Project name: Little Chippewa Creek Dam

Location: Ohio

Summary: Separation of spillway conduit joints due to foundation movement

The joints of the 48-inch reinforced concrete spillway pipe separated when foundation movement occurred during final stages of embankment construction. The failed spillway was removed, a new structure was constructed in a different location, and stabilizing berms were added to the embankment design.

Little Chippewa Creek Dam, known officially as "Chippewa Conservancy District Structure VIIc" is a high-hazard dam located about 3 miles northwest of the city of Orrville, Ohio. The 27-foot high embankment dam is a single-purpose, dry flood control structure designed in 1971 by the Soil Conservation Service under the authority of Public Law PL-566.

The embankment dam was designed with an upstream slope of 3H:1V and a downstream slope of 2.5H:1V. The site lies on the glaciated, moderately rolling Allegheny Plateau. The site was glaciated during a series of advances and retreats during the Wisconsin Stage of the Pleistocene Epoch. The foundation soils consist of glacial outwash deposits of layered sand, silt and clay.

Construction of the embankment dam started in July 1972. During spillway construction, the inspector noted the presence of soft, gray silt at the bottom of the excavation for the pipe. The foundation was overexcavated by 1 foot and backfilled with AASHTO No. 46 coarse aggregate. The pipe was installed on top of the aggregate and bedded in concrete. After the pipe joints were covered with 12-inch wide sheet metal shields, embankment material was backfilled around the pipe. Construction was suspended in late 1972 due to winter weather.

Construction of the embankment dam resumed in July 1973. Earthfill placement proceeded rapidly without incident until mid-August 1973. As the embankment dam was nearing completion, the downstream portion failed suddenly, severely damaging the spillway conduit. The downstream end of the conduit moved about 2.4 feet in the downstream direction. A 1.5-foot high bulge was observed in the stream bottom, and some cracks were observed in the slope below the dam. However, no settlement, cracking, or other distress of the embankment dam itself were observed.

The bottom of the pipe under the maximum earthfill height settled by about 1.5 feet, and the joints of one section of the pipe separated by more than 1 foot (figure B-49). The engineers investigating of the failure reported that the foundation soils under the

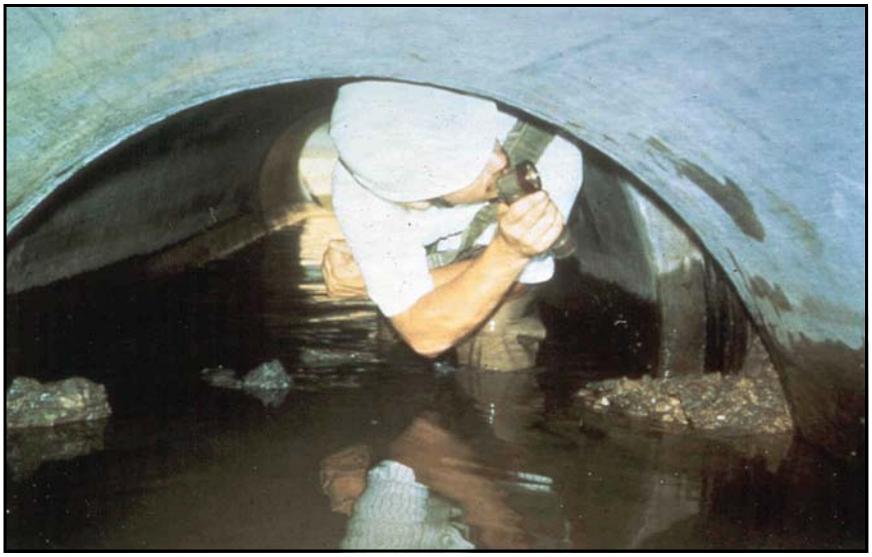

Figure B-49.—The pipe joints separated when foundation movement occurred during construction of the embankment dam.

pipe, visible through the open joints, were soft, wet silts that resembled "stiff grease." The engineer reported that no sands or gravels were encountered, and that the soil could be probed easily with a hand ruler to depths of 6 feet below the pipe. The report also indicated that the sheet metal shields on the outside of the joints appeared to support the embankment soils and minimized migration of soil into the open joints.

Because the nearly completed structure was capable of impounding water, but was unsafe, temporary modifications were required to prevent its failure during a storm. A 30-foot wide bypass channel was immediately excavated through the emergency spillway. Also, a 30-inch diameter CMP was temporarily installed inside of the failed concrete pipe to carry stream flow and to prevent flow of water over the open joints.

The permanent repairs included removal and relocation of the failed principal spillway conduit and appurtenances; reconstruction of the embankment dam in the vicinity of the failed spillway; installation of a new spillway at another location; addition of 70-foot-wide stabilizing berms to the upstream and downstream sides of the dam; addition of chimney drain across the reconstructed area of the embankment dam and around the new spillway pipe; and addition of a cut-off trench across the emergency spillway control section to seal the emergency breach channel.

Reconstruction of the embankment dam was successfully completed in late 1974.

Lessons learned

Embankment dams constructed on soft clay foundations may experience excessive settlement and spreading, and conduits associated with them may be damaged. Design of embankment dams on soft foundations must consider the undrained strength likely to be operative during construction and incorporate design measures, such as wide berms and special conduit joint details to address these problems.

Reference

Ohio DNR and *Phase 1 Inspection Report*, U.S. Army Corps of Engineers, 1981, which includes the 1973 "Report of Investigation of Structural Deficiency" as an appendix.

Project name: Loveton Farms Dam

Location: Maryland

Summary: Failure of an embankment dam by internal erosion along the spillway conduit

Loveton Farms Dam is a 23-foot high earth embankment dam in Baltimore County, Maryland. The dam was constructed in 1985 as a stormwater management structure to attenuate increased runoff due to commercial and residential development of the watershed. The embankment dam is a "dry structure" which does not normally impound any water. The spillway consists of large diameter (78-inch diameter) CMP constructed through the embankment dam. A vertical section of CMP about 16 feet high (riser) was constructed at the upstream end of the spillway pipe. Low flows pass through a small (1-foot) opening at the base of the riser. Flows in excess of the 100-year storm bypass the embankment dam via an emergency spillway channel excavated in the left abutment.

The embankment dam is essentially a homogeneous embankment constructed of local residual soils. These soils are micaceous silty fine sands and sandy silts weathered from the parent rock (Piedmont Geologic Province). They are classified as SM and ML under the Unified Soil Classification System. Typical liquid limits are about 30 percent, with a plasticity index of about 7.

The embankment dam failed less than a year after it was completed, when a relatively small storm filled the pool to the top of the riser. Failure was attributed to internal erosion of embankment fill along the outside of the pipe (figures B-50 and 51). The original spillway pipe was likely placed in a vertically sided trench excavated through the nearly completed embankment dam. This construction technique is not recommended, as it makes compaction of the soil under the sides of pipe very difficult.

Poorly compacted fill in this area results in poor support of the pipe, which causes excessive deformation of the pipe and may cause the joints to separate. In addition, the sides of the trench may tend to support the fill, allowing it to "bridge" across the excavation, preventing the fill from consolidating under its own weight. This can create areas of low soil density under the pipe where seepage can occur. In addition, differential settlement and hydraulic fracture can result.

The embankment dam was redesigned to include seepage controls. The structure was rebuilt in 1990 using essentially the same embankment dam and spillway

Figure B-50.—Loveton Farms Dam failure as viewed from upstream. Note that the walls of the failure area are nearly vertical. The construction records indicate that a large portion of the embankment dam was placed prior to installation of the CMP.

Figure B-51.—Loveton Farms Dam after failure as viewed from the downstream end of the 78-in diameter CMP spillway. Note that one of the antiseep collars, which were about 14 feet square, is visible in the breach.

configuration as the original (although a second riser was added at the upstream end of the spillway). However, a chimney filter was installed which ran axially along the embankment dam. A sand filter diaphragm was also constructed around the downstream portion of the pipe to control seepage and prevent internal erosion of embankment material along the sides of the conduit. The side slopes of the excavation through the remaining embankment dam were designed and constructed as 3H:1V to minimize problems with bridging of the fill. Compaction of the embankment material was carefully monitored and tested during the repairs. Powdered bentonite was added to the backfill under the pipe, because the bentonite would presumably swell to eliminate any voids.

In the years that followed the repair of the embankment dam, Maryland began to experience problems with other dams constructed with large diameter CMP spillways. In particular, it was noted that many of the joints between sections of the pipe were not watertight. This deficiency is primarily the result of deflection of the pipe, (the design standard allowed 5 percent of the pipe diameter), but poor construction techniques and manufacturing tolerances also contributed to the problem (figure B-52). Embankment dam owners were advised to carefully monitor their spillway conduits.

Accordingly, the owner of Loveton Farms Dam (a local government agency) scheduled inspections of the structure twice per year to document the condition of

Figure B-52.—The failed embankment dam was repaired with a new 78-in diameter CMP spillway. However, soon after reconstruction, this failed joint was discovered at the upstream end of the pipe near the riser. A large void was also observed in the adjacent embankment fill. Note the o-ring gasket has been displaced from the joint.

pipe. During an inspection in 1994, large voids (3 feet diameter and 20 feet long) were noted in the embankment around the upstream end of the pipe, and the first joint downstream of the riser had suddenly separated by about 0.1 m.

The embankment dam was determined to be unsafe, and the riser portion of the spillway was removed to minimize impounding of water until a more detailed inspection could be conducted. A more thorough investigation of the embankment dam utilizing seismic tomography revealed that nearly all of the embankment fill around the conduit was of low density. Since the soils are frost susceptible (silty sands and sandy silts of low plasticity), it is quite possible that freezing damaged the soils adjacent to the conduit. The melting of ice lenses that may have formed in the backfill would leave voids through which internal erosion could occur. Also, the formation of ice lenses can create forces large enough to deform the thin steel pipe, causing the joints between pipe sections to open.

The embankment dam was deemed to be unsafe and is scheduled to be removed.

Lessons learned

Use of large diameter CMP conduits in embankment dams should be avoided. Vertical trenches transverse to the embankment dam are never permissible, unless they are in rock and backfilled with concrete. Sloping the sides of excavations to no steeper than 2H:1V is always recommended.

Reference

Schaub, W., *Investigation of the Loveton Farms StormWater Management Pond*, prepared for Baltimore County Bureau of Engineering and Construction, June 1996.

Project name: McDonald Dam

Location: Montana

Summary: Steel lining of an existing outlet works conduit

McDonald Dam is located near Polson, Montana on the Flathead Indian Reservation. A section of the 6-foot diameter elliptical conduit (figure B-53) was removed and replaced. Installation of both 52- and 16-inch diameter bypass pipes was completed in the conduit replacement section, and a hydrostatic test of the 16-inch diameter bypass pipe was performed. The hydrostatic testing was performed in increments and eventually tested the entire lengths of the 52- and 16-inch pipes, as well as existing pipe installed in an earlier contract. Due to the existing 2-inch diameter air vent at the intake structure (tower), a portion of the pipe was pressure tested at 20 lb/in^2 instead of the 30 lb/in^2 required by the specifications.

An independent testing company performed dye-penetrant tests on the welds of all installed 52-inch diameter pipe sections (from sta. 5+79 to sta. 6+60.12).

The annular space around the pipes was grouted. Prior to beginning grouting, the 52-inch diameter pipe was anchored to the existing conduit to prevent the pipe from floating during the grouting operation. Anchorage was accomplished with ¾- by 7-inch diameter mechanical anchors placed through the steel liner and secured to the

Figure B-53.—Existing 6-foot diameter elliptical conduit.

invert of the existing conduit. A total of 11 anchors were used from stations 6+60 to 8+10. The annular space was grouted in two stages. The first stage was to a level just below the lower grout connections. The second stage—the remaining annular space—was grouted 24 hours later. Grouting pressure was limited to 5 lb/in^2. Air vents had been previously installed through the steel bulkheads at stations 6+02, 6+60, and 8+10. The vents at stations 6+02 and 6+60 were extended approximately 5 feet above the top of the conduit to allow placement of embankment to proceed prior to completion of the grouting.

Second stage grouting of the conduit from stations 5+79 to 6+02 began at the upstream grout connection, working downstream until grout had risen to the top of the air vent at station 6+02. The next day, it was observed that the grout had receded completely from the air vent standpipe. An additional 5 ft^3 of grout was pumped into the air vent. For several days prior to beginning the grouting, it was observed that water was flowing from the bottom edge of the bulkhead at station 8+10. The water was assumed to be entering the existing conduit through a joint in the concrete at approximately station 7+00. Grouting of the first stage proceeded from stations 6+60 to 8+10, in an effort to push the water ahead of the grout and out the bulkhead. The flow of water from the bulkhead stopped after the initial grout set, but then resumed several hours later, with just a trickle flowing from the bottom edge of the bulkhead. Problems were encountered with grouting of the second stage due to leakage of grout from the contact between the bulkhead and the pipe. The contractor attempted to use various fillers, but resorted to placement of a filet weld between the pipe and the bulkhead. Grouting of the second stage proceeded from stations 8+10 to 6+60, in an effort to minimize entrapment of air and dilution of grout in the downstream portion of the conduit. Grouting continued until grout was observed in the air vents at stations 6+60 and 8+10. The following day, the grout had receded in the vent at station 8+10. Additional grout was added at the vent, requiring less than 2 gallons to fill the vent. Seepage of water from the bulkhead did not occur after grouting was completed.

