

**CURRENT CREEK DAM TOE DRAIN REHABILITATION**  
**PROJECT ID: CEEN\_CPST\_010**

**by**

**EMP<sup>2</sup>**

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**A Capstone Project Final Report**

**Submitted to**

**Troy Ovard**  
**Central Utah Water Conservancy District**

**Department of Civil and Construction Engineering**  
**Brigham Young University**

**April 18, 2023**

## Executive Summary

**PROJECT TITLE:** CURRANT CREEK DAM TOE DRAIN REHABILITATION  
**PROJECT ID:** CEEN\_CPST\_010  
**PROJECT SPONSOR:** Central Utah Water Conservancy District (CUWCD)  
**TEAM NAME:** EMP<sup>2</sup>

The objective of the Currant Creek Dam Toe Drain Rehabilitation project was to evaluate and propose a solution for the damaged right toe drain at Currant Creek Dam. The primary objective was to have a preliminary design proposal completed by April 2023. The design proposal will be used by CUWCD to get approval for and to complete the rehabilitation project on behalf of the Bureau of Reclamation (USBR).

The relevant tasks of this project included the following: Researching and analyzing the current state of the toe drain, researching and applying known solutions in context of the project, proposing three alternative solutions, holding a meeting with CUWCD and USBR to narrow down the alternatives to a final preferred alternative, creating a preliminary plan set for the preferred alternative, and writing a final report.

Research on the current state of the toe drain showed that the current toe drain is difficult to access, and trenchless technologies are therefore not recommended. Three alternatives of 1) doing nothing 2) excavating in place and 3) installing a new drain downstream were proposed to CUWCD. CUWCD chose the third alternative for further design.

This final report contains details on the design of the preferred alternative. The preferred alternative addresses the long-term safety of the dam by ensuring that it will not fail due to dam saturation. It also addresses construction safety by excavating less of the dam face using a safe cut slope.

Slope stability analysis using Spencer's Method of Slices showed a safe cut slope of 1.5:1 for excavation during construction. While most excavation will occur downstream of the existing toe drain to lower excavation quantities, some excavation will cut further into the dam to install connecting gravel drains between the existing and new pipes. This final report includes a preliminary plan set for the preferred alternative of installing a new toe drain further downstream of the original pipe.

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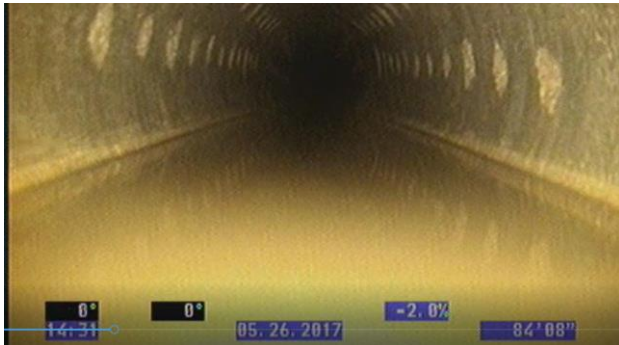
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## Introduction

Currant Creek dam is an earthen dam with a damaged right toe drain. A video inspection of the pipe taken in 2017 shows problems with the drains including bellied sections and blocked/damaged portions (likely from the original construction of the dam). See Figures 1 and 2. The last 20 years of measured data also show almost no measured flow in the drain. Furthermore, piezometer data shows a steady increase in the elevation of the phreatic surface at various points in the dam within the dam over the past several years, likely due to the right toe drain issues. All this caused CUWCD to worry about the long-term stability of the dam and its ability to properly drain. They want to preemptively rehabilitate the toe drain to improve drainage.



*Figure 1: Bellied portion of pipe at 84' from downstream manhole*



*Figure 2: Blocked portion of pipe at 170'*

Accessing the right toe drain for rehabilitation is difficult as there is only downstream access at a manhole. The rest of the pipe's location is unknown except for a possible location based on an assumed slope of 1.5% and the construction plans. The assumed location shows that the toe drain could be as deep as 26' at the end under the service road and dam face.

Due to these difficulties, CUWCD requested rehabilitation solutions. To provide the solution, the BYU capstone team researched and analyzed the current state of the toe drain, applied known solutions in context of the project, proposed three alternative solutions to CUWCD and USBR, narrowed down the alternatives to a final preferred alternative, and created a preliminary plan set for the preferred alternative.

**Schedule**

Table 1 shows the work schedule as actually completed. All deliverables were sent in email to Troy Ovard and Jared Hansen at CUWCD.

*Table 1: Work schedule*

| <b>Week of</b> | <b>Work Schedule</b>                                       | <b>Meetings with CUWCD</b>              | <b>Deliverables</b>                |
|----------------|------------------------------------------------------------|-----------------------------------------|------------------------------------|
| 11/13/2022     | Complete Statement of Work                                 |                                         | Statement of Work Document         |
| 11/20/2022     | (Thanksgiving break)                                       | -                                       | -                                  |
| 11/27/2022     | Research                                                   | Meeting with CUWCD and BLM (11/29/2023) | November Monthly Report            |
| 12/4/2022      | Research                                                   |                                         |                                    |
| 12/11/2022     | Research/Build existing Civil 3d Model                     |                                         | December Monthly Report            |
| 12/18/2022     | (Christmas Break)                                          | -                                       | -                                  |
| 12/25/2022     | (Christmas Break)                                          | -                                       | -                                  |
| 1/1/2023       | (Christmas Break)                                          | -                                       | -                                  |
| 1/8/2023       | Apply Research/Case studies                                |                                         |                                    |
| 1/15/2023      | Apply Research/Case studies/ Build existing Civil 3d Model |                                         |                                    |
| 1/22/2023      | Apply Research/Case studies/ Build existing Civil 3d Model |                                         |                                    |
| 1/29/2023      | Prepare Three Alternatives Report                          |                                         | January Monthly Report             |
| 2/5/2023       | Prepare Three Alternatives Report                          |                                         |                                    |
| 2/12/2023      | Prepare Three Alternatives Report                          | Three alternatives Meeting              | *Three alternatives Meeting Report |
| 2/19/2023      | Apply Research/Case studies                                |                                         |                                    |
| 2/26/2023      | Slope Stability Analysis/Plan Set Development              |                                         | February Monthly Report            |
| 3/5/2023       | Slope Stability Analysis/Plan Set Development              |                                         |                                    |
| 3/12/2023      | Slope Stability Analysis/Plan Set Development              |                                         |                                    |
| 3/19/2023      | Slope Stability Analysis/Plan Set Development              |                                         |                                    |
| 3/26/2023      | Slope Stability Analysis/Plan Set Development              |                                         | March Monthly Report               |
| 4/2/2023       | Prepare Final Report                                       |                                         |                                    |
| 4/9/2023       | Prepare Final Report                                       |                                         |                                    |
| 4/16/2023      |                                                            | Final presentation                      | Final Report                       |

\*Contains important reference documentation and reports for discarded alternatives. To be used as reference if the preferred alternative of relocating the drain downstream is discarded during final design planning.

## **Assumptions & Limitations**

### Model Assumptions and Limitations

The following information applies in detailing how the Civil 3d model and preliminary plan set were created:

- The elevation data used came as a processed 1-meter Bare Earth LiDAR DEM from [gis.utah.gov](http://gis.utah.gov) (see Appendix B for details). Vertical accuracy was 18.1 cm at the 95% confidence level.
- Stationing on the dam crest was based off of stationing in the original construction plans and was converted from Utah State Plane NAD27 to NAD83 using NOAA's NCAT online transformation tool.
- The drainage blanket location in the model was interpolated from the geologic cross sections from the USBR. The dam crest stations 15+00, 12+00, and 9+00 were all direct reflections of what the cross sections show, while all other stations were linear interpolations.
- The phreatic surface in the model was created from piezometer data using piezometers that had influence in zone 1, 2, or the blanket drain at stations 15+00, 12+00, and 9+00 in 2022.
- The piezometer coordinates came from Brig at CUWCD from the Asset Management department.
- The current toe drainpipe location was estimated based off of an assumed 1.5% slope, depth at manhole of 8 feet, and approximate direction according to original construction plans. The slope was estimated by taking the instantaneous slope measurement during the 2017 maintenance video every 4 feet and averaging the results. The depth of the manhole was measured in field but was done quickly and approximately and needs to be field verified.
- In the preliminary plan set, no engineering calculations were performed to determine the connectivity of the proposed pipe to the existing blanket drain. Locations of drainage gravel tying the blanket drain and existing toe drain to the proposed drain is recommended based off of similar work done at Red Fleet Dam. However, no calculations were performed by the BYU capstone team which explain where and how many of these connections are necessary.
- Pipe size was kept at 8 inches. The only analysis done to decide this size was comparing the potential flow through an 8 in pipe to measured flow in the left toe drain, which has never exceeded 20 gpm and averages around 4 gpm. Calculating flow in an 8-inch pipe using the Manning's equation results in a possible flow of 433 gpm, well above measured flows in the dam over the years.

Slope Stability Assumptions and Limitations

- Cross section 11+00 (of original dam stationing) was chosen for the slope analysis as it appeared to have the greatest cut depth and therefore required the most conservative design. The location of the blanket drain was interpolated using the available geological cross sections, which were available at sections 9+00 and 12+00.
- The cross-section composition was found by using the maximum dam cross section from the original plan set. The maximum cross section was laid on top of the existing cross section (from the LiDAR data, discussed in model assumptions above), and matching slopes were lined up. The slopes lined up almost exactly, reassuring us that the assumptions we made were accurate of existing conditions.
- Material properties for the cross section were not found in any of the construction or preconstruction documents, so they were assumed using the USCS classification average values. These can be found in Table 2 below.
- The slope stability was analyzed assuming pore pressure only in the core of the dam using the phreatic line in the model (discussed in model assumptions above).
- The force behind the dam was assumed to be that of the reservoir filled at the maximum water level, an elevation of 7683.2 ft. A lower water elevation could give different results.

*Table 2 Material Property Assumptions*

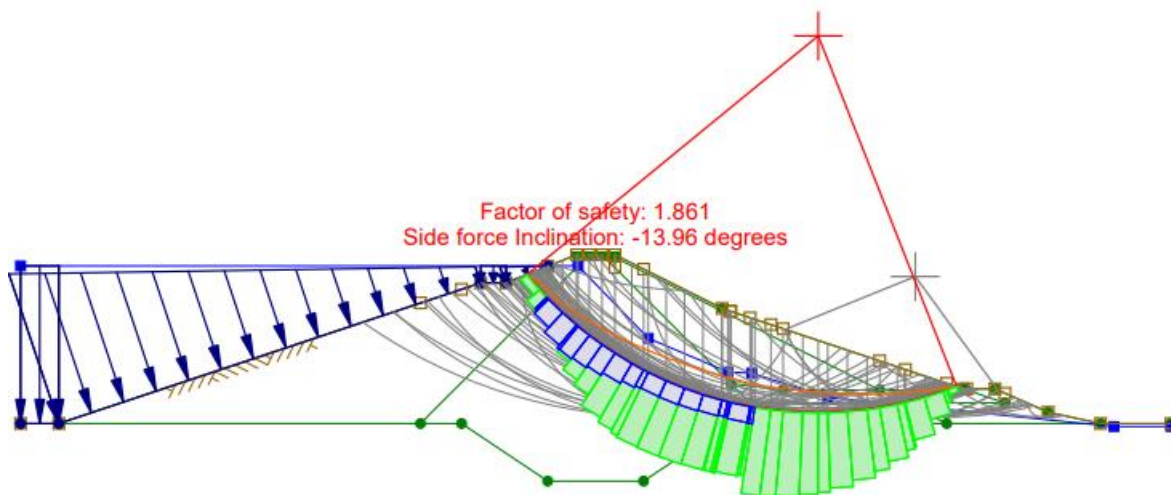
| Material | Given Description                              | Assumed USCS Class | Cohesion | Friction Angle |
|----------|------------------------------------------------|--------------------|----------|----------------|
| 1        | Selected Clay, Silt, Sand, gravel, and Cobbles | CH                 | 25       | 22             |
| 2        | Selected Sand, Gravel and Cobbles              | GM                 | 0        | 36             |
| 3        | Cobbles and Boulders                           | GW                 | 0        | 40             |

## Analysis, Results, & Design

The results in this section focus primarily on the analysis and resulting preliminary plan set of the third alternative from the “Three Alternatives Meeting Report”. For more details about other alternatives and the reasoning for selecting this alternative, refer to Appendix D. Appendix C contains the preliminary plan set in full.

### Analysis

To determine a safe cut slope, cross sections of the dam at station 11+00 were analyzed through iterations of different cut slopes and locations. Spencer’s Method of Slices was performed using an educational version of UTexas4. The dam was first analyzed at the existing cross section without any cuts, which came out to be a factor of safety of 1.828. Then the different cut locations and slopes were iterated through. As specified by the USBR, the factors of safety needed to range from 1.2-1.8 to meet standard. An example of the UTexas4 outputs can be found in Figure 3. The cut slopes scenarios and resulting safety factors analyzed were simplified since UTexas4 outputs are not very legible and are shown in Figure 4. They can also be found in Appendix C as a part of the preliminary plan set.



*Figure 3 Current Creek Dam Slope Stability UTexas4 Output*

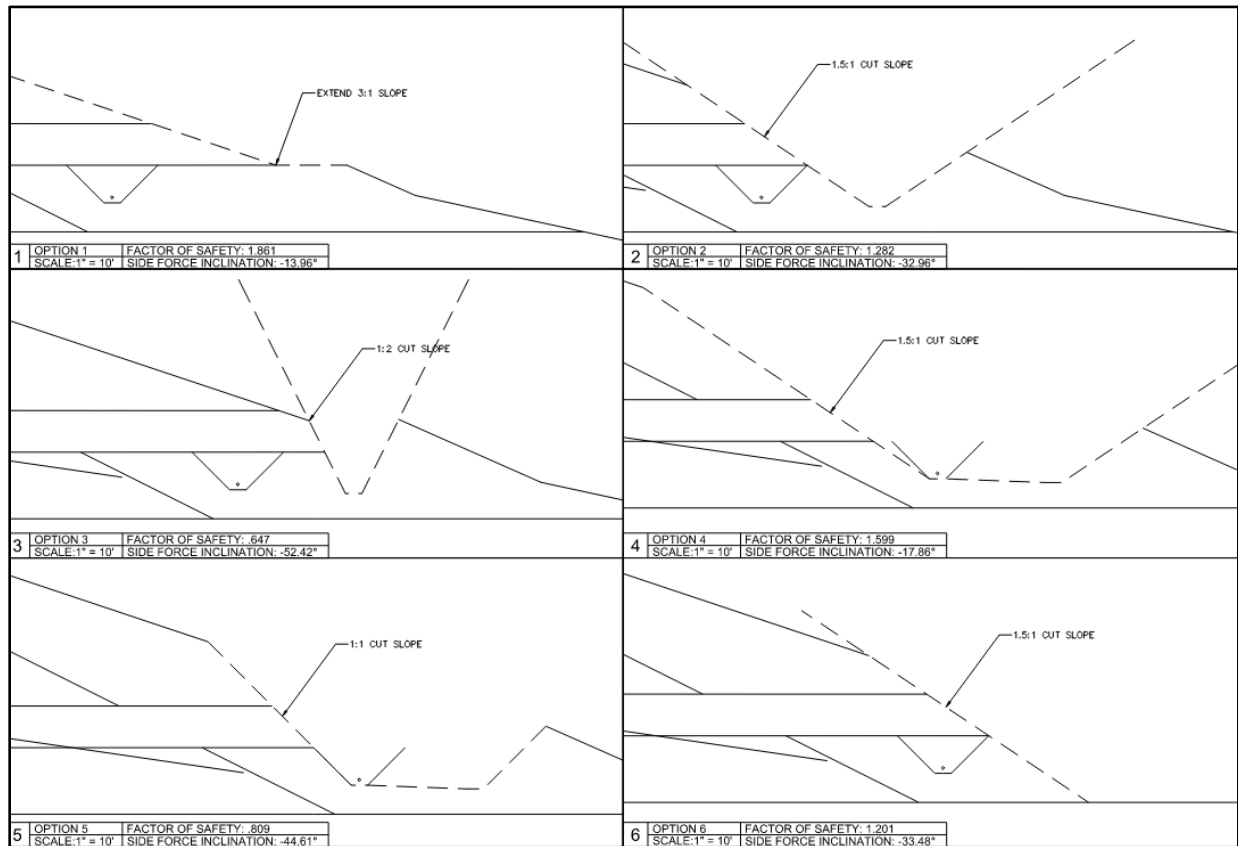


Figure 4: Simplified Utexas4 Slope Stability Scenario Results

**Results**

The cut slope analysis resulted in the conclusion that 1.5:1 was the safest cut slope that provided the minimum excavation. This was evaluated at two locations. Cutting at a new toe drain location provided a factor of safety of 1.282 and cutting at the existing toe drain provided a factor of safety of 1.599. The two cut sections were provided so that the new toe drain could be connected to the existing toe drain at selected location, allowing for better connectivity to the blanket drain. The cut slope and resulting toe drain installation is shown in figure 5. The sheet can also be referenced in Appendix C with the preliminary plan set.

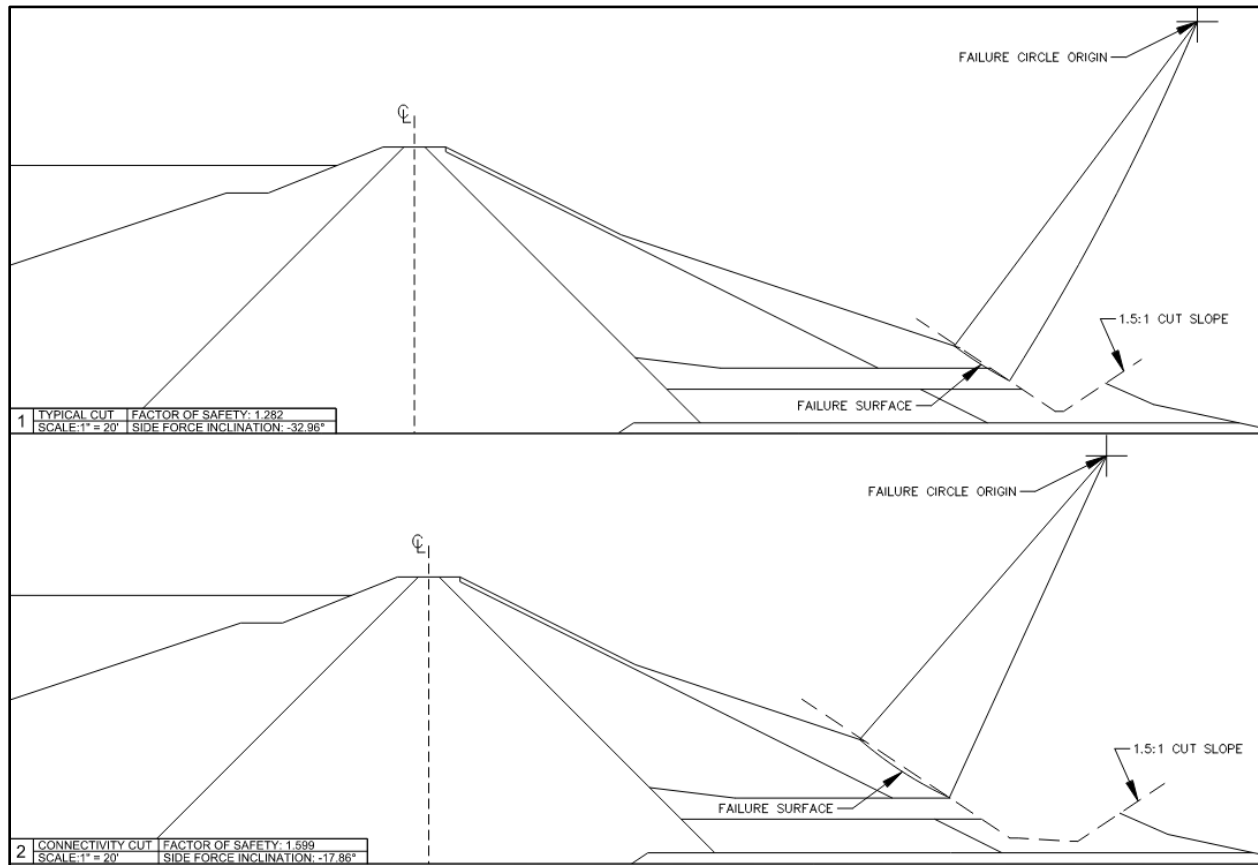


Figure 5: Cut Slope Factor of Safeties for a 1.5:1 Slope for Both the New and Existing Pipe

## Design

Using the cut slope of 1.5:1 at the approximate future location of the toe drain as shown in the plan view in Figure 6, the excavation quantity and materials were calculated. See Figure 7 for cut calculations along the proposed pipe profile. As shown in Table 3 approximately 11,000 cubic yards of material will be excavated depending on how far the pipe is extended toward the groin of the dam and how many gravel drains are extended to the existing pipe and blanket drain.

Total fill required varies based on the station and road vs dam location. Much of the road fill will be reused, but drainage gravel and sand type and quantity will need to be determined for laying the pipe itself. With a 3-foot square gravel envelope around the pipe the drainage gravel volume would be approximately 137 cubic yards. The washed sand to fill the trench would be approximately 573 cubic yards. See Table 3. Again, possible gravel blanket drain extension zones as shown in the preliminary plan set for blanket connectivity were not included in these approximate cut and fill volume calculations.

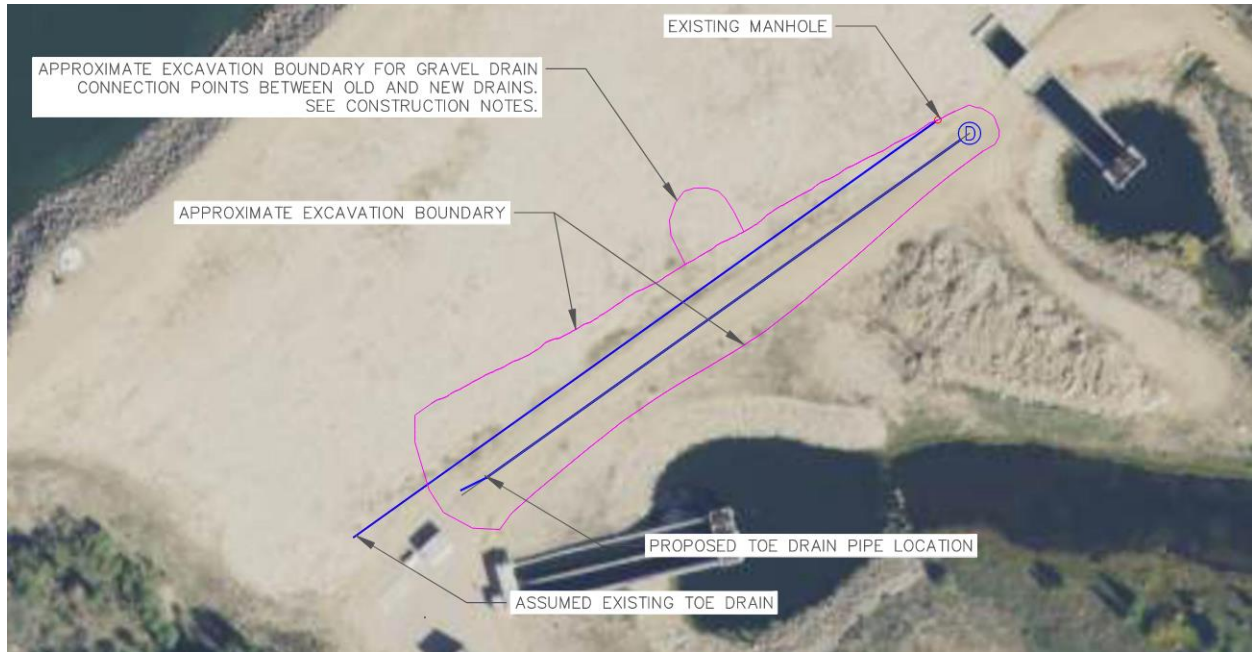


Figure 6: Plan View Showing Proposed and Current Toe Drain Locations

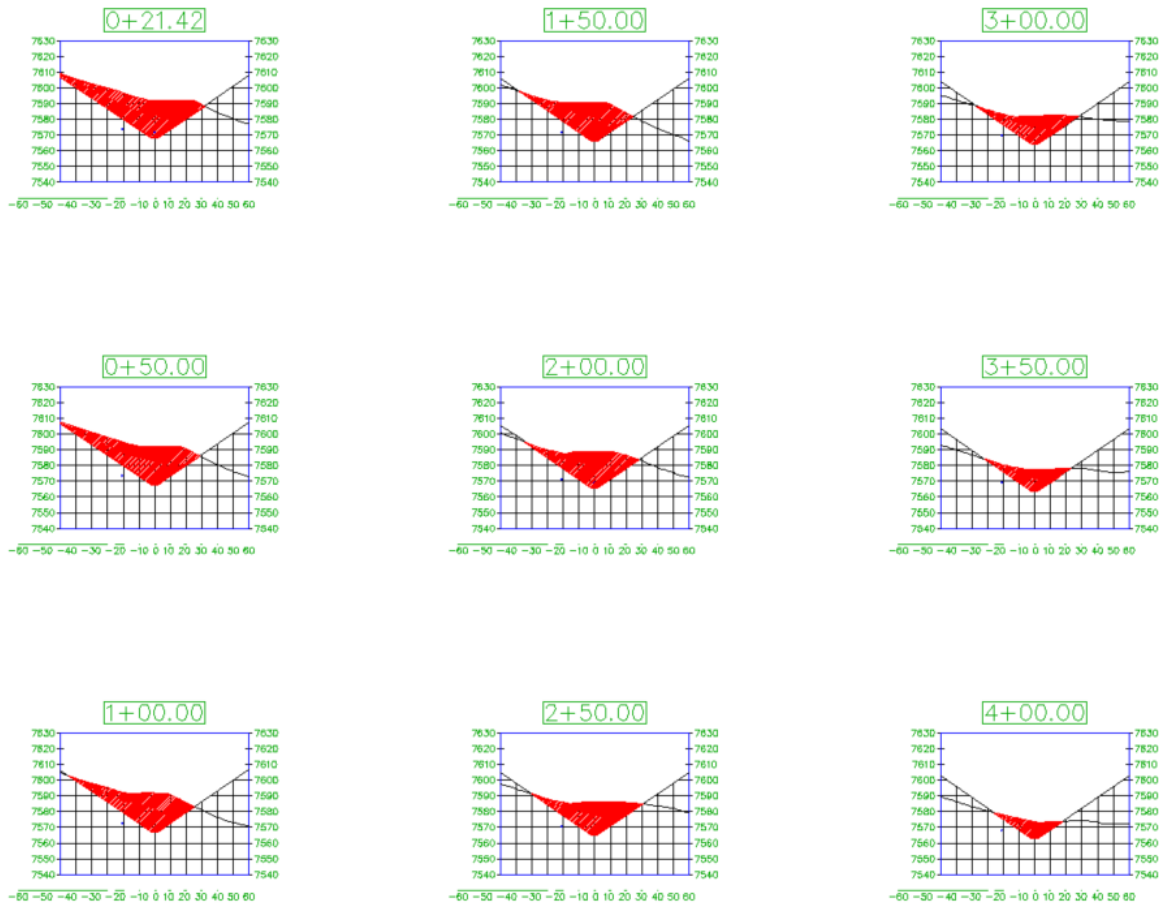


Figure 7: Cross Section Cut Calculations Along Proposed Pipe Profile

*Table 3: Approximate Cut and Fill Volume of Proposed Pipe*

| <b><u>Method</u></b> |                             | <b><u>Volume (Yd<sup>3</sup>)</u></b> |
|----------------------|-----------------------------|---------------------------------------|
| <b>Cut</b>           |                             | 11,000                                |
| <b>Fill</b>          | <b>Road/Dam replacement</b> | 10,290                                |
|                      | <b>Drainage Gravel</b>      | 137                                   |
|                      | <b>Washed Sand</b>          | 573                                   |

## **Related Issues**

The following issues may result from this project or if the project is not completed. They would need to be considered further before CUWCD implements the project.

1. **Safety:** If the project is not done, the dam could fail in the future and affect those who may live or work along Carrant Creek or those living downstream in Fruitland. If the project is executed, excavation on the dam will pose a risk during construction.
2. **Welfare:** If further analysis were to show that the water level would need to be lowered in the reservoir, this could limit the amount of water CUWCD would be able to deliver to those who rely on Carrant Creek Reservoir for water.
3. **Global Factors:** Carrant Creek Reservoir, part of the Central Utah Project, is part of a system that redirects water from the Colorado Basin and brings it into the Great Basin to be used by residents there. While this project would not significantly affect the amount of water being taken from that basin, the fact that it is tied in with this system could draw attention from concerned groups, both within and outside the state of Utah. The global perception of dams must also be considered. There exist many differing opinions on the necessity of dams, and a perceived lack of safety in dams can have a large effect on the public opinion of dams.
4. **Social Factors:** Should the water level of the dam be drawn down, recreation on the reservoir would be affected, leading to fewer users of the reservoir. Furthermore, construction processes, including sound pollution and sediment flowing downstream, could affect fish and wildlife, leading to fewer people using the area as a fishing location. Both of these influences on the region may draw negative reactions from those who use the region for recreation.
5. **Environmental factors:** All construction materials and processes have environmental costs, generally from the use of natural resources or releasing carbon dioxide into the atmosphere. Sound pollution, as mentioned in the discussion of social factors, would affect the wildlife in the region and likely keep them away. These environmental impacts would need to be limited as much as CUWCD can.
6. **Economic factors:** CUWCD would be paying for the project. As their revenue comes largely from Utah property taxes, the project could attract public attention. However, mitigation is always less expensive than repair. As such, rectifying the dam's drainage issue now would be less expensive than reparations would be should the dam fail. Additionally, assuming the previously listed factors lower the amount of visitors to Carrant Creek Reservoir, the Forest Service would lose money as a result of having fewer visitors in the region.

## **Lessons Learned**

The following list details the major difficulties the BYU capstone team encountered during this project and how they overcame those difficulties:

1. Technical difficulties with Civil 3d and UTexas4. The BYU capstone team had a rudimentary knowledge of how to work these programs and spent a large majority of time googling answers and collaborating to figure out how to use the programs in design and analysis.
2. No class experience in slope stability. The BYU capstone team did not have previous knowledge from class or work on slope stability analysis and had to meet with professors, learn from tutorials, and take a class offered at BYU to gain knowledge in this area.
3. Forecasting consequences of the “do nothing” alternative. The BYU capstone team did not have knowledge or experience in forecasting dam failure and considered a linear extrapolation of piezometer data a conservative approach. The team recognized and admitted the limitations to their forecasting.
4. Inability to implement theoretical trenchless technologies. The BYU capstone team researched many possible trenchless technologies that could have eliminated the need for trenching but found that many technologies had not been used on perforated pipes in a dam before. The solution was to stick with the more familiar method of trenching.

## **Conclusions**

This report has summarized the issue at Currant Creek Dam and has proposed a solution of excavating downstream of the existing toe drain and installing a new toe drain. The slope stability analysis performed yielded a safe cut slope of 1.5:1. This could be applied to excavate and install a new pipe as well as excavate further to the existing toe drain and install connecting gravel drains for blanket drain connectivity in several areas. A preliminary plan set in Appendix C shows details of the proposed rehabilitation plan. Specific recommendations for moving forward can be found in the Recommendations section below.

While these recommendations could be implemented immediately, the “do nothing” alternative described in Appendix D suggests 70+ years of functionality and therefore these recommendations do not have to be done immediately. This will be left to the discretion of CUWCD.

## **Recommendations**

The following recommendations should be implemented as this project begins:

1. CUWCD should take drill samples around the end of the blanket drain extent to determine how far it extends. This could affect exactly how close or far the new pipe will be installed. As described in the limitations section above, no calculations were performed to determine the blanket drain connectivity with the proposed new pipe. The number of connecting gravel drains along the existing pipe would need to be decided upon as well. Core samples could help determine this.
2. Use a locator and locate the exact depth, slope, and direction of the existing pipe. This may change the project bounds, including how much excavation of the dam would be needed.
3. While the slope stability analysis was performed at the maximum water surface elevation, it is recommended that the reservoir be lowered during construction to be conservative.
4. Only one cleanout is proposed at the end of the pipe, but if desired, a second could be placed in the middle of the pipe. This is not recommended as it would require a manhole in the middle of the service road.
5. Pumping may be required during excavation and was not calculated or analyzed.
6. Use a full walled HDPE pipe for the new toe drain as the depths and weight from equipment on the road will cause higher than average pressure on the pipe.
7. Use the preliminary plan set provided to start the construction and design process to receive approval from the USBR.

## **Appendix A – Team Member Resumes**

### **KAMDEN PETERSON**

208-943-1489 | kamdenpete@gmail.com

#### **Education**

**Bachelor's in civil engineering | April 2023 | Brigham Young University, Provo**

- Current GPA: 3.90
- Relevant course work completed: Hydraulics, Hydrology, and Advanced Culvert Hydraulics

#### **Work Experience**

**Asset Management Intern | Central Utah Water Conservancy District, Orem, UT | May 2022-Aug 2022**

- Built out 4 newly constructed or modified facilities in the District's asset management system
- Wrote 3 technical SOP documents for District operations
- Assisted in the project management of a Hazard Mitigation Plan update
- Assisted in writing a BRIC FEMA GO grant sub application
- Proposed and completed a field study project verifying functionality of 15 flumes and weirs
- Discovered 6 non-functioning flumes/weirs during project and proposed solutions

**Public Works Engineering Intern | City of Pleasant Grove, UT | Oct 2020-Sep 2021**

- Updated the sewer GIS system identifying neglected pipes saving thousands in future repairs
- Created concept plans and price estimates for a 40-acre city property for future park
- Instituted a standard operating procedure for updating the city's utility GIS program
- Collected information from multiple cities and prepared a storm water obstruction policy change

**Onsite Apartment Manager | Joseph Thomas Property Mgt., Provo, UT | May 2020-Sep 2021**

- Troubleshoot and solved maintenance problems including but not limited to plumbing, electrical, hardware and carpentry, and yard maintenance

**Electrician's Assistant | Brigham Young University, Provo, UT | Sep 2018-Oct 2020**

- Installed 10+ electrical systems for major remodels according to project plans
- Maintained electrical systems by troubleshooting and fixing electrical problems

**Soils Lab Technician Intern | Earthtec Engineering, Lindon, UT | May 2020-Aug 2020**

- Determined soil, asphalt, concrete, and fireproofing properties through routine lab tests
- Performed proficiency tests by reading and following national ASTM or AASHTO standards

#### **Other Experience/Skills**

##### **Computer:**

- ArcGIS Pro, VBA, Microsoft Office, AutoCAD, Civil3D, WMS, HEC-RAS 1D

##### **Flow Measurement:**

- StreamPro 2 Acoustic Doppler Current Profiler with WinRiver II software
- HACH FH950 Current Meter

**ALEX MASKOVICH**  
(480)-621-0748 alexmaskovich@gmail.com

**WORK EXPERIENCE**

**EPS Group, Mesa, AZ**

*Land Development Civil Engineer Intern (Summer 2022)*

- Add construction notes and quantities to drawings and address client comments on plan sets.
- Write preliminary due diligence reports for potential jobs.

**Southland Ind/TCM, Portland, OR**

*Project Engineer Intern (Summer 2021)*

- Create and finalize submittals for Operation and Maintenance (O&M's), LEED, etc.
- Document and map all valve tags for owner use.
- Prepare Requests For Information (RFI's).

**Pete King Construction, Phoenix, AZ**

*Intern (September 2021-Present)*

- Assist in creating product data and submittals.
- Calculate takeoffs and prepare estimates.

*Metal Framer (Summer 2019 & Summer 2020)*

- Interior and exterior framing for commercial buildings.
- Read layout for interior and exterior framing.

*Warehouse Worker (Summer 2018)*

- Organize and inventory warehouse.
- Deliver and receive jobsite materials.

**Brigham Young University, Provo, Utah**

*Telephone Operator (2018- 2022)*

- Answer difficult questions about the University.
- Show kindness and customer service to every person who contacts the University.

**The Church of Jesus Christ of Latter-day Saints, Little Rock, Arkansas**

*Full-Time Volunteer Representative (June 2016 – July 2018)*

- Lead groups of 10-12 volunteers, conduct weekly training meeting, and follow up on goals.
- Plan, organize and teach training sessions.
- Assist people in attaining their desired level of spirituality.

**Nally Construction Company, Phoenix, AZ**

*Warehouse lead (Summer 2015; Summer 2016)*

- Responsible for loading and unloading delivery trucks.
- Inventory and maintain the warehouse.
- Follow up on paint orders and shipments.

**EDUCATION**

**Brigham Young University, Provo, Utah**

*Currently Attending (Anticipated Graduation, April 2023)*

- 3.43 GPA currently.
- Civil Engineering Major.
- Experience working with Civil 3D, current Revit software, Excel, and ArcGIS.

**ACCOMPLISHMENTS**

*Boys Scouts of America –Eagle Scout; Early Morning Seminary Graduate-(perfect attendance my senior year); All Section Second Team Varsity Volleyball Setter 2015-2016 Season.*

**Braxton Porter**

825 E Utah Highlands Dr, Lehi, UT 84043  
(719) 360-3177 · braxtonporter18@gmail.com  
www.linkedin.com/in/braxton-porter

**Education**

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**Brigham Young University, Ira A Fulton School of Engineering** GPA 3.40 Provo, UT  
Bachelor of Civil and Environmental Engineering August 2020 - April 2023

- Recipient of 50% tuition Civil Engineering Scholarship
- BYU ASCE Student Chapter Member
- Course emphasis in Water Resources and Environmental Engineering

**Hard Skills**

- 
- CAD – Civil 3D, Revit, AutoCAD
  - GIS – ArcGIS pro
  - Programming & data analysis – Python, Rstudio, Excel, VBA
  - Fluent in Spanish

**Engineering Experience**

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**Brigham Young University, Civil Engineering Department** Provo, UT  
**Research Assistant** (Under Dr. Darrel Sonntag) April 2022 - Present

- Develop an SOP for data collection equipment
- Analyze data, and adjust values to reverse equipment drift
- Prepare and bid presentation for the 2023 CRC Real World Emissions Workshop

**Brigham Young University, Religion Department** Provo, UT  
**Research Assistant** (Under Dr. Kerry Muhlstein) Dec 2021 - April 2022

- Set standards and develop plans for the *Seila Pyramid Excavation* in Egypt

**Herriman City** Herriman, UT  
**Water Resources Engineer Intern** Jan 2022 – April 2022

- Assist the City Engineer in projects as needed
- Evaluate project costs and compare with predictions
- Perform city water fee audit, correcting the fees of 21% users

**Epic Engineering** Heber, UT  
**Structural CAD Technician** Feb 2021 – Feb 2022

- Update and establish new company plan standards
- Presenting information in an easily understandable manner for contractors
- Effectively communicate with the structural engineers to develop 3D models and blueprints

**Volunteer Experience**

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**The Church of Jesus Christ of Latter-Day Saints** Huancayo, Peru  
**Full-time Representative** June 2018 – May 2020

- Serve and teach the local people
- Managed, lead and guided other representatives as their assigned leader
- This service opportunity taught me in-valuable skills, one being the power of hard work

## MATTHEW EVANS

664 E 500 N, Provo, UT, 86404 · 952-484-5818  
matthewmeevans@gmail.com · www.linkedin.com/in/matthew-evans-64334a143/

### Education

GRADUATION: DECEMBER 2023

**B.S. CIVIL & ENVIRONMENTAL ENGINEERING, BRIGHAM YOUNG UNIVERSITY**

- GPA: 3.72
- Earned scholarships and grants totaling over \$16,000

### Work and Project Experience

**CIVIL ENGINEERING INTERN, BOWEN COLLINS & ASSOCIATES – DRAPER, UT**

**MAY 2022 – PRESENT**

- Assists in writing Water Master Plans for clients by generating tables and figures from water use data
- Wrote an application for a \$5 million Bureau of Reclamation grant (R23AS00005) to help fund the construction of a client's water reuse pipeline and pump station project
- Calls valve manufacturing companies to evaluate the ability of their valves to satisfy the demands of proposed projects
- Leads an effort to update a client city's water utility (PI, culinary, sewer) GIS maps using ArcMap and ArcGIS Pro
- Uses AutoCAD Civil 3D to make water system schematics, draft construction drawings, make changes to clients' standard drawings sets per redline request, and help in the design of pipelines and tanks
- Uses InfoWater Pro to delineate pressure zones in water system and design unidirectional flushing sequences

**ASSET MANAGEMENT INTERN, CENTRAL UTAH WATER CONSERVANCY DISTRICT – OREM, UT**

**MAY 2021 – APRIL 2022**

- 150+ hours in the field attending construction project meetings, inspections, and assisting in condition assessments
- Reviewed hundreds of engineering drawings of past and future construction for asset management purposes
- Generated replacement cost estimations of company assets to insure accurate cost predictions for future capital replacement projects
- Conducted a case study analyzing the costs and benefits of certifying the company's administration campus under LEED O+M v4 certification

**STUDENT AND MENTOR, FIRST ROBOTICS COMPETITION TEAMS 2169 AND 6844**

**TEAM 2169: 2013 – 2017, TEAM 6844: 2017-2018**

Participated in a high school robotics competition on a highly competitive team and mentored a team in college.

- Led the mechanical department, which oversaw the design and build of a 120 lb. competition robot
- In 2017, Received Chairman's Finalists recognition (one of six in the world), the highest award in the program
- Was the lead design and build mentor on a first-year robotics team and worked and worked under an extremely tight budget (less than \$1000). The team qualified for the world championship and placed as the highest performing rookie in the division.

