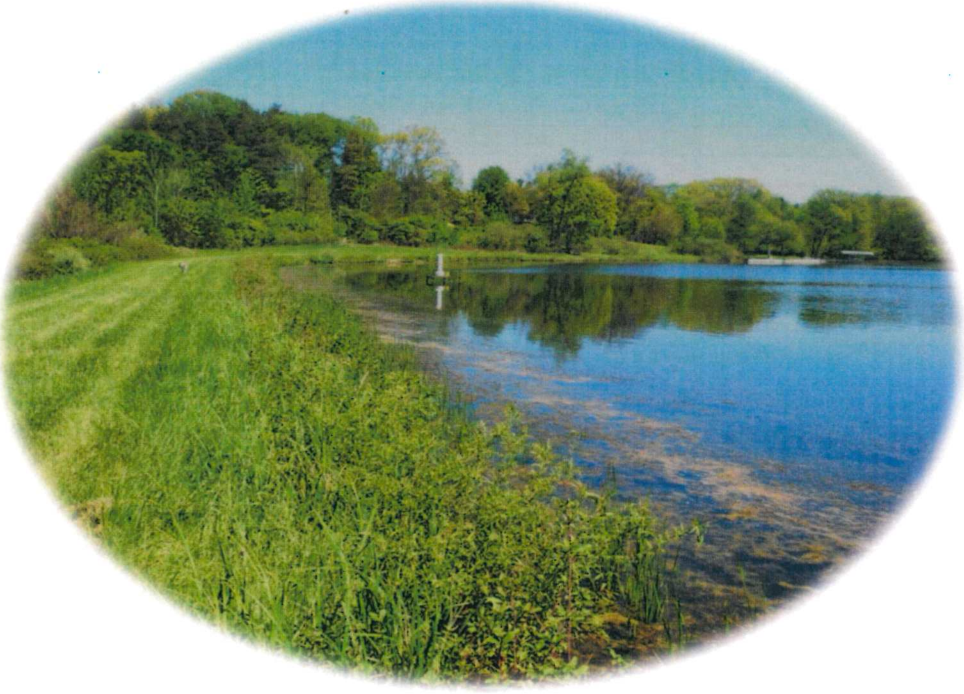
Faussett Lake Dam Inspection Report

Dam Identification No.: 307
Hazard Potential: Low
Section 33 ; T.04N. – R.05E.
Deerfield Township, Livingston County, Michigan
Located on the Yellow River
Per Part 315, Act 451 of 1994



Prepared for:

Walker Reservoir, LLC
4300 Faussett Road
Howell, MI 48855
Email address: gregoryna2003@yahoo.com

Prepared By:

Spicer Group, Inc.
230 S. Washington
Saginaw, Michigan 48607
(989) 754-4717

Inspected By:

Richard V. Graham III, P.E. #62039

STATE OF MICHIGAN
RICHARD VADE GRAHAM, III
PROFESSIONAL ENGINEER
No. 6201062039
LICENSED PROFESSIONAL ENGINEER

12/22/22

Date of Inspection: August 9, 2022
Date of Final Report: December 22, 20222

Project No. 124613SG2017

TABLE OF CONTENTS

I. INTRODUCTION

The Faussett Lake earthen dam and outlet was inspected pursuant to the requirements of Part 315, Dam Safety, Natural Resources and Environmental Protection Act, Act 451 of 1994. Spicer Group conducted the inspection of the dam on August 9, 2022, as requested by the party responsible for the dam, Walker Reservoir, LLC. The scope of this inspection is to identify conditions that constitute an existing or potential hazard to the dam. The identification of potential hazards is limited to the field visual inspection, review of previous dam inspection reports, review of a 2015 structural report, review of previous plans and general computations. The contents of this report are not to be treated as a detailed engineering evaluation.

This inspection report will serve as a supplement to previous inspections performed on the dam. Previous inspection reports, drawings, sketches, calculations, etc. will be referred to as part of this inspection report. A summary of the design, construction, maintenance, and subsequent inspections of the dam are outlined in the Project Information section of this report. The terms satisfactory, fair, poor, and unsatisfactory will be used to describe the conditions of the dam. The following is a brief definition of each term.

SATISFACTORY

No existing or potential dam safety deficiencies are recognized. Acceptable performance is expected under all loading conditions (static, hydrologic, seismic) in accordance with the applicable regulatory criteria or tolerable risk guidelines.

FAIR

No existing dam safety deficiencies are recognized for normal loading conditions. Rare or extreme hydrologic and /or seismic events may result in a dam safety deficiency. Risk may be in the range to take further action.

POOR

Dam safety deficiency is recognized for loading conditions which may realistically occur. Remedial action is necessary. POOR may also be used when uncertainties exist as to critical analysis parameters which identify a potential dam safety deficiency: further investigations and studies are necessary.

UNSATISFACTORY

Dam safety deficiency is recognized that requires immediate or emergency remedial action for problem resolution. Reservoir restrictions may be necessary until problem resolution.

II. CONCLUSIONS AND RECOMMENDATIONS

A. Overall Condition

Visual inspection of the dam indicates the dam and its companion structures are in fair overall condition. However, there are a few deficiencies or maintenance items that should be monitored and/or corrected. The following list is a summary of the areas of concern that were observed during the visual inspection.

B. Observed Deficiencies/Prioritized Recommendations

Specific deficiencies and recommended corrective measures, listed in order of priority are as follows:

1. There is continuing seepage on the downstream slope on the western end of the earthen dam. The seepage is slowly running onto the drive at the base of the slope. No piping was noted or erosion was noted during this inspection.

   Recommend: As recommended in the 2022 geotechnical report (included in Appendix E) a drained buttress should be implemented to remediate seepage and stabilize the earthen embankment. Continue to routinely monitor seepage until remedial action is taken.

2. The 36” corrugated metal pipe (CMP) outlet is corroding and the outlet headwall is failing. Minor settlement was observed at the downstream toe of the embankment along the alignment of the outlet pipe.

   Recommend: Continue to monitor the corrosion annually until repair or replacement can be made to the pipe and headwall. Monitor the soil over the length of the top of the outlet pipe for evidence of additional settlement or sinkholes.

3. Because of its location, it is difficult to assess the condition of the inlet structure and determine the condition of the water valve at the cold water draw down structure.

   Recommend: Continue to monitor the inlet as needed. If operable, exercise the draw down valve at least once per year. It may be pertinent to engage the services of an experienced dive team to film the structure and provide condition documentation, if it is not replaced as part of the outlet pipe project. Place a heavy wire grate or baffle system around the inlet to prevent floating materials or woody debris from entering the inlet and obstructing flow.

4. There are a few animal burrows along the upstream face of the earthen dam, which were marked during the inspection.

   Recommend: Fill in the animal burrows. Determine if these are muskrats and if they can be trapped and removed. Please see the attached fact sheet regarding the identification and remediation of animal burrows contained in Appendix F.

C. Further Detailed Studies and/or Investigations

We recommend continuing to design and ultimately construct the necessary repairs to the embankment and outlet structure. Further, continue to pursue the option of establishing a court ordered lake level though Part 307 as a means to complete and find the necessary repairs and to manage the dam and lake level.

D. Hazard Potential Classification

The hazard potential classification of the Faussett Lake Dam is currently listed as a low hazard potential dam. Based on our inspection, we do not recommend changing this classification.

The hazard potential classification is only an indication of the potential for loss of life and economic loss due to failure of the dam. The hazard potential classification is not an indication of the stability or integrity of the dam.

III. PROJECT INFORMATION

A. General Description of Dam

The Faussett Lake Dam was constructed from 1955 to 1963. Soil for the construction of the dam was obtained from the agricultural area of the parcel. The dam consists of an earthen embankment, a drop structure outlet and two (2) emergency spillways. The crest of the dam is approximately 560 feet long and 19 feet in elevation. The upstream slope is constructed at a 2H:1V and the downstream slope is 3H:1V with a top width of 10 feet on average. The embankment is mainly covered with grass and is mowed at least three (3) times per year. The drop structure spillway is a 24-inch diameter CMP, encased in reinforced concrete. A 24-inch CMP traverses under the earthen dam until about 40 to 50 feet from the outlet at which point a 36-inch CMP collects the water for final discharge into the stream. The drop structure has a steel plate with a buoy on top to protect and serve as a warning as to location of the structure. The lake also has a low flow draw down valve at this location which is operated manually at least once per year. There is a water valve located inside a 4-foot by 4-foot concrete block structure immediately east of the outlet structure which is reported to have been used for irrigation purposes in the past.

B. Purpose of Dam

The Faussett Lake Dam was originally constructed for agricultural and residential purposes. Currently, the lake is developing into a large parcel residential area and a platted subdivision is proposed. There are also a few lakeshore homes being constructed. The lake is used for boating, fishing, water recreation, and wildlife observation.

C. Available Design, Construction and Maintenance Information.

There is no design data available for this dam. A geotechnical analysis was completed by SME in 2022 and is included in this report. Also, it should be noted that a Special Assessment District (SAD) has been established.

D. Previous Inspection Reports

1997 Dam Inspection Report – Gary F. Croskey, P.E.
2002 Dam Inspection Report – Jonathan A., and Andrew R. Blystra, P.E.
2007 Dam Inspection Report – Spicer Group, Inc., Saginaw
2012 Dam Inspection Report – Spicer Group, Inc., Saginaw
2015 Structural Inspection Condition report – Spicer Group, Inc., Saginaw
2017 Dam Inspection Report – Spicer Group, Inc., Saginaw

Copies of these dam inspection reports and relevant information are on file with the owner, and/or the Michigan Department of Environment, Great Lakes, and Energy (ELGE) – Dam Safety Unit.

IV. FIELD INSPECTION

Spicer Group performed a visual inspection of the dam on August 9, 2022. Photographs were taken and a field inspection checklist was completed in the field and office summarizing the inspection. The checklist and photographs are included in the appendix of this report. The following is a summary of the visual observations made during the inspection:

A. The inlet structure appears to be in fair condition.

B. Alignment along the top of the dam is satisfactory.

C. The upstream embankment is covered with rip rap, grass, reeds, some small shrubs, and is maintained by mowing to within a few feet of the water.

D. Seepage was visible along the downstream face of the dam on the western one third of the structure. The driveway along the toe of slope in this area is wet. No boils were noted but the area is wet.

E. Seepage was observed at the toe of the downstream slope west of the outlet structure. The seepage does not appear to have increased, but should be continued to be monitored in case it increases.

F. Toe drains have water running out of them. The outlet is cleared of vegetation and root mats allowing the water to flow freely.

G. The low flow draw down valve has not been operated this year and it is recommended that it should be operated to determine its condition after it has been viewed by a dive team.

H. The outlet functions well, outlet channel and approaches are free of debris.

I. There is corrosion of the 36-inch outlet pipe.

J. There appears to be settlement directly above the outlet pipe at the downstream toe of the embankment. Based upon conversations with the Owner, this area of settlement corresponds with the location where the outlet pipe was added onto in order to widen the access road.

K. Ground cover vegetation is established well in the auxiliary spillways.

L. The cracks in the downstream headwall for the 36-inch outlet are becoming larger and the structure should be repaired.

M. The dam is accessed by a maintained private drive.

N. Repair and maintenance are done by the son on an as needed basis, but the family wishes for a maintenance contract on the structure.

O. The shoreline consists of rock protection, brush and natural vegetation.

V. STRUCTURAL STABILITY

The overall structural stability of the dam is fair, based on visual inspection. The embankments show no signs of potential failure. The outlet structure appears to be in fair condition.

VI. HYDROLOGY AND HYDRAULICS

A. Available Design Data, Hydrologic Design Data

Hydrologic information provided by EGLE has been obtained and is included in the appendix of this report. EGLE calculated the 100-year peak inflow into Faussett Lake Dam to be 250 cfs and the 1% chance flood volume is estimated at 340 acre-feet. The maximum storage for this impoundment is 800 acre-feet. Hydraulic capacity calculations are included in Appendix C.

B. Contributing Drainage Area

EGLE estimates the contributing drainage area to be 3.8 square miles. EGLE estimates the surface area of Faussett Lake at normal flow to be 74 acres. This provides a ratio of 32.9 to 1 of contributing watershed drainage area to impoundment area.

C. Design Flood Determination

Low hazard potential dams must be capable of passing the 100-year flood or the flood of record, whichever is greater. The estimated EGLE 100-year flood peak discharge is 250 cfs.

D. Existing Spillway Capacity

The primary spillway has a maximum spillway capacity of 50 cfs. The maximum spillway capacity for the primary spillway can only be reached with 2 feet of head (See hydraulic calculations in Appendix C). During normal conditions, there is 2 feet of freeboard with a normal storage of 500 acre-feet. Maximum storage is reported to be 800 acre-feet. The 1% chance flood storage volume is 340 acre-feet. Therefore the 100-year flood volume can be stored in the impoundment without overtopping.

Previous dam inspections and this one noted two emergency spillways, one on each end of the dam. The one on the west end of the structure is easily discerned. The eastern spillway should be checked for grading to accurately define its location and extent.

E. Flood of Record

The flood of record occurred in 1975. Per the owner, this rain event caused the failure of three (3) dams upstream which caused the reservoir level in Faussett Lake to rise and overtopped the dam by about 6 inches. Both emergency spillways activated in this event.

F. Routing of Spillway Design Flood

Due to the fact the dam can store the required storm event and passed the flood of record, routing of the inflow hydrographs is not necessary.

VII. OPERATION AND MAINTENANCE

A. Assessment of Operating Equipment and Procedures

The gate valve is typically operated annually. However, due to the complexity to access the valve, it has not been operated in a few years. Other than the valve, there is no other operating equipment.

B. Evaluation of Current Maintenance Plan

A copy of the Faussett Lake Operation and Maintenance Plan can be obtained from the Walker Reservoir LLC.

In addition to the items outlined in the operation and maintenance plan, all metal surfaces should be inspected on a regular basis. Sandblasting, cleaning, and painting should be performed as necessary. Further, continue to maintain the vegetation on the upstream and downstream slopes twice per year.

VIII. EMERGENCY ACTION PLAN

Because the Faussett Lake Dam is classified as a low hazard dam, an Emergency Action Plan is not required.

APPENDIX A

EGLE FLOW RATES
DAM INSPECTION CHECKLIST

Malburg, Andra L.

From:	Koko, Kristopher R.
Sent:	Wednesday, December 28, 2022 9:28 AM
To:	Malburg, Andra L.
Subject:	FW: Flood or Low Flow Discharge Request

Kris Koko | Design Engineer
SPICER GROUP, INC.
Office: 734-823-3308 | Cell: 989-798-7251 www.spicergroup.com Stronger. Safer. Smarter. Spicer

-----Original Message-----
From: Bowser, Joseph C. <joseph.bowser@spicergroup.com>
Sent: Monday, December 19, 2022 10:41 AM
To: Koko, Kristopher R. <kristopherk@spicergroup.com>
Subject: FW: Flood or Low Flow Discharge Request

-----Original Message-----
From: EGLE-wrd-qreq <EGLE-wrd-qreq@michigan.gov>
Sent: Thursday, December 15, 2022 9:36 PM
To: Bowser, Joseph C. <joseph.bowser@spicergroup.com>
Subject: RE: Flood or Low Flow Discharge Request

We have processed the discharge request submitted by email on November 22, 2022 (Process No. 20220701), as follows:

Yellow River Drain at Faussett Dam, Dam ID 307, Section 33, T4N, R5E, Deerfield Township, Livingston County, has a drainage area of 3.8 square miles. The design discharge for this dam is the 1% chance (100-year) flood. The 50%, 20%, 10%, 4%, 2%, 1%, 0.5%, and 0.2% chance peak flows are estimated to be 40 cubic feet per second (cfs), 80 cfs, 110 cfs, 160 cfs, 200 cfs, 250 cfs, 300 cfs, and 380 cfs, respectively. The 1% chance flood volume is estimated to be 340 acre-feet. (Watershed Basin No. 32C Shiawassee).

Please include a copy of this letter with your inspection report or any subsequent application for permit. These estimates should be confirmed by our office if an application is not submitted within one year. If you have any questions concerning the discharge estimates, please contact Ms. Susan Greiner, Hydrologic Studies and Floodplain Management Unit, at 517-927-3838, or by email at: GreinerS@michigan.gov. If you have any questions concerning the hydraulics or the requirements for the dam safety inspection report, please contact Mr. Luke Trumble of our Dam Safety Unit at 517-420-8923, or by email at: TrumbleL@michigan.gov.

Low flows are provided in a separate email.

-----Original Message-----

From: EGLE-Automated <EGLE-Automated@michigan.gov>
Sent: Tuesday, November 22, 2022 1:45 PM
To: EGLE-wrd-qreq <EGLE-wrd-qreq@michigan.gov>
Subject: Flood or Low Flow Discharge Request

Requestor: Joseph Bowser
Company: Spicer Group
Address: 125 Helle Blvd. Suite 2
City/State: Dundee/Michigan
ZIP Code: 48140
Phone: 7342657552
Date: 11/22/2022
50 percent
20 percent
10 percent
4 percent
2 percent
1 percent
0.5 percent
0.2 percent
Monthly 95 percent exceedance
Monthly 50 percent exceedance
Monthly Mean
90 Day, 10 year (90Q10)
Lowest monthly 95 percent exceedance
Lowest monthly 50 percent exceedance
Harmonic Mean
Flow Exceedance Curve
Contact Agency:
Contact Person: Susan Greiner
Watercourse: Yellow River
Local Name:
County: Livingston
City/Township: Deerfield Township
Section: 33
Town: 04N
Range: 05E
Location: Faussett Lake Dam #307
FFR1: Dam
Email: joseph.bowser@spicergroup.com

GENERAL INFORMATION / DESCRIPTION	
Name of Dam:	Faussett Dam
Dam Identification Number	Dam ID # 307
Project Number:	130521SG2021
County:	Livingston
Township	Deerfield Township
Town - Range:	T4N – R5E
Quarter Section of Section:	Section 33
Impounded Lake, Stream, or River:	Faussett Lake
Height of Dam:	17 ft
Length:	560 ft
Outlet Pipe(s) Size:	24 inch into a 36 inch CMP
Normal Head on Dam:	8 ft
Impoundment Size:	74 acres
Maximum Storage Capacity:	800 acre-ft
Normal Storage:	500 acre-ft
Purpose of Dam:	Recreation / Residential
Owner:	Walker Reservoir LLC
Contact Name:	Nancy Gregory
Owner Address:	4300 Faussett
	Howell, MI 48855
Owner Phone Number:	517-546-3112
Operator:	Walker Reservoir, LLC
Contact Name:	As above
Operator Address:	
Operator Phone Number:	
Hazard Potential Classification: (Circle 1)	High – 3 yr Significant – 4 yr [✓ **Low – 5 yr**]
Emergency Action Plan (EAP) Required?	No
Is there an existing EAP?	No
Name of the Emergency Coordinator:	N/A
Emergency Coordinator Address:	N/A
Emergency Coordinator Phone Number:	N/A

Date of Inspection:	August 9, 2022
Photographs:	Yes
Tie down stationing along top of dam:	No
FIELD INSPECTION	
EARTH EMBANKMENT	
A.) Settlement	
1.) depressions	Above the outlet pipe at the downstream toe of slope
2.) sinkholes	None Observed
3.) ruts and paths	Minor
B.) Slope Stability (upstream/ downstream slopes)	
1.) Irregularity in alignment	None Observed
2.) Movement	
a.) sloughing	Upstream- 2 places/Downstream-None Obs
b.) cracks	None Observed
c.) slides	None Observed
d.) slumps	None Observed
e.) beaching	None Observed
C.) Slope and Crown Protection	
1.) Vegetative Cover	Grass, well established
2.) Trees and brush	Minor Shrubs
3.) Erosion From Surface Runoff	None Observed
4.) Wave protection	Vegetation and riprap present
D.) Seepage, Boils, Piping	
1.) At contact points with all concrete structures	None Observed
2.) Along downstream slopes	Yes
3.) Along downstream toe	Yes
4.) Potential seepage areas	
a.) trees	None
b.) animal burrows	Along upstream slope
5.) Drainage Systems	
a.) toe drains	Present and draining
b.) filters	None
c.) ditches	Yes, water present

SPILLWAY AND OUTLET WORKS	
A.) Hydraulic Capacity	
1.) Principal Spillway	Acceptable
2.) Auxiliary Spillway	Acceptable
3.) Powerhouses	N/A
4.) Other diversions or outlets, withdrawals	N/A
B.) Control Gates & Operating Mechanisms	
1.) Structural Members	N/A
2.) Connections	N/A
3.) Hoists	N/A
4.) Cables	N/A
5.) Power Supply	N/A
6.) Gate Seals	N/A
C.) Stop Logs and Stop Log Channels	
1.) Leakage	N/A
2.) Deterioration	N/A
3.) Corrosion	N/A
D.) Obstruction to Flow	
1.) Approach Channel	Open
2.) Trash Racks	Yes
3.) Outlet Channel	Clear of debris
4.) Sedimentation	N/A
E.) Drawdown Facilities	
1.) Operation	Valve not operated this year.
F.) Energy Dissipation	
1.) Stilling Basins	N/A
2.) Plunge Pools	Acceptable
3.) Baffles	N/A
4.) Sills & Spillway Aprons	N/A
5.) Erosion Protection	Riprap and vegetation

G.) Pipes	
1.) Joint Separation	None Observed
2.) Leakage	Observed at outlet of 36 inch
3.) Protective Coatings	No coatings
4.) Settlement	None Observed
5.) Displacement	No displacement observed
H.) Sketch of Outlet Works	
CONCRETE & MASONRY STRUCTURES	
A.) Surface Conditions	
1.) Spalling	Yes
2.) Cavitation	None Observed
3.) Cracks	Yes
a.) Displacement and Separation	Yes on headwall for 36-inch CMP
b.) Seepage	Yes at headwall for 36-inch CMP
4.) Efflorescence	None Observed
B.) Joints (Monolith)	
1.) Sealant or Fillers	None Observed
2.) Movement	Yes
3.) Seepage	None Observed
OPERATION AND MAINTENANCE	
A.) Operation	
1.) Seasonal or Fluctuating Pond Levels	No
2.) Operation Records	With operators
3.) Periodic Drawdowns	None
4.) Instrumentation	None in place
B.) Maintenance	
1.) Repairs	
2.) Periodic Maintenance	
a.) Wood	None
b.) Metal	Monitor Corrosion
c.) Concrete	New headwall
d.) Soils	As needed by operator
e.) Electrical	None
f.) Mechanical	Water valve operator annually
3.) Operation and Maintenance Plan	With operator

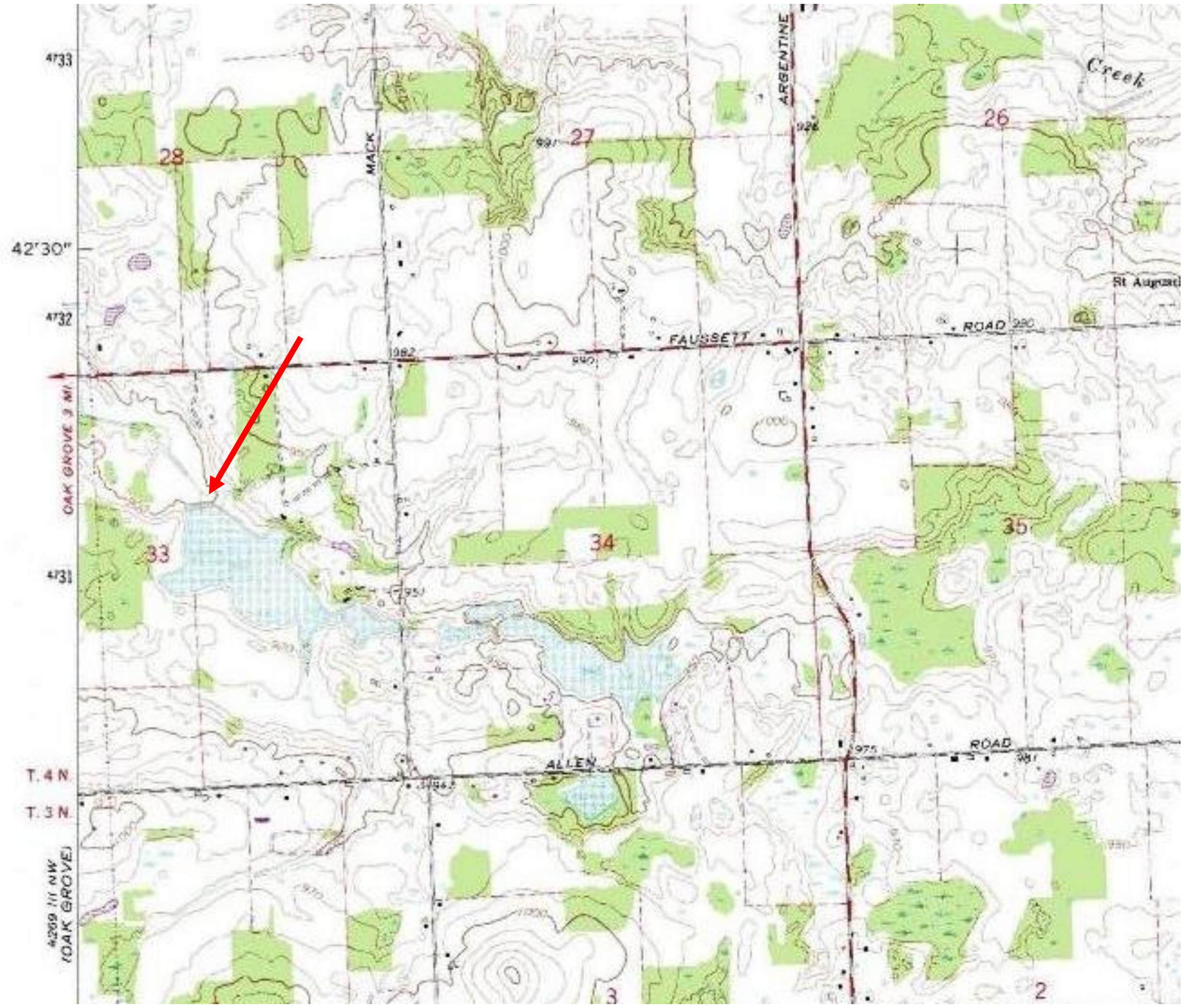
4.) Site Security	Private property
C.) Emergency Action Plan	N/A
1.) Warning systems	N/A
2.) Notification Networks	N/A
GENERAL AREAS	
A.) Reservoir	
1.) Unique Features	None
2.) Dead trees, debris in reservoir	None Observed
B.) Shoreline	
1.) Erosion	Small amounts present on East side of dam
2.) Vegetation	Good vegetation for wave energy dissipation
3.) Wave Protection	Riprap and vegetation
C.) Upstream Watershed	
1.) Historic Development	Wooded/ agricultural/ residential
2.) Present Development	Wooded/ agricultural/ residential
3.) Proposed Development	Residential
D.) Downstream Floodplain	
1.) Flood Protection	Natural, low, wide floodplain
2.) Historic, present, proposed development	Wooded, farms, residential
3.) Channel restrictions	Road crossing on Faussett Rd.
E.) Structural Stability	
1.) Embankment	Fair
2.) Drop structure	Could not visualize
3.) Outlet structure / pipe(s)	24” to 36” - Fair
4.) Overall	Fair
Further Detailed Studies?	Yes
If yes, explain	See report page 5
GENERAL COMMENTS/OVERALL CONDITIONS	Fair
Recommend Changing Hazard Classification?	YES [✓ **NO**]

DAM INSPECTION PREPARATION CHECKLIST	
Requested / Received information from EGLE?	Yes
Construction Plans, specifications,	
Operations and Maintenance records,	With operator
Design Calculations, Any Information	With EGLE
Requested/ Received from OWNER?	
Construction Plans, specifications,	With operator and EGLE
Operations & Maintenance records,	With operator
Design Calculations, Any Information	N/A
Set Date of Inspection with owner?	Yes
Weather, Owner Availability?	Sunny, 80° F, operator present
Operation of Gates/ Outlet Works?	Last operated last year
What special instrumentation will be necessary?	
Piezometers, Alignment Markers	N/A
Benchmarks, Established Elevations	N/A
Equipment:	
Survey (level or hand level, tripod, level rod):	Leica GG04
Measuring equip. (tapes, rules, sounding lines):	X
Marking Paint/ Keel	X
Paper/ Pencil	X
Calculator	X
Scale	X
Map	X
Measuring Wheel	X
Seepage (bucket and timer)	
Camera/ Film	X
Binoculars	
Probe (undercutting) / Level Rod	X
Rubber Boots, Waders	X
Survey Field Book	X
File Folder	X
Boat	
Shovel	X
Hammer	X
Crack measuring device	X
Specialized Equipment (piezometers, etc.)	
Safety Equipment:	
Life Jacket	
Hard hats	X
Steel toed shoes	X
Lights	

Interview with Owner	Yes
Comments, Date	8/9/2022
Inspection Checklist/ Forms	X
POST INSPECTION CHECKLIST	
Interview with Owner/ Operator	Yes
Insure everything on inspection checklist is answered	
Call Emergency Coordinator	N/A
What is needed in EAP for the County?	N/A
Schedule a meeting with Owner and coordinator	
Prepare preliminary EAP (see EAP form)	N/A
Inundation map	
List of possible flooded facilities, structures, etc.	
Time frame of flood wave	
Meeting with Emergency Coordinator	N/A
Final EAP	N/A
Make revisions per results of meeting	
Preliminary Dam Inspection Report	8/9/2022
Submit preliminary dam inspection report to owner	December 2022
Final Dam Inspection Report	December 2022
Make revisions per owners request, if any	Yes

APPENDIX B

SITE LOCATION MAP



Location Map for Faussett Lake Dam (ID # 307) located in Deerfield Twp, Livingston County, Section 33; T.4N. – R.5E.; Located on the Yellow River.   Dam located by red arrow.

APPENDIX C

HYDRAULIC INFORMATION

Faussett Dam

Spillway Capacity

Service Spillway

Use weir equation: $Q = c\,p\,h^{3/2}$

when c = crest coefficient of 2.9

p = Perimeter of inlet $= 2\pi r$

h = head $= 2$ ft

$Q = 2.9(2\pi \times 1\text{ ft})(2\text{ ft})^{1.5} = 51$ cfs (Use 50 cfs)

Emergency Spillways

Left Abutment: $Q = CLH^{3/2}$

$= 2.7(24)(1.5)^{3/2} = 119$ cfs

Right Abutment: $Q = CLH^{3/2}$

$= 2.7(2.8)(1.5)^{3/2} = 13.8$ cfs

Total capacity at top of dam ≈ 307 cfs

APPENDIX D

PHOTOGRAPHS

DESCRIPTION: Animal burrow in bank

SEVERITY: Medium

DESCRIPTION: Animal burrows in bank

SEVERITY: Medium

DESCRIPTION: Looking east at top of embankment, bank well vegetated

SEVERITY: Low

DESCRIPTION: Looking east at vegetation growth in lake, bank well vegetated

SEVERITY: Low

DESCRIPTION: Animal burrows in bank

SEVERITY: Medium

DESCRIPTION: Looking south at vegetation in lake

SEVERITY: Low

DESCRIPTION: Looking east at upstream bank, well vegetated

SEVERITY: Low

DESCRIPTION: Looking downstream at spillway

SEVERITY: Low

DESCRIPTION: Looking from east, downstream slope of embankment

SEVERITY: Low

DESCRIPTION: Looking west at rock placed on bank, bank well vegetated

SEVERITY: Low

DESCRIPTION: Looking north at embankment and spillway, bank well vegetated

SEVERITY: Low

DESCRIPTION: Looking south at saturated ground on downstream embankment

SEVERITY: Low

DESCRIPTION: Looking east at toe drain on bottom of embankment

SEVERITY: Low

DESCRIPTION: Looking west at saturated ground on down-stream slope of embankment

SEVERITY: Low

DESCRIPTION: Looking east at downstream slope of embankment

SEVERITY: Low

DESCRIPTION: Sinkhole over outfall pipe

SEVERITY: Low

DESCRIPTION: Looking east at downstream toe of embankment, auxiliary spillway

SEVERITY: Low

DESCRIPTION: Looking upstream at headwall failure

SEVERITY: High

DESCRIPTION: Downstream watercourse

SEVERITY: Low

DESCRIPTION: looking downstream at culvert under access road

SEVERITY: Low

DESCRIPTION: Looking south at small area of seepage beginning near auxiliary spillway

SEVERITY: Medium

DESCRIPTION: Standpipe Control Structure

SEVERITY: Low

DESCRIPTION: Looking upstream transition from RCP to CMP in outlet

SEVERITY: Low

APPENDIX E

2015 STRUCTURAL INSPECTION REPORT

2022 SME GEOTECHNICAL REPORT

Faussett Lake Dam (ID#307)

Section 33; T.04N. - R.05E.

Deerfield Township, Livingston County, MI

Spicer Group was enlisted to inspect the outlet structure (36-inch corrugated metal pipe and concrete headwall) and provide recommendations on implementing repairs to this structure. Additionally, we were requested to provide a maintenance plan to reflect what should be done on the structure for five year time periods of 2016 to 2020, 2021 to 2025, 2026 to 2030, and 2031 to 2035 which will assist in long term planning for the earthen dam and its structures. This will assist in assuring the dam's structural integrity and provide for a longer lifespan and assist in providing residents of the lake with information for planning, financing repairs, and maintenance in the future. This document is not based on a comprehensive, detailed engineering study of the structure, but is to be used as a guidance document for future planning.

Abbreviations Used:
CMP – Corrugated Metal Pipe
DW-HDPE – Dual Wall High Density Polyethylene Pipe
MDEQ – Michigan Department of Environmental Quality
RCP – Reinforced Concrete Pipe
H – Horizontal
V – Vertical

Maintenance Items- Outlet Structure

Spicer Group has provided two preliminary cost estimates for correcting the dam's outlet structure. These two cost estimates can be seen in Appendix B. The first cost estimate involves the removal of the existing concrete headwall structure and replacing it with stone riprap. The disturbed area will have a heavy 16-ounce non-woven geo-textile fabric with plain riprap on top of the fabric. This riprap is described as angular limestone or field stone that is 12 to 18 inches in its greatest diameter. The spillway that it enters, immediately downstream of the pipe should be at least 9 to 10 feet in width. The end of the pipe should have an end section with a 2H: 1V slope to match into the existing bank. The riprap should extend above the outlet pipe by at least 6 feet, and wrap around the sides of the pipe. We have included a detail in Appendix C to show an example of how this should be completed. The addition of the riprap will correct the erosion issue on the western side of the headwall structure as well. The end section will also aid in providing a flow transition from the pipe to the stream and hopefully prevent erosion downstream of the pipe.

The second cost estimate includes the removal and replacement of the existing concrete headwall structure.

Both cost estimates include the replacement of the low flow draw down valve structure; the application of a coating onto the metal water valve will not correct the existing corrosion issue or resist the accelerated corrosion. The replacement of this valve is not too costly and should be a task that is easy to complete. The concrete block structure that houses the valve should be replaced with a new structure providing easy access and weather proofing to improve longevity and decrease future maintenance.

Additionally, from reviewing past dam inspection reports and performing a cursory assessment during the site inspection on June 5th, it is apparent that the seepage issue on the downstream western face of the dam needs further assessment as the condition has not improved over time. The seepage issue is outside of the scope of work proposed in the letter agreement and we can only highly recommend the evaluation of this seepage area. We feel that attention to this issue is essential considering this could create a potential for dam failure. If this seepage is a concern to be addressed, we will want to discuss this with you in more detail; the option for adjusting the scope of services and fee structure is available. This type of investigation will require the services of a geo-technical engineering firm with experience in dams with seepage conditions.

Maintenance Recommendations

Maintenance recommendations over the next **five years (2016 – 2020):**

1. Complete a video camera assessment through the inlet / outlet structure (from the lake to the end of the outlet pipe in the creek) to determine the integrity of these structures.
2. Continue to monitor the corrosion of the 36-inch corrugated metal pipe (CMP) outlet structure; we recommend replacing the 36-inch CMP outlet structure and using a more hydraulically efficient pipe such as a dual wall high density polyethylene pipe (DW-HDPE) or a reinforced concrete pipe (RCP). These types of pipe are corrosion resistant and have a longer life spans than a CMP. Additionally, the pipe size can be decreased in diameter because the interior of these pipes are smoother (less friction). It may also be possible to "slip-line" the pipe or to use some of the newer lining technology and techniques that can be completed without removing the existing pipe. This will add life to the pipe but it has to be done by an experience contractor.
3. Complete the repair of the outlet structure by either replacing the headwall or removing the headwall and putting an end section on the pipe with placement of riprap as shown on the detail provided in Appendix C.

4. The low flow water valve must be replaced during this time frame. Once the low flow valve has been replaced, manually operate the low flow water valve annually to keep it functioning well. This valve is not extremely expensive and should be replace during this first 5 year time frame. Also, a weather resistant housing should be constructed for it.
5. Continue to monitor the seepage along the downstream face of the dam for soil slumping, piping, and boils. Before the seepage becomes progressively worse, we recommend beginning a study on the cause of the seepage. It would be prudent to approach a few geo-technical firms and send them a "Request for Qualifications" to obtain information from them as to their experience with dams and seepage. A study by a geo-technical firm will obtain a better understanding of the structure and existing soil profile. Also, we feel a seepage study to determine actual quantitative measurements of the ground flows will be beneficial. The application of meters to measure flows or use of ground penetrating radar should provide data to help realize the extent and urgency of this seepage issue. Obtaining quantitative data will also assist in the design phase to correct this issue as it will determine the type(s) of solutions or seepage management practices to incorporate to provide for longevity of the dam.
6. Continue to monitor the upstream slope of the dam where beaching has occurred. Monitor for active erosion after significant wind events, primarily from the south, southeast, or southwest. Restore protection of the upstream face of the dam with riprap or another forms of wave protection. Besides riprap, log booms or coir logs can be used to absorb the wave energy and lessen the erosive forces.
7. Removal of the brush, vines, and small trees within the western emergency spillway to improve the integrity and flow capacity of this overflow spillway.
8. The required five year inspection will need to be performed. (Required by the MDEQ for low hazard priority dams such as the Faussett Lake Dam).
9. Once all studies and assessments are completed, determine which actions will need to have a Joint MDEQ Permit filled out. These permits are valid for 5 years; it would be effective to plan to line up the tasks and enter all onto one permit if possible so the work can be done over a longer time period. This will save on permit application fees.
10. Periodic inspections and vegetation maintenance will need to be performed by the dam maintenance operator(s).