The plugs installed in the grout connections were ground down flush with the interior pipe surface and seal welded, per specification requirements. Several days later, it was noticed that water was seeping from two of the connection points. These connections were in the vicinity of the concrete joint in the existing conduit approximately at station 7+00. Presumably, a small void had developed between the pipe and the surrounding grout, allowing a path for the water to seep into the existing conduit at the joint. The leaking grout connections were welded again, which effectively stopped the seepage.

The 2-inch diameter air vents extending through the bulkheads at stations 6+03 and 6+60 (used for grouting of the annular space) were removed. The 2-inch diameter pipe was removed at the threaded couplers (10 inches below the top of the conduit) and the void repaired using sand and cement dry-pack.

The uncoated interior surfaces of the newly installed 52-inch diameter pipe and damaged paint coating on the existing pipe were sand blasted and painted. These areas consisted of the weld joints, grout connections, and miscellaneous scrapes and gouges throughout the length of the conduit. Paint coatings were applied with two applications of DeVoe High Build Epoxy. Mil thickness readings taken indicated the first coat was 7 mils and the second coat, applied the next day, checked out at only 14 mils. The paint subcontractor subsequently returned and applied a third coat, to build the coating thickness to the 16 mils required by the specifications. Final applications of DeVoe High Build Epoxy were made on weld areas and damaged paint surfaces of the 54-inch diameter pipe, which completed all work associated with the pipe.

Lessons learned

- Grouting operations must be well planned and closely monitored.
- Grouting operations may require field adjustment to accommodate any seepage encountered.

Reference

Bureau of Reclamation, *Technical Report of Construction for McDonald Dam Modification (Draft)*, Flathead Construction Office, Ronan, Montana, October 2000.

Project name: Medford Quarry Wash Water Lake Dam

Location: Maryland

Summary: Failure of an embankment dam due to internal erosion along the conduit

Medford Quarry Wash Water Lake Dam is a 26-foot high, significant hazard embankment dam. Downstream hazards include roadways, railroad tracks, and a residence. The embankment dam is essentially an offstream basin, and nearly all inflow is pumped into the basin from a nearby wash plant.

The embankment dam was constructed in 1988 as a "temporary sediment basin." Although an engineer prepared plans for the structure, no engineering supervision was provided during construction. Local materials (decomposed shale and erodible silts) were used to construct the embankment dam. A CMP spillway with conventional antiseep collars was constructed in a trench excavated into the foundation and backfilled with low plasticity silts and decomposed rock fragments from the excavation (figure B-54).

The structure was placed into use the following year, and it failed upon first filling. When the pool level was only about 2 to 3 feet deep, flow along the outside of the pipe resulted in loss of the adjacent soils by internal erosion (figure B-55).

Figure B-54.—The embankment dam was constructed with antiseep collars.

Figure B-55.—When the pool level of this embankment dam was only about 2 to 3 feet deep, flow along the outside of the CMP resulted in loss of the adjacent soils.

The spillway structure was removed and replaced and has performed satisfactorily.

Lessons learned

This significant hazard embankment dam was improperly designed and constructed as a "temporary sediment basin" (which has less rigid construction requirements) and did not have proper inspection during construction.

Reference

Maryland Dam Safety Division, Dam file No. 318.

Project name: Olufson Dam

Location: Washington

Summary: Outlet works conduit failure

Olufson Dam was a privately owned embankment dam located in Pierce County, near Gig Harbor, Washington that experienced an outlet works conduit failure. The embankment dam was 18 feet high, with a storage capacity of 15 acre-feet and 21 acre-feet at the top of the dam. The principal spillway consisted of a 2-foot square, concrete, drop inlet conduit. An open channel in the abutment served as an emergency spillway. The embankment dam was constructed in the 1960s without the benefit of engineering plans. The owner did all the work himself, including placing earthfill and mixing his own concrete onsite. Conditions exposed by the failure suggest that the elements of the construction that required skill were substandard. In particular, the concrete work suffered from inadequate cement content, poor overall mix gradation, and improper reinforcing. Thick steel cable was substituted, in part, for conventional reinforcing steel. Likewise, these cables were improperly positioned in the conduit section thus minimizing its enhancement of the tensile load capacity of the conduit. To limit concrete volumes, it appeared the owner had embedded bricks, rocks and concrete rubble into the walls as a filler during concrete pours. This practice, termed cyclopean concrete construction, has been successfully used in large gravity structures, but was inappropriate for thin walled, concrete box conduits.

On December 11, 1996, a sinkhole 20 feet in diameter and 17 feet deep opened up in the crest of the embankment dam (figure B-56). At the time the sinkhole developed, the property on which the embankment dam sat was uninhabited due to the recent death of the property owner. The sinkhole was discovered by neighbors walking the streambed to investigate the cause of muddy streamflows. This was fortuitous in that the sinkhole was discovered before it lead to an embankment failure. The sinkhole appeared to have resulted from a collapse in the top section of the cast-in-place box culvert that served as the principal reservoir outlet. The failed segment of the conduit allowed overlying masses of embankment soil, over time, to repeatedly drop into the conduit, where flows then flushed the soil downstream. This sequence of events was supported by the record of stream flows in a downstream gauging station. The gauge record shows normal flows interrupted by a series of near zero creek flows immediately followed by short, abnormally high channel discharges. The zero flows are interpreted as incidences of soil masses falling into and plugging the conduit. The following anomalous high flows represent a blowing out of the plug and a release of backwater in the conduit and inlet tower upstream of the plug.

Figure B-56.—Sinkhole in the dam crest the night of December 11, 1996.

As an immediate response to the threat of an embankment dam breach, county maintenance staff filled the sinkhole with some 200 yards of angular cobbles and boulders. The State dam safety staff saw no viable alternative to the county's scheme to address the immediate crisis. Finer grained soils would likely have been sluiced through the top of the collapsed box conduit. This could have worsened the situation by plugging what limited outlet capacity remained after sediment had largely blocked the conduit. Nonetheless, it was obvious that the rockfill was but an interim measure, and immediate follow-up action was necessary to lower the reservoir and permanently resolve the public safety threat. Three days of pumping were necessary to lower the reservoir to allow excavating a trapezoidally shaped breach of the embankment dam (figure 57). The floor of the breach was armored with a geotextile fabric and capped with much of the rock originally dumped into the void the night of the failure. To improve fish passage, an attempt was made to include a number of pools along the breach channel at the direction of the Washington State Department of Fish and Wildlife. The Washington State Water Quality staff assisted in blanketing the disturbed sections of the embankment dam with hay to minimize further sediments entering the water course.

Damage downstream was limited to the streambed. Primarily, it occurred in the form of stream habitat degradation from sediment deposition. Many of the salmon eggs in this fish-producing stream were smothered under sediments for several hundreds of yards downstream of the embankment dam. As bad as it was, the emergency action prevented a likely failure of the embankment dam. Thus, the

Figure B-57.—View of upstream dam crest nearing completion of breach.

possible threat of loss of life was averted along with extensive damage to property abutting the streambed.

Lessons learned

An examination of the failed conduit through the embankment dam revealed it to be of poor quality with minimal reinforcing. What reinforcing was provided, consisted of misplaced, steel cable rather than conventional deformed bars. Given the construction of the conduit, it is remarkable that it functioned for over 30 years.

This failure reinforces the concern that conduits have a definite service life, measured in decades. At the end of that service life, they require retrofitting for their continued satisfactory functioning. A failure to do so, risks a failure of the embankment dam. Proper care taken in the design and construction can materially increase the conduit service life. Conversely, poor workmanship may reduce it.

Periodic inspection of conduits is required to confirm that they are structurally sound, and to provide timely notice of a developing problem with age.

Reference

Washington State Department of Ecology, Project file.

Project name: Pablo Dam

Location: Montana

Summary: Removal and replacement of an existing outlet works

Pablo Dam is located near Polson, Montana on the Flathead Indian Reservation. The embankment dam is an earthfill structure consisting of a main dam and dikes, which flank both sides of the dam, south and north. The crest elevation of the main dam is at 3220, and the dikes are at 3217. The main dam has a structural height of 43 feet, a crest length of 10,550 feet, a crest width of 20 feet, a 3:1 upstream slope, and a 2:1 downstream slope. The north dike has a crest length of 5850 feet, and the south dike has a crest length of 10,250 feet. The crest width of both dikes is 12 feet.

Pablo Dam was constructed in three phases over 24 years. In 1911, the embankment was constructed to elevation 3202. The second construction in 1918 raised the embankment dam to elevation 3209, and the final construction in 1934 raised the dam to the present elevation 3220. Pablo Dam is an offstream structure that is fed by the Pablo Feeder Canal. The purpose of the embankment dam is to impound water for irrigation. The reservoir has a capacity of 28,400 acre-feet at elevation 3211.0.

The original outlet works was situated at the maximum section of the dam and consisted of a 42-foot high concrete intake structure with two 3- by 5-foot slide gates. The original outlet works consisted of three box shaped conduits. The middle and south conduits were 172 feet long and 4.5 feet wide by 5 feet high. The north conduit was about 136 feet long and 3.0 feet square. This north conduit was abandoned prior to the third phase of original construction.

Differential settlement between the intake tower and the outlet works conduits caused some offset in "sliding joints." This settlement was expected, as "sliding joints" (no reinforcement crossing the joint) were included in the original design. However, continued settlement of the intake structure and the first 13 feet of the conduits required grouting of the foundation shortly after construction. No further settlement has been detected in the last 50 plus years. The first sliding joint is displaced vertically about 2 inches and sprays water at high reservoir head. Mortar filling in all sliding joints was disbonded, cracked, and deteriorating. Tensile cracks were also discovered along the length of the conduit. Water was commonly leaking from both the cracks and the sliding joints, and there are signs of possible internal erosion of embankment material occurring in a few areas. Spalling concrete had been discovered in the walls of the conduits. The concrete in the center wall at the downstream end of the conduits was deteriorated, resulting in exposed aggregate and rebar.

Dam safety modifications were begun in 1993, consisting of injection of polyurethane grout into cracks and conduit joints. A two-man crew from McCabe Brothers Drilling of Idaho Falls, Idaho, mobilized to the job site. They installed ventilation ductwork into the two outlet works conduits and began drilling injection holes in the south conduit. Existing cracks (mostly at construction joints) upstream of station 1+27 were injected with polyurethane resin grout to stop leakage through the cracks. This was done prior to repair of spalled concrete in the conduits. The subcontractor used a ratio of polyurethane to water of 1.3:1, which effectively stopped 90 percent of the seepage. However, after completing injection of cracks in the south conduit, seepage began to migrate downstream and appear in cracks that were previously dry.

During drilling of the injection holes, two voids were discovered, one in the crown of each conduit at station 0+13. The voids were approximately 12 inches deep and 24 inches wide and seemed to be connected to each other. Old construction drawings showed this as the location where concrete counterfort walls, which support the intake tower, meet the conduits. No voids were found behind any of the other cracks. The voids at station 0+13 were injected with polyurethane. As injection of the south conduit was completed, some migration of polyurethane was noted through the crown and divider wall of the middle conduit.

In mid-November, McCabe Brothers Drilling completed injecting polyurethane resin into cracks in the outlet works conduits. They injected a total of 305 gallons into the two conduits (the specified quantity was 50 gal). As the injection operation progressed from upstream to downstream, cracks that had been previously dry near the canal outlet began to seep water. Therefore, these cracks were injected also. Because the seepage appeared to be following the exterior of the conduits and exiting farther downstream, the seepage continued to be unfiltered and may increase the internal pressures in the embankment. A decision was made to install weep drains in the conduit and to construct a filter collar about the exterior of the walls. A modification to the contract was issued to provide for this additional work.

After the polyurethane injection was completed, the conduits were unwatered and inspected. Repair areas were marked, and the contractor began chipping out and preparing the surfaces of the repair areas for epoxy-bonded concrete. Approximately 30 small repairs and one large repair at the conduit outlet (splitter wall) were done to complete the conduit repairs option of the work. Smaller and shallow areas were repaired using an approved two-part epoxy material. Larger areas were repaired with epoxy-bonded concrete.

During an inspection of the interior of the conduits in April 2001, it was discovered that material had been deposited inside the middle conduit near an opening in a construction joint. This was occurring through a hole in the floor of the middle conduit at a construction joint near station 1+30. Approximately 1 ft^3 of silt and fine sand were deposited on the floor. However, this deposit was observed during the winter when no irrigation releases are made. More deposition may have occurred during irrigation season that was washed downstream and not observed. Consequently, the total volume of material could have been much greater than the 1 ft^3 observed in 2001. Reclamation theorized that plugging this opening could result in redirecting the erosion through a different hole or crack in the conduit. Also, redirecting the erosion might cause a more dangerous path to develop along the foundation contact of the conduits, and a piping exit might develop downstream of the embankment dam. If the exit point were located within the outlet channel, early detection would be very difficult.

Another area of concern was the condition of the north conduit that was reportedly plugged at each end prior to the final raise of Pablo Dam in 1932, but was never confirmed. Therefore, it could be possible that a nearly full reservoir head could exist at the end of the north conduit, which was less than 100 feet from the downstream toe of the dam. After much discussion between all involved parties, it was decided to completely remove and replace the original outlet works.

