Repair of Voids Behind Spillways, Conduits, Canals, Tunnels, and Siphons

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Report 3 of 3 on Void Causes, Detection, and Repair

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14. ABSTRACT
Voids form as a consequence of the deteriorated state or poor design of the structures they are associated with. There are a variety of different mechanisms that can lead to the formation of voids adjacent to dam structures (U.S. Bureau of Reclamation, 2019). Determining the appropriate action to be taken after a void has been detected involves the careful consideration of many different factors. Selection of the appropriate method is always site specific. Designers must consider factors such as the type of materials used in the original design as well as common design / construction practices at the time of construction. Some other important factors include downstream impacts, extent and location of damage, size and shape of conduit or spillway adjacent to the void, type of material being eroded, size of dam, and cost of repair. This report details each of the different repair, renovation, and replacement techniques for structures associated with void formation, and provides guidance on the appropriate action to be taken. Additionally, the report provides the results of laboratory testing on permeable low density cellular concrete and foam transported sand as innovative methods to repair voids.

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Acronyms and Abbreviations

ACI	American Concrete Institute
ASTM	ASTM International
Reclamation	Bureau of Reclamation
CSL	Concrete and Structural Laboratory
CIPP	Cured In-Place Pipe
EAP	Emergency Action Plan
FTS	Foam Transported Sand
PLDCC	Permeable Low-Density Cellular Concrete
TSC	Technical Service Center

Measurements

%	percentage
ft	feet
gpm	gallons per minute
in	inches
in/hr	inches per hour
lb.	pounds-force
pcf	pounds per cubic foot
psi	pounds per square inch
k	permeability coefficient

Contents

Executive Summary

Once a void has been identified and located, it is important to enact timely remediation to limit further growth and minimize potential harm to a structure. There are many different options available for remediation, from grout to filter placement or even replacement. Selecting the right tool for a particular remedial approach requires a thorough understanding of the benefits and drawbacks for each technique.

This report is Part 3 of a three-part series written to provide tools for identifying the causes of void formation and where those voids are more likely to form, different tools and analysis techniques to locate voids, and methods for remediating voids.

Report ST-2024-21045-01: Void formation has been attributed to many causes, from soil settlement and frost heave, to movement of materials caused by scour processes or internal erosion. In many cases it is likely a combination of factors. Identifying the processes by which a void formed often occurs as a post-failure forensic analysis, which helps to inform future work. This report identifies geotechnical conditions and soil/rock types that may be more susceptible to void formation, elucidates on design or construction practices that have a higher probability of leading to void formation, and comments on observable indicators of void formation. The report concludes with a series of questions to help prioritize inspections from inventory-wide to structure-scale based on likely void locations.

Report ST-2024-21045-02: Detection of voids along conduits has been attempted in the past using thermography and the use of a crawler with a sophisticated wall-tapping system. While progress has been made, underlying difficulties exist that have resulted in only limited adoption of these systems. Developing a set of methods and analysis techniques that can help locate voids in a variety of different mediums (concrete, metal, etc.) in a variety of different conditions (dry, wet, below/above the water table, etc.) is a valuable tool for use in facility inspections. This report details a number of non-destructive tools that can be used for void detection.

Report ST-2024-21045-03: Determining the appropriate action to be taken after a void has been detected involves the careful consideration of many different factors. Selection of the appropriate method is always site specific. Designers must consider factors such as the type of materials used in the original design as well as common design / construction practices at the time of construction. Some other important factors include downstream impacts, extent and location of damage, size and shape of conduit or spillway adjacent to the void, type of material being eroded, size of dam, and cost of repair. This report details each of the different repair, renovation, and replacement techniques for structures associated with void formation, and provides guidance on the appropriate action to be taken. This report also details laboratory research performed on select repair methodologies used to address voids in a proof-of-concept research approach.

Introduction

Internal erosion and the subsequent formation of void space within embankment dams is one of the leading causes of dam failures and accidents in the United States (FEMA, 2005). Case histories show that internal erosion is often associated with seepage along conduits, tunnels and spillways. Modern design techniques, such as filter diaphragms, have proven to be effective in controlling seepage and preventing the migration of particles that can lead to internal erosion. Unfortunately, many dams in the U.S, along with their outlet works and spillways, were designed and built without the benefit of modern design and construction techniques. Furthermore, many these structures are aging and deteriorating, increasing the need for repair, renovation, or replacement in some serious cases.

Voids form as a consequence of the deteriorated state or poor design of a structure. There are a variety of different mechanisms that can lead to the formation of voids adjacent to dam structures (U.S. Bureau of Reclamation, 2019). Void formation and the detection of voids are detailed in preceding reports.

Determining the appropriate action to be taken after a void has been detected involves the careful consideration of many different factors. Selection of the appropriate method is always site specific. Designers must consider factors such as the type of materials used in the original design as well as common design / construction practices at the time of construction. Some other important factors include downstream impacts, extent and location of damage, size and shape of conduit or spillway adjacent to the void, type of material being eroded, size of dam, and cost of each method. The first section of this report details each of the different repair, renovation, and replacement techniques for various structures. The second section of this report details laboratory testing that was performed on select repair methodologies.

Repair, Renovation, and Replacement Methods

This section will discuss each of the different repair, renovation, and replacement techniques for voids and their associated causes. First, it is important to understand how each of these terms (repair / renovation / replacement) are defined for the purposes of this report. Each of these terms are defined as follows:

Repair: Maintenance that supports an existing structure's continued safe operation. Generally, this involves returning the structure to its original condition/design characteristics.

Renovation: The existing structure or its existing condition is improved upon to meet current safely standards, or otherwise has original design characteristics/features altered.

Replacement: The removal of an existing structure and replacement with a new structure.

Repair, renovation, and replacement methods are typically paired with seepage mitigation measures such as filters, drains, cutoff walls, or berms. The purpose of these features is to control or reduce seepage, thereby minimizing the risk of internal erosion and the formation of voids. U.S. Army

Corps of Engineers, (1993), FEMA (2011), and U.S. Bureau of Reclamation, (2014) detail the various types of seepage mitigation techniques and designs.

As with any repair, renovation, or replacement method, the installation of seepage mitigation measures should be part of the design if these features are not currently installed, functioning properly, or up to current design standards. In the following sections, "grout" is defined as a cementitious grout material composed primarily of cement, water, and fine aggregate.

Repair Methods

Voids can develop on the outside (conduits, siphons, and tunnels) or underside (spillways and canal) of structures in response to a variety of subsurface conditions including structure undermining related to seepage and piping, joint infiltration, and invert erosion / corrosion. Filling voids with grout can re-establish foundation support, but grouting is not recommended as the sole solution or as a long-term solution to the problem. Void filling can alter seepage flow regimes and may actually increase the seepage gradient, leading to the formation of new voids in adjacent areas. When seepage is the known cause of void formation, grouting must always be used in conjunction with a seepage control method i.e., barriers or filters.

Conduits, Tunnels, and Siphons

Grouting

Grouting along the exterior of a conduit, tunnel, or siphon (internal structures) consists of first injecting grout into voids and sealing any crack or joint leaks within the internal structure. Grouting can be done from inside the internal structure or from the outside of the structure by drilling grout holes from the surface. Regardless of where the void is being filled from, any defects in the existing structure that are determined to have caused the void to form must be repaired prior to the void(s) being filled. Cracks and leaking joints in concrete structures are repaired using standard concrete repair methods (U.S. Bureau of Reclamation, 2015). Grouting operations are carried out using standard grouting procedures (U.S. Army Corps of Engineers, 1995: U.S. Army Corps of Engineers, 2017).

Grouting from inside an internal structure is generally more successful than grouting from the surface. Drilling into embankment material carries the risk of hydraulic fracture which will only worsen the problem. When drilling into an embankment, it is advisable to use non-pressurized drilling methods such as auguring whenever possible. Once the grout holes are drilled, grout is injected by gravity flow to prevent embankment damage. Standard drilling procedures specific to earth embankment features must be followed to prevent unintentional damage to the embankment (U.S. Army Corps of Engineers, 2014).

Spillways and Canals

Grouting

Undermining can result in displacement, misalignment, separated joints, deterioration, and eventually structural collapse (Ohio Department of Natural Resources, 1999). Void filling with grout beneath spillways or canals (surface structures) re-establishes contact between the structure's foundation and the underlying material. Any damage caused by the presence of a void must be repaired immediately to prevent further damage or re-initiation of internal erosion through the defect. Cracking and differential settling is especially serious in spillways or canals with high volume flows. Concrete repairs such as structural cracking and leaks are performed using standard concrete repair techniques (U.S. Bureau of Reclamation, 2015). In situations where voids near structure's foundation are not treated properly, overlays will only mask and delay a potentially serious problem (U.S. Bureau of Reclamation, 2017).