<u>Maintenance recommendations for the next **ten years (2021 – 2025)**:</u>

1. If not already completed, we recommend the replacement of the 36-inch CMP outlet structure with a DW-HDPE. The type of material and life expectancy for the system should be considered prior to any pipe replacement(s). For example, a CMP only lasts for approximately 50 years,

whereas a RCP can last for approximately 100 years or longer. It would be beneficial to obtain up to date cost estimates to assist with the decision making.

2. Depending on the results of the video inspection, determine if the existing 24-inch outlet pipe from the inlet to where it meets with the existing 36-inch CMP should be assessed for leakage, corrosion, and general life expectancy and begin planning for failure of the pipe. Again, look at current lining technologies for the pipes and obtain cost estimates from contractors for this type of work to assist with decision making.
3. Continue to exercise the low flow water valve annually.
4. Evaluate the outlet structure for erosion at this time to assure that the new outlet is working well.
5. Assess the inlet structure to assure it is in satisfactory condition for the next ten years. Items to address are the following: Is the trash rack in satisfactory condition and is the concrete of the inlet structure in satisfactory condition?
6. The seepage study should be completed by this time and a plan to address the seepage should be implemented at this time, unless already addressed prior to this timeframe. Implementation of this action may require a permit from the MDEQ for repairs. Spicer Group can assist in preparing the permit and coordinate obtaining the permit through the MDEQ.
7. Upon completion of the soil borings, the following options may be feasible: a clay / bentonite blanket, a liner on the upstream face of the dam to act as a physical barrier to slow the seepage down, or a bentonite trench injection to block the flow. Once the seepage has been controlled, we recommend the removal of topsoil and cattail rhizomes. Back fill the area with an engineer approved fill material that is layered / compacted to prevent any soil particle losses. A non-woven geo-textile fabric should be placed with plain riprap on top of the fabric. Complete an application of an MDEQ Joint Permit with the application of the clay / bentonite blanket on the upstream face of the dam (The drawdown of Faussett Lake is necessary to apply the material in a manner to assure a consistent layer on the upstream slope and lake bottom; drawdowns require a permit).
8. Monitor the upstream slope of the dam where beaching has occurred. Monitor for active erosion after significant wind events. Restore protection of the upstream face with riprap or another form of wave protection such as vegetation establishment or other energy absorption methods.
9. Remove any debris, brush or small trees from the emergency spillway, downstream channel of the dam, embankments, etc. that have accumulated/grown in the previous five years.
10. Five year inspection required by the Michigan Department of Environmental Quality (MDEQ).
11. Periodic inspections (annual) performed by the dam maintenance operator(s).

Maintenance recommendations for the next **time period (2026 to 2030)**:

1. Reassess studies completed to date and evaluate what future work may be needed. Meet with your engineer to review and update this plan.
2. Remove any debris, brush or small trees from the emergency spillway, downstream channel of the dam, embankments, etc. that has accumulated or grown since the last cleanout.
3. If the inlet / outlet pipes have not had any work done on them to date, they may need a video camera assessment again to determine the condition and if replacement or relining needs to be completed.
4. Exercise the low flow water valve annually.
5. Five year inspection required by the Michigan Department of Environmental Quality (MDEQ) and will be need to be completed during this time period.
6. Periodic (annual) inspections performed by the dam maintenance operator(s).

Maintenance recommendations for the next **time period (2031 to 2035)**:

1. Reassess studies completed to date and evaluate what future work may be needed. Meet with your engineer to review and update this plan.
2. Remove any debris, brush or small trees from the emergency spillway, downstream channel of the dam, embankments, etc. that has accumulated or grown since the last cleanout.
3. Exercise the low flow water valve annually.
4. Five year inspection required by the Michigan Department of Environmental Quality (MDEQ) and will be need to be completed during this time period.
5. Periodic (annual) inspections performed by the dam maintenance operator(s).

APPENDIX A

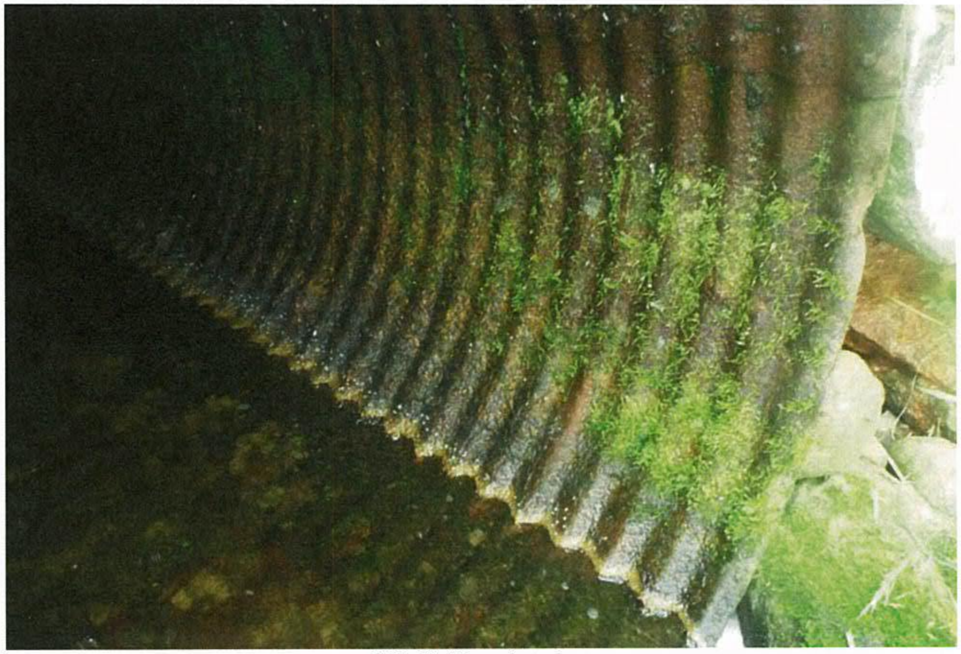
Seepage along western portion of downstream slope of Faussett Lake Dam.

Looking east along downstream slope of Faussett Lake Dam.

36" Outlet Structure and Concrete Headwall.

Looking west along downstream slope of Faussett Lake Dam.

4-foot by 4-foot concrete block, location of low flow draw down valve.



36" CMP outlet pipe.

4-foot by 4-foot concrete block, location of low flow draw down valve.

Downstream face of dam, looking northeast.

Downstream face of dam, looking upstream.

Downstream face of dam, looking west.

Downstream face of dam, looking north.

Cracked concrete headwall structure.

APPENDIX B

PRELIMINARY ESTIMATE OF COST

Faussett Lake Dam (ID #307)

Deerfield Township, Livingston County, Michigan

Concrete Structure Inspection Performed on June 5, 2015

Item No.	Estimated Quantity	Unit	Description	Unit Price	Amount
OUTLET STRUCTURE REPAIR					
1.	1	Lump Sum	Faussett Lake Flow Control / Water Diversion	$2,000.00	$2,000.00
2.	1	Lump Sum	Remove Existing Concrete Headwall Structure	$5,000.00	$5,000.00
3.	70	Sq. Yds.	Field Stone Plain Riprap; includes non-woven fabric	$60.00	$4,200.00
4.	60	Lin. Ft	24-inch DWHDPE	$55.00	$3,300.00
5.	1	Lump Sum	Replace Existing Valve Structure	$900.00	$900.00
6.	1	Lump Sum	Cleanup and Site Restoration	$2,600.00	$2,600.00
				SUB-TOTAL	**$18,000.00**
SOIL EROSION AND SEDIMENT CONTROL					
1.	75	Lin. Ft.	Silt Fence	$4.00	$300.00
2.	1	Lump Sum	Grading, Topsoil and Seed	$700.00	$700.00
				SUB-TOTAL	**$1,000.00**
SUB-TOTAL CONSTRUCTION COST--					**$19,000.00**
Survey, Engineering, Design					$3,500.00
MDEQ Permit & Fees					$1,200.00
SUB-TOTAL --					**$4,700.00**
TOTAL PRELIMINARY ESTIMATE OF COST ---					**$23,700.00**

PRELIMINARY ESTIMATE OF COST
Faussett Lake Dam (ID #307)
Deerfield Township, Livingston County, Michigan
Concrete Structure Inspection Performed on June 5, 2015

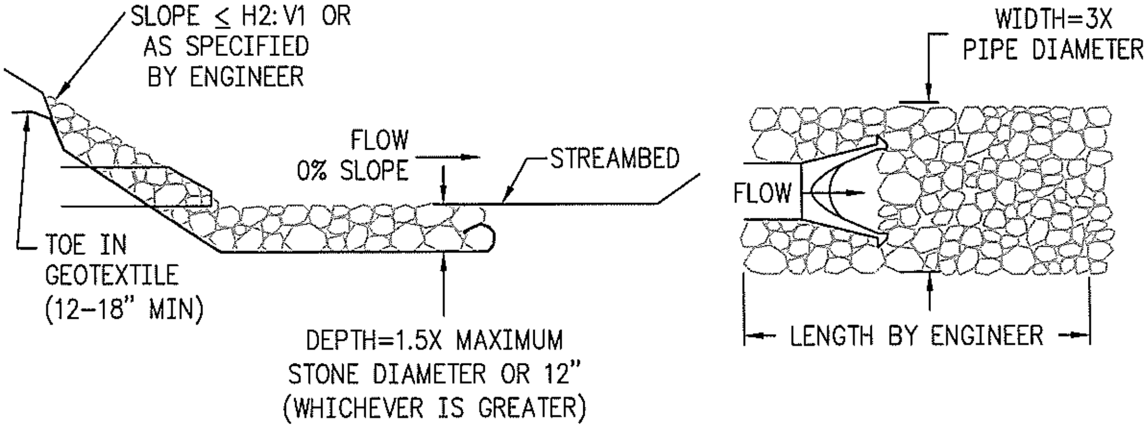
Item No.	Estimated Quantity	Unit	Description	Unit Price	Amount
OUTLET STRUCTURE REPAIR					
1.	1	Lump Sum	Faussett Lake Flow Control/Water Diversion	$2,000.00	$2,000.00
2.	1	Lump Sum	Remove Existing Concrete Headwall Structure	$5,000.00	$5,000.00
3.	1	Lump Sum	Replace Existing Concrete Headwall Structure *Including site work and mobilization*	$25,000.00	$25,000.00
4.	60	Lin. Ft	24-inch DWHDPE	$55.00	$3,300.00
5.	1	Lump Sum	Replace Existing 4 inch Valve	$900.00	$900.00
6.	1	Lump Sum	Cleanup and Site Restoration	$3,600.00	$3,600.00
				SUB-TOTAL	**$39,800.00**
SOIL EROSION AND SEDIMENT CONTROL					
7.	65	Sq. Yds.	Field Stone Plain Riprap	$60.00	$3,900.00
8.	75	Lin. Ft.	Silt Fence	$4.00	$300.00
9.	1	Lump Sum	Grading, Topsoil and Seed	$900.00	$900.00
				SUB-TOTAL	**$5,100.00**

SUB-TOTAL CONSTRUCTION COST	**$44,900.00**
Survey, Engineering, Design	$6,400.00
MDEQ Permit & Fees	$2,000.00
SUB-TOTAL	**$8,400.00**
TOTAL PRELIMINARY ESTIMATE OF COST	**$53,300.00**

PRELIMINARY ESTIMATE OF COST
Faussett Lake Dam (ID #307)
Deerfield Township, Livingston County, Michigan
Annual Operation & Maintenance

Item No.	Estimated Quantity	Unit	Description	Unit Price	Amount
VEGETATION MANAGEMENT					
1.	3	Lump Sum	Mowing, vegetation mangement of crown, embankments, toe of slope (3 times per year) (Lump sum each mowing event)	$450.00	$1,350.00
				SUB-TOTAL	**$1,350.00**
REPAIR OF DOWNSTREAM SLOPE / OUTLET STRUCTURE					
1.	1	Lump Sum	Seeding	$50.00	$50.00
2.	1	Lump Sum	Outlet Channel Cleanout	$500.00	$500.00
3.	10	Sq. Yds.	Plain Rip Rap *(as necessary)*	$60.00	$600.00
				SUB-TOTAL	**$1,150.00**
REPAIR OF UPSTREAM SLOPE EROSION					
8.	30	Sq. Yds.	Misc. Plain Riprap Protection *(as necessary)*	$60.00	$1,800.00
				SUB-TOTAL	**$1,800.00**
CLEANOUT OF EMERGENCY OVERFLOW CHANNEL					
9.	1	Lump Sum	Clear trees, vines, and brush, cut selected overstory from 95 feet of channel	Lump Sum	$1,500.00
10.	1	Lump Sum	Site clean up and seeding	$200.00	$200.00
				SUB-TOTAL	**$1,700.00**
				TOTAL	**$6,000.00**

APPENDIX C



Source Adapted from State of Michigan, Department of Management and Budget, SESC Guidebook

GEOTECHNICAL EVALUATION REPORT

FAUSSETT LAKE DAM
DAM ID 307
DEERFIELD TOWNSHIP, LIVINGSTON COUNTY, MICHIGAN

SME Project No. 088000.00
April 7, 2022

SME

3301 Tech Circle Drive
Kalamazoo, MI 49008-5611

T (269) 323-3555

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April 7, 2022

Mr. Richard V. Graham, PE
Project Manager
Spicer Group, Inc.
125 Helle Boulevard, Suite 2
Dundee, Michigan 48131

Via E-mail: richg@spicergroup.com

RE: Geotechnical Evaluation
Faussett Lake Dam
Dam ID 307
Deerfield Township, Livingston County, Michigan
SME Project No. 088000.00

Dear Mr. Graham:

The accompanying report summarizes the geotechnical evaluation performed by SME to assist Spicer Group in the development of the design of repairs and improvements to Faussett Lake Dam. This report presents the geotechnical information collected by SME, describes the repair concepts considered, and summarizes the slope stability analyses for the selected concept completed by SME to develop design details. This report also presents other pertinent design and construction considerations based on the geotechnical information collected and our experience with similar dams.

We appreciate the opportunity to be of service. If you have any questions or require additional information, please contact me.

Sincerely,

SME

Jeffery M. Krusinga, PE, GE
Principal Consultant

Distribution: Mr. Shawn Middleton, PE, CFM – Spicer Group
Via e-mail: shawnm@spicergroup.com (PDF file)

TABLE OF CONTENTS

1. INTRODUCTION

This report presents the results of the geotechnical evaluation performed by SME to assist Spicer Group (Spicer) with the development of plans for repairs and improvements to Faussett Lake Dam. Our services for the project were performed in general accordance with the scope outlined in SME Proposal No. P01020.20, dated August 20, 2020, and with a revised date of June 23, 2021. Spicer authorized SME's services for this project.

To assist with our evaluation, Spicer provided SME the following documents related to the dam:

- A report titled "Faussett Lake Dam Inspection Report", dated June 2007, prepared by Spicer.
- A report titled "Faussett Lake Dam Inspection Report", dated September 21, 2012, prepared by Spicer.
- A report titled "Faussett Lake Dam Inspection Report", dated May 15, 2017, prepared by Spicer.
- Sixty-one photographs of the dam obtained from Ms. Nancy Gregory, a member of the family that constructed the dam. Labels on these photographs indicate a range of photograph dates from October 1960 to June 1968. The earliest photographs depict conditions during original construction of the dam, while the later photographs depict the dam under operational conditions.
- A plan set (marked "Preliminary") for Fausset Lake Dam (Sheets DR 01 through DR 05, dated January 2022), prepared by Spicer. Sheet DR 04 of this plan set is titled "Existing Conditions" and contains topographic information (1-foot contours) and existing site features superimposed onto an orthophoto image.

1.1 GENERAL DAM INFORMATION

Faussett Lake Dam is located in Section 33 of Deerfield Township in Livingston County, Michigan. The dam is located on the Yellow River Drain and impounds Faussett Lake, which has a normal surface area of about 74 acres. The dam is situated about 1,700 feet south of Faussett Road, west of Mack Road and east of Latson Road. The location of the site is depicted on the Location Map inset contained on the Boring Location Diagram (Figure No. 1) included in Appendix A. Access to the dam site is provided via a private, unpaved driveway that extends south from Faussett Road.

The subject dam is regulated by the Dam Safety Unit, Water Resources Division, of the Michigan Department of Energy, Great Lakes, and Environment (EGLE) under the authority of Part 315, Dam Safety, of the Natural Resources and Environmental Protection Act, 1994 PA 451, as amended (Part 315). The dam is currently assigned a "low" hazard potential classification by EGLE, and is assigned Dam ID No. 307.

Faussett Lake Dam is an earthen embankment dam. The embankment is about 560 feet long, and the embankment crest is roughly 10 feet in width (minimum). The crest elevation of the dam ranges between about 906.5 feet and 907.5 feet. The water surface elevation of Faussett Lake was measured by Spicer as 904.99 feet on November 11, 2021. The conditions along the embankment crest are depicted on Images 1 and 2 below. The vertical height from the embankment toe to the crest is a maximum of about 19 feet. The upstream slope of the embankment has a reported inclination of 2:1 (horizontal to vertical). The downstream slope of the embankment has a maximum inclination of about 3:1. A private unpaved driveway extends along the toe of the embankment in an east-west direction, and provides access to a residence at the northwest corner of Faussett Lake.

A drop inlet structure in the impoundment serves as the primary spillway. The drop inlet consists of a 24-inch-diameter corrugated metal pipe (CMP) that is located within the reservoir a short distance upstream from the embankment. The top of the drop inlet is protected by a metal rail and is marked with a buoy (refer to Images 1 and 2 below). The drop inlet connects to a 24-inch-diameter CMP that extends under the embankment to within about 40 feet to 50 feet of the outlet, at which point a 36-inch-diameter CMP collects the water and conveys the flow to a concrete headwall at the discharge point into the downstream channel. Two secondary spillways, located at the east and west ends of the embankment, consist of grass-covered, fixed-crest weirs notched into the embankment. The combined flows of the dam spillways are required to pass the peak discharge associated with a 100-year storm (1-percent chance event). Based on the 2017 Faussett Lake Dam Inspection Report, the peak discharge associated with the 100-year storm is 250 cubic-feet per second (cfs), and the 1-percent chance flood volume is 340 acre-feet. The primary (drop inlet) spillway has a maximum capacity of 50 cfs. The 2017 Faussett Lake Dam Inspection Report indicates there is normal storage volume that exceeds the volume associated with the 1-percent chance flood volume.

Image 1: Looking east along the crest of the dam embankment (photo date: 12/7/2021).

Image 2: Looking west along the crest of the dam embankment (photo date: 12/7/2021).

The dam was constructed by the Gregory family between 1955 and 1963. The embankment was constructed from local borrow material and reportedly consists of clays. However, there have been no geotechnical borings previously completed at the site to verify embankment soil conditions. We are not aware of any construction drawings or design plans associated with the original construction of the dam, or any previous geotechnical analyses completed on the dam. The current listed owner of the dam is Walker Reservoir, LLC, with Ms. Nancy Gregory as the contact.

The impoundment (lake) behind the dam provides recreational opportunities, such as boating, fishing, swimming, and wildlife viewing. Several residences have been constructed on the lake since the construction of the dam. We understand a Special Assessment District (SAD) has been established to manage the costs for operation and maintenance of the dam.

1.2 PROJECT DESCRIPTION

The 2017 Faussett Lake Dam Inspection Report identified several deficiencies that require maintenance, repairs, or further study. Spicer has been retained to take the lead on addressing the deficiencies, except for further study of the seepage on the downstream face of the embankment at the western end. Spicer retained SME to evaluate the reported seepage condition and develop a concept-level measure (or measures) to address or repair this condition.

The area of subject seepage is located along the western portion of the downstream embankment, and generally located within the lower half of the embankment. The approximate area of the seepage is denoted on Figure No. 1. The seepage area is estimated to extend over an area measuring roughly 60 feet by 35 feet. The presence of seepage in this area was denoted in the 2007 Faussett Lake Dam Inspection Report, the 2012 Faussett Lake Dam Inspection Report, and the 2017 Faussett Lake Dam Inspection Report.

Images 3, 4, and 5 below depict the seepage area at the time of the field exploration associated with our evaluation. By the summer time of each year, cattails are present within the area of seepage. The embankment conditions at the time of our evaluation reflect conditions after completion of routine maintenance mowing of the embankment. The seepage area is noticeably soft and wet, and water from the seepage persistently flows into the small ditch between the toe of the embankment and the driveway. Based on our rough measurements in the field, we estimate the top of the seepage area is roughly at about elevation 898 feet.

Image 3: Looking east along the driveway at the toe of the embankment. The seepage area is visible to the right of the driveway in the background of the pink stake (photo date: 12/7/2021).

Image 4: Looking west along the driveway at the toe of the embankment. The seepage area is visible to the left of the driveway (photo date: 12/7/2021).

Image 5: Looking northeast at the seepage area. The blue bucket is located near the upstream edge of the seepage (photo date: 12/7/2021).

2. EVALUATION PROCEDURES

2.1 FIELD EXPLORATION

SME completed four borings (B1 through B4) along the embankment of the dam. Borings B1, B3, and B4 were completed on December 10, 2021. Boring B2 was completed on December 7, 2021. The approximate locations of the borings are depicted on Figure No. 1.

SME determined the planned number, depths, and locations of the borings. SME located the borings in the field by referencing existing site features. SME collected coordinates at the boring locations using a GPS unit with sub-meter accuracy. SME estimated the existing ground surface elevations at the boring locations to the nearest ½-foot based on site topographic information provided by Spicer.

Borings B1, B3, and B4 were drilled using a rotary-type drill rig mounted on an all-terrain vehicle (ATV). These borings were advanced using continuous-flight, hollow-stem augers and included soil sampling based upon the Split-Barrel Sampling procedure. Thin-walled Shelby tube samples (3-inch-diameter) were also collected at selected intervals from borings B1, B3, and B4. Recovered split-barrel samples were sealed in glass jars by the driller. The ends of the Shelby tube samples were sealed with wax in the field, and then the ends were capped with plastic caps.

An SME engineer advanced boring B2 manually by using a hand auger. Samples of cuttings collected from the auger bucket were sealed in plastic bags. A dynamic cone penetrometer (DCP) test was also performed in conjunction with the advancement of boring B2. The DCP test consists of steel rod with a 1⅛-inch-diameter conical tip that is driven into the subgrade with a sliding, 10-pound hammer falling 24 inches. The number of hammer blows required to advance the rod are recorded for every 6 inches of penetration, and these hammer blows (penetration resistances) are used to evaluate the consistency or relative density of the subgrade.

Groundwater observations in the boreholes were recorded during drilling and immediately after completion of drilling at each boring. The boreholes at borings B1, B3, and B4 were backfilled with bentonite-cement grout that was placed using the tremie method. The borehole at boring B2 was backfilled with bentonite chips.

Soil samples recovered from the field exploration were returned to the SME laboratory for further observation and testing.

2.2 LABORATORY TESTING

The laboratory testing program consisted of performing visual soil classification on recovered samples in general accordance with ASTM D-2488. Moisture content tests and hand penetrometer or Torvane shear tests were performed on portions of cohesive samples recovered from the borings. Loss-on-ignition (LOI) tests were performed on selected samples from the borings to evaluate the organic matter content of the soil. In addition, particle size analyses (sieve and loss-by-wash) were performed on three recovered samples to assist with soil classification. The Laboratory Testing Procedures in Appendix B provides descriptions of laboratory tests. Based on the laboratory testing, we assigned a group symbol to the various soil strata encountered based on the Unified Soil Classification System (USCS).

Upon completion of the laboratory testing, we prepared boring logs that include the soil descriptions, penetration resistances, and pertinent observations made during the field exploration. Results of the laboratory testing are also contained on the boring logs, except for the results of the particle size analyses which are contained on the Particle Size Distribution Reports in Appendix A (Figure Nos. A1 through A3). The existing ground surface elevations at the boring locations, as estimated by SME, are also provided on the boring logs, and the coordinates collected by SME at the boring locations are contained on the boring logs. The boring logs are included in Appendix A. Explanations of symbols and terms used on the boring logs are provided on the Boring Log Terminology sheet included in Appendix A.

The Standard Penetration Test (SPT) resistances (N-values) plotted on the logs for borings B1, B3, and B4 represent a modified N-value based on the correlation between the recorded SPT value and the measured hammer efficiency of the testing equipment (also shown on the boring logs). Specifically, the plotted N-values have been normalized to a 60 percent hammer efficiency (N_{60}).

Soil samples retained over a long time, even sealed in jars, are subject to moisture loss and are no longer representative of the conditions initially encountered in the field. Therefore, soil samples are normally retained in our laboratory for 60 days and are then disposed of, unless instructed otherwise.

3. SUBSURFACE CONDITIONS

3.1 SOIL CONDITIONS

The soil conditions encountered at the borings generally consisted of sand fill containing variable amounts of fines, such as silt and clay (with USCS classifications of “SM”, “SC”, “SP-SC”, and “SC-SM”), overlying natural lean clay (with USCS classification of “CL”) extending to the explored depths of the deeper borings (B1, B3, and B4). Soils consistent with the presence of a clay embankment or a clay embankment core **were not encountered** in borings B3 and B4, which were performed along the crest of the dam embankment.

The fill encountered at the borings is designated as “Embankment Fill” on the logs since the presence of this material is judged as associated with the construction of the embankment for the dam. At boring B4, located near the existing spillway and likely within the area of the original stream channel, the stratum below the Embankment Fill and above the underlying natural lean clay is designated as “Alluvium” since we believe this is material that was deposited naturally by water flowing in the original streambed and this material was not removed prior to placement of the overlying Embankment Fill.

The Embankment Fill at the two borings (B3 and B4) performed on the top or crest of the embankment extended to depths ranging from about 17 feet to 18 feet below the existing ground surface. In terms of elevation, the Embankment Fill at these two borings extended to about 889.5 feet. The Embankment Fill in these two borings was encountered in a very loose to medium dense condition, with N_{60} values ranging from 0 to 15 blows per foot of penetration (bpf). The sands associated with the Alluvium below the Embankment Fill in boring B4 were encountered in a loose to medium dense condition, with N_{60} values ranging from 6 to 12 bpf. The natural clays encountered below the Embankment Fill at borings B1, B3, and B4 exhibited a very stiff to hard consistency, with undrained shear strengths estimated from hand penetrometer tests ranging from 2.5 to greater than 4.5 kips per square-feet (ksf). Moisture contents in the clays ranged from about 10 to 14 percent.

At borings B1 and B2, the sands associated with the Embankment Fill extended to depths ranging from about 2.5 feet to 7 feet. Based on DCP resistances and one N_{60} value, the Embankment Fill was encountered in a very loose to medium dense condition at these two boring locations.

The soil profile described above and included on the appended boring logs is a generalized description of the conditions encountered. The stratification depths described above and shown on the boring logs are intended to indicate a zone of transition from one soil type to another. They are not intended to show exact depths of change from one soil type to another. The soil descriptions are based on visual classification of the soils encountered, and the results of laboratory tests. Soil conditions may vary between or away from the boring locations. Please refer to the boring logs for the soil conditions at the specific boring locations.

3.2 GROUNDWATER CONDITIONS

At boring B1, which was performed downslope of the seepage and near the ditch adjacent to the driveway, groundwater was encountered during drilling about 0.2 feet below the existing ground surface. After drilling at boring B1, groundwater was not observed in the borehole. The groundwater encountered at this boring is judged to be perched or trapped in the sands overlying the less permeable clays. At boring B2, groundwater was encountered at about 0.5 feet below the existing ground surface. At borings B3 and B4, groundwater was encountered during drilling about 4 feet to 6.5 feet below the existing ground surface. A reliable groundwater observation could not be collected in borings B3 and B4 after completion of drilling since these boreholes were tremie-grouted as the augers were extracted. Please refer to the boring logs for the specific groundwater conditions encountered in each boring.

Due to the flow of the groundwater from the reservoir, the depth or elevation of groundwater will vary across the dam embankment (measured perpendicular to the alignment of the dam crest), with the highest groundwater elevations expected nearest the reservoir or lake, and the lowest elevations expected closest to the embankment toe. However, these elevations depend on the permeability of the embankment fill which are in turn dependent on the gradation of fill.

The site groundwater levels, including perched conditions, should be anticipated to fluctuate throughout the year due to variations in precipitation, evaporation, surface runoff, and the surface water elevation in Faussett Lake. The groundwater conditions shown on the boring logs are based on the measurements made at the time the borings were drilled. Based on survey information provided by Spicer, the water surface elevation of Faussett Lake was 904.99 feet on November 11, 2021. During the SME field exploration on December 7, 2021, we estimated the seepage on the slope emanating at about elevation 898 feet, or about 7 feet below the lake level.

4. ANALYSIS AND RECOMMENDATIONS

4.1 GENERAL DISCUSSION

Noticeable seepage was emanating from the downstream slope of the embankment during our field exploration. The area of seepage is located within about the lower half of the embankment and toward the western end of the overall embankment. The conditions within this area are depicted on Images 3 through 5 above, and the approximate area where seepage is present on the downstream slope face is illustrated on Figure No. 1. At the time of our field exploration, this area was visibly wet and very soft, making even walking over this area difficult. Our engineer easily sank several inches into the subgrade while walking over the area. The downstream slope of the embankment had been mowed in the fall of 2021, prior to the SME field exploration performed in December 2021. However, photographs of the embankment contained in the referenced safety inspection reports indicate that cattails routinely grow in the area of seepage each summer This seepage has also been noted in safety inspection reports prepared by Spicer and dating back to 2007. Inspection reports from 1997 and 2002 are referenced in the 2007 Faussett Lake Dam Inspection Report prepared by Spicer, but copies of these previous reports were not provided for review to verify notes on potential seepage observations in 2002 and 1997.

The wet and soft conditions, described above, indicate the groundwater levels within the embankment (called the phreatic surface) is intercepting the surface of the embankment. Based on the soil conditions encountered in the borings performed for this evaluation, it is our opinion that seepage in the area of concern likely has been present since soon after the dam was constructed and the reservoir filled.

In general, seepage through or under dams is not necessarily an issue. Rather, it is the adverse consequences of seepage that are of concern. Earthen dams that are designed properly have drainage galleries and/or impermeable embankment cores designed to control or address seepage, and, more importantly, the potential adverse effects of seepage. Faussett Lake Dam, although reportedly constructed with clays, is not constructed as a “impermeable” clay embankment or with a clay embankment core based on the conditions encountered in borings B3 and B4. In addition, there are no observable or apparent drainage provisions that were incorporated into the embankment construction to address seepage, and there are no construction plans to verify details of embankment construction. There are mentions of “toe drains” in in the 2007, 2012, and 2017 Faussett Lake Dam Inspection Reports, but we believe this is a reference to drainage pipes/open ditches that run parallel to the driveway, between the embankment toe and the driveway, rather than internal drainage provisions that extend back into/under the embankment.

Based on the soil and groundwater conditions encountered in the borings performed for this evaluation, the source of the seepage water that emanates from the slope face is the reservoir or lake behind the embankment. Seepage that emanates from a slope face like the downstream embankment at Faussett Lake Dam is troubling because the presence of such seepage can lead to piping of the embankment soils. Piping is when seepage water that emanates from the slope face also carries with it particles of soil from the embankment. Active piping over time, if not addressed, can lead to internal erosion of the embankment and eventually, a sudden failure of a dam, especially during prolonged high water levels. The soil conditions encountered in boring B2, which was performed within the area of seepage, consisted of fine to medium silty sands in the upper approximate 6.5 feet of the profile. Silty sands are especially susceptible to piping since these soils are not cohesive and the fine silt particles can be transported (eroded) relatively easily by the seepage water as the seepage emanates from the slope face. Therefore, we agree with recommendations made in the 2007, 2012, and 2017 Faussett Lake Dam Inspection Reports that the seepage be studied and measures to address the seepage be implemented.

4.2 REPAIR CONCEPTS CONSIDERED

Based on our experience, we considered in concept two different repairs that could be implemented to address the seepage present at the dam. These two concepts are discussed in more detail in the following paragraphs.

The first concept involves the construction of a vertical barrier along the crest of the dam embankment. The top of the barrier would need to start just below the embankment crest (and above the normal lake level) and extend down and into the natural clays on which the embankment was constructed. Such a barrier would be composed of a relatively impermeable material, and because it would extend into the underlying relatively impermeable clays, would impede seepage that otherwise flows from the reservoir through the sandy embankment soils. The barrier for this concept could consist of a continuous, interlocking steel sheet pile wall or a continuous slurry wall.

The sheet pile wall would likely consist of steel sheets that would be installed by driving with an impact or vibratory hammer suspended from a crane. The overall width of the sheet pile wall for a Z-shaped sheet section would likely be about 12 inches to 16 inches. The slurry wall would consist of a mixture of the embankment soils, cement, and bentonite. The contractor would install the slurry wall in an approximate 3-foot-wide trench, with the soil, cement, and bentonite mixed together in-situ within the trench to form the slurry wall. For either type of barrier, the barrier would be required to toe into the underlying clays a minimum of 3 feet. A specialty geotechnical contractor would need to be retained to install the slurry wall, while a piling contractor would need to be retained to install the sheet pile wall.

The second concept involves the construction of a drained buttress on the downstream face of the embankment that would “cover” the area where seepage is present. The buttress material would be composed of a free-draining rock, such as riprap, placed on a non-woven geotextile fabric. The geotextile fabric would act as a filter to keep the embankment soils in place as the seepage water emanates from the embankment. The riprap serves as ballast to hold the fabric in place and to allow the seepage water to drain down to the toe of the embankment slope and be collected into a drainage gallery. This second concept does not attempt to stop or reduce the seepage, but only to address the potential negative consequences of adverse seepage (i.e., to mitigate the potential for piping).

Based on our experience with anticipated costs, we believe the second concept (the drained buttress) would be less expensive to design and implement than the first concept (the vertical barrier). However, for the barrier concept there should not be long-term maintenance required, whereas for the drained buttress, some cleaning of the recommended collector drain at the toe of the buttress may be necessary over time.

For the vertical barrier concept, relatively large equipment would need to access the crest of the embankment to construct the slurry wall or install the sheeting. The relatively narrow width of the embankment crest could make access for equipment prohibitive. A piling contractor or a specialty geotechnical contractor would need to be retained to install the barrier associated with the first concept. A typical earthwork contractor would likely be able to construct the drained buttress associated with the second concept. Therefore, based on expected relative cost, and on our anticipation that more earthwork contractors would be available to bid/perform the work compared to the number of piling contractors or specialty geotechnical contractors, we recommend the project team pursue the drained buttress option to address the seepage issue.

4.3 SLOPE STABILITY ANALYSES

To further the development of the design for the drained buttress concept, we evaluated slope stability at a cross section through the embankment at the location of the seepage. The location of this cross section (Cross Section A-A) is shown on Figure No. 1. We performed the stability analyses to evaluate the factor-of-safety (FOS) against slope failure at the cross section location for existing conditions, and to evaluate the FOS against slope failure for proposed conditions (i.e., after construction of a drained buttress).

The analysis of the stability of an embankment requires the determination of surface and embankment geometry, the phreatic surface or seepage conditions within the embankment, and soil properties (shear strength parameters and unit weight). SME developed the surface geometry at the cross section location from the topographic information contained on the referenced Existing Conditions plan sheet. SME developed the subsurface soil profile at the cross section location using the data obtained from borings B1, B2, and B3 performed as part of this evaluation. SME estimated the phreatic surface within the embankment (i.e., the line of seepage) from groundwater observations collected from borings B1, B2, and B3, and from the water surface elevation of the impoundment, which was surveyed as 904.99 feet on November 11, 2021 (we rounded this observation to 905.0 feet in our analyses). Soil properties, such as shear strength and total unit weight, were developed based on the samples collected during drilling, the laboratory testing performed, and our experience with similar soil conditions. For our slope stability analyses, we used drained soil shear strengths versus undrained soil shear strengths since in our experience, drained soil strengths control the analysis for long-term conditions where fine-grained soils are present. Table 1 below summarizes the soil parameters (total unit weight and shear strength) used in our slope stability analyses.

TABLE 1: SOIL PARAMETERS USED IN SLOPE STABILITY ANALYSES

MATERIAL DESCRIPTION	UNIT WEIGHT (PCF)	COHESION (PSF)	FRICTION ANGLE (DEGREES)
Embankment Fill – CLAYEY SAND (SC) – Very Loose	120	0	26
Embankment Fill – CLAYEY SAND (SC) and SAND with Clay (SP-SC) – Loose	120	0	28
Embankment Fill – SILTY SAND (SM) – Very Loose	120	0	26
Natural Lean CLAY (CL) – Very Stiff to Hard	130	200	28
RIPRAP	135	0	45

Notes:
1. The shear strengths shown are drained shear strengths and not undrained shear strengths.
2. PCF is pounds per cubic-foot and PSF is pounds per square-foot.
3. The Riprap only appears in the analyses where a drained buttress is present.

After the cross section geometry was developed, the subsurface stratigraphy applied, and the phreatic surface was estimated, we imported this model information into the slope stability program SLIDE. We then used the SLIDE program to evaluate the FOS of the slope at this cross section location. Based on the composition of the embankment, circular failure surfaces were considered for the slope stability analyses since this type of failure (as opposed to a sliding block failure) was considered most appropriate. Both the Bishop method and the Janbu method for evaluation of the FOS for theoretical circular failure surfaces were utilized in the slope stability analyses. For existing conditions at Cross Section A-A, we calculated a minimum FOS of 1.4 against slope failure. The critical failure surface, i.e., the theoretical circular failure surface with the lowest FOS, is shown on Figure No. C1 (Existing Conditions at Cross Section A-A) in Appendix C. We then altered this existing conditions slope stability model and added the geometry associated with the construction of a drained buttress on the face of the slope in the area of the seepage. With a drained buttress present, we calculated a minimum FOS of 1.7 against slope failure. The location of the critical failure surface is higher up on the embankment than the critical failure surface calculated for existing conditions. For the same critical failure surface determined for existing conditions, the FOS for this surface increases to 2.8 when the buttress is added to the slope stability model. The critical failure surface with the presence of the drained buttress is shown on Figure No. C2 (Cross Section A-A With Drained Buttress) in Appendix C. The same critical failure surface as determined for existing conditions but with the drained buttress present is also shown on Figure No. C2.