As an interim measure, a temporary patch was installed over the opening to prevent additional material from being eroded into the conduit while allowing for relief of water pressures. The patch consisted of filter fabric under a metal screen. During March 2002, the geotextile portion of the patch ruptured and approximately 0.5 ft^3 of silt and fine sand were deposited into the conduit. The patch was repaired soon after the rupture was discovered. Reservoir level restrictions were implemented in April 2003 and were to be kept in place until the removal and replacement modifications could be completed.

The construction of a new outlet works began in November 2004 and was completed in the by the spring of 2005. The major aspects of the work included:

- Construction of a cofferdam to maintain an area free of water during construction.

- Clearing, grubbing, and stripping prior to excavation.

- Removing existing embankment dam slope protection.

- Excavating embankment materials to accommodate construction of the new outlet works (Slopes transverse to the dam centerline were excavated at 4H:1V).

Figure B-58.—Pablo Dam nearing completion.

- Removal of the existing reinforced concrete intake structure, conduits, retaining walls, and apron.

- Constructing a lean concrete mudslab, on which to found the new outlet works.

- Constructing reinforced cast-in-place intake structure, conduit, retaining walls, and apron. The new conduit was double barreled with each barrel having a 6-foot 3-inch inside diameter. The exterior surface of the conduit was sloped at 1H:10V below springline and was curved above springline to provide a good surface to compact earthfill against. Each conduit joint was a treated as control joint with longitudinal reinforcement extending across the joint and 6-inch PVC waterstop.

- Installing two emergency guard gates and two regulating gates within the upstream intake structure.

- Constructing a chimney filter and drain system. The filter extended downstream and encased the outlet works conduit. Filter materials encasing the conduit consisted of sand processed to a specified gradation from an approved offsite source.

- Placing and compacting zoned earthfill in the embankment dam closure section.

- Replacing the embankment dam slope protection.

Figure 58 shows Pablo Dam as it was nearing completion.

Lessons learned

Sometimes repairs alone are not fully robust enough to address all the unknown erosional mechanisms existing within an embankment dam. Due to continued dam safety concerns, more extensive measures may be warranted.

Reference

Bureau of Reclamation, *Technical Construction Report-Pablo Dam Modification Contract No. CSKT/SOD 06*, August 1996.

Project name: Pasture Canyon Dam

Location: Arizona

Summary: Closed circuit television inspection of an outlet works conduit

Pasture Canyon Dam is located on the Hopi Indian Reservation in Arizona. Pasture Canyon Dam is a homogenous embankment dam with a height of 17 feet. The embankment dam crest is at elevation 4890.0 feet, 20 feet wide, and 632 feet in length. The embankment dam was apparently founded on pervious, sandy alluvium. No information was available as to its construction. The embankment dam was completed in 1920s or 1930s and modified in 1975. The 1975 modification included a 3-foot crest raise.

Appurtenant structures at the site include an uncontrolled earthen spillway and an outlet works. The outlet works is located within the embankment dam approximately 200 feet from the left abutment. The outlet works consists of a concrete intake structure, approximately 55 feet of 12-inch by 12-inch masonry conduit connected to 35 feet of 14-inch diameter CMP connected to 20 feet of 14-inch diameter concrete pipe. The intake structure contains a hand-operated slide gate. The discharge capacity of the outlet works has been estimated to be 9 ft^3/s.

The Bureau of Reclamation's Technical Service Center performed a CCTV inspection of the outlet works conduit at Pasture Canyon Dam in April 2004. The conduit was accessed for inspection, via an existing manhole located at the downstream end of the outlet works. The camera-crawler was inserted into the 14-inch diameter concrete pipe and was advanced upstream approximately 18 feet, where the concrete pipe ended and CMP began. Water clarity was somewhat poor and limited viewing throughout the conduit. The camera-crawler was advanced upstream within the CMP for approximately 35 feet, where the CMP ended and a masonry conduit began. The camera-crawler was advanced upstream within the masonry conduit for approximately 30 more feet, where numerous piles of sand were observed near the sidewalls of the masonry conduit. Figure B-59 shows a typical pile of sand near the sidewall of the conduit. In addition, just a few feet into the masonry conduit existed an open defect (crack) in the crown of the conduit. Figure B-60 shows the defect at the crown of the conduit. This defect allowed sand materials to enter the conduit. Based on observations made during the CCTV inspection, it was concluded that Pasture Canyon Dam was in the process of failing by internal erosion or backward erosion piping, and immediate action was required. Visual monitoring of the embankment dam and monitoring of the area around the outlet works was performed every 4 hours. The Bureau of Indian Affairs (BIA) imposed reservoir and gate operating restrictions at Pasture Canyon Dam.

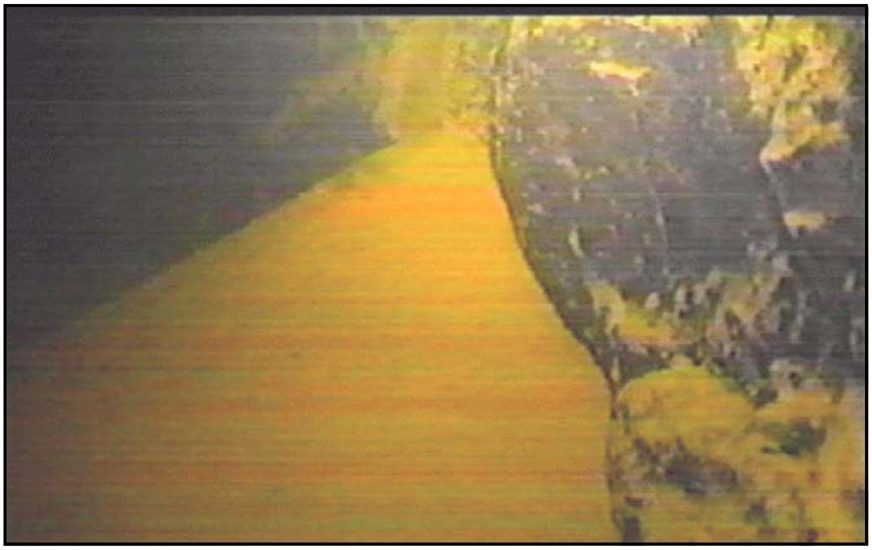

Figure B-59.—The camera-crawler encountered fine sandy materials that appeared to be entering the conduit through a defect in the sidewall. This view is looking upstream at the right side wall. The conduit had not been operated since the fall of 2003. These materials had collected within the conduit over the last 7 months.

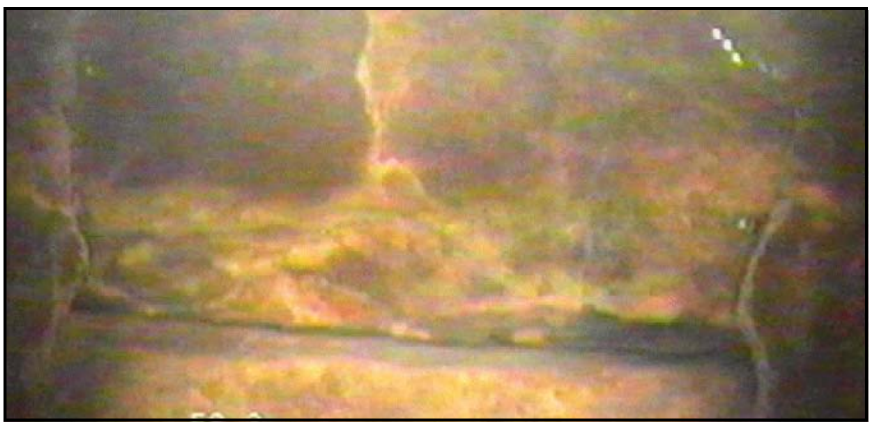

Figure B-60.—Just into the masonry conduit an open defect (crack), shown in the lower part of the figure, exists in the crown of the conduit. This defect allowed sandy materials to enter the conduit.

A contractor was mobilized to lower the reservoir level to elevation 4880.0 using high capacity, low head pumps (12-inch diameter). The reservoir drawdown was limited to 1 foot per day. After the reservoir was lowered to the desired elevation, a siphon was installed to provide downstream irrigation releases. Figure B-61 shows the siphon discharge irrigation releases. Based on the deteriorated condition of the conduit, the BIA decided to abandon the outlet works by grouting it closed. The

Figure B-61.—Siphon constructed over to crest of the embankment dam for discharging irrigation releases.

Figure B-62.—Grout mix being conveyed directly into the grout mixer from the transit mixer truck.

Bureau of Reclamation's Farmington Construction office accomplished the grouting using three 1½-inch diameter PVC schedule 40 pipes. The upstream end was plugged by an inflatable bladder. The downstream end was plugged using a burlap pig sealed with redi-mix dry-pack. The burlap pig acted as a filter to prevent grout leakage, and the dry-pack held the pig in place. Two grout plants were brought onsite. The grout mixers had a volume of 45 gallons each. The plants were powered by gasoline engine over a hydraulic system and were found to be very adequate for the grouting operations. Figure B-62 shows the grout plant used. Type II cement was utilized in the grout mix. The initial mix was 0.8:1 (water/cement ratio by volume). When it was determined that no problems were encountered with the grouting operations, the mix was reduced to 0.7:1. A grout fluidfier was used in the grout mix. Upon completion of the grouting operations, an ASTM C33 sand filter was installed at the downstream end of the conduit.

Lessons learned

- CCTV inspection equipment can be used to identify deteriorated areas within inaccessible conduits.
- Expedited dam safety actions require good communication between all interested parties and agencies involved.
- A siphon can be constructed quickly and inexpensively in order to provide downstream irrigation requirements.

Reference

Bureau of Reclamation, *Pasture Canyon Dam—Outlet Works Abandonment*, November 2004.

Project name: Piketberg Dam

Location: South Africa

Summary: Failure of an embankment dam by internal erosion resulting from hydraulic fracture of earthfill adjacent to the outlet conduit

In 1986, Piketburg Dam, a 40-foot high embankment dam was built across a minor tributary of the Verlore Vlei River, near the town of Piketberg in the Western Cape, South Africa. The new embankment dam was constructed over an existing dam at the site to increase storage. Figure B-63 shows a cross section of the embankment dam.

After 5 weeks following construction, during which water was pumped into the reservoir, and when it was almost full, major leakage suddenly appeared at the downstream toe of the embankment dam near the outlet. Within less than a day, the entire contents of the reservoir had been lost through a cavern adjacent to the outlet conduit.

Inspection after the event revealed a major tunnel through the entire width of the embankment dam along the outlet conduit. At the time of the inspection, the roof of the tunnel had collapsed over the entry and exit. The center portion of the tunnel beneath the dam crest, however, remained intact arching almost 33 feet across the tunnel. Large sinkholes were present in the upstream slope of the dam.

The embankment material was broadly graded from coarse gravel sizes to clay sizes, typically with a liquid limit of 28 percent and plasticity index of 9. Both residual soil (decomposed phyllite and greywacke) and transported soil were utilized. The latter

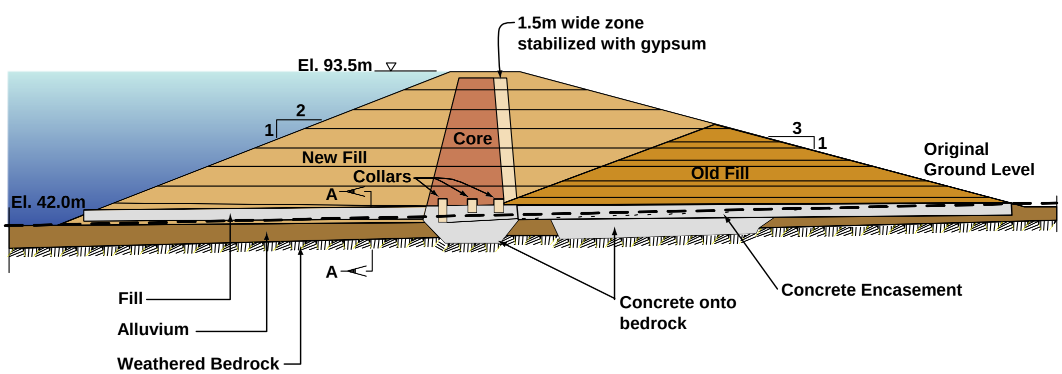

Figure B-63.—Cross section of the dam as designed.

included gravelly clay gully wash deposited in the form of an alluvial fan, as well as colluvium. Fine grained material was intended to be reserved for the designated core zone. The embankment dam design did not include provisions for either filtering or drainage of the core.

Tests showed that the earthfill material was dispersive. During construction, gypsum was added to portions of the core as a treatment for dispersivity. The new outlet conduit was laid roughly along the original ground surface, under the highest section of the new dam. A pipe was placed in the bottom of a wide slot cut through the old embankment dam. The pipe was laid in a trench dug into the lowest layers of compacted fill that had already been placed, and then encased in reinforced concrete. Across the new core's foundation cutoff, plus along one other section, the outlet trench was deepened to weathered bedrock prior to filling with concrete, to improve bearing upon soft material present under those sections. Concrete antiseep collars were found to have been cast over only the top and upper sides of the outlet encasement. The collars did not extend below the pipe encasement.

Breaching took place soon after filling began and before the reservoir was completely filled. This suggests that one or more concentrated leaks must have existed, to enable flow to reach the downstream toe so soon, long before the saturation front could have advanced very far into the earthfill. Failure started at the downstream toe in the vicinity of the outlet conduit, and the erosion tunnel terminated immediately adjacent to the upstream end of the outlet encasement.

