EM 1110-2-2902
31 December 2020

US Army Corps of Engineers®
ENGINEERING AND DESIGN

Conduits, Pipes, and Culverts Associated with Dams and Levee Systems

ENGINEER MANUAL

THIS PAGE INTENTIONALLY LEFT BLANK

DEPARTMENT OF THE ARMY
U.S. Army Corps of Engineers
441 G Street, NW
Washington, DC 20314-1000

*EM 1110-2-2902

CECW-EC

Manual
No. 1110-2-2902

31 December 2020

Engineering and Design
CONDUITS, PIPES, AND CULVERTS ASSOCIATED WITH DAMS AND LEVEE SYSTEMS

1. Purpose. This manual provides risk informed guidance for the life cycle of conduits, pipes, and culverts associated with U.S. Army Corps of Engineers (USACE) constructed dam and levee projects.

2. Applicability. This manual applies to all HQUSACE elements, major subordinate commands, districts, laboratories, and field operating activities having responsibilities for the design and construction of civil works projects.

3. Distribution Statement. This manual is approved for public release with unlimited distribution.

4. References. References are at Appendix A.

5. Records Management (Recordkeeping) Requirements. Records management requirements for all record numbers, associated forms and reports required by this regulation are included in the Army's Records Retention Schedule - Army. Detailed information for all record numbers, forms, and reports associated with this regulation are located in the Army's Records Retention Schedule - Army at https://www.arims.army.mil/arims/default.aspx.

6. Background. This manual updates the current EM 1110-2-2902 dated 31 March 1998. The manual is intended for designers and operators associated with design and operation and maintenance of pipes from proper selection to removal or abandonment of these structures.

FOR THE COMMANDER:

9 Appendixes
(See Table of Contents)

JOHN P. LLOYD
COL
Chief of Staff

*This manual supersedes EM 1110-2-2902, dated 31 March 1998.

THIS PAGE INTENTIONALLY LEFT BLANK

DEPARTMENT OF THE ARMY EM 1110-2-2902
U.S. Army Corps of Engineers
441 G Street, NW
CECW-CE Washington, DC 20314-1000

Manual
No. 1110-2-2902 31 December 2020

Engineering and Design
CONDUITS, PIPES, AND CULVERTS ASSOCIATED WITH DAMS AND LEVEE SYSTEMS

TABLE OF CONTENTS

Appendixes

Table List

Figure List

Chapter 1
Overview

1.1. Purpose. This manual provides guidance for the selection, design, installation, inspection, condition assessment, mitigation prioritization, maintenance, repair, rehabilitation, removal, or decommissioning of conduits, pipes, and culverts associated with USACE Civil Works dam and levee projects whether they convey fluids or gases, serve as encasements for utility lines, or intercept seepage. In general, designers should minimize the presence of pipes through engineered embankments to only those that are absolutely necessary for operation and safety (i.e., essential pipes) and that can be designed with a high standard of engineering to minimize the potential for uncontrolled release. Due to USACE's unique structures and their associated risks, criteria in this manual may exceed industry requirements. The criteria in this manual are minimum USACE requirements. These criteria may be exceeded by the designer to enhance resiliency as risk and economics dictate.

1.2. Delegated USACE Design Approval. Guidance in this manual supersedes all other previously published USACE guidance related to conduits, pipes, and culverts, except for those related to pipe sizing. Justifications for deviations and waivers from mandatory design standards in this manual must include a risk assessment. All deviations and waivers must be clearly identified in the decision documents/design reports and must be deliberately called out, within the review plan, as a specific charge for the review. All proposed deviations and waivers from these mandatory design standards, including rationale, must be documented in a memorandum approved by the respective District and Division Dam Safety Officer or Levee Safety Officer and concurred by the Dam Safety Oversight Group (DSOG) or Levee Safety Oversight Group (LSOG), whichever is appropriate. The DSOG or LSOG will ensure the appropriate USACE Community of Practice leader(s) or their designated representatives are included in the concurrence process. Review documentation will account for all decisions and rationale for deviations and waivers, including the memorandum documenting approval and concurrence.

1.3. Applicability. This manual applies to all conduits, pipes, and culverts (including their associated structures and attached appurtenances) that are elevated, within, beneath, or adjacent to the following USACE Civil Works projects: 1) embankment dams (hereafter referred to as dams); 2) concrete dams (only when founded on a soil foundation); and 3) levees (including embankments, floodwalls, and closures). It can also be used as a reference document by dam and levee owners or operators of non-USACE Civil Works projects. The term embankment refers to an earthen dam or levee; a floodwall is typically a concrete or steel structure used as a flood barrier. This manual provides guidance related to each phase of the pipe's life cycle (Figure 1-1) in an effort to extend its service life and reduce the probability of pipe-related system failures, whether by breach or interior ponding. Although the majority of design information has been consolidated into Chapter 4, other design considerations and requirements are distributed throughout the manual. The user of this Engineer Manual (EM) is responsible for seeking opportunities to incorporate the Environmental Operating Procedures (EOPs) whenever possible. A listing of EOPs is available on the USACE Headquarters website.

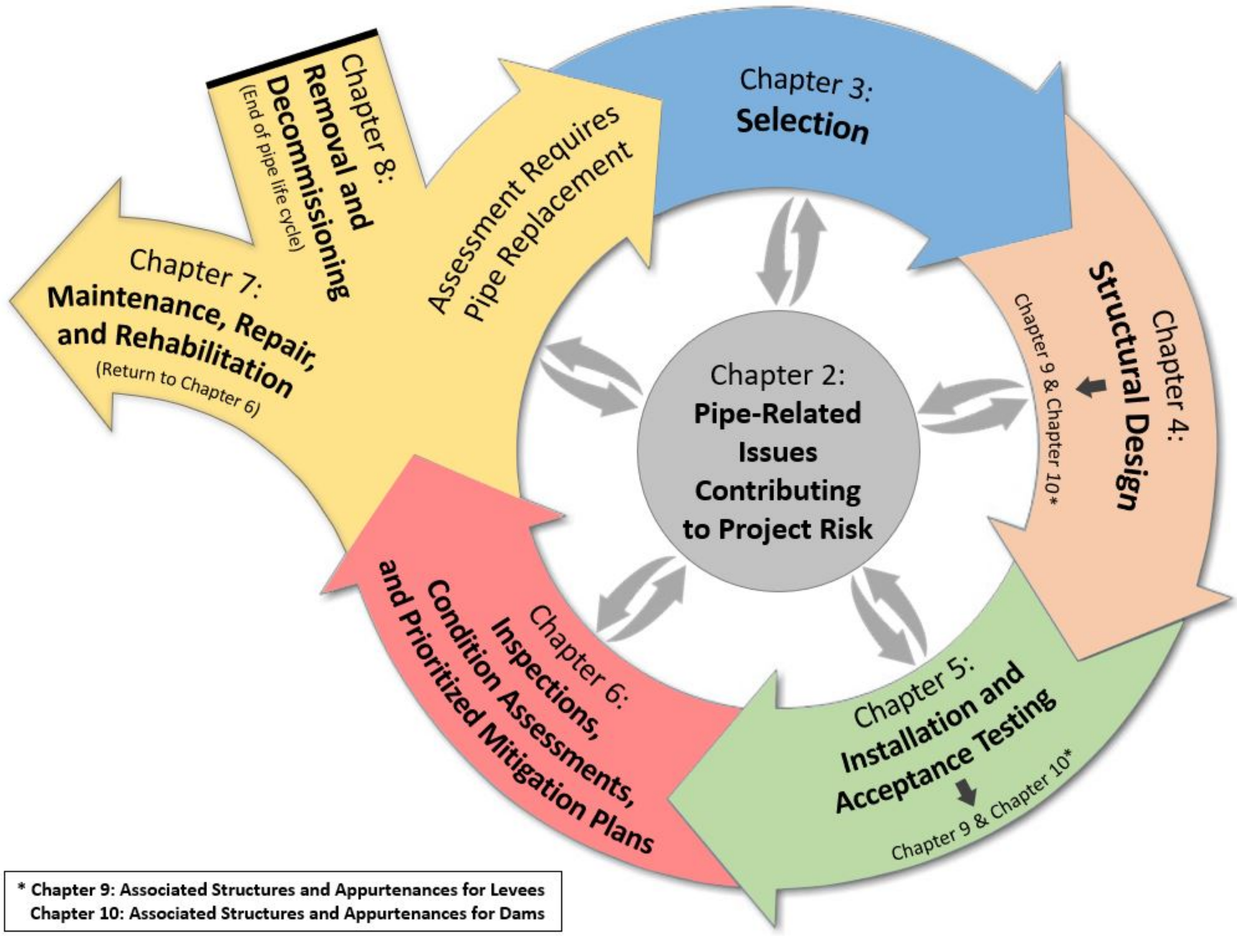


Figure 1-1. The chapters in EM 1110-2-2902 correlate with the life cycle of a pipe.

1.4. Distribution Statement. This manual is approved for public release; distribution is unlimited.

1.5. References. References are listed in Appendix A.

1.6. Terminology.

1.6.1. Dams and Levees. Dams and levees are typically structures constructed of soil, concrete, or a combination of the two. Reference ER 1110-2-1156 for comprehensive information and definitions related to dams and the most up-to-date levee safety guidance for comprehensive information and definitions related to levees.

1.6.2. Landside and Waterside. The landside (dryside) of an embankment or floodwall adjoins the leveed area and is on the opposite side of the source of flooding; it is comparable to the downstream side of a dam. The waterside is the side that experiences the hydraulic loading from the watercourse.

1.6.3. USACE Civil Works Project. This term generally refers to a USACE federally authorized and constructed flood risk management project such as a dam or levee that is operated and maintained by USACE or in cases of levees, by a non-federal sponsor.

1.6.4. Conduits, Pipes, and Culverts.

1.6.4.1. General. The terms “conduit,” “pipe,” and “culvert” are not clearly defined or standardized and are subject to personal bias, differing regional usage, and inconsistent industry terminology; however, within this manual, “pipe” is used to collectively refer to conduits, pipes, and culverts as defined below.

1.6.4.2. Conduit. A relatively large, typically reinforced, cast-in-place enclosed concrete structure used in the majority of dams and sometimes in levees for the conveyance of water. Conduits are generally equipped with gates for closure and discharge regulation (Figure 1-2).

(Courtesy of USACE Louisville District)
Figure 1-2. Cast-in-place conduit.

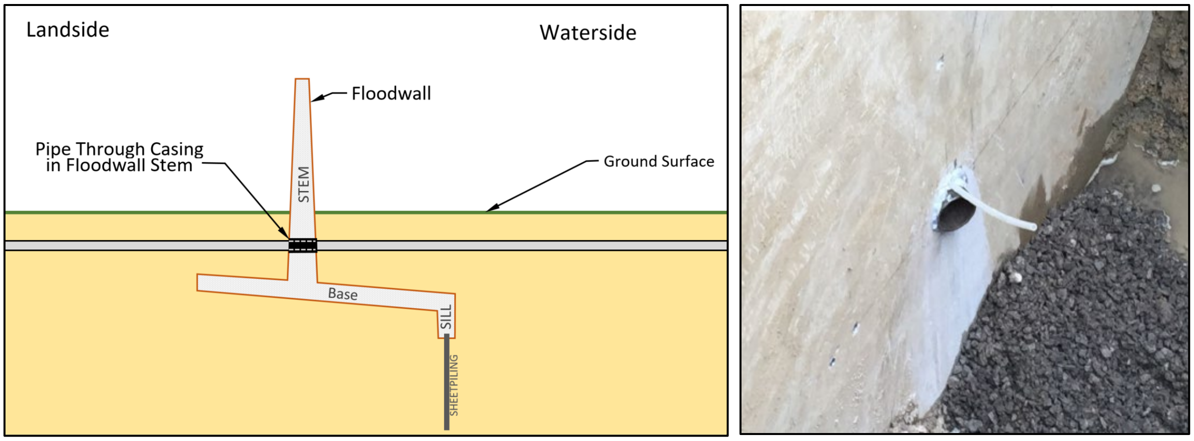
1.6.4.3. Pipe. A pipe is typically, but not always, a circular conveyance structure constructed of various materials that allows non-pressurized or pressurized flow to drain interior or reservoir areas, move fluids or gases, or pass utility lines (Figure 1-3 through Figure 1-5).

(Courtesy of USACE Louisville District)

Figure 1-3. Non-pressurized gravity fiber-reinforced pipe installation within an embankment.

(From America's Wetland Foundation Resource Center [left] and Courtesy of USACE New Orleans District [right])

Figure 1-4. Pressurized water pipe installation elevated over levee sideslopes and crest.



(Courtesy of USACE Louisville District)
Figure 1-5. Pipe casing epoxied through floodwall stem to pass a utility line.

1.6.4.4. Culvert. A culvert is typically a circular or box structure passing within or beneath a dam or levee, with the exception of open pipes beneath roadways, for the purpose of controlled or uncontrolled conveyance of water (Figure 1-6).

(Courtesy of USACE Alaska District)
Figure 1-6. Box culvert installation.

1.6.5. Pipe Nomenclature. Figure 1-7 shows a pipe cross-section depicting typical pipe nomenclature.

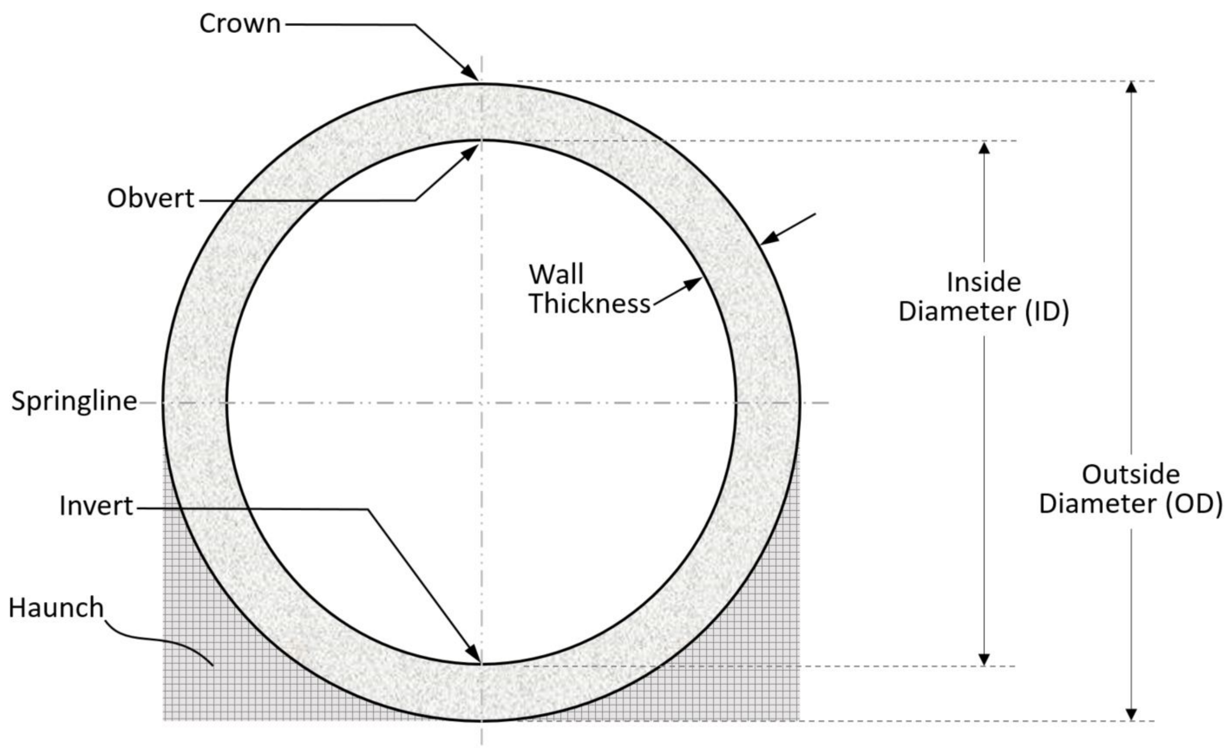


Figure 1-7. Typical pipe nomenclature.

1.6.6. Pipe System Anatomy. Figure 1-8 depicts the typical elements of a pipe system from a cross-section view.

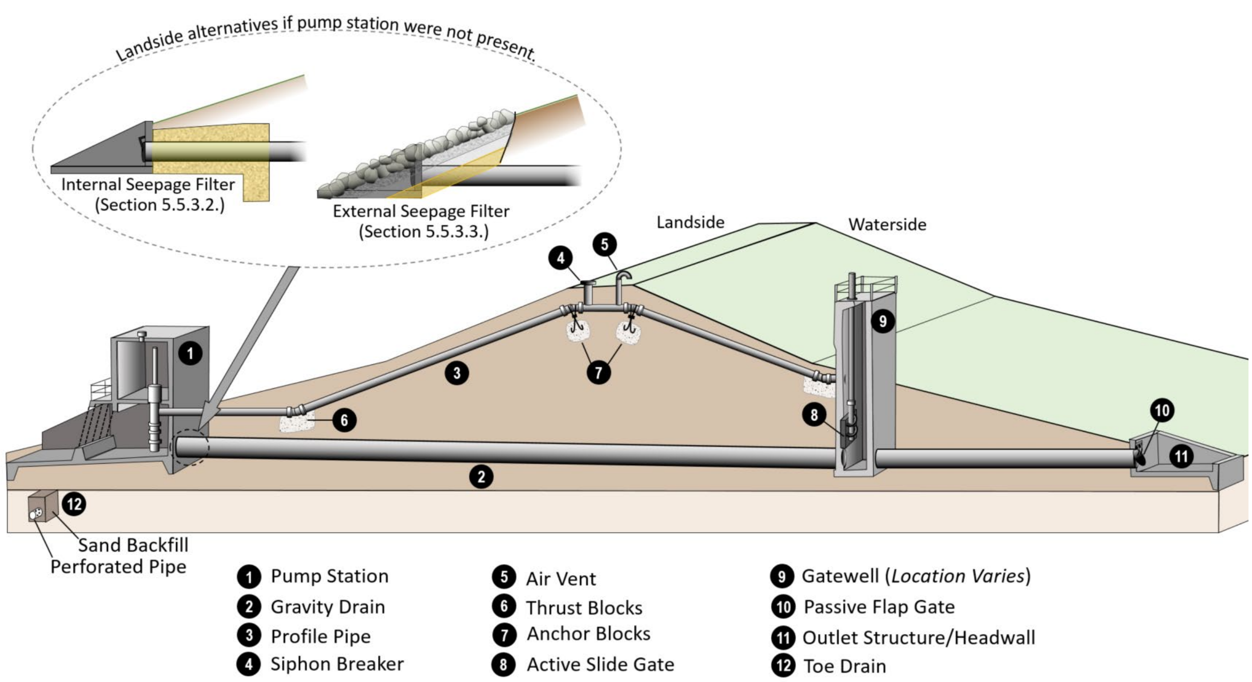


Figure 1-8. Typical associated structures and appurtenances common to pipes through levees.

1.6.7. Essential vs. Non-essential Pipes. Essential pipes are those necessary to the operation of a dam or levee system. Examples of essential pipe functions include regulating reservoir elevations; providing interior drainage using gravity and pump station discharge systems; powering appurtenances through electrical conduits; and providing pressure relief by intercepting seepage (i.e., toe drains or relief well collector system pipes). Non-essential pipes generally serve as utility crossings (commonly called third-party pipes) for water mains, sanitary sewers, liquid petroleum products, natural gas, or electrical, fiber optic, and other services. Non-essential pipes within, beneath, or adjacent to federally authorized projects require review and approval per 33 United States Code (USC) 408 ("Section 408") requirements.

1.6.8. Pipes that Can Impact the Integrity of a Dam or Levee. Whether considered essential or non-essential, pipes within, beneath, or adjacent to a USACE federally authorized and constructed dam or levee within its influence zone (reference Section 6.3) must be designed to meet the minimum requirements of this manual. In addition, new pipe installations, existing pipe relocations, or mitigation of pipe-related unfiltered exits (e.g., granular pipe backfill daylighting in a leveed area) crossing the proposed alignment must be evaluated prior to construction of a new USACE dam or levee.

1.6.9. Pipes that Cannot Impact the Integrity of a Dam or Levee. Pipes located outside a project's influence zone (reference Section 6.3), or those associated with a flood damage project such as a channel, should be designed to other federal agency or industry standards as appropriate (i.e., Federal Emergency Management Agency, state transportation agencies, ASTM International, American Association of State Highway and Transportation Officials, American Water Works Association, or other governing authorities). However, the guidance provided in this manual can be used for such pipes as well.

1.6.10. Levels of Adherence. The terms "will" and "must" denote mandatory requirements for compliance with this manual. The term "should" indicates a strong preference for a given criterion and a USACE recommendation. The term "may" indicates a criterion that is allowable.

1.7. Hydraulic Capacity. Pipe sizing is beyond the scope of this manual and should be performed by a hydraulic engineer according to EM 1110-2-1602 before referring to this manual (with the exception of sizing a new pipe being used to rehabilitate an existing pipe, as indicated in Chapter 7).

1.8. Requests for Altering Existing USACE Civil Works Projects with Respect to Pipes. The approval process for altering a federally constructed project is addressed in 33 USC 408 (see also Engineer Circular 1165-2-220 or its successor regulation); Appendix H contains suggested submittal requirements for an entity requesting such approval for alterations related to pipes. Modifications of USACE owned, operated, and maintained dam projects must follow ER 1110-2-1156.

THIS PAGE INTENTIONALLY LEFT BLANK

Chapter 2
Pipe-Related Issues Contributing to Project Risk

2.1. Purpose. Internal erosion accounts for nearly half of all embankment dam failures (Foster, Fell, and Spannagle), with many of those failures occurring along pipes. The intent of Chapter 2 is to introduce the concept of risk as characterized by USACE, and to provide examples that show how the presence of pipes associated with embankments or floodwalls has the potential to increase the overall project risk by increasing the probability of structure breach, or even causing interior ponding without a breach.

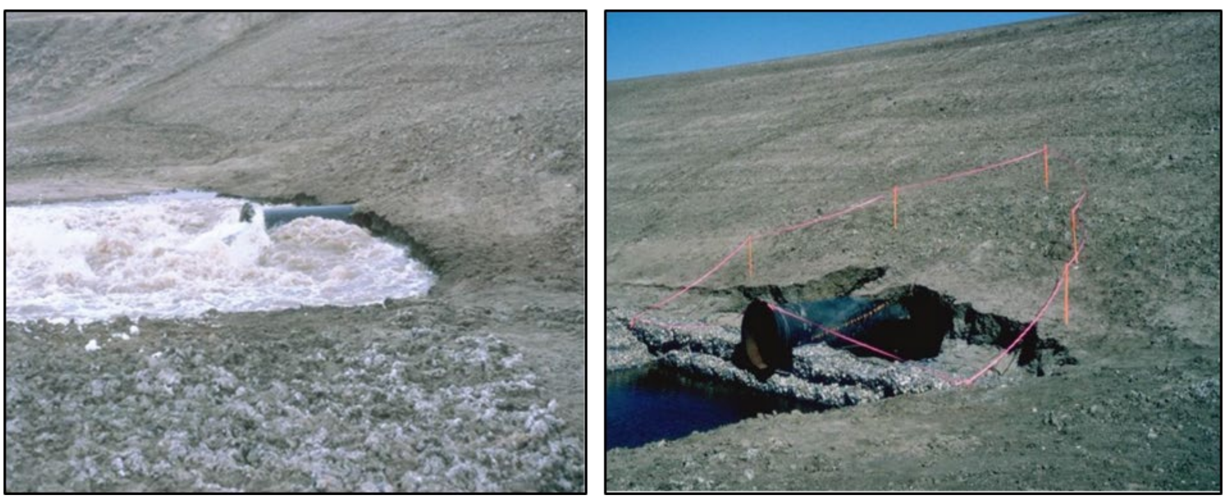
2.2. Introduction. USACE calculates the risk associated with the presence of a dam or levee as the function of three distinct quantitative components: 1) the probability that a certain water level – pool or flood – will occur (the hazard); 2) how the dam or levee will respond to the event – the likelihood of project failure (the performance); and 3) the loss of life, economic losses, and environmental impacts that could occur as a result of inundation (the consequences). It is not the intent of this chapter to explain the risk quantification process; instead, potential failure modes (PFMs) are used to explain the pipe-related vulnerabilities that influence the “performance” variable. Chapters 3 through 10 include PFM-related recommendations for reducing the likelihood of pipe-related system failures as a way of lowering the overall project risk.

2.3. Potential Failure Modes.

2.3.1. Overview. A PFM describes the chain of events leading to an embankment or floodwall breach that allows the uncontrolled release of water or to interior inundation without a breach, either of which results in adverse consequences. A pipe-related failure mode may involve the erosion of the embankment or foundation surrounding the pipe leading to a collapse of the embankment crest (Figure 2-1), a collapse of floodwall monolith(s), or an uncontrolled release of water without lowering the crest (Figure 2-2). It can also describe interior ponding caused by a blocked pipe or seized slide gate that prevents drainage or allows flood water to enter.

2.3.2. Potential Failure Mode Categories. Forensic studies of pipe-related dam and levee failures have identified many mechanisms that can lead to breach (Federal Emergency Management Agency [FEMA], 2005); within this manual, these mechanisms have been grouped into four broad PFMs (PFM-1 through -4). The remaining PFMs (PFM-5 through -7) cover cases where pipe-related system failures cause interior ponding without a breach, such as an inoperable gate. Flooding due to levee overtopping as a result of its hydraulic design or spillway flows associated with a dam is outside the scope of this manual.

(Courtesy of National Resource Conservation Service)
Figure 2-1. Embankment breach resulting in loss of pool caused by internal erosion along a pipe.



(Courtesy of the Association of State Dam Safety Officials ASDSO)

Figure 2-2. Internal erosion causes a release of the reservoir without collapsing the embankment.

2.3.3. Event Trees. In their most basic form, breaching PFMs must have at least three elements: 1) a load must be applied; 2) a flaw or defect must be present; and 3) the flaw must be able to initiate and progress to breach. Non-breaching PFMs are similar but do not result in failure of an embankment or floodwall. To better understand how each PFM leads from flaw to breach or inundation, it is helpful to expand the basic elements into an "event tree." The example event tree for PFM-4 (Figure 2-3) illustrates discrete steps and provides opportunities for risk-reducing actions (blue boxes) at various stages. Appendix B contains representative event trees for all PFMs listed in this chapter.

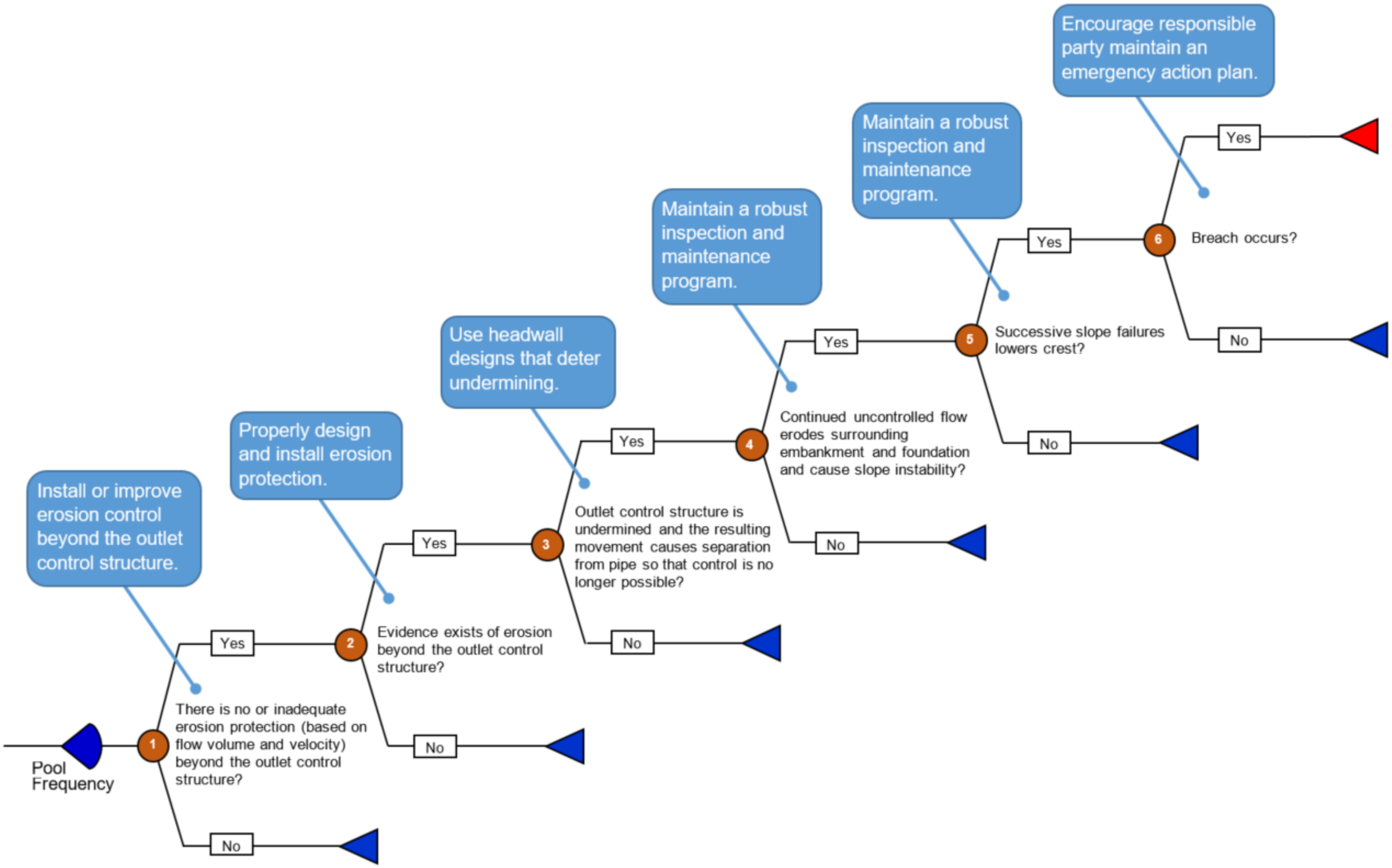


Figure 2-3. Event tree for PFM-4. (Notes in blue boxes describe methods to reduce the probability of breach.)

2.3.4. Pipe-specific Potential Failure Modes Leading to Breach.

2.3.4.1. Overview. The various ways embankments and floodwalls can fail due to pipes have been consolidated into four general mechanisms: PFM-1 through PFM-3 represent the internal erosion mechanisms most commonly associated with pipe-related embankment and floodwall failures, while PFM-4 describes a failure mechanism that initiates externally of the embankment. In general, these failure mechanisms also apply to floodwalls (particularly PFM-3). The specific and differing aspects of internal erosion are not covered in this manual but are comprehensively discussed in “Best Practices in Dam and Levee Safety Risk Analysis.” PFMs resulting in a breach prior to overtopping include: PFM-1: Internal Erosion along a Pipe; PFM-2: Internal Erosion from Leakage of a Pressurized Pipe; PFM-3: Internal Erosion into a Pipe; and PFM-4: External Erosion at a Pipe Outlet.

2.3.4.2. PFM-1 – Internal Erosion along a Pipe.

2.3.4.2.1. Failure Mode Description. Installing a pipe through an embankment has the potential to create several flaws in the form of preferential seepage paths (Figure 2-4). The most vulnerable area is below the pipe’s springline where soil compaction is difficult (Figure 2-5 and Figure 2-6). A designer may choose to fully encase the pipe in a CLSM to address this problem; however, this does not eliminate the potential for seepage. Even with a shrinkage-reducing admixture, CLSM may shrink enough during curing to produce gaps along the trench walls (Figure 2-7), thus providing a pathway for seepage and the opportunity for soil removal. Eventually, the water flow causes gross enlargement of the seepage path and an uncontrolled release of water, or the enlarged seepage path collapses and lowers the embankment or floodwall crest. A visual representation of the sequence of events related to PFM-1 is shown in Figure 2-8 through Figure 2-12.

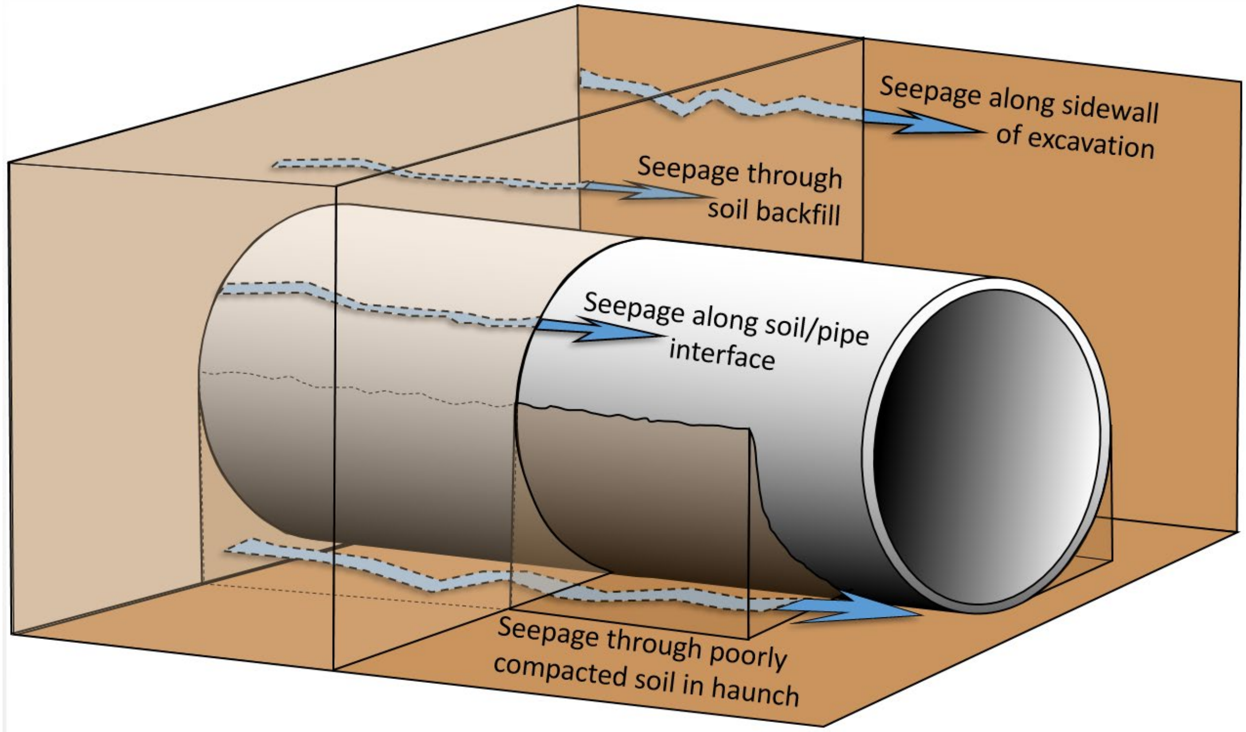
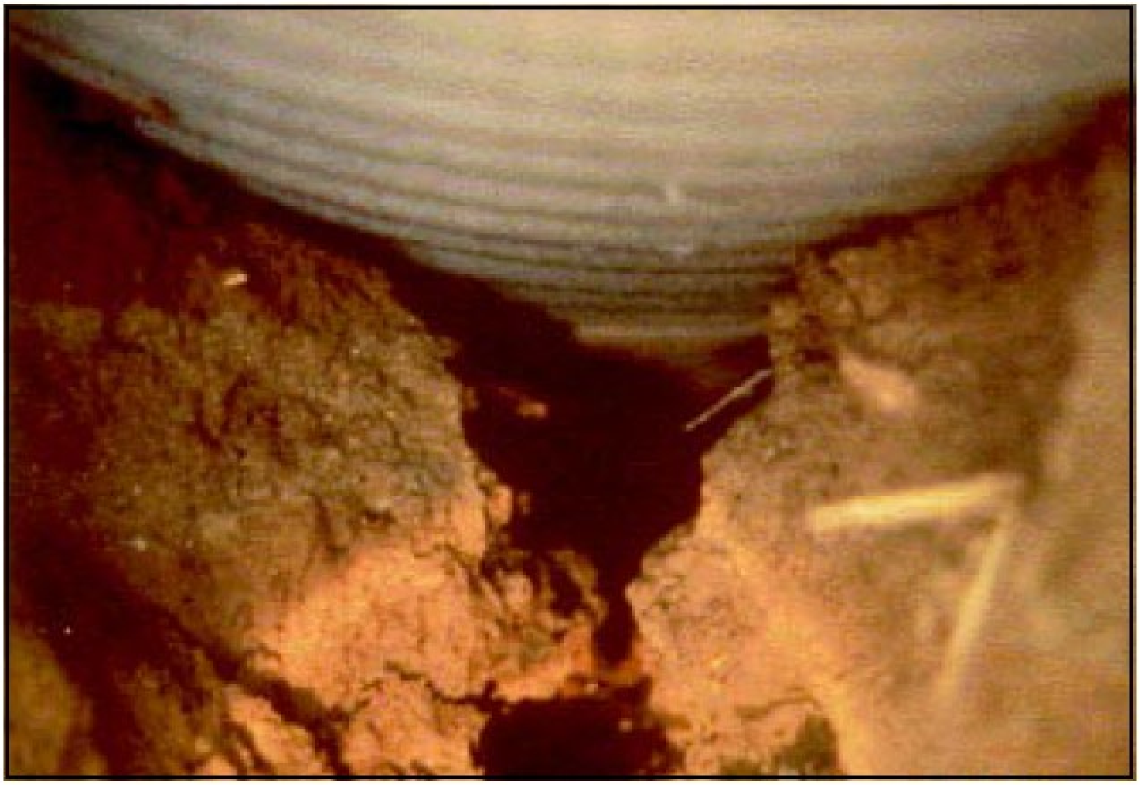


Figure 2-4. Soil backfill zones where potential flaws could promote PFM-1.

(Courtesy of USACE Louisville District)

Figure 2-5. Pipe placed on unprepared surface, promoting seepage within the haunch.



(From Low-Volume Roads Engineering, USDA-Forest Service, 2003)

Figure 2-6. Substantial void space beneath pipe caused by poorly placed backfill.

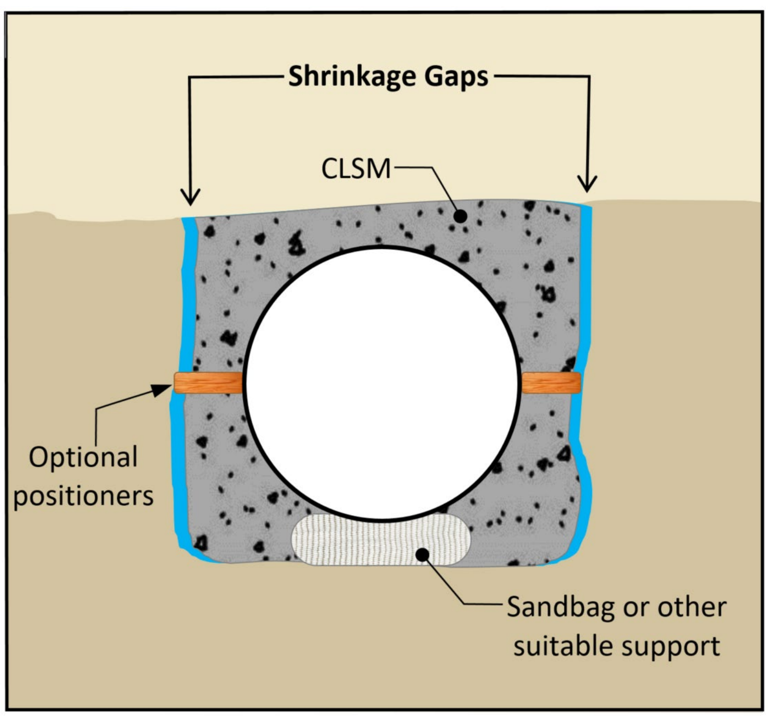


Figure 2-7. Potential flaw within CLSM-filled trench that may promote PFM-1.

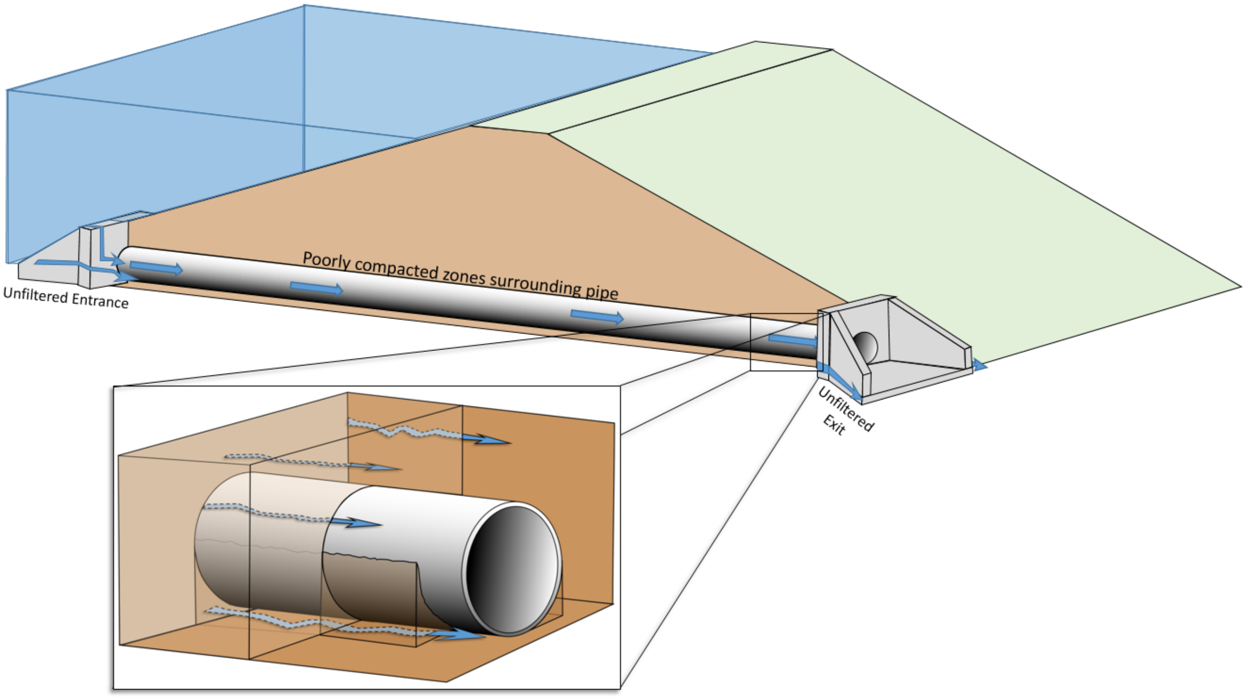


Figure 2-8. Unfiltered pipe ends allow seepage to initiate internal erosion.

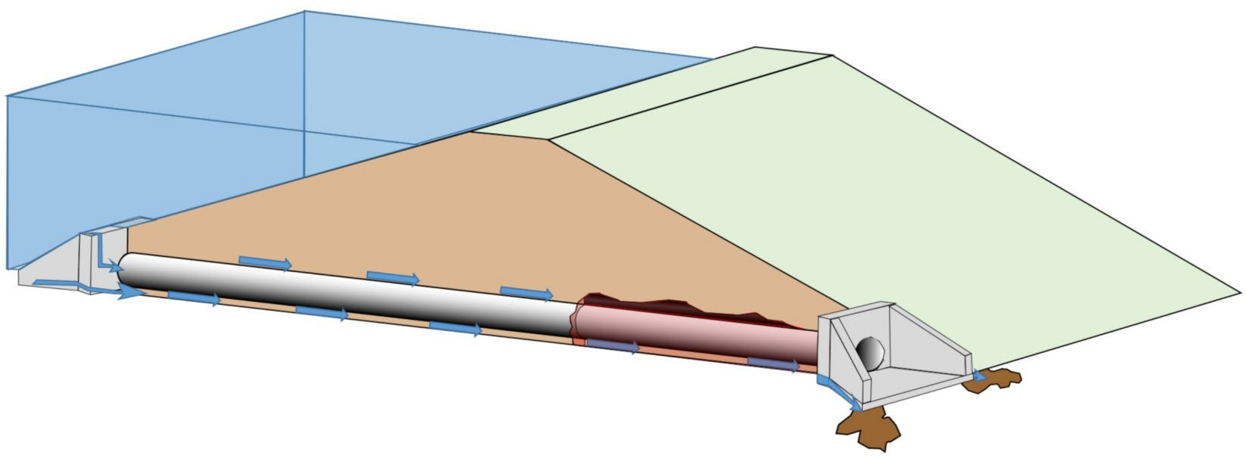


Figure 2-9. Soil removal begins and erodes along the pipe toward the water source.

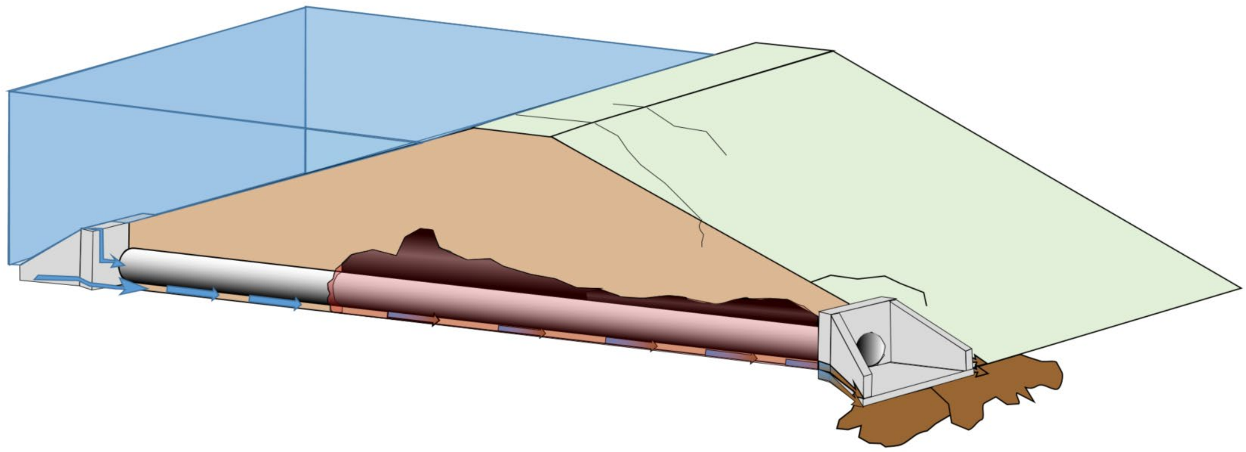


Figure 2-10. Progression of failure mode begins to create observable distress.

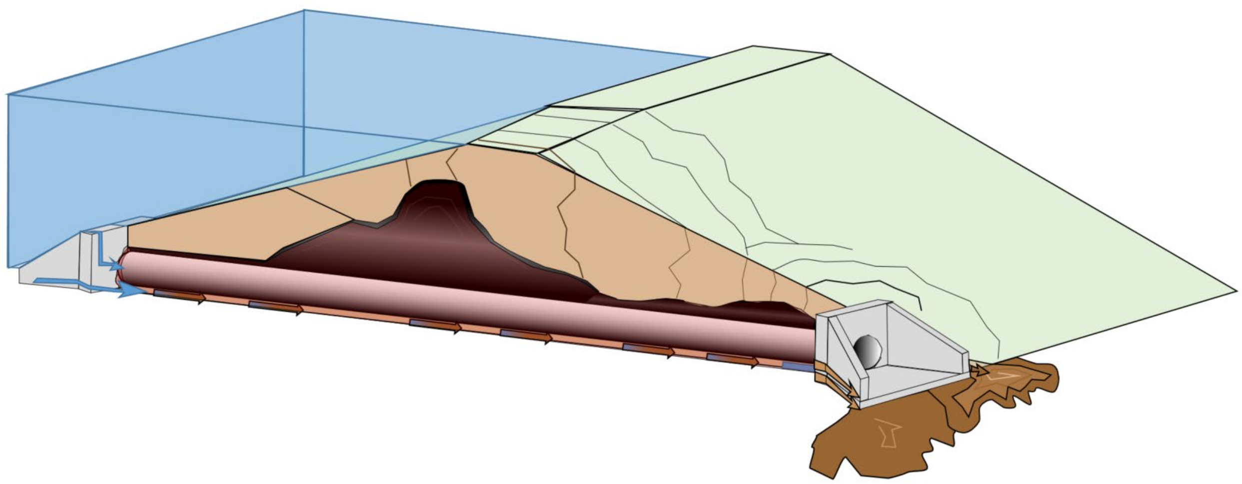


Figure 2-11. Continued progression increases distress and alters the embankment geometry.

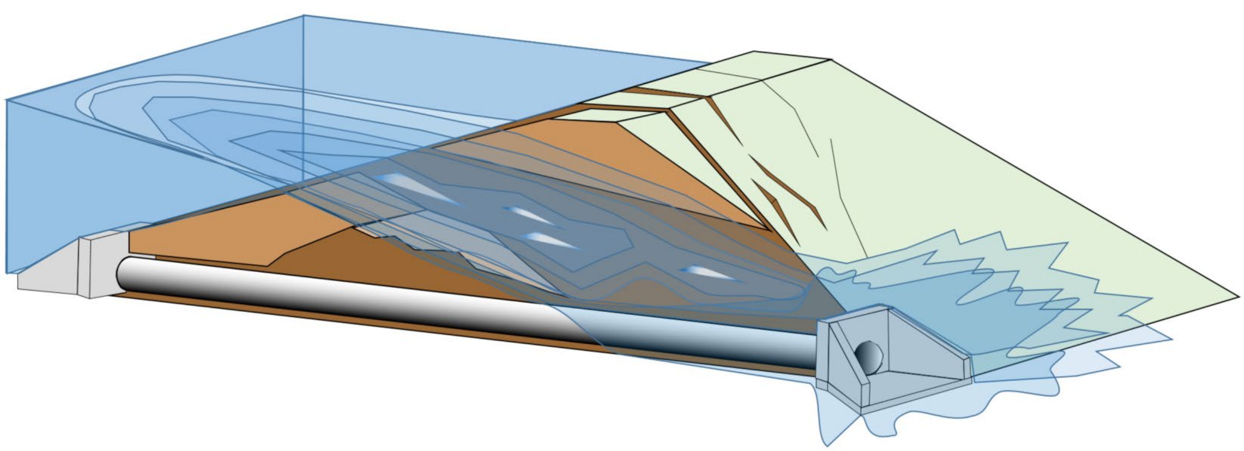


Figure 2-12. Intervention is not possible or unsuccessful and a collapse and breach occurs.

2.3.4.2.2. Proactive Measures and Intervention. Proactive measures include installing either an internal seepage filter (Figure 2-13) during typical trenched-construction, or an external filter along with a waterside buttress when a pipe is installed using trenchless methods (reference Figure 5-38). A properly designed filter behind the landside levee headwall intercepts and filters seepage along any of the pathways shown in Figure 2-4. Filters for dams must be designed according to "Filters for Embankment Dams" (FEMA, 2011). Observations during regular inspections that could indicate PFM-1 is in progress include fans of loose soil near the landside headwall (particularly after a high-water event) or depressions in the ground surface above the pipe alignment. Regular readings of instrumentation near or along the pipe can also help detect internal changes. Intervention during a high-water event typically consists of placing sand and gravel on the slope over the area of seepage or sandbagging around the base of the headwall to collect water and create an opposing hydraulic load to prevent or reduce the movement of soil.

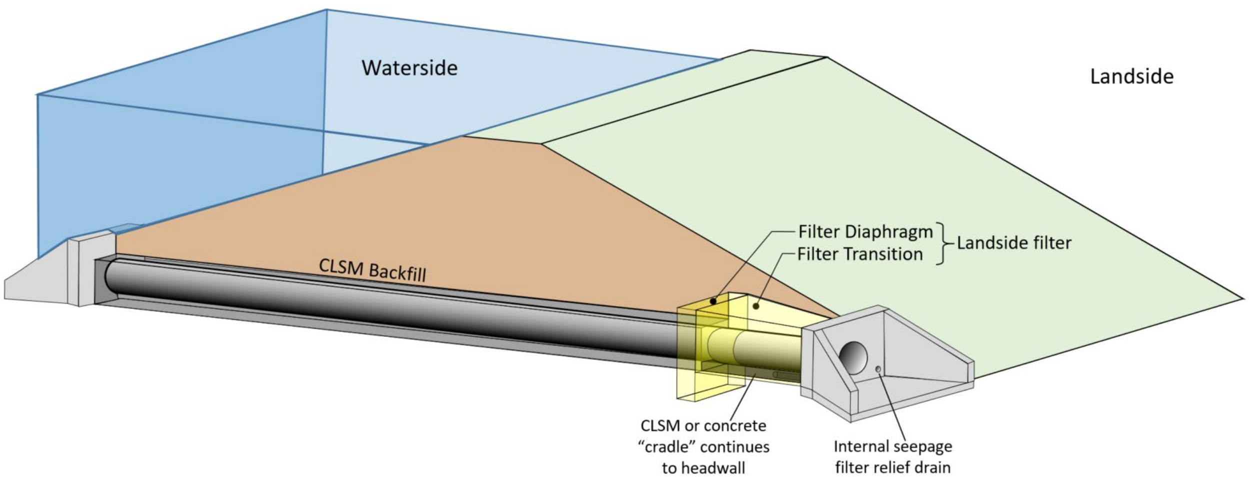


Figure 2-13. Internal seepage filter used to capture and safely release seepage along the pipe.

2.3.4.3. PFM-2 – Internal Erosion from Leakage of a Pressurized Pipe.

2.3.4.3.1. Failure Mode Description. Flaws such as defective connections, holes in the pipe, or separations/ruptures caused by excessive differential movement, can allow internal erosion to initiate by providing locations for leakage from pressurized pipes (e.g., water mains, forced sewer lines, penstocks, and obstructed gravity drains that become pressurized such as in Figure 2-14). If the escaping fluid is pressurized sufficiently to overcome the forces within the surrounding soil, it will fracture the soil along the path of least resistance, often along the pipe/soil interface, until it discharges on the ground surface. Once the full seepage path has been established, it is then possible for the flowing water to start removing soil from the embankment or foundation. Similar to PFM-1, gross enlargement of the seepage path will allow an uncontrolled release of water leading to an embankment or floodwall collapse that lowers the crest (Figure 2-15 to Figure 2-20).

(Courtesy of USACE Louisville District)

Figure 2-14. Trapped interior water (pressurizing pipe) exits through corrosion holes.

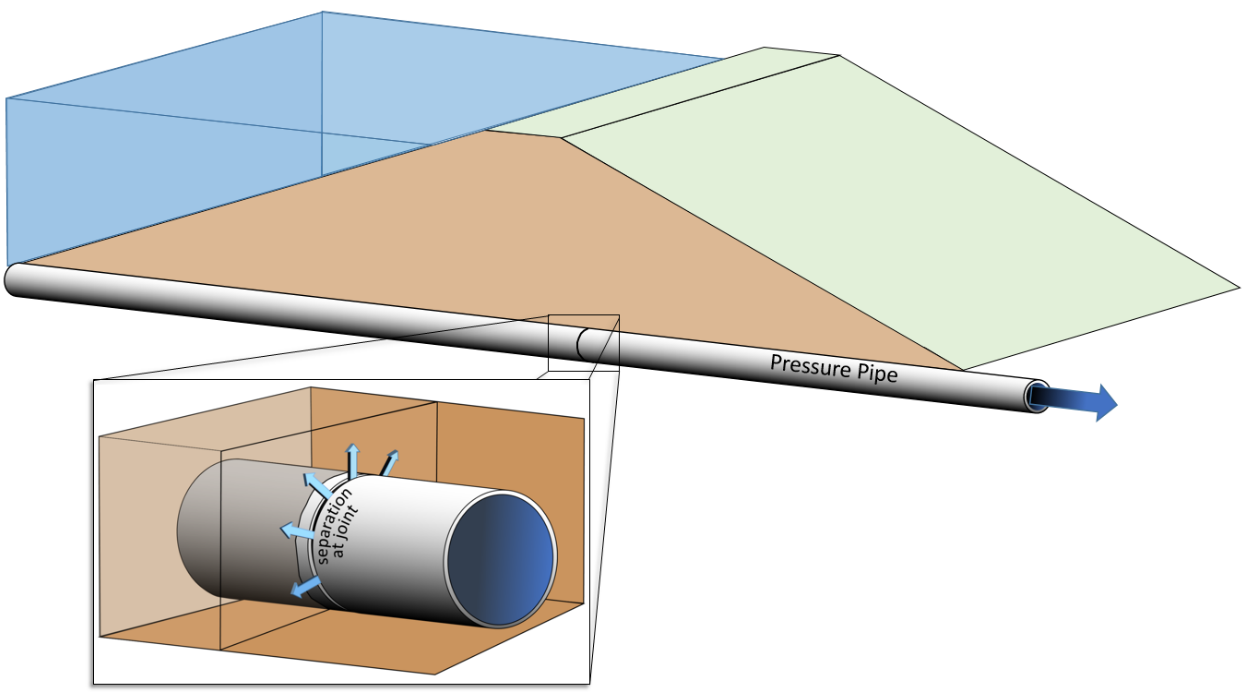


Figure 2-15. Flaw allows release of pressurized fluid into embankment.

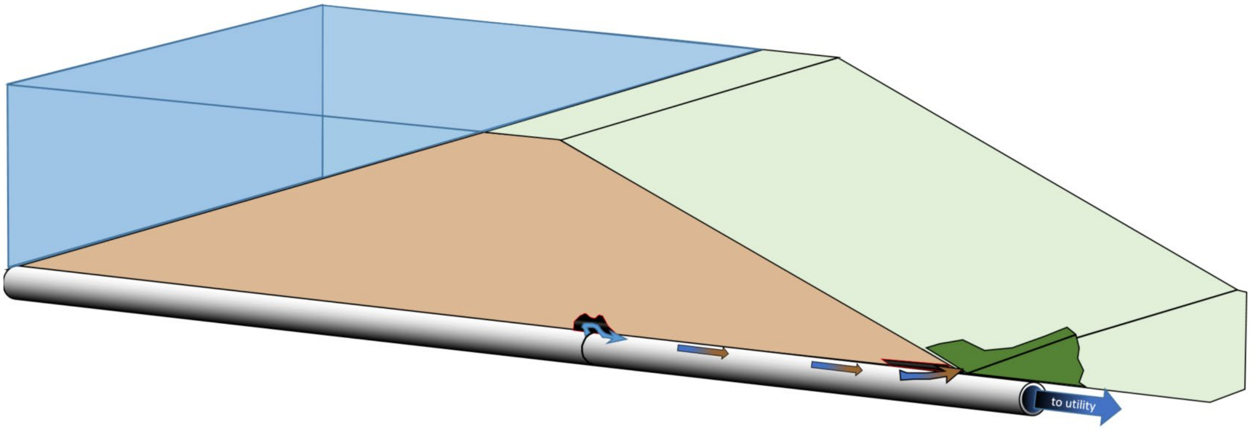


Figure 2-16. Pressurized fluid seeks an exit point and starts to saturate the embankment.

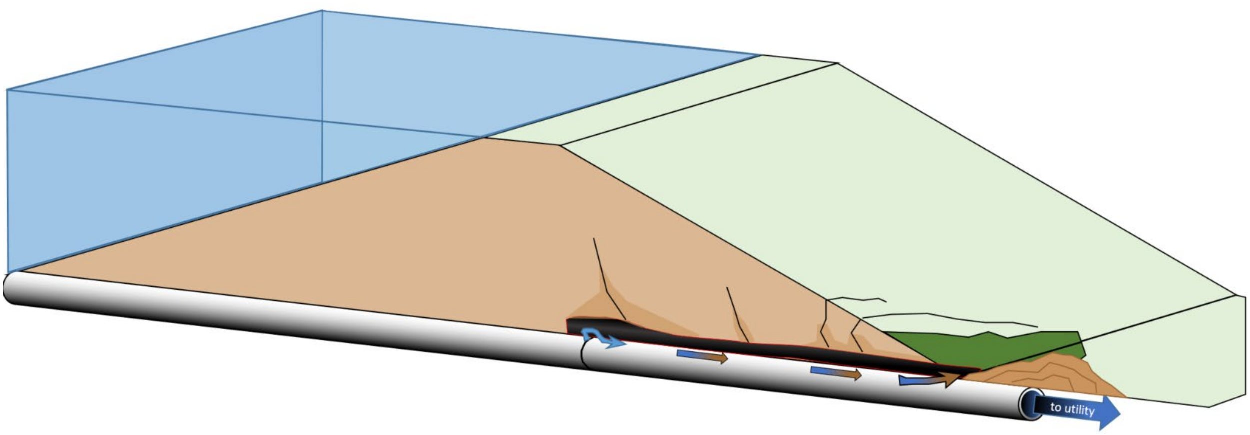


Figure 2-17. Saturation and removal of embankment material begins at the unfiltered exit.

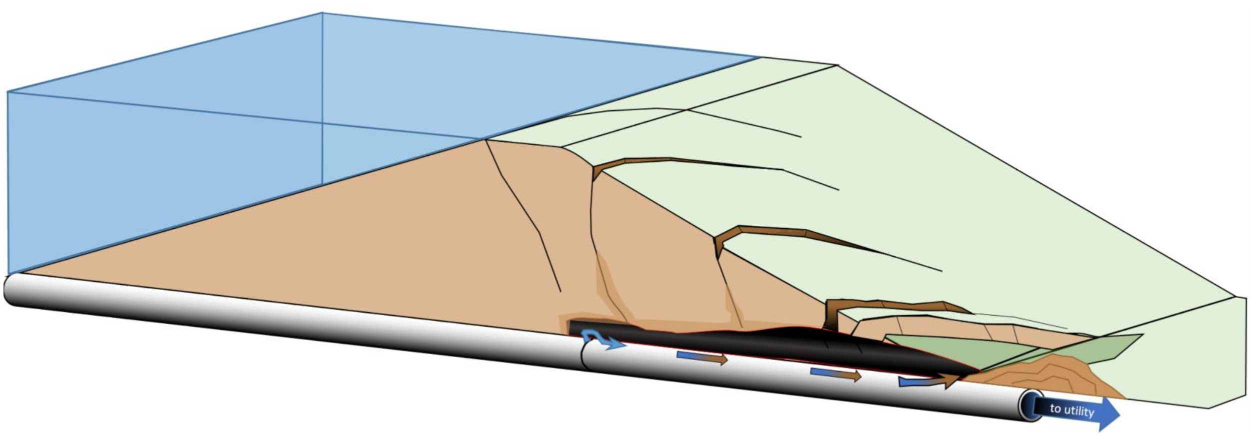


Figure 2-18. Continued progression increases distress and alters the embankment geometry.

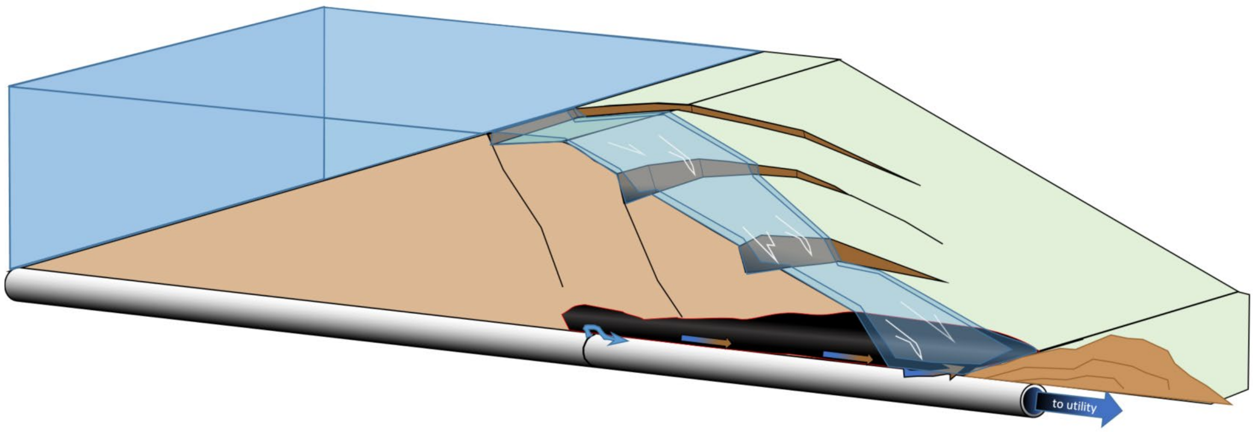


Figure 2-19. Lowering of the crest allows overtopping.

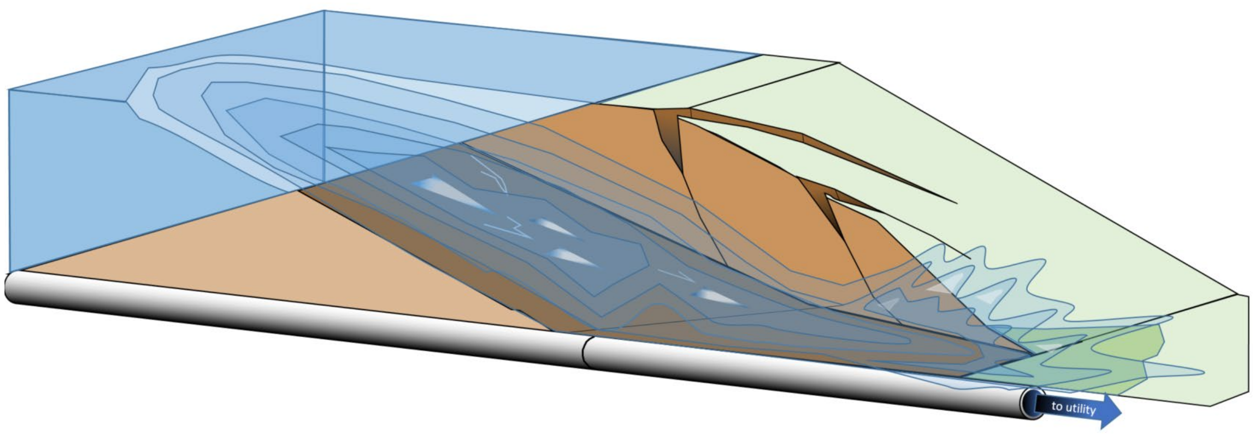


Figure 2-20. Intervention is not possible or unsuccessful and a collapse and breach occurs.

2.3.4.3.2. Proactive Measures and Intervention. To reduce the probability of PFM-2 occurring, non-essential pressure pipes are prohibited within an embankment unless the crest supports infrastructure that cannot be disrupted. Requiring new pressure pipes to include shutoff valves (reference Section 5.7.6) to address the potential release of fluids into or onto the embankment will provide the ability to stop flow and minimize damage. For existing pressurized utilities where valves do not exist and the release of pressurized fluid within the embankment cannot be stopped quickly, intervention may require the placement of an aggregate patch on the slope over the area of release. Well-graded aggregate will filter the seepage to prevent the loss of soil, while the weight of the riprap will keep the filter material in place. Some form of containment, such as stacked sandbags, may be required to prevent the patch material from unraveling down the slope. For releases near the toe, the size of the patch may become extensive.

2.3.4.4. PFM-3 – Internal Erosion into a Pipe.

2.3.4.4.1. Failure Mode Description. An opening in a pipe can be caused by a defective or improperly installed pipe connection (i.e., joint or connection to an associated structure), a rupture due to movement or deformation of the pipe, or a hole due to corrosion or abrasion. This type of flaw in a non-pressurized pipe can allow the surrounding soil to unravel into the pipe (Figure 2-21 and Figure 2-22). The unraveling process (loss of embankment material creating a stope - Figure 2-23 through Figure 2-25) can be accelerated by a hydraulic loading when there is a pool behind a dam or flooding against a levee; however, this failure mechanism can also progress over years of ordinary precipitation infiltration. When the stoping process results in the opening of a sinkhole on the waterside of the embankment below the water level during a flood event, an uncontrolled release of water will occur until the water falls below the opening or intervention is successful (Figure 2-26 through Figure 2-30). However, even without the threat of water entry, the presence of a sinkhole on the embankment slope could promote progressive slope failures and ultimately, a lowering of the crest.

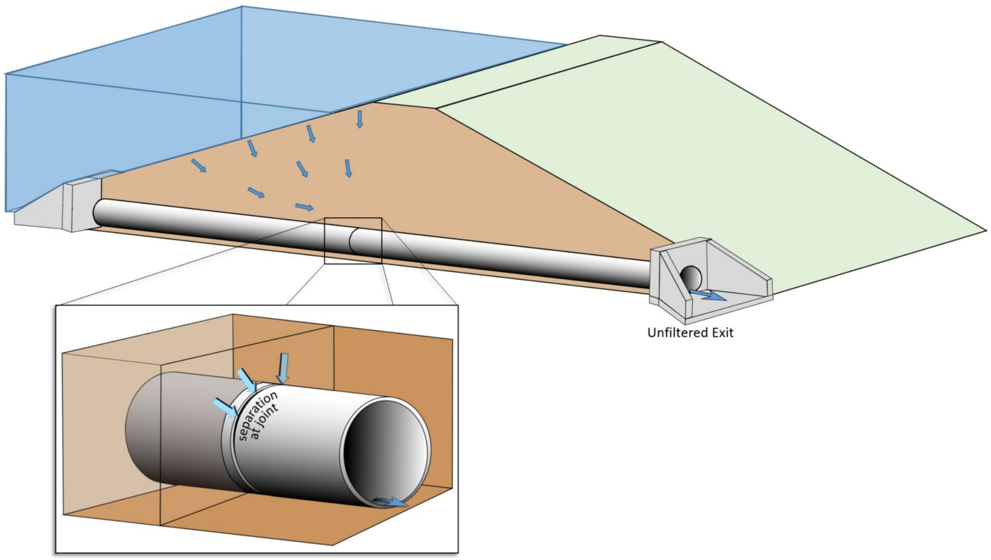


Figure 2-21. Excessive separation in any of the pipe joints allows soil infiltration.

(Courtesy of USACE St. Louis District)
Figure 2-22. Soil entering at joint separation.

(Courtesy of USACE Louisville District)
Figure 2-23. Holes in a pipe allowed continued soil loss to create a sinkhole in the levee crest.

(Courtesy of USACE Louisville District)
Figure 2-24. Opening in pipe behind headwall allows water to bypass the control appurtenance.

]

(Courtesy of USACE St. Louis District)

Figure 2-25. Sinkhole over a gravity drain, more than halfway up a 30-foot high embankment.

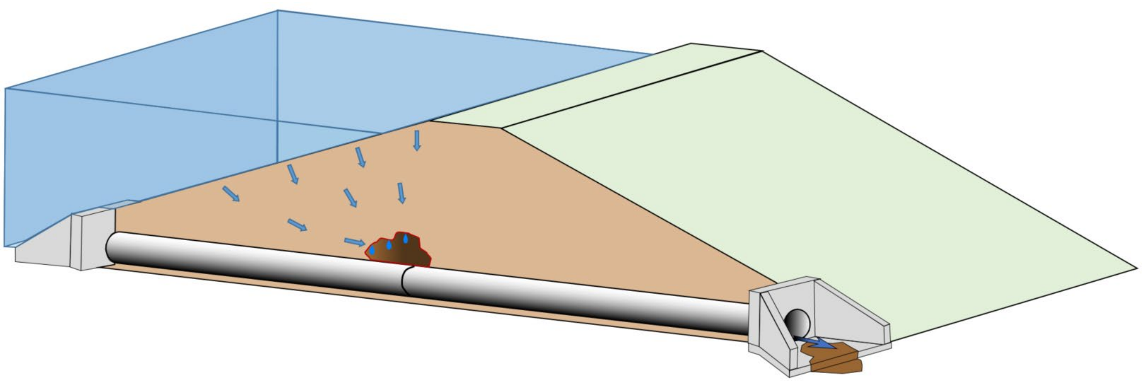


Figure 2-26. A void forms within the embankment as soil flows into the separation.

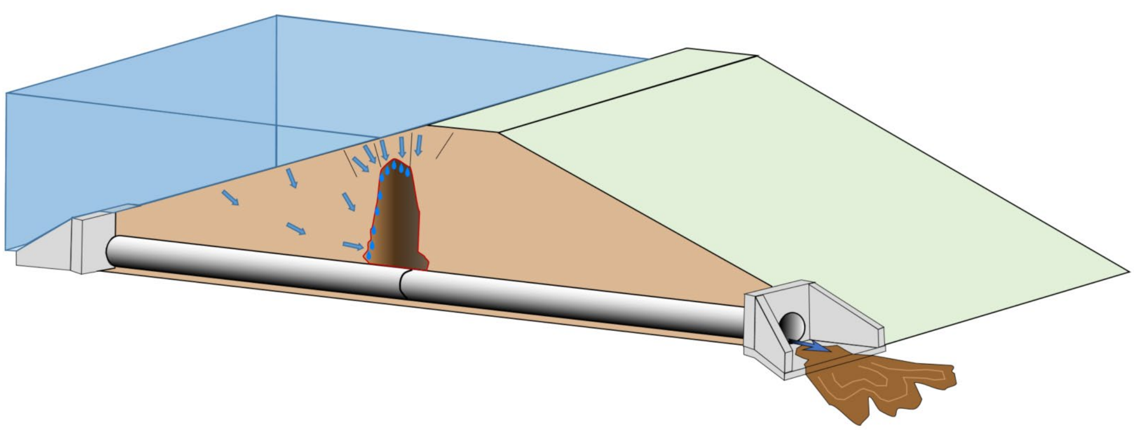


Figure 2-27. Stoping extends the void toward the embankment surface.

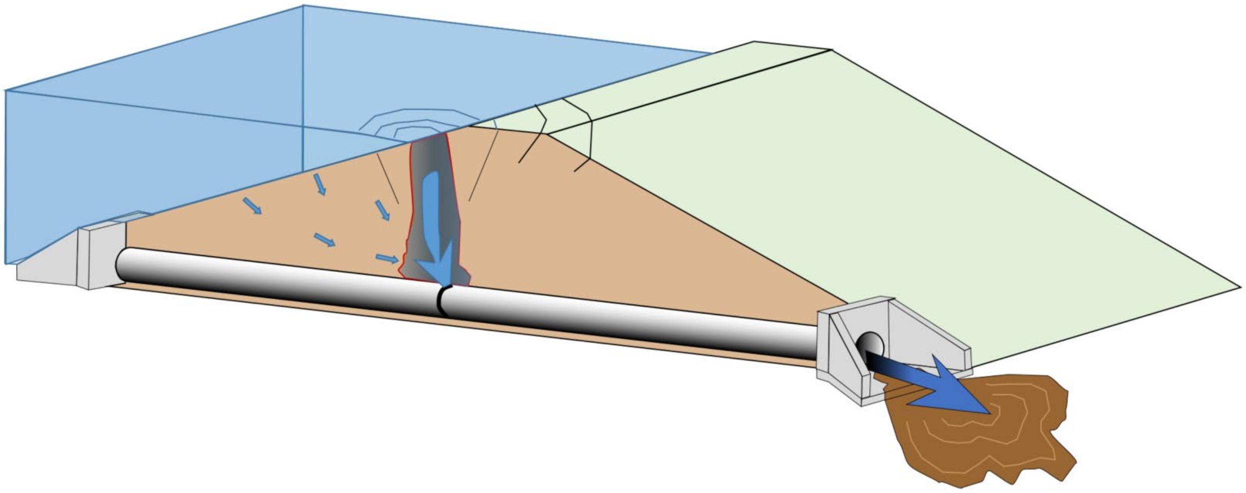


Figure 2-28. The void reaches the water source and initiates uncontrolled flow.

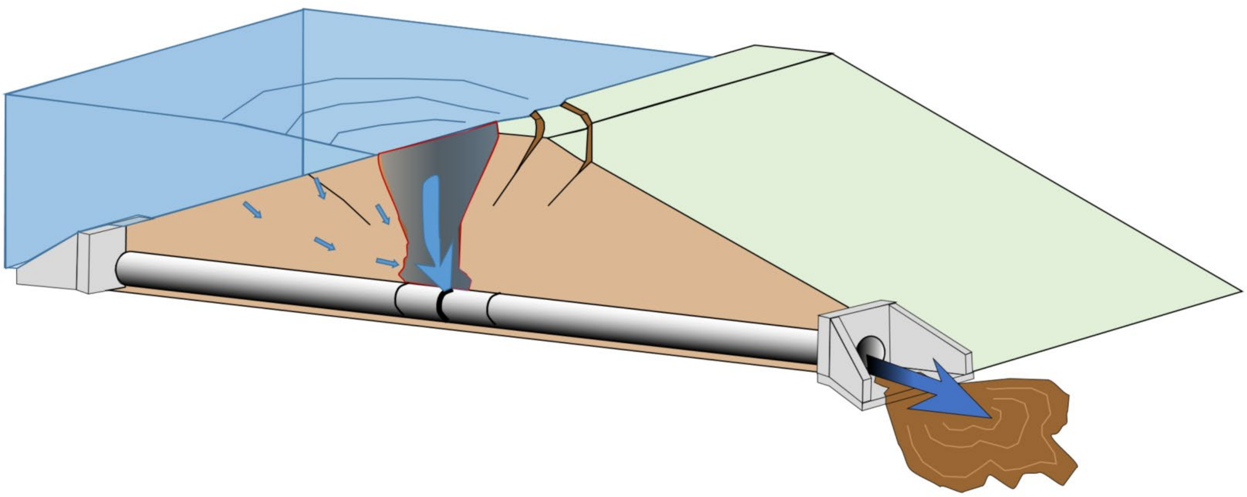


Figure 2-29. Uncontrolled flow destabilizes pipe sections and deforms crest.

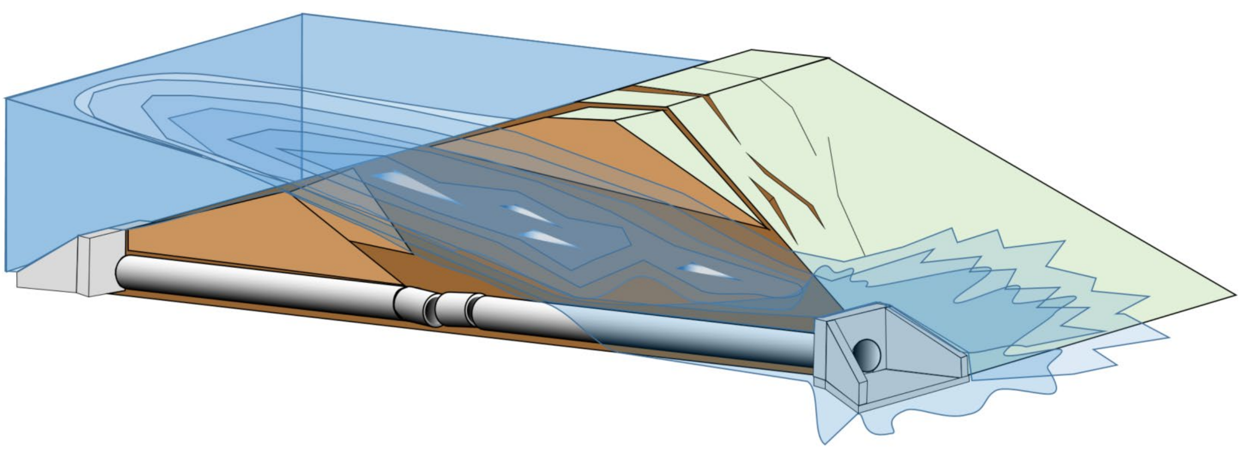


Figure 2-30. Increased soil removal rate causes embankment failures and eventual breach.

2.3.4.4.2. Proactive Measures. Routinely walking the embankment surface above pipes, particularly large-diameter pipes, may reveal indications of an underlying sinkhole, such as depressions or isolated areas of weak soil. Regularly assessing pipe inspection videos is a proactive measure which may reveal trends in joint movement, excessive separation of joints/connections, soil accumulation from infiltration, or staining of the joint/connection, alerting the inspector that PFM-3 has the potential to initiate or may have already initiated.

2.3.4.4.3. Intervention. The method of intervention is dependent on which embankment face the void daylights from and if it occurs during a flood event. For levees, sinkholes that open up on either side during non-flood periods (Figure 2-23) can be addressed by a repair, not intervention. Sinkholes that open up on the waterside of an embankment while hydraulically loaded typically require immediate intervention such as dumping appropriately-sized aggregate down the throat of the void to bridge the pipe opening, after which smaller aggregate or specialty products can be placed to further reduce or stop the flow. Intervention on the downstream or landside to contain active flow may include building a containment dike (Figure 2-61); although this type of intervention has been used successfully, it is unlikely to be successful on pipes larger than 36 inches in diameter or with high hydraulic heads.

2.3.4.4.4. Supplemental Failure Mode Description. Soil unraveling into unpressurized pipes running adjacent to a levee (e.g., toe drains [Figure 2-31], relief well collectors [Figure 2-32 and Figure 2-33], or storm water interceptors) also provides opportunities for embankment failure using the same soil removal mechanism. Instead of forming a void beneath the embankment surface, the soil lost near the landside toe either creates successive slope failures on the embankment until the crest is lowered or undermines the floodwall foundation (Figure 2-34 through Figure 2-39), or, in cases where a clay blanket overlies an aquifer, a hole is created in the blanket providing an unfiltered exit (Figure 2-40 through Figure 2-46).

(Courtesy of USACE Louisville District)

Figure 2-31. Collapsed toe drain accepting soil infiltration.

(Courtesy of USACE St. Louis District)
Figure 2-32. Soil loss from compromised relief well collector pipe connection to pump station.

(Courtesy of USACE St. Louis District)
Figure 2-33. Continued soil loss from compromised relief well collector pipe from Figure 2-32.

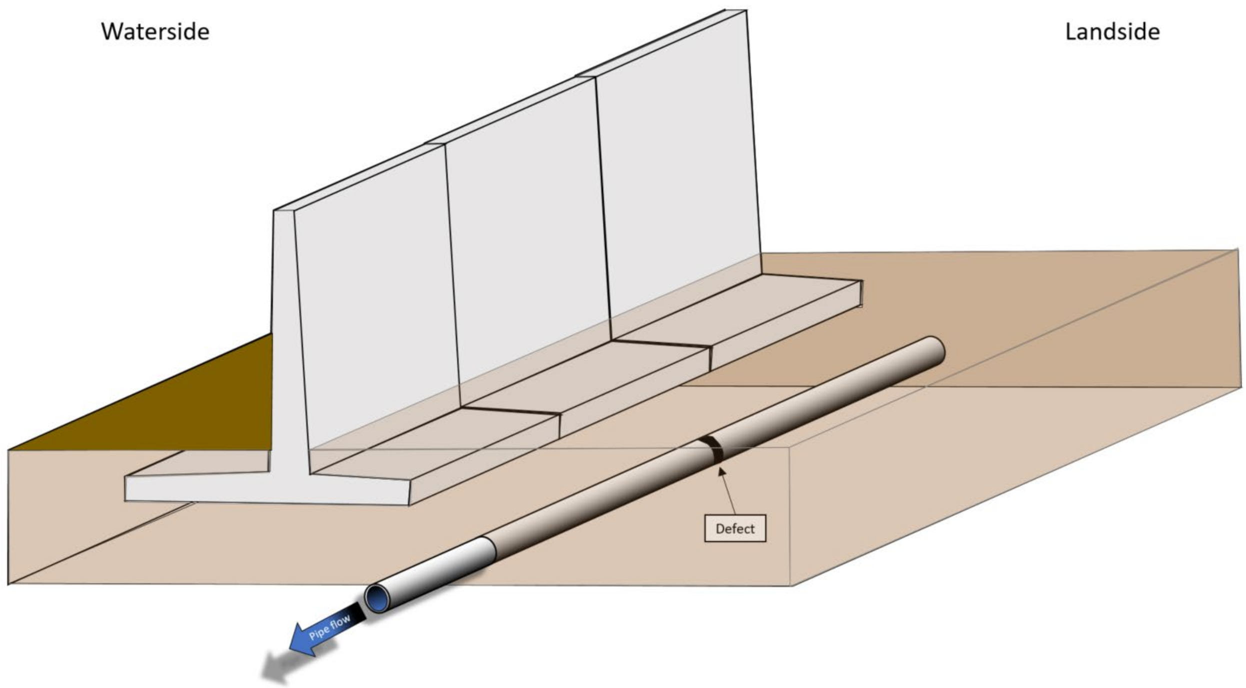


Figure 2-34. A pipe defect in the floodwall toe drain develops during installation or over time.

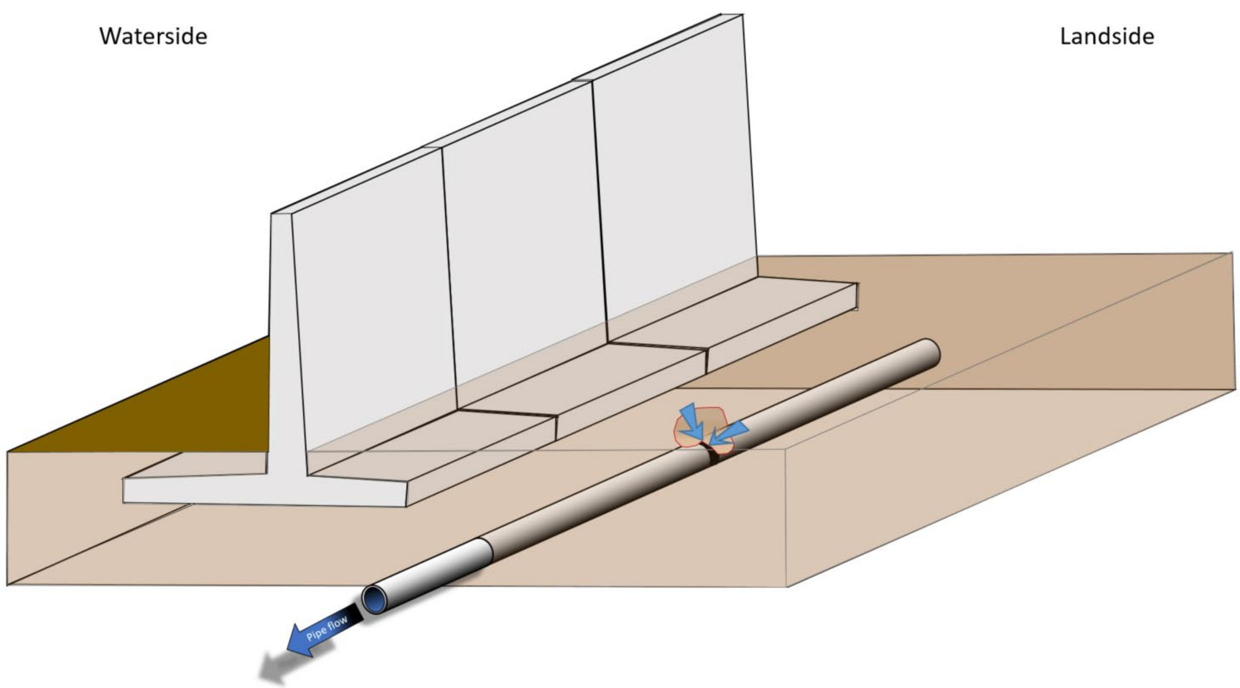


Figure 2-35. The surrounding soil infiltrates the toe drain defect and is carried away.

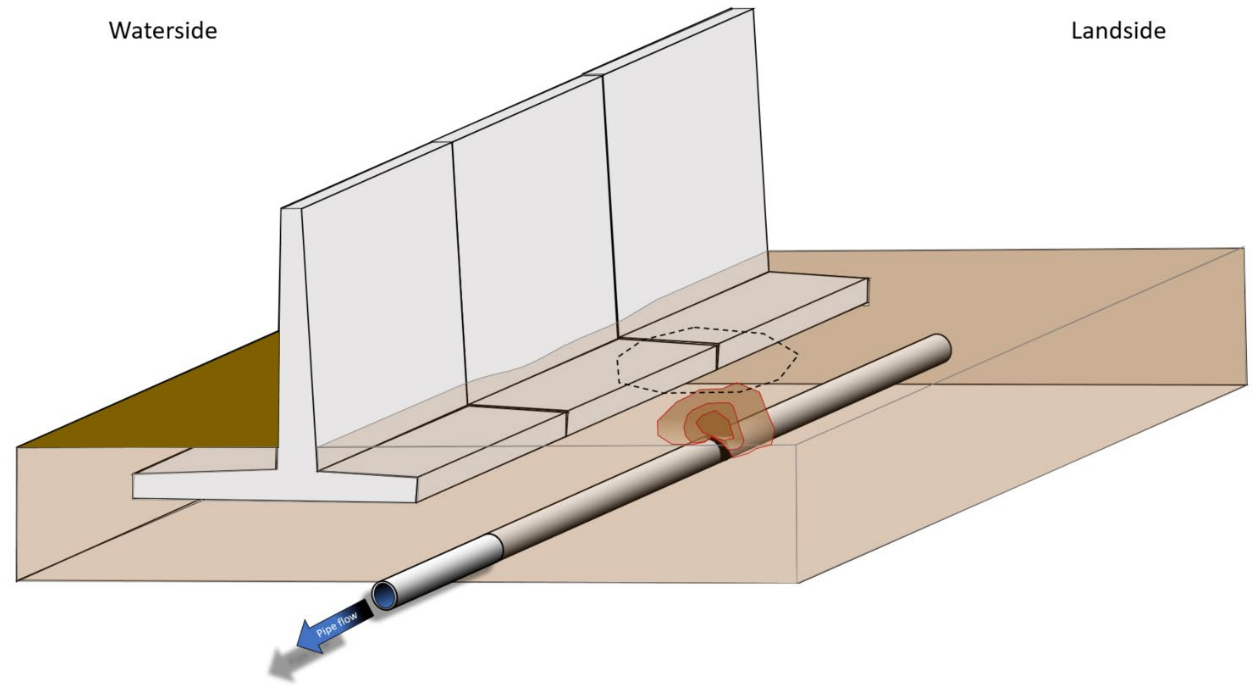


Figure 2-36. Continued soil loss removes supporting soil below and next to floodwall monoliths.

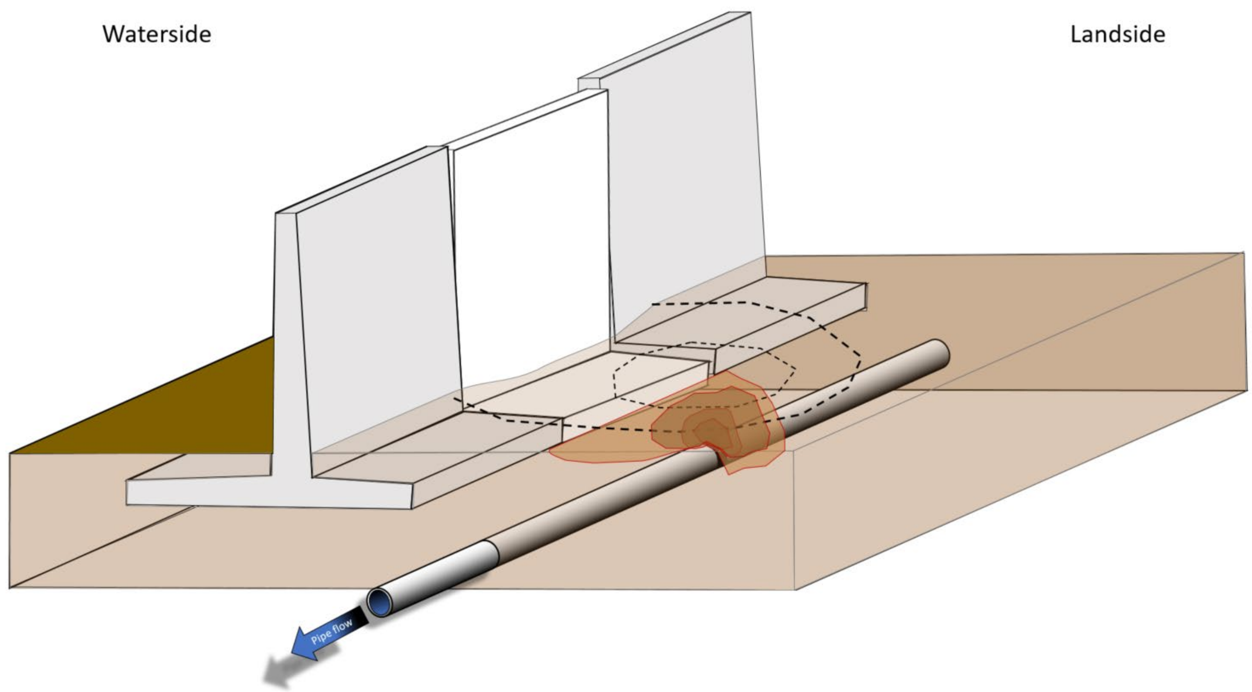


Figure 2-37. The tilting of the monoliths lowers the crest and/or forms openings in the wall.

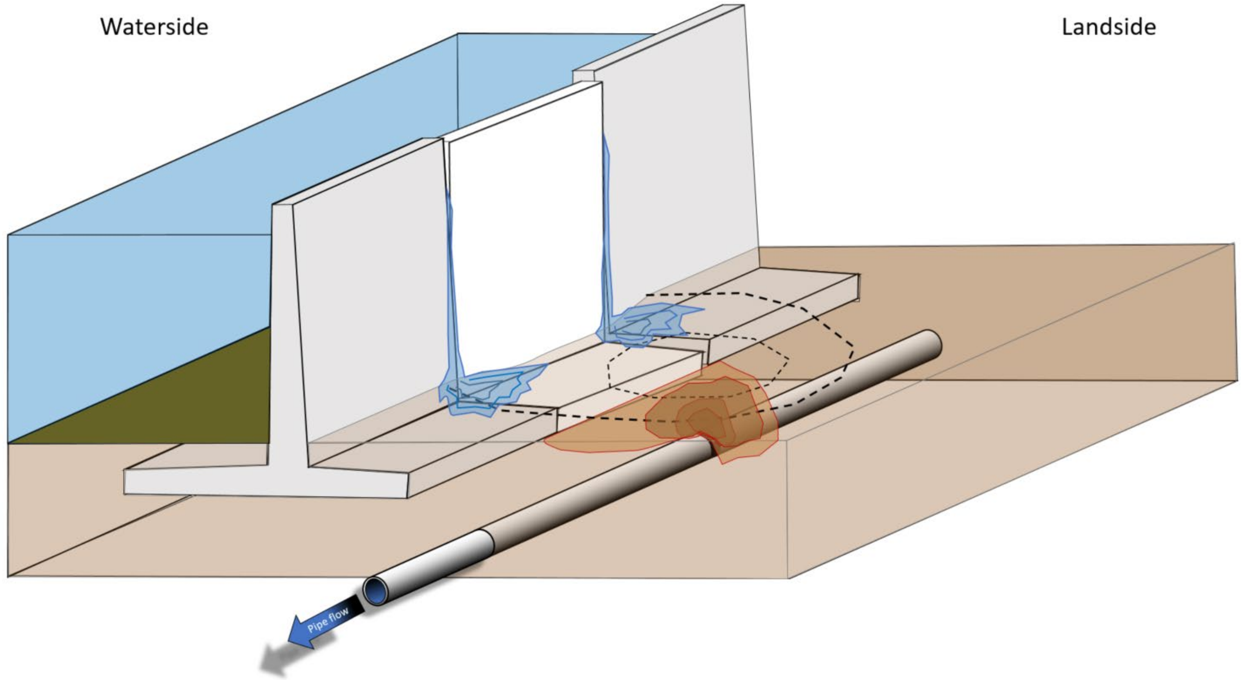


Figure 2-38. A flood loading accelerates the monolith tilting and initiates uncontrolled flow.

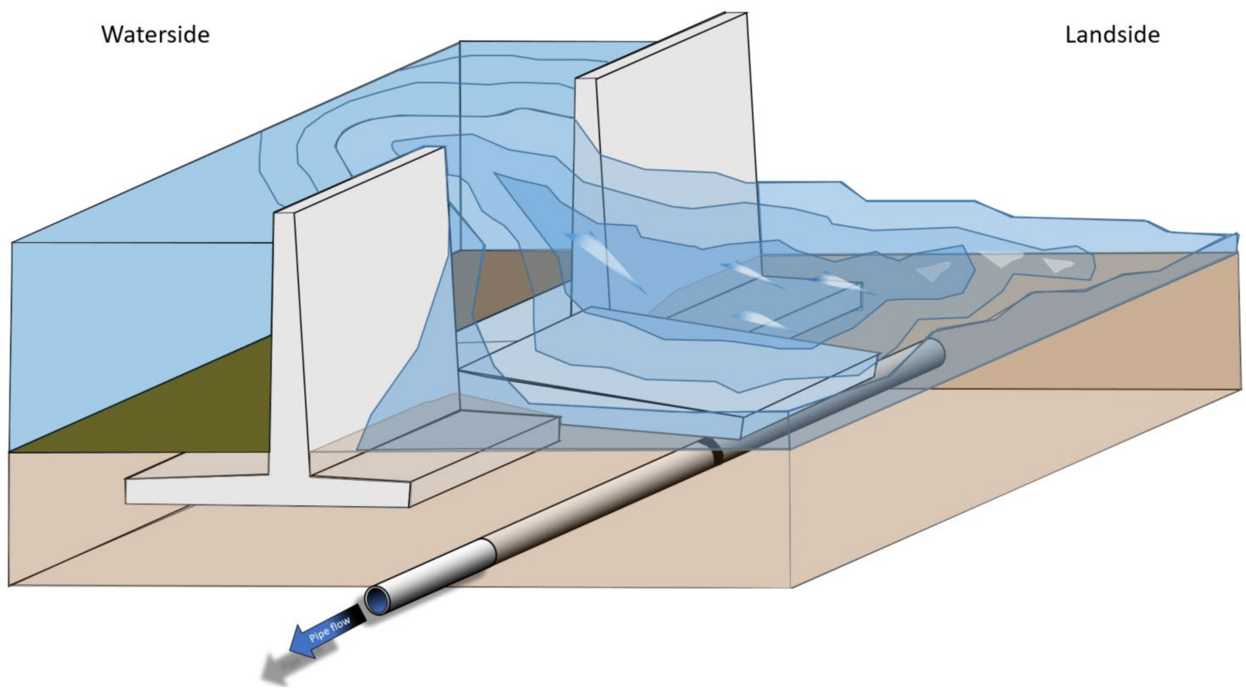


Figure 2-39. The monolith movement continues until the floodwall is breached.

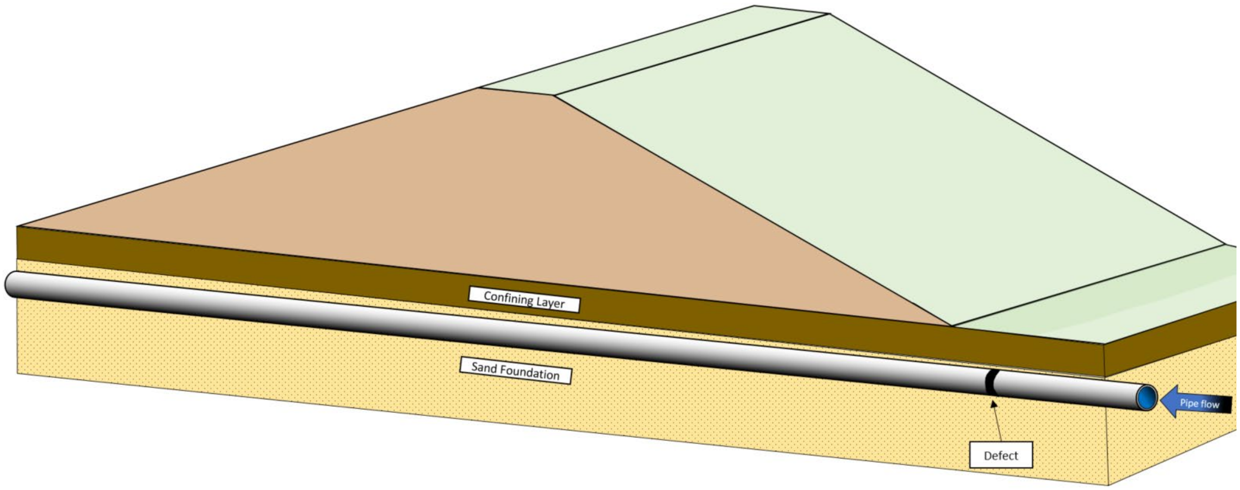


Figure 2-40. A pipe defect in the drain develops during installation or over time.

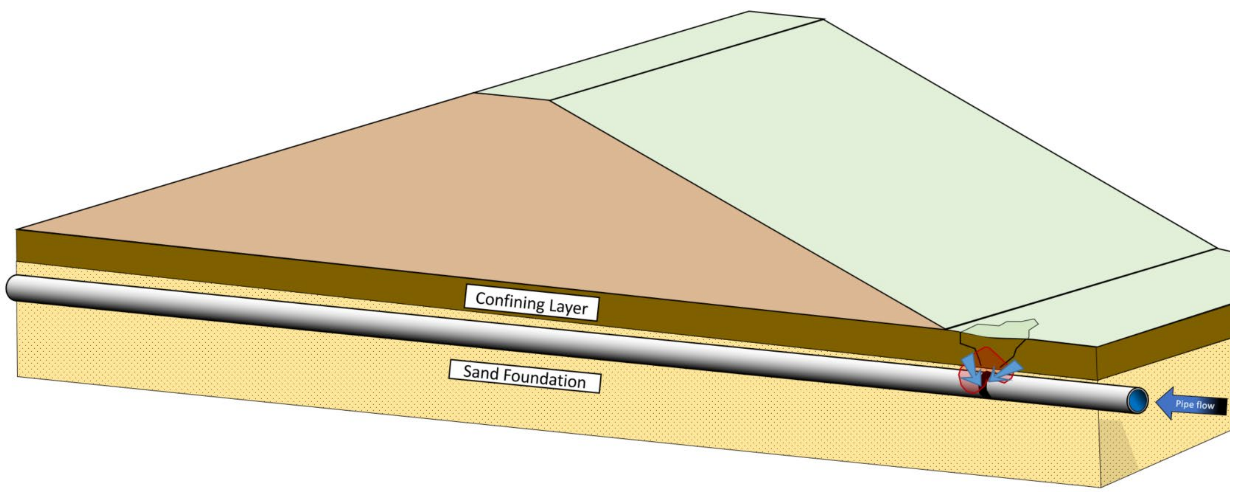


Figure 2-41. The defect allows soil infiltration that compromises the clay-confining layer.

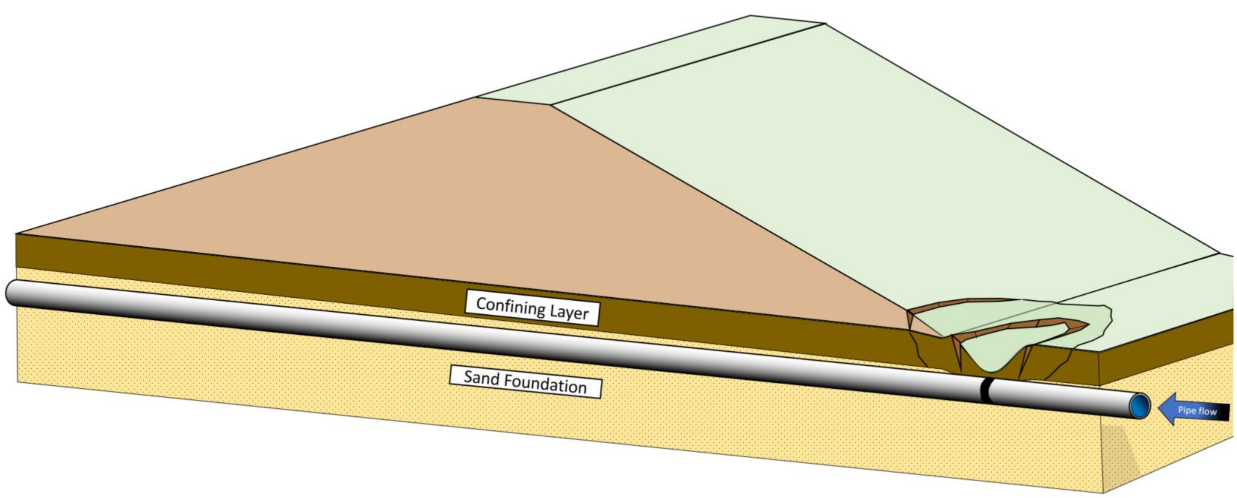


Figure 2-42. The compromised confining layer provides an unfiltered exit for the sand layer.

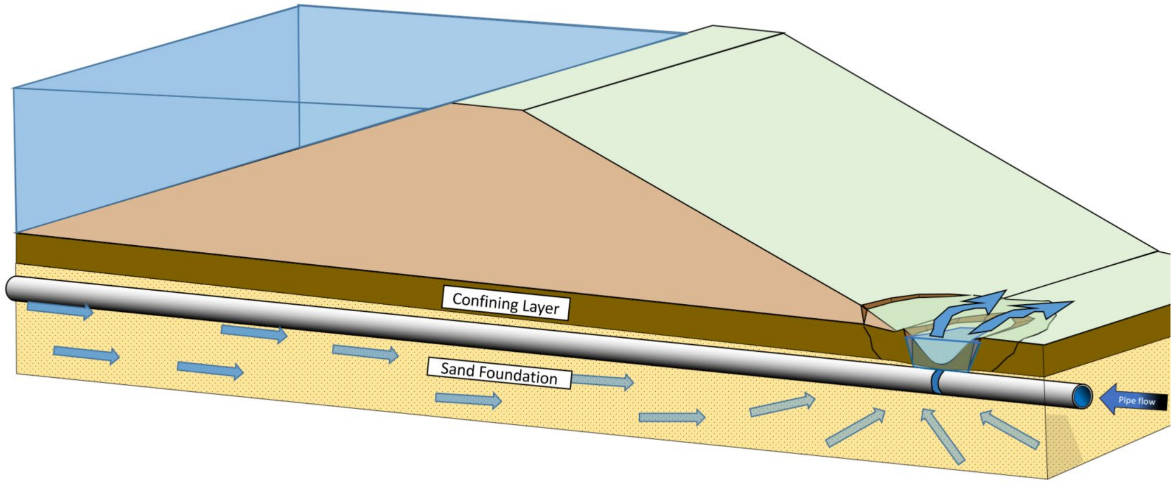


Figure 2-43. Flood event charges the foundation and creates flow through the confining layer.

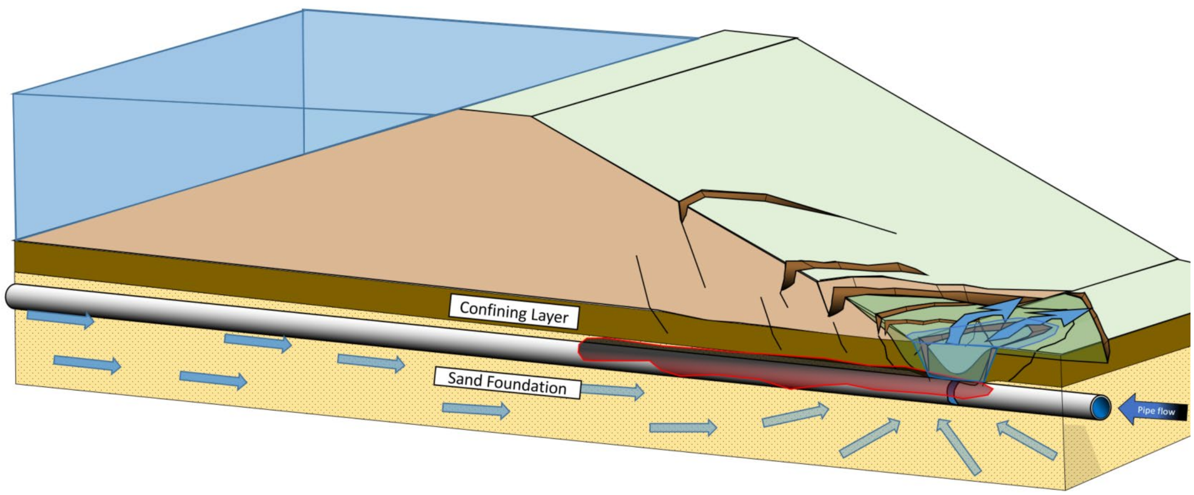


Figure 2-44. The uncontrolled flow removes soil along the pipe and deforms the embankment.

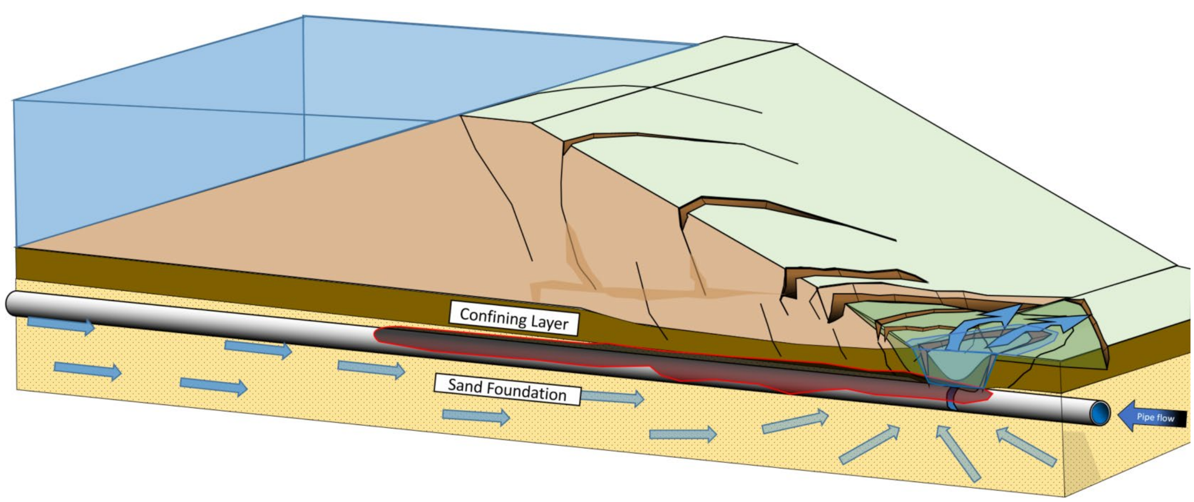


Figure 2-45. The continued loss of soil deforms the embankment until the crest is lowered.

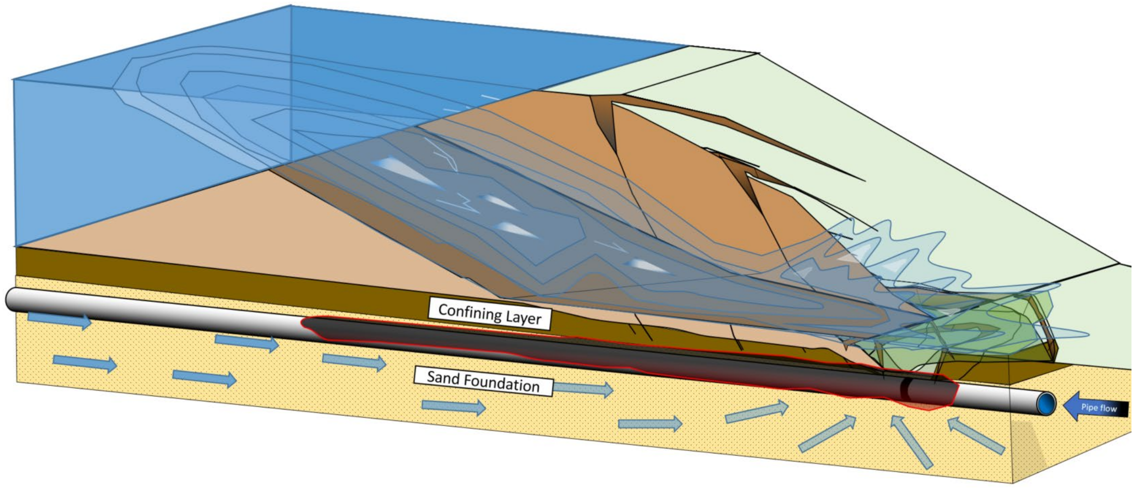


Figure 2-46. The lowered crest allows overtopping, leading to breach.

2.3.4.5. PFM-4 – External Erosion at the Pipe Outlet.

2.3.4.5.1. Failure Mode Description. Even if it does not occur within the embankment, erosion initiating around the pipe outlet can lead to embankment failure. Improperly designed and/or installed riprap or other revetment may allow water exiting the pipe to remove material at the embankment toe, creating progressive slope failures that can eventually work their way back to the crest. The crest can then collapse, allowing water to overtop the embankment, or a hydraulic loading could destabilize the remaining embankment section. The progression to failure is typically slow, and successful intervention is possible if detected prior to a flood loading (Figure 2-47 through Figure 2-53).

(Courtesy of USACE Louisville District)

Figure 2-47. Drainage pipe outlet erosion.

(Courtesy of USACE Louisville District)

Figure 2-48. Result of successive slope failures around pipes.

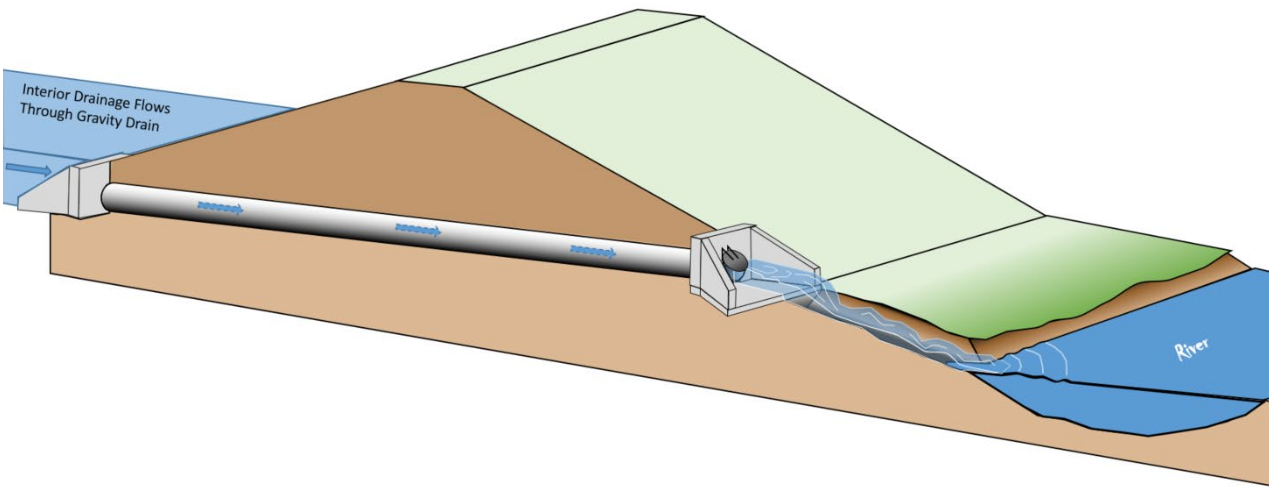


Figure 2-49. Lack of erosion control allows an erosion channel to form during releases.

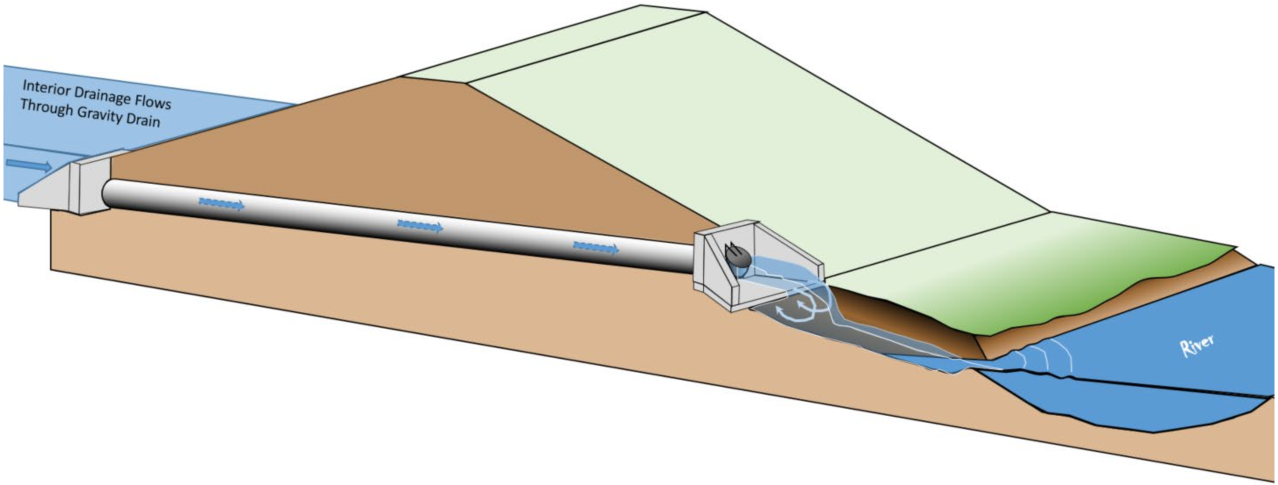


Figure 2-50. Continued erosion eventually begins to undermine the outlet structure.

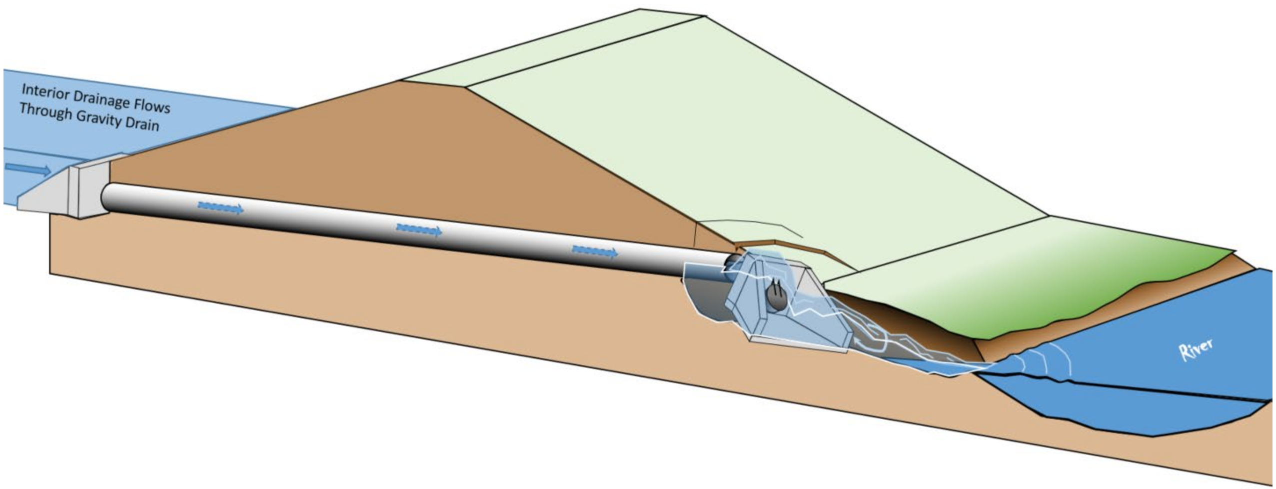


Figure 2-51. Excessive erosion destabilizes the headwall and causes a loss of outlet control.

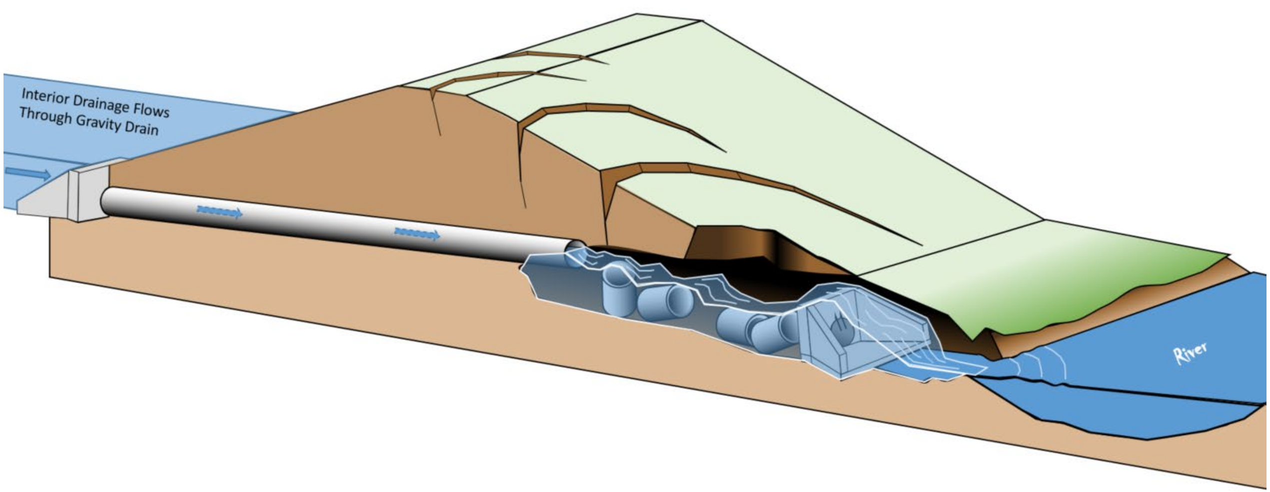


Figure 2-52. Continued erosion increases distress and alters the embankment geometry.

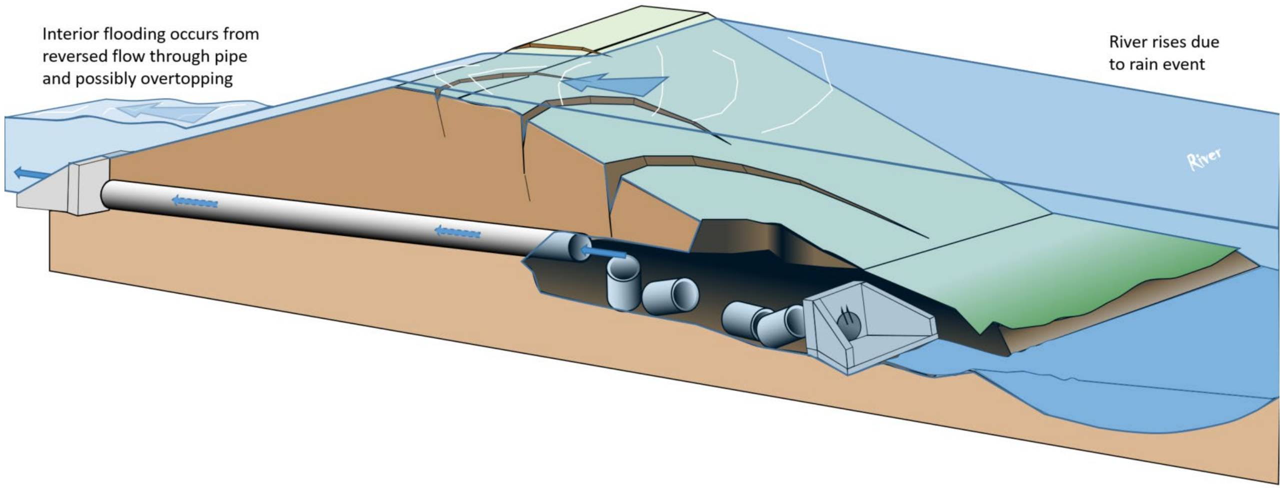


Figure 2-53. A lack of outlet control and lowered crest allow interior flooding.

2.3.4.5.2. Proactive Measures and Intervention. Regular inspections will reveal erosion, and the rating and associated narrative should indicate the urgency of the repair. Actions to correct deficiencies during normal water levels may be as simple as light regrading and riprap placement below outlet structures with minor erosion, whereas more advanced cases may require excavation by heavy equipment and outlet structure/partial pipe reconstruction/installation. Routine maintenance can typically prevent damage from this failure mechanism well before advanced stages are reached. If during a flood event there has been a dislodged headwall or gate, heroic intervention (i.e., crisis management) should include efforts to block both the waterside and landside ends of the pipe to stop flow and prevent pressurizing the pipe. In anticipation of the need for such intervention, the inlet headwall could be designed to incorporate slots for the placement of stoplogs or bulkheads.

2.3.5. Pipe-specific Potential Failure Modes Leading to Inundation without Breach.

2.3.5.1. Introduction. Not all PFMs result in breach. The following PFMs do not involve a breach but could still lead to interior ponding extensive enough to incur consequences; this would likely be limited to property damages (e.g., flooded homes, cars, etc.), but loss of life is also possible. PFMs resulting in inundation without breach include: PFM-5: Internal Restriction Causes Interior Ponding; PFM-6: External Restriction Causes Interior Ponding; and PFM-7: Appurtenance Malfunction Causes Interior Ponding/Flooding.