### Technical Skills

- Proficient in ArcMap, ArcGIS Pro, and InfoWater Pro
- Proficient in Microsoft Excel, Word, and PowerPoint
- Proficient in Adobe Acrobat and Revu Bluebeam
- Experienced in AutoCAD and AutoCAD Civil 3D
- Class experience with Python and Java
- Class experience with land surveying
- Exposure to HEC-RAS

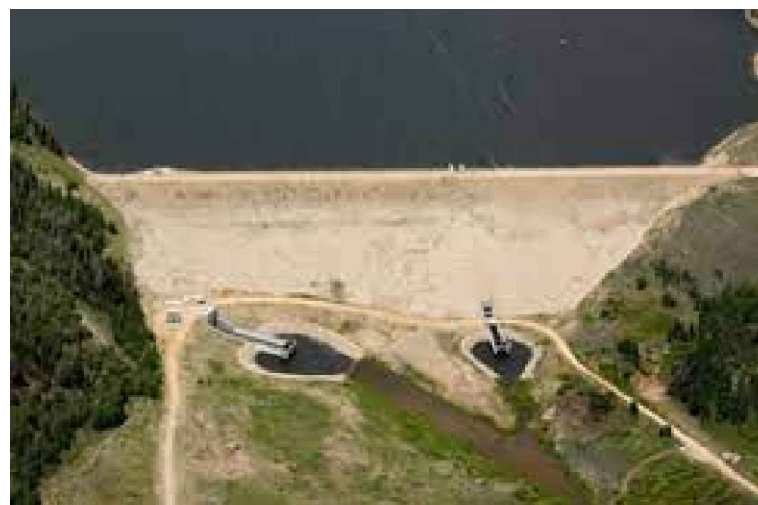
**Appendix B - LiDAR Source Information**

1 Meter Bare Earth LiDAR DEM

|                     |                                                                                                                                                                                                                     |
|---------------------|---------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| Year                | 2020                                                                                                                                                                                                                |
| File Format         | TIFF                                                                                                                                                                                                                |
| File Extension      | .tif                                                                                                                                                                                                                |
| Average File Size   | 3 MB                                                                                                                                                                                                                |
| Horizontal Accuracy | n/a                                                                                                                                                                                                                 |
| Vertical Accuracy   | 10.0cm RMSEz, and 18.1cm at the 95% Confidence Level                                                                                                                                                                |
| Contact             | UGRC - Rick Kelson - RKelson@utah.gov                                                                                                                                                                               |
| Stored at UGRC?     | Yes                                                                                                                                                                                                                 |
| Storage Path        | <a href="https://storage.googleapis.com/state-of-utah-sgid-downloads/lidar/central-southern-utah-2020/DEMs/">https://storage.googleapis.com/state-of-utah-sgid-downloads/lidar/central-southern-utah-2020/DEMs/</a> |
| Web Page            | <a href="https://gis.utah.gov/data/elevation-and-terrain/2020-lidar-central-southern-utah/">https://gis.utah.gov/data/elevation-and-terrain/2020-lidar-central-southern-utah/</a>                                   |

**Appendix C – Preliminary Plan Set**

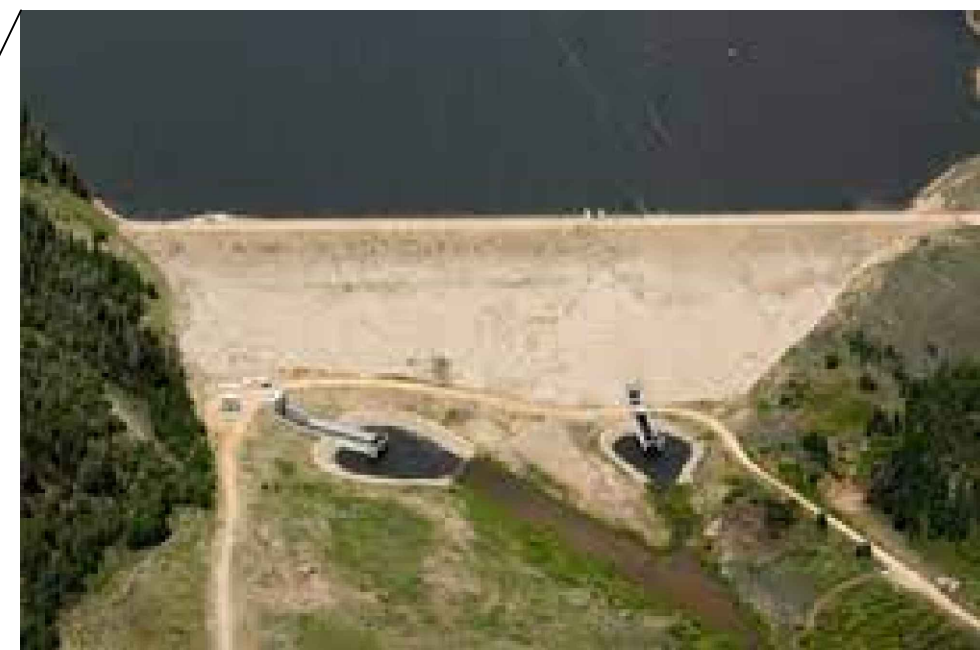
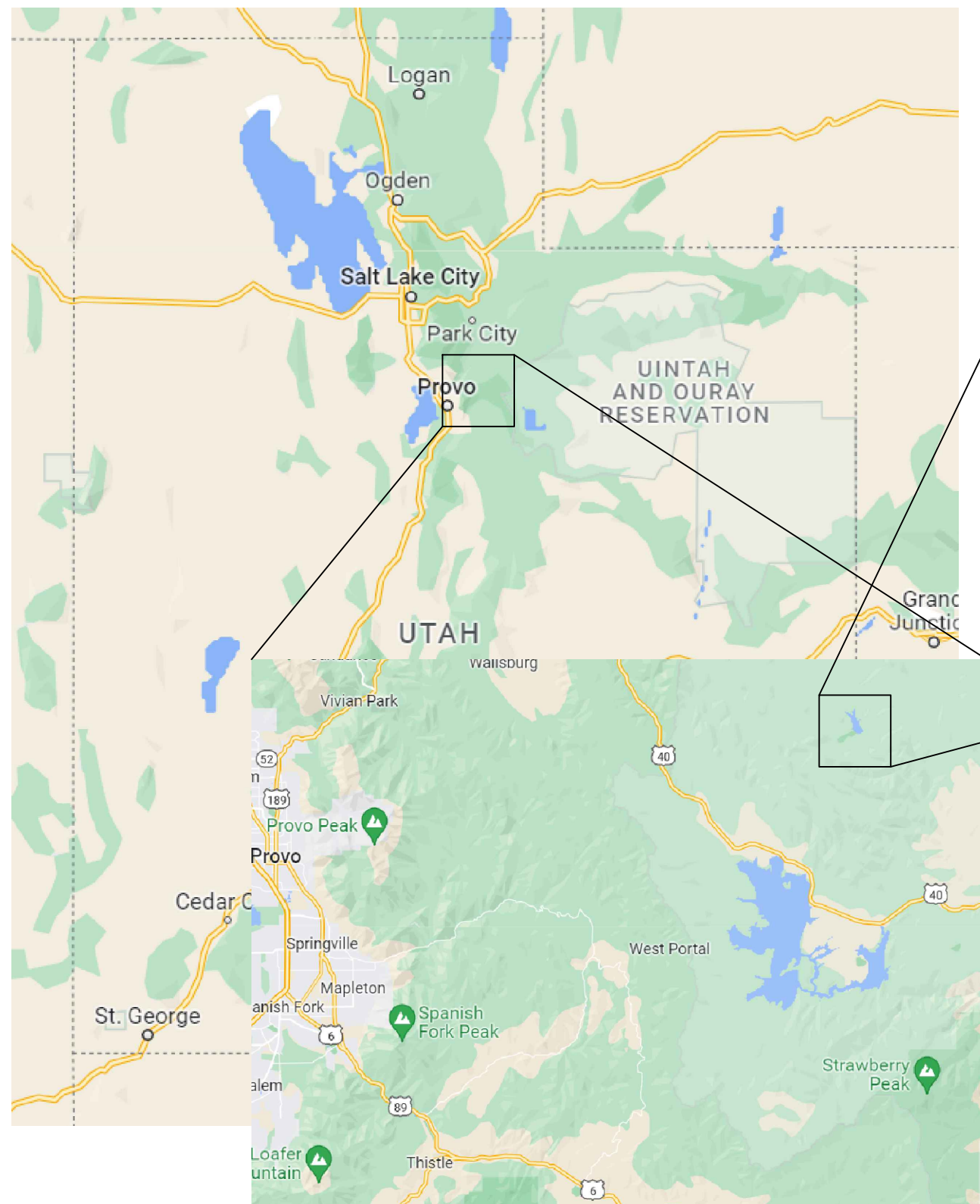
# Currant Creek Dam Toe Drain Replacement (Preliminary Design Drawings)



BYU  
Civil Engineering  
Capstone Team 10

---

PROVO CAMPUS  
1 UNIVERSITY HILL, PROVO, UTAH 84606



| SHEET CONTENTS |                    |
|----------------|--------------------|
| SHEET NUMBER   | TITLE              |
| C-01           | COVER SHEET        |
| C-02           | VICINITY MAP       |
| C-03           | PROJECT PLAN VIEW  |
| C-04           | EXISTING PROFILE   |
| C-05           | PROPOSED PROFILE   |
| C-06           | SLOPE STABILITY    |
| C-07           | CUT CROSS SECTIONS |
| C-08           | CUT SECTIONS       |
| C-09           | DAM STATIONS       |
| C-10           | DAM STATIONS       |
| C-11           | DAM STATIONS       |



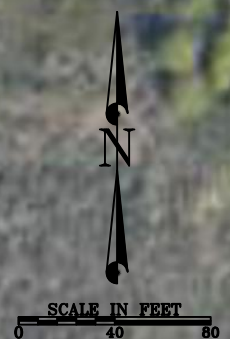
APPROXIMATE EXCAVATION BOUNDARY FOR GRAVEL DRAIN CONNECTION POINTS BETWEEN OLD AND NEW DRAINS. SEE CONSTRUCTION NOTES.

APPROXIMATE EXCAVATION BOUNDARY

EXISTING MANHOLE

PROPOSED TOE DRAIN PIPE LOCATION

ASSUMED EXISTING TOE DRAIN

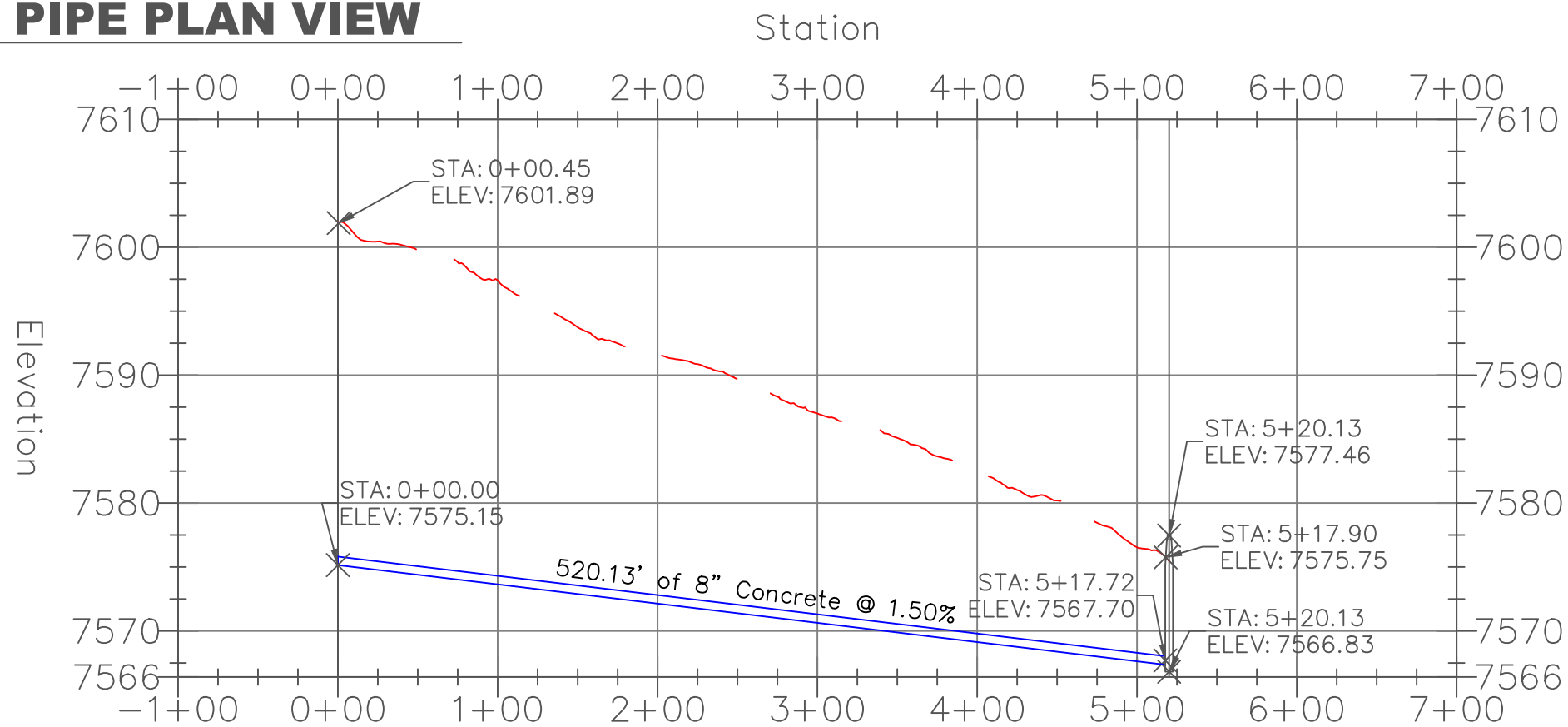




**1**

**EXISTING PIPE PLAN VIEW**

SCALE: 1" = 100'



NOTES:  
PROPOSED PIPE LOCATION  
DETERMINED FROM ASSUMED  
SLOPE, THE DOWNSTREAM  
MANHOLE LOCATION, AND  
ORIGINAL CONSTRUCTION PLANS.  
ACTUAL LOCATION UNKNOWN.

**2**

**EXISTING PIPE PROFILE**

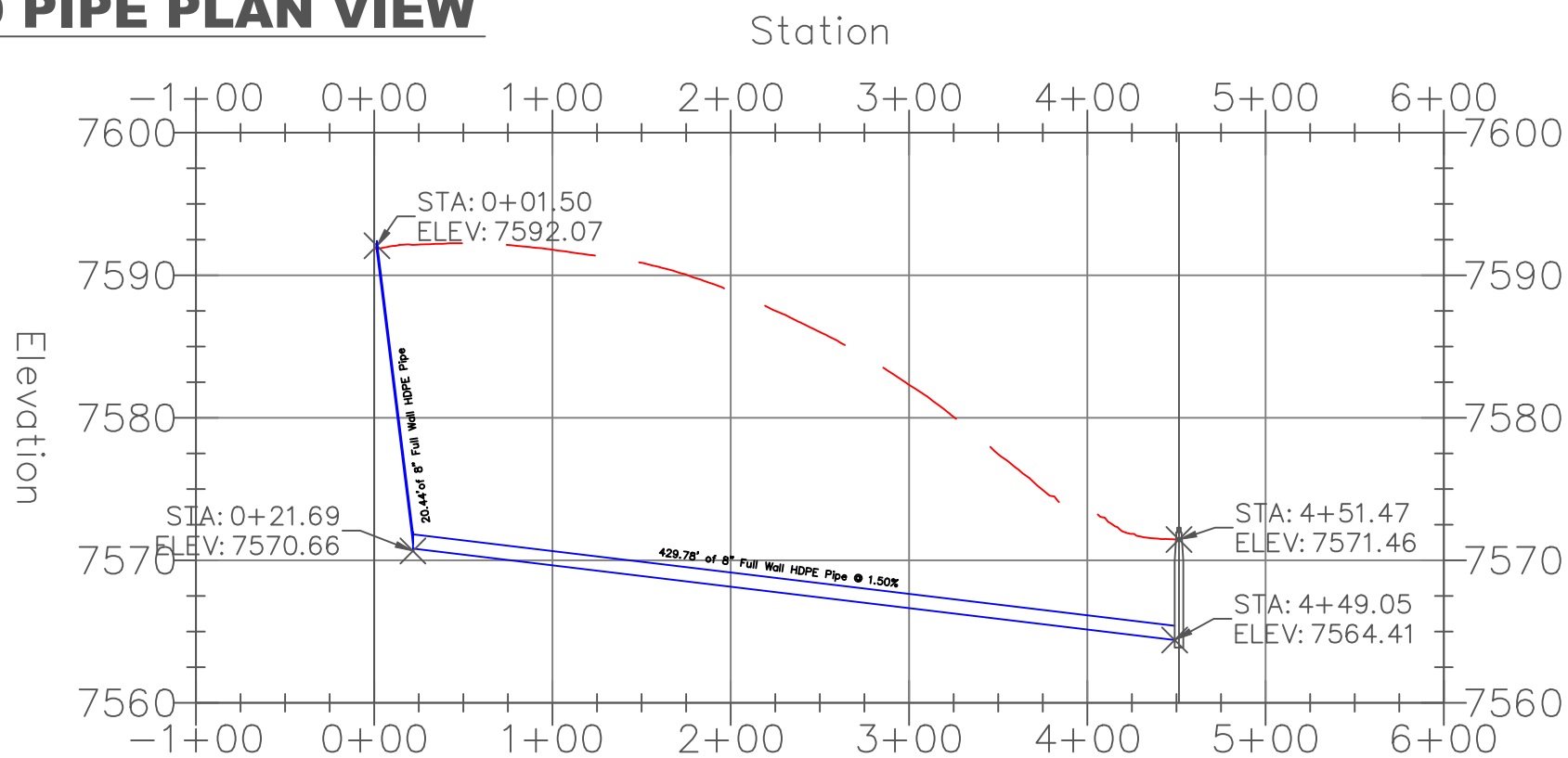
SCALE: 1" = 100'



**1**

**PROPOSED PIPE PLAN VIEW**

SCALE: 1" = 100'



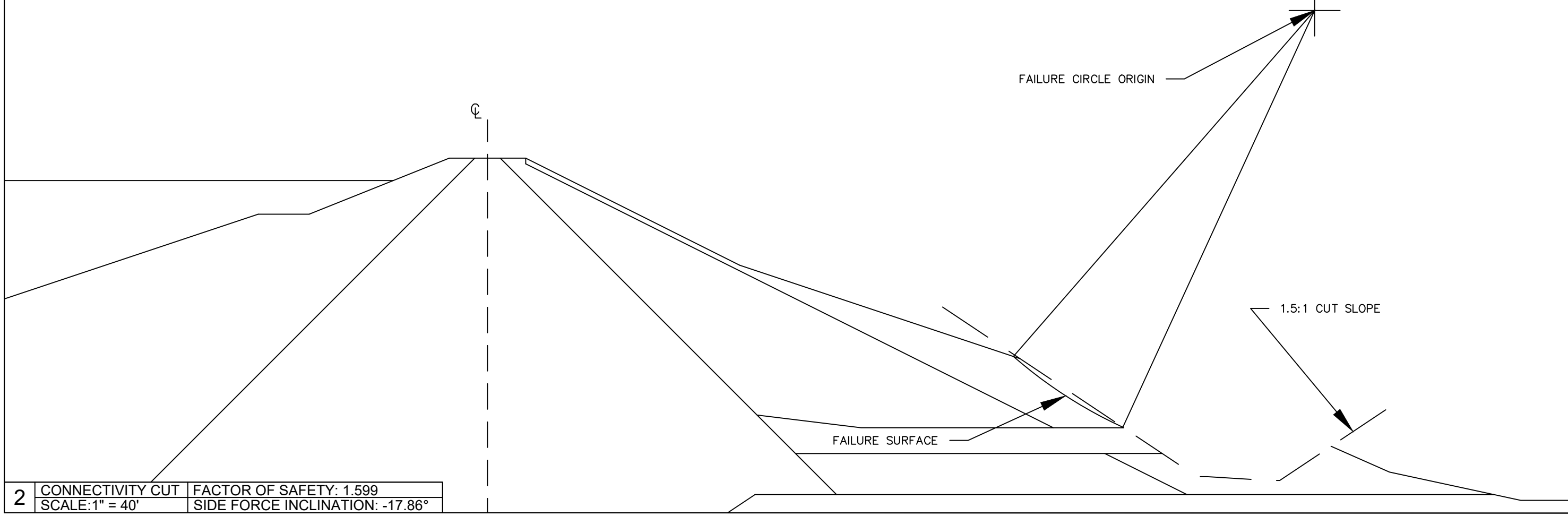
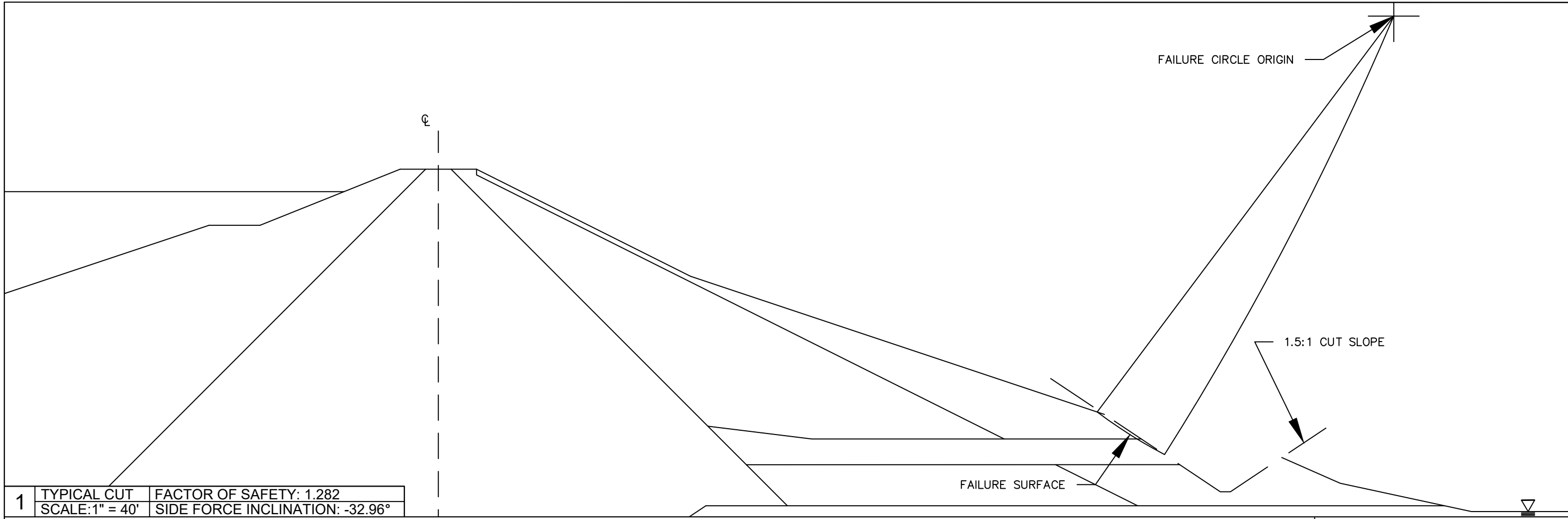
CONSTRUCTION NOTES:

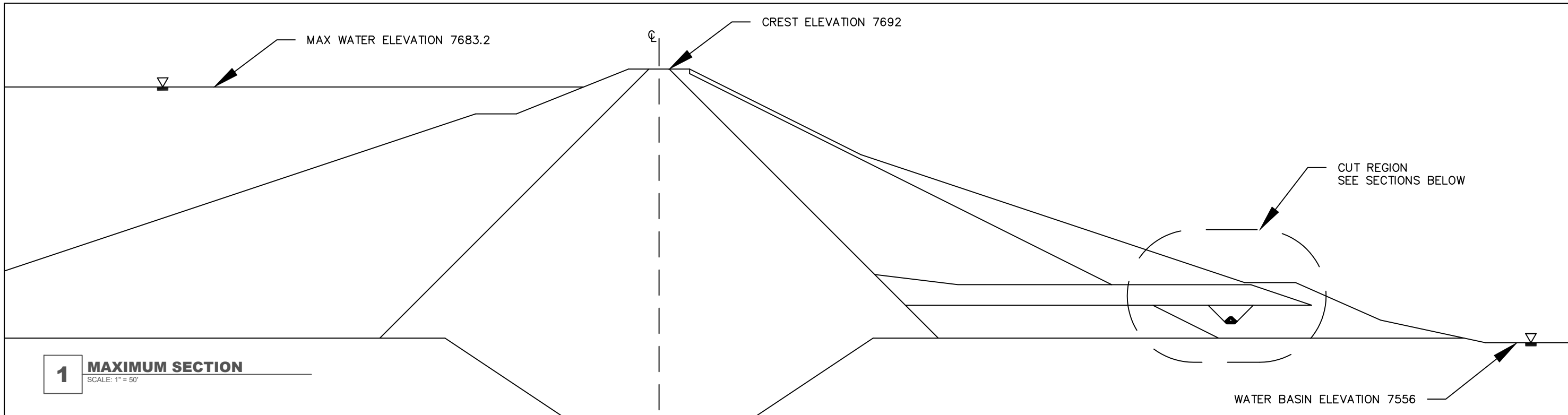
- 1) EXTENDED EXCAVATION LOCATIONS ARE TO BE DETERMINED BY BLANKET DRAIN CONNECTIVITY STUDY. SEE SHEET C-07 FOR DETAILS ON EXTENDED EXCAVATION.
- 2) STATIONING IS INDEPENDENT OF DAM CREST STATIONING AND APPLIES ONLY TO THE PROPOSED PIPE.
- 3) PIPE LOCATION IS APPROXIMATE AND IS RELATIVE TO THE EXISTING PIPE.
- 4) SEE SHEETS C-08 FOR SECTION VIEWS AND VOLUME QUANTITIES OF EXCAVATION.

**2**

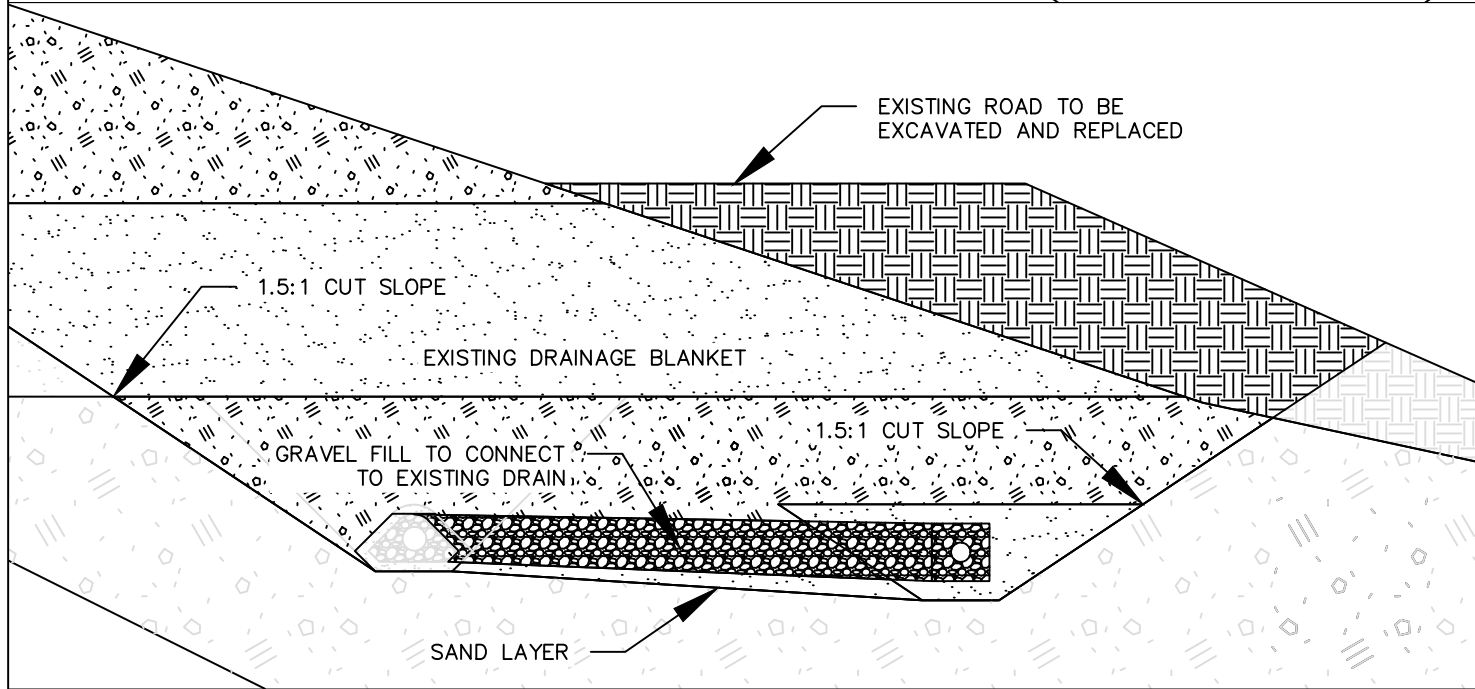
**PROPOSED PIPE PROFILE**

SCALE: 1" = 100'

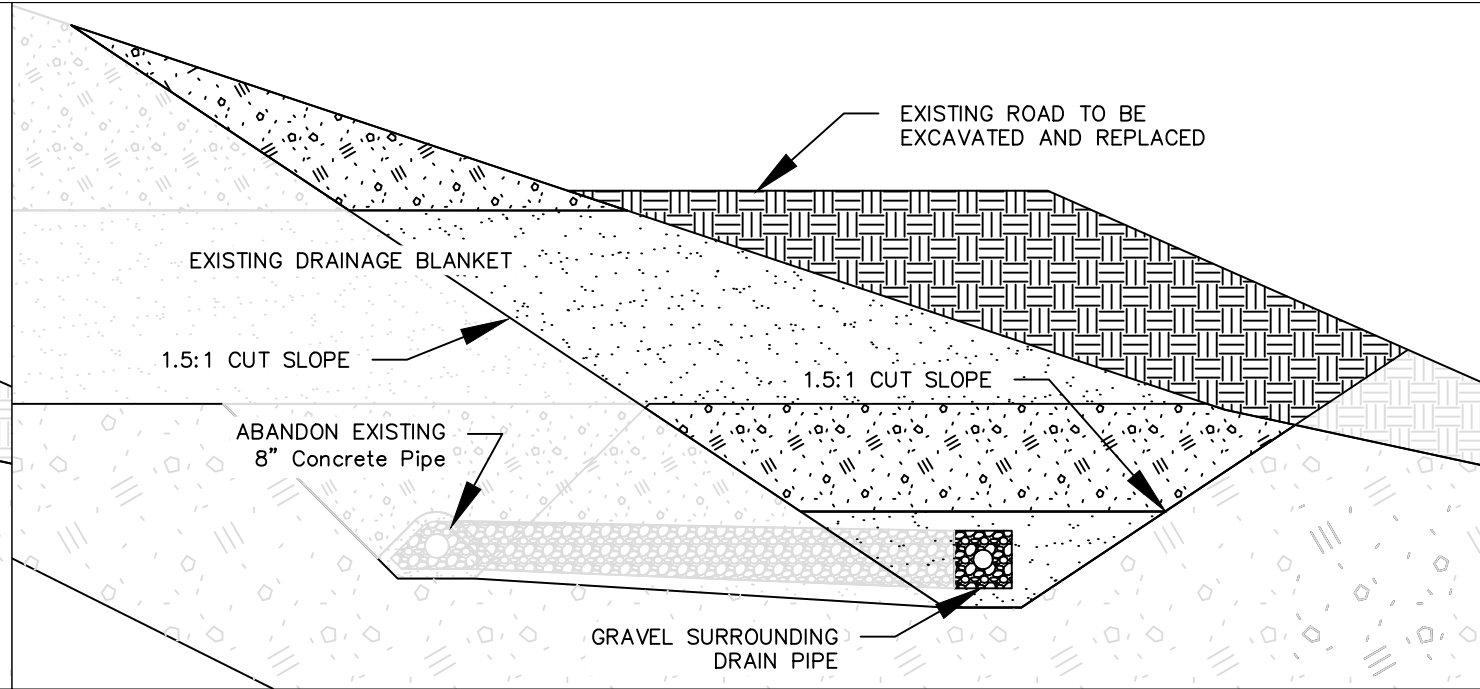




**1** MAXIMUM SECTION  
SCALE: 1" = 50'



**2** BLANKET CONNECTIVITY  
SCALE: 1" = 10'

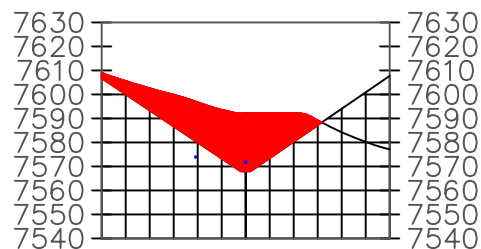


**3** TYPICAL TOE DRAIN  
SCALE: 1" = 10'

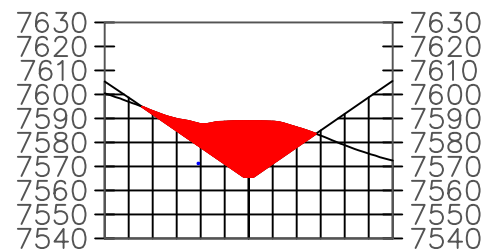
**CONSTRUCTION NOTES:**

- 1) BLANKET DRAIN CONNECTIVITY SHOWN IN VIEW 2 WILL ONLY BE USED AT SELECTED PREDETERMINED LOCATIONS, TYPICAL TOE DRAIN INSTALLATION WILL FOLLOW VIEW 3.
- 2) CROSS-SECTION BASED OFF OF APPROXIMATE MAXIMUM SECTION AT STATION 11+00, BLANKET DRAIN AND PIPE LOCATION MAY VARY.
- 3) EXISTING PIPE DEPTH IS APPROXIMATE BASED ON ORIGINAL DRAWING AND IS ANTICIPATED TO VARY.
- 4) NEW TOE DRAIN PIPE TO HAVE 1.5% MIN. SLOPE
- 5) INSTALLATION OF GRAVEL AROUND THE NEW TOE DRAIN PIPE WILL BE DONE WITH A "DOGHOUSE" TO ENSURE SAND IS NOT PLACED AROUND THE PIPE.
- 6) GRAVEL AND SAND ENVELOPES WILL BE NEW MATERIAL. ALL OTHER EXCAVATED MATERIAL SHALL BE STOCK-PILED AND REPLACED IN PREVIOUS CONDITION AND LOCATION.

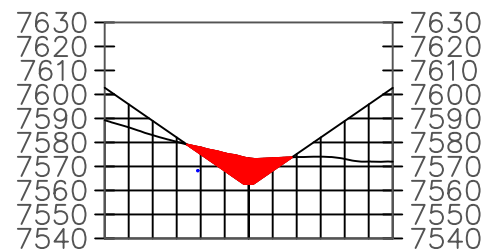
0+21.42



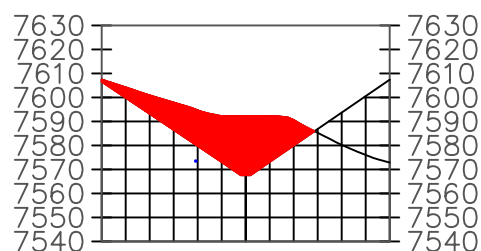
2+00.00



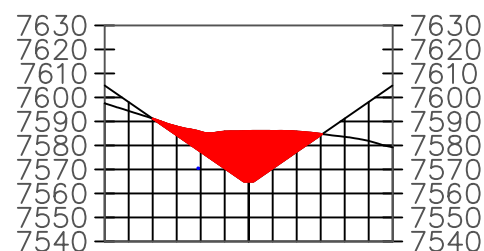
4+00.00



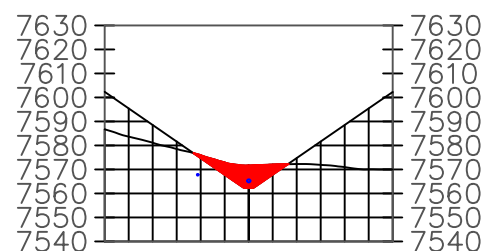
0+50.00



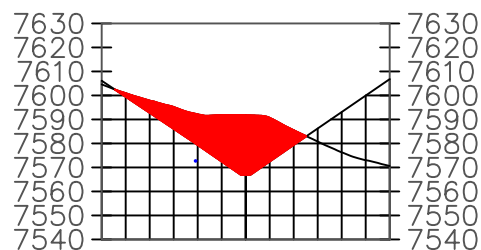
2+50.00



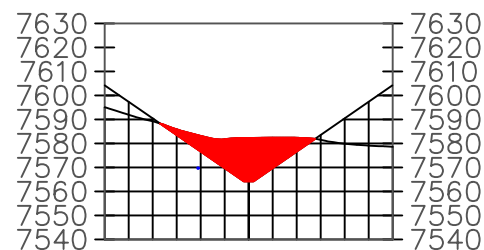
4+29.65



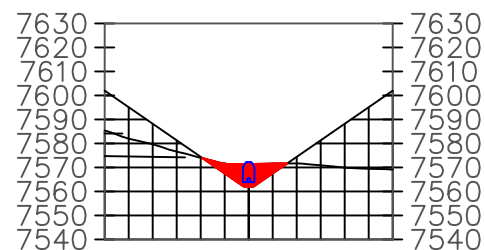
1+00.00



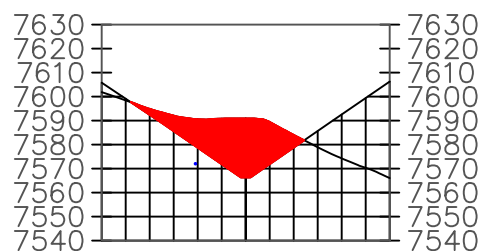
3+00.00



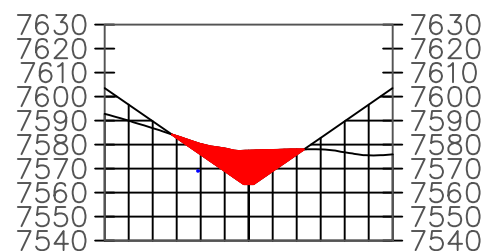
4+51.42



1+50.00



3+50.00



Material Table

| Station | Area    | Volume  | Cumulative Volume |
|---------|---------|---------|-------------------|
| 0+21.42 | 1230.24 | 0.00    | 0.00              |
| 0+50.00 | 1126.66 | 1247.30 | 1247.30           |
| 1+00.00 | 959.56  | 1931.68 | 3178.98           |
| 1+50.00 | 865.13  | 1689.53 | 4868.50           |
| 2+00.00 | 822.91  | 1563.01 | 6431.51           |
| 2+50.00 | 732.18  | 1439.90 | 7871.41           |
| 3+00.00 | 572.14  | 1207.71 | 9079.12           |
| 3+50.00 | 387.99  | 889.02  | 9968.13           |
| 4+00.00 | 255.64  | 595.96  | 10564.10          |
| 4+29.65 | 194.48  | 247.14  | 10811.23          |
| 4+51.42 | 177.04  | 149.79  | 10961.02          |

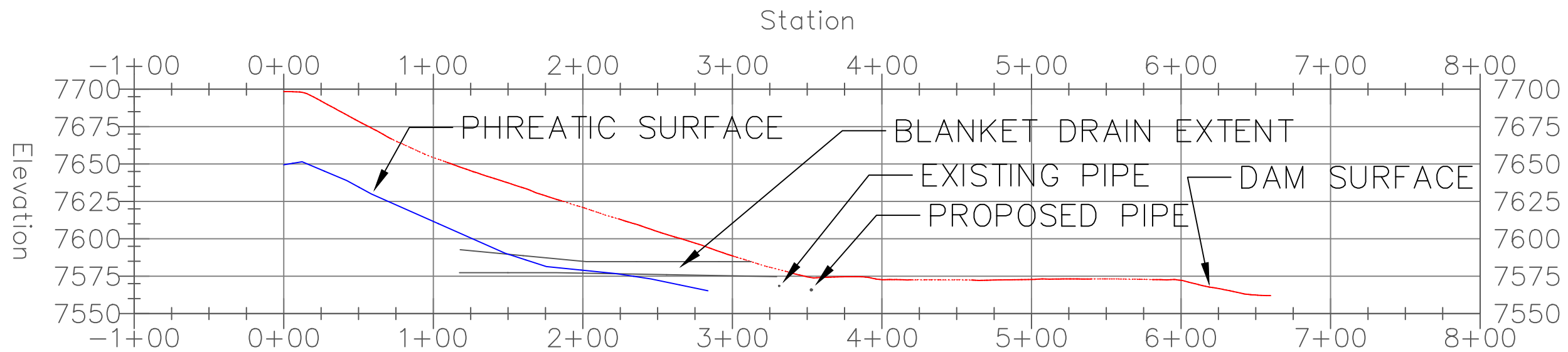


**1**

**DAM STATION PLAN VIEW**

SCALE: 1" = 100'

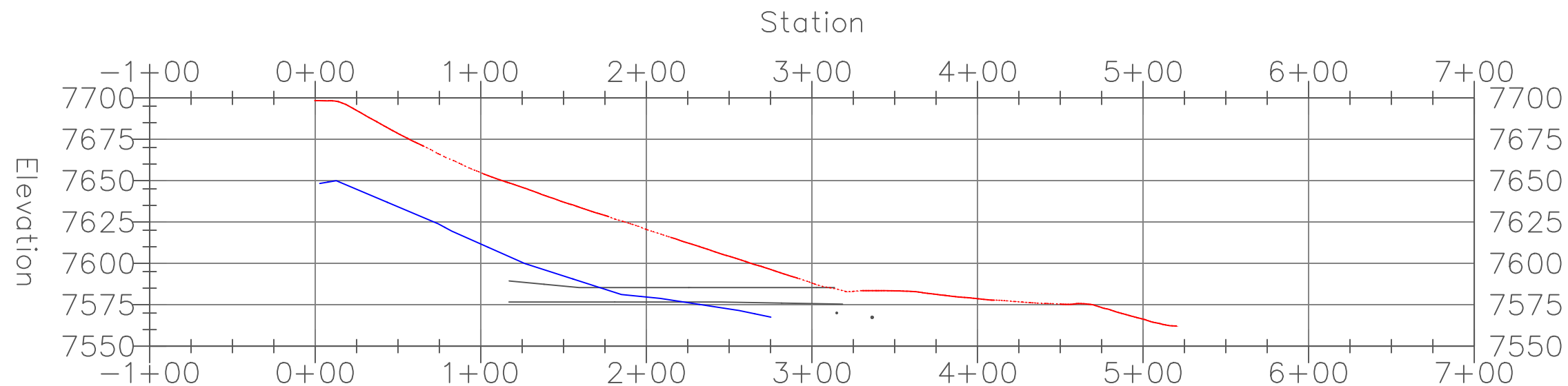
Off Dam Station 14+00 PROFILE



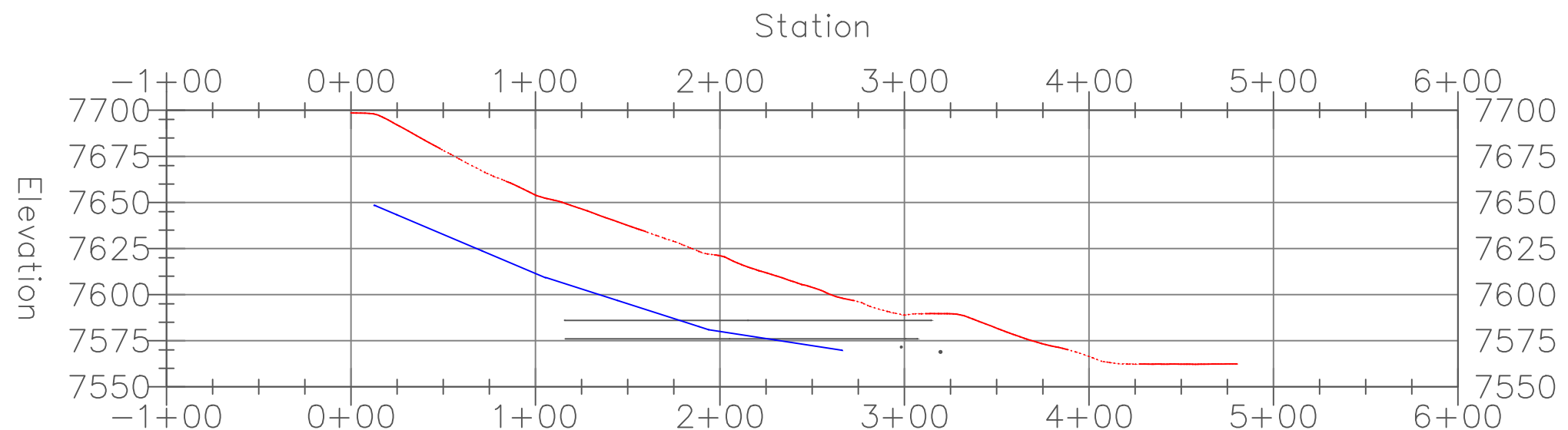
NOTES:

- 1) ALL STATIONS ARE BASED OFF OF ORIGINAL DAM CREST STATIONING FROM ORIGINAL CONSTRUCTION PLANS
- 2) ALL LABELS ABOVE APPLY TO ALL DAM CROSS SECTIONS

### Off Dam Station 13+00 PROFILE



### Off Dam Station 12+00 PROFILE



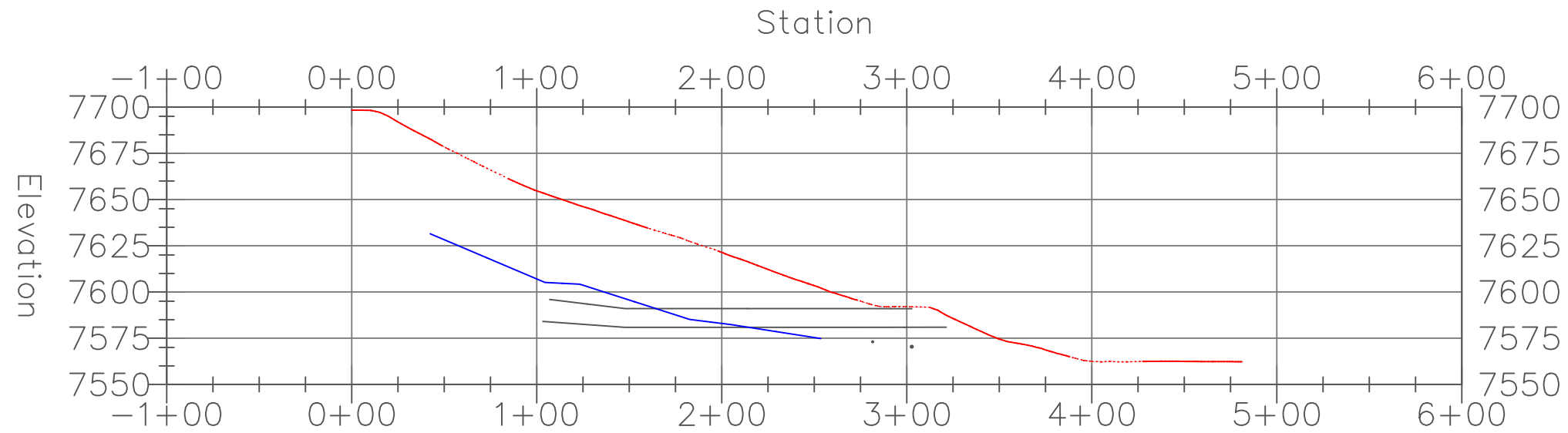
**NOTES:**

1) ALL STATIONS ARE BASED OFF OF ORIGINAL DAM CREST STATIONING FROM ORIGINAL CONSTRUCTION PLANS

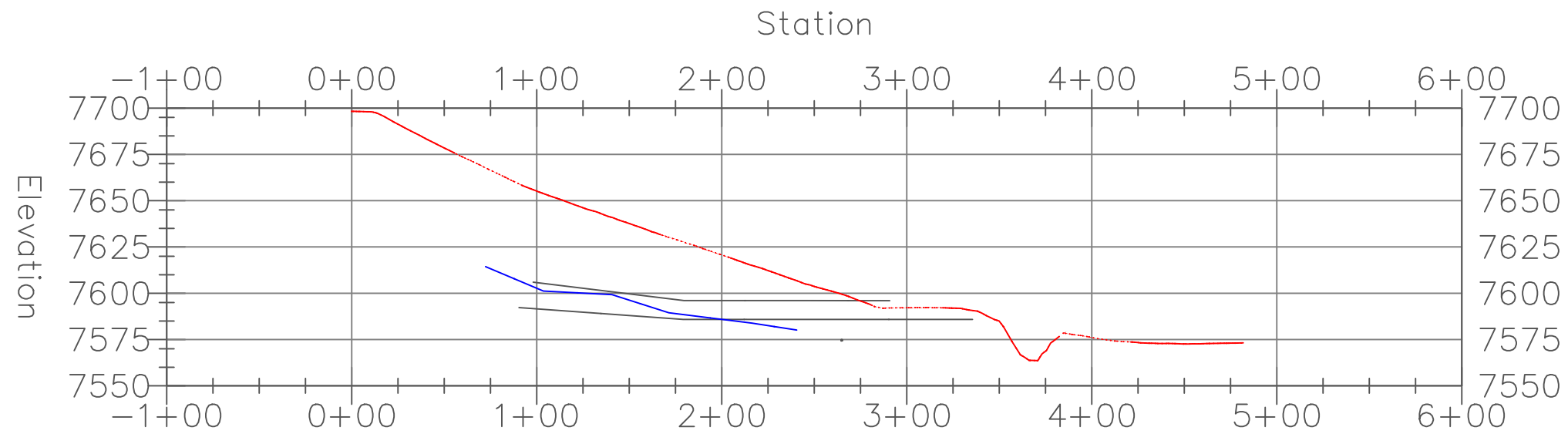
BYU CE CAPSTONE TEAM 10  
BRAXTON PORTER  
KAMDEN PETERSON  
MATT EVANS  
ALEX MASKOVICH

CURRENT CREEK DAM TOE REHABILITATION  
DAM CROSS SECTIONS STATION 12 & 13

# Off Dam Station 11+00 PROFILE



# Off Dam Station 10+00 PROFILE



**NOTES:**

1) ALL STATIONS ARE BASED OFF OF ORIGINAL DAM CREST STATIONING FROM ORIGINAL CONSTRUCTION PLANS

**Appendix D – Three Alternatives Report**

# THREE ALTERNATIVES MEETING REPORT

Prepared by:

**BYU Capstone Team 10 (Matthew Evans, Alex Maskovich, Kamden Peterson, and Braxton Porter)**

---

This document summarizes the research and calculations that the BYU capstone team 10 performed in preparation for the meeting with CUWCD and USBR personal on 2/15/2023. It proposes three alternatives for the Currant Creek Dam toe drain replacement project.