The initial concentrated leaks alongside the outlet conduit are postulated to have occurred due to hydraulic fracture of the earthfill by the rapid rise of the reservoir. The conduit appears to have allowed for a low stress zone to occur in the earthfill next to the wall of the encasement. In essence, the wall "shielded" the adjacent fill from the full weight of the overlying embankment. The stresses in this area were likely lower than the reservoir head. A somewhat compressible foundation material beneath the conduit could have assisted in the formation of the crack completely along the embankment dam's cross section. Once concentrated flow started, the dispersive nature of the embankment fill would have allowed for rapid erosion from the downstream exit of the crack, progressing upstream. Lack of any defensive designs for embankment cracking, such as a filter and drain, contributed to the failure.

Also contributing to the failure was the poor compaction of the earthfill material adjacent to the encasement wall that was found during the forensic investigations. Also thought to contribute to the failure were the potential for differential settlement of the new and old earthfill.

Figure B-64 shows a cross section through the conduit area after failure.

Lessons learned

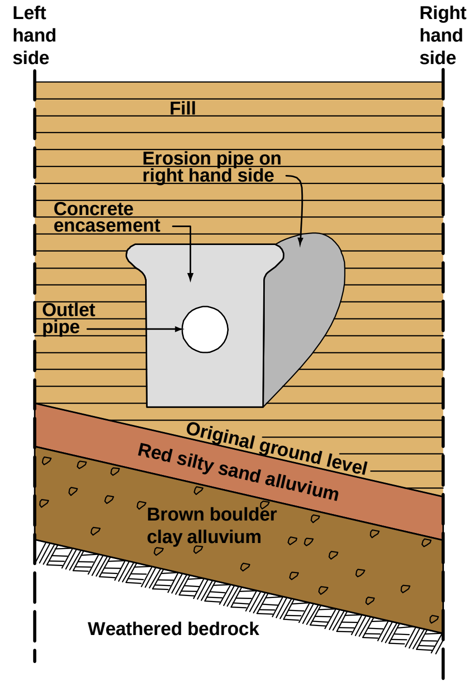

Figure B-64.—Cross section through the outlet conduit showing pipe, encasement and erosion tunnel.

The dam failure was likely caused by poor compaction of the soil adjacent to the outlet conduit. Factors contributing to the poor compaction include inclusion of antiseep collars and a poorly constructed concrete encasement. A compressible foundation may have assisted in the formation of a crack next to the conduit.

Key changes to the design and construction that would have likely prevented failure of the embankment dam include:

- Utilizing a conduit design that accommodated the likely settlements caused by the foundation.

- If the concrete casement around the conduit had used battered side slopes rather than vertical ones, compacting soils against the conduit would have created more positive pressures and lessened the potential for hydraulic fracture.

- Inclusion of a filter diaphragm.

- Elimination of the cutoff collars.

Reference

Wilson, Clive and Louis Melis, "Breaching of An Earth Dam in the Western Cape by Piping," *Geotechnics in the African Environment*, Blight et al. (eds), Balkema, Rotterdam, 1991.

Project name: Ridgway Dam

Location: Colorado

Summary: Grouting of cracks in an existing outlet works conduit

Ridgway Dam is a zoned earthfill embankment across the Uncompahgre River in Ouray County near Montrose, Colorado. The embankment dam has a maximum height of 335 feet above the streambed, and a crest length of approximately 2,460 feet. The river outlet works is located near the right abutment of the embankment dam and crosses the dam axis at station 11+69.07. Most of the outlet works was constructed under a Stage I contract during 1980 and 1981. The completed outlet works consists of a 50-foot long, 9-foot diameter diversion conduit, a drop inlet intake structure with a concrete plug, a 500-foot long, 9-foot diameter upstream conduit, a gate chamber with a 5- by 6-foot high pressure guard gate, 64-inch square regulating gates, a control house and equipment, and hydraulic jump stilling basin. The spillway consists of a gloryhole intake exiting into a 6.5-foot diameter conduit located on a shelf on the left abutment of the dam. The conduit exits into an open chute and then into a Type II hydraulic jump stilling basin. Figure B-65 shows an aerial view of Ridgway Dam.

Figure B-65.—Aerial view of Ridgway Dam, Colorado.

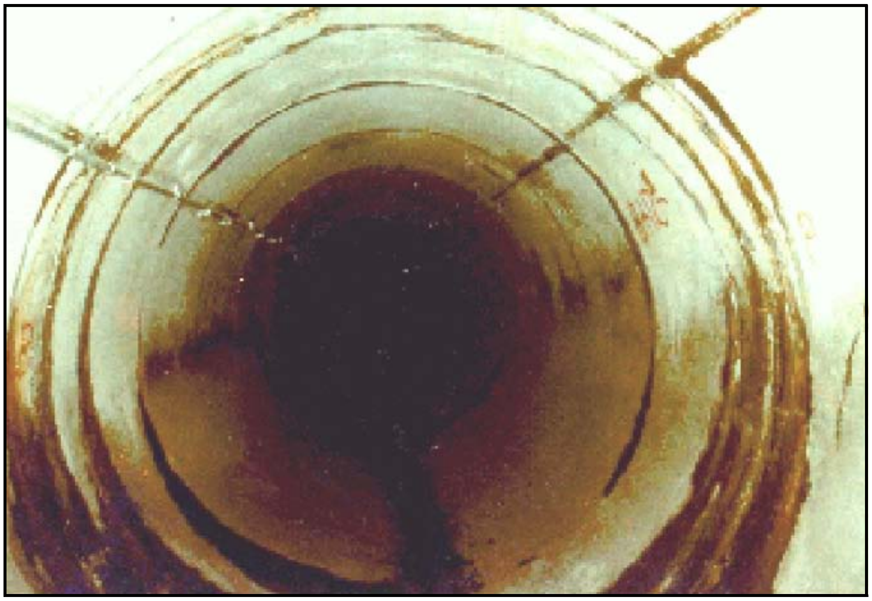

Figure B-66.—View of the interior of the outlet works after grouting.

Settlement and crack surveys were taken, in the upstream and downstream conduits, in January of 1986 and October of 1986. The maximum settlement recorded in January of 1986 was about 0.74 feet at station 11+24 in the downstream conduit. For reference, the station at the centerline of the gate chamber is located just downstream of the centerline of the embankment dam, station 7+50. In October of 1986, an additional settlement of about 0.22 feet was recorded at station 11+24. Settlements of other points in the conduit varied, seemingly based on the stiffness of the foundation. Maximum transverse cracking occurred in the upstream conduit at station 7+80 and station 8+80. The downstream conduit had very little transverse cracking, but a maximum joint opening of about ½ inch occurred at station 10+73. Longitudinal cracking occurred in the crown and invert between stations 6+50 and 7+65 and between stations 11+74 and 14+35 in the upstream and downstream conduits, respectively. The settlement and cracking occurred during or after constructing the dam embankment to elevation 6830 in the 1985 construction season and topping out of the dam in 1986 at elevation 6886.

The settlement and cracking of concrete in the upstream conduit was addressed by required injecting grout in and around the cracks to reduce water leakage and potential backward erosion piping or internal erosion of surrounding soils in the embankment dam, along with protecting the conduit reinforcement from corrosion. Polyurethane resin was used to grout the transverse cracks. Also, the polyurethane resin grout was considered (due to its more flexible characteristics over more rigid epoxy material) for grouting the longitudinal cracks. However, longitudinal cracks were grouted with epoxy material to provide a degree of structural stability. Approximately 500 feet of cracks were designated for grouting. The contractor grouted the cracks in February and March 1987 (figure B-66). Instrumentation was installed across selected cracks in the upstream conduit, which could be read in the gate chamber to track opening of the cracks and additional settlement. This type of tracking was selected because it is difficult and expensive to unwater and inspect the upstream conduit. The downstream conduit is accessible, and cracking is routinely inspected.

Lessons learned

- Settlement can occur even with best efforts to locate the conduit on competent foundation. Settlement along the alignment is not uniform and can result in cracking of the conduit.

- Grouting is an effective method for seating cracks and making the conduit watertight.

Reference

Bureau of Reclamation, Unpublished notes and file photographs.

Project name: Rolling Green Community Lake Dam

Location: Maryland

Summary: Sliplining of an existing spillway conduit using Snap-Tite® HDPE

Rolling Green Community Lake Dam failed in February 1999. Constructed in 1965, the 22-foot high, low hazard embankment dam contained a 24-inch diameter CMP spillway. The spillway riser had been gradually deteriorating, and the owner had attempted repairs at the top of the riser by use of a larger CMP sleeve and concrete grout. However, no repairs to the lower portion of the riser were attempted, and the base of the riser collapsed on February 6, 1999. A large portion of the embankment dam was washed away, leaving a void about 30 feet in diameter and 10 feet deep around the original riser location (figure B-67).

A CCTV inspection of the barrel portion of the CMP revealed that the remaining sections of pipe were in good condition. The engineer elected to slipline the existing 24-inch diameter barrel with 20-inch (outside diameter) Snap-Tite® pipe (SDR 32.5). The space between the two pipes was filled with a grout composed of fly ash and cement (compressive strength 2,500 lb/in^2), and a new aluminum riser was constructed within the upstream portion of the embankment dam.

Figure B-67.—When the 35-year old corrugated metal pipe riser collapsed, a large portion of the low hazard embankment dam was washed away. This left a void about 30 feet in diameter and 10 feet deep around the original riser.

A filter diaphragm was constructed around the downstream end of the pipe to control seepage along the outside of the conduit.

Lessons learned

The use of Snap-Tite® HDPE allowed for rapid installation of a slipliner at a low hazard facility. No specialized contractors were needed to heat fuse the joints of the slipliner.

Reference

Deegan, J., *Rolling Green Dam Completion Report*, 2001.

Project name: Round Rock Dam

Location: Arizona

Summary: Sliplining of an existing outlet works conduit using HDPE

Constructed in 1937 and enlarged in 1953, Round Rock Dam is a 35-foot high embankment dam, located about 3 miles from the town of Round Rock, Arizona on the Navajo Reservation. The original outlet works at Round Rock Dam was constructed by cut and cover methods. Both the upstream and downstream outlet works conduits were constructed of 24-inch diameter CMP. Reservoir releases are controlled by a 24-inch diameter slide gate in a concrete wet well located about 15 feet upstream of the dam crest. In 1991, the Bureau of Reclamation's Deficiency Verification Analysis identified the structural integrity of the outlet works conduits as a dam safety deficiency. CCTV inspection had detected corroded portions and joint separations in the CMP.

Reclamation designed a modification, to address this dam safety deficiency, which consisted of an 18-inch O.D. HDPE pipe that was sliplined (figures B-68 and B-69) and then grouted into both the upstream and downstream CMP conduits. The HDPE pipe was designed to withstand external loads, disregarding any additional

Figure B-68.—Heat fusion of an HDPE pipe joint.

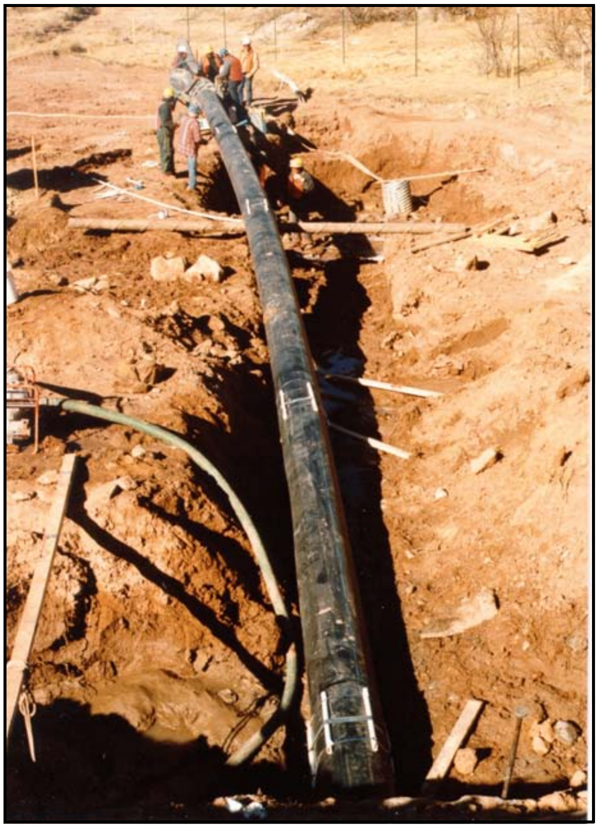

Figure B-69.—Inserting the HDPE slipliner into the existing CMP conduit.

support from the existing CMP. The design called for the HDPE pipe to withstand embankment fill loads of 38.2 feet, hydrostatic loads to 37 feet and a construction surcharge H-20 live load with a minimum of 5 feet cover. The HDPE pipe was also designed to withstand the loads associated with grouting. Installation of the liner was completed in the summer of 1994. The outlet works has operated without any liner-related incidents since that time. CCTV inspection of the slipliner was performed in May 2001, and the liner was found to be in good condition.

Lessons learned

Sliplining provides a low cost and less disruptive alternative to the conventional removal and replacement renovation method.

Reference

Bureau of Reclamation, *Outlet Works Video Inspection at Round Rock Dam—Bureau of Indian Affairs (BIA) Safety of Dams Program—Navajo Indian Reservation, Arizona*, July 26, 2001.

