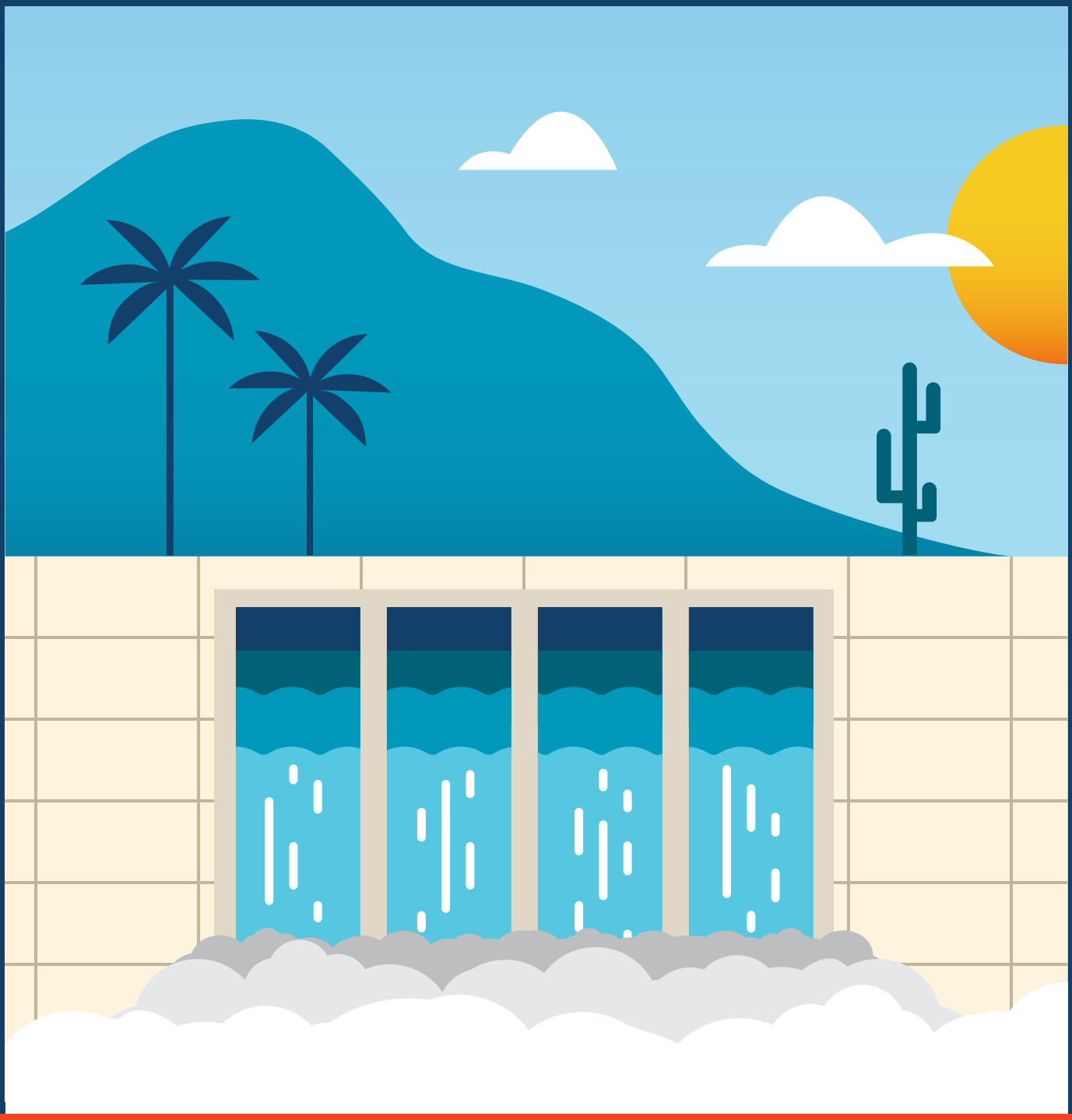
DAM SAFETY 2023

SEPTEMBER 17-21



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Foreword

The following is a compendium of technical papers which were presented at the 2023 ASDSO Annual Conference held September 17-21 at the Palm Springs Convention Center. All papers received by the publication date are included. Reports on other activities, such as the welcome remarks, administrative meetings and awards presentations will be published in the ASDSO E-News and distributed to all members.

Papers were chosen from over 150 abstract submittals by the conference program committee, chaired by John Ritchey of French and Parrello Associates. The ASDSO Board of Directors gave final approval to all conference speakers. ASDSO wishes to thank those who submitted abstracts, speakers, poster presenters, paper reviewers, and moderators for their contributions to the conference.

A special thanks to John Ritchey for overseeing the technical agenda and to program committee members Kallie Bauer, Eric Ditchey, John France, Nathaniel Gee, Matthew Lindon, Lee Mauney, Cory Miyamoto, David Griffin, and Gregory Paxson.

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Table of Contents

MONDAY, SEPTEMBER 18

Opening General Session: California Dam Safety

Sharon Tapia, P.E., Gary Lippner, Michael Anderson, and Anthony Meyers, CA Department of Water Resources 14

Session 1 - Decade Dam Failures Series Part I

1883 Disaster at English Dam
Cory A. Miyamoto, P.E., CA Department of Water Resources, Division of Safety of Dams; Keane Sommers, P.E., and Lacy Cannon, P.E., Nevada Irrigation District 15

100th anniversary of the 1923 Apishapa Dam failure, Colorado: an analysis of an underappreciated embankment dam settlement/cracking case history and the local impacts of a lost resource
Lee Mauney, P.E., CFM, and Keith Ferguson, P.E., HDR; Bill McCormick, Black & Veatch; and Mark Perry, P.E., Colorado Dam Safety 33

Hales Bar Dam: Many Firsts and Many Lessons
Eric Peterson, P.E., Tennessee Valley Authority; and Robert Hatton, P.E., Stantec Consulting Services, Inc. 67

Italian Dam Failures – Important Lessons Learned from Failure of Gleno Dam (1923) and Overtopping of Vajont Dam (1963)
Keil Neff, Ph.D., P.E., Stantec Consulting Services, Inc.; Nathaniel Gee, P.E., Tennessee Valley Authority; Bill Fiedler, P.E., HDR; and Scott Arends, P.E., Hazen & Sawyer 80

Gleno Dam - Italy's OG of Modern Dam Failures
Christopher Owen, Federal Energy Regulatory Commission 83

Session 2: Panel Discussion - Probabilistic Precipitation Frequency Tools for Dam Safety

Moderators – Greg Paxson, P.E., Schnabel Engineering; and Victoria Bahls, RTI International
Speakers: Debbie Martin, RTI International; Bill Kappel, Applied Weather Associates; and Kevin Griebenow, Federal Energy Regulatory Commission 84

Session 3: Seismic Considerations

Hardfill Dam Feasibility in a High Seismic Area
Arul Rajendram, Ph.D., Erik Newman, Ph.D., Terry Arnold, MS, Robert Green, MS, and Steven Haugaard, MS, AECOM; and Victor Gutierrez, MS, Santa Clara Valley Water District 85

Assessment of Aging Dams for Seismic Potential Failure Modes: A Delta Approach
Khaled Chowdhury, Ph.D., P.E., G.E., Tim O'Leary, P.E., Erica Medley, Ethan Dawson, Ph.D., David Serafini, P.E., G.E., Olivia Finlay, GIT, Tiffany Adams, Ph.D., P.E., and Georgette Hlepas, Ph.D., P.E., US Army Corps of Engineers 95

Numerical Modeling and Remediation Design of a Hydraulic Fill Embankment with Liquefiable Foundation Under Extreme Seismic Loading
Gauen R. Alexander, Ph.D., P.E., Bureau of Reclamation 124

Session 4: Processes and Innovations

Demystifying Alternatives Analysis for Federally Funded Projects
Katherine Sharpe, AICP, ENV-SP, Gannett Fleming, Inc. 125

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Beyond Emergency Action Plans: Leveraging Regulatory Requirements to Develop a Collaborative and Cross-Functional Hydroelectric Emergency Management Program
Andrew Hagen, Pacific Gas & Electric Company; and Jes Gearing, Gannett Fleming, Inc. 126

Using GIS as a Tool in Managing a National Dam Safety Program
Charles (Kirk) Cover, P.E., Federal Energy Regulatory Commission 127

Session 5: Fire and Ice

Dam Failures and Incidents Due To Dynamic Ice Loads
Paul Schweiger, P.E., CFM, and Boyd Howard, Gannett Fleming, Inc. 136

Frazzled – The 1972 Frazil Ice Failure of Keshena Falls Dam
Chad Krofta, P.E., CFM, Bureau of Indian Affairs; and Greta Backman, P.E., Geotechnical Engineer 137

The 70th Anniversary of the Tangiwai Railway Disaster – A volcanic dam break story
Juliana Rivera Alves Tabet, M. Sc., P. Eng., Hatch; Mark Baker, P.E., DamCrest Consulting 138

Session 6: Extreme Precipitation and the PMP

The "Perfect Storm": Can Atmospheric Models Improve Confidence In Probable Maximum Precipitation (PMP)?
Emilie Tarouilly, UCLA 153

Evaluating Spillway Capacity with Changing Data– The Effects of a Statewide PMP Study on a Portfolio of Dams
Amanda Hess, P.E., CFM, and Manogya Khanal, P.E., CFM, Gannett Fleming, Inc.; and Kurt Staller, P.E., Pennsylvania American Water 154

Evaluating Extreme Precipitation for Hydrologic Analysis for Dam Safety and Mining Operations: Considering Current and Future Conditions
Bill Kappel and Doug Hultstrand, Ph.D., Applied Weather Associates 155

Session 7: Construction Case Studies

Wanapum Dam Repair: A Challenge For Con-Tech Systems USA Inc.
Paul Krumm, Nicholson Construction Company 167

DMAD Dam Rehabilitation Phase II
Jed McFarlane, P.E., Ryan Cole, Ph.D., P.E., D.GE., Phil Gerhart, P.E., and Richard Buhler, P.E., Gerhart Cole, Inc; and Everett Taylor, P.E., and David Marble, P.E., Utah Division of Water Rights 168

Small Dams, Big Problems: A Case Study on a Complete Dam Reconstruction
Joseph Kudritz, P.E., and Brian Afek, P.E., Michael Baker International 169

Session 8: Risk Communication and Emergency Planning

From Translational to Transformational Flood Risk Communication: How USACE is Shifting its Communication About Dams, Levees, and Flooding
Katelyn Noland and Adoratia Purdy, US Army Corps of Engineers 179

Whittier Narrows Dam: Managing Risk Thru Communication: A Partnership Between USACE and The City of Pico Rivera
Melanie Ellis and Aaron Smith, US Army Corps of Engineers; and Javier Hernandez, City of Pico Rivera 180

Bourbanis Dam: The Spring 2022 Emergency Response and Moving Forward to Mitigate Future Risk
Zachary Herrmann, Houston Engineering, Inc.; and Christi Fisher, P.E., USDA - Natural Resources Conservation Service 182

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TUESDAY, SEPTEMBER 19

General Session 2: Colorado River Past, Present and Future 181

Session 9: The Dam Safety Toolbox

A Reservoir Restriction Toolbox
Jeremy Franz, P.E., Colorado Dam Safety; Brent Zundel, P.E., CFM, MT Department of Natural Resources & Conservation; and Steffen Krei, Dipl.Ing., Water & Soil Solutions 183

The Dam Safety Toolbox – A New Online Resource for the Dam Safety Industry from ASDSO
Jeremy Franz, P.E., Colorado Dam Safety; and Greg Richards, P.E., CFM, Gannett Fleming, Inc. 206

Session 10: CFD Modeling for Spillways and Outlet Works

Narrows Bypass Channel CFD Analysis – Modeling Challenges for Large CFD Models with Large Flows and Interpretation of Their 3-D Results (15 min)
Ahintha Kandamby, Ph.D., P.E., CFM, WSP; and Dusty Myers, P.E., Eagle Creek Renewable Energy 207

Solving complicated hydraulics in an old horseshoe outlet tunnel that needs some repairs through CFD analysis. (15 min)
Bryan Heiner, Bureau of Reclamation 208

Labyrinth Weir Spillway Design: Lessons Learned on the DMAD Dam Rehabilitation Project
Thayne Clark, P.E., and Jon Howe, EIT, Bowen Collins & Associates; and Everett Taylor, P.E., Utah Division of Water Rights 209

Garrison Dam - Iterative CFD Modeling to Evaluate Spillway Overlay Measures with 70 ft of Uplift Resistance
Chad Vensel, P.E., Curtis Miller, P.E., Brian Hall, P.E., D.WRE, Adam Wenck, P.E., and Korey Kadrmas, P.E., PMP, US Army Corps of Engineers 210

Session 11: Geotech Case Studies

Cracking of buttresses in an Ambursen Dam Spillway
Mussie Eyob and Prapa Haran, Federal Energy Regulatory Commission 226

Case History of 2002 Swift No. 2 Dam Failure
Nason McCullough, Ph.D., P.E., G.E., Jacobs 227

Yet Another Lesson Learned from Wolf Creek
Michael A. Brown, P.E., Tetra Tech, Inc.; and Dylan Grissom, P.E., US Army Corps of Engineers 238

Session 12: Geotech Analysis

Liquefaction Triggering Analysis of Construction and Long-Term Surcharge Loading on Earth Embankment Dams
Carlin Grundemann, P.E., GEI Consultants, Inc. 245

An Evaluation of Slope Stability of a Reclamation Dam using Three-Dimensional Limit Equilibrium Analysis
Tyler Quick, Bureau of Reclamation 246

The Impact of Vibration Loads on Liquefaction Susceptible Soils in Earth Embankment Dams
Carlin Grundemann, P.E., and Heidi Pence, P.E., GEI Consultants, Inc. 247

Session 13: Potpourri

National Inventory of Low-head Dams: completion and next steps
Rollin Hotchkiss, Ph.D., P.E., D.WRE, F.ASCE, Brigham Young University; Karl Kingery, Hydrometrics, Inc.; and Kathleen Hoenke, Southeast Aquatic Resources Partnership 248

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In a Room Full of Experts... Tips for Success from PFMA Recorders
Kathleen Darby, Ph.D., P.E., Jay Griffin, Melissa Setz, and Joe Goldstein, Geosyntec; and Cassandra Wagner, US Army Corps of Engineers 257

From Here to Recovery: The Softer Side of Effective Leadership
Leonard Greenberger, Public Relations Counselors 258

Session 14: Decade Dam Failures Series Part II

2003 Fuse Plug Failure at Silver Lake Dam, Michigan: A Look Back at the Lasting Impacts
Carmen Lepsch, P.E., Schnabel Engineering; James Pawloski, P.E., James T. Pawloski, PE, LLC; John France, P.E., D.GE., D.WRE, JWF Consulting LLC; and Greg Richards, P.E., CFM, Gannett Fleming, Inc. 259

1993 Split Rock Creek Dam Failure: Can I Get an Inspection?
Robb Roy, P.E., and Bill Kussmann, P.E., Hatch Associates Consultants, Inc. 262

2003 Emergency at Mount Carmel Dam, North Dakota
Carmen Lepsch, P.E., and Ben Webster, P.E., PMP, Schnabel Engineering; and Cory A. Miyamoto, P.E., CA Department of Water Resources, Division of Safety of Dams 263

Cart Path Only: Spillway Breaching at Black Creek Site 53 Dam
William Cantrell, P.E., Robert Hatton, P.E., and Stephen Karpenko, P.E., Stantec Consulting Services, Inc. 277

Session 15: Hydrologivariables

Is catchment area the dominant factor for PMF estimation in a supposedly hydrologically homogeneous region?
Miguel Wong, Ph.D., P.E., and Cory Anderson, P.E., Barr Engineering Co. 283

Distributed Curve Number Modeling for Runoff Volume Prediction
Michael Horst, The College of New Jersey 284

Nonlinear Watershed Behavior Using Variable Unit Hydrograph Parameters for California
Daniel Meyersohn, Ph.D., P.E., CA Department of Water Resources, Division of Safety of Dams; and Michael Bartels, P.E., US Army Corps of Engineers 297

Session 16: Risk Tools for State Dams

Development of a Consistent Framework: Risk Prioritization for 36 Unsafe, High Hazard Dams in Massachusetts
Amanda Barnett, P.E., Alex Gray, and Lissa Robinson, P.E., GEI Consultants, Inc.; David Ouellette, P.E., and William Salomaa, and Bryan Carignan, P.E., MA Office of Dam Safety 319

Oregon's Semi-Quantitative Risk Assessments for Dams with Limited Design and Analysis Information
Keith Mills, P.E., G.E., Tony Janicek, Ph.D., P.E., Arden Babb, P.E., and Bobby Voeks, P.E., OR Water Resources Department 334

7 High Hazard Dams Risk Assessment
Rick Scott, McMillen Inc.; Marc Ryan, Slate Geotechnical Consultants, Inc.; and John Falk, ID Department of Water Resources, Dam Safety 348

Session 17: Breach Modeling and Consequence Estimates

Evaluating Dam Breach Inundation for Largely Sedimented Reservoirs
Martin McCann, Jr., Jack R. Benjamin & Associates, Inc.; Ronaldo Borja and Yingxiao Liu, Stanford University; Gonzalo Castro, Consultant; Brad Hall, Brady McDaniel, and Andrey Shvidchenko, Northwest Hydraulic Consultants; Matt Farren, and Will Lukas, GEI Consultants Inc. 366

A Sensitivity Analysis Involving Simplified Approaches for Emergency Action Plan Development in Urbanized Settings
Joseph Machala, P.E., Hui Ming Shih, Ph.D., P.E., CFM, and Carlos Yvellez, EI, CDM Smith; Sushil Chaudhary, P.E., NM Office of the State Engineer - Dam Safety Bureau; and Charles Thompson, NM Office of the State Engineer - Dam Safety Bureau (Retired) 378

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PAR is not Risk. Why Proxies Don't Work.
Paul Risher, US Army Corps of Engineers; and Greg Paxson, P.E., D.WRE, Schnabel Engineering 379

Session 18: Geotechnical Exploration

Updates to USACE Drilling Guidance ER 1110-1-1807
Thomas Terry, P.E., P.G., and April Fontaine, US Army Corps of Engineers 380

Liquefaction Assessment in Gravels using Vs and DPT Blow count from Case Histories
Kyle Rollins, Ph.D., Brigham Young University; and Jashod Roy, Kiewit

Regulatory Perspective on Penetration Testing in Gravelly Soils
Alex Pires-Sturm, Ph.D., P.E., and Andrew Lutz, CEG. CA Department of Water Resources, Division of Safety of Dams 394

Session 19: DamFailures.org Updates

Shake, Quake, and Tremble – Seismic Events and their Impact on Dams (Recent Highlights from DamFailures.org)
Seth Thompson, P.E., Gannett Fleming, Inc.; Everett Taylor, P.E., Utah Division of Water Rights; Shane McWilliams, P.G., HDR; Cory A. Miyamoto, P.E., CA Department of Water Resources, Division of Safety of Dams; Jeffrey Munsey, Tennessee Valley Authority 405

Session 20: Floods, Waves, and Failures

Assessing Dam Performance During a Non-Failure Incident: Hurricane Fiona's Impact on Puerto Rico Dams
Patrick Miles, P.E., and Jacob Adkins, EIT, Freese and Nichols, Inc.; Molly Finster, Ph.D., and Carol Freeman, Argonne National Laboratory 410

An Alternate Approach for Developing Design Waves for Establishing Freeboard Requirements for Perched Reservoirs Exposed to Extreme Winds
Don Kingery, Scott Dunn, Luce Bassetti, Duane McClelland, and Rita Martins, Jacobs Engineering Group 411

Breach Evaluation of a Complex 11-Dam System Using Updated PMP with Rain-On-Mesh Hydrology in HEC-RAS
Kapil Dhital, P.E., CFM, Balbhadra Thakur, Ph.D., E.I., Alen Shrestha, E.I., and Marc Johnson, P.E., CFM, Ftn Associates Ltd 423

Session 21: Show Me the Money

Mississippi Rehabilitation Grant Efforts
William McKercher and Matt Iburg, MS Department of Environmental Quality 436

Oklahoma's Innovative Solutions to Funding Dam Safety Projects
Emma Moradi and Natalie Orbesen, Oklahoma Water Resources Board

The Corps Water Infrastructure Financing Program - A Solution to Making Water Infrastructure Affordable
Kevin McDonald, US Army Corps of Engineers 438

Session 22: Best of the Rest

Owner's Perspective: Addressing Seepage Issues During Dams Normal Operation
Mahsa Hedayati, Ph.D., P.E., Mark Ickert, P.E., and Marc Miller, P.E., Freese and Nichols Inc; Louie Verreault, P.E., and Dorota Koterba, P.E., Tarrant Regional Water District 439

Large Scale Peer Audit of Dam Instrumentation and Monitoring Program
Douglas S. Roy, P.E., and David J. Barile, P.E., GZA GeoEnvironmental, Inc. 450

Century Old, Run of the River Dam, Inspected for the First Time In 70 Years
Jeri Timm, Public Utility No. 1 of Okanogan County; and Catrin Bryan, M.Sc., P.E., McMillen Inc. 458

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Session 23: Project Delivery Options

Alternative Delivery Approaches to Constructing Dams
Del Shannon, P.E., Scott Jones, Ph.D., P.E., and Vincent Mercier, Ing,, P.Eng., PMP, Kiewit Engineering Group 459

North Michigan Creek Dam: Breathing Life into Aging Infrastructure Through Alternative Project Delivery
Nicole Young, P.E., Bureau of Land Management; and Cari Beenenga, P.E., Gannett Fleming, Inc. 469

Partnering – More than meetings. A philosophy for successful construction projects.
Michael J. Genduso, P.E., CDT, HDR 470

WEDNESDAY, SEPTEMBER 20

Session 24: What We Have Here is a Failure to Communicate: The Secret to Dam Modeling Success - Let's Discuss!

Moderator: Nathaniel Gee, P.E., Tennessee Valley Authority
Michele Lemieux, P.E., MT Department of Natural Resources and Conservation; Devon T. McLay, P.E., GeoEngineers, Inc.; Seth Thompson, P.E., Gannett Fleming; and Matthew Lindon, P.E., Loughlin Water Associates 480

Session 25: Evaluating Spillway Risks

Effort Put on the Right Path: Safety Evaluation Risk Assessments & Risk-Informed Decision-Making for Maple Grove Dam in Colorado
Hui Ming (Max) Shih, Ph.D., P.E., CFM, Joseph Machala, P.E., and Carlos Yvellez, EIT, CDM Smith; and Jarod Roberts, The Consolidated Mutual Water Company 487

Assessing Uplift at Existing Spillway Chutes in a Post Oroville World
Frederick (Rick) Lux, III, P.E., Aubian Engineering Inc.; John Trojanowski, P.E., Trojanowski Dam Engineering; and Keil Neff, P.E., Ph.D., Stantec Consulting Services, Inc. 495

SQRA Methods Bringing Clarity to a Spillway Design Flow Decision
Cory Anderson, P.E., Miguel Wong, Ph.D., P.E., and Brian Becker, P.E., Barr Engineering Co. 496

Session 26: Cutoff Walls and Grouting

Deep Foundations Institute - Guidelines for Selecting Cutoff Wall Systems
Mary Ellen B. Large, P.E., D.GE, Nicholson Construction Company; Stefano Valagussa, TREVIICOS; and Giovanni Bonita, Ph.D., P.E., P.Eng., P.G., GEI Consultants 507

Development of a Definition of Plastic Concrete for Cutoff Walls
David B. Paul, P.E., Tetra Tech; John Rice, Ph.D., Utah State University; and Dr. Karsten Beckhaus, Ph.D., Bauer AG 509

Seismic Acoustic Resonance (SAR) - A complimentary tool for mapping seepage through, beneath and around dams
Ryan Blanchard, Val Kofoed, and Mike Jessop, Willowstick Technologies, LLC 510

Session 27: Outlet Works

Anderson Dam Tunnel Project - Diversion System
Kevin Huynh, P.E., Theodore Feldsher, P.E., G.E., and John Roadifer, P.E., AECOM; and Ryan McCarter, P.E., Santa Clara Valley Water 511

Time-history analysis of Pyramid Dam Outlet Works Intake Structure as a basis for the future risk assessments.
Mohammadreza Mostafa, Ph.D., P.E., PMP, Stantec Consulting Services, Inc.; and David Sarkisian, P.E., CEG, CA Department of Water Resources 524

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Replacing a Siphon with a Low-Level Delivery Conduit on a Soft Compressible Foundation
Brandon Horrocks, P.E., Jacob Price, P.E., and McKay Harper, P.E., RB&G Engineering; and Bruce Barrett, P.E., Barrett Engineering 539

Session 28: Federal Policy Updates

NDSP BIL Implementation Progress/Updates
Jena'i Jordan, Federal Emergency Management Agency 546

Levees? Why Should States Care? Don't They Have Enough DAM Problems?
Michael L. Bachand and Jennifer Laux, US Army Corps of Engineers 547

Implementing Risk-Informed Decision Making in a Regulatory Dam Safety Program
Doug Boyer and David Capka, Federal Energy Regulatory Commission 548

Session 29: Extreme Precipitation and Climate Impacts

Climate Vulnerability Assessment for Possible Hot-Dry and Warm-Wet Future Climates for Dams in California
Debbie Martin, Victoria Bahls, and Kyle Chudler, RTI International; Nicole Novembre, Brava Engineering; Mel Schaefer and Bruce Barker, P.E., MGS Engineering 558

Reanalysis of a Record-Breaking Storm in the Colorado Rockies-Combining Meteorology with Hydrology to Find the Answer
Bill Kappel and Doug Hultstrand, Ph.D., Applied Weather Associates; and Mark Perry, P.E., Colorado Dam Safety 567

Development of Precipitation Frequency Areal Reduction Factors for Dams in British Columbia
Kyle Chudler, Debbie Martin, and Victoria Bahls, RTI International; and Mel Schaefer, MGS Engineering 566

Session 30: Seepage

Lessons Learned - Artesian Head at Earth Dams in Glacial Country
Donald R. Green, P.E., Michael Baker International, Inc. 591

South Fork Dam: Understanding and Mitigating Complex Geologic and Seepage Issues
Austin Martin, P.E., NV Division of Water Resources; E.J. Barben, P.E., and Cari Beenenga, P.E., Gannett Fleming, Inc. 592

Grindstone Canyon Dam Seepage Gallery Drain Cleaning
Gresh Eckrich, Yeh and Associates 593

Session 31: Grab N Go

The Argument for Mandatory Outlet/Drain Inspections on a 5-year Cycle.
Chadrick D. Hill, MT Department of Natural Resources and Conservation 604

Overtopping Potential for 550 Dams: Considering Storm Events with Varying Duration and Return Period
Jacob A. Smith, NC Department of Environmental Quality 605

Rapid Overtopping Evaluation for State-Regulated Dams in North Carolina
Lana Millard Waneka, P.E., and Michael Crouch, P.E., CFM, RTI International 606

The NRCS Model to Estimate Monetary Benefits of Watershed Dams from a Specific Storm or Series of Storms
Larry Caldwell, P.E., USDA – Natural Resources Conservation Service (Retired); and Gary Utley, USDA – Natural Resources Conservation Service (Retired) 607

SREL C3 – Construction Challenges and Solutions
Nagesh Malyala, P.E., Stantec Consulting Services, Inc.; Timothy Williams, P.E., G.E., Kimberly Brown, P.E., and Meredith Beswick, P.G., P.E., Kleinfelder; and Christopher Hamilton, US Army Corps of Engineers 624

Case Study: The Effects of Hot Temperatures on Deep Mix Method Seepage Barrier Construction
David C. Dean, P.E., G.E., SMUD/USACE; Christopher Hamilton, US Army Corps of Engineers; Alma Luna, P.E., G.E., Gannett Fleming, Inc.; and Doug Barber, Raito, Inc. 642

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Session 32: Hydraulic Modeling

Best Practices in Developing and Reviewing CFD Models for Spillways
Benjamin Israel Devadason, P.E., CFM, and Seth Thompson, P.E., Gannett Fleming, Inc.; and Abbas Dorostkar, Pacific Gas & Electric 650

Performance-based design of existing and future reservoir structures: An application of computational fluid dynamics modelling and machine learning techniques
Scott Dunn and Angelos Dimakopoulos, Jacobs Engineering Group 651

Showdown at Cowboy Dam
Chris Thornton, P.E., Colorado State University; David Hughes, Frank Lan, and Michael Zusi, AECOM; and Stan Ali, Contra Costa Water District 652

Session 33: Security and Resilience of Dams

Security and Resilience of Dams
Kate Schwartzer, PMP, Cybersecurity and Infrastructure Security Agency 653

Prioritizing a Portfolio using the Consequence-Based Top Screen
Michelle Yezierski, P.E., and J. Spencer Evans, Cybersecurity and Infrastructure Security Agency 654

Dam Security Panel Discussion
Kate Schwartzer, PMP, Cybersecurity and Infrastructure Security Agency 655

Session 34: Dam Removal

Programmatic and Project-Specific Approaches to Using Dam Removal Projects as Dam Safety Solutions
Meghan A. Quinn, Serena McClain, and Brian Graber, American Rivers 656

Wellsville Reservoir Dam Decommissioning: a HHPD Success Story
Melissa Menerey, CFM, and Mia Kannik, P.E., OH Department of Natural Resources, Dam Safety Program 657

Dam Removal: Improving Fish Passage, Removing Hazards, and Catalyzing Community Revitalization
Lee Holt, US Fish and Wildlife Service 658

Session 35: Construction

Underpinning Prairie: Micropiles Support Prairie Du Sac Dam - Part Two
Brian Barkauskas, Nicholson Construction Company 659

Bethany Dams Restoration and Rodent Burrow Prevention Project Case History
Youssef M. Awad and Daniel Bremerman, CA Department of Water Resources 660

A case study for embankment rehabilitation under full headwater pool: Challenges, surprises, and lessons learned
Adam J. Schneider, P.E., and Austin Rieder, P.E., Ayres; and Ryan Grondin, P.E., WEC Energy Group 661

Session 36: Decade Dam Failures Series Part III

Iowa Beef Processors Waste Pond Failure, 1993 - DFIC Decade Series
Douglas L. Johnson, P.E., Federal Energy Regulatory Commission 662

The Lake Overholser Dam Failure of 1923 - Devastation in Oklahoma City
Jonathan M. Pittman, P.E., Schnabel Engineering; Robb J. Roy, P.E., and Bill Kussmann, P.E., Hatch Associates Consultants, Inc. 670

Have Hope - The Eventful History of Hope Mills Dam, North Carolina
Jonathan M. Pittman, P.E., Mark Landis, P.G., P.E., and Adam Paisley, P.E., Schnabel Engineering 679

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Session 37: Design Considerations for Spillways

Constructability Considerations for Dams and Spillways
Brianna K. Crotty, P.E., Charles M. Johnson, P.E., S.E., and Marius Sima, Schnabel Engineering 682

Designing or Evaluating a Spillway? Chute Condition Matters!
Benjamin Israel Devadason, P.E., CFM, and Paul Schweiger, P.E., CFM. Gannett Fleming, Inc. 701

How Low Can You Go - Whittier Narrows Erodibility Study
Chris Thornton, P.E., and Jeff Ellis, Colorado State University; Ethan Thompson, US Army Corps of Engineers; and Daniel Rogers, Stantec Consulting Services, Inc. 702

Session 38: Sites Reservoir

Sites Reservoir
Derek S. Morley, P.E., Geosyntec Consultants 703

Session 39: Levee Design, Risk, and Permitting

The Engineer-of-Record's Role in Risk Informed Decision Making in Levee Design
Daniel N. Teak, P.E., and Kimberly Lukas, P.E., CPESC, HDR 704

Risks and Approach of Private Infrastructure on Public Levee ROW
Vicki B. Twerdochlib, P.E., CFM, HDR; and George Parizek, P.E., City of Omaha 705

Risk Informed Assessment to Support a Section 408 Authorization
Adam L. Hacker, P.E., and Matthias Lindenbauer, EIT, HDR; and Matthew Whelan, P.E., US Army Corps of Engineers 717

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Poster-Lightning Talk Presentations

Celebrating Success – The National Low Head Dams Inventory
Manuela Johnson, Christopher B. Burke Engineering, LLC

Stillwater Creek Watershed Dam Monitoring and Inspection Pilot Program
Sherry L. Hunt, Ph.D,. EIT, Jarrett Wise, Colton Shelton, Michael D. Buser, USDA - Agricultural Research Service; and Kevin Wagner, Oklahoma State University, Water Resources Center 728

An Evaluation of Uncertainty Associated with Generalized PMP Estimation Methods
Zahra Zahmatkesh, Ph.D., P.Eng., and Fuad Curi, P.Eng., KGS Group 729

Hydrologic Insights from 14,000 Square Miles: Unique Dynamics of the Coosa and Tallapoosa Site-Specific PMP & PMF Study
Patrick Miles, P.E., and Jacob Adkins, EIT, Freese and Nichols, Inc.; Ryan Abernathy, Southern Company Generation; and Bill Kappel, Applied Weather Associates 734

Savage River Dam, A Wild Ride: Developing 86 Miles of Inundation Mapping and Emergency Action Planning for 10 Municipalities/Counties Downstream of Savage River Dam.
Jeffery G. Powers, Kevin Chapman, and Kate Naughton, Hazen and Sawyer 735

Cementing the Future of Dam Safety
Joshua Gilman, Portland Cement Association 736

Hydraulics of beveled face stepped spillways
Megh Raj K C, MS, and Brian Crookston, Utah Water Research Laboratory, Utah State University 737

How Do We Get to This Gate?
Pete Nix, Tetra Tech 738

Impact of Groundwater Observations on Dam Design
Kyle R. Blakley, P.E., David Hayson, P.E., and Ben Schutt, Stantec Consulting Services Inc. 752

Coal mining tailings dam closure
Carina Paludo, Jair Carlos Koppe, and Anuar Bergamaschi Pires, Universidade Federal do Rio Grande do Sul 764

Design of a New Riparian Conduit through an Active Embankment Using Innovative Techniques to Mitigate Risks
Dami Adedapo, Kelly Fordyce, and Anne-Marie Hamilton, M.Eng., P.Eng., KGS Group 765

How to Exercise your Emergency Action Plans to be both impactful to dam owners and your downstream communities
Preston Wilson, Sr., Resilience Solutions Consulting 766

Was FEMA assistance equitable after the 2015 dam failures in Columbia, SC?
Ciara Horne, University of Virginia 767

Sensitivity analysis of input parameters to flood characteristics for historical dam failures using DSS-WISE Lite web-based system
Nuttita Pophet, Marcus McGrath, Mohammad Al-Hamdan, Ph.D., Paul Smith, and Seth Lambert, National Center for Computational Hydroscience and Engineering; Gokhan Inci, Federal Emergency Management Agency; and James Demby, Department of the Army 768

Digital Twins are Twice the Fun
Brian Schroeder, P.E., and Kyle M. Nicks, Collins Engineers Inc. 776

Estimated rates of failure for dams in the United States
Michael R. Dupuis, P.E., M.A.Sc., Lucas de Melo, and Glenn Rix, Geosyntec Consultants, Inc. 778

3D Detailing for Hydraulic Structures: Capabilities and Deficiencies
Cristian Franklin and Charles M. Johnson, Schnabel Engineering 791

Building a Digital Defect Timeline for Concrete Dams and Spillways – A Pathway to Predictive Monitoring for Dam Safety Engineers
Ryan T. Coppersmith, Niricson Software Inc 792

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Using the Simplified Stochastic Event Flood Modeling Approach to Support the Risk Informed Decision Making Method to Evaluate Dam Safety
Cory Anderson, and Omid Mohseni, Barr Engineering Co.; David Bowles; RAC Engineers and Economists, LLC; Mel Schaefer and Bruce Barker, MGS Engineering; and Michael Thelen, Consumers Energy Company 793

Informed Decision Making with Automated Instrumentation Data Reports - A Case History in Dam Safety Monitoring
Jordan T. Keeney, P.E., M.SCE, and Vik Iso-Ahola, Stantec Consulting Services, Inc.; and George Manole, Los Angeles County Department of Public Works 794

Assessing Dam Risk in Iowa: An Analysis of the Iowa Department of Natural Resources Dam Inventory
Abdulazeez R. Lawal, Iowa State University 795

Numerical Evaluation of Anti-Liquefaction Performance of Deep Soil Mixing Method under Embankment Dams
Araz Hasheminezhad, Iowa State University 796

Fracture Networks in Mississippian Carbonate Rocks: Investigation of Dam Bedrock Foundations in Northeastern Oklahoma
Melina Lazar and Daniel Lao Davila, Boone Pickens School of Geology at Oklahoma State University; and Steve Jacoby, Grand River Dam Authority 804

Escape Training Monotony by Hosting an Escape Room
Megan G. Garrett, P.G., US Army Corps of Engineers 805

How the City of Austin uses the ESRI Survey123 Connect App
Sergio Mendoza, P.E., CFM, CESCP, City of Austin - Watershed Protection Department 821

Closing the Gap: Combining numerical and analytical methods to predict gate closures
Adrian Strain and Brennan Smith, HDR 822

Levee Performance during Floods: A Finite Element Method Study of Transient Hydrodynamic Conditions
Audrika Nahian and Sadik Khan, Ph.D., P.E., Jackson State University 823

Using State-of-the Art Geophysical Technologies to Assess Earthen Embankments
Lincoln Steele, Majid Mirzanejad, and Vicki Scharnhorst, Tetra Tech Inc. 834

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

Opening General Session: California Dam Safety

SHARON TAPIA, P.E., GARY LIPPNER, MICHAEL ANDERSON, AND ANTHONY MEYERS, CA DEPARTMENT OF WATER RESOURCES

ABSTRACT

Following opening remarks from the ASDSO President, Mr. David Griffin, this session will welcome conference participants and provide a brief snapshot into "Strengthening California's Water Resource Management." Ms. Sharon Tapia, head of California's Division of Safety of Dams, will moderate the session. California is preparing for a future of more extreme weather events, and significant near- and long-term actions are needed to make sure the state is ready. Speakers will touch on events of the 2023 Winter Storms that impacted California, including tools used to predict atmospheric rivers and strategies implemented to manage excess floodwaters. Efforts to modernize existing water infrastructure given the extreme conditions faced by California – floods, droughts, and wildfires, will be presented. This includes plans for new and expanded reservoir storage, as well as rehabilitating existing dams to regain lost storage capacity. To support these efforts, the need for continued strengthening of California's dam safety program to support climate resilience and water supply reliability will also be discussed. Lastly, the Klamath River Renewal Project, the largest dam removal and river restoration project in U.S. history, will be highlighted. Through this opening general session, conference attendees will gain insight into how California is working to sustainably manage its finite water supply, while also working to restore, protect, and manage this resource across the State for generations of Californians to come.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

1883 DISASTER AT ENGLISH DAM – DFIC DECADE DAM FAILURE SERIES

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ABSTRACT

Eureka! The discovery of gold in California in 1848 and the ensuing California Gold Rush instigated an early period of dam building throughout the state. As hydraulic mining methods became more popular, more dams were needed to supply water for those seeking their fortune. English Dam, constructed in the mid-1850's on the Middle Yuba River, was one of those dams. Built between 1856-58 and repaired and raised in the mid-1870's, the dam was composed of three timber crib and rockfill structures. The middle dam, the largest of the three, was over 100-feet-tall and 300-feet-long. On the morning of June 18, 1883, two violent explosions were heard (Dynamite? Sabotage!), and a breach was seen in the upper cribwork of the middle dam. The breach continued to enlarge, and, within an hour, nearly the entire contents of the reservoir were emptied, estimated to be as much as 15,000 acre-feet of water. The resulting flood wave caused a rise of 40-feet at a point 43 miles below the dam, and resulted in at least seven fatalities, in addition to property damage and loss of the dam. The 1883 English Dam failure was followed shortly after by the 1884 Sawyer Decision (*Woodruff v. North Bloomfield Gravel Mining Company*), which brought an end to hydraulic mining in California, and is widely considered to be the state's first environmental law. This case study will (1) highlight information about the history and construction of English Dam, (2) describe the 1883 failure, including downstream consequences, and (3) briefly discuss dam building in California during the Gold Rush era and the fallout of the 1884 Sawyer Decision as it related to dam and levee safety.

I.BACKGROUND

The 1883 failure of the English Dam occurred at a time of great controversy over the practice of hydraulic mining in California. The collapse of the middle dam sent a wall of water downstream, causing at least seven fatalities and additional property damage. In retrospect, the impacts of the disaster and its contribution to the 1884 Sawyer Decision cannot be ignored. The failure of English Dam helped to usher in an end to hydraulic mining, forever changing the landscape of California for generations to come.

A. Site and Location

English Dam was located at the headwaters of the Middle Yuba River, roughly 21 miles northwest of Truckee, California on the historic lands of the Washoe tribe. The dam site (Figure 1) sits just upstream of Jackson Meadows Reservoir, and roughly six miles upstream of Milton Reservoir, both of which are owned and operated by the Nevada Irrigation District (NID). At an elevation of 6,140 feet [12], English Dam was located in the shadow of English Mountain (Elevation 8,373), the tallest peak in NID's watershed.

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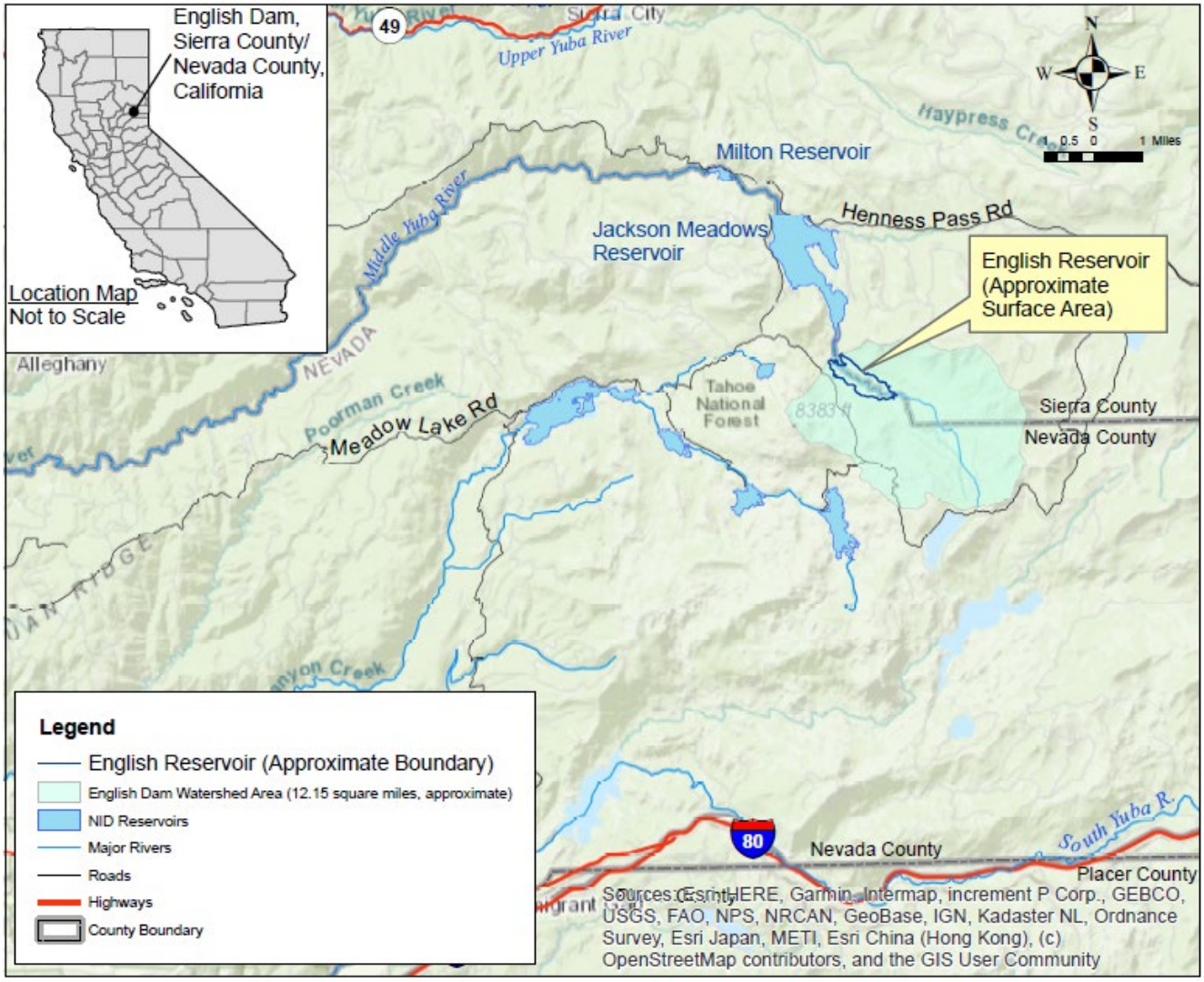


Figure 1. Site location map.

Also referred to as Rudyard Dam, the structure was built in approximately 1856-58 by the Sierra Nevada Water and Mining Company. As a large portion of the capital in this company was in English hands [15], it was often referred to as “the English Company”, and thus the dam came to be known colloquially as “English Dam”. The dam had a surface area of 395 acres, measured roughly two and a half miles long by one-half mile wide, and impounded approximately 15,000 acre-feet of water (Figures 2 and 3). The reservoir was supplied by snowmelt and runoff from a 12.1-square-mile drainage area [12].

English Dam was acquired in approximately 1868 by the North Bloomfield Gravel Mining Company, owners of the famous Malakoff Diggins claims. The reservoir was sold to the Milton Mining and Water Company (Milton Company) around 1872 or 1874 [1; 15].

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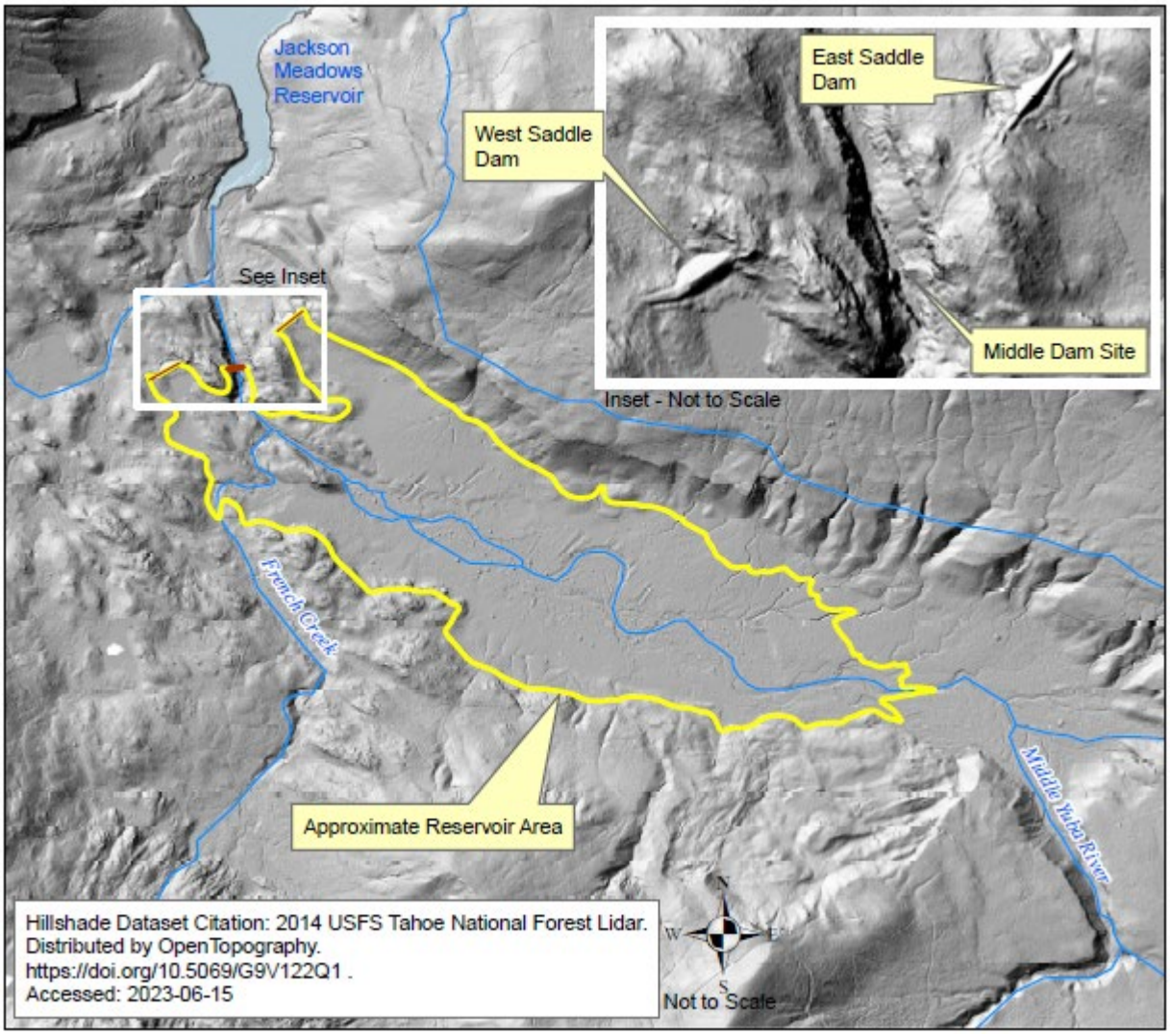


Figure 2. English Dam Site Vicinity Map.

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Figure 3. Overview of English Dam, c. 1871 (Source: California State University, Chico, Meriam Library Special Collections).

The dam served as water supply for the Milton Company's hydraulic mining operations located on the San Juan Ridge, and provided flows to roughly 80 miles of ditches with a carrying capacity of 2,800 miner's inches, or about 70 cubic feet per second (cfs). Claims operated by the Milton Company included workings at Badger Hill, Manzanita Hill, Birchville, and French Corral. Together, the Manzanita and French Corral claims grossed roughly $620,000 in 1878, providing a profit of $360,000 after expenses (nearly $11M in 2023) [15].

B. Dam Description

English Dam was composed of three timber crib and rockfill structures, including a main dam and two saddle dams (east and west). Historic photographs of the three structures are provided for reference in Figures 4-6. The Main Dam had an interior height of 100-feet, and exterior height of 131-feet. The base width was approximately 185-feet. The total length along the dam crest was estimated to be 331-feet, with a minimum bottom length of about 50-feet [1]. The original dam consisted of a rock-filled timber crib constructed of tamarack logs, 79-feet-high, 100-foot-thick at its base, and with an inner slope of 60-degrees [12]. The entire structure was faced with wooden planking.

In 1876-77, the structure was substantially repaired. Decayed portions of the timber crib were replaced with new timbers. At the same time, the dam was raised and thickened by building up a stone facing on the outside, laid up as a dry rubble wall at a slope of 44-degrees [12]. The total raise was approximately 14-feet above the original dam crest, with the upper 7-feet formed by a substantial timber crib work.

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The East and West Saddle Dams (also referred to as the north and south dams [15]) were reportedly 50-feet and 60-feet high and 150-feet and 200-feet long, respectively. Both dams were approximately 40-feet wide at their base. A spillway was provided with spill crest 14-inches below the top of the upper cribwork. Observation of the remaining dam structure as it exists today (2023) as well as review of historic photographs (Figure 3) appear to indicate that the spillway was located at the East Saddle Dam.



Figure 4. Overview of east saddle dam, c. 1871 (Source: California State University, Chico, Meriam Library Special Collections).

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Figure 5. Overview of west saddle dam, c. 1871 (Source: California State University, Chico, Meriam Library Special Collections).

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Figure 6. Overview of main dam, c. 1871 (Source: California State University, Chico, Meriam Library Special Collections).

C. Conditions at Failure

On June 17, 1883, the day before the failure, the reservoir level was reportedly 2.5-inches below the spillway crest (16.5-inches below the top of the upper cribwork) [12]. The dam had been inspected three days prior to the failure and had been found to be in satisfactory condition according to Milton Company Superintendent H.C. Perkins. There had been no signs of weakness or leakage in the structures following the raise and repair work completed in 1876-77. The water was raised annually to the spillway crest.

II.FAILURE AND CONSEQUENCES

A. Dam Failure

English Dam failed on June 18, 1883, at roughly 5 or 5:30 am. On that morning, Watchman George Davis, a Milton Company employee, heard two violent explosions. Upon reaching a point from which he could view the dam, he observed water pouring through a wide breach in the upper cribwork of the middle dam. As described by Davis in the *Nevada City Daily Transcript* two days after the failure, “It started by carrying off the upper wooden portion…It gradually crumbled the rest, stones and all, until nothing was left but the site. The water was an hour and a half running out” [10]. It was inferred that the dam breach had been caused by dynamite [12], though this claim was never proven. The resulting dam failure sent nearly 15,000 acre-feet of “water and debris crashing down the entire canyon of the Middle Yuba River, from the mountains all the way to Marysville and beyond” [10]. The initial breach produced an estimated discharge of over 166,000 cfs [1], and the resulting flood wave caused a rise of 40 feet at a point 43 miles below the dam [12].

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B. Downstream Flood Wave

On the morning of the failure, a warning message flashed along the stations of the Ridge Telephone Line:

"The English dam broke this morning at 5 o'clock. Warn everyone along the Middle Yuba River!" [9]

N.C. Miller, ditch superintendent, had received word from George Davis about the dam failure and rushed to warn those downstream. Miller dispatched the message and gave approximate time of arrival of the "hundred-foot wall of water" [6]. Reported to be the oldest long-distance line in the country [5; 18], the Ridge Telephone Line had been built in the late 1870's to help manage the long series of ditches and conveyances supplied by English Dam [15]. The line provided communication between the Milton Company's headquarters at French Corral up to Milton, just below English Dam (Figure 5).

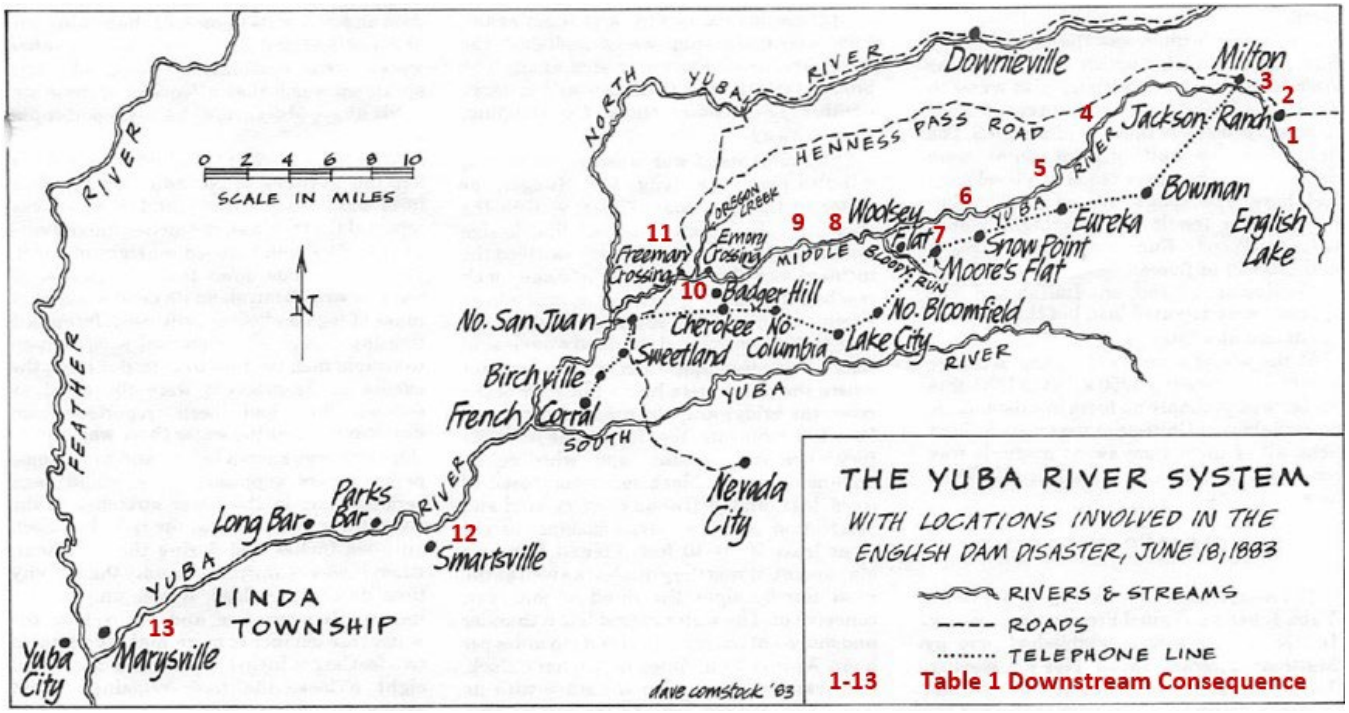


Figure 7. Yuba River and Ridge Telephone Line Stations (annotations added by authors) [15].

A thorough account of the ensuing flood wave and resulting damage is provided in a 1949 article that appeared in the *California Historical Society Quarterly* titled "The 1883 Flood on the Middle Yuba River," by Doris Foley and S. Griswold Morley [5]. Further details can be found in publications by the Nevada County Historical Society (NCHS) [9, 15], NID [10], and Bowie [1]. The timeline in Table 1 below has been summarized from these references to illustrate the timing and level of destruction wrought by the failure of the English Dam:

Table I - Flood Wave from English Dam Failure – Downstream Consequences

Location Reference	Time or Mile	Description
1	Unknown	Sheep herders tending to a flock belonging to W.P. Lipp of Wheatland, California were the first to experience the flood wave. While most of the sheep were driven clear of the raging waters, some were lost, together with a gun and some utensils.
2	3 miles downstream at Jackson Ranch	Operated by the Black brothers. Both managed to crawl out of bed just in time to escape. As reported in the Grass Valley Union after the disaster: *"The bottom land is strewn with trees and parts of trees smashed in all kinds of shapes. In many places, pieces of the*

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Location Reference	Time or Mile	Description
		trunks of trees, two or three feet through and eight to twelve feet long were peeled of all their bark as clean as could be done with an ax, and some trees 100 feet long are uprooted and peeled of all their bark and limbs…"
3	Between Milton and Jackson	Two bridges were destroyed along the Henness Pass Road
4	About six miles downstream	Ditch tender Tom Fairweather managed to escape with his new bridge thanks to the warning from N.C. Miller.
5	Roughly 12 miles downstream, north of Eureka and opposite Graniteville	Miners escaped by climbing the north bank of the river. One of two Reese brothers drowned despite efforts to save him.
6	Opposite Snow Point, about 16 miles downstream	A new tunnel was being driven for McKillon's (or McKillicum's, or McKillicon's) mine. The flood wave rapidly filled the tunnel, nearly trapping the miners inside. The men would likely have drowned had a new airshaft not just been completed, allowing them to escape.
7	Near Moore's Flat	The flood wave was reportedly as high as 60-feet.
8	About one mile above Foote's Crossing bridge	The flood wave destroyed a dam owned by the Eureka Lake Company. Between 24- to 30-feet high, the dam raised water into the San Juan Ditch that supplied the American mine at Sweetland. Two flumes were also destroyed.
9	At Horseshoe Bend	Five miners drowned.
10	At Emory's Crossing, three miles above Freeman's Crossing	12 cabins were carried away.
11	By 9 am, the flood wave had reached Freeman's Crossing, near Oregon Creek and the (present) California Highway 49 crossing of the Middle Fork Yuba River	Built in 1871, a four-span covered bridge existed at this location, measuring 240-feet-long, and resting on piers 15- to 18-feet above the water. • Upon receiving N.C. Miller's warning, Thomas Freeman guided his blind wife to higher ground. • Flood waters backed up into Oregon Creek, engulfing the covered bridge, lifting it from its foundation and carrying it 150-feet downstream. The Downieville Stage to Nevada City had crossed the bridge just moments before and was only 100-yards beyond the crossing when it was swept away. A month after the disaster, teams of oxen were used to drag and roll the bridge on planks and logs back to its original location.
12	Around 11 am, the flood wave reached Smartsville	A $5,000 stash of gold amalgam was washed away at the American Mine, along with 40 head of cattle.
13	By roughly 3 pm, the flooding reached Marysville, roughly 85 miles below the dam	The floodwaters carried a mass of mining debris, brush, trees, logs, and other debris. To the dismay of the locals, the observed rise at this point was estimated to be only 2-feet, 8-inches. Bets had been placed on how high the flood waters would rise. It was later revealed that a levee break near Linda, seven miles upstream, had flooded the Beeney ranch and largely spared Marysville from major damage.

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Location Reference	Time or Mile	Description
Not shown	Sacramento	The reported river rise was only 8-inches. The flood wave reached the State capitol in roughly 24 hours, with a total time in passing that point of 26 hours.

C. Fatalities and Property Damage

In total, at least seven fatalities were attributed to the dam failure and resulting flood wave [18]. Resulting property damage was estimated to be $40,000 to $50,000 ($1.2-1.5M in 2023), including a loss of $4,000 (2023 - $120k) for flooded wheat fields, at least 40 head of cattle and an undetermined number of sheep that were caught in the flood wave, along with approximately $5,000 (2023 - $151k) in gold amalgam that was swept away. The Milton Company suffered an estimated loss of $150,000 for the value of the dam (2023 - $4.5M), in addition to $75,000 in lost water revenue (2023 - $2.3M). The company reportedly paid all damage claims.

D. Whodunnit?

The 1883 failure of the English Dam occurred at a time of great controversy over the practice of hydraulic mining in California. Three other Sierra dams had already been attacked, reportedly by interests who objected to hydraulic mining activities. This included Brush Dam on the Yuba River, which had been burned, as well as the Birdsall Dam on the Bear River and the Alta Dam on Cedar Creek, both of which had been destroyed by dynamite [18].

In the days following event, Milton Company Superintendent H.C. Perkins publicly charged “The break was not accidental…the dam was blown up by powder” [18]. Suspecting sabotage, the Milton Company and Superintendent Perkins offered a $5,000 reward for information leading to the arrest of the responsible parties; no one was ever brought to trial. Soon after the failure, Watchman George Davis fell to death from a flume; foul play was again suspected but never proven.

Although mining officials feared more acts of sabotage would follow, the failure of English Dam was the last as the 1884 Sawyer Decision brought an end to hydraulic mining, thus stemming the controversy between mining and farming interests.

E*. English Dam Today*

English Dam was never rebuilt. A portion of the main dam and most of the two saddle dams still exist today (Figures 8 -10). Water rights to the Middle Yuba River, including the dam site, were acquired by NID in the 1920’s. The original dam site was considered again in the 1960’s during planning for NID’s Yuba-Bear Hydroelectric Power Project; however, construction of a new dam at the site was considered too costly. Furthermore, flows would be more effectively captured at the new (1965) Jackson Meadows Reservoir, just downstream [10]. NID continues habitat and hydrological restoration work at English Meadows (formerly inundated behind English Dam) to restore connectivity between the heavily incised Middle Yuba River and the floodplain. It is assumed that this disconnect is a result of the rapid draining of water following the dam’s failure.

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Figure 8. Remails of Middle English Dam looking southwest at the Middle Yuba River Canyon (Source: Nevada Irrigation District, Cameron Townsend).

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Figure 9. Remains of East Dam looking southwest (Source: Nevada Irrigation District, Cameron Townsend).

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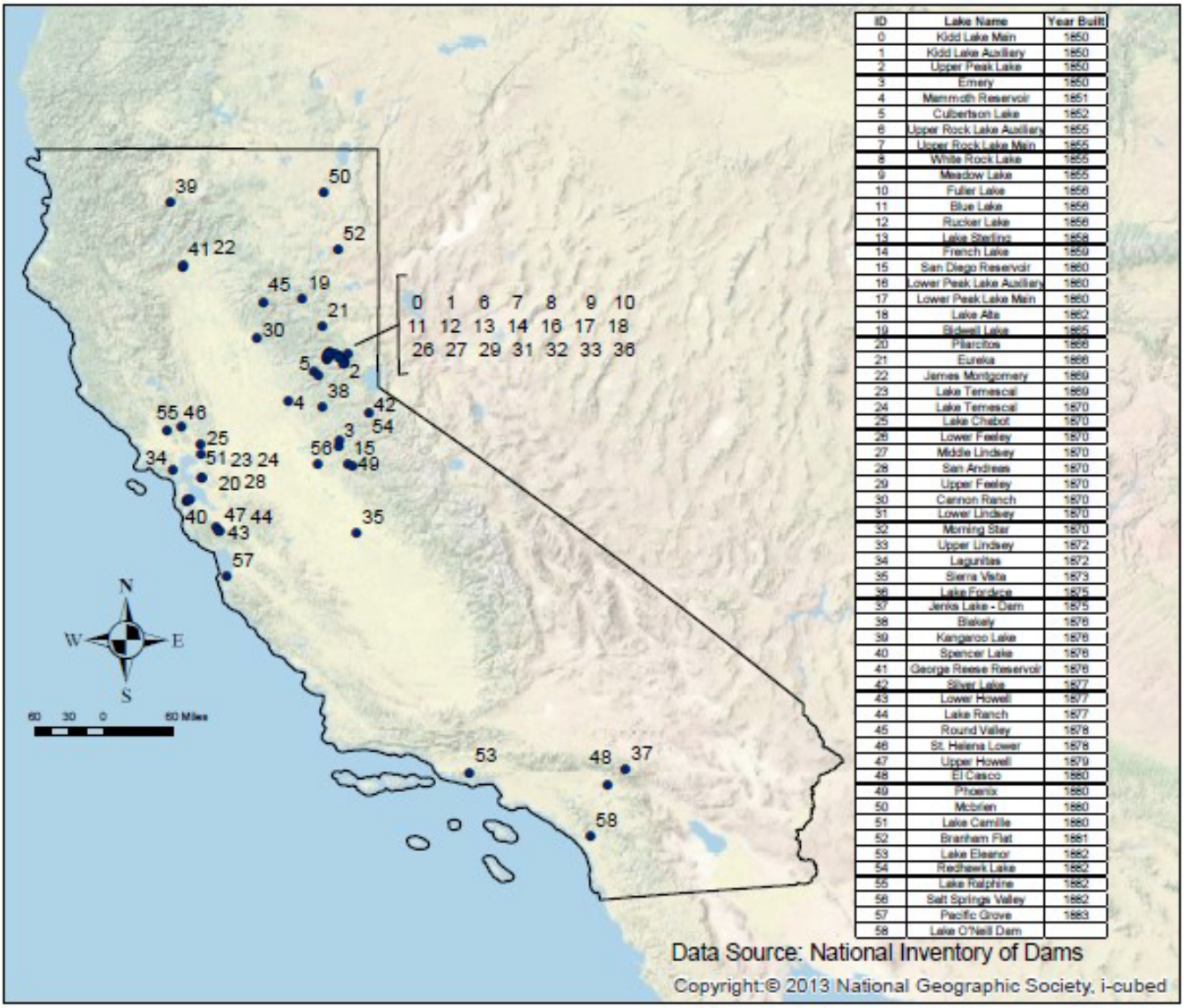
Figure 10. Remains of the East Dam looking upstream (Source: Nevada Irrigation District, Cameron Townsend).

III.FALLOUT

A. Early Dam Building in California

To consider the 1883 English Dam failure in better context, it is necessary to consider the momentous events that set its construction into motion. The 1848 discovery of gold by James Marshall at Sutter's Mill in Coloma on the South Fork of the American River forever changed the framework for constructing dams in California. Following Marshall's discovery and a second major gold strike in the Auburn Ravine five months later, thousands of people flocked to the region in hopes of striking it rich. The easy pickings were quickly diminished, and ambitious miners were compelled to design and construct more elaborate systems to separate the gold from the sand and gravel deposits. Prior to 1850, there were no engineered water systems in the foothills of California's Sierra Nevada mountains [11]. By 1884, over 58 dams had been built in California as catalogued in the National Inventory of Dams (Figure 11).

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ID	Lake Name	Year Built
0	Kidd Lake Main	1850
1	Kidd Lake Auxiliary	1850
2	Upper Peak Lake	1850
3	Emery	1850
4	Mammoth Reservoir	1851
5	Culbertson Lake	1852
6	Upper Rock Lake Auxiliary	1855
7	Upper Rock Lake Main	1855
8	White Rock Lake	1855
9	Meadow Lake	1855
10	Fuller Lake	1856
11	Blue Lake	1856
12	Rucker Lake	1856
13	Lake Sterling	1858
14	French Lake	1859
15	San Diego Reservoir	1860
16	Lower Peak Lake Auxiliary	1860
17	Lower Peak Lake Main	1860
18	Lake Alta	1862
19	Bidwell Lake	1865
20	Pilarcitos	1866
21	Eureka	1866
22	James Montgomery	1869
23	Lake Temescal	1869
24	Lake Temescal	1870
25	Lake Chabot	1870
26	Lower Feeley	1870
27	Middle Lindsey	1870
28	San Andreas	1870
29	Upper Feeley	1870
30	Cannon Ranch	1870
31	Lower Lindsey	1870
32	Morning Star	1870
33	Upper Lindsey	1872
34	Lagunitas	1872
35	Sierra Vista	1873
36	Lake Frodeca	1875
37	Jenks Lake - Dam	1875
38	Blakely	1876
39	Kangaroo Lake	1876
40	Spencer Lake	1876
41	George Reese Reservoir	1876
42	Silver Lake	1877
43	Lower Howell	1877
44	Lake Ranch	1877
45	Round Valley	1878
46	St. Helena Lower	1878
47	Upper Howell	1879
48	El Casco	1880
49	Phoenix	1880
50	Mcbrien	1880
51	Lake Camille	1880
52	Branham Flat	1881
53	Lake Eleanor	1882
54	Redhawk Lake	1882
55	Lake Ralphine	1882
56	Salt Springs Valley	1882
57	Pacific Grove	1883
58	Lake O'Neill Dam	

Figure 11. Locations of dams built in California prior to 1884.

Beginning in the early 1850's, the mountain watersheds in the area near English Dam were bustling with activity as miners and speculators were attempting to claim and develop the waters associated with them. Initially, pans and rockers were used to collect and extract the pay dirt from the ancient riverbeds, but competition was fierce, and construction was frantic. The need to process more material at a faster rate led to the construction of water conveyance features and the invention of the water cannon, leading to the birth of hydraulic mining. The first noteworthy attempt to convey water in an artificial channel was made at Coyote Hill in Nevada County in March of 1850 [10].

"Modern" hydraulic mining was first used in 1853 outside of Nevada City, California. The process used large canvas hoses to redirect water and dislodge gravel from large hillsides. [13] The resulting slurry was run through a sluice to extract the gold. Water delivered to a mining site would be shot though a nozzle onto the face of a cliff washing away tons of boulders, gravel, and dirt. The water cannons, or monitors, could "throw" 185,000 cubic feet of water in an hour with a velocity of 150 feet per second [14].

By 1869, there were 5,328 miles of artificial water courses for mining purposes in California. By the late 1860's, most of California's placer gold (gold eroded from a vein and located in a secondary deposit) was obtained by hydraulic mining and the most profitable mines, as a class, were those worked by the hydraulic process [3].

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B. Woodruff v. North Bloomfield Gravel Mining Co. and the Sawyer Decision

Concerns began regarding the tailings resulting from hydraulic mining soon after its inception. Land near downstream rivers was flooded and farms were destroyed. In the years leading up to the Sawyer Decision, it is estimated that the San Francisco Bay was filling with silt and debris at a rate of one foot per year as a result of hydraulic mining. Downstream rivers including the Feather, Yuba, and Sacramento had to be closed to riverboat traffic as a result of the increase in debris. In response to the fear of a rise in water levels, downstream residents built higher levees [4].

In 1882, Marysville resident Edward Woodruff filed suit against the North Bloomfield Gravel Mining Company due to the damages caused by hydraulic mining. In 1884, following the failure of English Dam in June 1883, Ninth Circuit Judge Lorenzo Sawyer issued what has become known as the Sawyer Decision [17]. The decision describes the damage caused by hydraulic mining and permanently enjoined the mining company against dumping any material into watercourses. Sawyer is often credited for handing down the first environmental decision from a judge in the history of the United States of America.

In the decision, Sawyer refers to the impact on levees resulting from the mining debris as well as the English Dam failure and the resulting flood as reasons for the need to outlaw the practice.

> *There could have been acquired no prescriptive right to extend the injury to other lands by continuing to send down other refuse matter from the mines, and raising the level of the bed of the river, by deposits of debris between the levees, higher and higher from year to year, thereby constantly and surely increasing the danger of breaking the levees, and discharging their augmented contents upon the surrounding country not yet destroyed. That an increase of these deposits, already elevated several feet above the level of the country outside the levees, must greatly enhance the danger, and in an increasing ratio, cannot fail to be obvious to the most superficial and least-informed observer. These barriers, upon which the present and future safety of Marysville and the adjacent country depends, are even now, with the present level of the debris confined within the levees, frail indeed, when compared with the forces of nature; liable at any time during our rainy season to be turned against them by any accidental obstruction to the currents of the flood. The temerity of those who trust their lives and fortunes to the protection afforded by these relatively feeble barriers during a flood is well calculated to excite wonder.*
>
> *The brief flood occasioned by the breaking of the English dam, in June last, afforded a striking illustration of what is liable hereafter to occur. This enormous deposit of debris in the Yuba, at and near Marysville, and in the streams in the mountains above, is a continuing, ever-present, and, so long as hydraulic mining is carried on as now pursued it will ever continue to be, an alarming and ever-growing menace, a constantly augmenting nuisance, threatening further injuries to the property of complainant, as well as the lives and property of numerous other citizens similarly situated. Against the continuous and further augmentation of this nuisance the complainant must certainly be entitled to legal protection. [17]*

The ruling effectively ended the practice of hydraulic mining in California. Although subsequent legislation has replaced the Sawyer Decision, the principles of the decision and impact it had can be seen in current legislation. The 1893 Caminetti Act modified the decision somewhat by allowing hydraulic mining in areas where a debris-retaining dam could be built to keep material from entering Sierra streams. The Rivers and Harbors Act of 1899 (33 U.S.C. Sec. 401 et seq.) provides the initial authority of the U.S. Army Corps of Engineers, and can be tied directly to the 1884 Sawyer Decision as it similarly makes it illegal to excavate or fill, or in any manner to alter or modify any navigable water of the United States without proper approval.

C. Lasting Environmental Impacts

The English Dam disaster and its contribution to the 1884 Sawyer Decision and subsequent environmental law in general cannot be ignored. The Sawyer Decision affirmed the courts' role in disputes between industry and private property owners, and balanced economic growth with benefit that industry brings to an area [16]. The impact of the decision can be seen today as the NID, the current owner and operator of several dams and reservoirs previously used to store water for hydraulic mining, works with Federal, State, and Local partners to restore the meadow that was inundated by the English Dam.

NID is in the process of implementing a meadow restoration project that includes constructing a series of native woody debris structures in the Middle Yuba River channel; raising the river channel and subsequently the water table to re-activate the meadow floodplain and improve hydrology. NID plans to treat and fill dewatering channels and headcutting tributaries to promote infiltration and absorption of surface flow, and reduce sediment transport. In addition, the project will thin 180 acres of dense understory forest vegetation on the slopes surrounding the meadow

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to improve forest health and decrease the risk of extreme wildfire. This project has included coordination with State and Federal regulatory agencies and locally affiliated Tribes, including the Washoe who have aboriginal rights to this area, in the planning and upcoming implementation stages of the restoration project. The Wildlife Conservation Board and the Sierra Nevada Conservancy are active partners working with NID to provide grant funding to realize the English Meadow Restoration Project. The agencies have provided over $2M collectively to support the expansion of forest management practices to create healthier and more resilient ecosystems in the Yuba River watershed.

Montane meadow environments are critical features throughout the high elevation of the Sierra Nevada region, providing refuge habitats for terrestrial and aquatic communities. Today we realize that these meadows are valuable and justify public investment to restore, monitor and manage them as parts of functional headwaters watersheds.

IV.CLOSING

The English Dam disaster serves as a unique case study in dam safety and environmental law. Although the cause of the failure will likely never be known, the impact of the resulting destruction is long lasting. The potential sabotage associated with the failure speaks to the need to provide adequate security to remote dam sites. Communication via telephone about the flood wave undoubtedly saved lives and highlights the need to have adequate emergency detection and response plans. The Sawyer Decision and subsequent Rivers and Harbors Act are two of the oldest state and federal environmental laws and trace their roots to the failure on June 18, 1883. As the NID and its partners in the watershed work to repair the meadow inundated by the dam and subsequently damaged by the failure, consideration must be given to the long lasting social and environmental impacts of the English Dam disaster.

V.ACKNOWLEDGMENTS

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

100th Anniversary of the 1923 Apishapa Dam Failure, Colorado An analysis of an underappreciated embankment dam settlement/cracking case history and the local impacts of a lost resource.

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ABSTRACT

Apishapa Dam was constructed to provide water for local farming communities in the arid plains of southeastern Colorado. It was a zoned earthfill (embankment) dam with a height of 115 feet and a normal storage capacity of more than 18,500 acre-feet at the spillway crest. The dam failed on the afternoon of August 22, 1923, when the reservoir reached a level two feet below the spillway crest and 12 feet below the dam crest. While no lives were lost, the resulting flood caused extensive damage downstream and limited water storage still impacts the area today. The dam experienced significant transverse cracking near each abutment during an initial partial filling of the reservoir in August of 1921. Attempts were made to repair the initial transverse cracks by sluicing material into each crack from the upstream slope of the dam. During the first full reservoir filling that resulted from a large storm event in the catchment basin above the dam in August of 1923, the embankment experienced an unusual sub-horizontal crack extending across the embankment from the right to the left abutment. Flow entering this crack at the reservoir surface on the upstream face near the right abutment and discharging from the downstream face near the lower left abutment caused a relatively rapid dam breach to develop. Remnants of the dam and spillway remain in place; the authors made a visit to the site, collected files from the state engineer's office and the Engineering News Record (ENR), and coordinated with local historians. This paper reviews the state of practice for design and construction of embankment dams at the time Apishapa Dam was built; previous forensic investigations; discusses its place in the timeline of similar failure modes, construction materials and methods, lessons learned; and provides a look at consequences from both the floodwave and loss of water resources for the local farming community.

I. INTRODUCTION: A HARD, DRY, AND LONELY LAND.

Traveling through the Arkansas River valley, one can easily be overcome by the vastness of the landscape. It is easy to overlook the scattered farm and ranch structures. On the near horizon are cedar covered mesas, and in the western distance the Greenhorns, the Sangre de Christos and Spanish Peaks. Eastward stretch the high plains as far as the eyes can see. It is a hard, dry, lonely land. Yet as you scan the scene, your eyes settle on something barely discernable in the sparse prairie grasses - a low linear mound, the site of a long-deserted irrigation ditch. There sticking up out of the grasses is a small wooden headgate topped with a steel hand wheel.

Irrigation in one form or another has been the life blood of southeastern Colorado long before the arrival of Europeans. The area south and east of Pueblo is known to anthropologist as the Chaquaqua Plateau. It was home to the Apishapa people who were hunter gatherers. They used bows, arrows, and the atlatl to harvested bison, deer, pronghorn, rabbits and small game. At the same time, they supplemented their diet by practicing various forms of irrigation horticulture, growing mostly squash, beans and maize.

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Archaeologists tell us that the Apishapa lived on the canon rims and farmed the canyon floors. These conclusions were derived mostly from the rock art they left behind (Huffman and Earley 2017; Figure 1). Various horizontal lines found on canyon walls have been interpreted to include solar, planting and travel related information, their occupation of the area taking place as early as A.D. 1200. It is documented that in the years following the Apishapa many Native American groups occupied or passed through the area.



Figure 1. Apishapa Rock Art (photo taken 11/2020, Mauney).

By the late 1880's much of the land that could be farmed under irrigation along the main stem of the Arkansas River as well as most of its tributaries had been occupied, first by explorations of Hispanic populations, then the French and English. Descriptions of small, irrigated plots and hay meadows have been documented in local histories. Title had been obtained by squatting, by purchase from original ranchers, or by homesteading.

The Apishapa River is a 139-mile-long tributary of the Arkansas River that flows from a source near West Spanish Peak in southern Colorado. It joins the Arkansas east of Fowler, Colorado. Like many western streams it has created small open valleys, twisting canyons and mini gorges. A picture of the Apishapa River canyon today, looking upstream from the site of the dam is shown on Figure 2. Spring snowmelt can bring heavy flows and summer monsoons flash floods. For the remainder of the year, the river dwindles and often flows intermittently.

In August 1898, Seneca Spencer and C. A. Watson filed a water right for the Omer Canal and Reservoir Company. The company was headquartered in the community of Omer, Colorado. The filing was for the spring, seepage and underflow waters of the Apishapa River in Pueblo and Otero Counties, for domestic and irrigation uses. After other local ditch development efforts, in May of 1910 a group incorporated the Omer Irrigation District, which set forth with plans for a dam across the Apishapa River. Over the years the Omer Canal received the adjudicated water rights. In 1905 a second canal company was created to open land on the north side of the Apishapa to irrigation. Originally named the Eureka Ditch, it was soon renamed the Van Skike Ditch for a local landowner, J. R. Van Skike.

After at least a couple of failed attempts, interests were consolidated into the Apishapa Consolidated Irrigation Company (ACIC), with a plan to finance the construction of the proposed dam on the sale of water shares. The proceeds of the offering were to pay for the new dam construction at the location shown on Figure 3, and for irrigation district development. Almost from the beginning, funding was a challenge, as the sale of water shares did not cover the cost of the dam or expansion of the irrigation system within the area downstream of the dam shown on Figure 4. It seems that at every critical juncture, decisions concerning money would drive cost cutting efforts and have both direct and indirect effects on forthcoming events, yet plans for the dam continued to evolve, and by 1915, a dam design was accepted.

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Figure 2. Apishapa River Canyon, looking upstream from Apishapa Dam site location (photo taken 11/2020, Mauney).

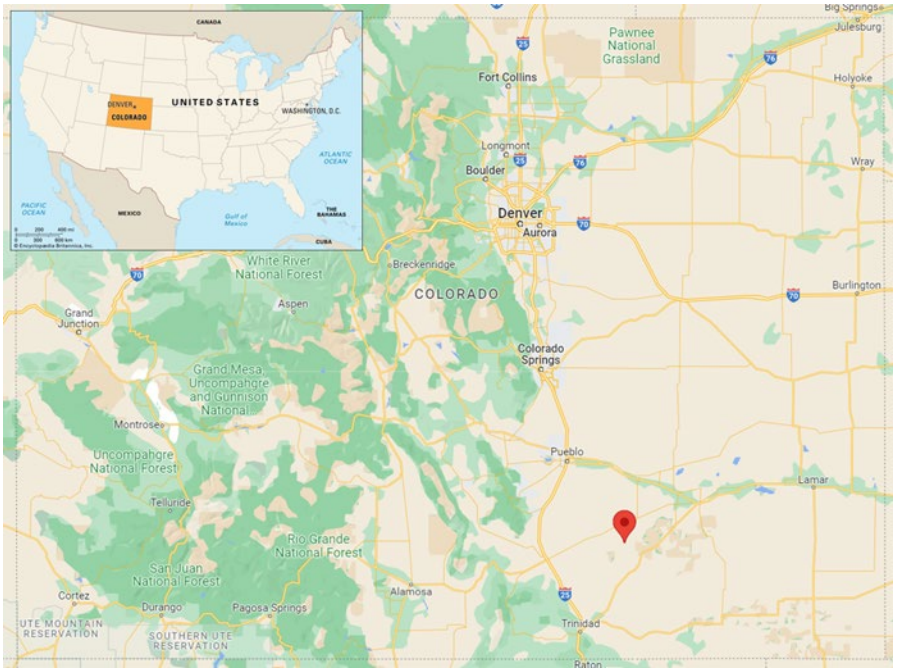


Figure 3. Apishapa Dam site location [37°44'55.8"N 104°08'41.2"W]

Apishapa dam was designed by Clair V. Mann, engineer and approved by the Colorado State Engineer on July 22, 1919. It was constructed by J.V. Stryker of Denver between August 1919 and early 1921. Issues with the dam developed from the very beginning. Figure 5 shows the reservoir hydrograph during the first 2 years of operation. The dam experienced significant settlement of the upstream slope of the embankment and transverse cracking in the upper portions of each abutment during an initial partial filling of the reservoir in August of 1921. Attempts were made to repair the initial transverse cracks by sluicing material into each crack from the upstream slope of the dam. The upstream slope was restored to its original configuration as engineers felt that all settlement of the dam had occurred, and the reservoir was ready for full operation. The reservoir filled rapidly in response to a storm in August of 1923. The dam failed on the afternoon of August 22, 1923, when the reservoir

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reached a level two feet below the spillway crest and 12 feet below the dam crest. Remnants of the dam and spillway remain in place today. The authors of this paper visited the remains of the structure, collected files from the Colorado State Engineer's Office (SEO), Engineering News Record (ENR), and local newspaper clippings from the time, and coordinated with local historical societies to gather maps and photographs.

This paper describes the dam design and construction, failure mechanism, and discusses its place in the timeline of similar failure modes, construction materials and methods, lessons learned and advancement of design and construction technology during the early to mid-1900s. While no lives were lost, the resulting flood caused extensive damage downstream and a lack of water storage still impacts the local area today. Here, we provide a look at consequences from both the flood wave and loss of water resources for the local farming community and attempt to bring to life the history and lessons to be learned from the story of Apishapa.

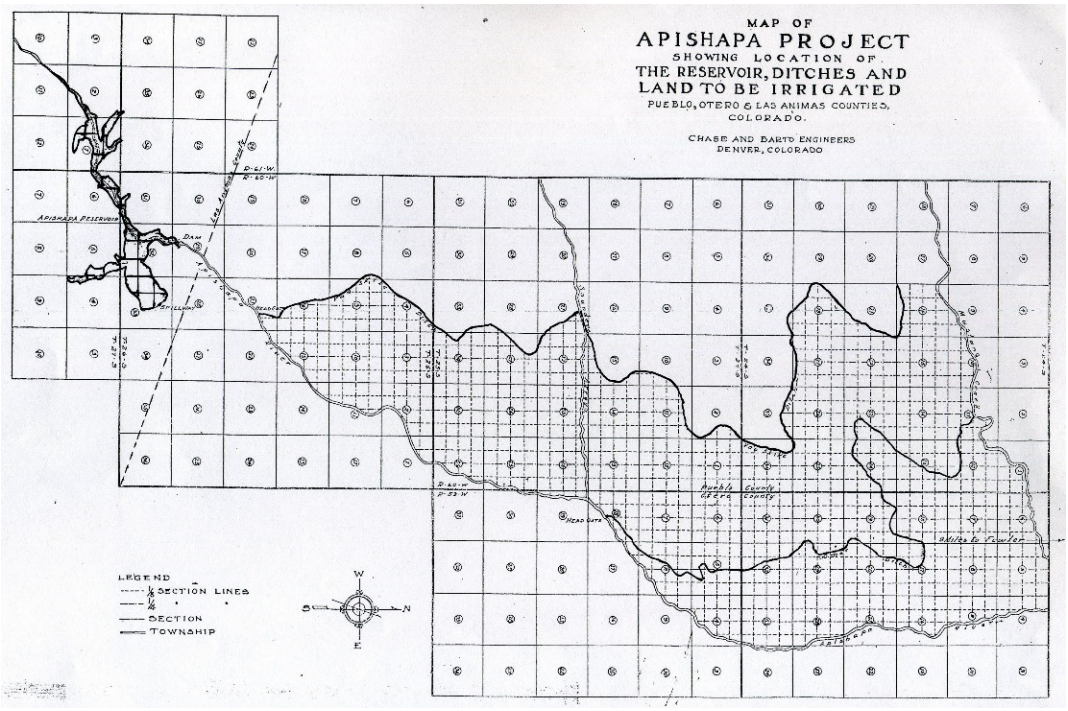


Figure 4. Map of Apishapa Project showing locations of the reservoir, ditches, and land to be irrigated (courtesy of Fowler Historical Society).

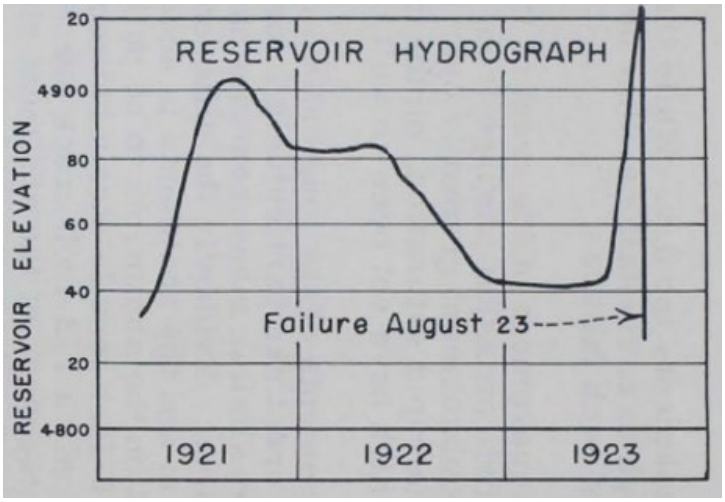


Figure 5. Reservoir Hydrograph showing Reservoir Levels during first two years of operation.

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II. OVERVIEW OF THE HISTORY OF EARTH DAM DESIGN AND CONSTRUCTION PRACTICES PRIOR TO APISHAPA DAM

Embankment dam construction around the world, including in the U.S. up to 1900 included few large dams. Sherard (1953), Wilson and Squier (1969), and Jansen (1983) have prepared excellent summaries of the history of dams that provide extensive background to understand what Sherard (1953) and Ferguson (2021) describe as the "empirically based design" and standard of care for construction of embankment dams in the late 1910s and early 1920s. Empirical design and construction methods were dependent on the precedent established by observing either the favorable or unfavorable behavior of dams and adopting design configurations from what were considered the best performing dams up to the time of the new dam's design and construction. Rational methods for analyzing or designing dams, while beginning to emerge in isolated locations around the world, were not generally available in the Empirical period of dam building that lasted until sometime into the late 1920's to early 1930's.

Jansen traces some of the earliest dam structures back to sometime between 5,700 and 2,700 BC on the Nile River (1983). Sherard traces the beginning of earthen dams to those in Ceylon (Sri Lanka) between 500 and 200 BC. He notes that extensive irrigation works were constructed in Ceylon and agriculture was highly developed. Their earth dams were very similar in appearance to modern structures. Amazingly, one of the structures of note was 11 miles long, 70 feet high and contained about 17,000,000 cubic yards of earth. Jansen notes that the common method of construction involved the placement of materials transported in baskets or other containers. Compaction was accomplished incidentally by the trampling feet of the carriers.

Dam building in India began in about 500 A.D. where structures over 1,000 years old are still in use. Jansen describes a dam built in the 11th century by Indian engineers that created a reservoir with an area of 650 square kilometers (250 square miles) in a valley about 32 kilometers (20 miles) southeast of the city of Bhopal (Madhya Pradesh). Although there are few details about the zoning (if any) in this dam, the lake was formed by two embankment dams with the largest having a height of about 27 meters (90 feet), was 92 meters (300 feet) long and had a base width of 92 meters (300 feet). The other was approximately 12 meters (40 feet) high, and 30 meters (100 feet) wide at the crest. These earthfill dams were noted as impressively watertight. The upstream and downstream slopes of these dams were faced with unmortared stones fitted together skillfully by the Indian masons. A spillway for the reservoir was excavated in a rock saddle. Water stains on the spillway sides mark the maximum reservoir level at about 1.8 meters (6 feet) below the top elevations of the dams. The successful functioning of the reservoir occurred for five hundred years when the higher dam was breached so that the fertile bed of the reservoir could be opened to cultivation. Details related to the earth materials used to construct the dam and any methods for placing and compacting the earthfill materials are unknown.

In the first 15 centuries A.D, the Japanese built about 30 dams higher than 15 meters (49 feet) that were all earthfill. Perhaps the most impressive of these is noted by Jansen (1983) as the Daimonike Dam, erected in 1128 near Nara. Its height was about 32 meters (105 feet) and was 79 meters (259) feet long. Again, details on the materials, configuration, and methods of construction of these dams are not presented by Jansen.

Puentes Dam, an earth dam with a height of 150 feet was completed at Estrecho de Rientes in Spain in 1789 that required 34 years to complete. It did not fill until February 1802 and the dam was breached the following April. This failure was a major deterrent to construction of large earth dams in the succeeding years, particularly in France and other parts of Europe. However, several moderate-sized earthfill dams were built in Europe, particularly in England and Germany in the late 1800's. The largest of these was the upper Barden Dam in England, which was built in 1876 to a maximum height of 125 feet.

A few small earth dams were built in the eastern United States beginning in the early part of the nineteenth century. For example, Druid Lake Dam was constructed for the Baltimore water supply system with a maximum height of about 118 feet in 1870. This was the highest dam in the US at the time. This height was exceeded by the Lower San Leandro (Chabot) Dam in California, completed in 1874, which had a structural height (dam crest to bottom of foundation excavation) of 145 feet. Jansen notes that a special feature of this earthfill structure was a central foundation trench excavated 9 meters (30 feet) below the streambed. In the bottom of the trench, three parallel concrete cutoff walls were built, each 0.9 meter (3 feet) thick and 1.5 meters (5 feet) high, with about half of the height anchored in the foundation and half protruding up into the dam. The dam contained a core zone which is about 27 meters (90 feet) wide at it bottom in the foundation trench. The slopes of the primary original embankment are 3(H):1(V) for the upstream, and 2-1/2(H):1(V) for the downstream. Within these limits the earth

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material was dumped by wagons and then sprinkled. Compaction was accomplished by the wagon wheels and by a band of horses which were led back and forth on the fill. A sluiced zone of earth and rock was placed against the 2 ½ to 1 downstream slope providing an overall final outer slope of 6.7(H):1(V).

Water supply systems for San Francisco and Oakland, California included four embankment dams over 80 feet in height constructed between 1865 and 1880. Sherard notes that in spite of the existence of these successful early structures, most American and European engineers remained leery of high earth dams through the end of the nineteenth century. An example of such skepticism is noted by a board of consulting engineers for the New York water supply system concluding in 1901 that:

> "…the maximum height to which an earth dam embankment, with its top 20 feet above the water line and with outside slope 2:1, can be built with safety, is 70 feet."

This opinion was based on observations of saturation lines in existing dams, concluding that the downstream slope of a dam exceeding this height would become saturated and unsafe.

Between 1900 and 1920, many earth dams were built due largely to the expansion of irrigation in the western United States. There was little experience and precedent, and there were no generally accepted methods available for design and construction of these dams. It was a period of experimentation (Sherard, 1953). Sherard further notes that by 1920, engineering literature was filled with descriptions of earth dam failures and the profession had become thoroughly alarmed by their high frequency.

The experience of the first two decades of the 1900's led to studies of the failures that were occurring, and dams were being designed and constructed in patterns that followed the successful structures. Because there were no means available for defining quantitatively the properties of fine-grained soils, none of the descriptions of dam failure in the literature included the soil type. Sherard concludes his discussion of dams build in the 1920's by noting that "As a consequence, the methods of construction which evolved were not influenced by soil properties and were adequate for the worst soil type."

1) *History of Embankment Slopes*

A number of homogeneous embankment dams were constructed in Colorado between 1905 and 1915 with 1.5(H):1(V) (horizontal: vertical) upstream slopes and 2:1 or 3:1 downstream slopes. Sherard (1953) notes that during the same time in California, the upstream slopes of embankment dams were usually flatter. In France, beginning in the later nineteenth and early twentieth centuries, it had become customary for embankment dams to be constructed with 1.5:1 upstream slopes and downstream slopes that were flatter. There was a belief that "a 1.5:1 dam always holds" even though there was evidence of slide failures that were occurring. By the early 1930's there was growing evidence that upstream slopes of dams may need to be flatter, perhaps flatter than 2:1.

Unfortunately, the Apishapa dam was designed and constructed at a time when there was still little understanding of shear strength, and the effects of seepage on slope stability. By the early 1930's rational design methods had begun to emerge, but they were slow to be implemented and much still needed to be learned by the practicing dam engineering community. For example, Terzaghi (1933) noted "that the maximum downstream slope of a dam without an impervious core is significantly flatter than the corresponding value for the upstream slope because of the downstream flow of percolating water. That this is not obvious and is only slowly being acknowledged is demonstrated by the fact that in the United States many homogeneous dams steeper downstream that upstream slopes." Proctor (1933) suggested that slopes varying from 2:1 to 4:1, depending on the foundation conditions may be needed but no specific means of slope selection was advocated.

2) *History of Construction/Compaction Methods*

The importance of compaction for embankment dams was first realized in Europe in the early part of the nineteenth century. Cattle and sheep were in general use as compaction tools in England by 1820. By 1850, heavy concrete and metal rollers came into use in Europe. This practice had been transplanted to California by emigrating engineers in the 1860's when the large water supply dams were constructed in San Francisco (Sherard, 1953).

The sheepsfoot roller was developed for road surface compaction in California about 1905. The first roller was constructed of a log, 3-feet in diameter and 8-feet long, into which 7-inch railroad spikes were driven. Its use was

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rapidly adapted for earth dams in California. For example, Drum Dam was constructed in 1907 with this new roller (Sherard, 1953).

While earth dams in California were being constructed in thin layers with heavy rollers in the later 1800's, other parts of the western U.S. were using less exacting construction methods. Sherard notes that the worst practice developed in Colorado. Here between 1905 and 1915, several of the world's largest earth dams were built by placing earth materials in 4-foot, unmoistened layers and compacted only by hauling equipment. At other locations throughout the west, between 1900 and 1920, methods of construction between the California and Colorado practices were adopted. Most dams were built in layers, 12-inches or less in thickness, some without rollers, relying only on the compaction by horses and wagons. Some embankments were being rolled in thin layers with heavy steam-powered road-roller having gross weights from 10 to 20 tons. Sherard notes that depending upon the width of the wheels, some rollers had weights per linear foot of roller which were comparable to and in some cases, considerably more than heavy sheepsfoot rollers in use in the early 1950's.

3) *Moisture Control of Embankment Materials*

Little is known about embankment dam moisture control procedures that were use in the U.S. and around the world prior to 1900. Sherard speculates that the disadvantages of constructing embankments with excessively wet or dry soils must have been recognized many years earlier, but it was not until the early 1900s that any recommendations for moisture control were published. One such guideline was published by B. Bassell, a California consulting engineer (1907):

> "Too much or too little (water) is equally bad and to be avoided. It is believed that only by experience is it possible to determine just the proper quantity of water to use with different classes of materials and their varying conditions. In rolling and consolidating the bank, all portions that have a tendency to quake must be removed at once…"

Wilson and Squier (1969) note that little attention was paid to the control of moisture contents in embankments prior to the work of Proctor (1933). They draw attention to guidance by Schuyler (1912) that did set forth the basic requirements for compaction and water content control. This, along with the guidance by Bassell would have been available to influence the design and construction of Apishapa Dam:

> "In building earth dams of any type, it is essential that the earth should be moist in order to pack solidly and if not naturally moist it must be sprinkled slightly until it acquires the proper consistency. An excess of moisture is detrimental. It should be placed in thin layers, and thoroughly rolled or tamped, and the surface of each layer should be roughened by harrowing or plowing before the next layer is applied. Droves of cattle, sheep, or goats are often used with success as tamping-machines for earth embankments. They are led or driven across the fresh made ground and the innumerable blows of their shop hoofs pack the soil very thoroughly."

Sherard then provides an interesting bit of information. First, he notes that there was little improvement over Bassell's concept (and by extrapolation, the guidelines by Schuyler) for the next 25 years (the period when Apishapa dam was built). He then notes that most engineers believed that some moisture should be added to dry materials, but no quantitative criteria evolved. Several rules of thumb for the lower limit of moisture content became widespread in use. One of the most common of these was to require that material have a sufficient moisture to allow it to be rolled into a cylinder about the size of a pencil without crumbling. Engineers today recognize this rule of thumb as being similar to the moisture content defining the Plastic Limit (PL) as part of the well know Atterberg limits created by Albert Atterberg in 1911 and later refined and standardized by Arthur Casagrande in support of Glennon Gilboy's doctoral thesis completed at MIT and published in 1928 under the direction of Terzaghi (Gilboy, 1928).

4) *Zoning and Drainage Provisions of Embankment Dams*

It is likely that the need for preventing seepage from reaching the downstream slope of an embankment dam was discovered soon after embankment dams began to be constructed. Sherard notes that the three principal means of keeping the saturation surface below the downstream slope included: a) a flat downstream slope, b) zoning within the dam, and c) drainage. Different areas of the world would give preference to one or more of these methods. In India for example, Sherard notes that dams were being constructed with downstream slopes flatter than upstream slopes. Before 1920, experiments by the Bombay Government concluded that the surface of

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saturation assumes a 4(H):1(V) (Horizontal:Vertical) slope from the reservoir surface and the downstream slope should be flatter than this saturation slope surface. Further, it was commonly believed that “It is better not to go in for drains and induce the passage of water through the dam.”

The first “zoned” earth dams appear to have been constructed in Europe in the early part of the nineteenth century. These dams incorporated an impervious core usually constructed out of clay or masonry. In the U.S., water supply dams constructed in the 1860’s by the City of San Francisco had thin vertical clay cores. They were designed by a young German engineer (H. Schussler). Sherard believed that these dams were “patterned after some European precedent”. Yarrow Dam constructed in England in the last part of the nineteenth century had a section like the San Francisco dams. Other types of zoned dams were also being experimented with including composite earthfill (upstream portion) and rockfill (downstream portion) sometimes with a separation of hand- or derrick-placed rock separating the zones at about a 1.5(H):1(V) slope. Other times, the earthfill and rockfill materials were not separated with any filters. Subsequently, composite earthfill/rockfill dams like Avalon and McMillan Dams on the Pecos River in New Mexico failed in the 1890’s as well as Scofield Dam in Utah (failure in 1928).

Most of the western U.S. dams constructed before 1920 were homogeneous, with a few having central masonry core walls and some having central or upstream impervious zones of clay. After 1920 it became increasing popular for the central cores of embankments to become thicker with heavy pervious outer zones also being used. By 1930, Sherard notes that zoning had progressed so far that homogeneous dams became the exception.

The value of gravel drains and rock toes was realized by engineers in California before the end of the nineteenth century. For example, Bear Gulch and Lake Frey Dam were provided with drains under the downstream slopes in the 1890’s. Also, an extensive downstream drainage system was built into Tabeaud in California in 1903. Sherard notes that Engineer G.S. Williams used closely spaced, longitudinal and transverse tile and gravel drains in the downstream half of dams in Wisconsin and Michigan before 1910.

While many prominent engineers had used downstream drains since 1900, their value was not generally recognized even into the 1920’s. There were well known engineers openly expressing contrary opinions on the incorporation of drainage into embankment dams such that at the time Apishapa was designed and constructed, there was no generally accepted best practice and the decision to incorporate drainage was the sole discretion of the designer. While the desirability of preventing the surface of saturation from reaching the downstream slope was almost universally being advocated, even as late as 1933, the well-known engineer Proctor claimed that seepage appearing on the downstream slope would not be injurious to embankments constructed according to his new methods of moisture-density control. Proctor (1933) wrote: “The usual custom of providing a porous section on the downstream side of an earthfill dam need not be followed. …It is not necessary to provide drainage for such fills … in fact, it is better that they retain all of the water possible to minimize shrinkage from drying.”

One final note is important here. Drainage, while sometimes being incorporated into the downstream portion of embankment dams in the early 1900’s, was empirically based. The practice of flow net construction was only beginning to be taught by Terzaghi at the Royal Ottoman College in Turkey in 1919, and the concept of filters was only beginning to be understood as part Terzaghi’s research. Hence, the ability to understand flow patterns, flow quantities, and seepage gradients into drainage systems and to provide adequate filtering of embankment and foundation soils adjacent to the drains did not exist in embankment dam engineering practice at the time Apishapa Dam was designed and constructed.

III. PLANS FOR A WATER SOURCE: APISHAPA DAM

1) *The Local Setting and Key Players*

The site was at a point where the river had cut a 200-foot-deep gorge, the canyon walls made up of massive sandstone layers with alternating lenses of shale and clay. At the base was a bedrock layer of Purgatory Sandstone, part of the Dakota formation. From the beginning, there were concerns about the type and quantity of suitable building materials available on site for construction.

Four engineers played important roles in the Apishapa Dam's construction.

1. From August 30, 1918, to September 18, 1920, Clair V. Mann, P.E. was engineer in charge of design and construction for the dam. He was on site during most of the earthwork. He was also an inspecting engineer for the state engineer, reporting to McCune. Mann left shortly after the end of construction. It

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was Mann who drew the "As Built" plans. Chase & Barto of Denver, Colorado provided surveying services and had drawn some earlier versions of the plans.

2. H. H. Knowlton, P.E. was the civil engineer who had helped draw the plans for the first proposed dam. H. H. was a landowner in the district, as well as a water share investor, and was sometimes referred to as the Mayor of the Van Skike District. He was elected and served in several positions as an officer of the irrigation company. His major role was that of corporate representative between the irrigation company, its hired engineers, contractors, and suppliers. He was involved in much of the day-by-day decision-making process involving the ACIC.
3. A. N. Dallimore, P.E., another district resident and civil engineer, served as the irrigation system engineer. He was responsible for the concrete sections of the diversion structures at the dam and most importantly he managed the operation of the irrigation system.
4. A. J. McCune, P.E. was employed as a consulting and inspection engineer. In his position he was responsible for many stages of early construction and approved many of the changes. In August 1918 he became the Colorado State Engineer but continued as a consultant to ACIC.

2) *Design and Construction of Apishapa Dam*

The design of the dam is shown in plan-view on Figure 6. A cross-section of the dam at its maximum section is shown on Figure 7, and a profile under the centerline axis of the dam across the valley is shown on Figure 8.

Overall, the design approved by the State Engineer and shown on these figures represents the best practices at the time for embankment design and construction as previously described. The dam design included construction of a foundation cutoff wall with sheet piling and abutment cutoff with conventional concrete made with Portland cement illustrated on the embankment profile under the dam crest centerline axis. The design included zoning intended to minimize seepage (foundation select fill in upstream 4/5ths of the dam, and coarse embankment materials downstream 1/5 of the dam), a drainage blanket and toe drain system with a tile drainpipe under the downstream shell intended to collect and discharge seepage that might develop through the dam, the dam foundation, and the dam abutments. These provisions in the design were progressive and reflected awareness by the designer (Mann) of dam design and construction practices as described previously in this paper. Foundation soils up to 30-feet in thickness were left under the entire footprint of the dam.

Our review of the specifications for construction of the dam indicate that they reflected best practices at the time for borrowing, placement, moisture conditioning, and compaction of embankment materials. For example, one section of the specifications for the embankment construction are shown on Figure 9. Embankment materials were to be placed and compacted in 12-inch lifts with:

> "a 10-ton road roller, passing at least twice over the entire filled area. In addition, the teams hauling material shall always be deployed so as to secure the greatest possible amount of compacting from wagon travel. If for any reason the Company shall choose to dispense with rolling, the greatest care shall then be exercised to secure the maximum amount of compacting by means of the teams and wagons."

The specifications also provided for treatment of the top of compacted layers by scarification, and moistening so that adequate integration and bonding between layers was achieved. The natural materials for embankment construction were known to be dry. The specifications clearly required the addition of water to the fill materials stating that the materials shall "be spread and sprinkled sufficient to insure thorough compacting" as decided by the Engineer. Photographs available for review suggest that the majority if not all the embankment material was placed and compacted by controlled routing of loaded wagons drawn by horses (see Figure 10). While a roller of the required weight may have been available, photos do not show it being used. Photographs lack evidence of any major effort to moisten the fill materials and post failure investigations describe the remaining embankment materials in the abutment areas, while placed in the proper lift thicknesses, were very dry.

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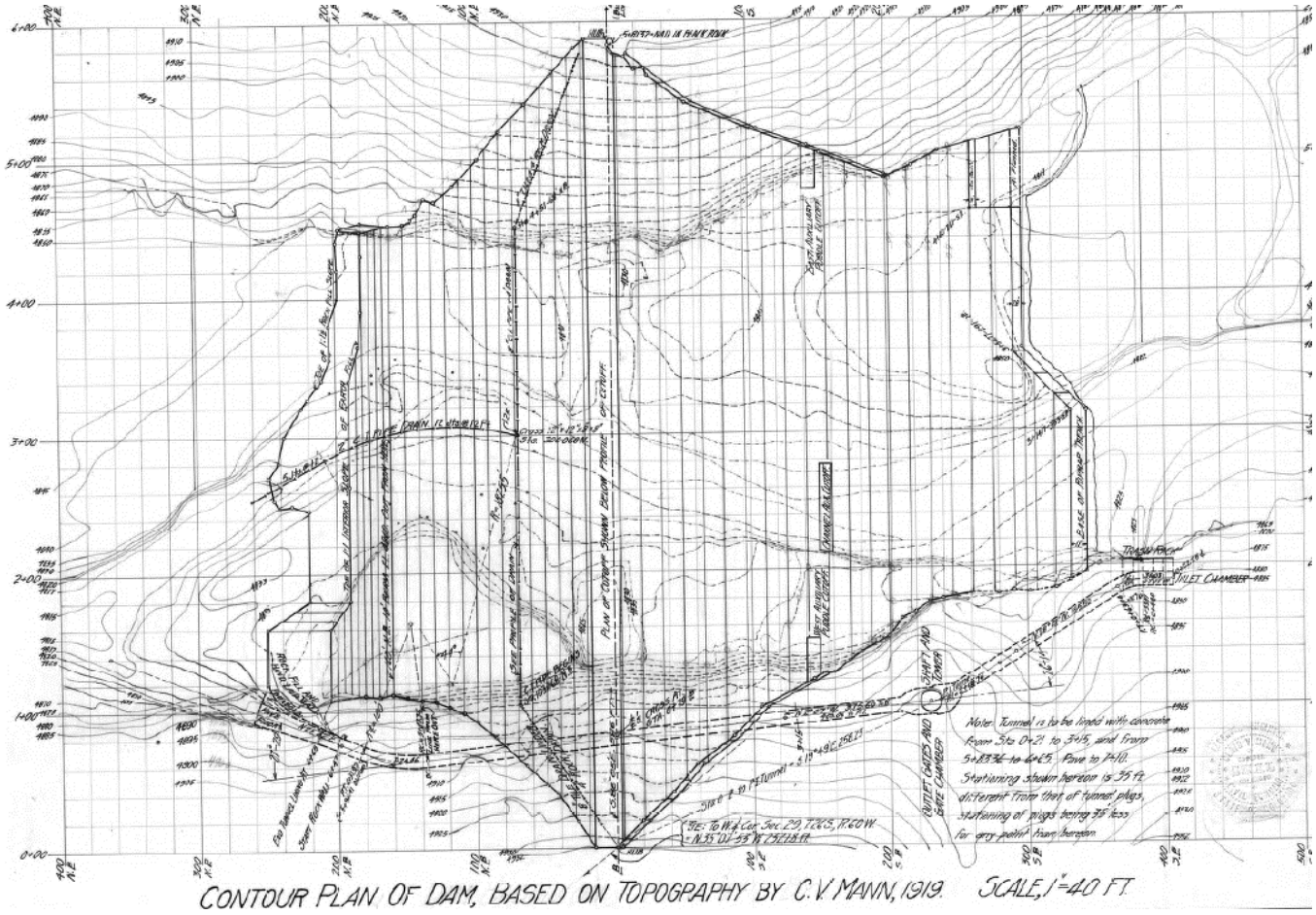


Figure 6. Plan View of Dam (upstream to right, north to the left)

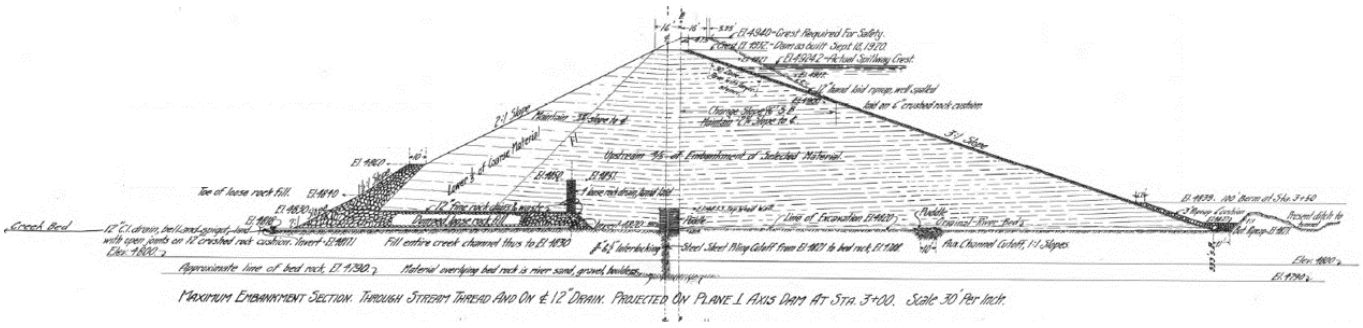


Figure 7. Typical Cross-section of Dam at Maximum Section (approx. Sta 3+00)

Sherard's investigation (1953) showed that there were two sources of materials used for construction of the embankment. The characteristics of the embankment as Sherard interpreted the constructed are shown on Figure 11, taken from Figure 5 of his 1953 publications. The lower embankment materials were obtained from borrow taken from within the valley bottom both upstream and downstream of the dam. The upper materials were taken from a borrow area on the eastern plateau above the dam. The access route from the borrow area on the plateau can be seen on Figure 10.

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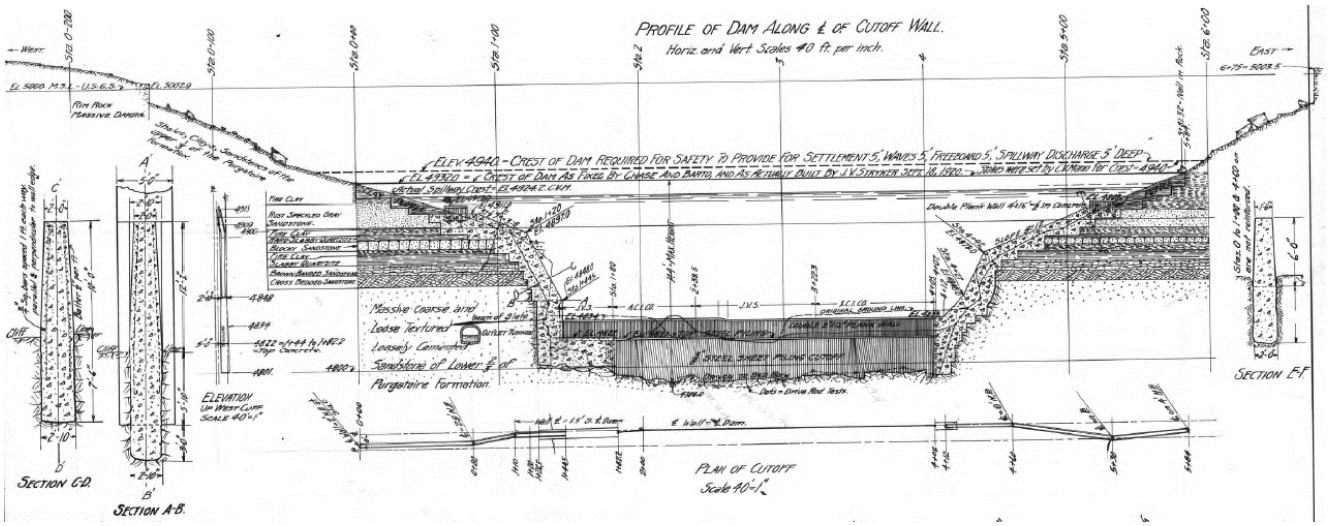


Figure 8. Profile of Dam Along Centerline Axis (looking downstream

Both the soil types used in the dam classify as sandy to very sandy low plasticity clay (CL – Unified Soil Classification System – see Figure 12). A compaction curve developed by the U.S. Bureau of Reclamation in support of Sherard's thesis showed that the Soil 1 materials had an optimum moisture content of around 22%. While a compaction curve was not developed for the Soil 2 materials, experience would suggest that the higher sand content and lower plasticity would have likely resulted in an optimum moisture content of 14 to 18 percent. In both cases, compaction substantially below the optimum moisture content would have resulted in brittle behavior (materials conductive to cracking). While information is not available, it is possible that the Soil 1 materials, compacted at low water content and to density well below the maximum density that could have been achieved, would have behaved in a normally consolidated manner (once saturated) with a relatively high compression index and even some amount of collapse potential upon wetting. Both materials were likely unsaturated at the time the reservoir was filled making the prediction of behavior complex and difficult at best.

The specification requirements summarized above, and other project specifications reflect a high level of expertise and experience in dam design for the time. In fact, many of the specifications and details on the drawing reflect approaches to design and for construction still in use today. However, design and construction technology available at the time was not sufficient to address several critical issues, some that directly led to failure of the dam, and others that would likely have led to failure or significant dam safety incidents over time:

1. Moisture control of the embankment fill that would have provided a combination of low permeability, predictable compressibility, and flexibility of the embankment materials.
2. Estimations of the engineering properties of the foundation soils that were left in place and prediction of the actual amount and pattern of settlement that would occur under the central portion, and the shells of the embankment.
3. Shaping of the abutment excavations to minimize the amount of differential settlement that would occur over relatively short distances under the crest of the dam. The settlement of the foundation materials adjacent to the abutment (along with steepness of the abutments), and under the upstream slope of the dam were likely the primary cause that led to transverse cracking through the crest of the dam, and longitudinal cracking under the upstream shell of the dam.
4. The sub-horizontal crack that developed through the dam leading to failure was likely caused by a complex mechanism related to a) settlement of the foundation materials, b) the settlement and possible collapse of the Soil 1 materials just above the embankment/foundation alluvium contact becoming wetted under the dam crest, and c) cross valley arching of the upper portion of the dam. Modern dam construction methods including removal of the foundation materials down to bedrock under a central zone of impervious material in the dam would have provided the necessary protection against development of the sub-horizontal cracking that occurred under the dam crest.
5. Adequate filter design of the embankment zones and downstream drainage provisions including a chimney drain.
6. Foundation and abutment grouting.

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76. METHOD OF CONSTRUCTING EMBANKMENT.-The dimensions, slopes and other details of the main earth embankment are shown on the Contract Drawings, and set out in paragraph 2 of these specifications. When the foundation has been prepared as specified in paragraph 61, and the auxiliary cut-off trenches, puddle wall, and drains begun or completed as the case may be, construction of the main earth embankment shall be begun. The upstream four-fifths of the fill shall be composed of "selected" material, and the downstream fifth of "coarse" material as specified in paragraph 75. This material shall be placed in layers not exceeding twelve inches in thickness, the same lying in horizontal position in a direction across the canon, but pitching in and downward to the center line of dam from each face, with a slope of 2 feet per hundred from upstream face, and 3 feet per hundred from downstreamface. Care shall be taken in bonding the two classes of material together to prevent the formation of a cleavage plane. To this end, successive horizontal layers shall break joints, and the two materials shall grade gradually one into the other. The layers coming into contact with the foundation shall be of the best, most elastic, durable, and impervious materials that can be found. The auxiliary cut-off trenches shall be filled with such material, well wetted, and compacted with heavy cast iron tamping bars where rolling or compacting with teams cannot be done. Where earth embankment comes into contact with the rock cliffs, the latter shall first be picked free from loose rock fragments, and the earth shall then be tamped, flooded with water, or puddled to insure the complete filling of all holes and seams.

Materials for embankment shall be hauled in dump wagons having at least 3½ inch tire treads, and shall be spread evenly in the successive layers, by hand shoveling or scraper-graders, so that when spread the material shall conform to the required lift. Before placing new material upon an old layer, or upon the foundation, the old layer or foundation, as the case may be, shall be dragged over with a weighted harrow, then properly sprinkled immediately ahead of placing the new fill, so that the new material shall always be placed upon the freshly harrowed and wetted surface of the old layer.

When the materials in any layer have been properly spread, and sprinkled sufficiently to insure thorough compacting, they shall be thoroughly rolled with a ten ton road roller, passing at least twice over the entire filled area. In addition, the teams hauling material shall always be deployed so as to secure the greatest possible amount of compacting from wagon travel. If for any reason the Company shall choose to dispense with rolling, the greatest care shall then be exercised to secure the maximum amount of compacting by means of the teams and wagons.

Water for sprinkling shall be furnished in considerable quantity, inasmuch as the filling materials will be quite dry. The water must be furnished through a suitable hose, attached to a system of water pipes supplied by tanks filled by pumps large enough to more than supply the water actually used in any one day. The tanks should be large enough to insure reserve supply in case the pumps get out of repair. The Engineer will always be the judge as to whether sufficient water is or is not being used to compact the materials.

Figure 9. Example of Project Specifications for Embankment Construction

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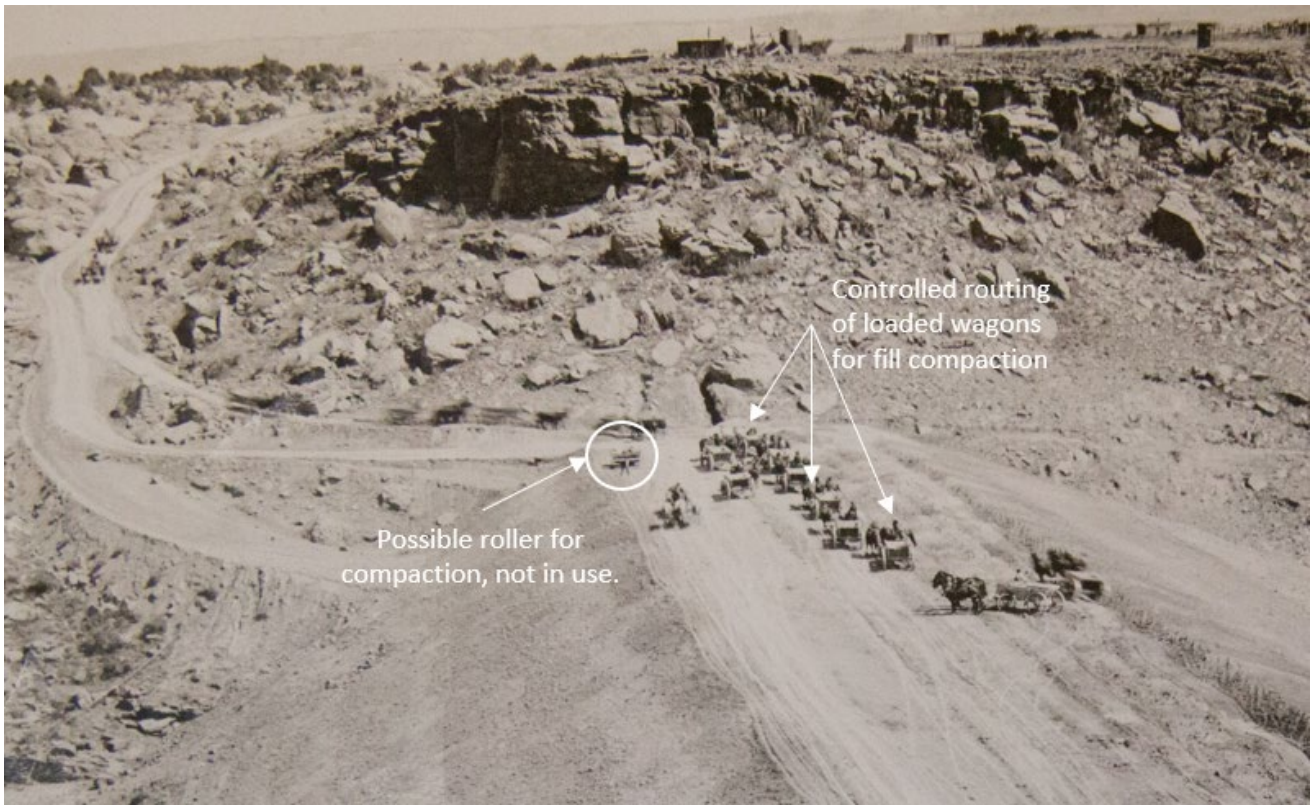


Figure 10. Photograph of loaded wagons compacting embankment materials, looking east (Fowler Historical Society).

IV. INVESTIGATION AND DESCRIPTION OF FAILURE EVENT

The most comprehensive study of the Apishapa dam failure was completed by Sherard as part of his Doctoral dissertation published by Harvard University and by the U.S. Bureau of Reclamation as Technical Publication 645 (Sherard, 1953). At the time of Sherard's work, the dam was one of six dams with a documented record of cracking that led to either failure of the dam or a significant dam safety incident. He summarized three factors that were generally associated with the (6) dam cracking failures as:

1. Low construction moisture content
2. Construction materials consisting of silts and silty clays
3. Steep abutments

A summary of the information he developed as part of his work on Apishapa was shown on Figures 5 and 6 of his report. Information from his Figure 5 was previously presented (see Figure 11) in describing the construction of the dam. Sherard identified, sampled, and tested the two different materials that were used to construct the dam as he summarized on a typical cross-section shown in the lower left corner of this figure. Soil type 1 was obtained from excavations within the stream channel just upstream from the dam consisting of slightly sandy to sandy and silty low plasticity clay (CL). Soil 2 was obtained from extensive shallow excavation of soils on the prairie above the right (east) abutment of the dam. These materials were a sandy to very sandy, silty, and slightly gravelly low plasticity clay (also CL) that was highly calcareous. A sample of these (Soil 2) materials taken from the borrow pits after failure lost 6 percent of its weight when allowed to set in water for 2 hours. His investigation postulated that the embankment was constructed on up to 30 feet of stream channel deposits like the material Type 1 described above. The consolidation/settlement properties of either the foundation stream deposits, or embankment materials were not determined as part of his investigation.

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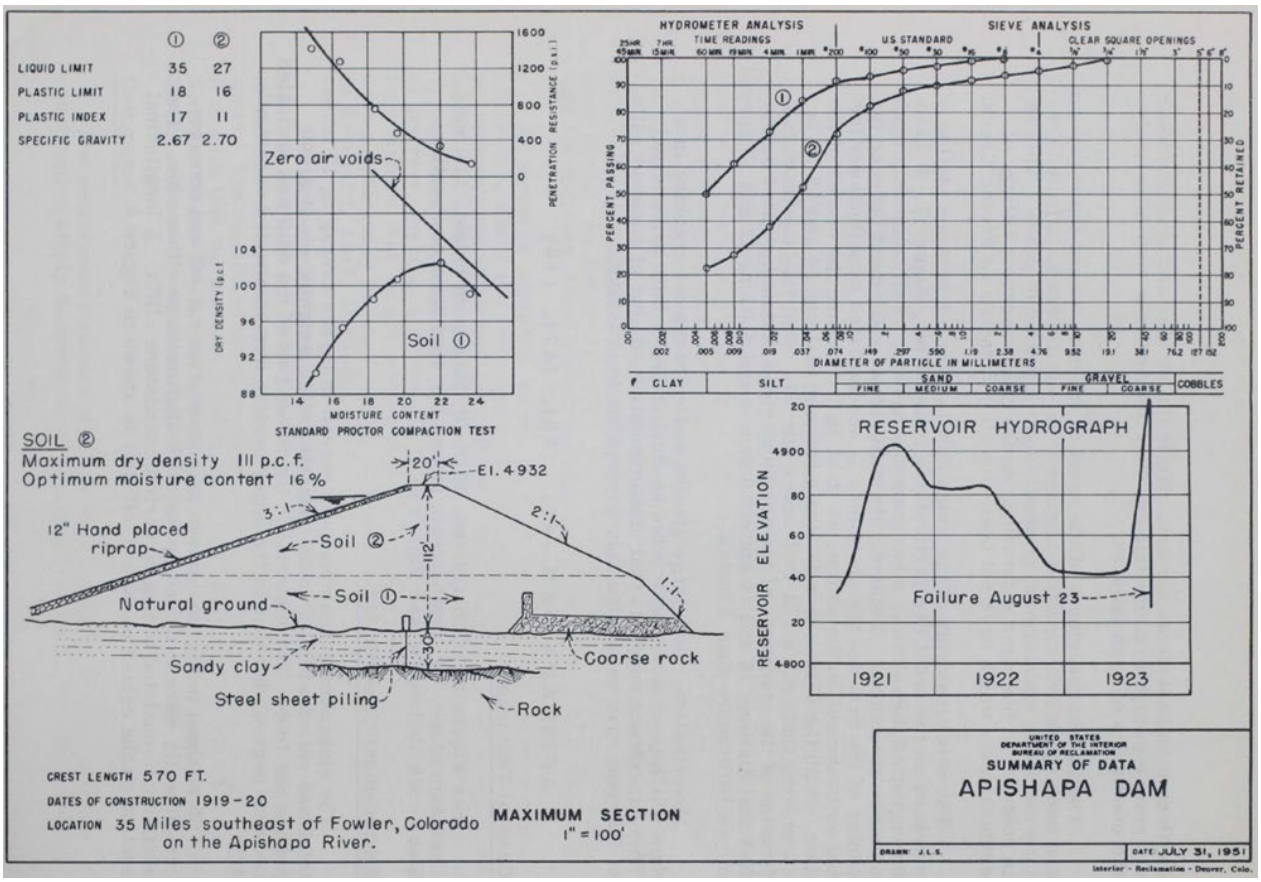


Figure 11. Apishapa Dam, Summary of Failure Investigation Data (Sherard, 1953, Figure 5)

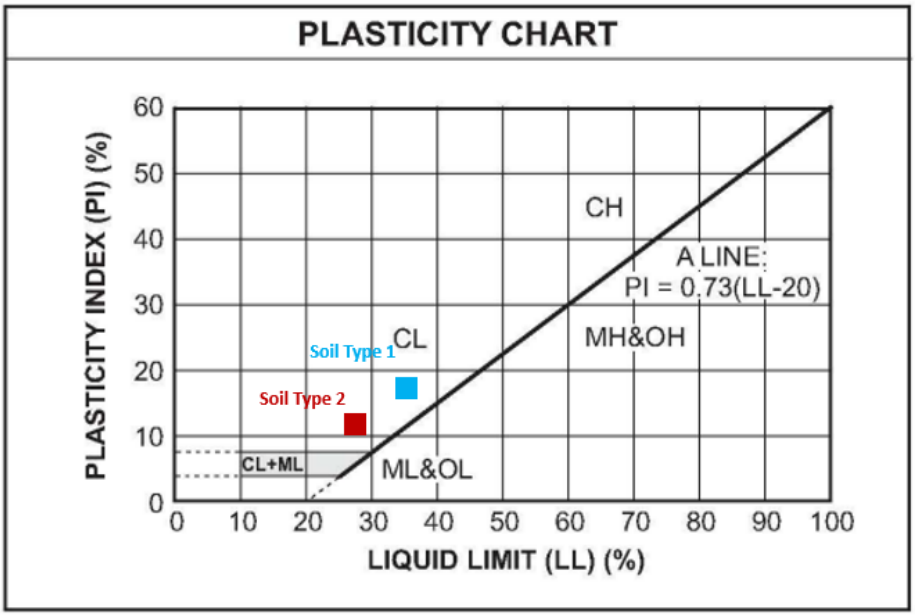


Figure 12. Plasticity Characteristics of Embankment Materials

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The following is a summary of Sherard's description of the failure of Apishapa Dam:

The reservoir hydrograph for the life of the dam is shown in the lower right corner of Figure 11. The reservoir reached a high in the fall of 1921 and then dropped until the summer of 1923, when a heavy rainstorm in the 700 square mile catchment above the dam sent the reservoir level to within 10 feet of the crest of the dam just before failure.

The dam suffered considerable settlement during the partial reservoir filling that occurred in 1921. Levels run on the embankment on May 10, 1922, established that the crest had settled a maximum of about 2.5 feet during this initial partial filling of the reservoir. Based on a dam height of 112 feet, the maximum settlement represented about 2% of the height of the dam. At the same time of the initial settlement episode, the upstream slope settled in a trough-like depression deeper than at the crest. The upstream slope settled vertically 3.5 feet at the center of the dam about 30 feet below the crest or at about the 1921 high water mark.

Following the maximum storage in 1921, two cracks opened transverse to the axis of the dam, parallel to and at distances of 80 and 90 feet, respectively, from the east and west abutments as shown on Figure 13 (Figure 6 from Sherard report). These cracks were over the shoulders of the rock cliffs where the fill height was much less. No record of the width of these cracks was found. Based on similar cracks in other dams, Sherard speculated that they were probably at least 2 to 3 inches wide at the crest. The crack on the east end extended from the crest to the water surface on the upstream slope and ran down the downstream slope. The west crack did not run down the downstream slope.

At the same time as the transverse cracks developed near the ends of the dam, a longitudinal crack was observed to open on the upstream slope parallel to the dam crest and a little above the 1921 high water mark. This crack was undoubtedly the direct results of the fact that the upstream slope settled 1 foot more than the crest at the center of the dam. These cracks were finally repaired by pumping water and earth in them until they were sealed. After the cracks were sealed, believing it was unlikely that further settlement would occur below the 1921 high water line, the company added more material to the upstream surface to compensate for the settlement and to bring the slope back to its original constructed dimension.

During the whole period before the failure, there had been considerable leakage through the stratified sandstone comprising the east abutment. This appeared downstream from the embankment and never touched the downstream slope. The quantity varied with the reservoir level reaching a maximum of about 2 second-feet just before the failure. This leakage was always clear and occasioned no concern for the stability of the dam. One interesting photograph captures the development of a sinkhole on the downstream face of the dam. While the location of this sinkhole (see Figure 14) is not known, the photograph suggests that it is near the right abutment, above the downstream coarse rock toe berm or blanket drain, or possibly over the abutment drain system at the upstream end of the blanket drain system. This feature provides a strong indication that the rockfill used in the downstream drain system in the dam did not adequately filter the overlying embankment materials.

As previously noted, the downstream shell included a drainage system and that system commenced flowing clear water at the first reservoir filling. Just prior to the failure, when the reservoir was nearly full, the total flow concentrated in the downstream toe drain was estimate at less than 1 second-foot. Careful investigations of the dam during the two weeks prior to the failure disclosed no sign of moisture appearing on the surface of the downstream slope.

As water rose above its previous high level on August 12, 1923, these was some tendency to settlement on the upstream slope at the west end of the dam following the old crack. New, fine cracks opened up there. The water level was gradually raised to elevation 4915 on August 22, when an unusual rain caused the reservoir to rise to elevation 4922 within 2 feet of the spillway crest by 3 p.m. of August 23, 1923, just before the failure.

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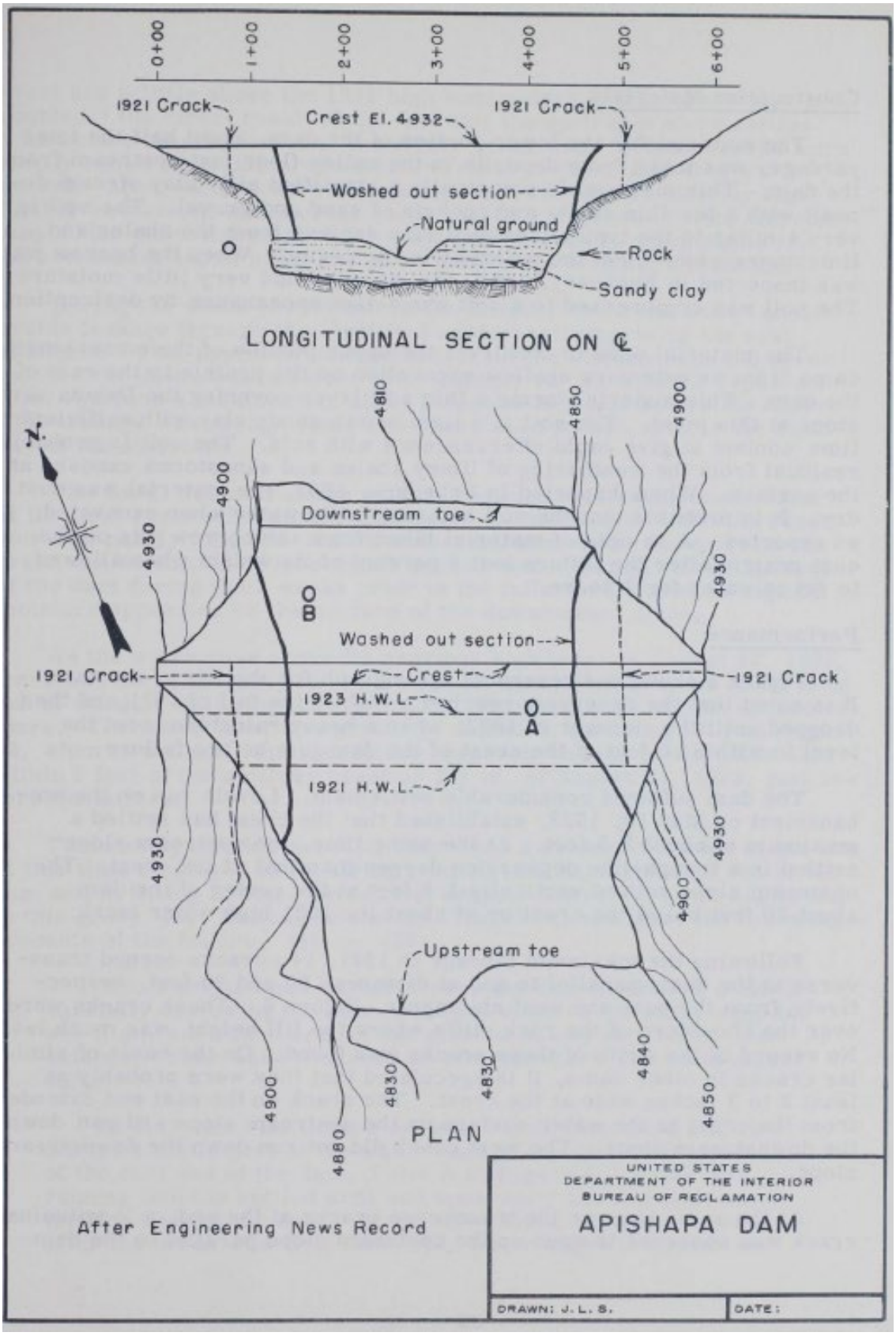


Figure 13. Apishapa Dam, Summary of Data (Sherard, 1953, Figure 6).

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Fortunately for describing the failure, there was a crew of workers at the site at this time employed in repairing a small leak near the west end of the dam and infilling in some settlement areas. There were two engineers in this group, Mr. Dallimore and Mr. Knowlton. They gave the following accounts of the failure (Engineering News Record, vol. 91, 1923). See Figure 15, which was taken from the ENR report:

> "Mr. Knowlton had waked across the dam at about 1 o'clock carefully observing the dam crest and down to the water edge to see if any additional cracks had started or any settling or subsidence. None were observed by him and there was no apparent danger.
>
> At 2:45, Mr. Dallimore made a trip of inspection and observed a settlement at the water edge at a point 100 feet west of the east end of the dam (Point A on Figure 15). Water was running into the settled area and apparently passing downward into the body of the dam. At the same time, or a few minutes after the break was first observed, water began to appear on the downstream slope some 200 feet west and 30 feet below the crest (Point B on Figure 15)."

This figure was developed from a photograph supplied to Mr. Mann by A.J. McCune (former state engineer of Colorado). To the approximate dimensions and dashed line showing the sub-horizontal crack through the dam, Mr. Mann added the stationing and elevation lines shown on this figure. This photograph was published in ENR, September 13, 1923: Vol. 91, No. 11).

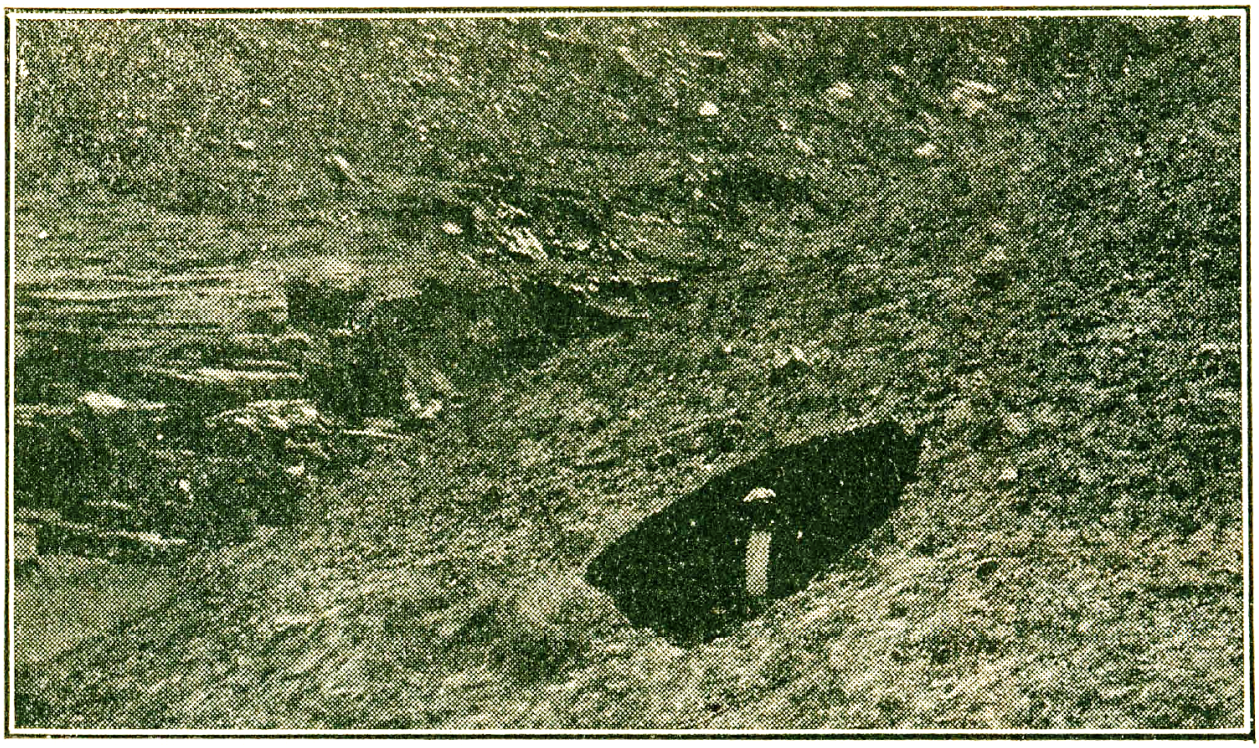


Figure 14. Sinkhole in Downstream Face of Dam near the Right Abutment (ENR, 1923; exact location not known)

A separate photograph from the record, likely taken some time between 2:45 and 3:00 pm shows seepage discharging from the downstream face of the dam near the mid right abutment area on Figure 16. A large quantity of seepage can be seen discharging from two locations near the lowest elevation of the sub-horizontal crack. In addition, it appears that this photograph shows a smaller amount of seepage discharging from a location higher up on the left (west) abutment near or at the location of the developing transverse crack that initially occurred in 1921. Sherard notes that the transverse crack in the left (west) abutment of the dam only develop along the upstream slope and into the dam crest in 1921. If this seepage did develop, it is a sign that the additional 1923 reservoir loading that caused the sub-horizontal crack to develop, also may have caused the transverse crack to develop completely through the dam. Either crack, or a combination of these two cracks certainly would have

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provided the flow necessary for the concentrated leak to initiate erosion in the dam that progressed rapidly to a full breach development. The limits of the breach section of the dam did not extend to the original transverse crack in the right abutment (see Figure 13, discussed further below) suggesting that the breach developed from flows through the sub-horizontal crack. If the sub-horizontal crack had not developed, it is possible, even likely that a breach would still have developed through the dam at the location of the transverse crack.

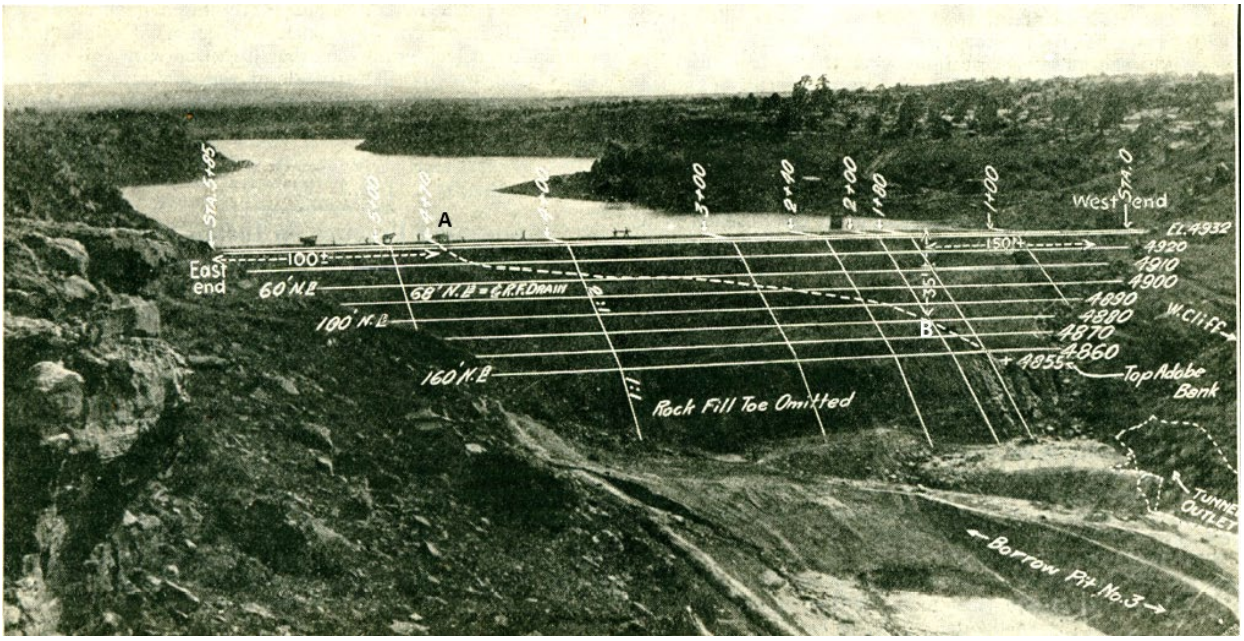


Figure 15. Photograph describing the locations where seepage was observed prior to dam failure (ENR, 1923).

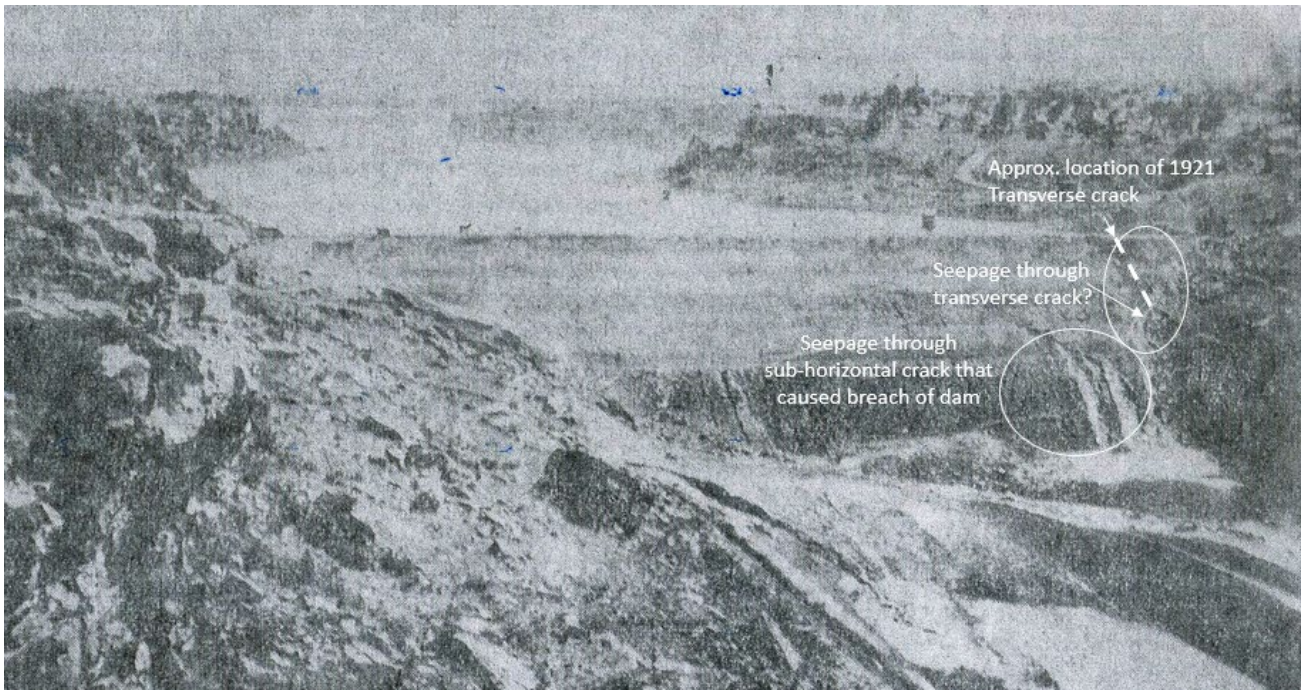


Figure 16. Photo showing large seepage flows from the sub-horizontal crack and also a location further up the left abutment at the possible transverse crack location that first occurred in 1921 (photo modified from Fowler Historical Society).

