from 1,000 to 3,000 feet. CIPP liners are best suited for existing conduits that are not severely damaged or deformed. For guidance on the use CIPP in conduit renovation applications, see section 12.2.

2.3 Metal

Metal pipes used in the construction of conduits have included:

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

These materials are discussed in the following sections.

2.3.1 Steel

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

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

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

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

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

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

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

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

- High compressive and tensile strength.

- Flexible and deformable under stress.

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

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

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

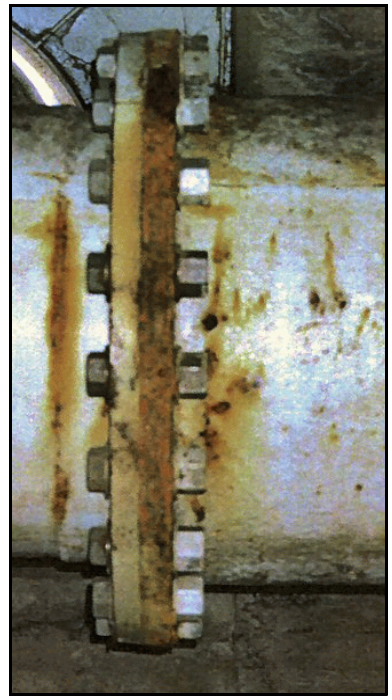

Figure 28.—Watertight joints can be provided for steel pipe by use of flange connections.

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

- Flanges provide a rigid connection to gates and valves.

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

The disadvantages of using steel pipe for conduits include:

- High material costs.

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

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

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

2.3.2 Ductile-iron.

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

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

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

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

The disadvantages of using ductile-iron pipe include:

- Ductile iron pipe is heavy and makes handling difficult.

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

- Requires cathodic protection in corrosive soils.

2.3.3 Cast-iron.

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

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

The disadvantages of using cast-iron pipe include:

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

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

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

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

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

2.3.4 CMP

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

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

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

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

The disadvantages of using CMP for conduits include:

- Deterioration has resulted in many embankment dam failures.

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

Figure 31.—A CMP conduit being installed.

- Joints are often incorrectly assembled in the field, resulting in nonwatertight joints.

- Not applicable for pressurized conduits due to lack of watertight joints.

- CMP is considered a flexible pipe, and flexible pipe design requires the earthfill surrounding the pipe to provide structural stability and support to the pipe. If the surrounding backfill does not provide adequate support, flexible pipes are subject to distortion and deflection.

- Circular shape and corrugations on the exterior surface makes compaction of the earthfill against the conduit difficult to achieve.

Chapter 3

Hydraulic Design of Conduits

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

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

3.1 Outlet works

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

3.1.1 Arrangement of control features

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

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

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

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

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

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

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

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

- Arrangement 4—Downstream control (figure 36)

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

3.1.1.1 Arrangement 1—Intermediate control with downstream access

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

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

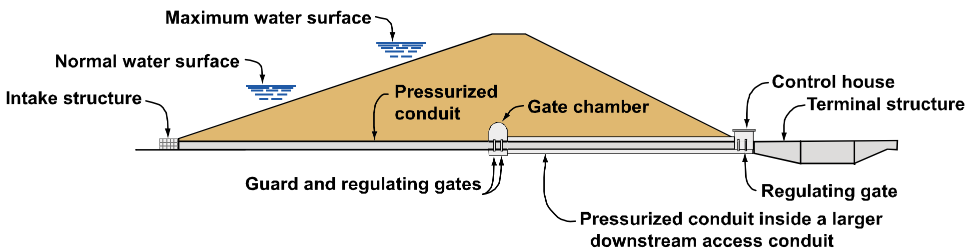

Figure 33.—Arrangement 1—Intermediate control with downstream access.—The control feature is located at an intermediate point within the conduit.

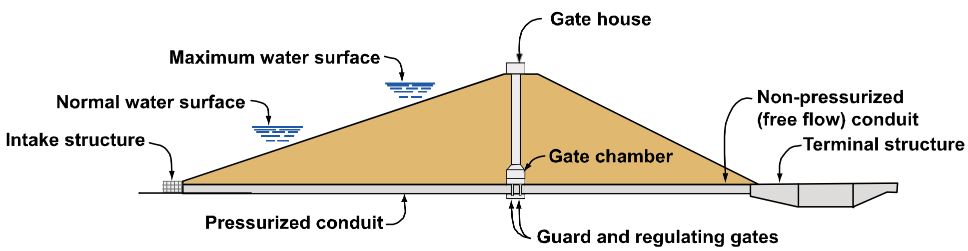

Figure 34.—Arrangement 2—Intermediate control without downstream access.—The control feature is located at an intermediate point within the conduit.