2.3.5.2. PFM-5 – Internal Restriction Causes Interior Ponding.

2.3.5.2.1. Description. PFM-5 is initiated by restricted flow through a pipe due to deformation (Figure 2-54), internal accumulation of debris (Figure 2-55) including ice buildup and/or root intrusions, or a combination of these causing landside ponding. Pipes may deform from installation-related distress such as passes of heavy equipment over the pipe before the minimum depth of soil cover was placed, or from being under-designed for the applied loads. Deformation may also occur if the pipe's hoop strength has been compromised by deterioration of the pipe wall, especially in metal pipes, decreasing the cross-sectional area and restricting flow.

(From Trenchless Technology Magazine, 2019)
Figure 2-54. Pipe deformation that could restrict flow and cause interior ponding.

(Courtesy of Lizotte Solutions)
Figure 2-55. Internal debris accumulation severely restricting flow.

2.3.5.2.2. Proactive Measures and Intervention. Proactive measures prior to a rainfall event include removal of debris within the drainage area and the installation of trash racks/debris screens near the pipe entrance. For cases where there is no debris screen or it is ineffective, removal of trapped debris from within a pipe (Figure 2-55) during a high water event is essentially impossible, and the only viable intervention technique is the use of portable pumps to remove interior water. Although relatively simple to deploy and operate, pumps require considerable energy and can create a fiscal burden in periods of extended operations, so the costs of proactive measures are likely justified in debris-heavy drainage areas.

2.3.5.3. PFM-6 – External Restriction Causes Interior Ponding.

2.3.5.3.1. Description. The purpose of a gravity drain through a levee is to allow water to freely exit the leveed area and prevent ponding. If debris blocks the landside entrance into a pipe, the flow of water is reduced or may even be prevented (Figure 2-56). The likelihood of excessive debris accumulating in the pipe entrance depends on the upstream environment, rainfall intensity, grade of the drainage area, and ability to remove the debris during an event. A heavy rainfall in a remote location with little chance of intervention is un-concerning if there is no debris upstream. Likewise, an abundance of debris may not be a concern if the project is in an area with sparse marginal rainfall that cannot mobilize the debris.

(Courtesy of Forest Hills CONNECTION, Marlene Berlin)

Figure 2-56. External debris accumulation severely restricting flow.

2.3.5.3.2. Proactive Measures and Intervention. Proactive measures prior to a rainfall event include removal of debris within the drainage area, removal of debris deposited near the pipe entrance from previous events, the installation of debris screens (Figure 2-57 and Figure 2-58), and oversizing the drainage feature for a reasonable degree of blockage. Intervention during a rainfall event involves attempting to remove accumulated debris at the pipe entrance so the area can freely drain. Sturdy grates installed flush with the headwall profile allow for cleaning at any time. Oversaturation and remote locations may make heavy machinery and successful intervention infeasible. In many cases portable pumps can be mobilized to the affected area to remove the water.

(From SPEC-NET, 2017)
Figure 2-57. Fabricated debris screen for poured-in-place headwall.

(From HYNDS Pipe Systems, 2019)
Figure 2-58. Precast headwall with matching contoured debris screen.

2.3.5.4. PFM-7 – Appurtenance Malfunction Causes Interior Ponding/Flooding.

2.3.5.4.1. Description. The appurtenances most likely to cause interior ponding or flooding are gates. Flap, slide, and other gates can become seized in various positions and either allow water to flow into the leveed area during a flood or prevent or hinder normal drainage. The standard operating condition for gates within a levee setting is in an open or raised position that allows the free release of water from the interior (leveed area). When seized open, they cannot prevent floodwater from entering the leveed area, and when seized closed they will prevent drainage and cause interior ponding.

2.3.5.4.2. Proactive Measures and Intervention. Regular inspections and proper lubrication of the gate mechanisms are proactive measures to preserve gate operability. Intervention in the case of floodwater entering the leveed area through an open pipe could be accomplished by dropping sandbags in front of the flap gate or down a gatewell in front of a slide gate (Figure 2-59 and Figure 2-60). Portable pumps could then be deployed to remove any seepage or ponded interior water. When waterside intervention is not possible or practical, a landside containment berm may successfully confine the ponding to a smaller area (Figure 2-61).

(Courtesy of USACE Louisville District)

Figure 2-59. Sandbags in front of flap gate.

(Courtesy of USACE Louisville District)

Figure 2-60. Sandbags in front of large-diameter slide gate.

(Courtesy of USACE St. Louis District)

Figure 2-61. Landside intervention using an aggregate berm to retain flow of 36-inch pipe.

Chapter 3
Selection

3.1. Introduction. This chapter describes the pipe selection process by detailing the strengths and weaknesses of various pipe materials and their associated joints. Selecting the most appropriate pipe material for a specific environment is an essential step in reducing the risk associated with the long-term performance of a pipe. Common pipe materials are described herein, along with a table listing factors to consider when selecting a new pipe or evaluating an existing one. The steel and concrete material types are broad and contain several more specific descriptive categories.

3.2. Potential Failure Modes Related to Pipe Selection. Selecting a pipe material without careful consideration of the type of fluid being conveyed, surrounding environment, bedload, earth pressures, frequency of use, and other factors may increase the probability of failure. Poor pipe selection may not only compromise the longevity of the pipe, but the stability of the associated embankment or floodwall. Choosing an inappropriate pipe material for the surrounding environment, such as placing an uncoated aluminum pipe within a cementitious backfill material or using steel pipe in a saline environment, can lead to accelerated corrosion and defects that can initiate a PFM. Generally, performance issues include internal erosion within or beneath an embankment or floodwall from a defect in a pressurized pipe (PFM-2) or internal erosion into a pipe through a defective connection or area of perforation through the pipe (PFM-3).

3.3. Pipe Materials.

3.3.1. General. For the purposes of structural design, all pipes are classified as either rigid or flexible. Rigid pipes (i.e., concrete, cast iron, and vitrified clay) are designed to withstand loads with low reliance on structural support from the surrounding soil; however, selecting a rigid pipe does not reduce the importance of following the proper backfilling techniques described in Chapter 5. Flexible pipes (i.e., corrugated steel, corrugated aluminum, ductile iron, plastic, welded seam steel, and fiberglass) are designed to mobilize the surrounding soil to help share the load acting on the pipe and can deflect more than two percent of the pipe diameter without structural distress. Adequate soil support along the sides of a flexible pipe is necessary to develop the required lateral passive support; therefore, proper backfill materials and placement methods are critical when selecting flexible pipes. Insufficient lateral support can lead to pipe collapse which can then lead to ruptured joints or barrels and the initiation of PFM-2 or PFM-3, finally leading to embankment and/or foundation failure.

3.3.2. Cast-in-Place Concrete Pipe (CiPCP). The use of CiPCP is typically limited to embankment dam outlet works, but it is occasionally used in levees when precast concrete sections are not a viable option or when reinforced concrete box (RCB) pipes are desired. CiPCP is typically reserved for high overburden loads or situations where substantial flow volumes are required, and cross-sectional area requirements are larger than what is available in precast sizes. When RCB is specified, attention to joint detail is critical to ensure no soil intrusion will be possible. When CiPCP is utilized, it must follow the guidelines and requirements outlined in EM 1110-2-2104.

3.3.3. Precast Reinforced Concrete Pipe (RCP).

3.3.3.1. General. Precast reinforced concrete pipe, commonly referred to as RCP, comes in a variety of sizes and shapes and is widely used in levee applications with a history of good performance when properly constructed. Precast RCP is manufactured in a controlled plant environment allowing for consistency and high construction quality and is typically manufactured in 8-foot segment lengths (Figure 3-1). Precast RCP also comes in box culvert shapes up to the size limitations noted in Table 3-3. Non-reinforced precast concrete pipe is never allowed in USACE embankment or floodwall applications due to its brittleness and lack of crack control.

(Courtesy of American Concrete Pipe Association)
Figure 3-1. Installation of precast RCP.

3.3.3.2. Non-Pressure and Low-Pressure Reinforced Concrete Pipe. Non-pressure and low-pressure RCP pipes are manufactured with reinforcement (i.e., bars, not cylinders), and include the sub-classifications listed below. It is important to note that the internal pressure listed is associated with the body of the pipe itself from laboratory or manufacturing plant tests and is not applicable to field test pressures. The appropriate type of joint must be specified to ensure the given design pressures are applicable for the entire pipe/joint configuration and also meet the in-field (reference Table 5-5) post-installation testing requirements. Reference Chapter 5 for more information regarding applicable pipe joint types by pipe material.

3.3.3.2.1. Round non-pressure RCP. Round non-pressure RCP is manufactured according to ASTM C76. Although they are classified as non-pressure, these pipes are acceptable for internal loadings up to 25 feet of head (10.8 psi).

3.3.3.2.2. Arch and elliptical non-pressure RCP (NP-RCP). Arch and elliptical non-pressure RCP (NP-RCP) are manufactured according to ASTM C506 (arch) and ASTM C507 (elliptical). These pipes are acceptable for internal pressure up to 25 feet of head (10.8 psi). Not all RCP producers supply arch and elliptical pipe that will accommodate a rubber gasket for required joint leakage criteria.

3.3.3.2.3. Low-pressure RCP (LP-RCP). Low-pressure RCP (LP-RCP) is manufactured according to ASTM C361 or American Water Works Association (AWWA) C302. ASTM C361 and AWWA C302 are considered interchangeable standards for LP-RCP. LP-RCP can be designed for internal pressures up to 125 feet of head (54.2 psi).

3.3.3.2.4. Reinforced Concrete Box Use in Levee and Dam Applications. If RCB is to be used for a levee or dam application, it must be of CiPCP construction with specialized design considerations for the joints; pre-cast RCB is not permitted because their joints have a history of leakage.

3.3.3.3. Concrete Pressure Pipe (CPP). CPP can be designed for a variety of internal pressures and must be specified where concrete pipe is desired and internal pressures will exceed 125 feet of head (54.2 psi). To ensure the pipe can handle large internal pressures, precast sections are further reinforced through a variety of methods, including the use of steel cylinders and steel bar wrapping. The following concrete pressure pipes are commonly used within the United States: PCCP manufactured according to AWWA C301; reinforced concrete cylinder pipe (RCCP) manufactured according to AWWA C300; reinforced non-cylinder pipe (RCNP) manufactured according to AWWA C302; and bar-wrapped cylinder pipe (BWCP) manufactured according to AWWA C303. BWCP is the only concrete pipe that is considered semi-rigid.

3.3.4. Vitrified Clay Pipe (VCP). Early versions of clay pipe (terra cotta) differ greatly from the higher quality VCP currently manufactured according to ASTM C700 (Figure 3-2). This is due to the process of vitrification, which is the progressive reduction of porosity of a ceramic composition resulting in a significant increase in density and mechanical bonding of the pipe's particles. VCP joint design has also improved with the advent of flexible compression joints. VCP is only applicable for non-pressurized environments because of its joint limitations with respect to pressurization. It is important to note that VCP has not routinely been used in USACE dams and levees since it was not considered a viable pipe material in the previous version of this manual; therefore, it does not have the same history of service as other pipe materials. While this does not preclude its use, it is a factor the designer should be aware of.

(Courtesy of National Clay Pipe Institute)
Figure 3-2. 30-inch VCP installation.

3.3.5. Corrugated Metal Pipe (CMP). are typically two types of CMPs used in USACE Civil Works Projects. The more common type is galvanized corrugated steel pipe (CSP), which is generally referred to as CMP in design plans and documents; the other is corrugated aluminum pipe (CAP).

3.3.5.1. Corrugated Steel Pipe (CSP). CSP corrugations in conjunction with a properly placed surrounding soil mass provide the structural stiffness that makes CSP a potentially viable option as a drainage structure within a levee or as part of an underseepage control system (i.e., toe drain, seepage collector); however, CSP should only be used for non-pressurized applications. CSP is normally galvanized with a zinc coating to improve corrosion resistance (Figure 3-3), but can also have an aluminized Type 2 coating or polymer coating to help extend the service life in some operating environments. CSP is manufactured according to ASTM A760 and ASTM A796. Corrugated steel box pipes are manufactured according to ASTM A964. A corrugated steel box culvert does not have a true box or rectangular shape (Figure 3-4); it is essentially a corrugated steel frame in which the pipe walls are structurally connected to a foundation to transfer load to the surrounding soil.

(Courtesy of National Corrugated Steel Pipe Association)
Figure 3-3. Large CSP at manufacturing facility.

(Courtesy of National Corrugated Steel Pipe Association)
Figure 3-4. Installation of a corrugated steel box culvert.

3.3.5.2. Corrugated Aluminum Pipe (CAP). CAP is similar to CSP with respect to available sizes, shapes, construction methodologies, and use in non-pressurized situations (Figure 3-5). However, while aluminum pipe is lighter and more corrosion resistant than steel, it is typically more susceptible to abrasive deterioration and historically has been more expensive than CSP. ASTM B745 is the governing specification for CAP. Corrugated aluminum box pipes are manufactured according to ASTM B864.

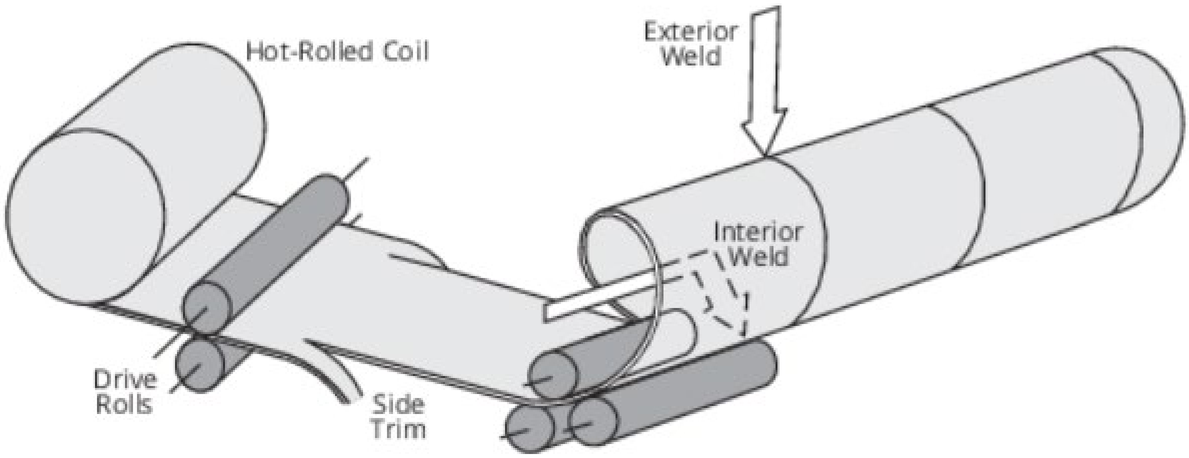
(Courtesy of the Aluminum Association)
Figure 3-5. Installation of a corrugated aluminum culvert.

3.3.6. Smooth Steel Pipe.

3.3.6.1. General. Smooth steel pipe is manufactured by two distinct processes which result in either a seamless (billet) or welded seam steel pipe (WSSP). Spiral-wound-seam and straight-seam are the two commonly manufactured sub-sets of WSSP. AWWA C200, ASTM A53, and ASTM A135 are the governing specifications for the manufacture of seamless and welded steel pipe.

3.3.6.2. Seamless (Billet) Steel Pipe. Seamless, smooth steel pipe starts with a solid round steel billet that is heated and then pierced with a plug. The hollow shell is rolled by a mandrel mill to reduce the outside diameter and wall thickness. The piece is reheated and further reduced to the specified dimensions using a stretch reducer while the rolling process lengthens the sections which are eventually cut to the desired length. Seamless steel pipe is typically only specified for high pressure applications, and, due to the high cost of manufacturing, is only cost competitive for small diameters (less than eight inches) and is not widely used in USACE dam or levee projects.

3.3.6.3. Spiral Welded Seam Steel Pipe. This pipe, also referred to as spiral wound steel pipe, is manufactured using a variety of methods, the most popular of which uses steel coils to form round pipe sections that are continuously welded along a seam. This is the most common method because of the nearly uninterrupted production and ease of creating specific lengths. Both the interior and exterior weld seams are smoothed during the manufacturing process to improve handling safety and flow resistance characteristics. This manufacturing process is depicted in Figure 3-6, with a completed spiral WSSP shown in Figure 3-7.



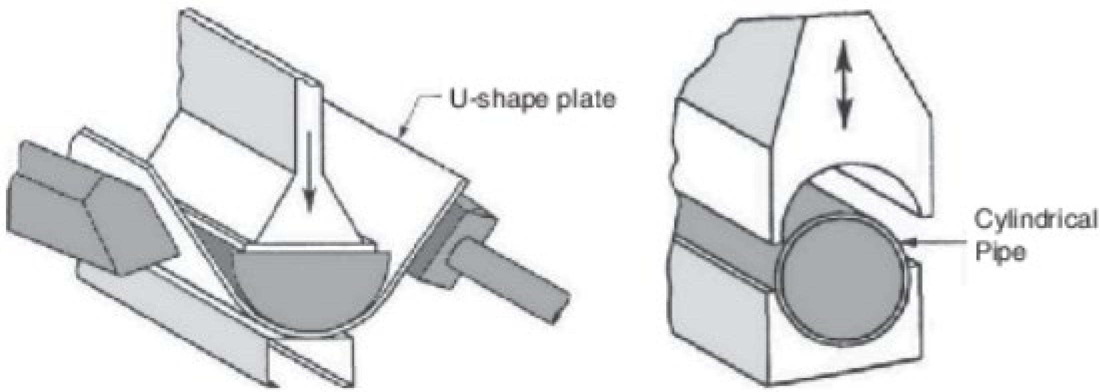
(Courtesy of AWWA)

Figure 3-6. Schematic of spiral welded seam steel pipe manufacturing process.

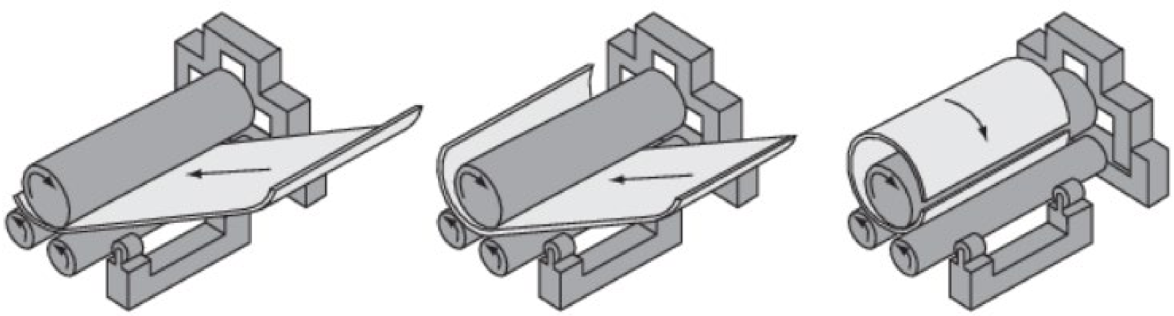
(Courtesy of National Association of Steel Pipe Distributors)

Figure 3-7. Completed spiral welded seam steel pipe.

3.3.6.4. Straight Welded Seam Steel Pipe. There are two basic manufacturing processes for constructing straight WSSP. The first uses a series of crimping rollers and presses (Figure 3-8) to form the basic circular shape. The pipe is then fed directly into a longitudinal seam welding machine. The second process uses a series of pyramid-style rollers to form the circular shape (Figure 3-9) that is welded longitudinally along the length of the pipe. A limiting factor is the length of the steel sheets used in the manufacturing process.



(Courtesy of AWWA)
Figure 3-8. Straight WSSP manufactured with presses.



(Courtesy of AWWA)
Figure 3-9. Straight WSSP manufactured with pyramid rollers.

3.3.7. Ductile Iron Pipe (DIP). DIP, typically used for water distribution systems, wastewater collection, and force main systems, became the successor of cast iron pipe when the manufacturing process was altered to include a magnesium treatment, making it stronger and lighter. A one-mil (0.001 inch) layer of seal coat is applied to the outer surface for corrosion protection, resulting in a finished product as shown in Figure 3-10. Figure 3-11 shows the installation of DIP with an enhanced polyethylene encasement for additional corrosion control in aggressive soil environments. The American National Standards Institute (ANSI)/AWWA C151/A21.51 is the governing standard for pressurized DIP. ASTM A746 is the governing specification for gravity flow DIP.

(Courtesy of Ductile Iron Pipe Research Association)
Figure 3-10. Finished DIP at manufacturing facility.

(Courtesy of Ductile Iron Pipe Research Association)
Figure 3-11. Installation of DIP encased in polyethylene.

3.3.8. Plastic Pipe.

3.3.8.1. General. Plastic pipes typically used within or beneath an embankment or floodwall project are manufactured with a thermoplastic material. There are some instances where thermoset (cured-in-place) plastic material is a viable option for slip lining as a rehabilitation method for existing deteriorated pipes (reference Chapter 7); however, only thermoplastic materials are considered applicable for new direct bury applications.

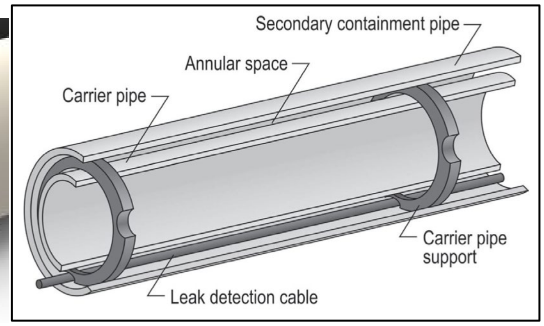
3.3.8.2. Thermoplastics. Thermoplastic materials can be repeatedly softened and hardened by heating and cooling without damaging their engineering properties. The most commonly used thermoplastic pipes associated with embankment and floodwall applications are high density polyethylene (HDPE), polypropylene (PP), and polyvinyl chloride (PVC). Thermoplastic pipes typically fall into one of four categories: solid wall, double-wall containment, single-wall corrugated (corrugated interior/exterior surfaces), or profile-wall corrugated (smooth interior/corrugated exterior). Profile-wall pipes are also commonly referred to as dual-wall pipes because of the two different patterns used on the inside and outside surfaces. The applicable specifications covering the manufacturing of various types of thermoplastic pipes are provided in Table 3-3.

3.3.8.2.1. Solid-wall HDPE (SW-HDPE). SW-HDPE is thermoplastic pipe that has a single wall with a smooth interior and exterior surface (Figure 3-12). The smooth interior surface provides a lower Manning's roughness coefficient and therefore allows for higher flow capacity when compared to a similarly sized corrugated pipe. The higher flow capacity is the primary reason it is used frequently for slip lining damaged pipes, as its smaller diameter is offset by greater flow.

(Courtesy of Plastics Pipe Institute)

Figure 3-12. Solid-wall HDPE at manufacturing facility.

3.3.8.2.2. Double-wall Containment HDPE. Double-wall containment HDPE consists of two solid-wall pipes and is used primarily in situations where leak containment is desired for hazardous or environmentally damaging fluids. The inner (carrier) pipe is used to transmit the flow and the outer (casing) pipe is designed to contain leaks. Double-wall containment HDPE can also be used for pressurized flow situations within or beneath an embankment or floodwall if additional leak protection is desired (Figure 3-13).



(From Fusion Australia, 2019 [left] and Idaho National Laboratory, 2012 [right])
Figure 3-13. Double-wall containment HDPE pipes.

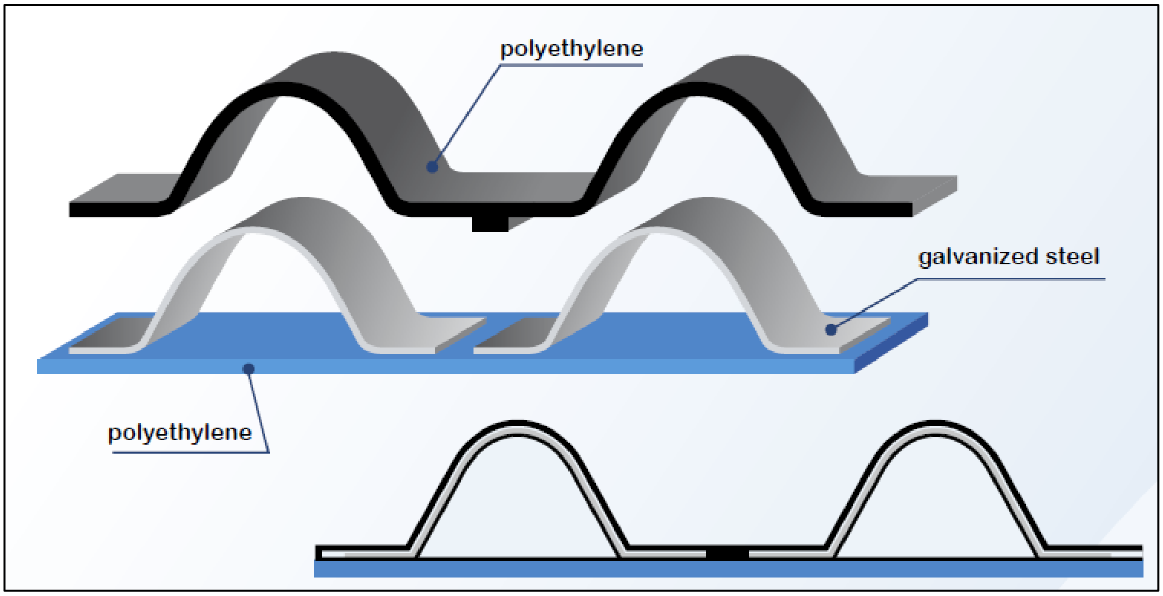
3.3.8.2.3. Single-wall Corrugated HDPE (C-HDPE). C-HDPE is corrugated both inside and out. The corrugation improves pipe stiffness and slows the flow velocity through the pipe section (Figure 3-14).

(From T.H. Rogers Lumber Company, 2019)
Figure 3-14. Single-wall corrugated C-HDPE.

3.3.8.2.4. Profile-wall (PW; also known as dual-wall) Thermoplastic Pipes (PW-HDPE, PW-PP, and PW-PVC). PW thermoplastic pipes have a corrugated exterior for added stiffness but a smooth interior surface for increased flow capacity. Figure 3-15 shows the manufacturing of a PW-HDPE pipe.

(Courtesy of Plastics Pipe Institute)
Figure 3-15. Manufacture of a PW-HDPE.

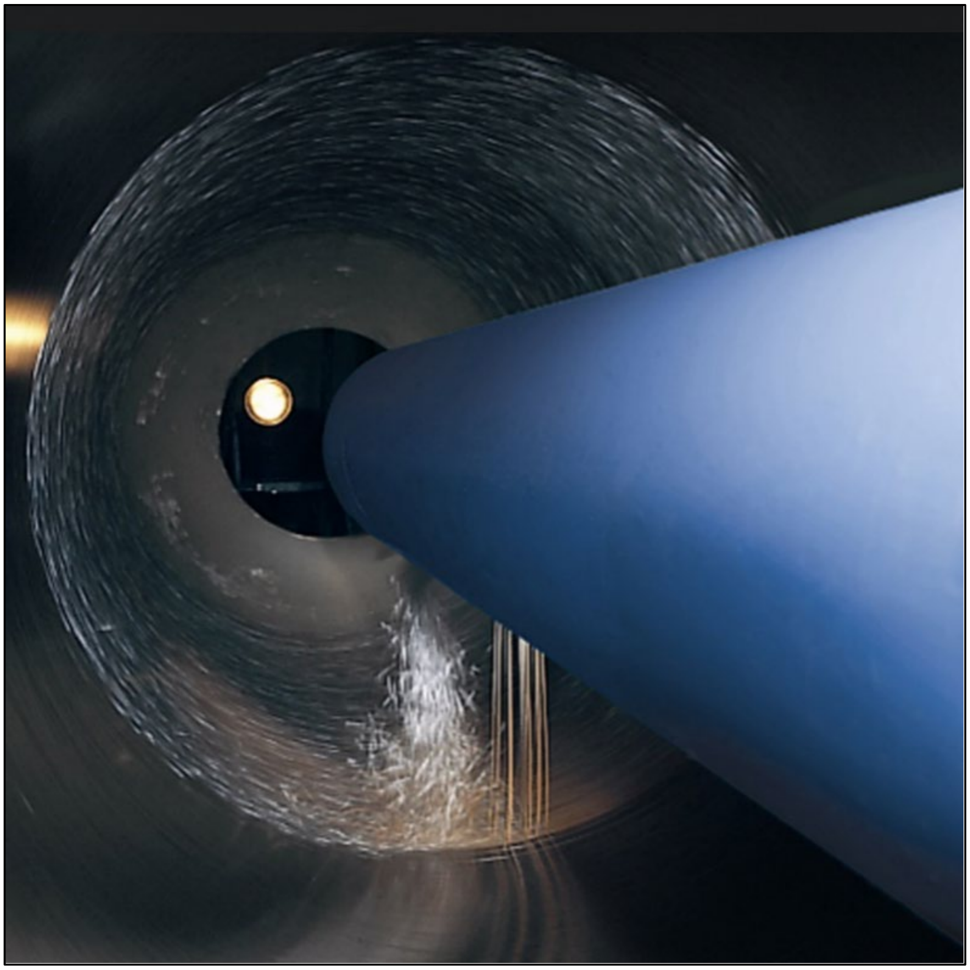
3.3.9. Steel-Reinforced Thermoplastic Pipe (SRTP). At the time of this manual's publishing, there are a limited number of SRTP pipe manufacturers in North America; however, foreign pipe manufacturing companies with U.S. suppliers can also provide this type of pipe. SRTP has many of the desirable corrosion and abrasion resistance properties of plastic pipe, with added strength and stiffness due to the inclusion of steel ribs encapsulated within the thermoplastic material. Multiple composite cross-sections are available depending upon the manufacturer; one example is depicted in Figure 3-16. The governing specification for SRTP is ASTM F2562. Because there is limited data on its long-term performance, designers considering SRTP are encouraged to review pertinent historic performance and research data that may be available through suppliers and other independent entities.



(From Paladeri SPA, 2016)

Figure 3-16. Example cross-section profile for SRTP.

3.3.10. Fiberglass Reinforced Pipe (FRP). FRP is made primarily of glass fiber reinforcements embedded in a cured thermosetting resin along with other additives such as aggregate (siliceous sand), platelet fillers, and dyes, depending upon the desired properties. The common industry terminology for this material is fiberglass pipe or glass fiber reinforced thermosetting resin pipe; however, FRP is also known as glass reinforced plastic, reinforced polymer mortar, and fiberglass reinforced epoxy, among other names. The manufacturing of FRP is governed by ASTM D3262. The production of a centrifugally-cast FRP is shown in Figure 3-17.



(Courtesy of Fiberglass Tank and Pipe Institute)
Figure 3-17. Centrifugally-cast FRP.

3.3.11. Other Pipe Materials. The pipe materials previously described cover the majority of installations within the United States, but a designer may choose to use a pipe material not covered within this manual. If so, the designer must consider all operating environment concerns for the selected pipe material and how they may relate to the PFMs discussed in Chapter 2. All pertinent design safety criteria must be met, as well as meeting a minimum service life of 100 years as outlined in Section 3.4.10. Reference Section 1.2 for guidance on requesting approval to use other pipe materials not included in this manual.

3.3.12. Engineering Performance and Characteristics. Table 3-1 provides a generalized comparison of various pipe materials for certain operating environments. Where feasible, pipe lengths should be maximized to limit the number of connections or joints to lower the risk associated with PFM-3. Reference Chapter 5 for joint details and installation procedures.

Table 3-1
Pipe material comparison (part 1 of 2)

<table>
<tr><th>Base Pipe Material[7]</th><th>Flexible or Rigid</th><th>Acceptable pH range for flow thru pipe</th><th>Acceptable Soil Resistivity Range (Ω-cm)</th><th>Acceptable Abrasion Environments[2]</th><th>Weight Range for 36" Circular Pipe (lbs./segment ft.)</th></tr>
<tr><td>Cast-in-Place Concrete (CiPCP)</td><td>Rigid</td><td>5.0 - 12.0</td><td>no limitations</td><td>Level 1, 2</td><td>custom</td></tr>
<tr><td>Non-Pressure RCP (NP-RCP)</td><td>Rigid</td><td>5.0 - 12.0</td><td>no limitations</td><td>Level 1, 2</td><td>525 -625</td></tr>
<tr><td>Low Pressure RCP (LP-RCP)</td><td>Rigid</td><td>5.0 - 12.0</td><td>no limitations</td><td>Level 1, 2</td><td>550 - 675</td></tr>
<tr><td>Concrete Pressure Pipe (CPP)[1]</td><td>See note 1</td><td>5.0 - 12.0</td><td>> 1,500 (note 4)</td><td>Level 1, 2</td><td>315 - 550</td></tr>
<tr><td>Vitrified Clay (VCP)</td><td>Rigid</td><td>no limitations</td><td>no limitations</td><td>all levels</td><td>420 - 480</td></tr>
<tr><td>Corrugated Steel (CSP - Galvanized)</td><td>Flexible</td><td>6.0 - 10.0</td><td>2,000 - 10,000</td><td rowspan="2">Level 1, 2</td><td rowspan="2">29 - 62</td></tr>
<tr><td>Corrugated Steel (CSP - Aluminized)</td><td>Flexible</td><td>4.5 - 9.0</td><td>> 1,500</td></tr>
<tr><td>Corrugated Steel (CSP - Polymer)</td><td>Flexible</td><td>3.0 - 12.0</td><td>> 250</td><td>Level 1, 2, 3</td><td>30 - 63</td></tr>
<tr><td>Corrugated Aluminum (CAP)</td><td>Flexible</td><td>4.0 - 9.0</td><td>≥ 500</td><td>Level 1, 2</td><td>11 - 20</td></tr>
<tr><td>Welded Seam Steel Pipe (WSSP)</td><td>Flexible</td><td>lining dependent[6]</td><td>coating dependent[6]</td><td>all levels</td><td>51 - 190</td></tr>
<tr><td>Ductile Iron (DIP)</td><td>Flexible</td><td colspan="2">refer to DIPRA guidelines</td><td>See note 3</td><td>150 - 275</td></tr>
<tr><td>Solid Wall HDPE (SW-HDPE)</td><td>Flexible</td><td>no limitations</td><td>no limitations</td><td>Level 1, 2, 3</td><td>53 - 148</td></tr>
<tr><td>Profile Wall (PW-HDPE, PW-PP)</td><td>Flexible</td><td>no limitations</td><td>no limitations</td><td>Level 1, 2, 3</td><td>18</td></tr>
<tr><td>Polyvinyl Chloride (PVC)</td><td>Flexible</td><td>no limitations</td><td>no limitations</td><td>Level 1, 2, 3</td><td>60 - 165</td></tr>
<tr><td>Steel Reinforced Thermoplastic (SRTP)[5]</td><td>Flexible</td><td>no limitations</td><td>no limitations</td><td>Level 1, 2, 3</td><td>30 - 50</td></tr>
<tr><td rowspan="2">Fiberglass Reinforced (FRP)</td><td rowspan="2">Flexible</td><td>1.0 - 9.0 (polyester resin)</td><td rowspan="2">no limitations</td><td rowspan="2">Level 1, 2, 3</td><td rowspan="2">75 - 101</td></tr>
<tr><td>> 9.0 (vinyl ester resin)</td></tr>
</table>

Notes:
1. CPP group includes: pre-stressed concrete cylinder pipe (rigid), RCP with cylinder (rigid), RCP w/o cylinder (rigid), and bar wrapped cylinder pipe (semi-rigid).
2. See Section 3.4.8 for descriptions associated with abrasion levels.
3. DIP with cement mortar lining sufficient for Abrasion Levels 1, 2. DIP with ceramic epoxy lining acceptable for Abrasion Levels 1, 2, 3.
4. Applicable if chlorides ≥ 400 ppm.
5. SRTP properties vary depending upon the manufacturer. The values in this table represent the ranges available among different manufacturers of SRTP.
6. The properties for internal linings and external coatings vary by manufacturer. Refer to manufacturer information for pertinent data.
7. Consult manufacturer for specific property information for PW-PVC and C-HDPE.

Table 3-1
Pipe material comparison (part 2 of 2)

Base Pipe Material	Allowable Internal Pressure for Pipe Material from Lab/Plant Certification (psi)	Standard Segment Length Range (ft.)	Manning's (n-value) Range[4]	Interior/Exterior Surface Pattern (smooth, rough, corrugated)	Leak Resistance Options
Cast-in-Place Concrete (CiPCP)	custom	custom	0.010 - 0.011	smooth - both	joints with waterstops
Non-Pressure RCP (NP-RCP)	< 10.8	8	0.010 - 0.011	smooth - both	gasketed joints
Low Pressure RCP (LP-RCP)	< 54.2	8	0.010 - 0.011	smooth - both	gasketed joints
Concrete Pressure Pipe (CPP)[1]	54.2 - 400	8 - 40	0.010 - 0.011	smooth interior smooth/rough ext.	gasketed joints
Vitrified Clay (VCP)	< 10.8	4 - 10	0.011 - 0.017	smooth - both	gasketed joints
Corrugated Steel (CSP - Galvanized)	< 10.8	20	0.011 - 0.037	1) corrugated int./ext. 2) corrugated. ext./ smooth int.	gasketed joints
Corrugated Steel (CSP - Aluminized)					
Corrugated Steel (CSP - Polymer)					
Corrugated Aluminum (CAP)	hydrostatic[2]	40	0.011 - 0.037	1) corrugated int./ext. 2) corrugated. ext./ smooth int.	gasketed joints
Welded Seam Steel Pipe (WSSP)	113 - 600	20 - 100	0.011 - 0.012	smooth - both	welded joints
Ductile Iron (DIP)	450	18 - 20	0.010 - 0.014	smooth - both	gasketed joints
Solid Wall HDPE (C-HDPE, SW-HDPE)	100 - 333	40 - 50	0.018 - 0.025 0.009 - 0.015	corrugated - both smooth - both	gasketed joints fused joints
Profile Wall (PW-HDPE, PW-PP)	< 5.0	20	0.009 - 0.015	corrugated exterior smooth interior	gasketed joints
Polyvinyl Chloride (PVC)	80 - 305	14 - 20	0.009 - 0.011	smooth - both	gasketed joints dual wall
Steel Reinforced Thermoplastic (SRTP)[3]	15 - 30	20	0.009 - 0.015	corrugated exterior smooth interior	gasketed or welded joints
Fiberglass Reinforced (FRP)	450	20	0.009 - 0.011	smooth - both	gasketed joints or laminated joints

Notes:
1. CPP group includes: pre-stressed concrete cylinder pipe (rigid), reinforced concrete cylinder pipe (rigid), reinforced non-cylinder pipe (rigid), and bar wrapped cylinder pipe (flexible/semi-rigid).
2. Maximum internal pressure is a function of depth of water within pipe. For a pipe that is 120 inches in diameter, the maximum internal pressure is 4.3 psi.
3. SRTP properties vary depending upon the manufacturer. The values in this table represent the ranges available among different manufacturers of SRTP.
4. Most values within this table represent laboratory values taken from FHWA Hydraulic Design of Highway Culverts, 3rd Edition (2012). Field values can result in a significant increase from laboratory Manning values depending upon the effects of abrasion, corrosion, deflection, and joint conditions.

3.4. Considerations for Selection.

3.4.1. General. Certain pipe materials may not be applicable for a given situation based upon available sizes, shapes, hydraulic characteristics, operating environment, or other restrictions. A designer may also choose to eliminate the use of a pipe material if it lacks sufficient performance information to provide a satisfactory level of confidence regarding the associated risk. Regardless of the type of pipe material selected, its design must consider the likely long-term operating condition of the pipe relative to the PFMs detailed in Chapter 2 and how conditions may change over time.

3.4.2. Pipe Function. A list of applicable pipe materials by function within USACE Civil Works projects related to embankments or floodwalls, along with potential concerns for each pipe function, is provided in Table 3-2. Some pipe materials are only acceptable for use in certain environments if modifications are made to them. Note that if SRTP is being used, the designer must coordinate directly with the pipe manufacturer/supplier to ensure it is applicable for the intended function and anticipated design loads.

Table 3-2
Viable pipe materials by pipe location/function

Pipe Location/Function[1]	Viable Pipe Materials	Potential Concerns
Pipes within/beneath an embankment or floodwall < 5psi (internal pressure)	All pipe materials, pending operating environment and internal pressures	Operating environment (pipe effluent, surrounding soil, flow abrasiveness, etc.), constructability, performance history, life-cycle cost, potential for internal pressures due to pipe closure gates, joint leakage, temperature issues
Pipes within/beneath an embankment or floodwall < 10.8 psi (internal pressure)	All pipe materials except CAP, C-HDPE, PW-PP, PW-HDPE, and PW-PVC subject to operating environment and internal pressure limits	Same as above subject to internal pressure limitations, joint performance
Pipes within/beneath an embankment or floodwall < 54.2 psi (internal pressure)	LP-RCP, CPP, WSSP, DIP, FRP, SW-HDPE, PVC, CiPCP	Same as above subject to internal pressure limitations, joint performance
Pipes within/beneath an embankment or floodwall > 54.2 psi (internal pressure)	CPP, WSSP, DIP, FRP, SW-HDPE, PVC, CiPCP	Same as above subject to internal pressure limitations, joint performance
Pump station discharge lines	WSSP, DIP, FRP, SW-HDPE	Differential settlement along pipe (joint flexibility), weathering exposure, corrosive environment, pressure limitations
Relief well collectors (solid wall)	All pipe materials pending operating environment and internal pressures	Operating environment, filter around pipe, constructability
Toe drains (perforated)	Perforated products of: CSP, CAP, VCP, PVC, PW-HDPE, PW-PP, PW-PVC	Clogging, corrosion/deterioration, collapse, filter around pipe
Slip lining deteriorated pipe	SW-HDPE, PVC, FRP, WSSP	Grouting procedures and pressures, modified flow characteristics
Third-party pipes (utilities, municipalities, irrigation, etc.)	These types of pipes are evaluated and reviewed on a case-by-case basis using the Section 408 process (reference Appendix H)	

1. Internal pressures as listed in Table 3-1.

3.4.3. Hydraulic Requirements. Pipe flow is affected by both the inlet and outlet conditions as well as the pipe characteristics. A pipe with interior corrugations may be desirable because it provides the necessary load carrying capability at a cost savings, but the increased Manning's roughness coefficient that results in lower flow capacities must be considered. Even pipes with no interior corrugations and identical inner diameters may have different flow rates based on the smoothness of the pipe walls and the differing Manning's roughness coefficients. The laboratory values for the Manning's roughness coefficient shown in Table 3-1 are not necessarily indicative of field values due pipe degradation after installation (e.g., from abrasion, corrosion, etc.). Sensitivity analysis with respect to varying Manning roughness coefficients is recommended during design. Designers should consult the wide range of design tools available (charts, software, etc.) specific to each pipe material to ensure hydraulic requirements are met. However, all applicable USACE criteria must be met regardless of which tools are used by the designer.

3.4.4. Available Standard Shapes and Sizes. An important factor to consider when selecting a pipe material is the available shapes and sizes for which it is manufactured. Figure 3-18 shows standard cross-sectional shapes for NP-RCP, but not all pipe materials are available in all shapes. Table 3-3 provides a comparison of pipe size ranges by cross-sectional shape along with the associated specifications that cover the manufacturing of the pipe and associated laboratory/plant certification testing.

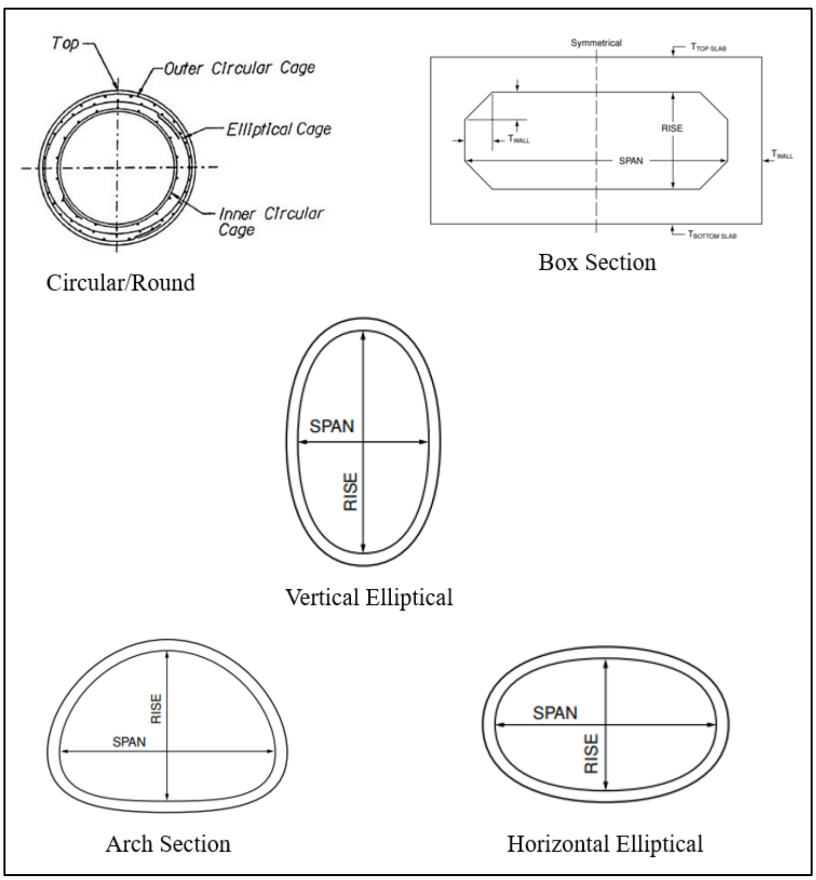


Figure 3-18. Standard pipe cross-sectional shapes for NP-RCP.

Table 3-3
Comparison of available pipe size range by material and shape

Pipe Material Description	AVAILABLE STANDARD PIPE SIZE RANGES (INCHES) BY CROSS-SECTIONAL SHAPE: Circular/Round Diameter	Arch[1] Equivalent Circular Diameter	Horizontal Elliptical[1] Equivalent Circular Diameter	Vertical Elliptical[1] Equivalent Circular Diameter	Box Shape: Span (Horizontal)	Box Shape: Rise (Vertical)	Applicable Specifications for Manufacturing and Lab/Plant Certification
Non-Pressure RCP (NP-RCP)	12 - 144	15 - 132	15 - 144	36 - 144	see Note 7		see Note 3
Low Pressure RCP (LP-RCP)		n/a	n/a	n/a	n/a	n/a	ASTM C361, C497
CPP - RCP with Steel Cylinder	30 - 144	n/a	n/a	n/a	n/a		AWWA C300
CPP - RCP w/o Steel Cylinder	12 - 144						AWWA C302
CPP - Prestressed Conc Cylinder Pipe	16 - 144						AWWA C301
CPP - Bar-Wrapped Cylinder Pipe	10 - 72						AWWA C303
Vitrified Clay (VCP)	4 - 42	n/a	n/a	n/a	n/a		ASTM C700
Fiberglass Reinforced (FRP)	8 - 156	n/a	n/a	n/a	n/a		see Note 4
CSP (Metallic Coated)	4 - 144	15 - 120	n/a	n/a	108 - 305	30 - 126	ASTM A760 (circular/round, arch) ASTM A964 (box)
CSP (Polymer Coated)					n/a	n/a	ASTM A762
Welded Seam Steel Pipe (WSSP)	24 - 144	n/a	n/a	n/a	n/a		see Note 5
Corrugated Aluminum Pipe (CAP)	4 - 120	15 - 120	n/a	n/a	105 - 305	30 - 126	ASTM B745 (circular/round, arch) ASTM B864 (box)
Ductile Iron Pipe (DIP)	3 - 64	n/a	n/a	n/a	n/a		ANSI/AWWA C151/A21.51 ASTM A746
Solid Wall HDPE (C-HDPE, HDPE)	4 - 65	n/a	n/a	n/a	n/a		see Note 6
Profile Wall (PW-HDPE, PW-PP)	4 - 60	n/a	n/a	n/a	n/a		see Note 6
Polyvinyl Chloride (PVC)	4 - 60	n/a	n/a	n/a	n/a		see Note 6
Steel Reinforced Thermoplastic (SRTP)	8 - 120	n/a	n/a	n/a	n/a		ASTM F2562
Cast-in-Place Concrete	all shapes and sizes for cast-in-place concrete are custom built						

NOTES
1. Arch and elliptical sections are typically sized by specifying an equivalent circular shape.
2. This table represents 'standard' sizes from applicable specifications. Individual manufacturers may have sizes available that vary from those listed within this table.
3. NP-RCP applicable manufacturing specifications: circular/round (ASTM C76), arch (ASTM C506), elliptical (ASTM C507). Laboratory/plant testing are covered by ASTM C443, C1628, and C497.
4. FRP applicable specifications: ASTM D3262 (non-pressure sewer), ASTM D3517 (pressure water), ASTM D3754 (pressure sewer and industrial). Non-circular shapes and sizes - ISO 16611:2017.
5. For WSSP, a specification doesn't exists that details standard sizes. Size limitations are determined by individual manufacturing limitations (steel sheet size, coil dimensions, etc.).
6. Thermoplastic pipe specifications: HDPE (ASTM D2737, D3350, F714), PP (ASTM F2881, F2764), PVC (ASTM F679, AWWA C900).
7. Precast non-pressure RCP is manufactured in box shapes of limited sizes, but only cast-in-place concrete box culverts are considered applicable for levee and dam applications.

3.4.5. Load Carrying Capacity. A pipe must have the ability to withstand all anticipated loading situations, such as an empty pipe experiencing the external loads applied by the surrounding soil and any additional potential live loads (i.e., vehicles, equipment), or a gravity drain pipe internally pressurized by water when a sluice gate is closed. Each pipe material has a specific structural design methodology, which is covered in Chapter 4. Thin-walled, corrugated pipes (i.e., steel, aluminum, plastic) as well as lightly reinforced concrete and VCP may not be used in embankments if they are expected to resist internal pressures greater than 10.8 psi (25 feet of head). Thermoplastic profile-wall pipes may not be used if internal pressures are likely to exceed 5 psi. Both flexible and rigid pipes can be designed for deep embedment, but the way the external load is distributed varies (reference Section 3.3.1).

3.4.6. Corrosion Environment.

3.4.6.1. General. Corrosion is the deterioration of a material's mechanical or physical properties that results from an electrochemical reaction with the material's environment. There are several different types of corrosion associated with metals; the more common types include: generalized surface corrosion, localized/pitting corrosion, galvanic corrosion (dissimilar metals), microbiological influenced corrosion (MIC), stress/fatigue corrosion, and stray current corrosion.

3.4.6.2. Causes of Corrosion. The corrosion of the interior of a pipe is mainly caused by differential aeration and/or localized galvanic corrosion, and the extent is dependent upon the effluent that is being carried through the pipe. Exterior corrosion of buried pipe is caused by a number of factors, including the corrosiveness of the surrounding soil, presence of chlorides/sulfates, soil pH, soil moisture content, and fluctuation of the water table. As a rule, higher-plasticity soils exhibit a greater potential to corrode buried metal pipes than lower- or non-plastic soils. The corrosion potential for soil is determined by measuring its resistivity, which is a measure of how much the soil conducts an electrical current. Soils with lower resistivity values are more corrosive to metal. The following soils are classified for relative corrosiveness, per AWWA M27:

3.4.6.2.1. Unusually Corrosive. Unusually corrosive soils (< 2,000 ohm-cm) include muck, peat, tidal marsh, clays with organic content, and adobe clay.

3.4.6.2.2. Severely Corrosive. Severely corrosive soils (2,000 – 4,500 ohm-cm) include clays, clayey loams.

3.4.6.2.3. Moderately Corrosive. Moderately corrosive soils (4,500 – 6,000 ohm-cm) include discolored sandy loams, clay/silt mixes, silt loams.

3.4.6.2.4. Lightly Corrosive. Lightly corrosive soils (> 6,000 ohm-cm) include sands, uniform-colored sandy loams, lightly textured silt loams, very well-draining loams.

3.4.6.3. Steel Corrosion. Soil or fluid with a pH less than 5.0 is considered acidic and can be very damaging to uncoated and/or unlined steel pipes. "Soft water" (less than 60 ppm calcium carbonate) is more damaging to CSP than "hard water" because it lacks the protective scaling effects from the higher calcium content. Specialized coatings can be applied to assist with corrosion resistance, but their effectiveness is heavily influenced by the abrasiveness of flow through the pipe. Coating the exterior of the steel pipe can provide additional service years and is recommended when installing steel pipe in a USACE embankment or floodwall project. AWWA M11 provides an excellent overview of both corrosion and coatings/linings for steel pipe and should be referenced.

3.4.6.4. Considerations for Steel Corrosion Control. The information in this section should be used in conjunction with Chapter 9 of the National Corrugated Steel Pipe Associations Design Manual, which provides guidelines for various operating environments. Cathodic protection systems can also be used as a measure to combat corrosion of steel pipes. One option for WSSP includes surface painting with a coal tar epoxy for corrosion control. If painted, USACE recommends that United Facilities Guide Specification (UFGS) 09 97 02, System 6-A-Z (Coal Tar Epoxy C-200a) be used as it has excellent adhesion for submergence in water/soil and has a good history of performance in brackish environments. For interior protection, cement-mortar linings for steel pipe tend to provide the best level of interior corrosion protection. Additional information relative to corrosion control is provided in Chapter 5.

3.4.6.5. Aluminum Corrosion. Numerous field studies for aluminum drainage pipes show that CAP tends to provide better waterside corrosion resistance than normal galvanized CSP for most environments. However, cementitious material must never be placed directly against an aluminum pipe or aluminum fittings or appurtenances due to aggressive corrosion, unless the aluminum has been coated with an approved primer. Bituminous coatings are not approved primers for aluminum because water can penetrate this type of coating within a few years and begin premature corrosion.

3.4.6.6. Ductile Iron Corrosion. For corrosive soil conditions and areas where stray current is a potential issue, DIP must be encased in enhanced polyethylene according to ANSI/AWWA C151/A21.51 and ASTM A674 (Figure 3-11). The enhanced polyethylene encasement system was developed to address the potential influence of anaerobic bacteria through MIC and the possibility of corrosion occurring under an intact polyethylene wrap. This system consists of three layers of linear low-density polyethylene fused into one. Locations where polyethylene wraps or other additional corrosion systems need to be added for DIP include soils that have low resistivity, anaerobic bacteria, differences in soil composition, or differential aeration. The Ductile Iron Pipe Research Association (DIPRA) maintains the most up-to-date guidance regarding corrosion control measures for DIP.

3.4.7. Flow Abrasion Environment. Deterioration along the invert of a pipe is a key contributor to shortening the service life of gravity drainage pipes because the abrasive flow "erodes" the invert and sides of the pipe wall resulting in perforations. This exposes the soil around the pipe, resulting in an unfiltered exit which can lead to internal erosion and an embankment failure.

3.4.8. Flow Abrasion Environment Categories. Categories of abrasive flow provide a basis for pipe selection to protect against this damaging mechanism. Table 3-1 indicates which pipe materials are applicable for each level of abrasive flow, which was determined through review of multiple independent research studies and best engineering judgment. There are countermeasures to combat abrasion including paving the pipe's invert and lower portion of its side walls, specifying a thicker pipe wall section than what is required to meet structural loads, or providing specialized abrasion resistance coatings. For moderate to severe abrasive environments, these countermeasures (i.e., paved invert, thicker pipe wall, coatings) will only provide minimal benefit to extending the life of the pipe; they are better suited for extending the service life in mildly abrasive environments. The abrasion environments are as follows:

3.4.8.1. Level 1 (Non-Abrasive). Level 1 indicates non-abrasive bedload (silt, clay, clear water) with a flow velocity < 5 feet/second. All pipe materials are adequate for this environment.

3.4.8.2. Level 2 (Mildly Abrasive). Level 2 indicates moderate, occasional abrasive bedload present (sands and/or gravels) with a flow velocity < 5 feet/second or non-abrasive bedload (silt, clay, clear water) with flow velocity between 5 and 15 feet/second.

3.4.8.3. Level 3 (Moderately Abrasive). Level 3 indicates routine, moderate bedload of sands, gravels with occasional small cobbles present with a flow velocity between 5 and 15 feet/second or non-abrasive bedload (silt, clay, clear water) with flow velocity > 15 feet/second.

3.4.8.4. Level 4 (Severely Abrasive). Level 4 indicates bedload of sands, gravels, small cobbles, or rocks routinely present with flows that commonly exceed 15 feet/second.

3.4.9. Cost Considerations. The designer should not allow a low initial cost to overshadow the larger considerations of long-term pipe performance, life cycle costs, and risk to the project and potential consequences. Likewise, a life cycle analysis for the pipe may reveal that a pipe with a high initial cost may not be the optimum choice (i.e., higher cost does not necessarily mean longer service life). The pipe material selection must meet the service requirements associated with the pipe's function over the life of the project. When considering life cycle costs, the designer must evaluate the pipe material, joint material, backfill material/placement method, installation confirmation testing, operating environment, and other pertinent information to ensure the best choice is made to minimize risk and ensure long-term safe and reliable operation.

3.4.10. Service Life. The term "service life" is often used interchangeably with the terms "design life" or "operating life." These terms can signify different time spans, but both essentially refer to the amount of time a pipe is in service before it reaches an unsatisfactory condition. This condition is defined differently depending upon the pipe's intended function. For example, an "unsatisfactory condition state" for a pipe extending within or beneath an embankment or floodwall may differ from that of a perforated pipe used as a toe drain adjacent to an embankment or floodwall. Designers should be aware that although a manufacturer or supplier may claim a certain service life for a pipe, it is impossible to make blanket statements without knowing how the pipe is designed and constructed, what its operating environment will be, and what the pipe's function is relative to the overall project. Selecting a pipe material based on a "promoted" numerical minimum service life is not recommended because such figures are based upon non-site specific generic environmental conditions. The application of protective coatings and/or liners can provide additional years of service, but only in the right type of environment. When designing a pipe for an embankment or floodwall with respect to life cycle, the design must meet the general requirements outlined in ER 1110-2-8159, which specifies a minimum project service life of 100 years for major infrastructure projects. This does not preclude any specific type of pipe material from being used, but it may require that replacement and/or slip lining of certain pipe materials be incorporated as part of the life cycle analysis in order to meet the design requirements outlined in the ER.

Chapter 4
Structural Design

4.1. Introduction.

4.1.1. Overview. Chapter 4 provides basic design concepts, load considerations, and other information relative to rigid and flexible pipe design, as well as specific design requirements for individual pipe material types. Pipe "design" refers to the process of determining the capacity of a pipe to resist all design loads without failure during its service life and is not intended to address the actual design associated with the manufacturing of pipe materials. As such, the structural design is the decision-making process of choosing the appropriate pipe for a given material type once the factored loads and applicable resistance criteria have been established.

4.1.2. Design Resources. A significant portion of the design information within this chapter comes from the American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) Bridge Design Specifications document and other industry association design and performance standards. It is important to note that the information within this manual that references specific calculations, figures, tables, and other pertinent information from reference and guidance documents such as ASTM specifications and American Water Works Association (AWWA) standards and manuals are subject to change as future versions of the documents are published. It is the responsibility of the designer to ensure the latest guidance is followed accordingly.

4.1.3. Pipe Inspection and Maintenance Access. Adequate access to the entire pipe for inspection and maintenance is an important feature that must be considered during the design process. The ability to routinely inspect a pipe either by physical walk-through for larger pipes (equal to or greater than 48 inches in diameter) or by means of remote equipment for smaller pipes (less than 48 inches in diameter) is required. This includes not only gravity pipes within or beneath an embankment or floodwall, but also other pipes such as those associated with seepage collection systems. Access can be granted though inlets/outlets or by installed manholes along the length of the pipe. Reference Chapter 5 and Chapter 6 for details regarding installation and inspection.

4.2. Chapter Considerations. Prior to starting the structural design of the pipe, it is assumed the designer has already determined the hydraulic requirements of the pipe (e.g., diameter, slope) and chosen the pipe material using the guidance from Chapter 3. Sections 4.5 through 4.8 cover basic structural design information associated with pipes including various pipe installation conditions, load factors, applicable modifiers, and different load case considerations. Sections 4.9 through 4.18 detail specific design procedures based on the pipe material; therefore, only certain sections of the chapter will apply given the designer's scenario.

4.3. USACE Dam Conduits. Dam outlet conduits (regulating outlets and tunnels) have special load conditions and design requirements that are only applicable when used as reservoir control features. Only CiPCP and CPP are considered acceptable for use as USACE dam conduits due to their typically high pressures and frequent usage. In cases where cavitation is a significant concern, these conduits may use steel liners as part of the design. CiPCP and CPP are also commonly used in levee applications. Other ancillary pipes associated with dams (e.g., seepage collection pipes) do not have the same pipe material restrictions. Penstocks also have specific design considerations. General information on penstocks is provided in Chapter 10.

4.4. Potential Failure Modes Related to Pipe Design. Proper structural design reduces the likelihood of excessive cracking, deformation, or collapse, which in turn reduces the likelihood of the PFMs related to providing an unfiltered exit along or through a pipe defect (PFM-2 and PFM-3), potentially leading to a breach. Additionally, collapse or deformation of a pipe may not lead to a breach but can still restrict flows and cause interior ponding (PFM-5), which has the potential to induce damages. Finally, proper design of the inlet and outlet structures per EM 1110-2-1602 reduces the probability of erosion initiating and leading to failure of the embankment or floodwall (PFM-4).

4.5. Rigid vs. Flexible Pipes. Pipes are generally categorized as either rigid or flexible. A flexible pipe is one that can deflect (i.e., transition from a circular to a more oval cross-section) at least two percent of its diameter without signs of structural distress. Flexible pipes move, or deflect, and transfer the overburden loads to the surrounding soil mass; thus, the pipe itself is not designed to carry the entire loading, and instead shares the load with the surrounding soil mass. Rigid pipes are made from materials with high rigidity that are designed to carry much of the overburden load, and therefore deflect very little into the surrounding soil. Although rigid pipes are much stiffer than flexible pipes, they still require properly placed backfill to prevent exceeding their load carrying capability. General design procedures for both rigid and flexible pipes are described within this chapter. Maximum pipe deflection, or ring deflection, limits for all flexible pipe materials are listed in Table 4-1. Pipe deflection (ring deflection) differs from joint deflection (reference Chapter 5).

Table 4-1
Flexible Pipe Deflection (Ring Deflection) Limits

Max. Allowable Deflection (%)	Pipe Material
5	CSP and CAP (inside diameter); WSSP (outside diameter for pipe with flexible linings/unlined and flexible coatings/uncoated); DIP with flexible or ceramic epoxy lining (outside diameter); HDPE, PP, PVC, SRTP (inside diameter); FRP (inside diameter)
3	WSSP (outside diameter for pipe with cement-mortar lining and flexible coating/uncoated); DIP (outside diameter)
2	WSSP (outside diameter for pipe with cement-mortar coating)
Other	BWCP (maximum allowable deflection equals the square of the inside diameter (in.) divided by 4,000, reference AWWA M9)

4.6. Rigid Pipe Installation Conditions.

4.6.1. General. Four standard installation conditions exist for estimating loads acting on buried rigid pipes and overviews for each are provided in the latest version of the Concrete Pipe Design Manual. The installation condition has a significant effect on the loads carried by a rigid pipe. The four rigid pipe standard installation conditions are 1) positive projecting embankment, 2) trench, 3) negative projecting embankment, and 4) trenchless (jacked or tunneled).

4.6.2. Positive Projecting Embankment. This installation condition is associated with placing a rigid pipe on the original ground or base of a compacted fill and then covering it with an earth fill embankment. For most installations of new pipes within USACE dam and levee embankments, the positive projecting embankment is the installation condition that should be assumed for calculating the earth loads acting on a rigid pipe. For this installation condition, the soil along the side of the pipe is assumed to settle more than the soil directly over the rigid pipe, resulting in additional soil load acting on the pipe. This additional load is accounted for by the use of a vertical arching factor (VAF) which is detailed in Section 4.9.2.1.2

4.6.3. Trench. This type of installation condition is commonly selected to estimate the earth loads for the structural design of pipes installed in relatively narrow trenches. For this situation, the calculated earth load is equal to the soil prism load minus the frictional forces on the side of the trench (AASHTO Equation 12.10.2.1-1). This is due to the assumption that the newly installed backfill will settle more than the existing soil on the sides of the trench; thus, the friction forces relieve the pipe of some of the soil burden. The VAF used for a trench installation is less than that associated with an embankment installation; however, because of the difficulty of controlling the maximum trench width in the field due to a variety of potential construction factors, any reduction in earth load acting on the pipe is usually ignored. Thus, trench installation conditions are recommended to be designed conservatively as a positive projecting embankment condition.

4.6.4. Negative Projecting Embankment. A negative projecting embankment installation condition is for a pipe that is installed in a shallow trench of such depth that the top of the pipe is below the natural ground surface or compacted fill and then covered with an earth fill embankment which extends above the original ground level. The details for estimating the earth loads for this type of installation are provided in the latest version of the Concrete Pipe Design Manual.

4.6.5. Trenchless (Jacked or Tunneled) Installation. An installation technique that avoids the need to open excavate the soil for a pipe. Reference Chapter 5 for details.

4.7. Non-pressurized vs. Pressurized Pipes. Pipes within or beneath embankments and floodwalls can be designed to handle both pressurized and non-pressurized applications. Non-pressurized pipes are typically gravity flow structures in which the pipe design is primarily controlled by external loads (soil and vehicular), although the service life of the pipe may be controlled by other performance issues such as wear along the invert of the pipe or corrosion. Pressurized pipes are either designed to meet the required internal pressures (water and surge pressure) and then checked against external loads or designed to meet the combination of the internal/external loads.

4.8. General Structural Design Considerations.

4.8.1. Background. The structural design of any pipe is required to safely withstand the loads acting on it while meeting required performance criteria. Loads acting on a buried pipe are dependent upon the following: the pipe material; how the pipe is installed within, beneath, or adjacent to the embankment or floodwall; the water table elevation; and the presence of any transient (live) loads. The amount of load taken by the pipe depends upon several factors, including its burial depth and the compactness of the material surrounding the pipe. Pressurized pipes must also be designed to safely withstand the internal working pressure plus a surge pressure.

4.8.2. Design Methodologies. At the time of this manual's publishing, the following pipe materials had an established LRFD design methodology which should be used for the structural design of new pipes: non-pressure/low-pressure reinforced concrete pipe (NP-RCP and LP-RCP); CPP; fiberglass reinforced pipe (FRP); all corrugated metal pipes (CMPs); gravity flow non-pressurized thermoplastic profile wall pipes; and CiPCP. Pipe materials without an approved LRFD methodology can still be used in USACE dam and levee applications, but all applicable design and performance criteria detailed in this chapter must be met. These pipes include: VCP, ductile iron pipe (DIP), pressurized thermoplastic solid wall pipes (SW-HDPE, SW-PP, and SW-PVC), welded seam steel pipe (WSSP), and SRTP. Thermoset (cured-in-place plastic material) pipe is used almost exclusively for rehabilitation of existing deteriorated pipes and is discussed separately in Chapter 7. The design of associated structures such as cast-in-place and precast concrete gatewells, manholes, and other hydraulic structures must be designed according to EM 1110-2-2104.

4.8.3. Load and Resistance Factor Design Overview.

4.8.3.1. Load and Resistance Factor Design Methodology. The structural design methodology of pipes follows the guidance outlined in the 8th edition of the AASHTO LRFD Bridge Design Specifications (September 2017), unless otherwise specified herein. In general, LRFD is a limit state design methodology where loads acting on a structure are increased by load factors, and the strength (or load carrying capacity) of the structure is reduced by resistance factors for a series of specific limit states (failure conditions). The factors are intended to account for potential deviations in actual loads and strength properties for the structure being designed. The basic equation for LRFD states that factored loads (left side of Equation 4-1) must be less than or equal to the factored resistance (capacity), shown on the right side of the equation:

Equation 4-1

$$\sum \eta_i \cdot \gamma_i \cdot Q_i \leq \phi \cdot R_n = R_r$$

Where:

η_i = load modifiers for ductility (D), redundancy (R), and importance (I)
γ_i = material/limit state specific load factors
Q_i = calculated, unfactored loads for specific limit states
ϕ = material/limit state specific resistance factor
R_n = nominal resistance for specific limit states
R_r = factored resistance for specific limit states

4.8.3.2. AASHTO Load and Resistance Factor Design Guidance. The structural design of many types of buried pipe is governed by Chapter 12 of the AASHTO LRFD Bridge Design Specifications document. Each type of pipe covered within the AASHTO document is required to meet design criteria for both strength (usually capacity) and service (deflection, crack control). Table 4-2 lists the limit states for most types of pipe covered in the AASHTO document, as well as the applicable sub-section.

Table 4-2
LRFD structural design of pipe materials for specified limit states

Description	Metal Pipes (Round and Vertical Arches)[1]	Reinforced Concrete Pipe[2]	Thermoplastic Pipe	Fiberglass Pipe
AASHTO Section	12.7	12.10	12.12	12.15
Strength Limit State #1	Wall Thrust	Flexure	Thrust Strain	Flexural Strain
Strength Limit State #2	Buckling	Shear	Buckling Strain	Buckling
Strength Limit State #3	Longitudinal Seam Resistance	Thrust	Combined Strain	Strain
Strength Limit State #4	Flexure[3]	Radial Tension	Flexural Strain	n/a
Service Limit State #1	n/a	Crack Control	Deflection[4]	Deflection[4]
Service and Handling	Flexibility Limit	n/a	Flexibility Limit	Flexibility Limit

Notes:
1. Applicable for steel and aluminum pipes. Rise-to-span ratio ≥ 0.3 for arch sections. Radius of curvature ≤ 13.0 feet for both round and arch sections. Section 12.7 also covers steel reinforced thermoplastic pipes.
2. Applicable for round, elliptical, and precast arches only. Precast box culverts and cast-in-place arches are covered in Section 12.11 of AASHTO. ASTM C361 is also considered an applicable reference that utilizes the same design methodology when using LRFD factors.
3. Only applicable to box shapes and deep-corrugation structures.
4. Reference Table 4-1.

4.8.4. Load and Resistance Factor Design Load Factors and Modifiers for Structural Pipe Design.

4.8.4.1. General. AASHTO's LRFD procedures require two adjustments to calculated unfactored loads. The first is pipe material/limit state-specific load factors (γ) that typically result in a higher factored load. The second adjustment is referred to as a load modifier (η) to account for ductility, redundancy, and operational importance. General information regarding load factors and modifiers is covered in Section 1.3 of the AASHTO document; specific applications of load factors/modifiers for buried structures (pipes) are covered in Section 12.5.

4.8.4.2. Load and Resistance Factor Design Load Factors.

4.8.4.2.1. Limit State Load Combinations. Buried pipes must be designed to meet load combinations associated with AASHTO-defined Strength Load Combinations I and II as well as Service Load Combination I, all of which have been summarized in Table 4-3. Loads to be considered must include forces associated with earth pressure (horizontal and vertical), pavement loads, live loads, and vehicular/rail live dynamic loads. The effects of buoyancy must be included when the invert of the pipe is below the water table. Earth surcharge loads, live load surcharges, and external water pressure (i.e., water above the ground surface) must be considered when warranted by construction and/or site conditions. When water is present above the ground surface and the pipe is empty or with less water pressure than associated external water, the pipe must be designed using a Strength Load Combination with external water pressure as the primary load. Earthquake forces only need to be considered when the pipe crosses an active fault (reference Section 4.8.5.1).

Table 4-3
LRFD buried pipes applicable load factors[1]

Limit State Load Combination	Permanent (Dead) Loads	Live Loads	Water Loads
Strength I	γ_p	1.75	1.00
Strength II	γ_p	1.35	1.00
Strength, Water	γ_p	n/a	γ_w
Service I	1.00	1.00	1.00

1. Earth surcharge, live load surcharge, and down drag loads must be evaluated as site conditions warrant.

4.8.4.2.2. Load Factor, γ_p. The load factor γ_p varies as a function of the pipe material for vertical earth pressure loads only. The load factor is also applied for horizontal earth pressures and earth surcharge loads, but the same value is used regardless of the pipe material. Table 4-4 shows the design values for γ_p as a function of pipe material and type of load.

Table 4-4
LRFD load factor (γ_p) for buried pipes

Load Type	Rigid Pipe[1]	All Others[2]	HDPE, PP, PVC	CMP, FRP[3]
Vertical Earth Pressure	1.3	1.95	1.3	1.5
Horizontal Earth Pressure (Active)	1.50 - all buried pipes			
Horizontal Earth Pressure (At Rest)	1.35 - all buried pipes			
Earth Surcharge Loads	1.50 - all buried pipes			

1. All rigid pipes from Table 3-1 including applicable footnotes.
2. Includes BWCP, WSSP, DIP, SRTP, and any others not covered by other categories.
3. Includes all FRP, CSP, CAP, and corrugated metal box culverts.

4.8.4.2.3. Live Load Effects – Depth of Fill. For single-span (individual) pipes, the effects of live loads can be neglected when the depth of fill over the top of the pipe exceeds eight feet and also exceeds the span length of the pipe. For multiple-span (e.g., double barrel, triple box) pipes, the live load effects can be neglected when the depth of fill exceeds the distance between the inside faces of the end walls. When the live load effects need to be taken into account, the loads are a function of the pipe material, depth of fill, and direction of travel relative to the pipe. This information is detailed in AASHTO Section 3.6.1.2.6.

4.8.4.2.4. Load Factor, γ_w. External water pressure is created by water loads on sections of pipes that are gated or sealed so there is no equalization of internal pressure. The maximum possible water load that can occur during any hydraulic condition must be used in the design. The load factor for water pressure is determined by the annual exceedance probability (AEP) associated with various water levels. The AEP can vary by location, embankment or floodwall height, spillway design, and other factors. Water load factors are based on four load categories applied to USACE hydraulic structures: 1) usual loads are frequent events with an AEP > 0.1 (commonly referred to as a 10-year event); 2) unusual loads are infrequent events with an AEP range between 0.0033 and 0.1 (300-year to 10-year event range); 3) extreme loading events are broken into two sub-categories with the first having an AEP range between 0.001 and 0.0033 (1,000-year to 300-year event); and 4) the more extreme event with an AEP < 0.001 (less frequent than a 1,000-year event).

- Usual: AEP > 0.1, use a value of γ_w=1.5 for external water loads
- Unusual: 0.0033 ≤ AEP < 0.1, use a value of γ_w=1.4 for external water loads
- Extreme 1: 0.001 ≤ AEP < 0.0033, use a value of γ_w=1.3 for external water loads
- Extreme 2: AEP < 0.001, use a value of γ_w=1.2 for external water loads

4.8.4.3. Load and Resistance Factor Design Load Modifiers.

4.8.4.3.1. Ductility Load Modifier. A ductility load modifier (η_D) must be included for strength limit state evaluations and has a range of 0.95 to 1.05 depending upon the situation. Pipe materials with an established LRFD design methodology that follows the general guidance of the AASHTO design specifications use a ductility modifier value of 1.0. The value of 0.95 is only used for situations where additional ductility-enhanced measures have been specified beyond what is required by the AASHTO design specifications. For more information regarding the specifics of this modifier, refer to AASHTO Section 1.3.3.

4.8.4.3.2. Redundancy Load Modifier. A redundancy load modifier (η_R) is also applied for strength limit state evaluations. For buried pipes, the redundancy load modifier equals 1.0 for earth fill loads, as well as live and dynamic loads. This is a modification from the 2017 AASHTO document where η_R was previously equal to 1.05 for earth fill loads.

4.8.4.3.3. Importance Load Modifier. A load modifier for operational importance (η_I) must be applied for strength and extreme event limit states. A value of η_I = 1.05 must be used for all pipes designed as part of a USACE dam and levee.

4.8.4.3.4. Load Modifiers for Construction Loads. All load modifiers (ductility, redundancy, and importance) use a value of 1.0 for construction loads.

4.8.5. Specialized Load Conditions.

4.8.5.1. Seismic Design Considerations.

4.8.5.1.1. General. None of the structural design methodologies for any of the pipe materials discussed herein addresses seismic design. In general, buried pipes have a better performance history during seismic events when compared to pipeline systems elevated above ground; therefore, buried pipes are typically designed for the routine load conditions of static (dead and live) loads only. However, depending upon the importance and location of the pipe, seismic design may be warranted. For example, seismic loads should be properly accounted for if the pipe crosses an active fault. A consistent, unified design methodology for seismic design of pipes does not currently exist; however, research by Anderson, Martin, Lam, and Wang (2008) provides a seismic evaluation methodology for buried pipes in highway embankments.

4.8.5.1.2. Risk Reduction for Seismic Loading. There are several mitigation measures that can be taken to reduce the likelihood of damage from seismic loading, including the following examples from an American Society of Civil Engineers (ASCE) Pipelines Proceedings paper, Seismic Design of Buried Steel Water Pipelines (2014).

- Increasing the wall thickness improves both the buckling and tensile resistance against seismic loading.

- Higher steel grades may improve performance, but material ductility and deformation capacity may be more important than grade strength.

- In areas where large permanent ground deformations are expected, it may be prudent to try to isolate the pipeline from ground movements by providing a tunnel around the pipeline.

- Improve ground conditions in landslide and liquefaction-prone areas.

- Specialized expansion joints and/or deflectable joints can be used as a mitigation measure to reduce axial stretching of the pipeline in areas where permanent ground motion is expected.

4.8.5.2. Maintenance Load Conditions. Additional load conditions may warrant consideration. For example, maintenance equipment is sometimes required to access the ends of the pipe to clear debris or move heavy gates which could cause the pipe to become overstressed when loaded at a shallow soil cover depth. Because a load case like this is not addressed in the various design reference manuals, it is up to the designer to account for these loads in the pipe's design or develop maintenance standards that ensure the pipe's continued safe operation.

4.8.6. Multiple Pipes in Same Trench.

4.8.6.1. General. Installing multiple pipes within the same trench is most often used where restrictive cover requirements preclude the use of a single pipe of larger diameter, which is not normally an issue with USACE embankments. Installing multiple pipes in the same trench may also allow the designer to reduce the calculated loads on the pipes since the weight of the overburden can sometimes be "shared." However, caution is recommended when applying this method because of the difficulty of controlling the maximum trench width and exact pipe placements in the field. Unless a significant cost savings can be realized and strict adherence to the trench design ensured, the designer should consider ignoring any reductions in earth loads acting on the pipe and use the positive projecting embankment condition instead. Using a controlled low strength material (CLSM) backfill requires less space around the pipes than soil backfill since no room is needed for compaction equipment and more evenly distributes the load. The method described herein provides conservative results for loads acting on the pipe when soil backfilled is used, and even more conservative results when CLSM is used. Once loads are calculated and factored accordingly, the design process follows the guidance for each specific pipe material as detailed throughout this chapter.

4.8.6.2. Loading Determination. The vertical earth load acting on multiple pipes within the same trench can vary from a single pipe in a trench installation to a projected embankment condition, or even a combination of both within the same trench (Sections 4.6.2 and 4.6.3, respectively). Each pipe must be analyzed separately and the transition width determined accordingly. The transition width is the width of a trench where the trench load equals the positive projecting embankment load. Although transition widths can be calculated as a function of fill height above the top of the pipe and installation bedding types can be found in the latest version of the Concrete Pipe Design Manual, it is recommended that the designer work with a geotechnical engineer and the American Concrete Pipe Association to determine the appropriate loading/bedding conditions and design. The general geometric relationships among three pipes in the same trench for CLSM and soil backfill are shown in Figure 4-1 and Figure 4-2, respectively. If B_{cC} (the outside diameter of the center pipe) plus 2Y (twice the width of the soil column between the pipes in the trench) is equal to or greater than the transition width for the middle pipe, then Pipe C is designed for a positive projected embankment condition. If the intermediate pipe spacing, Y, and the exterior pipe spacing to the trench wall, Z, are small compared to the outside pipe diameters (i.e., < $B_{cA}/6$ or $B_{cB}/6$), then the entire earth load is shared proportionately by the three pipes and designed as a trench condition. In addition, when the exterior pipe columns $B_{dA}/2$ or $B_{dB}/2$ are less than one-half of the transition width for either pipe (about 0.75 B_{cA} or B_{cB}), then a trench condition also exists. However, the positive projected embankment condition exists when the width of these exterior pipe columns exceeds the transition width for the pipe. The interior soil columns are analyzed in a similar manner.

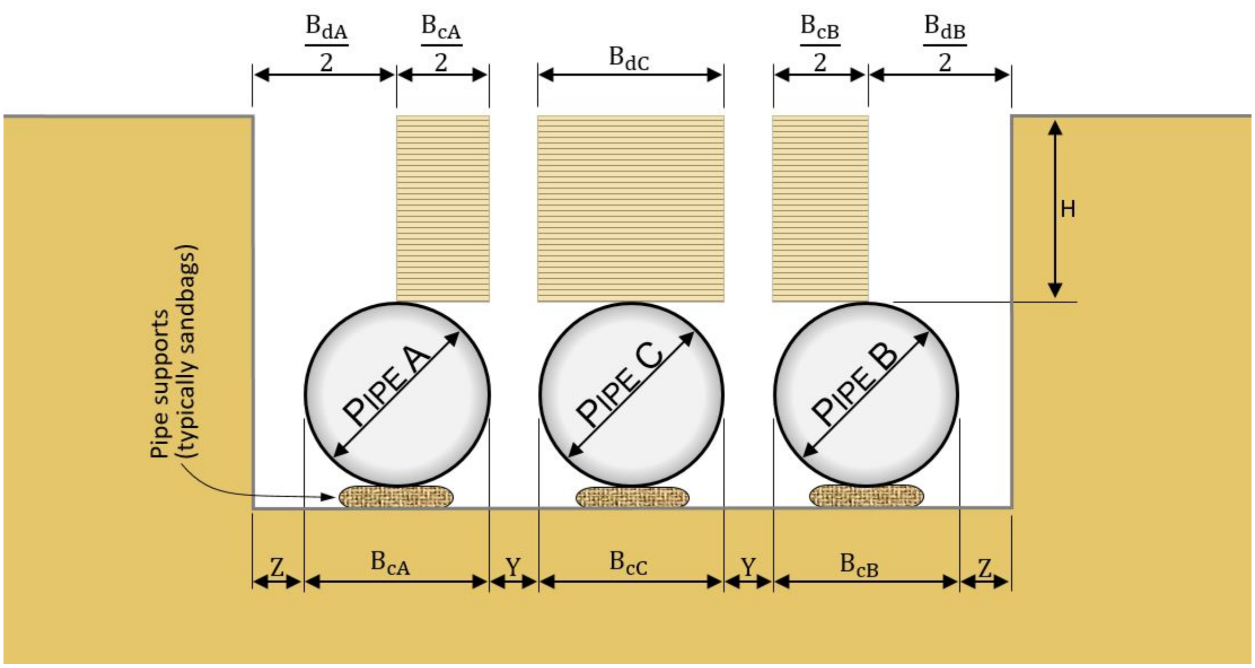


Figure 4-1. Multiple pipes within the same trench using CLSM backfill.

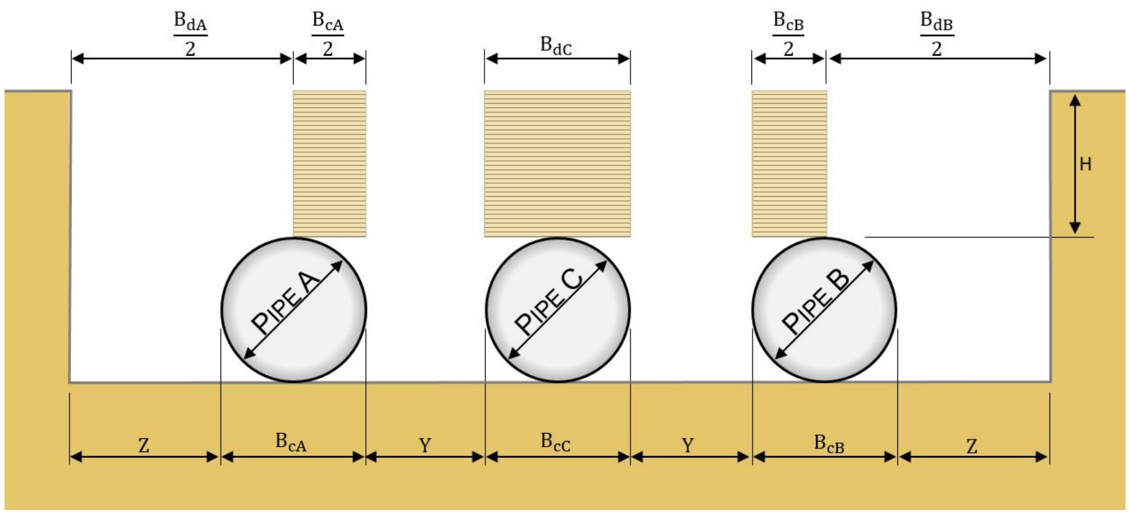


Figure 4-2. Multiple pipes within the same trench using soil backfill.

4.9. Precast Reinforced Concrete Pipe – Non-Pressure/Low-Pressure Precast Reinforced Concrete Pipe (NP-RCP/LP-RCP).

4.9.1. General. Reinforced concrete is one of the primary materials used for pipe drainage and water control applications in embankment and floodwall applications. Precast RCP in embankments and floodwalls is widely used today and historically has performed well when properly constructed and installed in the appropriate environment. Traditionally, precast RCP sections used for most gravity flow applications fall into this category.

4.9.2. Applicable Design Criteria for Precast Reinforced Concrete Pipe. For NP-RCP and LP-RCP, there are two design methods available. The most commonly used is the Indirect Design Method (IDM), which is based upon observed performance of precast RCP in a wide variety of environments. IDM can only be used when the internal pipe pressure is less than or equal to 30 feet of hydraulic head (13 psi) for a straight alignment. RCP joints must be designed to accommodate this internal pressure. It is a simplified method that evaluates a non-factored load (referred to as the $D_{0.01}$ load) acting on the pipe. The designer is required to select a pipe whose three-edge bearing capacity resulting in a 0.01-inch wide crack exceeds that of the computed $D_{0.01}$ load. It is considered applicable for the service load case for precast RCP. When these conditions are not met, the Direct Design Method (DDM) must be used. DDM is a much more intensive design effort that uses the LRFD approach. Both IDM and DDM are only applicable to NP-RCP/LP-RCP.

4.9.2.1. Indirect Design Method.

4.9.2.1.1. Indirect Design Method Design Equations. The equations governing IDM within the AASHTO guidance are covered in Section 12.10.4.3. Equation 12.10.4.3.1-1, shown below as Equation 4-2 and modified to include both hydraulic and installation factors, is the $D_{0.01}$ load for a precast RCP. The $D_{0.01}$ load is that which has been experimentally shown to cause the formation of a crack not exceeding a width of 0.01 inch. This is considered the service limit state for NP-RCP and LP-RCP. When CLSM is used as the bedding/backfill material, it should be considered a Type 1 installation.

Equation 4-2 (AASHTO Equation 12.10.4.3.1-1)

$$D_{0.01} = \left(\frac{12}{S_i}\right)\left(\frac{W_E+W_F}{B_{FE}} + \frac{W_L}{B_{FLL}}\right) \cdot I_f$$

Where:

$D_{0.01}$ = unfactored load causing 0.01-in. crack formation
S_i = internal pipe diameter (in.)
W_E = total unfactored earth load (kip/ft), as per AASHTO Equation 12.10.2.1-1
W_F = total unfactored fluid load in pipe (kip/ft), as per AASHTO Section 12.10.2.2
B_{FE} = earth load bedding factor as per AASHTO Section 12.10.4.3.2a or 12.10.4.3.2b
W_L = total unfactored live load for unit length of pipe (kip/ft), AASHTO Section 12.10.2.3
B_{FLL} = live load bedding factor (kip/ft), as per AASHTO Section 12.10.4.3.2c
I_f = installation factor (Type 1 = 1.1, All Other Types = 1.0)

4.9.2.1.2. Indirect Design Method Earth Load. W_E is commonly referred to as the prism load, which is the weight of the column of soil acting over the outside pipe diameter. It is computed as follows in Equation 4-3.

Equation 4-3 (AASHTO Equation 12.10.2.1-1)

$$W_E = VAF \cdot \gamma \cdot B_c \cdot H$$

Where:

VAF = vertical arching factor as per Table 4-5 for standard embankment installation type (Type 1 = 1.35, Type 2 and 3 = 1.4, Type 4 = 1.45). See AASHTO Section 12.10.2 for additional information.
γ = unit weight of soil (lb/ft^3) over top of the pipe (never less than 110 lb/ft^3)
B_c = outside-to-outside horizontal dimension of the pipe (ft)
H = height of fill over top of the pipe (ft)

Table 4-5
Standard embankment installation soils/minimum compaction requirements

Embankment Installation Type	Haunch[1, 3]	Adjacent Embankment[1, 3]
Type 1	CLSM or 95% SW	90% SW, 95% ML, or 100% CL
Type 2	90% SW or 95% ML	85% SW, 90% ML, or 95% CL
Type 3	85% SW, 90% ML, or 95% CL	85% SW, 90% ML, or 95% CL
Type 4	See note 2	See note 2

1. For USACE dam and levee applications, the material surrounding the pipe (haunch, bedding, backfill, etc.) should never be more pervious than the embankment material. Reference AASHTO Table 12.10.2.1-1 for standard embankment installation types.
2. No compaction required except if CL, which requires 85 percent.
3. SW (well graded sand), ML (low plasticity silt), CL (low plasticity clay).

4.9.2.1.3. Indirect Design Method Fluid Load. W_F should be based upon a unit weight of water unless a different fluid will be flowing through the pipe. It is computed by multiplying the inside area of the pipe (ft2) by 62.4 lb/ft3. For standard installations, the fluid weight is supported by the vertical earth pressure acting over the lower part of the pipe. Reference AASHTO Section 12.10.2.2 for more information.

4.9.2.1.4. Indirect Design Method Live Load. The influence of live loads (W_L) acting on a buried concrete pipe follows the guidance provided in AASHTO Section 3.6. The distribution of the live load through the earth fill is specified in AASHTO Sections 3.6.1.2.6 and 12.10.2.3. When the depth of the earth fill exceeds eight feet above the top of the pipe, live loads can be neglected for single span culverts. Procedures, equations, and tables for varying levels of earth fill less than eight feet are provided within the AASHTO document.

4.9.2.1.5. D-Load Selection. Once the D-load is calculated for the various earth fill and live loads acting on the pipe, the designer must specify a concrete pipe whose maximum D-load exceeds that of the calculated D-load. Precast RCP pipes consistent with ASTM specifications C76, C655, C506 (arch), and C507 (elliptical) use D-load methodology.

4.9.2.2. Direct Design Method.

4.9.2.2.1. General. When the design circumstances do not allow the use of IDM, NP-RCP and LP-RCP must be designed using DDM. DDM uses an LRFD-based approach by applying load and resistance factors to the design for strength limit states (i.e., flexure, thrust, shear, and radial tension) as per Section 12.10.4.2 of the AASHTO LRFD guidance. The inclusion of internal pressure when designing for low-head pressure pipe requires additional considerations beyond what is contained in AASHTO's document. ASTM C361 is applicable to the design of precast RCP in cases where the internal hydrostatic pressure does not exceed that associated with 125 feet of head (54.2 psi). ASTM C361 assumes reasonably good bedding support and does not address live load in its designs. The American Concrete Pipe Association should be contacted for additional guidance beyond what is provided in Appendix D of this manual.

4.9.2.2.2. Direct Design Method Design Tables for Precast Reinforced Concrete Pipe. The design procedures associated with DDM are the same as those used in the structural design of other reinforced concrete structures. A series of design tables within ASTM C361 has been developed for combinations of hydrostatic head, pipe dimensions, earth fill over the pipe, and reinforcing steel yield strengths.

- ASTM C361 Table 1 – Required Circumferential Reinforcement Area:

  Concrete design compressive strength of 5,000 psi
  Pipe diameters between 12 and 144 inches
  Circular shapes
  Wall thickness variations for each diameter
  Earth fill cover for 5, 10, 15, and 20 feet over top of the pipe
  Hydrostatic head for 25, 50, 75, 100, and 125 feet
  Reinforcement yield strength of 40,000 psi

- ASTM C361 Table 2 – Required Circumferential Reinforcement Area:

  Same as ASTM C361 Table 1 except reinforcement yield strength of 60,000 psi

4.9.2.2.3. Direct Design Method Load Conditions. NP-RCP/LP-RCP should be designed for three load conditions when using DDM for structural design. ASTM C361 Design Tables 1 and 2 are based upon the design methodology for each of these three load conditions as follows:

- Load Condition I – Internal pressure only

- Load Condition II – Earth load, pipe weight, and fluid weight with no internal pressure
- Load Condition III – External and internal loads acting concurrently

ASTM C361 ignores the effects of live load in its design; however, for shallow fills (less than eight feet), live loads are an important consideration and must be accounted for in the design. For the purposes of this manual, the unfactored live loads associated with AASHTO HL-93 must be used, as well as any construction loads that may exceed the HL-93 designation. Live loads are applicable for Load Condition II and Load Condition III.

4.9.3. Reinforced Concrete Box Culverts (RCB). RCB culverts are four-sided reinforced concrete pipes with a box/rectangular cross-sectional shape. They can be designed with multiple openings if necessary. Their rigidity and monolithic action make them an economic choice for construction of large pipes. CiPCP is the only viable RCB option for dams and levees since due to concern with the performance of precast RCB joints. RCB design should follow the guidance outlined in EM 1110-2104, with particular attention paid to the joint detail since it is critical to ensure no soil intrusion will be possible.

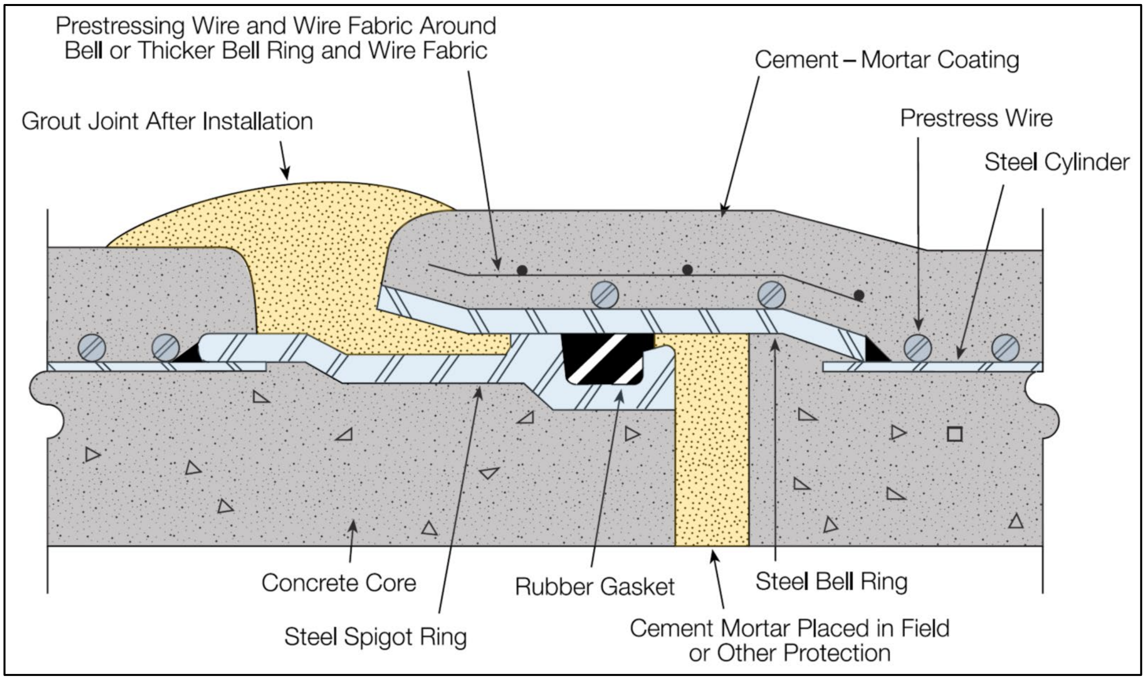
4.10. Concrete Pressure Pipe (CPP).

4.10.1. General. There are four types of CPP commonly used within the United States: 1) PCCP, required to meet AWWA C301 and C304; 2) RCCP, required to meet AWWA C300; 3) RCNP, required to meet AWWA C302; and 4) BWCP, required to meet AWWA C303. PCCP is the most common type of CPP. Prior to the introduction of PCCP, most concrete pressure pipe manufactured in the United States was RCCP. RCNP has primarily been used for low-pressure water supply lines.

4.10.2. Pre-Stressed Concrete Cylinder Pipe (PCCP).

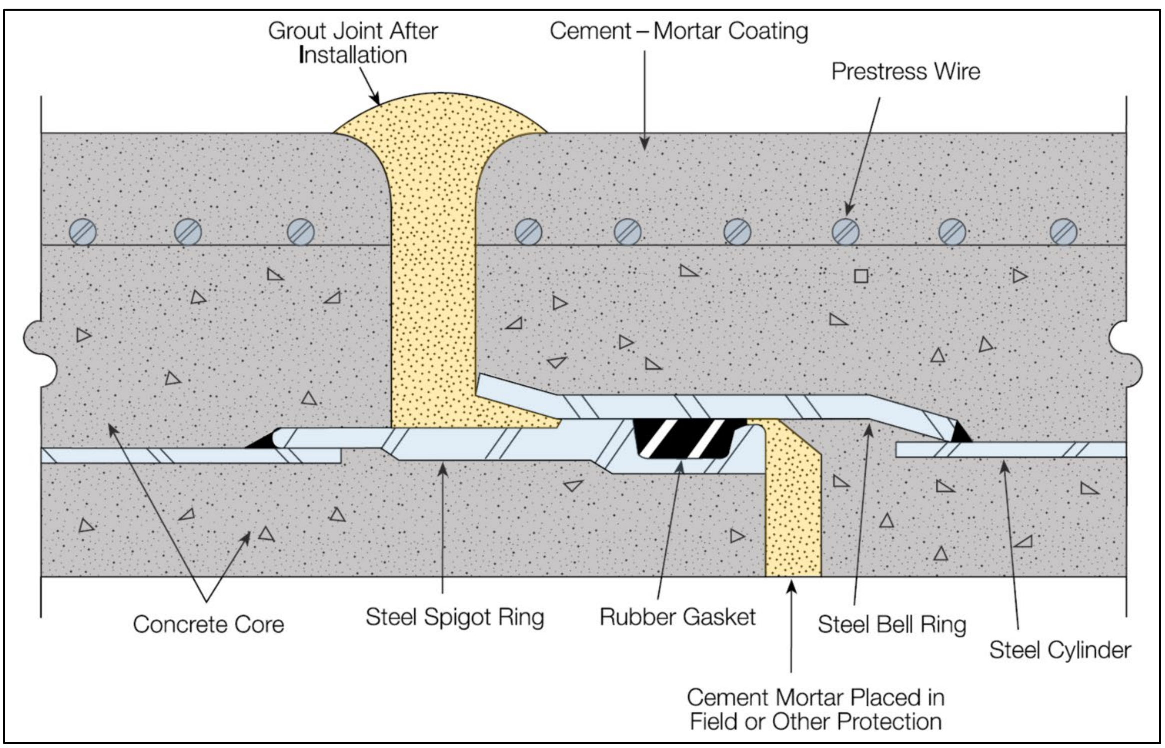
4.10.2.1. General. There are two types of PCCP: 1) lined-cylinder pipe with a core composed of a steel cylinder lined with concrete that is subsequently pre-stressed with high-tensile wire wrapped around the steel cylinder (Figure 4-3); and 2) embedded-cylinder pipe with a core composed of a steel cylinder encased in concrete that is subsequently pre-stressed with high-tensile wire around the exterior concrete surface (Figure 4-4). The design methodology for both types is a limit-state procedure based on limiting the circumferential thrust and bending moment resulting from internal pressure, external loads, and pipe/fluid weights.

4.10.2.2. Pre-Stressed Concrete Cylinder Pipe Limit States. Three sets of limit states are set for PCCP: serviceability, elasticity, and strength. The design procedures for PCCP are detailed in Section 8 of AWWA C304. This includes the required calculations for determining the maximum pressure, maximum thrust, burst pressure, and radial tension load in the pipe for the given loads. AWWA C304 Table 3 (embedded cylinder PCCP) and Table 4 (lined cylinder PCCP) are summary tables detailing the various limit states, design criteria, and applicable load combination for the given limit state, as well as necessary reference calculations.



(Courtesy of AWWA)

Figure 4-3. Lined-cylinder PCCP.



(Courtesy of AWWA)

Figure 4-4. Embedded-cylinder PCCP.

4.10.2.3. Pre-Stressed Concrete Cylinder Pipe Design Criteria. PCCP is designed to limit stress in the pipe concrete and reinforcement under simultaneous application of internal pressures and external loads. The design procedure for PCCP is detailed in AWWA Standard C304. The design methodology in AWWA C304 is an LRFD approach that uses both load and resistance factors for various load combinations and limit states.

4.10.2.4. Pre-stressed Concrete Cylinder Pipe Design Loads and Load Combinations.

4.10.2.4.1. Pre-stressed Concrete Cylinder Pipe Working Loads. Working loads included in the design of PCCP include pipe weight, fluid weight, external dead load (earth plus any external surcharge), and live loads (e.g., vehicular/rail, construction). Working loads for pipe weight, fluid weight, and dead load are computed in the same manner as outlined for NP-RCP/LP-RCP (reference Section 4.9.2). These working loads are briefly described in Section 2.3 of AWWA C304. Internal pressures include the working pressure, surge (transient) pressure, and internal field-test pressure, which is applicable after installation.

4.10.2.4.2. Internal Working Pressure. The internal working pressure for PCCP, P_w, is the greater of either P_g or P_s as shown in Equation 4-4.

Equation 4-4 (Equation 2-1, AWWA C304)

$$P_w = \max (P_g, P_s)$$

Where:

P_w = the internal working pressure
P_g = the maximum pressure associated with the hydraulic gradient
P_s = static head

4.10.2.4.3. Internal Surge (Transient) Pressure. The internal transient pressure, P_t, is the internal pressure in excess of P_w caused by rapid changes in pipe flow velocity (commonly referred to as surge pressure). The transient pressure should be provided to the pipe designer, but if no transient pressure is provided, the value of P_t must be at least the greater of $0.4 \cdot P_w$ or 40 psi as shown in Equation 4-5.

Equation 4-5 (Equation 2-2, AWWA C304)

$$P_t = \max (0.4 \cdot P_w, 40 \text{ psi})$$

Where:

P_t = the internal pressure in excess of P_w
P_w = the internal working pressure

4.10.2.4.4. Pre-stressed Concrete Cylinder Pipe Design Load Combinations. Factored loading and pressure combinations for PCCP are detailed in Section 3 of AWWA C304. PCCP is designed to meet 14 different load combinations listed herein (Equation 4-6 through Equation 4-19 with various load factors provided in AWWA C304 Table 1 for embedded-cylinder PCCP and AWWA C304 Table 2 for lined-cylinder PCCP). Reference Appendix A of AWWA C304 for additional detail regarding how the various load factors were developed for PCCP. The load factors herein are considered conservative, as per AWWA C304.

- Working Loads and Internal Pressures:

Equation 4-6 (Equation 3-1, AWWA C304)
$$W1: W_e + W_p + W_f + P_w$$

Equation 4-7 (Equation 3-2, AWWA C304)
$$W2: W_e + W_p + W_f$$

Equation 4-8 (Equation 3-3, AWWA C304)
$$FW1: 1.25 \cdot W_e + W_p + W_f$$

Where:

W_e = external dead load (lb/ft) as per Equation 4-3
W_p = weight of pipe (lb/ft)
W_f = weight of fluid (lb/ft)

- Working + Transient Loads and Internal Pressures – Load and Pressure Combinations:

Equation 4-9 (Equation 3-4, AWWA C304)
$$WT1: W_e + W_p + W_f + P_w + P_t$$

Equation 4-10 (Equation 3-5, AWWA C304)
$$WT2: W_e + W_p + W_f + W_t + P_w$$

Equation 4-11 (Equation 3-6, AWWA C304)
$$WT3: W_e + W_p + W_f + W_t$$

- Working + Transient Loads and Internal Pressures – Factored Load and Pressure Combinations:

Equation 4-12 (Equation 3-7, AWWA C304)
$$FWT1: \beta_1 \cdot (W_e + W_p + W_f + P_w + P_t)$$

Equation 4-13 (Equation 3-8, AWWA C304)
$$FWT2: \beta_1 \cdot (W_e + W_p + W_f + W_t + P_w)$$

Where:

W_t = transient load (lb/ft)
β_1 = 1.1 for embedded cylinder PCCP, 1.2 for lined cylinder PCCP

Equation 4-14 (Equation 3-9, AWWA C304)
$$FWT3: \beta_2 \cdot (W_e + W_p + W_f + P_w + P_t)$$

Equation 4-15 (Equation 3-10, AWWA C304)
$$FWT4: \beta_2 \cdot (W_e + W_p + W_f + W_t + P_w)$$

Equation 4-16 (Equation 3-11, AWWA C304)
$$FWT5: 1.6 \cdot (W_e + W_p + W_f) + 2.0 \cdot W_t$$

Equation 4-17 (Equation 3-12, AWWA C304)
$$FWT6: 1.6 \cdot P_w + 2.0 \cdot P_t$$

Where:

β_2 = 1.3 for embedded cylinder PCCP, 1.4 for lined cylinder PCCP

- Working Loads and Internal Field-Test Pressures:

Equation 4-18 (Equation 3-13, AWWA C304)
$$FT1: 1.1 \cdot (W_e + W_p + W_f + P_{ft})$$

Equation 4-19 (Equation 3-14, AWWA C304)
$$FT2: 1.1 \cdot \beta_1 \cdot (W_e + W_p + W_f + P_{ft})$$

Where:

β_1 = 1.1 for embedded cylinder PCCP, 1.2 for lined cylinder PCCP

4.10.2.5. Pre-stressed Concrete Cylinder Pipe Design Limit States.

4.10.2.5.1. General. PCCP is designed for both serviceability, elastic, and strength limit states. The serviceability limit states for PCCP are as follows: core-crack control, radial tension control, coating crack control, core compression control, and maximum pressure. The elastic limit states for PCCP are as follows: wire-stress control and steel cylinder stress control. Strength limit states for PCCP are as follows: wire yield strength, core compressive strength, bursting pressure, and coating bond strength.

4.10.2.5.2. Pre-stressed Concrete Cylinder Pipe Serviceability Limit States.

- Core Crack Control. The tensile strain at the inside surface of the core must meet the following guidelines for the given load combination:

W1: $\varepsilon_{w'} \leq 1.5 \cdot \varepsilon_{t'}$

FT1, WT1, and WT2: $\varepsilon_{k'} \leq 11.0 \cdot \varepsilon_{t'}$

Where:

$\varepsilon_{w'}$ = tensile strain limit in concrete core for working conditions only

$\varepsilon_{t'}$ = tensile elastic strain corresponding to tensile strength of concrete, f'_t

$\varepsilon_{k'}$ = tensile strain at concrete core at first visible cracking

- Radial Tension. The maximum radial tensile stress at the interface between the inner core and cylinder for an embedded cylinder PCCP is 12 psi for load combinations FW1 and WT3.

- Coating Crack Control. The tensile strain at the outside coating must meet the following criteria for the specified load combinations:

W1: $\varepsilon_{w'm} \leq 0.8 \cdot \varepsilon_{k'm}$
FT1, WT1, and WT2: $\varepsilon_{k'm} \leq 8.0 \cdot \varepsilon_{t'm}$

Where:

$\varepsilon_{w'm}$ = tensile strain limit of coating mortar for working conditions only
$\varepsilon_{k'm}$ = tensile strain limit of coating mortar at first visible cracking
$\varepsilon_{t'm}$ = tensile elastic strain of the coating mortar corresponding to the tensile strength of the coating mortar, $f'_{t\,m}$

- Maximum Pressure. The maximum internal pressure is limited to the following for the specified load combinations of embedded cylinder PCCP:

Embedded Cylinder PCCP	W1: P_o	
	WT1: min $(1.4 \cdot P_o, P_k')$	
Lined Cylinder PCCP	W1: $0.8 \cdot P_o$	
	WT1: min $(1.2 \cdot P_o, P_k')$	

Where:

P_o = decompression pressure (psi)
P_k' = maximum internal pressure limit for working plus transient conditions (psi)

4.10.2.5.3. Pre-stressed Concrete Cylinder Pipe Elasticity Service Limit States.

- Pre-stressing Wire Tensile Stress Control. The maximum tensile stress in the pre-stressing wire must remain less than the gross wrapping stress (f_{sg}) for load combinations FWT1, FWT2, and FT2. The maximum compression in the core must not exceed $0.75 \cdot f_c'$ for the same load combinations.

- Steel Cylinder Stress Control for Embedded Cylinder PCCP. The maximum tensile stress in the steel cylinder of an embedded-cylinder PCCP must be less than the design yield strength for the cylinder (f_{yy}) for load combinations WT1, WT2, and FT1. The tensile stress in the cylinder due to external loads only (with zero pressure) should remain below the compressive pre-stress in the cylinder for load combination WT3. This is to keep the separation of the cylinder from the outer core.

4.10.2.5.4. Pre-stressed Concrete Cylinder Pipe Strength Limit States.

- Wire Yield Strength. For load combinations FWT3 and FWT4, the maximum tensile stress in the pre-stressing wire must remain below the wire's yield strength (f_{sy}).

- Core Compressive Strength. For load combination FWT5, the maximum combined moment and thrust at the springline must remain less than the ultimate compressive strength of the core concrete.

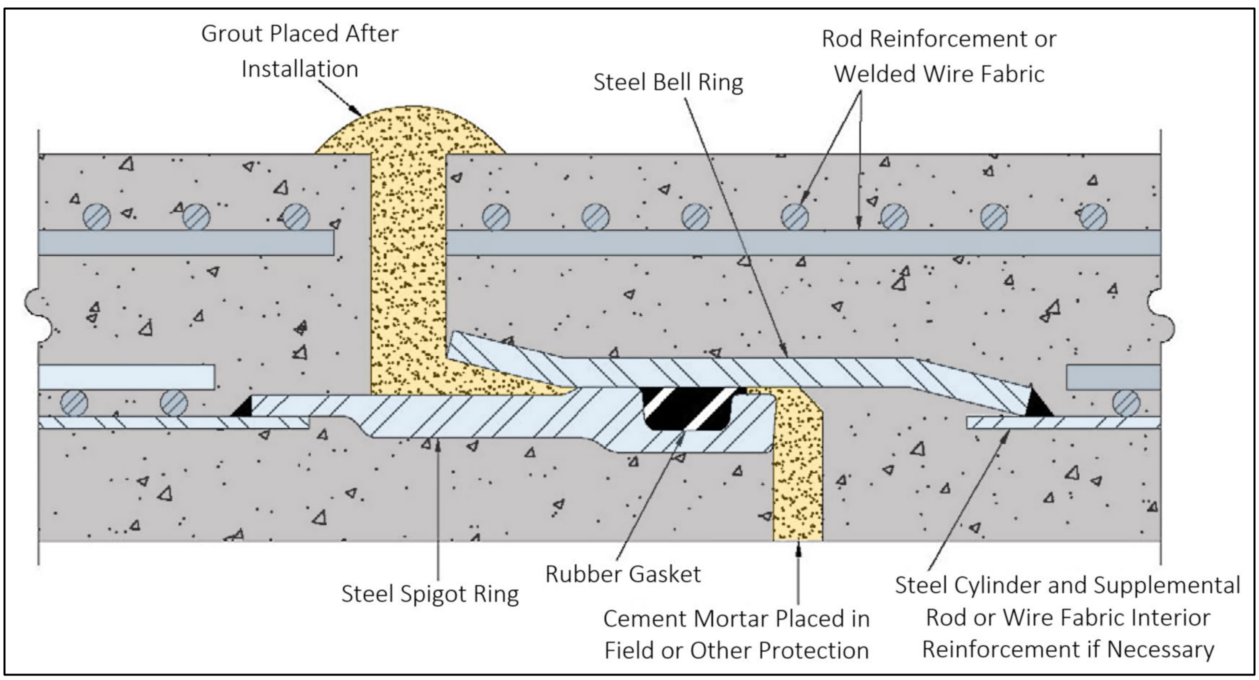
- Burst Pressure. The stress in the pre-stressing wire must remain lower than the specified minimal tensile strength of the wire for load combination FWT6.

- Coating Bond Strength. The minimum spacing for the pre-stressing wires in the same layer must be $d/d_s \geq 2$ for embedded cylinder PCCP and $d/d_s \geq 2.75$ for lined cylinder pipe, where d = center-to-center wire spacing and d_s = wire diameter. The maximum center-to-center spacing of the pre-stressing wires in the same layer must be 1.5 inches, except for lined-cylinder pipe with $d_s \geq 0.25$ inches, in which the maximum spacing for the pre-stressing wires must be one inch.

4.10.2.6. Pre-stressed Concrete Cylinder Pipe Material Properties and Design Procedures. The details regarding the manufacturing process and minimum material properties for PCCP are covered in Section 5 of AWWA C304. This includes the engineering properties of the concrete, mortar, steel cylinder, and pre-stressing wire. The concrete properties include: compressive strength (28-day), minimum compressive strength at time of wrapping, tensile strength, and modulus of elasticity. The mortar properties include: compressive strength, tensile strength, and modulus of elasticity. The steel cylinder properties include: yield strength, burst strength, and modulus of elasticity. Finally, the pre-stressing wire properties include: gross wrapping stress, wire yield strength, and modulus of elasticity.

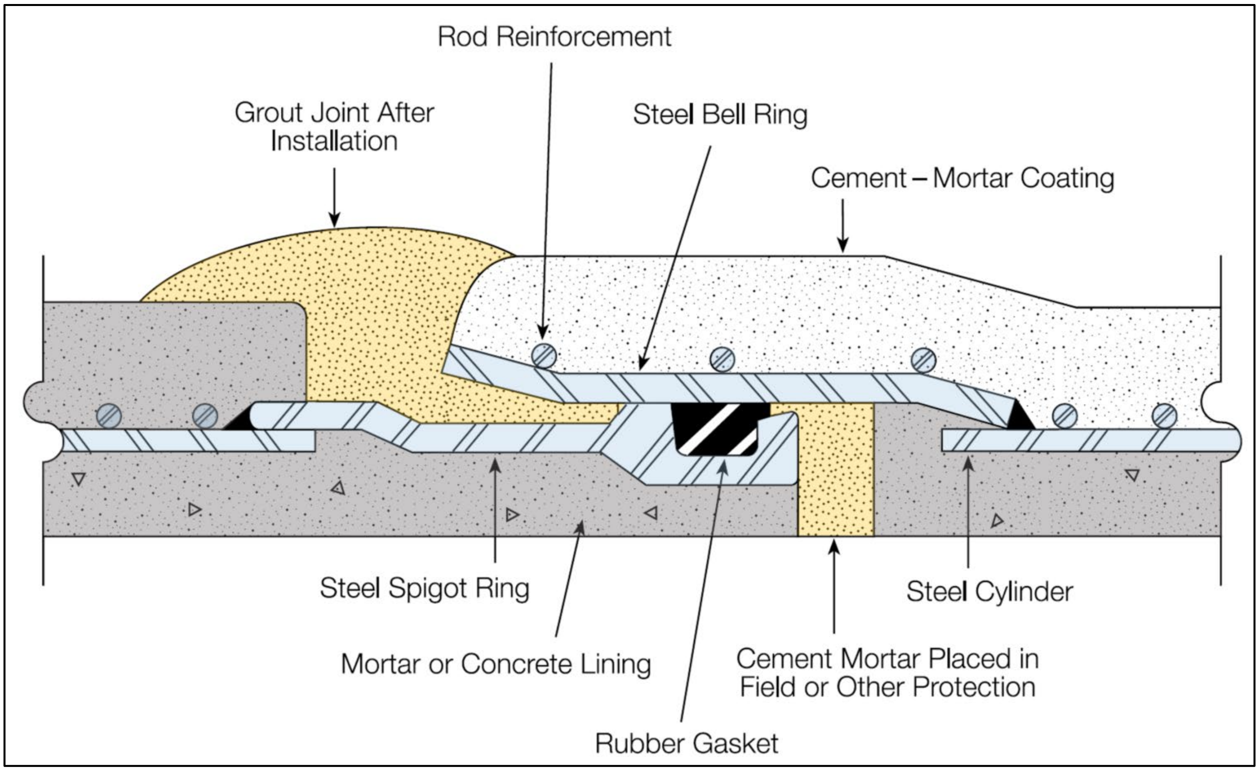
4.10.3. Reinforced Concrete Pressure Pipe.

4.10.3.1. General. The three types of reinforced concrete pressure pipe of concern within this manual are RCCP, RCNP, and BWCP. Most types of reinforced concrete pressure pipe are considered rigid pipes in which a steel cylinder is used as the primary means of reinforcing (Chapter 7 of AWWA Manual M9). RCCP is typically used for high pressure applications (> 54.2 psi) and is similar to PCCP except that a reinforcing cage is cast into the walls as opposed to pre-stressing wires. Figure 4-5 shows a detail of RCCP (Figure 2-5 in AWWA Manual M9). RCNP is used for lower pressure applications (< 54.2 psi) associated with water supply, sanitary/storm sewers, discharge lines, and drainage applications. BWCP uses a steel cylinder with an interior cement mortar lining and, like RCCP, is used for high pressure (> 54.2 psi) applications, but is more flexible and therefore sometimes referred to as semi-rigid (Figure 4-6 [Figure 2-7, AWWA Manual M9]).



(Courtesy of AWWA)

Figure 4-5. Detail of RCCP.



(Courtesy of AWWA)

Figure 4-6. Detail of BWCP.

4.10.3.2. Reinforced Concrete Pressure Pipe Design Criteria. The design procedures for RCCP and RCNP use similar methods and the governing document for their structural design is AWWA Manual M9. Because BWCP is considered semi-rigid, its design varies from other CPPs, but its design requirements are still covered by AWWA Manual M9. Each reinforced CPP type uses an LRFD design approach.

4.10.3.3. Reinforced Concrete Pressure Pipe Loads.

4.10.3.3.1. Internal Working Pressure. The internal working pressure for both RCCP and RCNP is calculated in the same manner as previously detailed for PCCP.

4.10.3.3.2. Surge Loads. Loading associated with surge pressure is a result of a change in fluid velocity while closing a shutoff valve. The velocity change causes waves to travel upstream and downstream from the point of origin, resulting in an increase or decrease in pressure. This is commonly referred to as water hammer, surge, or transient pressure and can be caused by a wide variety of pipe operations, including: normal pump start/stop, valve operations, pump shutoff due to power loss, air vent issues, pressure-relief valve malfunction, valve operations, and other issues. The equations governing the surge pressure calculations for reinforced CPP design are provided in Chapter 4 of the AWWA Manual M9.

4.10.3.3.3. External Loads. Applicable external loads for reinforced CPP design are covered in Chapter 5 of the AWWA Manual M9. Similar to the design of other pipes, external loads for reinforced CPP are a function of trench/embankment conditions. The external loads are calculated in a similar manner to those for NP-RCP and LP-RCP.

4.10.3.4. Reinforced Concrete Pressure Pipe Design Procedure (Reinforced Concrete Cylinder Pipe, Reinforced Concrete Non-cylinder Pipe).

4.10.3.4.1. Determine Circumferential Steel Area Required. The initial design step is to determine the required total circumferential steel area to resist internal pressure using the hoop tension equations for working pressure alone and for working pressure plus surge pressure. AWWA Manual M9 Equations 7-1 through 7-4 are used to determine the steel area.

4.10.3.4.2. Verify Tensile Strength of Concrete is Adequate. Using the hoop tension equation, the circumferential tensile stress in the concrete is calculated next for working pressure plus surge pressure. Equation 7-5 in AWWA M9 is used to calculate the tensile stress (RCNP only), and the result is then compared to the allowable tensile stress for the concrete. If the computed stress exceeds the allowable stress, then a higher strength concrete must be used.

4.10.3.4.3. Calculate External Loads. Next, calculate applicable external loads (earth load, pipe weight, fluid load, and any live load) acting on the pipe. The procedures for calculating external loads are covered in Chapter 5 of AWWA M9 and are similar to those for computing typical precast RCP design.

4.10.3.4.4. Compute Moment and Thrust Loads. Moment and thrust loads are calculated at the invert and springline of the pipe. The procedures for calculating these loads are detailed in Chapter 5 of AWWA M9. The designer is required to check radial tension and shear capacity of the pipe wall for the load condition that considers earth load, pipe weight, and live loads.

4.10.3.4.5. Determine Required Steel Area for Load Combinations. After the moment and thrust loads are computed, determine the required circumferential steel area for the invert and springline for the following load condition combinations:

- L1: combination of working pressure, dead loads, and live loads
- L2: combination of dead loads and live loads with no internal pressure
- L3: combination of working pressure, surge pressure, and dead loads (also applicable for field test load condition)
- L4: combination of earth load, pipe weight, and live load

4.10.3.4.6. Select Controlling Steel Area. Once the required steel area for load combinations is determined, select the controlling maximum steel area required for both the invert and springline calculations, which must be greater than or equal to the steel area required for hydrostatic design.

4.10.3.4.7. Select Bars/Fabric to Meet Steel Area Requirements. Check the concrete cover over the steel to make sure it meets the following guidelines. For pipes subject to AWWA C300, the rod reinforcement must equal at least 40 percent of the total circumferential steel area. Concrete cover and reinforcing spacing requirements are detailed in Chapter 7 of AWWA Manual M9 for various types of reinforced concrete pressure pipe.

4.10.3.4.8. Check American Concrete Institute (ACI) 318 Criteria. Finally, check the reinforcement areas to ensure the minimum requirements of ACI 318 are met for flexural reinforcement.

4.10.3.5. Bar-Wrapped Cylinder Pipe Design Procedure. The design procedure for BWCP is slightly different from RCCP and RCNP because it is considered a semi-rigid pipe; therefore, deflection criteria must be considered in the design process (reference Table 4-1). Design example calculations for BWCP are provided in AWWA M9 and the required design steps are summarized herein.

4.10.3.5.1. Select Steel Cylinder Thickness. The initial step in the structural design of BWCP is to select a steel cylinder greater than or equal to the minimum required by AWWA C303 criteria.

4.10.3.5.2. Calculate Required Circumferential Steel Area. The next step is to calculate the required circumferential steel area using the hoop tension formula for working pressure (Equation 7-1, AWWA M9) and working pressure plus surge pressure (Equation 7-2, AWWA M9). The larger steel area between these two must be selected.

4.10.3.5.3. Calculate Cylinder Steel Area. Once the required circumferential steel area is calculated, calculate the cylinder steel area and place the "remaining" required steel area in the bar by selecting a bar diameter and bar spacing using the following criteria:

- Area of bar reinforcement < 60 percent of total area of circumferential steel
- Area of bar reinforcement ≥ 0.23 in.^{2}/lf of pipe
- Center-to-center spacing of the bar ≤ 2 inches
- Area of bar reinforcement (in.^{2}/ft of pipe) must be numerically equal to at least 1 percent of the inside pipe diameter in inches
- Clear space between bars ≥ bar diameter
- Reinforcing bar diameter ≥ 7/32 of an inch

4.10.3.5.4. Calculate External Loads. Next, calculate external loads on the pipe using the information provided in Chapter 5 of AWWA M9.

4.10.3.5.5. Determine Allowable External Load. After calculating the external loads, determine if the total external load from the previous step exceeds the allowable external load value using AWWA M9 Design Table 7-3 or the maximum allowed external load as calculated using AWWA M9 Equation 7-24, which is based upon a pipe deflection limit.

4.10.3.5.6. Revise Pipe Design if Necessary. If required, provide additional external load-carrying capability by increasing the effective moment of inertia of the longitudinal pipe section or by improving the bedding material/compaction requirements. The effective moment of inertia may be increased by increasing the area or diameter of the bar reinforcement or increasing the coating thickness to a maximum of 1.25 inches over the bar wrap.

4.10.4. Special Considerations for Concrete Pressure Pipe. Similar to other pressurized pipes, thrust forces must be considered when designing a reinforced CPP that will be subjected to internal pressure. This occurs at changes in pipe alignment, changes in cross-sectional areas, and pipeline termination points such as bulkheads. The thrust forces must be restrained to protect the pipe joints. Thrust forces from angular joint deflections are resisted by passive soil resistance for buried pipes. Thrust forces at in-line fittings are typically restrained by frictional drag on the longitudinally-compressed joint. The fitting must be designed to transmit the anticipated thrust forces. Thrust blocks can be added to supplement the ability of the fitting to resist the forces. Details associated with thrust forces and thrust restart are covered in Chapter 9 of AWWA Manual M9.