Compaction Grouting

Compaction grouting is used to fill voids and re-compact soil material that has been loosened due to ground loss. Compaction grouting involves injecting a very stiff grout at high pressure for the purpose of densifying the adjacent soil. Due to the high pressures inherent to this technique, it is not recommended for injecting though an embankment. This method is used for foundation repair and void fill under surface structures (U.S. Army Corps of Engineers, 2017).

Filter berms are used to reduce the potential for internal erosion. Filter berms consist of a filter placed over the seepage exit in along with a weighted berm to resist seepage uplift pressures.

Isolated Removal and Replacement

Voids beneath a single panel in a canal or spillway can cause localized cracking or deterioration of the concrete. In many situations, it may be more technically and economically reasonable to completely replace the panel. This method involves removing the existing panel, compacting and grading the foundation, installing the new panel, and grouting the joints with a hydrophilic chemical grout, thus repairing the canal or spillway and returning it to its original condition.

Renovation Methods

Renovation techniques are typically implemented in order to return a structure to its original operating condition or better or otherwise alter the original design characteristics of the structure. This is done to prevent the re-initiation of internal erosion and the subsequent reformation of voids.

If a void has formed along a structure, that structure should always be considered for removal and replacement. If extensive grouting of large voids and further seepage mitigation will be required, renovation may not be economical. If the structure appears to be on the verge of collapse, this could be evidence of significant movement of material outside / underneath the structure in which case, replacement is typically the best option. Location of the void and the mechanism that allowed the void to be formed are also important factors to consider. For instance, if the void is located near the centerline of the dam and associated with a high amount of seepage flow, excavation and replacement must be considered. On the other hand, if a void has formed as a result of discharge through an internal structure and the void is located near an intake structure, renovation methods may be a better option. In cases when the structure has not been significantly deteriorated and voids can be filled in a cost-effective manner, renovation of the existing structure may be economically practicable.

The methods below describe several options for renovations that are indented to address various causes of void formation. Addressing existing voids is typically done via one of the "Repair" methods described above, in conjunction with the applicable renovation. If not applied correctly, in conjunction with repair of the existing voids, renovation methods can mask a potentially serious problem and cause more harm than good.

Conduits, Tunnels, and Siphons

Sliplining

Sliplining involves the installation of a new pipe into the existing internal structure and grouting the annular space (U.S. Army Corps of Engineers, 2017). Besides being less expensive, the primary advantage of sliplining over replacement is that sliplining doesn't require excavation of embankment material. The risks associated with excavation of embankment material are discussed in further detail in the "Replacement Methods" section.

The most common liner materials for sliplining applications are high-density polyethylene (HDPE) pipe liners, steel pipe liners, and polyvinyl chloride (PVC) pipe liners. Slipliners are designed under the assumption that the existing internal structure will eventually have no weight bearing capacity. The type of grout used for each job is chosen with the same assumption in mind. Other grout considerations include the ability of the grout to entirely encapsulate the slipliner within the existing structure without holes or void spaces, and the "float" capacity of the slipliner. During grouting, the slipliner tends to "float" on the highly viscous grout, making it nearly impossible to encase the entire slipliner without the use of spacers between the existing structure and the slipliner. FEMA (2005) provides information on choosing, installing, and maintaining slipliners. U.S. Army Corps of Engineers, 2017 provides more information on the type of grout used during the installation of slipliners.

Cured-In-Place Pipe (CIPP)

CIPP liners, typically comprising a resin-impregnated material, are best suited for existing internal structures that are not seriously damaged or deteriorated. CIPP liners are installed by one of two methods, the inversion method (ASTM F1216) or the pulled-in-place method (ASTM F1743). The inversion method involves inverting the flexible tube into the existing structure using hydrostatic head or air pressure. The pulled-in-place method involves pulling the liner in place and then inflating the liner using hydraulic head or air pressure. In both cases, the resin-impregnated liner is cured by circulating hot water or introducing controlled steam into the pipe.

Spillways and Canals

Lining Overlay

Lining overlays on surface structures are implemented as a means of preventing future void formation by eliminating or mitigating water infiltration. Overlays bind to the original liner, so material properties are an important consideration when choosing an overlay material. Available overlay materials include asphalt, concrete, geomembranes, or shotcrete. In addition to the binding properties of the material, other important considerations include cost, durability and constructability.

Replacement Methods

Replacement methods should always be considered when a structure is affected by a large void or sinkhole. Large voids and sinkholes are associated with structures that have been severely damaged or deteriorated. In these situations, removal and replacement is typically the most viable option.

Conduits, Tunnels, and Siphons

Excavation and Replacement

In most cases, removal and replacement of internal structures is a time consuming and costly venture compared to renovation methods. However, for some smaller dams or where operations can accommodate an interruption, removal and replacement of internal structures may be equally cost effective and efficient. In these cases, removal and replacement can be considered the better choice. Excavation allows for full removal of the structure. Any voids are fully repaired and seepage control measures are easily installed as part of the re-construction. Design modifications are made to meet current design standards (U.S. Bureau of Reclamation, 2012).

Generally, the process of removing and replacing internal structures consists of excavating the dam down to the existing structure, salvaging the excavated material, removing the existing structure, constructing a new structure, installing downstream filters, and replacing the embankment material. This process requires that either the reservoir is drained, or a cofferdam is constructed. This method will impact normal reservoir operations and may increase safety risks on the downstream community.

Excavations through embankment material increase the risk of hydraulic fracturing when the reservoir is subsequently refilled. Therefore, excavations must be wide enough to accommodate compaction equipment parallel to the new structure being installed. Filters or collars are installed or returned to new condition before the new conduit is installed.

Spillways and Canals

Voids beneath a single panel in a canal or spillway can be addressed as described above. When voiding becomes widespread or of sufficient magnitude that it cannot be reasonably addressed with isolated measures, removal and replacement of the entire canal or spillway can become a viable solution. In these cases, new structures should be designed and built to current standards.

Foundation grouting of voids under structures can never guarantee the entire space is filled. Removal and replacement ensures the voids are filled completely and allows for seepage mitigation methods to be installed which will prevent future internal erosion.

Emergency Repairs

An Emergency Action Plan (EAP) is required for all water impoundment structures in the United States. U.S. Bureau of Reclamation, (2020) provides detailed instruction on how to design an EAP and details items to include. The purpose of this section is to provide guidance on emergency actions related to filling voids and preventing the progression of seepage and internal erosion.

The first response to the observance of any of the following emergency situations is to initiate the appropriate emergency actions. In general, when a situation necessitates emergency action, the final step of an EAP is to contact a qualified engineer to carry out an investigation and begin designing an excavation and replacement plan.

Conduits, Tunnels, and Siphons

Sinkholes and Excessive Settlement

The appearance of sinkholes or subsidence of the embankment surface near or approximately above an internal structure can usually be attributed to internal erosion of embankment material. Both situations indicate an urgent problem that requires immediate emergency action. When a sinkhole appears, the first step is to begin lowering the reservoir level. If the sinkhole is actively forming and needs to be stabilized, the placement of a well graded sand and gravel mix can be attempted. This sand-gravel mix is meant to act as a makeshift filter which will eventually arrest the erosion or at least slow its progression.

Internal Erosion Through Embankment, Foundation, or Abutment

If an open pathway in the embankment is carrying embankment material and large amounts of water are accumulating on the downstream slope, a break in the conduit could be allowing water to discharge from the conduit. In the case of a pressurized conduit, a flow path has developed on the exterior of the conduit. In this situation, the reservoir level should be lowered and filter sand with gravel should be placed at the exit point of the seepage to control the progression of internal erosion.

Spillways and Canals

Seepage-related incidents near canals or spillways require immediate action to avoid failure. Due to the high flow rates associated with these structures, progression of an internal erosion failure from a new seepage to a breach in the canal or embankment dam can take less than an hour (U.S. Bureau of Reclamation, 2017). When new seepage is noticed, the first step is to lower the water level (canal) or shut off the flow (spillway). Material is placed over the seepage exit and entrance if possible, to control the progression of internal erosion.

Decision Making

The following "yes" or "no" questions will help guide a decision on which repair technique best suits any given situation. The "General Questions" should be considered along with "Specific" questions. Upon answering the questions, contact your Operations, Maintenance, and Repair representative or authors of this report with the answers to further discuss the specifics of the issue(s).