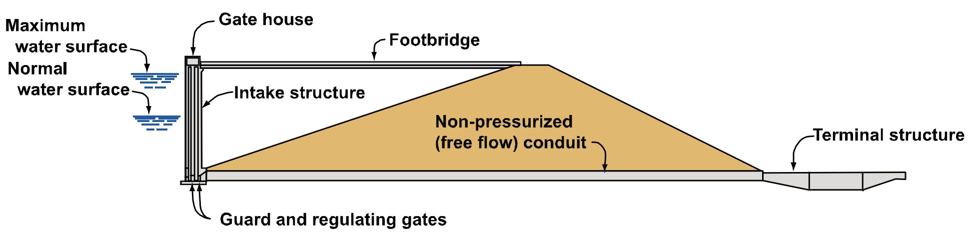

Figure 35.—Arrangement 3—Upstream control.—The control feature is located at the upstream end of the conduit.

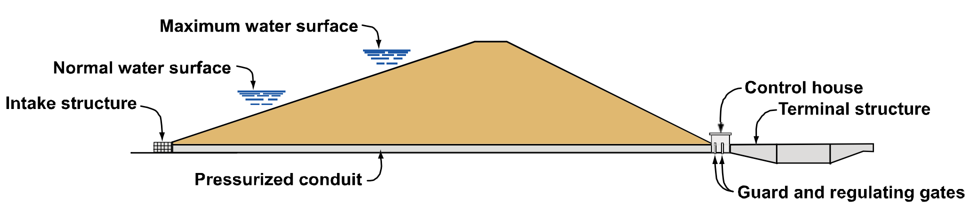

Figure 36.—Arrangement 4—Downstream control.—The control feature is located at the downstream end of the conduit.

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

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

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

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

3.1.1.2 Arrangement 2—Intermediate control without downstream access

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

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

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

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

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

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

3.1.1.3 Arrangement 3—Upstream control

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

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

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

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

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

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

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

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

3.1.1.4 Arrangement 4—Downstream control

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

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

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

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

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

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

3.2 Spillway

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

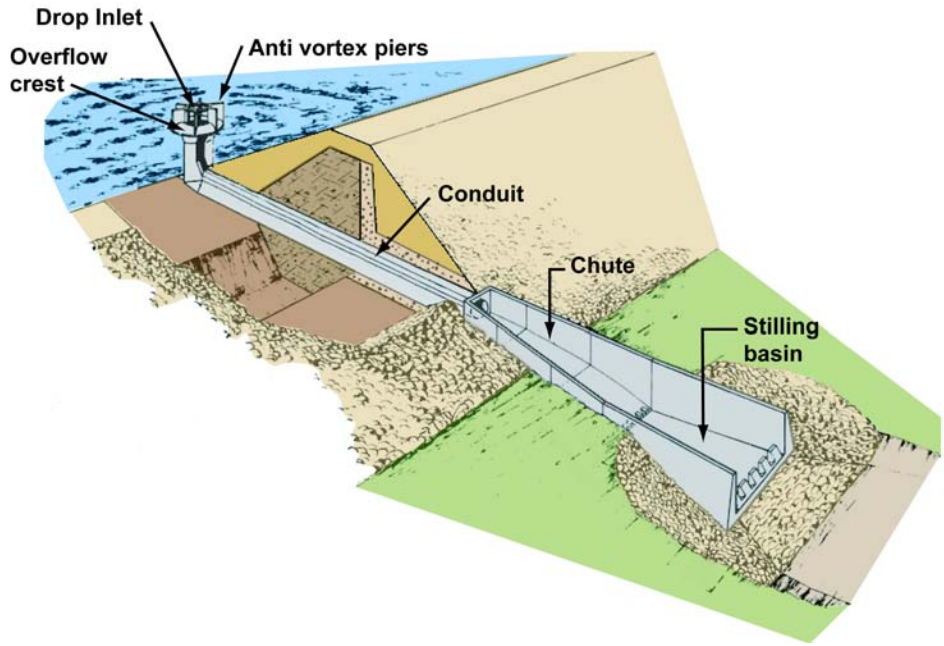

Figure 39.—A drop inlet spillway conduit through an embankment dam.

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

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

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

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

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

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

3.3 Power conduits

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

Figure 41.—Penstocks extending through an embankment dam.

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

3.4 Entrance and terminal structures

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

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

Figure 42.—Typical outlet works intake structure.