DISCLAIMER: No details of this report are approved by a licensed PE. All calculations are estimates and not to be used for design or construction.

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## KEY POINTS

- Linear extrapolation of piezometer data shows piping becoming an issue in 70 years. Further analysis of gradient increase within the dam could be done, likely showing a sooner piping date.
- Current drainage conditions indicate that an in-place toe drain replacement might not fix the drainage issue. Trenching in the current location could also result in depths over 26 feet deep.
- Relocating the toe drain pipe could result in less excavation if done down stream of the original toe drain. It could also allow improvement of the blanket drain and toe drain connectivity.
- Slope stability and flow nets with finite element models could be calculated to determine safety and feasibility of different excavation scenarios.
- Several trenchless remediation methods were considered but discarded due to complexity of blanket drain and toe drain materials and connectivity.

**1) DO NOTHING**

---

DESCRIPTION

The “Do Nothing” alternative consists of leaving the toe drain as is, without repair. It would by necessity include continual monitoring of the piezometers and flow rate from the toe drain.

---

ANALYSIS

The attached document titled “Projection of Phreatic Surface Depths at Currant Creek Dam” projects the rise in the phreatic surface if the measured trends at each piezometer were to continue. This is assuming that if the phreatic surface were to reach the dam face, piping could occur and cause failure in the dam. Of the piezometers analyzed, the phreatic surface at PZ-24 could reach the dam surface first around year 2089 if the water continued rising as measured over the last 20+ years. Other piezometer readings would reach the embankment surface between the years 2150-2300. These are only linear extrapolations and therefore give only simple estimates suggesting possible projections.

Many of the piezometers that are increasing in depth are upstream while the piezometers closer to the toe of the dam are more constant. This suggests there is some drainage occurring, but if those trends continued, the gradient through the dam could increase resulting in faster flow through the dam and higher likelihood of piping. No analysis was performed on gradient changes and likelihood to cause piping. If further analysis were desired, flow nets could be created within the dam using the finite element method to better estimate the gradient changes.

As shown in Figure 1, the blanket drain is inundated at station 15+00 on the dam (at the spillway). This could explain why PZ-24 would be the first to reach embankment surface. This is likely where the gradient would increase as well.

Station 15+00

High Res (Active Conservation) 7678  
Reservoir Elevations 7676.3-7676.56

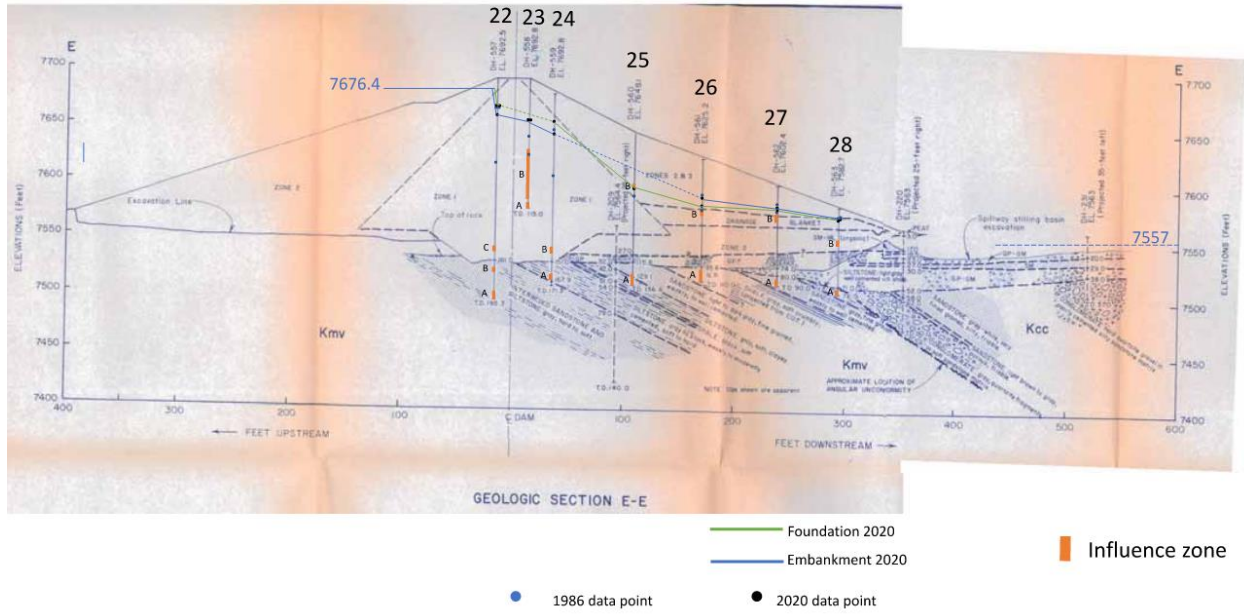


Figure 1: Phreatic surface at station 15+00

## RECOMMENDATION/CONSEQUENCES

As shown in the analysis, using linear extrapolation, piping could become a concern in approximately 70 years. More analysis using flow nets could be done which would give a better indication for piping before this time.

It appears that there are some drainage problems due to the rise in piezometers and inundation at station 15+00, and it is not recommended that the toe drain be left without remediation.

### 2) REPLACE TOE DRAIN IN ORIGINAL LOCATION

## DESCRIPTION

This alternative would consist of trenching along the location of the toe drain, removing the old toe drain, and replacing the drain with a new full-walled HDPE pipe with slots rather than holes.

## ANALYSIS

In considering this alternative, the current conditions of the toe drain are detailed below:

As shown in Figure 2, there has been no measured flow in the right toe drainpipe for the last 20 years. This could indicate a connectivity problem with the blanket drain, one that doesn't appear to be a problem on the left toe drain where flow is constantly measured. If the problem is connectivity with the blanket drain, replacing the pipe in place would not fix the problem.

Further indication of a problem with drainage is that of plant and moss growth on the right side of the dam face. See Figure 3 and Figure 4. This further indicates there is less drainage than on the left side of the dam.

A video inspection of the pipe taken in 2017 shows problems with the toe drain itself. There are several offset pipe joints in the pipe. After the jetter cleaned the pipe, water could be seen draining through those offset joints indicating the pipe might not be able to carry flow without it draining out. Further, there is significant pooling from 60 feet from the manhole to 105 feet (see Figure 6). This is a large dip in the pipe which could further prevent drainage. Then, at 170 feet there is a huge blockage shown in Figure 5 & Figure 7.

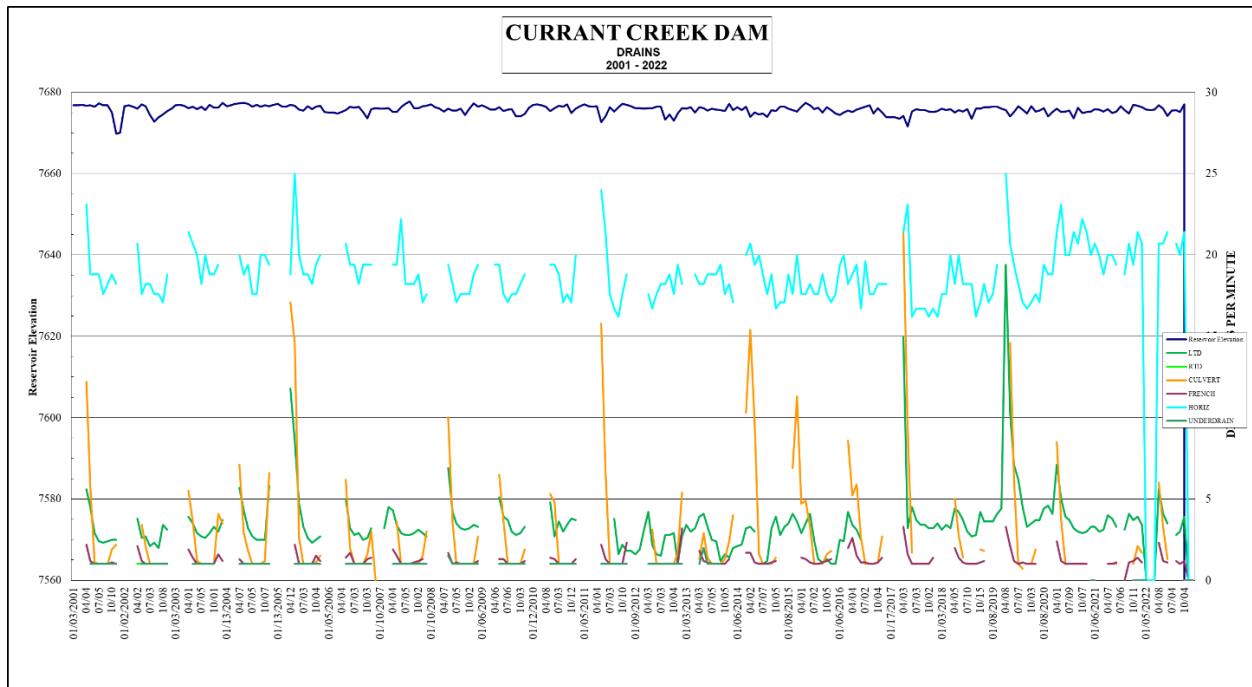


Figure 2: Measured Flow Rate in Drains (Taken from CUWCD's data)



**Figure 3: Significant growth on right side of dam face**



**Figure 4: Little plant growth on the left face of the dam**



Figure 5: Large Blockage around 170 feet

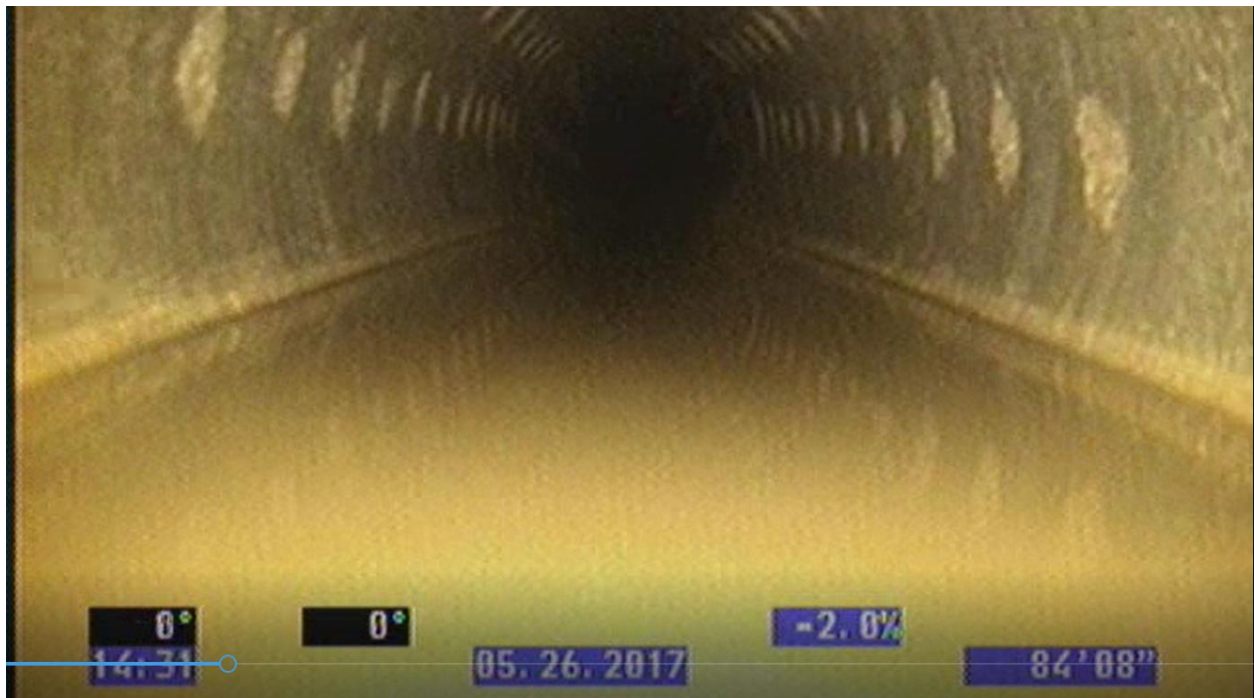


Figure 6: Standing water in large dip around 80 feet



Figure 7: Zoomed in image showing further blockage behind large blockage at 170 feet

In attempting to visualize where the toe drain is located, the BYU capstone team created a model in Civil 3D. Surface LiDAR data was taken from the UGRC Raster Data Discovery website ([raster.utah.gov](http://raster.utah.gov)). The blue pipe is the approximate location of the penstock shown to compare against the toe drain elevation. The horizontal surface is the top of the 10' thick blanket drain. Both elevations were estimated from the plans and adapted to the LiDAR elevations.

The slope of the toe drain was pulled from the 5-26-17 video inspection. Instantaneous slope values were recorded approximately every 2.5 to 3 feet. If rocks or other debris caused irregular jostling or positioning of the camera, the slope was not recorded. An average slope of 1.4 % was found within the first 170 feet. This slope is very approximate and a different viewer/different method for finding the average slope could yield different results. It does show that the pipe is likely in the recommended range of 1.5-2% slope.

Figure 8 & Figure 9 show profile views of station 12+00 and along the toe drain location to show possible depths of the pipe. If accurate this indicates the pipe is more than 26 feet deep at some locations.



---

## RECOMMENDATION/CONSEQUENCES

All the info in the Analysis section summarizes that there are some issues with fixing the pipe in place. First, the pipe could be over 26 feet deep at some parts and trenching would be difficult. Possible trenching methods could include putting in a sheet pile wall on the upstream side or sloping and bracing the trench. Second, with drainage conditions not ideal and possible connectivity problems with the blanket drain, it is not certain that simply replacing the pipe in place would yield results.

However, excavating to the toe drain would allow observation of the blanket drain and would allow for more analysis of the connectivity. Also, fixing the slope of the pipe and removing the blocked section could also yield results.

### **3) COMPLETE RELOCATION OF TOE DRAIN WITH PARTIAL BLANKET DRAIN REBUILD**

---

## DESCRIPTION

This alternative would consist of abandoning the existing toe drain and adding a new toe drain, possibly extending the existing blanket drain pending on the new toe drain's location.

---

## ANALYSIS

As described in the analysis section of alternative 2, there could be some connectivity issues with the current toe drain location. Further, excavating to the toe drain could be costly and difficult due to depths up to 26 feet. It could also be located underneath the penstock as mentioned in alternative 2, further complicating its excavation.

---

## RECOMMENDATION/CONSEQUENCES

To bypass the excavation presented in alternative 2, the existing toe drain could be abandoned. A new blanket and toe drain would be installed in a location that requires less excavation of the face of the dam downstream of the original toe drain (It is also possible to put the toe drain in the existing blanket drain if it extended further).

Execution further down the face of the dam, more in the service road, would result in less slope stability needed compared to alternative 2. However, only more detailed slope stability and gradient calculations would determine the method of slope stability and required reservoir elevation.

In our research we have found an example of this being done to Lake Whetstone Dam, located in Maryland. The pipe for the toe drain was completely abandoned and grouted in. The surface of the dam was excavated and stabilized with soil nails, to provide access for the installation of the new toe drain. The new blanket and toe drain were located close to the surface of the dam, so excavation was minimal compared to replacing the original toe drain. See Figure 10 and Figure 11 for locations of the new blanket and toe drain. See the attached document titled "Lake Whetstone Toe Drain Replacement" for more details.

# Toe Drain Pipe Location

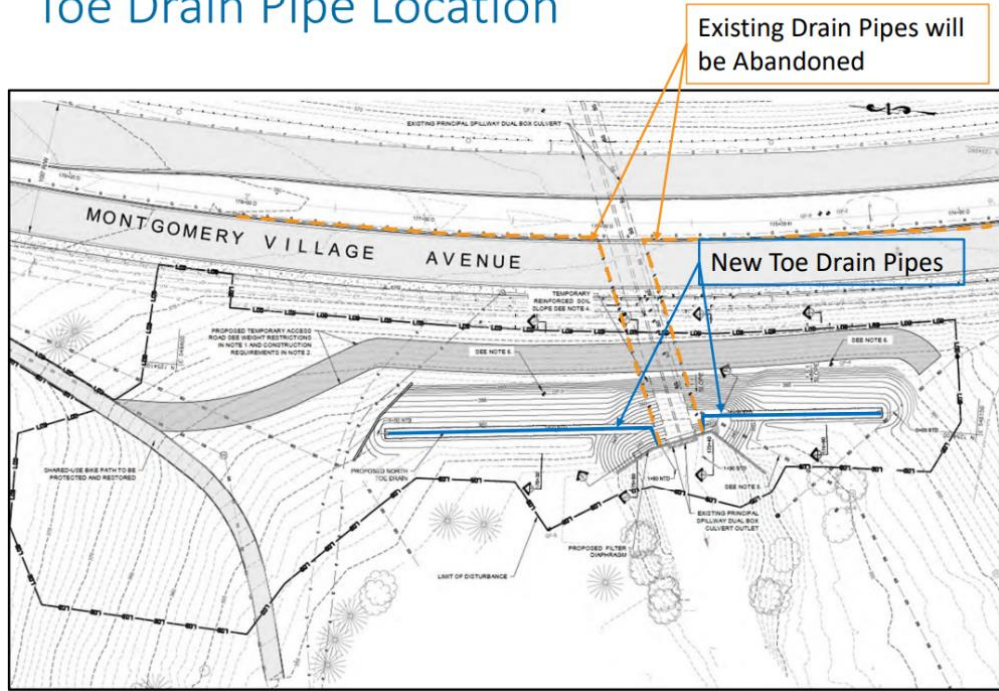


Figure 10. Lake Whetstone Toe Drain location

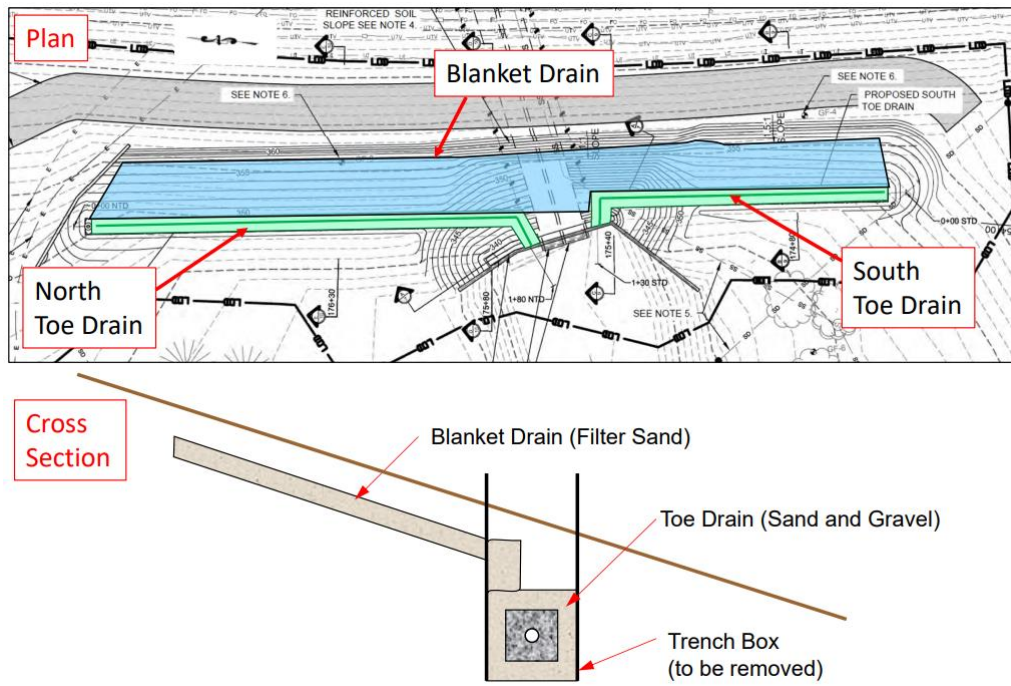


Figure 11. Lake Whetstone Blanket Drain plan view and cross section

## OTHER ALTERNATIVES

The following alternatives were researched early on but were rejected from further analysis for various reasons. They are here for reference only to indicate that they were considered.

- Boring
  - The team reached out to several boring companies for quotes and received no reply with details. In consultation with some engineers at Bowen and Collins, they explained that boring through blanket drain and toe drain material is difficult and easy to damage the connectivity.
- Other trenchless technology (pipe bursting)
  - The team talked with Randy Lee with Restored Pipe Systems about pipe bursting. Pipe bursting would require excavation every 100 feet. This requires a lot of excavation and might not fix the connectivity problem as explained in alternative 2.
- HDPE Insertion into the existing pipe
  - We could put a smaller HDPE into the existing pipe: See “ASCE Standard Guide for Insertion of Flexible Polyethylene Pipe Into Existing Sewers” and “Black Lake Dam Toe Drain Rehabilitation Case Study” documents. It has successfully been done before. However, with the large blockage at 170 feet, excavation might still be needed, and drainage issues might not be fixed as shown in alternative 2.

## RECOMMENDATIONS FOR PROJECT

The following recommendations are universal regardless of the alternative chosen and are to be used in developing the preferred alternative.

- Use final checklist found in the “Considerations for Embankment Dam Toe Drain Design” document to guide the final design recommendations. Guidelines for correct size, material and other maintenance aspects are considered.
- The “Guidelines for Drilling in and Near Embankment Dams and Their Foundations” document recommends having plugging material on hand to stop erosion and piping if it were to begin during the construction process.

## RELEVANT ATTACHED DOCUMENTS

The following documents are referenced in this report.

- Black Lake Dam Toe Drain Rehabilitation Case Study
  - Case study where HDPE pipe was inserted into an existing concrete toe drain pipe as remediation method.
- ASCE Standard Guide for Insertion of Flexible Polyethylene Pipe Into Existing Sewers
  - The ASCE standard and method for the Black Lake Dam Toe Drain Rehabilitation
- Guidelines for Drilling in and Near Embankment Dams and Their Foundations

- Division of Dam Safety and Inspections guidelines for drilling near embankment dams. Has recommendations for when doing work at a dam.
- Lake Whetstone Toe Drain Replacement
  - A case study presentation showing an example of complete toe drain relocation further downstream.
- Projection of Phreatic Surface Depths at Currant Creek Dam
  - A report prepared by the BYU capstone team showing linear projections of piezometer levels in the dam.
- Considerations for Embankment Dam Toe Drain Design
  - A report summarizing important considerations in toe drain design especially for maintenance perspective.

# REHABILITATION OF A TOE DRAIN

by

Robert L. Dewey<sup>1</sup>

## Abstract

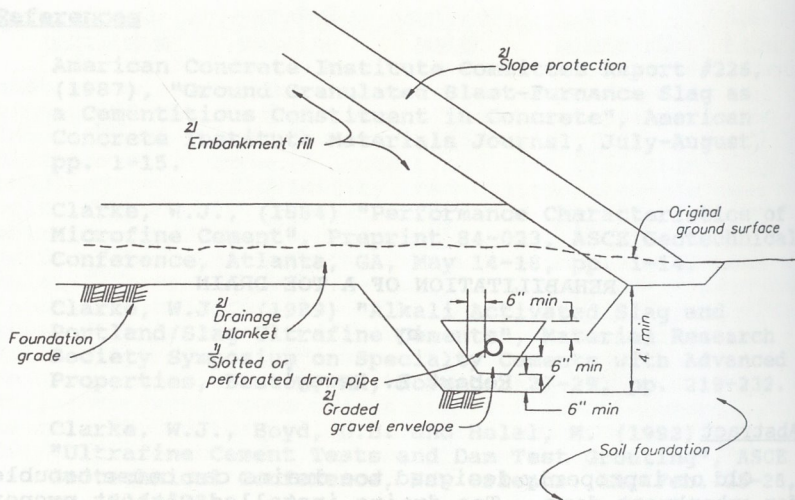
Old or improperly designed toe drains can cause trouble for embankment dams. Toe drains installed without proper filtering can provide an easy course for internal erosion and piping of foundation and/or embankment soil materials. Depending on location, a toe drain can be very difficult to access or unwise to abandon. The best solution for some systems is to attempt inplace rehabilitation. Other methods not as desirable as inplace rehabilitation are also presented to solve a toe drain problem. A rehabilitation is expected to eliminate the problem of piping foundation materials through the drain, yet maintain the capability of the toe drain to drain water and relieve foundation pore water pressures.

## Introduction

Embankment dam toe drains are commonly installed along the downstream toe of an embankment in conjunction with horizontal drainage blankets to relieve pore water pressures and to collect seepage discharging from the dam or foundation. The water collected is conveyed to an outfall pipe that discharges either into the spillway or outlet works stilling basin or into the river channel below the dam. Toe drains in old dams have been constructed in many different ways with many different materials. A typical system usually combines a perforated pipe surrounded by a filter zone of natural material (see figure 1). Corrugated or non-corrugated, plastic, metal, concrete or clay, perforated or non-perforated, jointed or continuous are some of the kinds of drain pipes found in

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1) There are many suitable drain pipes on the market. The requirements are adequate durability and strength. Pipe laid with open joints should not be used. Slots or perforations should meet design criteria.

2) Should meet filter criteria with adjacent material.

FIGURE 1. - Typical Toe Drain Installation

old dams. Sometimes no pipe was used and only open work gravel and cobbles were used for a toe drain. Filter zones around the pipe may or may not exist in old dams. A filter zone of natural material must satisfy filter criteria for the surrounding soil and must also be compatible with perforations of the drain pipe. Geosynthetics are now being used more often in toe drains as part of the filter.

Collapsed pipes and clogged filters are not uncommon in old dams and the complete lack of a toe drain is also not unusual. Old drainage systems were often built far beneath the dam without regard to access should maintenance repairs be necessary. Simply locating a toe drain without the benefit of any records of construction can be very difficult, if not impossible.

Toe drains that do not function properly can be a real threat to the safety of a dam. Many things can go wrong with toe drains. Flow through the drain can be impeded as a result of embankment settlement, drain section offset,

clogging, root penetration, or complete collapse of the pipe. Drains can hasten internal erosion (piping) if the surrounding filter is not properly designed or constructed, or if drain perforations are not properly sized. Slope failures, depressions, cavities, or boggy areas can result from an improperly functioning toe drain.

#### Dam Safety Modifications

Most often an improperly functioning drain is simply dug up and replaced. However, removal of a drain placed far beneath the dam may not be prudent, especially if excavation would remove a significant portion of the dam. Rehabilitation involves placement of a properly designed perforated drain pipe, along with replacement of a surrounding filter zone.

For poorly accessible drains, grouting them in place and constructing a new drain at the toe of the dam may be considered. The effects of this type of modification on groundwater flow, phreatic surface, and pore pressures in the downstream portion of the embankment and downstream foundation must be carefully evaluated. Changes in these conditions are almost certain.

Poorly accessible drains that need to be modified but cannot be grouted can often be rehabilitated in place. If the drain pipe is open or can be cleaned out, a smaller drain pipe can be inserted within the existing pipe. The American Society for Testing and Materials standard designation: F-585 describes such a procedure. This type of modification has been accomplished for the toe drain rehabilitation at Black Lake Dam on the Flathead Indian Reservation, Montana.

#### Case History - Black Lake Dam Toe Drain Rehabilitation

Evidence of material piping through the toe drain system at Black Lake Dam was found at the downstream end of the toe drain outfall pipe. Fine sand that is conspicuously absent on the ground anywhere else on the dam site accumulated at the outlet of the toe drain system. Pockets of fine sand naturally exist in the dam's right abutment which is mostly composed of openwork talus. High velocity seepage from the reservoir through the talus and right abutment of the dam carried the sand to the dam's drainage system. Continued internal erosion of the sand pockets could have resulted in the formation of large cavities, collapse, and breaching through the right abutment or right end of the dam.

The toe drain at Black Lake Dam was originally constructed with 4-foot long, 8-inch inside diameter solid concrete pipe sections placed so that about a 3/4-inch separation at the top of each joint would allow seepage to enter the drain. To limit transport of soil material into the open-jointed drain, tar paper was originally placed over each joint and a gravel envelope was placed around the pipe.

The drain is aligned parallel to dam centerline and located about halfway between the dam's downstream toe and the seepage cutoff trench, about 20 feet below original ground surface (see figure 2). When the dam was originally constructed, the drain was able to be placed in its current location because it was constructed prior to placement of the embankment fill. It effectively controls the phreatic line through the embankment, keeping water away from the dam's downstream face. It is located adjacent to an impervious blanket which directs flow passing by the seepage cutoff to the drain. Therefore, the location of the dam is ideally suited to drain any water that would reach the downstream portion of the dam and the foundation immediately below the dam; however, the drain pipe openings are too wide and have allowed soil to enter the pipe. Since the drain was a conduit which carried piped embankment and foundation materials, it was a threat to the safety of the dam and required rehabilitation. Because the drain was located so far under the dam, the rehabilitation was performed in place.

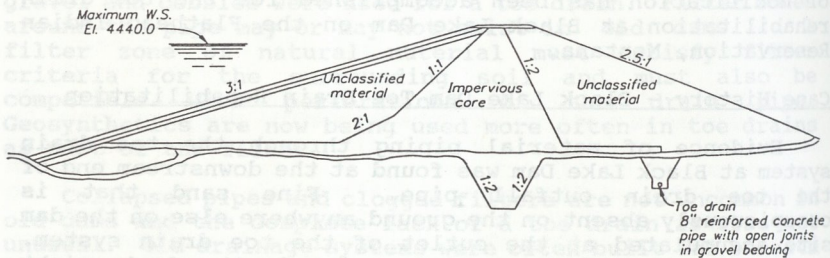


FIGURE 2. - Typical Cross Section of Black Lake Dam

The two basic functions of the Black Lake Dam toe drain rehabilitation were to stop piping and to maintain drainage. Piping was stopped by screening of the piped material through flexible perforated pipes inserted into the original drain. Adequate drainage was maintained through the perforations of the insert drain pipes.

Rehabilitation of the open-jointed toe drain of Black Lake Dam was implemented as a two-part process. For part 1, a 5-inch nominal diameter flexible perforated drain pipe was inserted into the larger, 8-inch inside diameter open-jointed drain pipe. The smaller insert drain pipe was designed with openings appropriately sized to allow for drainage, yet mitigate piping. Two excavations were made to access the upper ends of the open-jointed toe drain which was then cleaned and flushed. Two insert drain pipes were installed, one from each end of the exposed toe drain, so that space was left between the two insert drain pipes at the "tee" with the outfall pipe (see figure 3). (The outfall pipe is an 8-inch inside diameter nonperforated concrete pipe leading from the toe drain to a downstream discharge point to allow for release of water from the drain system). A flexible "formation packer" was affixed

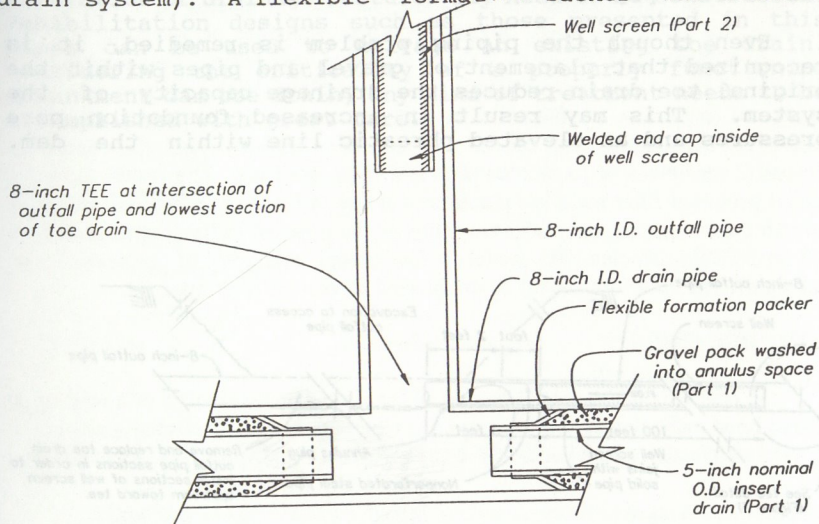


Figure 3. - Tee Detail

NO SCALE

at the end of each insert drain pipe near the tee with the outfall pipe to block the annulus between the insert drain and concrete pipe and direct flow from around the outside of the insert drain pipe through its perforations. Soil material that would have piped through the original toe drain is stopped by the restricting slot sizes of the insert drain pipe.

Rounded pea gravel was washed into the annulus space between the inside of the 8-inch concrete toe drain pipe and the outside of the 5-inch insert drain pipe. This gravel pack occupies a volume that would otherwise have filled with piped material. The gravel also enhances filtration capability.

Part 2 of the toe drain rehabilitation consisted of an additional means of eliminating piping of material through the drain system. A continuously slotted, steel well screen was inserted into the outfall pipe beneath the dam near the tee of the drain system (see figure 4). Plugs were installed so that flow in the toe drain system is forced to go through the well screen before traveling down the outfall pipe.

Even though the piping problem is remedied, it is recognized that placement of gravel and pipes within the original toe drain reduces the drainage capacity of the system. This may result in increased foundation pore pressures and an elevated phreatic line within the dam.

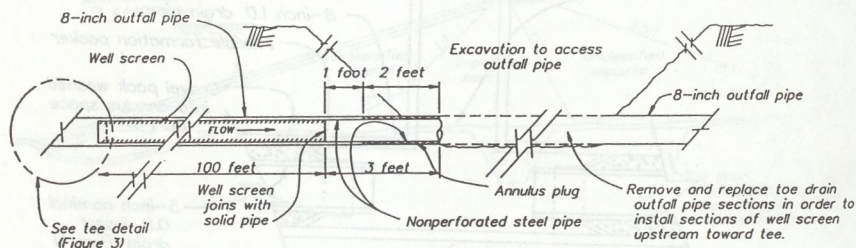


Figure 4. - Outfall Pipe Rehabilitation

(PIPE WALL THICKNESS NOT SHOWN)  
NO SCALE

Therefore, provisions have been made to construct a Stage II modification to Black Lake Dam should these consequences occur. Stage II involves construction of a drainage and filter system under a seepage berm over the downstream toe of the dam. It was decided to accomplish the Black Lake Dam modifications in stages because there is reasonable confidence that the Stage I toe drain rehabilitation will provide adequate drainage and may be all that is necessary to control seepage and to stop piping.

### Conclusion

It is sometimes better to rehabilitate a dam's downstream toe drain than to abandon or replace it. Abandonment by grouting in place could be a type of modification with serious negative side effects. Rehabilitation can be achieved to stop internal erosion while still maintaining the capability of the drain to control groundwater flow and relieve excess pressures. Inplace rehabilitation usually results in reduced drainage capacity, and secondary modifications should be considered to supplement drainage through the toe drain. Additional filtration or drainage features may need to be constructed. Rehabilitation designs such as those presented in this paper can be used to salvage an existing toe drain. Considering the criticality of a properly functioning embankment dam toe drain, any kind of treatment needs to be accomplished with great care.



## Designation: F585 – 16 (Reapproved 2021)

# Standard Guide for Insertion of Flexible Polyethylene Pipe Into Existing Sewers<sup>1</sup>

This standard is issued under the fixed designation F585; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon ( $\epsilon$ ) indicates an editorial change since the last revision or reapproval.

*This standard has been approved for use by agencies of the U.S. Department of Defense.*

## 1. Scope\*

1.1 This guide describes design and selection considerations and installation procedures for the rehabilitation of sanitary and storm sewers by the insertion of solid wall or profile wall or corrugated polyethylene pipe into an existing pipe and along its existing line and grade. The procedures in this guide are intended to minimize traffic disruption, surface damage, surface restoration and interruption of service.

1.2 The polyethylene piping product manufacturer should be consulted to determine the polyethylene piping product's suitability for insertion renewal as described in this guide.

1.3 The values stated in inch-pound units are to be regarded as standard. The values given in parentheses are mathematical conversions to SI units that are provided for information only and are not considered standard.

1.4 *This standard does not purport to address all of the safety concerns, if any, associated with its use. It is the responsibility of the user of this standard to establish appropriate safety, health, and environmental practices and determine the applicability of regulatory limitations prior to use. See 6.1, 7.1, and 8.1 for additional safety precautions.*

1.5 *This international standard was developed in accordance with internationally recognized principles on standardization established in the Decision on Principles for the Development of International Standards, Guides and Recommendations issued by the World Trade Organization Technical Barriers to Trade (TBT) Committee.*

## 2. Referenced Documents

### 2.1 ASTM Standards:<sup>2</sup>

[F412 Terminology Relating to Plastic Piping Systems](#)

[F1417 Practice for Installation Acceptance of Plastic Non-pressure Sewer Lines Using Low-Pressure Air](#)

[F1804 Practice for Determining Allowable Tensile Load for Polyethylene \(PE\) Gas Pipe During Pull-In Installation](#)

[F2620 Practice for Heat Fusion Joining of Polyethylene Pipe and Fittings](#)

2.2 *Other Documents:*<sup>3</sup>

[PPI Material Handling Guide](#)

## 3. Terminology

3.1 *Definitions*—Definitions are in accordance with Terminology [F412](#), unless otherwise specified.

## 4. Significance and Use

4.1 The procedures described in this guide are intended as a design and review aid for use by the design engineer in conjunction with manufacturer's recommendations for installing a polyethylene pipe using the insertion method.

## 5. Design and Selection Considerations

### 5.1 General Guidelines:

5.1.1 *Host Pipe Condition Assessment*—Prior to the selection of polyethylene pipe size and installation procedure, measures should be taken to determine in detail the condition of the host (original) sewer piping. A detailed examination should determine if the host sewer piping is structurally sufficient, and that any joint offsets or other host pipe defects will permit polyethylene pipe insertion.

5.1.2 The presence of obstructions should be determined (see [6.3](#)). Protrusions of lateral or service piping into the host sewer pipe, root growths, sedimentation, mineral deposits, or any combination of such obstructions, may require remedial work prior to inserting the polyethylene pipe.

5.1.3 To ensure against interference during insertion, the minimum annular clearance between the polyethylene pipe OD and the host pipe ID should be 10 % of the host pipe ID or 2 in. (50 mm) whichever is less. Greater annular clearance is acceptable. Outside diameter information should be obtained from the polyethylene pipe manufacturer.

5.1.4 The number of insertion excavations should be kept to a minimum and should coincide with areas where problems have been detected in the existing sewer (see [Section 7](#)).

<sup>3</sup> Available from Plastics Pipe Institute (PPI), 105 Decker Court, Suite 825, Irving, TX 75062, <http://www.plasticpipe.org>.

\*A Summary of Changes section appears at the end of this standard

5.1.5 Solid wall or profile wall or corrugated polyethylene pipe may be assembled at the time of insertion using heat fusion in accordance with Practice F2620, integral bell and spigot joints, or mechanical connections as appropriate for the polyethylene piping product.

5.1.6 In all cases, the hydraulic capacity of the pipeline should be assessed by an engineer to insure the reduction in flow area from an inserted pipe or deterioration or deformation, or both, of the host pipe does not adversely impact the hydraulic capacity or flow characteristics of the storm or sanitary sewer.

NOTE 1—It should be noted, in many instances, the pipe being retrofitted is not designed to flow at 100 percent capacity, which may provide additional options for downsizing of the inserted pipe.

5.2 Ancillary materials—Mechanical fasteners, sealants, grouts and other materials that are or are likely to be exposed to sanitary sewage, sewage gases or other corrosive elements of the sanitary sewer environment should be resistant to deleterious effects of the sanitary sewer environment. Sealants, grouts and other materials must be selected with the ability to adequately cure in underwater, sewage or other corrosive environments.

5.3 External Loading—Areas where the host pipe is or may be structurally compromised, or where some or the entire host pipe had been removed will subject the polyethylene pipe to external loads. Information about the resistance of polyethylene pipe (pipe stiffness (PS) or ring stiffness constant (RSC), and buckling resistance) to external hydrostatic and earth loads should be obtained from the pipe manufacturer, and is available in some ASTM polyethylene pipe specifications. Design information about the external load collapse resistance of polyethylene pipe is available in Handbooks and Technical Notes published by the Plastics Pipe Institute.

5.3.1 Hydraulic Loads—When the ground water level may be above the polyethylene pipe, the ground water level and its duration should be estimated by the design engineer, and the polyethylene pipe should be designed to withstand the estimated external hydrostatic pressure without collapsing.

5.3.2 Filling the Annulus—Filling the annulus between the host pipe and the polyethylene pipe with a cementitious grout or other structurally stable material increases the resistance of the polyethylene pipe to external hydrostatic or structural load, and may improve the overall external load capacity of the host pipe. Flexible pipe such as polyethylene pipe relies in part on materials that surround the pipe for external structural load resistance. Flexible pipe ring stiffness and the stiffness of materials surrounding the flexible pipe act together to support external loads. Structurally stable fill materials are materials that remain in place and resist movement from the rise and fall of groundwater around the pipeline. Fill materials may be able to penetrate into cracks and voids in the host pipe, and in combination with the inserted polyethylene pipe provide partial structural rehabilitation. Host pipe condition assessment per 5.1.1 should identify if filling the annulus is needed for structural reasons. Hydraulic load analysis per 5.3.1 should identify if filling the annulus is needed for hydraulic load resistance.

5.3.3 Point Excavation Encasement—At all points where the polyethylene pipe has been exposed, such as at excavations for polyethylene pipe insertion, or for service connections, or excavations at other points where structurally unsound host pipe is removed, the polyethylene pipe, fittings, and service connections should be encased in embedment that provides structural support for the polyethylene pipe. Polyethylene pipe embedment design information is available from organizations such as the Plastics Pipe Institute. If the annulus is not filled (5.3.2), stabilized embedment material should be used to seal the ends of the excavation against embedment migration into the annulus. Stabilized embedment material is at least 6 in. (150 mm) of concrete, or cement-stabilized sand, or other stable high-density material as specified by the design engineer. Preparations for placing of the encasement material include the removal of debris and soil along each side of the host pipe down to the spring line. After the encasement material has been placed and accepted by the design engineer, backfill is placed and compacted to the required finished grade in accordance with the design engineer’s specifications. At service connections, care should be taken to ensure compaction of earth beneath the lateral service pipe in order to reduce subsidence that can cause bending at the lateral connection.

5.3.4 Host Pipe Deterioration—Structural deterioration of the host pipe may continue after the polyethylene pipe has been installed. Uneven or concentrated point loading on the polyethylene pipe or subsidence of the soil above the host pipe may occur if the host pipe collapses or if large parts of the host pipe fall into the annulus between the host pipe and the polyethylene pipe. This can be avoided by filling the annular space between the inside diameter of the host pipe and the outside diameter of the polyethylene pipe. See 5.3.2.