General Repair / Renovations / Replacement Questions

1. Was the structure built before 1985?
2. Is a void detectable?
3. Did the dam design include any seepage mitigation measures?
4. Is there a large void or sinkhole associated with the structure?
5. Did a void form on the downstream side or near the crest of the embankment?
6. Is the structure in a severely deteriorated state?
7. Is material being carried in the leakage flow?
8. Is leakage flow increasing?

Questions Specific to Conduits, Tunnels, and Siphons

1. Is the structure accessible for man-entry?
2. Are there a large number of voids or excessive void space to fill?
3. Is this a small dam?
4. Can the reservoir be drained, or a cofferdam constructed?
5. Can excavations accommodate the machinery required to meet design standards?

Questions Specific to Canals and Spillways

1. Has undermining occurred?
2. Is the damaged localized?
3. Can the flow be cut off or diverted?

Case Histories

Applegate Dam

Issue Date: 1981

Source: Campbell, R., Bean, D., (1988)

Significant leakage observed through three monolith joints in the outlet conduit. Flow was measured at 3 – 5 gpm. The water was assumed to be flowing from the foundation material and into the joint through a break in the waterstop. Joints were grouted with hydrophilic chemical grout. No leakage was reported at the next inspection.

Solution: Repair

Arkabutla Dam

Issue Date: 2003

Source: FEMA (2005)

Problems with the dam started soon after its construction in 1943. Fine sands from the foundation were being washed through the joints during operation of the outlet works. For several years, steel wool was used to control erosion, but there were issues with keeping the wool in place. Eventually the conduit began to settle due to the loss of sand in the foundation. Grouting operations were undertaken to the fill multiple joints. Grouting operations continued through the years. In 1970, it was discovered that more sand was being eroded through the joints. Grouting operations continued to little avail.

In 2003, while a maintenance crew was in the conduit to replace a joint filler, a joint broke loose and began flowing a significant amount of sand and water. The conduit was immediately deemed unsafe. It was determined that the best course of action would be to line the pipe with a steel liner. However, due to cost concerns, it was proposed that an attempt be made to bolt a steel plate over the defective joint with gasketing underneath the plate to prevent sand from flowing through the joint. The plate was installed but during a later inspection, it was discovered that the plate had been displaced from over the joint. It was then decided that installing a steel pipe liner would be the only reliable solution to the problem.

Solution: Repair and subsequent Renovation

Balderhead Dam

Issue Date: 1967

Source: ICOLD (2016)

Soon after the reservoir was initially filled, sink holes developed in the crest of the embankment dam and material laden water issued from the drains downstream from the crest. The reservoir was drained, and the leakage stopped after 9 meters of drawdown. Investigations revealed sand filled cracks in the core that were initialed by hydraulic fracture.

Leakages and cracks were filled with grout and a diaphragm wall was constructed on the downstream side of the embankment. A large section of the core was also grouted to seal any potential leaks.

Solution: Repair

Clair Peak Dam

Issue Date: 2003

Source: FEMA (2005)

Investigation into a roadway collapse atop an embankment dam revealed voids adjacent to the dam's spillway pipes. An investigation revealed that the collapse and associated sinkholes were caused by internal erosion. A decision was made to construct a new spillway in different location. Excavation and removal of the old spillway structure was deemed impractical. Instead, the voids and old spillway piping were filled with cement and fly ash-based compaction grout.

Solution: Replacement

Como Dam

Issue Date: 1992

Source: FEMA (2005)

Increased seepage through the embankment dam caused dam safety concerns. Potential seepage through some areas of the conduit and the degraded condition of the redwood lined pipe led to the decision to line the entire conduit with a steel pipe slipliner. Rationale for this decision include increased overall structural integrity of the conduit, improved flow surface, prevention of seepage through the concrete conduit, and less time and money spent compared to removal and replacement.

Solution: Renovation

Dalewood Shores Dam

Issue Date: 1995

Source: FEMA (2005)

Sloughing of the upstream embankment slope caused damage to the upstream end of the embankment dam conduit. An inspection of the pipe revealed seepage into the conduit at two separate locations. The seepage was clear at the time (1993) and no further action was taken. In 1995, flow through the ruptured pipe increased and soil deposits inside the pipe indicated internal erosion of the embankment material. The decision was made to install HDPE slipliner inside the conduit.

Solution: Renovation

Lake Darling Dam

Issue Date: 1988

Source: FEMA (2005)

Instrumentation was being installed around the outlet works conduit within the embankment. During the installation, a large amount of grout was lost into an internal erosion feature while backfilling a borehole. The grout loss occurred at the contact between the embankment materials and the foundation materials which was sealed with bentonite. The conduits were inspected, and small voids were found beneath the conduit at several locations. The voids were all located near cracks in the conduit and were associated with seepage and internal erosion. Although the outlet works were scheduled to be replaced within five years, immediate action was deemed necessary. **The limited life expectancy of the outlet works was considered when designing these corrective actions**. Six relief wells were installed, cracks in the conduit were filled with an elastic filler, grouting was performed at the base and exterior of the outlet works, and a new floor with a filtered underdrain system was constructed.

Solution: Emergency Repair and subsequent Replacement

Matahina Dam

Issue Date: 1987

Source: ICOLD (2016)

An initial incident of internal erosion occurred in 1967. Differential settling led to erosion and leakage which stopped after 24 hours. It was concluded that the erosion was arrested in the rockfill and core material soon after it began. In 1987, an earthquake caused a crack to appear in the core of the right abutment. The earthquake appeared to have re-opened the same crack caused by differential erosion 20 years earlier. During remediation, a large void was found in the core and the rockfill downstream of the core was impregnated with eroded core and transition materials. Following this incident, the core was excavated and removed and replaced with a smooth contact core.

Solution: Replacement

Pablo Dam

Modifications Date: 1993

Source: FEMA (2005)

Differential settling caused offset in some of the conduit joints. This required grouting of the foundation shortly after construction. Water was observed leaking through the cracks along the length of the conduit and through some of its joints. Internal erosion of embankment material was considered very likely. Spalling concrete was found in the walls of the conduit and the downstream end of the conduit was heavily deteriorated. This resulted in aggregate and rebar being exposed within the conduit.

Cracks were filled with chemical grout to stop the leakage. As the cracks were filled upstream, leakage began to appear in previously dry cracks downstream of the repaired cracks. Two voids were discovered while filling the cracks. Construction drawings show that these voids formed where the concrete intake tower support walls meet the conduit. Voids were injected with chemical grout, and weep drains, and filter collars were installed on the downstream end of the conduit.

In 2001, it was discovered that material had been deposited in the middle of the conduit near a joint. Reclamation thought that plugging the crack could cause internal erosion through a different crack in the conduit and/or cause a more dangerous path to develop. The decision was made to completely remove and replace the original outlet works.

Solution: Replacement

Pine Creek Dam

Issue Date: 1970

Source: Campbell, R., Bean, D., (1988)

Water leakage through two joints was observed from within the conduit. An investigation revealed that the leak was caused by differential settling along the conduit. Grouting was performed in 1970 and 1976 from inside the conduit to repair the joints and any voids that had formed on the exterior of the conduit.

Solution: Repair

Pinery Dam (Bingham Lake Dam)

Source: Cesare, J.A., Brauer, D.J., (2004)

Continuous leakage flows measuring over 60 gpm and carrying embankment material were discovered exiting the outlet works conduit. The source of the leak was a 3-inch vertical offset at a pipe joint ~260 ft from the downstream end of the outlet conduit. Since draining the reservoir had to be a last resort measure, repairs were designed from the downstream end or underwater on the upstream side. The first repair attempt was done with a flexible tube lining that was set in place with epoxy resin from the downstream end. However, the flex tube was forced out of the conduit by the leakage flows resulting in a repair failure. The final repair was made using a HDPE slip liner with the void space between the liner and the original conduit being filled with grout.

Solution: Renovation

Porjus Dam

Issue Date: 1993

Source: ICOLD (2016)

A sinkhole was found in the upstream filter when the reservoir was at maximum capacity. Investigations concluded that hydraulic fracture had caused erosion through the core. Conditions for the progression of erosion were assessed as neutral. Grouting was performed around the sinkhole to seal the zone where leakage was concentrated. In 2004, a rockfill berm along the whole length of the downstream slope was constructed to resist leakage.

Solution: Repair, subsequent Renovation

Ridgway Dam

Issue Date: 1986

Source: FEMA (2005)

Settlement caused longitudinal and transverse cracking on the upstream and downstream sections of the conduit. All cracks were filled with grout and instrumentation was installed in the upstream section of the conduit.