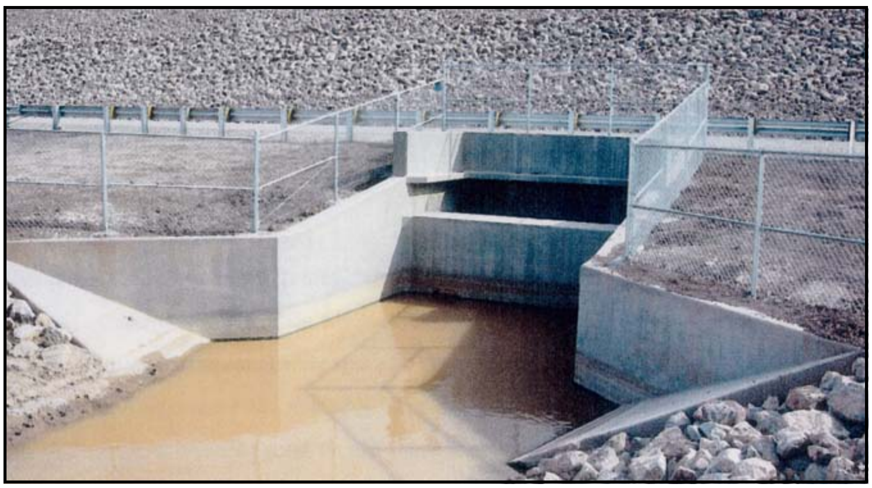

Figure 43.—Typical outlet works terminal structure.

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

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

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

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

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

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

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

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

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

Chapter 4

Structural Design of Conduits

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

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

- High velocity flow can result in cavitation or erosion.

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

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

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

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

4.1 Conduit shape

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

- To promote good compaction of earthfill against the conduit

- To eliminate or minimize the impacts of low density areas caused by difficulties in compaction of the earthfill

- To eliminate or minimize the potential for stress arching in the embankment dam leading to low density zones and hydraulic fracture of the dam

- To eliminate or minimize the potential for differential settlement leading to low density zones and maintain a positive embankment pressure on the conduit

- To allow access for periodic inspection by either man-entry or CCTV

- To allow for future repairs, renovation, or changes in operating requirements

- To allow for the most economical structural design while still addressing all of the items above considerations

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

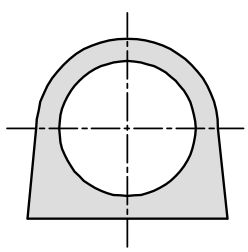

Figure 46.—Conduit shape A

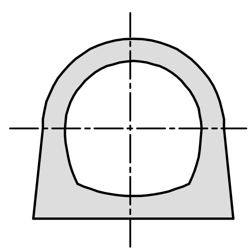

Figure 47.—Conduit shape B

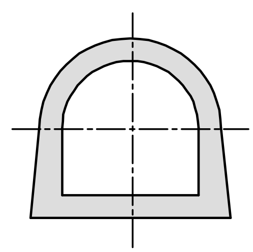

Figure 48.—Conduit shape C

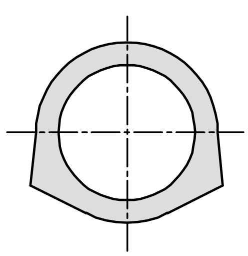

Figure 49.—Conduit shape D

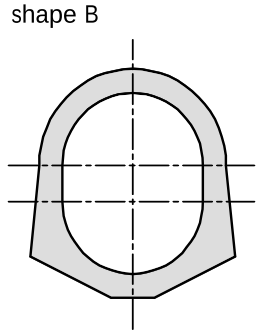

Figure 50.—Conduit shape E

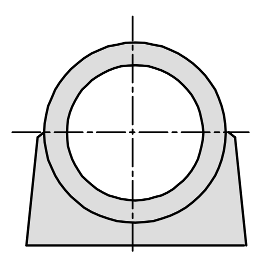

Figure 51.—Conduit shape F

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

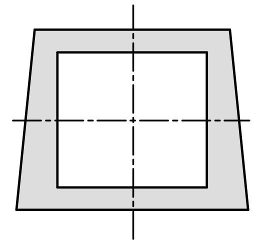

Figure 52.—Conduit shape G

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

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

4.1.1 Conduit shapes A, B, and C

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

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

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

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

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

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

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

4.1.2 Conduit shape D

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

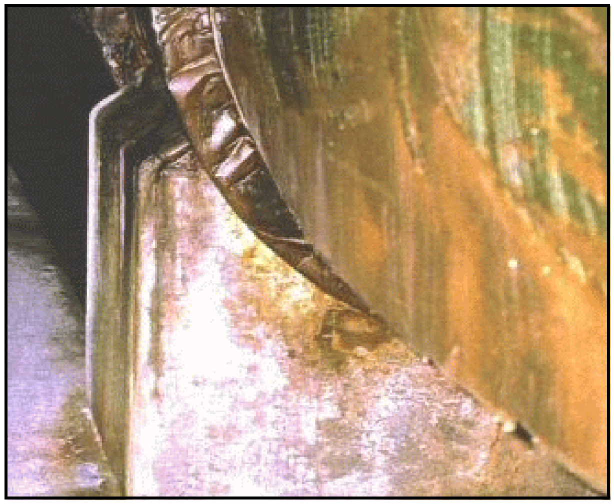

Figure 57.—Concrete saddles are used to support steel pipe located within a larger access conduit.