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The flowing water rapidly ate back on the fill, diagonally up the downstream face of the dam and finally to a point immediately opposite the first observed break in the water side. Desperate efforts to choke the break made by the workmen; among other things, a steel tank, 6-feet in diameter and 20 feet long, was rolled into the hole. It held an instant and passed through the hole.

In spite all the efforts to stop the inflow, it increased very rapidly, and it soon became apparent that the water entering the east end was the same water coming out of the downstream slope at the west end. A caving in of the top soon exposed the diagonal flow through the dam. The workmen left the dam at 3 p.m. when the "flow was beyond control."

By 5 p.m. the reservoir was empty and only short sections of the embankment at each abutment were still standing (see Figure 13). Fortunately, no loss of life occurred in the resulting flood.

The Engineer New-Record (1923, p. 422) contained an interesting description of the appearance of the portions of the embankment remaining after the failure provided by Messrs. B. M. Jones, and O. N. Floyd. The east abutment had an estimated 5,000 cubic yards of embankment materials that were left in the lower half of the remaining dam.

> "Wherever this material (soil 1?) was found, it appeared to be damp throughout. The light brown material in the upper part of the dam was very dry, so dry that whenever a small portion of it would fall off the nearly vertical exposed face and slide downs to the river below, a noticeable cloud of dust would rise. Each rolled layer was plainly visible. The top 2 or 3 inches of each layer was very hard somewhat resembling soft shale. The rest of the layer, while fairly compact, could readily be dug out with a stick. At one place a quantity of this material had caved and fallen on a dry ledge of rock. It resembled road dust except for the small amount of gravel and slabs of the hard layers. In walking over this dry material, pulverized by the fall, one's feet would sink into it 6 to 8 inches. This material apparently gave a very impervious fill. On the upstream slope of each exposed cross-section the line to which the water had penetrated could be followed for a distance down of 20 to 25 feet in elevation from the water surface. At the lower depth the water had gone into the fill only 5 or 6 feet. Aside from some wet spots where leaks occurred from seams in the canyon walls, this upper part of the dam appeared to have remained remarkably dry."

Sherard provided the following summary comments on the failure:

> "All the men who were associated with the dam agree that the failure started with the formation of a nearly horizontal crack which extended clear across the upper portion of the embankment. When first observed, this crack intersected the water surface at a point 100 feet from the east end and dipped slightly down toward the west end, intersecting the downstream slope 35 feet below the crest, 150 feet from the west end. There can be little doubt that the crack had considerable initial width before the water broke through the upstream slope. A few minutes after the water was observed entering the crack of the upstream slope at the east end, the flow through the crack emerging on the downstream slope was estimate at 2,000 second feet (information from Mr. H. H. Knowlton of Fowler, Colorado).
>
> The crack pattern and the greater settlement of the upstream slope than the crest supply clear evidence that the lower portion of the embankment settled more that the upper. The very dry and rigid upper portion undoubtedly tended to arch the 300 feet across the valley and could not settle with the lower portion resulting in the crack and subsequent failure.
>
> The role played by the soluble salts in the failure is very problematical. Colorado State Engineer, M. C. Hinderlider, believes that their action must have contributed in some degree. The observation of Messrs. Jones and Floyd that the water had penetrated the embankment only 5 feet at a distance of 25 feet below the water surface at the time of failure demonstrates that the salt content does not have an immediate radical effect on the permeability. It seems improbable that the large settlement of the lower portion of the dam, which was constructed of stream deposits, was due, in any large degree, to the solution of salts in the soil. It seems probable that water in a river running through a region where the soils have a high percentage of soluble materials would soon be saturated whereupon it could no longer dissolve soluble salts from an earth dam constructed of the river deposits. It is believed that the large settlements which occurred, and the subsequent cracking were due primarily to saturation of the initially dry embankment and the very steep rock abutments."

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V. CRACKING IN EMBANKMENT DAMS – HISTORY AND EVALUATION

Relatively shallow, isolated cracks occur in the upper/outer portions of embankment dams made of clay soils due to several conditions including:

1) Cycles of wetting and drying, and
2) Expansion and contraction associated with temperature variations particularly very cold temperatures.

This type of cracking is typically not serious as the depth of the cracks do not typically reach levels of water storage behind the dam providing a mechanism for initiation of a dam failure mode. Routine monitoring for, and rehabilitation of these cracks are a standard best practice for operation and maintenance. They are usually less than 1 inch wide, less than 2 to 5 feet in depth, propagate in various directions, and occur especially where the embankment crest and slopes lack healthy grass or earthfill cover.

Larger, well-defined cracks may indicate a more serious dam safety concern or problem. These types of cracks can be longitudinal and/or transverse to the axis of the embankment depending on the pattern of settlement within the dam and in the foundation beneath the dam. The occurrence of, and concern for cracking likely begun shortly after the practice of compacting the embankments improved with mechanized equipment and became a best practice in the late 1800's.

Compacting the embankment materials provided several advantages to the performance of the embankment dams including:

1) decreases in magnitude of settlement that would occur,
2) increases in the shear strength of the embankments and corresponding increase in stability,
3) reduction in embankment permeability and associated reduced seepage losses.

Compacting embankment materials also increased their stiffness and increased the potential for brittle cracking under reduced amounts of settlement. Watching for cracking, particularly during first filling of the reservoir was likely a best practice, as evidenced by the concern for and treatment of cracking that occurred at Apishapa dam during the first partial filling in 1921, and during the full reservoir filling when failure occurred as previously described in this paper.

Cracking of embankment dams were first described as a significant dam safety concern by Casagrande is his notes presented to the Boston Society of Civil Engineers in 1950. Undoubtably, Sherard's research of embankment dam cracking in 1953 was significantly influenced by Casagrande's concern.

The issue of cracking of embankment dams emerged sometime in the late nineteenth century and into the early 1900's as previously described, likely the result in efforts to compact embankment materials. The cracking at Apishapa dam represents one of the first known and documented cases of embankment cracking that became the focus of interest, research evaluation, communication of cracking hazards. A list of the known early failures or incidents involving cracking as outlined by Sherard (1953) is shown below:

- Lake Francis Dam, CA, 1903 (failure)
- Apishapa Dam, CO, 1923 (failure)
- Scofield Dam, UT, 1928 (incident)
- Rector Creek Dam, CA, 1947 (incident)
- Kern Dam, OR, 1948 (failure)
- Wister Dam, OK, 1949 (incident)
- Masterson Dam, OR, 1950 (incident)

While Sherard's early research noted the cracking concern as being associated with silts and silty clays, by 1969, the scope of cracking concern had expanded to all types of soils used in dam construction as illustrated in the following quote by Casagrande (1969):

> "It is not possible to prevent entirely the formation of substantial tension zones and transverse cracks in the top of the dams in the vicinity of the abutments, no matter what materials we use in the dam. Therefore, we must defend ourselves against the effects of cracks."

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The first methodology attempting to develop predictive models for cracking were develop by Leonard and Narain in 1963. The U.S. Army Corps of Engineers (USACE) invested in significant research related to the use of finite elements methods (FEM) for investigating stresses and the potential for cracking in embankment dams from about 1968 through 1974 (See for example Covarrubias, 1969; Kulhawy et al, 1969; Casagrande and Corarrubias, 1970; and Lefebvre and Duncan, 1974).

McCook and Grotrian (2011) describe Sherard's study of cracking and related seepage incidents in SCS (Soil Conservation Service, now the National Resource Conservation Service (NRCS)) dams (Sherard, 1972). They note that Sherard enlisted the assistance of Professor J.M. Duncan, and Dr. G. LeFebvre (University of California Department of Civil Engineering) in performing finite element analyses in an attempt to explain some of the mechanisms of dam cracking failures he had seen. The connection between the work published by Lefebvre and Duncan in 1974 and information on the FEM interests of Sherard described by McCook and Grotrian (2011) is not clear. It is possible that LeFebvre and Duncan could have used the models they were developing for the USACE to perform some analyses using the embankment properties Sherard found for the SCS dams where cracking occurred. As discussed further below, McCook and Grotrian performed FEM analyses of a low embankment dam on a compressible foundation in an attempt to reproduce the work of LeFebvre and Duncan using SIGMAW to analyze embankment stresses associated with settlement along the axis profile of a dam that could lead to cracking. Their studies assumed a relatively stiff embankment was being constructed on a compressible foundation with a relatively steep rock abutment (1.5H:1V). An example of their results compared to the early results by LeFebvre and Duncan they attribute to Sherard's request is shown on Figure 18.

A summary of notable work and publications related to cracking up through 1974 is provided below:

- Casagrande, 1950
- Sherard, 1953
- Sherard et al, 1963
- Leonards and Narain, 1963
- Bertram, 1967
- Covarrubias, 1969
- Kulhawy, Duncan and Seed, 1969
- Casagrande and Covarrubias, 1970
- Sherard, 1973
- LeFebvre and Duncan, 1974

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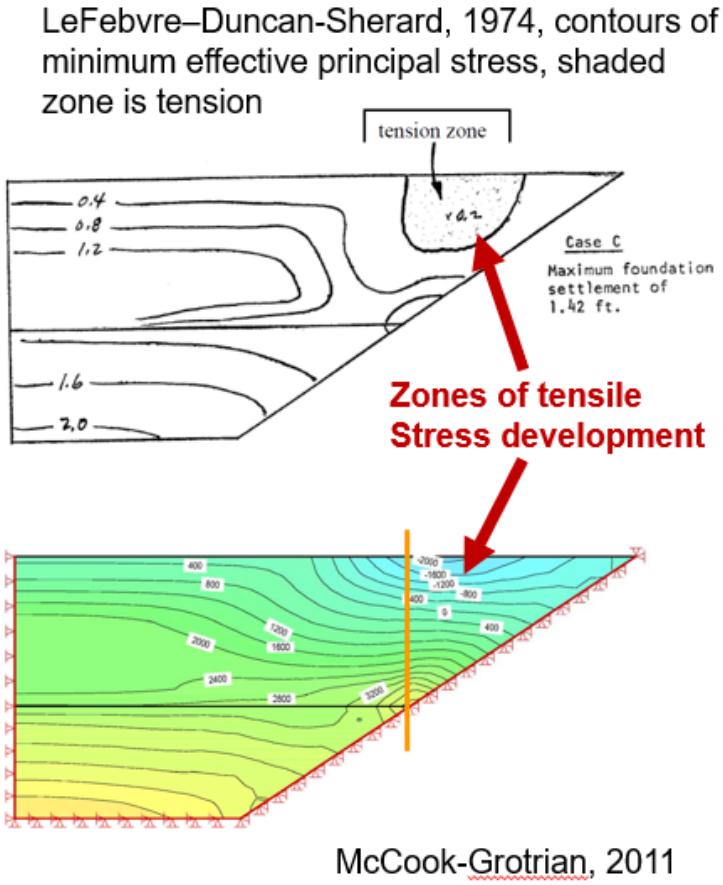


Figure 18. Comparison of tensile zone prediction between 1974 and 2011 FEM analyses of low embankment dam on compressible foundation.

At the request of the SCE, McCook noted (personal communication, 2013) Sherard later developed a simplified method for screening SCS dams for cracking potential base on observed “deflection ratios” from cracking failure case histories. Deflection ratios (DR) were estimated based on the measures vertical settlement (ΔV) over a distance (ΔL) along the axis of the dam:

$$DR = \Delta V/\Delta L. \qquad (1)$$

Sherard provided the following guidance on deflection ratios and cracking hazard (McCook and Grotrian, 2011):

DR >= 0.005 (.5%) dangerous and merits preventive measures for large, high hazard dams.
DR >= 0.01 (1%) dangerous for small and moderate hazard dams, likely cracking
DR >= 0.02 (2%) very likely cracking.

Cracking and/or hydraulic fracturing associated with vertical settlement of the dam axis that can lead to transverse cracking through the dam and initiation of concentrated leak potential failure modes has continued to be investigated since this work by various investigators in the late 1960’s through mid-1970’s as summarized in the paragraphs below. It is important to note that certain loading conditions such as earthquakes or settlement of the shells of dams can result in both vertical and horizontal deformations that can also lead to cracking. This is a special study case that requires consideration of the potential for tensile stress development under three-dimensional (3D) deformation patterns; a difficult and tedious problem that is more complicated than the two-dimensional (2D) problem of cross-valley settlement induced transverse cracking. Cracking associated with 3D deformation of dams will continue to challenge dam safety engineers until more advanced methodologies emerge in the coming years.

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Until then, engineers with must consider cracking hazards based on 2D evaluation methods and use judgement in assigning cracking potential associated with 3D loading conditions and deformation patterns.

In 2008, Fell and his fellow researchers assembled a comprehensive list of dams that had experienced cracking and the types of cracking mechanisms that were observed. A summary of these findings is provided in Table 1.

Table 1. Summary of Known Cracking Mechanisms and Number of Study Cases Identified by Fell, et. al (2008).

Cracking Mechanism	No. of Cases	Percentage
Differential settlement, cross-valley	20	35
Differential settlement, cross-section	6	11
Differential settlement, foundation	4	7
Differential settlement, embankment staging	0	0
Desiccation cracking	3	5
Closure section	3	5
Total	36	63

The work by McCook and Grotrian in 2011 represented the first of several evaluations using modern FEM software to assess embankment stresses and the potential for development of stress reduction conditions associated with differential settlement that could lead to embankment cracking. One of their primary purposes was to re-evaluate the DR criteria that Sherard had developed for initial screening of SCS (NRCS) dams. Their modeling results suggested that the DR criteria that Sherard had developed could probably be relaxed if one considers the use of more flexible soils in the embankment.

Ferguson, Soudkhah, and Sossenkina, 2013 created a series of 2D profile models (under the upstream slope, centerline axis, and under the downstream slope) of the Isabella auxiliary embankment dam (CA) to assess the potential for cracking/hydraulic fracturing of the embankment or foundation materials surrounding an outlet works penetration. The auxiliary dam outlet works had been constructed on a stiff foundation soil while a layer of relatively compressible soil was left in place adjacent to the twin 10-foot by 12-foot cast-in-place outlet conduits below the embankment dam. Results of their analyses indicated that there was a low potential for hydraulic fracturing through the embankment materials under the crest of the dam. Example results for a relatively still embankment material under sequential construction is shown on Figure 19. The results also showed a potential for cracking of the embankment materials above the conduit during construction but that by completion of construction, cracking within the embankment would have closed as the minor principal stresses increased from the embankment loads.

The work completed and documented in the He, Fell and Song (2019) publication was intended to provide some basis for estimating the potential for embankment cracking due to vertical differential settlement under the centerline axis of a dam. They performed a detailed assessment of the 20 known case histories where transverse cracking of embankment dams occurred (see Table 2). A 2D model with a profile configuration similar to that modeled by LeFebvre and Duncan as well as McCook and Grotrian but considering an embankment height of 50m (164 feet), abutment step at various locations (0.6, 0.8 and 0.9 percent of the embankment height), alternative abutment slopes (25, 45, and 60 degrees), and both drained and undrained behavior of the embankment materials. Properties of the foundation materials were varied to create post construction total settlements ranging from zero to 1 percent (0.5m or 1.64 feet) of the embankment height in the model. Both drained and undrained strength parameters were evaluated: drained $\phi' = 20^{\circ}$ and $c' = 209$ psf (pounds per square foot), and undrained $\phi = 0^{\circ}$ and $c = 2880$ psf. The soils were assumed to be able to carry some tension in undrained loading up to 4177 psf. A cross section of the model is shown on Figure 20. An example of the analysis results showing zones in the dam where horizontal tensile strains developed is shown on Figure 21.

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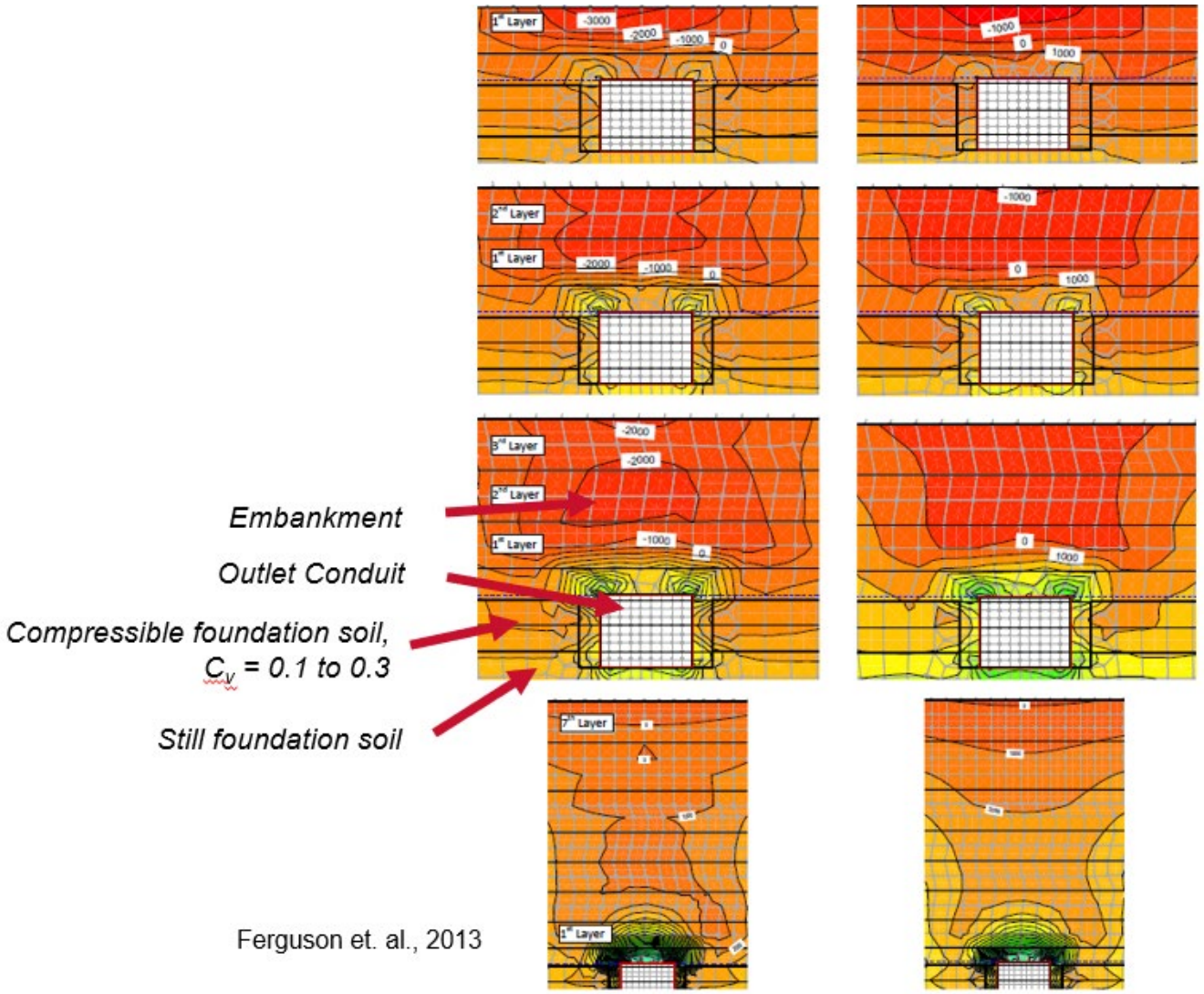


Figure 19. Example of Dam Axis FEM model showing effect of compressible foundation soils adjacent to outlet conduit on potential for cracking of embankment and hydraulic fracturing of foundation soils adjacent to conduit.

Results of the numerical simulations for an embankment on a compressible soil were summarized in a Table for use in estimating the depth of the tensile zone that would develop from the crest of the dam. The table from their paper that is applicable to the Apishapa dam is provided as Table 3 below. These analyses did not consider a step in the abutments and only included slope angels of 45 degrees (1H:1V). Therefore, they provide an approximation that must incorporate judgement.

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Table 2. Summary of Transverse Embankment Dam Cracking Case Histories (Table 8 from He, Fell, and Song 2019 publication)

Table 8. Case studies in which transverse cracking in embankment dams is described.

Dam	Country	Year constructed	Embankment type	Reference(s)
Rector Creek	USA	1946	Homogeneous earthfill	Sherard (1953), Leonards and Narain (1963)
Apishapa	USA	1919-20	Earthfill with rock toe	Sherard (1953)
Scofield	USA	1925-26	Earth and rockfill	Sherard (1953)
Lake Francis	USA	1899	Homogeneous earthfill	Sherard (1953)
Masterton	USA	1950	Earthfill with rock toe	Sherard (1953)
Kern	USA	1948	Earthfill with rock toe	Sherard (1953)
Portland	USA	1956	Homogeneous earthfill	Leonards and Narain (1963)
Woodcrest	USA	1953	Zoned earthfill	Leonards and Narain (1963)
Shell Oil	USA	1944-45	Homogeneous earthfill	Leonards and Narain (1963)
Stockton Creek	USA	1950	Earthfill with rock toe	Sherard (1973)
Boggy Creek	USA	1970	Homogeneous earthfill	Sherard et al. (1972)
Wister	USA	1946-48	Homogeneous earthfill	Sherard (1973)
East Branch	USA	1952	Zoned earthfill	Sherard (1973)
Virginia Ranch	USA	1961-63	Zoned earthfill	Sherard (1973)
Round Butte	USA	1962?-64	Central core earth and rockfill	Sherard (1973)
Duncan	Canada	1964-67	Zoned earthfill	Gordon and Duguid (1970)
Matahina	New Zealand	1964-66	Central core earth and rockfill	Gillon (2007)
Tullaroop	Australia	1958-59	Homogeneous upper, Zoned earthfill lower	Hunter, Fell, and McGrath (2007)
Buffalo	Australia	1964-65	Central core earth and rockfill	Newman and Foster (2006)
Red Willow	USA	1961	Homogeneous earthfill	US Bureau of Reclamation (2013, 2014).
Austrian	USA	1948?-50	Zoned earthfill	Harder, Bray, Volpe, and Rodda (1991); Pells and Fell, (2002, 2003); Forster and MacDonald (1998).

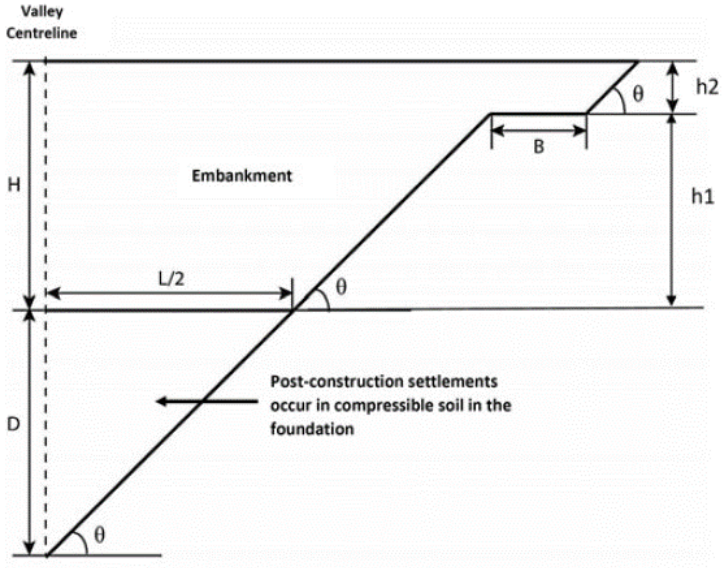


Figure 20. Cross-valley section model for embankment on a foundation with rock abutments and compressible soil in the valley showing definition of terms by He, Fell, and Song (2019)

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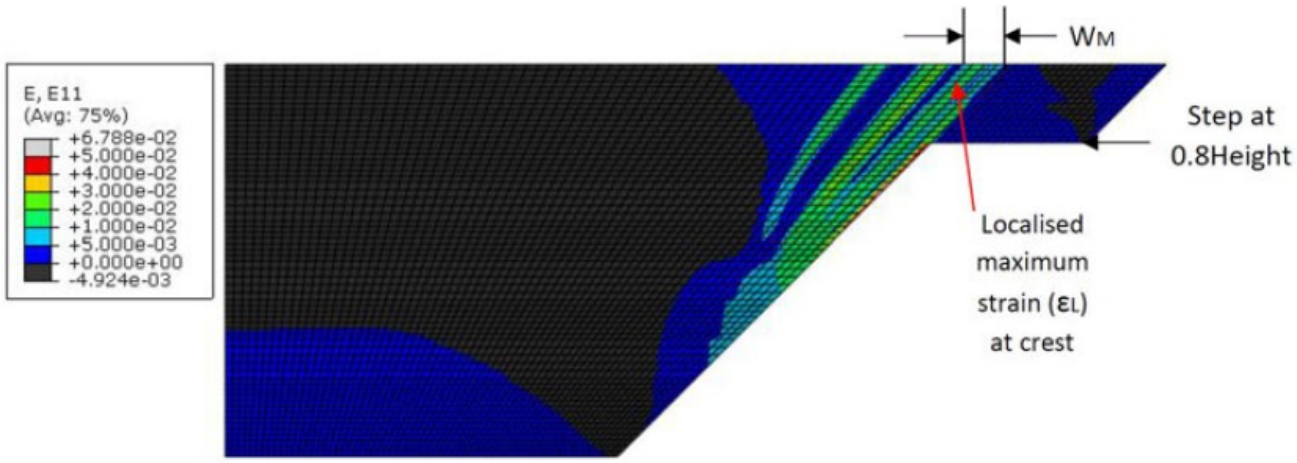


Figure 21. Example of He, Fell and Song analysis results – horizontal tensile strains for post-construction settlement.

Table 3. He, Fell, and Song numerical simulation results for embankment on valley filled with compressible soil with post-construction settlement.

Slope of step in foundation (α) (degrees)	Post-construction settlement (% height)	Localised tensile strain at crest (%)		Accumulated extension at crest (mm)[a]		Depth tensile zone from crest (% height)	
		Drained analysis	Undrained analysis[b]	Drained analysis	Undrained analysis[b]	Drained analysis	Undrained analysis[b]
45	0	0.7	0.1	20	2	3	5
	0.25	1.1	0.8[d]	30	24[d]	3	13[d]
	0.5	1.7	1.5	70	46	3	20
	1.0	2.0	2.5	110	75	3	20
30	0	0.7	0.1	15	2	3	5
	0.25	0.9	0.6[d]	25	19[d]	3	13[d]
	0.5	1.2	1.1	60	35	3	20
	1.0	1.6	1.6	100	105	3	20
15	0	0	0	0	0	3	3
	0.25	0.9	0.1[d]	25	2[d]	3	12[d]
	0.5	1.0	0.2	35	4	3	20
	1.0	1.0	0.2	40	4	3	20

Effects of post-construction settlements within the foundation on stresses and strains within the embankment.
[a]Undrained analyses are for upper 20% of embankment modelled with undrained strengths, with 40 kPa limit on tensile strength.
[b]Accumulated extension (Δc) = (localised tensile strain, ε_L) × (width of tensile strain zone, W_M).
[d]Figures obtained by linear interpolation.

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Looking back at the design of the dam, a simplified evaluation of the as constructed conditions, and the settlement and cracking that occurred at Apishapa dam shows the following important characteristics relative to cracking:

- abutment slope angles of about 64 degrees,
- steps (changes in abutment slopes to shoulders) occurring between 50% and 65% of the abutment height in both the right and left abutments, respectively,
- total settlements of up to 2.5 feet with estimated maximum vertical settlement of up to 2 feet over a desistance of 60 to 80 feet over the steep portions of the abutment slopes during the 1921 partial filling,
- estimated deflection ratios of 0.025 to 0.03 (2.5 to 3%) for the partial filling in 1921,
- significant settlement of the upstream slope during the 1921 partial filling with some likely upstream deflection of the dam crest that was not captured in the crest surveys (hence the DRs from the crest settlement measurements would have underestimated the movements that contributed to the transverse cracking events).
- No information is available to assess the potential for the sub horizontal cracking that occurred due to settlement of the foundation and lower portion of the dam that led to the concentrated leak failure mode that developed causing a breach of the reservoir.

The estimated deflection ratios summarized above contributed to Sherard's recommendation that DR >= 2% would likely result in cracking. Considering the other summary characteristics listed above, the numerical simulations by He, Fell and Song would suggest significant tensile zone development in each abutment, cracking depths of up to 20 to 30 feet, and crack top widths possibly as wide as 3 inches or more. It is interesting to note that the cracking depth predicted by the He, Fell and Song simplified cracking evaluation method would not have reached the water storage level in the reservoir consistent with the observed behavior during the 1921 cracking incident. However, this crack depth would have been below the reservoir level in 1923, and consistent with the photographic evidence of seepage through the left abutment transverse cracking highlighted shown on Figure 16. Perhaps most importantly, the simplified methods for predicting cracking of the dam even today do not provide a bases to estimate the occurrence of the sub horizontal crack that developed and the failure that ensued. To better understand that hazard, a full 3D model of the dam with appropriate foundation and embankment material properties would be necessary.

VI. CONSEQUENCES OF THE DAM FAILURE

Apishapa Dam had a height of approximately 115 feet above from streambed up to the dam crest, with about 8 feet of freeboard (C-153 construction plans). The reservoir stage and volume on August 23, 1923, prior to dam failure were reportedly 2 to 3 feet below the spillway and about 18,500 acre-feet (Engineering News-Records, August 30, 1923).

The dam failure occurred during daylight hours and under the watch of the dam's builders, between about 1 p.m. and 5 p.m. on August 23, 1923. Importantly those at the dam were able to get warning to downstream towns.

The Engineering News-Record (1923) reported a breach flow of about 60,000 cfs at the Apishapa River's confluence with the larger Arkansas River, approximately 30 miles downstream from the dam. The flood stage was reported as being 3 feet below the crest of the catastrophic 1921 Arkansas River flood. Engineering News-Record reported a peak stage from the Apishapa Dam failure floodwave at 14 feet below the 1921 flood crest at La Junta, Colorado, approximately 60 miles downstream from the dam.

Newspaper articles from 1923 reported a massive wall of water descending on Olney Springs, Manzanola, Rocky Ford and La Junta, Colorado (see map in Figure 22) (The Herald Democrat, August 23, 1923). The Herald Democrat said the Arkansas River was already bankfull from recent rains and caused fears of serious flooding of the towns and concern about a cascading failure of Hock Ford Dam at Rocky Ford. They reported that Apishapa Dam had been watched closely on August 23rd after heavy rains filled the reservoir and that one end of the dam washed out around 3 o'clock. Prompt warnings were reported as having saved large loss of life, according to local officials. They cited a 27-foot wall of water 26 miles upstream from Manzonala and headed towards the town.

Telephone communication was lost and cut the area off from further reports to the press after the floodwave hit. Low-lying areas were flooded, according to the county sheriff. Railway officials reported a 15-ft wall of water, one mile wide threatening the Sante Fe railroad bridge and Sante Fe highway. Farmhouses ("scores of houses"),

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buildings and implements (Fort Collins Courier, August 23, 1923), and livestock were carried along by the advancing floodwave. Again, the papers credit prompt warnings with averting human life loss.

The Georgetown Courier (Sept. 1, 1923) reported an 86-foot wall of water from the dam and a community "recovering from the havoc wrought by flood waters". The papers at the time estimated damages at around $600,000 (including the dam itself, built for around $250,000) (The Dolores Star, Sept. 7, 1923). Engineering News-Record (August 30, 1923) put the financial loss at $1,000,000. We estimate present worth of about $18,000,000, based on historical inflation rates from 1923 to 2023 (https://www.usinflationcalculator.com/). The Dolores Star (Sept 7, 1923) reported that the dam failure flood left several hundred persons homeless.

USGS Report 85-560 (Costa, 1985), Floods from Dam Failures, reported a peak breach flow of 242,000 cfs from Apishapa dam failure, citing an anonymous source. Correlation to the water depth of 105 feet, reservoir volume of 18,500 acre-feet, and HxV factor compared well with dam failure floods from other historical dam failures (see Figure 23, Apishapa data point circled in red). Costa (1985) reported an attenuated peak breach flow of 50,100 cfs at 57 miles downstream, likely somewhere near La Junta, Colorado.

As part of our research, Apishapa Dam breach parameters were estimated using Froehlich (2008) method. For the reported depth of water of 106 feet, dam height of 115 feet, and reservoir volume at failure of 18,500 acre-feet, Froehlich (2008) estimates a breach width of 231 feet and breach formation time of 0.76 hour, for an erosion rate of 2.9, where rates between 1.6 and 21 are considered reasonable. Reports from 1923 describe seepage exiting the dam face, with the water level falling slowly for an hour, then more and more rapidly until the reservoir had lost all 18,000 acre-feet of storage. The dam break occurred at 3 p.m. and at 5 p.m. the reservoir had drained (Engineering News-Record, August 30, 1923).

We simulated the Apishapa Dam failure flood using FEMA's DSS-WISE Lite web tool. The simulated flood inundation area is shown in Figure 24. We estimated the reservoir surface area at the time of failure as approximately 640 acres, in order to provide good agreement with the storage capacity curve that DSS-Wise generated from its DEM terrain model. This results in a maximum storage volume of 25,500 acre-feet at dam crest elevation of 4939 feet. The dam failure was simulated at a water surface 11 feet below that or elevation 4928 feet and 18,500 acre-feet, corresponding to the reports of the August 23, 1923 failure condition.

DSS-Wise estimated a peak breach flow of 313,000 cfs at the dam; 47,000 cfs at the Arkansas River confluence; 29,300 cfs at the Manzanola; 17,700 cfs at Rocky Ford; and 10,700 cfs at La Junta, Colorado, approximately 60 miles downstream. Our simulated breach flow is about 30 percent higher than Costa (1985), whereas the attenuated peaks downstream are considerably lower than the 50,100 cfs reported at La Junta and somewhat below the 60,000 cfs reported at the Apishapa-Arkansas River confluence. We note that the newspaper articles at the time reported concurrent flooding on the Arkansas River due to recent rains. Also, the higher peak flow at the dam may be explained by our use of Froehlich's breach formation time (0.76 hours), which seems to be faster than the reported failure time of about 2 hours.

Nevertheless, the simulation is reasonable for talking purposes and relative to the many sources of uncertainty here. DSS-Wise simulates depth x velocity (ft^2/s) values of over 500 immediately below the dam site, and values still over 100 where the Apishapa River canyon opens onto the farmed floodplain 25 miles downstream from the dam. Utilizing Reclamation Consequences Estimation Methodology (RCEM), assuming adequate warning, DV of 100 corresponds to fatality rates between 0.0002 and as high as 0.05, the latter of which lies at the data point from the 1921 Arkansas River flood in Pueblo, just upstream from the dam.

It truly seems fortunate that no lives were lost to the Apishapa Dam failure. We can only assume that many lives could have been lost had the dam failed at night or in other adverse circumstances, or if the downstream area near the dam was further developed. Based on the report of several hundred people being displaced from their homes, we can assume a population at risk and fatality rates around 50%, for little to no warning conditions, could have resulted dozens of lives being lost. Like many other successful interventions in historical dam failures, this demonstrates the effectiveness and need for early warning, even with limited communications in 1923.

As described in the introduction, two irrigation companies began the Apishapa Dam irrigation project -- the Omer Irrigation Company and the Van Skike Irrigation Company. They later merged and formed the Apishapa Consolidated Irrigation Company. Member/owners of ACIC mortgaged their individual properties to help finance the dam. Eventually they ran out of money and were forced to issue bonds. After the dam failed the bond company foreclosed on all 80 properties and would not allow individual landowners to buy back their property. The bond holder would only sell all 80 properties in one mass sale. Most of the original owners walked away but a few were able to lease their land and did so. Some 40 years later, descendants of a few (4 or 5) of the original landowners

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were able to arrange a purchase of the property. They were also the individuals who had been leasing the land from the bond holders (Larry Green, historian, email dated March 31, 2020).



Figure 22. Newspaper articles from 1923 reported a massive wall of water descending on Olney Springs, Manzanola, Rocky Ford and La Junta, Colorado (Google Earth).

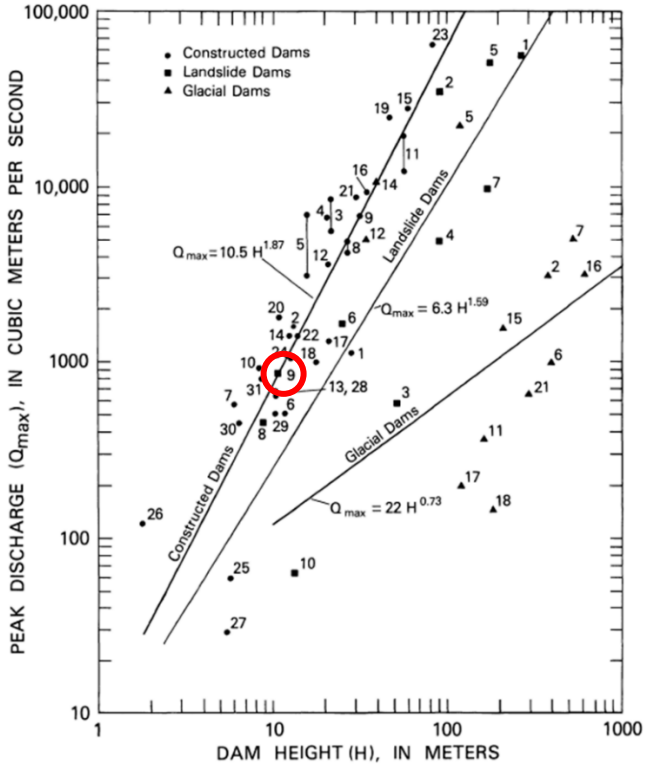


Figure 23. USGS Report 85-560 (Costa, 1985), Floods from Dam Failures, reported a peak breach flow of 242,000 cfs from Apishapa dam failure. Correlation to the water depth of 105 feet, reservoir volume of 18,500 acre-feet, and HxV factor compared well with dam failure floods from other historical dam failures (Apishapa data point circled in red).

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Figure 24. Apishapa Dam breach inundation area simulated using FEMA's DSS-WISE Lite web tool.

VII. KEY LESSONS LEARNED

1. **The design of Apishapa Dam was consistent with the best practices that were occurring in the U.S. for embankment dams in the 1918-1921 timeframe.** However, this case history provides insight into potential failure modes (PFMs) associated with inadequate empirically based design provisions that should be carefully considered when analyzing or conducting risk analyses on embankment dams designed and built within the same timeframe. The inadequate but empirically based provisions that were inadequate include a) moisture control for embankment materials, b) embankment zoning, c) settlement of embankment and foundation materials, d) filter design for the drainage provisions in the downstream shell of the dam, and e) lack of chimney filter/drain system.

2. **The Apishapa Dam failure is a key case history for bringing to light the potential for cracking within embankment dams and failure modes attributed to concentrated leak erosion (CLE) through embankment cracks.** This failure case history has been an important contributor to empirical and analytical methods for predicting the potential for cracking in embankment dams. This PFM was not recognized even when Sherard was conducting his investigation of Apishapa.

3. **Both transverse and sub horizontal cracking occurred during two separate reservoir loading conditions.** The transverse cracking in the upper portions of each abutment occurred first (1921) and happened under a partial reservoir filling to about 30 feet below the maximum storage level. Significant settlement of the upstream slope also occurred at the same time. Maximum settlement of 2.5 feet along the dam crest was measured which represents 2% of the maximum height of the dam. Transverse cracking does not appear to have contributed to the rapid failure of the dam that occurred in 1923. However, there is some indication that the transverse crack on the left abutment that had only partially developed across the crest of the dam in 1921 may have fully developed across the crest and downstream slope of the dam during the second reservoir filling that occurred in August of 1923. There may have been some seepage flows through that crack at the same time the sub horizontal crack developed. Flows through this transverse crack, if they occurred would have been capable of causing failure of the dam.

4. **The sub horizontal cracking that occurred during the second reservoir filling to within 2 feet of the spillway crest elevation was critical and was the loading condition that led directly to the failure of the dam.** This is an unusual crack configuration and the cause for the crack has not been fully established. It was likely caused by a combination of settlement within the foundation alluvium and lower portion of the

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embankment under the full reservoir conditions. The sub horizontal configuration suggests that some cross-valley arching of the upper portion of the dam may have occurred. The crack orientation is consistent with a principal stress path of the upper embankment materials transferring to the steep left abutment slope. The crack configuration also suggests that the foundation and lower embankment settlement was not uniform but concentrated under the left side of the maximum valley section of the dam.

5. **This case history strongly reinforces the importance of identifying and evaluating PFMs as part of design and first filling of a reservoir.** The initial filling of a reservoir is the first test that the dam will perform the function for which it was designed. A carefully managed first filling is crucial to the future success of a dam. According to Best Practices, "approximately two-thirds of all failures and one-half of all dam incidents occur on first filling or in the first 5 years of reservoir operation (Reclamation and USACE, 2019). Because the first filling of a reservoir is a critical phase in the life for the dam, it is vital for dam operators and engineers to have as much control over the first filling as possible allowing as much time as needed for appropriate surveillance, including the observation and analysis of instrumentation data. The first filling of a reservoir should be planned, controlled, and monitored.

6. **Forensic investigations are needed for major dam failures and incidents in order to determine the history of the contributing physical and human factors, and the culminating physical failure modes and mechanisms.** Without the ENR articles, Sherard thesis, newspaper clippings and local museums, much of this incident would have been lost to history. Mr. Mann was forthcoming with the dams community following the failure and authored at least one ENR article. There were eyewitnesses to the failure (and those that arrived on scene shortly thereafter) and downstream impacts, and their accounts were preserved. While a formal, independent forensic investigation made up of a team of recognized industry experts could have potentially developed and shared many lessons to be learned from this failure, possibly preventing future dam failures, without the reports that exist today, little would be known of this event.

7. **Timely warning and rapid public response are critical to saving lives during dam failures and incidents.** The historic record of dam failures clearly shows that no warning or evacuation before failure can have a fundamental influence on loss and survival outcomes for people located in inundation zones. If not for the observation and warning of the failure, life loss would have been likely. Like many other successful interventions in historical dam failures, this demonstrates the effectiveness and need for early warning, even with limited communications in 1923.

8. **Dam incidents and failures can fundamentally be attributed to human factors.** Human factors can include pressure from non-safety goals, such as delivering water, generating power, reducing cost, increasing profit, maintaining property values, meeting schedules, protecting the environment, providing recreational benefits, building and maintaining relationships, personal goals, and political goals. Provisions in the design of Apishapa Dam were progressive for the time and reflected awareness by the designer (Mann) of dam design and construction practices. In fact, many of the specifications and details on the drawings reflect approaches to design and for construction still in use today. However, design and construction technology available at the time was not sufficient to address several critical issues, some that directly led to failure of the dam, and others that would likely have led to failure or significant dam safety incidents over time. In addition, changes were made during construction that altered the design concepts and implementation of the specifications. It appears from some of the documents that Clair Mann did not agree with some of the changes. While the changes did not themselves appear to lead to failure, it does demonstrate how critical the Engineer of Record (EOR) position is and the constant pressure that exists between implementation of best practices and costs that continue to exist today.

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VIII. ACKNOWLEDGMENTS

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- Larry Green (Pueblo County Historic Society) and Doug Baxter (Pueblo Archaeology & Historical Society) for arranging a visit to the dam site.
- Fowler Historical Society for many images of Apishapa Dam and construction
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- Dam Failures and Incidents Committee

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Hales Bar Dam: Many Firsts and Many Lessons

ERIC PETERSON, PE, MANAGER DAM SAFETY GEOTECHNICS, TVA; ROBERT HATTON, PE, PROJECT MANAGER, STANTEC

ABSTRACT

In 1905, the Chattanooga and Tennessee River Power Company began construction of the first privately-owned dam in the United States to span a major, navigable river: The Tennessee. Hales Bar Dam was intended to alleviate notorious navigation issues in the Tennessee River Gorge downstream of Chattanooga and generate hydroelectric power. Construction was to take place at a site chosen for its topography; geologic investigation was considered unnecessary on the presumably sound Bangor Limestone. Eight years, five contractors, and 109 deaths later, the landmark dam and record-breaking lock facility were completed. A troubling death toll and leakage of up to 1,700 cfs, which equates to 10% of the average river flow, remained. By 1963, under the ownership of TVA, continued modification and remediation efforts of Hales Bar Dam were deemed impractical. The once record sized lock was insufficient for contemporary river traffic and despite extensive geologic mapping and geotechnical repairs, the leakage was nearly as significant as the leakage observed in 1913. Hales Bar Dam was replaced by Nickajack Dam six river miles downstream in 1967. Hales Bar contributed two generating units to the new dam, but the lock and spillway were demolished to clear the navigation channel. Its legacy helped shape current safety and site exploration practices and is a case worth studying for dam safety engineers looking to the future.

INTRODUCTION

The Tennessee River Gorge (Figure 1), regionally known as the Grand Canyon of the Tennessee River, is located downstream of Chattanooga, Tennessee. The Tennessee River Gorge includes 27,000 acres of land carved through the Cumberland Mountains by 27 miles of the Tennessee River [1]. Today, the Gorge is the fourth largest river canyon east of the Mississippi River and begins approximately five miles downstream of downtown Chattanooga, Tennessee and extends for 27 river miles to Hales Bar Dam Marina near Nickajack Lake.

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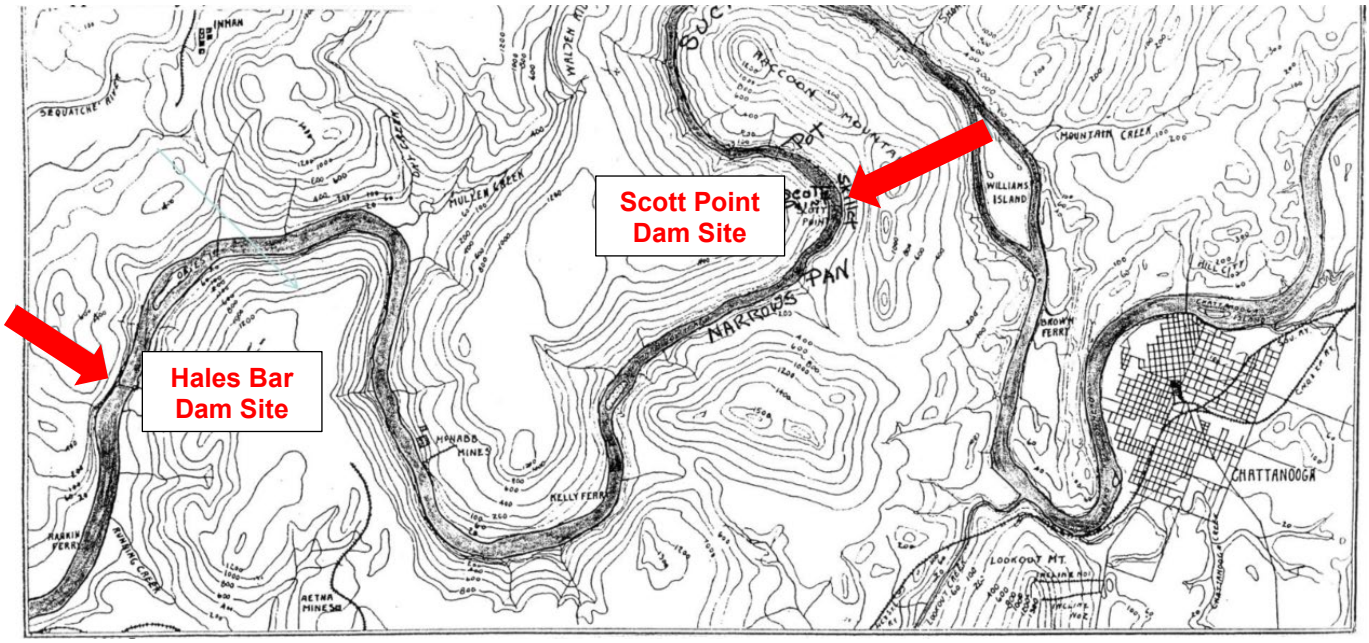


Figure 1. The Tennessee River Gorge Looking West Near Signal Mountain [1]

Prior to the construction of Hales Bar Dam, this stretch of the Tennessee River contained numerous vortexes referred to as “suck holes”. These “suck holes” developed where swirling currents formed in the turns of the Tennessee River that ultimately obstructed navigation. With congressional approval, the Chattanooga and Tennessee River Power Company formed and set out to construct a new dam to solve the perilous navigation challenges of this stretch of the river and to generate hydroelectric power. The new dam would be called Hales Bar Dam.

DAM SITE SELECTION

The Hales Bar Dam site was chosen in 1904 at the southwest end of the Tennessee River Gorge. Initially, the Chattanooga and Tennessee River Power Company selected a site known as Scott Point (Figure 2). However, it was determined that a dam at Scott Point would not inundate the lower gorge and the pool would impact the city of Chattanooga [2]. The Hales Bar Dam site was then selected at a location 33 river miles below Chattanooga. The Hales Bar Dam was ultimately selected solely due to the favorable topography at the site [2].



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Figure 2. Hales Bar Dam Site Selection [2]

HALES BAR DAM DESIGN

The design of Hales Bar Dam was considered cutting edge at the time. The design for the dam included a concrete dike overflow section that was 1,300 feet long, the highest single lift lock in the world, a significant powerhouse (60,000 horsepower), and an earthen dike that included a concrete core wall [2] (Figure 3). The design did not consider the site geology and assumed that the narrow channel at the dam site was indicative of more erosion resistant rock that would form the foundation of the new dam.

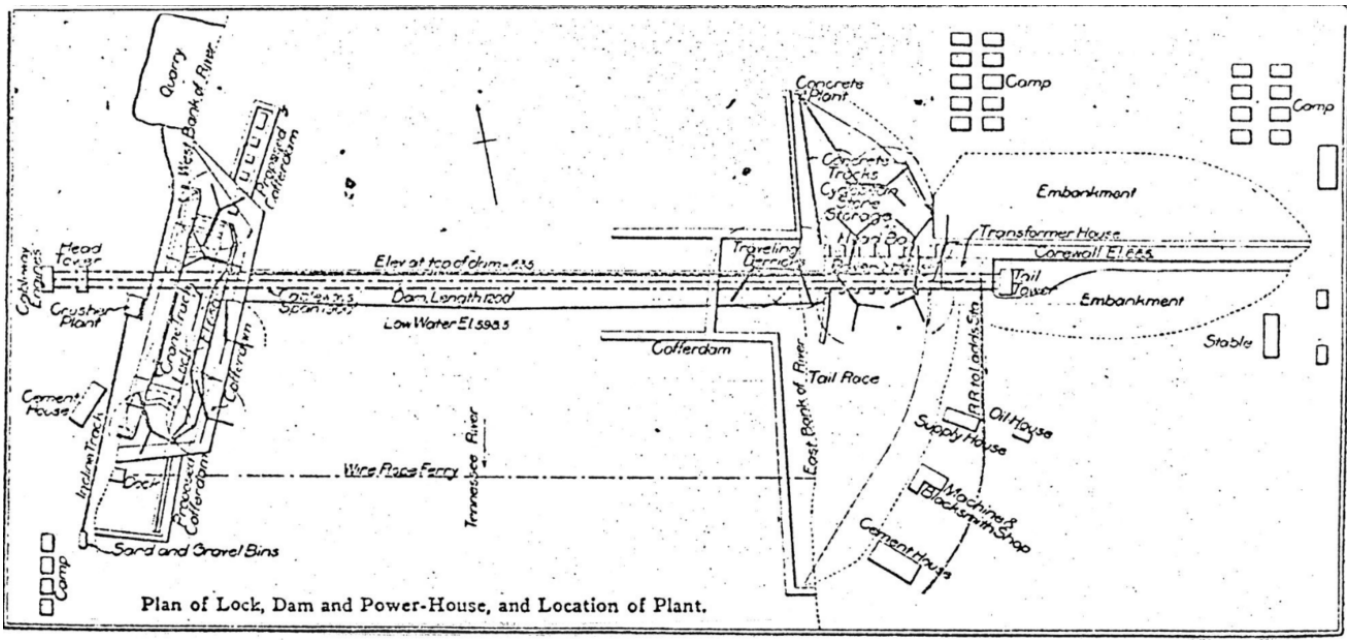


Figure 3. Hales Bar Dam Plan View [2]

The main concrete overflow weir portion of the dam was designed to be approximately 1,200 feet wide and consisted of a triangular section that included an interior galley (Figure 4) [2]. The concrete overflow weir section rose over 110 feet above the original riverbed.

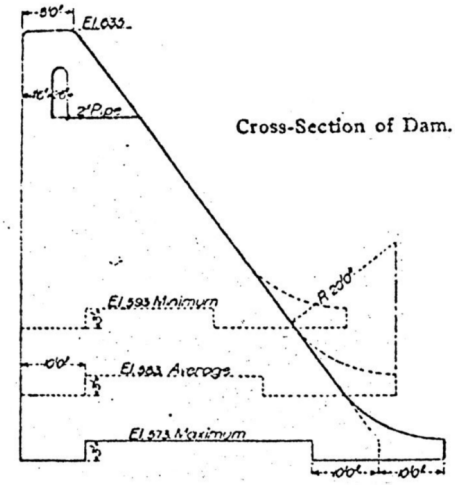


Figure 4. Concrete Overflow Weir Section [2]

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CONSTRUCTION

The W.J. Oliver Co. of New York began construction of Hales Bar Dam in October 1905. Oliver's original bid was $1.5 million for a two-year construction schedule. Mobilizing to the site with two steam shovels, Oliver constructed the 100,000 cubic yard earthen dike to a height of 75 feet around a concrete core wall that ranged in height from 7 to 85 feet (Figure 5) [2]. In an attempt to meet the schedule, a crew of about 500 men worked day and night, six days a week [3].

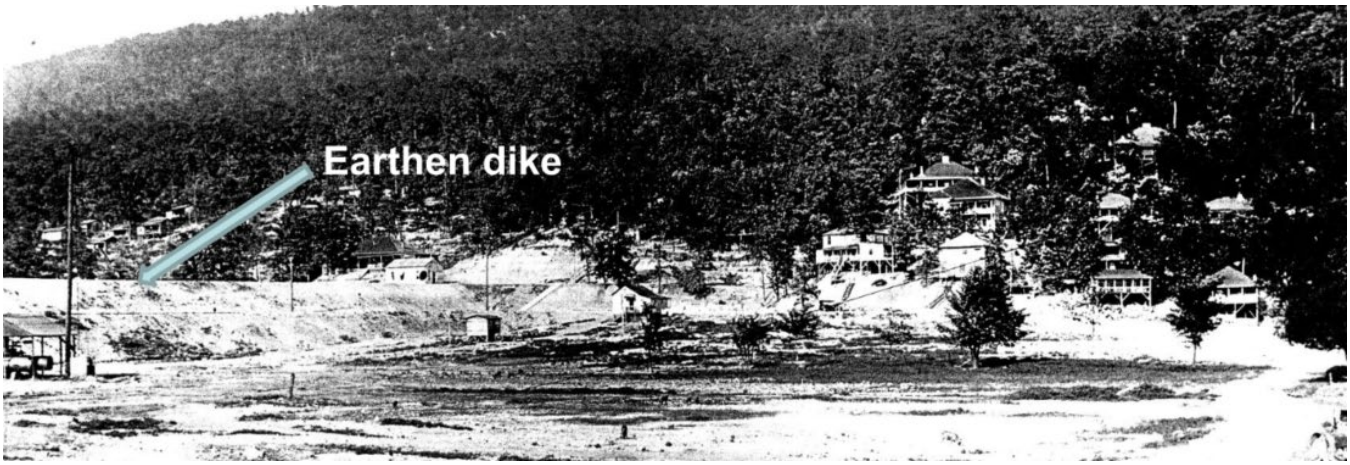


Figure 5. Earthen Dike Following Completion [2]

In 1906, Oliver attempted to construct the first cofferdam and was unsuccessful in keeping the area dewatered due to foundation leakage (Figure 6) [2]. Following the challenges of dewatering work areas, Oliver abandoned the job in November 1907.

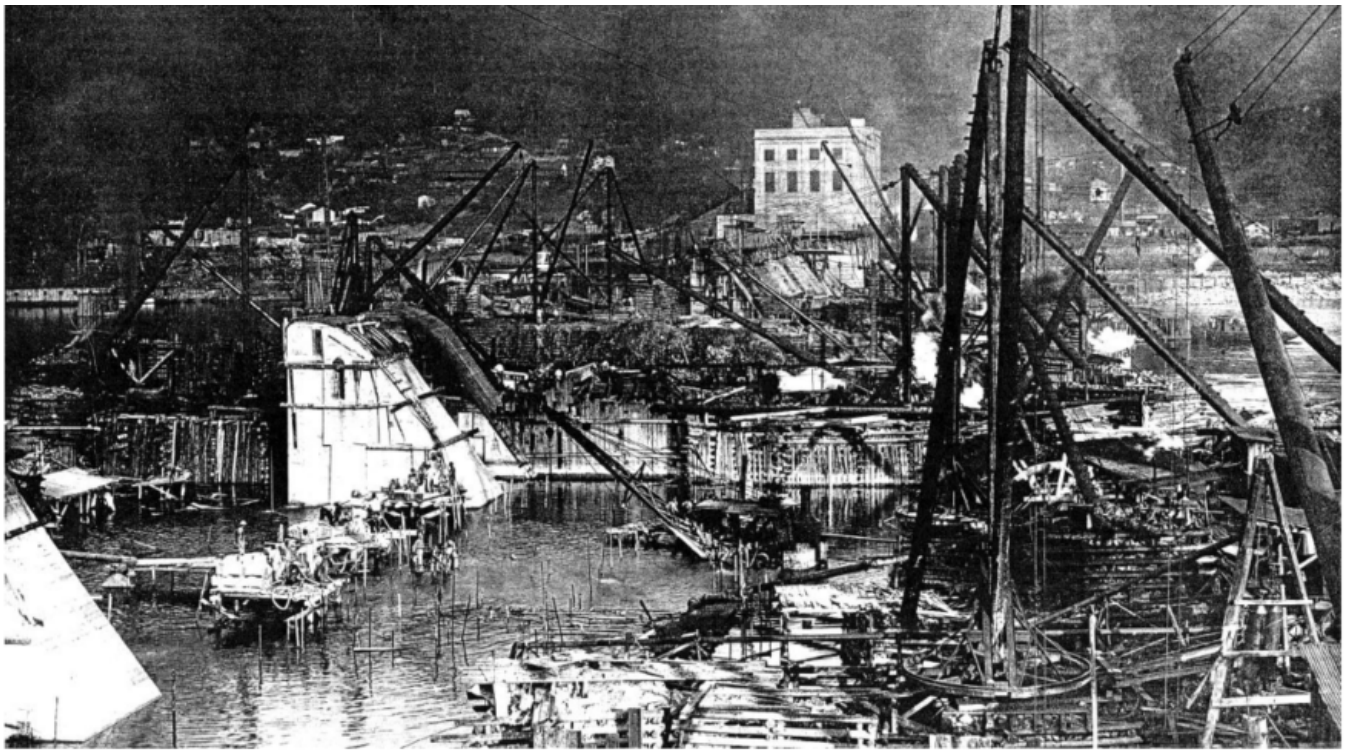


Figure 6. Flooded Cofferdam at Hales Bar Dam During Construction [2]

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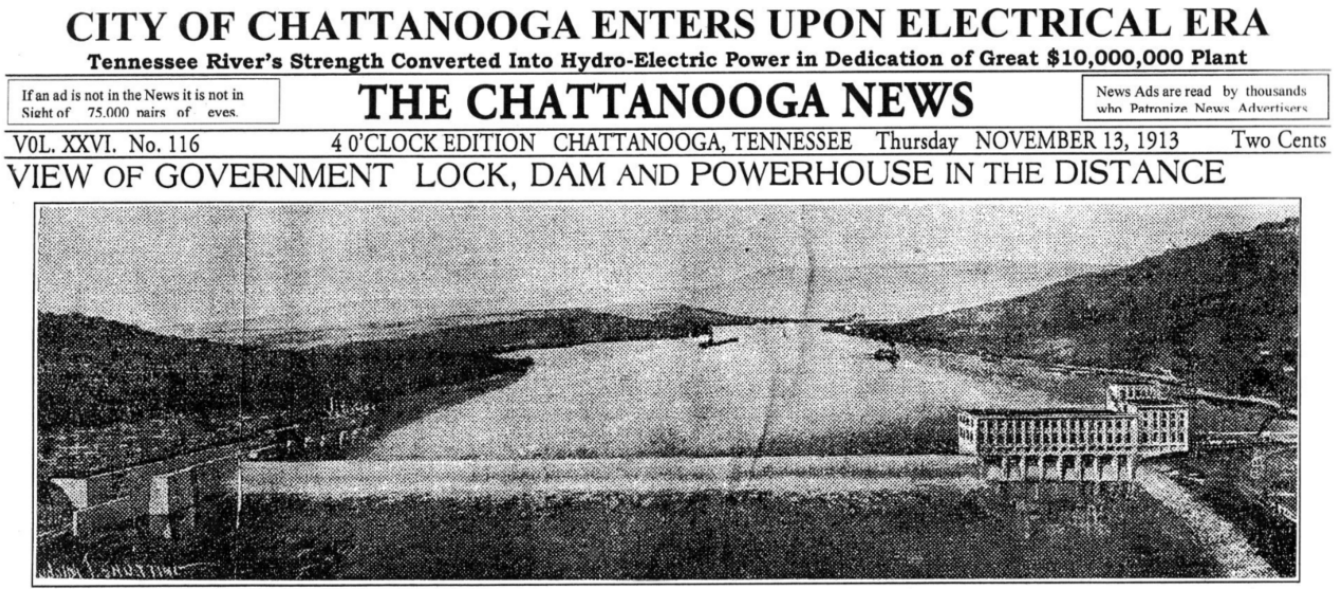
Following Oliver's failed attempts to construct a cofferdam, several other contractors experienced the same challenges and fate. The list below summarizes the construction timeline [2]:

- T.J. Shea took over the project following Oliver, but also failed to successfully manage the underseepage problem and abandoned the job in March 1908.
- Ballie-Dumary took over the project immediately, and then Flaharty, both abandoning the project in December 1909.

In summary, between 1905 – 1910, four separate Contractors had abandoned the project because of the construction challenges associated with the foundation underseepage.

In January 1910, Jacobs and Davis took over the project and began drilling reinforced concrete caissons and initiated an extensive foundation grouting plan. The caissons on the upstream side of the dam were 40 x 45 feet and 30 x 32 feet on the downstream side. The foundation grouting program consisted of both two and six-inch diameter holes.

Construction of the facility required 900,000 cubic yards of soil excavation and 175,000 cubic yards of rock excavation. The powerhouse consisted of a 98 x 240-foot substructure that reached 75 feet below the original riverbed. The main dam was 65 feet wide at the base, varied in height from 45 to 65 feet above bedrock elevation, and had an operating pressure head of 41 feet. The lock had a 41-foot lift making it the highest in the nation when it opened. The lock was only 265 feet and quickly became the shortest lock on the Tennessee River. The lock and dam construction project were finally completed with the lock opening in November 1913 and power conveyed to the city of Chattanooga (Figure 7)[2][3]. Ultimately, the construction of Hales Bar Dam exceeded a total cost of $10 million and took eight years to complete [2].



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Figure 7. Hales Bar Dam Prior to Water Overtopping the Concrete Weir Section [2]

Soon after construction had been completed, wooden flashboards were added to the crest of the overtopping concrete weir section to increase the operating pool by three feet (Figure 8) [2].

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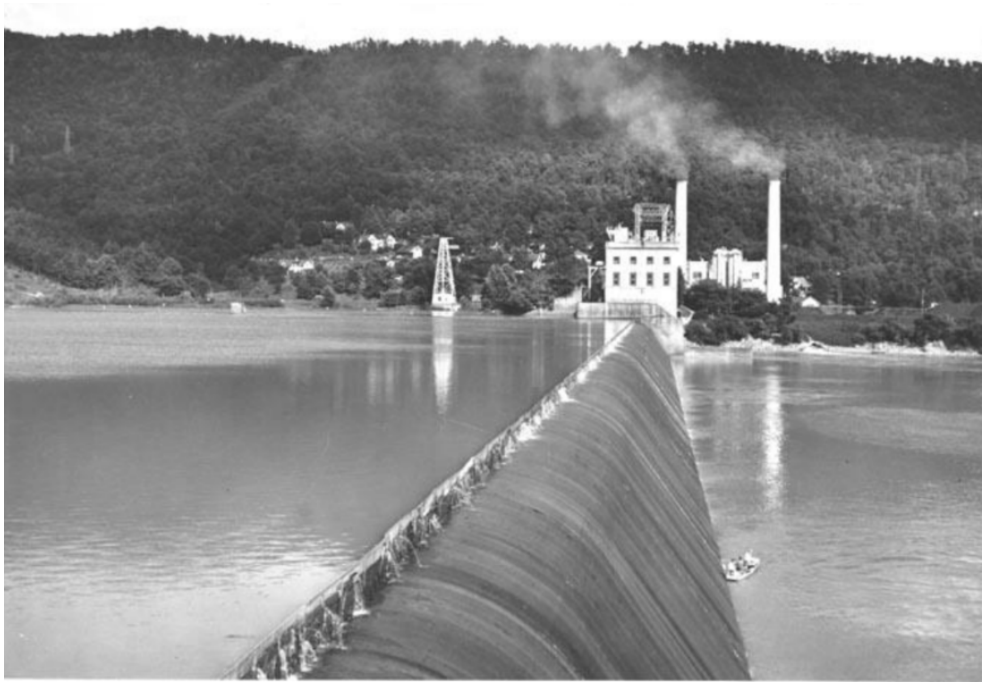


Figure 8. Hales Bar Dam Prior to Water Overtopping the Concrete Weir Section [2]

SITE GEOLOGY

The Hales Bar Dam site was selected based on the local topography. The site was located in a narrow channel of the river and the foundation conditions were assumed to be more resistant. The site was founded on the Mississippian Bangor Limestone formation where the shallow channel was structurally controlled by two faults (Figure 9) [2].

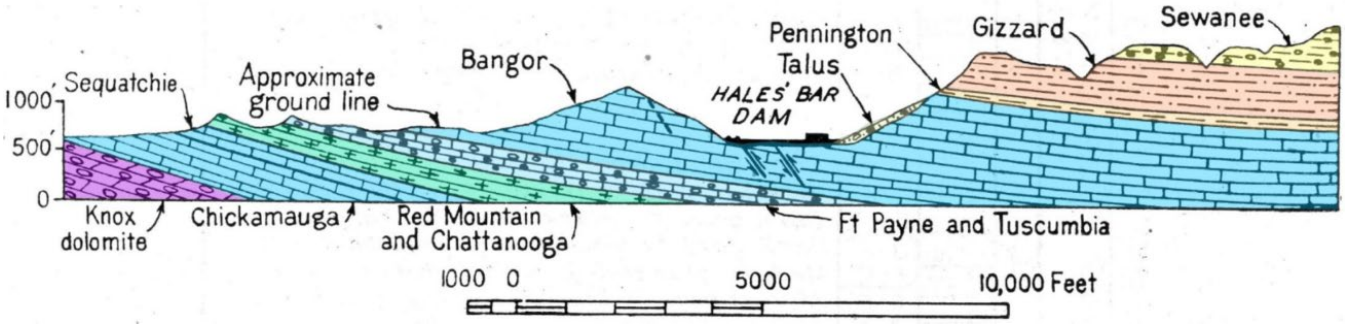


Figure 9. Geologic Profile of the Hales Bar Dam Site [2]

After more than two decades of geologic study, it was determined that the Bangor limestone possessed numerous clay-filled cavities as well as open cavities [2]. Over time, the clay-filled cavities proved to be the more difficult problem to solve. Refer to Figure 10 which shows a geologic section showing the faults at the site (blue arrows), open cavities (white space), and the clay-filled cavities (black).

Foundation leakage was primarily contributed to the washing out of shales in horizontal bedding planes, solution of calcites, and to a lesser degree, faulting. The most difficult to remedy was the numerous vertical solutions channels present within the limestone [4].

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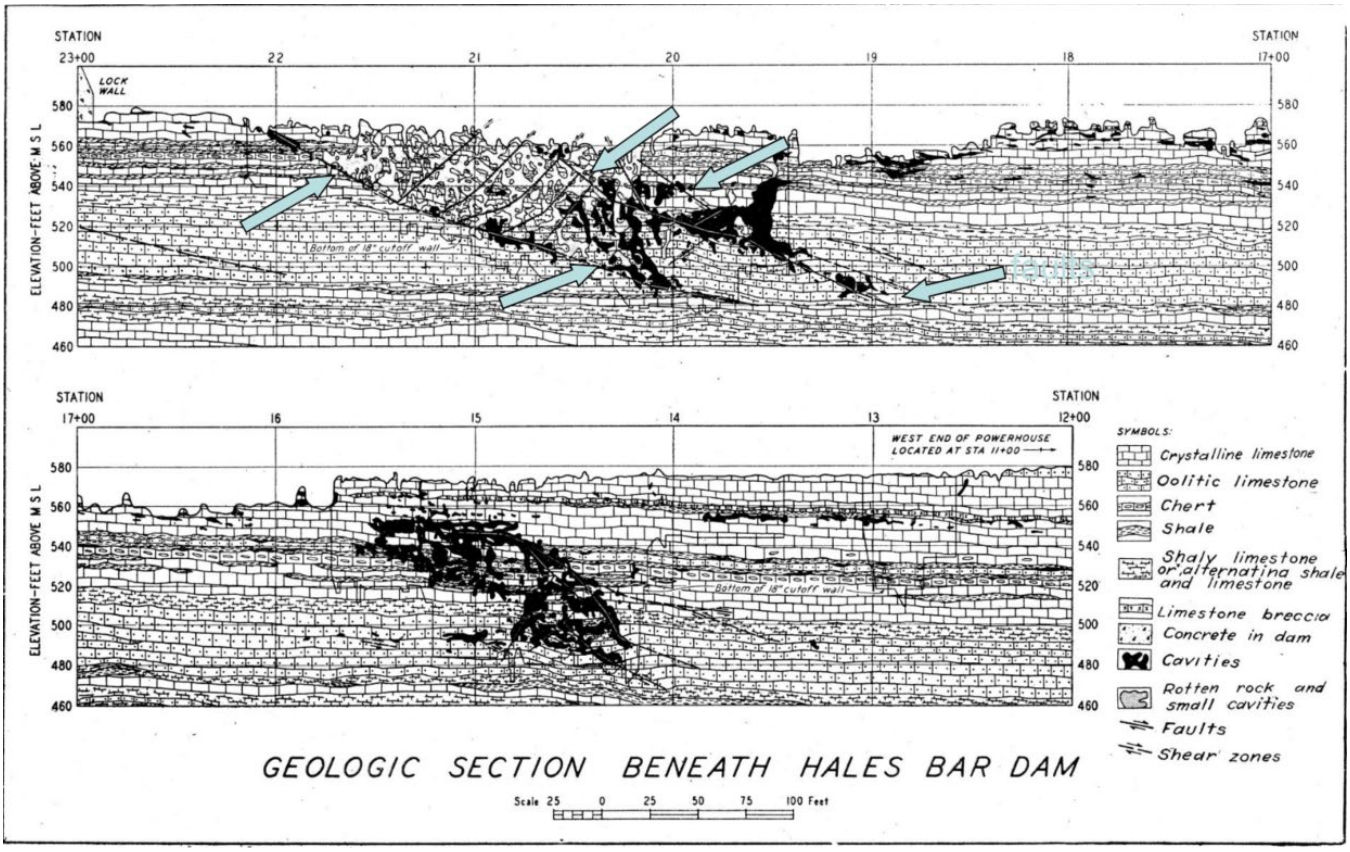


Figure 10. Geologic Section of the Hales Bar Dam Site [2]

STOPPING THE LEAKAGE

Not long after construction was completed, excessive leakage was observed. The excessive leakage posed issues with power generation and threatened foundation bearing integrity [4]. Initial attempts to stop the leakage around the eastern abutment, 1914, included dumping rock proved unsuccessful. The following year, rags were placed over suction holes that were located in the riverbed below the dam and concrete was pumped over the rag-covered suction holes. Once leakage was stopped in one hole it reappeared in another. The leaking suction holes were attempted to be filled with various materials including hay bales, mattresses, wire, and carloads of corsets [2].

To help identify and stop leakage through the exposed features in the bedrock, the project implemented the first use of divers (Figure 11). The divers identified leaks both upstream and downstream of Hales Bar Dam. From 1915 – 1919, the leaks were treated by dropping rags in the holes and then covering the holes with tremie concrete [2].

Leakage continued and the suction hole treatment became a futile effort so in 1919 the owners began drilling grout holes from the gallery inside the concrete overflow section of the dam. Hot asphalt was injected into the grout holes in an attempt to treat the foundation voids. This approach was selected because it allowed the plugging agent to be injected into flowing water with injection pressures as high as 200 pounds per square inch (psi). 6,266 linear feet of grout holes were drilled and a total of 78,324 cubic feet of hot asphalt grout was injected into the dam foundation [2]. This foundation treatment program appeared successful until 1922.

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EXCAVATIONS MUST GO 50 FEET UNDER THE RIVER

Even after the work is started there will be many different features of the job to consider. In building the dam excavation will have to be made 50 feet under the river bed before solid rock is struck. It is said that there is a layer of sand fifty feet deep under the river through which the water penetrates. All of this sand will have to be removed in the immediate section of the dam, as the water would work under a cofferdam through the sand unless the shells went clear through to solid rock. Before the work on the dam can start, the river will have to be pumped dry of the water in the immediate vicinity of the section of which the dam is being constructed. Appliances and machinery will have to be put in place, which will prevent any section of the coffer dam from giving away and catching the men who are working on the foundations of the dam, fifty feet below the surface of the river bed.

Dillan Hemstread was the main diver at the Time of building the Dam. Diving to get the base (tiles) etc, in the proper place.

Greatest Care Necessary at All Times.

Figure 11. Divers Investigate Leakage at the Hales Bar Dam Site [2]

Excessive leakage was continuously observed at the site from 1922 to 1929 reaching the same levels as observed in 1919 prior to the asphalt grouting effort. It was determined that the asphalt grouting was only effective in the top 10 to 15 feet of the bedrock. In 1930 – 1931 an exploration program was implemented to identify flow conduits and to help quantify the volume of leakage (Figure 12). Using dyes and oils flow conduits were mapped and the leakage was estimated to vary between 100 and 1,720 cubic feet per second (cfs). The leakage observed was approximately 10% of the Tennessee River's normal flow [2].

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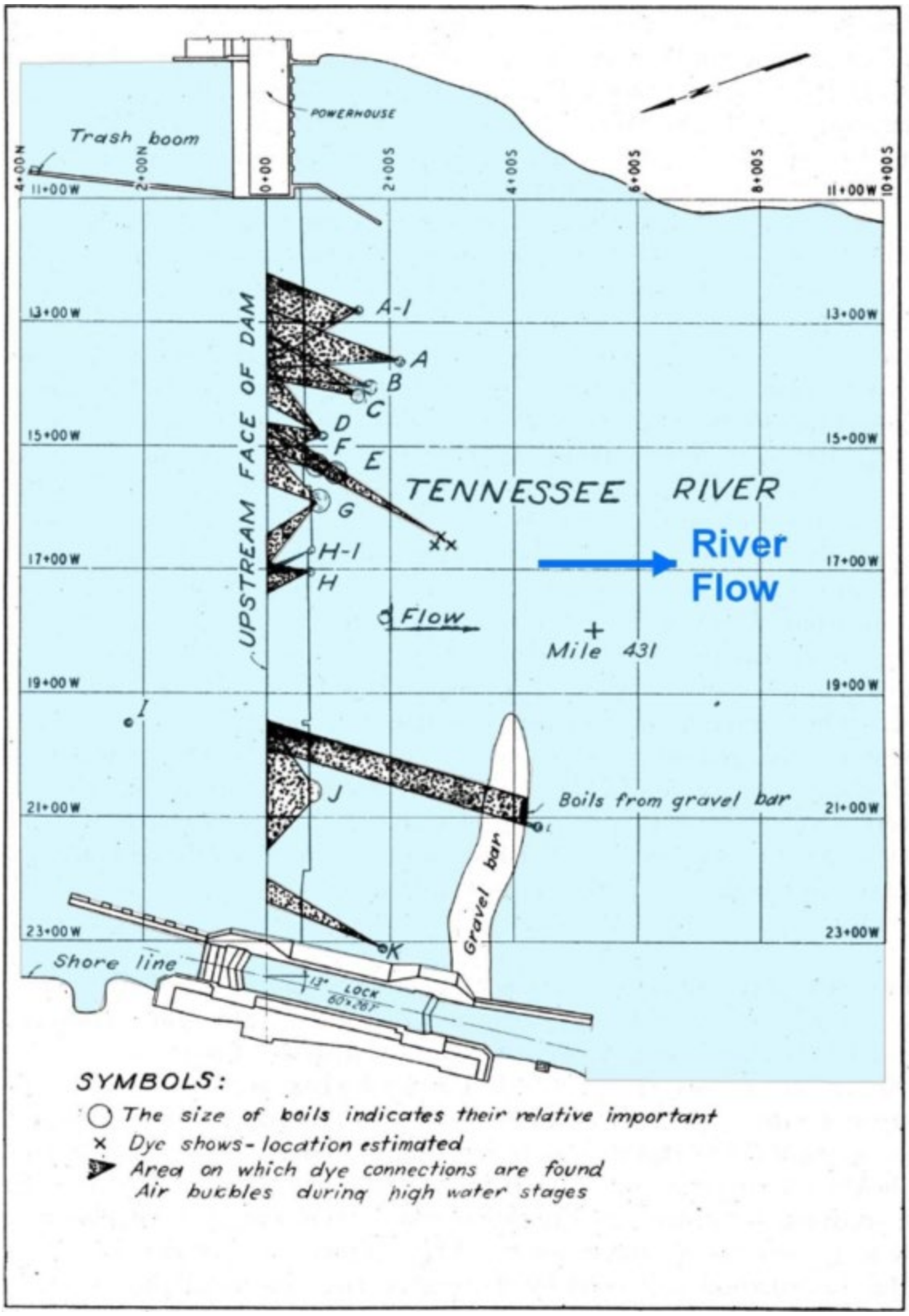


Figure 12. Flow Conduits Mapped at the Hales Bar Dam Site [2]

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The Tennessee Valley Authority (TVA) purchased the dam in August 1939 and launched an investigation in November 1940 using three-inch diamond drill holes along the upstream face of the dam. The TVA then installed a seepage cutoff wall along the dam's baseline (Figure 13) [2].

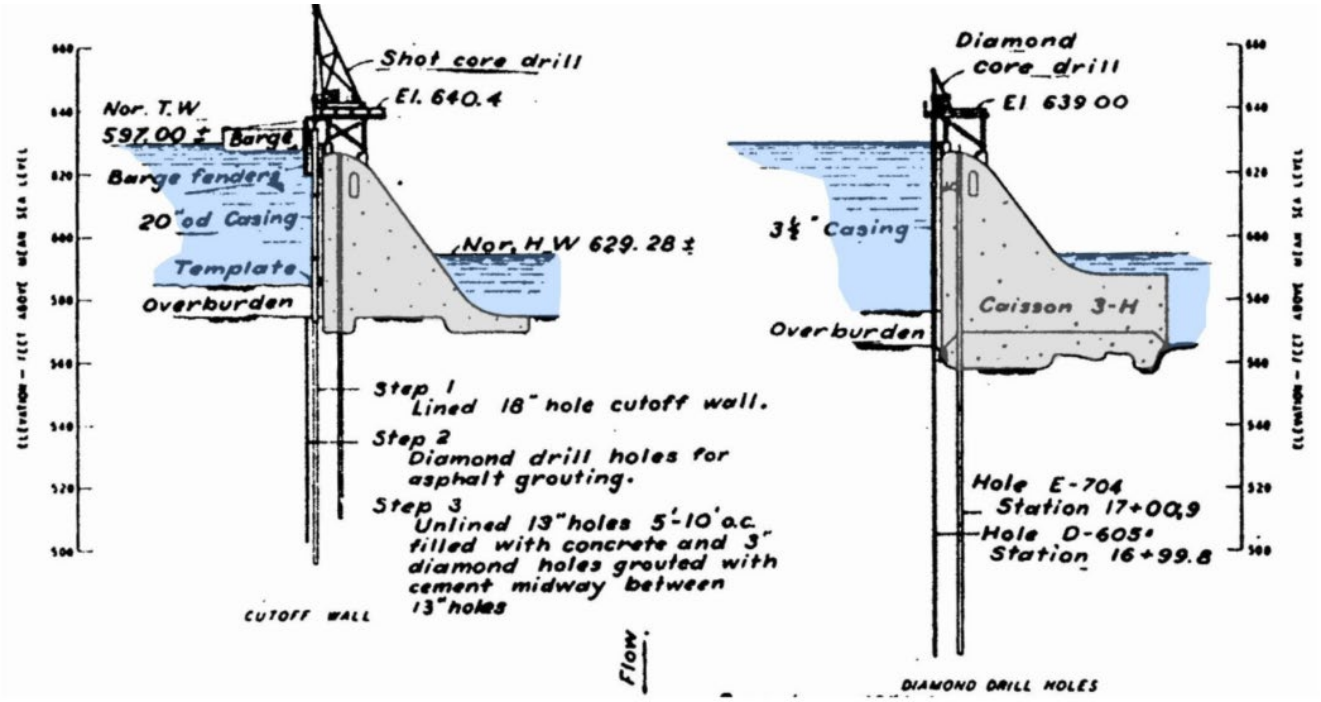


Figure 13. Seepage Cutoff Installed by TVA [2]

Shortly after (1941), TVA began drilling 750 18-inch diameter Calyx holes, removing the cores, and backfilling the holes to a depth up to 103 feet below the riverbed. A three-step process was used by the TVA to create a redundant deep cutoff outlined below (Figure 14) [2]:

1. The main staggered 18-inch diameter cutoff was installed.
2. Diamond drill holes on 10-inch centers were filled with asphalt.
3. 13-inch diameter holes drilled at 10-inch centers filled with concrete along with three-inch diameter grouted holes between the 13-inch holes.

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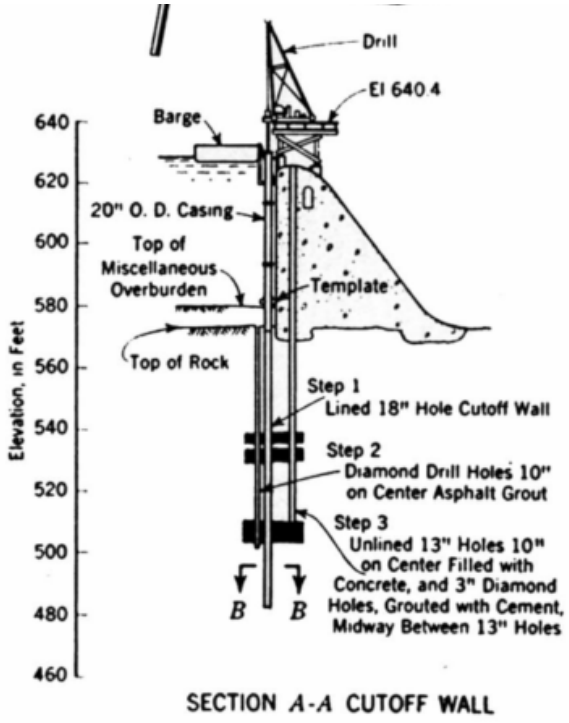


Figure 14. Seepage Cutoff Installed by TVA in 1941 [2]

A drilling template was used for the 18-inch diameter holes that was reset to stagger adjacent holes to create the seepage cutoff beneath the dam. Refer to Figure 15 for a schematic showing the sequence and installation phases for the cutoff which was installed from 1940 - 1944.

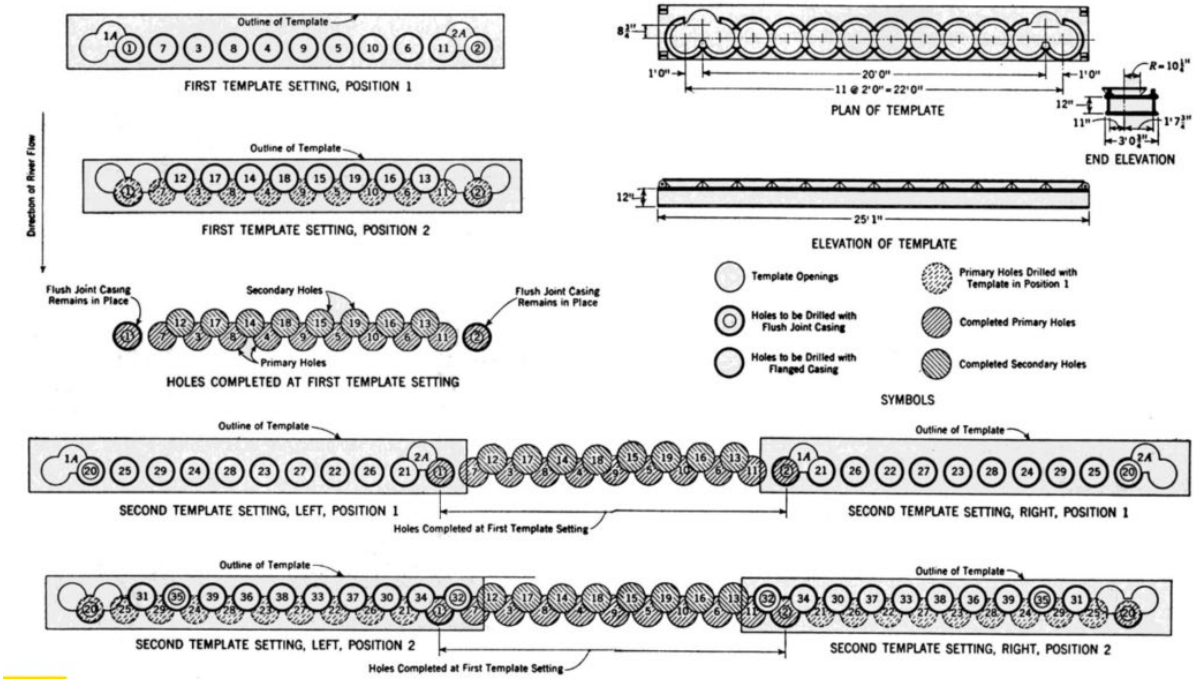


Figure 15. Installation Schematic of Seepage Cutoff Installed by TVA in 1940 - 1944 [2]

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TVA continued to operate and update Hales Bar Dam through 1963. Over time, seepage paths were monitored and further mapped (Figure 16). The logging of cores, current meter measurements (up to 4.5 feet per second), and mapping of sand boils were leveraged to develop an extensive mapping of the principal seepage paths through the foundation [2]. The mapping revealed multiple interconnected systems of cavities in the dam's foundation.

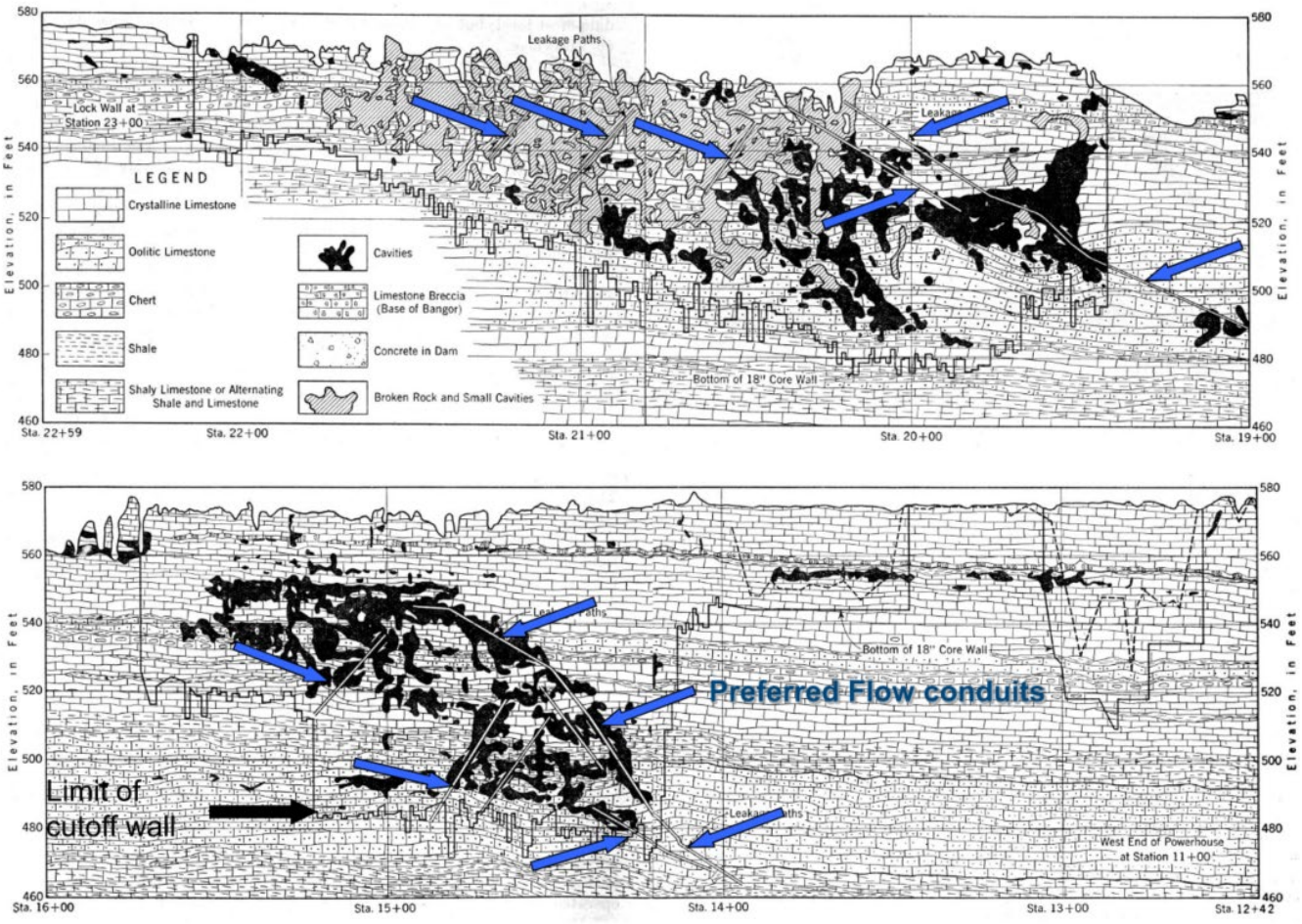


Figure 16. Mapped Seepage Paths [2]

THE FATE OF HALES BAR DAM

TVA finally announced the abandonment of Hales Bar Dam in April 1963 due to the ever-persistent challenge of increasing leakage at the site along with the addition of new, larger navigation locks along the Tennessee River. An alternative site, Nickajack Dam, was selected 6.4 miles downstream of Hales Bar.

The foundation leakage at Hales Bar Dam was attributed to both open cavities and clay-filled cavities. The cavities proved to be intricately connected by joints, shears, and faults that were enlarged by solutioning. The foundation treatment measures proved effective for the open cavities and temporarily for the clay-filled cavities. However, over time, the clay was washed out due to increased pressure from the grout curtain and cutoff resulting in the development of new cavities that enlarged over time. These cavities then became new flow conduits through the foundation of the dam.

Many lessons have been learned from the perpetual foundation leakage problems of Hales Bar Dam. Site selection of future projects of similar nature included geological study and geotechnical exploration. Additionally, modern advancements of cutoff walls and grout curtains can be attributed to lessons learned from Hales Bar Dam.

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Robert Hatton, PE. Robert is a Project Manager for Stantec for various dam safety, civil, and marine projects across the country. His responsibilities include project management, design, planning and coordinating geotechnical investigations and laboratory testing programs, construction quality assurance (CQA) management, internal erosion assessments, operations and maintenance manual development, risk-informed decision making and planning, inspections, and slope stability and seepage analyses. He is experienced in the design and analysis of earthen embankments, concrete dams, spillways, flood control structures, locks, sluice and service gates, and other civil works projects. Robert is experienced with the modification, rehabilitation, and repair of existing locks and dams. Robert is most passionate about helping dam Owners come up with unique solutions to their dam safety challenges that improve the safety and performance of their facilities.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

Italian Dam Failures – Important Lessons Learned from Failure of Gleno Dam (1923) and Overtopping of Vajont Dam (1963)

KEIL NEFF, PE, PHD, PRINCIPAL ENGINEER, STANTEC; NATHANIEL GEE, PE, DAM SAFETY MANAGER, TVA; BILL FIEDLER, PE, SENIOR DAM SAFETY ADVISOR, HDR; SCOTT ARENDS, PE, SENIOR ASSOCIATE, HAZEN & SAWYER

ABSTRACT

Two significant dam catastrophes occurred last century in Northern Italy: Gleno Dam in 1923 and Vajont Dam in 1963. Both of these failures resulted in high loss of life, emphasizing the importance of dam safety in the world, and leading to future practices to prevent such catastrophic failures.

Gleno Dam, a multiple arch buttress dam, was constructed between 1916 and 1923 primarily to produce hydroelectric power. Forty (40) days after first filling, a buttress on the dam cracked and subsequently failed monoliths in the middle section of dam. This fast-forming sunny-day breach caused a large flood wave to flood several villages below, killing at least 365 people.

One of the tallest dams in the world at 860-feet, the double-curved, thin-arch Vajont Dam was constructed between 1957 and 1960. During initial filling in 1963, a landslide caused a mega-tsunami in the lake in which over 40,000 acre-feet of water overtopped the dam. This large overtopping volume caused a flood wave estimated at over 800 feet tall just below the dam which propagated downstream causing massive flooding and destruction to the valley below. Several villages and towns were destroyed with an estimated 1,900 to 2,500 deaths.

An overview of these catastrophes and the important lessons learned will be presented, linking to Italian Dam Safety as well as discussing the human factors that contributed to the catastrophes, leveraging previously developed information from prior papers and presentations.

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DAM SAFETY PALM SPRINGS
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Gleno Dam - Italy's OG of Modern Dam Failures

CHRISTOPHER OWEN, FEDERAL ENERGY REGULATORY COMMISSION

ABSTRACT

At the beginning of World War I, a strong sentiment to liberate Italian speaking populations in the Alps from the Austro-Hungarian Empire existed within the Italian population and political factions. Therefore, Italy declared war on Austria-Hungary on May 23, 1915.Being behind Austria-Hungary in development, Italy knew they needed more energy to fight the war and determined hydroelectric power was the answer. Many rich capitalists started to build dams in Northern Italy. At this time there was turmoil in the area between the rich capitalists and communist workers. In 1916, the Vigano family began construction of the Gleno Dam above the Scalve valley. The communists in the area were protesting in the factories and sabotaging many projects. During construction, the Vigano family had issues with poor quality materials and workmanship. When workers expressed concerns over the safety of the dam, the family fired them and sent them away. Due to funding issues in 1921, the family change the approved plans from constructing a concrete gravity dam to a multiple-arch dam while construction was in progress. This resulted in a mixed dam. They received approval for this plan after they completed the dam. The dam was approximately 722 feet long and 141 feet high with 25 total arches. On October 22, 1923, the dam was fully complete and its reservoir full after heavy rains when several leaks developed. On December 1, 1923, at 7:15 am, the central section of the dam collapsed, releasing 3,648 acre-feet of water into the villages in the valley. The official death toll counted 356 lives lost as a result of the flood, as well as massive damage. After an investigation, officials determined the dam failed due to poor materials and construction, but there were many rumors that the failure was an act of sabotage. After 80 years, the mayor of one of the flooded villages decided to investigate the rumors and discovered that the initial investigation overlooked many facts. His investigation determined that the cause of failure was a bomb placed at the base of the dam by Italian political fractions. This dam failure is evidence that one should not start an investigation with a preconceived idea of how a dam failed, but should investigate all potential facts, even if they are controversial, to determine the cause of failure.

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Panel Discussion - Probabilistic Precipitation Frequency Tools for Dam Safety

MODERATORS – GREG PAXSON, P.E., SCHNABEL ENGINEERING; AND VICTORIA BAHLS, RTI INTERNATIONAL

SPEAKERS: DEBBIE MARTIN, RTI INTERNATIONAL; BILL KAPPEL, APPLIED WEATHER ASSOCIATES; AND KEVIN GRIEBENOW, FEDERAL ENERGY REGULATORY COMMISSION

ABSTRACT

Probable maximum precipitation (PMP) is now a common concept for dam safety practitioners and historically the industry has considered this such an extreme event that tying probability to this event was both difficult and unnecessary; the PMP was considered an upper bound used for design and evaluation of high hazard dams. However, as the dam safety community continues to advance into the use of risk informed decision making, there is a need to estimate the probability of extreme storms to inform our risk estimates. This panel will explore the best-practices for estimating probabilities of extreme rainfall events for watersheds along with industry needs and potential paths forward to meet those needs.

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DAM SAFETY PALM SPRINGS
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Hardfill Dam Feasibility in a High Seismic Area

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ABSTRACT

A hardfill dam is a variation of RCC that distinguishes itself by utilizing on-site aggregate in its natural state, without the need for any additional treatment except for the removal of large particles. The feasibility of a 300-foot high hardfill dam was evaluated for a site located in California with relatively high seismicity. Several hardfill dams with heights up to 500 feet have been constructed around the world since the late 1980's, including in areas of high seismicity. The foundation was judged adequate to support the dam and the rock quality was good enough to provide hardfill aggregate. No active faults were identified at or close to the dam site. Based on the results from investigation and design analyses using linear and nonlinear methods, the hardfill dam was concluded to be a technically feasible dam for the proposed site. The eventual decision not to proceed with the hardfill dam considered the lack of precedent for this dam type in California. This paper will discuss potential next steps designers and dam safety regulators could take to construct a hardfill dam in highly seismic areas.

I.INTRODUCTION

A reservoir expansion project at a site in California includes construction and operation of a new dam and ancillary facilities upstream of an existing reservoir. The alternatives assessment considered multiple dam types including earthfill, hardfill, roller compacted concrete (RCC), concrete face earthfill, and asphalt core earthfill. The highest ranked alternative, the hardfill dam was selected for the feasibility evaluation.

Hardfill dam construction is similar to RCC but can include lower quality aggregate, a wider gradation range, and higher fines content. The resulting hardfill material can be weaker than RCC but has higher stiffness and strength than earthfill. Hardfill dam geometry has a symmetrical trapezoidal shape resulting in lower dynamic tensile stresses within the body of the dam and lower static and dynamic shear stresses on the foundation compared to a RCC or concrete gravity dam, which typically has a vertical or near-vertical upstream face. This allows hardfill dams to be founded on competent, yet relatively lower strength, foundation than RCC or concrete dams.

The hardfill dam concept was first experimented with in the 1980's by various engineers on small cofferdams and dams, notably Rwedat Dam, a 79-foot-tall hardfill dam completed in Morocco in 1988 [1]-[2]. The idea was further developed in the 1990s. In 1992, Pierre Londe and Michel Lino proposed the concept of the symmetric concrete-faced hardfill dam [3]. Marathia Dam, completed in Greece in 1993, was the first dam completed using this concept. Since then, the concept has been used worldwide for construction of dams and is now internationally accepted. Hardfill dams have been built over both soft/weathered rock and soil foundations, although the dams built over soil foundations are generally of lower heights. Most of the constructed dams have cementitious contents between 100 and 250 pounds/cubic yard (lb/CY). Some of those dams did not use any pozzolans in the mix design. Pozzolan content in the other dams ranged from about 10% to 50% with an average of 35% of the cementitious content.

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The hardfill dams that have been built to date in the U.S. (Taum Sauk, and four smaller dams on the Ohio River) are located in low- to moderate-seismicity areas [2]. However, hardfill dams with heights up to 540 feet have been designed and constructed in other parts of the world in various seismic environments. Based on review of the available data, design PGA values for hardfill dams have been up to about 0.9g. Review of existing earthquake data shows that several hardfill dams have been shaken by earthquakes of magnitudes up to 6.6, with estimated PGAs of up to 0.5g. The distribution of earthquake magnitudes within 31 miles (50 km) of existing hardfill dams is presented in Figure 1 [4]. No published case histories were located indicating that any hardfill dam has experienced significant earthquake-induced damage.

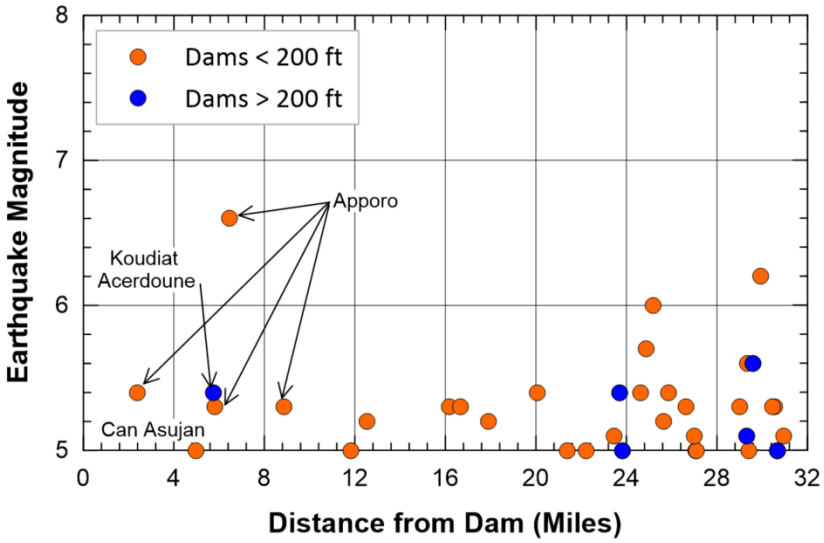


Figure 1. Distribution of earthquakes within 31 miles (50 km) of hardfill dams.

II.DAM FOUNDATION CHARACTERIZATION

For feasibility study of the subject hardfill dam in California, field investigation to characterize the foundation and embankment materials included soil and rock borings and surficial geophysical surveys (Figure 2). The borings included borehole geophysical surveys, dilatometer testing, Lugeon (packer) testing, televiewer surveys, piezometer installations, and collection of samples for laboratory testing. Laboratory testing of interest for the dam foundation included unconfined compressive strength and point load testing.

Based on the results of the data collection effort, the rock underlying most of the proposed dam primarily consists of Franciscan sandstone with interbedded siltstone logged as moderately weathered and medium strong to strong. A small portion of the dam at the top of the left abutment will be underlain by Franciscan mélange, consisting of sheared siltstone/claystone and a large greenstone block. Weathering generally decreases with depth. The bedding dips east to northeast (into the left abutment slope). The rock is overlain by landslide deposits or a veneer of residual soil or colluvium on the hillslopes, and the floor of the valley contains alluvium and sediments from the existing reservoir.

The dam foundation surface was developed by identifying the depth in each exploratory boring within or near the dam footprint where the in-place rock became predominantly slightly weathered or moderately to slightly weathered. In some borings, this depth point was very clear while in other borings the decision also included review of the Rock Quality Designation (RQD) and recovery. If these rock quality indicators showed a marked increase at a certain depth within the range of slightly to moderately weathered rock, then that depth was chosen as the foundation depth. These points were connected, and the resulting surface was smoothed to create an excavatable foundation surface shown on Figure 2.

A potential failure mode (PFM) workshop was conducted to identify possible failure modes, both credible and non-credible, that could affect the safety of a hardfill dam. Some of the credible failure modes included seepage through the foundation causing erosion of the foundation, and differential settlement causing cracking (separation) at the foundation contact, leading to erosion of the foundation. The identified failure modes were considered in the design.

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The orientation of bedding and discontinuities in the predominately sandstone foundation is likely to produce an irregular foundation surface. The foundation surface would be treated to reduce potential for cracking of the hardfill, removal of weak material, and backfilling any depressions with dental concrete. Based on results from Lugeon testing and televiewer survey, a two-row grout curtain is proposed along the upstream toe of the dam to reduce the hydraulic conductivity of the foundation and minimize seepage under the dam, thus preventing the piping of foundation materials. Grout holes will be oriented to intersect as many foundation discontinuities as possible.

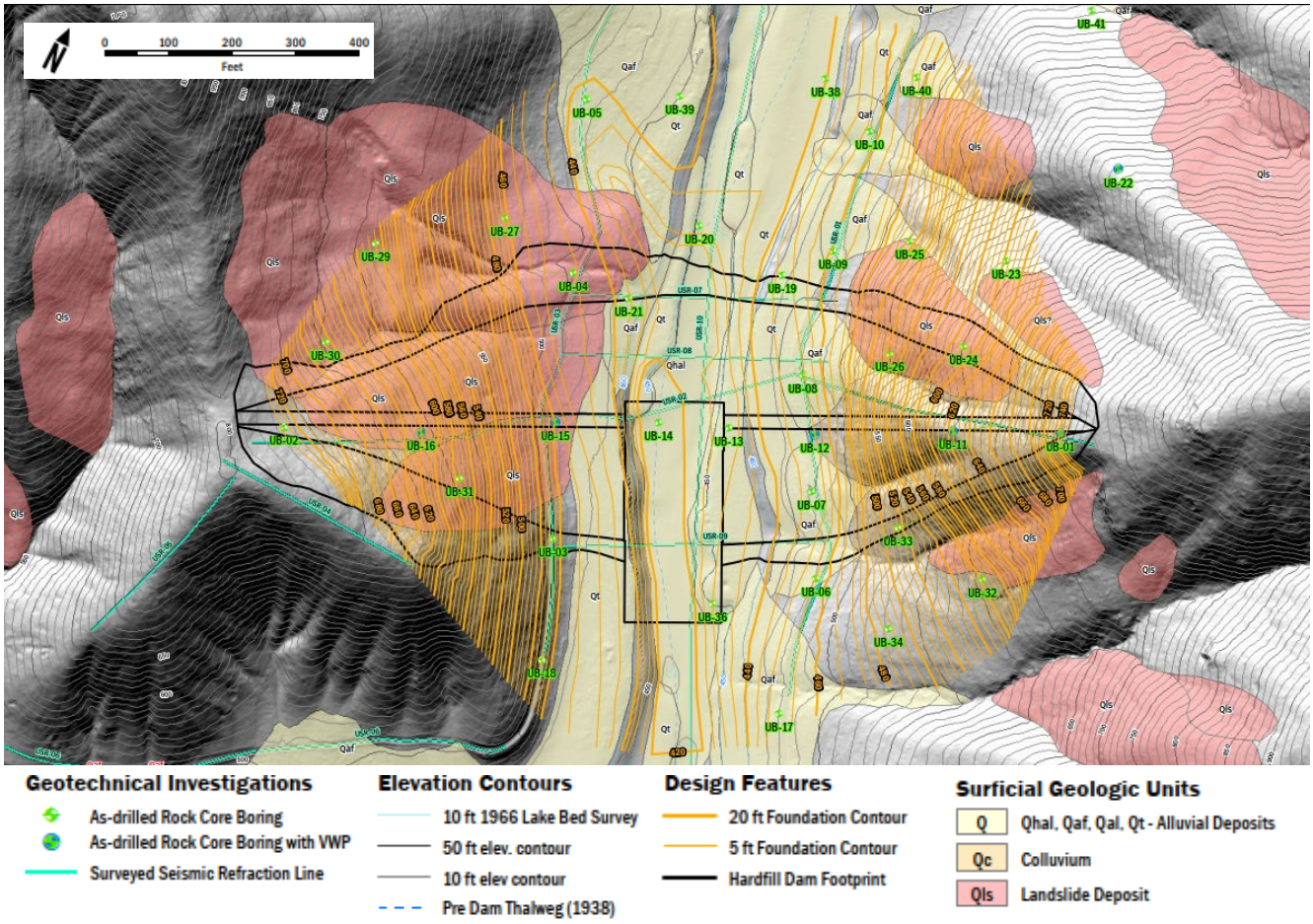


Figure 2. Map of exploration locations, contours of slightly to moderately weathered rock.

III.HARDFILL MATERIAL DEVELOPMENT AND CHARACTERIZATION

Rock borrow areas within a half mile upstream and downstream of the proposed California dam site were identified. The borrow area field investigation involved rock coring and geophysics. Samples were also recovered for laboratory testing of rock strength, specific gravity, absorption, Los Angeles (LA) abrasion tests at different revolutions, sodium sulfate soundness, and petrographic examination. The borrow area evaluation concluded that the quantity of hardfill aggregate materials required for the proposed hardfill dam is available on site, and that the quality of the material was generally acceptable for use as a hardfill aggregate. Borrow excavation would be performed in a manner to produce material consisting primarily of sandstone with lesser amounts of siltstone. Thicker siltstone layers would be selectively excavated and wasted.

Processing of borrow aggregate for hardfill would be limited to crushing and screening for removing oversized materials. The strength of hardfill materials depends on cementitious content, aggregate quality, an established aggregate processing plan that can be consistently implemented for all borrow areas, allowable nominal maximum size and percentage of fines, and the degree of potential breakdown of aggregate during mixing, placing, and compacting of the hardfill mix.

As the hardfill dam has a trapezoidal shape, seismic-induced stresses within the dam are expected to be uniform, resulting in very small tensile stresses. Therefore, the required strength of a hardfill dam does not need to

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be as high as that of an RCC dam of similar height. Aggregates that strictly meet typical acceptance requirements in terms of gradation, absorption, abrasion resistance, specific gravity, and durability (ASTM C33, C131, C127) are necessary for RCC dams. However, for a hardfill dam, these acceptance requirements can be relaxed due to its symmetrical shape. Furthermore, permeability is not a significant factor since seepage mitigation measures at the upstream face, such as reinforced concrete facing, geomembrane, or grout-enriched vibrated RCC, are incorporated into the design. The material properties used for the proposed dam will be verified through a variety of tests, including laboratory mix designs and a test fill during the design phase. The hardfill test fill program can be used to evaluate the aggregate (rock type, weathering, gradation), cementitious content, and water content required to achieve the design hardfill properties. The results of the design test fill will be used to inform the specification requirements. A test fill will also be required during construction to verify the contractor's proposed means and methods to meet the specification requirements.

IV.DAM FEATURES

For the California dam site example, a new hardfill dam would be constructed upstream of the existing reservoir to provide a storage capacity of 140,000 acre-feet. Figures 3 and 4 show the plan and typical sections of the proposed dam. The new dam would have a maximum height of 320 feet, a crest 30 feet wide and 1900 feet long, a width of about 550 feet at the maximum section, and a symmetrical trapezoid-shaped cross section with an upstream concrete face slab that minimizes water penetration into the dam body. The dam body would consist of roller compacted cemented sand and gravel materials.

The hardfill dam would include a stepped concrete spillway constructed within the middle section of the dam. A stilling basin would be located at the toe of the spillway to dissipate energy prior to releasing flows back to the river channel. Reinforced concrete training walls would be constructed on each side of the spillway and stilling basin to contain the discharge flows.

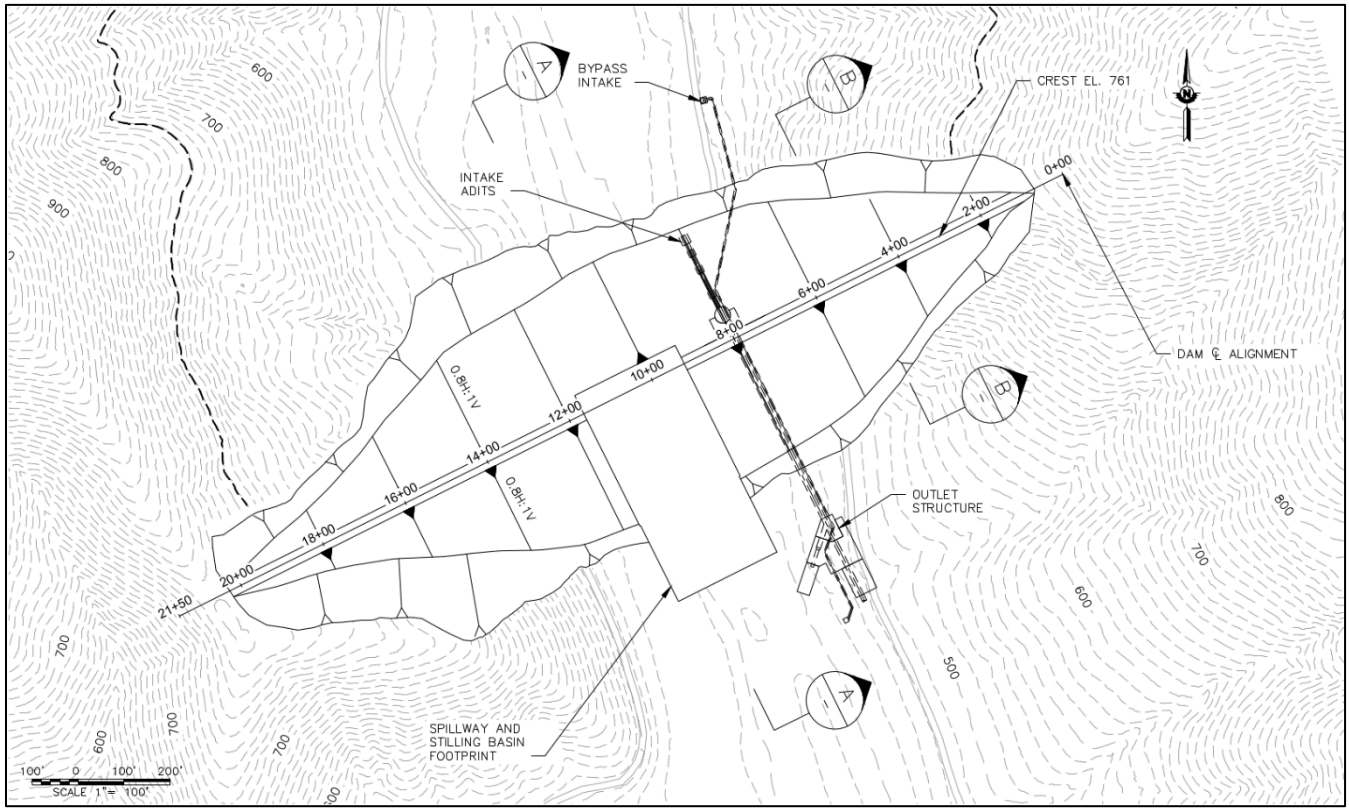


Figure 3. Plan view of proposed hardfill dam.

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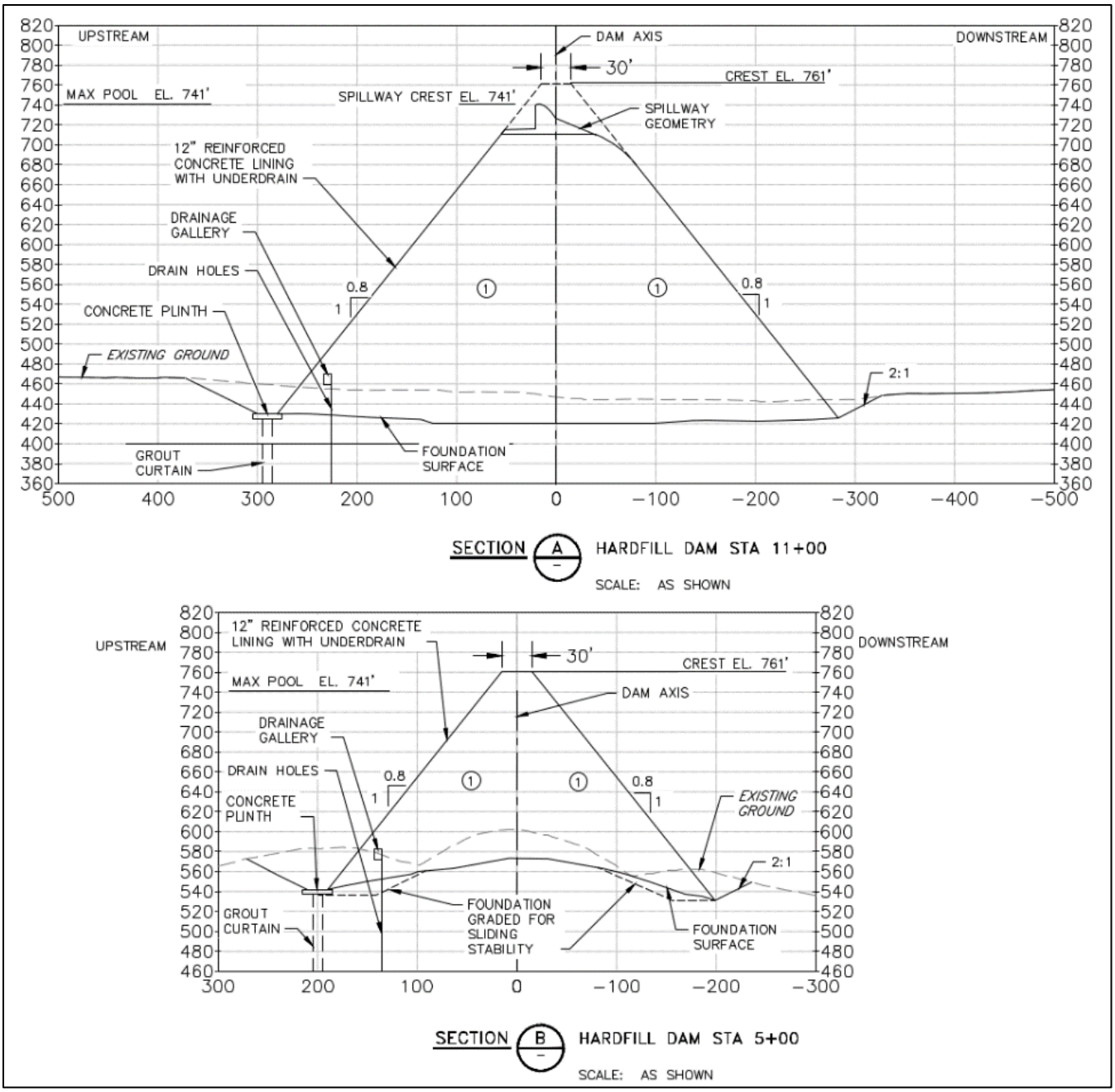


Figure 4. Typical sections of proposed hardfill dam.

V.SEISMIC HAZARD EVALUATION

The regional and local faulting studies in the area of the California dam were focused on assessing the potential for a large shear or an active fault within the footprint of the proposed dam. Although numerous localized bedrock shear zones were identified in core samples recovered from the exploratory borings, that shearing is interpreted to be related to accretion of marine deposits and subsequent deformations, and not related to movement associated with the current tectonic setting. No active faults have been identified at or close to the proposed dam site based on the geologic field investigations and is further supported by the lack of historical seismic activity at the site.

Seismic loading criteria were developed based on the results from deterministic and probabilistic seismic hazard analyses. Multiple earthquakes including, maximum credible earthquake (MCE), 2,500-year, 5,000-year, 10,000-year, and reservoir-induced earthquake (Figure 5) were developed with corresponding performance criteria.

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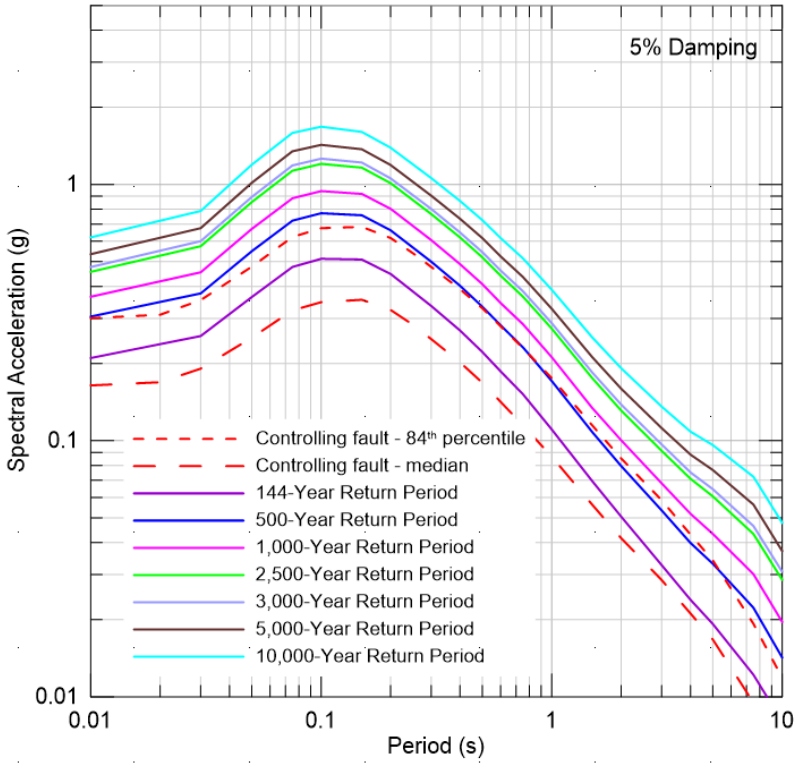


Figure 5. Response spectra for MCE (84th percentile) and range of probabilistic seismic levels.

VI.DAM ANALYSIS

Dam analyses consisted of both static and dynamic evaluations to assess the performance of the dam under various loading conditions. The static analysis presented in this paper focuses solely on settlement during construction and first filling. The seismic performance of the hardfill dam poses a complex problem, requiring a nonlinear analysis approach to capture the tensile behavior especially when subjected to high levels of seismic excitation. However, for a feasibility-level design analysis, linear time-history analyses offer valuable insights into dam performance in a relatively efficient manner. Although the validity is limited to cases where the material behavior is predominantly linear, they still serve as a useful tool to provide information on the main characteristics of the dam's dynamic response and allow for qualitative damage estimates. Both linear and nonlinear seismic analyses were conducted as discussed below.

ANSYS Analysis - Static (Settlement)

A 3D ANSYS Release 19.2 [5] model was used to understand the settlement of the dam during construction and first filling under the usual (static) load combination (Figure 6). The settlement at the dam/foundation interface is presented in Figure 7 for the end of construction case. The maximum settlement of the dam ranges from 1.5 to 3.5 in for different foundation conditions (sandstone or siltstone). Distortions in the transverse direction were less than 0.1/300 inch/inch and no apparent concentrations in strain are visible at the stratigraphic transitions modeled in foundation modulus. Based on these results, the dam is considered to have adequate safety regarding settlement.

ANSYS Analysis - Seismic

The seismic performance of the dam was first analyzed using 2-D finite element models (FEM) using the general-purpose finite element software ANSYS. A 3-D FEM was not performed to evaluate seismic performance of the dam considering the relatively wide valley, relatively uniform foundation conditions, and symmetrical cross section of the dam.

The linear time-history analysis approach, similar to the expedient solution proposed by Clough [6], assumes a massless foundation and models hydrodynamic effects as an added mass of water moving with the dam. The design ground motion was applied at the bottom fixed boundary of the foundation domain. This method of analysis

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was chosen for feasibility level design because the model runs much faster than nonlinear FEMs considering foundation mass and water-dam-foundation interaction.

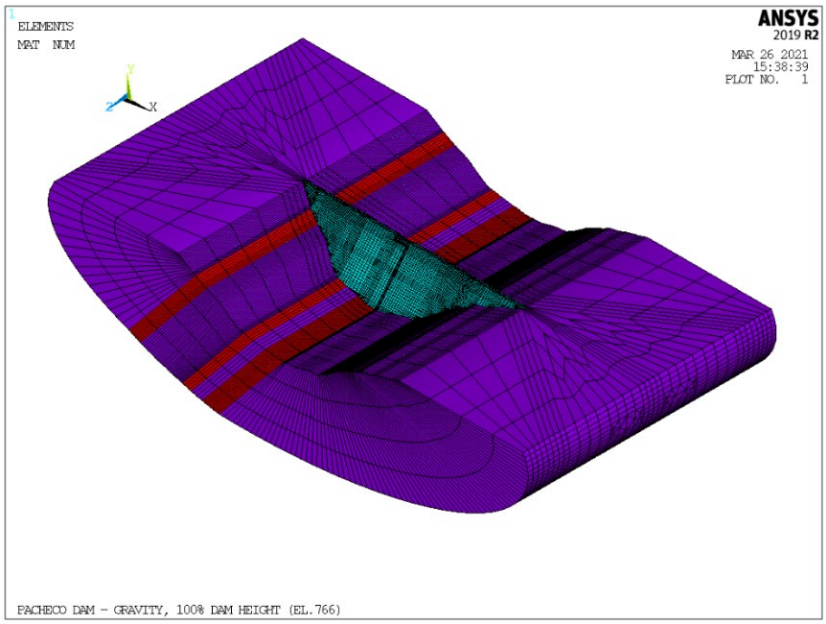


Figure 6. 3D ANSYS model for settlement estimation.

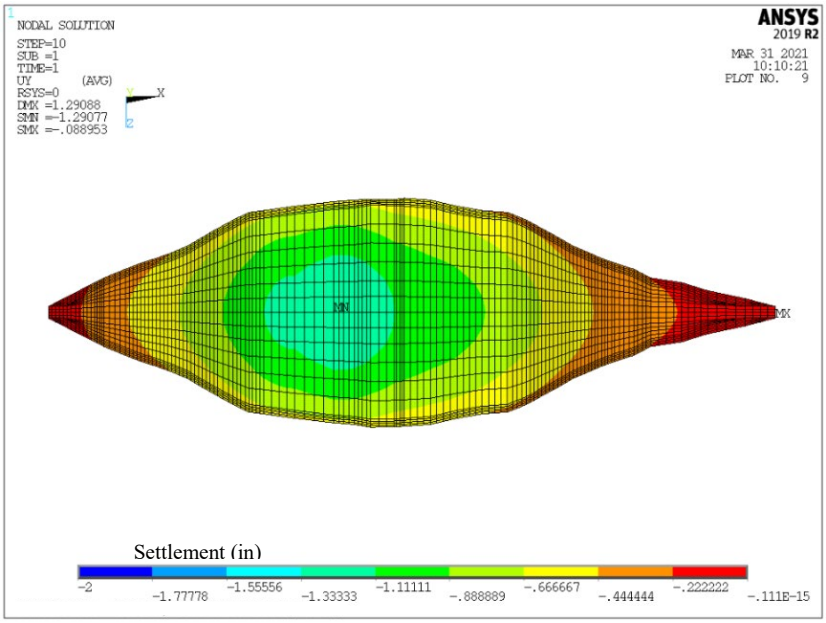


Figure 7. Dam/foundation interface settlement, end of construction.

Figure 8 shows positive vertical stress (tension) is present for critical time steps in one of the considered earthquakes, (Christchurch event). At the critical time steps the base pressure was output from the ANSYS analyses and the uplift pressure was added to the results along the base. The sum of the two pressures yields the contact pressure at the dam base illustrated in these plots. The vertical stress at the base for these critical time steps can be seen in Figure 9. The presence of tension at the base (Figure 9) indicates that a crack may develop at the upstream toe of the dam. Note that the analysis assumes complete bond between dam and rock. This approach sometimes results in unreasonably high stresses at local points near the foundation, especially in the upstream toe

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of the dam. The computed tensile stresses are judged not indicative of the actual stress-state but instead is a result of simplifications or assumptions made in the model and therefore acceptable [7].

The results from the analysis show that the overall computed stresses in the dam are less than the allowable stresses, cracking is expected to occur along the dam/foundation interface at the upstream toe for the MCE seismic load combination, and the proposed hardfill dam has adequate safety against overturning for the analyzed load combinations.

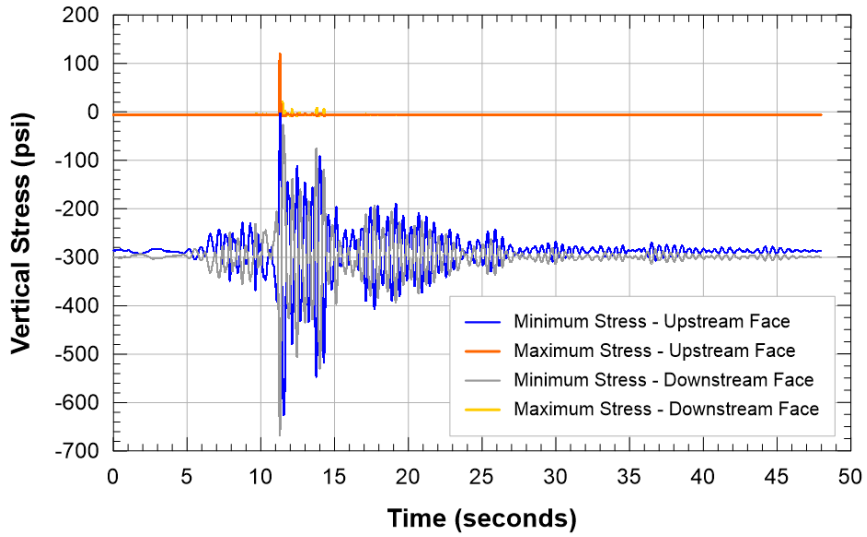


Figure 8. Maximum vertical stresses on upstream and downstream faces, Christchurch earthquake.

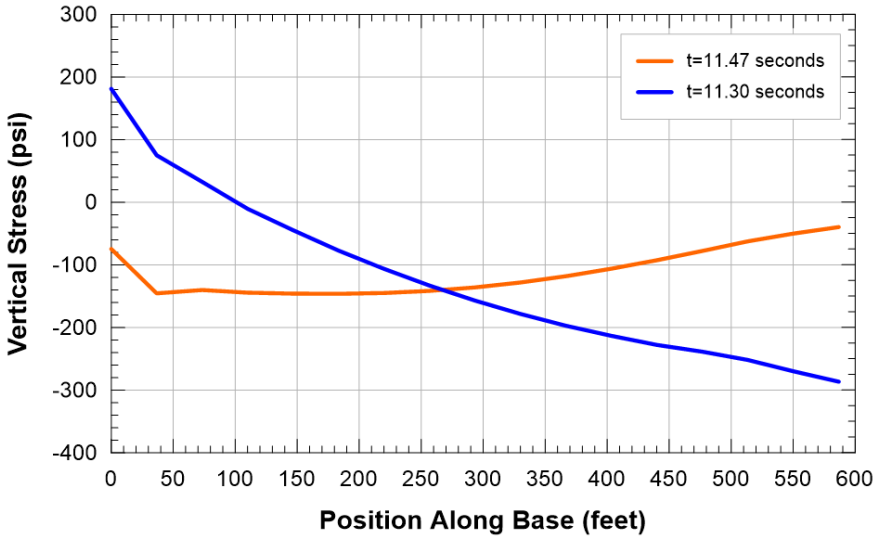


Figure 9. Vertical stresses along base (negative in compression), Christchurch earthquake at two critical time steps.

LSDYNA Analysis- Seismic

The purpose of the LSDYNA [8] analysis was to check the behavior of the structure for the 10,000-year level of shaking where nonlinearity could play a greater role in the response. The geometry of the finite element model used for the LSDYNA simulations is similar to that used for the ANSYS simulations presented earlier. The seismic excitation was prescribed at the bottom of the model with non-reflective boundary conditions on the left and right sides of the model (Figure 10). Input time histories ("within" motions at the base of the LSDYNA model) were developed by running a 1D deconvolution analysis in DEEPSOIL [9] using the same dynamic properties for the rock

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that are used in the LSDYNA analysis. The results from the analyses for the 10,000-year return period confirmed results from the ANSYS analysis indicating that the dam is stable.

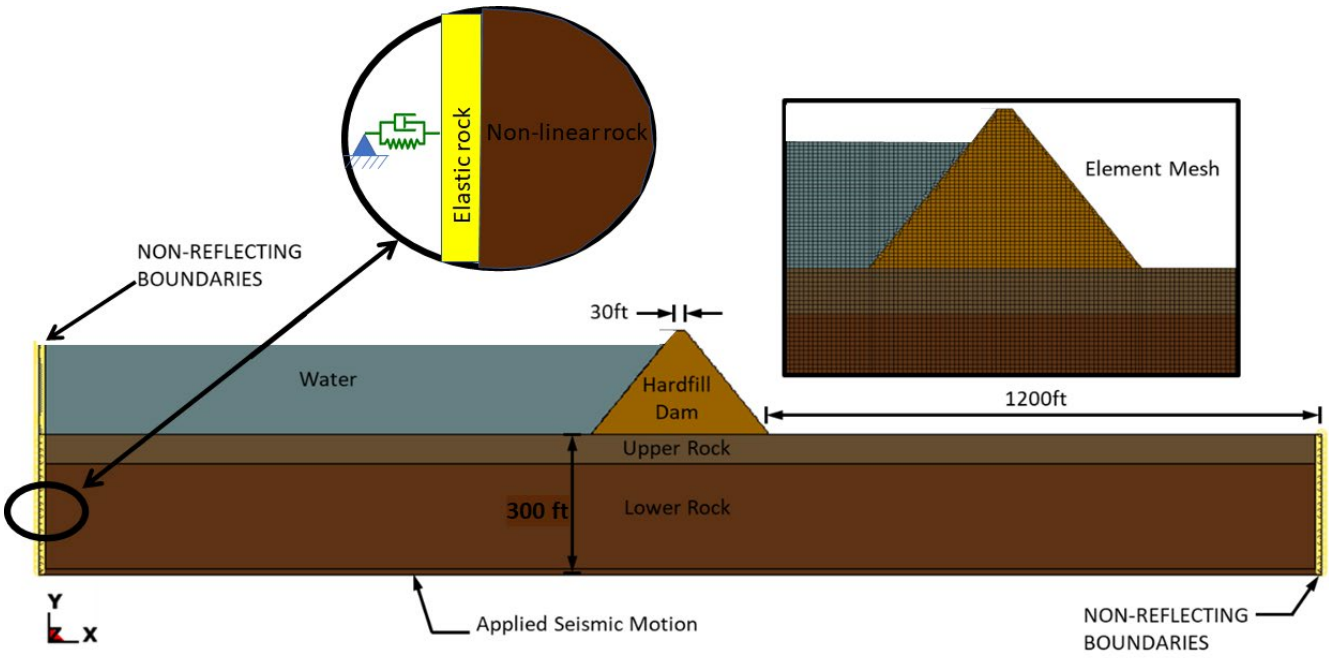


Figure 10. LSDYNA model of dam and foundation.

VII.FINDINGS

Hardfill dams with heights up to 540 feet have been designed and constructed around the world in various seismic environments since the late 1980's. A review of existing earthquake data shows that several hardfill dams have experienced shaking from earthquakes of magnitudes up to 6.6, with estimated PGAs of up to 0.5g. However, no published case histories indicate that any hardfill dam has experienced significant earthquake-induced damage.

The foundation for the proposed dam primarily consists of moderately weathered Franciscan sandstone with interbedded siltstone. Seepage through the foundation causing erosion was identified as one of the potential failure modes. To address this, a two-row grout curtain along the upstream toe was proposed.

Rock borrow areas near the dam site were identified through field investigations. As the hardfill dam is trapezoidal in shape, seismic-induced stresses within the dam are expected to be uniform, resulting in very small tensile stresses. Consequently, the required strength of hardfill would not need to be as high as for a RCC dam of similar height. Processing of borrow aggregate for hardfill would be limited to crushing and screening to remove oversized materials.

The 3D Finite Element static analysis showed that settlement during construction and first filling is expected to be minimal and acceptable. The 2D linear seismic analysis indicated that overall computed stresses in the dam are less than the allowable stresses and has adequate safety against overturning. However, cracking is expected to occur along the dam/foundation interface near the upstream toe in a limited area. Nevertheless, the proposed hardfill dam is expected to be seismically stable. The results from the nonlinear analyses for one of the largest seismic loadings confirmed the findings derived from the linear analysis.

Based on the results of the geologic investigations, the design analyses, and a constructability assessment, the hardfill dam type was concluded to be a technically feasible dam type alternative for the proposed site.

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VIII.FUTURE OF HARDFILL DAMS IN CALIFORNIA

The hardfill dam is a technically feasible dam. However, the eventual decision not to proceed with hardfill dam considered the lack of precedent for this dam type in California, and the lack of documented performance of existing hardfill dams over 300 feet high under strong earthquake shaking.

Potential next steps designers and dam safety regulators could take to construct a hardfill dam in highly seismic areas include the following:

1. Reach out to current hardfill dam owners and discuss their experience during planning, design, and construction and past dam performance under seismic and normal conditions. Arrange technical meetings with these dam owners, dam designers in California, and dam safety regulators to share and gather information.
2. Collaborate with universities to build models of hardfill dams of various heights with instrumentation and subject them to low to high ground motions in a centrifuge. Develop numerical models to simulate centrifuge test conditions and compare the results from numerical models to actual performance observed in centrifuge tests. This comparison can help designers and regulators identify any potential issues with numerical models in predicting the seismic performance of the hardfill dams.
3. Look for an opportunity to build a hardfill dam less than 200-ft tall in moderately to high seismic areas.

IX.ACKNOWLEDGMENT

The authors would like to thank M. Forrest, L. Handfelt, and J. Roadifer for their input provided during the feasibility design process. The authors also gratefully acknowledge the contributions of Y. Alostaz for performing the nonlinear analyses in LSDYNA and S. Jones and D.J. Bianco for performing analyses in ANSYS.

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Assessment of Aging Dams for Seismic Potential Failure Modes: A Delta Approach

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ABSTRACT

The state of practice in design and construction of embankment dams has been evolving for over a hundred years with the invention of construction equipment and knowledge of dam design and performance. The importance of seismic design considerations for embankment dams was not fully recognized prior to the near failure of the Lower San Fernando Dam and poor performance of the Upper San Fernando dam during the 1971 San Fernando earthquake. However, many embankment dams were designed and constructed prior to the current seismic design standards. A qualitative approach was developed to assess and screen embankment dams for seismic potential failure modes (PFMs). This delta (Δ) approach consists of comparing the historical design and construction methods originally used for the existing embankment dam being evaluated to the current state of practice. The term delta (Δ) refers to missing features, as well as design and construction deficiencies between historical practice and the modern state of practice for addressing seismic Potential Failure Modes (PFMs). Several factors are documented in this evaluation process with explanations based on the current state of practice for the design and construction of embankment dams and defenses against seismic PFMs. The larger the delta (or the difference with the current state of practice), the higher the potential for poor performance of the embankment dam during an earthquake. The results can be used to help assess and screen seismic PFMs of existing embankment dams for additional investigation and evaluation to reduce uncertainty and better characterize the performance risk and the need for interim risk-reduction measures. This delta (Δ) approach provides an effective tool to communicate the potential overall system performance deficiencies in a transparent manner.

I. INTRODUCTION

Most embankment dams in the US were designed and constructed prior to the development of modern seismic design standards and construction methods. For the USACE portfolio of dams, 72 percent were constructed prior to 1970; 25 percent were constructed between 1970 and 1989; and less than 3 percent were constructed after 1990. Although the near failure of the Lower San Fernando Dam occurred in 1971, the state of practice in embankment dam design and construction continued to improve for several more decades and continues to improve

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to this day. Performance- or risk-informed design approaches for new modern dams are now required to meet performance objectives (both functional and safety) during and after an earthquake. A modern embankment dam with state of practice design and construction procedures to address non-seismic PFMs typically provides higher resiliency for seismic PFMs, as many common standard of practice design and construction considerations for a modern dam also improve seismic performance. However, re-assessment of existing embankment dams that were not designed or constructed for seismic loading, as per the current standard of practice, continues to be imperative as well as a challenging task.

An assessment of an existing embankment dam requires knowledge of the evolution of the state of practice of embankment dam design and construction, as well as the state of practice of geotechnical earthquake engineering, seismology, and geology. In the absence of an adequate understanding of these factors, the performance evaluation of an existing dam during an earthquake, that was not designed or constructed for such an earthquake event, may be unconservative and over optimistic. As earthquake events are generally rare, this inadequate engineering practice may sometimes continue without embankment dams being subjected to strong shaking. Several parts of the US, such as California, Oregon, Washington, Alaska, Tennessee, and Missouri are located within high seismic hazard potential zones. With each passing year without a strong earthquake event, the probability of strong shaking from large earthquakes such as the Cascadia Subduction Zone in the Pacific northwest and California, the Alaska subduction zone in Alaska, the San Andreas Fault and other active faults in California, and the New Madrid fault zone is increasing. However, these quiet periods in potential active seismic zones also negatively impact the urgency in the evaluation and decision-making process that could be improved with transparent communication between the practitioners and decision makers for dam safety.

An innovative qualitative approach, known as the delta (Δ) approach, was developed to assess embankment dams for seismic related potential failure modes (PFMs) to assist in screening and strategy for additional investigations, analyses, and evaluations. This delta (Δ) approach is based on comparing the current state of practice for design and construction that effectively addresses seismic related PFMs with the design considerations, features, and construction methods for the existing embankment dams being evaluated. Several factors are documented in this evaluation process with explanations based on the current state of practice regarding the design, construction, and seismic PFMs. This delta approach requires the practitioners to (1) develop a better understanding of the history and evolution of the state of practice in engineering and construction of embankment dams, (2) develop an improved understanding of an existing embankment dam, and then (3) assess any deficiencies compared to modern embankment dam design and practice.

Figure 1 shows a balance scale to demonstrate the basic concept of this delta (Δ) approach, which highlights the gap between the conditions of an existing dam compared to a properly designed and constructed modern dam. A poorly constructed embankment dam may include many deficiencies that would indicate a greater delta (Δ) or higher vulnerability during an earthquake. As congruency of an existing dam to current practice increases, the balance levels and the delta decreases.

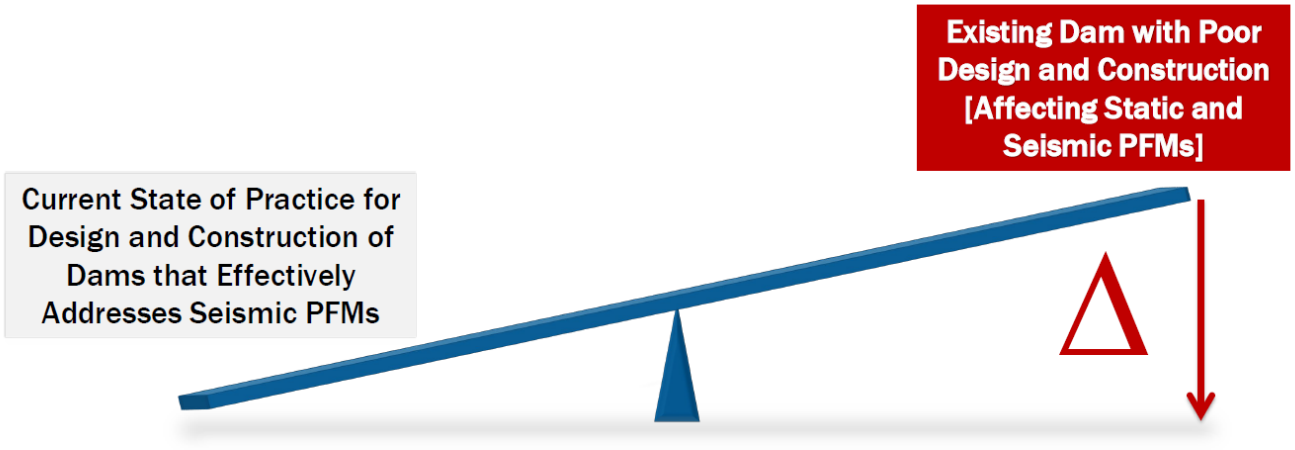


Figure 1: Balance Scale to Demonstrate the Basic Concepts of the Delta (Δ) Approach

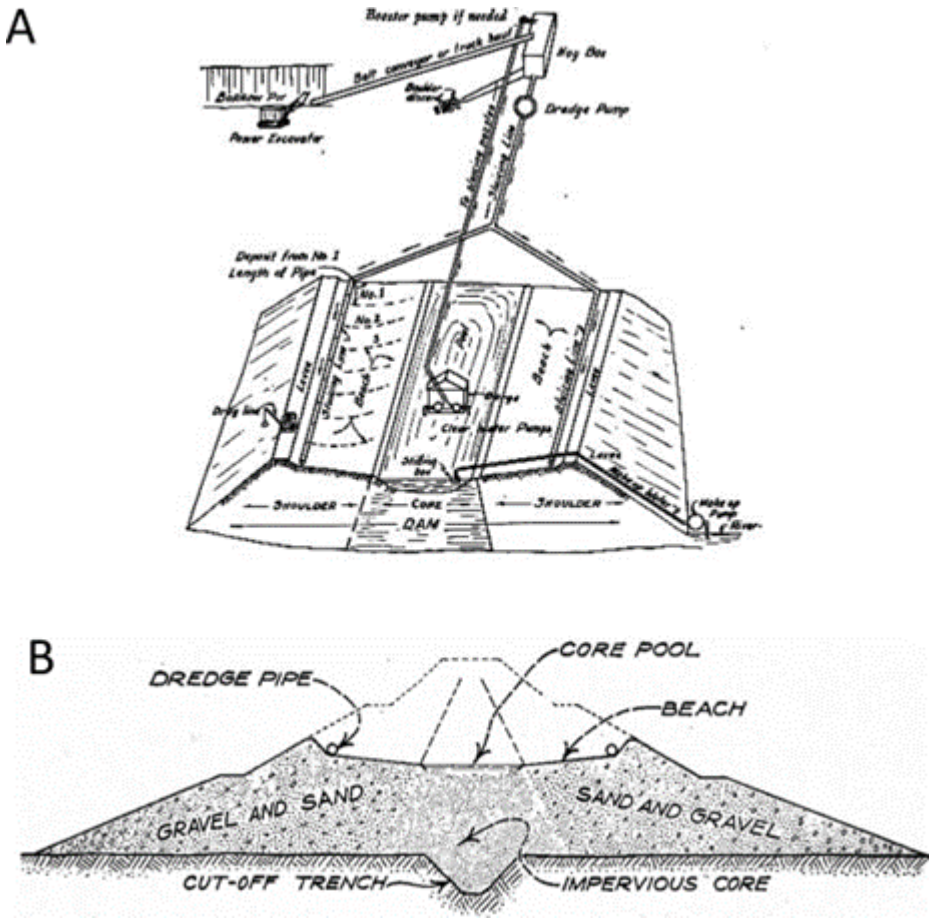
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II. EMBANKMENT DAM CONSTRUCTION – HISTORICAL PERSPECTIVES

Federal involvement in embankment dam construction in the US dates to 1820 (Billington et al, 2005). However, most of the large embankment dams in service in high seismic hazard potential regions were built in the twentieth century. Embankment dam design approaches and construction methods have evolved throughout the twentieth century based on lessons learned from embankment construction and their performance as well as advances in construction equipment. The different phases of embankment dam construction can be grouped into three major phases:

- Dumping Phase (dumped soil, hydraulic fill, or semi-hydraulic fill)
- Spreading Phase (spread soils in thicker lifts and leveling)
- Compacting Phase (use of thinner lifts and soil-type specific compactor)

Figure 2 shows common designs, techniques, and assumptions used in hydraulic fill embankments, which was a common construction technique for many dams constructed in the US in the nineteenth and twentieth century. Figure 2A shows typical hydraulic fill placement technique where sluiced material is separated and sent to both the upstream and the downstream areas. From there, sediment is discharged from opposing levees onto "beaches" to form the shoulders (shells). The fine-grained materials slowly settle out in a central pool to form the core pool. The clay core of the embankment serves as the key element of any embankment dam (Plummer and Dore, 1940). In Figure 2B, as material is discharged away from the upstream and downstream levees, the coarse-grained materials settle first with the fine-grained materials pooling and settling out in the core area. The assumption is that coarse, well-drained sand and gravel will form the shells (Morgan 1951).



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Figure 2: Common designs, techniques, and assumptions used in hydraulic fill embankments.
[Source: Cato and Rogers, 2018, after Plummer and Dore, 1940 and Morgan 1951]

Spreading compaction is a common construction method employed in many of the existing dams that were constructed prior to the mid-1960s, and in some cases, the practice continued after the mid-1960s. In the spreading compaction method, soils are spread in thicker lifts, and then a dozer with two tracks (such as D8 or D9 dozers) is used to level the surface to be prepared for the next layer of soil dump and leveling. As leveling is needed for the entire surface, this method requires several passes (such as 3 to 4 passes) of the dozer to complete one coverage of levelling. This is not a deliberate method of compaction and should not be relied on for compaction beyond the surficial few inches of these thicker layers. It also likely develops a potential condition of contrasting layers of hydraulic conductivities between the surficial few inches with the thicker layer, which provide favorable conditions for void redistribution in loose layers during liquefaction. Figure 3 shows the use of the spreading compaction method in construction of gravelly sand to sandy gravel shell zones of Folsom Wing Dam completed in 1955 in Sacramento County, California.



Figure 3: Spreading compaction at Folsom Wing Dam using D8 dozer to level the soil layers in thicker lifts (USACE)

Figure 4 through 6 show the evolution of construction of Isabella Dam in Kern County, California. Figure 4 shows the use of soil-filled carts carried by dozers to compact the embankment during the original construction that was completed in 1953. During the recent seismic retrofit of the Lake Isabella Auxiliary Dam completed in 2023, vibratory rollers were used for coarser-grained shell and filter and drain layers (Figure 5) and sheepsfoot rollers were used for the clayey core zone (Figure 6). Vibration in coarser-grained soils allow proper packing of particles and increased relative density. Similarly, use of sheepsfoot rollers for plastic fine-grained soils at or wet of the soil's optimum moisture content allows the particles to reorient, resulting in reduced hydraulic conductivity and increased strength.