Solution: Repair

Rolling Green Community Lake Dam

Issue Date: 1999

Source: FEMA (2005)

The dam's spillway riser was significantly deteriorated. Repairs were done to the upper portion of the riser, but no repairs done to the lower portion of the riser. Because no repairs were done to the lower portion of the riser, the base of the riser collapsed, and a large portion of the embankment was washed away leaving a 30 by 10 ft void around the original riser location. A CCTV inspection was performed and concluded that the remaining pipe was in good condition. The existing spillway barrel was sliplined and a new riser was constructed. A new filter diaphragm was constructed around the downstream end of the pipe to control seepage along the outside of the conduit.

Solution: Renovation

Round Rock Dam

Issue Date: 1991

Source: FEMA (2005)

Reclamation identified corroded portions of the CMP and noted separations in several of its joints. The upstream and downstream portion of the CMP was sliplined and designed to withstand external loads.

Solution: Renovation

Sardis Dam

Issue Date: 1974

Source: FEMA (2005)

A sinkhole appeared above the monolith at the junction of the intake tower and the upstream end of the transition monolith. An investigation revealed that the intake tower was founded on piles, but the transition monolith was not. This caused enough differential settling to rupture the copper waterstop between the two monoliths and allowed water and material to flow though the joint.

Grout holes were drilled around the entire perimeter of the joint and into the surrounding soil. Grout was pumped through the holes to fill any voids along the transition and to seal the water stop.

Solution: Repair

St. Louis Recreation Lake Dam (actual name withheld)

Source: FEMA (2005)

During construction of the dam, a PVC diversion pipe was installed in the base of the embankment to prevent impoundment of water. The pipe was meant to be temporary and was to be filled once construction was complete. However, after completion of the dam the owner proposed a new design that included keeping the pipe in place and installing a valve on the downstream end of the pipe so it could be used control the lake level in the future. The pipe was inspected with CCTV, which revealed a separated joint and a collapse in the pipe. The pipe was immediately abandoned and filled with grout.

Solution: Repair

St. Pardoux Dam

Issue Date: 1991

Source: ICOLD (2016)

A routine dam inspection revealed some increased pore pressures and leakage within the downstream slope of the embankment. An investigation revealed that suffusion was occurring within the fill and that there was some drainage flowing through the sandy layers in the fill. A diaphragm wall was installed at the crest of the embankment, foundation grout curtains were deepened, and drainages were added to each of the abutments.

Solution: Renovation

Turtle Lake Dam

Issue Date: 1997

Source: FEMA (2005)

A sinkhole was discovered above the lower portion of the downstream conduit. An investigation revealed that portions of the outlet works had partially collapsed. The conduit downstream of the embankment toe was excavated, but the engineering team did not want to do any excavation within the embankment itself. CCTV inspection showed potential for offsets in the alignment of the conduit. HDPE sliplining was chosen as the preferred remediation method.

Solution: Renovation

Waterbury Dam

Issue Date: 1999

Source: FEMA (2005)

Seepage of water and material through the rockfill zone of the embankment suggested internal erosion was occurring within the embankment. Voids were located and filled using a filter injection process. Seepage conditions did not stabilize calling for further action to be taken. A filter drain was placed along the interface of the conduit and embankment material surrounding the downstream end of the conduit. 160 feet of the conduit was excavated and inspected. At the upstream end, a filter diaphragm was installed at the crown of the conduit and a drain was wrapped around the remainder of the conduit.

Solution: Repair

Willow Creek Dam

Issue Date: 1996

Source: FEMA (2005)

A large sinkhole was found at the crest of the embankment dam directly above the outlet works tunnel. The hole was temporarily stabilized by filling it with sand and gravel. Excavation revealed that the hole extended through 40 feet of bedrock to a large cavity surrounding the concrete tunnel lining. The voids around the tunnel were filled with tremie grout and the excavated bedrock was filled with concrete. CIPP lining was installed within the downstream section of the tunnel.

Solution: Emergency Repair and Renovation

Wister Dam

Issue Date: 1949

Source: FEMA (2005)

Heavy rain caused the reservoir to reach capacity. Shortly after, muddy water was observed on the downstream slope of the embankment dam. Flow was estimated to be up to 9,000 gal/min at the highest point over the course of three days. The spillway gates were opened, and the reservoir dropped 13ft exposing large tunnels that had formed on the upstream slope. Once the reservoir had dropped further, the erosion tunnels were excavated and filled with grout. In addition to filling the tunnels, a steel sheet pile wall was installed along with upstream and downstream berms and drains.

Solution: Renovation.

Laboratory Testing

This project also reviewed select repair technologies and their potential effectiveness in filling voids. While grouting with traditional materials and processes is generally well understood, this study reviewed the use of permeable low-density cellular concrete and foam transported sand as two additional potential repair options. These two methods were identified for investigation primarily as they would theoretically provide a permeable medium that may be advantageous in select scenarios such as filling abandoned drains where the is a desire for the medium to allow some water flow and be filter compatible.

Permeable Low-Density Cellular Concrete

The American Concrete Institute (ACI) 523.1 (ACI 523.1, 2006) defines low-density cellular concrete is a mixture of cement, water, and preformed foam. Permeable low-density cellular concrete, or PLDCC, is a low-density cellular concrete where the air voids created by the foam are at least partially interconnected, giving the cellular concrete high permeability characteristics. The low-density characteristics also provide for lighter in-place material weights, reducing loads when utilized above buried elements, behind walls, or similar. While exact mix proportions vary, foam is typically mixed, by volume, at 70% to 80%, yielding batch densities in the range of 25 to 35 pcf.

As it relates to the use of PLDCC in Reclamation facilities, the primary research goal was to utilize PLDCC in a proof-of-concept type arrangement such as the application of filling abandoned drains. Based on this scenario, it was desired to evaluate the material at different head pressures, i.e. pumping the material up into a sloped drain, and also how the material interacts with other material within the void, such as water or sand/rock.

PLDCC Testing Program

To evaluate the PLDCC and its potential effectiveness in applications such as filling abandoned drains, testing was designed to evaluate material properties at varying levels of pressure head, and in cavities with debris and water within to evaluate the PLDCC's interaction with these elements within the pipes. To achieve this, the following test setups were created; all material was tested in 3-in diameter clear polyvinyl-chloride (PVC) pipe sections.

- 32-foot vertical (no material within pipe)
- 24-foot vertical (no material within pipe)
- 8-foot vertical (no material within pipe)
- 8-foot horizonal (no material within pipe)
- 8-foot horizontal with water within
- 8-foot horizontal with debris within (concrete sand and ¾-in aggregate)

After placement, samples were extracted and tested for density (ASTM C495, 2019), compressive strength (ASTM C495, 2019), and permeability; sample locations were within the tested sections. Batch densities were obtained on each batch mixed, prior to any pumping operations. Control samples were cast in concrete cylinder molds, obtained at the end of the pump hose, immediately prior to the start of pumping into each setup.

All PLDCC was mixed and cast on the same day. The grout utilized consisted of 100-percent cement mixed with water at a 0.50 water/cementitious (w/c) ratio. Foam utilized to create the PLDCC was AQUAERiX manufactured by Aerix Industries. The foam and grout were mixed at 3:1 (75% foam) with a foam density of 2.5 pcf to achieve a target PLDCC plastic density of approximately 30 pcf. The 24-foot vertical set-up was filled last with all remaining PLDCC material, resulting in filling only to approximately 19ft. Photograph 1 through Photograph 6 show general set-up and pumping operations.

PLDCC compressive strength testing was performed in general conformance with ASTM C495 with the exception that all specimens were tested at the same load rate, regardless of time of failure; ASTM C495 does not specify a loading rate, only a time window during which failure should occur. All samples were tested at the same load rate in order to test all specimens under the same conditions. Similarly, due to the variability of each specimen, all specimens would have required duplicate specimens to determine the approximate failure time prior to testing of the "official" specimen, for which there was insufficient material. The load rate was approximately 2.0 kips/min, which yielded failure within the specified time for samples at average density.

Permeability testing was performed using a falling head apparatus under atmospheric conditions with a constant exit head. This method was utilized due to its simplicity and the ability to test in-situ specimens within the same PVC pipes in which they were originally pumped. All specimens were vacuum saturated for a minimum of 48 hours prior to testing. Testing consisted of filling a graduated standpipe and measuring the change in head over time; each specimen was tested for approximately 1 hour. The test apparatus is illustrated in Photograph 7. Consideration was given to testing ASTM D2434 (ASTM D2434, 2022) and ASTM D5084 (ASTM D5084, 2024); however, these methods require testing apparatuses that were not conducive to pumping the material into the PVC set-ups and testing in-situ samples.