From our slope stability analyses, we developed several observations and conclusions:

1. The minimum FOS for existing conditions is 1.4 (refer to Figure C1). The U.S. Army Corps of Engineers (USACE) provides guidelines in their Slope Stability Engineer Manual (EM 1110-2-1902, dated October 31, 2003) for acceptable factors of safety for slopes associated with dams. Specifically, Table 3-1 provides guidelines for factors of safety for various cases for new dams. Based on our review of this table, the required minimum FOS for the Long-Term, Steady State conditions is 1.5. Our stability analyses at the cross section location where seepage is present indicate the recommended minimum FOS is not met (1.4<1.5), although there could be circumstances when a lower FOS is considered acceptable.
2. The soils encountered within borings B2 and B3, which were performed through the Embankment Fill at the location of the cross section analyzed, encountered very loose to loose sands. Typically, when such loose soils are present within an embankment, a relatively low FOS is calculated when performing slope stability analyses. The calculated FOS at the cross section analyzed is higher than we might typically expect because the maximum inclination of the downstream slope in this area is between about 3½:1 and 3:1, which is a relatively modest inclination compared to many embankment slopes that may range in inclination from 2½:1 to 2:1.
3. Even though the existing FOS for the embankment at the location of the cross section is only slightly less than the recommended minimum in the USACE reference noted above, modifications to the embankment in the area of the seepage are required to address the risk of piping due to the presence of seepage emanating from the slope face. Slope stability models do not account for the risk of piping failures.
4. When a drained buttress is added to the slope stability model, the minimum FOS against slope failure is 1.7, which is greater than the recommended minimum in the USACE reference noted above. With the addition of the buttress, the FOS for the critical failure surface based on existing conditions increases from 1.4 to 2.8. This is because the weight and location of the buttress increase the resisting forces for the passive slices analyzed in the slope stability model.
5. The addition of the drained buttress accomplishes two things: the risk of piping is mitigated, and the minimum FOS against slope failure is raised to above the recommended minimum outlined in the USACE reference noted above.

4.4 DESIGN AND CONSTRUCTION CONSIDERATIONS

The recommended geometry of the drained buttress is shown on Figure No. C2 in Appendix C. The geometry shown is based on extending the riprap of the buttress up 2 feet (vertical) above the highest location where seepage is present on the slope. We estimate the seepage is present as high as about elevation 898 feet on the slope face. Therefore, the buttress must extend up to at least elevation 900 feet. The location and elevation of the seepage should be verified at the time of construction to verify that elevation 900 feet is suitable for the top of the buttress. The level bench that forms the top of the buttress must extend away from the slope face until it reaches the point above where seepage is first present.

The surface of the buttress can then slope down toward the driveway. At the hinge point at the toe of the slope, we recommend the vertical height of the buttress be a minimum of 2.5 feet as shown on Figure No. C2. The buttress can then be sloped down at a maximum inclination of 2:1 to match existing grades on the south side of the existing ditch. We anticipate the toe of the buttress will be located about 8 feet back from (south of) the edge of the existing ditch. We recommend the east and west edges of the buttress extend a minimum of 5 feet beyond the area where seepage is currently present, and then the riprap of the buttress extend down on a maximum 2:1 slope to match existing embankment grades.

We recommend the riprap of the buttress meet the gradational and material requirements of MDOT Plain Riprap. We recommend that the riprap be composed of suitable limestone and that the use of crushed concrete or natural stone not be allowed for the riprap. Durability test data should be reviewed for the limestone proposed for use to verify suitable quality for the stone.

The riprap of the buttress should be placed on a suitable non-woven geotextile fabric that is anchored into a trench at the top of the buttress. The fabric should be unrolled down the slope (perpendicular to the slope contours) from the anchor trench, with one piece of fabric extending from the top to the bottom of the buttress. The fabric should be suitably overlapped as the fabric is placed east-west along the buttress. Refer to MDOT specifications for guidelines for suitable geotextile fabric based on the use of Plain Riprap. Care should be taken when placing the riprap so as to not tear or damage the fabric. Prior to placement of the geotextile fabric, we recommend surface vegetation below the area of the buttress be removed. We anticipate such removal will need to be performed from the bottom of the slope with a long-reach excavator since we do not recommend heavy equipment traverse the slope face due to soft and wet conditions present. After stripping of surficial vegetation, we recommend a minimum 4-inch-thick layer of MDOT Class II Granular Material be placed on the slope face in the area of the buttress to form a cushion on which to place the non-woven geotextile.

Seepage that emanates from the embankment is expected to flow down the slope face (below the riprap) to the bottom of the buttress. We recommend the seepage not be allowed to simply emanate from the downstream toe of the buttress and run overland into the existing ditch since icy conditions and ice build-up may be experienced once the seepage is exposed at the toe during cold periods of the year. Instead, we recommend a below-grade collector drain be installed at the toe of the buttress to collect the seepage water before it emanates from below the buttress. This collector drain should extend along the entire length of the buttress (east-west) and discharge into the existing collector ditch.

During our evaluation, we observed the water level in the impoundment to be relatively close to the crest elevation of the embankment. Specifically, our review of the existing topographic information indicates the crest of the existing embankment in the area of our borings (B3 and B4) to vary between about 906.5 feet and 907.5 feet, while the water surface elevation of the impoundment during “normal” conditions was surveyed as being at 905.0 feet. This means there is only 1.5 feet of freeboard along portions of the embankment during “normal” conditions. We recommend a uniform elevation be established for the crest of the dam embankment. We also recommend determination of a suitable amount of freeboard be considered as part of the evaluation of the dam. Freeboard could be increased by raising the existing crest of the embankment and/or by lowering the normal impoundment level.

Our scope did not include review of the existing spillway capacity or review of storage volume of the reservoir to evaluate whether the dam spillway system can safely pass various design storm events while maintaining a suitable amount of freeboard. We understand Spicer will be performing analyses associated with review of spillway capacity and storage volume. Please contact us if we can be of assistance with these analyses.

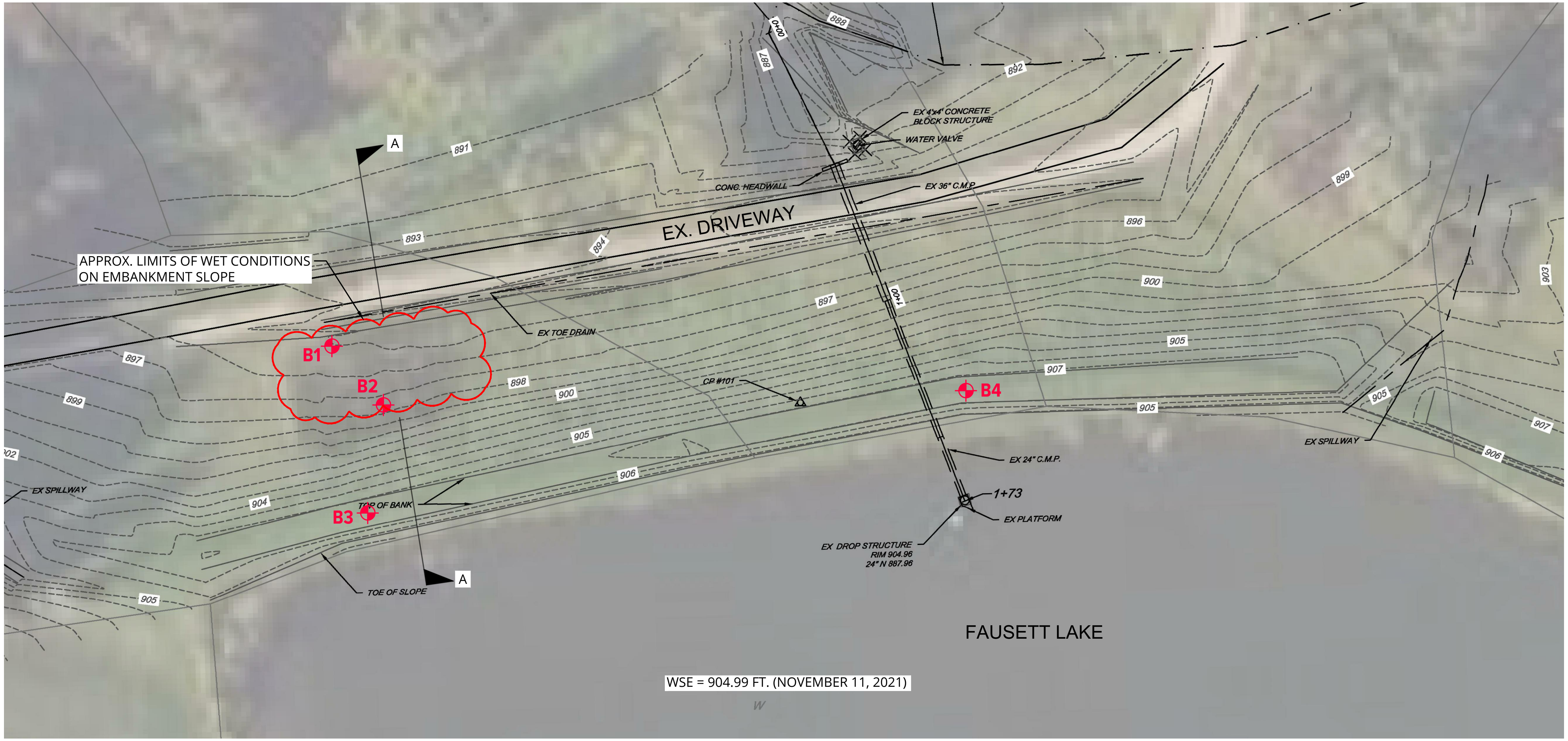
We recommend the repair plans to be developed by Spicer be provided to SME for our review and comment. The purpose of this review will be to verify the recommendations provided by SME are correctly interpreted and to verify the details on the plans by Spicer are consistent with the recommendations of this report.

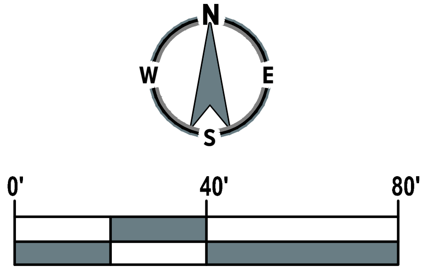
5. SIGNATURES

Report Prepared By:	Report Reviewed By:
Jeffery M. Krusinga, PE, GE Principal Consultant	Timothy H. Bedenis, PE, D.GE Chief Geotechnical Engineer

APPENDIX A

FIGURE NO. 1: BORING LOCATION DIAGRAM
BORING LOG TERMINOLOGY
BORING LOGS (B1 THROUGH B4)
PARTICLE SIZE DISTRIBUTION REPORTS (FIGURE NOS. A1, A2, AND A3)



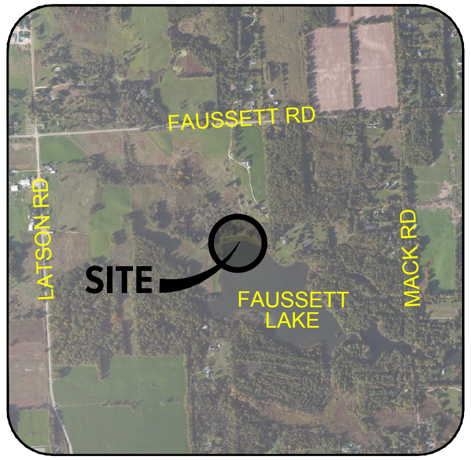


GRAPHIC SCALE: 1" = 40'

NOTE:
1. BASE MAP FOR DIAGRAM DEVELOPED FROM AN "EXISTING CONDITIONS" PLAN (SHEET DR 04, DATED JANUARY 2022) PREPARED BY SPICER GROUP.
2. CROSS SECTION A-A SHOWN ON THIS DIAGRAM INDICATES THE CROSS SECTION LOCATION UTILIZED IN SME'S SLOPE STABILITY MODEL.

LEGEND

⌖	APPROXIMATE BORING LOCATION



LOCATION MAP
NOT TO SCALE

\\Sme-inc\pz\WIP\088000.00\CAD\Design Files\BLD\Rev0\088000.00-BLD.dwg

PLOT DATE: Mar 31, 2022 - 2:30pm - cindy.rocha-ceron

SME
www.sme-usa.com

Project
FAUSSETT LAKE DAM

Project Location
DEERFIELD TOWNSHIP, LIVINGSTON COUNTY, MICHIGAN

Sheet Name
BORING LOCATION DIAGRAM

No.	Revision Date

Date: 03/25/2022
CADD: CRC
Designer: JMK
Scale: AS NOTED
Project: 088000.00
Figure No.: 1

DRAWING NOTE: SCALE DEPICTED IS MEANT FOR 11" X 17" AND WILL SCALE INCORRECTLY IF PRINTED ON ANY OTHER SIZE MEDIA

BORING LOG TERMINOLOGY

UNIFIED SOIL CLASSIFICATION AND SYMBOL CHART

COARSE-GRAINED SOIL (more than 50% of material is larger than No. 200 sieve size.)			
GRAVEL More than 50% of coarse fraction larger than No. 4 sieve size	Clean Gravel (Less than 5% fines)		
	▨	GW	Well-graded gravel; gravel-sand mixtures, little or no fines
	▨	GP	Poorly-graded gravel; gravel-sand mixtures, little or no fines
	Gravel with fines (More than 12% fines)		
	▨	GM	Silty gravel; gravel-sand-silt mixtures
	▨	GC	Clayey gravel; gravel-sand-clay mixtures
SAND 50% or more of coarse fraction smaller than No. 4 sieve size	Clean Sand (Less than 5% fines)		
	▨	SW	Well-graded sand; sand-gravel mixtures, little or no fines
	▨	SP	Poorly graded sand; sand-gravel mixtures, little or no fines
	Sand with fines (More than 12% fines)		
	▨	SM	Silty sand; sand-silt-gravel mixtures
	▨	SC	Clayey sand; sand–clay-gravel mixtures
FINE-GRAINED SOIL (50% or more of material is smaller than No. 200 sieve size)			
SILT AND CLAY Liquid limit less than 50%	▨	ML	Inorganic silt; sandy silt or gravelly silt with slight plasticity
	▨	CL	Inorganic clay of low plasticity; lean clay, sandy clay, gravelly clay
	🟩	OL	Organic silt and organic clay of low plasticity
SILT AND CLAY Liquid limit 50% or greater	▨	MH	Inorganic silt of high plasticity, elastic silt
	▨	CH	Inorganic clay of high plasticity, fat clay
	🟩	OH	Organic silt and organic clay of high plasticity
HIGHLY ORGANIC SOIL	🟩	PT	Peat and other highly organic soil

OTHER MATERIAL SYMBOLS

■ Topsoil	□ Void	▨ Sandstone
▨ Asphalt Concrete	▨ Glacial Till	▨ Siltstone
▨ Aggregate Base	■ Coal	▨ Limestone
▨ Portland Cement Concrete	▨ Shale	▨ Fill

LABORATORY CLASSIFICATION CRITERIA

GW	$C_U = \frac{D_{60}}{D_{10}}$ greater than 4; $C_C = \frac{D_{30}^{2}}{D_{10} \times D_{60}}$ between 1 and 3	
GP	Not meeting all gradation requirements for GW	
GM	Atterberg limits below "A" line or PI less than 4	Above "A" line with PI between 4 and 7 are borderline cases requiring use of dual symbols
GC	Atterberg limits above "A" line with PI greater than 7	
SW	$C_U = \frac{D_{60}}{D_{10}}$ greater than 6; $C_C = \frac{D_{30}^{2}}{D_{10} \times D_{60}}$ between 1 and 3	
SP	Not meeting all gradation requirements for SW	
SM	Atterberg limits below "A" line or PI less than 4	Above "A" line with PI between 4 and 7 are borderline cases requiring use of dual symbols
SC	Atterberg limits above "A" line with PI greater than 7	

Determine percentages of sand and gravel from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size), coarse-grained soils are classified as follows:

Less than 5 percent……………………….……..GW, GP, SW, SP
More than 12 percent………………….….…….GM, GC, SM, SC
5 to 12 percent……………...……..Cases requiring dual symbols

- SP-SM or SW-SM (SAND with Silt or SAND with Silt and Gravel)
- SP-SC or SW-SC (SAND with Clay or SAND with Clay and Gravel)
- GP-GM or GW-GM (GRAVEL with Silt or GRAVEL with Silt and Sand)
- GP-GC or GW-GC (GRAVEL with Clay or GRAVEL with Clay and Sand)

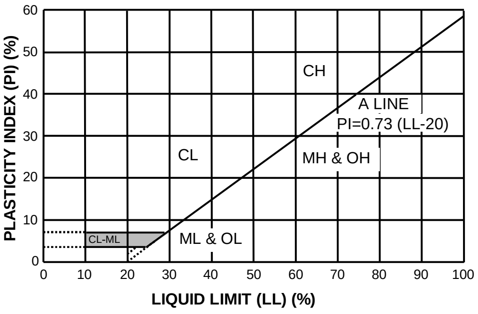
If the fines are CL-ML:

- SC-SM (SILTY CLAYEY SAND or SILTY CLAYEY SAND with Gravel)
- SM-SC (CLAYEY SILTY SAND or CLAYEY SILTY SAND with Gravel)
- GC-GM (SILTY CLAYEY GRAVEL or SILTY CLAYEY GRAVEL with Sand)

PARTICLE SIZES

Boulders		-	Greater than 12 inches
Cobbles		-	3 inches to 12 inches
Gravel-	Coarse	-	3/4 inches to 3 inches
	Fine	-	No. 4 to 3/4 inches
Sand-	Coarse	-	No. 10 to No. 4
	Medium	-	No. 40 to No. 10
	Fine	-	No. 200 to No. 40
Silt and Clay		-	Less than (0.074 mm)

PLASTICITY CHART



VISUAL MANUAL PROCEDURE

When laboratory tests are not performed to confirm the classification of soils exhibiting borderline classifications, the two possible classifications would be separated with a slash, as follows:

For soils where it is difficult to distinguish if it is a coarse or fine-grained soil:

- SC/CL (CLAYEY SAND to Sandy LEAN CLAY)
- SM/ML (SILTY SAND to SANDY SILT)
- GC/CL (CLAYEY GRAVEL to Gravelly LEAN CLAY)
- GM/ML (SILTY GRAVEL to Gravelly SILT)

For soils where it is difficult to distinguish if it is sand or gravel, poorly or well-graded sand or gravel; silt or clay; or plastic or non-plastic silt or clay:

- SP/GP or SW/GW (SAND with Gravel to GRAVEL with Sand)
- SC/GC (CLAYEY SAND with Gravel to CLAYEY GRAVEL with Sand)
- SM/GM (SILTY SAND with Gravel to SILTY GRAVEL with Sand)
- SW/SP (SAND or SAND with Gravel)
- GP/GW (GRAVEL or GRAVEL with Sand)
- SC/SM (CLAYEY to SILTY SAND)
- GM/GC (SILTY to CLAYEY GRAVEL)
- CL/ML (SILTY CLAY)
- ML/CL (CLAYEY SILT)
- CH/MH (FAT CLAY to ELASTIC SILT)
- CL/CH (LEAN to FAT CLAY)
- MH/ML (ELASTIC SILT to SILT)

DRILLING AND SAMPLING ABBREVIATIONS

2ST	–	Shelby Tube – 2" O.D.
3ST	–	Shelby Tube – 3" O.D.
AS	–	Auger Sample
GS	–	Grab Sample
LS	–	Liner Sample
NR	–	No Recovery
PM	–	Pressuremeter
RC	–	Rock Core diamond bit. NX size, except where noted
SB	–	Split Barrel Sample 1-3/8" I.D., 2" O.D., except where noted
VS	–	Vane Shear
WS	–	Wash Sample

OTHER ABBREVIATIONS

WOH	–	Weight of Hammer
WOR	–	Weight of Rods
SP	–	Soil Probe
PID	–	Photo Ionization Device
FID	–	Flame Ionization Device

DEPOSITIONAL FEATURES

Parting	–	as much as 1/16 inch thick
Seam	–	1/16 inch to 1/2 inch thick
Layer	–	1/2 inch to 12 inches thick
Stratum	–	greater than 12 inches thick
Pocket	–	deposit of limited lateral extent
Lens	–	lenticular deposit
Hardpan/Till	–	an unstratified, consolidated or cemented mixture of clay, silt, sand and/or gravel, the size/shape of the constituents vary widely
Lacustrine	–	soil deposited by lake water
Mottled	–	soil irregularly marked with spots of different colors that vary in number and size
Varved	–	alternating partings or seams of silt and/or clay
Occasional	–	one or less per foot of thickness
Frequent	–	more than one per foot of thickness
Interbedded	–	strata of soil or beds of rock lying between or alternating with other strata of a different nature

DESCRIPTION OF RELATIVE QUANTITIES

The visual-manual procedure uses the following terms to describe the relative quantities of notable foreign materials, gravel, sand or fines:

Trace	–	particles are present but estimated to be less than 5%
Few	–	5 to 10%
Little	–	15 to 25%
Some	–	30 to 45%
Mostly	–	50 to 100%

CLASSIFICATION TERMINOLOGY AND CORRELATIONS

Cohesionless Soils

Relative Density	N_{60} (N-Value) (Blows per foot)
Very Loose	0 to 4
Loose	5 to 10
Medium Dense	11 to 30
Dense	31 to 50
Very Dense	51 to 80
Extremely Dense	Over 81

Cohesive Soils

Consistency	N_{60} (N-Value) (Blows per foot)	Undrained Shear Strength (kips/ft^2)
Very Soft	<2	0.25 or less
Soft	2 - 4	> 0.25 to 0.50
Medium	5 - 8	> 0.50 to 1.0
Stiff	9 - 15	> 1.0 to 2.0
Very Stiff	16 - 30	> 2.0 to 4.0
Hard	> 30	> 4.0 or greater

Standard Penetration 'N-Value' = Blows per foot of a 140-pound hammer falling 30 inches on a 2-inch O.D. split barrel sampler, except where noted. N60 values as reported on boring logs represent raw N-values corrected for hammer efficiency only.

Revised 10/06/20

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BORING B1

PAGE 1 OF 1

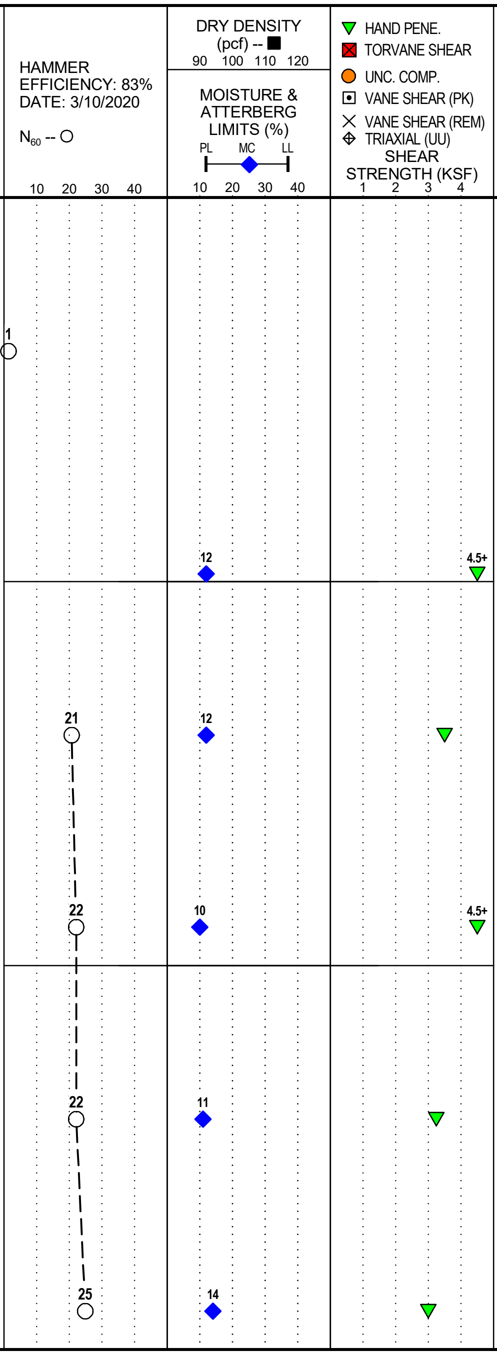
BORING DEPTH: 15 FEET

PROJECT NAME: Faussett Lake Dam
PROJECT NUMBER: 088000.00
CLIENT: Spicer Group
PROJECT LOCATION: Deerfield Township, Livingston County, Michigan

DATE STARTED: 12/10/21
COMPLETED: 12/10/21
BORING METHOD: Hollow-stem Augers
DRILLER: RM
RIG NO.: 531-CME55-ATV
LOGGED BY: EFG
CHECKED BY: JMK

NORTHING: 437545 FT
EASTING: 13256884 FT
ELEVATION: 895± FT

ELEVATION (FEET)	DEPTH (FEET)	SYMBOLIC PROFILE	PROFILE DESCRIPTION	SAMPLE TYPE/NO. INTERVAL	RECOVERY LENGTH (INCHES)	SPT BLOWS PER SIX INCHES	REMARKS
895 ▽	0		**EMBANKMENT FILL** CLAYEY SAND- Frequent Fine Silty Sand Layers- Occasional Topsoil Seams- Dark Gray and Gray- Wet- Very Loose (SC)	SB1	18	0 0 1	
892.5	2.5		LEAN CLAY with Sand- Brown and Gray- Hard to Very Stiff (CL)	3ST2	24		
890	5						
				SB3	18	6 7 8	
887.0	8.0		LEAN CLAY with Sand- Gray- Hard to Very Stiff (CL)	SB4	18	5 7 9	
885	10						
				SB5	18	6 7 9	
				SB6	18	5 8 10	
880.0	15.0		END OF BORING AT 15.0 FEET.				



GROUNDWATER & BACKFILL INFORMATION

	DEPTH (FT)	ELEV (FT)
▽ **DURING BORING:**	0.2	894.8
▼ **AT END OF BORING:**	Note 3	

BACKFILL METHOD: Note 4

NOTES:
1. The indicated stratification lines are approximate. The in-situ transitions between materials may be gradual.
2. The colors depicted on the symbolic profile are solely for visualization purposes and do not necessarily represent the in-situ colors encountered.
3. The augers were pulled after completion of drilling, and groundwater was not observed in the open borehole.
4. The open borehole was backfilled by the tremie method with bentonite-cement grout from the bottom of the borehole to 3 feet below the ground surface and with bentonite chips from 3 feet to the ground surface.

BORING B2

PAGE 1 OF 1

BORING DEPTH: 8 FEET

PROJECT NAME: Faussett Lake Dam

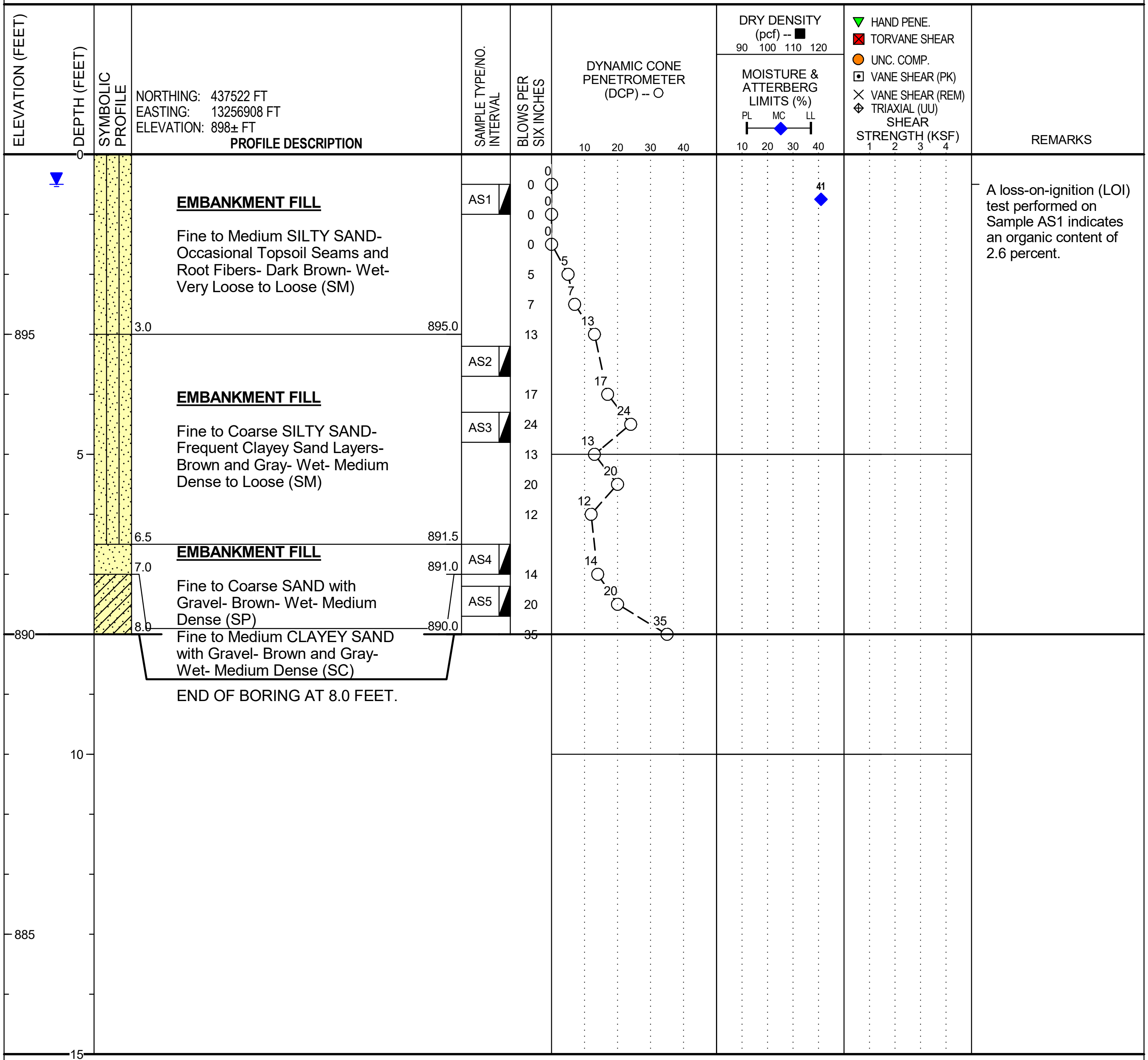
PROJECT NUMBER: 088000.00

CLIENT: Spicer Group

PROJECT LOCATION: Deerfield Township, Livingston County, Michigan

DATE STARTED: 12/7/21 **COMPLETED:** 12/7/21 **BORING METHOD:** Hand Auger

FIELD REPRESENTATIVE: MWB **EQUIPMENT:** Hand Auger **LOGGED BY:** MWB **CHECKED BY:** JMK



DEPTH (FT)	ELEV (FT)	PROFILE DESCRIPTION
0.0–3.0	895.0	**EMBANKMENT FILL** Fine to Medium SILTY SAND- Occasional Topsoil Seams and Root Fibers- Dark Brown- Wet- Very Loose to Loose (SM)
3.0–6.5	891.5	**EMBANKMENT FILL** Fine to Coarse SILTY SAND- Frequent Clayey Sand Layers- Brown and Gray- Wet- Medium Dense to Loose (SM)
6.5–7.0	891.0	**EMBANKMENT FILL** Fine to Coarse SAND with Gravel- Brown- Wet- Medium Dense (SP)
7.0–8.0	890.0	Fine to Medium CLAYEY SAND with Gravel- Brown and Gray- Wet- Medium Dense (SC)

END OF BORING AT 8.0 FEET.

REMARKS: A loss-on-ignition (LOI) test performed on Sample AS1 indicates an organic content of 2.6 percent.

GROUNDWATER & BACKFILL INFORMATION

	DEPTH (FT)	ELEV (FT)
▼ **AT END OF BORING:**	0.5	897.5
CAVE-IN OF BOREHOLE AT:	4.0	894.0

BACKFILL METHOD: Bentonite Chips

NOTES: 1. The indicated stratification lines are approximate. The in-situ transitions between materials may be gradual.
2. The colors depicted on the symbolic profile are solely for visualization purposes and do not necessarily represent the in-situ colors encountered.

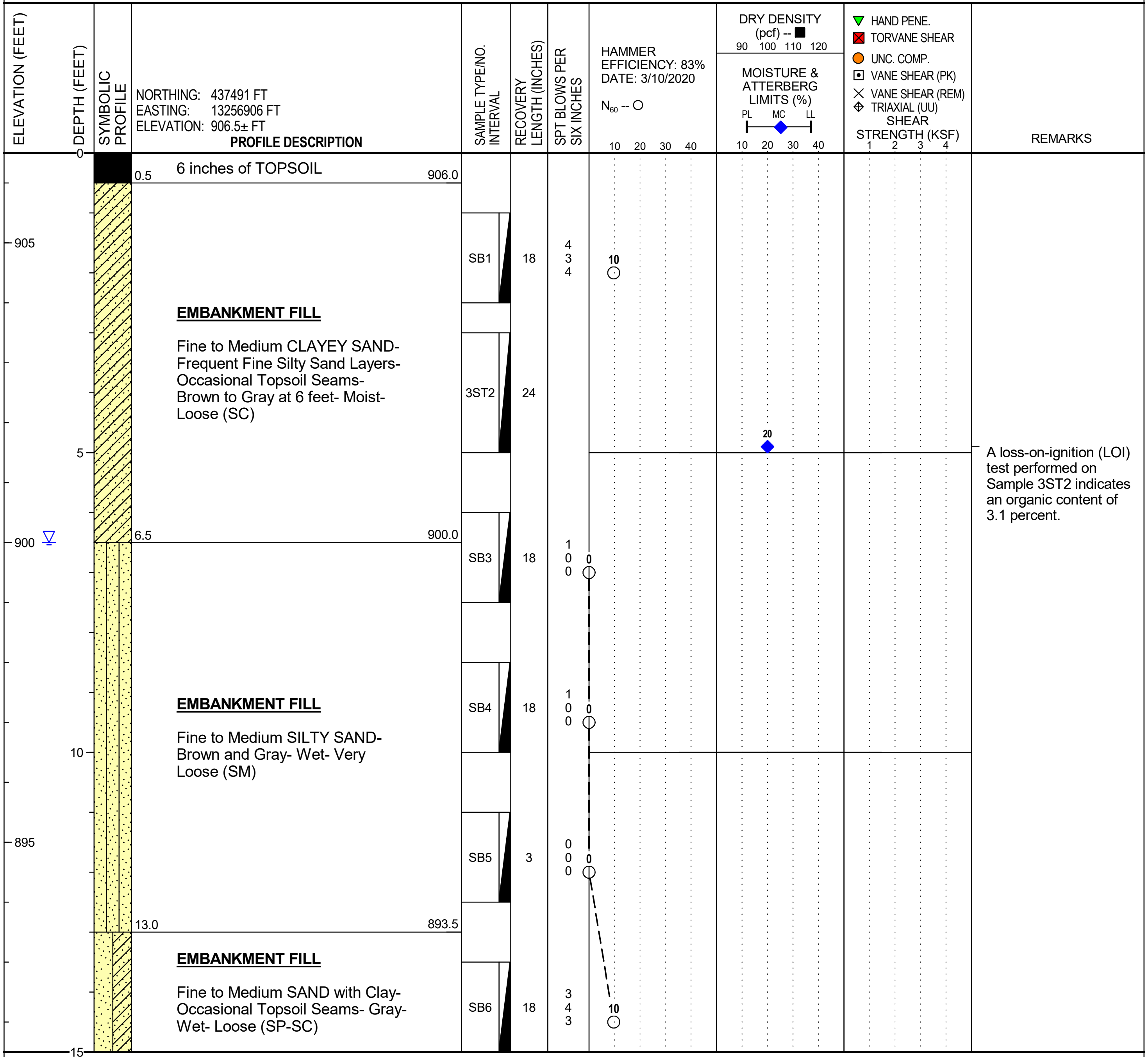
SME

BORING B3

PAGE 1 OF 2

BORING DEPTH: 25 FEET

PROJECT NAME: Faussett Lake Dam
PROJECT NUMBER: 088000.00
CLIENT: Spicer Group
PROJECT LOCATION: Deerfield Township, Livingston County, Michigan
DATE STARTED: 12/10/21
COMPLETED: 12/10/21
BORING METHOD: Hollow-stem Augers
DRILLER: RM
RIG NO.: 531-CME55-ATV
LOGGED BY: EFG
CHECKED BY: JMK



NORTHING: 437491 FT
EASTING: 13256906 FT
ELEVATION: 906.5± FT

PROFILE DESCRIPTION

Depth (ft)	Description	Elev. (ft)
0.5	6 inches of TOPSOIL	906.0
6.5	**EMBANKMENT FILL** Fine to Medium CLAYEY SAND- Frequent Fine Silty Sand Layers- Occasional Topsoil Seams- Brown to Gray at 6 feet- Moist- Loose (SC)	900.0
13.0	**EMBANKMENT FILL** Fine to Medium SILTY SAND- Brown and Gray- Wet- Very Loose (SM)	893.5
	EMBANKMENT FILL Fine to Medium SAND with Clay- Occasional Topsoil Seams- Gray- Wet- Loose (SP-SC)	

SAMPLE TYPE/NO.	RECOVERY LENGTH (INCHES)	SPT BLOWS PER SIX INCHES	N_{60}	MC (%)
SB1	18	4 3 4	10	
3ST2	24			20
SB3	18	1 0 0	0	
SB4	18	1 0 0	0	
SB5	3	0 0 0	0	
SB6	18	3 4 3	10	

REMARKS: A loss-on-ignition (LOI) test performed on Sample 3ST2 indicates an organic content of 3.1 percent.

GROUNDWATER & BACKFILL INFORMATION

	DEPTH (FT)	ELEV (FT)
▽ **DURING BORING:**	6.5	900.0
▼ **AT END OF BORING:**	21.0	885.5

BACKFILL METHOD: Note 4

NOTES:
1. The indicated stratification lines are approximate. The in-situ transitions between materials may be gradual.
2. The colors depicted on the symbolic profile are solely for visualization purposes and do not necessarily represent the in-situ colors encountered.
3. Groundwater observation at completion recorded inside the hollow-stem augers before grouting the borehole.
4. The borehole was backfilled by the tremie method with bentonite-cement grout from the bottom of the borehole to 5 feet below the ground surface and with bentonite chips from 5 feet to the ground surface.