5.4 Axial Bending:

5.4.1 Solid wall polyethylene pipe is relatively flexible such that the barrel of the pipe may be curved during installation. It will accommodate reasonable irregularities in line and grade. Excessive pipe barrel bending during handling and installation that may cause the pipe to kink should be avoided. Axial (longitudinal) bends induced during the insertion step, in transporting pipe lengths from assembly sites to job sites, or permanent bends to accommodate line or grade changes, should not be less than the minimum bending radius in Table 1. The bending radius is the inside radius of curvature.

TABLE 1 Minimum Bending Radii for Solid Wall Pipe

| Dimension Ratio, DR                                | Minimum Bending Radius |
|----------------------------------------------------|------------------------|
| <9                                                 | 20 × Pipe OD           |
| >9 to 13.5                                         | 25 × Pipe OD           |
| >13.5 to 21                                        | 27 × Pipe OD           |
| >21 to 26                                          | 34 × Pipe OD           |
| >26 to 32.5                                        | 42 × Pipe OD           |
| >32.5                                              | 52 × Pipe OD           |
| Fitting, Flange or MJ Adapter in Bend <sup>A</sup> | 100 × Pipe OD          |

<sup>A</sup>Because fittings, flange and MJ Adapter connections are rigid compared to the pipe, the minimum bend radius is 100 times the pipe OD when a fitting flange or MJ adapter is present in the bend. The bend radius should be limited to 100 × OD for a distance of about 5 times the pipe OD on either side of the fitting, flange or MJ adapter location.

5.4.2 Axial bending of bell and spigot joined or coupled profile wall or corrugated polyethylene pipe is generally limited by the allowable angular displacement of bell and spigot or spigot and coupling joints. The manufacturer should be contacted for information.

5.5 Installation Force and Length:

5.5.1 Butt fusion joined solid wall polyethylene pipes are typically installed by pulling the polyethylene pipe into the host pipe. The length of polyethylene pipe to be inserted by pulling in will seldom be limited by the strength of the pipe itself. Pulling load limitations will usually be in the load capacity of the winch and cable and the length of the pulling cable. Smaller diameter pipe 24 in. (610 mm) or less can usually be handled with winches having about 12-tons-force (110-KN) pulling capacity. Higher capacity pulling equipment, or a combination of pulling and pushing may be applicable to larger pipe, longer lengths, where the host pipe is in poor condition, or where there are bends or offsets that restrict the passage of the polyethylene pipe being inserted.

NOTE 2—For pull in installation, Practice F1804 may be useful for estimating the allowable tensile load for solid wall pipe.

5.5.2 Profile wall or corrugated pipe with bell and spigot joints is inserted by pushing. Typically, the pushing distance is limited by the joint’s resistance to telescoping. Pipe pushing load information should be obtained from the manufacturer.

6. Inspection and Cleaning

6.1 Confined Space Safety—Entry manholes or the existing pipe are usually considered entry into a confined space. Observe appropriate confined space entry requirements in accordance with local, state and federal requirements.

6.2 Preliminary Investigation—Visual inspection to determine the condition of the host piping may be practical for large diameter pipes over short distances; however, closed-circuit TV inspection equipment is generally preferred to provide details of problem areas, such as of offset joints, crushed walls, obstructions, and to locate service and other lateral connections and protrusions.

6.3 Remove obstructions, debris, and protruding service lines that interfere with inserting the polyethylene pipe.

6.4 Before the insertion operation, it may be desirable to pass a test-head of the same diameter as the polyethylene pipe through the host pipe to ensure free passage. Test-heads may be made from a short section of the polyethylene pipe with pulling cables attached to both ends so that the test head may be pulled back out if blocked by an obstruction. After passing the test head through the host pipe, the test head should be inspected for damage.

7. Excavation

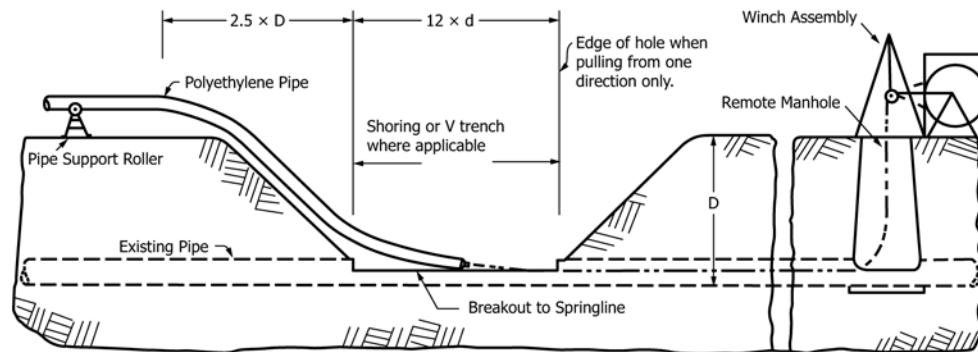
7.1 Excavation Safety—Observe all local, state, or federal regulations for excavation safety together with other applicable laws and ordinances covering public and private access, and the protection and safety of the public and property.

7.2 Insertion Excavations—For butt fused polyethylene pipes, the insertion excavation down to the springline of the host pipe should have an entry slope grade of at least 2½ to 1. The length of the level excavation should be at least 12 times the diameter of the polyethylene pipe being inserted. The excavation should be as narrow as possible, consistent with the diameter of the pipe, in situ soil, height of water table, and length of the host sewer (see Fig. 1). For single joints being pushed or jacked into position, the length and width of the excavation should at least accommodate the pipe joint laying length (typically 20 ft) and jacking equipment if necessary, (see Fig. 2).

7.3 Lateral Connection Excavations—These excavations should be located where the lateral pipe connection to the host sewer can be exposed. These points are located during preliminary investigation in accordance with 6.2. Tapping methods that are remote from the actual point of connection may be used when approved by the design engineer. Provision for sealing and anchoring the connection should be included in remote tapping methods.

8. Storage, Handling, and Joining of Polyethylene Pipe

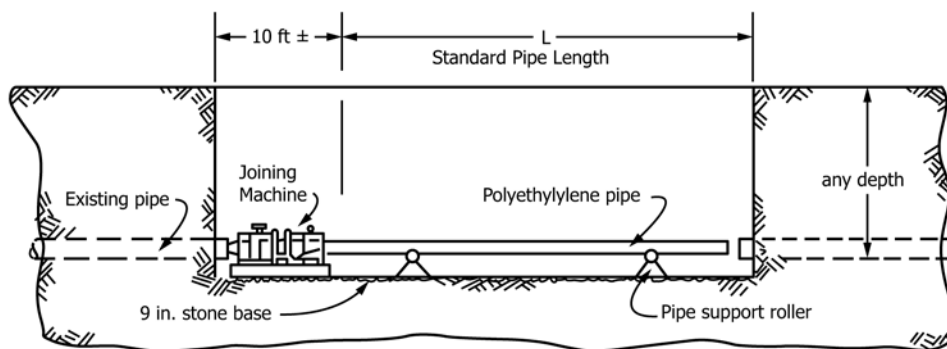
8.1 Unloading and Storage:



D = Depth to invert  
d = Diameter of insert pipe

NOTE—Width of excavation will depend on shoring method or depth of existing pipe, or both. To permit proper flow, break out existing pipe to springline only.

FIG. 1 Insertion Hole and Pull-in Detail



NOTE 1—To maintain existing flow, pump and by pass will be necessary.  
 NOTE 2—Use appropriate shoring where required.

FIG. 2 Push Method Hole Detail

8.1.1 Observe manufacturer’s unloading, handling and storage recommendations. General information on polyethylene pipe unloading, handling and storage is available in the PPI Material Handling Guide.

8.1.2 Depending on size, polyethylene pipe lengths are packaged for commercial transport in bundles, layered lengths (strip loads), or individual lengths. Polyethylene pipe should be unloaded from the truck as packaged, and handled with properly rated lifting equipment such as forklifts or lifting boom equipment. Wide fabric slings and spreader bars should be used with lifting boom equipment. Chain or wire ropes are not recommended because pipe can slip and cause injury or damage.

8.1.3 The pipe storage area should be level ground, free of stones, debris, or litter of any type which could create points of loading on pipe in contact with it. Pipe should be stored as packaged (in bundles) for transport, and not removed from packaging until needed for installation. If pipe packages (bundles) are stacked, the stacks should not be more than 4-6 ft in height, and pipe packages (bundles) should be removed from the stack and placed on the ground before removing pipes from the package.

8.1.4 Individual pipes may be placed in a pyramidal pile of no more courses than recommended in the manufacturer’s literature and chocked to prevent roll-out of the bottom layer. The bottom courses should have the ends of the pipe covered to prevent dirt or debris or small animals from entering the pipes.

8.1.5 Care should be taken to minimize handling damage. Black polyethylene pipe need not be protected from sunlight or weather for indefinite storage periods. Non-black polyethylene pipe such as grey or green is generally suitable for unprotected outdoor storage for at least 6 months. Manufacturer’s recommendations for extended storage should be observed.

8.2 *Inspection:* Pipe should be inspected for damage upon receipt at the jobsite, and immediately prior to joining. Receiving damage should be noted on receiving documents and reported to the supplier for disposition. Damage such as serious abrasion, cutting, or gouging of the outside or inside surface extending to more than 10 % of the wall thickness in depth, or kinking due to excessive or abrupt bending should be cut out and discarded.

8.3 *Joining:*

8.3.1 Butt fusion in accordance with Practice F2620 is the primary method for joining solid wall pipe into appropriate lengths. Butt-fusion requires training and experience to develop appropriate joining skills and should only be undertaken by persons that have been trained in the process.

8.3.2 Profile wall pipe is joined using gasketed spigot and bell joints, gasketed couplings, extrusion welding, heat fusion in accordance with Practice F2620, or electrofusion. Extrusion welding is used to supplement gasketed spigot and bell joints and gasketed couplings when axial restraint is necessary.

8.3.3 Before using joining equipment in the trench or manhole, evaluate the atmosphere in accordance with local, state, or federal regulations to determine the presence of flammable or toxic vapors.

8.3.4 Mechanical connections such as flanges, mechanical joint adapters, OD clamps may be used for connections to other pipes, or appurtenances are required. See Fig. 3. Mechanical connections should be evaluated by the engineer for annular clearance and axial load resistance. All mechanical joints should be supported to distribute shear and bending loads that may cause flexing away from the area of the joint. The potential for pull-out arising from thermal contraction should be considered in the design of an insertion-joined pipeline using mechanical connections.

9. Insertion and Termination

9.1 *Insertion*—Pipe may be pulled (see Fig. 1) or pushed (see Fig. 4) into the existing piping. Pull and push techniques can sometimes be combined for the placement of unusually heavy-walled or exceptionally long lengths of polyethylene pipe, or where equipment load capacity limits the use of the pull technique, pushing or a push-pull combination technique may be used to extend the insertion distance.

9.1.1 Whether pulling or pushing installation is used, a tapered nose-cone or pulling head to guide the pipe end past minor obstructions and prevent entry of debris is necessary. See Fig. 5 and Fig. 6. A nose cone or pulling head is usually open to allow flow into the insertion pipe.

9.1.2 Whether pulling or pushing installation is used, a tapered nose-cone or pulling head to guide the pipe end past minor obstructions and prevent entry of debris is necessary.

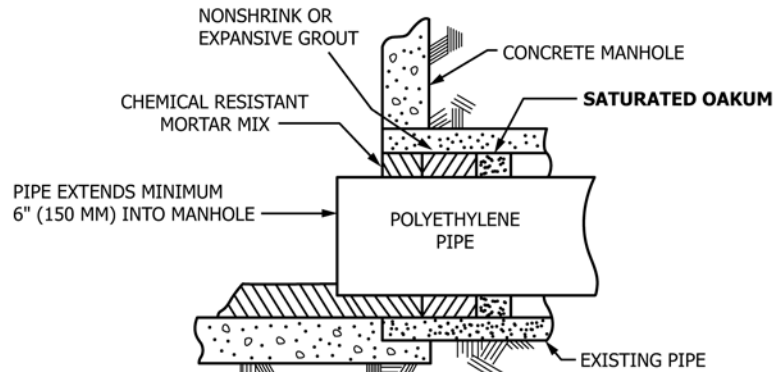
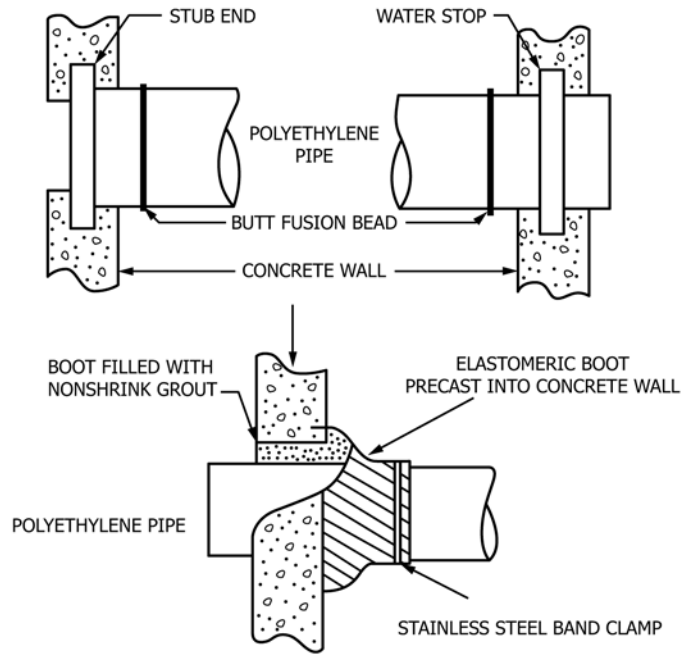


FIG. 3 Typical Manhole Seal

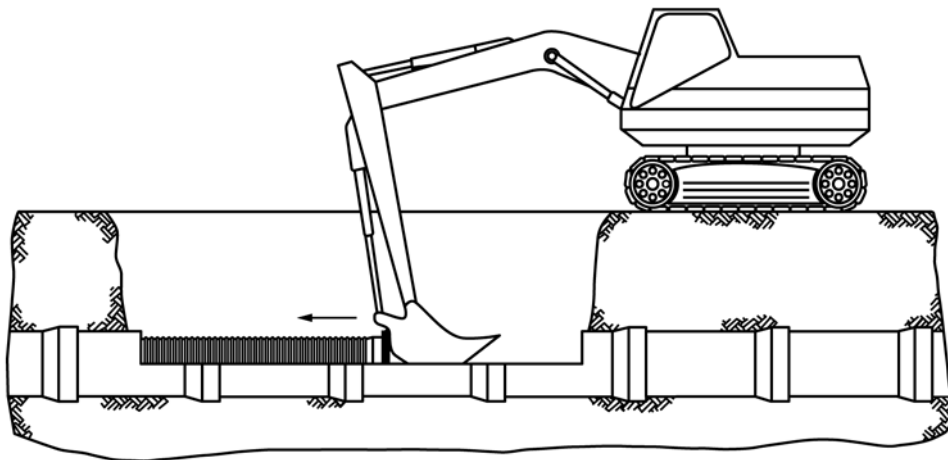


FIG. 4 Push Method for Bell and Spigot

See Fig. 5 and Fig. 6. A nose cone or pulling head is usually open to allow flow into the insertion pipe.

9.1.3 It may also be necessary to install temporary guards over the edges of the existing pipe at the inlet end to protect the

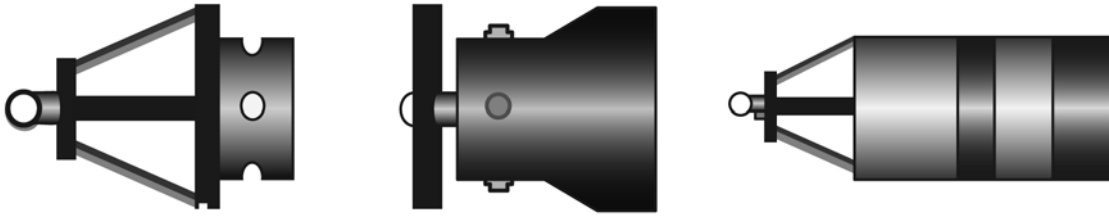


FIG. 5 Pulling Head

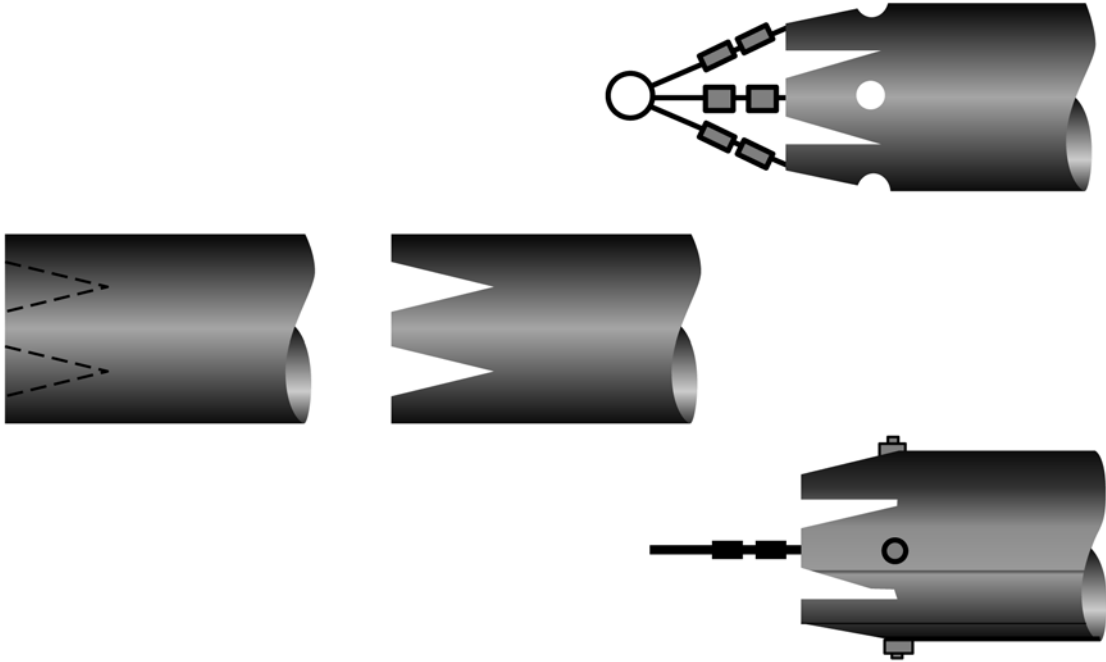


FIG. 6 Field Fabricated Pulling Head

polyethylene pipe against gouging and excessive surface abrasion during insertion. Split rings of polyethylene pipe may be useful for this purpose.

9.1.4 Fused lengths of solid wall polyethylene pipe may be pulled into place using a cable and winch arrangement. See Fig. 1.

9.1.4.1 For pulling or combination push-pull insertion, a cable from a winch or pulling device at a manhole or shaft the end of the insertion length is fed through the existing piping and attached to a nose cone or pulling head on the polyethylene pipe. The winch or pulling device is usually at surface grade; therefore, a pulley frame is installed at the base of a manhole or shaft to direct the cable from vertically down the manhole or shaft, then horizontally through the existing piping. The pulley frame should be constructed to withstand axial loads required to insert the pipe, and vertical loads from the winch at top of the manhole or shaft. The winch is usually powered; but a hand-operated winch may be sufficient for the insertion of small piping. Load controls are desirable to prevent overstressing the equipment in case of a blockage. When pulling equipment can exert greater tensile load than the pipe can safely withstand, a break-away device should be installed between the nose cone or pulling head and the winch cable. Practice F1804 may be used to determine the pipe's allowable tensile load, ATL.

9.1.5 The insertion operation often proceeds with a “slip-stick” action. Therefore, a means of coordinating the feed of pipe into the entrance with winch operation is desirable. The pulling operation will tend to stretch the pipe, and excessive stretching (more than 1.5 %) should be avoided. The pulling speed should not exceed about 360 ft/h (110 m/h). Slower speeds will be necessary under more difficult conditions. Once started, pulling should continue without interruption until completed.

9.1.6 On reaching the exit point, the pipe should be pulled beyond this point as advised by the coordinator at the entrance point. Stretching of about 1 % of the total length pulled will often be observed. This stretching will be recovered over a period of time. A 24 h relaxation period is recommended. Additionally, pipe length change due to temperature change may be observed. This can be as much as 1 in./100 ft/10 °F (20 mm/30 m 5 °C) difference in temperature between the pipe before and after installation and this should be allowed for in the length of insertion pipe used. Before insertion, polyethylene pipe temperature and existing pipe temperature should be measured with a pyrometer or infrared temperature gauge, and the temperature difference used to determine pipe shrinkage due to temperature reduction. To assure that the ends of the polyethylene pipe extend past the entrance and exit of the existing pipe after the stabilization period, the polyethylene

pipe should be pulled past the end of the existing pipe by an amount equal to estimated stretching length plus temperature reduction length.

$$L_A = 0.01L + \left( \frac{L\Delta T}{12000} \right) \quad (1)$$

- $L_A$  = Length to extend past existing pipe exit, ft.
- $L$  = Polyethylene pipe length, ft.
- $\Delta T$  = Temperature difference between polyethylene pipe and existing pipe, °F.

9.1.7 *Push Insertion*—Butt fused lengths of polyethylene pipe may be pushed into place using a choke cable. See Fig. 7. For bell and spigot joined pipe, each joint of pipe may be pushed in place, see Fig. 4. When bell and spigot-jointed pipe is installed using the pushing method, flow in the host line is permitted. Optimum conditions are achieved when the flow is at or below the springline. Maintaining the flow will reduce pushing force requirements and eliminate the need for bypass pumping. Insertion is accomplished in an incremental process. When a joint is lowered into the insertion pit, it is joined to the previous pipe. This pipe joint and previous joints are pushed up the line to make room for the next pipe joint. This process is continued until length of the host pipe has been lined. This technique also permits the lining of curve sections using short lengths of pipe. After the recommended 24 h relaxation period following the insertion of the polyethylene pipe, each individual service connection and lateral can be added to the new system. One common method of making these connections involves the use of a wrap-around service saddle. The saddle is placed over a hole that has been cut through the liner pipe, and the gasket assembly is then fastened into place with stainless steel bands. Additional joint integrity can be obtained by extrusion welding of the lap joint created between the saddle base and the liner. By extending the saddle into the branch opening, the service lateral can then be connected into the saddle. Once the lateral has been connected,

the entire area can be stabilized by following standard direct burial procedures. For pressure applications, lateral connections can be made using sidewall fusion of branch saddles onto the liner. As an alternate, a molded or fabricated tee may be fused or flanged into the liner at the point where the lateral connection is required (see Fig. 2).

9.1.7.1 If the work is done during warm weather, contraction of the polyethylene pipe may be observed. This can be as much as 1 in./100 ft/10 °F (20 mm/30 m/5 °C) difference in temperature between the pipe before and after installation. Before insertion, polyethylene pipe temperature and existing pipe temperature should be measured with a pyrometer or infrared temperature gauge, and the temperature difference used to determine pipe shrinkage due to temperature reduction. To assure that the ends of the polyethylene pipe extend past the entrance and exit of the existing pipe after the stabilization period, the polyethylene pipe should be pulled past the end of the existing pipe by an amount equal to estimated stretching length plus temperature reduction length.

$$L_B = \left( \frac{L\Delta T}{12000} \right) \quad (2)$$

- $L_B$  = Length to extend past existing pipe entrance, ft.
- $L$  = Polyethylene pipe length, ft.
- $\Delta T$  = Temperature difference between polyethylene pipe and existing pipe, °F.

9.2 *Termination and Service Connections:*

9.2.1 After insertion, the polyethylene pipe should be allowed to stabilize undisturbed for a 24 h period.

9.2.2 The annular space between the liner pipe and the original pipe may be filled with grout or other material if required by the design engineer. Grouting will stabilize the line against flotation off-grade and collapse due to external ground water pressure. During the grouting process the installer should take care not to exceed the allowable grouting pressure of the pipe. The grouting pressure can be calculated using Love’s

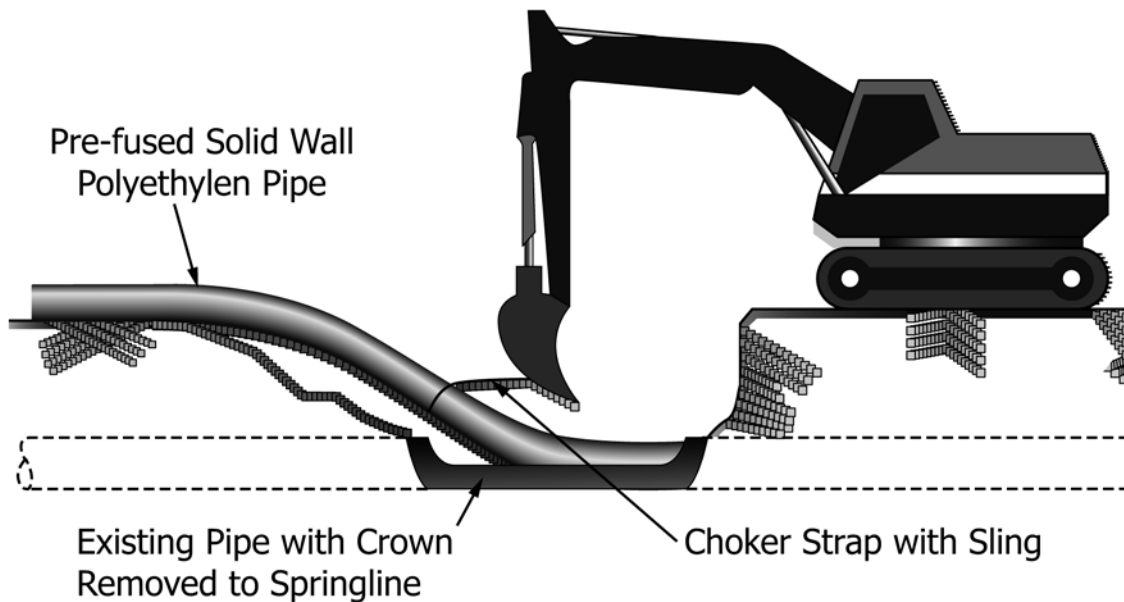


FIG. 7 Alternate Push Technique

Equation with appropriate safety factors or the manufacturer can be consulted. It is suggested that the installer place standpipes and other means of regulating grout pressure in the line.

9.2.3 At entries and exits from manholes, the annular space between the original and inserted pipes should be sealed. The sealing compound used must be recommended for underwater application and have elastomeric properties. Products used should be approved by the engineer. Typical manhole connections are shown in Fig. 3.

9.2.4 Following the insertion of the polyethylene pipe, each individual service and lateral connections can be added to the new system

9.2.4.1 Service and lateral connections that are not in manholes require a bell-hole excavation, and removal of the top half of the existing pipe to expose the polyethylene pipe.

9.2.4.2 One common method of making these connections involves the use of a wrap-around service saddle. A hole is cut through the polyethylene pipe, a gasket placed around the hole, the saddle is placed over the gasket, and the assembly is secured with stainless steel bands. Additional joint integrity can be obtained by extrusion welding of the lap joint created between the saddle base and the polyethylene. The service or lateral can then be connected to the saddle.

9.2.4.3 Once the service or lateral has been connected, the entire area should be stabilized by following standard direct burial procedures. For pressure applications, lateral connections can be made using sidewall fusion of branch saddles onto the polyethylene pipe. As an alternate, a molded or fabricated

tee may be fused or flanged into the polyethylene pipe at the point where the lateral connection is required (see Fig. 8).

**10. Finishing and Restoration**

10.1 Whenever necessary, manholes should be rebenched to suit the contours of the new pipe.

10.2 Shafts should be backfilled in accordance with local requirements to minimize subsidence and permit surface restoration in accordance with the terms of the contract.

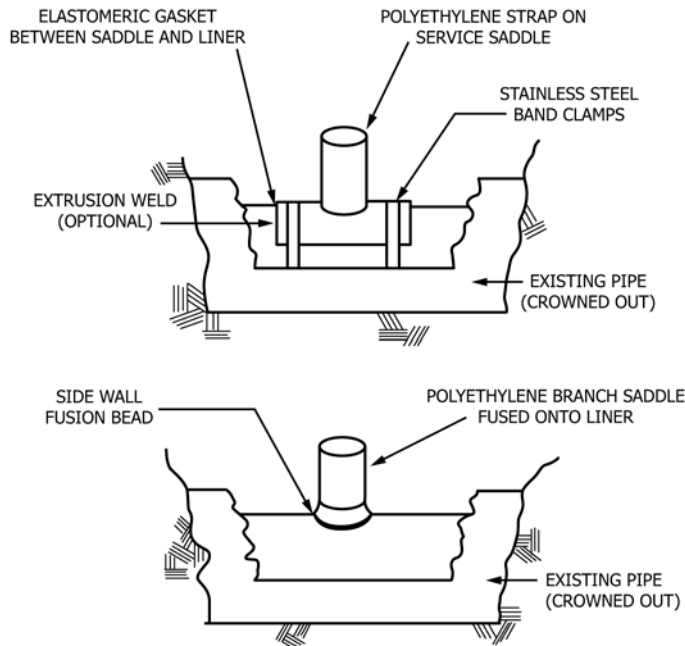
**11. Inspection and Acceptance**

11.1 The installation may be inspected visually if appropriate or by closed-circuit TV if visual inspection cannot be accomplished. Variations from the true line and grade may be inherent because of the condition of the original piping. Minor distortion of the inserted pipe may be observed, but localized dimpling, reverse bending of the pipe walls, or flattening resulting in more than a 10 % reduction in diameter should be corrected. No infiltration of ground water should be observed. All service entrances should be accounted for and be unobstructed. A summary report with photographs should be submitted to the engineer.

11.2 When the engineer has deemed that a leakage test is required, Practice F1417 should be observed.

**12. Keywords**

12.1 conduit; insertion renewal; polyethylene pipe; rehabilitation; sanitary sewer; sliplining; storm sewer; trenchless



**FIG. 8 Lateral Service Connections**

## SUMMARY OF CHANGES

Committee F17 has identified the location of selected changes to this standard since the last issue (F585–13) that may impact the use of this standard.

(1) Inclusion of a new 5.1.6 and new Note 1 to clarify hydraulic capacity and design issues.

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**Federal Energy Regulatory Commission  
Division of Dam Safety and Inspections**

**GUIDELINES FOR DRILLING  
IN AND NEAR EMBANKMENT DAMS  
AND THEIR FOUNDATIONS**

**Version 3.1 – Approved for Public Release  
June 2016**



# **GUIDELINES FOR DRILLING IN AND NEAR EMBANKMENT DAMS AND THEIR FOUNDATIONS**

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# **GUIDELINES FOR DRILLING IN AND NEAR EMBANKMENT DAMS AND THEIR FOUNDATIONS**

## **1.0 INTRODUCTION/PURPOSE**

### **1.1 Objective**

The primary purpose of this document is to provide guidance for drilling in and near embankment dams and their foundations. Of special emphasis is the prevention of damage to the embankment, structures, and their foundations from hydraulic fracturing, heave, erosion, filter/drain contamination, or other mechanisms during drilling-related activities.

The need for any investigation (drilling, testing, etc.) at a dam site should have been presented to and accepted by FERC prior to developing an investigation program requiring drilling activities in or adjacent to a dam. In addition, it should have been demonstrated that any potential damage to the structure created by the drilling and associated processes is outweighed by the need for the drilling data. It is not the purpose of this document to provide an all-inclusive guidance or best practices on considerations for the development of a subsurface exploration or investigation program for a dam.

A guiding principle inherent in any potential dam investigation or testing is DO NO HARM. In developing investigation plans it is important to identify the potential risks and develop and implement plans to mitigate, manage, or avoid those risks.

These guidelines are appropriate for FERC-regulated embankment dam or other earthen water retaining structures (levee, canal embankment, etc.) – any earthen structure that’s responsible for holding back water or serves to provide direct support to the feature or element that is holding back the water, including its foundation.

Much of the information in Sections 3.0 and 4.0 of these guidelines have been taken from or modified from the following documents:

1. U.S. Army Corps of Engineers, “Drilling in Earth Embankments and Levees”, ER 1110-1-1807, Washington, DC, December 31, 2014.
2. Bureau of Reclamation, “Guidelines for Drilling and Sampling in Embankment Dams”, Denver, CO, August 2010.

### **1.2 Scope**

Much of the information contained in this guideline has principles and applications to other forms and purposes of investigation, maintenance, construction, modification, or other activity that physically penetrates the dam or foundation, including:

- Test pits/trenches
- Drilling holes/borings and probes

- Dynamic loads/pulses/blasts
- Excavations, including grading/regrading and foundation installations/construction
- Utility installation, including buried conduits, utility vaults, utility poles, etc.
- Concrete removal/demolition
- Drain and relief well cleaning/maintenance
- Toe drain/drainage feature modifications/repair
- Penetrations, including conduits, horizontal drilling activities, etc.
- Grouting or other pressure injection/testing activities
- Removal of large vegetation, trees, and root balls

These apply to any area subject to seepage pressures, stability influences or have the potential to cause harm to the water retaining structure or its foundation.

## 2.0 BACKGROUND

There is a very real potential for damaging structures during the drilling process if these guidelines are not followed. Damage created by hydraulic fracturing during the drilling process (use of inappropriate drilling methods), improper in-situ sampling techniques, and/or unacceptable methods of completing (backfilling) borings can open seepage paths which could create conditions conducive to internal erosion (piping) and ultimately dam failure. Although not particularly well documented, there are a number of case histories that have highlighted the potential dangers that can happen as the result of improper planning, using improper drilling methods in dams, not having the appropriate drilling equipment and contingency plans, not having knowledgeable field staff present on-site during the drilling operations, and other factors (France, 2002).

There is also some not well publicized guidance on precautions in developing investigation programs, precautions on appropriate drilling methods, and other 'rules of thumb' that are important to consider and others that should be avoided.

Drilling in embankments often does not provide conclusive data related to seepage and piping problems within a structure. The chance of finding a disturbed zone in a dam by drilling is small, and there could be great risk. Piezometers can be installed to monitor seepage problems, but they are only effective if the problem area is known. One case for drilling into embankments could be to collect samples to evaluate filter criteria of transition zones within the structure. This can be accomplished with shallow drilling, preferably above the phreatic surface in the dam and sometimes at angles into the structure to target transition zones. Holes could be drilled from the crest or downstream shell of the structure, index tests performed, and soil samples obtained. Care must be taken during drilling to be sure that internal drainage features are not damaged or contaminated. If drilling must be performed in a dam subject to seepage and piping problems, seepage flows need to be monitored continuously during the investigation, and drilling fluids need to be controlled as discussed in the section on drilling.

Dams with seepage problems may require investigation to determine the condition, location, or even whether drains exist in the structure. In these cases, test pit excavations may be attempted. However, the possibility for piping of the foundation into an excavation or drill hole could exist

and should be carefully assessed. Some dams already may have evidence of critical gradient development at the toe or into drains or manholes. **Drilling at the toe of the dam is risky even if seepage is not evident.** If there is concern about the occurrence of piping, a contingency plan must be developed. For example, for test pitting at the toe, if critical gradient piping is a concern, materials to stop progressive erosion in the trench must be ready. For this situation, it is recommended to stockpile fine (C33 concrete sand) and coarse processed aggregates and geotextiles at or near the site to filter and plug the excavation. If drill holes must be advanced under a critical gradient condition, one should consider the construction of drill berms at the toe.

Liquefaction investigations often require drilling through the shell or crest of a dam to perform standard penetration tests (SPTs) in embankment core and/or unconsolidated foundation materials under the structure. Testing can also be performed at the downstream toe, but these soils often are not consolidated like those under the dam, and at times it is preferable to test the material under the structure. However, holes are often drilled in alluvium at the toe of a structure. Materials also can be investigated in accessible test pits to evaluate the density of the soil. In some cases, drilling can be performed from the crest of the dam as long as the cutoff trench or wall is not penetrated. Access roads may be required on the downstream slope or, in rarer occurrences, on the upstream slope if water levels allow.

The preferred method of determining SPT results in loose sands below the water table is by fluid rotary drilling where the mud pressures and hydrostatic forces can be used to stabilize the sands. However, in locations that include concerns with possible hydraulic fracturing, use of hollow-stem augers (HSA) is preferred.

In 2000, a FEMA-sponsored workshop was convened with a group of experts with respect to dam safety issues associated with seepage through embankments and their foundations (FEMA, 2000). As part of that workshop, the participants offered the following recommendations relative to the investigation and monitoring of seepage problems and concerns:

- Although actual investigation practices vary widely, it was the consensus of the workshop participants that the recommended state-of-the-practice should be that drilling should not be done in the core of an existing embankment dam unless absolutely necessary, and then only with carefully planned precautions and dry drilling (e.g., auger) methods. The risk of hydraulic fracturing is too great to support drilling in the core without appropriate justification.
- It was the consensus of the workshop participants that drilling or test pitting should not be done at the downstream toe of a dam with water stored in the reservoir, without contingency plans and stockpiling of weighted filter materials (e.g., sand and gravel) to be used in the event of a seepage incident. It is also essential that such explorations be completed with the on-site presence of experienced personnel with the knowledge to react appropriately to any seepage incidents that may occur.
- It was the consensus of the workshop participants that they generally advised against installing piezometers in an embankment core, unless there were very compelling reasons for the instruments. The workshop participants felt that, in most cases, piezometers in the

core do not provide significant additional understanding of the performance of the dam beyond that which can be obtained from piezometers in the upstream and downstream shells, which are much safer locations for the instruments.

- Piezometers are tools whose careful installation and subsequent data interpretation, in conjunction with other investigative techniques, may provide valuable information in diagnosing seepage conditions. However, the limitations of what the piezometers record must be recognized, and the piezometer data must be used in conjunction with other information (e.g., seepage rates, seepage locations, etc.) to correctly diagnose seepage conditions. Since piping channels in embankments are often relatively long, narrow features, it is highly unlikely that piezometers will be located at exactly the correct locations to provide direct data regarding the piping phenomenon.

### **3.0 PLANNING/PROJECT INFORMATION**

When planning an investigation program, the first consideration is if the need for the data to be collected justifies the cost and potential risk to the structure created by the data collection process. A determination of potential consequences if no action is taken should be made. These consequences should include both risk and likelihood for worsening conditions, which could drive up future cost of remediation if required. When and where possible, the determination of consequences should be performed with available data. However, a scaled down investigation program may be required before an adequate assessment can be performed.

If data collection is justified, a multidiscipline exploration team should be formed to determine exploration components required to adequately address the data needs. The exploration team should consist of engineers, geologists, and others with the requisite knowledge and experience in planning and conducting field exploration programs for dams. The exploration team should thoroughly discuss data needs and investigation plans to ensure compatibility.

A thorough search of all available records should precede any investigation program. Sources of information that could be useful in evaluating the need to collect additional data include:

- Geologic mapping, logs, and reports from previous investigations and construction
- Owner and FERC project files
- Supporting Technical Information (STI) document
- Current and past consultant files
- Records of design and construction, including photographs
- Archived records
- Project records at field offices and at the project site.

The exploration program should consider:

- Purpose of the investigation
- Cost of the exploration
- Required sample type and size (disturbed or undisturbed)
- Acceptable drilling and investigative methods

- Depth, diameter, and inclination of drilling required
- Materials to be drilled and sampled
- Utilities, surface and underground obstacles, and accessibility
- Location of any seepage cutoff walls, blankets and drainage features and pipes
- Dam foundation geometry and drilling hazards
- Instrumentation and completion requirements

The investigation may also require clearances, permits, and traffic control plans. The investigation schedule must allow time to obtain clearances and permits. In most cases, National Environmental Policy Act (NEPA) compliance activities will be required. Under the National Historic Preservation Act, some sites may require inspection by an archeologist and a permit from the State Historic Preservation Officer (SHPO).

## **4.0 DRILLING ACTIVITIES**

### **4.1 General**

Drilling into, in close proximity to, or through dams and their foundations may pose significant risk to the structures. Water, compressed air, and various drilling fluids have been used as circulating media while drilling through dams and their foundations. Although these methods have been successful in accomplishing the intended purposes, there have been incidents of damage to embankments and foundations (Sherard, 1973). While using air (including air with foam), there have been reports of loss of circulation with pneumatic fracturing of the embankment as evidenced by connections to other borings and blowouts on embankment slopes. While using water and drilling mud as the circulating medium, there have been similar reports of erosion and/or hydraulic fracturing of the embankment or foundation materials. Conversely, there have been cases where heave, borehole collapse, and significant disturbance have occurred while drilling in granular materials below the groundwater level. This typically has been the result of not using a proper drilling fluid to balance the water pressures in the soil or using high energy systems that induce heave in order to evacuate the cuttings. There is a delicate balance between too much induced fluid pressure that will cause hydraulic fracture and not enough fluid pressure that will result in borehole instability, heave, or significant sample disturbance. Other potential damaging effects include: creating preferential seepage paths due to improper backfilling, inadequate protection of embankment from drilling fluids during foundation rock coring, erosion and widening of cracks, and inadvertently clogging filters or drains with drilling fluid or grout.

All drilling and associated activities that use fluid or other circulation or stabilization media need to be evaluated for the potential to hydraulically fracture the embankment or foundation. These activities include but are not limited to the use of drilling fluids, backfilling borings after completion, backfill grouting of instrumentation, backfill grouting of casings, water testing for permeability, piezometer rehabilitation, etc. The risk will vary with the selected methods and the site conditions. Every drilling operation must be well thought out and must have benefits of successful completion that confidently outweigh the risk of potential negative impacts.

## 4.2 Drilling Hazards

The following is a brief discussion of some common drilling hazards that must be considered, evaluated, and mitigated for in developing and implementing an exploration program.

### 4.2.1 Hydraulic Fracturing

Excessive pressures from water, air, drilling fluid, or grout can fracture embankment and foundation materials. Hydraulic fracturing problems have occurred while drilling in embankments as evidenced by reports of loss of fluid circulation, blowouts into nearby borings, seepage of drilling fluids on the face of the embankment, and other similar situations. Hydraulic fracture can occur in both cohesive materials and cohesionless materials, and bedrock. It has been found that in soils, hydraulic fracturing can occur when the borehole pressure exceeds the lowest total confining stress (minimum principal stress,  $\sigma_3$ ) plus some additional strength (Sherard, 1986). The additional strength can be approximated by the undrained shear strength of the soil. The minor principal confining stress ( $\sigma_3$ ) in a normally consolidated soil with a level ground condition is typically the horizontal stress, which can be reasonably estimated. However, the minor principal confining stress in and under an embankment is difficult to determine and can vary significantly from idealized geostatic conditions. Effects from the side slope geometry, piezometric surface, abutment configuration, foundation rock geometry, embedded structures, compaction stress, and settlement history all are significant and can influence in-situ stress conditions. Typical drilling methods that use circulation fluids can quickly create induced fluid pressures that exceed the minimum confining stress. This often occurs when the return path for the fluid clogs or blocks off and the induced fluid pressures quickly increase. The use of non-pressurized stabilizing fluids is preferable, yet in some subsurface conditions, hydraulic fracture can occur under gravity pressure. Low stress zones may exist within and under embankments. It is possible for the confining stress in these locations to be much less than the gravity pressure exerted by a drilling fluid or grout.

Certain embankment locations and conditions have a higher potential for hydraulic fracturing due to geometric configurations that create zones of low confining stress. Sherard 1973 and 1986 are good references that provide a comprehensive evaluation of the issues along with numerous case histories. Locations and conditions where hydraulic fracturing by drilling media is more likely to occur and have the higher potential of damaging the structure include the following:

- Near and over steep abutments that create low confining or tensile stress conditions.
- Adjacent to rock overhangs on abutments.
- Adjacent to buried structures or abrupt foundation geometry change that creates a differential settlement condition and a zone of lower soil stress transfer.
- Adjacent to conduits where narrow zones of soil backfill were placed between the structure and rock face.
- Dam cores that can experience more settlement than the adjacent shells.
- Dams in very narrow valleys. Arching keeps full confining stresses from developing.
- Near abutments where abrupt changes in geometry occur.

- In areas where the embankment is subject to differential settlement due to large differences in thickness of adjacent compressible foundation or embankment soils.

Accurately estimating in-situ embankment stresses can be difficult for the conditions listed above. In some cases, it may be helpful to calculate static stresses including seepage forces within the embankment. The results of such computations can aid in evaluating the maximum applied drilling fluid pressures or static grouting head for borehole backfill. However, with any such computation, judgment is required in applying the results.

Additional references on hydraulic fracturing are included in Appendix A.