4.1.3 Conduit shape E

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

4.1.4 Conduit shape F

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

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

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

4.1.5 Conduit shape G

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

The advantages of a box shape include:

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

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

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

The disadvantages of a box shape include:

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

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

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

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

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

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

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

4.1.6 Cradles and bedding

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

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

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

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

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

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

Figure 60.—Precast concrete using bedding as support.

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

4.2 Structural design and construction considerations

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

Loading conditions typically analyzed include:

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

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

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

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

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

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

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

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

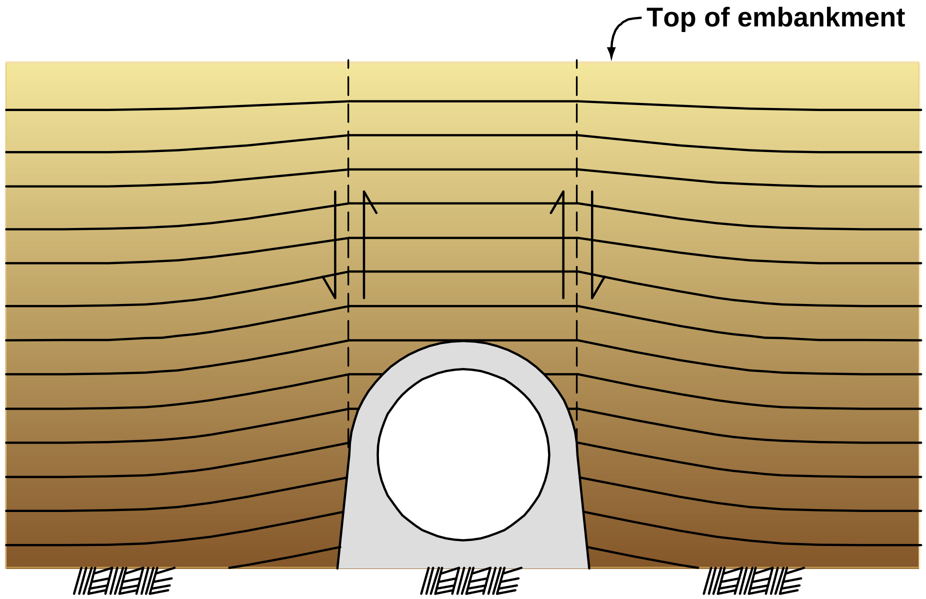

Figure 61.—Conduit constructed prior to earthfill placement. Friction factors increase embankment load on the conduit as adjacent earthfill settles more than earthfill overlying the conduit.