Project name: St. Louis Recreation Lake Dam (actual name withheld by request of owner)

Location: Missouri

Summary: Conduit abandonment by grout injection

In 2000, a 118-foot high embankment dam was constructed in the St. Louis area to create a 325-acre recreation lake. The lake is an integral part in an upscale land development. When the construction permit application was submitted to the State for approval, the designer included a 16-inch diameter PVC diversion pipe in the base of the embankment dam to prevent impoundment of water while the dam was being built. The construction permit was ultimately approved with the condition that the pipe would be filled with grout when the embankment dam was completed.

During construction, some foundation problems were discovered that required grouting of the foundation. The contractor was allowed to construct the lower portion of the embankment dam, and the grouting contractor was allowed to drill through it to grout the foundation. During this process, the contractor apparently observed a small amount of grout flowing from the outlet end of the temporary PVC diversion pipe (However, this was not reported to the State until after a problem with the pipe was later discovered).

As the work on the embankment dam neared completion and it came time to abandon the temporary diversion pipe with grout, the owner (through his designer) requested permission to alter the plans. Instead of filling the pipe with concrete, the owner proposed to retain the pipe and place a valve on the downstream end of the pipe. Their argument was that this would allow them to use the PVC pipe to lower the lake level in the future. The state Dam Safety Program balked at this, and it quickly became a contentious issue between the State and the dam owner. In an effort to assess the condition of the PVC pipe prior to making a final decision the State used a remotely operated video camera to examine the interior of the pipe.

With the owner and his engineer present, a video camera was inserted into the downstream end of the pipe. At approximately 450 feet upstream of the pipe outlet, a separated joint was observed (figure B-70). Just upstream of that joint, the pipe had collapsed, leaving a space only a few inches high at the bottom (figure B-71) The pipe was immediately abandoned by completely filling it with grout.

The owner later admitted that if the State had not been able to demonstrate why it did not want a valve on the downstream end of the pipe, he would have tried to coerce the State to allow the pipe to remain by contacting his legislators or by going

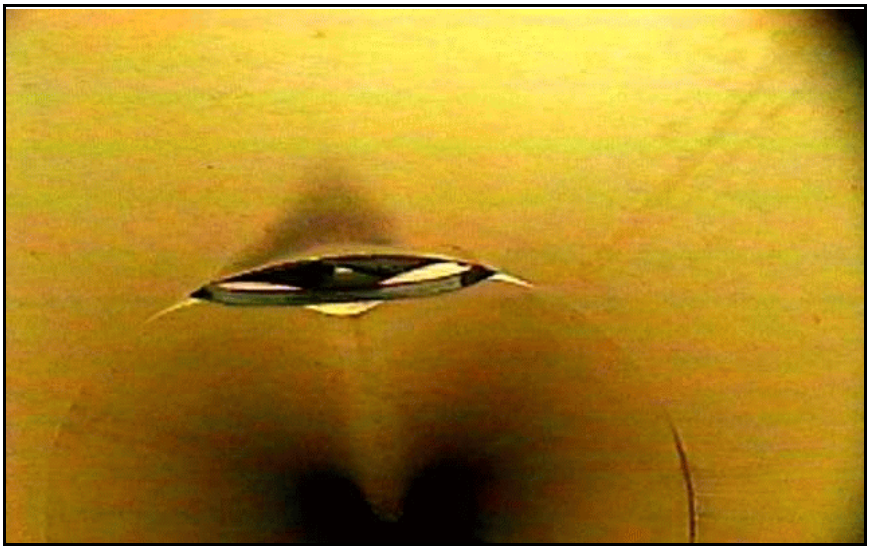

Figure B-70.—An open joint was discovered in a 16-inch PVC temporary diversion pipe under the 118-foot high embankment dam.

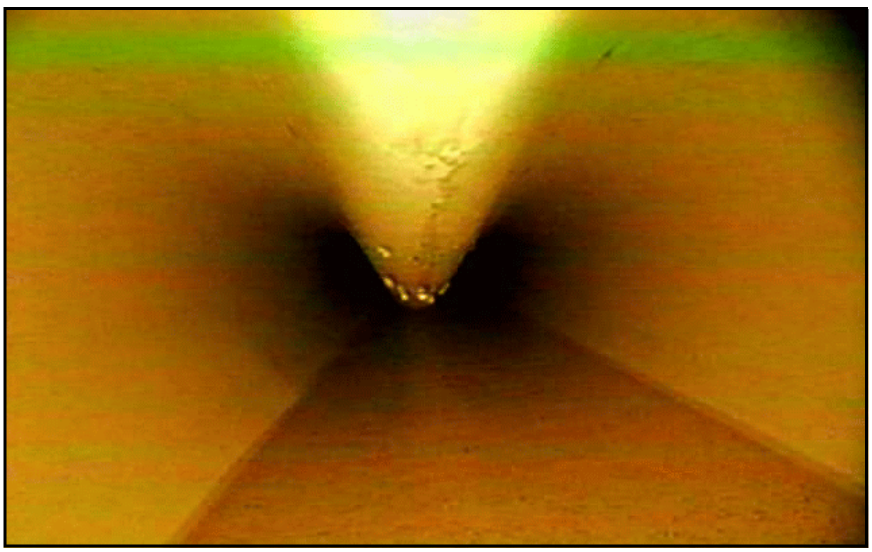

Figure B-71.—A portion of the temporary PVC diversion pipe was found to be severely deformed.

to court. If they had proceeded with the modification, this would have resulted in an unsafe pipe, subjected to full reservoir of more than 100 feet of head pressure running through the base of the embankment dam, with a valve at the downstream end. The presence of the separated pipe joint could ultimately have resulted in a disaster.

The cause of failure of the PVC diversion pipe was not determined. The cause may have been the result of the grouting work, poor construction practices, faulty pipe, or a combination of these factors.

Lessons learned

Internal inspection of all conduits within an embankment dam should be conducted at the end of construction to ensure that the conduits are not excessively deformed and that they will perform as intended. PVC pipe is not recommended for use in high or significant hazard embankment dams, unless it is fully encased in reinforced cast-in-place concrete. A better design would allow the permanent spillway to also function as a temporary diversion and would avoid the use of temporary conduits, which are intended to be abandoned in place.

Reference

Personal communications with Mr. Jim Alexander, P.E., Program Director, Chief Engineer, Water Resources Program, P.O. Box 250, Rolla, Missouri 65402.

Project name: Salmon Lake Dam

Location: Washington

Summary: Man-entry and underwater inspections of an outlet works conduit

Salmon Lake Dam is an offstream embankment dam located above the town of Conconully in Okanogan County, Washington. The Salmon Lake Dam reservoir (Conconully Lake) was a natural lake prior to construction of the embankment dam in 1921. The embankment dam is an earthfill structure with a structural height of 54 feet, a crest length of 1,260 feet at elevation 2325.1 (2330.25 original datum), and a crest width of 14 feet. The reservoir has an active storage capacity of 10,540 acre-feet and a surface area of 310 acres at spillway crest and normal reservoir elevation 2318.68 (2324.25 original datum).

The uncontrolled automatic siphon spillway and gate tower are located in the left abutment of the embankment dam. A trashrack prevents debris from entering the spillway. The siphon spillway consists of a trashrack intake structure, a vertical shaft, and a "goose-neck" transition section that leads to the downstream outlet conduit. The spillway discharge capacity at reservoir water surface elevation 2318.68 (2324.25 original datum) is 400 ft^3/s.

Outlet releases are controlled by two 3-foot by 4-foot 6-inch cast-iron slide gates with two hand-operated gate lifts. The gates have a combined discharge capacity of about 500 ft^3/s at reservoir elevation 2318.68 (2324.25 original datum) when operated separately from the spillway. The outlet works consists of a trashrack intake structure, a 4-foot 6-inch diameter upstream conduit, a control tower containing the slide gates, a 4-foot 6-inch diameter downstream conduit, and transition into an open cut channel. The concrete conduit has a minimum concrete thickness of 9 inches. Both the outlet works and spillway share a common downstream conduit with a transition immediately downstream from the gate.

Man-entry inspection of the downstream conduit was preformed in November 2000 and revealed poor quality concrete on eroded lift lines where, in at least one location, a ruler could be inserted beyond the thickness of the concrete. The most severe damage to the concrete was located approximately 110 feet upstream from the outlet portal.

A decision was made to perform concrete repairs on the poorest areas the downstream conduit. In preparation of the repair work in March 2001, a tap was inserted into a crack in the conduit approximately 190 feet upstream of the outlet portal in order to control seepage that was entering the conduit. Figure B-72 shows

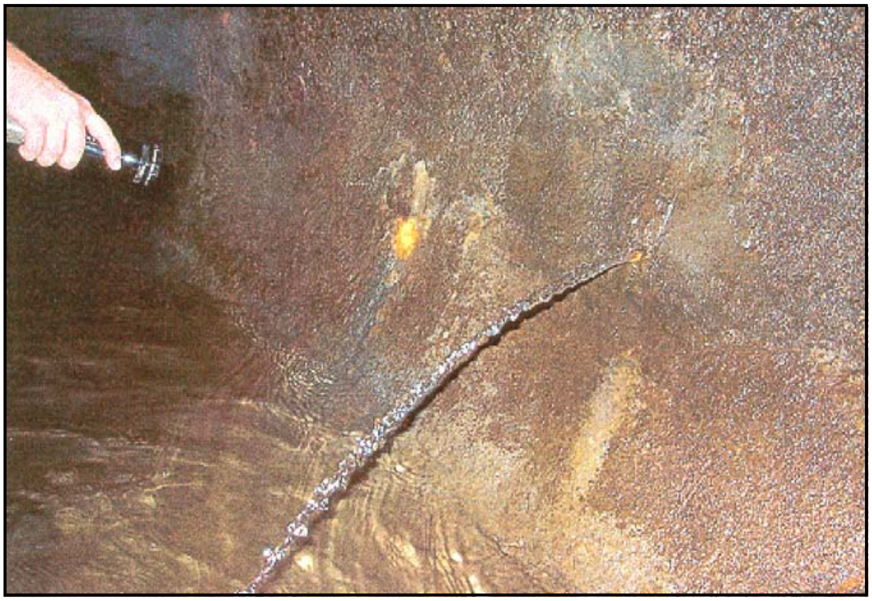

Figure B-72.—Seepage entering the downstream conduit.

the seepage into the conduit prior to installation of the tap. At this location, it was discovered that a void approximately 12 inches in depth existed behind the conduit wall. The concrete in this section of the conduit, which had previously been described as good, was determined to have 3 to 4 inches of somewhat sound concrete, backed by approximately 6 inches of loosely bonded aggregate or rubble. In subsequent explorations, it was determined that the void extended a minimum of 3 feet upstream and 4 feet downstream from the tap and was approximately 3 feet high.

The repairs amounted to jackhammering the eroded areas and soft spots and patching holes in the concrete. Repairs were made at locations identified during man-entry inspections and at some other locations where poor quality concrete was encountered. A largely unsuccessful attempt was made to grout the void behind the conduit wall located about 190 feet upstream of the outlet portal. This was attempted using a hand grout pump, which proved to be inadequate (partially due to the existing seepage gradients). One observation during concrete repairs was that only hoop steel was encountered in the concrete conduit.

Additional man-entry inspection was performed in May 2001 following the completion of the concrete repairs with the intent of evaluating possible modification alternatives. The concrete deterioration and erosion in the interior of the downstream conduit was considered unusual and possibly an indication of concrete with low strength and poor durability. The current condition of the concrete indicates that deterioration and leakage into and out of the conduit will worsen with time. The condition of the concrete may also be indicating a potential for loss of structural strength over time. Water tests were performed to determine if the water in the reservoir and the conduit were acidic or contained anything that would be detrimental to the concrete. The water tests results were negative, and it was concluded that the condition of the concrete could be a result of poor consolidation and possibly other poor construction practices.

The upstream conduit had never been inspected, since the reservoir would need to be drawn down to the level of the intake structure. In lieu of reservoir drawdown, an underwater inspection was performed in September 2001 to assess the condition of the concrete. The divers found the concrete in the intake structure to be in very good condition with no signs of deterioration. The concrete in the upstream conduit did not appear to be in as good of condition as the intake structure. The divers inspected and videotaped approximately 80 linear feet of the conduit. A knife was used to probe cracks and deteriorated areas in the concrete. Figure B-73 shows the diver using a knife to probe a crack in the conduit. In general, the crown of the upstream conduit was in the best condition, with most of the concrete being smooth and free of voids. The floor of the conduit is mostly smooth to $^1/_8$-inch relief. The sides of the conduit were in the poorest condition with concrete relief being $^1/_8$ to $^1/_4$ inch thick with localized areas to $^3/_4$ inch thick. Some areas of unconsolidated concrete were observed on the sides of the upstream conduit, but did not appear to be as severe as what was observed on the downstream conduit. The divers also inspected the upstream sides of the two 3-foot by 4-foot 6-inch cast-iron slide gates. During the inspection, the divers took care not to stir up particles on the invert, to avoid reducing the water visibility to near zero.

Due to the poor condition of the concrete within the downstream conduit and voids on the outside of the conduit, a decision was made to perform outlet works modifications to mitigate the existing dam safety deficiencies. In 2003, a 48-inch inside diameter steel liner was installed, and the annulus between the steel liner and existing concrete conduit was backfill grouted. As part of the outlet works modifications, a filter collar was installed around the downstream end of the conduit. The upstream conduit was determined to be of adequate condition to continue in service without repair, but underwater inspection should be made at regular 6-year intervals.