Photograph 1.—Addition of foam to cementitious grout in mixing drum.

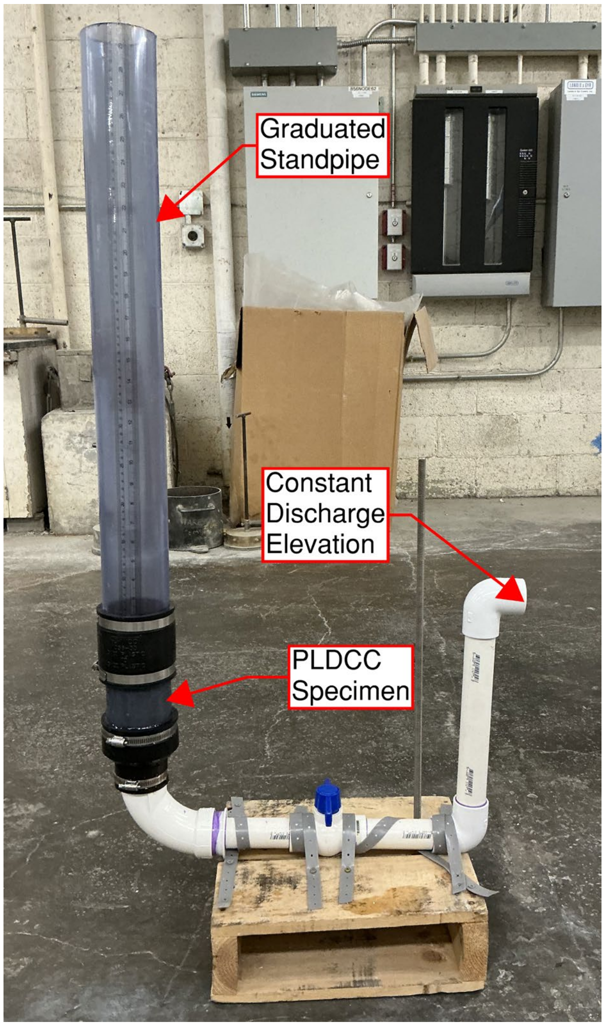
Photograph 2.—Fully mixed PLDCC in drum mixer.

Photograph 3.—PLDCC transferred from drum mixer to grout pump.

Photograph 4.—PLDCC test set-ups with vertical pipes attached to permanent ladder.

Photograph 5.—PLDCC pumping into vertical 32-foot set-up.

Photograph 6.—PLDCC pumping into horizontal 8-foot set-ups.



Photograph 7.—Falling head permeability testing apparatus.

PLDCC Findings

Test results for the PLDCC are presented in Table 1 through Table 5 and Figure 1 and Figure 2.

Table 1.—Neat Grout and PLDCC Batch Plastic Densities.

Batch	Density (pcf)	Average Density (pcf)	Density Standard Deviation	Density Coefficient of Variation (%)
Neat Grout	112.5	NA	NA	NA
PLDCC-1	30.6	30.5	0.10	0.32
PLDCC-2	30.4			
PLDCC-3	30.4			
PLDCC-4	30.5			

Table 2.—PLDCC Compressive Strength Specimen Results.

Sample ID	Specimen ID	Elevation (ft)	Dry Density (pcf)	28-Day Compressive Strength[1] (psi)
32 FT	Control	NA	27.3	110
	32-0.5	0.5	111.4	4400
	32-5	5.0	67.1	1610
	32-10	10.0	57.1	1030
	32-15	15.0	37.7	270
	32-20	20.0	34.8	200
	32-25	25.0	34.7	510
	32-30	30.0	25.1	140
24 FT	Control	NA	39.6	220
	24-0.5	0.5	74.0	1060
	24-5	5.0	51.4	630
	24-10	10.0	44.2	570
	24-15	15.0	37.0	470
	24-18.5[2]	18.5	28.6	290
8 FT Vertical	Control	NA	38.5	260
	8V-0.5	0.5	47.8	660
	8V-6.5	6.5	28.9	280
8 FT Horizontal	Control	NA	49.1	120
	8H-0.5	0.5	45.1[4]	NA
	8H-6.5	6.5	49.8[4]	NA
8 FT H20[3]	Control	NA	38.4	280
8 FT Debris[3]	Control	NA	38.1	260

[1]Compressive strength testing was performed in general conformance with ASTM C495 with the exception that all samples were tested at the same load rate, regardless of time of failure. The load rate was approximately 2.0 kips/min, which yielded failure within the specified time for samples at average density.
[2]The maximum sample height from sample 24 FT was 18.5-feet due to limited material during placement.
[3]Test Specimens, in addition to Control specimens, could not be obtained from these set-ups due to voids present after curing, reference Visual Observations for additional information.
[4]Density results are an approximation due to variable geometry of sample.

Table 3.—PLDCC Control Specimen Data Summary.

Average Control Dry Density (pcf)	38.5
Control Dry Density Standard Deviation	6.34
Control Dry Density Coefficient of Variation (%)	16.5
Increase from Average Batch Dry Density (%)	26.2
Average Control Compressive Strength (psi)	210
Control Compressive Strength Standard Deviation	68.2
Control Compressive Strength Coefficient of Variation (%)	32.7

Table 4.—PLDCC Permeability Specimen Results.

Sample ID	Specimen ID	Elevation (ft)	Density[1] (pcf)	Flow Rate[2] (in^3/hr.)	Hydraulic conductivity[2], k (in/hr.)
32 FT	32-0.5	0.5	98.7	0.380	0.034
	32-5	5.0	70.5	0.340	0.030
	32-10	10.0	58.9	0.415	0.038
	32-15	15.0	32.5	6.270	0.582
	32-20	20.0	46.3	0.829	0.075
	32-25	25.0	38.2	3.583	0.329
	32-30	30.0	27.7	10.640	1.029
24 FT	24-0.5	0.5	69.2	0.380	0.034
	24-5	5.0	51.6	0.671	0.059
	24-10	10.0	47.8	0.362	0.033
	24-15	15.0	37.7	2.375	0.217
	24-18.5	18.5	29.1	18.240	1.859
8 FT Vertical	8V-0.5	0.5	48.5	0.407	0.036
	8V-6.5	6.5	29.1	41.040	4.553

[1]Density of permeability samples obtained by subtracting PVC weight and volume. Permeability specimen densities should be considered approximate.
[2]Permeability testing was performed with a falling head apparatus. Permeability coefficient, k, is defined as $k=\frac{2.303*a*L}{A*t}*log10(\frac{h1}{h2})$.

Table 5.—PLDCC Average Dry Density at Elevation Specimen Results.

Sample ID	Specimen ID	Elevation (ft)	Average Dry Density (pcf)
32 FT	32-0.5	0.5	105.1
	32-5	5.0	68.8
	32-10	10.0	58.0
	32-15	15.0	35.1
	32-20	20.0	40.5
	32-25	25.0	36.5
	32-30	30.0	26.4
24 FT	24-0.5	0.5	71.6
	24-5	5.0	51.5
	24-10	10.0	46.0
	24-15	15.0	37.4
	24-18.5	18.5	28.8
8 FT Vertical	8V-0.5	0.5	48.2
	8V-6.5	6.5	29.0