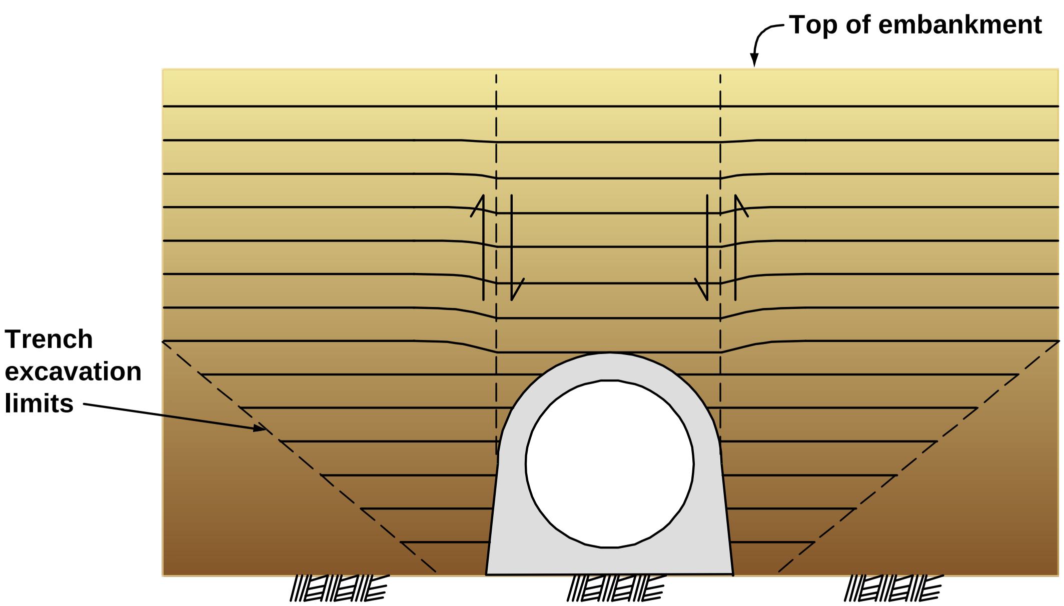

Figure 62.—Conduit constructed within a trench excavated into the embankment. Friction factors decrease embankment load on the conduit as earthfill over conduit settles downward relative to adjacent embankment.

4.2.1 Concrete

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

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

4.2.1.1 Reinforced cast-in-place concrete

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

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

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

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

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

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

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

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

> buildings to ensure long term durability in aggressive wet/dry, freeze/thaw environments.
>
> The NRCS is developing updated concrete design guidance under contract to a consultant. This guidance will adopt current ACI codes and explain their application to different NRCS concrete structures. The guidance will also include concrete joint design details and numerous example problems. The expected publication date is 2006.

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

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

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

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

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

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

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

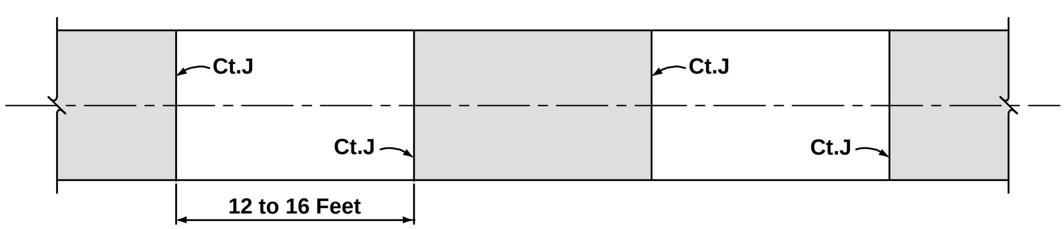

Figure 64.—Concrete placement for a reinforced cast-in-place conduit.

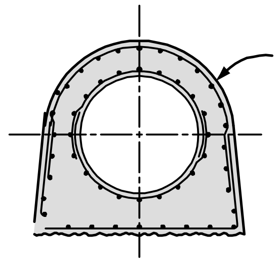

Figure 65.—Concrete placement for a reinforced cast-in-place conduit.

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

4.2.1.2 Precast concrete

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

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

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

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

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

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

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

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

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

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

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

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

4.2.2 Plastic

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

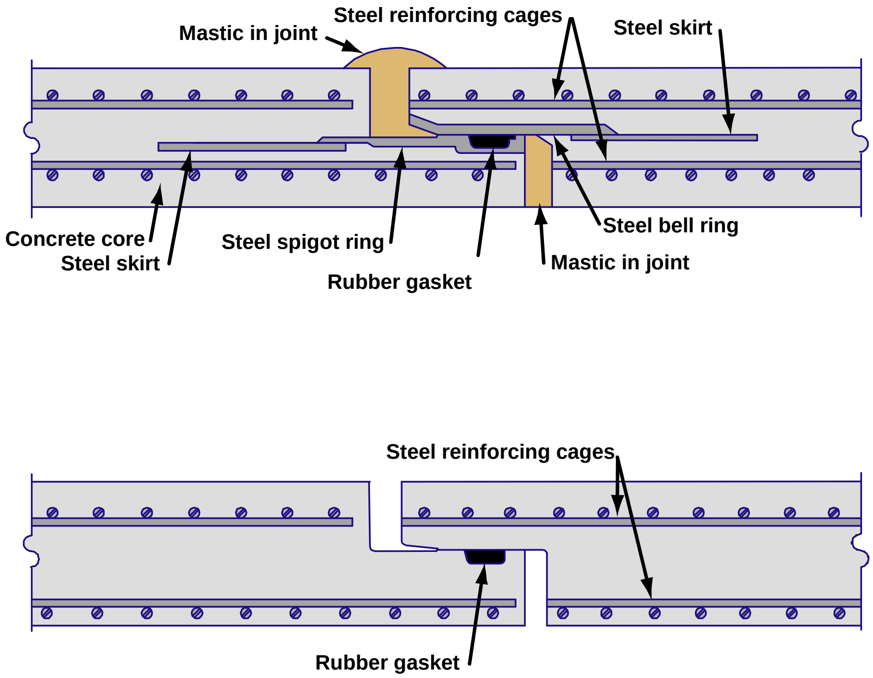

Figure 66.—Reinforced concrete pipe (RCP) details. The top figure illustrates pipe with steel joint rings, and the bottom figure illustrates a concrete joint.