4.11. Vitrified Clay Pipe (VCP).

4.11.1. General. VCP has not been used in recent USACE embankment or floodwall projects because of performance issues associated with old terra cotta clay pipes. As such, previous USACE guidance did not identify VCP as a viable pipe material option. Changes to the manufacturing process (vitrification) and improved joint design of contemporary VCP makes it superior to the older terra cotta clay material. Therefore, VCP is now considered a viable option for pipes associated with USACE dams and levees, especially in highly corrosive environments or situations where abrasive flow is expected. The designer should be aware that because of the previous restrictions, modern VCP does not have a long history of use in USACE projects.

4.11.2. Applicable Design Criteria and Standards for Vitrified Clay Pipe.

4.11.2.1. General. The design procedures for VCP are governed by the latest version of the VCP Engineering Manual. When a VCP is designed for use within or beneath a USACE embankment or floodwall, a minimum safety factor of 1.5 must be met. Meeting this criterion is accomplished by using an acceptable combination of VCP and bedding classification for the vertical load acting on the pipe. It is important to note that when placing a VCP within or beneath a levee/dam embankment or floodwall, the material surrounding the pipe should never be more pervious than the adjacent embankment material/natural ground with the exception of materials used for as part of the pipe's seepage filter.

4.11.2.2. Vertical Loads Acting on Vitrified Clay Pipe. It is important to note that the overall process for estimating loads using the procedures outlined in the VCP Engineering Manual should result in a conservative estimate of the vertical load if the procedures are followed correctly. Loads are calculated either using the Marston methodology (traditional soil prism load) or the modified Marston technique based upon Spangler's work. The VCP Engineering Manual describes the appropriate methodology to use for various installation/construction conditions.

4.11.2.3. Calculating the Field Supporting Strength of Vitrified Clay Pipe. The load a VCP can safely support varies according to the bedding class. The load factor (not the same as LRFD load factor) for VCP structural design is the ratio of the supporting strength of the pipe to its three-edge bearing test strength. The three-edge bearing test represents the base value for each VCP and consequently a load factor equivalent to a value of 1.0. To determine the field supporting strength, the load factor for the given bedding class and VCP is multiplied by the minimum bearing strength provided for the VCP from three-edge bearing test results, as outlined in ASTM C700. The load factors for various bedding materials are provided in the VCP Engineering Manual.

4.11.2.4. Determining the Safety Factor. The safety factor for VCP structural pipe design is calculated by dividing the field supporting strength by the vertical load acting on the pipe. The procedures for calculating the vertical load acting on the pipe are outlined in the VCP Engineering Manual. In addition, Appendix D of this manual provides a VCP design example using CLSM backfill.

4.12. Corrugated/Ribbed Metal Pipe (CMP).

4.12.1. General. CMP is an overarching term that includes both CSP and CAP; however, CAP has had limited use in USACE projects while CSPs have been used extensively over many years. While the structural design for each follows the same procedures outlined within this manual, their individual engineering properties result in differing advantages and disadvantages depending upon the operating environment.

4.12.2. Applicable Design Criteria and Standards for Corrugated/Ribbed Metal Pipe. The structural design of CMPs is governed by Section 12.7 of the AASHTO LRFD guidance. This includes the structural design of both steel and aluminum corrugated circular and arch sections, as well as SRTP (reference Section 4.18). Reference Table 4-1 for CAP and CSP deflection limits. There are multiple strength limit states for which CMPs must be designed, including thrust (wall area), buckling strength, and seam resistance.

4.12.3. Strength Limit States for Corrugated/Ribbed Metal Pipe.

4.12.3.1. Thrust (Wall Area).

4.12.3.1.1. General. CMPs must be designed such that the factored wall resistance is sufficient to resist the factored thrust load. The computation for factored thrust follows the procedures in Section 12.7.2.2 of the AASHTO document, shown here as Equation 4-20.

Equation 4-20 (AASHTO Equation 12.7.2.2-1)

$$T_L = \gamma_{DL}\left[\frac{P_{FD}\,S}{2}\right] + \gamma_{LL}\left[\frac{P_{FL}C_L F_1}{2}\right]$$

Where:

T_L = factored thrust load per unit of length (kip/ft)
C_L = $l_w \leq S$ (width of culvert on which the live load is applied parallel to span, ft)
l_w = live load path length at depth H as specified in AASHTO Section 3.6.1.2.6
S = culvert span (ft)
γ_{DL} = 1.95 dead load factor as per AASHTO Table 3.4.1-2
P_{FD} = dead load vertical crown pressure as per Section 12.12.3.4 AASHTO (ksf) with VAF=1.0 and D_o=S
γ_{LL} = 1.75 live load factor as per AASHTO Table 3.4.1-1
P_{FL} = live load vertical crown pressure as per Section 12.12.3.4 AASHTO (ksf)
F_1 = $(0.75S/l_w) \geq F_{min}$

$F_{min} = (15/12S) \geq 1.0$

4.12.3.1.2. Long Span Corrugated/Ribbed Metal Pipes. Long-span corrugated metal structures have a deeper corrugation profile compared to other CMPs and require special features such as longitudinal stiffeners or reinforcing ribs. AASHTO Section 12.8 provides a definition of what constitutes a long-span corrugated metal structure, including associated cross-sectional shapes. Horizontal ellipses, low profile arches, high profile arches, inverted pear shapes, and pear arch sections are applicable long-span cross-sectional shapes. For long-span CMPs, F_1 must be calculated using AASHTO Equation 12.7.2.2-5.

4.12.3.1.3. Live Load Effects. The live load effects on the pipe can be neglected when the top of the CMP is embedded more than eight feet deep in the embankment and the embedment depth is greater than the span of the pipe. The distribution of the live load through the soil structure at depths exceeding eight feet makes these loads insignificant compared to dead loads.

4.12.3.1.4. Load Modifiers. Once the factored thrust load (T_L) is computed, load modifiers to account for ductility, redundancy, and operational importance must be applied. For CMP, the load modifier applied should be determined by Equation 4-21.

Equation 4-21

$$\eta_{cmp} = \eta_D \cdot \eta_R \cdot \eta_I = 1.0 \cdot 1.0 \cdot 1.05 = 1.05$$

4.12.3.1.5. Pipe Selection. Once the final thrust load is computed (factored thrust load with modifier applied), a CMP should be selected that is sufficient to resist the factored load. Once this is done, the engineering properties for the selected pipe must be used to ensure they meet the remaining strength limit states.

4.12.3.2. Thrust/Wall Buckling Resistance. The factored thrust resistance must exceed the factored thrust load using the engineering properties from the selected pipe, per Equation 4-22. The lesser value between the pipe material yield strength (F_y) and critical buckling stress (f_{cr}) must be used in the calculation.

Equation 4-22

$$\eta_{cmp} T_L < \phi\, A\, (F_y \text{ or } f_{cr})$$

Where:

ϕ = resistance factor for wall area (1.0 for all CMPs except long-span structures and tunnel-liner plate structures where 0.67 must be used)

A = wall area for selected pipe (in.^{2})

F_y = yield strength of pipe material (ksi)

f_{cr} = critical buckling stress (ksi) using AASHTO Equation 12.7.2.4-1 or 12.7.2.4-2 depending upon the pipe diameter or span of the plate structure

4.12.3.3. Seam Resistance. For CMPs fabricated with longitudinal seams, the factored resistance of the seam must be sufficient to develop the factored thrust (T_L) in the pipe wall.

4.12.3.4. Handling and Installation.

4.12.3.4.1. General. Similar to other flexible pipes, a flexibility factor (FF) value is calculated to evaluate loads for transport and placement. The flexibility factor for CMPs is computed using Equation 4-23.

Equation 4-23 (AASHTO Eq. 12.7.2.6-1)

$$FF = \frac{S^2}{E_m I}$$

Where:

S = diameter of pipe or span of plate structure (in.)
E_m = modulus of elasticity for pipe material (ksi)
I = moment of inertia for pipe section ($in.^4$)

4.12.3.4.2. Flexibility Factor. The computed flexibility factor should not exceed the values shown in Table 4-6 (AASHTO Table 12.5.6.1-1) and Table 4-7 (AASHTO Table 12.5.6.2-1) for the given materials and installation conditions.

Table 4-6
Flexibility factor limits for CMP and structural plate pipe materials (Courtesy of AASHTO)

Material	Corrugated Size (inches)	FF Limit (inch/kip)
Steel Pipe	0.25	43
	0.5	43
	1.0	33
Aluminum Pipe	0.25 & 0.5 (0.060 thickness)	31
	0.25 & 0.5 (0.075 thickness)	61
	0.25 & 0.5 (all others)	92
	1.0	60
Steel Plate	6.0 x 2.0 (Pipe)	20
	6.0 x 2.0 (Pipe-Arch)	30
	6.0 x 2.0 (Arch)	30
Aluminum Plate	9.0 x 2.5 (Pipe)	25
	9.0 x 2.5 (Pipe-Arch)	36
	9.0 x 2.5 (Arch)	36
Steel-Reinforced Plastic	n/a	68

Table 4-7
Flexibility factor limits for spiral rib metal pipe and pipe arches (Courtesy of AASHTO)

Material	Installation Condition	Corrugated Size (inches)	FF (inch/kip)
Steel	Embankment	0.75 x 0.75 x 7.5	$217 I^{0.333}$
		0.75 x 1.0 x 11.5	$140 I^{0.333}$
	Trench	0.75 x 0.75 x 7.5	$263 I^{0.333}$
		0.75 x 1.0 x 11.5	$163 I^{0.333}$
Aluminum	Embankment	0.75 x 0.75 x 7.5	$340 I^{0.333}$
		0.75 x 1.0 x 11.5	$175 I^{0.333}$
	Trench	0.75 x 0.75 x 7.5	$420 I^{0.333}$
		0.75 x 1.0 x 11.5	$215 I^{0.333}$

4.12.4. Corrugated/Ribbed Metal Pipe Design Manuals. Design manuals for both steel and aluminum CMP products provide a broad design overview with helpful guidance tables detailing available sizes and shapes. The vast majority of CMPs used in USACE embankment and floodwall projects are round and arch culvert styles. The design manuals provide a more detailed look into the various options available, along with additional options such as concrete lining for additional invert durability. Steel and aluminum pipe manufacturers' websites provide the most up-to-date product listings of available shapes and sizes and should be referenced when designing CMPs.

4.12.5. Minimum Cover Requirements. Minimum cover requirements are provided in AASHTO Section 12.6.6.3.

4.12.6. End Treatment. Designing the ends of CMPs requires additional consideration beyond the ring compression design used for the main body of the pipe because the potential exists for unbalanced soil loading in addition to other hydraulic forces associated with flow, uplift, and scour consideration. Unbalanced loading is applicable when the pipe is skewed relative to the embankment alignment. Unbalanced soil conditions are considered insignificant for most drainage applications where relatively small pipe diameters (less than 48 inches) are used or the skew angle is minimal (less than 10 degrees). If the alignment of a large diameter (48 inches or larger) CMP is skewed relative to the embankment alignment more than 10 degrees, refer to the information in the latest version of the NCSPA Design Manual.

4.12.7. Metal Box Culverts. AASHTO has adopted a simplified design method for corrugated metal box culverts based upon the geometric limit determined through a series of historical studies. This method limits the span length to 25.42 feet and rises up to 10.5 feet. Cover limits range from 1.4 feet to a maximum of 5 feet. Table 4-8 (Table 7.9 of the NCSPA Manual) provides the geometry limits using this methodology. If the metal box culvert falls within these size limits, AASHTO Section 12.9 is the applicable design guide. Metal box culverts that exceed any of the dimensions within the table must be designed using more rigorous finite element modeling techniques. ASTM A964 covers the material, geometric, and wall section properties of corrugated steel box culverts. There are five different types of corrugated steel box culverts, each with slightly different shapes and varying structural stiffness. ASTM

B864 is the specification that covers the same properties for corrugated aluminum box culverts. There are also multiple types of corrugated aluminum box culverts with varying cross-sections.

Table 4-8
Geometric limits for corrugated steel box culverts (Courtesy of NCSPA)

Standard Corrugated Steel Box Culvert Geometry Limits[1]		
Dimension	Minimum	Maximum
Span	8 ft – 0 in.	25 ft – 5 in.
Rise	2 ft – 6 in.	10 ft – 6 in.
Crown Radius	n/a	24 ft – 9.5 in.
Haunch Radius	2 ft – 6 in.	n/a
Included Angle of Haunch	50º ∞	70º ∞
Leg Length (to bottom of plate)	0 ft – 4.75 in.	5 ft – 11 in.

[1] See NCSPA Manual for associated figures and dimension references.

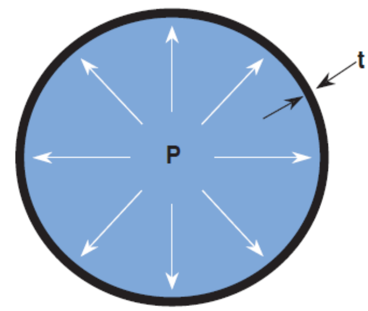
4.13. Welded Seam Steel Pipe (WSSP).

4.13.1. General. Non-corrugated (smooth) steel pipe is currently manufactured by two methods, welded and seamless. Previously, WSSP was determined to be inferior to seamless steel pipe because the welded seam was considered the weakest portion of the pipe. However, this no longer an issue due to significant improvements in the welding and forging process. The major benefit of WSSP compared to seamless steel pipe is a much larger selection of sizes and lengths, as well as it being far more cost effective than seamless steel pipe; therefore, only the design of WSSP is covered in this chapter. WSSP can be manufactured with different linings and coatings based on the operating environment.

4.13.2. Applicable Design Criteria and Standards for Welded Seam Steel Pipe. WSSP structural design, addressed in AWWA Manual M11, is similar to several other flexible pipes that are primarily used for pressure applications (i.e., ductile iron, fiberglass, etc.) in that it is typically designed to handle internal pressures. The procedure in AWWA M11uses an allowable stress design (ASD) methodology, where the yield strength of the steel is reduced by a safety factor to an allowable value which must exceed the pressures from the calculated unfactored loadings. As of the publication of this manual, the AASHTO LRFD specifications do not have an outlined procedure for WSSP. The reference to structural plate pipe in the AASHTO document is for corrugated pipe plate sections, not for WSSP.

4.13.3. Welded Seam Steel Pipe Design and Load Considerations.

4.13.3.1. Internal Pressure. When designing a welded seam steel pipe, the main concern is the ability to safely withstand internal pressures. Once the final internal pressure is determined (i.e., working pressure plus any surge loading pressure), the wall thickness required to safely resist the loading is calculated. Chapters 3 and 4 of AWWA Manual M11 address the process to determine internal working pressures and pipe wall thickness. The wall thickness selected must be sufficient to safely resist the tensile hoop stress and is estimated for both of the long-term internal pressures (working plus surge allowance) using a 50 percent reduction in yield strength as a safety factor for working pressure and 75 percent of yield strength for test and transient pressures. The larger of the two calculated thickness values must be used. The calculation (Equation 4-1 from AWWA M11) is reproduced as Equation 4-24.



Equation 4-24

$$t = (P \cdot D_o) / (2 \cdot S)$$

Where:

- t = net pipe wall thickness (in.)
- P = internal pressure (working + surge, psi)
- D_o = outside diameter of pipe (in.)
- S = allowable design stress (psi)

4.13.3.2. Transportation and Handling. The next step is to evaluate the selected thickness for handling loads. The equations for determining the thickness required for transportation and handling are in Equation 4-25 through Equation 4-27 (D_n represents the nominal pipe diameter):

Equation 4-25 (Equation 4-2, AWWA M11)

$$t = D_n/288$$

[Pipes < 54-inch inside diameter (no lining/flexible linings)]

Equation 4-26 (Equation 4-3, AWWA M11)

$$t = (D_n + 20)/400$$

[Pipes ≥ 54-inch inside diameter (no lining/flexible linings)]

Equation 4-27 (Equation 4-4, AWWA M11)

$$t = D_n/240$$

[Cement mortar lined pipe/flexible coating]

The larger of the required pipe wall thicknesses from internal pressures versus handling loads is carried forward in the design. AWWA M11 specifically notes that an allowance for corrosion is not required due to the availability of high-quality coatings and cathodic protection systems in use today. This can be modified if necessary, depending on the environment in which the pipe is placed.

4.13.3.3. External Loads. Next, the pipe is evaluated for external loads. For buried pipes, this will be a function of the pipe stiffness and passive soil resistance under and adjacent to the pipe. AWWA M11 discusses the Marston loading theory and provides the relevant equations, but also states that the soil prism method (weight of the soil column above the pipe) is a conservative design method that is easier to calculate and can be used. The manual also notes that over time, soil loads may approach the soil prism load due to consolidation. The soil prism (dead) load is estimated using Equation 4-28.

Equation 4-28 (Equation 5-3, AWWA M11)

$$W_c = (\omega \cdot H_c \cdot D_c)/12$$

Where:

- W_c = soil prism load along length of pipe (lb/ft)
- ω = unit weight of the soil (lb/ft^3)
- H_c = height of fill above top of pipe (ft)

D_c = outside diameter of coated pipe (in.)

4.13.3.4. Live Load. The effects of the live loads (WL) on the pipe are a function of the depth of soil cover over the top of the pipe. Table 4-9 (AWWA Table 5-1) shows estimated live loads with 50 percent impact factors for both wheel and rail loading as a function of height of soil cover over the top of the pipe. Heavy construction equipment passing over top of the pipe should be evaluated when necessary. If the soil cover height is at least three feet above the top of the pipe, construction equipment can usually pass over the pipe without issue; however, additional analysis may be required. Methods to estimate loads are provided in Chapter 5 of the AWWA M11 Manual if required.

Table 4-9
Live load estimates for buried pipes (AWWA M11 Manual Table 5-1) (Courtesy of AWWA)

Highway HS-20 Loading[1]		Railroad E-80 Loading[1]	
Cover Height (ft)	Load[2] (lb/ft^2)	Cover Height (ft)	Load[2] (lb/ft^2)
1	1,800	2	3,800
2	800	5	2,400
3	600	8	1,600
4	400	10	1,100
5	250	12	800
6	200	15	600
7	176	20	300
8	100	30	100

1 Neglect live load when less than 100 lb/ft^2; use dead load only.
2 Other HS and E loads can be calculated by applying a ratio such as 25/20 to HS-20 for HS-25 loading.

4.13.3.5. Deflection Criteria. Once the external loads (dead and live) are accurately calculated, the pipe needs to be checked for deflection criteria. The estimated deflection is calculated using the modified Iowa formula. A design example for WSSP is provided in Appendix D. AWWA M11 Manual Equation 5-4 (modified Iowa formula) is used to calculate the predicted deflection for the combination of loads and pipe information. The allowable deflection limits for WSSP are provided in Table 4-1.

4.13.3.6. Controlled Low Strength Material (CLSM) Bedding. The AWWA M11 Manual also provides design considerations for CLSM as a bedding material. Care must be taken during placement and curing of the CLSM to avoid floatation of the pipe and excessive ring deflection. To this end, CLSM should be placed in lifts while the pipe is secured from movement.

4.13.3.7. Buckling Strength. The final check is an evaluation of the buckling strength of the pipe. The equation for this is provided in Chapter 5 of AWWA Manual M11 using procedures outlined by Moore, et al. Several different safety factors are embedded in the buckling equation (AWWA Equation 5-7), including a 0.9 factor to account for variability in stiffness of the compacted soil, as well as reducing the allowable buckling pressure by a value of two.

4.13.4. Special Design Considerations for Welded Seam Steel Pipe.

4.13.4.1. Unbalanced Force Design. Buried steel pipe must be designed for unbalanced thrust forces. These typically occur at changes in pipe alignment, changes in cross-sectional areas, and pipeline termination points. The necessary calculations associated with thrust are provided in Chapter 8 of the AWAA M11 Manual. The chapter also covers thrust resistance design (e.g., thrust blocks, thrust restrained joints).

4.13.4.2. Other Design Situations. Chapter 9 of the AWWA Manual covers design of steel pipes on supports. Supports detailed in the manual include saddles, ring girders, concrete piers, and concrete anchors. The chapter also includes design examples for other situations.

4.14. Ductile Iron Pipe (DIP).

4.14.1. General. DIP is primarily used for water utility distribution systems and has not routinely been used as a gravity drainage structure in USACE embankment and floodwall projects; however, this does not preclude its use as a viable pipe material for gravity flow applications. The hydraulic characteristics of DIP are similar in nature to WSSP. Because DIP has primarily been used for water distribution systems, bedload has usually not been a long-term performance issue. Historical performance issues typically involve external corrosion as opposed to interior degradation.

4.14.2. Applicable Design Criteria and Standards. The Ductile Iron Pipe Research Association (DIPRA) is the main source for DIP design and general information. The design standards for DIP (ANSI/AWWA C150/A21.50 and ASTM A746) are based on its behavior as a flexible pipe. DIP is designed separately for both internal pressures and external loads, and the design standard is considered more conservative than a combined (internal with external at the same time) loading condition.

4.14.3. Factor of Safety Design. The DIP design standard employs a nominal safety factor approach for the anticipated loads versus required strength requirements. The nominal safety factor of 2.0 (essentially a load factor) is applied to the internal pressure, with the minimum wall thickness selected to meet the factored pressure. For external loads, a safety factor of 1.5 is required for ring yield strength, with a minimum of 2.0 based upon ultimate ring strength. Finally, the design ring deflection criteria requires a safety factor of 2.0 based upon deflection test data causing failure in a cement-mortar lining. LRFD methodology was not available for design of DIP at the time of this manual's publishing.

4.14.4. Ductile Iron Pipe Specification Standards. ANSI/AWWA C150/A21.50 and ASTM A746 provide the required standards for manufactured DIP including tolerances for diameter, length, wall thickness, weight, coatings, and test acceptance requirements. Acceptance tests required include a tension test to determine the yield strength and a Charpy impact test to measure toughness of the material. Requirements related to delivery, foundry records, and other factors are also addressed in the standards and/or specifications.

4.14.5. Design and Load Information for Ductile Iron Pipe.

4.14.5.1. Basis of Design. The design guidance methodology results in the required pipe wall thickness for the corresponding design pressure. A series of pre-calculated load tables and a variety of other supporting information to assist with the design are provided in ASTM A746 and

ANSI/AWWA C150/A21.50. The equations governing the loading tables and overall design are also detailed in the previously noted design specifications. The basic design steps are as follows: design for internal pressures (static pressure and surge pressure); design for bending stress due to external loads (earth and live); select the larger resulting net wall thickness; add a 0.08-inch service allowance to the wall thickness; check against deflection criteria with USACE hydraulic factor applied (Table 4-1); add the standard casting tolerance; select proper class from pressure class table.

4.14.5.2. Internal Pressure. The design internal pressure for DIP uses the same equation as previously listed for WSSP (Equation 4-24) of this manual. For DIP, a typical surge allowance of 100 psi (considered an appropriate value for most situations) is added, but higher surge values can be used. This value is then doubled to serve as the safety factor for this design. The modified allowable design stress (S) from Equation 4-24 is 42,000 psi for DIP.

4.14.5.3. External Load Design. External load design checks for two conditions: ring bending stress and ring deflection. The external load design uses the soil prism earth loading above the pipe (P_e), as per Equation 4-29. This result is then combined with a truck load (P_t), as per Equation 4-30, and an impact factor to estimate the overall load in the design. The maximum design ring bending stress for DIP is 48,000 psi, which provides a minimum safety factor of 1.5 based upon minimum ring yield strength and at least 2.0 for minimum ultimate ring strength. Equation 4-31 is used to calculate the load required to develop a bending stress of 48,000 psi at the pipe springline. Reduction factors for truck loads are provided in Table 4-10.

Equation 4-29

$$P_e = (w \cdot H)/144$$

Where:

P_e = soil prism pressure load (lb/in.2)
w = unit weight of the soil (lb/ft3)
H = height of soil column over top of pipe (ft)

Equation 4-30

$$P_t = (R \cdot F \cdot C \cdot P)/(b \cdot D)$$

Where:

P_t = truck pressure load (lb/in.2)
R = reduction factor as per Table 4-10
F = 1.5 (impact factor)
C = surface load factor for single concentrated wheel load (Equation 4-30)
P = wheel load (typically 16,000 lb)
b = 36 inches (effective pipe length)
D = outside diameter of pipe (in.)

Equation 4-31

$$(P_e + P_t) \le P_v = \frac{f}{3\left(\frac{D}{t}\right)\left(\frac{D}{t} - 1\right)\left[K_b - \frac{K_x}{\frac{8E}{E'\left(\frac{D}{t} - 1\right)^3} + 0.732}\right]}$$

Where:

P_v = allowable external pressure load (lb/in.2)
f = design max bending stress (48,000 psi)
D = outside pipe diameter (in.)
t = net thickness (in.)
K_b = bending moment coefficient
K_x = deflection coefficient
E = modulus of elasticity (24,000,000 psi)
E' = modulus of soil reaction (psi)

The surface load factor, C, is a measure of how the wheel load at the surface is distributed through the soil to the pipe. Equation 4-32 is used to calculate this factor. Pre-calculated values for the surface load factor are provided for a variety of cover depths in ANSI/AWWA C150/A21.50. This equation provides a reasonable estimate for estimating a single load acting over three feet of pipe length. This is considered adequate when the pipe will not be exposed to trucks where the load is distributed over multiple axles (highway loads). When highway loads must be evaluated for DIP, it is suggested that AASHTO Section 3.6.1.2 be followed.

Equation 4-32

$$C = 1 - \frac{2}{\pi}\arcsin\left[H\sqrt{\frac{A^2 + H^2 + 1.5^2}{(A^2 + H^2)(1.5^2 + H^2)}}\right] + \frac{2}{\pi}\left(\frac{1.5AH}{\sqrt{A^2 + H^2 + 1.5^2}}\right)\left[\frac{1}{A^2 + H^2} + \frac{1}{1.5^2 + H^2}\right]$$

Where:

H = depth of cover (ft)
A = outside radius of pipe (ft)

Table 4-10.
Reduction factors for truck load calculations. (Courtesy of DIPRA)

Pipe Diameter (in.)	Cover Depth < 4 (ft)	Cover Depth 4-7 (ft)	Cover Depth 7-10 (ft)	Cover Depth > 10 (ft)
3-12	1.00	1.00	1.00	1.00 for all pipe diameters
14	0.92	1.00	1.00	
16	0.88	0.95	1.00	
18	0.85	0.90	1.00	
20	0.83	0.90	0.95	
24-30	0.81	0.85	0.95	
36-64	0.80	0.85	0.90	

4.14.5.4. Determining Design Values for Soil Variables/Coefficients. The design methodology for DIP was developed for use with gravity/pressurized sewer applications and not necessarily to be an integral part of a levee embankment or floodwall; thus, the values for various soil parameters outlined within the DIP design procedures and ASTM A746 likely are not applicable for levee and dam projects. The pipe designer must work with a qualified geotechnical engineer to establish appropriate values for the bending moment coefficient (K_b), deflection coefficient (K_x), and modulus of soil reaction (E') when DIP is used for levee and dam applications.

4.14.5.5. Wall Thickness Selection. Once the external loads are computed, the required wall thickness can be selected and compared to that which is required based upon internal pressure. The larger of the two should be selected (internal pressure design versus external pressure design thickness) and must be checked for deflection criteria (reference Table 4-1). An additional 0.08 inches should be added to the wall thickness as a service allowance (Equation 4-33).

4.14.5.6. Deflection Check. Tests have shown that maximum deflection for DIP provided in Table 4-1 will provide a safety factor of at least 2.0 with regard to failure of the lining. The calculation used for estimating the maximum trench load to meet the safety factor criteria is shown as Equation 4-33. The minimum thickness required to meet the deflection criteria is then compared to that which was computed from the internal pressure/external loading stress design, and the larger one is selected.

Equation 4-33

$$P_v = \frac{\Delta x/D}{12K_x}\left[\frac{8E}{\left(\frac{D}{t_1}-1\right)^3} + 0.732E'\right]$$

Where:

$t1$ = minimum thickness (t + 0.08 in.)

Δx = design deflection (in.)

Pv, Kx, E, E', and D as defined in Equation 4-31

4.14.5.7. Casting Tolerance. Once the minimum thickness is determined, an additional allowance for casting tolerance is added to account for the potential for a minus deviation of the nominal size. The casting tolerance is a function of the pipe diameter as follows: 3- to 5-inch diameter (0.05-inch casting tolerance), 10- to 12-inch (0.06-inch), 14- to 42-inch (0.07-inch), 48-inch (0.08-inch), and 54- to 64-inch (0.09-inch).

4.14.5.8. Selection of DIP from Pressure Class Table. Once the final thickness is determined by adding the casting tolerance, the designer is required to select an acceptable DIP from the various pressure class tables available within the specified guidance documents (ASTM A746 or ANSI/AWWA C150/A21.50). The pressure class tables are available for different lining materials (i.e., cement mortar, flexible). Specialized thickness tables for DIP are also available within ANSI/AWWA C150/A21.50 when the required thickness is too large for the standard design tables.

4.14.5.9. DIPRA Design Tools. DIPRA has numerous resources to assist with DIP design including four online calculators that cover design on supports, hydraulic analysis, thickness design, and thrust restraint. As is the case with any available software, the designer needs to have confidence in the inputs and verify the outputs provided by these design tools to ensure they match the conditions anticipated in the field. Designers must also ensure all applicable safety factor criteria are satisfied when using these design aids. Scanned versions of various design support documents as well as a variety of technical publications are available on the DIPRA website.

4.14.6. Special Considerations Associated with DIP. Unless specified otherwise, DIP is manufactured with a Portland cement-mortar lining that conforms to the standards outlined in AWWA C104. The cement-mortar liner is typically used for various types of water conveyance and has a maximum service temperature range of 150˚ (with an asphaltic shock coating) to 212˚F (without an asphaltic shock coating). Specialized liners (e.g., petroleum asphalt, ceramic quartz filled epoxy) are available when cement-mortar is not the most appropriate alternative, such as in highly acidic flow, alkali waste environments, and septic wastewater applications.

4.15. Thermoplastic Pipe.

4.15.1. General. The provisions for the structural design of thermoplastic pipes are covered in Section 12.12 of the AASHTO LRFD document and are considered applicable for the structural design of gravity flow, non-pressurized thermoplastic pipe including profile wall versions of HDPE, PP, and PVC. The design guidance within AASHTO is not applicable to pressurized thermoplastic pipe design. For pressurized thermoplastic pipe design, refer to the most recent applicable design specifications and handbooks for the thermoplastic material being used.

4.15.2. Design Requirements for Profile Wall Thermoplastic Pipe. The structural design of thermoplastic pipe must be evaluated for both service and strength limit states. The only service limit state evaluated is associated with deflection. Three strength-based limit states must be satisfied for the structural design of thermoplastic pipe: thrust, global/localized buckling, and combined strain. Section properties required to assess both the service and strength limit states for thermoplastic pipe should be determined from cut sections of pipe or obtained from the pipe manufacturer.

4.15.3. Service Limit State for Profile Wall Thermoplastic Pipe. In order to meet this required service limit state (deflection criteria) for profile wall thermoplastic pipe, the total deflection (Δ_t) should be less than or equal to the allowable deflection shown in Table 4-1. The calculation for total deflection is taken from Section 12.12.2.2 of the AASHTO Manual and is reproduced here as Equation 4-34.

Equation 4-34

$$\Delta_t = \left[\frac{D_o K_B (D_L P_{sp} + C_L P_L)}{1000\left(\frac{E_p I_p}{R^3} + 0.061 M_s\right)}\right] + \varepsilon_{sc} D$$

Where:

Δt = total deflection of pipe (in.)
D_o = outside diameter of pipe (in.)
K_B = bedding coefficient (typically a value of 0.1 is used)
D_L = deflection lag factor (typically a value of 1.5 is used)
P_{sp} = soil prism load pressure at pipe springline (psi) as per AASHTO Section 12.12.3.7
C_L = live load distribution coefficient as per AASHTO Section 12.12.3.5
P_L = service live load on the pipe (psi) as per AASHTO Section 12.6.1
E_p = short-term modulus of pipe material as per AASHTO Table 12.12.3.3-1 (ksi)
I_p = moment of inertia per unit length (in.4/in.)
R = radius of pipe from center to centroid of pipe wall (in.)
M_s = secant constrained soil modulus from Section 12.12.3.5 (ksi)
D = diameter to centroid of pipe wall (in.)
ε_{sc} = service compressive strain as per AASHTO Section 12.12.3.10.1c

4.15.4. Strength Limit State for Profile Wall Thermoplastic Pipe.

4.15.4.1. General. Profile wall thermoplastic pipe must be designed to resist axial thrust, general/local buckling, and combined strain in order to satisfy all the strength limit states. Total compressive strain in thermoplastic pipe can cause yielding or buckling, and total tensile strain can cause cracking. The mechanical design properties for the structural design of thermoplastic pipe are taken from AASHTO Table 12.12.3.3-1. For the design of thermoplastic pipes used in USACE embankments or floodwalls, the 75-year properties are applicable for dead loads from the table. For live loads, the initial properties from the table should be used.

4.15.4.2. Axial Thrust. The factored thrust load (T_u) is calculated using AASHTO Equation 12.12.3.5-1 (Equation 4-35). This includes the addition of load factors with respect to earth and live loads.

Equation 4-35

$$T_u = \frac{[\eta_{EV}\left(\gamma_{EV}K_{\gamma E}K_2 P_{sp}VAF + \gamma_W Pw\right) + \eta_{LL}\gamma_{LL}P_L C_L F_1 F_2]D_o}{2}$$

Where:

T_u = factored thrust per unit length (lb/in.)
η_{EV} = 1.05 (earth load importance modifier for pipes in USACE embankments /floodwalls)
γ_{EV} = 1.3 vertical earth pressure load factor (see Table 4-3)
$K_{\gamma E}$ = 1.5 (installation factor)
K_2 = coefficient for variation in thrust around the pipe circumference; 1.0 for thrust at the springline, 0.6 for thrust at the crown of the pipe
VAF= vertical arching factor per AASHTO Equations 12.12.3.5-3 and 12.12.3.5-4
P_{sp} = soil pressure prism at pipe springline (psi)
γ_W = load factor for hydrostatic pressure (see Section 4.8.4.2.4)
Pw = hydrostatic water pressure at the springline of the pipe (psi)
η_{LL} = 1.05 (live load importance modifier for pipes in USACE dams/levees)

γ_{LL} = 1.75 (live load factor for Strength Load Case I)
P_L = live load pressure with dynamic load allowance (psi) using guidance from AASHTO Section 3.6.1.2
C_L = live load distribution coefficient using AASHTO Equation 12.12.3.5-5
D_o = outside diameter of pipe (in.)
F_1 = live load factor 1 calculated using AASHTO Equation 12.12.3.5-6
F_2 = live load factor 2 calculated using AASHTO Equation 12.12.3.5-8

4.15.4.3. Earth Load. It is important to note that the calculation of P_{sp} does not include hydrostatic pressure. It is the pressure due to the weight of material above the pipe and should be based upon the wet density for the soil above the water table and buoyant density below the water table.

4.15.4.4. Resistance to Axial Thrust. In order to determine the pipe's resistance to axial thrust, the factored compressive strain, ε_{uc}, must be computed using Equation 4-36.

Equation 4-36 (AASHTO Equation 12.12.3.10.1c-1)

$$\varepsilon_{uc} = \frac{T_u}{1000\left(A_{eff} \cdot E_p\right)}$$

Where:

ε_{uc} = factored compressive strain due to thrust
T_u=factored axial thrust load as per Equation 4-33 (lb/in.)
A_{eff} = effective area of pipe wall per unit of length (in.^{2}/in.) as per AASHTO Section 12.12.3.10
E_p = modulus of elasticity (as per AASHTO Table 12.12.3.3-1, ksi)

4.15.4.5. Compressive Strain. The factored compressive strain due to axial thrust must satisfy Equation 4-37 in order to meet the necessary design requirements for this limit state:

Equation 4-37 (AASHTO Equation 12.12.3.10.1d-1)

$$\varepsilon_{uc} \leq \phi_T \, \varepsilon_{yc}$$

Where:

ϕ_T = 1.0 (resistance factor for axial thrust for thermoplastic pipe)
ε_{yc} = factored compressive strain limit for pipe material as per AASHTO Table 12.12.3.3-1 (varies from 2.6 to 4.1 percent depending upon pipe material type)

4.15.4.6. Global Buckling Strain Limits. The equations required to evaluate the pipe section for global buckling are provided in AASHTO Section 12.12.3.10.1e. The factored compression strain due to thrust with the incorporation of localized buckling effects must satisfy Equation 4-38 (AASHTO Equation 12.12.3.10.1e-1).

Equation 4-38 (AASHTO Equation 12.12.3.10.1e-1)

$$\varepsilon_{uc} \leq \phi_{bck} \, \varepsilon_{bck}$$

Where:

ϕ_{bck} = 0.7 (global buckling resistance factor for thermoplastic pipe)

ε_{bck} = nominal strain capacity for general buckling as per AASHTO Equation 12.12.3.10.1e-2)

4.15.4.7. Flexural Strain. The calculation for factored flexural strain is based upon an empirical relationship between strain and deflection and is calculated using Equation 4-39.

Equation 4-39 (AASHTO Equation 12.12.3.3-1)

$$\varepsilon_f = \gamma_{EV} \cdot D_f \left(\frac{c}{R}\right)\left(\frac{\Delta f}{D}\right)$$

Where:

ε_f = factored strain due to flexure

γ_{EV} = 1.3 (load factor for earth fill for thermoplastic pipe)

D_f = shape factor as per AASHTO Table 12.12.3.10.2b-1

c = larger of the distance from neutral axis of profile to the extreme innermost or outermost fiber (in.)

R = radius from center of pipe to centroid of pipe profile (in.)

Δ_f = reduction of vertical diameter due to flexure per AASHTO Equation 12.12.3.10.2b-4 (in.)

D = diameter to centroid of pipe profile (in.)

4.15.4.8. Combined Strain. If the summation of the axial thrust strain (ε_{uc}) and bending strain (ε_f) produces a tensile strain on the extreme fiber of the pipe wall, then Equation 4-40 must be satisfied.

Equation 4-40 (AASHTO Equation 12.12.3.10.2b-1)

$$\varepsilon_f - \varepsilon_{uc} < \phi_f \, \varepsilon_{yt}$$

Where:

ϕ_f = 1.0 (flexure resistance factor for thermoplastic pipe)

ε_{yt} = service long-term tension strain limit of the pipe wall material as per AASHTO Table 12.12.3.3-1

If the summation of the axial thrust strain (ε_{uc}) and bending strain (ε_f) produces a compressive strain on the extreme fiber of the pipe wall, then Equation 4-41 must be satisfied.

Equation 4-41 (AASHTO Equation 12.12.3.10.2b-2)

$$\varepsilon_f + \varepsilon_{uc} < \phi_T \, (1.5 \cdot \varepsilon_{yc})$$

4.15.5. Other Design Considerations for Thermoplastic Pipe. The flexibility limit for thermoplastic pipe is evaluated for handling and installation requirements. The allowable flexibility limit for thermoplastic pipe is 95.0 in./kip; therefore, FF < 95.0 is required to meet the flexibility limit. The flexibility limit calculation for thermoplastic pipe is shown as Equation 4-42.

Equation 4-42 (AASHTO Equation 12.12.3.6-1)

$$FF = \frac{S^2}{EI}$$

Where:

- FF = flexibility factor
- S = pipe diameter (in.)
- E = initial modulus of elasticity (ksi)
- I = moment of inertia ($in.^4/in.$)

4.16. Fiberglass Reinforced Pipe (FRP).

4.16.1. General. The structural design of FRP is governed by Section 12.15 of the AASHTO LRFD guidance, which is based upon Chapter 5 of the AWWA Manual M45, Fiberglass Pipe Design. The methodology generally follows the methodology associated with Spangler's deflection theory, Molin's bending equation, and constrained buckling analysis.

4.16.2. Design Requirements for Fiberglass Reinforced Pipe. The structural design of FRP must be evaluated for both service and strength limit states. The only service limit state evaluated is associated with deflection. Three strength-based limit states must be satisfied for FRP: flexure, global buckling, and flexibility limit. Section properties required to assess both the service and strength limit states for FRP should be determined by referencing the manufacturer's published nominal thickness and the nominal diameter requirements of ASTM D3262 Tables 2 and 3. FRP wall thickness must be determined according to ASTM D3567 due to the variable nature of FRP manufacturing including the amount, type, and orientation of the fiber reinforcement.

4.16.3. Fiberglass Reinforced Pipe Service Limit State – Deflection Criteria. The allowable design deflection must not exceed the value shown in Table 4-1. The estimated deflection for FRP, taken as a reduction of the vertical diameter when deflecting, is governed by AASHTO Equation 12.15.5.2-1 (reproduced here as Equation 4-43). This is similar to the deflection equation used for profile wall thermoplastic pipe, but is slightly modified by eliminating the thrust component and calculating the soil prism pressure at the crown of the pipe. Compressive thrust is ignored in the calculation due to the high modulus associated with solid wall FRP.

Equation 4-43 (AASHTO LRFD Equation 12.15.5.2-1)

$$\Delta_t = \left[\frac{D_o K_B (D_L P_{sp} + C_L P_L)}{1000\left(\frac{E_{cf} I_p}{R^3} + 0.061 M_s\right)}\right]$$

Where:

- Δ_t = total deflection of pipe (in.)
- D_o = outside diameter of pipe (in.)
- K_B = bedding coefficient (typically a value of 0.1 is used)
- D_L = deflection lag factor (typically a value of 1.5 is used)
- P_{sp} = soil prism load pressure at top of the pipe (psi)

C_L = live load distribution coefficient as per AASHTO Section 12.12.3.5
P_L = service live load on the pipe (psi) as per AASHTO Section 12.6.1
E_{cf} = circumferential flexural modulus as defined in AASHTO Section 12.15.3.1 (determined by pipe stiffness tests as per ASTM D3262)
I_p = moment of inertia per unit length (in.^{4}/in.)
R = radius of pipe from center to centroid of pipe wall (in.)
M_s = constrained soil modulus (ksi) from AASHTO Table 12.12.3.5-1

4.16.4. Fiberglass Reinforced Strength Limit States.

4.16.4.1. Flexure. The equation to check flexure (ring bending) is AASHTO Equation 12.15.6.2-1 and is reproduced here as Equation 4-44.

Equation 4-44 (AASHTO Equation 12.15.6.2-1)

$$\varepsilon_f \leq \phi_f S_b$$

Where:

ε_f = factored long-term strain due to flexure, determined by AASHTO Equation 12.12.3.10.2b-3
ϕ_f = 0.9 (resistance factor for flexure for FRP)
S_b = long-term ring bending strain as determined by AASHTO Section 12.15.3.2 (ASTM D3681)

4.16.4.2. Global Buckling. The global buckling check must satisfy AASHTO Equation 12.15.6.3-1 (Equation 4-45). Loads to be considered for this calculation include hydrostatic groundwater, soil pressure, and live loads.

Equation 4-45 (AASHTO Equation 12.15.6.3-1)

$$\varepsilon_{fb} \leq \phi_{bck}\, \varepsilon_{bck}$$

Where:

εfb = factored buckling strain demand as per AASHTO Equation 12.15.6.3-2
ϕbck= 0.63 (resistance factor for global buckling for FRP)
ε_{bck}= nominal strain capacity for general buckling as determined by AASHTO Equation 12.12.3.10.1e-02 with modifications outlined in AASHTO Section 12.15.6.3.

4.16.4.3. Fiberglass Reinforced Pipe Flexibility Limit. The flexibility limit for FRP is evaluated for handling and installation requirements. The allowable flexibility limit for FRP is 95.0 in./kip; therefore, FF < 95.0 is required to meet the flexibility limit. The flexibility limit calculation for FRP is shown as Equation 4-46.

Equation 4-46

$$FF = S^2/E_{cf}\, I$$

Where:

FF = flexibility factor
S = pipe diameter (in.)
Ecf = circumferential flexural modulus as per AASHTO Section 12.15.3.1 (determined by pipe stiffness tests according to ASTM D3262)
I = moment of inertia (in.4/in.)

4.17. Cast-in-Place Concrete Pipe (CiPCP).

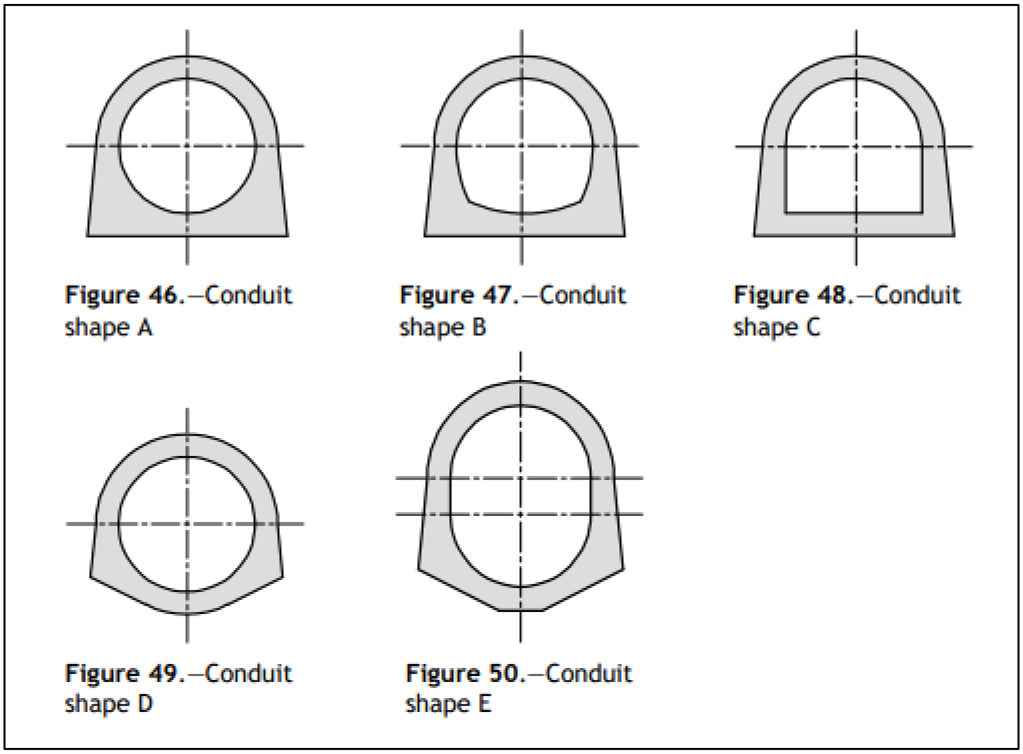
4.17.1. Introduction. CiPCP is a reinforced concrete pipe that is custom made and formed on-site. The use of CiPCP for conveyance of water through USACE embankment and floodwall projects has primarily been as an outlet works for large flood control or hydropower dams. There has been limited use of CiPCP in USACE levee projects since the full, widespread development of the precast RCP industry. In general, it is almost always more economical to use precast RCP compared to CiPCP, but there are certain situations where precast RCP will not suffice to meet the required criteria. For example, more recent USACE projects with specialized design and/or construction criteria have required the use of CiPCP.

4.17.2. Cast-in-Place Concrete Pipe Design Methodology. Reinforced concreted design for CiPCP must follow the guidance covered in the most up-to-date version of EM 1110-2-2104. EM 1110-2-2104 uses LRFD methodology for different levels of loading recurrence (usual, unusual, and extreme). The recurrence level for these load events is shown graphically in Figure 3-1 in EM 1110-2-2104. EM 1110-2-2104 also covers the applicable load combinations to be considered for CiPCP, as well as required load factors.

4.17.3. Cast-in-Place Concrete Pipe Design Loads. Design loads to consider include internal fluid pressure, vacuum pressure, surge pressure, external hydrostatic loads, embankment loads, surcharge loads, construction loads, and other specialized loads. The Marston theory of embankment pressures has typically been used for vertical earth loads acting on a positive projecting embankment CiPCP. This theory provides a conservative approach to estimating loads when the pipe is in a positive projected condition. Foundation pressure is assumed to act uniformly across the full width of CiPCP. Uplift pressure is calculated as uniform pressure at the base of the CiPCP when checking flotation. Because of the ratio of vertical-to-horizontal pressure, the most severe loading condition generally occurs when the pipe is empty and the surrounding soil is in a natural drained condition. Higher level, detailed soil-structure interaction computer modeling can be used for multi-dimensional and time-dependent analysis if deemed necessary. Prior to using modeling results for the CiPCP design, a comparison of modeling loads versus those from more traditional approaches should be done in consultation with experienced geotechnical engineers.

4.17.4. Cast-in-Place Concrete Cross-Section. The main considerations for shape selection are the ability to adequately compact against the pipe during construction, minimization of the potential for stress arching within the embankment, minimization of potential for differential settlement, allowing access for inspection/future repairs, and providing the most economical design that meets project requirements. The most economical CiPCP cross-section depends upon the consideration of all design factors and site conditions and the judgment of the designer. Figure 4-7 (Figures 46-50 from FEMA 484) shows cross-sectional shapes typically used for

CiPCP in embankment dams. Box shaped CiPCP have also been used in embankment and floodwall applications, but USACE prefers making the exterior faces of the vertical side walls slightly sloped (thicker at the base of the vertical wall compared the top section) in order to allow improved compaction against the structure. Reinforcement requirements for CiPCP should also follow the guidance outlined in EM 1110-2-2104.



(Courtesy of FEMA)

Figure 4-7. Typical CiPCP cross-section shapes.

4.18. Steel Reinforced Thermoplastic Pipe (SRTP).

4.18.1. Background. SRTP is a composite steel/thermoplastic pipe that is relatively new to the United States market at the time of this manual's publishing. Engineering requirements regarding the manufacture and installation of SRTP are dictated by ASTM F2562. The structural design of SRTP is governed by Section 12.7 of the AASHTO LRFD Bridge Design Specification, which is the same section that governs CMP. This is because the main load carrying capability of SRTP pipe is the steel ribs that are encapsulated within thermoplastic material.

4.18.2. Steel Reinforced Thermoplastic Pipe Engineering Properties. The engineering properties of SRTP must be provided directly by the manufacturer for structural design. While the steel ribs are the main load carrying section of the pipe, the thermoplastic profile braces the steel ribs by preventing buckling or distortion under load. The thermoplastic liner also distributes the load between the steel ribs. While SRTP may provide advantageous combined engineering properties associated with plastic (non-corrosive, abrasion resistant) and steel (strength), it is important that the designer understand this is the newest type of pipe material included within this manual. It has a limited operating history compared to other pipe materials and its long-term performance has not yet been established.

4.18.3. Long-Term Strain. A structural evaluation of the composite section of SRTP is required to ensure the tensile strains within the profile do not exceed the long-term strain capacity for the thermoplastic material used in the construction of the pipe. In order for this evaluation to occur, the pipe manufacturer must provide the results of a three-dimensional finite element (FE) analysis of the profile that has been calibrated against a minimum of two full-scale tests. The predicted long-term tensile strains within the profile from the FE analysis must remain below the allowable limits for the applicable thermoplastic material as provided in Table 12.12.3.3-1 of the AASHTO LRFD Bridge Design Specification.

4.18.4. Critical Compressive Stress. A stub compression test following the procedures covered in AASHTO T341 must be provided to establish the critical buckling stress (f_{cr}) for the SRTP wall profile section. The lower value of f_{cr} from either the stub test results or as calculated following AASHTO LRFD Bride Design Specification Section 12.7.2.4 must be used as the allowable limit of compressive stress. The applicable stroke rates associated with the stub compression test to ensure long-term interaction of the steel reinforcement with the thermoplastic profile are detailed in Section 12.7.2.7 of the same document.

4.18.5. Cover Requirements. Most manufacturers of SRTP have minimum and maximum cover requirements as a function of pipe diameter. Minimum cover typically ranges from one to three feet, whereas maximum cover ranges from 25 to 50 feet. Consult the individual manufacturer's limitations for cover requirements.

4.19. Other Pipe Materials. It is unlikely that a pipe material other than those covered in this chapter will be used in a new design of a pipe associated with a USACE embankment or floodwall unless it is a new product produced after publication of this manual. Reference Section 1.2 for guidance on requesting approval if a new pipe material/composite section is planned to be used.

THIS PAGE INTENTIONALLY LEFT BLANK

Chapter 5
Installation and Acceptance Testing

5.1. Introduction. Installation is the placement of new pipes elevated, within, beneath, or adjacent to a new or existing embankment or floodwall. This chapter is structured to follow a typical construction sequence and includes guidance and requirements on: on-site water management; pipe location; trenched and trenchless installation; and acceptance testing and inspection. In USACE's descending order of preference, new pipes should be aligned to pass over floodwalls, through closures, through floodwall stems, over embankments, beneath floodwalls/embankments, and finally, through embankments. When penetration of the embankment is unavoidable, the risk must be reduced by requiring that the pipe cross as high in the embankment as possible (Figure 5-1, as an example). With the exception of toe drains and relief well collector pipes related to dams, the information in Chapter 5 applies only to levees.

5.2. Potential Failure Modes Related to Installation. The incorrect installation of or damage to a pipe and its surrounding backfill may provide opportunities for potential failure modes (PFMs), as discussed in Chapter 2, to initiate. Breach may occur despite the fact that internal erosion failure mechanisms are typically slow to progress and may not be observed until substantial internal damage has occurred. For example, improper storage of pipes onsite may create defects that are not immediately apparent but can lead to a PFM. Other seemingly benign issues, such as inadequate water management, can cause a dirty working environment that can contaminate pipe joint connections and prevent adequate joint sealing, leading to embankment soil loss through the joint (PFM-3). Improper trench backfilling practices can provide the flaws necessary to initiate PFM-1. If pipe segments are not properly joined, the release of pressurized fluids (PFM-2) or the infiltration of embankment soils into the pipe (PFM-3) are more likely to occur.

5.3. Water Management during Installation.

5.3.1. Background. Water management during pipe installations is necessary to prevent schedule delays, increased costs, and lower quality installations. For example, water ponding in the pipe trench can prevent adequate backfill compaction, which may lead to vertical and/or differential settlement between pipe segments.

5.3.2. Groundwater Management. Excavations which extend below the groundwater table can cause a "quick" condition in the bottom of the trench, which occurs when the upward water pressure reduces the soil's effective stress and it begins to behave more like a fluid. This loss of strength creates an inadequate supporting surface for the pipe. The depth of the bottom of the excavation in relationship to the groundwater table, foundation conditions, expected quantity of flow, and size of excavation will dictate the groundwater management method. The information required for selecting and designing a groundwater control system can be found in UFC 3-220-05.

5.3.3. Surface Water Management. Surface water is any water found on the ground surface, regardless of its source (e.g., storm runoff; melting snow or ice; overland flows from streams, rivers, lakes, and reservoirs). Two of the methods for controlling surface water include earthwork/regrading to divert water around the construction site or cofferdams, which are used in larger bodies of water to divert water away from the site. Requirements for design and construction of a temporary cofferdam are found in ER 1110-2-8152.

5.3.4. Floodwater Management. The installation or replacement of a pipe through a full cross-sectional embankment excavation will require an emergency closure plan if a cofferdam around the work are is not used. It is the responsibility of the levee sponsor to submit an emergency closure plan to the respective United States Army Corps of Engineers (USACE) District office for approval. It is typically required that the plan include trigger elevations/river stages to indicate when construction of the closure is to begin. The trigger elevations should be supported by historical evidence for the rate-of-river rise correlated to man-hours needed to construct the closure. Emergency closure materials (e.g., stoplogs, portable dams, bladders, soil, aggregates, sandbags, plastic sheeting, and necessary placement equipment) should be stockpiled onsite or in a readily accessible location based on the construction schedule and the site-specific flood characteristics. As an alternative to an emergency closure, a USACE-approved cofferdam to the full height of the levee may be constructed.

5.4. Pipe Location Considerations.

5.4.1. Typical Pipe Locations and Restrictions. Pipes considered essential to the function of the levee, such as pump station discharge pipes, gravity drains, and electrical conduits to power appurtenances may pass within or beneath an embankment assuming the precautions and requirements detailed within this chapter are followed. Non-essential (third-party) pipes are required to be elevated over the embankment crest to reduce the likelihood of initiating internal erosion through potentially flawed backfill (PFM-1); limited exceptions to this requirement are discussed in the next section. Any pipe, whether essential or non-essential, may pass through floodwall elements (stem, key, or sheet piling) or beneath floodwalls, provided its placement adheres to the precautions and requirements of this chapter to avoid seepage concerns. To reduce the number of active penetrations through a levee embankment or its foundation, new pipes that serve as replacements for existing non-essential pipes within or beneath an embankment should be rerouted in an elevated position over the embankment and the original pipe removed or decommissioned (reference Chapter 8).

5.4.2. Exceptions to Non-Essential Pipe Location Restrictions.

5.4.2.1. Example Exceptions for Pipe Location within an Embankment. Non-essential pipes should be located as close to the crown of the levee as possible, taking into consideration the PFMs that may occur given the loading applied on the ground surface (reference Chapter 2). In cases where the levee crest supports infrastructure, such as a highway that cannot be obstructed by an elevated placement or disrupted by an open-cut installation (Figure 5-1), non-essential pipes can be approved to pass within the embankment. The carrier pipe must pass through a casing pipe installed using the trenchless methods discussed in Section 5.6.

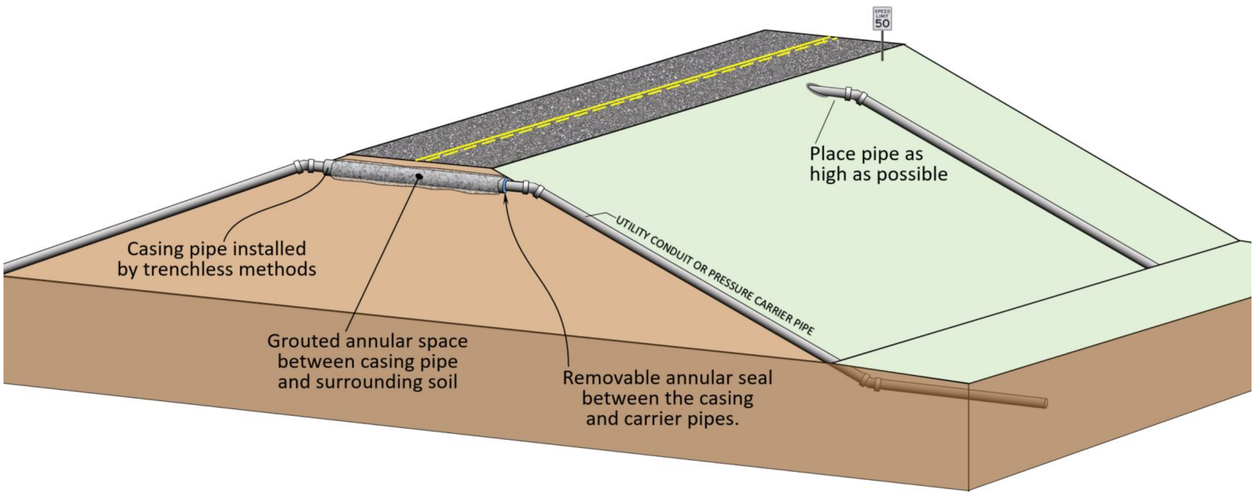


Figure 5-1. Example of allowable, non-essential pipe crossing beneath a highway.

5.4.2.2. Example Exceptions for Pipe Location beneath an Embankment. Non-essential pipes may be allowed to pass beneath a levee using horizontal directional drilling (HDD) since the installation submittal package under 33 USC 408 will be subject to review by the USACE Risk Management Center, reducing the likelihood of failure due to hydro-fracturing or internal seepage along the pipe. However, before considering trenchless placement within the foundation, the respective USACE District must require that the levee sponsor/owner provide compelling justification for why the pipe cannot be installed in an elevated placement. An example of a compelling reason would be the need to protect a high-pressure natural gas line where exposure above ground may otherwise pose a safety concern or violate federal regulations. Another more common reason may be that the pipe must remain below a deep frost depth.

5.4.3. Elevated Embankment Pipes.

5.4.3.1. General. Elevated pipes are positioned higher than the embankment ground surface (i.e., trenching is not required for installation). Exposed elevated pipes must be protected from or designed to withstand debris impact. While not required, it is recommended that elevated pipes be designed to allow vehicular passage beneath or over them.

5.4.3.2. Passable (Raised). Pipes elevated to allow passage beneath them (Figure 5-2 and Figure 5-3) should be protected from accidental vehicular impact with structures such as guard rails or bollards. Footers and foundations for the support and protective structures should be designed to minimize their depth of penetration in the embankment.

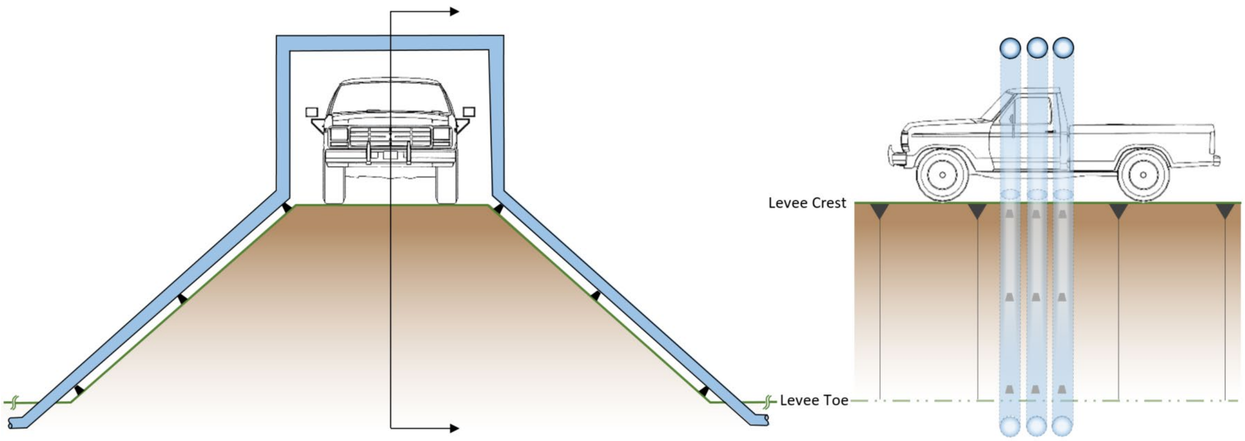


Figure 5-2. Elevated pipes providing vehicular passage (pipe support and protection structures removed for clarity).

(Photo courtesy of Dreamstime.com, Sherry Young)

Figure 5-3. Example of passable elevated pipes.

5.4.3.3. Passable (Overbuilt). Elevated pipes on the ground surface can be made passable by overbuilding the embankment to cover them with compacted soil (Figure 5-4). Figure 5-5 is an example of an overbuild that accommodates two small-diameter water lines. Overbuilding the embankment above the pipes while maintaining the slope geometry may make the crest too narrow to continue to accommodate vehicular traffic (Figure 5-6). This requires the overbuild to include covering the sloped portion of the pipe(s) to provide lateral support for additional soil on the crest if restoration of the drivable width is desired. If the crest is wide enough to continue supporting traffic after the overbuild, it is not required that the sloped portion of the pipe be covered (Figure 5-7). The overbuild geometry should be rounded to aid mowing (typically flatter than 10H:1V slopes) and prevent “high-centering” of vehicles or trailers if the crest supports traffic.

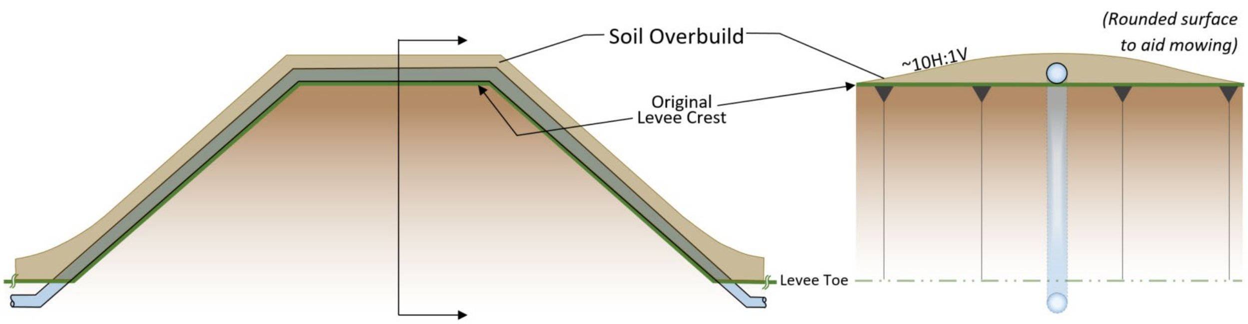


Figure 5-4. Profile and section of overbuilt pipe.

(Courtesy of USACE Louisville District)
Figure 5-5. Overbuilt section to accommodate water lines.

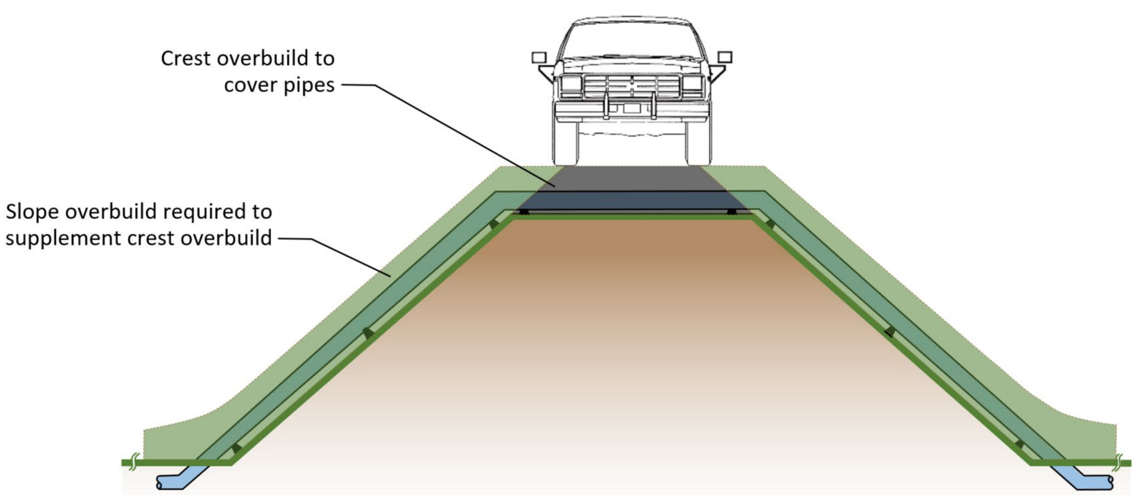


Figure 5-6. Normal crest widths (roughly 10 to 12 feet) will typically require overbuilt slopes.

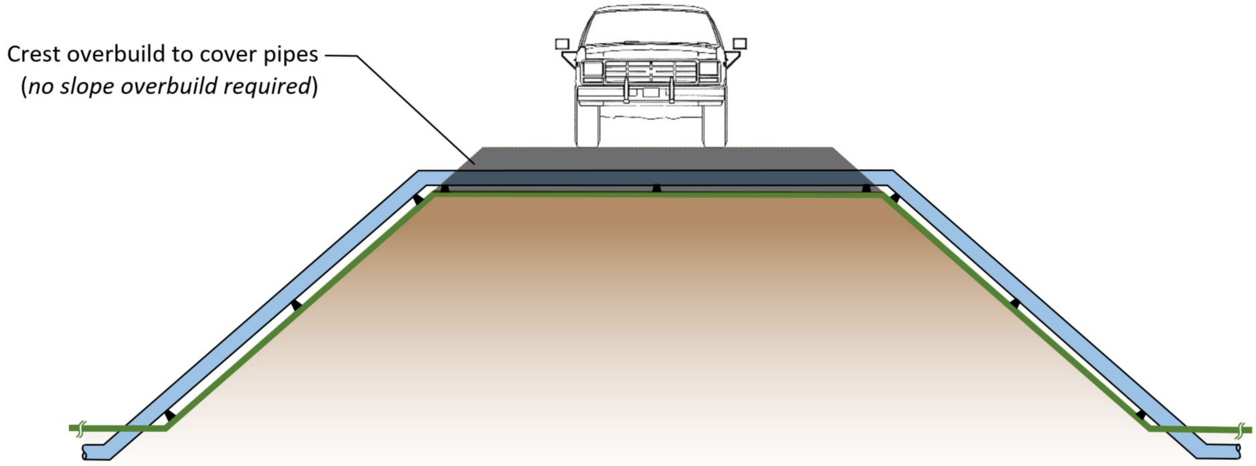


Figure 5-7. Some crests are wide enough that overbuilding the slope is not required.

5.4.3.4. Non-passable. A pipe is non-passable when vehicular clearance is not provided beneath it and overbuilding the embankment to allow passage over the pipe is not an option (Figure 5-8 and Figure 5-9). An alternative to raising or overbuilding the pipe is to reroute traffic around the pipe crossing (Figure 5-10 and Figure 5-11).

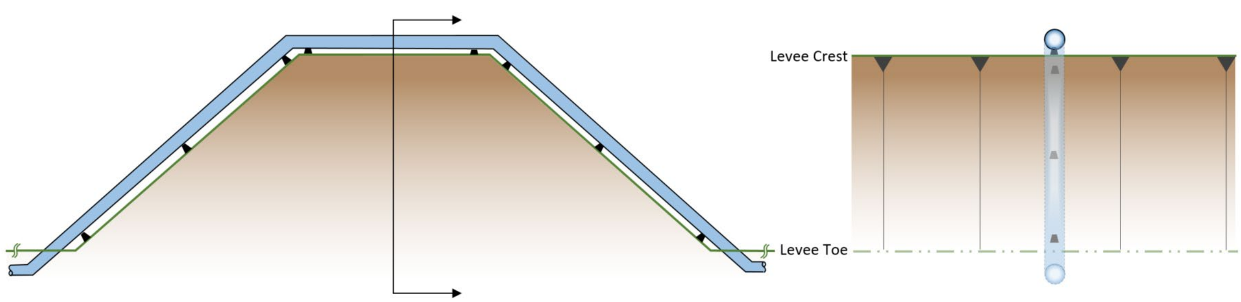


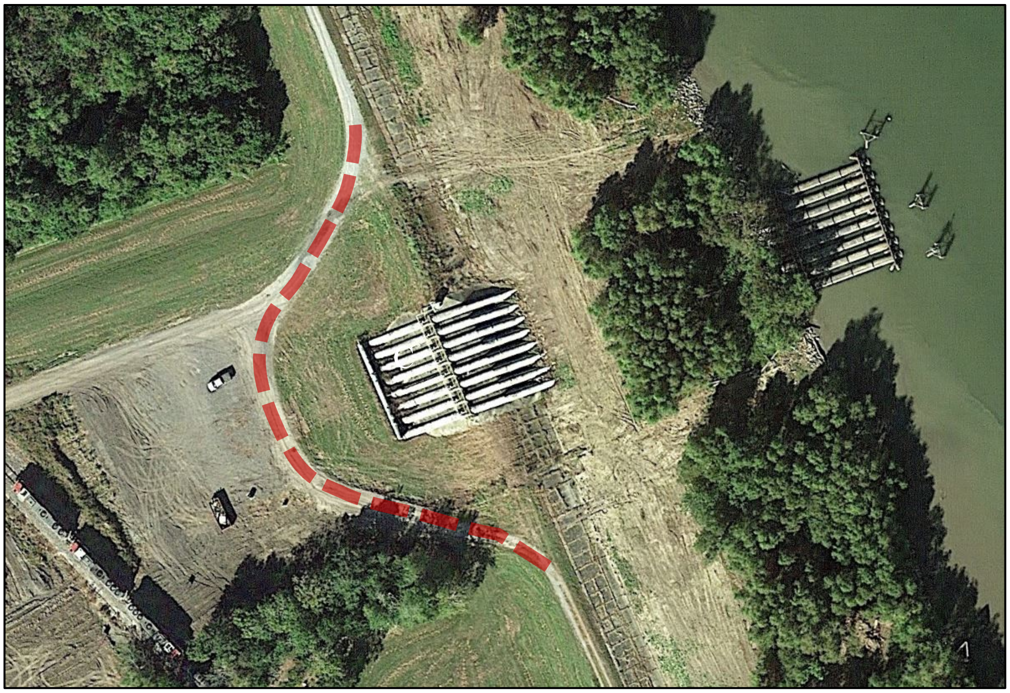
Figure 5-8. Elevated non-passable pipes prevent vehicular crossing.

(Courtesy of USACE Huntington District)

Figure 5-9. Example of non-passable elevated pipes.

(From FOX 8 WVUE-TV, Plaquemines Parish, LA)

Figure 5-10. Vehicular traffic routed around large diameter siphon pipes over levee crest.



(From Google Earth, 2018; Map data ©2018Google)

Figure 5-11. Aerial view of access road being detoured around siphon pipes.

5.4.4. Within Embankment Pipes.

5.4.4.1. General. Pipes considered to be within an embankment include those that essentially go straight through, such as gravity drain pipes, and those that follow the levee profile, such as pump station discharge pipes; therefore, they are known as "through" and "profile" pipes, respectively. Electrical conduits to power essential appurtenant structures are most accurately classified as profile pipes, but are excluded from this discussion.

5.4.4.2. Through Pipes. Of the two pipe types classified as being within an embankment, through pipes provide a greater opportunity for seepage and seepage-related PFMs since they typically daylight at each end of the embankment, provide a more direct connection from the waterside to the landside, and experience more frequent and greater hydraulic loadings (Figure 5-12).

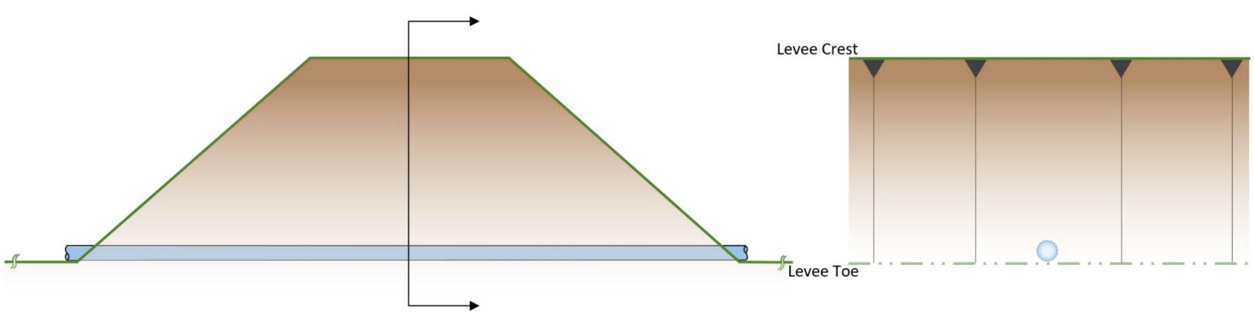


Figure 5-12. Pipe "through" a levee embankment.

5.4.4.3. Profile Pipes. Seepage from the waterside to the landside of an embankment along a profile pipe is less likely than along a through pipe since the pipe typically does not daylight, the hydraulic loading is less, and the water level must at least reach the highest pipe invert (below the crest). This means that most profile pipes will rarely see flood events high enough or with a long enough duration initiate seepage. For new pump stations, USACE recommends that the discharge pipes be elevated as high in the embankment as fiscal restraints allow, understanding that larger, more expensive pumps are required to increase the pumping head. The transverse (horizontal) portion of a replacement pump station discharge pipe may not be located lower than the pipe it is replacing or have less than four feet of cover at it ends (Figure 5-13); this may require overbuilding the embankment above the pipe.

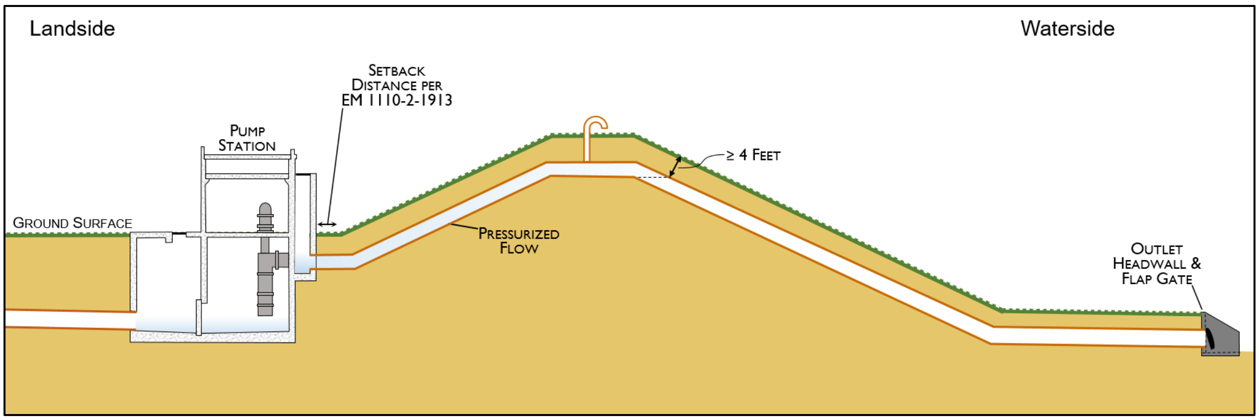


Figure 5-13. Profile pipe originating at a pump station.

5.4.5. Beneath Embankment Pipes. Pipes may be installed in the foundation beneath a levee by trenching (Section 5.5) or trenchless methods (Section 5.6), assuming the proper authorization has been obtained. Installations using trenching methods are usually not technically challenging, but do require large excavations to safely reach a working platform as well as comprehensive emergency closure plans that will likely include requirements to stockpile large amounts of materials. In contrast, trenchless installations entail less excavation and their emergency closure plans require less effort, material, and manpower, but they are more technically challenging and require specialized review of design documents to deter hydraulic fracturing. Existing pipes beneath a proposed new levee alignment must be inspected and meet the USACE condition assessment requirements discussed in Chapter 6 prior to allowing them to remain in place.

5.4.6. Adjacent to Embankment Pipes. Pipes installed adjacent to a USACE project are typically toe drains or relief well collectors. They usually run along the levee toe within the influence zone (defined in Chapter 6) to intercept and redirect seepage. Designs for new toe drains and relief well connector pipes must include man-entry cleanout/access structures no more than 300 feet apart to facilitate maintenance and inspections.

5.4.7. Pipes Crossing Existing Floodwalls and Closure Sills. USACE prefers that new pipes crossing existing floodwalls be installed over the top of the floodwall without contacting the wall (Figure 5-14) or through the nearest closure sill. However, there may be cases where the pipe needs to pass through: 1) the stem of a T- or L-wall or an I-wall section where sheet piling is embedded in concrete; 2) the key of a T- or L-wall; or, 3) through the seepage barrier sheet piling below an I-wall or key of a floodwall (Figure 5-15 and Figure 5-16). In these scenarios, a permanent casing must be installed or formed by coring an acceptably smooth hole. Pipe crossings that require mounting fixtures to the floodwall must not have the potential to significantly damage the concrete or reinforcing steel or to inhibit flood fighting. Reference Section 5.7 and EM 1110-2-2502 for pipes through new floodwalls.

(Courtesy of USACE Louisville District)

Figure 5-14. Supported (left) and freestanding (right) pipes over a floodwall.

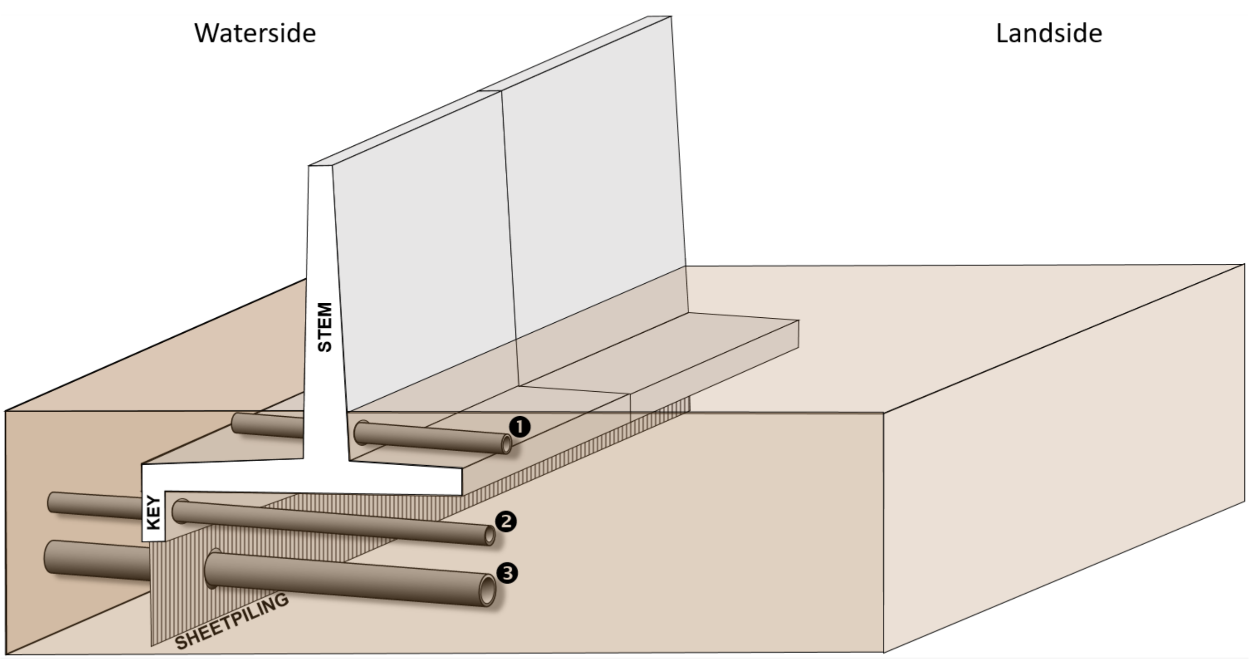


Figure 5-15. Potential pipe penetration locations.

(Courtesy of USACE Louisville District)

Figure 5-16. Penetrations through floodwall key (left and pipe #2 in Figure 5-15) and sheet piling (right and pipe #3 above).

5.5. <u>Trenched Placement</u>.

5.5.1. General. Trenched placement involved excavating soil along a path, preparing the excavation to receive a pipe, installing the pipe, and backfilling around the pipe. Although relatively straight forward, the precautions and restrictions specified in this manual must be adhered to in order to reduce the probability of a pipe-related failure mode since a hydraulic load will eventually be applied.

5.5.2. Trenched Placement Considerations. Prior to excavating the trench, the designer should determine if CLSM or soil backfill is desired since it will impact the trench dimensions. If CLSM is used, the trench excavation must provide the minimum dimensions described in Section 5.5.10.1., with the sides of the trench acting as the formwork. If soil is used, the excavated trench must be wide enough to accommodate the hand-operated equipment used at an angle to compact the soil within the pipe haunches. Unless the loads were determined assuming a positive projecting embankment condition, the designer should consult Chapter 4 to verify the soil loads based on the trench width. During new levee embankment construction, it is generally preferred to partially construct the new embankment so that a trench may be cut through uniformly compacted fill. The partially completed embankment should be constructed high enough that the backfill over the pipe will be sufficient to protect it from equipment loads once embankment construction resumes. The bottom of the excavated trench should be tested to verify soil and groundwater design assumptions.

5.5.3. Trenched Placement Restrictions. With the exception of trench boxes, deep temporary shoring of pipe trenches through an embankment is prohibited unless approved by the respective USACE District. The soil disturbance associated with vibratory or impact hammers for deep shoring, such as sheet piling or piles, could damage otherwise acceptable embankments and the removal of sheet piling can leave gaps in the embankment or foundation (Figure 5-17). While trench boxes are less likely to disturb embankment soil, they can create lines of disturbance that must be addressed to prevent creating a preferential seepage path. In addition, field personnel must ensure no damage to the soil surrounding the trench occurs when the trench box is advanced, leaving the side walls temporarily unsupported. Indications of trench wall instability or soil distress, such as sloughs, bulging walls within the excavation, or tension cracks near the edges of the excavation must be addressed immediately, and any compromised embankment soil reworked. Trench backfill may never be more permeable than the surrounding material; in addition, conventional, clean, crushed-stone bedding is not allowed within USACE levees as it allows substantial soil infiltration and large seepage flows.

(Courtesy of USACE Louisville District)

Figure 5-17. Arrows indicate soil gaps created when excavation shoring was removed.

5.5.4. Settlement Management. In cases where significant settlement along the pipe alignment is anticipated (reference EM 1110-1-1904), the preferred method of mitigation is pre-loading the foundation to reduce post-construction settlement. Reducing the post-construction settlement reduces stress on the pipe connections and the chance of producing a sag in the pipe alignment that perpetually holds water. In cases where preloading is not an option, it is sometimes possible to address settlement-related issues by varying the vertical trench alignment to create a "hinge point" (Figure 5-18). This approach ensures positive drainage is maintained regardless of settlement and is preferred to radial-camber installations since a cambered alignment has the potential to retain water if the actual settlement is less than expected. If camber is used, consult the pipe manufacturer to determine the pipe's maximum acceptable deflection and strain and the joint's maximum degree of curvature. The pipe connection to the structure must be flexible and occasionally observed during settlement to ensure the connection remains secure.

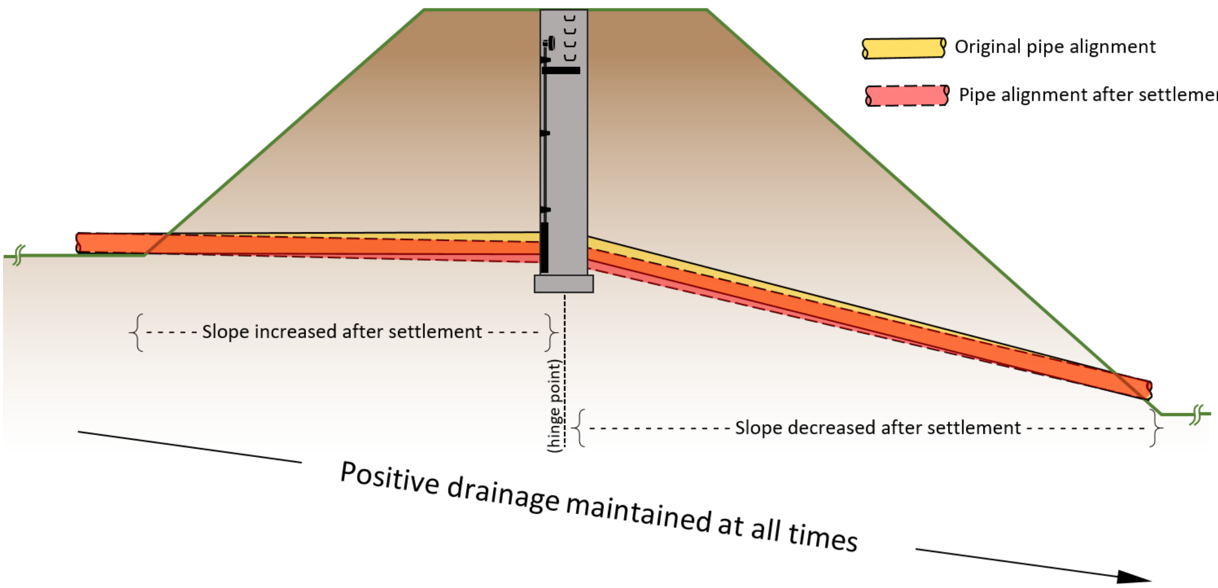


Figure 5-18. Vertical pipe alignment to compensate for foundation settlement.

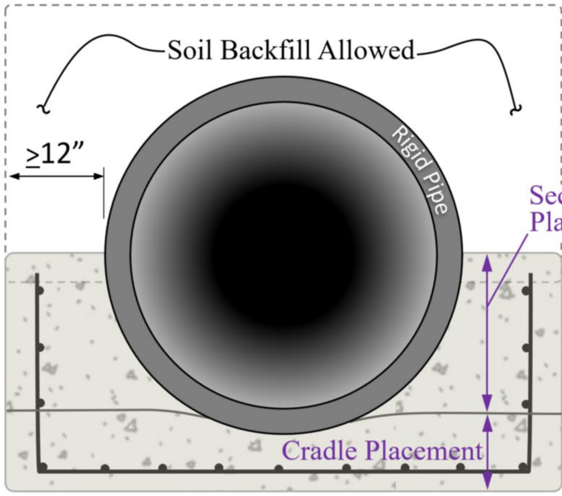
5.5.5. Cementitious Pipe Cradles.

5.5.5.1. General. Foundation conditions within an excavated pipe trench can vary, and combined with the weight of the embankment above can result in differential settlement of the pipe, typically affecting the joints. To reduce the effects of differential settlement, cementitious pipe cradles can be formed in the trench bottom to bridge soft zones. Pipe cradles are not synonymous with pipe supports; a pipe cradle is a continuous, formed structure or shaped excavation in the bottom of the trench, whereas pipe supports are discrete elements that raise the pipe off the ground.

5.5.5.2. Rigid Pipe Support. When supporting rigid pipes, it is recommended that a pipe cradle consisting of two concrete placements be used to bring the top of the cradle to the pipe's springline (Figure 5-19, left). This provides a flat surface to compact against if the design includes soil backfill and also prevents the need to compact soil within the haunch. If the designer chooses to use a secondary placement to reach the springline, then some vibratory effort is prudent to ensure the tightest portion of the haunch is filled since the coarse aggregate within the concrete could impede its flow. It is further recommended that the cradle include reinforcement and a shallow trough in the center to help maintain the pipe's alignment (Figure 5-19, right). CLSM can also be used to create a cradle; however, it is often too fluid to form the shallow cradle.

5.5.5.3. Flexible Pipe Support. Flexible pipes may be placed on cementitious cradles, but the remainder of the backfill must be either all soil or all CLSM because flexible pipes cannot tolerate the shearing forces produced by differential backfill.

5.5.5.4. Cradle Extent. If a cementitious pipe cradle is used, it is recommended that it be placed along the full extent of the pipe. This will help prevent pipe cracking or stress concentrations in joints from differential support conditions. Note that filter material extends beneath the cradle within the diaphragm portion (reference Section 5.5.2.2.). For flexible pipes fully encased in CLSM, only the cradle portion directly beneath the pipe is carried through the seepage filter to the headwall.



(Courtesy of USACE Los Angles District (right))

Figure 5-19. Cross-section (left) and photo (right) of cradle-supported rigid pipe.

5.5.6. Soil Pipe Cradles. Soil cradles can be used to ensure properly compacted soil in the pipe haunches as soon as the segments are placed; however, if used they must be cut to the springline of the pipe. In addition, the shape of the cradle must conform to the radius of the pipe so that a seepage path is not created (Figure 5-20). These requirements make soil cradles an infrequently used option. Bell and spigot joints may be used with soil cradles as long as bell holes are cut into the cradle (Figure 5-21) and are filled with a cementitious material (preferably CLSM). The bell holes need to be large enough to allow the spigot of the next pipe segment to be positioned without touching the soil and contaminating the joint.

(Courtesy of USACE Louisville District)

Figure 5-20. Manual construction of a soil pipe cradle for a 66-inch diameter pipe.

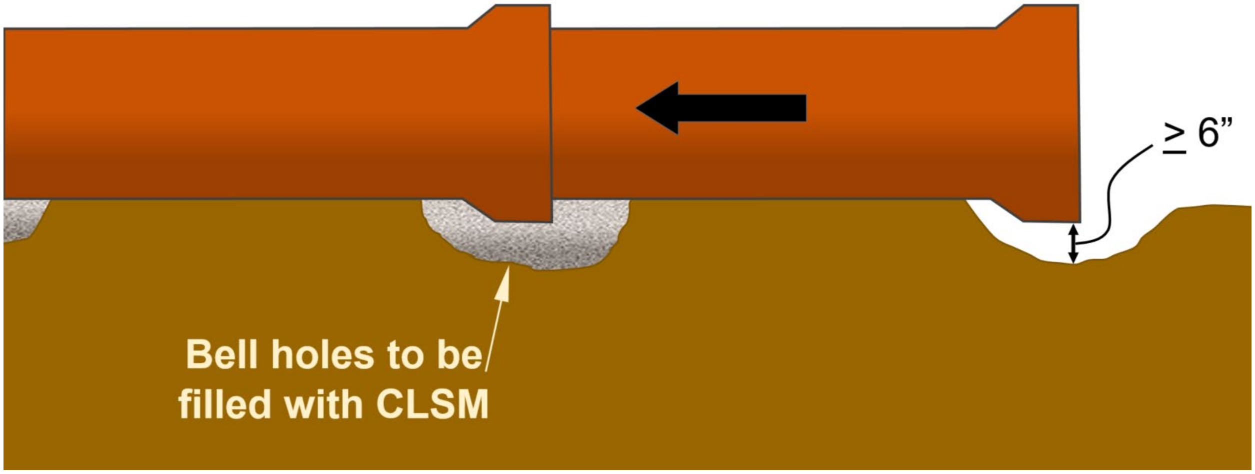


Figure 5-21. Holes prevent point-loading of the bells and allow uniform support of the barrel.

5.5.7. Elevated (Supported) within Trench. To receive CLSM backfill, pipes within trenches must be elevated on supports to meet both the bell and barrel minimum clearances to allow the CLSM to flow beneath the pipe and completely fill the haunches. Sandbags or compacted soil pads are the preferred pipe supports, but other non-rigid supports such as wood or Styrofoam blocks are acceptable if their stiffness is less than that of the pipe material. Supports should be placed from L/4 to L/5 from the pipe ends, where L is the pipe segment length (Figure 5-22).

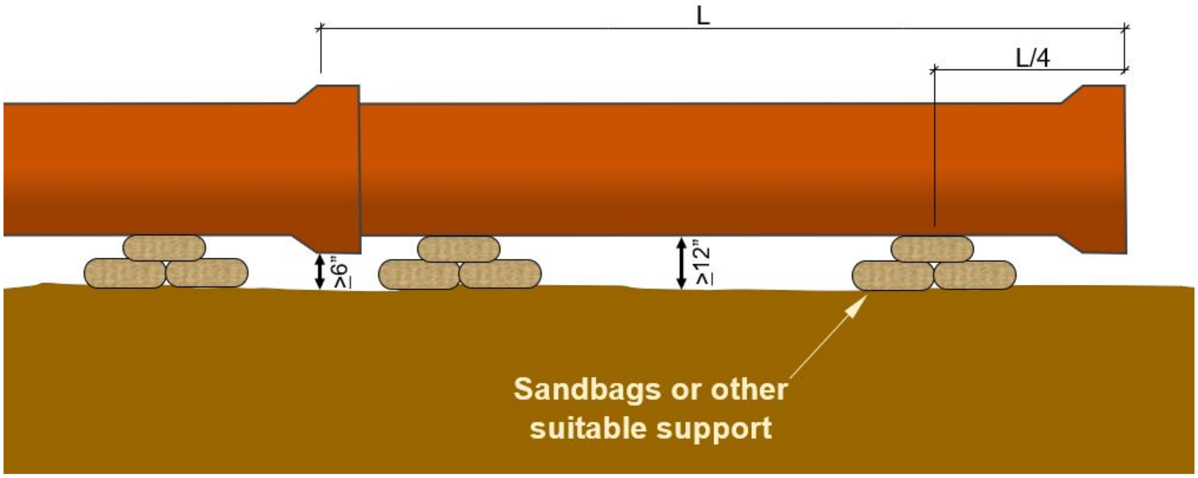


Figure 5-22. Typical configuration for pipe support before CLSM placement.

5.5.8. On Ground within Trench. Pipes should be placed directly on the ground when soil backfill is used so that the bottom of the pipe can be used as a backstop to compact the soil against (Figure 5-23). Poor compaction within the haunches can lead to internal erosion issues, as described in FEMA 484. It is recommended that the pipe segments be secured to prevent movement during backfill compaction; even seemingly heavy pipes can be moved by compaction forces. It is prohibited to place the pipe on the ground if CLSM backfill will be used since the CLSM would not be able to fully encapsulate the pipe and seepage paths within the smallest parts of the haunch may be created.

(Courtesy of FEMA)
Figure 5-23. Compaction of Soil Backfill in the Pipe Haunches.

5.5.9. Seepage Filters.

5.5.9.1. General. Pipe installations, whether trenched or trenchless, have the potential to create preferential seepage paths from the waterside to the landside. Seepage filters address this issue through relieving local pore pressures by allowing the passage of water while preventing the migration of soil particles (internal erosion) when the levee is loaded. There are two different types of acceptable seepage filters: 1) internal seepage filters are preferred and are required for new through pipes that daylight near the landside embankment toe, regardless of the chosen trench backfill material; 2) external seepage filters are typically used as a repair option for existing pipes (reference Section 7.4.10.); however, their use is required on new trenchless gravity pipes installed within an embankment. Filter designs for dam conduits must comply with the latest version of either the Federal Emergency Management Agency's (FEMA's) "Filters for Embankment Dams" or EM 1110-2-1901.

5.5.9.2. Seepage Risks and Remediation. For pipes that pass beneath an embankment but do not daylight, a preferential seepage path to the surface can be created when a pipe is installed using "cut-and-cover" methods that penetrate a clay blanket. A trench excavation can compromise a confining layer (Figure 5-24 A and B) and promote a form of PFM-1 where the walls of the trench or improper backfill provide the seepage path. Properly backfilling the trench with cohesive soil (Figure 5-24 C) or CLSM (Figure 5-24 D) should greatly reduce the possibility of PFM-1 occurring. The designer must also determine if a filter is required based on the specific project conditions. While not a filter, seepage "collars" were often used in the past to reduce seepage along pipes; however, they are now prohibited since they can be difficult to compact backfill around, leaving the potential of preferential seepage paths through loose soil.

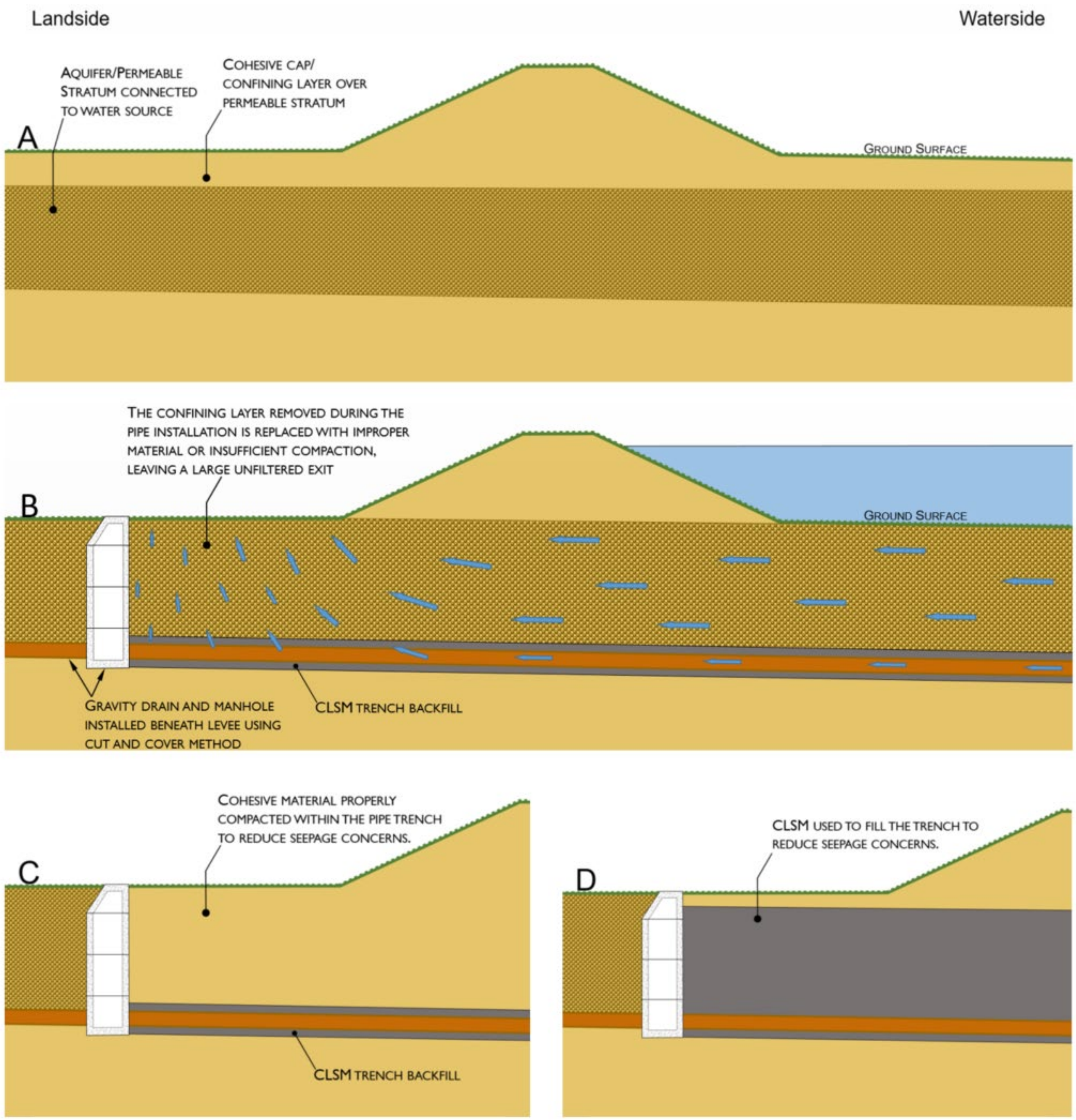


Figure 5-24. Seepage concern and remediation along excavation for non-daylighting pipes.

5.5.9.3. Internal Seepage Filter Details.

5.5.9.3.1. Diaphragm and Transition. An internal seepage filter is comprised of two seamlessly joined elements: 1) a filter diaphragm having a cross-sectional area larger than the pipe trench; and 2) a transition zone connecting the diaphragm to the headwall to provide filter drainage (Figure 5-25 through Figure 5-29). The diaphragm is designed to be large enough to cover all potential seepage paths associated with installing the pipe, including: seepage along the pipe exterior; seepage within the soil backfill trench or along the CLSM backfill-trench wall interface; and seepage through defects caused by movement of the trench walls after the trench was excavated. The diaphragm width (perpendicular to the pipe) must be at least as wide as the trench depth, but not less than three feet in any direction beyond the edges of the trench. The filter diaphragm is the only portion of the filter that must extend beneath the pipe and/or cradle. It must also satisfy FEMA's encapsulation requirement in FEMA's "Filters for Embankment Dams." The transition zone must surround the pipe and cradle on both the sides and top. The H/2 measurement in Figure 5-26 satisfies FEMA confinement requirements to prevent blowout should the full hydrostatic loading reach the seepage filter and the filter become completely clogged. The seepage filter material should comply with the latest version of either FEMA's guidance or EM 1110-2-1901, but ASTM C-33 concrete sand is typically used.

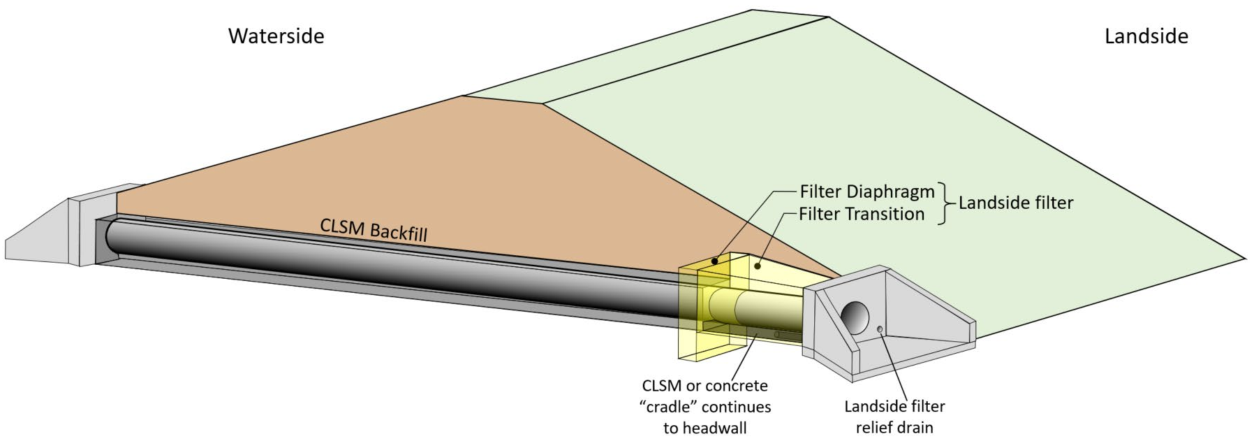


Figure 5-25. View of a gravity pipe through an embankment with an internal seepage filter.

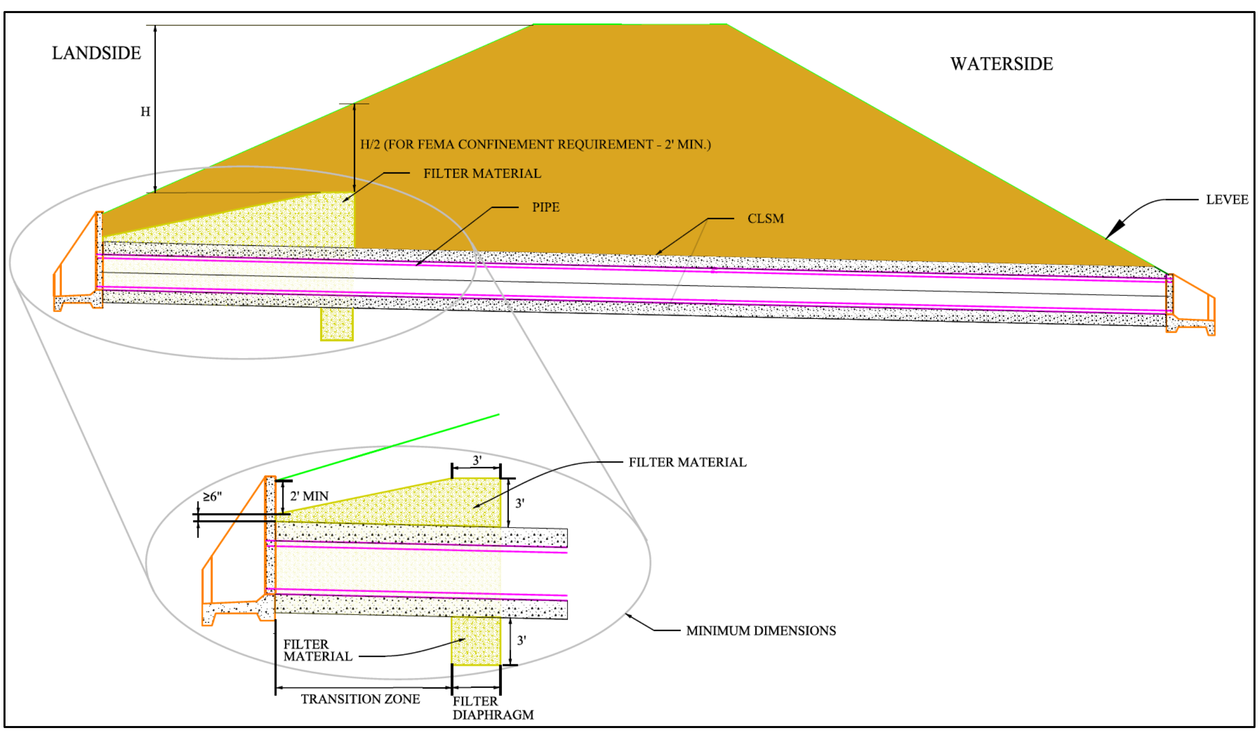


Figure 5-26. Elevation view of pipe and landside internal seepage filter.

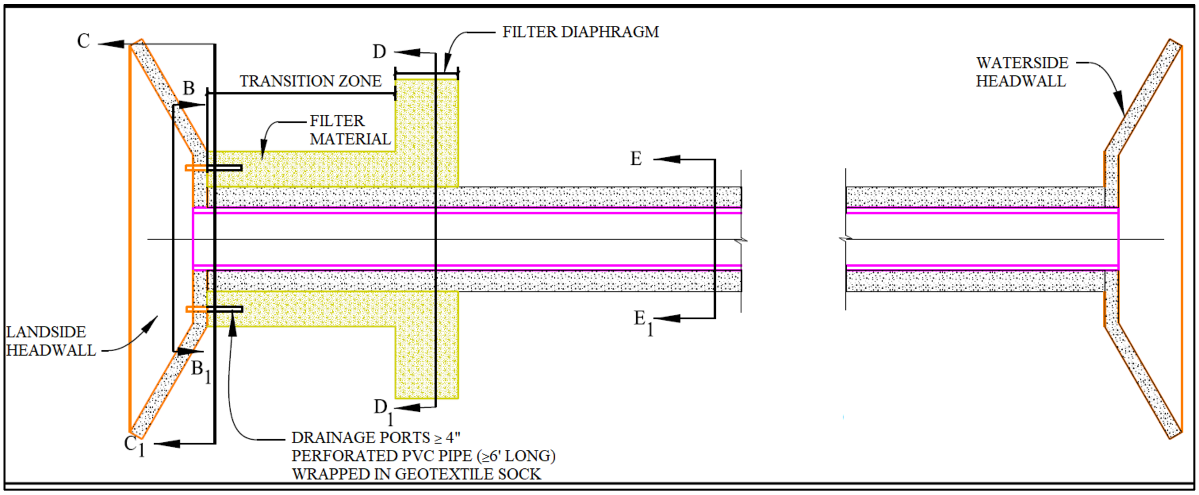


Figure 5-27. Plan view of landside seepage filter.

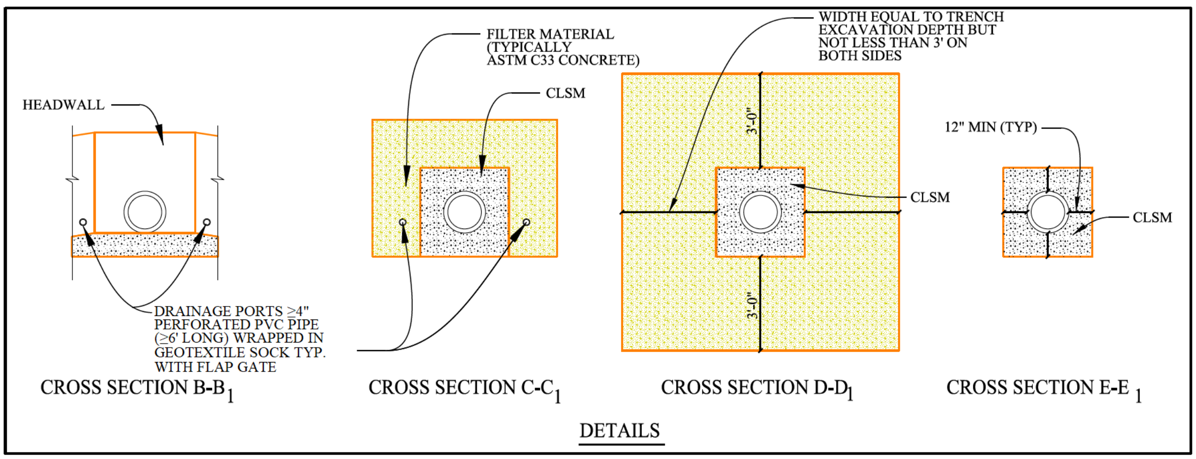


Figure 5-28. Details of landside internal seepage filter.

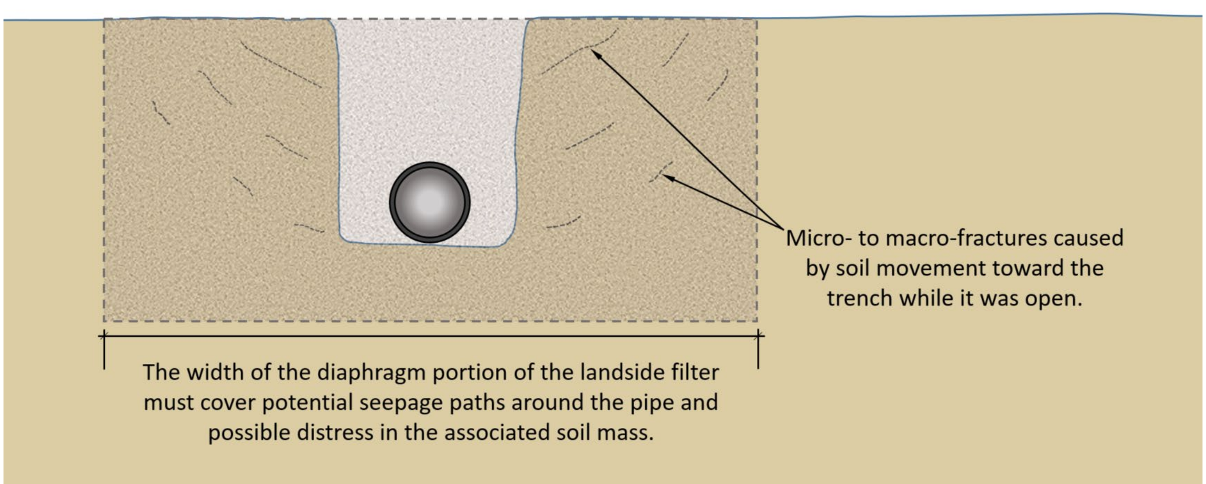


Figure 5-29. Cross-section of filter diaphragm, wide enough to cover potential soil defects.

5.5.9.3.2. Drainage Ports. At least two drainage ports, one on each side of the pipe and each no less than four inches in diameter, are required to be installed through the headwall to drain water collected by the internal seepage filter. The ports should be placed as low as possible on the headwall to minimize the water level on the other side. The drainage ports require flap gates or check valves on the headwall to prevent charging the filter if water ponds on the landside of the pipe. Each drainage pipe should be slotted or ported non-metallic material no less than six feet long and wrapped with geotextile filter socks with the waterside end capped. In cases where the face of the headwall does not provide enough room for ports, they may exit the wing walls (Figure 5-30); however, they must remain encased in the filter material.

(Courtesy of Louisville District)

Figure 5-30. Landside headwall with seepage filter drainage ports.

5.5.9.4. External Seepage Filter Details. External seepage filters serve the same function as internal seepage filters, but are used around new pipes installed in an embankment by trenchless methods (Table 5-3) or as a surficial remediation for existing pipes where the more invasive construction of an internal seepage filter may not be warranted. A filter is required if seepage around either end of an existing pipe has been observed to carry soil particles. Installation of an external filter requires some embankment excavation to seat the filter layers and provide sufficient height so that the weight of the armoring (riprap) results in sufficient filter confinement. Filter confinement should be based on an analysis that assumes that the full hydraulic head is applied to the clogged upstream side of that filter. Figure 5-31 represents a three-layer filter system encapsulating the pipe and covered with armoring; however, some external filters may only require two layers, depending on the size and gradation of the chosen armoring. Each filter layer beneath the armoring must be at least six inches thick. External filters may also be constructed around existing pipes not having a headwall. Filter design must comply with the latest version of either FEMA's "Filters for Embankment Dams" or EM 1110-2-1901. The size and angularity of the armoring should be chosen to provide stability against movement from surface runoff, velocity of adjacent flowing river/streams, wave action, mowers, pedestrian traffic, etc.

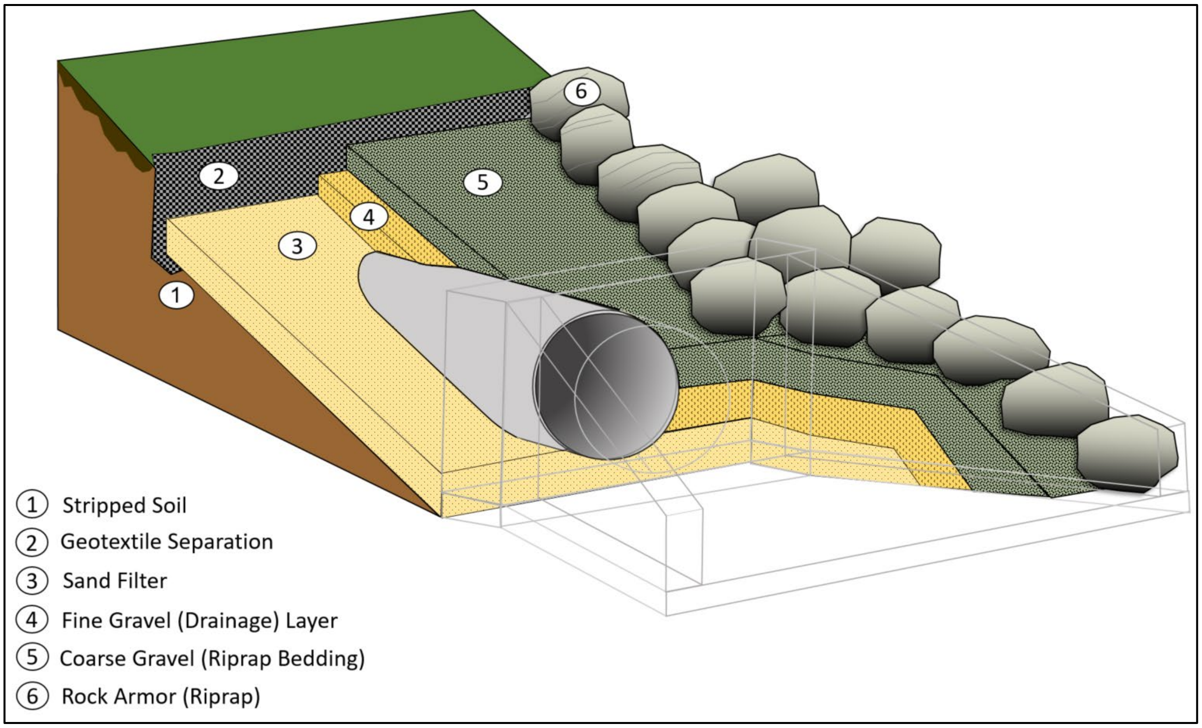


Figure 5-31. Details of External Seepage Filter (with or without a headwall).

5.5.10. Pipe Joint Types.

5.5.10.1. General. Joints for all pipes installed within the influence zone of a dam or levee must be designed to accommodate lateral and longitudinal movement while minimizing leakage (according to the appropriate ASTM testing method), which may cause embankment erosion from either exfiltration or infiltration. A well designed and installed pipe joint is expected to limit vertical and transverse displacement of pipe segments relative to one another, accommodate rotation and longitudinal movement, provide consistent flow, and allow easy installation, all while minimizing leakage as the embankment settles (FEMA, 2005).

5.5.10.2. Pipe Joint Installation Overview. Pipe segments must be joined according to the manufacturer's instructions to better ensure their success and longevity. In general, pipe segments are placed only after pipe supports or cradles have been properly prepared. Pipe placement should begin at the lower elevation and continue uphill, which uses gravity to help home the joints and maintain a seating force. General descriptions for each of the four major joint types follows and Table 5-1 shows the applicable standards for each pipe material and joint connection type.

5.5.10.3. Bell and Spigot Joints. Bell and spigot, tongue and groove, and similar joint types must be fully homed to properly engage the gasket (Figure 5-32 and Figure 5-33). Figure 5-34 shows an acceptable manual method for installing/homing bell and spigot pipes. Bells are necessary on smaller-diameter reinforced concrete pipes (RCP) to accommodate the required reinforcing bars essential to maintain the joint's integrity due to the forces of homing the next segment and to resist sheering forces after the pipe is loaded with the embankment overburden. A gap exceeding the manufacturer's maximum allowable separation may allow soil intrusion or water exfiltration and initiate PFM-2 or PMF-3. The "X" dimension of the bell and spigot joint in Figure 5-32 represents the gap that must be measured to determine if the gasket is fully engaged or not. Bell and spigot joint design differs significantly among manufacturers; designers should always consult the manufacturer for specific dimension requirements. Profile gaskets are non-round gaskets that must be oriented correctly on the pipe to provide a competent seal. ASTM C1619 covers elastomeric joint seals for concrete pipe.

Table 5-1
Applicable Joint Standards by Pipe Type and Joint Type

Pipe Material[1]	Joint Type[2]			
	Bell & Spigot, Compression, & Push-on	Coupler	Solvent or Heat Fused & Welded	Mechanical & Restrained
NP-RCP (<10.8psi)	ASTM C443, ASTM C1628	N/A	N/A	N/A
LP-RCP (<54.2 psi)	ASTM C361	N/A	N/A	N/A
CPP -PCCP -RCCP -RCNP -BWCP	AWWA M9 AWWA C301 AWWA C304 AWWA C300 AWWA C302 AWWA C303	N/A	N/A	N/A
VCP	ASTM C425 ASTM C1208	N/A	N/A	N/A
FRP	AWWA M45, ASTM D4161, ASTM F477	AWWA M45 ASTM D4161	AWWA M45, including laminated	AWWA M45
CSP	AASHTO M36/ ASTM A760	AASHTO M36/ ASTM A760	N/A	See Couplers
CAP	AASHTO M36/ ASTM A760, ASTM B788	AASHTO M36/ ASTM A760 ASTM B745 ASTM B788	N/A	See Couplers
WSSP	AWWA M11	AWWA M11	AWWA M11, AWWA C206	AWWA M11
DIP	ANSI/AWWA C111/A21.11	N/A	N/A	ANSI/AWWA C111/A21.11
SW-HDPE	ASTM D3212, ASTM F477	N/A	ASTM D2321, ASTM F2620	AWWA C219, ASTM F1924
PW-HDPE	ASTM D3212, ASTM F477	N/A	N/A	N/A
Pipe Material[1]	Joint Type[2]			
	Bell & Spigot, Compression, & Push-on	Coupler	Solvent or Heat Fused & Welded	Mechanical & Restrained
PVC	ASTM D3212, ASTM D2774, AWWA C605	N/A	ASTM D2321, ASTM D2774, AWWA C605	AWWA C219, ASTM F1924, AWWA C605
SRTP	ASTM F913, ASTM D3212	N/A	N/A	ASTM F913, ASTM D3212
CiPCP	FEMA 484 – monolith (contraction) joints with water stops (EM 1110-2-2102) and construction joints.			
RCB	ASTM C1677	N/A	N/A	N/A

Notes:
1. See Abbreviations in the Glossary for pipe material acronyms.
2. AASHTO R82 is an overall pipe joint selection guideline for most of the listed pipe materials.