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BORING B3

PAGE 2 OF 2

BORING DEPTH: 25 FEET

PROJECT NAME: Faussett Lake Dam

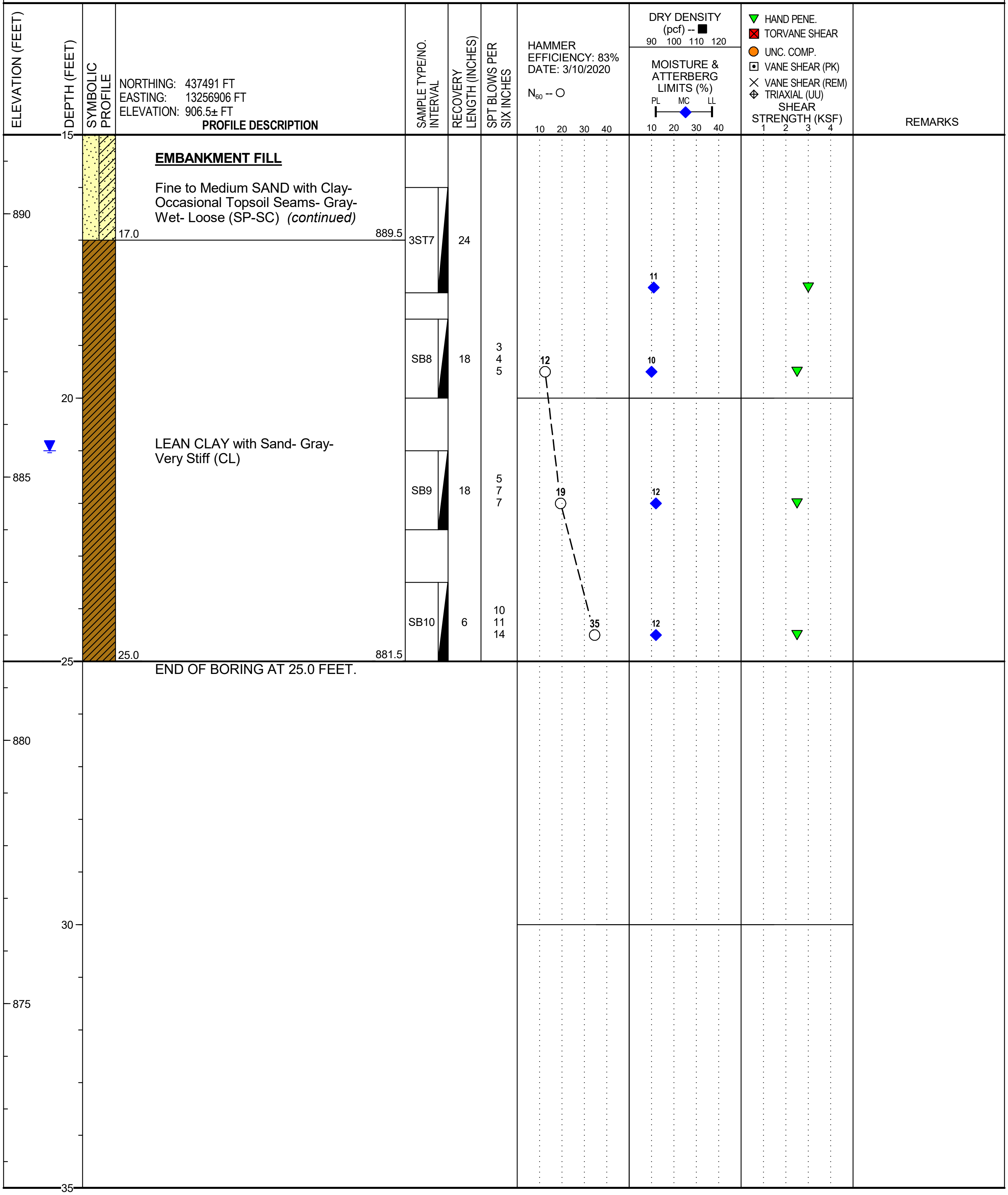
PROJECT NUMBER: 088000.00

CLIENT: Spicer Group

PROJECT LOCATION: Deerfield Township, Livingston County, Michigan

NORTHING: 437491 FT
EASTING: 13256906 FT
ELEVATION: 906.5± FT

HAMMER EFFICIENCY: 83%
DATE: 3/10/2020



EMBANKMENT FILL

Fine to Medium SAND with Clay- Occasional Topsoil Seams- Gray- Wet- Loose (SP-SC) *(continued)*

17.0 — 889.5

LEAN CLAY with Sand- Gray- Very Stiff (CL)

25.0 — 881.5

END OF BORING AT 25.0 FEET.

SAMPLE TYPE/NO. INTERVAL	RECOVERY LENGTH (INCHES)	SPT BLOWS PER SIX INCHES	N_{60}	MOISTURE (%)
3ST7	24			11
SB8	18	3 4 5	12	10
SB9	18	5 7 7	19	12
SB10	6	10 11 14	35	12

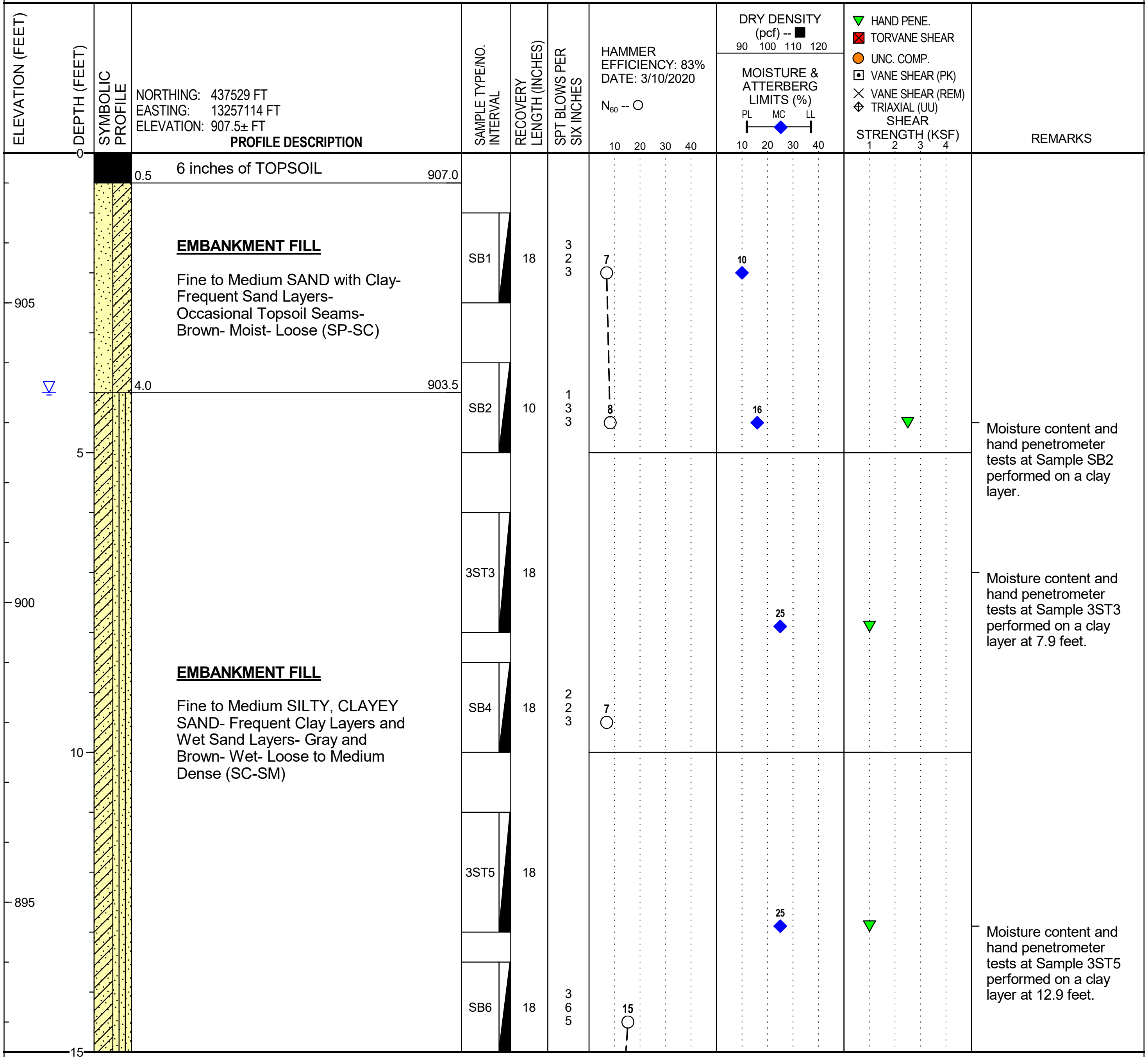
SME

BORING B4

PAGE 1 OF 2

BORING DEPTH: 30 FEET

PROJECT NAME: Faussett Lake Dam
PROJECT NUMBER: 088000.00
CLIENT: Spicer Group
PROJECT LOCATION: Deerfield Township, Livingston County, Michigan
DATE STARTED: 12/10/21
COMPLETED: 12/10/21
BORING METHOD: Hollow-stem Augers
DRILLER: RM
RIG NO.: 531-CME55-ATV
LOGGED BY: EFG
CHECKED BY: JMK



SAMPLE TYPE/NO. INTERVAL	RECOVERY LENGTH (INCHES)	SPT BLOWS PER SIX INCHES
SB1	18	3 2 3
SB2	10	1 3 3
3ST3	18	
SB4	18	2 2 3
3ST5	18	
SB6	18	3 6 5

0.5 6 inches of TOPSOIL 907.0

EMBANKMENT FILL

Fine to Medium SAND with Clay- Frequent Sand Layers- Occasional Topsoil Seams- Brown- Moist- Loose (SP-SC)

4.0 903.5

EMBANKMENT FILL

Fine to Medium SILTY, CLAYEY SAND- Frequent Clay Layers and Wet Sand Layers- Gray and Brown- Wet- Loose to Medium Dense (SC-SM)

Moisture content and hand penetrometer tests at Sample SB2 performed on a clay layer.

Moisture content and hand penetrometer tests at Sample 3ST3 performed on a clay layer at 7.9 feet.

Moisture content and hand penetrometer tests at Sample 3ST5 performed on a clay layer at 12.9 feet.

GROUNDWATER & BACKFILL INFORMATION

	DEPTH (FT)	ELEV (FT)
▽ **DURING BORING:**	4.0	903.5
▼ **AT END OF BORING:**	20.0	887.5

BACKFILL METHOD: Note 4

NOTES:
1. The indicated stratification lines are approximate. The in-situ transitions between materials may be gradual.
2. The colors depicted on the symbolic profile are solely for visualization purposes and do not necessarily represent the in-situ colors encountered.
3. Groundwater observation at completion recorded inside the hollow-stem augers before grouting the borehole.
4. The borehole was backfilled by the tremie method with bentonite-cement grout from the bottom of the borehole to 5 feet below the ground surface and with bentonite chips from 5 feet to the ground surface.

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SME

BORING B4

PAGE 2 OF 2

BORING DEPTH: 30 FEET

PROJECT NAME: Faussett Lake Dam

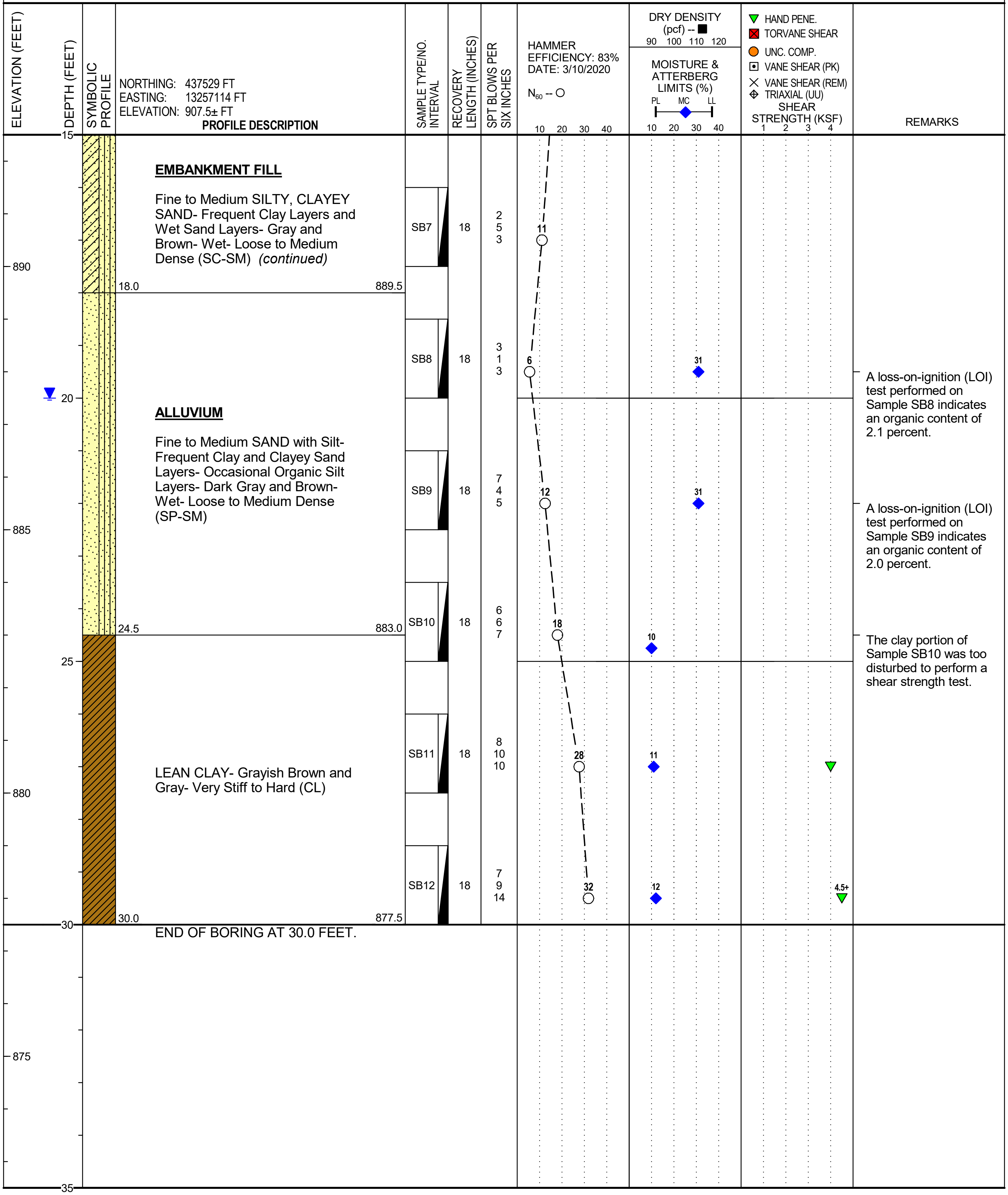
PROJECT NUMBER: 088000.00

CLIENT: Spicer Group

PROJECT LOCATION: Deerfield Township, Livingston County, Michigan

NORTHING: 437529 FT
EASTING: 13257114 FT
ELEVATION: 907.5± FT

HAMMER EFFICIENCY: 83%
DATE: 3/10/2020



EMBANKMENT FILL

Fine to Medium SILTY, CLAYEY SAND- Frequent Clay Layers and Wet Sand Layers- Gray and Brown- Wet- Loose to Medium Dense (SC-SM) *(continued)*

18.0 889.5

ALLUVIUM

Fine to Medium SAND with Silt- Frequent Clay and Clayey Sand Layers- Occasional Organic Silt Layers- Dark Gray and Brown- Wet- Loose to Medium Dense (SP-SM)

24.5 883.0

LEAN CLAY- Grayish Brown and Gray- Very Stiff to Hard (CL)

30.0 877.5

END OF BORING AT 30.0 FEET.

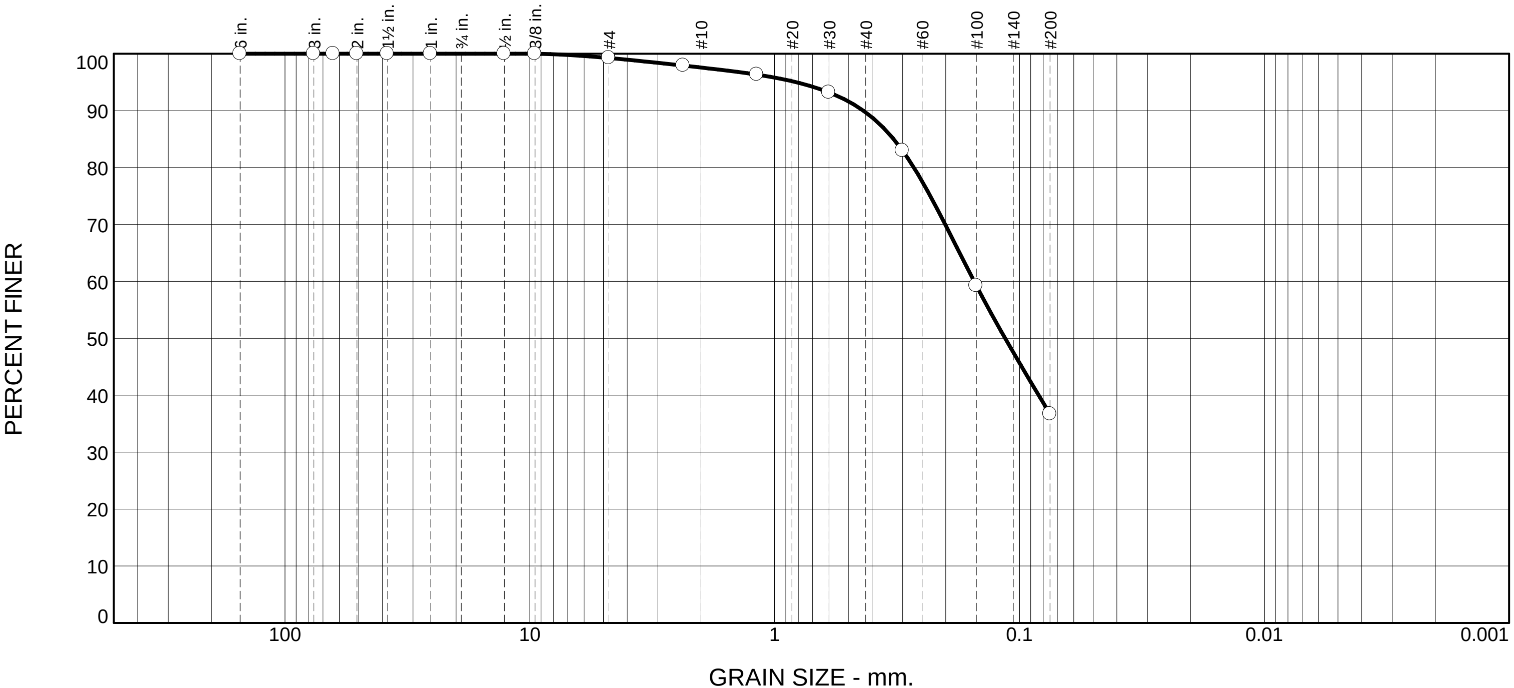
SAMPLE TYPE/NO.	RECOVERY LENGTH (INCHES)	SPT BLOWS PER SIX INCHES	N_{60}	MC (%)	SHEAR STRENGTH (KSF)
SB7	18	2 5 3	11		
SB8	18	3 1 3	6	31	
SB9	18	7 4 5	12	31	
SB10	18	6 6 7	18	10	
SB11	18	8 10 10	28	11	
SB12	18	7 9 14	32	12	4.5+

A loss-on-ignition (LOI) test performed on Sample SB8 indicates an organic content of 2.1 percent.

A loss-on-ignition (LOI) test performed on Sample SB9 indicates an organic content of 2.0 percent.

The clay portion of Sample SB10 was too disturbed to perform a shear strength test.

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.7	1.7	7.9	53.0	36.7	

Test Results (ASTM C136 & ASTM C117)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
6	100.0		
3	100.0		
2 1/2	100.0		
2	100.0		
1 1/2	100.0		
1	100.0		
1/2	100.0		
3/8	100.0		
#4	99.3		
#8	97.9		
#16	96.3		
#30	93.2		
#50	83.0		
#100	59.2		
#200	36.7		

* (no specification provided)

Material Description

Fine to Medium SILTY SAND

Atterberg Limits (ASTM D 4318)

PL= **LL=** **PI=**

Classification

USCS (D 2487)= SM **AASHTO (M 145)=**

Coefficients

D_{90}= 0.4340 D_{85}= 0.3264 D_{60}= 0.1532
D_{50}= 0.1144 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Date Received: **Date Tested:** 12/10/2021

Tested By: Errol Gilbert, CET

Checked By: Jeff Krusinga, PE, GE

Title: Chief Consultant

Location: Boring B2
Sample Number: AS1 **Depth:** 0.5 to 1 feet

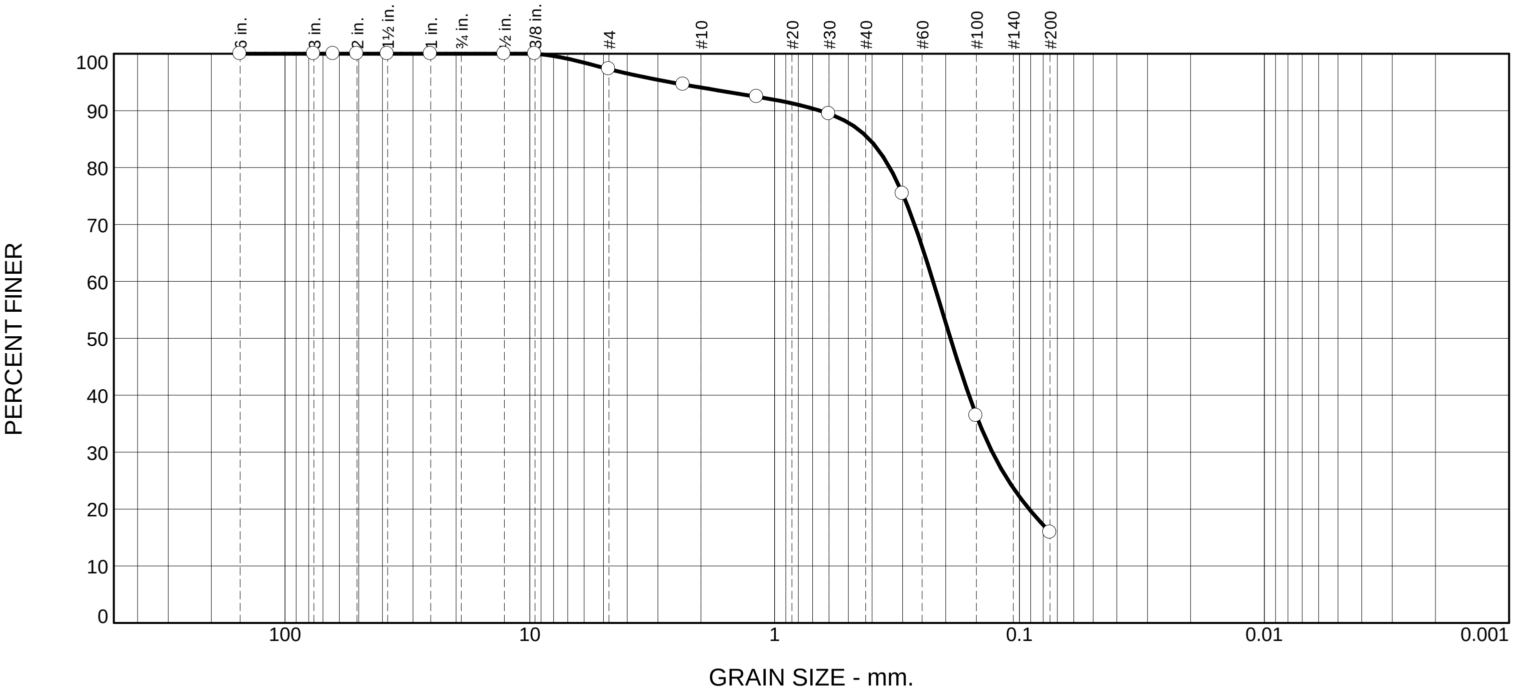
Date Sampled:

SME

Client: Spicer Group
Project: Faussett Lake Dam

Project No: 088000.00 **Figure** No. A1

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	2.7	3.2	8.5	69.7	15.9	

Test Results (ASTM C117 & ASTM C136)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
6	100.0		
3	100.0		
2 1/2	100.0		
2	100.0		
1 1/2	100.0		
1	100.0		
1/2	100.0		
3/8	100.0		
#4	97.3		
#8	94.6		
#16	92.4		
#30	89.4		
#50	75.4		
#100	36.4		
#200	15.9		

* (no specification provided)

Material Description

Fine to Medium SILTY SAND

Atterberg Limits (ASTM D 4318)

PL= **LL=** **PI=**

Classification

USCS (D 2487)= SM **AASHTO (M 145)=**

Coefficients

D_{90}=	0.6544	D_{85}=	0.4108	D_{60}=	0.2253
D_{50}=	0.1913	D_{30}=	0.1291	D_{15}=	
D_{10}=		C_u=		C_c=	

Remarks

Date Received: **Date Tested:** 1/26/2022

Tested By: Errol Gilbert, CET

Checked By: Jeff Krusinga, PE, GE

Title: Chief Consultant

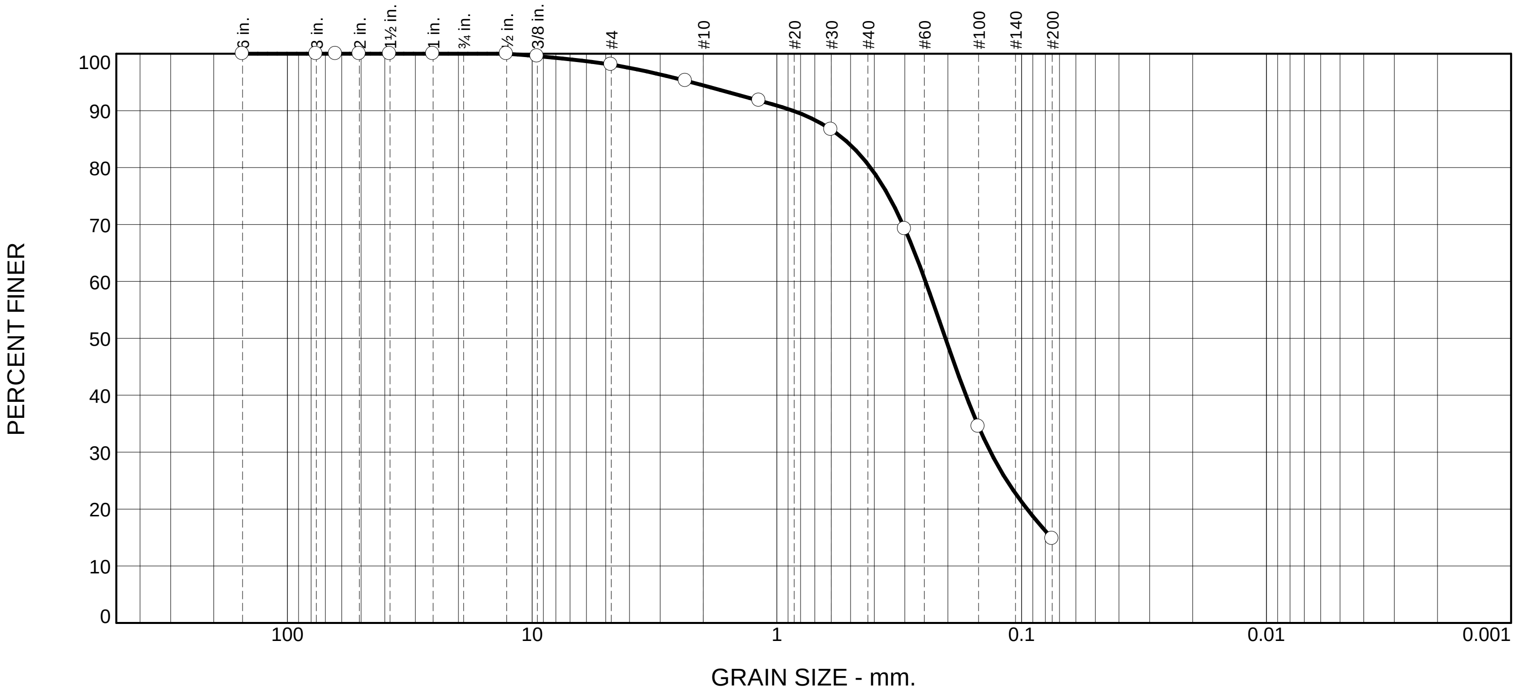
Location: Boring B3
Sample Number: SB4 **Depth:** 8.5 to 10 feet **Date Sampled:**

SME

Client: Spicer Group
Project: Faussett Lake Dam

Project No: 088000.00 **Figure** No. A2

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	1.9	3.7	13.8	65.8	14.8	

Test Results (ASTM C136 & ASTM C117)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
6	100.0		
3	100.0		
2 1/2	100.0		
2	100.0		
1 1/2	100.0		
1	100.0		
1/2	100.0		
3/8	99.6		
#4	98.1		
#8	95.3		
#16	91.8		
#30	86.7		
#50	69.2		
#100	34.5		
#200	14.8		

* (no specification provided)

Material Description

Fine to Medium, SILTY, CLAYEY SAND

Atterberg Limits (ASTM D 4318)

PL= **LL=** **PI=**

Classification

USCS (D 2487)= SC-SM **AASHTO (M 145)=**

Coefficients

D_{90}=	0.8604	D_{85}=	0.5323	D_{60}=	0.2471
D_{50}=	0.2048	D_{30}=	0.1339	D_{15}=	0.0757
D_{10}=		C_u=		C_c=	

Remarks

Date Received: **Date Tested:** 1/26/2022

Tested By: Errol Gilbert, CET

Checked By: Jeff Krusinga, PE, GE

Title: Chief Consultant

Location: Boring B4

Sample Number: SB4 **Depth:** 8.5 to 10 feet

Date Sampled:

SME

Client: Spicer Group

Project: Faussett Lake Dam

Project No: 088000.00 **Figure** No. A3

APPENDIX B

GENERAL COMMENTS

LABORATORY TESTING PROCEDURES

IMPORTANT INFORMATION ABOUT THIS GEOTECHNICAL-ENGINEERING REPORT

GENERAL COMMENTS

BASIS OF GEOTECHNICAL REPORT

This report has been prepared in accordance with generally accepted geotechnical engineering practices to assist in the design and/or evaluation of this project. If the project plans, design criteria, and other project information referenced in this report and utilized by SME to prepare our recommendations are changed, the conclusions and recommendations contained in this report are not considered valid unless the changes are reviewed, and the conclusions and recommendations of this report are modified or approved in writing by our office.

The discussions and recommendations submitted in this report are based on the available project information, described in this report, and the geotechnical data obtained from the field exploration at the locations indicated in the report. Variations in the soil and groundwater conditions commonly occur between or away from sampling locations. The nature and extent of the variations may not become evident until the time of construction. If significant variations are observed during construction, SME should be contacted to reevaluate the recommendations of this report. SME should be retained to continue our services through construction to observe and evaluate the actual subsurface conditions relative to the recommendations made in this report.

In the process of obtaining and testing samples and preparing this report, procedures are followed that represent reasonable and accepted practice in the field of soil and foundation engineering. Specifically, field logs are prepared during the field exploration that describe field occurrences, sampling locations, and other information. Samples obtained in the field are frequently subjected to additional testing and reclassification in the laboratory and differences may exist between the field logs and the report logs. The engineer preparing the report reviews the field logs, laboratory classifications, and test data and then prepares the report logs. Our recommendations are based on the contents of the report logs and the information contained therein.

REVIEW OF DESIGN DETAILS, PLANS, AND SPECIFICATIONS

SME should be retained to review the design details, project plans, and specifications to verify those documents are consistent with the recommendations contained in this report.

REVIEW OF REPORT INFORMATION WITH PROJECT TEAM

Implementation of our recommendations may affect the design, construction, and performance of the proposed improvements, along with the potential inherent risks involved with the proposed construction. The client and key members of the design team, including SME, should discuss the issues covered in this report so that the issues are understood and applied in a manner consistent with the owner's budget, tolerance of risk, and expectations for performance and maintenance.

FIELD VERIFICATION OF GEOTECHNICAL CONDITIONS

SME should be retained to verify the recommendations of this report are properly implemented during construction. This may avoid misinterpretation of our recommendations by other parties and will allow us to review and modify our recommendations if variations in the site subsurface conditions are encountered.

PROJECT INFORMATION FOR CONTRACTOR

This report and any future addenda or other reports regarding this site should be made available to prospective contractors prior to submitting their proposals for their information only and to supply them with facts relative to the subsurface evaluation and laboratory test results. If the selected contractor encounters subsurface conditions during construction, which differ from those presented in this report, the contractor should promptly describe the nature and extent of the differing conditions in writing and SME should be notified so that we can verify those conditions. The construction contract should include provisions for dealing with differing conditions and contingency funds should be reserved for potential problems during earthwork and foundation construction. We would be pleased to assist you in developing the contract provisions based on our experience.

The contractor should be prepared to handle environmental conditions encountered at this site, which may affect the excavation, removal, or disposal of soil; dewatering of excavations; and health and safety of workers. Any Environmental Assessment reports prepared for this site should be made available for review by bidders and the successful contractor.

THIRD PARTY RELIANCE/REUSE OF THIS REPORT

This report has been prepared solely for the use of our Client for the project specifically described in this report. This report cannot be relied upon by other parties not involved in the project, unless specifically allowed by SME in writing. SME also is not responsible for the interpretation by other parties of the geotechnical data and the recommendations provided herein.

LABORATORY TESTING PROCEDURES

VISUAL ENGINEERING CLASSIFICATION

Visual classification was performed on recovered samples. The appended General Notes and Unified Soil Classification System (USCS) sheets include a brief summary of the general method used visually classify the soil and assign an appropriate USCS group symbol. The estimated group symbol, according to the USCS, is shown in parentheses following the textural description of the various strata on the boring logs appended to this report. The soil descriptions developed from visual classifications are sometimes modified to reflect the results of laboratory testing.

MOISTURE CONTENT

Moisture content tests were performed by weighing samples from the field at their in-situ moisture condition. These samples were then dried at a constant temperature (approximately 110º C) overnight in an oven. After drying, the samples were weighed to determine the dry weight of the sample and the weight of the water that was expelled during drying. The moisture content of the specimen is expressed as a percent and is the weight of the water compared to the dry weight of the specimen.

HAND PENETROMETER TESTS

In the hand penetrometer test, the unconfined compressive strength of a cohesive soil sample is estimated by measuring the resistance of the sample to the penetration of a small calibrated, spring-loaded cylinder. The maximum capacity of the penetrometer is 4.5 tons per square-foot (tsf). Theoretically, the undrained shear strength of the cohesive sample is one-half the unconfined compressive strength. The undrained shear strength (based on the hand penetrometer test) presented on the boring logs is reported in units of kips per square-foot (ksf).

TORVANE SHEAR TESTS

In the Torvane test, the shear strength of a low strength, cohesive soil sample is estimated by measuring the resistance of the sample to a torque applied through vanes inserted into the sample. The undrained shear strength of the samples is measured from the maximum torque required to shear the sample and is reported in units of kips per square-foot (ksf).

LOSS-ON-IGNITION (ORGANIC CONTENT) TESTS

Loss-on-ignition (LOI) tests are conducted by first weighing the sample and then heating the sample to dry the moisture from the sample (in the same manner as determining the moisture content of the soil). The sample is then re-weighed to determine the dry weight and then heated for 4 hours in a muffle furnace at a high temperature (approximately 440º C). After cooling, the sample is re-weighed to calculate the amount of ash remaining, which in turn is used to determine the amount of organic matter burned from the original dry sample. The organic matter content of the specimen is expressed as a percent compared to the dry weight of the sample.

ATTERBERG LIMITS TESTS

Atterberg limits tests consist of two components. The plastic limit of a cohesive sample is determined by rolling the sample into a thread and the plastic limit is the moisture content where a 1/8-inch thread begins to crumble. The liquid limit is determined by placing a ½-inch thick soil pat into the liquid limits cup and using a grooving tool to divide the soil pat in half. The cup is then tapped on the base of the liquid limits device using a crank handle. The number of drops of the cup to close the gap formed by the grooving tool ½ inch is recorded along with the corresponding moisture content of the sample. This procedure is repeated several times at different moisture contents and a graph of moisture content and the corresponding number of blows is plotted. The liquid limit is defined as the moisture content at a nominal 25 drops of the cup. From this test, the plasticity index can be determined by subtracting the plastic limit from the liquid limit.

GRAIN SIZE DISTRIBUTION ANALYSIS

COARSE-GRAINED (GRANULAR) SAMPLES WITH LOW FINES CONTENT

Grain size distribution tests performed on granular samples involves oven-drying a representative sample of soil and washing out the fines (passing the No. 200 sieve) with tap water. The sample retained on the No. 200 sieve is then oven-dried, cooled and sieved on a series of stacked sieves beginning with the largest sieve on top and progressing to the smallest on the bottom. The portions of the sample retained on each sieve are then weighed and used to develop the grain size distribution curve in the report for each sample tested.

FINE-GRAINED (SILT OR CLAY) SAMPLES OR COARSE-GRAINED SAMPLES WITH HIGH FINES CONTENT

Particle size distribution tests performed on fine-grained or coarse-grained samples with a high fines content involves oven-drying a representative sample and mixing the sample with a liquid deflocculant to disperse the soil particles. The slurry is placed in a graduated cylinder and shaken to suspend the soil particles in the slurry. The graduated cylinder is then placed on a tabletop; a calibrated hydrometer is floated in the slurry to determine its density. The hydrometer measurements are made at selected time intervals as the soil in the cylinder settles and slurry density decreases. When the hydrometer measurements are completed, the slurry is poured onto a No. 200 sieve and the fines are washed out with tap water. The sample retained on the No. 200 sieve is then oven-dried, cooled and sieved on a series of stacked sieves beginning with the largest sieve on top and progressing to the smallest on the bottom. The portions of the sample retained on each sieve are then weighed and used with the hydrometer data to develop the grain size distribution curve in the report for each sample tested.

WET/DRY DENSITY TESTS

Wet/dry density tests involve extracting a representative soil sample from either a Shelby tube or sample liner, trimming the ends perpendicular to the length of the sample and measuring the length and diameter. The sample is then weighed, oven-dried and weighed again after drying. The wet density is equal to the wet weight of the sample (prior to drying) divided by the volume, while the dry density is the dry weight of the sample divided by the volume.

UNCONFINED COMPRESSIVE STRENGTH TESTS

In addition to the hand penetrometer and Torvane tests, unconfined compression tests were performed to better estimate the undrained shear strength of selected cohesive samples recovered from either Shelby tubes or liners taken in conjunction with the Standard Penetration Test. In the unconfined compression test, the unconfined compressive strength of a soil sample is determined by axially loading the soil sample at a slow, constant rate of strain. The unconfined compressive strength is the maximum compressive stress in the soil sample, up to 15 percent strain. Theoretically, the undrained shear strength of the cohesive sample is one-half the unconfined compressive strength. The undrained shear strength presented on the boring logs is reported in units of kips per square-foot (ksf).