An understanding of the evolution in embankment dam design and construction is an essential part of

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evaluation for seismic related

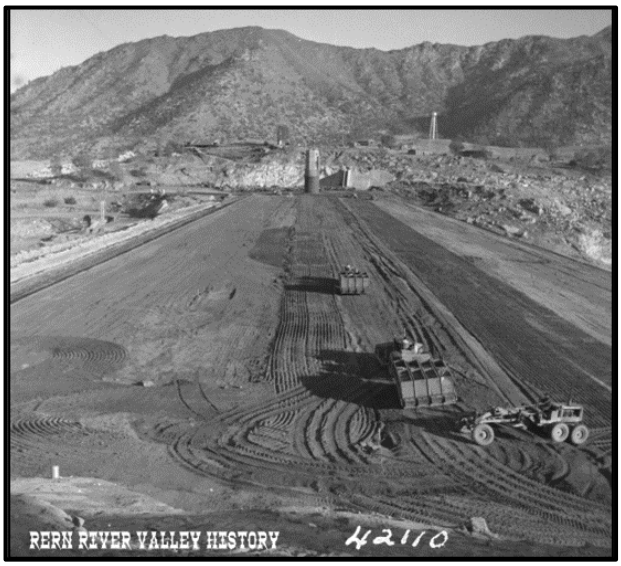


Figure 4: Use of soil-filled cart pulled by a dozer during construction of Isabella Dam (completed in 1953)



Figure 5: Use of vibratory roller to compact coarser-grained soils placed in thinner lifts in shell, filter, and drainage zones in Isabella Dam for USACE seismic retrofit project (completed in 2023)



Figure 6: Use of sheepsfoot roller to compact finer-grained soils placed in thinner lifts in core zones in Isabella Dam for USACE seismic retrofit project (completed in 2023)

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III. MODERN DAM DESIGN AND CONSTRUCTION MEASURES FOR IMPROVED SEISMIC RESILIANCY

Any new embankment dam with adequate design features should be demonstrated to have acceptable performance for seismic PFMs. The design should be developed, and construction measures should be implemented to ensure dam safety and functional reliability for design seismic loading conditions.

If an embankment dam is constructed using the current state of practice protocols for foundation preparation, embankment dam materials, zoning, and construction to design for non-seismic PFMs, it is likely to show higher resiliency against seismic PFMs. Seed (1979 and 1981) and FEMA (2005) provide lists of defensive measures for improving the seismic performance of embankment dams. Since then, several embankment dams were remediated to provide improved seismic performance. Most of these embankment dam remediation projects were implemented in California by utility companies (such as Pacific Gas and Electric Company [PG&E]) or water resource agencies (such as USBR).

The following are commonly used standard state of practice protocols for embankment dam construction that also improves seismic resiliency:

1. Removal and/or treatment (e.g. ground improvement) of soft, weak, or potentially liquefiable materials beneath the embankment core and shell. The depth of alluvium removal or in-situ improvement should be based on bedrock depth and alluvium characteristics. The removal should be extended down to dense or stiff soils (such as $N_{1,60,CS} \geq 30$) or bedrock to reduce the susceptibility to liquefaction and strength loss.
2. Perform additional foundation preparation by removing unsuitable materials and performing adequate grouting to improve seepage conditions. Multi-row grouting with close monitoring should be the preferred method for mitigation of underseepage through foundation layers.
3. Provide filters on fractured foundation rock to prevent concentrated leak erosion along the soil-rock interface or internal migration of the embankment into open defects in the foundation rock, if fractures are opened by an earthquake.
4. Provide wide engineered filter and transition zones adjacent to the core that are not susceptible to cracking to act as a crack stopper and provide adequate particle retention (i.e., self-healing).
5. Provide adequate drainage zones to safely lower the phreatic surface in the downstream shell, to manage leakage through core, and safely transport the water to a downstream exiting conveyance system.
6. Provide high-quality, free-draining rockfill shells, if feasible. Upstream filter zones are not mandatory as long as the shell and core are properly designed and constructed to be filter compatible. However, if present, properly designed filter-compatible wide filter zones upstream of the core are expected to help choke-off or dampen any cracks that may open and to prevent movement of particles eroded from the core.

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7. Construct a wide central core zone, constructed by emplacing plastic clayey materials, that are not susceptible to cracking, in controlled thinner lifts, using appropriate compaction equipment (such as a sheepsfoot compactor) and moisture conditions, to achieve a specified relative compaction and flare the core at the abutment contacts to provide longer seepage/leakage pathways. Performing suitable monitoring and preparing documentation of field construction/emplacement is important.

8. Construct filter, transition, and shell zones in thin lifts with filter compatible materials (across core-filter-transition-shell) using a vibratory compactor to achieve specified relative density for adequate compaction to reduce susceptibility to cracking and liquefaction. Perform suitable monitoring and prepare documentation of field construction/emplacement.

9. Use crest details and downstream slope protection that will reduce erosion potential from limited overtopping depths.

10. Stabilize and/or remove potentially unstable slopes and landslides near the dam abutments and appurtenant structures and around the reservoir rim to prevent slides into the reservoir that may block or damage conduits, spillways, or control towers or trigger seiches.

11. Provide special details for treating the embankment-foundation interface if the potential for fault movement in the foundation exists. In case of a mapped or potential fault within the dam footprint, other design measures such as a wider and efficient filter and drainage system, adequate freeboard, and a wider embankment may be needed. Not all potential fault movements can be suitably mitigated. Location, orientation, displacement mechanisms, and potential offset magnitude should be considered in evaluation of faults.

12. Shape foundation contacts to avoid abrupt changes in profile, overhangs, or large "stairsteps" by trimming and/or “dental” grouting, etc. that may lead to low-stress zones in the embankment, cracking, and concentrated leak erosion.

13. Provide filters or other measures to prevent concentrated leak erosion at the interface between the embankment and non-erodible embedded structures or features such as conduits, spillways, abutment walls, etc.

14. Provide adequate freeboard to account for post-construction settlements as well as potential settlement, slumping, or fault movements during an earthquake.

These standard design features and construction procedures are more routine for a new embankment dam. Unfortunately, most embankment dams in the US were constructed prior to these recent developments in design approaches (both seismic and non-seismic) and construction practice.

IV. KEY “DELTA’ AREAS FOR EVALUATION – AN EXAMPLE ZONED EMBANKMENT DAM

An example zoned embankment dam is presented in this paper to explain the delta (Δ) approach. Eleven primary areas of delta evaluation have been identified in this example embankment dam. As the purpose of this example is to explain the areas of potential differences or deficiencies, the topics and discussions are based on experience from multiple Federal and non-Federal embankment dams, not specific to any particular dam for all issues. The following are twelve areas of delta that would be useful to communicate the current conditions of the dam for decision making for the next steps.

Delta 1: Purpose, Critical Features, and Consequences
Delta 2: Design Standards/Criteria
Delta 3: Foundation Conditions (Geologic and Site Preparation)
Delta 4: Embankment Core (Construction and Performance)

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Delta 5: Filter and Drainage Zones
Delta 6: Embankment Shell Zones
Delta 7: Embankment-Abutment Contact Area
Delta 8: Instrumentation and Monitoring
Delta 9: Seismic Freeboard
Delta 10: Appurtenant Structures and Systems with Significance in Post-Earthquake Response and Performance
Delta 11: Post-Earthquake Response Preparedness and Ability
Delta 12: Site-Specific Issues

V. DESCRIPTION OF KEY DELTA AREAS: "EXISTING DAM' VS. "WHAT IF DESIGNED AND CONSTRUCTED NOW"

An evaluation of the delta approach based on differences between the current state of practice and the example existing dam is presented in a tabular format with description of (1) the current state of practice procedures and/or criteria with potential significance for seismic performance, (2) design, construction, and performance aspects of the existing dam, and (3) discussion on the impacts of such differences. Tables 1 through 11 presents examples based on common standards and protocols and potential deficiencies in embankment dams. In performing delta (Δ) approach evaluations of embankment dams, these tables should be considered as a useful example, however, the tables would need to be adapted to match project feature-specific conditions and standards and guidelines for each specific dam.

Delta-1: Purpose, Critical Features, and Consequences

Table 1. Example Evaluation of a Hypothetical Dam: Delta-1: Purpose, Critical Features, and Consequences

Item	General Information	Evaluation Considering Current State of Practice
Dam Type	Zoned Embankment Dam	
Design Period	1953-1955	Design in early 1950s did not include seismic considerations.
Construction Period	1957-1959	In the US, the era of large dam construction occurred between 1930s and 1960s. The use of vibratory rollers in dams started in mid-1960s. An embankment dam built in 1950s is likely to be poorly compacted, as it pre-dates use of modern construction methods and equipment.
Purpose of the Dam	Flood control, water supply for drinking and irrigation, hydroelectric, recreation	Functionality of this dam after an earthquake should be carefully considered, as it impacts drinking water and electricity for a large population. The impact of loss of irrigation and recreation may have a long-term effect on the local economy.

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Item	General Information		Evaluation Considering Current State of Practice
Number of Dams in the System and Downstream Levees	The subject dam is the main embankment dam. The system includes a concrete spillway structure adjacent to the dam, and 10 saddle embankment dams with heights ranging from 30 to 150 feet.		A system consisting of multiple dams has the potential for higher vulnerabilities from different dams, which in the absence of adequate resources is difficult to maintain. The overall risk may be higher.
Reservoir Capacity	1.1 Million acre-ft		A large reservoir capacity indicates that it will be difficult to drawdown in case of damage and consequence of loss of capacity will be adverse.
Maximum Dam Height (ft)	300 feet (measured from downstream toe to crest) 280 feet (measured from stream bed to crest)		If the stream bed is elevated from its original elevation at construction time, this suggests sedimentation at the toe. These sediments may indicate loss of actual reservoir capacity. It may provide a thick upstream surficial layer of very loose deposits, looser than original surficial soil or rock. It may indicate potential for longer runout distance in looser deposit than original rough surface.
Freeboard	The 10th percentile reservoir pool elevation annual duration exceedance results in 10 feet of freeboard The 50th percentile annual duration exceedance results in 60 feet of freeboard The summertime conservation pool results in 15 feet of freeboard		This reservoir stores spring runoff from snow melt and is located in a wet winter region. Therefore, both the summertime conservation pool and the 10th percentile exceedance pool elevations are high. The very low freeboard indicates that the dam was designed without understanding the need for adequate freeboard.
Downstream population at risk	During Design: Less than 1 thousand	Current: About 1.1 million (2020)	The dam was initially constructed, when the downstream population was low. If the dam was built today with the current large population at risk, it would have to adhere to much higher standards.
Critical Appurtenant Works [e.g. Spillway (Elevation and height above D/S toe), Low Level Outlet Works, and other Critical and Lifeline Features]	During Design: (1) Concrete Spillway: EL. XYZ (2) Unlined emergency spillway. (3) No low-level outlet, except hydroelectric tunnel with XY,000 cfs capacity (XX months to lower pool from spillway elevation to low level elevation).	Current: (1) Concrete Spillway: EL. XYZ (2) Unlined emergency spillway (never used). (3) No low-level outlet, except hydroelectric tunnel with 5,000 cfs capacity (6 months to lower pool from spillway elevation to low level elevation). (4) Critical and Lifeline Features Downstream:	(1). If there is no low-level outlet to lower the pool elevation below El. XYZ (only ZZ feet below crest), the dam should have a higher resiliency against an earthquake loading. This is critical, if coincident pool is high during an earthquake. (2). A large number of critical civil infrastructure has been built since construction of the dam. This infrastructure is important for the entire region, not only the immediate downstream area.

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Item	General Information		Evaluation Considering Current State of Practice
	(4) Critical and lifeline features downstream: 1 major highway. (5) Access to dam through left abutment, which abuts a concrete spillway. Right abutment abuts to rock slope and not accessible through roadways.	2 major highways (part of evacuation routes), 3 major hospitals, one major power plant, multiple data centers for major telecommunication and high-tech industry.	(3) If reservoir drawdown capacity is impaired due to an earthquake, emergency spillway may be needed to reduce potential for overtopping. An unlined emergency spillway, which has not been tested, may indicate potential issues.

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Delta-2: Design Standards/Criteria

Table 2. Example Evaluation of a Hypothetical Dam: Delta-2: Evaluation of Design Standards/Criteria

Evaluation Topic	Current Criteria or State of Practice [with Potential Significance for Seismic Performance]	Existing Dam Design, Construction, and Performance	Remarks on Potential Impacts of Differences Between Current Criteria/State of Practice and the Existing Dam
I. Design criteria for seismic loading	Operating Basis Earthquake ground motion (OBE-GM) at 475 years return period and the performance objective of little or no damage and no interruption of function; Maximum Design Earthquake Ground Motion (MDE-GM) with performance objective of no uncontrolled release or impounded water or liquid-borne solids. Site MDE return period is less than 1500 years (Cascadia Subduction Zone).	No design standards for seismic loading were used in the original design and construction in the 1950s. No national or local standards were-available.	(1) This embankment dam was designed and constructed without considering any seismic criteria. Potential impacts of a Cascadia Subduction Zone earthquake were discovered by geologist and sesimologists in late 1990s to early 2000s. (2) Because the dam was designed and constructed without seismic criteria and inadequate construction techniques, there is a potential for poor seismic performance.
II. Site investigations and parameter development for seismic design	Site investigation and site characterization with focus on potential contributing factors for poor seismic performance including fault study, liquefaction, soft soils, key embankment design features and construction criteria, abutment conditions, foundation cutoff, freeboard requirement, etc.	The original site investigation information is inadequate for modern seismic site characterization and seismic analyses. Subsequent investigations were also inadequate, as these were not performed for seismic PFMs, and did not consider state of practice protocols for seismic site invetsigations. .	Without reliable site-specific data, any seismic assessment is mostly based on engineering judgment. Site-specific seismic focused site characterization is essential for fact-based seismic analyses and evaluations.
III. Aftershock events	For critical project features, the performance requirements for analysis level earthquake ground motions are also applicable for aftershock events. Critical project features are engineering structures, natural site conditions, or operating equipment and utilities, etc., whose failure or damage during or immediately following an earthquake could	This dam site is located in a seismic environment, where a subduction zone earthquake may be followed by large crustal earthquakes or crustal earthquakes may follow significant aftershocks.	As the dam was not designed for seismic loading, the vulnerability to aftershocks may be high, if performance is marginal for mainshock events.

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Evaluation Topic	Current Criteria or State of Practice [with Potential Significance for Seismic Performance]	Existing Dam Design, Construction, and Performance	Remarks on Potential Impacts of Differences Between Current Criteria/State of Practice and the Existing Dam
	result in direct loss of life or significant economic losses due to loss of project service or functionality, emergency response and repair, disruption of lifeline facilities or access, and direct property damages.		
IV. Appurtenant Structures	If appurtenant structures or equipment (such as spillway gates and piers, inlet towers, outlet towers and tunnels, electrical and mechanical equipment) is required to lower the pool elevation after a major earthquake (such as after experiencing MDE-GM), it must be functional or operational after the earthquake event.	No design standards for seismic loading were used in the original design and construction of the appurtenant structures in the 1950s. No national or local standards were-available.	Appurtenant structures need to be functional after an earthquake that may result in damage to the dam requiring rapid pool lowering.
V. Other			

Delta-3: Foundation Conditions (Geologic and Site Preparation)

Table 3. Example Evaluation of a Hypothetical Dam: Delta-3:
Evaluation of Foundation Conditions (Geologic and Site Preparation)

Evaluation Topic	Current Criteria or State of Practice [with Potential Significance for Seismic Performance]	Existing Dam Design, Construction, and Performance	Remarks on Potential Impacts of Differences Between Current Criteria/State of Practice and the Existing Dam
I. The presence of mapped or potential faults in dam footprint or proximity	An active fault in the dam footprint or proximity may indicate the potential for near source effects such as (1) permanent foundation offset displacement and (2) strong ground motions (fling and pulse). Features such as wider drainage features, berm, etc. to address permanent offset	A mapped splay fault is located under the main embankment and near the right abutment of the dam. It has potential to rupture or offset with the main fault rupture. No design features were included to address potential fault rupture hazard.	In the absence of design features to minimize impacts of a fault offset, rupture, or near-source ground motions, the risk of potential failure is increased. In addition to immediate seismic deformations, delayed impacts such as the development of backward erosion piping (BEP)

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Evaluation Topic	Current Criteria or State of Practice [with Potential Significance for Seismic Performance]	Existing Dam Design, Construction, and Performance	Remarks on Potential Impacts of Differences Between Current Criteria/State of Practice and the Existing Dam
	and strong ground motions are utilized in modern design.		due to lack of adequate filter capability is also possible.
II. The presence of potentially liquefiable alluvium in the foundation	The state of practice for modern dam construction is removal of potentially liquefiable or cyclically weakening soils from the foundation and replacement by engineered fill to establish a competent foundation beneath the embankment. Ground improvement techniques (such as densification) can also be utilized.	The existing dam was constructed without removal of alluvium from the foundation. Shallow foundation layers underlying both the upstream and downstream shells consist of from 10 to 25 feet of potentially liquefiable sandy and gravelly soils.	The presence of potentially liquefiable and/or cyclic weakening soils is one of the most common flaws in embankment dams, and can contribute to upstream and/or downstream seismic deformations ranging from limited freeboard loss to flow slides with larger loss of freeboard (overtopping).
III. The presence of stream deposits, irregular rock, or other source(s) of significant geologic contrasts within the dam footprint	Many dams are constructed across rivers or streams, which likely consist of looser and/or softer deposits along the rivers/streams and denser or stiffer materials on the banks. The transition areas near the channels are locations of potentially significant stiffness contrasts. These areas are potential locations for transverse crack development. In the current state of practice, loose deposits from these channels are removed prior to placement and construction of the dam. Backfill is performed to reduce or prevent abrupt transition between loose and stiff zones.	The dam was constructed across a stream without removal of all loose/soft foundation materials. The width of the stream is about one-third (1/3rd) of the dam length. The stream is unaltered at the downstream toe area.	There are potential locations of transverse cracking near the known locations of contrasting geologic units. The existing channel is also very close to the downstream toe, which provides an easier freeface exit for deformed foundation and embankment soil movement (Figure 7).
IV. The presence of active or old landslides adjacent to the dam	There may be active or old landslides within the reservoir rims (distant from the dam or in proximity of the dam). An earthquake could potentially trigger these landslides. A landslide could trigger	An existing active landslide with slow movement (creep) is located near the right abutment of the dam.	The potential landslide mass volume is significant to (1) deposit sediments at the upstream of the dam and block the low-level outlet and spillway gates, (2) create waves, which may result in overtopping if coincident pool

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Evaluation Topic	Current Criteria or State of Practice [with Potential Significance for Seismic Performance]	Existing Dam Design, Construction, and Performance	Remarks on Potential Impacts of Differences Between Current Criteria/State of Practice and the Existing Dam
	overtopping by seiche and/or direct impact on dams. In current state of practice, active pre-existing landslides are evaluated for potential triggering during an earthquake event. Pre-existing landslides are either removed or mitigated prior to construction of a dam.		is high, (3) create openings or cracks in the right abutment area, which may result in rapid erosion of materials.
V. Foundation preparation and grouting	Proper foundation preparation (compaction, removal of unsuitable materials, etc.), drainage filters over fractured rock, and multi-line sequential grouting are performed in modern dams.	Only one line of grouting was performed and the grouting operation was not well documented. Downstream seepage is a regular concern, and monitoring to detect potential accelerating seepage is not regularly performed and/or reported.	Inadequate grouting may indicate an existing deficiency for BEP-type failure modes, conditions that may be exacerbated due to an earthquake.
VI. Abutment mapping, inspection and treatment. Monitoring of abutting fill placement. Etc.	Embankments are usually constructed with special considerations for compaction near steep abutment slopes and/or localized overhangs. Rock slopes are sculpted and prepared (e.g. trimming overhangs, slush grouting, excavation of poor materials, etc.) to provide good abutment contact conditions with embankment materials.	The embankment was constructed on a steep rock slope without removal of rock ledges or other abutment treatments.	Seismic displacements of the embankment at and near a steep rock abutment could lead to the potential for transverse through-cracking or contact gapping during an earthquake. This can lead to potential development of piping, and can be exacerbated by seismic deformations and displacements as well as post-earthquake settlements due to densification of loose/uncompacted embankment materials.
VII. Other			

Three Example Dams with Potential Foundation and Embankment Deficiencies:

Figures 7, 8, and 9 shows three examples of zoned embankment dams that have different types of potential distress conditions that may develop due to an earthquake. These examples are intended to illustrate how plans,

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sections and other figures can be used as supplemental tools to document and communicate differences between the design of an existing dam versus the current state of practice. Sometimes, potential deficiencies in an existing embankment dam can be documented by adding notes to plans, sections, and other figures.

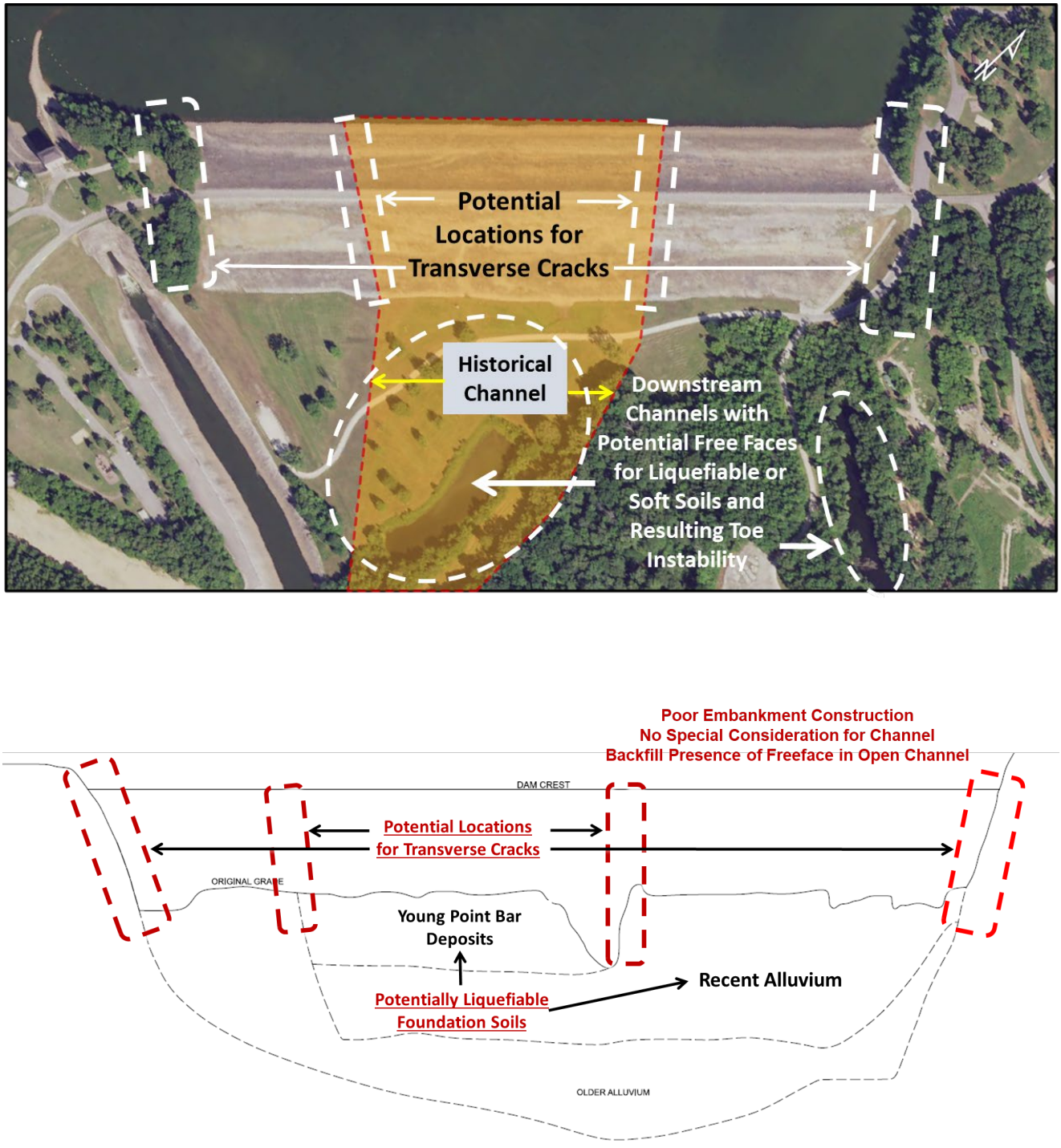


Figure 7: Example Dam 1 - Planview (top) and cross-section (bottom) of an embankment dam showing potential areas of distress due to an earthquake

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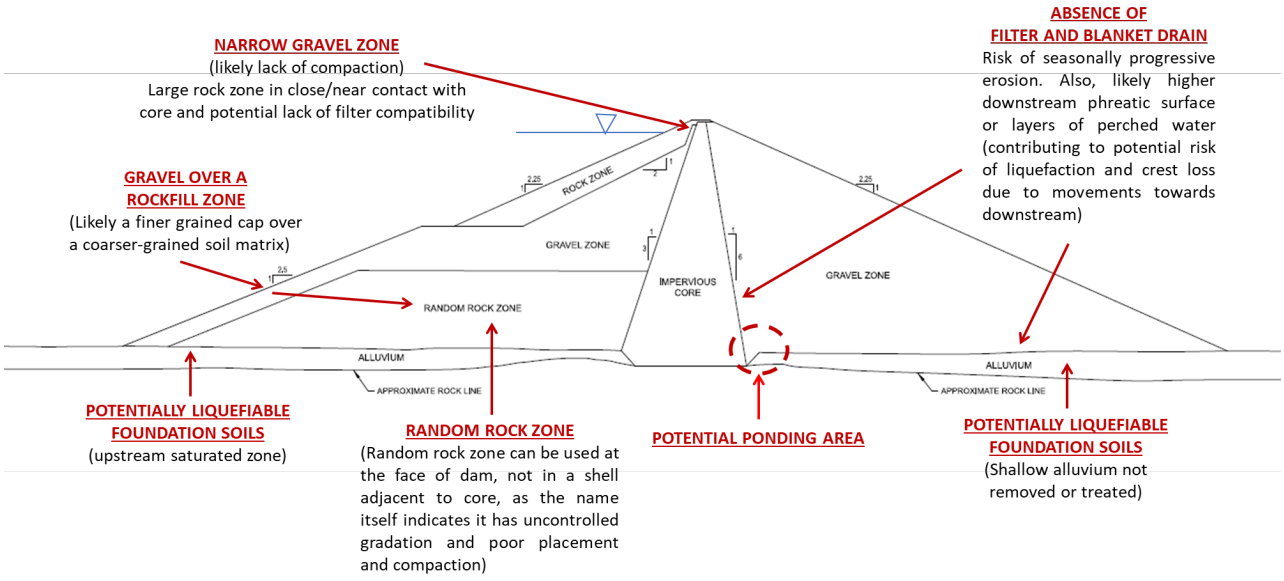


Figure 8: Example Dam 2 – Cross-section of an embankment dam showing potential areas of distress due to an earthquake

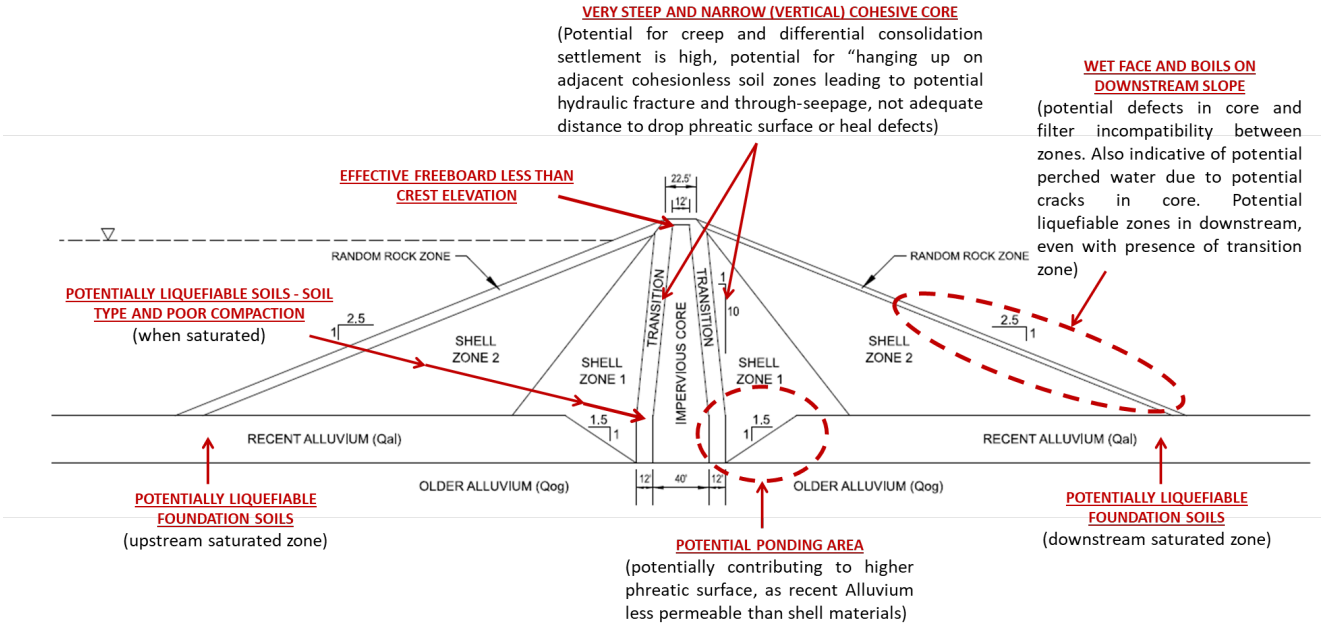


Figure 9: Example Dam 3 – Cross-section of an embankment dam showing potential areas of distress due to an earthquake

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Delta-4: Embankment Core (Design, Construction, and Performance)

Table 4. Example Evaluation of a Hypothetical Dam: Delta-4:
Evaluation of Embankment Core (Design, Construction, and Performance)

Evaluation Topic	Current Criteria or State of Practice [with Potential Significance for Seismic Performance]	Existing Dam Design, Construction, and Performance	Remarks on Potential Impacts of Differences Between Current Criteria/State of Practice and the Existing Dam
I. Embankment Core: material type, lift thickness, compaction equipment, moisture conditions	Clayey materials (not too high plasticity or too low plasticity and with lower sensitivity) placed in thinner lifts (≤ 12 - inches), compaction moisture conditions with slightly higher than optimum moisture content (OMC) (< +5 percent of OMC) and "kneading" compaction (compacted with sheepsfoot roller.).	Clayey core was constructed with moisture +10 percent of optimum with thick lifts (24-inches) and spread-compacted with 1-pass of D8 dozer.	The embankment core has a very high water content and was compacted poorly and potential for deformation during an earthquake may be high due to low strength. It may also be potentially prone to shrinkage and cracking upon drying. Seismic loading might exacerbate cracking.
II. Embankment Core: width at top, upstream and downstream slopes	Dam cores with wider crest widths (such as 30 feet or more) and flatter slopes (such as 0.5H:1V or flatter).	Dam core has a relatively narrow crest width (12 feet) with very steep slopes (0.17H:1V or 1H:6V).	A narrow vertical core may "hang up" on adjacent cohesionless soil zones, increasing the risk of cracking and/or hydraulic fracture. A core with a wider crest and flatter slopes would provide additional protection against potential cracking, creep and BEP (pre- and post-earthquake) after an earthquake. Also, a wider core with a flatter slope would provide additional protection against erosion if shell materials are removed during an earthquake (exposed core slope).
III. Embankment core - presence of potentially liquefiable materials	Dam cores should be constructed of plastic fine soils with low potential for liquefaction.	Embankment core of a portion of the dam was constructed with low plasticity silt to silty sand with poor compaction effort and thicker lifts due to lack of suitable borrow sources.	Potentially liquefiable core would have higher potential for deformations including freeboard loss, cracking, etc.
IV. Other			

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Delta-5: Filter and Drainage Zones

Table 5. Example Evaluation of a Hypothetical Dam: Delta-5:
Evaluation of Filter and Drainage Zone (Design, Construction, and Performance)

Evaluation Topic	Current Criteria or State of Practice [with Potential Significance for Seismic Performance]	Existing Dam Design, Construction, and Performance	Remarks on Potential Impacts of Differences Between Current Criteria/State of Practice and the Existing Dam
I. Presence of downstream filter and filter geometry and location	Downstream filter and blanket drains are essential components of modern embankment dams.	Downstream filter zone was included in design.	
II. Downstream filter – material type, lift thickness, compaction equipment, moisture conditions	Coarse-grained soils compacted with vibratory roller with adequate relative compaction such that particle breakage (and increase of finer particles) is prevented, and liquefaction potential is low. A wider filter zone indicates that proper compaction equipment can be used. A wider filter zone may have materials to prevent cracks to widen and deteriorate.	Filter layer is narrow (7 feet), constructed with thicker lifts of 24-inches and spread compacted using one pass of a D8 dozer.	Poorly compacted filter layer would have potential for liquefaction and deformations.
III. Filter compatibility between filter zone and adjacent transition and/or shell zone materials. Full continuity of filter compatibility from core to exit drain.	Filter material gradations should have compatibility with both core and shell materials.	Even though the particles are progressively larger from core to filter to shell, these layers do not meet modern filter compatibility criteria.	In the absence of filter compatibility, migration of particles from core to filter and filter to shell may occur. Finer particle migration may result in a higher phreatic surface downstream (e.g. higher potential for liquefaction) and potential BEP conditions.
IV. Presence of downstream blanket drain and collection system	Blanket drains safely convey water that seeps through core and filter to downstream toe area.	A downstream blanket drain was not included in the design and was not constructed.	In the absence of a properly designed blanket drain, the potential for a higher phreatic surface in the downstream shell can lead to the potential for liquefaction and other conditions that contribute to seismic deformations and freeboard loss.

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Evaluation Topic	Current Criteria or State of Practice [with Potential Significance for Seismic Performance]	Existing Dam Design, Construction, and Performance	Remarks on Potential Impacts of Differences Between Current Criteria/State of Practice and the Existing Dam
V. Downstream drain – material type, lift thickness, compaction equipment, moisture conditions	Coarse-grained soils compacted with vibratory roller with adequate relative compaction such that particle breakage (and increase of finer particles) is prevented, and liquefaction potential is low. An adequately thick drainage layer is required to provide drainage capacity. The drainage layer is compacted with higher relative compaction or relative density requirements than the filter zone considering location of the drainage layer at the base of downstream shell, which may indicate greater contributions to seismic deformations.	A downstream blanket drain was not included in the design and was not constructed.	Same impacts as E4.
VI. Filter compatibility between foundation-drain and between drain and shell	Filter compatibility between foundation and blanket drain materials is an important modern criterion.	Filter compatibility between the foundation and blanket drain materials and between drain and shell do not meet modern filter criteria.	In the absence of filter compatibility between blanket drain and foundation layer and between blanket drain and shell, banket drain may get clogged and the downstream phreatic surface may rise, leading to potential for liquefaction and other conditions that can contribute to seismic deformations and potential for freeboard loss.
VII. Presence of upstream filter/transition zone	Upstream filter/transition layer, if designed, needs to be filter compatible with both shell and core. A transition layer may allow use of coarser shell materials (such as rockfill), which might otherwise not be filter compatible with the core. These zones may help in crack stopping and healing.	An upstream transition/filter layer is present.	The transition/filter layer that is present is susceptible for deformations, which could create longitudinal cracking along the core-transition interface and may potentially expose the core, if soil deforms.
VIII. Upstream filter/transition zone – material type,	Coarse-grained soils compacted with vibratory roller with adequate relative compaction such that particle breakage (and	Filter layer is narrow (7 feet), constructed with thicker lifts of 24-inches and spread	Poorly compacted filter layer would have potential for liquefaction. Subsequent static settlement may result

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Evaluation Topic	Current Criteria or State of Practice [with Potential Significance for Seismic Performance]	Existing Dam Design, Construction, and Performance	Remarks on Potential Impacts of Differences Between Current Criteria/State of Practice and the Existing Dam
lift thickness, compaction equipment, moisture conditions	increase of finer particles) is prevented, and liquefaction potential is low. A wider filter zone indicates that proper compaction equipment can be used. A wider filter zone may have materials to prevent cracks to widen and deteriorate.	compacted using one pass of D8 dozer.	differential deformations. Differential settlements between the narrow filter and the two adjacent zones can lead to cracking.
IX. Other			

Delta-6: Embankment Shell Zones

Table 6. Example Evaluation of a Hypothetical Dam: Delta-6:
Evaluation of Shell Zone (Design, Construction, and Performance)

Evaluation Topic	Current Criteria or State of Practice [with Potential Significance for Seismic Performance]	Existing Dam Design, Construction, and Performance	Remarks on Potential Impacts of Differences Between Current Criteria/State of Practice and the Existing Dam
I. Upstream shell – material type, lift thickness, compaction equipment, moisture conditions, numbers of passes, etc.	Coarse-grained soils compacted with vibratory roller such that materials are suitably densified with relatively thinner lifts (12-inches to 18-inches) and sufficient passes. Adequate compaction provides dense packing, which helps reducing seepage concerns, erosion potential, liquefaction potential, and seismic deformation potential.	Shell was constructed with thicker lifts of 36-inches to 48-inches and spread compacted using one levelling pass of a D8 dozer. Materials are sandy gravel to gravelly sand.	Poorly compacted shell zone would have potential for liquefaction and resulting deformations. Large deformations (movements) may expose and/or unbrace the core, which usually has steep slopes and may not be stable once exposed. Larger slide displacements can carry away the crest section of the dam. Lesser sliding displacements can cause crest loss accompanied with cracks and blocky failures occur. Case history: Lower San Fernando Dam in 1971.
II. Presence of different sub-layers within upstream shell	Construction of shell zones with thinner lifts with vibratory roller should reduce potential for looser sub-layers. Suitable compaction should be performed for all lifts during construction; one or more looser lifts could pose a potential hazard.	Construction techniques of shell zone (thicker lifts with inadequate compaction efforts) indicates that potential for looser sub-layers is high.	It is common to find denser and looser soil sub-layers with varying thicknesses in upstream shells due to poor compaction efforts in older dams compacted with tracked or rubber tired non-vibratory compaction equipment which produces poor compaction of potentially liquefiable soil types, especially in lower portions of a compaction lift.

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Evaluation Topic	Current Criteria or State of Practice [with Potential Significance for Seismic Performance]	Existing Dam Design, Construction, and Performance	Remarks on Potential Impacts of Differences Between Current Criteria/State of Practice and the Existing Dam
			This often results in loose and potentially liquefiable soils, and also to layered soils potentially vulnerable to void redistribution and resulting low post-liquefaction residual strengths.
III. Downstream shell – material type, lift thickness, compaction equipment, moisture conditions, numbers of passes, etc.	Coarse-grained soils compacted with vibratory roller such that materials are suitably densified with relatively thinner lifts (12-inches to 18-inches) and sufficient "passes". Adequate compaction provides dense packing, which helps reducing seepage concerns, erosion potential, liquefaction potential, and seismic deformation potential.	Shell was constructed with thicker lifts of 36-inches to 48-inches and spread compacted using one "levelling" pass of D8 dozer. Materials are Sandy Gravel to Gravelly Sand.	Poorly compacted shell zone would have potential for liquefaction and resulting deformations. Large deformations (movements) may expose and/or unbrace the core, which usually has steep slopes and may not be stable once exposed. Larger slide displacements can carry away the crest section of the dam. Lesser sliding displacements can cause crest loss accompanied with cracks and blocky failures occur. Downstream deformations may be less than upstream; however, would depend on site-specific conditions.
IV. Presence of different sub-layers within downstream shell	Construction of shell zones with thinner lifts with vibratory roller should reduce potential for looser sub-layers. Suitable compaction should be performed for <u>all</u> lifts during construction; one or more "looser" lifts could pose a potential hazard.	Construction techniques of shell zone (thicker lifts with inadequate compaction efforts) indicates that potential for looser sub-layers is high. Due to absence of filter and proper drainage, downstream phreatic surface is higher than expected.	In is common to find denser and looser soil sub-layers with varying thicknesses in downstream shells due to poor compaction efforts in older dams compacted with tracked or rubber tired non-vibratory compaction equipment which produces poor compaction of potentially liquefiable soil types, especially in lower portions of a compaction lift. This often results in loose and potentially liquefiable soils, and also to layered soils potentially vulnerable to void redistribution and resulting low post-liquefaction residual strengths.
V. Other			

<u>Delta-6 Examples:</u>

The importance of the dam core, and not just the shell zones, for the seismic performance of embankment dams has not been fully appreciated until more recently. Many older dams have thin cores with steep sides. In many

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cases, even though the core was constructed relatively well, the shell zones were constructed simply by dumping or spreading, i.e. without deliberate compaction effort. Modern measures such as thinner lifts, proper compaction equipment, proper soil gradations, and proper moisture conditions were not used. Figure 10 shows results from nonlinear seismic analysis of the Lower San Fernando Dam upstream flow slide case history, where both the clayey core as well as the upstream shell lost significant strength due the 1971 San Fernando earthquake (Chowdhury et al., 2019). Seismic analysis with residual strength for both the core, as well as the shell zones, resulted in a good match with the observed performance for this iconic case history. Figure 11 shows similar seismic analyses of an existing dam, where a flow slide occurs in the upstream shell, exposing the core to direct contact with reservoir loading (Chowdhury et al., 2021). The remaining core slope is steep and may not be stable, as the core is also expected to develop transverse and longitudinal cracks.

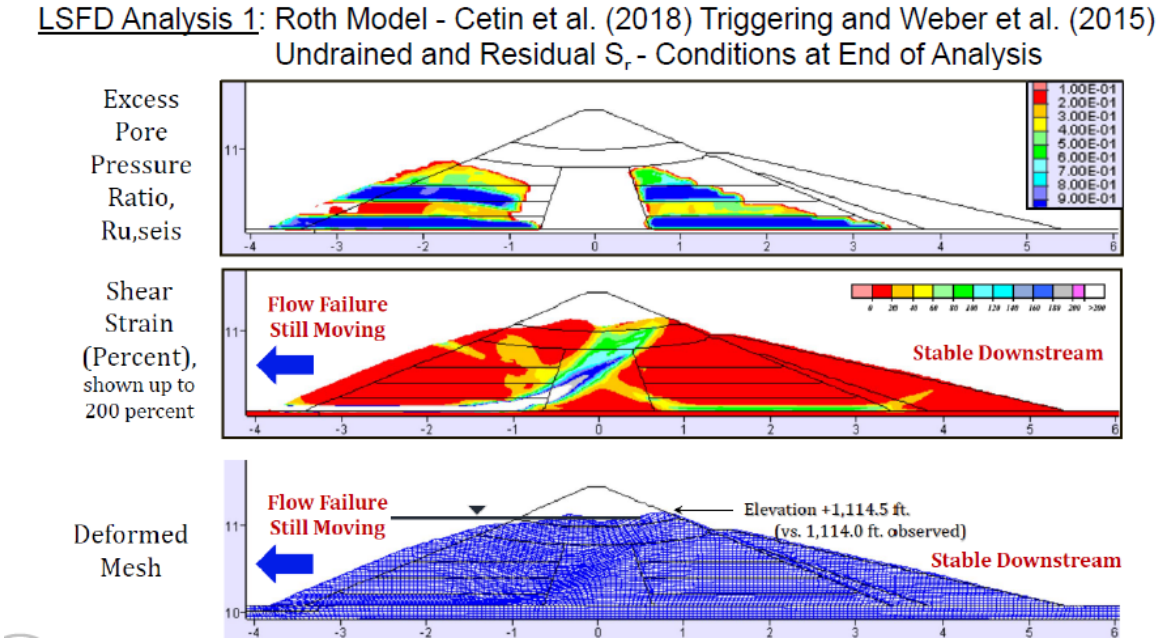


Figure 10. Excess pore pressure ratio, shear strain, and deformed mesh from the Lower San Fernando Dam Analysis matching closely with the remaining hill crest elevation (Chowdhury et al., 2019)

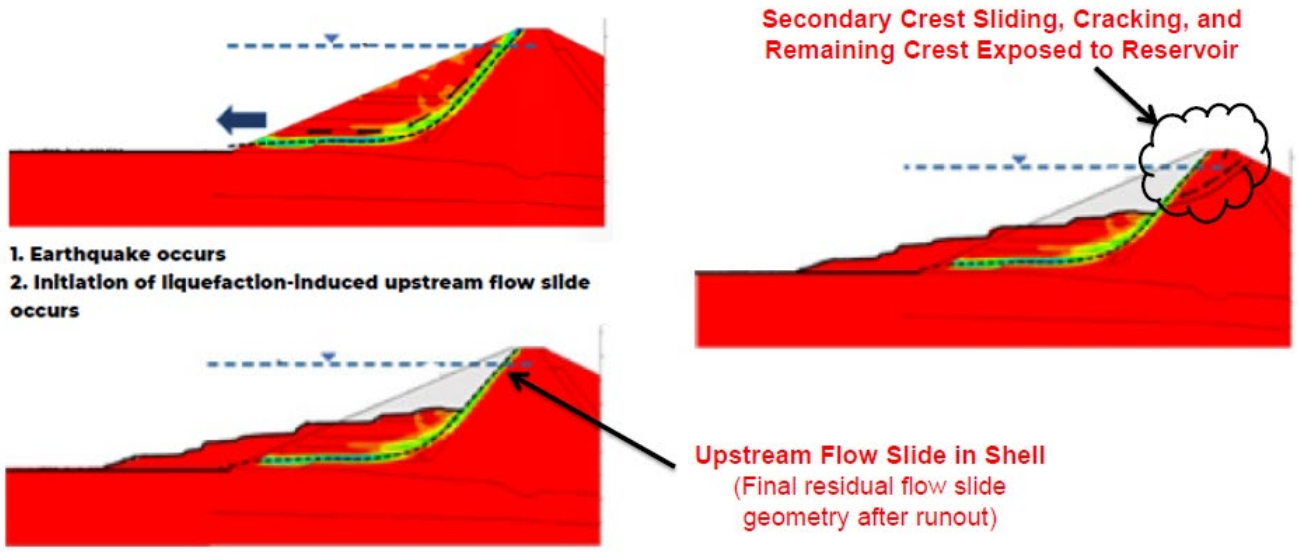


Figure 11. Semi-schematic illustration of seismic soil-liquefaction induced upstream flow slide potential failure (Chowdhury et al., 2021)

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Delta-8: Instrumentation and Monitoring

Instrumentation for monitoring dam performance is a critical and essential feature of the dam design. Without an appropriate instrumentation and monitoring program, potential developing distress conditions may go undetected. These pre-existing distress conditions will, in almost all cases, be exacerbated during an earthquake. An existing instrumentation and monitoring program may indicate potential distress conditions that may exacerbate during an earthquake. In delta (Δ) approach evaluation, this category does not directly show a delta (Δ) between the current state of practice and the existing dam, however, it shows the results of inadequate design and construction reflecting existing Δ, that may exacerbate during an earthquake.

Table 8. Example Evaluation of a Hypothetical Dam: Delta-8:
Evaluation of Instrumentation and Monitoring

Evaluation Topic	Current Criteria or State of Practice [with Potential Significance for Seismic Performance]	Existing Dam Design, Construction, and Performance	Remarks on Potential Impacts of Differences Between Current Criteria/State of Practice and the Existing Dam
I. Strong Motion Instrumentation	Strong motion instruments are located at various locations on the dam and in surrounding area to provide (1) reference or baseline ground motions at the site and (2) dynamic response of a site showing impacts of ground motions propagating through foundation and embankments. These instruments are important for post-earthquake response planning and execution, as well as future evaluation of dam performance.	No seismic instrument has been installed at the site.	Without seismic instrumentation, emergency data on seismic loading at the site and an assessment of dynamic response of the dam will need to be estimated based on regional seismic data from USGS, in-person observation, and other instrument monitoring.
II. Instrumentation data for phreatic surface	Instrumentation at the crest, downstream, and toe are important to evaluate the phreatic surface on the downstream side of a dam.	Existing (limited) piezometers indicate higher than expected phreatic surface on the downstream slope.	The high phreatic surface, even with filter and blanket drain, may indicate filter incompatibility and clogging of filters and blanket drains. Higher downstream phreatic surface may increase liquefaction potential and increased crest loss due to potential embankment deformations towards downstream in addition to potential deformations towards upstream.
III. Instrumentation data for deformations	Instrumentation data for deformations may help monitor any slope stability issues prior to an earthquake and identify internal damage due to an earthquake. Some damage may not manifest on surface.	Existing instruments indicate a deeper shear plane with slow movements.	The existing instrument data may indicate slower movement of the sliding mass in static conditions, however, any movement along pre-existing sliding planes may be exacerbated due to an earthquake.

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Evaluation Topic	Current Criteria or State of Practice [with Potential Significance for Seismic Performance]	Existing Dam Design, Construction, and Performance	Remarks on Potential Impacts of Differences Between Current Criteria/State of Practice and the Existing Dam
IV. Instrumentation data for settlement	Settlement monitors near the abutments, and along the crest (along interfaces of different zones), and transverse to the dam axis are important to monitor and characterize settlements in different zones.	Existing settlement monitors indicate settlement of upstream transition and shell zones. Cracks are also visible on surface near these differential settlement locations.	Differential settlement across the interface and cracks on surface indicate post-construction settlements. These may indicate pre-earthquake existing shear planes with potential weaker interfaces. Transverse cracks across different zones and abutment may be exacerbated during an earthquake.
V. Observations from dam site	Detailed observations and recording by experienced personnel may help to identify issues that may require further inspection, data collection and instrumentation, and evaluation.	A hummocky slope surface of a poorly constructed dam.	This may be an indication of poor compaction of the dam, which with cycles of saturated conditions and subsequent drying may create uneven settlement of the surficial portions of the shell. However, in some cases, these may indicate material loss (such as development of piping erosion, segregation of finer particles, etc.). These are indicative of poor dam design, construction, and maintenance, which may exacerbate during an earthquake.
VI. Other			

Delta-9: Seismic Freeboard

Table 9. Example Evaluation of a Hypothetical Dam: Delta-9: Evaluation of Seismic Freeboard

Evaluation Topic	Current Criteria or State of Practice [with Potential Significance for Seismic Performance]	Existing Dam Design, Construction, and Performance	Remarks on Potential Impacts of Differences Between Current Criteria/State of Practice and the Existing Dam
I. Allowable freeboard	Freeboard for non-seismic conditions is determined based on potential post-construction settlement and hydrologic conditions. Freeboard needs to be checked against potential	Only 10 feet (~3 percent of height) freeboard available for this 300-foot tall dam for 3 to 6 months of a year.	The freeboard is inadequate and potential for overtopping due to earthquake-induced crest loss and cracking may be high, considering other potential deficiencies in embankment design and construction for

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	crest loss due to an earthquake. Available seismic freeboard should be demonstrated (by analyses) to be able to safely retain the reservoir after potential seismic deformations (deviatoric and volumetric) and seismic cracking, etc. for design-level seismic events. Note that the California DSOD recommends at least 0.05H+5 ft of freeboard for new dams. Cracking depth needs to be evaluated along with coincident pool.		seismic PFMs.
II. Continuity of low permeability core safely above the maximum reservoir elevation within the freeboard depth.	Within the freeboard depth, the low permeability core materials are usually extended up to the crest to slow down erosional damage due to overtopping.	The top of the core is 7 feet below the crest elevation. Erodible sand and gravel materials are in this near crest area.	The available freeboard after an earthquake should be measured from top of impervious core, i.e., 7 feet less than crest level.

Delta-10: Appurtenant Structures

Table 10. Example Evaluation of a Hypothetical Dam: Delta-10: Evaluation of Appurtenant Structures

Evaluation Topic	Current Criteria or State of Practice [with Potential Significance for Seismic Performance]	Existing Dam Design, Construction, and Performance	Remarks on Potential Impacts of Differences Between Current Criteria/State of Practice and the Existing Dam
I. Spillway with capability to lower reservoir elevation in an emergency scenario	Regular side channel spillway is an important feature of embankment dams to provide flood control	Gated spillway system is available. Spillway capacities are suitably adequate.	An earthquake may damage spillway capability, if not properly designed and constructed (such as crack in rock-concrete lining)
II. Location of outlet towers/inlet tower compared to spillway	Ideally intake tower or outlet tower with tunnels should not be in close proximity of spillway. 0	Intake tower in the dam is in close proximity to spillway gates.	Earthquake damage to intake tower may impact the gates spillway.
III. Presence of low elevation outlet tunnel/conduit [elevation, condition, and expected survivability/availability after an earthquake]	A low level outlet is important for lowering reservoir pool in case of emergency.	No existing low level outlet available.	The ability of lowering reservoir pool to prevent further damage of the dam using low level outlet is not available.
IV. Special care around tunnel, pipe, outlet, etc. [such as backfill]	Any tunnel, utility crossing, etc. require special considerations such as backfill type, compaction, grouting, etc.	Tunnel backfill is granular even in core and poor compaction due to shape of the tunnel, etc.	Potential for liquefaction, differential settlement, and backward erosion piping (BEP).

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Evaluation Topic	Current Criteria or State of Practice [with Potential Significance for Seismic Performance]	Existing Dam Design, Construction, and Performance	Remarks on Potential Impacts of Differences Between Current Criteria/State of Practice and the Existing Dam
V. Other			

Delta-11: Post-Earthquake Response Preparedness and Ability

Table 11. Example Evaluation of a Hypothetical Dam: Delta-11:
Evaluation of Post-Earthquake Response Preparedness and Ability

Evaluation Topic	Current Criteria or State of Practice [with Potential Significance for Seismic Performance]	Existing Dam Design, Construction, and Performance	Remarks on Potential Impacts of Differences Between Current Criteria/State of Practice and the Existing Dam
I. Expected post-earthquake operational capability of control tower equipment, etc. at MDE-GM	The outlet structure should be reliably safe for use in the immediate aftermath of a seismic event. Equipment in inlet or outlet towers are anchored to assure operational capability after an earthquake.	No assessment of equipment suitability was performed and these are not anchored.	Loss of ability to operate an outlet tower could happen after an earthquake
II. Post-earthquake inspection and remediation plan	Post-earthquake response protocols are documented to identify, assess, and respond to potential damage to arrest potential failure mode prevent uncontrolled release).	No specific response protocol is available considering the existing dam's characteristics.	Post-earthquake activities may be challenging due to the absence of proper planning. This highlights the need for good performance of the dam during an earthquake to reduce dependence on success of post-earthquake response.
III. Emergency Action Plan (EAP)	An EAP is a formal document that identifies potential emergency conditions at a dam and specifies actions to be followed to minimize loss of life and property damage (see FEMA P-64). Post-earthquake activities and actions are usually included planned and resources are identified, and preparations are taken as per EAP for an earthquake event resulting in poor performance of a dam.	No EAP for the dam is available.	Absence of an EAP for an older dam constructed in the 1950s should not be considered acceptable.
IV. Other			

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Delta-12: Site-Specific Considerations

Every dam has unique site-specific issues that are not usually covered or anticipated by commonly used criteria or standards. These site-specific issues are related to geologic conditions, borrow sources, material processing, workmanship, construction practice, designer, and construction team skills, etc. The examples provided in Delta 1 through 11 do not constitute a comprehensive list for areas of delta approach evaluation. Independent evaluation of site-specific (or dam-specific) conditions is an important aspect of this Delta (∆) approach.

VI. CONCLUSIONS

The Delta approach is a qualitative tool to assess the contributing factors for potential performance of an existing dam during an earthquake. It requires a comprehensive qualitative evaluation of the system including the dam foundation, embankment, abutments, spillway capacity, consequences, etc. It requires practitioners to understand historical dam design and construction practices and techniques that are inadequate, and the current state of practice that appropriately accounts for potential seismic PFMs. The Delta (∆) approach provides a transparent communication tool between practitioners and decision makers.

The results can be used to help screen seismic PFMs of existing embankment dams for additional investigation and evaluation to reduce uncertainty and better characterize the performance risk and the need for interim risk-reduction measures. The Delta (∆) approach can help to evaluate which contributing factors can be further investigated and analyzed using current state of practice investigations and analytical tools (such as foundation and embankment conditions). Also, there may be conditions that would benefit from modification decisions based on engineering judgment, as the deficiencies are so apparent and further analysis may not add much value (such as need to anchor the electrical and mechanical equipment in an inlet or outlet tower to provide post-earthquake functional capability).

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Dr. Khaled Chowdhury has over 23 years of experience in evaluation, design, and construction of infrastructure projects. He currently provides technical leadership on several major dams and levees evaluation and design projects nationwide, addressing static and seismic potential failure modes. Khaled contributed or is currently contributing to development of several USACE and California DWR guidance and regulatory documents on dams and levees. Khaled earned his PhD from the University of California, Berkeley under supervision of Professor Raymond B. Seed. Khaled's research and practice areas include site characterization, soil liquefaction, residual strength, seismic deformation analyses, seepage cutoff walls, and levee and dam design and construction and he has about 30 technical papers on these topics. Khaled is a member of seismic cadre of the USACE, which has been formed to evaluate seismic risk of USACE dams. Khaled is the lead instructor for USACE Prospect Course on seismic stability of embankment dams.

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Tim O'Leary has 28 years of geotechnical and dam and levee safety-related experience. He provides technical expertise with respect to geotechnical engineering and dam and levee safety risk assessment and risk management, develops methodology and tools, and prepares risk-informed guidance documents. He was a primary author of the joint-U.S. federal agency document Best Practices in Dam and Levee Safety Risk Analysis and the U.S. Federal Emergency Management Agency's manual P-1032 Evaluation and Monitoring of Seepage and Internal Erosion. He has a Bachelor of Science degree in Civil Engineering and a Master of Engineering degree in Geotechnical Engineering from the University of Louisville.

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Erica Medley is a Certified Engineering Geologist and Registered Geologist in the state of Oregon. She has worked for the U.S. Army Corps of Engineers for 11 years as an engineering geologist and is currently employed at the USACE Risk Management Center in Lakewood, Colorado. Medley graduated with a BS in Geological Sciences from Indiana University, a MS in Geological Sciences from Portland State University, and a ME in Geotechnical Engineering and Engineering Geology from Missouri Science and Technology. Erica has demonstrated expertise in site characterization, geotechnical engineering, field investigations, geologic mapping, and risk assessments.

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David Serafini is both a Registered Professional Civil Engineer and Registered Geotechnical Engineer in the State of California. David has over 21 years of USACE experience between the Dam Safety Modification Mandatory Center of Expertise, the Sacramento District, and the South Pacific Division Dam Safety Production Center. He started his USACE career working on both the MilCon and Civil Works programs. He has since worked primarily on dam and spillway evaluations and modifications including Isabella Dam, Success Dam, Tule River Raise, and the Folsom Joint Federal Project. He also provided key support during and following the Oroville Dam Spillway Incident. David has served as the Geosciences Branch Chief for the SPD DSPC and was the Lead Engineer on the Isabella Dam Safety Modification Project (DSMP) prior to joining the DSMMCX.

Ethan Dawson, PhD, Senior Geotechnical Engineer, South Pacific Division – Dam Safety Production Center in Sacramento, USACE.

Ethan Dawson has over 25 years of experience in numerical modeling of geotechnical structures including dams, tunnels, deep foundations, pipelines and waterfront retaining structures. Ethan specializes in seismic deformation analyses, soil dynamics, and constitutive modelling. Ethan Dawson earned his BS degree from Princeton University and MS and PhD from the University of Minnesota. Ethan was a student of Peter Cundall, the author of the FLAC, FLAC3D, UDEC and PFC programs, and has made significant contributions to the development of FLAC and FLAC3D. After working at Itasca Consulting Group, he worked at Dames & Moore/URS/AECOM for 20 years before joining the USACE in 2020. Ethan is a member of seismic cadre of the USACE, which has been

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formed to evaluate seismic risk of USACE dams.

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Dr. Tiffany Adams is a registered professional engineer with more than 25 years of geotechnical and dam and levee safety-related experience. She provides technical expertise with respect to geotechnical engineering and dam and levee safety risk assessment and risk management, serving as a technical specialist in embankment dam and levee performance under static and seismic loading. Her experience is predominately focused on analysis, technical oversight, and third-party expert review of site and material characterization, seepage evaluation, and stability evaluation and deformation modeling under static and seismic conditions. She has a B.S. degree in Civil Engineering from Oregon State University and M.S. and Ph.D. degreed in Civil Engineering with a geotechnical specialty from Virginia Tech.

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Olivia Finlay is a Department of Army Fellow and Geologist in Training at the USACE Walla Walla District in Walla Walla, Washington. Olivia earned undergraduate degrees in mathematics and geology from Western Washington University. Olivia is assisting USACE HQ and Districts in many enterprise-based tasks.

Georgette Hlepas, PhD, PE, Geotech, Geology, and Materials Community of Practice Lead at USACE Headquarters

Dr. Georgette Hlepas leads the US Army Corps of Engineers (USACE) Geotechnical, Geology, and Materials Community of Practice (GGM CoP). She has over 15 years of experience working as a geotechnical engineer on a variety of large civil works modification projects including dams, levees, and confined disposal facilities. Dr. Hlepas has focused much of her career on advancing the state-of-practice in project performance monitoring and performance evaluation. Dr. Hlepas is currently serving as a Board of Directors member for the USSD, the Chairperson for the USSD Monitoring of Dams and their Foundations Committee and the Surveillance and Monitoring US representative for ICOLD. In addition, she is a member of multiple committees at DFI. She has a master's degree in civil engineering from Columbia University in New York, a doctorate degree in civil engineering from the University of Illinois at Chicago and is a licensed Professional Engineer in the state of Illinois.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

Numerical Modeling and Remediation Design of a Hydraulic Fill Embankment with Liquefiable Foundation Under Extreme Seismic Loading

GAUEN R. ALEXANDER, PH.D., P.E., BUREAU OF RECLAMATION

ABSTRACT

Reclamation studied options for the improvement of a legacy dam in Washington with a hydraulic fill core and thick deposits of soft liquefiable sediments in the foundation. Deformations driven by seismic strength loss dominate the risk at this facility. To evaluate potential remediation options, Reclamation used finite difference numerical modeling (FLAC2D) to investigate various options to increase the stability of this structure during seismic loading. Numerous modeling challenges were encountered including unusual bedrock geometry, extreme deformations impacting boundary conditions, and the frequent need for model rezoning. Reclamation overcame these challenges and successfully identified two remediation alternatives, deep soil mixing (DSM) and installation of a new toe shear key. A field test section for the deep soil mixing alternative was successfully implemented.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

Demystifying Alternatives Analysis for Federally Funded Projects

KATHERINE SHARPE, AICP, ENV-SP, GANNETT FLEMING, INC.

ABSTRACT

Federally funded infrastructure projects have unique requirements for analyzing alternatives that go beyond engineering criteria. Dam owners need to understand how to develop, evaluate and select alternatives that will be approved for federal funding, in order to access the significant new funding for dam safety programs provided by the two recent multibillion-dollar federal infrastructure laws. We will present considerations for designing an alternatives analysis that meets criteria established in the National Environmental Policy Act (NEPA) as well as federal dam funding agency guidance. Our presentation will include the less common categories of alternatives that may need to be considered (e.g., decommissioning, non-structural, floodproofing and relocation), and how these vary by federal agency, funding source, and project type. We will discuss the role of risk analysis in alternatives evaluation. Benefit-cost analysis is a key aspect of alternatives analysis but is not widely understood. We will cover what can and can't be included in a benefit-cost analysis of alternatives, and how to capture benefits without a straightforward dollar value, such as ecosystem service benefits. We will highlight recent changes to ecosystem services guidance by federal agencies including FEMA, USACE and NRCS.Nature-based solutions, or actions to improve the health of ecosystems and benefit society, are receiving new emphasis in federal alternatives analysis guidance. We will discuss the umbrella concept of nature-based solutions, and where practitioners can find guidance on the integration of nature-based solutions into alternatives formulation and evaluation. Our presentation will cite the nature-based solutions initiatives described in Executive Order 14072, Strengthening the Nation's Forests, Communities, and Local Economies, and the subsequent November 2022 report on accelerating nature-based solutions published by the White House Council on Environmental Quality.We will demonstrate alternatives analysis best practices using recent examples from federal watershed planning projects in Arizona and Hawaii that exhibit the implementation of federal guidance on alternatives analysis design, risk analysis and benefit-cost analysis of alternatives, and the integration of nature-based solutions.

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DAM SAFETY PALM SPRINGS
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Beyond Emergency Action Plans: Leveraging Regulatory Requirements to Develop a Collaborative and Cross-Functional Hydroelectric Emergency Management Program

ANDREW HAGEN, PACIFIC GAS & ELECTRIC COMPANY; AND JES GEARING, GANNETT FLEMING, INC.

ABSTRACT

In 2017, following the Oroville Dam Spillway Failure, California's Governor Brown signed Senate Bill 92 (SB 92), which established new dam safety requirements, into law. This legislation requires that dam owners of Significant, High, or Extremely High Hazard Dams (as classified by the California Department of Water Resources Division of Safety of Dams, DWR DSOD) receive approval of all inundation maps by DWR DSOD and then submit Emergency Action Plans (EAPs) to the California Governor's Office of Emergency Services (Cal OES) for approval. Cal OES' additional requirements for EAP compliance compelled Federal Energy Regulatory Commission (FERC)-regulated dam owners to collectively rewrite their federally approved EAPs to meet state compliance—a significant task, but not one without its benefits, as Pacific Gas and Electric (PG&E) discovered. These new state requirements mandate not only a collaborate development process between dam owners and impacted jurisdictions, but, for PG&E, they served to jumpstart a cross-functional collaboration between previously siloed divisions within its own organization.During this presentation, PG&E will discuss how SB 92 strengthened its EAP program by not just the redevelopment of its documents, but by also better integrating the program into PG&E's other electric and gas emergency programs. Additionally, this presentation will discuss how the Cal OES' compliant revised EAPs facilitated a faster and more coordinated response between PG&E and its emergency management agency partners during the series of atmospheric river storms that struck California in January of 2023. Finally, actionable tips for cross-functional collaboration will be provided to help other dam owners reimagine their own EAP programs. Through processes like these, regulations can be leveraged by dam owners everywhere to streamline stakeholder outreach, strengthen internal relationships, and better serve the communities surrounding our hydroelectric facilities.

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SEPTEMBER 17-21, 2023

Using GIS as a Tool in Managing a National Dam Safety Program

C. KIRK COVER, P.E., PROGRAM MANAGER, FEDERAL ENERGY REGULATORY COMMISSION

ABSTRACT

The Federal Energy Regulatory Commission (FERC) has oversight of energy infrastructure throughout the United States, including jurisdictional responsibility for non-federal hydroelectric projects. FERC licenses approximately 1,600 hydroelectric projects, and dam safety staff from the Office of Energy Projects (OEP) regulate more than 2,900 structures. We have embraced geographic information system (GIS) technology as an important tool in the dam safety decision making process. This paper describes our experience with GIS and how we integrate this technology with our dam safety daily workflow.

FERC began its GIS program in the mid 1990's. Our Compliance and Dam Safety staff spent the next 15 years creating standards and working with licensees on adequate GIS submittals for project boundaries, project asset locations, and danger reach analyses. Our GIS program took a significant leap forward in 2014, when we moved our data to an enterprise geodatabase on the Esri GIS platform. FERC's enterprise GIS provides a central data store, and gives multi-user edit capability to staff in headquarters and our five regional offices. Most significantly, we use Esri's software tools to build web based applications that help us manage compliance and dam safety with real-time data awareness.

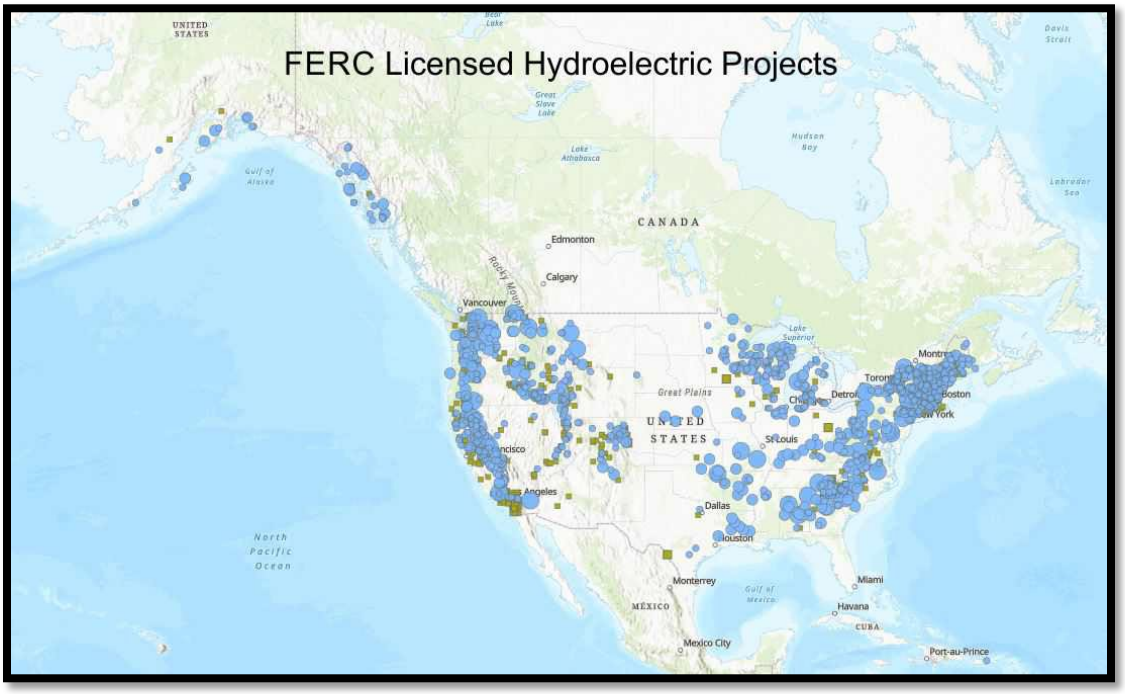
This paper explains how we built our GIS program using off-shelf software technology and describes our dam safety Situational Event Monitor and award winning High Flow Dashboards. FERC uses these applications to track natural hazards such as fires, droughts, weather, and flood events and their potential impact to hydroelectric projects and dams. The High Flow Dashboard includes a data capture form used by staff when dam operators notify us of high flows. Finally, the paper outlines the basic building blocks required for a GIS program focused on hydropower projects and dam safety, and FERC's in-the-works enhancements that will allow real-time notifications, advanced forecasting, and post-event analysis.

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I. WHO IS FERC?

The Federal Energy Regulatory Commission (FERC) was created as an independent agency of the federal government by Congress in 1920. Under the Federal Power Act, FERC is presently organized under the Department of Energy, and acts independently to regulate key operating functions of the natural gas, electric utility, hydroelectric power, and oil pipeline transportation industries. The Commission's strategic direction is to maintain strong oversight, safety, and environmental programs so competitive markets can evolve and satisfy our nation's energy needs.

The Office of Energy Projects (OEP) at FERC is responsible for engineering and environmental review of hydroelectric and gas energy projects. Most of the GIS work at FERC takes place in its hydroelectric program where it has regulatory responsibility for more than 1,600 non-federal hydroelectric projects. These projects come in all sizes and shapes and are dispersed throughout the contiguous United States, Alaska, and Puerto Rico. Projects vary in size from a few kilowatts occupying several acres of land to hundreds of megawatts occupying thousands of acres.



II. NEED FOR GIS

FERC has broad responsibilities for regulating hydroelectric projects. Engineering staff are stationed in five regional offices and must inspect hydroelectric dams and project features. Similarly, environmental staff inspect public recreation and environmental resources associated with projects. All hydroelectric projects have a project boundary that describes and shows the lands under federal jurisdiction. Projects can have multiple reservoirs, dams, pipelines, powerhouses, and any combination therein. Many projects date back to the early 20th century and by today's standards have poor-quality drawings and inadequate documentation.

FERC began its GIS program in the mid 1990's to assist with reviewing project impacts to federal lands. More than 25 percent of the regulated hydroelectric projects are located on federal property, and licensees are assessed approximately $7.75 million annually for land use charges. The FERC works with many federal agencies to track and administer the occupancy of these lands, and we need to assure the acreage is correct for assessing fees that are returned to the U.S. Treasury. While we knew issues such as dam safety, land management, threatened and endangered species, and recreation share equal weight in regulating projects we chose to begin our GIS program with the goal of collecting an accurate accounting of project boundaries and federal lands occupied.

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FERC's GIS effort began with a small, lower cost GIS system and focused on developing baseline data for projects located on federal lands. As technology progressed and allowed the software to migrate from UNIX workstations to a Windows based platform, we bought more licenses for staff to validate and collect data about our projects, including dams and reservoir areas. We began the tedious process of verifying dam locations with greater accuracy. We knew the potential of GIS but were aware that early-on the technology came with a high cost. Early on, it was too cost prohibitive to obtain funding for a central managed GIS, so we made the decision to push ahead with de-centralized data collection and validation. We knew that enterprise system costs would steadily decline, and our data acquisition and validation efforts would not be wasted.

III. GROWING AN ENTERPRISE GIS

Implementing a full-fledged GIS system throughout an organization can be a slow process. FERC began with an initiative to inventory and develop a geo-database for 428 hydroelectric project boundaries located on 186,000 acres of federal lands. We forged ahead and started converting engineering drawings to electronic format (TIFF based raster files) and began digitizing project boundaries. This effort included contractor assistance to scan and digitize drawings, begin verifying dam location data, and train FERC staff.

FERC manages approximately 30,000 exhibit drawings that were stored in aperture card (microfilm) format. Each drawing had to be converted to a raster image, and we found the scanning, geo-referencing, and digitizing process time consuming. The poor quality of many engineering drawings and the inability to achieve a reasonable accuracy when converting the project boundary raster images to vector data using a standard coordinate system hindered our progress.

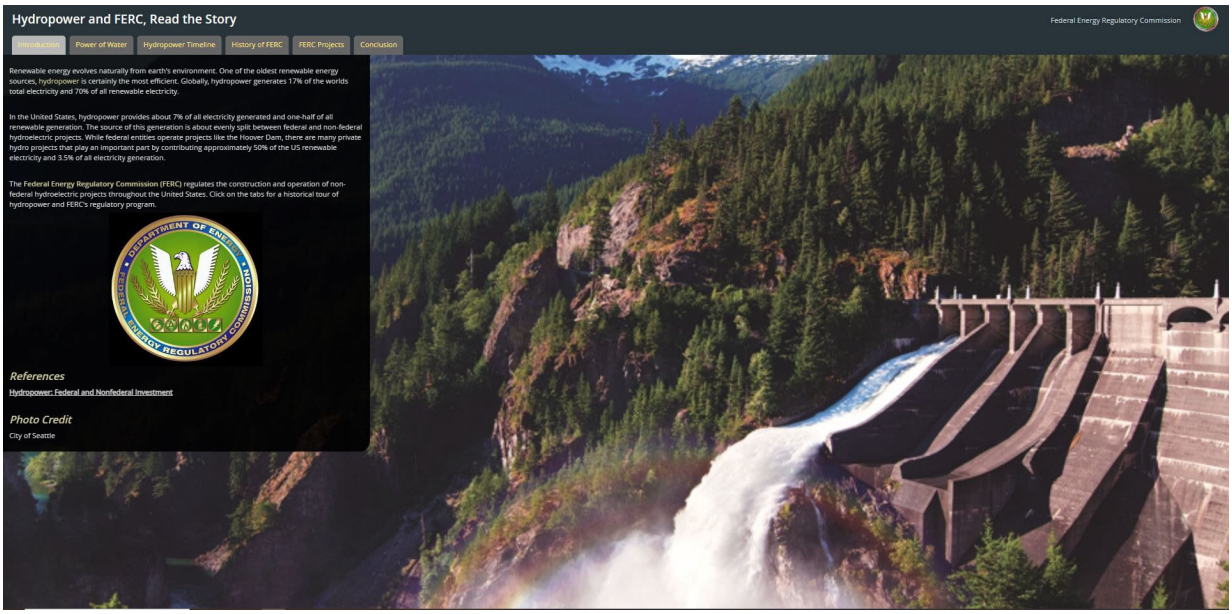
By early 2000, many advances in hardware and software had taken place and we made the decision to switch to a Windows-based GIS system. The initial learning curve for new software and new operating system was a minor setback compared to the progress we made by switching to a more up-to-date GIS technology. We also learned without the best available data it would be impossible to develop a successful GIS program, and to move the GIS program forward required changes to FERC's regulations. We promulgated regulations to require licensees to provide drawings and data in electronic format and issued standards for submittals.[1] Each project boundary submittal is verified by staff engineers and federal lands are cross-referenced with accounting records. We have been very satisfied with the map standards and checks and balances we developed to verify project boundary data. On the dam safety side, we developed in-house standards to validate dam locations and each record is tagged with a latitude/longitude accuracy descriptor.

Each GIS software license comes with access to ArcGIS OnLine (AGOL), an Esri hosted cloud-based mapping and analysis solution. While planning the migration to an enterprise GIS database format we deployed our first mainstream GIS application in AGOL using the Esri's story map format[2] as a public outreach project. To give the app exposure, we worked with FERC's Office of External Affairs and added a link to FERC's official web site. Our next effort consisted of creating a dashboard to monitor flooding and high flow events at dam sites. The original design and development took place in AGOL while we were migrating to the GIS enterprise system.

[1] (https://www.ferc.gov/sites/default/files/2020-04/drawings-guide.pdf)

[2] https://ferc-oep.maps.arcgis.com/apps/MapSeries/index.html?appid=447e5523f6bc4a759c5667791d5af074

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IV. CURRENT GIS SYSTEM ARCHITECTURE

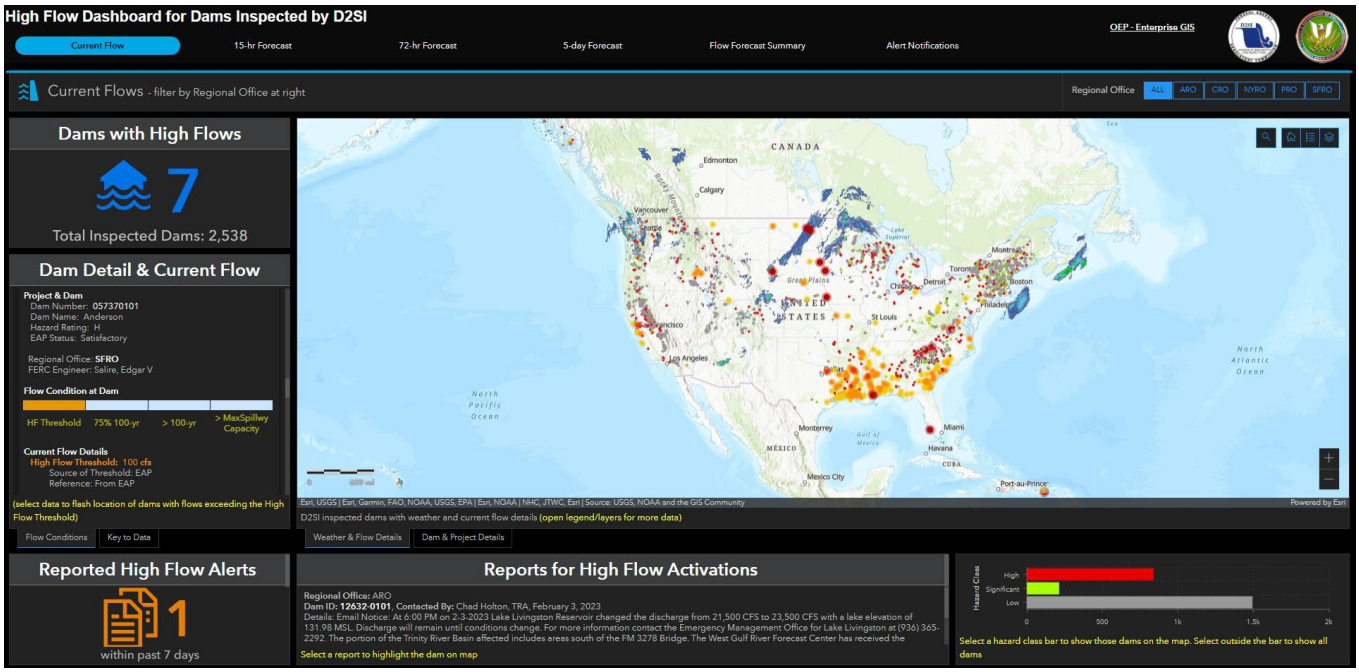
FERC's current GIS architecture centers around Esri's enterprise software running on dedicated network servers. When first deployed we found random performance bottlenecks we could not rectify. Only after we switched from Esri's ArcMap software to the 64-bit ArcGIS Pro did we realize significant improvements in speed. From this process we learned a valuable lesson in working closely with our IT Support staff and taking advantage of Esri technical support to solve problems. Most importantly, we have developed a suite of GIS applications that reside on in our GIS environment behind a firewall secured network.

The GIS system is fully integrated with FERC's data operations and security is a top priority. The software supports a wide range of geo-databases and we have coupled with in-house data formats to create a wide range of geospatial data. A key tool for any GIS system is the ability to move data quickly and efficiently between systems. Built-in GIS software tools allow external files to be easily integrated or appended to existing geodata. In addition, we rely heavily on Esri's data interoperability tool to refresh our enterprise data with routing updates from our in-house databases.

FERC has expanded the use of GIS in the review of new hydropower license applications, especially with environmental resource and justice related impacts. FERC's hydropower compliance staff use forecasting data from the National Drought Mitigation Center (https://droughtmonitor.unl.edu) to assess potential impacts of droughts on resources around hydroelectric projects. Our dam safety engineers use GIS to assist with various engineering reviews such as danger reach analyses. However, our best use of GIS has been to integrate dam safety project data with readily available live data from Esri's living atlas, government web sites, etc. and create monitoring dashboards in our enterprise GIS system. The original high flow dashboard created in AGOL had limitations and was migrated to the portal for security and useability reasons and is a daily tool used by dam safety staff. The dashboard monitors live stream gauge flows, forecasts from the National Water Model (NWM)[3], and tropical storms. Flow data is updated multiple times each day using python scripts executed by scheduled batch files with the option to execute manually by dam safety staff. In addition, a data entry form allows staff to record reports of high flow events at dam sites.

[3] https://water.noaa.gov/about/nwm

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V. STEPS FOR GIS SYSTEM GROWTH

Currently FERC is involved in a multi-year program to move our GIS architecture to a managed cloud system. This move is a long-term commitment, and we are fortunate the agency made this decision to migrate FERC's data operations from managed servers to a cloud architecture. Nevertheless, the process has taken several years of planning, design, and testing. FERC is in the final stages of quality control testing that requires tight coordination between users, IT staff, and the software provider. The flowchart below illustrates the sequence for building FERC's enterprise GIS system.

PHASE 1

Individual GIS software licenses

Data stored on network drives; dispersed & individual data sets

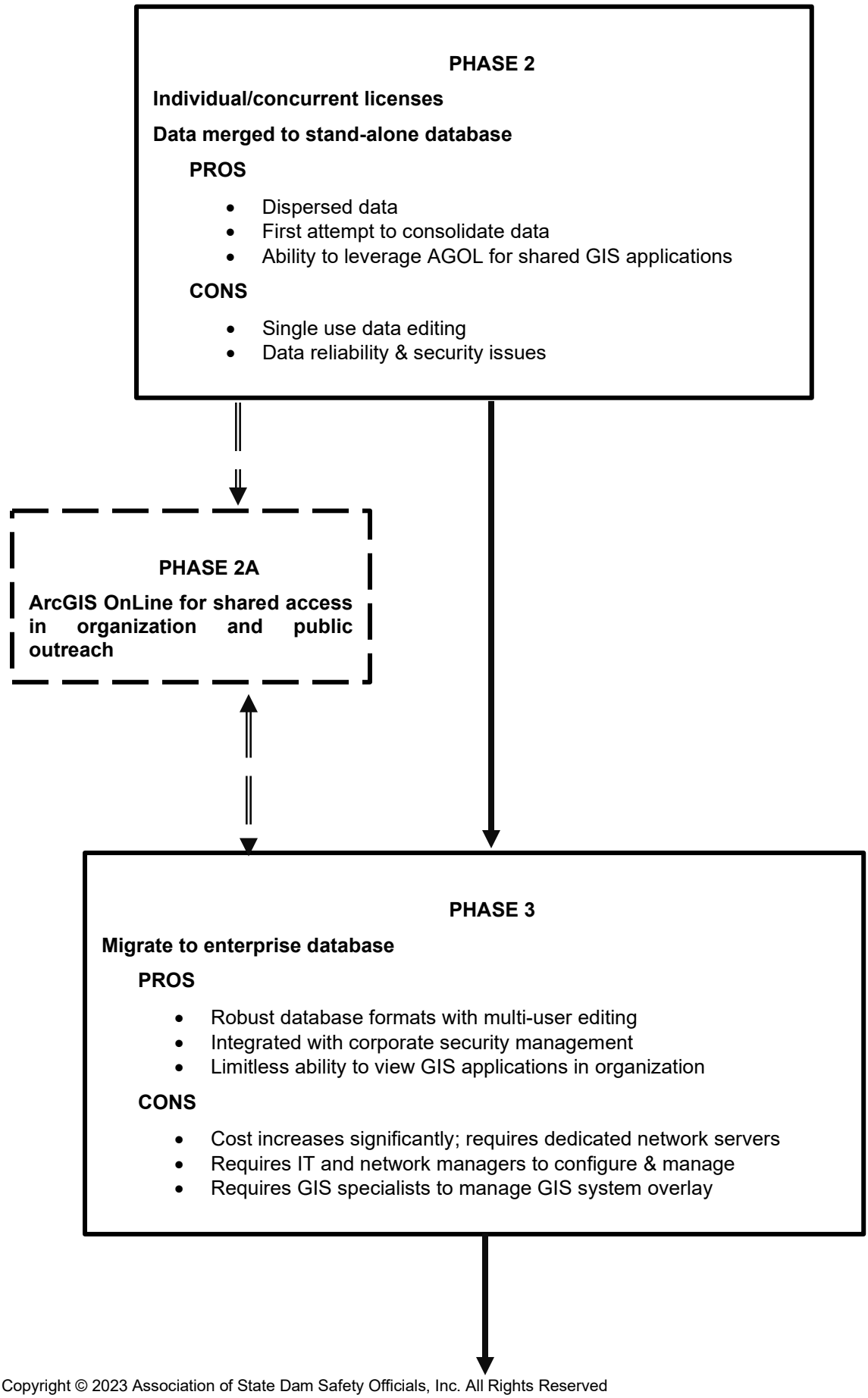
PROS

- Least cost GIS buy-in
- Enables technology training "learning curve" to begin

CONS

- Hard to manage authoritative, dispersed data
- Un-coordinated data editing if many users
- Data reliability & security issues

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PHASE 4

Migrate to cloud environment

PROS

- No difference in user GIS experience
- Eliminate in-house managed servers & hardware costs
- Long-term agency cost savings

CONS

- Requires very careful planning and lead time to migrate GIS operations
- Requires skilled IT managers to set-up and manage system
- None for GIS users

Implementing a GIS program with an enterprise geodatabase system is highly desirable and has many advantages over systems that rely on personal geodatabases. However, in many instances development, funding, and deployment can overwhelm organization resources if not fully prepared. A phased approach allows for more complex or advanced computing needs to be incorporated once a basic system is up and running successfully. FERC has followed the phased approach process in building its successful GIS program.

VI. GIS SYSTEM FUTURE ENHANCEMENTS

FERC initially deployed its enterprise GIS platform in 2016 and annually updated the software with Esri technical support. Through the year as program priorities change, we make incremental database enhancements and create additional GIS applications. We also look for ways to take advantage of product enhancements such as Esri's Arcade expression language which allows customizations to dashboards and web applications. Our migration to a cloud environment should be completed in late 2023, and we expect have the capability to monitor real-time situational events and broadcast alert notifications to staff. This requires a separate server/software layer and should only be attempted once a fully functional and validated enterprise GIS is operational.

While testing to complete GIS deployment in a cloud environment FERC is implementing a process to archive data from the high flow dashboard in the geodatabase. Long-term, this data will be used to validate flows at dams, monitor licensee reporting of high flows, and potentially assist in validating high flow thresholds and other dam safety parameters. For now, we rely on publicly available data feeds from Esri's living atlas, NOAA, NWS, USGS, and the US Drought Monitor and have found them to be stable and reliable for our needs. However, we expect to tackle issues such as advance forecasting for weather, floods, seismic and other natural events as technology enhancements become available.

VII. RECOMMENDATIONS FOR GIS SUCCESS

Have a strategic plan. Developing a well thought out GIS system requires lots of hard work and planning and will consume time and dollar resources not anticipated. Initial planning of an enterprise GIS system requires equal amounts of political and technical capital. Given their complexity, the best planned enterprise GIS system will omit something that will add delays to deployment. Most organizations do not have technicians on staff capable of handling the initial system deployment and management of an enterprise GIS system and should consider vendor support for start-up operations.

Formalize requirements linked to GIS. GIS system planners should always link GIS with organizational needs and critical agency functions whenever possible. This dependency is critical to receiving management approval and annual budget support for GIS. For example, a regulatory agency should link GIS to regulatory

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requirements while a government service agency should link its GIS system programs that will increase services. A for-profit organization will always look to build a GIS system that will increase operating profits.

Consider Phased Growth. Utilizing a phased development approach for GIS system growth (scalable architecture) should receive serious consideration. In addition to providing phased funding for expensive equipment, software and training, this strategy allows technology advances to be included over time and is more palatable to IT operating and maintenance budgets. A phased approach also allows for more complex and unforeseen computing needs to be incorporated once the basic system is up and running successfully.

While customized or proprietary products may be necessary to support a growing GIS program, try to avoid them until a stable system is deployed. GIS software is very complex, frequently updated, and requires intermittent software patches. GIS applications depending on customized products require constant attention to insure compatibility with system updates and patches.

Persistence and patience pays off. GIS system planners should have a visionary plan with breakpoint goals that produce visible achievements. They should develop a game plan that includes alternate paths for growing the system. A GIS system that is strategically planned and has a solid base will move forward and reap rewards for the organization.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

Dam Failures and Incidents Due To Dynamic Ice Loads

PAUL SCHWEIGER, P.E., CFM, AND BOYD HOWARD, GANNETT FLEMING, INC.

ABSTRACT

A search of dam failure databases such as those maintained by the National Performance of Dams Program and the Association of State Dam Safety Officials shows that they did not include any dam failures due to dynamic ice loads until Spencer Dam failed in 2019. Before the failure of Spencer Dam, it appears that the dam safety industry did not generally consider that ice runs could fail dams. This is supported by the fact that current dam safety best practices do not include procedures for evaluating dams for stability during ice runs. Both the Bureau of Reclamation and the U.S. Army Corps of Engineers Best Practices and Risk Methodology that are used by many dam engineers do not include potential failure modes related to ice runs. While static ice pressure against structures has long been documented as a design loading condition, and while the transportation industry has researched ice impact loading on structures such as bridge piers in rivers, the Dam Safety Industry generally lacks knowledge of how ice runs may impact the safety of dams in cold weather regions and how these dams should be evaluated using dynamic ice loads. This presentation will describe examples of dam failures and significant incidents related to both static and dynamic ice loads with an emphasis on dynamic ice loads. These examples will be used to illustrate the unique combinations of extreme loads that can occur during a dynamic ice event and provide important information to help engineers formulate new potential failures modes for dams in cold weather regions. Considerations will be provided for evaluating the stability of dams subject to dynamic ice loads. Lessons learned from the evaluation of a significant concrete gravity dam subject to dynamic ice loads will be shared and discussed.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

Frazzled – The 1972 Frazil Ice Failure of Keshena Falls Dam

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ABSTRACT

In the spring of 1972, the turbulent, rain swollen waters of the Wolf River cascaded over the granitic bedrock falls above Keshena Falls Dam within the Menominee Indian Reservation, Wisconsin. As air temperatures plummeted in the days following heavy rains and break up of winter ice, the cascading waters were super-cooled, turning the river into a nearly 300-foot-wide frazil ice machine. The flowing ice particles, fragments of the region's glaciated past, buckled the 13-foot-high timber-crib Keshena Falls Dam, itself a remnant of log drives on the Wolf River dating to the 19th Century. The breach released frigid water and ice towards the town of Keshena, WI located less than one mile downstream. The falls of the Wolf River were once the "drum" which drew the sturgeon, a traditional source of sustenance for the Tribe, home to spawn within the Tribe's lands. Since the dam failure, the falls have instead become a damaging source of flooding as a large portion of the ice flows generated by the upper falls are no longer obstructed by the dam enroute to town. Furthermore, a hydroelectric plant which once operated at the dam providing electricity to tribal members has also been abandoned. This presentation will examine the cause and impacts of this little-known dam failure based on research of historical documentation available from the Wisconsin Department of Natural Resources Dam Safety Program and interviews with tribal officials. The results of this research highlight a dam failure case history which continues to impact the downstream community over 50 years after the event. The results also highlight ice loading, an often-overlooked potential dam failure mechanism, and this presentation will provide a discussion of the primary engineering considerations associated with ice loading on dams.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

Decade Dam Failure Series

The 70th Anniversary of the Tangiwai Railway Disaster – A Volcanic Dam Break Story

JULIANA TABET, M.SC., P.ENG., SR. DAM ENGINEER, HATCH LTD.; AND MARK BAKER, P. E., INDEPENDENT CONSULTANT, DAMCREST CONSULTING

ABSTRACT

The Tangiwai Railway Disaster, which occurred on Christmas Eve in 1953, remains the most devastating rail accident in New Zealand's history. Of the 285 persons on board, 151 lost their lives when the train heading from Wellington to Auckland plunged into Whangaehu River shortly after the collapse of the Tangiwai railway bridge. The accident was caused by the break of a tephra dam, a natural dam formed by volcanic material, which held back Mount Ruapehu's crater lake. The resulting lahar, a fast-moving, mud-like flood, swept down the Whangaehu River, causing the destruction of the bridge's piers and causing its beams and deck to fall into the river. The efforts of a passerby to warn the oncoming train kept this disaster from become even more catastrophic. This article recounts the harrowing tale of the Tangiwai Railway Disaster, delves into the intricate and dynamic geomorphic and hydraulic processes that led to the dam's collapse, and examines the valuable lessons learned by New Zealand, including the implementation of an early warning system. Furthermore, as this case study focuses on the failure of a natural dam, the article also begins with a brief introduction to other types of significant natural dams such as landslide dams, avalanche dams, ice runs, and Glacial Lake Outburst Floods (GLOFs), to raise awareness among readers.

I.INTRODUCTION

1) Natural Dams

In natural settings, various mechanisms contribute to the formation of dams. These include landslide dams, glacial ice dams, neoglacial moraine dams, volcanic dams, fluviatile dams, eolian dams, coastal dams, and organic dams. Among them, landslides, glacial ice, and moraine dams pose the greatest threats to both people and property [1, 2]. A brief description of each type is presented below.

Landslides Dams: The most common type of mass movements are rock and debris avalanches, rock and soil slumps and slides, and mud, debris, and earth flows. Heavy rainfall, snowmelts, and earthquakes are common triggers for dam formation. These dams typically have a short lifespan, with approximately 27% failing within a day and 50% failing within 10 days of their formation. Overtopping is the primary cause of failure [1].

Glacial Ice Dams (GLOF): Glacial ice dams impound water within, beneath, or behind masses of glacial ice. Failures of glacial ice dams occur periodically, with typical return periods ranging from 1 to 10 years. Several failure mechanisms have been observed, including erosion of tunnels under or through the ice, hydrostatic flotation, slow plastic yielding of ice due to hydrostatic pressures, crack propagation under shear stresses from glacier flow and hydrostatic pressure, water overflow erosion, subglacial melting by volcanic heat, and weakening of the ice dam caused by earthquakes.

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The type of ice that forms the dam impacts the failure mechanism, with warmer temperate glacier dams being less dense and more prone to fractures compared to colder ice dams. Warmer dams tend to drain subglacially or englacially, while colder dams typically drain supraglacially or marginally.

Neoglacial Moraine Dams: Late-neoglacial moraine dams are in steep mountains where the advances and retreats of glaciers leaves behind piles of debris and rocks that create a dam. The term moraine stands for any accumulation of unconsolidated material. These dams are hazardous due to their geological youth, lack of established vegetation, the steep slopes – often exceeding 40 degrees -, and potential instability caused by deteriorating frozen core material. Global warning could have a serious impact on these cores, potentially accelerating and aggravating the instability by loss of frozen material. The most common failure mechanism is overtopping and breaching triggered by waves generated by ice or rock falls or snow or rock avalanches. Other failure mechanisms include melting of frozen core material, piping, and seepage.

Volcanic Dams: Volcanic dams are natural barriers created by volcanic activity which holds or temporarily restricts the flow of water in existing rivers. They can be formed by the flow of molten lava or by deposition of pyroclastic material and debris. Tephra is the name of any fragmented material produced by a volcanic eruption while pyroclastic material is a mixture of lava blocks, ash and gas. Interestingly, Crater Lakes, which often hold larger volumes of water for longer periods of time are not considered volcanic dam [3].

Crater Lakes: A volcanic crater lake is a depression formed by explosive activity or collapse during an eruption. These lakes are filled primarily by precipitation, melted ice and snow, and some groundwater circulation. Water loss from these lakes occurs through evaporation, seepage, and, if the water level reaches the rim, surface overflow.

Several notable examples of crater lakes exist worldwide, such as Crater Lake in Oregon, known for its remarkable depth of 1,949 ft, making it the deepest lake in the United States, and Lake Toba in Indonesia, the largest crater lake globally, measuring 62 miles by 19 miles with a depth of 1,657 ft [4].

The processes of seepage and overflow may lead to erosion of the rim or tephra barrier and produce a dam break or outburst, releasing a large volume of water and debris, called Lahars. [Wikipedia]. Lahars are also formed by eruptions, by snow and ice melting by the heat produced by the volcano or by rain [5].

2) Lahars

Lahars are swiftly flowing mixtures of volcanic ash, rock fragments, water, snow, and ice [6]. These matrix-rich debris flows exhibit a thick slurry consistency, like fresh concrete, with a density ranging from 112 to 143 lb/ft^3 and sediment accounting for 50 to 75% of the volume. Lahars are more viscous than water and can reach velocities exceeding 43 mph or 65 ft/s. The entrainment of sediment in lahar flows leads to increased peak discharge, with bulking factors of 1.8-2.5 observed for past lahars at Mount Ruapehu. The damage inflicted on structures during a lahar event depends on factors such as the depth and velocity of the flow, as well as the design and construction capacity of the structure to withstand the erosive force of sediment-laden flows containing boulders up to 1m or more in diameter [7,8].

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Photo 1 - Mount Ruapehu lahar, following eruption 24 September 1995 by Lloyd Homer (copyright GNS Science)



Photo 2 - Lahar deposits in Whangaehu Valley, Ruapehu, following Ruapehu eruption on the night of Tuesday 25 September 2007 by Vern Manville (copyright GNS Science / EQC)



Photo 3 - Whangaehu River boulders on April 28, 1975, by Brad Scott (copyright GNS Science)



Photo 4- Lahar deposits in Whangaehu Valley, Ruapehu on September 26, 2007, by Vern Manville (Copyright GNS Science / EQC)

3) Mount Ruapehu

At Mount Ruapehu, three types of natural dam formations are observed: a Crater Lake formed by a depression or collapse of the rim, a glacial ice dam created by glacier fields with englacial tunnels feeding the Whangaehu River, and a volcanic dam formed by tephra material blocking the overflow into the Whangaehu River following the 1995-96 eruptions.

II.MOUNT RUAPEHU & ITS ERUPTIONS

Mount Ruapehu, located in the North Island of New Zealand, is the country's most active volcano. It is situated in Tongariro National Park, which is also home to two other active volcanoes, Ngauruhoe and Tongariro [9]. Tongariro National Park was designated as a UNESCO World Heritage site in 1990 and is one of the oldest national parks in the world, established in 1887 [10, 11]. The park features three major peaks: Tahurangi (Ruapehu) at 9,177 ft, Te Heuheu at 9,039 ft, and Paretetaitonga at 9,026 ft.

Mount Ruapehu stands out as the only North Island mountain with glaciers [9]. There are six glacier tongues located around the Crater Lake, and during the summer months, crevasses form around the lake, leading to ice blocks sliding into the lake, particularly along the cliffed margins [12]. The volcano is a popular tourist attraction, offering a major ski resort and hiking trails.

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As a stratovolcano, also known as a composite volcano, Mount Ruapehu has been formed by layers of lava and tephra (volcanic material) accumulated from past eruptions. It has steep slopes in a conical shape, featuring a summit crater [13].

Major eruptions recorded in history happen to be exactly 50 years apart 1895, 1945 and 1995. Minor eruptions are more frequent, with approximately 60 occurring since 1945 [14]. A comprehensive list of eruptive activity is presented in [15]. Typically, steam bursts occur every 1-3 years, phreatic eruptions (steam and rock) every 5-10 years and larger phreatomagmatic (molten magma) events, like the one of 1995-1996, occur every 50 years [11, 16]. The last typical short-lived event occurred in September 2007.

Volcanic hazards associated with Mount Ruapehu include ashfall, gases, lahars, ballistic projectiles, lava flows, pyroclastic flows, and debris avalanches. It is important to note that volcanic ash is different from the ashes found in fireplaces or left behind by burned paper. Volcanic ash is made of rock fragments, minerals, and volcanic glass, with particle sizes ranging from the size of a grain of sand to as fine as a grain of clay. It is highly abrasive, mildly corrosive, conductive of electricity when wet, and insoluble in water [16, 17].

Typical eruptions from Mount Ruapehu are likely to result in light ashfall, with thicknesses generally less than 2” [17]. Ashfall in this range (3/16” – 4” thick) can have various impacts on daily life, including health issues (eye and lung irritation), infrastructure disruptions (roads, rail, hydropower generation and transmission) environmental problems (contamination of water bodies, damage to crops) [18]. These impacts highlight the importance of preparedness and safety measures when dealing with volcanic ash hazards.

III.CRATER LAKE

Situated at the center of the peaks is the Crater Lake, a volcanic crater formed as a result of volcanic activity. This approximately circular depression measures about 1000 ft in diameter and is 58 ft deep, according to measurements from Google Earth. The presence of the lake serves as a steam condenser and somewhat masks the volcanic activity of Mount Ruapehu [19].

The Crater Lake undergoes thermal cycles with the water temperature fluctuating between approximately 15°C and 40°C in a period of 6-12 months; it is usually lukewarm, but during eruptions it may reach boiling point. Historical records indicate that the lake was observed boiling in 1886. On rare occasions, the lake freezes, as reported in 1886 and 1926 [19].



Photo 5 – Mount Ruapehu Crater Lake (Source: GeoNet / Copyright of GNS Science - https://static.geonet.org.nz/info/images/volcano/Ruapehu-Crater-Lake.jpg) – the red arrow indicates the Whangaehu River

The water is very acidic with a pH smaller than one. It contains sulfur, chloride, and magnesium. The correlation between the sulphates and chlorides provides valuable insights into the level of the volcano’s activity [20, 21]. The color of crater lakes varies markedly according to the temperature and chemistry of the water, and the type and concentration of particles suspended within it.

The refilling of Crater Lake, following eruptions or collapse/break of the tephra dam, depends on a range of factors, including snowfall, rainfall, melting, lake temperature, and volcanic activity [7].

IV.THE WHANGAEHU RIVER & ITS LAHARS

The Whangaehu River, which translates as "muddy river mouth," is also known locally as the Sulphur Stream, owing to its strong sulfur smell. The water of the river carries a high level of sediment, giving it the sensation of swimming in dense saltwater where one can easily float for an extended period. Occasionally, the river water exhibits a discolored appearance due to its sulfurous origin, appearing yellowish or brownish [6].

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Originating from the Crater Lake, the river's outlet is located on the southeast rim – see Photo 5. It flows westward through a gorge and alluvial fan before taking a curve and turning south. After a journey of 100 miles from its source, the Whangaehu River finally reaches the Tasman Sea.

The fact that the Crater Lake is fed during the summer months by copious quantities of melt water from the surrounding icefields, and its level normally remains constant for considerable periods, suggests the possibility of a natural outlet for water, apart from seepage into the ground-water system.

Historically, most authors agreed that the Crater Lake fed the Whangaehu River through a tunnel under the ice fields. O'Shea [12] believed that the major outlet was in the form of an ice tunnel or ice-bridged ravine, often concealed by the extensive overhang of ice walls bordering the lake's southern edge [19].

One of the earliest documented lahar events was recorded over 160 years ago by Rev. Richard Taylor on February 13, 1861. Starting at 6 am and lasting approximately 2 hours, the flood swept away the bridge connecting Palmerston North to New Plymouth, approximately 42 miles downstream of Tangiwai. An eyewitness, a Maori (native indigenous people of New Zealand), described the surface as a thick mass of snow, ice, timber, and debris. At the time, Rev. Richard Taylor speculated that the flood waters originated from the Crater Lake, which turned out to be an accurate guess, considering the mountain had not been climbed until 1877. He noted the water was milky and in fact a diluted sulphuric acid, and surely from a volcanic origin [5, 6].

In January of 1922, Mr. L.M. Lennard first observed the ice caves in the Crater Lake. In 1937, a local guide by the name Roy Sheffield swam in the lake and entered the cave, where observed a waterfall feeding into the Whangaehu River [12, 19]. Leakage through the rim/dam can be chemically detected in the stream water [22].

During the 1945 eruption, the lava flow is believed to have blocked the existing ice tunnel, and expelling the water out of the crater, leaving it empty. Refilling then took place to a level above the pre-eruption one. On December 24, 1953, the water pressured became too great for the lava plug, and it let go, causing the release of another lahar. This lahar caused the train accident described below.

In the 1995-96 eruptions, a tephra dam ("Tephra Dam") was deposited in the southeast rim. As this section was the lowest point in the rim, the water in the crater was raising to levels which were much higher than ever before. In 2007, the water pressure was too large for the weak tephra dam to resist, and it gave way, releasing a volume of water twice as large as the one from 1953.

Currently, the Crater Lake water level maintains at the same level, as the water flows over the southeast rim.

V.THE DISASTER - CHRISTMAS EVE 1953

1) The Dam Break at Crater Lake & its released Lahar

The tragic events of December 23,1953 can be traced back to the eruption of Mount Ruapehu in 1945. The eruption, a phreatomagmatic event, produced a tholoid (magma dome) to rise in the center of the Crater Lake. The tholoid pushed the hot water into the pre-existing cave, enlarging it. Hot water was observed gushing out through the outlet into the Whangaehu River at a rate of 50 ft^3/s, far exceeding the usual flow rate of 2 or 3 ft^3/s. Suddenly the hot water ceased to flow. It is believed that lava blocked the cave/outlet at that time leaving an almost empty 1,000 feet deep crater and a covering of ash and debris over the mountain [5, 19].

The lake refilled to a level 20 to 25 ft higher than the 1945 pre-eruption levels and stabilized in August 1953 [22]. The water level remained stable for four months. Mr. and Mrs. Wood from Auckland visited the lake on the morning of the accident and reported the presence of a cave at the south end of the lake [5].

On 24 December 1953, a stage may have been reached when the ice wall between the re-forming tunnel and the lake could no longer hold back the high volume of lake water. Consequently, it gave way suddenly, and the water above the rock rim rushed out, enlarging the tunnel as it passed [19].

It is believed that while the lake was discharging through the first formed cave – believed to be about 13-16ft in diameter [7] -, the water was also seeping through the volcanic ash and scoria alongside to form another channel beneath the ice at a lower level. Erosion of this channel may have weakened the ash barrier, while cracking movements in the ice above could have triggered a sudden collapse of the ash barrier. An observed ice bridge in the Whangaehu Glacier was likely thin, and when the mass of water from the Crater Lake surged down the channel, the vibrations and pressure may have caused the ice to crack and shatter or a temporary blockage beneath the ice to increase pressure until it gave way [12, 23].

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No volcanic activity was recorded, but vibrations were noted about 8:00 pm to 9:00pm. A local tremor of very small magnitude was recorded at the Chateau Tongariro seismic station, which was uncharacteristic of a local earthquake. [12] The recording time agrees with witness accounts of a roaring sound. [5, 19 or 23]. It is believed that the vibration was due to the waters bursting through the previously blocked tunnel/outlet.

The lahar is believed to have reached the Tangiwai bridge sometime between 10:10pm and 10:15 pm [5, 23]. It took the wave 2 hours and 15 minutes for the wave to reach the Tangiwai bridge at an estimated flow velocity of 32 ft/s. The lahar density was calculated by Mr. James Healy to be 100 lb/ft^3 [14]. The lahar at the Tangiwai bridge was a mixture of water, ice, mud, sand, and boulders, some as large as 4 ft in diameter [5]. The Crater Lake water level dropped 26 ft, with 20ft of the drop occurring in the first 150 minutes. The flow varied from a maximum peak of 30,000 ft^3/s to 1,300 ft^3/s. The last 6 ft discharged relatively slower [5].



Photo 6 – Mount Ruapehu Crater Lake Outlet after the Tangiwai Disaster on December 28, 1953 (Copyright GNS Science)

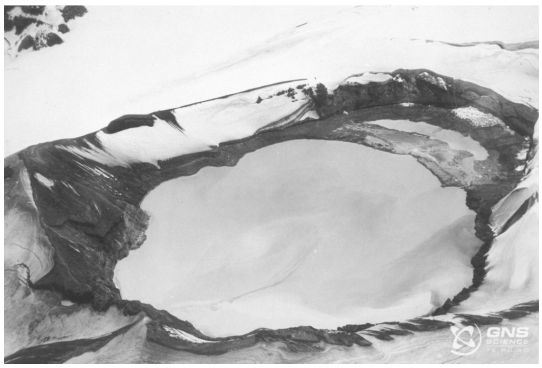


Photo 7 – Mount Ruapehu Crater Lake after Tangiwai on December 28, 1953 (Copyright GNS Science)



Photo 8 – Ruapehu Crater Lake outlet after the Tangiwai disaster on Dec 28, 1953 – the red arrow indicates the breached 1945 tephra layer (Copyright GNS Science)

When the lahar reached the bridge, it is believed to have washed away Pier 4, leading to the collapse of spans 3 and 4 into the river. The water level at the bridge rose by 15 ft to 17 ft [5]. It is believed that span 3 girders were struck by the waves, as the flood water was measured above the elevation of the bottom flange of girder 3. Lateral and diagonal forces on the pier then caused it to fracture at a construction joint, and the upper portion was then forced upwards and sideways. When the lower portion of the pier was relieved of most, if not all, of the vertical forces from above it was swept away by the flood. As a result, the upper portion of the pier and spans 3 and 4 were then unsupported and fell into the raging waters [5, 23].

At 10:22pm, approximately 5 to 10min after the collapse of the piers, and at the lahar peak, the train arrived at the bridge [5].

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2) The Disaster

Just moments before the train arrive at the bridge, Cyril Ellis, accompanied by his wife and mother-in-law, was driving from east to west, on the road parallel to the train tracks, State Highway 46 – the Volcanic Loop Highway. Upon reaching the road bridge, he noticed that it was nearly submerged, with only the handrails visible. When he saw the express train approaching the nearby rail bridge, he realized, even though he didn't check, that the rail bridge couldn't possibly be safe to cross. He took a torch (flashlight) and ran toward the train, jumping the fence on his way. He desperately waved his torch at the conductor, and shouted, but the conductor didn't see him in time to completely stop the train before reaching the bridge. Later at the inquire, it was concluded that the emergency brakes were applied at 700ft before the bridge and that dry sand was used to increase wheel traction, creating sparks witnessed by passersby. The calculated required stopping distance for a train travelling at a speed of 50 mph to completely stop was 1177 ft [6, 24].

The locomotive along with its tender, five second class wagons and one first class wagon fell into the raging waters. Of the 285 persons in the train, 151 lost their lives. Of those 151, only 131 bodies were recovered. Thanks to the heroic efforts of the conductor and firefighter, of the actions of the brave passerby and many others, a worst disaster was averted.

3) Christmas Day

When the sun rose, the full extent of the disaster became apparent. The violence of the raging flood waters was clearly displayed in the river area and banks. The vegetation on the banks was laid flat toward downstream, and there was clay covering in everything. The train wagons were spread downstream of the bridge and were completely destroyed. One wagon, A 1920, was carried 1.5 miles downstream and is believed to have been catapulted over the road bridge or squeezed through the abutment and the riverbank, as the wagon would not have fit underneath it. The first wagon, just behind the tender, was bent at a 90-degrees angle.

Graham Steward, the photographer present at the scene, described the aftermath in the preface of his book "The Tangiwai Disaster – A Christmas Eve Tragedy" (1972):

"What greeted us I will never forget: first, a strange silence – an aerie atmosphere – which hung over and around the disaster scene. Before our eyes was unbelievable destruction, as if a giant Gulliver had flung this mighty railway train of steel into the river. The concrete and steel bridge had been smashed beyond comprehension. Wrecked carriages were scattered unbelievably in all directions. Seats, blankets, suitcases, children's toys, some still wrapped in festive paper, and many personal belongings, all lay half buried in silt and mud wherever one stepped."



Photo 9 - The scene at the site, looking toward the west - Dec 25, 1953, Mount Ruapehu in the background (Copyright Graham Stewart)

Search parties were organized, and many volunteered. The scene was horrific. Many bodies were stripped of clothing, some were found hanging on the survived vegetation on the top of the banks. Briefcases, gifts, toys, and other items were scattered in the downstream area. The bodies were heavy with the sedimented water. Most victims died by drowning, but the evidence of injuries from the boulders carried by the floodwaters was visible in their deformed bodies.

4) The Board of Inquire

Following the tragic Tangiwai disaster, a board of inquiry was promptly appointed to investigate the accident. Their main tasks were to find the cause of the accident, whether anybody was responsible for it, and what steps should be taken to prevent similar accidents.

The board went into great depth to investigate and bring forth the details of design, construction, and maintenance of the rail bridge, and into the operations and maintenance of the rail line itself. As part of their

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investigation, they studied the origin of the flood and characteristics of the Whangaehu River, the damage to the bridge and rational behind its failure.

Regrettably, crucial background information regarding the rail bridge, including as-built plans and construction records, was lost in a fire. The only available documentation consisted of typical plans that were used at the beginning of construction that lacked the final details of the bridge's foundations.

Regarding the cause of the accident, the board concluded that it resulted from the sudden release of a large volume of a mixture of water and ash from the Crater Lake though an englacial ice tunnel combined with large blocks of ice which collapse from the glacier above. The flood collected / uplifted enormous amounts of sand, silt and boulders and was very violent. The flood destroyed the bridge before the train arrived.

With regards to assigning blame for the tragedy, the board concluded that no individual or entity could be held responsible. The bridge had been designed, constructed, and maintained in a reasonable manner, and the trains were operated and maintained properly as well.

The board of inquire recommended the following protective measures to be implemented immediately and kept until effective warning measures could be implemented: installation of a telephone at the bridge, reduction of train speeds over the bridge to 6 mph and stationing a person at the bridge to signal drivers and prohibit passage until an "all clear" signal was received. This protocol was to remain in effect until alternative and effective warning systems were established.

Overall, the board of inquiry diligently fulfilled its mandate by thoroughly investigating the incident, providing comprehensive findings, and making valuable recommendations to prevent future disasters.

VI. LESSONS LEARNED & EVOLUTION OF THE ERUPTION DETECTION SYSTEMA AND THE LAHAR ALERT AND WARNING SYSTEMS

The first warning device

The Board of Inquiry finalized its report in April 1954. By November of the same year, a comprehensive investigation on flood warning systems was conducted, focusing on several types of warning devices. The report aimed to address the operational requirements specific to the local conditions, which included factors such as isolation, extreme climatic changes, prolonged dampness, vulnerability to damage, atmospheric corrosion, and the need for continuous reliability despite extended periods of no operation [24]. The alarm needed to be reliable, have a fail-safe operation, it should always carry a message indicating that “the river is safe”, it should be a closed circuit, the device should break an electrical circuit to activate the alarm, a mechanical fuse that fractures a wire was considered the most reliable option, the power source should be located at or upriver from the detector, or an alternate route should be used for powering the detector separately from the indicating circuit, and the system should be trouble free.

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Various types of devices were investigated. The chosen device consisted of a waterproof concrete chamber with fixed probes installed at specific heights through holes in the concrete wall. The device was installed by January 1957 at the Whangaehu River 17 miles downstream of the Crater Lake [25]. It provided a 30-minute warning time before the flood reached the rail bridge [24]. Table 1 presents the flood warning levels that were set.

Table 1 – Flood warning levels

Level 1	A flood which will occur once or twice each year
Level 2	A flood reaching ½ the height of the 1953 flood
Level 3	A flood reaching ¾ the height of the 1953 flood
Level 4	A flood of the same height as the 1953 flood



Photo 10 - Rail lahar warning system, Whangaehu River, Taupo Volcanic Zone by Lloyd Homer (Copyright: GNS Science)

The 1969 & 1975 failure

In 1969 Mount Ruapehu erupted once again, generating a lahar that failed to register at Level 1. The warning system had failed. Again, in the 1975 another eruption occurred, releasing a lahar which travelled at an average of 16 mph, reaching the warning system gauging site (17 miles from the Crater Lake) in just 8 minutes. The peak flood velocity was estimated to be 27 mph and the peak flow 176,600 ft^3/s. Within 2 or 3 minutes the Whangaehu River rose from normal flow to above the gauge maximum at circa 5 m above normal river level [25]. This time the alarm was successful, but no warning was given, the alarm only alerted the rail station of the incoming flood waters, there was no integration with other stakeholders.

The Lahar Warning System (LWS)

After the 1969 and 1975 eruptions, it became evident that an improved warning system was necessary, particularly for those residing on the slopes of Mount Ruapehu, particularly in the Whakapapa Ski Area.

In 1984 the Lahar Warning System (LWS) was installed in Dome Shelter by government departments, the Ministry of Works, Dept of Scientific & Industrial Research (forerunner of GNS Science) and the Department of Lands & Survey (forerunner of DOC). The system consisted of a seismometer sensor and radio communications, which would trigger an automatic alarm when detecting significant differences in seismic signals between the volcano and a remote peak in Tongariro Forest. The Dome Shelter was a wood structure the size of a shed, located in the top of Mount Ruapehu.

The 1995 LWS failure

In 1995, another eruption occurred, causing damaged to the Dome Shelter and the LWS, and exposing its inadequacy. Valuable lessons were learned from this event, including the need to have multiple stations with sensors and more robust communication systems (Private conversation with Harry Keys). The system was then reengineered as the Eruption Detection System (EDS).

The Eruption Detection System (EDS)

The primary purpose of the Eruption Detection System (EDS) was to identify eruptions on Mount Ruapehu and provide skiers in valleys at the Whakapapa Ski Area with timely warnings via sirens and voice message of an approaching lahar, so they could make their way to higher ground and out of danger. This system also ensured that response agencies were promptly notified via SMS, email, and page messages.

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Whakapapa Village Lahar Alarm and Warning System (VLAWS)

The VLAWS was designed after the lessons learned from the 1995 eruption and the upgraded EDS and commissioned in 2013. The purpose of the VLAWS was to detect lahars travelling down the Whakapapanui River on the northern flanks of Ruapehu and provide residents, visitors, and businesses in Whakapapa Village with a 20-minute warning via sirens and voice message of an approaching lahar, so they can leave to designated evacuation areas. The system also provides alerts via SMS, email, and page messages to response agencies.

The VLAWS is integrated with the EDS and an outstation at the Skippers Fence, equipped with a tripwire, a geophone, a cabinet with UHF radio link to the base station and a fiber optic power cable from Schuss House. Triggering of either the tripwire by breakage (drop in voltage due to tripwire breakage) or the geophone provides a "confirmation" that a lahar has arrived at this location. An Acoustic Flow Monitor (AFM) at the Skippers Fence was upgraded in 2016 and 2018.

The 1995-96 changes to the Crater Lake

After the 1995 eruptions, significant changes occurred in the Crater Lake rim and upper Whangaehu Valley due to erosion and the accumulation of a thick layer of tephra (ash, scoria, and lava blocks) over the crater area. Parts of the southeastern rim, between Pyramid Peak and the outlet and the spur to the west of the outlet, and the ridge between the Pyramid and J Peak were eroded by cascading lake water and the explosions that occurred during the eruptions. These changes led to a 40% enlargement of the crater lake. Its walls south of the Pyramid thinned and parts of the crater rim lowered. Such changes brought a concern about the stability of the rim, and size and path for future lahars.[7].

Tephra Dam was the name given to the built up of material that blocked the natural outlet to the Whangaehu River. Unlike in 1953, there was no longer an ice tunnel restricting the flow out of the Crater Lake. Therefore, the peak outflow from the lake into the Whangaehu River was expected to be unrestricted and significantly larger than in the 1953 flood [16]. The expectation was that as the Crater Lake started to refill and came close to overflow the Tephra Dam, internal seepage flow was likely to occur through the permeable tephra, which was expected to lead to piping erosion in the tephra layer, weakening it, and leading, eventually to a rapid breach and discharge of the lake into the Whangaehu River. Based on the 1953 precedent, overtopping of the dam, channel erosion and slow lowering of the lake was deemed unlikely [7].

The growing concern with a 1953 repeat event

The concern over the possibility of a repeat of the 1953 event was significant. In April 1996, a debriefing workshop was conducted following the eruptions in 1995. The workshop focused on the changes observed in the Crater Lake and developed a plan to investigate and address these concerns [7]. The plan consisted of two phases: Phase I (1997) included survey and geological/geomorphological mapping, oblique photography, sampling and testing the southeastern rim and stability and dam break analysis, and Phase II (1998) involved a refinement and more in-depth analysis of the hydraulic modelling and inundation studies, with estimation of peak flow depths, and assessment of possible infrastructure and environmental damages. Based on the findings of these studies, engineering options were assessed to mitigate the concerns [7]. Six categories of options were considered:

1- Allow lahar to occur naturally: develop alarm and response system, improve land use planning but no engineering intervention at crater, etc.
2- Allow lahar to occur but intervene in lahar flood zones to reduce its size and/or confine it.
3- Aim to prevent or reduce a lahar by hardening or perforating tephra barrier e.g., grouting, weir, tunnelling, culvert.
4- Prevent or reduce a lahar by excavating a trench through the 1995-1996 tephra barrier using techniques such as bulldozer, snow groomer, explosives, sluicing.
5- Prevent a lahar and reduce lake volume by excavating trench into underlying lava at the lake outlet.
6- Defer, prevent, or reduce a lahar by other options like siphoning or barrier truss.

Most stakeholders favored options to allow the lahar to occur naturally while developing an alarm and response systems and improving land use planning. These options were developed with knowledge of work conducted at crater lakes and similar situations elsewhere around the world [22].

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The Eastern Ruapehu Lahar Alarm and Warning System (ERLAWS)

The New Zealand Department of Conservation (DOC) prepared an Assessment of Environmental Effects (AEE) report to provide mitigation measures to the Minister of Conservation [7]. After reviewing the Minister of Conservation approved the design and installation of a public lahar alarm system and recommended the drafting up of a contingency plan for warning and response actions by responsible agencies [7].

The alarm system, named the Eastern Ruapehu Lahar Alarm and Warning System (ERLAWS), was installed May 2000. The ERLAWS was a best-practice alarm system, which comprised of a network of sensors, acoustic flow monitors (AFMs), dual-pathway telemetry to a base station at Tokaanu power station (Genesis Energy), and near real-time interfaces to pagers, telephones, and the internet. While the geophones and the tripwire across the dam were reliable sensors, the pressure transducers used to help monitor lake level proved to be unreliable for alarming [22].

The goal of the ERLAWS was to provide an early warning (two hours in advance of a lahar reaching Tangiwai) and notify all stakeholders of the impending danger. Its objective was to detect the actual dam break as well as a large lahar coming down the upper Whangaehu valley. A response plan was developed by the local district councils with major support from DOC, the NZ police, the Fire Service and the other agencies and companies involved. This included a succession of exercises as the lake level rose (private communication with Harry Keys).

Further observational measures

An observational approach was also taken as it was deemed to provide many benefits such as: provide a more definite warning of the impending lahar and give responsible authorities, transport, and utility agencies more time to plan and initiate an appropriate response. An automatic lake level recorder was installed. A bund (embankment) to prevent overflow into the Waikato stream was constructed [7].

The March 2007 success story

In March 2007 the tephra dam broke and released the waters from the crater lake, but this time, the warning system worked. The predictions were in very good agreement with the actual lahar (quite impressive!). The response plan worked very well, no one was hurt, very little damage occurred to infrastructure and the highway and rail bridges were operational within a few hours of closing as part of the response plan.

With the Tephra Dam gone, the water level at the crater lake is now controlled by the outlet into the Whangaehu River. The concerns now, besides the usual eruption generated lahars (and the eruptions itself), would be lahars produced by the collapse of the rim (hard rock) or landslide of parts of the crater rim into the crater lake. Such landslides were part of the scenarios developed before the dam break, but the probability and risks were assessed as low compared to the main dam break process (private communication with Harry Keys).

The September 2007 lesson

Another eruption followed in September 25 the same year, a phreatic eruption, which produced a snow rich lahar, which did not trigger ERLAWS, even though its peak discharge was approximately 60,000 ft^3/s. ERLAWS detected the lahar, but the apparently the snow damped the lahar due to its low bulk density and the system didn't trigger [32].

Retirement of ERLAWS

After 20 years of service, the Eastern Ruapehu Lahar Alert and Warning System (ERLAWS) was retired in June 2022. ERLAWS was built to mitigate the lahar risk from the tephra dam created by the 1995 and 1996 eruptions of Ruapehu. It successfully detected the lahar caused by the tephra dam's collapse on 18 March 2007. However, with the tephra dam gone, the water level at the crater lake is now controlled by the outlet into the Whangaehu

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River. A new lahar detection and warning system for the Whangaehu River is operated by Genesis Energy and Horizons Regional Council [27].

Status of Emergency Management

The Department of Conservation (DOC) developed an Initial Response Plan (IRP) [28] for volcanic activity in Tongariro National Park (TNP) which is one of two documents it uses to guide its approach to volcanic risk management and response. The IRP addresses the initial response to an eruption in the TNP while the Guidelines for DOC Volcanic Risk Management in TNP [29] addresses the escalation and de-escalation of volcanic unrest and the associated management actions that accompany the changing volcanic climate.

GNS is the responsible body for monitoring volcanic activity, setting Volcanic Alert Levels (VALs), and issuing Volcanic Activity Bulletins via GeoNet [29]. The IRP serves as the DOC's response guide in the event of an eruption in TNP. The response is triggered by the activation of the TNP Volcanic Alert Network (TNP VAN) which includes:

- *The Ruapehu Eruption Detection System (REDS) and Whakapapa Ski Area Lahar Alert and Warning System (WLAWS)*

  The REDS detects eruptions from the crater lake using acoustic and seismic sensors. If an eruption is detected, it triggers the WLAWS system, which consist of a siren and a voice message instructing people to move to out of the valley into higher ground.

- *The Whakapapa Village Lahar Alert and Warning System (VLAWS)*

  WLAWS uses a tripwire in Skippers Canyon to detect lahars travelling down the Whakapapanui Stream. This system will activate if REDS is activated.

- *The Tongariro Eruption Detection System (TEDS)*

Based on past eruptions, it takes 1-5 minutes for a lahar to reach the top Whakapapa Ski field and 15 minutes to reach the bottom; as for the Whakapapa Village, it takes 25 minutes [28].

Monitoring

Mount Ruapehu is monitored by the Institute of Geological and Nuclear Science (GNS) through a program called GeoNet (www.geonet.org.nz). The instrumentation includes seismometers, web cameras, and continuous GPS stations to record ground deformations. [14, 16]. A supervisory control and data acquisition system (SCADA) monitors the eruption detection data inputs from GeoNet. It sends eruption and lahar, all clear, testing, and diagnostic alerts via email and email-to-SMS to emergency response groups and stakeholders.

GNS's monitoring program also measures the temperature at the Crater Lake, performs monthly gas and water chemistry analysis, mostly collected manually by staff. GNS has recently installed two permanent gas monitoring stations on the eastern and western sides of the volcano.

Public Safety

The Department of Conservation (DOC) has undertaken various initiatives to promote public safety and awareness of the volcano hazards. Educational materials, including posters, have been distributed throughout the Whakapapa Village and Ski Area. However, despite these efforts, public tests of the Eruption Detection System in Whakapapa ski area revealed that a persistent minority of up to 20% of individuals persists in remaining in lahar paths even one minute after warning broadcasts [33]. Ensuring effective response to the warning system at Mount Ruapehu present challenges due to the transient nature of ski area visitors and their lack of familiarity with volcanic hazards. Detailed interviews with staff and public, as well as observations during "open" and "blind" exercises serve as important research tools [35].

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Infrastructure Improvements

Since the Tangiwai disaster of 1953, significant efforts were made to enhance the resilience of vulnerable infrastructure to lahars. By January 6, 1954, a temporary rail bridge replacing the collapsed one was in operation. This temporary bridge was eventually replaced with a permanent bridge in 1957. The road bridge was temporally replaced with a bailey bridge and later reconstructed. In response to lahar predictions by scientists, Transit New Zealand raised the road bridge by 6.5 ft in 2005 [32, 34].

To mitigate the risk of distributary flows crossing into the Tongariro catchment and affecting the State Highway 1 bridge and Tongariro River, a 985-ft-long bund structure, reaching a height of 13.8 ft, was constructed during 2001-2002 [32, 34].

VII.CONCLUSIONS/EPILOGUE

Natural dams present significant hazards, particularly in the case of Mount Ruapehu, where the complex geomorphology and dynamic nature of its rims, comprised of various layers of lava deposits, eruption debris, ice, and snow, pose unique challenges. The location of this site on top of an active volcano within a national park and heritage site further compounds these challenges.

Although predicting eruptions remains a difficult task, the presence of the Crater Lake provides valuable insights into the volcano's activity and proximity to an eruption. Thus, monitoring the Crater Lake is crucial to the monitoring of lahars, and the instrumentation scattered across Mount Ruapehu plays a vital role in providing timely alarms and warnings, especially to those in the hazardous Whakapapa region.

Effective emergency response and preparedness plans in place, regularly updated and accompanied by proper training, have proven their effectiveness as demonstrated during the March 2007 event.

Furthermore, public education is essential, albeit challenging when the target group is continuously changing, as is the case with visitors to the Whakapapa ski areas. While the alarm and warning system will alert individuals within the hazard zone, understanding and heeding the warning is of utmost importance.

Infrastructure, including road and rail bridges, is particularly vulnerable to lahars where they exist. It is imperative to either locate these structures outside the path of destruction or design them to withstand such hazards.

The use of monitoring systems to detect volcano activity and lahar flows are challenging. Volcanos are dynamic, unpredictable, locating and protecting the instrumentation is difficult and dangerous to maintenance staff. New generations of technologies offer additional capacity and reliability. Ongoing installation, operation, testing, and maintenance of these systems is critical to protecting the public from the hazards of lahars.

Numerous lessons have been learned since the tragic events of the Christmas Eve in 1953. These lessons have been transformed into action, evolving over the years, culminating in the successful prediction of the Tephra Dam break in March 2007. The efforts of the individuals involved were commendable, encompassing accurate predictions of the event (timing, water level, failure mode, peak flow, and bulking factor), effective monitoring triggering the alarm and warning systems, seamless interagency application of the emergency response plan, and, most importantly, the prevention of loss of life.

VIII.ACKNOWLEDGMENT

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

The "Perfect Storm": Can Atmospheric Models Improve Confidence In Probable Maximum Precipitation (PMP)?

EMILIE TAROUILLY, UCLA

ABSTRACT

The flood that would result from the greatest depth of precipitation "meteorologically possible", or Probable Maximum Precipitation (PMP) is used to ensure the safety of nuclear power plants, among other high-risk structures. Historically, PMP has been estimated by scaling (extrapolating) depth-area-duration relationships obtained from severe historical storms, following guidelines from the so-called Hydrometeorological Reports (HMRs). Over the last decade, frameworks that leverage numerical weather prediction models to predict precipitation resulting from the addition of moisture (called relative humidity maximization, or RHM) have been developed. Incorporating current understanding of precipitation processes in those model-based methods represents an important advance. Nonetheless, model-based PMP still relies on key assumptions: (1) that severe historical storms achieved maximum efficiency (moisture conversion to precipitation), such that only moisture needs to be maximized and (2) that maximizing moisture (i.e., saturating the atmosphere) near the target basin is realistic and consistently maximizes precipitation. Numerical weather prediction models allow us to re-evaluate those assumptions and perform scenario analyses to develop physically-based guidelines on how to reliably maximize storms. Additionally, as the use of model-based tools introduces new challenges such as model uncertainty, our scenarios include different model setups and parametrizations that aim to characterize the magnitude of this uncertainty.Focusing on the Feather River basin in California, we downscale the most severe historical storms from ERA5 reanalysis using the WRF model. Using these high-resolution simulations, we seek to identify key attributes of these storms (storm orientation, convection and large-scale convergence) that control precipitation efficiency and we characterize the nonlinear precipitation response to the addition of moisture in our simulations. In so doing, we highlight that PMP would be better presented as an ensemble of values, such that uncertainty can be communicated, rather than a single estimate, and develop guidance for the engineering community on how to consistently maximize storms.

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DAM SAFETY PALM SPRINGS
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Evaluating Spillway Capacity with Changing Data– The Effects of a Statewide PMP Study on a Portfolio of Dams

AMANDA HESS, P.E., CFM, GANNETT FLEMING, INC.

ABSTRACT

With 30 high hazard dams spread throughout Pennsylvania, the Pennsylvania American Water Company seized the opportunity to re-evaluate the spillway capacity of their dams based on updated estimates of Probable Maximum Precipitation (PMP). Previous studies had evaluated spillway capacity based on the PMP from Hydrometerological Reports 51 and 52 and concluded that only 6 out of 30 of their high hazard dams passed greater than 50% of the PMF. Only 1 if the dams passed 100% of the PMF. Based on the broad finding that many of the dams required significant modifications to pass the spillway design flood, the Pennsylvania American Water Company embarked on a major dam modification program aimed at updating all their high hazard dams to passing the full PMF and meet current dam safety requirements. The rehabilitation program started in 2008 and is on-going with nearly half of the dams completed or in final design.The 2019 Pennsylvania PMP study developed updated precipitation depths for various durations for the entire state. In conjunction with the study the Pennsylvania Department of Environmental Protection established application guidelines for using the precipitation depths to generate the PMF. The Pennsylvania American Water Company commissioned a study that included the re-analysis of the Probable Maximum Storm for the 30 high hazard dams as well as updated hydrologic analysis to establish the PMF based on the new application guidelines. This presentation will explore the effects of the statewide PMP study on spillway capacity of their high hazard dams and the effects the statewide PMP study has had on the dam rehabilitation program.

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DAM SAFETY PALM SPRINGS
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Evaluating Extreme Precipitation for Hydrologic Analysis for Dam Safety and Mining Operations: Considering Current and Future Conditions

BILL KAPPEL, CHIEF METEOROLOGIST/PRESIDENT, APPLIED WEATHER ASSOCIATES AND DOUG HULTSTRAND, PHD, SENIOR HYDROMETEOROLOGIST, APPLIED WEATHER ASSOCIATES

Quantifying the effects of extreme precipitation and changing meteorological conditions on mining operations and planning for closure conditions and design is critically important to ensure safe and efficient operations today and in the future. Applied Weather Associates (AWA) has completed 100's of site-specific meteorological analyses and climate change assessments world-wide, many for complex mining operations. These studies have included determination of Probable Maximum precipitation (PMP), precipitation frequency, and Annual Exceedance Probability (AEP) for both current conditions and climate change projections. Results have been used to determine spillways adequacy, understanding water balance, evaluations of extreme precipitation depths and seasonality, and input for evaluation of hydrologic runoff responses.

In order to provide a complete picture that includes both current conditions and future conditions, AWA has completed several climate change assessments that incorporate both the site-specific data from current observations and future climate change projections. These climate change assessments accurately consider the site-specific characteristics related to precipitation and other meteorological factors, building off the work AWA has completed for each location. AWA has been able to provide detailed evaluations of potential future conditions, along with uncertainty analyses that allows for more accurate characterization of potential climate change outcomes. This is very important for operations to determine where and how to best allocate funding for future conditions and ensure ongoing safe operations.

INTRODUCTION

Understanding future climate change projections and how those might affect temperature, moisture, and extreme precipitation is critical for tailings dam design and dam safety evaluations. Climate change projections are available with global coverage and provide the parameters required to evaluate how these critical meteorological inputs may change through time.

Applied Weather Associates (AWA) has completed numerous climate change and effect on hydrology assessments world-wide. These cover just about every conceivable climate situation from the driest location on earth in the Atacama Desert to Arctic cold in Yukon Territories, to tropical wet climates in Indonesia through the Rocky Mountains in Colorado. Figure 1 displays the locations where climate change assessments have been completed by AWA. Each of these assessments utilizes standard processes to evaluate daily temperature, dew points, and precipitation projections through trend analyses, frequency quantification, and ratio comparisons.

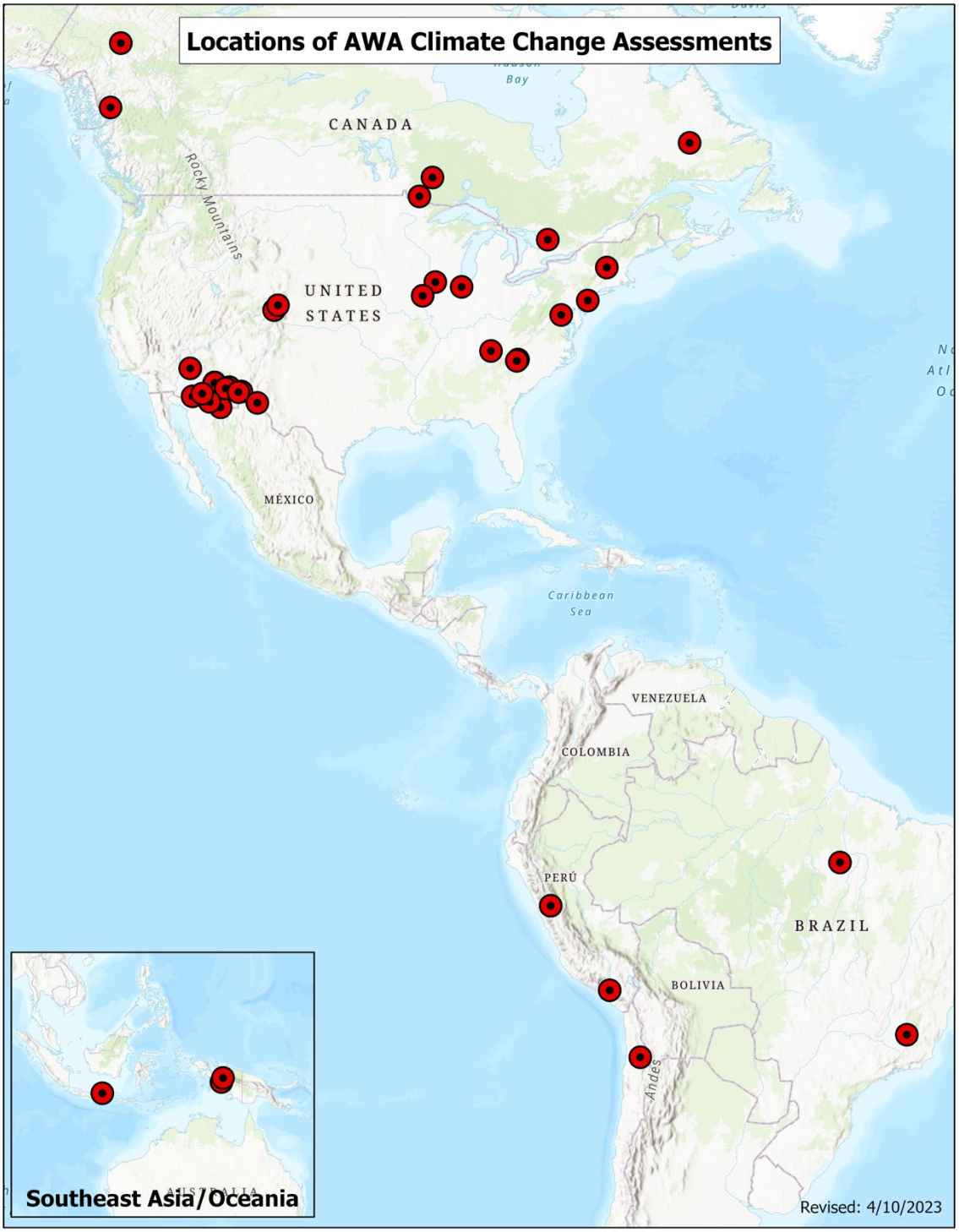


Figure 1. Climate Change Assessment Locations.

Climate Change Projection Background

Climate is changing, always has been changing, and will continue to change as long as the energy received from across the Earth's surface and atmosphere is not distributed evenly over time. Accounting for future changes in climate is important to quantify changes that may affect hydrologic outcomes and loading conditions at a dam. Understanding the effects of climate change for a given location in a more accurate manner can reduce future

risks associated with these changes and ensure infrastructure is designed to safely manage potential future changes.

Unfortunately, quantification of the amount and rate of change at any given location for any specific meteorological parameter is not explicitly quantifiable and instead has to be modeled based on our incomplete understanding of the Earth's climate system and future atmospheric composition. Therefore, model projections that utilize our current understanding of the Earth climate system are developed. The climate projections are based on our best quantification of physical understanding of numerous atmospheric parameters and how those affect weather and climate through time and space. However, because our quantification of these parameters are incomplete (and at times inaccurate) and because we currently have a limited understanding of the various interactions and feedbacks, the climate projections represent possible outcomes. None of which can be considered truth, but instead should be treated as "what if" scenarios representing some number of possible outcomes.

To better address these significant limitations, numerous iterations and sensitivity analyses for various atmospheric parameters are performed so that a suite of ensembles is produced to represent a wide range of potential outcomes. From this output, inferences can be made, with more confidence given when ensemble outcomes converge on a common projection.

Another layer of uncertainty within the climate change projections relates to the assumption applied for future emissions scenarios related to human activities and how those may affect the climate system. Future emissions scenarios have two major areas of uncertainty. First, emission scenarios are an estimate of what we think will occur through time. This of course is unknown as there are many internal and external factors that can influence the amount of emissions produced through time. Second, our understanding and quantification of how the Earth's climate will respond to any given greenhouse gas emission scenario is limited. Both uncertainties introduce errors into the climate projections.

Finally, the Global Climate Change Models (GCMs) are computationally intensive and are therefore run at low resolution both in time and space. As a result, the resolution of the GCMs is inadequate to capture the significant spatial variations. To overcome this, projections from GCMs are downscaled using a statistical process into regional downscaled model projections. Regional Downscaled Climate Change Models (RCMs) are downscaled and are what were utilized for this climate change analysis. Given all the limitations and uncertainties noted above, it is still useful to evaluate RCMs to understand the range of potential outcomes that could occur through time over the basin.

Global Climate Change Models

GCMs produce realizations of the Earth's climate on a generally coarse scale of around 1000km by 1000km. Because the scale is so coarse, a single GCM grid may cover vastly differing landscape (from very mountainous to flat coastal plains for example). Within one GCM grid, the climate can vary significantly and therefore many of the unique meteorological and hydrological aspects within that grid are smoothed out and not represented properly.

Regional Downscaled Climate Change Models

RCMs and Empirical Statistical Downscaling applied over limited areas result in a much finer resolution. These are therefore able to capture the spatial and temporal variations related to a site-specific region or basin. The downscaling methods are driven by GCMs, where the RCM is nested within the overall GCM and utilizes the GCM to set the initial boundary conditions. These are then downscaled using either the statistical methodology which compares observational data to adjust climate change outputs, or the RCM based on a meteorological model interface that is used to initialize weather models. The RCM process can provide projections of future climate conditions on a much smaller scale (e.g., < 50km by 50km) supporting more detailed site-specific information allowing for adaptation assessment and planning. An example of different climate model resolutions across the region are shown in Figure 2.

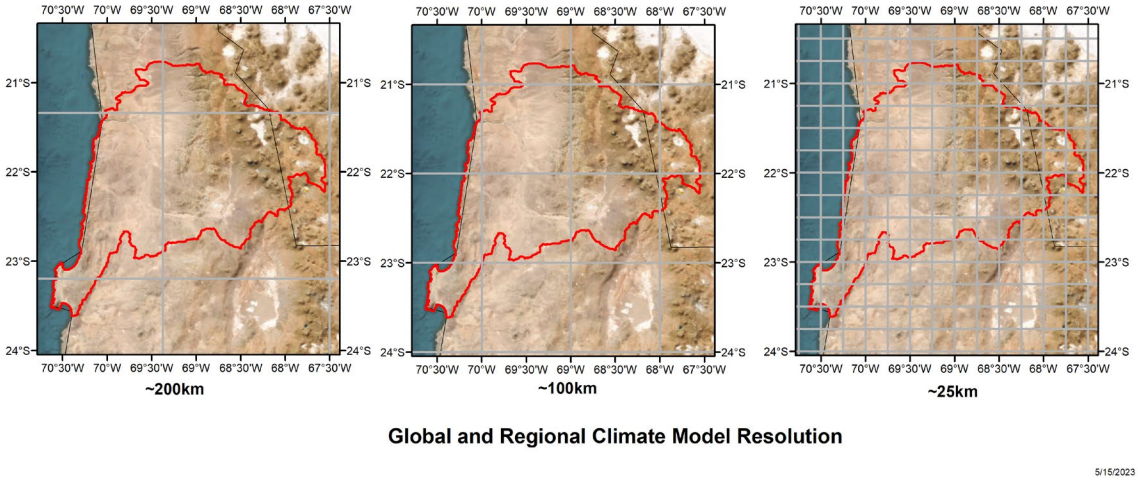


Figure 2. Example of global and regional climate model resolutions across a region. The red outline is an example basin with variable meteorology and terrain.

METHODS AND ANALYSIS

Future climate change projections are determined from Regional Downscaled Climate Model (RCM) outputs specifically evaluated for the location. The Global Climate Models (GCMs), also referred to as General Circulation Models, are developed by various governmental, academic, and research agencies around the world in coordination with the Intergovernmental Panel on Climate Change (IPCC). These are utilized to set the boundary conditions and input for the RCMs. The different scenarios are described by Shared Socioeconomic Pathways (SSPs). SPPs are scenarios of projected socioeconomic global changes and greenhouse gas concentration trajectory. SSPs are scenarios of projected socioeconomic global changes out to 2100.

As part of the IPCC analysis, four pathways were applied for climate modeling: SSP 2.6, SSP 4.5, SSP 6.0, and SSP 8.5 (IPCC, 2021). The various pathways considered different climate futures, depending on the volume of greenhouse gases (GHG) emitted in the years to come. Climate change studies that evaluate future temperature and precipitation projections most often utilize the middle emission scenario (SSP 4.5) and the most extreme emission scenario (SSP 8.5). These provide a bracket of the projections that utilize the most likely outcome (SSP 4.5) and the most unlikely outcome (SSP 8.5).

For this example, climate model projections gridded outputs were investigated for the three scenarios: i) historic, ii) SSP 4.5, and iii) SSP 8.5. The historical period is based on daily climate change information from 1950 through 2014, and the SSP periods are based on daily climate change projections from 2015 through 2100. The NASA Earth Exchange Global Daily Downscaled Projections (NEX-GDDP-CMIP6) dataset, a gridded daily time-series set of projections are extracted, aggregated, and applied for the climate change analysis. AWA evaluated the climate model projections in this example using statistical methodologies to quantify changes in precipitation frequency and maximum precipitation for the 1-day, 2-day, 3-day, 4-day, 5-day, monthly, and annual durations for the area covering the study region (Figure 2).

The climate change study used two investigations. The first method investigated climate change for mean monthly and annual climatologies. The second method was precipitation frequency analysis based on L-moments methods described in Hosking and Wallis (1997) and utilized the R-statistical software packages ‘lmom’ and ‘lmomRFA’ developed by Hosking (Hosking 2015a, and Hosking 2015b).

The NASA Earth Exchange Global Daily Downscaled Projections (NEX-GDDP-CMIP6) dataset is comprised of thirty-five global downscaled climate scenarios derived from the GCM runs conducted under the Coupled Model Intercomparison Project Phase 6 (CMIP6) and across two of the four “Tier 1” greenhouse gas emissions scenarios (SSP 4.5 and 8.5). The CMIP6 GCM runs were developed in support of the Sixth Assessment Report of the IPCC AR6) (Thrasher et. al, 2021; Thrasher et. al, 2022). The purpose of this dataset is to provide a set of global, high resolution, bias-corrected climate change projections that can be used to evaluate climate change

impacts on processes that are sensitive to finer-scale climate gradients such as moisture availability, cloud formation, and temperature changes and the effects of local topography on climate conditions.