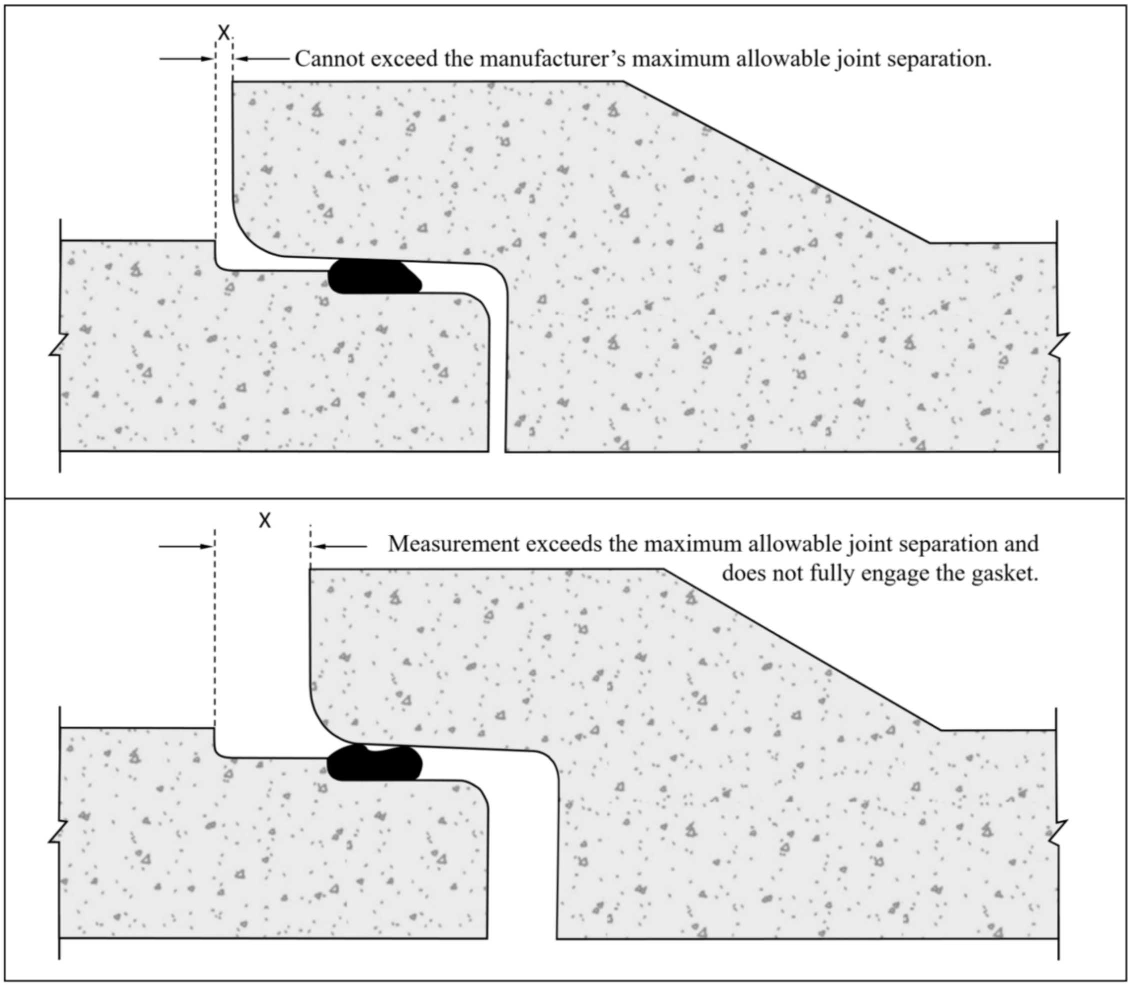
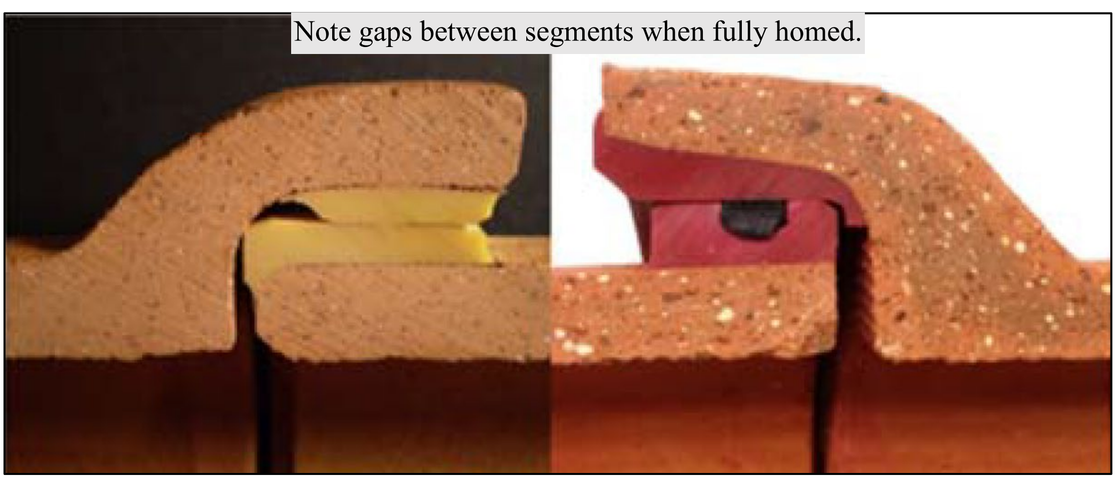


Figure 5-32. RCP bell and spigot joint (top, correctly homed; bottom, gasket not fully engaged).



(Courtesy of Gladding, McBean and Logan Clay Products)

Figure 5-33. Polyurethane (left) and Gasket (O-ring) and polyester (right) compression joints.

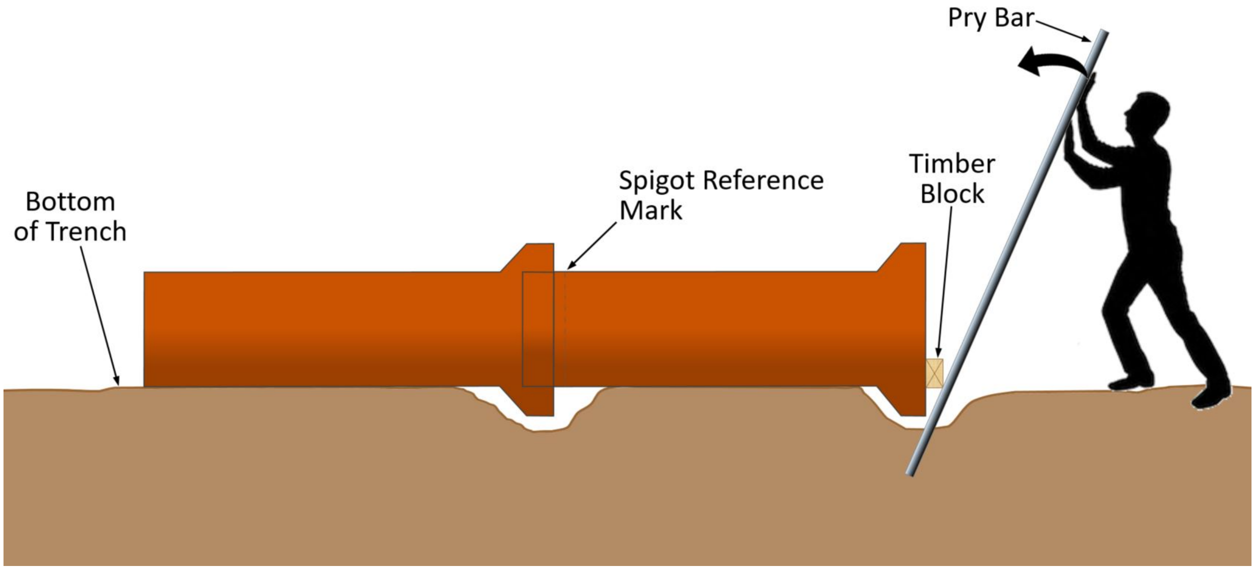


Figure 5-34. PVC Pipe – bar and block assembly.

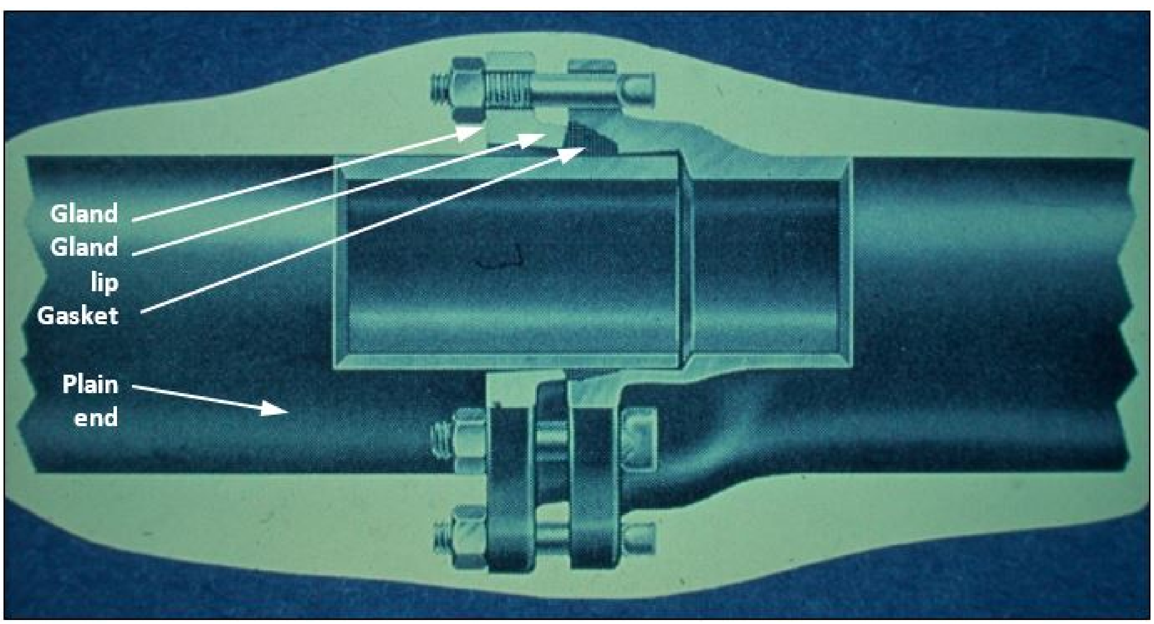
5.5.10.4. Coupler Joints. A coupler is a short piece of pipe with socketed ends having inner diameters slightly larger than the outer diameters of the two pipe segments it is designed to join. The coupler typically uses heat or chemical welding to complete the joint. If the pipe is corrugated, ensure corrugations of the coupling band match corrugations on the pipe ends. Larger diameter corrugated metal pipes often must be assembled in a specific sequence determined by the manufacturer. Use suitable end protection, such as wood timbers, to protect the pipe end during installation. The joint manufacturer should be consulted on how to minimize leakage through the joints.

5.5.10.5. Fused or Welded Joints. Fusing or welding relies on the strength of a solidified material once it has been melted and joined with another like material. Circumferential fillet welds for lap joints need to account for additional stresses that may develop due to thermal loadings from the welding process when the pipes are anchored or fixed on both sides of the joint. Improperly fused joints cannot be repaired and therefore must be cut out and the fresh ends fused together using specialty equipment (Figure 5-35).

5.5.10.6. Mechanical Joints. Mechanical joints may be temporary or permanent, but most are designed to be disassembled (Figure 5-36). Many mechanical joints are designed to allow relative movement of the two ends in one degree of freedom while restricting movement in one or more directions; this is intended to allow flexibility while minimizing joint leakage.

(From FEMA, 2007)

Figure 5-35. Apparatus for heat fusing solid-wall high density polyethylene (SW-HDPE) pipe.



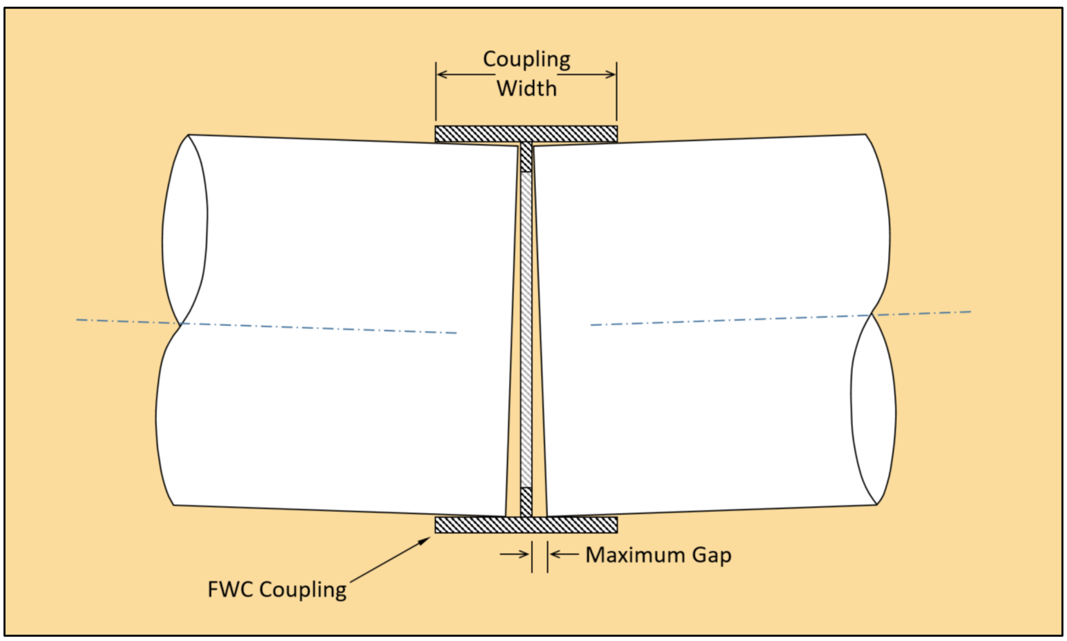
(Courtesy of DIPRA)

Figure 5-36. Ductile iron pipe (DIP) mechanical joint, section view.

5.5.11. Pipe Connections to Associated Structures and Appurtenances. Chapter 9 covers pipe connections to associated structures and appurtenances in levees and Chapter 10 covers similar connections in dams.

5.5.12. Pipe Connections to Special Pipe Fittings and Other Pipe Types.

5.5.12.1. Special Fittings. Changes in pipe alignment should be made with special curved fittings to ensure the pipe segment is fully engaged within the connection. A less desirable but acceptable alternative to a specialized fitting is to create long-radius curves by deflecting pipe segments at the joints to create small angular changes while ensuring none of the deflections exceed the manufacturer's limits. Joint deflection angles exceeding the manufacturer's limits increase the likelihood that the joints may separate greater than the allowable limit, letting pressurized fluid escape or soil to infiltrate. Figure 5-37 is an example of one in a series of deflected joints required to make a large radius bend. Beveling one or both ends in a joint requires a factory designed joint (except for fused joints) to minimize leakage and may not be economical. Connections between different pipe sizes are handled by special fittings, the installation of which must closely follow the manufacturer's recommendations. Connections to other special pipe fittings, such as taps, also require joints designed to minimize leakage.



(Courtesy of HOBAS Pipe-USA)

Figure 5-37. Deflected fiberglass reinforced pipe (FRP) joint to make large radius bend.

5.5.12.2. Other Pipe Types. The installer must contact the pipe manufacturers for guidance on joining two dissimilar pipe materials to prevent leakage and avoid initiating PFM-1 or PFM-2. Typically, external bands or clamps are used to hold the segments of dissimilar pipe types together and to secure a gasket, but since this is not optimal, connections between dissimilar materials within the levee inspection limits as determined in Section 6.3. are strongly discouraged. Each industry has numerous connection options such as concrete encasement, couplers, and flanges and should be contacted by designers for guidance.

5.5.13. Thrust Blocks for Pressurized Pipes. Pressurized flow at a change in a pipe's alignment creates a hydrostatic thrust force that can damage pipe joints. A common method for resisting these forces for pipes founded in soil is the installation of concrete thrust blocks that rely on passive soil pressure for the reaction force. Mechanical couplers should be installed near the thrust block to further protect the pipe joint integrity. Protection may also be in the form of additional wrapping and bonding for FRP and plastic pipes. Gravity flow pipes are typically straight and unpressurized, but even they may require restraint if the fluid velocity is high enough to induce a thrust force where there is a change in alignment. Thrust blocks should be designed according to the Construction Industry Research and Information Association (CIRIA) Report 128, Guide to the Design of Thrust Blocks for Buried Pressure Pipelines.

5.5.14. Anchor Blocks for Pressurized Pipes. Similar to thrust blocks, anchor blocks are buried within the embankment and resist fluid thrust forces using their weight and the surrounding soil resistance. However, anchor blocks resist the force from the opposite side and are used where there is inadequate soil cover for a thrust block. Anchor blocks may also be used to support elevated pipes on embankment side slopes to prevent the pipe from moving down the slope. The block burial depth is a function of the resisting force.

5.5.15. Joint Harnesses. In areas where gasketed-type joints and non-restraining couplings are subject to thrust loading, harnesses can be used across the joint. Harness assemblies and tie rod sizes vary based on the pipe diameters being utilized and the pressures being experienced. AWWA M11 provides sizing information for joint harness assemblies and tie rods for given pipe diameters and select common design pressures.

5.5.16. Control of External Corrosion on Underground Metallic Pipes.

5.5.16.1. General. Pipe segments with an anti-corrosion coating should be accompanied by similarly protected fittings, accessary joints, etc. so that all components of the pipe system have a relatively equal service life. Discontinuity (holiday) detectors should be used according to ASTM D5162 to detect and repair coating flaws prior to backfilling.

5.5.16.2. Cathodic Protection. Metal pipes require supplementary (cathodic) corrosion protection (CP) beyond their surficial protective coating if an investigation of the surrounding environment indicates conditions promoting accelerated corrosion. The pipe materials, functions, and environments particularly vulnerable to accelerated attack are itemized in UFC 3-570-01, but a CP specialist should be engaged to determine if a supplemental protection system, such as galvanic anodes (passive) or impressed current (active), is required. Newly installed metallic pipes may experience accelerated corrosion from stray currents (interference) if placed near an existing pipe or structure using a CP system or other voltage gradients. Existing active CP systems near the alignment of the proposed pipe should be investigated and appropriate mitigation action taken if necessary.

5.5.16.3. Galvanic Anode Protection. Galvanic (sacrificial) anodes are physically connected to a pipe and placed in the surrounding backfill; however, the system no longer provides protection once the anode material is depleted. Reinstituting protection would require excavating the embankment to replenish the anode field; therefore, other alternatives should be considered due to the invasiveness of this procedure.

5.5.16.4. Impressed Current Protection. An impressed current system also uses sacrificial anodes, but a constant power source allows the anode field to be installed beyond the embankment toe, making replenishing the depleted field far less challenging than the passive alternative.

5.5.17. Pipe Locators. The location and orientation of all new pipe installations in the vicinity of a USACE embankment or floodwall, except for gravity drains daylighting at both embankment toes, must be identified by above ground signage and either tracer (locator) wire or tracer (marking) tape. Above ground signage information should include: project stationing at pipe crossing; top of pipe elevation (including datum); pipe diameter; products that are carried in the pipe; and pipe owner and/or emergency contact information. If signage is not possible, an energized or non-energized tracer wire along the pipe's crest should be installed so an above-ground device can detect its location. Tracer tape is typically a color-coded plastic ribbon placed several feet above the pipe so that excavations within the area will alert observers to the presence of a pipe before damaging it.

5.5.18. Backfill Alternatives.

5.5.18.1. Controlled Low Strength Material Backfill. Although soil backfill is permitted, CLSM is preferred since its properties are better controlled to provide a more consistent and less permeable product in defense against PFM-1 and PFM-3. Flexible pipes cannot tolerate the shearing forces caused by backfilling with dissimilar materials (such as CLSM to the springline and soil above that); therefore, when using CLSM, it must fully encase the pipe at least 12 inches all around. Rigid pipes can withstand these shearing forces and therefore soil may be used above the springline, assuming at least 12 inches of CLSM surrounds the lower half. While CLSM typically flows well during placement, stiffer consistencies may require vibration to ensure the space below the springline is completely filled, but inducing bleed water should be avoided. Restraining the pipe against movement is recommended since discharges from a chute or bucket can cause an uneven load or push the pipe laterally, and flotation can occur subtly without warning. Flotation can be prevented by either creating a temporary formwork that provides pressure from above (Figure 5-38, left) or a weighted/anchor system (Figure 5-38, right). Figure 5-39 shows weights (often concrete blocks) being hung over a pipe (A), the first lift of CLSM being placed so that the weights become anchors once the material has cured (B), the second lift being placed to a level that will not induce flotation (C, which sufficient anchorage may allow to be above the springline), and the final placement if the height of the second left was limited (D). CLSM placement for profile pipes can be limited to the horizontal alignment as shown in Figure 5-40. A recommended CLSM mix design is provided in Table 5-2.

(Courtesy of Louisville District)

Figure 5-38. Methods to prevent flotation during CLSM placement.

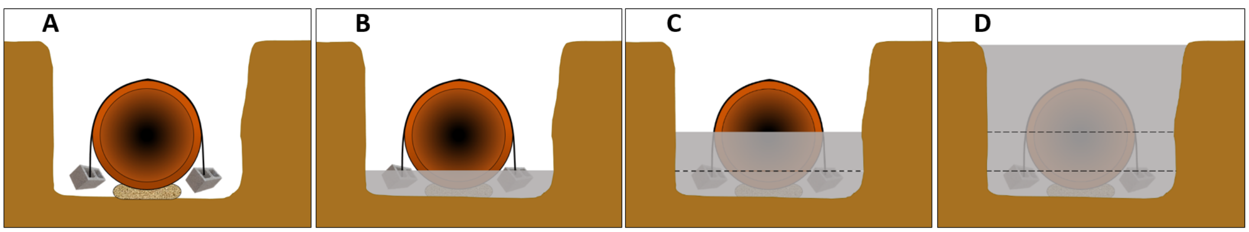


Figure 5-39. Anchoring method to prevent flotation.

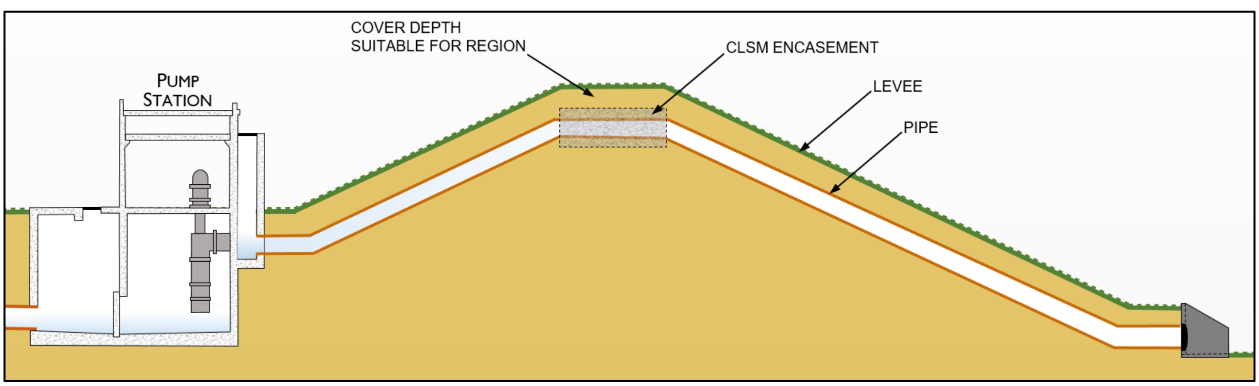


Figure 5-40. CLSM placement for profile pipes.

Table 5-2
Example mix design limits for one cubic yard of CLSM

MATERIAL	DOSAGE
Portland Cement (ASTM C150 - Type I or II)	80-100 lbs. (Air temp. >32° F) or 150-175 lbs. (Air temp. <32° F)
Fly Ash (ASTM C618 - Class F, C, or N)	200-400 lbs. (Air temp. >32° F) or 200-300 lbs. (Air temp. <32° F)
Sand (ASTM C33)	2000-3000 lbs. (Depends on air, water & cementitious materials)
Potable Water	Dosed to create a water-to-cement ratio of 1.0 to 1.3
Air Content (ASTM C260)	10% to 20%
Unit Weight	110 pcf to 126 pcf
Shrinkage Reducing Material	Bentonite dosed at half the weight of the Portland Cement used, -- or -- a Shrinkage Reducing Admixture dosed at a rate recommended by the admixture manufacturer.

Note: A compressive strength between 30 and 300 psi, as tested according to ASTM C39/C1019, is desired so that it is roughly compatible with soil, yet remains easily excavatable by mechanical equipment.

5.5.18.2. Soil Backfill. Although CLSM is the preferred trench backfill to reduce the likelihood of PFM-1 and PFM-3, soil backfill can be considered acceptable with proper precautions and placement techniques as long as the respective USACE District approves the placement materials and specifications. When soil backfill is used, it cannot be more permeable than the surrounding soils, except for toe drains. Achieving adequate soil compaction even in ordinary conditions with heavy machinery is sometimes challenging, but proper compaction within the pipe haunches using hand-operated equipment is difficult and critical to prevent creating a preferential seepage path. It is recommended that the pipe be braced since compaction below the springline requires an angled approach (Figure 5-23) which can move and lift the pipe. Pipe joints must be wrapped with filter fabric when backfilling with soil to reduce the likelihood of infiltration, even when joint gaskets are present. Soil backfill is not permitted for elevated (supported) pipes in trenches because there is no way to adequately compact soil beneath them. Figure 5-41 demonstrates unacceptable backfilling around a glass-reinforced pipe for multiple reasons: 1) the pipe was elevated but soil backfill was used; 2) the pipe was supported by concrete blocks, a material harder than the glass-reinforced pipe; and, 3) the soil was loosely dumped into the trench with compaction being applied only surficially. The result of this unacceptable backfilling example is that voids and uncompacted soil surrounds most of the pipe, increasing the likelihood of PFM-1. Figure 5-23 demonstrates proper compaction techniques if soil backfill is to be used.

(Courtesy of Louisville District)

Figure 5-41. Unacceptable pipe installation methods increasing the chance of harmful seepage.

5.6. Trenchless Installations.

5.6.1. General. Trenchless installations for non-essential pipes within USACE embankments are only allowed in the scenario described in Section 5.4.2.; essential pipes are not subject to this limitation (e.g., such as when additional gravity drains are required). The criteria within this section is applicable to all forms of trenchless installation unless specifically stated otherwise. Drilling (which includes trenchless installations) within or beneath USACE embankments and floodwalls must follow the requirements described in ER 1110-1-1807.

5.6.2. Installation Methods. Trenchless installations (Table 5-3) consist of steerable and non-steerable methods that produce an overcut, creating an annular space between the excavated soil and the outside of the pipe. For the purposes of this manual, slip-lining methods are considered a trenchless rehabilitation method (reference Chapter 7).

Table 5-3
Comparison of Trenchless Installation Methods

Installation Method	Pipe Diameter Range (in.)	Maximum Installation Length (ft)	Radial Overcut (in.)	Pipe Material	Typical Accuracy / Tolerances
Pilot Tube (ASCE MOP No. 133)	4 to 48	500 (300 to 400 typical)	¼ to ½	WSSP, LP-RCP, RCNP, RCCP, FRP, VCP	¼-inch line and grade over 400 lf
Pipe Ramming (ASCE MOP No. 115)	4 to 147	400 (250 typical)	¾ to 1½	WSSP (open-ended)	Line and grade to within 1 percent of drive length for non-guided methods
Horizontal Auger Boring (ASCE MOP No. 106)	8 to 96	800 (175 to 225 typical)	¼ to ½	WSSP (open-ended), FRP	Line and grade to within 1 percent of drive length for traditional methods
Open-Shield Pipe Jacking	42 or greater	Unlimited, if using intermediate jacking stations	½ to 1½	WSSP, LP-RCP, RCNP, RCCP, FRP	Line and grade to within 2 inches
Earth Pressure Balance Pipe Jacking	42 or greater	Unlimited, if using intermediate jacking stations	½ to 1½	WSSP, LP-RCP, RCNP, RCCP, FRP	Line and grade to within 1 inch
Microtunneling (ASCE Standard 36-15)	12 to 126 (24 to 48 typical)	Unlimited, if using intermediate jacking stations	¼ to 1½	WSSP, LP-RCP, RCNP, RCCP, FRP, DIP, PVC, VCP	Line and grade to within 1 inch
Horizontal Directional Drilling (ASCE MOP 108)	2 to 60 10 (Mini-HDD)	Over 2,000 (600 Mini-HDD)	2 to 6	WSSP, HDPE, PVC, DIP	Line and grade to within 1% of the drill length. If secondary survey coils are used, line and grade accuracy may approach 0.2 ft./10 ft. of borehole depth, even with magnetic interference. In most HDD installations, a horizontal tolerance of 5 feet of the alignment and 2 feet above and 10 feet below the designed profile is acceptable.
Direct Pipe	30 to 60	7,500 [theoretical] (500 to 6,500 typical)	½ to 5	WSSP	± 2 inches with manned entries; ± 10 feet with limited man entries.

5.6.3. Drilling Fluid Pressures and Hydraulic Fracture.

5.6.3.1. General. The installation methods described in Table 5-3 require introduction of water-based lubricants within an annular "overcut" space between the pipe and the soil to facilitate pipe installation and remove drill cuttings. The annular space must be grouted as outlined in Section 5.6.4.7. The drilling fluids or grout that are introduced can cause the borehole to fail through uncontrolled expansion or hydraulic fracturing; therefore, the procedure discussed in this section must be followed.

5.6.3.2. Minimum Depth Requirements. Pipes installed within USACE dam or levee earthen embankments must have a minimum depth of cover below the lowest portion of the crest of at least nine feet, or a depth-to-crown to bore-diameter ratio equal to or greater than three; whichever is greater. Similarly, pipes installed beneath embankments must have a depth-to-crown to bore-diameter ratio of equal to or greater than three to prevent surface disturbances. In addition, the minimum depth of cover must accommodate short- and long-term loads imposed on the pipe. For all methods that use pressurized drilling fluid or grout, the minimum depth of cover is determined by establishing the depth required to maintain the required factor of safety against hydraulic fracture using the estimated operational drilling pressures.

5.6.3.3. Influence of Mud Pressures. The cavity expansion theory was first developed by Vesic (1972) and then further developed for use in horizontal directional drilling (HDD) projects by researchers at Delft Geotechnics, Luger and Hergarden (1988). The theory is that as the annular fluid pressure increases, the borehole radius will expand. Initially the deformation will be elastic, but as the pressure increases, the deformation will become plastic. As the zone of the plastic deformation increases to the ground surface, blowout will occur and drilling fluid will flow to the surface, creating a "frac out" or "inadvertent return."

5.6.3.4. Hydraulic Fracture Capacity (Delft Equation). Equation 5-1 through Equation 5-5 were developed by Luger and Hergarden (1988) and have previously been recommended by USACE for estimating the maximum total mud pressure (CPAR, 1998).

Equation 5-1

$$p_{max} = p'_{max} + u$$

Equation 5-2

$$p'_{max} = (p'_f + c \cdot \cot\emptyset) \cdot \left\{ \left(\frac{R_0}{R_{p,max}} \right)^2 + Q \right\}^{\frac{-\sin\emptyset}{1+\sin\emptyset}} - c \cdot \cot\emptyset$$

Equation 5-3

$$p'_{lim} = (p'_f + c \cdot \cot\emptyset) \cdot Q^{\frac{-\sin\emptyset}{1+\sin\emptyset}} - c \cdot \cot\emptyset$$

Equation 5-4

$$p'_f = \sigma'_0(1 + \sin\emptyset) + C \cdot \cos\emptyset$$

Equation 5-5

$$Q = \frac{(\sigma'_0 \cdot \sin\emptyset + c \cdot \cos\emptyset)}{G}$$

Where:

p_{max}	=	maximum total mud pressure (psf)
p'_{max}	=	maximum effective mud pressure (psf)
p_{lim}	=	limiting pressure when $R_{p,max}$ approaches infinity (psf)
p'_f	=	effective mud pressure where plastic deformation initiates (psf)
σ'_0	=	effective confining stress (psf)
c	=	cohesion (psf)
$\emptyset$	=	angle of internal friction (degrees)
Q	=	a function of the shear modulus and effective stress (unit less)
G	=	shear modulus (psf)
R_0	=	initial radius of the borehole (feet)
$R_{p,max}$	=	radius of the plastic zone (feet)
u	=	initial pore water pressure (psf)

5.6.3.5. Use of the Delft Equation**.**

5.6.3.5.1. General. There are several known issues with the Delft equation (Equation 5-2) that should be considered by designers before it is used: There are errors in the published equations in the often-referenced USACE CPAR-98-1. The equations in this document have since been corrected.

- The equation does not account for anisotropic ground stress conditions where vertical and horizontal stresses are not equal ($k_o \neq 1$) or for very shallow bores where tensile failure may dominate.

- Because of its simplifying assumptions, the equation significantly overestimates the maximum allowable pressure of the drilling fluid.

Most practitioners assume that the effective confining stress in the equation is equal to the effective vertical stress. This is not correct since in most subsurface situations the effective vertical stress is not the minor principle stress. Many factors affect the stress conditions in the ground (e.g., historical loadings, desiccation, surface geometry). Additional information on in-situ stress can be found in Schmertmann (1985). Typically, the vertical effective stress, $\acute{\sigma}_v$, can be easily determined, but the determination of the major and minor effective confining stresses and their orientations is more difficult. In the absence of performing advanced soil investigations and stress analyses, it is conservative to assume that the effective confining pressure, $\acute{\sigma}_0$, is equal to the minor principle stress, $\acute{\sigma}_h$, under normally consolidated conditions, as determined by Equation 5-6.

Equation 5-6

$$\acute{\sigma}_0 = \acute{\sigma}_h = (k_0 \cdot \acute{\sigma}_v)$$

Where:

$\acute{\sigma}_v$	=	vertical effective stress (psf); $\gamma \cdot z - u$
u	=	pore pressure (psf)
$\emptyset$	=	drained friction angle of the soil (degrees)
k_0	=	at rest earth pressure coefficient (unit less); $\sim 1 - \sin\emptyset$
γ	=	unit weight of the soil (pcf)
z	=	depth of soil cover over pipe (feet)

5.6.3.5.2. Maximum Plastic Radius, $R_{p,max}$. The radius of the plastic zone $R_{p,max}$ must be selected by the designer. Practitioners concerned about limiting fluid releases to the surface have used 1/2 the distance to the ground surface for clays and 2/3 the distance for sands. These values were selected assuming that no damage would occur outside the zone established by $R_{p,max}$; however, these $R_{p,max}$ values are meant to produce limiting pressures that will prevent drilling fluid surface release, but not necessarily localized hydraulic fracture. Staheli (2010) and Rostami (2017) have both shown the calculation of P_{max} is not sensitive to $R_{p,max}$ beyond a few feet from the borehole and that there is no correlation to the actual fracture pressure. Instead of assuming that plastic deformation is permissible, it is preferred that deformation be limited to the elastic range where specifying $R_{p,max}$ is not required.

5.6.3.5.3. Drained vs. Undrained Conditions. During drilling, the loading from the annular drilling fluid pressure will be drained for pervious materials and undrained for impervious material. The Delft equation includes both cohesion and friction angles which presents the potential to erroneously use both drained and undrained strengths together. For evaluating low permeability soil layers, $\phi'=0$ should be assumed and only the undrained strength (cohesion) should be used. For this condition, the equation for the maximum effective mud pressure reduces to Equation 5-7.

Equation 5-7

$$p'_{max} = \sigma'_0 + c$$

For high permeability soil layers, $c = 0$ should be assumed and only the effective friction angle (ϕ') should be used. For this condition, the maximum effective mud pressure should be limited to equal p'_f, where plastic deformation initiates according to Equation 5-8.

Equation 5-8

$$p'_{max} = p'_f = \sigma'_0 \cdot (1 + \sin \emptyset)$$

To get the maximum total pressure that can be compared to measured pressures, the pore pressure needs to be added, as shown in Equation 5-1.

5.6.3.5.4. Maximum Pressure in Rock. For fractured rock with open joints or karst rock with solution features, the drilling fluid pressures can be easily transmitted through the openings to the overlying soils. For these conditions, the maximum allowable pressures should be determined assuming the properties of the overlying soil and the equations presented above. For solid intact rock, the maximum effective mud pressure can be estimated by assuming the rock behaves as a strong soil and is equal to the sum of the minor effective principle stress plus the tensile strength of the rock, as in Equation 5-9.

Equation 5-9

$$p'_{max} = \sigma'_3 + T$$

Where:

σ'_3 = minor effective principle stress (psf)

T = tensile strength of the rock (psf)

The maximum total mud pressure can be determined by adding the pore pressure to the maximum effective pressure as shown in Equation 5-1. If the calculated maximum total fracture pressure (p_{max}) is greater than the total vertical stress (σ_v), then P_{max} should be set to σ_v in order to prevent the possibility of vertical ground heave.

5.6.3.6. Factor of Safety.

5.6.3.6.1. **Clays and Silt**. The allowable maximum total mud pressure is determined by dividing p_{max} (Equation 5-1) by a minimum factor of safety which must be maintained along the entire borehole length. These limiting pressures must be estimated prior to construction and clearly stated in the project contract documents or in the contractor submittals. Higher factors of safety may be warranted in more complex topography or stratigraphy, or in soft clays. Where the operational mud pressures may exceed the allowable maximum drilling pressure, the pipeline must be set at a deeper elevation or in a higher strength formation to increase the factor of safety. When drilling in clay or silt Equation 5-7 is used to determine p'_{max}. A minimum factor of safety of 1.5 shall then be applied to p_{max} to determine the allowable maximum total mud pressure.

5.6.3.6.2. **Sand.** When drilling in sands the risk of damage from hydraulic fracture may be significantly different than in clays. In sands, a hydraulic fracture will typically not adversely affect the seepage or stability performance of the structure unless there are specific seepage control features such as relief wells or drains that could be blocked by drilling fluid. The focus on sands is to prevent the drilling fluid pressure from damaging an overlying layer that is more critical to the levee or dam performance. This can be done by limiting the allowable maximum total mud pressure to the lower pressure calculated when using Equation 5-2 (using c=0 and $R_{p,max}$ equal to ½ the distance from the bore to the bottom of the clay layer) to determine p'_{max} and applying a factor of safety of 1.5 to p_{max} or the pressure calculated using Equation 5-8 for p'_{max} and applying a factor of safety of 1.0 to p_{max}. Equation 5-2 is intended to limit plastic deformation to the zone defined by $R_{p,max}$ while Equation 5-8 limits all deformation to the elastic range.

5.6.3.7. Monitoring Drilling Fluid Pressures. Real-time continuous monitoring of the actual total borehole mud pressures must be performed during drilling and the drill operator must keep the annular pressures low enough to maintain the required factor of safety at all times. For some methods (e.g., HDD and direct pipe), the drilling fluid may exert pressure on the soils all the way back to the return exit. Simply comparing the measured pressures to the allowable maximum mud pressure at the drill bit location for these methods is insufficient to prevent hydraulic fracture. If the drill head enters a stronger material, the allowable maximum pressure of weaker material along the drill path may still control, and pressures may not be able to be increased in the stronger material.

5.6.3.8. Speed and Continuity of Drilling.

5.6.3.8.1. General. Based on methods presented in the North American Society for Trenchless Technology's (NASTT) Good Practices Guidelines, the rate of advancement (drilling) must be controlled in order to maintain line and grade; and for HDD methods to reduce the potential for pressure buildup in the annular space due to clogging of the return fluid path with solids. For HDD, maintaining good circulation and low annular pressures is best accomplished by pumping adequate volume of drilling fluid, based on soil characteristics and volume of excavated soil, for any given reach. Drilling fluid flow factors may range from 3 to 5, or higher for high plasticity, highly consolidated clays. Flow factors can be lower for non-cohesive soils. The contractor should estimate the maximum penetration rate over the range of possible drilling fluid pumping rates anticipated during installation and manage drilling speeds to not exceed this limit. The penetration rate should be fast enough to prevent over-reaming of the hole.

5.6.3.8.2. Method for Estimating Penetration Rate of Bore Based on Achieving Good Bore Cleaning.

Step 1. Determine volume of solids to be removed for the next pipe section. For example, assume a 31-foot drill pipe section, reaming a 12-inch diameter bore to a 24-inch diameter bore.

$$\begin{aligned}\text{Volume of Solids} &= [\text{Area of 24-in. diam. bore} - \text{Area of 12-in. diam. bore}] \times \text{Pipe Segment Length} \\ &= [(\pi \cdot r^2)_{outer} - (\pi \cdot r^2)_{inner}] \times \text{Pipe Segment Length} \\ &= [(3.1416 \times 1)\text{ft}^2 - (3.1416 \times 0.25)\text{ft}^2] \times 31\text{-ft} = 73.0\ \text{ft}^3 \text{ of solids}\end{aligned}$$

Step 2. Determine volume of drilling fluid required to be introduced during installation of pipe section to achieve good bore cleaning. Assume a flow factor of 5 for this calculation.

$$\begin{aligned}\text{Volume of Drilling Fluid} &= \text{Volume of Solids} \times \text{Flow Factor} \\ &= 73.0\ \text{ft}^3 \times 5 = 365\ \text{ft}^3 \text{ (or 2,730 gal for 31 feet)}\end{aligned}$$

Step 3. Determine maximum penetration rate, based on actual flow rate of drilling fluid achievable with pumping system, for pipe section. Assume flow rate is 250 gpm for this calculation.

$$\begin{aligned}\text{Penetration Rate} &\leq \text{Flow rate / (Volume of Fluid / Pipe Segment Length)} \\ &\leq 250\ \text{gpm / (2,730 gal / 31ft)} = 2.84 \text{ feet per minute}\end{aligned}$$

5.6.4. Seepage Mitigation.

5.6.4.1. General. Seepage evaluations must be considered on a case-by-case basis because seepage characteristics are highly dependent on soil/rock properties and geometry. Trenchless installations require excavations at the entry and exit areas in addition to an “overcut” or annular space between the pipe and the soil. These techniques can result in surficial disturbances or unfiltered seepage pathways along the pipe and at the entry/exit areas if left untreated. In order to address these seepage concerns, the following requirements and defensive measures must be followed and applied.

5.6.4.2. Geotechnical Explorations. Vertical borings advanced from the ground surface must comply with the requirements in ER 1110-1-1807 and be offset at least 25 feet from the pipe centerline to reduce the risk of creating a connection (seepage path) to the ground surface if improperly backfilled.

5.6.4.3. Requirements for Trenchless Advancement within Levees. As a defensive measure against undesirable seepage that could occur through an imperfectly-grouted annular space, gravity pipes placed within (through) earthen embankments by trenchless methods must incorporate a landside external seepage filter (reference Section 5.5.9.4.) and a waterside buttress, providing at least three feet of thickness (Figure 5-42). Installation methods through an embankment do not require a setback distance; however, the phreatic surface must be below the planned horizontal bore.

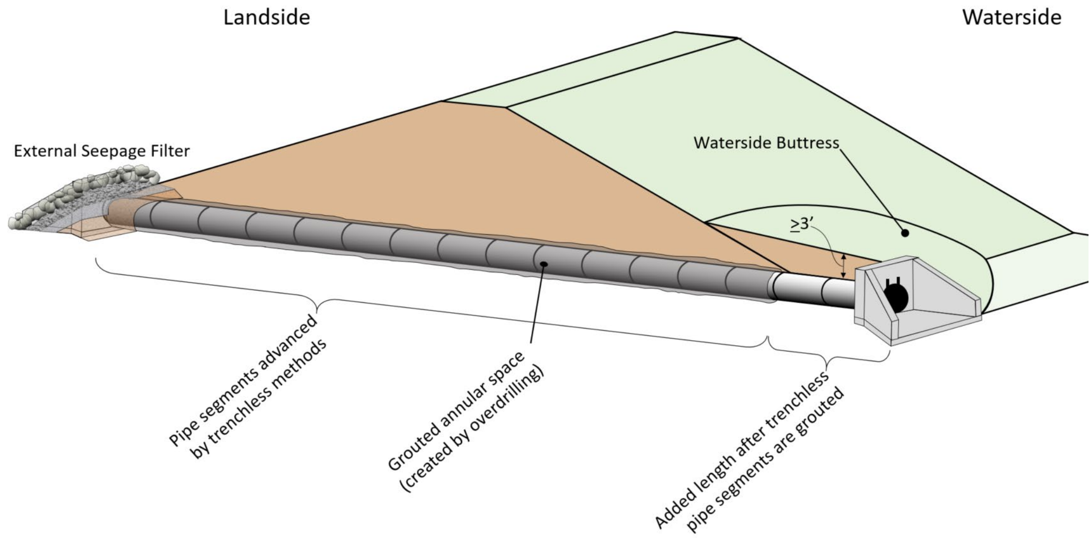


Figure 5-42. Waterside buttress required for trenchless pipe installation within embankment.

5.6.4.4. Requirements for Trenchless Advancement beneath Levees. For installations beneath embankments, drilling and grouting fluid pressures often exceed the maximum allowable drilling fluid pressures near the entry and exit locations due to their shallow depth. As such, the entry and exit pits must be located at the greater of 300 feet or 20 times the embankment height from the levee centerline to ensure the horizontal gradient is low enough to protect against the progression of backwards-erosion piping should the vertical gradient allow initiation. In addition, the effective vertical stress must provide a factor of safety of at least 1.6 against heave/uplift (Equation 5-10).

5.6.4.5. Special Considerations for Trenchless Installations. Trenchless installations are not permitted through zoned earthen embankments or seepage mitigation features (e.g., cutoff walls, seepage blankets, relief wells, internal filters/blanket drains). If any seepage mitigation feature is above or beside the proposed trenchless pipe alignment, the designer must ensure the advancement technique will not impact the feature.

5.6.4.6. Excavations at Entry and Exit Areas. Excavations for trenchless installations must be designed, excavated, and maintained in a stable condition. Seepage into excavations must be controlled to prevent erosion of soils, which may require dewatering well points, dewatering wells, or other methods as necessary. Entry and exit areas must be backfilled with a low-permeability cohesive soil or CLSM placed around the pipe to prevent a preferred seepage path. The designer must also consider the need for emergency closure of the entry or exit areas during high water events.

5.6.4.7. Annulus Grouting. For HDD and direct pipe installations, grouting the near surface sections of the annulus around the pipeline must be performed to reduce the likelihood of a preferential seepage path; however, grouting pressures must be carefully controlled to minimize risks of hydraulic fracture. On both ends, grouting must be performed to the bottom of the cohesive blanket overlying a pervious layer or until a minimum factor of safety of 1.6 (Equation 5-10) is achieved. A higher factor of safety may be warranted in some situations (e.g., complex topography or subsurface stratigraphy). The grout must be introduced according to ER 1110-1-

1807 and the quantity used equivalent to the theoretical overcut volume. Additional grouting must be performed where field volumes are less than theoretical.

Equation 5-10

$$FS = \frac{h_s \gamma'}{H_w \gamma_w} \geq 1.6$$

Where:

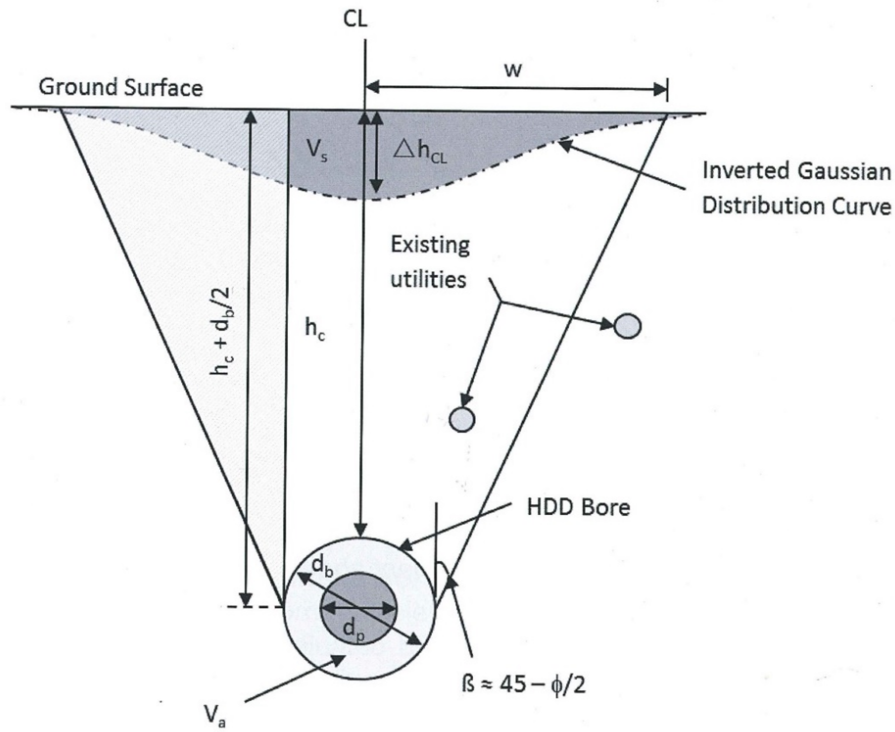
h_s = grout depth (feet)
γ' = effective or buoyant unit weight of grout
H_w = Height of excess head above the bottom of the grout (feet) or head difference between flood elevation and bottom of grout (feet)
γ_w = unit weight of water (pcf)

5.6.5. Settlement Requirements.

5.6.5.1. General. Surface settlement may occur due to over-excavation or collapse of the overcut between the pipe and the soil. Settlement must be evaluated so that reasonable limits can be placed on trenchless installations methods (e.g., depth of bore, spoil removal, maximum annular overcut). Settlement risk is generally higher for bores or portions of bores which are not filled with drilling fluids; therefore, lubrication fluids must be used. For pipe ramming installations, spoil must not be removed within 50 feet of the leading edge of the pipe.

5.6.5.2. Settlement Tolerances. Maximum tolerable settlement will depend on site-specific conditions and must be approved by the respective USACE District prior to pipe installation. The maximum foundation settlement due to trenchless installations should be one inch for typical levee embankments; however, this may be more stringent if there are nearby settlement sensitive structures (e.g., interstates, bridges, buildings). For floodwalls, the maximum settlement should be one-half inch with no more than one-quarter inch differential settlement between adjacent floodwall monoliths. Rotational displacements of the floodwall due to trenchless installations must be less than one-half inch, as measured at the top of the wall.

5.6.5.3. Estimating Settlement. Surficial settlement due to a trenchless pipe installation may be evaluated using the methods outlined in NASST's Horizontal Directional Drilling, Good Practices Guidelines, as summarized here. This approach models the shape of settlement above a bore as a trough approximated by an inverted Gaussian distribution curve. This curve is a normal probability distribution curve modified by assuming that the limits of the settlement trough are defined within the lines, beginning at the springline on both sides of the bore and ending at the ground surface, inclined at opposing vertical angles of 45-ϕ/2 degrees (Figure 5-43). The maximum settlement above the bore centerline, Δh_{CL}, is estimated using Equation 5-11 and Equation 5-12, which assumes a percentage of the bore's annular volume is translated to the surface ($V_s = V_a \cdot C_f$), with the correction factor (C_f) being based on the soil or rock conditions. For stiff to hard clays, cohesive sands, rock, and very dense or cemented sands, $C_f \approx$ 0 to 0.25. For medium clay and medium dense to dense sand, $C_f \approx 0.50$. For soft clay and loose sand, $C_f \approx 0.75$. For very soft, squeezing clays, and very loose sands, the settlement trough volume correction factor may approach 1.0, but can be greater than 1.0 due to consolidation of soil under tooling.



(Courtesy of Bennett Trenchless Engineers)

Figure 5-43. Graphical representation of settlement trough above HDD bore.

Equation 5-11

$$\Delta h_{CL} = \frac{V_S}{w}$$

Equation 5-12

$$w = \frac{d_b}{2} + \left(h_c + \frac{d_b}{2}\right) \cdot \tan(45 - \frac{\emptyset}{2})$$

Where:

- CL = Centerline of pipe
- w = Settlement trough half width
- Δh_{CL} = Settlement trough depth at centerline (maximum settlement)
- V_a = Volume of the annulus per unit length of bore length = $\pi/4 \cdot (d_b^2 - d_p^2)$
- C_f = Correction factor based on soil type
- V_S = Settlement trough volume per unit of bore length (V_a x C_f)
- h_c = Depth of clearance above crown of bore (not the pipe)
- d_b = Diameter of the bore
- d_p = Diameter of the product pipe
- $\emptyset$ = Drained friction angle of the soil
- β = 45−ϕ/2, empirical estimated angle from bore springline to ground surface which defines settlement trough width.

Two factors can affect the percentage of annular volume that is translated to the surface: soil mass loosening and soil strength (arching). To account for these effects, a correction factor is applied to the annular volume, V_a. For stiff to hard clay, cohesive sands, rock, and very dense or cemented sands, the volume is typically reduced by 25 percent or less. For medium dense to dense sand, and medium clay, the volume is typically reduced by 50 percent or less. For very soft, squeezing clays and very loose sands, the settlement trough volume may approach 100 percent of the annular volume; however, it can be greater than 100 percent due to consolidation of soil under tooling. The percentage of annular volume that may contribute to settlement trough volume will increase as soil consistency decreases.

5.6.6. Problematic Subsurface Conditions.

5.6.6.1. General. Problematic subsurface conditions may be encountered during trenchless installations. A subsurface exploration program should be used to assess installation impacts due to problematic subsurface conditions and determine the appropriate installation method. Some of the more common problematic soil conditions are described as follows.

5.6.6.2. Raveling Ground. Raveling ground consists of poorly consolidated or cemented materials that can stand up for several minutes to several hours after being cut, but then begin to slough, slake, or scale off. Residual soils or sands with small amounts of binder may be fast raveling below the water table but slow raveling above. Stiff-fissured clays may be slow or fast raveling, depending upon the degree of overstress. Raveling ground conditions can lead to over-excavation for certain trenchless methods. Site investigations needed for identifying raveling ground include sieve analysis, undrained shear strength tests, standard penetration tests, stress states, and groundwater elevation measurements (CPAR, 1995).

5.6.6.3. Running Ground. Clean, dry, cohesionless materials are typically classified as running ground and will run like granulated sugar until the slope flattens to its angle of repose. Apparent cohesion in moist sand, or weak cementation in any granular soil, may allow the material to stand for a brief period before eventually raveling. Such behavior is called cohesive running. Pipe ramming, open-shield pipe jacking, and horizontal auger boring methods are not suitable for running ground conditions as they can lead to over-excavation. Site investigations necessary for determining running ground conditions include sieve analysis and tests to determine soil classification.

5.6.6.4. Flowing Ground. Soils without enough clay content to provide adequate cohesion to prevent flowing when present below the water table include silts, sands, and gravels. Ground flow may also occur in highly sensitive clays when disturbed. Sensitive clays are those soils with lower strengths in remolded condition compared to undisturbed conditions. Soil sensitivity (S_t) is defined as the ratio of undisturbed to remolded undrained shear strengths: $S_t = S_u$ undisturbed / S_u remolded. Information needed to determine the potential for flowing ground includes sieve analysis, Atterberg limits, natural moisture content, sensitivity of clays, vane shear tests, and SPT blow counts on undisturbed and remolded specimens (CPAR, 1995). Pipe ramming, open-shield pipe jacking, and horizontal auger boring methods are not allowed in cohesionless soils below the water table.

5.6.6.5. Squeezing Ground. Squeezing ground extrudes plastically into the bore without visible fracturing or loss of continuity and without a noticeable increase in water content. The nature of squeezing is a ductile, plastic yield and flow due to overstress, typically in ground with low frictional strength. The rate of squeeze depends on the degree of overstress. Squeezing can occur in clays having a soft to medium consistency. Stiff to hard consistency clays under high overburden stresses may move into the excavation through a combination of raveling at the excavation surface and squeezing at some distance behind the free surface. Typical tests and information used for determining squeezing properties include undrained shear strength, consolidation tests, SPT blow counts, and stress state (CPAR, 1995).

5.6.6.6. Swelling Ground. Swelling ground increases in volume as it absorbs water during boring and expands slowly into the bore. Swelling soils include highly pre-consolidated clays with plasticity indexes over about 30, generally containing significant percentages of montmorillonite. When these soils are identified, it is critical to have a large overcut to allow the swelling to occur without binding the pipe. Water with polymer and/or bentonite additives should be used in the slurry make-up tank in order to reduce the loss of water into the clay, and therefore avoid swelling potential due to clay's high affinity for adsorption. Site investigation data that are useful for evaluating swelling potential include soil classification, Atterberg limits, clay mineralogy, soil suction information, and consolidation test results (CPAR, 1995).

5.6.6.7. Contaminated Groundwater or Soil. The potential for contaminants must be identified during site investigation to address environmental requirements. Proper soil and groundwater sampling and handling procedures should be followed during test drilling to ensure accurate results, especially if volatile organic compounds are anticipated. The site investigation should be extensive in areas of potential contamination (e.g., areas near gas stations, chemical plants, or buried fuel tanks) (CPAR, 1995).

5.6.6.8. Mixed-Face Conditions. Because of the serious potential for line and grade deviations and stability problems, the borehole alignment should avoid mixed-face conditions if possible (CPAR, 1995). Mixed-face conditions have distinct variations in material properties and behavior within the cross-sectional area of the borehole and should be avoided as they can present challenges to alignment, grade control, and stability of the face. For example, a borehole advancing through an interface of soft ground overlying rock would tend to force the drilling face into the softer material. Geophysical surveys complemented with borings can provide reliable indications of the top of the rock or hard layers.

5.6.6.9. Cobbles and Boulders. Pipe ramming and horizontal auger boring methods are not considered suitable for ground conditions that that are predominately comprised of cobbles and boulders with little soil. In addition, these methods must not be used where cobbles or boulders greater than 0.3 times the casing diameter are present. To properly assess the risk that cobbles and boulders present to drilling, a subsurface investigation must be tailored to measure cobble and boulder volume ratios, sizes, and properties (e.g., unconfined compressive strength, abrasivity). This may require rock coring or large volume samples (e.g., test pits, deep excavations) and the determination of soil matrix properties (e.g., grain size, cohesion, strength, density, permeability, abrasiveness). Local knowledge of site geology is also important since cobbles and boulders cannot be easily sampled at great depths (Hunt, 2017).

5.6.6.10. Rock. Vertical joints and fractures and zones of soft and hard rock may facilitate hydraulic fracture due to the high fluid pressures required in more competent zones, which may also be inadvertently used in the less competent zones. Problems may also arise from large voids related to solutioning processes (e.g., karst openings in carbonate formations) or thinly bedded shales where fluid is lost to the formation through pre-existing fractures and voids. This fluid loss mechanism is referred to as "formational fluid loss" and occurs at pressures below those that would initiate hydraulic fracture. Swelling shales can pose similar issues to swelling soils, and laminated shales with weak planes can exhibit significantly less capacity than suggested by cavity expansion theory (Equation 5-1). Pipe ramming is not suitable where refusal conditions are anticipated or if the entire pipe perimeter may be engaged with solid rock.

5.6.7. Work Plan Requirements.

5.6.7.1. General. A comprehensive evaluation and/or technical review is required of the work plan and associated construction documents based on alterations to USACE Civil Works Projects per 33 USC 408. At a minimum, the work plan must be in full compliance with the drilling program plan (DPP) referenced in ER 1110-2-1807.

5.6.7.2. Contractor Qualifications. The contractor should provide a list of projects where trenchless installations related to dams and levees have been successfully completed by the company. Other information that should be provided is a list of key personnel with resumes, such as a driller who has successfully completed at least two trenchless installations in the past five years of similar size, type, location, project environment (e.g., urban work, river crossing), pipeline diameter, and length of installation.

5.6.7.3. Entry and Exit Pit/Shaft Design. Excavation of entry and exit areas must include stability and seepage evaluations that indicate compliance with the requirements outlined in EM 1110-1-1913 and EM 1110-2-1902. Temporary shoring is not permitted within USACE dam or levee embankments or their respective foundations. Pit and shaft designs must be able to resist forces from pipe installation, disturbance of utilities behind thrust blocks, and weight of installation equipment.

5.6.7.4. Drill Path Design. Drill path design must include the planned pipe alignment, location of entry and exit points or excavations, results of the subsurface investigation, settlement evaluation of the embankment or floodwall, hydraulic fracture evaluation and annular grouting plan (reference Appendix E), drilling fluid design, pre-construction surveys and project stationing, required minimum clearances from existing structures, right of way lines, and diameter of the trenchless bore and drill pipe. Geotechnical borings must comply with ER 1110-1-1807 and penetrate to an elevation at least 30 feet below the depth of the proposed drill path to provide information for design modifications and anticipated deviations during construction. Pipe design calculations must include installation and long-term service loads, pipe material, pipe thickness, joint design or connections, and corrosion/protective coatings.

5.6.7.5. Pipe Locating and Tracking. Pipe locating and tracking is required when drilling within or beneath a USACE embankment or floodwall. Risk and design parameters will dictate the accuracy required for the bore.

5.6.7.6. Contingency and Decommissioning Plan. Documented procedures to address inadvertent fluid losses (hydraulic fracture), a high water event during drilling operations (reference Section 5.3.4.) and decommissioning (in the form of a decommissioning plan) must be completed prior to initiation of work. If nighttime drilling activities are necessary, lighting along the pipe alignment must be provided so that inadvertent surface releases can be observed. If a drill hole beneath a levee must be decommissioned, the hole must be completely backfilled with grout to prevent future subsidence. Pipes installed by trenchless methods must incorporate shutoff valves as outlined in Section 5.7.6.

5.6.7.7. Requirements for Protecting Existing Structures and Site Features. The work plan must include procedures for preservation of existing property from damage and for employing construction methods that will not produce damaging vibrations, soil movement, soil loss, or instability of existing structures.

5.6.7.8. Construction Methods and Equipment. The work plan should include the schedule of activities associated with drilling fluid and spoil management, the boring procedure, the size of the working/layout areas, traffic control requirements, and a safety plan.

5.6.7.9. Quality Control and Quality Assurance. A USACE representative and/or independent construction specialist must be provided full-time access to the drill cab and all instruments to adequately observe and document the drilling parameters and downhole annular pressures. Submittal requirements should include daily logs of pressure measurements and cutting head locations at frequent intervals. Several types of tests may be required or desirable for waterlines, including air hydrostatic tests of each joint (e.g., if pipes are fuse-joined), and leakage test of the assembled pipeline sections to the applicable ASTM standard (Table 5-4).

5.6.7.10. Ground Movement Monitoring. A detailed plan for monitoring possible ground surface movements caused by trenchless pipe installations is required. This requirement may be waived by the respective USACE District if the depth of drilling is sufficiently deep and measurable surface settlements are not anticipated. The plan should address the equipment, method, and frequency of survey measurements and special monitoring requirements. A baseline survey of the ground surface above the pipe alignment must be performed before the start of construction to establish pre-existing conditions. In addition, existing conditions of potentially affected structures, utilities, and facilities must be documented. The plan should include frequency and formatting of reports as well as action levels and contingency plans (ASCE 36-15, 2015). Volumetric monitoring of excavated spoils must be continuously performed to reduce the likelihood of over-excavation that could lead to surface deformation.

5.6.7.11. Site Restoration. The work plan must include a method to restore surfaces affected by the work to their preconstruction condition.

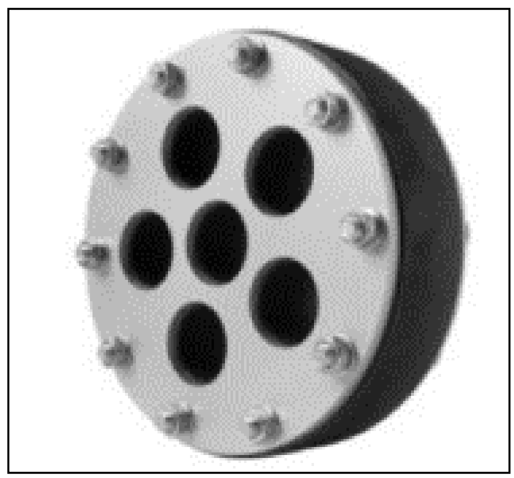
5.6.7.12. Post Construction Documentation. The work plan must outline the post-construction documents that will be provided to USACE upon completion of the project. As-built drawings including both alignment and profile must be provided upon completion of trenchless installations. Drawings must be constructed from actual field conditions using raw data. Post construction documentation must include the date, start, and finish times of pipe installation, volume of spoils removed, number of pipes installed and distances bored, lubrication and grout quantities, and other important information (e.g., settlement).

5.7. Methods for Crossing Floodwalls and Closure Sills.

5.7.1. General. Pipe installations may be required through new floodwalls or closure sills (EM 1110-2-2502), or as approved alterations of existing floodwalls or sills per guidance in 33 USC 408. To prevent leaving a direct connection between the waterside and landside, the annular space between the casing and the carrier pipe must be sealed by either mechanical seals or grouting. Mechanical seals are recommended over grouting to ensure easier future removal of the carrier. An example of an annular seal is shown in Figure 5-44. Multi-pipe seals accommodate two or more pipes through the same circular opening with all spaces between the pipes completely sealed (Figure 5-45). The use of two seals is preferred, but if only one seal is used it must be placed on the waterside.

From SPEC-NET, 2017)

Figure 5-45. Casing with a surface-mounted flange and annular seal on existing floodwall.



(Courtesy of Westatlantic Tech Corp.)

Figure 5-44. Typical annular end seal for multiple pipes

5.7.2. New Floodwalls and Closure Sills. At locations in new floodwalls where pipes are anticipated to pass, installing a casing pipe with a centerline flange (Figure 5-46) or a well-formed sacrificial blockout before the concrete is poured should provide a hole smooth enough that a mechanical annular seal can be used to provide a tight seal. If the blockout does not provide a sufficiently smooth surface, a casing pipe must be grouted in place (consult EM 1110-2-2502 for reinforcing bar details). It is recommended that a casing pipe be placed within the form work before casting the concrete for a new closure sill in lengthy structures. For existing pipes passing through a new floodwall stem or closure sill, temporarily remove enough sections of the pipe to construct the stem or sill with a continuous casing if possible, or use a gasketed split casing (Figure 5-47) around the pipe if it cannot be temporarily removed.

(Courtesy of GPT Industries)

Figure 5-46. Pipe casing with a centerline flange, designed for cast-in-place applications.

(Courtesy of Ironhed LLC)

Figure 5-47. Split pipe casings (gaskets are not shown).

5.7.3. Order of Preference for Pipes Crossing Existing Floodwalls. When it is necessary for a pipe to cross an existing floodwall, the respective USACE District must determine the situational appropriateness for reducing risk when approving a pipe crossing alteration per guidance in 33 USC 408. Generally, the likelihood of endangering the system increases as the crossing gets deeper and the excavation gets closer to the wall. The most desirable crossing scenario would be a pipe routed over the floodwall without contact, but anchor straps may be required and are acceptable assuming their installation does not damage the reinforcing members (❶ in Figure 5-48). This typically requires inconsequential shallow trenching above the floodwall footing to reach the wall before daylighting. If routing the pipe over the floodwall is not possible, the next most desirable location is through the floodwall above the footing (❷ in Figure 5-48). The risk is mitigated by running the carrier pipe through a casing pipe and sealing the ends; an example is shown as Figure 5-47. When it is necessary to locate the pipe beneath the floodwall, the entry and exit pits should be located outside the influence zone, and a casing pipe with end seals installed between the pits (❸ in Figure 5-49). If the floodwall foundation includes a deep concrete or sheetpile seepage barrier, the riverside excavation will have to penetrate the influence zone to expose it (❹ in Figure 5-50). It is recommended that this type of installation include a riverside concrete collar as described in Section 5.7.5.2. This is the least desirable scenario because it requires removing supporting soil, so proper backfilling of the excavation is critical.

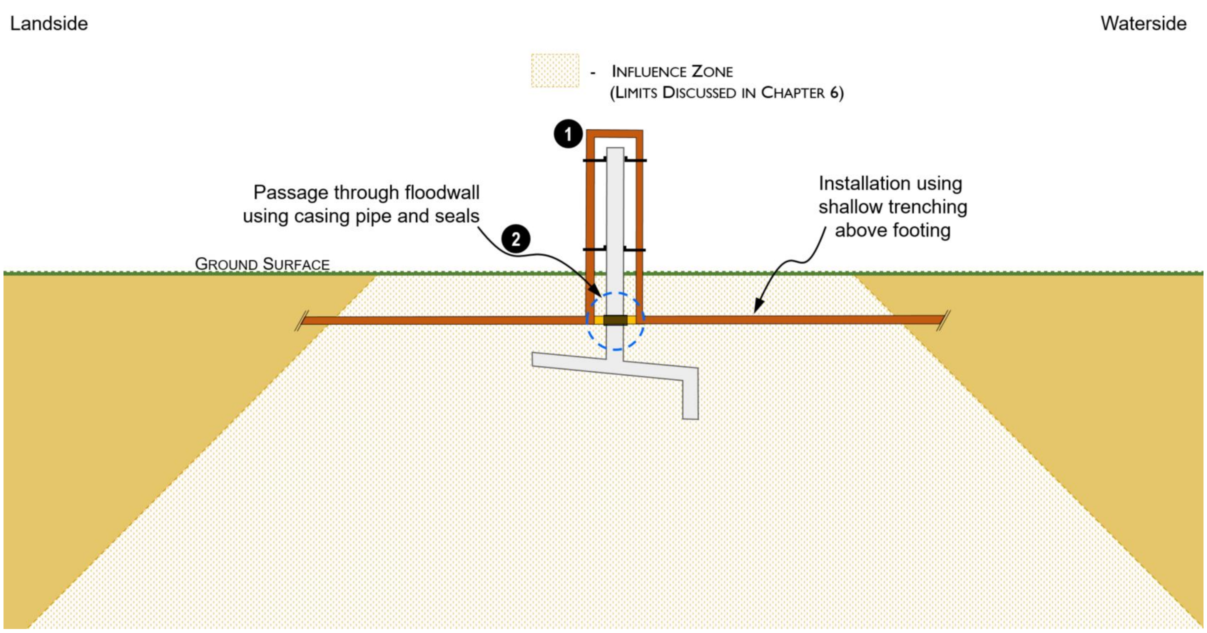


Figure 5-48. First and second most preferred pipe crossing scenarios.

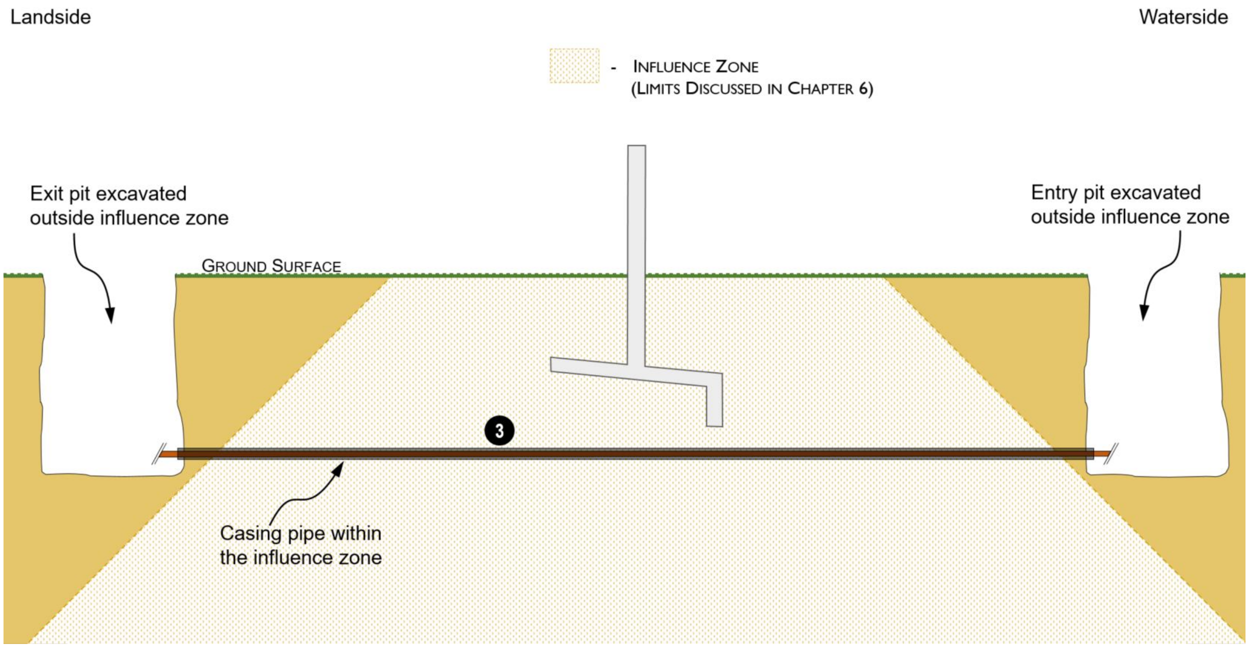


Figure 5-49. Third most preferred pipe crossing scenario.

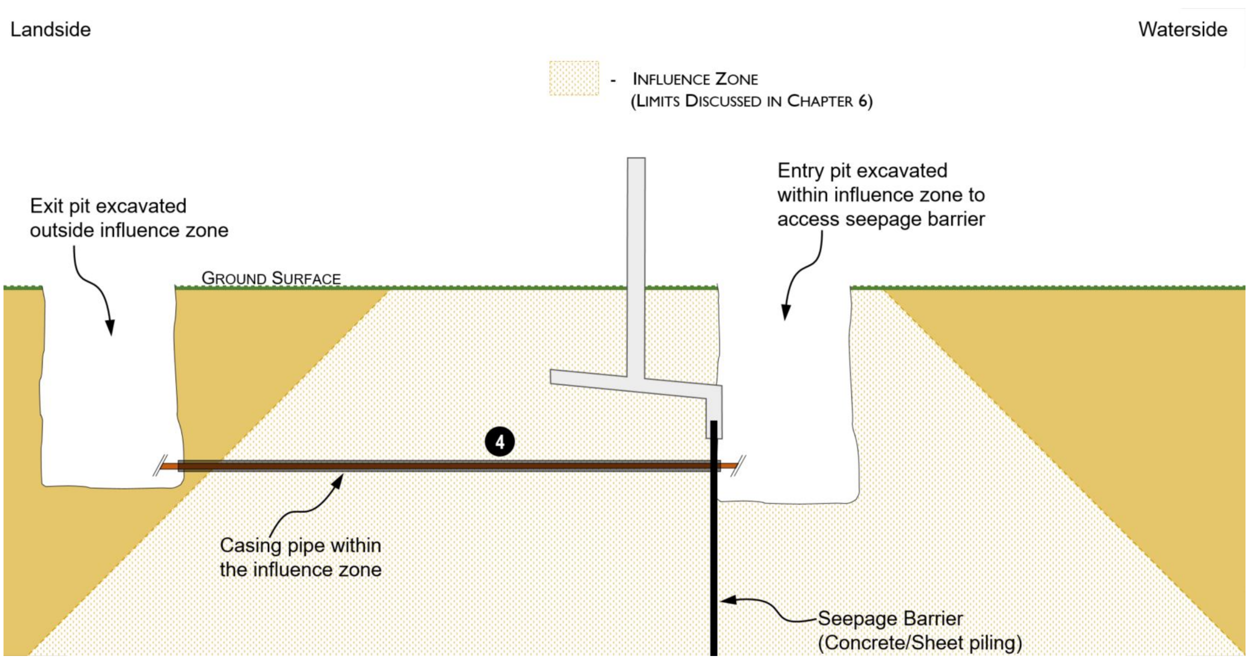
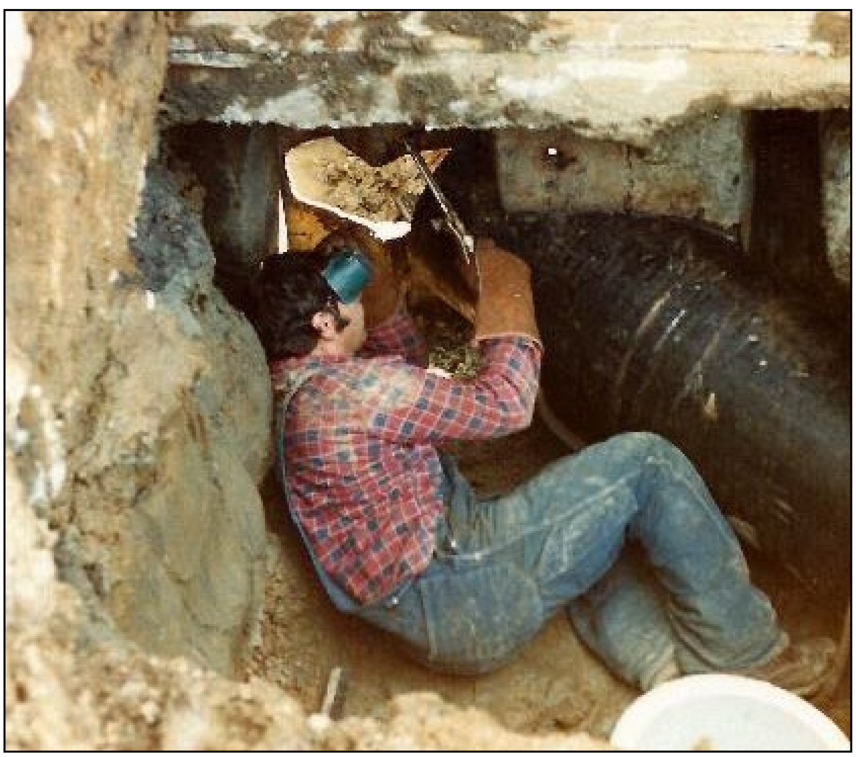


Figure 5-50. Least preferred pipe crossing scenario.

5.7.4. Existing Floodwalls. Casings are not required when a smooth hole is cored from the waterside (critical side) to the landside with no breakouts such that a mechanical seal will make full contact and provide a tight seal. Removing the concrete by rotary percussion tools or jackhammering is not recommended because they leave a rough/irregular opening in which a casing pipe must be installed. If the proposed pipe is greater than eight inches in diameter and requires cutting two adjacent primary (tension) reinforcement elements in the existing floodwall stem, a structural modification to the floodwall may be required. During the 33 USC 408 review/approval process, the respective USACE District office will determine if a structural modification of the stem is required. Possible modifications include installing a structural buttress or counterfort to the wall or structural bridging across the hole that redistributes the bending moments and shear. The installation also requires mounting an external flanged casing on the concrete surface with either an elastomeric sealant or grout between the flange and the wall face (Figure 5-41). Concrete saddles should be anchored in the floodwall base using adhesive anchors to absorb any thrust forces if the hole is near the stem base.

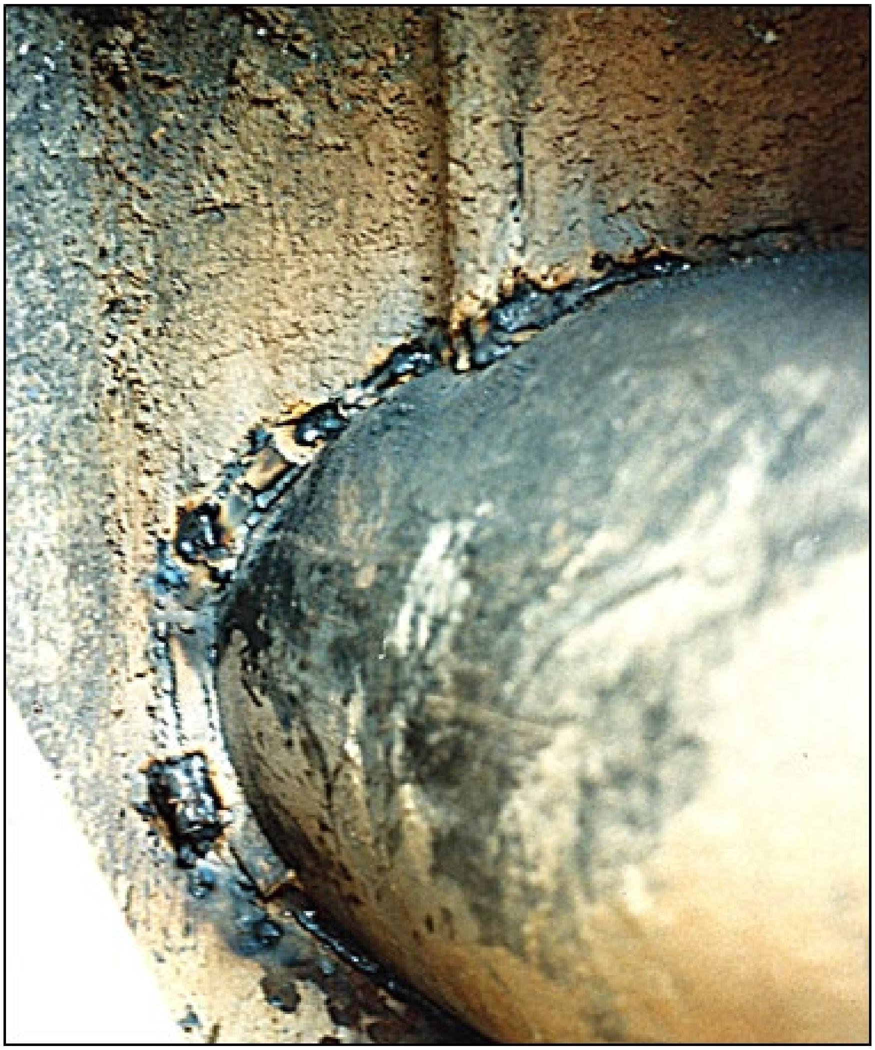
5.7.5. Penetrations through Floodwall Sheet Piling.

5.7.5.1. General. Sheet piling associated with new or existing floodwalls, often used as a seepage cutoff, is typically constructed with hot-rolled Z-shaped structural sheet piling that form angular corrugations. Therefore, openings cut through the sheet piling to pass a pipe will likely encounter surfaces oriented in more than one direction, unless the pipe is small enough to fit between the corrugations. For existing sheet piling, it may be difficult to properly weld the casing pipe to the rough, non-circular opening (Figure 5-51 and Figure 5-52). This may not be the case for new construction where the sheet piling penetration can be made under more ideal conditions.



(Courtesy of USACE Louisville District)

Figure 5-51. Welder cutting a hole in I-wall sheet piling for pipe passage.



(Courtesy of USACE Louisville District)

Figure 5-52. Casing pipe through rough opening in floodwall sheet piling.

5.7.5.2. Casing Pipe to Sheet Pile Connection. When passing pipes less than 10 inches in diameter, the pipe should be aligned such that its centerline falls at the sheet pile interlock. In this scenario, a nearly circular opening can be torch-cut to allow a close fit with the steel casing pipe. The casing pipe should be welded to the sheet pile according to American Welding Society's AWS D1.1. For carrier pipes 10 inches in diameter or larger, the casing pipe should be welded to the sheet pile and the connection encased in a reinforced concrete collar to account for the potential lack of quality in the welding due to the rough cut and difficult working conditions (Figure 5-51). The collar should be located on the waterside and fully encompass the carrier pile to sheet pile connection. It should also be bonded to the sheet pile utilizing headed concrete anchors as shown in (Figure 5-53). An annular seal must be installed between the steel casing and the carrier pipe on both sides of the penetration.

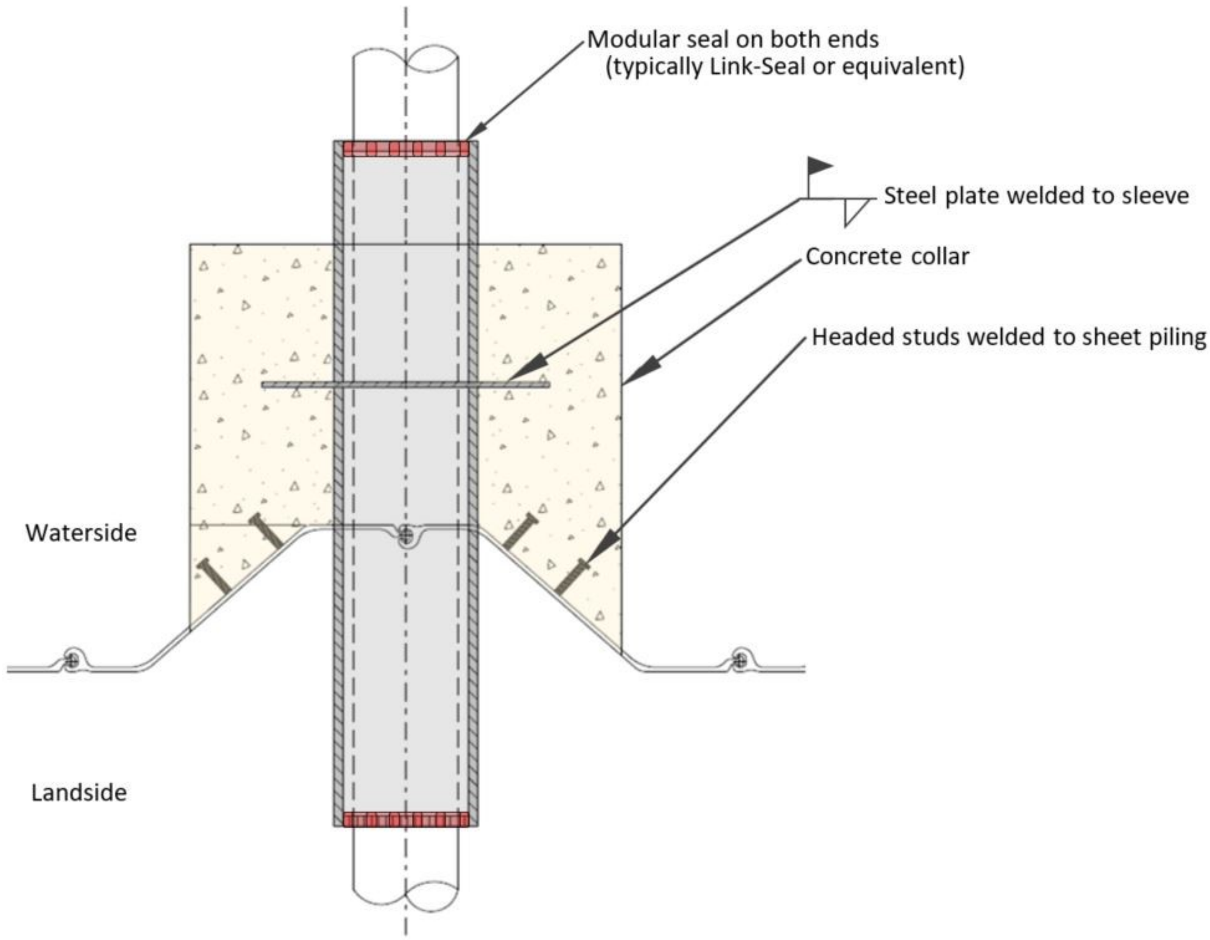
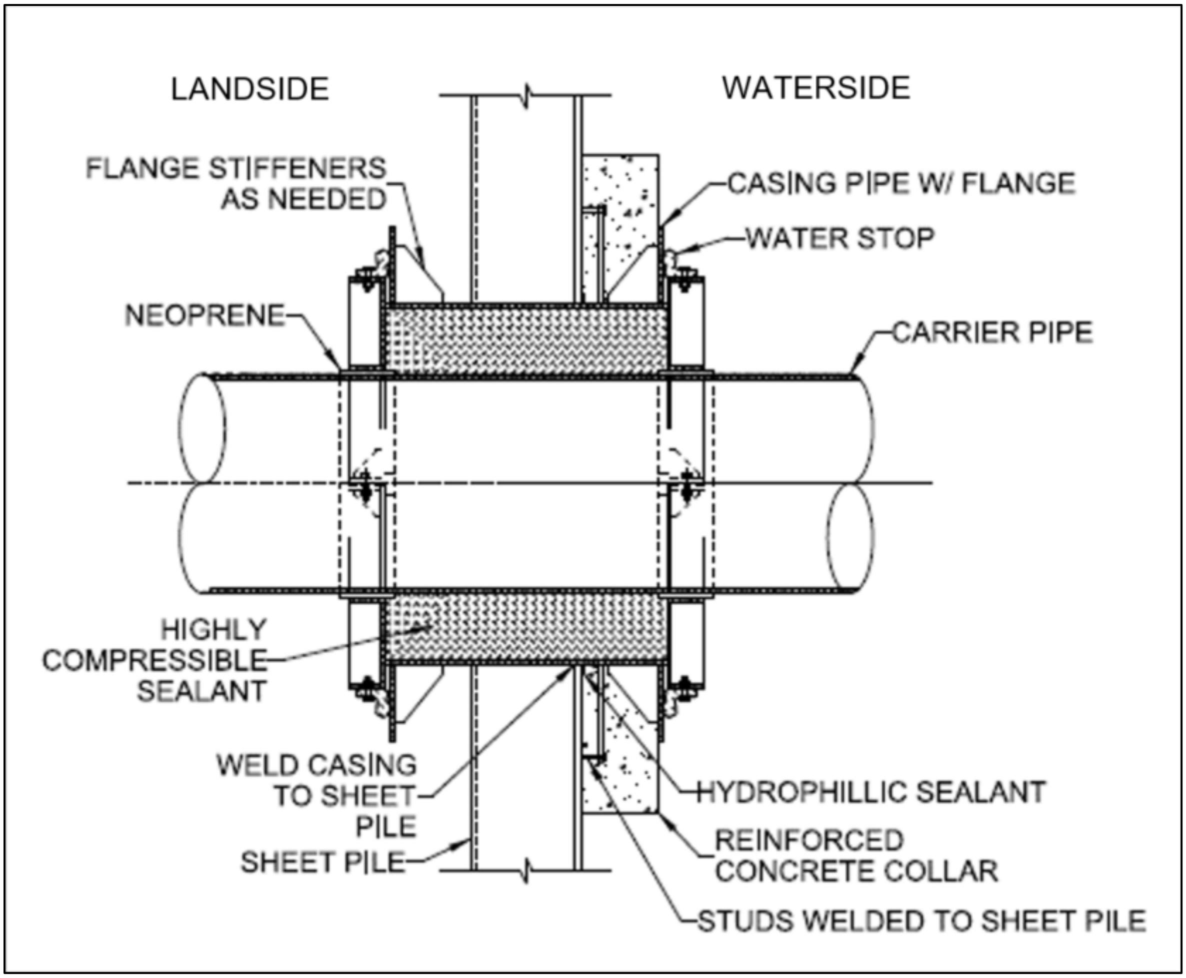


Figure 5-53. Plan view of concrete collar detail for pipes through sheet piling.

5.7.5.3. Differential Settlement. Pipe penetrations through the floodwall sheet pile must be able to resist the possibility of differential settlement between the floodwall and the pipe. This is typically common in pile-founded floodwalls where the piles support the floodwall, but the surrounding soil and pipe settle over time. If differential movement is anticipated, L-Type waterstops should be used instead of annular seals (Figure 5-54). This permits the carrier pipe to slide vertically within the casing, whereas annular seals are tightened to compress against the walls between the two, thereby restricting differential movement. Lateral movement is restricted by welding flanges to the casing pipe, which also serve as a sealing face for the waterstop. The waterstop must be positioned so it is pre-compressed on both sides of the barrier by a minimum of 1/8-inch or according to the manufacturer's recommended preset. The annular space must also be sized to accommodate the differential movement.



(Courtesy of USACE New Orleans District)

Figure 5-54. Pipe through sheet piling using L-type waterstops for differential movement.

5.7.6. Existing Closure Sills. Casings for new pipes through existing sills must be installed using one of two methods. Method 1: Saw cut to the depth required and install the casing from the ground surface (Figure 5-55), after which the reinforcing bars must be repaired and the concrete replaced. Method 2: Core a hole through the sill from excavated pits adjacent to the sill, using the cored hole as the casing. This method typically requires a specialty contractor, but generally results in a shorter shutdown of the roadway crossing.

(Courtesy of USACE Louisville District)
Figure 5-55. New pipe installation through existing closure sill.

5.7.7. Pipe Shutoff Valves. To prevent problems associated with a pressure pipe placed within or beneath an embankment, through a floodwall stem/key/sheet piling, or beneath a floodwall or closure sill, shutoff valves must be installed to stop the pipe flow. If the pipe flow is from landside to waterside, a shutoff valve must be installed between 15 and 50 feet of the landside toe; for flow in the opposite direction, the shutoff valve must be installed outside the influence zone but no more than 50 feet from the landside toe or face of wall. This requirement is not applicable to coastal levees because the probability of needing to operate a shutoff valve during a short-term storm surge event is remote. The respective USACE District may deviate from this distance requirement, as necessary. A secondary valve is recommended on the opposite side in each of these cases to isolate the portion beneath the levee, but is not required. Shutoff valves must be located outside any pipe casing in a concrete box enclosure with a secure, manhole-type, watertight (waterside) access cover. Since new pressure pipes must be elevated over an embankment, their waterside valve boxes may be placed on the embankment crest to provide access during high water.

5.8. <u>Acceptance Testing and Inspection.</u>

5.8.1. General. Acceptance testing formalizes the approval of newly installed or rehabilitated pipes using inspections and field testing so that the pipe owner does not rely solely on the manufacturer's QA/QC factory testing. USACE requires an inspection of each installed pipe section after the trench has been backfilled but before the embankment is placed. Post-installation inspections provide a baseline for subsequent in-service inspections. Section 5.8.2. provides a checklist, in approximate construction sequence, of typical considerations for preparing construction contract specifications. Table 5-4 lists relevant standards by pipe material that should be used for acceptance testing.

Table 5-4
Relevant Standards and Guidance for Pipe Acceptance Testing by Pipe Material

Pipe Material*	Guidance	Primary Topic
NP-RCP (< 10.8 psi)	AASHTO M 170/ASTM C76	Manufacturing
	AASHTO M 207/ASMT C507 (elliptical)	Manufacturing
	AASTHO R 73	Acceptance
	ASTM C969	Field testing, joints
	ASTM C1103	Field testing, joints
	ASTM C1479	Installation
	ASTM C443	Manufacturing, joint design & testing
	ASTM 497	Manufacturing, joint testing
LP-RCP (< 54.2 psi)	ASTM C361	Manufacturing, joint design
	ASTM C497	Manufacturing, joint testing
	ASTM C969	Field testing, joints
	ASTM C1103	Field joint testing
	AWWA M9	Installation, field joint testing
CPP	AWWA M9	Installation, Field joint testing
PCCP	AWWA C301 AWWA C304	Manufacturing, joint design, structural design
RCCP	AWWA C300	Manufacturing, joint design
RCNP	AWWA C302	Manufacturing, joint design
BWCP	AWWA C303	Manufacturing, joint design
VCP	ASTM C12	Installation
	ASTM C828	Field joint testing
FRP	AWWA M45	Manufacturing, design, installation
	ASTM D3262	Manufacturing, joint design
	ASTM D3839	Installation, field joint testing
CSP	AASHTO M 36/ASTM A760	Manufacturing, joint design
	AASHTO LRFD Bridge Construction Specifications, Section 26	Installation
	ASTM A798	Installation, including joints
WSSP	ANSI/AWWA C604	Installation, field joint testing
	ASTM A53	Field welding & testing of joints
	ASTM A135	Field welding & testing of joints
	AWWA M11	Design, including joint design; installation
	AWWA C206	Field welding & testing of joints
CAP	AASHTO M196/ASTM B745	Manufacturing, joint design
	ASTM B788	Installation, including joints
	ASTM F2487	Field testing, joints (applies to CAP)
DIP	ASTM A716	Manufacturing
	ASTM A746	Design, factory joint testing
	AWWA C600	Installation, field joint testing
SW-HDPE	ASTM D2321	Installation, including joints

Pipe Material*	Guidance	Primary Topic
	ASTM D3212	Manufacturing, joint design
	ASTM F1417	Acceptance & field joint testing (non-pressurized pipe)
	ASTM F2164	Field joint testing (pressurized pipe)
	ASTM F714	Manufacturing
	AWWA C906	Manufacturing
	AWWA M55 (Joints)	Manufacturing, joint design & field joint testing
PW-HDPE	ASTM D2321	Installation, including joints
	ASTM D3212	Manufacturing, joint design
	ASTM F2306	Manufacturing
	ASTM F2487	Field joint testing
	ASTM F3058	Field joint testing
PW-PP	ASTM D2321	Installation, including joints
	ASTM D3212	Manufacturing, joint design
	ASTM F2764	Manufacturing, joint design
	ASTM F2881	Manufacturing, joint design
	ASTM F3058	Field joint testing
PVC	ASTM D2321	Installation, including joints
	ASTM D2774	Installation, including joints
	ASTM D3034, (Type PSM[2])	Manufacturing, joint design
	ASTM F679	Manufacturing, joint design
	ASTM F794, (Profile)	Manufacturing, joint design
	ASTM F913	Manufacturing, joint design
	ASTM F949	Manufacturing, joint design
	ANSI/AWWA C605	Installation, field joint testing
SRTP	ASTM D2321 (applies to SRTP)	Installation, including joints
	ASTM F2562	Manufacturing
CiPCP	FEMA 484	Monolith contraction joints with water stops (EM 1110-2-2102) and construction joints
RCB (Precast Reinforced Conc. Box Culvert)	AASHTO R 73	Acceptance
	ASTM C1433	Manufacturing
	ASTM C1577 (AASHTO LRFD)	Manufacturing
	ASTM 497	Manufacturing, joint testing

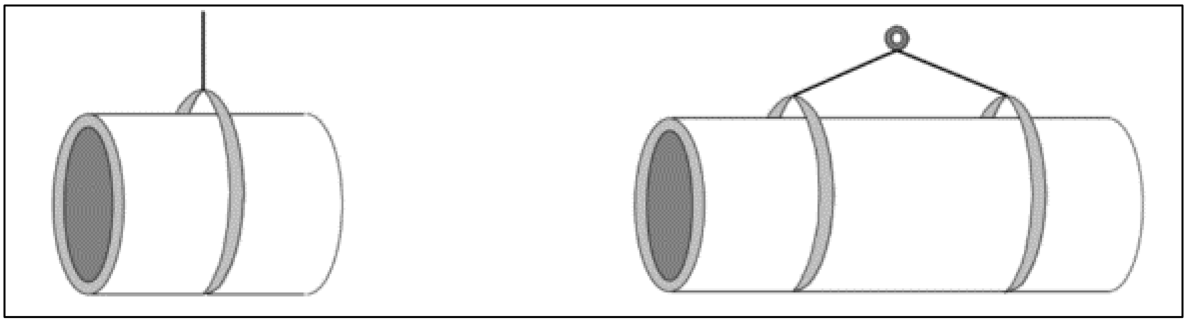
* See abbreviations in the glossary for pipe material acronyms.

5.8.2. Checklist of Installation Considerations. The following guidance covers the portion of the pipeline within the levee inspection limits as determined in Section 6.3.

5.8.2.1. Certification Stamp. All pipes and fittings must have a manufacturer's certification stamp stating that the material conforms to the specification and/or guidance governing the manufacture of the particular pipe material; otherwise, it must be rejected.

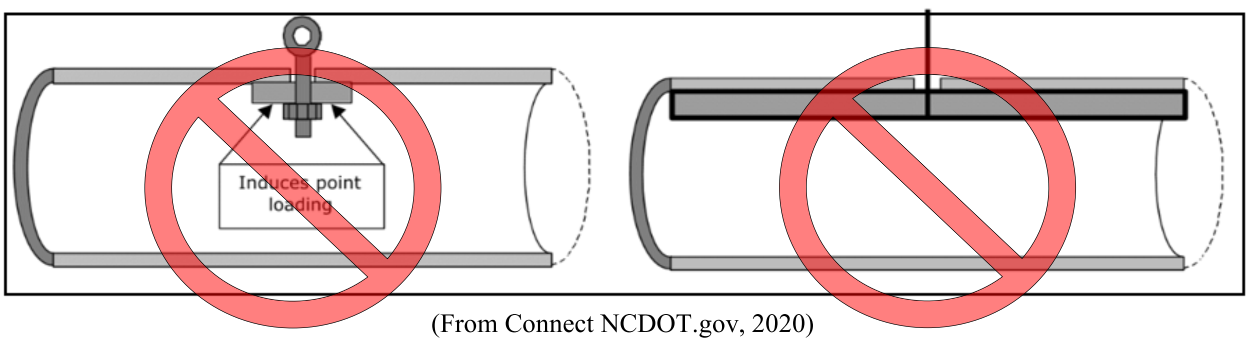
5.8.2.2. Visual Inspection. Visually inspect all pipe segments and appurtenances upon arrival at the job site to ensure that they were not damaged during transit before accepting their delivery.

5.8.2.3. Pipe Handling. Lift pipes according to the manufacturer's directions (from delivery vehicles and into trenches) to prevent excessive bending stresses or damage to protective coatings (Figure 5-56 and Figure 5-57).



(From Connect NCDOT.gov, 2020)

Figure 5-56. Acceptable sling methods for lifting pipes.



(From Connect NCDOT.gov, 2020)

Figure 5-57. Lifting holes are unacceptable for pipes associated with dams and levees.

5.8.2.4. Pipe Protection. Protect pipes from impact damage, such as scratching and cracks, and store on level ground in the manufacturer's packaging. Follow the manufacturer's recommendations for allowable stack heights, supports, and exposure (temperature and ultraviolet).

5.8.2.5. Field Repair of Damaged Pipes. Follow manufacturer's guidance for field repair of damaged pipes, if the repair is not expected to affect performance; otherwise, replace the pipe. Defects vary by pipe material, but common defects include: fractures and cracks; coating holidays or damage; and surface defects indicating mixing, molding, or other manufacturing deficiencies. Pipe sections with joint damage must be replaced.

5.8.2.6. Hydrostatic Testing. USACE requires that each joint be tested hydrostatically to determine whether it exceeds the maximum joint leakage specified by the pipe's applicable ASTM or other guidance (Table 5-4).

5.8.2.7. Installation Order. Ensure that match-marked pipe segments are installed in the correct order.

5.8.2.8. CLSM Backfill Testing. CLSM backfill must be tested according to ASTM C1019. Compacted soil backfill must be tested for moisture-density relationships meeting the requirements of ASTM D698 or ASTM D1557 (determined by the respective USACE District). Field density tests are determined according to ASTM D1556 or D6938 (determined by the respective USACE District). Replace CLSM backfill if its strength exceeds construction criteria or re-compact soil backfill not meeting these criteria.

5.8.2.9. Walk-through and Remote Inspection. Reference Section 6.5. for walk-through or remote inspection guidance for installed pipe. Laser profiling, governed by ASTM F3095, is also an acceptable technique. Inspection technology such as laser crack measurement, electric current leak detection, or other new technology must be approved by the respective USACE District. Document inspection results to create a baseline for future inspections.

5.8.2.10. Inspection Items. Inspect each segment of pipe for: alignment; settlement or sags; excessive joint offsets or separations; buckling, bulging, and deformation; protective coating damage; seal or weld separations; and other damage. Ring deflection that may occur during installation must not exceed the limits in Table 4-1. Reinstall segments with installation deficiencies. Repair or replace damaged segments.

5.8.2.11. Assessment of Identified Defects. Identified defects on an installed pipe must be assessed by an engineer for potential impacts to embankment or floodwall performance (reference Section 6.7.).

5.8.3. Post-Construction Inspection.

5.8.3.1. General. A post-construction inspection of pipes within the inspection limits of a levee as determined in Section 6.3. must be performed no sooner than 30 days after completion of the project to assess backfilling, grading, paving, placement of concrete structures, etc.

5.8.3.2. Deflection Testing. For deflection testing on newly installed flexible pipes after 90 days, use an industry standard laser profiler, pipe mandrel, or manual measurements (reference Section 7.5.2.3.) to determine if deflections exceed the limits stated in Table 4-1. Use Equation 5-13 to calculate percentage deflection. Pipe segments with deflections exceeding the values in Table 4-1 must be reinstalled.

Equation 5-13. Deflection Calculation.

$$[(\text{MMD} - \text{CMD})/\text{CMD}] \times 100 = \%\ \text{deflection}$$

Where:

MMD = minimum of the eight diameter measurements at each location every 10 feet along the length of the pipe, or any location where deflection, bulging, buckling, or racking is evident.

CMD = original certified mean diameter as provided by the pipe supplier.

5.8.3.3. Mandrels. Pipe mandrels for use on new pipes must be nine-armed (or more odd-numbered), non-adjustable, fixed-arm mandrels (Figure 5-58).

(From Sewer Equipment Company of Florida, 2020)
Figure 5-58. New pipe mandrel for small to medium sized pipes.

5.9. Construction Reporting and Documentation. Post-construction documentation is a valuable reference for future inspections, risk assessments, and alterations. As the pipe installation progresses, field documentation must be assembled as part of a post construction report. Conditions including joint gaps, tears, misalignment, cracks, and deformations must be noted and reviewed by the condition assessor (reference Section 6.7.). The post construction report should include the following items:

5.9.1. Manufacturer Documentation. Manufacturer documentation should include: design drawings for pipe joints; design calculations, proof of design testing, and inspection records for pipes and fittings; and, drawings, design calculations, and specifications for appurtenances (such as slide gates, flap gates, valves, pumps, and pre-formed or precast associated structures, such as gate wells, manholes, headwalls).

5.9.2. Acceptance Testing Documentation and Inspection Reports. Acceptance testing documentation and inspection reports should be developed as described in Section 5.8., including any additional quality assurance/quality control information.

5.9.3. Testing and Inspection Personnel and Equipment Information. Testing and inspection personnel and equipment information should include a statement of the field accuracy achieved for all measurements, including tolerances. The report typically includes a narrative about required field/measurement calibration and provides proof that all calibration procedures were followed when collecting data within the report.

5.9.4. As-built Drawings and Field Data. As-built drawings and field data should be provided in a digital format, including the pipe's alignment profile and elevations. Tracking equipment, including method or confirmatory procedure, used to capture the data should also be included.

5.9.5. General Soil Characteristics. General soil characteristics of the embankment material and pipe backfill (if soil) should be documented, such as the Unified Soil Classification System (USCS) Soils Classification per ASTM D2488 and density.

5.9.6. Engineer's Evaluation. The engineer's evaluation of installed condition should be documented, typically including certification that the pipe and backfills were installed consistent with the final approved plans and specifications.

5.9.7. Recommendations for Operation and Maintenance. Reference Table 7-1 for pipe material cleaning specifications to maintain a pipe's operational condition.

THIS PAGE INTENTIONALLY LEFT BLANK

Chapter 6
Inspections, Condition Assessments, and Prioritized Mitigation Plans

6.1. Introduction. As pipes associated with United States Army Corps of Engineers (USACE) dams and levees age, deterioration not only affects their performance but also poses increasing risk to the public through the potential for project failure. Therefore, pipes are inspected and their condition assessed on a recurring basis so that any potential impact to the integrity of a USACE dam or levee can be evaluated regularly. These condition assessments are also critical to making decisions related to pipe repair, rehabilitation, removal (with or without replacement), or decommissioning needs. Unless specifically stated otherwise, discussion herein of inspection techniques refers to pipes associated with both USACE dams and levees, whereas discussion of inspection limits and prioritized mitigation plans only refers to pipes associated with USACE levees.

6.2. Potential Failure Modes Revealed by Inspections.

6.2.1. Overview. The presence of pipes within, beneath, or adjacent to dams and levees can increase the risk of embankment failure by providing opportunities for the loss of embankment or foundation material that would not otherwise exist. Pipe inspections are intended to document the condition of a pipe; these observations may also inform the inspector that a PFM has initiated or has the potential to initiate. Chapter 2 discusses PFMs related to pipes through dams and levees; internal pipe inspections can reveal evidence of flaws leading to the initiation of PFM-2 or PFM-3, while evidence of activity related to PFM-1 and PFM-4 must be observed externally.

6.2.2. Interior Inspections. Inspections of pressurized pipes or pipes that can sometimes become pressurized can inform the presence of flaws that may allow pressurized fluids to escape into the surrounding embankment and either remove embankment material or cause slope instability (PFM-2). Although internal inspections of pressurized pipes are possible and often mandated by state or federal regulations, discovering a point of leakage is not as easy as in gravity drains. Inspections of gravity drains may also detect the initiation of PFM-3 by documenting the presence of penetrating corrosion, open pipe joints, open connections to associated structures, or soil accumulation. These flaws provide unfiltered points of entry into a pipe where embankment material can be lost. Visual observations of these flaws vary from stained or wet areas running down the pipe interior to actively running water or accumulation of soil deposits. Observed joint gaps do not necessarily indicate a flaw as some joint types have allowable separation when properly homed.

6.2.3. Exterior Inspections. Exterior inspections involve walking and observing the ground above the pipe for isolated areas of soft soil, depressions, or sinkholes that may indicate a loss of material into the pipe (PFM-3), or for wet soft ground or soil deposits near the headwall that could indicate an external seepage path along the pipe (PFM-1) or an exit location for a leak in a pressurized pipe (PFM-2). An observation of discharged soil particles indicates the need to evaluate and potentially install an external seepage filter to interrupt the failure mode. Leakage from a pressurized pipe (PFM-2) may not be obvious during an internal inspection and could allow prolonged releases that may go undetected until there are surficial indications, such as saturated or sloughing slopes, wet/ponded areas near the toe, or lush vegetation. Observed externally, evidence of escaping fluid can easily be distinguished from PFM-1 if discovered when the levee is not loaded. Erosion at the outlet (initiation of PFM-4) is easily observable, but the continued operational adequacy of the outlet control must be verified by ensuring that the headwall did not move enough to prevent proper gate operation or cause a pipe connection to separate. Excavation to expose pipes to allow for exterior inspections and a more thorough understanding of the pipe condition may be necessary and is usually performed when pipes are located at a shallow depth (Figure 6-1).

(Courtesy of Louisville District)
Figure 6-1. Examples of excavation to allow exterior pipe inspection.

6.3. Special Considerations for Levee Toe Drain Inspections. There are occurrences within the USACE levee portfolio in which toe drains were installed without the effort of a design analysis to prove their necessity to the project. As such, USACE may require inspection of features that do not serve an essential function. Since toe drain inspections are usually expensive and can be difficult or impossible to access in some cases, the relevant District may suggest that the project owner review the design documentation to see if the toe drain was designed for site-specific usage. If it was not, the relevant District may request that the owner perform a seepage analysis, at its own expense, which will be reviewed by the District to determine if the feature may be decommissioned in place. In cases where USACE is the project owner, the relevant District will determine the need for a seepage analysis and perform and review the analysis, as appropriate.

6.4. Inspection Limits.

6.4.1. Real Estate Rights. Although all or portions of a pipe requiring inspection may be located outside of the levee/floodwall right-of-way, inspection of the pipe may be conducted according to the real estate rights associated with the pipe. Access to these pipes should be permitted either under their own specific easement or allowed as part of the levee or floodwall right-of-way. The owner of the pipe must be familiar with the real estate instruments associated with the pipe to ensure that inspections are conducted consistent with the terms of the easement. Utility/pipeline easements generally allow the owner of the pipe to construct, operate, maintain, alter, replace, and inspect the pipe(s) within the pipe's easement limits. In cases where a records search has been conducted by a real estate professional or an attorney and an easement for a particular pipe does not appear to exist, the pipe owner should obtain legal services to acquire an easement from the landowner. In some cases, the pipe may be subject to an easement by prescription, which is an easement acquired by continued use without legal permission of the landowner for a legally defined period of time. Laws regarding prescriptive easements vary by state and the pipe owner should consult an attorney to assess whether they may be applicable. The "required" inspection limits shown in Figure 6-2 through Figure 6-10 assume no right-of-way issues were encountered. All opportunities to resolve right-of-way disputes should be investigated so that any pipe or portion of a pipe within the influence zone is inspected; however, until resolved, the inspection limits must legally terminate vertically at disputed rights-of-way.

6.4.2. Inspection Limits Overview. There are two reasons for establishing pipe inspection limits: the first is to maintain the structural integrity of the levee system, and the other is to prevent inundation of the leveed area by maintaining its operational adequacy; these limits are not necessarily the same. The following limits are considered minimums and it is recommended that the respective USACE District determine if there is a need to increase these limits on a case-by-case basis based on site-specific knowledge. In addition to pipes associated with existing levees, existing pipes along the proposed alignment of new levees must be inspected within these limits and meet USACE assessment requirements prior to allowing the pipe to remain in place.

6.4.3. Inspection Limits Related to Structural Integrity. The area within which pipe-related defects could impact the structural integrity of the levee or hinder access along it is considered the "influence zone." The limits of the influence zone were established assuming the loss of soil into a pipe produces a stope (Figure 6-1) that advances more or less vertically to the surface before the interior sides start to successively slough over time toward its angle of repose. The 1H:1V (45°) slope angle of the zone limits may be more steep than the angle of repose for a specific soil in question, especially with loose fine-grained soils or dry sands; however, it is anticipated the sloughing process will be gradual, allowing the issue to be detected and addressed before crossing over into the influence zone limits. As shown in the following figure, a minimum horizontal distance from the structure is included before the downward slope begins. This not only provides a buffer, but also provides sufficient O&M and emergency access adjacent to the embankment or floodwall should a sinkhole develop.

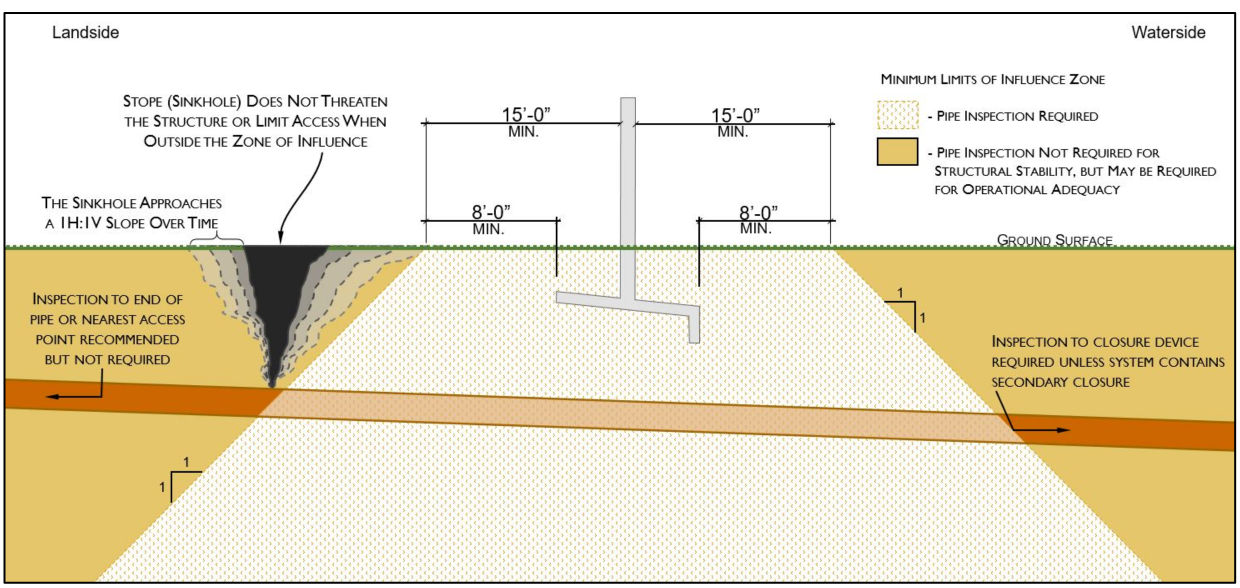


Figure 6-2. Example for purpose of influence zone limits.

6.4.4. Inspection Limits Related to Operational Adequacy. Operational adequacy here refers to a system's mechanical ability to keep the leveed area from becoming inundated. The inspection limits used to ensure structural stability will not necessarily ensure operational adequacy because it is possible for water to bypass the embankment or floodwall without affecting the integrity of the structure. Figure 6-2 shows a vulnerable area outside the influence zone, between the waterside inspection limits and the outlet headwall, where a defect in the pipe can allow water to bypass the closure device and enter the leveed area. In cases where there is no secondary (backup) closure device, USACE requires that the waterside inspection limits be extended beyond the influence zone limits, defined in Section 6.4.2 to the closure device.

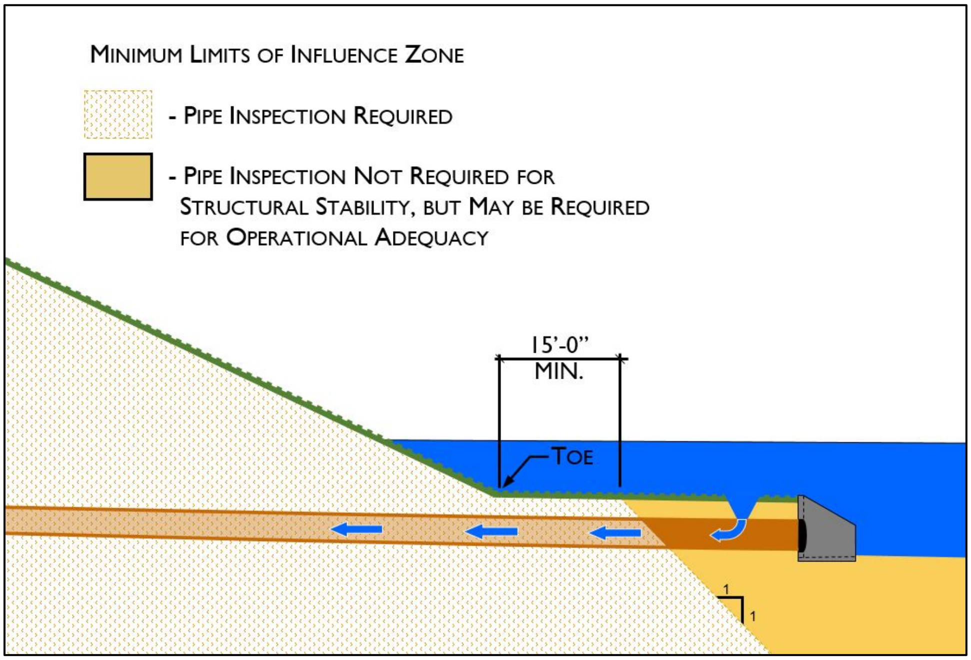


Figure 6-3. Structural integrity inspection limits can fail to inform operational adequacy.

6.4.5. Gravity Pipes Through and Beneath Levee Embankments. Gravity pipes penetrating the levee embankment cross section are inspected from headwall to headwall (Figure 6-3, upper pipe). Gravity pipes beneath levee embankments or floodwalls which do not daylight at the landside levee toe are inspected to 15 feet at a minimum, with a projected 1H:1V slope below the ground surface (Figure 6-3, lower pipe). Access to gain entry into pipes may be required beyond the project rights-of-way. Figure 6-4 and Figure 6-5 show the minimum limits of inspection for pipes passing beneath floodwalls.

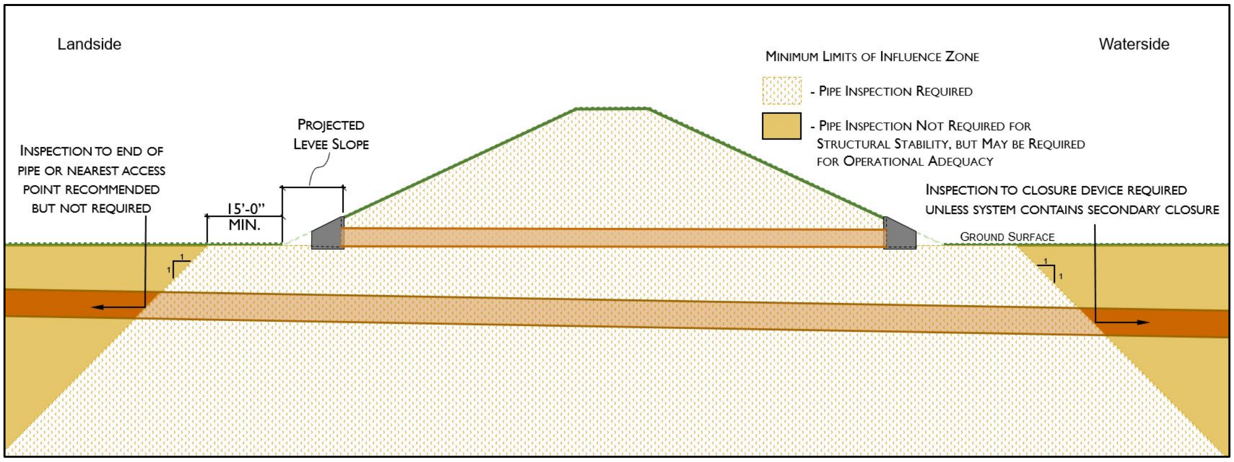


Figure 6-4. Gravity pipes through and beneath a levee embankment.

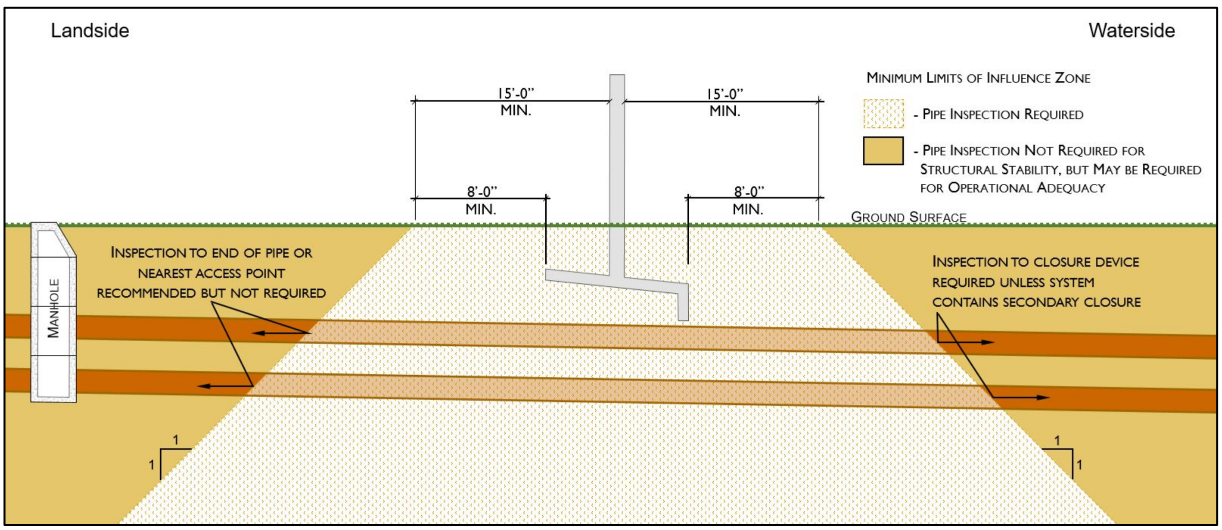


Figure 6-5. Gravity pipes beneath a floodwall (T-wall).

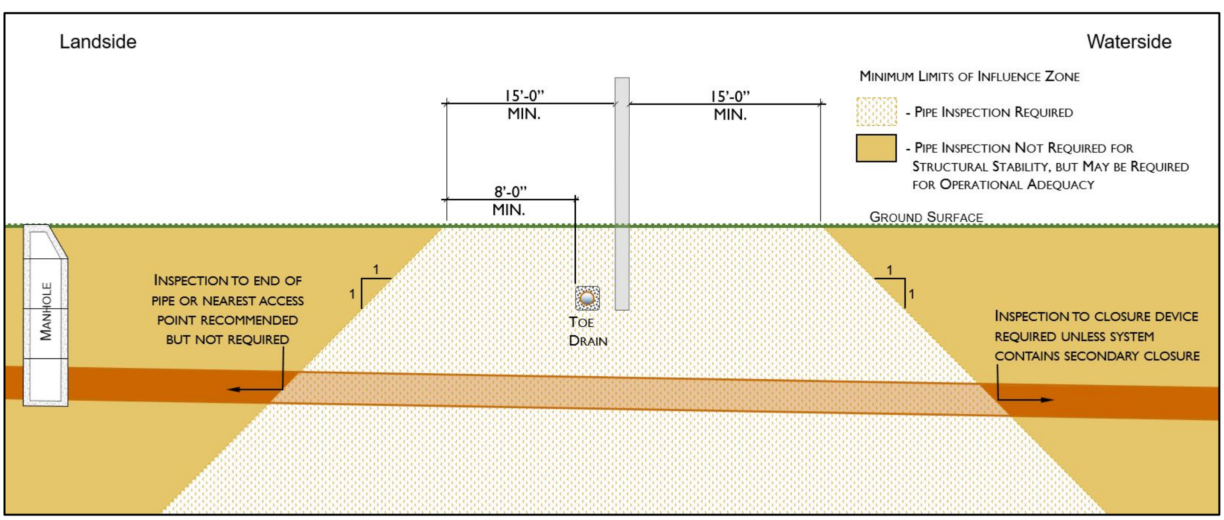


Figure 6-6. Gravity pipes beneath a floodwall (I-wall).

6.4.6. Gravity Pipes that Do Not Cross a Levee Alignment. Pipes running adjacent to a levee embankment or floodwall that are within 15 feet horizontally of the levee toe or floodwall (I-wall) face, within eight feet of the floodwall (T-wall) foundation or toe drain, or below a 1H:1V slope from that point, require inspection. Examples of these types of pipes include toe drains, collector systems, and third-party pipes Figure 6-7 and Figure 6-8).

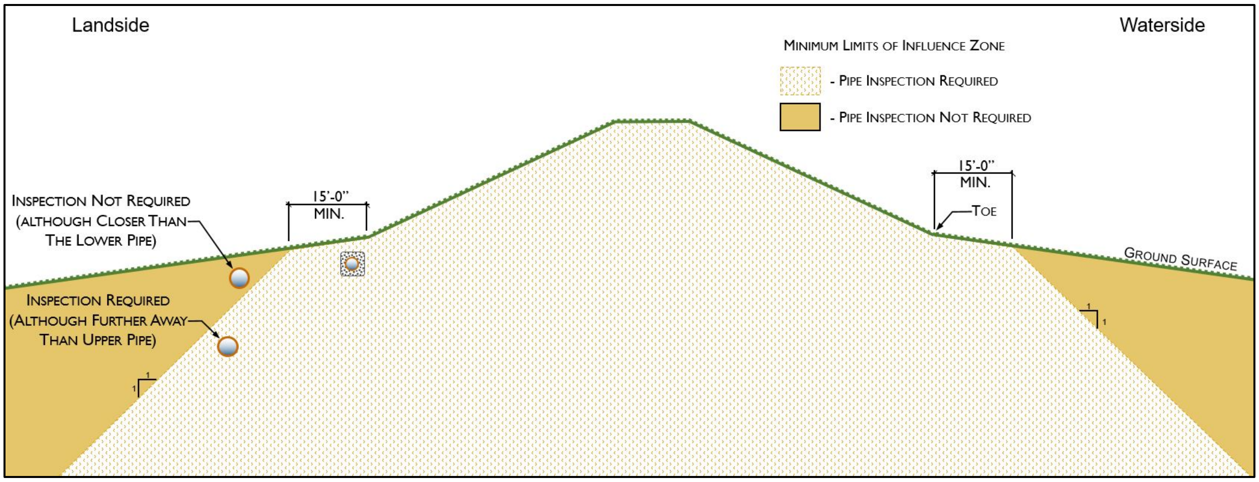


Figure 6-7. Gravity pipes adjacent to a levee.