Lessons learned

- Man-entry inspection should be used to evaluate the condition of the conduit for both temporary repairs and permanent renovations.

- Where feasible, divers should be used to perform underwater inspections of conduits that cannot be dewatered.

Figure B-73.—The diver used a knife to probe cracks in the concrete. Divers prefer knives with blunt ends in underwater inspection, since it is less likely that a hole could accidently be poked into their dry suits.

Reference

Bureau of Reclamation, *Report of Findings—Spillway and Outlet Works Conduit Modifications—Corrective Action Alternatives*, February 7, 2002.

Bureau of Reclamation, *Design Summary—Salmon Lake Dam Modifications, Okanogan Project, Washington*, October 2003.

Project name: Sardis Dam

Location: Mississippi

Summary: A sinkhole developed over an outlet works conduit due to material being eroded through a joint

Sardis Dam is a hydraulic fill embankment dam constructed by the Corps of Engineers and was placed in service in 1940. Sardis Dam is 15,300 feet in length with an average height of 97 feet. The outlet works is located in the left abutment. The outlet works consists of an intake tower with four gated passages, and these passages transition in a 64-foot long monolith to a single "egg" shaped reinforced concrete conduit. The 18.25- by 16-foot conduit is founded on fine Tertiary sand and was cast in place. The walls of the conduit are 3.25 feet thick. The conduit consists of 17 monoliths, each 30 feet in length. Copper waterstops were placed at each monolith joint. The conduit discharges into a concrete stilling basin, which has baffle blocks for energy dissipation.

In December 1974, a sinkhole occurred above the monolith joint at the junction of the intake tower and the upstream end of the transition monolith. Figure B-74 shows the location of the sinkhole. Sinkhole investigation revealed that the intake tower is founded on piles, but the transition is not founded on piles. This allowed the transition monolith to settle about 1 inch more than the intake tower. This differential settlement was enough to rupture the copper waterstop. With water in the conduit being free flowing (nonpressurized conduit), and water pressure outside the conduit being near lake stage, a large pressure differential existed across this joint. This large pressure differential caused flow through the joint after the waterstop ruptured. The water flowing through the joint carried enough material to eventually cause the sinkhole to occur.

The solution to this problem was to fill the sinkhole with impervious material and to drill grout holes in this monolith joint all the way through the concrete into the surrounding soil along the entire perimeter of the joint. The holes were drilled from inside the transition. Neat cement grout was then pumped through these holes to fill any voids outside the transition and to seal the waterstop as well as possible. Prior to grouting, the gates were closed and sealed with saw dust, air compressors and a grouting machine were set up in the backfill area of the stilling basin, and supply lines were run up the conduit to the transition monolith to be grouted. The lake stage was at its normal level for that time of the year, and the elevation of the lake was about 20 feet above the invert of the conduit. Twelve grout holes were drilled in the monolith joint, four in the invert, four in the crown, and two in each side wall. Each hole was installed as follows: (1) A hole about 2 inches in diameter was drilled

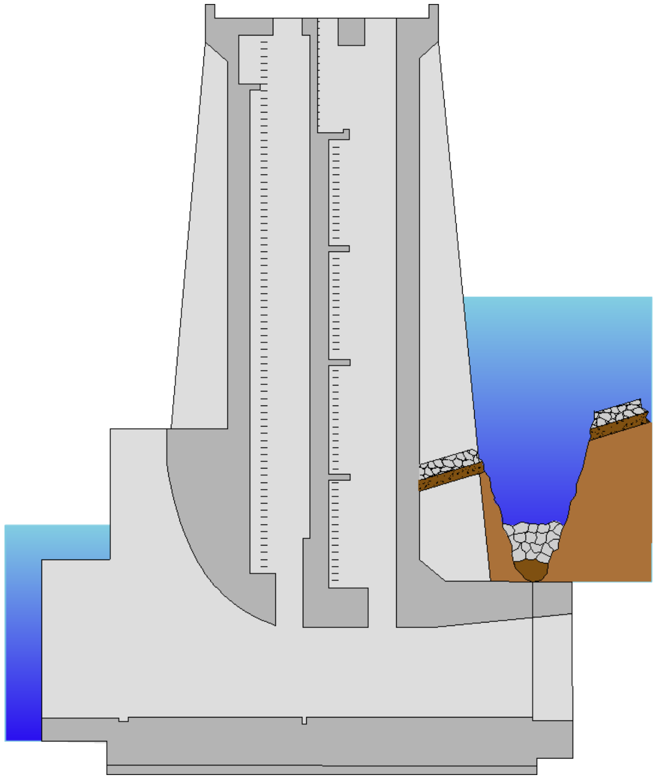

Figure B-74.—A sinkhole occurred above the monolith joint at the junction of the intake tower and the upstream end of the transition section.

with a jack hammer to a depth of about 2 feet, and (2) a 1½-inch pipe with a ball valve on the upper end was then grouted into the hole. After the grout had set up, a jack hammer with bit small enough to go inside the 1½-inch pipe was used to drill the rest of the way through the conduit to the foundation or backfill material. When foundation or backfill material was encountered, the holes would start to flow. The jack hammer with bit was then removed, and the ball valve was closed to prevent the hole from flowing. A water jet pipe was used to clean out the grout pipe just prior to grouting. At the end of the grouting operation, the ball valves were removed, and caps were placed on the grout pipes.

The first hole grouted took 12 cubic feet, and the take per hole decreased for each succeeding hole. A total of 40.5 cubic feet of neat cement grout was pumped around the joint.

The cement grout was anticipated to be brittle, and there was some concern that any additional settlement of the foundation could cause this neat cement to crack. However, this repair was completed more than 30 years ago and no other sink holes have developed.

Lessons learned

The problem was caused by a broken waterstop, which was the result of differential settlement. To prevent this on future designs, the monolith joints should be designed so that there can be essentially no differential settlement at the monolith joint.

Reference

Sardis Lake Project, Little Tallahatchie River, Mississippi; Dam, Outlet Works, and Spillway; Periodic Inspection Report No.2, Supplement D, September 1971.

Project name: Sugar Mill Dam

Location: Georgia

Summary: Siphon spillway failure

Sugar Mill is a residential subdivision that was developed in the early 1990s in north Fulton County, Georgia (Atlanta metropolitan area). A central amenity of the development was an existing reservoir impounded by an old earthen embankment dam with inadequate spillway capacity.

In addition to widening the earthen emergency spillway, five PVC siphon pipes were installed in trenches excavated through the crest of the embankment. The design called for the pipes to be bedded in concrete. Control valves were installed in the siphons at the top of the embankment dam inside of manhole structures.

In 2002, the owner noted the presence of water flowing out of a hole in the embankment adjacent to the siphons, approximately 15 feet downstream of the valve manhole. The owner contacted the design engineer, who performed exploratory investigations in an attempt to locate the source of the seepage. The engineer recommended installation of a drainage system to control the seepage. However, this did not work, and the seepage situation continued to get worse. In 2003, during a storm, the owner attempted to operate the siphon spillways, and found the manholes full of water and that the seepage had substantially increased.

An internal CCTV inspection of the siphons found no problems with the PVC pipes. The engineer suspected that flow was occurring under the pipes and advised the owner to replace the siphons. Upon excavation and removal of the pipes, it was found that the original contractor had not achieved adequate placement of the concrete cradle, resulting in voids under the center of each siphon. Constant flow through these voids under the pipes resulted in internal erosion of the underlying embankment soils.

Lessons learned (adapted from Wilson and Monroe)

- Siphon spillways do not always work as expected, especially when constructed by an inexperienced contractor. Thorough construction oversight is required.

- A filter should be used in conjunction with conduit penetrations through embankment dams.

References

Wilson, Charles, and Joseph Monroe, *Dam Surgery—Repairs to Sugar Mill Dam, Fulton County, Georgia*, presented at the ASDSO Southeast Regional Conference, Norfolk, Virginia (entire reference is not available), 2004.

Sugar Mill Community Association, Minutes of Board of Directors' meetings: April 18, 2002; May 7, 2002; and January 14, 2003

Project name: Turtle (Twin) Lake Dam

Location: Montana

Summary: Sliplining of an existing outlet works conduit using HDPE

Constructed in 1932 by the U.S. Indian Irrigation Service (now Bureau of Indian Affairs), Twin or Turtle Lake Dam is a 20-foot-high embankment dam, located about 4 miles southeast of Polson, Montana on the Flathead Indian Reservation. The original outlet works at Turtle Lake Dam was constructed by cut and cover methods. The outlet works conduits were constructed of concrete pressure pipe in approximately 4-foot long sections. The upstream conduit and the first 40 feet of the downstream conduit were 21-inch diameter pipe. The vertical wet well shaft at the upstream edge of the dam crest separates the upstream and downstream conduits and houses a 24-inch diameter slide gate, which regulates discharges. The remainder of the downstream conduit (340 feet, much of which extends downstream of the toe of the embankment dam) is 18-inch diameter.

During the 1996-1997 winter, a sinkhole was discovered above the lower reaches of the downstream conduit. Investigations showed that a root ball had partially plugged the outlet, and portions of the pipe had partially collapsed. During the late spring of 1997, the Bureau of Reclamation designed a temporary repair for the deteriorating outlet works. Because of the small size of the conduit and the concern for seepage coming into the conduit, it was decided that a watertight liner should be used. The conduit downstream of the embankment dam toe was excavated, but it was not desirable to excavate in the embankment dam itself. For this reason and because remotely CCTV inspection seemed to indicate potential for offsets and changes in the conduit alignment, an HDPE sliplining was proposed.

A 16-inch O.D. HDPE pipe was sliplined (figures B-75 and B-76) into the existing downstream conduit beneath the dam up to the regulating gate, and the annulus between the existing and new pipes was grouted with a cement grout with superplasticizer. Downstream of the dam toe, the pipe transitions to a 22-inch O.D. pipe, which extends downstream to the original portal location.

The HDPE-lined outlet works at Turtle Lake Dam has been in operation since modification without further incident. A CCTV inspection was conducted in April 2001. The inspection indicated that the HDPE slipliner was performing well.

Figure B-75.—HDPE slipliner being installed at Turtle (Twin) Lake Dam. Note the HDPE grout lines welded onto top of new liner. The end flange was used to attach the two different sized liners together.

Figure B-76.—Upstream end of the HDPE slipliner modified to act as a pulling head.

Lessons learned

Sliplining provides a low cost and less disruptive alternative to the conventional remove and replacement renovation method.

Reference

Cooper, Chuck, Ernest Hall, and Walt Heyder, *Case Histories Using High Density Polyethylene Pipe for Slip-Lining Existing Outlet Works and Spillways*, October 2001.

Project name: Upper Red Rock Site 20 Dam

Location: Oklahoma

Summary: Failure of an embankment dam by internal erosion resulting from hydraulic fracture of earthfill adjacent to the flood control conduit

Upper Red Rock Site 20 Dam was a low hazard earthfill embankment structure constructed by the NRCS for flood control in 1973. The embankment height was about 31 feet and it contained about 61,000 cubic yards of earthfill. The principal spillway conduit is a 36-inch diameter reinforced concrete pipe. The embankment dam was constructed in an area of Oklahoma now known to have a high concentration of dispersive clays. The embankment soils classify as CL in the Unified System with an LL of about 35 and a PI of about 15. The dispersive clays are produced from weathering of Permian or Pennsylvanian age shales of marine origin. The sodium rich parent material produce dispersive clays that are highly erodible.

The embankment dam failed in 1986 by internal erosion when the reservoir filled suddenly to a reservoir elevation higher than the reservoir had previously ever impounded water. At the time of the failure, the reservoir was about 1.6 feet below the top of the crest of the embankment dam. The embankment dam had impounded water continuously at a lower reservoir elevation for most of its history, until the rainfall event that filled the reservoir to this unprecedented higher elevation.

Most often, failures similar to this one occur when the reservoir fills suddenly soon after completion of the embankment dam. This failure occurred however when a "second first filling" type of event occurred. The site had previously filled to a pool level corresponding to about one-third of the embankment dam height (the dam is a flood control single purpose reservoir), and maintained that pool for most of the 13 years of its life. A large rainfall event caused water to rapidly fill the reservoir and water flowed through the auxiliary spillway, but not over the crest of the embankment dam in 1986. Cracks in the earthfill in the upper part of the embankment dam allowed water to find a pathway through unsaturated dispersive clay fill used to construct the embankment dam. The crack(s) in the embankment dam quickly eroded and a breaching type failure of the dam resulted. The cracks in the embankment dam were thought to be a result of a combination of hydraulic fracture and desiccation. The failure tunnel was located about 40 feet to the side of the conduit that penetrated the embankment dam, in the area of the old stream channel.

The differential settlement that helped to create stress conditions favorable to hydraulic fracture was probably associated with the presence of a channel through the embankment dam that had relatively steep side slopes and the presence of the principal spillway conduit in the vicinity. Internal erosion failures in similar embankment dams constructed by the NRCS in Oklahoma were often near principal spillway conduits. Differential settlement is a primary contributor to conditions favorable to hydraulic fracture.

The failure of the embankment dam was observed as it occurred from an aerial survey of the site. Figure B-77 shows the failure of the embankment dam as it occurred. Water in the reservoir had risen to about 1.6 feet below the crest of the embankment dam following about 19 inches of rainfall which had occurred over several days. Water had flowed over the crest of the auxiliary spillway, but had not overtopped the embankment dam. Water was observed to be flowing through a tunnel developing in the embankment dam at about 40 feet to the side of the conduit location. Water entered the upstream slope of the embankment dam at about the maximum reservoir level in several locations and exited the downstream slope of the embankment dam through an erosion tunnel. Water exited the downstream slope about one-third of the way up from the toe of the embankment dam. The tunnel that developed in the embankment dam eroded quickly and drained the pool to about one-half of the embankment height.