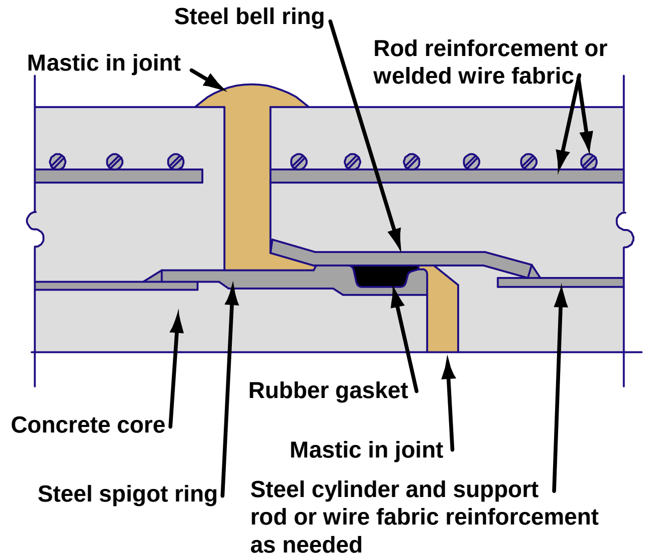

Figure 67.—Reinforced concrete cylinder pipe (RCCP) details.

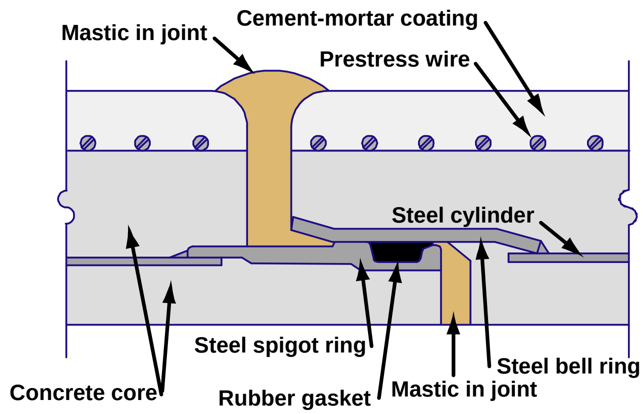

Figure 68.—Prestressed concrete cylinder pipe (PCCP) details (lined cylinder).

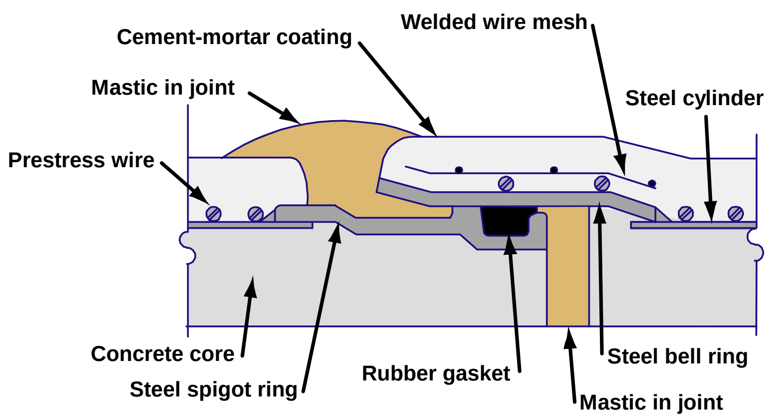

Figure 69.—Prestressed concrete cylinder pipe (PCCP) details (embedded cylinder).

For guidance on design and construction parameters pertaining to:

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

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

4.2.3 Metal

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

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

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

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

4.3 Watertightness

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

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

- Conduit joints
- Barriers within joints

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

4.3.1 Conduit joint

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

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

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

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

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

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

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

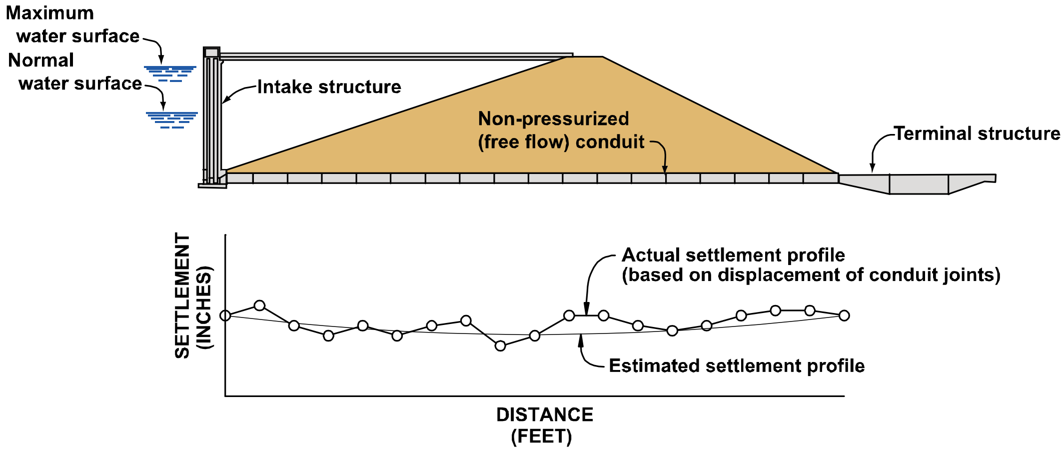

Figure 71.—Actual embankment dam settlement can differ from predicted settlement.

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

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

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

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

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

4.3.1.1 Reinforced cast-in-place concrete

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

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

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

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

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

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

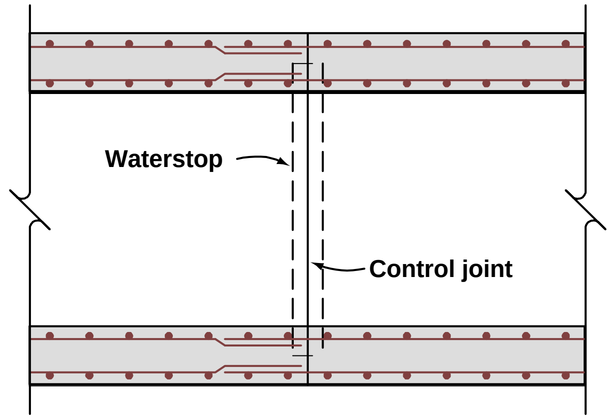

Figure 73.—Typical control joint used in reinforced cast-in-place concrete conduit construction. Longitudinal reinforcement is continuous through the joint.

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

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

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

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

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

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

The advantages of using temperature instruments include:

- They allow for improved timing of construction activities, resulting in shorter durations between placements.
- Temperatures in specific critical locations within the conduit can be measured.
- More accurate representation of concrete strength is possible.
- Strength measurements can be obtained any time.

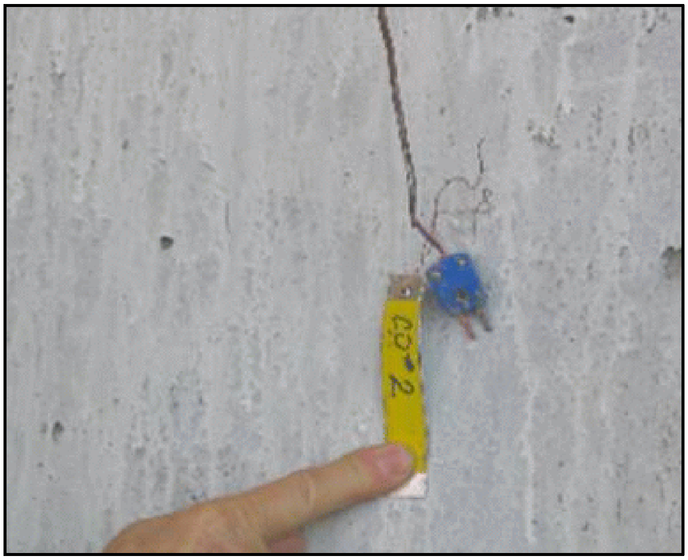

Figure 74.—An identifying tag used with a thermocoupler.

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

4.3.1.2 Precast concrete

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

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

4.3.1.3 Plastic

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

4.3.1.4 Metal

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

4.3.2 Barrier within joints

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

4.3.2.1 Reinforced cast-in-place concrete

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

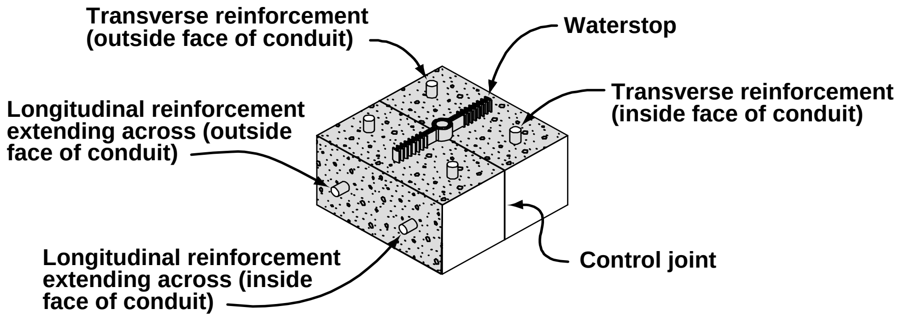

Figure 75.—Waterstop is placed across the joints of conduits to stop water from coming through the joint.