#### **4.2.2 Artesian Conditions/Blowout**

In situations where the presence of higher fluid pressures in the subsurface materials is suspected, either at the ground surface or at depth, it may be necessary to install a surface casing to control artesian pressures if the pressures are anticipated to be significant and/or derived directly from reservoir head. Surface casing of slightly larger diameter than the augers or drill string to be used is grouted in place and allowed to set prior to advancing the borehole to depth. If flow from the borehole occurs, the surface casing provides a means of controlling it by blocking off the space between the augers/drill rods and well casing. When the static water level is very near the ground surface or artesian conditions prevail, one should consider elevating the drilling rig on a temporary drill berm to raise the drill hole collar elevation. In extreme cases, the berm should consist of filter zones. Specific details such as height of the drill pad and amount of surface casing must be developed on a case-by-case basis dependent upon specific conditions present at the site. Even if artesian pressures are not expected at a given site, potential risk requires contingency plans be in place in case these conditions arise.

If holes must be advanced at the toe of a dam that has a critical gradient condition, planning and precautions should be developed. In all cases, issues of this nature should be identified and addressed by the exploration team prior to commencement of work. In these areas, it is necessary to maintain a positive hydrostatic pressure on the drill hole to prevent a “blowout.” In instances when higher pressures are not anticipated, the addition of commercial densifiers to the drill mud may successfully address the concern.

#### **4.2.3 Erosion**

The introduction of drilling fluids into cracks, either existing or formed by hydraulic fracture, can potentially cause erosion along the crack walls. This will enlarge the crack and could lead to an increased potential for internal erosion. Existing subsurface cracks are common in many dams and are often the result of differential settlement. The locations most at risk for existing cracks are typically the same areas that have low confining stress and have the highest risk for hydraulic fracture to occur.

#### **4.2.4 Contamination of Filter/Drainage Features**

In addition to hydraulic fracturing, the use of drilling fluids can pose a contamination risk for internal drainage features if the drill fluid or sealing grout migrates into and clogs the drain or filter materials. Avoid drilling near drains or seepage blankets that may become contaminated by fluids. If drain penetration is justified, special provisions must be taken to prevent contamination. Special provisions may also be required for protecting the drainage features while backfilling the hole (such as placement of filter material through the zone of the drain or filter and installing lower and upper seals).

#### **4.2.5 Heave and Sample Disturbance**

Drilling programs that include performing in-situ tests or undisturbed sampling may require the use of drilling fluid to offset the confining stress relieved by the drilling of the hole. There have been cases where the failure to prevent stress relief or heave of granular soils below the water table have led to invalid in-situ test results and subsequently invalid interpretation of the subsurface conditions. This has occurred for both tests performed in drill holes and test performed in casings installed by methods that did not control heave or disturbance.

BOR (1999) contains methods to deal with heaving sands while drilling and performing Standard Penetration Tests. If high quality undisturbed samples of fine grained soils are required for shear strength testing, then drilling mud may be required to prevent the soil from failing in undrained triaxial extension. See Ladd and DeGroot (2004) for a discussion on clay sample disturbance due to drilling.

Prior to embarking on any drilling activity, the exploration team should consider, at a minimum, these potential drilling hazards and develop the drilling plans to avoid or mitigate these hazards. If the hazards cannot be avoided, then the risks must be evaluated and mitigated in the drilling plan.

### **4.3 Drilling Methods**

There are numerous drilling methods available to perform geotechnical investigations. The American Society of Testing and Materials (ASTM D6286) provides a comprehensive guide for drilling methods and groups individual practices for eight drilling methods (ASTM, 2006). Other good texts on drilling include The Bureau of Reclamations Earth Manual, Part I, Third Edition, Chapter 2 (BOR, 1998), the Australian Drilling Manual (ADI, 1992), and the National Drill Association Drilling Manual (NDI, 1990). Details of these drilling methods are not discussed in-depth in this guide.

Nine major drilling methods are briefly discussed below. Table 1 provides a quick reference to each method. All drilling methods that use air or fluid media have the potential to create hydraulically-induced fractures. Air drilling methods use high pressures and are well known for causing fracturing with air traveling long distances. Therefore, drilling with air as the drilling medium should never be considered when there is potential to encounter the core of an embankment dam.

*The drilling methods listed below are in order of preference for use in drilling and sampling in embankment dams. Only the first three are considered preferred methods.*

All drilling programs in dams should be designed to minimize the need for any drilling fluid such as air, gas, water, mud, polymers, slurries or any other drilling fluid that could pressurize the borehole soils. If the drilling objective can be performed using dry methods such as augers or sonic drilling they should be employed in lieu of methods that require fluids. If drilling fluids must be used due to the drilling objective or the subsurface conditions, the drilling plan must contain an analysis of the potential to cause damage and a plan that covers the measures that will be used to minimize the risk (see Section 4.8 for additional information). The use of pressurized air or foam should only be considered when drilling in materials that will not transmit pressures to the soil core or other critical features or when the air pressure is reliably isolated from the borehole soils. Drilling in an open graded rockfill shell may be an example of when using air may be appropriate. All drilling programs that propose the use of stabilizing or circulating fluids or other media will require an additional level of review.

- 1. Hollow-Stem Auger** – Hollow-stem auguring (HSA) is a preferred method of drilling in the core and most other areas of an embankment dam without restriction. Blowout prevention measures, such as sealable surface casing, should be used prior to advancing augers in areas where there is potential to encounter artesian conditions. If no fluid is added to the auger column, it does not pressurize the embankment and no potential for hydraulic fracturing exists. However, for SPT testing, it may be required to add some fluid to stabilize loose sands and gravels. In instances when groundwater is encountered or fluids are added to the process, the auger string should be raised and lowered slowly to avoid pressurization, negatively and positively, respectively, of any open hole. Using a hollow-stem auger permits sampling in the embankment and allows sampling/testing of the foundation through the auger's hollow-stem which acts as casing. Continuous sampling is described in ASTM D6151 (ASTM, 2008). Small diameter cores of 3 to 4 inches in diameter can be taken in 5-foot-lengths using the split inner sampling barrel. High quality, undisturbed samples can be taken with larger diameter HSA (6-inch ID and larger) in acrylic liners that provide samples suitable for laboratory testing.
- 2. Sonic Drilling** ASTM D6914 (ASTM, 2010) – Sonic (vibratory) drilling is a preferred method of drilling in the core and other areas of embankment dams. This method uses a double casing system and vibrating drill head to set up standing waves or resonance to the drill steel to advance the boring. This method of drilling is favored due to its lack of drill fluid and rapid speed of drilling. The drilling process first advances a core barrel. The core barrel is removed, and the sample is extruded while the outer casing is then advanced to the end of the sampling run. There are no cuttings generated, and there is some compaction of soil around the annulus of the drill. Crowd-in and crowd-out bits are used depending on the formation. Some water (static water, not under pressure) is required for dry cohesive formations to lubricate the drill stem. The cores, typically 4 to 5 inches in diameter, are useful for lithology determination and samples may be adequate for standard engineering properties laboratory analysis, but does not meet criteria for many laboratory tests requiring undisturbed samples (Dustman, et al, 1992). Since there

is uncertainty as to the extent of disturbance to the adjacent foundation material from the vibratory drilling process, sonic drilling should not be used if SPT, undisturbed sampling, and certain in-situ testing are required.

- 3. Cable Tool or Churn Drilling** ASTM D5783 (ASTM 2000a) – Cable tool or churn drilling, with minor restriction, is a preferred method of drilling in embankment dams. This is an older method of drilling that is infrequently used. Drill action is by up and down movement of the drill string and jars (bit). The drill string is regularly pulled and a bucket-grab tool is inserted to remove/sample the cuttings. Water is often added to the hole to mix the cuttings into slurry. SPTs can be completed below the bottom of the casing. This method of drilling is rated high in desirability because it does not use a full column of drilling fluid and, therefore, has low potential for fracturing. Drilling speed is fairly comparable to HSA drilling. One variation of this “chop and drive” technique employs continuous circulation of water to bring cuttings to the surface and should not be used in the core of an embankment dam.
- 4. Dual Rotation Drilling** ASTM D5781 (ASTM 2000b) – Dual rotation drilling is not a preferred method for drilling in embankment dams, and its use in embankment core material must be approved by FERC prior to use. The dual rotary drilling method advances both the casing and the drill string/bit separately. The upper and lower rotary drives feed independently by use of separate hydraulic cylinders. Distances between the bit tip and casing shoe are adjustable. With the bit advancing ahead of the shoe, drilling becomes more aggressive. These bit to shoe relationships allow the pressurized drilling medium to come in contact with the unprotected hole wall, and potential for hydraulic fracturing increases. When drilling in embankment core material, the bit should not be advanced ahead of the shoe. In those instances when the bit advances ahead of the shoe they should be recorded on the daily drill report and, subsequently, geologic log for future reference. In all cases, use of clear water or air as a drilling medium is not allowed in embankment core material. Fluid pump pressure must remain low and pressures carefully monitored when this method is used in or near the embankment core. When starting circulation, pumping should be increased gradually to reduce the occurrence and increase the ability to observe evidence of hydraulic fracturing. A pressure relief valve set to the maximum allowable pressure is required.
- 5. Fluid Rotary Drilling** ASTM D5783 (ASTM 2000c) – Fluid rotary drilling is not a preferred method for drilling in embankment dams, and its use in embankment core material must be approved by FERC prior to use. This drilling method uses a rotary cutting bit with circulation of water or drilling mud (bentonite or polymer). Cuttings are returned to the surface and dropped in settling tanks. Ideal bentonite drill mud mixtures do not exceed 72 lb/ft and have 60- to 70-second marsh funnel viscosities; however, higher viscosities may be necessary where artesian conditions are encountered. Casing is often advanced with the boring. In all cases, use of clear water as a drilling medium should not be allowed in embankment core material. Fluid pressure must be very low and carefully monitored when this method is used in or near the embankment core. When starting circulation, pumping should be increased gradually to reduce the occurrence and increase the ability to maximum allowable pressure is recommended.

Fluid rotary is the preferred method for SPT testing for liquefaction (see ASTM D6066), where it is recommended to keep the hole full of fluid during the test to stabilize sands. Since drilling fluid is being used, this method has a high potential for hydraulic fracturing. Raising and lowering drill bits, sampling tools, and drill rods should be done slowly so as not to induce negative fluid pressures or increase fluid pressures.

- 6. Becker Drilling/Penetration Testing** – Becker drilling is not a preferred method for drilling in embankment dams, and its use in embankment core material must be approved by FERC prior to use. Becker drilling may be one of two methods. The closed bit system advances a closed bit by means of hammering with a double acting diesel hammer. This method frequently is used in coarser grained material where SPT data likely would be invalid. The open bit method advances an open bit by using of the double acting diesel hammer. In this method, disturbed samples may be collected. High-pressure air is forced down the outer annulus of the dual casing system and returned up the inner casing. The returning air carries soil cutting up to the ground surface. Open bit Becker drilling is prohibited when drilling in or near the core section of an embankment dam.
- 7. Wire Line and Casing Advancer** ASTM D5876 (ASTM 2000d) – Wire line and casing advancer systems are not preferred methods for drilling in embankment dams, and their use in embankment core material must be approved by FERC prior to use. These drilling systems use fluid rotary action to remove the cuttings with the exception that the fluid flows up the annulus between the rods and the borehole wall. In all cases, use of clear water as a drilling medium should not be allowed in embankment core material. Fluid pressure must be very low and carefully monitored when this method is used in or near the embankment core. When starting or restarting circulation, pumping should be increased gradually to reduce the occurrence and increase the ability to observe evidence of hydraulic fracturing. A pressure relief valve set to the maximum allowable pressure is recommended. Since fluid is circulated up the annulus between the soil and drill rod, there is increased chance of blocking circulation and possible fracturing. The drill rods act as casing and are equipped with a cutting bit. Either a core barrel or cleanout bit lock into the lead section of the drill rods and is latched by wire line. This results in rapid drilling and reduced rod trip time during coring operations. Some wire line drilling systems have soil core barrels, but their success is limited. Wire line diamond drilling is the primary method of rock core drilling (see ASTM D2113 on Diamond Drilling (ASTM 1999)). Typically, augers, casing, or other methods are used to set a protective casing through the embankment and foundation soils and then fluid rotary drilling is used to core and water test the foundation rock.
- 8. Drill Through/Drive Casing Advancer** ASTM D5872 (ASTM 2000e) - Drill through/drive casing advancers are not preferred methods for drilling in embankment dams and their use in embankment core material should not be considered. The drills have a casing driver (hammer) and a rotary rock bit or down hole hammer that may be rotated through the casing hammer. Down-the-Hole hammers (DTH) and air are used in coarse boulders deposits and hard rock while rock bits and fluids might be used in dirtier gravel cobble soils. One version of DTH, known as ODEX, has a swing out bit which over-reams the

hole for the casing. Air flow to circulate cuttings has to be rather high, but can be reduced by introduction of foam. To minimize fracturing when drilling with air, the drill bit should be held just inside the casing so a protective seal remains at the bottom of the casing. This practice is not possible when using ODEX, which requires the bit to advance before the casing.

- 9. Air Rotary** ASTM D5782 (ASTM 2000f) - Air rotary is not a preferred method for drilling in embankment dams and its use should not be considered in embankment core material. This class of drilling is very similar to drill through drive casing systems except the hole may be left open (uncased) exposing the complete borehole wall to air flow. Without the protection casing provides, the possibility exists for circulation blockage, possible fracturing, and degradation/opening/erosion of any weak seam exposed along the sides of the borehole. One example of this type is the air track drill.

Table 1 – Drilling in Embankment Dams – Drilling Methods

|                             | <b>Drilling Methods</b>               | <b>Restriction</b>                                                                                                                                               | <b>Recommendations</b>                                                                                                                                                |
|-----------------------------|---------------------------------------|------------------------------------------------------------------------------------------------------------------------------------------------------------------|-----------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| Preferred Drilling Methods  | Auger                                 | None                                                                                                                                                             | Raise and lower auger string slowly when fluid in hole                                                                                                                |
|                             | Sonic/Vibratory                       | None                                                                                                                                                             | Core not suitable for higher level laboratory testing                                                                                                                 |
|                             | Cable Tool/Churn                      | Chop and drive variation not allowed                                                                                                                             | Samples are of cuttings and are highly disturbed                                                                                                                      |
| Restricted Drilling Methods | Dual Rotation                         | Approval of drilling method required                                                                                                                             | Monitor fluid pressure closely<br><br>Use pressure relief valves to cap fluid pressure<br><br>Increase pump pressure gradually<br><br>Monitor fluid viscosity closely |
|                             | Fluid Rotary                          | Clear water as drilling media not allowed                                                                                                                        |                                                                                                                                                                       |
|                             | Becker                                | Fluid pressure must be very low                                                                                                                                  |                                                                                                                                                                       |
|                             | Wireline/Casing Advancers             | Bit must not be advanced beyond shoe<br><br>Open bit methods are not allowed                                                                                     |                                                                                                                                                                       |
| Prohibited Drilling Methods | Drill Through/ Drive Casing Advancers | Not allowed in or near the core of embankment dams. Approval of drilling method required for other areas. Will only be considered in extraordinary circumstances | Not allowed in or near the core of embankment dams. Approval of drilling method required for other areas. Will only be considered in extraordinary circumstances      |
|                             | Air Rotary                            |                                                                                                                                                                  |                                                                                                                                                                       |

There are some general procedures that should be followed when using drilling fluids to limit the risk of damage:

- Tools should be sized and designed to minimize the likelihood of the return flow clogging.
- Methods that require the cuttings to flow through a small annulus between the tools or casing and the borehole wall should not be used.
- Fluid discharges from the bit should always be upward, not downward into the formation material or lateral into the sidewalls that could lead to excessive disturbance or erosion.
- Lower and raise drill tools slowly to avoid pressure changes in the drill hole; this is especially important when using tools with restricted annulus space below the groundwater as the pressure changes are more severe and can lead to suction and surging problems.
- Drilling feed rate must be slow enough to avoid crowding the bit and, thus, minimize the chance of inducing fracturing. The bit must be of a design such that pressure buildup is minimized.
- Drilling media properties, pressure, and return should be continuously monitored. A floating needle pressure valve is required to record maximum pressure spikes that can occur instantaneously and are often unnoticed.
- When media circulation is required, a pressure controlled release (“pop off”) valve should be on the pump.
- In some conditions, casing can be advanced ahead of the drilling bit to reduce the risk of hydraulic fracturing by confining the drilling fluids within the casing.
- Great care should be taken during washing of the hole.
- Casing should be pushed or driven and not jetted. Except in special circumstances, casing must precede the drilling.
- When core drilling rock, the embankment or foundation soil above top of rock must be protected and isolated from the circulating drilling fluid. Fractures in the bedrock must be considered as potential flow paths in contact with the overlying soil.
- A pause or suspension in drilling operations (breaks, meals, overnight/weekend, etc.) should not leave the borehole in a critical state that could result in damage to the embankment.

#### **4.4 In-Situ Testing/Sampling**

The actual process of advancing the boring is not the only potential hazard that can lead to hydraulic fracturing and other adverse impacts of the drilling, sampling, disturbance, and

performance of the structure. Raising and lowering drill rods, casing, or other drill steel too quickly can induce significant positive or negative fluid pressures.

In-situ testing that includes applying hydraulic pressures through static head (falling head or constant head permeability tests) or pressure induced head (packer pressure tests, etc.) can result in excessive hydraulic pressures that could lead to hydraulic fracturing. In-situ testing and sampling methods and procedures must be aware of the potential to create these conditions. The Bureau of Reclamations' *Engineering Geology Field Manual* is an excellent reference to assist in determining applied and total hydraulic fluid pressures from in situ tests (BOR, 1998).

#### **4.5 Hole Completion**

All boreholes and other penetrations (including direct push sampling, Cone Penetration Test soundings, Standard Penetration Testing, Becker Penetration Testing, etc.) in and around embankment dams must be sealed after completion. Completing a borehole by backfilling with drill cuttings is not acceptable. There are a variety of acceptable methods to complete a borehole.

All boreholes and similar penetrations in the impervious portions of an embankment dam and their foundations must be backfilled by tremie-placed cement-bentonite grout or bentonite pellets/chips, except when an alternative backfill method compatible with instrument installation is approved. The drilling plan must address the possibility of confined and separate groundwater aquifers and demonstrate safe completion which avoids cross-contamination and leakage. The grout must be designed to obtain strength equal to or greater than the soil or rock. Note that some instrumentation installations may require additional considerations for the grout strength. Gravity grouting techniques should be used for backfilling boreholes.

For borings that penetrate zones with low confining stress it is possible to induce hydraulic fracturing even from gravity pressure alone. When grouting borings in these locations or if significant grout losses are observed, the grout backfilling should be done in stages allowing the grout to set between stages.

For pervious portions of the dam (drainage features, filters, etc.), the borehole must be backfilled by tremie placement of granular materials that are sized to provide drainage without being susceptible to migration through the pervious embankment or foundation materials or segregation during placement.

Lutenegger, et.al. (1995) is a good source for borehole backfill guidelines.

Special procedures and materials may be required for installation of instrumentation in boreholes.

Borehole completion is often not well documented. Recommended inclusions in borehole completion documentation include intervals of various backfilling materials, calculated volume of material necessary to fill each interval, and actual volume of material required to fill each interval. Detailed records of borehole completion are important and, as in the case of backfill

material volumes significantly higher or lower than calculated, may be indicative of conditions significantly different than anticipated.

Below are some general guidelines that can be considered in borehole completion.

- **High Solids Bentonite Grout** - Tremie grouting with high solids bentonite is an acceptable method of completing boreholes in embankment dams. Mixes which yield 20 to 30 percent solids should be used. Stage up tremie grouting methods should be used in the embankment with the casing (i.e. hollow-stem augers, rods, etc.) pulled incrementally to ensure hole wall stability. The bentonite slurry should always be injected through a tremie pipe to ensure the best possible placement and most thorough borehole completion.
- **Neat Cement Grout** - Neat cement grout is another acceptable method of completing boreholes in embankment dams. The best results are achieved when the mix consists of 5 to 7 gallons of water to one sack, 94 lbs of Type I or Type II Portland cement (using higher water contents may result in excessive shrinkage, cracking, and bleed water). Commonly, the addition of up to 3 percent powdered bentonite by dry mass of cement is used for pumping ease and to reduce shrinkage and cracking after curing – although a myriad of other compounds are also available. Additives such as calcium chloride or carboxylic acid can be used to control set times, but shrinkage factor must be considered. Using type K cement or adding up to 1 percent gypsum or aluminum powder by weight will give the cement expansive properties, which may be advantageous in embankment dams where internal seepage is an issue. As with the bentonite grout, stage up tremie grouting methods should be used in the embankment core with the casing pulled incrementally to ensure borehole wall stability. The grout should always be injected through a tremie pipe to ensure the best possible placement and most thorough borehole completion.
- **Bentonite Pellets or Chips** - The use of bentonite pellets or chips may be an acceptable method of completing boreholes in embankment dams. However, there are some conditions under which bentonite pellets or chips should not be considered and only tremie grouting is acceptable. Bentonite pellets or chips, including those treated to retard or delay flocculation, should not be used in cases where there is a chance the depth of water in the hole could slow the bentonite fall and allow flocculation prior to the bentonite reaching hole bottom. Additionally, even in a dry hole, there must be adequate annular space available to allow the bentonite to fall to the borehole bottom without bridging. It is advisable to always place both solid bentonite and grout through a tremie pipe.
- **Instrument Installations** - Instrumentation installations require special completions. For piezometers, sand packs are placed in the influence zone and a bentonite seal is placed above the sand pack to prevent any contamination of the sand pack from sealing materials placed above it. The bentonite seal is typically bentonite pellets. A common error in placing the seal is not allowing bentonite time to hydrate. Pellets should be allowed a minimum of 1 to 2 hours to hydrate prior to placing additional backfill material above the

seal. Alternatively, piezometers can be installed in fully grouted holes. While it is possible to place two piezometers in a typical 4-inch inside diameter hollow stem auger or casing, only one piezometer is recommended, and no more than two instruments should be allowed in a single boring. Difficulty in providing a good seal between multiple riser pipes may result in communication between influence zones. Other instrument installations (slope inclinometer casing, geophysical casing, etc.) will require additional considerations.

#### 4.6 Drilling Personnel

Because of the potential to do harm, drilling in a dam should only be performed by experienced and qualified personnel. This includes the lead drill rig operator and the engineer or geologist who is the on-site representative responsible for the drilling program and the safety of the dam. Schedule, budget, and other issues should be considered secondary to the safety and integrity of the structure and those potentially impacted by its compromise.

Drill rig operators must have a minimum of 5 years of experience drilling with the equipment and procedures described in the drilling program. When the drilling plan includes drilling in or in the vicinity of dam or appurtenant structure foundations or abutments or within an embankment dam, the drill rig operators must have demonstrated embankment dam drilling experience clearly indicated in their resume.

All drilling activities must be conducted in the presence of a qualified geotechnical engineer or engineering geologist who will be responsible for maintaining the integrity of the structure and the inspection of the drilling operation. Qualified is by combination of education, training, and experience as indicated in Table 2.

Table 2 – Minimum Qualifications of Responsible On-site Personnel

| <b>Factor</b> | <b>Low Hazard Dams</b>                                                                                                                            | <b>Significant and High Hazard Dams</b>                     |
|---------------|---------------------------------------------------------------------------------------------------------------------------------------------------|-------------------------------------------------------------|
| Education     | Minimum B.S. in Civil Engineering or Geology (or licensed as a professional engineer, professional geologist, or certified engineering geologist) |                                                             |
| Training      | Independent study or formal training in the identification and mitigation of drilling hazards in embankment dams                                  |                                                             |
| Experience    | Minimum of two years of general drilling experience                                                                                               | Minimum of four years of embankment dam drilling experience |

While there are many inspectors with significant years of experience with drill procedures, classifying soils and rock, and in-situ testing methods, they may only have limited knowledge and experience with dams and may be unaware of potential damage to critical dam features caused by certain drilling procedures. Therefore it is critical that a combination of education,

training, and experience be demonstrated and clearly shown on the resume of the geotechnical engineer or geologist inspecting the work.

The project manager directing the drilling program must also be an experienced geotechnical engineer that is a licensed professional engineer or a licensed professional geologist or certified engineering geologist with at least ten years' experience in dams-related work.

Both the drill rig operator and the on-site geotechnical engineer/engineering geologist must also be familiar with these guidelines. It is essential that drill rig operators and the geotechnical engineer/engineering geologist be well trained and aware of the causes of and the problems resulting from hydraulic fracturing and artesian conditions and have the equipment, materials, and experience to correct and remediate damage to the embankment and foundation.

#### **4.7 Other Considerations**

**Emergency Communications** - No dam should be drilled or investigated without a thorough review of the Emergency Action Plan (EAP). FERC-regulated dams have EAPs in place. The EAP lists the key individuals who should be contacted and informed of proposed activities. There are documented case histories where drilling has caused incidents with dams and knowledge of the EAP and good communications were key contributors to safely solving the problems.

**Monitoring** - During drilling operations, the dam embankment should be continuously inspected and monitored using appropriate procedures and instrumentation at the dam site. The proposed monitoring should be used to evaluate any impacts from of the drilling activity and assist in detecting any unanticipated changes. The type of monitoring (piezometer, inclinometers, etc.), frequency of readings, and purpose for monitoring should be carefully considered. If appropriate, threshold limits could be determined for specific drilling scenarios. It may be necessary to perform daily inspections of the dam for a period of time after the drilling operations have concluded.

**Reporting** - All incidents of damage or potential damage related to drilling and associated activities for dams must be reported. If a sudden loss of drill media occurs during any embankment drilling within the core, drilling must be stopped immediately. Action should be taken to stop the loss of drill fluid. The reason for loss should be determined and if hydraulic fracturing may have been the reason for the fluid loss, FERC should be notified immediately.

**Construction/Remediation Drilling Activities** - Drilling activities performed during construction or remediation phases are often overlooked as opposed to drilling that occurs under the traditional exploration phases. There are numerous examples of dams which required remediation after reservoir filling and the embankment or foundation was damaged. Many of these dams required remedial grouting immediately after construction, and the grouting contractor used air drilling, rapidly resulting in fracturing of blankets and foundations. Jet grouting contractors drill holes with very high air/fluid pressures at rapid rates. Contractors want to drill fast, but drilling fast may cause blockage and loss of the circulating fluid and hydraulic fracturing. It is imperative that, for remediation construction projects, and instrumentation

installation contracts, project geologists and engineers identify drilling methods and confirm they are appropriately screened to avoid damage to the dam or foundation. If there is concern, a team should be formed to review the drilling methods and ensure the contract documents have appropriate provisions to avoid damage to the dam and foundation.

**Exemptions** - Drilling required for immediate emergency measures where delays required to develop the drilling plans and to obtain the necessary reviews and acceptances would result in unacceptable risk of damage or failure, may be exempted from the requirements to prepare a drilling plan, as approved by the Regional Engineer. Emergency drilling should be appropriately expedited but should follow the general guidelines presented in this guideline.

#### **4.8 Evaluation of Potential Risks**

The licensee must thoroughly evaluate the risks associated with the proposed drilling and indicate how they intend to mitigate them. Among other topics, the potential risks of causing hydraulic fracturing of the embankment, as well as the potential risks of causing seepage, instability, or other potential dam safety issues as a result of the proposed drilling program must be evaluated and addressed. The risk evaluation must include an assessment of the potential impact of the drilling operations and the location of the boreholes in relation to areas of the dam that may be more susceptible to hydraulic fracturing, as discussed in Section 4.2.1.

Aside from comparing the planned drilling locations with the areas of the embankment and soil types that are more susceptible to hydraulic fracturing, the proposed drilling procedures must also be evaluated with respect to their likelihood of causing hydraulic fracturing or other dam safety issues. This includes the instrumentation installation procedures, borehole completion/abandonment procedures, and emergency procedures if a potential dam safety issue is identified during the drilling. Special attention should be given to highlighting the specific procedures and contingency plans that will be utilized to protect the dam from potential hydraulic fracturing and other potential risks.

#### **5.0 DRILLING PROGRAM PLAN (DPP)**

An approved Drilling Program Plan (DPP) is required for any exploration drilling, instrument installation, or remediation drilling (including grouting) work to occur on an embankment dam, in proximity of the dam in which the drilling methods could pose a risk to the dam, or the dam's foundation and abutments. DPPs shall be prepared and reviewed by experienced geotechnical engineers and/or engineering geologists familiar with subsurface exploration techniques and methods. It is paramount that all existing subsurface information is thoroughly evaluated and understood by the exploration team prior to developing a plan for additional drilling. In order to understand and communicate subsurface conditions and estimate drilling risk, the existing subsurface information must be assimilated into essential plan and section drawings showing proposed drill holes and depths, target sample areas and proposed instrumentation. The DPP must also comply with good environmental practices and comply with site environmental provisions/restrictions, which may need coordination with DHAC and outside agencies.

The DPP must be reviewed and accepted by the FERC Regional Engineer prior to beginning the

drilling program. Depending on the particular dam and scope of the project involved, the review process may also require additional coordination with FERC headquarters staff in Washington, D.C. and/or DHAC. As stated in our Annual Letter, this plan must be submitted for our review a minimum of **30 days** prior to beginning the drilling work. However, licensees are encouraged to inform the FERC project engineer of the planned drilling program and begin discussions with him or her regarding the proposed drilling well in advance of this deadline.

In addition, the licensee is encouraged to set up either a face-to-face meeting or conference call with the Regional Engineer and headquarters staff, as appropriate, once the specifics of the proposed drilling program have been developed. Ideally, this meeting should take place as soon as possible but no later than *a minimum of two weeks* prior to submission of the DPP. The purpose of this meeting and early coordination with the FERC project engineer is to ensure that both the licensee and FERC share a common understanding of the requirements of the project and the DPP, and there are no delays associated with FERC's review or potential issues with the plan.

FERC's primary concern in evaluating the licensee's DPP will be ensuring that the planned drilling program will "do no harm" to the existing dam. A thorough, well-organized, and well-developed DPP, including the various items highlighted in these guidelines, will assist FERC in its review by demonstrating that the licensee fully understands the risks associated with the drilling program, and is taking the appropriate measures to mitigate them.

In general, the DPP must include the following information, as a minimum:

1. Name and description of project.
2. Purpose of site disturbing activity.
3. Description of the proposed site exploration activity (drilling, test pitting, etc.). Include plan view showing location of activity (ies), proposed drill hole depths, sampling intervals, insitu testing, and instrument installations.
4. Describe and show anticipated site conditions. Show location of known subsurface conditions and features. Describe subsurface units. Describe understanding of ground water conditions and phreatic surface, including the potential to encounter artesian conditions. Use cross sections and profiles to graphically illustrate.
5. Describe proposed equipment, methods, and processes. For example, for any activity that introduces a fluid in or near the water retaining feature or its foundation, detail how fluid pressures will be measured and monitored. For example, for falling head permeability tests, show how the introduction of a column of water will not cause excess water pressures in the embankment that could lead to hydraulic fracturing. Likewise, for grouting of boreholes, describe how if staged grouting will be required and how the maximum height of grout column will be determined to prevent hydraulic fracturing.
6. Identify project personnel and qualifications/experience, including resumes.

7. Risk identification and mitigation plan. Identify and describe potential risks imposed by site disturbing activities. Identify and describe risk mitigation plan. For example, for any activity at the toe of a water retaining feature, describe the risk mitigation plan should unexpected artesian conditions be encountered.
8. Identify communication plan with names and phone numbers. Include a list of emergency equipment and supplies to have on site (phone/radio, filter materials, grout materials, light plant, etc.).
9. Provide an overall schedule and duration of drilling activities.

Specific requirements for the DPP are included in Appendix B.

## **6.0 REPORTING REQUIREMENTS**

The DPP should provide details on the documentation, logging, and submission of drilling data. The field inspector's boring log should be submitted to FERC within 24 hours after completion of backfilling the boring. When feasible, draft field boring logs should be submitted daily, along with daily work logs. Since there is always a possibility that some changes will need to be made in the field due to the specific subsurface conditions encountered, the DPP should describe how changes and deviations from the approved DPP will be communicated and coordinated with FERC. Also, any significant differences from expected conditions which could be an indication of a potentially serious dam safety issue must be reported immediately to the FERC Regional Engineer.

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## APPENDIX A

### ADDITIONAL HYDRAULIC FRACTURING REFERENCES

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## **APPENDIX B**

### **DRILLING PROGRAM PLAN (DPP) OUTLINE FOR DRILLING IN AND NEAR EMBANKMENT DAMS**

Drilling in and near embankment dams must subscribe carefully to the “do no harm” philosophy. Dams are not places for standard investigation techniques; they require different drilling procedures because there are significant risks. Incorrect drilling, grouting, or borehole abandonment procedures could lead to damage of the structure by hydrofracturing, erosion, drain contamination or other mechanisms, during drilling operations, instrumentation installation, borehole completion, and borehole abandonment. All design and field personnel need to understand the existing subsurface conditions and potential problems and damage that the drilling could trigger.

An approved Drilling Program Plan (DPP) is required for any exploration drilling or remediation drilling (including grouting) work to occur in or near an embankment dam. DPPs shall be prepared and approved by experienced geotechnical engineers and/or engineering geologists familiar with subsurface exploration techniques and methods.

The following outline describes the basic information that should be developed and included in the DPP that is to be submitted by the licensee. Additional information, discussion, and recommendations on the items presented in Appendix B are provided in the guidelines. It is strongly recommended that the DPP follow the following organizational structure.

#### **1. Purpose**

The purpose of the drilling program needs to be clearly defined and summarized in the plan. The DPP should provide sufficient discussion, details, and figures to ensure that the proposed exploration will accomplish its goals and prevent damage to the dam. The need for any investigation (drilling, testing, etc.) at a dam site should have been presented to and accepted by FERC prior to developing a DPP program. In addition, it should have been demonstrate that any potential damage to the structure created by the drilling and associated processes is outweighed by the need for the drilling data.

#### **2. Existing Information**

Before preparing a DPP, the licensee or its consultant should review the subsurface, design, and construction information available in the Supporting Technical Information Document (STI) and their files to properly evaluate the risks associated with the proposed drilling program. The information review typically includes, but is not limited to:

- Subsurface profiles and piezometric conditions;
- Geologic mapping, logs, and reports portraying information from previous investigations and construction;
- Foundation reports;
- Embankment construction reports;

- As-built drawings;
- Archived records;
- Construction reports;
- Construction photos;
- Instrumentation plans; and
- Available laboratory analyses.

Based on this review, a summary of the existing information should be included in the DPP.

### **3. Essential Geologic and Engineering Drawings**

The DPP should include a complete set of drawings depicting the current subsurface conditions. This detailed set of foundation and embankment drawings typically requires a plan drawing showing all previous subsurface investigation locations, profile drawings, and sections of the embankment in the areas of proposed exploration. The sections should be drawn to scale (no vertical exaggeration) and should show the locations and depths of the proposed borings along with all available factual information and appropriate geologic or engineering interpretations. The information on the plan, profile and sections should be detailed, include all available data significant to the planned explorations, and be supplemented by additional discussion in the text of the DPP, as appropriate. At a minimum, the following information should be included, as applicable:

- Embankment zones, including added berms, filters, blankets, and drains;
- Estimated extent of any other zones of interest;
- Details of subsurface material classifications, including relevant laboratory test results such as Atterberg Limits, grain size analyses, and dispersivity test results, as applicable;
- Geologic contacts and continuity supported by all nearby drilling and sampling details;
- Contours of the top of rock or any other layer of particular interest;
- Piezometer locations showing screened influence zones and recorded piezometric levels tied to the reservoir water level. Whether or not the dam includes active piezometers, the estimated phreatic surface through the embankment should be clearly shown on all relevant cross-section drawings included in the DPP. In addition, the basis for determination of the estimated phreatic surface should be clearly described in the DPP.
- Inclinator locations showing any shear zones or areas of deformation;
- Standard Penetration Test (SPT) blow counts or other in-situ test results;
- Geophysical data, where useful (e.g. downhole and/or crosshole shear wave velocity profiles);
- Seepage areas tied to geologic units; and
- Location of all structures, including seepage control features, outlet works, etc.

### **4. Drilling Scope and Methodology**

The plan should thoroughly describe the scope and methods that will be used for the drilling program. At a minimum, the following information should be included:

- Number, location, depth, diameter, and inclination of the proposed borings;
- Drilling and sampling methods, including a description of the drilling equipment to be used (e.g. track-mounted vs. truck-mounted drill rig). The DPP should include justification for the proposed methods and equipment based on the expected subsurface conditions. In particular, if any drilling fluids will be used to advance the borings, the DPP must include a detailed explanation of why these procedures must be used, how the potential for hydraulic fracturing will be mitigated, and how continuous monitoring of the fluid pressures will be accomplished during the drilling. The allowable fluid pressures so as to prevent hydraulic fracturing should be included in the DPP, along with supporting calculations, as appropriate.
- List of ASTM standards and methods that will be followed to perform the drilling.
- Anticipated materials to be drilled and sampled;
- Required sample types (disturbed or undisturbed), sizes, and anticipated depths;
- Procedures for identifying underground utilities, and other surface or subsurface obstacles prior to the drilling; and
- Site Access and accessibility of the boring locations (see paragraph 11). .

## **5. Field and Laboratory Testing Program**

The DPP should provide information on the proposed testing program, which should include both field and laboratory testing. A detailed description of the in-situ testing proposed at each boring should be provided, including the type, location (depth), and specific testing method(s) (i.e. ASTM standards, etc.) to be used. The plan should also describe the anticipated laboratory testing program.

## **6. Instrumentation Installation**

If instrumentation is being installed in one or more borings, the materials, location, and procedures that will be used to construct and install the proposed instrument should be described in the DPP. Appropriate figures including installation details for the instruments should also be drafted and included in the plan. For piezometers and monitoring wells, these details should include the following items, at a minimum:

- Installation depth;
- Pipe material type, length, and diameter, as well as the methods that will be used to centralize the pipe;
- Depth of screened interval and the slotted screen size;
- Type, gradation, depth range, and annular thickness of the filter/drain pack material. The DPP must demonstrate that the proposed filter/drain pack material will adequately meet filter and drainage compatibility criteria with both the surrounding embankment soils and the slotted screen size of the piezometers/wells.
- Type, mixture, depth range, and annular thickness of the bentonite or cement grout seal, as applicable;
- Procedures for monument installation or other near-surface (i.e. within the upper five feet) abandonment methods, as applicable; and
- Procedures for developing the piezometers/wells. In particular, if water or air pressures

will be introduced, the DPP must include reasons why these pressures must be used in order to develop the piezometer/well and indicate how this will be implemented so as to avoid causing any damage to the piezometer/well or surrounding embankment. The DPP must indicate how continuous monitoring of the fluid pressures will be accomplished during the development process, state an allowable fluid pressure that will not be exceeded, and include supporting calculations, as appropriate.

## **7. Monitoring**

The DPP should provide details on any proposed monitoring and evaluation of the drilling activity. The plan should describe the type of monitoring (piezometer, inclinometers, etc.), frequency, and purpose for monitoring. If appropriate, threshold limits could be determined for specific drilling scenarios.

## **8. Emergency Procedures**

A discussion should be provided as to what materials and methods will be used to prevent damage to the dam should problems such as loss of drilling fluids, artesian pressures or seepage be encountered during the explorations. The plan should include an emergency contact list and personnel notification flow chart.

## **9. Borehole Completion**

All boreholes in and around embankment dams should be sealed after completion. Completing a borehole by backfilling with drill cuttings is not acceptable. The proposed materials (grout mix) and field procedures that will be used to backfill the borehole should be described in the DPP, along with the estimated quantities required to backfill the borehole. Additional information on backfilling of boreholes is provided in the guidance.

## **10. Personnel Experience**

The DPP should clearly indicate the specific personnel that will be on site either performing or observing the drilling work, and their respective roles and responsibilities. Resumes for all of the relevant project personnel (including the project manager, field geologist/engineer, and lead driller) should be included in the DPP or submitted prior to start of work. The level of experiences required for each of the specific personnel performing the work is described in the guidelines.

## **11. Site Access, and Environmental Consideration**

The DPP should include information on the proposed procedures to access the boring locations, which may include details for constructing and maintaining access roads and for mitigating any adverse impacts that might be caused by its construction. The DPP, if applicable, should address any adverse impact to the embankment stability or seepage from the construction of access roads within the footprint of the dam. For access roads which will be constructed through areas of previously undisturbed ground, additional consultation with FERC's Division of Hydropower

Administration and Compliance (DHAC) will be required prior to FERC approval of the DPP. The DPP should describe the procedures for identifying underground utilities, and other surface or subsurface obstacles prior to the drilling.

## **12. Documentation and Coordination**

The DPP should provide details on the documentation, logging, and submission of drilling data. Since there is always a possibility that some changes will need to be made in the field due to the specific subsurface conditions encountered, the DPP should describe how changes and deviations from the approved DPP will be communicated and coordinated with FERC. Also, any significant differences from expected conditions which could be an indication of a potentially serious dam safety issue must be reported immediately to the FERC Regional Engineer.

In addition, the DPP should include an overall schedule and duration of drilling activities.

## **13. Evaluation of Potential Risks**

The DPP must document the licensees' assessment of the risks associated with the proposed drilling and indicate how they intend to avoid or mitigate them. Among other topics, this section should address the risks of causing hydraulic fracturing of the embankment, as well as the risks of causing erosion, blowout, contamination of drainage materials, or other potential dam safety issues as a result of the proposed drilling program. The DPP should also outline the nearby instruments whose behavior will be monitored during the investigation, their expected response, and contingency plans for unexpected response.