CORROSION TESTS

The soil corrosion tests may include measuring the electrical resistivity, pH and concentrations of soluble chlorides and sulfates. Soil samples tested are generally taken from a composite of two or more selected soil samples with generally similar visual characteristics. The electrical resistivity of the selected soil samples was performed on natural-state and saturated samples using a Miller multi-combination meter with a soil box configured in a four-pin arrangement. pH tests are conducted in general accordance with Brighton Analytical's method reference EPA 150.1. The soil samples for the soluble sulfates and chlorides were prepared at a water-to-soil ratio of 2:1 and tested in general accordance with Brighton Analytical's method reference SW846-9056.

MOISTURE-DRY DENSITY RELATIONSHIPS (COMPACTION) TESTS

Moisture-dry density tests involve the preparation of a bulk soil sample by compacting the sample at a given energy into a calibrated mold with a known volume of 0.0333 cubic feet at various moisture contents. A graph of the moisture content vs. dry density is developed, which results in an inverted U-shaped curve. The maximum dry density is the peak of the curve and the corresponding moisture content is the optimum moisture. Two methods can be performed, namely:

STANDARD PROCTOR METHOD

This method involves a standard energy of 12,400 ft-lbs per cubic foot of soil volume to compact the sample. The sample is compacted in three layers of equal thickness using a 5.5-pound hammer dropped 12 inches using 25 blows per layer.

MODIFIED PROCTOR METHOD

This method involves a modified energy of 56,000 ft-lbs per cubic foot of soil volume to compact the sample. The sample is compacted in five layers of equal thickness using a 10-pound hammer dropped 18 inches using 25 blows per layer.

SPECIFIC GRAVITY TESTS

This test involves the determination of the ratio of the weight of a known volume of soil particles in air to weight of the same volume of water in air. The test is performed by oven drying a soil sample and placing the sample with water into a calibrated pycnometer, boiling the soil/water mixture, filling the pycnometer with distilled water to its calibration mark, weighing the pycnometer and soil/water mixture and measuring the temperature of the mixture. The specific gravity is equal to the weight of the dry soil particles multiplied by the specific gravity of distilled water at the temperature measured for the soil/water mixture divided by the sum of the weight of the dry soil particles plus the weight of the pycnometer, soil/water mixture plus the weight of the pycnometer plus water from the calibration curve developed for the pycnometer.

DIRECT SHEAR TESTS

A bulk samples is compacted in a direct shear mold at a specified density and moisture content. Shear tests are then performed using the direct shear procedure. The direct shear test is performed at several overburden pressures or normal stresses that represent approximate potential stresses in the proposed construction. Values of both peak friction angle and residual friction angle are determined from the tests for each overburden pressure. The results of the direct shear tests are tabulated and plotted on the Direct Shear Test Plots in Appendix A.

CONSOLIDATION TESTS

Consolidation tests are used to evaluate the magnitude and rate of consolidation of soil when it is restrained laterally and drained on the top and bottom while subjected to vertical load applied in controlled increments. The range of test loads applied is generally selected to represent the anticipated vertical stress conditions resulting from existing conditions and the proposed construction. Plots of the percent strain vs. log pressure are constructed from the data to assess consolidation characteristics, while the rate of consolidation is evaluated from plots of deformation vs. time for each vertical load increment.

PERMEABILITY TESTS

The permeability of either relatively undisturbed or compacted soils can be determined by various laboratory test equipment including a triaxial cell, permeameter mold or from a liner sample. The type of permeability equipment used and test performed will be based on the soil type being evaluated.

CLAY, SILT AND OTHER LOW PERMEABLE SOIL SAMPLES

For samples with relatively low permeability characteristics, an undisturbed or compacted soil sample is placed in a triaxial cell. Prior to performing the permeability test, the sample must be fully saturated by forcing water into the sample using a backpressure (water under pressure from an air supply) which is slightly less than the cell pressure. Once the sample is saturated, water is forced through the top of the sample with pressure from an air supply (which is slightly less than the cell pressure) and water forced out of the bottom of the sample is measured in a burette. The volume of water displaced from the sample is recorded with time and from that information, the coefficient of permeability is calculated. This method is a constant head permeability test.

SAND SAMPLES

Due to the nature of relatively clean granular soils, the use of a triaxial cell is generally not practical and the permeability of these types of soils is typically determined from either a liner sample (either recovered directly from a split-spoon in the field or a sample compacted in the liner) or a bulk sample compacted in a 6-inch diameter permeameter mold. A falling head permeability test can be performed on most granular samples by filling a standpipe with water and measuring the head drop with time. For highly permeable soils, the rate of drop in a falling head test may be too rapid to obtain reliable volume and time measurements. Thus, a constant head test will be required where a constant head of water is maintained, and the volume of water discharged from the sample is measured with time.

TRIAXIAL TESTS

Triaxial tests were conducted on samples trimmed from Shelby tubes or liners. There are several types of triaxial tests which can be performed and each are described below:

UNCONSOLIDATED-UNDRAINED TRIAXIAL TEST METHOD

The strength and stress-strain relationships of a cylindrical soil sample are determined for a sample subjected to a selected confining fluid pressure in a triaxial chamber. No drainage of the sample is permitted during the test and the sample is sheared in compression at a constant rate of axial deformation. The peak stress measured for the sample is recorded, up to a maximum 15 percent strain. At least three triaxial tests are performed at various confining fluid pressures to model in-situ stress conditions for loading. A plot of the Mohr circles at failure stress for each confining pressure is included in Appendix A.

CONSOLIDATED-DRAINED TRIAXIAL TEST METHOD

The strength and stress-strain relationships of a cylindrical soil sample are determined for a sample subjected to a selected confining fluid pressure in a triaxial chamber. The sample is isotropically consolidated prior to applying axial loads and sheared in compression at a slow constant rate of axial deformation while allowing the sample to drain. The peak stress measured for the sample is recorded, up to a maximum 15 percent strain. At least three triaxial tests are performed at various confining fluid pressures to model in-situ stress conditions for loading. A plot of the Mohr circles at failure stress for each confining pressure is included in Appendix A.

CONSOLIDATED-UNDRAINED TRIAXIAL TEST METHOD

The strength and stress-strain relationships of a cylindrical soil sample are determined for a sample subjected to a selected confining fluid pressure in a triaxial chamber. The sample is isotropically consolidated prior to applying axial loads and sheared undrained in compression at a constant rate of axial deformation. Pore water pressure measurements can also be measured during the shearing of the sample. The peak stress measured for the sample is recorded, up to a maximum 15 percent strain. At least three triaxial tests are performed at various confining fluid pressures to model in-situ stress conditions for loading. A plot of the Mohr circles at failure stress for each confining pressure is included in Appendix A.

DENSITY TESTS ON ROCK CORES

Density tests involve trimming the ends of an intact rock core sample perpendicular to the length of the sample and measuring the length and diameter. The sample is then weighed and the weight is divided by the volume to calculate the density.

UNCONFINED COMPRESSIVE STRENGTH TESTS ON ROCK CORES

Unconfined compression tests were performed to estimate the compressive strength of selected rock core samples. Representative rock cores were selected and cut perpendicular to the length of the sample on both ends to a specified length with a wet saw. In the unconfined compression test, the unconfined compressive strength of a rock core sample is determined by axially loading the rock core sample at a slow, constant rate of strain. The unconfined compressive strength is the maximum compressive stress in the rock core sample or the load applied when a predetermined amount of strain is achieved.

Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, you can benefit from a lowered exposure to problems associated with subsurface conditions at project sites and development of them that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed herein, contact your GBA-member geotechnical engineer. Active engagement in GBA exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Understand the Geotechnical-Engineering Services Provided for this Report

Geotechnical-engineering services typically include the planning, collection, interpretation, and analysis of exploratory data from widely spaced borings and/or test pits. Field data are combined with results from laboratory tests of soil and rock samples obtained from field exploration (if applicable), observations made during site reconnaissance, and historical information to form one or more models of the expected subsurface conditions beneath the site. Local geology and alterations of the site surface and subsurface by previous and proposed construction are also important considerations. Geotechnical engineers apply their engineering training, experience, and judgment to adapt the requirements of the prospective project to the subsurface model(s). Estimates are made of the subsurface conditions that will likely be exposed during construction as well as the expected performance of foundations and other structures being planned and/or affected by construction activities.

The culmination of these geotechnical-engineering services is typically a geotechnical-engineering report providing the data obtained, a discussion of the subsurface model(s), the engineering and geologic engineering assessments and analyses made, and the recommendations developed to satisfy the given requirements of the project. These reports may be titled investigations, explorations, studies, assessments, or evaluations. Regardless of the title used, the geotechnical-engineering report is an engineering interpretation of the subsurface conditions within the context of the project and does not represent a close examination, systematic inquiry, or thorough investigation of all site and subsurface conditions.

Geotechnical-Engineering Services are Performed for Specific Purposes, Persons, and Projects, and At Specific Times

Geotechnical engineers structure their services to meet the specific needs, goals, and risk management preferences of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client.

Likewise, geotechnical-engineering services are performed for a specific project and purpose. For example, it is unlikely that a geotechnical-engineering study for a refrigerated warehouse will be the same as one prepared for a parking garage; and a few borings drilled during a preliminary study to evaluate site feasibility will not be adequate to develop geotechnical design recommendations for the project.

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project or purpose;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, the reliability of a geotechnical-engineering report can be affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying the recommendations in it. A minor amount of additional testing or analysis after the passage of time – if any is required at all – could prevent major problems.

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read the report in its entirety. Do not rely on an executive summary. Do not read selective elements only. *Read and refer to the report in full.*

You Need to Inform Your Geotechnical Engineer About Change

Your geotechnical engineer considered unique, project-specific factors when developing the scope of study behind this report and developing the confirmation-dependent recommendations the report conveys. Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the elevation, configuration, location, orientation, function or weight of the proposed structure and the desired performance criteria;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project or site changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept* *responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface using various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing is performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgement to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team through project completion to obtain informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, they are not final, because the geotechnical engineer who developed them relied heavily on judgement and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* exposed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a continuing member of the design team, to:

- confer with other design-team members;
- help develop specifications;
- review pertinent elements of other design professionals' plans and specifications; and
- be available whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction-phase observations.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for information purposes only.* To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. This happens in part because soil and rock on project sites are typically heterogeneous and not manufactured materials with well-defined engineering properties like steel and concrete. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually provide environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures.* If you have not obtained your own environmental information about the project site, ask your geotechnical consultant for a recommendation on how to find environmental risk-management guidance.

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, the engineer's services were not designed, conducted, or intended to prevent migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration. Confront the risk of moisture infiltration*** by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

GBA GEOPROFESSIONAL BUSINESS ASSOCIATION

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e-mail: info@geoprofessional.org www.geoprofessional.org

APPENDIX C

SELECTED OUTPUT FROM SLIDE (FIGURE NOS. C1 AND C2)

FIGURE NO. C1

EXISTING CONDITIONS AT CROSS SECTION A-A,
STEADY-STATE SEEPAGE CONDITIONS

FAUSSETT LAKE DAM
DEERFIELD TOWNSHIP, LIVINGSTON COUNTY, MICHIGAN

GLOBAL STABILITY MODEL EXISTING EMBANKMENT WITH LONG-TERM DRAINED SOIL SHEAR STRENGTH PARAMETERS

Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Embankment Fill - CLAYEY SAND (SC) - Very Loose	🟨	120	0	26
Embankment Fill - CLAYEY SAND (SC) and SAND with Clay (SP-SC) - Loose	🟩	120	0	28
Embankment Fill - SILTY SAND (SM) - Very Loose	🟧	120	0	26
Natural LEAN CLAY (CL) - Very Stiff to Hard	🟪	130	200	28

Method Name	Min FS
Bishop simplified	1.500
Janbu simplified	1.393

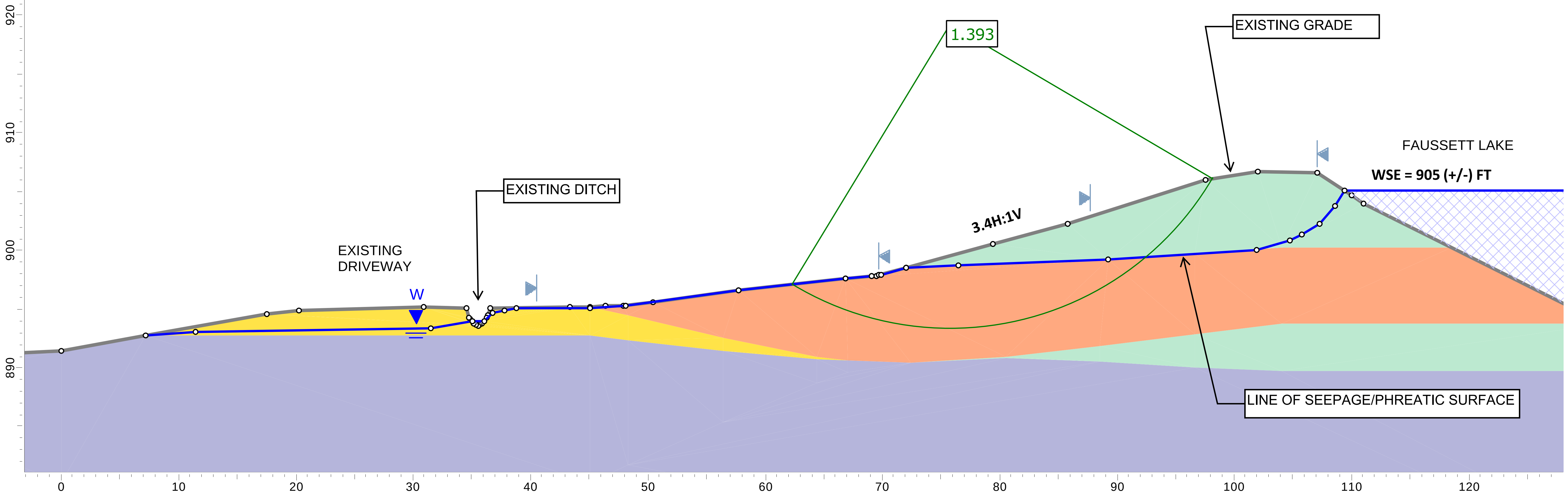
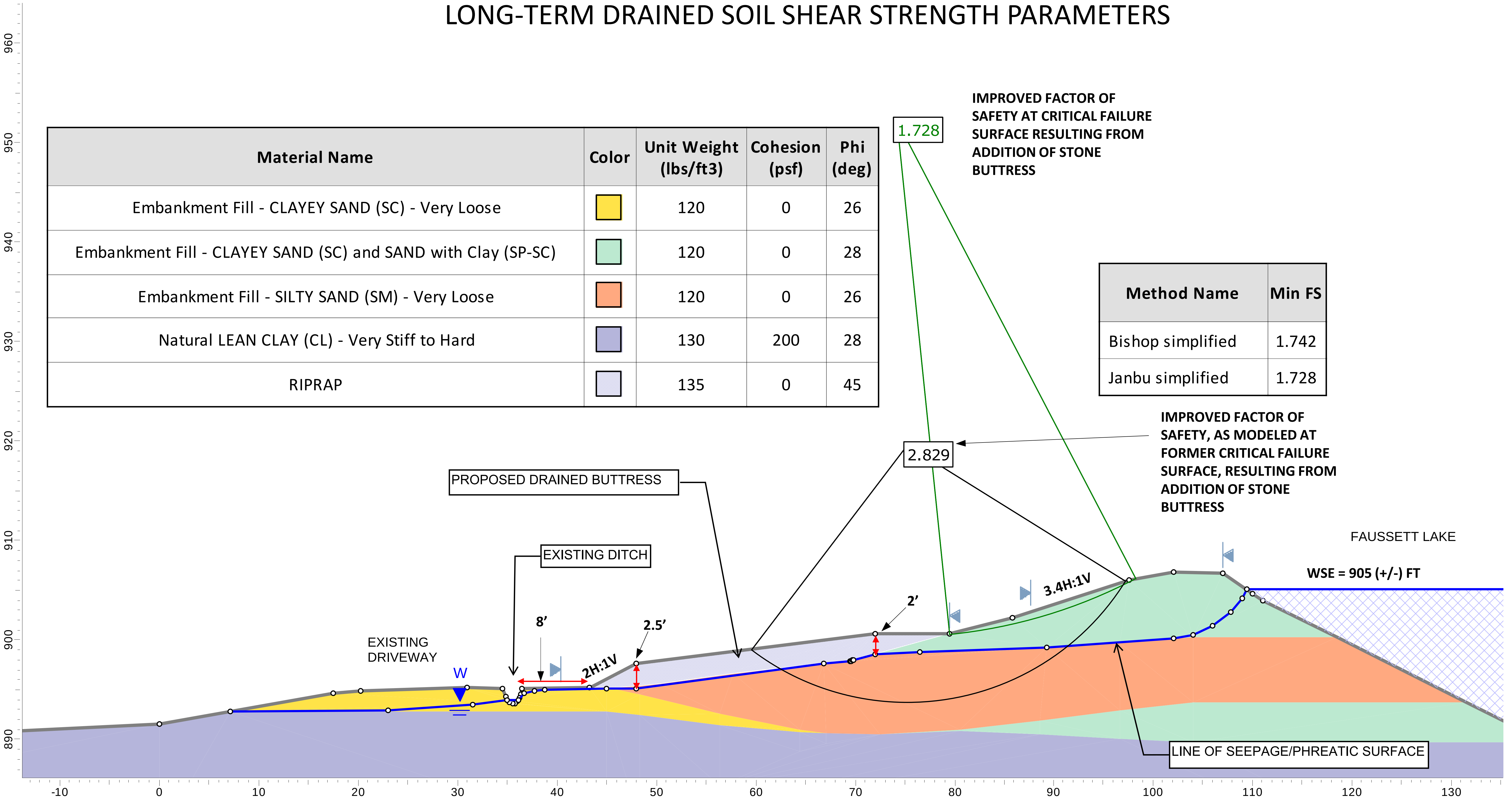


FIGURE NO. C2

**CROSS SECTION A-A WITH DRAINED BUTTRESS,
STEADY-STATE SEEPAGE CONDITIONS**

**FAUSSETT LAKE DAM
DEERFIELD TOWNSHIP, LIVINGSTON COUNTY, MICHIGAN**

GLOBAL STABILITY MODEL
STONE BUTTRESS AT TOE WITH
LONG-TERM DRAINED SOIL SHEAR STRENGTH PARAMETERS



Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Embankment Fill - CLAYEY SAND (SC) - Very Loose	🟨	120	0	26
Embankment Fill - CLAYEY SAND (SC) and SAND with Clay (SP-SC)	🟩	120	0	28
Embankment Fill - SILTY SAND (SM) - Very Loose	🟧	120	0	26
Natural LEAN CLAY (CL) - Very Stiff to Hard	🟪	130	200	28
RIPRAP	⬜	135	0	45

Method Name	Min FS
Bishop simplified	1.742
Janbu simplified	1.728

IMPROVED FACTOR OF SAFETY AT CRITICAL FAILURE SURFACE RESULTING FROM ADDITION OF STONE BUTTRESS

IMPROVED FACTOR OF SAFETY, AS MODELED AT FORMER CRITICAL FAILURE SURFACE, RESULTING FROM ADDITION OF STONE BUTTRESS

Passionate People Building
and Revitalizing our World

SME

APPENDIX F

DAM MAINTENANCE INFORMATION

Dam Ownership
Fact Sheet

TOPIC:
RODENT CONTROLS

Rodents such as the groundhog (woodchuck), muskrat, and beaver are attracted to dams and reservoirs, and can be quite dangerous to the structural integrity and proper performance of the embankment and spillway. Groundhog and muskrat burrows weaken the embankment and can serve as pathways for seepage. Beavers are attracted to running water and may try to plug the spillway and raise the pool level. Rodent control or eradication is essential in preserving a well-maintained dam.

GROUNDHOG

The groundhog is the largest member of the squirrel family. Its coarse fur is a grizzled grayish brown with a reddish cast. Typical foods include grasses, clover, alfalfa, soybeans, peas, lettuce, and apples. Breeding takes place during early spring (beginning at the age of one year) with an average of four or five young per litter, one litter per year. The average life expectancy is two or three years with a maximum of six years.

Occupied groundhog burrows are easily recognized in the spring due to the groundhog's habit of keeping the burrow "cleaned out." Fresh dirt is generally found at the mouth of active burrows. Half-round mounds, paths leading from the den to nearby fields, and clawed or girdled trees and shrubs also help identify inhabited burrows and dens.

When burrowing into an embankment, groundhogs stay above the phreatic surface (upper surface of seepage or saturation) to stay dry. The burrow is rarely a single tunnel. It is usually forked, with more than one entrance and with several side passages or rooms from 1 to 12 feet long.

GROUNDHOG CONTROL

Control methods should be implemented during early spring when active burrows are easy to find, young groundhogs have not scattered, and there is less likelihood of damage to other wildlife. In later summer, fall, and winter, game animals will scurry into groundhog burrows for brief protection and may even take up permanent abode during the period of groundhog hibernation. Groundhogs can be controlled by using fumigants or by shooting. Fumigation is the most practical method of controlling groundhogs. Around buildings or other high fire hazard areas, shooting may be preferable. Groundhogs will be discouraged from inhabiting the embankment if the vegetal cover is kept mowed.

Gas cartridges may be purchased at garden supply and hardware stores. Information about the use and availability of gas cartridges may be obtained from county extension offices, or the U.S. Department of Agriculture.

MUSKRAT

The muskrat is a stocky rodent with a broad head, short legs, small eyes, and rich dark brown fur. Muskrats are chiefly nocturnal. Their principal food includes stems, roots, bulbs, and foliage of aquatic plants. They also feed on snails, mussels, crustaceans, insects, and fish. Usually three to five litters, averaging six to eight young per litter, are produced each year. Adult muskrats average one foot in length and three pounds in weight. The life expectancy is less than two years, with a maximum of four years.

Muskrats can be found wherever there are marshes, swamps, ponds, lakes and streams having calm or very slowly moving water with vegetation in the water and along the banks. Muskrats make their homes by burrowing into the banks of lakes and streams or by building “houses” of bushes and other plants. Their burrows begin from 6 to 18 inches below the water surface and penetrate the embankment on an upward slant. At distances up to 15 feet from the entrance, a dry chamber is hollowed out above the water level. Once a muskrat den is occupied, a rise in the water level will cause the muskrat to dig farther and higher to excavate a new dry chamber. Damage (and the potential for problems) is compounded where groundhogs or other burrowing animals construct their dens in the embankment opposite muskrat dens.

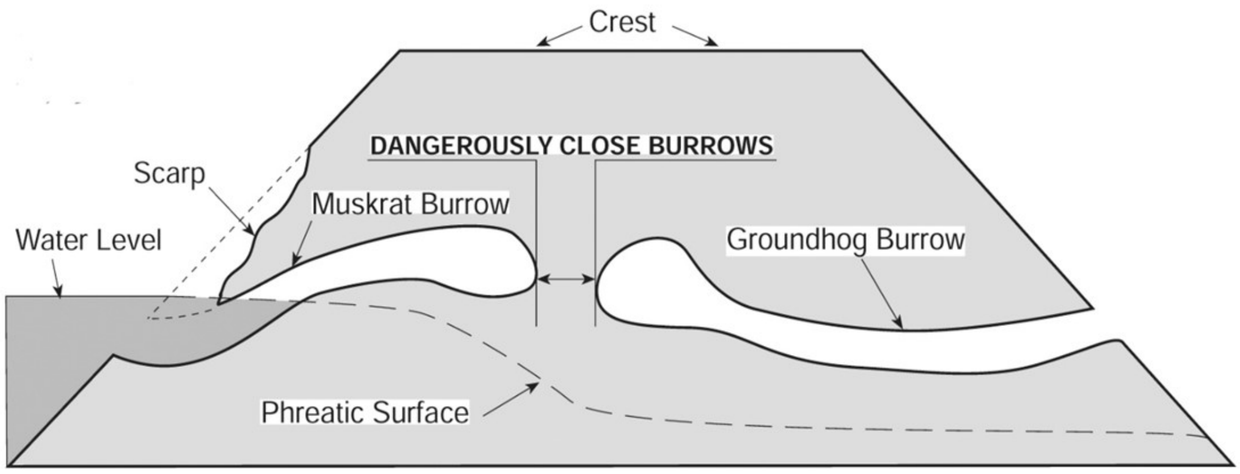
MUSKRAT CONTROL

Barriers to prevent burrowing offer the most practical protection to earthen structures. A properly constructed riprap and filter layer will discourage burrowing. The filter and riprap should extend at least 3 feet below the water line. As the muskrat attempts to construct a burrow, the sand and gravel of the filter layer caves in and thus discourages den building.

Heavy wire fencing laid flat against the slope and extending above and below the water line can also be effective. Eliminating or reducing aquatic vegetation along the shoreline will discourage muskrat habitation. Where muskrats have inhabited the area, trapping is usually the most practical method of removing them.

ELIMINATING A BURROW

The recommended method of backfilling a burrow in an embankment is mud-packing. This simple, inexpensive method can be accomplished by placing one or two lengths of metal stove or vent pipe in a vertical position over the entrance of the den. Making sure that the pipe connection to the den does not leak, the mud-pack mixture is then poured into the pipe until the burrow and pipe are filled with the earth-water mixture. The pipe is removed and dry earth is tamped into the entrance. The mud-pack is made by adding water



to a 90 percent earth and 10 percent cement mixture until a slurry or thin cement consistency is attained. All entrances should be plugged with well-compacted earth and vegetation reestablished. Dens should be eliminated without delay because damage from just one hole can lead to failure of a dam or levee.

BEAVER

Beaver do not necessarily burrow into dams but they will try to plug any spillways, outlets and channels with running water with their cuttings, mud, rocks and debris. Routinely removing the cuttings is one way to alleviate the problem but beaver can rebuild their obstructions overnight. Beaver may also establish large intrusive lodges on the banks or lakes formed by dams. Trapping beaver may be done by the owner during the appropriate season but beaver can migrate up and down a stream or river system and proliferate where habitat is good.

HUNTING AND TRAPPING REGULATIONS

Because hunting and trapping rules and regulations vary from state to state the appropriate State Wildlife Agency should be consulted to ensure compliance with state regulations.

RESOURCES

FEMA #473: FEMA Technical Manual For Dam Owners, "Impact of Animals on Earthen Dams"

FEMA Flyer, "Dam Owner's Guide to Animal Impacts on Earthen Dams," FEMA #L-264.NA

ASDSO Resources
The ASDSO website houses national guidelines on dams. Go to:
DamSafety.Org/ManualsandGuidelines

For more information, videos and tools for dam owners go to:
DamOwner.Org

Watch for training in your area sponsored by ASDSO or your State Dam Safety Office.

Access your state's Dam Safety Program by clicking your state at:
DamSafety.Org/States

APPENDIX G

ASDSO FACT SHEET

Dam Ownership
Fact Sheets

TABLE OF CONTENTS

Dam Ownership
Fact Sheet

TOPIC:

OWNERSHIP RESPONSIBILITY & LIABILITY

Dams are owned and operated by individuals, private and public organizations, and the government. The responsibility for maintaining a safe dam rests with the owner. A dam failure resulting in an uncontrolled release of the reservoir can have a devastating effect on persons and property downstream. Tens of thousands of public and private dam owners in the United States have exposure to liability for the water stored behind their dams. Safely maintaining a dam is a key element in preventing a failure and limiting the liability that an owner could face.

Public safety around dams is also the responsibility of the owner. Dams can create a hazardous environment and dangerous hydraulic features. Dam owners need to consider issues with accessibility by the public to the dam and the surrounding area.

DAM FAILURE

The failure of a dam has the potential for loss of life and catastrophic impact on communities, private property and public works downstream. The data shows that there are approximately 10 to 20 failures per year involving uncontrolled release of reservoirs. Failure of even small dams can result in serious injuries, fatalities, disruption of business operations, damage to critical infrastructure and other extensive property damage.

> *"In today's litigious society it is safe to assume that in the case of catastrophic dam failure, extensive litigation will ensue. Any competent lawyer, representing the victims, will sue all possible wrong doers in seeking redress…including…the owners and operators of the facility, and…architects, engineers, contractors, sub-contractors, and consultants involved in the original construction and any subsequent modifications…."*
>
> *— Denis Binder, Professor of Law, Chapman University*

LOSS FROM FAILURE

The cost of dam failure is difficult to assess because flooding can affect large areas, often beyond the floodplain areas where flood insurance is required. The dam owner loses a valuable asset and faces reconstruction costs and possible liability for downstream damages. Local communities may be directly impacted due to building damage, injuries, fatalities, lost water supply, damaged transportation systems and infrastructure, and lost recreational assets.

Common law holds that the storage of water is a hazardous activity.

COMPLIANCE

Compliance with government or professional standards does not absolve an owner from liability, but it does establish a minimum standard of care to be used by owners. The extent of liability in any situation depends on the facts of the case and how those facts are interpreted by a judge or jury.

Consequently, actions that result in owner liability in one state may not result in liability in different states. In general, a dam owner is required to use "reasonable care" in the operation and maintenance of a dam and reservoir.

STRICT LIABILITY AND NEGLIGENCE

The extent of an owner's liability will vary from state to state, depending on the statutes and case law precedents. The concept of strict liability imposes liability on a dam owner for damages that occur regardless of the cause of failure. The alternative theory of negligence considers the degree of care employed by the owner in constructing, operating and maintaining a dam. Historically, courts have sought to compensate those injured by a dam failure. When assessing liability, the standard of care exercised by an owner will be closely examined. The standard of care should be in proportion to the downstream hazards involved. Where the risk is great, owners must be especially cautious. In many cases, a dam regulated by the federal government or a state dam safety program must be designed to withstand an unprecedented flood or earthquake.

RISK MANAGEMENT

An essential and logical part of an organization's management program is the control of potential losses that may arise. To manage risks, an owner can utilize a combination of standard operating procedures, employee training, regular maintenance, emergency preparedness and liability insurance.

A dam owner can take several actions to protect against financial loss. Technical guidance and information is available from your state's Dam Safety office.

Each dam should have:

- A state dam safety permit (if applicable).
- An operation plan, documented regular maintenance plan and emergency action plan.
- Documented periodic inspections.
- Warning signs and controlled access.

RESOURCES

ASDSO Resources

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State Attorney General's Office
State Office of Emergency Services

Dam Ownership
Fact Sheet

TOPIC:

HOW TO PROCURE THE SERVICES OF A PROFESSIONAL ENGINEER

DAM MAINTENANCE & OWNER RESPONSIBILITY

The responsibility for maintaining a safe dam rests with its owner. The owner must understand the laws and regulations associated with proper dam maintenance and the procedures for keeping these structures safe. Dam owners are also responsible for maintaining safety at and around their dam. It is important to limit trespassing including considering fencing alternatives along high spillway walls and reducing access around dangerous water hydraulics. Proper operation, maintenance, repair and rehabilitation of a dam are key elements in preventing a failure, limiting your liability and maintaining your water resource. One of the most important procedures for ensuring proper maintenance of the dam is procuring the services of a Professional Engineer. A Professional Engineer is one who has been certified by the state and the industry according to their tested ability, schooling and experience.

WHY DO I NEED AN ENGINEER?

All dams meeting government regulatory definitions – no matter what their size or level of engineering – will deteriorate with time. Periodic inspection, proper maintenance and occasional repair and rehabilitation are inevitable. An owner needs the expertise of an engineer to perform inspections or evaluate and undertake corrective measures at a dam. An engineer can investigate the problem and recommend a course of action which may include the design of corrective measures and the preparation of construction plans and specifications. The engineer also can assist in selecting a contractor and will provide valuable construction inspection services.

QUESTIONS TO ASK WHEN HIRING

It is essential to select someone with a Professional Engineer (P.E.) license, with a background in civil engineering, who is competent and experienced in the field of dam safety.

Important criteria to look for in a prospective engineer include the following:

- ✓ A licensed Professional Engineer in your state;
- ✓ A minimum of 10 years of experience in dam design, maintenance, safety and construction;
- ✓ A knowledge of the rules and regulations governing dam design and construction in the state where the dam is located;
- ✓ Specific experience in the problem area—hydrology, hydraulics, structural, soils, seismic, seepage, and geotechnical engineering.

HOW DO I CHOOSE AN ENGINEER WHO IS BEST FOR MY NEEDS?

It is important to use the *Qualification-Based* approach to selecting an engineer. *Qualification-Based* means that the knowledge, experience and ingenuity of the engineer are the determining factors in making the selection. This strategy is advantageous when the owner is uncertain about the exact problem or the best solution to the problem. When *Qualification-Based* selection is used, several engineering firms submit their technical qualifications, experience with similar projects, reputation with existing clients and any other factors pertaining to the specific project. The owner then selects the three to five most qualified firms to make brief presentations outlining a cost-effective and innovative approach to the problem. Based upon these presentations, the owner chooses the most qualified engineer to develop a scope of work. When agreement on the scope of work is achieved, the engineer and the owner negotiate a price that is fair and reasonable to both parties. If an agreement cannot be reached, negotiations start with the second-ranked engineer. In this selection process, price is a factor, but only after the most qualified engineer has been identified.

Fee-Based selection means the engineer's fee is the only determining factor in making the selection. This is not the recommended selection procedure. It is only advantageous when the owner, in conjunction with their State Dam Safety Program, knows exactly what is needed and can clearly define the scope of work before meeting with an engineer. In this case, the engineer is requested to prepare the designs and bid documents or conduct investigations as the owner specifies. A strict *Fee-Based* selection often means the engineer selected may not be qualified to do the work, especially if the bidding is open to anyone and/or the scope of work is poorly defined.

Carefully consider your selection of an engineer. A little work on your part in selecting the engineer may save you money in the future.

FOR YOUR CONSIDERATION

Request references and a portfolio from the engineer. Contact the references of owners and contractors to discuss the engineer's performance. Look at projects that have been completed under the engineer's leadership. Request to review state files of projects an engineer has undertaken to see if the process went smoothly. Maintain an open line of communication with regulatory agencies, particularly your State Dam Safety Program. They may be unable to recommend one engineer over another but they can give an assessment of their previous work. Discuss an engineer's recommended course of action to verify that regulatory requirements will be satisfied. Educate yourself in the basics of dam safety and be knowledgeable regarding the laws you must meet.

RESOURCES

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Dam Ownership
Fact Sheet

TOPIC:

EMERGENCY ACTION PLANNING

Emergency Action Plans Help Dam Owners as Well As People Downstream.

WHAT IS AN EMERGENCY ACTION PLAN?

An Emergency Action Plan (EAP) is a written document that identifies incidents that can lead to potential emergency conditions at a dam, identifies the areas that can be affected by the loss of reservoir and specifies pre-planned actions to be followed to minimize property damage, potential loss of infrastructure and water resource and potential loss of life because of failure or mis-operation of a dam.

The dam owner is responsible for development, maintenance, and exercise of the EAP; however, there are guidelines, tools and assistance available to help owners. City, county and state emergency management directors and state dam safety officials stand ready to partner with dam owners to create and exercise EAPs. An owner can tap into this technical and emergency management expertise and can get additional support by using state and national educational materials, EAP forms and examples, and step-by-step guidelines.

An EAP takes time, focus and dedication. The time is now. The focus is on saving lives. The dedication is to public safety.

The dam owner initiates the EAP process and both emergency responders and owners will be users of the EAP. The completed document should have had input from emergency managers, state dam safety officials, leaders of downstream communities and, directly or indirectly, everyone who may be responsible for the proper implementation of the EAP. It is important that the dam owner stays involved throughout the entire process.

Emergency Action Plans are a public safety benefit for all citizens.

KEY POINTS ABOUT EAPS

- An EAP must clearly specify the dam owner's responsibilities to ensure timely and effective action. Responsibilities of dam owners include: surveillance (monitoring the condition of the dam) and notification (phoning local or state emergency management agency officials in charge of emergency response).
- EAPs are developed by dam owners working with local emergency response managers, dam safety engineers, and state dam safety officials.
- Inundation maps are a key component of the EAP. Inundation maps show areas that may have to be evacuated in a dam emergency. The maps facilitate notification by displaying flood areas and estimated travel times for the floodwaters. New, two-dimensional technologies are available to create inundation maps of areas below dams.
- Dam owners and local emergency responders are primary users of EAPs. A Standard Operating Plan (SOP) is a related document that outlines the normal, non-emergency operation of a dam and is a document for the dam owner and his staff and not a public emergency document.
- Public awareness is a critical component of emergency planning. Many people do not know they may live or work near a dam. Public awareness of an EAP will enhance its effective implementation.
- The EAP defines events that trigger emergency actions.
- An EAP includes a notification flowchart with names and numbers of who will call whom and in what priority.
- Emergency events at dams are infrequent. Training and exercises of EAPs help maintain readiness.
- EAPs should be updated at least once per year and following any changes or new information such as changes in downstream development or new contact information. EAPs should be exercised at least every five years.

DAM OWNER RESPONSIBILITIES

All potentially hazardous dams benefit from some type of an Emergency Action Plan. Obviously dams with a potential for loss of life or damage to infrastructure or high value property in the event of failure (typically identified by regulators as High-Hazard Potential) would be a higher priority and would require a more sophisticated and detailed plan. The regulatory agency responsible for dam safety will probably have criteria for the type and detail of EAP required and the required priority if major repairs are also needed (in many states, dam owners are legally obligated to provide EAPs for certain dam hazards). Regardless of the requirement for a recorded or documented EAP by the Dam Safety Regulatory Program, every dam owner is strongly encouraged to develop some type of EAP that can be used to implement emergency action response in the event of a dam incident.

Regardless of state or federal regulatory requirements, dam owners are responsible and liable for dam operations and any related incidents. EAPs can actually limit a dam owner's liability in the field and in the courtroom because it shows the proper diligence and reasonable actions expected by the law and the dam industry.

Time and effort must be devoted to creating an EAP, filing it appropriately with state and local officials, updating plan details, testing the plan's assumptions and functionality, and following its procedures in an emergency. Completion of an EAP demonstrates that a dam owner is actively attempting to prevent and mitigate harm to persons and property.

Owners must provide an inundation map as part of the EAP.

HOW TO GET STARTED

Contact your state dam safety regulatory office and your consulting engineer.

You can locate your state's office by visiting the ASDSO website (www.damsafety.org/states). ASDSO can point you toward its industry membership to assist in locating a consulting engineer.

Contact the state and local emergency management coordinator.

The primary means of notification to the public is the National Weather Service (NWS). The NWS has the Congressional mandate for issuing flood warnings, which include dam failure. The NWS has a well-established warning infrastructure that includes access to the Emergency Alert System, Weather Radio network, and Internet-based mechanisms.