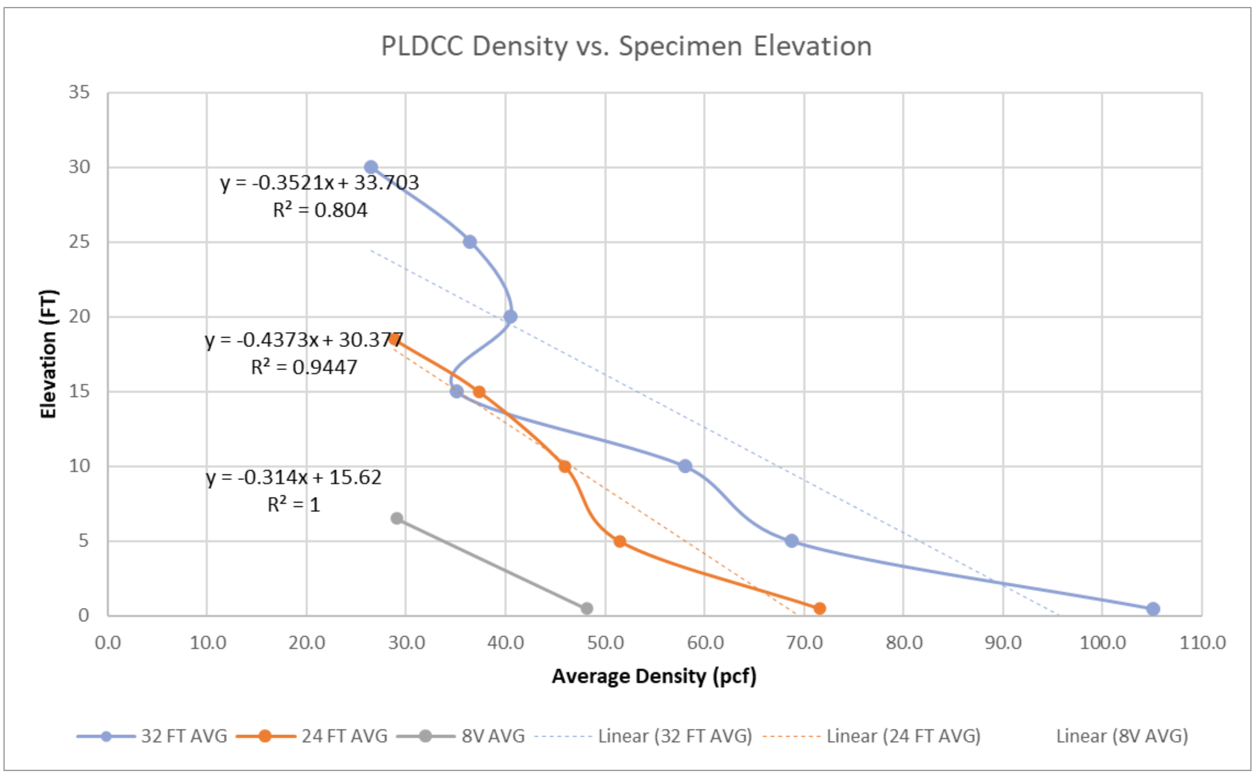


Figure 1.—Average PLDCC specimen density vs. elevation.

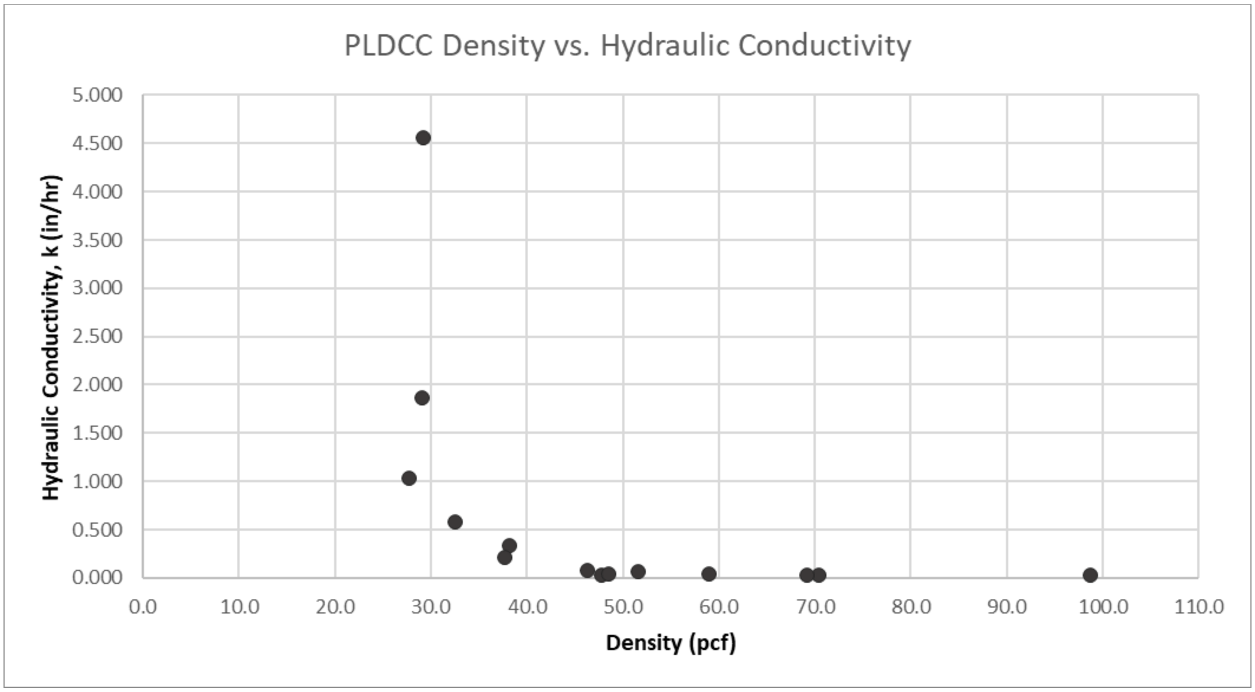


Figure 2.—PLDCC Density vs. Hydraulic Conductivity.

PLDCC Notes and Visual Observations

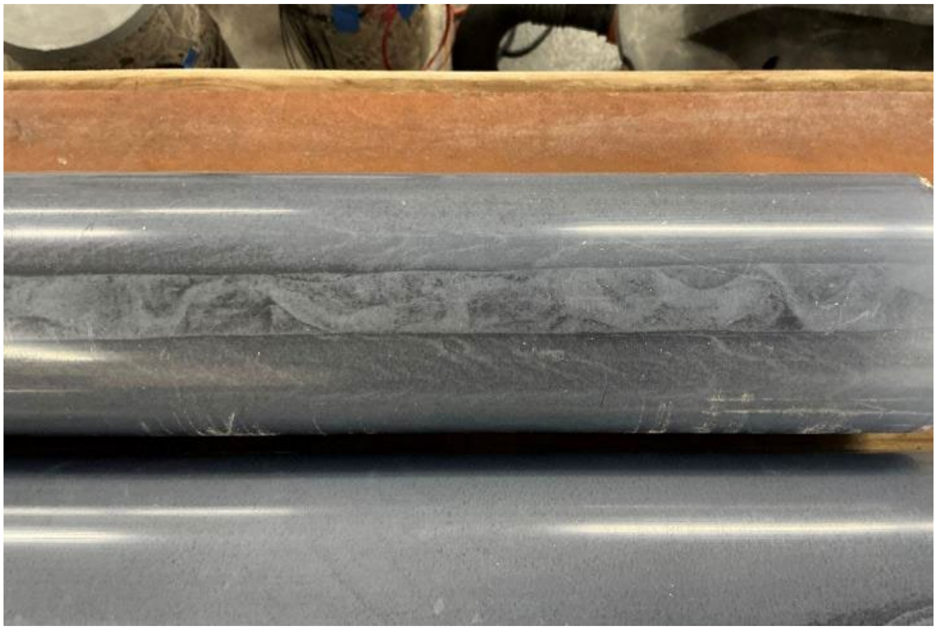
Upon full curing, the specimens were extracted from each set-up and visually reviewed.

32-foot Vertical

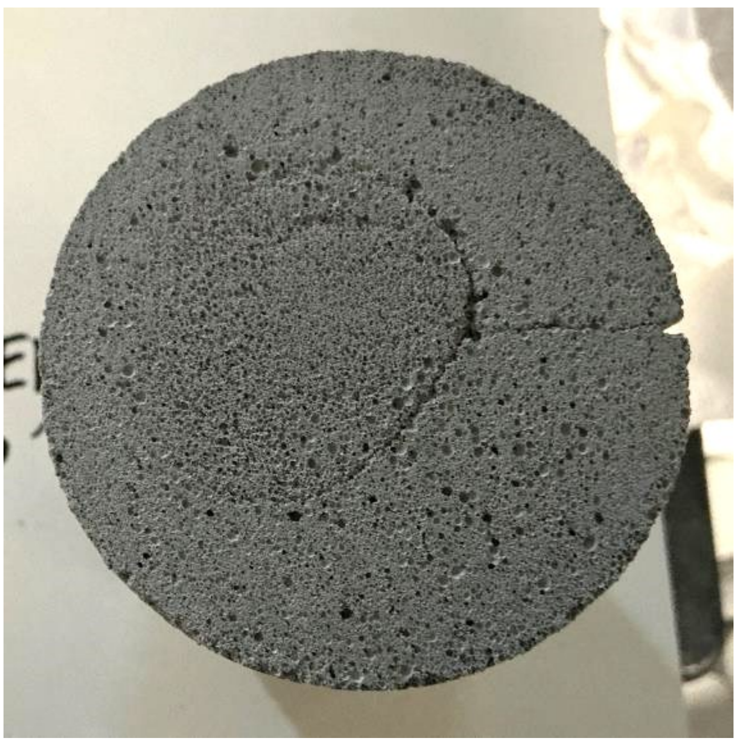
- This sample was placed in two batches; initial pumping operations stopped at an elevation of approximately 15 ft while an additional PLDCC batch was made. Placement was made to approximately 31 ft.
- Change in density and porosity was apparent from the lowest sample, 32-0.5 (Photograph 7) to the highest sample, 32-30 (Photograph 8).
- Varying densities from 15 ft to 25 ft; specimens 32-15, 32-20, and 32-25, this corresponds to stopping and restarting of pumping operations at approximately 15 ft.
- Color variation within pipe (Photograph 9) from 15ft to 20 ft indicates non-uniform flow during pumping; this corresponds to stopping and restarting of pumping operations at approximately 15 ft.
- Compressive strength samples 32-15 and 32-20 appeared to have non-homogenous sections approximating lift lines when viewed prior to testing (Photograph 10). After testing, the failures occurred on a plane in alignment with the non-homogenous sections (Photograph 11 and Photograph 12); this corresponds to stopping and restarting of pumping operations at approximately 15 ft.
- Upon final curing, the uppermost extents of the PLDCC did not appear to condense or collapse (Photograph 13).