# Lake Whetstone Dam Toe Drain Repair

Public Meeting

February 16, 2021



**DEPARTMENT OF  
ENVIRONMENTAL  
PROTECTION**  
MONTGOMERY COUNTY • MARYLAND

## Agenda

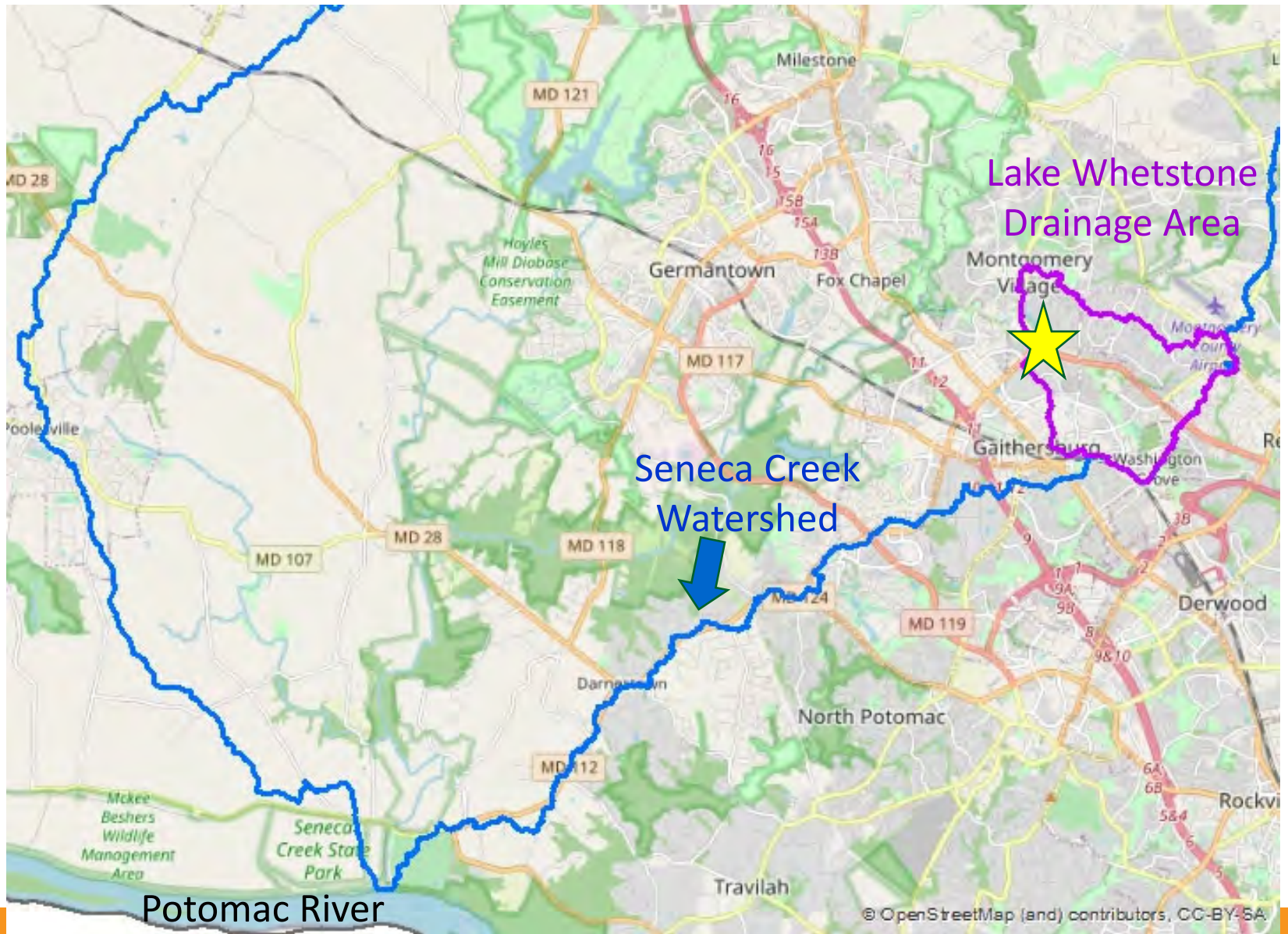
- What is the Project?
- Construction Location
- Toe Drain Repair Overview
- What to Expect
- Questions



# Project Location



# Managing Stormwater Protects Streams



# County Stormwater Facility Maintenance

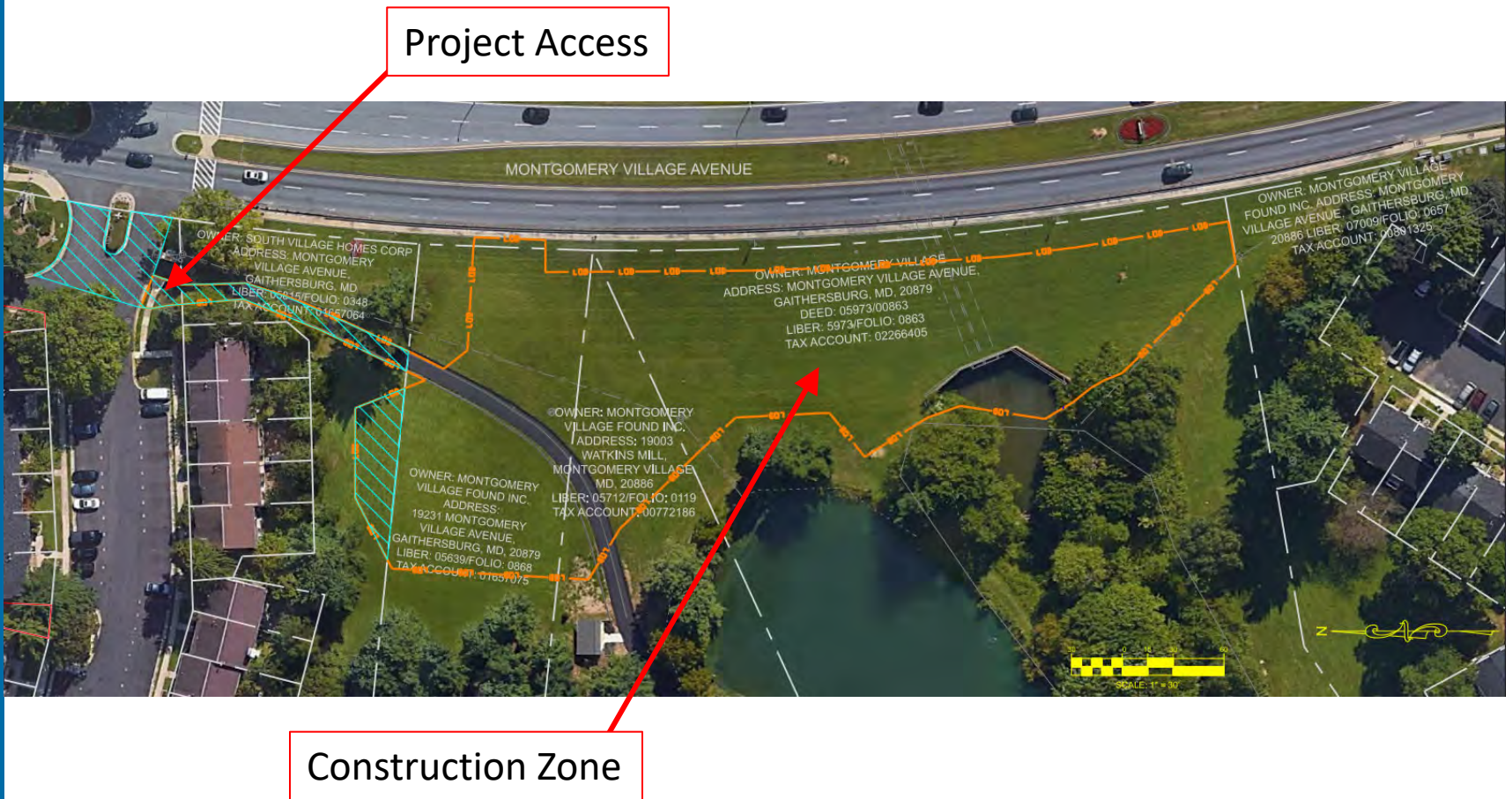
- Over 14,000 stormwater facilities inspected by County
- Capital Projects – Structural Maintenance
  - Dredging
  - Slip-lining
  - Dam failures
    - Barrel replacements
    - Riser replacements
  - Safety improvements
  - Retrofits
  - Repairs





# Project Impact Limits

- Project will replace drains over the course of several months
- Construction zone shown in orange (limit of disturbance)



# Project Impact Limits

- Temporary orange construction fence will be installed around the lake during the project work
- Lake will be drawn down by about 4 feet from normal levels

NO LAKE ACCESS DURING CONSTRUCTION

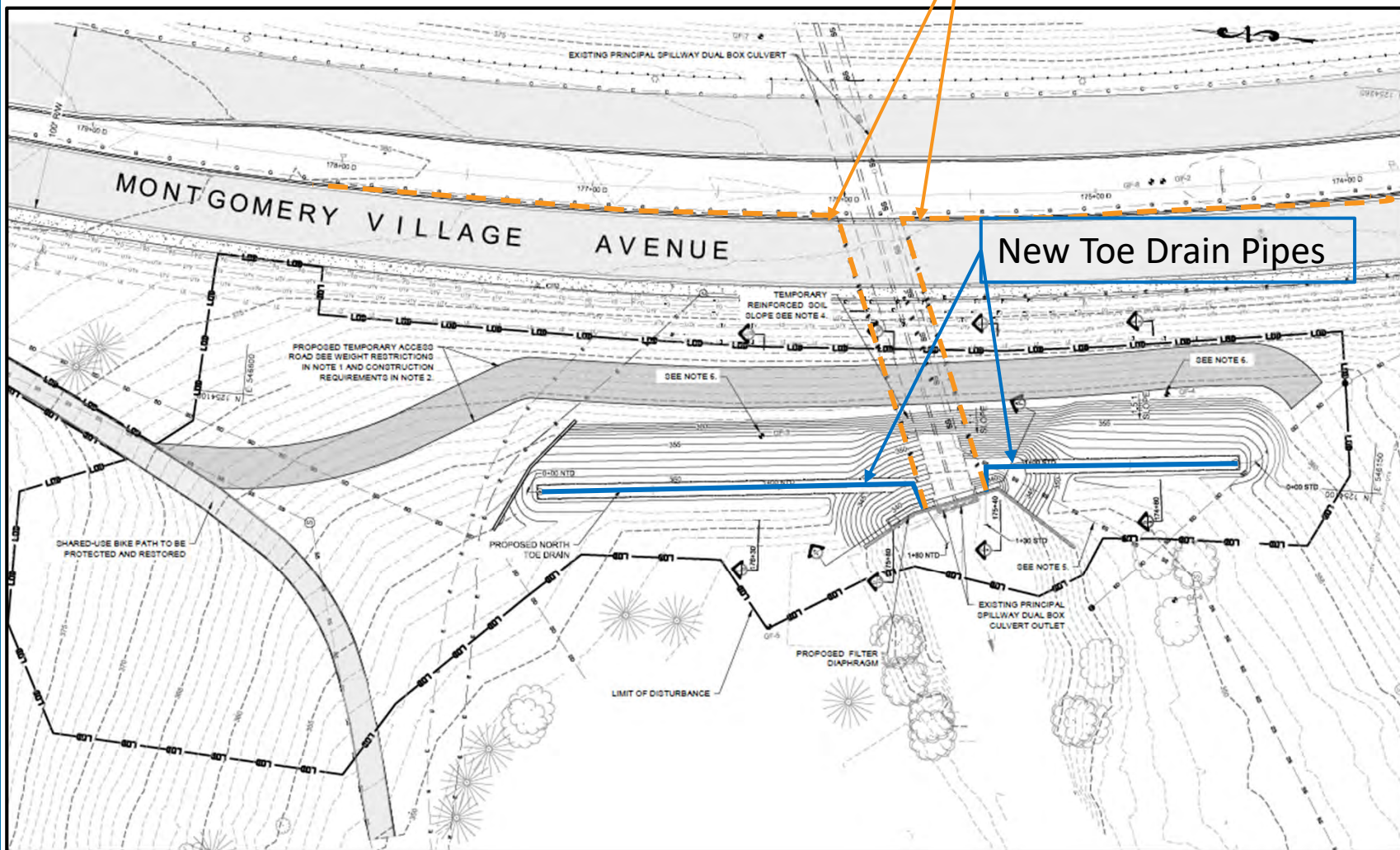
Temporary Orange  
Construction Fence



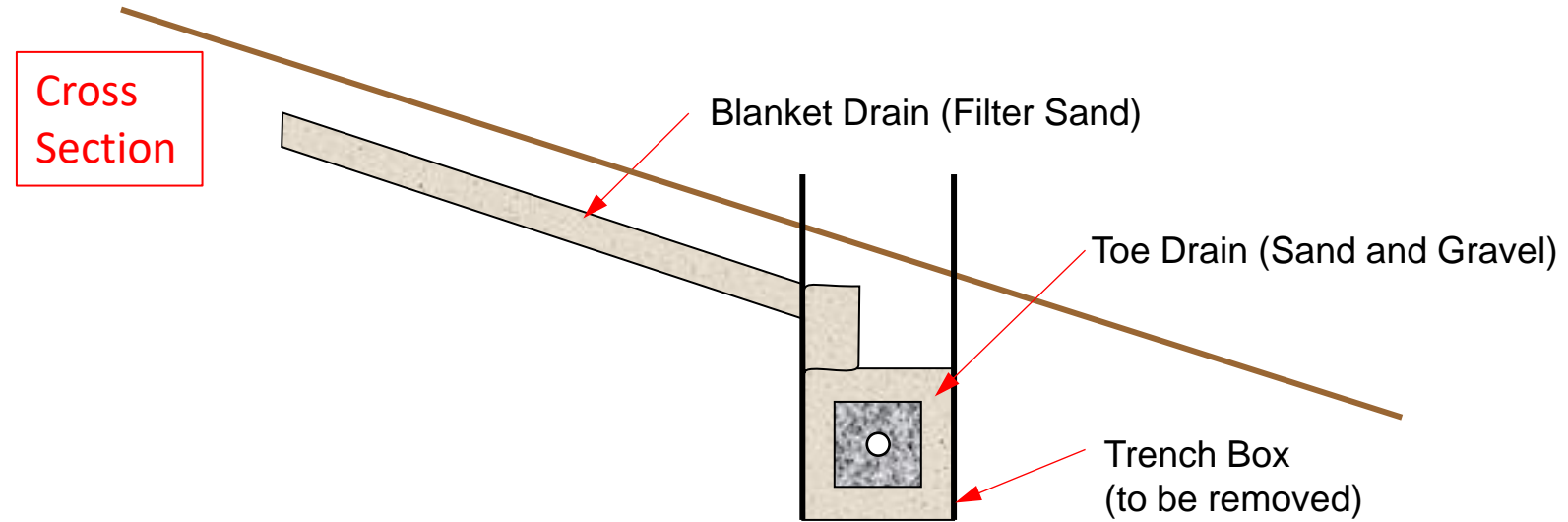
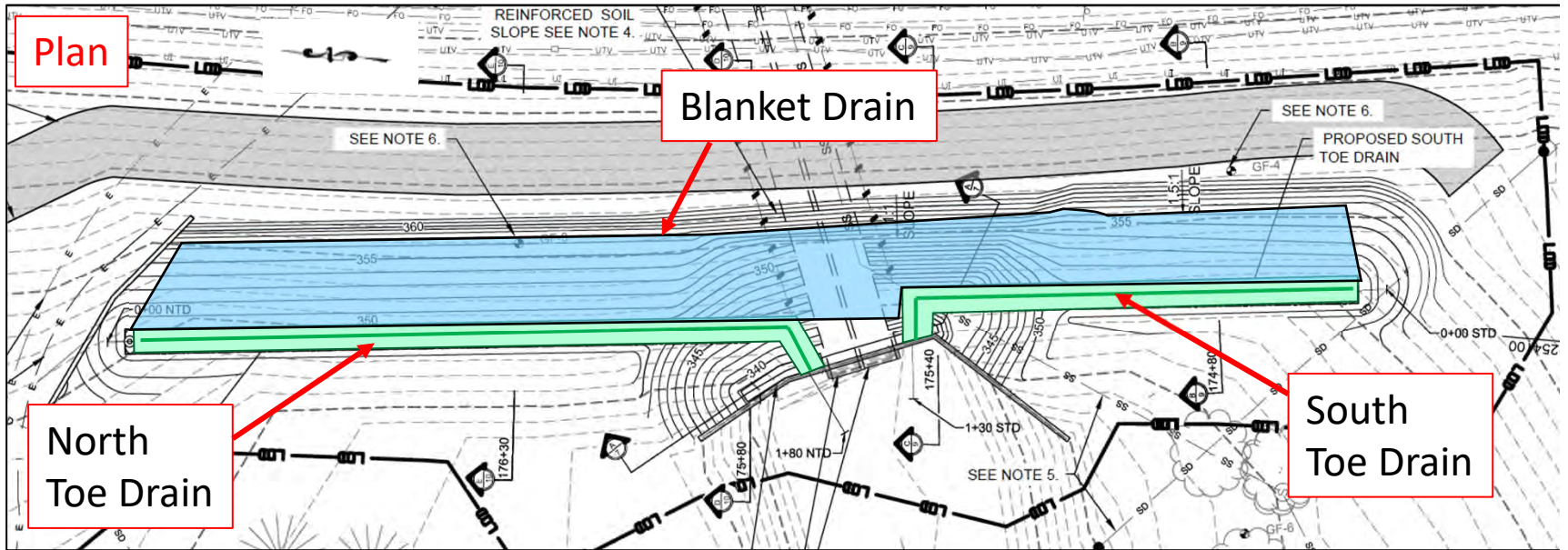
# Toe Drain Pipe Location

Existing Drain Pipes will be Abandoned

New Toe Drain Pipes



# New Toe Drains and Blanket Drain



# Drain Construction Example

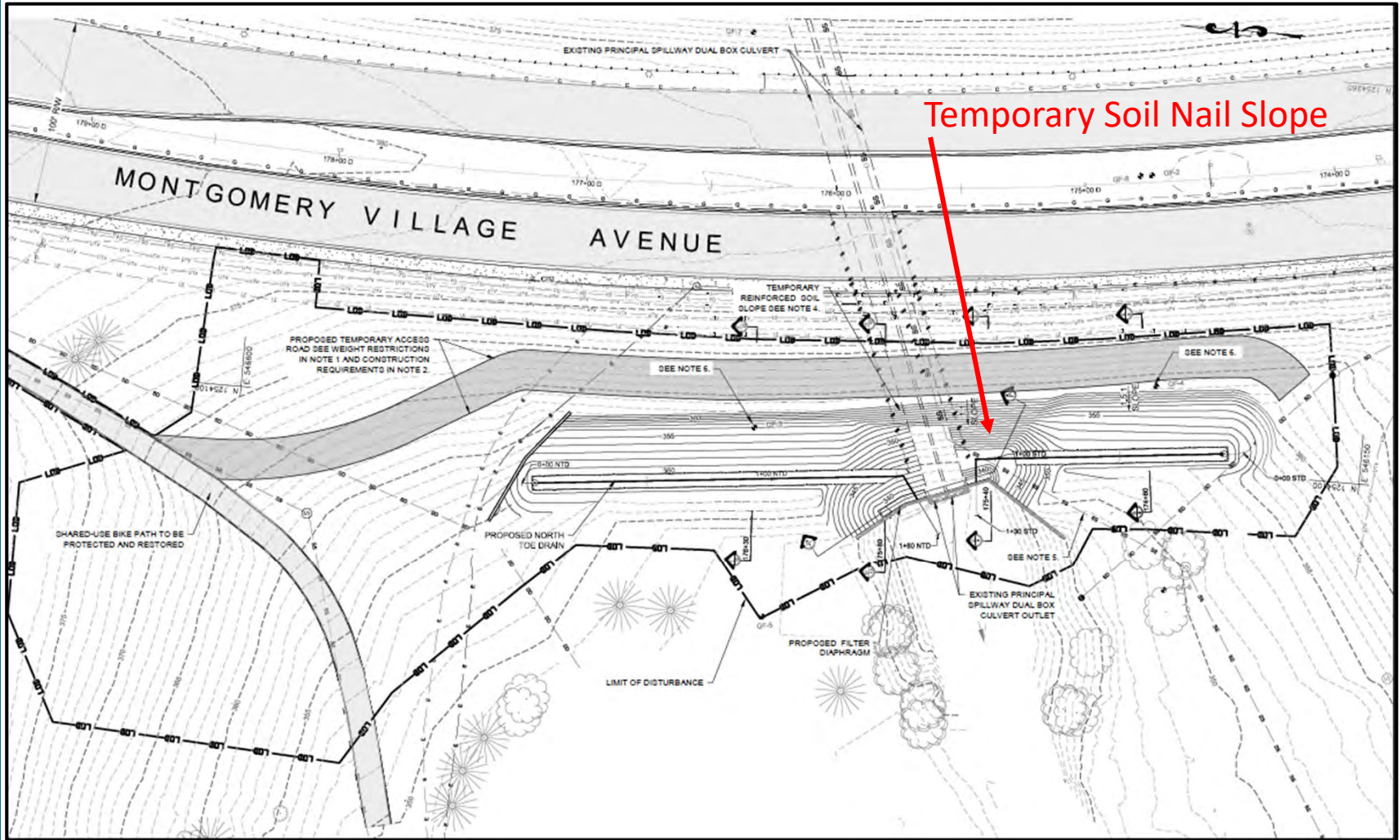


# Toe Drain Pipe Outlets



New Toe Drain Pipes to be installed through the Headwall (Same Location as Existing Drains)

# Drain Excavation Temporary Stabilization

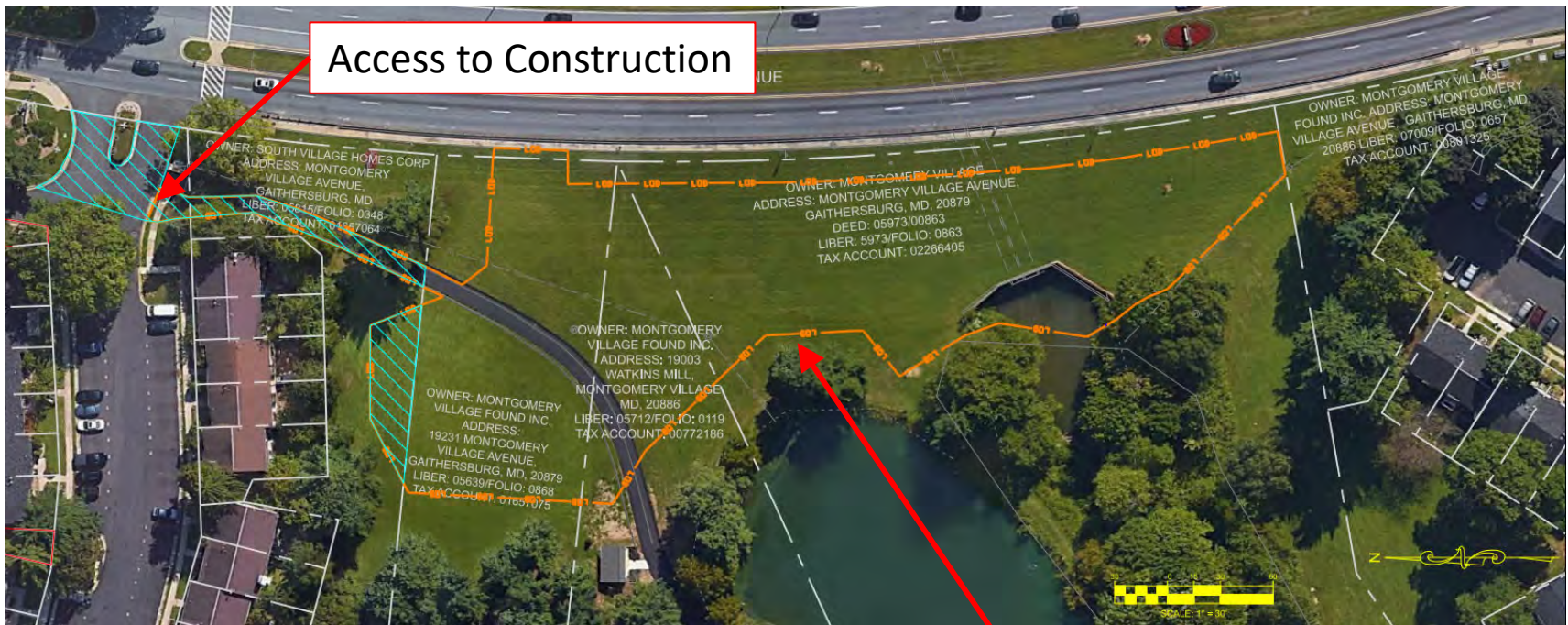


# Soil Nail Construction Example



# Project Impacts: Access to Work Area

- Construction zone shown in orange (limit of disturbance)
- Access to construction zone shown in blue (temporary easement)

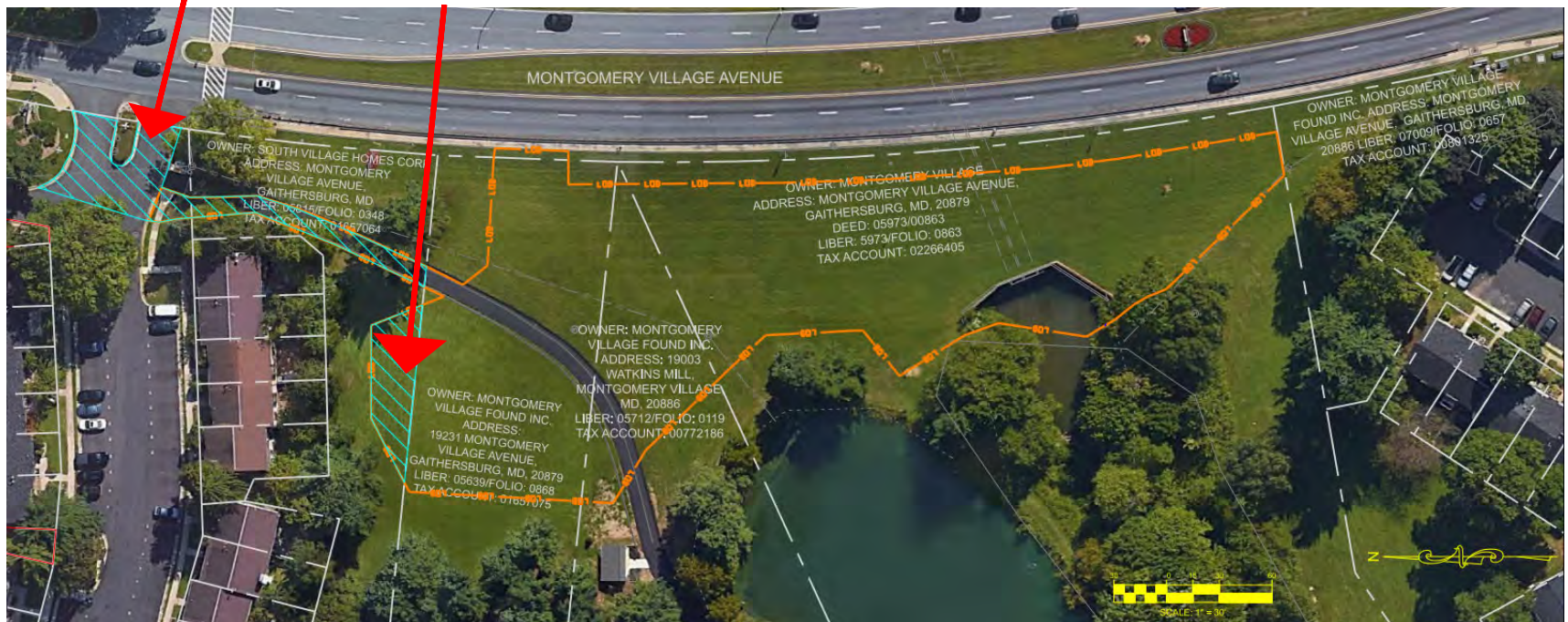


Construction Zone

# Project Easements

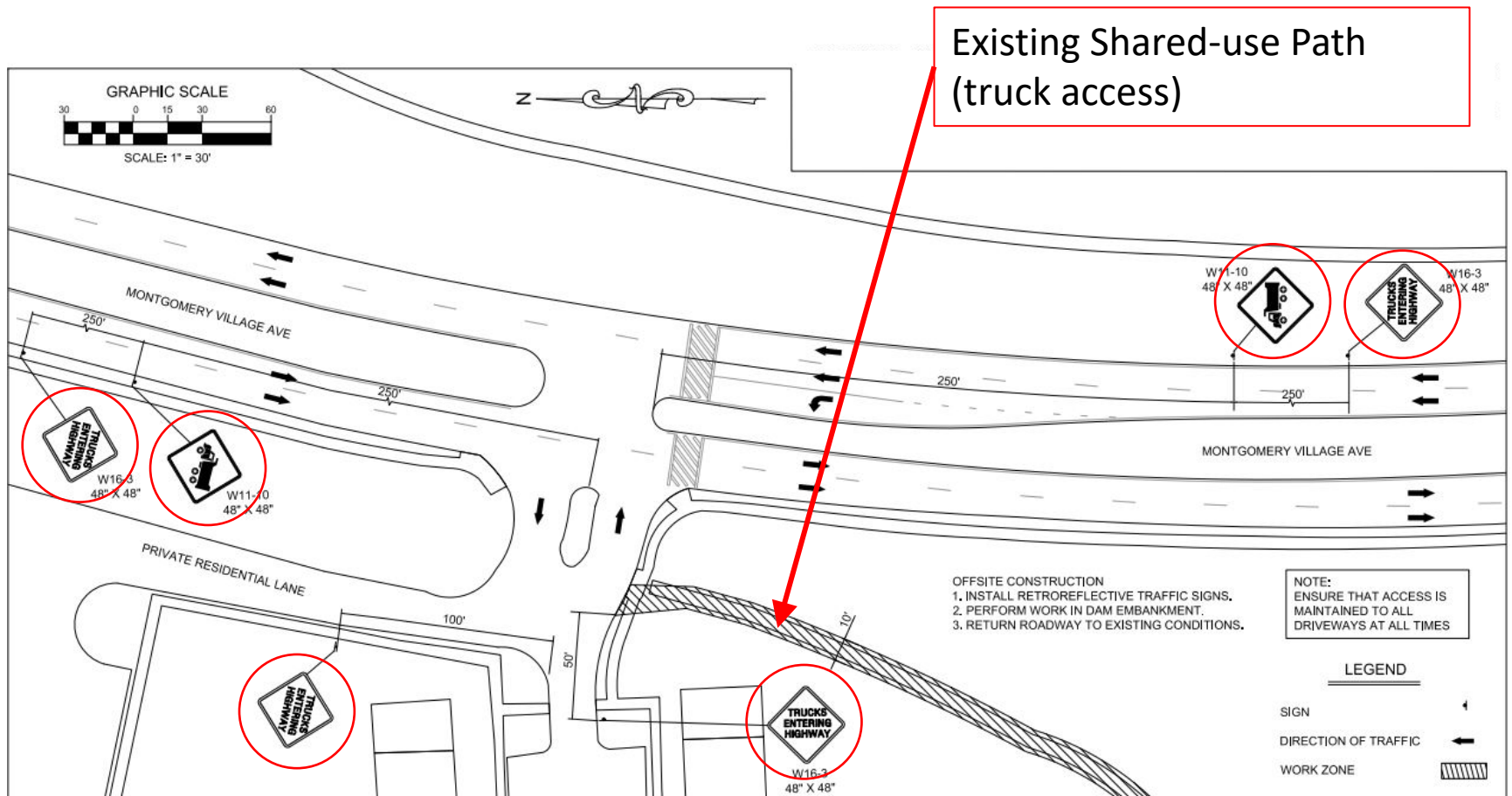
- Temporary construction easement has been granted by South Village Homes (blue hatching below)

Temporary Easement



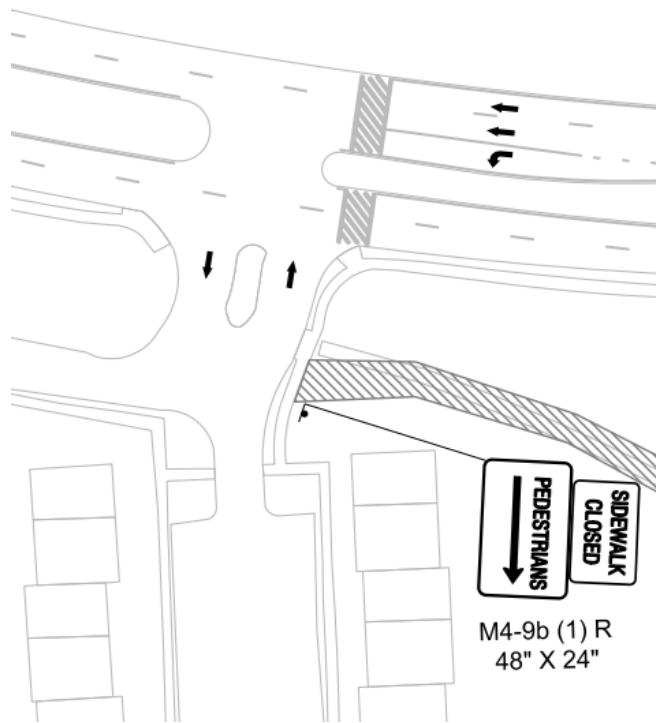
# During Dam Repair

- Trucks access from Montgomery Village Ave. via existing shared-use path
- Warning signs along Montgomery Village Ave and community side roads
- No travel lanes or sidewalk will be closed.



# Shared-Use Path Closure

- Shared-use path will require a detour for several months
- Sidewalk connectivity through neighborhood will be maintained





# Toe Drain Repair Schedule

- ✓ Design and Permitting      Completed
- ✓ Permits Secured              Completed - November 2020
  
- Project Bidding                Bids Received – Processing Award
  
- Construction                  June to December 2021

# What to Expect During Construction

- **Resident Notification**

- MVF Newsletter
- Project Sign

- **Duration**

- About 6 months

- **Construction Hours**

- Monday through Friday, 7AM – 4PM

- **Safety**

- Fencing around construction and lake – no lake access
- Follow applicable COVID-19 precautions at time

- **Traffic**

- Minor from entering and exiting construction traffic and contractor parking during the day
- Pedestrian Detour to Pavilion
- WSSC – separate project



## Other DEP Programs for Residents

- RainScapes – [rainscapes.org](http://rainscapes.org)



- Tree Montgomery – [treemontgomery.org](http://treemontgomery.org)



- Organic Lawn Care

- [www.montgomerycountymd.gov/lawns](http://www.montgomerycountymd.gov/lawns)

- Residential Energy Program

- [www.montgomerycountymd.gov/green/energy/home](http://www.montgomerycountymd.gov/green/energy/home)

- Volunteer

- [www.montgomerycountymd.gov/water/volunteer](http://www.montgomerycountymd.gov/water/volunteer)

# Questions?

Contact: Christy Ciarametaro

Watershed Project Planner

Christy.Ciarametaro@montgomerycountymd.gov

240-777-7720

Project Website:

<https://www.montgomerycountymd.gov/water/restoration/Lake-whetstone-dam-toe-drain-repair.html>

OR

Google “Lake Whetstone Toe Drain”



Working together for a cleaner, greener,  
economically vibrant community

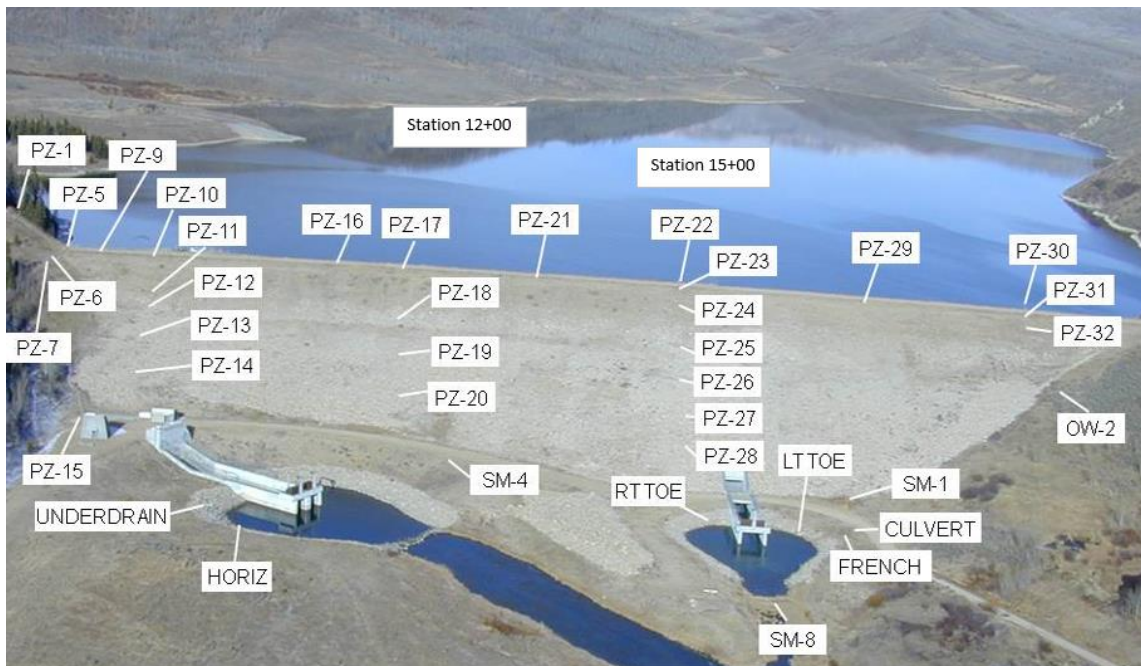
# Projection of Phreatic Surface Depths at Currant Creek Dam

## PZ-18 to PZ-28

Prepared by: Matthew Evans

Using piezometer data provided by CUWCD

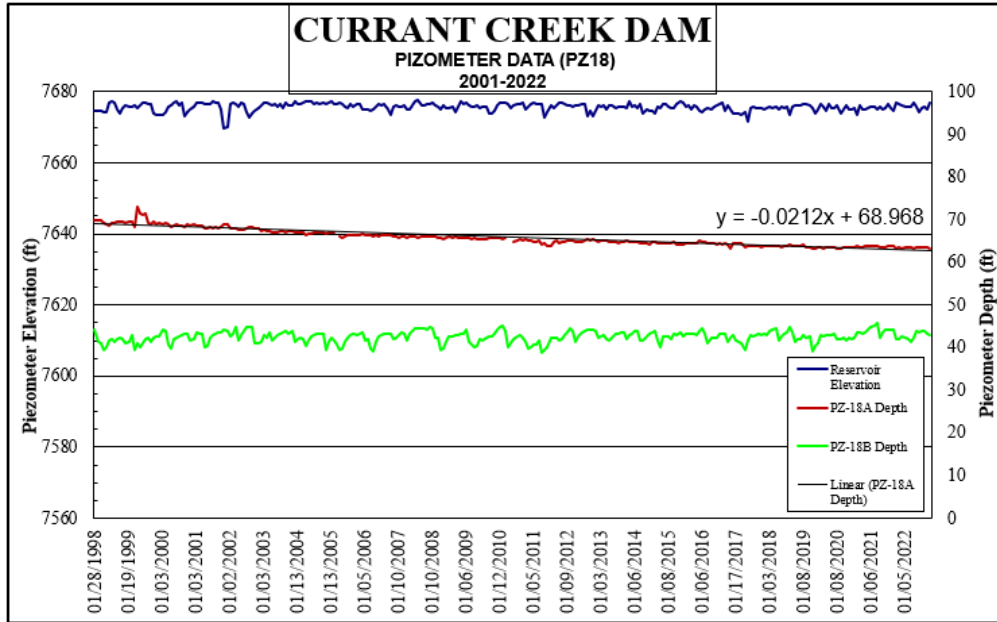
With the right toe drain (RT TOE) not functioning, it would be expected that the area around stations 12+00 to 15+00 would be most at risk of increasing water levels. This is evident by the vegetation growing around those regions of the dam, particularly at toe of the dam. Below is an image of the dam (provided by CUWCD) with the piezometers labeled. This analysis will look at the piezometer data for those at the aforementioned stations. This covers PZ-18 to PZ-28.



The following are graphs display water depths per piezometer data. It is understood that if no other failure modes do not happen first, piping in a dam will occur at least when the phreatic surface reaches the surface of the embankment at a point. Using a linear regression, projections of when the phreatic surface depth will be 0 ft, as well as other intervals, will be provided. The equation for that regression is provided in each graph, with the variable  $x$  being months since January 1998 (PZ data is collected monthly by CUWCD). All but one piezometer station has at least two locations within the dam being monitored. The regression is used on the point that would first reach depth = 0 ft. Reservoir surface levels are also provided for reference. Some comments will be provided where there is something to note.

## Piezometer Data at Station 12+00 (PZ-18 to PZ-21)

### PZ-18

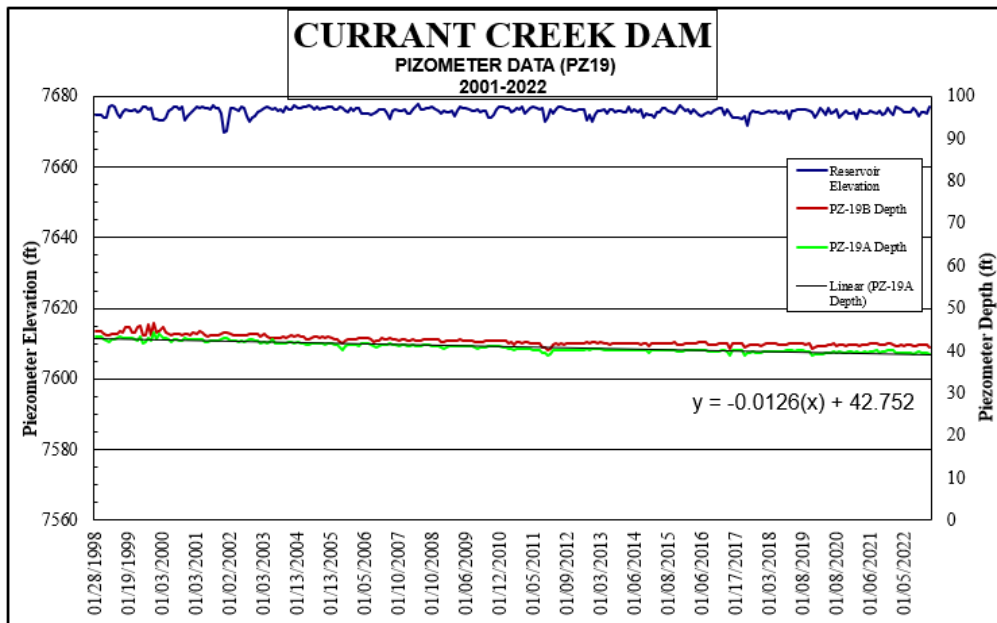


**PZ18**

|            |      |      |      |
|------------|------|------|------|
| Depth (ft) | 50   | 25   | 0    |
| Year       | 2072 | 2170 | 2269 |

Oddity: PZ-18B, the upper portion, is stable while depth of PZ-18A, the foundation zone, is trending downward.