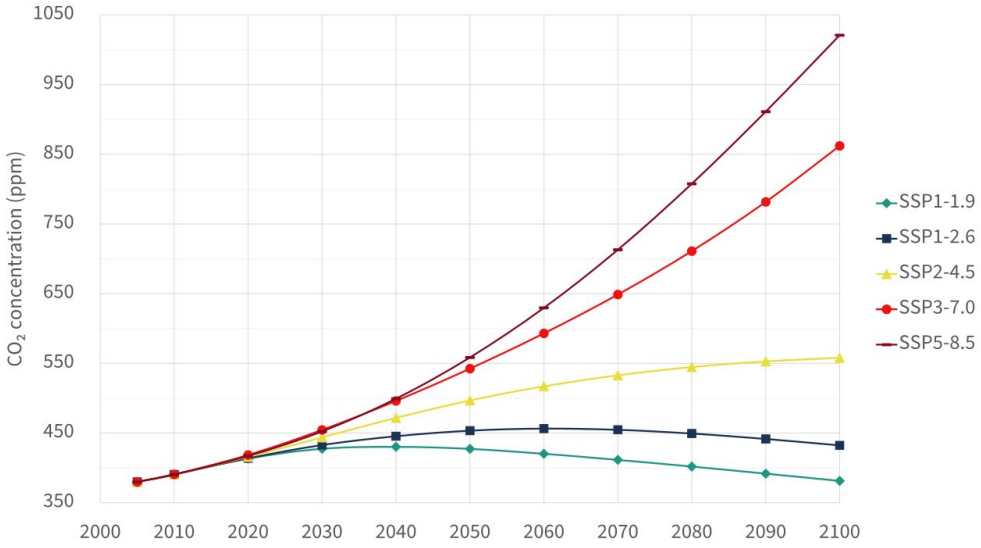


Figure 3. Shared Socioeconomic Pathways (SSP) trajectories (from IPCC 2021).

The key climate model parameters used in this analysis were precipitation (Ppt), air temperature (Ta), and dew point temperature (Td). The parameters of relative humidity (RH) and Ta were used to derive the estimates of dew point (Td). The NEX-GDDP-CMIP6 dataset consists of thirty-five models, of these thirty-five models, twenty-six models had the parameters and projections needed for the climate change analysis (Figure 5). An example of the modeled daily climate projection parameters of precipitation (Ppt), temperature (Ta), and relative humidity (RH) are shown in Figure 4 and the grid resolution covering the region are displayed in Figure 5. The climate projections historical period is based on daily data from 1950 through 2014, and the future periods are based on daily data from 2015 through 2100.

Model #	MODEL NAME	*Relative Humidity (hurs)*			*Precipitation (pr)*			*Temperature (tas)*		
		HISTORICAL	SSP45	SSP85	HISTORICAL	SSP45	SSP85	HISTORICAL	SSP45	SSP85
1	ACCESS-CM2	1950-2014	2015-2100	2015-2100	1950-2014	2015-2100	2015-2100	1950-2014	2015-2100	2015-2100
2	ACCESS-ESM1-5	1950-2014	2015-2100	2015-2100	1950-2014	2015-2100	2015-2100	1950-2014	2015-2100	2015-2100
4	CanESM5	1950-2014	2015-2100	2015-2100	1950-2014	2015-2100	2015-2100	1950-2014	2015-2100	2015-2100
5	CESM2-WACCM	1950-2014	2015-2100	2015-2100	1950-2014	2015-2100	2015-2100	1950-2014	2015-2100	2015-2100
6	CESM2	1950-2014	2015-2100	2015-2100	1950-2014	2015-2100	2015-2100	1950-2014	2015-2100	2015-2100
7	CMCC-CM2-SR5	1950-2014	2015-2100	2015-2100	1950-2014	2015-2100	2015-2100	1950-2014	2015-2100	2015-2100
8	CMCC-ESM2	1950-2014	2015-2100	2015-2100	1950-2014	2015-2100	2015-2100	1950-2014	2015-2100	2015-2100
9	CNRM-CM6-1	1950-2014	2015-2100	2015-2100	1950-2014	2015-2100	2015-2100	1950-2014	2015-2100	2015-2100
10	CNRM-ESM2-1	1950-2014	2015-2100	2015-2100	1950-2014	2015-2100	2015-2100	1950-2014	2015-2100	2015-2100
11	EC-Earth3-Veg-LR	1950-2014	2015-2100	2015-2100	1950-2014	2015-2100	2015-2100	1950-2014	2015-2100	2015-2100
12	EC-Earth3	1950-2014	2015-2100	2015-2100	1950-2014	2015-2100	2015-2100	1950-2014	2015-2100	2015-2100
13	FGOALS-g3	1950-2014	2015-2100	2015-2100	1950-2014	2015-2100	2015-2100	1950-2014	2015-2100	2015-2100
14	GFDL-CM4_gr1	1950-2014	2015-2100	2015-2100	1950-2014	2015-2100	2015-2100	1950-2014	2015-2100	2015-2100
15	GFDL-CM4_gr2	1950-2014	2015-2100	2015-2100	1950-2014	2015-2100	2015-2100	1950-2014	2015-2100	2015-2100
16	GFDL-ESM4	1950-2014	2015-2100	2015-2100	1950-2014	2015-2100	2015-2100	1950-2014	2015-2100	2015-2100
17	GISS-E2-1-G	1950-2014	2015-2100	2015-2100	1950-2014	2015-2100	2015-2100	1950-2014	2015-2100	2015-2100
21	INM-CM4-8	1950-2014	2015-2100	2015-2100	1950-2014	2015-2100	2015-2100	1950-2014	2015-2100	2015-2100
22	INM-CM5-0	1950-2014	2015-2100	2015-2100	1950-2014	2015-2100	2015-2100	1950-2014	2015-2100	2015-2100
23	IPSL-CM6A-LR	1950-2014	2015-2100	2015-2100	1950-2014	2015-2100	2015-2100	1950-2014	2015-2100	2015-2100
26	MIROC-ES2L	1950-2014	2015-2100	2015-2100	1950-2014	2015-2100	2015-2100	1950-2014	2015-2100	2015-2100
27	MIROC6	1950-2014	2015-2100	2015-2100	1950-2014	2015-2100	2015-2100	1950-2014	2015-2100	2015-2100
28	MPI-ESM1-2-HR	1950-2014	2015-2100	2015-2100	1950-2014	2015-2100	2015-2100	1950-2014	2015-2100	2015-2100
29	MPI-ESM1-2-LR	1950-2014	2015-2100	2015-2100	1950-2014	2015-2100	2015-2100	1950-2014	2015-2100	2015-2100
30	MRI-ESM2-0	1950-2014	2015-2100	2015-2100	1950-2014	2015-2100	2015-2100	1950-2014	2015-2100	2015-2100
33	NorESM2-MM	1950-2014	2015-2100	2015-2100	1950-2014	2015-2100	2015-2100	1950-2014	2015-2100	2015-2100
34	TaiESM1	1950-2014	2015-2100	2015-2100	1950-2014	2015-2100	2015-2100	1950-2014	2015-2100	2015-2100

Figure 4. Subset of twenty-six CMIP6 models and the parameters and projections needed for the climate change analysis.



Figure 5. Example Climate model projections of Ppt, Ta, and Td from Model 1 of the 26 models investigated. The historical period represents climate model calibration period.

FINDINGS AND CONCLUSIONS

The monthly and annual climatologies are derived to describe the average monthly and annual conditions over the defined climate projection period. The historical monthly climate projection period is based on daily data from 1950 to 2014. The SSP4.5 and SSP8.5 monthly climate projections were created for three twenty-five-year periods: i) 2025-2050, ii) 2050-2075, and iii) 2075-2100 of daily data. The percent change for each month for each projection period were based on the historical climate projection reference period.

Precipitation Frequency Analysis

AWA used the precipitation frequency analysis L-moment statistics method instead of product moment statistics, which decrease the uncertainty of rainfall frequency estimates for more rare events and dampens the influence of outlier precipitation amounts from extreme storms (Hosking and Wallis, 1997). The projections were applied assuming stationarity. Methods to account for non-stationarity in projections were not addressed, the projections were applied assuming stationarity. For the precipitation frequency analysis, AWA utilized the daily climate model projections to perform frequency analysis on the 1-day, 2-day, 3-day, 4-day, and 5-day durations.

AWA evaluated the future climate change projections, SSP4.5 and SSP8.5, for three twenty-five-year periods: i) 2025-2050, ii) 2050-2075, and iii) 2075-2100 of Annual Maximum Series (AMS) data for each duration. The percent change for each duration for each projection period were based on the historical climate projection reference period.

AWA identified, extracted, and quality controlled maximum precipitation projections for the twenty-six models and three projection scenarios. The AMS were then subjected to frequency analysis methods (Hosking and Wallis, 1997). L-moment statistics were computed for annual maximum data for each projection and duration. Goodness of fit measures were evaluated for five candidate distributions: generalized logistic (GLO), generalized extreme value (GEV), generalized normal (GNO), Pearson type III (PE3), and generalized Pareto (GPA). An L-Moment Ratio Diagram was prepared based on L-Skewness and L-Kurtosis pairs for each duration. The weighted-average L-Skewness and L-Kurtosis pairing were found to be near the GEV distribution for all projections.

The GEV distribution was selected because: i) This is the most common distribution used for precipitation frequency studies (e.g., NOAA Atlas 14, Perica, 2015) ii) the GEV was identified on both the 1-day, 5-day, 90-day, and Annual goodness-of-fit measures, and iii) using the same distribution ensures a more direct comparison to more rare values of the frequency curve. The GEV is a general mathematical form that incorporates Gumbel's Extreme Value (EV) type I, II and III distributions for maxima. The parameters of the GEV distribution are the ξ (location), α (scale), and k (shape). The Gumbel EV type I distribution is obtained when k = 0. For k > 0, the distribution has a finite upper bound at ξ + α /k and corresponds to the EV type III distribution for maxima that are bounded above. For k < 0, this corresponds to the Gumbel EV type II distribution.

The uncertainty analysis for deriving the frequency curve and uncertainty bounds were conducted as follows. The frequency distributions were randomly permuted, and data were simulated from the selected frequency distribution. The procedure is described in Hosking and Wallis (1997) and Hosking (2015b), except that the permutation of frequency distributions is a later modification, intended to give more realistic sets of simulated data (Hosking, 2015b). From each permutation the sample mean values and estimates of the quantiles of the regional growth curve, for non-exceedance probabilities are saved. From the simulated values, for each quantile specified the relative root mean square error (relative RMSE) is computed as in Hosking and Wallis (1997). The error bounds are sample quantiles of the ratio of the estimated regional growth curve to the true at-site growth curve of the ratio of the estimated to the true quantiles at individual sites (Hosking, 2015b).

In order to separate Summer season and Winter season precipitation events that are controlling of the yearly precipitation regime in the example region, the 1-day, 2-day, 3-day, 4-day, and 5-day annual maximum were also extracted for the Summer season (June - October) and for the Winter season (November – May). The summer and winter AMS data were used to perform L-moment frequency analysis methods as described above. Comparisons of percent change were made among model projections for 10-year through 1,000-year recurrence intervals, beyond this the uncertainty in probability distributions estimates is large. Figure 6 shows and example of the results for Model 1 1-day precipitation frequency analysis for All season (mixed storm distribution), Summer season, and Winter season for the historic, SSP 4.5, and SSP 8.5 projections.

***** 1-Day Precipitation**

	10yr	50yr	100yr	500yr	1000yr	Pct Change					Average
Historical	56.6	72.1	79.3	97.7	106.4	-	-	-	-	-	
SSP45	66.3	78.0	82.2	90.4	93.4	17%	8%	4%	-7%	-12%	**2%**
SSP85	70.1	81.6	85.6	93.3	96.0	24%	13%	8%	-5%	-10%	**6%**

***** 1-Day Summer**

	10yr	50yr	100yr	500yr	1000yr	Pct Change					Average
Historical	34.5	49.2	57.0	80.0	92.5	-	-	-	-	-	
SSP45	44.5	64.7	75.0	104.1	119.2	29%	32%	32%	30%	29%	**30%**
SSP85	45.8	64.6	73.6	97.4	109.1	33%	31%	29%	22%	18%	**27%**

***** 1-Day Winter**

	10yr	50yr	100yr	500yr	1000yr	Pct Change					Average
Historical	56.3	71.5	78.4	95.7	103.7	-	-	-	-	-	
SSP45	65.8	79.4	84.6	95.4	99.6	17%	11%	8%	0%	-4%	**6%**
SSP85	69.6	82.2	86.7	95.9	99.2	24%	15%	11%	0%	-4%	**9%**

Figure 6. Example results for 1-day precipitation frequency analysis for climate projection from Model 1.

Uncertainty

Measurement, modeling, and simulation of many meteorologic components can be highly uncertain, mainly because fundamental dynamics of many processes cannot be measured and modeled accurately (Kampf et al., 2020). Most meteorologic processes are not observed in detail, consequently accurate mathematical representation of the variables spatial and temporal processes, model initial boundary layer conditions, and physical processes, cannot be represented accurately. Mantovan and Tondini (2006) have identified sources of water balance uncertainties as: (i) data uncertainty, (ii) model parameter uncertainty, (iii) model structure uncertainty, and (iv) natural uncertainty. Data uncertainty can be reduced with more observational data and higher quality observations. Model parameter uncertainty can be reduced as more accurate observational data is included representing the parameters being investigated. Model structure uncertainty can be reduced as model are better parametrized, calibrated, and tested.

Results of Analysis

The modeled precipitation frequency results have a large variability that can be attributed to the uncertainty inherent with GCM and RCM projections. The different climate models used for the region are subject to significant components of future climate uncertainty in climate models and the uncertainty is manifested by the range of climate futures indicated by the CMIP6 ensemble of projections (McSweeney and Jones, 2016; Masson-Delmotte et al., 2021).

The median of the twenty-six models projects an increase in mean annual temperature (2.9 °C to 4.0 °C) and slight increase in mean annual precipitation (8% - 10%) (Figure 7). Monthly average temperature shows an increase by 2100 for both the SSP 4.5 and SSP 8.5 projections for all four periods investigated: i) 2015-2100, ii) 2025-2050, iii) 2050-2075, and iv) 2075-2100 (Figure 8). Numeric values representing the change in temperature all four periods investigated are shown in Table 1.

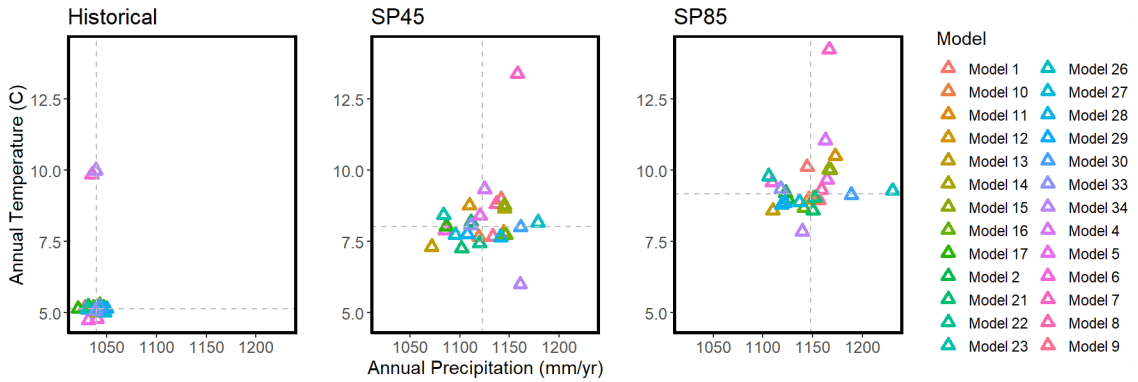


Figure 7. Comparison of mean annual temperature and mean annual precipitation for the three climate projection periods. The grey dashed lines represent the median value for annual average temperature and precipitation. Note two of the models are significant outliers based on their model parametrization and characteristics of atmospheric responses. However, all models are considered to have the same probability of outcome.

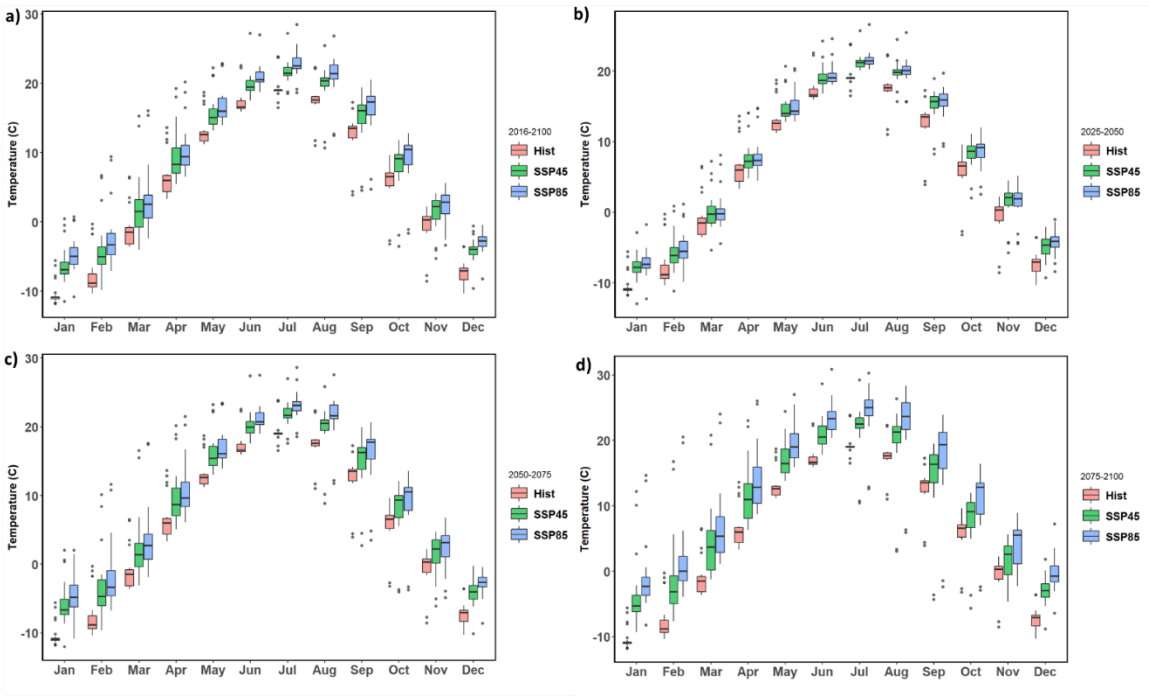


Figure 8. Monthly temperature normal compared to climate change temperature. Results are based on daily normal calculations a) 2015-2100, b) 2025-2050, c) 2050-2075, and d) 2075-2100.the plots show the median, 25th, and 75th percentile and all projections from all 26 models.

Table 1. Monthly temperature (C) for current historical climate (1950-2014) and two SSP future projections from 2015 through 2100.

	Historical		SSP45		SSP85		Mean Delta		Median Delta	
	Mean	Median	Mean	Median	Mean	Median	SP45	SP85	SP45	SP85
January	-10.4	-10.9	-6.2	-6.9	-4.7	-5.0	***4.2***	***5.7***	***4.0***	***5.9***
February	-7.7	-8.8	-4.0	-5.0	-2.4	-3.3	***3.6***	***5.3***	***3.8***	***5.5***
March	-1.1	-1.5	2.0	1.5	3.2	2.6	***3.1***	***4.3***	***3.0***	***4.1***
April	6.4	6.0	9.4	8.3	10.3	9.4	***3.0***	***3.9***	***2.3***	***3.5***
May	13.1	12.6	15.8	15.1	16.8	16.0	***2.7***	***3.7***	***2.5***	***3.4***
June	17.2	16.6	19.9	19.4	20.9	20.5	***2.7***	***3.7***	***2.8***	***3.9***
July	19.2	19.0	21.7	21.4	22.8	22.5	***2.5***	***3.6***	***2.4***	***3.5***
August	17.5	17.6	19.8	20.3	21.1	21.4	***2.4***	***3.6***	***2.7***	***3.8***
September	12.6	13.5	15.0	16.0	16.2	17.3	***2.3***	***3.6***	***2.5***	***3.8***
October	5.8	6.6	7.9	9.1	9.2	10.5	***2.1***	***3.4***	***2.6***	***3.9***
November	-0.6	0.3	1.3	2.2	2.4	2.8	***1.9***	***2.9***	***1.9***	***2.5***
December	-7.3	-7.1	-4.1	-4.0	-2.9	-2.8	***3.2***	***4.4***	***3.1***	***4.3***

Conclusions

This climate change analysis investigated CMIP6 projections. The projections were evaluated using statistical methodologies to quantify changes in precipitation frequency and changes in monthly temperature and precipitation climatologies. The results have large variability that can be attributed to the uncertainties and limitations inherent in climate model projections and the physical representation of meteorological parameters such as precipitation. For hydrologic simulation and sensitivity, AWA generally recommends the ensemble median SSP 4.5 climate change projections for temperature and precipitation. The results for this example study demonstrate that the climate change projections during the winter season are within the range of uncertainty in the observational period and therefore no changes outside of the observational record is expected. For the

summer season, there is a trend toward more frequent heavier rainfall amounts and these effect of increased summer precipitation should be evaluated.

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DAM SAFETY PALM SPRINGS
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Wanapum Dam Repair: A Challenge For Con-Tech Systems USA Inc.

PAUL KRUMM, NICHOLSON CONSTRUCTION COMPANY

ABSTRACT

The Wanapum Dam is a hydroelectric project located on the Columbia River downstream (south) from Vantage, WA, where Interstate 90 crosses the Columbia from Grant County into Kittitas County. It is owned by the Grant County Public Utility District (PUD). On February 27, 2014, the PUD discovered a 65-ft long by 2" wide crack that spanned the full length of Monolith 4, which required an emergency action of lowering the water level 26 ft to reduce the pressure on the gated spillway structure.Con-Tech Systems (CTS®) was awarded for the supply of DCP Strand and Bar anchors, By Nicholson Construction. The length of the multiple, 61 strands, per anchor, and the underwater placement and stressing of the 3" DCP bar anchors made this very challenging.A total of thirty-five, 61-strand anchors, with length up to 260 feet, were installed through the monolith piers with lock-off loads of 2,500 Kips. Post-Tensioning the anchors was performed with a 2,200-ton single stroke jack with a stroke length of 35 inches; maximum elongation measured 21.3 inches for the longest anchor. In addition to the strand anchors, 69 solid 3" diameter bar anchors were installed, of which most were installed below water through the ogee spillway on the upstream side of the radial gates.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

DMAD Dam Rehabilitation Phase II

JED MCFARLANE, P.E., RYAN COLE, PH.D., P.E., D.GE., PHIL GERHART, P.E., AND RICHARD BUHLER, P.E., GERHART COLE, INC; AND EVERETT TAYLOR, P.E., AND DAVID MARBLE, P.E., UTAH DIVISION OF WATER RIGHTS

ABSTRACT

DMAD Dam is located along the Sevier River near the town of Delta, Utah. Although original construction records are not available, it was raised to its current geometry in 1959. The embankment is about 1,200 ft in length with a structural height of 37 feet and has a storage volume of about 11,000 acre-feet. The reservoir provides storage for irrigation diversions and cooling water for the Intermountain Power Plant. In June of 1983, the east end of the embankment and original spillway failed during a large flood event. The spillway was replaced, and embankment repaired in 1984. In 2006, main portions of the dam were rehabilitated to address embankment seepage. During rehabilitation efforts loose liquefiable materials combined with high seepage pressures were found under the existing spillway suggesting the potential for the existing spillway to be unusable following a major earthquake. Geotechnical modeling demonstrated that densification of these loose materials could potentially stabilize the spillway structure to meet minimum standards. Various ground improvement and mitigation methods were considered, and compaction grouting was found to be the most reasonable, minimally invasive, and relatively low-cost mitigation solution. Given concerns with the level of mitigation needed to improve the site soils and access logistics for the equipment, a full-scale compaction grouting program was developed and implemented with pre- and post-improvement assessments using cone penetration soundings. Results from the compaction grouting program combined with other hydraulic and operational concerns led the owner, regulators and design team to move forward with replacing the spillway. The project commenced in the summer of 2021 and was completed in the spring of 2022. Some key features included: a) lowering the crest of the spillway apron 10 feet and flattening the spillway chute to 3.3H:1V to increase capacity and stability of the spillway, b) removing approximately ten feet of liquefiable soils below portions of the spillway and basin, and c) buttressing the remaining liquefiable soils by filling the plunge pool and scour hole with structural fill and riprap. Implementation of these mitigation measures demonstrated minimum regulatory standards were met and increased seismic resiliency of the spillway.

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DAM SAFETY PALM SPRINGS
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Small Dams, Big Problems: A Case Study on a Complete Dam Reconstruction

JOSEPH KUDRITZ, P.E., PROJECT MANAGER, AND BRIAN AFEK, P.E., DEPARTMENT MANAGER,
MICHAEL BAKER INTERNATIONAL, INC.

ABSTRACT

Often, small dam structures are overlooked due to physical size but have similar deficiencies and risks as large-scale structures. This case study will show the decision-making processes (from preliminary design through construction) utilized at two small state-owned dams to address deficiencies and navigate a restrictive project site with a limited budget. This case study provides an example of how a small-scale dam can dictate the rehabilitation efforts and how they can be drastically different than large-scale projects.

I. INTRODUCTION

As part of a statewide capital improvement project, the Ohio Department of Natural Resources (ODNR) improved the Mount Gilead State Park Upper and Lower Dams in order to bring the facility into compliance with current ODNR Dam Safety Regulations. The two dams are situated within in Mount Gilead State Park, which is located in the Village of Mount Gilead, Ohio, and are owned and operated by ODNR Division of Parks and Watercraft. The lakes are connected in series, with the Upper Lake discharging directly into the Lower Lake, ultimately discharging into Sam's Creek.

The origins of the lakes date back to 1841 when the upper lake was formed after a dam composed of rock, wood, and soil was constructed to provide water power for a private sawmill. The lake also served as a recreation area for the local residents. The property was eventually purchased by Ohio Department of Agriculture in 1928 with the intention to build a fish and game reserve. After being purchased, Ohio Department of Agriculture decided to rehabilitate the Upper Dam and create a larger lake by constructing a new Lower Dam.



Figure 1. Mt. Gilead Upper (right) and Lower (Left) Lakes Prior to Construction (Source: Google Earth)

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Prior to the start of this rehabilitation project, the Upper Dam was classified by ODNR Dam Safety as a Class 2 (intermediate hazard) dam consisting of a 90 feet long, 9 feet high, sharp-crested concrete spillway that is flanked on either side by a 15-foot high, 470 feet long earthen embankment. The Lower Dam was classified as a Class 1 (high hazard) dam featuring a 90-feet long, 12-feet high concrete ogee spillway that is centered on a 525-feet long, 20-feet high earthen embankment. Prior to rehabilitation the surface area of the Lower Lake was approximately 12.7 acres and impounded approximately 132 acre-feet of water at the embankment crest.

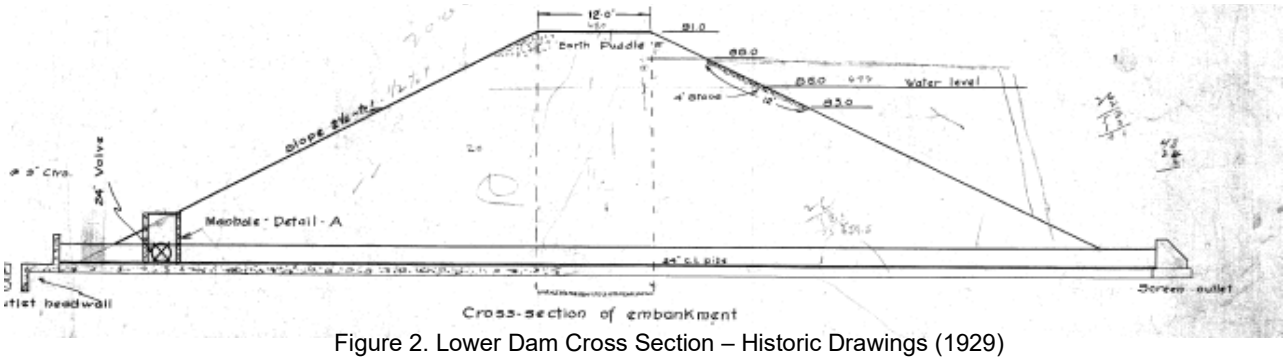


Figure 2. Lower Dam Cross Section – Historic Drawings (1929)


Figure 3. Lower Dam Prior to Rehabilitation (2014)
(Source: Michael Baker International, Inc.)


Figure 4. Upper Dam Prior to Rehabilitation (2014)
(Source: Michael Baker International, Inc.)

II. INSPECTIONS AND ASSESSMENTS


Figure 5. Typical Condition of Training Wall (2014)
(Source: Michael Baker International, Inc.)

ODNR Dam Safety performed an inspection of the Lower Dam in 2012 and an engineering consultant was retained to perform an assessment of both the Upper and Lower Dams, which was completed in 2013. The inspection and assessments revealed structural deficiencies of the Upper and Lower spillways, deficiencies within the Lower Dam embankment, and inadequate hydraulic capacity of the Lower Dam.

Deficiencies of the Upper and Lower Dams were similar in nature. Both spillways were functional, however the concrete for the training walls, wingwalls, and portions of the ogee spillways showed significant scaling, delamination, and areas of exposed steel reinforcement; all likely caused by exposure to decades of freeze thaw cycles. Concrete cores were obtained and tested from the spillway trainings walls at each dam and the results revealed low unconfined compressive strengths.


Figure 6. Boils at toe of dam partially obscured by brush/dead fall (2014)
(Source: Michael Baker International, Inc.)

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In addition to structural inspections/evaluations, visual inspections of the Lower Dam were performed to assess the condition of the embankment. Active seepage was observed at the toe of the embankment near the right abutment as well as significant damage to the embankment crest and downstream slope caused by a pedestrian usage. The seep, located within the footprint of the historic stream channel, fed a wetland along the toe of the embankment. Subsurface explorations were performed at multiple locations within the embankment and along the downstream and upstream toe. Laboratory testing was performed on the collected data and provided insights on the composition and material properties of the embankment. The subsurface exploration revealed occasional thin laminated sand lenses within the embankment fill, loosely compacted soils through the embankment, inconsistent profiles through the embankment, and different foundation soils below the right and left embankment sections. Borings and investigations also revealed a high accumulation of lake bed sediment within the lake. Initial seepage and stability analyses for the dam revealed borderline, satisfactory results for the embankment however, the spillway sliding capacity did not meet minimum requirements.

A 24-inch CIP gravity lake drain and valve vault was installed through the Lower Dam's right embankment section as part of the original dam construction. A replacement siphon lake drain and valve vaults were installed through the left embankment section in 2009. The original lake drain and valve vault were not formally abandoned after the installation of the siphon lake drain and poses a severe risk for internal erosion.



Figure 7. Lower dam embankment (Source: Michael Baker International, Inc.)

An initial hydrologic and hydraulic (H&H) analysis was performed to analyze the Lower Dam. Since the dam is classified as a Class 1 High Hazard structure, the Probable Maximum Flood (PMF) must be conveyed through the spillway without overtopping the embankment (since overtopping embankment protection has not been installed). The results of the analysis indicated the dam could only pass 25% of the PMF. To meet current ODNR Dam Safety regulations in its current configuration, sufficient overtopping protection would be required.

Because of the deficiencies identified, ODNR engaged Michael Baker International, Inc. (Michael Baker) to further evaluate the existing dams and to provide recommendations and alternatives to bring the Lower Dam into compliance with current ODNR Dam Safety regulations. Michael Baker was also retained to provide final design for the preferred alternative and construction administration services throughout construction.

III. PRELIMINARY DESIGN

Michael Baker was tasked to perform Preliminary Design services to fix the noted deficiencies at the Upper and Lower Dams. The Preliminary Design services included the following: additional subsurface investigations, evaluation of potential borrow areas, structural, geotechnical, and H&H analyses, detailed alternative evaluations, cost estimates, and preliminary design plans for the recommended alternative.

Through progress meeting meetings with the client, it was determined that the focus of the rehabilitation efforts would mainly be on the Lower Dam (the most critical dam). Two alternatives were considered for the upper dam; breach the dam or deregulate the dam and modify it into a sediment forebay. After considering the alternatives for the Upper Dam, ODNR chose to deregulate the Upper Dam. Deregulation was achieved by reducing the spillway crest height by 2.8 feet and reducing the embankment height by approximately 5 feet; thus decreasing the normal pool elevation and maximum storage capacity. In order to maintain the integrity of the spillway and the existing embankment, the Upper Dam training walls would require rehabilitation.

For the Lower Dam, ODNR not only wanted to bring the dam into compliance but also desired to create more usable reservoir area for recreation by increasing the depth of water in the lower lake, which would offset the costs to dredge the reservoir. Three alternatives were considered for the Lower Dam rehabilitation: 1) replace the existing embankment (with a higher embankment) and spillway while increasing normal pool by 2.5 feet, 2) replace the existing spillway and embankment, retain normal pool and dredge the lake, and 3) rehabilitate and stabilize the existing ogee spillway by providing concrete overlay and anchors, provide RCC overtopping protection to the embankment, and dredge the lower lake. Rehabilitation alternatives 1 and 2 both considered replacing the existing ogee spillway with a labyrinth spillway to provide the needed hydraulic capacity to safely convey the PMF.

Each design alternative was evaluated with regards to the construction costs, the effectiveness of each alternative to remediate the existing dam deficiencies, schedule implications, and the design life of the alternative. After considering all factors, it was determined that the most cost effective alternative would be design alternative

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1. This design alternative provided adequate hydraulic capacity to convey the PMF without overtopping the embankment, addressed the legacy issues within the existing embankment (existing penetrations, embankment construction deficiencies, and seepage concerns), provided additional lake volume by raising normal pool, and reduced environmental impacts since dredging was not included.

IV. FINAL DESIGN

The preferred design alternative and preliminary designs for the Upper and Lower Dams were advanced through final design to produce construction documents. A labyrinth type spillway was accepted by ODNR for the replacement spillway at the lower dam. The labyrinth spillway geometry, elevations of the principal and auxiliary spillway crest, and number of cycles were refined during final design and additional details were prepared. The labyrinth spillway was designed to satisfy two requirements: mimic existing flows up to and including the 100-year discharge and convey the PMF through the spillway without overtopping the new embankment. To meet spillway stability requirements, 12 post-tensioned soil anchors were designed within the spillway slab (which were reduced to 8 post-tensioned rock anchors during construction). A sheet pile cutoff wall extending approximate 15 feet below the bottom of the spillway slab was included to further protect against underseepage concerns.

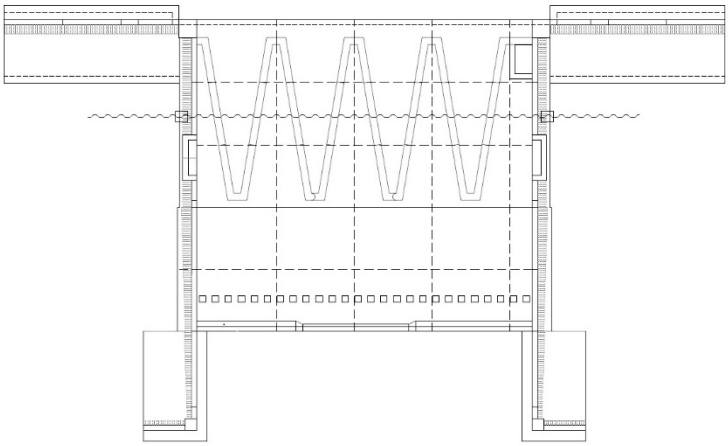


Figure 8. Lower dam labyrinth spillway plan – Construction Drawings (Source: Michael Baker International, Inc.)

A new control tower was designed on the spillway slab to eliminate the need for a separate penetration and potential seepage path through the embankment. The control tower contained two sluice gate and a stop log system mounted on a concrete bulkhead to provide ODNR greater flexibility to regulate pool levels.

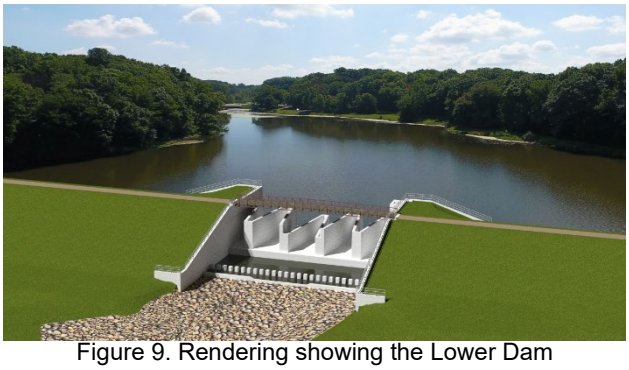


Figure 9. Rendering showing the Lower Dam (Source: Michael Baker International, Inc.)

As noted during the subsurface investigation and preliminary design phases, there were significant concerns regarding the lower dam embankment that included thin laminated sand lenses, loosely compacted soils through the embankment, inconsistent profiles through the embankment, and different foundation soils below the right and left embankment sections. During construction the team also identified abandoned field tiles and abandoned concrete remnants that were not noted on the as-built drawings. To address these issues, the lower dam embankment was completely removed to the original foundation and re-constructed using the existing embankment material and on-site borrow located near the right abutment. Since the borrow site was in close proximity to the dam, there was minimal hauling of material and no off-site embankment fill was necessary to construct the dam. The new embankment was constructed to meet current best practices that incorporated chimney and blanket drains to properly filter and manage seepage through the dam. To reduce the likelihood of excessive settlement and transverse cracking, the embankment was placed within the existing footprint of the existing embankment. Additionally, the new embankment crest was constructed 10 feet higher than the previous embankment to provide additional storage capacity to contain the PMF.

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Additional site features were also incorporated to minimize normal wear and tear on the embankment including rock revetment on the upstream slope, an aggregate path on the embankment crest, pedestrian path along the downstream right abutment, and pedestrian bridge over the spillway.

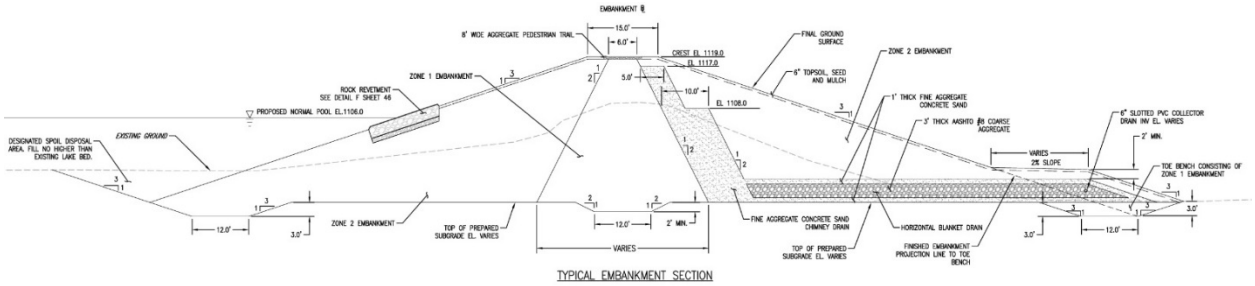


Figure 10. Lower Dam typical embankment section – Construction Drawings
(Source: Michael Baker International, Inc.)

V. CONSTRUCTION

The project was successfully bid in the late spring of 2016 and was awarded to Mosser Construction, Inc. of Fremont, Ohio. The contractor mobilized to the site in August of 2016 and completed construction in 2017.



Figure 11. Demolishing top of spillway to drain lake
(Source: Michael Baker International, Inc.)

Since the project involved the complete reconstruction of the lower dam, the contractor was required to lower the two lakes at approved rates so as to not de-stabilize the embankment slope or reservoir side slopes. The contractor elected to drain the upper lake by utilizing the existing lake drain and the Lower Lake was drained by demolishing the top of the concrete weir incrementally; allowing for the water to drain without exceeding acceptable rates. The incremental demolition to the spillway was repeated multiple times until the lake was low enough to breach the dam embankment and create a diversion channel that was used during Phase I of construction. This method of lowering the lake also allowed for fish to pass over the lowered spillway into the receiving creek where the fish were able to be netted and relocated to an adjacent water body.

Once both lakes were drained, the contractor began installation of their Phase I water control measures; which included the construction an earthen diversion berm to protect the work area and a diversion channel to route incoming stream flow. In order to install the diversion berm, the contractor had to remove existing lake bed sediment and expose a competent foundation for the diversion berm and the new embankment upstream toe. Utilizing a long reach excavator and haul trucks, the sediment was completely removed offsite. To maintain the project schedule,



Figure 12. Removal of lake bed sediment
(Source: Michael Baker International, Inc.)



Figure 13. Installation of diversion channel and berm
(Source: Michael Baker International, Inc.)

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the contractor elected to remove sediment shortly after draining the reservoir, which made sediment removal difficult given the material was not completely drained and became fluid when pushed with an excavator or dozer. The contractor explored alternate stabilization techniques, such as use of lime, but ultimately decided to advance along the embankment toe from left to right abutment as shown in Figure 13. The sediment was removed by trucks and placed just offsite. After the sediment was removed, a diversion berm was constructed upstream of the proposed dam and a diversion channel was constructed around the right abutment to protect the spillway work area throughout Phase 1 of construction (see Figure 14).

Following the successful installation of the Phase I water control measures, the contractor completely demolished the lower dam spillway and excavated the embankment to subgrade elevations. After the partial excavation, the sheet pile cutoff wall was installed using a vibratory pile driver. In order to continue to work through the winter months, the contractor initiated cold weather concreting techniques by adding admixtures to the concrete to help with curing and strength gains. To protect the subgrade and concrete from freezing, the contractor utilized ground thawing units in conjunction with thermal blankets. This technique proved to be successful as the contractor was able to install all the concrete cutoff walls and begin spillway slab placement by the start of spring.



Figure 14. Subgrade protected with heaters and blankets
(Source: Michael Baker International, Inc.)



Figure 15. Winter concrete placement
(Source: Michael Baker International, Inc.)

At the start spring, the contractor focused on installation of the spillway slabs. Prior to placement of each slab, the subgrade was approved and then covered by a concrete mud mat to protect the work and provide a stable base to construct the spillway. The placement of the spillway slabs required the contractor to accommodate several features including: steel anchor sleeves and anchor pockets for post-tensioned anchors (to be installed after spillway slab construction), sheet pile connections, keys at the base of the labyrinth spillway walls (required to provide the necessary clearance between the waterstop and the slab reinforcement), and the intricate waterstop placement. At the labyrinth cycle locations, the contractor utilized mechanical couplers for the reinforcement to allow access for the drill rig during anchor installation as the spillway geometry limited the available space for the drill rig (See Figure 16 for the completed spillway slab).

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Following spillway slab placement, the contractor moved on to construction of the stilling basin slabs and stilling basin drainage system. The drainage system was composed of filter drainfill aggregate and slotted PVC pipe, which outlets into the stream channel (See Figure 17). As a requirement of the project, drainfill was required to have less than 5% fines after compaction and periodic gradation testing was performed to verify that the drainfill met this requirement. Stilling basin slabs were constructed similar to the spillway slabs and were completed in May of 2017 (See Figure 18).



Figure 17. Installation of Stilling Basin Drainage
(Source: Michael Baker International, Inc.)

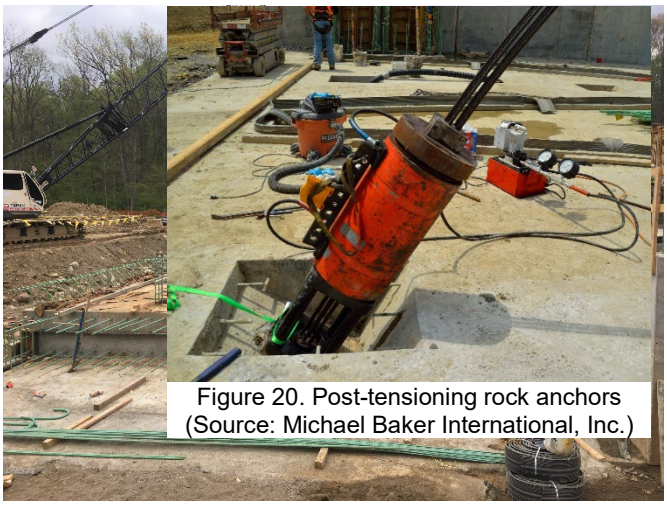


Figure 18. Stilling Basin Slab Construction
(Source: Michael Baker International, Inc.)

Based on the spillway geometry and loading conditions, post-tensioned anchors were required to improve resistance against sliding. Anchor pockets were cast into the spillway slab to accept the 12 post-tensioned soil anchors with a total strand length of 80 feet were indicated in the original design. The prime contractor's subcontractor had difficulty installing the spillway anchors into the glaciated foundation soils, which resulted in significant construction delays. This was generally due to running sands within the hole and unbalanced head pressures causing issues with both grouting and reaching hole depths. Additional borings onsite were drilled to determine the variation of glacial soils being encountered. To prevent further delays in construction, the project team elected to install post-tensioned rock anchors into the underlying bedrock that was approximately 135 feet below the spillway slab. New anchor strands were ordered since the total strand lengths increased from 80 feet to 180 feet. Since the rock anchors had more capacity than the soil anchors, the total number of anchors was reduced from 12 post-tensioned soil anchors to 8 post-tensioned rock anchors and the unused anchor pockets were filled with concrete. After the anchors were installed and holes were grouted, the anchors were stressed and accepted. Since the new spillway and increase embankment size would increase the loads of the foundation soils, the settlement of the spillway was expected. The settlement was monitored through periodic surveying of alignment pins that have been set into the concrete structure. After the rate of settlement decreased, the anchors were re-stressed after the spillway was loaded with embankment and spillway settlement occurred.



Figure 19. Drilling Spillway Anchors
(Source: Michael Baker International, Inc.)

With the spillway and stilling basin slabs constructed, the contractor focused on the construction of the training walls, stilling basin walls, and control tower. Care was taken by the contractor to ensure that all PVC waterstops that were installed within the base slabs were connected to the vertical joints of the training walls to ensure a watertight structure. As dictated by the Contract Documents, the contractor was permitted to perform only butt-welds in the field while complex unions were required to be fabricated by the waterstop supplier. In addition, all waterstop unions were required to be inspected by the contractor before and after installation to ensure waterstops were installed correctly to ensure the structure was watertight.

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Figure 21. Water control through the spillway
(Source: Michael Baker International, Inc.)

Once the spillways training walls were constructed and have been properly cured, Phase 2 of the water control plan was implemented that involved conveying normal stream flow through the spillway structure (See Figure 21). With the streamflow diverted through spillway, the embankment was reconstructed evenly on both sides of the structure so as to prevent differential loading and settlement of the structure. The embankment was constructed with premium low-permeability clay used for the core and available borrow material used for the shell. A chimney drain was installed immediately downstream of the core, and a blanket drain with a collection pipe at the downstream toe. Vibrating wire piezometers were installed within the left and right embankment section (relative to the spillway) at various zones within the embankment. The piezometers cables from the left and right embankment were collected to an instrument panel located on either side of the spillway. As the embankment was placed, piezometer and settlement readings were collected and monitored. The information gathered was used to track the embankment placement and monitor pore water pressures within the embankment and foundation soils.



Figure 22. Blanket drain installation
(Source: Michael Baker International, Inc.)



Figure 23. Embankment construction
(Source: Michael Baker International, Inc.)

As the spillway structure was loaded with embankment, the contractor elected to begin construction of the spillway weir up to a horizontal control joint located approximately 1-foot below the spillway weir crest, where the remaining one foot would be poured after the settlement period (See Figure 24). This was done to avoid having the weir crest settle below the targeted normal pool and/or settle unevenly. The contractor continued to divert normal flows through unfinished spillway weir panels, the control tower, and/or bypass pumping. Multiple storm events occurred during this span in construction while the labyrinth spillway weirs were constructed; however, the contractor scheduled the work around these storms to ensure no damage to the new spillway structure (See Figure 24 for streamflows diverted through an unfinished spillway wall).



Figure 24. Partial construction of the spillway weirs
(Source: Michael Baker International, Inc.)

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After the embankment was fully constructed and the rate of settlement diminished, the Contractor re-tensioned the rock anchors and the anchors were greased, capped, and then the pockets were filled with concrete. The Contractor then poured the remaining spillway weir crest to the final spillway elevation. By delaying this final pour until after the settlement period, it was possible to obtain a constant crest elevation across the weir at the designed crest elevation. After the spillway weir was constructed and the concrete was adequately cured, the reservoir was refilled in accordance with the designated refill plan that incorporated a gradual refilling as well as hold elevations to monitor the dam.



Figure 28. Upper dam spillway after rehabilitation
(Source: Michael Baker International, Inc.)



Figure 29. Upper dam after reservoir refilling
(Source: Michael Baker International, Inc.)

While the Lower Dam was rehabilitated, repairs to the Upper Dam were performed to satisfy the following objectives: decommission the upstream dam in accordance with dam safety regulations and to serve as a forebay to control sediment prior to discharging into the Lower Lake. To decommission the structure, the structure needed to be modified to reduce the storage volume of the reservoir and reduce the total height of the embankment to 10 feet or less. To accomplish these requirements, the spillway crest was partially removed and a new spillway crest was formed 2.8 feet below the existing weir. The embankment crest was also lowered by approximately 5 feet so that the total embankment height was less than 10 feet. The final configuration was designed to safely convey the 100-year design storm through spillway structure prior to overtopping the embankment. Additional modifications were made to the upper dam to extend the life of the structure. The existing training walls exhibited severe structural deterioration and needed to be repaired. Rather than replacing the structures, rehabilitation designs focused on re-facing the training walls to give a consistent aesthetic look with the rest of the site. Tieback anchors that were connected to a deadman anchor in the embankment were installed to meet stability requirements since the stability of the existing walls were expected to be compromised given the degree of deterioration.



Figure 27. Upper Dam training wall facing
(Source: Michael Baker International, Inc.)

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Mr. Afek is a department manager and project manager with Michael Baker International, Inc. He has over 16 years of professional experience specializing in dam inspection, dam analysis, design of dam rehabilitation components, and construction management for high hazard dams. He has served as the project manager for dam rehabilitation projects throughout the Great Lakes Region. Mr. Afek graduated with a B.S. in Civil Engineering from the Ohio State University and is a licensed Professional Engineer in the states of Ohio, Pennsylvania, and Louisiana.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

From Translational to Transformational Flood Risk Communication: How USACE is Shifting its Communication About Dams, Levees, and Flooding

KATELYN NOLAND AND ADORATIA PURDY, US ARMY CORPS OF ENGINEERS

ABSTRACT

The U.S. Army Corps of Engineers (USACE) Dam and Levee Safety Programs are rich with technical knowledge and information about how dams and levees work, how they can fail, and what the impacts are if failure occurs. To date, the programs have placed emphasis on getting the details right and translating information for public consumption. Far too often this translation process is arduous and still leaves the agency focusing more on what it wants to communicate instead of framing information and communication to promote action and collaboration to address flooding. This session will provide an overview of a 24-month study USACE completed with HDR-Gannett Fleming on how to improve its communication practice and integrate social and behavioral science-based approaches to engage communities and individuals in flood risk decisions. Presenters will discuss how USACE has updated its communication initiatives based on this study and explore approaches that may be relevant to others in the flood risk community.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

Whittier Narrows Dam: Managing Risk Thru Communication: A Partnership Between USACE and The City of Pico Rivera

MELANIE ELLIS AND AARON SMITH, US ARMY CORPS OF ENGINEERS; AND JAVIER HERNANDEZ, CITY OF PICO RIVERA

ABSTRACT

Whittier Narrows Dam, owned and operated by the US Army Corps of Engineers, is the centerpiece of flood risk infrastructure within Los Angeles County. Located in the nation's most populous county, Whittier Narrows Dam provides flood risk reduction benefits to approximately 1 million people within 25 municipalities. USACE is currently designing for a modification to address both stability concerns and a hydrological deficiency. This significant project is scheduled to be completed within 4-5 years. Communication plays an important role in supporting meaningful dialogue with those who collectively work to manage flood risk, plan for, and respond to emergencies, as well as increasing awareness of the importance dams play in people's lives.Communicating to a large, ethnically diverse population in an urban setting with considerable competing priorities (earthquakes, drought, fires, congestion, affordability) is no small task. USACE continually seeks opportunities to engage in meaningful dialogue with elected officials, emergency planners and responders, as well as the public at large. Along the way, USACE formed a valuable and important partnership with the City of Pico Rivera, located immediately downstream of the dam and situated within the highest risk areas due to both spillway releases and breach waters. City officials quickly grasped the importance of engaging in meaningful dialogue to improve their emergency preparedness and increase the understanding of the importance Whittier Narrows Dam plays in the continued health of their community. This foresight led to innovative communication and multimedia strategies employed by Pico Rivera. The City wanted to give its residents this vital information given urgency to address the identified deficiencies. Providing real time updates from USACE through Social Media platforms, or timely updates through newsletters, provides City residents with the most accurate information available which is crucial in a time when everyone is receiving information from unofficial sources that is often exaggerated creating unnecessary panic. This presentation will highlight how the partnership continues to play a vital role in helping manage flood risk and sharing the importance of dams in our society.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

General Session:Colorado River Past, Present and Future

Moderator: Nathaniel Gee

ABSTRACT

The Tuesday General Session will inform dam safety professionals about the history and current state of the Colorado River operations and water policy Experts will discuss the importance of the Colorado River and its impact on the growth of the United States. The audience will learn more about what it means to be a State within the Upper and Lower Basins and speakers will bring their unique perspectives from both viewpoints. Topics will range from historic water issues facing decisionmakers in both Basins, how the recent drought has impacted each Basin differently, and how current runoff, hydrology and operations issues have affected life in the Basins.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

Bourbanis Dam: The Spring 2022 Emergency Response and Moving Forward to Mitigate Future Risk

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ABSTRACT

The spring flood of 2022 came seemingly out of nowhere. By early April, an uneventful spring runoff season was already beginning to subside in the Tongue River Watershed. Conditions rapidly changed when much of North Dakota received a late season snowstorm in mid-April with snowfall accumulations anywhere from 12 inches to upwards of 30 inches within the Tongue River Watershed. Snow rapidly melted in late April/early May due to spring rains, causing 10 flood control dams within the watershed to nearly reach, and in some cases exceed, storage capacity.One such dam that exceeded capacity was Bourbanis Dam, located in eastern Cavalier County, ND. The dam is owned and operated by the Pembina County Water Resource District (PCWRD). The flood pool quickly rose to the auxiliary spillway, resulting in a partial failure of the earthen grass-lined spillway. Bourbanis Dam is considered a high-hazard structure, meaning that failure has potential to result in loss of life downstream. Also compounding concern were downstream dams already at capacity as well as severe flooding occurring further downstream along the Tongue River including the community of Cavalier, ND. Rapid, emergency response was undertaken to reduce risk of a full dam failure. Luckily, the efforts were successful.While the immediate risk was resolved, the PCWRD was then faced with the questions of “what’s next?”. There were two main considerations: 1) how to mitigate the immediate risk of a partially failed structure, and 2) how to ensure the long-term viability of flood control within the watershed. The talk will provide background on conditions leading up to the 2022 flood event, actions taken during the emergency response, and next steps that the PCWRD is taking to move forward.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

A Reservoir Restriction Toolbox

An ASDSO Dam Design & Construction Committee Topic Paper

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ABSTRACT

A Dam Design and Construction Committee (DDCC) task group recently studied various methods that can be used to establish reservoir restrictions if unsafe conditions are observed at a dam. This paper will provide an overview of methods, including simple, intermediate, and advanced approaches, that can be employed to establish the safe storage level. The appropriate levels of analysis should be based on the conditions observed at the structure and information available to evaluate its performance. The paper also explores the life cycle of a reservoir restriction including setting a restriction level, revisions to the restriction level, lifting the restriction to allow full storage, and issuance of zero storage restrictions and breach orders. Several case studies are included to demonstrate the approach for the various risk and condition levels.

I. INTRODUCTION

The core function of dam safety regulators is to determine the amount of water that can be safely impounded by a dam or water retaining structure. The most effective means to protect the downstream public when a dam has deficiencies is to restrict the impounded storage. It is this authority of dam safety regulators to impose reservoir storage restrictions that have formed a foundation of trust that the public has placed in regulators.

The ability to store water is often considered a property right. That property 'right' is often critical to the solvency of the owner as it supports their means of production (e.g.: irrigation, power production, etc.) or allows them to provide services to their customers (e.g.: water utility, recreation, etc.). The ability to store water supports a healthy and functional society. Imposition of a storage restriction is tantamount to 'taking' that property right away, analogous to condemnation or eminent domain acquisition of real estate. In the case of water storage 'rights', the right is borne indirectly by the risk imposed on those living downstream of the dam. Given the critical nature of water storage on the owner's means of production, the proper function of society, and the risk borne by the downstream public, it is incumbent on dam safety officials to provide reasonable and prudent justification for imposing storage restrictions when critical dam safety issues arise. The goal is to establish an effective reservoir restriction (i.e., restricted pool level) based on available evidence utilizing transparent and technically defensible analyses and best practices to reduce the risk to a tolerable level while, if possible, also allowing some level of

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storage for beneficial use. *It is important to note that the beneficial use of water must be held secondary to the safety of the downstream public*.

Once a dam safety deficiency is identified or observed, and after any immediate threat to the safety of the structure is mitigated, the dam safety regulator must determine what amount of storage the dam can safely impound. Imposing a reservoir storage restriction is the most common way to reduce the risk of dam failure because it reduces the pressure on all components of the dam and reservoir system. Reducing the impounded volume and hydraulic head also reduces the potential energy stored in the system and therefore reduces the magnitude of damage downstream that could result from a dam breach, reducing the consequences. It also creates more retention volume in which flood water could be buffered if the outlet capacity is lower than the reservoir inflow. Imposing a restriction reduces the threat to the downstream public and allows for an engineered solution to be implemented.

Breach orders and zero storage restrictions may be warranted in some extreme cases because they minimize the risk to the downstream public. However, this level of restriction is not frequently warranted and may be overly punitive to the dam owner in many cases because it removes most, and sometimes all, of the beneficial uses of the reservoir.

Any amount of reservoir restriction can have undesirable impacts outside of dam safety. In addition to the primary purpose of the reservoir, secondary benefits are also impacted (e.g., recreation at a water storage reservoir) and reduces benefits of other stakeholders besides the owner such as customers, neighbors, and other project partners. The over-use of draconian measures may draw scrutiny from detractors, and reduce the overall credibility and effectiveness of the regulatory program. Restrictions must, therefore, be utilized judiciously and with adequate justification. The use of transparent methodologies supported by evidence and analysis consistent with industry standards allows regulators to maintain the trust of downstream populations, and also build trust with owners which will garner responsiveness to address dam safety issues in a timely and effective manner in the future.

The purpose of this paper is to describe a tiered approach that regulators can utilize to establish defensible reservoir restriction levels. The tiers include consequence reduction with simple, intermediate, and a more advanced, risk-based approach that regulators can use depending on the conditions observed at the structure and information available to evaluate its performance. Case studies[1] are presented that represent real-world scenarios where regulators utilized various tiers to justify restrictions. This paper also explores other considerations related to storage restrictions including enforceability, the life cycle of a reservoir restriction, and issuance of zero storage restrictions and breach orders.

II. CONSEQUENCE REDUCTION

When setting a storage restriction, the reduced impounded storage volume and hydraulic head decreases the pressure on the structure and potential energy stored by the system which will result in a smaller breach flood. This reduction in consequences may result in a reduced hazard posed downstream. If the stored energy can be reduced to the point that a breach flood no longer results in unacceptable consequences, then this may be an appropriate way to establish the reservoir restriction level.

Regulators must also abide by statutory limits to their authority. For example, Colorado statutes generally establish a limit on dam safety regulatory authority to apply primarily to dams exceeding 10 feet in height, or dams which impound a reservoir storage volume of 100 acre-feet or surface area of 20 acres. Setting the storage restriction to reduce the dam/reservoir to less than this size may be viewed as regulatory overreach unless the structure is still deemed to pose a significant or high hazard to the downstream public at the reduced size.

[1] Case studies presented herein are based on real situations, but details and names may have been modified in an attempt to provide anonymity.

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III. SIMPLE OBSERVATIONAL APPROACH

The simplest approach to establishing a reservoir restriction level relies solely on visual or instrumented observations of the behavior of the structure. When a dam exhibits an easily observable manifestation of distress, the reservoir level that initiates that behavior can be used for the reservoir restriction. Several examples include the following when observed to correspond to a specific reservoir level:

- Cloudy seepage.
- Increasing seepage over time.
- Phreatic surface changes observed in piezometers.
- Unfiltered seepage emanating on the downstream face.
- Spillway or other outlet structures deteriorated to the point that they are deemed unsafe to pass flow.

This approach relies on regular inspection and collection/analysis of monitoring data to identify the specific reservoir elevation below which physical manifestations of distress disappear. For example, if cloudy seepage is observed in drain flows when the reservoir pool exceeds a certain elevation, then this simple observational approach should be to lower the water elevation until seepage flow is sufficiently reduced or cloudiness abates. Documentation of these observations could serve as the sole basis for the restriction.

Since the simple approach may be based on limited data and employ simplifying assumptions, it should be conservative. At a minimum, the adequacy of the simple observational restriction should be determined. For example, if the pool level is lowered sufficiently to eliminate physical manifestations of distress (e.g., cloudy seepage), but a high-frequency rainfall event is sufficient to load the dam beyond acceptable limits, then the initial restriction may not be adequate. This would require stepping up to the intermediate tier to calculate the expected inflow volume of a frequent flood event that is necessary to store without triggering the undesirable response.

The U.K. Environment Agency 2017 Guide to drawdown capacity for reservoir safety and emergency planning [5] Table 2.3 identifies drawdown elevations required to avert failure for several failure mechanisms. The document gives the following overall guidance considered generally appropriate:

> *"The decision of how far to draw the reservoir down in an emergency should be based on advice by an inspecting engineer. The advice is likely to be to lower the water level until the symptoms of failure cease, and then lower it by a further amount to provide a margin of safety, for example to prevent flood inflows reinitiating the failure mode. In most cases, lowering a reservoir by one-third of its depth will significantly reduce the risk of failure progressing (this is equivalent to roughly halving the hydrostatic force)."*

This 'one-third depth' rule of thumb for establishing a restriction level is a good starting point when signs of distress are noted, but detailed observations of changing response with changing reservoir level are not available since it removes significant pressure from the system. It also removes a significant portion of reservoir storage since most reservoir storage exists in the upper third of the reservoir pool as illustrated in a typical stage-storage curve represented by the volume of an inverted cone as presented below. This curve shows that more than 60% of reservoir storage exists in the upper third of the reservoir pool.

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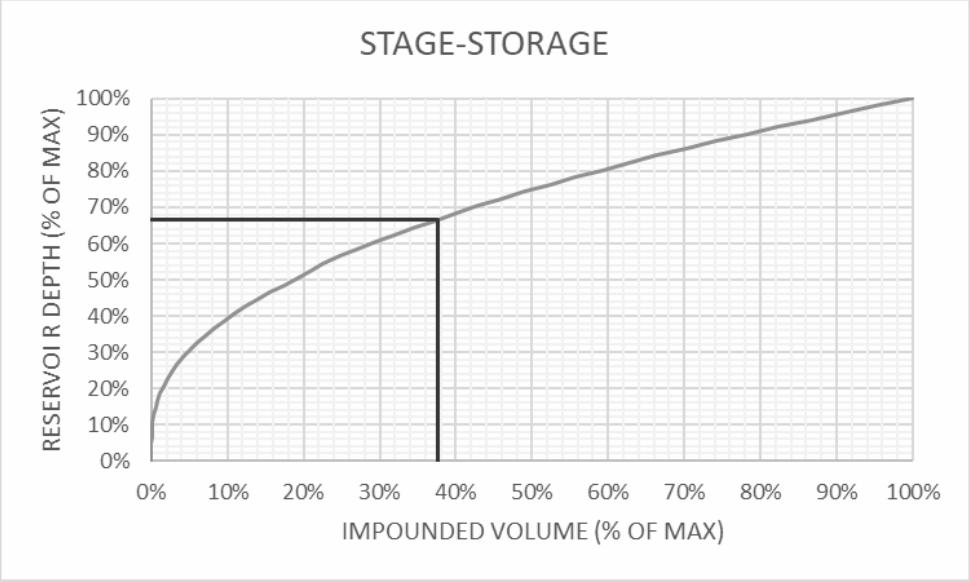


Figure 1: Typical reservoir stage-storage relationship (inverted cone)

Whatever type of simple analysis is used to justify the restriction, the regulator should clearly document and describe the observations that were utilized to set the restriction level in their communication to the dam owner. This will allow the dam owner to hire an engineer with adequate scope to evaluate the root cause of the restriction, investigate the site as needed, analyze the mechanisms that are believed to contribute, and design remedial measures accordingly.

A few case studies are described here to demonstrate this simple observational approach.

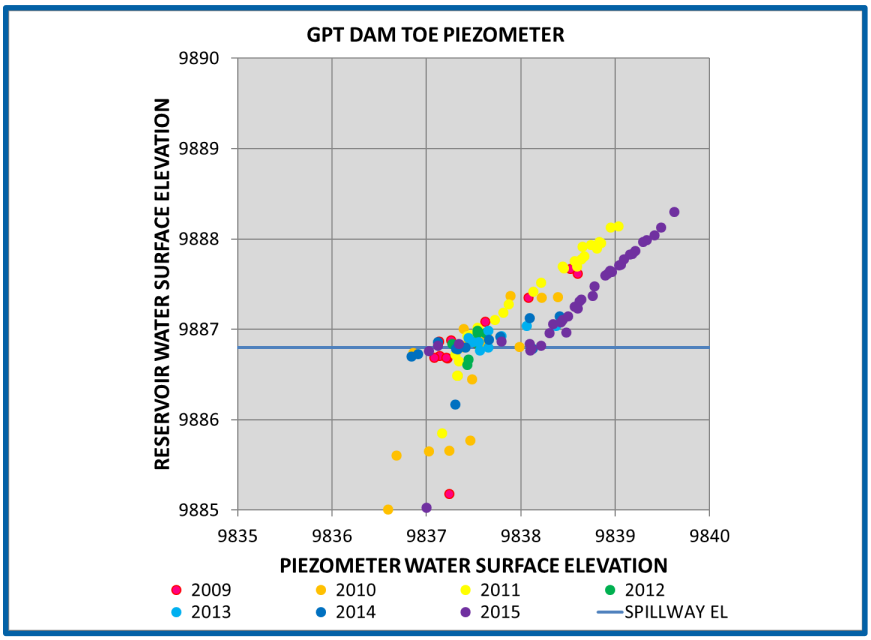
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CASE STUDY

Duck Meadow Dam
Simple Observational Approach
Change in Character of Piezometer Response

Hazard: High
Dam Height: 60-ft
Impounded Volume: 800 ac-ft
Spillway Configuration: Reinforced Concrete Chute over Embankment

Duck Meadow Dam is an embankment dam in a narrow mountain valley. Although not ideal, the valley width and site constraints have resulted in both a service and emergency spillway being constructed on the embankment. The service spillway is a concrete-lined chute and the emergency spillway immediately adjacent is an RCC-lined chute. Over the course of several years of diligent monitoring performed by the owner, piezometers within the downstream shell located immediately adjacent to the service spillway which normally showed little to no influence from reservoir level began showing a significant response in pore water pressures when the reservoir water level exceeded the spillway entrance elevation. Plots of monitoring data from one of the piezometers is provided below which shows a clear change in behavior once the reservoir water surface exceeds the spillway elevation where the piezometer response transitions to a nearly a 1:1 response.



In addition to the change in behavior, the response was trending in an unfavorable direction year-over-year. The plot above shows several years of data collection and analysis indicated that the issue was getting worse which was particularly evident from 2011 to 2015. Further site investigation revealed that a seepage entry point had developed below the service spillway base slab which was transmitting seepage underneath the slab. A preferential flow path had developed under the slab which was conveying water to the piezometer.

The void acting as a seepage path indicated a loss of intimate contact of the spillway base slab with the underlying foundation. A storage restriction was issued to maintain the reservoir level 4-ft below the service spillway slab to allow for some flood routing capacity without re-initiating the seepage pathway.

Setting the restriction at 4-ft below the service spillway in this case was not based on any engineering calculations. It was established based on the lowest elevation achievable by lowering the adjacent RCC-lined emergency spillway. The emergency spillway modification was accomplished rapidly under an emergency contract to allow the owner time to hire an engineer to design a permanent solution. The owner's responsiveness to monitoring and maintaining the reservoir below the service spillway and their readiness to deploy other emergency mitigation features also weighed into establishing the restriction level.

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CASE STUDY

Muddy Waters Dam
Simple Observational Approach
Increasing Drain Flows

Dam Height: 80 ft
Impounded Volume: 1,500 Acre-ft
Hazard Class: High

This privately owned, high-hazard dam impounds a relatively small volume of water (less than 1,500 AF at full pool) but has such high seepage rates that it has filled only a handful of times since original construction in the 1950s. The downstream face is dry and free-draining; seepage exits primarily through glacial moraine material located along the right abutment contact. Several seepage entrance locations (sinkholes) have been observed within the reservoir pool, but none adjacent to the dam. A significant seepage upwelling located approximately 150 yards downstream of the toe requires careful monitoring. The dam is in a high-elevation basin that requires off-road vehicles for access and has no cell service. Failure would inundate much of a small town within 15 – 30 minutes with essentially no warning time. A second town would also be inundated with greater warning time.

A number of years after construction, the dam was likely damaged during a grouting project that used dynamite in the grout holes to attempt to increase uptake. It is also likely that the toe drain pipe was partially crushed shortly after construction and was built with bell-and-spigot style open joints.

For the first time in decades, the reservoir filled in back-to-back years in the 2010s – an event that was similar to a "first fill." The next year, sediment was observed in the toe drain for the first time. Engineering inspection and documentation up to this point had historically been relatively good. However, the dam changed ownership and within a couple of years, documentation was poor and of limited value. After more careful monitoring was ultimately re-established, sediment was consistently observed in the toe drain in subsequent years. Available data also indicated that the toe drain flow rate was increasing at lower reservoir elevations, as compared to flow rates for the same reservoir elevation prior to back-to-back fills.

The spillway crest and maximum normal operating pool are at elevation 5118 ft. Engineering inspection reports prior to the back-to-back filling stated that the toe drain began to flow when the reservoir reached elevation 5095 feet, with an approximately 1-month lag time. Recent data indicate that the toe drain still flowed between 5 – 30 GPM at this elevation. Recent observations did not detect sediment in the toe drain if flow was reduced below approximately 20 GPM.

From a dam safety perspective, a drain flow of 20 GPM is still considerable for a dam of this size. Recent data indicate that at reservoir elevations of 5090 feet, corresponding toe drain flows are 10 GPM or less, with no sediment. Historical (pre-back-to-back filling) data indicate that drain flows were minimal or eliminated at or below this reservoir elevation.

In order to minimize drain flows and the potential for material transport, and to avoid hydraulically loading the dam above the elevation at which drain flows are known to occur, the regulatory agency implemented a reservoir level restriction of 5090 feet, representing a 28-foot and approximately 1,000-AF restriction. The actual impact is much less, since, even without the pool level restriction, the reservoir almost never fills completely. For example, in the 15 years preceding the restriction, the reservoir did not even reach the restricted pool elevation in one-quarter of the years.

No additional safety margin was applied to this restriction, as further decreases would make the reservoir non-viable for irrigation. However, the owners were required to install remote monitoring of the reservoir level given the difficult access. In addition, an analysis was made to understand the potential for a storm even to unexpectedly raise the reservoir.

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CASE STUDY

Sadenbeck Dam
Simple Observational Approach
Staged Restriction based on Multiple Deficiencies

Dam Height: 30-ft
Impounded Volume: 851 Acre-ft
Hazard Class: High

This 30 foot high (crest to low-level outlet) earthen embankment dam has a reservoir volume of 851 AF at permitted pool level. The dam was built and permitted in 1988 for irrigation purposes. The 1,000 foot long dam is constructed as a zoned embankment dam with a sand shell and a 3 to 6 foot thick clay core layer as an impermeable barrier on the upstream face extending 30 feet from the upstream toe on the reservoir floor. The clay core layer is protected by a granular gravel filter and a rip rap cover. The dam has one low-level outlet which also includes a service spillway. Neither of the outlet gates were operable due to major damage and sediment buildup. The emergency spillway crest was partially overgrown and limited in capacity. The dam includes a toe drain constructed as a single PVC drainage pipeline (4”), wrapped in a geotextile filter with manholes every 200 feet. The estimated seepage gradient is 4.4H:1V from the permitted pool level at the service spillway crest elevation to the dam toe near the low-level outlet. The dam is considered a high-hazard dam based on reservoir volume and a potential loss of life due to the proximity to a major highway approximately 750 feet downstream. The highway grade is lower than the permitted water level and a potential breach flood would likey overtop the highway. The ownership remained unclear due to legal implications after two years of operation which left the dam abandoned for decades until finally the state gained ownership in 2017.

Reservoir restrictions were imposed in several steps by state officials. The first restriction limited the pool level by approximately 2-feet after wet spots were observed near the toe drain manholes during an inspection in 2005. The local watershed association, an organization for maintaining small privately owned water bodies, volunteered to execute the restriction and took responsibility to maintain the dam financed by the county. The wet spots disappeared after cleaning the clogged drainage pipe. This restriction also served to provide sufficient storage for the reservoir to impound a 1/100 annual exceedance probability flood event without the need to operate the non-functional outlet gates.

After the state obtained ownership of the abandoned dam in 2017, a full inspection was ordered including a sub-surface investigation, PMF evaluation, seepage monitoring, and review of stability calculations. The geotechnical investigation observed artesian groundwater near the low-level outlet during drilling. Calculations showed a very high likelihood of a toe failure (heave) which led to the second reservoir restriction and drawdown to gage height 16.4 feet, 6.5-ft from the already limited pool level in early 2019 as an emergency action response. The second restriction considered a PMF-event as hydraulic impact without causing overtopping and safe phreatic level near the toe in order to reduce pressure and seepage flow. Further, the reduced pool level is now below the highway grade which decreased the likelihood of overtopping the highway. The drawdown reduced the seepage gradient to 8H:1V. The reservoir volume decreased by nearly 50% (851 AF to roughly 405 AF). The flood retention volume doubled from 324 AF to 730 AF, and the modified Spillway capacity is able to handle a PMF event without overtopping.

The dam remains in a restricted state since the deficiencies (damage to outlet gates and sediment buildup) are not yet resolved. The owner has no interest in regaining the full capacity of the dam and ordered a feasibility study to repurpose the reservoir as flood control reservoir or recreational pond.

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IV. INTERMEDIATE EVALUATION APPROACHES

Intermediate approaches utilize engineering analyses, calculations or modeling to establish a reservoir restriction. This may involve several disciplines including hydrologic, hydraulic, geotechnical, structural, mechanical or other types of calculations as described below.

A. Hydrologic

It may be that a spillway or outlet works is deemed unsafe to convey any flow. In this case, hydrologic calculations can be performed to determine how low the reservoir pool must be maintained in order to capture the flood of concern without activating the spillway and/or outlet. Hydrologic modeling can be used to estimate the volume of runoff anticipated for storms of varying frequencies. This information provides the regulator the ability to determine what probability of loading is acceptable.

When the tributary area is large, the dam/reservoir may not be able to fully retain an inflow design flood (IDF) established for the structure, particularly when the IDF is a very rare event (e.g., the Probable Maximum Flood), however, a significant level of protection and risk reduction may be gained simply by restricting the reservoir pool such that the dam can store the full volume of the 1/100 annual exceedance probability flood. This reduced probability of spillway flows for a relatively short time period may be acceptable. This provides the owner with some time to hire an engineer and contractor to design and construct a new spillway.

A common practice to inform decisions based on hydrologic loading is to develop a hydrologic loading curve to plot the peak reservoir level vs. annual exceedance probability of the flood event. This can be used to estimate probability of loading for risk-based analysis of hydrologically driven potential failure modes. Different curves can be developed based on different initial conditions in the reservoir to represent various restriction levels or operational scenarios as illustrated by the figure below..

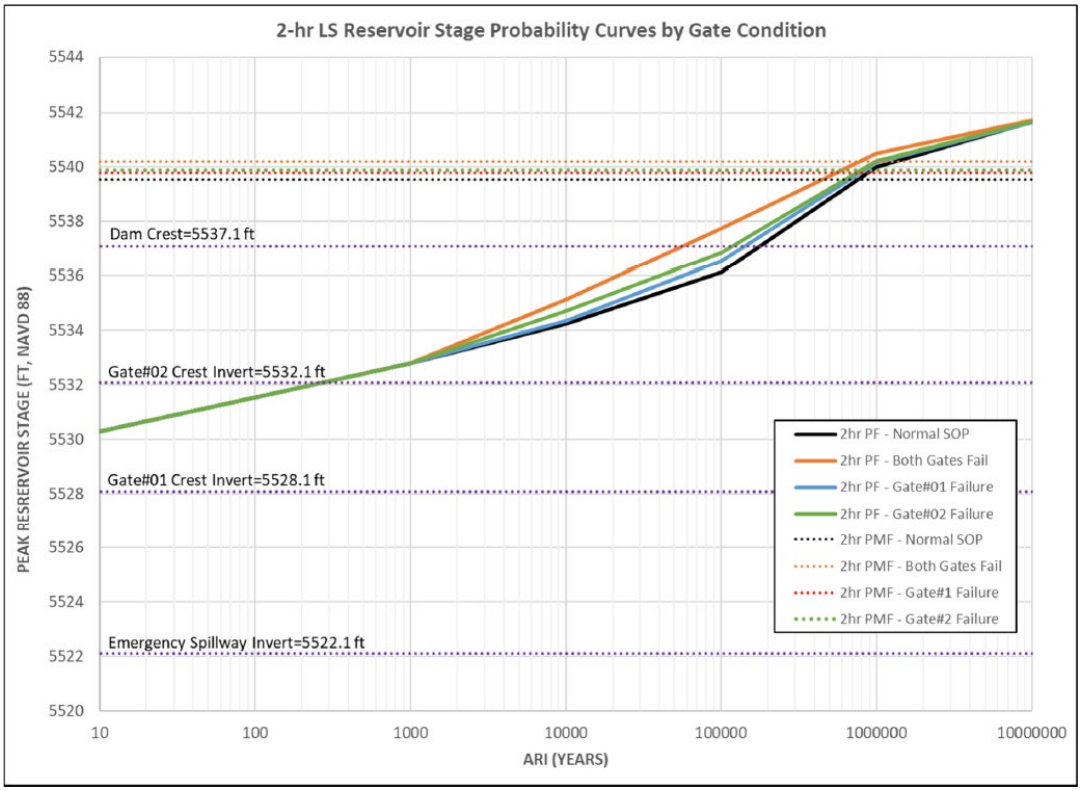


Figure 2: Example Hydrologic Loading Curve

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In some cases, it may be acceptable to use a more frequent hydrologic event to establish a safety margin for loading the spillway. Some factors that could be used to justify this decision include: owner actively pursuing improvements so period of exposure will be limited, dam accessibility and ability to utilize low level outlet works to augment spillway capacity, owner's willingness and ability to rapidly deploy other means to lower reservoir including pumps/siphons, and reduced downstream consequences at the lower reservoir level, amont others.

B. Hydraulic

Hydraulic modeling can be used to estimate flow velocity, depth, shear stress, cavitation potential and other hydraulic parameters which may indicate initiation of a failure mode. This could be used to determine what amount of flow, if any, can be allowed through a structure with a known defect. A combination of hydraulic and hydrologic modeling can be used to establish a reservoir level appropriate for outlet works restrictions such as limited spillway flows. Example calculations could include soil erosion and head-cutting for an un-lined spillway (shear force, stream power, etc.) or cavitation potential for high velocity, long duration flow events in a concrete-lined spillway chute.

The U.K. Environment Agency 2017 Guide to Drawdown Capacity for Reservoir Safety and Emergency Planning [5] provides comprehensive guidance for assessing and evaluating the outlet works capacity required to evacuate reservoir volume in an emergency. The outlet works must be designed to reliably perform under the maximum required discharge. USBR ACER TM-3 [8] provides similar guidance.

C. Geotechnical

Seepage modeling can be used to estimate phreatic level within the embankment, potential seepage exit points, and hydraulic gradients. Hydraulic gradients can be used to determine the likelihood of internal erosion or backward erosion piping of the embankment or foundation soils. The modeling can vary input reservoir level to determine the sensitivity of output to loading and identify what reservoir level reduces the gradient to the point that it is no longer a concern as estimated factor of safety agains particle detachment, or to the point that the seepage is no longer expected to reach an unfiltered exit.

There are several federal publications that establish standards for seepage and slope stability factors of safety. For example, the US Army Corps of Engineers Risk Management Center has established toolboxes for various geotechnical issues including internal erosion [6]. Internationally, the German Federal Institute for Hydraulic Engineering has established that a hydraulic gradient of 1V:10H is generally considered to be safe if certain geometrical conditions apply [1]. References like these can be utilized to establish safe conditions in combination with seepage modeling.

Slope stability modeling can also be utilized to determine the factor of safety against sliding of a particularly concerning failure surface. This modeling can utlilize monitoring data from piezometers for phreatic surface within the embankment or a seepage model as described above could be utilized to evaluate the range of phreatic level in the dam based on varying reservoir level input to see how it effects the calculated factor of safety for a failure envelope of concern.

D. Structural

Modeling of structural performance under various load combinations can be used to estimate the factor of safety against deflection or failure. Strength reduction factors can be used if the structural components are known to be compromised in any way, such as reduced section due to corrosion or general deterioration. Once factors of safety drop below acceptable levels for the category of load case (usual, unusual or extreme), a restriction may be warranted. This may involve reducing the reservoir level to reduce the load on the structure. It may also involve limiting other loads on the structure when possible (weight restricted bridges are an example). Structural

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calculations can also be used in combination with any of the other disciplines listed here to establish the load limit and justify a reservoir restriction.

E. Mechanical

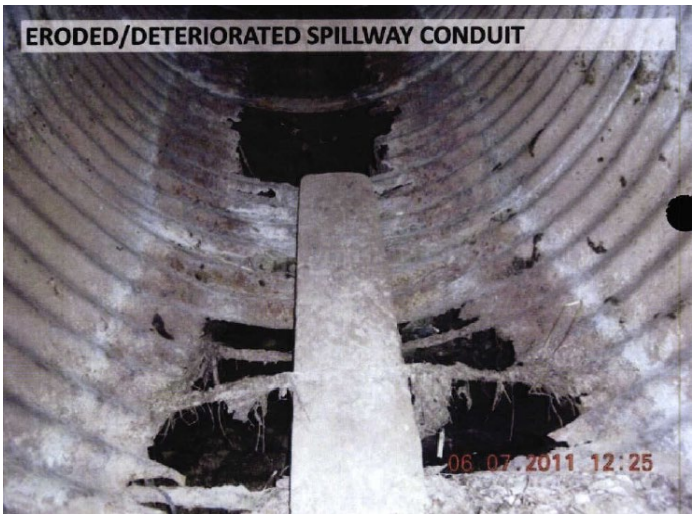
Mechanically operated features like gates and valves within a dam/reservoir system can also be evaluated for performance. Gates and valves should be designed to operate reliably under full reservoir head.

Standard operating procedures may also be warranted where mechanically induced loading can result in stressing the system. Closing gates or valves too rapidly may result in a water hammer where transient pressures spike within closed pipelines that must be accounted for in proper design. Any limitations in operation should be clearly incorporated into the standard operating procedure for the structure, even if they don't warrant a reservoir restriction.

CASE STUDY

Parshall Dam - Damaged Spillway
Intermediate Evaluation Approach
Hydrology Calculations Used to Establish Storage Restriction Level

Hazard: Significant
Dam Height: 20-ft
Impounded Volume: 300 ac-ft
Spillway Configuration: Corrugated Metal Pipe Culvert Through Embankment



Parshall Dam is a significant hazard dam with a corrugated metal pipe culvert spillway laid on a steep slope within the embankment. A routine inspection revealed that the pipe had deteriorated to the point that spillway flows were exposed to the surrounding soil and had eroded a scour hole within the embankment. In its damaged state, the spillway was deemed unsafe to pass any flow.

The prescriptive inflow design flood (IDF) for the significant hazard dam was the flood generated by the 50% Probable Maximum Precipitation (PMP) event. A hydrology study was performed by the regulator which estimated the runoff response and potential hydrologic loading of the reservoir to develop an hydrologic loading curve. The modeling was performed for various events from the 1/100 annual exceedance probability (AEP) event up to the 50% PMP event. The starting water surface in the reservoir was varied in the hydrology model for each event modeled to determine what level event would activate the spillway at various starting water levels. The reservoir did not have sufficient capacity to route the prescriptive IDF without activating the spillway, even with zero storage. However, it did have capacity to capture the 1/100 annual exceedance probability rainfall event if the reservoir was held to a maximum pool fo 3-ft below the spillway. A restriction was set at that level so the dam could retain the volume of the 1/100 flood without initiating flow in the spillway which was deemed adequate given the dam owner's due diligence to address the problem quickly.

Within 4 years, the dam owner was able to hire an engineer to design a replacement spillway, a new spillway was constructed, and the storage restriction was lifted. The CMP culvert spillway was completely removed from the embankment and replaced with an open channel spillway located on the abutment, a safe distance away from the dam, restoring the flood routing capacity of the dam and reservoir to route the prescriptive IDF and eliminating a potential failure mode of erosion of the embankment during spillway activation.

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CASE STUDY

Stovepipe Dam
Intermediate Evaluation Approach
Maximum Depth of Flow Allowed in Spillway

Dam Height: 100 ft
Impounded Volume: 20,000 Acre-ft
Hazard Class: High

This large earthen dam impounds approximately 20,000 AF at normal full pool. This high-hazard dam is owned by a public agency with dedicated professional engineers on staff, but a water users association is responsible for day-to-day operations. Failure would inundate farms/ranches, roads/interstate, and small towns (populations of approximately 2,000 people), with the flood wave reaching the towns in 2 – 4 hours.

When originally constructed during the New Deal era, it was completed with an unlined spillway excavated into weathered gneiss in the left abutment. After being damaged upon activation, a concrete lining was installed in the mid-1940s; this same spillway is still in service and has reached or exceeded its design life.

The spillway has suffered from moderate concrete delamination and spalling, joint deterioration, and exposed water stops. Regular maintenance has included concrete repairs and joint sealing. For approximately 20 years, the owner has made an operational recommendation to minimize spillway flows to avoid further damage.

About five years ago, inspection by the owner's engineers and, subsequently, a specialty concrete firm revealed much more extensive deficiencies in the spillway – including joint faulting and extensive sub-slab voids – than was previously understood. The owner and regulatory agency immediately began working together, and the owner's engineers developed an operation plan to minimize depth of flow over the spillway.

The owner's engineer reviewed records of historical performance and concluded that minimal damage to the concrete would be expected at flows less than 100 CFS, corresponding to 0.5 ft of water depth over the ogee crest. Previous performance and engineering analyses indicate that the outlet works have a maximum operational discharge rate of 350 CFS that is sustainable on a continual basis. While the outlet's maximum capacity is approximately 700 CFS, cavitation damage is expected if flows exceed 350 CFS. The combined project capacity in this scenario is therefore 100 CFS over the spillway + 350 CFS through the outlet = 450 CFS combined.

The reservoir level restriction is based on passing the 1% annual exceedance chance (100-year) inflow hydrograph safely, without damage to the concrete spillway. In order to pass the 1% event under these criteria, the reservoir level was restricted to 4 ft below the spillway crest, equating to more than 3,000 AF of storage loss.

The owner and regulator agreed that more aggressive restrictions were not warranted because:

- The project has a relatively large outlet capacity (up to 700 CFS, though damage is expected above 350 CFS).
- The spillway is founded on Precambrian gneiss that is believed to be generally competent. Although further investigations would be needed, it is believed that, in the event the spillway fails, the rock foundation would be effective at delaying upstream erosion and preventing uncontrolled release of stored water.
- Planning efforts are underway to repair or replace the spillway, so the period of risk exposure is limited.

The pool level restriction may not affect water deliveries in most years. The usable capacity is reduced significantly, but it is still slightly more than the total volume of water marketed/delivered from the reservoir in a year. The restriction requires considerably more active management from the dam operator (to adjust releases and avoid entering the restricted pool) and may impact carryover storage from year to year. The modeling to support this restriction also assumed that the low level outlet gate is not operated to augment spillway flows during the event.

In this example, it is important to note that the dam owner, under the guidance of licensed engineers, proactively developed a reservoir level restriction and proposed it to the regulatory agency for review and approval.

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CASE STUDY

Buck Brook Dam
Intermediate Evaluation Approach
Seepage, Hydraulic & Structural Evaluation

Dam Height: 55-ft
Reservoir Volume: 1,000 Acre-ft
Hazard Class: High Hazard

Buck Brook dam was constructed in the early 1960s in a narrow mountain valley consisting of glacial till deposits. In the early 1980s, a hydrology study indicated it's spillway was inadequate to pass the prescriptive Inflow Design Flood (IDF). In order to pass the IDF, an overlay on the downstream face of the dam was proposed with roller compacted concrete (RCC). The RCC protection was installed butting up to the dam's service spillway, constructed of conventionally cast in place reinforced concrete chute.

Inspections of the service spillway since its construction, showed a steady decline in its condition. Issues observed include ASR/AAR, general concrete deterioration, spalling and delamination, hollow areas observed when sounding with a rock hammer, wall deflections, offset construction joints, and seepage emanating from construction joints. In the mid-2000s, seepage through construction joints near the bottom of the spillway were to exhibiting artesian pressure. This pressure has appeared to increase in the past two decades.

In 2014, the owner performed a semi-quantitative risk assessment (SQRA) on their entire portfolio of high- and significant-hazard dams. At that time, failure of the spillway was deemed to pose a moderate risk. This evaluation was purely qualitative as they did not have the budget/scope to perform structural calculations to support their conclusions. The SQRA did identify that the low-level outlet works, consisting of gasketed reinforced concrete pipe, posed a high risk of failure due to potential for internal erosion to carry embankment material into the pipe. The low-level outlet works became the focus of improvements at the structure based on this conclusion.

Since the SQRA, regular inspections of the dam revealed continued deterioration of the service spillway chute. Given the ongoing concerns the regulator had about the spillway stability, particularly following the Oroville incident in 2017, an issue-specific potential failure modes analysis was performed by the regulator in 2020 to address the structural stability of the service spillway chute. Two failure modes were evaluated: floatation of the spillway base slab under no-flow condition, and slab-jacking of the spillway base slab during a flow event. To support the risk analysis, a seepage model was created and calibrated based on observed seepage conditions in the spillway. This allowed for structural flotation calculations to be performed which revealed a nominally acceptable factor of safety. For the slab jacking potential failure mode, hydraulic calculations were performed to estimate the stagnation pressure to estimate the likelihood of slab jacking initiation during a flow event. The likelihood of each failure mode was evaluated to be moderate, but only with medium confidence. The analysis made recommendations for decreasing likelihood, including a storage restriction, and also recommendations to increase confidence. Neither were implemented immediately due to the risk plotting in an acceptable range, and allowing the dam owner to develop a plan of action.

In 2021-22, based on the prioritization of the SQRA, an outlet works rehabilitation project was performed which installed a CIPP Liner, and a new intake structure/gate. This project required draining the reservoir to provide access for construction. Upon first filling in the spring of 2023, seepage emanating around outlet conduit penetration through RCC overtoppoing protection was observed. Seepage pressures under the spillway base slab also appeared to increase with the 'fountains' increasing several inches indicating additional pressure under the slab.

Based on the increase in pore pressure under service spillway slabs, and the analysis performed in the 2020 study, the factor of safety against flotation was deemed to be to reduced below tolerable limits and therefore increase the likelihood of the failure mode to 'high' requiring immediate action. Inspections were performed as the reservoir was drawn down to determine the reservoir level where pressure under the spillway was reduced. This elevation was utilized to institute a formal storage restriction.

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CASE STUDY

Buck Brook Dam
Intermediate Evaluation Approach
Seepage, Hydraulic & Structural Evaluation





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V. RISK-BASED APPROACH

Risk-based analysis can also be used as a basis for reservoir restriction. As a prerequisite for this approach, regulators must have a thorough understanding of risk and the agency policy makers must establish what level of risk is tolerable to the general public. The details of risk analysis are beyond the scope of this document. For more details, please refer to sources such as the 2019 U.S. Bureau of Reclamation (USBR) and U.S. Army Corps of Engineers (USACE) Best Practices in Dam and Levee Safety Risk Analysis [7], and the U.K. Environment Agency 2017 Guide to Drawdown Capacity for Reservoir Safety and Emergency Planning [5] Section 8.3.

In general, the risk-based approach involves identifying a potential failure mode (PFM) of concern, estimating the breach flood, and the likely consequences that would be realized. The risk-based approach requires describing the sequence of events that must occur leading to failure of the dam and the resulting consequences. This is commonly referred to as a PFM "Event Tree." A thorough understanding of each 'node' in the event tree and a comprehensive understanding of the site conditions is required in order to understand the range and likelihood of behavior that can be expected with any level of confidence.

At its most basic level, risk analysis involves estimating the likelihood of the failure mode resulting in the defined consequences. Estimating likelihood can be as simple as an overall estimation of the likelihood of the overall event (qualitative), or can be broken down into estimated individual nodal probabilities (semi-quantitative) and may even use variable site conditions and material properties to estimate nodal probabilities (fully quantitative). The more advanced levels of analysis of semi-quantitative and fully quantitative are generally more amenable to setting reservoir storage restrictions due to the ability to quantify/estimate the relative impact that each node has on the overall probability of the PFM leading to consequences in an objective manner.

Consequences could be relatively benign such as failure to perform as intended or could be as catastrophic as loss of life. From a regulatory perspective, the lower consequences of failure to perform as intended is not likely to result in a reservoir storage restriction since the consequences are only realized by the dam owner. However, this may lead a dam owner to self-impose a restriction on their facility as it would prevent them from fulfilling their core mission (water delivery in the case of a water utility, recreation in the case of park owners, etc.). Regulatory restrictions of reservoirs typically are limited to failure modes that would result in consequences to the general public which result from an uncontrolled release of impounded water. Note that this includes events that may not lead to loss of life.

In the risk-based approach, a reservoir restriction is warranted if the calculated risk posed by the reservoir at full pool elevation exceeds the established tolerable limit for the regulatory agency or dam owner. Risk analysis can be used to evaluate several approaches to reduce that risk including reduced loading by a reservoir restriction, increasing the likelihood of detection or intervention by requiring more frequent monitoring, and reduced consequences through a reduced breach flood (due to reservoir storage restriction) or early warning system program which can lead to more advance warning to evacuate the population at risk.

The benefit of using semi-quantitative risk analysis is that individual nodal probabilities can be adjusted to reflect the increasing or decreasing likelihood due to the proposed intervention until the overall risk is reduced to a tolerable level. That can include the reduced probability of occurrence of any nodes in the event tree, reduced consequences, or most likely a combination of both. To establish a reservoir restriction, interventions or mitigations are added until the overall risk is reduced to a tolerable level.

Without a thorough understanding of the site, structure, and downstream conditions, the risk-based approach generally requires conservative assumptions of nodal probability estimates. These assumptions should be documented and incorporated into a confidence statement about the overall likelihood estimate. If the risk posed by the PFM exceeds tolerable limits and the confidence is high, then intervention and/or reservoir restriction is warranted. For similar instances where confidence is low, intervention and/or reservoir restriction may still be

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warranted, but other investigations and/or analyses can be initiated concurrently to increase understanding of the site or loading conditions which will improve confidence in the risk assessment. This additional analysis and/or data acquisition should be incorporated into a re-evaluation of the likelihood of the potential failure mode and can result in refinement or elimination of the intervention and/or reservoir restriction. Data collection and analysis can be incorporated into a compliance plan providing the owner with a clear path forward to address the concern as they work toward reduction of the risk and removal of the reservoir restriction.

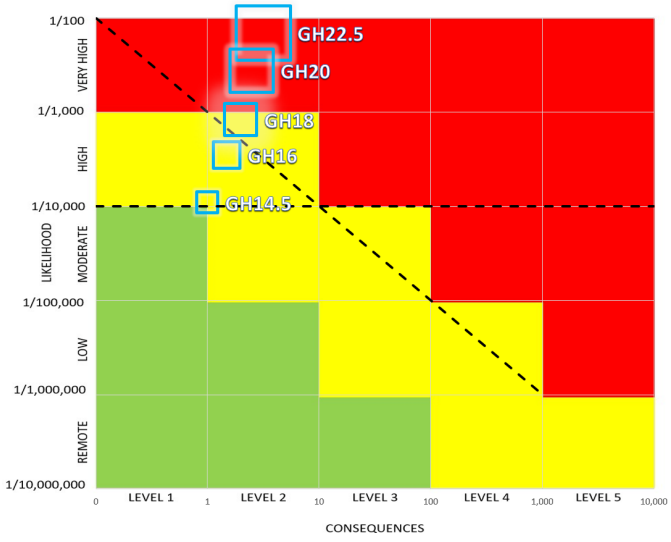
CASE STUDY

Lion Pool Dam - Seepage Incident *Risk-Based Approach* Seepage Gradient, Flow Velocity and Consequence Reduction to Establish Storage Restriction

Dam Height: 25-ft
Reservoir Volume: 2,400-AF
Hazard Class: High
Dam Type: Homogeneous Rolled Earth
Foundation:Soil

A recent inspection of this dam revealed a seepage entry pont on the upstream slope of the dam at approximately mid-height (GH 14.5). Following the inspection, an issue-specific PFMA was performed to evaluate the likelihood and consequences of failure. The PFMA was informed by evaluation of loading probability, likelihood of contact erosion by comparing seepage gradients and velocities to embankment/foundation soil information, and estimated life loss for various levels of storage allowed. The PFMA was performed with the dam owner present to inform them of the factors being weighed into the likelihood and consequences. Through evaluation of various reduced reservoir levels, the hydraulic load was reduced, decreasing the seepage gradient successively until velocities were reduced below the threshold where erosion would continue. The reduced storage volume also reduced the severity of a hypothetical breach flood, thereby reducing consequences as well. The result was that a tolerable level of risk was calculated for an 8-ft storage restriction.

A plot of the f-n chart for this PFM is presented below showing reduced risk and consequences as the reservoir pool is lowered. The box size also represents some level of uncertainty that exists at higher pool levels compared to reduced (restricted) pools.



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VI. OTHER CONSIDERATIONS

Utilizing engineering calculations or observations to establish a safe storage level is sometimes the easiest part of managing reservoir storage restrictions. Other non-physical factors often play into the decision as well, including public communication, organizational and human factors.

Dams with higher risk/consequences always demand more rigorous analysis and documentation to support the restricted level. The higher the level of scrutiny that the restriction will receive, the stronger the case should be made. Many times, decisions must be made despite a dearth of information. In these cases, reasonable, yet conservative assumptions should be made and documented. Detailing the basis for the assumptions used in the analysis supporting the restriction level provides the opportunity for additional data to be collected to increase confidence in the decision. This may result in a basis for reducing or increasing the restriction. If the dam owner collects new information, that data and analysis should be provided to the regulator who can review it when considering whether it warrants a revision to the storage restriction. In these cases, the regulator is justified in making conservative assumptions in the interest of public safety. The burden of proof to provide additional data or analysis should fall on the dam owner. The same level of rigor should apply to high-profile dams/reservoirs, particularly if they provide great value to the community in the form of water supply or economic benefit.

In some high-profile cases, a public information campaign may be warranted. This could be as simple as setting up a briefing with community leaders to communicate the dam safety concerns and describe the measures being taken to address those concerns. It could also be very complex and include setting up informational websites where the concerned public can go for information. These cases should necessarily include informing dam safety organizational leadership and potentially the agency public information officer so they can provide accurate information in response to inquiries from the press and general public.

For contentious restriction orders, it may be beneficial for the regulator to hire a third-party engineer to perform an independent review of the supporting analysis used to justify the restriction. An independent review could lend credibility to the restriction by providing an objective opinion on the justification of the assumptions, validity of the analysis/results, and reasonableness of the conclusions of the regulator.

In addition to loading conditions and consequences commonly identified in dam safety regulation, site accessibility during extreme loading events should be considered when analyzing the overall risk and establishing a storage restriction. During extreme flood events, dams must operate near the limits of their design. The level of event that the dam is designed for often exceed thresholds of other adjacent infrastructure including roads and bridges used to access the structure. When this is the case, the dam should be designed to function assuming that operating staff and equipment cannot mobilize to the site to operate gates or otherwise intervene. Critical passages include bridges, roads along rivers or through canyons, unpaved or unimproved roads, roads below steep slopes prone to mudslides or rockfall, and roads in forested areas should all be considered. Dams with single access roads are more vulnerable than dams with multiple access roads on safer ground. These access issues should be considered for dams that are not staffed continuously for monitoring or operations as they may greatly reduce the probability of incident detection or intervention during an emergency. All modes of access including helicopter may be required during an emergency. A frank assessment of that possibility should be evaluated when considering a restriction.

Mobility around the dam site should also be considered. Extreme rainfall may make maintenance access roads impassable due to mud. All-weather access is needed at or near critical areas of the dam including the: crest, toe, low spots along the reservoir perimeter, outlets, drains, spillway, and powerhouse. A dam should always be accessible with heavy equipment at any time, especially under extreme weather conditions. If a dam is regularly inaccessible during a certain season (closed mountain roads in winter) the reservoir operation should be adjusted accordingly.

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In higher elevations, snowpack may influence reservoir accessibility. If reservoir storage during spring runoff is a loading condition that poses an unacceptable risk, dam owners should work with land managers and road maintenance crews to ensure access roads are cleared of snow sufficiently as early in the spring as needed. Alternately, tracked vehicles such as snowcats or snowmobiles should be available to provide access to monitor and operate the structure. Heavy equipment and material stockpiles may need to be stored on-site for use during an emergency.

CASE STUDY

Spoon Dam
Other Considerations
Zero Storage → Breach Order

Dam Height: 135-ft
Impounded Volume: 35,000 Acre-ft
Hazard Class: High
Tributary Area: 650 sq-mi
Dam Tyhpe: Concrete Faced Rockfill

Spoon dam was a concrete-faced rockfill dam constructed in the early 1900s on the main stem of the Spoon River. Unfortunately, this intermittent stream never produced runoff regularly enough to serve as reliable irrigation water for the farmland below. The structure fell into disrepair due to lack of beneficial use and lack of productivity of the farmland it was intended to serve. Occasionally, the dam would capture runoff from large storms, but those storms also produced a significant sediment load which began to reduce the active storage the reservoir could impound, further diminishing its value.

The dam had settlement issues due to poor foundation preparation/shaping of the steep canyon walls and poor compaction of the rockfill which resulted in cracking of the upstream facing, the dam's impermeable barrier. The dam nearly failed in the late 1980s when a large flood filled the reservoir inducing seepage through the cracked upstream face. Total seepage was estimated to be on the order of 100 cfs, and it nearly failed the embankment due to internal erosion. Complete dam failure was averted by the quick lowering of the spillway channel which allowed rapid reduction in the reservoir level. A storage restriction was issued shortly thereafter limiting the reservoir to gage height 100-ft citing an inadequate spillway, deteriorated upstream concrete facing and poor condition of the outlet works.

No further action was taken until 2012, when the spillway was lowered significantly, but other issues remained including the deteriorated upstream facing and poor condition of the outlet works. Due to lack of action on these issues, the storage restriction was revised to a zero storage restriction in 2014. As part of the zero storage restriction, a compliance plan was implemented between the regulator and dam owner requiring remedial actions on a schedule toward addressing the safety issues. Despite the spillway lowering, given the large tributary basin and significant sedimentation in the reservoir pool, the dam still had insufficient capacity to safely route the Inflow Design Flood (IDF).

Given the difficulties surrounding the structure, particularly the flashy and sporadic nature of the tributary basin and its unreliable production of water for irrigation, and the expensive repairs required to remove the storage restriction and regain full storage, the structure ownership changed. The new owner did not meet the requirements of the compliance plan which led to the regulator issuing a breach order requiring removal of the dam.

In 2018, a wildfire burned approximately 12-percent of the tributary basin. The potential for more runoff from a partially burned basin pushed the risk posed by the structure to intolerable levels, and emergency action was deemed necessary. State statutes allowed the regulator to take control of a dam and reservoir under emergency conditions and that authority was exercised in this case. The regulator designed an engineered breach and emergency funds were utilized to execute a contract with a local contractor to breach the dam.

The ability to rapidly reduce the storage in the reservoir is another consideration when establishing a storage restriction. The "Guide to Drawdown Capacity for Reservoir Safety and Emergency Planning [5] describes this requirement in detail. The outlet works must be robust enough and maintained well enough to ensure that this drawdown capacity is available under the most extreme loading conditions. Otherwise, failure of the outlet works

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during emergency drawdown should be evaluated as a potential failure mode. Reclamation document ACER TM-3 [8] also provides recommendations for reservoir evacuation capability.

Predictability of the critical loading that initiates a failure mode of concern should also be taken into consideration. For the cases of seismic loading, there is little to no chance of advance warning of the loading. Hydrologic loading may not provide much warning either if the basin is small and the critical loading event is a small convective thunderstorm. Dams with larger basins fed by snowmelt may have significantly more warning as the snowpack in the basin can be monitored and runoff is more predictable.

The loss of utilities could lead to severe problems during an emergency. Power outages are common during flood events through inundated power stations or damaged overhead lines. With a loss of power the electric-driven gates, valves, and computer-controlled equipment may not be operable. These critical functions should include redundant emergency backup power and/or manual override options.

Dam owner disposition should also be considered when establishing a storage restriction. Most dam owners are conscientious and recognize their responsibility for the safety of the downstream public. But, unfortunately, there are some that take less care and view regulators as bureaucrats standing in the way of their business. In the case of the latter, extra care is warranted in educating the owner and their staff about the reason behind the restriction and their liability should something happen. In extreme cases, where the owner is not expected to be responsive, a more punitive restriction may be necessary until they demonstrate that the dam safety deficiencies are being taken seriously and they have implemented a plan to address them.

Zero storage restrictions should be issued when the dam is deemed unsafe for any storage. A zero storage restriction typically requires the owner to maintain the low level outlet gate open. But zero storage does not mean zero risk. There is always the possibility that a hydrologic even could be captured by the reservoir which would load the structure. This possibility should be considered when considering a zero storage restriction and whether a breach order is warranted.

Breach orders are a last resort. They are generally reserved for projects that are no longer viable for the owner financially. Unfortunately, breaching a structure is a complicated process and not without cost. If the owner does not have the financial resources to design and construct a breach, or the will to expend those resources on a project that will have no financial return, other means of compelling compliance with the breach order may be necessary. In some cases, court orders and financial penalties may be needed. State statutes that govern these processes vary dramatically from state to state. Regulators should work closely with the Attorney General of their state to determine what possible courses of action is available to them. These actions should be supported with the types of engineering analysis described herein.

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VII. RESTRICTION REVISION/REMOVAL

When a regulatory authority or dam owner imposes a reservoir restriction to address unsafe conditions, it is a best practice to clearly identify what conditions must be met to lift or revise the restriction. It is generally appropriate to lift or revise a reservoir restriction after sufficient physical or operational interventions take place, or as new data is collected or analysis is performed to demonstrate that the dam will perform safely under the expected loading conditions. This can allow the reservoir restrictions to be reconsidered. If unsafe conditions no longer exist and the project meets current design and safety standards, the reservoir restriction is no longer warranted. If unsafe conditions are partially addressed through the implementation of mitigation or re-analysis with better data or more sophisticated techniques, the reservoir restriction may be reassessed under the new conditions using the methods discussed herein.

Physical interventions such as repairs or modifications to a dam may warrant reconsideration or removal of a reservoir restriction. For example, the reservoir at the FERC-regulated Swinging Bridge Dam in New York was drawn down in 2005 following the discovery of a sinkhole. After investigating the cause of the sinkhole, physical interventions included addressing the sinkhole-initiating water leakage at the penstock through the dam by filing the penstock with concrete and repairing the sinkhole and the damaged embankment. Following this repair, the reservoir restriction was lifted in July 2007.

Operational interventions are generally measures that reduce the time to detect potentially hazardous conditions paired with a plan for preventative or mitigating actions. Operational intervention is often less expensive than physical intervention. However, it is not always possible for operational interventions alone to address the unsafe conditions triggering a reservoir restriction. Appropriate intervention measures should be selected based on the project, the identified unsafe conditions, the consequences of failure, and engineering judgment.

Detection time may be decreased by additional instrumentation and monitoring designed to provide early warning. Such additions may include more frequent visual inspections, added instrumentation such as those indicated in the FERC Engineering Guidelines Chapter 9 [4], provided in the table below, and regular review of available forecasting and flow monitoring resources. Any additional instrumentation should be selected to provide information useful and relevant to detection of potential hazards at a project.

Operational intervention may be directed in a standard operating plan (SOP) for reservoir drawdown or other remediation measures following “triggering” observations related to potential unsafe conditions. Following detection of an unsafe condition, emergency intervention techniques and appropriate emergency response can stop or minimize the consequences of a dam failure [3].

Along with reservoir restriction, common emergency interventions may include temporary measures to increase freeboard, mitigate active seepage, prevent or redirect erosion, and increase outflow capacity [2]. Operational intervention to address a reservoir restriction may include advanced preparation and planning for these emergency measures such as improving downstream notification and stockpiling tools and materials such as sand or gravel in an accessible location.

Outside of emergency response, most issues that warrant a reservoir restriction are slowly developing. This may warrant developing a path toward compliance with deadlines and consequences. This ‘compliance plan’ approach can be utilized to provide the dam owner with clear objectives, milestones and time to hire an engineer, collect additional information, perform analysis, design mitigation, and construct improvements.

Refilling a reservoir after an extended restriction should be performed in a cautious and measured manner. In some cases, refiling will load new or modified features for the first time. Prolonged operation at less than full level may also result in changes to the system which must be evaluated for proper performance once re-loaded. A first

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filling and monitoring plan should be developed by an engineer to identify the frequency of monitoring and visual inspection, and possibly hold points to evaluate performance when a reservoir restriction is lifted.

TYPICAL INSTRUMENTATION AND MONITORING USED IN EVALUATING CAUSES OF COMMON PROBLEMS/CONCERNS [1]

PROBLEM/CONCERN	TYPICAL INSTRUMENTATION
Seepage or leakage	Visual observation, weirs, flowmeters, flumes, calibrated containers, observation wells, piezometers
Boils or piping	Visual observation, piezometers, weirs
Uplift pressure, pore pressure, or phreatic surface	Visual observation, observation wells, piezometers
Drain function or adequacy	Visual observation, pressure and flow measurements, piezometers
Erosion, scour, or sedimentation	Visual observation, sounding, underwater inspection, photogrametric survey
Dissolution of foundation strata	Water quality tests
Total or surface movement (translation, rotation)	Visual observation, precise position and level surveys, plumb measurements, tiltmeters
Internal movement or deformation in embankments	Settlement plates, cross-arm devices, fluid leveling devices, pneumatic settlement sensors, vibrating wire settlement sensor, mechanical and electrical sounding devices, inclinometers, extensometers, shear strips
Internal movement or deformation in concrete structures	Plumblines, tiltmeters, inclinometers, extensometers, jointmeters, calibrated tapes
Foundation or abutment movement	Visual observation, precise surveys, inclinometers, extensometers, piezometers
Poor quality rock foundation or abutment	Visual observation, pressure and flow measurements, piezometers, precise surveys, extensometers, inclinometers
Slope stability	Visual observation, precise surveys, inclinometers, extensometers, observation wells, piezometers, shear strips
Joint or crack movement	Crack meters, reference points, plaster or grout patches
Stresses or strains	Earth pressure cells, stress meters, strain meters, overcoring
Seismic loading	Accelerographs
Relaxation of post-tension anchors	Jacking tests, load cells, extensometers, fiber-optic cables
Concrete deterioration	Visual observation, loss of section survey, laboratory and petrographic analyses
Concrete growth	Visual observation, precise position and level surveys, plumb measurements, tiltmeters, plumblines, inclinometers, extensometers, jointmeters, calibrated tapes, petrographic analyses
Steel deterioration	Visual observation, sonic thickness measurements, test coupons

[1] Appropriate remedial measures should be taken for all problems and concerns. Possible remedial measures for a wide variety of problems and concerns are discussed in EPRI (1986), National Research Council (1983), ASCE (1975 and 1988) and USACE (1986a).

Table 1: FERC Recommended Instrumentation & Monitoring for Evaluating Dam Problems/Concerns

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CASE STUDY

Plains Dam
Restriction Revision & Removal
Refill Monitoring Plan

Dam Height: 35-ft
Impounded Volume: 24,000 Acre-ft
Hazard Class: High

Plains Dam and Reservoir is a large irrigation reservoir serving a significant amount of farmland. The geology in the area consists of deep deposits of Eolian sands with relatively low fines content. The dam was constructed with these sandy soils and included a reinforced concrete facing system which served as the dam's impermeable barrier.

The reservoir has had multiple issues over the past half-century including seepage, a spillway that was too small to pass the prescriptive inflow design flood (IDF), and erosion of the dam crest and downstream slope due to wave action exacerbated by the smooth upstream dam facing. Acknowledging these problems, the dam owner has operated the structure under a self-imposed storage restriction since the mid-1960s. A formal storage restriction was issued by the regulator in 1990.

Over the past 15 years, the owner has been progressively addressing the issues identified in the storage restriction. In 2007, a new spillway was completed to pass the prescriptive IDF. And in 2022, both a wave breaker system was installed on the upstream concrete face and a filtered toe drain was installed at the location of the most troublesome seepage.

Acceptance of the 2022 construction and removal of the storage restriction was issued in early 2023 under the condition that reservoir filling be closely monitored to assess the performance of the structure under the increased hydraulic load which the dam has not experienced in nearly 60 years. The owner's engineer prepared the fill and monitoring plan which was reviewed and approved by the regulator. The fill plan requires fill rates to be controlled and reservoir level hold points where dam performance can be evaluated. It also delineates frequent visual observations and monitoring of instrumentation along with observing the performance of the wave break system after wind events. Monitoring data is collected by the owner, evaluated by their engineer, and transmitted to the regulator for review.

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VIII. CONCLUSION

The authority to impose reservoir restrictions is a powerful tool in the regulatory toolbox to maintain the safety of the public. While regulators can and should consider the loss of resources that the dam owner will realize due to a storage restriction, they must hold public safety paramount. Utilizing well-documented analysis, appropriate data, industry-standard practices, and engineering judgment in exercising this authority is required to maintain the trust that has been placed in regulators by both the general public and dam owners.

The various tiers of analysis described herein should provide regulators with several defensible approaches to establish the safe storage level based on the conditions present at a dam. That data and analysis can be provided to dam owners, the public, or policy makers to explain why the storage restriction is warranted in a transparent manner. It can also establish the basis for a path toward lifting the restriction that the dam owner can follow to mitigate the safety concerns and re-establish their ability to use the reservoir to its full potential, once all safety issues have been addressed.

Reservoir storage is clearly required and beneficial for a properly functioning society. The proper implementation of reservoir restrictions and the subsequent improvements to remove those restrictions can help us work toward a future where all dams are safe.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

The Dam Safety Toolbox – A New Online Resource for the Dam Safety Industry from ASDSO

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ABSTRACT

The Association of State Dam Safety Officials (ASDSO) is excited to share the new online Dam Safety Toolbox (DamToolbox.org)! This wiki-style website allows users to easily access current, state-of-the-practice guidance and resources on all aspects of dam engineering. The goals of the website are to:1. Advance the industry toward the use of standard practices, 2. Serve as a resource to educate and support dam safety professionals including regulators, consultants, and owners, 3. Serve as an authoritative reference source for regulators when providing comments on technical aspects of designs to consulting engineers,4. Serve as a resource for expert consultants who are submitting designs to regulators who may not have experience with the issue being addressed,5. Aid dam owners in understanding the basics of dam safety and identifying qualified engineers, and6. Serve as a mechanism for states to share tools and technical information with each other. This presentation will provide an overview of the development, format, and functionality of the Dam Safety Toolbox website. Opportunities to contribute to the website and get involved in the long-term expansion and maintenance of this exciting resource will also be shared.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

Narrows Bypass Channel CFD Analysis – Modeling Challenges for Large CFD Models with Large Flows and Interpretation of Their 3-D Results

AHINTHA KANDAMBY, PH.D., P.E., CFM, WSP

ABSTRACT

The Narrows Dam is located on the Yadkin River in Badin, North Carolina. The dam consists of a main dam section and a bypass spillway section. The dam's bypass spillway structure is located on the left side of the dam when looking downstream and consists of ten (10) Stoney gates. Each gate is 28 feet high and 35 feet wide. The bypass spillway structure is approximately 430 feet long and has a concrete crest elevation of 480.84 feet NAVD 1988 (513.0 Yadkin datum). The bypass spillway chute (channel) is approximately 1,200 feet long and approximately 103 feet wide at its narrowest point. The Probable Maximum Flood (PMF) for Narrows Dam is 414,500 cfs, which includes the capacity of the bypass spillway (about 242,455 cfs) and main dam (about 172,045 cfs). The reservoir water surface elevation during the PMF event is 517.64 feet NAVD 1988 (549.8 feet Yadkin datum). The normal pool level is 508.94 feet NAVD 1988 (541.1 Yadkin datum).Following a high flow event occurred between November 13-15th in 2020, that the bypass channel experienced about 118,000 cfs, operations staff performed an inspection in accordance with the Narrows Dam Safety Surveillance and Monitoring Plan and noticed a few rocks on top of the east wall of the bypass chute located at the downstream end of the rock backfill section. Recognizing the need to investigate further to ensure the structural integrity, CFD modeling was proposed to reevaluate the capacity of the spillway and evaluate the turbulent dynamic pressure forces on the spillway chute training walls and highly turbulent plunging pool.The discussion highlights the essential steps needed to develop a complex skewed spillway chute geometry with a large vertical elevation drop between upstream and downstream. The model is further complicated by the necessity to model the full domain with large discharge capacity, gate operational sequence, and high speed thin sheet flows in the chute and associated flow splashing due to bends. A major part of the modeling effort was calibrating the CFD model against a physical model and identifying the model sensitivity to externally applied Manning's Roughness values. The discussion also explains the challenges faced during the model development and how they were resolved using efficient CFD modeling and 3-D meshing techniques using the FLOW-3D software and its numerical tools.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

Solving complicated hydraulics in a old horseshoe outlet tunnel that needs some repairs through CFD analysis.

BRYAN HEINER, BUREAU OF RECLAMATION

ABSTRACT

Many dams throughout the US are currently at or beyond their original design life. Many methods are employed to upkeep and maintain these facility to extend their design life well into the future. One potential area of concern for some of these older dams is the outlet works and tunnels that pass low-medium flow releases into the downstream channel. Repeatedly repairs and upgrades are required to outlet works to mitigate degraded concrete or tunnel linings, sedimentation at the intakes or damage caused from cavitation or abrasion. The paper will focus on a facility that went through major modifications in the 90's that is again suffering from damage in the downstream chute. The problems will be highlighted, followed by the planned actions to remedy the issues. CFD analysis will be presented to show how often complex issues and be solved through simple methods.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

Labyrinth Weir Spillway Design: Lessons Learned on the DMAD Dam Rehabilitation Project

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ABSTRACT

DMAD Reservoir is the penultimate reservoir on the Sevier River, located near the town of Delta, Utah. With a storage capacity of approximately 11,000 acre-ft, it primarily serves as intermediate storage prior to conveying irrigation water to the greater Delta area and cooling water to the Intermountain Power Plant. The 7,270 square-mile drainage basin upstream of the DMAD Dam required the spillway to pass a probable maximum flood (PMF) of 48,300 cfs, a significantly larger flow than that permitted by the old spillway's approximate 18,000 cfs capacity. Insufficient capacity of the old spillway coupled with geotechnical concerns regarding liquefaction of the foundation soils underneath the spillway and continual scour of the immediate downstream spillway channel resulted in the replacement of the DMAD Dam spillway. This rehabilitation work commenced in the summer of 2021 and was completed in the spring of 2022. Various spillway configurations were evaluated to pass the required PMF, including an auxiliary fuse plug of varying layouts and controlled failure alternatives. In order to address the challenges associated with the potential uncontrolled downstream erosion and the need to pass the PMF flows, a new labyrinth weir spillway design was ultimately selected and constructed. The 8-cycle labyrinth spillway increased the effective weir length to 1,200 ft but was contained within a 300 ft span. CFD and 2D river models were produced in order to optimize the spillway configuration. The deep downstream tailwater shown through the modeling process allowed for a significantly reduced stilling basin design, thus saving the project stakeholders cost and time in construction. The earthen channel downstream of the stilling basin was also equipped with a deep riprap cutoff wall to prevent headcut erosion from progressing up to the stilling basin structure.One lesson learned and re-emphasized during the construction of this project was the importance of planning ahead for emergency reservoir control and the implementation of a reservoir management plan. A local 1,000-year 24-hr flood event nearly compromised the project site when the overexcavation was at its deepest levels. Quick action to follow the Reservoir Management Plan procedures and construct an emergency upstream cofferdam ultimately saved the owner and contractor time and money while preventing extensive downstream damages from what could have been an uncontrolled failure of the dam through the project site.

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DAM SAFETY PALM SPRINGS
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Garrison Dam - Iterative CFD Modeling to Evaluate Spillway Overlay Measures with 70 feet of Uplift Resistance

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ABSTRACT

Differential uplift forces on the Garrison Dam concrete spillway lower chute and stilling basin slabs of up to 70 feet are expected during infrequent flood events, including the 1,150,000 cubic feet per second probable maximum flood event (PMF), and exceed existing uplift resistance. Uplift resistance configurations were iteratively developed and included “smooth” and stepped concrete overlays up to 16 feet thick (i.e., mass gravity resisting force) and associated stilling basin modifications. These configuration geometries were unique with atypical entrance and exit transitions, which resulted in uncertain hydraulic performance and adequacy. As such, computational fluid dynamics (CFD) modeling was undertaken to provide an iterative “proof of concept” understanding of the hydraulic adequacy of various modification configurations. The results of CFD simulations provided preliminary confirmation of the potential hydraulic adequacy of the overlay measure to mitigate a potential failure mode associated with the excessive uplift forces. Simulation result adequacy ranged from marginal for a smooth chute overlay without stilling basin modifications to near complete for a stepped chute overlay with stilling basin modifications.

I. INTRODUCTION

Garrison Dam is classified as a high hazard potential dam located in central North Dakota and is one of six U.S. Army Corps of Engineers (USACE) owned and operated mainstem dams on the Missouri River Mainstem Reservoir System. The authorized purposes of the dam are flood risk management, hydroelectric power, water supply, water quality control, irrigation, recreation, navigation, and fish and wildlife. Construction of the dam began in fall of 1947 and was completed in spring of 1954.

Garrison Dam consists of a zoned earthfill dam, outlet works, spillway, and reservoir, as presented in Figure 1. The main embankment is generally aligned east to west and is approximately 12,000 feet long with a maximum height of 210 feet. The dam has a crest width of 60 feet with a design crest elevation of 1,875 feet local project datum (LPD). The outlet works includes an approach channel, intake structure, powerhouse with five power tunnels, three regulating (flood-control) tunnels, a flood tunnel stilling basin, and a tailrace discharge channel.

The approximately 3,000-foot-long spillway is excavated in the left abutment with a design crest elevation of 1,825 feet LPD. The chute-type spillway structure consists of a crest structure (including 28 Tainter gates and a bridge), concrete lined chute, and a stilling basin. The chute transitions from a width of 1,336 feet at the crest structure to 800 feet in the upper spillway chute and maintains the 800-foot width through the lower spillway chute and stilling basin. The upper chute is sloped at 0.75 percent while the lower chute is sloped at 20 percent. The stilling basin is approximately 200 feet long and includes 10-foot-tall baffle blocks and a 12-foot-tall stepped end sill. Figure 2 presents an isometric view of the existing conditions spillway configuration. The original design discharge capacity of spillway was 827,000 cubic feet per second.

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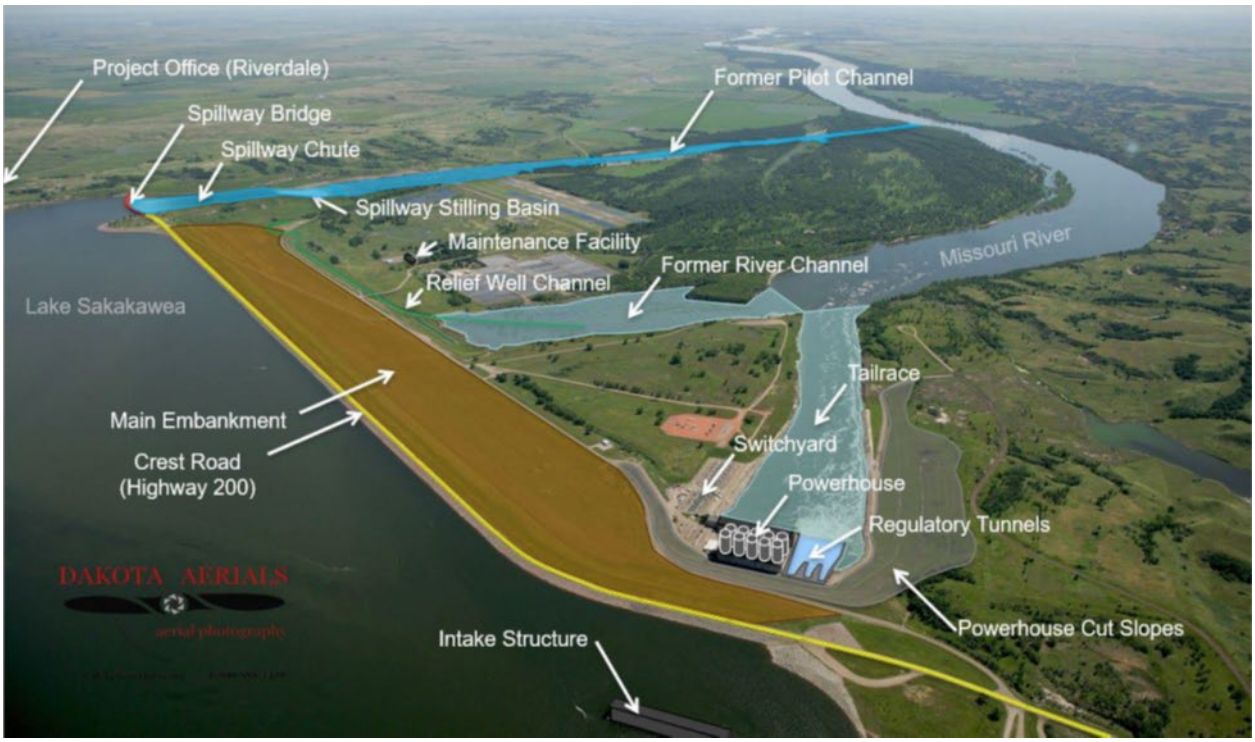


Figure 1. Overview of Garrison Dam and its appurtenant features

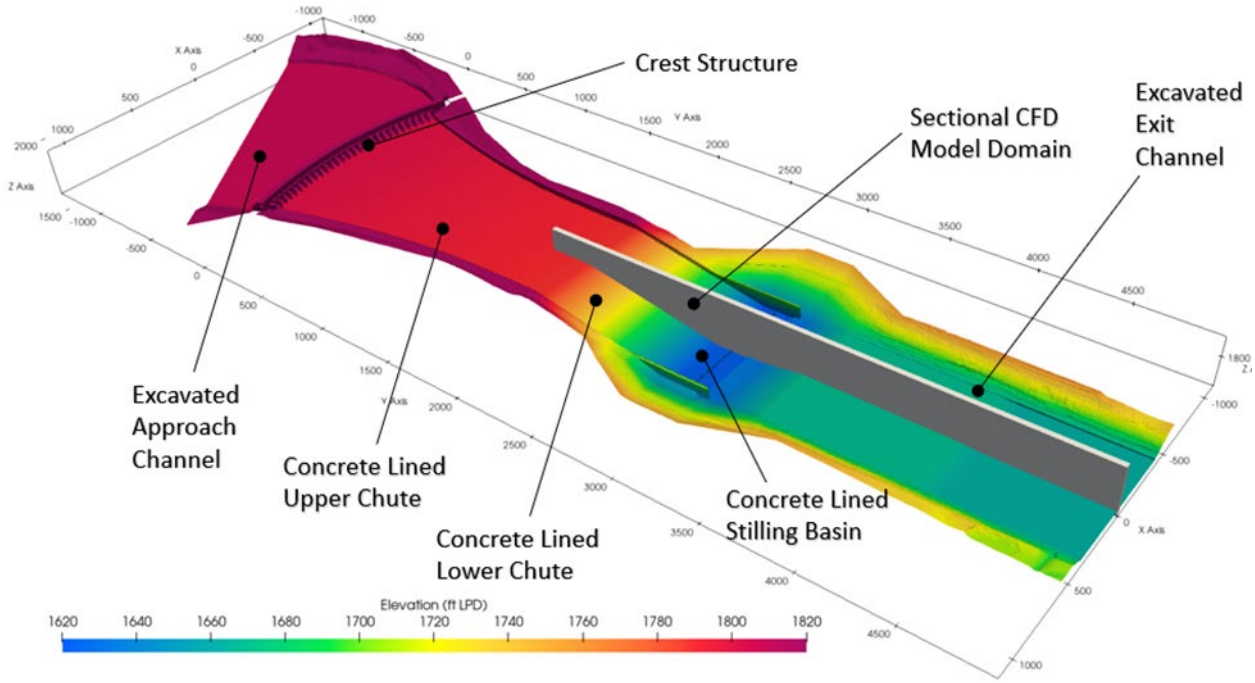


Figure 2. Isometric view of the existing conditions spillway configuration (note shift between CFD and Design Stationing: CFD Station [feet] + 3,850 [feet] = Design Station [feet]).

The Garrison Dam probable maximum flood (PMF) / inflow design flood (IDF) was updated in 2020 as part of an existing conditions risk assessment (ECRA), which was the first update to the Garrison PMF since the release of the 1946 Definite Project Report. The purpose of the 2020 update was to establish an updated PMF for Garrison Dam based on significant amounts of new data, new policy, updated methodologies, and dam safety

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best practices. The spillway discharges associated with the 2020 PMF update pose intolerable life safety risks. As a result of the 2020 reasonable high PMF estimate, dam freeboard decreased from 13.8 feet to 1.4 feet and spillway discharges increased to approximately 1,150,000 cubic feet per second, which is approximately 40 percent greater than the original spillway design discharge of 827,000 cubic feet per second.

Tailwater within the exit channel downstream of the spillway is controlled by the Missouri River. It was recognized during original design that the Missouri River, downstream of the dam, could be subject to significant geomorphic changes and associated scour during the project design life. As such, two downstream channel configurations and associated tailwater rating curves were considered as part of original design evaluations:

i. An initial retrogression configuration representative of conditions shortly after construction completion.
ii. An ultimate retrogression configuration representative of maximum expected channel scour resulting from floods and reservoir releases during the project design life.

The geology proximate to the spillway lower chute and stilling basin is generally characterized by compacted sediments and lignite beds of the Fort Union Group. Bedding is generally horizontal with thicknesses ranging from a few inches to more than 15 feet. Testing indicates that the clay and silt stratum has properties intermediate between a rock and a soil and exhibits characteristics similar to a soft shale lacking cementation. The lignite beds are jointed and cracked with some thicker beds capable of transmitting water and associated pressures due to high permeability. Two such thicker beds are located proximate to the stilling basin and lower chute foundations.

II. RISK CHARACTERIZATION

Garrison Dam currently has a total of six risk-driving potential failure modes (PFMs) that are associated with the spillway. This paper is focused on the intolerable life safety risks associated with spillway failure during infrequent flood events due to excessive uplift forces and subsequent hydraulic jacking potential in the lower chute and stilling basin (PFM 8). Geologic features under the lower chute and stilling basin can transmit the downstream tailwater pressure head to the base of the spillway chute and stilling basin upstream of the hydraulic jump. This would be expected to result in differential uplift pressures of up to approximately 70 feet. Drainage features intended to relieve up to 50 percent of differential uplift pressures do not appear to be reliable due to their age, current poor condition, and potential to be inundated during infrequent flood events. The inundation of drainage features appeared to be likely based on computational fluid dynamics (CFD) modeling completed in support of existing risk characterizations. Without drainage uplift relief, the capacity of remaining uplift resistance elements (slab mass and anchorage) would be exceeded during infrequent flood events with discharges of approximately 400,000 cubic feet per second (i.e., approximately 50 percent of the original spillway design capacity) and greater. This would result in hydraulic jacking, headcutting, and ultimately failure of the spillway control structure. A failure of either the Garrison embankment or spillway during a maximum high pool would result in widespread, long-lasting, and catastrophic consequences impacting 11 states and approximately 2,000 river miles (not including backwater areas up tributaries) extending from North Dakota to Mississippi.

III. RISK MITIGATION MEASURES

Following the identification and risk characterization of the risk-driving failure modes, a dam safety modification study (DSMS) was initiated by the USACE to develop risk mitigation plans (i.e., risk reduction alternatives). Potential PFM 8 risk mitigation measures were brainstormed, evaluated, and ultimately the team selected measures for risk management plan (RMP) development (i.e., conceptual design) to address the intolerable risk associated with the excessive uplift forces. Brainstorming included measures associated with uplift resistance as well as uplift reduction. Uplift resistance measures included the construction of chute and stilling basin overlays (i.e., gravity mass), anchorage, and a combination of both elements. The DSMS schedule was constrained and the opportunity to refine and advance measure development was limited. The combination of required significant uplift resistance and challenging foundation conditions made measures that included anchorage less favorable at this level of design advancement. As such, the only uplift resistance measure selected was associated with a mass overlay without anchors. However, it is anticipated that anchorage will be considered and evaluated as part of future design advancements and refinements.

The uplift profile under the lower chute and stilling basin was not defined in the DSMS. The uplift under the stilling basin slabs was conservatively assumed to be an equal distribution of the full tailwater depth associated with the PMF for ultimate retrogression discharge channel conditions. The uplift under the lower chute slabs was

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assumed to be equal to the PMF full tailwater depth at the entrance to the stilling basin (Sta 66+50) and linearly decrease to zero at the elevation of the tailwater (Sta 62+50). As such, the uplift resistance was only necessary in the lower half of the lower chute and portions of the stilling basin. On this basis, two general overlay configurations were considered:

i. An overlay focused on the entire approximately 800-foot-long lower chute; and
ii. An overlay focused on only the approximately 400-foot lower half of the lower chute.

These overlay configurations posed a design challenge given that an overlay of the entire lower chute would more closely conform with a typical design, but would not be as economical as an overlay focused on only the lower half of the lower chute. However, the hydraulic performance and design of an overlay focused on only the lower half of the lower chute would have greater uncertainty and require greater evaluation. Given the substantial economic differences between these two general overlay configurations and the unnecessary redundancy associated with the overlay of the entire lower chute, the team decided to pursue the overlay focused on the lower half of the lower chute.

IV. HYDRAULIC DESIGN

1) Design Overview

The most significant design challenge associated with an overlay focused on the lower half of the lower chute (lower chute overlay) was establishing a geometric layout that provided the overlay thickness required to resist uplift forces but also provide satisfactory hydraulic performance within the existing stilling basin. Additionally, hydraulic transition considerations into and out of the overlay presented challenges. Pertinent preliminary hydraulic design criteria also included:

i. Limiting potential basin modifications to those associated with baffle blocks and end sill to avoid the potentially significant challenges and complications of global stilling basin modifications (e.g., modification of the existing approximately 100-foot-tall training walls, etc.);
ii. Preventing hydraulic jump sweepout (associated with the revised PMF for ultimate retrogression discharge channel conditions) to avoid potential excessive downstream channel erosion and headcutting concerns; and
iii. Maintaining existing lower chute and stilling basin training walls without modification.

Based on preliminary existing uplift resistance estimates and the previously noted assumed uplift profile, a maximum overlay thickness in the lower chute of about 16 feet was estimated to be required at the entrance to the stilling basin (Sta 66+50). This resulted in significant encroachment into the stilling basin and required a transition from the lower chute overlay to the stilling basin. Based on the CFD modeling completed as part of the ECRA, the existing stilling basin configuration was only marginally capable of containing the hydraulic jump in the basin for the revised PMF discharge. As such, encroachment in the stilling basin had the potential to increase risk associated with a basin sweepout PFM and potential basin encroachment was considered a critical design element to minimize.

Initial formulation of the lower chute overlay configuration included a smooth profile; however, it was recognized that the incoming energy to the existing stilling basin may not be adequately dissipated within the basin due to potential overlay encroachment. As such, a stepped overlay profile was also postulated as an attempt to increase energy losses within the chute such that the existing stilling basin configuration or a basin configuration with relatively minor modifications (i.e., limited to baffle blocks and end sill) would provide adequate hydraulic performance and energy dissipation within the basin. These two general overlay configuration types are further described in subsequent sections.

2) Smooth Overlay Profile

The initial smooth overlay profile configuration included a tie-in to the existing chute slab at Sta 62+50, extension along a constant slope of approximately 15 percent to achieve a 16-foot overlay thickness at Sta 66+50, and continued extension to a tie-in point within the stilling basin at approximately Sta 67+50. The overlay upstream transition location (i.e., proximate to Sta 62+50) was not of great concern at this level of design advancement given the relatively minor differences between the existing lower chute slope (20 percent) and overlay slope (approximately 15 percent). However, it is recognized that this transition, if advanced, will need to be further evaluated as part of future design advancements and refinements. The overlay downstream transition location (i.e., between Sta 66+50 and Sta 67+50) encroached approximately 100 feet into the existing 200-foot-long stilling basin

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and was not anticipated to produce adequate hydraulic performance as it pertains to the preliminary design criteria and risk characterization. As such, a vertical curve was developed to transition discharges from the lower chute overlay to the stilling basin over a shorter distance to reduce encroachment.

The vertical curve was designed based on U.S. Bureau of Reclamation (Reclamation) guidance for a free-discharging jet, which includes consideration of jet support while maintaining positive pressures along the curve (Reclamation, 1987). The recommended curve is a function of geometric and hydraulic conditions entering the curve including the floor slope angle, flow depth, and velocity head. The incoming hydraulic conditions to the downstream transition were estimated based on the existing conditions CFD modeling completed as part of the ECRA and indicated a flow depth of approximately 20 feet and velocity of approximately 70 feet per second. Using this information in combination with the floor slope angle (approximately 15 percent), the vertical curve was estimated and found to encroach approximately 60 feet into the basin. Although encroachment continued to persist, this configuration appeared more likely to provide adequate hydraulic performance than the initial configuration.

Figure 3 presents the adopted lower chute smooth overlay profile with the revised downstream transition. This configuration was carried forward as part of further design advancement and evaluation.

3) Stepped Overlay Profile

Given the uncertainty associated with the hydraulic performance of the smooth profile due to reduced basin length as a result of stilling basin encroachment, a stepped overlay profile was also developed. The intent of the stepped profile was to initiate additional energy losses within the chute such that potential stilling basin encroachment would be less critical to achieving adequate hydraulic performance entirely within the basin. However, maintaining flow support, step engagement, and positive pressures throughout the overlay were of critical importance to the hydraulic performance of the proposed configuration and had to be balanced with the concept of chute energy dissipation.

An initial stepped overlay profile configuration was developed that mimicked the smooth overlay profile with the top of the vertical step faces intersecting the smooth overlay profile. Given the lack of precedence for a stepped configuration with concave entrance and convex exit transitions as well as a unit discharge of approximately 1,400 cubic feet per second per foot, typical industry stepped spillway design guidance and recommendations did not appear to be entirely reliable.

Despite the dearth of reliable guidance for the proposed stepped overlay configuration and associated hydraulics, the Folsom Dam auxiliary spillway design and evaluation completed by the USACE and Reclamation provided a precedence for stepped spillways with similarly large unit discharges (approximately 1,800 cubic feet per second per foot) and entrance velocities (approximately 80 feet per second). As such, the Folsom auxiliary spillway configuration was used as a basis for selecting initial step heights for the stepped overlay for this preliminary evaluation. This included both the entrance transition step height variation as well as the typical step heights. Entrance step heights varied between 0.5 feet and 3.0 feet between Sta 62+50 and Sta 63+55 with typical step heights of 3.0 feet downstream of Sta 63+55.

Several aspects of stepped spillway hydraulic performance (e.g., energy dissipation, cavitation potential, entrained air concentrations and associated flow bulking, etc.) were not explicitly considered and evaluated as part of the development and evaluation of the stepped overlay configuration. However, it is recognized that these aspects will need to be further evaluated as part of future design advancements and refinements.

Figure 6 presents the adopted lower chute stepped overlay profile. This configuration was carried forward as part of further design advancement and evaluation.

4) Stilling Basin Modifications

The stilling basin encroachment associated with both lower chute overlay configuration types was anticipated to also necessitate modifications to the existing stilling basin baffle blocks and end sill to achieve adequate hydraulic performance. The adequacy of the existing stilling basin baffle blocks and end sill were also evaluated against USACE (1992) and Reclamation (1984) guidance using the incoming hydraulics to the stilling basin. The incoming flow depth (approximately 13.0 feet), velocity (approximately 110.0 feet per second), and Froude number (approximately 5.3) were estimated using the SpillwayPro software package (Reclamation, 2019). It is noteworthy that the CFD modeling completed during the ECRA indicated a greater flow depth (approximately 18.3 feet) and lesser velocity (approximately 81.2 feet per second) than estimated using SpillwayPro (Reclamation, 2019). This can be attributed to flow turbulence and recirculations predicted by the CFD model proximate to the hydraulic jump toe. The SpillwayPro (Reclamation, 2019) incoming hydraulic estimates were adopted to provide a more

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conservative estimate of baffle block and end sill heights, which were estimated using USACE (1992) and Reclamation (1984) guidance and compared with existing dimensions, as presented in Table 1.

Table 1. Comparison of existing and recommended (USACE [1992] and Reclamation [1984]) baffle block and end still heights.

Feature	Existing Conditions	USACE Guidance	Reclamation Guidance
Baffle Block Height (ft)	10.0	15.2	19.8
End Sill Height (ft)	12.0	6.6	17.2

The data presented in Table 1 further indicated that the existing baffle block and end sill dimensions were not anticipated to be adequate to properly contain the hydraulic jump and meet the preliminary design criteria noted above. As such, the modified baffle block and end sill heights advanced as part of the proposed measures were based on Reclamation guidance (1984), as a conservative approach. However, given the unique lower chute overlay configurations and associated hydraulic uncertainty, the team decided to increase the heights of the baffle blocks and end sill to 25 feet to provide additional conservatism and increase confidence in hydraulic performance adequacy appropriate for this level of study. The baffle blocks and end sill geometries will be optimized, along with the overlay, as part of future design advancements and refinements.

Although it was not anticipated to be influential to the hydraulic performance and adequacy of the modified chute and stilling basin configuration, a 4-foot-thick stilling basin overlay was also proposed. The purpose of this overlay was to provide uplift resistance across the entire stilling basin through a combination of weight and anchorage; however, the exact magnitude and location of required uplift resistance were not evaluated at this level of design advancement. As such, this element will be further evaluated as part of future design advancement and refinements.

Figure 5 presents the adopted lower chute smooth overlay profile with the modified stilling basin configuration. It should be noted that this configuration was also coupled with the lower chute stepped overlay profile. These configurations were carried forward as part of further design advancement and evaluation.

5) Design Uncertainty

Given the various aforementioned lower chute and stilling basin modification configurations, a total of five potential configurations were considered for further evaluation:

i. Configuration No.1 – Lower chute smooth overlay with existing stilling basin (refer to Figure 3)
ii. Configuration No.2 – Lower chute smooth overlay, stilling basin overlay, modified baffle blocks, and existing end sill (refer to Figure 4)
iii. Configuration No.3 – Lower chute smooth overlay, stilling basin overlay, modified baffle blocks, and modified end sill (refer to Figure 5)
iv. Configuration No.4 – Lower chute stepped overlay, stilling basin overlay, modified baffle blocks, and existing end sill (refer to Figure 6)
v. Configuration No.5 – Lower chute stepped overlay, stilling basin overlay, modified baffle blocks, and modified end sill (refer to Figure 7)

These configurations were sequenced with a progression in modification to the lower chute and stilling basin in order of scope and scale of necessary modifications for each overlay configuration type (i.e., smooth or stepped) with configuration No.1 being the least extensive and configuration No.5 being the most extensive. Note that the progression of modifications in Figures 3 through 7 is shown with each newly added feature to a particular configuration displayed in light green hatching and subsequent figures display previously added modifications in darker green hatching. The hydraulic performance and adequacy of these configurations was not well understood and difficult to predict using first principles, however, anticipated and necessary risk reduction hinged upon the performance of these configurations. As such, a “proof of concept” understanding was necessary to reduce uncertainty and advanced hydraulic modeling using CFD modeling was recommended.

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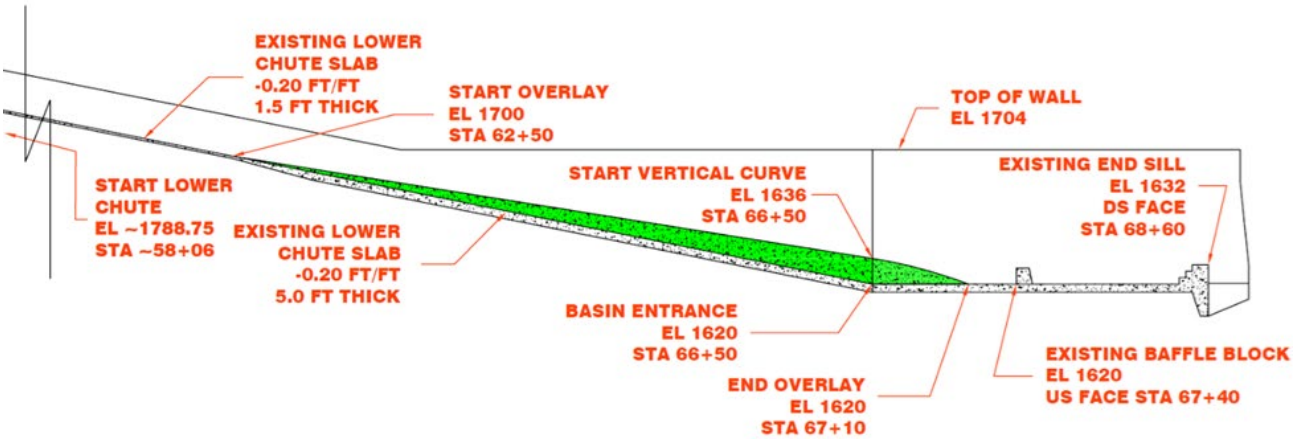


Figure 3. Profile view of Configuration No.1 – Smooth lower chute overlay.

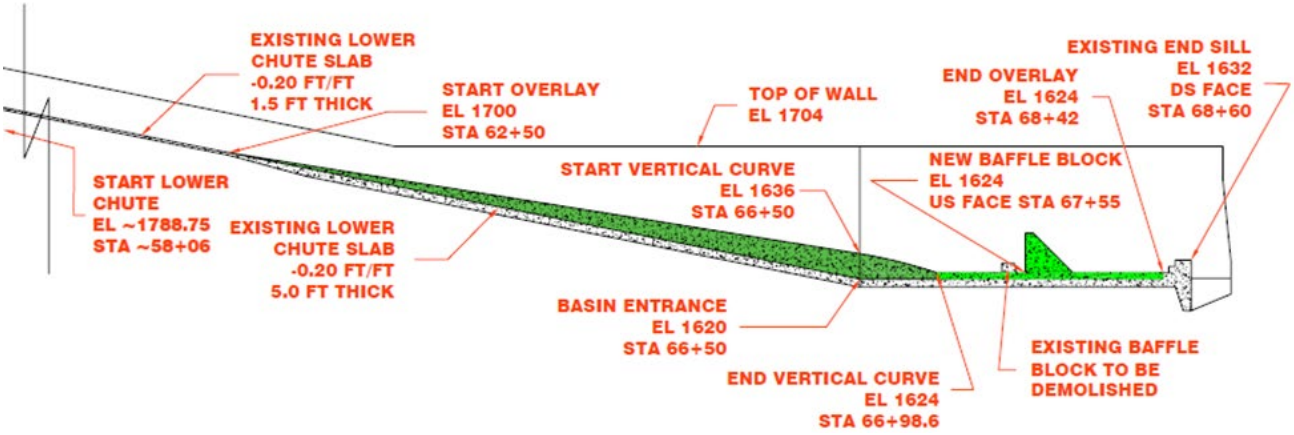


Figure 4. Profile view of Configuration No.2 – Smooth lower chute overlay, stilling basin overlay, and modified baffle block.