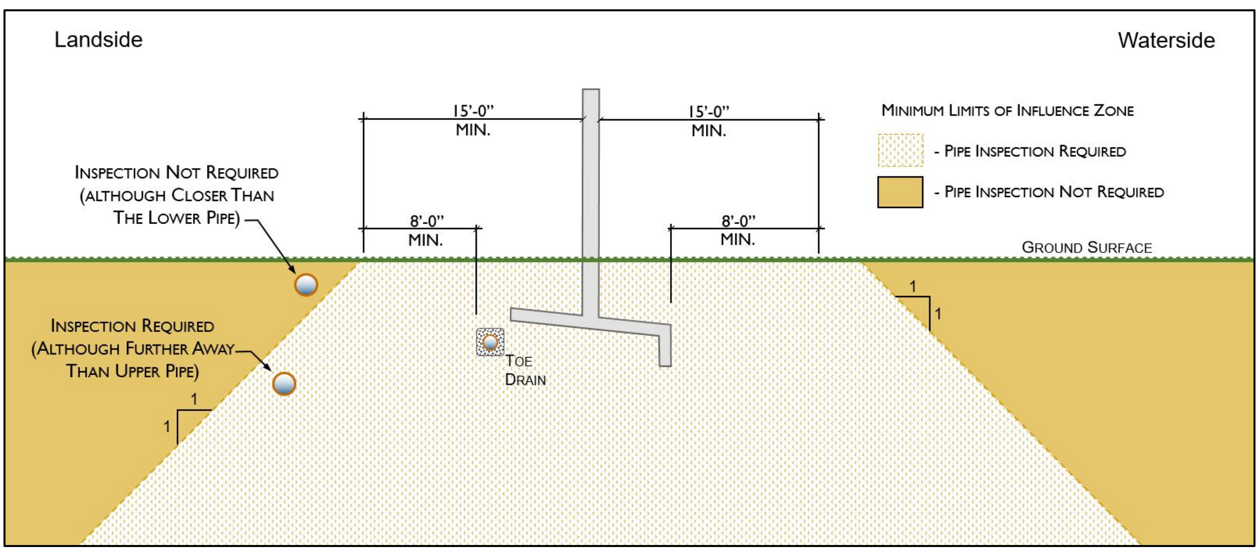


Figure 6-8. Gravity pipes adjacent to a floodwall (T-wall).

6.4.7. Discharge Pipes from Pump Stations (Through or Profile). Discharge pipes from pump stations are inspected from inside the pump station starting at the flange connection and ending at the end of the discharge line at the headwall/gatewell (Figure 6-9 and Figure 6-10). Pipe access is possible through outlet headwalls, gatewell discharge locations, and sometimes air vents/siphon breakers (Figure 6-11, reference Section 9.4.4.1.). The discharge pipes must be inspected from the pump station all the way to the outlet structure, regardless of whether they are interrupted by a gatewell. This applies to the entire waterside pipe length, even for the portion beyond the 15-foot limit/distance from the levee toe.

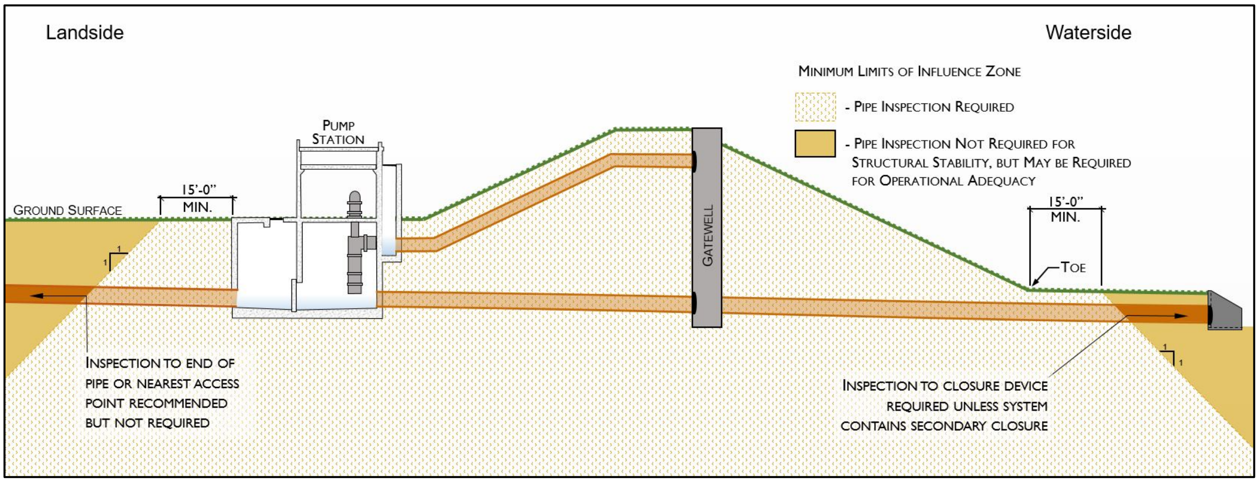


Figure 6-9. Pump station discharge pipes into gatewell.

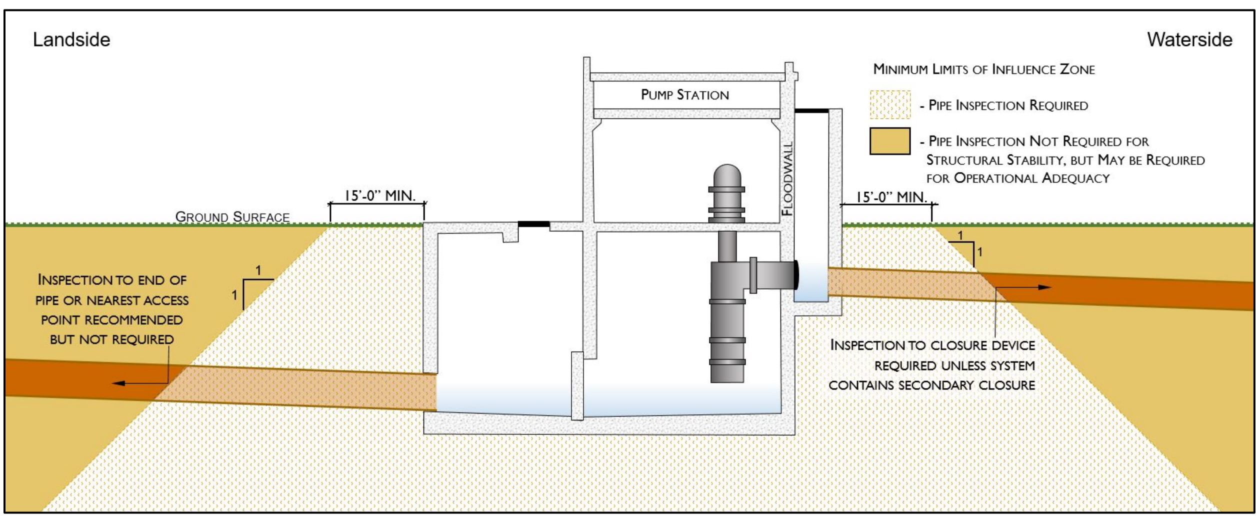


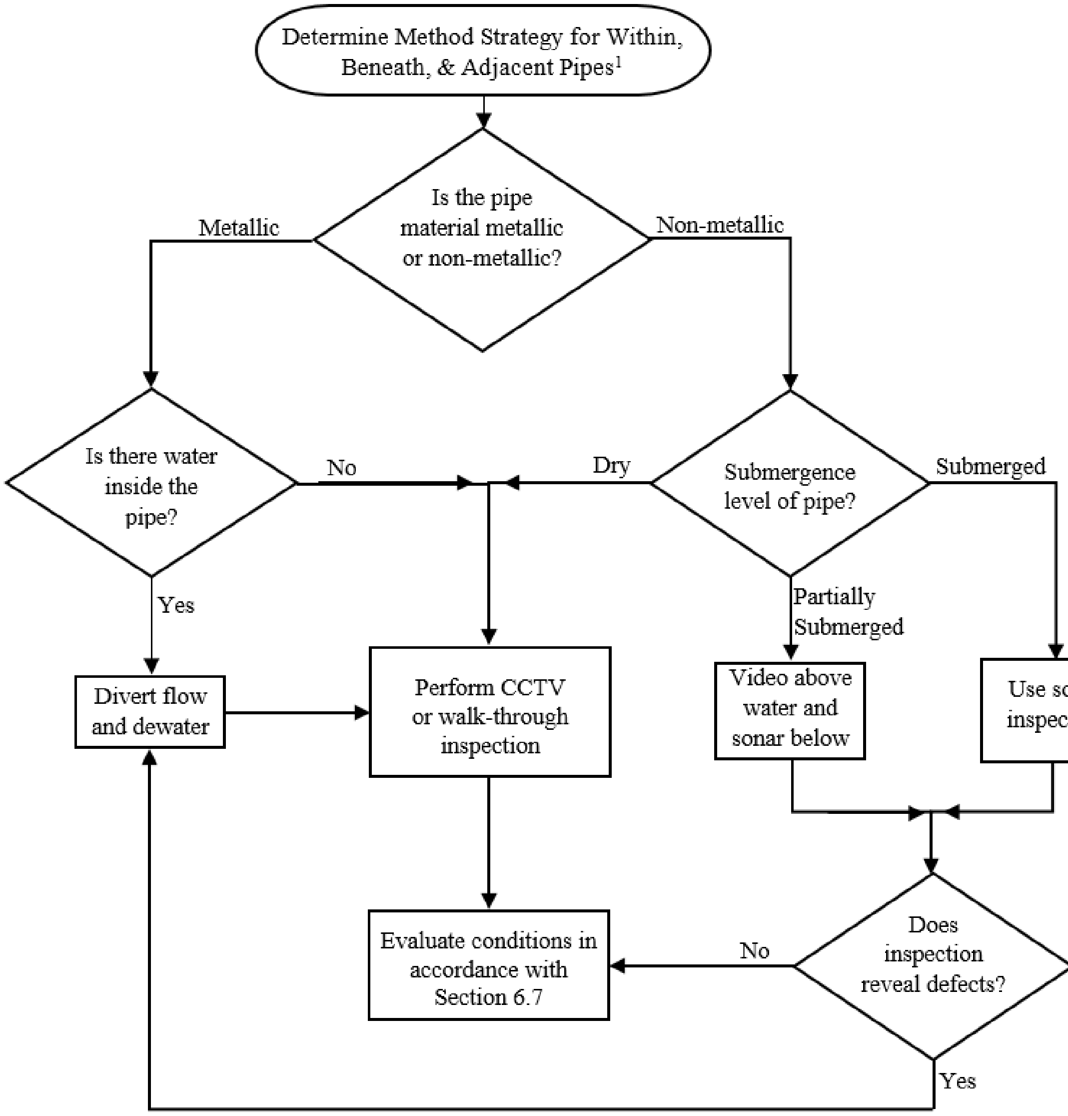
Figure 6-10. Pump station integral with floodwall and associated pipes.

(Courtesy of USACE Louisville District)
Figure 6-11. Potential inspection access points for pump station discharge pipes.

6.5. Inspection Methods.

6.5.1. General Planning. An inspector should consider the following before starting or scheduling a pipe inspection: high water may prevent access due to pumping operations or pipe submergence; freezing temperatures may produce ice within the pipe that hinders visibility or access; seasonal nesting or spawning of certain species inhabiting culverts could affect access; height of vegetation that could obscure visibility and hide ground surface defects; and sediment and debris can obscure the invert of the pipe, which is typically in the worst condition.

6.5.2. Pipe Size and Submergence Factors. Pipes less than 48 inches in diameter are typically considered too confined for a “walk-though” inspection and are therefore most often inspected using remote cameras or another comparable methods approved by the respective USACE District. Pipes larger than 48 inches in diameter are easier for man-entry but other factors, such as air quality, may necessitate or provide a preference for remote cameras. The appropriate inspection method is determined by the presence or absence of water in the pipe and the pipe material. Figure 6-12 is used to determine proper inspection methods based on the pipe material and setting.



Note 1: Excluding third-party pipes

Figure 6-12. Inspection method strategy for essential pipes.

6.5.3. Walk-through Method. When safe, a walk-through inspection is preferred for pipes associated with both dams and levees, since it provides the inspector the ability to touch and test areas of interest as well as make very accurate measurements. Flows up to six inches deep in the pipe invert can be tolerated when the water is clear enough that the condition of the bottom of the pipe can be seen. The direction of inspection should be conducted opposite of the flow, from downstream to upstream, to prevent creating turbid conditions that may obscure the invert. Walk through inspections for pipes both associated with dams and levees should be performed to the standards of National Association of Sewer Service Companies' (NASSCO) Pipeline Assessment Certification Program (PACP), or an organization with equivalent standards. Closed-circuit television (CCTV), video recording, or digital still photos should be used to document defects, as described in Section 6.4.

6.5.4. Remote Methods.

6.5.4.1. CCTV. Pipes with dry or nearly dry interiors can be remotely inspected using CCTV cameras mounted on tracked or wheeled vehicles. Push-type CCTV cameras can also be used to remotely inspect sloped or vertical pipes when access will not allow tracked or wheeled CCTV equipment. Metallic pipes (i.e., steel, aluminum, ductile iron, and cast iron) are subject to corrosion which can only be adequately observed when completely dewatered.

6.5.4.2. Sonar. Sonar is the preferred method for inspecting fully submerged non-metallic pipe (i.e., reinforced and non-reinforced concrete, vitrified clay, fiberglass reinforced, and plastic pipes) when it is not practical to dewater the pipe. However, the quality of the visual record obtained by sonar is less detailed than CCTV. A sonar inspection portrays offsets and distortions in the interior pipe profile as well as sediment build up in the pipe invert. As such, sonar will not clearly reveal the presence of fractures without offsets or open cracks with soil visible. Sonar inspections are rarely used on pipes subject to corrosion (i.e., steel, aluminum, ductile iron, and cast iron) due to the inability of sonar to determine surface corrosion without measurable section loss. Debris or sediment in the pipe will limit the sonar's capability to produce useful images. Pipes should be cleaned before a sonar inspection, if possible.

6.5.4.3. CCTV and Sonar Hybrid. Partially-submerged pipes that cannot be dewatered are inspected using CCTV inspection above water and sonar inspection below water. When sonar inspection of a submerged pipe indicates that the pipe cross-sectional profile deviates from the as-built condition, the pipe must be dewatered and CCTV inspected.

6.5.5. Advances in Technology. As new pipe inspection technologies become available, they will be considered as alternative methods for determining the internal condition of levee pipes. Proposed new methods of inspection must be approved by the respective USACE District prior to use to ensure adequate and accurate documentation can be captured and still provide protection of the pipes.

6.6. Inspection Requirements.

6.6.1. Walk-through Requirements.

6.6.1.1. Inspector Qualification. Inspectors must be trained and certified by the NASSCO PACP, or an organization with equivalent standards. Inspectors should also be knowledgeable about pipe design and installation, how pipe defects can lead to PFMs, and pipe joint and structure connection details. Inspectors should be familiar with site-specific details about the type of pipe, limitations of pipe joint movement, and plan and vertical alignment. The inspector should review construction and inspection history prior to the inspection.

6.6.1.2. Cleaning. Any pipe with debris, sediment, or other obstruction that inhibits the inspection of the pipe must be cleaned prior to inspection (reference Chapter 7).

6.6.1.3. Equipment and Tools. The following are required for walk-through pipe inspection:

6.6.1.3.1. Inspection Form. Inspection form for documenting inspection and pipe defects (reference Section 6.7).

6.6.1.3.2. Permits. All appropriate permits (if applicable) in addition to air monitoring, communication, and rescue equipment required for safe entry. This includes confined space training and equipment when necessary.

6.6.1.3.3. Light Source. Bright, high intensity light source, preferably intensity-adjustable to provide visual clarity of defects. Lighting during the inspection should be adequate to fully illuminate, but not overly illuminate, the entire pipe. Excessive lighting or an overly-adjusted camera iris can result in a flaring of the image and exaggeration of pipe joint displacement or other pipe conditions.

6.6.1.3.4. Camera. Camera to document all defects with digital video and/or still photos.

6.6.1.3.5. Measurements. Method for determining the distance from the entry point (i.e., headwall, manhole, or sluice gate) to the nearest 0.1 foot for accurate location of defects within the pipe.

6.6.1.3.6. Global Positioning System. Global Positioning System equipment capable of determining coordinates of the inlet and outlet locations to six-decimal accuracy, preferred in decimal format (e.g., 38.845972 instead of 38˚ 50' 45.5"). Ensure sufficient satellite connection before recording readings.

6.6.2. Remote Requirements.

6.6.2.1. Inspection Operator Qualifications. The individual performing the inspection is required to demonstrate their qualifications by providing training and experience records. Individuals operating the remote inspection equipment must be trained and certified by the NASSCO PACP, or an organization with equivalent standards. A minimum of one year of experience with pipe inspections using the NASSCO's PACP or an equivalent industry standard is required. It is recommended that personnel demonstrate experience with levee or dam pipes. All inspections using remote equipment are required to meet the minimum visibility and operation requirements of NASSCO's PACP or an equivalent industry standard.

6.6.2.2. Cleaning. Debris, obstructions, and sediment must be cleaned to provide an unobstructed view of the pipe's interior before video or other remote inspections are conducted. CCTV inspection is adequately accomplished through shallow depths (up to four to six inches) of clear water. If the water is turbid or too deep to permit a clear view of all pipe and joint surfaces, dewatering or diversion is required. Reference Chapter 7 for more information on cleaning pipes.

6.6.2.3. CCTV. CCTV is required to have the following capabilities or features to execute and record the inspection. Additional pipe inspections using specialized equipment may be required to document specific measurements (e.g., detected offset joints or metal loss due to corrosion).

6.6.2.3.1. Light Source. Bright, high-intensity light source that travels with the camera. Ability to control the light intensity to control glare is an important feature that can improve the quality of the video images. Lighting during the inspection must be adequate to fully illuminate, but not overly illuminate, pipe joints and individual points of interest (at a right angle to the direction of travel) for an accurate assessment. Excessive lighting or an overly-adjusted camera iris can result in a flaring of the image and exaggeration of pipe joint displacement or other pipe conditions.

6.6.2.3.2. Camera. Color, high definition (720p or better) resolution camera with remote focus, zoom, pan, and tilt capability.

6.6.2.3.3. Video. Video image digitally recorded using a current digital multimedia video format.

6.6.2.3.4. Measurements. Footage meter with capability to record footage reading on the video at all times. Ability to record and display distance from starting point within 0.1-foot accuracy.

6.6.2.3.5. Travel speed. Maximum travel speed through the pipe not exceeding 25 feet per minute.

6.6.2.3.6. Still Images. Ability to take still images in .jpg or .png format of all significant defects observed during the inspection.

6.6.2.3.7. 360-degree Joint Views. Ability to stop traversing and record a 360-degree view of each joint.

6.6.2.3.8. Defect Recording. Ability to stop traversing if any defects are suspected and record a detailed video inspection.

6.6.2.4. Sonar. Sonar equipment must be specifically adapted using multi-frequency sound waves to locate and map irregularities by creating continuous sonar images recorded in “real time” mode. Sonar equipment must utilize digital, multi-frequency profiling in order to model the submerged portion of the pipe. Using a rotating transducer, the sonar unit must transmit an acoustic signal toward the pipe walls in a radial fashion. The time delay between transmission and reception of reflected pulse echo is used to determine the distance from the transducer to the surface that reflected the pulse.

6.6.2.5. CCTV and Sonar Hybrid. Inspection equipment must be positioned in the pipe according to the manufacturer’s recommendations and able to complete a 360-degree inspection of the pipe circumference at one-inch intervals along the length of the pipe. During the inspection, the following information must be clearly and continuously displayed on the periphery of the screen, monitor, and CCTV recording: starting location ID, ending location ID, and distance from access point.

6.7. Inspection Documentation.

6.7.1. General. Pipe inspections must be documented either using an inspection form during a walk-through inspection similar to the one shown in Figure 6-13 or a digital program that records the pipe condition during a remote inspection. For both walkthrough and remote inspections, PACP (or equivalent) ratings must be used to rate the defects of the pipe. Below is a list of the information that is required to be obtained as part of a pipe inspection and provided in a report for each pipe:

- Narrative report (Section 6.7.2)
- Date of inspection
- Inspector(s) name(s)
- Nearest dam or levee as-built station number to pipe that is being inspected
- Coordinates of the inlet and outlet locations to six-decimal accuracy, preferred in decimal format (e.g., 38.845972 instead of 38˚ 50’ 45.5”)
- Segment length (from inside wall of adjacent manholes to nearest 0.1 foot)
- Description of pipe purpose
- Type of pipe (material, diameter, shape, segment length)
- Inspection method (walk-through or remote inspection)
- Inside diameter of pipe
- Defect, PACP (or equivalent) rating
- Clock position of defect relative to the position of the hour hand of a clock
- Distance from reference (starting) end of pipe
- Defect measurements to establish height, width, and depth, if possible
- Picture of each defect

6.7.2. Narrative Report. The CCTV and/or sonar operator/inspector must provide a brief narrative report that summarizes the following for each pipe inspected: the pipe location; start and end points; conditions of the inspection; equipment used; general condition of the pipe; and specific defects with location.

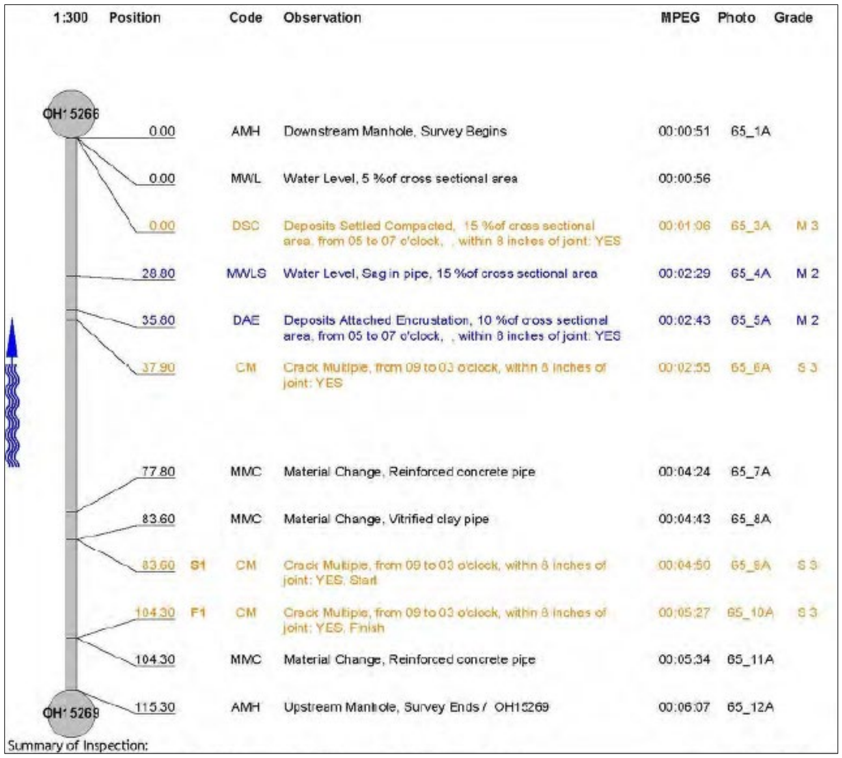
6.7.3. Submittals. Regardless of whether an inspection is performed by an inspection company or a USACE District, a digital report must be developed (and submitted, in the case of an inspection company) including the documentation in Section 6.4., paper inspection log (similar to Figure 6-13) or digital inspection log (similar to Figure 6-14 and meeting requirements of NASSCO's PACP or organization with equivalent standards), narrative report (Section 6.7.2.), and inspection video. The inspection video must be configured in a current digital multimedia video format. Digital files must be configured to have the ability to use all features of the CCTV player, including fast forward capability. Submission should include all raw and native data formats. The following items must be submitted after completion of all required inspection activities:

- Clearly labeled and organized electronic inspection videos.
- Electronic still-capture pictures and/or sonar images of significant defects.
- Printed and/or electronic inspection form or logs with information noted in Section 6.4. (Figure 6-13 and Figure 6-14).
- Aerial map locating deficiencies with the as-built pipe stationing or local name of culvert (Figure 6-15). The location of the embankment waterside and landside toe, or location of the floodwall centerline, must be clear and/or labeled on the aerial map. The map must also show the direction of CCTV camera travel. Sonar inspection defects are also mapped in a similar manner.
- A summary table showing the stationing and results of each pipe inspection rating based on NASSCO's PACP or equivalent industry standard.

SEGMENT/SYSTEM NAME:						
Inspection Details	Inspected by (Name, Certification):					
	Description:					
	Station:				Date of Inspection:	
	Type of Pipe:	Segment length (e.g. from inside wall of adj. manholes):			Pipe Diameter:	
	Inlet Coordinates:		Outlet Coordinates:			Accuracy:
	Inspected via:					
	Sketch (Path of Inspection, approx. Location of defect, location of pipe in relation to river portion of pipe under levee, etc.)					

Deficiency Details	PACP Defect Code	Location (ft.):	Time Stamp	Comments	Photo	Clock Position
Results Summary:						
Rating:						

Figure 6-13. Example inspection form for use during a walk-through inspection.



1:300	Position		Code	Observation	MPEG	Photo	Grade
OH15268	0.00		AMH	Downstream Manhole, Survey Begins	00:00:51	65_1A	
	0.00		MWL	Water Level, 5 %of cross sectional area	00:00:56		
	0.00		DSC	Deposits Settled Compacted, 15 %of cross sectional area, from 05 to 07 o'clock, , within 8 inches of joint: YES	00:01:06	65_3A	M 3
	28.80		MWLS	Water Level, Sag in pipe, 15 %of cross sectional area	00:02:29	65_4A	M 2
	35.80		DAE	Deposits Attached Encrustation, 10 %of cross sectional area, from 05 to 07 o'clock, , within 8 inches of joint: YES	00:02:43	65_5A	M 2
	37.90		CM	Crack Multiple, from 09 to 03 o'clock, within 8 inches of joint: YES	00:02:55	65_6A	S 3
	77.80		MMC	Material Change, Reinforced concrete pipe	00:04:24	65_7A	
	83.60		MMC	Material Change, Vitrified clay pipe	00:04:43	65_8A	
	83.60	S1	CM	Crack Multiple, from 09 to 03 o'clock, within 8 inches of joint: YES, Start	00:04:50	65_9A	S 3
	104.30	F1	CM	Crack Multiple, from 09 to 03 o'clock, within 8 inches of joint: YES, Finish	00:05:27	65_10A	S 3
	104.30		MMC	Material Change, Reinforced concrete pipe	00:05:34	65_11A	
OH15269	115.30		AMH	Upstream Manhole, Survey Ends / OH15269	00:06:07	65_12A	

Summary of Inspection:

Figure 6-14. Example digital log from remote inspection.

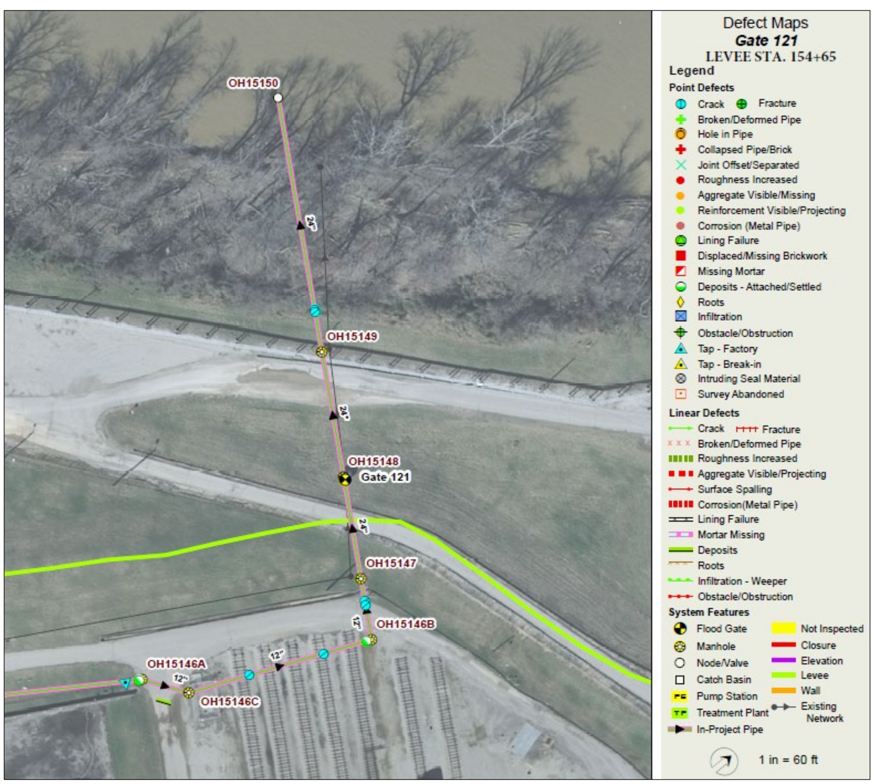


Figure 6-15. Example aerial map.

6.8. Condition Assessment.

6.8.1. General. Once the respective USACE District office has completed or received the submittals listed in Section 6.7.3, a USACE condition assessor will perform an assessment of each pipe based on the pipe inspection documentation (Section 6.8). There is no specific format needed for this assessment, but the individual pipe condition ratings should be shared with the project owner to facilitate the development of a cooperative prioritized mitigation plan (Section 6.9). For pipes associated with levees, each pipe's condition assessment, along with external conditions observed during an inspection, will be used to rate the overall condition of the pipes (based on the worst graded pipe in the system) according to the current levee inspection guidelines.

6.8.2. Condition Assessor Qualifications. The condition assessor assigned to determine the pipe inspection rating is typically a USACE engineer or other qualified person adequately trained and experienced to ensure appropriate and consistent ratings are assigned for each pipe inspection. Assessors must be certified by NASSCO's PACP or an equivalent industry standard; NASSCO maintains a category of structural defects specifically related to levees and dams. Because of the potential consequences, the acceptance standards for pipes associated with dams and levees are more stringent than those for storm or sanitary sewers not in dams or levees. Assessors should have the ability to review the dam or levee system as-built drawings and specifications before reviewing the inspection reports and inspection video or assigning a rating for each pipe.

6.8.3. Pipe Condition Assessment Grade. The condition assessor will use the inspection report along with the defects to determine the overall condition for each pipe. The condition assessor must determine the portion of the pipe that is likely to impact the structural integrity and operational adequacy of the dam or levee (reference Section 6.4); pipe defects within the portion of the pipe determined to be relevant by the condition assessor can generally be graded in correlation with the highest PACP (or equivalent rating system) structural defect grade. Whenever necessary, the condition assessor will use his or her training and experience to assign the most appropriate structural or O&M rating, which may differentiate from the recommended ratings in Table 6-1 and Table 6-2. For pipes associated with levees, each pipe's latest recorded condition assessment, along with external conditions observed during an inspection, will be used to assign the overall inspection item rating for pipes in the USACE levee inspection checklist.

Table 6-1
PACP structural defect grade correlation to required action

Defect Codes per NASSCO PACP [1] or other Organization with Equivalent Standards (reference Section 6.3 for inspection limits)		NASSCO PACP Defect Grade for Dams and Levees	Required Action
- Any crack hinge code - Any fracture, broken, or hole code - Collapse - Any flexible deformation > 10% - Any brick deformation ≥ 10% - Any rigid deformation Code - Any joint offset, separated, or angular code - Dropped invert	- Any weld failure code - Any point repair defective code - Missing brick - Any surface damage reinforcement code - Missing mortar medium and large - Surface damage missing wall - Surface damage corrosion (without further inspection or section loss ≥ 25%)[2]	4 or 5	Mitigate
- Crack longitudinal, multiple, or spiral - Any flexible deformation ≤ 10% - Any brick deformation < 10% - Displaced brick - Surface damage corrosion (with inspection and section loss < 25%)[2]	- Surface damage surface spalling - Any point repair code (non-defective) - Any surface damage aggregate code - Missing mortar small - Any lining feature code	3	Monitor
All other codes		1 or 2	Continue Inspection Frequency

[1]Defect codes are explicitly defined by the latest version of NASSCO's PACP manual. Some codes only pertain to specific pipe materials, while others indicate a location within a pipe segment.
[2]Additional inspection is recommended for observed corrosion to determine the extent of section loss using devices to measure remaining pipe wall thickness (e.g. ultrasonic thickness measuring instrument).

Table 6-2
PACP O&M defect grade correlation to required action

Defect Codes per NASSCO PACP [1] or other Organization with Equivalent Standards (reference Section 6.3 for inspection limits)		NASSCO PACP Defect Grade for Dams and Levees	Required Action
- Any deposits code ≥ 25% blockage - Any roots medium or ball code - Any infiltration runner or gusher code	- Any obstruction code ≥ 25% blockage - Any intruding seal code ≥ 25% - Any tap defective code	4 or 5	Maintenance & Repair
- Any roots tap code - Any obstruction code 15% - 20% blockage - Any intruding seal code 15% - 20% blockage	- Any tap intruding code - Any deposits code 15% - 20% blockage - Any infiltration dripper code	3	Monitor
All other codes		1 or 2	Continue Inspection Frequency

[1]Defect codes are explicitly defined by the latest version of NASSCO's PACP manual. Some codes only pertain to specific pipe materials, while others indicate a location within a pipe segment.

6.8.4. Required Mitigation and Maintenance of Pipes Graded 4 or 5. Pipe mitigation measures to correct structural defects include repair, rehabilitation, removal (with or without replacement), or decommissioning. Other pipe defects may be able to be corrected through routine maintenance (e.g., jetting, root and ice removal per Section 7.4). A pipe condition assessment will aid in choosing the most appropriate measure; however the following criteria must be considered:

6.8.4.1. Collapsed Pipes. Pipes that have collapsed per the latest version of the NASSCO's PACP manual cannot be rehabilitated or decommissioned and must be removed (and replaced, as necessary).

6.8.4.2. Pipes Exceeding Maximum Deflection. Pipes that have exceeded their maximum percent deflection per Table 4-1 but have not yet collapsed should be rehabilitated (reference the slip lining methodologies describe in Section 7.5.2).

6.8.4.3. Internal Soil Erosion or Voids. If internal soil erosion or voids adjacent to an existing pipe are identified during an inspection or are strongly suspected through the observation of surface depressions or sinkholes, the probability of PFM-1 and PFM-3 occurring eliminates repair, rehabilitation, and decommissioning as reasonable alternatives and the pipe must be removed (and replaced, as necessary). In some cases, pressure grouting of voids adjacent to the pipe may be authorized as an interim risk reduction measure (or for emergency repair) while other alternatives are evaluated; however it must follow the regulations outlined in ER 1110-1-1807. When alternate mitigation options are not practical from a cost or technical perspective, pressure grouting may be the only feasible long-term option.

6.8.4.4. Mitigation Methods. Refer to Chapter 7 for further information on pipe repair and rehabilitation, and Chapter 8 for removal and decommissioning.

6.9. Prioritized Mitigation Plan.

6.9.1. General. Levee sponsors often face the challenge of maintaining or extending the life of project-related pipes using limited resources, and therefore need to prioritize their efforts to address pipes that pose the most risk. USACE and levee sponsors should work together to develop and continuously update a planned approach to manage all levee-related risk, including pipes. Typically, the risk management strategy for pipe mitigation is initially limited to a pipe-focused prioritization plan that would not consider risk from any other sources, a point that must be clearly communicated to the sponsor. This prioritized pipe list can then be used to help determine the order of overall management activities for the levee.

6.9.2. Evaluating Risk.

6.9.2.1. Overview. A pipe-focused risk management strategy prioritizes mitigation efforts by considering several aspects of each pipe and targeting actions that efficiently lower risk rather than relying solely on condition assessment results. As covered in Chapter 2, risk is a function of the probability of a certain hydraulic loading, response of the project to that loading, and the consequences of failure while loaded.

$$\text{Risk} = \overbrace{\text{Likelihood of Loading x Likelihood of Poor Performance}}^{\text{Likelihood of Failure}} \text{ x Consequence of Failure}$$

The term “likelihood of failure” is often used to consolidate the loading and performance terms and is necessary for plotting risk. Based on this equation, the chance of flood loading and the associated consequences are both given equal weight to the condition of the pipe, which means that mitigating a pipe with a better rating over one with a worse rating may provide more risk reduction because that pipe is more frequently loaded and has higher associated consequences.

6.9.2.2. Considerations for Evaluation. To create a preliminary prioritized mitigation plan for the pipes in a levee system, the time and money required to produce a detailed and comprehensive mitigation plan must be replaced by prompt and sound engineering judgment based on available information. The following aspects of the pipe and system must be considered during prioritization to help the levee sponsor manage mitigation funds in the most efficient manner:

6.9.2.2.1. Condition. A pipe with a greater number of higher graded structural defects will have a higher likelihood of causing a levee failure; this includes associated structures and appurtenances (reference Chapter 9). This consideration informs the performance variable and will require the most engineering judgment to evaluate.

6.9.2.2.2. Return Frequency. The flood return frequency for the landside invert elevation will determine when it is possible that water may begin uncontrolled flow into the leveed area through or around a specific pipe. A pipe that is loaded more frequently is usually more likely to fail than one that is loaded less frequently, and is more likely to cause consequences than pipes founded higher in elevation. Once determined, these values should not require reevaluation for future prioritization plans unless significant watershed changes occur.

6.9.2.2.3. Topography. Where detailed inundation mapping is unavailable, it may be necessary to roughly estimate from existing topographic information. Questions to consider include the following: if water does flow through the pipe into the leveed area, will it immediately begin to spread and inundate a wide area? Will it remain within a drainage channel and at what return frequency will the water exceed the channel and begin to inundate a larger area? Are there return frequency break points where inundation significantly changes? And, will flow through pipes located near the upstream end of the system cause overland flow towards the downstream end of the leveed area? These are hydraulic loading considerations that may also impact consequence considerations.

6.9.2.2.4. Location. What is the pipe's proximity to critical infrastructure (i.e., hospitals, schools, airports, military facilities), and could uncontrolled flow cut off evacuation routes or endanger lives? What is the population at risk for different water levels? Is the pipe located on the upstream end where a breach could produce higher flow velocities through populated areas as it follows the downstream grade, or is it near the downstream end of the system where a breach would typically cause backwater flooding to a lower level? These considerations relate to consequences.

6.9.2.2.5. Diameter. Pipes with larger diameters are generally considered harder to flood fight unless an accessible gatewell exists where sandbags or other suitable material can be introduced to stop or greatly reduce the flow. This informs both the performance and loading variables since the percentage of the pipe's cross-sectional area passing water must be considered.

6.9.2.2.6. Filtering. Was an internal seepage filter (reference Section 5.5.9.3) installed around the pipe to address PFM-1? A filter will not stop flow that enters the pipe, but it can address seepage along the pipe.

6.9.2.2.7. Cost. Once developed, the preliminary prioritized pipe mitigation plan must then be balanced with the cost of addressing each pipe and the levee sponsor's budget constraints. Depending on the cost disparities associated with properly addressing each pipe, it is possible that the pipe presenting the most risk will not be addressed first. A simplified pipe-focused prioritized mitigation plan is shown in Figure 6-16.

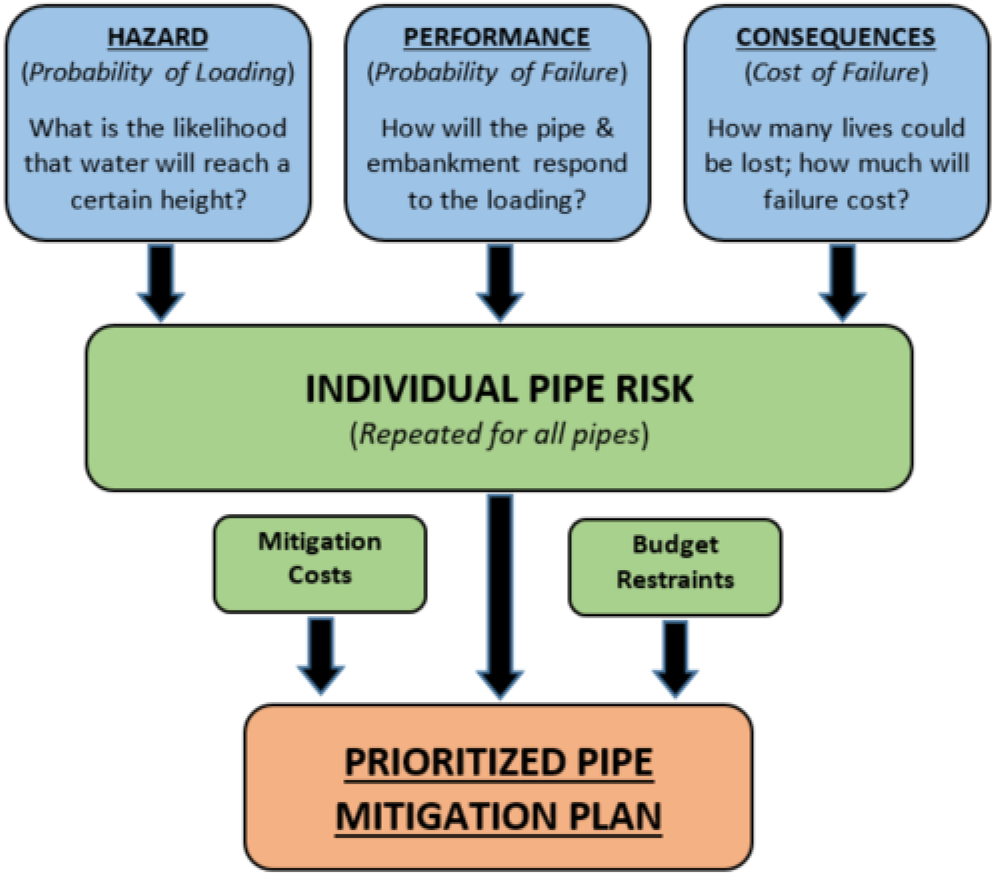
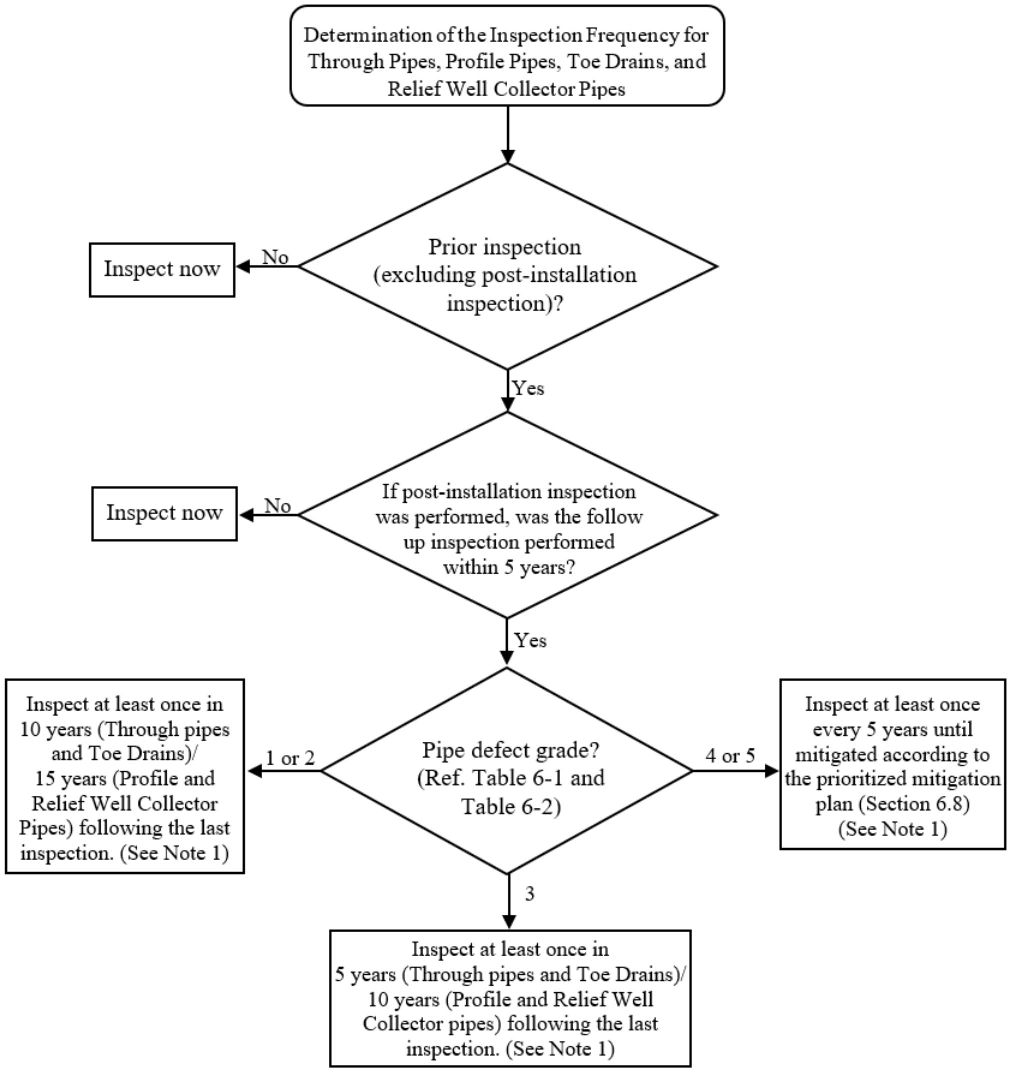


Figure 6-16. Example risk management approach to a prioritized mitigation plan.

6.10. Inspection Frequency.

6.10.1. Essential Pipes. Essential pipes must be inspected at recurring intervals; the limits of those inspections for levees are covered in Section 6.4, dam essential pipes must be inspected in their entirety. The intervals between inspections indicated in Figure 6-17 may be more frequent based on consideration of the factors listed in Section 6.9.2.2, but may not be extended. In addition to these factors, the respective USACE District or levee sponsor may determine an unscheduled inspection is required during a flood event, after a flood or seismic event, or because of sabotage or other unusual events. A post-installation inspection is required immediately after installing a new pipe system, and a follow-up inspection is required within the first 5 years since the continued consolidation of the surrounding soil may shift the pipe alignment and compromise pipe connections. After the follow-up inspection, the frequency for future inspections will be determined by the pipe's condition assessment rating (Figure 6-17). Updates or changes to pipe inspection frequencies must be included in the project's operations and maintenance manual when revised.



Note 1: Reference Section 6.8.2.2. 6.8.2.2. for factors that may be considered to adjust inspection frequencies. The intervals shown in the boxes related to the PACP grades are maximums and must not be exceeded.

Figure 6-17. Inspection frequency determination for essential pipes within levees.

6.10.2. Nonessential Pipes. Typically, nonessential pipes are third-party pipes, such as utility pipes or crossings, that are neither owned nor operated by the levee sponsor or USACE. The most common third-party pipes are for water distribution, force-main sewers, gravity sewers, natural gas distribution, and hazardous liquid transmission. Some third-party pipes serve as casings for utilities (electric, fiber optics, etc.) and cross levees within a larger casing pipe. Neither the utility carrier pipe nor its larger casing pipe requires regular inspections as long as the ends of the larger casing pipe are sealed and the utility carrier pipe is continuous. The inspection requirements and frequency for nonessential pipes are either governed by a specific Code of Federal Regulations (CFR) or a mutually agreed upon arrangement between the pipe owner, the levee sponsor, and USACE. Gas pipelines and hazardous liquids transmission pipelines are regulated under 49 CFR Parts 192 and 195, respectively, and their inspections are determined by the U.S. Department of Transportation Pipeline and Hazardous Materials Safety Administration. Inspection intervals for new third-party pipes crossing a levee that are not regulated under a CFR will be established as part of the approval documents through the 33 USC 408 process. USACE Districts should work with existing third-party pipe owners and the levee sponsor to establish pipe inspection schedules. USACE Districts should use the third-party inspection information to assess and assign the pipe condition ratings and ensure any recommended actions are communicated to the appropriate pipe owner through the levee sponsor.

Chapter 7
Maintenance, Repair, and Rehabilitation

7.1. Introduction. A pipe typically deteriorates as it ages, but proper maintenance, repair, and rehabilitation can extend its service life. A condition assessment, based on a pipe inspection report, will determine which of these actions should be taken (reference Section 6.7). Regular maintenance will typically minimize the need for repairs and rehabilitation can restore a pipe to its original design capacity.

7.2. Potential Failure Modes Related to Maintenance, Repair, and Rehabilitation. An assessment of a pipe's condition or an indication that one of the PFMs described in Chapter 2 has initiated may lead to the need for maintenance, repair, or rehabilitation. Maintenance, such as high-pressure cleaning or root or ice removal, is required to ensure the operational adequacy and promote the longevity of the pipe. However, aggressive maintenance could damage protective coatings or create holes and lead to PFM-3. Likewise, rehabilitation of a gravity pipe by slip lining often increases the outlet flow velocity and could initiate erosion at this end of a gravity drain (PFM-4). Finally, deformed or collapsed pipes that can cause interior ponding (PFM-5) require repair, rehabilitation, or replacement.

7.3. Maintenance.

7.3.1. General. Maintenance of pipes and their appurtenances (e.g., flap gates, slide gates, valves) should typically be routine and planned according to the project's operation and maintenance manual. The most common maintenance activity involves debris removal (cleaning); however, painting of metallic appurtenant parts, lubrication of moving parts, and other minor tasks to keep the pipe and its appurtenances in good working condition are also important.

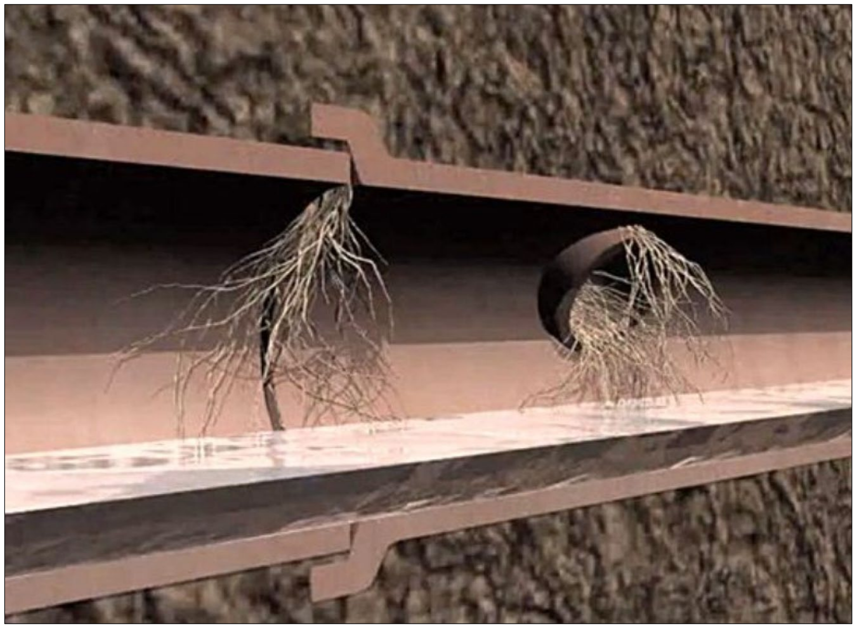
7.3.2. Cleaning. Non-aggressive cleaning following the procedures and restrictions in Table 7-1 removes loose debris, root intrusion, ice accumulation, or biofouling, all of which reduce a pipe's hydraulic capacity and can inhibit appurtenance operation. The magnitude of these issues depends on pipe gradient, water velocity, pipe smoothness, the nature and quantity of debris, bedload material, density of vegetation in the drainage channel, and temperature, among other factors. Careful debris cleaning may be accomplished using small excavators at the pipe inlet/outlet as well as high-pressure flushing, root augers, and ice melting tools within the pipe. Associated structures can be cleaned using vacuum trucks. Hydro-jet nozzles can remove sediment, debris, and deposits (e.g., grease or pipe scale) from a pipe's interior (Figure 7-1). Hot water is more effective for removing biofouling than cold water. Remote cleaning technology is recommended over manual methods to reduce risk of operator injuries.

(From Drains CO, UK, 2019)
Figure 7-1. Hydro-jet nozzle head.

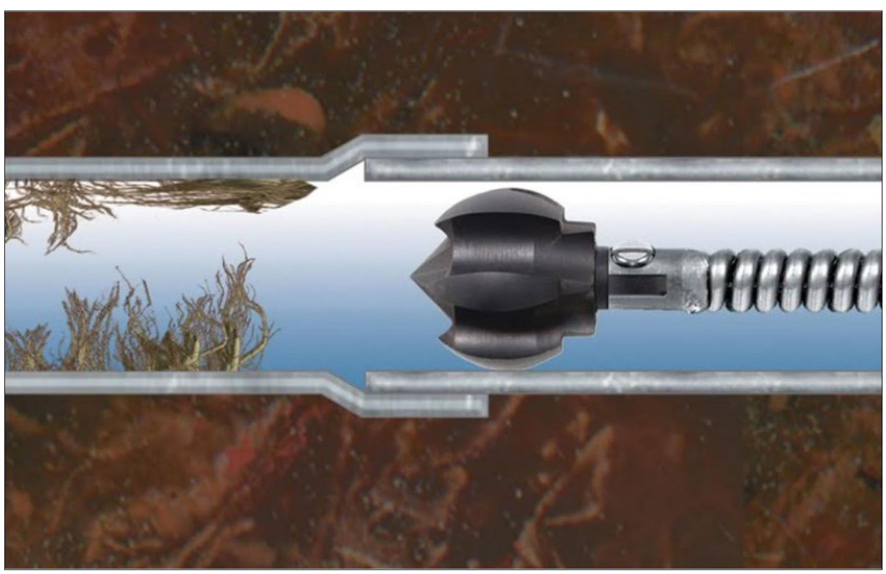
Table 7-1
Typical Pipe Material Cleaning Specifications

Pipe Material	Non- & Low-Pressure Reinforced Concrete (NP-RCP/LP-RCP) and Concrete Pressure (CPP)	Vitrified Clay Pipe (VCP)	Fiberglass Reinforced Pipe (FRP)	Corrugated Steel Pipe (CSP) (galvanized, aluminized or polymer)	Welded Seam Steel Pipe (WSSP) (cement, epoxy or polyurethane lined)	Corrugated Aluminum Pipe (CAP)	Ductile Iron Pipe (DIP)	Plastic Pipe – High Density Polyethylene, Polypropylene and Polyvinyl Chloride (HDPE, PP & PVC)
Cleaning Methods	Jetting; hydro-mechanical, rotational, power rodders, grinding type, and bucket cleaning used when jetting is not fully effective	Jetting; hydro-mechanical, rotational, power rodders, grinding type, and bucket cleaning used when jetting is not fully effective	Jetting; pigging; flushing	Jetting	Jetting using fan jet nozzles	Jetting	Jetting using fan jet nozzles	Jetting using fan jet nozzles
Maximum Nozzle Orientation	Any angle and distance to pipe wall	Any angle and distance to pipe wall	30 degrees to pipe wall and nozzle no closer than 2.5 inches	30 degrees to pipe wall and nozzle no closer than 2 inches	30 degrees to pipe wall and nozzle no closer than 2.5inches	30 degrees to pipe wall and nozzle no closer than 2 inches	30 degrees to pipe wall and nozzle no closer than 2 inches	6 to 15 degrees to pipe wall and nozzle no closer than 2 inches
Nozzle Positioning in Pipe	Center in pipe especially when < 15-inch ID moving in a continuous motion	Center in pipe and no restrictions on stationary nozzle dwell time	Center in pipe and move in a continuous motion	Center in pipe and move in a continuous motion	Center in pipe and move in a continuous motion	Center in pipe and move in a continuous motion	Center in pipe and move in a continuous motion	Forward facing nozzles and not to exceed 60 seconds in a stationary position
Maximum Water Pressure (psi)	3000	5000	See Special Notes	1800	1800	5000	1800	See Special Notes
Flow Rate (gpm) or Nozzle Movement Speed (fpm)	100 gpm	125 gpm; fpm must be determined by equipment operator	32 – 65 fpm	20 – 30 fpm	20 – 30 fpm	20 – 30 fpm	20 – 30 fpm	40 fpm
Steam Cleaning	Contact the manufacturer	Contact the manufacturer	NOT RECOMMENDED	< 250 degrees F	< 250 degrees F	< 250 degrees F	< 250 degrees F	NOT RECOMMENDED
Special Notes	Additional passes to ensure effective cleaning may be required	Additional passes to ensure effective cleaning may be required.	Reference: Industry Guideline for Water Jet Cleaning of Plastic Pipes, July 2009; improve cleaning by increasing water amount and nozzle sizes used	Improve cleaning results by increasing amount of water used, not increasing pressure which requires increase in size & number of nozzle inserts	Take care at joints to not dislodge liner material	Improve cleaning results by increasing amount of water used, not increasing pressure which requires increase in size and number of nozzle inserts	Additional passes to ensure effective cleaning may be required. Reference guidelines for Pressure Cleaning the Internal Diameter of DIP, DIPRA, May 2016	Reference: Industry Guideline for Water Jet Cleaning of Plastic Pipes, July 2009

7.3.3. Root Removal Using Augers. The use of mechanical augers within a pipe is the best way to remove roots; however, this is typically only a temporary measure since their presence indicates cracks or openings in the pipe's joints or connections to taps, which may also have allowed the movement of soil into the pipe (PFM-3). The installation of a liner pipe is a proven way to prevent roots from continually intruding through defects in the original/host pipe. Figure 7-2 and Figure 7-3 show root intrusion and a root auger.



(From Euro Plumbing & Sewer, LLC, 2017)
Figure 7-2. Pipe root intrusions.



(From General Pipe Cleaners, 2019)
Figure 7-3. Root auger within pipe.

7.3.4. Ice Removal. Steam, hot water flushing, and electric heaters can be used to remove ice from pipes and gates in cold weather environments. In addition, bubbler systems can help preemptively prevent water from freezing within pipes or gatewell structures. Ice buildup reduces or completely eliminates a pipe's hydraulic capacity (PFM-5) and can prevent gate operations (PFM-7). Figure 7-4 shows severely restricted flow due to a loss in a pipe's cross section, and Figure 7-5 is an example of a slide gate seized by ice. Ice removal is typically performed in preparation of spring flooding and not during the winter months.

(Courtesy of USACE St. Paul District)
Figure 7-4. Ice-filled pipe.

(Courtesy of USACE St. Paul District)
Figure 7-5. Ice preventing gate from opening.

7.4. Repair.

7.4.1. General. Repairs to existing pipes are performed when a pipe inspection and condition assessment (reference Section 6.7) reveal an issue that can adversely impact the integrity of the embankment or floodwall (reference Section 6.3). Issues detected outside of the influence zone should also be addressed when they can impact the proper operation of the dam or levee system (e.g., a defect between the headwall and the influence zone that could allow flooding of the interior). Repairs temporarily correct relatively small, discontinuous areas of distress such as corrosion, concrete spalling, exposed concrete rebar, open joints, and minor cracking and are intended to extend the service life of a pipe. A repair may be necessary when time or money restrictions prevent a longer-term correction; however, the pipe must function adequately until a decision about rehabilitation, replacement, or decommissioning can be made. Repairs to brick pipes are not allowed; they must be rehabilitated, replaced, or decommissioned.

7.4.2. Reinforced Concrete Pipe (RCP) and Concrete Pressure Pipe (CPP) Repairs. The conditions and associated recommended repairs for various issues with RCP and CPP are shown in Table 7-2. In addition to the repair methods listed in Table 7-2 specified for CPP, it is also possible to repair a considerable length using internal or external application of an epoxy-saturated carbon fiber. Because pipes are most often buried, more common repairs made with carbon fiber are done from the interior. American Water Works Association (AWWA) C305 covers the repair of pre-stressed concrete cylinder pipe using carbon fiber. Similar applications listed in AWWA C305 can also be used for other types of CPP. For applications in pipes within or beneath dams and levees, a decision to apply carbon fiber at or over joints in the pipeline must be based on whether future settlement and movement of joints is likely to occur.

Table 7-2
Repairs for RCP and CPP

Condition	Considerations	Repair Methods*
Crack or fractures	• Width, length, location, environmental conditions. • Small cracks (less than 0.05-inches wide) and cracks that do not penetrate the wall typically need no remediation. • Cracks (fractures) larger than 0.05 inches in width are candidates to be sealed to protect reinforcement from degradation. • For RCP, see ASTM C1840 for complete evaluation criteria; for CPP, see American Concrete Pressure Pipe Association (ACPPA) Repair Guide at ww.acppa.org.	• Non-shrink cementitious grout • Chemical grout (RCP only) • Epoxy adhesives • Flexible sealants
Joint – end damage (cracks/ fractures or areas of joint broken or missing)	• Width of crack/fracture, size of damaged area. • If joint sealing surface is compromised, repair is required. • For RCP, see ASTM C1840 for evaluation criteria and further repair details. • For CPP, joint must be made suitable for the pipe working pressure by gasket sealing or welding; see ACPPA Repair Guide at www.acppa.org.	• Joint seals (typically RCP) • Preformed mastic sealant (RCP only) • Chemical grout (RCP only) • Foam grout (RCP only) • Gel grout (RCP only) • Gasket/or weld joint ring (CPP only)
Joint – separation (ends of pipe separated)	• For RCP: Joint gap exceeding 0.75A in gasket pipe must be repaired as long as no embankment subsidence above (A = *Note). • For CPP: Consider if flexibility must be maintained at the joint.	• Joint seals (typically RCP) • Preformed mastic sealant (RCP only) • Chemical grout (RCP only) • Foam grout (RCP only) • Gel grout (RCP only) • Welded butt strap (CPP only)
Joint infiltration	• Remediate any joint experiencing infiltration as long as no embankment subsidence above. • For RCP, use the joint evaluation criteria in ASTM C1840 for other joint issues.	• Joint seals (typically RCP) • Preformed mastic sealant (RCP only) • Chemical grout (RCP only) • Foam grout (RCP only) • Gel grout (RCP only) • Weld joint ring/butt strap (CPP only)
Wall damage (slabbing, spalling)	• Size and depth of damage. • Damage that exceeds the depth of the innermost reinforcement layer (or penetrates the cylinder of CPP) must be remediated or replaced.	• Cementitious materials to reestablish pipe wall thickness • Polymer coatings • Patch weld cylinder (CPP only)
Hole through wall	• Repair is always required as long as no embankment subsidence above.	• Joint seals (typically RCP) • Preformed mastic sealant (RCP only) • Chemical grout (RCP only) • Foam grout (RCP only) • Gel grout (RCP only) • Patch weld cylinder (CPP only)

* Post Installation Evaluation and Repair of Installed Reinforced Concrete Pipe (consult American Concrete Pipe Association for additional information on RCP pipe repairs and ASTM C1628-05); “A” represents the maximum separation distance at which the gasket remains compressed between the two pipe sections.

7.4.3. Vitrified Clay Pipe (VCP) Repairs. The conditions and associated recommended repairs for various issues with VCP are shown in Table 7-3.

Table 7-3
Repairs for VCP

Condition	Considerations	Repair Methods*
Cosmetic imperfections (i.e., surface cracks, scratches, chips, blisters or laminations)	• No repair necessary.	• No matter the age of the pipe, no action is needed unless soil or infiltration is visible.
Crack or fracture with infiltration	• Repair always required as long as no embankment subsidence above.	• Chemical grout is an option to repair pipe with water infiltration. Follow grout manufacturer's recommendations.
Crack or fracture with soil visible	• Repair is not permitted; replacement is always required.	• When soil is visible in the pipe, replacement of the damaged pipe section is required. A shielded rubber coupling (Figure 7-6) is used to connect the new section of pipe to the existing sections. Follow grout manufacturer's recommendations.

*Consult the manufacturer for additional details on repair methods and replacement requirements for VCP in addition to the National Clay Pipe Institute (NCPI), Vitrified Clay Pipe Engineering Manual, 2017.

(Courtesy of Mission Rubber Company)
Figure 7-6. Shielded rubber coupling.

7.4.4. Corrugated Steel Pipe (CSP) Repairs. The conditions and associated recommended repairs for various issues with CSP are shown in Table 7-4.

Table 7-4
Repairs for CSP

Condition	Considerations	Repair Methods
Corrosion with no holes	• Repair	• Spray on protective coating, bituminous coating, or concrete paving.
Wall separation at riveted or bolted joints	• Repair	• Joint sealing using injected polyurethane grout.
Deteriorated inverts	• Repair as long as no embankment subsidence above; if > 5% ring deflection, remove and replace.	• Steel patch – divert water, clean pipe, and internally brace areas to prevent deformation or collapse during repair. • Reinforced concrete patch – divert water, clean pipe, lay reinforcement, and add concrete paving. • If necessary, internally brace areas to prevent deformation or collapse during repair.

7.4.5. Corrugated Aluminum Pipe (CAP) Repairs. The conditions and associated recommended repairs for various issues with aluminum pipe are shown in Table 7-5.

Table 7-5
Repairs for CAP

Condition	Considerations	Repair Methods*
Crack or tear	• Repair is always required.	• Weld according to AWS D1.2.
Hole	• Repair as long as no embankment subsidence above; if so, replace.	• Install a rivet or a bolt with a neoprene washer in the hole. Rivets and bolts must comply with ASTM B745, or replace pipe.
Gouge or scratch	• If remaining wall thickness < ASTM B744 Table 1, repair.	• Weld according to AWS D1.2.
Loose or damaged bolts or rivets at seams	• Repair as long as no embankment subsidence above; if so, replace.	• Replace the rivet or bolt with one that complies with ASTM B745 along with neoprene washer or replace pipe.

*AWS D1.2 is the American Welding Society's D1.2 Structural Welding Code – Aluminum.

7.4.6. Welded Seam Steel Pipe (WSSP) Repairs. The conditions and associated recommended repairs for various issues with WSSP are shown in Table 7-6).

Table 7-6
Repairs for WSSP

<table>
<tr><th>Condition</th><th>Considerations</th><th>Repair Methods</th></tr>
<tr><td>Damaged cement mortar lining</td><td>• Hairline cracks need no repair.</td><td>• N/A</td></tr>
<tr><td>Damaged cement mortar lining</td><td>• Large cracks or missing lining as defined in AWWA C205.</td><td>• Repair per AWWA C205.</td></tr>
<tr><td>Damaged polyurethane lining or coating</td><td>• As described in AWWA C222.</td><td>• Repair per AWWA C222.</td></tr>
<tr><td>Damaged epoxy lining or coating</td><td>• As described in AWWA C210.</td><td>• Repair per AWWA C210.</td></tr>
<tr><td>Damaged polyethylene coating</td><td>• As described in AWWA C209 and C214.</td><td>• Repair per AWWA C209 and C214.</td></tr>
<tr><td>Gouges</td><td>• Measure thickness and assure the remaining thickness is adequate for design.</td><td rowspan="3">• Depending on severity of condition – use one of the following:
• Small areas can frequently be repaired using a plug weld (with or without a backing plate). Guidelines are dictated by the weld procedure chosen.
• Areas too large to plug weld can be repaired by either welding a cover plate over the area using a fillet weld or cutting out the area and welding in a patch plate using a complete joint penetration (CJP) weld. Guidelines are dictated by the weld procedure chosen.</td></tr>
<tr><td>Holes</td><td>• Pipe may be repaired depending on pressure design, size of hole, and whether there is embankment subsidence above.</td></tr>
<tr><td>Corrosion</td><td>• Surface corrosion may be cleaned and recoated. A qualified corrosion engineer should be consulted if there are pervasive corrosion issues within the piping system. Repairing corrosion damage without addressing the source of corrosion is not recommended.</td></tr>
</table>

7.4.7. Ductile Iron Pipe (DIP) Repairs. The conditions and associated recommended repairs for DIP are shown in Table 7-7.

Table 7-7
Repairs for DIP

Condition	Considerations	Repair Methods
Circumferential/"crack around" beam break	• Repair as long as no embankment subsidence above and diameter of pipe allows for man-entry repair.	• Install a mechanical joint repair sleeve/clamp.
Longitudinal split or crack	• Repair as long as no embankment subsidence above and diameter of pipe allows for man-entry repair.	• Install a mechanical joint repair sleeve/clamp.
Pitted pipe or corrosion "blow-out hole"	• Repair as long as no embankment subsidence above and diameter of pipe allows for man-entry repair.	• Install a mechanical joint repair sleeve/clamp and encase all repair products as well as existing exposed piping per AWWA C105/A21.5.
Holes and/or punctures	• Repair as long as no embankment subsidence above and diameter of pipe allows for man-entry repair.	• Install a mechanical joint repair sleeve/clamp.
Leaking bell/spigot connection	• Repair as long as no embankment subsidence above and diameter of pipe allows for man-entry repair.	• Install bell joint leak clamps internally.
Leaking mechanical joint/cracked bell	• Repair as long as no embankment subsidence above and diameter of pipe allows for man-entry repair.	• Install a mechanical joint bell repair sleeve.

7.4.8. Thermoplastic – Polyvinyl Chloride (PVC), High-Density Polyethylene (HDPE), and Polypropylene (PP) Repairs. The repair methods associated with solid-wall PVC and HDPE pipes and profile-wall HDPE and PP pipes are shown in Table 7-8 and Table 7-9.

Table 7-8
Repairs for solid-wall PVC and Solid Wall HDPE pipe

Condition	Considerations	Repair Methods
Puncture in both pipe walls	• Repair is not permitted; remove and replace.	• Remove damaged section and install new pipe with two electro-fusion (EF) couplings; reference Figure 4.* • Remove damaged section and reinstall new pipe with two pullout restraint couplings with internal stiffeners; reference Figure 18.*
Damaged section	• Repair is not permitted; remove and replace.	• Remove and replace with EF saddle or replace pipe segment with new section and two EF couplings; reference Figures 3* or 5.* • Remove saddle or replace; reference Figures 16,* 17,* and 23.*

*For more information on repairs along with details/figures, refer to Municipal Advisory Board - MAB-4, Basic HDPE Repair Options.

Table 7-9
Repairs for Profile Wall HDPE and Profile Wall Polypropylene (PP)

Condition	Considerations	Repair Methods*
Excessive joint gap, rolled gasket, or damaged joint	• Repair	• Joint seal • Welding • Pressure injection of an acceptable flexible chemical grout
Cracks	• Repair	• Joint seal • Welding • Mechanical repair sleeve
Punctures or minor damage	• Damage should not exceed the width of one corrugation; if it does, repair.	• Joint seal • Welding • Pressure injection of an acceptable flexible chemical grout • Mechanical repair sleeve • Mastic banding • Concrete collars • Split band coupler • Slip coupler

*Consult pipe manufacturer for specific details associated with the repair method selected.

7.4.9. Fiberglass Reinforced Pipe (FRP) Repairs. Typically, FRP manufacturers assist with determining the severity of the damage along with the repair requirements. The repairs associated with FRP pipes are shown in Table 7-10.

Table 7-10
Repairs for FRP

Condition	Considerations	Repair Methods
Impact crack or craze	• Repair is usually required.	• Grind damage and repair with a glass and resin layup per the manufacturer's recommendations.
Hole	• Repair is always required.	• Repair the structural integrity with a glass and resin layup per the manufacturer's recommendations.
Gouge or scratch	• Interior scratches penetrating through the liner or exterior scratches/gouges penetrating into the glass layer must be repaired.	• Grind damage and repair with a glass and resin layups per the manufacturer's recommendations.

7.4.10. Repair for Pipe-Related Seepage. If seepage around either end of an existing pipe shows evidence of soil particle movement, an external seepage filter can be used. External seepage filters serve the same function as internal seepage filters, but are used as a surficial remediation for existing pipes where more invasive and complicated construction of an internal seepage filter may not be warranted. Reference Section 5.5.9.4 and Figure 5-31 for information on the use of external seepage filters.

7.5. Rehabilitation.

7.5.1. General. Rehabilitation of a pipe is performed when the pipe is systemically deteriorated and where it is supported by the condition assessment process (reference Section 6.7). Rehabilitation is typically less expensive than pipe replacement since it does not involve disturbing the existing embankment; it also restores the hydraulic capacity to near its original condition. The selection and design of an appropriate pipe rehabilitation method is dependent on existing pipe condition, its hydraulic capacity, and the purpose of the pipe within an embankment. Pipe rehabilitation is typically conducted using a trenchless method involving the use of internal liners such as slip lining, spray-in-place (SIPP) liners, close-fit liners, and pipe bursting and splitting. However, not all rehabilitation methods are acceptable by USACE for use in pipes that penetrate embankments (reference Section 7.5.4.1).

7.5.2. Slip Lining.

7.5.2.1. Background. Slip lining is the most common type of trenchless rehabilitation and is acceptable for use where continuous or segmental pipe can be inserted by either pulling or pushing it into the host pipe and the minimum clearances for grouting the annular space between the host and liner pipe can be maintained. Slip lining can be used in cases where the host pipe is deformed, badly damaged from corrosion, or has offset joints or cracks allowing infiltration/exfiltration. However, if there are significant distortions of the host pipe, a much smaller-diameter liner may be needed that would not provide adequate discharge capacity; open cut and replacement of the pipe is required in such cases. USACE-approved materials for slip lining are spiral wound PVC, solid-wall HDPE, and FRP. Other materials may be used per the approval method described in Section 1.2.

7.5.2.2. Design Requirements and Assumptions.

7.5.2.2.1. Hydraulic Capacity. The liner's hydraulic capacity must meet the needs of the draining area while maintaining a one-inch minimum average annular space between the host pipe and liner to allow for grouting. If these requirements are not met, an open cut and replacement or decommissioning of the host pipe is necessary. The Manning's roughness coefficient, along with the hydraulic capacity of the host and liner pipes, must be evaluated to determine the pre- and proposed post-rehabilitation hydraulic capacity. Laboratory Manning's roughness coefficients for new pipe materials are provided in Chapter 3 (Table 3-1b).

7.5.2.2.2. Structural Capability. It is assumed that the host pipe will no longer provide any structural capability; therefore, the slip liner pipe must carry the full in-situ loads.

7.5.2.2.3. Live and Dead Load Effects. Live and dead load effects on the slip liner must be determined when the pipe is located under an embankment and/or with vehicular traffic, using the water loading to the top of the levee to determine the external hydrostatic pressure. Reference Sections 4.15 and 4.16 for the applicable load calculations for the liner material to be installed.

7.5.2.2.4. Buckling and Flotation. Buckling and flotation potential due to grout pressures must be checked.

7.5.2.2.5. Allowable Deflection. Allowable long-term deflection of the slip liner must be five percent or less of the original liner diameter (reference Table 4-1). Reference Sections 4.15 and 4.16 for the applicable load calculations for the liner material to be installed.

7.5.2.2.6. Funneled Inlets. If the liner pipe is determined to be inlet-controlled, there must be a funneled end or a modified inlet added to the headwall to assist in facilitating the water into the pipe since the invert will be higher after re-lining (Figure 7-7 and Figure 7-8). Figure 7-9, Figure 7-10, and Figure 7-11 are examples of funneled inlets that can be attached to a liner pipe or an inlet headwall.

7.5.2.2.7. Exit Velocities. If the liner pipe is determined to be outlet-controlled, an evaluation of the exit velocities must be conducted to design the riprap protection, increase the size/thickness of the existing outlet riprap, or add energy dissipaters/baffle blocks in the outlet stilling pad.

7.5.2.2.8. Thermal Expansion and Contraction. Thermal expansion and contraction for liner pipes must be considered during design to accommodate this type of movement at the headwalls.

7.5.2.2.9. Gateswells and Headwalls. Where a slip liner connects to a gatewell or headwall, it cannot project beyond the end of the host pipe. Trimming of the liner pipe must be delayed for 28 days after completion of grouting.

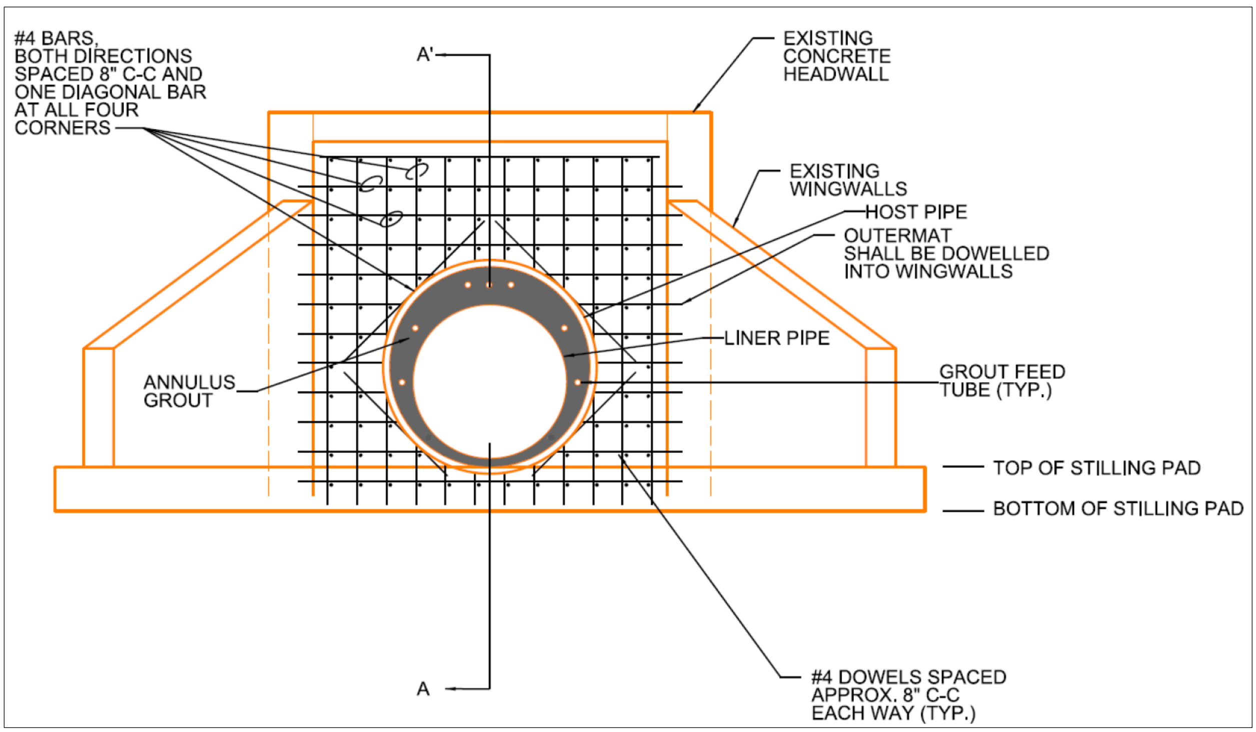


Figure 7-7. Elevation view of a modified inlet for a slip lined pipe.

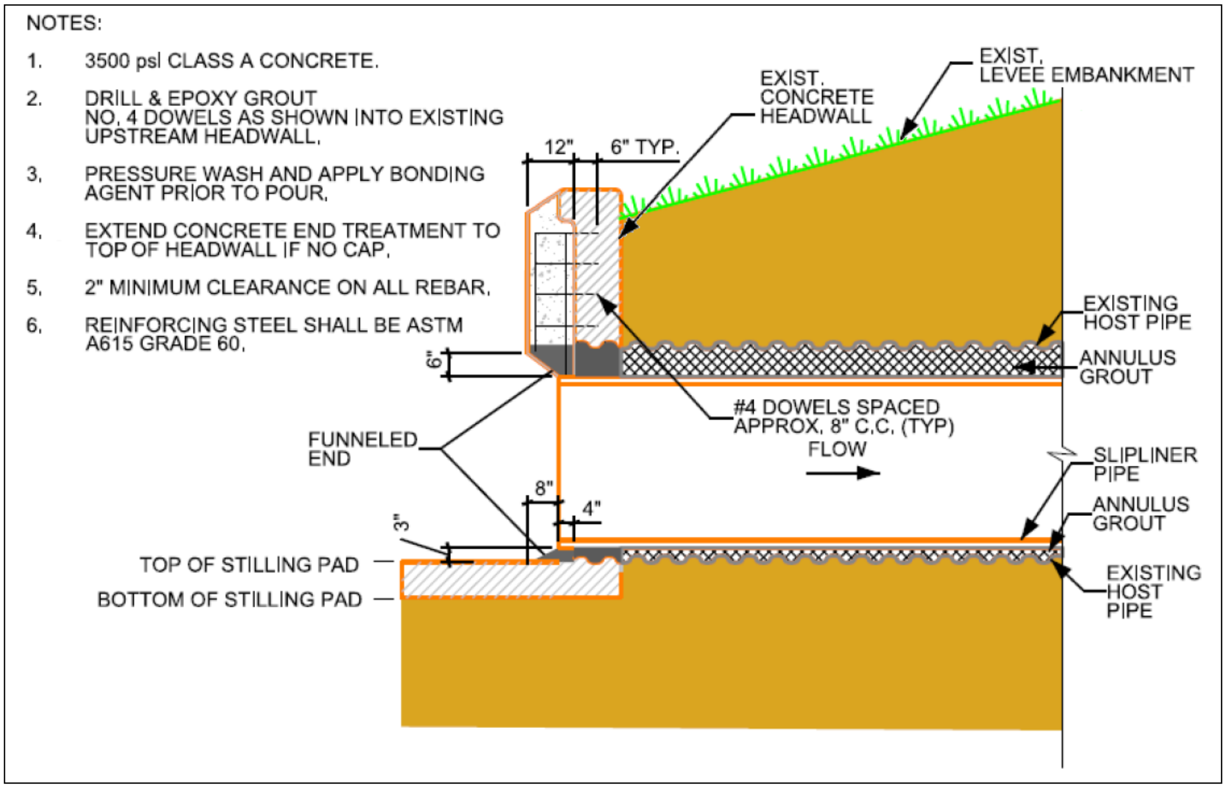
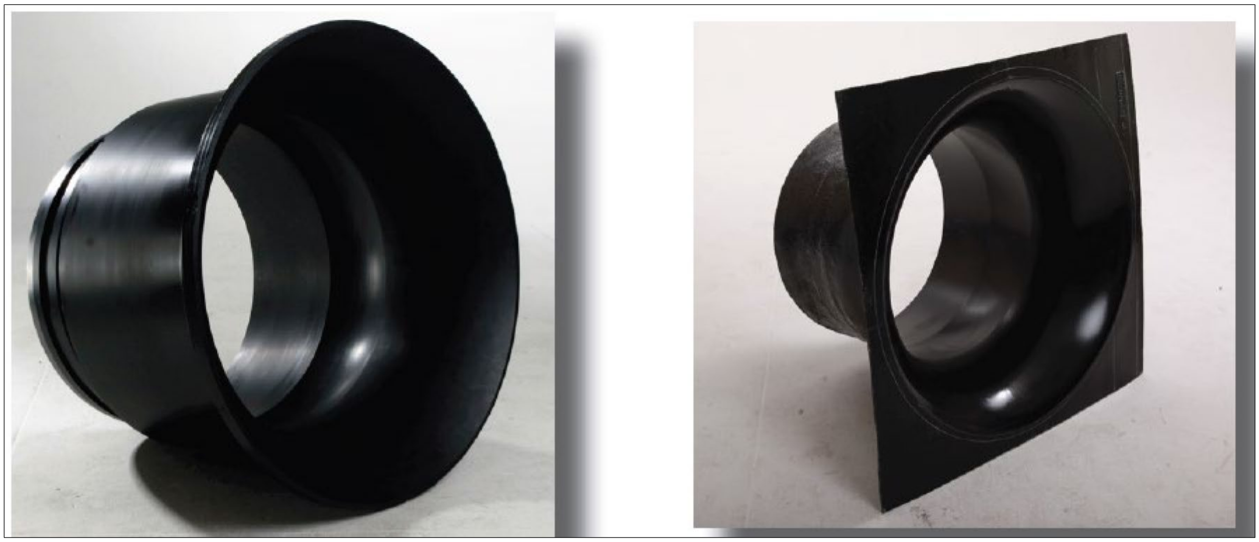


Figure 7-8. Section A-A' view of a modified inlet.

(Courtesy of USACE Louisville District)

Figure 7-9. Finished modified inlet for a slip-lined pipe.



(From Snap-Tite/ISCO Industries, 2019)
Figure 7-10. Types of Snap-Tite Hydro-Bells.

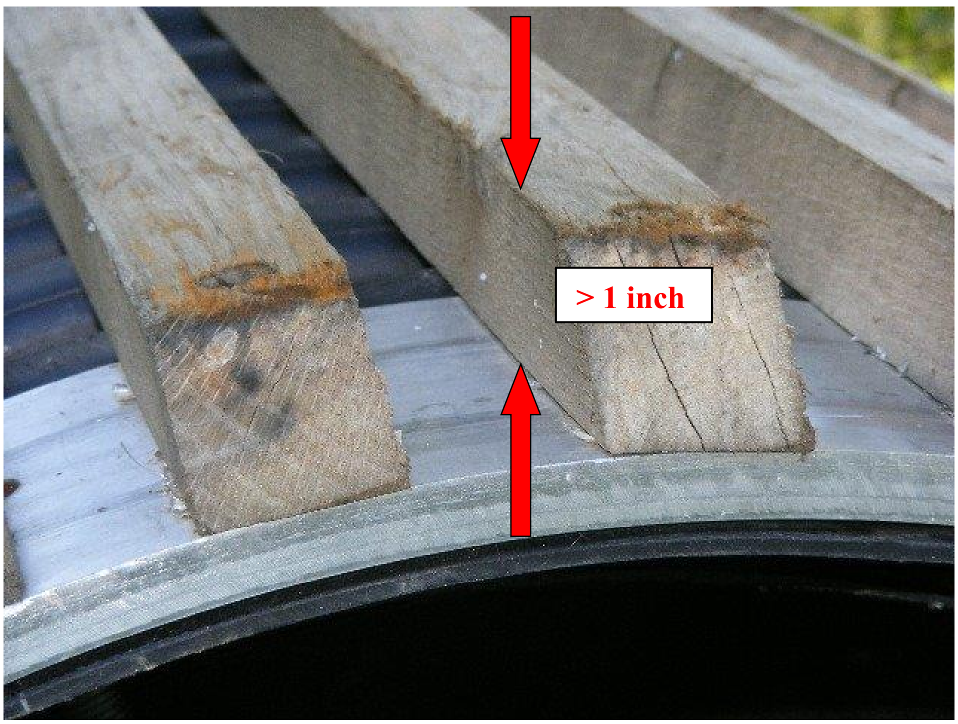
(Courtesy of Indiana Reline)
Figure 7-11. Inner flow maximizer.

7.5.2.3. Preparation.

7.5.2.3.1. Water Control Measures. Measures to control or bypass water normally flowing through a host pipe must be taken so that slip lining can be performed with both the host pipe and liner pipe dewatered and relatively dry.

7.5.2.3.2. Cleaning. The host pipe must be thoroughly cleaned (reference Section 7.3.2.) prior to installing the slip liner to remove debris and protrusions to reduce friction during liner insertion.

7.5.2.3.3. Clearance Limits for Host Pipes Smaller than 36-inches. The clearance limits for a liner pipe fitting inside a host pipe smaller than 36 inches in diameter must be determined using a liner pipe mandrel. A liner pipe mandrel must be of the same pipe material and joint length as the proposed slip liner, and its diameter must be a minimum of two inches greater than the proposed slip liner pipe outside diameter, which can be accomplished using wood strips (Figure 7-12 and Figure 7-13). The liner pipe mandrel is pulled through the entire host pipe to ensure that the diameter and alignment of the host pipe are sufficient to allow insertion of the slip liner (Figure 7-14). The segment of liner pipe used as a mandrel may not later be used as a permanent slip liner segment.



(Courtesy of USACE Louisville District)
Figure 7-12. Wood strips along outside of mandrel.

(Courtesy of USACE Louisville District)
Figure 7-13. Typical mandrel pipe segment with one-inch thick wood strips to ensure minimum space for grout placement.

(Courtesy of USACE Louisville District)
Figure 7-14. Mandrel insertion into host pipe to be slip lined.

7.5.2.3.4. Clearance Limits for Host Pipes 36-inches or Larger. The clearance requirements for a liner pipe fitting inside a host pipe 36 inches in diameter or larger can be verified through man-entry by measuring the diameter at 10-foot intervals within the pipe from the 12:00 to 6:00 position, 3:00 to 9:00 position, and two other measurements midway between them (i.e., 1:30 to 7:30 and 4:30 to 10:30). These measurements can be conducted by laser profiling or by liner pipe mandrelling as stated above to ensure that the diameter and alignment of the host pipe are sufficient to allow insertion of the slip liner. Diameter measurements on corrugated pipe are made from the innermost-to-innermost corrugations.

7.5.2.3.5. Wood Blocking. Wood blocking must be placed in the crown of the host pipe prior to insertion of the slip liner to maintain the desired spacing during grouting operations (Figure 7-15). In addition, wood blocking can be placed under the pipe to aid with insertion of the slip liner and to ensure the invert is as low as possible in the host pipe while maintaining the minimum one-inch annular space to allow for the liner pipe to be fully encapsulated by grout.

(Courtesy of USACE Louisville District)

Figure 7-15. Blocking in crown area of host pipe.

7.5.2.3.6. Intermittent Shims. Intermittent shims installed along the invert of a bituminous-coated host pipe or use of a sub-aqueous lubricant should be used to lessen the difficulty of liner insertion during warm weather due to the softening and adhesion of the coating material. Lubricants must be approved by the pipe manufacturer.

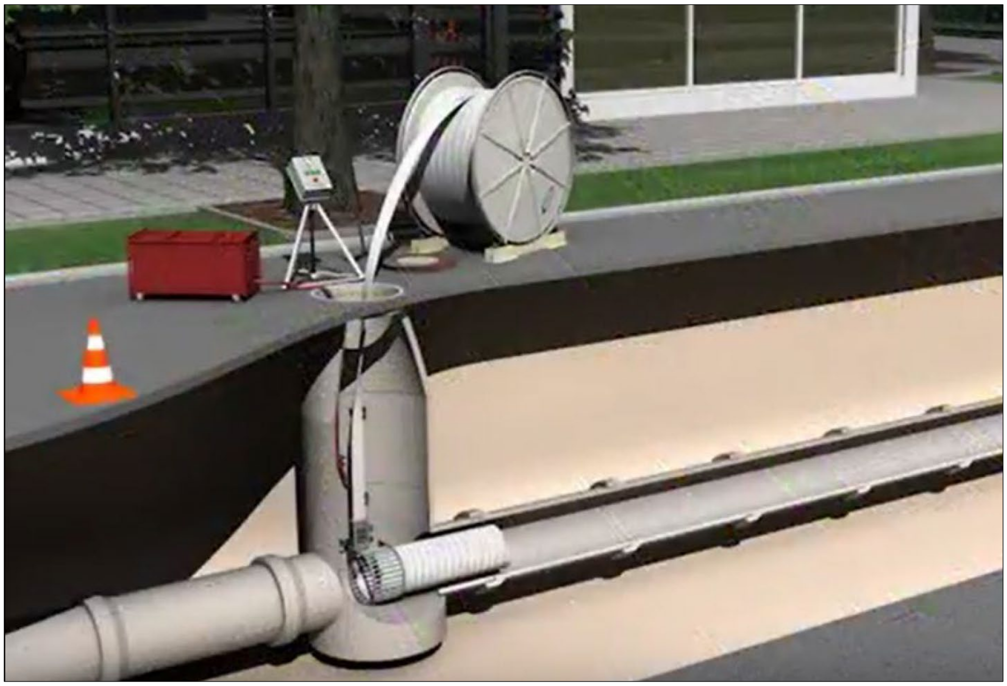
7.5.2.4. Continuous Liner Pipe Method.

7.5.2.4.1. Overview. Fixed-diameter spiral wound pipe lining, or the continuous liner pipe method, involves insertion of continuous PVC profile strips (reference ASTMs F1697, F1698, and F1741) fed from a spool inside a host pipe and is suitable for pipes six to 180 inches in diameter. This is accomplished using either: 1) a fixed-location spiral winding machine (for pipes 6 to 42 inches in diameter) placed at the inlet headwall (ground surface or manhole access) of the host pipe so that the liner is pushed through to the other end, typically down the decline from landside to riverside to get a gravity assist (Figure 7-16 and Figure 7-17); or 2) a mobile winding machine (for pipes 32 to 200 inches in diameter, both round and non-round) is placed inside the pipe and the machine is moved through the host pipe as the liner is wound directly into place (Figure 7-18 and Figure 7-19). Both methods build the liner in one operation and in live flow conditions, at a fixed diameter and with a required minimum gap for grouting.

7.5.2.4.2. Continuous Liner Pipe Details. When installing a continuous liner pipe, the joints consist of a single mechanical interlock between profile strips supplemented with sealant that is connected continuously as the profile PVC liner is wound into the pipe. The inside and outside surfaces of the spiral wound pipe are flush to help prevent the liner from getting caught inside the deteriorated host pipe, and the liner must have a lower Manning's roughness coefficient than the host pipe to maintain the required flows. The joints must meet the leakage requirements for pipes within or beneath a dam or levee (reference ASTM F1735). Spiral wound slip lining is often used for odd shapes or ovality and tight curves, when hydraulic capacity analysis permits the use of a liner pipe whose diameter is significantly less than the host pipe (to avoid snagging), or when access to the host pipe is difficult, such as from a man-hole.

(Courtesy of USACE Louisville District)

Figure 7-16. Setup at headwall access to install a fixed-diameter spiral wound pipe liner.



(From Sekisui SPR Americas, LLC, 2018)
Figure 7-17. Manhole access for installation of fixed-diameter spiral wound liner.

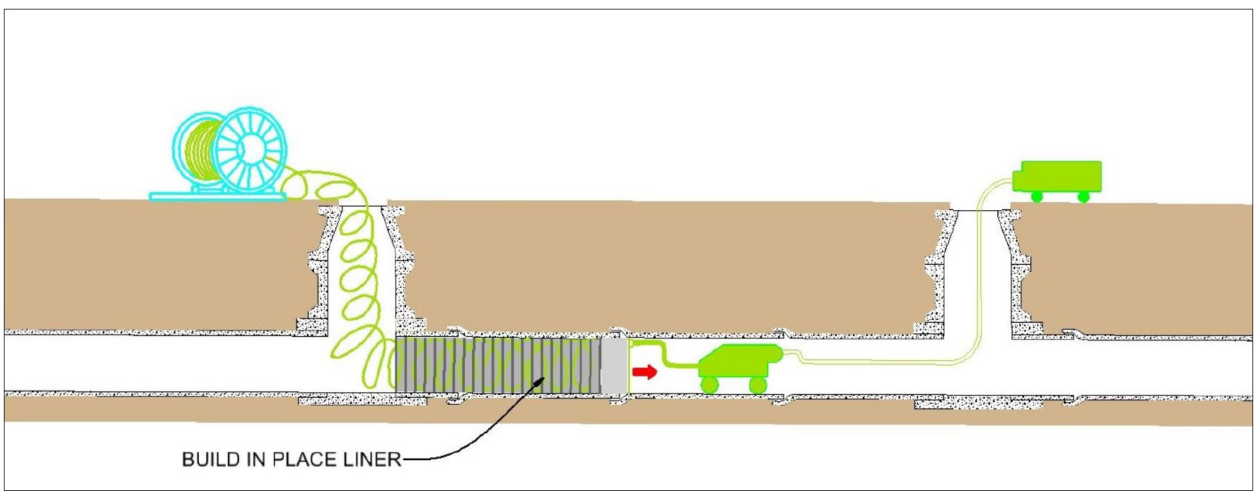
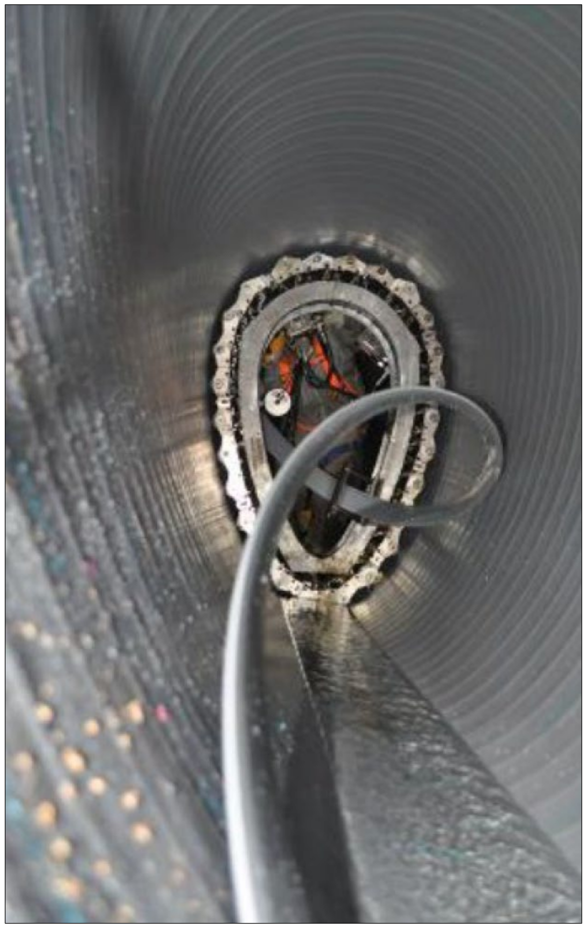


Figure 7-18. Typical setup for a mobile/transverse winding spiral liner installation.



(From Sekisui SPR Americas, LLC, 2019)
Figure 7-19. Non-round mobile/transverse spiral wound liner installation.

7.5.2.5. Segmental Liner Pipe Method.

7.5.2.5.1. Overview. Segmental slip lining is performed using discrete pipe joint lengths, often of 10 or 20 feet. Individual segments of pipe must be placed in alignment with the preceding segment, joined together by threaded, snap together, or welded joints segment by segment, and pushed into the host pipe one at a time. Connections between pipe segments are flush and do not increase the outside diameter nor reduce the inside diameter of the slip liner. Figure 7-20 shows a segmental pipe being installed from a headwall access location. Figure 7-21 is a cross-section of a typical slip-lined pipe and Figure 7-22 shows an actual segmental liner being installed.

7.5.2.5.2. Segmental Liner Pipe Requirements. USACE-approved segmental liner pipe materials include: 1) solid-wall HDPE pipe (ASTM F585, F714, and D3350) with smooth interior and exterior surfaces joined by either butt fusion welds (ASTM D3261 and D2657) or push-together joints with gasketed interlocking machined grooves (ASTM D3212 and ASTM F477); and 2) FRP pipe (ASTM D3262, Piping Material – Cell-Type 1, Liner 2, Grade 3). Joints must be tight enough to prevent grout from entering inside of the slip liner and must meet the performance requirements of ASTM D4161.

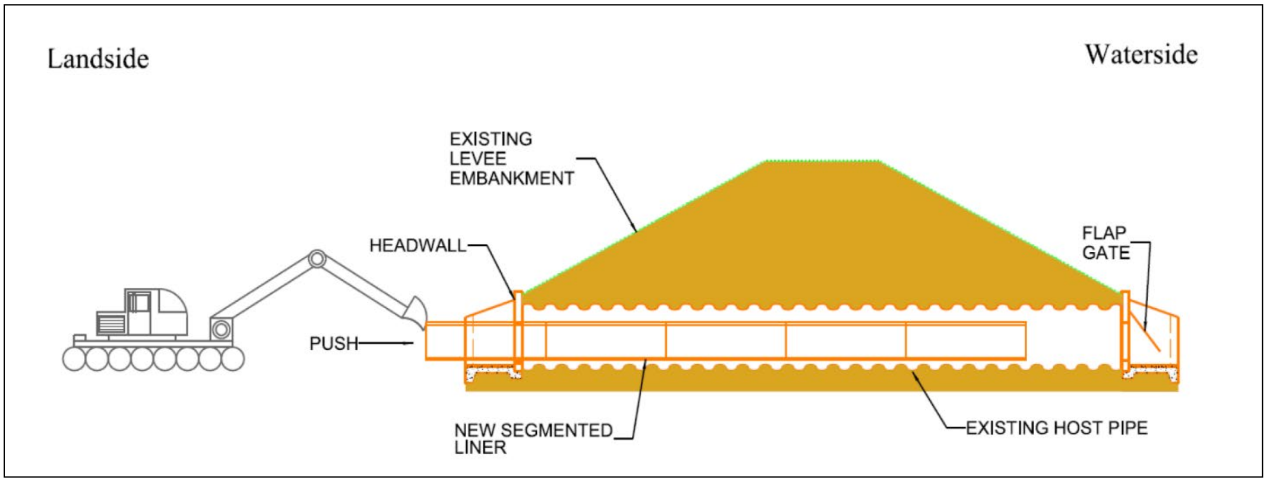


Figure 7-20. Typical headwall access for installation of a segmental liner.

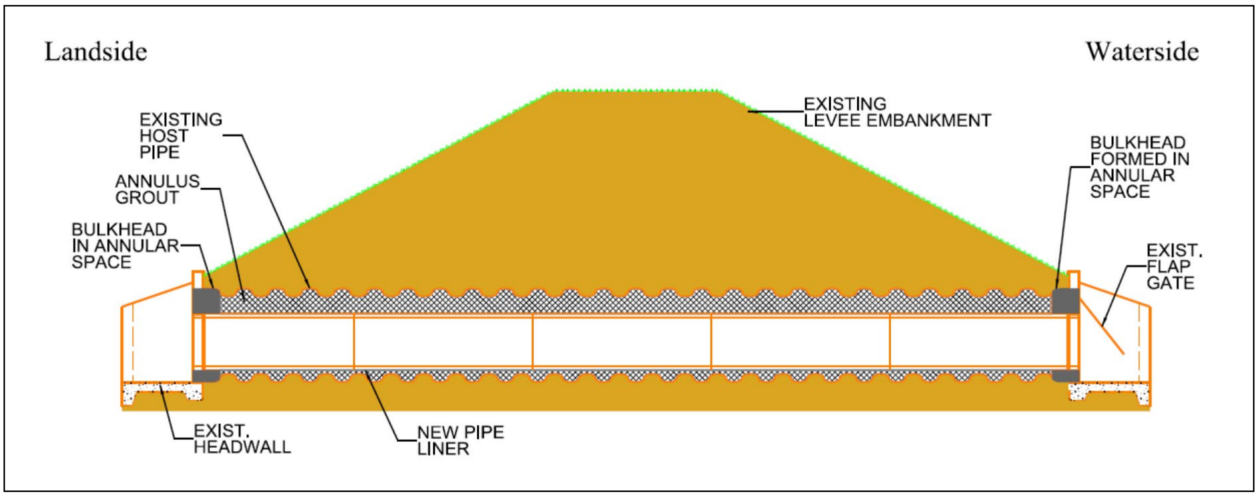


Figure 7-21. Typical installed liner in a host pipe with external grout.

(Courtesy of Indiana Reline)

Figure 7-22. Installation of two previously joined slip liner segments using headwall access.

7.5.2.6. Partial Slip Lining. Partial slip lining of a host pipe is permissible during rehabilitation if the remainder of the pipe is in satisfactory condition based on the condition assessment, as discussed in Section 6.7. There must be a smooth transition between the two different diameters to prevent the build-up of debris and provide hydraulic continuity.

7.5.3. Annular Space Grouting.

7.5.3.1. General. For continuous and segmental slip lining, grout must be used to fill the annular space between the slip liner and the existing host pipe. Structural grout is required when the load calculations indicate that the slip liner cannot support the loads against buckling and deflection on its own (reference Table 4-1); otherwise, a non-structural grout can be used. Reference Sections 4.15 and 4.16 for the applicable load calculations for the liner material to be installed. Note: the placement of non-structural grout may require the use of specialty equipment and/or contractors. Many ready-mix plants, for example, will not be able to supply grout that includes a foam additive since this product is mixed onsite using specialized equipment and placed immediately after mixing. In this case, a pipe slip-lining specialist experienced in placing foamed grout is more likely to have the equipment and expertise required to properly complete annular space grouting.

7.5.3.2. Non-structural (Cellular Foam) Grout. Non-structural grout used for continuous and segmental slip lining should have a 24-hour penetration resistance of no less than 100 psi (per ASTM C403) and a 28-day compressive strength of approximately 300 psi (per ASTM C495). This lower compressive strength grout reduces the cost when only infilling of the annular space is needed. A foaming agent must be incorporated into the grout mix and the grout density must be approximately 45 pcf (per ASTM C138). To ensure that grout flows adequately through the annular space, grout viscosity must be 20 seconds or less (per ASTM C939). The foaming agent will increase the flowability of the grout as compared to structural (unfoamed) grout, but its compressive strength will be lowered as a result. An example mix design for a non-structural grout is shown in Table 7-11.

Table 7-11
Example non-structural grout mix design

Material	Amount
Cement (per ASTM C150)	1750 lb/cy
Fly ash	Optional, will lower early strength if substituted for cement
Water	960 lb/cy
Sand	376 lb/cy
Foaming agent admixtures	Manufacturer's recommended dosage

7.5.3.3. Structural Grout. Structural grout used for continuous and segmental slip lining must have a 24-hour penetration resistance of no less than 100 psi (per ASTM C403) and a 28-day compressive strength of greater than 3,000 psi (per ASTM C942). To ensure that grout flows adequately through the annular space, grout viscosity must be 20 seconds or less (per ASTM C939). An example mix design for a structural grout is shown in Table 7-12.

Table 7-12
Example structural grout mix design*

Material	Amount
Cement (ASTM C150)	1200 lb/cy
Fly ash	600 lb/cy
Water	900 lb/cy
Sand	400 lb/cy
HRWRA (high range water reducing Admixture)	Manufacturer's recommended dosage
SRA (shrinkage reducing admixture)	Manufacturer's recommended dosage

*A pre-packaged structural grout product may be used as long as it meets the above criteria for strength, flow, and positive expansion.

7.5.3.4. Grout Injection. Drilling additional injection holes from the interior surface of the slip liner to facilitate grouting is prohibited. After the liner pipe is placed in the host pipe, the following steps must be taken to install the annular space grout:

7.5.3.4.1. Bulkheads. Bulkheads (usually about two feet thick) are installed at each end of the pipe by placing a very low slump mix between the slip liner and the host pipe by tamping, rodding, and/or ramming the material into place. A cubic yard of bulkhead mix typically consists of 750 pounds of cement, 2,400 pounds of sand, and just enough water to produce a one-inch slump. The mix must be packed around several two-inch diameter injection ports and air vents. For large annular spaces, concrete blocks or bricks can be used with the bulkhead material to reduce the amount of material needed (Figure 7-23). Bulkheads are hand-finished with a Portland cement-based grout to a Class C concrete surface per American Concrete Institute (ACI) 347.3R-13.

(Courtesy of USACE Louisville District)

Figure 7-23. Concrete blocks and brick used to fill annular space in bulkhead zone.

7.5.3.4.2. Grout Injection and Exit Air Vent Ports. Grout injection and exit air vent ports (usually PVC) should be attached to the exterior of the liner pipe prior to installation of the bulkhead material.

7.5.3.4.3. Annular Space. The annular space between the bulkheads should be low-pressure grouted through tubes that extend through the bulkheads (Figure 7-24 and Figure 7-25). Grout should be injected at one end of the pipe (the inlet end or higher elevation end) and flow through the annular space toward the outlet end (lower elevation end).

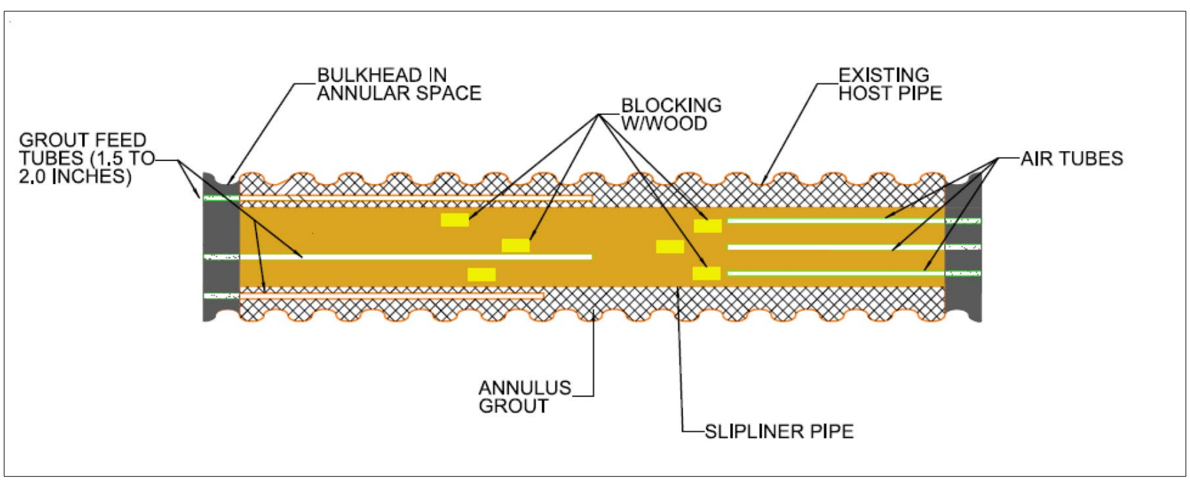


Figure 7-24. Plan view of bulkheads and annular space grouting.

(Courtesy of IPC Sales)
Figure 7-25. Grout ports through bulkhead.

7.5.3.4.4. Grouting System. The grouting system must have gauges and monitoring devices to determine if grouting can be single-staged or multi-staged due to the potential for collapsing the liner pipe.

7.5.3.4.5. Determining Grouting Effectiveness. Grout injection continues until the estimated volume of grout is injected, exhausted grout recovered at each vent is not less than 85 percent of the density of the freshly injected grout, and the grout installer and USACE representative recommend ceasing grouting operations. Grouting effectiveness is determined when the grout begins flowing out of vent tubes installed through the bulkhead, which are plugged when consistent grout begins to flow out of each tube.

7.5.3.4.6. Grouting Considerations. Grout will typically seal most open joints and/or exits from the host pipe through defects and fill voids adjacent to the exterior of the pipe, stabilizing the surrounding soil mass. At locations where the host pipe is damaged, there is a risk of hydraulic fracturing of the surrounding soil if the grout pressures are too high. Typically, pressures in the pipe annulus are limited to 5 psi. In some conditions there may still be a potential for fracturing where there is insufficient soil cover over the pipe. The pressures in the annulus will increase as the grouting is completed and the final vent tube is closed. Pumping must be controlled to prevent pressures in the annular space from exceeding 5 psi, the estimated hydraulic fracture pressure divided by a factor of safety of 1.5, or the manufacture's specified maximum pressure (whichever is lowest). The fracture pressure for the soil can be estimated using Equation 7-1 through Equation 7-4:

Equation 7-1

$$P_f = \frac{(\sigma_{min} + c)}{144}$$

Equation 7-2

$$\sigma_{min} = (k_0 \cdot \sigma'_v) + u$$

Equation 7-3

$$k_0 = 1 - \sin \varphi$$

Equation 7-4

$$\sigma'_v = \gamma \cdot z - u$$

Where:

P_f	=	estimated hydraulic fracture pressure (psi)
σ_{min}	=	minimum total principle stress (psf)
σ'_v	=	vertical effective stress (psf)
u	=	pore pressure (psf)
c	=	undrained strength of the soil (psf)
φ	=	drained friction angle of the soil (degrees)
k_0	=	at rest earth pressure coefficient (unitless)
γ	=	unit weight of the soil (pcf)
z	=	depth of soil cover over pipe (feet)

7.5.3.4.7. Ambient Temperature. If the ambient temperature during grouting is below 32 degrees Fahrenheit, the grout mix temperature must be 60 degrees or higher and an interior pipe heater used for 24 hours to help the grout hydration process. In addition, insulation or concrete blankets must be used to cover the ends of a pipe with less than 24 inches of soil cover. A soluble reactive silicate concrete treatment should be applied over the bulkhead material and the entire headwall surface to help prevent freeze/thaw damage.

7.5.4. Other Rehabilitation Methods and Evolving Technologies.

7.5.4.1. General. Liner methods that do not allow space for pressurized grouting between the deteriorated host pipe and new liner are not permitted for gravity drains in USACE dams and levees. The inability to grout the annular space means that any voids created by the loss of embankment soil through pipe defects would not be filled, promoting PFM-1. However, if the host pipe has no penetrating deterioration and no visible soil at the joints, and deflection is less than five percent (reference Table 4-1), SIPP and close-fit liners (e.g., cured-in-place pipe liners [CIPP]) may be approved on a case by case basis by the respective USACE District. USACE does not allow the use of either pipe bursting or pipe splitting in dams and levees because of the possibility of creating a void around the new pipe that is not filled with soil introduces the risk that the internal erosion PFMs (PFMs-1, -2, and -3) may initiate and lead to failure of the dam or levee where no internal erosion issues previously existed.

7.5.4.2. Spray-in-Place Pipe Liner. SIPP liners are used in non-pressure pipes and are typically applied by a machine passing through the host pipe spraying lining material radially onto the interior wall (Figure 7-26, reference ASTM F3182 and ASCE MOP 132), or otherwise can be manually applied (Figure 7-27). Cement mortar, resinous materials such as epoxy and polyurethane, or carbon fiber-reinforced polymer are common seamless lining materials with no surface voids or ridges. SIPP liners which contain polyurea or polyurethanes with methylene diphenyl diisocynate (MDI) contain isocyanates which are toxic and their use should be carefully considered. The liner thickness is controlled by the forward and rotational speed of the machine and the viscosity of the liner fluid. All of these liner materials are suitable for use on cast iron, ductile iron, welded seam steel, and concrete pipes.

(From TunnelTalk.com, 2008)

Figure 7-26. Mechanical application of SIPP liner.

(Courtesy of Warren Environmental Inc.)
Figure 7-27. Manual application of epoxy SIPP liner.

7.5.4.3. Close-Fit Liners. Close-fit liners such as CIPP involve the installation of a thermoplastic pipe approximately equal to or slightly larger than the internal diameter of the host pipe. There are multiple types of close-fit liners such as cured-in-place (Figure 7-28, reference ASTM F1743, F2016, and F2019), fold and form (Figure 7-29, reference ASTM F 1867 and ASTM F1871), and swagelining and rolldown (reference ASCE MOP 132). The designer must be aware that adhesion between the liner and host pipe is critical and requires special surface cleaning.

(From JTV Inc., 2017)
Figure 7-28. Fabric tube advances through host pipe as pressure is applied during the inversion process.

(From WaterWorld Magazine, 2012)
Figure 7-29. Fold-and-form liners.

7.6. Replacement or Relocation Considerations. Before determining whether to repair or rehabilitate a pipe, the costs associated with total pipe replacement should be evaluated. Reference Chapters 3 and 4 for pipe replacement information. The major considerations when choosing pipe replacement or rehabilitation are size, depth below the ground surface, proximity to other structures, disruption to public life, and criticality of service. Pipe replacement does offer the opportunity to resize the new pipe if a larger pipe is needed to accommodate significantly larger flow than the original design; it also allows for grade or alignment changes. Where a replacement pipe can be relocated to avoid penetrating an embankment, the designer should carefully consider the relocation alternatives to prioritize avoiding the PFMs associated with pipes within an embankment or under a floodwall.

7.7. Section 408 Considerations for Pipe Repair/Rehabilitation. The respective USACE District may review documentation related to proposed repair or rehabilitation methods to determine if such actions are subject to 33 USC 408 and require approval. Reference Appendix H for a list of suggested documentation for Section 408, if needed.

7.8. Post-Installation Inspection and Acceptance of Work. Upon completion of repairs or rehabilitation, the work must be inspected according to Chapter 6 and the results provided to the respective USACE District. The results of that inspection are used as part of the decision to accept the work as complete, remediate if the work is not acceptable and can be corrected, or remove, decommission, or replace the pipe if the work is not acceptable and cannot be corrected.

Chapter 8
Removal and Decommissioning

8.1. Introduction. This chapter provides information on and general procedures for removal and in-place decommissioning of existing pipes, including methods to reduce the probability of pipe-related PFMs. Within this manual, decommissioning is defined as withdrawing a pipe from service by acceptably infilling the pipe with an approved material.

8.2. Potential Failure Modes Related to Removal or Decommissioning. Removal of a pipe followed by the proper placement of soil backfill eliminates the possibility of any of the PFMs listed in Chapter 2, and is recommended if the pipe has widespread advanced deterioration that has allowed a substantial loss of embankment material. For example, proper decommissioning eliminates the possibility of both pressurized fluid escaping from a pipe (PFM-2) and internal erosion into a pipe defect (PFM-3); however, it cannot repair compromised soil around the pipe. Pipes that are decommissioned must incorporate a seepage filter, whether internal or external to the embankment (reference Section 5.5.9.), to reduce the potential for internal erosion adjacent to the pipe (PFM-1).

8.3. Special Requirements for Dams and Toe Drains.

8.3.1. Special Requirements for Dams. For pipes passing design flows through a dam, removal or decommissioning will only be approved after an alternative means of passing the design flows is in place. The required removal or decommissioning limits may vary depending on how the discharge capabilities of the reservoir are modified. Additionally, alterations must follow the study and review procedures outlined in ER 1110-2-1156.

8.3.2. Special Requirements for Toe Drains. Prior to removing or decommissioning a toe drain, a seepage evaluation must be performed using the minimum requirements in EM 1110-2-1913 for levees or EM 1110-2-1901 for dams. This may require that alternate seepage mitigation measures (e.g., cutoff walls, relief wells, landside berms, new toe drains) meeting the EM requirements be installed prior to toe drain removal or decommissioning. Figure 8-1 shows the surface expression (sinkhole) as a result of internal erosion (PFM-3) into two collapsed six-foot diameter corrugated steel pipes. The tremendous loss of soil necessitated an open-cut excavation to remove the pipes (Figure 8-2) since the embankment had been so severely compromised that annular grouting associated with slip lining could not have filled the void completely.

(Courtesy of USACE Louisville District)
Figure 8-1. Sinkhole in levee due to collapsed pipes.

(Courtesy of USACE Louisville District)
Figure 8-2. Removal and replacement of collapsed pipes.

8.4. Horizontal Removal and Decommissioning Limits. The horizontal removal or decommissioning limits for a pipe are the same as the influence zone limits referenced in Figure 6-2 through Figure 6-10 and ensure that any remaining pipe segment or associated structures will not threaten the integrity of a USACE embankment or floodwall or hinder operation, maintenance, inspection, or potential flood fighting activities. The respective USACE District may approve varying these limits.

8.5. Pipe Removal.

8.5.1. Excavation Requirements. Open cut excavation and pipe removal is required if the pipe's condition prohibits decommissioning (reference Section 6.7.).

8.5.2. Verification. Final excavation widths, depths, side slopes, and excavation shoring removal must be verified in the field by the respective USACE District. Additional excavation beyond what was anticipated may be required if field observations reveal evidence of distress or internal erosion adjacent to the pipe. Field or laboratory testing methods (e.g., unit weight/density test data, soil moisture content) may be required to suitably delineate and remove any soft, loose, or partially eroded zones that could threaten the embankment or floodwall if left in place.

8.6. Pipe Decommissioning.

8.6.1. General. In most cases, pipe decommissioning (or partial decommissioning and removal) is more feasible than a full pipe removal due to technical and/or cost considerations or disruption to public services. Decommissioning (or partial decommissioning and removal) may be approved by the respective USACE District unless it is prohibited by the conditions detailed in Section 8.6.2.

8.6.2. Conditions that Prohibit Decommissioning. Decommissioning is not permitted if the pipe or pipes do not meet the condition requirements outlined in Section 6.7.

8.6.3. Seepage Mitigation. Decommissioned pipes must be surrounded by an external filter to prevent particle migration along the outside of the pipe (reference Section 5.5.9.4.) and must comply with the most recent version of either the Federal Emergency Management Agency's (FEMA) "Filters for Embankment Dams" or EM 1110-2-1901.

8.6.4. Associated Structures. Associated structures must be evaluated to ensure they are in satisfactory condition and will not introduce unacceptable risk to the dam or levee if left in place and decommissioned along with the pipe. Similar to pipes, associated structure openings must be blocked or bulkheaded in such a manner as to ensure the interior is completely infilled with approved material (reference Section 8.6.7). Protruding structures that could hinder infilling and facilitate voids (e.g., gates, gate frames, operator stems, ladders, landings, grates) must be removed prior to infilling.

8.6.5. Decommissioning Plan. A decommissioning plan must include (see Appendix H):

- General description and location of the existing pipe and limits of decommissioning.
- Consequences of pipe decommissioning (e.g., interior drainage plan or other significant impacts).
- Existing pipe as-built drawings and specifications, when available.
- Pipe condition assessment as a result of the internal inspection (reference Chapter 6 for inspection and condition assessment guidance).
- Performance history (e.g., previous inspections, surface expressions over the pipe alignment, history of ruptures or leaks).
- Preparations for pipe decommissioning and treatment of associated structures (reference Section 8.6.4.).
- Infilling equipment, material, and procedures to include theoretical volume of pipe and associated structures.
- Waste disposal requirements.
- Verification and documentation requirements.
- Seepage evaluation and mitigation measures if decommissioning a toe drain or relief well.

8.6.6. Preparations for Pipe Decommissioning.

8.6.6.1. Bulkheads and Venting. After diverting flow through the pipe to be decommissioned, bulkheads are placed at the ends (Figure 8-3) to support the infilling material. Air vent ports must be installed through the bulkhead to allow air and any interior water to be released during the infilling process. Additional bulkheads or grout pipes may be required at intervals along the pipe depending on the length of the pipe and infilling method. Inflatable bladders placed inside the pipe may be used as a bulkhead if man-entry is not required. However, the circumferential stresses applied must be evaluated to ensure the pipe is not damaged and the bladder remains stable during infilling. If inflatable bladders are used to bulkhead the end of a pipe, they must be removed after the pipe has been decommissioned. In order to prevent pipe or joint damage, or leakage into the embankment or foundation, it is recommended that the vents be a minimum of 150 percent larger than the injection port diameter.

(Courtesy of Mobile Concrete and Grout of Alaska)
Figure 8-3. Typical infill and vent ports through a temporary bulkhead.

8.6.6.2. Pipe Cleaning and Inspection. Pipe cleaning, inspection, and a condition assessment must be performed prior to pipe decommissioning (reference Chapter 6 and Chapter 7). As-built dimensions and true shape should be confirmed during this step.

8.6.7. Infilling Material.

8.6.7.1. Non-structural Grout. Non-structural grout is commonly used for decommissioning pipes and must meet the requirements outlined in Section 7.5.3.2. Other materials, such as those listed below, may also be permitted if supported by an engineering evaluation and approved by the respective USACE District.

8.6.7.2. Portland Cement Concrete (PCC). Conventional concrete consisting of a combination of Portland cement, water, and aggregate may be used as infilling material. PCC must contain a shrinkage reducing admixture at the manufacturer's recommended dosage to minimize shrinkage. Accumulated bleedwater, tested according to ASTM C232, is recommended to be less than two percent. The use of 25 to 50 percent pozzolans (fly ash or ground blast furnace slag) is recommended to reduce the heat of hydration associated with a straight-cement mixture. Similarly, an alkali-silica reactivity expansion of less than 0.10 percent at 16 days per ASTM C1567 is recommended in order to limit unwanted internal expansion. Mixture design, slump, and admixtures must be compatible with placement techniques (e.g., tremie, pumping).

8.6.7.3. Cellular Concrete. Cellular concrete is composed of cement, water, and sometimes a fine aggregate, with a foam additive chosen to achieve a specified density. Cured densities of cellular concrete can range from 20 to 150 pcf and compressive strengths can range from 30 to 5,000 psi. A minimum density of 30 pcf and a compressive strength between 50 and 100 psi are recommended.

8.6.7.4. Sand Slurry Mixture. Sand slurry mixtures may be used for toe drain decommissioning; however, they are not acceptable for infilling pipes penetrating embankments due to their high hydraulic conductivity and tendency to develop cracks if the host pipe degrades. Sand slurry mixtures are preferred where toe drain decommissioning requires large volumes of material and access ports are widely spaced. Sand slurry mixtures are prepared by mixing sand with water and/or polymer additives, which allow the sand to remain in suspension and be hydraulically placed into a pipe. Specially prepared bulkheads are necessary to retain the sand slurry mixture and allow water to pass through the bulkhead while retaining the sand component. As water dissipates from the mixture, additional sand slurry is delivered to the pipe until the water is fully dissipated.