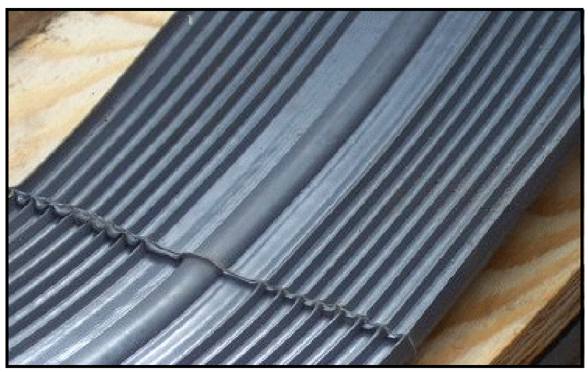

Figure 76.—Typical waterstop used in conduit construction to prevent the movement of water through joints. The ends of this waterstop have been spliced together.

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

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

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

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

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

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

4.3.2.2 Precast concrete

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

Chapter 5

Foundation and Embankment Dam

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

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

5.1 Excavation and foundation preparation

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

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

5.1.1 Rock foundation

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

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

5.1.2 Soil foundation

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

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

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

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

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

5.2 Cracking and hydraulic fracture of embankment dams

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

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

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

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

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

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

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

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

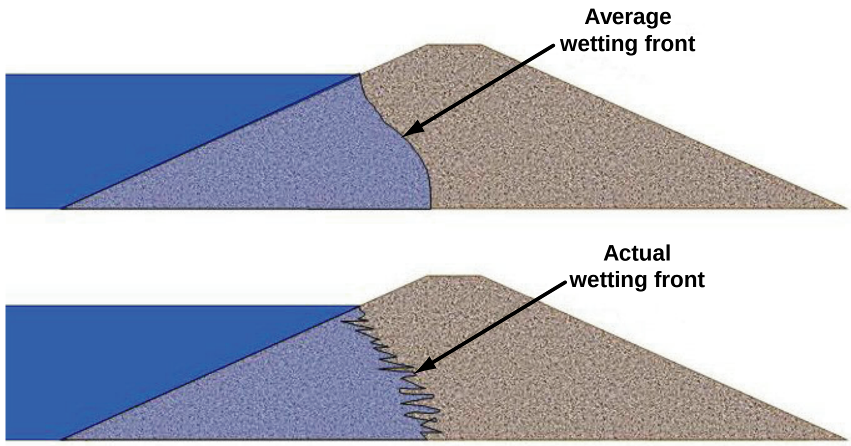

Figure 81.—On first filling, a high hydraulic gradient exists in the embankment dam as a wetting front moves through the dam. The wetting front will not be smooth. Projections will exist due to the different permeability of the embankment dam. The water pressure pushing against the soil can easily be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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

5.3 Selection and compaction of backfill

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

5.3.1 Selection of backfill material to be placed against conduit

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

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

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

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

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

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

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

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

5.3.2 Compaction of backfill material against conduit

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

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

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

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

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

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

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

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

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

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

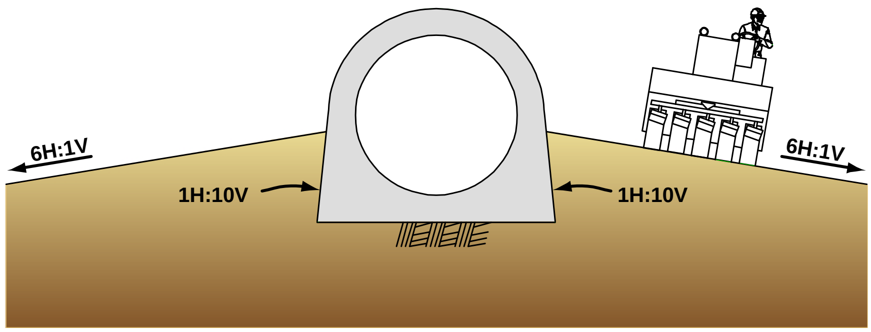

Figure 83.—Recommended earthfill ramp and conduit side slopes.

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

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

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

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

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

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

5.3.3 Dispersive clay backfill

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

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