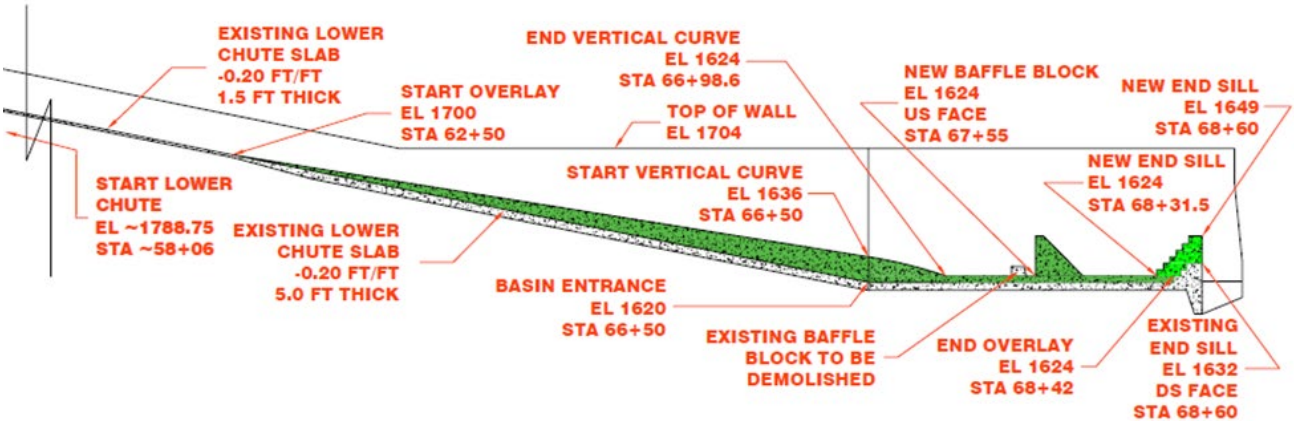


Figure 5. Profile view of Configuration No.3 – Smooth lower chute overlay, stilling basin overlay, modified baffle block, and modified end sill.

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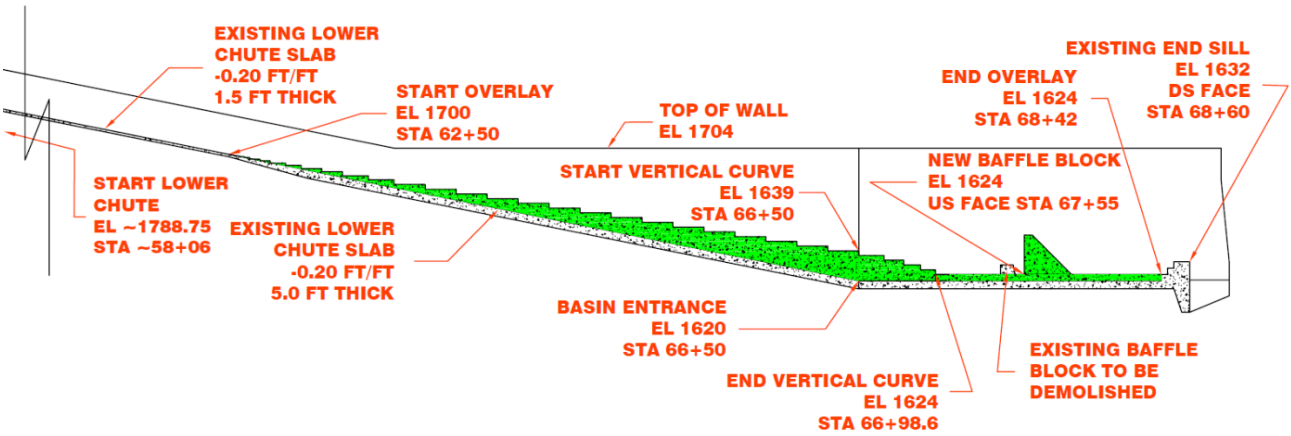


Figure 6. Profile view of Configuration No.4 – Stepped lower chute overlay, stilling basin overlay, and modified baffle block.

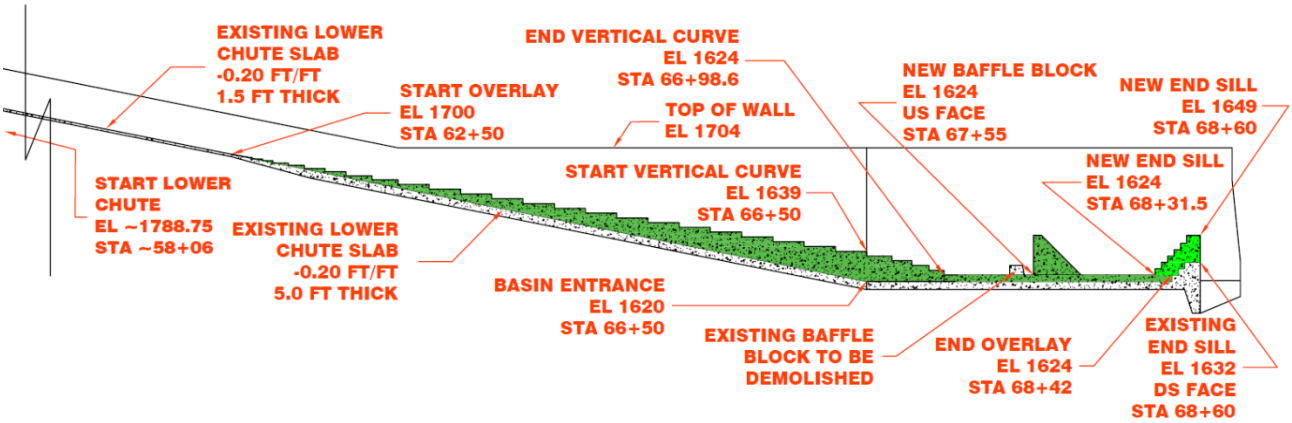


Figure 7. Profile view of Configuration No.5 – Stepped lower chute overlay, stilling basin overlay, modified baffle block, and modified end sill.

V. CFD MODELING

1) Overview

Existing and modification configurations were evaluated using CFD for a 64-foot-wide (prototype) sectional flow volume domain comprising a portion of the upper chute, the lower chute, the stilling basin, and a portion of the exit channel. An isometric view of the model domain is presented in Figure 2 and a profile view of the model domain and other pertinent information is presented in Figure 8. An iterative (i.e., stepwise) modeling approach was initially proposed in which each configuration would be simulated, evaluated, and refined prior to conducting additional simulations. However, project schedule constraints ultimately required that all configurations were simulated and evaluated concurrently without opportunity for refinement.

The CFD software utilized for this evaluation was OpenFOAM version 1906. InterFoam, a multiphase solver packaged within the OpenFOAM CFD model, was utilized for this analysis. An OpenFOAM CFD simulation is controlled by a set of governing equations, which must be solved for each control volume that comprises the simulation domain. The mass conservation and momentum conservation equations in an Eulerian frame, commonly referred to as the Navier-Stokes equations, are used to describe how the velocity and pressure of a moving fluid are related. For this evaluation, Reynolds averaging of the solution variables in the instantaneous Navier-Stokes equations was used (Reynolds-averaged Navier-Stokes equations).

The basic requirements for the CFD model include a numerical mesh of the flow domain, boundary conditions, and model parameters including initial conditions. The OpenFOAM mesh utility, snappyHexMesh, was utilized to

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generate a 3-dimensional mesh containing hexahedra and split-hexahedra cells. The mesh conforms to the surfaces by iteratively refining a starting mesh and morphing until convergence with the input surfaces. The mesh cell count for the sectional model was approximately 13 million cells based on boundary surface, water phase fraction, and air phase fraction cell sizes of 0.25 meters, 0.50 meters, and 1.00 meters, respectively.

Model boundary conditions included inlet, outlet, bottom, top, and side wall boundaries. The inlet boundary was modeled using a specified volumetric flow rate boundary condition (i.e., approximately 18,000 cubic feet per second for the 64-foot-wide sectional domain [the PMF discharge]) as well as flow depths and velocities sourced from the CFD modeling completed as part of the ECRA. The outlet boundary was modeled using a specified pressure boundary condition (i.e., tailwater elevation/depth associated with ultimate retrogression exit channel configuration). The top of the domain was modeled to represent atmosphere conditions (i.e., free inflow and outflow of the air phase under constant pressure head). The bottom flow surfaces were modeled with a wall function, which relates the local wall shear stress to the mean velocity, turbulent kinetic energy, and rate of dissipation using a wall roughness height. The side walls were modeled as a slip boundary to reflect a zero gradient velocity condition for this sectional model approach.

The solver was applied to the multi-phase volume-of-fluid model with two Eulerian phases (water and air). Air and water density values of 1 kilogram per cubic meter and 1,000 kilograms per cubic meter, respectively, were adopted. Air and water kinematic viscosity values of 1.48e-5 square meters per second and 1.00e-6 square meter per second, respectively, were adopted. The kEpsilon turbulence model was utilized. A second order finite volume discretization scheme was implemented. The High Performance Computers (HPCs) at the USACE Engineer Research and Development Center (ERDC) Department of Defense Supercomputing Resource Center were utilized for computing the simulations.

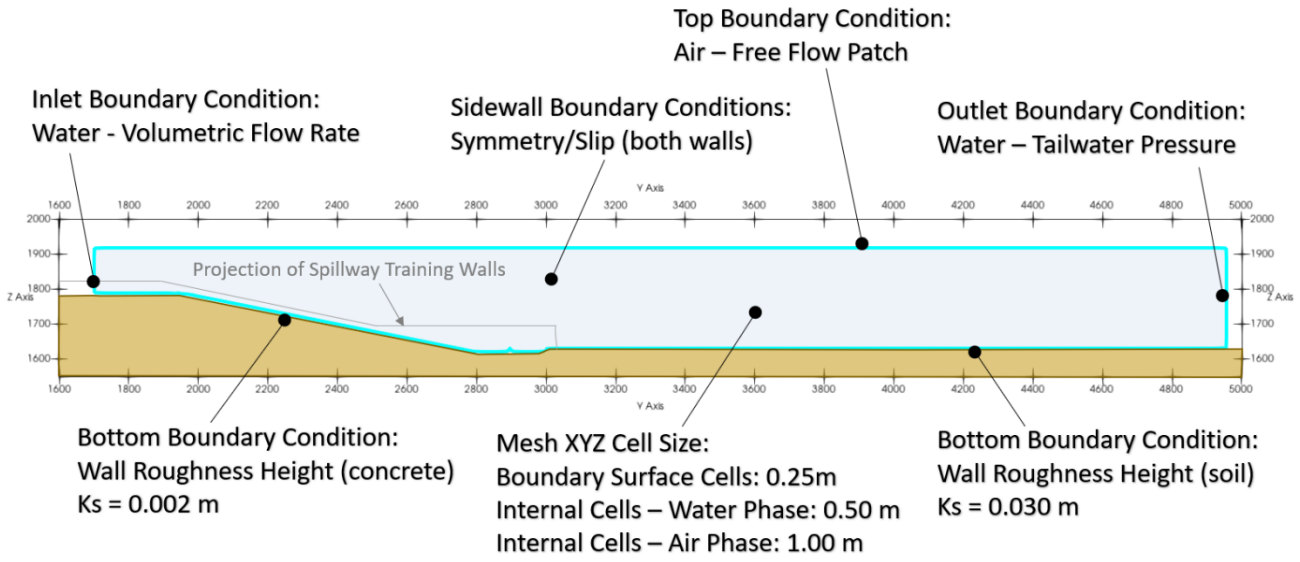


Figure 8. Profile view of CFD model domain and boundary conditions (note shift between CFD and Design Stationing: CFD Station [feet] + 3,850 [feet] = Design Station [feet]).

2) Results

In general, the CFD modeling results were successful in providing “proof of concept” understanding and preliminary confirmation of the hydraulic adequacy of the various modification configurations evaluated during the DSMS. A key outcome of the study was the sequential addition of features within each configuration simulation. This was beneficial in presenting an iterative approach to understanding the hydraulic effect and performance as each spillway feature was added to provide a more comprehensive understanding of their collective hydraulic performance. Simulation result adequacy appeared to range from marginal for the lower chute smooth overlay with existing stilling basin to near complete for the lower chute stepped overlay with stilling basin modifications. Profile views of existing and modified configuration simulations are presented in Figures 9 through 14 with additional discussion, as follows:

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i. Configuration No.1 translated the hydraulic jump farther downstream (refer to Figure 10), as compared to the existing configuration (refer to Figure 9), and appeared to provide marginally adequate hydraulic performance based on the preliminary design criteria noted above.

ii. Configuration No.2 appeared to significantly improve hydraulic adequacy (refer to Figure 11), as compared to the existing configuration and Configuration No.1, but produced a larger surface roller, which can likely be attributed to the height of the baffle block. This roller appeared to be stable, but may not provide adequate hydraulic performance, particularly, at the basin training walls.

iii. Configuration No.3 appeared to produce slightly more favorable (refer to Figure 12), but similar hydraulic performance to that of Configuration No.2.

iv. Configuration No.4 appeared to produce appreciable energy dissipation within the chute with no obvious indications of inadequate stepped spillway hydraulic performance (refer to Figure 13). This configuration also appeared to significantly improve hydraulic adequacy as compared to Configuration No.1, No.2, and No.3, however, the surface roller persisted but with reduced magnitude, which may provide more adequate hydraulic performance.

v. Configuration No.5 appeared to produce slightly more favorable (refer to Figure 14), but similar hydraulic performance to that of Configuration No.4.

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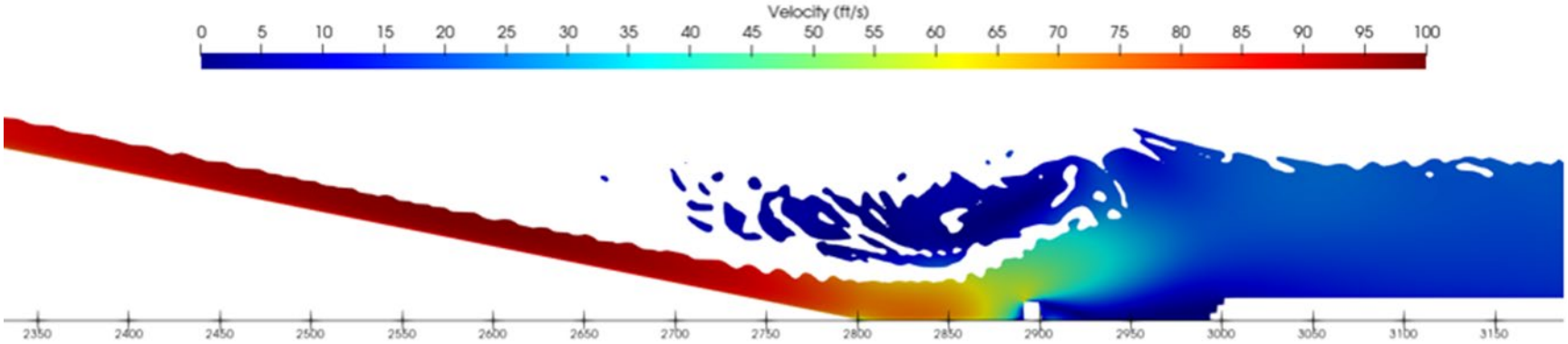


Figure 9. Profile view of CFD modeling results for the existing lower chute and stilling basin configuration (note shift between CFD and Design Stationing: CFD Station [feet] + 3,850 [feet] = Design Station [feet]).

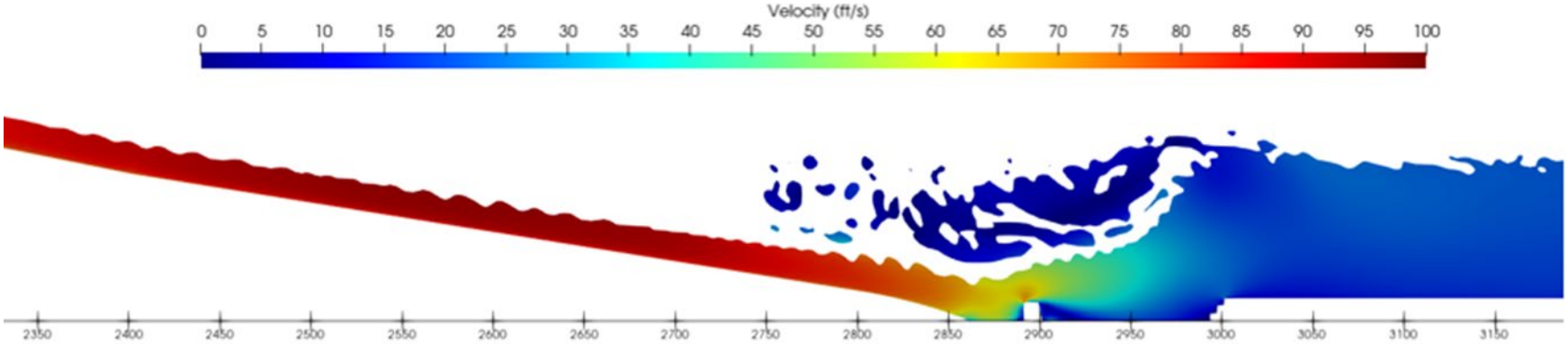


Figure 10. Profile view of CFD modeling results for Configuration No.1 (note shift between CFD and Design Stationing: CFD Station [feet] + 3,850 [feet] = Design Station [feet]).

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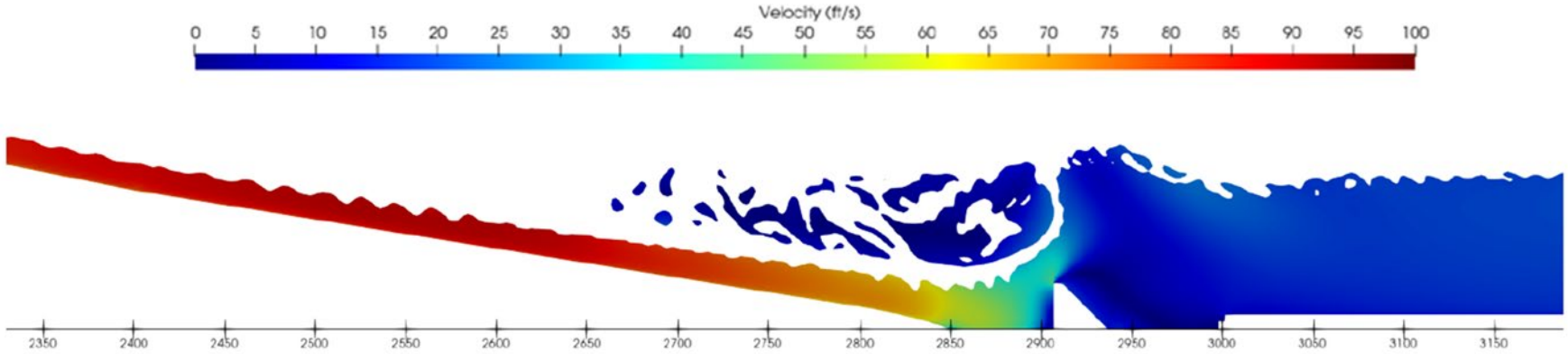


Figure 11. Profile view of CFD modeling results for Configuration No.2 (note shift between CFD and Design Stationing: CFD Station [feet] + 3,850 [feet] = Design Station [feet]).

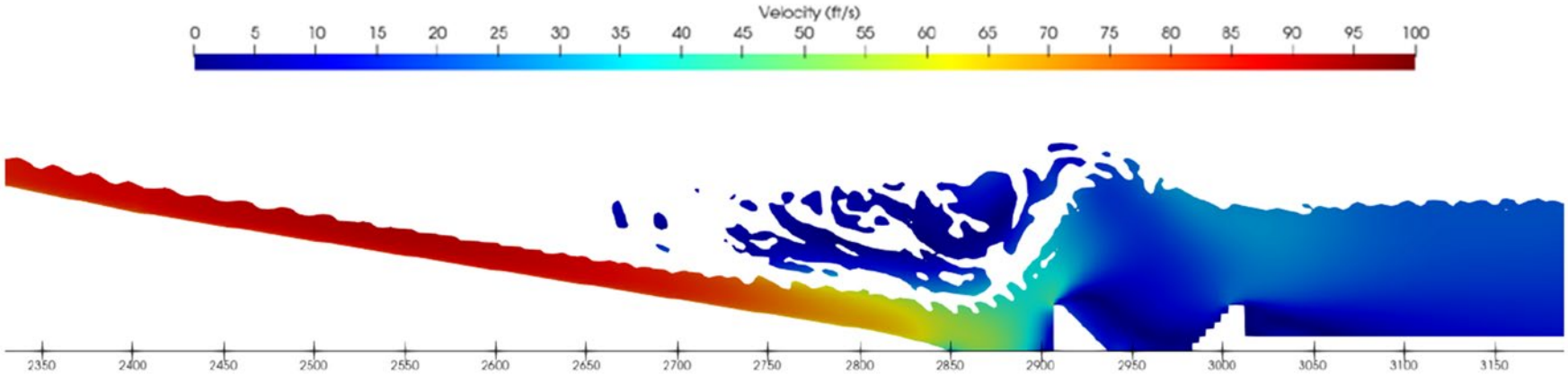


Figure 12. Profile view of CFD modeling results for Configuration No.3 (note shift between CFD and Design Stationing: CFD Station [feet] + 3,850 [feet] = Design Station [feet]).

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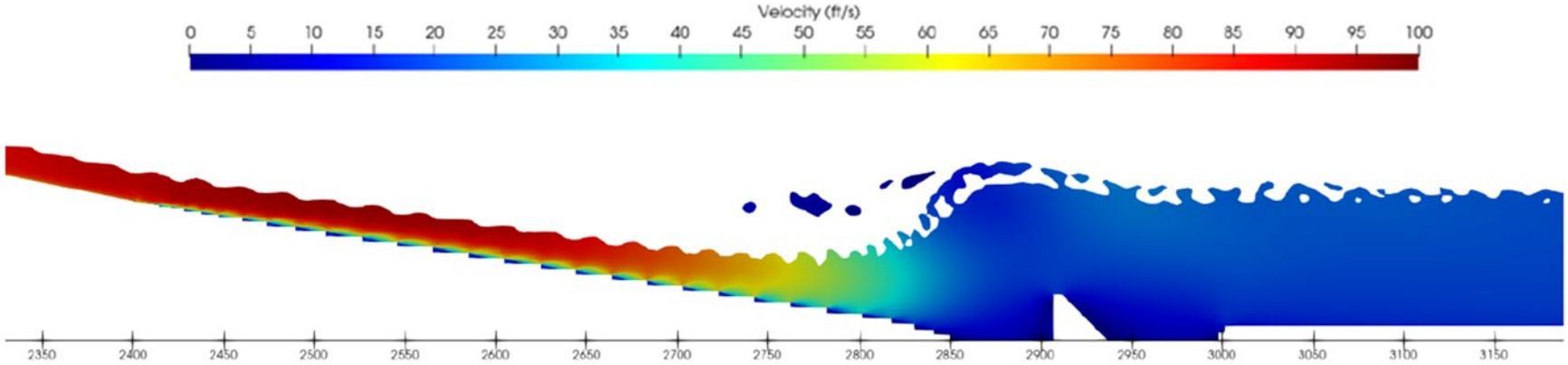


Figure 13. Profile view of CFD modeling results for Configuration No.4 (note shift between CFD and Design Stationing: CFD Station [feet] + 3,850 [feet] = Design Station [feet]).

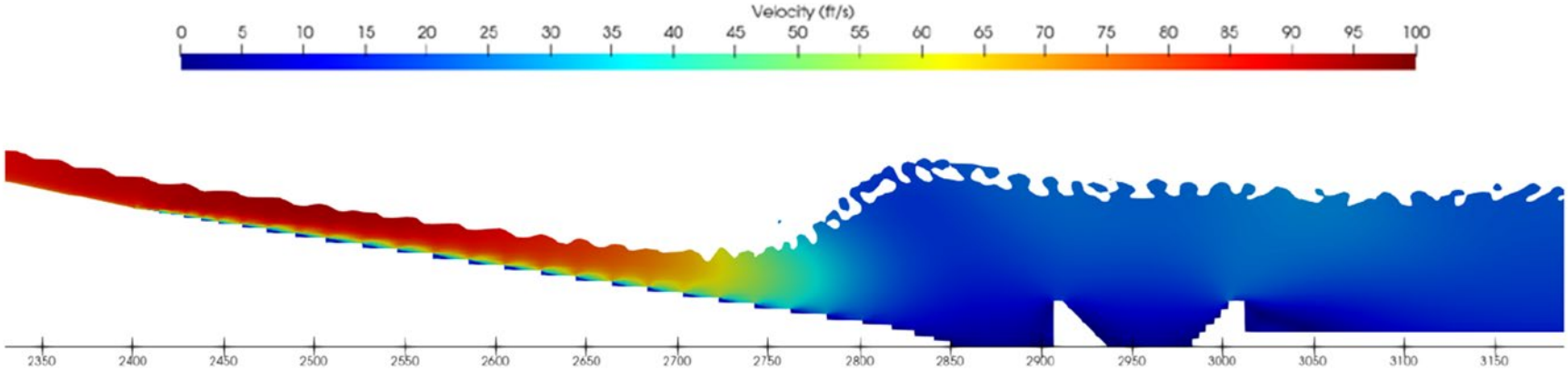


Figure 14. Profile view of CFD modeling results for Configuration No.5 (note shift between CFD and Design Stationing: CFD Station [feet] + 3,850 [feet] = Design Station [feet]).

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VI. CONCLUSIONS AND PATH FORWARD

The CFD hydraulic modeling results demonstrated the proposed overlay and stilling basin modifications should be sufficient to provide adequate hydraulic performance and mitigate PFM 8 (hydraulic jacking due to uplift). The uplift reduction measure consisting of a mass gravity overlay (no anchorage) provided a "proof of concept" understanding for this preliminary level of study. However, the uplift reduction concept will be more rigorously studied in future design phases to refine the spillway lower chute and stilling basin geometry through optimization of the combined overlay gravity mass thickness and anchorage as required to mitigate uplift forces on this structure.

Future hydraulic design of the lower chute and stilling basin will include physical modeling to supplement the additional planned CFD modeling. Additional hydraulic design evaluation and refinements planned for future study include these considerations:

- Maximum overlay thickness (anticipated to include some degree of anchorage)
- Overlay transitions
- Overlay profile type
- Full suite of discharges with "high" and "low" tailwater conditions

These refinements will continue to consider the overlay encroachment issue with the goal of avoiding modifications to the stilling basin monolith walls. Future design will follow risk-informed design guidance to ensure risks such as PFM 8 in the spillway lower chute and stilling basin are tolerable for the final project modification.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

Cracking of buttresses in an Ambursen Dam Spillway

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ABSTRACT

Ambursen dams often exhibit buttress cracks that have been attributed to thermal expansion of the upstream slabs. At the Colliersville Dam in New York State, all of the buttresses cracked shortly after construction. At this dam, the reservoir submerges the upstream slabs and the downstream rollway faces south. This paper aims to assess the role of thermal expansion of the downstream rollway slab in contributing to the buttress cracking phenomenon.

To analyze this Ambursen dam, a finite element analysis was conducted utilizing the SAP2000 software. One typical buttress of the buttresses of the Colliersville dam was analyzed. The buttress was modelled as a 3-D shell element to realistically describe the deformation and possible cracking of concrete. The analysis investigated the potential formation of tensile cracks within the buttresses due to the temperature variations on the downstream slab. A temperature variation load was applied to the buttress on the downstream side, in addition to dead load, hydrostatic load, and uplift.

The results of the finite element analysis indicate that temperature fluctuations in the downstream rollway slab can cause expansion and shrinking of the slab that can lead to the development of tensile cracks in the buttresses of the Ambursen dam. The results of the analysis showed that in the areas where the buttresses exhibited cracking, the tensile stress exceeded the tensile capacity of the concrete. The buttresses are minimally reinforced and the tensile resistance from the reinforcing bars was not considered in the analysis. The results of analysis suggest that the thermal expansion of the downstream rollway slab is significantly contributing to the observed buttress cracking phenomenon at the Colliersville Dam.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

Case History of the 2002 Swift No. 2 Dam Failure

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ABSTRACT

Swift No. 2 Dam failed suddenly in the early morning of April 21, 2002, following 44 years of operation. The 38 foot tall embankment failed in under 4 hours, releasing flows up to 9,000 cfs and resulting in damage to the adjacent highway, powerhouse, and switchyard but no injuries or fatalities occurred. Following failure of the embankment, an extensive post-failure investigation found that a seepage path and backward erosion piping through erodible sandy soils was able to progress under the dam due to a roof formed by a basalt flow. The void that resulted from the backward erosion piping was ultimately estimated to be approximately 7 acres in size and up to several feet in height, that led to a "blowout" of the toe, collapse of the basalt and overlying embankment, and breaching of the embankment dam. This paper summarizes the history of the project, the causes that led to the failure as determined from the post-failure investigation, the impacts of the failure, and probable failure mode as determined by the post-failure investigation.

I.INTRODUCTION

The Swift Hydroelectric Complex is located on the north fork of the Lewis River, approximately 60 miles northeast of Portland, Oregon (Figure 1). The complex includes Swift No. 1 Dam and Reservoir that provides water for the Swift No. 1 Powerhouse. The Swift No. 1 Powerhouse discharges into the Swift No. 2 Power Canal and Forebay, bypassing the Lewis River. The Swift No. 2 Dam and Forebay provides water for the Swift No. 2 Powerhouse, which discharges into the Yale Reservoir. Swift No. 1 Project is owned and operated by PacifiCorp, Swift No. 2 Project is owned and operated by Cowlitz County PUD, with both projects under the jurisdiction of the Federal Energy Regulatory Commission (FERC).

On April 22, 2002, after 44 years of operation, the Swift No. 2 dam embankment failed in under 4 hours releasing 9,000 cfs of flow resulting in damage to the adjacent highway and Swift No. 2 Powerhouse and switchyard. Following failure of the embankment, a post-failure investigation was completed to determine the cause of the failure (CH2M HILL, 2003). It was found that the failure originated from a backward piping internal erosion mechanism that connected a location in the forebay, about 600 feet upstream of the dam, to the tailrace downstream of the dam. This seepage path, once formed, was able to continue due to a roof formed by an overlying basalt flow. The void was ultimately estimated to be approximately 7 acres in size and up to several feet in height. The void transmitted water pressures from the reservoir and resulted in a "blowout" of the toe, resulting in breaching of the embankment.

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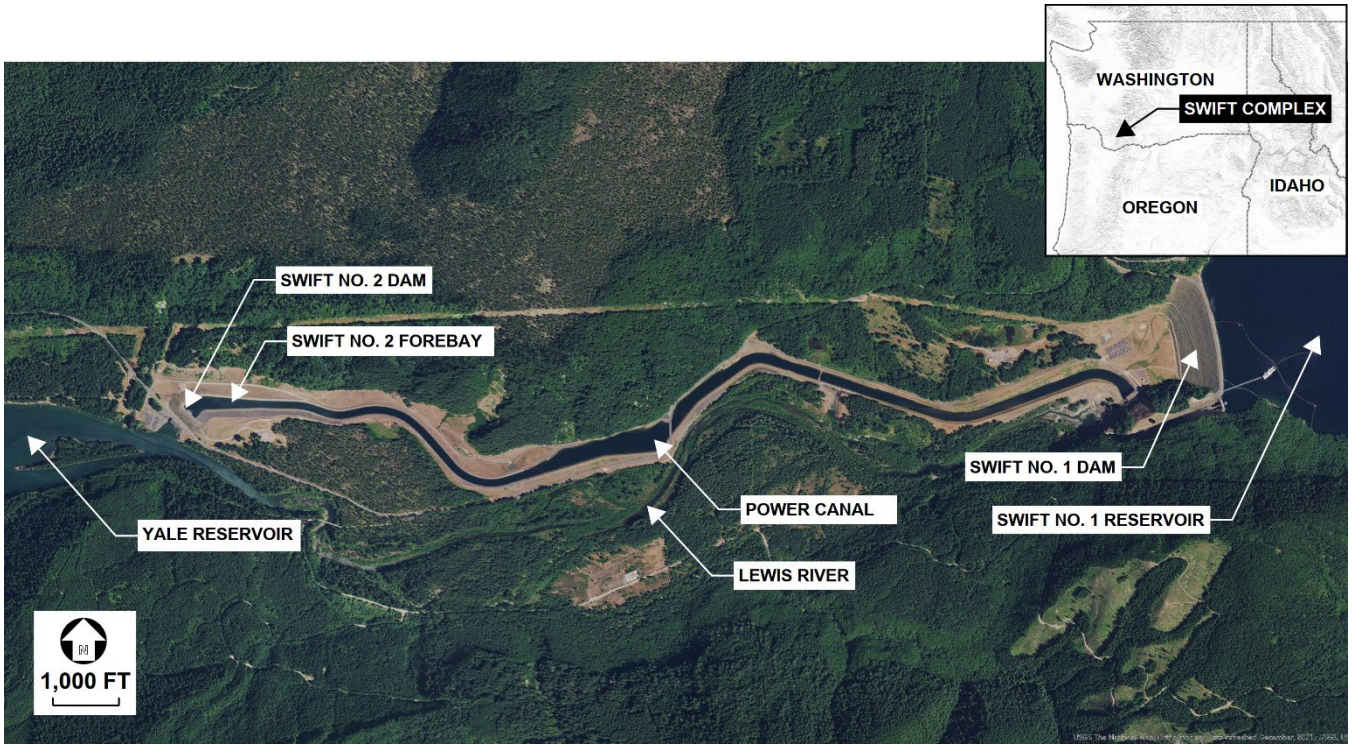


Figure 1. Swift Complex Location and Site Plan (Map services and data from U.S. Geological Survey, National Geospatial Program)

II.GEOLOGY

Swift No. 2 Dam is located in Washington on the western slope of the Cascade Range south of Mount Saint Helens, in a region dominated by volcanic activity, and is underlain by the following geologic formations (Figure 2):

- Ohanapecosh Formation (To) is the oldest bedrock unit at the site, and is composed of slightly metamorphosed interstratified rock consisting of lava flows, breccia, tuff, and volcaniclastic rocks more than 3.1 miles thick and 45 to 31 million year old.
- Swift Creek Assemblage (Qsh) overlies the Ohanapecosh Formation and consists chiefly of unconsolidated deposits of pyroclastic flows, lahars, and alluvium. This formation has a thickness of up to 650 feet and is 36,000 to 12,000 years old.
- Pre-Basalt Lewis River Alluvium underlies the Cave Basalts, and varies from sandy, gravel-cobble alluvium to sand and sandy silt that were part of the main channel or floodplain deposits of the Lewis River. In the area under the Swift No. 2 Dam and Forebay significant deposits of well-graded sand with gravel and silt that were likely from historic Lewis River overbank flood deposits. Some of these deposit were identified during the geotechnical boring for the original dam design, described as “packed fine to medium sand with little silt” and “medium sand with some gravel”.
- Cave Basalt (Qcb) is reported as a single basalt flow that originated from the southern flank of Mount Saint Helens and flowed into the Lewis River Valley. The flow is up to 80 feet thick and is about 1,900 years old. The flow overlies the Swift Creek Assemblage and Pre-Basalt Lewis River Alluvium. The upper surface of the basalt flow is hummocky, with irregularities up to 20 feet. Tree molds and log casts are common, along with pressure ridges and collapsed tumuli. The Cave Basalt is known to contain extensive lava tubes through the flow, with more than 5.5 miles of documented lava tubes within the overall formation.
- Post-Basalt Alluvium overlies the Cave Basalt and was likely deposited by the Lewis River shortly after the Cave Basalt flow.

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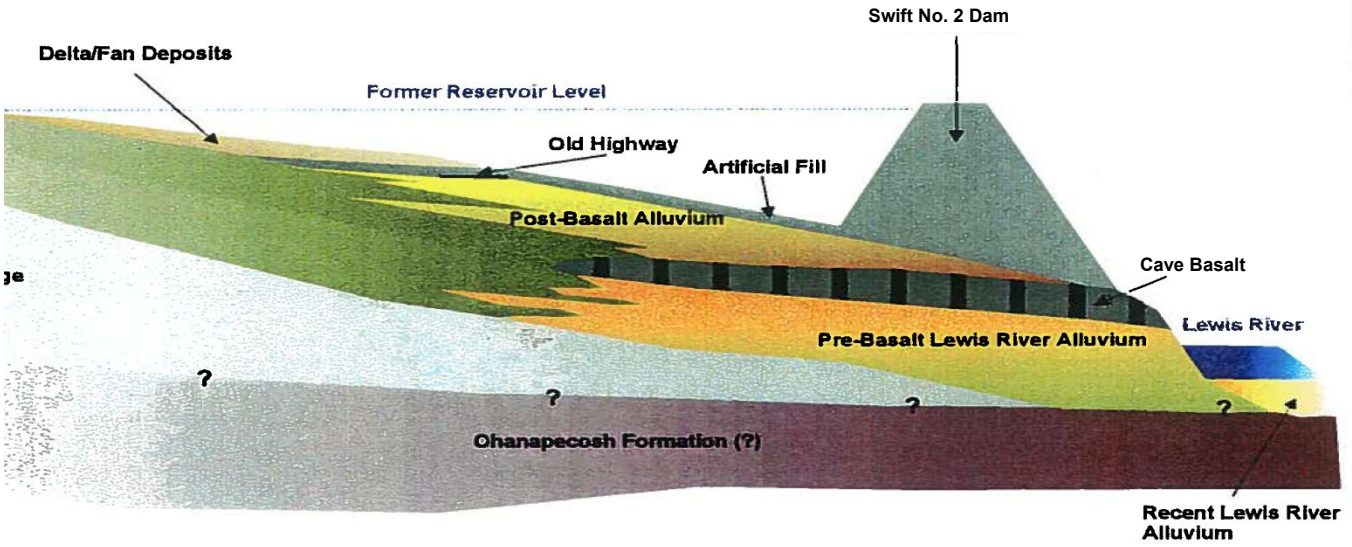


Figure 2. Swift No. 2 Dam Geologic Cross Section

III.SWIFT NO. 2 DESIGN AND CONSTRUCTION

Swift No. 2 Dam was an 83 foot tall earth embankment dam (Figures 3 and 4 prior to the failure, and located at the downstream end of a 16,700 ft long earth lined power canal fed by discharge from the Swift No. 1 project (Figure 1). The project was designed in 1956-1957 and construction was completed in December 1958. The project included a board of consultants that met six times during design and construction, and included A.H. Ayers, Dr Arthur Casagrande, Julian Hinds, and E.R de Luccia.

The power canal embankment varied in height up to approximately 80 feet, and the canal bottom was blanketed with low-permeability silty sand material (Zone 1) that was tied into the upstream toe of the Swift No. 2 Dam (Figure 4). This material consisted of well-graded silty sand with not less than 12 percent passing the No. 200 sieve and was placed throughout the length of the canal, with a thickness that generally varied from 12 to 54 inches. The native overburden above the basalt was thinnest near the Swift No. 2 Dam, and the blanket thickness was increased to 11 feet at the upstream toe of the Swift No. 2 Dam.

The forebay embankment (Swift No. 2 Dam) was a zoned earth embankment dam with Zone 2 being the flow barrier for the embankment dam, consisting of well-graded silt, sand, and gravel material with more than 5 percent passing the No. 200 sieve. The interior of the embankment was a well-graded sand and gravel with less than 5 percent passing the No. 200 sieve (Zone 3), and the downstream shell was a rock fill section with basalt gravel and boulders (Zone 4). No transition filter zones were incorporated into the design per the drawings. The upstream slope was 2.0(H):1.0(V), and the downstream slope was 1.5(H):1.0(V).

Swift No. 2 Powerhouse was a concrete and steel structure 60 feet wide by 144 feet long and contained two vertical-shaft Francis turbines rated at 35MW at a design hydraulic head of 128 feet. Two 16 foot diameter, 250 foot long penstocks extended from the intake structure to the powerhouse. During normal operations, the flow in the power canal was about 8,000 cfs. The full length of the power canal had a capacity of 2,400 acre-feet at the normal water surface elevation of 603 ft.

As part of the design, fourteen boreholes were advanced along the power canal through the Post-Basalt Alluvium, the Cave Basalt, and the underlying Pre-Basalt Lewis River Alluvium. Three of the borings were in the embankment foundation; these borings had 100-percent recovery of the Cave Basalt formation, and no mention of cavities. The Swift No. 2 Dam was placed on both the Cave Basalt and Post-Basalt Alluvium. In locations where

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the basalt was exposed, or had thin soil cover, foundation preparation consisted of clearing, grubbing, excavation, and water-jetting to expose the basalt surface, cracks, and crevices. Water sluicing of granular materials was then used to fill cracks and crevices that were inaccessible to compaction equipment, and projecting rock was removed. This foundation preparation was required over about two-thirds of the power canal and embankment footprint.

Foundation grouting was performed, but was limited to the area of the intake structures, penstocks, and a cutoff trench 350 feet east and west of the intake structure centerline. Several grout holes penetrated the Cave Basalts, but due to a large variation in grout takes and budgetary concerns, most of the grout holes were 15 to 20 feet deep, which did not penetrate the Cave Basalt. The board of consultants concurred with this approach and noted that the grouting was primarily to improve the foundation and not to provide a water barrier. One construction shift report noted that two of the grout holes encountered small cavities immediately below the Cave Basalts, but no additional action was taken. Only primary grouting was performed, no intermediate grouting was conducted.

The power canal was first filled in December 1958. Following initial filling, numerous seepage areas were observed at various locations along the toe of the three-mile long canal embankment. The canal was dewatered in July 1959 for inspection, then refilled. The canal was again dewatered for repair of sinkholes and cracks for about 20 days in August 1959. In 1973, the canal was drained due to leakage appearing in the bank about 800 feet southeast of the intake structure; during this drawdown numerous sinkholes were documented within a mile of Swift No. 2 Dam (Figure 5), as well as twelve springs/wet spots (not shown) mapped between 1 and 2 miles upstream of Swift No. 2 Dam. A 300-400 foot section of the embankment was repaired in 1975, and during this time the sinkholes were also backfilled. The power canal was then refilled, and the project remained in operation until the Swift No. 2 Dam failure in April 2002.

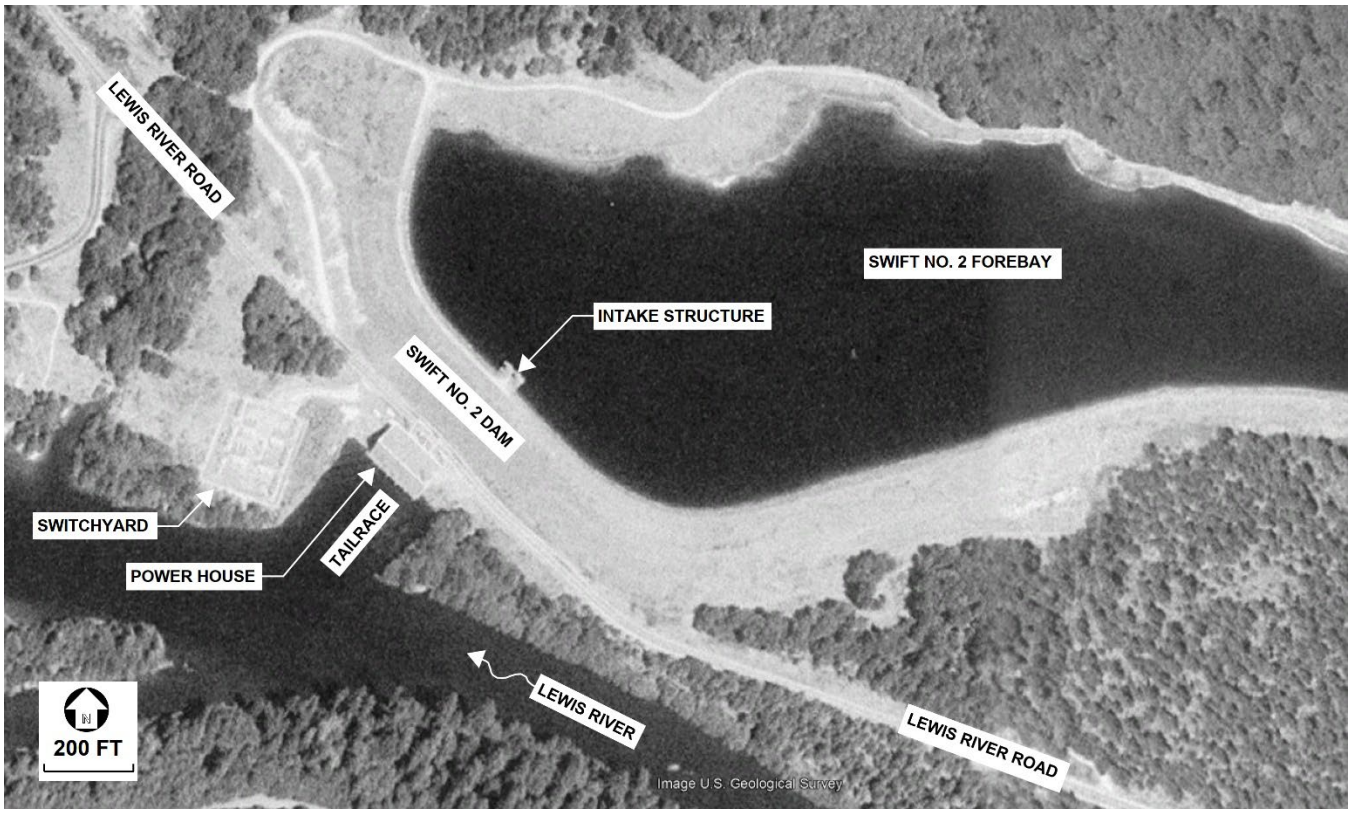


Figure 3. Swift No. 2 Dam Site Prior to Failure (US Geological Survey, 1994)

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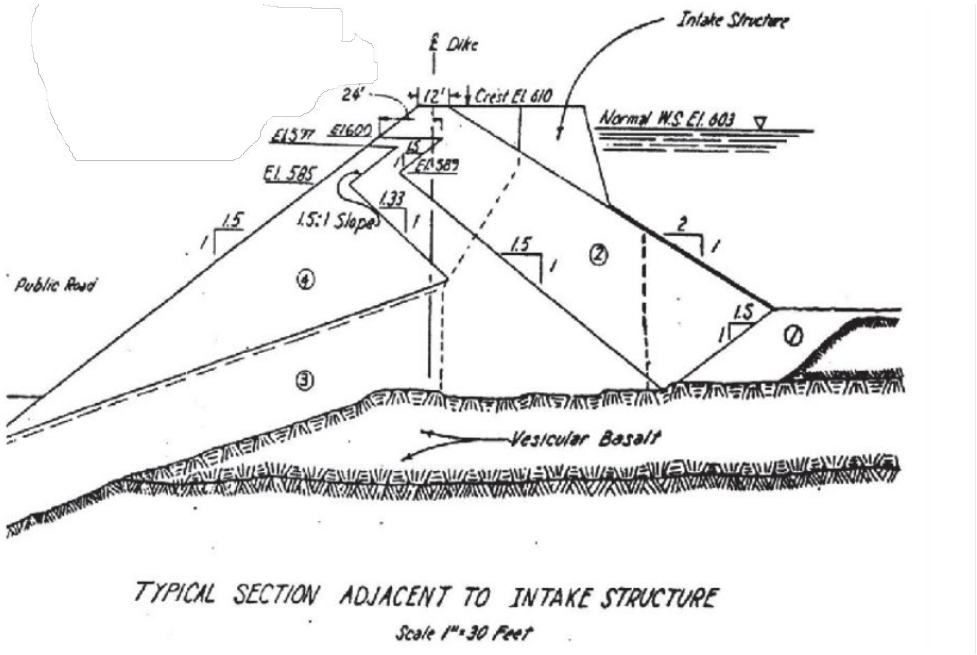


Figure 4. Swift No. 2 Dam Typical Section (Bechtel, 1957)

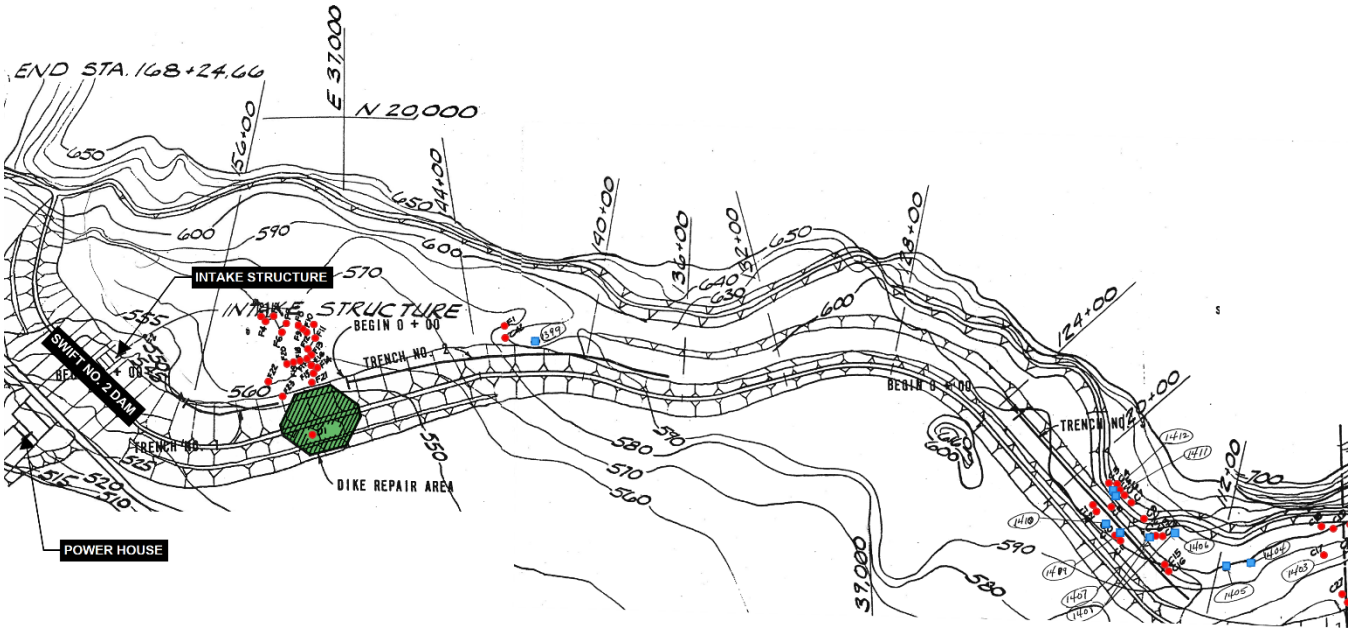


Figure 5. Swift No. 2 Sinkholes and Repairs within 1 Mile of Swift No. 2 Dam
Red Circles: 1975 Documented Sinkholes
Green: 1975 Power Canal Embankment Repair
Blue Squares: 2002 Post-Failure Documented Sinkholes

IV.HISTORIC PERFORMANCE

1) 1958 Initial Filling

Leakage was observed emerging from the Cave Basalts in borrow area cutbanks along the north bank of the Lewis River, and in some location along the power canal embankment toe. Leakage rates of 118 cubic feet per second were measured and considered unacceptable. The canal was dewatered and 50 to 75 sinkholes and a

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crack were observed in the side and bottom of the canal, with one crack noted in the bottom of the forebay about 100 feet upstream of the intake structure. These sinkholes were concentrated upstream of Swift No. 2 Dam, between stations 79+00 and 89+00 and between stations 97+00 and 110+00 (these are located between 1 and 2 miles upstream of Swift No. 2 Dam, and not shown on Figure 5).

The sinkholes and crack were repaired by sluicing and backfilling with silty sand and gravel in August 1959, and the canal was placed back in service. The repairs reduced the leakage, though some flows still occurred. Regular inspection noted no unusual or significant change in the leakage following these repairs.

2) 1973 Power Canal Repairs

"Mud boils" and leakage observed on the downstream side of the power canal embankment at station 151+00 (Figure 5) led to dewatering of the canal on May 21 and 22, 1973. Inspection of the canal found 65 sinkholes ranging in size for 2 to greater than 20 feet in diameter between stations 66+00 and 151+00 (the sinkholes downstream of station 110+00, and within a mile of Swift No. 2 Dam, are shown in Figure 5).

A review board was formed consisting of Dr Arthur Casagrande, Julian Hinds, and E.R. de Luccia. Explorations revealed that the finer grained material of the Zone 1 blanket had eroded into the underlying Cave Basalt. It was noted that water was "cascading" into the Yale Reservoir along the Lewis River bank. In addition, tunnels were discovered through the embankment at the contact between the soil and basalt. The tunnels were found filled with coarse gravel with no fines, indicating that the fines had been eroded and the gravel has collapsed into the tunnels. These observations led to the decision to remove a section of the embankment.

An approximately 140 foot wide section of the embankment near station 151+00 (Figure 5) was removed down to bedrock and the bedrock surface was cleaned with water jetting. A D9 dozer with a ripper was used to break up the Cave Basal and exposed two caverns that were 10 to 15 feet across and 8 feet deep. Fissures and cracks in the basalt were exposed and plugged with sand and gravel using high-pressure water jets, and then capped with concrete. The caverns were filled with approximately 60 cubic yads of lean concrete.

V.FAILURE ON APRIL 21, 2002.

On April 21, 2002, a section of the dam failed (Figure 7) immediately northwest of the intake structure within a period of a few hours, in the night and very early morning hours, with no forewarning of the failure. The water in the Power Canal drained into Yale Reservoir, destroying all but the concrete foundation and walls of the Swift No. 2 Powerhouse. The adjacent switchyard was severely damaged as well.

The chronology of events on the morning of April 21:

- **2:18am**: a security guard inspects the site and observes nothing unusual
- **3:00am**: a security guard drives by the site and observes nothing unusual
- **3:35 to 3:45am**: water was noted by an unidentified driver to be flowing across Lewis River Road (Highway SR 305); this was reported to a security guard at 5:35am or 5:45am
- **4:00am**: Swift No. 2 Powerhouse high sump alarm was noted at the offsite control center
- **5:25am**: Swift No. 2 Power Canal pool elevation telemetry reported lower than normal water levels had occurred in the Power Canal; an operator went to do a check at Swift No. 2
- **5:40 to 5:45am**: a security guard reported a lot of water flowing across Lewis River Road toward the powerhouse
- **5:45am**: an operator was onsite and reported that the canal embankment had failed and a torrent of water was flowing across Lewis River Road and into the Powerhouse
- **6:30am**: photos were taken of the failure
- **6:30-6:33am**: a seismograph network recorded a series of seismic signals as the embankment failed, the seismicity returned to background levels by 7:15am
- **6:55am**: an estimated 200-foot wide washout with a 10 foot high wall of water was flowing across Lewis River Road
- **7:30am**: Cowlitz County Deputy Sheriff arrived onsite

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Figure 7. Swift No. 2 Dam Photographs of the Failure (Cowlitz County PUD, used with permission)

VI. POST-FAILURE INVESTIGATION

A significant post-failure investigation was initiated following the failure to determine the most probable failure mechanism. The investigation work included:

- Surveying and photography of post-failure surface conditions
- Detailed review and evaluation of site geology and geologic history
- Geologic mapping using test trenches, test pits, and exposed surfaces
- Geotechnical explorations including test pits, soil borings, and rock coring
- Field tests and laboratory testing of soil and rock samples
- Analyses of geotechnical and hydraulic conditions
- Thorough review of hundreds design and construction files
- Smoke testing of lava tubes and voids

The post-failure investigation identified the following key failure features (Figure 8), including:

- Formation of a scoured amphitheater located at the breach. An almost continuous horizonal void was discovered under the basalt at the northeast edge of the amphitheater, up to 5 ft in vertical dimension (Figure 9). The amphitheater was the result of a scour hole during the failure, that was filled back in as the breach flows subsided; it was estimated that the scour hole was up to 40 feet deeper than the post-failure ground surface.
- A large sinkhole along the north side of the forebay, about 500 feet upstream of the breach. A lava tube up to 12 feet in diameter was discovered at the bottom of the sinkhole (Figure 9). Smoke was injected into the lava tube to confirm that the sinkhole and lava tube had a direct connection to the amphitheater; smoke reached the amphitheater in about 30 minutes, and the volume of airflow suggested a void size of 1 to 2 acres and averaging 1 to 4 feet high (Figure 11) under the Cave Basalts underlying the forebay. It was further determined that this was just a portion of the void, and through additional boreholes it

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was estimated that the void was up to 7 acres in size and extended farther downstream below the switchyard, indicated by seepage flow exiting from under the Cave Basalt downstream of the switchyard (Figure 8). Deposits of a well-graded sand with silt and gravel was identified along the perimeter of the void and was 3 feet or more feet in thickness; this deposit was likely overbank flood deposits from the Lewis River prior to the basalt flow.

- A tension crack led from the sinkhole in an arc to the breach area. Where the crack entered the breach area, an oval-shaped 16 inch diameter tunnel/void was observed in the native well-graded sand and gravel layer beneath the canal lining (Zone 1). This tension crack was likely due to subsidence of the Cave Basalt into the void.
- Twelve sinkholes were discovered in the canal after the failure (Figure 5) up to 5 feet in diameter upstream of the embankment.

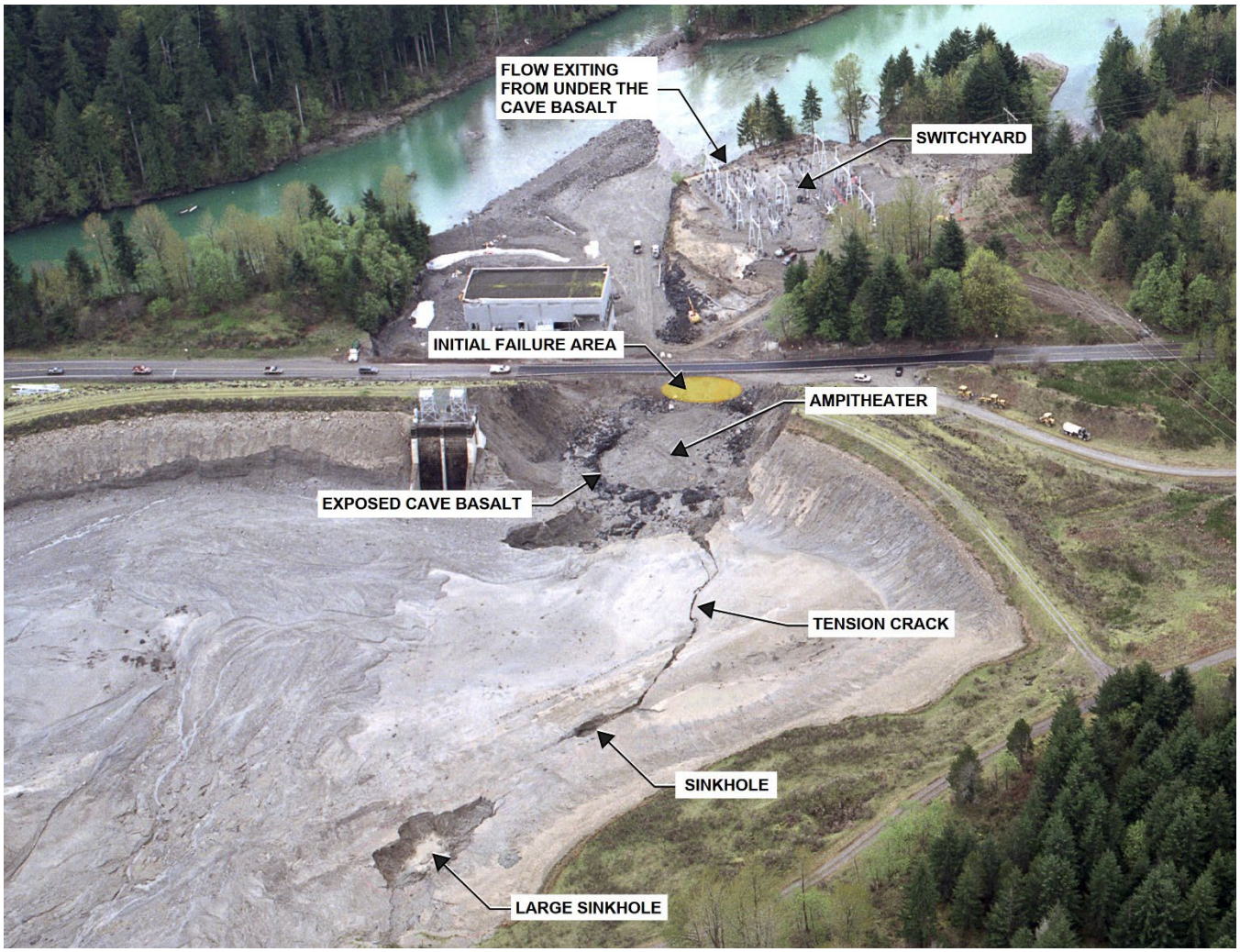


Figure 8. Post-Failure Observations (Cowlitz County PUD, used with permission)

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Figure 9. Cavity under the basalt on the northeast side of the amphitheater, up to 5 feet high and probed up to 25 feet with steel rods (Ken Green, used with permission)



Figure 10. Lava Tube (approximately 10 to 12 feet wide) at the base of the large sinkhole (Ken Green, used with permission)

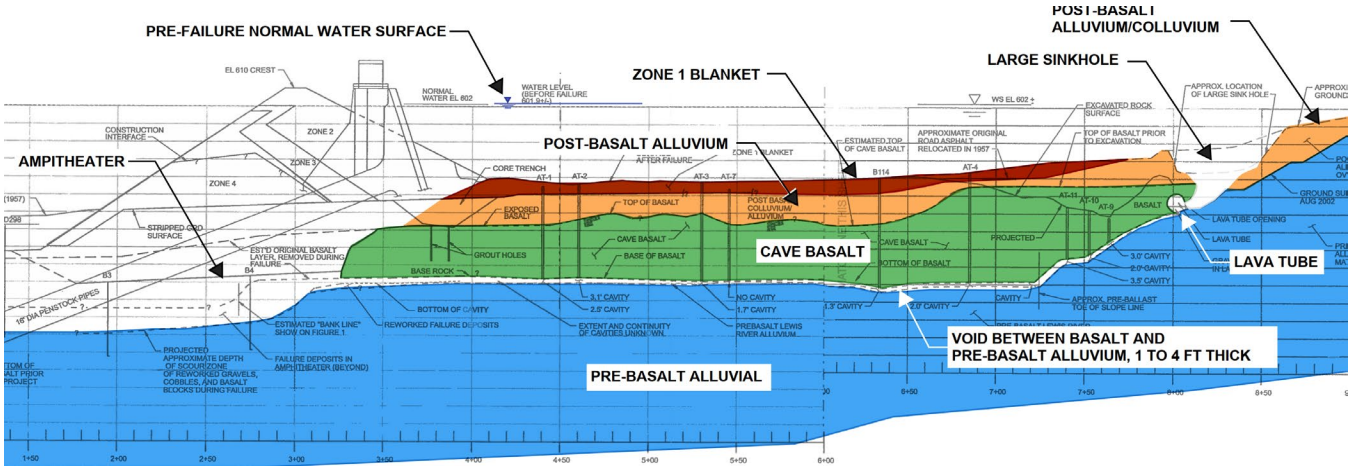


Figure 11. Post-Failure Cross-Section through the Failed Embankment and Large Sinkhole

VII. FINDINGS

The progression of the probable failure mode as determined based on the post-failure investigation:

1. The historic Lewis River flowed through and adjacent to the existing dam location, and resulted in pre-basalt alluvial deposits, including relatively uniform thickness and extensive overbank sand deposits. This overbank deposit of well-graded sand with silt and gravel was up 3 feet or more and thickness and was likely relatively erodible.
2. During formation of the Cave Basalts that covered the pre-basalt alluvium, a lava tube formed in the Cave Basalts underlying the forebay 1,900 years ago.
3. The construction and operation of Swift No. 2 provided a hydraulic head for seepage flow.
4. A seepage flow path connected from the Forebay to the Tailrace; going through the Canal Liner (Zone 1) and Post-Basalt Alluvium, into a lava tube or other voids, then through the Pre-Basalt Alluvium, and ultimately exiting at a free face into the tailrace downstream of the embankment (this seepage path followed the void shown in Figure 11). The seepage path, or a portion of it, was discovered to have exited into Yale Reservoir just downstream of the switchyard (Figure 8).
5. A hydraulic exit gradient was adequate to initiate backward erosion piping at the free face exit into Yale Reservoir, and there were no natural or constructed features in place to arrest the backward erosion,

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so piping continued along the seepage path and the void enlarged and allowed piping to continue. The Cave Basalt formation provided a roof that allowed the erosion to continue.

6. After 44 years of reservoir operation, the backward erosion piping progressed to the lava tube, and then progressed vertically to the floor of the canal, resulting in the formation of the large sinkhole. At this time there was now a void along the full seepage path from the sinkhole in the forebay to the tailrace.
7. With a direct connection between the tailrace and reservoir forebay, the water flow increased which further enlarged the void in the Post-Basalt Alluvium, Pre-Basalt Alluvium, and Zone 1 Canal Liner (Figure 11) to form the void that was ultimately observed during the post-failure investigation.
8. The flow rate increased until the water pressures at the toe of the dam caused a blowout of the toe, resulting in collapse of the embankment and breach of the dam. In addition, the basalt partially collapsed into the void due to the weight of the embankment, further progressing the breach failure.
9. The breach flow created the scour hole observed in the amphitheater, filled the powerhouse with debris, destroyed the highway, and damaged the switchyard.

VIII.CONCLUSIONS

The failure of Swift No. 2 Dam in 2022 came about suddenly on the morning of April 21st. Although there were indications of seepage in other locations along the canal as indicated by the 1958 and 1973 repair work, it was not envisioned that a seepage path existed under the Cave Basalts beneath the Swift No. 2 Dam that connected the tailrace to the reservoir forebay. The post-failure field investigation revealed not only a flow path, but erodible well-graded sand with silt and gravel deposits under portions of the basalt and a significant network of voids up to 7 acres in size under the basalt. The Cave Basalt formation was able to maintain a roof that allowed piping to progress and continue until a connection to the forebay had been reached. The water pressure in the void increased until it was sufficient to cause uplift and fracturing of the basalt at the toe of the embankment resulting the “blowout” and loss of support under the basalt which further led to caving, settling, and eventually breaching of the embankment.

There were several lessons-learned that we feel can benefit current dam owners and engineers, as well as provide valuable lessons for future design:

1. Erosion and piping can take decades to progress into a failure; Swift No. 2 Dam was in operation for 44 years prior to failure. Though there were historical indications of piping in the canal due to sinkholes and seepage, these were independent of the failure that was observed in 2002, and it took 44 years for the backward erosion piping under the dam to manifest into the failure.
2. There were no records of identified features prior to failure that could have indicated a failure mechanism was forming. However, the failure does highlight the importance of diligent observations and periodic surveys to attempt to identify features of concern that could be an indication of backward erosion piping or other failure mechanisms. Relatively recent advances in technology has made observations more accurate and efficient to collect, using tools such as multi-beam bathymetric surveys to identify sinkholes below a body of water, and the use of drone based photogrammetry and LiDAR to identify changes in the ground surface and provide high-resolution aerial photographs. These tools were not available, or nearly as sophisticated as they currently are, but can now be used on a relatively frequent basis to supplement onsite visual observations from engineers, operators, and/or maintenance staff.
3. Re-evaluation of design and construction records and past performance is a valuable exercise. Following the failure, a detailed review of design and construction records and past performance was undertaken. During this review, several observation were made that on their own may not have been a concern, but when combined together might have indicated a concern with internal erosion. These included sandy soils with little silt or gravel, a grout curtain that was not completed across the dam foundation, and the 1958 and 1973 sinkholes in the canal upstream of the dam. Hindsight helps, but all of these provide an indication that pipeable materials were present, water pressures were adequate to pipe materials, and there were no intervening features under the dam to prevent backward erosion piping from progressing if there were defects in the basalt that could allow erosion of the canal liner

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IX.ACKNOWLEDGMENTS

The author gratefully acknowledges the contributions of Ken Green of Jacobs Engineering Group in documenting the failure of Swift No. 2 in the CH2M HILL (2003) report, and the mentorship he provided to the author in the process; and gratefully acknowledges Gary Huhta, General Manager of Cowlitz County PUD, for his approval to publish this important case history.

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DAM SAFETY PALM SPRINGS
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Yet Another Lesson Learned from Wolf Creek Dam

MICHAEL A. BROWN, PE, NATIONAL PRACTICE LEAD FOR DAMS & LEVEES, TETRA TECH INC.; AND DYLAN GRISSOM, PE, RISK CADRE LEAD, US ARMY CORPS OF ENGINEERS, NASHVILLE DISTRICT

ABSTRACT

Wolf Creek Dam is part of the US Army Corps of Engineers (USACE), Nashville District portfolio, located along the Cumberland River in southern Kentucky.

Construction began in 1941, and the reservoir, Lake Cumberland, was impounded in 1952. There was a construction delay of several years (1942 to 1947) due to World War II. The dam is founded on karstic limestone. Wolf Creek Dam has a gravity portion with 37 concrete monoliths, and an embankment portion constructed of well compacted clay.

Seepage was detected by 1962 in the form of significant wet areas along the downstream toe. Two sinkholes were discovered in 1968, both on downstream area adjacent to the powerhouse switchyard, and muddy show in tailrace.

Several remediation contracts were performed from 1975 to 1979 to include a series of grout curtains and a diaphragm wall constructed by ICOS. The ICOS wall was constructed at the crest of the dam along the upstream shoulder of US Route 127.

There have been several lessons learned regarding the ICOS wall of Wolf Creek Dam, probably the most famous lesson was foreseen by Ralph Peck who stated that the first barrier wall was neither long enough nor was it deep enough.

This lesson learned technical paper will discuss the features of construction allowed to remain in place after completion of ICOS diaphragm wall project. The features were located right of the interface of the embankment and gravity portions of the dam along the crest. They were paved over with bituminous hot mix. Due to increasing traffic loads on US Route 127, settlement, rutting and cracking occurred and were documented and described in several annual inspections, periodic assessment reports and numerous dam safety incident reports. Cracks, rutting, and differential settlement were observed in the roadway at the crest along both north and south bound lanes. Some of the problem areas were understood. However, the cause and full extent were not revealed until the design of the remediation began in 2016 and historical construction documents were discovered and reviewed.

This technical paper will tell the story and communicate the lessons learned.

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I. INTRODUCTION

The U.S. Army Corps of Engineers (USACE) operates and maintains approximately 740 dams and associated structures nationwide that provide significant, multiple benefits to the nation—its people, businesses, critical infrastructure, and the environment. Wolf Creek Dam is part of the Nashville District portfolio, located along the Cumberland River in southern Kentucky. The project authorization is for flood control and hydropower, but other missions performed include recreation, water supply and water quality. US Route 127 traverses the entire dam crest and the traffic loads have been steadily increasing.

Wolf Creek Dam impounds Lake Cumberland, which is the largest reservoir east of the Mississippi River. Lake Cumberland has a total storage of 6.089M ac-ft, when pool is at elevation 760 MSL. The spillway is a controlled ogee with a crest elevation of 723 MSL and using 10 tainter gates, which have a height of 37 feet and are 50 feet wide. Design discharge of the spillway is 553,000 cfs. Outlet works include six sluice gates, with dimensions of 4 feet by 6 feet, with hydraulically operated slide gates, with design discharge capacity of 9,800 cfs.

Construction of Wolf Creek Dam began in 1941 and was soon halted due to United States entry into World War II. Dam construction was restarted in 1947 and reservoir impoundment was begun in 1952 and completed in 1953. A point of interest; during construction was a decision was made to allow elements of a tower crane to remain in place within the embankment right of the interface between the embankment and gravity portions of the dam. By the 1960s wet areas were noticed, muddy flows were showing up in the tailrace, and in 1968 two sink holes opened adjacent to the power plant switchyard.

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The seepage, piping and sink hole issues prompted the Corps to begin a remedial grouting program in 1968, which lasted until 1975 with 300,000 cubic feet of grout solids being placed in the ground. In 1975 the ICOS diaphragm combination wall began. At the time, it was a cutting-edge approach and the world record holder for many years. The ICOS wall was constructed at the crest of the dam along the upstream shoulder of US Route 127. Follow up point of interest; the alignment of the ICOS diaphragm wall was altered to jog upstream to avoid the tower crane element allowed to remain in place post initial dam construction. The combination diaphragm wall consisted of primary and secondary piles with progressively smaller diameter casings vertically telescoped through the embankment. Primaries are 26 inches in diameter encased in steel and socketed into rock. Each primary was cored to a depth 20 to 50 feet below top of rock elevation, pressure tested and grouted. The secondary elements were excavated by machines which tracked between the primary elements. Tremie concrete was placed between the primary elements to form the secondary elements. There was concern about the bond between the concrete of the secondary elements and the steel of the primary elements. The length and depth of the wall was also a concern prompting Ralph Peck to comment that the diaphragm wall was neither long enough nor was it deep enough.

By 2005 plans began to rehabilitate the dam due to evidence showing increased seepage into the karst foundation, cracks along the crest, evidence of settlement, increased size and number of wet areas immediately downstream, soft zones, and cold water in the piezometers. The failure modes of primary concern were embankment foundation seepage and piping, embankment stability, foundation seepage and piping in the concrete gravity section, and embankment seepage and piping.

TREVIICOS – Soletanche JV was awarded the barrier wall contract, and the new foundation remediation project began in 2007 starting with grout curtains upstream and downstream of the proposed barrier wall alignment. The area of concern for this paper was referred to as Critical Area 1. Critical Area 1 had already begun showing signs of differential settlement and subsequent cracking. Traffic volume, due to construction, aggravated the condition even further. Several small maintenance projects were performed on the area prior to and during the most recent barrier wall contract to remediate until a permanent fix could be designed and constructed. Maintenance included installation of a makeshift catch basin for stormwater conveyance and various crack sealing efforts. The condition of the area was cited multiple times in annual inspection reports.

II. DAM SAFETY INCIDENT: OBSERVED VOID DETECTED AT CREST OF DAM

Due to the amount of differential settlement in the crest right of the gravity portion of the dam, one of the post barrier wall miscellaneous contracts was for Roadway Restoration. A portion of the project design was for a structural slab similar to a highway bridge approach slab, only larger. As part of the design of the remediation in 2015, historical construction documents were discovered and reviewed. It was at this time that the design team realized that features from the ICOS wall construction were allowed to remain in place. The features discovered include guidewall, slurry trench, construction platform and pile caps. Concrete thicknesses, elevations and bearing depths differed. For example, the pile cap was bearing on top of pile, which was bearing on top of rock some 280 feet down approximately at elevation 450 MSL, while the construction pad was relatively thin and bearing on clay approximately at elevation 772 MSL.

Information regarding the remaining construction features was provided in the design documents to inform the bidding contractors of the existing conditions. The contract was awarded to Bluegrass Contracting and construction began in early 2016. During the instrumentation relocation phase, the Contractor observed a void through one of the construction joints of the existing concrete features.

The reported void was located along and under the upstream edge of US Route 127 at the crest of the dam. The void was first observed by the Roadway Restoration Project Contractor (Bluegrass Contracting) in the afternoon of 17 May 2016 and reported by Matthew Short to Michael Brown, Soils and Dam Safety Section, Geotechnical Engineer. Later that same day, after arriving on site, Mr. Brown met with Mr. Short, Chris Marlow (Powerhouse Supervisor), Bernie Kearns, and other construction staff of the Resident Engineer Office, and was briefed regarding the void.

The reported depression was located along the upstream edge and under US Route 127 at Station 37+73 and approximate elevation of 772 above MSL See Figure 1. This is also the area of a jut out of the ICOS wall due to a tower crane structure, used during the original construction of the dam, being left in place. The Contractor's crew discovered the void while relocating the instrumentation cable for piezometer CW-25T. The crew had trenched

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through the existing asphalt to run the cable and noticed the void in a joint of the existing concrete, which was cast for the construction of the old ICOS wall work platform and guide wall. At the time of discovery, the void appeared to run longitudinally primarily along the upstream edge of the roadway. It appeared to follow an existing construction joint.

Prior to District engineering staff arrival, the Contractor had removed more of the concrete slab, which was believed to be the old ICOS slurry trench. Once the additional concrete was removed, a better view was possible to see into the void and look beneath the concrete. In the upstream direction, there was approximately a 24-inch vertical void extending to the downstream face of the old ICOS guide wall. In the right direction, a void was visible for approximately 10 feet with a separation below the concrete of approximately 10 inches. In the downstream direction (under the northbound lane of Route 127), the void appeared to be approximately 3 to 4 inches in height and measures 6 to 8 feet. Looking left, towards the existing catch basin, the void was 14 to 16 inches deep and extended to approximately 1 foot.

The throat was about 8 to 10 inches in diameter. The powerhouse supervisor, Chris Marlow, pushed a shovel into this hole as far as he could reach and experienced no resistance. The shovel head and a portion of the handle was covered with water and saturated mud.

Near end of the construction day, it was decided that more of the concrete slab should be removed. The Contractor was to remove a 6-foot x 5-foot section of the existing concrete, to allow for better inspection of the throat and to aid in determining where the clay fill material was escaping.

During the following morning, the Roadway Restoration Project Contractor (Bluegrass Contracting) had begun saw cutting in preparation for removal of a portion of the existing ICOS work platform. This removal was to facilitate better assessing the extent of the void. On site on 18 May 2016, along with the Contractor employees, were District Office Engineering staff, construction staff from the Resident Engineers office and powerhouse operations staff.

Removal of the existing concrete work platform was made difficult due to the thickness of concrete and steel reinforcement. The concrete saw was only capable of cutting to a depth of 6 inches, so some rebar remained uncut. The limitations of the saw made removal by a track hoe very difficult. The Contractor was able to remove a portion of the slab and an observation trench was excavated. See Figure ?. The trench dimensions were 2 feet wide (upstream to downstream), 4 feet deep and 7 feet long.

The condition of the void appeared to be the same as observed on 17 May 2023 (See Figure ?) with the exception of the upstream area throat. This area of the void appeared much larger than previously reported. The horizontal dimensions measured 10 feet left to right and 9 feet downstream to upstream. The void observed under the existing ICOS downstream guide wall. From the edge of the concrete, the void appeared to have a maximum extent of 9-foot upstream skew. Measuring the depth of the void was hampered due to the inability to look down into the hole. Engineering staff dipped the hole with a shovel and felt what appeared to be a bottom but was unsure if it was the bottom or a side slope. The side clay material on the walls of the void was the consistency of muddy paste. Residue on the blade of the shovel was highly saturated clay. The depth was 7 feet below top of guide wall elevation.

As stated earlier, the ICOS wall alignment deviated in the upstream direction in this area to avoid an existing tower crane feature that was allowed to remain in place during the initial dam construction. Due to this misalignment, the top of the ICOS wall was not capped and is currently visible within the upstream slope rip rap. The surface of the work platform of the new barrier wall in this area was wet and stained with what appeared to be clay material. Between the ICOS wall and the new barrier wall work platform was (from top to bottom) the upstream rip rap slope, sheet pile wall, new buttress shot rock covered with rip rap and 3 feet X 3 feet gabion baskets as shown in Figure ?.

The existing grade of the roadway (See Figure ?) and existing ICOS guide wall and work platform are shown in Figure ?. This condition directed storm water to this containment space (ponding) and the water found its way into the existing crack and the existing clogged catch basin. See Figure ?. The progression of the void appeared to be progressing in the upstream direction, developing a large void beneath the downstream ICOS guide wall. The team was unable to observe a crack due to visibility issues. However, in our opinion, the pathway most likely continued under the guide wall and through a crack in the ICOS wall, eventually daylighting in the upstream slope rip rap. This opinion is based on the top of ICOS wall that is exposed to the elements, and the wet and stained condition of the newer barrier wall work platform.

On 19 May 2016 the area was again inspected by Michael Brown and Vanessa Bateman. No further changes of the area were noted. The Contractor had set sandbags and millings around the area for protection. A scope of

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work for additional excavation to remove more slab was provided to the Contractor by Construction Division staff. Engineering Division's understanding on site this day was that slab removal of the larger area would begin on Monday 23 May 2016 due to the need to make sure the appropriate equipment was on the job and due to forecasted rain on Friday, 20 May 2016. The roadway had been milled down to a condition that would not direct water into this feature. Figure ? below shows the start of the preparation around the area to impede water runoff. Sandbags were placed around the catch basin drainage grate.

The catch basin was located adjacent to the area of concern and was thought to have been related to the water runoff that had apparently caused the problem. Water flowed from the catch basin along a 4-inch PVC conduit that exited the slope and ran down to the support of exaction sheet pile wall on the outside of the slope. Figure ? shows a view of this conduit as it exited the slope from the new barrier wall work platform below this area. This conduit appeared to be plugged and did not appear to be carrying water at this time. Also, due to excessive settlement of this area, the conduit appeared to be sloped in the wrong direction. Water did exit the rip rap at the new barrier wall work platform below this and other areas. Water appeared to be related to runoff due to rain.

Other factors that appeared to be driving this event were cracks that are present on either side of the old ICOS wall. The cracks on the upstream side of the ICOS wall had been tracked for several years and were one of the factors for the Roadway Restoration Project. The new roadway surface was graded such that runoff will not be concentrating and flowing toward these features. Milling of the asphalt pavement exposed existing cracking at the edge of the downstream guide wall and the ICOS wall. This crack was, however, protected by the roadway and appeared to be caused by differential settlement. The ICOS wall, built between 1975 and 1979, is founded in rock and the embankment has settled, and separated from beneath the guide wall slab. Cracks are visible in Figure ?.

On Friday 20 May 2016, it did indeed rain. The Contractor continued with their asphalt milling operation but did not perform any construction activities related to the void. There were no construction activities related to the void over the weekend.

Construction activities related to the void resumed on Monday 23 May 2016. Most of the day was devoted to saw cutting concrete. See Figure ?. The saw cutting was subcontracted out to a concrete cutting company from Elizabethtown, Kentucky. However, some slab removal continued to be performed by Bluegrass Contracting on this day.

On 24 May 2016 the area was again inspected by Michael Brown and Vanessa Bateman. Bluegrass Contracting continued their concrete slab removal exposing the roadway sub grade. Once the sub grade material was exposed, engineering personnel were able to better observe the void. See Figure ?. The void extended beneath the work platform slab and partially underneath the old ICOS guide wall. It also extended to the right of void approximately 15 feet and to the left of void for approximately 23 feet at varying depths. The maximum depth was approximately 7 feet. However, loose unstable clay material was undercut excavated to a maximum depth of 9 feet directly under the location of the catch basin as shown in Figure ?. It appeared as if water had entered from the existing longitudinal cracks at the surface and through the catch basin area. Water then exited through the ICOS wall into the upstream rip rap slope.

Prior to Engineering Division staff departure, a scope of work was developed in consultation with the Contractor, Construction Division staff from the Resident Engineers Office, and Engineering Division staff. Flowable fill was chosen as the backfill material, and a mix design was developed in conjunction with the above and the concrete supplier.

On 25 May 2016 flowable fill was placed beginning around noon. See Figure ?. Dylan Grissom monitored the placement due to an early departure by Michael Brown caused by illness. Upon Mr. Grissom's arrival to the site at approximately 1400, he met with onsite USACE staff who briefed him on the work previously completed. Bluegrass Contracting had completed placement of four trucks of flowable fill, each delivering 8 cubic yards, for a total of 32 cubic yards. The flowable fill material consistently produced a slump of 8 to 8.5 inches.

Prior to fill placement, two 3-inch diameter vent bore holes were drilled (See Figure ?) through the existing concrete to aid penetration of flowable fill into the void. The bore holes (looking downstream) were located at the left and right extremes of the void. A concrete vibrator was used to aid movement of flowable fill beneath the slab and into the void. See Figure ?. Contractor personnel paid special attention to the edges of the excavation near the guide wall to ensure the flowable fill was entering the void to the best extent practical. As the vibration progressed, air bubbles were observed coming to the surface, indicating that the procedure was aiding in consolidation and movement of the fill into the remaining voids. Hand tools were also utilized to force the material into harder to reach areas.

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After vibration operations, the core holes were checked with a flashlight to observe whether flowable fill was entering the space below. The left core hole did show flowable fill in the bottom. The right core hole did not have an appreciable amount. However, there was some apparent mortar observed in the bottom. This could have been dropped off the bottom of the vibrator, as the Contractor placed it down the core hole momentarily, or it could have seeped through the open graded gravel below. However, substantial movement of flowable fill was not observed into the right core.

The Contractor placed three additional 8-cubic yard loads of flowable fill to cap the excavation. As each truck was placed, the crew worked the material with the concrete vibrator to ensure even placement and consolidation of the fill.

The final placement and consolidation was completed around 1715 as a rain shower moved through the area. Figure ? shows the site condition as work was completed for the day.

III. CLOSING: LESSONS LEARNED

The primary lesson learned from this case study is that leaving temporary construction features in place, both original to the project and those used during any subsequent dam modification, should be avoided whenever practicable. In the case of Wolf Creek, both the tower crane as well as the remnant features from the 1970s-era barrier wall contributed to improper compaction of the backfill resulting in cracking and differential settlement along the dam crest. This settlement would likely have occurred to some degree without traffic loads, but the cyclic loading of passenger cars and semi-trucks transitioning from the concrete dam section onto the embankment in this area exacerbated the problem. Installation of an approach slab at the time of the ICOS wall construction as opposed to the use of flexible pavement may have reduced the total amount of settlement over the subsequent decades.

Remnant features should be identified and carefully considered during risk assessments, particularly those involving embankment dams. Irregularities within the embankment can focus stresses for settlement and create preferential pathways for development of internal erosion related failure modes. These features may not always be readily apparent on construction drawings, so it is important to cross-reference completion reports or construction records to aid in identifying features.

Some examples of potential failure modes that could be associated with remnant features in embankments are differential settlement leading to transverse cracking and concentrated leak erosion; development of a low stress zone along an embedded upstream to downstream oriented feature also leading to concentrated leak erosion; differential settlement leading to a reduction in height of the damming surface and increased frequency of overtopping.

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construction engineering, engineering risk assessments, interim risk reduction measures (IRRM), and dam and levee safety modifications. Notable projects include Project Engineer for Wolf Creek Dam Foundation Remediation and Center Hill Dam Foundation Remediation projects, both were DSAC-1 dams. Dam safety engineer and FOB construction supervisor for the Mosul Dam Task Force which provided technical assistance to the Government of Iraq to mitigate dam safety issues associated with Mosul Dam.

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Dylan Grissom serves as the Risk Cadre Lead for the USACE Nashville District. Dylan is a licensed professional engineer in Tennessee and holds a Bachelor of Civil Engineering from Auburn University and a Master of Science in Risk Management from Notre Dame of Maryland University. He has over 14 years' experience with USACE as a geotechnical engineer, working on the Wolf Creek and Center Hill Dam major rehabilitation projects, the Kentucky Lock addition, and numerous flood damage reduction studies. In his current role, he leads a team of professionals in dam and levee risk analysis on infrastructure projects across the US.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

Liquefaction Triggering Analysis of Construction and Long-Term Surcharge Loading on Earth Embankment Dams

CARLIN GRUNDEMANN, P.E., GEI CONSULTANTS, INC.

ABSTRACT

Following the failure of the Edenville Dam, reconstruction design efforts included an updated Potential Failure Modes Analysis (PFMA). New Potential Failure Modes (PFMs) were identified related to static liquefaction triggered by a variety of load cases. These load cases included surcharge loads from flooding, construction equipment crest surcharge loads, toe excavations and long-term maintenance equipment crest surcharge loads. PFMs were related to static liquefaction of the embankment soils and foundation soils. The PFMA team identified these loads as potential load cases that could lead to failures but did not evaluate if the magnitude of those loads were sufficient to trigger static liquefaction. Therefore, the reconstruction design team checked for a strain weakening response due to the change in stress to determine if that load is sufficient to weaken the soils (e.g. trigger liquefaction). This evaluation step is an important intermediate step necessary to rule out PFMs or address potential failure concerns in existing and newly constructed earth embankment dams when foundation soils and/or embankment soils are susceptible to static liquefaction.Reconstruction design for Secord Dam, Smallwood Dam and Edenville Dam followed this process for diligent consideration of these types of failure modes and to be critical of a load's ability to trigger static liquefaction. The geotechnical data (boring logs, cross sections, lab data, etc.) from those three dams and a discussion on the analysis approach is presented.

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An Evaluation of Slope Stability of a Reclamation Dam using Three-Dimensional Limit Equilibrium Analysis

TYLER QUICK, BUREAU OF RECLAMATION

ABSTRACT

Three-dimensional (3D) effects can have significant impacts on the stability of earthen embankment dams. These effects can be dramatic for dams in narrow valleys, dams where geometry or material properties vary over short distances, embankment-abutment contacts areas, and sites with very weak layers. 3D effects are generally neglected in the traditional two-dimensional (2D) slope stability analyses. Two-dimensional analysis assumes a plane-strain condition that is not representative of most embankments. However, most assessments are still done using 2D methods, with 3D effects generally only considered qualitatively. While the conservative estimates of factor of safety (FS) produced by 2D analyses are generally considered acceptable in practice, overestimating the potential for slope instability can lead to overdesign and increased costs when designing new dams or modifications to existing dams. For this reason, assessing 3D effects is important when estimating the risk of dam instability and the need for dam modifications. To demonstrate the influence of 3D effects on stability, a case study is presented in which slope stability of a Reclamation Dam was evaluated using both 2D and 3D methods. Materials in both the foundation and dam embankment have been found to be susceptible to significant strength loss during earthquakes. As a result, post-seismic stability of the dam is a concern. 2D post-seismic stability analyses performed using limit equilibrium methods (LEM) and finite difference methods (FDM) indicated significant potential for post-seismic instability at the dam (FS < 1). Because the dam is located in a narrow canyon, and it was considered worthwhile to assess potential 3D effects on stability of the dam. To this end, 3D LEM analyses were also conducted. Results were compared to results from the 2D LEM and FDM analyses to evaluate 3D effects on slope stability of the dam. The FS from 3D LEM were significantly larger than those from 2D analysis, with increases as high as 100 percent compared to FS at the critical 2D section. In contrast to the 2D analysis where FS for some cases were much lower than 1, FS from 3D LE analysis were all greater than 1, indicating post-seismic stability would likely be acceptable. As a result, while 2D analyses suggest potential risk of post-seismic stability at the dam, incorporating 3D analyses into evaluation of this dam could lead to smaller estimates of post-seismic risk. This analysis demonstrates the importance of considering 3D effects when evaluating slope stability of earthen embankments.

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DAM SAFETY PALM SPRINGS
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The Impact of Vibration Loads on Liquefaction Susceptible Soils in Earth Embankment Dams

CARLIN GRUNDEMANN, P.E., SENIOR GEOTECHNICAL ENGINEER, GEI CONSULTANTS, INC; AND HEIDI PENCE, P.E., SENIOR ENGINEER, GEI CONSULTANTS, INC.

ABSTRACT

Data from grouted Vibrating Wire Piezometers (VWPZs) and a vibration monitor at Secord Dam during Steel Sheet Pile (SSP) installation are presented along with conclusions about the sensitivity of loose, saturated foundation soils from vibration loads at Secord Dam. The pore pressure response based on horizontal and vertical proximity of the load at the SSP tip relative to the VWPZ sensor is presented. This data may be used to help predict maximum porewater pressure response and assign a porewater pressure threshold for vibratory load in proximity to liquefaction susceptible soils.

I. INTRODUCTION – PROJECT BACKGROUND

Following the failure of Edenville Dam on the Tittabawassee and Tobacco Rivers in Beaverton, MI, liquefaction failure modes are considered in earth embankment dams with heightened awareness. One load case that could potentially lead to liquefaction triggering is vibratory loading from construction equipment, specifically vibratory hammers for steel sheet pile (SSP) installation. Often SSP is installed as a hydraulic cutoff in granular embankments, whose soils can be susceptible to liquefaction. The pore pressure response of those liquefaction susceptible soils under vibratory loads is largely undocumented. Therefore, as part of the SSP cutoff wall installation at Secord Dam, which is upstream of the Edenville Dam on the Tittabawassee River in Gladwin, MI, a series of grouted vibrating wire piezometers (VWPZs) were installed and monitored near real-time during SSP driving to monitor pore pressure response and allow the porewater pressure to dissipate within the embankment by pausing SSP driving activities when pore pressures approached Response Values defined by the Temporary Construction Safety Monitoring Plan (TCSMP). A vibration monitor was also installed on an adjacent concrete training wall to monitor the zone of influence in which the vibrations were felt by the dam structure.

The Secord Dam is located in Gladwin County on the Tittabawassee River, a tributary of the Saginaw River, and is approximately 41 river miles upstream of the City of Midland in Gladwin County, Michigan. The facility is owned by Gladwin County and operated by Four Lakes Task Force (FLTF). Construction of the dam was completed in 1925 to provide storage and headwater level control for the purpose of hydroelectric power generation, which has since been decommissioned. From left to right[1], the Project consists of a 650-foot-long left embankment and an approximately 350-foot-long right embankment with a minimum dam crest elevation (El.) of 757.52 feet. Normal headwater and tailwater pools at the dam are El. 750.3 feet and 704.3 feet, respectively. The right and left embankments have productive longitudinal clay tile drains under both crests with transverse finger drains existing

[1] All references to left and right herein are with respect to looking in a downstream direction.

[2] All references to elevation herein are with respect to North American Vertical Datum of 1988 (NAVD88) unless otherwise noted.

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along a toe drain trench. The Secord Dam is classified as having a high hazard potential. An aerial image highlighting the main dam structures is provided in Figure 1.

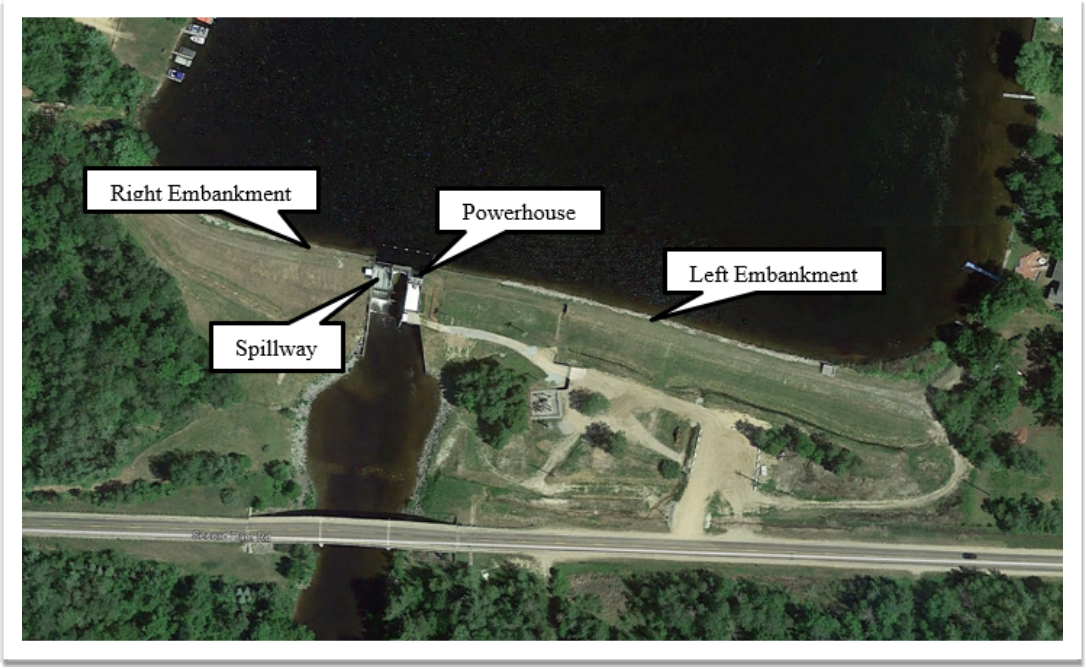


Figure 1. Aerial Image of Secord Dam.

II. LIQUEFACTION SUSCEPTIBLE SOILS

At Secord Dam, the native lacustrine soils, in both the left and right embankment, adjacent to the powerhouse were excavated down to the footing elevation to construct the powerhouse, which bears upon glacial till. Upon completion of the spillway and powerhouse construction on glacial till, the abutments were backfilled with undocumented fill to bring the dam up to the existing crest height. In the dam fill, there are loose granular fill (SP, SP-SM, SM, SC, and ML) layers with uncorrected blow counts, N, ranging from 7 to 13 bpf (equivalent clean sand corrected and normalized blow count, N1,60cs-Sr, 8 to 14 bpf). These loose to medium dense materials are finite in extents (parallel to the dam longitudinal axis) and are not continuous along the dam alignment.

III. POTENTIAL SOIL LIQUEFATION TRIGGERS

Two key components must be present for liquefaction induced strength loss instability: (1) a triggering mechanism, and (2) liquefaction-susceptible soils. Soils susceptible to liquefaction generally have three key characteristics: (1) granular soils, (2) loose soils with an uncorrected N-Value less than 10 and (3) are saturated or submerged. Given the loose nature of the granular backfill adjacent to the training walls and location of the phreatic surface, there was a potential for liquefaction identified within 100 lineal feet of the primary spillway and powerhouse training walls. Several plausible loading conditions were identified during the Supplemental Potential Failure Modes Analysis (SPFMA) that could potentially cause liquefaction triggering and undrained material strength loss at Secord Dam. Rapid embankment filling, deep excavations along the dam toe, large low frequency vibrations and equipment surcharge loads are forms of rapid loading that can trigger soil liquefaction and destabilize embankments composed of loose, saturated cohesionless materials. Specifically at Secord Dam, these loading conditions include vibrations induced during SSP installation, crane surcharge loading during SSP installation, crest gate/stop log installations, and toe excavations.

An instrumentation monitoring program was developed as a risk-reduction measure to monitor excess porewater pressure indicating potentially unstable conditions during these loading conditions. Excess positive porewater pressures can be generated by soil particle mobility, as the loose soil particles try to move into a more

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stable arrangement. If excess porewater pressure dissipates too slowly, soil liquefaction due to undrained loading may result in embankment destabilization.

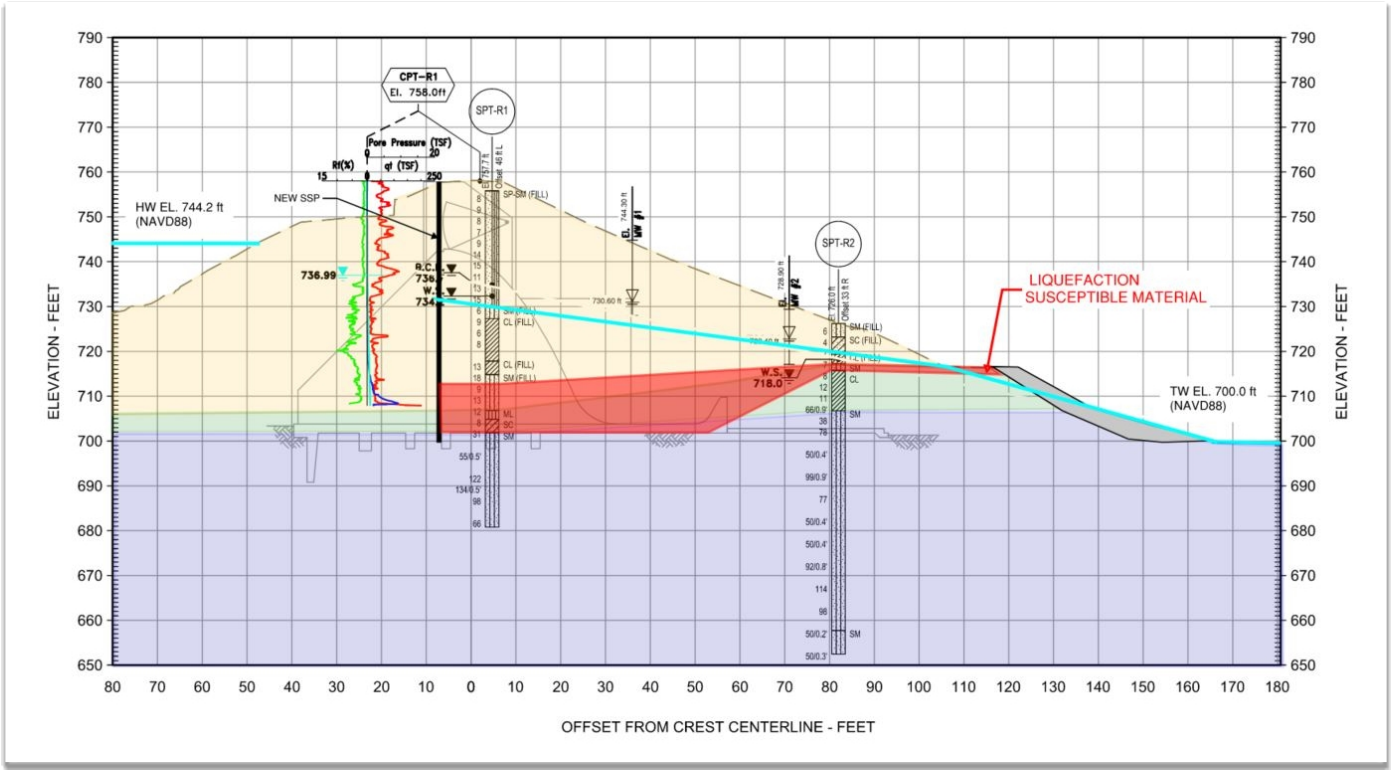


Figure 2. Cross-Section of Right Embankment

A SVR 24 VM crane-suspended vibratory hammer was utilized to drive the SSP at Secord Dam. The vibratory hammer has a variable moment (phase shifter) that adjusts the position of the eccentric masses to limit resonance. This allows the vibration case to vibrate at the maximum frequency and reduces disturbance to surrounding soils. The maximum frequency is 2300 rpm (38 Hz) with a centrifugal force of 154 tons creating a maximum eccentric moment of 2083 in.lbs.

IV. INSTRUMENTATION MONITORING

An automated seismograph was installed on the right upstream training wall to monitor vibrations observed within the concrete structure during vibratory SSP installation. Response values varied depending on the type of hammer utilized during the SSP installation. Response values were set based on industry standard limits for type of structure, age of structure, and type of construction. Instantel's Micromate and seismograph were used to automatically monitor the construction induced vibrations.

Automated grouted-in-place vibrating wire piezometers (VWPZ) and standpipe piezometers were installed approximately 5 feet from the SSP alignment within liquefaction-susceptible fill and native soils discovered during the field exploration program. The purpose of this instrumentation program was to measure the porewater pressure (u) response during pile driving operations. Response values were assigned to limit porewater pressure increases and thereby reduce the risk of liquefaction instability. Response values were assigned based on porewater pressure as a function of the overburden stress ($\gamma_s z$), or the ratio R_u.

$$\Delta u_{max,threshold} = R_u * \gamma_s z \quad (1)$$

Where R_u is the ratio of pore-water pressure to overburden stress.

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Two vibrating wire piezometers (Geokon Model 4500S-350kpa) and one standpipe observation well with a 5-ft screen were installed to monitor the change in porewater pressure based on the TCSMP Response Values. The vibrating wire piezometers and the observation well were installed in a 4-inch diameter borehole backfilled with cement-bentonite grout mixture to manufacturer specifications. The open standpipe observation well was instrumented with a hanging VWPZ. The sensor tip elevations and bottom of screen elevations are presented in Table I.

TABLE I

Instrumentation Response Values

Instrument ID	Bottom of Sensor/Screen Installation Elevation (Feet)	Baseline Elevation (Feet)	Response Values (Warning Level, Action Level, in Feet)
VWPZ1A	704.6	738.0	N/A (Installed for long-term monitoring)
VWPZ1B	735.6	744.5	746, 747.6 (3.1 ft increase)
VWPZ2	711	731.9	736.4, 740.9 (9 ft increase)
MWL-C-2	713.1	718.5	720.9, 726.6 (8.1 ft increase)

Note: Temporary Construction (Drawdown) Headwater = El. 744.2 ft

A plan view showing the location of the instruments relative to the SSP alignment is presented in Figure 3. Each SSP was given a discrete number for reference during construction, beginning with R1 adjacent to the primary spillway right training wall.

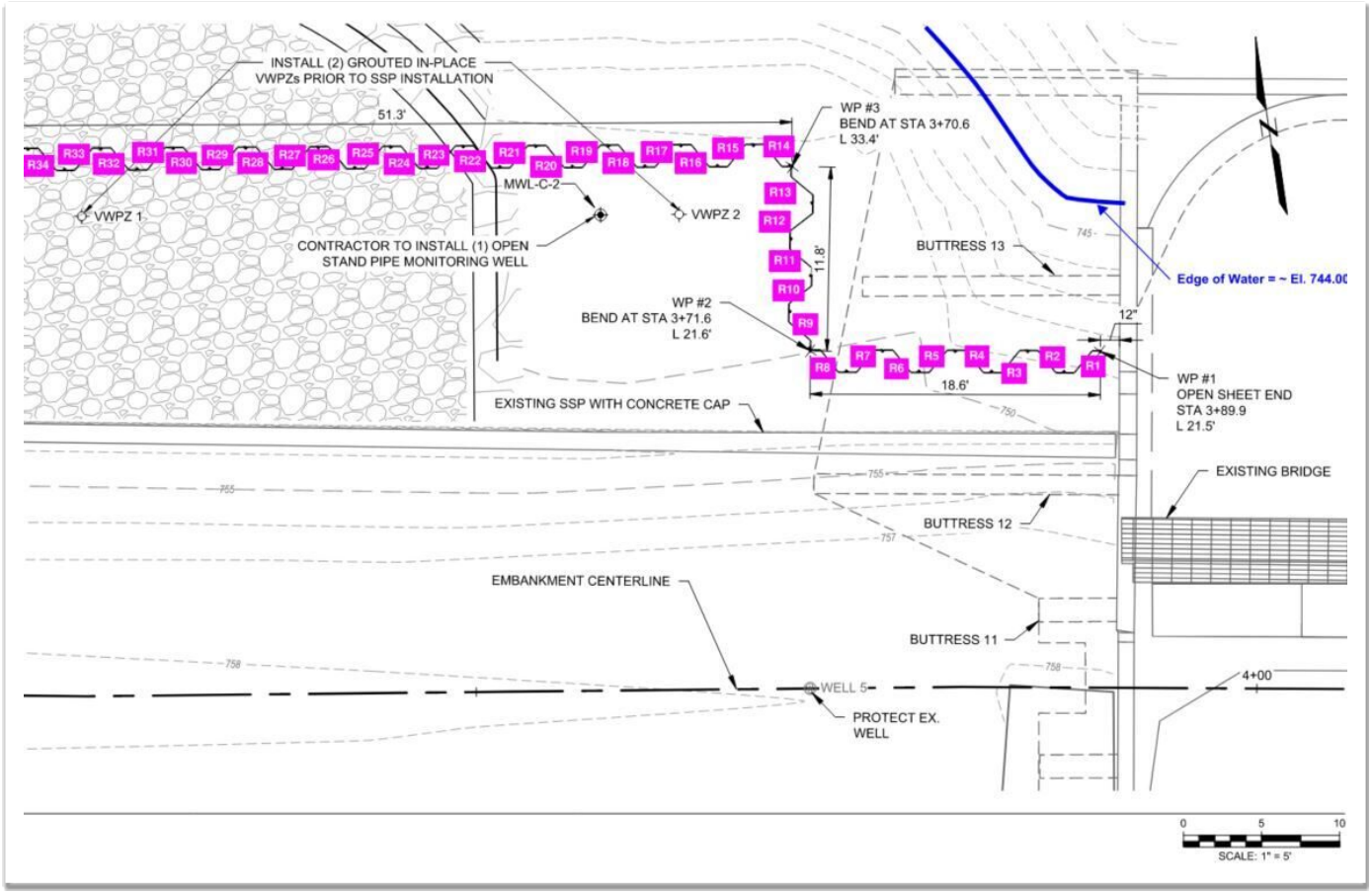


Figure 3. Instrumentation Locations and Right Embankment SSP Alignment of Secord Dam.

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VWPZ2 was installed to monitor porewater pressures within a relatively loose soil stratum near the bottom of embankment fill soils where there was a potential for a deep-seated liquefaction-type failure to occur. Observation well MWL-C-2 was installed for long-term monitoring but was useful to compare porewater pressure changes in an open-standpipe type instrument and a fully grouted-in-place instrument during construction. The nested vibrating wire instruments at VWPZ1 were to designed monitor two different layers; the embankment soils and foundation soils. All the above-mentioned instruments were installed at least a week prior to the start of SSP installation to establish a baseline under normal conditions. Figure 4 provides a profile view of the instruments in the right embankment, superimposed on a profile of the existing SSP, right training wall and buttress structure.

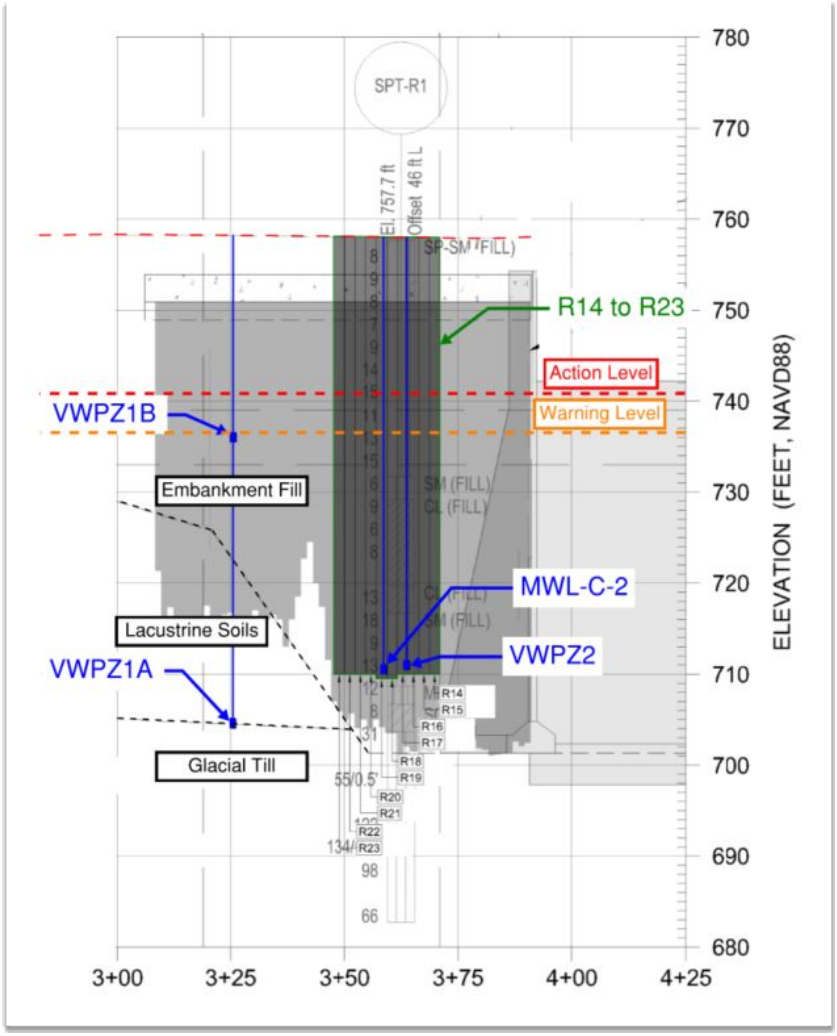


Figure 4. Profile View of Instrumentation Locations and Right Embankment SSP Alignment of Secord Dam in Elevation.

The instruments were automated using a Campbell Scientific CR6 datalogger connected to a cellular modem and an onsite visual/audio alerting system (See Figure 5).

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Figure 5. Instrumentation Automation at Secord Dam.

V. INSTRUMENTATION RESULTS AND OBSERVATIONS

The following sections present the results of the near real-time monitoring during the SSP installation. Figure 6 presents the porewater pressure responses for approximately 6 weeks representing the time when SSP were initially installed or re-struck within 100 feet of the VWPZs and standpipe.

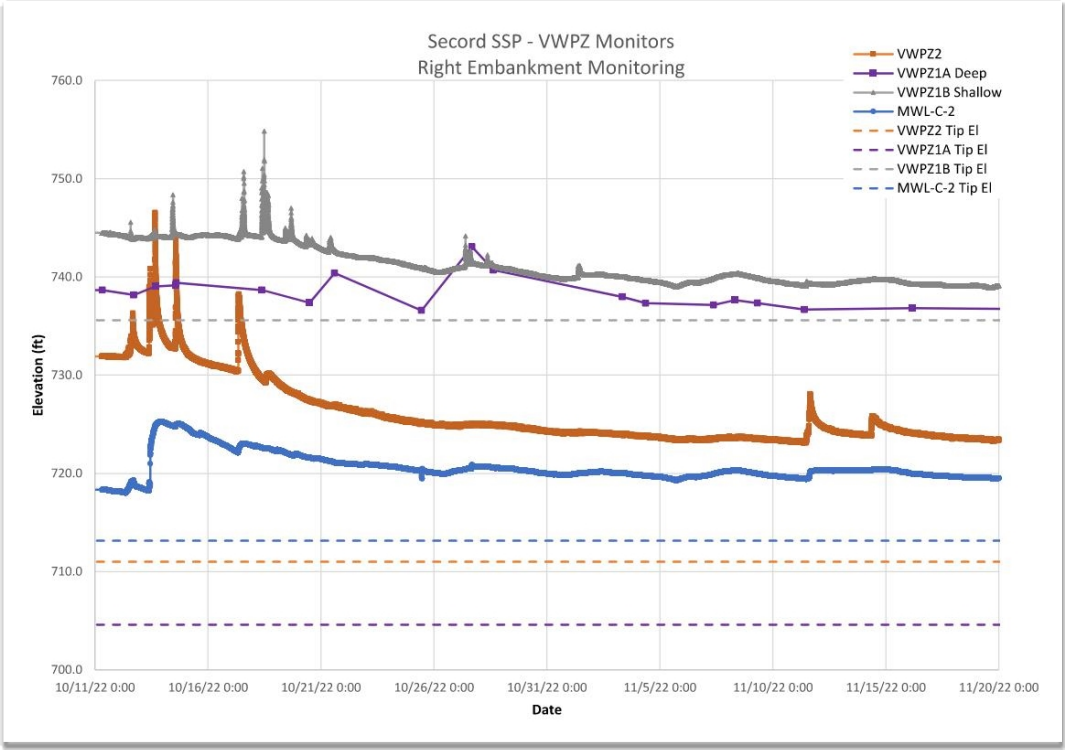


Figure 6. Time History of Porewater Pressure Response to SSP Installation and Dissipation.

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A. Comparison Between Hanging VWPZ in an Open Standpipe and Grouted Vibrating Wire Piezometer

There is a stark contrast in the responsiveness of the instruments between the open standpipe (MWL-C-2) and the vibrating wire piezometers, specifically VWPZ2. Initially the open standpipe hanging piezometer reported an increase in pore pressure and a subsequent dissipation effect when the vibratory load was applied and then stopped. At 10:38 am on October 13, 2022 during the installation of sheets R18 & R19 with the variable moment hammer, the open standpipe reported an increase from El. 718.9 to El. 721.9 (+2.0 feet) and effectively stayed at a higher elevation for the duration of the right embankment SSP installation. The initial response of the open standpipe correlated to a similar increase in the grouted VWPZ from El. 734.6 to El. 737.4 (+2.9 feet). At that time the SSP tip was 28 feet below the ground surface at El. 722 (G.S. EL. 750) which was approximately 10 feet above the installed instruments. At the peak of the pore pressure increase, the sheeting was approximately at a tip depth of 37 feet below the ground surface at El. 713 which is approximately 1 foot to 2 feet above the VWPZ2 sensor and at the same elevation as the MWL-C-2 screen.

VWPZ2 was offset approximately 5.7 feet downstream and to the left from R18 & R19 and MWL-C-2 was similarly offset approximately 4.9 feet downstream from the SSPs being installed. The responses are presented in Figure 7.

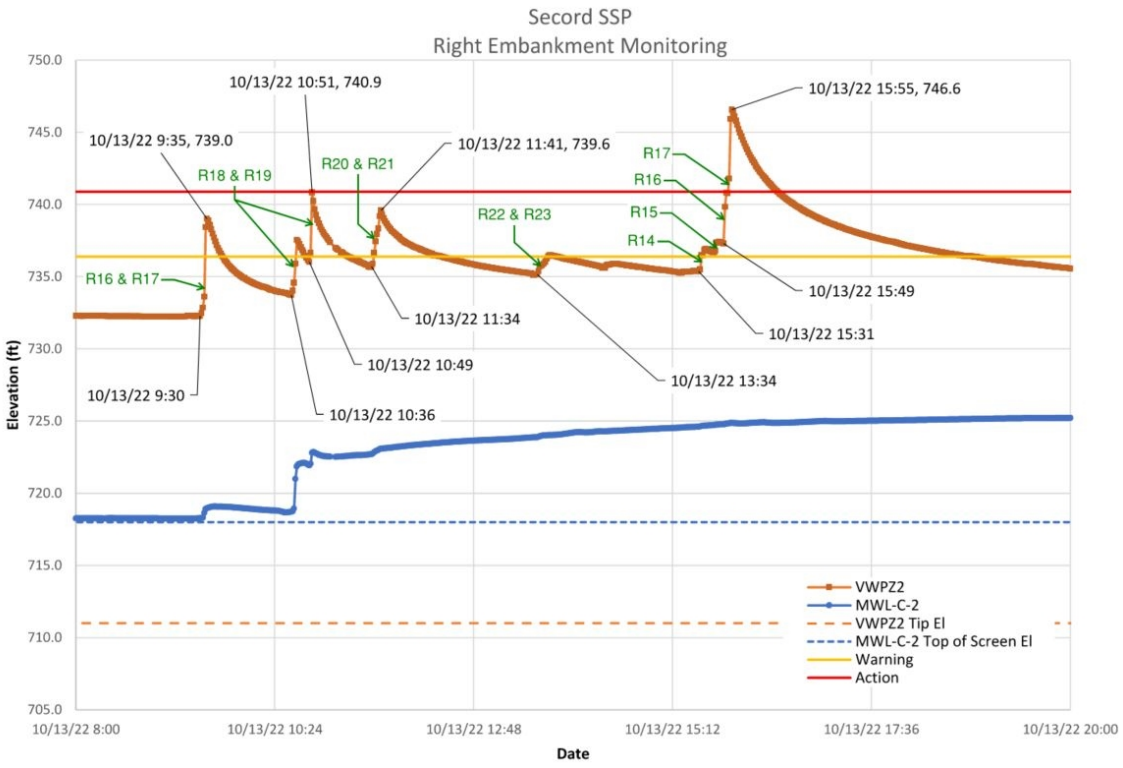


Figure 7. Comparison between the response in MWL-C-2 open standpipe and VWPZ2 vibrating wire piezometer on October 13, 2022.

B. Summary of Observations on October 13, 2022

Data from October 13, 2022 was selected for these plots and analysis given the close proximity of construction activity to the instruments and availability of onsite video footage of the SSP installation that allowed us to correlate time-history of SSP installation with sheet tip depth, vector distance to the instrumentation and embankment response. On this day multiple increases in porewater pressure were recorded by the instruments onsite upon advancing the SSP in segments. SSP driving efforts were stopped whenever a response value was exceeded to allow the porewater pressures to dissipate. During the installation of R18 & R19, the warning level was exceeded, and work was paused. After resuming the installation of R18 & R19 porewater pressures quickly increased in excess

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of the warning level however, the contractor was allowed to proceed with caution unless an Action level response was exceeded. The first action level response requiring work to stop until pressures dissipated was recorded in VWPZ2 at approximately 15:53 during the use of the variable moment vibratory hammer. The tip elevation of sheet R17 was at approximately 702.5 NAVD88. VWPZ2 is offset 4.6 ft downstream from SSP R17 with an installed elevation of 711. Figures 8 and 9 illustrate the resulting porewater pressure changes.

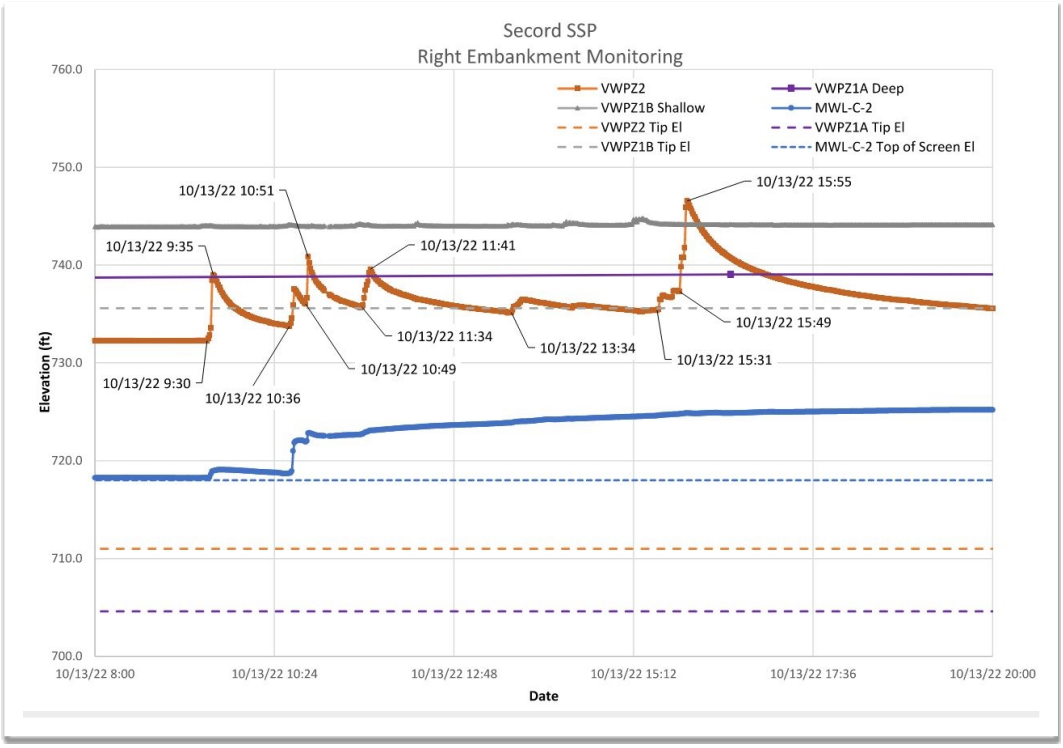


Figure 8. PWP Response in all VWPZs on October 13, 2022

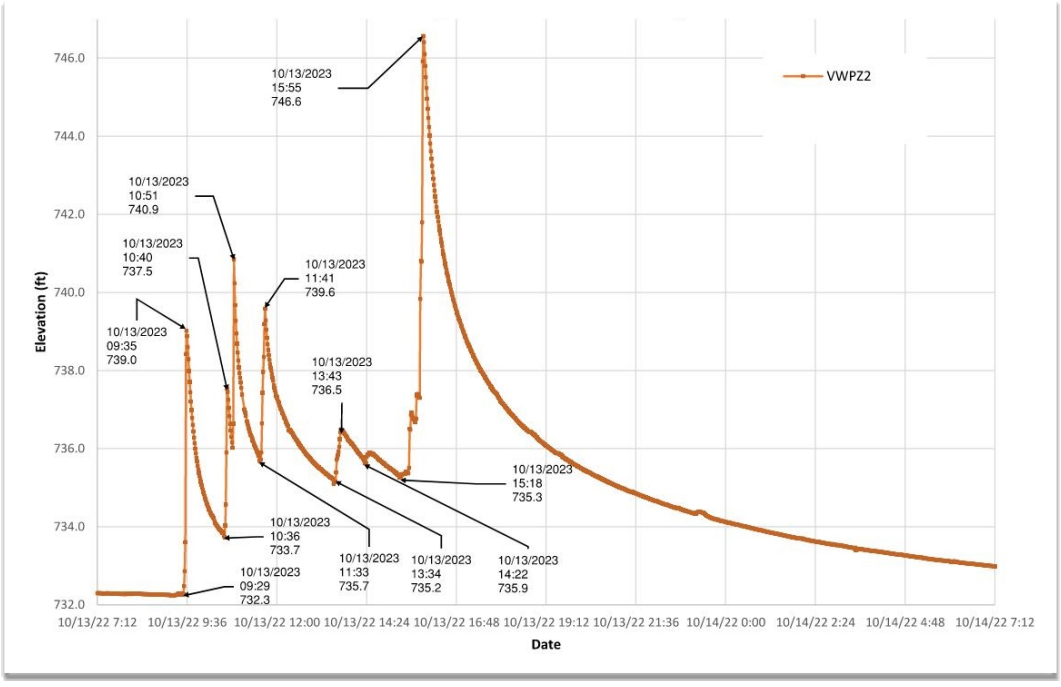


Figure 9. PWP Response on October 13, 2022 and October 14, 2023, VWPZ2 only.

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C. Interpretation of Data

A series of SSP installations were tracked in detail to monitor the pore pressure response in relation to tip depth of the sheet, as a function of the vector distance. Data from October 13, 2022 was used to illustrate the responses for several sheets installed in close proximity to the VWPZ2 sensor. The proximity of the sheet tip from Sheets R16 through R23 were plotted vs the change in pore pressure as the sheets were driven and can be seen in Figures 10 and 11 below.

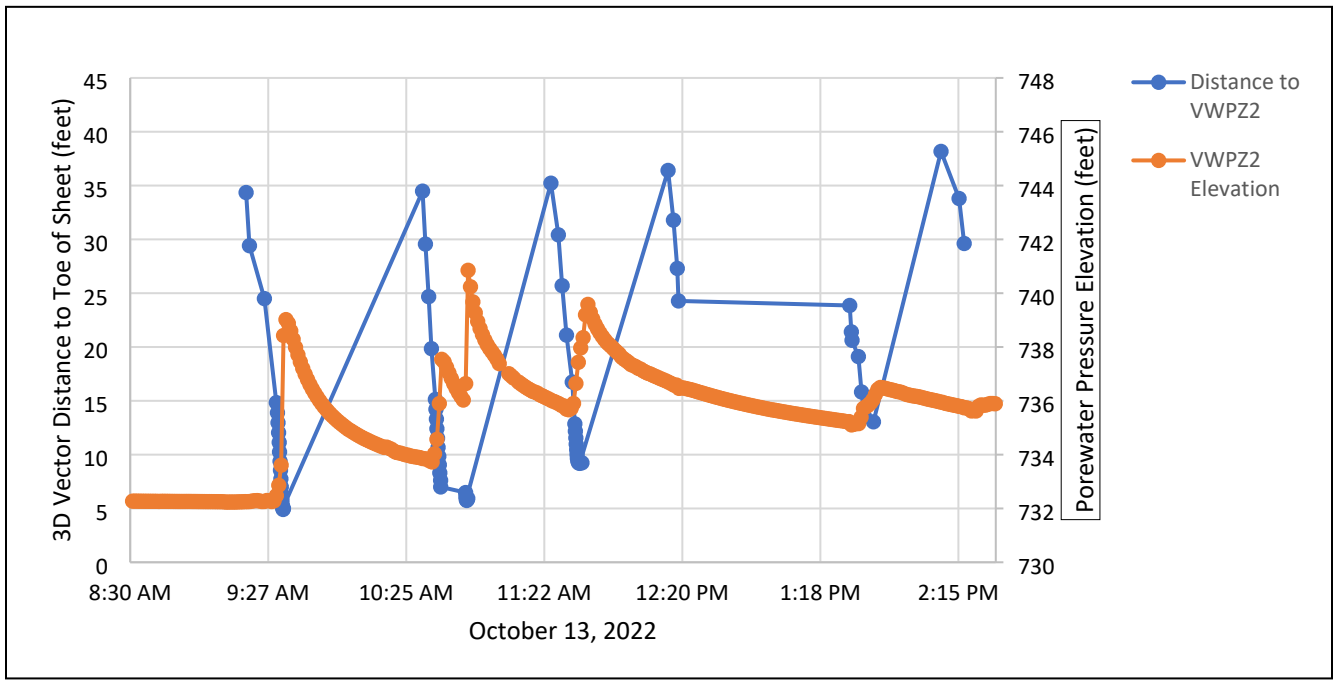


Figure 10. Change in PWP vs Distance of Toe of Sheet to Instrument

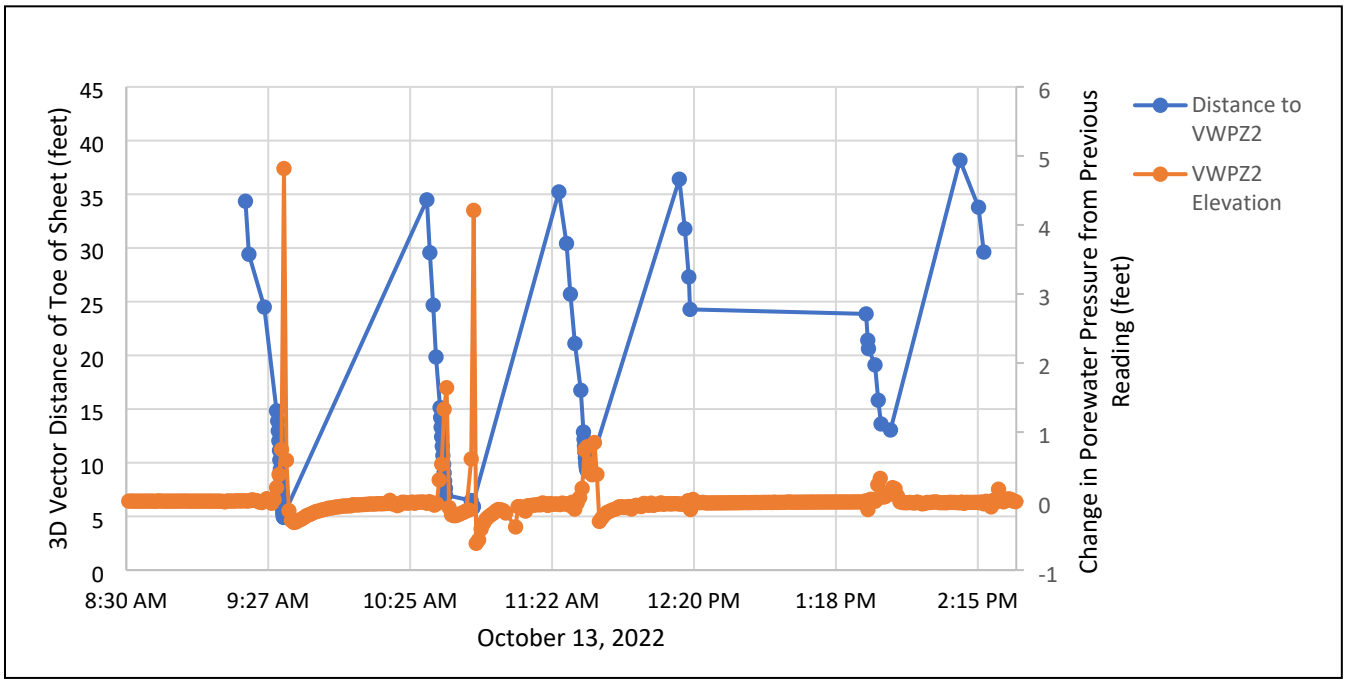


Figure 11. Change in PWP from Previous Reading & Distance of Toe of Sheet to Instrument

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Consistently, PWP begins to be impacted by the vibratory load induced upon the embankment when the sheet tip is within approximately 15 feet of the sensor. The sensor location itself is not important spatially, however it does represent a point in space that can be used as a reference point. In other words, it represents a point in the embankment where one may be interested in observing the PWP response at, say, the embankment foundation. Using that premise, it can be said that the load induced on this reference point begins to generate a PWP response at a distance, B', in this case within approximately 15 feet, as shown in Figure 12 below.

The change in PWP (Δu) increases exponentially as the sheet tip approaches the sensor elevation (vertical distance). When the sheets are within ~5 feet vertically the Δu begins to exhibit an exponential increase in change in PWP. It could be said that outside of this 5-foot range, the Δu behaves linearly and that the horizontal:vertical impact is 1:1.

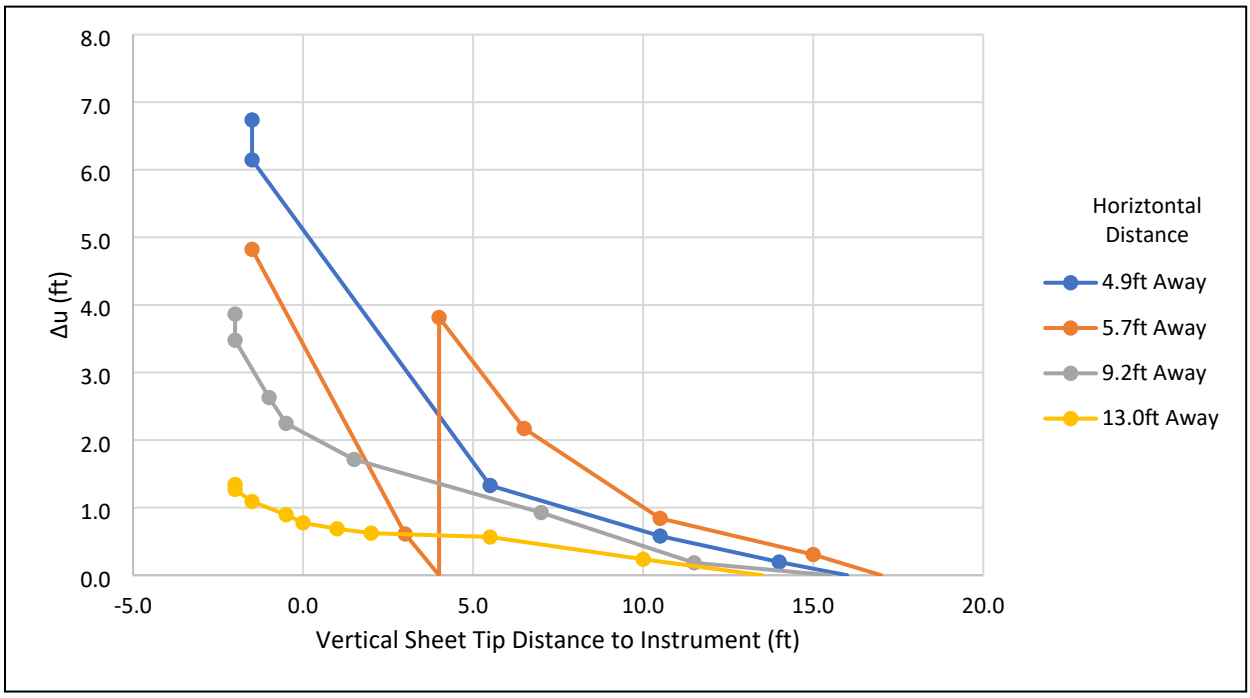


Figure 12. Change in Pore Pressure vs. Vertical Tip Distance

Sheets R18 & R19 (horizontally offset 5.7 feet) caused a porewater pressure response in VWPZ2 that exceeded the Action Value and triggered stop work order, thereby allowing the porewater pressures to dissipate before resuming the installation which is reflected in the data. For this reason, the sheet pair R18 & R19 have been omitted from further derivation of the data to avoid confusion. The remaining SSP pairs were driven to a 40-ft depth without pausing installation to let pore pressures dissipate.

D. Distance Factor, B'

To further understand the response and to see the initial response in more detail, the vector distance is broken down to vertical and horizontal components. The components of the vector distance are plotted when pore pressures began to increase and again at the maximum recorded pore pressures in Figure 13, below. This provides insight to the importance of distance in relationship to the initial PWP pressure elevation (vertical distance) as compared to the distance in plan view (horizontal distance).

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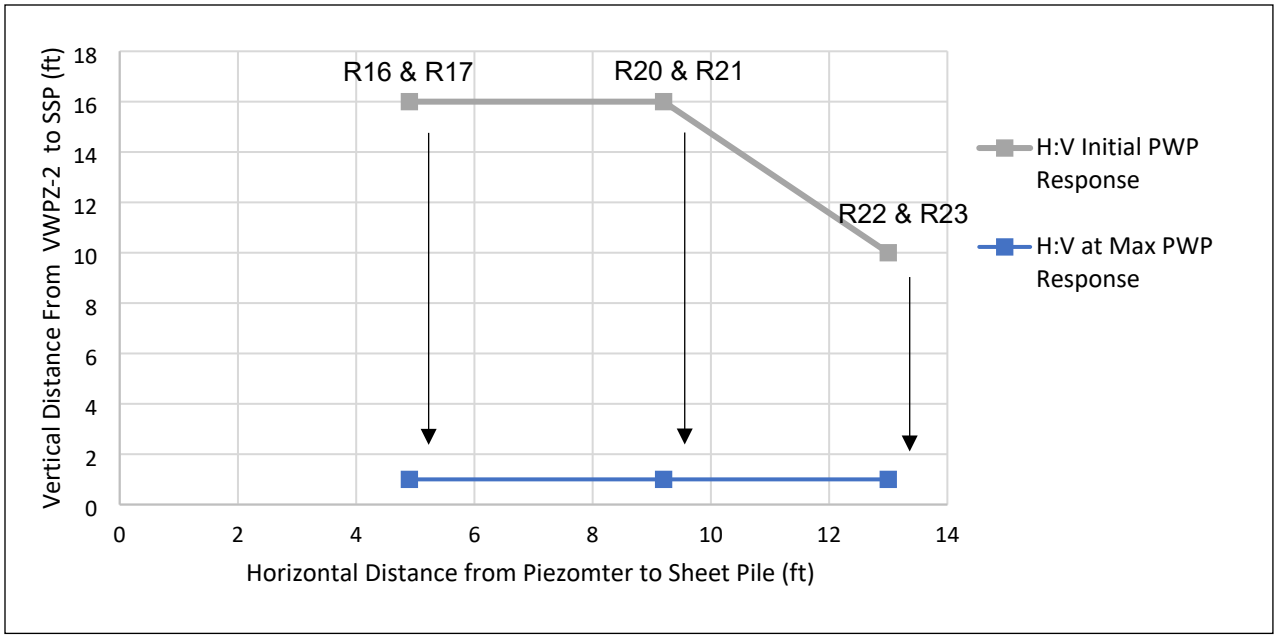


Figure 13. Horizontal and Vertical Distance from VWPZ2 at Initial and Final Porewater Pressure Response

The relationship between the horizontal distance of the SSP tip and the final change in porewater pressure (μ_{max}), can be expressed as a function of the slope of the line in Figure 14.

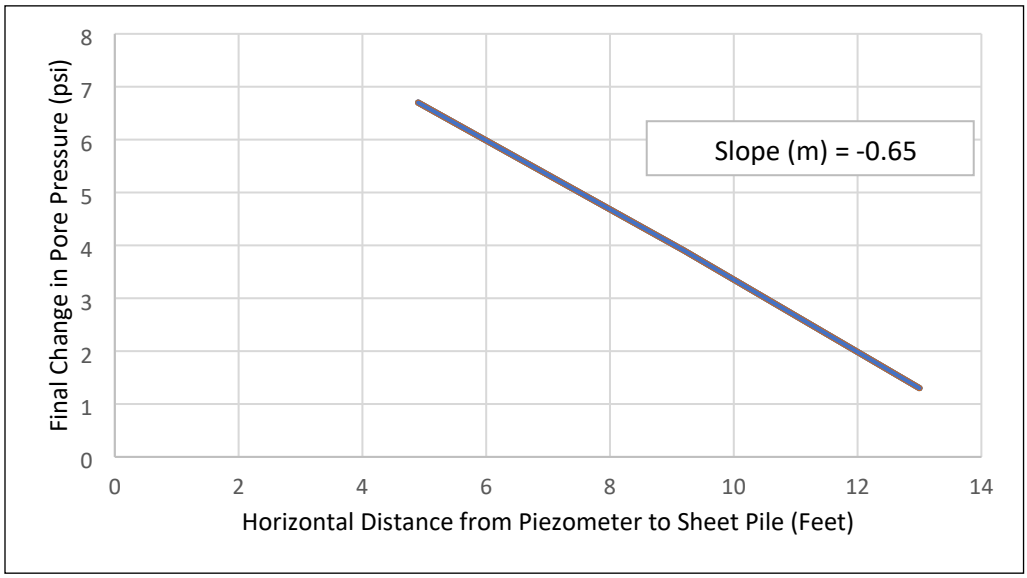


Figure 14. Horizontal Distance from VWPZ2 at Final Porewater Pressure Response

The expected porewater pressure response can be estimated based on this data using the site-specific conditions (m) as distance factor B' and the horizontal proximity of the VWPZ to the sheet pile tip (D_h).

$$\Delta u_{max} = B' / D_h \qquad (2)$$

Where $\Delta\mu_{max}$ is the increase in porewater pressure from initial.

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E. Vibrating Monitor during October 13, 2022

The vibration monitor's primary purpose was to monitor the right upstream training wall critical structure during vibration induced construction activity. During the pore pressure increases mentioned in the previous section the vibration monitor recorded a maximum particle velocity of 0.06 in/s. It is important to note that the vibration monitor was placed on the concrete training wall and was not intended to monitor for ground vibrations in the soils. The vibration monitor was at a horizontal distance of approximately 28 feet from the first sheet pile pair R16 & R17 and 45 feet from the last sheet pile pair R23 & R24 plotted in Figure 15.

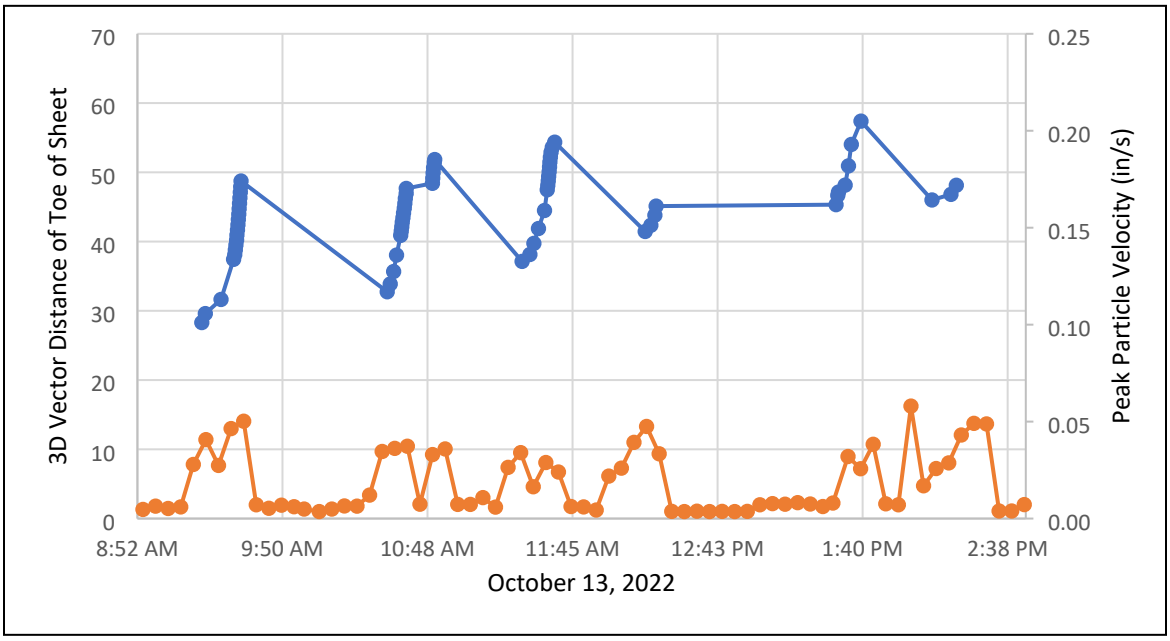


Figure 15. Peak Particle Velocity vs 3D Vector Distance of Toe of Sheet to Vibration Monitor

Figure 16 represents good correlation between porewater pressure response and vibrations transmitted through the soil to the concrete structure. There is no apparent variation in the dissipation of vibrations within the vector distance range of this data set.

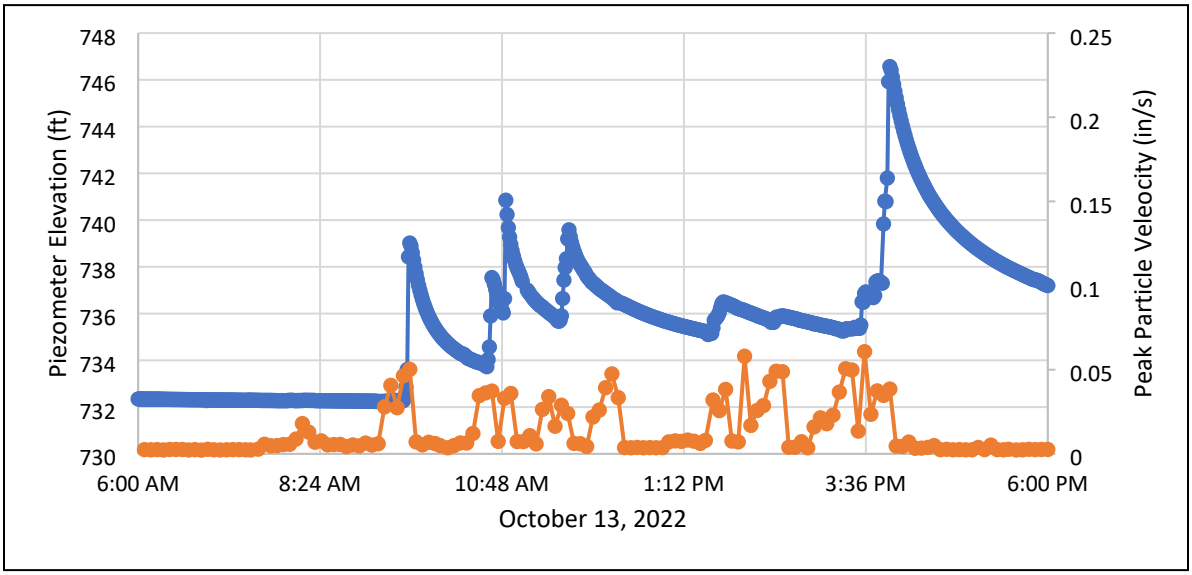


Figure 16. Peak Particle Velocity vs VWPZ2 Pore Pressure Response

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VI. SUMMARY AND CONCLUSIONS

The open standpipe piezometer acted like a weep-drain during sheeting installation. Pressure head elevation within the standpipe was slow to dissipate, therefore is not a good real-time indicator of porewater pressure response in the soil unit. The open standpipe would require bailing water that infiltrated during the SSP installation to allow the well to properly recharge to match embankment conditions. This could be done periodically but did not occur in this case due to project and safety constraints until after the SSP installation effort was completed.

To understand the correlation between SSP installation and effect of porewater pressures we offer the following key conclusions:

1. The pore pressure in the embankment at our site is impacted by the pile driving at a 3D vector distance of about 15 feet from the pile being installed. It is expected that the radius of influence will vary at other sites as a function of sand density and fines content.
2. The pore pressure rises at an exponentially faster rate near and below the same elevation being measured. This means that pore pressure rise in the embankment above the grouted vibrating piezometer is not represented at the elevation of the piezometer.
3. With these two orthogonal conditions, the pore pressure rise can be considered an ovoid with the long axis being vertical and elongating as the pile penetrates deeper.
4. Additionally, the pore pressure dissipates starting immediately after the vibratory hammer is turned off and follows a typical falling head condition. When evaluating stability, the pile diving sequence and actual production rate will define the lateral extent of the modified pore pressure conditions, especially when pore pressures are not fully dissipated to initial conditions between each pile drive.
5. With the above conclusional statements and when evaluating temporary embankment stability, it is important to consider which regions of the slip surface are impacted by the raised pore pressure and three-dimensional effects associated with a laterally limited condition along the slip surface.

R_u is a reasonable approach to define Response Values, so long as it is more conservative than the static load slope stability case and does not exceed 0.2, as a general rule of thumb. Alternatively, consider the energy dissipation affects through various soil types and various hammer energy. Data from this paper supports general alignment with the logarithmic dissipation of vibration intensity versus scaled energy ($\sqrt{(energy)}/distance$) as it relates to vertical proximity. However, we found the maximum change in pressure was generally linear in the horizontal 5-foot to 15-foot range. This relationship is of more use since for a simple analysis we would consider the full height of the ovoid influenced by the sheet pile installation.

With additional case studies and similarly applied data, a more robust correlation between vibratory source distance from liquefaction susceptible soils and the porewater pressure response can be made. However, this small data set may be used as an indicator to provide guidance for similar installations where vibratory loads are induced in embankments with sensitive granular deposits or fill. The zone of influence of the vibratory load is limited, in this case, to within 15 feet of the sensor. A site factor, C, can be applied to correlate this data and site-specific ratio of horizontal to vertical proximity magnifications for other earth embankments using a test pile and similar monitoring prior to production installation (see Figure 17). This site factor would have to be established with additional research and monitoring efforts.