8.6.8. Infilling Procedures and Equipment.

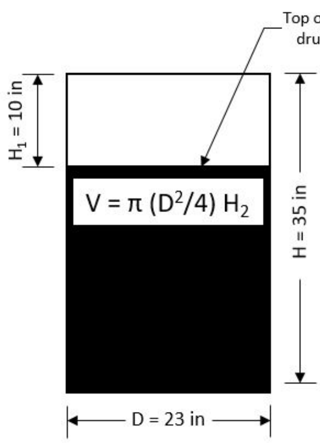
8.6.8.1. Specialty Products, Equipment, and Contractors. The placement of some infilling materials may require the use of specialty equipment and/or contractors. Many ready-mix plants, for example, will not be able to supply an infilling material that includes a foam additive since this product is mixed onsite using specialized equipment and placed immediately after mixing. In this case, a pipe slip-lining specialist experienced in placing foamed grout is more likely to have the equipment and expertise required to properly decommission a pipe. Other infilling materials may also have related specialists better suited for pipe decommissioning than a bulk material supplier.

8.6.8.2. Infill Ports and Air Vents. Infilling ports are typically constructed at one or more of the inspection access locations discussed in Section 6.3 and the grout ports discussed in Section 7.5.3.4, and they generally correspond with bulkhead locations. Drilling through an embankment to install infill ports and air vents is prohibited unless approved by the respective USACE District and the requirements of ER 1110-1-1807 are met.

8.6.8.3. Pumps.

8.6.8.3.1. General. Infilling equipment must be capable of continuously pumping material at the planned placement rate and the pump must be properly sized to prevent clogging. Delivery line discharge pressures during infilling must be maintained at less than 5 psi to prevent damaging the host pipe. This maximum discharge pressure criterion is based on experience with annular grouting during slip lining efforts and is considered applicable to pipe infilling activities. Pressures at the bulkhead (or arrangement of valves and pressure gauges used to control the quantity and pressure of the infill material) must be continually monitored to ensure significant pressure spikes do not occur, which may indicate improper pumping techniques or blockages in the pipe. If the discharge pressures exceed 5 psi, the injection process must be altered (e.g., slow the injection rate, stop pumping and evaluate the pipe or air vents for blockages, or reduce the mixture viscosity).

8.6.8.3.2. Potential Pumping Concerns. Damage to pumping equipment and interruptions to pumping are undesirable; therefore, the following proactive measures and contingency preparations are recommended. Pump hoses (delivery lines) should be laid as straight as possible to reduce straining the pump. Small hose diameters and hot conditions are incentives to reduce the pumping distance so that clogging from premature curing of the material in the lines does not occur. The pumping contractor should maintain spare parts onsite in case pump repairs are required after the process has started. Flowmeters are preferred and should be used, but when not used, the pump must be “calibrated.” Pump calibration is the process of equating the number of pump strokes to the volume of material pumped. Pump calibration is performed using a 50-gallon drum (or similar volume apparatus), as detailed in Figure 8-4.



Step 1. Determine pump calibration factor, CF

CF = volume pumped into barrel / no. strokes
= [(π x D^2/4) x $H_{2)}$] / no. strokes

where,
π = 3.1416
D = inside diameter of drum
H = Total height of drum, measured from inside
H_1 = Free Distance at end of stroke, between top of drum and material
$H_2 = H - H_1$

CF = [(3.1416 x 23^2/4) x 25] / 8 = 1,298.36 in^3 per stroke (0.75 ft^3 per stroke)

Step 2. Determine the actual volume used, V

V_{actual} = No. strokes x CF
= 8 strokes x 0.75 ft / stroke = 6 ft^3

Figure 8-4. Pump calibration calculations.

8.6.8.4. Delivery Lines. Flexible hoses are the most common delivery lines, but rigid pipe systems are sometimes used. Sizing of delivery lines must consider the injection material, infilling volume, anticipated injection rate, and access requirements. Depending on the pump, injection hose, and injection mixture, it is possible to use delivery lines for pumping material distances of 500 feet or more. Where conventional PCC or cellular concrete is used, a four-inch minimum diameter delivery line is recommended. The delivery line diameter and pump must be sized relative to the maximum size of the coarse aggregate to prevent clogging. Typically, the coarse aggregate is limited to one-third of the smallest inside diameter of the pump or delivery line; however, it is recommended that the diameter be no less than 1.5 inches for grout. Additional considerations for grouting are found in EM 1110-2-3506. Tremie lines should be removed from the pipe to be decommissioned during infilling activities. Tremie lines may remain only if they are completely infilled.

8.6.8.5. Horizontal Injection. In cases where the pipe is accessible from both sides of the USACE dam or levee, horizontal injection is accomplished by placement of a vent port at the highest point through the bulkhead on the inlet side of the pipe and introducing material at the low point of the pipe on the outlet side. Horizontal grout injection must be accomplished soon after pipe cleaning, inspection, and repairs are completed. Typically, infilling material is introduced through an injection tube placed through the downstream bulkhead. However, infilling may also be accomplished by placing a grout tube (or series of grout tubes) through the upstream bulkhead and filling from the low point to the high point to prevent entrapped air pockets. In either case the infilling material flows toward the upstream (inlet) bulkhead (toward the vent port or exit tube), forcing air through the uppermost portion of the pipe. Material is introduced until return flows are observed (reference Section 7.5.3.4).

8.6.8.6. Vertical Injection. In some cases, vertical access to an existing pipe already exists or can be made relatively easily by open-cut excavation (e.g., a pump station discharge pipe placed up and over a levee embankment) or by using the air vent/siphon breaker access. Vertical grout injection must be completed soon after pipe cleaning, inspection, and repairs are completed. Where material is injected vertically down through the embankment with delivery lines penetrating into an existing pipe, vertical vents must also be located at the top of the pipe and at the highest point along the alignment in order to allow entrapped air to escape during filling. Material is introduced until the theoretical or field-measured pipe volume is installed.

8.6.9. Verification.

8.6.9.1. General. Once infilling is complete, the percentage of filling must be determined by comparing the pumped volume with the theoretical or field-measured pipe volume, while considering the accuracy of the measurements used. Material pumped in excess of the actual pipe volume may cause damage to the pipe or jointed connections, and/or hydraulic fracture of the embankment. Pumped volumes less than the actual pipe volume may indicate that voids remain in the pipe. In either case, differences between the pumped and actual volumes must be evaluated by the respective USACE District. Where excessive material is judged to have been used during infilling activities, damage may be difficult to quantify, and open-cut excavations could be warranted to investigate possible damages or to completely remove the pipe. The determination to require open-cut excavation is made by the respective USACE District and may require a risk assessment. Verification must also incorporate review of construction reports, review of as-built documents/test reports, and instrumentation results. As-built documentation requirements are detailed in Section 8.8.

8.6.9.2. Construction Observation. Construction observations should include visual observation of the dam or levee structure and pipe prior to, during, and after construction. The field conditions, complexity, and schedule will determine the frequency and duration of monitoring. Monitoring is typically performed by a USACE representative familiar with the project or a third party hired by the levee sponsor.

8.6.9.3. Instrumentation. Monitoring instruments are necessary in order to determine infilling pressures, flowrates, and volumes. Flowmeters are available with accuracies of up to approximately 0.2 percent and are recommended for estimating volumes where there is a high degree of certainty regarding the fill volume. If the use of a flowmeter is not permissible (e.g., when pumping large aggregate mixtures), pump calibration is performed as outlined in Section 8.6.8.2. On more complex projects (e.g., grouting small voids adjacent a pipe to be decommissioned), other instrumentation may be required, such as piezometers for monitoring sudden increases in pore water pressure which could be indicative of hydraulic-fracture or loss of material. Piezometers, or other instruments that require initial readings, must be installed prior to decommissioning in order to establish “baseline” conditions as outlined in EM 1110-2-1908.

8.6.9.4. Documentation Review. Upon completion of pipe decommissioning, the contractor performing the work must submit documentation to include quality assurance test reports (e.g., compressive strength test data) required by the project specifications. The documentation must include any modifications from the original design drawings. USACE personnel will perform a review of the documentation and consolidate comments identifying issues that need to be resolved. The submittal documentation must include a comparison of the total volume pumped into the pipe to the theoretical or field-measured volume of the pipe. Documentation that is determined to be insufficient may lead to complete removal of the pipe, regardless of the pipe's condition.

8.7. Section 408 Considerations for Pipe Removal/Decommissioning. The respective USACE District may review documentation related to proposed removal or decommissioning methods to determine if such actions are subject to 33 USC 408 and require approval. Reference Appendix H for a list of suggested documentation for Section 408, if needed.

8.8. As-Built Documentation. Pipe removal or decommissioning must be documented upon completion of the project in a report provided by the contractor's quality assurance representative and should include the items listed in Table 8-1. The report is a valuable reference in the future if there are concerns with the performance of the USACE dam or levee as it relates to the decommissioned pipe(s).

Table 8-1
As-built documentation requirements

Item	Pipe Removal	Pipe Decommissioning
1. Project Modification Details		
Cleaning and Inspection Results	Note 1	✓
Pipe Repair Requirements		✓
Design Drawings and Specifications	✓	✓
2. Construction Documentation		
Equipment, Procedures, Materials	✓	✓
Construction Photos (Prior, During, & Post-Construction)	✓	✓
Quality Control & Assurance Documents	✓	✓
Field Observation Reports	✓	✓
3. As-Built Documentation		
As-Built Plans, Specifications	✓	✓
Infilling Volume (Actual vs Theoretical)		✓
Instrumentation Results and Evaluation		Note 2

1. May be required to assess existing pipe and determine if removal is warranted.
2. Instrumentation includes equipment for pump calibration, flowrate and pressure measurements, or other instrumentation as directed by the USACE District.

THIS PAGE INTENTIONALLY LEFT BLANK

Chapter 9
Associated Structures and Appurtenances for Levees

9.1. Introduction. Chapter 9 discusses common types of associated structures and appurtenances related to pipes in levees; their purpose, function, typical location, and design criteria; references to design and inspection resources; and the likelihood of causing a breach if they do not function properly or are poorly maintained. Associated structures (e.g., pump stations, gatewells, headwalls) are directly connected to the pipe and often provide man-entry access to the pipe or control the entry and exit of water flow. Appurtenances are mechanical flow-controlling and closure devices (e.g., sluice gates, flap gates, pumps) typically found within associated structures.

9.2. Potential Failure Modes Related to Associated Structures and Appurtenances for Levees. The connection of a pipe to an associated structure is a vulnerability since these are typically constructed and sealed in the field and lack the control and consistency of a factory connection. As minute differential movements between the two structures occur, it can lead to openings in the connection and allow soil infiltration (PFM-3); regular pipe inspections should detect this issue. Without regular inspections or attention to operation, and maintenance requirements, appurtenances can become blocked with debris (PFM-6) and gates are more likely to become seized in a position that allows either floodwaters to enter a leveed area or prevents interior water from draining (PFM-7). Figure 9-1 shows a failed closure system because debris was not removed before closing the gate. This also demonstrates the need to design appurtenant structures using the motor's stall torque.

(Courtesy of USACE Louisville District)

Figure 9-1. Failed closure system due to damaged sluice gate stem.

9.3. Associated Structures.

9.3.1. Pump Stations. Pump stations are typically concrete structures that house the pumps and associated piping responsible for removing interior drainage from the landside ponding area that accumulates due to a closed outlet condition on a gravity pipe. Some levee systems have multiple stations that are responsible for pumping not only stormwater, levee underseepage, and relief well flow, but sanitary sewage or industrial effluent as well. The preferred location for a pump station is at the levee toe (Figure 9-2), which reduces the height of the pump column and the depth of the building, and sometimes allows the ponding area to drain directly into the pump station intake. The levee centerline can be considered for the location of a pump station if the operating floor would be submerged during normal conditions (Figure 9-3). Figure 9-4 and Figure 9-5 show cross-sections of a pump station at the levee landside toe that pumps water within (up-and-over) the embankment towards the waterside headwall or into a gatewell structure located near the levee centerline, respectively. Comprehensive general information, design requirements, and inspection guidance are located in EM 1110-2-3104 and EM 1110-2-3105.

(Courtesy of USACE Louisville District)

Figure 9-2. Pump station at landside toe.

(Courtesy of USACE Louisville District)
Figure 9-3. Pump station at levee centerline.

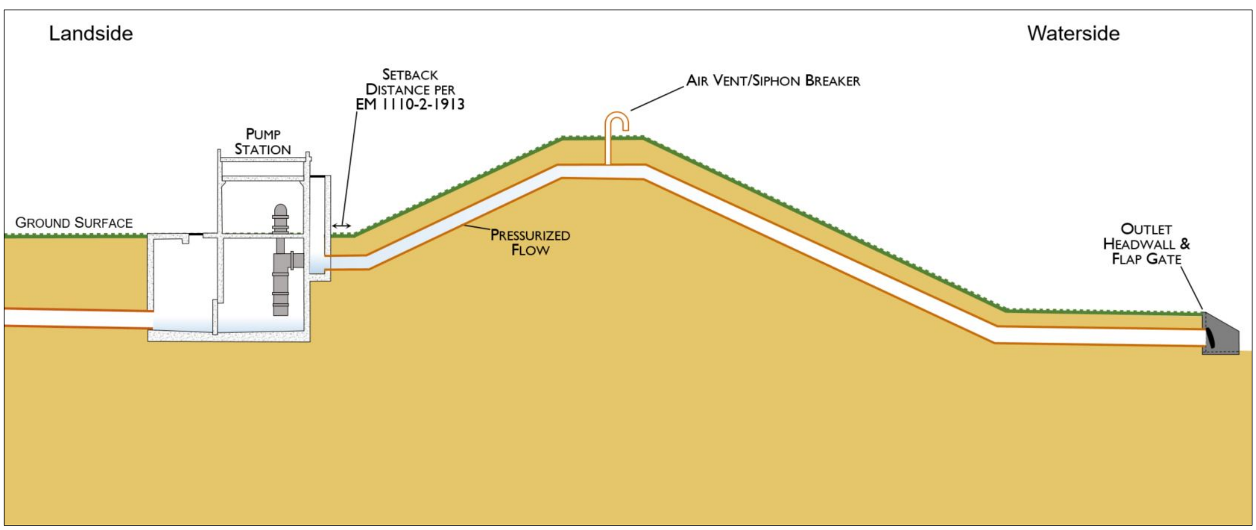


Figure 9-4. Pump station cross-section near landside toe.

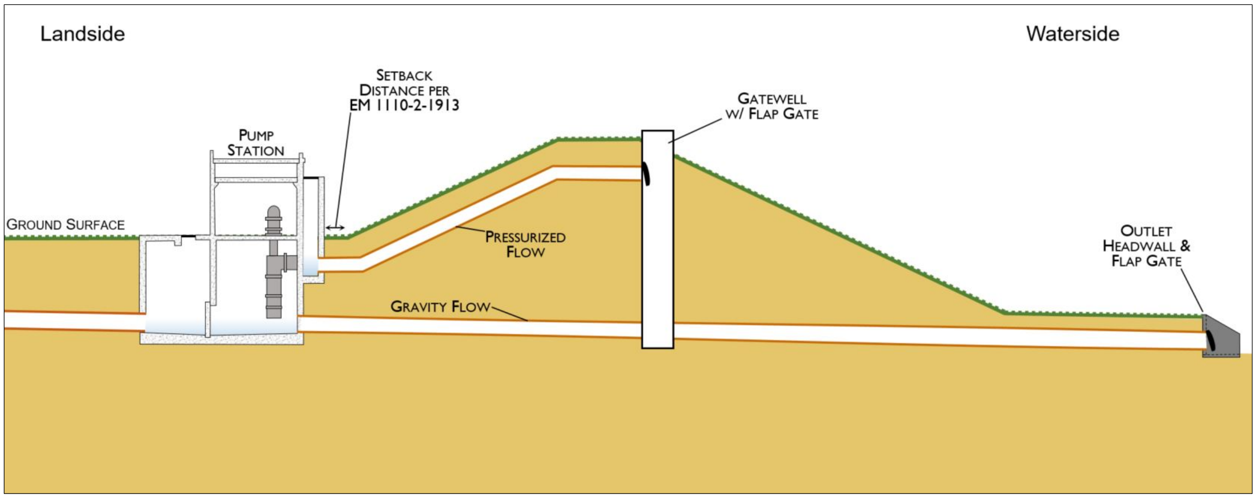


Figure 9-5. Pump station cross-section with gatewell discharge.

9.3.2. Gatewells.

9.3.2.1. General. Gatewells are typically concrete structures but can also be a vertical corrugated steel pipe (CSP), usually located on the waterside of a levee system. Gatewells typically house active gates that are closed during flood events to prevent backflow through the pipe. Precast concrete gatewells or corrugated metal gatewells may be used in lieu of cast-in-place concrete if designed and detailed to satisfy the loading and functional requirements of the levee system, and if the joints are designed to prevent soil infiltration.

9.3.2.2. Design. If an electric motor is used to operate an appurtenance within an associated structure, the appurtenance wall anchors must be designed to resist the stall torque specified by the motor manufacturer. Failure to do so may result in a failed closure system that could induce interior ponding (seized in the closed position) or allow interior flooding (seized in the open position). The connection of a pipe to a gatewell is vulnerable to differential settlement since the seal may separate and allow soil infiltration; therefore, its design must permit the anticipated connection movement without failing. In cases where the pipe rests on a concrete cradle, the designer should determine if doweling the cradle and gatewell together is required and/or appropriate. Consult EM 1110-2-2104 for embankment gatewell design and EM 1110-2-2502 when a floodwall is constructed integral with the gatewell.

9.3.2.3. Location. Waterside gatewells must have an operations platform (location of actuator or manual controls to lower the gate) a minimum of one foot higher than the height of the levee or floodwall to allow access at all times. Gatewells located immediately next to the levee crest provide the advantage of easy access regardless of the river level (Figure 9-6); otherwise, a platform/bridge or an exterior ladder reachable by boat must be installed to access the top of the gatewell (Figure 9-7 and Figure 9-8). There must also be access down to the bottom to the pipe and gate. If a bridge is constructed, consider that tall gatewells may require large piers within the embankment, creating the potential for additional seepage paths. For new construction, gatewells should be located at the waterside edge of the levee crest (Figure 9-9), eliminating the need for boat or platform/bridge access during a flood event.

(Courtesy of USACE Huntington District)
Figure 9-6. Gatewell near levee crest.

(Courtesy of USACE Louisville District)
Figure 9-7. Gate actuators requiring access by boat.

(Courtesy of USACE Louisville District)

Figure 9-8. Gatewell within levee embankment on waterside.

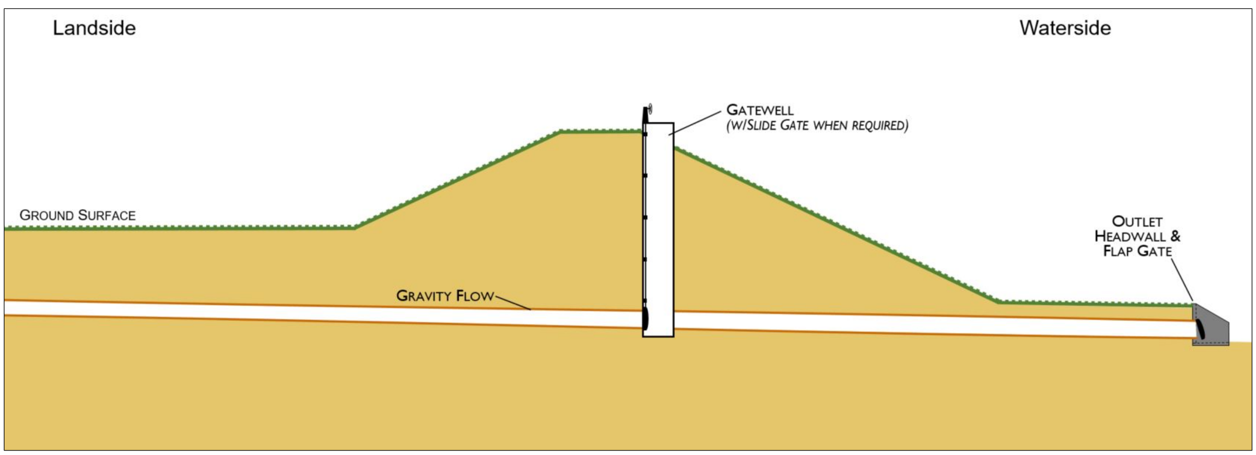


Figure 9-9. Cross-section of gatewell adjacent to waterside crest in levee embankment.

9.3.2.4. Areas of Concern. All gatewell joints, whether cast-in-place construction joints or connections between stacked precast elements, must have a waterstop to prevent the infiltration of embankment material into the gatewell (PFM-3). Similar to pump stations, movement of a gatewell due to instability or flotation could affect the performance of the levee by compromising the pipe-gatewell. A poorly installed or compromised connection could allow material loss through the defect, leading to internal erosion. Typically, gatewells are very deep in order to service the connecting drainage pipe and therefore the internal erosion process may be active for years, removing many yards of embankment material before a surface expression is observed. Reference EM 1110-2-2002 for additional information on gatewells.

9.3.3. Catch Basins.

9.3.3.1. General. Catch basins function by draining the nearby surface water into a pipe that transports the water out of the leveed area by gravity flow (Figure 9-10). Catch basins are usually precast concrete structures and are required to be sealed to prevent soil infiltration. Additional information for catch basins can be found in EM 1110-2-2002.

(Courtesy of USACE Louisville District)

Figure 9-10. Typical catch basin at toe of levee.

9.3.3.2. Areas of Concern. Catch basins are normally located no closer to the levee than the landside toe and the probability of instability is usually negligible since the catch basin is below ground. The connection of a pipe to the catch basin is critical to minimizing infiltration and reducing the initiation of PFM-3. Fortunately, catch basins are typically founded near the ground surface, which allows for quick identification if soil loss is occurring. A catch basin inlet also has the potential of becoming blocked by debris and backing up water within the leveed area, contributing to interior ponding (PFM-6).

9.3.4. Manholes.

9.3.4.1. General. Manholes are primarily used to access a pipe, and while not typical, manholes associated with pipes through levees can contain gates. Manholes within 15 feet of the landside or waterside toe are required to have joints that prevent soil infiltration. Additional information for manholes can be found in EM 1110-2-2002.

9.3.4.2. Areas of Concern. The primary risk driver for manholes is associated with pipe connections. Connections not installed per ASTM C1821 and ASTM C923 could allow infiltration of the embankment material (PFM-3). Operating devices within manholes are often neglected since they are out of sight, increasing the chance for interior ponding due to a malfunctioning appurtenance (PFM-7). The operating device may be accessible from the ground surface by simply removing the manhole cover; however, some may require operation by entry to a lower level within the manhole using an interior ladder (Figure 9-11).

(Courtesy of USACE Louisville District)
Figure 9-11. Typical manhole with sluice gate.

9.3.5. Headwalls.

9.3.5.1. General. Headwalls function to recess the inflow or outflow end of a pipe into the fill slope to improve flow conditions, anchor the pipe, support gates, and control erosion and scour from the pipe outflow area (PFM-4). Headwalls are typically constructed using concrete (Figure 9-12 through Figure 9-14) and should use wingwalls for added stability. All new pipes are required to have a headwall on both ends. New landside headwalls must have sufficient area on their face to install gated drains associated with an internal seepage filter as discussed in Section 5.5.9.3. To meet coverage requirements related to the internal filter, the height of the landside headwall above the pipe crown may have to be taller than most precast models; therefore, standard U.S. Department of Transportation headwalls may need to be modified. Reference Chapter 17 of EM 1110-3-136, EM 1110-2-2104, and EM 1110-2-2002 for additional information and design requirements for headwalls.

(Courtesy of USACE Louisville District)
Figure 9-12. Typical headwall with wingwalls and flap gate.

(Courtesy of USACE Louisville District)
Figure 9-13. Typical flat headwall with flap gate.

(Courtesy of USACE Louisville District)
Figure 9-14. Typical headwall with wingwalls and flap gate on concrete slope.

9.3.5.2. Areas of Concern. Erosion control should be provided to prevent headwall undermining, as this could lead to pipe failure and possibly a levee breach. The condition of a headwall could affect the performance should the concrete be allowed to deteriorate to a point that it can no longer support a gate. Failure of this structure's connection to the pipe and/or its appurtenance (typically a passive gate) would allow flood waters to enter the leveed area (PFM-7), or promote erosion at the levee toe from undirected, uncontrolled outflow, eroding embankment soil and potentially causing a levee failure (PFM-4). Regular inspections and repairs when necessary can reduce the likelihood of failure of the levee.

9.3.6. Junction Box. Junction boxes are similar to manholes but are typically not large enough for physical entry. Because of their smaller structure size, junction boxes house small appurtenances like shutoff valves instead of sluice or flap gates. Reference Section 5.7.6 for specific location requirements for shutoff valves. Junction boxes are usually precast concrete structures and are required to be sealed to prevent soil infiltration. Reference to EM 1110-2-2002 for additional information on junction boxes.

9.4. Appurtenances.

9.4.1. Gates.

9.4.1.1. General. Gravity pipes must be equipped with gates to control flow. Active gates are manually operated while passive gates automatically respond to rising flood waters. All gates must be inspected prior to high water events to ensure no debris will hinder their closure. Passive gates can address issues of rapidly rising floods automatically, but at the risk of an obstruction impeding operation if not inspected prior to an event. While active gates are necessary for reliability to reduce the likelihood of interior flooding, it is at the cost of requiring personnel to operate them in a timely manner to avoid interior ponding.

9.4.1.2. Selection. Selection of an active and/or passive gate is determined using Table 9-1 and depends on pipe diameter and rate of river rise. The closure configurations stated in Table 9-1 are requirements for new construction, but are also highly recommended for retrofitting existing pipes. Regardless of active or passive gate selection, the gates must be designed for the expected hydraulic head. These flood rise scenarios are associated with required closure configurations to reduce the likelihood of interior flooding, given the conditions shown. Any deviations from Table 9-1 or Section 9.4.1.4. must be approved per the requirements in Section 1.2.

Table 9-1
Closure type requirements for pipes

Flood Rise[1]	Pipe Diameter (inches)	Closure Configuration
Fast	< 36	(1) Passive
Fast	≥ 36	(1) Passive AND (1) Active
Slow	< 36	(1) Passive OR (1) Active
Slow	≥ 36	(1) Passive and (1) Active -OR- (2) Active

1. Fast flood rise refers to floods capable of rising to a flood stage with less than 12 hours prediction time.

9.4.1.3. Active Gates.

9.4.1.3.1. General. Active gates have greater reliability of operation than passive gates but have the disadvantage of requiring personnel and equipment to operate them (unless outfitted with an automated or remote actuator). Active gates are used when the rate of rise of water is slow enough to allow ample time for complete operation of the gates (minimum 12-hour flood prediction time according to EM 1110-2-1913) and must be in a gatewell (reference Section 9.3.2.).

9.4.1.3.2. Sluice (Slide) Gates. Sluice gates, typically made of structural steel, can handle large head differentials (Figure 9-15) and are considered active gates. Sluice gates are usually left fully opened to pass flows for open outlet conditions or fully closed to reduce the risk of interior flooding in a closed outlet condition. Intermediate positions are possible but should be avoided as this may encourage debris accumulation. Sluice gates are usually on guides and are operated with a gear either with a manual driven handle or electric actuator, typically placed above the gate. Along the sealing edge of a cast iron sluice gate, seating wedges can be installed to allow for adjustments to better seal the gate in the closed position. However, newer fabricated sluice gates typically use slides instead of wedges. The levee system's O&M manual dictates the water level at which the sluice gate is operated. There are two types of sluice gate orientations:

- A seating head orientation refers to a gate installed such that the water pressure pushes the gate onto a supporting structure (e.g., the concrete wall of a gatewell).
- An unseating head configuration is the opposite of a seating head configuration, in which the water pressure pushes the gate away from a supporting structure.

(Courtesy of USACE Louisville District)
Figure 9-15. Installation of sluice gate.

9.4.1.3.3. Sluice Gate Orientation. Sluice gates have a design limit for both orientations, where the seating head differential is significantly higher than the unseating head differential and is therefore preferred. However, if the gate location is restricted to one side of the gatewell for access or clearance, it may be necessary to install a sluice gate in an unseating head orientation. If an unseating head gate is needed on the landside of a levee, the gatewell must be built to the height of the levee since otherwise it would allow floodwaters to fill the gatewell up to the levee height. A conventional closure sill, where the gate closes below invert of the pipe (Figure 9-16), may be required for unseating head sluice gates in order to maximize the head differential. If a conventional closure sill can be avoided, a flush bottom sill is ideal to prevent the collection of water and debris which could obstruct the closure of the gate.

(Courtesy of USACE Louisville District)
Figure 9-16. Conventional-closure sill for sluice gate.

9.4.1.3.4. Sluice Gate Considerations. F-type wall thimbles are most commonly used around the wall opening for mounting sluice gates, which aligns the gate, protects against gate distortion, and forms the opening during the wall pour. E-type wall thimbles are used on unseating head sluice gates (Figure 9-17). Consult Table 9-1 for circumstances that require the use of active gates, such as sluice gates, on a pipe. Reference ETL 1110-2-584, EM 1110-2-6054, and EM 1110-2-3105 for additional information and design requirements for sluice gates.

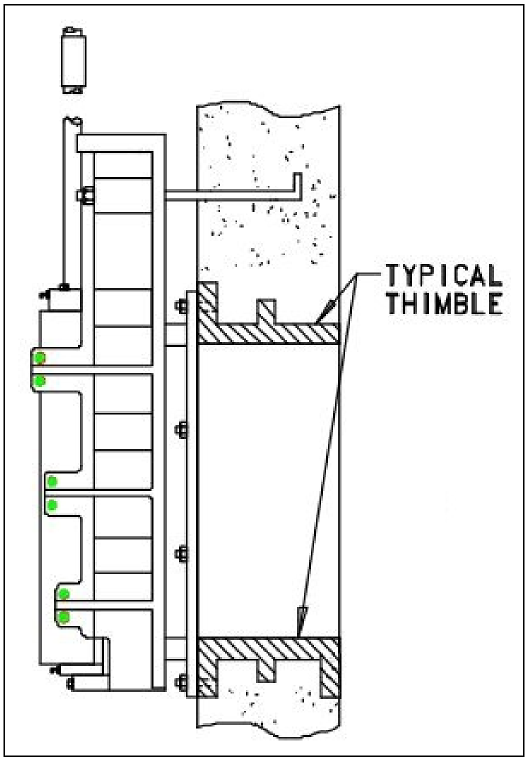


Figure 9-17. Wall thimbles, F-type (top) and E-type (bottom).

9.4.1.4. Passive Gates.

9.4.1.4.1. General. Passive, or automatic, gates are required in cases where the water is likely to rise to the action stage (typically identified in the schedule of operations) with less than a 12-hour flood prediction time. Passive gates are also required on slower-rising bodies of water where visits from operating personnel prior to events to verify nothing is blocking the gate from closing are not practical due to excessive travel distances or poor access conditions.

9.4.1.4.2. Flap Gates. Typically located on outlet headwalls, flap gates operate by rotating freely on a hinge in response to water pressure (Figure 9-18). These metal or composite gates are usually circular in shape but can be rectangular for large pipes and are used on gravity and pump discharge pipes. During normal conditions (i.e., open outlet), the interior flow has enough head differential to open the gate, when properly lubricated, to allow the flow to pass out of the pipe. During a flood (i.e., closed outlet), the pressure from the flood waters on the waterside keeps the flap gate shut/sealed; hence, no active intervention is necessary.

(Courtesy of USACE Louisville District)
Figure 9-18. Typical flap gate.

9.4.1.4.3. Flap Gate Design. New or replacement flap gates must use a double hinge and require a frame with a batter of 2.5 degrees for gates larger than 48 inches and 5 degrees for gates 48 inches and smaller (Figure 9-19). The batter aids gate function by using gravity to increase the required head to open the gate during normal conditions, minimizing debris accumulation and providing a better seal. Consult Table 9-1 to determine when to use a passive gate. Reference ETL 1110-2-584, EM 1110-2-6054, and EM 1110-2-3105 for additional information and design requirements for flap gates.

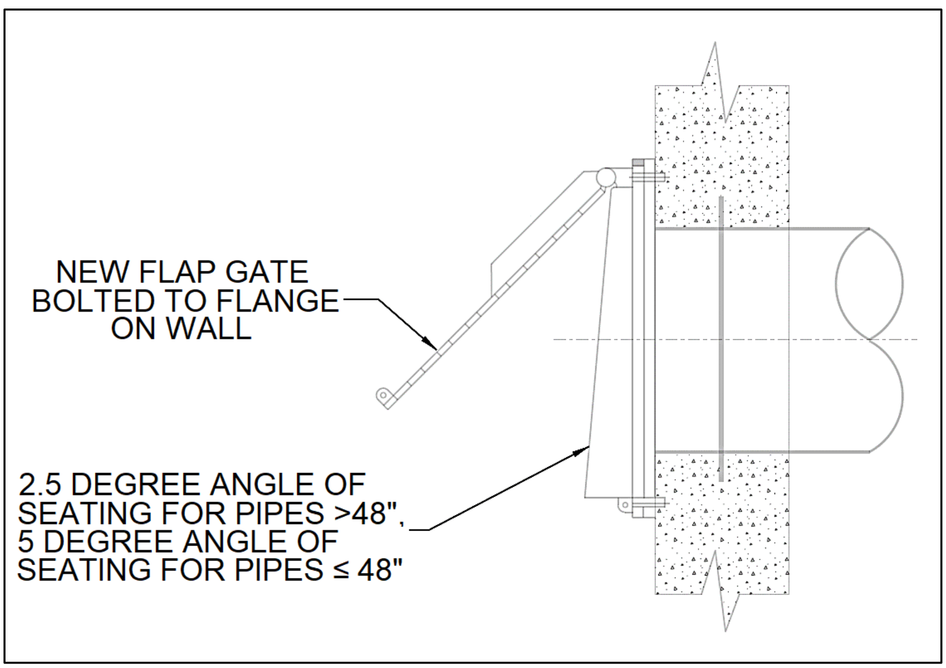


Figure 9-19. Flap gate detail showing batter.

9.4.1.4.4. Duckbill Check Valves. Duckbills are passive gates that can be installed in a recessed or protruding orientation. Inline duckbills are made of an elastomer without any stiffening elements and are located entirely within the pipe (Figure 9-20). Protruding duckbills are made of an elastomer with a stiffening element in the top of the valve for support (Figure 9-21 and Figure 9-22). Instead of a flap gate hinge, the duckbill's outlet end is machine rolled to create a tight seal. However, during normal conditions (i.e., open outlet), the water head inside the pipe can open the bottom portion of the duckbill and allow outward flow. During a flood event (i.e., closed outlet), the water head on the exterior of the pipe is greater than the interior and keeps the duckbill closed. Duckbill manufacturers are able to provide specific design considerations, maintenance requirements, and inspection guidance. Consider the following factors prior to selection of this type of closure.

- At the time of this manual's publication, the material cost of duckbills was lower compared to metal flap gates for pipes less than 36 inches in diameter, comparable in price for pipes between 36 and 48 inches, and higher for pipes greater than 48 inches.

- If a protruding duckbill is selected, it is very important that there is a minimum vertical clearance below the pipe of 10 percent of the pipe diameter so that the duckbill will not contact the bottom of the outlet concrete apron. Failure to do so will result in deformation of the duckbill in a sustained open position due to sag, which requires costly and time consuming remediation to correct (Figure 9-23).

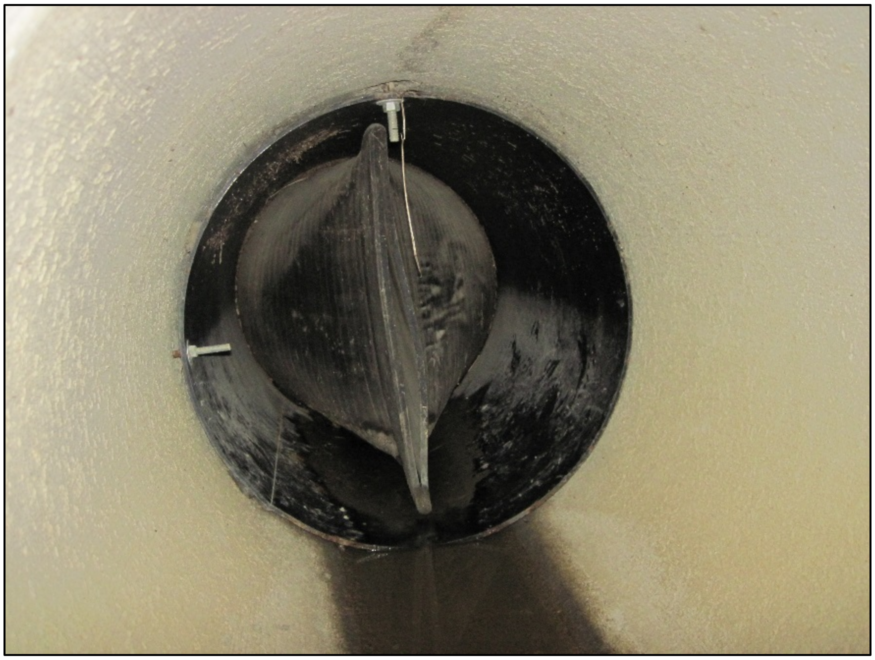
- When a protruding duckbill is placed at a headwall, the elastomer material is typically exposed to ultraviolet rays. This exposure will break down the material and increase the rate of deterioration, where a metal flap gate can simply be repainted for corrosion resistance. A recessed placement can reduce ultraviolet exposure. The material has a tendency to harden over time, reducing flexibility and increasing the likelihood of leaks due to cracking. Replacement costs should be factored into O&M in the event the duckbill no longer functions as it was intended to.

- Duckbills have a history of attracting nuisance species who consume the material, creating holes. Repair of these holes can be costly and is typically performed using a special patch system up to a certain size dependent on the manufacturer's recommendations.

- Because of the nature of the machine-rolled end and the difficulty involved with opening the duckbill, special tools may be required to remove any trapped debris in the duckbill and will hinder inspection of the pipe itself, although this is generally a rare occurrence. The debris removal process is simpler with a protruding duckbill.

- Duckbills are not built for fire survivability, which can cause issues in agricultural areas if the local landowners practice crop burning.

- Duckbills may be a better option in areas where theft or vandalism of metal flap gates is a realistic possibility, as duckbills are not usually targeted.



(Courtesy of USACE Louisville District)
Figure 9-20. Inline duckbill check valve viewed from the interior and facing the outlet.

(Courtesy of USACE Louisville District)
Figure 9-21. Exterior/protruding duckbill check valve inside gatewell.

(Courtesy of USACE Louisville District)
Figure 9-22. Exterior/protruding duckbill check valve at headwall.

(Courtesy of USACE Louisville District)
Figure 9-23. Exterior/protruding duckbills with deformation due to sag.

9.4.1.5. Exceptions to Required Closures. In rare scenarios, it may not be necessary to install a closure device on a pipe through a levee. If the pipe meets one of the following conditions, a closure device is not required:

9.4.1.5.1. Exception 1. Closures are not required if under normal conditions, the pipe allows for the normal tidal flux of water or allows for the normal migration of aquatic species to go upstream and downstream of the pipe for environmental reasons. This will likely be documented through the project's National Environmental Protection Act (NEPA) documentation and in the Design Documentation Report (DDR).

9.4.1.5.2. Exception 2. Closures are not required when pipes are sized such that during flood events, up to the top of levee loading, the volume of water passing through the pipe does not produce an unacceptable risk of damage or life loss within the leveed area. These types of pipes are typically designed to provide environmentally beneficial flows even during flood events (Figure 9-24). The necessary hydrologic and hydraulic studies will likely be performed and documented through the project's NEPA documentation and in the DDR.

(Courtesy of USACE Jacksonville District)

Figure 9-24. Pipe without a gate for canal water level control.

9.4.2. Pumps. Pumps are commonly used to dewater the leveed area into a pipe and to prevent interior ponding during a high-water event. They are usually housed within a pump station (Figure 9-25), although it is possible to install pumps in an underground structure for smaller ponding areas (Figure 9-26). EM 1110-2-3105 contains additional information and design requirements for pumps, as well as inspection guidance. The levee system's O&M manual will also contain information on the proper operation of the pumps and routine maintenance instructions. Failure of a pump to operate can impact interior ponding of a levee system during a flood event (PFM-7). Installation of a trash screen at the intake reduces the chance of large debris entering the pump, and routine maintenance of the pump allows for longer service life and optimal operation.

(Courtesy of USACE Louisville District)
Figure 9-25. Typical pump intake in pump station sump area.

(Courtesy of USACE Louisville District)
Figure 9-26. Underground pump station structure.

9.4.3. Transition Pipes. The transition from a pump to the discharge pipe is made using a pump-specific fitting or pump elbow. Design information and special considerations for the fitting of discharge pipes are discussed in EM 1110-2-3105. Selection and design of pump station discharge pipes are covered in Chapter 3 and Chapter 4, respectively. PFMs associated with the design and installation of discharge pipes are discussed in Chapter 4 and Chapter 5, respectively.

9.4.4. Air Vents and Siphon Breakers.

9.4.4.1. General. When a pump is turned off and the profile discharge pipe is no longer pressurized, the flow can reverse back to the pump, potentially damaging it by causing it to spin in the opposite direction and siphon flow back into the leveed area (PFM-7). Backflow or back-siphoning can be prevented by installing an air vent or siphon breaker (Figure 9-27) on the crest, typically near the waterside coupling. Air vents are used if the pipe system does not operate as a siphon, while siphon breakers are used when it does and usually use a valve. When the pumps are turned off and the pressure drops, these devices automatically open to the outside environment and introduce air into the pipe at the highest point to cause the siphoning vacuum to break. Consider sizing air vents or siphon breakers to accommodate a point of access for inspection equipment. Pump station discharge pipes are difficult to inspect when they are within (profile) the embankment. The air vent or siphon breaker is a good location for inspection access if sized greater than six to eight inches. For colder climates, ice may block the appurtenance regularly, requiring removal. Reference EM 1110-2-3105 for specific design parameters and requirements.

(Courtesy of USACE Louisville District)

Figure 9-27. Typical pump station air vents.

9.4.4.2. Redundancy. If emergency measures are necessary for the prevention of backflow in new installations, a sluice gate, or valve can be installed at the outlet of the discharge pipe. Flap gates, though not required for elevated pipes, also reduce the chance of backflow by not allowing the vacuum to pull water from the discharge/outlet end of the pipe. Ensuring that the backflow prevention is free of debris and opened to the outside environment reduces the likelihood of interior flooding.

9.4.5. Valves on Pressurized Pipes. Valves are typically associated with third-party pipes carrying various types of fluids or gases. Reference Section 5.7.6 for location requirements. The type of valve selected will depend on the corrosivity of the flow, the configuration, the material of the pipe, the location of the pipe (e.g., elevated, within, beneath, or adjacent to a levee), and the means of closure desired. Valve manufacturers should be consulted for design and installation considerations. Valves are necessary to isolate the portion of the pipe near the levee where a problem may exist. If a third-party pipe ruptures or leaks, the levee could lose large amounts of material due to high pressures or could slough off if the levee is significantly saturated by the leak. Once an issue is observed, the likelihood of embankment or floodwall failure can be reduced by closing the valves to isolate the zone of leakage and make a repair. Regular maintenance, per manufacturers' recommendations, must be performed to ensure that valves will properly operate.

9.4.6. Other Appurtenances. When considering an appurtenance not discussed herein, consult EM 1110-2-6054 for metal structures and EM 1110-2-2002 for concrete structures.

9.5. Connections to Associated Structures.

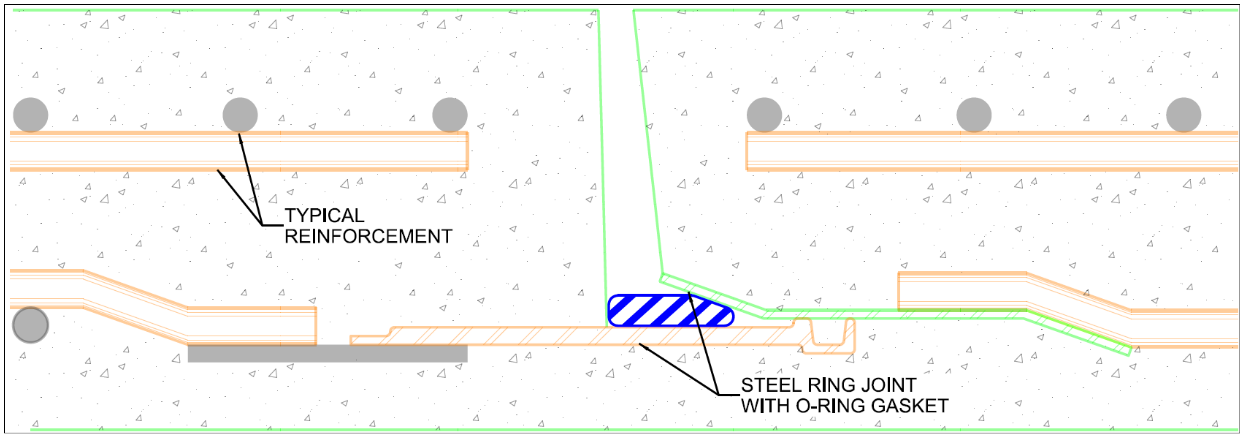
9.5.1. General. The connection of a pipe to an associated structure is critical to ensure the integrity of the levee; however, these connections are often stressed due to the high soil overburden adjacent to the connections in the levee. The connection to the structure can provide high shear forces on the pipe directly outside of the structure, which must be accounted for in the connection design. In addition, embankment material must be prevented from moving out of or into pipe connections of all types to reduce the probability of initiating PFM-3 (Figure 9-28). Pipe-to-structure connections must be installed to reduce the likelihood of embankment material loss.

(Courtesy of USACE St. Louis District)
Figure 9-28. Manhole with a problematic connection.

9.5.2. Non-Pressure and Low-Pressure Reinforced Concrete Pipe (RCP) to Associated Structure Connections.

9.5.2.1. Cast-in-Place and Precast. The connection between RCP and other structures in levees must meet the requirements of ASTM C923. RCP connections to cast-in-place structures must include a waterstop meeting EM 1110-2-2102 requirements. Grouted or mastic connections are not acceptable in levees for RCP. For connections to cast-in-place structures, a thimble with a waterstop gasket should be used around the exterior of the thimble and casted with the structure when it is formed and poured. For connections to precast structures, a resilient connection (booted with a cast-in compression gasket) meeting ASTM C923, ASTM C1478, ASTM F2510, or ASTM C1821 should be used.

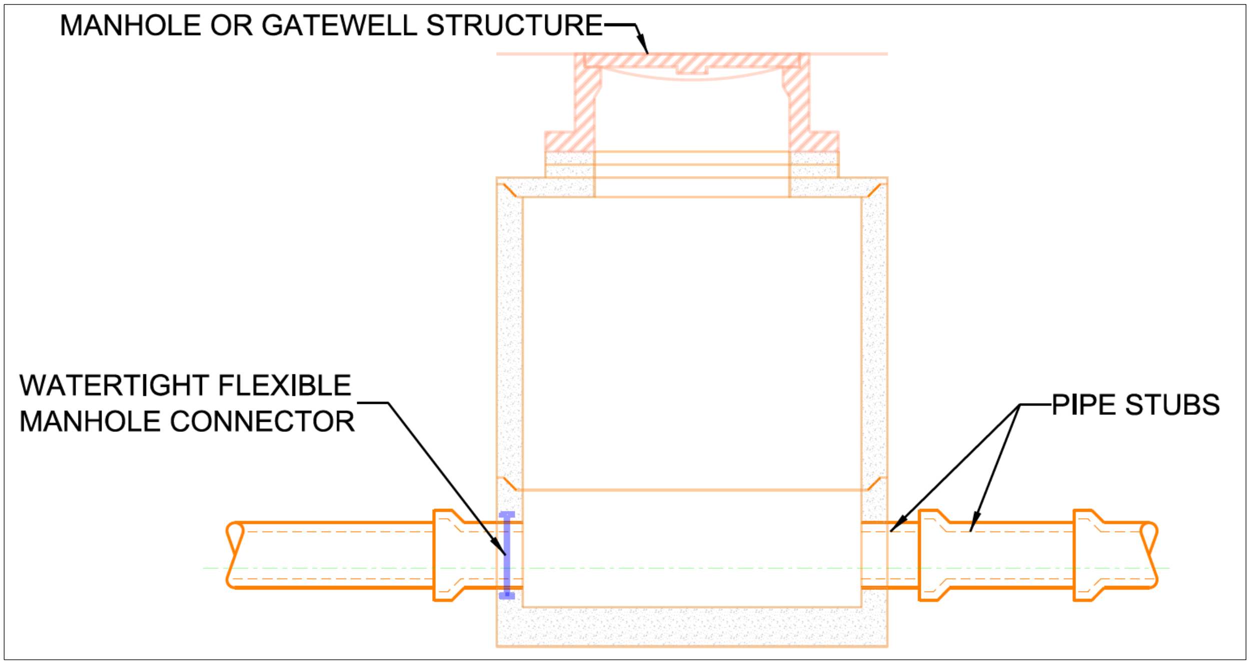
9.5.2.2. Steel Rings. Steel bell and spigot rings are steel rings that can be used to reinforce the joint end of RCP. These steel rings offer flexibility and additional shear strength at connections, where differential settlement between the structure and the pipe is a concern (Figure 9-29). Steel ring joints are also less susceptible to damage during installation than pipe ends without reinforcement. Steel ring joints must meet the requirements of ASTM C361 and American Water Works Association (AWWA) C302. When steel ring joints are not used at the connection, a short concrete pipe (12 to 24 inches) must be laid through the wall of the gatewell or intake structure, and the wall must be cast around the pipe. Two half-lengths of pipe must be used at each connection (upstream and downstream) to an associated structure to provide flexibility, which helps to relieve the high soil shear forces. All joints between RCP and other structures must be wrapped on the exterior with an external band gasket meeting the requirements of ASTM C877.



(Courtesy of American Concrete Pipe Association)
Figure 9-29. Steel ring joints with O-ring gasket.

9.5.3. Concrete Pressure Pipe (CPP) to Associated Structure Connections. With the exception of internal valves and sluice gates, pressure pipes are generally not connected to other associated structures or appurtenances since they must be capable of holding pressurized fluid. However, its high hydrostatic head capacity and traditional steel joint configuration have proven CPP is a viable pipe for many applications where there is a potential for higher pressures, whether frequent or rare. Special structure closure pieces can be fabricated to implement structure connections similar to RCP. Additionally, flanges for CPP are fabricated from steel to match the drilling pattern for any flanged valve, active gate, or similar structure. Gasket joints to match existing pipe can be formed of steel and rolled or milled to any diameter and tolerance. The steel end is then welded to the CPP closure in circular pipes and is provided with cement mortar coating and lining, as appropriate for the joint and installation method.

9.5.4. Vitrified Clay Pipe (VCP) to Associated Structure Connections. Reference Chapter 9 of the Vitrified Clay Pipe Engineer Manual for requirements for connections between VCP and associated structure. For levee applications, one short-length pipe with one flexible connector required for levee applications is required to provide necessary flexibility. The required method to provide pipe flexibility at a connection to another structure in levees is shown on the left in Figure 9-30; the right shows a traditional connection for non-levee applications. When the manhole connector is utilized as one of the points of flexibility, no mortar may be placed between the pipe and the wall of the concrete structure. Failure to center the pipe or apply the mortar in the opening will diminish the ability of the connector to compensate for differential settlement.

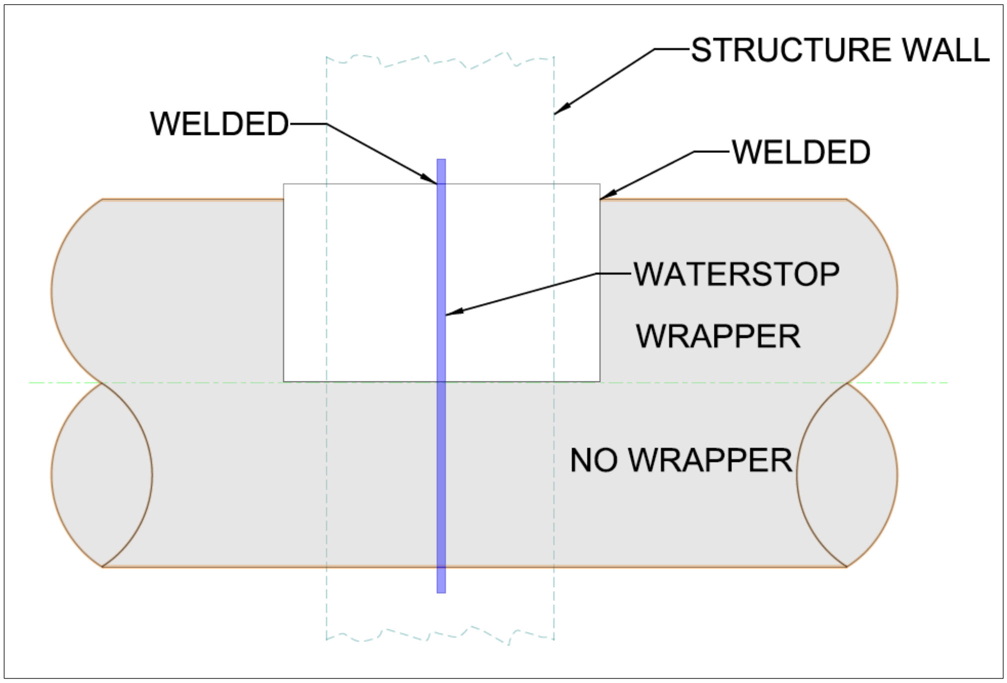


(Courtesy of National Clay Pipe Institute)

Figure 9-30. VCP segment with flexible connector for levees (left); stub joints for non-levee applications (right).

9.5.5. Corrugated Steel Pipe (CSP) to Associated Structure Connections. Connections of associated structures to CSP are similar to CSP pipe segment-to-segment connections, with the exception of the use of waterstops in the structure connection. Concrete headwalls must be made with an embedded CSP stub. The pipe end is anchored to the headwall or beam with 0.75-inch diameter anchors.

9.5.6. Welded Seam Steel Pipe (WSSP) to Associated Structure Connections. Similar to CPP, pressure pipes should not be connected to non-pressurized structures. Flanges for WSSP are fabricated from steel to match the drilling pattern for any flanged valve, active gate, or similar structure. Gasket joints to match the existing pipe can be formed of steel and rolled or milled to any diameter and tolerance. The steel end is then welded to the WSSP closure and provided with a coating and a lining, as appropriate for the joint and installation method. In some applications, a seep ring is needed and may be welded to the WSSP (Figure 9-31). Design of the ring must be done according to AWWA M11. Reference Chapter 7 of the AWWA Manual to determine when the wrapper plate shown is required (it is usually not).

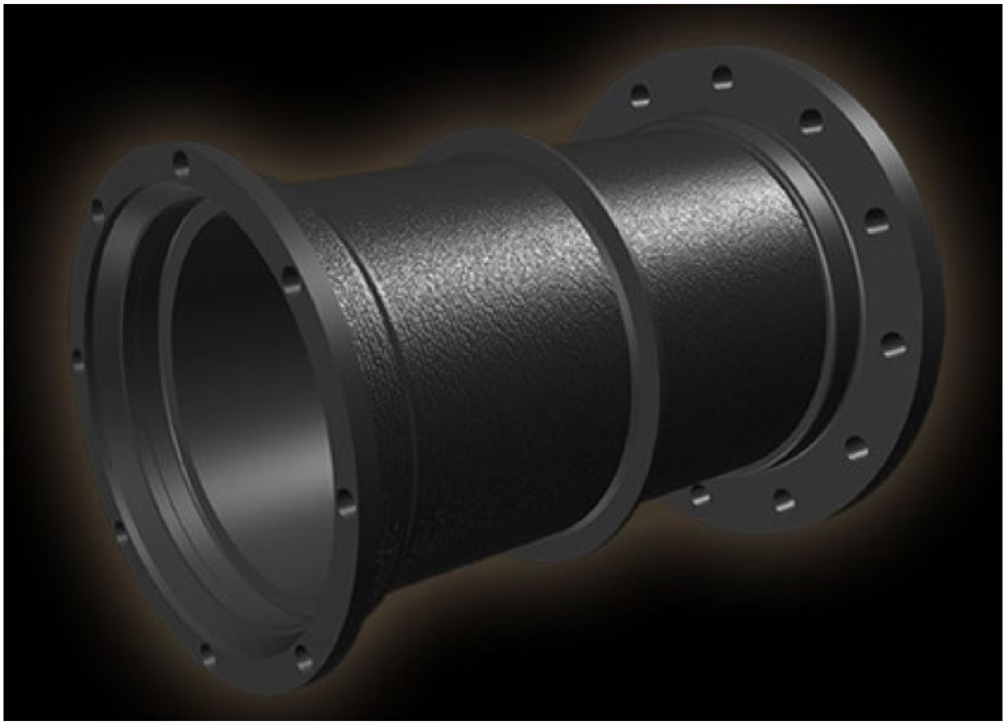


(Courtesy of Steel Fabricators Pipe Association)

Figure 9-31. Welded seam steel pipe connection to structure.

9.5.7. Corrugated Aluminum Pipe to Associated Structure Connections. Because of corrosive activity between aluminum and concrete, USACE does not allow aluminum pipe connections to associated cementitious structures in levee applications.

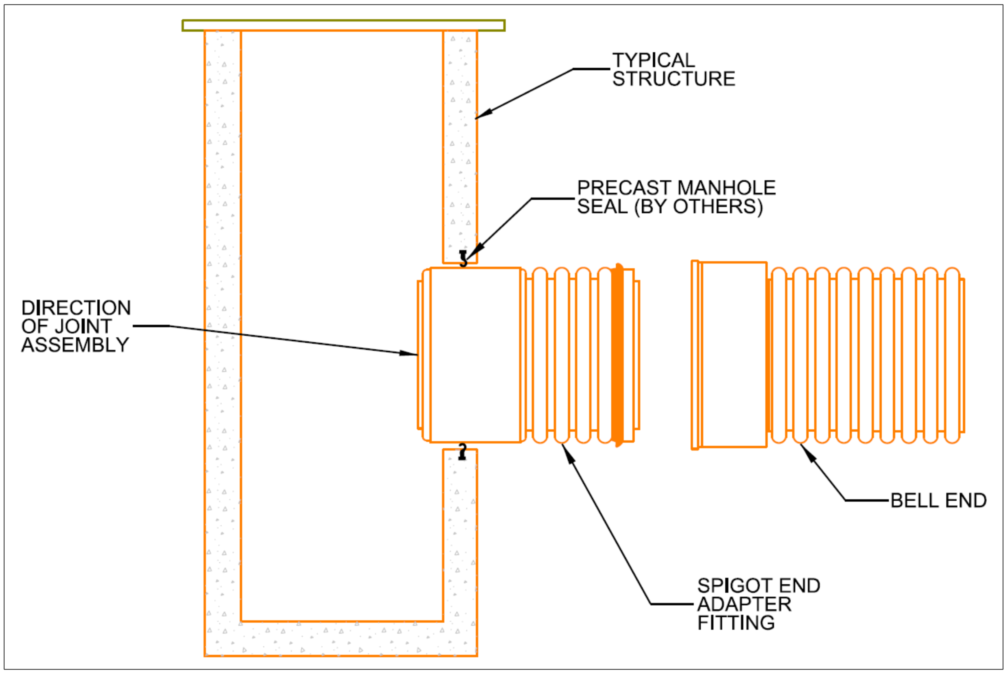
9.5.8. Ductile Iron Pipe (DIP) to Associated Structure Connections. In pressure applications for DIP, "full-body" fittings and thick walls should be used, as defined in and meeting all requirements of ANSI/AWWA C110/A21.10. For other DIP applications, "short-body" fittings and thin walls should be used, as defined and meeting all requirements of ANSI/AWWA C153/A21.53 (Figure 9-32). Smaller diameter pipes correlate to thinner walls; see the referenced standards for clarification. Reference ANSI/AWWA C116/A21.16 for information on the interior and exterior surfaces of ductile iron and gray-iron fittings.



(Courtesy of Ductile Iron Pipe Research Association)
Figure 9-32. DIP wall fitting.

9.5.9. Thermoplastic Pipe.

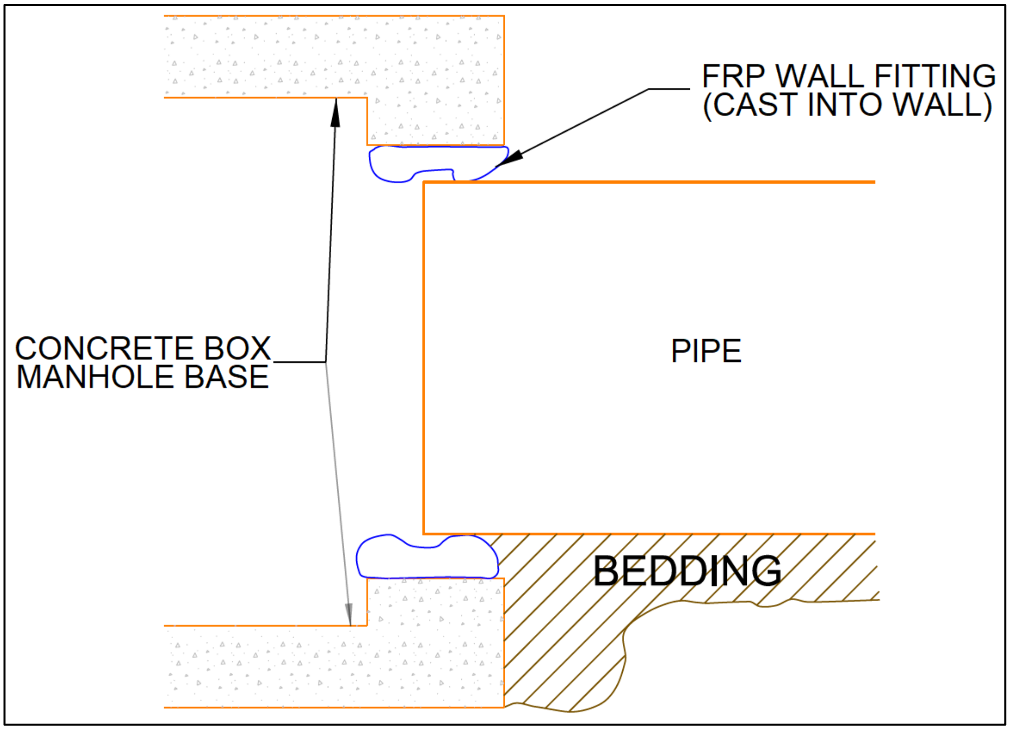
9.5.9.1. Corrugated High Density Polyethylene (HDPE) Pipe to Associated Structure Connections. Grouted joints between single-wall and profile-wall HDPE pipes prevent soil infiltration. Gaskets or boots must be used to construct joints between pipe sections and other structures. Compression gaskets are cast into the structure wall around the opening of the pipe. A smooth-walled adapter band on the pipe forms the seal with the compressible rubber gasket. Boot rings are anchored to the edges of the opening in the concrete structure and to a smooth-walled adapter band placed around the pipe section (Figure 9-33). For both boots and compression gaskets, the first pipe section coupler must be no more than 18 inches from the connection to the structure, and adequately supported or encased in a CLSM (reference Section 5.5.10.). ADS TN 1.04 and ADS TN 5.04 offer information regarding connection options and installation recommendations.



(Courtesy of Advance Drainage Systems)
Figure 9-33. Typical plastic pipe connection detail.

9.5.9.2. Solid-Wall High Density Polyethylene to Associated Structure Connections. Grouted joints including chemical grouts (excluding oakum) between solid-wall HDPE pipes prevent soil infiltration. Compressive rubber seals and other flexible connection fittings are used to connect HDPE and other plastic pipe to other structures and can reduce the likelihood of embankment material loss. Fitting designs vary, so it is important to follow the manufacturer's guidance.

9.5.9.3. Fiberglass Reinforced Pipe (FRP) to Associated Structure Connections. FRP can be connected to concrete structures using cast-in gaskets cast into the opening, expandable boot-seal connections for precast structures, cast-in FRP coupling wall fittings, or waterstops grouted into place in the opening (Figure 9-34). Connections must use cast-in connections to reduce the likelihood of embankment material loss and must be restrained, flexible joints capable of resisting longitudinal forces while accommodating rotation movement. Connections to other pipe types (reference Section 5.5.12.) such as stubs extending from existing structures may be accomplished by several methods. Custom fabricated mechanical couplings capable of connecting pipes of different outer diameters may be used.



(Courtesy of Advanced Drainage Systems)
Figure 9-34. Typical FRP connection detail.

9.6. Section 408 Considerations for Associated Structures and Appurtenances for Levees. The respective USACE District may review documentation related to proposed modifications of associated structures and appurtenances if such actions are subject to 33 USC 408 and require approval. Reference Appendix H for a list of suggested documentation for Section 408, if needed.

Chapter 10
Associated Structures and Appurtenances for Dams

10.1. Introduction. Chapter 10 discusses the function, references relevant design guidance, and addresses concerns with a dam's outlet works (to include the conduit and its associated structures and appurtenances (Figure 10-1 and EM 1110-2-2400). The outlet works of a dam consist of several elements designed to control the release of water from a reservoir. An associated structure is a structure within the outlet works that is directly connected to the conduit, while an appurtenance is a device within or attached to an associated structure. The identified associated structures and appurtenances in this chapter are integrated into the conduit system and are integral to its function. Each structure and appurtenance can also contribute to failure of the embankment.

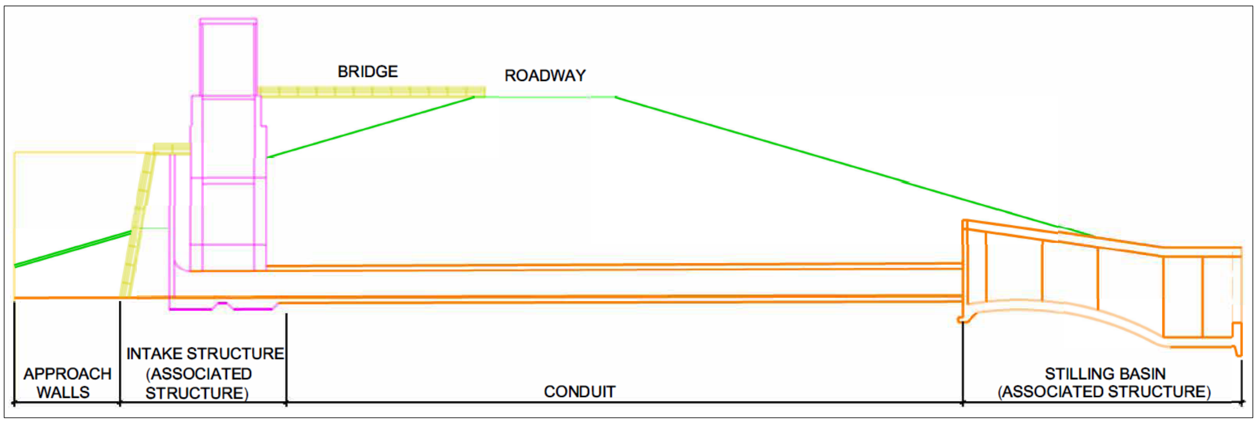


Figure 10-1. Typical configuration of a dam and outlet works.

10.2. Potential Failure Modes Related to Associated Structures and Appurtenances for Dams. Conduits are often risk drivers in dams due to PFMs related to the loss of embankment material through a conduit defect. All PFMs described in Chapter 2 may be applicable to conduits discussed in this chapter. PFM-1 is more likely to occur in dams that do not have filtering systems, such as dams with "chimney drains" or sand envelopes around the conduit. A conduit through a dam can become pressurized during large releases and can contribute to PFM-2. PFM-3 is possible at construction joints of associated structures and at connections between associated structures and conduits that are not cast-in-place. The inability of the stilling basin to adequately reduce the energy at the outlet can contribute to erosion at the toe of a dam and eventual dam failure due to PFM-4.

10.3. Associated Structures.

10.3.1. General. Associated structures for dams are components that reduce turbulent flow from an outlet structure (stilling basin), control flow through a conduit (control tower), provide hydroelectric power (penstocks), or allow flow into a spillway (shaft spillway). The following discussions cover several common types of associated structures used with conduits in dams; their function, typical location, and design criteria; references to design and inspection resources; and concerns related to dam failure.

10.3.2. Control Tower. The control tower or intake structure is typically constructed on the upstream side of a dam (near the dam toe) and is usually free-standing with bridge access from the dam (Figure 10-2). The intake structure houses the appurtenances that perform several different functions in the dam outlet works system. Control towers can serve as the entrance to the conduit, allowing the flow of water from the reservoir to the tailwater downstream to regulate reservoir levels. To reduce such concerns, alternate foundation systems or alignments should be explored. Additional information and design considerations for control towers are found in EM 1110-2-2104 and EM 1110-2-2400. Control towers are inspected according to ER 1110-2-1156. Reference Section 10.5 for discussions related to the control tower and conduit connection.

(Courtesy of USACE Louisville District)

Figure 10-2. Typical reservoir control tower.

10.3.3. Shaft Spillway. A shaft spillway, such as a morning glory inlet, is a vertical or sloping shaft (conduit) with a bell-shaped inlet that passes water beneath the dam to the tailwater to reduce the probability of overtopping (Figure 10-3, Figure 10-4, and Figure 10-5). EM 1110-2-1603 addresses the hydraulic design of shaft spillways as well as their advantages and disadvantages. In some scenarios, the dam geometry or layout of the land near the dam may require a shaft spillway configuration as opposed to an open-cut or gated spillway. The probability of dam failure associated with PFM-1, PFM-2, and PFM-3 can be greatly reduced when it is practical to install the shaft spillway through the underlying rock instead of the overlying soil, which essentially eliminates the issue of soil migration and internal erosion. If the shaft passes through the dam, the tunnel is subjected to the same PFMs as a conduit through a dam.

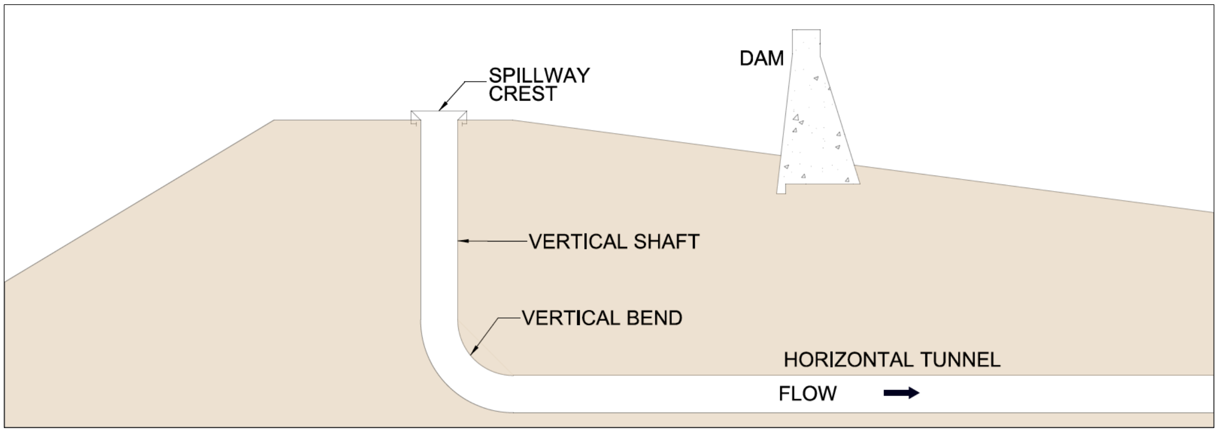
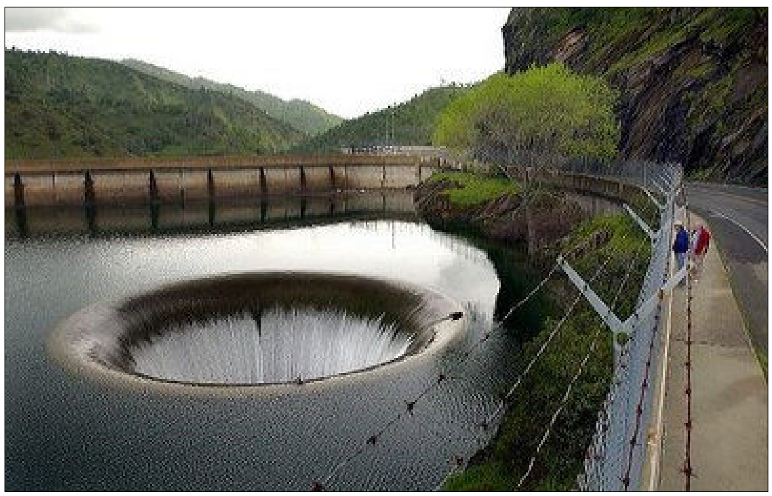


Figure 10-3. Diagram of a vertical shaft spillway.



(Courtesy of Solano Irrigation District, CA)
Figure 10-4. Morning glory inlet in operation.

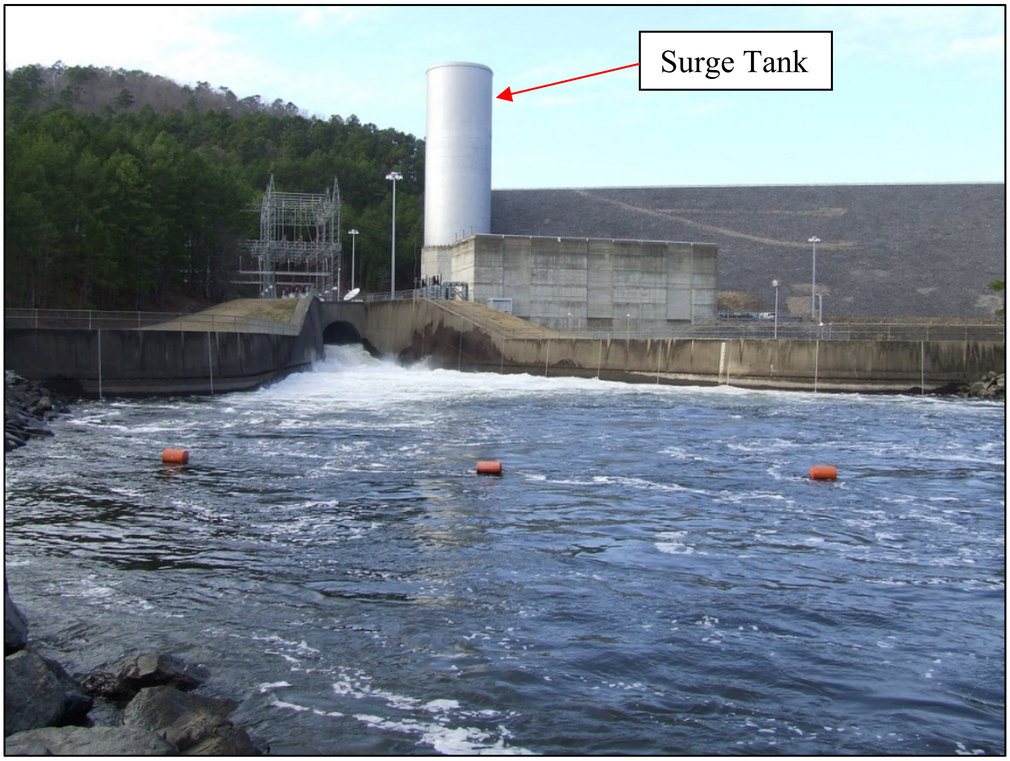
(Courtesy of USACE Huntington District)
Figure 10-5. Morning glory inlet.

10.3.4. Penstocks and Surge Tanks.

10.3.4.1. General. Penstocks are another form of pipe and are typically used for hydroelectric generation in dams. They are equipped with surge tanks to prevent water hammer within piping and gate systems and to regulate flow through a turbine to generate electricity (Figure 10-6 and Figure 10-7). They can be built within the embankment or outside of the dam on the slope from the crest to the toe. Penstocks are usually concrete within the dam and use steel and welded/bolted joints where the dynamic pressures are higher (i.e., near the power house). Design information for penstocks and surge tanks is found in EM 1110-2-3001 and additional information can be found in the American Society of Civil Engineers (ASCE) Manual of Practice (MOP) 79.

(Courtesy of USACE Seattle District)

Figure 10-6. Penstocks outside of embankment.



(Courtesy of USACE Vicksburg District)
Figure 10-7. Hydroelectric dam surge tank.

10.3.4.2. Areas of Concern. The main concern with penstocks is the pressure associated with their operation. Within a dam, this pressurized pipe could allow pressurized water to create a seepage path through the dam or along the outside of the pipe and/or saturate an area of the embankment (PFM-2). Ideally, penstocks are constructed outside of the dam, as this location greatly reduces the likelihood of failure of the embankment and facilitates easier inspection of the joints. If the configuration of the dam does not allow for construction of the penstock on the slope surface, regularly scheduled inspections can still identify defects with the pipe, whether at the joint or in the pipe walls.

10.3.5. Energy-Dissipating Structures.

10.3.5.1. General. An energy-dissipating structure is generally required at the conduit outlet to prevent downstream channel degradation. The specific type of structure needed is determined by hydraulic model studies, but typically such structures include abrupt expansions into high pressure conduits. Examples can include hydraulic jump stilling basins, flip bucket plunge pool structures, valves, deflectors, and baffle block structures (Figure 10-8 and Figure 10-9).

(Courtesy of USACE Louisville District)
Figure 10-8. Stilling basin.

(Courtesy of USACE Louisville District)
Figure 10-9. Baffle blocks.

10.3.5.2. Design. The stability of energy-dissipating structures must meet EM 1110-2-2100 requirements unless they are pile founded, in which case they must meet EM 1110-2-2906 requirements. Guidance on spillway-type stilling basins can be found in EM 1110-2-2200. The structural design for energy-dissipating structures must be according to EM 1110-2-2104, and the hydraulic design must meet in EM 1110-2-1602 requirements.

10.3.5.3. Areas of Concern. The function of energy-dissipating structures is to eliminate the conditions leading to PFM-4, so it is important that they maintain good structural condition to avoid erosion at the outlet. It is not unusual for large debris to flow through the dam conduit and cause damage to these structures. An in-depth inspection, which involves dewatering to identify defects, is needed to make repairs and reduce the likelihood of future erosion in the outlet area (reference ER 1110-2-1156). Particular attention is given to the joints and any cracks within the floor or walls of the structure, as they can form unfiltered exits. The transition between the downstream end of the conduit and the energy-dissipating structure is important and can lead to issues leading to PFM-3. Because of the turbulent flow within the structure, it is necessary to isolate the flow and the conduit with a construction joint and waterstop.

10.3.6. Water Supply Pipes. In rare cases, it may be necessary to install a water line within the dam. Refer to the considerations mentioned in ER 1110-2-1156 regarding the installation of pressure lines within dams for more information and precautions. To reduce the likelihood of a pressure pipe rupture saturating the embankment, the water supply pipes must be installed as shallow as the frost depth will allow to make surface expressions of distress more readily observable, and to reduce the amount of embankment disturbed if excavation of the pipe is required. Additionally, a shutoff valve must be located before the pipe enters the dam embankment to allow for shutting off flow if leaking is observed or a rupture occurs.

10.4. Appurtenances.

10.4.1. Gates.

10.4.1.1. Service Gates. Several types of gates are used within the control tower to control water level releases from a reservoir. During normal operation, service gates are raised or lowered to allow for flow to pass through the conduit to the tailwater. Service gates are permanently installed within the tower and operate along slots within its concrete walls (Figure 10-10). Typically there is a passageway for each service gate within the control tower, and all passageways connect at the entrance of the conduit. For each gate, an independent air vent must be installed with an intake invert elevation above the probable maximum flood elevation to avoid cavitation damage to the metal and concrete of the conduit. Each gate is designed according to EM 1110-2-1602.

(Courtesy of USACE Louisville District)
Figure 10-10. Service gate.

10.4.1.2. Emergency Gates. If a service gate becomes inoperable, an emergency gate can be used to block the affected passageway to stop uncontrolled flow. For guidance regarding the need for emergency gates, reference EM 1110-2-2400. A service gate stuck in the fully open position could lead to longer than desired durations of turbulent flow or downstream flooding. Continued high velocity flows could lead to thinning of the conduit walls from cavitation damage and the wearing of energy-dissipating structures. This could lead to the loss of embankment material (PFM-2) and erosion of the dam toe (PFM-4). Conversely, a service gate stuck in the closed position could lead to spillway flow or overtopping of the dam. The likelihood of dam failure associated with gates is commonly reduced by constructing the dam with two or more passageways, with a service and emergency gate in each passageway. Regularly scheduled inspections should assess the condition of the gates to address any concerns with performance.

10.4.2. Air Vents. Air vents are usually constructed within the walls of a control tower (Figure 10-11). The vents originate at an opening in the top of the conduit just downstream of the service gates and terminate with an invert elevation above the probable maximum flood, opening to the outside atmosphere. Reference EM 1110-2-1602 for design guidance regarding air vents. A properly operating air vent can prevent the dam conduit from becoming pressurized, while an obstructed air vent could pressurize the conduit and create conditions leading to PFM-2. Regularly scheduled inspections (adhering to ER 1110-2-1156) reduce the likelihood of this PFM by identifying issues for repair.

(Courtesy of USACE Louisville District)

Figure 10-11. Air vent downstream of service gate.

10.4.3. Bypass Valves.

10.4.3.1. General. Since the entrance to a conduit is typically located at the bottom of the reservoir, releases through the service gate can introduce low temperature water with low oxygen content to the tailwater, which can negatively impact downstream wildlife. To remedy this issue, some control towers have multi-level gate systems that allow water from different elevations of the reservoir (at varying temperatures and oxygen levels) into a bypass well and eventually through the bypass valves (Figure 10-12). The bypass valves allow the mixed water to enter into the conduit along a passageway wall. During times of low release, the bypass valves can be used in lieu of the service gates to limit wear on the service gates. For guidance regarding bypass valves, reference EM 1110-2-2400.

(Courtesy of USACE Louisville District)

Figure 10-12. Bypass knife gate (left) and bypass valve (right).

10.4.3.2. Areas of Concern. Given the nature of bypass valves as well as their relatively small size in comparison to service gates, they usually do not allow enough flow to create any concerns for the conduit. However, they are important as they are typically used to regulate reservoir levels during normal operation of the dam. Excessive flows due to mechanical failure of the bypass valve could threaten the integrity of the bypass pipe. Regularly scheduled inspections must be performed to monitor changes in the valves' condition over time.

10.5. Connections to Associated Structures and Appurtenances.

10.5.1. General. Connection locations for a conduit through a dam usually occur at the control tower on the upstream end of the conduit and at the stilling basin on the downstream end. The connection between the control tower and conduit is typically cast-in-place, and at times, is constructed at the same time as the conduit. At the stilling basin end of the conduit, a construction joint is placed between the stilling basin structure and the conduit. The joint itself must be constructed with a waterstop. EM 1110-2-2400 provides additional guidance regarding connections. For other pipes through embankments not associated with the operation of the dam, connections may be constructed using the methods described in Section 9.5.

10.5.2. Areas of Concern. Inspectors must investigate if concrete defects such as cracks and spalls in the conduit could contribute to connection failure. Collapse of the tower connection could lead to loss of embankment, while collapse of the floor could obstruct the flow through the conduit during dam releases. Inspections conducted as required by ER 1110-2-1156 should catch these issues early and allow for repairs to prevent failure of the connection or of the tower floor. Other connection concerns can become apparent if a control tower and conduit are founded at different elevations on different materials, as this can result in differential settlement between the two structures. Seismic design is also important to consider because of the nature of the tower movements with respect to the adjoining embankment.

Appendix A
References

Required References

Public Laws and Statutes

33 U.S.C. § 408, Taking Possession of, Use of, or Injury to Harbor or River Improvements (https://www.govinfo.gov/app/details/USCODE-2011-title33/USCODE-2011-title33-chap9-subchapI-sec408).

Regulations

49 CFR Part 192, Transportation of Natural and Other Gas by Pipeline: Minimum Federal Safety Standards (https://www.govinfo.gov/app/details/CFR-2011-title49-vol3/CFR-2011-title49-vol3-part192).

49 CFR Part 195, Transportation of Hazardous Liquids by Pipeline (https://www.govinfo.gov/app/details/CFR-2011-title49-vol3/CFR-2011-title49-vol3-part195).

DoD Directives, Instructions, Regulations, Standards and Other Publications

UFC 3-220-05, Dewatering and Groundwater Control (https://www.wbdg.org/ffc/dod/unified-facilities-criteria-ufc/ufc-3-220-05).

Corps of Engineers Publications

Engineer Regulation (ER) 1110-2-1150, Engineering and Design for Civil Works Projects (https://www.publications.usace.army.mil/Portals/76/Publications/EngineerRegulations/ER_1110-2-1150.pdf).

ER 1110-2-1156, Safety of Dams – Policy and Procedures (https://www.publications.usace.army.mil/Portals/76/Publications/EngineerRegulations/ER_1110-2-1156.pdf).

ER 1110-2-8152, Planning and Design of Temporary Cofferdams and Braced Excavations (https://www.publications.usace.army.mil/Portals/76/Publications/EngineerRegulations/ER_1110-2-8152.pdf?ver=bJHoQNne3hHCndWfleteHg%3d%3d).

ER 1110-2-8157, Responsibility for Hydraulic Steel Structures (https://www.publications.usace.army.mil/Portals/76/Publications/EngineerRegulations/ER_1110-2-8157.pdf?ver=45UrI9qnWBu6bYl2SnUiiA%3d%3d).

ER 1110-2-8159, Life Cycle Design and Performance (https://www.publications.usace.army.mil/Portals/76/Publications/EngineerRegulations/ER_1110-2-8159.pdf?ver=2013-09-08-233429-950).

Engineer Manual (EM) 385-1-1, Safety and Health Requirements Manual (https://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_385-1-1.pdf?ver=7Cpck_22Ct_-w6QSGyOKVQ%3d%3d).

EM 1110-2-1413, Hydrologic Analysis of Interior Areas (https://www.publications.usace.army.mil/Portals/76/Users/227/19/2019/EM_1110-2-1413.pdf?ver=vXkP1EpOfhBDNqKoYxfSMw%3d%3d).

EM 1110-2-1421, Groundwater Hydrology (https://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-1421.pdf?ver=2013-09-04-070801-780).

EM 1110-2-1601, Hydraulic Design of Flood Control Channels (https://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-1601.pdf).

EM 1110-2-1602, Hydraulic Design of Reservoir Outlet Works (https://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-1602.pdf?ver=2013-09-04-161103-333).

EM 1110-2-1603, Hydraulic Design of Spillways (https://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-1603.pdf).

EM 1110-2-1901, Tunnels and Shafts in Rock (https://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-1901.pdf).

EM 1110-1-1904, Settlement Analysis (https://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-1-1904.pdf).

EM 1110-2-1913, Design and Construction of Levees (https://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-1913.pdf).

EM 1110-2-1914, Design, Construction, and Maintenance of Relief Wells (https://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-1914.pdf).

EM 1110-2-2002, Evaluation and Repair of Concrete Structures (https://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-2002.pdf).

EM 1110-2-2100, Stability Analysis of Concrete Structures (https://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-2100.pdf).

EM 1110-2-2102, Water Stops and Other Preformed Joint Materials for Civil Works Structures (https://www.publications.usace.army.mil/portals/76/publications/engineermanuals/em_1110-2-2102.pdf).

EM 1110-2-2104, Strength Design for Reinforced Concrete Hydraulic Structures (https://www.publications.usace.army.mil/portals/76/publications/engineermanuals/em_1110-2-2104.pdf).

EM 1110-2-2200, Gravity Dam Design (https://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-2200.pdf).

EM 1110-2-2300, General Design and Construction Considerations for Earth and Rock-Fill Dams (https://www.publications.usace.army.mil/portals/76/publications/engineermanuals/em_1110-2-2300.pdf).

EM 1110-2-2400, Structural Design and Evaluation of Outlet Works (https://www.publications.usace.army.mil/Portals/76/publications/EngineerManuals/EM_1110-2-2400.pdf).

EM 1110-2-2502, Retaining and Flood Walls (https://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-2502.pdf).

EM 1110-2-2906, Design of Pile Foundations (https://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-2906.pdf).

EM 1110-2-3001, Planning and Design of Hydroelectric Power Plant Structures (https://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-3001.pdf).

EM 1110-2-3104, Structural and Architectural Design of Pumping Stations (https://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-3104.pdf).

EM 1110-2-3105, Mechanical and Electrical Design of Pump Stations (https://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-3105.pdf).

EM 1110-2-3506, Engineering and Design, Grouting Technology (https://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-3506.pdf).

EM 1110-2-6054, Inspection, Evaluation and Repair of Hydraulic Steel Structures (https://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-6054.pdf).

EM 1110-3-136, Drainage and Erosion Control Mobilization Construction (https://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-3-136.pdf?ver=2013-09-04-161200-380%20).

Engineer Circular (EC) 1165-2-220, Policy and Procedural Guidance for Processing Requests to Alter U.S. Army Corps of Engineers Civil Works Projects Pursuant to 33 USC 408 (https://www.publications.usace.army.mil/Portals/76/Publications/EngineerCirculars/EC_1165-2-220.pdf).

USACE, 1998. CPAR-GL-98-1, Construction Productivity Advancement Research, Installation of Pipelines beneath Levees using Horizontal Directional Drilling (https://apps.dtic.mil/sti/pdfs/ADA345211.pdf).

CHNLPRO Version 2.0 [Computer software]. Army Engineer Waterways Experiment Station, Coastal Hydraulics Lab; Vicksburg, MS.

Conroy, P., Latorre, C., and Wakeley, L., 2002. Guidelines for Installation of Utilities Beneath Corps of Engineers Levees Using Horizontal Directional Drilling Techniques. USACE Technical Report, ERDC/GSL TR-02-9 (http://hdl.handle.net/11681/10539).

Eff, K., 1960. Effects of Loads on Buried Pipes. USACE Office of the Chief of Engineers; Vicksburg, MS.

Maynord, S. T., Hebler, M. T., and Knight, S. F., 1998. User's Manual for CHANLPRO, PC Program for Channel Protection Design. Army Engineer Waterways Experiment Station, Coastal Hydraulics Lab; Vicksburg, MS (https://ntrl.ntis.gov/NTRL/dashboard/searchResults/titleDetail/ADA351838.xhtml).

Simicevic, J. and Sterling, R. L., 2001. TTC Technical Report #2001.04. USACE Engineer Research and Development Center (ERDC); Vicksburg, MS (https://citeseerx.ist.psu.edu/viewdoc/download?doi=10.1.1.509.6025&rep=rep1&type=pdf).

Other Federal Agency Publications

Federal Emergency Management Agency (FEMA), 2005. FEMA 484, Technical Manual: Conduits through Embankment Dams, Best Practices for Design, Construction, Problem Identification and Evaluation, Inspection, Maintenance, Renovation, and Repair (https://www.fema.gov/media-library-data/20130726-1515-20490-8766/fema484.pdf).

FEMA, 2007. Technical Manual: Plastic Pipe Used in Embankment Dams, Best Practices for Design, Construction, Problem Identification and Evaluation, Inspection, Maintenance, Renovation, and Repair (https://www.fema.gov/media-library-data/20130726-1633-20490-0008/femap_675part1.pdf).

FEMA, 2011. Filters for Embankment Dams – Best Practices for Design and Construction (https://www.fema.gov/media-library-data/1522783448418-8294f1e44e9509a1313f3930e5519d0e/filters_embankment_dams_508_update.pdf).

Federal Highway Administration (FWHA). FHWA-HIF-12-026, Hydraulic Design of Highway Culverts (https://www.fhwa.dot.gov/engineering/hydraulics/pubs/12026/hif12026.pdf).

U.S. Department of the Interior and USACE, 2015. Best Practices in Dam and Levee Safety Risk Analysis, A Joint Publication by U.S. Department of the Interior, Bureau of Reclamation, and U.S. Army Corps of Engineers (https://www.usbr.gov/ssle/damsafety/risk/BestPractices/Chapters/1-BestPracticesCover.pdf).

U.S. Bureau of Reclamation, March 2007. Water Operation and Maintenance Bulletin No. 219 (https://www.usbr.gov/assetmanagement/WaterBulletins/219mar07.pdf).

Other Publications

American Association of State Highway and Transportation Officials (AASHTO), 2017. AASHTO LRFD Bridge Design Specifications, 8th Edition (https://www.transportation.org/).

AASHTO M 36, Standard Specification for Corrugated Steel Pipe, Metallic-Coated, for Sewers and Drains (https://www.transportation.org/).

AASHTO M 196, Standard Specification for Corrugated Aluminum Pipe for Sewers and Drains (https://www.transportation.org/).

AASHTO M 197, Standard Specification for Aluminum Alloy Sheet for Corrugated Aluminum Pipe (https://www.transportation.org/).

AASHTO M 207, Standard Specification for Reinforced Concrete Elliptical Culvert, Storm Drain, and Sewer Pipe (https://www.transportation.org/).

AASHTO M 218, Standard Specification for Steel Sheet, Zinc-Coated (Galvanized), for Corrugated Steel Pipe (https://www.transportation.org/).

AASHTO M 252, Standard Specification for Corrugated Polyethylene Drainage Pipe (https://www.transportation.org/).

AASHTO M 274, Standard Specification for Steel Sheet, Aluminum-Coated (Type 2), for Corrugated Steel Pipe (https://www.transportation.org/).

AASHTO M 294, Standard Specification for Corrugated Polyethylene Pipe, 300- to 1500-mm (12- to 60-in.) Diameter (https://www.transportation.org/).

AASHTO R 82, Standard Practice for Pipe Joint Selection for Highway Culvert and Storm Drains (https://www.transportation.org/).

AASHTO, 2002. Standard Specifications for Highway Bridges, 17th Edition (https://www.transportation.org/).

Advanced Drainage System (ADS) TN 1.04, MH Adapter Guide – SaniTite HP and HP Storm Pipe (https://www.ads-pipe.com/engineering-resources).

ADS TN 5.04, HDPE and HP Storm Connections to Manholes and Structures for Storm Sewer Applications (https://www.ads-pipe.com/engineering-resources).

American Concrete Institute (ACI) 305R, Guide to Hot Weather Concreting (https://www.concrete.org/publications.aspx).

ACI 306R, Guide to Cold Weather Concreting (https://www.concrete.org/publications.aspx).

ACI 306.1, Standard Specification for Cold Weather Concreting (https://www.concrete.org/publications.aspx).

ACI 347.3R-13, Guide to Formed Concrete Surfaces (https://www.concrete.org/publications.aspx).

American Concrete Pipe Association (ACPA), 2007. Concrete Pipe Design Manual (https://www.concretepipe.org/wp-content/uploads/2014/09/cp-manual.pdf).

ACPA, undated. Installation and Inspection of Concrete Pipe: Joining/Joints.

American Concrete Pressure Pipe Association (ACPPA), 2018. Concrete Pressure Pipe: Repair Guide (https://acppa.org/wp-content/uploads/2020/04/ACPPA-6288_Repair-Manual-Updates-Web.pdf).

American Iron and Steel Institute (AISI), 2007. Welded Steel Pipe Design Manual: Merits, Design Standards, Technical Data and References (https://www.steeltank.com/Portals/0/pubs/Welded%20Steel%20Pipe%2010.10.07.pdf).

Anderson, D.G., Martin, G.R., Lam, I., and Wang, J.N., "Seismic Analysis and Design of Retaining Walls, Buried Structures, Slopes, and Embankments," National Cooperative Highway Research Program Report 611; Transportation Research Board; Washington, D.C., 2008 (http://www.ce.memphis.edu/7137/PDFs/Seismic%20Manual/nchrp_rpt_611.pdf).

American National Standards Institute (ANSI)/ American Water Works Association (AWWA) C110/A21.10, Ductile-Iron and Gray-Iron Fittings (https://www.awwa.org/publications/standards).

ANSI/AWWA C116/A21.16, Protective Fusion-Bonded Coatings (https://www.awwa.org/publications/standards).

ANSI/AWWA C151/A21.51, Ductile Iron Pipe, Centrifugally Cast (https://www.awwa.org/publications/standards).

ANSI/AWWA C153/A21.53, Ductile-iron Compact Fittings (https://www.awwa.org/publications/standards).

ANSI/AWWA C200, Steel Water Pipe, 6 In. (150 mm) and Larger (https://www.awwa.org/publications/standards).

American Public Works Association (APWA), 1999. Uniform Color Code (https://www.apwa.net/Library/Resources/Uniform-Color-Code.pdf).

Ariaratnam, S.T., and Beljan, I.J., 2005. Post-Construction Evaluation of HDD Installations. Practice Periodical on Structural Design and Construction, ASCE: Vol. 10, No. 2, pp. 115-126 (https://ascelibrary.org/doi/full/10.1061/%28ASCE%291084-0680%282005%2910%3A2%28115%29).

American Society of Civil Engineers (ASCE), 2008. ASCE Manuals and Reports on Engineering Practice No. 115, Pipe Ramming Projects. Edited by Najafi, M (https://ascelibrary.org/doi/book/10.1061/9780784409794).

ASCE, 2014. ASCE Manuals and Reports on Engineering Practice No. 108, Pipeline Design for Installation by Horizontal Directional Drilling, Second Edition. Edited by Skonberg, E. and Munidi, T. (https://ascelibrary.org/doi/book/10.1061/9780784413500).

ASCE, 2015. Standard ASCE/CI, Standard Design and Construction Guidelines for Microtunneling (https://ascelibrary.org/doi/book/10.1061/9780784413630).

ASCE, 2016. ASCE Manuals and Reports on Engineering Practice No. 132, Renewal of Potable Water Pipes. Edited by Najafi, M. and Perez, M. (https://ascelibrary.org/doi/book/10.1061/9780784413746).

ASCE MOP 79. ASCE Manuals and Reports on Engineering Practice No. 79, Steel Penstocks (https://ascelibrary.org/doi/book/10.1061/9780784412169).

American Society for Testing and Materials (ASTM) A53, Standard Specification for Pipe, Steel, Black and Hot-Dipped, Zinc-Coated, Welded and Seamless (https://www.astm.org/Standard/standards-and-publications.html).

ASTM A135, Standard Specification for Electric-Resistance-Welded Steel Pipe (https://www.astm.org/Standard/standards-and-publications.html).

ASTM A139, Standard Specification for Electric-Fusion (Arc)-Welded Steel Pipe (NPS 4 and Over) (https://www.astm.org/Standard/standards-and-publications.html).

ASTM A716, Standard Specification for Ductile Iron Culvert Pipe (https://www.astm.org/Standard/standards-and-publications.html).

ASTM A746, Standard Specification for Ductile Iron Gravity Sewer Pipe (https://www.astm.org/Standard/standards-and-publications.html).

ASTM A760, Standard Specification for Corrugated Steel Pipe, Metallic-Coated for Sewers and Drains (https://www.astm.org/Standard/standards-and-publications.html).

ASTM A796, Standard Specification for Structural Design of Corrugated Steel Pipe, Pipe-Arches, and Arches for Storm and Sanitary Sewers and Other Buried Applications (https://www.astm.org/Standard/standards-and-publications.html).

ASTM A849, Standard Specification for Post-Applied Coatings, Pavings, and Linings for Corrugated Steel Sewer and Drainage Pipe (https://www.astm.org/Standard/standards-and-publications.html).

ASTM A964, Standard Specification for Corrugated Steel Box Culverts (https://www.astm.org/Standard/standards-and-publications.html).

ASTM B744 - Standard Specification for Aluminum Alloy Sheet for Corrugated Aluminum Pipe (https://www.astm.org/Standard/standards-and-publications.html).

ASTM B745, Standard Specification for Corrugated Aluminum Pipe for Sewers and Drains (https://www.astm.org/Standard/standards-and-publications.html).

ASTM B788, Standard Practice for Installing Factory-Made Corrugated Aluminum Culverts and Storm Sewer Pipe (https://www.astm.org/Standard/standards-and-publications.html).

ASTM B790, Standard Specification for Structural Design of Corrugated Aluminum Pipe, Pipe-Arches, and Arches for Culverts, Storm Sewers, and Other Buried Conduits (https://www.astm.org/Standard/standards-and-publications.html).

ASTM B864, Standard Specification for Corrugated Aluminum Box Culverts (https://www.astm.org/Standard/standards-and-publications.html).

ASTM C12, Standard Practice for Installing Vitrified Clay Pipe Lines (https://www.astm.org/Standard/standards-and-publications.html).

ASTM C76, Standard Specification for Reinforced Concrete Culvert, Storm Drain, and Sewer Pipe (https://www.astm.org/Standard/standards-and-publications.html).

ASTM C138, A Standard Test Method for Density (Unit Weight), Yield, and Air Content (Gravimetric) of Concrete (https://www.astm.org/Standard/standards-and-publications.html).

ASTM C150, Standard Specification for Portland Cement (https://www.astm.org/Standard/standards-and-publications.html).

ASTM C361, Standard Specification for Reinforced Concrete Low-Head Pressure Pipe (https://www.astm.org/Standard/standards-and-publications.html).

ASTM C403, Standard Test Method for Time of Setting of Concrete Mixtures by Penetration Resistance (https://www.astm.org/Standard/standards-and-publications.html).

ASTM C425, Standard Specification for Compression Joints for Vitrified Clay Pipe and Fittings (https://www.astm.org/Standard/standards-and-publications.html).

ASTM C443, Standard Specification for Joints for Concrete Pipe and Manholes, Using Rubber Gaskets (https://www.astm.org/Standard/standards-and-publications.html).

ASTM C495, Standard Test Method for Compressive Strength of Lightweight Insulating Concrete (https://www.astm.org/Standard/standards-and-publications.html).

ASTM C506, Standard Specification for Reinforced Concrete Arch Culvert, Storm Drain, and Sewer Pipe (https://www.astm.org/Standard/standards-and-publications.html).

ASTM C507, Standard Specification for Reinforced Concrete Elliptical Culvert, Storm Drain, and Sewer Pipe (https://www.astm.org/Standard/standards-and-publications.html).

ASTM C655, Standard Specification for Reinforced Concrete D-Load Culvert, Storm Drain, and Sewer Pipe (https://www.astm.org/Standard/standards-and-publications.html).

ASTM C700, Standard Specification for Vitrified Clay Pipe, Extra Strength, Standard Strength, and Perforated (https://www.astm.org/Standard/standards-and-publications.html).

ASTM C877, Standard Specification for External Sealing Bands for Concrete Pipe, Manholes, and Precast Box Sections (https://www.astm.org/Standard/standards-and-publications.html).

ASTM C881, Standard Specification for Epoxy-Resin-Base Bonding Systems for Concrete (https://www.astm.org/Standard/standards-and-publications.html).

ASTM C923, Specification for Resilient Connectors Between Reinforced Concrete Manhole Structures, Pipes, and Laterals (https://www.astm.org/Standard/standards-and-publications.html).

ASTM C928, Standard Specification for Packaged, Dry, Rapid-Hardening Cementitious Materials for Concrete Repairs (https://www.astm.org/Standard/standards-and-publications.html).

ASTM C939, A Standard Test Method for Flow of Grout for Preplaced-Aggregate Concrete (Flow Cone Method) (https://www.astm.org/Standard/standards-and-publications.html).

ASTM C942, Standard Test Method for Compressive Strength of Grouts for Preplaced-Aggregate Concrete in the Laboratory (https://www.astm.org/Standard/standards-and-publications.html).

ASTM C990, Standard Specification for Joints for Concrete Pipe, Manholes and Precast Box Sections Using Preformed Flexible Joint Sealants (https://www.astm.org/Standard/standards-and-publications.html).

ASTM C1107, Standard Specification for Packaged Dry, Hydraulic-Cement Grout (Nonshrink) (https://www.astm.org/Standard/standards-and-publications.html).

ASTM C1478, Standard Specification for Storm Drain Resilient Connectors Between Reinforced Concrete Storm Sewer Structures, Pipes, and Laterals (https://www.astm.org/Standard/standards-and-publications.html).

ASTM C1619, Standard Specification for Elastomeric Seals for Joining Concrete Structures (https://www.astm.org/Standard/standards-and-publications.html).

ASTM C1628, Standard Specification for Joints for Concrete Gravity Flow Sewer Pipe, Using Rubber Gaskets (https://www.astm.org/Standard/standards-and-publications.html).

ASTM C1821, Standard Practice for Installation of Underground Circular Precast Concrete Manhole Structures (https://www.astm.org/Standard/standards-and-publications.html).

ASTM C1840, Standard Practice for Inspection and Acceptance of Installed Reinforced Concrete Culvert, Storm Drain, and Storm Sewer Pipe (https://www.astm.org/Standard/standards-and-publications.html).

ASTM D1056, Standard Specification for Flexible Cellular Materials – Sponge or Expanded Rubber (https://www.astm.org/Standard/standards-and-publications.html).

ASTM D1784, Standard Specification for Rigid Poly (Vinyl Chloride) (PVC) Compounds and Chlorinated Poly (Vinyl Chloride) (CPVC) Compounds (https://www.astm.org/Standard/standards-and-publications.html).

ASTM D2321, Standard Practice for Underground Installation of Thermoplastic Pipe for Sewers and Other Gravity-Flow Applications (https://www.astm.org/Standard/standards-and-publications.html).

ASTM D2657, Standard Practice for Heat Fusion Joining of Polyolefin Pipe and Fittings (https://www.astm.org/Standard/standards-and-publications.html).

ASTM D3034, Standard Specification for Type PSM Poly (Vinyl Chloride) (PVC) Sewer Pipe and Fittings (https://www.astm.org/Standard/standards-and-publications.html).

ASTM D3212, Standard Specification for Joints for Drain and Sewer Plastic Pipes Using Flexible Elastomeric Seals (https://www.astm.org/Standard/standards-and-publications.html).

ASTM D3261, Standard Specification for Butt Heat Fusion Polyethylene (PE) Plastic Fittings for Polyethylene (PE) Plastic Pipe and Tubing (https://www.astm.org/Standard/standards-and-publications.html).

ASTM D3262, Standard Specification for "Fiberglass" (Glass-Fiber-Reinforced Thermosetting-Resin) Sewer Pipe (https://www.astm.org/Standard/standards-and-publications.html).

ASTM D3350, Standard Specification for Polyethylene Plastics Pipe and Fittings Materials (https://www.astm.org/Standard/standards-and-publications.html).

ASTM D4161, Standard Specification for "Fiberglass" (Glass-Fiber-Reinforced Thermosetting-Resin) Pipe Joints Using Flexible Elastomeric Seals (https://www.astm.org/Standard/standards-and-publications.html).

ASTM D6103, Standard Test Method for Flow Consistency of Controlled Low Strength Material (https://www.astm.org/Standard/standards-and-publications.html).

ASTM D6938, Standard Test Methods for In-Place Density and Water Content of Soil and Soil-Aggregate by Nuclear Methods (Shallow Depth) (https://www.astm.org/Standard/standards-and-publications.html).

ASTM F477, Standard Specification for Elastomeric Seals (Gaskets) for Joining Plastic Pipe (https://www.astm.org/Standard/standards-and-publications.html).

ASTM F585, Standard Guide for Insertion of Flexible Polyethylene Pipe Into Existing Sewers (https://www.astm.org/Standard/standards-and-publications.html).

ASTM F679, Standard Specification for Poly(Vinyl Chloride) (PVC) Large-Diameter Plastic Gravity Sewer Pipe and Fittings (https://www.astm.org/Standard/standards-and-publications.html).

ASTM F714, Specification for Steel Polyethylene (PE) Plastic Pipe (DR-PR) Based on Outside Diameter (https://www.astm.org/Standard/standards-and-publications.html).

ASTM F794, Standard Specification for Poly(Vinyl Chloride) (PVC) Profile Gravity Sewer Pipe and Fittings Based on Controlled Inside Diameter (https://www.astm.org/Standard/standards-and-publications.html).

ASTM F949, Standard Specification for Poly(Vinyl Chloride) (PVC) Corrugated Sewer Pipe With a Smooth Interior and Fittings (https://www.astm.org/Standard/standards-and-publications.html).

ASTM F1697 – Standard Specification for Poly(Vinyl Chloride) (PVC) Profile Strip for Machine Spiral-Wound Liner Pipe Rehabilitation of Existing Sewers and Conduits (https://www.astm.org/Standard/standards-and-publications.html).

ASTM F1698, Standard Practice for Installation of Poly (Vinyl Chloride) (PVC) Profile Strip Liner and Cementitious Grout for Rehabilitation of Existing Man-Entry Sewers and Conduits (https://www.astm.org/Standard/standards-and-publications.html).

ASTM F1735, Standard Specification for Poly (Vinyl Chloride) (PVC) Profile Strip for PVC Liners for Rehabilitation of Existing Man-Entry Sewers and Conduits (https://www.astm.org/Standard/standards-and-publications.html).

ASTM F1741, Standard Practice for Installation of Machine Spiral Wound Poly (Vinyl Chloride) (PVC) Liner Pipe for Rehabilitation of Existing Sewers and Conduits (https://www.astm.org/Standard/standards-and-publications.html).

ASTM F1743, Standard Practice for Rehabilitation of Existing Pipelines and Conduits by Pulled-in-Place Installation of Cured-in-Place Thermosetting Resin Pipe (CIPP) (https://www.astm.org/Standard/standards-and-publications.html).

ASTM F1871, Standard Specification for Folded/Formed Poly (Vinyl Chloride) Pipe Type A for Existing Sewer and Conduit Rehabilitation (https://www.astm.org/Standard/standards-and-publications.html).

ASTM F1867, Standard Practice for Installation of Folded/Formed Poly (Vinyl Chloride) (PVC) Pipe Type A for Existing Sewer and Conduit Rehabilitation (https://www.astm.org/Standard/standards-and-publications.html).

ASTM F2016, Standard Practice for Rehabilitation of Existing Pipelines and Conduits by the Inversion and Curing of a Resin-Impregnated Tube (https://www.astm.org/Standard/standards-and-publications.html).

ASTM F2019, Standard Practice for Rehabilitation of Existing Pipelines and Conduits by the Pulled in Place Installation of Glass Reinforced Plastic (GRP) Cured-in-Place Thermosetting Resin Pipe (CIPP) (https://www.astm.org/Standard/standards-and-publications.html).

ASTM F2510, Standard Specification for Resilient Connectors Between Reinforced Concrete Manhole Structures and Corrugated Dual- and Triple-Wall Polyethylene and Polypropylene Pipes (https://www.astm.org/Standard/standards-and-publications.html).

ASTM F2562, Specification for Steel Reinforced Thermoplastic Ribbed Pipe and Fittings for Non-Pressure Drainage and Sewerage (https://www.astm.org/Standard/standards-and-publications.html).

ASTM F3182 - Standard Practice for the Application of Spray-Applied Polymeric Liners Inside Pipelines for Potable Water (https://www.astm.org/Standard/standards-and-publications.html).

ASTM F2620, Standard Practice for Heat Fusion Joining of Polyethylene Pipe and Fittings (https://www.astm.org/Standard/standards-and-publications.html).

AWWA C104, Cement Mortar Lining for Ductile Iron Pipe and Fittings (https://www.awwa.org/publications/standards).

AWWA C105/A21.5, Polyethylene Encasement for Ductile-Iron Pipe Systems (https://www.awwa.org/publications/standards).

AWWA C111, Rubber-Gasket Joints for Ductile-Iron Pressure Pipe and Fittings (https://www.awwa.org/publications/standards).

AWWA C205, Cement-Mortar Protective Lining and Coating for Steel Water Pipe 4 In. (100 mm) and Larger – Shop Applied (https://www.awwa.org/publications/standards).

AWWA C209, Cold-Applied Tape Coatings for Steel Water Pipe, Special Sections, Connections, and Fittings (https://www.awwa.org/publications/standards).

AWWA C210, Liquid-Epoxy Coatings and Linings for Steel Water Pipe and Fittings (https://www.awwa.org/publications/standards).

AWWA C214, Tape Coatings for Steel Water Pipe (https://www.awwa.org/publications/standards).

AWWA C222, Polyurethane Coatings and Linings for Steel Water Pipe and Fittings (https://www.awwa.org/publications/standards).

AWWA C300, Reinforced Concrete Pressure Pipe, Steel-Cylinder Type (https://www.awwa.org/publications/standards).

AWWA C301, Pre-stressed Concrete Pressure Pipe, Steel-Cylinder Type (https://www.awwa.org/publications/standards).

AWWA C302, Reinforced Concrete Pressure Pipe, Non-cylinder Type (https://www.awwa.org/publications/standards).

AWWA C303, "Concrete Pressure Pipe, Bar-Wrapped, Steel-Cylinder Type (https://www.awwa.org/publications/standards).

AWWA C304, "Design of Pre-stressed Concrete Cylinder Pipe (https://www.awwa.org/publications/standards).

AWWA, 2008. AWWA M9, Concrete Pressure Pipe, Third Edition (https://www.awwa.org/publications/standards).

AWWA, 2017. AWWA M11, Steel Pipe – A Guide for Design and Installation, Fifth Edition (https://www.awwa.org/publications/standards).

AWWA, 2013. AWWA M27, External Corrosion Control for Infrastructure Sustainability, Third Edition (https://www.awwa.org/publications/standards).

AWWA M41, Ductile Iron Pipe and Fittings (https://www.awwa.org/publications/standards).

AWWA, 2013. AWWA M45, Fiberglass Pipe Design, Third Edition (https://www.awwa.org/publications/standards).

American Welding Society (AWS) D1.2, Structural Welding Code – Aluminum (https://www.aws.org/standards).

Baumert, M.E., Allouche, E.N., and Moore, I.D., 2005. Drilling Fluid Considerations in Design of Engineered Horizontal Directional Drilling Installations. ASCE International Journal of Geomechanics, December, 2005 (https://ascelibrary.org/doi/10.1061/%28ASCE%291532-3641%282005%295%3A4%28339%29).

Bennett, R.D., Guice, L.K., Khan, S., and Staheli, K., 1995. CPAPR-GL-95-2, Guidelines for Trenchless Technology: Cured-in-Place Pipe (CIPP), Fold-and-Formed Pipe (FFP), Mini-Horizontal Directional Drilling (Mini-HDD), and Microtunneling. U.S. Army Engineer Waterways Experiment Station (WES) and Trenchless Technology Center (TCC) at Louisiana Tech University (https://hdl.handle.net/11681/4704).

Bennett and Wallin, 2008. Step-By-Step Evaluation of Hydrofracture Risks for HDD Projects. Proceedings of the 2008 No-Dig Conference. Dallas, Texas, April 27-May 2, 2008 (https://member.nastt.org/products/).

California Department of Transportation, 2014. Highway Design Manual (https://dot.ca.gov/programs/design/manual-highway-design-manual-hdm).

Construction Marketing Committee, American Iron and Steel Institute, 1996. Steel Plate Engineering Data - Volume 3, Welded Steel Pipe (https://www.steeltank.com/Portals/0/pubs/SPED_VL3.pdf).

Elwood, D.E.Y., Moore, I., and Xia, H., 2007. Hydraulic Fracture Experiments in a Frictional Material and Approximations for Maximum Allowable Mud Pressure, GeoEngineering Centre at Queens – RMC, Queens University, Kingston, Ontario, Canada (https://citeseerx.ist.psu.edu/viewdoc/download?doi=10.1.1.534.3181&rep=rep1&type=pdf).

Foster, M., Fell, R., and Spannagle, M., 2000. The statistics of embankment dam failures and accidents. Can. Geotech. J. 37: 1000–1024 (2000) (https://cdnsciencepub.com/doi/10.1139/t00-030).

Goerz, et. al., 2018. A New Perspective in Hydro Fracture Analysis. 2018 TAC/NASST-NW Tunnelling & Trenchless Conference (https://nastt.org/).

Clay Griffin, President, Rembco Geotechnical Contractors, Inc., P.O. Box 23009, Knoxville, TN 37933.

HOBAS Pipe-USA, 2016. FRP Pipe for Direct Bury Installation - Gravity Service (http://www.hobaspipe.com/fileadmin/user_upload/Direct_Bury_March_2019.pdf).

HOBAS Pipe-USA, 2016. The Complete Hobas Guide (http://www.hobaspipe.com/fileadmin/user_upload/HOBAS_Guide_2016.pdf).

Howard, A., 2015. Pipe Installation 2.0, 2nd Edition. Relativity Publishing (https://amsterhoward.com/book).

Hunt, S.W., 2017. Tunneling in Cobbles and Boulders. Breakthroughs in Tunneling Short Course, Chicago, IL, August 14-16, 2017 (https://tunnelingshortcourse.com/2017-presentations/hunt-tunneling-in-cobbles-and-boulders.pdf).

Karamanos, S., Keil, B., and Card, R.J., 2014. "Seismic Design of Buried Steel Water Pipelines," American Society of Civil Engineers Pipelines 2014 Conference Proceedings, Portland, OR (https://ascelibrary.org/doi/10.1061/9780784413692.091).

Luger, H.J., and Hergarden, A.M., 1988. Directional Drilling in Soft Soil: Influence of Mud Pressures. International Society of Trenchless Technology, No-Dig Conference.

North American Society for Trenchless Technology (NASTT), 2017. NASTT's Horizontal Directional Drilling (HDD) Good Practices Guidelines, 4th Edition (https://member.nastt.org/products/).

NASTT, 2018. NASTT's Introduction to Trenchless Technology, New Installation Methods Good Practices Guidelines, 1st Edition (https://member.nastt.org/products/).

National Clay Pipe Institute (NCPI), 2017. Vitrified Clay Pipe Engineering Manual (https://www.ncpi.org/assets/ncpi-engineering-manual.pdf).

NCPI, undated. Tips for Installing Vitrified Clay Pipe (https://www.ncpi.org/assets/tips-brochure-2016.pdf).

National Corrugated Steel Pipe Association (NCSPA), 2008. Corrugated Steel Pipe Design Manual, First Edition (https://ncspa.org/wp-content/uploads/2016/07/2ndedncspsacspdm.pdf).

NCSPA, 2013. Installation Manual for Corrugated Steel Pipe and Structural Plate (https://r7x5k9u8.stackpathcdn.com/wp-content/uploads/2016/07/NCSPAInstallationManualFINAL01022014.pdf).

NCSPA, 2017. Corrugated Steel Pipe Design Manual (https://ncspa.org/ncspa-csp-design-manual-survey/).

Pipeline Research Council International, Inc., 1995. Installation of Pipelines by Horizontal Directional Drilling, an Engineering Design Guide (https://www.prci.org/Research/DesignMaterialsConstruction/DMCProjects/CNST-1-3/3169/18295.aspx).

Plastics Pipe Institute (PPI), 2012. Handbook of Polyethylene Pipe, Second Edition (https://plasticpipe.org/publications/pe-handbook.html).

PPI, 2018. MAB-4-2018, Basic HDPE Repair Options (https://plasticpipe.org/pdf/mab-basic-hdpe-repair-options-mab4.pdf).

Plastics Industry Pipe Association (PIPA) of Australia Limited, 2009. Industry Guidelines, Water Jet Cleaning of Plastics Pipes, Issue 1.2 (https://www.iplex.com.au/assets/Uploads/ec9c641256/POP205.pdf).

PVC Pipe Association, 2013. Handbook of PVC Pipe Design and Construction. Industrial Press, Inc., New York (https://www.uni-bell.org/Resources/Handbooks).

Robison, P., and Sparks, A., 2015. Direct Pipe Levee Crossing Design – Mitigating Hydraulic Fracture Risk. NASTT 2015 No-Dig Show. Denver, Colorado, March 15-19, 2015 (https://member.nastt.org/products/).

Rostami, A., 2017. Prediction and Evaluation of Annular Pressure in Horizontal Directional Drilling. Ph.D. Thesis, Department of Civil and Environmental Engineering, University of Alberta, Canada (https://era.library.ualberta.ca/items/b1238128-d5d5-426a-91ef-9444ae2407b1).

Schmertmann, J.H., 1985. Measure and Use of the Insitu Lateral Stress. The Practice of Foundation Engineering – a Volume honoring Jorj O. Osterberg. The Department of Civil Engineering, Northwestern University.

Staheli, K., Bennett, R.D., O'Donnell, H., and Hurley, T., 1998. CPAR-GL-98-1, Installation of Pipelines Beneath Levees Using Horizontal Directional Drilling (HDD). U.S. Army Engineer Waterways Experiment Station (WES) and O'Donnell Associates, Inc (https://hdl.handle.net/11681/4691).

Staheli K., 2010. Effectiveness of Hydrofracture Prediction for HDD Design. NASTT No-Dig Show 2010 (https://member.nastt.org/products/).

Stuedlein, A.W., and Meskele, T., 2013. Analysis and Design of Pipe Ramming Installations. Final Report SPR 710. School of Civil and Construction Engineering, Oregon State University. Prepared for Oregon Department of Transportation (https://www.oregon.gov/ODOT/Programs/ResearchDocuments/SPR710_PipeRamming.pdf).

Thompson, J., 1993. Pipejacking and Microtunnelling, 1st Edition. CRC Press (https://www.routledge.com/).

Thorley, A. R., and Atkinson, J.H., 1994. Guide to the design of thrust blocks for buried pressure pipelines. Construction Industry Research and Information Association (CIRIA), Report 128, London (https://kupdf.net/download/ciria-128pdf_5969b56adc0d60cc46a88e82_pdf\).

Vesic, A.S., 1972. Expansion of cavities in infinite soil mass. ASCE Journal of the Soil Mechanics and Foundations Division, Vol. 98: 265-290 (https://cedb.asce.org/CEDBsearch/record.jsp?dockey=0127466).

Warner, J., 2004. Practical Handbook of Grouting, Soil, Rock, and Structures. Wiley (https://www.wiley.com/).

Xia, H., 2009. Investigation of Maximum Mud Pressure within Sand and Clay during Horizontal Directional Drilling. Ph.D. Thesis, Queen's University, Kingston, Ontario, Canada, January 2009 (https://qspace.library.queensu.ca/bitstream/handle/1974/1659/Xia_Hongwei_200901_PhD.pdf?sequence=1).

Yan, X., Sayem, S., Allouche, E., and Alam, S., 2015. Experimental Examination of the Mathematical Model for Predicting the Borehole Pressure during Horizontal Directional Drilling. ASCE Pipelines 2015 Conference (https://ascelibrary.org/doi/10.1061/9780784479360.011).

Zipperer, Michael P., VP Specialty Products, Chief Engineer, Mechanical Research & Design, Inc., 7411 Center Road, Manitowoc, WI 54220.

Related References

Corps of Engineers Publications

EM 1110-2-2102, Waterstops and Other Preformed Joint Materials for Civil Works Structures (https://www.publications.usace.army.mil/portals/76/publications/engineermanuals/em_1110-2-2102.pdf).

Other Federal Agency Publications

FHWA, 2000. Durability Analysis of Aluminized Type 2 Corrugated Metal Pipe (https://www.fhwa.dot.gov/publications/research/infrastructure/hydraulics/97140/index.cfm).

U.S. Environmental Protection Agency, 1999. EPA 832- F-99-032, Collection Systems O&M Fact Sheet – Trenchless Sewer Rehabilitation (https://www3.epa.gov/npdes/pubs/rehabl.pdf).

U.S. Forest Service, 2005. Summary of Trenchless Technology for Use with USDA Forest Service Culverts. Forest Service Technology & Development Program (https://www.fs.fed.us/eng/pubs/pdf/05771201.pdf).

Other Publications

AASHTO M36, Standard Specification for Corrugated Steel Pipe, Metallic-Coated, for Sewers and Drains (https://www.transportation.org/).

AASHTO M196, Standard Specification for Corrugated Aluminum Pipe for Sewers and Drains (https://www.transportation.org/).

AASHTO T341, Standard Method of Test for Determination of Compression Capacity for Profile Wall Plastic Pipe by Stub Compression Loading (https://www.transportation.org/).

ACI 318, Building Code Requirements for Structural Concrete (https://www.concrete.org/publications.aspx).

American Lifelines Alliance, 2005. Guidelines for the Design of Buried Steel Pipe (https://www.americanlifelinesalliance.com/pdf/Update061305.pdf).

American Society of Civil Engineers, 1984. Guidelines for the Seismic Design of Oil and Gas Pipeline Systems (https://cedb.asce.org/CEDBsearch/record.jsp?dockey=0042895).

ASTM A674, Standard Specification for Polyethylene Encasement for Ductile Iron Pipe (https://www.astm.org/Standard/standards-and-publications.html).

ASTM B744, Standard Specification for Aluminum Alloy Sheet for Corrugated Aluminum Pipe (https://www.astm.org/Standard/standards-and-publications.html).