Figure B-77.—Aerial view showing the failure of Upper Red Rock Site 20 Dam.

Lessons learned

The incident demonstrated that although failures by hydraulic fracture may be most common in first filling incidents, a potential for failures still exists years after the first filling. The soils were known to be highly dispersive, but designers thought that by placing the dispersive soils above the elevation of the permanent pool, the threat to the embankment dam could be reduced. Embankments constructed of dispersive clays are extremely susceptible to internal erosion failures unless protected by chimney filter zones.

Reference

Oklahoma NRCS State Office files.

Project name: Waterbury Dam

Location: Vermont

Summary: Design and construction of a filter diaphragm around an existing outlet works conduit

Waterbury Dam is 150 feet high across much of a valley except over the original river gorge, where it approaches 190 feet high. The embankment dam consists of a wide central impervious core (CL and ML) flanked by sand and gravel (SM and GM) shells upstream and downstream. A large rock fill zone extends across the toe of the main embankment section. The dam is founded on a thick glacial silt deposit beneath the western two-thirds of the embankment dam, and directly on the schist bedrock beneath the eastern third of the dam.

The internal erosion of embankment dam and/or foundation silts into and through the rockfill zone attracted attention in the late 1970s. A section of the rockfill toe along the western portion of the embankment dam was reconstructed and treated by a filter injection process in the mid-1980s. At that time, a separate internal erosion condition was revealed in the portion of the embankment dam over the original river gorge. Internal voids were treated using the filter injection process.

From 1985 through the late 1990s, seepage conditions within the gorge area of the dam failed to completely stabilize. In 1999, investigations concluded that additional remedial action was needed. The remediation included placing a filter drain to intercept any seepage that might occur along the interface between the conduit and the embankment materials around the downstream end of the existing outlet conduit.

The outlet conduit consists of a horseshoe-shaped reinforced concrete conduit placed within a bedrock excavation. Within the impervious core zone, the conduit is mostly within the confined bedrock excavation, and impervious material was compacted using hand tampers. The east wall of the excavation consists of a near-vertical excavated rock wall with some localized overhanging rock ledges. These conditions led to the concern for potential internal erosion along the conduit or along the interface with the steep rock surface.

Exposure of the conduit required a large excavation into the downstream sand and gravel shell (20,000 yd^3) and rock fill (10,000 yd^3) zones of the embankment dam. Hand-placed riprap from the dam surface was stripped, processed to remove fine materials, and eventually replaced on the embankment surface. Within the shell, excavation slopes were 1.5H:1V, and within the rock fill slopes were 1H:1V. In spite of several very hard rains, there were no stability problems. About 160 feet of the conduit, entirely within the rockfill zone, was exposed.

The drain, consisting of a coarse inner zone and a fine outer filter zone, was wrapped around the outer surface of the exposed conduit. At the upstream end of the excavation, an expanded filter diaphragm was placed from the eastern bedrock surface to the western rock fill excavation surface. The diaphragm extended about 5 feet above the crown of the conduit.

The drain consisted of a coarse drainage fill (AASHTO No. 7 stone) to act as a carrying medium for collected seepage, and a finer sand zone (Vermont concrete sand) to act as a filter to prevent the infiltration of fines from the surrounding rockfill materials. The 18-inch thick course zone was placed around the drainpipe and filled the irregular space between the overhanging eastern bedrock wall and the conduit surface. The fine filter zone was also a minimum of 18 inches thick, but was broadened to fill the excavation wedge along the western side of the conduit. See figure B-78 for cross section.

The filter and inner coarse drainage fill zone were designed to meet filter criteria with respect to each other, and the filter was designed to handle the silts within the fine matrix material within the adjacent rock fill zone. The low-plasticity glacial silts, whether foundation silts, core material, or rock fill matrix fines, have always been the primary concern with respect to internal erosion. A relatively fine filter material is required to meet filter criteria.

The exact limits of the rockfill zone were not known, although some of the original drawing information suggested that the proposed excavation would completely penetrate the rock fill to the contact with the shell zone. As the excavation progressed, it became apparent that the rock fill zone would not be completely penetrated as hoped. Therefore, it was decided to install a horizontal drain through the remaining wedge of rockfill to intercept seepage before it dispersed into the rockfill zone. The drain was drilled within the confined space between the west side of the conduit and the bedrock surface. Drilling was very difficult due to the presence of an unknown concrete plug adjacent to the conduit. However, the drain was completed, and seepage flows were intercepted near the upstream limits of the rockfill zone. The discharge pipe from the horizontal drain was extended to the downstream toe of the dam, along with the drainage pipe placed at the bottom of the conduit drainage zone along both the east and west sides of the conduit.

Because of the confined spaces and irregular bedrock surfaces, the drainage and filter zones were placed in thin lifts and compacted with hand-operated vibratory compaction equipment (figure B-79). Although the work was time consuming, the uniformly graded materials were very easy to place and compact. To accommodate

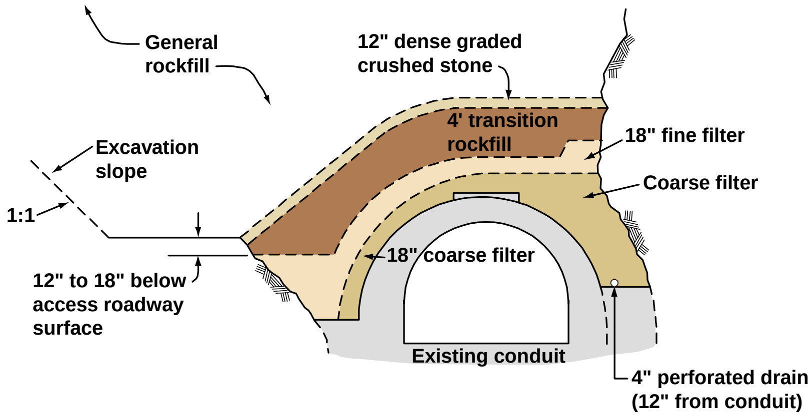

Figure B-78.—Cross section showing filter zone and conduit.

Figure B-79.—Filter material being placed in thin lifts and compacted with hand-operated vibratory compaction equipment.

construction, the zones were expanded on both the east and west sides of the conduit to reach to the exposed bedrock surfaces paralleling the conduit before switching back to rockfill materials. A transition zone of smaller rockfill was placed immediately adjacent to the filter before going to general rock fill backfill materials.

Lessons learned

- The designers must be actively involved onsite during the construction phase as the actual subsurface conditions are revealed. Designers should not rely on construction personnel to make critical judgments about the need for field changes—especially where filter and drain features are involved.

- The highest risk to the embankment dam occurred when the excavation was at the maximum extent, and before the new drains were installed. The contractor was required to prepare a contingency plan for mobilization of a horizontal drilling specialist to minimize the time the excavation had to be open to the maximum depth. When it was determined that a horizontal drain would be required, the drilling contractor was mobilized. The drilling contractor was onsite by the time the excavation was completed, resulting in minimum impact on the duration of the critical phase of the excavation.

- This type of construction, due to subsurface conditions, is difficult to investigate. Since as-built records from that construction era were lacking, a high degree of uncertainty was involved. Anticipating field changes is required, as well as carrying a higher contingency than might be necessary for above-ground construction.

- Any evidence of slime bacteria deposits should be addressed as a potentially serious problem for filter and drain features.

Reference

U.S. Army Corps of Engineers, Summary Report CENAB-EN-GF, *Waterbury Dam—Seepage Control Modifications*, January 30, 2003.

Project name: Willow Creek Dam

Location: Montana

Summary: Lining of an existing outlet works conduit using CIPP

Willow Creek Dam is located in western Montana. The 84-foot high embankment dam impounds a reservoir of 32,300 acre-feet, used primarily for irrigation. The outlet works consists of a 54-inch diameter concrete-lined tunnel through the right abutment, with guard and regulating gates provided within a gate shaft upstream of the dam axis. The embankment dam and outlet works were originally constructed between 1907 and 1911, and were modified in 1917 and 1941. The Bureau of Reclamation owns the embankment dam, and the Greenfields Irrigation District operates it.

A large sinkhole was discovered on the crest of the embankment dam in June 1996. The sinkhole was located directly above the outlet works tunnel, about 50 feet downstream from the gate shaft and near the dam axis. Earth materials were found to be eroding periodically from a 1-inch weep hole in the tunnel sidewall. Four weep holes in the tunnel lining were sealed, and the sinkhole was temporarily stabilized by backfilling with sand and gravel materials. The reservoir was gradually lowered using the outlet works, for a total drawdown of 27 feet.

Excavation of the embankment dam revealed the sinkhole extended through 40 feet of bedrock to a large cavity surrounding the concrete tunnel lining. Tremie grout was used to fill the voids around the tunnel, followed by the placement of backfill concrete to the excavated bedrock surface at elevation. The embankment dam was restored by the placement of a filter blanket on the excavated foundation, followed by the placement of compacted glacial till materials to dam crest elevation, and replacement of the slope protection on the downstream face.

The outlet works tunnel was originally excavated in 1907 by hand-drilling and blasting, with considerable water and soft materials encountered. Heavy timber beams and posts with timber lagging were used to support the tunnel excavation throughout its length, resulting in a square, excavated opening for the circular tunnel lining. Although it is unclear how the concrete tunnel lining was actually constructed, the specifications called for a uniform concrete thickness of 8 inches, without reinforcing bars, and the placement of "puddled fill" outside the tunnel lining to the excavated surface. Such construction could have resulted in a significant quantity of fine grained backfill material along the outlet works tunnel.

The downstream tunnel lining was severely damaged in 1958, when maximum outlet releases of 550 ft^3/s were reported. Approximately 70 feet of unreinforced concrete in the tunnel invert was removed by the flow, beginning 10 feet downstream from the regulating gate, and the foundation rock was eroded to a depth of 3 feet. A 10- to 15-foot long section of the tunnel crown was also removed, revealing a large void surrounding the tunnel. The structural damage is believed to have been initiated by negative pressures resulting from an insufficient air supply to the downstream tunnel during maximum releases, due to an undersized, 6-inch diameter air vent pipe. Repairs included replacement of the missing concrete invert and crown, and placement of rubble fill outside the tunnel lining. Weep holes were later drilled in the tunnel lining for pressure relief.

The development of the large tunnel cavity was probably a combination of overexcavation during construction, gradual erosion of the puddled fill and soft bedrock materials, and collapse of the harder bedrock materials into the tunnel following the lining failure. The sinkhole may have developed gradually by the internal erosion of glacial till embankment materials through open joints and fractures in the bedrock, progressive collapse or "stoping" of the bedrock into the void below, and erosion of earth materials through open cracks and weep holes in the tunnel lining.

Continued concern for the long term stability and structural integrity of the downstream tunnel lining, and the potential for renewed erosion of earth materials through open cracks and joints (despite grouting efforts) resulted in the consideration of tunnel lining options. The downstream tunnel extends 429 feet from the regulating gate to the downstream portal, where outlet releases enter a diffusion-type stilling basin. A structural lining was required for the first 100 feet of tunnel, which seemed to be the most susceptible to future problems, since it included the portion damaged in 1958, the sinkhole location, a significant longitudinal crack along the crown, and continuing seepage from various other open cracks and joints, and was located directly below the wide embankment dam crest.

The configuration of the stilling basin at the downstream portal, and grade changes within the tunnel (including one of over 3 degrees), made the proposed installation of a rigid structural lining more difficult. So the search for alternatives to rigid linings focused on CIPP systems, originally introduced in the United States by Insituform Technologies in 1977. A CIPP lining consists of a flexible, resin-impregnated, needled polyester felt tube, which is expanded under hydrostatic head, and cured by the circulation of heated water. Construction access through the outlet works gate shaft, for installation of a CIPP lining from the upstream end of the tunnel, would have been severely affected by the gate house and existing mechanical equipment, including the gate operator and stem, air vent pipe, ladders, and landings. Installation of a CIPP lining by the inversion method from the downstream portal would have resulted in the exposure of the entire unreinforced concrete lining to high water temperatures, and the requirement of an additional 329 feet of waste tube material. A finite element analysis of potential thermal stresses within the 8-inch concrete lining, using an ABAQUS computer program, predicted large tensile stresses sufficient to produce extensive cracking, which was unacceptable. Use of an alternative low temperature resin, with a curing temperature of only 80 °F, would avoid thermal stresses and produce acceptable results, but would require special handling and a longer curing period.

Reclamation prepared design specifications for a partial tunnel lining using CIPP and issued them in May 1997. A construction contract was awarded to the low bidder, Western Slope Utilities, Inc. (WSU) of Breckenridge, Colorado, in July 1997. An InLinerUSA licensee, WSU was experienced in the pulled-in-place installation method for linings up to 36 inches in diameter, and obtained the services of an InLinerUSA representative with the required experience for large diameter linings.