To approximate the pore pressure response due to the sheet pile driving we offer the following initial algorithm.

$$B' = (E)(C) \quad (3)$$

Where B' is the Distance Factor as a function of Energy (E) and Site Factors (C).

$$u_{max} = \Delta u_{max} - u_i \quad (4)$$

Where μ_{max} is the final porewater pressure, Δu_{max} based on B' from Eq. 2 and $\mu_{initial}$ is the initial porewater pressure.

Response Values can be assigned as based on $\Delta u_{max,threshold}$:

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$$u_{max,allowable} = R_u * \gamma_s z > u_{max}$$

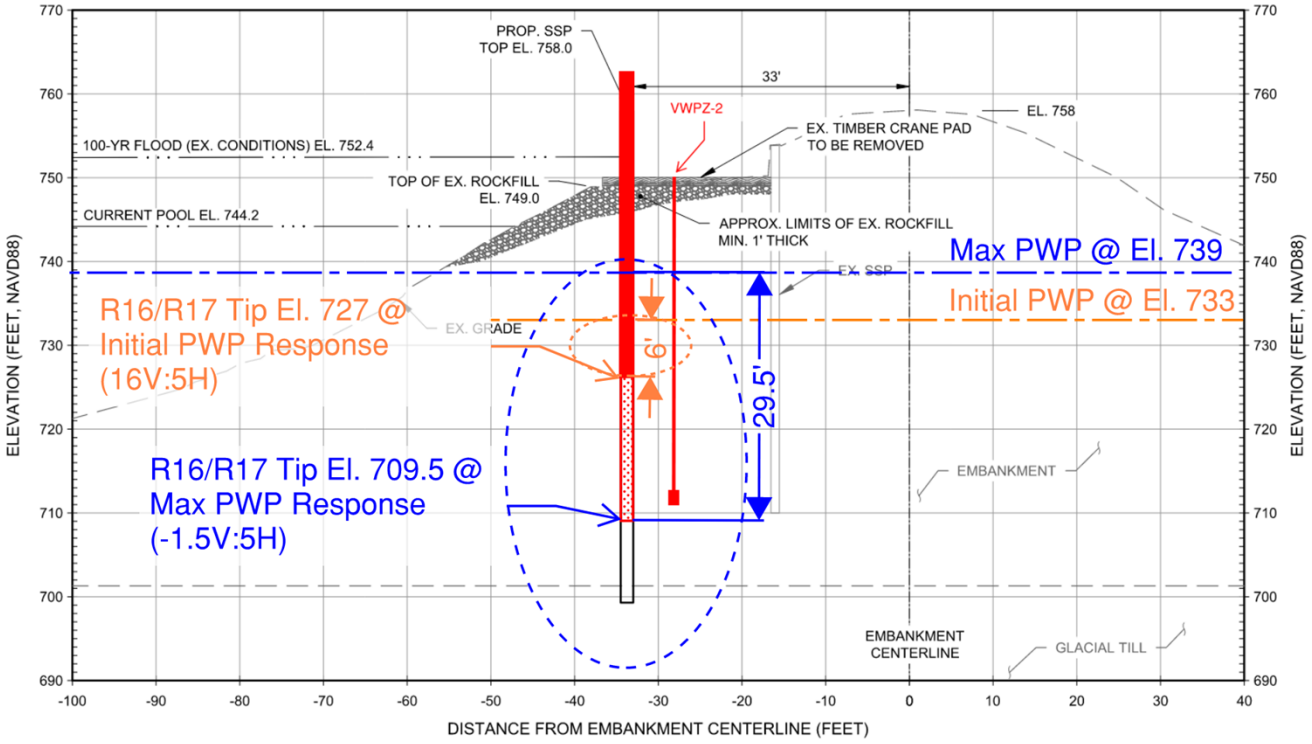


Figure 17. Cross-Section View at VWPZ-2 and Illustration of Porewater Pressure Responses at Initial and Maximum Response for R16 & R17.

Consider a diameter, equal to B', to be the zone in which a significant porewater pressure response can be expected. Depending upon the geometry of a given earth embankment section, the impacted zone of an earth embankment may or may not extend to create a continuous zone of instability. Therefore, even if those soils were to become liquified, the zone may only occupy a limited portion of the slip surface rather than the majority.

Variables to consider in application are:

1. The moment induced by the hammer used for driving the SSP.
2. The energy delivered by the hammer based on the SSP section and length dimensions.
3. The density and continuity of the soil and resulting effective dampening of vibratory energy.
4. Confining cohesive layers or other structures such as partial hydraulic barriers or portions of the spillway works which may limit drainage and dissipation of the porewater pressures.

Future study considerations:

1. Monitor the vibrating wire piezometers more frequently than 1-minute intervals to better correlate with SSP tip elevation intervals.
2. Install vibrating piezometers in an array perpendicular to the line of sheets and parallel to the line of sheets to analyze pressure radius and vector component importance.
3. Compare pore pressure increases in cohesive vs cohesionless materials.
4. Install geophones at ground surface adjacent to vibrating wire piezometer locations to establish vibration dissipation with respect to distance. Alternatively, downhole geophones could be installed at the vibrating wire piezometer elevation.

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VII. ACKNOWLEDGMENT

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Carlin Grundemann, P.E. is a registered professional engineer with 18 years of experience in design, construction, and rehabilitation of levees, earth embankment dams and stabilization of non-water retention earth embankments and bluffs. Her project work includes hydroelectric structures, earth embankments, landfills, power plants, building foundations, pavement, rivers and watersheds for both public and private clients throughout the United States. She has served as program director and/or lead design engineer for multi-year geotechnical engineering contracts with MDTMB/MDNR, MDOT, The US Army Corps of Engineers (USACE), Detroit and Chicago Districts, and the Michigan Department of Environment, Great Lakes and Energy (EGLE). Her technical background includes risk assessment, stabilization design and construction, and surveillance monitoring of levees, dams, embankments, and bluffs with loose or soft soils exhibiting surficial erosion, seepage, seepage-induced sloughing and global failures adjacent to or in-line with major bodies of water. She served as the lead geotechnical engineer for Secord Dam, Smallwood Dam and Edenville Dam rehabilitation designs for the Four Lakes Task Force, which led to this paper. She led a design team who was awarded the “2022 Governor’s Award for Historic Preservation” by the State of Michigan for stabilization of a lakeshore bluff and relocation of a historic building at

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National Inventory of Low-head dams: Completion and Next Steps

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ABSTRACT

The Task Force to create a National Inventory of Low-head Dams was established in 2020 to locate all low-head dams in the U.S. using latitude and longitude. The purpose of this paper is to describe lessons learned and next steps for the work. This 100% volunteer work effort, producing an alpha version of the inventory, is nearing completion; the final product will include a map similar to the National Inventory of Dams map with latitude and longitude at a minimum. If funded by the 2022Water Resources Development Act bill, the U.S. Army Corps of Engineers will produce a beta version of the inventory with additional information. Additionally, three proposals to define risk and improve safety are being developed.

I.INTRODUCTION

This paper describes the initial (alpha version) national inventory of low-head dams and documents its creation and summarizes the gathered data. A website showing the location of low-head dams can be found at https://urldefense.proofpoint.com/v2/url?u=https-3A__sarp.maps.arcgis.com_apps_webappviewer_index.html-3Fid-3D1cab6b600069461da86a416173a585b2&d=DwMFaQ&c=euGZstcaTDllvimEN8b7jXrwqOf-v5A_CdpgnVfiiMM&r=1ASHJfkDLutOeNRh3b7iQfoaCf-EKe_gD6U3UI_JuI0&m=HmvB6o2Nj79-kz_3Rq6BO7L-mjdZ0d8QyBb_9xULTP4&s=mFibai3yVtB_kGWgU9AqhiczL_YuIUOZTL5SfRH9OzM&e=. An overview of the current working version of this database, as of June 20, 2023, is shown in Figure 1.

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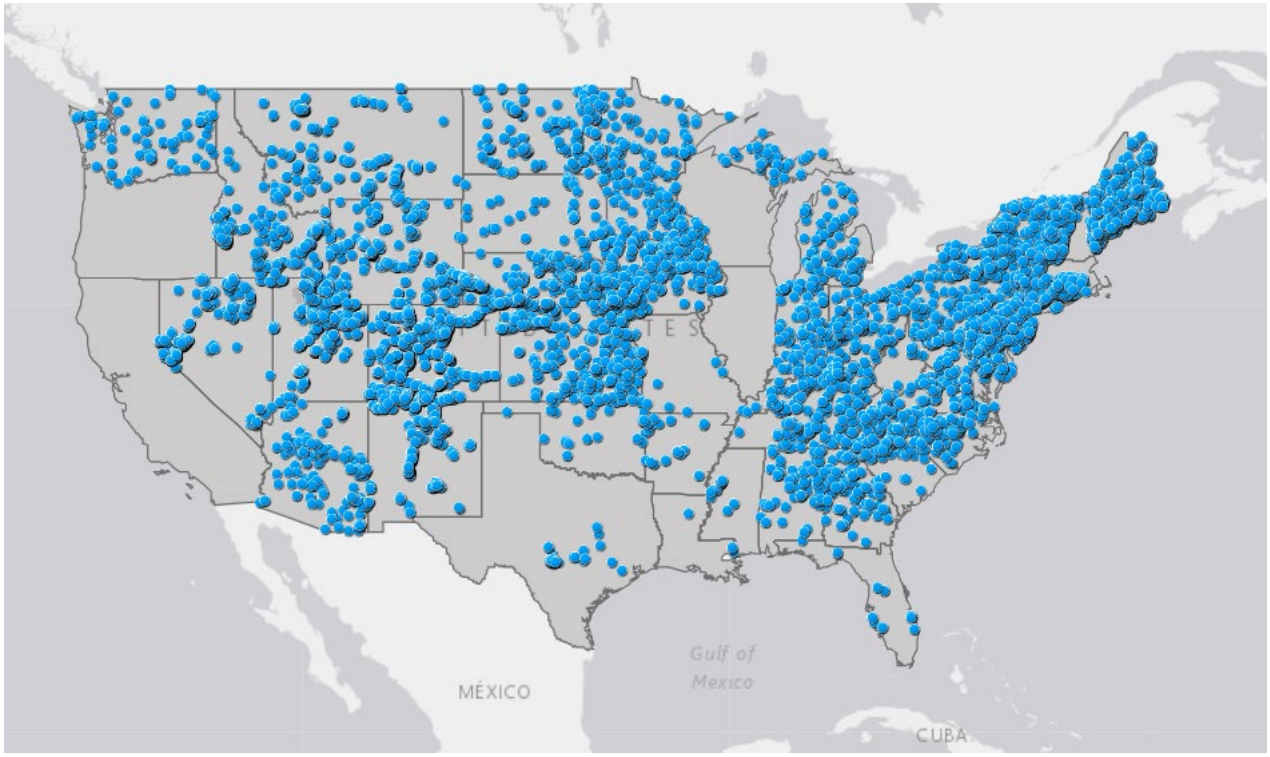


Figure 1. Map of national inventory of low-head dams, alpha version (August 2023).

The dangers associated with submerged hydraulic jumps downstream from low-head dams (LHD) are now well known [1,2]. Most of these structures, defined to raise upstream water levels for diversion, are not part of the National Dam Inventory, nor are they regulated or inspected as part of State dam safety programs, leaving them "non-jurisdictional." Until now there has been no widespread inventory of LHDs in the United States.

A National Task Force to create an inventory of LHD was created in early 2020. This task force was authorized and organized as a subcommittee under the Association of State Dam Safety Officials (ASDSO) Safety Around Dams Committee. It quickly grew to incorporate members from other organizations as well. Other contributing organizations that joined the effort to inventory the nation's low head dams included the Environmental and Water Resources Institute (EWRI) of the American Society of Civil Engineers (ASCE), and the U.S. Society on Dams (USSD). As the outreach effort grew, other organizations were also brought in as collaborators. These collaborating organizations included American Rivers (AR), American Whitewater (AW), and the Southeast Aquatic Resources Partnership (SARP). Support was also received from private consulting firms and several universities. Members of the Task Force, and their associated organizations, are listed in the Appendix.

II.TASK FORCE ORGANIZATION

Work on creating the inventory was initially carried out in five subcommittees, later reorganized, and now constituted as described in Table 1.

TABLE 1.

Task Force Subcommittees

Subcommittee	**Purpose**
States' Data and Regional Efforts (States)	Request and receive LHD inventories from state agencies
Google Earth Pro (GEP)	Locate LHDs using GEP
Smartphone Applications (Apps)	Create apps for documenting LHDs in the field
Machine Learning (ML)	Determine if ML can reliably locate LHDs

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Data management (DM)	Archive, plot, and document data

III.SUBCOMMITTEE WORK AND DATA COMPILATION

States' Data and Regional Efforts (States) Subcommittee

The States subcommittee initiated the identification of LHDs. The subcommittee was organized and officers were assigned to the subcommittee. Each subcommittee member was given several states to reach out to and request inventory data. The outreach effort started by reaching out to states with known inventories (e.g., Indiana, Colorado, etc.). Several states had already compiled information about their LHD inventories prior to formation of the Task Force. When data was requested from these states, the data was generously shared with the Task Force by each state's Dam Safety Program. Many states did not have an inventory that separated LHDs from regulatory dams, and stated that they did not have regulatory authority over LHDs in their state. As a result, data was not gathered from some states. Information from an ASDSO states survey performed in 2014 and again in 2022 were used to identify states that likely had data so that they could be contacted. While not all states were contacted, the states that were contacted and which had data were helpful in providing information. In general, the contact at most states was that individual state's dam safety program representative. In some cases, the contact at the state was a representative from the state's ecology/fish and wildlife program.

After the States' subcommittee finished their initial survey and data acquisition effort, they moved into assembling data from regional entities. This data was primarily gathered from the Southeast Aquatic Resources Partnership (SARP) and The Nature Conservancy (TNC). Both of these organizations had already developed a significant resource of stream obstruction data with the focus of restoring stream connectivity. Data in their inventories provided valuable raw materials for identifying low head dams. The data from these two organizations have been gathered and maintained over the course of many years from local agencies, state and county agencies, fish and wildlife departments, water rights databases, and other resources.

SARP "is a regional collaboration of natural resource and science agencies…developed to strengthen the management and conservation of aquatic resources in the southeastern United States" (https://southeastaquatics.net/; accessed 8 June 2023). Their foundational work has been to compile a nationwide inventory of aquatic barriers that include dams, LHD, and road crossings. Over the course of developing the LHD inventory, SARP has expanded its efforts from identifying stream obstructions in 15 southeast states to all of the 50 states as well as Puerto Rico and the US Virgin Islands. Their network of fish and wildlife partners have been crucial in identifying low head dams. SARP developed algorithms to filter their data to identify sites that had a high likelihood of being low head dams. Filtered datasets that identified the most likely candidates for LHD were shared with the Task Force and a Quality Control (QC; see below) effort was then performed on that data. Most of the states in this alpha version of the LHD inventory incorporate SARP data (Table 2).

All data gathered under the States' subcommittee was reviewed for quality control and organized by the Data Management subcommittee.

Google Earth Pro Subcommittee

It was known at the creation of the LHD Inventory Committee that not all states had inventories of the LHDs in their state and that additional data would need to be either gathered and processed or created. As a way to initially get inventories started in states without any current data, the use of Google Earth Pro was identified as a way to create new LHD inventory data. Since all 50 states have relatively good satellite imagery data, including historical satellite image data, that is available for free use on Google Earth Pro, it was an easily usable and accessible way to find LHDs. A subcommittee to use these methods to identify dams and to create data was formed. That subcommittee developed and then utilized a process to identify and document low head dams using a "yes, maybe, no" identification classification. This process was described as a "Brute Force" method. While initially it was intended to fill data gaps for states where other data could not be obtained, it later became a major source of the data found in the inventory.

Initially, the "Brute Force" method was used to identify LHDs in states where either the State's subcommittee had identified a lack of information or where it was known that there was a lack of information. The "Brute Force" method was employed by engineering students at several universities to systematically identify potential LHDs.

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Each state that the Brute Force method was used on was searched by county and those counties were aggregated into a statewide file. To make LHD identification easier, shape files of watercourses from the U.S. Geological Survey National Hydrographic Dataset (NHD) were brought into Google Earth Pro and the students followed the streamlines to find low head dams. Students attached a green pin to locations they were certain a LHD was present; a yellow pin when unsure, and a red pin for those locations that had a remote chance of being a LHD. These pin assignments were then reviewed by volunteers with extensive experience in water resources engineering and LHD identification. During that QC review, red pins were removed from the dataset, so that green and yellow pins remained. In addition to students, several private individuals and members of the Google Earth Pro Subcommittee used the Brute Force Method to identify LHDs.

Smartphone Application (Apps) Subcommittee

The Apps subcommittee created two versions of software for field use. These two software versions were initially tested and implemented for users to identify LHDs when out in the field. It was initially hoped that a widespread grassroots effort of kayakers, fishermen, and recreational water users would have access to these apps and that the data gathered by using them would augment the data obtained from other sources. However, the grassroots effort needed for data to be provided has not yet taken hold. One of their applications, which is based on ArcGIS Survey 123 (Figure 2), is available for download upon request. Minor development and testing updates to the app are also ongoing.

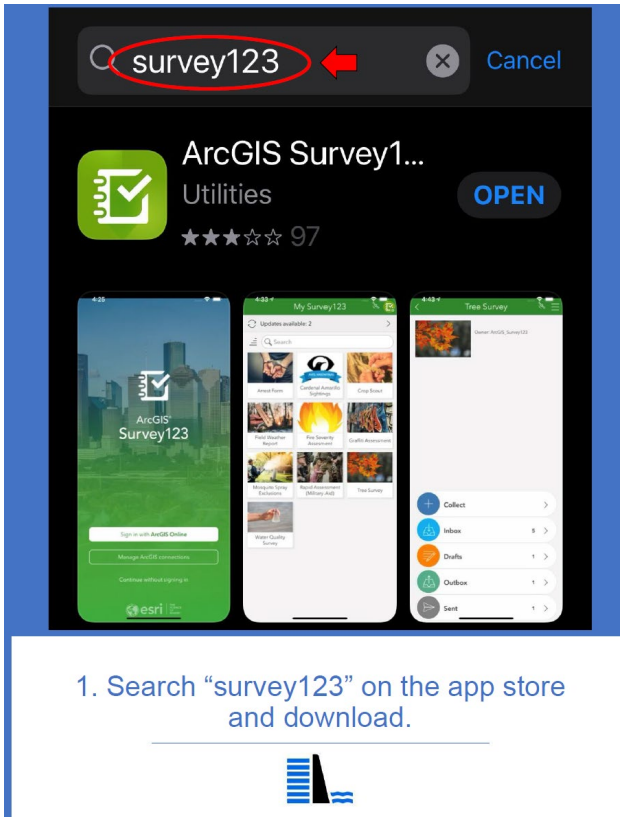


Figure 2. Introductory Page for Survey 123 Application

Machine Learning Subcommittee

The Machine Learning subcommittee was created with the plan that it would more quickly and effectively create data for later QC review than is able to be created using the Brute Force method. The subcommittee reached out to several universities, summarized current research, and collaborated with university programs as they developed different algorithms that could be used to identify LHDs. Initially, the subcommittee reviewed different sources of data that could be used for LHD identification and tested whether the dams could be reliably identified using a machine learning algorithm. Data sources reviewed included aerial imagery, LiDAR, thermal imagery, and NISAR

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(NASA-ISRO Synthetic Aperture Radar) data. Outreach was also made to Google and to Digital Globe to determine whether either would be willing to help develop a machine learning process to better identify low head dams. Neither invitation was accepted.

These efforts had mixed results. It was determined that thermal imagery and NISAR data did not provide an accurate enough level of resolution to properly identify low head dams. Some success was made in using LiDAR to identify low head dams and a proof-of-concept method was developed. The LiDAR data used required a high level of computational data and a high resolution of raw data for identification of low head dams. Since there is incomplete and often low-resolution LiDAR over much of the US, it was determined that the use of LiDAR for LHD identification would need to wait until better quality LiDAR is available over a wider area. Significant success was made in the use of aerial imagery for LHD identification. The Polis Center at IUPUI created an algorithm using high quality imagery that was able to identify LHDs with an over 90% success rate using ortho-imagery [3]. At Utah State University, a graduate student tried to also use machine learning algorithms available in ArcGIS and lower quality aerial imagery to identify LHDs. Her algorithm was able to accurately identify a majority of low head dams in the area [4]. Vinay and Hotchkiss [5] documented efforts to use machine learning and Google Engine to locate LHDs. While successful in identifying LHDs, the prohibitively high number of false positive identifications rendered the algorithm unusable. Ultimately, the machine learning subcommittee's efforts, in collaboration with others, proved that machine learning can be used for low head dam identification. However, some additional research is needed and updates to data resources including LiDAR and the National Hydrologic Dataset are needed to further identify low head dams. Efforts by the subcommittee to expand the use of the tools developed by IUPUI and USU were unsuccessful due to data limitations and a lack of funding resources.

Data Management Subcommittee

As the number of located LHD increased, a data management subcommittee was formed to organize all of the data, provide secure backups, and document collection efforts for each state. This data management subcommittee was assigned four tasks. First, it was important that the data be gathered and organized. Second, the data must be checked through a quality control process to limit duplicate, erroneous, or otherwise spurious data. Third, the data must be able to be protected against being lost. Fourth, that data must be made presentable and usable. Each of these tasks were undertaken and performed.

The Data Management subcommittee gathered data from all of the other subcommittees and organized it by state. Data was separated into "raw" data and "qc" data. In that way, the inventory includes all of the raw data that was submitted or created as well as adjustments that were made to that data and the final quality-controlled version of the data. Data is organized by date so that when new data is added it can be differentiated from data already in the database.

To develop the integrity of the data, the Data Management subcommittee, in collaboration with the Google Earth Pro subcommittee, created a method of performing quality control on the data gathered. This quality control method ultimately resulted in a single statewide file for each state having LHD data.

To protect the data, regular backups are performed onto the data and it is stored at two physical remote backup locations. These backups are performed regularly bimonthly and after major updates. Data is primarily stored on a Dropbox folder and access is limited to specific persons who are known to the subcommittee chair. Links for widespread access to the database were not created due to security concerns and some users were only granted the ability to "view" and "download" data and not to "upload" data.

To present the data, an ArcGIS Online Geodatabase was created to present both raw and QC statewide data files. The final presentation format of the data was modeled after the National Inventory of Dams (NID) so that there can be consistency for data users. The ArcGIS Online Geodatabase presents the Low Head Dam Inventory based on location. It is currently hosted by SARP.

IV.SUMMARY

As of June 20, 2023. Approximately 11,115 LHDs have been identified in the United States. These dams were identified through a collaborative, fully volunteer effort. The effort was originally created by ASDSO in collaboration with EWRI in 2020. Since the inventory focused solely on identifying the locations of these structures, they have not been grouped by risk posed to the public, miles of aquatic habitat blocked, public function, or by any other method. Some efforts to utilize and apply this data are ongoing through other agencies. The effort to identify LHDs in the United States continues. The creators of this initial (alpha version) inventory are hopeful that this inventory

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will be expanded to fill existing data gaps, that it will provide valuable information to the public and state and local agencies, and that it will help protect public safety.

V.CURRENT AND FUTURE WORK

There are two possibilities for improving the alpha version of the LHD inventory to a beta or a final version. First, the Federal Emergency Management Agency (FEMA) is in the process of creating requests for proposals for several dam safety initiatives, including three for LHDs. If funded, support will be available to add more data to the inventory, develop automatic warning communication to water recreationists, and determine the likelihood that a 'killer current' will form at LHD. Secondly, the 2022 Water Resources Development Act (WRDA) contained a provision for $30 million for the US Army Corps of Engineers to produce a national LHD inventory containing more detailed information than found in the alpha version. At the moment, the disposition of the funding is unknown.

It is currently anticipated that the existing LHD Inventory will be expanded to be used by the FEMA and USACE efforts described above. Since the status of these efforts are still being developed, the alpha version of the inventory may have additional data added to it. It is also intended to be presented on the ASDSO Safety Around Dams website and distributed to national fire and rescue agencies so that water users and first responders can be made aware of potential hazards. Other agencies, such as fish and wildlife agencies, dam owners, and state agencies may also use the data for prioritization of rehabilitation or removal projects. Generally, the inventory data gathered during this effort is intended to be a resource to persons and agencies interested in protecting public safety and improving river ecosystems.

TABLE 2.

Inventory Data Source by State

STATE	DATA SOURCE		
	State Inventory (Dam Safety/ Ecology Dept)	Google Earth Brute Force Methods	Obstructions (SARP/TNC)
Alabama			X
Alaska			
Arizona	X	X	
Arkansas			X
California			
Colorado	X		
Connecticut		X	X
Delaware	X	X	X
District of Columbia		X	
Florida			X
Georgia			X
Hawaii			
Idaho		X	
Illinois		X	
Indiana	X		
Iowa	X		
Kansas	X	X	
Kentucky	X		X
Louisiana			X
Maine			X
Maryland			X
Massachusetts			X
Michigan		X	
Minnesota	X		
Mississippi			X
Missouri			X
Montana		X	
Nebraska	X	X	
Nevada	X	X	
New Hampshire			X

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New Jersey		X	X
New Mexico		X	
New York			X
North Carolina	X		X
North Dakota	X		
Ohio	X		
Oklahoma			X
Oregon	X		X
Pennsylvania	X	X	X
Rhode Island		X	X
South Carolina			X
South Dakota			X
Tennessee	X		X
Texas			X
Utah		X	
Vermont		X	X
Virginia	X		X
Washington		X	X
West Virginia			X
Wisconsin		X	
Wyoming		X	

APPENDIX: Members of National Task Force to Create a National Inventory of Low-head Dams

Position	Name	Firm or Agency
Taskforce Officers		
Chair	Rollin H. Hotchkiss	Brigham Young University
Vice Chair	Manuela Johnson	Christopher B. Burke Engineering, LLC
Vice Chair	Brian Crookston	Utah State University
Vice Chair	Jessie Thomas-Blate	American Rivers
Vice Chair	Hattie Johnson	American Whitewater
Secretary	Jacob Nielsen	Hansen, Allen & Luce, Inc.
Subcommittee Chairs		
States' Data and Regional Efforts	Stephanie Williams	HDR
Google Earth Pro	Benjamin Devadason	Gannet Fleming, Inc.
Smartphone Applications	Lisa Dietrich	GHD
Machine Learning	Manuela Johnson & Karl Kingery	Christopher B. Burke Engineering, LLC & Hydrometrics, Inc.
Data Management	Karl Kingery	Hydrometrics, Inc.
Task Force Members		
Member	Ana Avendano	Golder Associates Colombia
Member	Andy Kulik	Gannett Fleming, Inc.
Member	Benjamin Israel Devadason	Gannett Fleming, Inc.

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Member	Beth Knudsen	Wright Water Engineers, Inc.
Member	Brandon Lanthier	Geosyntec Consultants
Member	Bruce Rogers	USACE (Retired)
Member	Caitlin Arnold	Utah State University
Member	Chad Ballard	Plummer
Member	Chuck Davis	Beaver Creek Hydrology
Member	David Hernandez	HDR
Member	Desiree Tullos	Oregon State University
Member	Edward Kern	Stantec
Member	Elizabeth Landowski	Gannett Fleming, Inc.
Member	Jacob Davis	USACE
Member	Jarlene Ninive	HDR
Member	Jon Jones	Wright Water Engineers, Inc.
Member	Jordan Keeney	Stantec
Member	Karl Kingery	Hydrometrics
Member	Ken Smith	State of Indiana
Member	Kevin Nielsen	Jacobs
Member	Lisa Dietrich	GHD
Member	Marc Schneider	Worthington Products, Inc.
Member	Matthew C. Lindon	Loughlin Water Associates
Member	Nicole Mee	Wade Trim
Member	Paul Schweiger	Gannett Fleming, Inc.
Member	Samuel Plaza	Wright Water Engineers, Inc.
Member	Shin Miyawaki	Jacobs
Member	Stephanie Williams	HDR
Member	Travis Ford	HDR
Member	Tony Wahl	USBR
Member	Kathleen Hoenke	SARP
Member	Dennis Richards	AAWRE
Member	Mark Killgore	Commonwealth of Virginia
Member	Lee von Gynz-Guethle	West Consultants
Member	Paul Cocca	USACE

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SEPTEMBER 17-21, 2023

In a Room Full of Experts… Tips for Success from PFMA Recorders

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ABSTRACT

In planning and preparing for potential failure mode analysis (PFMA) and other risk analysis (RA) workshops, emphasis is often put on the selection of the facilitator and subject matter experts, with lesser attention given to the selection of the note-taker/recorder. However, the role of the recorder is critical to the success of the workshop and the final report documentation used by dam safety decision-makers. The recorder is often responsible for not only capturing the expert elicitation discussions, basis, and conclusions of the PFMA or RA, but also for efficiently locating supporting project information pertinent to the ongoing discussions during the workshop. This presentation will summarize input from a variety of mid-career to senior engineers that have served as PFMA or RA recorders and facilitators to identify common "best practices" and tips on "what works" for recorders, as well as key qualifications of an ideal note-taker/recorder. The presentation will also include perspectives from dam owners on how recorders can contribute to a successful PFMA or RA workshop.

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DAM SAFETY PALM SPRINGS
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From Here to Recovery: The Softer Side of Effective Leadership

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ABSTRACT

Too often, leaders feel they must be strong, tough and decisive, that to admit any weakness will only undercut them and make them ineffective. Until I lost my mind on the morning of October 17, 2021, as the results of months of prescription drug abuse, I know I felt that way.But after a week on a psych ward, five weeks in rehab, 20 weeks of intensive outpatient therapy, and more than a year of sobriety, I've come to understand that all of that is exactly wrong.In this session, I'll share these hard-won lessons -- the importance of being willing to seek help, of being present in the lives of those you care about and need the most, of self-care, and so much more -- so attendees can understand the role that their "softer sides" can play in becoming more effective leaders.Participants will understand the importance of the "softer side" of leadership, receive practical tips on what constitutes the "softer side" of leadership, and learn how to hone their soft leadership skills.

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DAM SAFETY PALM SPRINGS
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2003 Fuse Plug Spillway Failure at Silver Lake Dam, Michigan: A Look Back at the Lasting Impacts

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EXTENDED ABSTRACT

The emergency fuse plug spillway at Silver Lake Dam in Michigan failed on May 14, 2003, resulting in a nearly complete release of the reservoir. The dam was modified in 2002 to increase spillway capacity with construction of the fuse plug spillway. A fuse plug spillway is an engineered earth embankment section designed to fail sacrificially to prevent failure of the main dam structure. Silver Lake Dam is located in the Dead River Basin, a tributary to Lake Superior, in Michigan's Upper Peninsula. The main stem for the Dead River extends 25 miles with five impoundments which are part of a hydropower generation system licensed by the Federal Energy Regulatory Commission (FERC) and owned and operated by a regional utility provider. Silver Lake is the upstream most impoundment in this system, and it is used for water storage and does not have power generation facilities.

The original project features include a main earthen dam with a concrete spillway, and four dikes constructed in saddles and low points along the reservoir rim. The primary spillway located at the main dike is a 100-ft long ungated concrete ogee spillway, divided into 10 bays, with a single bay having a lower concrete crest elevation that can be raised with stop logs. As noted above, the fuse plug spillway was added in 2002. Following a rainfall event 3 days prior, the combination of the following circumstances resulted in premature fuse plug operation for a much smaller storm event than was intended by the design:

- The low-level outlet released a low discharge relative to its capacity. The low-level outlet should have been fully opened based on the operational rules for the reservoir pool elevation assumed during the design of the fuse plug.
- The stoplogs in the adjustable crest of the spillway were set at an elevation higher than the fuse plug pilot channels. Given the flow conditions and pool elevations, the stoplogs at the spillway should have been removed to lower the pool elevation to increase capacity.
- The fuse plug pilot channels were set lower than the fixed spillway crest in the other 9 bays, which is a critical design failure of the overall system.

Based on the independent investigations, it is likely that had the low-level outlet been fully opened or the stoplogs been set lower, the activation of the fuse plug and the resulting failure could have been avoided for this storm event. However, even if operation of the reservoir had been conducted in a way that averted failure during the May 2003 flood, it is likely that failure still would have occurred during a future storm event. Omissions and errors in the design and construction of the new fuse plug spillway would have resulted in the activation of the fuse plug for a more frequent storm event than intended.

The elevated reservoir level activated the fuse plug pilot channel, initiating the erosion of the fuse plug. The fuse plug pilot channels were designed and constructed at a lower elevation than the concrete service spillway, resulting in premature operation of the fuse plug. Consequently, erosion of the fuse plug would have always initiated before any water flowed over the concrete spillway crest, which highlights a flaw in the design of the overall system.

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The low setting of the fuse plug pilot channels relative to the service spillway is attributed as the root cause of the operation of the fuse plug spillway. Once the fuse plug activated, the erosion did not stop at the base of the fuse plug, as envisioned in the design, but rather extended into the foundation and downstream emergency spillway channel, resulting in release of nearly the entire reservoir. The fuse plug did not include a base slab or other features to prevent the deep foundation erosion that was judged to be the root cause for the release of Silver Lake [7].

The resulting flood cascaded downstream along the Dead River toward Lake Superior and the city of Marquette. Downstream flood impacts included the cascading failure of Tourist Park Dam, evacuation of approximately 1,800 individuals, widespread damage to infrastructure, economic losses, and severe environmental consequences.

- As the magnitude of the flood was realized, Emergency Action Plans for all 5 dams along the Dead River were activated resulting in the safe and effective evacuation of approximately 1,800 individuals. In the successful implementation of the Emergency Action Plans, there were no deaths, injuries, or lost individuals as a result of the failure of the fuse plug and subsequent flooding and devastation. The 2008 article in the *Journal of Dam Safety,* "Planning for Emergencies: Lessons Learned from Silver Lake," provides an in-depth perspective and details about the response of Emergency Management agencies [6].
- The flood wave safely passed over 3 dams on the Dead River before culminating in the failure of Tourist Park Dam that was overwhelmed by the flood waters, resulting in embankment erosion and release of the reservoir. Tourist Park Dam and hydropower production was rebuilt and restored in 2012 for $4.8 million [3].
- Following the failure of Tourist Park Dam, the Presque Isle coal-fired power plant situated at the mouth of the river was flooded and became non-operational. Repairs to the power plant following the flood were completed within months, restoring an important source of electric power to the region and local communities. The power plant was subsequently shut down in 2019, as it was replaced by 2 natural gas power plants to meet carbon dioxide reduction goals [1], [4].
- There was extensive damage done to the roads, bridges, public infrastructure, and the economy totaling nearly $100 million. Nine bridges were damaged or destroyed from the flooding [5], [6].
- The torrent of water cascading down the river resulted in extreme erosion and destruction of the river ecosystem. In the 20 years since the flood, Michigan Department of Natural Resources has led an effort not only to repair damage but also restore and improve the river ecosystem through natural channel design using wetlands, vegetation, and serpentine channel redevelopment [2].
- Silver Lake Dam reconstruction was completed in 2008, and the reservoir was restored to full storage capacity in 2013 [5].

Events such as this provide the opportunity to investigate, research, and study dam failures. There were numerous lessons learned from the fuse plug failure and the subsequent flood.

- **Understanding of Foundation Erosion Mechanisms.** An understanding of the fuse plug erosion progression mechanism and the need for an erosion-resistant base for operation-as-intended are critical in the selection of fuse plug locations and design of the fuse plug structure. Limiting the depth of erosion during operation-as-intended is critical for the safe operation of the spillway.
- **Emergency Action Plans.** The successful implementation of an Emergency Action Plan was demonstrated following the failure at Silver Lake. These are effective tools for protecting the safety of downstream communities during a dam failure event and in immediate response to the situation.
- **Service Spillway Operation.** The operation requirements of the system, including the low-level outlet and stoplogs, were not well communicated, which resulted in fuse plug activation during a smaller flood than the design intended. The operation rules assumed in the design compared to those implemented following the rehabilitation were not aligned; this resulted in the operation of the reservoir pool, outlet works, and stoplogs in a manner that was inconsistent with design assumptions. Clear operation rules for pool elevation and spillway operation are essential for safe operation of the dam.
- **Auxiliary Spillway Design.** During the fuse plug activation, the stoplogs were set at a higher elevation than the crest of the fuse plug. In the configuration at Silver Lake Dam, the stoplog bay becomes the

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primary spillway only when the stoplogs are removed. Primary and auxiliary spillway crests should be carefully designed and constructed to ensure appropriate spillway activation and usage. Furthermore, the need for human intervention and operation to ensure passage of a flood through an auxiliary spillway should be avoided where possible.

- **Application of Best Practices.** The failure of the fuse plug at Silver Lake Dam highlights the need for engineers and regulators to disseminate and understand best practices information related to dam design, construction, and operation of facilities. In the design and construction of fuse plugs, non-erodible foundations are critical to safe and effective operation.

Shortly after the failure, efforts were underway to investigate the incident including those by personnel from FERC. Investigations by independent consultants also provided impartial evaluations of the incident. Through site observations and assessment of the failure, reports were developed. These comprehensive documents regarding the incident are available to the public at: https://www.ferc.gov/dam-safety-and-inspections/dead-river-project-p-10855-silver-lake-incident.

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DAM SAFETY PALM SPRINGS
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1993 Split Rock Creek Dam Failure: Can I Get an Inspection?

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ABSTRACT

The Split Rock Creek Dam is located 3 miles north of Jasper, Minnesota, and was constructed in 1938 out of Sioux Quartzite rock to create a recreational lake. The dam was owned by the State of Minnesota. In May of 1993, the Split Rock Creek flooded due to heavy rains and broke through the earthen dike on the west side of the dam. The lake was drained to repair the dam following the failure.The town of Brandon, South Dakota, located 23 miles downstream on Split Rock Creek, brought a lawsuit against the State of Minnesota for approximately $400,000 dollars for damage to its golf course, claiming that Minnesota had failed to adequately inspect the dam in keeping with state regulations. A judge from Pipestone County, Minnesota, dismissed the lawsuit, ruling that the Minnesota Department of Natural Resources was not liable because the state's regulations make the timing of inspections "subject to the availability of staff and funds". The legislature was still in session when the structure failed. Lawmakers allotted $350,000 to the Minnesota Department of Natural Resources to quickly repair the dam. Even if the lawsuit had been thrown out, this failure brings an interesting discussion on regulations and lawsuits. Where availability of staff and funds is limited, the State of Minnesota was not required to inspect the dam. The City of Brandon, South Dakota apparently was convinced that there was a defect with the dam which could be observed (and was not) and thus was a potential reason for the failure and downstream damages. However, simply performing inspections may not have prevented the failures. Would a higher frequency of inspections have prevented this failure? Or was the dam loaded beyond design limits during the flood event?This presentation will discuss the history of the failure and discuss the role that dam safety inspections play for a rural dam such as this one and lessons learned from the proposed lawsuit.

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DAM SAFETY PALM SPRINGS
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2003 Emergency at Mount Carmel Dam, North Dakota

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ABSTRACT

On March 29, 2003, a local high school student taking pictures made a discovery that helped divert a potential disaster. Substantial erosion had occurred at the main spillway of Mount Carmel Dam, which is located near Langdon, North Dakota. Flows from the internal erosion (piping) event were discharging beneath the chute spillway structure in an uncontrolled release and exiting through a large hole that had developed at the bottom of the dam embankment. Swift action was required to safeguard the remaining reservoir supply that served a population of over 5,000 people and to prevent a complete failure of the dam. Emergency response included voluntary evacuation of two families, monitoring of downstream roads and bridges that might be impacted, and staging of equipment and supplies ready to be utilized in the event of a flood. Emergency earth and sheet pile cofferdams were constructed to prevent further erosion, and an out-of-use and buried spillway conduit was reopened in order to safely lower the reservoir and avert larger consequences. Resulting forensic analyses and failure mode evaluations identified filter incompatibility, inadequate seepage cutoffs, and adverse effects due to freeze-thaw action as likely contributing factors. The damaged spillway structure was ultimately removed and replaced with one that addressed design flaws identified during the incident investigation. The incident at Mount Carmel Dam also instigated further evaluation at several similar spillway structures around the state. This case study will: (1) highlight the factors leading up to the erosion failure; (2) describe the incident, including the emergency response and investigation; (3) provide details about the reconstruction and repairs at the dam; and (4) highlight the impact to other similar spillway structures around the state. The authors also incorporated available firsthand accounts from individuals familiar with the event.

I.INTRODUCTION

Mount Carmel Dam is located approximately 10 miles north of Langdon, North Dakota, and just 6 miles south of the Canadian border. The dam was constructed in 1970 for water supply and recreational uses on the Upper Little South Pembina River. The dam is owned by Cavalier County Water Resource District (formerly the Cavalier County Water Management District). Since the dam's reconstruction following the 2003 event, Langdon Rural Water has also been involved in the operation of the dam, namely monitoring and reading instrumentation. A regional location map of Mount Carmel Dam is shown in Figure 1.

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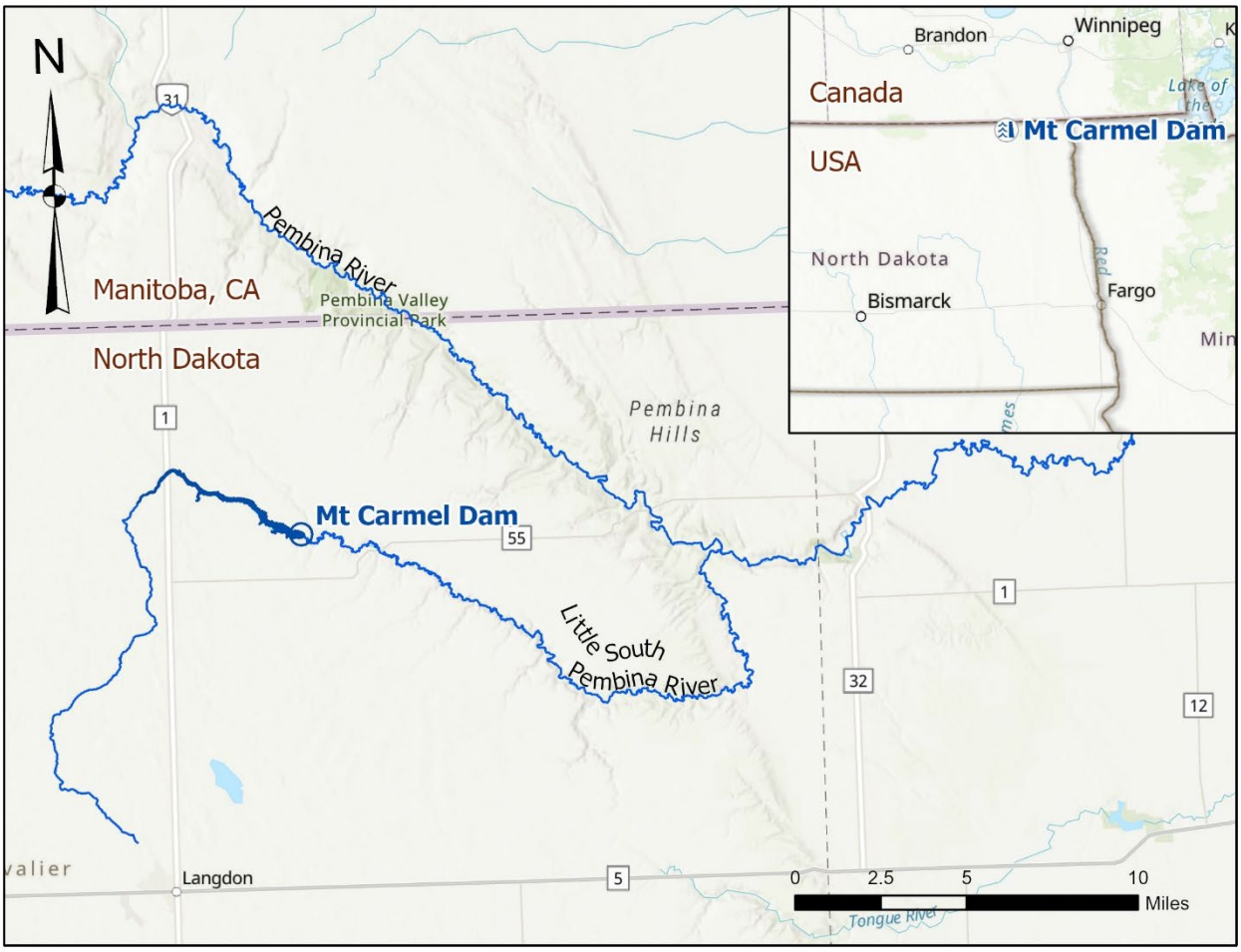


Figure 1. Location Map of Mount Carmel Dam in North Dakota.

Mount Carmel Dam was constructed in 1970 – 1971 at a cost of more than $300,000, equivalent to nearly $2.2 million in 2023 [1, 6]. The earthen embankment dam was originally built 42 feet high and 670 feet long. In original construction, the principal spillway inlet was a reinforced drop inlet riser that discharged into a conduit located at the approximate center of the dam and a grass-lined emergency spillway [1, 3]. The original principal spillway plays a role in the emergency response and operation of the dam during the incident described in this paper. Following initial completion and filling of the dam, seepage was observed in the right abutment near the downstream toe. This was remediated with a grouting program, and the seepage reduced.

In 1995, the dam was modified with a 2-foot increase to normal pool to increase reservoir storage and bring the dam into compliance with dam safety regulations. Modifications included raising the embankment 4 ft; widening and raising the emergency spillway; constructing a new reinforced concrete principal spillway consisting of a drop inlet, chute, and stilling basin; constructing outlet works with low-level drawdown capabilities; and decommissioning the original principal spillway. A concrete bulkhead was constructed on the upstream end of the original principal spillway to remove it from service. These modifications increased reservoir storage and brought the dam into compliance with dam safety regulations.

An aerial view and photos of the site are presented in Figures 2 through 5.

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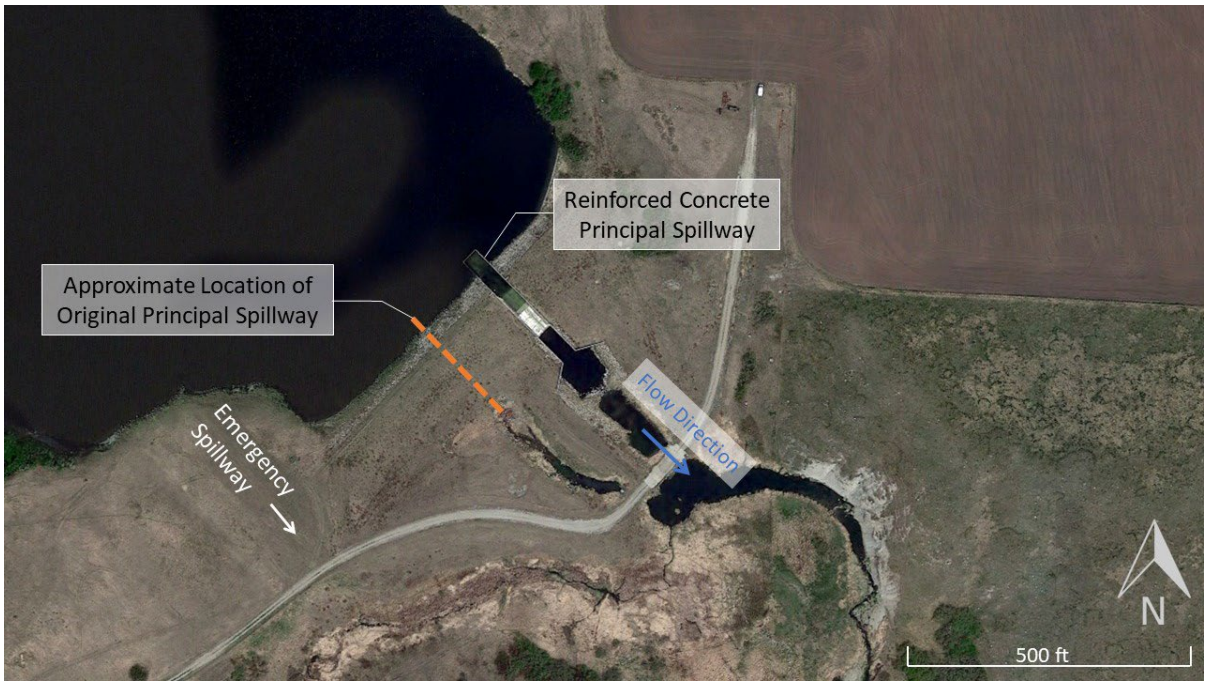


Figure 2. Aerial photo of Mount Carmel Dam with main components of structure (Source: Google Earth, imagery collected May 2019).



Figure 3. Construction of sloping spillway chute showing completed stilling basin in background – August 24, 1995 (Source: ND Department of Water Resources).

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Figure 4. Exposed backfill face of spillway during construction with sidewall drain pipe outlets visible prior to placement of drainage material – September 16, 1995 (Source: ND Department of Water Resources).



Figure 5. Completed dam and spillway just after construction completion – July 24, 1996 (Source: ND Department of Water Resources).

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II.INCIDENT DESCRIPTION

On March 29, 2003, a local high school student taking pictures for a class project discovered water flowing uncontrollably under the spillway of Mount Carmel Dam and exiting from a large void on the downstream face [5, 6]. The student quickly alerted local authorities, allowing for a prompt response to the pending disaster. “We were very fortunate the boy discovered it when he did,” recounted Bill Hardy, Cavalier County Water Resources Board, in the April 10, 2003, issue of the Grand Forks *Herald*. “Under the worst-case scenario, the entire spillway could have been washed out.”

Upon closer inspection of the dam, it was discovered that the void under the dam extended from the upstream wall to the downstream toe of the spillway chute. An aerial view of the damaged embankment, captured on March 29, 2003, is provided in Figure 6.

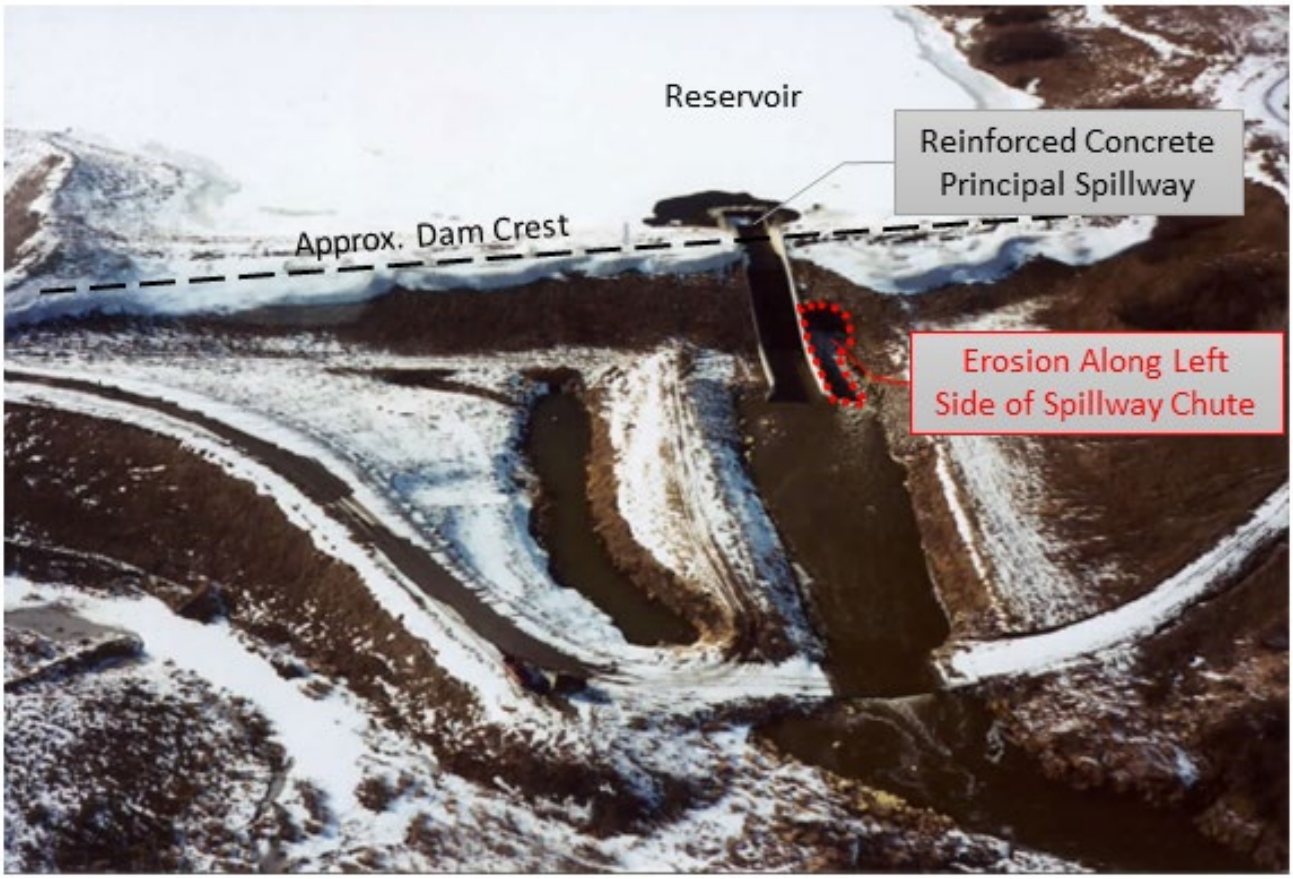


Figure 6. Aerial view of erosion along the left side of the spillway chute exiting at the downstream toe – March 29, 2003 (Source: ND Department of Water Resources).

Significant erosion was clearly visible at the downstream toe of the embankment (Figures 7 and 8), with a large volume of soil having been displaced from both under and adjacent to the structure. It appeared that the erosion left approximately 45 ft of the downstream section of the chute was unsupported, with the remainder of the chute also undermined to an indeterminate extent (Figures 7 through 10) [5].

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Figure 7. Erosion at downstream wall of spillway, viewed looking upstream – March 30, 2003 (Source: ND Department of Water Resources).



Figure 8. Water flowing beneath the eroded spillway chute – March 30, 2003 (Source: ND Department of Water Resources).

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Figure 9. Extensive voiding resulting in a large extent of unsupported spillway chute – March 31, 2003 (Source: ND Department of Water Resources).



Figure 10. Investigating the voiding – March 31, 2003 (Source: ND Department of Water Resources).

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Further review indicated that the erosion and undermining of the principal spillway was likely initiated on or around March 28, 2003. The North Dakota State Water Commission (SWC) [1] judged that the spillway was not actively flowing when the event initiated, and the pool level was likely several inches below the crest when the seepage under the spillway began. SWC estimated that 700 to 800 acre-feet of water were released from the reservoir during the erosion event, with a peak flow rate of approximately 200 to 500 cfs [6]. There was a soil berm present around the upstream spillway inlet (Figure 11) that likely contributed to a reduction of flow as the event progressed and the pool elevation decreased.



Figure 11. Water flowing into void beneath the compromised spillway chute, note small berm in front of spillway – March 30, 2003 (Source: ND Department of Water Resources).

When SWC personnel arrived on site on March 29, 2003, the reservoir level had dropped about 2.5 to 3 ft, and flow rates through the erosion hole had reduced significantly from the apparent high point. The conditions of the dam as observed on April 1, 2023, several days after the incident was first discovered, are shown in Figure 12 [1].

[1] Note: The North Dakota (ND) Department of Water Resources was formerly the ND State Water Commission, as referred to herein.

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Figure 12. A view of the downstream face several days after the incident – April 1, 2003 (Source: ND Department of Water Resources).

Fortunately, no deaths or injuries resulted from the incident, nor was any downstream damage reported from the initial release. No stream gage data could be located for the incident to evaluate the outflow hydrograph resulting from the event.

The high school student who discovered and reported the incident received a letter of thanks from the Governor of North Dakota, John Hoeven. Governor Hoeven noted that this student's actions likely prevented the total loss of the Mount Carmel Dam, which may have resulted in the potential loss of life downstream and the loss of the water supply source for the city of Langdon and the surrounding area [13].

Jason Boyle, State Dam Safety Engineer for Minnesota's Dam Safety Program, was an engineer with the North Dakota Dam Safety Program at the time of the 2003 Mount Carmel Dam incident. In speaking with the authors 20 years after the incident, he recalled visiting the site shortly after the discovery of the void and making several trips to the dam site throughout the investigation and repair process. At roughly four hours from his office in Bismarck, the trip was not one to be made lightly. Mr. Boyle noted that he was actively involved with the emergency response effort including tracking financial metrics as the state was heavily involved. Stopping the erosion from progressing farther and safeguarding water supplies for the local population were the most urgent concerns, after which temporary and permanent repairs could be implemented. Mr. Boyle also recalled the state's evaluation of other similar structures, paying close attention to the potential for any vulnerabilities. In reflecting on the 2003 incident, he stressed the need to follow up on any items that seem irregular or out of place during routine inspections. If a change is noticed, it should be chased down and addressed to prevent a potential dam failure [2].

III.INCIDENT RESPONSE AND INVESTIGATION

When the incident was reported on March 29, 2003, SWC responded quickly. Despite the flow rate through the eroded hole having reduced significantly, SWC initiated emergency response actions immediately. Later that day and/or the following day, the North Dakota Division of Emergency Management (ND DEM) was notified, and an emergency declaration was made. In addition, state resources from SWC, Department of Transportation, Highway Patrol, State Radio, National Guard, Civil Air Patrol, DEM, and Office of the Governor were alerted and/or engaged for support. The American Red Cross and other local volunteers were also engaged for action to support emergency response [13].

Two families were asked to voluntarily evacuate the potential downstream inundation area, and materials and equipment to support a potential flood fight were staged near the area. Roads that could potentially be affected were closed, and warnings were provided to the public. Qualified personnel were dispatched to the site to evaluate

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actions that may be necessary to reduce the likelihood of dam failure and characterize potential downstream effects from a dam failure. Local water supply considerations were also assessed during the emergency response period, as dam failure would have resulted in only a 2- to 3-month supply of drinking water for affected users. Other provisions put into place included identifying pumps and water transport tankers that could be rapidly deployed if needed and issuing a Flash Flood Warning in the area [13].

A *Situation Report* from the ND DEM dated March 31, 2003, indicates that technical support was requested from the U.S. Army Corps of Engineers (USACE) and the Natural Resources and Conservation Service. It also states that 85,000 sandbags had been staged in towns within the potential flood area, and four bridges, six roads, and a culvert had been identified within the potential flooding area. The report also indicates that a contractor was on site performing emergency response activities [13].

Between March 30 and April 2, 2003, the reservoir was lowered. To accomplish this, the previously abandoned 66-inch diameter concrete pipe that had served as the original principal spillway was reactivated by removing the concrete bulkhead installed in 1995. Beginning shortly after the incident, a barrier was constructed upstream of the spillway to reduce flow and limit additional damage from erosion. On March 30, 2003, an initial berm was constructed with soil on top of existing riprap around the principal spillway inlet. The soil fill was reportedly borrowed from the main embankment to the left of the spillway chute. Between April 7 and 17, 2003, after the reservoir had been lowered, a sheet pile cofferdam and soil infill were constructed to isolate the principal spillway from the reservoir [14].



Figure 13. Sheet pile cofferdam upstream of box-inlet drop spillway during installation – April 14, 2003 (Source: ND Department of Water Resources).

On April 1, 2003, three USACE engineers (two geotechnical and one hydraulic) visited the site. Several key observations were made including an eroded opening on the downstream left side of the principal spillway structure. The estimated dimensions of the opening were 30 feet wide (parallel to the dam centerline), 35 feet long (in the upstream to downstream direction), and 6 feet high. The sheet pile cutoff wall, which was installed as part of the replacement spillway in 1995, was observed to be approximately 4 feet deep beneath the spillway structure and displaced in some locations. On the upstream side of the dam, small openings and collapsed soil were observed on the left and right sides of the spillway. Tension cracks were observed in the soil at the crest that indicated subsidence of the soil materials as it collapsed into the voids resulting from the erosion beneath the spillway [16].

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Figure 14. Erosion and deformed sheet pile below the spillway following the incident – April 1, 2003 (Source: ND Department of Water Resources).

GEI Consultants, Inc. (GEI) was engaged in July 2003 to generally identify potential failure mechanisms and provide recommendations for rehabilitation of the dam. Identified contributing factors were based on a review of existing available information and observations from a site visit on July 30, 2003. In general, it was concluded that seepage paths may have developed underneath the spillway structure due to site-specific conditions (i.e., freeze/thaw), construction-related issues (i.e., poor compaction and gaps between insulation panels), and design-related considerations (i.e., inadequate seepage cutoff depth, filter incompatibility, and highly permeable coarse aggregate placed under the spillway for subgrade stabilization). As a result, seepage flows overwhelmed the spillway underdrain system, traveled along seepage pathways, exacerbated erosion underneath the spillway, and exited at the left side of the spillway [1].

IV.REPAIR AND RECONSTRUCTION

Following the Mount Carmel Dam incident, it was determined that the spillway structure would be entirely rebuilt along the same spillway alignment. Based on the findings from the investigation identifying potential failure mechanisms, the following recommendations for future design modifications generally included the following key elements:

- Provisions to mitigate against the adverse impacts of freeze/thaw on the spillway slab.
- Deepening and widening the seepage cutoff to reduce exit gradients.
- Installing appropriate filters and drains (consisting of natural aggregate materials, not geosynthetics) with hydraulic capacity to pass anticipated seepage flows and the ability to mitigate against soil particle migration. Materials should be placed in a manner to prevent segregation and adverse changes in density.
- Consideration of structure footings to resist flotation.
- Design and construction of a new service spillway incorporating the elements listed above.

Design and construction of the replacement spillway was underway in 2004, one year after the incident. GEI prepared a rehabilitation design, and construction of the replacement spillway began in June 2004; the construction was completed by Industrial Builder Incorporated of West Fargo, North Dakota. Construction commenced with an

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effort to have an operable spillway prepared to handle spring runoff the following year. The main elements of the rehabilitation and reconstruction were:

- **Replacing the principal spillway and restoring normal operation of the reservoir.** The new spillway is a box-inlet drop spillway with a Saint Anthony Falls type stilling basin. Seepage control consisting of fine and course filter material; sheet pile along the perimeter of the drop inlet spillway is included in the rehabilitation. A frost management system consisting of rigid insulation is also included.
- **Providing rapid drawdown capabilities.** Drawdown capabilities established to drawdown to the upper 10 feet of the reservoir were accomplished through modifications to the original spillway outlet works to provide regulated releases and increase low-level drawdown capacity of the reservoir.
- **Restoring the low-level outlet.** The low-level outlet drawdown pipe was restored following the incident.
- **Improving seepage and embankment slope stability.** Repairs of the eroded embankment from the incident were completed at the same time that additional embankment modification improvements were made.

The rehabilitation of the dam cost approximately $2.5 million and continues to perform as per the intended design.

The 2003 Mount Carmel Dam event led to a proactive response to other similar structures around the state built similarly to Mount Carmel Dam. The consultant who evaluated the dam following the incident and designed the repairs (GEI) was also hired to review and evaluate three similar dams: Sweetbriar Dam, Big Coulee Dam, and Epping Dam, as they each had similar designs to Mount Carmel Dam. To make a thorough inspection of each of these dams, the stilling basins were pumped out. The inspection of the dams resulted in major repairs at Sweetbriar Dam. Although no work was completed at Big Coulee or Epping Dams immediately after the GEI inspections, substantial repair projects were completed at both structures in 2016; both projects included structural repairs to the spillways and remediation of the spillway drainage systems.

At Sweetbriar Dam fine materials were found indicating evidence of internal erosion; since the dam's first inspection as part of the Phase 1 program, there have been concerns reported about uncontrolled seepage around the spillway. Immediate recommendations for repair included installation of core hole filters in the spillway slab, improvement of toe drainage, and installation of a downstream cofferdam to raise the downstream hydraulic gradient. These repairs were completed in December 2004 following the recommendations. Additional modifications were completed in 2010 including construction of a filter collar around the spillway structure, installation of toe drains and filters, and reconstruction of the reinforced concrete stilling basin wing walls due to deterioration.

By taking a proactive approach to considering dams of similar construction and with similar features, the likelihood of a future problem or incident developing was reduced. The safety of each of these dams and the downstream communities was improved by the proactive approach in review and rehabilitation where necessary.

V.LESSONS LEARNED

In reviewing this case study, the reader is offered the following lessons to be learned for further consideration:

- **All dams need an operable means of drawing down the reservoir.** During the response to the incident, the previously abandoned outlet had to be reactivated to draw down the reservoir and complete repairs. https://damfailures.org/lessons-learned/all-dams-need-an-operable-reservoir-drain-system/.
- **Dams in cold regions should account for ice.** Freeze/thaw impacts were identified as one of the contributing factors that adversely affected the spillway slab, creating a pathway for seepage and erosion. https://damfailures.org/lessons-learned/dams-in-cold-regions-should-account-for-ice/.
- **Emergency Action Plans can save lives and must be updated, understood, and practiced regularly to be effective.** In response to the potential complete loss of the dam, two families were voluntarily evacuated, roads were closed, and other precautions were put in place to safeguard drinking water supplies. https://damfailures.org/lessons-learned/emergency-action-plans-can-save-lives-and-must-be-updated-understood-and-practiced-regularly-to-be-effective/.

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- **Forensic investigations are needed for major dam failures and incidents in order to determine the history of the contributing physical and human factors, and the culminating physical failure modes and mechanisms.** A consulting firm and multiple federal agencies investigated the root causes of the incident. The findings of this investigation were implemented to prepare a rehabilitation design for Mount Carmel Dam as well as to evaluate similar structures. The collective findings instigated further evaluation of three other similar dams in the state. This follow-up work resulted in major repairs at Sweetbriar Dam that showed evidence of internal erosion at the spillway. Significant repairs at two other dams were completed later (2016) to address similar concerns. The immediate implementation of the lessons learned from Mount Carmel were applied to improve dam safety objectives across the state. https://damfailures.org/lessons-learned/forensic-investigations-are-needed-for-major-dam-failures-and-incidents/.
- **High and significant hazard embankment dams should have internal filter and seepage collection systems.** Adequate internal filter and drain systems may have mitigated contributing factors that allowed seepage and internal erosion to develop. https://damfailures.org/lessons-learned/internal-filter-and-seepage-collection-systems/.
    - See also: https://damfailures.org/lessons-learned/seepage-along-penetrations-through-embankment-dams-should-be-controlled-using-a-filter-diaphragm-instead-of-anti-seep-collars/.
- **Intervention can stop or minimize the consequences of a dam failure. Warning signs should not be ignored.** The incident was noticed prior to the full loss and breach of the dam. Emergency measures were put in place to reduce the risk of complete dam failure.

VI.ACKNOWLEDGMENT

The authors gratefully acknowledge the contributions of Karen Goff, Dam Safety Program Manager of North Dakota's Dam Safety Program for the gracious sharing of information regarding this incident. In addition, the authors wish to thank Jason Boyle, State Dam Safety Engineer for Minnesota's Dam Safety Program, for his recounting of the incident during his tenure with the North Dakota Dam Safety Program.

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DAM SAFETY PALM SPRINGS
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Cart Path Only: Spillway Breaching at Black Creek Site 53 Dam

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I. EXTENDED ABSTRACT

Golfers are familiar with the term “Cart Path Only” which instructs golfers the keep motorized golf carts on the designated paths to avoid damaging the grass surfaces on the course. A similar term should be applied for grass-lined spillways which also require uniform, well-maintained vegetated surfaces. Unlike golf courses, the consequences of discontinuities and surface disturbances of grass-lined spillways are much greater than the bumps felt riding in the golf cart. Poor maintenance of the vegetative cover and post-construction modifications such as road cuts, fences, drainage ditches, woody vegetation, etc. can concentrate the flow in an earth cut spillway and accelerate erosion. These were the circumstances surrounding the 1983 Black Creek Site 53 Dam incident in Mississippi. In May of 1983, two significant storm events occurred within one week of each other causing the spillway to activate and eventually breach. It was determined that the breach was initiated and accelerated by a field road that was added after construction of the spillway that crossed the spillway exit channel perpendicular to the flow. This paper will review the history of Black Creek Site 53 Dam and discuss the root causes for the development of this issue. The primary focus of the paper will be to review how the discontinuity and poor maintenance of the spillway led to the spillway failure. The paper will highlight the importance of earth-cut spillway inspections, maintenance, and owner education to prevent catastrophic failures.

Black Watershed Structure Y-36-53 Dam (Black Creek 53), located on Chicopa Creek in Garden Chapel, MS, was completed in 1964 to plans developed by the USDA. See Figure 1 for a location map of the site. The purpose for Black Creek 53 was to reduce flooding in the Yazoo River. The structure itself consists of a 43-foot earthen embankment with the ability to impound 6,611 ac-ft and is classified as Low Hazard due to the low potential for impacts to human safety upon failure. In 1979 and 1980, modifications were made to the outlet works and the embankment was resurfaced.

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Figure 1. Black Creek Site 53 Location Map

Black Creek Site 53 consists of a 43-foot-high flood control earth embankment on a 19 square mile watershed. The earth-cut auxiliary spillway was 100-feet wide with 3H:1V side slopes. It consists of an inlet reach of approximately 140 feet on a two percent adverse slope, a level crest section 20 feet in length, and an exit reach of approximately 160 feet on a 2.7 percent slope (Schweiger, et al. 2008). The three constructed reaches were apparently well vegetated at the time of the flow. (Schweiger, et al. 2008)

The constructed portion of the auxiliary spillway exits into a drainage channel forming a natural channel to carry the flow to the valley below the dam (Figure 2). A roadway constructed along the spillway side of the draw crossed the constructed spillway at the outlet forming a near vertical drop of approximately six feet onto the roadway and another drop of approximately eight feet to the floor of the drainage channel. (Schweiger, et al. 2008)

The site geology is somewhat unusual. The spillway was constructed through a loess material. Approximately six feet below the surface of the constructed spillway was a lens of non-cohesive sand and gravel on the order of one to three feet thick. Below this sand-gravel lens was another 10 to 12 feet of loess underlain by another layer of highly erodible non-cohesive material. Near the valley floor elevation was an erosion resistant shale (Figure 2). (Schweiger, et al. 2008)

On May 20, 1983, a storm event released nine inches of rain. This followed a storm event that released five inches of rain earlier that week. The first storm raised the reservoir water surface and decreased the flood storage available to contain runoff from the second storm. The second storm generated enough runoff to activate the auxiliary spillway and caused it to discharge flow for over 60 hours. The maximum head on the spillway was 5.1 feet and the peak unit discharge was 84.7 cfs per foot. The final breach through the spillway was approximately 40 feet deep, 160 feet wide, and over 500 feet long (Schweiger, et al. 2008). The breach effectively drained the flood control pool. It was estimated that 187,000 cubic yards of spillway material was eroded. A photo of Black Creek Site 53 following complete breaching of the auxiliary spillway is presented in Figures 3 and 4. (Temple, 1989)

The mode of breach was apparently the upstream movement of two vertical overfalls based in the highly erodible non-cohesive materials (Temple, 1989). The roadway at the spillway exit initially concentrated the flow attack at points where the erodible material had minimum cover at the roadway cut and the floor of the draw. Once exposed, the non-cohesive material was easily removed undercutting the loess and allowing it to break off and be washed away as large blocks. (Schweiger, et al. 2008)

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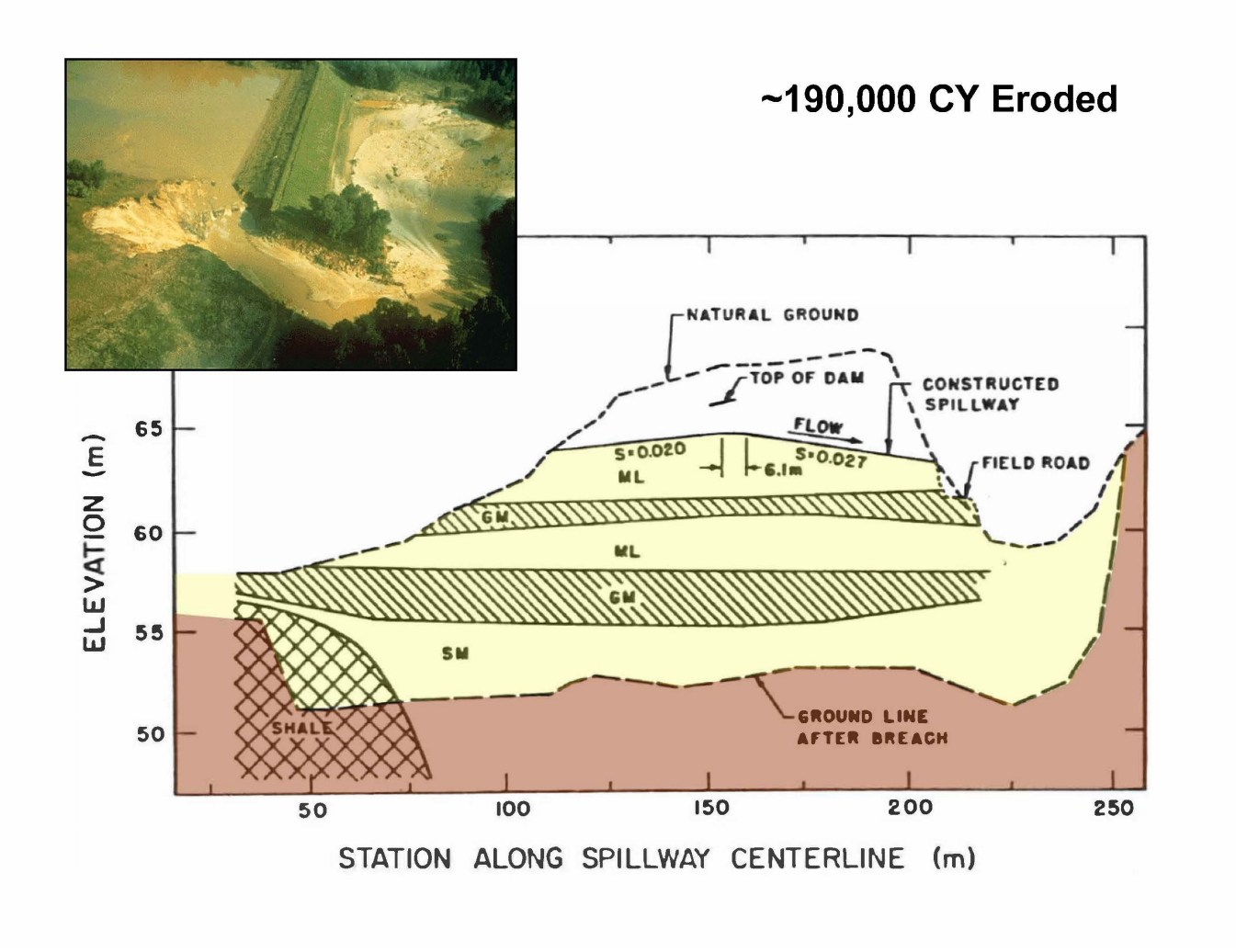


Figure 2. Black Creek Site 53 Auxiliary Spillway Profile and Geology (Gannett Fleming, 2017)

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Figure 3. Black Creek Site 53 Following Breach of Auxiliary Spillway (Schweiger, et al. 2008)



Figure 4. Black Creek Site 53 Following Breach of Auxiliary Spillway

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Previous studies have determined the mode of breach was apparently the upstream movement of two vertical overfalls based in the highly erodible non-cohesive materials. The roadway at the spillway exit initially concentrated the flow attack at points where the erodible material had minimum cover at the roadway cut and the floor of the draw. Once exposed, the non-cohesive material was easily removed undercutting the loess and allowing it to break off and be washed away as large blocks. (Schweiger, et al. 2008)

The field road was not on the original plans and was likely constructed by non-technical parties unaware of its possible impacts on the spillway operation. Black Creek Site 53 Dam has taught us the importance of involving the proper technical parties prior to modifying grass lined spillways. The incident has also showed the importance of understanding material variations in the observed failure process and the need for thorough geologic investigation and evaluation in spillway siting. (Schweiger, et al. 2008)

The following recommendations involving earth-cut spillways should be considered to avoid failures similar to Black Creek Site 53 Dam.

- Knowledge of the underlying material should be gathered through a geologic investigation and evaluation.
- Owner's knowledge and understanding of the potential impacts spillway modifications can have on their structure.
- Development of an Operations, Maintenance, and Inspection Manual with approval from a dam safety engineer.
- Routine maintenance to the structure so it can operate to the design standards.
- Routine inspections performed by a dam safety engineer.
- Limiting or prohibiting public access onto the structure can help avoid hazardous impacts to the structure and additional maintenance cost to the site.

Routine maintenance, inspections, and owner education of earth-cut spillways are important in avoiding catastrophic failures of spillways. The performance of earth-cut spillways can be greatly impacted by modifications that cause high flow concentration, especially in non-cohesive material. It is important to have a good understanding of the geologic features underlying an earth-cut spillway and important to get any spillway modifications approved by a dam safety engineer.

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DAM SAFETY PALM SPRINGS
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Is catchment area the dominant factor for PMF estimation in a supposedly hydrologically homogeneous region?

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ABSTRACT

One of the primary drivers for rehabilitation of dams is to address inadequate spillway capacity. Determination of the inflow design flood (IDF) or design flow for a dam facility is governed by a few considerations, but in many instances it boils down to a balancing act between applicable dam safety rules, relevant industry standards, risk assessment and rehabilitation cost.Diving one level down into the specifics behind the determination of the IDF determination, it is the estimate of the probable maximum flood (PMF) that consistently serves as a key reference, in particular for high hazard dams, and this holds true whether the evaluation is deterministic, probabilistic, or somewhere in between. It is assumed many times that catchment area will be a key input for estimation of the PMF, and relationships between PMF and catchment area are available in the literature for initial ballpark estimates. Going this route is reasonable, especially if comparing to values from other locations with similar climatic and hydrologic conditions, including land use. However, recent work brings up some questions about the assumption that catchment area is indeed a dominant factor.This presentation will summarize the results of estimating the PMF for 4 dam facilities, namely Elm Lake Dam, Corsica Dam and Richmond Dam in South Dakota, and Lake Bronson Dam in Minnesota, with catchment areas ranging between 91 and 547 square miles. The study watersheds have common characteristics: low topographic relief, primarily agricultural, and recorded snowmelt peaks usually larger than the observed rainfall-driven floods. We followed the same methodology for PMF estimation at the 4 sites, starting with development and calibration of HEC-HMS hydrologic models, incorporation of updated values for probable maximum precipitation (PMP) -per work by the state of North Dakota in 2021-, and development and application of HEC-RAS models. The PMF estimates to share will suggest that parameters like the shape of the watershed, or the magnitude and associated likelihood of the flood of record, can be equally or more important than catchment area.

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Distributed Curve Number Modeling for Runoff Volume Prediction

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ABSTRACT

Rainfall-runoff modeling is commonly employed by hydrologic engineers with one of the primary goals being the determination of runoff volume. Runoff volume can be calculated from the total depth of rainfall, the overall amount of losses and the watershed area. In hydrologic engineering modeling the Natural Resources Conservation Services (NRCS) Curve Number (CN) method is commonly used to determine the losses from an associated rain event. Traditionally, application of the CN method involved determining one "average" representative CN for a watershed with the underlying assumption that losses occur uniformly over the entire landscape of the watershed. This assumption has obvious flaws and therefore the presumption of one representative CN for a watershed is inherently incorrect and frequently will lead to erroneous predictions of runoff volume when compared to actual data associated with historical rainfall-runoff measurements. Version 6 of the Hydrologic Engineering Center's River Analysis System (HEC-RAS) allows users to input a spatially distributed CN layer in combination with a precipitation boundary condition for the purpose of simulating the rainfall-runoff phenomenon. By using a distributed CN layer as opposed to one representative value, runoff volume predictions theoretically will prove more reliable. A study was conducted which compared runoff volume results of 45 simulations using an average CN as part of the NRCS methodology vs simulations using a spatially varied CN as part of a HEC-RAS simulation vs actual results using historical data obtained from the National Weather Service (NWS) (rainfall data) and the United States Geologic Survey (USGS) (runoff data). The HEC-RAS simulations produced an average runoff volume error of 67% with 21 of 45 simulations overestimating the total runoff volume. The NRCS simulations had an average runoff volume error of 81% with 31 out of 45 simulations overestimating the total runoff volume. A subsequent analysis was performed to compare the theoretical CN based on soil type and land use used as part of the NRCS simulations, to both the average CN calculated based on analysis of the runoff hydrograph produced from the HEC-RAS simulations, and the event specific CN determined from analysis of the runoff hydrograph and rainfall values associated with the historical data collected from the NWS and USGS. Results showed that using a spatially distributed CN approach yielded better results than the theoretical average CN, when compared to the storm specific CN.

I. INTRODUCTION

Rainfall-runoff modeling gained popularity in the 20th century after the inception of unit hydrograph theory, proposed by Sherman (1932), and the development of various unit hydrograph methods including Snyder's, Clark's, and the Natural Resources Conservation Services (NRCS) (Snyder 1938, Clark 1945, Mockus 1957). Both Snyder's and Clark's methods were adopted by the United States Army Corps of Engineers (USACE), whereas the NRCS method has gained widespread acceptance and use throughout the professional engineering community. Regardless of which unit hydrograph method is chosen for the hydrologic analysis, a preliminary analysis must first be performed which estimates the amount of hydrologic abstractions (losses) which occur through the duration of a given rain event. A series of loss methods have been used including: the NRCS's Curve Number method (SCS 1973), Horton (1940), and Green & Ampt (1911), amongst others. The NRCS's Curve Number method is the most commonly used method for large scale hydrologic analyses which require estimating the amount of losses occurring during any given rain event. The Curve Number is a single value ranging between 0-100 based on soil type,

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landuse, and antecedent moisture condition. Lower Curve Numbers assume high infiltration rates and less surface water runoff, whereas higher Curve Numbers yield less infiltration and more surface water runoff. Initial Curve Numbers based on soil type and landuse are determined from tables provided in the SCS-TR-55 manual (SCS 1986). Adjustments can be made to the initial estimate based on conversion factors to antecedent moisture conditions I or III. Application of the Curve Number method is accomplished using equation 1.

$$Q = \frac{(P_t - I_a)^2}{(P_t + .8S)} \qquad (1)$$

Where:
Q = total depth of direct runoff (in)
P_t = total depth of rainfall (in)
I_a = Initial Abstraction (in) = 0.2S
S = Maximum Potential Storage (in) = 1000/CN – 10
CN = Curve Number

The result of equation 1 is the determination of excess precipitation from a given storm event. The associated losses are computed as the total rainfall minus the total runoff (excess precipitation) over a given time frame. As seen in equation 1, only one Curve Number value is used to determine the excess precipitation. The intention of the method as stated by the SCS (1973) is that runoff should be computed for areas exhibiting constant landuse and soil type throughout the corresponding area (ie: 1 curve number). However, soil type and landuse are rarely constant across large areas and therefore the general application applied by the professional community was to determine 1 area-averaged representative Curve Number for use in equation 1. This application, besides not being consistent with the intention of the method, often times will lead to erroneous calculations of excess precipitation when compared to historical data. For example, a watershed which is 50% impervious with a Curve Number of 99 and 50% wooded in good condition with soil type A (Curve Number equals 30) would result in an area-average Curve Number of approximately 65. The theoretical amount of runoff produced from this watershed using a Curve Number of 65 will often times be much less than the actual amount of runoff occurring should field measurements be taken. This is due to the impervious area actually producing near total runoff whereas the theoretical amount due to use of the average Curve Number producing something much less.

Recent advancements in both computing power and software capability offer a new way to simulate the rainfall-runoff process. High-resolution digital elevation models (DEMs) (3D terrain representations of the land surface), in combination with land coverage and soil type layers, allow users to simulate the rainfall-runoff process by first determining the amount of excess precipitation that would occur at specific locations throughout the watershed, based on the localized Curve Number, and then compute the path that individual water particles travel through a watershed using two-dimensional hydrodynamic equations (conservation of momentum and conservation of mass). This is a fundamental shift in the way that runoff as a result of a rain event can be determined. While hydrologic methods, such as the NRCS unit hydrograph method, rely on empirically based equations which were developed based on a limited set of historical data and which utilize general watershed characteristics, hydrodynamic models use the actual watershed topography with corresponding (localized) land cover characteristics (i.e., infiltration potential based on specific soil type, routing based on surface roughness, etc.) to route volumes of water through the watershed and to the outlet. Hydrodynamic models allow users to overlay, on top of the terrain, a mesh consisting of a network of cells. Two-dimensional conservation laws solved using matrix algebra as part of an iterative finite volume solution algorithm are then used to determine how any water collected in one cell will move to an adjacent cell based on the change in elevation between cells as well as the amount of resistance that the water is encountering within and between adjacent cells. Rainfall can be introduced into the system as a boundary condition and as volumes of water collect in specific areas throughout the watershed, based on the underlying topography, the water is then routed from cell to cell. Eventually, excess rainfall accumulates in river systems and flow characteristics (runoff hydrograph, depth at a point, etc.) can be measured at the watersheds outlet. Version 6 of the Hydrologic Engineering Center's River Analysis System (HEC-RAS) is one software package with this capability. All Geographic Information System (GIS) processing is handled directly within HEC-RAS, including importing the corresponding terrain information and then overlaying both a Curve Number layer, for the purpose of determining how much rainfall at specific locations throughout the watershed will result in runoff, and a Manning's n layer, which is used to account for surface resistance once excess rainfall has accumulated and routing of the

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runoff commences. In short, hydrodynamic routing attempts to realistically mimic the actual rainfall-runoff process using math and physics as opposed to using empirically derived equations which hydrologic methods are based on.

The goal of this study was to compare the total runoff depth for various watersheds and numerous historical rain events using both a standard uncalibrated hydrologic method (NRCS) as well as an uncalibrated hydraulic model, via HEC-RAS, and then compare the results to actual data recorded from a United States Geologic Survey (USGS) gaging station located at each watershed's outlet for the purpose of determining if one method is more reliable than the other.

II. METHODOLOGY

The overall process for completing this study consisted of first identifying usable watersheds and their historical rainfall and runoff data and then determining any inputs necessary to complete the hydrologic and hydraulic modeling. Each step is described below.

A. Acquisition of Historical data

To stay consistent with the assumption of unit hydrograph theory that the rainfall is uniformly distributed (spatially) over the entire watershed for each time increment, watersheds were limited to a size no greater than 44 square miles. While this area may be considered large for typical rainfall-runoff modeling, only historical rain events which produced a peak flow equal to or greater than a 25-year return period were used in the analysis with the assumption that rain events producing this level of runoff most likely were spatially large and encompassed the entire watershed. Return periods were based on USGS regression equation estimates obtained from StreamStats (https://streamstats.usgs.gov/ss/) via the Peak-Flow Statistics → Regression Based Scenarios Report. Each of the watersheds used in the study was in New Jersey, and historical runoff data was accessed from the USGS National Water Information System: Web Interface (https://waterdata.usgs.gov/nwis/sw). To reflect current land use conditions, only hydrographs found since the year 2000 were used. A total of 29 watersheds were used for this study.

Flow records from the 29 watersheds were inspected and the dates associated with any peak flow approximately equal to or exceeding a 25-year return period were recorded. The runoff hydrograph from each rain event was obtained and the baseflow was removed from the flow record so that an analysis of peak flows could be conducted on the resulting direct runoff hydrograph. Since neither the hydrologic nor hydraulic models accounted for baseflow it was important to remove baseflow from the historical event to properly compare the resulting peak flows.

National Weather Service (NWS) gages were used to acquire historical rainfall data (in hourly increments) for these corresponding dates. Gage locations were mapped in a GIS software and then overlaid with each of the watershed locations (Figure 1) to determine Thiessen polygons (Viessman & Lewis, 2003). The gage weighting for each watershed was then computed. Once the percent contribution was found from any rain gages either within or near the watershed, the average hourly rainfall amount occurring over the entire watershed was calculated.

A total of 45 historical rain events were used for this study. Eighteen watersheds had one historical storm, eight watersheds had two historical storms, two watersheds had three historical storms, and one watershed had five historical storms used for the analysis.

B. Hydrologic Modeling

Due to its widespread use and acceptance, the NRCS methodology was used to predict the runoff hydrograph for each watershed based on the corresponding historical precipitation. The Hydrologic Engineering Center's Hydrologic Modeling System (HEC-HMS) was used for the simulation. A minimal amount of input was required to produce the simulation. Specifically, the watershed area, loss parameter (Curve Number), lag time, peak rate factor, and rainfall were the required inputs. The watershed area was obtained from the USGS record associated with each gage. The average basin Curve Number was determined using GIS. A land use layer obtained from the New Jersey Geographic Information Office was overlaid with a hydrologic soil grouping layer obtained from the Soil

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Survey Geographic Database (SSURGO) via the NRCS website (https://websoilsurvey.sc.egov.usda.gov/App/WebSoilSurvey.aspx) and then merged with Curve Numbers found in the Technical Release-55 tables (SCS 1986) to generate a CN layer for each watershed. Layer statistics within GIS were used to determine the basin average Curve Number. As stated previously, while using an area averaged Curve Number is commonly employed in many practical engineering applications associated with hydrologic analysis of watersheds, it is not the proper application of the Curve Number method. Instead, partial runoffs should be calculated over areas of uniform land use and soil type (i.e., constant Curve Number) and then the incremental runoffs should be combined to determine the overall runoff for the watershed (Hawkins et al, 2009). However, as stated previously, most hydrologic engineering studies lump sum soil type and land use characteristics to determine one basin averaged Curve Number which is then used to develop runoff from one basin as opposed to a series of sub-basins. Using a basin averaged Curve Number was employed by the authors to try to best mimic the typical approach that the professional community uses for hydrologic rainfall-runoff modeling. The lag time was calculated using Equation (2) and the "standard" peak rate factor of 484 was used for the analysis. For the lag time calculation both the longest hydraulic length and average watershed slope were determined using GIS software.

$$T_l = \frac{L^{.8}(S+1)^{.7}}{1900\sqrt{slope}} \quad (2)$$

Where:

T_l = Lag Time (hrs)
L = Longest Hydraulic Length (ft)
S = Storage Potential = 1000/CN – 10
CN = Curve Number
$Slope$ = average watershed slope (%)

The final input necessary to conduct the hydrologic simulation was the incremental rainfall, which was obtained from NWS gages as described in the previous section. Once all of the required inputs were determined, a HEC-HMS model was developed using a computation interval of 15 minutes (i.e. duration (D) of the unit hydrograph) for all simulations. Please note that a 15-minute computation interval divides the hourly rainfall total into 4 equal amounts. The total runoff depth for each simulation was documented and compared to the estimated runoff depth from the USGS gaging station for each storm event.

C. Hydraulic Modeling

HEC-RAS Version 6 was used to perform hydraulic modeling of the rainfall-runoff response for each storm event. A DEM, obtained from the New Jersey Office of Geographic Information, was imported into HEC-RAS to establish the underlying terrain for each watershed. A shapefile obtained from StreamStats, using the USGS flow gage location as the outlet point, was loaded into HEC-RAS and used to determine the boundary of the 2D flow area (i.e., watershed boundary), which the corresponding rainfall would be distributed over. A Curve Number layer, obtained via GIS as previously described, was input to calculate the localized hydrologic abstractions associated with each storm event.

It is important to note that HEC-RAS determines losses based on the precipitation applied to each cell within the 2D mesh in combination with the corresponding Curve Number at the specific location over the terrain. This is the correct approach for Curve Number modeling (Hawkins et al., 2009) and a significant change in the way the Curve Number methodology is applied when compared to typical hydrologic approaches which use one average Curve Number applied to the entire watershed. Subsequently, there will be differences in the overall volume of direct runoff throughout the watershed for each method. Additionally, it is important to note that HEC-RAS does not determine losses based on water moving into a specific cell in the form of surface runoff from an adjacent cell. Instead, the only losses that are calculated are a result of rainfall being applied to specific cells.

Once the excess rainfall was determined in each cell, the corresponding volume of water was computed as the depth of excess rainfall times the area associated with a specific cell. The location where the volume of water pools was then determined based on the underlying topography of that cell, with water initially filling up at the lowest elevation levels within each cell. Once the volume and location of the water were determined, two-dimensional

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hydrodynamic equations were used to route the flow to adjacent cells. The direction of motion for the routing was based on the change in elevation between adjacent cells as well as the amount of surface resistance that the flow is encountering. Water will not begin to move out of a specific cell until the depth of water within the cell becomes higher than the elevation along the border of an adjacent cell. Surface resistance was accounted for using a land cover layer and corresponding Manning's n values associated with each type of land use. The land cover layer was obtained from the New Jersey Office of Geographic Information and was based on land use conditions in 2015.

Any type of hydraulic flow modeling requires boundary conditions which are used to establish starting criteria for the first iteration of the solution algorithm at each time step. A time series of incremental hourly rainfall depths was used as the boundary condition which served the purpose of adding water into the system. Once the water accumulated and flow developed, a second boundary condition was also required to properly model the flow leaving the system. The second boundary condition, "normal depth", was chosen along the downstream perimeter of the 2D mesh. A normal depth boundary condition requires the friction slope of the water profile at the downstream boundary to be input. The friction slope for each river was determined based on the slope of the terrain in the river as it approached the outlet.

HEC-RAS allows the user to select the two-dimensional hydrodynamic equation set used for the solution of the simulation. For all simulations, the Shallow Water Equations Eulerian Method (SWE-EM) (i.e. – full Saint Venant equations) were chosen to perform the hydraulic computations. A computational time step of one minute was chosen for each simulation.

Once each simulation was completed, hydraulic outputs including velocity, depth, hydrographs, etc. can be viewed at any location within the watershed boundary. Numerical integration of the hydrograph at the outlet to the watershed was used to determine the overall depth of runoff for each rain event. The depth of runoff was documented and compared to the estimated depth of runoff from each historical event.

III. RESULTS AND DISCUSSION

Table 1 shows the watershed characteristics for each of the 29 watersheds used in this study. Characteristics shown in the table were used to develop the hydrologic simulation of each rain event. Table 2 shows the historical runoff/volume data and the results from the hydrologic and hydraulic model simulations.

Table 1 Watershed Characteristics

Station #	Area (sq mi)	LHL (ft)	Slope (%)	Curve Number	# of storms analyzed
1367800	15.8	59358	2.98	68	1
1379699	24.3	97892	4.5	77	1
1381400	13.9	46652	3.88	69	1
1381500	29.7	72080	3.62	73	1
1389550	7.8	36256	2.87	85	2
1394500	25.9	62325	2.56	85	2
1396190	31.0	67896	2.84	76	2
1398000	25.5	55465	1.9	81	2
1398500	26.5	79249	3.61	71	2
1398900	40.8	92309	3.4	72	1
1399500	32.2	116771	2.82	75	2
1406050	16.0	60533	1.51	72	1
1407290	6.4	25319	1.99	74	3
1408000	44.0	85110	1.15	75	5
1408120	34.9	116523	1.09	64	4
1410150	8.1	36093	1.11	58	2
1410784	15.1	40688	0.77	74	1
1411300	30.8	83121	0.63	67	1

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1411456	9.8	33953	0.69	68	1
1412800	28.0	49249	0.92	76	1
1439800	22.8	80806	3.37	73	1
1443280	13.0	34079	3.47	73	1
1445000	31.0	60843	3.76	78	1
1455090	9.6	52247	3.63	79	1
1463740	4.0	23384	1.32	85	1
1466500	2.4	24268	0.94	57	1
1467081	9.0	46446	1.03	87	2
1475001	6.2	25066	1.2	80	1
1477120	26.9	59415	1.38	75	1

Table 2 Historical Storm Data and Modeling Results

Station #	Storm Date	Total Rainfall Depth (in)	Estimated Runoff Depth (in)	Hydrologic Modeling Runoff Depth (in)	Hydraulic Modeling Runoff Depth (in)	Storm Specific Curve Number	Hydrologic Modeling (Theoretical) Curve Number	Hydraulic Modeling Average Curve Number
1367800	8/28/2011	6.7	3.30	3.16	1.37	69	68	48
1379699	8/28/2011	7.1	3.42	4.41	2.88	68	77	62
1381400	8/28/2011	9.3	3.75	5.47	4.22	55	69	59
1381500	8/28/2011	9.3	3.15	5.97	2.65	51	73	47
1389550	9/6/2008	3.8	0.90	2.23	1.49	65	85	75
	8/28/2011	8.2	3.50	6.43	4.75	60	85	70
1394500	4/15/2007	5.9	4.70	4.25	2.35	89	85	65
	8/28/2011	8.4	6.30	6.55	4.25	83	85	65
1396190	10/9/2005	6.4	1.95	3.78	2.02	57	76	58
	8/28/2011	5.9	2.39	3.26	1.33	66	76	53
1398000	4/27/2007	1.9	1.82	0.54	0.41	99	81	77
	8/28/2011	5.0	4.61	2.95	2.34	97	81	74
1398500	10/9/2005	6.4	1.16	3.27	2.30	47	71	61
	8/28/2011	8.1	2.86	4.68	3.70	55	71	63
1398900	8/28/2011	6.2	2.50	3.14	5.77	65	72	96
1399500	8/28/2011	8.2	1.00	5.25	4.04	36	75	65
	9/8/2011	5.4	0.70	2.74	1.08	48	75	53
1406050	8/28/2011	7.6	4.58	4.34	3.18	74	72	61
1407290	8/28/2011	7.0	2.97	4.05	3.77	64	74	71
	4/30/2014	3.5	1.58	1.24	0.82	79	74	66
	5/5/2017	2.3	0.95	0.49	0.34	84	74	69
1408000	8/28/2011	6.5	5.24	3.67	3.13	90	75	69
	12/27/2012	4.0	1.70	1.62	1.59	76	75	74
	5/27/2018	4.6	1.51	2.13	1.63	66	75	68
	8/13/2018	4.8	0.68	2.29	1.44	51	75	64
	11/25/2018	2.6	1.23	0.69	2.13	85	75	96

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1408120	3/14/2010	3.8	1.09	0.87	0.56	68	64	57
	5/1/2014	3.5	0.37	0.71	0.15	56	64	48
	8/13/2018	4.8	0.59	1.45	1.16	50	64	60
1410150	8/13/2000	2.5	0.70	0.14	0.27	76	58	64
	8/28/2011	4.4	0.28	0.88	1.70	45	58	71
1410784	8/28/2011	4.6	1.63	2.05	0.92	68	74	57
1411300	8/28/2011	4.8	0.63	1.64	1.18	50	67	60
1411456	8/28/2011	4.6	2.23	1.60	0.73	76	68	54
1412800	8/28/2011	6.6	2.09	3.92	2.66	57	76	63
1439800	8/28/2011	6.7	2.77	3.67	3.04	64	73	67
1443280	8/28/2011	5.9	1.97	3.01	0.81	61	73	45
1445000	8/28/2011	5.6	1.43	3.25	2.05	57	78	65
1455090	8/28/2011	5.8	0.94	3.46	2.05	49	79	63
1463740	8/28/2011	5.4	4.61	3.77	2.64	93	85	73
1466500	8/28/2011	4.8	0.40	1.00	0.91	45	57	55
1467081	8/28/2011	4.6	2.96	3.20	1.27	84	86	63
	4/15/2007	5.8	3.73	4.31	1.31	81	86	54
1475001	8/28/2011	4.6	2.23	2.55	2.10	76	80	74
1477120	8/28/2011	5.9	3.00	3.17	2.29	73	75	65

The overall depth of direct runoff was calculated for each historical event with both the corresponding hydrologic and hydraulic simulations of each event. The depth of direct runoff was determined from the historical event by subtracting baseflow from the storm hydrograph by assuming a linear increase in baseflow from the time of rise to the inflection point on the recession limb of the hydrograph (McCuen 2004). Once the baseflow was removed, the depth of runoff was determined through numerical integration of the direct runoff hydrograph and then dividing the resulting volume by the corresponding watershed area. The depth of runoff from the hydrologic modeling was determined based on the watershed's theoretical Curve Number and the total depth of rainfall for each storm event. The depth of runoff from the hydraulic model was determined through numerical integration of the simulated runoff hydrograph and then dividing the resulting volume by the area of the corresponding watershed.

An analysis was performed for the depth of direct runoff to determine how well the simulated results match the historical data. Figure 1 shows the relationship between the modeled runoff depth to the actual runoff depth with the 1:1 line, 100% overprediction and 50% underprediction lines shown. As shown in Figure 1 the majority of the hydrologic modeling results (31 out of 45) overpredicted the amount of direct runoff, whereas less than half (21 out of 45) of the results from the hydraulic model overpredicted the amount of direct runoff. The amount of percent error associated with the runoff depth predictions was approximately 62% (raw) for the hydrologic modeling and 20% (raw) for the hydraulic modeling. The absolute value of the percent error averages was approximately 81% (hydrologic) and 67% (hydraulic). The root mean square error for each situation was 1.40 in (hydrologic) and 1.32 in (hydraulic). The historical runoff depth for all historical events was 2.26 in. The results of the simulations show that in general using a distributed curve number approach (hydraulic modeling) tends to result in less error compared to the lumped sum approach (hydrologic), when computing the amount of direct runoff as a function of the given Curve Number. However, the level of improvement measured for the distributed approach compared to the lump sum method does not appear to be significant for this data set.

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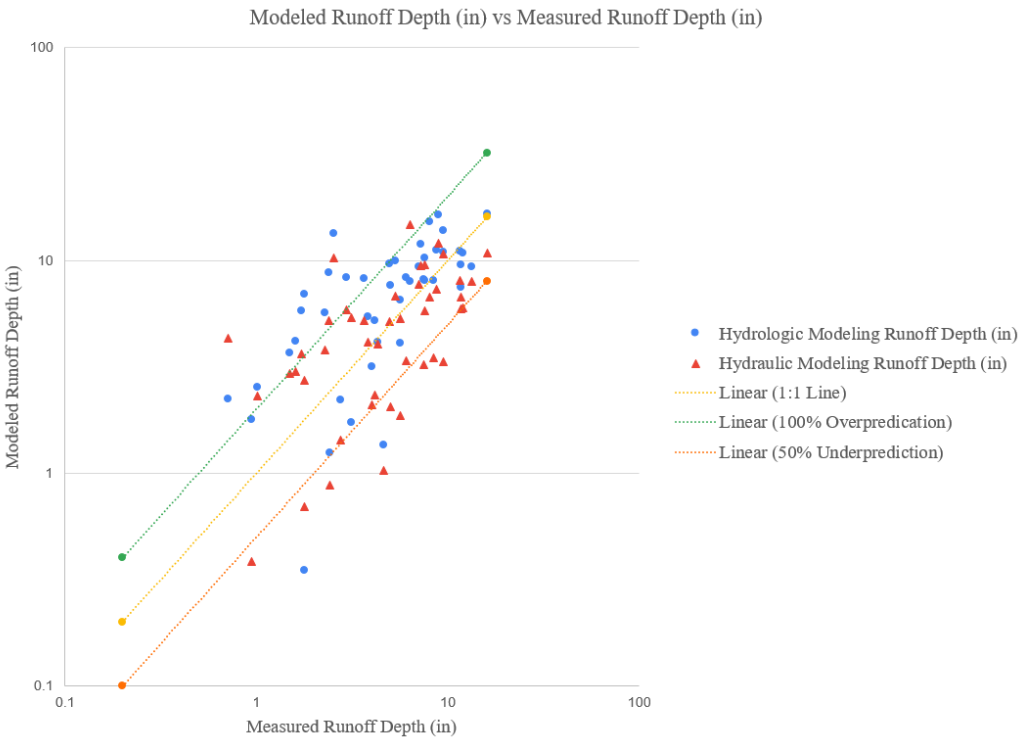


Fig. 1 Comparison of Simulated Runoff Depth (in) for each Model to Historical (Estimated) Runoff Depth (in)

Additional analysis was conducted to determine if the results for the depth of direct runoff improved as a function of various watershed, rainfall, or flow characteristics (Figures 2-5). Figures 2 (watershed area), 3 (longest hydraulic length), 4 (average watershed slope) do not show any relationship to improve the predicted runoff depth to the historical runoff depth. Figure 5 (rainfall return period) does show that as the return period for the rain event increases to at least a 50 yr event the associated error for the predicted runoff depth falls within the 50% underprediction - 100 % overprediction bounds. Additionally, as the rainfall return period reaches a 200 yr event, the hydraulic modeling results have minimal error (approximately -16% and 12% errors, compared to 29% and 46% errors for the same events using the hydrologic modeling approach (theoretical Curve Number)). While this is only two (2) data points, it does suggest that for rain events which truly are uniformly distributed over the entire watershed (which is a reasonable assumption for a 200 yr rainfall event), using a distributed Curve Number approach may result in much better predictions of runoff depths.

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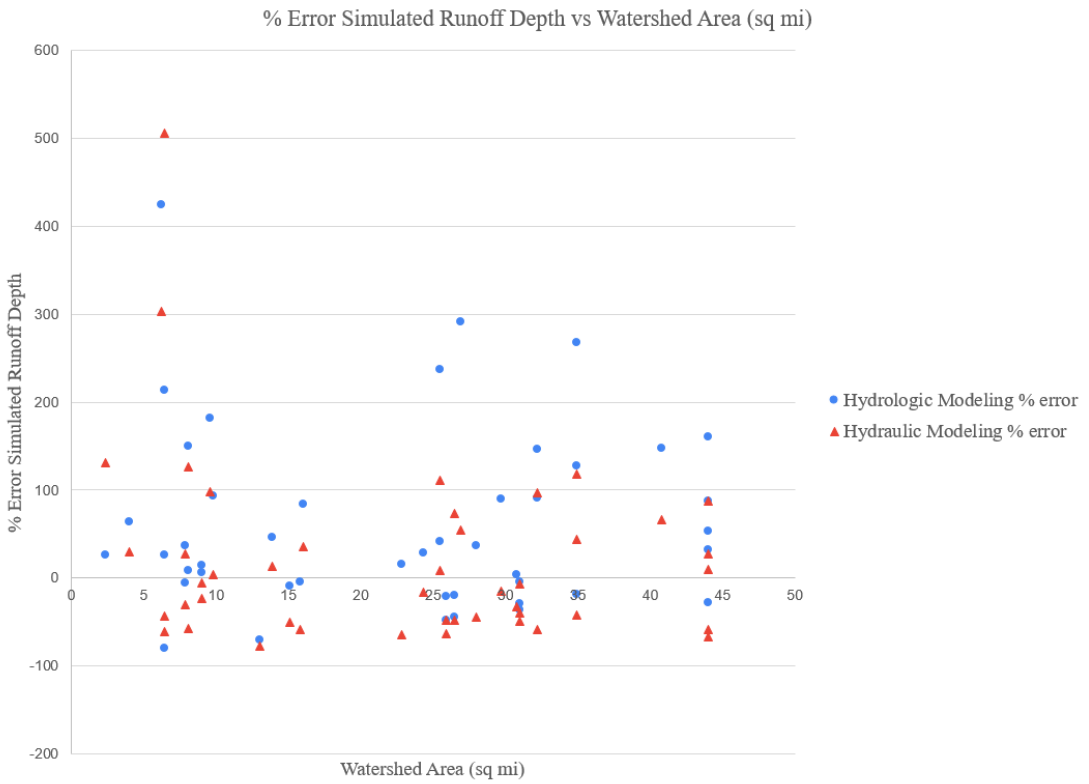


Fig. 2 Runoff Depth % Error vs Watershed Area (sq mi)

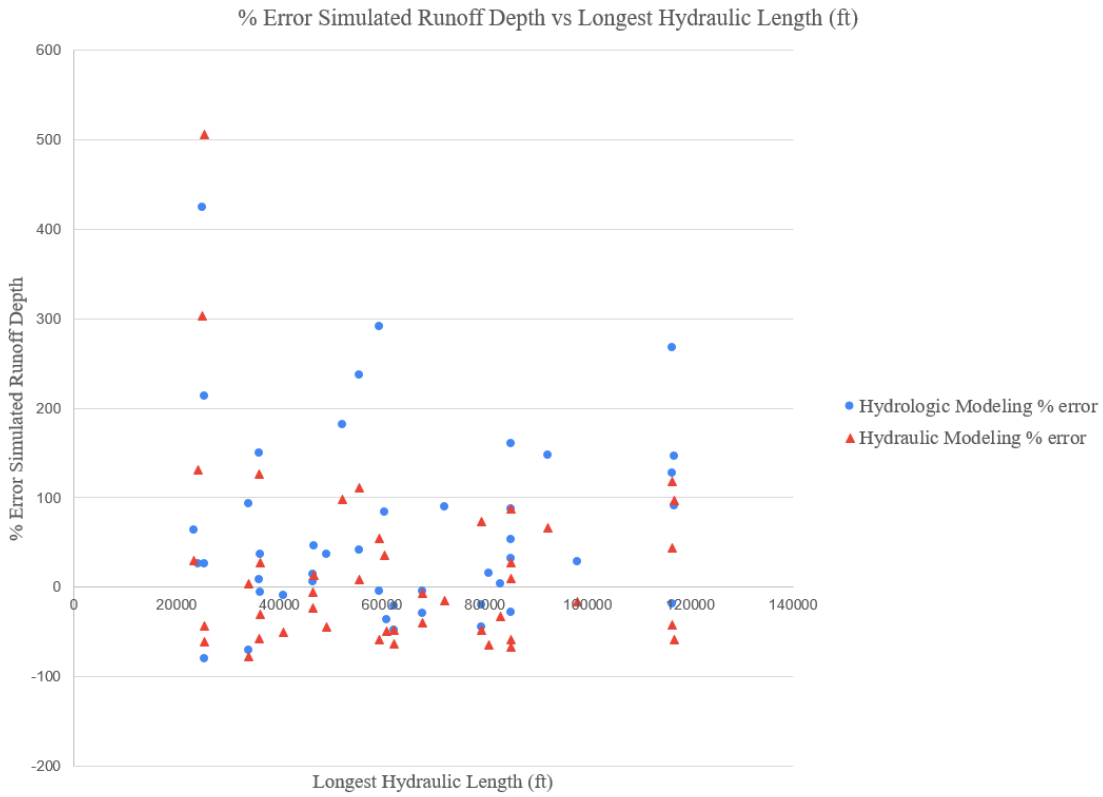


Fig. 3 Runoff Depth % Error vs Longest Hydraulic Length (ft)

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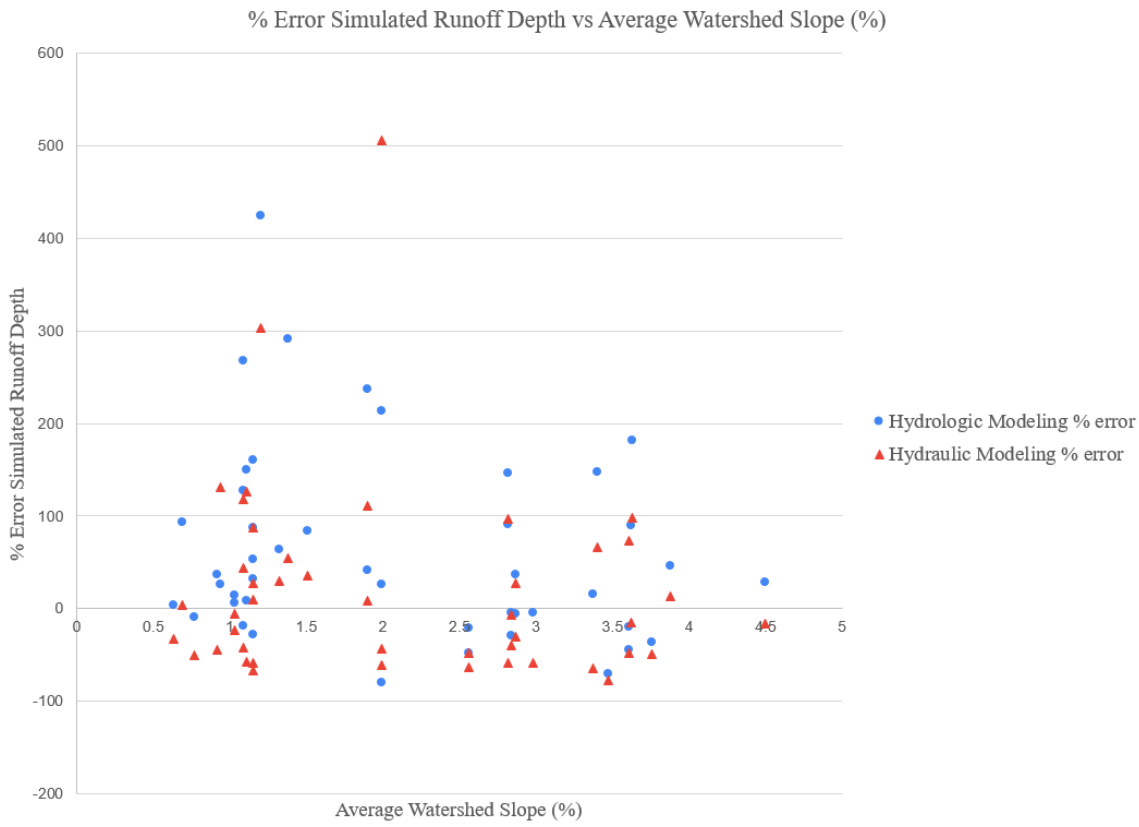


Fig. 4 Runoff Depth % Error vs Average Watershed Slope (%)

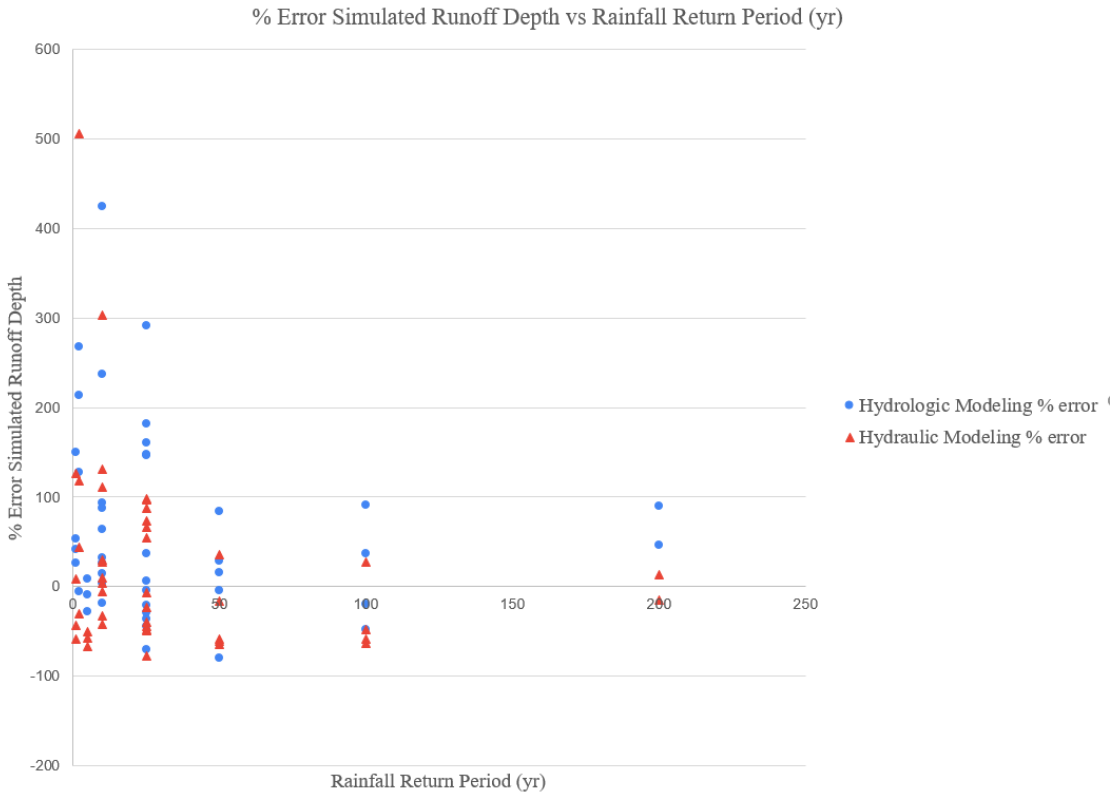


Fig. 5 Runoff Depth % Error vs Rainfall Return Period (yr)

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Based on the calculation of direct runoff for both the historical event and hydraulic modeling simulation, the representative Curve Number was back-calculated for each situation using Equation (1). The depth of direct runoff was determined as previously described. The total depth of rainfall was determined for each rain event was found by summing the hourly incremental values. Both the initial abstraction (Ia) and the maximum storage potential (S) are functions of the Curve Number and therefore the equation can be solved directly to determine the Curve Number for each event. The back-calculated Curve Number for both the historical data and the hydraulic model assumed the Initial Abstraction to Storage Potential (Ia/S) ratio of 0.2.

Figure 6 shows the relationship of the storm specific Curve Number to the theoretical (hydrologic modeling) and hydraulic modeling "average" Curve Number. To reiterate, the average Curve Number shown in Figure 6 for the hydraulic model was not specifically used as part of the simulation. Instead, the hydraulic model used cell specific Curve Numbers based on location specific soil type and land use to determine the runoff within each individual cell (Hawkins et al, 2009). Analysis of Figure 6 shows that the majority (31 out of 45) of the theoretical Curve Numbers used for the hydrologic modeling were overestimated when compared to the storm specific (back-calculated) Curve Number. Conversely the majority (24 out of 45) of the "average" Curve Numbers determined as part of the hydraulic model tended to underestimate the actual storm specific Curve Number. The average storm specific Curve Number across all events was 67, whereas the average hydrologic and hydraulic modeling Curve Numbers were 74 and 64 with RMSEs of 15.20 and 14.99 respectively. The fact that the storm specific Curve Numbers did not match the theoretical Curve Numbers was not surprising as this has been shown to occur in many previous studies (Schneider & McCuen, 2005, Elhakeem & Papanicolaou, 2009, Tedela et al, 2012). The results of the runoff depth / Curve Number analysis help to explain why the hydrologic modeling tended to overpredict the observed peak flows at a greater occurrence than the hydraulic modeling.

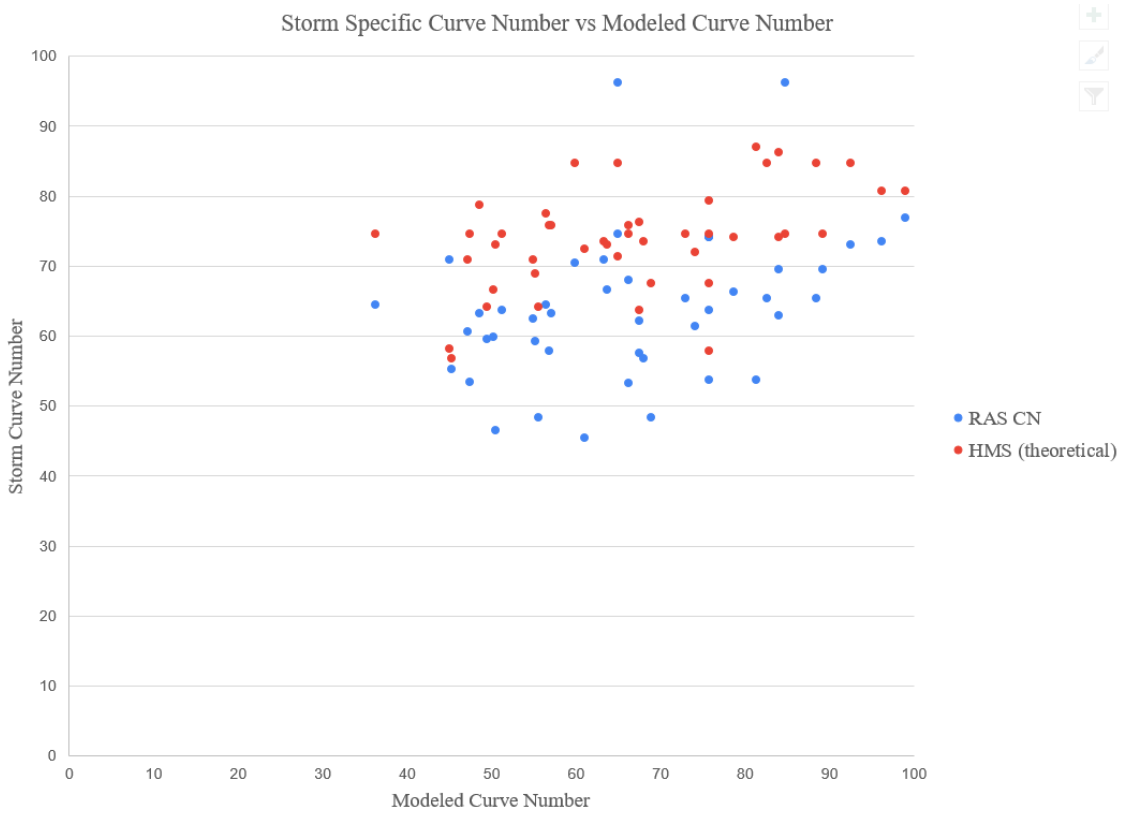


Fig 6. Curve Number Comparison, Historical vs Modeled

IV. CONCLUSIONS

A study was completed to compare results from hydrologic and hydraulic rainfall-runoff modeling to actual recorded data obtained from historical rain events for the purpose of determining if one method proved more reliable than the other for determining the depth of runoff. Results show that the hydrologic model tended to overestimate the runoff

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depth (81% absolute value, 62% raw) compared to the hydraulic model (67% absolute value, 20% raw). Additionally, the back-calculated "average" Curve Numbers associated with the distributed approach of the hydraulic model were in general closer to the actual storm specific back-calculated Curve Numbers, than the theoretical Curve Numbers were used in the hydrologic modeling. No correlation could be found between the overall percent error with each modeling approach and specific watershed characteristics such as area, longest hydraulic length or slope. In conclusion, while neither technique proved to be exceptional at replicating historical data, the hydraulic modeling did tend to perform better. Additionally, for modeling larger return period events, which is typical in the design and analysis of hydraulic structures, while the hydraulic modeling performed better, both modeling techniques were shown to produce less overall error.

V. ACKNOWLEDGEMENTS

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Nonlinear Watershed Behavior Using Variable Unit Hydrograph Parameters for California

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ABSTRACT

Nonlinear watershed response to rainfall has been recognized as an important component in hydrologic routing, especially for large synthetic storms that are commonly used in dam safety evaluations. Recent updates to the US Army Corps of Engineers Hydrologic Engineering Center's Hydrologic Modeling System (HEC-HMS) software includes a new method referred to as the variable Clark unit hydrograph that expands the classical Clark unit hydrograph by allowing the time of concentration and storage coefficient to vary in response to changes in excess rainfall intensity throughout the simulation. This new method allows modelers to simulate the nonlinear dynamic response more accurately within a watershed when subjected to large excess rainfall intensities. HEC partnered with the California Department of Water Resources, Division of Safety of Dams (DSOD) to develop a method to estimate variable Clark unit hydrograph parameters throughout the state of California for use within dam safety applications. Using results from 2D diffusion wave simulations, variable Clark unit hydrograph parameters were derived for a range of excess rainfall intensities using 16 watersheds in California. The results were then synthetized to develop regression equations that quickly allow estimation of changes in time of concentration and storage coefficient needed for hydrologic routing. Predicted variable Clark parameters derived from these equations were evaluated through a validation process demonstrating that the equations improve predictions compared to classical Clark unit hydrograph theory.

I. INTRODUCTION

Watershed response to rainfall scenarios is fundamental in hydrologic dam safety studies to compute inflows to reservoirs, estimate reservoir stages, and evaluate spillway performance. Reservoir inflows associated with various storms depend to a great extent on geomorphic characteristics of the watershed and temporal distribution of precipitation.

Most hydrologic methods for estimating watershed runoff response used in practice rely on two fundamental concepts: the time of concentration and the unit hydrograph. The time of concentration is the time required for surface water to travel from the most hydrologically distant point of the watershed, along a hydraulic path, to the watershed outlet. The unit hydrograph represents the response of a watershed, expressed as a hydrograph at the outlet, resulting from an excess rainfall of one inch uniformly distributed over the entire watershed. Based on a

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computed time of concentration, a unit hydrograph and appropriate precipitation loss and baseflow models, it is possible to perform a hydrologic routing to estimate watershed runoff.

Hydrologic routing models currently used in dam engineering practice assume the time of concentration to be constant for the duration of the storm and independent of the variation in rainfall intensity or discharge. This assumption greatly simplifies hydrologic computations since a constant time of concentration results in a stationary unit hydrograph. This in turn allows for a linear convolution of the unit hydrograph for a specific excess rainfall distribution.

The assumption of a constant time of concentration, however, is not necessarily correct. Variations in rainfall intensity result in changes in discharge and flow velocity, both as channel and overland flow, implying variations in flow travel time, and thus in the magnitude of the time of concentration. Existing formulas and regional studies performed to estimate the time of concentration are usually based on small and frequently occurring events, which may not be appropriate for flows associated with more extreme and infrequent rainfall scenarios used for dam safety evaluations.

II. CLARK UNIT HYDROGRAPH TRANSFORM AND WATERSHED NONLINEARITIES

Hydrologic prediction of large runoff events is usually performed using routing models available in computer codes such as HEC-HMS (U.S. Army Corps of Engineers, 2021). A common and widely accepted approach for hydrologic routing is based on the Clark unit hydrograph (Clark, 1945) using the time-area (TA) concept. The TA represents the response of a watershed to an instantaneous application of a unit depth of excess rainfall (usually one inch) when basin storage effects are ignored. Figure 1 illustrates the concept of the TA. Isochrone lines drawn on the watershed map represent lines of equal travel time to the watershed outlet. The TA curve is the distribution of drained area as the flow reaches the outlet, which is represented by the histogram shown in the same figure.

The response hydrograph at the watershed outlet is computed by a transfer function that transforms inflows (direct or excess rainfall) to outflows. The transfer function accounts for basin storage effects, which diffuse discharge, reduce peak flow, and increase flow duration. Without storage effects, the shape of the resulting response hydrograph will be similar to the shape of the time-area curve. Storage effects are applied at the watershed outlet and assumed by Clark (1945) to vary linearly with respect to outflow. Since storage effects are applied at the outlet, the Clark unit hydrograph method is considered a lumped method.

The implicit assumption in the Clark method is that of linearity in the unit hydrograph to variations in rainfall. For example, if two inches of instantaneous precipitation are applied over the watershed instead of one inch, the response hydrograph at the outlet would be estimated by increasing the ordinates of the unit hydrograph by a factor of two. A consequence of the assumed linearity between the unit hydrograph and rainfall is that the peak of the unit hydrograph is time invariant, that is, the time of its peak does not change with variations in rainfall intensity.

While the assumed linearity between the unit hydrograph and rainfall has proven to be successful in simulating recorded storms, it has been observed that watershed response to rainfall is not necessarily linear. For example, Minshall (1960) noted that the peak of the unit hydrograph tends to occur at earlier times as rainfall intensity increases, suggesting that as the rainfall intensity increases the time of concentration would decrease. Rodriguez-Iturbe and Valdes (1979) indicated that the unit hydrograph, and thus the time of concentration, varies during and between storms. Bras and Rodriguez-Iturbe (1989) discussed the dependency of flow travel velocity on direct rainfall intensity and duration. They showed that the time to peak discharge is a power function of the mean effective rainfall intensity. Saghafian and Julien (1995) and Saghafian et al. (2002) investigated variable time of concentration models but did not account for basin diffusion effects.

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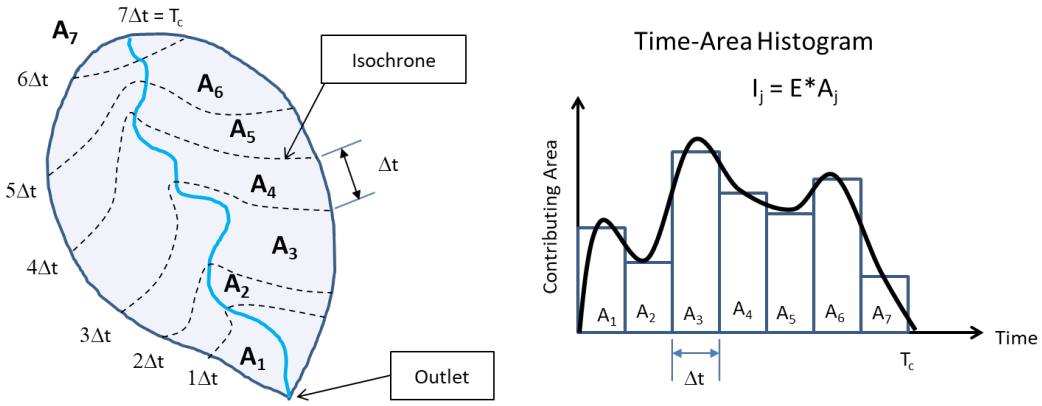


Figure 1. Isochrone map and associated time-area histogram; the time-area histogram represents the distribution of drained area with time as surface water reaches the outlet.

The authors have independently investigated watershed nonlinearities by accounting for variations in time of concentration and time-area curves during large storms (Meyersohn, 2016) and simulating watershed response using two-dimensional (2D) runoff transform methods (Bartles, 2017; Bartles & Fleming, 2016). The studies by the authors demonstrated that the time of concentration decreases as a function of excess rainfall intensity. More recently, Michailidi et. al., (2018) investigated 30 Mediterranean basins and showed that the time of concentration follows a negative power trend with excess rainfall intensity. Further advances in basin nonlinear response to changes in rainfall intensity have been investigated by Barbero et al. (2022), Nagy et al. (2022), and Jacobs (2022).

Even though nonlinear watershed response has been recognized by academia and research, it has not been fully incorporated in engineering practice. In an attempt to use reasonable runoff parameters for the large, extreme storm events used in dam safety evaluations and account for nonlinear behavior, the US Army Corps of Engineers (USACE) and the California Division of Safety of Dams (DSOD) have adopted interim recommendations to address variability in time of concentration and peaking of the unit hydrograph. For example, the USACE recommends reducing calibrated Tc and R values to achieve an increase in peak values of the unit hydrograph between 25 and 50 percent (U.S. Army Corps of Engineers, 1991). This action shifts the time of peaking to earlier times. The DSOD adopted a simpler strategy of reducing the time of concentration by 35 percent. However, the applicability of these rules of thumb has not been fully analyzed and it is unknown whether these changes in Tc and R would result in over- or under-prediction of flows.

III. VARIABLE UNIT HYDROGRAPH STRATEGY

The Clark unit hydrograph method can be modified to simulate basin nonlinearities by varying both Tc and R with excess rainfall intensity during simulation of extreme precipitation events. This will allow the unit hydrograph to vary spatially and temporally in response to rainfall demands (Bartles, 2014; Meyersohn, 2016). Bartles and Fleming (2016) showed that the variable Clark unit hydrograph can reproduce results that are similar to those obtained using more complex 2D routing methods efficiently and with lower computational demands.

The current version of the Hydrologic Engineering Center, Hydrologic Modeling System (HEC-HMS) software includes a 2D engine that solves the St. Venant Equations using physically measurable characteristics to route water on overland surface (U.S. Army Corps of Engineers, 2022). This engine makes use of an implicit finite volume algorithm which allows for advantages such as:

- Larger computational time steps than explicit methods,
- Improved stability and robustness over traditional finite difference and finite element techniques,
- Efficient wetting and drying of 2D cells, and

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- Subcritical, supercritical, and mixed flow regimes.

Unstructured or structured computational meshes can be utilized within this engine that include triangular, square, rectangular, or even eight-sided ground surface elements. Computational cells and cell faces are pre-processed to contain detailed hydraulic property tables including elevation-volume and elevation-conveyance relationships, amongst others. This type of model is often referred to as a "high resolution subgrid model" (Casulli, 2008).

The 2D engine can be used to better recreate anticipated nonlinear runoff responses when subjected to large amounts of precipitation when compared to unit hydrograph transform methods (Bartles, 2017). However, 2D overland transform methods require additional data, are significantly more computationally intensive than unit hydrograph transform methods and may not be cost-effective in engineering practice.

Nevertheless, 2D overland transform methods can provide valuable information regarding the behavior of a watershed under high direct rainfall intensity, allowing insight into how rainfall affects the unit hydrograph. A useful application of 2D overland transform methods is to investigate how the flow at the watershed outlet varies when subjected to single pulses of excess rainfall. The hydrograph at the watershed outlet resulting from the application of a pulse of 1 inch of excess rainfall over one-hour duration, for example, would be equivalent to the unit hydrograph. Similar response hydrographs can be obtained for a range of excess rainfall pulses of different intensities. Because 2D transform methods inherently accommodate temporal and areal variations in flow velocity fields in their formulation, the resulting response hydrograph to excess rainfall pulses would reflect the watershed nonlinear behavior.

In order to develop recommendations to be used in practice, the strategy employed in this paper is to use the 2D diffusion wave transform method to investigate how the time of concentration and the storage coefficient would vary during a storm based on the response of watersheds to excess rainfall pulses of different intensity. The development of variable Clark unit hydrograph parameters follows these steps:

1. Selection of suitable watersheds through the state of California.
2. Calibration of loss, baseflow, and 2D transform parameters at each watershed location.
3. Estimation of Clark parameters (Tc and R) of these watersheds through calibration.
4. Development of response hydrographs for these watersheds for excess rainfall intensity pulses of 0.25, 0.50, 1.0, 2.0, 4.0, 5.0, and 6.0 inches per hour (in/h) using the 2D diffusion wave transform method available in HEC-HMS.
5. Evaluation of Clark unit hydrograph parameters that would reproduce the response hydrographs computed in the previous step.
6. Develop regression equations for Tc and R relating watershed physical characteristics to each excess rainfall intensity.
7. Validation of these regression equations using independent data.

The calibrated Clark parameters estimated during step 3 are obtained from rainfall and flow measured from historical storms. Calibrated Clark parameters are referred to in this paper as index parameters. Calibrated parameters are considered adequate for similar storms but are not suitable for the large hypothetical storms used for dam safety evaluation.

It is important to mention that in steps 3 and 5, the default time-area histogram included within HEC-HMS Clark unit hydrograph transform was used, without modification, within all modeling domains. No attempts were made to develop watershed-specific time-area histograms as this step would not be practical. The simplified default time-area histogram is commonly used in practice during model calibration and runoff evaluation of large storms.

Study Locations

California has a land area of approximately 155,000 square miles (mi^2) and contains an extremely diverse geography which ranges from the Pacific Coast in the west to the Sierra Nevada mountains in the east. In

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the northwestern portion of the state, coniferous forests are prevalent while the Mojave Desert can be found in the southeast. Additionally, the approximately 50-mile wide and 450-mile long Central Valley stretches across the center of the state. Similar to the geography, the climate within California is also tremendously varied. A Mediterranean climate is prevalent throughout the Pacific Coast, the Central Valley, and the foothills of the Sierra Nevada, while moist temperate rainforests can be found in the north. Additionally, arid desert regions are widespread in the southern half of the state while snowy alpine areas can be found in the Sierra Nevada mountains.

Sixteen watersheds upstream of either U.S. Geological Survey (USGS) stream gages or USACE reservoirs were chosen throughout California to develop variable Clark unit hydrograph parameter regression equations. Watersheds were selected such that they are not affected by impaired flow due to the presence of upstream reservoirs or water diversions. However, if impairment was present, their effects were quantified and demonstrated to be minimal. Figure 2 shows the locations of the 16 watersheds selected for this study. Details of the selected watersheds and storms used for calibration are presented in Table 1. Watershed areas range from 8 to 113 mi^2 with peak measured flows from the selected calibration storms in the range of 997 to 25,500 cubic feet per second (cfs). Each of the 16 selected watersheds was calibrated to only one storm event.

Data Sources

Several sources of information were used to perform calibration of the selected watersheds. Flow hydrographs were obtained from stream gages operated by the USGS. Measured streamflow data at USGS gages were downloaded from the National Water Information System (website: https://waterdata.usgs.gov/nwis.) Two of the selected watershed outlets are located at dams, Bear Dam and Owens Dam, both operated by USACE. Inflow at the reservoirs impounded by these dams were downloaded from the Corps Water Management System (CWMS) servers at the Sacramento District (SPK).

Watershed boundaries and streams were acquired from the USGS National Hydrography (website: https://www.usgs.gov/core-science-systems/ngp/national-hydrography.) The Watershed Boundary Dataset (WBD) was used to demarcate the approximate extents of each modeling domain while the National Hydrography Dataset (NHD) was used to identify streams within each modeling domain.

10-meter digital elevation models (DEM) were acquired from the USGS National Elevation Dataset (NED) (website: https://viewer.nationalmap.gov/launch/) for each modeling domain. Land use classifications were also acquired from the Multi-Resolution Land Use Consortium (MRLC) (website: https://www.mrlc.gov/data/nlcd-2016-land-cover-conus) for each modeling domain.

Gridded precipitation data was obtained from multiple sources, that included: a) Multi-Radar Multi-Sensor (MRMS) Quantitative Precipitation Estimate (QPE); b) Real-time Mesoscale Analysis (RTMA); c) Parameter-elevation Regressions on Independent Slopes Model (PRISM); d) Next-generation Radar (NEXRAD) QPE; e) National Oceanic and Atmospheric Administration (NOAA) National Centers for Environmental Prediction (NCEP) Stage I; and f) Analysis of Record for Calibration (AORC). The gridded precipitation data source that was used for each modeling domain depended upon the age of the event used for model calibration. For instance, MRMS data were used for events that occurred after 2015. However, in all instances, sub-daily precipitation was initially used to best capture the temporal distribution of precipitation.

When large under or overpredictions of precipitation were found relative to runoff observations, the Normalizer tool within HEC-HMS was used to create normalized precipitation grids. In all cases where normalization was required, PRISM data was used to correct the sub-daily precipitation. PRISM data has been shown to provide accurate estimations of daily precipitation accumulation within ungaged locations, such as mountainous locations or rural areas. This is possible since PRISM data accounts for spatial variations in climate due to elevation, terrain orientation, terrain steepness, moisture regimes, coastal proximity, and inversion layers, amongst other factors (Daly et al., 2017).

When calibration required estimation of runoff from snowmelt, temperature data was used to estimate the heat transfer into or out of the snowpack. Similar to precipitation, multiple sources of gridded temperature

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data were used within this modeling effort. Depending upon the year of the event used for model calibration, either RTMA or AORC temperature data was used. For instance, RTMA data was used for events that occurred after 2012. Snow data was used to initialize state variables within the modeling domain and calibrate snowmelt process when necessary. Two sources of gridded snow data were used within this modeling effort. The first source of snow data came from NOAA's Snow Data Assimilation System (SNODAS) and the second source of snow data came from the University of Arizona (UofA SWE).

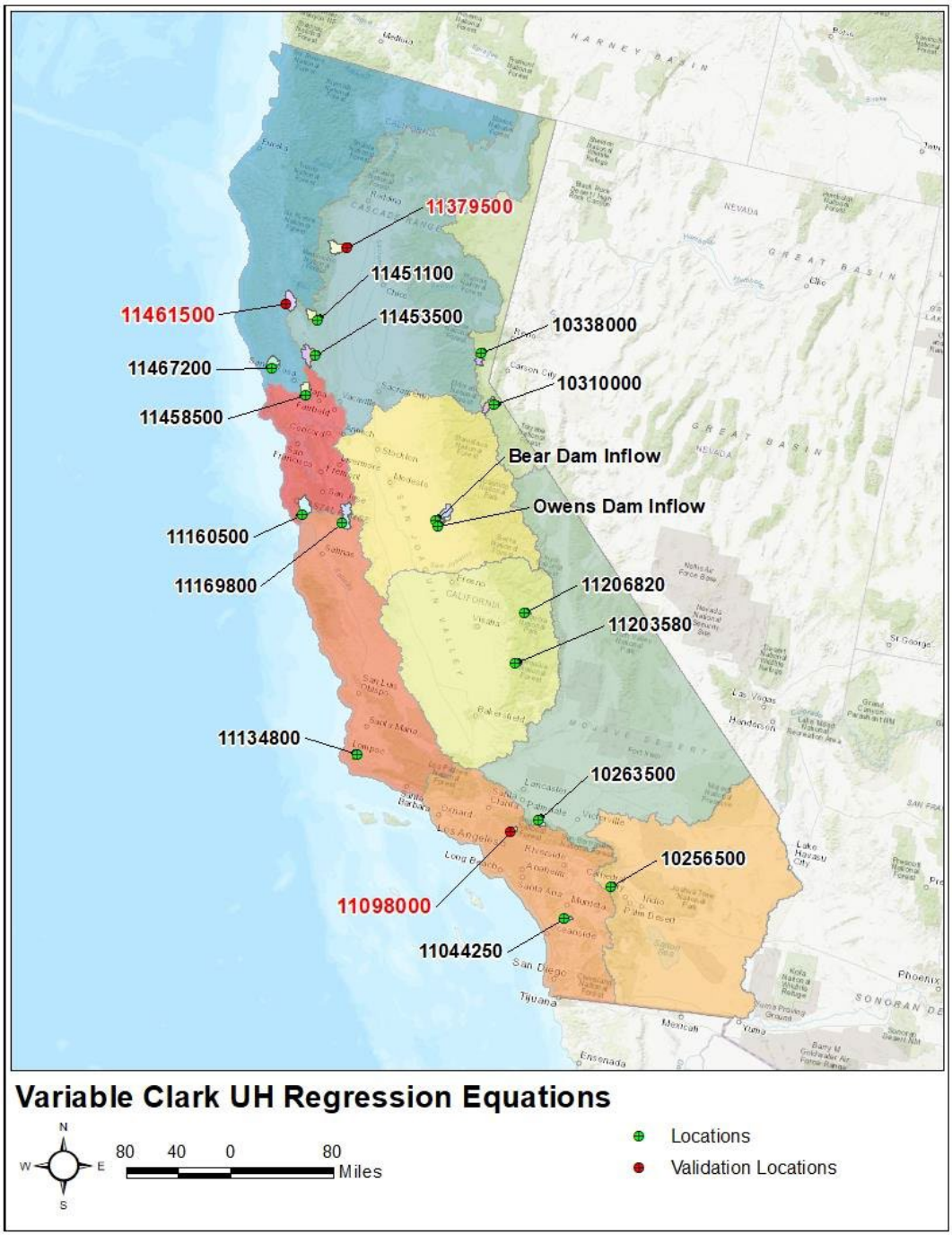


Figure 2. Location of watersheds selected for the study. Number next to symbols on the map represent the USGS Stream Gage Number. Numbers and symbols in red denote gages used for validation.

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Table 1 - Watersheds and Storms Used in this Study							
No.	**Watershed Name**	**Lat**	**Long**	**USGS Stream Gage Number**	**Area (mi^2)**	**Calibration Event**	**Peak Flow (cfs)**
1	Austin C Nr Cazadero	38.506	-123.070	11467200	63	Feb 2019	21,300
2	Sonoma C A Agua Caliente	38.323	-122.494	11458500	58	Dec 2005-Jan 2006	20,300
3	Coyote C Nr Gilroy	37.078	-121.494	11169800	109	Jan 2017	11,500
4	San Lorenzo R A Big Trees	37.044	-122.072	11160500	106	Feb 2017	19,000
5	Miguelito C A Lompoc	34.632	-120.465	11134800	12	Feb 1998	2,660
6	Rainbow C Nr Fallbrook	33.407	-117.201	11044250	10	Dec 2010	4,010
7	Snow C Nr White Water	33.871	-116.681	10256500	11	Feb 2019	3,170
8	NF Cache C A Hough Spring Nr Clearlake Oaks	39.165	-122.620	11451100	60	Feb 2019	10,100
9	Putah C Nr Guenoc	38.779	-122.517	11453500	113	Jan 2017	24,000
10	Bear Dam Inflow	37.368	-120.228	NA	72	Mar 2018	25,500
11	Owens Dam Inflow	37.314	-120.186	NA	26	Mar 2018	10,500
12	SF Tule R Nr Chololllo Campground Nr Porterville	36.048	-118.654	11203580	20	Feb 2017	1,510
13	Marble Fork Kaweah R Ab Horse C Nr Lodgepole	36.611	-118.702	11206820	8	Jul 2015	997
14	W Fk Carson Rv At Woodfords	38.770	-119.833	10310000	65	Jan 1997	8,100
15	Truckee R Nr Truckee	39.296	-120.205	10338000	46	Jan 1997	11,900
16	Big Rock C Nr Valyermo	34.421	-117.839	10263500	23	Jan 2005	2,250

The Gridded Data Import tool within HEC-HMS was used to import the gridded precipitation, temperature, and snow data. During the import process, each data set was reprojected to a common coordinate reference system, resampled to a 2 km x 2 km grid size, and clipped to the boundary of the modeling domain.

Model Development

HEC-HMS was used to simulate the rainfall-runoff processes within each modeling domain. HEC-HMS has been successfully used to solve a wide array of possible hydrologic problems including large river basin water supply, extreme flood hydrology, and small urban watershed runoff, amongst other uses (U.S. Army Corps of Engineers, 2021).

Model Discretization

10-meter digital elevation models (DEM) were used as the basis for all watershed and stream delineations and physical parameter estimations. For the 2D diffusion wave equation models, a 2D mesh was first created within HEC-RAS for each modeling domain. Breaklines were used to align cell faces with prominent topographic features like roadways, embankments, and stream centerlines. Computation points were added in order to create approximately 1000 to 5000 cells, depending upon the size and complexity within each

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modeling domain. To create 2D meshes with variable computation point spacing, improve accuracy, and reduce computation times, Mesh Refinement Regions were also used within some modeling domains. Following the creation of each 2D mesh within HEC-RAS, the resultant information was imported to HEC-HMS.

Modeling Methods

Watershed calibration of modeling domains were carried out with the following methods. For precipitation losses, the Deficit and Constant Loss method was used. Baseflow was modeled using the Linear Reservoir method using two layers. Finally, when snow melt was necessary, the Gridded Temperature Index snowmelt method was used. Since each modeling domain was not subdivided into smaller basins, channel routing was not necessary for calibration. No losses or baseflow were considered for the 2D diffusion wave equation models when subjected to single pulses of excess rainfall.

Model Calibration

Model performance for each of the 16 selected watersheds was evaluated by comparing computed hydrographs results against measured hydrographs at each location. Model parameters were altered to minimize the differences between computed and observed hydrograph shape, peak flow, peak timing, and discharge volume. Differences less than or equal to 15 percent between computed and observed peak flow rates and flow volumes were desired. Statistical metrics to assess the adequacy of the calibration effort was quantified using the Ratio of the Root Mean Square Error to the Standard Deviation (RSR), the Nash-Sutcliffe Efficiency (NSE), and the Percent Bias (PBIAS). Table 2 shows statistics performance ratings for the three metrics used in this study.

Table 2 – Statistics Performance Ratings used in Calibration

Performance Rating	NSE	RSR	PBIAS (Absolute values)
Very Good	0.65 to 1.0	0.0 to 0.60	Less than 15
Good	0.55 to 0.65	0.60 to 0.70	15 to 20
Satisfactory	0.4 to 0.55	0.70 to 0.80	20 to 30
Unsatisfactory	Less than 0.40	More than 0.80	More than 30

NSE: Nash-Sutcliff Efficiency
RSR: Root Mean Square Error to the Standard Deviation
PBIAS: Percent Bias

Examples of model calibration are presented in Figure 3, which shows measured and computed hydrographs for six of the 16 watersheds considered in this study. The results of the calibration efforts are presented in Table 3, which shows for each of the 16 watersheds the calibrated time of concentration and storage coefficient, as well as the resulting statistical metrics and their performance ratings. In general, metrics show calibrated basins show hydrographs that are satisfactory or very good. The 2D diffusion wave equation models were also calibrated to the same storms, but the results are not shown here.

The calibrated values of Tc and R for each of the 16 watersheds are referred in this paper to as the index Tc and R, or Tc, index and R, index. The magnitude of the Tc and R pairs for each excess rainfall pulse reflect the behavior of each watershed to storms with maximum excess rainfall intensity comparable to that of the storm used in calibration. The maximum excess rainfall intensity of the storms selected for calibration are also included in Table 3. They range from 0.20 to 1.16 in/h with an average value of 0.51 in/h.