Photograph 8.—Section of 32-0.5 specimen with little to no voids visible indicating high density, low porosity.

Photograph 9.—Section of 32-30 specimen with voids visible indicating low density, high porosity.



Photograph 10.—Portion of sample 32 with color variation in the vicinity of specimens 32-15 and 32-20.



Photograph 11.—Strength specimen 32-20 indicating non-homogenous section and “lift lines”.

Photograph 12.—Strength specimen 32-20 after testing with failure occurring at “lift lines”.

Photograph 13.—Strength specimen 32-20 after testing with failure occurring at “lift lines”.

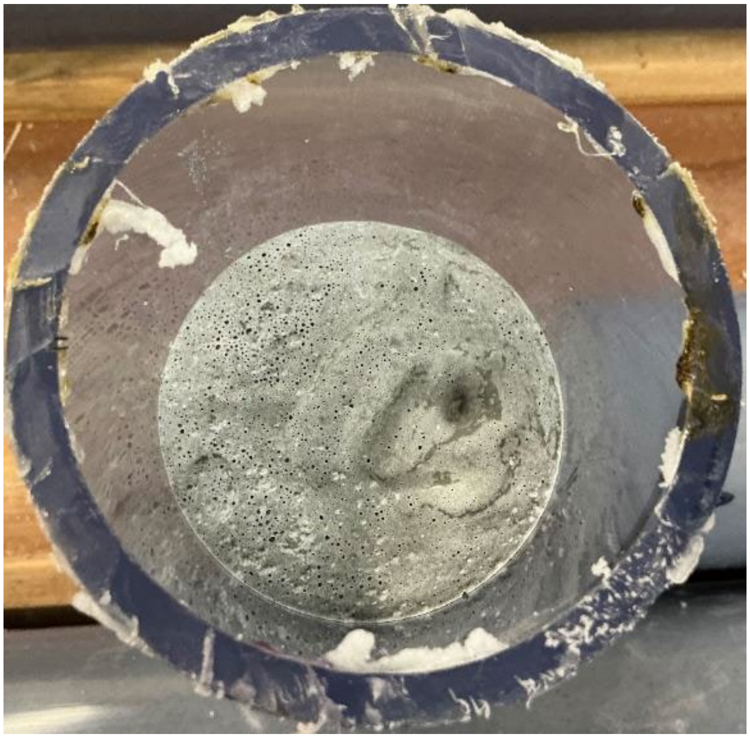
Photograph 14.—Non-collapsed upper extents of 32-foot set-up, approximate elevation 31ft.

24-foot Vertical

- This sample was placed in one batch with all remaining material; placement was made to approximately 20 ft.
- Change in density was apparent from the lowest sample, 24-0.5 (Photograph 14) to the highest sample, 24-18.5 (Photograph 15).
- Upon final curing, the uppermost extents of the PLDCC did not appear to condense or collapse (Photograph 16).

Photograph 15.—Section of 24-0.5 specimen with little to no voids visible indicating high density, low porosity.

Photograph 16.—Section of 24-18.5 specimen with voids visible indicating low density, high porosity.



Photograph 17.—Non-collapsed upper extents of 24-foot set-up, approximate elevation 20ft.

8-foot Vertical

- This sample was placed in one batch with all remaining material; placement was made to approximately 7.5 ft.
- Change in density was apparent from the lowest sample, 8-0.5 (Photograph 17) to the highest sample, 8-6.5 (Photograph 18).
- Upon final curing, the uppermost extents of the PLDCC appeared to condense or collapse approximately 2 in (Photograph 19).

Photograph 18.—Section of 8-0.5 specimen with fewer voids indicating higher density, lower porosity.

Photograph 19.—Section of 8-6.5 specimen with greater voids indicating lower density, higher porosity.

Photograph 20.—Collapsed upper extents of 8-foot set-up, approximate elevation 7ft.

8-foot Horizonal-Empty

- This sample was placed in one batch with all remaining material; placement was made until PLDCC was discharging from the vent hole at the end of the pipe (Photograph 20).
- Upon curing, it was noted that the density varied from the top of the specimens to the bottom, i.e. vertically through the 3 in pipe cross section.
    - An air void was present at the top and the specimens were notably more dense and generally free of foam air voids at the bottom (Photograph 21).

Photograph 21.—PLDCC discharging from vent hole of 8-foot horizontal-empty sample.

Photograph 22.—PLDCC specimen from 8-foot horizontal-empty, with air void and lower density at top, and higher density at bottom.

8-foot Horizontal-Water

- This sample was placed in one batch with all remaining material; placement was made until PLDCC was discharging from the vent hole at the end of the pipe. During placement, clean water was discharged from the vent hole (Photograph 22).
  - The pipe was also propped up at the vent end resulting in different results for specimens at 0.5ft (pump end) and 6.5ft (vent end).
- Upon curing, it was noted that the density varied from the top of the specimens to the bottom, i.e. vertically through the 3 in pipe cross section.
  - Near the vent end, water was present in the sample and came out during specimen cutting. After draining of the water, a void was present on the bottom of the 8-6.5 specimens (Photograph 23). The specimens displayed a slightly higher density immediately above the void filled with water with decreasing density toward the top.
  - Near the pump end, an air void was present at the top and the specimens were notably more dense and generally free of foam air voids at the bottom (Photograph 24).

Photograph 23.—Clean water discharging from vent hole during placement of PLDCC.

Photograph 24.—PLDCC specimen from 8-foot horizontal-water, with water-filled void at bottom pipe at vent end.

Photograph 25.—PLDCC specimen from 8-foot horizontal-water, with air void and lower density at top, and higher density at bottom.

8-foot Horizontal-Debris

- This sample was placed in one batch with all remaining material; placement was made until PLDCC was discharging from the vent hole at the end of the pipe.
- Upon curing, it was noted that the encapsulated limited amounts of rock and sand debris (Photograph 25). Areas of rock and sand remained unbound (Photograph 26); unbound debris appeared most common at locations where greater quantities of debris were initially present.

Photograph 26.—PLDCC specimen from 8-foot horizontal-debris, with air void and lower density at top, and partially encapsulated coarse and fine aggregate debris at bottom.

Photograph 27.—PLDCC specimen from 8-foot horizontal-debris, with unbound coarse and fine aggregate debris material.

Foam Transported Sand

Foam transported sand, or FTS, is a process by which foam is used as a transport medium for sand. The foam and sand are mixed to create a slurry-like mixture enabling the material to be pumped or "injected" into areas such as sub-surface voids where sand cannot be deposited in a traditional manner, such as dumping. While exact mix proportions vary, foam is typically mixed, by volume, at 20% to 40%, yielding batch densities in the range of 65 pcf to 80 pcf.

Similar to the use of PLDCC in Reclamation facilities, the primary research goal for FTS was to utilize FTS in a proof-of-concept type arrangement such as the application of filling abandoned drains.

FTS Testing Program

To evaluate the foam transported sand and its potential effectiveness in applications such as filling abandoned drains, testing was set-up similar to the PLDCC to evaluate material properties at varying levels of pressure head, and in cavities with debris and water within. Since confined samples cannot be reasonable extracted from the sand specimens, the primary objective of this testing was to evaluate the pumpability of the foamed sand, the magnitude of sample settlement within the PVC pipes after a length of time, and the material's interaction with water and debris. A similar testing protocol was developed per below; all material was pumped into 3-inch diameter clear polyvinyl-chloride (PVC) pipe. After placement, samples were visually reviewed.