Know your:
State Dam Safety Officials &
State and Local Emergency
Management Coordinator

TYPICAL EAP COMPONENTS

- Basic Dam Characteristics
- EAP Plan Overview
- Roles & Responsibilities
- Event Detection
- Emergency Level Determination
- Notification & Communication Flowcharts
- Expected Actions
- Termination
- EAP Maintenance Plan (Review, Exercise & Update)
- Appendices including Inundation Maps for Evacuations

WHO IS RESPONSIBLE?

Dam Owners/Operators

- Identification of emergency at dam
- Initial notifications
- Implementation of repairs
- Security and technical assistance on site

Local Emergency Management and Responders

- Public warning
- Possible evacuation
- Shelter plan activated
- Rescue and recovery
- State of Emergency declaration
- Termination of emergency status

State Emergency Management

- Aid affected area when requested
- Coordinate specialized assistance
- Notify appropriate state agencies
- Determine who does what in an emergency

RESOURCES

ASDSO Resources

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DHS/FEMA Resources

DHS and FEMA make several publications and videos available to dam owners through FEMA.gov and DHS.gov (search "dam safety")

Dam Ownership
Fact Sheet

TOPIC:
EARTH DAM FAILURES

Owners of dams, operating personnel, and maintenance personnel must be knowledgeable of the potential problems which can lead to dam failure. These people regularly view the structure and, therefore, need to be able to recognize potential problems so that failure can be avoided. If a problem is noted early enough, an engineer experienced in dam design, construction, and inspection can be contacted to recommend corrective measures, and such measures can be implemented.

IF THERE IS ANY QUESTION AS TO THE SERIOUSNESS OF AN OBSERVATION, AN ENGINEER EXPERIENCED WITH DAMS SHOULD BE CONTACTED.

Acting promptly may avoid possible dam failure and the resulting catastrophic effect on downstream areas.

Since only superficial inspections of a dam can usually be made, it is imperative that owners and maintenance personnel be aware of the prominent types of failure and their telltale signs. Earth dam failures can be grouped into three general categories: overtopping failures, seepage failures, and structural failures. A brief discussion of each type follows.

OVERTOPPING FAILURES

Overtopping failures result from the erosive action of water on the embankment. Erosion is due to uncontrolled flow of water over, around, and adjacent to the dam. Earth embankments are not designed to be overtopped and therefore are particularly susceptible to erosion. Once erosion has begun during overtopping, it is almost impossible to stop. A well vegetated earth embankment may withstand limited overtopping if the dam's crest is level, the downstream slope of the dam is uniform with a consistent slope gradient, and there are no bare areas or undulations along the surface of the dam. The owner should closely monitor the reservoir pool level during severe storms.

SEEPAGE FAILURES

All earth dams leak to some extent and this is known as seepage. This is the result of water moving slowly through the embankment and/or percolating slowly through the dam's foundation. This is normal and usually not a problem with most earthen dams if measures are taken to control movement of water through and under the dam. If uncontrolled, seepage can progressively erode soil from the embankment or its foundation, resulting in failure of the dam. Typically, erosion of embankment soil begins at the downstream side of the dam and progressively works toward the reservoir eventually developing a path to the reservoir which is referred to as "piping." Piping action can be recognized by an increased seepage flow rate, the discharge of muddy or discolored water, sinkholes on or near the embankment, and possibly a whirlpool at the surface of the reservoir. Once a whirlpool (eddy) is observed, failure of the dam may follow. As with overtopping, fully developed piping is virtually impossible to control and will likely cause failure.

Seepage can also cause dam failure by saturating the embankment, thus weakening the dam, or by increasing internal pressure within the embankment. Saturation and internal pressure within the dam are difficult to determine without proper instrumentation.

STRUCTURAL FAILURES

Structural failure typically refers to the collapse of non-earthen embankment dams such as those made from concrete, masonry, or other materials not consisting of a soil matrix. In addition, failure of a dam's appurtenant structures such as a concrete chute spillway slab, gate structures and components, or other such features may lead to failure of the dam itself. Earthen dams do not tend to collapse or fail catastrophically on their own except where earthquakes of significant magnitude are prevalent or other erosive forces weaken the structure. Large cracks in an earthen embankment, major settlement, and major slides may require emergency measures to ensure safety, especially if these problems occur suddenly. If this type of situation occurs, the lake level should be lowered, the appropriate state and local authorities notified, and professional advice sought. If the observer is uncertain as to the seriousness of the problem, a qualified professional engineer with experience in dam safety should be contacted immediately.

The three types of failure previously described are often interrelated in a complex manner. For example, uncontrolled seepage may weaken the soil and lead to a structural failure. A structural failure may shorten the seepage path and lead to a piping failure. Surface erosion may result in structural failure.

Minor defects such as cracks in the embankment may be the first visual sign of a major problem which could lead to failure of the structure. The seriousness of all deficiencies should be evaluated by someone experienced in dam design and construction. A qualified professional engineer can recommend appropriate permanent remedial measures.

RESOURCES

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Dam Ownership
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TOPIC:

EMBANKMENT INSTABILITIES

The dam embankment and any appurtenant dikes must safely contain the reservoir during normal and flood conditions. Cracks, slides, and depressions are signs of embankment instability and should indicate to the owner that maintenance or repair work may be required. When one of these conditions is detected, the owner must retain an experienced professional engineer to determine the cause of the instability. A rapidly changing condition or the sudden development of a large crack, slide, or depression indicates a very serious problem, and the state dam safety agency should be contacted immediately. A professional engineer must investigate these types of embankment stability problems because a so-called “home remedy” may cause greater and more serious damage to the embankment and eventually result in unneeded expenditures for unsuccessful repairs.

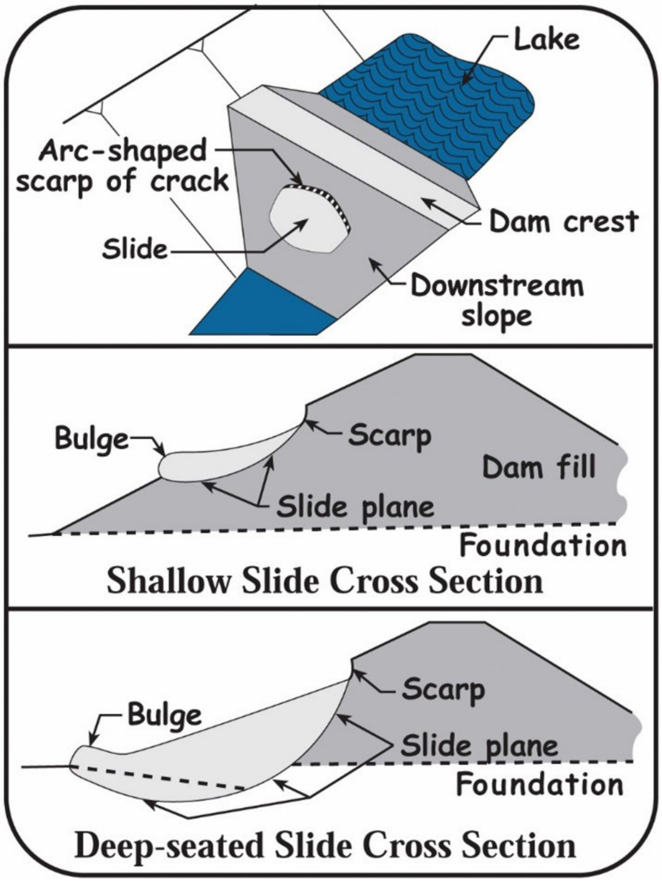
CRACKS

Short, isolated cracks are commonly due to drying and shrinkage of the embankment surface and are not usually significant. They are usually less than 1 inch wide, propagate in various directions, and occur especially where the embankment lacks a healthy grass cover. Larger (wider than 1 inch), well-defined cracks may indicate a more serious problem. There are generally two types of these cracks: longitudinal and transverse. Longitudinal cracks extend parallel to the crest of the embankment and may indicate the early stages of a slide on either the upstream or downstream slope of the embankment. They can create problems by allowing runoff to enter the cracks and saturate the embankment which in turn can cause instability of the embankment. Transverse cracks extend perpendicular to the crest and can indicate differential settlement within the embankment. Such cracks provide avenues for seepage through the dam and could quickly lead to piping, a severe seepage problem that will likely cause the dam to fail. If the owner finds small cracks during inspection of the dam, they should document the observations, and seal the cracks to prevent runoff from saturating the embankment. The documentation should consist of detailed notes (including the location, length, approximate elevation, and crack width), photographs, sketches, and possibly monitoring stakes. The crack must then be monitored during future inspections. If the crack becomes longer or wider, a more serious problem such as a slide may be developing. Large cracks indicate serious stability problems. If one is detected, the owner should contact the state dam safety agency and retain an engineer to investigate the crack and prepare plans and specifications, if necessary, for repairs. When muddy flow discharges from a crack, the dam may be close to failure. The emergency action plan should be initiated immediately and the state dam safety agency contacted.

SLIDES

A slide in an embankment or in natural soil or rock is a mass movement of material. Some typical characteristics of a slide are an arc-shaped crack or scarp along the top and a bulge along the bottom of the slide (see drawing). Slides may develop because of poor soil compaction, the gradient of the slope being too steep for the embankment material, seepage, sudden drawdown of the lake level, undercutting of the embankment toe, or saturation and weakening of the embankment or foundation.

Slides can be divided into two main groups: shallow and deep-seated. Shallow slides generally affect the top 2 to 3 feet of the embankment surface. Shallow slides are generally not threatening to the immediate safety of the dam and often result from wave erosion, collapsed rodent burrows, or saturated top soil. Deep-seated slides are serious, immediate threats to the safety of a dam. They can extend several feet below the surface of the embankment, even below the foundation. A massive slide can initiate the catastrophic failure of a dam. Deep-seated slides are the result of serious problems within the embankment.



Small slides can be repaired by removing the vegetation and any unsuitable fill from the area, compacting suitable fill and adding topsoil to make the embankment uniform, and establishing a healthy grass cover. If a shallow or deep-seated slide is discovered, the state dam safety agency should be contacted and an engineer retained to investigate the slide. Plans and specifications may need to be prepared for its repair depending on the findings of the investigation.

DEPRESSIONS

Depressions are sunken areas of the abutment, toe area, or embankment surface. They may be created during construction, or may be caused by decay of buried organic materials, thawing of frozen embankment material, internal erosion of the embankment, or settlement (consolidation) of the embankment or its foundation. To a certain degree, minor depressions are common and do not necessarily indicate a serious problem. An embankment with several minor depressions may be described as hummocky. However, larger depressions may indicate serious problems such as weak foundation materials, poor compaction of the embankment during construction, or internal erosion of the embankment fill.

Depressions can create low areas along the crest, cracks through the embankment, structural damage to spillways or other appurtenant structures, damage to internal drainage systems, or general instability of the embankment. They can also inhibit maintenance of the dam and make detection of stability or seepage problems difficult. The owner should monitor depressions during the regular inspection of the dam. All observations should be documented with detailed notes, photographs, and sketches. Minor depressions can be repaired by removing the vegetation and any unsuitable fill from the area, adding fill and then topsoil to make the embankment uniform, and finally establishing a healthy grass cover. An engineer should be retained to investigate large depressions or settlement areas. Plans and specifications may need to be prepared for its repair depending on the findings of the investigation.

IMPORTANCE OF INSPECTION

Stability problems can threaten the safety of the dam and the safety of people and property downstream. Therefore, stability problems must be detected and repaired in a timely manner. The entire embankment should be routinely and closely inspected for cracks, slides, and depressions. To do this thoroughly, proper vegetation must be regularly maintained on the embankment. Improper or overgrown vegetation can inhibit visual inspection and maintenance of the dam. Accurate inspection records are also needed to detect stability problems. These records can help determine if a condition is new, slowly changing, or rapidly changing. A rapidly changing condition or the sudden development of a large crack, slide, or depression indicates a very serious problem, and the state dam safety agency should be contacted immediately.

RESOURCES

ASDSO Resources

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Dam Ownership
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TOPIC:
SEEPAGE THROUGH EARTHEN DAMS

Areas downstream from dams are not usually natural springs, but most likely seepage exiting from the dam's embankment. Even if natural springs exist, they should be treated with suspicion and carefully observed. Flows from ground-water springs in existence prior to the reservoir would probably increase due to the pressure caused by the pool of water behind the dam.

All dams have some seepage as the impounded water seeks paths of least resistance through the dam and its foundation. Seepage must, however, be controlled to prevent erosion of the embankment or foundation materials or damage to concrete structures.

DETECTION

Seepage can emerge anywhere on the downstream face, beyond the toe, or on the downstream abutments at elevations below normal pool. Seepage may vary in appearance from a "soft," wet area to a flowing "spring." It may show up first as an area where the vegetation is lush and darker green. Cattails, reeds, mosses, and other marsh vegetation often become established in a seepage area. Another indication of seepage is the presence of rust-colored iron bacteria. Due to their nature, the bacteria are found more often where water is discharging from the ground than in surface water. Seepage can make inspection and maintenance difficult. It can also saturate and weaken portions of the embankment and foundation, making the embankment susceptible to earth slides.

If the seepage forces are large enough, soil will be eroded from the foundation and be deposited in the shape of a cone around the seepage outlet. If these "boils" appear, professional advice should be sought immediately. Seepage flow which is muddy and carrying sediment (soil particles) is evidence of "piping," and could very possibly cause failure of the dam. Piping can occur along a spillway and other conduits through the embankment, and these areas should be closely inspected. Sinkholes may develop on the surface of the embankment as internal erosion takes place. A whirlpool in the lake surface may follow and then likely a rapid and complete failure of the dam. Emergency procedures, including downstream evacuation, should be implemented if this condition is noted.

Seepage can also develop behind or beneath concrete structures such as chute spillways or headwalls. If the concrete structure does not have a means such as weep holes or relief drains to relieve the water pressure, the concrete structure may heave, rotate, or crack. The effects of the freezing and thawing can amplify these problems. It should be noted that the water pressure behind or beneath structures may also be due to infiltration of surface water or spillway discharge.

A continuous or sudden drop in the normal lake level is another indication that seepage is occurring. In this case, one or more locations of flowing water are usually noted downstream from the dam. This condition, in itself, may not be a serious problem, but will require frequent and close monitoring and professional assistance.

CONTROL

The need for seepage control will depend on the quantity, content, and location of the seepage. Reducing the quantity of seepage that occurs after construction is difficult and expensive. It is not usually attempted unless the seepage has lowered the pool level or is endangering the embankment or appurtenant structures. Typical methods used to control the quantity of seepage are grouting or installation of an upstream blanket. Of these methods, grouting is probably the least effective and is most applicable to leakage zones in bedrock, abutments, and foundations. These methods must be designed and constructed under the supervision of a qualified professional engineer experienced with dams. Controlling the volume of the seepage or preventing seepage flow from removing soil particles from the embankment is extremely important. Modern design practice incorporates this control into the embankment through the use of cutoffs, internal filters, and adequate drainage provisions. Control at points of seepage exit can be accomplished after construction by installation of toe drains, relief wells, or inverted filters. Weep holes and relief drains can be installed to relieve water pressure or drain seepage from behind or beneath concrete structures. These systems must be designed to prevent migration of soil particles but still allow the seepage to drain freely. The owner must retain a professional engineer to design toe drains, relief wells, inverted filters, weep holes, or relief holes.

MONITORING

Regular monitoring is essential to detect seepage and prevent a potential dam failure. Knowledge of the dam's history is important to determine whether the seepage condition is in a steady or changing state. It is important to keep written records of points of seepage exit, quantity and content of flow, size of wet area, and type of vegetation for later comparison. Photographs provide invaluable records of seepage. All records should be kept in the operation, maintenance, and inspection manual for the dam. The inspector should always look for increases in flow and evidence of flow carrying soil particles, which would indicate that a more serious problem is developing. Instrumentation can also be used to monitor seepage. V-notch weirs can be used to measure flow rates, and piezometers may be used to determine the saturation level (phreatic surface) within the embankment.

Regular surveillance and maintenance of internal embankment and foundation drainage outlets is also required. The rate and content of flow from each pipe outlet for toe drains, relief wells, weep holes, and relief drains should be monitored and documented regularly. Normal maintenance consists of removing all obstructions from the pipe to allow for free drainage of water from the pipe. Typical obstructions include debris, gravel, sediment, and rodent nests. Water should not be permitted to submerge the pipe outlets for extended periods of time. This will inhibit inspection and maintenance of the drains and may cause them to clog.

RESOURCES

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Dam Ownership **Fact Sheet**

TOPIC:

INTERNAL EROSION OF EARTH DAMS

WHAT IS INTERNAL EROSION?

Internal erosion (called "piping" by dam engineers) of an earth dam takes place when water that seeps through the dam carries soil particles away from the embankment, filters, drains, foundation or abutments of the dam.

If the seepage that discharges at the downstream side of the dam carries particles of soil, an elongated cavity or "pipe" may be eroded backward (working upstream) toward the reservoir through the embankment, foundation or abutment. When a backward-eroding pipe reaches the reservoir, a catastrophic breaching of the dam can occur. Internal erosion usually takes place in episodes of erosion and discharge of muddy water interspersed with periods of clear-water discharge or no discharge at all depending on head and flow. Internal erosion may be taking place even if there is no visible discharge of water or if the water that is discharging from the soil on the downstream side of a dam is not muddy. Chemicals, salts, dissolved and suspended solids and dispersive clays can also erode unnoticed from the inside of a dam. The only way to monitor this, in the absence of visible erosion or sand boil deposits, is to send samples to a lab for testing.

Failure at a dam caused by internal erosion of the soil abutment.

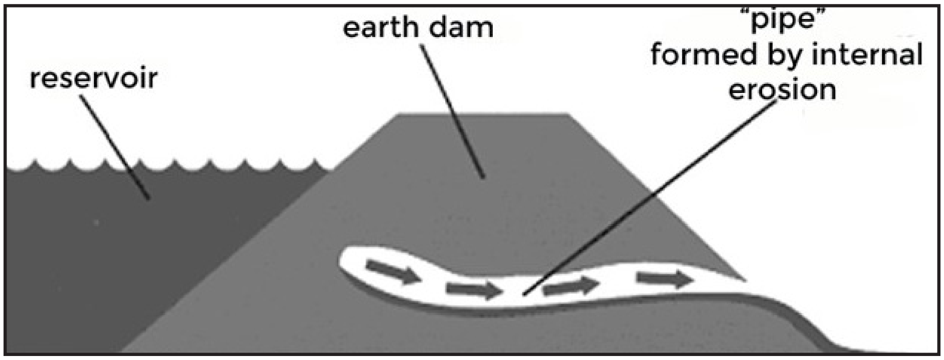

Illustration of internal erosion (piping) in an earth dam embankment.

INTERNAL EROSION BASICS

Internal erosion is one of the most common causes of failure of earth dams.

There may be no external evidence, or only subtle evidence, that it is taking place.

A dam may breach within a few hours after evidence of the internal erosion becomes obvious. Internal erosion may develop the first time water is impounded behind a dam, or it may develop slowly over many years.

Higher water surface elevations and pressure may exacerbate or initiate internal erosion.

You cannot assume that your dam is safe against internal erosion just because it has performed satisfactorily for many years.

Internal erosion failures are often associated with "penetrations" of dams, such as outlet pipes buried in the embankment, rodent activity, and concrete spillways that cross the embankment.

An experienced dam engineer may be able to detect the subtle signs of internal erosion during routine periodic inspections, but you should be aware of what signs to look for between inspections.

If you do observe signs of internal erosion, you should get help from an experienced dam engineer.

Above: Seepage that developed where the root ball of a tree pulled out of the ground near the downstream toe of the dam.

Above: Sinkhole on the crest of an earth dam. The reservoir was lowered and a cavity was found under the sinkhole.

Above: Corroded corrugated-metal outlet pipe removed from a dam that had developed large sinkhole.

Left: Failure of an earth dam by internal erosion along concrete outlet pipe.

SIGNS OF A DEVELOPING SITUATION

What to Look For:

- ✓ Water discharging on the downstream slope of an earth dam or within a few hundred feet downstream from the dam. Look for any accumulation of sediment downstream from the discharge.
- ✓ Water flowing along the outside of a pipe, concrete spillway, or other structure that penetrates the embankment.
- ✓ Water discharging near the roots of a living or dead tree.
- ✓ Corrosion or deterioration of the visible portion of a low-level outlet pipe or other structure that penetrates the embankment.
- ✓ Trees that are uprooted on the embankment or abutments or in the valley bottom immediately downstream from the dam.
- ✓ Water emanating from animal borrows.
- ✓ Dead trees (the rotting roots of which may become avenues of internal erosion) on the embankment or abutments or in the valley bottom immediately downstream from the embankment.

WHAT TO DO

As soon as possible, contact your qualified Professional Engineer or dam safety consultant to inspect the dam and then call your state dam safety engineer.

Research the history of seepage in previous dam inspection and monitoring reports. Look for changes of flow quality and quantity.

SIGNS OF IMMINENT DANGER

What to Look For:

- ✓ Muddy water or large flow of clear water discharging (1) from soil anywhere on the downstream side of the dam, (2) next to a spillway, pipe or other structure that penetrates the embankment or abutments, or (3) from drain pipes in the embankment. Muddy water discharging from the downstream side of a dam or from a drain or low-level outlet pipe, which may indicate that the dam is failing.
- ✓ Sinkholes or subsidence anywhere on the embankment or an abutment. Water flowing into a sinkhole below the reservoir surface on the upstream slope of a dam is especially dangerous.

WHAT TO DO

Immediately call your emergency management, public safety officials or 911 for imminent dangers.

Activate your Emergency Action Plan and call your engineer and the State Dam Safety Program.

RESOURCES

All guidelines and tools for owners are available at the ASDSO website for owners: ***DamOwner.org***

To view an animation of a piping failure, go to ASDSO's YouTube site.

Dam Ownership
Fact Sheet

TOPIC:

GROUND COVER

The establishment and control of proper vegetation are an important part of dam maintenance. Properly maintained vegetation can help prevent erosion of embankment and earth channel surfaces, and aid in the control of groundhogs and muskrats. The uncontrolled growth of vegetation can damage embankments and concrete structures and make close inspection difficult. Grass vegetation is an effective and inexpensive way to prevent erosion of embankment surfaces. If properly maintained, it also enhances the appearance of the dam and provides a surface that can be easily inspected. Roots and stems tend to trap fine sand and soil particles, forming an erosion-resistant layer once the plants are well established.

Grass vegetation may not be effective in areas of concentrated runoff, such as at the contact of the embankment and abutments, or in areas subjected to wave action.

COMMON PROBLEMS

Bare Areas

Bare areas on an embankment are void of protective cover (e.g. grass, asphalt, riprap etc.). They are more susceptible to erosion which can lead to localized stability problems such as small slides and sloughs. Bare areas must be repaired by establishing a proper grass cover or by installing other protective cover. If using grass, the topsoil must be prepared with fertilizer and then scarified before sowing seed. Types of grass vegetation that have been used on dams are bluegrass, fescue, ryegrass, alfalfa, clover, and redtop. One suggested seed mixture is 30% Kentucky Bluegrass, 60% Kentucky 31 Fescue, and 10% Perennial Ryegrass. Once the seed is sown, the area should be mulched and watered regularly.

Erosion

Embankment slopes are normally designed and constructed so that the surface runoff will be spread out in a thin layer as "sheet flow" over the grass cover. When the sod is in poor condition or flow is concentrated at one or more locations, the resulting erosion will leave rills and gullies in the embankment slope. The erosion will cause loss of material and make maintenance of the embankment difficult. Prompt repair of the erosion is required to prevent more serious damage to the embankment. If erosion gullies are extensive, a registered professional engineer may be required to design a more rigid repair such as riprap or concrete. Minor rills and gullies can be repaired by filling them with compacted cohesive material. Topsoil should be a minimum of 4 inches deep. The area should then be seeded and mulched. Not only should the eroded areas be repaired, but the cause of the erosion should be addressed to prevent a continued maintenance problem.

Footpaths

Paths from animal and pedestrian traffic are problems common to many embankments. If a path has become established, vegetation in this area will not provide adequate protection and a more durable cover will be required unless the traffic is eliminated. Gravel, asphalt, and concrete have been used effectively to cover footpaths. Embedding railroad ties or other treated wood beams into an embankment slope to form steps is one of the most successful and inexpensive methods used to provide a protected pathway.

Vehicle Ruts

Vehicle ruts can also be a problem on the embankment. Vehicular traffic on the dam should be discouraged especially during wet conditions except when necessary. Water collected in ruts may cause localized saturation, thereby weakening the embankment. Vehicles can also severely damage the vegetation on embankments. Worn areas could lead to erosion and more serious problems. Ruts that develop in the crest should be repaired by grading to direct all surface drainage into the impoundment. Bare and eroded areas should be repaired using the methods mentioned in the above sections. Constructed barriers such as fences and gates are effective ways to limit access of vehicles.

Improper Vegetation

Vegetation that hides the embankment surface, preventing early detection of cracks and erosion, is not recommended. Crown vetch is an example of this type of vegetation. It is a perennial plant with small pink flowers. It is also not effective in preventing erosion.

Vines and woody vegetation such as trees and brush also hide the embankment surface preventing early detection of cracks and erosion. Tall vegetation also provides a habitat for burrowing animals.

All improper vegetation must be removed from the entire embankment surface. Any residual roots that are larger than 3 inches in diameter must be removed. All roots should be removed down to a depth of at least 6 inches and replaced with a compacted clay material; then 4 inches of topsoil should be placed on the disturbed areas of the slope. Finally, these areas must be seeded and mulched to establish a proper grass cover.

MAINTENANCE

Embankments, areas adjacent to spillway structures, vegetated channels, and other areas associated with a dam require continual maintenance of the vegetal cover. Removal of improper vegetation is necessary for the proper maintenance of a dam, dike or levee. All embankment slopes and vegetated earth spillways should be mowed at least twice a year. Reasons for proper maintenance of the vegetal cover include unobstructed viewing during inspection, maintenance of a non-erodible surface, discouragement of burrowing animal habitation, and aesthetics. Common methods for control of vegetation include the use of weed trimmers or power brush-cutters and mowers. Chemical spraying to kill small trees and brush is acceptable if precautions are taken to protect the local environment. Some chemical spraying may require proper training prior to application.

RESOURCES

ASDSO Resources

The ASDSO website houses national guidelines on dams. Go to: *DamSafety.Org/ManualsandGuidelines*

For more information, videos and tools for dam owners go to: *DamOwner.Org*

Watch for training in your area sponsored by ASDSO or your State Dam Safety Office.

Access your state's Dam Safety Program by clicking your state at: *DamSafety.Org/States*

Dam Ownership **Fact Sheet**

TOPIC:

TREES AND BRUSH

The establishment and control of proper vegetation is an important part of dam maintenance. Properly maintained vegetation can help prevent erosion of embankment and earth channel surfaces and aid in the control of groundhogs and muskrats. The uncontrolled growth of vegetation can damage embankments and concrete structures and make close inspection difficult.

TREES AND BRUSH

Trees and brush should not be permitted on embankment surfaces or in vegetated earth spillways. Extensive root systems can provide seepage paths for water. Trees that blow down or fall over can leave large holes in the embankment surface that will weaken the embankment and can lead to increased erosion. Brush obscures the surface limiting visual inspection, providing a haven for burrowing animals, and inhibiting the growth of grass vegetation. Tree and brush growth adjacent to concrete walls and structures may eventually cause damage to the concrete and should be removed.

STUMP REMOVAL & SPROUT PREVENTION

Stumps of cut trees should be removed so vegetation can be established and the surface mowed. Small stumps may be entirely removed if removal does not require extensive excavation into the embankment which could compromise the structural integrity of the dam. If the stump is of sufficient size where complete removal would require significant excavation into the embankment, then the stump should be ground down to about 6 inches below the surface. All other woody material should also be removed or ground down to about 6 inches below the ground surface. The cavity should be filled with well-compacted clay soil with a surface dressing of top soil to promote a vigorous grass cover.

Stumps of trees in riprap should be cut as close to the rock layer as possible and then chemically treated so they will not form new sprouts. Certain herbicides are effective for this purpose and can even be used at water supply reservoirs if applied by licensed personnel. These products should be applied in strict coherence with local and state herbicide regulations. Other instructions found on the label should be strictly followed when handling and applying these materials. Only a few commercially available chemicals can be used along shorelines or near water.

Tree roots growing into the dam's earth embankment causing failure.

EMBANKMENT MAINTENANCE

Embankments, areas adjacent to spillway structures, vegetated channels, and other areas associated with a dam require continual maintenance of the vegetal cover. Grass mowing, brush cutting, and removal of woody vegetation (including trees) are necessary for the proper maintenance of a dam, dike, or levee. All embankment slopes and vegetated earth spillways should be mowed at least twice per year: once in the late spring and then during fall when the growing season subsides. Aesthetics, unobstructed viewing during inspections, maintenance of a non-erodible surface, and discouragement of burrowing animal habitation are reasons for proper maintenance of the vegetal cover.

Methods used in the past for control of vegetation but now are considered unacceptable include chemical spraying and burning. Acceptable methods include the use of weed whips or power brush-cutters and mowers. Chemical spraying to first kill small trees and brush is acceptable if precautions are taken to protect the local environment.

It is important to remember not to mow when the embankment is wet. It is also important to use proper equipment for the slope and type of vegetation to be cut. Also, always follow the manufacturer's recommended safe operation procedures.

Slope Mower

Properly maintained embankment.

RESOURCES

ASDSO Resources

The ASDSO website houses national guidelines on plant impacts on dams.
DamSafety.Org/ManualsandGuidelines

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DamSafety.Org/States

DHS / FEMA Resources

DHS and FEMA make several publications and videos available to dam owners through:
FEMA.gov and *DHS.gov*
(search "dam safety")

Dam Ownership
Fact Sheet

TOPIC:
RODENT CONTROLS

Rodents such as the groundhog (woodchuck), muskrat, and beaver are attracted to dams and reservoirs, and can be quite dangerous to the structural integrity and proper performance of the embankment and spillway. Groundhog and muskrat burrows weaken the embankment and can serve as pathways for seepage. Beavers are attracted to running water and may try to plug the spillway and raise the pool level. Rodent control or eradication is essential in preserving a well-maintained dam.

GROUNDHOG

The groundhog is the largest member of the squirrel family. Its coarse fur is a grizzled grayish brown with a reddish cast. Typical foods include grasses, clover, alfalfa, soybeans, peas, lettuce, and apples. Breeding takes place during early spring (beginning at the age of one year) with an average of four or five young per litter, one litter per year. The average life expectancy is two or three years with a maximum of six years.

Occupied groundhog burrows are easily recognized in the spring due to the groundhog's habit of keeping the burrow "cleaned out." Fresh dirt is generally found at the mouth of active burrows. Half-round mounds, paths leading from the den to nearby fields, and clawed or girdled trees and shrubs also help identify inhabited burrows and dens.

When burrowing into an embankment, groundhogs stay above the phreatic surface (upper surface of seepage or saturation) to stay dry. The burrow is rarely a single tunnel. It is usually forked, with more than one entrance and with several side passages or rooms from 1 to 12 feet long.

GROUNDHOG CONTROL

Control methods should be implemented during early spring when active burrows are easy to find, young groundhogs have not scattered, and there is less likelihood of damage to other wildlife. In later summer, fall, and winter, game animals will scurry into groundhog burrows for brief protection and may even take up permanent abode during the period of groundhog hibernation. Groundhogs can be controlled by using fumigants or by shooting. Fumigation is the most practical method of controlling groundhogs. Around buildings or other high fire hazard areas, shooting may be preferable. Groundhogs will be discouraged from inhabiting the embankment if the vegetal cover is kept mowed.

Gas cartridges may be purchased at garden supply and hardware stores. Information about the use and availability of gas cartridges may be obtained from county extension offices, or the U.S. Department of Agriculture.

MUSKRAT

The muskrat is a stocky rodent with a broad head, short legs, small eyes, and rich dark brown fur. Muskrats are chiefly nocturnal. Their principal food includes stems, roots, bulbs, and foliage of aquatic plants. They also feed on snails, mussels, crustaceans, insects, and fish. Usually three to five litters, averaging six to eight young per litter, are produced each year. Adult muskrats average one foot in length and three pounds in weight. The life expectancy is less than two years, with a maximum of four years.

Muskrats can be found wherever there are marshes, swamps, ponds, lakes and streams having calm or very slowly moving water with vegetation in the water and along the banks. Muskrats make their homes by burrowing into the banks of lakes and streams or by building "houses" of bushes and other plants. Their burrows begin from 6 to 18 inches below the water surface and penetrate the embankment on an upward slant. At distances up to 15 feet from the entrance, a dry chamber is hollowed out above the water level. Once a muskrat den is occupied, a rise in the water level will cause the muskrat to dig farther and higher to excavate a new dry chamber. Damage (and the potential for problems) is compounded where groundhogs or other burrowing animals construct their dens in the embankment opposite muskrat dens.

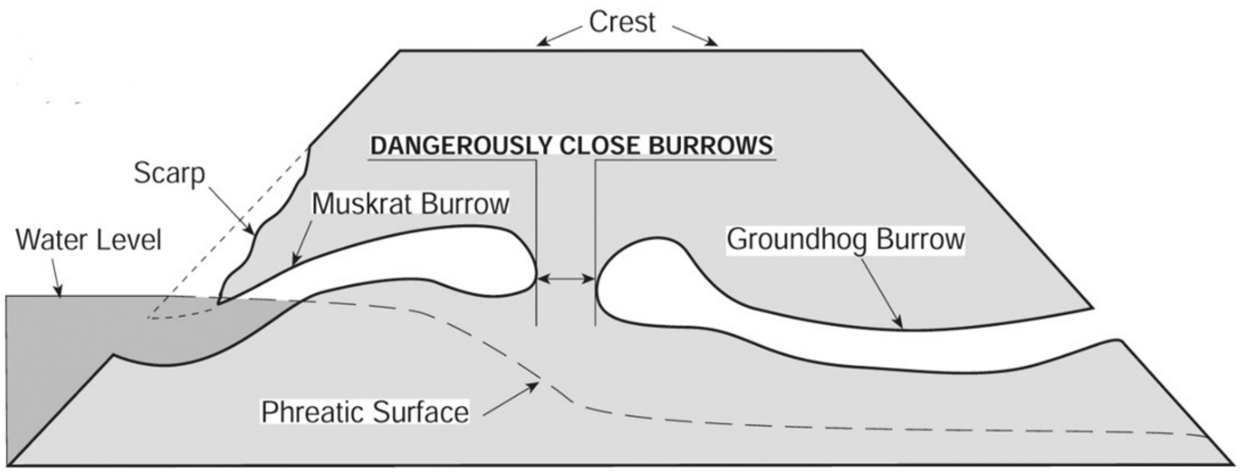
MUSKRAT CONTROL

Barriers to prevent burrowing offer the most practical protection to earthen structures. A properly constructed riprap and filter layer will discourage burrowing. The filter and riprap should extend at least 3 feet below the water line. As the muskrat attempts to construct a burrow, the sand and gravel of the filter layer caves in and thus discourages den building.

Heavy wire fencing laid flat against the slope and extending above and below the water line can also be effective. Eliminating or reducing aquatic vegetation along the shoreline will discourage muskrat habitation. Where muskrats have inhabited the area, trapping is usually the most practical method of removing them.

ELIMINATING A BURROW

The recommended method of backfilling a burrow in an embankment is mud-packing. This simple, inexpensive method can be accomplished by placing one or two lengths of metal stove or vent pipe in a vertical position over the entrance of the den. Making sure that the pipe connection to the den does not leak, the mud-pack mixture is then poured into the pipe until the burrow and pipe are filled with the earth-water mixture. The pipe is removed and dry earth is tamped into the entrance. The mud-pack is made by adding water



to a 90 percent earth and 10 percent cement mixture until a slurry or thin cement consistency is attained. All entrances should be plugged with well-compacted earth and vegetation reestablished. Dens should be eliminated without delay because damage from just one hole can lead to failure of a dam or levee.

BEAVER

Beaver do not necessarily burrow into dams but they will try to plug any spillways, outlets and channels with running water with their cuttings, mud, rocks and debris. Routinely removing the cuttings is one way to alleviate the problem but beaver can rebuild their obstructions overnight. Beaver may also establish large intrusive lodges on the banks or lakes formed by dams. Trapping beaver may be done by the owner during the appropriate season but beaver can migrate up and down a stream or river system and proliferate where habitat is good.

HUNTING AND TRAPPING REGULATIONS

Because hunting and trapping rules and regulations vary from state to state the appropriate State Wildlife Agency should be consulted to ensure compliance with state regulations.

RESOURCES

FEMA #473: FEMA Technical Manual For Dam Owners, "Impact of Animals on Earthen Dams"

FEMA Flyer, "Dam Owner's Guide to Animal Impacts on Earthen Dams," FEMA #L-264.NA

ASDSO Resources

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Dam Ownership
Fact Sheet

TOPIC:
SPILLWAY CONDUIT SYSTEM PROBLEMS

Many dams have conduit systems that serve as principal spillways. These conduit systems are required to carry normal stream and small flood flows safely past the embankment throughout the life of the structure. Conduits through embankments are difficult to construct properly and can be extremely dangerous to the embankment if problems develop after construction. Conduits are usually difficult to repair because of their location within the embankment. Also, replacing conduits requires extensive excavation. In order to avoid difficult and costly repairs, particular attention should be directed to maintaining these structures. The most common problem noted with spillway conduit systems is undermining of the conduit. This condition typically results from water leaking through pipe joints, seepage along the conduit or inadequate energy dissipation at the conduit outlet. The typical causes of seepage and water leaking through pipe joints include any one or a combination of the following factors: loss of joint material, separated joints, misalignment, differential settlement, conduit deterioration, and pipe deformation. Problems in any of these areas may lead to failure of the spillway system and possibly dam failure.

UNDERMINING

Undermining is the removal of foundation material surrounding a conduit system. Any low areas or unexplained settlement of the earthfill in line with the conduit may indicate that undermining has occurred within the embankment. As erosion continues, undermining of a conduit can lead to displacement and collapse of the pipe sections and cause sloughing, sliding or other forms of instability in the embankment. As the embankment is weakened, a complete failure of the conduit system and, eventually the dam may occur.

Seepage along the conduit from the reservoir can occur because of poor compaction around the conduit. If seepage control devices have not been installed, the seepage may remove foundation material from around the conduit and eventually lead to undermining.

In addition, undermining can occur as the result of erosion due to inadequate energy dissipation or inadequate erosion protection at the outlet. This undermining can be visually observed at the outlet of a pipe system and can extend well into the embankment. In this case, undermining can lead to other conduit problems such as misalignment, separated joints and pipe deterioration. An extensive discussion on outlet erosion control as it relates to undermining of the pipe outlet can be found in the “Outlet Erosion Control Structures” fact sheet.