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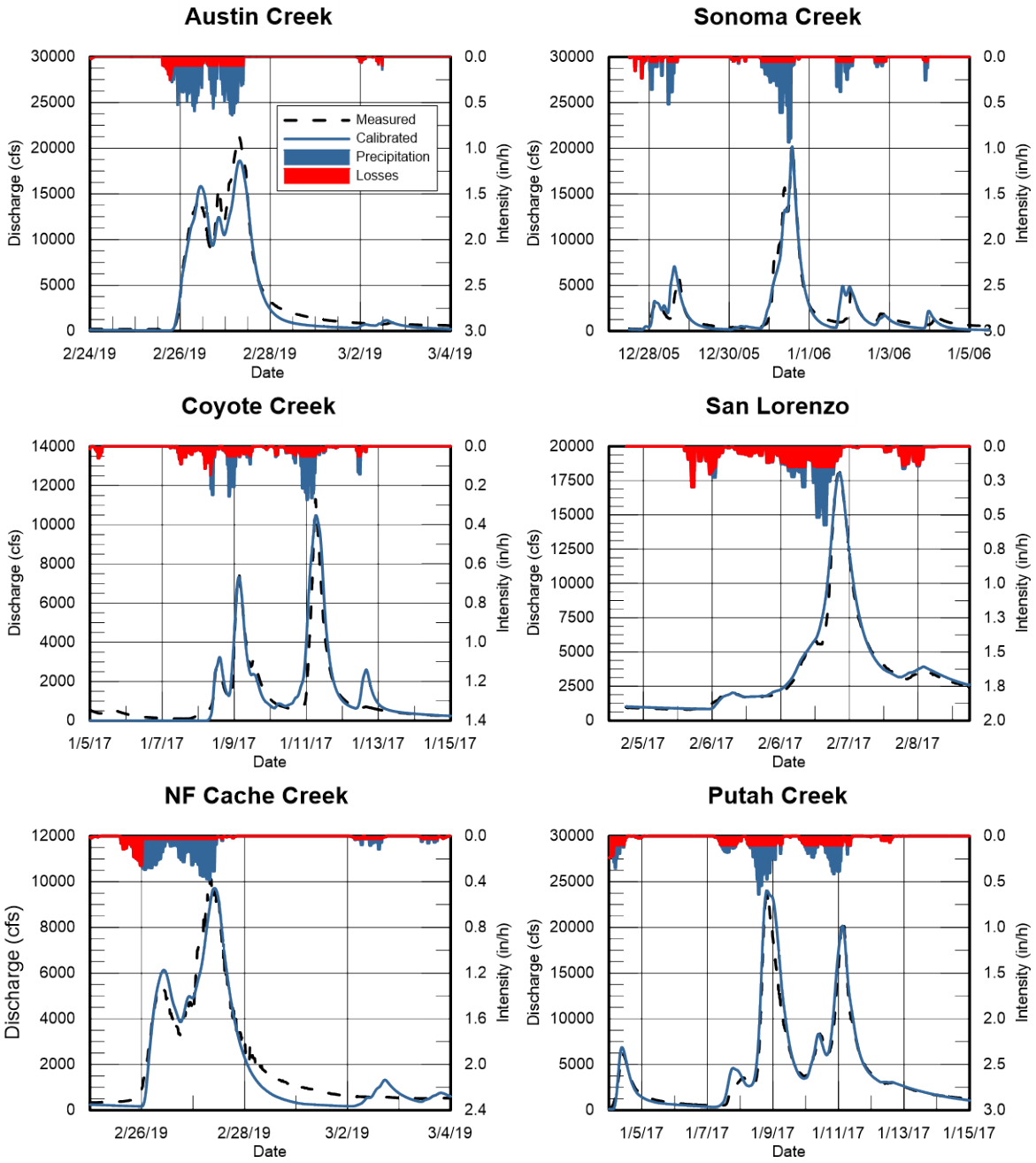


Figure 3. Calibration examples of six of the 16 watersheds selected for evaluation.

Watershed Response to Single Pulses of Excess Rainfall

Calibrated Clark and 2D diffusion wave watershed models were then subjected to single, 1-h-duration pulses of excess rainfall with pulses in the range of 0.25 in/h to 6 in/h. The hydrographs resulting from the Clark method were compared against those from the 2D diffusion wave transform method at each study location. Clark unit hydrograph parameters were systematically modified to minimize the differences with hydrographs resulting from the 2D model. For simplicity, the default time-area histogram included within the HEC-HMS Clark unit hydrograph transform was used within all modeling domains. Differences in time of peak and peak flow rates less than 1 hour and 3 percent, respectively, between the Clark unit hydrograph and 2D diffusion wave results were desired.

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Table 3 - Results of Watershed Calibration							
No.	Watershed Name	Tc (h)	R (h)	Max. Excess Rainfall Intensity (in/h)	Statistical Metrics		
					RSR	NSE	PBIAS
1	Austin C Nr Cazadero	3.0	5.0	0.48	0.3	0.92	-14.51
2	Sonoma C A Agua Caliente	1.6	4.3	0.68	0.4	0.81	-6.79
3	Coyote C Nr Gilroy	7.0	3.5	0.24	0.4	0.84	-0.93
4	San Lorenzo R A Big Trees	5.75	3.5	0.52	0.2	0.96	-1.62
5	Miguelito C A Lompoc	3.2	1.5	0.56	0.4	0.50	66.67
6	Rainbow C Nr Fallbrook	0.5	1.0	0.56	0.5	0.79	26.95
7	Snow C Nr White Water	4.0	1.5	0.64	0.5	0.71	-10.27
8	NF Cache C At Hough Spring Nr Clearlake Oaks	1.5	7.5	0.20	0.3	0.88	-2.54
9	Putah C Nr Guenoc	4.5	5.0	0.52	0.2	0.97	2.55
10	Bear Dam Inflow	2.5	1.5	1.16	0.4	0.81	-6.93
11	Owens Dam Inflow	2.5	0.5	1.16	0.3	0.90	10.98
12	SF Tule R Nr Chololllo Campground Nr Porterville	4.0	5.0	0.20	0.4	0.88	2.03
13	Marble Fork Kaweah R Ab Horse C Nr Lodgepole	0.75	1.5	0.12	0.7	0.57	4.77
14	W Fk Carson Rv At Woodfords	4.0	8.0	0.21	0.5	0.75	0.87
15	Truckee R Nr Truckee	7.0	8.0	0.68	0.39	0.84	8.53
16	Big Rock C Nr Valyermo	2.5	2.5	0.20	0.49	0.76	-1.65

Figure 4 shows an example of how Clark response hydrograph compares to response hydrographs from the 2D diffusion wave transform method for the Austin Creek near Cazadero watershed. The figure shows results for pulses of 1 in/h and 4 in/h. Clark Tc and R parameters were varied to minimize the error in peak flow, coincide the timing of the peak, and have similar slopes on the rising and falling limbs of the hydrograph. The results show how the peak flow for the 4 in/h pulse is more than fifteen times the peak flow for the 1 in/h pulse. This reflects the nonlinear behavior captured by the 2D transform.

Figure 5 shows the combined set of calibrated Clark response hydrographs for excess rainfall intensity pulses from 1 to 6 in/h for the same watershed. The plot on the left shows the response hydrographs with both axes in arithmetic scale. Response hydrographs for 1 in/h and 4 in/h, presented in Figure 4, are denoted with red lines in Figure 5 for reference. Several important insights can be deduced from this example. First is that the peak of the response hydrographs occurs at earlier times as the excess rainfall intensity increases. For example, the peak hydrograph flow for 4 in/h occurs 4 hours earlier than the peak for 1 in/h. Second, and as expected, is that the increase in peak flow is not linear. The peak flow for 4 in/h is about 45,700 cfs while the peak flow for 1 in/h is about 3,050 cfs.

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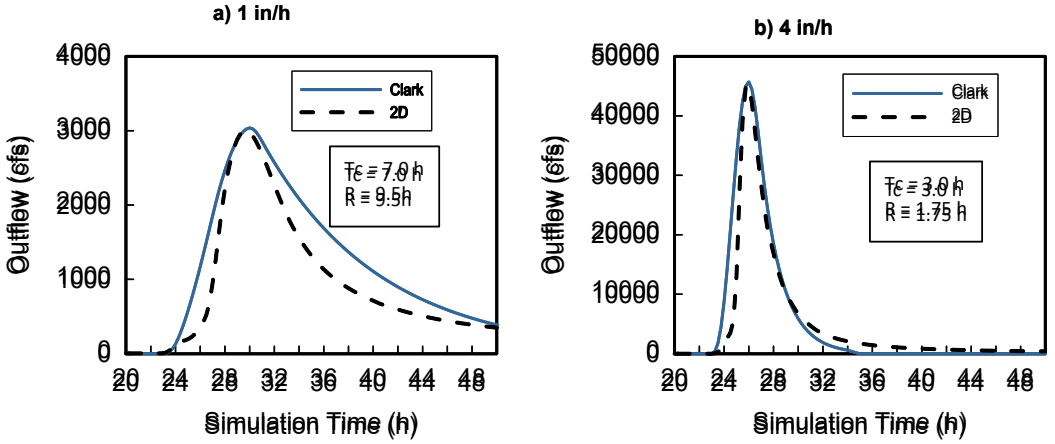


Figure 4. Examples of variable Clark parameter calibration for Austin Creek watershed for two excess rainfall intensity pulses.

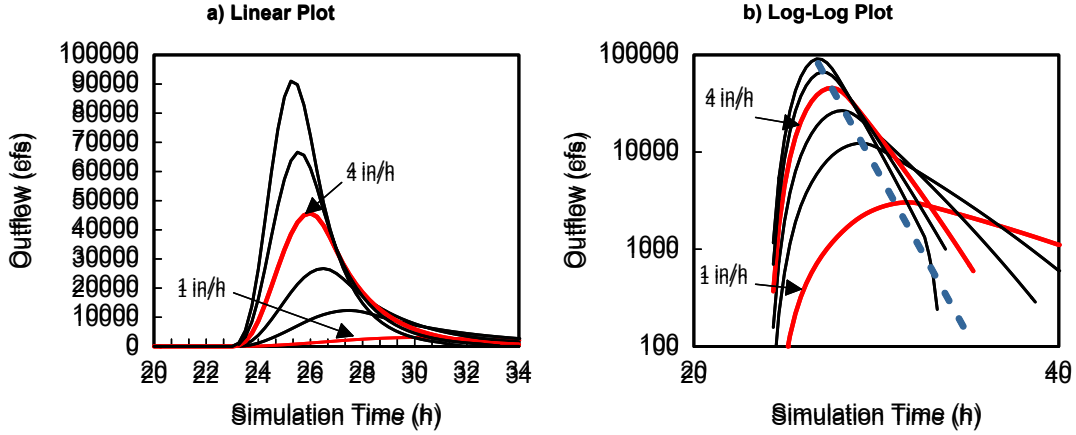


Figure 5. Combined set of response hydrographs for excess rainfall intensity pulses of 1 to 6 in/h for the Austin Creek Watershed. Plot on the left shows axes in linear scale, plot on the right shows axes in logarithmic scale. Response hydrographs for 1 in/h and 4 in/h are shown in red.

The plot on the right of Figure 5 shows the same response hydrographs but with both axes drawn in logarithmic scale. The peaks of the response hydrographs follow an approximate straight line, as indicated with the dashed line. A consequence of this nonlinear behavior is that for rare storms, where excess rainfall intensity exceeds the peak intensity of storms used for calibration, the linear convolution normally associated with the Clark unit hydrograph method would be incapable of capturing the large flows peaks derived from methods that capture nonlinear behavior.

Figure 6 presents the variation of the time of concentration and the storage coefficient with respect to the excess rainfall intensity for six of the 16 subject watersheds. In this figure, horizontal and vertical axes are plotted in logarithmic scale. The plots show that the relationship between the time of concentration and the storage coefficient with the excess rainfall intensity follows a power relationship, that is, the trend is linear in a log-log plot. Similar trends were noted by Meyersohn (2016) and Michailidi et. al., (2018) using different evaluation techniques. The inset on each graph shows the equation for the power relationship. The behavior for the other 10 watersheds used in the present study is similar to those in Figure 6 but not shown.

Figure 7 presents the variation of peak flow of response hydrographs against excess rainfall intensity. Peak flow increases following a power relationship with excess rainfall intensity. It is interesting to know that the increase in flow is highly nonlinear; for each log cycle of excess rainfall Intensity increase, peak flow increases by several log cycles. The exponential increase is visually depicted in Figure 5.

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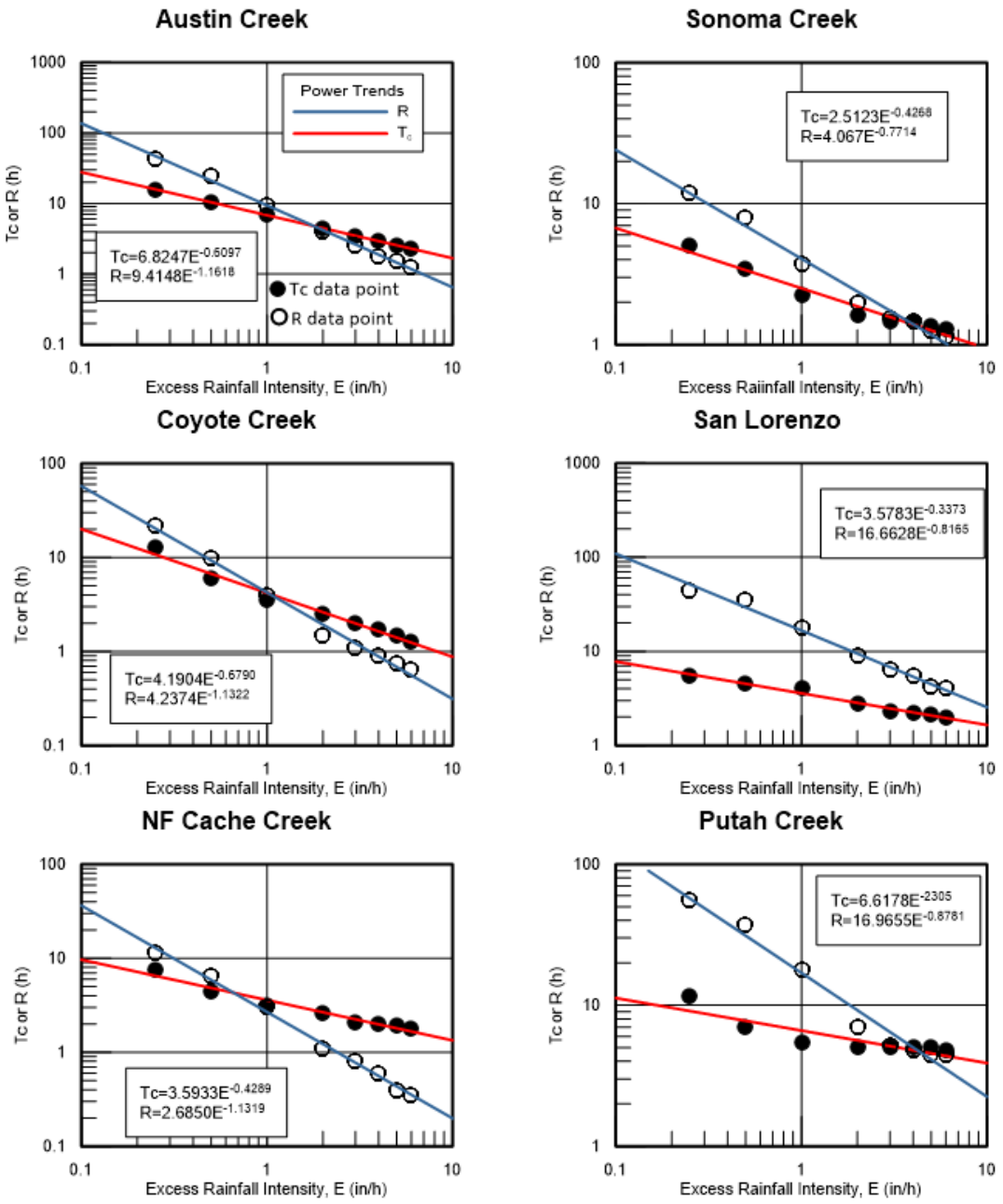


Figure 6. Variation of Tc and R with excess rainfall intensity for six of the 16 selected watersheds. Lines show regression trendlines and symbols show data points. The reduction in Tc and R follows a power relationship. Insets in each plot show power regressions.

IV. DEVELOPMENT OF REGRESSION EQUATIONS FOR THE VARIATION OF Tc AND R

The variation of Clark parameters with excess rainfall intensity is influenced by a variety of factors including drainage area, physical characteristics of the watershed, and stream network characteristics amongst others. In order to develop regression equations that predict how Clark unit hydrograph parameters change with excess rainfall intensity throughout the state of California, dimensionless ratios of Tc and R with respect to index values were developed. Thus, Tc and R ratios are expressed by the following equations:

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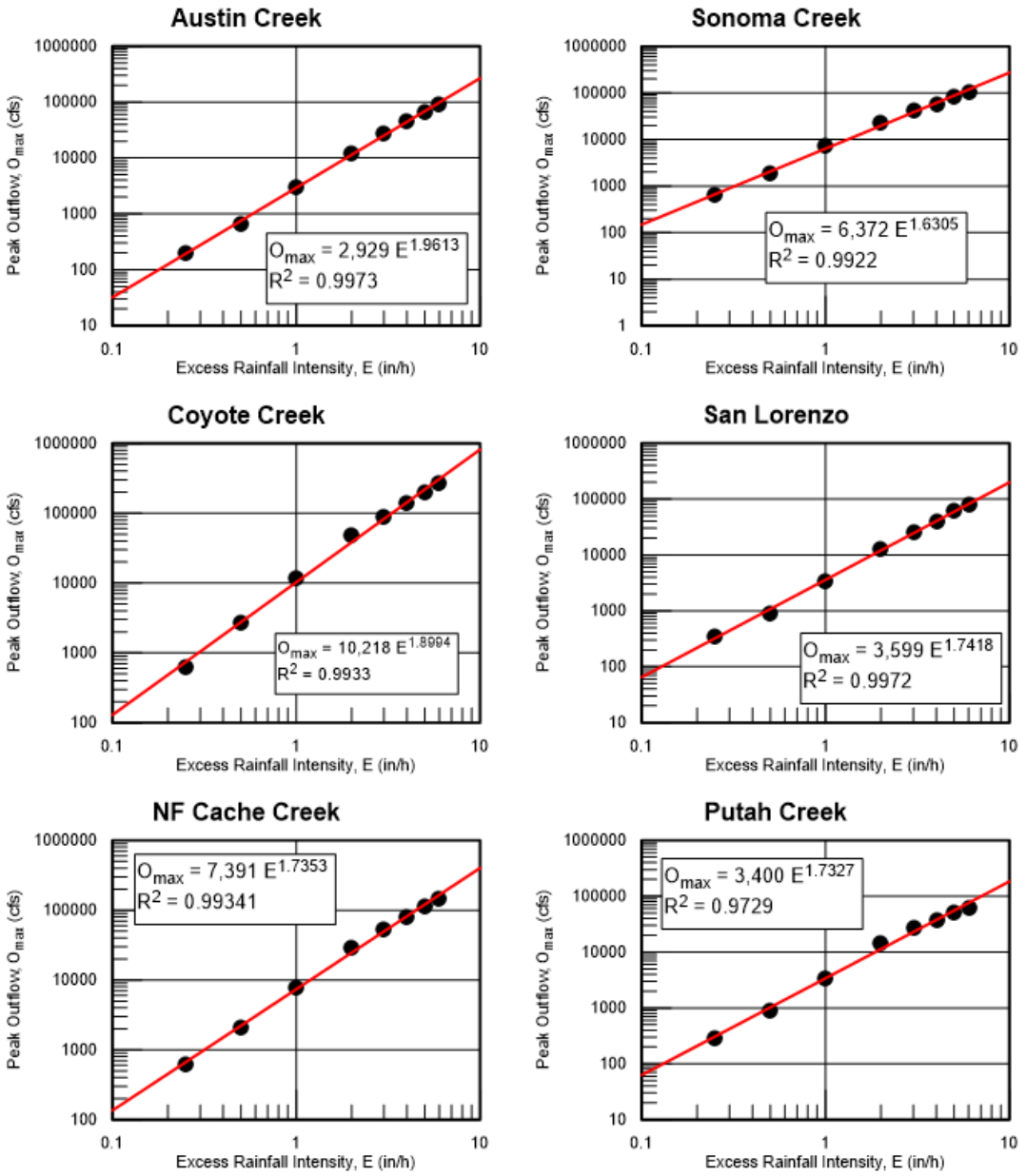


Figure 7. Variation of peak flow with excess rainfall intensity for six of the 16 selected watersheds. Peak flow increase follows a power relationship. Inset in each plot shows the power regression and the coefficient of determination, R^2.

$$Tc, ratio = \frac{Tc}{Tc,index} \quad (1)$$

$$R, ratio = \frac{R}{R,index} \quad (2)$$

where Tc, index and R, index represent the values of Tc and R obtained through calibration, which are presented in Table 3.

The statistical software R Statistical Language (R Core Team, 2021) was used to develop regression equations relating the Clark index parameters to physical characteristics of the watersheds for each of the excess rainfall intensity pulses used in the study.

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Two multiple linear regression models were considered within this study. The first model was of the form:

$$Y = a + b1X1 + b2X2 + \cdots bnXn \quad (3)$$

where Y is the dependent variable (e.g., Tc, ratio or R, ratio); a is a regression constant; *X1*, *X2, …, Xn* are independent variables (e.g., watershed characteristics); and *b1, b2, …, bn* are regression coefficients. The second model was of the form:

$$Log_{10}Y = a + b1Log_{10}X1 + b2Log_{10}X2 + \cdots bnXLog_{10}n \quad (4)$$

The regression constant and coefficients were determined utilizing the method of least squares. When using multiple linear regression with several watershed characteristics, some of the proposed characteristics may have little to no effect on the dependent variable of interest. When this occurs, these characteristics were removed from consideration (U.S. Army Corps of Engineers, 1982). In order to simultaneously evaluate the usefulness of the regression models and avoid overfitting, an adjusted coefficient of determination (adjusted R^2) was used. Adjusted R^2 ranges between -∞ and 1, where and adjusted R^2 = 1 is optimal. This metric increases when a new term improves the regression model more than would be expected by chance. Adjusted R^2, or $\bar{R}^2$, is computed using the following equation:

$$\bar{R}^2 = 1 - (1 - R^2)\frac{n-1}{n-p-1} \quad (5)$$

where p is the total number of explanatory (independent) variables, n is the sample size, and R^2 is the "classical" coefficient of determination. t-statistic tests and their associated p-values were used to determine whether there was a statistically significant relationship between an independent variable and the dependent variable of interest, that is whether or not the unknown coefficient(s) within Equations (3) or (4) are significantly different from zero. Regressions with p-values ≤ 0.05 were desired. Details on how the regression equations were developed and the watershed physical characteristics selected are presented in Bartles and Meyersohn (2023).

V. REGRESSION RESULTS

Regression equations were developed using all available data from the 16 watersheds and the results of the 2D diffusion wave equation. Regionalization of the regression equations was not attempted to maintain a reasonable sample size, reduce uncertainty in the results, and make the regression equations easier to apply. Additionally, the same predictive variables were used for each excess rainfall intensity to facilitate the use of the regression equations. This parameter selection also mitigates any potential for sharp inflection points in the output or increasing ratios of Tc and R as excess rainfall intensity increases for reasonable parameter values.

The physical parameters that produced acceptable adjusted R^2 values for each excess rainfall intensity are:

- DA, the Drainage Area expressed in square miles.
- $S_{10\text{-}85}$, the average slope along the $L_{10\text{-}85}$ stream. $L_{10\text{-}85}$ is a subset of the longest watercourse of the watershed, L, beginning at a point representing ten percent of L and extending to a point representing eighty-five percent of L.
- L_{ca}, the Centroidal Flowpath Length. It is the length along the longest watercourse from the watershed outlet to a point on the stream nearest the watershed centroid. This parameter is in miles.
- BR, the Basin Relief. It is the difference between the highest elevation on the drainage divide and the elevation of the outlet point of the subbasin. This parameter is in feet.
- RR, the Relief Ratio. It is the ratio of the Basin Relief, BR divided by the length of the longest watercourse of the watershed, L.

A summary of the regression equations for each excess rainfall intensity is presented in Table 4.

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Table 4 – Regression Equations to estimate Tc, ratio and R, ratio	
Excess Rainfall Intensity (in/h)	**Equations**
0.25	$log_{10}(Tc, ratio) = 0.04082 - 0.35461 * log_{10}DA - 0.53746 * log_{10}S_{10-85}$
	$R, ratio = -1.01528 + 1.7684 * L_{ca} - 0.00316 * BR + 120.4306 * RR$
0.50	$log_{10}(Tc, ratio) = -0.09866 - 0.42538 * log_{10}DA - 0.5683 * log_{10}S_{10-85}$
	$R, ratio = -1.40573 + 1.2471 * L_{ca} - 0.00208 * BR + 78.6917 * RR$
1.0	$log_{10}(Tc, ratio) = -0.2412 - 0.5586 * log_{10}DA - 0.6837 * log_{10}S_{10-85}$
	$R, ratio = -0.26226 + 0.6196 * L_{ca} - 0.00118 * BR + 38.7834 * RR$
2.0	$log_{10}(Tc, ratio) = -0.3810 - 0.5353 * log_{10}DA - 0.6581 * log_{10}S_{10-85}$
	$R, ratio = -0.5070 + 0.2515 * L_{ca} - 0.00063 * BR + 16.721 * RR$
3.0	$log_{10}(Tc, ratio) = -0.4809 - 0.5387 * log_{10}DA - 0.6703 * log_{10}S_{10-85}$
	$R, ratio = 0.489 + 0.1709 * L_{ca} - 0.00046 * BR + 11.3057 * RR$
4.0	$log_{10}(Tc, ratio) = -0.5446 - 0.5107 * log_{10}DA - 0.6573 * log_{10}S_{10-85}$
	$R, ratio = 0.4668 + 0.13062 * L_{ca} - 0.00037 * BR + 8.4999 * RR$
5.0	$log_{10}(Tc, ratio) = -0.5425 - 0.5204 * log_{10}DA - 0.6457 * log_{10}S_{10-85}$
	$R, ratio = 0.4814 + 0.10512 * L_{ca} - 0.00033 * BR + 7.2064 * RR$
6.0	$log_{10}(Tc, ratio) = -0.5408 - 0.5403 * log_{10}DA - 0.6452 * log_{10}S_{10-85}$
	$R, ratio = 0.5215 + 0.0824 * L_{ca} - 0.00029 * BR + 5.9854 * RR$

Validation of Regression Equations

The predictive capabilities of the regression equations were evaluated through a validation process. Three independent watersheds, not included in the set of 16 watersheds used preciously, were selected for validation. Information about the three validation watersheds is presented in Table 5. The first step was to calibrate the three selected watersheds. Hydrologic models for each of the validation watersheds were constructed using the Clark Unit Hydrograph transform and constant Clark parameters estimated through calibration. Table 6 shows the result of the calibrations. These calibrated parameters are considered the index values. Then, variable Clark parameters were estimated using the regression equations in Table 4 with the index Tc and R from calibration.

Since there are no available recorded storms with rainfall intensities comparable to those commonly used in dam safety (i.e., Probable Maximum Precipitation, PMP), we used the 2D diffusion wave transform method to generate hydrographs that can be used to assess the benefits of the variable Clark transform method. Therefore, the 2D diffusion wave parameters for the validation watersheds were subjected to a calibration process using the same storms indicated in Table 5.

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Table 5 - Watersheds Selected for Validation							
No.	**Watershed Name**	**Lat**	**Long**	**USGS Stream Gage Number**	**Area (mi^2)**	**Event**	**Peak Flow (cfs)**
17	EF Russian R Nr Calpella CA	39.246	-123.130	11461500	92.5	Dec 2005-Jan 2006	14,600
18	Arroyo Seco Nr Pasadena CA	34.222	-118.178	11098000	16.1	Jan 2010	4,230
19	Elder C near Paskenta CA	40.025	-122.510	11379500	92.3	Feb 2019	12,700

Table 6 - Results of Calibration for Verification Watersheds							
No.	**Watershed Name**	**Tc (h)**	**R (h)**	**Max. Excess Rainfall Intensity (in/h)**	**Statistical Metrics**		
					RSR	**NSE**	**PBIAS**
17	EF Russian R Nr Calpella CA	5.0	8.0	0.40	0.30	0.90	-4.50
18	Arroyo Seco Nr Pasadena CA	1.6	0.75	0.66	0.20	0.96	11.9
19	Elder C near Paskenta CA	4.0	3.0	0.20	0.2	0.95	-0.41

It is important to note that the regression equations could result in values of Tc and R larger than calibrated Clark parameters at low excess rainfall rates. This is because the regression equations were derived from a range of excess rainfall intensity pulses and not from recorded storms. Therefore, the magnitude of Tc and R computed from the regression equations needed to be limited to values less than or equal to their calibrated values. As an example, Figure 8 shows the variation of Tc and R from the regression equations for the EF Russian River Watershed. In this case, maximum values of Tc and R are capped at 5 and 8 h, respectively.

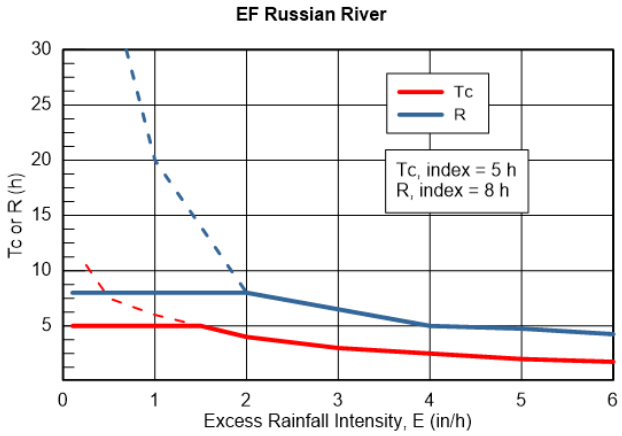


Figure 8. Example of computed variation of Tc and R with excess rainfall intensity for the EF Russian River watershed. Magnitude of Tc and R is capped at their calibrated (index) values.

Following the calibration of the 2D diffusion wave transform parameters, several large hypothetical storms, were simulated for each validation watershed and the results used to verify whether the variable Clark parameters improve predictive capabilities compared to traditional constant Clark parameters models. Because there are not recorded storms of comparable magnitude as those used in dam safety evaluations

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(e.g., Probable Maximum Precipitation, PMP; 1/1,000 Annual Exceedance Probability, AEP), results from the 2D diffusion wave transform method were used for comparison.

The first verification was to compare the predictive capabilities of variable and constant Clark parameters with an approximation of the PMF (Probable Maximum Flood) simulated for the EF Russian River watershed using a rainfall hyetograph from a recent dam safety study (U.S. Army Corps of Engineers, 2009). The 72-h PMP at this location is about 30.5 in. The results of the simulations are presented in Figure 9, which shows hydrographs resulting from the 2D diffusion wave transform, the variable Clark method, and the constant Clark method. Note that the peak excess rainfall intensity of about 2.1 in/h is approximately 5 times larger than that which was realized during model calibration (0.4 in/h), indicating Tc and R were reduced during the simulation run for the variable Clark method. The results show that the hydrograph computed using the variable Clark methods closely resembles the one computed with the 2D diffusion wave transform, closely matching peak flow, time of peak, and general hydrograph shape. The peak flow from the variable Clark method is only 8.2 percent higher than the 2D model. In contrast, the constant Clark method underpredicts peak flow by about 27 percent and the resulting hydrograph is significantly different than the one from the 2D model. The results show that the variable Clark method improves predictive capabilities over the traditional constant Clark method for large infrequent storms. Not including the reduction of Clark transform parameters for these large precipitation events could underestimate peak inflows and reservoir stage.

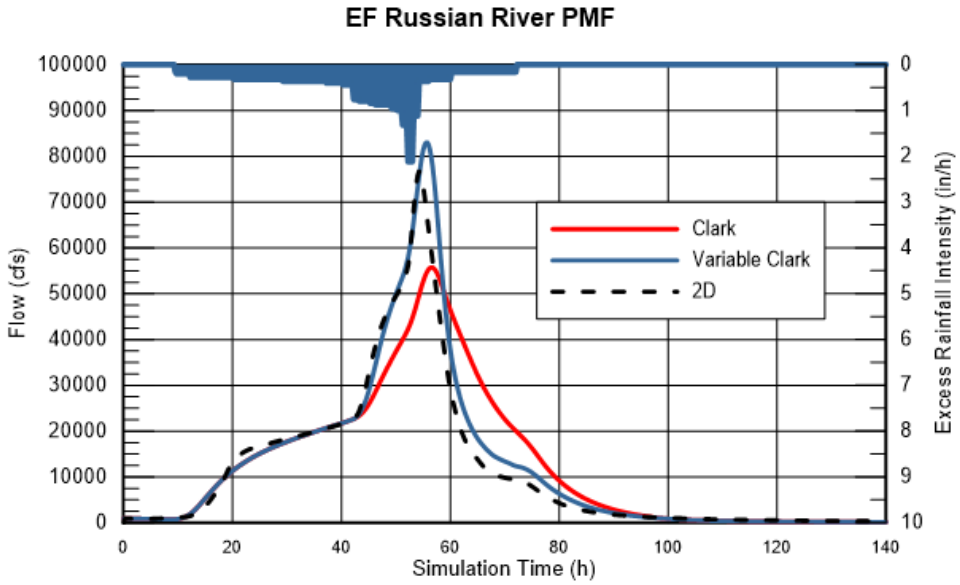


Figure 9. Comparison of hydrographs computed using constant Clark method, variable Clark method, and 2D diffusion wave transform method for a PMP storm – EF Russian River watershed.

The second verification involves a shorter, 24h-duration event, with an annual exceedance probability (AEP) of 1/1,000 for the Arroyo Seco watershed. Depth-duration data were obtained from Atlas 14, Volume 6 (National Weather Service, 2014). The total 24h rainfall for this event is about 14.6 in. The storm hyetograph was constructed using the same temporal distribution as the January 2010 storm used to calibrate the watershed. Figure 10 presents the results of the simulations. The hydrograph computed from the variable Clark method closely resembles the hydrograph computed with the 2D diffusion wave transform method, with similar peak flow magnitude and peak timing. The error in peak flow is only 2.8 percent. In contrast, the constant Clark method underpredicts peak flow by about 29 percent, and the hydrograph shape is quite different than the reference 2D hydrograph. Note that the peak excess rainfall intensity of 4.8 in/h is approximately seven times larger than the peak excess rainfall intensity realized during model calibration (0.66 in/h), indicating that variable Clark parameters were indeed triggered during the variable Clark simulation.

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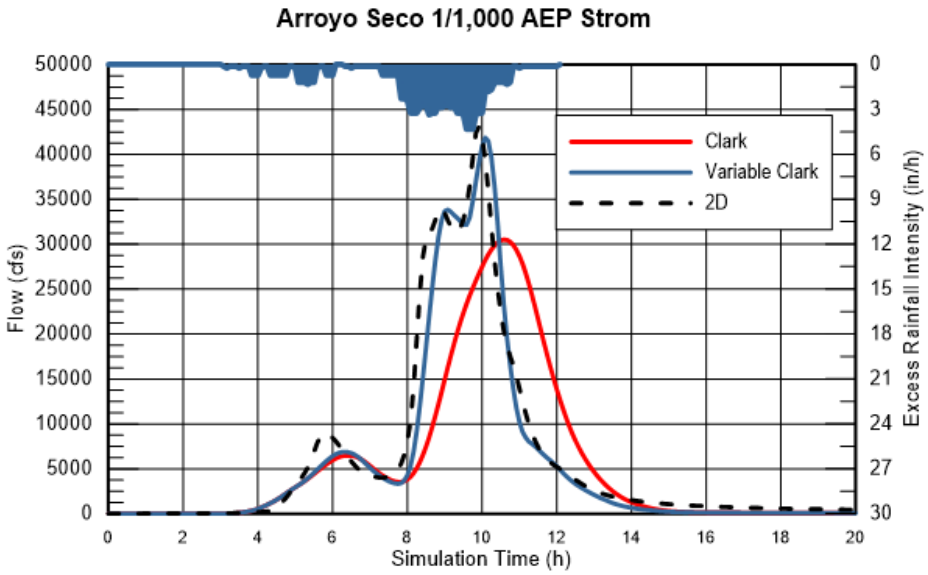


Figure 10. Comparison of hydrographs computed using constant Clark method, variable Clark method, and 2D diffusion wave transform method for a 12-h, 1/1,000 AEP storm – Arroyo Seco watershed.

The last verification was simulated on the Elder Creek watershed using a 72h duration storm with an AEP of 1/1,000. The depth-duration was obtained from the Atlas 14, Volume 6 as well. The 2019 temporal rainfall pattern was used to distribute the precipitation in time. Figure 11 shows the results. The excess rainfall intensity hyetograph shows a peak excess of about 0.48 in/h which is not much higher than the excess rainfall intensity used for calibration (0.2 in/h). The results show that the constant and variable Clark models provide quite similar hydrographs with peak discharges about 11 percent lower than the 2D diffusion wave transform. The similarity of the constant and variable Clark methods is a consequence of the variable Clark parameters being capped at their calibrated values for most of the peak excess rainfall pulses. Nevertheless, the results are well within the bounds of acceptability.

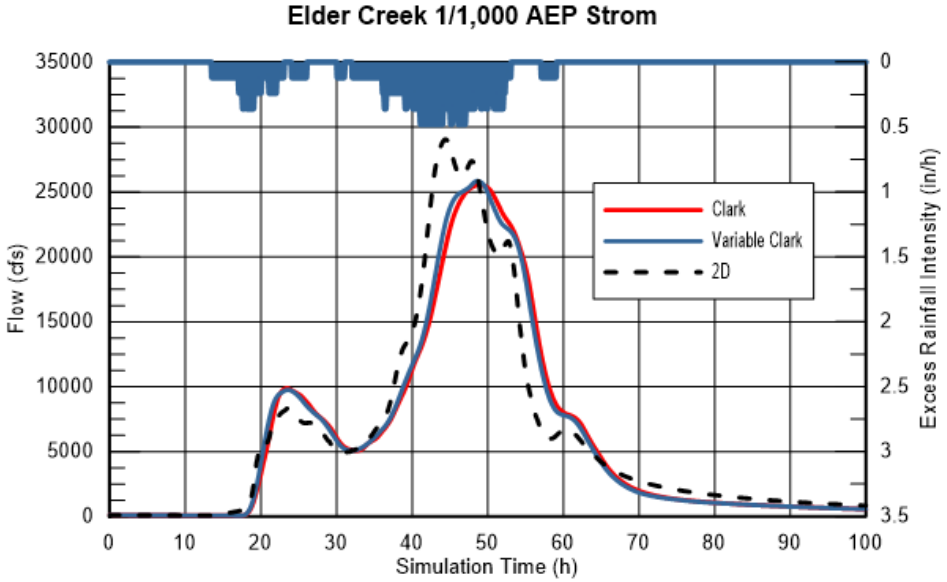


Figure 11. Comparison of hydrographs computed using constant Clark method, variable Clark method, and 2D diffusion wave transform method for a 72-h, 1/1,000 AEP storm – Elder Creek watershed.

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Goodness of Fit

The regression equations presented previously represent the most likely Tc or R value for a given location. However, the regression equations contain uncertainty which cannot be completely eliminated. This uncertainty stems primarily due to the difficulty in simulating the complex hydraulics from a 2D diffusion wave transform using only two parameters (Tc and R). Table 7 shows model statistical parameters of the regression equations for each of the excess rainfall intensities. While some p-value statistics are larger than 0.05, the regression equations produce acceptable model fits for the excess rainfall rates when predicting ratios of Tc and R relative to an index parameter.

Table 7 – Regression Equations Model Statistics

Rainfall Intensity (in/h)	Tc, ratio			R, Ratio		
	$\bar{R}^2$	F-Statistic	p-value	$\bar{R}^2$	F-Statistic	p-value
0.25	0.2509	3.512	0.06033	0.5263	6.556	0.00713
0.5	0.3156	4.459	0.03354	0.4625	5.303	0.01471
1	0.4532	7.216	0.00779	0.3570	3.777	0.04055
2	0.4272	6.594	0.01054	0.2927	3.069	0.06891
3	0.4382	6.851	0.00929	0.3142	3.291	0.05811
4	0.3838	5.671	0.01695	0.2695	2.845	0.08233
5	0.3449	4.949	0.02523	0.2441	2.615	0.09924
6	0.3288	4.674	0.02955	0.1792	2.092	0.15480

Figure 12 shows a comparison of Tc, ratio and R, ratio obtained from the 2D calibration and from the regression equations for excess rainfall intensities of 1, 2, and 4 in/h. Even though the data points do not follow the perfect fit (diagonal line), the validation examples presented above show that the regression equations are capable of successfully reproducing extreme precipitation events when compared to the 2D transform model.

Despite the less-than-ideal model statistics, the validation runs show that the regression equations are capable to reproducing hydrographs from 2D transform models, especially when excess rainfall intensities are large enough to trigger reduction in Tc and R. Model statistics can be improved by considering additional watershed to the pool of 16 selected for this study. In addition, calibrating watersheds to more than one storm would increase confidence in the magnitude of Tc, index and R, index, improving the model statistics as well.

A significant finding from the validation examples presented in this section is that the constant Clark unit hydrograph method, which is often calibrated to smaller storm events, may underpredict flows when compared to the variable Clark method or the 2D diffusion wave transform method for storms that generate higher excess rainfall intensity pulses. This implies that constant Clark parameters developed from calibration might be inadequate for extremely large storms used in dam designs. Since the constant Clark method has been widely used for hydrologic dam safety evaluations, it is quite likely that these evaluations underpredict both inflow to reservoirs and reservoir stages.

VI. CONCLUSIONS

The nonlinear response of watersheds to large excess rainfall intensity is explored by means of 2D diffusion wave equation methods implemented in the hydrologic computer software HEC-HMS. Response hydrographs to excess rainfall pulses can be reproduced using the Clark transform method, whereby Clark parameters, Tc and R, are reduced with increasing excess rainfall intensity. The magnitude of Clark parameters decreases with increasing excess rainfall intensity following a power relationship.

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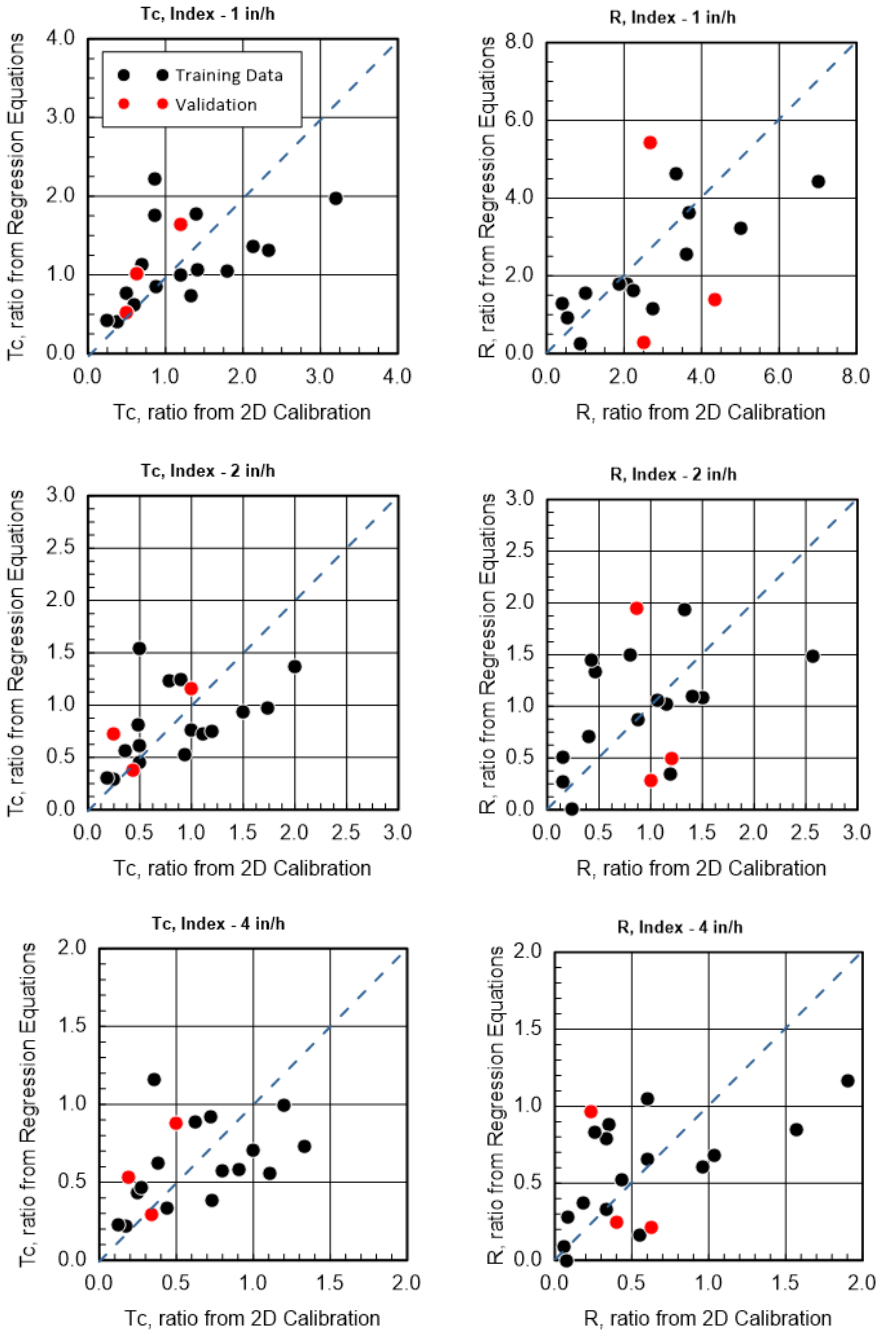


Figure 12. Comparison of Tc, index and R, index values from the 2D calibration and regression equations for excess rainfall intensities of 1, 2 and 4 in/h. Black dots show training data from the 16 watersheds. Red dots show data from the three validation watersheds. Diagonal dashed lines show a perfect fit.

To facilitate the estimation of the reduction of Clark parameters with increasing excess rainfall intensity, regression equations for the state of California are presented for a range of excess rainfall intensities. The regression equations use physical watershed parameters to estimate the variation of time of concentration and storage coefficient for excess rainfall intensities in the range of 0.25 to 6.0 in/h. Variable Clark transform parameters should be used when performing hydrologic routing on extreme precipitation events, such as the Probable Maximum Precipitation (PMP), or rainfall scenarios where peak excess rainfall intensity exceeds the ones used for calibration or for the development of local regression equations.

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Verification of the variable Clark method shows improved predictive capabilities when compared to results from the traditional constant Clark transform method. Not accounting for the reduction of Clark parameters for the large events use for dam safety could underestimate peak reservoir inflows and reservoir stage.

The regression equations presented in this paper were derived from an analysis of only 16 watersheds. Potential expansion of the dataset by including additional watersheds and performing calibration to more than one storm per watershed would improve the statistical significance of the regression equations. While the work presented in this paper applies only to California, the methodology can be expanded to cover other geographic areas where well instrumented watershed information is available.

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Development of a Consistent Framework: Risk Prioritization for 35 Unsafe, High Hazard Dams in Massachusetts

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ABSTRACT

Dam owners/regulators are responsible for addressing risks related to unsafe dams such as loss of life from predictable and unpredictable catastrophic events including dam failure. Aging infrastructure, limited resources, and increasing development along downstream reaches complicate this responsibility. Funding from the FEMA High Hazard Potential Dams (HHPD) Grant Program is helping owners/regulators address risks associated with dam safety issues.

The Massachusetts Department of Conservation & Recreation Office of Dam Safety (MA ODS) realized the importance of the FEMA HHPD program in addressing dam safety issues related to dams in Poor or Unsafe condition in Massachusetts. From a large portfolio of more than 300 high hazard dams, MA ODS identified 35 dams that would qualify for HHPD funding. With the FEMA funding award, MA ODS embarked on a portfolio screening level risk analysis for the 35 dams. The two objectives were to develop a risk prioritization matrix to allow for qualitative evaluation and comparison of the 35 dam and to rank the dams in order of priority for rehabilitation, partial breach, or removal (i.e., repair or removal). Consistency and efficiency were predominant goals in developing the risk screening program for the 35 high hazard dams. The approach was especially challenging given the inconsistencies in previous studies.

MA ODS will use the findings of risk informed dam prioritization to apply for future HHPD grant funding to repair/remove the dams, starting with those that pose the greatest risk to the public. This paper provides a summary of a combined quantitative and qualitative risk analysis framework developed to rank the 35 high hazard dams in the MA ODS portfolio based on likelihood of failure and consequences.

I. INTRODUCTION

In 2019 the Federal Emergency Management Agency (FEMA) began awarding High Hazard Potential Dam (HHPD) Grant Program funds to states for the repair or removal of high hazard dams. The main objective of the High Hazard Potential Dam (HHPD) grant program is to provide technical, planning, design, and construction assistance to dam owners for the repair or removal of eligible high hazard dams [3].

For a dam to be eligible for HHPD funding the dams must meet four key factors: 1) located in a state with a dam safety program, 2) classified as a high hazard dam, 3) have an Emergency Action Plan, and 4) fail to meet dam safety standards and pose an unacceptable risk to the public. For a state or organization to be considered for grant funding the state and communities are required to have an approved Hazard Mitigation Plan (HMP) and participate in the National Flood Insurance Program (NFIP).

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Dam owners can evaluate the risk associated with their dams, or a portfolio analysis can be performed for a group of dams. Massachusetts has over 300 high hazard potential dams. The Massachusetts Office of Dam Safety combed through this portfolio of dams and identified a total of 35 dams that are eligible for HHPD funds. Once the HHPD eligible dams were identified, GEI embarked on a risk prioritization for a portfolio of dams which may qualify for HHPD funding. The two objectives were to develop a risk prioritization matrix to allow for qualitative evaluation and comparison of the 35 dam and rank the dams in order of priority for rehabilitation, partial breach, or removal.

To perform a risk prioritization for the portfolio of 35 dams we set out to complete the following tasks:

1. Evaluate the likelihood of failure for each dam under static, seismic, and hydrologic loading conditions.
2. Develop a dam break model for each dam using DSS-WISE™ and estimate consequences including population at risk (PAR), economic/environmental losses, and community impact.
3. Compile a risk-based prioritization matrix based upon likelihood and consequences of failure.
4. Rank the dams based upon level of risk to the public.

While completing these tasks it was important for the project to be as objective, reproducible, and systematic as possible. The portfolio analysis, which involved ranking each dam according to likelihood of failure and consequences, will serve as a foundation for future FEMA HHPD grant applications for repair or removal of dams in Massachusetts.

II. LIKELIHOOD OF FAILURE ANALYSIS

A. Review of Existing Information

For each dam in the portfolio, the project involved review of the most recent Phase I Inspection Reports, Emergency Action Plans (EAPs), and inundation maps. When available, review also included Phase II Investigation and Alternatives Analysis Evaluation Reports, Follow-up Inspection Reports, MA General Law (M.G.L.) Chapter 253 applications (applications for permits required in MA to construct, repair, materially alter, breach or remove a dam), and information from readily available online sources.

While reviewing the existing information, we compiled key dam characteristics in a Microsoft Excel spreadsheet with quantitative and qualitative information (i.e., dam names, vertical datums, dam and spillway elevations, embankment, crest, and spillway dimensions, normal pool elevations, factors of safety, previously performed studies and analyses, previously identified deficiencies, etc.). To inform the likelihood of failure designation for a given dam, the compilation included a summary of findings from previous structural stability, hydrology & hydraulic, and seismic analyses.

B. Development of Likelihood of Failure

Consistent with the FEMA HHPD guidelines [3], the risk prioritization process involved designating “likelihood of failure” using qualitative terms of “Low, Moderate, High, and Very High” as follows:

- Very High – Failure is likely to occur in less than 10 years.
- High – Failure is likely to occur in 10 to 100 years.
- Moderate – Failure is likely to occur in 100 to 1,000 years.
- Low – Failure is likely to occur in > 1,000 years.

The bullets above present time frames, which suggest the periods within which a failure might occur starting from present day. In other words, the time frames above do not appear to represent annual exceedance probabilities such as a 100-year event, which typically indicates a 1% chance of occurrence in any given year.

For each dam we estimated the dam’s likelihood of failure under the following three failure modes:

- Static Failure (fair weather breach and failure at normal pool).
- Hydrologic Failure (dam breach at peak Spillway Design Flood (SDF) water level).

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- Seismic Failure (fair weather breach and failure at normal pool under seismic conditions).

To assess likelihood of failure across the portfolio of dams in a consistent manner, we developed an approach that included a list of evaluation criteria for each failure mode and each dam. Our approach recognized that not all dams had comprehensive studies, dams with comprehensive studies varied by the type and extent of the analyses, and each dam was unique with the structures, materials, construction, and age.

We assigned one likelihood of failure category for each of the three modes. The assessment included quantitative information such as slope stability Factors of Safety (FS) and hydraulic capacity of the dam, as well as qualitative information provided by dam section (embankments, spillway, dikes, etc.) observations, and other descriptions of dam deficiencies.

If key information was missing, we elected to increase the likelihood of failure to the next higher category (i.e., “Low” was increased to “Moderate" for seismic if factors of safety were not available). A likelihood of “Low” was assigned only for dams where we identified sufficient and appropriate information in the existing technical reports that supported this ranking. Evaluation criteria for static, hydrologic, and seismic failure modes are provided in Table 1.

Table 1. Relative Likelihood of Failure Criteria

Relative Likelihood of Failure	Static Failure[1]	Hydrologic Failure[2]	Seismic Failure[3]
Very High	$1 \geq FS$ OR Most recent inspection indicates severe deficiencies that are likely to cause failure of the dam in the short-term	Dam can pass less than a 10-year storm without overtopping OR $1 \geq FS$	$1 \geq FS$ AND Subsurface investigation identified the presence of liquefiable soils
High	$1 < FS \leq 1.1$ OR Most recent inspection indicates one or more major deficiencies related to dam condition	Dam can pass between a 10-year and a 100-year storm without overtopping OR $1 < FS \leq 1.1$	$1 \geq FS$ AND Subsurface investigation found that liquefaction of the soil is not anticipated
Moderate	$1.1 < FS \leq 1.2$ OR Most recent inspection indicates one or more significant deficiencies related to dam condition	Dam can pass between a 100-year and 500-year storm without overtopping OR $1.1 < FS \leq 1.2$	$1 < FS \leq 1.1$ OR Subsurface investigation found that liquefaction of the soil is not anticipated
Low	$1.2 < FS$ OR Most recent inspection indicates no significant deficiencies	Dam can pass between a 500-year and a ½ Probable Maximum Flood storm without overtopping OR $1.2 < FS$	$1.1 < FS$ OR Subsurface investigation found that liquefaction of the soil is not anticipated

Notes:

1. Static Failure – Deficiency descriptions and examples
   a. "Severe” includes high rates of seepage, actively failing or collapsed portions of the structure, and large sink holes.
   b. “Major” includes areas of major erosion, uncontrolled seepage, major concrete spalling or deterioration, and heavy presence of trees or other mature vegetation on embankments.
   c. "Significant” includes bare soils, mortar missing from masonry walls, lack of erosion protection on embankments, cracked concrete, and presence of concrete efflorescence.
2. Hydrologic Failure designations were based on a review of the capacity to pass the SDF in the Phase I reports. If the dam could not pass the SDF, we determined the annual recurrence interval storm that the dam could pass without overtopping by reviewing previous studies provided by MA ODS FEMA Flood Insurance Studies, and/or USGS StreamStats estimates.
3. Seismic Failure – The likelihood of a large seismic event in Massachusetts is relatively low. Most dams have a 2% Probability of Exceedance in 50 years peak ground acceleration of less than 0.2g. However, we determined our likelihood of failure value assuming an earthquake occurs.

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C. Summary of Results

Table 2 summarizes the estimated likelihood of failure for the 35 dams based on the static, hydrologic, and seismic scenarios. Table 3 provides a summary of the number of dams assigned to each likelihood of failure for each failure mode.

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Table 2. Likelihood of Failure Results Summary Table

Dam No.	Likelihood of Failure		
	Static Failure Mode	Hydrologic Failure Mode	Seismic Failure Mode
1	Very High	High	High
2	High	Moderate	High
3	Very High	High	High
4	Moderate	Moderate	High
5	Moderate	High	Moderate
6	Very High	Very High	High
7	High	High	Moderate
8	High	Moderate	High
9	High	Moderate	High
10	Moderate	Moderate	Very high
11	High	Moderate	Moderate
12	Very high	High	High
13	High	Very High	High
14	Very High	Very High	Moderate
15	Moderate	Moderate	High
16	High	High	High
17	High	Moderate	Moderate
18	Moderate	High	Low
19	High	Moderate	Moderate
20	Very High	Very High	High
21	Very High	Moderate	High
22	High	Moderate	Moderate
23	Very High	Moderate	High
24	High	Moderate	Moderate
25	High	High	Moderate
26	High	High	High
27	High	Moderate	Moderate
28	High	Moderate	High
29	Very High	Very High	High
30	Very High	Very High	High
31	High	High	Moderate
32	High	High	Low
33	High	Very High	Moderate
34	Very High	High	High
35	High	Moderate	Moderate

Table 3. Likelihood of Failure Results Summary Table

Likelihood of Failure	Failure Mode		
	Static	Hydrologic	Seismic
Very High	11	7	1
High	19	12	19
Moderate	5	16	13
Low	0	0	2

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III. HYPOTHETICAL DAM BREACH MODELING

A. DSS-WISE™ Standard Approach Model Set Up

We used the FEMA DSS-WISE™ modeling program [1] to simulate the hypothetical failure of each dam for three failure modes: static, seismic, and hydrologic. DSS-WISE™ is a web-based, automated two-dimensional dam-break modeling and mapping software platform that facilitates efficient model setup and a stable model environment that can handle unsteady flows and wet/dry conditions. DSS-WISE™ uses the full shallow water governing equations and produces geospatial files compatible with ArcGIS and FEMA HAZUS. DSS-WISE™ was designed and is maintained by The National Center for Computational Hydroscience and Engineering at the University of Mississippi (UM-NCCHE).

The terrain within the DSS-WISE™ program is automatically generated from digital elevation data available for each region. Data includes, for example, web-based USGS 1/3 arc-second (i.e., approximately 10 meters) National Elevation Dataset (NED) tiles, and from Light Detection and Ranging (LiDAR) survey tiles of varying detail derived from the United States Geological Survey (USGS). The NED tiles are the base layer for the DSS-WISE™ Digital Elevation Model (DEM) and, in locations where detailed LiDAR is available, DSS-WISE™ incorporates the LiDAR into the DEM. The DEM horizontal datum is referenced to the North American Datum of 1983 (NAD 83) and the vertical datum is referenced to the North American Vertical Datum of 1988 (NAVD88). We converted dam and reservoir elevations originally referenced to the National Geodetic Vertical Datum of 1929 (NGVD) or a local/site datums to NAVD88 based on the National Geodetic Survey Coordinate Conversion and Transformation Tool [8], a conversion indicated on the project drawings, or information provided by local town officials.

We developed inputs to DSS-WISE™ based on information from Phase I and Phase II Inspection reports, EAPs, and/or Hydrology & Hydraulic (H&H) studies. Information about our standard approach to the DSS-WISE™ software inputs for reservoirs & dams, breach parameters, and simulation parameters are presented in more detail below.

B. DSS-WISE™ Inputs

In DSS-WISE™, the “Reservoir Details” input parameters generate an interpolated bathymetric surface upstream of the dam using each dam’s stage and storage information. To develop the bathymetric surface, we input the elevation (ft, NAVD88) and storage (acre-ft) values associated with normal pool and maximum pool (i.e., top of dam). The failure pool elevation and storage are also entered.

The “Dam Details” input parameters generate the inline structure (i.e., dam, spillway, or dike) that will be breached. The dam/spillway/dike crest line is drawn in the DEM to match the length noted in the Phase I reports. We then input the dam’s crest elevation, structure height, and structure type (embankment, gravity, or concrete arch).

Breach parameters include breach geometry and time to failure, which is also referred to as formation time. The DSS-WISE™ program can perform two types of breach simulations: a reservoir-type and a hydrograph-type, with each breach type allowing options for breach geometry and time to failure.

A reservoir-type simulation models the reservoir impounded by the dam. The dam breach is modeled by entering a user-defined average breach width and breach formation time, as discussed below. During a reservoir-type simulation the outflow hydrograph is calculated as the breach widens, water flows through the breach, and discharges downstream.

In a hydrograph-type simulation the reservoir impounded by the dam is not modeled and the user provides the breach hydrograph directly. As described in detail below, the flow discharges downstream from the user-defined breach hydrograph [9]-[10]. We used a reservoir-type simulation for the static and seismic failure modes and a hydrograph-type simulation for the hydrologic failure mode. The DSS-WISE™ program does not allow water surface elevations to be higher than the top of dam elevation. Since many of the SDFs often result in water surface elevations that exceed top of dam elevation (i.e., overtopping), a hydrograph-type simulation provides an appropriate alternative method for modeling SDFs for the hydrologic scenario.

In addition to the type of breach simulation (i.e., reservoir- and hydrograph-type) DSS-WISE™ further refines breach input by the type of dam. This approach is consistent with guidance from Federal Energy Regulatory Commission (FERC) and FEMA, which describes factors for consideration in selecting appropriate breach

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parameters that include dam type, dam dimensions, and dam materials of construction [4]. Time to failure is also selected based on dam type.

Typically, dam breach geometry is based on dam type with two primary types of construction: embankment and concrete. Earthen embankments represented 80%, and concrete dams represented 20% of the dams in this study. Earthen embankment dams can fail by internal piping (i.e., static and seismic scenarios) or overtopping (i.e., hydrologic scenario) with breach progression resulting from erosion of the embankment material. Concrete dam failures are typically modeled as rectangular breach geometry (or if the channel is constrained, the slope of the valley walls), with breach width based on the concrete dam type (i.e., gravity, arch, buttress). Breach progression at concrete dams typically occurs as a structural failure of an entire dam section. Information on breach geometry and time to failure of earthen embankment and concrete dams for static, seismic, and hydrologic scenarios is provided below.

C. Static Failure Mode

The term “static” in this study describes normal loading conditions at a dam as represented by water levels at the normal pool and absent seismic activity. We set the failure pool to the normal pool elevation and normal pool storage. For earthen embankment dams, we used a partial breach failure type function in DSS-WISE™ with a reservoir-type breach. Partial breach failure types in DSS-WISE™ allow the user to select from seven embankment dam breach equation methods to calculate the average width (ft) and formation time (hr) of the breach. For earthen embankment dams, we selected the Von Thun & Gillette [11] breach parameter routine to estimate the breach geometry for each earthen embankment dam. When selecting the Von Thun & Gillette breach equation, DSS-WISE™ presents two formation time options: “erosion resistant” and “easily erodible.” We used the average of these two formation times as the breach formation time for the static failure mode of embankment dams. For concrete gravity dams, we followed the FERC Chapter II Engineering Guidelines [6], which suggests an average breach width equal to or less than 0.5 times the crest length. Due to the reported “Poor” or “Unsafe” condition of the dams in the portfolio, the absence of detailed drawing information about dam construction, and that we did not visit the dams in person, we conservatively failed the entire length of the spillway/dam crest at concrete gravity structures. It is possible that additional information about the concrete gravity dams could justify smaller breach geometry thereby reducing the inundation boundary. Breach formation time (i.e., time to failure) for concrete dams typically ranges from 0.1 to 0.3 hours [6]. For this project, we assumed a formation time of 0.2 hours for concrete dams.

D. Seismic Failure Mode

The term “seismic” in this study indicated that a seismic event would be occurring at the dam at the time of failure. While a seismic event would potentially influence a consequence analysis, we also elected to have a seismic event influence the dam breach analysis by decreasing the formation time compared to the static failure mode. For the seismic failure mode, we assumed a “nearly instantaneous” breach formation of 0.1 hours due to the unexpected nature of seismic events. For the seismic failure mode, the initial water surface was set at the normal pool (same as static) and the breach geometry parameters were assumed to be the same as the static failure breach parameters.

E. Hydrologic Failure Mode

The hydrologic failure mode involved a combination of high downstream flows resulting from a significant rainfall event. In Massachusetts, all existing high hazard dams have a SDF that is either the 500-year annual recurrence interval storm event or the ½ probable maximum flood (PMF) depending on the dam size of small, intermediate, or large [7]. Since high flows from the rainfall event would flood downstream areas in the absence of dam failure, we needed to identify the incremental impacts from the dam breach (i.e., failure minus non-failure). Therefore, we ran two hydrologic models for each dam: failure and non-failure to derive the incremental impacts. Delineating the two inundation boundaries is important for consequence analysis where the population at risk is only computed for the incremental impacts (i.e., incremental rise equals failure water surface elevation minus non-failure water surface elevation). We used a partial breach failure type with a hydrograph-type breach for the hydrologic models. The hydrologic failure breach width and formation time were the same as the static failure breach width and formation time.

Due to limitations with the DSS-WISE™ software in modeling the hydrologic failure mode, we developed a framework for flood flow inputs. For the hydrologic hydrograph templates, we used the previously estimated spillway design flood flows and peak breach flows from the Phase I and Phase II Inspection reports and EAPs, as

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available. We developed outflow hydrograph templates, Tables 4 and 5 below, for the hydrologic failure and non-failure models.

Table 4. Hydrologic Failure Hydrograph Template

Elapsed Time (hours)	Flow (cfs)
0	0
11	0
12	Spillway Design Flood
12 + Static Formation Time	Peak Failure Flow
13 + Static Formation Time	Spillway Design Flood
15 + Static Formation Time	0

Table 5. Hydrologic Non-Failure Hydrograph Template

Elapsed Time (hours)	Flow (cfs)
0	0
11	0
12	Spillway Design Flood
15	0

F. Simulation Parameters

DSS-WISE™ has several simulation parameters like the domain buffer distance (i.e., model extent), cell size for computation (larger cells produces faster computation times but results are coarser), maximum simulation duration, and editable Manning's n-values.

We started with a domain distance (i.e., model extent) of 5 miles and a simulation duration of 3 days and increased or decreased the domain distance and duration until an incremental rise (i.e., failure water surface elevation minus non-failure) of 2 ft or less was achieved at the downstream end of the model for the three flow scenarios (i.e., static, seismic, and hydrologic). According to Chapter II of the FERC Engineering Guidelines [6], dam failure analysis should be routed downstream to a point where the incremental effect of failure no longer constitutes a threat to life or property. The FERC guidelines indicate that, while the 2-ft rise guideline is not an absolute decision-making point, the consequences of failure are not expected to cause a probable loss of life when the incremental rise on downstream structures is approximately 2 ft or less [6]. We also considered the following parameters in model development and in evaluating the downstream end of the model:

- Depth*Velocity.
- Hydrograph peak achieved.
- Dam failure flows (i.e., inundation boundary) were within the natural channel.
- Dam failure flows (i.e., inundation boundary) were contained within a large downstream reservoir.
- Dam failure flows (i.e., inundation boundary) entered a tidal bay or the ocean.

To obtain the most detailed results available from DSS-WISE™, we used the smallest cell size of 15 ft by 15 ft for computation. We used the DSS-WISE™ default Manning's n-values.

IV. DAM FAILURE CONSEQUENCE ANALYSIS

A. Population at Risk

Population at Risk (PAR) is a term used by FEMA and other dam safety organizations to quantify the downstream population at risk from dam failure. The PAR is typically documented in numbers of persons at risk [3]. We used the DSS-WISE™ Human Consequence Module (HCOM) software platform [1] and the LandScan Daytime and Nighttime data to develop the PAR estimates.

The DSS-WISE™ software platform provides a method to evaluate human consequences for dam failure scenarios using DSS-WISE™ HCOM: Assessment of Human Consequences of Dam-Break Floods. The DSS-WISE™ HCOM software is recommended by FEMA for automated assessment of the consequences of dam

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failure. We also found that the software provided a consistent and systematic approach to apply to the 35 dams in the portfolio.

The LandScan population density grids were an important data source for estimating PAR given that people are transient and may be in different places during the day (e.g., work, school, etc.) and night (e.g., home). Two LandScan population density grids were provided within the DSS-WISE™ HCOM software to represent daytime and nighttime populations.

We ran the DSS-WISE™ HCOM assessment with the static, seismic, hydrologic failure, and hydrologic non-failure model scenarios for each dam. To estimate PAR for the static and seismic failure modes, we used DSS-WISE™ HCOM LandScan estimates for daytime and nighttime PAR. To estimate PAR for the hydrologic failure mode, we subtracted the hydrologic non-failure LandScan PAR estimate from the hydrologic failure LandScan PAR estimate to derive the incremental daytime and nighttime PAR associated with the dam failure. For each dam, the failure mode (i.e., static, seismic, hydrologic) with the largest PAR was identified and subsequently used in the economic and environmental consequence assessments.

In addition to PAR, we also reviewed flood wave arrival time and critical infrastructure as additional criteria for evaluating risk associated with the dam failure. We used the flood wave arrival time at the first structure downstream as the estimated warning time for the nearest PAR. We also evaluated the presence of critical infrastructure that may be impacted by dam failure along the reach downstream of the dams. We used ArcGIS to identify critical infrastructure situated within the inundation boundary. Critical infrastructure included mapped layers available from the state-based geospatial data portals such as emergency management-related services (e.g., police, fire, etc.), hospitals, schools, and other infrastructure with potential for high density of people, sensitive populations, and/or representative of a critical facility on which the community depends.

B. Economic Losses

In addition to PAR, economic losses are identified as additional criteria to use in performing risk analysis. We used the FEMA HAZUS software [2] to develop estimates of economic loss from dam failure. The FEMA HAZUS program is a GIS-based desktop application that provides decision support software for estimating potential losses from flood events. For each dam, we input to the FEMA HAZUS program the maximum depth raster from the DSS-WISE™ model failure mode (i.e., static, seismic, hydrologic) that resulted in the largest PAR.

The FEMA HAZUS program results provided an estimated monetary value of building loss (building, content, inventory) and business interruption (income, relocation, rental income, wage). For dams where the hydrologic failure produced the largest PAR, we subtracted the hydrologic non-failure economic loss estimate from the hydrologic failure economic loss estimate to derive the incremental economic loss associated with the dam failure.

The FEMA HAZUS program focuses on buildings and does not include other infrastructure (i.e., roads, bridges, utilities, etc.) in the economic loss estimate. Despite this potential limitation, using the FEMA HAZUS economic loss estimate provided us with a consistent approach to determine a relative economic loss value across the 35 dams in the portfolio.

C. Environmental Losses

Similar to economic losses, environmental losses provided additional criteria to use in performing risk analysis. FEMA recommends categorizing environmental losses from a dam failure by the severity of the impact and the amount of time required for remediation. We used these guidelines [3] to develop a systematic approach to evaluate environmental losses from dam breaches. For the evaluation, we took a two-step approach. We used FEMA HAZUS to estimate the debris tonnage resulting from the dam failure and used readily available on-line sources to identify listed land use activity indicative of the presence of potential contamination sources. We considered these two areas for their influence in causing potential severe and long-term consequences, and for the ability to perform the analysis in a reproducible, systematic, and consistent manner for the portfolio of 35 dams.

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The tons of debris generated by a dam failure was estimated for each dam using the FEMA HAZUS program. In the FEMA HAZUS program, we input the maximum depth raster from the DSS-WISE™ model for the failure mode (i.e., static, seismic, hydrologic) with the largest PAR to estimate the tonnage of debris generated from building structures. HAZUS was limited to estimating debris from building structures and did not consider other debris sources such as sediment or vegetation. We used this estimate as a proxy for the total amount of debris generated from a dam failure. For dams where the hydrologic failure resulted in the largest PAR, we subtracted the hydrologic non-failure debris estimate from the hydrologic failure debris estimate to derive the incremental amount of debris associated with the dam failure.

We also considered potential contamination sources in the environmental loss evaluation. We downloaded readily available point shapefiles for contamination sources such as waste disposal sites, wastewater treatment plants, and facilities/businesses recognized by the states as using or storing regulated/hazardous waste from the Massachusetts, Rhode Island, New York, and Vermont state geodata portals. Since the potential contamination data were provided as points and some of the parcels were large, we created a 250-foot polygon around each point feature to ensure consideration relative to the inundation boundaries. In our analysis of environmental losses, we used aerial imagery to consider the presence of materials (i.e., stockpiles, drums, dumpsters, junk cars, etc.) stored outside of buildings and exposed to the elements, factors that would increase the potential for environmental risk. We also made observations on condition of the surrounding area, recent site redevelopment, and other details that might increase or reduce the potential for contamination in the event of a flood.

D. Development of Consequences Evaluation Criteria

In general accordance with FEMA guidelines [3], we used the qualitative terms of Low, Moderate, High, and Very High to quantify consequences. We estimated the relative consequences of failure of each dam under the following three categories:

- PAR
- Economic Losses
- Environmental Losses

To assess the relative consequences of failure across the portfolio in a consistent manner, we developed an approach that included a list of evaluation criteria for each consequence category and each dam.

Our approach recognized that we would be assigning one consequence of failure category (Low, Moderate, High, Very High) for each of the three categories (PAR, economic, and environmental). Evaluation criteria for the relative consequence of failure for PAR, economic losses, and environmental losses are provided in Table 6.

Table 6. Relative Consequence of Failure Criteria

Relative Consequence of Failure	Population at Risk (PAR)	Economic Losses	Environmental Losses
Very High	PAR $\geq$ 1,000 OR 100 $\leq$ PAR < 1,000 and critical facilities	$\geq$ $1 Billion	Very high-risk potential contamination source OR $\geq$ 10,000 tons of debris
High	100 $\leq$ PAR < 1,000 OR 10 $\leq$ PAR < 100 and critical facilities	$100 Million $\leq$ Losses < $1 Billion	High-risk potential contamination source OR 1,000 $\leq$ tons of debris < 10,000
Moderate	10 $\leq$ PAR < 100 OR 1 $\leq$ PAR < 10 and critical facilities	$10 Million $\leq$ Losses < $100 Million	Moderate-risk potential contamination source OR 10 $\leq$ tons of debris < 1,000

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Relative Consequence of Failure	Population at Risk (PAR)	Economic Losses	Environmental Losses
Low	1 ≤ PAR < 10 without critical facilities	< $10 Million	No known contamination sources OR < 10 tons of debris

E. Summary of Results

Table 7 is the Summary Dam Consequences Table with the estimated relative consequences of PAR, economic losses, and environmental losses for the 35 dams included in the evaluation. Table 8 provides a summary of the number of dams assigned to each relative consequence of failure for each consequence category.

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Table 7. Consequences Results Summary Table

Dam No.	Consequences		
	Population at Risk (PAR)	Economic Losses	Environmental Losses
1	High	Moderate	Moderate
2	Moderate	Low	Moderate
3	High	Low	Moderate
4	High	Moderate	High
5	Very High	Very High	Very High
6	Moderate	Low	Moderate
7	Moderate	Low	Low
8	Very High	High	Very High
9	Very High	High	Very High
10	Very High	High	High
11	High	Moderate	High
12	Moderate	Low	Moderate
13	High	Low	High
14	Moderate	Low	Low
15	Very High	Very High	Very High
16	Very High	High	High
17	High	High	High
18	Moderate	Low	Moderate
19	Moderate	Low	Low
20	High	Low	Low
21	Very High	High	High
22	Very High	High	Very High
23	Very High	High	High
24	Moderate	Low	Low
25	Moderate	Low	Low
26	High	Low	Very High
27	Very High	Moderate	Moderate
28	High	Moderate	Moderate
29	Moderate	Low	Low
30	Moderate	Low	Low
31	High	Moderate	Moderate
32	Low	Low	Low
33	Moderate	Moderate	Moderate
34	High	Moderate	Moderate
35	High	Low	Moderate

Table 8. Consequences Results Summary Table

Relative Consequence of Failure	Consequences		
	PAR	Economic	Environmental
Very High	10	2	6
High	12	8	8
Moderate	12	8	12
Low	1	17	9

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V. RISK MATRIX AND RANKINGS

A. Risk Matrix

To understand and visualize the risks associated with each dam, we plotted each dam on the FEMA risk matrix [3]. Fig. 1 shows the relationship between the PAR (X-axis) and the combined likelihood of failure (Y-axis) for each dam. We selected the greatest PAR value across the three failure modes (static, seismic, and hydrologic) for each dam, and plotted them on a logarithmic scale. To develop the combined likelihood of failure value we assigned a numerical value to the static, seismic, and hydrologic likelihood of failure values and added the static, seismic, and hydrologic values together for each dam. The numerical values were "Low = 1," "Moderate = 2," "High = 3," or "Very High = 4". The combined likelihood of failure (LOF) where based on LOF < 3 is "Low," 3 ≤ LOF< 6 is "Moderate," 6 ≤ LOF< 9 is "High," and LOF ≥ 9 is "Very High."

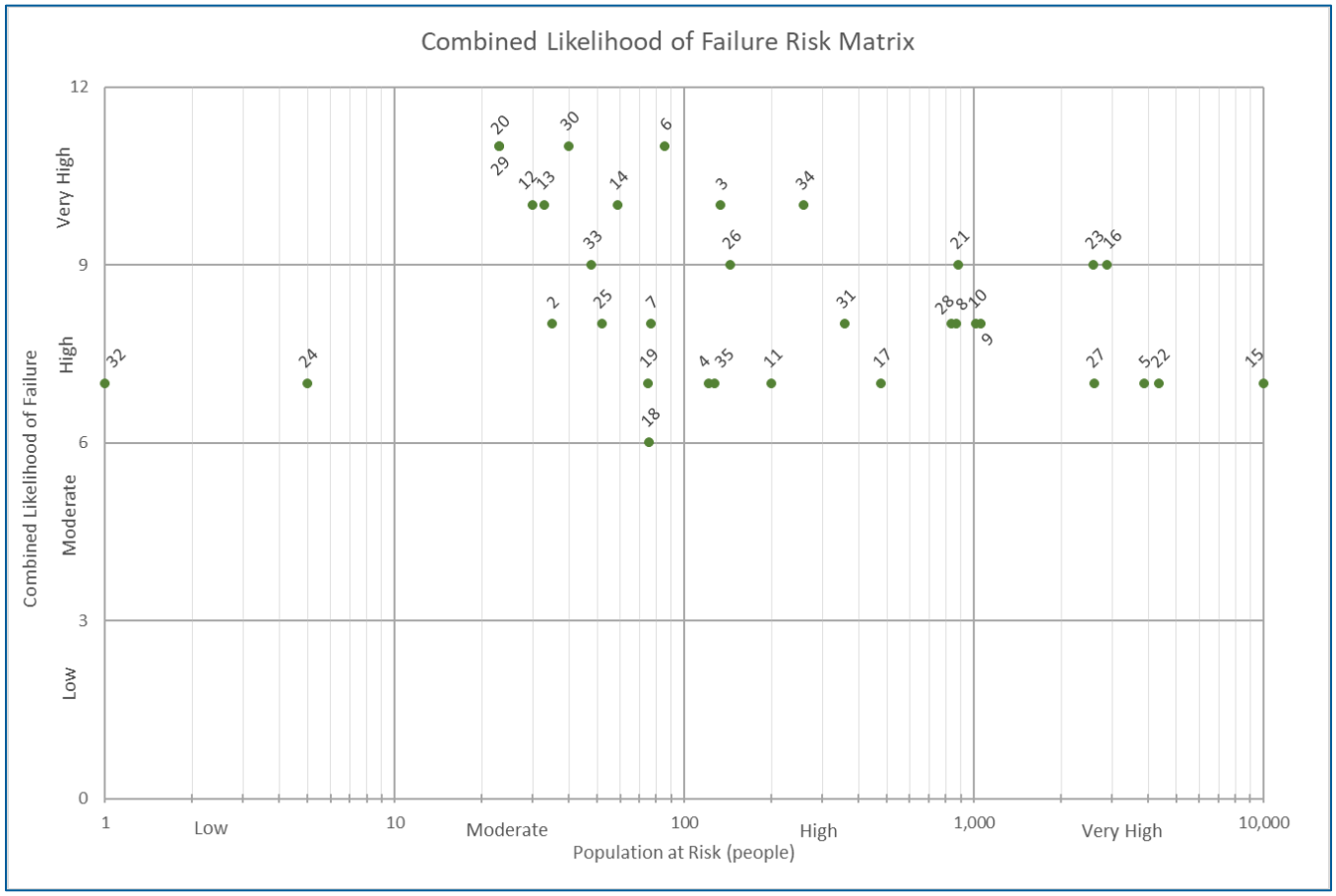


Figure 1. Combined likelihood of failure risk matrix.

B. Development of Ranking Based on Risk

The FERC Chapter 18 Engineering Guidelines dated December 2021 [5] state that "there are three primary elements when considering dam safety risk:

1. Frequency/Probability of loading
2. Likelihood of dam failure (response) given the loading
3. Adverse consequences"

When evaluating the risk of the 35 dams in the portfolio, we calculated the relative risk by multiplying the likelihood of failure for each failure mode (i.e., static, seismic, and hydrologic) by the population at risk. The likelihood of failure is an estimate of the annual exceedance probability (AEP). Table 9 shows the approximate AEP value used in the risk equation based on the qualitative terms. We used the qualitative likelihood terms of "Low, Moderate, High, and Very High" assigning one likelihood of failure for each of the three modes. We added the static risk, seismic risk, and hydrologic risk to determine the numeric relative risk of each dam. We then

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ranked the dams based on their relative risk. Figure 2 shows a portion of the spreadsheet used to identify the top ten ranking dams in the HHPD portfolio based on risk.

Table 9: Approximate AEP Values for the Qualitative Likelihood of Failure Terms

Qualitative Term	Description of Annual probability of Failure	~AEP used in the Risk Calculations
Very High	Failure is likely to occur in less than 10 years.	0.2
High	Failure is likely to occur in 10 to 100 years.	0.02
Moderate	Failure is likely to occur in 100 to 1,000 years.	0.002
Low	Failure is likely to occur in > 1,000 years.	0.0003

Ranking	Dam No.	Static Risk Static LOF * Static PAR	Seismic Risk Seismic LOF * Seismic PAR	Hydrologic Risk Hydrologic LOF * Hydrologic PAR	Combined Risk (addition)
1	23	309.00	31.32	5.19	345.51
2	10	1.69	169.00	2.04	172.73
3	16	43.56	43.56	57.82	144.94
4	15	6.46	64.92	20.16	91.53
5	5	4.45	4.45	77.46	86.36
6	22	57.78	5.78	8.74	72.30
7	21	56.20	5.60	1.78	63.58
8	28	16.78	16.78	1.65	35.21
9	1	24.40	2.44	3.78	30.62
10	3	26.60	2.68	0.70	29.98

Figure 2. Top ten ranking dams in the HHPD portfolio based on risk.

VI. CONCLUSION

The portfolio analysis, which involves ranking each dam according to likelihood of failure and consequences, will serve as a foundation for future FEMA HHPD grant applications for repair or removal of dams. Having a risk prioritization of a portfolio of dams can help the state of Massachusetts make funding decision in the future based on risk. Whether the funding comes from future HHPD grant awards or other funding sources.

The next steps of our work will include working with Massachusetts to apply for HHPD funds this year, HHPD dam stakeholders reach out and education about the HHPD grant program and performing studies to fill in data gaps for MA ODS. The HHPD grant funds will be a new source of funding for dam owners in Massachusetts, but these funds are crucial in keeping the dams and people downstream of dams in Massachusetts safe.

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DAM SAFETY PALM SPRINGS
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Oregon's Semi-Quantitative Risk Assessments for Dams with Limited Design and Analysis Information

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ABSTRACT

With risk assessment now an important dam safety function, Oregon dam safety staff have developed an assessment protocol for use on State regulated dams to support FEMA grant processes. Fifteen high hazard dams with known or suspected major safety deficiencies have been assessed, including quantitative evaluation of events associated with failures, loadings in those events, and loss of life from catastrophic failure. Events assessed in this project include flood, high water earthquake, and landslides above the reservoir or dam. The project evaluated seven general failure modes based on statistical frequency, including modes that have caused most historical U.S. incidents plus the seismic and landslide risks. Dam specific failure modes are also considered. The methods were influenced by a similar risk evaluation project for rapidly moving landslides and loss of life in Oregon. The use of historical failure frequencies is a more established means of determining risks.

Existing file information was comprehensively reviewed with inspection elements based in information and gaps in the dam safety program files for those dams. The product is a consistent procedure and includes a rating of confidence in the information. Results are calculated in terms of expected loss of life on an annualized basis and shown in this paper on a figure developed following the federal guidelines for dam safety template. Parts of the risk assessment process are applicable to all dam safety regulators. This process would benefit from more detailed failure investigations added to dam failure statistics, and from additional input from experts.

I. INTRODUCTION

The FEMA High Hazard Potential Dam (HHPD) grant program and the new Model State Dam Safety Program both recognize the need for risk assessments, especially for deficient high hazard potential dams. The Oregon Dam Safety Program used most of its fiscal year (FY) 2019 HHPD grant to develop a comparative risk assessment process for consistent evaluation of the risks posed by eligible State regulated dams. The 2015 FEMA publication *Federal Guidelines for Dam Safety Risk Management* [1] provides definitions and conceptual guidance for risk analysis by dam safety programs and was used to develop methods. **"Risk"** in the simplest terms for dam safety applications, and for this work, is the product of the likelihood (probability) of a sudden dam failure and the consequences of that dam failure or uncontrolled release of water [2].

The goal of this project was to develop a simple and repeatable semi-quantitative risk analysis (SQRA) that is a defensible means to prioritize dams for additional actions, and to apply this to Oregon high

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hazard dams in Poor or Unsatisfactory Condition as defined in the National Inventory of Dams[1]. This assessment process was developed for 15 dams that meet the FEMA eligibility criteria for FY 19. Work on this project began in 2020 just as the pandemic hit which complicated the comprehensive inspection portions of this project. The team working on this project were the four engineers that staff the Oregon dam safety program and the authors of a full report to FEMA and this summary paper. We fit this work between our normal dam safety work, and this project allowed an increase in staffing that later was supported through the state budget process. Development of methods included review of draft methods by two experienced engineering firms, one hydrologic and hydraulic, the other geotechnical. Changes were made to the proposed methods after this review. A full report was submitted to FEMA on March 30, 2023 [3]. The full report includes much more detail, with assumptions and all data tables that are too detailed for this summary paper.

The Oregon dam safety program and its files date back to 1929, so dam design and/or inspection information is available. Thirteen out of the 15 dams are embankments. Most were designed as homogenous, with a single conduit, without spillway gates, and with few other appurtenant structures. Comprehensive file review and inspections were performed for each dam, with consistent data tables completed. Methods are probabilistic and comparative, not deterministic. Dam failure is analyzed as event driven, to determine likely loading and especially reservoir levels of concern. Understanding of extreme events, and recurrence of these events were the key element of this project.

II. EXTREME EVENTS AND FAILURE STATISTICS

Most dam failures occur in extreme events so most of the risk of dam failure is event driven. The extreme events that are most likely to cause dam failure in Oregon are flood events, earthquakes. The potential dam failure triggering events in Oregon and the Pacific Northwest have significant differences from much of the rest of the country. Oregon event and vulnerabilities are summarized as follows.

Floods: Oregon's two largest historical floods occurred in 1861 and 1903, not during the life of the 15 dams in this project. There is very little data on the precipitation leading up to these flood events, or on specific peak flood flows. The 1861 storm occurred up and down the west coast. This storm was a long duration atmospheric river event (very different mechanism and moisture source from the eastern two-thirds of the country). The 1903 event was an extreme intermountain arid region thunderstorm. Given the short period of record and the lack of extreme floods in Oregon there is low confidence in more rare extreme floods. It is unknown how Hydrometeorological Report No. 57 (HMR 57 for the Pacific Northwest) represents the meteorological physics of atmospheric rivers, or how it can be used to determine flood recurrence, so an alternate approach is needed for this analysis.

While internal erosion observations and incidents can occur at typical reservoir levels, most internal erosion failures occur at the highest reservoir level experienced by that dam [*5*]. These will be referred to as non-overtopping floods. Based on stream gage data and other information, it is also unlikely any of the dams in the project have experienced greater than a 0.2 percent annual exceedance probability (AEP) (500-year) flood. Therefore, it is reasonable to use a reservoir level calculated at the 0.2 percent AEP inflow to make a consistent comparison of risk for these dams as described later in this report.

Earthquakes: The western part of the state is subject to a very large Cascadia Subduction Zone earthquake, the last one occurring in 1700 (determined by Tsunami records from Japan and buried forests in NW estuaries). There has been no very large earthquake in Oregon since that time (two earthquakes near magnitude 6 occurred in 1993). Most Oregon dams were designed before there was any awareness of the occurrences of subduction zone earthquakes in the Pacific Northwest. Phase I

[1] POOR A dam safety deficiency is recognized for loading conditions that may realistically occur. Remedial action is necessary. Uncertainties exist as to critical analysis parameters and further investigations and studies are necessary. UNSATISFACTORY A dam safety deficiency is recognized that requires immediate or emergency remedial action for problem resolution.

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Inspection Reports were completed by the U.S. Army Corps of Engineers and the Water Resources Department from 1971 to 1982 and used a ground acceleration of 0.05 standard gravitational acceleration (g) for analysis of these dams. The 2500-year return earthquake accelerations are now 0.6 to 1 g at coastal Oregon dams. The Cascadia earthquake has about a14-20 percent occurrence probability every 50 years (0.3 percent annual occurrence) with eastern Oregon faults believed to have been last active in the past up to 16,000 years ago.

Landslides: Landslides occur in both floods and landslides. In western Oregon, landslides occur at great density in high precipitation events [4] and will likely occur during large earthquakes. Oregon's geologic units closer to the coast tend to have weaker rocks and very steep slopes, further increasing seismic and landslide risk. More so than during precipitation, the landslides that occur in earthquakes, can be very large rockslides and debris avalanches capable of causing large waves in reservoirs.

Statistics of Historical Failure and Incidents: Worldwide statistics of Historical Failure and Incidents are reported in [*5*], with additional information from Oregon dam failure records and from Oregon's risk analysis of rapidly moving landslides (*5*). Figure 1 below provides a summary of historical failure statistics up to 1986. Most failures of embankment dams have been attributed to overtopping (48%) and internal erosion (46.5%). The historical failure rates for overtopping and internal erosion are similar and much higher than failure rates for embankment slope movement or seismic deformation.

376 Geotechnical engineering of dams, 2nd edition

Table 8.1 Overall statistics of embankment dam failures, large embankment dams, up to 1986 and excluding dams constructed in Japan pre 1930 and in China (adapted from Foster *et al.*, 1998, 2000a).

	Number of cases		*% failures (where known)*		*Average probability of failure (10^{-3})*	
Mode of failure	*All failures*	*Failures in operation*	*All failures*	*Failures in operation*	*All failures*	*Failures in operation*
Inadequate spillway capacity	46	40	36	34	4	3.5
Malfunction of gate	16	15	12	13	1.5	1.5
Subtotal overtopping & appurtenant failures	62	55	48	47	5.5	5
Internal erosion through embankment	39	38	30	33	3.5	3.5
Internal erosion through foundation	19	18	15	15	1.5	1.5
Internal erosion from embankment into foundation	2	2	1.5	1.5	0.2	0.2
Subtotal internal erosion[(1)]	59	57	46.5	48.5	5	5
Downstream slides	6	4	5	3.5	0.5	0.4
Upstream slides	1	1	1	1	0.1	0.1
Subtotal slides	7	5	6	4.5	0.6	0.5
Earthquake/liquefaction	2	2	1.5	1.5	0.2	0.2
Unknown mode	8	7				
Total no. of failures[(1)]	136	124			12	11
Total no. of failures (where mode of failure known)	128	117				
No. of embankment dams	11192	11192				

Notes: [(1)]Subtotals and totals do not necessarily sum to 100% as some failures were classified as multiple modes of failure.

Table 1: Historical failures from Geotechnical Engineering of Dams [5]

Safety Issues observed on Oregon's State Regulated Dams: Most of the dams in the project were designed as homogenous embankments without internal filters or filtered drain exits. Only one of the HHPD project dams was built after 1975. Based on investigations on other dams in Oregon's regulatory portfolio, these dams likely have the following characteristics:

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- Under compacted with material generally placed dry of optimum moisture
- Single zone design (Homogenous), with irregular material placement
- Historical lack of plant and animal control or conduit and valve maintenance
- No inflow design flood calculations or design records for spillway stability
- No as built drawings

III. POTENTIAL FAILURE MODES FOR COMPARATIVE ANALYSIS

Flood overtopping, spillway failure, and internal erosion comprise 94 percent of historical failures and therefore make up the majority of the modes of failure for this project. Seismic vulnerabilities are of special concern in Oregon because of the Cascadia Subduction Zone and recurrence of extreme events that was not understood when the most dams were designed. And Oregon has experienced thousands of landslides during events as frequent as a 2 percent AEP flood (*5*). The same PFMs developed for all of the dams in this project and then each dam was individually evaluated with the same consistent methods.

Historical information on occurrence of dam failure is essential for a comparative risk analysis. A comparative analysis refers to the comparison of two or more processes, documents, data sets or other objects. This risk assessment uses a comparative analysis using known risk factors to compare safety elements at each dam to each other, and to an “average” or standard Oregon embankment dam. For each event type, the available data is summarized as it relates to risk of dam failure. Much more detail including assumptions are found in the full report.

Extreme flood related modes: Forty eight percent of historical large dam failures were due to overtopping or a gate failure [5]. This is the most common general failure mode for dams. Thirteen of the 15 dams are embankments with no overtopping protection. Only one of the dams has a spillway gate. Failure can also occur in an extreme flood event without overtopping by erosion of the spillway head cutting to the dam or the reservoir. Analysis of extreme flooding is the most important element of risk analysis of dams.

There are multiple ways spillway capacity can be reduced so that a lesser flow will cause the dam to overtop. These include but are not limited to trees and debris blocking the spillway, ice blocking the spillway or making gates, or intentional actions. Erosion of the spillway that head-cuts to the reservoir or undercuts the dam can also cause failures and near failures (The Oroville Dam spillway incident is an example).

PFM 1. Overtopping from flow only

Breach mechanism: Usually erosion of embankment increasing as overtopping increases, with a head cut developing and moving toward the reservoir. Final failure by sliding or collapse.

PFM 2. Overtopping from spillway blockage

Breach mechanism: Usually erosion of embankment increasing as overtopping increases, with a head cut developing and moving toward the reservoir. Final failure by sliding or collapse.

PFM 3. Spillway failure eroding to reservoir

Breach mechanism: There are many erosion processes that can begin with, direct erosion, slab jacking, or rock block lifting. For failure, this process must head cut back to the dam or reservoir.

Internal Erosion Most non-overtopping dam failures occur during or close to the period of highest reservoir level experienced by that dam [5]. These non-overtopping related dam failures are usually caused by internal erosion. Internal erosion is the cause of almost half (46.5 percent) of historical dam failures. Based on information in [5] the annual frequency of internal erosion incidents for large dams has been about 2×10^{-4} each, for within the embankment, and a near equal risk for internal erosion due next to conduits or walls. The frequency above includes accidents. For this report we are only looking at failures after the first fill, so for each of the two locations in an embankment, this annual frequency is 4×10^{-5}, a frequency that appears consistent with historical Oregon internal erosion failure occurrence.

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The Oregon risk analysis considered two general internal erosion locations.
1) The upper portion of the dam because of animal, vegetation, and desiccation impacts.
2) Around the conduit or at the base of the dam. Defects in compaction are common around the conduit and seepage are frequently identified during routine inspections. This location is critical since internal erosion at the conduit/foundation makes up about half of internal erosion failures.

PFM 4. Internal erosion concentrated leak from damage in the upper 1/3 of the dam

Breach mechanism: Piping through the dam by enlargement of a defect in embankment materials that had been unsaturated and form a roof, followed by collapse of the crest above the pipe and reservoir flow through the area where the crest collapsed. Failure may also be by landslide by leakage and increase in pore pressure due to burrow, decomposed root related void, or defect or climate related crack or defect.

PFM 5. Internal erosion at the conduit/foundation

Breach mechanism: Any of the four potential breach mechanisms with the conduit or a difference in material properties resulting in a roof. This will be followed by crest collapse into the void and overtopping of the reservoir, then failure by flow erosion or sliding failure.

Damage from Earthquake: USGS research in the late 1980's found compelling evidence of great subduction zone earthquakes in western Oregon. There was no recognition of significant earthquake hazard when most Oregon dams were constructed. As a result, most were designed for very small seismic loading and most state regulated dams have not had an analysis for seismic these vulnerabilities.

PFM 6. Seismic Deformation

Breach mechanism. Earthquake shaking causes the dam to deform, lowering elevations along the crest and causing cracking though the dam. The actual breach may be from overtopping and rapid erosion of a weakened structure, or from concentrated leak erosion through cracks.

Landslides on the west coast: The Pacific Coast states have mountainous terrain that in some locations are prone to very large rapidly moving landslides. The most catastrophic dam incident in the modern western world was the 1963 Viaont Dam overtopping flood in northern Italy (another mountainous region with weak rock). That dam overtopped without dam failure and killed about 2,500 people. LiDAR technology proves to be very effective at identification of landslides features including scars, scarps, and deposit. LiDAR can also be used to gather other watershed level information, especially on tree cover and slope steepness. This is illustrated in Figure 1, which is a LiDAR image from Skookum dam and reservoir in the Oregon coastal mountains. For scale, the dam is 700 feet long. The landslide area, because it was covered by dense forest, was not recognized until 2014, with the dam removed in 2015.

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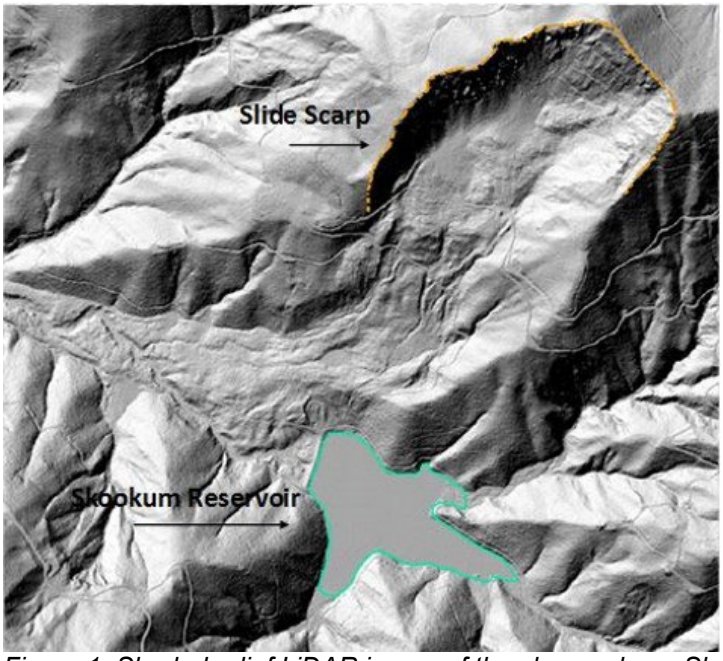


Figure 1: Shaded relief LiDAR image of the slopes above Skookum dam. A large landslide scar is apparent above the dam with debris avalanche deposits below the dam.

PFM 7. Landslide into dam or embankment
Breach Mechanism: Very large landslide occurs on the slopes directly above the reservoir or dam. Dam may fail by landslide damage, or dam may not fail landslide will displace reservoir water and result in extreme flooding downstream

PFM 8. Dam Specific (Any significant risk potential not already addressed and identified during inspections, comprehensive review of files or other tasks associated with the project)
Breach Mechanism: Dam specific. This PFM estimates the probability of failure due to any other dam specific issue that could cause dam failure or unsafe release of water. A probability was assigned based on each specific case.

IV. SAFETY ELEMENTS AND RISK ASSESSMENT

This assessment considers the event causing extreme dam loading, an estimate of the probability of failure in that event based on failure statistics and presence or lack of safety elements, population in vulnerable areas, the persons likely remaining in highest risk locations after a breach. All 8 PFMs were evaluated for every embankment dam. A requirement for this analysis was to only use data that was available in the dam safety files or that could be readily obtained through external inspection and measurements on the dam. The general steps used for this assessment are as follows:

1. Determine magnitude and recurrence of extreme events in Oregon:
    a. Floods
    b. Earthquakes
    c. High water level with little freeboard
2. Determine “standard” dam characteristics and departure for these characteristics, especially for internal erosion and spillway stability issues.
3. Dam safety file review of inspections for safety related issues
4. Detailed field inspections, including embankment material and spillway evaluation, and differences from the approved design
5. Determine inflow floods and normal operating condition freeboards at 0.2 percent AEP
6. Complete risk assessments with consistent ratings and spreadsheet analyses approach.
7. Determine the annualized risk of failure for the most typical modes of failure in Oregon, and additional modes of failure because of a dam specific vulnerability.

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8. Are risks consistent with the statistics of historic failures and incidents?
9. Estimate the warning time based on monitoring, observation frequency, and expected time to failure
10. Estimate annualized loss of life based on risk of failure and population at risk (PAR)
11. Determine highest priority dams for safety actions (analysis, rehabilitation, or removal)

Evaluating Safety Elements: We define safety element as any design, construction, maintenance, operation or emergency response element that reduces risk of dam failure or dangerous reservoir releases**.** Many safety elements are details and not readily analyzed by deterministic methods. For this project we consider a safety element to be any known or likely design, construction, maintenance, operational, monitoring or emergency preparation action that is likely to have an effect on a mode of failure or early identification and communication of likely failure or other flooding related to the dam. The comprehensive file review compiled Excel spreadsheets on all these elements for every dam, especially those that could be easily measured. A summary of these elements follows:

Design and construction administration: File review evaluated reports, if any, that document geologic and geotechnical investigation and interpretation; design review and approval; on site engineers construction administration; project completion reports and if accurate as built drawings were submitted. Any design modifications to the dam were review for these same elements.

Embankment Material: The review evaluated specifications in the contract documents and then in the project completion report, including documentation of compaction and moisture content and strength testing. The review also evaluated the potential for liquefiable or internal erosion prone materials, and placement and compaction practices.

Embankment Dimensions: Basic parameters such as height, width, crest width, storage, design freeboard were recorded.

Internal Drainage and Filters: Is the dam homogenous or zoned, and are there filters, filter diaphragms, toe drains and filtered exits? There were few, if any, of these safety features on the 13 embankment dams evaluated in this project.

Spillway Sufficiency: Dimensions of control section and spillway gradient were measured in the field inspection portion of this project and compared to the design drawings. If there were hydrologic and hydraulic calculations, they were reviewed. Spillway channel material was evaluated during the field inspections, as was the presence and condition of a debris boom or other means to pass or control floating material.

Conduit Material and Durability: The size and material of the conduit specified in design was compared to the conduit that exists in the dam now. Field inspections evaluated the material and condition of the conduit as compared to the original design, with special attention to corrugated metal pipe conduits because of their susceptibility to corrosion and very short safe life spans.

General Maintenance and Control of Flora and Fauna: Past dam safety program inspections (there were between 3 and 25 engineers' inspections for each dam) were evaluated for embankment defects, which in Oregon are most commonly related to plants and animals and damage done near the surfaces of the dam. Inspections also evaluated operation of the valve, how frequently cycled, and is it even operational or is it just partly operational. Inspection records include leakage through or around the conduit.

Emergency Preparation including Dam Access and Monitoring: Evaluation of the EAP, its date and in current format, whether it has been exercised. Several of the dams are in mountain locations where winter road access is blocked by snow. A few dams have real time reservoir level gages for identification of potential restriction level exceedances or overtopping. Most of the dams do not have any instrumentation for monitoring reservoir level, drainage flows, or anything else.

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All data from the comprehensive file review were evaluated for reliability based on the criterial found in the full paper. Data are assigned one of three confidence levels:

Reliable – The design engineer submitted as-builts or a formal survey and analysis or similar.
Average – There is an approved design that includes data for this part of the dam and no discrepancies between design and inspection information.
Minimal – There are no approved design documents or there are discrepancies.

Assessments for PFMs 1, 2 and 3: Data required for overtopping analysis included: Spillway dimensions; watershed area above the dam; and LiDAR data of the watershed above the dam. This analysis includes hydrologic and hydraulic analysis of reservoir storage and spillway capacity. Other agency staff developed regression equations up to 0.2 % AEP. This information required the following general steps to determine the inflow and flood recurrence where the dam overtops:

A. Hydrologic Model Development
B. Hydraulic Model Development
C. Stochastic Model Development & Simulations
D. Probability of Overtopping and Hydraulic Risk

The hydraulic characteristics of the reservoir, dam, and downstream topography were defined using the HEC-RAS, version 5.0.5 [6]. For each dam, a 1D, 2D, or combined 1D/2D hydraulic model was developed. Models were extended downstream at least one mile to ensure that any errors associated with model boundary conditions did not influence the results of the area of interest. When sufficient information was available, models were calibrated with gage data or calibrated to the 100-year base flood elevation shown on the FEMA Flood Insurance Rate Map using the 1% AEP as inflow to the model. Staff developed a program using Visual Basic for Applications (VBA) in Microsoft Excel that communicates with the HECRASController API to automate model simulations and extract model output. The program then runs multiple Monte Carlo simulations which provide a means to account for the epistemic (knowledge) uncertainty and the aleatory (natural variability) uncertainty and ultimately provides the probability of overtopping during the selected event.

The HEC-RAS model described was used to define the inflow discharge versus reservoir elevation relationship for each dam. The average, lower, and upper limits of the relationship were defined as follows:

- Average: average volume hydrograph with expected spillway coefficients
- Lower Limit: a low-volume hydrograph with high spillway coefficients
- Upper Limit: a high-volume hydrograph with low spillway coefficients

To achieve this, for each dam, the program ran the HEC-RAS model for a range of flows corresponding to exceedance probabilities determined from a regional regression model with a peak discharge calculator. Flow AEPs ranged from the 1% AEP to 0.01% AEP. For each AEP, the models were run at the average, upper confidence level (UCL), and lower confidence level (UCL). Maximum and minimum values were also set. The recurrence of the flow that would cause overtopping was then determined or approximated, embankment dam failure risk is assigned a value from 0.1 to 0.2 depending on dam material and embankment thickness.

Data for blockage analysis (PFM 2) includes the following:

- Watershed characteristics, especially the area of forest with trees taller than the width of the spillway, most determined from LiDAR data,
- Potential for ice jam movement and for ice on gates, and
- Presence and condition of the log boom.

The probability is estimated through an index value that is a multiplier of a rating for the each of the risk factors above.

Data for spillway chute erosion potential analysis (PFM 3) includes:

- Spillway channel material,
- Historical spillway erosion,

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- Historical spillway leakage, and
- The presence and function of spillway drains.

All dams were compared using an index rating:

Spillway Stability Index = (Material)(Erosion)(Drain)(High Velocity)

As with PFM 2, the failure probability for PFM 3 was estimated through an index value that is a multiplier of a rating for the each of the risk factors above. As an example, index values for PFM 3 ranged from 2 to 500. The 500 was a special case because of a drop inlet spillway made of a failing corrugated metal pipe, with the highest assessed annual failure risk in the risk evaluation project. No other HHPD dam had anywhere near as high a rating for spillway stability.

Assessments for PFM 4 and 5: Key calculated data for these PFMs were the hydraulic gradient(s) at the 0.2% inflow flood and current dam dimensions. Other data are index values or condition ratings. This project used an index of soil internal erosion potential. The Erosion Rate Index, $\mathbf{I_{HET}}$ [5] is the hole erosion index for a solid compacted to 95% dry density at optimum moisture content and can be grouped into six categories. The $\mathbf{I_{HET}}$ is highest for dispersive soils, which are rare in Oregon. For other soils, the highest risk has a Unified Classification of silty sands (SM) with less than 30% fines. (Note that this is also a material that can be prone to liquefaction (both seismic and static). A material factor ***(M)*** was used to modify the Erosion Rate Index, since construction practices greatly influence potential for internal erosion. For homogenous dams, soil moisture for compaction and consistent compaction all affect internal erosion potential [5] and information from this reference included relative importance for a number of the index values developed for this project.

None of the dams in this project has a soil filter or a fully filtered exit. For the upper part of the dam only, damage after construction can occur due to vegetation, animal burrows, and cracks from desiccation or freezing and are given a damage rating index. The higher the damage rating, the greater the potential for concentrated leak erosion. The freeboard at the 0.2 % AEP inflow was used for consistency, and is a level not experienced by these dams. The less the freeboard, the greater the likelihood of flow into desiccation cracks, burrows, and other damaged embankment soils that are concentrated near the surface of the dam [7]. The index value for internal erosion in the upper 1/3 of the dam was developed for this project by the dam safety program, and is as follows:

$$\text{Index Value} = \boldsymbol{IE_s} = \frac{(\boldsymbol{H})(\boldsymbol{D})}{(\boldsymbol{L})(\boldsymbol{M})(\boldsymbol{F})}$$

IE_s – Upper one-third of dam (surface) dam internal erosion rating
H – Height (at 0.2 % AEP) from water level at 0.2 % AEP to max depth of defects on downstream face, or 10 feet, whichever is higher
L – Length of near surface flow path (with H/L being the hydraulic gradient)
M - Material and construction practice rating
D – Surface damage rating
F – Freeboard at the 0.2 % AEP reservoir level (adjusted if freeboard under 0.5 feet)

For internal erosion, we compared all dams with the index for an average dam. As an example, the range of index values for internal erosion in the upper 1/3 of the dam was 0.06 to 3.96, with two dams having this as the controlling PFM. For the highest index the annual risk of failure was estimated to be between one and two orders of magnitude above the average dam, so to the nearest order of magnitude would be 1.0E-3.

Data for internal erosion at the base of the dam include, in addition, a conduit defect rating and seepage history rating, and does not use surface damage or freeboard. There is a somewhat similar index equation for failure at the base of the dam, as described in the full paper [3]).

Assessment for PFM 6: While rigorous seismic deformation analysis is complex and still may not be precise, there is a simplified deformation method developed by J. Swaisgood [8]. He presents a case history evaluation as a tool for immediate assessment of a structure that has undergone seismic loading,

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and a method for checking the results of complex deformation analyses against actual, historic dam behavior during earthquake. The total Swaisgood database has 82 dam incidents, including 4 from recent subduction zone events, however, no liquefaction cases were included in the regression equations he developed.

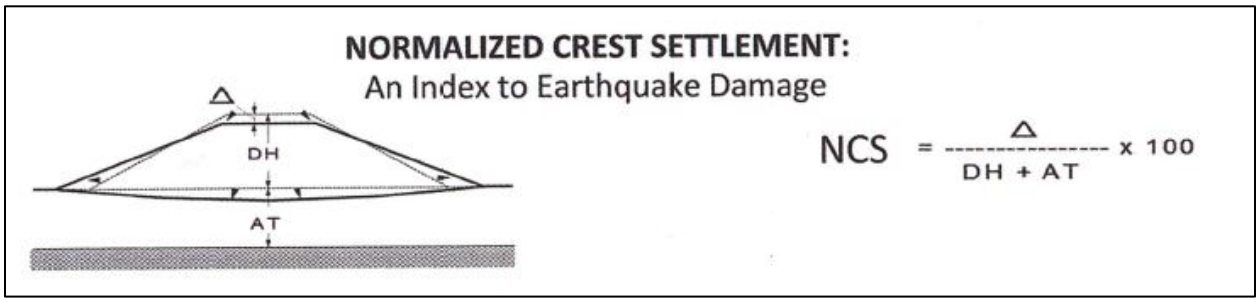


Figure 2: Normalized Crest Settlement [7]

We used the design ground acceleration in the USGS charts for both seismic regions. The reservoir level during the earthquake is likely around a normal operating level of the reservoir, as an earthquake at during an extreme flood would be an even more remote event. Data required initially were the height of the dam and the depth to rock, and earthquake acceleration and magnitude. We estimate the normalized crest settlement (NCS) from the maximum considered earthquake (MCE) for each dam site for 500-year (2% exceedance in 50 years) and 2,500-year (10% exceedance in 50 years) events. In addition to the methods in Swaisgood, materials were grouped into four vulnerability categories: LQ – liquefiable (alluvial larger basin); SS – strain softening (silts) NM – normal HD – high density. Finally, a probability of failure was assigned based on the calculated freeboard remaining after the Swaisgood deformation was deducted. We then calculated freeboard remaining the Swaisgood calculated deformation was subtracted and assigned failure probability based on the freeboard remaining. Estimated failure probabilities based on remaining freeboard are included in the full paper.

Assessment for PFM 7: The lead author, while working for the state department of forestry, was responsible for assessing landslide risk across the landscape [4]. Based on this work, the primary data used to determine risk of large rapidly moving landslides that could cause wave overtopping or spillway blockage included:
1) The general geologic unit or sub-unit's vulnerability to large rapidly moving landslides.
2) Steep slopes above the dam or reservoir, with slope breaks of 40% and 70 %.
3) Identification and size of rapidly moving landslides in that geologic unit or sub-unit, with the potential for overtopping based on the size of those rapidly moving landslide scars.
4) Landslide area as a determination of landslide geomorphology (arcuate or angular scarps with scour or deposit downslope) that are evident on LiDAR, remains of toe or debris flow or avalanche runout

The evaluation is an index value that includes acreage (above the reservoir) of each of the data factors. We compared landscape features to the conditions above the Skookum dam (Figure 1) which had just been removed due to landslide risk and which as estimated at annual failure at 1.0E-3 (only one dam in this project had similar risk).

Assessment for PFM 8: Dam specific modes were determined for concrete dams, and in some cases for unusual stability issues with the penetrating conduit. Dam specific modes included:

1. S*liding along the base of a concrete dam;*
2. *Toppling of a concrete dam from extreme flood eroding a dam where the spillway drops 90 feet onto soft rock at the base of the dam; and*
3. *Unusual structural issues in the conduits through embankment dams.*

Loss of Life Estimates: The PAR is the portion of the population downstream of dam that is affected by a dam failure. The PAR is not an estimate of expected life loss in an actual failure scenario. There are several factors that will normally result in life loss much under the PAR. Federal Risk

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Guidance [9] was reviewed and used in part to make these assessments of expected loss of life as follows:

***Access to the dam in critical periods, and frequency of direct observations*:** The evaluation included road access during all periods when an extreme event could occur or otherwise when dam breach would be more likely. A number of dams in this study have no access in winter months because of snow cover. In these areas, the extreme flood events occur in winter and spring. Compounding this issue, most of the dams do not have an operator and are not in public view, so identification of a potential failure may not occur.

***Instrumentation for monitoring of potential failure modes*:** Three of the 15 dams have real time reservoir level instrumentation that is monitored by the dam owner and the dam safety program. This allows for timely inspection for high flow related potential failure modes. The other 12 dams have no instrumentation for remote monitoring. Remote monitoring for advanced warning of internal erosion, seismic and landslide potential failure modes would be problematic on any dam.

Speed of failure progression after identification: Dams of sand and sand like material are likely to have the fastest progression of failure from problem initiation to breach. Well compacted rockfill dams with clay cores should have the slowest progression from failure initiation to full breach.

Distance from the dam to persons at risk: The greater the distance from the dam to the locations where people are present the longer the arrival time of dangerous flooding, and therefore the greater the potential for effective evacuation to take place.

The velocity and depth of the breach flows and the types of structures: As flow depth and velocity increases the risk to persons increases greatly. Most of the EAP's for these dams include information on depth and velocity.

These factors and how they were considered when estimating loss of life are discussed in detail in the report. For this project, we focused on the owner's ability to identify a potential failure, the distance to the population at risk, and a categorical rating for depth and velocity and time to breach to develop the reduction factor. This factor was multiplied by the PAR to determine expected life loss in a dam breach. For this project it ranged from 0.05 to 0.4. and required significant judgment. Note that with additional information and resources there are software programs to aid in this type of evaluation [10].

V. RESULTS AND DISCUSSION

We compiled all information in data spreadsheets for each dam, then into an analysis spreadsheet, which are shown in the full paper [3]. The analysis for calculation so all key safety elements (or lack thereof) and index values could be compared. We calculated or estimated an annual risk or failure for each PFM, unless that PFM was not applicable to the dam (concrete dams, and for landslides, areas with gentle stable slopes). Risk of Failure was determined to an order of magnitude only, as data is insufficient for more precise calculations. PFMs were only carried forward if they had the highest order of magnitude risk of failure. Risks that are at least one order of magnitude less than the highest risk PFM for that dam are not considered to be a principal failure mode. For convenience, if PFMs had the same annual risk of failure they were added together.

The annual probability of failure for each dam (most significant only, unless two PFMs had equal probability) and the adjusted PAR (expected life loss) were used to calculate annualized per capita life loss risk, which is the probability associated with loss of life given the event and is calculated as follows:

$$\text{Annualized Per Capita Life Loss Risk} = \text{Probability of Failure} \times \text{Loss of Life Estimate}$$

As with annual risk of each PFM, the combined risk of failure is determined to an order of magnitude and only considers the highest risk modes of failure. All risks may be one or two orders of magnitude off, depending on the confidence in the data behind the estimates. The summary risks for the 15 dams are shown in Figure 3 below, with the confidence indicated by the very approximate error bars.

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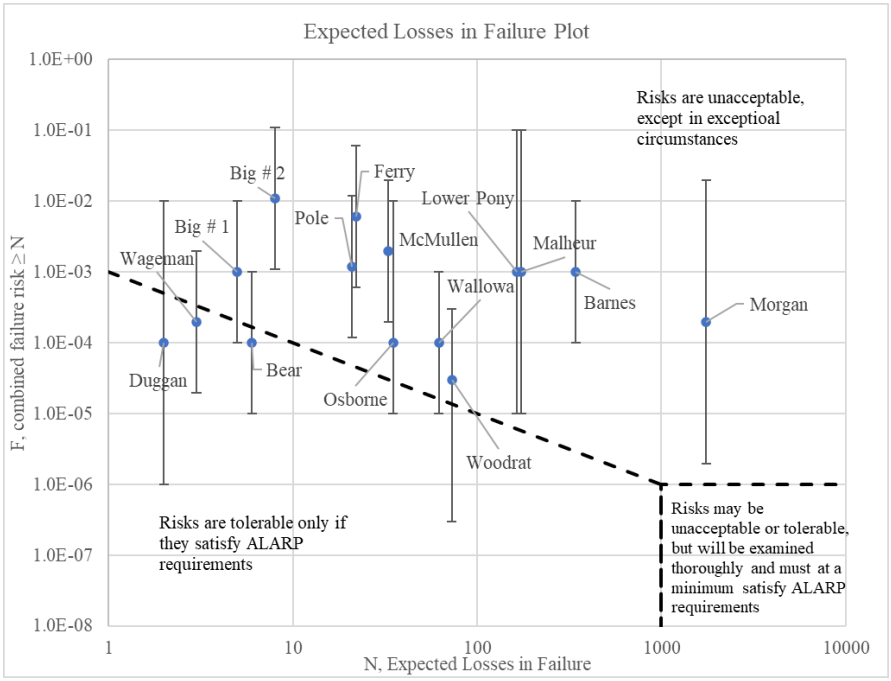


Figure 3: F-N plot showing estimated number of fatalities as a function of the probability of failure (U.S. Army Corps of Engineers [1,10], showing the best estimate and estimated risk range. ALARP stands for as-low-as-reasonably-practicable and is described in the aforementioned reference.

Figure 3 shows that a number of dams far exceed the Federal risk guidelines, with three of the dams close to the federal tolerable guidelines. Overtopping without debris blockage and internal erosion around the conduit were the PFM responsible for at least part of the risk for six dams each, with seismic deformation the highest risk factor for four of the dams (a few dams had multiple equivalent risks). Much more detail can be found in the full paper [3]. From these results, PAR and likely warning and evacuation have a very great influence on the determination of the dams with “unacceptable” risks based on the federal guidelines. Unacceptable risk is a term that, for different jurisdictions, could be different, and more discussion and standardization could be helpful.

Basic data determination for these methods is fairly simple. Evaluation of confidence in the data is more complex and necessitates use of an experienced engineer. We used judgement to correlate index values with risk. Over time any improvement in risk estimates using these methods may require thorough expert forensic investigation of actual dam failures. Most historical dam failure investigations generally did not collect as much data as would be needed to further improve these methods. Finally, this is a prioritization tool for dams needing rehabilitation or removal. Deterministic analysis is necessary for any rehabilitation work on these dams. While it can be completed with a small dam safety staff, this risk analysis work may compete with and reduce other essential dam safety functions in a program with limited staffing.

VI. SUMMARY AND RECOMMENDATIONS

An SQRA was completed in accordance with the scope of work contained in the Oregon FEMA HHPD FY19 grant application. The analysis resulted in a risk-based ranked list of dams that is being used to prioritize deterministic analysis, rehabilitation or removal timeframes, and where owner action is insufficient, enforcement on these high hazard dams. The analysis indicates that the major safety issue affecting Oregon dams is overtopping during flood events, internal erosion at the base of the dam (especially the conduit), and seismic deformation only for those dams closest to the Cascadia Subduction Zone. Several of the dams evaluated under this risk analysis have an apparent risk of failure much higher than the accepted tolerance level outlined in FEMA’s *Federal Guidelines for Dam Safety Risk*

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Management (FEMA P-1025). It should be noted that special inspection, analysis and/or design work is underway to address safety related issues with almost all of the dams evaluated in this risk analysis, most by the dam safety program with special state funding.

Based on the results and due to the semi-quantitative nature of this analysis, there are limitations to this analysis that should be mentioned. Reservoir level was a key factor in determining risk for most of the PFMs. As result, the major limitations are related to the hydrologic inputs to the analysis. The limitations and means to address them are as follows:

- The analysis is limited to one event duration, the 24-hour precipitation event.
- Oregon needs up-to-date precipitation estimates with spatial and temporal distributions.
- Some of the peak discharge estimates are likely incorrect; the regression model underestimates the peak discharge in at least one Oregon geographic region.

Recommendations for future actions are as follows:

1. The dam owner needs to have deterministic engineering analysis conducted for mitigation of the highest risk mode(s) of failure.
2. The dam safety program is finalizing contracts for a precipitation frequency analysis and also a related Probable Maximum Flood analysis for Oregon, which could be used to help determine more accurate extreme flood inflows, outflows and overtopping risks in the future. This will improve confidence in the overtopping type PFM results when available.
3. The dam owner needs to develop more specific timeframes for rehabilitation or removal of the highest risk dams. Interestingly, most of these were rated in Poor condition, not Unsatisfactory condition.
4. The dam safety community should improve and modify these methods as necessary to allow comparative risk analysis for other dam safety programs. We suggest a team of nationwide experts by area of expertise be assembled to improve the reliability of risk assessments.
5. All dam failure investigations should include an evaluation of all the key safety elements used or not used on the dams that failed in future dam failure investigations.

VII. ACKNOWLEDGEMENTS

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IX. AUTHOR BIOGRAPHIES

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DAM SAFETY PALM SPRINGS
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7 High Hazard Dams Risk Assessment

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ABSTRACT

The objective of the Risk Assessment Study was to develop Potential Failure Modes (PFMs) for seven privately owned high hazard dams regulated by the Idaho Department of Water Resources (IDWR), and from that effort, prioritize those features most needing repair, rehabilitation, and/or replacement. The objective was also to categorically describe the Risk Reduction Measures (RRMs) that could be utilized to reduce the acceptable risk to downstream populations, and to prepare corresponding appraisal level cost estimates for each recommended RRM to describe how these seven high hazard dams can benefit most from future intervention.

The Risk Assessment Study was commissioned by the IDWR Dam Safety Program with funding assistance provided by DHS-FEMA though the High Hazard Potential Dams (HHPD) grant program. The study successfully accomplished the following:

- Understand which dams would benefit the most from future available funding opportunities.
- Prioritize which dams present the greatest risk to downstream population.
- Rank the importance of recommendations that identify risk reduction measures.
- Identify dam safety deficiency priorities and estimated remedial cost(s).

I. INTRODUCTION

A. Purpose

In September 2021, Idaho Department of Water Resources (IDWR) issued a Request for Qualifications to develop a Failure Mode/Effects Analysis for seven aging high hazard dams. The IDWR Dam Safety Program had previously identified this group of non-federal dams due to existing structural, mechanical, or operational deficiencies; or because the original design and (or) construction methods do not meet currently accepted safety standards for high hazard potential dams (HHPD); particularly with respect to perceived risks to downstream populations (PAR).

McMillen, Inc. (McMillen) was selected by IDWR from a group of four qualified engineering consultants to perform the assessment. McMillen proposed performing a semi-quantitative risk analysis (SQRA) to accomplish the stated objectives listed in the IDWR Scope of Work. The SQRA captures key information and provides the rationale that supports a summary report with recommendation(s) for reducing or otherwise mitigating risks. Other outcomes include a better understanding of each project's deficiencies through the examination of the primary risk-drivers, better prioritization of studies, more focused inspections, and improved surveillance and monitoring. The SQRA focuses on reasonably credible potential failure modes (PFMs) to determine which scenarios are considered the most significant at the dam, and thus does not pursue developing the entire list of less likely events with corresponding annual probabilities-of-occurrence smaller than approximately 10^{-6}. 10^{-6} was selected as a threshold level because this value is considered tolerable regardless of the number of life loss.

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B. Description of Dams

Blacks Creek Dam – Blacks Creek Dam is an earthen embankment dam with an overall height of 51.5 feet, a hydraulic height of 45 feet, crest length of approximately 1,400 feet, and crest width that varies between 8 and 12 feet. The reservoir has a reported storage capacity of 3,640 acre-feet. The tallest portion of the dam is built across a 300-foot wide steep-sided river canyon carved into the basalt bedrock. The remaining length of the dam embankment sits on the surface of the surrounding plain and tapers from approximately 20 feet tall at the edge of the canyon until the crest meets the abutment. The dam was originally constructed between 1900 and 1905 and was raised 7 feet in 1915 to its current elevation.

A concrete spillway was constructed on the right abutment in 1915. Thirteen spillway piers provide 12 openings (bays) and each pier has slots on the upstream side to allow flashboards to be installed; however, a practice currently prohibited by IDWR. The spillway channel is excavated in bedrock and is discharged off the basalt cliff into the creek channel approximately 200 feet downstream of the dam. The flowline of the spillway channel slopes towards the embankment and downstream creek channel. A short training wall exists along the left side of the spillway to direct the flow downstream to the creek channel. The training wall was built with stacked rocks mortared together, and with an unreinforced concrete facing.

A formed concrete low-level outlet is located to the left of the maximum section. A slide gate at the upstream end controls outflow and is operated manually with a crank located on the dam crest.



Photo 1: Blacks Creek Dam crest and downstream face looking towards the right abutment.

Crowther Mill Dam – Crowther Mill Dam is an earthen embankment dam with structural height of 90 feet and hydraulic height of 85 feet impounding 960 acre-feet of water. The crest is approximately 40 feet wide with an upstream slope approximately 3:1 (H:V) and downstream slope approximately 2:1 (H:V). A two-lane dirt road crosses the crest. A concrete cutoff wall is shown on the 1948 drawings but is not documented in construction photos or records. The dam foundation is shown on the 1948 drawings as consisting of “layers of blue clay.”

The primary spillway is a 5-foot diameter steel drop-inlet pipe with a wall thickness of 1/4-inch. The vertical drop (glory hole) transitions to a 4-foot diameter pipe inclined at or near a 45-degree angle which passes through the earthen embankment and discharges into a stilling pond at the dam toe. The drop inlet is set approximately 5 feet below the dam crest. An earthen channel exists at the left abutment and is blocked with a low-height earthen berm (fuse plug) across the channel width. The fuse plug allows the drop inlet to operate between 3 and 5 feet below the dam crest before the fuse will overtop and wash out. The owners report that the auxiliary spillway has never been utilized.

The low-level outlet is a 23-inch steel pipe encased in 10 inches of reinforced concrete with anti-seepage collars every 25 feet. The outlet is controlled by an upstream gate valve from a valve house on the dam crest. The outlet pipe was video inspected in 1998 and it was reported that the pipe had settled 6 inches and there was water leaking through joints at four locations. Leakage inside the outlet pipe was witnessed during the 2022 site inspection.

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Photo 2: Crowther Dam downstream face looking towards the right abutment.

Deep Creek Dam – Deep Creek Dam is an earthfill embankment dam that is 96 feet tall and 1,100 feet long with a reported reservoir storage capacity of 5,540 acre-feet. The crest is approximately 20 feet wide, the upstream slope is approximately 3:1 (H:V), and the downstream slope is approximately 2:1 (H:V) with two benches. A one-lane public road exists across the top of the dam crest to access boat launch facilities located on the left shoreline of the reservoir. Vehicles must also travel across the entrance to the emergency overflow spillway. According to borings performed and described in a 1976 investigation report, the dam has two zones: "the upstream three-fourths is predominantly silty clay, with some layers of plastic clay, and thin, random lenses of limestone fragments; and the downstream one-fourth is primarily clayey gravel and clay-sand mixtures". The 1976 report found the dam to be unstable and recommended constructing a downstream buttress. This approximately 32-foot-tall buttress was constructed in 1987 and included the addition of a toe drain. In 1994 the dam crest was raised approximately 2 feet to its current elevation of 5,194.0 feet to provide additional recommended freeboard.

A 30-inch outlet conduit is located at the maximum section of the dam and is controlled by a gate valve with a control located in a gate house at the dam crest. The original gate valve was replaced in 1983 with a butterfly valve that is still in operation today. The original conduit was slip-lined with high density polyethylene (HDPE) in 1990. The outlet works discharges into a concrete intake structure for municipal water supply.

An uncontrolled unlined emergency spillway is excavated into the left abutment hillside and discharges water down the hillside into the creek channel a few hundred feet downstream from the dam.



Photo 3: Deep Creek Dam looking towards the left abutment.

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Fish Creek Dam – Fish Creek Dam is a concrete multiple arch buttress dam, constructed between 1915 and 1922. It is approximately 92 feet tall at the maximum section and 1,725 feet long. The dam presently consists of 69 vertical buttress walls supporting 67 barrel arches inclined in an upstream-to-downstream direction at an angle of approximately 45 degrees. Due to safety concerns about the structural integrity of the concrete dam, two (2) of the original 69 barrel arches were removed that resulted in lowering the maximum reservoir elevation by approximately 20 feet and reducing storage capacity from 12,743 to 5,515 acre-feet. Of the remaining 67 arches, there are 19 arches with a vertical height of 33 feet or greater. These 19 arches comprise the following:

- Seven (7) “High Bay” arches, approximately 90 feet high with two sets of horizontal (lateral) struts spanning the distance between the buttress wall(s).
- Three (3) arches located to the right (east) of the High Bay arches, each approximately 50 feet high with a single set of horizontal struts between the buttress wall(s).
- Nine (9) arches located to the left (west) of the High Bay arches, each approximately 33 feet high with a single set of horizontal struts between the buttress wall(s).

In addition to those features identified above, there exist another eleven (11) arches taller than 33 feet that exhibit horizontal struts.



Photo 4: Fish Creek Dam view from the right abutment.

Mountain Home Dam – Mountain Home Dam is an earthen embankment dam approximately 48 feet tall, 3,700 feet long, and with an approximately 12-foot-wide crest. The upstream slope is approximately 2.5:1 (H:V) and the downstream slope is approximately 1.5:1 (H:V) impounding a reservoir with 5,470 acre-feet storage capacity. A paved 28-foot-wide county road crosses the downstream face at approximately half the dam height, effectively forming a buttress against the downstream toe.

The dam was originally constructed in 1884 to a height of 11 feet. It was raised in 1902 to approximately 15 feet. It was then raised another 33 feet to its current height between 1915 and 1920. Major repairs were made to the dam in 1970, including adding the 28-foot-wide roadway berm to the downstream face, armoring the upstream face with riprap, and re-grading the dam crest to a uniform elevation.

The outlet consists of mortared rock at the foundation contact both upstream and downstream of the centrally positioned gate well located near the maximum height of the embankment. In 1967 the original outlet gate was replaced with a 36-inch round slide gate. Installation of the new gate included the addition of a 6-foot length of 36-inch diameter steel pipe and gate frame, both set into place at the downstream end of the gate shaft (wet well) and secured with concrete. Water releases currently are controlled manually using this mechanical feature.

An uncontrolled open channel earthen spillway is located at the left end of the embankment. At this location, the dam crest elevation drops about 6 feet onto the underlying abutment rock. The spillway entrance matches this

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elevation for about 75 feet and then terminates at a small embankment approximately 4 feet above the spillway crest until it meets the slope of the left abutment. The spillway channel passes downstream through a small dip in the county road and disperses flow into the field south of the dam toward Rattlesnake Creek.



Photo 5: Mountain Home Dam looking towards left abutment.

Strong Arm Dam (Treasureton Reservoir) – Strong Arm Dam is an earthen embankment dam approximately 40 feet tall with corresponding reservoir storage 1,710 acre-feet. The crest is approximately 500 feet long and 10 feet wide. The dam was originally constructed prior to 1920 to an approximate height of 25 feet and was subsequently raised in 1920 and again in the 1940s. Several other repairs and modifications were performed on the dam. A buttress was added to the downstream face in 2019 in response to seepage and stability concerns.

A concrete spillway structure with flashboards slots is built in the right abutment. A spillway channel is cut into the right abutment hillside and discharges water a few hundred feet downstream. The owner typically installs flashboards in the concrete spillway, often to a height that leaves just two (2) feet of freeboard between the maximum water surface and the dam crest.

The dam was originally built with a concrete low-level outlet tunnel through the embankment. The outlet resembles a “wet well,” consisting of a 24-inch diameter “black dipped pipe” installed from the reservoir to the cross-sectional center of the dam where it intercepts a vertical concrete gate shaft fitted with a slide gate against the downstream side of the shaft. When the gate is placed in a closed position, water fills the vertical concrete shaft to equal that of the reservoir surface. The lower (downstream) portion of the concrete outlet tunnel was slip-lined with a grouted, 30-inch diameter corrugated metal pipe in 2003. In 2019, the outlet conduit was extended with HDPE when the downstream buttress was constructed.



Photo 6: Strongarm No. 1 Dam looking towards the left abutment.

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Winchester Dam – Winchester Dam was constructed in approximately 1910 to impound water for log storage and lumber mill operations. The dam is a 40-foot-high earth and rockfill structure with a crest length of 218 feet. As reported, an approximately 2-foot-thick concrete core wall is located near the original downstream edge of the dam's crest and is shown on the design drawings to extend from the foundation to the top of dam and across the full width of the embankment. A small portion of the top of the core wall is presently exposed to view.

The design drawings show a narrow dam crest, approximately 10 feet or less but no dimension is given. According to records, the dam initially leaked so much that material was added on the upstream slope in an effort to control the leakage. Later, additional material was added to the embankment; the first to accommodate the construction of U.S. Highway 95 across the crest, and periodically for a number of years to reduce leakage until the present crest width of approximately 140 feet was achieved. The highway was later re-routed to occupy its present location approximately one mile east of the dam.

The original dam was constructed with a 36-inch low-level concrete outlet (LLO) conduit at the foundation interface. The original outlet conduit was determined to be leaking soon after construction and reportedly was plugged and abandoned. However, despite all these efforts, the dam continued to leak at the downstream toe. In 1978, an investigation of the leakage was performed with several borings through the embankment including one that is reported to have been drilled through the abandoned LLO. Tracer dye was introduced through several borings that showed communication with seepage at the downstream toe in less than one hour. They estimated 300 to 400 gallons per minute were exiting the downstream toe under the spillway and another 50 to 100 gallons per minute were exiting in a primarily horizontal band extending approximately 20 feet east of the spillway. The conclusion was that significant leakage was occurring through both the abandoned LLO and portions of the concrete core wall.

According to a 1948 IDWR report, the concrete core wall was damaged when a portion of the original timber spillway failed causing erosion of the downstream embankment slope. Enough material was removed from the downstream side of the core wall to expose the base of the wall. The loss of lateral support reportedly caused the wall to bulge downstream and crack horizontally. Repairs were reportedly made by using two steel cables and eyebolts inserted though the wall, anchoring the cables to twin 8-inch x16-inch vertical timbers, and pulling it back into place using turnbuckles. Embankment material was replaced by bulldozing it from the nearby abutment hillside.

A gated concrete spillway with two erodible fuse plugs is located in about the center of the dam crest. The outlet works consist of a 12-inch diameter HDPE siphon pipe controlled by a valve at the downstream toe to the right of the spillway. Estimated storage capacity is approximately 1,425 acre-feet covering about 98 acres at the normal maximum water surface elevation of 3,904.6 feet above mean sea level (MSL). The drainage area above the dam is approximately 13 square miles.

Remedial grouting to reduce observed leakage at the downstream toe was performed in March and July 1980 but apparently unsuccessful in stopping or reducing leakage. A detailed report from the March 1980 grouting operation was included in the historical documentation. A detailed plan for the July 1980 grouting operation was also included.



Photo 7: Winchester Dam looking across downstream face towards the right abutment.

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C. Methodology

The following tasks were performed in the assessment of each dam.

Document Review – The McMillen project team reviewed documentation provided by IDWR prior to performing the site inspections and SQRA workshops to gain an understanding of each dam and its historical issues. This included reviewing available original design drawings and specifications, historical photos, correspondence, IDWR site inspections and trip reports, and all other available supplemental information. These documents helped the project team form a basic yet comprehensive understanding for each dam.

Site Visit – The project team performed a site visit at each dam, with the dam owners present. The purpose of the site visit was to visually assess each dam's present condition and to look for any visible defects or evidence of anomalous behavior that would be relevant to the SQRA workshop. The site visit included inspection of all observable external portions of the dams including the crest, upstream and downstream embankment slopes, left and right abutments, and the area located immediately downstream of the dam (toe). The inspections also examined appurtenant structures including, but not limited to emergency spillways, outlet controls, and monitoring instruments and other features deemed relevant to the SQRA workshop. This also included a firsthand look at downstream population, upstream canal diversions, and other developed infrastructure.

Semi-Quantitative Risk Analysis Workshop – The SQRA workshops were all performed in a similar manner, with the same team from McMillen for the six embankment (earthen) dams. One additional structural subject matter expert was included for evaluation of Fish Creek Dam, which is a concrete multiple arch buttress structure. The team evaluated each dam using the following steps:

1. **Brainstorming**. The first step in identifying potential failure modes was to have the team members exchange their ideas regarding the possible ways that the dam could fail. This type of review session was conducted for each loading condition (normal, flood, seismic).
2. **Screening of Potential Failure Modes**. Additional screening of credible failure modes was performed to evaluate which were significant and should be carried forward into the SQRA. This process focused on the loading initiators that would result in the highest risks. In the interest of time, the total number of fully developed potential failure modes was typically limited to 4 to 6 per dam based on the team's decision to focus on more credible events corresponding approximately to annual occurrence probabilities of 10^{-6} or greater (i.e., failure modes more likely to occur).
3. **Develop Failure Likelihood and Consequence Estimates**. The assemblage of credible failure modes identified in the previous step were then fully described from initiation to failure. This was done in a bullet format that described each individual node of the failure progression. Development of each failure mode included a list of relevant facts and conditions as well as lists of those factors that make one failure mode more likely to occur than another potential event. Based on the discussions and developed factors the team members individually and anonymously assigned estimates for the likelihood of average annual failure. The estimates were then arranged according to order of magnitude only, ranging between 1/10 (1×10^{-1}) to less than 1/10,000,000 (1×10^{-7}).
4. **Develop Confidence Estimates**. Confidence estimates for the failure likelihood and consequence estimates for each failure mode were developed by the team using high, moderate, or low based on the available information used to estimate failure likelihoods and consequences.
5. **Develop Potential Risk Reduction Measures and Management Actions**. The team identified one or more risk reduction measures (RRMs) for each listed failure mode. Some of the more typical RRMs included: more frequent comprehensive dam safety inspections, additions to the surveillance and monitoring instruments, improvements to the emergency action plan (EAP), operational changes, enhanced maintenance procedures, the completion of follow up studies to evaluate currently unknown conditions or suspected deficiencies, and the implementation of physical modifications and structural improvements. RRMs included both relatively inexpensive short term (interim) efforts, and more costly long term (permanent) activities.
6. **Portray Risk Estimates**. Risk is generally defined to be a combination of estimated failure probability and the resulting life loss consequences that would occur if the failure proceeded without intervention. More specifically, risk estimates for water storage dams can be portrayed and plotted on risk matrices; a procedure which is further discussed in Sections II and III below.

Appraisal Level Cost Estimates – Appraisal level cost estimates were developed for each recommended RRM associated with a structural (physical) modification. Appraisal level cost estimates serve the purpose of alternative comparison relative to one another and are not appropriate for establishing construction cost budgets.

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To identify construction budgets realistically, at least 30% design efforts are required which was beyond the scope of the assessment.

To develop appraisal level cost estimates, the general construction steps for each RRM were listed, rough quantities were determined, and a simple cost per quantity was determined. The cost per quantity was assumed to include the cost of construction, contract administration, and field costs. Once the construction costs were calculated, a percentage of the construction cost was added for each of construction oversight (10%), design (5-15% depending on the construction cost), contingency (typically 15%), and unlisted items (10%). Additional engineering analyses were also estimated and included as appropriate.

II. SEMI-QUANTITATIVE RISK ANALYSIS

The SQRA approach is a risk categorization system that assigns likelihood of failure (probability), and consequential categories to potential failure modes. The assignment of probability is based on existing data, both generalized and specific to the dam, and on available consequence estimates developed for the inundation zone downstream from each dam. As illustrated in Figure 1, the SQRA utilizes a risk matrix approach to assess individual potential failure modes as well as the total risk for a project. The dashed line shown in Figure 1 was taken from United Staters Bureau of Reclamation (USBR) public protection guidelines and represents the tolerable risk limit for societal risk associated with dams. The SQRA method provides a relative risk categorization system that is an important primary tool to help organize dam safety priorities. SQRA is also a valuable tool to determine if higher level studies would be beneficial for specific potential failure modes or if structural modifications are immediately warranted.

The McMillen team performed a SQRA for each dam. Idaho statutes do not address SQRA procedures, so an approach was followed that resembled the Federal Energy Regulatory Commission (FERC) and USBR methodologies. SQRAs differ from more formalized quantitative risk analyses in that the probability estimates of failure are grouped according to orders of magnitude only. Quantitative risk analyses will estimate a likelihood of occurrence for each node in the failure mode event tree and the risk is the product of each of those estimates. Likewise, the loss of human life (consequence) is estimated by orders of magnitude for an SQRA.

Due to time constraints, the SQRA was limited to one day for each dam. The seven dams to be assessed had been pre-selected by IDWR due to age, known problems, and design and (or) construction deficiencies; therefore, the SQRA focused primarily on these issues. By addressing the known issues first, the SQRAs were able to be successfully completed within the allotted timeframe.

For each dam, the SQRA results are portrayed in two risk matrices: one showing the estimated risks for the existing conditions, and the other showing the estimated risks incorporating the identified and(or) recommended risk reduction measures (RRMs). As an example, Figures 1 and 2 show the two respective risk matrices for Crowther Mill Dam. The gray circles represent the estimated probability of failure for each identified PFM with its associated consequence estimate. The red circle represents the accumulated PFMs for the dam. The red square represents the range of estimates to be expected. Section III (below) further describes how the matrices are interpreted and used to prioritize risk assessment for a portfolio of dams.

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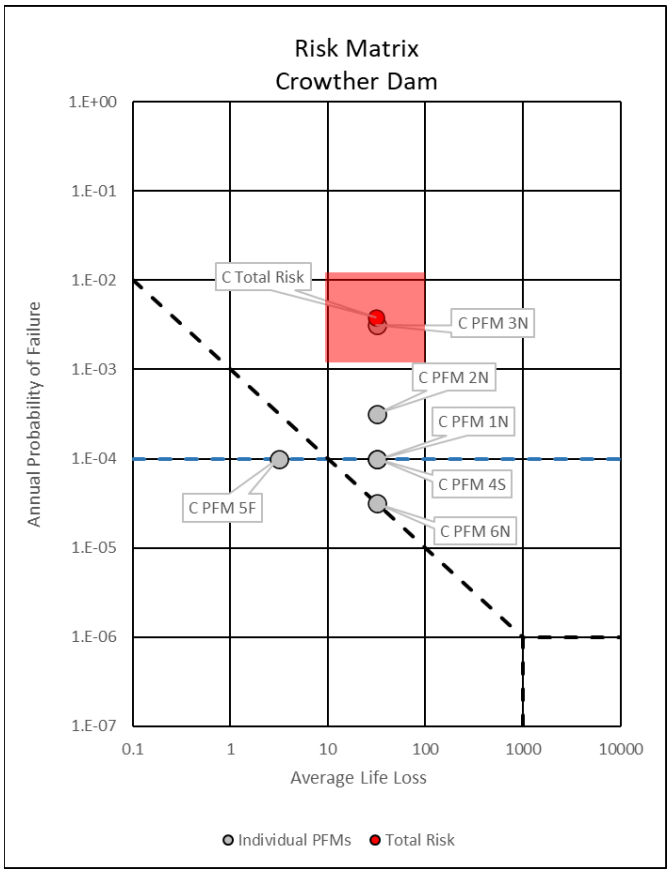


Figure 1 Risk Matrix, Existing Conditions – Crowther Mill Dam

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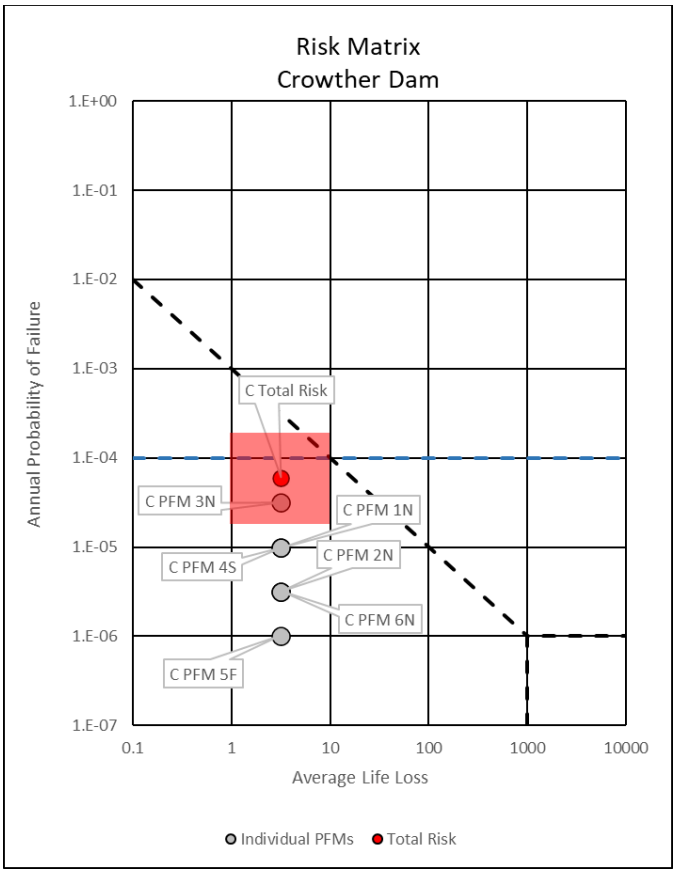


Figure 2 Risk Matrix, RRMs Implemented – Crowther Mill Dam

III. PORTFOLIO RISK ASSESSMENT

The combined results of the risk assessment for each dam are shown on Figure 3. Figure 3 plots the cumulative risk for each dam; referred to here as the total or aggregate risk. As illustrated, on the figure, the total risk that corresponds to all of the seven dams exceed the tolerable risk guidelines (dashed line on the graph). The risk at Strong Arm Dam plots just above the tolerable risk guidelines while the risks associated with the other six dams are higher.

In reviewing the risk matrix, the total risk is measured by the perpendicular distance from the sloping tolerable risk threshold line. Thus, the highest and lowest overall risk (annual probability of failure multiplied by average annual life loss) is estimated at Winchester and Strong Arm dams, respectively. The remaining five dams all have estimated total risks within an order of magnitude of each other. Fish Creek and Winchester Dams have the highest probability of failure (annual probability of failure between 10^{-1} and 10^{-2}), but Deep Creek Dam has the highest average annualized potential life loss (PLL) (more than 100).

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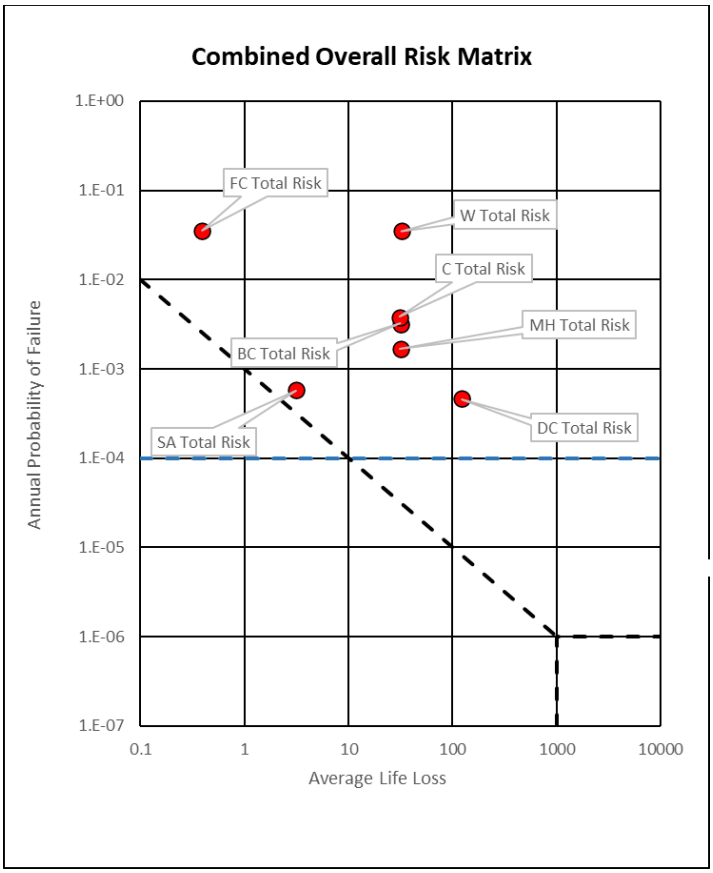


Figure 3 Combined Risk Matrix – All Dams in the Portfolio

Figure 4 portrays the estimated total risks with the recommended RRMs implemented. As shown, if the recommended RRMs are implemented for this portfolio of dams, total risks at each dam are subsequently estimated to be at or below the tolerable risk guidelines. For each identified RRM, a corresponding reduction in risk was estimated. Risk reduction is accomplished by lowering the estimated probability of failure, or by lowering the estimated consequence of failure (i.e. hazard), or some combination of both.