- 32-foot vertical (no material within pipe)
- 8-foot vertical (no material within pipe)
- 8-foot horizonal (no material within pipe)
- 8-foot horizontal with water within
- 8-foot horizontal with debris within (concrete sand and ¾-inch aggregate)

All FTS was mixed and placed on the same day. The sand utilized for the testing was a natural sand meeting the requirements of fine aggregate per ASTM C33-24, prior to mixing the sand was prepared to 3% free-moisture. The foam utilized to create the FTS was ARX-Transport manufactured by Aerix Industries. The foam and sand were mixed at 1:3 with a foam density of 2.5 pcf to achieve a target FTS density of approximately 75 pcf.

FTS Findings and Visual Observations

Test results for the FTS are presented in Table 6 and Table 7.

Table 6.—FTS Batch Plastic Density.

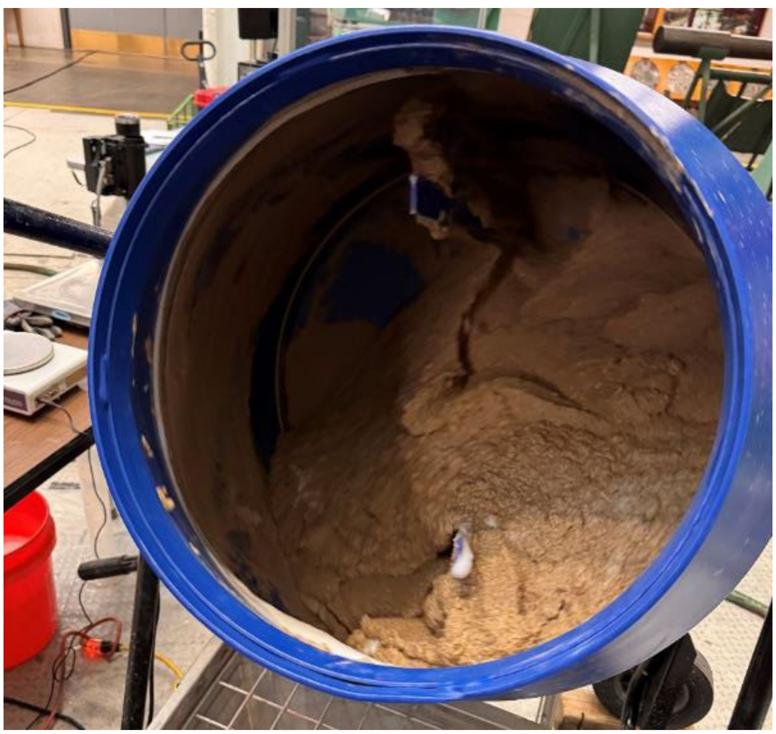
Batch	Density (pcf)
1	71.2

Upon batching of the FTS, multiple attempts to pump the material were made but were unsuccessful. Visual review of the material indicated that the material was likely too cohesive to properly fill and transfer through the pump being utilized. As a result, only the 8-foot vertical set-up was placed; the material was placed within the pipe by hand from the top of the pipe. The final placement height was marked and tracked over time based on the assumption that the sand will self-consolidate as the foam dissipates; the findings from this analysis are presented in Table 7.

Table 7.—FTS Elevation Over Time.

Reading	Duration (hrs.)	Elevation (in)	Percent Drop of Initial Elevation (%)
0	0.0	84.00	NA
1	24	83.25	0.89
2	48	82.625	1.64
3	118	82.125	2.23
4	286	82.125	2.23
5-Manipulated[1]	NA	66.50	20.83

[1]Upon stabilization of the sand elevation, the column was manipulated via external vibration to induce further consolidation, if possible.



Photograph 28.—Mixed foamed sand or FTS.

Laboratory Testing Discussion and Recommendations

The laboratory testing indicates that the PLDCC is highly susceptible to consolidation through not only head pressure, but also pumping pressures. The testing indicated a 26% increase from average Batch density to average Control sample density, taken immediately prior to pumping and at the end of the pump hose, respectively. Furthermore, all samples showed a consistent increase in density when pumped into each test set-up and placed under head pressure. Specimens obtained at the bottom of the 32-foot set-up, 32-0.5, had an average density of approximately 105 pcf, approximately 94% of the neat grout density of 112 pcf; indicating that under sufficient pressure, the foam is not fully stable, and the placed material will approach the neat grout density. On average, the density of the material at the lowest point increased by approximately 2.7 pcf per foot of elevation pumped (head pressure); however, the increase in density was greater within the lowest 5-feet of all test set-ups. Densities at the top of each set-up were also consistently lower than Batch and Control densities, indicating that some volume of foam migrated upward through and concentrated in the upper portion of each pipe during curing. Irregularities in the density and strength of specimens 32-15, 32-20, and 32-25 are consistent with the starting and stopping of pumping at 15 ft and are likely a result of non-uniform flow through the pipe cross-section when pumping was resumed. This created non-homogeneous sections and weaker failure planes within the specimens. Permeability testing, while relative, indicates low permeabilities for densities greater than 40 pcf. While the permeability testing was performed as a falling head test under atmospheric conditions, it can be reasonably assumed that material with densities greater than 40 pcf will not be free-draining or behave similar to traditional filter material. It is recommended that additional permeability testing with more traditional rock or soil permeability test set-ups be performed to validate these findings.

Under no external pressure during curing, the change in density through the cross-section of horizontal set-ups also indicates that the foam was not stable within the solution and migrated up and out of the grout during curing. The PLDCC was significantly affected by the presence of water when pumped into the horizontal 8 FT-H2O set-up. While clean water was initially discharged during pumping, significant water remained within the sample as it was more dense than the PLDCC and settled to the bottom of the pipe. These results indicate there is limited effectiveness for PLDCC to fully displace water present within a void, and that the water, when drained, will leave a secondary void. The PLDCC did encapsulate some debris and the debris did not appear to affect the pumping operations or the final cured product.

While not the focus of this report, PLDCC, utilizing the same AQUAERiX foam manufactured by Aerix Industries, was utilized on a Reclamation project with the intent of filling voids with a low density, permeable, erosion resistant material. Ultimately, this project was partially abandoned due to structural distress caused by the self-weight of the PLDCC. This failure was not investigated further at the time; however, it is plausible that a PLDCC densification occurred, similar to the findings discussed herein, that contributed to the structural distress.

It is recommended that Aerix, the PLDCC foam manufacturer be contacted to discuss these results and provide updated guidance, if any, on mixture proportioning. Based on these initial results, PLDCC does not appear to be a viable solution for scenarios where permeable, low-density materials are desired but pumping under pressure will be required, or where the void contains water, due to the effects of pressure and water on the PLDCC properties.

The FTS was unable to be pumped, limiting the overall findings of this work. While the FTS was batched in general accordance with readily available technical guidance from Aerix, it is recommended that they be contacted directly for follow-up guidance and troubleshooting of the inability to pump the material. The testing that was performed indicated that the material did not fully self-consolidate after pumping. External manipulation was required to achieve a degree of consolidation approximately equivalent to the 25% foam at which the FTS was batched. It is recommended that this be investigated further upon achieving a mixture that can be adequately pumped.

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										Repair		Renovation			Replacement	
Project Name	**Issue Date**	**Conduit / Tunnel / Siphon**	**Canal / Spillway**	**Seepage Mitigation Added**	**Large Void / Sinkhole**	**Void on Downstream side**	**Structure Severely Deteriorated**	**Increasing Flow**	**Material in Leakage**	**Grouting**	**Compaction Grouting**	**Sliplining**	**CIPP**	**Lining Overlay**	**Excavation & Replace**	**Abandonment**
Applegate Dam	1981	X						X	X	X						
Arkabutla Dam	2003	X						X		X		X				
Balderhead Dam	1967	X		X	X			X	X	X						
Clair Peak Dam	2003	X			X											X
Como Dam	1992	X					X	X				X				
Dalewood Dam	1995	X						X	X			X				
Lake Darling Dam	1988	X		X						X						
Matahina Dam	1987														X	
Pablo Dam	1993	X		X			X		X						X	
Pine Creek Dam	1970	X							X	X						
Pinery Dam		X						X	X			X				
Porjus Dam	1993	X		X	X					X						
Ridgeway Dam	1986	X								X						
Rolling Green Dam	1999	X	X	X	X		X					X			X	
Round Rock Dam	1991	X										X				
Sardis Dam	1974	X			X					X						
St. Louis Dam							X									X
St. Pardoux Dam	1991	X			X	X		X	X							
Turtle Lake Dam	1997	X			X	X	X					X			X	
Waterbury Dam	1999	X		X											X	
Willow Creek Dam	1996	X			X					X		X				
Wister Dam	1949			X	X			X	X			X				