Installation of seepage control devices is required as a preventative measure to control seepage along the conduit and undermining. Regular monitoring of conduit systems must include visual observation and notation of any undermining or any precursors. These precursors usually include pipe deformation, misalignment and differential settlement, pipe deterioration, separated joints and loss of joint material.

PIPE DEFORMATION

Pipe deformations are typically caused by external loads that are applied on a pipe such as the weight of the embankment or heavy equipment. Collapse of the pipe can cause failure of the joints and allow erosion of the supporting fill. This may lead to undermining and settlement. Pipe deformation may reduce or eliminate spillway capacity. Pipe deformation must be monitored on a regular basis to ensure that no further deformation is occurring, that pipe joints are intact and that no undermining or settlement is occurring.

SEPARATED JOINTS AND LOSS OF JOINT MATERIAL: JOINT DETERIORATION

Conduit systems usually have construction and/or section joints. In almost every situation, the joints will have a water stop, mechanical seal and/or chemical seal to prevent leakage of water through the joint. Separation and deterioration can destroy the watertight integrity of the joint. Joint deterioration can result from weathering, excessive seepage, erosion or corrosion. Separation at a joint may be the result of a more serious condition such as foundation settlement, undermining, structural damage or structural instability. Deterioration at joints includes loss of gasket material, loss of joint sealant and spalling around the edges of joints. Separation of joints and loss of joint material allow seepage through the pipe. This can erode the fill underneath and along the conduit causing undermining, which can lead to the displacement of the pipe sections. Separated pipe joints can be detected by inspecting the interior of the conduit. A regular monitoring program is needed to determine the rate and severity of joint deterioration. Joint separations should be monitored to determine if movement is continuing.

CONDUIT DETERIORATION

Deterioration of conduit material is normally due to the forces of nature such as wetting and drying, freezing and thawing, oxidation, decay, ultra-violet light, cavitation and the erosive forces of water. Deterioration of pipe materials and joints can lead to seepage through and along the conduit and eventually failure of conduit systems. Additional information on deterioration can be found on the “Problems with Concrete Materials”, “Problems with Metal Materials”, and “Problems with Plastic (Polymer) Materials” fact sheets.

DIFFERENTIAL SETTLEMENT

Removal or consolidation of foundation material from around the conduit can cause differential settlement. Inadequate compaction immediately next to the conduit system during construction would compound the problem. Differential settlement can ultimately lead to undermining of the conduit system. Differential settlement should be monitored with routine inspections and documentation of observations.

MISALIGNMENT

Alignment deviations can be an indication of movement, which may or may not be in excess of design tolerances. Proper alignment is important to the structural integrity of conduit systems. Misalignment can be the direct result of internal seepage flows that have removed soil particles or dissolved soluble rock. Misalignment can also result from poor construction practices, collapse of deteriorated conduits, decay of organic material in the dam, seismic events or normal settlement due to consolidation of embankment or foundation materials. Excessive misalignment may result in other problems such as cracks, depressions, slides on the embankment, joint separation and seepage. Both the vertical and horizontal alignment of the conduit should be monitored on a regular basis.

MONITORING AND REPAIR

Frequent inspection is necessary to ensure that the pipe system is functioning properly. All conduits should be inspected thoroughly once a year. Conduits that are 24 inches or more in diameter can be entered and visually inspected with proper ventilation and confined space precautions. Small inaccessible conduits may be monitored with video cameras. The conduits should be inspected for misalignment, separated joints, loss of joint material, deformations, leaks, differential settlement and undermining. Problems with conduits occur most often at joints, and special attention should be given to them during the inspection. The joint should be checked for separation caused by misalignment or settlement and loss of joint-filler material. The outlet should be checked for signs of water seeping along the exterior surface of the conduit. Generally, this is noted by water flowing from under the conduit and/or the lack of foundation material directly beneath the conduit. The embankment surface should be monitored for depressions or sinkholes. Depressions or sinkholes on the embankment surface above the spillway conduit system develop when the underlying material is eroded and displaced. Photographs along with written records of the monitoring items performed provide invaluable information.

Effective repair of the internal surface or joint of a conduit is difficult and should not be attempted without careful planning and proper professional supervision. Various construction techniques can be applied for minor joint repair and conduit leakage, but major repairs require a plan be developed by a professional engineer experienced in dam spillway construction.

RESOURCES

ASDSO Resources
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DHS/FEMA Resources
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FEMA.gov and *DHS.gov*
(search "dam safety")

Dam Ownership
Fact Sheet

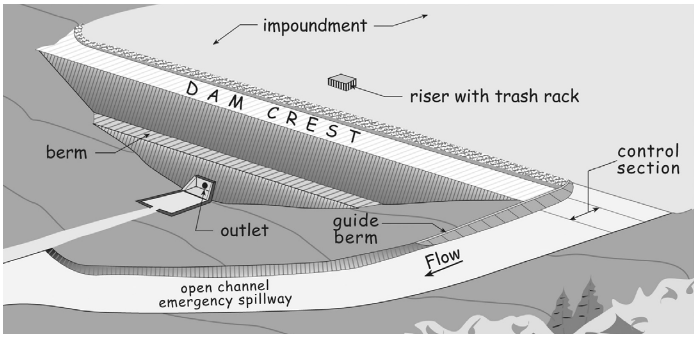
TOPIC:
OPEN CHANNEL SPILLWAYS (EARTH AND ROCK)

Open channels are often used as the emergency spillway and sometimes as the principal spillway for dams. A principal spillway is used to pass normal inflows, and an emergency spillway is designed to operate only during large flood events, usually after the capacity of the principal spillway has been exceeded.

For dams with pipe conduit principal spillways, an open channel emergency spillway is almost always required as a backup in case the pipe becomes clogged. Open channels are usually located in natural ground adjacent to the dam and can be vegetated, rocklined, or cut in rock.

DESIGN

Flow through an emergency spillway does not necessarily indicate a problem with the dam, but high velocity flows can cause severe erosion and result in a permanently lowered lake level if not repaired. Proper design of an open channel spillway will include provisions for minimizing any potential erosion. One way to minimize erosion is to design a flatter channel slope to reduce the velocity of the flow. Earthen channels can be protected by a good grass cover, an appropriately designed rock cover, concrete or various types of erosion control matting. Rock-lined channels must have adequately sized riprap to resist displacement and contain an appropriate geotextile fabric or granular filter beneath the rock. Guide berms are often required to divert flow through open channels away from the dam to prevent erosion of the embankment fill. If an open channel is used for a principal spillway, it must be rock-lined or cut in rock due to more frequent or constant flows.



Downstream View of Open Channel Spillway

Many States have requirements, based on hazard classification, for how often an earth (grass-lined) or a rock-lined emergency spillway should be used prior to maintenance procedures. It is important to check the guidelines or regulations in your State.

MAINTENANCE

Maintenance should include, but not be limited to, the following items:

Grass-covered channels should be mowed at least twice per year to maintain a good grass cover and to prevent trees, brush and weeds from becoming established. Poor vegetal cover can result in extensive and rapid erosion when the spillway flows. Repairs can be costly. Reseeding and fertilization may be necessary to maintain a vigorous growth of grass.

One suggested seed mixture is 30% Kentucky Bluegrass, 60% Kentucky 31 Fescue, and 10% Perennial Ryegrass.

Trees and brush must be removed from the channel. Tree and brush growth reduces the discharge capacity of the spillway channel. This increases the lake level during large storm events which can lead to overtopping and failure of the dam.

Erosion in the channel must be repaired quickly after it occurs. Erosion can be expected in the spillway channel during high flows, and can also occur because of rainfall and runoff, especially in areas of poor grass cover. Terraces or drainage channels may be necessary in large spillway channels where large amounts of rainfall and runoff may concentrate and have high velocities. Erosion of the side slopes may deposit material in the spillway channel, especially where the side slopes meet the channel bottom. In small spillways, this can significantly reduce the discharge capacity. This condition often occurs immediately after construction before vegetation becomes established. In these cases, it may be necessary to reshape the channel to provide the necessary capacity.

All obstructions should be kept out of the channel. Open channel spillways often are used for purposes other than passage of flood flows. Among these uses are reservoir access, parking lots, boat ramps, boat storage, pasture and cropland. Permanent structures (buildings, fences, etc.) should not be constructed in these spillways. If fences, bridges or other such structures are absolutely necessary, they should cross the spillway far enough upstream or downstream from the control section so that they do not interfere with the flow Construction of any structures in or across the channel may require prior approval from the State.

Weathering of rock channels can be a serious problem and is primarily due to freeze/thaw action. Deterioration because of sun, wind, rain, chemical action and tree root growth also occurs. Weathered rock is susceptible to erosion and displacement during high flows; therefore, rock channels are often designed with 1 to 3 feet of earth with a grass cover over the rock surface to help insulate the rock from the effects of freeze/thaw action.

MONITORING

Open channel spillways should be monitored for erosion, poor vegetal cover, growth of trees and brush, obstructions, and weathering and displacement of rock. Monitoring should take place on a regular basis and after large flood events. It is important to keep written records of observations. Photographs provide invaluable records of changing conditions. All records should be kept in the operation, maintenance, and inspection manual for the dam.

RESOURCES

ASDSO Resources
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Dam Ownership
Fact Sheet

TOPIC:

OPEN CHANNEL SPILLWAYS (CONCRETE CHUTES AND WEIRS)

Concrete chutes and weirs are used for principal spillways and emergency spillways. The principal spillway is used to pass normal flows, and the emergency spillway provides additional flow capacity during large flood events. If the principal spillway for a dam is a concrete weir and/or chute, the flow capacity may be large enough that an emergency spillway is not needed. Unlike grass-lined channel spillways that should always be located on natural ground, a concrete weir or chute may be located on the dam, but must be properly designed so that the integrity of the dam is not endangered.

The main components of a concrete chute spillway are the inlet structure, control section, discharge channel, and outlet erosion control structure. The inlet structure conveys water to the control section. The control section is the highest point in the channel and regulates the outflow from the reservoir. It is usually located on or near the crest of the dam. The control section may consist of a concrete weir or may simply be the most elevated slab in the floor of the chute. The discharge channel is located downstream of the control section and conveys flow to the outlet erosion control structure. This structure is designed to dissipate most of the erosive energy of the flow before it enters the downstream channel.

OVERALL DESIGN AND SAFETY CONSIDERATIONS

Alignment

For good hydraulic performance, abrupt changes should be avoided. This applies to sudden changes in vertical elevation of the chute floor, abrupt widening or narrowing of the chute, and sharp turns in the chute. Anything that will abruptly disrupt or change the direction of the flow in the chute will reduce flow capacity and will place more stress on the concrete. The best performance is obtained when the distribution of flow is even across the channel.

Settlement and Movement

Abnormal settlement, heaving, deflections, and lateral movement of the sidewalls or floor slabs of the spillway can occur. Movements are usually caused by a loss of underlying material, excessive settlement of the fill, or the buildup of water pressure behind or under the structure. Any abnormal settlement, heaving, deflections or lateral movement in the concrete spillway should be immediately investigated by a registered professional engineer knowledgeable about dam safety. As necessary, plans and specifications for repair to the spillway should also be promptly developed and implemented by a registered professional engineer.

The concrete sidewalls and floor of the chute must have enough strength to withstand water loads, soil/fill loads, uplift forces, weathering, and abrasion. The forces of weathering, movement of abrasive materials by water flowing in the spillway, or cavitation may cause surface defects or more serious concrete deterioration.

The freeze-thaw cycle is the most damaging weathering force acting on exposed concrete. The concrete's durability and resistance to weathering and deterioration will be determined by the concrete mix, age of the concrete, and proper sealing of the joints. Typical problems with concrete structures include scaling, spalling, honeycombing, bugholes, and popouts. Please refer to the "Problems with Concrete Materials" fact sheet for further explanation of these problems and more details about concrete durability and design. Plans and specifications for repair of structural cracks, or other structural problems, should be developed and implemented by a registered professional engineer so that the integrity of the spillway and/or embankment is not jeopardized.

Undermining

Undermining of the chute may occur at any point along its length. The chute may become undermined at the inlet and/or outlet due to an inadequate cutoff wall or erosion protection. Erosion beneath and alongside the spillway may also be caused by seepage and inadequate drainage. Undermining and erosion will lead to settlement of the undermined portions of the chute. If the concrete spillway is located on the embankment, undermining and collapse of portions of the chute will jeopardize the safety of the dam. If the spillway is located in the abutment, erosion and lowering of the lake level may result. A registered professional engineer should be hired to develop plans and specifications to repair undermining of the chute.

Cutoff Wall and Endwall

A cutoff wall should be placed at the entrance to the concrete chute to prevent the flow approaching and entering the chute from flowing beneath and undermining the floor slabs. Undermining of the chute can cause cracking and collapse of the slabs as the underlying material is eroded away. In addition, a cutoff wall is necessary at the downstream end of the chute to prevent undermining by flows exiting the chute and entering the downstream channel. The cutoff wall or endwall should be founded on bedrock or have adequate support to provide stability and prevent undermining of the wall itself.

Outlet Erosion Control Structure

The discharge at the outlet may exit the chute at a high velocity. Based on the anticipated velocity, energy, and volume of flow, a structure may be needed to protect the spillway and/or dam from erosion and undermining. Please refer to the "Outlet Erosion Control Structures" fact sheet for more detailed information.

Seepage

The rate and content of flow from weep holes and relief drains must be monitored and documented regularly. Muddy flow may indicate erosion of fill material along the spillway or piping through the embankment. The presence of soil particles or muddy flow from the drains indicates that the filter or underdrainage is not functioning properly and is allowing the migration of soil particles from the embankment. Sudden increases in flow, or muddy flow from the drains should be immediately investigated by a registered professional engineer to determine the cause and severity of the problem. Plans and specifications to properly control the seepage and repair the drain(s) and embankment should also be developed and carried out under the direction of a registered professional engineer.

In addition to monitoring the amount of flow, normal maintenance consists of removing all obstructions from drain holes and pipes to allow free drainage. Typical obstructions include debris, gravel, sediment and rodent nests. Water should not be permitted to submerge the pipe outlets for extended periods of time. This will inhibit inspection and maintenance and may cause the drains to clog. Also see the "Seepage Through Earthen Dams" fact sheet for more information.

Underdrainage and Weep Holes

Weep holes, relief drains and underdrains must be included with the concrete chute to relieve excessive water pressure or infiltration from behind the walls and floor. The drainage system for the chute should consist of correctly placed and sized drainage holes, perforated pipes, and filter and bedding materials, such as sand and gravel. Seepage can occur through the dam, along the contact between the embankment and the concrete chute, or through open joints and cracks. Uncontrolled seepage flow along the structure can erode the underlying fill material (undermining) which may cause cracking or buckling of the slabs. Excessive pressure behind the walls and floor of the chute can cause cracking and heaving of the concrete. The freeze-thaw cycle can increase the amount of stress and strain on the concrete and can also cause heaving, cracking and additional serious damage to the structure. Weep holes, relief drains, and underdrainage for a concrete chute spillway should be designed by a registered professional engineer.

RESOURCES

ASDSO Resources

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Dam Ownership
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TOPIC:
OUTLET EROSION CONTROL STRUCTURES (STILLING BASINS)

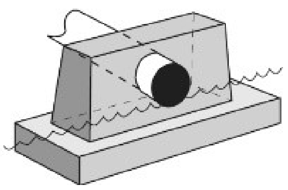
Water moving through the spillway of a dam contains a large amount of energy. This energy can cause erosion at the outlet which can lead to instability of the spillway. Failure to properly design, install, or maintain a stilling basin could lead to problems such as undermining of the spillway and erosion of the outlet channel and/or embankment material. These problems can lead to failure of the spillway and ultimately the dam. A stilling basin provides a means to absorb or dissipate the energy from the spillway discharge and protects the spillway area from erosion and undermining. An outlet erosion control structure such as a headwall/endwall, impact basin, United States Department of the Interior, Bureau of Reclamation Type II or Type III basin, baffled chute, or plunge pool is considered an energy dissipating device. The performance of these structures can be affected by the tailwater elevation. The tailwater elevation is the elevation of the water that is flowing through the natural stream channel downstream during various flow conditions.

A headwall/endwall, impact basin, Type II or Type III basin, and baffled chute are all constructed of concrete. Concrete structures can develop surface defects such as minor cracking, bugholes, honeycombing, and spalling. Concrete structures can have severe structural defects such as exposed rebar, settlement, misalignment and large cracks. Severe defects can indicate structural instability.

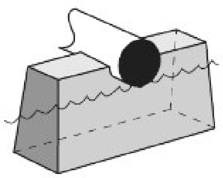
HEADWALL/ENDWALL

A headwall/endwall located at or close to the end of the discharge conduit will provide support and reduce the potential for undermining. A headwall/endwall is typically constructed of concrete, and it should be founded on bedrock or have an adequate foundation footing to provide support for stability. A headwall/endwall can become displaced if it is not adequately designed and is subject to undermining. Displacement of the headwall/endwall can lead to separation of the spillway conduit at the joints which could affect the integrity of the spillway conduit. If a concrete structure develops the structural defects mentioned in the opening paragraphs, or if the discharge spillway conduit does not have a headwall/endwall, then a registered professional engineer should be contacted to evaluate the stability of the outlet.

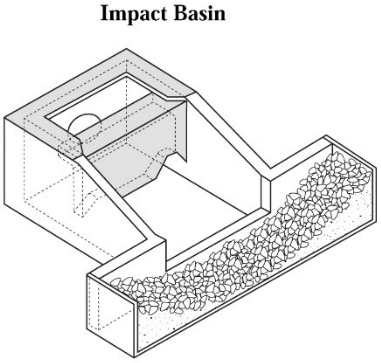
Headwall



Endwall

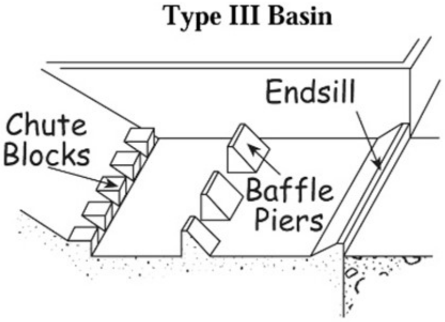


IMPACT BASIN



A concrete impact basin is an energy dissipating device located at the outlet of the spillway in which flow from the discharge conduit strikes a vertical hanging baffle. Discharge is directed upstream in vertical eddies by the horizontal portion of the baffle and by the floor before flowing over the endsill. Energy dissipation occurs as the discharge strikes the baffle, thus, performance is not dependent on tailwater. Most impact basins were designed by the United States Department of Agriculture, Natural Resources Conservation Service and the United States Department of Interior, Bureau of Reclamation. If any of the severe defects that are referenced in the opening paragraphs are observed, a registered professional engineer should be contacted to evaluate the stability of the outlet.

U.S. DEPARTMENT OF INTERIOR, BUREAU OF RECLAMATION TYPE II AND TYPE III BASINS

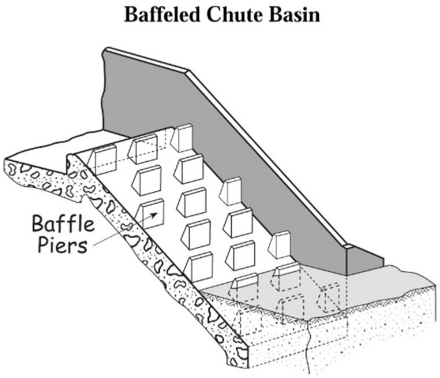


Type II and Type III basins reduce the energy of the flow discharging from the outlet of a spillway and allow the water to exit into the outlet channel at a reduced velocity. Type II energy dissipators contain chute blocks at the upstream end of the basin and a dentated (tooth-like) endsill. Baffle piers are not used in a Type II basin because of the high velocity water entering the basin. Type III energy dissipators can be used if the entrance velocity of the water is not high. They contain baffle piers which are located on the stilling basin apron downstream of the chute locks. Located at the end of both the Type II and Type III basins is an endsill. The endsill may be level or sloped, and its purpose is to create the tailwater which reduces the outflow velocity. If any of the severe defects associated with concrete structures are observed, a registered professional engineer should be contacted to evaluate the stability of the basin.

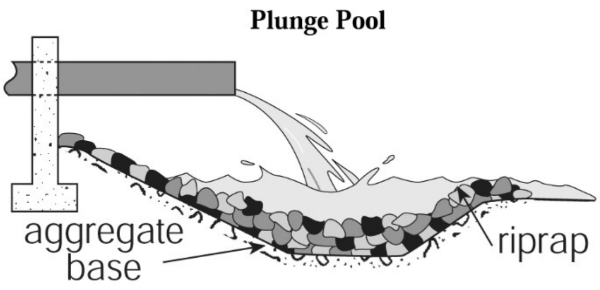
BAFFLED CHUTE

Baffled chutes require no initial tailwater to be effective and are located downstream of the control section. Multiple rows of baffle piers on the chute prevent excessive acceleration of the flow and prevent the damage that occurs from a high discharge velocity. A portion of the baffled chute usually extends below the streambed elevation to prevent undermining of the chute. If any of the severe problems associated with concrete that are referenced in the opening paragraphs are observed, a registered professional engineer should be contacted to evaluate the stability of the outlet.

PLUNGE POOL



A plunge pool is an energy dissipating device located at the outlet of a spillway. Energy is dissipated as the discharge flows into the plunge pool. Plunge pools are commonly lined with rock riprap or other material to prevent excessive erosion of the pool area. Discharge from the plunge pool should be at the natural streambed elevation. Typical problems may include movement of the riprap, loss of fines from the bedding material and scour beyond the riprap and lining. If scour beneath the outlet conduit develops, the conduit will be left unsupported and separation of the conduit joints and undermining may occur. Separation of the conduit joints and undermining may lead to failure of the spillway and ultimately the dam. A registered professional engineer should be contacted to ensure that the plunge pool is designed properly.



RESOURCES

Additional information about related topics can be found on the following fact sheets:

- Inspection of Concrete Structures
- Spillway Conduit System Problems
- Open Channel Spillways (Concrete Chutes and Weirs)
- Problems with Concrete Materials

ASDSO Resources
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DHS/FEMA Resources
DHS and FEMA make several publications and videos available to dam owners through:
FEMA.gov and *DHS.gov*
(search "dam safety")

Dam Ownership
Fact Sheet

TOPIC:
LAKE DRAINS

TYPES OF DRAINS

Common types of drains include the following:

- A valve located in the spillway riser.
- A conduit through the dam with a valve at either the upstream or downstream end of the conduit.
- A siphon system (often used to retrofit existing dams).
- A gate, valve, or stoplogs located in a drain control tower.

USES OF DRAINS

The following situations make up the primary uses of lake drains:

Emergencies

Should serious problems ever occur to threaten the immediate safety of the dam, drains may be used to lower the lake level to reduce the likelihood of dam failure. Examples of such emergencies are as follows: clogging of the spillway pipe which may lead to high lake levels and eventually dam overtopping, development of slides or cracks in the dam, severe seepage through the dam which may lead to a piping failure of the dam, and partial or total collapse of the spillway system.

Maintenance

Some repair items around the lake and dam can only be completed or are much easier to perform with a lower than normal lake level. Some examples are: slope protection repair, spillway repairs, repair and/or installation of docks and other structures along the shoreline, and dredging the lake.

Winter Drawdown

Some dam owners prefer to lower the lake level during the winter months to reduce ice damage to structures along the shoreline and to provide additional flood storage for upcoming spring rains. Several repair items are often performed during this winter drawdown period. Periodic fluctuations in the lake level also discourage muskrat and beaver habitation along the shoreline. Muskrat burrows in earthen dams can lead to costly repairs.

In addition to providing means of regulating normal pool level, typical riser structures include means to operate a reservoir drain.

COMMON MAINTENANCE PROBLEMS

Common problems often associated with the maintenance and operation of lake drains include the following:

- Deteriorated and bent control stems and stem guides.
- Deteriorated and separated conduit joints.
- Leaky and rusted control valves and sluice gates.
- Deteriorated control towers.
- Deteriorated ladders in control towers.
- Clogging of the drain conduit inlet with sediment and debris.
- Inaccessibility of the control mechanism to operate the drain.
- Seepage along the drain conduit.
- Erosion and undermining of the conduit discharge area because the conduit outlets significantly above the elevation of the streambed.
- Vandalism.
- Development of slides along the upstream slope of the dam and the shoreline caused by lowering the lake level too quickly.

OPERATION AND MAINTENANCE TIPS

All gates, valves, stems and other mechanisms should be lubricated according to the manufacturer's specifications. If you do not have a copy of the specifications and the manufacturing company cannot be determined, then a local valve distributor may be able to provide assistance.

The lake drain should be operated at least twice a year to prevent the inlet from clogging with sediment and debris and to keep all movable parts working easily. Most manufacturers recommend that gates and valves be operated at least four times per year. Frequent operation will help to ensure that the drain will be operable when it is needed. All valves and gates should be fully opened and closed at least twice to help flush out debris and to obtain a proper seal. If the gate gets stuck in a partially opened position, gradually work the gate in each direction until it becomes fully operational. Do not apply excessive torque as this could bend or break the control stem, or damage the valve or gate seat. With the drain fully open, inspect the outlet area for flow amounts, leaks, erosion and anything unusual.

All visible portions of the lake drain system should be inspected at least annually, preferably during the periodic operation of the drain. Look for and make note of any cracks, rusted and deteriorated parts, leaks, bent control stems, separated conduit joints, or unusual observations.

A properly designed lake drain should include a headwall near the outlet of the drain conduit to prevent undermining of the conduit during periods of flow. A headwall can be easily retro-fitted to an existing conduit if undermining is a problem at an existing dam. A properly designed layer of rock riprap or other slope protection will help reduce erosion in the lake drain outlet area.

Drain control valves and gates should always be placed upstream of the centerline of the dam. This allows the drain conduit to remain depressurized except during use, therefore reducing the likelihood of seepage through the conduit joints and saturation of the surrounding earth fill.

For accessibility ease, the drain control platform should be located on shore or be provided with a bridge or other structure. This becomes very important during emergency situations if high pool levels exist.

Vandalism can be a problem at any dam. If a lake drain is operated by a crank, wheel or other similar mechanism, locking with a chain or other device or off-site storage may be beneficial. Fences or other such installations may also help to ward off vandals. The recommended rate of lake drawdown is one foot or less per week, except in emergencies. Fast drawdown causes a build-up of hydrostatic pressures in the upstream slope of the dam which can lead to slope failure. Lowering the water level slowly allows these pressures to dissipate.

MONITORING

Monitoring of the lake drain system is necessary to detect problems and should be performed at least twice a year or more frequently if problems develop. Proper ventilation and confined space precautions must be considered when entering a lake drain vault or outlet pipe. Items to be considered when monitoring a lake drain system include the stem, valve, outlet pipe and related appurtenances. Monitoring for surface deterioration (rust), ease of operation, and leakage is important to maintain a working lake drain system. If the stem or valve appears to be inoperable because of deterioration or if the operability of the lake drain system is in question, because the valve does not completely close (seal) and allows an excessive amount of leakage, then a registered professional engineer or manufacturer's representative should be contacted. Photographs along with written records of the monitoring items performed provide invaluable information.

CONCLUSION

An operable lake drain accomplishes the following:

1. Makes for a safer dam by providing a method to lower the lake level in an emergency situation.
2. Allows the dam owner to have greater control of the lake level for maintenance, winter drawdown and emergency situations.
3. Meets the requirements of state dam safety laws.

RESOURCES

For further information on evaluating the condition of the lake drain systems see the "Spillway Conduit System Problems," "Problems with Metal Materials," "Problems with Plastic (Polymer) Materials," and "Problems with Concrete Materials" fact sheets.

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Dam Ownership
Fact Sheet

TOPIC:
DESIGN AND MAINTENANCE OF TRASHRACKS FOR PIPE AND RISER SPILLWAYS

The principal spillway for dams can be one of several designs. The proper operation of these spillways is an important part of maintaining the overall safety of the dam. Pipe and riser, drop inlet, and slant pipe spillways are susceptible to obstruction and damage by floating debris such as leaves, branches, and logs. One device used to ensure that these spillways operate correctly is a trashrack. Trashracks are designed to keep trash and other debris from entering the spillway and causing damage.

COMMON PROBLEMS

Trashracks usually become plugged because the openings are too small or the head loss at the inlet causes material and sediment to settle out and accumulate. Small openings will cause debris such as twigs and leaves to accumulate on the trashrack bars. This buildup will cause progressively larger debris to accumulate against the trashrack bars. Ultimately, this will result in the complete blockage of the spillway inlet.

Pipe and riser spillways can also become blocked by a buildup of debris in the spillway. This type of blockage occurs when no trashrack is in place, or if the openings are too large.

In many spillway systems, the size of the outlet conduit is smaller than the size of the inlet. Therefore, it is incorrect to assume that debris which passes through the inlet will not obstruct the flow through the outlet. Large debris, such as logs and tree limbs, can become lodged in the transitions in the spillway. This reduces the capacity of the spillway and could cause damage. An obstructed outlet pipe can be a major problem because removal of large debris from inside the spillway can be very difficult.

A partially blocked spillway reduces the capacity of the spillway and may also create a higher than normal pool level. The combination of these two factors can dramatically reduce the discharge & storage capacity of the dam. A reduction in the discharge & storage capacity of a dam increases the likelihood that the dam will be overtopped during a severe storm event. Overtopping, for even a short period of time, can cause damage to the embankment and possibly failure of the dam. If the dam has an emergency spillway, a blocked principal spillway will cause more frequent flows in the emergency spillway. Since emergency spillways are usually grass lined channels designed for infrequent flows of short duration, serious damage is likely to result.

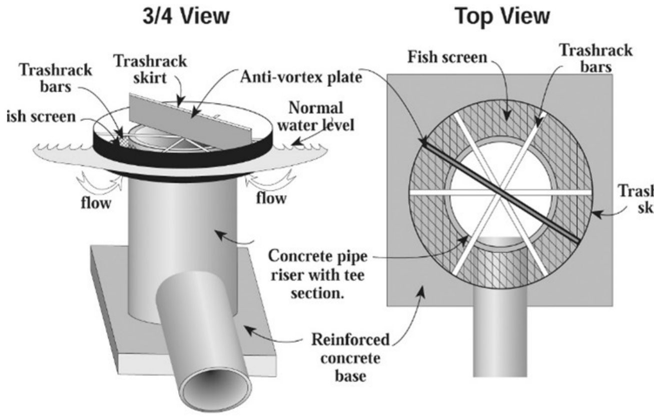
TRASHRACK DESIGN

A well-designed trashrack will stop large debris that could plug the conduit, but allow unrestricted passage of water and smaller debris. The larger the outlet conduit, the larger the trashrack opening should be. In the design of a trashrack, the openings should be sized so that they measure one-half the nominal dimension of the outlet conduit.

For example, if the outlet pipe is 18 inches in diameter, the trashrack openings should be the effective equivalent of 9 inches by 9 inches; if the outlet conduit is 3 feet by 5 feet, the trashrack openings should be the effective equivalent of 18 inches by 18 inches. This rule applies up to a maximum trashrack opening of two feet by two feet. For an outlet conduit with a nominal dimension of 12 inches or less, the trashrack openings should be at least 6 inches by 6 inches. This prevents large debris from passing through the inlet and blocking the outlet conduit while allowing smaller debris (leave, sticks, etc.) to flush through the spillway system. The trashrack should be securely fastened to the inlet. The connection must be strong enough to withstand the hydrostatic and dynamic forces exerted on the trashrack during periods of high flow.

FISH PROTECTION

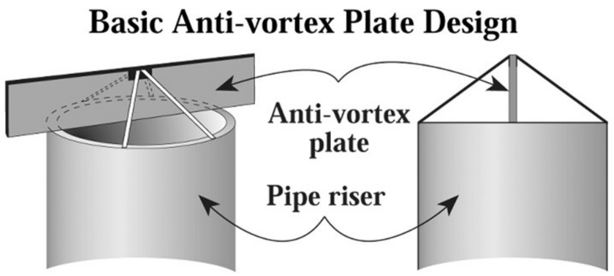
Many owners are concerned about losing fish through trashracks that have large openings. If this is a concern, a metal plate surrounding the riser or drop inlet which extends above and below the normal pool level should be installed. See Figure below. On the bottom of the plate, a metal screen should be attached and connected to the riser pipe. The solid plate at the water level will prevent the fish and floating debris from passing over the crest of the riser. The underwater screen will keep the fish from moving under the metal plate and through the spillway. The underwater screen will not become blocked because most of the debris floats on the water surface. If this design is used, the area between the inside of the cylinder and the outside of the riser must be equal to or greater than the area inside the riser.

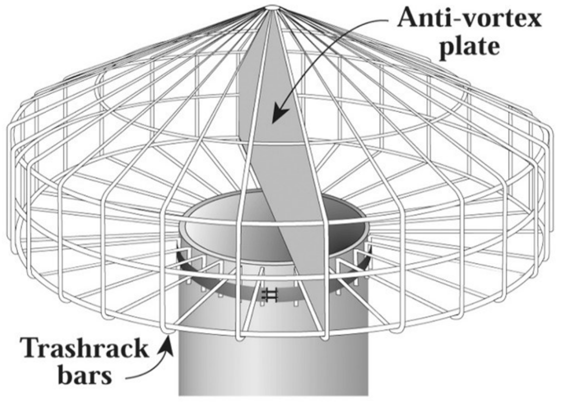


Trash Rack Design with Fish Protector Screen

ANTI-VORTEX DEVICES

An anti-vortex device can easily be incorporated into most trashrack designs. A common anti-vortex device is a flat metal plate which is placed on edge and attached to the inlet of the spillway. See Figure above. The capacity of the spillway will be increased by equipping the trashrack with an anti-vortex plate. The anti-vortex plate increases capacity by preventing the formation of a flow inhibiting vortex during periods of high flow.





MAINTENANCE

Maintenance should include periodic checks of the trashrack for rusted and broken sections and repairing as needed. Trashracks should be checked frequently during and after storm events to ensure they are functioning properly and to remove accumulated debris. Extreme caution should be used when attempting to remove accumulated debris during periods of high flow.

CONCLUSION

The benefits of a properly designed and maintained trashrack include the following:

- ✓ Efficient use of the existing spillway system that will maintain the design discharge/storage capacity of the dam and prevent overtopping.
- ✓ Prevention of costly maintenance items such as the removal of debris from the spillway, repair or replacement of damaged spillway components, and the repair of erosion in emergency spillway.
- ✓ A reduction in the amount of fish lost through the spillway system if a fish screen is used.

RESOURCES

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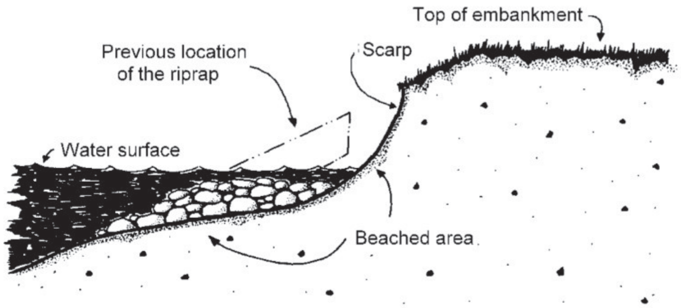
Dam Ownership
Fact Sheet

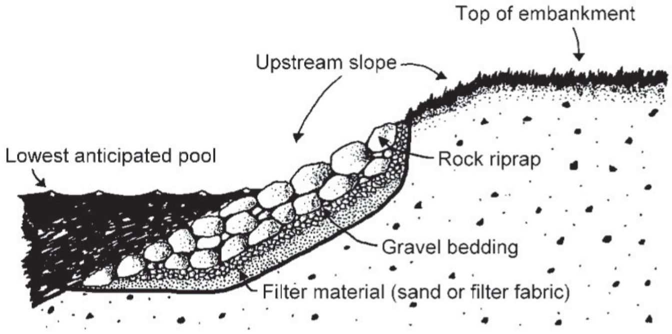
TOPIC:
UPSTREAM SLOPE PROTECTION

Slope protection is usually needed to protect the upstream slope against erosion due to wave action. Without proper slope protection, a serious erosion problem known as "beaching" can develop on the upstream slope.

The repeated action of waves upon a vegetated embankment surface over time may erode embankment material and deposit it farther down the slope, creating a "beach." The amount of erosion depends on the predominant wind direction, the orientation of the dam, the steepness of the slope, water level fluctuations, boating activities, and other factors. Further erosion can lead to cracking and sloughing of the slope which can extend into the crest, reducing its width. When erosion occurs and beaching develops on the upstream slope of a dam, repairs should be made as soon as possible. The upstream face of a dam is commonly protected against wave erosion by placement of a layer of rock riprap over a layer of bedding and a filter material. Other material such as concrete facing, soil-cement, fabri-form bags, slush grouted rocks, steel sheet piling, and articulated concrete blocks can also be used. Vegetative protection combined with a berm on the upstream slope can also be effective.



Beaching



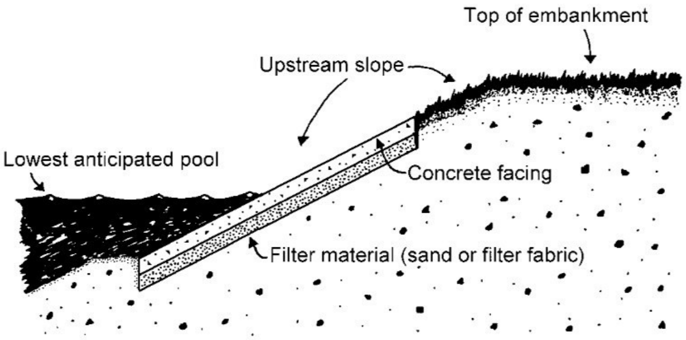
Rock Riprap

ROCK RIPRAP

Rock riprap consists of a heterogeneous mixture of irregular shaped rocks placed over gravel bedding and a sand filter or geotextile fabric. The smaller rocks help to fill the spaces between the larger pieces forming an interlocking mass. The filter prevents soil particles on the embankment surface from being washed out through the spaces (or voids) between the rocks. The maximum rock size and weight must be large enough to break up the energy of the maximum anticipated wave action and hold the smaller stones in place. If the rock size is too small, it will eventually be displaced and washed away by wave action. If the riprap is sparse or if the filter or bedding material is too small, the filter material will wash out easily, allowing the embankment material to erode. Once erosion has started, beaching will develop if remedial measures are not taken.