ASTM B746, Standard Specification for Corrugated Aluminum Alloy Structural Plate for Field-Bolted Pipe, Pipe-Arches, and Arches (https://www.astm.org/Standard/standards-and-publications.html).

ASTM B788, Standard Specification for Installing Factory-Made Corrugated Aluminum Culverts and Storm Sewer Pipe (https://www.astm.org/Standard/standards-and-publications.html).

ASTM B789, Standard Specification for Installing Corrugated Aluminum Structural Plate Pipe for Culverts and Sewers (https://www.astm.org/Standard/standards-and-publications.html).

ASTM C12, Standard Practice for Installing Vitrified Clay Pipe Lines (https://www.astm.org/Standard/standards-and-publications.html).

ASTM C30, Test Methods for Vitrified Clay Pipe (https://www.astm.org/Standard/standards-and-publications.html).

ASTM C857, Standard Practice for Minimum Structural Design Loading for Underground Precast Concrete Utility Structures (https://www.astm.org/Standard/standards-and-publications.html).

ASTM D2321, Standard Practice for Underground Installation of Thermoplastic Pipe for Sewers and Other Gravity-Flow Applications (https://www.astm.org/Standard/standards-and-publications.html).

ASTM D2683, Standard Specification for Socket-Type Polyethylene Fittings for Outside Diameter-Controlled Polyethylene Pipe and Tubing (https://www.astm.org/Standard/standards-and-publications.html).

ASTM D2737, Standard Specification for Polyethylene (PE) Plastic Tubing (https://www.astm.org/Standard/standards-and-publications.html).

ASTM D2774, Standard Practice for Underground Installation of Thermoplastic Pressure Piping (https://www.astm.org/Standard/standards-and-publications.html).

ASTM D2997, Standard Specification for Centrifugally Cast Fiberglass (Glass-Fiber Reinforced Thermosetting-Resin) Pipe (https://www.astm.org/Standard/standards-and-publications.html).

ASTM D3261, Standard Specification for Butt Heat Fusion Polyethylene (PE) Plastic Fittings for Polyethylene (PE) Plastic Pipe and Tubing (https://www.astm.org/Standard/standards-and-publications.html).

ASTM D3262, Standard Specification for Fiberglass (Glass Fiber Reinforced Thermosetting Resin) Sewer Pipe (https://www.astm.org/Standard/standards-and-publications.html).

ASTM D3350, Standard Specification for Polyethylene Plastics Pipe and Fittings Materials (https://www.astm.org/Standard/standards-and-publications.html).

ASTM D3517, Standard Specification for Fiberglass (Glass-Fiber Reinforced Thermosetting-Resin) Pressure Pipe (https://www.astm.org/Standard/standards-and-publications.html).

ASTM D4161, Standard Specification for Fiberglass (Glass-Fiber Reinforced Thermosetting-Resin) Pipe Joints Using Flexible Elastomeric Seals (https://www.astm.org/Standard/standards-and-publications.html).

ASTM F585, Standard Guide for Insertion of Flexible Polyethylene Pipe into Existing Sewers (https://www.astm.org/Standard/standards-and-publications.html).

ASTM F667, Standard Specification for 3 through 24 in Corrugated Polyethylene Pipe and Fittings (https://www.astm.org/Standard/standards-and-publications.html).

ASTM F905, Standard Practice for Qualification of Polyethylene Saddle-Fused Joints (https://www.astm.org/Standard/standards-and-publications.html).

ASTM F2164, Standard Practice for Field Leak Testing of Polyethylene (PE) Pressure Piping Systems Using Hydrostatic Pressure (https://www.astm.org/Standard/standards-and-publications.html).

ASTM F2306, Standard Specification for 12 to 60 inch [300 to 1500 mm] Annular Corrugated Profile Wall Polyethylene (PE) Pipe and Fittings for Gravity-Flow Storm Sewer and Subsurface Drainage Applications (https://www.astm.org/Standard/standards-and-publications.html).

ASTM F2764, Standard Specification for 6 to 60 inch [150 to 1500 mm] Polypropylene (PP) Corrugated Double and Triple Wall Pipe and Fittings for Non-Pressure Sanitary Sewer Applications (https://www.astm.org/Standard/standards-and-publications.html).

ASTM F2881, Standard Specification for 12 to 60 inch [300 to 1500 mm] Polypropylene (PP) Dual Wall Pipe and Fittings for Non-Pressure Storm Sewer Applications (https://www.astm.org/Standard/standards-and-publications.html).

ASTEM F3183, Standard Practice for Guided Side Bend Evaluation of Polyethylene Pipe Butt Fusion Joint, 2016 (https://www.astm.org/Standard/standards-and-publications.html).

ASTM F3190, Standard Practice for Heat Fusion Equipment (HFE) Operator Qualification on Polyethylene (PE) and Polyamide (PA) Pipe and Fittings (https://www.astm.org/Standard/standards-and-publications.html).

ASTM M252, Standard Specification for Corrugated Polyethylene Drainage Pipe (https://www.astm.org/Standard/standards-and-publications.html).

ASTM M294, Standard Specification for Corrugated Polyethylene Pipe, 300 to 1500 mm (12 to 60 in) Diameter (https://www.astm.org/Standard/standards-and-publications.html).

ASTM M330, Standard Specification for Polyethylene Pipe, 300 to 1500 mm (12 to 60 in) Diameter (https://www.astm.org/Standard/standards-and-publications.html).

AWWA C105, Polyethylene Encasement for Ductile-Iron Pipe Systems (https://www.awwa.org/publications/standards).

AWWA C150, Thickness Design of Ductile-Iron Pipe (https://www.awwa.org/publications/standards).

AWWA C950, Fiberglass Pressure Pipe (https://www.awwa.org/publications/standards).

Bednar, L., 1989. Transportation Research Board 1231, Plain Galvanized Steel Drainage Pipe Durability Estimation with a Modified California Chart, Washington, DC (https://onlinepubs.trb.org/Onlinepubs/trr/1989/1231/1231-007.pdf).

Bonds, R., 2017. DIPRA Technical Publication, Ductile Iron Pipe Joints and Their Uses (https://www.dipra.org/phocadownload/new/Features-JointsandUses.pdf).

Boschert, J. and Howard, A., 2014. "Importance of Haunching," ASCE Pipelines 2014 Conference Proceedings, Portland, OR (https://ascelibrary.org/doi/10.1061/9780784413692.036).

Ductile Iron Pipe Research Association (DIPRA), 2016. DIPRA Technical Publication, Design of Ductile Iron Pipe (https://www.dipra.org/ductile-iron-pipe-resources/technical-publications).

DIPRA, 2018. DIPRA Technical Publication, the Design Decision Model for Corrosion Control of Ductile Iron Pipelines (https://www.dipra.org/ductile-iron-pipe-resources/technical-publications).

Harries, K.A., and Sweriduk, M., 2013. Hydrostatic Pressure Tests of Spray-Applied Epoxy Lining System. Report to Warren Environmental. Watkins Haggart Structural Engineering Laboratory, University of Pittsburgh (https://www.warrenenviro.com/wp-content/uploads/2020/07/Multi-ASTM-Hydrostatic-Pressure-Testing_Kent-A.-Harrie.pdf).

Noll, J., and Westrich, M., undated. Rehabilitation and Relining of Culverts. Contech Construction Products Inc. (https://www.conteches.com/knowledge-center/pdh-article-series/rehabilitation-and-relining-of-culverts).

PPI, 2012. PPI Technical Report 33, Generic Butt Fusion Joining Procedure for Polyethylene Gas Pipe (https://plasticpipe.org/pdf/tr-33_generic_butt_fusion_joining_gas_pipe.pdf).

PPI, 2009. Polyethylene Piping Systems: Field Manual for Municipal Water Applications (https://plasticpipe.org/pdf/mid-pe-field-manual-municipal-water-applications.pdf).

Image Credits

Figure 1-4. “solutions_b_clip_image016.jpg.” America’s Wetland Foundation Resource Center, 2012, http://www.americaswetlandresources.com/background_facts/detailedstory/current.html

Figure 2-54. “damaged-culvert-cleveland.jpg.” Trenchless Technology Magazine, 2019, https://trenchlesstechnology.com/downsized-culvert-solves-urgent-problem-increases-flow-cleveland/

Figure 2-57. “eja280514_img01.jpg.” SPEC-NET Building Index, 2017, https://www.spec-net.com.au/press/0514/eja_280514.htm

Figure 2-58. “Wingwall Grille.jpg.” HYNDS Pipe Systems, 2019, https://www.hynds.co.nz/product/hynds-wingwall-grilles/

Figure 3-13 (left). “Agru-Polyflo-Dual-Containment.265-300x169.” Fusion Australia, 2019, https://fusionaus.com/manufacturers/agru/

Figure 3-13 (right). Cadwallader, L. and Pinna, T. “Features of double containment piping.” *Reliability Estimation for Double Containment Piping,* 20th ANS Topical Meeting on the Technology of Fusion Energy. Idaho National Laboratory, 2012, https://inldigitallibrary.inl.gov/sites/sti/sti/5558900.pdf

Figure 3-14. Hancor regular solid single wall pipe 10 ft, HDPE. T.H. Rogers Lumber Company, 2019. http://www.throgers.com/PIPE_SOLID_DRAIN_CORR_3X10FT_p/8310849.htm

Figure 3-16. Cross-section profile for SRTP. Paladeri SPA, 2016, https://www.paladeri.it/

Figure 5-41. “pro150709_img01.jpg.” SPEC-NET Building Index, 2017, https://www.spec-net.com.au/press/0709/pro_150709/Projex-Group-UGA-external-wall-sleeve-to-seal-pipe-penetrations

Figure 5-50. Acceptable sling methods for lifting pipes. Connect NCDOT Drainage User Manual, 2020, https://connect.ncdot.gov/resources/Materials/Pages/Drainage-User-Manual-Text.aspx?Method=DM-05-A

Figure 5-51. Lifting holes are unacceptable for pipes associated with dams and levees. Connect NCDOT Drainage User Manual, 2020, https://connect.ncdot.gov/resources/Materials/Pages/Drainage-User-Manual-Text.aspx?Method=DM-05-A

Figure 5-52. "gauge_aluminumSM.png." Sewer Equipment Company of Florida, 2020, http://www.sewerequipmentco.com/deflection.html

Figure 7-1. "drain-unblocking-water-jetting.png." Drains CO, UK, 2019, https://www.drainsco.co.uk/drain-jetting/

Figure 7-2. "tree-roots-300x219." Euro Plumbing & Sewer, LLC, 2017, http://europlumbingtoday.com/2017/05/19/crystal-lake-plumber/

Figure 7-3. General Pipe Cleaners ClogChopper®. General Pipe Cleaners (a division of General Wire Spring Co.), 2019, https://drainbrain.com/en/products/clogchopper/

Figure 7-10. Types of Snap-Tite Hydro-Bells. Snap-Tite/ISCO Industries, 2019, https://culvert-rehab.com/products/hydrobell/

Figure 7-18. "What are Spiral Wound Liners? Trenchless Sewer Pipe Lining" [digital image from video]. Sekisui SPR Americas, LLC, 2018, https://sekisuispra.com/pipe-rehab-technology-spiral-wound-liners/trenchless-pipe-lining/

Figure 7-20. Non-round mobile/transverse spiral wound liner installation. Sekisui SPR Americas, LLC, 2019, http://sekisuispra.com/pipe-rehab-technology-spiral-wound-liners/trenchless-pipe-lining/spr/

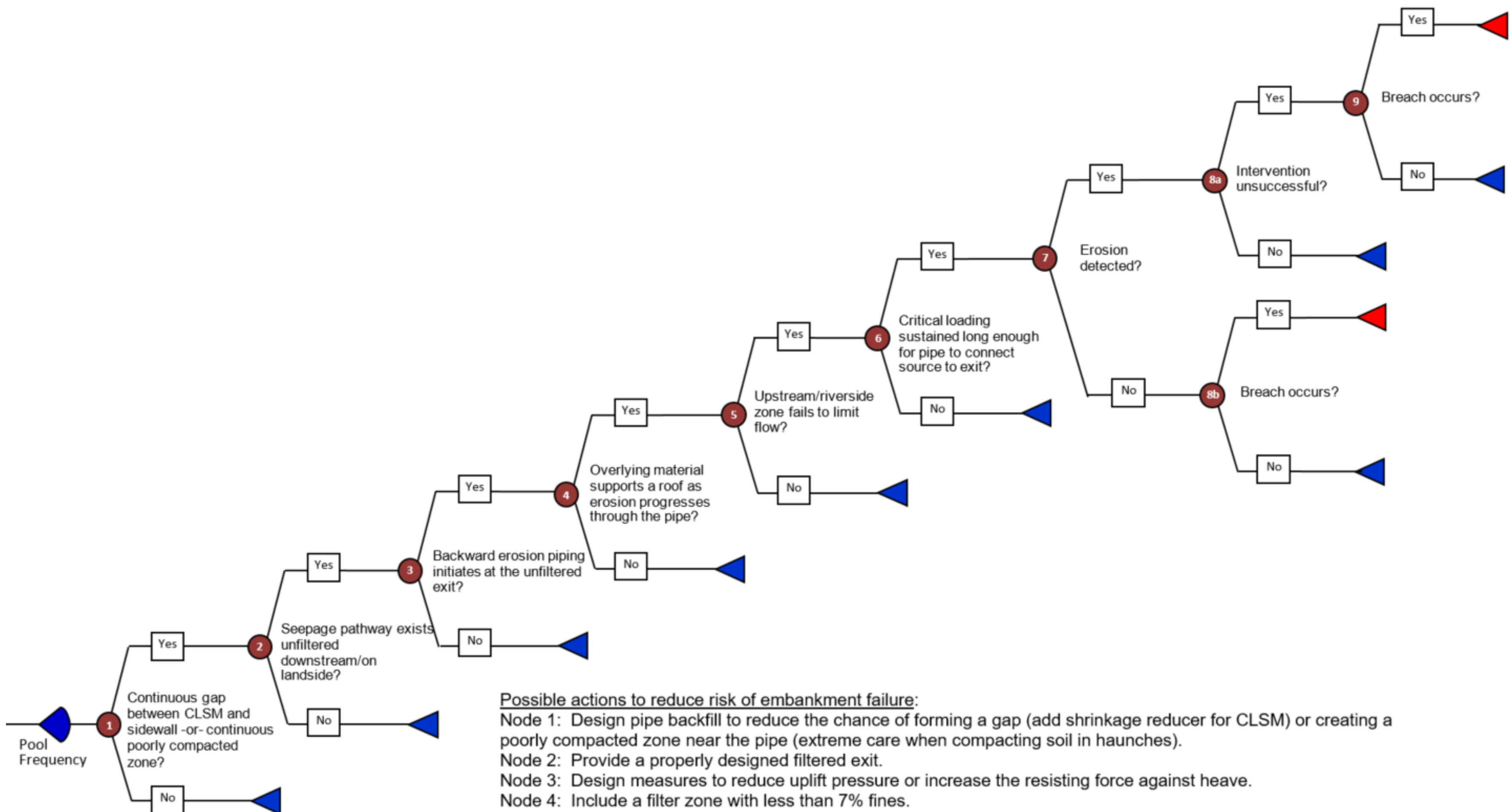
Figure 7-27. "The art of the spray-on membrane." TunnelTalk, 2008, https://www.tunneltalk.com/Spray-on-waterproofing-Finding-real-application.php

Figure 7-29. "The Inversion Process by JTV Inc." JTV Inc., 2017, http://jtv-cipp.com/stormdrain-cipp-lining-florida.html

Figure 7-30. Fold-and-form liners. WaterWorld Magazine, 2012, https://www.waterworld.com/drinking-water/distribution/article/16193026/trenchless-water-main-rehabilitation-design-considerations

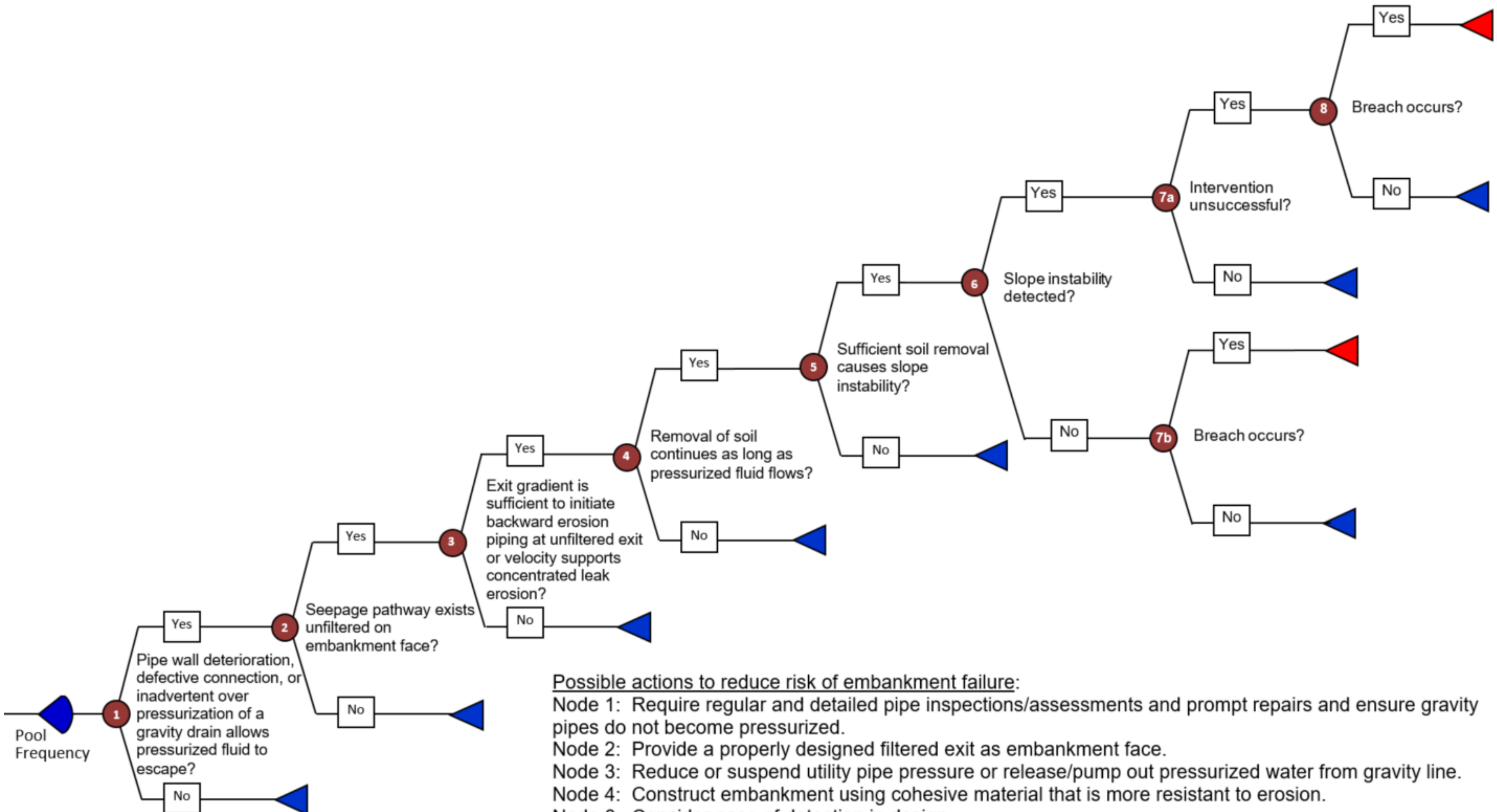
Figure 9-15. "SS-250_Fab-300x300.jpg." watermanusa.com, Waterman, 2018, https://watermanusa.com/products/awwa-slide-gates/ss-250-slide-gates/

Appendix B
Representative Event Trees



<u>Possible actions to reduce risk of embankment failure</u>:
Node 1: Design pipe backfill to reduce the chance of forming a gap (add shrinkage reducer for CLSM) or creating a poorly compacted zone near the pipe (extreme care when compacting soil in haunches).
Node 2: Provide a properly designed filtered exit.
Node 3: Design measures to reduce uplift pressure or increase the resisting force against heave.
Node 4: Include a filter zone with less than 7% fines.
Node 5: Provide a granular upstream flow limiter.
Node 6: Lower pool – raise pipe to reduce chance of reaching critical loading.
Node 7: Consider ease of detection in design.
Node 8a: Consider ease of intervention in design.
Node 8b/9: Encourage responsible party to maintain an emergency action plan.

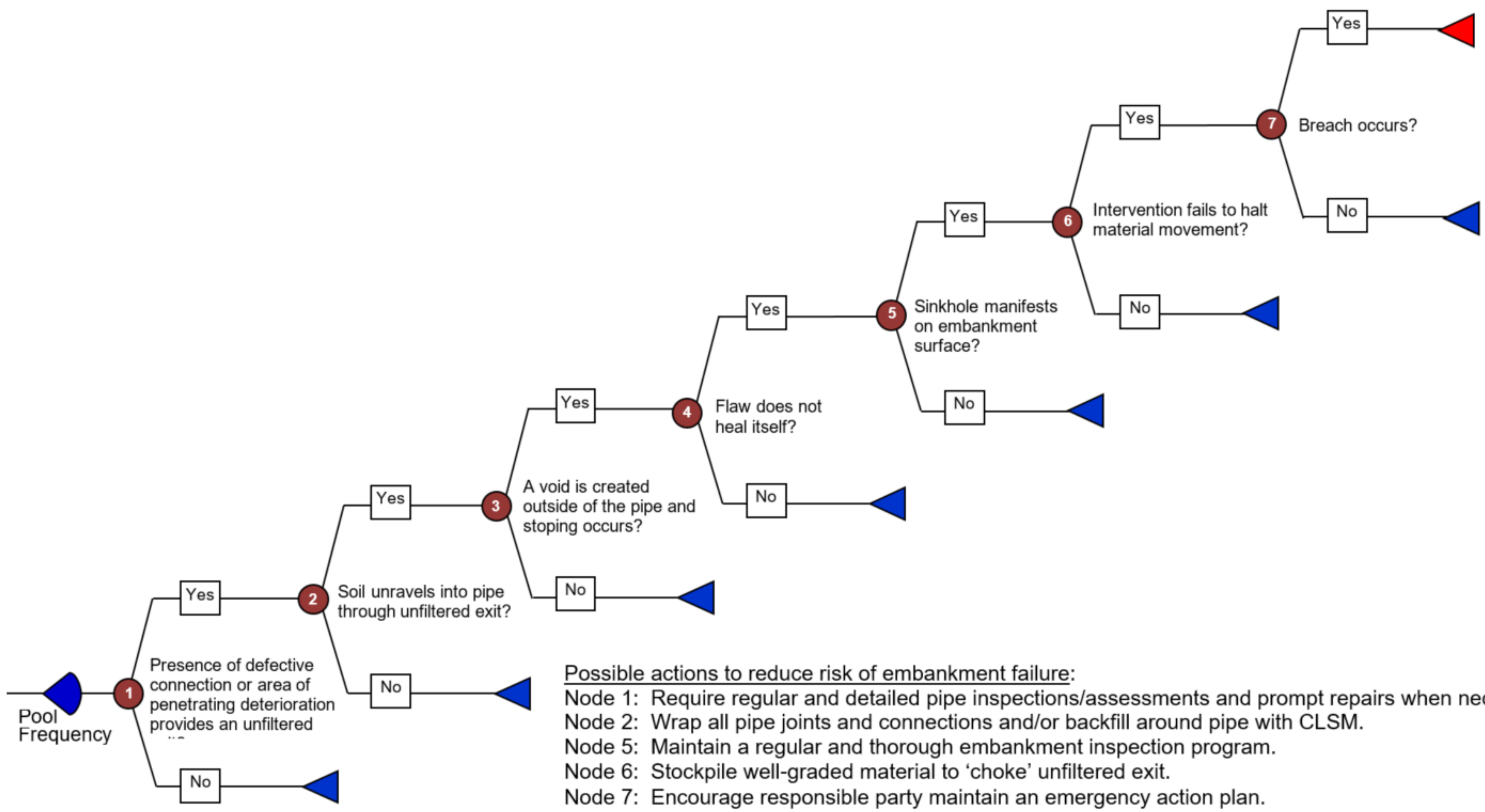
Representative event tree for PFM-1 (Internal Erosion along a Pipe Leads to Breach)



<u>Possible actions to reduce risk of embankment failure</u>:

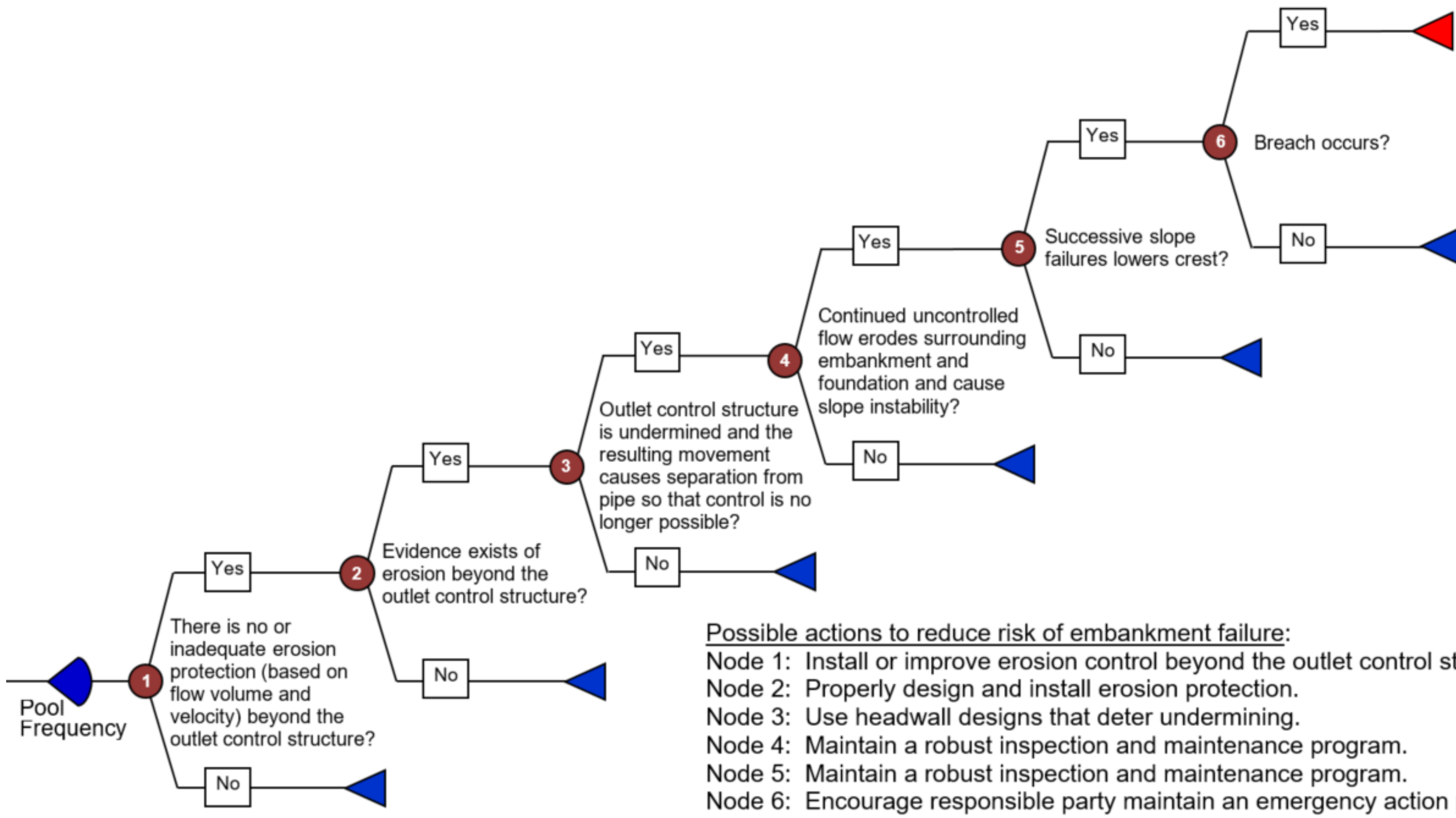
Node 1: Require regular and detailed pipe inspections/assessments and prompt repairs and ensure gravity pipes do not become pressurized.
Node 2: Provide a properly designed filtered exit as embankment face.
Node 3: Reduce or suspend utility pipe pressure or release/pump out pressurized water from gravity line.
Node 4: Construct embankment using cohesive material that is more resistant to erosion.
Node 6: Consider ease of detection in design.
Node 7a: Consider ease of intervention in design.
Node 7b/8: Encourage responsible party to maintain an emergency action plan.

Representative event tree for PFM-2 (Internal Erosion from Leakage of a Pressurized Pipe Leads to Breach)



Possible actions to reduce risk of embankment failure:
Node 1: Require regular and detailed pipe inspections/assessments and prompt repairs when necessary.
Node 2: Wrap all pipe joints and connections and/or backfill around pipe with CLSM.
Node 5: Maintain a regular and thorough embankment inspection program.
Node 6: Stockpile well-graded material to ‘choke’ unfiltered exit.
Node 7: Encourage responsible party maintain an emergency action plan.

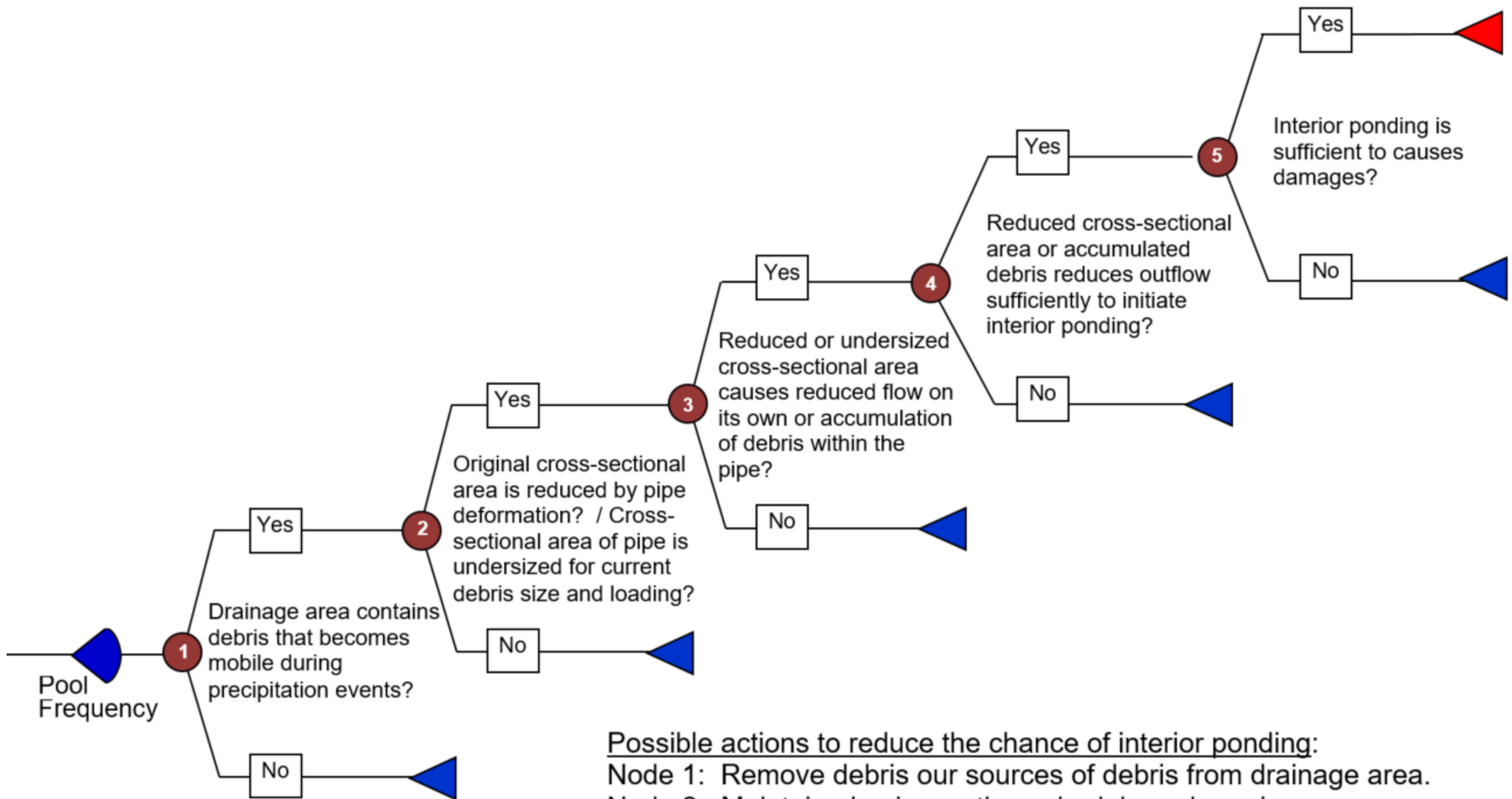
Representative event tree for PFM-3 (Internal Erosion into a Pipe Leads to Breach)



<u>Possible actions to reduce risk of embankment failure</u>:
Node 1: Install or improve erosion control beyond the outlet control structure.
Node 2: Properly design and install erosion protection.
Node 3: Use headwall designs that deter undermining.
Node 4: Maintain a robust inspection and maintenance program.
Node 5: Maintain a robust inspection and maintenance program.
Node 6: Encourage responsible party maintain an emergency action plan.

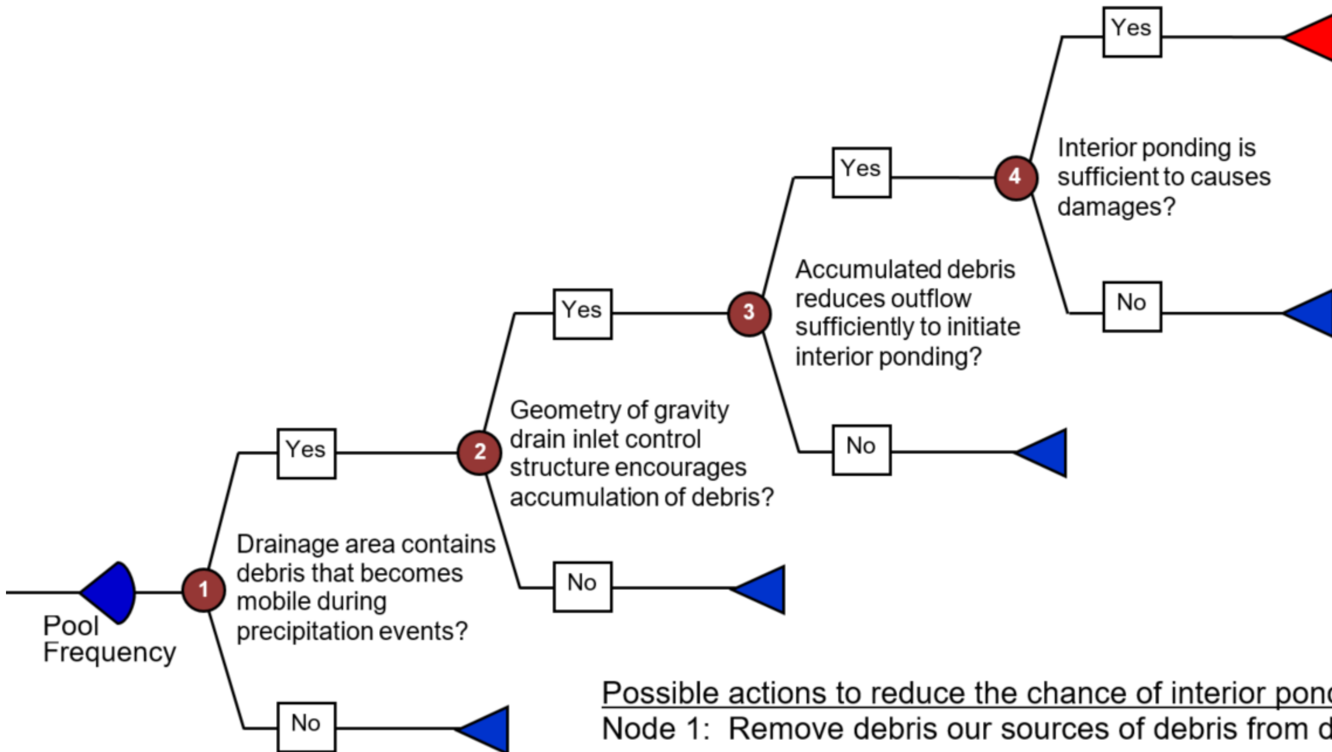
<u>Note</u>: *The secondary result of the loss of control as described in Node 3 is that interior flooding is possible with even minor flood events if intervention is not performed or is unsuccessful.*

Representative event tree for PFM-4 (External Erosion at a Pipe Outlet)



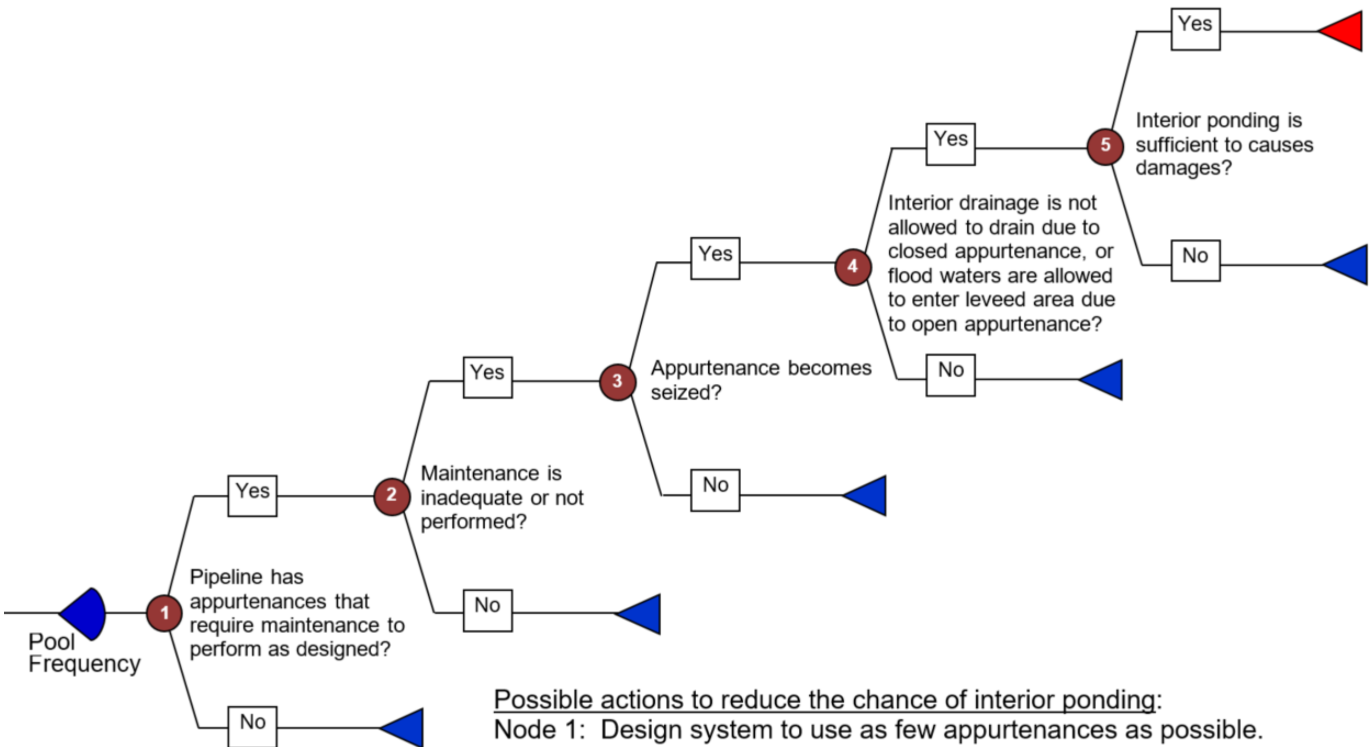
<u>Possible actions to reduce the chance of interior ponding</u>:
Node 1: Remove debris our sources of debris from drainage area.
Node 2: Maintain pipe inspection schedule and repair as necessary.
Node 3: Install a debris capture system to prevent debris from entering pipe.
Node 4: Prepare a separate debris removal procedure (if feasible).
Node 5: Include the removal of interior water as part of an emergency action plan.

Representative event tree for PFM-5 (Internal Blockage Causes Interior Ponding)



Possible actions to reduce the chance of interior ponding:
Node 1: Remove debris our sources of debris from drainage area.
Node 2: Design inlet control structure to reduce likelihood of accumulating debris.
Node 3: Prepare a separate debris removal procedure.
Node 4: Include the removal of interior water as part of an emergency action plan.

Representative event tree for PFM-6 (External Blockage Causes Interior Ponding)



Possible actions to reduce the chance of interior ponding:
Node 1: Design system to use as few appurtenances as possible.
Node 2: Maintain pipe and appurtenance inspection and maintenance schedule and repair as necessary.
Node 3: Fully operate each appurtenance on a strict schedule and address as necessary.
Node 4: Include the removal of interior water or riverside pipe sealing as part of emergency action plan.
Node 5: Include the removal of interior water as part of an emergency action plan.

Representative event tree for PFM-7 (Appurtenance Malfunction Causes Interior Ponding)

Appendix C
Pipe Association and Federal Agency Contact Information

Pipe Association Contacts
RCP and CIPP:
American Concrete Pipe Association (ACPA)
info@concretepipe.org
972-506-7216
https://www.concretepipe.org/

Trenchless:
North American Society for Trenchless Technology (NASTT)
info@nastt.org
888-993-9935
https://www.nastt.org/

Midwest Society for Trenchless Technology (MSTT)
888-817-3788
http://www.mstt.org/

CSP/CMP:
National Corrugated Steel Pipe Association (NCSPA)
info@ncspa.org
972-850-1907
https://ncspa.org/

PVC:
PVC Pipe Association (PVCPA)
info@uni-bell.org
972-243-3902
https://www.uni-bell.org/

HDPE:
Plastics Pipe Institute (PPI)
469-499-1044
https://plasticpipe.org/

Polypropylene:
Advanced Drainage Systems, Inc.
800-821-6710
https://www.ads-pipe.com/

FRP:
Fiberglass Tank & Pipe Institute
info@fiberglasstankandpipe.com
918-809-6292
http://www.fiberglasstankandpipe.com/

DIP:
Ductile Iron Pipe Research Association (DIPRA)
info@dipra.org
205-532-4267
https://www.dipra.org/

Steel Pipe:
Steel Plate Fabricators Association (SPFA)
info@steeltank.com
847-438-8265
https://www.steeltank.com/

American Spiral Weld Pipe
205-325-7701
866-442-2797
https://american-usa.com/

National Association of Steel Pipe Distributors (NASPD)
361-574-7878
https://naspd.com/

Allied Corrosion Industries
info@alliedcorrosion.com
770-425-1355
https://www.alliedcorrosion.com/

VCP:
National Clay Pipe Institute (NCPI)
262-742-2904
https://www.ncpi.org/

CPP:
American Concrete Pressure Pipe Association (ACPPA)
support@acppa.org
714-801-0298
https://acppa.org/

Precast:
National Precast Concrete Association (NPCA)
317-571-9500
https://precast.org/

CI:
Cast Iron Soil Pipe Institute (CISPI)
224-864-2910
https://www.cispi.org/

Aluminum:
The Aluminum Association
info@aluminum.org
703-358-2960
https://www.aluminum.org/

Federal Agency Contacts

Tennessee Valley Authority
Dam Safety
tvainfo@tva.com
865-632-2101
https://www.tva.gov/

Federal Energy Regulatory Commission
Division of Dam Safety and Inspections
202-502-8700
https://www.ferc.gov/

THIS PAGE INTENTIONALLY LEFT BLANK

Appendix D
Design Examples

D1. Non-Pressure/Low-Pressure Precast RCP (Indirect Design Method) Design Example

Given: A 36-inch diameter precast circular RCP is to be installed with 5 feet of cover and embedded in CLSM. The soil unit weight over and around the CLSM is 120 lbs/ft^3. It is a type B-wall pipe, which based on the ASTM C76 Standard has a 4-inch wall thickness.

Find: The required pipe strength associated with the 0.01-inch D-load.

Design Example Notes:

1. For a CLSM installation, the bedding factor and vertical arching factor for the installation are assumed the same as a Type 1 installation. For Type 1installation, the installation factor is set equal to 1.1 for the D-load equation.

Solution:

D1 Step 1. DETERMINE THE EARTH LOAD BEDDING FACTOR

The dead load bedding factor (B_{FE}) is defined based on the installation type and the pipe size. For a Type 1 installation and 36-inch pipe size, B_{FE} is equal to 4.0 according to AASHTO Table 12.10.4.3.2a-1.

D1 Step 2. DETERMINE THE EARTH LOAD (W_E) BASED ON EQUATION 4-3

$$W_E = \mathrm{VAF} \cdot \gamma \cdot B_c \cdot H \qquad \text{Equation 4-3}$$

The vertical arching factor (VAF) depends on the type of installation used. For a Type 1 installation, VAF is equal to 1.35. The unit weight of the soil fill (γ) is given as 120 lbs/ft^3. The out-to-out horizontal dimension of the pipe (B_c) is computed as follows:

$$B_c = (D_i + 2\cdot\text{pipe wall thickness})/12 = (36 + 2\cdot4.0)/12 = 3.67 \text{ feet}$$

The pipe is buried under 5 feet of soil fill (H = 5.0 feet).

$$W_E = 1.35 \cdot 120 \cdot 3.67 \cdot 5.0 = 2{,}973 \text{ lb/ft}$$

D1 Step 3. DETERMINE THE INTERNAL FLUID LOAD W_F

The internal fluid load is equal to the internal area of the pipe multiplied by the unit weight of the fluid, γ_f =62.4 lb/ft^3 is used for water.

$$W_F = A_f \cdot \gamma_f$$

The area of the pipe section is equal to:

$$A_f = \pi \cdot \frac{D_i^2}{4} = \pi \cdot \frac{36^2}{4} = 1{,}017.9 \text{ in}^2 = 7.07 \text{ ft}^2$$

$$W_F = 7.07 \cdot 62.4 = 441 \frac{\text{lb}}{\text{ft}}$$

D1 Step 4. DETERMINE THE LIVE LOAD BEDDING FACTOR, B_{FLL}

Since live loads are present the live load bedding factor B_{FLL} is defined from Table 12.10.4.3.2c-1 of AASHTO, based on the pipe diameter and the fill height. For a 36-inch pipe diameter and 5 feet of fill height, B_{FLL} is equal to 2.2.

D1 Step 5. DETERMINE THE LIVE LOAD

The AASHTO LRFD Bridge Design Specification (Section 3.6) must be followed for calculating the live load on the pipe. Based on AASHTO Section 3.6, an HL93 design includes loading from either a design truck or design tandem. In this example, both calculations are developed and the most critical is used.

D1 Step 5.1 CALCULATE THE TRUCK LOAD

Based on AASHTO Section 3.6.1.2.2, the loads applied per axle are 8 kips and 32 kips for the front and the two rear axles, respectively. The distance between the front and the middle axle is 14 feet while the distance between the middle and the last axle varies from 14 to 30 feet. The transverse spacing of wheels is 6 feet.

The tire contact area for truck and tandem wheels is calculated based on AASHTO 3.6.1.2.5, and is assumed to be a single rectangle, whose width is 20 inches and whose length is 10 inches.

For traffic parallel to the culvert span, the first step is to check if the load applied from the tires, at the level of the pipe, are overlapping.

D1 Step 5.1.1 - Check for overlapping in the transverse to the pipe span direction

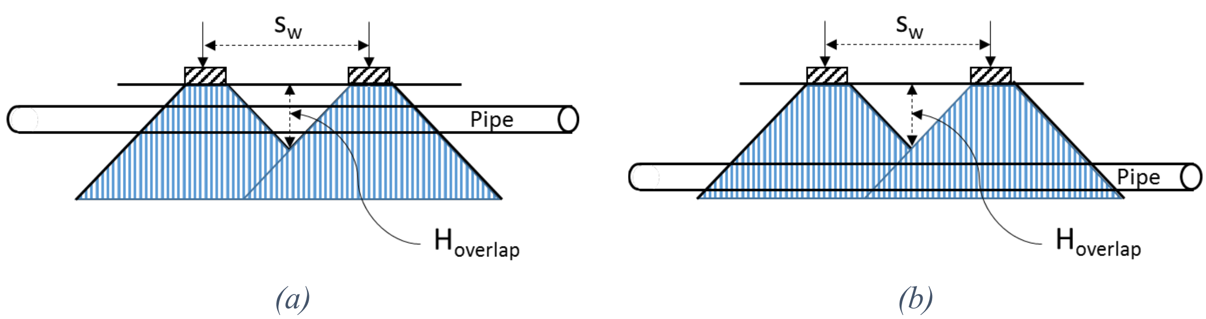


Figure D-1. Load Distribution for the Direction Transverse to the Pipe Span.

Wheel loads from a single axle are applied at a spacing of 6 feet center-to-center, and assumed to spread through the soil at a ratio of LLDF to 1, where LLDF is the live load distribution factor defined in Table 3.6.1.2.6a-1 of AASHTO. For a 36-inch diameter concrete pipe, the LLDF is:

$$\mathrm{LLDF} = 1.15 + \left(\frac{36-24}{96-24}\right)\cdot(1.75-1.15)$$

$$\mathrm{LLDF} = 1.25$$

Upon reaching the pipe, AASHTO assumes a further distribution through the pipe of $0.06 \cdot D_i$. Thus, the depth where the wheels from a single axle would overlap would be equal to $H_{overlap}$ as shown in Figure D-1.

$$H_{overlap} = \frac{s_w - \frac{w_t}{12} - 0.06\cdot\frac{D_i}{12}}{\mathrm{LLDF}}$$

$$H_{overlap} = \frac{6 - \frac{20}{12} - 0.06\cdot\frac{36}{12}}{1.25} = \frac{4.153}{1.25} = 3.32\ \text{feet}$$

Since the $H_{overlap} < 5$ feet, it means that the wheel loads are overlapping at the pipe level. The live load patch width at height H is therefore equal to:

$$w_w = s_w + \frac{w_t}{12} + H\cdot\mathrm{LLDF} + 0.06\cdot\frac{D_i}{12} = 6 + \frac{20}{12} + 5\cdot1.25 + 0.06\cdot\frac{36}{12} = 14.1\ \text{feet}$$

D1 Step 5.1.2 - Check for overlapping in the direction parallel to the pipe span direction

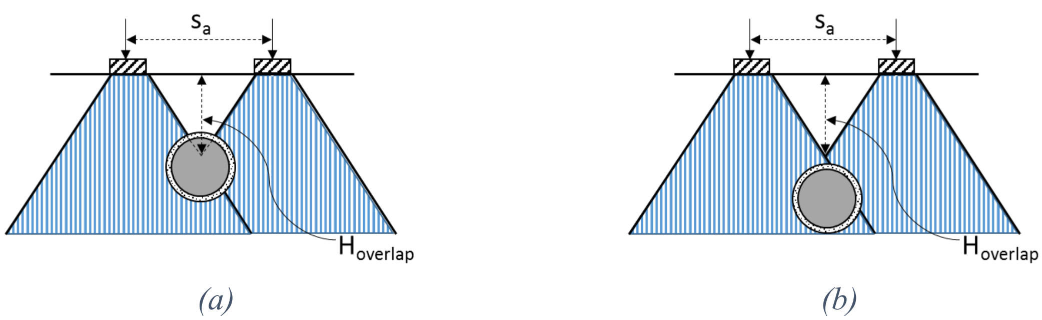


Figure D-2. Load Distribution for the Direction Parallel to the Pipe Span.

In the direction parallel to the pipe span, the distance between the two heavy axles is 14 feet, and there is no load distribution through the pipe in this direction. Thus, the depth where the axle loads interact is:

$$H_{overlap} = \frac{s_a - \frac{l_t}{12}}{LLDF} = \frac{14 - \frac{10}{12}}{1.25} = 10.53 \text{ feet}$$

Since the $H_{overlap}$> 5 feet, there is only one single axle load over the pipe at a time, as shown in Figure D-2(a). The live load patch width at height H is therefore equal to:

$$l_w = \frac{l_t}{12} + H \cdot LLDF = \frac{10}{12} + 5 \cdot 1.25 = 7.1 \text{ feet}$$

Therefore, the area for the applied live load at the level of the pipe is equal to:

$$A_{LL} = w_w \cdot l_w = 14.1 \cdot 7.1 = 99.85 \text{ ft}^2$$

D1 Step 5.1.3 – Calculate applied load pressure at the pipe

To define the vertical live load applied at the top of the pipe from the HL-93 truck, the equation presented in Section 3.6.1.2.6b of the AASHTO is used.

$$P_L = \frac{P \cdot \left(1 + \frac{IM}{100}\right) \cdot m}{A_{LL}}$$

The impact factor (IM) for buried components is calculated based on AASHTO Article 3.6.2.2.

$$IM = 33 \cdot (1.0 - 0.125 \cdot D_E) \geq 0\%$$

$$IM = 33 \cdot (1.0 - 0.125 \cdot 5) = 12.375$$

The multiple presence factor (m) based on AASHTO Article 3.6.1.1.2 is defined from Table 3.6.1.1.2-1, and it is equal to 1.20 for a single loaded lane, which is the governing load for buried pipe when traffic is parallel to the span. Since there is overlapping of the loading from both wheels, but not from the axles, the overall load (P) applied from the truck at the ground level is equal to the single axle load of 32 kips.

$$P_L = \frac{P \cdot \left(1 + \frac{IM}{100}\right) \cdot m}{A_{LL}} = \frac{32{,}000 \cdot \left(1 + \frac{12.375}{100}\right) \cdot 1.20}{99.85} = 432.2 \text{ lb/ft}^2$$

D1 Step 5.1.4 – Calculate the live load applied on the pipe due to a truck load

To determine the live load applied on the pipe from a truck, multiply the pressure by the minimum of the external dimeter of the pipe or l_w. For this example:

$$W_{Ls} = P_L \cdot B_c = 432.2 \text{ lbs/ft}^2 \cdot 3.67 \text{ ft} = 1{,}585 \text{ lbs/ft}$$

D1 Step 5.2 CALCULATE THE TANDEM LOAD

Based on AASHTO Section 3.6.1.2.3, the loads applied for tandem design are a pair of 25-kip axles spaced 4 feet apart. The transverse spacing of the wheels is the same as for the truck design and equals 6 feet. The tire contact area for truck and tandem wheels are based on AASHTO 3.6.1.2.5, and is assumed to be a single rectangle, whose width is 20 inches and whose length is 10 inches.

D1 Step 5.2.1 - Check for overlapping in the direction transverse to the pipe

For traffic parallel to the culvert span, the first step is to check if the load applied from the tires, at the level of the pipe, are overlapping. Since the distance s_w for the tandem axles is equal to the single truck axle, this step is the same as Section 5.1.1 of this example.

D1 Step 5.2.2 - Check for overlapping in the direction parallel to the pipe

The live load distribution factor, LLDF, is defined with linear interpolation, based on Table 3.6.1.2.6a-1, and is the same as it is for the single axle. (LLDF = 1.25).

In the direction parallel to the pipe span, the distance between the tandem axles is 4 feet. Thus, the depth where the axle loads interact is:

$$H_{overlap} = \frac{s_a - \frac{l_t}{12}}{LLDF} = \frac{4 - \frac{10}{12}}{1.25} = 2.53 \text{ feet}$$

Since the $H_{overlap} < 5$ feet, it means there is overlapping from both axles at the pipe level. The live load patch width at height H is therefore equal to:

$$l_w = s_a + \frac{l_t}{12} + H \cdot LLDF = 4 + \frac{10}{12} + 5 \cdot 1.25 = 11.08 \text{ feet}$$

Therefore, the area for the applied live load at the level of the pipe is equal to

$$A_{LL} = w_w \cdot l_w = 14.1 \cdot 11.08 = 156.23 \text{ ft}^2$$

D1 Step 5.2.3 – Calculate the pressure applied on the pipe

To define the vertical live load applied at the level of the pipe from the tandem axles, the equation presented in Section 3.6.1.2.6b of AASHTO is used.

$$P_L = \frac{P \cdot \left(1 + \frac{IM}{100}\right) \cdot m}{A_{LL}}$$

The impact factor (IM) for buried components is calculated using AASHTO Section 3.6.2.2, and is the same for the single axle and tandem axles.

$$IM = 33 \cdot (1.0 - 0.125 \cdot D_E) \geq 0\%$$

$$IM = 33 \cdot (1.0 - 0.125 \cdot 5) = 12.375$$

The multiple presence factor (m) based on AASHTO Article 3.6.1.1.2 is defined from Table 3.6.1.1.2-1, and it is equal to 1.2 for a single loaded lane, which is the governing load for buried pipe when traffic is parallel to the span. Since there is overlapping of the loading from both axles and wheels the overall load (P) applied from the truck at the ground level is equal to 50 kips.

$$P_L = \frac{P \cdot \left(1+\frac{IM}{100}\right) \cdot m}{A_{LL}} = \frac{50{,}000 \cdot \left(1+\frac{12.375}{100}\right) \cdot 1.20}{156.23} = 431.6\ \mathrm{lb/ft^2}$$

D1 Step 5.2.4 – Calculate the live load applied on the pipe from a tandem load

To determine the live load applied on the pipe due from a tandem load, multiply the pressure by the minimum of the external diameter of the pipe (l_w).

$$W_{Lt} = P_L \cdot B_c = 431.6\ \mathrm{lbs/ft^2} \cdot 3.67\ \mathrm{ft} = 1{,}584\ \mathrm{lbs/ft}$$

D1 Step 5.2.5 - Select the more critical live load

The more critical live load is defined as the maximum value when comparing W_{Ls} and W_{Lt} loads. Thus, W_L = 1,585 lbs/ft for this example.

D1 Step 6. CALCULATE THE $D_{0.01}$ LOAD USING EQUATION 4-2

$$D_{0.01} = \left(\frac{12}{D_i}\right) \cdot \left(\frac{W_e + W_f}{B_{fe}} + \frac{W_L}{B_{fLL}}\right) \cdot I_f = \left(\frac{12}{36}\right) \cdot \left(\frac{2973+441}{4.0} + \frac{1585}{2.2}\right) \cdot 1.1 = 577\ \mathrm{lb/ft/ft}$$

D2. Non-Pressure/Low-Pressure Precast RCP (Direct Design Method) Design Example

Given: A 36-inch diameter circular concrete pipe with a pressure head of 35 feet is to be installed in a positive projecting embankment condition in clay soils with 5 feet of cover. The clay soil has a unit weight of 120 lbs/ft^3. The wall thickness of the pipe is 4 inches per ASTM C361 and the manufacturer. This is considered a normal operating condition.

Find: The required inside and outside reinforcement for the pipe.

Properties:

t = 4.0 pipe wall thickness (inches)

f'_c = 5,000 concrete compressive strength (psi)

f_y = 60,000 steel yield strength (psi)

Solution:

Per ASTM C361, the pipe must be evaluated for three load conditions:

- Load Condition 1: Internal Pressure Only

- Load Condition 2: Earth Load, Pipe Weight, and Fluid Weight with No Internal Pressure (This is the same load condition used or gravity pipe.)
- Load Condition 3: External and Internal Loads Acting Concurrently

D2 STEP 1. LOAD CONDITION 1 – INTERNAL PRESSURE ONLY

D2 Step 1.1 FIND THE MINIMAL STEEL AREA REQUIRED FOR INTERNAL PRESSURE ONLY PER ASTM C361 X2.4

$$A1_s = \frac{6 \cdot (0.433 \cdot H_w) \cdot D_i}{f_s}$$

Where: $A1_s$ = total area of steel required for internal pressure (in^2/ft)

H_w = hydrostatic head (ft)

D_i = internal diameter (in)

f_s = allowable tensile stress in the reinforcement (lbs/in^2)

$f_s = 17{,}000 - 35 \cdot H_w = 17{,}000 - (35 \cdot 35) = 15{,}775$ psi

$$A1_s = \frac{6 \cdot [(0.433) \cdot 35] \cdot 36}{15{,}775}$$

$A1_s = 0.208\ in^2/ft$

D2 Step 1.2 VERIFY THE TENSILE STRESS IN THE CONCRETE IS NOT EXCEEDED PER X2.4.7.1 OF ASTM C361

$$f_{ct} = \frac{0.433 \cdot H_w \cdot D_i}{2 \cdot t_w}$$

Where: t_w = pipe wall thickness (in)

f_{ct} = tensile stress in the concrete

Max $f_{ct} = 4.5\sqrt{f'_c}$

$f_{ct} = \frac{0.433(35)(36)}{2(4.0)} = 68.2$ psi

Max $f_{ct} = 4.5\sqrt{5000} = 318$ psi

68.2 psi < 318 psi Verified (O.K.)

$A1_s = 0.208\ in^2/ft$ Minimum steel area (inside and outside reinforcement) required for Load Condition 1 (Internal Pressure Only)

D2 STEP 2. LOAD CONDITION 2: EARTH LOAD, PIPE WEIGHT, AND FLUID WEIGHT WITH NO INTERNAL PRESSURE

D2 Step 2.1 DETERMINE THE LOADS ON THE PIPE

The loads will be calculated in the same fashion as for Example D1, with the exception that the self-weight of the pipe must also be calculated since its load must be accounted for when it is not tested in three-edge bearing.

D2 Step 2.1.1 - Determine the earth load based upon Equation 4-3

$$W_E = VAF \cdot \gamma \cdot B_c \cdot H$$

The vertical arching factor (VAF) depends on the type of installation used. Since the pipe is being installed in clay, assume a Type 4 installation. Per this manual, the VAF for a Type 4 installation is 1.45. The unit weight of the soil cover fill is given and it is equal to 120 lbs/ft^3.

B_c is the maximum outside horizontal dimension of the pipe in feet:

$$B_c = (D_i + 2 \cdot \text{pipe wall thickness})/12 = (36 + 2 \cdot 4.0)/12 = 3.67 \text{ feet}$$

The pipe is buried under 5 feet of fill (H = 5 ft).

$$W_E = VAF \cdot \gamma \cdot B_c \cdot H = 1.45 \cdot 120 \cdot 3.67 \cdot 5 = 3{,}190 \frac{lb}{ft}$$

D2 Step 2.1.2 - Determine the internal fluid load W_F

The internal fluid load is equal to the internal area of the pipe multiplied by the unit weight of the fluid, γ_f = 62.4 lb/ft^3 for water.

$$W_F = A_f \cdot \gamma_f$$

The area of the pipe section is equal to:

$$A_f = \pi \cdot \frac{D_i^2}{4} = \pi \cdot \frac{36^2}{4} = 1{,}017.9\ in^2 = 7.07\ ft^2$$

$$W_F = 7.07 \cdot 62.43 = 441 \frac{lb}{ft}$$

D2 Step 2.1.3 – Determine the pipe weight

Find the mean radius (r_m) and circumference (C_m) at the mid-point of the pipe wall:

$r_m = \frac{D_i}{2} + \frac{t}{2}$ $\qquad$ $r_m = \frac{36}{2} + \frac{4.0}{2}$ $\qquad$ $r_m = 20.0$ inches

$C_m = 2\pi \cdot r_m$ $\qquad$ $C_m = 2\pi \cdot (20.0)$ $\qquad$ $C_m = 125.66$ inches

Find the volume of concrete (Vol_c) per foot of pipe:

$Vol_c = \frac{C_m \cdot t \cdot 12}{1728}$ $\qquad$ $Vol_c = \frac{125.66 \cdot 4.0 \cdot 12}{1728}$ $\qquad$ $Vol_c = 3.49\ ft^3$

Find the weight of the pipe (W_p):

$W_p = Vol_c \times \gamma_c$ $\qquad$ $W_p = 3.49 \times 150$ $\qquad$ $W_p = 523.60$ lbs/ft

Where:

W_p = weight of one foot length of pipe (lbs/ft)

γ_c = unit weight of concrete (150 lbs/ft^3)

D2 Step 2.1.4 - Determine the live load on the pipe W_L

Since this is the same size pipe, with the same height of cover as in example D1, rather than repeat the long calculation process for live load, use the value obtained in Example D1.

$$W_L = 1{,}585 \text{ lbs/ft}$$

In order to calculate the moments, shears, and thrusts in the pipe, the length of the live load pressure in the direction of the span of pipe must be determined. This can also be pulled directly from Example D1 (Step 5.1.2):

$$l_w = 7.1 \text{ feet}$$

D2 Step 2.2 CALCULATE THE STEEL REQUIREMENT FOR THE INSIDE STEEL CAGE FOR LOAD CONDITION 2

D2 Step 2.2.1 - Find the moments and thrusts at the invert of the pipe

Direct design for concrete pipe is usually performed using commercially available pipe design software; however, it can also be done manually by using design coefficients to calculate pipe wall loads (moment, shear, and thrust). These coefficients can be found in ASCE 15-98, which also includes additional information beyond what is found in ASTM C361 and AASHTO LRFD Bridge Design Specifications. For ease of calculation, the design coefficients for moment and thrust at the pipe invert have been repeated in Table D-1.

Table D-1. Design Coefficients for Moment and Thrust for Circular RCP at Pipe Invert

m – moment n – thrust		Installation Types							
		Type 1		Type 2		Type 3		Type 4	
Applicable Location	**Load Type**	C_{mi}	C_{ni}	C_{mi}	C_{ni}	C_{mi}	C_{ni}	C_{mi}	C_{ni}
Invert	W_p	0.225	0.077	0.227	0.077	0.230	0.077	0.235	0.077
	W_E	0.091	0.188	0.122	0.169	0.150	0.163	0.191	0.128
	W_F	0.088	-0.445	0.111	-0.437	0.133	-0.425	0.160	-0.403
	$*W_{L1}$	0.075	0.250	0.107	0.205	0.136	0.199	0.185	0.152
	$**W_{L2}$	0.165	-0.046	0.189	-0.035	0.211	-0.023	0.237	-0.004
Springline	W_p	-0.091	0.249	-0.091	0.249	-0.097	0.271	-0.101	0.287
	W_E	-0.077	0.500	-0.090	0.500	-0.103	0.500	-0.127	0.504
	W_F	-0.064	-0.068	-0.070	-0.068	-0.081	-0.063	-0.095	-0.057
	$*W_{L1}$	-0.065	0.500	-0.078	0.513	-0.126	0.497	-0.121	0.495
	$**W_{L2}$	-0.154	0.500	-0.160	0.500	-0.155	0.496	-0.168	0.492

* L1 designation is for a deeply buried pipe with a uniformly distributed live load
* L2 designation is for a pipe with a relatively narrow load distribution over the crown of the pipe

The moment, thrust, and shear values are calculated as follows:

$$M_i = C_{mi} \cdot W_i \cdot r_m$$
$$N_i = C_{ni} \cdot W_i$$
$$V_i = C_{vi} \cdot W_i$$

From Table D-1, the moment and thrust coefficients for a Type 4 installation are:

$C_{mie} = 0.191$ $C_{nie} = 0.128$ coefficients for earth load moment and earth load thrust
$C_{mip} = 0.235$ $C_{nip} = 0.077$ coefficients for pipe load moment and pipe load thrust
$C_{mif} = 0.160$ $C_{nif} = -0.403$ coefficients for fluid load moment and fluid load thrust

Table D-1 has two rows for the live load coefficients. The values of W_{L1} are for when the live load has spread to a length of twice the diameter of the pipe and is acting uniformly above it. The values of W_{L2} are for when the live load is acting as a concentrated load at the top of the pipe with a spread length no longer than 10% of the outside diameter of the pipe, as shown in Figure D-3. For spread lengths in between, the live load coefficients may be linearly interpolated.

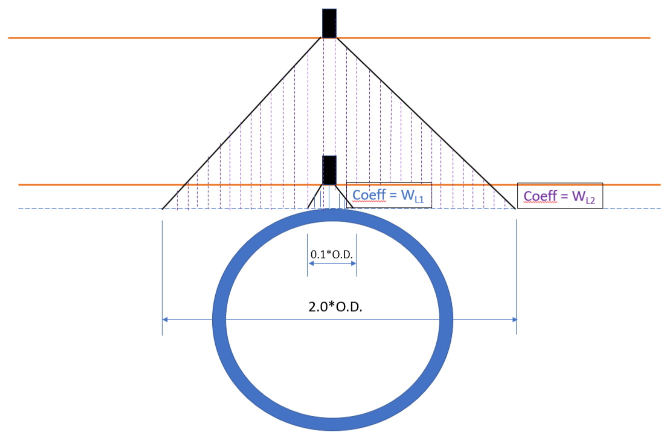


Figure D-3. Application of Live Load Coefficients for Circular RCP at Pipe Invert.

$C_{miL1} = 0.185$ $C_{niL1} = 0.152$ coefficients for live load moment and thrust at 2·O.D.
$C_{miL2} = 0.237$ $C_{niL2} = -0.004$ coefficients for live load moment and thrust at 0.1·O.D.
Interpolate to find the appropriate live load moment and thrust coefficients for the governing live load spread.

The live load spread length from Example D1 is: $l_w = 7.1$ feet

The minimum spread length is $0.10 \cdot O.D. = 0.10 \cdot 3.67 = 0.367$

The maximum spread length is $2.0 \cdot O.D. = 2.0 \cdot 3.67 = 7.33$

Live load moment coefficient:

$$C_{miL} = C_{miL2} - \left(\frac{l_w - l_{min}}{l_{max} - l_{min}}\right)(C_{miL2} - C_{miL1})$$

$$C_{miL} = 0.237 - \left(\frac{7.1 - 0.367}{7.33 - 0.367}\right)(0.237 - 0.185)$$

$$C_{miL} = 0.187$$

Live load thrust coefficient:

$$C_{niL} = C_{niL2} + \left(\frac{l_w - l_{min}}{l_{max} - l_{min}}\right)(C_{niL1} - C_{niL2})$$

$$C_{niL} = -0.004 + \left(\frac{7.1 - 0.367}{7.33 - 0.367}\right)(0.152 - (-0.004))$$

$$C_{niL} = 0.146$$

Using the equations provided above, the moments and thrust at the pipe invert are as follows:

For earth load:

$M_E = C_{mie} \cdot W_E \cdot r_m$ $\qquad$ $N_E = C_{nie} \cdot W_E$

$M_E = 0.191 \cdot 3190 \cdot 20.0$ $\qquad$ $N_E = 0.128 \cdot 3190$

M_E =12,190 in-lbs/ft N_E = 408 lbs/ft

For pipe load:

$M_p = C_{mip} \cdot W_p \cdot r_m$ $N_p = C_{nip} \cdot W_p$

$M_p = 0.235 \cdot 524 \cdot 20.0$ $N_p = 0.077 \cdot 524$

M_p = 2,462 in-lbs/ft N_p = 40 lbs/ft

For fluid load:

$M_F = C_{mif} \cdot W_F \cdot r_m$ $N_F = C_{nif} \cdot W_F$

$M_F = 0.160 \cdot 441 \cdot 20.0$ $N_F = -0.403 \cdot 441$

M_F = 1,411 in-lbs/ft N_F = -178 lbs/ft

For live load:

$M_L = C_{miL} \cdot W_L \cdot r_m$ $N_L = C_{niL} \cdot W_L$

$M_L = 0.187 \cdot 1585 \cdot 20.0$ $N_L = 0.146 \cdot 1585$

M_L = 5,922 in-lbs/ft N_L = 231 lbs/ft

D2 Step 2.2.2 - Find the moments, thrusts and shears at the critical location for shear

The critical location for shear is not directly at the invert, but roughly between 20 to 25 degrees from the invert. Table D-2 provides the appropriate moment, thrust and shear coefficients to be used at the critical location for shear.

Table D-2. Design Coefficients (Moment, Thrust) for Circular RCP at Critical Shear Location.

m – moment, n – thrust, v - shear		**Installation Types**					
		Type 1			**Type 2**		
Location	**Load Type**	C_{mi}	C_{ni}	C_{vi}	C_{mi}	C_{ni}	C_{vi}
Shear Critical	W_p	0.119	0.198	0.425	0.119	0.198	0.386
	W_E	0.066	0.238	0.157	0.088	0.235	0.225
	W_F	0.063	-0.395	0.153	0.078	-0.370	0.217
	W_L	0.80*C_{mi}@ 0 deg.	0.070	0.210	0.80*C_{mi}@ 0 deg.	0.070	0.270
m – moment, n – thrust, v - shear		**Installation Types**					
		Type 3			**Type 4**		
Location	**Load Type**	C_{mi}	C_{ni}	C_{vi}	C_{mi}	C_{ni}	C_{vi}
Shear Critical	W_p	0.118	0.198	0.386	0.118	0.198	0.386
	W_E	0.109	0.242	0.288	0.145	0.217	0.374
	W_F	0.094	-0.344	0.268	0.118	-0.319	0.344
	W_L	0.80·C_{mi}@ 0 degrees	0.070	0.310	0.80·C_{mi}@ 0 degrees	0.070	0.350

From Table D-2, the moment and thrust coefficients for a Type 4 installation are as follows:

$Cv_{miE} = 0.145$ $Cv_{niE} = 0.217$ $Cv_{viE} = 0.374$ coefficients for earth load
$Cv_{mip} = 0.118$ $Cv_{nip} = 0.198$ $Cv_{vip} = 0.386$ coefficients for pipe load
$Cv_{miF} = 0.118$ $Cv_{niF} = -0.319$ $Cv_{viF} = 0.344$ coefficients for fluid load
$Cv_{miL} = 0.8 \cdot 0.187 = 0.150$ $Cv_{niL} = 0.070$ $Cv_{viL} = 0.350$ coefficients for live load
(Note - 0.187 is the value for the live load moment at the invert, 0 degrees)
Using the equations provided earlier in this example problem, the moments, thrust, and shears at the critical shear location are as follows:

For earth load:

$Mv_E = Cv_{miE} \cdot W_e \cdot r_m$	$Nv_E = Cv_{niE} \cdot W_E$	$V_E = Cv_{viE} \text{ x } W_E$
$Mv_E = 0.145 \cdot 3190 \cdot 20.0$	$Nv_E = 0.217 \cdot 3190$	$V_E = 0.374 \cdot 3190$
$Mv_E = 9{,}251$ in-lbs/ft	$Nv_E = 692$ lbs/ft	$V_E = 1{,}193$ lbs/ft

For pipe load:

$Mv_p = Cv_{mip} \cdot W_p \cdot r_m$	$Nv_p = Cv_{nip} \cdot W_p$	$V_p = Cv_{vip} \cdot W_p$
$Mv_p = 0.118 \cdot 524 \cdot 20.0$	$Nv_p = 0.198 \cdot 524$	$V_p = 0.386 \cdot 524$
$Mv_p = 1{,}236$ in-lbs/ft	$Nv_p = 104$ lbs/ft	$V_p = 202$ lbs/ft

For fluid load:

$Mv_F = Cv_{miF} \cdot W_F \cdot r_m$	$Nv_f = C_{niF} \cdot W_F$	$V_F = Cv_{viF} \cdot W_F$
$Mv_F = 0.118 \cdot 441 \cdot 20.0$	$Nv_F = -0.319 \cdot 441$	$V_F = 0.344 \cdot 441$
$Mv_F = 1{,}041$ in-lbs/ft	$Nv_F = -141$ lbs/ft	$V_F = 152$ lbs/ft

For live load:

$Mv_L = Cv_{miL} \cdot W_L \cdot r_m$	$Nv_L = C_{niL} \cdot W_L$	$V_L = Cv_{viL} \cdot W_L$
$Mv_L = 0.150 \cdot 1585 \cdot 20.0$	$Nv_L = 0.070 \cdot 1585$	$V_L = 0.350 \cdot 1585$
$Mv_L = 4{,}750$ in-lbs/ft	$Nv_L = 111$ lbs/ft	$V_L = 555$ lbs/ft

D2 Step 2.2.3 - Calculate the total moment, thrust, and shear values

For service loads:

$M_s = M_E + M_p + M_F + M_L$	$N_s = N_E + N_p + N_F + N_L$
$M_s = 12{,}190 + 2{,}462 + 1{,}411 + 5{,}922$	$N_s = 408 + 40 + (-178) + 231$
$M_s = 21{,}985$ in-lbs/ft	$N_s = 501$ lbs/ft

For ultimate loads:

Load Factors (as per ASTM C361):

LD = 1.6 Load factor for dead loads (including: earth, pipe, and fluid)

LL = 1.75 Load factor for live loads

LDC = 1.0 Load factor for thrusts when they are compressive (in this example the earth, pipe, and live loads are compressive, and the fluid load is tensile)

LV = 1.3 Load factor for all dead loads when calculating shear

Ultimate moments and thrusts needed for flexural steel calculation:

$M_u = LD \cdot (M_E + M_p + M_F) + (LL \cdot M_L)$

$M_u = 1.6 \cdot (12{,}190 + 2{,}462 + 1{,}411) + (1.75 \cdot 5{,}922) = 36{,}064$ in-lbs/ft

$N_u = LDC \cdot (N_E + N_p + N_L) + (LD \cdot N_F)$

$N_u = 1.0 \cdot (408 + 40 + 231) + (1.6 \cdot -178) = 396$ lbs/ft

$N_{uc} = LD \cdot (N_E + N_p + N_F) + (LL \cdot N_L)$

$N_{uc} = 1.6 \cdot (408 + 40 + (-178)) + (1.75 \cdot 231) = 839$ lbs/ft

Note: When the pipe wall is checked for sufficient compressive strength, this higher factored compressive thrust is used.

Moments, thrusts, and shear values needed for the shear calculation:

$M_{uv} = LV \cdot (Mv_E + Mv_p + Mv_F) + (LL \cdot Mv_L)$ $N_{uv} = LV \cdot (Nv_E + Nv_p + Nv_F) + (LL \cdot Nv_L)$

$M_{uv} = 1.3 \cdot (9251 + 1236 + 1041) + (1.75 \cdot 4750)$ $N_{uv} = 1.3 \cdot (692 + 104 - 141) + (1.75 \cdot 111)$

$M_{uv} = 23{,}290$ in-lbs/ft $N_{uv} = 1{,}046$ lbs/ft

$V_u = LV \cdot (V_E + V_p + V_F) + (LL \cdot Vv_L)$

$V_u = 1.3 \cdot (1{,}193 + 202 + 152) + (1.75 \cdot 555)$

$V_u = 2{,}982$ lbs/ft

D2 Step 2.2.4 - Find the amount of reinforcement required for ultimate load flexure

$$A2_{si} = \frac{\left[g\varphi_f d - N_u - \sqrt{g[g(\varphi_f d)^2 - N_u(2\varphi_f d - t) - 2M_u]}\right]}{f_y}$$

Note: Since Appendix X2 of ASTM C361 does not include a flexural steel equation, Equation 12.10.4.2.4a-1 from the AASHTO LRFD Bridge Design Specifications is used.

Where:

$\varphi_f = 0.95$ strength reduction factor for flexure per Appendix X2 ASTM C361

$d = t - 0.875 - 0.3$ depth from the compression face of the concrete to the centroid of the reinforcing steel (based on equation in Appendix X2 of ASTM C361) (in)

$d = 4.0 - 0.875 - 0.3 = 2.825$ inches

$g = 0.85 \cdot b \cdot f`c$

$b = 12$ unit strip width (in)

$g = 0.85 \cdot 12 \cdot 5000$

$g = 51{,}000$ lbs/in

$$A2_{si} = \frac{\left[51000(0.95)(2.825) - 396 - \sqrt{51000[51000((0.95)(2.825))^2 - 396(2(0.95)(2.825) - 4.0) - 2(36064)]}\right]}{60000}$$

$A2_{si} = 0.231\ in^2/ft$

D2 Step 2.2.5 - Determine the maximum amount of reinforcement allowed without concrete compression failure.

$$A_{smax} = \left(\frac{\left[\frac{55000g`\varphi_f d}{(87000+f_y)}\right] - 075N_{uc}}{f_y}\right)$$ (Equation X2.20 of ASTM C361)

Where:

$$g` = b \cdot f`_c\left[0.85 - 0.05\ \frac{(f`_c - 4000)}{1000}\right]$$ (Equation X2.21 of ASTM C361)

$0.65\ b \cdot f`_c < g` < 0.85\ b \cdot f`_c$

$$g` = (12)(5000)\left[0.85 - 0.05\ \frac{(5000-4000)}{1000}\right]$$

$g` = 48{,}000$

$$A_{smax} = \left(\frac{\left[\frac{55000(48000)(0.95)(2.825)}{(87000+60000)}\right] - 075(839)}{60000}\right)$$

A_{smax} = 0.793 in^2/ft > 0.231 in^2/ft

$A2_{si}$ is less than the maximum allowable reinforcing steel, so OK

D2 Step 2.2.6 - Determine the amount of reinforcement needed to resist cracking

$$A2_{scri} = \frac{B_1}{F_{cr}30000\varphi_f d}\left[\frac{M_s + N_s\left(d - \frac{t}{2}\right)}{i*j} - C_1 bt^2\sqrt{f'_c}\right]$$ (Equation X2.4 of ASTM C361)

Where:

F_{cr} = crack control factor set at 1.0 for an average crack width of 0.01 inch

C_1 = crack control coefficient based on type of reinforcement, set at 1.5 for welded wire reinforcement

$j = 0.74 + 0.1\frac{e}{d}$ $\qquad$ j_{max} = 0.9

$i = \frac{1}{1 - \frac{jd}{e}}$

$e = \frac{M_s}{N_s} + d - \frac{t}{2}$

$B_1 = \sqrt[3]{\frac{t_b s_l}{2n}}$

Where:

t_b = clear cover over reinforcement (in) (set at 1 inch)
s_l = spacing of circumferential reinforcement (assumed 4 inches)
n = number of reinforcement layers (use "1")

Thus:

$B_1 = \sqrt[3]{\frac{(1)(4)}{2(1)}}$ $\qquad$ B_1 = 1.26

$e = \frac{21{,}985}{501} + 2.825 - \frac{4.0}{2}$ $\qquad$ e = 44.71

$j = 0.74 + 0.1\frac{44.71}{2.825}$ $j = 2.32 > 0.9$ (use $j = 0.9$)

$$i = \frac{1}{1-\frac{(0.9)(2.825)}{44.71}}$$ $i = 1.06$

$$A2_{scri} = \frac{1.26}{(1.0)(30000)(0.95)(2.825)}\left[\frac{21985+501\left(2.825-\frac{4.0}{2}\right)}{1.06*0.9} - (1.5)(12)(4.0)^2\sqrt{5000}\right]$$

$A2_{scri} = 0.048\ in^2/ft < 0.231\ in^2/ft$ Steel required for crack control is less than the flexural reinforcing steel, so OK

D2 Step 2.2.6 - Determine if reinforcing stirrups are required

D2 Step 2.2.6.1 - Determine if stirrups are required for radial tension

$$A_{smax} = \left(\frac{b}{12}\right)\left(\frac{16r_sF_{rp}\sqrt{f'_c}\left(\frac{\varphi_v}{\varphi_f}\right)F_{rt}}{f_y}\right)$$ (Equation X2.17 of ASTM C361)

Where:

Φ_v = strength reduction factor for shear and radial tension = 0.9

F_{rp} = process and material factor for radial tension strength = 1.0

r_s = radius to the inside reinforcement (in)

$r_s = 36/2 + 1 = 19$

$F_{rt} = 1 + 0.00833 \cdot (72 - S_i)$ for 12 in $\leq S_i \leq$ 72 in

$F_{rt} = \frac{(144-S_i)^2}{26000} + 0.80$ for 72 in $\leq S_i \leq$ 144 in

$F_{rt} = 1 + 0.00833 \cdot (72 - 36)$ $F_{rt} = 1.3$

$$A_{smax} = \left(\frac{12}{12}\right)\left(\frac{16(19)(1.0)\sqrt{5000}\left(\frac{0.9}{0.95}\right)1.3}{60000}\right)$$

A_{smax} = 0.441 in^2/ft > 0.231 in^2/ft $A2_{si}$ is less than the maximum allowable reinforcing steel for radial tension, so OK

D2 Step 2.2.6.2 - Determine if reinforcing stirrups are required for diagonal tension

$$V2_c = 2 \cdot \varphi_v \cdot b \cdot d \cdot F_{vp}\sqrt{f'_c}\left(\frac{F_d F_{ex}}{F_c}\right)$$ (Equation X2.10 of ASTM C361)

Where:

d = distance from compression face to centroid of tension reinforcement (in)
b = width of section = 12 inches
F_{vp} = process and material factor for shear strength = 1.0

$$F_d = 0.8 + \frac{1.6}{d}$$

$$F_d = 0.8 + \frac{1.6}{2.825}$$ F_d = 1.366

$$F_{ex} = 2.2 \cdot (1 - 2.75 \cdot (\varepsilon 2_{xu})^{0.25})$$

$$\varepsilon 2_{xu} = \frac{\left(\frac{M_{uv}}{0.9d}\right) + 0.5V_u \cos\theta_v - 0.4N_{uv} - 0.5N_{up}}{E_s A_s}$$ (Equation X2.14 of ASTM C361)

$\theta_v = 37/F_d = 37/1.366 = 27$ degrees

$$\varepsilon 2_{xu} = \frac{\left(\frac{23{,}290}{0.9\,(2.825)}\right) + 0.5(2{,}982)\cos(27) - 0.4(1{,}046) - 0.5(0)}{(29000000)(0.231)}$$ $\varepsilon 2_{xu}$ = 0.0015

Note: We are evaluating Load Condition 2, so the internal thrust from fluid pressure is zero for this condition.

$F2_{ex} = 2.2(1 - 2.75(0.0015)^{0.25})$ $F2_{ex}$ = 1.009

$$F_c = 1 + \frac{d}{2 \cdot r}$$

F_c = 1.078

$$V2_c = 2(0.9)(12)(2.825)(1.0)\sqrt{5000}\left(\frac{(1.366)(1.009)}{1.078}\right)$$

$V2_c$ = 5,518 lbs/ft > 2,982 lbs/ft   Shear capacity is greater than the applied shear, so the pipe wall is satisfactory for shear.

$A2_{si}$ = 0.231 in^2/ft   Inside steel area required for Load Condition 2

D2 STEP 2.3 CALCULATE THE STEEL REQUIREMENTS FOR THE OUTSIDE STEEL CAGE FOR LOAD CONDITION 2

D2 Step 2.3.1 - Find the moments and thrusts near the springline of the pipe.

From the Table D-1, the moment and thrust coefficients for a Type 4 installation are:

C_{mse} = -0.127 C_{nse} = 0.504 coefficients for earth load moment and earth load thrust
C_{msp} = -0.101 C_{nsp} = 0.287 coefficients for pipe load moment and pipe load thrust
C_{msf} = -0.095 C_{nsf} = -0.057 coefficients for fluid load moment and fluid load thrust
C_{miL1} = -0.121 C_{niL1} = 0.495 coefficients for live load moment and thrust at 2·O.D.
C_{miL2} = -0.168 C_{niL2} = 0.491 coefficients for live load moment and thrust at 0.1·O.D.
Interpolate to find the appropriate live load springline moment and thrust coefficients for the governing live load spread.

The live load spread length from Example D1 is: l_w = 7.1 feet

The minimum spread length is 0.10 · O.D. = 0.10·3.67 = 0.367

The maximum spread length is 2.0 · O.D. = 2.0·3.67 = 7.33

Live load moment coefficient:

$$C_{miL} = C_{miL2} - \left(\frac{l_w - l_{min}}{l_{max} - l_{min}}\right)(C_{miL2} - C_{miL1})$$

$$C_{miL} = -0.168 - \left(\frac{7.1 - 0.367}{7.33 - 0.367}\right)(-0.168 - 0.121)$$

C_{miL} = -0.123

Live load thrust coefficient:

$$C_{niL} = C_{niL2} + \left(\frac{l_w - l_{min}}{l_{max} - l_{min}}\right)(C_{niL1} - C_{niL2})$$

$$C_{niL} = 0.491 + \left(\frac{7.1 - 0.367}{7.33 - 0.367}\right)(0.495 - 0.491)$$

C_{niL} = 0.495

Applying these coefficients to the appropriate equations:

For earth load:

$MS_E = C_{mse} \cdot W_E \cdot r_m$ $NS_E = C_{nse} \cdot W_E$

$MS_E = -0.127 \cdot 3190 \cdot 20.0$ $NS_E = 0.504 \cdot 3190$

MS_E = -8,103 in-lbs/ft NS_E = 1,608 lbs/ft

For pipe load:

$MS_p = C_{msp} \cdot W_p \cdot r_m$ $NS_p = C_{nsp} \cdot W_p$

$MS_p = -0.101 \cdot 524 \cdot 20.0$ $NS_p = 0.287 \cdot 524$

$MS_p = -1058$ in-lbs/ft $NS_p = 150$ lbs/ft

For fluid load:

$MS_F = C_{msf} \cdot W_F \cdot r_m$ $NS_F = C_{nsf} \cdot W_F$

$MS_F = -0.095 \cdot 441 \cdot 20.0$ $NS_F = -0.057 \cdot 441$

$MS_F = -838$ in-lbs/ft $NS_F = -25$ lbs/ft

For live load:

$MS_L = C_{msL} \cdot W_L \cdot r_m$ $NS_L = C_{nsL} \cdot W_L$

$MS_L = -0.123 \cdot 1585 \cdot 20.0$ $NS_L = 0.495 \cdot 1585$

$MS_L = 3,899$ in-lbs/ft $NS_L = 785$ lbs/ft

D2 Step 2.3.2 - Find the total moment, thrust, and shear values

For service loads:

$MS_s = MS_E + MS_p + MS_F + MS_L$ $NS_s = NS_E + NS_p + NS_F + NS_L$

$MS_s = -8103 - 1058 – 838 - 3899$ $NS_s = 1608 + 150 – 25 + 785$

$MS_s = -13,898$ in-lbs/ft $NS_s = 2,518$ lbs/ft

For ultimate loads:

LD = 1.6 Load factor for dead loads (including: earth, pipe, and fluid)

LL = 1.75 Load factor for live loads

LDC = 1.0 Load factor for thrusts when they are compressive (in this example the earth, pipe, and live loads are compressive, and the fluid load is tensile)

LV = 1.3 Load factor for all dead loads when calculating shear

Moments and thrusts needed for flexural calculation:

$MS_u = LD \cdot (MS_E + MS_p + MS_F) + (LL \cdot MS_L)$

$MS_u = 1.6 \cdot (-8,103 -1,058 -838) + (1.75 \cdot -3,899)$

$MS_u = -22,822$ in-lbs/ft

$NS_u = LDC \cdot (NS_E + NS_p + NS_L) + (LD \cdot NS_F)$

$NS_u = 1.0 \cdot (1608 + 150 + 785) + (1.6 \cdot -25)$

$NS_u = 2,503$ lbs/ft

$NS_{uc} = LD \cdot (NS_E + NS_p + NS_F) + (LL \cdot NS_L)$

$NS_{uc} = 1.6 \cdot (1608 + 150 + (-25)) + (1.75 \cdot 785)$

$NS_{uc} = 4{,}147$ lbs/ft

Note: When the pipe wall is checked for sufficient compressive strength, this higher factored compressive thrust will be used.

D2 Step 2.3.3 - Find the amount of reinforcement required for ultimate load flexure

Note: The moments at the springline are negative to show that the bending stress is outward and not inward. However, it is the absolute value of the moment that is critical for determining the flexural reinforcement area, so we will use the absolute (positive) value for the springline moments.

$$A2_{so} = \frac{\left[g\varphi_f d - NS_u - \sqrt{g[g(\varphi_f d)^2 - NS_u(2\varphi_f d - t) - 2|MS_u|]}\right]}{f_y}$$

Note: Since Appendix X2 of ASTM C361 does not include a flexural steel equation, Equation 12.10.4.2.4a-1 from the AASHTO LRFD Bridge Design Specifications is used.

Where:

$\varphi_f = 0.95$	strength reduction factor for flexure per Appendix X2 ASTM C361
$d = t - 0.875 - 0.3$	depth from the compression face of the concrete to the centroid of the reinforcing steel (based on equation in Appendix X2 of ASTM C361) (in)

$d = 4.0 - 0.875 - 0.3 = 2.825$ in.

$g = 0.85 \cdot b \cdot f'c$

$b = 12$	unit strip width (in)

$g = 0.85 \cdot 12 \cdot 5000$

$g = 51{,}000$ lbs/in

$$A2_{so} = \frac{\left[51000(0.95)(2.825) - 2503 - \sqrt{51000[51000((0.95)(2.825))^2 - 2503(2(0.95)(2.825) - 4.0) - 2(22822)]}\right]}{60000}$$

$A2_{so} = 0.116\ in^2/ft$

D2 Step 2.3.4 - Determine the maximum amount of reinforcement allowed without concrete compression failure

$$A2_{smaox} = \left(\frac{\left[\frac{55000g`\varphi_f d}{(87000+f_y)}\right] - 075NS_{uc}}{f_y}\right) \qquad \text{(Equation X2.20 of ASTM C361)}$$

Where:

$$g` = b \cdot f`_c \left[0.85 - 0.05\ \frac{(f`_c - 4000)}{1000}\right] \qquad \text{(Equation X2.21 of ASTM C361)}$$

$0.65 \cdot b \cdot f`_c < g` < 0.85 \cdot b \cdot f`_c$

$$g` = (12)(5000)\left[0.85 - 0.05\ \frac{(5000-4000)}{1000}\right]$$

$g` = 48{,}000$

$$A2_{smaxo} = \left(\frac{\left[\frac{55000(48000)(0.95)(2.825)}{(87000+60000)}\right] - 075(4147)}{60000}\right)$$

$A2_{smaxo} = 0.751\ in^2/ft$

$A2_{so}$ is less than the maximum allowable reinforcing steel, so OK

D2 Step 2.3.5 - Determine the amount of reinforcement needed to resist cracking

$$A_{scr} = \frac{B_1}{F_{cr}30000\varphi_f d}\left[\frac{MS_s + NS_s\left(d-\frac{t}{2}\right)}{i*j} - C_1 bt^2\sqrt{f`_c}\right] \qquad \text{(Equation X2.4 of ASTM C361)}$$

Where:

F_{cr} = crack control factor set at 1.0 for an average crack width of 0.01 inches

C_1 = Crack control coefficient based on type of reinforcement, set at 1.5 for welded wire reinforcement

$j = 0.74 + 0.1\frac{e}{d}$ $\qquad$ $j_{max} = 0.9$

$$i = \frac{1}{1-\frac{jd}{e}}$$

$$e = \frac{MS_s}{NS_s} + d - \frac{t}{2}$$

$$B_l = \sqrt[3]{\frac{t_b s_l}{2n}}$$

Where:

t_b = clear cover over reinforcement = 1 inch
s_l = spacing of circumferential reinforcement (assume 4 inches)
n = number of reinforcement layers (use "1")

Thus:

$$B_l = \sqrt[3]{\frac{(1)(4)}{2(1)}} \qquad B_l = 1.26$$

$$e = \frac{13898}{2518} + 2.825 - \frac{4.0}{2} \qquad e = 6.34$$

$$j = 0.74 + 0.1\frac{6.34}{2.825} \qquad j = 0.964 > 0.9, \text{ so use } j = 0.9$$

$$i = \frac{1}{1 - \frac{(0.9)(2.825)}{6.34}} \qquad i = 1.67$$

$$A2_{scro} = \frac{1.26}{(1.0)(30000)(0.95)(2.825)}\left[\frac{13898 + 2518\left(2.825 - \frac{4.0}{2}\right)}{1.67 * 0.9} - (1.5)(12)(4.0)^2\sqrt{5000}\right]$$

$A2_{scro}$ = -0.152 in^2/ft < 0.116 in^2/ft — Steel required for crack control is less than the flexural reinforcing steel, so OK

Note: If crack control is adequate for the inside steel, then it would be expected to be adequate for the outside steel provided that the concrete cover is the same.

D2 Step 2.3.6 - Determine if reinforcing stirrups are required

If stirrups are not required at the invert area, then they will not be required at the springline area, since the radial tension stresses from bending are less at the springline, and the applied shear is less at the springline.

A_s = 0.116 in^2/ft — Outside steel area required for Load Condition 2

D2 STEP 3. LOAD CONDITION 3: EXTERNAL AND INTERNAL LOADS ACTING CONCURRENTLY

Note: Moments, thrusts, and shears have already been calculated for the external loads; thus, there is no need to calculate them again.

D2 Step 3.1 DETERMINE THE THRUST IN THE PIPE WALL RESULTING FROM THE INTERNAL PRESSURE

$N_{pr} = -f_{ct} \cdot t \cdot b$

Where:

N_{pr} = tensile thrust force resulting from internal pressure (lbs/ft)

f_{ct} = tensile stress in the concrete from internal pressure (psi) – see Load Condition 1

t = wall thickness (in)

b = unit width (in)

$N_{pr} = -68.2 \cdot 4.0 \cdot 12$

N_{pr} = -3,273 psi

D2 Step 3.2 CALCULATE THE STEEL REQUIREMENTS FOR THE INSIDE STEEL CAGE FOR LOAD CONDITION 3

D2 Step 3.2.1 - Determine the thrust values for use in the design equations.

Service Load Thrust:

$N3_s = N_s + N_{pr}$

Where:

$N3_s$ = Total service load thrust at invert for Load Condition 3 (psi)

$N3_s = 501 – 3,273$

$N3_s$ = -2772 psi

Ultimate Load Thrust for Flexure:

$N3_u = N_u + (LI \cdot N_{pr})$

Where:

$N3_u$ = Total factored thrust at invert for Load Condition 3 (psi)

LI = 1.5 Load Factor for Internal Pressure

$N3_u = 396 + 1.5 \cdot (-3,273)$

$N3_u$ = -4,514 psi

Ultimate Pressure Thrust at Critical Location for Shear:

$N_{up} = LV \cdot N_{pr}$

Where:

N_{up} = Total factored internal pressure thrust critical shear location for Load Condition 3

$N_{up} = 1.3 \cdot -3{,}273$

N_{up} = -4,256 psi

D2 Step 3.2.2 - Find the amount of reinforcement required for ultimate load flexure

$$A3_{si} = \frac{\left[g\varphi_f d - N3_u - \sqrt{g[g(\varphi_f d)^2 - N3_u(2\varphi_f d - t) - 2M_u]}\right]}{f_y}$$

Note: Since Appendix X2 of ASTM C361 does not include a flexural steel equation, Equation 12.10.4.2.4a-1 from the AASHTO LRFD Bridge Design Specifications is used.

Where:

$\varphi_f = 0.95$	strength reduction factor for flexure as per Appendix X2 of ASTM C361
$d = t - 0.875 - 0.3$	depth from the compression face of the concrete to the centroid of the reinforcing steel (based on equation in Appendix X2 of ASTM C361) (in)

$d = 4.0 - 0.875 - 0.3 = 2.825$ inches

$g = 0.85 \cdot b \cdot f'c$

b =12	unit strip width (in)

$g = 0.85 \cdot 12 \cdot 5000$

g = 51,000 lbs/in

$$A3_{si} = \frac{\left[51000(0.95)(2.825) - (-4514) - \sqrt{51000[51000((0.95)(2.825))^2 - (-4514)(2(0.95)(2.825) - 4.0) - 2(36064)]}\right]}{60000}$$

$A3_{si} = 0.290\ in^2/ft$

D2 Step 3.2.3 - Determine the maximum amount of reinforcement allowed without concrete compression failure

Note: The additional steel reinforcement needed for Load Condition 3 is to resist the tensile force from internal pressure. Since the moment is the same in Load Condition 3 as it is in Load Condition 2, as long as the maximum amount of reinforcement is not exceeded in Load Condition 2, then it will not be exceeded in Load Condition 3 since the additional steel is not being added to balance compressive forces from bending.

D2 Step 3.2.4 - Determine the amount of reinforcement needed to resist cracking

Since the resultant thrust is negative, use Equation X2.5 of ASTM C361.

$$A3_{scri} = \frac{B_1}{F_{cr}30000\varphi_f d}\left[1.1M_s - 0.6N3_s d - C_1 bt^2\sqrt{f'_c}\right]$$ (Equation X2.5 of ASTM C361)

Where:

F_{cr} = crack control factor set at 1.0 for an average crack width of 0.01 inches

C_1 = Crack control coefficient based on type of reinforcement, set at 1.5 for welded wire reinforcement

$B_1 = \sqrt[3]{\frac{t_b s_l}{2n}}$

Where:

t_b = clear cover over reinforcement = 1 inch
s_l = spacing of circumferential reinforcement (assume 4 inches)
n = number of reinforcement layers (use "1")

Thus:

$B_1 = \sqrt[3]{\frac{(1)(4)}{2(1)}}$ $B_1 = 1.26$

$$A3_{scri} = \frac{1.26}{(1.0)(30000)(0.95)(2.825)}\left[1.1(21985) - 0.6(-2772)(2.825) - (1.5)(12)(4.0)^2\sqrt{5000}\right]$$

$A3_{scri} = 0.133\ in^2/ft < 0.290\ in^2/ft$ Steel required for crack control is less than the flexural reinforcing steel, so OK

D2 Step 3.3 DETERMINE IF REINFORCING STIRRUPS ARE REQUIRED

D2 Step 3.3.1 – Check first for radial tension requirements

Note: Radial tension is a function of the bending stress in the pipe wall. Since only the tensile stress in the pipe wall is increased in Load Condition 3, as long as there is not a problem with radial tension in Load Condition 2, there will not be a problem with radial tension in Load Condition 3.

D2 Step 3.3.2 – Next, check for diagonal tension requirements

$V3_c = 2 \cdot \varphi_v \cdot b \cdot d \cdot F_{vp}\sqrt{f'_c}\left(\frac{F_d . F3_{ex}}{F_c}\right)$ (Equation X2.10 of ASTM C361)

Where:

d = distance from compression face to centroid of tension reinforcement (in)

b = width of section = 12 inches

F_{vp} = process and material factor for shear strength (1.0)

$F_d = 0.8 + \frac{1.6}{d}$

$F_d = 0.8 + \frac{1.6}{2.825}$ $F_d = 1.366$

$F_{ex} = 2.2(1 - 2.75(\varepsilon_{xu})^{0.25})$

$\varepsilon 3_{xu} = \frac{\left(\frac{M_{uv}}{0.9d}\right) + 0.5V_u \cos\theta_v - 0.4N_{uv} - 0.5N_{up}}{E_s A_s}$ (Equation X2.14 of ASTM C361)

$\theta_v = 37/F_d = 37/1.366 = 27$ degrees

$\varepsilon 3_{xu} = \frac{\left(\frac{23290}{0.9\,(2.825)}\right) + 0.5(2982)\cos(27) - 0.4(1046) - 0.5(-4{,}256)}{(29000000)(0.290)}$ $\varepsilon_{xu} = 0.00129$

$F3_{ex} = 2.2(1 - 2.75(0.00129)^{0.25})$ $F3_{ex} = 1.053$

$F_c = 1 + \frac{d}{2r}$ $F_c = 1.078$

$V3_c = 2(0.9)(12)(2.825)(1.0)\sqrt{5000}\left(\frac{(1.366)(1.053)}{1.078}\right)$

$V3_c$ = 5,757 lbs/ft > 2,982 lbs/ft Shear capacity is greater than the applied shear, so the pipe wall is satisfactory for shear.

$A3_{si} = 0.290\ in^2/ft$ Inside steel area required for Load Condition 3

D2 Step 3.4 CALCULATE THE STEEL REQUIREMENTS FOR THE OUTSIDE STEEL CAGE FOR LOAD CONDITION 3

D2 Step 3.4.1 - Determine the thrust values for use in the design equations

Service Load Thrust:

$NS3_s = NS_s + N_{pr}$

Where:

$NS3_s$ = Total Service Load Thrust at Springline for Load Condition 3 (psi)

$NS3_s = 2{,}518 - 3{,}273 = -755$ psi

Ultimate Load Thrust for Flexure:

$NS3_u = N_u + (LI \cdot N_{pr})$

Where:

$NS3_u$ = Total factored thrust at invert for Load Condition 3 (psi)

$LI = 1.5$ Load factor for internal pressure

$NS3_u = 2{,}503 + 1.5\cdot(-3{,}273) = -2{,}407$ psi

D2 Step 3.4.2 - Find the amount of reinforcement required for ultimate load flexure

Note: The moments at the springline are negative to show that the bending stress is outward and not inward. It is the absolute value of the moment that is critical for determining the flexural reinforcement area, so use the absolute (positive) value for the springline moments; however, the compression and tensile thrust forces must be keep the correct sign.

$$AS3_{so} = \frac{\left[g\varphi_f d - NS3_u - \sqrt{g[g(\varphi_f d)^2 - NS3_u(2\varphi_f d - t) - 2|MS_u|]}\right]}{f_y}$$

Note: Since Appendix X2 of ASTM C361 does not include a flexural steel equation, Equation 12.10.4.2.4a-1 from the AASHTO LRFD Bridge Design Specifications is used.

$$A3_{so} = \frac{\left[51000(0.95)(2.825) - (-2407) - \sqrt{51000[51000((0.95)(2.825))^2 - (-2407)(2(0.95)(2.825) - 4.0) - 2(22822)]}\right]}{60000}$$

$A3_{so} = 0.176$ in^2/ft

D2 Step 3.4.3 - Determine the maximum amount of reinforcement allowed without concrete compression failure

Note: The additional steel reinforcement needed for Load Condition 3 is to resist the tensile force from internal pressure. Since the moment is the same in Load Condition 3 as it is in Load Condition 2, as long as the maximum amount of reinforcement is not exceeded in Load Condition 2, then it will not be exceeded in Load Condition 3 since the additional steel is not being added to balance compressive forces from bending.

D2 Step 3.4.4 - Determine the amount of reinforcement needed to resist cracking

Since the resultant thrust is negative, use Equation X2.5 of ASTM C361.

$A3_{scri} = \frac{B_1}{F_{cr}30000\varphi_f d}\left[1.1MS_s - 0.6NS3_s d - C_1 bt^2\sqrt{f'_c}\right]$ (Equation X2.5 of ASTM C361)

Where:

F_{cr} = crack control factor set at 1.0 for an average crack width of 0.01 inches

C_1 = Crack control coefficient based on type of reinforcement, set at 1.5 for welded wire reinforcement

$B_1 = \sqrt[3]{\frac{t_b s_l}{2n}}$

Where:

t_b = clear cover over reinforcement = 1 inch
s_l = spacing of circumferential reinforcement (assume 4 inches)
n = number of reinforcement layers (use "1")

Thus:

$B_1 = \sqrt[3]{\frac{(1)(4)}{2(1)}}$ $B_1 = 1.26$

$A3_{scri} = \frac{1.26}{(1.0)(30000)(0.95)(2.825)}\left[1.1(13898) - 0.6(-755)(2.825) - (1.5)(12)(4.0)^2\sqrt{5000}\right]$

$A3_{scri} = -0.06\ in^2/ft < 0.176\ in^2/ft$ Steel required for crack control is less than the flexural reinforcing steel, so OK

Note: If crack control is adequate for the inside steel, then it would be expected to be adequate for the outside steel provided that the concrete cover is the same.

D2 Step 3.4.5 - Determine if reinforcing stirrups are required for outer cage

Note: If stirrups are not required at the invert area, then they will not be required at the springline area, since the radial tension stresses from bending are less at the springline, and the applied shear is less at the springline.

$AS3_{so} = 0.176\ in^2/ft$ Outside steel area required for Load Condition 3

D2 Step 4. DETERMINE THE FINAL REQUIRED STEEL AREA FOR THE PIPE

Take the highest total required steel area from the three load conditions

Load Condition 1 – $A1_{sTot} = 0.208\ in^2/ft$
Load Condition 2 – $A2_{sTot} = A_{si} + A_{so} = 0.231 + 0.116 = 0.347\ in^2/ft$
Load Condition 3 – $A3_{sTot} = A3_{si} + A3_{so} = 0.290 + 0.176 = 0.466\ in^2/ft$

Load Condition 3 results in the highest required steel areas.
$A_{si} = 0.290\ in^2/ft$

$A_{so} = 0.176\ in^2/ft$

Note: Load Condition 2 is the same load condition used for gravity pipe. Thus, if this were a gravity pipe design the engineer could simply follow the requirements for Load Condition 2 and avoid the extra reinforcement required to accommodate the internal pressure.

Further Note: The steel areas for a 36-inch pipe with 5 feet of cover taken from ASTM C361 are as follows:

With 25 feet of head: $A_{si} = 0.12\ in^2/ft$
$A_{so} = 0.08\ in^2/ft$

With 50 feet of head: $A_{si} = 0.17\ in^2/ft$
$A_{so} = 0.13\ in^2/ft$

The calculated resulting steel areas for a 36-inch pipe, at 5 feet of cover, with 35 feet of head, are higher than these values. Thus, for installations that do not use high quality granular backfill, or that experience live load, the steel reinforcement areas found in ASTM C361 are considered unconservative.

D3. <u>Vitrified Clay Pipe (VCP) Design Example</u>

Given Information: Unit Weight of Embankment Material = 120 lbs/ft^3
Cover Depth from Top Embankment to Top VCP = 30 feet
Transition Width = 100 inches
Bedding Material – CLSM
Bedding Factor = 2.8 for CLSM
Interior Pipe Diameter = 36 inches
Pipe Wall Thickness = 4 inches

Pipe Specification: ASTM C700 VCP with minimum three-edge bearing strength of 6,000 lbs/LF

Find: Using the procedures highlighted in this document and detailed in the latest version of the VCP Design Manual, determine the trench load, field supporting strength of the pipe installation, and the associated safety factor associated with the selected pipe. The effects of any live load are considered negligible given the cover depth exceeds 8 feet.

D3 Step 1. CALCULATE THE ESTIMATED EARTH LOAD ON VCP

Use the Modified Marston Equation for a rigid pipe in an embankment with CLSM side fill to calculate the load on the pipe. For more information in using the Modified Marston Equation for VCP, refer to the latest version of the VCP Design Manual.

$$W_c = C_d \cdot \omega \cdot B_c \cdot B_d$$

$$C_d = \frac{1 - e^{-2K\mu'\left\{\frac{H}{B_d}\right\}}}{2 \cdot K \cdot \mu'}$$

Where:

C_d = load calculation coefficient

W_c = earth load on pipe, lbs/LF

K = ratio of active horizontal pressure at any point in fill to the vertical pressure

μ' = coefficient of sliding friction between fill material and trench sides

H = vertical height from top of pipe to upper surface of fill, feet

ω = unit weight of fill material, lbs/ft^3

B_c = breadth of pipe (interior diameter plus twice the pipe wall thickness), feet

B_d = breadth of ditch measured at the top surface of pipe, feet

The value of C_d is calculated using $K \cdot \mu' = 0.11$ for fine-grained backfill as per the VCP Design Manual.

$$C_d = \frac{1 - e^{-2(0.11)\left\{\frac{30}{\left(\frac{100}{12}\right)}\right\}}}{2(0.11)} = 2.49$$

$$W_c = (2.49) \cdot (120) \cdot (44/12) \cdot (100/12) = 9{,}130 \text{ lbs/LF}$$

D3 Step 2. ***CALCULATE THE FIELD SUPPORTING STRENGTH (FSS)***

The FSS is computed by multiplying the three-edge bearing strength (6,000 lbs/LF) provided in the given information by the bedding factor. The bedding factor for CLSM is 2.8.

$$FSS = (6{,}000 \text{ lbs/LF}) \cdot (2.8) = 16{,}800 \text{ lbs/LF}$$

D3 Step 3. ***CALCLUATE THE SAFETY FACTOR***

The safety factor for this example is calculated by dividing the FSS by the vertical load acting on the pipe as shown in the following equation:

$$\text{Safety Factor} = FSS / W_c$$

$$\text{Safety Factor} = 16{,}800 / 9{,}130 = 1.84$$

As a reminder, the value for W_c (earth load only) was used since the cover depth exceeds 8 feet and any live load effects can be considered negligible at this depth.

D4. Corrugated Aluminum Pipe (CAP) Design Example

Using LRFD procedures, design a 36-inch diameter CAP under 30 feet of a homogeneous embankment with a density of 120 lb/ft^3. The design follows the procedures outlined within this document and ASTM B790. Any live load is neglected since the cover height exceeds 8 feet.

D4 Step 1. ***CALCULATE THE EARTH LOAD ACTING ON THE PIPE***

The earth load pressure (P_{FD}) is calculated by multiplying the unit weight of the fill over the top of the pipe by the height of the soil column.

$$P_{FD} = H \cdot \gamma$$

Where:

P_{FD} = dead load vertical crown pressure from soil prism, lb/ft^2

H = height of the soil column, feet

γ = unit weight of the soil fill, lbs/ft^3

$$P_{FD} = (30 \text{ ft}) \cdot (120 \text{ lb/ft}^3) = 3{,}600 \text{ lb/ft}^2$$

D4 Step 2. CALCULATE THE FACTORED THRUST LOAD IN THE PIPE WALL

The factored thrust in the pipe wall is calculated using Equation 4-24 of this document (AASHTO Equation 12.7.2.2-1) with any live load neglected due to the cover depth being greater than 8 feet.

$$T_L = \gamma_p \left[\frac{P_{FD}\, S}{2}\right] + \gamma_{LL} \left[\frac{P_{FL} C_L F_1}{2}\right]$$

Where:

T_L = factored thrust load per unit length of wall, lb/LF

γ_p = vertical earth pressure factor = 1.95 (see Table 4-3)

S = pipe span (outside diameter of pipe), feet

$$T_L = (1.95) \cdot [(3{,}600 \cdot 3)/2] + 0 = 10{,}530 \text{ lb/LF}$$

D4 Step 3. APPLY LOAD MODIFIER TO FACTORED THRUST LOAD

The factored thrust load is next multiplied by the combined load modifier (η_{cmp}) for ductility, redundancy, and operational importance. As per Section 4.10.3.1.3 of this document, the applicable combined load modifier for corrugated metal pipe (including aluminum) is 1.1. This is referred to as the 'demand' side of equation 4-25 (left side) and represents the 'demand' thrust load which the pipe must be designed to resist.

$$(\eta_{cmp}) \cdot (T_L) = (1.1) \cdot (10{,}530 \text{ lb/LF}) = 11{,}583 \text{ lb/LF}$$

D4 Step 4. SELECT CAP TO MEET DEMAND THRUST LOAD

From ASTM B790, choose an ALCLAD 3004-H32 alloy CAP with helical seams and 2 2/3 x ½ corrugations, a 0.060-inch wall thickness. It has the following engineering properties as per ASTM B790:

$A = 0.775$ in^2/LF	cross-sectional area per linear feet of pipe
$F_y = 20{,}000$ lb/in^2	minimum yield strength as per ASTM B790
$Fu = 27{,}000$ lb/in^2	minimum tensile strength as per ASTM B790
r = 0.1712 inch	radius of gyration
$I = 1.892 \times 10^{-3}$ in^4/in	moment of inertia for cross-section
k = 0.22	soil stiffness factor for good side fill compacted to a minimum of 90% standard density

D4 Step 5. DETERMINE IF CRITICAL BUCKLING STRESS IS LESS THAN F_y

Since the pipe diameter (span length) of 36 inches is less than the value calculated below (73.4 inches), use the f_{cr} formula shown for computing the critical buckling stress for the pipe, as per AASHTO Section 12.7.2.4.

$$\frac{r}{k}\sqrt{\frac{24 \cdot E}{Fu}} = \frac{0.1712}{0.22}\sqrt{\frac{24 \cdot (10{,}000{,}000)}{27{,}000}} = 73.4 \text{ inches} > 36 \text{ inches}$$

$$f_{cr} = Fu - \left(\frac{Fu^2}{48 \cdot E}\right) \cdot \left(\frac{k \cdot s}{r}\right)^2 = 27{,}000 - \left(\frac{27{,}000^2}{48 \cdot 10{,}000{,}000}\right) \cdot \left(\frac{0.22 \cdot 36}{0.1712}\right)^2 = 23{,}750 \text{ lb/in}^2$$

D4 Step 6. CALCULATE THE WALL RESISTSNCE WITH MINIMUM OF F_y OR f_{cr}

Since F_y (20,000 lb/in2) < f_{cr} (23,750 lb/in2), use F_y in computing the wall resistance as per Equation 4-25 of this document. The resistance factor for the wall area (ϕ) is 1.0 since the CAP is not considered a long-span pipe and has no tunnel liner plate.

$$(\eta_{cmp}) \cdot (T_L) < \phi \cdot A \cdot (\text{minimum } F_y, f_{cr})$$

$$11{,}583 \text{ lb/LF} < (1.0) \cdot (0.775 \text{ in}^2\text{/LF}) \cdot (20{,}000 \text{ lb/in}^2)$$

$$11{,}583 \text{ lb/LF} < 15{,}500 \text{ lb/LF} \qquad \text{check is } \underline{\underline{\text{OK}}}$$

The selected CAP (ALCLAD 3004-H32) meets the necessary strength requirements.

D4 Step 7. CHECK PIPE STIFFNESS FOR HANDLING AND INSTALLATION

Next, check that the pipe has sufficient stiffness to withstand the forces applied during shipping and placement. The flexibility factor (FF) is computed using Equation 4-26 of this document.

$$FF = \frac{s^2}{E \cdot I} = \frac{36^2}{(10{,}000{,}000) \cdot (0.001892)} = 0.0685 \text{ in/lb} \qquad \text{for 0.06-inch wall thickness}$$

Since the computed FF value exceeds the allowable (0.031 in/lb) for the given pipe as per ASTM B790, choose a CAP with a thicker wall section. Try 0.075" thick pipe instead. The new FF for the thicker wall section is as follows:

$$FF = \frac{s^2}{E \cdot I} = \frac{36^2}{(10{,}000{,}000) \cdot (0.002392)} = 0.0542 \text{ in/lb} \qquad \text{for 0.075-inch wall thickness}$$

The allowable FF value for the 0.075-inch wall thickness CAP is 0.061 in/lb as per ASTM B790 which exceeds the computed value of 0.0542 in/lb. Therefore, use an ALCLAD 3004-H32 alloy that complies with ASTM B745 and has helical seams and 2 2/3 x ½ corrugations with 0.075-inch wall thickness.

D5. Fiberglass (FRP) Design Example

Design a 36-inch diameter FRP using LRFD procedures given the following situation:

Depth of pipe burial (soil column height) H = 27 feet
Unit weight of the soil (W_s) = 120 lb/ft^3
Poisson's ratio for embankment material (ν) = 0.4
HL-93 live load classification

The following pipe properties are considered applicable for the design:

Nominal inside diameter of FRP (D_i) = 36 inches
Outside diameter of FRP (D_o) = 38.3 inches
Total wall thickness (T_t) = 0.83 inches
Liner thickness (T_L) = 0.04 inches
Structural wall thickness (T_{str}) = $T_t - T_L$ = 0.83 – 0.04 = 0.79 inches
Mean diameter (D_m) = $D_o - T_{str}$ = 38.3 – 0.79 = 37.51 inches
Mean radius (R_m) = 0.5 · D_m = 0.5 · 37.51 = 18.76 inches
Cross-sectional area of pipe wall per unit of length, A_g = 47.04 in^2
Minimum pipe stiffness = 72 lb/in^2
Long-term bending strain (S_b) = 0.0094 in/in (AASHTO 12.15.3.2)

D5 Step 1. CALCULATE PIPE STIFFNESS (PS) WITH 5% DEFLECTION

$$PS = \frac{E_{cf} \cdot I_p}{0.149 \cdot {R_{m5}}^3} = \frac{(1{,}900{,}000) \cdot (0.041)}{(0.149) \cdot (19.23)^3} = 73.52 \text{ psi} > 72 \text{ psi requirement} \quad \underline{\underline{\text{OK}}}$$

Where:

$$I_p = \frac{{T_{str}}^3}{12} = \frac{(0.79)^3}{12} = 0.041 \text{ in}^4/\text{in} \qquad \text{moment of inertia for pipe profile}$$

E_{cf} = 1,900,000 lb/in^2 circumferential flexural modulus (AASHTO 12.15.3.1)

$R_{m5} = R_m \cdot 1.025 = 18.76 \cdot 1.025 = 19.23$ inches (mean radius at 5% deflection)

D5 Step 2. CALCULATE VERTICAL SOIL PRESSURE (P_{sp}) ACTING ON CROWN OF FRP

$$P_{sp} = \frac{H \cdot Ws}{144} = \frac{27 \cdot 120}{144} = 22.50 \text{ psi}$$

D5 Step 3. CALCULATE LIVE LOAD PRESSURE (P_L) ACTING ON FRP

As per AASHTO Section 3.6.1.2.6a, since the height of the fill over the pipe exceeds 8 feet and also exceeds the diameter of the pipe, the effects of live load are considered insignificant for this example; thus, the value for P_L is set equal to 0.

D5 Step 4. CHECK DEFLECTION REQUIREMENT (SERVICE LIMIT STATE)

As per AASHTO Equation 12.15.5.2-1, total deflection is determined as follows:

$$\Delta_t = D_o \cdot K_B \cdot \frac{[(D_L \cdot P_{sp}) + (C_L \cdot P_L)]}{1000 \cdot \left[\left(\frac{E_{cf} \cdot I_p}{R_m{}^3}\right) + 0.061 \cdot M_s\right]}$$

Where:

Δ_t = total deflection under load, inches
D_o = 38.3 inches — outside diameter of pipe
K_B = 0.1 — bedding coefficient as per AASHTO 12.12.2.2
D_L = 1.5 — deflection lag factor as per AASHTO 12.12.2.2
C_L = 1.00 — live load coefficient as per AASHTO 12.12.3.5
M_s = 1.906 kip/in^2 — composite constrained soil modulus (AASHTO Table 12.12.3.5-1) with linear interpolation for Si-95 soil type and compaction level

P_{sp}, P_L, E_{cf}, I_p, and R_m have already been defined and/or calculated previously. For this calculation, the value for E_{cf} should be in kip/in^2.

$$\Delta_t = D_o \cdot K_B \cdot \frac{[(D_L \cdot P_{sp}) + (C_L \cdot P_L)]}{1000 \cdot \left[\left(\frac{E_{cf} \cdot I_p}{R_m{}^3}\right) + 0.061 \cdot M_s\right]} = (38.3) \cdot (0.1) \cdot \frac{[(1.5 \cdot 22.50) + (1.00 \cdot 0)]}{1000 \cdot \left[\left(\frac{1{,}900 \cdot 0.041}{18.76^3}\right) + 0.061 \cdot 1.906\right]}$$

Δ_t = 1.01 inches

Δ_A = 0.05 · 36 inches = 1.8 inches

Since $\Delta_t \leq \Delta_A$ — deflection check (service limit state) is <u>OK</u>

D5 Step 5. CHECK FLEXURE REQUIREMENTS (STRENGTH LIMIT STATE)

The factored long-term flexural strain must not exceed the factored long-term flexural resistance as per AASHTO Section 12.15.6.2.