For design purposes, the existing tunnel was assumed to be in a “fully deteriorated” condition (due to the longitudinal crack in the crown) and subject to internal pressure under maximum discharge conditions. Design loads included a 10-foot external fill height on the tunnel crown, a 10-foot external hydrostatic head on the tunnel invert, and a maximum internal pressure of 20 lb/in^2. The CIPP was designed to carry the external loads with no contributing support from the circular tunnel lining with a factor of safety of 2.0. An ovality reduction factor, based on the average minimum and maximum diameters of the tunnel lining, was included to properly estimate the stiffness of the elliptically deflected pipe. For internal loads, the CIPP was designed as a thin-walled cylinder with uniform pipe wall stresses, using a hoop stress equation for plastic pipe.

An epoxy vinyl ester resin was selected over a polyester resin for greater strength and longevity. Design properties for the resin included an initial flexural modulus of 300,000 lb/in^2 and an initial flexural strength of 5,000 lb/in^2 for external loads, and an initial tensile strength of 3,000 lb/in^2 for internal loads. To characterize the long term performance of the CIPP over the minimum 50-year design life, a 33-percent creep reduction was assumed for the flexural modulus and flexural strength, and a 50-percent hydrostatic stress regression was assumed for the tensile strength. The final design thickness for the CIPP was 1.06 inches, including an additional 5 percent thickness to provide sufficient resin to fill the interior felt of the calibration hose, which was to remain in place.

The contractor began mobilizing equipment at the embankment dam on August 8, 1997. A steel platform was installed 12 feet above the bottom of the regulating gate shaft, and a steel elbow section was centered within the upstream end of the tunnel, to support a short flexible hose for a water column. One end of a 110-foot long calibration hose, consisting of a single layer of felt fabric with a watertight polyurethane coating, was carefully lowered down the shaft and through the flexible hose and elbow, where it was turned inside out and securely fastened to the outside of the elbow. A winch and roller were set up at the gate house doorway, and a second roller was positioned at the bottom of the shaft. The tunnel surfaces were swept clean, and utility lines (for lighting, ventilation, and water circulation) were established within the shaft.

The resin-filled tube was delivered to the site on August 11 in a refrigerated truck (figure B-80). The nonwoven fabric tube was manufactured in Houston, Texas at InLinerUSA headquarters, and the resin was added in Alma, Colorado at a "wet-out" plant used by WSU. Total weight of the liner was 10,000 pounds. The liner was removed from the refrigerated truck using a truck-mounted winch, and was carefully fed into the tunnel at the downstream portal and slowly pulled upstream. The liner was pulled into final position in the tunnel within about 1.5 hours and was securely fastened to the steel elbow outside the calibration hose. Reservoir water from the upstream gate shaft was pumped into the water column to begin inversion of the calibration hose under a 1-foot head. Within 20 minutes, the calibration hose had been turned inside out and extended the full length of the liner, pressing the liner tightly against the tunnel surface. Two perforated water supply hoses inside the calibration hose were used to circulate heated water from a heat exchanger truck under the full 12-foot head.

Return water temperatures at the truck reached 135 °F in 2 hours and were held constant for 4 hours, and then were raised to 175 °F within 1 hour and were held constant for 6 hours for curing the resin. After curing was completed, the circulating water was gradually cooled to 100 °F in 4 hours, finishing by noon on August 12. Epoxy vinyl ester resin contains styrene, a possible carcinogen, which is released during the curing process. Styrene vapors are heavier than air, and potentially flammable and explosive. Installers and inspectors must follow OSHA regulations pertaining to workers in hazardous and confined spaces. Fresh air had to be introduced into the tunnel before the contractor could cut a small hole in the end of the hardened liner to release the water.

The waste water was fully contained within the downstream stilling basin to permit final cooling to 70 °F, removal of resin residue from the water surface, and dissipation of dissolved styrene.

Both ends of the liner were trimmed using chain saws and circular saws, and a 0.5-inch deep groove was provided around the periphery to accommodate installation of end seals. Amex 10/WEKO seals were used, each consisting of a 14.5-inch wide rubber seal with three stainless steel bands spread by a hydraulic expanding device to ensure a tight fit. The work was completed on August 15, one week after site mobilization, for a total cost of about $145,000. Subsequent laboratory tests on field samples confirmed the design parameters for tensile and flexural properties.

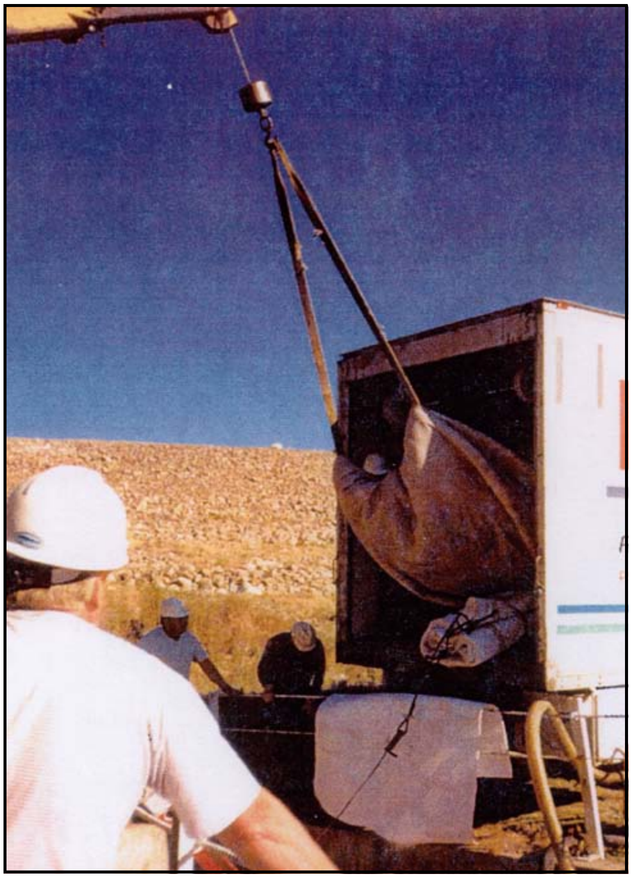

Figure B-80.—Lowering of the CIPP liner into the stilling basin, so it can be winched up the tunnel.

The CIPP installation has been performing satisfactorily since completion of construction.

Lessons learned

- CIPP can be used for conduit renovation.

- Use of CIPP can be applicable to conduits with changes in invert slope. CIPP provides a conduit lining with minor loss of flow cross-sectional area.

Reference

Hepler, Tom, Ron Oaks, and Roger Torres, *Sinkhole Development and Repairs at Willow Creek Dam, Montana*, ASDSO Conference, 1997.

Project name: Wister Dam

Location: Oklahoma

Summary: Near failure of an embankment dam due to internal erosion

The descriptions of this case history are extracted from several articles written by Sherard, including his 1986 article, and an article by Rutledge and Gould (1973). Arthur Casagrande (1950) also discussed this case history.

Lake Wister is located in the San Bois Mountains on the Poteau River in far eastern Oklahoma. The Tulsa District Corps of Engineers designed and built the project. Construction began in April 1946, and the project was placed in full flood control operation in December 1949. The embankment dam is a rolled, impervious earthfill with rock-protected slopes. The embankment dam was constructed as a homogeneous clay fill without a chimney filter. At the time the embankment dam was constructed, chimney filters were not a standard design element in major dams as they are now. The embankment dam is 5,700 feet long and rises to a maximum height of 99 feet above the streambed. Later tests on soils from the embankment dam conclusively demonstrated that the clays were highly dispersive. The bedrock in the area is Pennsylvanian age shale known to commonly produce residual soils with dispersive properties.

Heavy rains caused the reservoir to fill quickly beginning in January 1949. When water had risen to a height of about 60 feet, muddy water was seen discharging on the downstream slope of the embankment dam. The quantity of flow was initially estimated at 2,200 gal/min, and it increased to about 8,000-9,000 gal/min in the next several days. The spillway radial gates were opened, and within 3 days, the reservoir had dropped about 13 feet. This exposed tunnels on the upstream slope, through which the water was entering the embankment dam. The tunnels were about 2 feet in diameter and extended along the upstream face of the embankment dam for a distance of about 300 feet, at about the same elevation on the slope. Dye was injected into a vortex on the upstream slope, and the test showed the water was flowing along a nearly horizontal seam in the embankment dam for a distance of about 740 feet, with a head on the tunnel at the time flow began of only about 13 feet (a gradient of a little over 50:1). The dye tests showed the time for flow to travel this distance was less than 13 minutes, a velocity of about 1 foot per second. Figure B-81 below shows an idealized sketch of the embankment cross section with the flow path causing the erosion identified.

After the reservoir level had dropped farther, the erosion tunnels exposed were excavated and plugged, and several remedial measures were implemented, including

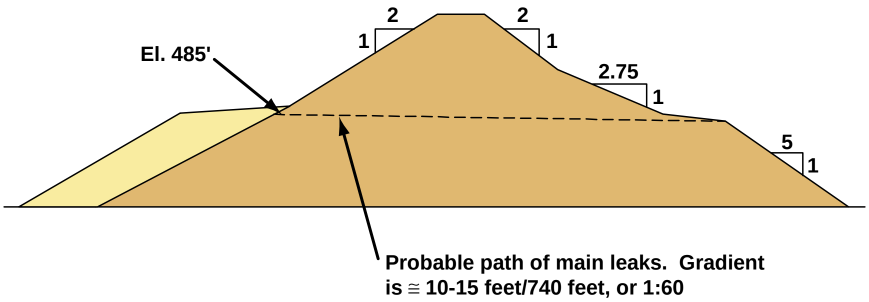

Figure B-81.—Cross section of Wister Dam showing probable path for internal erosion through embankment. The length of the flow path was about 740 feet, and the head on the entrance tunnels was less than 15 feet.

extensive grouting, a steel sheet pile wall, and additional upstream and downstream berms and drains. After completion of the remedial work, the embankment dam has been in operation continuously with little trouble. A major renovation program was finished in 1990. The renovation included a slurry panel wall installed for the full height of the embankment.

Sherard concluded that the cause of the leakage path in the embankment dam could only be attributed to hydraulic fracture in the embankment. The flow path developed immediately above the closure section on the embankment dam, also just above the old stream channel. Aggravating this condition was the fact that the right bank of the stream channel (viewed downstream) consisted of a bedrock shelf that contributed to differential settlement. A plan view of the area of the leak is shown in figure B-82. A longitudinal profile of the area where the leak developed is shown as figure B-83. Note that this view is looking upstream.

The embankment dam was compacted to a relatively high dry density corresponding to about 97 percent of the maximum Standard Proctor dry density, and it was compacted at about optimum water content. The soils were silty, relatively low in plasticity, and highly dispersive. Compaction at these conditions probably resulted in a somewhat brittle fill likely to crack when subjected to differential settlement.

Lessons learned

The incident demonstrated that even well constructed embankment dams built by what was then state-of-the-art technology are susceptible to hydraulic fracture and internal erosion if they are not protected by internal chimney filters. Modern embankment dam design concepts include protective filters for all significant and high hazard embankment dams, and even low hazard dams if constructed of

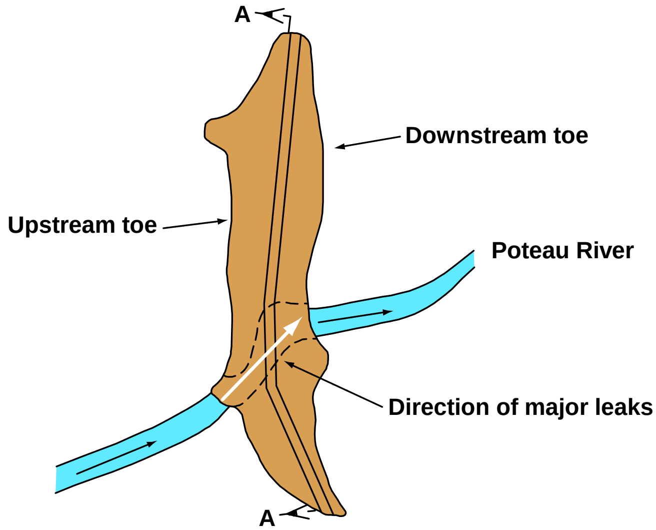

Figure B-82.—Plan View of Wister Dam showing flow path for internal erosion tunnels in fill.

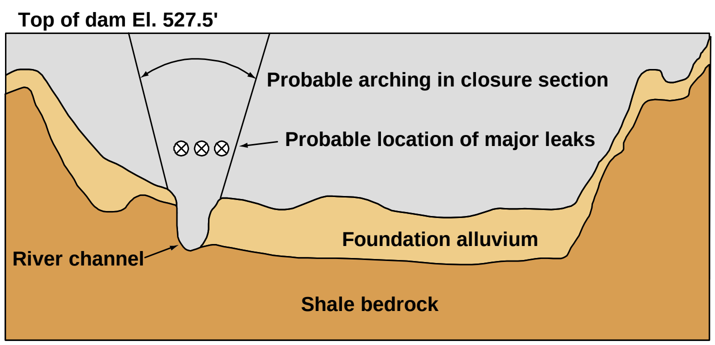

Figure B-83.—Cross section A-A from figure B-82. Profile along centerline of embankment viewed upstream.

problematic soils, such as dispersive clays. This case history also illustrates the increased potential for arching and hydraulic fracture associated with closure sections in embankment dams.

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Rutledge, Philip, and James P. Gould, *Embankment Dam Cracking*, Casagrande Volume, John Wiley and Sons, New York, 1973, pp. 272-353.

Sherard, James L., "Hydraulic Fracturing in Embankment Dams," *Journal of Geotechnical and Geoenvironmental Engineering*, October, 1986.