The dam owner should expect some deterioration (weathering) of riprap. Freezing and thawing, wetting and drying, abrasive wave action, and other natural processes will eventually break down riprap. Its useful life varies with the characteristics of the stone used. Stone for riprap should be rock that is dense, well cemented, abrasion resistant, and angular in shape to resist deterioration and create an interlocking barrier. Vegetative growth within the slope protection is undesirable because it can displace stone and disturb the filter material. Heavy undergrowth prevents an adequate inspection of the upstream slope and may hide potential problems. For additional information, see the "Trees and Brush" fact sheet.

Sufficient maintenance funds should be allocated for the addition of riprap and the removal of vegetation. Severe erosion or reoccurring problems may require a registered professional engineer to design a more effective slope protection.



Soil-Cement

SOIL-CEMENT

Soil-Cement consists of a well compacted mixture of soil, Portland cement, and water compacted to a high density. The relative proportions of soil, cement and water in the soil-cement mixture are based on the results of laboratory tests on specially prepared specimens to determine its durability and strength properties over a range soil gradations and cement contents. A soil-cement mixture of adequate durability and strength can be designed, and slope protection constructed, using almost any type of soil. Soil-cement can be placed by either the "plating" or "stair-step" method. The plating method of placement consists of one or more lifts of soil-cement placed parallel to the slope. The plating method can be considered for use on small dams where wave action is not severe. Even for small dams this method is not considered for areas where significant wave action is expected.

The stair-step method of construction consists of placing the soil-cement in horizontal lifts of 6 to 9 inches. An approximate 9-inch spread thickness results in a 6-inch compacted thickness. The width of each lift is generally 8 to 10 feet to accommodate placing and compaction equipment. See Figure. Use of pneumatic-tired rollers or steel drum rollers is the most common used compact soil-cement in the stair-stop method.

RCC (roller compacted concrete) has also been used as slope protection and is designed and installed similar to soil cement. Consideration should be given that other methods maybe more appropriate in freeze-thaw regions of the country. *Bureau of Reclamation Design Standard No. 13, Embankment Dams, Chapter 17: Soil Cement Slope Protection* and the Portland Cement Association Soil -Cement Construction Handbook are good references for design engineers.

INSPECTION AND MONITORING

Regular inspection and monitoring of upstream slope protection is essential to detect any problems. It is important to keep written records of the location and extent of any erosion, undermining, or deterioration of the riprap, wave berm or other slope protection. Photographs provide invaluable records of changing conditions. A rapidly changing condition may indicate a very serious problem, and appropriate dam safety officials should be contacted. All records should be kept in the operation, maintenance, and inspection manual for the dam.

RESOURCES

The ASDSO website houses guidelines on dams. Go to:
DamSafety.Org/ManualsandGuidelines

For more information, videos and tools for dam owners go to:
DamOwner.Org

Natural Resources Conservation Service Technical Releases can be found at:
https://directives.sc.egov.usda.gov/

Access your state's Dam Safety Program by clicking your state at:
DamSafety.Org/States

Dam Ownership **Fact Sheet**

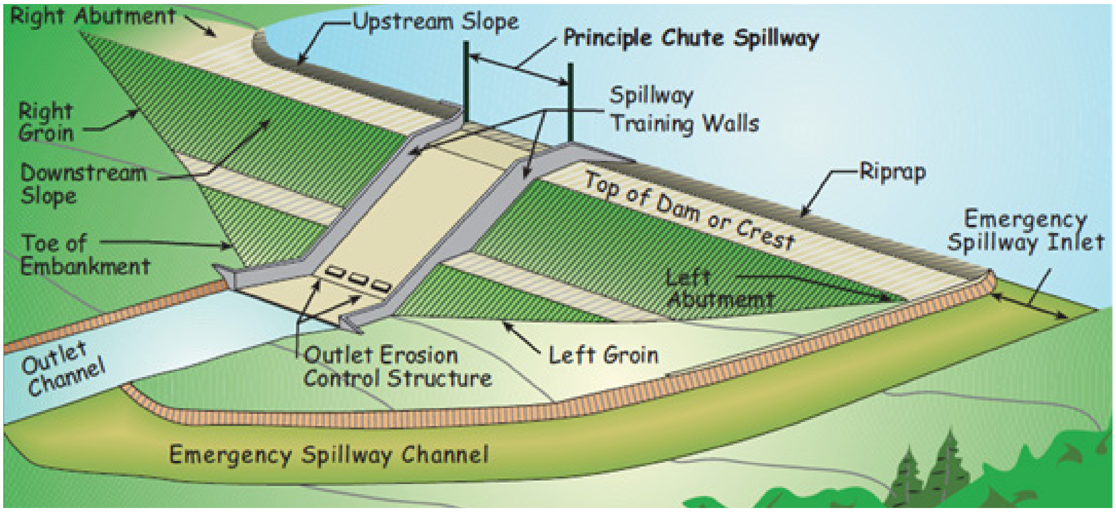
TOPIC:

COMMON PROBLEMS FOR SMALL DAMS WITH CONCRETE CHANNEL SPILLWAYS

The State Dam Safety Program has inspection requirements for state regulated dams. A dam, like any man-made structure, will change and deteriorate over time. Keeping a dam in good condition will allow better inspections and easier maintenance. Proper inspection and maintenance will help prevent small problems from turning into larger, more costly repairs. The following paragraphs and pictures address common problems that have been noted during inspections.

EARTHEN EMBANKMENTS

The establishment and control of proper vegetation is an important part of dam maintenance. Properly maintained vegetation can help prevent erosion of embankment and earth channel surfaces, and aid in the control of groundhogs and muskrats. Embankment slopes are normally designed and constructed so that the surface drainage will be spread out in a thin layer as “sheet flow” over the grass cover. When the sod is in poor condition or flow is concentrated at one or more locations, the resulting erosion will leave rills and gullies in the embankment slope.



A dam safely passes a flood event by a combination of storing water in the lake and passing water through its spillways. Earthen embankments are not designed to have floodwaters overtop them. An emergency spillway should not pass over the crest of the dam; it should be located in the abutment area.

TREES AND BRUSH

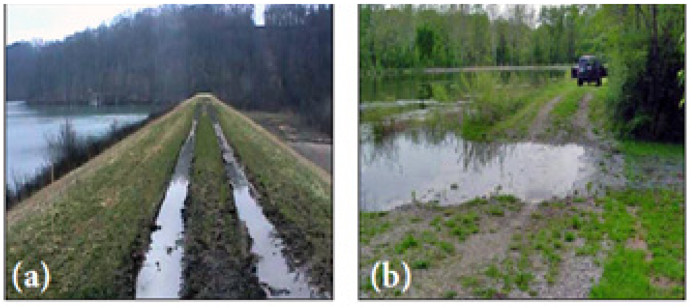
Trees and brush must not be permitted on embankment surfaces or in vegetated earth spillways. Extensive root systems can provide seepage paths for water. Trees that blow down or fall can leave large holes in the embankment. Brush hinders visual inspection, provides a haven for burrowing animals, and retards growth of grass vegetation.

UPSTREAM SLOPE

Slope protection may be needed to protect the upstream slope against erosion. Erosion can lead to cracking and sloughing, which can extend into the crest. Muskrats and groundhogs can also damage the slope. The upstream face of the dam is commonly protected against wave erosion by placement of a layer of rock riprap over a layer of bedding and a filter material.

An excellent grass cover will reduce erosion and is easily maintained.

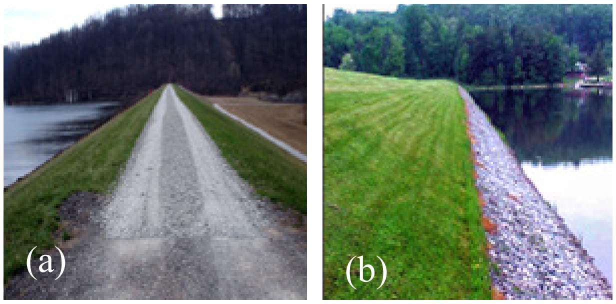
(a) Embankments covered with trees and brush makes inspections difficult. (b) Cleared of trees and brush, this embankment is much easier to inspect, but needs a good vegetal cover.



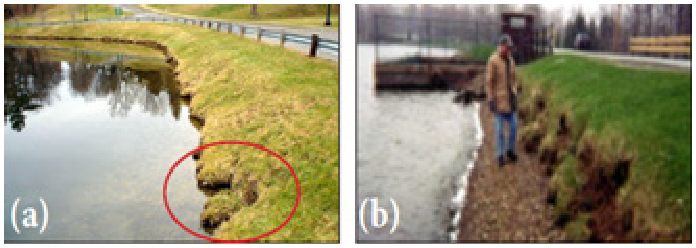
(a) Embankment crests with vehicle ruts will collect water and weaken the embankment. (b) Crest beginning to overtop in low area. This could lead to erosion and failure of the dam.

CREST

Vehicular traffic should be discouraged, especially during wet conditions, to avoid ruts. Water collected in ruts may cause localized saturation, thereby weakening the embankment. Ruts can develop into low areas. Low areas on the crest increase the likelihood that a dam will be overtopped during severe floods. Earthen embankments are not designed to be overtopped. Should the dam overtop, floodwaters will concentrate in the low area, increasing the likelihood of erosion of the crest and downstream slope. Severe erosion can lead to failure of the embankment. A well-vegetated earth embankment may withstand limited overtopping if its crest is level and water flows over the crest and downstream slope as an evenly distributed sheet without becoming concentrated.



(a) Crest with a good cover that will reduce the effects of vehicular traffic.
(b) This slope is well maintained with rock riprap along the shoreline.



(a) Collapsed muskrat burrows increase shoreline erosion and sloughing. (b) This shoreline has eroded due to lack of proper erosion protection leading to sloughing and cracking of the slope.

(a) A spillway outlet area as originally constructed. (b) Severe erosion has undermined the spillway outlet and threatens the overall structural safety of the spillways.

CONCRETE SPILLWAYS

A concrete weir or chute is often used as a principal spillway for dams. The principal spillway is the first spillway to experience flow after a storm when the pool rises above the normal pool level. For the spillway to be effective, it must be clear of obstructions, in good structural condition, and on a solid foundation. A spillway must remain unobstructed to maintain its flow capacity. Obstructions such as fish screens, walkways, vegetation, and bridge piers should be cleared from the spillway inlet. Loss of flow capacity could cause the dam to overtop and fail. The spillway must remain in good structural condition to ensure that spillway flow stays within the spillway and does not cause erosion that could cause the spillway to fail. Concrete surfaces should be visually examined for structural problems due to weathering, stress, chemical attack, erosion, and other destructive forces. Structural problems are indicated by cracking, exposure of reinforcing bars, and large areas of spalled concrete. Even if the spillway is in good structural condition, seepage under the spillway or erosion at the outlet or along the sides can cause the spillway to fail. Spillway floor slabs and walls should be checked for erosion of underlying base material known as undermining. Indicators of problems with seepage and erosion under the spillway include misalignment at joints and large cracks.

EMERGENCY AKA AUXILIARY SPILLWAYS

The emergency or auxiliary spillway is the second spillway to experience flow during a flood event. For many common dams, the emergency spillway consists of a grass-lined, earthen open channel. An open channel can convey much more flow than a pipe spillway, so it is important to keep the spillway free of obstructions. Obstructions reduce the flow capacity and could cause the dam to overtop and fail. Permanent structures including buildings, fences, and roadway embankments for access across the spillway should not be constructed in the spillway. Earthen channels should be protected by a good grass cover, an appropriately designed rock cover, concrete, or other various types of erosion control matting. Grass-lined channels should be mowed at least twice per year to maintain a good grass cover and to prevent trees, brush, and weeds from becoming established. Poor vegetal cover can result in extensive and rapid erosion when the spillway flows.

Left: A tree is obstructing this spillway.
Right: Notice the landscaping directly behind the table. Obstructions in the spillway reduce the capacity to convey water. They also can collect debris, further diminishing the capacity of the spillway.

This bridge blocks much of the spillway, reducing its ability to convey water.

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This open-channel emergency spillway is clear of trees, brush and other obstructions. Also note the good grass cover.

Dam Ownership **Fact Sheet**

TOPIC:
INSPECTION OF CONCRETE STRUCTURES

Dams, dikes, and levees must not be thought of as part of the natural landscape, but as manmade structures which must be designed, inspected, operated, and maintained accordingly. Routine maintenance and inspection of dams and appurtenant facilities should be an ongoing and active process to ensure that structural failures do not occur which can threaten the overall safety of the dam. The information provided in this fact sheet pertains entirely to the inspection of concrete structures used at dams. The intention is to help dam owners become more aware of common problems that are typically encountered with concrete so that they can more readily address the seriousness of a condition whenever it arises.

STRUCTURAL INSPECTIONS

Concrete surfaces should be visually examined on a periodic basis for spalling and deterioration due to weathering, unusual or extreme stresses, erosion, cavitation, vandalism, and other destructive forces. Structural problems are indicated by cracking, exposure of reinforcing bars, large areas of broken-out concrete, misalignment at joints, undermining and settlement in the structure. Rust stains that are noted on the concrete may indicate that internal corrosion and deterioration of reinforcement steel is occurring. Spillway floor slabs and upstream slope protection slabs should be checked for erosion of underlying base material otherwise known as undermining. Concrete walls and tower structures should be examined to determine if settlement and misalignment of construction joints has occurred.

WHAT TO LOOK FOR

Cracking

Concrete structures can exhibit many different types of cracking. Deep, wide cracking is due to stresses which are primarily caused by shrinkage and structural loads. Minor or hairline surface cracking is caused by weathering and the quality of the concrete that was applied. The results of this minor cracking can be the eventual loss of concrete, which exposes reinforcing steel and accelerates deterioration. Generally, minor surface cracking does not affect the structural integrity and performance of the concrete structure. Cracks through concrete surfaces exposed to flowing water may lead to the erosion or piping of embankment or foundation soils from around and/or under the concrete structure. In this case, the cracks are not the result of a problem but are the detrimental condition which leads to piping and erosion.

Example of cracking.

Structural cracking of concrete is usually identified by long, single or multiple diagonal cracks with accompanying displacements and misalignment. Cracks extending across concrete slabs which line open channel spillways or provide upstream slope wave protection can indicate a loss of foundation support resulting from settlement, piping, undermining, or erosion of foundation soils. Piping and erosion of foundation soils are the result of inadequate underdrainage and/or cutoff walls. Items to consider when evaluating a suspected structural crack are the concrete thickness, the size and location of the reinforcing steel, the type of foundation, and the drainage provision for the structure.

Seepage

Seepage at the discharge end of a spillway or outlet structure may indicate leakage of water through a crack. Proper underdrainage for open channel spillways with structural concrete floors is necessary to control this leakage. Flows from underdrain outlets and pressure relief holes should also be observed and measured. Cloudy flows may indicate that piping is occurring beneath or adjacent to the concrete structure. This could be detrimental to the foundation support.
Concrete surfaces adjacent to contraction joints and subject to flowing water are of special concern especially in chute slabs. The adjacent slabs must be flush or the downstream one slightly lower to prevent erosion of the concrete and to prevent water from being directed into the joint during high velocity flow.

Poor Drainage

All weep holes should be checked for the accumulation of silt and granular deposits at their outlets. These deposits may obstruct flow or indicate loss of support material behind the concrete surfaces. Tapping the concrete surface with a hammer or some other device will help locate voids if they are present as well as give an indication of the condition and soundness of the concrete. Weep holes in the concrete are used to allow free drainage and relieve excessive hydrostatic pressures from building up underneath the structure. Excessive hydrostatic pressures underneath the concrete could cause it to heave or crack which increases the potential for accelerated deterioration and undermining. Periodic monitoring of the weep hole drains should be performed and documented on a regular and routine basis to ensure that they are functioning as designed.

Inspection of intake structures, trash racks, upstream conduits, and stilling basin concrete surfaces that are below the water surface is not readily feasible during a regularly scheduled inspection. Typically, stilling basins require the most regular monitoring and major maintenance because they are holding ponds for rock and debris, which can cause extensive damage to the concrete surfaces during the dissipation of flowing water. Therefore, special inspections of these features should be performed at least once every five years by either dewatering the structure or when operating conditions permit. Investigation of these features using experienced divers is also an alternative.

Weep holes with screens to control seepage.

PREPARING FOR AN INSPECTION

Before an inspection of the dam's concrete facilities is performed, it is recommended that a checklist be developed that includes all the different components of the spillway and/or outlet works. The checklist should also include a space for logging any specific observations about the structure and the state of its condition. Photographs provide invaluable records of changing conditions. A rapidly changing condition may indicate a very serious problem and documentation of prior inspections is very helpful in making this determination If there are any questions as to the seriousness of an observation the state dam safety agency, or a registered professional engineer experienced with dams, should be contacted.

RESOURCES

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TOPIC:
PROBLEMS WITH CONCRETE MATERIALS

Visual inspection of concrete will allow for the detection of distressed or deteriorated areas. Problems with concrete include construction errors, disintegration, scaling, cracking, efflorescence, erosion, spalling, and popouts.

CONSTRUCTION ERRORS

Errors made during construction can include adding improper amounts of water to the concrete mix, inadequate consolidation, and improper curing can cause distress and deterioration of the concrete. Proper mix design, placement, and curing of the concrete, as well as an experienced contractor are essential to prevent construction errors from occurring. Construction errors can lead to some of the problems discussed later in this fact sheet such as scaling and cracking. Honeycombing and bugholes can be observed after construction.

Honeycombing can be recognized by exposed coarse aggregate on the surface without any mortar covering or surrounding the aggregate particles. The honeycombing may extend deep into the concrete. Honeycombing can be caused by a poorly graded concrete mix, by too large of a coarse aggregate, or by insufficient vibration at the time of placement. Honeycombing will result in further deterioration of the concrete due to freeze-thaw cycles because moisture can easily work its way into the honeycombed areas. Severe honeycombing should be repaired to prevent further deterioration of the concrete surface.

Bugholes is a term used to describe small holes (less than about 0.25 inch in diameter) that are noticeable on the surface of the concrete. Bugholes are generally caused by too much sand in the mix, a mix that is too lean or excessive amplitude of vibration during placement. Bugholes may cause durability problems with the concrete and should be monitored.

DISINTEGRATION AND SCALING

Disintegration can be described as the deterioration of the concrete into small fragments and individual aggregates. *Scaling* is a milder form of disintegration where the surface mortar flakes off. Large areas of crumbling (rotten) concrete, areas of deterioration which are more than about 3 to 4 inches deep (depending on the wall/slab thickness), and exposed rebar indicate serious concrete deterioration. If not repaired, this type of concrete deterioration may lead to structural instability of the concrete structure. A registered professional engineer must prepare plans and specifications for repair of serious concrete deterioration. For additional information, see the “Concrete Repair Techniques” fact sheet.

Disintegration can be a result of many causes such as freezing and thawing, chemical attack, and poor construction practices. All exposed concrete is subject to freeze-thaw cycles, but the concrete’s resistance to weathering is generally determined by the concrete mix and the age of the concrete.

Concrete with the proper amounts of air, water, and cement, and a properly sized aggregate, will be much more durable. In addition, proper drainage is essential in preventing freeze-thaw damage. When critically saturated concrete (when 90% of the pore space in the concrete is filled with water) is exposed to freezing temperatures, the water in the pore spaces within the concrete freezes and expands, damaging the concrete. Repeated cycles of freezing and thawing will result in surface scaling and can lead to disintegration of the concrete. Hydraulic structures are especially susceptible to freeze-thaw damage since they are more likely to be critically saturated. Older structures are also more susceptible to freeze-thaw damage since the concrete was not air entrained. In addition, acidic substances in the surrounding soil and water can cause disintegration of the concrete surface due to a reaction between the acid and the hydrated cement.

CRACKS

Cracks in the concrete may be structural or surface cracks. Surface cracks are generally less than a few millimeters wide and deep. These are often called hairline cracks and may consist of single, thin cracks, or cracks in a craze/map-like pattern. A small number of surface or shrinkage cracks is common and does not usually cause any problems. *Surface cracks* can be caused by freeze-thaw cycles, poor construction practices, and alkali-aggregate reactivity. Alkali-aggregate reactivity occurs when the aggregate reacts with the cement causing crazing or map cracks. The placement of new concrete over old may also cause surface cracks to develop. This occurs because the new concrete will shrink as it cures. Surface cracks in the spillway should be monitored and will need to be repaired if they deteriorate further. Structural cracks in the concrete are usually larger than 0.25 inch in width. They extend deeper into the concrete and may extend all the way through a wall, slab, or other structural member. *Structural cracks* are often caused by settlement of the fill material supporting the concrete structure, or by loss of the fill support due to erosion. The structural cracks may worsen in severity due to the forces of weathering. A registered professional engineer knowledgeable about dam safety should investigate the cause of structural cracks and prepare plans and specifications for repair of any structural cracks.

EFFLORESCENCE

A white, crystallized substance, known as efflorescence, may sometimes be noted on concrete surfaces, especially spillway sidewalls. It is usually noted near hairline or thin cracks. Efflorescence is formed by water seeping through the pores or thin cracks in the concrete. When the water evaporates, it leaves behind some minerals that have been leached from the soil, fill, or concrete. Efflorescence is typically not a structural problem. Efflorescence should be monitored because it can indicate the amount of seepage finding its way through thin cracks in the concrete and can signal areas where problems (i.e. inadequate drainage behind the wall or deterioration of concrete) could develop. Also, water seeping through thin cracks in the wall will make the concrete more susceptible to deterioration due to freezing and thawing of the water.

EROSION

Erosion due to *abrasion* results in a worn concrete surface. It is caused by the rubbing and grinding of aggregate or other debris on the concrete surface of a spillway channel or stilling basin. Minor erosion is not a problem but severe erosion can jeopardize the structural integrity of the concrete. A registered professional engineer should prepare plans and specifications for repair of this type of erosion if it is severe.

Erosion due to cavitation results in a rough, pitted concrete surface. Cavitation is a process in which subatmospheric pressures, turbulent flow and impact energy are created and will damage the concrete. If the shape of the upper curve on the ogee spillway is not designed close to its ideal shape, cavitation may occur just below the upper curve, causing erosion. A professional engineer should prepare plans and specifications for repair of this type of erosion if the concrete becomes severely pitted which could lead to structural damage or failure.

SPALLING AND POPOUTS

Spalling is the loss of larger pieces or flakes of concrete. It is typically caused by sudden impact of something dropped on the concrete or stress in the concrete that exceeded the design. Spalling may occur on a smaller scale, creating popouts. Popouts are formed as the water in saturated coarse aggregate particles near the surface freezes, expands, and pushes off the top of the aggregate and surrounding mortar to create a shallow conical depression. Popouts are typically not a structural problem. However, if a spall is large and causes structural damage, a registered professional engineer should prepare plans and specifications to repair the spalling.

INSPECTION AND MONITORING

Regular inspection and monitoring is essential to detect problems with concrete materials. Concrete structures should be inspected a minimum of once per year and after any significant weather event. The inspector should also look at the interior condition of concrete spillway conduit. Proper ventilation and confined space precautions must be considered when entering a conduit.
It is important to keep written records of the dimensions and extent of scaling, disintegration, efflorescence, honeycombing, erosion, spalling, popouts, and the length and width of cracks. Structural cracks should be monitored more frequently and repaired if they are a threat to the stability of the structure or dam. Photographs provide invaluable records of changing conditions. A rapidly changing condition may indicate a very serious problem, and the State Dam Safety Agency should be contacted immediately. All records should be kept in the operation, maintenance, and inspection manual for the dam.

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TOPIC:
CONCRETE REPAIR TECHNIQUES

Concrete is an inexpensive, durable, strong and basic building material often used in dams for core walls, spillways, stilling basins, control towers, and slope protection. However, poor workmanship, construction procedures, and construction materials may cause imperfections that later require repair. Any long-term deterioration or damage to concrete structures caused by flowing water, ice, or other natural forces must be corrected. Neglecting to perform periodic maintenance and repairs to concrete structures as they occur could result in failure of the structure from either a structural or hydraulic standpoint. This in turn may threaten the continued safe operation and use of the dam.

CONSIDERATIONS

Floor or wall movement, extensive cracking, improper alignments, settlement, joint displacement, and extensive undermining are signs of major structural problems. In situations where concrete replacement solutions are required to repair deteriorated concrete, it is recommended that a registered professional engineer be retained to perform an inspection to assess the concrete's overall condition and determine the extent of any structural damage and necessary remedial measures.

Typically, it is found that drainage systems are needed to relieve excessive water pressures under floors and behind walls. In addition, reinforcing steel must also be properly designed to handle tension zones and shear and bending forces in structural concrete produced by any external loading (including the weight of the structure). Therefore, the finished product in any concrete repair procedure should consist of a structure that is durable and able to withstand the effects of service conditions such as weathering, chemical action, and wear. Major structural repairs that require professional advice are not addressed here.

REPAIR METHODS

Before any type of concrete repair is attempted, it is essential that all factors governing the deterioration or failure of the concrete structure are identified. This is required so that the appropriate remedial measures can be undertaken in the repair design to help correct the problem and prevent it from occurring in the future. The following techniques require expert and experienced assistance for the best results. The method of repair will depend on the size of the job and the type of repair required

1. The Dry-Pack Method: The dry-pack method can be used on small holes in new concrete which have a depth equal to or greater than the surface diameter. Preparation of a dry-pack mix typically consists of about 1-part portland cement and 2-1/2-parts sand to be mixed with water. You then add enough water to produce a mortar that will stick together. Once the desired consistency is reached, the mortar is ready to be packed into the hole using thin layers.

2. Concrete Replacement: Concrete replacement is required when one-half to one square foot areas or larger extend entirely through the concrete sections or where the depth of damaged concrete exceeds 6 inches. When this occurs, normal concrete placement methods should be used. Repair will be more effective if tied in with existing reinforcing steel (rebar). This type of repair will require the assistance of a professional engineer experienced in concrete construction.
3. Replacement of Unformed Concrete: The replacement of damaged or deteriorated areas in horizontal slabs involves no special procedures other than those used in good construction practices for placement of new slabs. Repair work can be bonded to old concrete by use of a bond coat made of equal amounts of sand and cement. It should have the consistency of whipped cream and should be applied immediately ahead of concrete placement so that it will not set or dry out. Latex emulsions with portland cement and epoxy resins are also used as bonding coats.
4. Preplaced Aggregate Concrete: This special commercial technique has been used for massive repairs, particularly for underwater repairs of piers and abutments. The process consists of the following procedures: 1) Removing the deteriorated concrete, 2) forming the sections to be repaired, 3) prepacking the repair area with coarse aggregate, and 4) pressure grouting the voids between the aggregate particles with a cement or sand-cement mortar.
5. Synthetic Patches: One of the most recent developments in concrete repair has been the use of synthetic materials for bonding and patching. Epoxy-resin compounds are used extensively because of their high bonding properties and great strength. In applying epoxy-resin patching R 07/08/99 mortars, a bonding coat of the epoxy resin is thoroughly brushed onto the base of the old concrete. The mortar is then immediately applied and troweled to the elevation of the surrounding material.

Before attempting to repair a deteriorated concrete surface, all unsound concrete should be removed by sawing or chipping and the patch area thoroughly cleaned. A sawed edge is superior to a chipped edge, and sawing is generally less costly than mechanical chipping. Before concrete is ordered for placing, adequate inspection should be performed to ensure that (1) foundations are properly prepared and ready to receive the concrete, (2) construction joints are clean and free from defective concrete, (3) forms are grout-tight, amply strong, and set to their true alignment and grade, (4) all reinforcement steel and embedded parts are clean, in their correct position, and securely held in place, and (5) adequate concrete delivery equipment and facilities are on the job, ready to go, and capable of completing the placement without addition unplanned construction.

CONCRETE USE GUIDELINES

In addition to its strength characteristics, concrete must also have the properties of workability and durability. Workability can be defined as the ease with which a given set of materials can be mixed into concrete and subsequently handled, transported, and placed with a minimal loss of homogeneity. The degree of workability required for proper placement and consolidation of concrete is governed by the dimensions and shape of the structure and by the spacing and size of the reinforcement. The concrete, when properly placed, will be free of segregation, and its mortar is intimately in contact with the coarse aggregate, the reinforcement, and/or any other embedded parts or surfaces within the concrete. Separation of coarse aggregate from the mortar should be minimized by avoiding or controlling the lateral movement of concrete during handling and placing operations. The concrete should be deposited as nearly as practicable in its final position. Placing methods that cause the concrete to flow in the forms should be avoided. The concrete should be placed in horizontal layers, and each layer should be thoroughly vibrated to obtain proper compaction.

All concrete repairs must be adequately moist-cured to be effective. The bond strength of new concrete to old concrete develops much more slowly, and the tendency to shrink and loosen is reduced by a long moist-curing period. In general, the concrete repair procedures discussed above should be considered on a relative basis and in terms of the quality of concrete that one wishes to achieve for their construction purpose. In addition to being adequately designed, a structure must also be properly constructed with concrete that is strong enough to carry the design loads, durable enough to withstand the forces associated with weathering, and yet economical, not only in first cost, but in terms of its ultimate service. It should be emphasized that major structural repairs to concrete should not be attempted by the owner or persons not experienced in concrete repairs. A qualified professional engineer experienced in concrete construction should be obtained for the design of large scale repair projects.

CRACK REPAIR

The two main objectives when repairing cracks in concrete are structural bonding and stopping water flow. For a structural bond, epoxy injection can be used. This process can be very expensive since a skilled contractor is needed for proper installation. The epoxy is injected into the concrete under pressure, welding the cracks to form a monolithic structure. This method of repair should not be considered if the crack is still active (moving). For a watertight seal, a urethane sealant can be used. This repair technique does not form a structural bond; however, it can be used on cracks that are still active. Cracks should be opened using a concrete saw or hand tool prior to placing the sealant. A minimum opening of 1/4 inch is recommended since small openings are hard to fill. Urethane sealants can be reapplied since they are flexible materials and will adhere to older applications. All of the factors causing cracking must be identified and addressed before repairing the concrete to prevent the reoccurrence of cracks.

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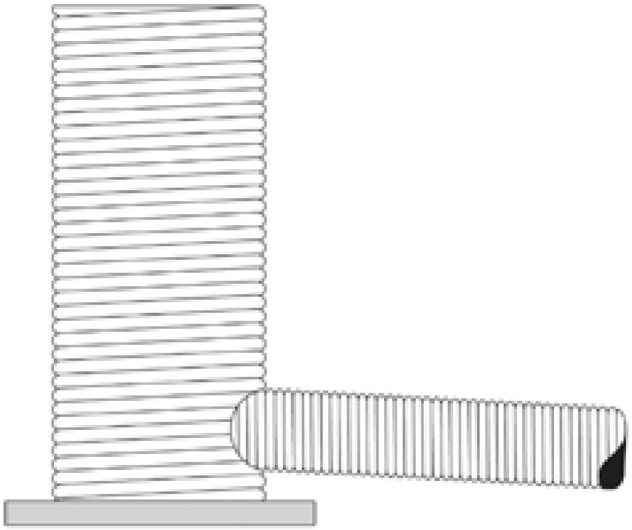
TOPIC:

PROBLEMS WITH METAL MATERIALS

Corrosion is a common problem for spillway conduits and other metal appurtenances. Corrosion is the deterioration or breakdown of metal because of a reaction with its environment. Exposure to moisture, acidic conditions, or salt will accelerate the corrosion process. Acid runoff from strip-mined areas will cause rapid corrosion of metal conduits. In these areas, conduits made of less corrodible materials such as concrete or plastic should be used. Soil types also factor into the amount of corrosion. Clayey soils can be more corrosive than sandy soils since they are poorly drained and poorly aerated. Silts are somewhere in between clays and sands. Some examples of metal conduits include ductile iron, smooth steel, and corrugated metal.



Example of a corrugated metal pipe and riser spillway.

Corrugated metal pipe is <u>not recommended</u> for use in dams since the service life for corrugated metal is only 25 to 30 years, whereas the life expectancy for dams is much longer.

In areas of acidic water, the service life can be much less. Therefore, corrugated metal spillway conduits typically need to be repaired or replaced early in the dam's design life, which can be very expensive.

Conduit coating is an effective way of controlling corrosion of metal conduits if used properly. It is relatively inexpensive and extends the life of the conduit. Some examples of coatings include cement-mortar, epoxy, aluminum, or polyethylene film. Asphalt (bituminous) coatings are not recommended since their service life is usually only one or two years. Coatings must be applied to the conduit prior to installation and protected to ensure that the coating is not scratched off. Coatings applied to conduits in service are generally not very effective because of the difficulty in establishing an adequate bond.

Corrosion can also be controlled or arrested by installing cathodic protection. A metallic anode such as magnesium (or zinc) is buried in the soil and is connected to the metal conduit by wire. Natural voltage current flowing from the magnesium (anode) to the conduit (cathode) will cause the magnesium to corrode and not the conduit. However, sufficient maintenance funds should be allocated for the regular inspection of this active system.

If corrosion is allowed to continue, metal conduits will rust out. The spillway must be repaired before water flows through the rusted-out portion of the conduit and erodes the fill material of the embankment. Continued erosion can lead to failure of the dam. Sliplining can be an economical and effective method of permanently restoring deteriorated spillways. During sliplining, a smaller diameter pipe is inserted into the old spillway conduit and then grout is used to fill in the void between the two pipes. If sliplining the spillway is not feasible, the lake may need to be drained and a new spillway must be installed. A registered professional engineer should be retained to develop and submit plans and specifications for any major modifications such as spillway sliplining or replacement.

Corrosion of the metal parts of the operating mechanisms such as lake drain valves and sluice gates can be effectively treated by keeping these parts lubricated and /or painted. If the device has not been operated in several years, a qualified person (i.e. manufacturer's representative or registered professional engineer) should inspect it to determine its operability. Caution must be used to prevent the mechanism from breaking. A registered professional engineer may be needed to prepare plans and specifications for repair if the device is determined to be inoperable.
Regular inspection and monitoring is essential to detect any problems with metal materials. Coatings on metal pipes should be inspected for scratched and worn areas. The inspector should also look for corrosion inside the spillway conduit. Proper ventilation and confined space precautions must be considered when entering the spillway conduit system. If using cathodic protection, regular inspections are required to verify that the system is working properly. It is important to keep written records of the amount of surface rust, pitting, and corrosion on any metal surface. Areas of thin metal should be monitored more frequently and repaired or replaced if they rust out. Photographs provide invaluable records of changing conditions. A rapidly changing condition may indicate a very serious problem, and the State Dam Safety Agency should be contacted immediately. All records should be kept in the operation, maintenance, and inspection manual for the dam.

RESOURCES

ASDSO Resources
The ASDSO website houses national guidelines on dams. Go to:
DamSafety.Org/ManualsandGuidelines

For more information, videos and tools for dam owners go to:
DamOwner.Org

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Access your state's Dam Safety Program by clicking your state at:
DamSafety.Org/States

Dam Ownership
Fact Sheet

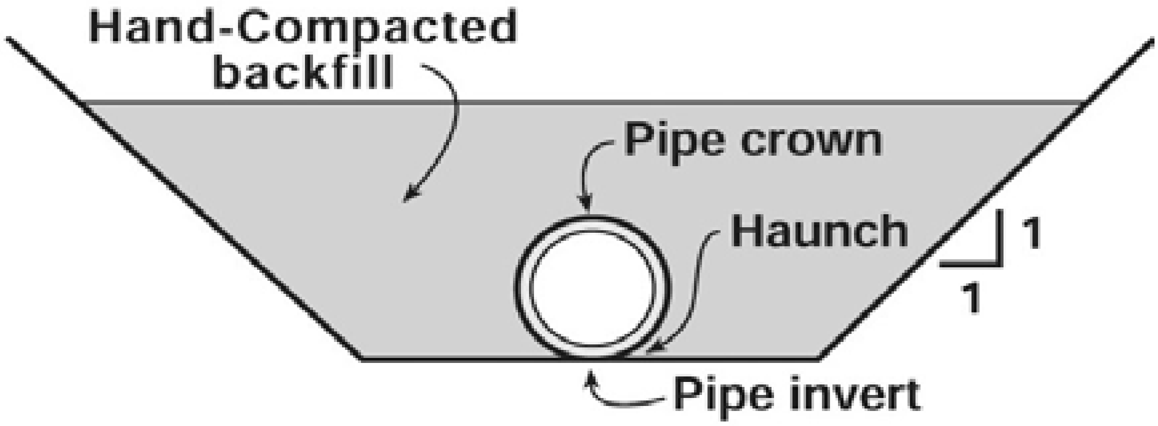
TOPIC:

PROBLEMS WITH PLASTIC (POLYMER) MATERIALS

Plastics are often used as spillway and lake drain pipes in dam construction and repair. The most common plastic pipes are high-density polyethylene (HDPE) and polyvinyl chloride (PVC). The advantages of using plastic pipe include excellent abrasion resistance, chemical corrosion resistance, low maintenance, and long-life expectancy. Naturally occurring chemicals in soils will not degrade plastic pipe and cause it to rot or corrode. Plastic pipes are also much easier to handle and install compared to heavier concrete and steel pipes.

Plastic pipes are considered flexible, and they get their strength from the material and the surrounding backfill whereas rigid pipes, such as concrete, get their strength from the material and the pipe structure. Backfill around plastic pipes must be properly compacted and in full contact with the pipe. It is important to take special care in the haunch area to prevent the pipe from lifting off the subgrade and disrupting vertical alignment.

Symmetric backfilling is also required to prevent the pipe from being out of lateral alignment. When designing a new spillway system, a registered professional engineer will be required to specify the correct type of pressurized plastic pipe that can be used. The pipe must be able to withstand the pressures from the weight of the embankment without crushing or buckling. The joints must also be watertight. Not all plastic pipe will meet these requirements.



Cross-section of plastic pipe in trench.

As with other plastic materials, ultraviolet light degradation can be a problem. Photo-degradation can cause plastic to become brittle and crack.

Carbon black is the most effective additive to enhance the photo-degradation resistance of plastic materials. Pipes containing carbon black can be safely stored outside in most climates for many years without damage from ultraviolet exposure.

Plastic pipes can be affected by liquid hydrocarbons such as gasoline and oil. If hydrocarbons come in contact with plastic pipe, they will permeate the pipe wall causing swelling and loss of strength. However, if the hydrocarbons are removed, the effects are reversible.

Regular inspection and monitoring is essential to detect any problems with plastic materials. Plastic pipes should be inspected for deformation and cracking. The inspector should also look at the interior condition of the spillway pipe. Proper ventilation and confined space precautions must be considered when entering the spillway pipe system. It is important to keep written records of pipe dimensions to note deformation and the length and width of cracks. Photographs provide invaluable records of changing conditions. A rapidly changing condition may indicate a very serious problem, and the State Dam Safety Agency should be contacted immediately. All records should be kept in the operation, maintenance, and inspection manual for the dam.

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