$\varepsilon_f \leq \varphi_f \cdot S_b$ — flexure requirement

Where:

ε_f = factored long-term strain due to flexure (in/in) as per AASHTO 12.12.3.10.2b-3
φ_f = resistance factor for flexure = 0.9 as per AASHTO Table 12.5.5-1
S_b = long-term ring bending strain (in/in) as per AASHTO Section 12.15.4

$$\varepsilon_f = \gamma_{EV} \cdot \eta_I \cdot D_f \cdot \left(\frac{c}{R_m}\right) \cdot \left(\frac{\Delta_A}{D}\right) = (1.5) \cdot (1.05) \cdot (4.5) \cdot \left(\frac{0.395}{18.76}\right) \cdot \left(\frac{1.8}{37.51}\right) = 0.0072 \text{ in/in}$$

$\varphi_f \cdot S_b = (0.9) \cdot (0.0094) = 0.0085$ in/in   Since $0.0072 \leq 0.0085$ flexure check is OK

D5 Step 6. CHECK GLOBAL BUCKLING (STRENGTH LIMIT STATE)

Similar to the check for flexural strain, the factored global buckling strain must not exceed the factored resistance buckling strain as shown, but there are additional supporting calculations in order to complete this check. Following the guidance in AASHTO Section 12.15.6.3, the calculations are as follows:

$$\varepsilon_{fb} \leq \varphi_{bck} \cdot \varepsilon_{bck} \qquad \text{buckling requirement}$$

Where:

$$\varepsilon_{fb} = \frac{R_m \cdot \left[\left[\frac{\gamma_{EV} \cdot \eta_I \cdot H_w \cdot \gamma_w}{144}\right] + \gamma_{EV} \cdot \eta_I \cdot P_{sp} \cdot R_w + \gamma_{LL} \cdot \eta_{LL} \cdot C_L \cdot P_L\right]}{1000 \cdot E_{cf} \cdot A_g}$$

$R_w = 1 - (0.33) \cdot (H_w / H) = 1 - (0.33) \cdot (0/27) = 1.0$ applicable only if $0 \leq H_w \leq H$

Where:

γ_w = unit weight of water (lb/ft^3) = 62.4 lb/ft^3
γ_{LL} = load factor for live load as per AASTO 12.5.4
E_{cf} = 1,900 kip/in^2 for this calculation

All other terms previously defined and/or calculated

$$\varepsilon_{fb} = \frac{18.76 \cdot [0 + (1.5 \cdot 1.05 \cdot 22.5 \cdot 1.0) + 0]}{1000 \cdot 1900 \cdot 47.04} = 0.0000074 \text{ in/in}$$

The nominal strain capacity for general buckling (ε_{bck}) is calculated using AASHTO Equation 12.12.3.10.1e-2 with modifications for fiberglass pipe as outlined in AASHTO Section 12.15.6.3 as follows:

$$\varepsilon_{bck} = \frac{1.2 \cdot C_n \cdot (E_{cf} \cdot I_p)^{1/3}}{A_g \cdot E_{cf}} \cdot \left[\frac{\varphi_s \cdot M_s \cdot (1 - 2\nu)}{(1-\nu)^2}\right]^{2/3} \cdot R_h$$

Where:

$$R_h = \frac{11.4}{11 + \frac{D}{12 \cdot H_E}} = \frac{11.4}{11 + \left(\frac{38.3 - 0.79/2}{12 \cdot 27}\right)} = 1.0255$$

$$\varepsilon_{bck} = \frac{1.2 \cdot 0.55 \cdot (1900 \cdot 0.041)^{1/3}}{47.04 \cdot 1900} \cdot \left[\frac{0.9 \cdot 1.906 \cdot (1 - 2 \cdot 0.4)}{(1 - 0.4)^2}\right]^{\frac{2}{3}} \cdot 1.0255 = 0.0000313 \text{ in/in}$$

$$\varepsilon_{fb} \leq \varphi_{bck} \cdot \varepsilon_{bck}$$

$$0.0000074 \text{ in/in} \leq (0.63) \cdot (0.0000313)$$

0.0000074 in/in ≤ 0.0000197 in/in global buckling check is OK

D5 Step 7. CHECK FLEXIBILITY LIMIT FOR HANDLING AND INSTALLATION

This check is completed by calculating the flexibility factor (FF) for the pipe and then ensuring it is less than the flexibility limit (FL). This is done per AASHTO Section 12.15.6.4.

$$FF = \frac{{D_m}^2}{E_{cf} \cdot I_p} = \frac{37.51^2}{1{,}900 \cdot 0.041} = 18.06 \text{ in/kip}$$

FL = 95.0 in/kip as per AASHTO Section 12.5.6.3

Since 18.06 < 95.0 flexibility limit check is OK

D6. Polypropylene Pipe (Thermoplastic) Design Example

Embankment information:

- Depth of embankment fill over top of pipe (H_E) = 30 feet
- Depth of water above springline of pipe (H_w) = 0 feet
- Embankment unit weight (γ_s) = 120 lb/ft^3
- Poisson's ratio for embankment material (ν) = 0.4
- Constrained soil modulus (M_s) = 25 kip/in^2

Pipe geometric design properties (36-inch nominal diameter):

- Outside diameter (D_o) = 41.43 inches = 3.4525 feet
- Inside diameter (D_i) = 36.05 inches
- Diameter to centroid (D) = 38.81 inches
- Radius to centroid of pipe wall (R) = D/2 = 19.405 inches
- Gross area of pipe wall (A_g) = 0.529 in^2

Effective area of pipe (A_{eff}) = 0.357 in^2
Moment of inertia (I_p) = 0.606 in^4

Pipe material design properties:
Short-term modulus of elasticity (E_{ps}) = 175 kip/in^2
Long-term modulus of elasticity (E_{pl}) = 28 kip/in^2
Tensile limiting strain for combined strain (ε_{yt}) = 0.025 in/in
Factored compressive strain limit (ε_{yc}) = 0.037 in/in

Miscellaneous factors:
Plate buckling coefficient (k) = 4.0
Bedding coefficient (K_b) = 0.1
Deflection lag factor (D_L) = 1.5
Coefficient for thrust variation at the crown ($K2_c$) = 0.6
Coefficient for thrust variation at the springline ($K2_s$) = 1.0

Applicable load factors and modifiers:
Earth load factor (γ_{EV}) = 1.3
Live load factor (γ_{LL}) = 1.75
Combined redundancy and importance load modifier (η_{ri}) = (1.05) · (1.05) = 1.1
Installation factor (K_E) = 1.5

Resistance factors:
Thurst (φ_T) = 1.0
Soil stiffness (φ_s) 0.9
Global buckling (φ_{bck}) = 0.67
Flexure (φ_f) = 1.0

D6 Step 1. CALCULATE PIPE STIFFNESS (PS)

$$PS = \frac{E_{ps} \cdot I_p}{0.149 \cdot R_o^3} = \frac{175{,}000 \cdot 0.606}{0.149 \cdot 19.405^3} = 97.44 \text{ lb/in}^2$$

D6 Step 2. DETERMINE LOADS ACTING ON PIPE

D6 Step 2.1 SOIL PRISM PRESSURE

$$P_{sp} = (H_E + 0.11 \cdot D_o) \cdot \gamma_s \qquad \text{since } H_w < 0.5 \cdot D_o$$

$$P_{sp} = (30 + 0.11 \cdot (3.4525)) \cdot (120) = 3{,}645.57 \text{ lb/ft}^2 = 25.31 \text{ lb/in}^2$$

D6 Step 2.2 HYDROSTATIC WATER PRESSURE

$P_w = 0$ since the water level is below the springline of the pipe

D6 Step 2.3 LIVE LOAD

Since the crown of the pipe is more than 8 feet below the top of the embankment, then the effects of the live loads are considered insignificant for this example.

D6 Step 3. CALCULATE THRUST IN PIPE

D6 Step 3.1 HOOP STIFFNES FACTOR (S_H)

$$S_H = \frac{\varphi_S \cdot M_S \cdot R}{E_{pl} \cdot A_g} = \frac{0.9 \cdot 25{,}000 \cdot 19.405}{28{,}000 \cdot 0.529} = 29.45$$

D6 Step 3.2 VERTICAL ARCHING FACTOR (VAF)

$$VAF = 0.76 - 0.71 \cdot \left(\frac{S_H - 1.17}{S_H + 2.92}\right) = 0.76 - 0.71 \cdot \left(\frac{29.45 - 1.17}{29.45 + 2.92}\right) = 0.14$$

D6 Step 3.3. CALCULATE SERVICE THRUST LOADS

The service thrust load in the pipe wall at the crown (TSC) is computed using the following equation:

$$T_{SC} = (K_{2c} \cdot VAF \cdot P_{sp} + P_L \cdot C_L \cdot F_1 \cdot F_2) \cdot \frac{D_o}{2}$$

$$T_{SC} = (0.6 \cdot 0.14 \cdot 25.31 + 0) \cdot \frac{41.43}{2} = 44.04 \text{ lb/in}$$

The service thrust load in the pipe wall at the springline (T_{SS}) is computed with the same equation as T_{SC} except that $K_{2s} = 1.0$ is used to replace K_{2c} as follows:

$$T_{SS} = (1.0 \cdot 0.14 \cdot 25.31 + 0) \cdot \frac{41.43}{2} = 73.40 \text{ lb/in}$$

D6 Step 3.4 CALCULATE FACTORED THRUST LOADS

The factored thrust load in the pipe wall at the crown (T_{UC}) is computed as follows:

$$T_{UC} = [\eta_{ri} \cdot (\gamma_{EV} \cdot K_E \cdot K_{2c} \cdot VAF \cdot P_{sp} + \gamma_w \cdot P_w) + \eta_{LL} \cdot \gamma_{LL} \cdot P_L \cdot C_L \cdot F_1 \cdot F_2] \cdot \frac{D_o}{2}$$

$$T_{UC} = [1.1 \cdot (1.3 \cdot 1.5 \cdot 0.6 \cdot 0.14 \cdot 25.31 + 0) + 0] \cdot \frac{41.43}{2} = 94.47 \text{ lb/in}$$

The factored thrust load in the pipe wall at the springline (T_{US}) is computed with the same equation as the crown (T_{UC}), but $K_{2s} = 1.0$ replaces K_{2c} in the equation as follows:

$$T_{US} = [1.1 \cdot (1.3 \cdot 1.5 \cdot 1.0 \cdot 0.14 \cdot 25.31 + 0) + 0] \cdot \frac{41.43}{2} = 157.45 \text{ lb/in}$$

D6 Step 4. CHECK THURST STRAIN LIMIT STATE

D6 Step 4.1 CALCULATE SERVICE THRUST STRAIN

The service compressive strain at the crown of the pipe due to thrust (ε_{SC}) is calculated using AASHTO Equation 12.12.3.10.1c-2 as follows:

$$\varepsilon_{SC} = \frac{T_{SC}}{1000 \cdot A_{eff} \cdot E_{pl}} = \frac{44.04}{1000 \cdot 0.357 \cdot 28} = 0.00441 \text{ in/in}$$

The service compressive strain at the springline of the pipe due to thrust (ε_{SS}) is calculated using the same equation as follows:

$$\varepsilon_{SS} = \frac{T_{SS}}{1000 \cdot A_{eff} \cdot E_{pl}} = \frac{73.40}{1000 \cdot 0.357 \cdot 28} = 0.00734 \text{ in/in}$$

D6 Step 4.2 CALCULATE FACTORED THRUST STRAIN

The factored compressive strain at the crown of the pipe due to thrust (ε_{UC}) is calculated using AASHTO Equation 12.12.3.10.1c-1 as follows:

$$\varepsilon_{UC} = \frac{T_{UC}}{1000 \cdot A_{eff} \cdot E_{pl}} = \frac{94.47}{1000 \cdot 0.357 \cdot 28} = 0.00945 \text{ in/in}$$

The factored compressive strain at the springline of the pipe due to thrust (ε_{US}) is calculated using the same equation as follows:

$$\varepsilon_{US} = \frac{T_{US}}{1000 \cdot A_{eff} \cdot E_{pl}} = \frac{157.45}{1000 \cdot 0.357 \cdot 28} = 0.01575 \text{ in/in}$$

D6 Step 4.3 CHECK FACTORED THRUST STRAIN AGAINST STRAIN LIMIT

The maximum factored thrust strain (ε_{US}) calculated in the previous sub-step must not exceed the factored compression strain limit for the pipe wall material as per AASHTO Equation 12.12.3.10.1d-1 as follows:

$$\varepsilon_{US} \leq \varphi_T \cdot \varepsilon_{yc}$$

0.01575 in/in ≤ (1.0)·(0.037 in/in) = 0.037 in/in check is OK for thrust strain limit

D6 Step 5. CHECK GLOBAL BUCKLING STRAIN LIMIT STATE

D6 Step 5.1 DETERMINE THE CORRECTION FACTOR (R_h) FOR BACKFILL

Using AASHTO Equation 12.12.3.10.1e-3, determine the value for R_h for input into buckling equation.

$$R_h = \frac{11.4}{11+\frac{D}{12 \cdot H_E}} = \frac{11.4}{11+\left(\frac{38.81}{12 \cdot 30}\right)} = 1.0263$$

D6 Step 5.2 CALCULATE THE GLOBAL BUCKLING STRAIN CAPACITY

The global buckling strain capacity (ε_{bck}) is computed using AASHTO Equation 12.12.3.10.1e-2 as follows:

$$\varepsilon_{bck} = \frac{1.2 \cdot C_n \cdot \left(E_{pl} \cdot I_p\right)^{1/3}}{A_{eff} \cdot E_{pl}} \cdot \left[\frac{\varphi_s \cdot M_s \cdot (1-2\nu)}{(1-\nu)^2}\right]^{2/3} \cdot R_h$$

$$\varepsilon_{bck} = \frac{1.2 \cdot 0.55 \cdot (28 \cdot 0.606)^{1/3}}{0.357 \cdot 28} \cdot \left[\frac{0.9 \cdot 25 \cdot (1-2 \cdot 0.4)}{(1-0.4)^2}\right]^{\frac{2}{3}} \cdot 1.0263 = 0.939 \text{ in/in}$$

D6 Step 5.3 COMPARE FACTORED GLOBAL BUCKLING STRAIN CAPACITY vs. FACTORED GLOBAL STRAIN LOADING

The factored global buckling strain loading (ε_{UC}) must not exceed the factored global buckling strain capacity as per AASHTO Equation 12.12.3.10.1e-1 as follows:

$$\varepsilon_{UC} \leq \varphi_{bck} \cdot \varepsilon_{bck}$$

0.00945 in/in ≤ (0.7)·(0.939 in/in) = 0.6573 in/in global buckling strain check is OK

D6 Step 6. CHECK COMBINED BENDING AND THRUST LIMITS

D6 Step 6.1 DETERMINE DEFLECTION CRITERIA

The maximum allowable deflection (Δ_A) is 5 percent of the inside diameter of the pipe which is computed as follows:

$$\Delta_A = (0.05)\cdot(36.05 \text{ inches}) = 1.8025 \text{ inches}$$

The allowable deflection due to bending deformation (Δ_f) is computed using AASHTO Equation 12.12.3.10.2b-4 as follows:

$$\Delta_f = \Delta_A - (\varepsilon_{SS})\cdot(D) = 1.8025 - (0.00734)\cdot(38.81 \text{ inches}) = 1.576 \text{ inches}$$

D6 Step 6.2 CALCULATE THE FACTORED FLEXURAL STRAIN

The initial step is to determine the shape factor (D_f) by linear interpolation or extrapolation of AASHTO Table 12.12.3.10.2b-1 for the calculated pipe stiffness (calculated in D6 Step 1) coupled with the pipe embedment material and compaction level. Using linear extrapolation, a value of 2.8 for the shape factor is estimated for this pipe and embedment scenario.

The next step is to calculate the factored flexural strain (ε_f) using AASHTO Equation 12.12.3.10.2b-3 as follows:

$$\varepsilon_f = \gamma_{EV} \cdot D_f \cdot \left(\frac{c}{R}\right) \cdot \left(\frac{\Delta_f}{D}\right) = 1.3 \cdot 2.8 \cdot \left(\frac{1.407}{19.405}\right) \cdot \left(\frac{1.576}{38.81}\right) = 0.01072 \text{ in/in}$$

Where:

c = distance from neutral axis to inner or outer most fiber (maximum)

D6 Step 6.3 CHECK THE COMBINED STRAIN FOR TENSION AND COMPRESSION

The combined axial and bending strain where flexure causes tension must satisfy AASHTO Equation 12.12.3.10.2b-1 as follows:

$$\varepsilon_f - \min(\varepsilon_{UC}, \varepsilon_{US}) \le \varphi_f \cdot \varepsilon_{yt}$$

$$0.01072 - 0.00945 \le (1.0)\cdot(0.025)$$

$0.00127 \le 0.025$ combined strain check for tension is <u>OK</u>

The combined axial and bending strain where flexure causes compression must satisfy AASHTO Equation 12.12.3.10.2b-2 as follows:

$$\varepsilon_f + \max(\varepsilon_{UC}, \varepsilon_{US}) \le \varphi_T \cdot (1.5 \cdot \varepsilon_{yc})$$

$$0.01072 + 0.01575 \le (1.0)\cdot(1.5 \cdot 0.037)$$

$0.02647 \le 0.0555$ combined strain check for compression is <u>OK</u>

D6 Step 7. CHECK DEFLECTION CRITERIA

The total deflection (Δ_t) due to loading must not exceed the allowable deflection (Δ_A) for the pipe. The total deflection due to loading is calculated using AASHTO Equation 12.12.2.2-2 as follows:

$$\Delta_t = \left[\frac{K_B \cdot (D_L P_{sp} + C_L P_L) D_o}{1000\left[\frac{E_{pl} I_p}{R^3} + 0.061 M_s\right]}\right] + \max(\varepsilon_{SC}, \varepsilon_{SS}) \cdot D_o$$

$$\Delta_t = \left[\frac{0.1 \cdot (1.5 \cdot 25.31 + 0) \cdot 41.43}{1000 \cdot \left[\frac{28 \cdot 0.606}{19.405^3} + (0.061 \cdot 25)\right]}\right] + (0.00734) \cdot (41.43) = 0.407 \text{ inches}$$

Next, compare the total deflection due to loading against the allowable deflection computed previously in order to satisfy AASHTO Equation 12.12.2.2-1 as follows:

$$\Delta_t \leq \Delta_A$$

0.407 inches ≤ 1.8025 inches deflection check is <u>OK</u>

D6 Step 8. HANDLING AND INSTALLATION REQUIREMENTS

The calculation for the handling and installation requirements involves computing the flexibility factor (FF) for the selected pipe and comparing against the allowable limit (FL = 95 in/kip). FF is computed using AASHTO Equation 12.12.3.6-1 as follows:

$$FF = \frac{{D_i}^2}{E_{ps} \cdot I_p} = \frac{36.05^2}{175 \cdot 0.606} = 12.25 \text{ in/kip}$$

FL = 95.0 in/kip as per AASHTO Section 12.5.6.3

Since 18.06 < 95.0 flexibility limit check is <u>OK</u>

D7. <u>Welded Seam Steel Pipe (WSSP) Design Example</u>

Design Requirements:

- Design will use allowable stress design (ASD) philosophy
- Pipe has an inside diameter of 48” with 0.5” thick cement-mortar lining and tape coating
- Water will be pumped through the pipe and over a levee embankment at 100 psi working pressure with a 20 psi surge allowance.

- Pipe will be buried 12 feet at the deepest point and 3 feet at the crown of the levee embankment. There is an access road on the crown of the levee embankment that allows vehicular access.
- The levee embankment consists of a compacted granular material to 90% standard proctor. The unit weight of the soil is 120 lbs/ft^3.

D7 Step 1. SELECT AN EXISTING PIPE TO MEET INITIAL REQUIREMENTS

In order to meet the 48-inch inside diameter requirement, we will select a WSSP with a 49.5-inch outside diameter made of A36 carbon steel with a minimum yield strength of 36,000 psi. This type of pipe is manufactured such that it is readily available and high quality.

D7 Step 2. DETERMING THE REQUIRED WSSP WALL THICKNESS

The allowable stress for working pressure is 50% of the minimum yield strength or 18,000 psi. Surge pressure is a short-term loading, thus, the allowable stress for surge pressure load case is 75% of minimum yield strength or 27,000 psi. The load case which requires the greater pipe wall thickness will be selected.

Equation 4-27 of this manual is used to calculate the wall thickness for both load cases as follows:

$$t = (P \cdot D_o) / (2 \cdot S)$$

For working pressure: $t_w = (100 \cdot 49.5) / (2 \cdot 18{,}000) = 0.1375$ inch

For surge pressure: $t_s = (120 \cdot 49.5) / (2 \cdot 27{,}000) = 0.11$ inch

The working pressure load case controls; therefore, move forward with a minimum wall thickness of 0.1375 inch.

D7 Step 3. CHECK WSSP FOR HANDLING REQUIREMENTS

Since the pipe has a cement-mortar lining, Equation 4-30 of this manual is used to determine if the 0.1375 wall thickness is sufficient where D_n is the nominal diameter of the pipe.

$$t_h = D_n / 240 = 48/240 = 0.2 \text{ inch}$$

Since 0.2” > 0.1375”, move forward with 0.2 inch wall thickness and check deflection criteria.

D7 Step 4. CALCULATE EXTERNAL LOADS ACTING ON WSSP

The deflection of the pipe is determined by calculation the loading acting on the pipe and then determining if it results in deflection that exceeds the allowable amount. Use Equation 4-31 of this manual to estimate the loading from the soil column above the pipe for both the maximum and minimum depths.

$W_c = (\omega \cdot H_c \cdot D_o)/12$

$W_c = (120 \cdot 12 \cdot 49.5)/12 = 5{,}940$ lb/in at 12 feet of cover depth

$W_c = (\omega \cdot H_c \cdot D_o)/12$

$W_c = (120 \cdot 3 \cdot 49.5)/12 = 1{,}485$ lb/in at 3 feet of cover depth

Next, add the live load using HS-20 loading criteria. For the scenario with 12 feet of depth, the effects of the live load are negligent as per Section 4.6.4.2.3 of this manual. For the burial depth of 3 feet at the crown of the levee embankment where the access road is located, the effects of vehicular traffic must be included along with the soil prism load with 3 feet of cover. Using Table 5-1 of AWWA M11, the live load for 3 feet of cover for HS-20 loading is 600 lb/ft^2. The load per unit of length of pipe for both the cover depths is as follows:

$$W_{12} = \frac{W_c}{12} + \frac{W_L \cdot D_o}{144} = \frac{5{,}940}{12} + 0 = 495 \text{ lb/in}$$

$$W_3 = \frac{W_c}{12} + \frac{W_L \cdot D_o}{144} = \frac{1{,}485}{12} + \frac{600 \cdot 49.5}{144} = 330 \text{ lb/in}$$

D7 Step 5. CALCUATE PREDICTED DEFLECTION

The predicted deflection is calculated using Equation 5-4 of AWWA M11. In its basic form, it represents the load divided by the summation of the pipe stiffness and soil stiffness. This must be calculated for both the minimum and maximum cover depths for this example.

$$\Delta x = D_l \cdot \frac{K \cdot W \cdot r^3}{EI + 0.061 \cdot E' \cdot r^3}$$

Where:

$D_l = 1.0$ (deflection lag factor set equal to 1.0 for soil prism load methodology)
$K = 0.1$ (bedding constant)
W = load per unit length of pipe from D7 Step 4
$r = (D_c - t - t_l - t_c)/2 = (49.5 - 0.2 - 0.5)/2 = 24.4$ in (mean radius of the pipe)
$E_s = 30{,}000{,}000$ psi modulus of elasticity for steel
$E_c = 4{,}000{,}000$ psi modulus of elasticity for cement-mortar lining
E' = soil stiffness for pipe embedment, lb/in^2 (Table 5-3, AWWA M11)
$I_s = t^3/12 = (0.2)^3/12 = 0.000667$ in^4 (moment of inertia of the pipe wall)
$I_L = (0.5)^3/12 = 0.0104$ in^4 (moment of inertial of the cement-mortar lining)
$EI = E_s I_s + E_c I_L = (30{,}000{,}000) \cdot (0.000667) + (4{,}000{,}000) \cdot (0.0104) = 61{,}700$ lb-in^2

$$\Delta x3 = 1.0 \cdot \frac{0.1 \cdot 330 \cdot (24.4)^3}{61{,}700 + 0.061 \cdot 1{,}000 \cdot (24.4)^3} = 0.506 \text{ inch} \quad \text{(3-ft cover depth deflection)}$$

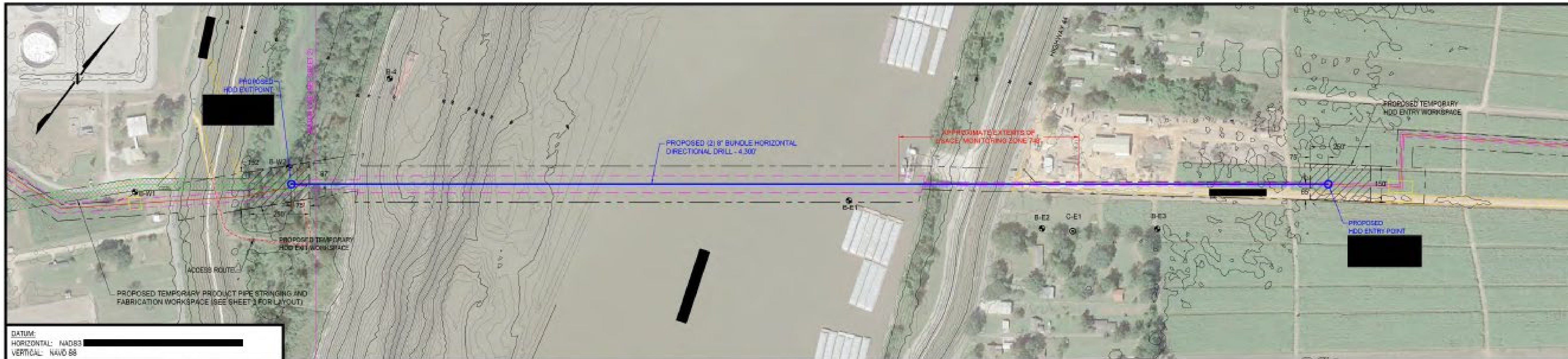
$$\Delta x12 = 1.0 \cdot \frac{0.1 \cdot 495 \cdot (24.4)^3}{61{,}700 + 0.061 \cdot 1{,}600 \cdot (24.4)^3} = 0.486 \text{ inch} \quad \text{(12-ft cover depth deflection)}$$

Predicted deflection of 0.506 inch controls. As per Section 4.11.3.5, the maximum allowable deflection for WSSP with cement-mortar lining with flexible coating is 3% of the pipe diameter, which equals (0.03)·(48 inches) = 1.44 inches. Since 0.506 < 1.44, a 49.5-inch outside diameter WSSP with a minimum wall thickness of 0.2 inches and made of A36 carbon steel will satisfy this design.

THIS PAGE INTENTIONALLY LEFT BLANK

Appendix E
Trenchless Installations (Example Hydraulic Fracture Evaluation and Annual Grouting Plan)

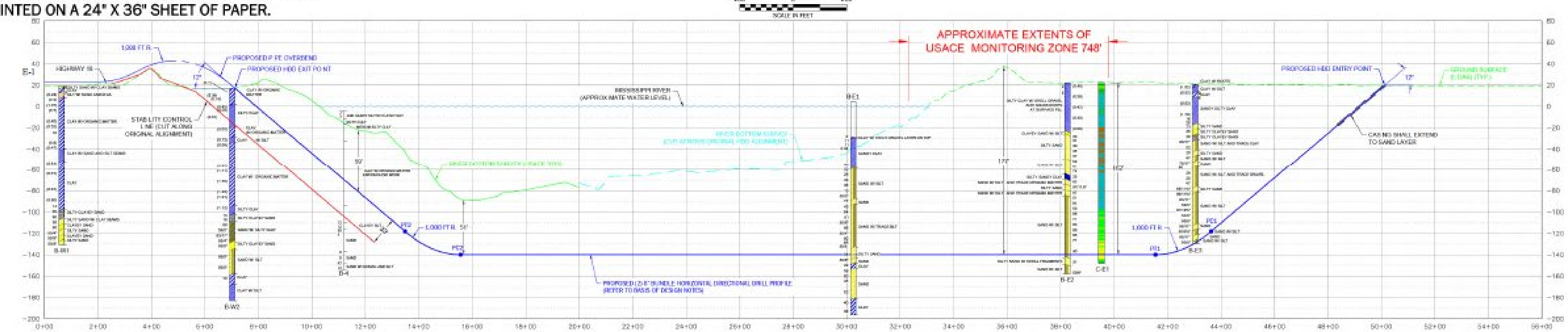
Appendix E – HDD Design Drawings



NOTE: THIS IS A FULL SIZE DRAWING THAT IS INTENDED TO BE PRINTED ON A 24" X 36" SHEET OF PAPER.

PLAN

SCALE IN FEET



DIRECTIONAL DRILL DATA [redacted] RIVER HDD		
DESCRIPTION	STATION * (FT)	ELEVATION (FT)
ENTRY @ 12°	50+12.50	19.80
P.C.1 (12.00° @ 1,000 FT R.)	43+63.03	-118.15
P.T.1	41+55.12	-140.00
P.C.2 (12.00° @ 1,000 FT R.)	15+56.43	-140.00
P.T.2	13+48.52	-118.15
EXIT @ 12°	7+12.50	17.04
HORIZONTAL DISTANCE – 4,300.00 FT		
DIRECTIONAL DRILL PIPE LENGTH – 4,331.77 FT		

* THE STATIONING IS BASED ON AN ARBITRARY REFERENCE POINT

RECOMMENDED TOLERANCES	
ITEM	TOLERANCE
PILOT HOLE ENTRY ANGLE	INCREASE ANGLE UP TO 1° (STEEPER), BUT NO DECREASE IN ANGLE ALLOWED.
PILOT HOLE ENTRY LOCATION	AS PER COORDINATES PROVIDED BY COMPANY WITH NO CHANGES WITHOUT COMPANY APPROVAL.
PILOT HOLE EXIT ANGLE	INCREASE ANGLE UP TO 1° (STEEPER) OR DECREASE UP TO 2° (FLATTER).
PILOT HOLE EXIT LOCATION	UP TO 20 FEET BEYOND OR 10 FEET SHORT OF THE EXIT STAKE. BETWEEN 5 FEET LEFT AND 5 FEET RIGHT OF CENTERLINE.
PILOT HOLE DEPTH	UP TO 2 FEET ABOVE THE DESIGN DRILL PROFILE ALLOWED. UP TO 10 FEET BELOW THE DESIGN DRILL PROFILE ALLOWED.
PILOT HOLE ALIGNMENT	SHALL REMAIN WITHIN 5 FEET LEFT OR RIGHT OF THE HDD ALIGNMENT.

LEGEND

SPT (N) (SHEAR STRENGTH KSF)

TYPE OF SOIL

BORING LOCATION

CPT LOCATION

MAJOR CONTOUR - 10' INTERVAL

MINOR CONTOUR - 2' INTERVAL

PROFILE

HORIZONTAL SCALE IN FEET

VERTICAL SCALE IN FEET

CPT LEGEND:

1 - SENSITIVE, FINE GRAINED SOILS
2 - ORGANIC SOILS, PEATS
3 - CLAYS-CLAY TO SILTY CLAY
4 - SILT MIXTURES-CLAY SILT TO SILTY CLAY
5 - SAND MIXTURES-SILTY SAND TO SANDY SILT
6 - SANDS-CLEAN SAND TO SILTY SAND
7 - GRAVELLY SAND TO SAND
8 - VERY STIFF CLAY TO CLAYEY SAND
9 - VERY STIFF FINE GRAINED SOILS

[redacted]

BASIS OF DESIGN:

1. PRODUCT PIPES WILL CONSIST OF TWO (2) 8.625" O.D. X 0.5" W.T., API 5L X-60 PIPE WITH 10-12 MILS OF FUSION BONDED EPOXY (FBE) AND A MINIMUM OF 40 MILS OF ABRASION RESISTANT OVERLAY (ARO).
2. THE MAXIMUM ALLOWABLE OPERATING PRESSURE (MAOP) = 1,440 psi.
3. THE ASSUMED MAXIMUM OPERATING TEMPERATURE = 85° FAHRENHEIT.
4. THE MINIMUM ALLOWABLE THREE JOINT RADIUS SHALL NOT BE LESS THAN 500 FEET.

NOTES:

1. CONTRACTOR SHALL ADHERE TO THE SPECIFICATIONS AND REQUIREMENTS PER [redacted] SPECIFICATIONS, CONTRACT DOCUMENTS AND SPECIAL PERMIT CONDITIONS, EXCEPT AS NOTED ON THIS DRAWING.
2. CONTRACTOR IS RESPONSIBLE FOR CALLING [redacted] AND LOCATING ALL UNDERGROUND UTILITIES PRIOR TO BEGINNING CONSTRUCTION. IF ANY UTILITY IS LOCATED WITHIN 15 FEET OF THE DESIGNED HDD PROFILE AND ALIGNMENT, CONTRACTOR SHALL OBTAIN APPROVAL FROM [redacted] PRIOR TO INITIATING HDD OPERATIONS.
3. IT IS THE CONTRACTOR'S RESPONSIBILITY TO IDENTIFY AND PROTECT ANY FOREIGN UTILITY THAT MAY BE AFFECTED BY THE HDD OPERATIONS.
4. PLACEMENT OF THE HDD RIG IS NOT FIXED BY THE DESIGNATION OF THE ENTRY AND EXIT POINTS. THE USE OF DUAL HDD RIGS DURING CONSTRUCTION MAY BE AT THE DISCRETION OF THE HDD CONTRACTOR, TO BE APPROVED BY THE PROJECT TEAM.
5. ALL EQUIPMENT MUST ACCESS THE SITE ALONG THE CONSTRUCTION RIGHT-OF-WAY OR FROM APPROVED ACCESS ROADS.
6. WORKSPACE: MAXIMUM WORKSPACE LIMITS ARE DEPICTED. RESTRICT CLEARING TO THE WORKSPACE INDICATED AT THE ENTRY AND EXIT POINTS AND PRODUCT PIPE STRINGING AND FABRICATION WORKSPACE ALONG THE CONSTRUCTION RIGHT-OF-WAY. CLEARING BETWEEN THE ENTRY AND EXIT POINTS REQUIRES PRIOR APPROVAL FROM THE ENVIRONMENTAL INSPECTOR AND IS LIMITED TO THE AMOUNT NECESSARY TO STRING SURVEY WIRES AND INSTALL PUMPS AND PIPING TO OBTAIN WATER (WHERE APPROVED).
7. WATER SOURCE: DRILL WATER AND HYDROSTATIC TEST WATER SHALL BE OBTAINED FROM AN APPROVED SOURCE.
8. HYDROSTATIC TEST: PRE-INSTALLATION AND POST-INSTALLATION HYDROSTATIC TESTS SHALL BE CONDUCTED IN ACCORDANCE WITH THE HYDROSTATIC TEST PLAN. TEST WATER SHALL BE SAMPLED AND TESTED IN ACCORDANCE WITH PERMIT REQUIREMENTS. THE TEST WATER SHALL BE DISCHARGED IN AN UPLAND AREA INTO AN EROSION CONTROL STRUCTURE OF STRAW BALES AND/OR SILT FENCES, GEOTEXTILE FILTER BAG, OR COLLECTED IN A TRUCK AND HAULED TO AN APPROVED DISPOSAL SITE. UPON COMPLETION OF DEWATERING AND DRYING, A CALIPER PIG SURVEY SHALL BE COMPLETED IN ACCORDANCE WITH THE CONTRACT DOCUMENTS.
9. SPILL-PREVENTION: REFUELING OF ALL EQUIPMENT SHALL BE COMPLETED IN ACCORDANCE WITH THE SPCC PLAN.
10. EROSION AND SEDIMENT CONTROL: CONTRACTOR SHALL SUPPLY, INSTALL AND MAINTAIN SEDIMENT CONTROL STRUCTURES IN ACCORDANCE WITH CONTRACT DOCUMENTS. CONTRACTOR SHALL INSTALL ADDITIONAL EROSION CONTROL STRUCTURES AS DIRECTED BY THE ENVIRONMENTAL INSPECTOR.
11. INSTALLATION: THE PIPE SECTION FOR THE DRILLED CROSSING SHALL BE MADE UP WITHIN THE APPROVED CONSTRUCTION RIGHT-OF-WAY AT THE DRILL EXIT POINT AS SHOWN. AFTER THE PILOT HOLE IS COMPLETE, CONTRACTOR'S ACTUAL DRILL PROFILE SHALL BE SUBMITTED TO [redacted] FOR APPROVAL. CONTRACTOR SHALL ASSESS THE NEED FOR AND SUPPLY APPROPRIATE BALLAST DURING PULLBACK.
12. DRILLING FLUID DISPOSAL: CONTRACTOR SHALL DISPOSE OF EXCESS DRILLING FLUID AS DIRECTED BY THE COMPANY REPRESENTATIVE IN ACCORDANCE WITH PERMIT CONDITIONS. UNDER NO CIRCUMSTANCES SHALL DRILLING FLUID BE DISPOSED OF IN WATER BODIES OR WETLANDS. ANY DRILLING FLUID WHICH INADVERTENTLY SURFACES AT POINTS OTHER THAN THE ENTRY OR EXIT POINTS SHALL BE CONTAINED AND COLLECTED TO THE EXTENT PRACTICAL AND DISPOSED OF AS DIRECTED BY THE COMPANY REPRESENTATIVE IN ACCORDANCE WITH PERMIT CONDITIONS.
13. CLEANUP/STABILIZATION/RESTORATION: ALL DISTURBED AREAS SHALL BE RETURNED TO THE ORIGINAL CONTOURS. DISTURBED AREAS SHALL BE SEEDED AS SPECIFIED IN THE CLEAN-UP AND RESTORATION REQUIREMENTS. IF THE TERRAIN ALLOWS AND ACCESS IS PERMITTED, CONTRACTOR SHALL UTILIZE LOW GROUND PRESSURE EQUIPMENT OR OTHER EQUIPMENT APPROVED BY OWNER, TO FACILITATE CONTAINMENT AND CLEAN-UP OF ANY INADVERTENT RETURNS THAT OCCUR DURING THE HDD INSTALLATION PROCESS.
14. GEOTECHNICAL DATA: BORE HOLES ARE OFFSET FROM THE PIPELINE CENTERLINE AS SHOWN ON THE PLAN VIEW. THE GEOTECHNICAL INFORMATION PROVIDED ON THIS DRAWING IS A GENERAL SUMMARY. REFER TO THE APPLICABLE GEOTECHNICAL REPORT IN THE CONTRACT DOCUMENTS FOR MORE DETAILED INFORMATION [redacted]
15. LIDAR DOWNLOADED FROM [redacted]
16. BASE FILES, SURVEY DATA, AND AERIAL IMAGE PROVIDED BY [redacted]

REFERENCES	
DRAWING NUMBER	REFERENCE DRAWING TITLE
[redacted]	[redacted]

REVISIONS					
NO.	DESCRIPTION	BY	DATE	CHK'D	APP'D
[redacted]	[redacted]	[redacted]			

[redacted]

Design [redacted]	Date [redacted]
Drawn [redacted]	Date [redacted]
Checked [redacted]	Date [redacted]
Approved [redacted]	Date [redacted]

[redacted]

PLAN AND PROFILE

[redacted]

Project No. [redacted]

Drawing No. -

Sheet 1 of 2

DATUM:
HORIZONTAL: NAD83 [redacted]
VERTICAL: NAVD88

NOTE: THIS IS A FULL SIZE DRAWING THAT IS INTENDED TO BE PRINTED ON A 24" X 36" SHEET OF PAPER.

PLAN

200 0 200
SCALE IN FEET

NOTES:

1. ALL EQUIPMENT MUST ACCESS THE SITE ALONG THE CONSTRUCTION RIGHT-OF-WAY OR FROM APPROVED ACCESS ROADS.
2. LIDAR DOWNLOADED FROM [redacted]
3. BASE FILES, SURVEY DATA, AND AERIAL IMAGE PROVIDED BY [redacted]

[redacted]

LEGEND
- BORING LOCATION
- MAJOR CONTOUR - 10' INTERVAL
- MINOR CONTOUR - 2' INTERVAL

REFERENCES	
DRAWING NUMBER	REFERENCE DRAWING TITLE
[redacted]	[redacted]

REVISIONS					
NO.	DESCRIPTION	BY	DATE	CHK'D	APP'D
[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]

[redacted]

[redacted] Design	[redacted] Date
[redacted] Drawn	[redacted] Date
[redacted] Checked	[redacted] Date
[redacted] Approved	[redacted] Date

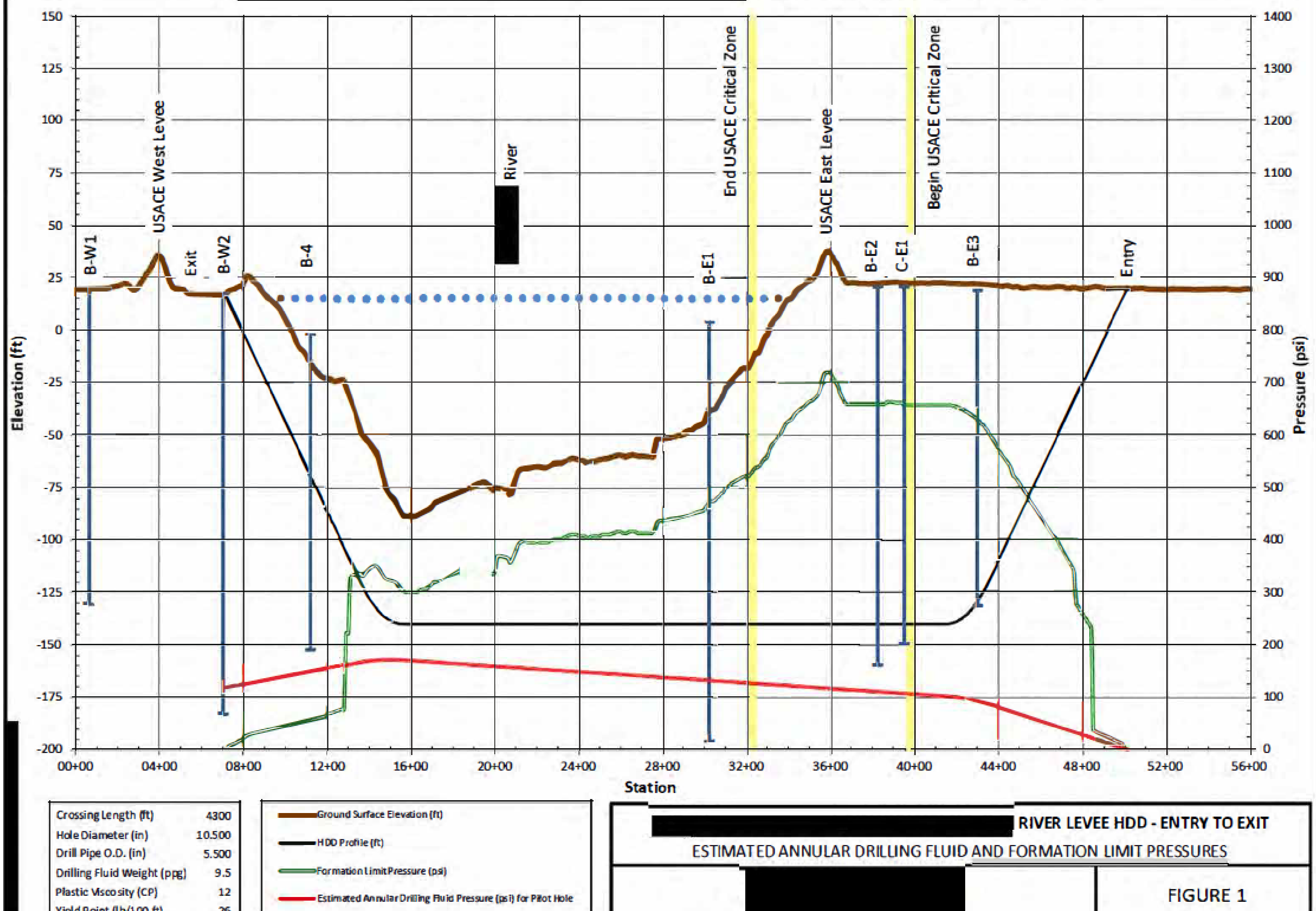
[redacted]

[redacted]
STRINGING WORKSPACE
[redacted]

Project No. [redacted]
Drawing No. -
Sheet 2 of 2

Appendix E – Hydraulic Fracture Analysis

[redacted] RIVER LEVEE HDD - ENTRY TO EXIT



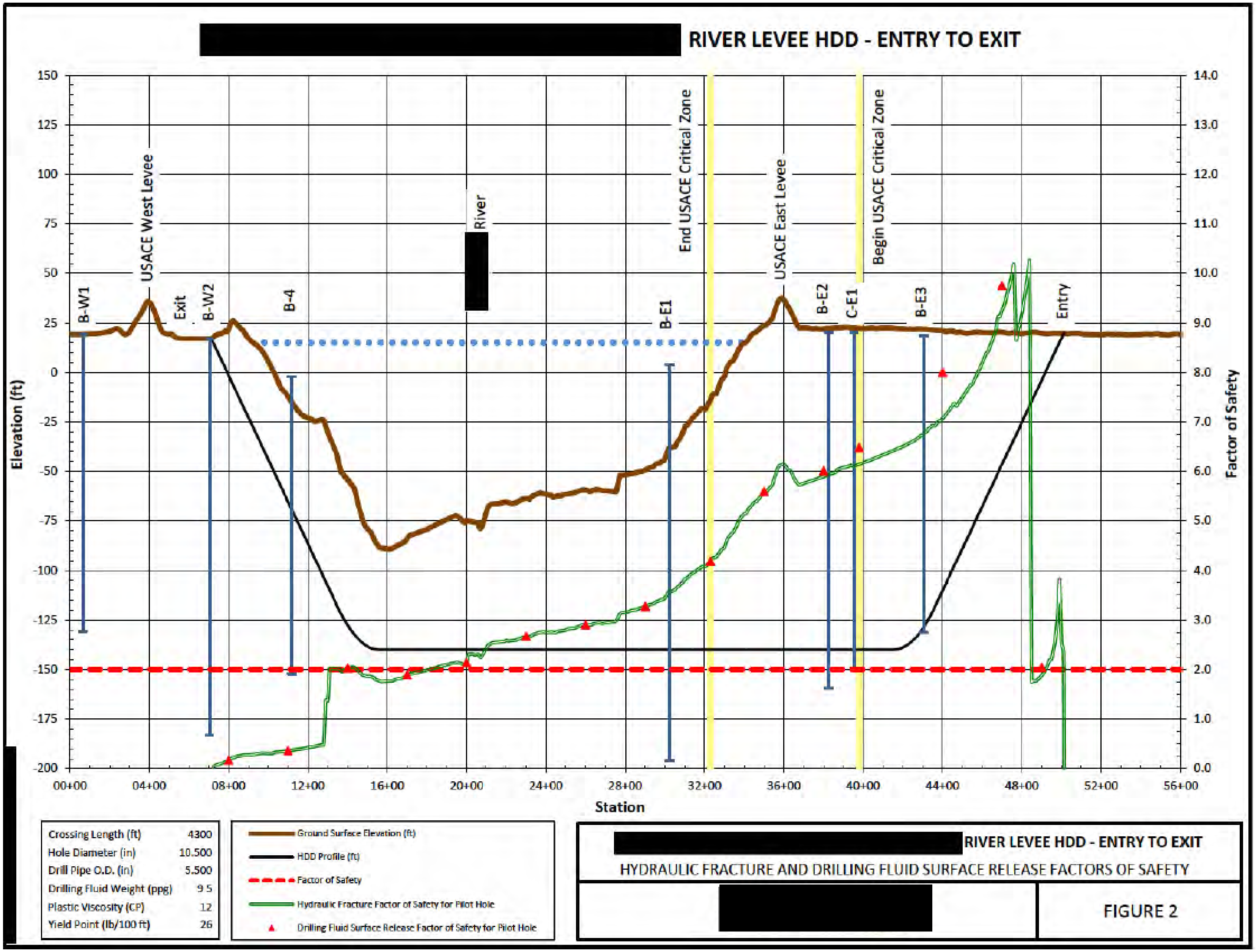
Crossing Length (ft)	4300
Hole Diameter (in)	10.500
Drill Pipe O.D. (in)	5.500
Drilling Fluid Weight (ppg)	9.5
Plastic Viscosity (CP)	12
Yield Point (lb/100 ft)	26

[redacted] RIVER LEVEE HDD - ENTRY TO EXIT

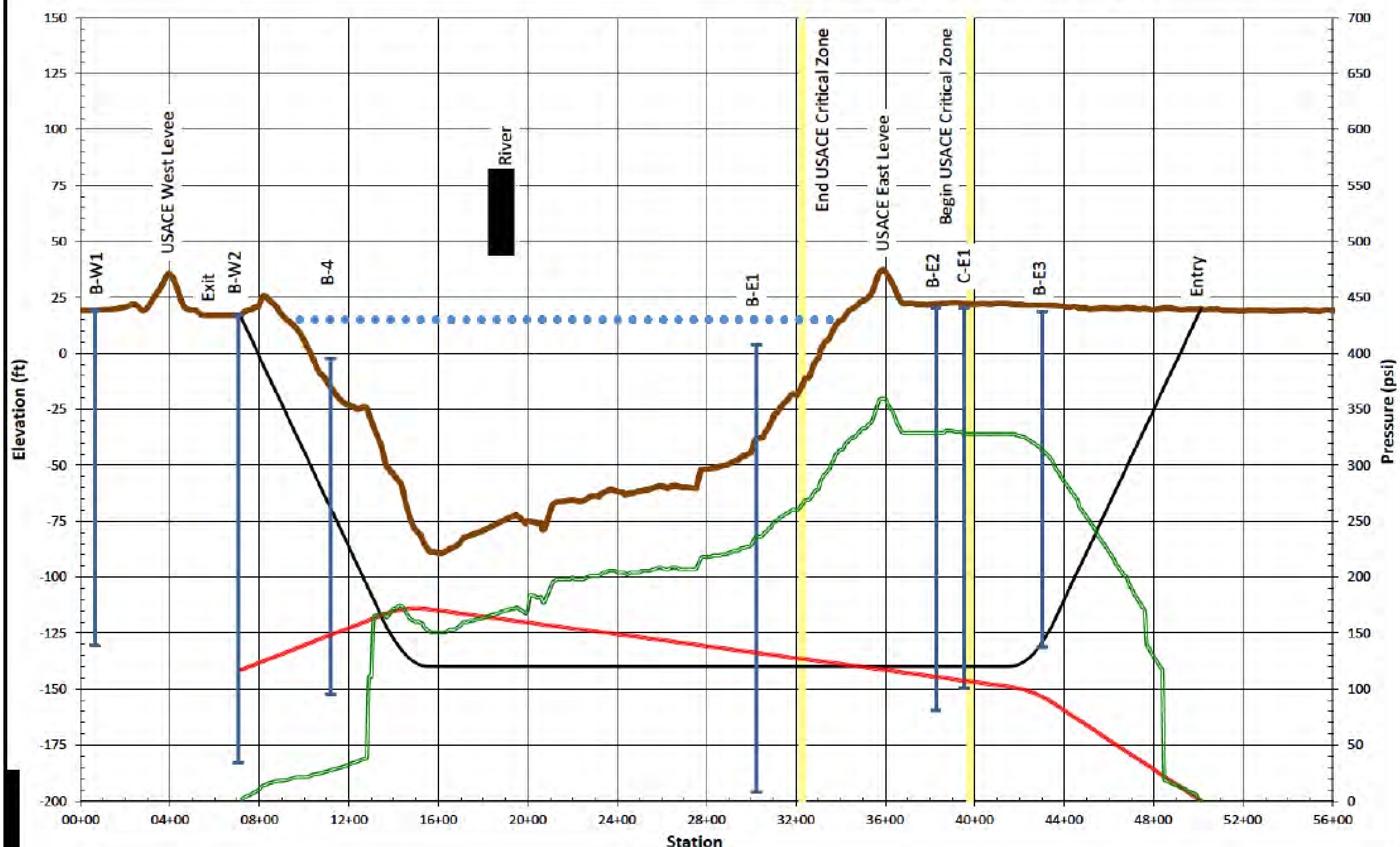
ESTIMATED ANNULAR DRILLING FLUID AND FORMATION LIMIT PRESSURES

[redacted]

FIGURE 1


[redacted] RIVER LEVEE HDD - ENTRY TO EXIT
B-W1
USACE West Levee
Exit
B-W2
B-4
River
[redacted]
B-E1
End USACE Critical Zone
USACE East Levee
B-E2
C-E1
Begin USACE Critical Zone
B-E3
Entry
Elevation (ft)
Factor of Safety
Station
150
125
100
75
50
25
0
-25
-50
-75
-100
-125
-150
-175
-200
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
00+00
04+00
08+00
12+00
16+00
20+00
24+00
28+00
32+00
36+00
40+00
44+00
48+00
52+00
56+00
Crossing Length (ft) 4300
Hole Diameter (in) 10.500
Drill Pipe O.D. (in) 5.500
Drilling Fluid Weight (ppg) 9.5
Plastic Viscosity (CP) 12
Yield Point (lb/100 ft) 26
Ground Surface Elevation (ft)
HDD Profile (ft)
Factor of Safety
Hydraulic Fracture Factor of Safety for Pilot Hole
Drilling Fluid Surface Release Factor of Safety for Pilot Hole
[redacted] RIVER LEVEE HDD - ENTRY TO EXIT
HYDRAULIC FRACTURE AND DRILLING FLUID SURFACE RELEASE FACTORS OF SAFETY
[redacted]
FIGURE 2

[redacted] RIVER LEVEE HDD - ENTRY TO EXIT



Crossing Length (ft)	4300
Hole Diameter (in)	10.500
Drill Pipe O.D. (in)	5.500
Drilling Fluid Weight (ppg)	9.5
Plastic Viscosity (CP)	12
Yield Point (lb/100 ft)	26

- Ground Surface Elevation (ft)
- HDD Profile (ft)
- Estimated Annular Drilling Fluid Pressure (psi) for Pilot Hole
- Allowable Drilling Fluid Pressure (psi)

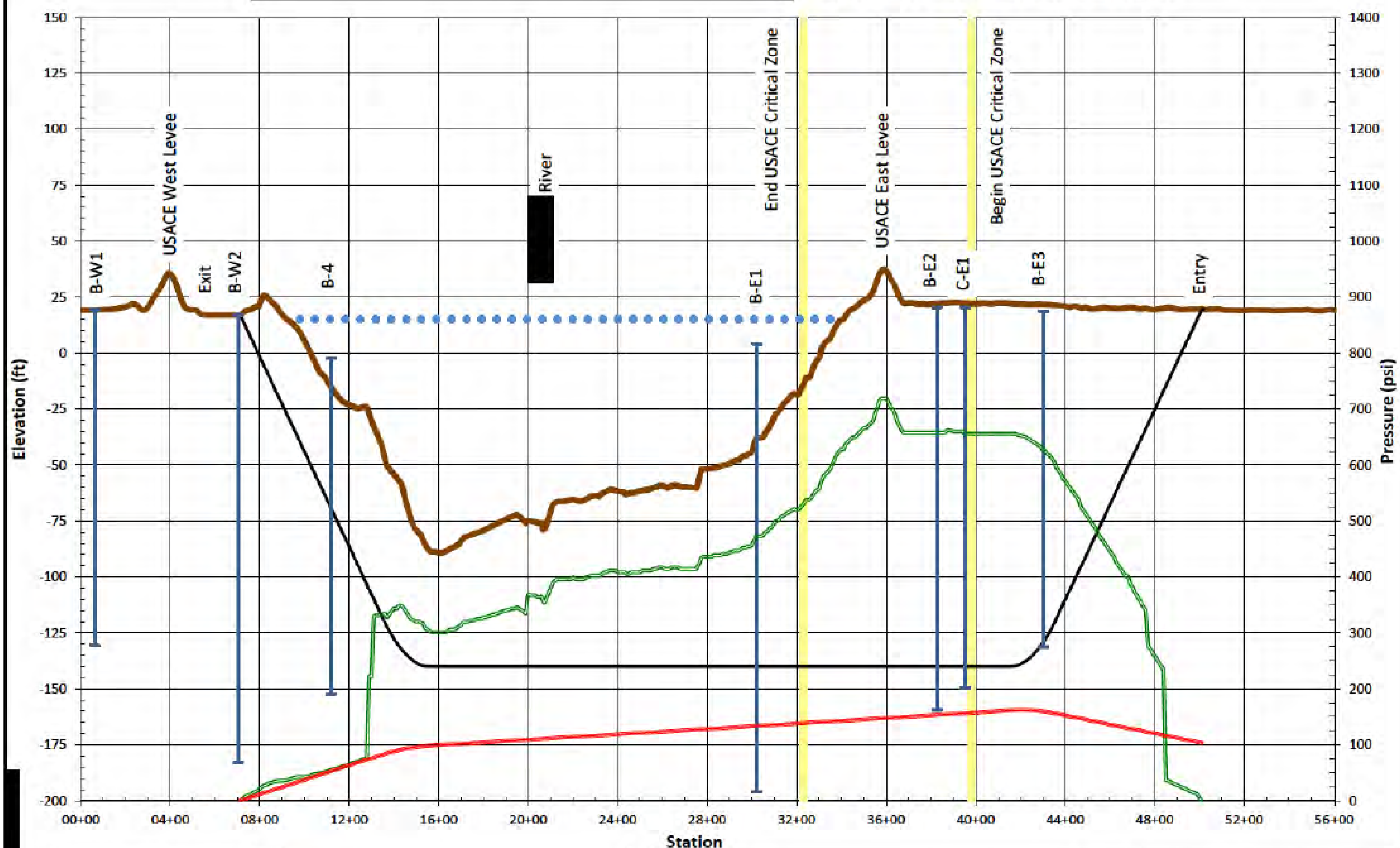
[redacted] RIVER LEVEE HDD - ENTRY TO EXIT

ESTIMATED AND ALLOWABLE ANNULAR DRILLING FLUID PRESSURES

E-8 [redacted]

FIGURE 3

[redacted] RIVER LEVEE HDD - EXIT TO ENTRY



Crossing Length (ft)	4300
Hole Diameter (in)	10.500
Drill Pipe O.D. (in)	5.000
Drilling Fluid Weight (ppg)	9.5
Plastic Viscosity (CP)	12
Yield Point (lb/100 ft)	26

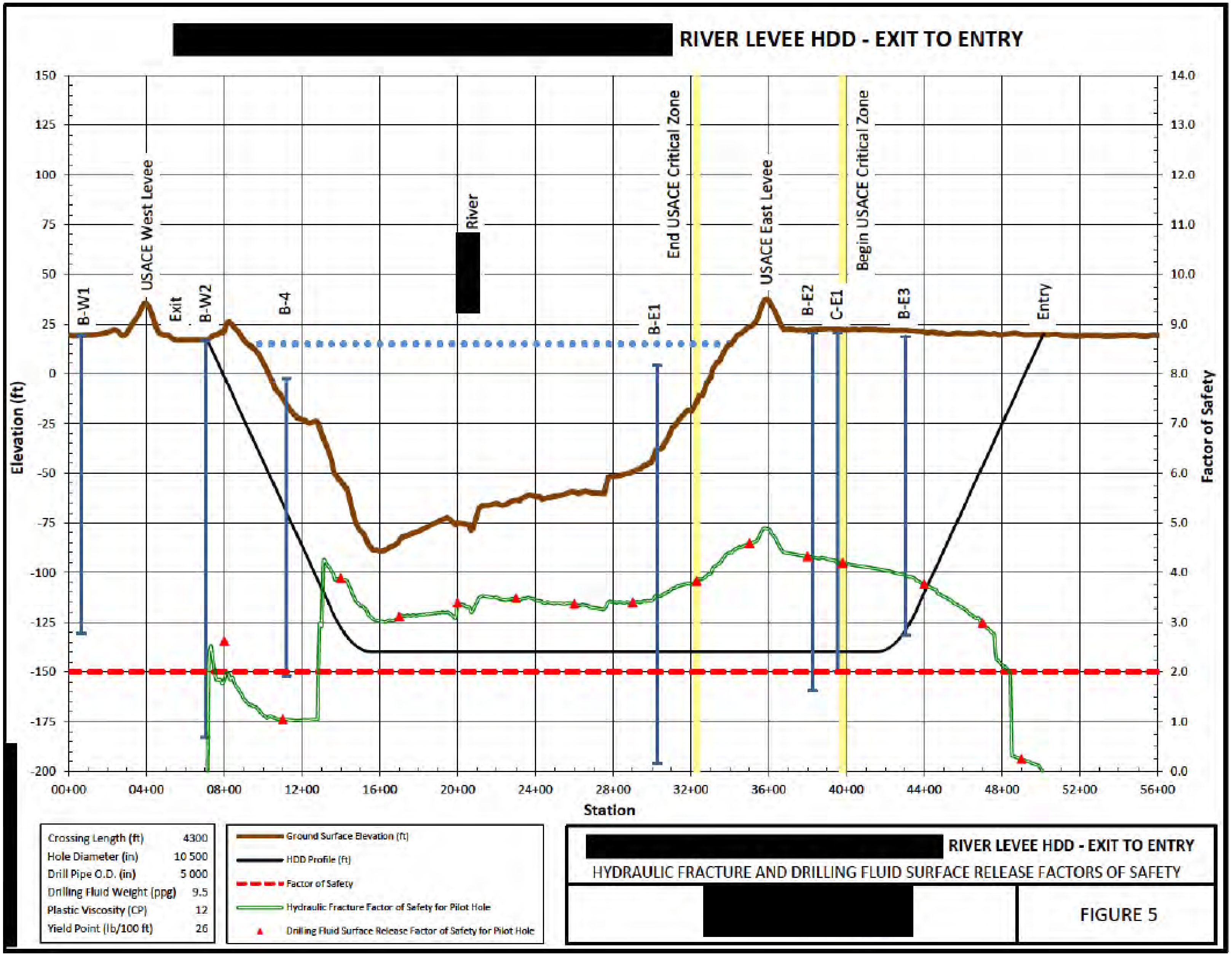
Ground Surface Elevation (ft)
HDD Profile (ft)
Formation Limit Pressure (psi)
Estimated Annular Drilling Fluid Pressure (psi) for Pilot Hole

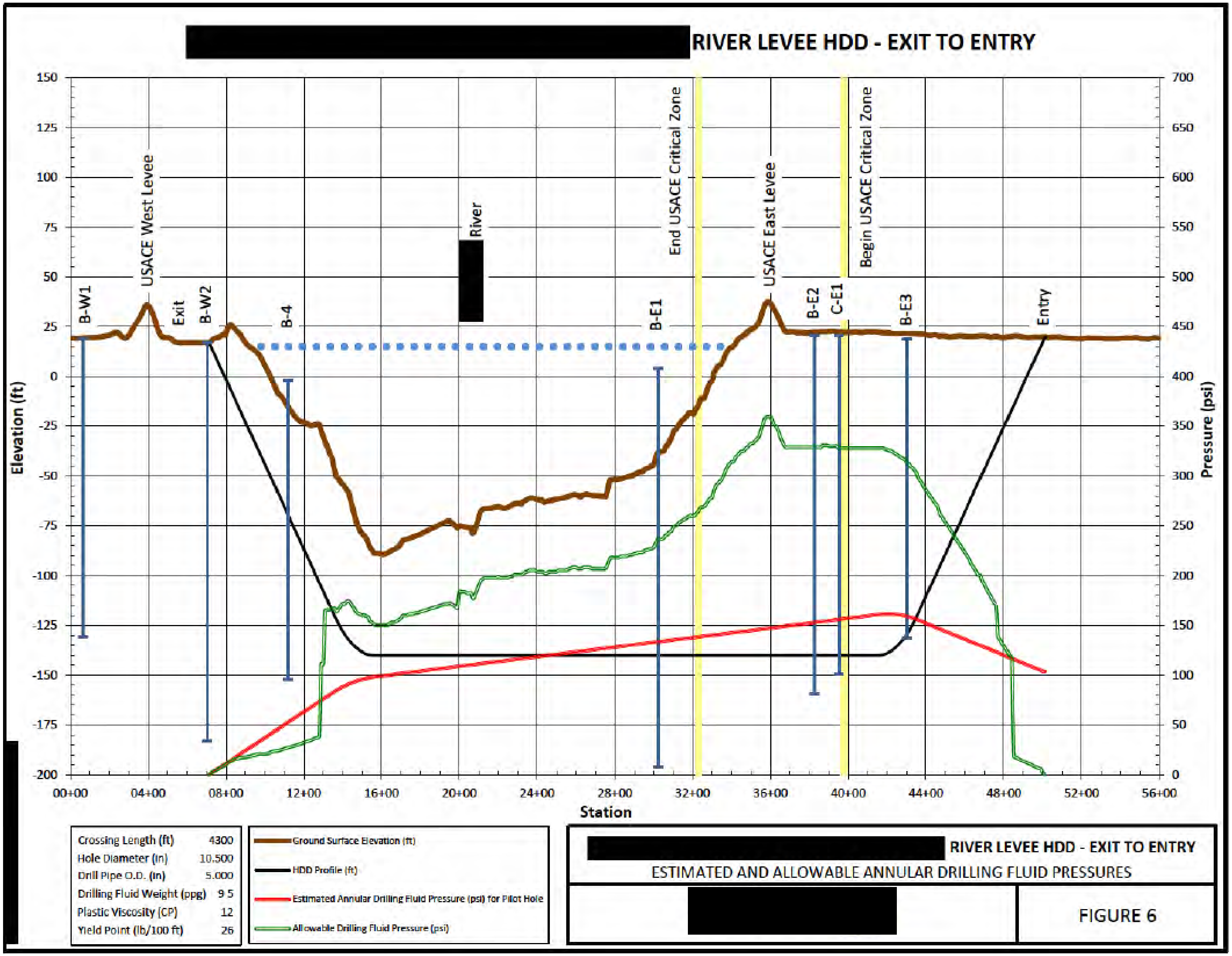
[redacted] RIVER LEVEE HDD - EXIT TO ENTRY

ESTIMATED ANNULAR DRILLING FLUID AND FORMATION LIMIT PRESSURES

[redacted]

FIGURE 4


[redacted] RIVER LEVEE HDD - EXIT TO ENTRY
Elevation (ft)
Factor of Safety
Station
150
125
100
75
50
25
0
-25
-50
-75
-100
-125
-150
-175
-200
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
00+00
04+00
08+00
12+00
16+00
20+00
24+00
28+00
32+00
36+00
40+00
44+00
48+00
52+00
56+00
B-W1
USACE West Levee
Exit
B-W2
B-4
River
[redacted]
B-E1
End USACE Critical Zone
USACE East Levee
B-E2
C-E1
Begin USACE Critical Zone
B-E3
Entry
Crossing Length (ft) 4300
Hole Diameter (in) 10.500
Drill Pipe O.D. (in) 5.000
Drilling Fluid Weight (ppg) 9.5
Plastic Viscosity (CP) 12
Yield Point (lb/100 ft) 26
Ground Surface Elevation (ft)
HDD Profile (ft)
Factor of Safety
Hydraulic Fracture Factor of Safety for Pilot Hole
Drilling Fluid Surface Release Factor of Safety for Pilot Hole
[redacted] RIVER LEVEE HDD - EXIT TO ENTRY
HYDRAULIC FRACTURE AND DRILLING FLUID SURFACE RELEASE FACTORS OF SAFETY
[redacted]
FIGURE 5


[redacted] RIVER LEVEE HDD - EXIT TO ENTRY
Elevation (ft)
Pressure (psi)
Station
B-W1
USACE West Levee
Exit
B-W2
B-4
River
B-E1
End USACE Critical Zone
USACE East Levee
B-E2
C-E1
Begin USACE Critical Zone
B-E3
Entry
Crossing Length (ft) 4300
Hole Diameter (in) 10.500
Drill Pipe O.D. (in) 5.000
Drilling Fluid Weight (ppg) 9.5
Plastic Viscosity (CP) 12
Yield Point (lb/100 ft) 26
Ground Surface Elevation (ft)
HDD Profile (ft)
Estimated Annular Drilling Fluid Pressure (psi) for Pilot Hole
Allowable Drilling Fluid Pressure (psi)
[redacted] RIVER LEVEE HDD - EXIT TO ENTRY
ESTIMATED AND ALLOWABLE ANNULAR DRILLING FLUID PRESSURES
[redacted]
FIGURE 6

Example Calculation Data

Entry to Exit					
Station	HDD Elevation	AP	FLP	HF FOS	Allowable
5012.5	19.9				
5010.0	19.4	0.3	0.8	2.4	0.4
5000.0	17.2	1.7	4.2	2.5	2.1
4990.0	15.1	3.0	11.3	3.8	5.7
4980.0	13.0	4.3	13.1	3.0	6.5
4970.0	10.9	5.6	14.8	2.6	7.4
4960.0	8.7	6.9	16.6	2.4	8.3
4950.0	6.6	8.3	18.3	2.2	9.2
4940.0	4.5	9.6	20.9	2.2	10.5
4930.0	2.4	10.9	22.7	2.1	11.3
4920.0	0.2	12.2	24.4	2.0	12.2
4910.0	-1.9	13.5	26.2	1.9	13.1
4900.0	-4.0	14.9	27.9	1.9	14.0
4890.0	-6.1	16.2	29.7	1.8	14.8
4880.0	-8.3	17.5	31.4	1.8	15.7
4870.0	-10.4	18.8	33.2	1.8	16.6
4860.0	-12.5	20.1	35.8	1.8	17.9
4850.0	-14.6	21.5	37.6	1.8	18.8
4840.0	-16.8	22.8	233.3	10.2	116.6
4830.0	-18.9	24.1	239.7	9.9	119.9
4820.0	-21.0	25.4	246.0	9.7	123.0
4810.0	-23.1	26.7	252.0	9.4	126.0
4800.0	-25.3	28.1	258.0	9.2	129.0
4790.0	-27.4	29.4	263.8	9.0	131.9
4780.0	-29.5	30.7	272.2	8.9	136.1
4770.0	-31.6	32.0	277.8	8.7	138.9
4760.0	-33.8	33.4	339.1	10.2	169.5
4750.0	-35.9	34.7	346.0	10.0	173.0
4740.0	-38.0	36.0	352.8	9.8	176.4
4730.0	-40.1	37.3	359.5	9.6	179.8
4720.0	-42.3	38.6	366.2	9.5	183.1
4710.0	-44.4	40.0	372.8	9.3	186.4
4700.0	-46.5	41.3	382.3	9.3	191.2
4690.0	-48.6	42.6	388.7	9.1	194.4
4680.0	-50.8	43.9	400.6	9.1	200.3
4670.0	-52.9	45.2	401.2	8.9	200.6
4660.0	-55.0	46.6	407.4	8.7	203.7
4650.0	-57.2	47.9	413.5	8.6	206.7
4640.0	-59.3	49.2	419.5	8.5	209.8
4630.0	-61.4	50.5	425.5	8.4	212.8
4620.0	-63.5	51.8	434.4	8.4	217.2
4610.0	-65.7	53.2	440.3	8.3	220.1
4600.0	-67.8	54.5	446.1	8.2	223.1
4590.0	-69.9	55.8	451.9	8.1	226.0
4580.0	-72.0	57.1	457.7	8.0	228.9
4570.0	-74.2	58.5	463.4	7.9	231.7
4560.0	-76.3	59.8	469.1	7.8	234.6
4550.0	-78.4	61.1	474.8	7.8	237.4
4540.0	-80.5	62.4	483.5	7.7	241.8
4530.0	-82.7	63.7	489.5	7.7	244.7
4520.0	-84.8	65.1	495.4	7.6	247.7
4510.0	-86.9	66.4	501.3	7.6	250.7
4500.0	-89.0	67.7	507.2	7.5	253.6
4490.0	-91.2	69.0	513.0	7.4	256.5
4480.0	-93.3	70.3	518.8	7.4	259.4
4470.0	-95.4	71.7	524.6	7.3	262.3
4460.0	-97.5	73.0	538.0	7.4	269.0
4450.0	-99.7	74.3	543.6	7.3	271.8
4440.0	-101.8	75.6	549.3	7.3	274.6
4430.0	-103.9	76.9	554.9	7.2	277.4
4420.0	-106.0	78.3	560.5	7.2	280.2
4410.0	-108.2	79.6	566.0	7.1	283.0
4400.0	-110.3	80.9	571.6	7.1	285.8
4390.0	-112.4	82.2	577.1	7.0	288.5
4380.0	-114.5	83.6	585.3	7.0	292.6
4370.0	-116.7	84.9	590.7	7.0	295.4
4360.0	-118.8	86.2	596.1	6.9	298.1
4350.0	-120.8	87.5	606.1	6.9	303.0
4340.0	-122.8	88.7	611.4	6.9	305.7
4330.0	-124.6	89.9	616.8	6.9	308.4
4320.0	-126.3	91.0	619.4	6.8	309.7
4310.0	-127.9	92.1	624.7	6.8	312.4
4300.0	-129.4	93.1	627.4	6.7	313.7
4290.0	-130.9	94.0	632.6	6.7	316.3
4280.0	-132.2	95.0	635.3	6.7	317.6
4270.0	-133.4	95.8	637.9	6.7	319.0

Exit to Entry					
Station	HDD Elevation	AP	FLP	HF FOS	Allowable
712.5	17.0				
720.0	15.4	1.0	2.2	2.3	1.1
730.0	13.3	2.3	5.7	2.5	2.9
740.0	11.2	3.6	8.1	2.3	4.1
750.0	9.1	4.9	9.7	2.0	4.9
760.0	6.9	6.2	11.3	1.8	5.7
770.0	4.8	7.4	13.7	1.8	6.9
780.0	2.7	8.7	16.1	1.8	8.1
790.0	0.6	10.0	17.7	1.8	8.9
800.0	-1.6	11.3	20.9	1.8	10.5
810.0	-3.7	12.6	24.9	2.0	12.4
820.0	-5.8	13.9	27.3	2.0	13.6
830.0	-7.9	15.2	28.1	1.8	14.0
840.0	-10.1	16.5	31.3	1.9	15.6
850.0	-12.2	17.8	32.0	1.8	16.0
860.0	-14.3	19.1	32.7	1.7	16.4
870.0	-16.4	20.4	34.2	1.7	17.1
880.0	-18.6	21.7	34.9	1.6	17.5
890.0	-20.7	23.0	35.7	1.6	17.8
900.0	-22.8	24.3	35.6	1.5	17.8
910.0	-24.9	25.6	36.3	1.4	18.2
920.0	-27.1	26.9	37.0	1.4	18.5
930.0	-29.2	28.2	37.8	1.3	18.9
940.0	-31.3	29.5	39.3	1.3	19.7
950.0	-33.4	30.8	40.0	1.3	20.0
960.0	-35.6	32.1	41.5	1.3	20.8
970.0	-37.7	33.4	42.3	1.3	21.1
980.0	-39.8	34.7	42.2	1.2	21.1
990.0	-41.9	35.9	42.1	1.2	21.1
1000.0	-44.1	37.2	42.0	1.1	21.0
1010.0	-46.2	38.5	42.8	1.1	21.4
1020.0	-48.3	39.8	42.7	1.1	21.4
1030.0	-50.4	41.1	45.4	1.1	22.7
1040.0	-52.6	42.4	46.9	1.1	23.4
1050.0	-54.7	43.7	47.3	1.1	23.6
1060.0	-56.8	45.0	48.0	1.1	24.0
1070.0	-58.9	46.3	48.8	1.1	24.4
1080.0	-61.1	47.6	50.0	1.0	25.0
1090.0	-63.2	48.9	51.1	1.0	25.6
1100.0	-65.3	50.2	52.3	1.0	26.1
1110.0	-67.4	51.5	53.1	1.0	26.6
1120.0	-69.6	52.8	55.0	1.0	27.5
1130.0	-71.7	54.1	55.8	1.0	27.9
1140.0	-73.8	55.4	57.0	1.0	28.5
1150.0	-76.0	56.7	58.2	1.0	29.1
1160.0	-78.1	58.0	59.0	1.0	29.5
1170.0	-80.2	59.3	60.1	1.0	30.1
1180.0	-82.3	60.6	61.6	1.0	30.8
1190.0	-84.5	61.9	62.8	1.0	31.4
1200.0	-86.6	63.2	65.0	1.0	32.5
1210.0	-88.7	64.4	66.5	1.0	33.2
1220.0	-90.8	65.7	67.7	1.0	33.8
1230.0	-93.0	67.0	68.8	1.0	34.4
1240.0	-95.1	68.3	70.3	1.0	35.1
1250.0	-97.2	69.6	72.1	1.0	36.1
1260.0	-99.3	70.9	73.6	1.0	36.8
1270.0	-101.5	72.2	75.1	1.0	37.5
1280.0	-103.6	73.5	77.3	1.1	38.7
1290.0	-105.7	74.8	222.5	3.0	111.2
1300.0	-107.8	76.1	222.1	2.9	111.0
1310.0	-110.0	77.4	329.0	4.3	164.5
1320.0	-112.1	78.7	331.4	4.2	165.7
1330.0	-114.2	80.0	331.3	4.1	165.7
1340.0	-116.3	81.3	333.7	4.1	166.8
1350.0	-118.5	82.6	333.5	4.0	166.8
1360.0	-120.5	83.8	334.4	4.0	167.2
1370.0	-122.5	85.0	328.3	3.9	164.2
1380.0	-124.3	86.2	333.5	3.9	166.8
1390.0	-126.1	87.3	338.6	3.9	169.3
1400.0	-127.7	88.4	343.7	3.9	171.9
1410.0	-129.2	89.4	342.8	3.8	171.4
1420.0	-130.6	90.3	347.8	3.9	173.9
1430.0	-132.0	91.2	349.2	3.8	174.6
1440.0	-133.2	92.1	345.7	3.8	172.9
1450.0	-134.3	92.8	339.7	3.7	169.9
1460.0	-135.3	93.6	333.6	3.6	166.8

ANNULAR SOLIDS DURING REAMING

Hole Diameter (in)	10.5	Reamer Diameter (in)	24	Joint Length (ft)	31.5

Flow Rate (gpm)

Reaming Time (min/joint)	Penetration Rate (ft/min)	100	125	150	175	200	225	250	275	300	325	350
1	31.5	85.7%	82.7%	80.0%	77.4%	75.0%	72.7%	70.5%	68.5%	66.6%	64.8%	63.1%
2	15.8	75.0%	70.5%	66.6%	63.1%	59.9%	57.1%	54.5%	52.1%	49.9%	47.9%	46.1%
3	10.5	66.6%	61.5%	57.1%	53.3%	49.9%	47.0%	44.4%	42.0%	39.9%	38.0%	36.3%
4	7.9	59.9%	54.5%	49.9%	46.1%	42.8%	39.9%	37.4%	35.2%	33.3%	31.5%	29.9%
5	6.3	54.5%	48.9%	44.4%	40.6%	37.4%	34.7%	32.4%	30.3%	28.5%	26.9%	25.5%
6	5.3	49.9%	44.4%	39.9%	36.3%	33.3%	30.7%	28.5%	26.6%	25.0%	23.5%	22.2%
7	4.5	46.1%	40.6%	36.3%	32.8%	29.9%	27.5%	25.5%	23.7%	22.2%	20.8%	19.6%
8	3.9	42.8%	37.4%	33.3%	29.9%	27.2%	25.0%	23.0%	21.4%	20.0%	18.7%	17.6%
9	3.5	39.9%	34.7%	30.7%	27.5%	25.0%	22.8%	21.0%	19.5%	18.1%	17.0%	16.0%
10	3.2	37.4%	32.4%	28.5%	25.5%	23.0%	21.0%	19.3%	17.9%	16.6%	15.6%	14.6%
11	2.9	35.2%	30.3%	26.6%	23.7%	21.4%	19.5%	17.9%	16.5%	15.4%	14.3%	13.5%
12	2.6	33.3%	28.5%	25.0%	22.2%	20.0%	18.1%	16.6%	15.4%	14.3%	13.3%	12.5%
13	2.4	31.5%	26.9%	23.5%	20.8%	18.7%	17.0%	15.6%	14.3%	13.3%	12.4%	11.6%
14	2.3	29.9%	25.5%	22.2%	19.6%	17.6%	16.0%	14.6%	13.5%	12.5%	11.6%	10.9%
15	2.1	28.5%	24.2%	21.0%	18.6%	16.6%	15.1%	13.8%	12.7%	11.7%	10.9%	10.2%
16	2.0	27.2%	23.0%	20.0%	17.6%	15.8%	14.3%	13.0%	12.0%	11.1%	10.3%	9.7%
17	1.9	26.0%	22.0%	19.0%	16.7%	15.0%	13.5%	12.3%	11.3%	10.5%	9.8%	9.1%
18	1.8	25.0%	21.0%	18.1%	16.0%	14.3%	12.9%	11.7%	10.8%	10.0%	9.3%	8.7%
19	1.7	24.0%	20.1%	17.4%	15.3%	13.6%	12.3%	11.2%	10.3%	9.5%	8.8%	8.3%
20	1.6	23.0%	19.3%	16.6%	14.6%	13.0%	11.7%	10.7%	9.8%	9.1%	8.4%	7.9%
21	1.5	22.2%	18.6%	16.0%	14.0%	12.5%	11.2%	10.2%	9.4%	8.7%	8.1%	7.5%
22	1.4	21.4%	17.9%	15.4%	13.5%	12.0%	10.8%	9.8%	9.0%	8.3%	7.7%	7.2%
23	1.4	20.6%	17.2%	14.8%	12.9%	11.5%	10.4%	9.4%	8.6%	8.0%	7.4%	6.9%
24	1.3	20.0%	16.6%	14.3%	12.5%	11.1%	10.0%	9.1%	8.3%	7.7%	7.1%	6.7%
25	1.3	19.3%	16.1%	13.8%	12.0%	10.7%	9.6%	8.7%	8.0%	7.4%	6.9%	6.4%
26	1.2	18.7%	15.6%	13.3%	11.6%	10.3%	9.3%	8.4%	7.7%	7.1%	6.6%	6.2%
27	1.2	18.1%	15.1%	12.9%	11.2%	10.0%	9.0%	8.1%	7.5%	6.9%	6.4%	6.0%
28	1.1	17.6%	14.6%	12.5%	10.9%	9.7%	8.7%	7.9%	7.2%	6.7%	6.2%	5.8%
29	1.1	17.1%	14.2%	12.1%	10.5%	9.4%	8.4%	7.6%	7.0%	6.4%	6.0%	5.6%
30	1.1	16.6%	13.8%	11.7%	10.2%	9.1%	8.1%	7.4%	6.8%	6.2%	5.8%	5.4%

Annular Solids Less Than 20%
Annular Solids Between 20% and 30%
Annular Solids Greater Than 30%

Annular Solids (%) = Annular Volume (gal)/(Annular Volume (gal)+(Flow Rate (gpm)*Reaming Time (min))*100

[redacted]

ANNULAR SOLIDS DURING REAMING

24-inch Reaming Pass

GROUTING PLAN

Scope of Work:
Grouting contractor will work in conjunction with the other on-site contractors (HDD contractor and Mainline Contractor) to coordinate Grouting Contractor's work on both the entry and exit sides of HDDs mentioned above. Grouting Contractor will install annular grout using a Corps of Engineers (USACE) approved bentonite grout mix into the annular space between the reamed hole and the pipeline to mitigate possible seepage through the annular space.

Prior to starting any work, Grouting Contractor will make necessary utility locates. Grouting Contractor will then work in conjunction with the Survey crew(s), mainline contractor and HDD contractor to make sure that the work can be completed without interruption of other construction tasks, such as testing the HDD section, and to ensure there are no delays in the tie in of the drills.

Grouting Contractor will use a water jet technique utilizing a three (3) inch dual nozzle to locate the production pipe from entry or exit point to the proposed grout plug length every forty (40) feet. Once the production pipe has been identified, a designed probe rod will be injected approximately three (3) feet, seven (7) inches from edge of production pipe. Once this process has been completed, Grouting Contractor will then directionally drill a tremie pipe with a diameter of two (2) feet, seven-eighth (7/8) to the location of the proposed annular plug. At this point, a grout pump will then be attached to the tremie pipe and Grouting Contractor will proceed with grouting of the annulus with a COE approved bentonite grout mix starting at the proposed length and working our way back toward the entry/exit site. The pump to extraction rate is 2.17 cubic foot to linear foot.

During grouting activities there will be continuous on-site monitoring of the flow and back pressure at the grout pump. For purposes of calculating the proportionate volume of bentonite grout required for each annular grout plug, the following calculation is used:

$$FS = \frac{h_s \gamma'}{H_w \gamma_w} > 1.6$$

Where:

h_s = *grout depth (feet)*

γ' = *effective or buoyant unit weight of grout (pcf)*

H_w = *Height of excess head above the bottom of the grout (feet) or head difference between flood elevation and bottom of grout (feet)*

γ_w = *unit weight of water (pcf)*

The borehole annulus volume will be calculated and reported by taking the inside diameter of the final ream pass (to be determined) less the outer diameter of the production pipe (eighteen inches). Once the final ream pass size is known a cubic yard per foot can be calculated to determine the amount of grout required. Once the appropriate amount of grout has been injected into the annular space and is observed at the entry/exit holes, a minimum of four hours will be required for the grout mix to settle prior to the final backfill of the entry/exit holes. Grout is expected to be pumped at approximately 115 PSI and will be calibrated daily.

THIS PAGE INTENTIONALLY LEFT BLANK

Calculations to ensure cnoveyance capacity and erosion protection after slip lining.

Project # 07192-2016 Project Location 15+50 Brevoort Levee

Pipe Length =	96	ft			
Host Pipe Diameter =	60	in	=	5	ft
Liner Pipe Diameter =	50.477	in	=	4.206	ft
Slope =	0.0104				
K =	1.49				

Conveyance Capacity

$$Q = \frac{1}{n} KAR^{2/3}S^{1/2}$$

Q = Flow (cfs)
n = Manning's Coefficient for the Pipe Material
K = Conversion Factor
A = Area (ft^2)
R = Hydraulic Radius, Area / Wetted Perimeter (ft)
S = Slope

Host Pipe (CMP)

A = 19.635 ft^2
P = 15.708 ft
R = 1.250 ft
n = 0.024

$$Q = \frac{1}{0.024}(1.49)x(19.635)x(1.25)^{2/3}x(0.0104)^{1/2}$$

Q = 144.254 cfs

Liner Pipe (HDPE)

A = 13.897 ft^2
P = 13.215 ft
R = 1.052 ft
n = 0.012

$$Q = \frac{1}{0.024}(1.49)x(13.897)x(1.052)^{2/3}x(0.0104)^{1/2}$$

Q = 181.972 cfs

Flow of Liner Pipe is greater than Host Pipe, therefore OK.

Designing for Erosion

Q = VA
V = Q/A

Q = Flow (cfs)
V = Velocity (ft/s)
A = Area (ft^2)

*Using Manning's equation to compute max flow, assume no tailwater condition and no head pressure.

V = (181.972)/(13.897) = **13.095 ft/s**

Using Max Velocity of Liner Pipe, 13.095 ft/s, input into USACE ERDC Software "ChannelPro"

Output of "ChannelPro" indicates Class 2 riprap

THIS PAGE INTENTIONALLY LEFT BLANK

Appendix G
Case Study of Properly Decommissioning/Infilling an Existing Pipe

Project Name: Canton Lake Auxiliary Project

Location: Canton, Oklahoma

Description: Decommissioning of Toe Drainage System (~2 miles) with Sand Slurry Mixture

Summary: The project required approximately two miles of an existing drainage system to be decommissioned. The old toe drainage system consisted of nearly 2 miles (10,680 feet) of bituminous coated corrugated steel pipe that varied from 24 to 42 inches in diameter. The system also had a series of 23 corrugated steel pipe risers, or manholes, spaced every 400 to 600 feet, which varied in depth from 10 to 30 feet. The slope of the old toe drainage system varied from 0.15 to 1.5 percent, with the majority less than 0.26 percent. Two hundred and thirty eight relief wells were connected to the toe drain system via 6 inch lateral lines. The relief wells were previously capped and abandoned. The pipes converged at a manhole and flowed into the new toe drainage system, and then discharged into the outfall channel.

The depth of the toe drainage system made removal very costly and would have necessitated a large excavation, which would have introduced significant risk to the foundation of the dam. Filling the toe drainage system with filter compatible sand was determined to be an economical way to eliminate the potential collapse of the disposal berm when the old corrugated metal pipes rusted out, which could lead to internal erosion into the pipe. This method also removes the risks involved with excavation and removal of the system. As a precursor to the production work, a mockup section was required to be constructed in order to demonstrate the proposed filling procedures.

Full-Scale Mockup

In order to evaluate the infilling process, the contractor performed a full scale mockup to prove that the means and methods of construction could adequately fill the drainage pipes with the filter sand. The mockup testing was performed at a nearby sand quarry. The test setup consisted of 600 feet of 24 inch diameter bituminous coated corrugated steel pipe set at a typical slope for the project, with a 6 foot tall riser on the lower end. Six inch PVC pipes were placed every 50 feet to simulate finger drains and to serve as observation holes; reference Figure G-1.

The filter sand was mixed with water and a polymer additive (Polyore), which allowed the sand to remain in suspension and be hydraulically placed into the pipe; reference Table G-1. The specified gradation for the filter sand is shown in Table G-2. The mockup test showed that placement of the sand slurry using the tremie method was not very effective nor necessary. The mockup testing adequately filled 450 feet of the pipe from one application location; as confirmed by removal of a small pipe section; reference Figure G-2.

The mixture design used for the mockup consisted of the following:

Table G-1. Mixture Distribution per Cubic Yard

Material	Weight (lbs)	% by Volume
Sand	2,303	52.2
Potable Water	798	47.4
Polyore	5.30	<0.5

Figure G-1. Filter Sand Slurry Mockup

Figure G-2. Removed section for analysis (100-ft from end)

Table G-2. Filter Sand Gradation

Sieve No.	Actual % Passing	Required
3/8”	100	100
No. 4	97	95 – 100
No. 8	89	80 – 100
No. 16	69	50 – 85
No. 30	34	25 – 60
No. 50	9	5 – 30
No. 100	2	0 – 10
No. 200	0.7	0 - 3

Mockup Results / Lessons Learned

The main points noted throughout the mockup process were as follows:

- Full tremie of the concrete hose through the pipe was not needed
- The material will continue to flow as long as a constant supply of mixer trucks is maintained
- Pumping rate can be increased
- The pipe will be nearly full of sand at the conclusion of placement

Filter Sand Slurry Production

The sand slurry was mixed in an onsite batch plant and the slurry was transported from the batch plant to the application locations by cement mixer trucks. The old toe drain system was video inspected prior to being filled with the filter sand slurry to ensure that the pipe was free of an obstructions.

Table G-3 shows the sand slurry mixture used in production. The optimal amount of Polyore additive was found to be approximately 4.4 pounds per cubic yard of sand. Overall approximately 4,356 tons of filter sand, and 542 buckets (15,718 pounds) of Polyore additive were used to fill the old toe drainage system.

Although the sand slurry mockup results proved that the sand slurry could flow as much as 450 feet, actual performance had mixed success. Problems were attributed to the presence of standing water in the pipes. Initially bulkheads were placed on both upstream and downstream ends of the pipe. Sand slurry was then pumped into the system with mixed success. The production plan had to be altered several times.

The final application process removed the upstream bulkheads and poured the sand slurry directly into the manholes. Intermediate holes, 18 inches in diameter, were then drilled between the toe drain man holes for additional application locations to ensure that the pipe was adequately filled with filter sand. The actual volume of sand slurry used was measured and compared to the theoretical volume to determine if the pipe had been adequately filled, or identify where additional intermediate holes were needed. The final installation plan is shown schematically in Figures G-3 and G-4.

On average, 30 to 35 cubic yards of sand slurry could be placed per hole before the sand began to back up in the pipe, and the riser was filled. This helped estimate how many intermediate holes were needed, and where to drill them. Initially, 57 cased, 18 inch diameter intermediate holes were drilled, which allowed placement of 2,102 cubic yards of sand slurry out of a theoretical 2,426 cubic yards (87%). Placement notes were reviewed and locations were mapped that could potentially accept more slurry. 39 additional intermediate holes were drilled, and over 300 cubic yards of additional sand slurry was placed. The theoretical volume placed as compared to the estimated pipe volume varied from approximately 99 to 143 percent.

Table G-3. Production Sand Slurry Mixture Design per Cubic Yard

Material	Weight (lbs.)	% by Volume
Filter Sand	2,220	50.3
Potable Water	830	49.3
Polyore Additive	2.9 – 5.8	< 0.5

A permanent bulkhead was constructed just upstream from the manhole where the old toe drain system drains in to the new toe drain system. The bulkhead was constructed of stainless steel wire grating with 2 layers of US 670 woven filter fabric to allow water to pass while retaining the filter sand. The new toe drain system was video inspected and cleaned in July 2015 after completion of the filling of the old toe drain system, and found to be in good condition. The permeable bulkhead was inspected and was doing well at holding back the filter sand in the old system while still allowing water to pass from the old system to the new one.

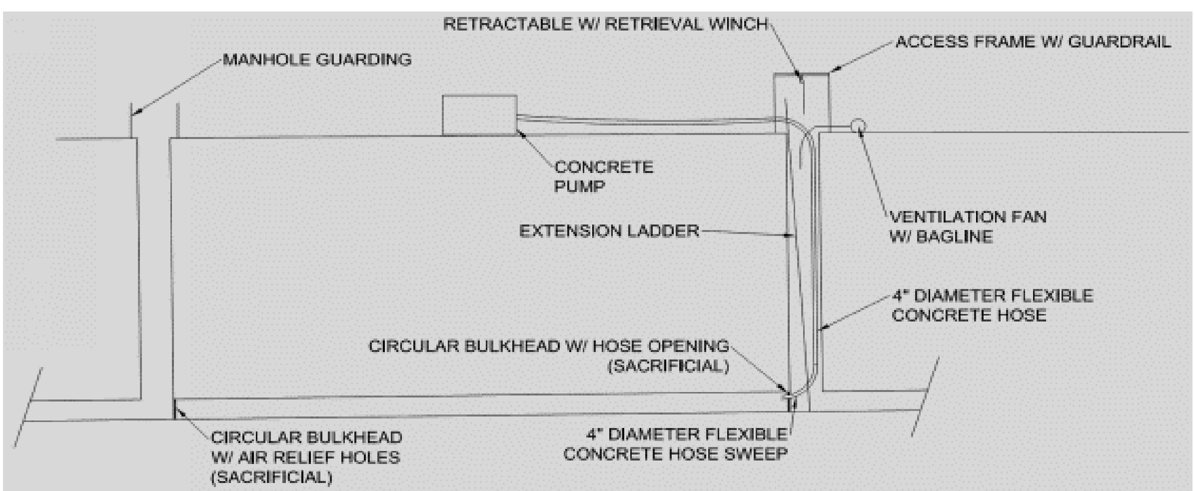


Figure G-3. Pumping Plan

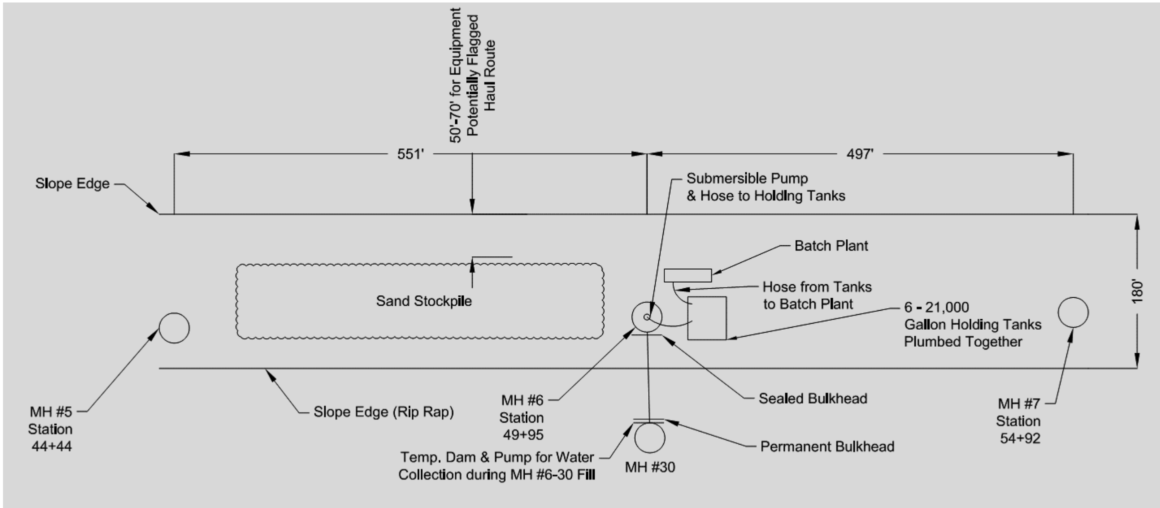


Figure G-4. General Arrangement of Batching and Environmental Setup

Lessons Learned:

1. The sand slurry mixture resulted in a cost savings as compared with cementitious grout.
2. The full-scale mock-up testing proved beneficial for optimizing slurry placement techniques.
3. Removal of small pipe sections aided in evaluating the volume of material placed within the pipe.

THIS PAGE INTENTIONALLY LEFT BLANK

Appendix H
Suggested Submittal Items for Pipe-Related Section 408 Alteration Requests

The respective USACE District must verify whether the activity conducted by a levee sponsor on a pipe requires a formal Section 408 permission or not. If not, some of the content in this appendix may still be information that should be coordinated and reviewed between the levee sponsor and USACE. Monitoring is typically performed by a USACE representative familiar with the project or a third party hired by the levee sponsor.

The following is a suggested list of submittals that USACE should be reviewing for approval of a pipe-related alteration request.

Installations:

- **Elevated Placement**
        - Pipe material and diameter to be used
        - Details of valves if it is a pressure pipe within 50 feet of levee toes or floodwall foundations.
        - Additional rights-of-way if associated structure is outside existing limits.
        - Pipe annular sealing details.
        - Cross-section showing the existing as-built information along with the proposed placement of the new pipe and a plan with levee right-of-way at the location.
        - Embankment As-built stationing where crossing is proposed.
        - Notes on drawings stating the related District's backfill and compaction requirements (typically referencing ASTMs D2488 and D698/1557).
        - Plan view and cross-sections of the post-construction slopes showing that the finished grading provides the recommended 10H:1V slopes surrounding the pipe.
        - Proposed testing of pressure pipe joints to meet 0% leakage between the valves based on the appropriate ASTM for the pipe material being used.
        - Working pressure of the pipe.
- **Within Placement and/or Removal**
    - **Open-cut installation**
        - Pipe material and diameter to be used.
        - Lateral limits and geometry of excavation, including shoring methods and right-of-way limits.
        - Notes on drawings stating the related District's backfill and compaction requirements (typically referencing ASTMs D2488 and D698/1557).
        - Embankment as-built stationing where crossing is proposed.
        - Emergency closure plan in case of flooding.
        - Evaluation to determine if a gatewell is needed using Chapter 9 in EM 1110-2-2902.
        - Details of internal seepage filter.

        - Prohibition against beginning excavating until a closure device is on site (gate, inflatable bladder, or other device deemed appropriate by the related District).
        - Evidence that the chosen pipe can handle the anticipated loads.
        - Requirements that photos be taken before, during and after installation.
        - USACE suggestion for onsite camera during installation to understand construction.
    - **Horizontal trenchless installation**
        - Pipe material and diameter to be used.
        - Proposed trenchless installation method, including entry and exit pit details.
        - Embankment as-built stationing where crossing is proposed.
        - Details of external seepage filter and waterside buttress.
        - Evaluation to determine if a gatewell is needed using EM 1110-2-2902.
        - Evidence that the chosen pipe can handle the anticipated loads and installation method.
        - Prohibition against beginning advancement until a closure device is on site (gate, inflatable bladder, or other device deemed appropriate by the related District).
- **Beneath Placement**
    - A drilling program plan (DPP) as required by ER 1110-1-1807.
- **Adjacent to Placement**
    - Pipe material and diameter to be used.
    - Backfill gradation to be used around the toe drain with filter compatibility analysis using the native soil.
    - Cross-section locating the toe drain along the embankment/floodwall.

Repair:

- Details of the repair method based on the pipe material.
- Access information depending on pipe diameter – man or remote entry.
- Photos taken before, during and after repair.
- The testing method for the area of the liner pipe repaired (per the applicable ASTM) for 0% leakage.

Rehabilitation:

- The liner type, diameter, and pipe joint lengths to be used.
- The modified inlet system to be installed on the liner, if the liner inside the host pipe is inlet controlled (per the H&H calculations using the diameter and liner Manning's n).
- The H&H calculations and design for the outlet erosion protection per Appendix F.
- Proposed method of re-establishing connection(s) to associated structures.
- Joints, gaskets, proposed resins, coatings, and other pertinent information as applicable.
- Written confirmation that the contractor has coordinated grouting procedures with the grout installer and the liner pipe manufacturer.

- Plan for addressing buoyancy during grouting.
- Method for preventing damage to the liner pipe using guide rails, pipe invert paving, or other applicable methods to assist with liner placement.
- The method of testing of the liner pipe (per the applicable ASTM) for 0% leakage between the beginning and ending of the liner pipe as it relates to the dam or levee system.
- Evidence that the chosen pipe can handle the anticipated loads. .
- Detailed cross-section of the levee with the liner pipe installed and labeled.
- Construction schedule along with anticipated start and completion dates.
- Photos taken before, during and after rehabilitation.

Decommissioning:

- The existing pipe video to determine its overall condition after cleaning and prior to filling
- The calculated quantity of grout needed to completely fill the pipe.
- Mix design of material to fill the pipe (reference EM 1110-2-2902 for example).
- Verification that USACE Hydrology and Hydraulics staff was consulted to determine if a pipe can be permanently decommissioned based on not creating any adverse flooding conditions within the leveed area.
- Photos taken before, during and after decommissioning.

Associated Structures and Appurtenances for Levees:

- **Connections**
    - Construction plans and specifications with details of the appurtenance/associated structure, the connection between the pipe and the structure, and applicable manufacturer information.
- **Pumps**
    - Design calculations for pumps showing they have the capacity to meet the demand and have the appropriate backflow prevention measures in place and the appropriate factor of safety is met.
- **Gatewells**
    - Verification that the design satisfies the requirements of EMs 1110-2-2100 and 1110-2-2104.
- **Outlet Measures**
    - Design calculations for the outlet velocities at the headwall to indicate that erosion control meets all requirements.
- **Gates**
    - Design calculations for gates showing that they can withstand the total differential head (seating or un-seating) and plans showing actuators, stem guides, shafts, bolt patterns, coatings, etc.

THIS PAGE INTENTIONALLY LEFT BLANK

Glossary
Abbreviations and Terms

AASHTO	American Association of State Highway and Transportation Officials
ACI	American Concrete Institute
AEP	Annual Exceedance Probability
ASCE	American Society of Civil Engineers
ASD	Allowable Strength Design
ASDSO	Association of State Dam Safety Officials
ASTM	Also ASTM International, formerly American Society for Testing and Materials
AWWA	American Water Works Association
BWCP	Bar-Wrapped Cylinder Pipe
CAP	Corrugated Aluminum Pipe
CCTV	Closed Circuit Television
CD	Compact Disc
CECW-EC	Corps of Engineers, Civil Works, Engineering and Construction
CFR	Code of Federal Regulations
CIP	Cast Iron Pipe
CIPP	Cured In-Place Pipe
CiPCP	Cast-in-Place Concrete Pipe
CIRIA	Construction Industry Research and Information Association
CLSM	Controlled Low Strength Material (a.k.a. flowable fill)
CPP	Concrete Pressure Pipe
CMP	Corrugated Metal Pipe (See CSP)
CSP	Corrugated Steel Pipe (a.k.a. Corrugated Metal Pipe, CMP)
DDM	Direct Design Method

DDR	Design Documentation Report
DIP	Ductile Iron Pipe
DOT	Department of Transportation
DPP	Drilling Program Plan (for HDD)
DVD	Digital Video Disc
EC	Engineer Circular
EM	Engineer Manual
ER	Engineer Regulation
FE	Finite Element
FEMA	Federal Emergency Management Agency
FRP	Fiberglass Reinforced Pipe
fpm	feet per minute
HAZUS	Hazards U.S.
H&H	Hydrologic and Hydraulic
HDPE	High Density Polyethylene Pipe
HDD	Horizontal Directional Drilling
ICW	Inspection of Completed Works
IDM	Indirect Design Method
IPM	Interior Ponding Mode
LP-RCP	Low-pressure Reinforced Concrete Pipe
LRFD	Load and Resistance Factor Design
MAB	Municipal Advisory Board
MIC	Microbiology Influenced Corrosion
NASSCO	National Association of Sewer Service Companies
NCPI	National Clay Pipe Institute

NCSPA	National Corrugated Steel Pipe Association
NLD	National Levee Database
O&M	Operation and Maintenance
PACP	Pipeline Assessment Certification Program
PCC	Portland Cement Concrete
PCCP	Pre-stressed Concrete Cylinder Pipe
pcf	pounds per cubic feet
PE	Polyethylene
PFM	Potential Failure Mode
PFMA	Potential Failure Mode Analysis
psf	pounds per square foot
PVC	Polyvinyl Chloride
PW	Profile Wall
RCB	Reinforced Concrete Box Culvert
RCP	Precast Reinforced Concrete Pipe
RCCP	Reinforced Concrete Cylinder Pipe
RCNP	Reinforced Non-cylinder Pipe
RMC	USACE Risk Management Center
ROW	Rights-of-Way
SIPP	Spray In-Place Pipe
SRTP	Steel Reinforced Thermoplastic Pipe
SW	Solid Wall
WSSP	Welded Seem Steel Pipe
USACE	United States Army Corps of Engineers
USDA	United States Department of Agriculture

USC	United States Code
UV	Ultraviolet
VCP	Vitrified Clay Pipe

Acceptance Testing: Measurements, assessments, measurements, and observations made to determine if the requirements of a specifications or contract have been met.

Action Stage: The first stage, usually defined by a river gauge, which requires an operator to perform a task for flood risk management purposes.

Active Gates: Non-automatic gates that require manual operation.

Actuator: A mechanism, commonly found on sluice gates, which allows the user to operate the gate into a closed or open position.

Adjacent to (Pipes Running along Embankment or Floodwall Toe): Pipes located approximately parallel to the embankment centerline; they do not cross the embankment.

Air Vent: An opening in the top of a levee pump station discharge line, in which the pipe system does not operate as a siphon, preventing backflow. In a dam control tower, it is the opening in the roof of the conduit, adjacent to the service gate, which opens to the atmosphere, typically above the probable max flood.

Allowable Stress Design: A design method that does not place a safety factor on design loadings in calculations using the strength of the material.

Angle of Repose: The angle of repose is the steepest angle relative to the horizontal plane to which a material can be piled without slumping. At this angle (ranging from 0° to 90°), the material on the slope face is on the verge of sliding.

Annual Exceedance Probability (AEP): The probability that a certain threshold may be exceeded at a location in any given year, considering the full range of possible values, an if appropriate, incorporation of project performance.

Appurtenance: A device within an associated structure.

Associated Structure: A structure directly connected to a pipe that is constructed to serve a project function.

Bearing Strength: The non-destructive limit of pipe load, as determined by 3-edge bearing test method, used to determine field supporting strength.

Bedding: Bedding is what the invert of the pipe is placed on. Note, bedding must not be more permeable than the embankment or surrounding material.

Bedload: Pertaining to this document, the term bedload describes particles in a flowing fluid (usually water) that are transported along the bed (invert) of the pipe.

Beneath (Pipe in Foundation): A pipe installation location that is under the original ground surface regardless of depth.

Bending: A deformation of the pipe before buckling.

Billet: A type of rounded weld.

Blockout: To create a box or barrier within a formed area to prevent the concrete from entering another area.

Buckling: A material failure of a pipe under a loading.

Bulkhead: A barrier placed on the end of a pipe segments used to retain material. Typically bulkheads are used when slip lining or decommissioning a pipe.

Buoyancy: An object becomes buoyant when an upward force exerted by a fluid opposes the weight of an immersed object. The danger associated with buoyancy is that it can move or destabilize structures (pipes, pump stations, gatewells, manholes, etc.).

Casing Pipe: An exterior pipe that is used to enclose and surround a carrier pipe. Casing pipes are commonly used to install or protect utility lines. This is sometimes also referred to as a double wall pipe.

Catch Basin: An associated structure that allows the inflow of surface runoff into a drainage pipe.

Cavitation: The formation of vapor cavities in a liquid, small liquid-free zones, which are the consequence of forces acting upon the liquid. It usually occurs when a liquid is subjected to rapid changes of pressure that cause the formation of cavities in the liquid where the pressure is relatively low. Cavities result in section loss, decreasing strength of the material.

Closure: An opening in the embankment/floodwall that allows traffic (either vehicle or pedestrian) to pass through during normal conditions. During flooding, it is closed with a structure or sandbags to prevent flooding of the interior. Floodwall closures typically have slotted recesses in the walls for metal components as well as a closure sill for anchorage (typically a concrete slab extending the entire width of the closure).

Cofferdam: A temporary watertight structure built to exclude water from a construction site.

Composite Gate: A gate made of more than one type of material.

Control Tower: An associated structure, usually related to a dam that houses the gates to control the water level of the reservoir.

Controlled Low Strength Material (CLSM): CLSM refers to a mixture of Portland cement, fly ash, water, and admixtures (2) that forms a low-shrinkage soil replacement material with flowability characteristics that allow it to spread easily at relatively low pressures within trenches.

Counterfort: A reinforced concrete structure built against the floodwall to counteract flood loading. In this manual, the counterfort provides stability to a wall whose structural integrity may have been compromised by installing a large diameter pipe through the stem.

Coupling: A device used to connect pipe segments.

Crack: A break line is visible on the surface but is not visibly open and not gap is visible between the edges of a crack.

Dead Load: The load imposed on the pipe that is determined by depth and width of the trench at the top of the pipe, as well as the unit weight of the backfill material.

Decommissioning (Decommissioned): Withdrawing a structure in an embankment or foundation from service by acceptably filling in with an approved material. The structure and embankment/foundation conditions must meet the requirements outlined in Chapter 8 of this manual.

Deflection: The decrease of the vertical diameter of a flexible pipe, normally expressed in percentage.

Delivery Line: High pressure hose or rigid pipe utilized for infilling pipes or other structures to be decommissioned. In general, the smallest lines that will not generate excessive backpressure due to friction are recommended.

Designer: The engineer responsible for performing relevant calculations and developing the plans and specifications.

Direct Design Method: A design method for determining pipe size and thickness for expected loadings.

Direct Pipe: Direct Pipe is a proprietary trenchless installation method that combines features of HDD and utilizes a MTBM connected to the leading edge of an assembled length of pipe and a pipe thruster to jack the pipeline into place, similar to, but in the opposite direction of HDD pullback operations. As with microtunneling, the slurry collection/recycling system and pressure control features at the excavation face reduce the potential for drilling fluid loss. Direct pipe installations can be launched from the surface or from a shallow shaft and navigated along a steeper-sloped route than is typically achieved with traditional micro-tunneling/pipe jacking methods. There is no current ASCE MOP for EPB Pipe Jacking.

Drilling Fluid Flow Factor: Factor applied to the volume of solids when determining volume of drilling fluid required for good bore cleaning. The flow factor accounts for the affinity of an earth formation for water (e.g. clay has an affinity for water and expands into the bore as it takes on water).

Drilling Program Plan: A report that is required prior to any drilling, sampling, grouting, or any other invasive in-situ testing or exploration, including activities related to investigation, maintenance, and remediation.

Dual Wall Pipe: Typically a thermoplastic pipe whose wall that has two different types of profiles (corrugated exterior and smooth interior). Dual wall pipe is used when additional stiffness is required (corrugated side) but flow characteristics associated with a smooth surface are needed. This is also commonly referred to as profile wall pipe.

Duckbill: A passive gate appurtenance made of an elastomer fitted at the end of a pipe. The end of the appurtenance is rolled which allows gravity flow during normal conditions, but will block flow during flooding conditions.

Elevated Pipes: Pipes above the ground surface, which includes any exposed pipes higher than the cross-sectional design of the embankment or covered pipes that increase the height of surface geometry.

Embankment: Typically a trapezoidal-shaped mound of soil built in lifts and compacted to a minimum relative density for the purpose of retaining a reservoir (a dam built perpendicular to the flow of water) or to prevent floodwater from causing consequences (a levee built parallel to the flow of water). Embankments may be homogeneous or zoned. A zoned embankment may have a central core of low-permeability soil flanked by a higher-permeability material, such as gravel and stone to protect the core.

Embankment Load: The load imposed on the foundation of a levee or dam due to the weight of the embankment material.

Emergency Gate: A type of active that gate commonly found in dam projects that require personnel to close during service gate failure.

Encasement: Special materials, their placement and configuration which are designed to fully surround the pipe, and develop a field supporting strength which exceeds that developed by other commonly used installation and bedding techniques.

Essential Pipes: Pipes necessary for the operation of a project such as those that function to regulate reservoir elevations, provide interior drainage using gravity and pump station discharge systems, and provide pressure relief by intercepting seepage (toe drains or relief well collector system pipes).

Event Tree: Graphic consisting of a series of nodes, branches, and endpoints that describe the sequence of events that occur from initiation of a failure mode to a breach.

Exfiltration: The process of water migrating or leaking out of a pipe. The process poses a risk to the dam or levee since the escaping water may erode embankment soil over time, putting the levee at risk of failure.

Flap Gates: A type of passive gate that uses the water head differential to seal a pipe against backflow.

Flood Fighting: Physical measures taken before and during a flood to maintain existing flood risk management projects distressed by the flood loading.

Floodwall: A concrete or steel structure used as a barrier. Floodwalls typically come in one of three configurations; I-Wall, T-Wall, and L-Wall. I-Walls are vertical gravity walls, usually constructed with sheet piling embedded to a depth for seepage and stability. L-Walls and T-Walls (inverted T-type) can also be installed with sheet piling, but are only needed as a seepage measure.

Foundation: The soil or rock of the original ground surface beneath the embankment prism.

Fracture: A break line that has become visibly open and a gap can be seen although the pipe walls are still in place and not able to move.

Frac Out: See inadvertent return.

Gatewell: An associated structure that houses active or passive gates and can provide a working platform in order to operate active gates during a flooding event.

Geotextile: A drainage fabric which allows the flow of water while preventing the movement of soil.

Gravity Pipe: A non-pressure pipe designed with the intent to pass flow by using a shallow slope during normal conditions. During flood conditions, gravity pipes are normally closed off with a gate.

Grout: This is a Portland cement based "flowable" material used to fill the annular space between a host and liner pipe or decommission a pipe.

Haunch: The areas bordered by the barrel outside diameter, the vertical tangents from the pipe springline, and the horizontal tangents from the bottom of the pipe.

Headwall: An associated structure used to recess the inflow or outflow end of a pipe into the fill slope to improve inlet and outlet flow conditions, anchor the pipe, and control erosion and scour from the pipe outflow area

Home the Joints or "Homing": The process of assembling a bell-and-spigot type joint. One pipe is pushed into the other to the proper depth, so the gasket is fully engaged, but the joint is not damaged from pushing too deep.

Hydraulic Head: The height difference between the design flood and the pipe invert elevation at the gate location, causing water pressure on the gate.

Inadvertent Return: The unintended transfer of drilling mud to the surface during boring machine operations by way of fractures or fissures that occur naturally, rather than as a result of boring operations.

Indirect Design Method: A widely used empirical method for selecting and specifying pipes, using D-Loads.

Infiltration: The process of soil-bearing water migrating or leaking into a non-pressurized pipe. It poses a risk to the dam or levee since the loss of soil into the pipe may, over time, put the levee at risk of failure. This should have been defined in Chapter 2 PFMs.

Influence Zone: The influence zone is the area where defects in a pipe could impact the integrity of a dam or levee or restrict access for O&M or emergencies. Reference Section 6.3 for zone limits.

Inlet Controlled: A pipe flow condition where flow leaves the pipe faster than it can enter.

Interim Risk Reduction Measure: Effective, interim actions taken to reduce flood risks while longer term solutions are planned and implemented.

Intervention: Actions taken to prevent an embankment or floodwall breach. Intervention efforts are possible at all stages of a failure mode, and, in extreme cases, can continue after a breach to reduce the severity of flooding. Successful intervention does not mean that the structure will not require later rehabilitation or repairs.

Jetting: A maintenance procedure that uses high pressure water to clean debris and settlement out of a pipe.

Leveed Area: The area behind a levee system or segment that has a reduced risk to damages from flooding because of the construction of the levee.

Levee Sponsor: A public entity that has responsibility for operation and maintenance of a levee system or discrete portion of a levee system.

Levee System: A manmade barrier that does not cross a watercourse with the principle function of excluding flood waters from a limited range of flood events from a portion of the flood plain (referred to as "leveed area"). A levee system is composed of one or more levee segments and other features that collectively are integral to excluding flood waters from the leveed area.

Liner Pipe Mandrel: (Not to be confused with New Pipe Mandrel) A mandrel using the same pipe material and joint length of the proposed slip liner where the diameter must be a minimum of two inches greater than the proposed slip liner pipe outside diameter.

Load and Resistance Factor Design (LRFD): A design method that does place a safety factor on design loadings in calculations using the ultimate strength of the material.

Mandrel: A specified pipe joint segment manually inserted into a host pipe to determine if the diameter and alignment of the host pipe is sufficient to allow insertion of the slip liner pipe.

Manhole: An associated structure that provides access to the pipe. Manholes associated with levees can also contain active gates necessary for operation during flooding scenarios.

Manually Operated: Typically applies to active gates and refers to the need for project personnel or maintenance to physically perform an action to open or close the gate at the gate's location. The action can include a manual hand cranking mechanism or electronic actuator with controls located at the gate or a remote location

New Pipe Mandrel: (Not to be confused with Liner Pipe Mandrel). A new pipe mandrel is made of a nine-armed, non-adjustable, fixed arms where the diameter is not less than the allowable percent deflection of the pipe being inspected.

Non-federal sponsor: Refers to a non-federal interest, as defined in the Flood Control Act of 1970, as amended (42 USC 1962d-5b (b)), that has provided assurances or executed a binding agreement for the provision of items of local cooperation for a USACE project, including, as applicable, operation and maintenance.

Non-Essential Pipes: Pipes not listed as essential that typically serve as utility crossings for water mains, sanitary sewers, petroleum products, natural gas, electrical, fiber optic, and other services and typically referred to throughout this manual as third-party pipes.

Open-Shield Pipe Jacking: Open-shield pipe jacking refers to a type of steerable tunnel boring machine (TBM) in which the cutting face of the shield is open, allowing for man-entry and direct access to the soil face. Excavation may be by hand or by mechanical processes and will typically result in a small overcut either from excavation tooling that leads the casing or from a shield with an overcut ring. Spoil removal can be by manual or machine methods (e.g. conveyor, muck skip). Open-shield tunnel boring machines are developed for tunnel excavations or milling in various earth conditions. Due to the open and visible working face, the soil can be mined and documented easily. To facilitate pipe installation, the soil is overexcavated. Lubrication fluids consisting of a mixture of water and bentonite or bentonite/polymer are pumped into the annular space (between the casing and earth) through pre-installed injection ports to facilitate pipe installation. Open shield pipe jacking is suited for a wide range of earth materials; however, is not well suited where groundwater is present. There is no current ASCE MOP for open-shield pipe jacking.

Overbuilt Pipes: Pipes that are located on the surface of an embankment and covered with compacted embankment material to protect the pipes from live loads and frost heave.

Overbuild: Embankment material that is placed over pipes to allow the passage of pedestrians and vehicular traffic.

Overtopping/Overtopped: When water levels exceeds the top of an embankment or floodwall and begins to enter into the leveed area. The overtopping may be continuous (still-water elevation higher than the crest of structure) or intermittent (caused by waves or surges). Overtopping does not necessarily result in a breach.

Passive Gates: Gates that do not require an operator, more or less automatic using flood waters to seal the gate to a surface and preventing flow.

Penstock: An enclosed pipe that delivers water to hydro turbine systems.

Pipe Cradle: A reinforced concrete pad placed beneath the pipe before installation to provide support for the pipe.

Pipe Condition Assessment Rating: The rating given to a pipe by a qualified USACE assessor, taking into consideration the pipe defect ratings, location of defects relevant to the levee, and likelihood of the defects to compromise the integrity of the levee.

Pipe Ramming: A trenchless method for installation of pipes, typically steel.

Pipe Supports: Non-rigid means of elevating a pipe above the trench bottom so that placed CLSM backfill will flow beneath the pipe. Sandbags are commonly used.

Potential Failure Mode: A mechanism, regardless of its probability of occurring, which has the potential to progress from a flaw to a breach of an embankment or floodwall. Overtopping without breach is not considered a potential failure mode.

Probable Maximum Flood: The most severe flood that is considered reasonably possible at a site as a result of meteorological and hydrologic conditions.

Profile Pipe: A pipe that follows the cross section of the embankment, installed parallel to both slopes and the crest. It may be placed over or within the embankment.

Project: Project refers to a dam or levee system within the USACE portfolio of projects.

Real Property: Any interest in land, including leaseholds, easements, and rights of-way, together with the improvements, structures, and fixtures located thereon.

Rehabilitation: Permanently restore a pipe to its original capability.

Restrained Joint: A pipeline connection that allows longitudinal rotational movement but not longitudinal axial movement. [Howard, 2015] The joint can withstand internal pressure without separating longitudinally.

Ring Deflection: The change in a pipe's diameter, expressed as a measurement or percentage, from its manufactured diameter (sometimes referred to as ovaling). The maximum ring deflection is typically a reduced measurement on the vertical axis due to soil loading above the pipe, but could also be an increased measurement on the horizontal axis as the pipe deforms into the surrounding soil.

Risk: A measure of the probability and severity of undesirable consequences.

Risk Assessment: Analysis that defines the nature of the risk, its probability, and the consequences, either quantitatively or qualitatively (or a combination).

Right-of-Way: The legal right, established by usage or grant, to pass along a specific route through ground or property belonging to another.

Schedule of Operation: A map or table indicating actions required based on the flood stage elevation for the project. Actions can include inspecting a passive gate, closing/opening an active gate, installing a road closure, operating a pump station, or constructing a sandbag wall.

Seat (Seating) Head: Head measured from the surface of the water to the centerline of the gate that exerts force against the gate to push the seats (the part of the gate where the cover slides against or contacts the frame) together.

Service Gate: A type of active that gate commonly found in dam projects that require personnel to raise and lower in order to control the elevation of the reservoir for flood storage or normal operations.

Service Life: A term used to represent the amount of time a pipe is in use before it reaches a state of failure. Similar terms include "design life" and "operating life." These are considered interchangeable terms for the purposes of this document.

Shaft Spillway: An associated structure constructed as a vertical or sloping shaft (conduit) with a bell-shaped inlet that passes water beneath the dam to the tailwater to reduce the probability of overtopping.

Siphon Breaker: An appurtenance on a pump station discharge pipe, usually equipped with an operating valve that opens the pipe to the atmosphere to prevent backflow. Required on pipe systems that operate as a siphon

Slide Gate: A type of active gate that uses an actuator to traverse the threads of a metal stem or rod to open or close the gate. Installed without wedges.

Slip lining: The installation of a smaller pipe into a larger host pipe, grouting of the annular space between the two pipes and sealing the ends. This is considered a trenchless rehabilitation in this manual.

Sluice Gate: A type of active gate that uses an actuator to traverse the threads of a metal stem or rod to open or close the gate. Installed with wedges for leak resistance.

Special Fittings: Pipe devices other than standard pipe segments and their joints, such as taps, wyes, tees, valves and gates, and transitions to other pipe sizes or material types.

Springline: The vertical midpoint of a pipe. External forces below the springline (groundwater pressure, soil pressure) work to lift the pipe while those above the springline work to drive it downward. The springline is an important aspect of a pipe when considering encasing the pipe in soil, since soil compaction below this point is very difficult and improper techniques can jack the pipe out of position.

Stirrups: Reinforcement, typically in concrete, used to resist shear and torsional forces.

Stoping: Stoping is common when soil is removed below the ground surface (typically through an unfiltered exit), creating a growing void that works its way upward as the sides shear away and the roof incrementally collapses to the bottom of the void where the material is carried away. These voids can develop in both saturated and unsaturated environments and eventually result in the opening of a sinkhole on the ground surface.

Surge Tank: An associated structure placed at the downstream end of a penstock that absorbs sudden rises of pressure as well as providing extra water during brief drops in pressure.

Thimble: A short piece of pipe with flanged ends.

Third-Party Pipes: Pipes owned and maintained by parties not affiliated with the dam or levee owner or with USACE. These pipes often transport pressurized fluids such as water, fuel, or natural gas or communication cables.

Through Pipe: Refer to within embankment pipes. A pipe that goes through the embankment.

Thrust Block: A concrete block cast between a pipe and the existing soil to resist the thrust created when the pipe changes directions or changes size. (Howard, Pipe Installation 2.0, 2015)

Transition: Locations within a dam structure or levee system where there is a change in geometry, material type or between structure types.

Trenchless: An installation or rehabilitation technique that avoids the need to open excavate the soil for a pipe.

Unfactored Loads: Design loads that are not altered by a safety factor.

Unseat (Unseating) Head: Head measured from the surface of the water to centerline of gate that exerts force on the gate as to pull the seats (the part of the gate where the cover slides against or contacts the frame) apart.

Utility Carrier: A larger pipe used to encompass a few to several smaller pipes associated with a utility (i.e. electric wiring, fiber optic cables, networking cables, etc.)

Valve: An appurtenance that comes in a variety of mechanisms with the primary purpose to shut off pipe flow on demand.

Vitrification: Clay mineral particles are heated to high temperatures to become mechanically bonded into an inert, chemically stable compound.

Water Hammer: A pressure surge or wave caused when a fluid in motion is forced to stop or change direct suddenly.

Watertight: A pipe wall or joint that does not allow the passage of water through it.

Within Embankment Pipe: This term is used to describe the location of pipes which are found within or passes through the dam or levee design cross-section.