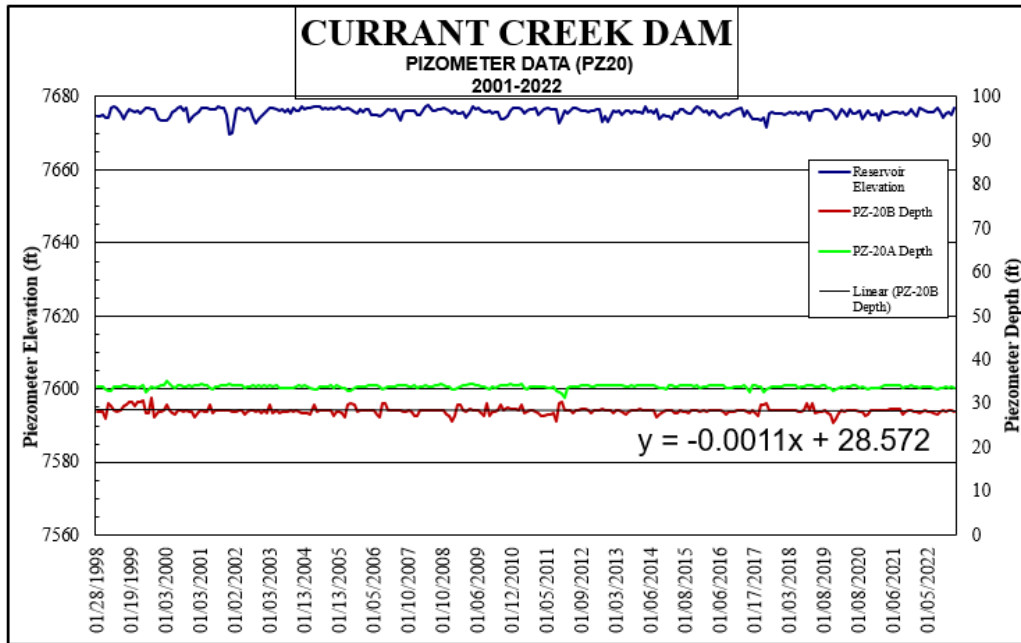
### PZ-19



**PZ19**

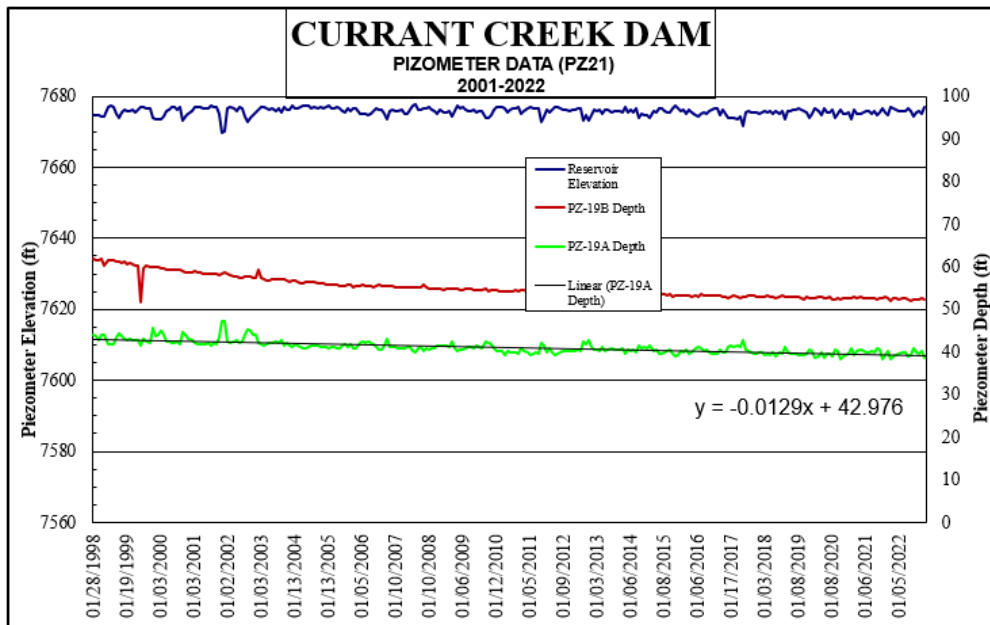
|            |      |      |      |
|------------|------|------|------|
| Depth (ft) | 30   | 15   | 0    |
| Year       | 2082 | 2181 | 2280 |

### PZ-20



**PZ20**  
PZ20 is virtually horizontal. projections of lower depths extend to thousands of years

### PZ-21



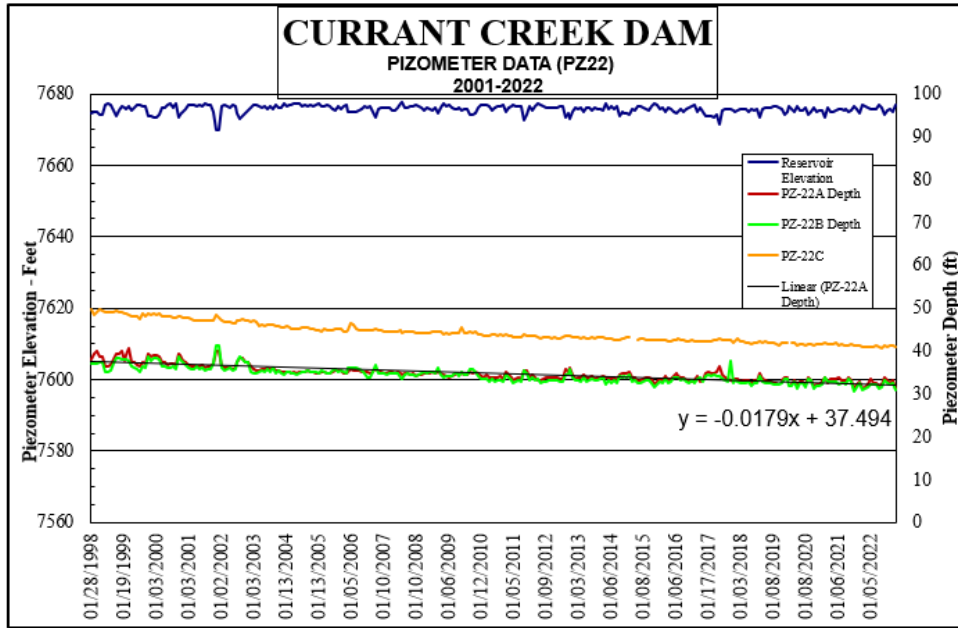
**PZ21**

| Depth (ft) | 30   | 15   | 0    |
|------------|------|------|------|
| Year       | 2081 | 2179 | 2275 |

PZ18, PZ19, and PZ21 all have similar projections, which may suggest that those piezometers are at least consistent. It suggests a fairly even hydraulic gradient

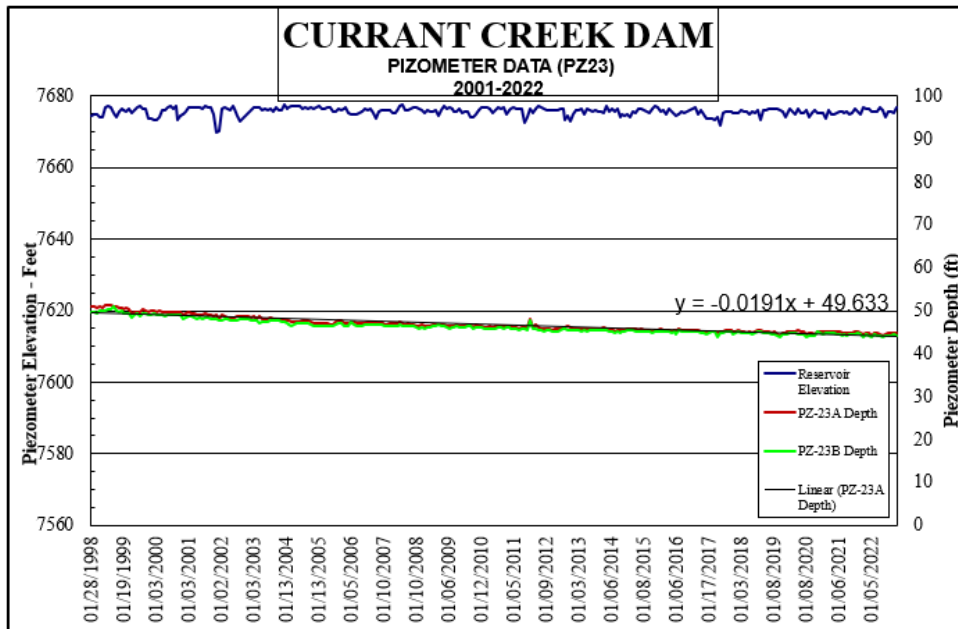
## Piezometer Data at Station 15+00 (PZ-22 to PZ-28)

### PZ-22



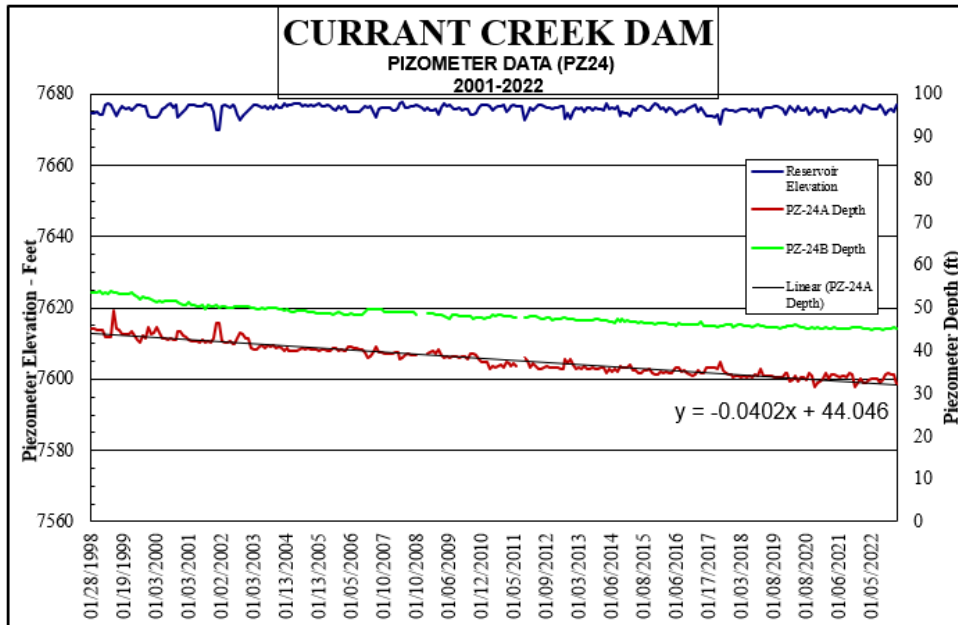
| PZ22       |      |      |      |
|------------|------|------|------|
| Depth (ft) | 20   | 10   | 0    |
| Year       | 2079 | 2126 | 2172 |

### PZ-23



| PZ23       |      |      |      |
|------------|------|------|------|
| Depth (ft) | 40   | 20   | 0    |
| Year       | 2040 | 2127 | 2215 |

### PZ-24

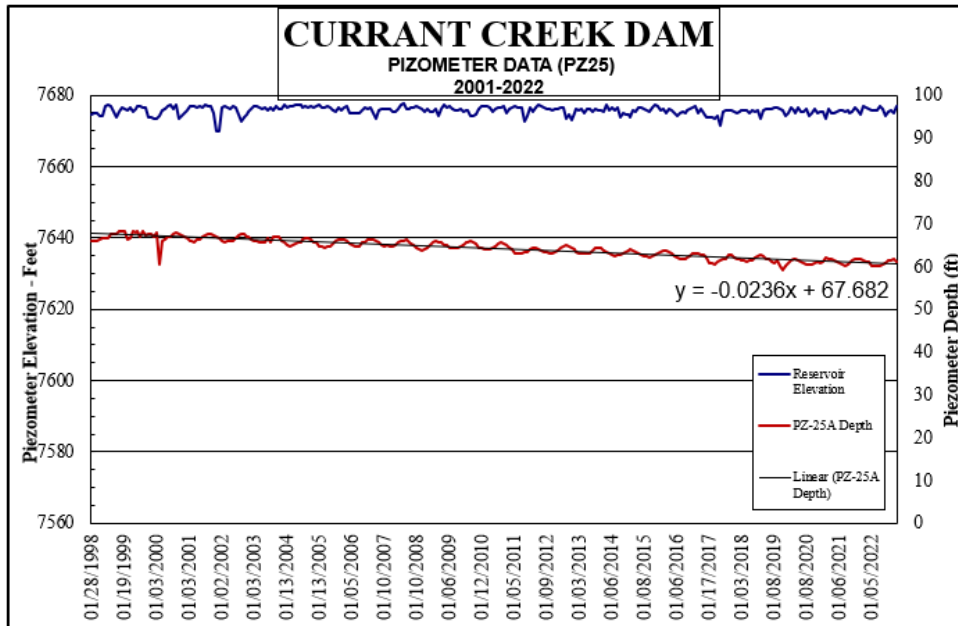


**PZ24**

| Depth (ft) | 20   | 10   | 0    |
|------------|------|------|------|
| Year       | 2048 | 2069 | 2089 |

According to this analysis, PZ-24 would reach a depth of 0 ft first.

### PZ-25

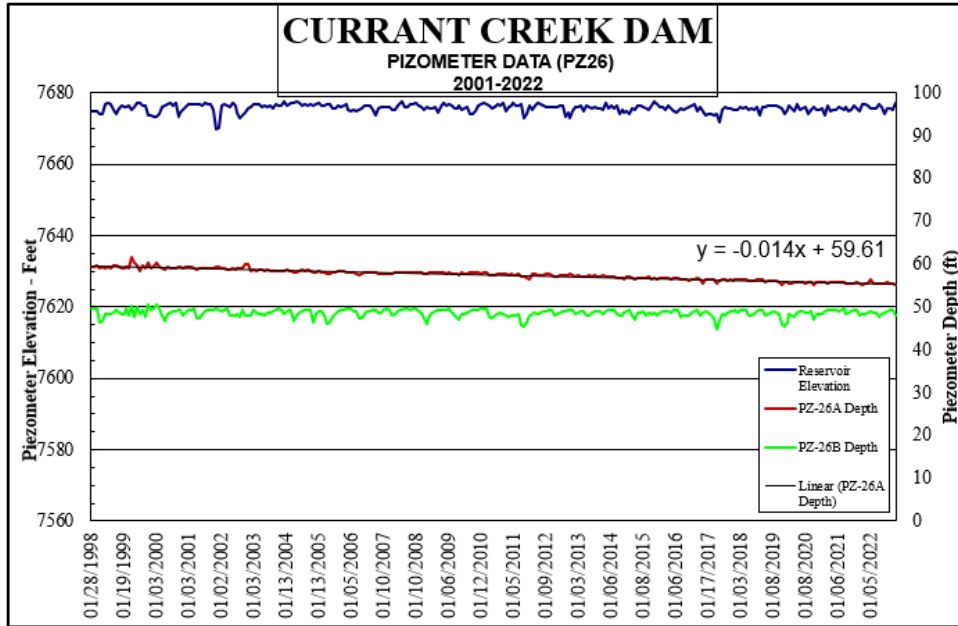


**PZ25**

| Depth (ft) | 50   | 25   | 0    |
|------------|------|------|------|
| Year       | 2060 | 2149 | 2237 |

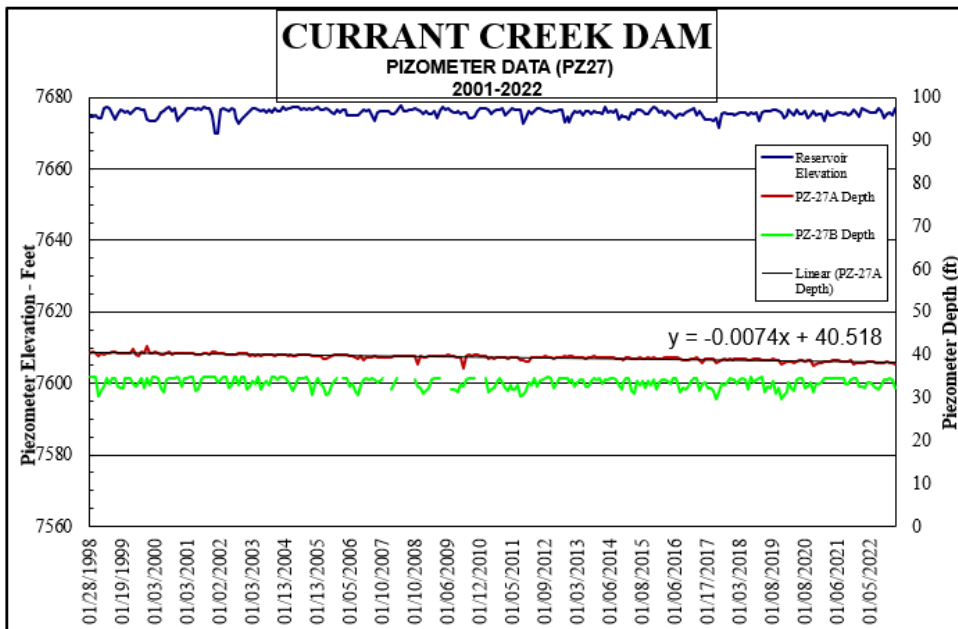
Data from PZ-25B has not been recorded since 2007.

### PZ-26



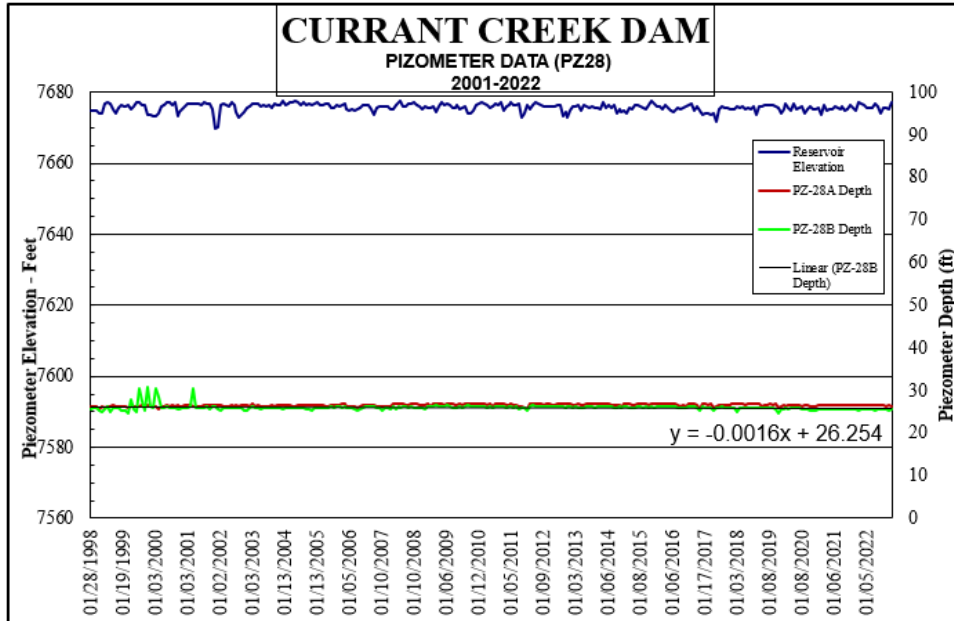
| PZ26       |      |      |      |
|------------|------|------|------|
| Depth (ft) | 40   | 20   | 0    |
| Year       | 2115 | 2234 | 2353 |

### PZ-27



| PZ27       |      |      |      |
|------------|------|------|------|
| Depth (ft) | 30   | 15   | 0    |
| Year       | 2116 | 2285 | 2454 |

# PZ-28



## PZ28

PZ28 is virtually horizontal.

Generally, this analysis predicts phreatic surface depth to be 0 ft earlier for upstream locations on embankment, and later times for downstream portions, suggesting the hydraulic gradient increases in downward slope as you travel downstream

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# Considerations for Embankment Dam Toe Drain Design: An Inspection and Maintenance Perspective

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**Abstract**—One of the key components in modern embankment dam design is the toe drain system, which reduces pore pressures in the downstream face and toe of the embankment and conveys seepage away from the dam. While toe drain design is not new to the dam building industry, challenges associated with the long-term requirements for modern inspection and maintenance techniques have not been well documented. Inspection and maintenance of toe drain systems is an important component of embankment dam safety, since the potential for internal erosion may exist within an inadequate system. It is important for design engineers to familiarize themselves with inspection and maintenance issues commonly encountered in the field, and to account for them in the design. The purpose of this study is to “close-the-loop” of knowledge regarding challenges observed in the field and how they relate to what is designed in the office. This paper will highlight issues found in toe drain systems related to environmental factors and common obstructions; pipe material, size, slope, bends, and depth; filtering and perforations; inspection wells, seepage monitoring instrumentation, and terminations; vehicular access; security and protection; and construction considerations. Work published on the topic will be incorporated and synthesized with case studies from high-hazard dams in the Bureau of Reclamation’s dam inventory to highlight specific problems associated with various types of toe drain designs. Based on these findings, a summary of considerations will be given for embankment toe drain design.

## I. INTRODUCTION

Toe drains are typically located along the downstream toe of embankment dams for the purpose of capturing seepage from the embankment, abutments, and foundation, and conveying it safely to a discharge area downstream [1] [2]. When properly designed, constructed, and maintained, they provide a filtered exit that **alleviates pore-water pressures inside the embankment, thereby reducing the risk of dam failure by internal erosion** [3]. It is therefore important that dam safety programs include routine monitoring, inspections, and maintenance of these drains to assess whether they are meeting their intended purpose [4].

Work by others on this subject has focused primarily on the essential components of toe drain design, while only briefly addressing design considerations for inspection and maintenance equipment. The intent of this paper is to synthesize and reaffirm the findings found in these publications from an inspection and maintenance perspective; as well as provide additional considerations based on various observations and challenges encountered with this equipment in the field. A brief review of modern inspection and maintenance techniques will be presented, followed by a summary of considerations that should be evaluated when designing, constructing, and maintaining toe drain systems.

## II. TOE DRAIN MAINTENANCE CONSIDERATIONS

Before addressing design considerations for toe drains, a brief summary of the purposes and techniques behind inspections and maintenance will be discussed.

### A. Importance of Dam Safety Inspections on Toe Drains

Risk-based dam safety analysis is based on the use of potential failure modes, which are defined as particular sequences of events that, if allowed to develop, will eventually lead to the failure of a dam [5]. **In embankment dams, a failure may occur if the embankment becomes overly saturated from a high phreatic surface, leading to an increase in pore-water pressures, a decrease in effective stress, and hence a decrease in the stability of the dam** [6]. Most embankment dam failures, however, are ultimately caused by **internal erosion** mechanisms of one form or another, which typically begin at unfiltered exit points for seepage through the embankment, abutments, or foundation [7].

Current design practices include the use of properly filtered toe drain systems, which provide ample protection against these potential failure modes. In addition to providing a preferential filtered exit for seepage, they collect the seepage and convey it away from the toe of the dam, consequently lowering the phreatic surface in the embankment (Figure 1) [2],[8].

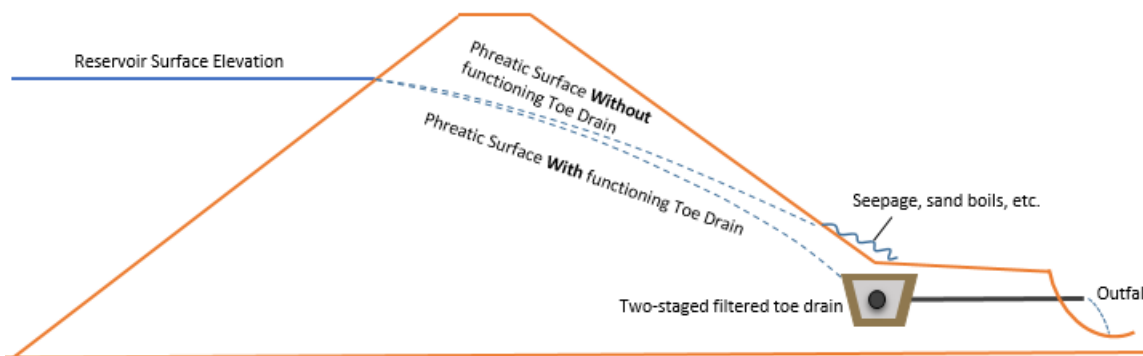


Figure 1 – General profile of phreatic surface in a dam with and without a functioning toe drain.

It is important to note, however, that most of the embankment dams in use today were not designed to modern standards, and that when improperly designed, constructed, or maintained, any toe drain system can actually create an inherent risk to the dam. A toe drain decreases the length of the flow path through an embankment, thereby increasing the overall hydraulic gradient through the dam. It also creates a horizontal exit gradient, which has a much lower factor of safety against internal erosion than a critical (vertical) gradient [9]. Without proper filtering, it provides a preferential and unfiltered exit for seepage to carry eroded material from the embankment directly to the outfall downstream. Furthermore, if a toe drain becomes plugged due to pipe collapse (Photo 18), sedimentation (Photo 22 and Photo 23), roots (Photo 9), etc., it can no longer serve its intended purposes of providing a preferential filtered seepage exit, and relieving pore pressures within the embankment [10]. Therefore, regardless of when or how a particular embankment dam was built, it is essential to have a regular inspection program in place to detect toe drain problems before they become dam safety issues.

There are several notable incidences of inadequate or dysfunctional toe drain systems contributing to potential failure modes [11]. One such case study is the near-failure of Arthur V. Watkins Dam in 2006. The original toe drain, an unfiltered open-jointed concrete pipe, was installed after seepage and piping were observed near the downstream toe during the first fill following the dam's construction in 1964. Despite these precautionary measures, the dam almost failed in November 2006 near the same location due to piping through the foundation; only through emergency interventions was the embankment saved from fully breaching. While poor foundation conditions were ultimately blamed for this incident, it was also noted that the toe drain openings were plugged with sediment and mineral encrustation, which had gradually accumulated since construction [12]. The drains were no longer serving their intended purpose, and seepage was instead exiting through another, more preferential path. Prior to this near-failure, the drains had never been inspected, and the seepage through the foundation was exiting below the surface of a heavily-vegetated canal, so no early warning signs of this failure mode were apparent [13]. Had routine inspections of the toe drain taken place, the developing problem may have been detected and corrected in a timelier manner. This incident highlights the importance of (1) designing a properly filtered toe drain system, and (2) implementing a routine visual monitoring program that addresses all potential failure modes of a particular dam [4].

### **B. Modern Inspection Techniques**

Once constructed, toe drain systems should be inspected regularly to verify that they are operating properly [14]. Modern practice for inspecting toe drain systems typically involves the use of a closed-circuit television (CCTV) system connected to a steerable remotely operated vehicle (ROV), or "camera-crawler." A maneuverable camera mounted to the ROV (Photo 1) captures video footage of the interior of the pipe being inspected [15], and the footage is transmitted back through a retractable cable to a control center where the operator can view and record it for technical evaluation and documentation [8]. Typical ROVs with their cameras attached range between 18 and 36 inches in length and between 4 and 8 inches in diameter. In instances where pipes are too small or otherwise unnavigable for an ROV, fixed or self-levelling push cameras (1 to 3 inches in diameter) are typically used instead (Photo 1), although they are not nearly as maneuverable.

There are currently several such pipe inspection systems commercially available, each with their own unique capabilities and limitations. The equipment employed by the Provo Area Office, and used for the case studies detailed in this report, includes an Envirosight® SuperVision® SV150 (large) and SV90 (small) ROV (Photo 1), and a Pierpoint®/SPX® P330+ Flexiprobe® push camera. The ROVs are 6-wheel-drive stainless steel vehicles equipped with a detachable high-definition pan-tilt-zoom camera mounted at the front end; and are connected to a control panel and monitor via a ¼-inch-diameter, 1,000-foot-long Teflon-shielded cable. Among the various attachments available, they are supplied with interchangeable wheels to accommodate pipes as small as 6 inches and as large as 36 inches in diameter, with the capability of lifting the ROV up to 1.25 inches off of the invert. The push camera is equipped with both a fixed (1-inch-diameter) and a self-leveling

(3-inch-diameter) camera, attached to a 500-foot-long push rod cable and a cased monitor. Both systems allow for video output to a computer for recording purposes.

These and other such systems are designed primarily for municipal water, stormwater, and wastewater pipe inspections, which generally do not present the same challenges that many toe drain systems do, such as long reach lengths, steep gradients, sharp turns, abrupt changes in invert elevation, deep sediment, etc. It is important, therefore, for design engineers to understand both the capabilities and limitations of the inspection equipment available that could be used for monitoring the systems after their construction to ensure proper maintenance over their lifespan.

### C. Modern Maintenance Techniques

As previously mentioned, if not properly maintained, toe drains may become entirely obstructed, thereby impeding full camera-crawler inspections (Photo 8-Photo 10, Photo 22, Photo 23, Photo 61, Photo 62, Photo 64, Photo 65, and Photo 73). If there is a dam safety issue related to the inspection of a toe drain system, then the system may need to be carefully cleaned to establish a baseline against which future inspections may be compared. Any sedimentary material should be sampled and analyzed to determine its source, and a new baseline inspection should be conducted.



Photo 1 – From rear to front: the “large” ROV, “small” ROV, and push camera, with an iPhone for scale.



Photo 2 – View of a jetting unit used for toe drain cleanings.

Currently, typical maintenance of toe drain systems involves the use of a high-pressure water jetting tool, commonly referred to as a “jetter”. The jetter usually consists of trailer-mounted tank, pump, and motorized hose reel (Photo 2) and a retractable, reinforced hose with a cleaning head attached to the end. The head is equipped with rear-facing nozzles which propel the head and hose forward through a pipe, and sweep material back out as the hose is retracted (Photo 3 and Photo 4).



Photo 3 – View of a gravel pile obstructing access in a toe drain.

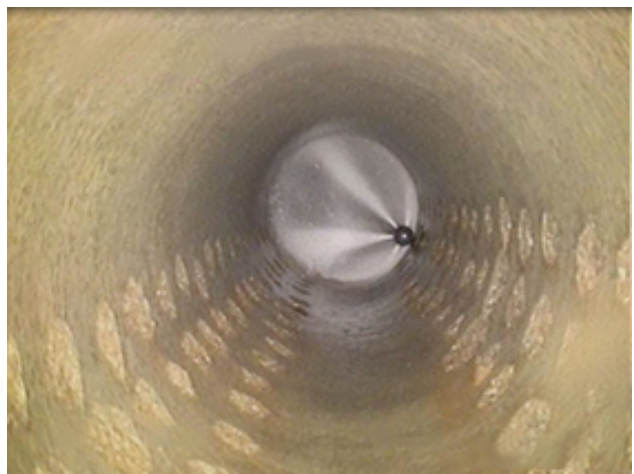


Photo 4 – View of the location of the gravel pile in Photo 3 following the cleaning. Notice there is no more gravel or sediment.

Depending on the material being removed as well as the pipe’s material, size, condition, and location (i.e. within a dam or not), various cleaning heads are available for use (Photo 5), each with their own purposes and operating techniques. Some

self-center in the pipe, while others are dragged along the invert. Roots within a pipe may require a cutting head before they can be removed, while mineral encrustation may require a tool such as the chain flail. A spinning head (Photo 5) has been found to be the most versatile tool for general toe drain cleaning applications.

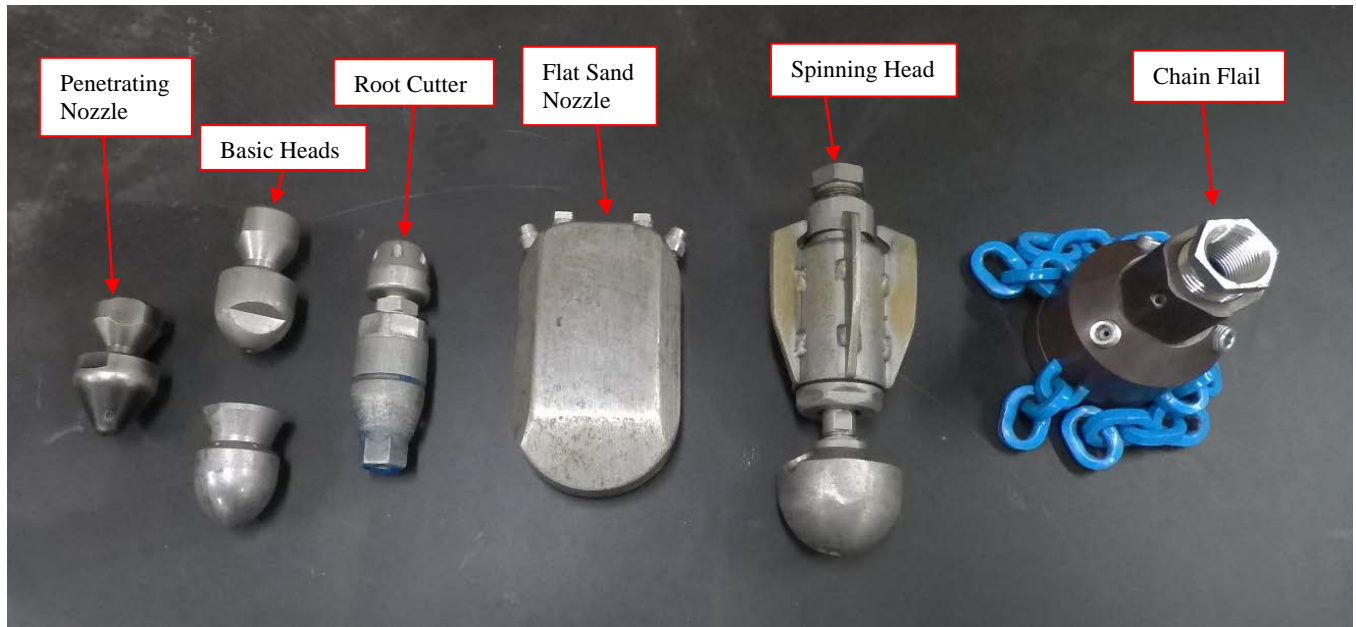


Photo 5 – Photo of various commercially available jetting head nozzles.

**D. Dam Safety Inspections vs. Routine Maintenance**

Similar to camera-crawlers, jetters are designed primarily for municipal water systems, and they pose serious risks for toe drain systems. To provide enough force to propel the hose forward through the pipe, water pressure at the nozzles may reach as high as 4,000 psi. Because of this high pressure, extreme care must be used when operating the equipment. If it is mishandled, operated in poor pipe conditions, or used by untrained operators, serious injury to personnel or damage to the structure may occur. This specifically introduces a risk to embankment dams, as it could potentially scour out embankment material surrounding a toe drain that was damaged, broken, poorly-designed, or poorly constructed (Photo 6 and Photo 7). It should therefore be noted that the objective of preparing a toe drain system for a baseline inspection is not necessarily to clean the pipes completely. Where necessary, material may be left in the drain if there is a risk associated with removing it and if its location, type, and quantity are well-documented in the baseline condition inspection report.



Photo 6 – View of a gap near the invert of a drain outfall, exposing the surrounding material.



Photo 7 – A jetter quickly eroded out material in this area and developed a much larger hole.

The guiding principle in Reclamation’s 2005 *Guidelines for Embankment Drain Inspections, Evaluation and Follow-Up Activities* is that all decisions related to the inspection or maintenance of existing drains within dams is to “DO NO HARM” [8]. Based on this philosophy, any decision regarding how to approach an inspection or cleaning should be made by those

familiar with dam safety principles, based upon careful consideration of potential failure modes and all known site conditions (e.g. location, filter material, pipe material, performance history, etc.). Clear documentation should be prepared prior to proceeding regarding the purpose of the cleaning (e.g. the need to inspect beyond a removable obstruction, or the need to establish a baseline to determine the source and rate of sediment accumulation). If the purpose for the cleaning is related to a dam safety concern, then the documentation should include any relevant failure modes. Finally, cleaning activities should be monitored with CCTV equipment, either simultaneously or in frequent intervals, by someone trained in dam safety.

The principle examiner has the responsibility of knowing when to proceed with a cleaning, when to post-pone it until further analysis and planning has taken place, and when to cancel it altogether to avoid “doing harm” to personnel or to the facility. In summary, a clear understanding of the signs and causes of dam failures related to toe drains should be required of anyone performing inspections or maintenance on embankment dam toe drain systems.

### III. TOE DRAIN DESIGN CONSIDERATIONS

When designing toe drain systems, several factors should be considered beyond fulfilling the drainage needs of embankment dams. As discussed, these systems require frequent inspections and occasional cleanings to remain functional. Based on observations and challenges faced by those maintaining the toe drain systems, the following considerations are recommended for implementation into the design process.

#### A. Environmental Factors and Common Obstructions

It is important that designers become familiar with the ways in which toe drain systems can be degraded by their surrounding environment; if appropriate preventative measures are omitted from a design the consequences could lead to costly corrective action [16]. While a full list of issues cannot be adequately covered in this paper, the items listed below are most common.

**Corrosion** - Pipe corrosion is a concern that can affect toe drains, particularly if the drain pipe is placed along an abutment and collects groundwater [17]. In these situations, the chemistry of the abutment material and the groundwater should be determined, the type of corrosion predicted, and a pipe material selected to prevent or minimize the corrosion [18].

**Sediment and Gravel** - Sediment and gravel are the most common obstructions found in toe drains, especially when the pipes are being inspected for the first time since construction (Photo 8, Photo 22, Photo 23, and Photo 73) [17]. It is not always obvious how or where material is introduced into a toe drain. If the outlet is submerged by tailwater, material can potentially wash in over time (Photo 56). Occasionally, toe drain systems are not properly cleaned following construction. Most often, however, surrounding material gets in through joint gaps, collapses, damaged areas of pipe, or oversized perforations.

**Roots** - Roots from deep-rooted vegetation sometimes infiltrate toe drain systems, thereby limiting their effectiveness and inhibiting inspections (Photo 9). If root growth is not removed promptly when small, it can offset conduit joints, damage pipe, or completely obstruct the drain. If left for too long, it may require excavation and pipe replacement to be removed. As a preventative measure, deep-rooted vegetation within 25 feet of the drain alignment should be eradicated as part of the regular operation and maintenance of the dam [19].



Photo 8 – View of gravel at the base of a cleanout, likely introduced during construction, creating a permanent obstruction for inspection equipment.



Photo 9 – View of a root-ball completely plugging a drain.

**Mineralization** - When seepage water high in dissolved minerals enters a toe drain system, the minerals tend to fall out of suspension and accumulate over time. This can lead to clogged perforations and cementitious build-up in the pipe, which ultimately creates an obstruction to flow and hinders inspection equipment (Photo 10 and Photo 11). The Bureau of Reclamation's *Ground Water Manual* provides more detail into common causes of mineralization in drains [20].



Photo 10 – Mineral encrustation created a permanent obstruction preventing inspection access.



Photo 11 – Pressurized water entering through some of the perforations, while others are plugged up by sediment or mineral encrustation.

**Biofouling** - When ferrous minerals such as iron pyrite are present in soil, and that soil is subject to intermittent wetting and drying, these minerals oxidize and attract certain bacteria in a process known as biofouling. This can happen within a toe drain system if any sections of pipe straddle the oxidation zone [21]. Bacterial growth can clog the perforations in collector pipe, reducing flow efficiency. It can also obscure the pipe from view with CCTV equipment, or even plug the pipe entirely, preventing full access. If left for too long, bacteria has actually deteriorated pipes (Photo 12 and Photo 13), which can help initiate internal erosion. Toe drain design should therefore account for this risk to maximize the functionality and lifespan of the system. Where practical, embankment materials containing ferrous material should be avoided.



Photo 12 – View of biofouling along an abutment drain prior to cleaning.



Photo 13 – View of a deteriorated invert of pipe, similar in location as that shown in Photo 12, after biofouling was removed.

### **B. Pipe Materials**

Choosing a proper toe drain pipe material is especially important in regards to dam safety. A material well-suited to its environment will contribute to the longevity and sustainability of a toe drain system, whereas a poorly-chosen material could lead to a collapse, providing an easy path for seepage to initiate internal erosion [22]. One of the more common factors to consider is overburden pressure, both from the embankment material and from live loading due to construction or other traffic. A few commonly used pipe materials are listed below, along with the effects they each have on inspection and maintenance efforts.

**Clay-Tile Pipe** – A commonly used pipe material in older dams, clay-tile pipe drains were primarily installed before modern toe drain design and construction criteria were developed, and when the material was inexpensive and readily available. The main problem with clay-tile is that it is brittle, and can easily crack and break apart if improperly loaded (Photo 14), such as from vehicular traffic above the drains. Once cracked, the pipe could become dislodged during a cleaning, which would initiate erosion of the surrounding material (Photo 6 and Photo 7). Furthermore, because most clay-tile pipe is laid open-jointed, joint offsets are very common (Photo 28). These offsets can limit access and create challenges for both inspection and maintenance equipment. Therefore, it is recommended that clay-tile be avoided altogether for use in toe drain systems.



Photo 14 – Cracked clay-tile pipe.

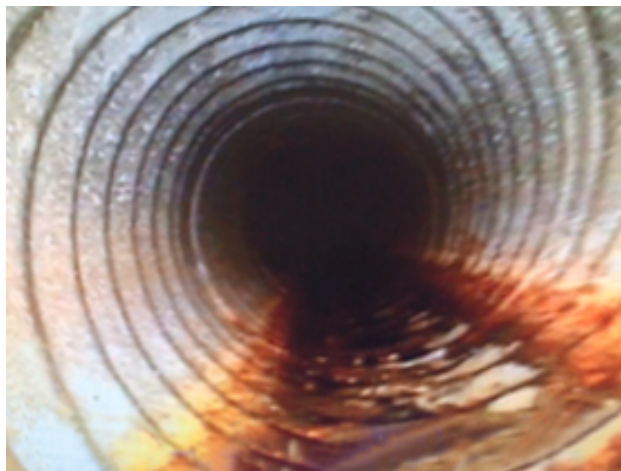


Photo 15 – View of corrosion on the invert of a toe drain which is allowing water to escape prior to discharging at the outfall.

**Corrugated Metal Pipe (CMP)** – CMP has historically been used in toe drain systems, although its use is more commonly limited to less-critical features such as outfall pipe segments or inspection wells [23]. As in most applications, CMP is subject to corrosion due to the interaction between the metal, the surrounding material, seepage, and water flowing through the pipe [24]. Its service life is therefore much shorter than that of other components of an embankment dam and its drainage system. Once CMP rusts through, whether along the invert of an outfall pipe or at the base of an inspection well, the holes that form can allow collected seepage to leak back out into the drainage envelope. This prevents the outflow from being properly observed or quantified, an important component of seepage monitoring at embankment dams. If the filter material around the pipe or inspection well is inadequate, these holes can also become unfiltered exit points. Furthermore, due to the high operating pressure of typical jetters, cleanings may exacerbate corroded conditions, further damaging corroded areas and widening existing holes (Photo 15). For these reasons, many agencies now limit the use of CMP to low hazard dams, while the Bureau of Reclamation prohibits its use in embankments altogether [25].

**Asbestos Cement (AC) and Concrete Pipe** – Both AC and concrete pipe are a pipe material commonly used in older dams, and both have been found to perform well over time. They are more conducive to inspections, because the rough cement provides enough traction for inspection equipment even when inclined or wet. However, AC is no longer manufactured due to health hazards, and concrete pipe presents its own challenges. Because they are connected bell-and-spigot, offsets can often occur between conduit sections, creating gaps for material to enter the pipe, and making inspection and maintenance efforts more difficult (Photo 29). As with clay tile pipe, concrete pipe is also brittle and can break, whether during construction (Photo 16) or due to overburden pressure (Photo 17). If this material is used in toe drain designs, the concrete strength should be specified to withstand the weight of material placed upon it, and any damaged section should be replaced during construction.



Photo 16 – Broken area near the crown of a bell-and-spigot connection between sections of concrete pipe, caused during construction.



Photo 17 – View of longitudinal cracking in a concrete pipe, caused by overburden pressure.

**Plastic Pipe** – Plastic pipe has become more common in modern toe drain design because of its versatility and resistivity to corrosion. High-density polyethylene (HDPE) is more ubiquitous, although other materials such as Poly-Vinyl Chloride (PVC) have also been used. One of the main concerns with plastic pipe, however, is that its structural strength tends to be less than that of other materials. The Bureau of Reclamation began using single-walled, corrugated plastic pipe in toe drain systems in the 1980s [17]. In 1999 Reclamation began conducting CCTV inspections and discovered that approximately one-half of these toe drains with the single-walled materials had either seriously deformed or had collapsed entirely (Photo 18 & Photo 19). Further, at one facility rodents burrowed into single-walled HDPE and initiated sediment transport into the toe drains (Photo 20).



Photo 18 – View of a buckled corrugated HDPE drain pipe.



Photo 19 – View of a buckled corrugated HDPE drain pipe, likely caused from vehicular traffic traversing the drains.

In 2009, Reclamation performed studies that compared the strengths and failure modes of both perforated and non-perforated plastic pipe, resulting in a number of findings and recommendations [26]. Most of the aforementioned collapses were with single-walled HDPE or Schedule 40 PVC pipe [10] [26]. While Schedule 80 PVC and double-walled HDPE are better options, Reclamation now highly recommends the use of solid wall HDPE, welded at the joints (Photo 21) [26]. This pipe, while relatively new for use in toe drain systems, appears to be structurally suitable for typical overburden pressures, flexible enough to accommodate most alignments, and versatile in the options it provides for perforations (Table 1). Because it uses welded joints rather than mechanical ones, it also prevents offsetting due to construction or settlement.

From an inspection and maintenance perspective, plastic pipes offer very little traction to ROVs, especially with travel up steeper slopes. Even drains with mild slopes (0.5%) can be too slick for inspection equipment if seepage flow is present (Photo 31). If plastic pipe is to be used, then an additional access point should be provided at the upstream end of the drain to allow for downstream travel.



Photo 20 – View of holes caused by rodents burrowing through a single-walled HDPE pipe.



Photo 21 – View of solid walled HDPE pipe being fused at the joints.

Table 1 – Comparison table between HDPE and PVC pipes [26].

| Product | Type       |             | Advantages                                                      | Disadvantages                                                                             | Recommended |
|---------|------------|-------------|-----------------------------------------------------------------|-------------------------------------------------------------------------------------------|-------------|
| HDPE    | Solid      |             | Strong, welded joints; flexibility of perforation size and type | Highest cost; special ordered, or hand-drilled after-market addition of perforations      | Highly      |
|         | Corrugated | Single-wall | Economical                                                      | Poor historic performance; weak                                                           | No          |
|         |            | Double-wall | Economical; successful applications; large perforations sizes   | Low strength; maximum recommended cover will often be exceeded in a toe drain application | Moderately  |
| PVC     | Solid      | Wellscreen  | Strong                                                          | Small perforation aperture                                                                | Moderately  |
|         |            | Drainpipe   | Economical                                                      | Weak, brittle                                                                             | No          |
|         | Corrugated | Double      | Economical                                                      | Weak, brittle                                                                             | No          |

### C. Pipe Size

**Flow Capacity** – Toe drains are typically designed such that the maximum seepage flow fills less than 75% of the inside diameter of the pipe [17]. However, in addition to handling seepage directly from the reservoir, the pipes should also be sized to handle inflows related to groundwater, precipitation, and snow melt. Considering this, it is recommended that drain pipes be designed with a maximum seepage depth no greater than 25% of the inside diameter to compensate for inherent uncertainty in seepage analysis, and for potential changes in seepage behavior [32].

**Accessibility** – While many toe drain systems in existing dams have sufficient flow capacity, they are too small to accommodate inspection and maintenance equipment, especially when material is present inside. Pipes with significant material accumulation have created blockages that have been extremely difficult to remove (Photo 22 and Photo 23). Consideration must be given to long-term accessibility of the pipe with debris present. While inspection and maintenance equipment generally does not have difficulty negotiating its way through well-constructed and well-maintained 8-inch-diameter drain pipe (which is the commonly recommended minimum pipe diameter for toe drains [17]), a 10-inch minimum pipe diameter is preferable from an inspection standpoint.



Photo 22 – View of a pipe that was completely full of sediment. Notice the stratification of sediment indicating that material movement and deposition has been occurring slowly over time.



Photo 23 – View of a gravel pile in a 10-inch pipe that was large enough to completely lodge a jetter head, requiring mechanical extraction.

**Transitions** – Access issues occasionally occur at transitions between pipe sizes. When these transitions are concentric (i.e. when the centerlines of both pipes are the same), an insurmountable barrier is created, permanently preventing access with inspection and maintenance equipment (Photo 24 - Photo 26). While there are many ways around this, eccentric transitions (i.e. when the inverts of both pipes are aligned) are far more accommodating and are therefore recommended (Photo 27). Where needed, grout can be used to fill the gaps created by these transitions (Photo 27).



Photo 24 – A broken portion of 10-inch-diameter concrete toe drain pipe being replaced with concentrically-placed 12-inch-diameter HDPE.

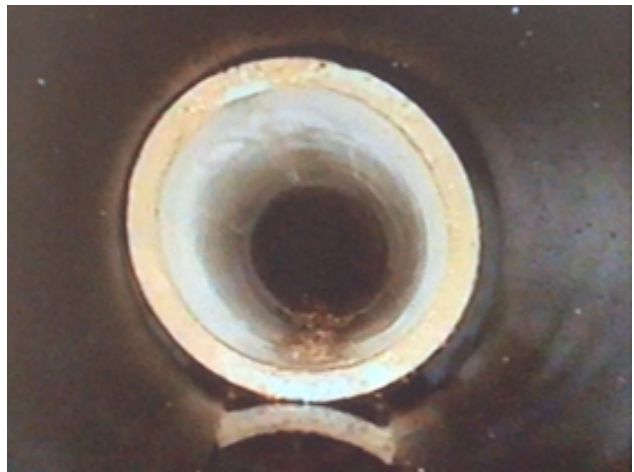


Photo 25 – View of the lip created by joining two concentrically-placed pipes, creating a permanent inspection barrier.

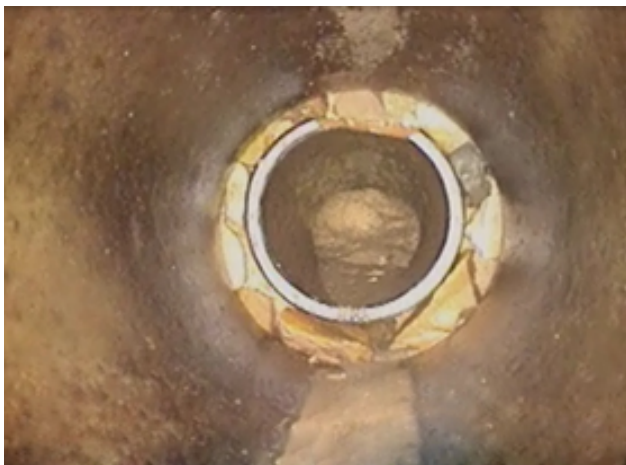


Photo 26 – Concentric pipe transitions can create an access barrier.



Photo 27 – View of an eccentrically constructed transition.

**Offsets** – As previously mentioned, offset joints between pipe sections present another obstacle for inspections and maintenance. These offsets decrease the available pipe area for inspection equipment to access the pipe, in some cases permanently limiting access of the drain (Photo 28 and Photo 29). This is primarily a problem with open-jointed or loosely-connected bell-and-spigot connections, which can easily become dislodged during construction or due to settlement. If these connections are to be used, then consideration should be given to ensure that conduit joints are well connected, and that the pipe diameter is large enough (a minimum of 10 inches) to allow for inspection and maintenance access even if small offsets occur at the joints.



Photo 28 – Offset in a clay tile pipe which prevented further inspection of the toe drain.



Photo 29 – Offset in a concrete pipe which prevented further inspection of the toe drain.

#### **D. Pipe Slope**

**Flow velocity** – While pipe gradients are site dependent, it is generally preferable that toe drains be self-cleaning so that any sandy or clayey material washes downstream to a settling basin instead of accumulating in the pipe. Not only does this help identify and quantify sediment transport through the toe drain system, but it also lessens the need for high-pressure cleanings, which pose a risk to the system and should be avoided whenever possible. In order for a toe drain to be self-cleaning, pipe slope should be such that seepage velocities reach approximately 3 to 5 feet per second [34].

**Traction** – Accessibility is another important factor to consider when choosing the slope of a toe drain. Inspections and cleanings typically begin at the outfall and proceed in the upstream direction, but if the slope is too steep, ROVs have difficulty maintaining traction as the friction from the cable dragged behind them increases. Likewise, jetter heads have increasing difficulty travelling uphill as the friction from the length of a full and pressurized hose behind them increases. Tests have shown that inspection access difficulties occur at slopes of 10° or greater [35], so the Bureau of Reclamation recommends a maximum slope of 5°. This is not always possible, however, since many abutment drains have to be much steeper.

Ultimately, the maximum slope depends on the distance from the access point and the friction provided by the pipe, which is a function of its material and the amount of water, sediment, and biofilm present. In dry concrete pipe, for example, slopes as high as 34° have been ascendable (Photo 30), whereas wet corrugated HDPE or CMP is difficult to navigate at even minimal (0.3°) slopes (Photo 31).



Photo 30 – View of inspection equipment accessing a steep (34°) abutment drain. Access was ultimately limited by a significant change in slope, and subsequent offset, at the top of the incline.



Photo 31 – View of a corrugated double wall HDPE pipe with minimal slope (0.3°), which was so slick that access with inspection equipment was severely limited.

If there is a possibility that access will be difficult due to the slope or material of the pipe being specified, then access points at the upstream ends should be included in the design, since downhill access is far easier than uphill access. This provides the additional advantage of allowing cleaning operations to be conducted from an outfall, while a camera-crawler monitors the cleaning from the upstream direction (Photo 4).

**Slope Changes** – As with pipe sizing, an abrupt change in slope between pipe sections can also present an obstacle for inspection or cleaning equipment (Photo 30). Not only do abrupt slope changes present gaps and offsets, but depending on the pipe diameter, they can also high- or low-center an ROV. It is therefore recommended that slope changes (as with pipe bends discussed below) be limited to 22.5° at any given location.

#### **E. Pipe Bends**

Perhaps the most common challenge faced when navigating through toe drain pipes is abrupt turns due to the length of typical ROVs and the inability for jetter heads to steer (Photo 41). Navigating these corners becomes easier as the pipe diameter increases, but regardless of pipe size, cable and hose friction around corners can halt inspections and cleanings as well, and can even damage the equipment (Photo 32 and Photo 33).



Photo 32 – Damaged cleaning hose from going around a sharp 90° elbow.



Photo 33 – A damaged CCTV cable caused by too much friction.

Turns are unavoidable in most toe drain systems, but layouts can be designed with gradual turns so that inspection and cleaning equipment can navigate them more easily and the amount of friction on the cable and hose can be reduced (Photo 34 and Photo 35) [17]. It is recommended that bends no greater than 22.5° be used in toe drain systems, and that a 5-foot-long straight segment be placed between each bend (Photo 35). There is some leniency with larger (i.e. 24-inch and greater) pipes, because they provide more space for ROVs to turn; but if anything sharper than a 45° elbow must be used in a layout, an additional access point should be placed at that location instead.



Photo 34 – A sharp bend in a toe drain outfall pipe which prevented inspection and cleaning equipment access.



Photo 35 – New alignment of the pipe shown in Photo 34, allowing inspection access.

**The 270° Rule** – When designing toe drain systems, it is important to note that pipe bends are not the only sources of friction on cables and hoses. Both inspection and maintenance operations are typically limited to a combined total of 270° in bends between the cable/hose reel and the ROV/jetter head. Beyond this, the friction on the cable or hose becomes too great, and movement in either direction becomes limited. This “270° Rule” should account for the accessibility and configuration of each access point, as well as the bends in the layout itself. If more bends are required than this threshold will allow, then more access points will be needed (Figure 2).

$$\begin{array}{r}
 45^\circ \text{ (trailer reel to inspection well)} \\
 90^\circ \text{ (top of inspection well to toe drain)} \\
 + 90^\circ \text{ (to enter the toe drain)} \\
 \hline
 \text{Total: } 225^\circ \text{ (between the reel and jetter head)}
 \end{array}$$

Bends available in drain segment layout:  $270^\circ - 225^\circ = 45^\circ$

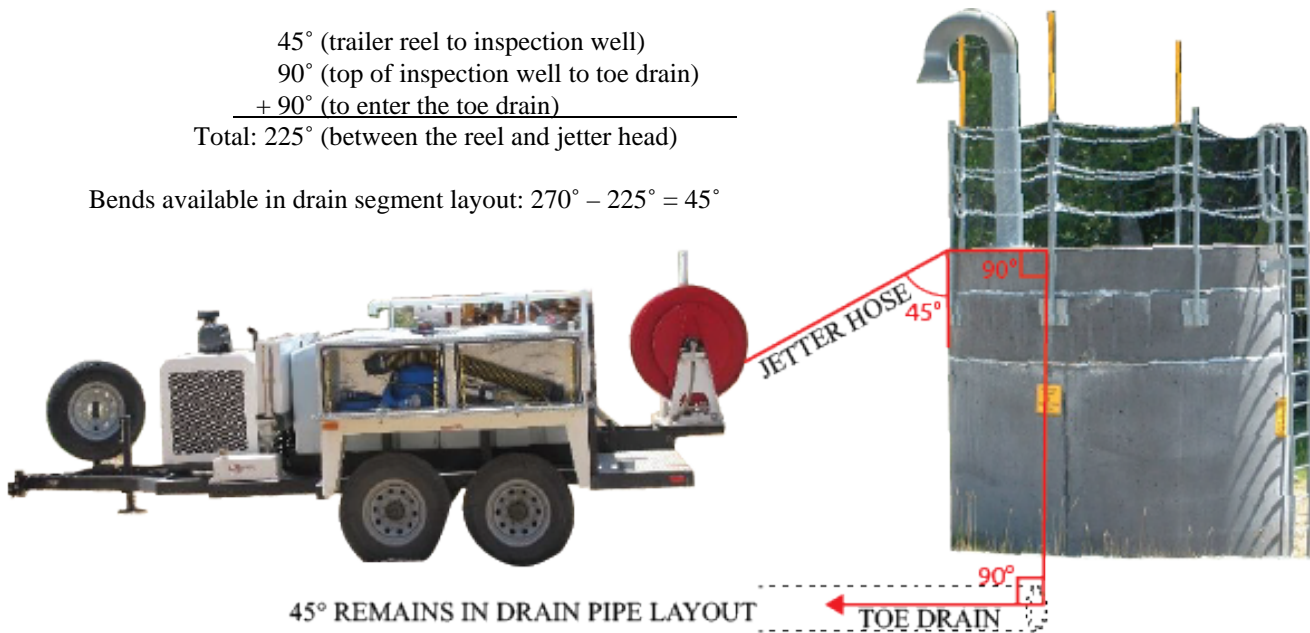


Figure 2 – Diagram illustrating the “270° Rule” for pipe bends. No more than 270°-worth of bends per access point should exist in a toe drain layout, including the angles required to access the drain. In this example, the drain system is limited to 45°-worth of bends, due in part to how tall the inspection well manhole is.

Another way to address this issue is to minimize bends at the access point. For example, if an inspection well has been built at or above waist height, the angle created between the cable/hose reel and the manhole adds additional friction that can be avoided (Figure 2). Sometimes this is not feasible due to snow conditions or road access locations (Photo 36) [36], but where possible, access points should be relatively low to the ground to improve accessibility (Photo 37).



Photo 36 – An inspection well constructed to this height would create substantial friction on both the inspection and maintenance equipment.



Photo 37 – An inspection well constructed next to an upslope access road, and low enough to allow easy access for inspection and maintenance equipment.

### F. Pipe Depth

Another important consideration in toe drain design is the depth of the toe drain pipe. While this does not significantly affect accessibility from an inspection or maintenance standpoint, it could impact the lifecycle cost of the dam if portions of the system ever needed to be repaired or replaced due to damage, collapse, corrosion, etc. to address dam safety issues.

Repairs can be expensive if the toe drain is not reasonably accessible by excavation (Photo 38 and Photo 39) [22]. Slip-lining is discouraged since this would interfere with the system’s ability to collect seepage. Therefore, while the required depth of a toe drain system is inevitably site specific, toe drains should be kept in relatively close proximity to the embankment surface, such that unnecessarily deep excavations into the downstream embankment will not be required.

**Modifications** – It should be noted that while toe drains may have been initially designed and constructed correctly, modifications to the dam may have adverse effects on the system. For example, seismic stability or filter/seepage collection berms are often added to embankment toes to reduce the risk to the dam. However, when the existing toe drain system is left in place it ends up being buried far deeper than its original design (Photo 39). Not only does this make the drain less accessible for repairs or replacement, but it increases the overburden pressure on the pipe, which can lead to collapse (Photo 18, Photo 19, Photo 65, and Photo 74). Therefore, when making such modifications, an assessment should be made of the existing pipe’s ability to withstand the additional overburden. As necessary, it should be replaced with stronger pipe, relocated downstream, and/or supplemented with a more accessible drainage system.



Photo 38 – These toe drains were buried almost thirty feet below the base of the toe, and would require significant excavation into the embankment to replace the drains.



Photo 39 – View of a seismic berm constructed over the existing toe drains, placing over 80’ of material above the drains.

### G. Filtering and Perforations

Understanding proper filter compatibility is an essential component of modern toe drain design, as an inadequate filter can contribute to various forms of internal erosion (Photo 40) [7]. While numerous resources are available that address proper filter design, the intent of this section is to highlight how different design methodologies affect inspection and maintenance activities.



Photo 40 – View of a sinkhole which formed above a toe drain outfall pipe with large perforations and no filter.



Photo 41 – View of a 90° elbow in 24-inch pipe. Inspection equipment navigated this bend well enough, but the jetter could not due to its inability to steer.

**Single-stage filters** - Single-stage filters generally consist of a homogenous, broadly graded filter material (typically an ASTM C 33 fine concrete aggregate gradation sand [27]) immediately surrounding the drain pipe (Figure 3). These filters are relatively simple and inexpensive to construct, which makes them appealing for most designers and contractors, but they have their drawbacks. The grain size of the material must be small enough to filter the surrounding embankment or foundation material against piping, but large enough not to migrate into the toe drain pipe [28]. Consequently, the openings in the drain pipe must be small as well. If single-stage filters are to be used, slots cut into the pipe are recommended over drilled or punctured perforations. The smaller the openings, however, the less efficient the drain becomes, and the easier the openings can become plugged from biofouling or mineralization. Furthermore, high pressure cleaning nozzles tend to scour and displace small filter material located near perforations and joint openings. Therefore, single-stage filters are not recommended for high-hazard dam applications, although they may be adequate for non-critical low-hazard dams [27] [28].

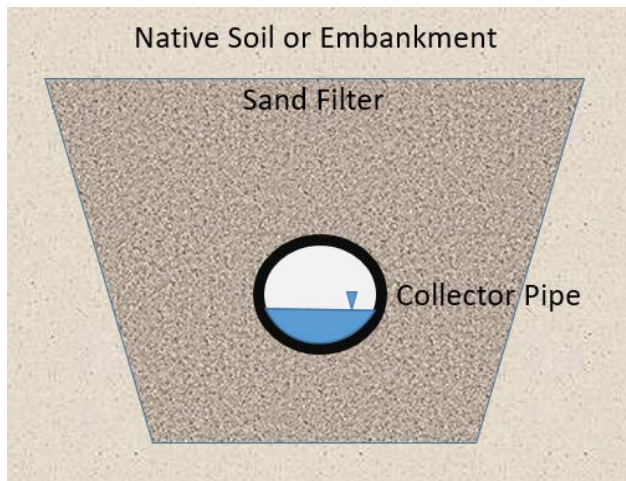


Figure 3 – Single-stage filter consisting of a homogeneous sand filter envelope.

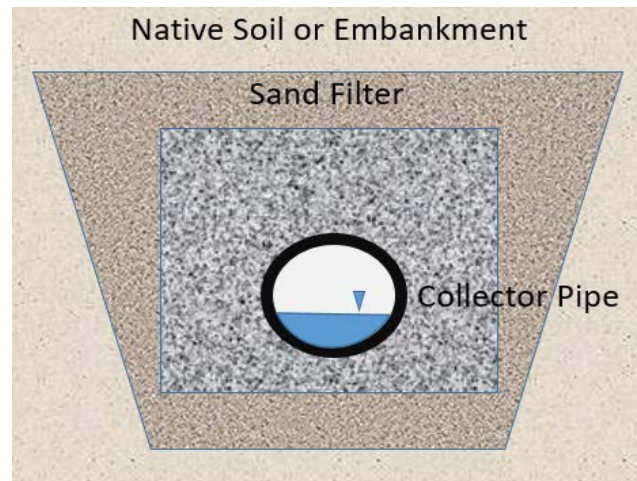


Figure 4 – Two-stage filter with the drainage material designed to be constructed with a slip form.

**Two-stage filters** – Two-stage filters consist of a fine-grained filter material surrounding a larger-diameter drainage material, which encloses the toe drain collection pipe (Figure 4). Although more expensive and more difficult to construct,

this design provides several advantages over the single-stage filter. The larger drainage material allows for larger perforation sizes, which increase drainage efficiency and decrease the chances of clogging [27] [29]. It is also less prone to damage from cleaning operations. Thus, a well-designed two-stage filter with appropriately-sized pipe perforations is recommended.

**Geofabric** – Reclamation tests have shown that the use of geofabric as one of the stages in a two stage filter can be successful when installed correctly [30]. However, these tests were performed over a 30 day period in ideal laboratory conditions, and did not account for the potential of the geofabric becoming clogged due to biofouling or mineralization. Considering this, the use of geofabric in two-stage filters is not recommended; however, if geofabric is to be used, then these drawbacks should be well understood and accounted for in the design [27] [30] [31].

**Perforation Design** – Perforations in toe drain pipes are typically sized based on the diameter of the surrounding material, and their frequency or density is chosen based on the seepage inflow required [28]. However, consideration should also be given to the effective drainage area of the pipe perforations *after* they have become clogged due to biofouling or mineralization. The Natural Resources Conservation Service (NRCS) *Reference on Design of Embankment Drains* uses 30% of the total perforation area in computing effective inflow capacity for circular perforations, and 60% for rectangular slots [32]. It is also important to note that for solid wall pipe, pipe strength decreases proportional to the reduction in surface area resulting from the perforations. This is less true for corrugated pipe, since its strength is in the ribs while perforations are typically placed in the troughs [26].

Perforation patterns typically consist of rows of holes placed at an internal angle of either 45° or 60°, although numerous other patterns have historically been used. Because of this, the rotational placement of the pipe is an important factor to consider. Many perforated pipes are positioned such that the rows of perforations are near the bottom, providing a small trough along the invert to convey seepage downstream [27] (Photo 30). Generally speaking, perforations should be placed along collector pipes low enough to capture seepage at the phreatic surface, but high enough to prevent any collected seepage from leaking out before reaching the outfall or measurement device (Photo 24). The same holds true for outfall pipes, though non-perforated pipe is recommended for these sections since their sole purpose is to transport seepage, not collect it.

## **H. Inspection Wells**

Inspection wells allow entry by personnel, may contain flow measurement instrumentation, and provide inspection and maintenance equipment access to one or more drains [35].

**Spacing** – The spacing between these access points is an important consideration due to limitations of inspection and maintenance equipment. Camera-crawlers are generally capable of pulling up to between 500 and 1,000 linear feet of cable, depending on the equipment type and drain conditions. Jettors, however, are limited to approximately 500 feet due to the weight of the water in the hose, the friction along the invert, and any friction around bends. It is therefore recommended that access points be spaced no more than 500 feet apart [27]. Furthermore, as previously mentioned, it is highly recommended that access points be provided at the upstream ends of a toe drain system as well as at the outfall to better facilitate inspections and cleanings.

**Ventilation** – Inspection wells are a very useful, albeit costly part of most toe drain systems, and they require a few considerations regarding personnel safety. First and foremost, because an inspection well is a structure with limited means of entry or egress, is not designed to be continuously occupied, and has the potential to contain a hazardous atmosphere, it falls under the definition of a “Permit Required Confined Space” [36]. This can be mitigated by using sufficient mechanical ventilation to circulate the air prior to and during entry [37]. Portable ventilation fans with retractable ducts are commercially available for his purpose. However, it is recommended that a permanent ventilation duct be installed inside the inspection well, reaching from the bottom of the well and terminating above the lid in a goose neck configuration (Photo 36 and Photo 37). This allows for natural ventilation once the lid is opened, and if necessary, a fan can be used to mechanically ventilate without the need for additional equipment.

**Ladders and Hatches** – In addition to air supply, nearly all inspection wells require a ladder to provide access to the floor of the structure. Ladders for inspection wells are typically fabricated out of stainless or galvanized steel and are permanently fastened to the wall of the structure. Rungs or steps are required to be corrugated, dimpled, coated with skid resistant material, or otherwise treated to minimize slipping [38]. Ladder rungs must stand off the face of the structure a minimum of seven inches and have a minimum clearance of 30 inches to any obstruction on the climbing side, perpendicular to the rungs [39].

While Occupational Safety and Health Administration (OSHA) does not specifically require an extension or grab bar for ladders used in inspection wells with hatches, it is good practice to specify the installation of such a device on all inspection wells [40]. These devices attach to the upper rungs of the ladder and can be extended 42 inches above the hatch opening, allowing personnel to grab and support themselves while transitioning onto the ladder (Photo 43). When incorporating such a device into the design of an inspection well, the position of the hatch in the lid must be considered. Most lids are constructed of precast concrete, with either an embedded hatch or a formed rectangular channel for installing a hatch. The corners of the hatch opening are offset from the outer perimeter of the lid by at least 6 inches to avoid the rebar embedded

around the perimeter. This results in the rectangular opening being 6 to 8 inches away from the interior circular wall of the manhole. This poses a problem if the ladder rungs are placed only 7 inches from the wall, because the extendable bar will hit the inspection well lid and not be aligned with the hatch opening. Doing so also creates a dangerous overhang when climbing in and out of the inspection well. Consequently, ladder supports should be extended away from the wall so that the extendable grab bars have sufficient clearance and the ladder can be immediately accessed when entering and exiting the inspection well (Photo 44).



Photo 42 – Permanent ladder extension mounted on the exterior of an inspection well.



Photo 43 – View of a manhole lid with hydraulic supports and an extendable grab bar.

**Fall Protection** – Although comparatively rare, inspection wells are occasionally required in embankment material where the depth to the toe drain is 24 feet or more. Such cases currently require that the ladder has either a cage or fall protection system, although cages will no longer be allowed in the near future [41]. Ladder fall protection systems consist of a rail or cable mounted to the center of the ladder rungs, which extends the length of the ladder (Photo 45). A cam-lock trolley device is attached to the rail and secured to a wearable safety harness via a lanyard. In the event of a slip, the cam locks onto the rail or cable and prevents the wearer from an unrestrained fall. These systems are required to extend a minimum of 42 inches above the ladder, so in the case of a deep inspection well with a hatch, a removable extension rail should be used, with the ability to attach to brackets on the ladder for storage in the inspection well when the hatch is closed.



Photo 44 – Ladder rungs extending 16” from the inside wall (rather than 7”), ensuring that the extendable safety rail clears the hatch.



Photo 45 – View of a ladder fall protection system attached to a ladder.

**Seepage Collection** – Finally, it has been recommended that an “underground dam” be installed around the inspection wells to force seepage into the drains and wells so it can be quantified [35]. Placing controlled low strength material (CLSM) on the downstream side of the inspection wells; placing a two-stage filter on the upstream portion of the inspection wells; and drilling holes on the upstream side of the inspection well to capture any additional pore pressure has proven effective (Photo 46 and Photo 47). As always, care must be taken to ensure that all seepage exits are filtered, including those around an inspection well.



Photo 46 – View of a downstream CLSM berm used to force seepage into inspection well and drains.



Photo 47 – View of perforations on upstream side of inspection well to capture as much seepage as possible.

### I. Seepage Monitoring Instrumentation

**Flow Separation** – One essential characteristic of toe drain design is to allow for the measurement of both seepage flow and material transport. Monitoring stations are typically installed at or near each outfall, as well as at any other access/measurement point throughout the system (including within inspection wells), for the purpose of isolating various areas of the embankment to quantify and locate potential problems [42]. The toe drain system at one particular facility was originally designed such that flow from the left and right abutments combined at a tee and exited from the same outfall downstream (Figure 5). It was assumed that the flow was equally divided between the two sides, but when this junction was replaced as part of a modification to the outfall weir box (Figure 6), it was discovered that in fact 95% of the flow was coming from the left side. The result has been a greater focus on seepage monitoring on that side.

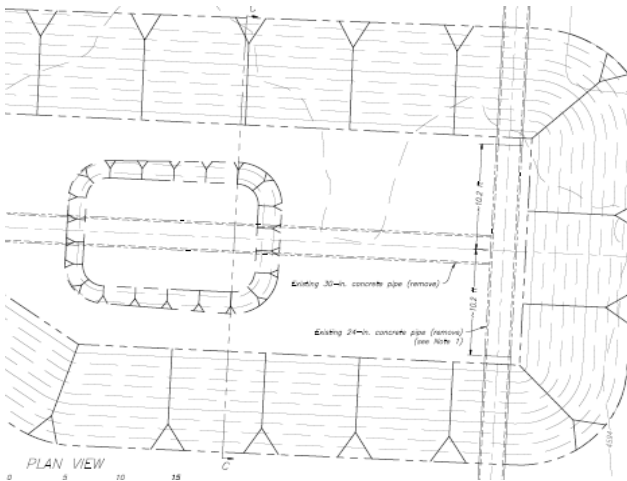


Figure 5 – Original layout of the Hyrum Dam toe drain system, which combined flows from each side of the dam into a single outfall pipe via a tee junction.

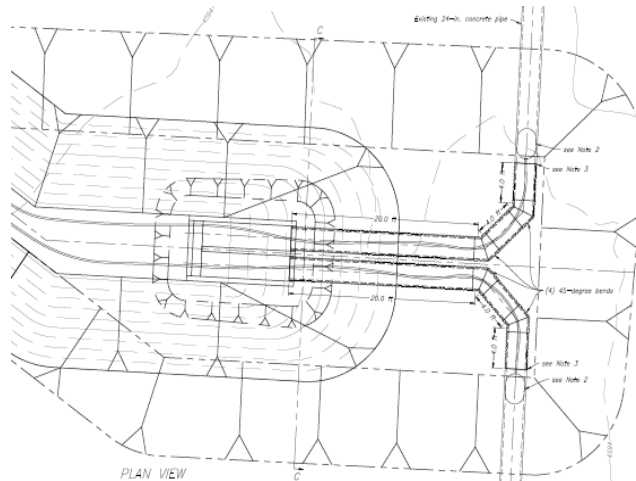


Figure 6 – Plan view of a redesigned outfall layout at Hyrum Dam, which replaced the tee with two sweeping 90-degree bends, effectively dividing the flow from the two sources for improved seepage monitoring.

**Backwater from Weirs** – Flow measurements are typically obtained through the use of weirs, flumes, or simply buckets and stopwatches; while transported sediment is typically collected in settling basins and monitored over time [4]. Sometimes flow measurement weirs are installed such that seepage flow backs up into the outfall pipe or collection system, essentially turning the toe drain system itself into a long settling basin (Photo 48). This condition should be avoided as any potential material being transported into the toe drains will settle out within the pipes themselves, rather than in the settling basin, so sediment transport can no longer be effectively quantified. Inspection progress becomes difficult due to the buildup of sediment, and while most ROVs are designed to operate underwater under low head, slow-moving water also tends to accumulate algae and suspended particles and debris, which drastically reduce visibility. Occasionally, measurement weirs are placed so close to outfalls that they prevent an ROV from being inserted into the pipe altogether. Furthermore, high-

pressure jetters are almost useless when their nozzles are submerged, so any ability to collect and quantify sediment in this manner is also nullified. It is therefore recommended that wherever possible, flow measurement devices should never be designed or installed such that they back water up into the toe drain system, or prevent access with inspection equipment (Photo 49).



Photo 48 – View of water backed up into a toe drain system because of the measurement weir configuration.



Photo 49 – View of a free-flowing toe drain outfall with sufficient distance between the pipe and weir inverts.

**Sluicing** – Toe drain settling basins are seldom designed with accommodations for maintenance. Weir plates are typically embedded or welded in place, and while this ensures a leak-proof measurement system, it does not allow the plate to be removed for the purpose of cleaning out the toe drain system. It is therefore recommended that toe drain monitoring stations be designed to allow the settling basin to be sluiced as needed, either with a removable weir or a drain valve.

**Encasement** – Measurement stations should be covered so that material cannot be blown or washed in, thereby producing a false indication of sediment accumulation from the toe drains (Photo 50 and Photo 51) [42]. These covers should allow for easy access to the monitoring station, so heavy access doors should be avoided. Aluminum lids with locking support mechanisms and hydraulic supports have proven to be an effective option (Photo 43 and Photo 51).



Photo 50 – View of an unprotected settling basin at the outfall of a toe drain system.



Photo 51 – A vault was constructed around the settling basin shown in Photo 50 to prevent foreign material from entering.

## J. Terminations

**Cleanouts** – Cleanouts provide indirect access to the drain system for inspection and maintenance equipment only [35]. They are a relatively inexpensive alternative for providing access for inspection and maintenance equipment, but they require special considerations during design and construction. As mentioned elsewhere in this paper, care must be taken during construction to prevent material from entering the system through the cleanout (Photo 8). Whether installed at the upstream terminus of a pipe or mid-section, the cleanout should be connected in such a way as to allow for inspection and cleaning equipment to enter and exit easily, avoiding steep grades, drop-offs, hard angles, etc. (Photo 52 and Photo 53) [35].



Photo 52 – View of two 22.5° bends coupled to provide a cleanout at the upstream terminus.



Photo 53 – View of a typical cleanout and lid installed at the upstream terminus of a toe drain system.

**End Caps** – If the upstream end of a toe drain pipe does not require access, it should instead be capped to prevent unwanted material from entering. Specialized and perforated end caps are recommended for use (Photo 54). Additionally, care should be taken during construction to ensure a temporary endcap is used to prevent material from entering the drain (Photo 55).

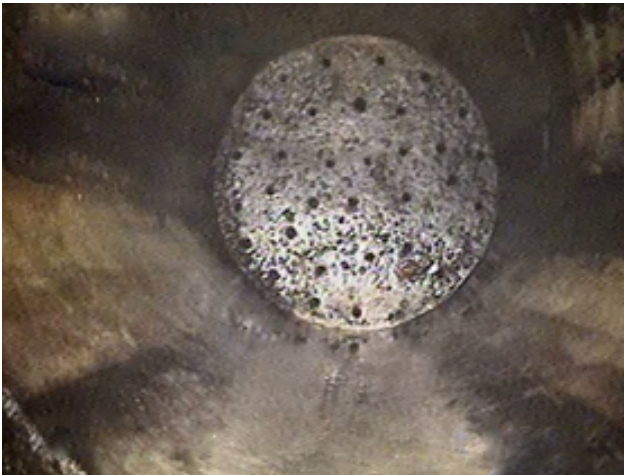


Photo 54 – View of a drain end cap with perforations to allow water flow.



Photo 55 – View of a temporary drain end cap using a generic five-gallon bucket.

**Outfalls** – Toe drain outfalls are sometimes placed low enough that they permit tailwater from the settling basin or discharge channel to enter the drainage system. This is a concern because the backflow can be laden with sediment or debris during high releases, and this material deposits in the drains when the tailwater recedes (Photo 56). Not only does this hinder inspection access, but it also makes it difficult to determine whether sediment is actually moving through the toe drain. Wherever possible, the outfall should be placed higher than the anticipated tailwater levels, but if backflow conditions are anticipated, then a flap gate is recommended to prevent sediment and debris from entering the outfall pipe (Photo 57).



Photo 56 – View of sand deposits in the toe drain outfall.



Photo 57 – View of an aluminum flap gate.

### ***K. Vehicular Access***

Sometimes the layout of a toe drain system is conducive to toe drain inspections, but the access points are still somewhat inaccessible due to the limitations of the trailers or vehicles upon which the inspection and maintenance equipment are mounted and controlled. Close access to the inspection wells can eliminate a substantial amount of friction between the cable or hose and the terrain between the vehicle and the access point, thereby increasing the overall accessibility of the drains. Therefore, it is highly recommended that access roads be included in a design to provide vehicular access up to any location requiring inspection or maintenance access (Photo 58 and Photo 59).



Photo 58 – View of an access ramp to a toe drain inspection well.



Photo 59 – View of the distance (approximately 350') between an access manhole (with a person standing on top of the cover and circled), and the trailer access road below. The distance and terrain that must be covered by the equipment prior to getting into the inspection well creates friction and limits the inspection capabilities.

### ***L. Security and Protection***

Security is a feature that was not considered when many dams were constructed, and many dam features are currently being retrofitted to meet modern security needs. While toe drains are not the most accessible or vulnerable features of a dam, they can still provide access to sensitive portions of a dam. Inspection wells and other access points should therefore be equipped such that they can be secured against unauthorized entry.

***Animals*** – Oftentimes rodents and other small animals have been known to inhabit toe drain pipes, entering the systems through the outfalls (Photo 60) and even burrowing through the drains themselves (Photo 20). While this may seem harmless, they can build nests and introduce debris which impedes inspections (Photo 61). Sometimes these animals die in the pipe, creating even larger and more permanent obstructions. At one particular dam, a deceased rodent was encountered immediately beyond a 90° elbow, creating an inspection barrier and blocking visibility (Photo 62). Beyond the carcass, there

appeared to be a full pipe blockage that could not be accessed or observed more closely due to the obstruction. Removal of the carcass and the blockage with the jetter proved impossible due to poor maneuverability around the 90-degree bend. This will likely require excavation, a much costlier endeavor over a simple cleaning. It is therefore recommended that screens, grates, or flap gates be installed at toe drain outfalls to prevent unwanted animal access (Photo 57 and Photo 63).



Photo 60 – View of a beaver encountered during a toe drain inspection.



Photo 61 – View of a rodent nest blocking inspection access to the rest of the drain.



Photo 62 – View of a deceased rodent blocking inspection access to the drain.



Photo 63 – View of toe drain outfall screen.

#### IV. TOE DRAIN CONSTRUCTION CONSIDERATIONS

A significant amount of sediment and gravel has been encountered during the initial inspections of many of Reclamation's high-hazard dam toe drains. While it is not always clear how this material was introduced into the drain, large cobble sized material, or even debris in a drain with no obvious openings to native soil, are an indication that at least some of the material was introduced during construction (Photo 8, Photo 64, and Photo 65). In many cases, debris introduced in the drains during construction is too large to be removed via cleaning methods, and creates permanent inspection obstructions unless removed through excavation.



Photo 64 – View of a broken crown of a drain pipe that was lodged during modification of the toe drain.



Photo 65 – Portion of crushed pipe and gravel sized material, blocking inspection.

### A. Trenching

The filter configurations shown in Figure 3 and Figure 4 are typically constructed with either a rectangular or a trapezoidal cross section, which is dictated by the type of trench and formwork used (Photo 66 and Photo 67). Trapezoidal cross-sections result from trenches with sloped sides, and generally provide more capacity for seepage collection; but they also require more excavation and filter materials to construct. Rectangular cross-sections result from vertical trenches and therefore require less excavation, but they are relatively smaller in cross section and may provide less seepage capacity. It should be noted that slipform shoring panels can be used to construct the secondary drainage filter shown in Figure 4. When an excavation deeper than five feet is required, the slopes of the trench must be shored back for personnel safety. In such cases, it is likely less expensive to use a trapezoidal sand filter coupled with a rectangular gravel cross section, as this will require less drainage material to be brought onsite.



Photo 66 – View of contractor using slipform shoring panels to construct a two-stage filter. Notice the filter will have a trapezoidal sand filter and a rectangular drainage material cross section.



Photo 67 – View of a trapezoidal two stage filter with a rectangular drainage material zone using slipform shoring panels (similar to design in Figure 4).

### B. Compaction

As mentioned earlier, a commonly observed issue especially with bell-and-spigot pipe is the development of gaps at the joints between pipe sections, caused primarily by localized settlement due to poor compaction beneath the pipe. These gaps can allow material to easily move into the drain pipe, expose the surrounding material to scour from high-pressure jetters, and create potential snags for inspection and cleaning equipment (Photo 28, Photo 29, Photo 68, and Photo 69).

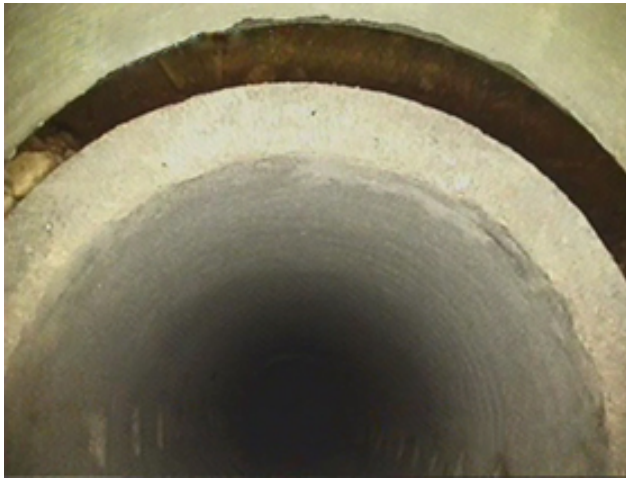


Photo 68 – View of a large joint opening through which overlying material had fallen, creating a void.



Photo 69 – View of material accumulated on the invert of the pipe due to another large joint opening.

Localized settlement can also lead to excessive undulations in the pipe profile, causing seepage flows to pool and/or exit the system before reaching the outfall, prompting false conclusions about seepage conditions in the dam. At one particular facility the toe drain outfalls were consistently dry, so it had always been assumed that the phreatic surface was lower than the collector drains along the abutments. However, the initial inspection revealed undulations in the pipe, accompanied by high water marks that indicated seepage flow, as well as deep deposits of sediment that hindered inspection access and had to be removed in order to establish a baseline for future monitoring (Figure 7, Photo 70 through Photo 73). It is therefore important that pipe joints are properly connected during construction, and that the bedding is properly compacted according to industry standards so as to minimize localized settlement, offsetting, and undulations along the pipes [43].

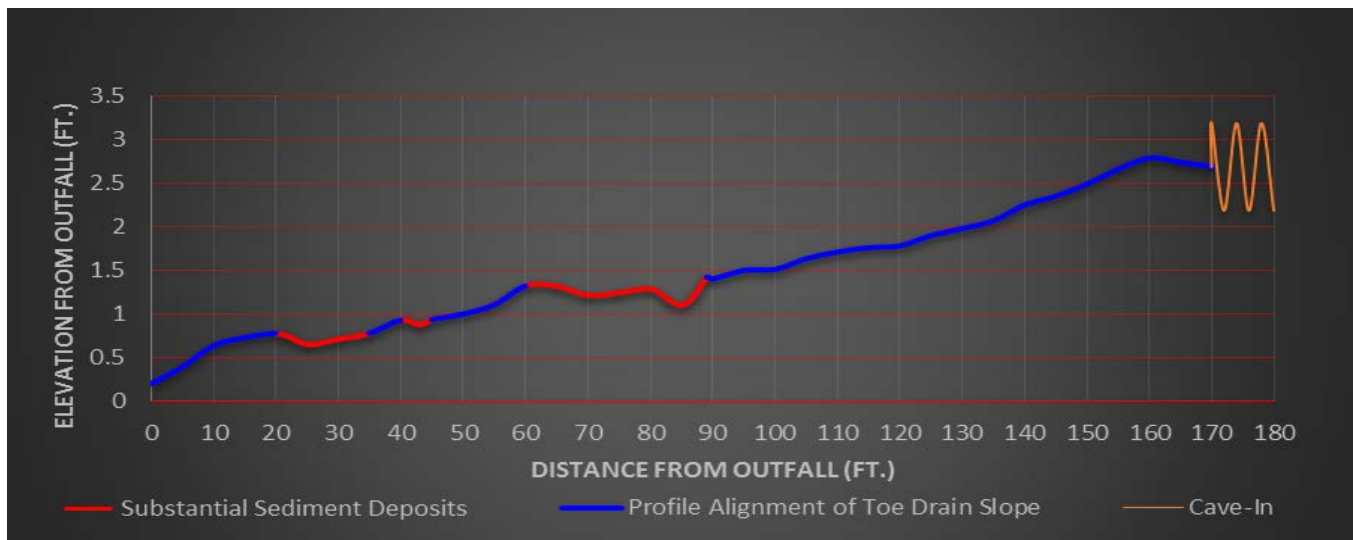


Figure 7 – Profile of a drain with sags which allowed sediment to accumulate and seepage to discharge through conduit openings prior to discharging (Photo 56 – Photo 61). The drain was never thought to be problematic as it had never been observed flowing.



Photo 70 – View of a joint opening which allowed seepage to dissipate. Notice signs of water level in the upstream conduit.



Photo 71 – Sediment gradually accumulates as the slope of the conduit decreases.



Photo 72 – Slope of conduit continues to decrease, while depth of sediment increases.



Photo 73 – View of the sediment deposits and high water level.

### C. Backfilling

The backfill around the pipe must also be well compacted, especially when the drain passes under a roadway. Many of the lower strength plastic pipes rely on this compaction for support [26]. However, damage caused by construction equipment is often more likely to crush pipes than soil loading, and the risk of this happening is at its highest while backfilling. Care should be taken during the initial backfill to only traverse the drain pipe if necessary, and then only with tracked equipment or roller compactors (which have a low ground pressure compared to trucks). Also, inspectors should closely monitor the drain during backfilling operations to ensure that pipe sections are being connected properly and kept free of debris. Furthermore, the construction specification should require the contractor to record and submit CCTV video footage of the system after approximately 5 feet of backfill has been placed and compacted, and then again once construction is complete [44] [45] [46]. Any debris encountered during these inspections should be removed, and any damaged sections should be replaced, both at the contractor's expense.

In one toe drain outfall pipe, a sizeable hole was encountered in the crown through which a few large cobbles had protruded (Photo 74). The ROV was only able to access beyond this point once the damaged pipe section was excavated and removed. A short distance further upstream, the broken crown piece was encountered and once again prevented inspection access, requiring further excavation to remove it (Photo 75). This damage likely occurred either during compaction of the backfill material, or due to vehicular traffic along an access road that crosses the outfall alignment. The backfill material around the concrete outfall pipe evidently contained large cobbles, which exerted concentrated loads on the pipe and ultimately punctured through. Two-stage filters used in modern toe drain design eliminate this problem around collector drains, but outfall pipes are typically backfilled with native material that may contain larger material. It is therefore recommended that backfill material around toe drain outfalls be no greater than 3 inches, to prevent punctures due to concentrated loads.



Photo 74 – Damaged crown of an outfall pipe that prevented inspection access of the toe drain.



Photo 75 – View of a piece of a broken crown which permanently obstructed access.

## V. CONCLUSIONS

Toe drain systems are a critical component of embankment dams; when properly designed, constructed, and maintained, they reduce pore-water pressures in the embankment and safely convey filtered seepage downstream. Maintaining these systems requires consistent and detailed inspections using CCTV equipment and ROVs; and where appropriate, occasional cleanings using high-pressure water jetting units. The guiding principle regarding all decisions related to the inspection or maintenance of existing drains within dams is to "**DO NO HARM**", and inspection and maintenance on toe drains should only be performed by experienced personnel trained in dam safety [8]. Design engineers, inspectors, and operators must have a thorough understanding of inspection and maintenance processes and the potential challenges associated with them to ensure full functionality of a toe drain system throughout its design life. Given the important role toe drains play in handling seepage through embankment dams, any additional up-front cost associated with facilitating routine inspections and long-term maintenance is easily justifiable [26]. As described in this paper, there are many such factors that should be considered when designing a toe drain system. These considerations are summarized below:

### A. *Dam Safety Inspections and Maintenance*

- What type of equipment will access the drain system for inspections?
- What are the dimensions, maneuverability limitations, and cable/hose length limitations of this equipment?

### B. *Environmental Factors and Common Obstructions*

- What is the potential for corrosion, mineralization, root growth, and/or biofouling due to the surrounding conditions in the foundation, abutment, and embankment?
- How corrosive is the native soil and water?
- Do embankment or foundation materials contain ferrous material, and if so, can they be avoided for use?

### C. *Pipe Materials*

- Given the environmental conditions, what pipe material would be best suited to prolong the design life of the drainage system?
- Is the pipe strength adequate to handle the anticipated overburden and traffic loading?
- Will the pipe material provide enough traction for inspection equipment under wet and dry conditions?
- If using a smooth interior HDPE pipe, will access at the upstream terminal end of the drain be provided?

### D. *Pipe Size*

- Are the drains designed to handle a maximum seepage depth no greater than 25% of the inside diameter?
- Is the pipe diameter at least 10 inches, and larger if possible, in order to accommodate inspection and maintenance equipment?
- Are all connections between different pipe materials and/or sizes smoothly transitioned to allow for inspections and maintenance?

#### **E. Pipe Slope**

- Is the pipe slope steep enough to allow the system to self-clean?
- Are any slope changes greater than 22.5°, or otherwise could prevent access to the entire drain?
- Are any portions of the system with slopes steeper than 5° accessible from upstream terminal end of the drain?

#### **F. Pipe Bends**

- Are any bends in the toe drain system greater than 22.5°?
- Is there at least 5 feet of straight pipe between each bend?
- If a bend greater than 45° is required, is an inspection well specified at that location instead?
- Are there less than 270°-worth of bends per drain reach in the drain layout?
- Should access be provided at the upstream terminal end of the drain?

#### **G. Pipe Depth**

- Are the toe drains deep enough to intercept seepage, but shallow enough to be easily excavated if necessary?
- Are the drains situated such that significant excavations into the downstream embankment will not be required if they must be replaced?

#### **H. Filtering and Perforations**

- In high hazard dams, is a two-stage filter being used?
- What perforation size best meets the filter or drainage material surrounding the pipe?
- What perforation pattern will be used to allow for adequate drainage?
- Is a particular pipe configuration required for proper placement of the perforations?

#### **I. Inspection Wells**

- Is the spacing between inspection wells approximately 500 feet to accommodate inspections and maintenance?
- Do inspection wells, including manholes and vaults, meet OSHA ventilation and accessibility requirements?
- How will sediment transport through the drain be monitored, collected, and removed?

#### **J. Seepage Monitoring Instrumentation**

- Can flow rates and sediment from each toe drain reach be collected and measured independently?
- Are flow measurement devices placed such that they will not back water up into the drains?
- How will weir plates be removed, or otherwise how will the water and material be sluiced, to accommodate inspection and maintenance activities?
- Are monitoring stations covered to prevent material from entering the settling basin?

#### **K. Terminations**

- Do all cleanouts have locking, weatherproof covers?
- Do upstream terminal ends have end caps?
- If backwater conditions are present at the toe drain outfall, is a flap gate warranted?

#### **L. Vehicular Access**

- How will inspection and maintenance equipment reach the inspection wells and monitoring stations?

#### **M. Security and Protection**

- How will access points be secured against unauthorized entry by trespassers?
- Is a screen or flap valve specified at the terminal downstream end of the drain to prevent unwanted animal activity?

#### **N. Construction Considerations**

- What type of trenching will be required due to the depth of drain?
- How will the bedding and backfill be compacted, and to what specification?
- Is backfill material free of cobble sized material?
- If using bell-and-spigot connections, how will these connections remain intact during and after construction?

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