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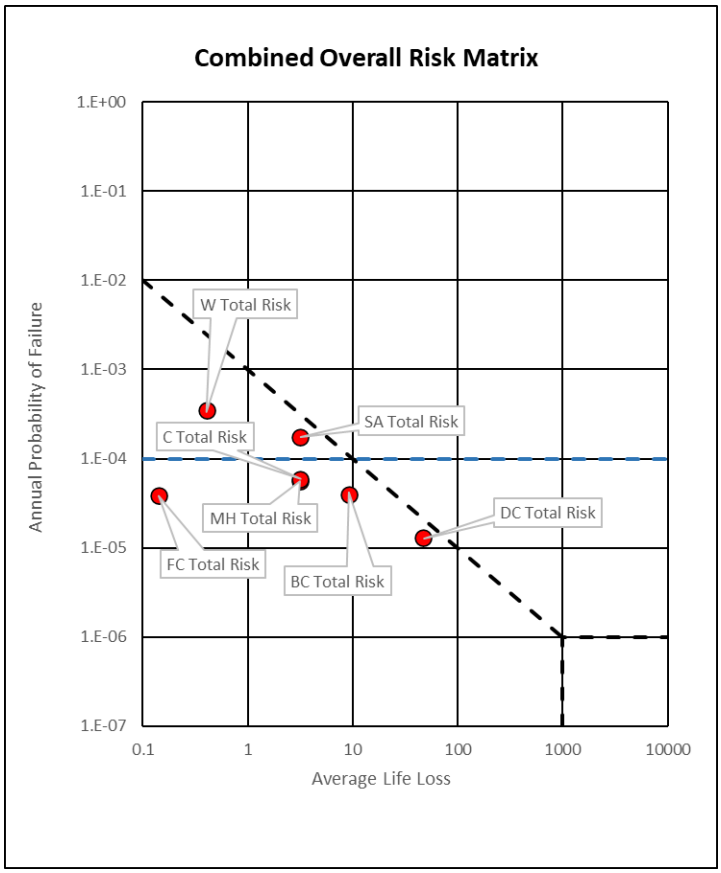


Figure 4 Combined Risk Matrix, RRMs Implemented – All Dams

IV. CONCLUSIONS

The portfolio-wide risk assessment shows that the estimated risks at all seven dams considered in this study exceed the tolerable risk guidelines; therefore, risk reduction measures are warranted. Appendix A summarizes the recommended risk reduction measures for each project dam along with their associated appraisal level cost estimates. The table also shows the corresponding reduction in risk associated with each of the recommended measures. As previously noted, risk reduction is accomplished by lowering the estimated probability of failure, or by lowering the estimated consequence of failure (i.e., hazard), or some combination of both.

To help illustrate risk reduction versus remediation costs, a cost-to-risk reduction (CRR) ratio was proposed. The total risk reduction can be quantified by orders of magnitude reductions for both annual failure probability and average life loss. For example, if one considers Winchester Dam, the implementation of all the RRMs can reduce the annual failure probability by two orders of magnitude. That, coupled with a corresponding reduction in the average life loss of 1.75 orders of magnitude, the total risk reduction for Winchester Dam could be up to 3.75 orders of magnitude. As shown in Table 1, the lower the CRR ratio the better the cost-to-benefit value.

Table 1 Cost/Risk Reduction Ratios

Dam	Failure Probability Reduction (Order of Magnitude)	Consequence Reduction (Order of Magnitude)	Total Remediation Costs	CRR Ratio
Winchester Dam	2	1.75	$1.7 m	0.45
Mountain Home Dam	1.5	1	$1.9 m	0.76
Blacks Creek Dam	.5	2	$2.1 m	0.84

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Dam	Failure Probability Reduction (Order of Magnitude)	Consequence Reduction (Order of Magnitude)	Total Remediation Costs	CRR Ratio
Crowther Mill Dam	1	1.75	$3.5 m	1.3
Deep Creek Dam	1.5	.5	$8.9 m	4.5
Fish Creek Dam	3	.25	$37 m	11.4
Strong Arm Dam	.5	0	$840 k	1.7

With the completion of this important assessment, IDWR and FEMA have been provided the necessary supporting information for distribution to the dam owner(s) as they pursue funding opportunities that will allow implementation of recommended modification, repair,, and(or) other non-structural Risk Reduction Measures to reduce downstream population at risk to within accepted risk guidelines.

V. APPENDIX

Appendix A – Summary of Risk Reduction Measures

VI. ACKNOWLEDGMENT

The authors gratefully acknowledge the Federal Emergency Management Agency (FEMA) who funded this study through the High Hazard Potential Dam (HHPD) grant program.

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Rick Scott, PE, has over 40 years of experience with an emphasis in dam safety. His background is in heavy civil and geotechnical engineering including design, construction, and investigations. Rick has been employed at McMillen Inc. for 5 years, Prior to that he served as a Principal Dam Safety Engineer for Damwatch Engineering Ltd. located in New Zealand. He retired from the Bureau of Reclamation in 2015. His 33-year tenure with Reclamation included Manager of Safety, Security, and Dam Safety for the Upper Colorado Region where he had responsibility for over 60 dams located in the western United States. He has visited and inspected dams throughout the Western United States, New Zealand as well as Australia, the Philippines and Vietnam.

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Appendix A – Summary of Risk Reduction Measures

Dam	Recommended Risk Reduction Measures	PFMs Affected	Impact on Probability of Failure	Impact on Consequences	Appraisal Level Cost Estimate
Fish Creek Dam	FC-RRM-1: Repair and update the public early warning alert system	1N, 2N, 3S, 4F	None	Reduce by approximately 1 order of magnitude	< $50,000
	FC-RRM-2: Perform concrete structural repairs.	1N, 2N, 3S, 4F	Reduce by approximately 3 orders of magnitude for PFMs 1N and 2N and 2 orders of magnitude for PFM 3S	None	Replace 10 struts: < $50,000 Reinforce 22 buttress walls: $47,000,000 Reinforce 21 arches: $5,400,000 Alternatives analysis and DSHA: $200,000
	FC-RRM-3: Modify the bays of the dam to a concrete gravity structure.	1N, 2N, 3S, 4F	Reduce by approximately 3 orders of magnitude for PFMs 1N and 2N and 2 orders of magnitude for PFM 3S	None	Fill 21 bays: $37,000,000 Alternatives analysis and DSHA: $200,000
Mountain Home Dam	MH-RRM-1: Implement focused surveillance accompanied by staff training	1N, 2N, 3N, 4S, 5F, and 6F	None	None	< $10,000
	MH-RRM-2: Install a public early warning alert system MH-RRM-3: Install remote monitoring system for the reservoir elevation and downstream flows	1N, 2N, 3N, 4S, 5F, and 6F	None	Reduce by approximately 1 order of magnitude	< $100,000
	MH-RRM-4: Enlarge and regrade the spillway intake the spillway channel	5F and 6F	Reduce by approximately 2 orders of magnitude for PFM 5F and 1 order of magnitude for PFM 6F	None	MH-RRM-4: $350,000 (Including updated PMF and PMP analysis)

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Dam	Recommended Risk Reduction Measures	PFMs Affected	Impact on Probability of Failure	Impact on Consequences	Appraisal Level Cost Estimate
Mountain Home Dam	MH-RRM-5: Armor the downstream toe				MH-RRM-5: $700,000
	MH-RRM-6: Install a downstream filter and toe drain	1N, 2N, and 3N	Reduce by approximately 1-1/2 orders of magnitude	None	$700,000
Blacks Creek Dam	BC-RRM-1: Implement focused surveillance accompanied by staff training	1N, 2N, 3N, 4S, and 5F	None	None	< $10,000
	BC-RRM-2: Install a public early warning alert system BC-RRM-3: Install remote monitoring system for the reservoir elevation and downstream flows	1N, 2N, 3N, 4S, and 5F	None	Reduce by approximately ½ order of magnitude	< $100,000
	BC-RRM-4: Replace the spillway intake structure BC-RRM-5: Perform general embankment crest repairs and add parapet wall	5F	Reduce by approximately 3 orders of magnitude	None	BC-RRM-4: $1,400,000 BC-RRM-5: $560,000 (Both including updated PMF and PMP analyses)
	BC-RRM-6: Implement tunnel repairs for the low level outlet works conduit	2N	Reduce by approximately 1 order of magnitude	None	< $10,000
Strong Arm Dam	SA-RRM-1: Implement focused surveillance accompanied by staff training	1F, 2S, 3F, 4N, 5N, and 6O	None	None	< $10,000
	SA-RRM-2: Develop a Standard Operating Procedure operations manual	6O	None	None	< $50,000
	SA-RRM-3: Increase the spillway capacity	1F, 3F, and 6O	Reduce by approximately 1 order of magnitude for PFMs 3F and 6O and at	None	$820,000

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Dam	Recommended Risk Reduction Measures	PFMs Affected	Impact on Probability of Failure	Impact on Consequences	Appraisal Level Cost Estimate
			2 orders of magnitude for PFM 1F		(Including updated PMF and PMP analysis)
Deep Creek Dam	DC-RRM-1: Install remote monitoring system for the reservoir elevation and downstream flows DC-RRM-2: Install early warning traffic control system on I-15	1N, 2N, 3F, 4S, 5N, and 6N	None	Reduce by approximately ½ order of magnitude	< $100,000
	DC-RRM-3: Construct a downstream filter and drain DC-RRM-4: Add piezometers to downstream embankment	1N, 2N, 4S, 5N, and 6N	Reduce by approximately 2 orders of magnitude for PFM 1N and 5N, 1 order of magnitude for PFMs 2N and 6N, and ½ order of magnitude for PFM 4S	None	DC-RRM-3: $1,100,000 DC-RRM-4: $25,000 each (assume 4 total)
	DC-RRM-5: Enlarge spillway or construct an auxiliary spillway DC-RRM-6: Construct a parapet wall	3F and 4S	Reduce by approximately 1-1/2 orders of magnitude for PFM 3F (note no additional reduction for PFM 4S due to already low probability of failure)	None	DC-RRM-5: $7,300,000 (Including updated PMF and PMP analysis) DC-RRM-6: $410,000
Crowther Mill Dam	C-RRM-1: Install remote monitoring system for the reservoir elevation and downstream flows	1N, 2N, 3N, 4S, 5F, and 6N	None	Reduce by 1 order of magnitude	< $100,000
	C-RRM-2: Repair the drop inlet spillway	3N	Reduce by 1 order of magnitude	None	Localized repairs: $300,000 Slip lining: $1,000,000
	C-RRM-3: Abandon drop inlet spillway and construct new spillway	2N, 3N, and 5F	Reduce by 2 orders of magnitude	None	$2,500,000 (Including updated PMF and PMP analysis)
	C-RRM-4: Construct a downstream filter and drain	1N, 2N, 4S, 5F, and 6N	Reduce by 1 order of magnitude	None	$500,000 Liquefaction analysis: $50,000

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Dam	Recommended Risk Reduction Measures	PFMs Affected	Impact on Probability of Failure	Impact on Consequences	Appraisal Level Cost Estimate
					Geotechnical investigation: $50,000
Winchester Dam	WIN-RRM-1: Install remote monitoring system WIN-RRM-2: Early Warning System on Hwy 95 WIN-RRM-3: Review/Revise inundation mapping	1N, 2N, 3F, and 4F	None	Reduce by 2 orders of magnitude for PFMs 1N, 2N, and 3F. Reduce 1 order of magnitude for PFM 4F	WIN-RRM-1 and WIN-RRM-2: < $100,000 WIN-RRM-3: < $50,000
	WIN-RRM-4: Downstream filter and drain	1N, 2N, and 3F	Reduce by 2 orders of magnitude for each PFM.	None	$2,400,000
	WIN-RRM-5: Cutoff wall or impermeable liner	1N, 2N, and 3F	Reduce by 2 orders of magnitude for each PFM.	None	$1,500,000 (Including alternatives analysis)
	WIN-RRM-6: Construct ogee crest emergency spillway	4F	Reduce by 2 orders of magnitude.	None	$50,000

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DAM SAFETY PALM SPRINGS
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Evaluating Dam Breach Inundation for Largely Sedimented Reservoirs

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ABSTRACT

This paper reports on a dam breach and inundation study for a dam in which 90 percent of the reservoir is now filled with sediment that has accumulated over its more than 100-year life. In this case, regulatory requirements indicated that the dam breach analysis should assume the reservoir is entirely filled to its maximum operating level with water. This condition drastically overestimates the potential area of inundation. To address this problem, three very different methods were used to model the outflow from a breach and the flow of water and sediment from the reservoir. The methods varied from simplified kinematics analysis to sophisticated particle analysis. The methods were 1) geotechnical kinematic analysis, 2) smoothed particle hydrodynamics (SPH) method, and 3) hydraulic analysis using HEC-RAS and mudflow analyses using FLO-2D. The use of three alternative methods addresses model uncertainty concerns. The SPH method is a computational method for simulating fluid flows. The results of the three methods showed the inundation remained in the upper reaches of the downstream channel away from populated areas.

I. INTRODUCTION

Following the Oroville spillway incident, the State of California passed Senate Bill 92 in 2017 that required owners to prepare dam breach inundation maps for all dams[1] regulated by the California Department of Water Resources, Division of Safety of Dams (CA DSOD). The regulations provide specific requirements for performing dam breach and inundation analysis and for reporting the study results (e.g., maps) to DSOD (CA DSOD, 2018).

Among the requirements stipulated in the CA DSOD regulations, an owner is to conduct and submit a dam breach and inundation analysis assuming the full impoundment is modeled as water. However, the circumstances with respect to the dam evaluated in this study are extremely conservative and an unrealistic assessment of the potential hazard from a hypothetical failure since the reservoir is approximately 90 percent filled with sediment.

The regulations (CA DSOD, 2018) offer dam owners the opportunity to perform an analysis to evaluate the effect of impounded sediments on the downstream hazard. The regulations state,

> "If an owner contends that impounded sediment should not be modeled as water, the dam owner may request department acceptance of a sediment release modeling approach."

[1] With the exception of Low Hazard Dams.

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In response to this statement, the owner discussed the option of evaluating the breach and inundation of the dam, accounting for the sediment in the reservoir with DSOD staff and elected to go forward with an analysis that considers the effect of sediment. This paper describes the analyses that were performed and the results.

II. THE DAM

The dam evaluated in this study is a concrete gravity dam that is 68 feet in height. It was constructed in 1890 and had an original permitted storage capacity of 1,840 acre-feet. It has an overflow spillway located at the center of the dam, with 4-ft high stoplogs that can be installed to provide additional storage capacity. Through time the influx of sediment has resulted in approximately a 90% loss of the as-built impoundment volume to sediment deposition within the reservoir. The existing pool volume is about 146 ac-ft.

III. APPROACH

In dam safety it is not a common practice to model the release of impounded sediment from water storage reservoirs as part of dam breach and inundation studies. Generally, accumulated sediment is assumed to be all water. From the perspective of estimating the area that would be inundated this is conservative as the water will flow downstream and laterally further than water with suspended sediments or mudflows. As such, a standard-of-practice for modeling the release of water and impounded sediments has not been established for water supply dams. We note, however, there are guidelines for consideration of tailings. For example, FEMA's dam safety guidelines on inundation mapping do not mention dam breach modeling and inundation mapping associated with impounded sediments (FEMA, 2013).

To model the runout of impounded sediments, three alternative methods were used to account for uncertainty in performing these types of analyses. The methods are:

1. Geotechnical Kinematic Analysis
2. Smoothed Particle Hydrodynamics Method
3. Hydraulic Analysis Using HEC-RAS and Mudflow Analysis FLO-2D

The methods differ in terms of their approach to the problem and in terms of their modeling capabilities. For example, the geotechnical kinematic analysis is only applicable to the evaluation of the sediment runout, it cannot model the release of the free water in the reservoir, or the mixing of impounded sediments (mudflow) which the FLO-2D method can do. Each of the methods is summarized later in this section.

The following steps were taken to evaluate the flood hazard associated with a hypothetical dam breach accounting for the release of the free water and impounded sediments. The steps are:

1. Determination of Sediment Parameters – Based on available sediment data, estimates are made of the properties of the impounded sediments that are required in the sediment runout analysis.
2. Release of Free Water – In this step, calculations are performed to estimate the inundation that would occur when the current free water storage is released.
3. Sediment Runout Analysis – Calculations are performed to estimate the runout of the impounded sediments when the dam is breached. Three different methods are used to model the runout of the sediment. In this calculation no direct interaction of the free-water and the impounded sediments is considered. However, the fact the sediments are saturated is included in the sediment properties that are used in the analysis.
4. Sediment Laden Water Analysis – The final analysis performed considers the release of the impounded sediments with free water (referred to here as the sediment laden water) and their routing downstream. In this analysis the water is 'bulked' to account for the addition of some of the reservoir sediment to the volume of free water that will be released. Increasing water discharge to account for a concentration of sediment in the flow is known as bulking (Gusman, et al., 2008).
5. Breach and Inundation Results – This final step involves drawing from the results of the prior analyses to define the flood scenario that would result from a hypothetical breach of the dam when the free water and sediments are released.

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A. Geotechnical Kinematic Analysis

The geotechnical analysis consists of two parts. The first part uses slope stability analyses to estimate the volume of sediment that would become unstable and begin to move downstream in the event of a dam breach. A limit equilibrium slope stability analysis was performed to estimate possible failure surfaces where the factors of safety were 1.0 or less in the sediment once the dam is removed.

The second part of the geotechnical analysis is a kinematic assessment that estimates the runout of the sediments once they become unstable. This is a plane-strain, two-dimensional (2D) analysis to estimate the distance the sediment will travel if the dam is instantaneously removed. It is a kinematic method described in Davis, et al. (1988) (see Figure 1). This approach is commonly used to back calculate the residual shear strength of failed embankments (Davis, et al, 1988; Olson, 2000), such as the Lower San Fernando Dam. In these cases, the runout distance is known, and the residual shear strength of soil is calculated. In the present analysis, the soil shear strength is estimated from the available sediment data and a forward analysis is performed to estimate the deformation (runout of the soil mass). Two strengths are estimated for the sediments, a peak strength and a remolded strength. Initially the movements are controlled by the peak strength which gradually decreases to the remolded strength. For the analyses performed, the sediment strength was conservatively assumed to be the remolded strength starting at the time of the dam failure.

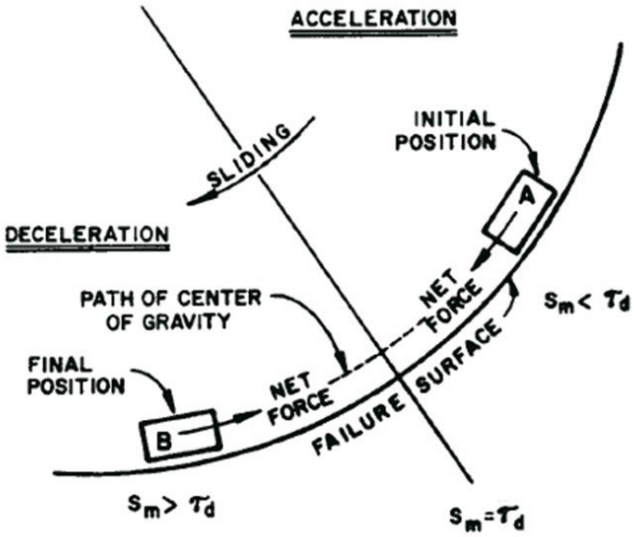


Figure 1 Schematic illustration of the geotechnical runout analysis.

The geotechnical analysis does not address the release of the free water in the reservoir or the interaction of the water and sediment.

B. Smoothed Particle Hydrodynamics Method

The smoothed particle hydrodynamics (SPH) method is a mesh-free method, that overcomes the extreme mesh distortion that afflicts mesh-based methods (e.g., finite element solution). Despite its name, SPH is a continuum method and not a discrete method. The word “particle” in the name does not refer to a physical particle such as a piece of gravel or a grain of sand, but rather, it is a material point that carries with it continuum properties such as volume and mass, as well as continuum variables such as stresses and strains. It derives its efficiency by introducing kernels that allow interaction between neighboring particles. The SPH method can capture the dominant kinematical features of granular flow making it suitable for modeling and simulating the sediment release.

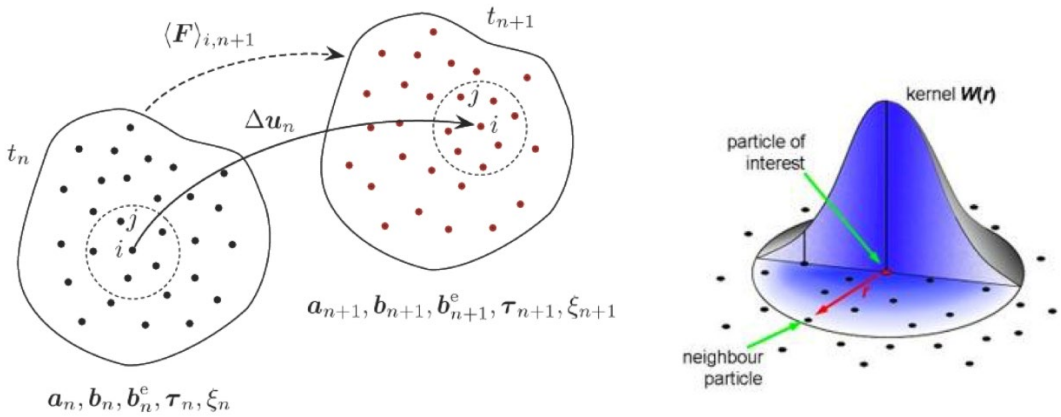
The SPH method was originally developed independently by Gingold and Monaghan (1977) and Lucy (1977) for astrophysical problems. In recent years it has become a popular choice of modelers for simulating debris and granular flow phenomena. McDougall and Hungr (2004) were the first to use the SPH method for simulating the propagation of flow-like landslides. They proposed a framework capable of incorporating more sophisticated rheological models, as well as the influence of topography and material entrainment along the runout path. Crosta, et al. (2009) extended the McDougall and Hungr formulation to three-dimension (3D) using a more elaborate approach to simulate material entrainment and base erosion. Maeda et al. (2006) investigated the impact of

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seepage forces on granular flows with the SPH method. Validation studies of the SPH method have been performed by Favero, et al. (2018) and Favero, et al. (2020) for dry granular flows and for debris flows, respectively.

Each particle (material point) interacts with its neighboring particles depending on the relative distance between the particles (see Figure 2); the closer the two particles, the stronger the interaction between them.

The SPH method was originally developed for problems in astrophysics where no boundaries exist. In order to use it for boundary-value problem simulations, one must use model solid boundaries to indirectly impose essential boundary conditions. This implies that sediment particles may not penetrate the boundary particles, so some degree of friction must be provided between the flowing sediment and the boundary.



(a) (b)

Figure 2. Schematic of the SPH method: (a) particle i interacts with neighboring particle j inside the compact support of the kernel (denoted by the dashed circle; (b) "particle of interest" i and its associated kernel function W(r).

The analysis of sediment flows using the SPH method is carried out in two phases. In the first phase, the dam is breached allowing the sediment to mobilize. The analysis is done in 3D accounting for the irregular geometry of the reservoir in the vicinity of the dam. Because each particle is associated with a certain volume and because this volume does not change with deformation, the total volume of the sediment that flows out of the dam can be estimated from the number of particles that move past the dam. From the motion of the sediment particles within the reservoir, the extent of the mobilized zone in the longitudinal and transverse directions of the reservoir is estimated as well.

The second phase of the analysis simulates the runout of the released sediment downstream. The volume of sediment that flows past the dam (estimated from the first phase of the simulation) is allowed to flow into the creek. The runout distance is estimated by tracking the motion of the particles as the sediment moves downstream.

C. Hydraulic and Mudflow Analyses – HEC-RAS and FLO-2D

II. A one-dimensional (1D) unsteady HEC-RAS was used to simulate the dam breach water outflow hydrograph and flood wave routing downstream of the dam along the creek channel (USACE, 2010). The mudflow analysis used FLO-2D, which is a quasi 2D volume conservation model that can simulate non-Newtonian super-concentrated sediment flows (mudflows and landslides) (FLO-2D Software, Inc., 2009). Simulation of mudflows is based on a set of rheological parameters (sediment concentration, viscosity, yield stress, and laminar flow resistance) characterising the modeled water/sediment mixture.

IV. RESULTS

This section presents the result of a series of dam breach scenarios. The scenarios considered vary from all water cases to various water-sediment combinations. The first two scenarios considered assume the reservoir is comprised of water for the original volume and for the current volume. The results of these scenarios provide general context for the conservative nature of the all water assumption. The next set of calculations estimate the runout of the impounded sediments in the reservoir. Using the three methods of analysis described in the previous section, a comparison of the estimated sediment runout can be made. Lastly, sediment laden water analyses are performed.

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A. Original Reservoir Volume

As noted at the outset of the paper the DSOD requirements indicate a dam breach analysis should be carried out for the original storage volume of the reservoir, assuming it is all water, ignoring the accumulated sediment. To perform the analysis, an unsteady HEC-RAS model was developed to simulate the dam breach outflow hydrograph. The reservoir was assumed to be at its maximum possible water-surface elevation during a sunny day failure.

A sunny day failure of the dam was modeled using the piping failure mode in HEC-RAS. A piping failure, as modeled in HEC-RAS, begins as a leak in the dam resulting in eventual failure of the dam structure. The failure propagates both vertically (up and down) and horizontally during the breach development.

The outflow hydrographs were computed in HEC-RAS using the breach parameters in Table 1. The outflow hydrograph captures the complete dam failure event and reservoir outflow and drainage. The peak outflow from the dam breach is 50,090 cfs located 0.1 miles downstream of the dam. Figure 3a shows the outflow hydrograph for this case.

Table 1

Dam Breach Parameters for the Original Storage Volume

Parameter	**Value**
Center station (ft)	225
Breach Failure Mode	Piping
Breach Elevation (ft NAVD88)	338
Final bottom width (ft)	35
Final bottom elevation (ft NAVD88)	327
Dam crest elevation (ft NAVD88)	344.6
Maximum water surface elevation (ft)	341.4
Breach left side slope	0.65
Breach right side slope	0.65
Breach weir coefficient	3
Full formation time (hour)	0.3

The inundation from this analysis overtops the downstream creek channel and causes flooding in local communities that are approximately 5.5 miles downstream. The total area that is flooded is estimated to be 1,880 acres.

B. Current Reservoir Volume

For the current condition the reservoir topography and water storage volume were based on a recent bathymetric field survey. Volume balance runs were made to verify the cross-sections that represent the correct lake volume for all elevations of interest. The estimated water volume of the reservoir is 146 ac-ft. The breach parameters listed in Table 1 were used for this case as well. A sunny day failure of the dam was modeled using the piping failure mode in HEC-RAS.

The outflow hydrograph was taken at the downstream extent of the dam and reservoir model. The outflow hydrographs capture the complete dam failure event and reservoir outflow and drainage. The peak outflow from the dam breach is 5,057 cfs located 0.1 miles downstream of the dam. Figure 3b shows the outflow hydrograph for this case.

The flow from this dam breach is retained in the downstream creek channel and thus there is no flooding in downstream communities. However, this analysis does not consider the sediment runout or the effect of the sediments on downstream flows.

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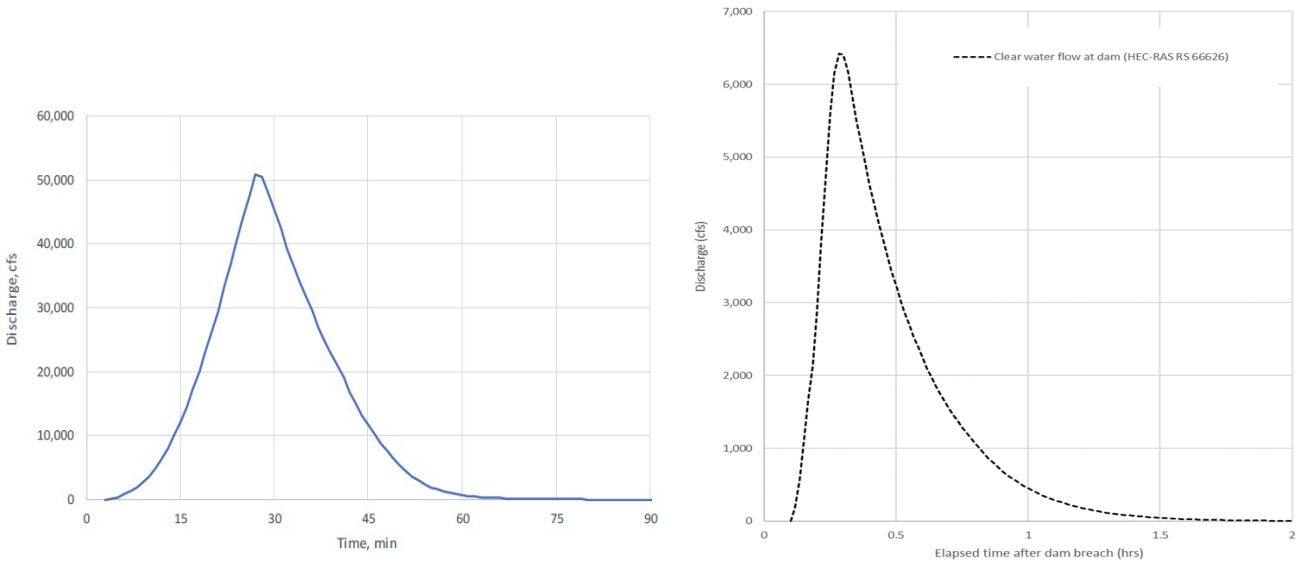


Figure 3. Dam breach outflow hydrograph for a) original storage volume, and b.) current storage volume.

C. Sediment Runout Analysis

As described in Section III, three methods are used to estimate the sediment runout for a hypothetical breach of the dam. This section describes the results of the sediment runout analysis for the three methods.

Geotechnical Kinematic Analysis - As described in Section III the geotechnical kinematic analysis involves a kinematic assessment of the impounded sediment once the support of the dam is removed as a result of breach. For purposes of these calculations, it is assumed the dam breach occurs instantaneously and the entire dam is removed. No interaction with the fresh water in the reservoir is modeled.

Stability of the sediment was evaluated using SLOPE/W, a slope stability analysis software program developed by GEO-SLOPE International, Ltd. (version 9.0.3.15488). The Spencer analysis method (Spencer, 1967) with built-in optimization, which satisfies both moment and force equilibrium, was used. The limit equilibrium analysis resulted in the failure surfaces and factors of safety presented in Table 2.

A factor of safety of 1.0 corresponds to a failure surface that extends about 2,300 ft behind the dam,[2] slightly beyond the extent of the current reservoir bounds. The failure surface (with FS=1.0) was mostly within the fine-grained sediment but extended slightly into the coarse-grained sediment at the far extent of the failure surface. The failure surface did not penetrate the alluvium layer.

Four failure surfaces whose factors of safety were less than 1.0 were chosen to perform the kinematic analysis because the largest failure does not necessarily correlate to the farthest toe runout. These failure surfaces define different volumes of sediment that could be mobilized in the event of a dam failure. Table 2 summarizes the four failure surfaces considered.

Table 3 reports the results of the geotechnical kinematic analysis for the four failure surfaces that were analyzed. Failure surface 3 (see Table 2) moved the furthest downstream. The toe runout for this case is about 4,400 ft downstream. Using the length of the dam of 275 ft to define the width of the sediment, an approximate estimate of conservative estimate of the maximum runout corresponds to about 430,000 cubic yards of sediment moving past the dam and into the creek (This likely overestimates the volume of sediment that is released from the reservoir because it assumes remolded sediment strength at the start of the failure.)

[2] Following completion of the geotechnical analysis, it was recognized the use of the remolded shear strength in the limit equilibrium analysis was overly conservative since the initial instability of the sediments will be determined by the peak strength of 280 psf. The limit equilibrium analysis was re-run using the peak strength showed the maximum extent behind the dam where there is a factor-of-safety of 1.0 is 600 ft rather than the 2,300 ft calculated using the remolded shear strength of 30 psf (Farren, 2019). Using a maximum distance of 600 ft corresponds to a failure surface between surfaces 1 and 2 (see Table 2).

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Table 2

Summary of the Limit Equilibrium Results

Failure Surface	Extent Behind the Dam (ft)	Approximate Corresponding Factor of Safety	Cross Sectional Area (ft^2)
1	425	0.1	16,400
2	1,233	0.2	43,000
3	1,619	0.3	49,500
4	1,934	0.4	52,700

Note, if the peak shear strength of the sediments was used to estimate the failure surface, the estimated toe runout would be in the range of the results for failure surfaces 1 and 2; 2,500 to 3,900 ft.

Table 3

Results of the Geotechnical Kinematic Analysis Based on a 30 psf Sediment Strength

Failure Surface	Extent of Initial Failure Mass Behind the Dam[1] (ft)	Time of Runout (s)	Initial X-Y Coordinates of Center of Gravity[2] (ft)	Final X-Coordinate of Center of Gravity[3] (ft)	Lateral Movement of Center of Gravity[4] (ft)	Estimated Toe Runout[5] (ft)
1	425	54.3	185, 305	-866	1,051	2,531
2	1,233	94.7	515, 309	-1,371	1,886	3,928
3	1,619	97.7	620, 311	-1,436	2,056	4,387
4	1,934	101.9	637, 311	-1,322	1,959	4,288

Notes:

[1]The extent of the initial failure mass behind the dam is the horizontal distance from the dam to the back of the failure mass.

[2]The initial center of gravity is listed as x and y coordinates, where the x-value is the distance upstream from the dam. The y-value is the elevation.

[3]The final x-coordinate of the center of gravity is the horizontal distance from the dam for the final resting place of the failure mass. Values downstream of the dam are negative and upstream are positive.

[4]The lateral movement of center of gravity is the horizontal movement from the initial center of gravity to the final center of gravity.

[5]The estimated toe runout is the distance of the farthest extent of sediment from the dam. It was estimated by interpolating between the failure stage geometries.

SPH Method – In the first phase of the SPH analysis, the dam is breached allowing the sediment to mobilize. As assumed in the geotechnical analysis, the failure of the dam is instantaneous. The total number of sediment particles coming out of the dam is 16,255, which is equivalent to 214,504 cubic yards of sediment.

In the second phase of the analysis, the runout of the released sediments downstream is estimated. The runout distance is estimated by tracking the motion of the particles as the sediment moves downstream

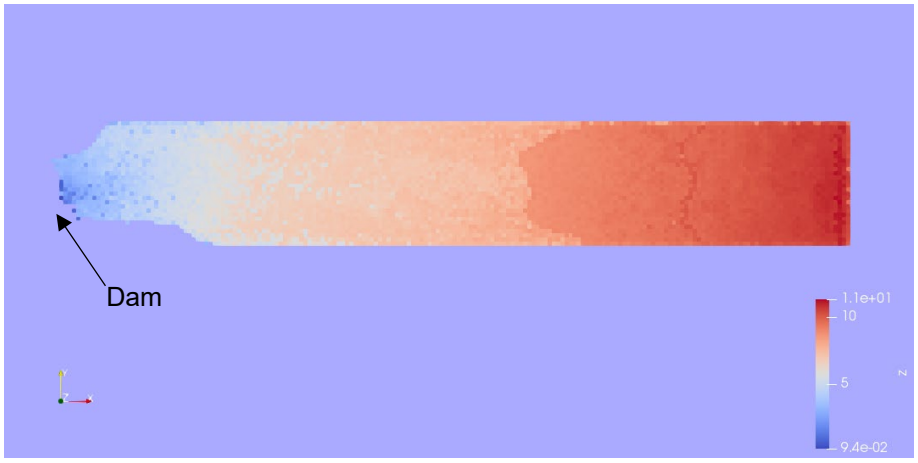
The density of the sediment is estimated to be 93 pcf. This value is inferred from the sediment coring data. Young's modulus of elasticity and Poisson's ratio are 20 MPa and 0.4, respectively, which are consistent with what Fávero Neto and Borja (2018) used in their SPH simulation of granular flow. These parameters are not particularly critical in the simulation because the overall behavior of the sediment is dominated by plastic deformation.

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Figure 4 shows the final profile of the sediment in the reservoir at the end of the simulation. Near the dam (see Figure 4a), the top surface of the sediment drops by 34 ft, leaving a very thin layer of sediment at the bottom of the reservoir. The thickness of this layer grows with distance from the dam into the reservoir. At a distance of nearly 1,000 ft from the dam, the top surface of the sediment drops by nearly 14 ft, and beyond a distance of 1,247 ft, the drop in the surface elevation is approximately zero. From this figure, beyond a distance of about 1,300 feet, the top surface elevation of the sediment remains essentially unchanged.



(a)



(b)

Figure 4. Contours of final free surface elevation for estimated in the SPH analysis: (a) water-sediment mixture (top), and (b) solid sediment.

The model for the creek consists of 126,270 boundary particles. The particle size is chosen to be 3 m, and the number of sediment particles is 6,160.

The flow of sediment in the creek downstream is carried out until the velocity of all particles is zero and no more movement is detected. The runout distance is calculated from the final position of the tip particle relative to the dam. For an undrained shear strength of 30 psf, the runout distance is calculated to be 5,154 ft.

FLO-2D Mudflow Analysis - As summarized in Section III the analysis of the sediment runout was performed using FLO-2D (FLO-2D Software, Inc. 2009). Unlike the geotechnical kinematic analysis and the SPH analysis, the FLO-2D simulations model in detail the geometry of the channel downstream of the dam. The FLO-2D model reach is about 5 miles long including the creek channel and overbank areas.

In the FLO-2D simulations of the sediment runout the dam breach was modeled as an instantaneous failure of the entire dam, similar to the geotechnical kinematic analysis. The estimated mudflow peak discharge from this dam breach 9,000 cfs. The runout distance for the toe of the sediment is approximately 6,300 ft.

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Sediment Results Summary - Three methods were used to estimate the runout of impounded sediments. The initial application of the three methods used the same yield strength for the sediments. In addition, all of the methods assumed the entire dam breached instantaneously. However, the methods differed in terms of how the downstream channel was modeled. In the geotechnical kinematic analysis, the only feature of the downstream channel that was considered was the slope. Other hydraulic characteristics such as the channel path (turns), cross-section and roughness (Manning's n) were not modeled. In the SPH analysis the slope of the downstream channel was modeled. The channel was assumed to be straight with varying cross-sections and the channel roughness was not modeled. In the FLO-2D analysis the downstream channel was modeled in detail based on available topographic data to define the channel cross-sections and hydraulic characteristics.

Table 4 summarizes the results of the three methods in terms of the runout distance of the toe of the sediment. The table reports the results for the base case in which the three methods used the same yield strength for the sediment and the results of the sensitivity calculations where the yield strength was varied. Given the differences in the methods, the results are similar and support a conclusion that the runout of sediments is limited and in this case sediment flows do not approach populated locations.

Table 4

Summary of the Sediment Runout Analysis Results

No.	Method	Sediment Toe Runout Distance[1] (ft) Base Case / Sensitivity Case
1	Geotechnical Kinematic Analysis	4,387 / 7,104[2]
2	Smoothed Particle Hydrodynamics Method	5,154 / 7,152[2]
3	FLO-2D Mudflow Analysis	~5,800 / Over 20,000[3]

[1]Distance from the dam

[2]The sensitivity case assumed a remolded shear strength of 15 psf.

[3]This result is based on a yield strength that is more than an order of magnitude less than the base case of shear strength of 30 psf. The mudflows continue beyond this point at very low rates; flows remain in the channel.

D. Sediment Laden Water Analysis

In this section the release of sediment laden water is considered. The analysis was performed using HEC-RAS. Hyperconcentrated flow (or mud flood) typically has a sediment concentration between 20-40% by volume. Fluid properties and sediment-transport characteristics change for hyperconcentrated flow as large volumes of sediment can be transported throughout the water column, and the mixture no longer behaves strictly as a Newtonian fluid. Standard Newtonian fluid based hydraulic and sediment transport models, such as HEC-RAS, are often applied for hyperconcentrated flows (WEST Consultants, 2011). However, it has been demonstrated that such an approach may produce erroneous results, particularly for unsteady flow modeling (Travis et al. 2012). A specialized model that accounts for the water/sediment mixture rheological properties, such as FLO-2D, is a preferred tool for simulating hyperconcentrated flows.

Mudflows and debris flows typically have a sediment concentration between 40-55% by volume, although some researchers use up to 65% as the upper limit. Mudflows contain high concentrations of silt and clay, while debris flows contain a high coarse material content including boulder and woody debris. The sediment/water mixture is a slurry not unlike wet concrete. As described by O'Brien (2006), during typical mudflow events, clear-water flows arrive first from the drainage basin, followed by a surge or "frontal wave" of mud and debris. Mudflows and debris flows acts as a non-Newtonian fluid, and basic hydraulic and sediment transport equations do not apply. If detailed modeling of such environments is required, a model with specific debris/mud flow capabilities, such as FLO-2D, should be used rather than standard hydraulic models, such as HEC-RAS.

Water/sediment mixtures with more than 55% sediment concentration by volume are generally considered landslides rather than debris flows.

HEC-RAS was used to route the free water dam breach hydrograph. The free water hydrograph was increased ("bulked") to account for the presence of sediments in the flowing water. To provide conservative results, the maximum possible bulking factor for normal streamflows was used in the present analysis (about 20% of sediment by volume in the water-sediment mixture). The sediment-laden breach flows are conveyed within the downstream channel and thus do not spill into populated floodplain areas.

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E. Downstream Inundation

The results of the sediment runout analysis and the sediment-laden water analysis provide insight into the downstream flood hazard posed by a dam breach. When a breach occurs the free water and the impounded sediment will be released. Based on the FLO-2D calculations the sediment will move much slower than the sediment laden water. For example, the toe of the sediment runout will take 30 minutes to travel 3,500 ft downstream of the dam. The sediment laden water will take approximately 12 minutes to travel this same distance. Within 24 minutes the peak of the sediment laden water will have moved past this point.

Of the various analyses performed to estimate the hazard from a dam breach, accounting for the sediment in the reservoir, a single scenario must be selected for inundation mapping purposes. Based on the results of the sediment runout analyses that were performed, the sediments will travel a relatively short distance from the dam (less than 7,000 feet). As a result, for approximately 7,000 feet (or less) below the dam, the creek channel will have a substantial amount of sediment (mud) deposited in it.

In addition to the release of the sediment, sediment laden water will also be released at the time of a breach. Calculations show that sediment laden water will move downstream faster than the released sediments (which represents the largest volume of material that will be released). Once the sediment laden water moves more than about 3,500 feet downstream, these flows will have moved ahead of the released sediment and continue down the creek channel. For this reason, the results of the sediment runout analysis as computed by FLO-2D based on the 30 psf sediment strength model results and the sediment laden water analysis (HEC-RAS analysis) was used for flood mapping purposes.

V. SUMMARY

In the dam safety profession, it is not a common practice to model the release of impounded sediment from water storage reservoirs as part of dam breach and inundation studies. Current dam safety guidelines on inundation mapping do not mention dam breach modeling and inundation mapping associated with impounded sediments. Typically, accumulated sediment is assumed to be all water, which is conservative. In the case of the dam-reservoir considered in this study, given the current storage volume of the reservoir, such an assumption is conservative, but also unrealistic.

To account for the release of impounded sediments as a result of a hypothetical breach of the dam, a series of alternative methods were used to evaluate the inundation that would occur when the water and sediment are released. The analyses that were performed considered the release of the water only as an upper-bound estimate of the possible inundation, the release of the impounded sediments (mudflow), and the release of sediment laden water. To model the runout of impounded sediments, three alternative methods were used to account for uncertainty in these analyses.

The results of the different sediment runout analyses that were performed show that following a breach of the dam, the sediments travel a relatively short distance downstream; less than 7,000 feet. At the same time that sediments are released, the free water in the reservoir will also be released. As the free water in the reservoir mixes with the sediments, a sediment laden water mix will also flow downstream. Calculations for the release of the sediment laden water show these materials will move downstream much faster than the released sediments (which represents the largest volume of material that will be released). Once the sediment laden water moves more than approximately 3,500 feet downstream, these flows will have moved ahead of the released sediment and continue down the creek channel. The flowrate of the sediment laden water remains in the creek channel and as a result inundation of populated areas does not occur.

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VII. AUTHOR BIOGRAPHIES

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

A Sensitivity Analysis Involving Simplified Approaches for Emergency Action Plan Development in Urbanized Settings

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ABSTRACT

There has been an on-going effort to increase the compliance rate of dam owners with High and Significant hazard potential dams in New Mexico to have Emergency Action Plans. A streamlined approach with acceptable, conservative assumptions has been developed utilizing an automated spreadsheet to develop simplified dam breach hydrographs, performing hydraulic routing in DSS-WISE, conducting a population at risk (PAR) analysis in HCOM, and automatically produced inundation maps with a system of programmed tools. All tools create a standardized methodology with a consistent result and mapping format. These tools work well in rural area. However, what about dams in urbanized areas with complex hydraulics and flow bifurcations? Can DSS-WISE create comparable breach inundation mapping extents in urban areas compared to traditional, detailed hydraulic models such as HEC-RAS 2D? A pilot study was implemented in coordination with the New Mexico Office of the State Engineer – Dam Safety Bureau, to compare multiple approaches for two high hazard dams (Arroyo del Oso Dam and John Robert Dam) within the City of Albuquerque, New Mexico. Both dams were assessed by comparing a simplified top-of-dam breach hydrograph versus the traditional, empirically derived breach parameters generating a dam breach hydrograph. Each breach hydrograph approach was compared to two HEC-RAS 2D approaches involving either raising the terrain within each structure footprint downstream or overwriting building footprints within the land cover layer with high Manning's n-values, both with enforced computation mesh around each structure's perimeter. Simulation results also tested to see if dam breach inundation extents varied by considering downstream bridges and structures fully open or fully closed by debris. The standard settings of DSS-WISE were also utilized for a comparison to all HEC-RAS 2D results. Results indicate that within the urbanized setting of Albuquerque, New Mexico, the selected HEC-RAS 2D model created a much more realistic propagation of the breach hydrograph and detailed inundation extents relative to DSS-WISE which includes default, course internally programmed terrain. Further analysis also indicated negligible differences by either assuming downstream structures are fully open or fully clogged or if downstream structures are depicted by raised terrain or elevated Manning's values within each building footprint. These results indicate urbanized settings may be simplified further in hydraulic routing model development for the creation of dam breach inundation mapping for Emergency Action Planning and potentially pass on additional savings to the dam owner.

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DAM SAFETY PALM SPRINGS
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PAR is not Risk. Why Proxies Don't Work.

PAUL RISHER, US ARMY CORPS OF ENGINEERS; AND GREG PAXSON, P.E., D.WRE, SCHNABEL ENGINEERING

ABSTRACT

As the dam and levee safety community moves toward the application of risk informed decision making (RIDM), we should be sure to appropriately apply the principles and tools. It's important to be aware of two issues that often come up when trying to simplify the effort: 1) Hazard classification is not the same as life safety risk2) Population at risk (PAR) is not the same as potential life loss.To make risk-informed decisions, we need more resolution in our life loss estimates than what can be inferred from hazard classification or PAR. The tools to estimate potential life loss from flooding continue to improve, making these estimates more reliable. These tools also provide a means to scale the effort of estimating life loss to the decisions they inform.This presentation will include data collected by the US Army Corps of Engineers related to their dam and levee portfolios to demonstrate why PAR is not well correlated to life safety risk or life loss consequences. In addition, specific examples will be used to illustrate the differences between PAR, life safety risk, and life loss consequences. Existing and new tools for estimating life loss will be discussed.

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DAM SAFETY PALM SPRINGS
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Updates to USACE Drilling Guidance ER 1110-1-1807

THOMAS TERRY, P.E., P.G., AND APRIL FONTAINE, US ARMY CORPS OF ENGINEERS

ABSTRACT

There are many examples of poor investigation techniques causing damage to dams and levees. Care must be taken when approaching this type of work to minimize the risk of hydraulic fracture of this critical infrastructure. USACE ER 1110-1-1807 is a critical document for planning safe investigations near dams and levees. This ER went through a major overhaul and was most recently published in 2014, but many lessons learned since then have resulted in another update this year. Some of the most critical updates include: • Establishes a minimum FS for hydraulic fracture of 1.3. Previous version was silent. • Requires documentation of two methods to calculate that FS. There are several potential equations that can be used, but none are ideal. Previous version was silent on method and did not require calculation submissions. • Applied to not just embankments, but all dams and levees. • Changed Drilling Program Plan to Drilling and Invasive Program Plan since this ER also applies to test pits and other non-drilling activities. • Added a license requirement for preparer of DIPPs (not previously required). • Clarified experience requirements for drillers. • Clarified the construction exemption – for drilling that has undergone ATR/SAR. • Clearly stated that approval of a DPP (now DIPP) is a federal action that triggers NEPA. This presentation will summarize the changes and rationale behind them.

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DAM SAFETY PALM SPRINGS
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Liquefaction Assessment in Gravels using Vs and DPT Blow count from Case Histories

KYLE ROLLINS, PROFESSOR, BRIGHAM YOUNG UNIVERSITY AND JASHOD ROY, GEOTECHNICAL ENGINEER, KIEWIT ENGINEERING

ABSTRACT

Gravelly soils have liquefied at multiple sites in at least 27 earthquakes over the past 130 years. These gravels typically contain more than 25% sand which lowers the permeability and makes them susceptible to liquefaction. Typical Standard Penetration Tests (SPT)- or Cone Penetration Test (CPT_-based correlations can be affected by large gravel particles and lead to erroneous results. To deal with these problems, we have developed liquefaction triggering curves for gravelly soils based on (1) shear wave velocity (V_s) and (2) a 74-mm diameter dynamic cone penetrometer (DPT), less affected by gravel particles. These correlations are based on case histories where gravelly soils did and did not liquefy in past earthquakes. The new probabilistic triggering curves reduce the range between 15 and 85% probability of liquefaction in comparison to previously developed curves. The V_s-based liquefaction triggering curves for gravels shift to the right relative to similar curves based on sands. Good agreement has been obtained with liquefaction factor of safety computed by the DPT and Becker penetration tests (BPT) at sites in Idaho subjected to the 1985 M_w 6.9 Borah Peak earthquake.

I. INTRODUCTION

Liquefaction of loose saturated granular soils results in significant damage to civil infrastructure such as dams, bridges, roadways, pipelines, and ports in nearly every earthquake. Liquefaction and the resulting loss of shear strength can lead to landslides, lateral spreading, loss of vertical and lateral bearing support for foundations, and excessive foundation settlement and rotation. Direct and indirect economic losses resulting from liquefaction are substantial costs to society. A significant number of gravel liquefaction case histories have occurred during more than 27 earthquake events over the past 130 years. Assessing the potential for liquefaction of gravelly soils in a reliable, cost-effective manner has always posed a great challenge for geotechnical engineers. Liquefaction assessment is particularly important for older dams that were constructed on gravelly soil foundations or with poorly compacted gravel shells before the potential for liquefaction in gravels was recognized. Likewise, many ports around the world were constructed of gravelly soils or rockfill which was believed to be immune to liquefaction. For these projects, assessing the potential for liquefaction and determining appropriate remedial measures can become multi-million-dollar decisions. These decisions involve both life-safety and regional economic issues. Over the past 15 years, gravel liquefaction has caused significant damage to ports in Greece, Chile, Ecuador, and New Zealand. In addition to these large projects, gravel liquefaction must be routinely considered for a myriad of small to medium projects throughout the world.

Early numerical modeling by Seed et al. [1] found that excess pore pressure generated by an earthquake would dissipate as fast as they generated if the hydraulic conductivity of the gravel was greater than about 0.3 centimeters per second (cm/s). This suggested that clean gravels would be unlikely to liquefy. The range of grain size distribution curves for gravels that have liquefied in field case histories is plotted in Fig.1 [2]. Although gravel contents often

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exceed 50%, they are typically well-graded sandy gravels with more than 30% sand. Roy and Rollins [2] found that these sandy gravels typically had hydraulic conductivities less than 0. 3 cm/s because the sand filling the void space between the gravel particles was reducing the hydraulic conductivity of the mixture. This allows excess pore pressures to generate during earthquake shaking.

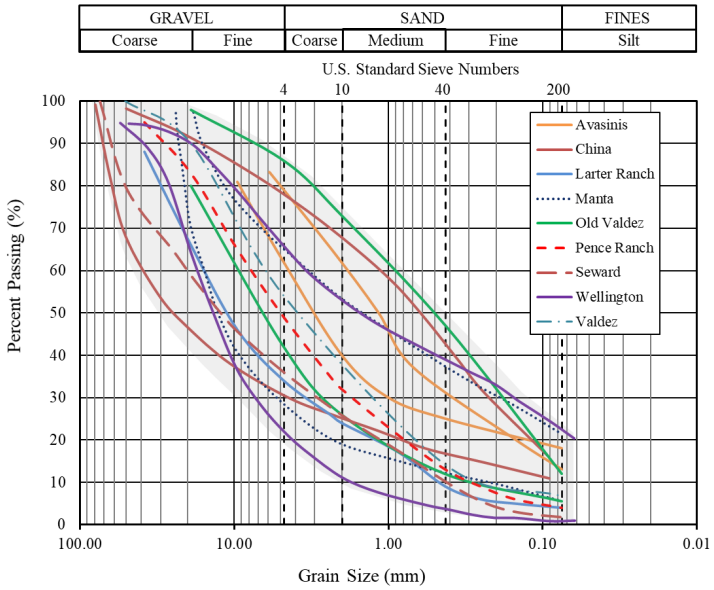


Figure 1. Grain-size distribution curves for gravelly soils that have liquefied in field case histories [2].

II. PENTRATION TEST METHODS

Typical laboratory investigation techniques have generally proven to be ineffective for characterizing gravelly soil due to the cost and difficulty of extracting undisturbed sample from gravelly deposits using freezing techniques [3]. In addition, the large particle size of gravels can lead to artificially high penetration resistance values from traditional in situ tests such as the cone penetrometer (CPT) test and the Standard Penetration (SPT) test [4]. The 168 millimeter (mm) diiameter Becker Penetration Test (BPT) [5, 6] reduces the potential for artificially high penetration values; however, this method is relatively expensive and is not available outside of North America. In addition, the method requires a correlation between the BPT blow count and the SPT blow count which leads to greater uncertainty relative to methods that are directly correlated with field liquefaction resistance.

As another approach for gravelly soils, Chinese engineers in the Chengdu region, faced with widespread gravel deposits, developed a Dynamic Cone Penetrometer (DPT) with a 74-millimeter (mm) or 3 inch diameter cone tip for site characterization. The methodology is a large-size implementation of the lightweight dynamic cone penetrometer that is used extensively for assessment of compaction of soils in pavement applications [7] and different cone geometries are also known as dynamic probing in Europe [8]. In the Chinese version of the DPT, the cone tip is driven continuously with a 120 kilogram (kg) hammer dropped from one meter (equivalent to 340 pound weight dropped from 30 inches) and is capable of penetrating medium to dense gravel and cobbles. DPT soundings can be easily performed with conventional SPT drilling rigs or even simple tripod systems, making it viable worldwide. In contrast to the straight sides of the BPT, the cone tip tapers back to a 60 mm drill rod to reduce rod friction. At 74 mm, the DPT diameter is 50% larger than the SPT and 110% larger than a standard 10 cm^2 CPT; however, it is still 55% smaller than the BPT. Although the BPT provides the largest diameter to particle size ratio of all tests, the DPT is superior to the SPT or CPT and could be a reasonable solution in many cases depending on the gravel size and percentage.

Based on field case histories of gravel liquefaction in the M_w 7.9 Wenchuan earthquake, Cao et al. [3] developed probabilistic liquefaction triggering curves for gravels based on the DPT blow count. However, these curves are

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based on relatively few data points from one earthquake and a geologic environment [3]. Because of the limited number of data points and the possibility of false negatives (sites where liquefaction may have occurred but did not produce surface manifestation), the individual triggering curves (85 to 15% probability of liquefaction) are spread over a large range. In contrast, more mature probabilistic liquefaction triggering curves for sands based on the CPT [9] have more closely grouped probability curves because of the larger size of the data set. In addition, the Cao et al. [3] triggering curves were developed for a single event of *Mw* 7.9 without incorporating any correction to the seismic demand by using the Magnitude Scaling Factor (*MSF*). Thus, applicability of these curves would become questionable for evaluating the liquefaction potential of gravelly soils for other seismic events of different Magnitude. Although existing *MSF* models developed for sand liquefaction [10] can be used, it is unclear whether they are appropriate for gravel liquefaction using the DPT.

Therefore, it becomes crucial to add more case histories to the DPT database with different earthquake magnitudes and geologic settings to develop an improved DPT-based liquefaction triggering procedure and a gravel-based MSF curve.

III. SHEAR WAVE VELOCITY METOHDS

As an alternative to penetration resistance testing, in-situ measurement of shear wave velocity (V_s) is a popular way of characterizing the liquefaction resistance of soil deposits. V_s is a basic mechanical property of soil materials, directly related to the small strain shear modulus (G_0), that has been used to evaluate liquefaction resistance in sand. The use of V_s as a field index of liquefaction resistance is soundly based on the fact that both V_s and liquefaction resistance are similarly, but not proportionally, influenced by void ratio, effective confining stresses, stress history and geologic age [10]. In addition, V_s is considerably less sensitive to the problems of soil compression and reduced penetration resistance when fines are present, compared with SPT and CPT methods. Moreover, V_s requires only minor corrections for fines content (*FC*) at least for sands [11]. The primary advantage of the in-situ V_s approach is that testing can be performed at sites where borings are not possible, or the penetration test results may be unreliable. Hence, V_s measurement can be considered as a reliable and economical alternative to overcome the difficulties of penetration testing within gravelly strata.

The traditional methods of measuring V_s requires a penetrometer or an instrumented borehole to measure the travel time of shear waves at various depths. A downhole test requires one borehole to measure the vertically propagating wave, while a cross-hole test requires at least two boreholes to directly measure the horizontally propagating wave [12]. These invasive test methods are usually quite expensive owing to the cost of drilling, casing, and grouting boreholes. In the last two decades, some advanced non-invasive test methods (Spectral Analysis of Surface Waves (SASW) and Multichannel Analysis of Surface Wave (MASW) have been developed, which indirectly define the V_s through the surface wave dispersion characteristics of the ground [12, 13, 14]. These non-invasive test methods have significantly reduced the cost of in-situ V_s estimation and made soil exploration possible at sites where penetration is not possible or economically feasible.

Although the Andrus and Stokoe [13] V_s-based liquefaction triggering curves for sand contained some gravel case histories, no liquefaction triggering curves based exclusively on gravel performance were available until recently [15]. SPT-V_s correlations developed by Ohta and Goto [16] for sands and similar correlations by Rollins et al. [17] for gravelly soils suggest that gravel profiles with higher shear wave velocities might still be susceptible to liquefaction than would be predicted by triggering curves based on sand.

After collecting a database of gravel case histories from the 2008 Wenchuan earthquake, Cao et al. [18] developed probabilistic liquefaction triggering curves for gravels based on Vs using logistic regression techniques However, these curves were based on only 47 data points (19 liquefaction and 28 no liquefaction points) that refer to a single earthquake and a similar geological environment [18]. Because of the limited number of data points and the possibility of false negatives (sites where liquefaction may have occurred but did not produce surface manifestation), the individual triggering curves (15% to 85%) widely separated as was the case for the DPT case histories. In contrast, V_s-based probabilistic liquefaction triggering curves for sands [11] have more closely grouped probability curves because of the larger size of the data set. Moreover, the Cao et al. [18] triggering curves were developed for a single event of M_w 7.9 without proposing any correction to the seismic demand for different earthquake magnitudes. Thus, it becomes difficult to use these curves for other magnitude events without using a Magnitude Scaling Factor developed for sand liquefaction [10]. Therefore, additional effort was necessary to collect

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more V_s data from the gravel liquefaction sites to improve the existing V_s-based liquefaction triggering curves for gravelly soils.

IV. COLLECTION OF ADDITIONAL FIELD CASE HISTORY DATA

In the present study, a larger database consisting of 174 V_s data points and 137 DPT data points has been compiled by collecting additional data points from seven different countries around the world where gravel liquefaction did or did not take place in 17 major earthquake events and adding them to the existing data points from China [3, 19]. Case histories with no liquefaction in Italy, Greece, and New Zealand were strategically identified, tested, and then added to the database to help constrain the position of the liquefaction triggering curves. For each case history, the cyclic stress ratio (CSR) has been obtained by using the simplified equation,

$$CSR = 0.65\ (a_{max}/g)\ (\sigma_{vo}/\ \sigma'_{vo})\ r_d \quad (1)$$

originally developed by Seed and Idriss [20] where a_{max} is the peak ground acceleration, σ_{vo} is the initial vertical total stress, σ'_{vo} is the initial vertical effective stress, and the r_d value in Eq. 1 was updated to include the effect of both depth and earthquake magnitude using the equation,

$$r_d = e^{[\alpha(z)+\beta(z)M_w]} \quad (2)$$

where:

$$\alpha(z) = -1.012 - 1.126sin(^{z}/_{11.73} + 5.133) \quad (3)$$

$$\beta(z) = 0.106 + 0.118sin(^{z}/_{11.28} + 5.142) \quad (4)$$

and z is the depth in meters, based on the work by Golezorkhi [21] and Idriss [22].

Peak ground accelerations *(PGA or a_{max})* for every location were taken from the literature or from USGS Shake Maps [23] where necessary as employed by Idriss and Boulanger [24] for their CPT database. Besides *CSR*, the moment magnitude (M_w) has been considered as another independent seismic variable for obtaining the liquefaction potential of gravelly soils. Values of M_w were found from available references regarding the appropriate earthquake. The data set contains a wide distribution of M_w ranging from 5.3 to 9.2 as well as *PGA* ranging from 0.17 to 0.6 g.

A. DPT Blow Count Corrections

The DPT blow count, N_{120}, represents the number of hammer blows to drive the penetrometer 30 cm deep with a 120 kg hammer dropped from a height of 1 m. Raw blow counts are typically reported at every 10 cm penetration but are multiplied by three to get the equivalent N_{120} for 30 cm of penetration. Based on 1200 hammer energy measurements, Cao et al. [25] found that the Chinese DPT provided an average of 89% of the theoretical free-fall energy. Since the energy delivered by a given hammer (E_{Hammer}) was different than the energy actually supplied by a Chinese DPT hammer ($E_{Chinese\ DPT}$), it was sometimes necessary to correct the measured blow count. In this study, the correction was made using the simple linear adjustment ratio suggested by Seed et al. [26] for SPT testing

$$N_{120} = N_{Hammer}\left(\frac{E_{Hammer}}{E_{Chinese\ DPT}}\right) \quad (5)$$

where N_{Hammer} is the number of blows per 0.3 m of penetration obtained with a hammer delivering an energy of E_{Hammer}. In addition, Cao et al. [1] recommend an overburden correction factor, C_n, to obtain the normalized N'_{120} value using the equation

$$N'_{120} = N_{120}C_n \quad (6)$$

Where,

$$C_n = \left(\frac{100}{\sigma'_{v0}}\right)^{0.5} \leq 1.7 \quad (7)$$

and σ'_o is the initial vertical effective stress in kN/m^2. In the current study, a limiting value of 1.7 was added to be consistent with the C_n used to correct penetration resistance from other in-situ tests [10]. For each case history a critical layer was selected below the water table and with the lowest ratio of blow count divided by CSR over at least one meter. All the critical liquefaction layers were located at depths less than about 14 m which is consistent with other liquefaction case history databases and with blow counts typically less than about 20.

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B. Shear Wave Velocity Corrections

The V_s values obtained by various in-situ methods were corrected for overburden pressure to obtain V_{s1} using the equation:

$$V_{S1} = V_S \left(\frac{P_a}{\sigma'_{v0}}\right)^{0.25} \tag{8}$$

where σ'_{vo} is the initial vertical effective stress, and P_a is atmospheric pressure approximated by a value of 100 kPa as suggested by Sykora [26] and adopted by Youd et al. [10]. These normalized V_{s1} profiles based on the V_s testing were then plotted as a function of depth and a critical depth with the lowest average ratio of V_{s1}/CSR over a length of at least one meter. Once again, the critical layers based on V_{s1} were all shallower than 14 m.

V. DEVELOPMENT OF PROBABILISTIC TRIGGERING CURVES

Using the expanded database, a new set of probabilistic liquefaction triggering curves has been developed by logistic regression analysis based on V_{s1} and on N'_{120}. The triggering equations developed in the present study include the earthquake magnitude as an independent variable.

A. Probabilistic DPT-Based Liquefaction Triggering Curves

The logistical regression analysis was carried out using M_w, N'_{120}, and Ln(*CSR*) as independent variables and the following equation was developed to calculated the Cyclic Resistance Ratio (CRR)

$$CRR = exp\left[\frac{1.32M_w - 0.0008{N'}_{120}^{3} - \ln\left(\frac{1-P_L}{P_L}\right)}{5.2}\right] \tag{9}$$

where P_L is the probability of liquefaction expressed as a fraction. If a given probability and M_w of 7.5 is used in Eq. 9, a plot of *CRR* vs. N'_{120} can be produced for a given probability. Fig. 2 provides a plot of *CRR* vs. N'_{120} for M_w 7.5 for various P_L values. CSR and N'_{120} data points for each liquefaction and no liquefaction case history are also shown in Fig. 2 relative to the triggering curves proposed by Rollins et al. [28].

B. Probabilistic V_s-Based Liquefaction Triggering Curves

Logistical regression analysis was also carried out using M_w, V_{s1} and Ln(*CSR*) as independent variables and the following equation was developed to calculate the Cyclic Resistance Ratio (CRR)

$$CRR = exp\left[\frac{1.438M_w - 3.8x10^{-7}{V_{s1}}^{3} - \ln\left(\frac{1-P_L}{P_L}\right)}{4.026}\right] \tag{10}$$

where P_L is the probability of liquefaction expressed as a fraction. If a given probability and M_w of 7.5 is used in Eq. 10, a plot of *CRR* vs. V_{s1} can be produced for a given probability. Fig. 3 provides a plot of *CRR* vs. N'_{120} for M_w 7.5 for various P_L values proposed by Rollins et al. [29].

The new probabilistic triggering curves with liquefaction probabilities of 15% to 85% are plotted in Fig. 4(a) with solid lines along with similar curves developed by Cao et al. [18] with dashed lines to draw a distinct comparison between the two triggering procedures. For lower values of V_{s1} (around 150 m/s), the *CRR* for the new 50% probability of liquefaction curve is about 0.10 while it is only 0.04 for the Cao et al. [18] curves. This adjustment produces much better agreement with observed field performance. This higher *CRR* value at small velocities are also more typical of that predicted by the V_s-based triggering curves developed by Kayen et al. [11]. Case history data points where gravels did not liquefy in Fig. 3 from the Port of Wellington and ports in Argostoli, Italy, have had a significant effect in constraining the lower branch of the triggering curves to move upwards. Likewise, the triggering curves at the higher range of V_{s1} values have been tightened relative to the curves developed by Cao et al. [3] owing to additional “no liquefaction” data points from Chengdu, L’Aquila, Italy and Valdez, Alaska. Additional data points would certainly be desirable to define the shape of the curve better in this range of V_{s1} values.

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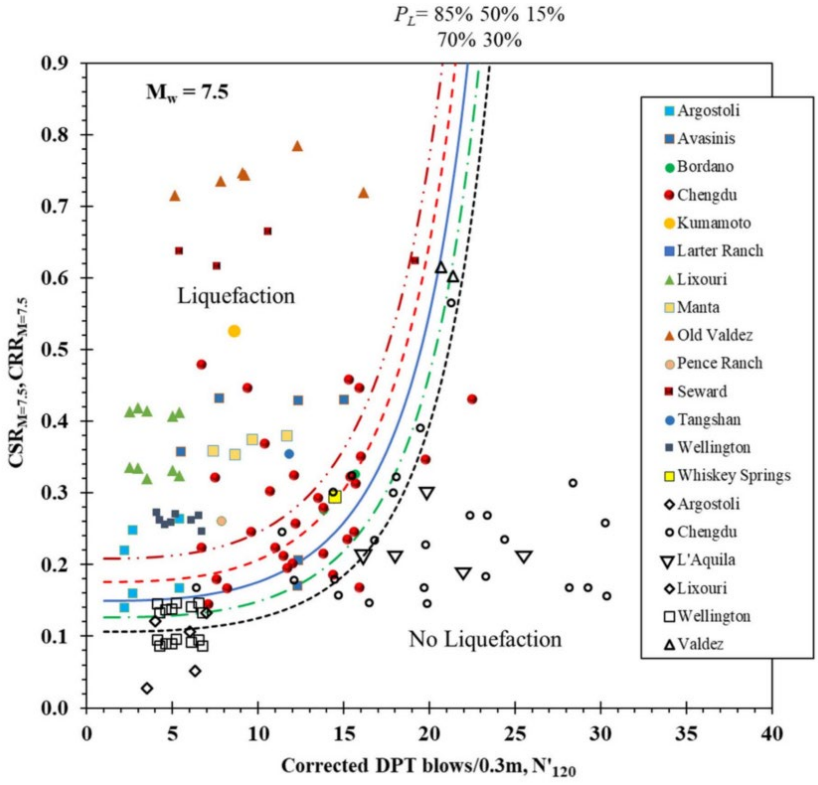


Figure 2. Plot of *CRR* vs. N'_{120} for a M_w 7.5 earthquake with various probabilities of liquefaction based on expanded DPT-based database proposed by Rollins et al. [28].

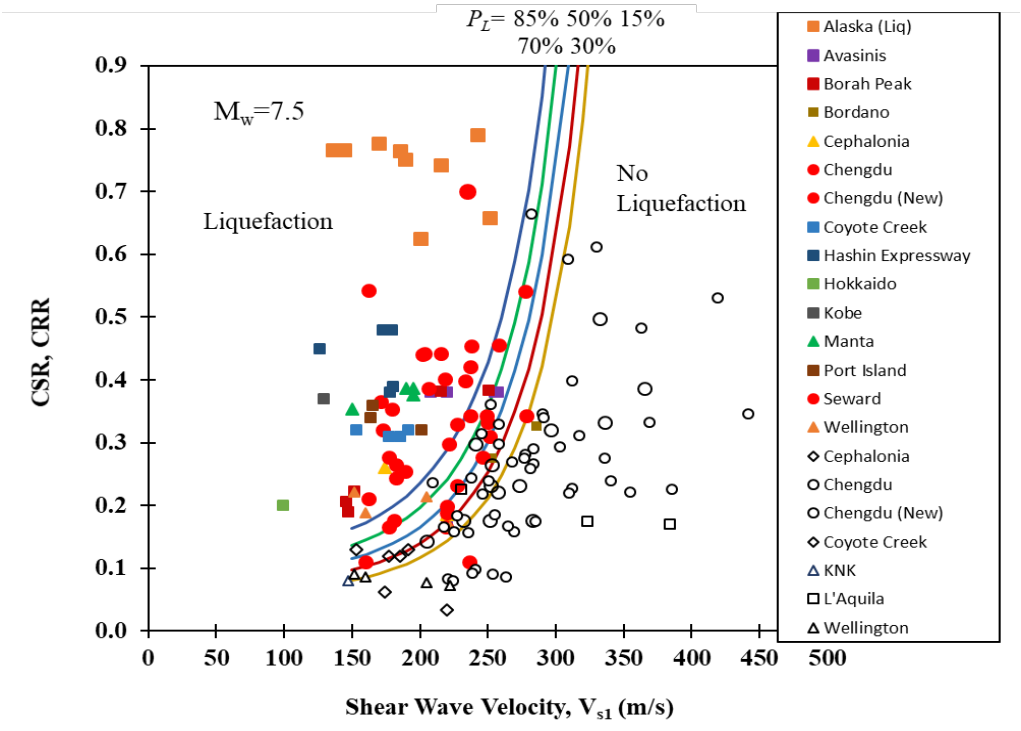
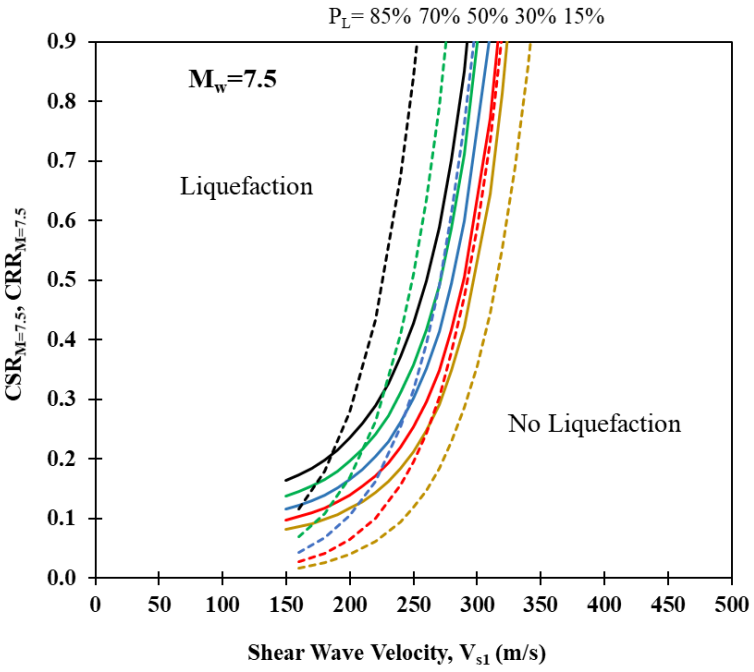
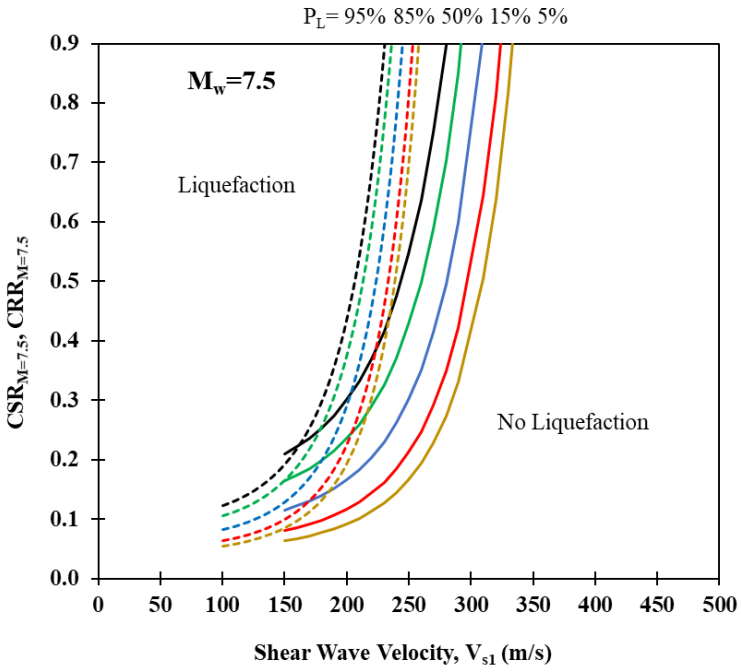


Figure 3. Plot of CRR vs. V_{s1} for a M_w 7.5 earthquake with various probabilities of liquefaction based on expanded Vs -based database collected by Rollins et al. [29].

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(a)



(b)

Figure 4. Revised liquefaction triggering curves from this study (solid lines) (a) relative to triggering curves originally proposed by Cao et al. [18] (dashed lines) and (b) relative to triggering curves proposed by Kayen et al. [11] for sands (dashed lines).

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As shown in Fig. 4(a), for V_{s1} values above 200 m/s, the P_L = 50% curve for the new regression is very similar to that for the Cao et al. [18] regression. However, the addition of new liquefaction points has pulled the new P_L=85% curve to the right while the addition of no-liquefaction data points has pulled the new P_L=15% curve to the left, relative to the Cao et al. [18] curves. Moving the new P_L=15% curve to the left is important because this curve is often recommended for deterministic evaluations [11]. However, the slope of the new set of curves from this study remains almost the same as for the Cao et al. [18] curves. Overall, the spread between the triggering curves for various probabilities of liquefaction is substantially reduced for the new triggering curves relative to the Cao et al. [18] curves. This result is consistent with the concept that the increased number of data points reduces the uncertainty that develops when an individual data point plots in an unexpected position. Furthermore, the addition of data points where liquefaction did not occur has helped constrain the triggering curves on the "no liquefaction side" in critical locations.

Fig. 4 (b) provides a comparison is provided between the newly developed triggering curves for gravel and the curves developed by Kayen et al. [11] for sand. To plot the triggering curves for Kayen et al. [11], an average effective vertical stress of 100 kPa, and fines content of 6% has been assumed to keep the values within a reasonable range. Although the probabilistic liquefaction triggering curves for gravel developed in this study are similar to those for sands [11] at lower V_{s1} values typical of looser gravels, the curves diverge as V_{s1} increases. For example, V_{s1} equals 275 m/s for the proposed P_L = 50% curve for gravel in this study at a CRR of 0.5 in comparison with a V_{s1} of only 225 m/s for the P_L = 50% curve for sand proposed by Kayen et al. [11]. This indicates that the probabilistic triggering curves for gravels from this study shift to the right relative to similar curves developed for sands as V_{s1} increases. This result indicates that gravels can still liquefy at V_{s1} values that would be high enough to preclude liquefaction in a sand. This does not mean that gravels are more or less likely to liquefy than sand, it simply means that for a comparable level of shaking, a higher V_{s1} is necessary to obtain the same probability of liquefaction for a sandy gravel as that for a sand. This result is consistent with liquefaction case histories in gravels reported by several investigators [15, 18, 30, 31] where V_s-based triggering curves for sands would have incorrectly predicted no liquefaction. Similar results have also been observed in laboratory testing (e.g. [32, 33]).

VI. DEVELOPMENT OF MAGNITUDE SCALING FACTORS

Most liquefaction triggering curves adjust the *CSR* for earthquake magnitude using a Magnitude Scaling Factor (MSF) to obtain an equivalent *CSR* for a M_w of 7.5 using the equation,

$$CSR_{M=7.5} = CSR/MSF \tag{11}$$

As a part of the present study, we have developed new *MSF* models specifically for gravelly soils that will simplify liquefaction evaluation at gravel sites for different magnitude earthquakes; however, more data from other earthquakes would be desirable to improve the data set. The *MSF* for triggering analyses using the DPT blow counts can be computed as a function of magnitude with the best-fit exponential equation,

$$MSF = 7.258exp(-0.264M_w) \tag{12}$$

Likewise, the *MSF* for triggering analyses using V_{s1} can be computed as a function of magnitude with the best-fit exponential equation.

$$MSF = 10.667exp(-0.316M_w) \tag{13}$$

These *MSF* curves are plotted and compared with several other *MSF* vs. M_w curves in Fig. 5. It can be observed that the *MSF* curves developed for gravelly soil fall about mid-way between the *MSF* vs. M_w curves for sand suggested by Idriss and endorsed by the NCEER/NSF liquefaction workshop [10] at the high end and the Kayen et al. [11] curve at the low end. Hence, the proposed *MSF* curves for gravel appear to be reasonably consistent with existing *MSF* curves for sands.

Based on these *MSF* equations, the *CSRs* for all the case history data points have been converted to CSRs at M_w = 7.5 and plotted with the newly developed triggering curves as shown in Figs. 2 and 3. Generally, the data points fall on the correct sides of the P_L = 50% curves for liquefaction and no liquefaction.

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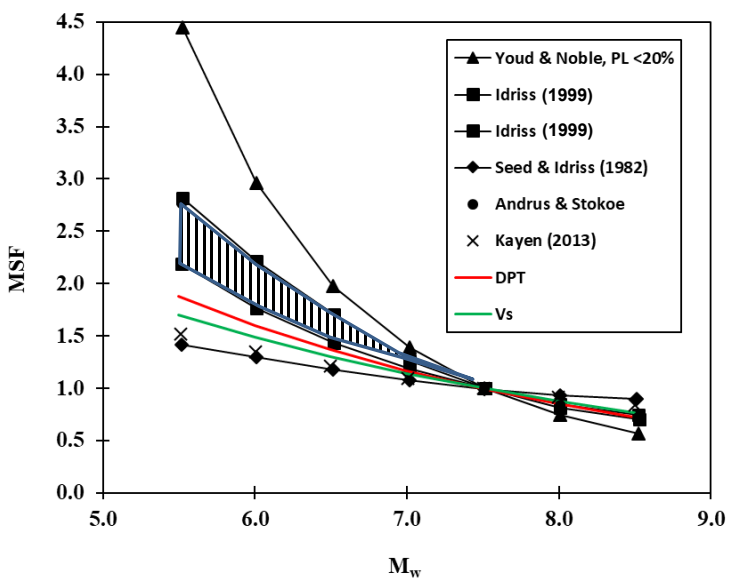


Figure 5. Comparison of *MSF* curves from logistical regression analysis of gravel liquefaction case histories based on *Vs* and DPT triggering curve with *MSF* curves proposed previously for sand [28, 29].

VII. COMPARISION OF BPT AND DPT LIQUEFACTION FACTOR OF SAFETY

To provide a comparison between the liquefaction factor of safety computed by the Becker Penetrometer (BPT) and the Dynamic Cone Penetrometer (DPT) in the field, we performed DPT test holes within a few meters of locations where BPT tests were previously performed [14,34] at liquefaction case history sites in Idaho subjected to the M_w 6.9 1985 Borah Peak Earthquake. Four DPT tests were performed at the Larter Ranch and Pence Ranch sites, while two were performed at the Whiskey Springs site. Peak ground accelerations at these sites were between 0.4 and 0.5 g. The critical liquefaction layers for the DPT were very similar to those identified by the BPT holes. For the critical layers in each hole at each site, the average Cyclic Stress Ratio (CSR) was computed using Eqs. 1 and 11. The Cyclic Resistance Ratio (CRR) was computed for the DPT holes using Eq. 9. However, the CRR for the BPT holes was based on the equivalent SPT N_{60} value reported by Andrus [14, 34] that was obtained using a correlation between equivalent sand SPT N_{60} value and the BPT N_{BC} developed by Harder and Seed [5]. With the equivalent sand SPT N_{60}, the $(N_1)_{60}$ was determined and the CRR calculated using the Idriss and Boulanger [24] approach. Table 1 provides a summary of the average CSR, CRR and factor of safety against liquefaction (FS_L) for the sites at Larter Ranch, Pence Ranch, and Whiskey Springs obtained from the companion DPT and BPT holes. Although there is some variation in the CRR as would be expected for natural soil deposits, the average percent difference in the FS_L from the DPT and BPT methods for the three sites range from only 4% to 17% with an overage average of 11%. The worst agreement was for the Whiskey Springs site where only two holes were available for comparison. Considering that both methods were developed from completely different data set with different methods for normalizing blow counts, this close agreement provides confirmation of the validity of both methods.

TABLE 1

Comparison of DPT- and BPT-based Liquefaction Assessment at 1985 Borah Peak Gravel Liquefaction Sites

Summary of DPT Testing				Summary of BPT Testing				
Site	Avg. CSR	Avg. CRR	Avg. FS_L	Site	Avg. CSR	Avg. CRR	Avg. FS_L	% Difference in BPT and DPT FS_L
Larter Ranch	0.26	0.11	0.42	Larter Ranch	0.26	0.10	0.38	10
Pence Ranch	0.28	0.12	0.43	Pence Ranch	0.28	0.115	0.41	4
Whiskey Springs	0.28	0.175	0.63	Whiskey Springs	0.28	0.15	0.54	17
Average			0.49	Average			0.44	11

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VIII. SUMMARY AND CONCLUSIONS

In this study, probabilistic liquefaction-triggering curves for gravelly soils based on the Dynamic Cone Penetration (DPT) test blow count (N'_{120}) and shear-wave velocity (Vs) presented that can be used for liquefaction evaluation of gravelly soils for a wide range of earthquake magnitudes, tectonic settings, and geological environments. These curves are a significant step forward compared to those developed by Cao et al. [3,19], as the total number of data points has increased significantly. The N'_{120} and Vs1 data were compiled from various sites around the world where liquefaction or no liquefaction case histories of gravelly soils were observed during several earthquake events in the past. The expanded data set consisted of 174 Vs data points and 137 DPT data points from 17 different earthquakes in 10 different countries in a variety of geological environments. Additional case histories from future events should help improve the triggering curves as we move forward. Comparisons were also made between the liquefaction factor of safety obtained from the BPT and DPT at gravel liquefaction sites from the 1985 Borah Peak earthquake. Based on the results of the field studies and data analysis performed in this study, the following conclusions were drawn:

1. The increased number of liquefaction and no-liquefaction data points in the expanded data set better constrain the probabilistic liquefaction-triggering curves. Relative to the Cao et al. [26] curves for V_s and the Cao et al. [3] curves for DPT, this shifted the P_L = 85% curve to the right and P_L = 15% curve to the left. The reduction in the range between the P_L = 85% and 15% curves indicate a considerable decrease in uncertainty, because false negative data points have less impact on the expanded data set. Shifting the P_L = 15% curve to the left is significant because this probability curve has been recommended for deterministic analyses (e.g. [11]).
2. At lower V_{s1} values (≈150 m/s) and DPT blow counts less than 7, typical of looser gravels, the proposed triggering curves for gravel in this study start at a higher range of CSRs compared to the curves developed by Cao et al. [3, 19]. This modification was necessary to produce agreement with the no-liquefaction points from the field case histories and brought the CSR values in line with the V_{s1} values for sand as predicted by the Kayen et al. [11] probability curves.
3. Simplified *MSF* versus moment magnitude M_w equations were developed exclusively for gravel liquefaction. The *MSF* versus M_w curves plot about midway between similar curves proposed for sand. These results suggest that the effect of magnitude on liquefaction resistance is similar, but slightly different, for both sands and sandy gravels.
4. Although the new probabilistic triggering curves for gravel are similar to those for sands [18] at low V_{s1} values typical of loose gravels (≈150 m/s), they shift to the right as V_{s1} values increase. This indicates that gravels can still liquefy at values that would preclude liquefaction for sands. Therefore, using V_s-based triggering curves for sand when encountering gravels could incorrectly estimate gravel susceptible to liquefaction as being non-liquefiable.
5. The factor of safety against liquefaction computed using the DPT and BPT methods at gravel liquefaction sites in the 1985 Borah Peak Earthquake were within 11% of one another despite the fact that the liquefaction resistance for these two methods are based on entirely different data sets and normalization procedures. This close agreement between these two methods tends to validate the reliability of both methods.

IX. ACKNOWLEDGEMENTS

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Bio-sketch

Kyle Rollins received his BS degree from Brigham Young University and his Ph.D. from the University of California at Berkeley. After working as a geotechnical consultant, he joined the Civil Engineering faculty at BYU in 1987, following his father who was previously a geotechnical professor. He has supervised more than 130 graduate students and published over 200 papers. His research has involved liquefaction assessment of gravels, lateral resistance of piles and pile groups, passive resistance of bridge abutments, lightweight cellular concrete for retaining structures, and various soil improvement techniques. His studies typically involve full-scale testing to determine "ground truth" behavior. Prof. Rollins was the chair of the Geo-Institute technical committee on soil improvement, and ASCE has recognized his work with the Huber research award, the Wellington prize, and the Wallace Hayward Baker award. In 2009, he was the Cross-Canada Geotechnical lecturer for the Canadian Geotechnical Society, and he is the Cross-USA lecturer for the ASCE Geo-Institute for this coming year. He received the Utah Governor's medal for science and technology and the Osterberg Innovation Award from the Deep Foundation Institute.

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Jashod Roy received his B.E. in civil engineering from Jadavpur University in India in 2014. After completion of his B.E., he worked for Fluor Corporation during 2014-2015 as a structural engineer. In 2017, he completed his M.Tech from IIT Bombay with a specialization in geotechnical engineering. Further, he received his Ph.D. from Brigham Young University in 2022 in the field of Geotechnical Engineering under the supervision of Prof. Kyle Rollins. His research during his academic years primarily involved analysis and design of combined pile raft foundation under earthquake loading, liquefaction assessment of gravelly soils based on in-situ tests, and numerical studies. Currently, he is working as a geotechnical engineer for Kiewit in Omaha, Nebraska.

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Regulatory Perspective on Penetration Testing in Gravelly Soils

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ABSTRACT

The California Division of Safety of Dams (DSOD) regulates an inventory of over 1,200 dams. Many of these dams are founded on or constructed from gravelly soils, which can be challenging to characterize and evaluate using conventional techniques. Although most practitioners acknowledge the potential for penetration test values to be artificially elevated in gravelly soils, there is a broad range of opinions regarding the best practices for identifying gravel interference and applying corrections to site-specific data. This paper provides guidance for anticipating and evaluating gravel interference on penetration testing at dams.

Special provisions should be incorporated into geotechnical investigation plans for sites where gravelly soils are suspected or known to be present. The scope of these provisions can vary depending on the investigation objective, but the overall purpose is to ensure that enough contextual information is recorded to evaluate data quality. Once this information is available, the penetration test data should be evaluated for gravel interference by considering the geotechnical and geologic circumstances at each test interval.

We have also found probe-to-particle-diameter ratio to be an effective screening parameter that is preferable to any rule-of-thumb criterion based on gravel percentage alone. After data validation is complete, the higher confidence data can be applied to engineering analyses. This paper demonstrates how a unified geotechnical and geologic approach to gravelly soil characterization is beneficial for dam safety evaluations.

I. INTRODUCTION

The California Division of Safety of Dams (DSOD) has jurisdictional authority for over 1,200 non-federal dams throughout California with a wide range of ages, designs, physical conditions, expected loading conditions, and risk profiles. Dam owners and their engineering consultants commonly perform geotechnical investigations to support design and construction of new dams or the enlargement, repair, alteration, or removal of existing dams. One duty of DSOD's engineers and engineering geologists is to oversee geotechnical site investigations for all phases of a project by reviewing work plans, conducting field inspections, and evaluating data quality and reliability. This work not only contributes to positive dam safety outcomes for individual projects, but in aggregate it provides DSOD with important insights into how the range of available investigation equipment, methods, and strategies are applied to different projects and site conditions across California. The possibility of gravel interference can be a significant challenge for practitioners where dams are founded on or constructed from gravelly soils, particularly for engineering evaluations of stability or liquefaction susceptibility. The goal of this paper is to leverage DSOD's experience reviewing a variety of projects across California and provide our perspective on the best strategies for collecting and validating penetration resistance data at gravelly soil sites to obtain reliable data for use in engineering analysis.

Penetration testing is a widely used method for characterizing in-situ properties of cohesionless soil in dam embankments and foundations in California. The Standard Penetration Test (SPT) is advantageous because it provides a standardized measure of penetration resistance (blow counts) for use in engineering evaluations and allows users to examine soil samples recovered from each test interval. One disadvantage to the SPT is the widely recognized observation that gravel-sized particles can interfere with equipment and produce elevated blow counts (ASTM Standard D6066-11). A larger diameter penetrometer such as the Modified California sampler (ModCal) or

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Becker Penetration Test (BPT) can reduce the potential for gravel interference, but the measured penetration resistance must be converted to an equivalent SPT blow count for use in most engineering analyses, introducing additional uncertainty (Daniel et al., 2003; Rogers, 2006). Larger diameter penetrometers are also susceptible to interference and elevated blow counts if gravels are large enough or abundant enough to interfere with the continuum penetration process[1]. Regardless of the equipment used, gravel interference can cause the material properties (e.g., density, stiffness, strength, etc.) of gravelly soil to be over-estimated in critical engineering analyses.

In this paper, we refer to particles larger than the No. 4 sieve (4.75 mm, or about 0.2 inches) as "gravel" and use the term "gravelly soil" to describe any earth material containing a measurable quantity of gravel. This definition of gravelly soil encompasses a wide range of gradations from poorly graded gravels to gap-graded soils (gravels surrounded by sandy or fine-grained matrix) to well-graded mixtures of sand and gravel (Figure 1). Soil can also be stratified such that the gradation of a gravelly soil may vary significantly with depth or alternate with layers containing few or no gravels. A key challenge of penetration testing in gravelly soil is the need to identify and correct for gravel interference without directly knowing how different gradations affect each test interval.

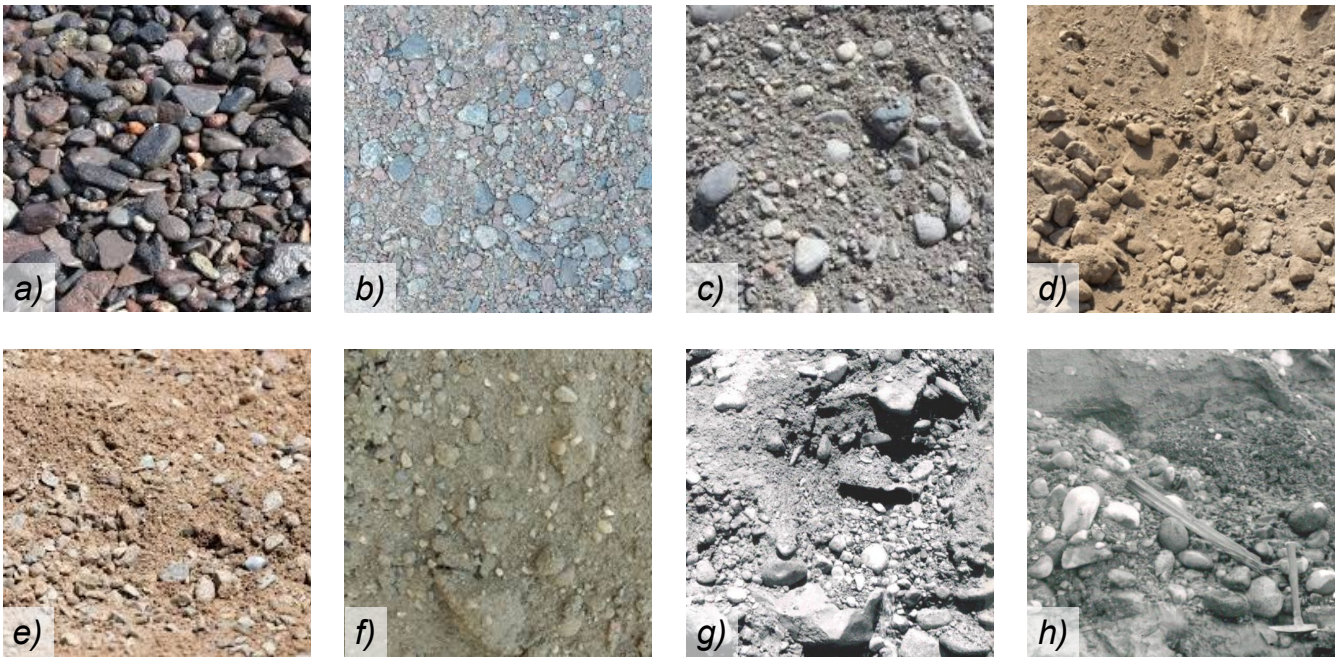


Figure 1. Example images of gravelly soils.

Gravels can interfere with and resist the advancement of open-ended (e.g., split spoon) penetrometers when: (1) gravel blocks the sampler opening, (2) gravel resists soil movement up into the sample barrel, or (3) gravel contacts the outside of the penetrometer (Figure 2A). As the penetrometer advances, it fails a volume of soil that is typically assumed to extend 5 to 10 times the probe tip diameter in all directions. Ideally, this failure bulb is representative of the continuum soil mass. However, if gravel become large enough or abundant enough relative to the size of the penetrometer the measured penetration resistance is instead representative of the discrete particles, which can lead to an inaccurate characterization of soil properties. An open-ended penetrometer with a partially or completely blocked opening typically requires more hammer blows to advance through the test interval because soil that would otherwise enter the sample barrel must fail around the outside of the sample barrel instead. Similarly, gravel inside the sample barrel can wedge or drag along the inner wall and increase the amount of soil required to fail outside the penetrometer and require more hammer blows to advance through the test interval. Lastly, in-place gravel can wedge or drag against the outside of the penetrometer and require additional hammer blows to fail the surrounding soil and overcome added frictional resistance. All these interference mechanisms are functions of relative size and abundance.

Gravel interference can also affect closed-ended penetrometers; although, the mechanism is slightly different

[1] Although it could be more appropriate to refer to "cobble interference" for the BPT, we use the term gravel interference herein to refer to the influence of particle size and abundance for all types of penetrometers.

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than for a split-spoon penetrometer. Because the closed-ended penetrometer is designed to fail the entire volume of soil in a test interval, the interference occurs when gravel (or cobbles) act to increase the length or tortuosity of failure paths for matrix materials (Figure 2B). As with split-spoon penetrometers, gravel interference on closed-ended penetrometers occurs when particles are large enough or abundant enough to interfere with the continuum penetration process.

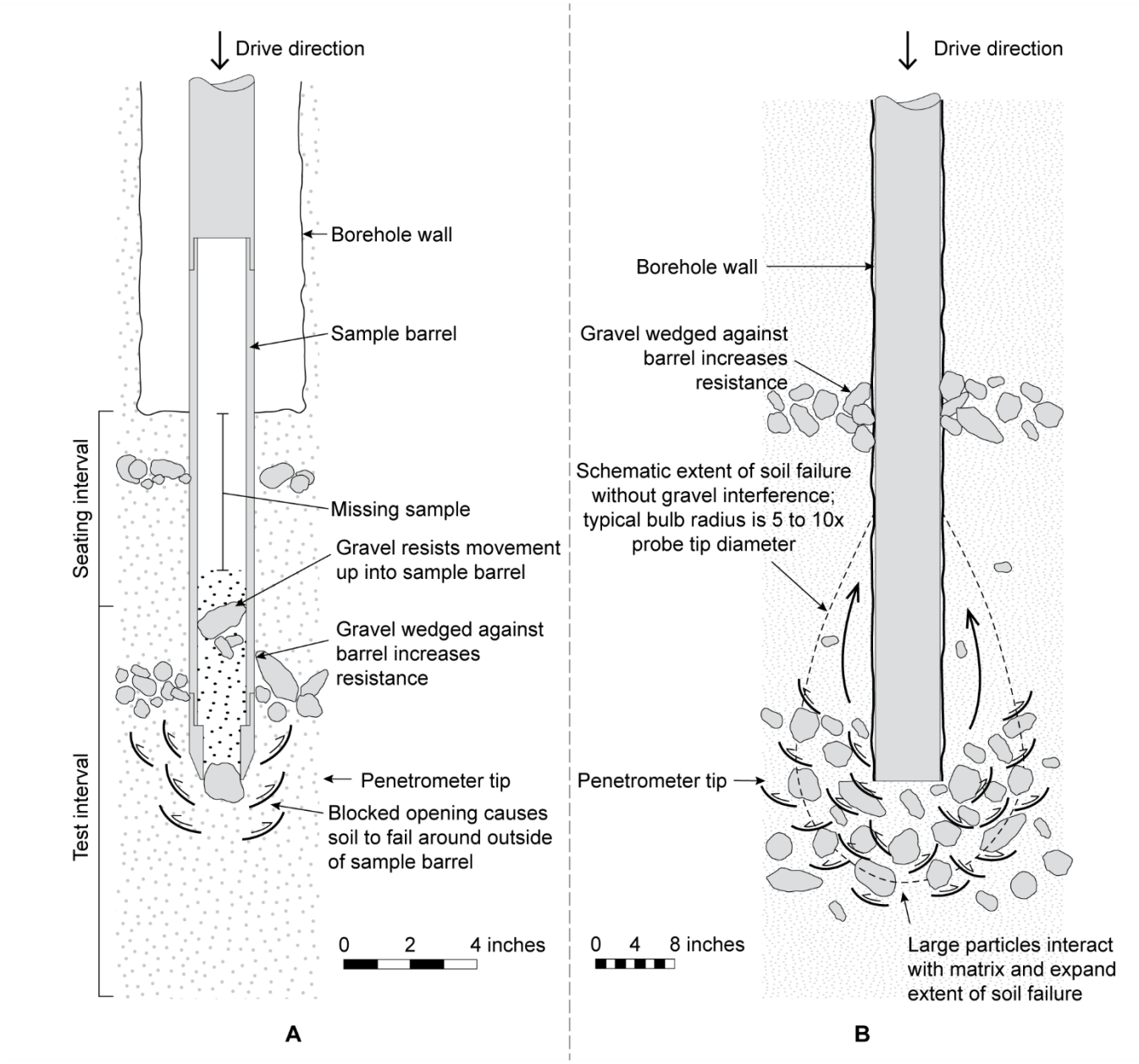


Figure 2. Schematic diagram of gravel interference mechanisms on A) open-ended and B) closed-ended penetrometers.

There is a broad range of opinions regarding the best practices for identifying and adjusting for gravel interference for the variety of open and closed-ended penetrometers used at dam sites in California (Figure 3). Although it may not always be possible to accurately quantify the effect of gravel interference relative to a nominal standard, there are many tools and strategies available in standard practice that can optimize the amount of reliable penetration resistance data collected during a site investigation and help guide important decisions about data quality. The appropriateness of these (and other) approaches depends significantly on specific site conditions as well as the quality and documentation of contextual information.

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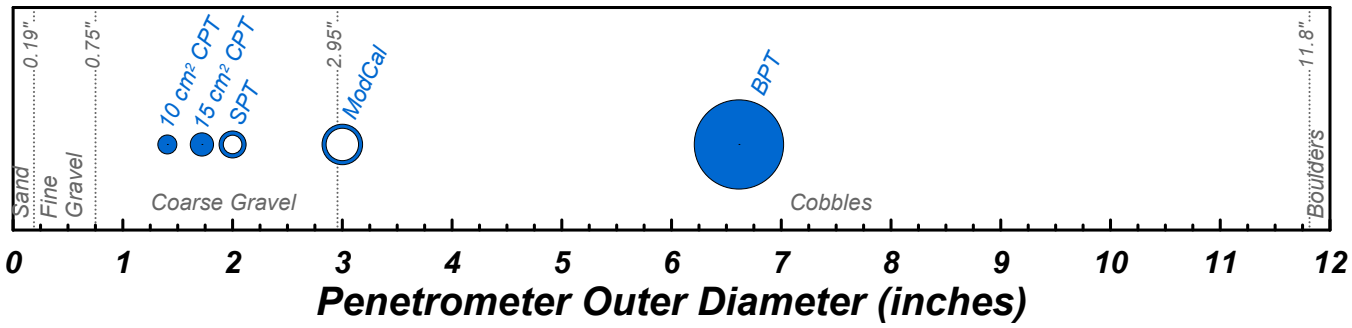


Figure 2. Open and closed-ended penetrometers used at dam sites in California showing penetrometer outer diameter shown in comparison to USCS soil sizes.

II. DATA COLLECTION

A. Investigation Planning

Efforts to minimize the impact of gravel interference on penetration test data should start in the planning stage by defining the investigation objectives and anticipating the types of engineering analyses that may need to be completed. Analyses that might incorporate penetration resistance data should be identified in advance and associated with specific borings where additional measures may be needed to evaluate gravel interference. This important step is commonly overlooked and should be prioritized, especially at dam sites where gradation information may be limited due to incomplete records or absence of previous exploration. In regions where gravelly soils are commonly encountered it is prudent to assume that gravelly soils will be encountered unless there is sufficient geologic information to rule out that possibility.

Designing a site investigation with an assumption of gravelly soils ensures that appropriate information can be collected to evaluate gravel interference. This provides an opportunity for suitable equipment to be mobilized and for field personnel to be provided with proper training, procedures, and work plans. Once underway, it may be possible to relax specific provisions related to gravel interference if site conditions warrant. This approach is preferable to collecting penetration test data from gravelly soils that cannot be adequately validated or used with confidence during the analysis phase.

A phased approach to geotechnical investigations can improve the quality of penetration testing in gravelly soils where one or more initial borings are used to understand the location, stratification, or gradation of gravelly soils at a site. This information can be used to select equipment and methods for penetration testing in subsequent borings. Drilling methods that can produce near-continuous soil samples are particularly useful for initial characterization of gravelly soils, such as sonic core borings and hollow-stem auger borings (using a continuous sample tube).

B. Drilling Boreholes

Rig geologists or engineers should be in position to directly observe drilling operations and communicate with the drill operator about drilling conditions and equipment responses to gravelly soil. Drillers using mud rotary equipment should be prepared to adjust the drilling rate, fluid output, flushing media, and drill bit to successfully clear gravel cuttings from the bottom of the hole and minimize excessive erosion of the borehole wall. It is possible for test intervals with no gravel fraction to be influenced by gravel if sampling equipment interacts with gravel in sloughed material at the base of a hole.

C. Penetration Testing

Open-Ended Penetrometers

All else being equal, geotechnical investigations that implement ASTM Standard D6066-11 are more likely to collect penetration test data that can be considered reliable and used confidently in stability and liquefaction triggering evaluations. This standard was withdrawn by ASTM in 2020 because it had not been updated within the standard review period; however, the guidance in this standard remains valid and appropriate for dam safety evaluations. For penetration testing in gravelly soil, ASTM Standard D6066-11 (§12.10.1) recommends recording blow counts in 0.1-ft intervals (typically implemented in California using 1-inch intervals). Short-interval blow counts can be challenging to record in real time, but it is relatively simple to document the rate of advance with video to ensure accuracy. Farrar (2016) describes several methods for recording short-interval blow counts and drive per blow.

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This procedure is useful but may be inadequate for evaluating gravel influence unless other key provisions in ASTM Standard D6066-11 are also followed. For example, the amount of sample recovered from each test interval is critical to interpreting per-inch blow count data and needs to be recorded (§12.12). Incomplete sample recovery may indicate that the sampler was blocked by gravel. Test intervals with no sample recovery (or poor recovery) should be re-driven using a sample retainer or a larger diameter split-spoon sampler (§6.3.2.2).

Consider performing near-continuous penetration tests (with appropriate clean-out intervals) to increase the total number of test intervals, particularly in gravelly soils that might be interstratified with other materials or have significant variation in the quantity and size of gravel. Decreasing the vertical spacing between penetration test intervals can increase opportunities to collect reliable data without increasing the number of borings.

While ASTM Standard D6066-11 includes some recommendations for penetration testing in gravelly soil, it was primarily developed for liquefaction assessment of clean sands. As such, there are certain provisions that may need to be relaxed when working with gravelly soil. For example, the standard does not recommend using larger-diameter ModCal samplers for determining penetration resistance, but this practice can increase sample recovery in gravelly soils and reduce the likelihood of gravel interference by increasing the probe-to-particle diameter ratio. This practice requires an additional step to convert the ModCal blow courts to equivalent SPT blow counts, which introduces some uncertainty (Daniel et al., 2003; Rogers, 2006). It can be advantageous to alternate between SPT and ModCal samplers in gravelly soil to realize the benefits of both approaches.

Closed-Ended Penetrometers

The Cone Penetration Test (CPT) provides an abundance of information for performing engineering analyses at dam sites; however, it has several disadvantages when used in gravelly soils. The outer diameter of both the 10 cm^2 and 15 cm^2 CPT are smaller than the SPT (Figure 3), which makes this equipment more susceptible to gravel interference. CPT operators should monitor for rod chatter and report it in their logs as an indicator of likely gravel interference. Because CPTs don't collect samples, they should be paired with lithologic information from an adjacent boring that has been described at an appropriate level of detail.

The BPT is the largest diameter penetrometer used for dam projects in California (Figure 3) and is the least susceptible to gravel interference, but its high cost generally limits its use to the most critical or well-funded projects. Numerous methods have been proposed to interpret BPT results and account for the variable effects of shaft friction on penetration resistance (Harder and Seed, 1986; Sy and Campanella, 1994; DeJong et al., 2017). These various interpretation methods can produce significantly different results and contribute to conflicting interpretations about embankment stability. DSOD has observed that the direct measurement of tip energy offered by the recently developed instrumented BPT (iBPT) (DeJong et al., 2017) is typically the most reliable of the available interpretation methods, however, the conversion of iBPT data to equivalent SPT blow counts remains a significant source of uncertainty. A reliable approach is to collect SPT blow counts from an adjacent boring and perform a site-specific verification of the standard 1.8 correlation factor (Ghafghazi et al., 2017) in test intervals where the SPT blow counts are free of gravel interference or can be reliably adjusted. If a site-specific verification is not possible, sonic core borings or other borings that recover near-continuous samples should be performed to provide contextual information for this non-sampling penetrometer.

D. Sample Description

It is important to characterize the gravel fraction of every sample recovered from an open-ended penetration test. Key parameters for evaluating gravel influence include field estimates of the coarse gravel fraction, the fine gravel fraction, and the maximum gravel size. Descriptions of gravel angularity or roundness, shape, and hardness are also highly relevant. Visual assessments of gravel fraction typically reference percent by volume and should be estimated to the closest 5%. Any samples recovered using liners or rings and not selected for laboratory-based particle size analysis should be extruded and described using visual standards. As noted above, any data collected using a close-ended penetrometer need to be paired with lithologic information from an adjacent boring that has been described at an appropriate level of detail.

Laboratory sieve analyses should also be performed on representative soil samples to confirm the visual field estimates and provide quantitative information on particle size. As discussed later, the 85th percentile particle size (D_{85}) can be a useful parameter for identifying gravel interference.

Photographing samples before removing them from the split-spoon sampler is an effective and cost-efficient method to document the stratigraphic context of each penetration test interval. Sample photographs should be taken in natural or even lighting at an appropriate resolution for each test interval where per-inch blow count data will be used to identify and correct for gravel influence. Boring logs and photographs from adjacent sonic core

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borings can also be used to evaluate the relative presence or absence of gravel in a specific test interval.

E. Corroborative Testing

Material properties can be directly measured to corroborate penetration resistance data at gravelly soil sites. For example, large-diameter ring density tests provide a reliable measurement of in-situ density that can be compared to density estimates from empirical correlations with penetration resistance. If the independent methods provide comparable results, this can be evidence that penetration resistance data is free of gravel interference or has been appropriately adjusted. The drained friction angle is another parameter that can be measured directly and estimated via empirical correlation to corroborate penetration test results in gravelly soils. Special provisions must be followed to ensure laboratory strength tests are also free of gravel interference, however, a discussion of this topic is beyond the scope of this paper.

III. DATA VALIDATION

A. Identifying Gravel Interference

The existing guidance for identifying and adjusting for gravel interference is focused on the SPT and CPT. For the SPT, Seed et al (2003) recommends that SPT blow counts can be corrected for gravel interference "when the non-gravel (finer than about 0.25-inch diameter) fraction of the soil represents greater than about half of the overall soil mix/gradation." Idriss and Boulanger (2008) make a slightly different recommendation that SPT blow counts can be reasonably corrected "for gravel contents up to about 15 - 20%." Both sets of guidance focus on adjusting or “correcting” SPT blow counts, which implies that gravel interference should be expected when performing SPTs in these conditions. Additionally, both recommendations include thresholds for size (0.2 or 0.25 inches) and abundance (20 or 50%), reflecting the importance of both parameters when evaluating gravel interference. While these guidelines may be generally reasonable, they were developed specifically for the SPT and are not directly applicable to other penetrometers.

For the CPT, Pires-Sturm and DeJong (2022) used a centrifuge to conduct CPTs in soils of varying gradation to develop practical guidance for identifying gravel interference at the field scale. The study found that the 85th percentile particle size (D_{85}) was a better indicator of gravel interference than the median particle size (D_{50}). The study also used the non-dimensional probe-to-particle-diameter (D_{probe}/D_{85}) ratio to predict gravel interference and found that penetration resistance is elevated for D_{probe}/D_{85} ratios lower than about 10, significantly elevated for D_{probe}/D_{85} ratios lower than about 5, and approaches refusal for D_{probe}/D_{85} ratios lower than about 2.

A unified framework for identifying gravel interference is possible if we can apply the non-dimension approach of Pires-Sturm and DeJong (2022) to the guidance of Seed et al (2003) and Idriss and Boulanger (2008). Based on discrete element modeling (DEM) by Daniel et al. (2008), we expect open-ended penetrometers will behave in a plugged fashion when driving in gravelly soils; therefore, it is reasonable to define D_{probe} as the SPT outer diameter (2 inches) rather than the inner diameter (1.375 inches). As such, Seed et al (2003) would predict gravel interference for the SPT at D_{probe}/D_{50} ratios lower than 8, while Idriss and Boulanger (2008) would predict gravel interference for D_{probe}/D_{85} ratios lower than about 10 (assuming D_{85} is about 0.2 inches, larger than the No. 4 sieve per the USCS definition of gravel). These thresholds for the SPTs are consistent with the recommendations of Pires-Sturm and DeJong (2022) for CPTs, and it may be reasonable to adopt similar criteria for identifying gravel influence on other penetrometers.

Table 1 presents the maximum D_{85} values for the common penetrometers used in California to maintain D_{probe}/D_{85} ratios lower than 10, 5, and 2, respectively. These D_{probe}/D_{85} ratios are not only a helpful starting point for evaluating the likelihood of gravel interference but are also a useful framework for planning a geotechnical site investigation to ensure appropriately sized penetrometers are specified in work plans and mobilized to site.

The D_{probe}/D_{85} ratio should be used in conjunction with the other contextual information to help identify gravel interference. In some cases, the visual and laboratory estimates of particle size may not be representative for sampling methods that scalp or crush the largest particles. This could produce a larger D_{probe}/D_{85} ratio, leading to an underprediction of gravel interference. In one project under DSOD review, construction photographs of abundant cobbles showed that a potentially liquefiable foundation material is much coarser than indicated by gradation estimates on recovered sonic core from the same deposit (Figure 1g and h). This contextual information provided a key piece of justification for identifying elevated iBPT blow counts in this material.

The D_{probe}/D_{85} ratio should not be the sole parameter used to validate penetration test data in gravelly soils, as it is based on extrapolation of limited test data, uses only a single measure of particle size to predict interference,

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and likely does not capture the complex differences between open-ended and closed-ended penetrometers. This is a developing area of research, and we expect that our perspective will evolve with future work.

Table 1: Threshold D_{85} particle size for identifying gravel interference on various penetrometers.

Penetrometer	D_{probe} (inches)	Gravel Interference Less Likely [$D_{probe}/D_{85} > 10$]	Gravel Interference Possible [$10 > D_{probe}/D_{85} > 2$]	Gravel Interference Likely [$D_{probe}/D_{85} < 2$]
10 cm^2 CPT*	1.4	$D_{85} < 0.14''$ [coarse sand]	$0.70'' > D_{85} > 0.14''$ [coarse sand to fine gravel]	$D_{85} > 0.70''$ [fine gravel]
15 cm^2 CPT*	1.7	$D_{85} < 0.17''$ [coarse sand]	$0.85'' > D_{85} > 0.17''$ [coarse sand to coarse gravel]	$D_{85} > 0.85''$ [coarse gravel]
SPT**	2.0	$D_{85} < 0.20''$ [fine gravel]	$1.0'' > D_{85} > 0.20''$ [fine to coarse gravel]	$D_{85} > 1.0''$ [coarse gravel]
ModCal**	3.0	$D_{85} < 0.30''$ [fine gravel]	$1.5'' > D_{85} > 0.30''$ [fine to coarse gravel]	$D_{85} > 1.5''$ [coarse gravel]
BPT/iBPT*	6.625	$D_{85} < 0.66''$ [coarse gravel]	$3.3'' > D_{85} > 0.66''$ [fine gravel to cobbles]	$D_{85} > 3.3''$ [cobbles]

** No reliable procedures are available to correct data from closed-ended penetrometers (see §IIIB).*
*** Data correction can be attempted for open-ended penetrometers (see §IIIB and Figure 4).*

B. Adjusting for Gravel Interference

Open-Ended Penetrometers

Penetration test data collected in gravelly soils using SPT or ModCal samplers need to be screened and validated prior to use in critical engineering evaluations. Collection of short-interval (per-inch) blow count data has become relatively common at dam sites in California, though it has been less common for practitioners to consistently document the contextual information needed to validate these data. It is important to use all available data to critically evaluate each test interval to decide if it should be accepted, adjusted, or excluded before proceeding with engineering analyses. Figure 4 provides a flowchart to guide users through this process.

Test intervals with a low likelihood of gravel interference can be accepted and reliably used in engineering analyses. These test intervals should have nearly complete sample recovery (80 to 100%) and a D_{probe}/D_{85} ratio larger than about 10. For test intervals meeting these conditions, any changes in penetration rate can likely be attributed to variability of the material. Incomplete sample recovery may indicate that the soil is too loose to be retained, or it may indicate that the sampler tip was completely or partially blocked by gravel. If a test interval with incomplete recovery was not re-driven to verify the absence of gravel, then it may be necessary to assume that the sampler tip was obstructed and either adjust or exclude the data from analysis.

There are several, well-established methods for adjusting penetration test data using short-interval blow counts. A common treatment is to plot cumulative blow counts on one axis and cumulative penetration depth on the other (Seed et al., 2003). For this method, plots showing abrupt or significant increases in penetration rate are typically interpretated as evidence of gravel influence. A common error occurs when practitioners conclude that a steady, near-constant penetration rate is reliable evidence of no gravel interference—this is not necessarily true, and if short-interval blow counts are consistently high, then gravel interference may be significant. Incremental blow counts can be also plotted with cumulative penetration depth (Ghafghazi et al., 2017) to identify and adjust the portions of each test interval with elevated blow counts.

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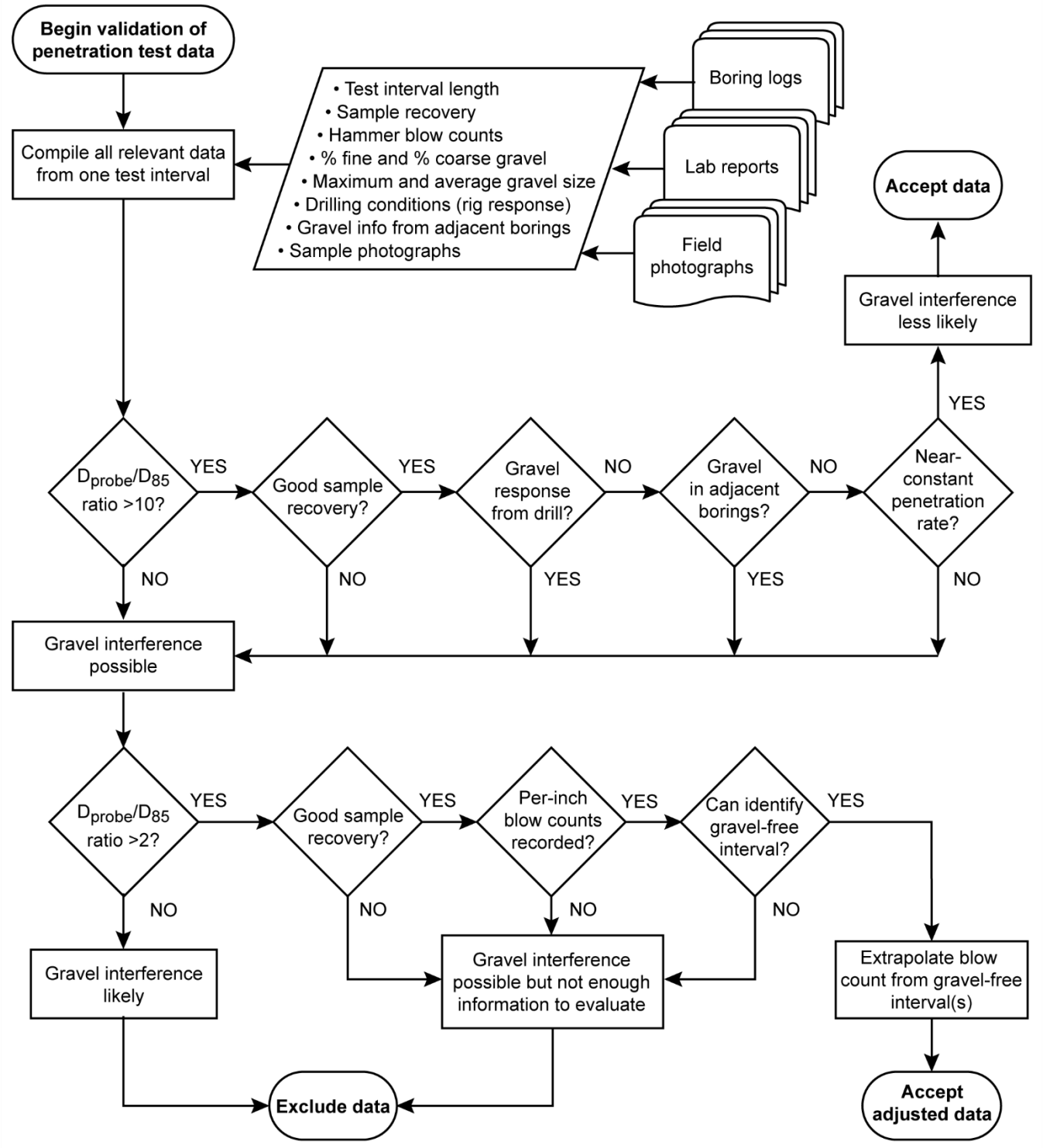


Figure 4: Flowchart for evaluating gravel interference on opened-ended penetrometers in a single test interval.

We find it is often necessary to exclude SPT or ModCal data from analysis instead of performing an adjustment. Existing guidance for plotting short-interval blow counts often leads practitioners to apply adjustments to many test intervals outside of the gravel size and abundance thresholds recommended in literature. Based on the findings of Pires-Sturm and DeJong (2022), it is reasonable to exclude any data with D_{probe}/D_{85} ratios lower than about 2. It

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may also be prudent to exclude data when there is uncertainty due to a lack of information following guidance of Terzaghi and Peck (1967) who advised that "gaps in available information must be filled by the most unfavorable assumptions compatible with the known facts."

Closed-Ended Penetrometers

There are no commonly referenced procedures for adjusting CPT or iBPT data for gravel interference. While Pires-Sturm and DeJong (2022) developed an equation to correct CPT penetration resistance as a function of the D_{probe}/D_{85} ratio, this correction is based on limited centrifuge data and is not appropriate for use on dam safety evaluations without further research. In the absence of an accepted adjustment procedure, it is prudent to exclude any CPT or iBPT test intervals where gravel interference is suspected.

IV. FUTURE WORK

The current standard of practice could be significantly improved by developing sound guidance for identifying gravel interference for the range of penetrometers used to characterize gravelly soil (Figure 3). D_{probe}/D_{85} appear to be a useful framework to account for gravel size and abundance, but additional work is needed to verify the thresholds proposed in Table 1. This could be accomplished via a combination of discrete element modeling, centrifuge testing, and field-scale application. Additional probe-to-particle-diameter thresholds may need to be developed for other gradation metrics (i.e., D_{50} or D_{100}). The effects of secondary parameters such as gravel shape, angularity, and hardness should also be studied.

The current standard of practice could also be improved by developing new approaches for correcting gravel interference on closed-ended penetrometers. It may be possible to develop closed-form equations that incrementally reduce measured penetration resistance as function of gradation; however, the use of a functional form would still require lithologic information from adjacent borings.

V. CONCLUSIONS

Penetration testing is a widely used method for characterizing in-situ properties of cohesionless soil in dam embankments and foundations in California. Gravel interference on penetration testing is commonly overlooked or underestimated but can have a significant effect on engineering evaluations of stability or liquefaction susceptibility. In DSOD's experience, the most effective efforts to generate reliable penetration test data in gravelly soil will consider the possibility of gravel interference when planning a geotechnical investigation, mobilize appropriate penetrometer equipment, and observe and document all relevant geotechnical and geologic conditions for each test interval. These measures can often be relaxed where materials are found to have no significant gravel content but are difficult or impossible to re-create where materials are found to have more gravel than expected.

Gravel interference on penetration testing is primarily a function of gravel size and abundance relative to the diameter of the penetrometer equipment. The particle-probe diameter ratio is a helpful starting point where a D_{probe}/D_{85} ratio lower than 10 may indicate that gravel interference is likely and a D_{probe}/D_{85} lower than 2 may indicate that gravel interference cannot be reliably adjusted or corrected and may need to be discarded. Reliable blow count corrections are typically well supported by additional contextual information about sample recovery, fine and coarse gravel fractions, and drilling conditions (rig response). Reliable penetration data from closed-ended penetrometers is typically well supported by similar contextual information from adjacent geotechnical borings.

VI. ACKNOWLEDGEMENTS

Any opinions, findings, and conclusions or recommendations expressed in this paper are those of the authors and do not necessarily reflect the views of the California Department of Water Resources, Division of Safety of Dams.

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DAM SAFETY PALM SPRINGS
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Shake, Quake, and Tremble – Seismic Events and their Impacts on Dams

Recent Highlights from DamFailures.org

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I.EXTENDED ABSTRACT

In September 2015, the Association of State Dam Safety Officials (ASDSO), in cooperation with the Federal Emergency Management Agency (FEMA), released the *Lessons Learned from Dam Incidents and Failures* website (DamFailures.org). The primary goal of this website is to convey educational information relating to and resulting from dam failures and incidents in an innovative, user-friendly manner. Since its release, over 350,000 users worldwide have accessed the site to learn more about dam engineering and design practices, human factors as they relate to dam incidents, emergency planning and response, operation and maintenance, and other dam safety topics.

As part of the ASDSO Dam Failures and Incidents Committee, the DamFailures.org Subcommittee currently oversees maintenance and development of the website. Thanks to continued FEMA funding, the website content is being significantly expanded and diversified. This session will consist of a brief introduction to and update on ASDSO's DamFailures.org website followed by presentations from authors that focus on seismic incidents at dams and associated lessons learned. Each of the presentations will be based on content available on DamFailures.org . Presentations are as follows:

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1) Lesson Learned: Dams located in seismic areas should be evaluated for liquefaction, cracking, potential fault offsets, deformations, and settlement due to seismic loading. (Everett Taylor, P.E.)

Earthquakes occur throughout the world with unknown frequency and potentially devastating effects. Due to the consequence of their failure, seismic evaluation and design should be included when determining the location and design of a new dam or reviewing the design and function of an existing one. The evaluation process should consider the type of dam to construct. Earthfill and rockfill dams typically perform better than concrete dams under seismic loading because of the nature of their makeup. Though earth and rockfill dams are inherently more resistant that does not make them immune to the effects of earthquake loading. Deformations to the crest, cracking of the embankment, damage to a core wall, and damage to the spillway are a few potential failure modes that an earth or rockfill dam may experience during an earthquake. For further information regarding observed performance at dams during earthquakes see the United States Committee on Large Dams' (USCOLD) *Observed Performance of Dams During Earthquakes (Volumes 1 through 3).*

Designers should consider the potential impacts earthquakes may have on their project and include defensive design measures to counteract damage as best possible and prevent catastrophic failure.

https://damfailures.org/lessons-learned/dams-located-in-seismic-areas-should-be-evaluated-for-liquefaction-cracking-potential-fault-offsets-deformations-and-settlement-due-to-seismic-loading/

2) Case Study: Hebgen Dam and Quake Lake (Shane McWilliams, P.G.)

Just before midnight on August 17, 1959, in southwest Montana, a Richter Scale Magnitude (M_L) 7.3 earthquake caused a block of the earth's crust 125 square miles in area, including Hebgen Lake, to subside along established faults north of the reservoir. Up to 21 feet of vertical displacement was measured along the segment of the Hebgen Fault near the dam. The earthquake caused a massive rockslide to cross the Madison River and up the opposite canyon wall. Rockslide debris dammed the river, forming Quake Lake just downstream of Hebgen Dam.

As water began rising behind the slide, officials feared that the mass would be overtopped and breached or that water in Quake Lake would rise to the toe of Hebgen Dam, potentially destabilizing that dam. Working around the clock, a discharge channel was excavated through the slide mass, lowering the lake to a safe level. Ground deformations from the earthquake also damaged Hebgen Dam, cracking the concrete core wall, damaging the spillway, and inducing a seiche on Hebgen Dam reservoir, causing waves to overtop the earthfill dam.

The failure of the dam and uncontrolled release of Hebgen Lake would have been catastrophic for the downstream population at risk. Although severely damaged, Hebgen Dam did not fail and was later repaired.

https://damfailures.org/case-study/hebgen-dam-montana-1959/

3) Case Study: Sheffield Dam (Cory Miyamoto, P.E.)

Sheffield Dam is one of the earliest recorded case histories of earthquake-induced failure of an embankment dam. Constructed in 1917 near Santa Barbara, California, the 25-foot-high, 720-foot-long earth dam failed on the morning of June 29, 1925, after being subjected to shaking from a M_L 6.3 earthquake event roughly 7 miles northwest of the dam site. Peak ground accelerations at the dam site were estimated in 1968 to be 0.15g, but may have been at least 0.25g, based on more recent estimates.

The strong ground shaking caused a roughly 300-foot-wide section of the embankment to slide downstream for nearly 100 feet, releasing 30 million gallons of water contained in the reservoir (~92 acre-feet). The collapse of the dam caused flooding 1- to 2-feet deep in the lower parts of the city. Although the earthquake caused extensive damage to Santa Barbara and the surrounding areas, including 13 deaths, no casualties were attributed to the dam failure.

Later investigations into the cause(s) of the collapse identified liquefaction at the base of the dam as a key contributing factor in the failure.

https://damfailures.org/case-study/sheffield-dam-california-1925/

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4) Case Study: Langley Dam (Jeff Munsey)

The failure of Langley Dam, located near Aiken, South Carolina, is one of the more noteworthy and surprising far field (epicentral distance ~176 km) effects of the M_L 7.0, August 31, 1886, Charleston, SC earthquake. Langley Dam, a relatively small earthen mill dam, was constructed on Horse Creek and completed in 1871. Almost immediately after shaking from the Charleston earthquake ceased, Langley Dam failed and the discharged water damaged two railways, which caused two separate train derailments. Two of the fatalities attributed to the Charleston earthquake were from crewmembers on the two trains.

Historical accounts of the dam failure integrated with recently completed geotechnical investigations and site response modelling are used to explain the surprising failure of Langley Dam. Henry W. Grady, who was managing editor of the Atlanta Constitution newspaper, wrote an important historical account of the Langley Dam failure. Mr. Grady left Atlanta for Charleston within hours of the main shock. His detailed trip accounts provide a unique perspective on the aftermath of the Charleston earthquake. In particular, his description of the post-failure condition of Langley Dam is critical in understanding how the dam failed.

The foundation conditions and geologic setting of Langley Dam, and the materials and methods used to construct the dam played key roles in its failure. Recognition of the geologic conditions that contributed to the failure of Langley Dam has implications for seismic design of modern structures in similar settings in Georgia and South Carolina.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

Assessing Dam Performance During a Non-Failure Incident: Hurricane Fiona's Impact on Puerto Rico Dams

PATRICK MILES, P.E., AND JACOB ADKINS, EIT, FREESE AND NICHOLS, INC.; MOLLY FINSTER, PH.D., AND CAROL FREEMAN, ARGONNE NATIONAL LABORATORY

ABSTRACT

Evaluating dam performance after significant flooding events – even those not resulting in dam failure – provides opportunity for dam owners and community representatives to evaluate the impact of dams on flooding and related consequences, leading to a better understanding of dam risk and the need to enhance emergency preparedness.On September 18, 2022, Hurricane Fiona made landfall in Puerto Rico dumping more than 20 inches of rain across much of the island and resulting in widespread flooding and storm-related damages. Many communities were left without power and/or clean water for several days. The island's thirty-six dams survived the flooding without major failure. However, FEMA is interested understanding the influence – both positive and negative – that the dams and their operations may have had on flooding during the event and how the emergency management personnel responded. This presentation will discuss FEMA's Incident Specific Technical Assistance (ISTA) in Puerto Rico, specifically highlighting work that is evaluating dam performance during Hurricane Fiona in three watersheds – Rio de La Plata, Rio Grande de Loiza, and Rio Jacaguas. These watersheds include a total of seven dams and represent numerous affected communities across the island. Project efforts developed a suite of hydrologic and hydraulic models calibrated to the Hurricane Fiona rainfall to compare flood impacts with and without the dams in place.Through a robust post-event data collection process, conducted in close collaboration with federal, regional, and state partners and local stakeholders, FEMA and the ISTA project team conducted more than 30 interviews and site visits, developed detailed models, performed spatial mapping, and created illustrative graphics to represent hurricane-related flooding and an understanding of the dams' influence on flood damages. From these findings, the project team determined a number of lessons learned, best practices, challenges, and potential mitigation strategies to support Puerto Rican dam owners and downstream communities at risk for flooding to better plan and prepare for future extreme weather events, thus improving overall resilience. The positive benefits of performing the Puerto Rico ISTA study will be highlighted, as well as specific actions that resulted from the effort.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

An Alternate Approach for Developing Design Waves for Establishing Freeboard Requirements for Perched Reservoirs Exposed to Extreme Winds

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ABSTRACT

This paper describes an alternative approach to developing design waves for perched reservoirs with extreme winds. Although the focus of the paper is on off-channel reservoirs, the approach can be generalized to all reservoirs. The authors describe how to leverage computational fluid dynamics (CFD) to model changes in the wind field over the surface of the reservoir and provide a wind map for input to a two-dimensional wave growth model. Additionally, we recommend capping the drag coefficient in wave growth models based on a review of available data collected under extreme wind conditions.

Results compared and contrasted with more conventional approaches demonstrate how advanced numerical modeling tools can provide conservative design waves with less uncertainty than standard approaches and potentially lower construction costs.

I.INTRODUCTION

Off-channel reservoirs are used for a variety of purposes, including pumped-storage hydropower, emergency drought reserves, industrial and mining processes, and wet-season storage for water supply, seasonal fish releases, irrigation, and municipalities. Key advantages of off-channel reservoirs are they can reduce flooding when filled during periods of high river or channel flow, preserve or enhance instream flows when stream flows drop, and provide storage or hydropower benefits without obstructing environmentally sensitive waterways. A disadvantage of off-channel reservoirs is they are typically costly to construct. They are commonly built by constructing earthen embankments around most or all of the reservoir perimeter, usually using soil from inside the reservoir footprint. This results in a large ratio of embankment length to storage volume and unit storage cost.

The heights of these embankments are determined such that during the extreme design storm event, overtopping flows due to wave action on the embankments are insufficient to erode their landside slopes, which could lead to damage and possible failure of the embankments.

Reservoir freeboard (the height of the embankment crest above the reservoir surface) is affected by rain falling on the reservoir, precipitation runoff from any contributing watershed and within the highest embankment rim, operational (pumped or gravity fed) inflows, operational drawdown, active and passive spillway and outlet releases during storms, and wind setup that increases water levels on the downwind side of the reservoir. Wave action will also be on the downwind side of the reservoir, where water levels will be highest.

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The volume of soil required to construct impounding embankments is a significant contributor to the cost of the reservoir, and the design freeboard can significantly impact the embankment volume. For example, for a reservoir with a 40-foot-high embankment with a 15-foot-wide crest and 2.5:1 (horizontal to vertical) side slopes, raising the embankment crest by one foot to provide one additional foot of freeboard would add nearly 5% to the volume of the embankment. With the same shape, a one-foot raise to a 20-foot embankment would increase the earthen volume by 9%. A volume of a 12-foot-high embankment would increase the volume by more than 14%. For a reservoir with a large surface area, raising a long embankment could cost millions or tens of millions of dollars.

In the past, freeboard requirements for wind setup and runup from wind-driven waves were addressed using empirical equations that are based on limited data sets and simplifying assumptions. Many of these relationships are based on waves produced by lower or moderate wind speeds, with little to no data from extreme hurricane-force wind events. Application of these equations to size waves from extreme winds requires extrapolations outside the range of the data on which the equations were developed.

More recently, there has been increased use of physics-based numerical wave and hydrodynamic models in place of empirical approaches. These numerical models can address factors such as variable bathymetry, variable wind fields, reservoir shoreline geometry, bottom friction, and associated effects. The ability to address these site-specific effects reduces uncertainties associated with application of generic empirical equations to site-specific conditions. However, it remains uncertain how accurately the numerical models represent physical processes, especially wave growth and wind setup. Uncertainties in both empirical and numerical modeling methods can result in either overly conservative freeboard requirements, with additional construction costs, or conversely, too small of freeboard for the design conditions with safety implications.

The authors describe an alternate approach to modeling the reservoir response to extreme wind events utilizing Computational Fluid Dynamics (CFD). The approach leverages recent data related to momentum transfer between wind and water under extreme winds. This provides a more robust method of estimating freeboard requirements for off-channel reservoirs subject to hurricane force winds and reduces uncertainties associated with modeling wave growth and wind setup. Wave and water level predictions for a generic off-channel reservoir under hurricane force winds are presented using both the alternate approach presented in this paper and a more traditional numerical modeling approach for comparison.

II.WAVE AND WIND SETUP PREDICTION METHODS

The minimum reservoir freeboard is the vertical difference between the maximum reservoir water surface during a design flood and the embankment crest. The height of the embankment must prevent reservoir wave action from causing intolerable overtopping or damage to the embankment and its components.

For an off-channel reservoir, the downwind water surface elevation is generally determined by the water surface prior to the storm event, the amount of precipitation falling on the reservoir, water evacuated from the reservoir during the storm by operations and passive spillway releases, and wind setup that increased the local water surface. Wind setup is a function of the wind speed, fetch (overwater distance that the wind acts upon), variable water depth over that fetch, and the resistance of the bottom to return flow as the wind piles the water up on the downwind embankment.

Wave action in the reservoir is also directly related to wind. Waves grow by the transfer of momentum from the wind to the water due to shear stresses at the boundary between the wind and the water surface. As the water surface roughens by waves, it affects the boundary layer between wind and water. The waves at any location in the reservoir are a function of the wind speed, fetch, and antecedent wave conditions, which are affected by the wind duration. Other factors that can affect wave conditions include topographic features, reservoir geometry and bathymetry in the reservoir. The topography and reservoir geometry can limit the straight-line fetch and shield some portions of the reservoir from the wind. Bathymetric features can transform waves through processes including refraction/diffraction and breaking of the waves.

Analysis of potential wave overtopping depends on adequate prediction of water levels and wave conditions at the embankment. The following discusses common traditional methods for evaluating wind setup and wave growth in reservoirs, uncertainties, and recent research related to the interaction of winds and water during extreme wind events. It is followed by an alternative approach that reduces some of these uncertainties.

A.Wind Setup

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Common methods of estimating wind setup include simplified equations relating setup to wind speed, fetch, and average water depth as well as more robust and sophisticated methods such as physics-based numerical models.

Simplified equations are often used for wind setup for the types of off-channel reservoirs addressed in this paper due to their relatively simple geometries. These equations essentially balance the forces due to wind on the water surface and bottom friction with the change in water surface between the upwind and downwind ends of the reservoir.

There are several equations that can be used to calculate wind setup in a lake or reservoir. Probably the most widely used equation for US reservoirs is the Zeider Zee equation, described in the USACE Engineering Manual, *Hydrologic Engineering Requirements for Reservoirs* [1]:

$$S = \frac{U^2 F}{1400 D}$$

where:

S = wind setup, feet

U = average windspeed, miles per hour

F = fetch, miles

D = average depth along the fetch line, feet

This equation was developed based on data collected in the Zuiderzee, a large lake in the Netherlands, and is based on an assumed drag coefficient that relates wind velocity to shear stress on the water surface. It is one of three methods presented in [1], the Zeider Zee equation normally results in the highest estimate of wind setup and can result in overly excessive estimates for reservoirs or lakes with shallower water.

This type of approach is essentially a one-dimensional approach which balances the forces along the line of the fetch and represents the bathymetry as a single average value.

Numerical hydrodynamic models represent a more robust approach that can consider the reservoir's hydrodynamics in multiple dimensions. They can allow variations in wind to be modeled over the entire reservoir, model the influence of varying bathymetry on return currents that are generated in response to the wind forcing, allow variations in bottom roughness to be specified, and allow specification of the drag coefficient between the wind and water which will change with wind speed as waves develop in the reservoir.

B.Wave Growth

1) Empirical Equations

Many of the early approaches to estimating waves involved equations largely based on field data collected in the North Sea during the Joint North Sea Wave Project [2]. As with wind setup, these empirical equations are largely a function of wind speed, fetch length, and water depth.

It is noted that the maximum wind velocity for data in [2] was limited to on the order of 34 mph (15 m/s), well below the range of extreme hurricane force winds.

Figure 1 presents plots of predictions of significant wave height (top) and peak wave period (bottom) for wind speeds ranging from 10 to 150 mph from a range of wave growth equations. Predictions in Figure 1 are based on a fetch of 2.5 miles and a water depth of 15 feet. Both wave heights and wave periods impact the amount of overtopping the waves can cause, with higher wave heights and higher wave periods resulting in increased overtopping rates. As shown in Figure 1, the calculated wave heights and wave periods diverge widely for wind speeds associated with a major Category 3 hurricane (wind speeds greater than 110 mph), suggesting a large amount of uncertainty associated with this method when applied to higher winds.

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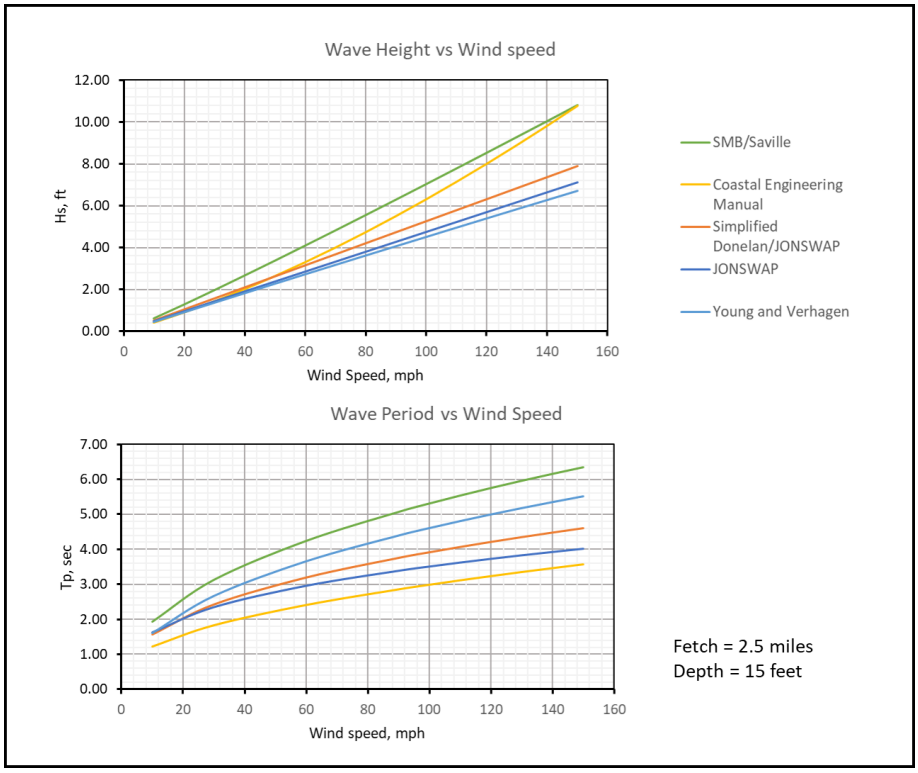


Figure 1. Comparison of Empirical Wave Growth Equations

2) Numerical Wave Models

As with wind setup, numerical models provide advantages over the empirical wave growth equations for estimating waves in reservoirs. Advantages include the ability to model wave growth and transformations, such as refraction/diffraction in response to bathymetric changes, wave breaking in shallow water, and white-capping due to overly steep waves in high winds. Both wave breaking and white capping represent mechanisms that dissipate wave energy and limit wave heights.

Common wave growth models include models developed by the USACE (STWAVE and CMS-Wave), Delft Hydraulics (SWAN), and the Danish Hydraulic Institute (DHI) (MIKE21-SW). These models are spectral wave models that are generally comparable, but with some differences in how they model different mechanisms associated with wave growth and dissipation.

DHI's MIKE21-SW model was used to demonstrate the alternate wave modeling approach proposed in this paper.

3) Wave Modeling Approach

The first step toward modeling wave growth is establishing appropriate wind conditions for the site. Design winds may be based on specific guidance documents or site-specific data that can be analyzed to characterize appropriate extreme event conditions. After the design wind conditions are determined, several adjustments may need to be made for application in wave growth models. These include adjustments for wind duration, elevation above ground level of the wind speed, and differences between wind blowing over land and over water. General requirements and/or guidelines for these adjustments are described below. Simplified guidelines for making these adjustments are described in the Coastal Engineering Manual [3]. These adjustments are typically appropriate for both empirical and numerical modeling approaches.

Duration. Wind data are often reported based on the averaging time the measurements represent. For example, the peak 2-minute average wind over a 10-minute period will be higher than the 10-minute average wind over that period, and the peak 3-second gust will be higher than the peak 2-minute average wind. The averaging time of the wind required for wave modeling is related to the time required for the wave field to develop within the reservoir, which is a function of the wind speed and the fetch it acts over. The duration required to fully develop the wave field can be determined based on methods described in [3].

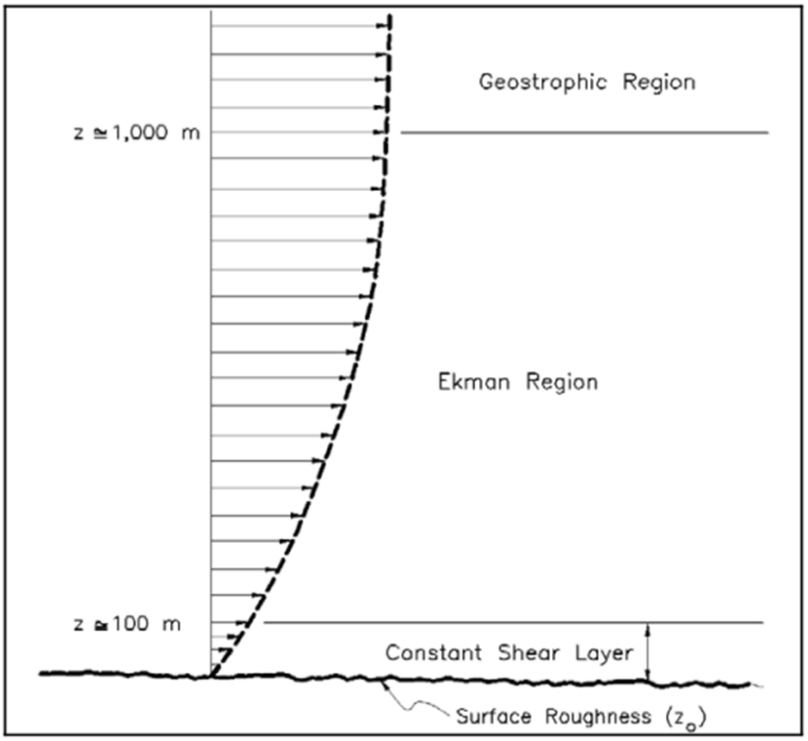
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Elevation. Figure 2 illustrates the change in wind speed throughout the atmospheric boundary layer. The standard measurement height for land-based wind measurements is 10 meters above the ground level and relationships used to model wave growth for the models considered here are based on 10-meter wind speeds. Wind data collected at other elevations can over- or under-estimate the impact of the wind speed on wave growth and must be adjusted to an equivalent 10-meter wind height.

Overland/Overwater. The shape of the constant shear layer shown in Figure 2 is influenced by the roughness of the surface below it and has an influence on the drag/shear stress that the wind exerts on it. The larger the gradient of the wind, the larger the shear between the wind and water will be, resulting in more energy transferred into wave growth. Changes in surface roughness (for example differences between the roughness of the land associated with land-based wind measurements and the surface of the water) will therefore affect the shape of the near-surface wind profile and either result in higher or lower wind speeds over the water.

Figure 3 shows the ratio of overwater to overland wind speeds based on a study of winds in the Great Lakes. As shown in Figure 3, for low wind speeds, the transition from land to water results in an increase in wind speed. For high winds, however, the transition from land to water results in lower wind speeds, likely a result of the increased roughness in the water surface due to waves that develop from the higher winds. The transition of the boundary layer from an overland to a fully developed overwater profile takes place over a distance on the order of about 10 miles. Most off-channel reservoirs are significantly smaller and overwater wind profiles will fall within this transition zone. This transition is complex and USACE Coastal Engineering Manual [3] guidance is to increase wind speeds by 20 percent if the water body is less than 10 miles long, regardless of wind speed.

After appropriate adjustments are made, the adjusted wind speeds can be used to determine design wave conditions within the reservoir, either using simplified empirical equations or through use of numerical models.



From USACE Coastal Engineering Manual [3]

Figure 2. Wind Profile in Atmospheric Boundary Layer.

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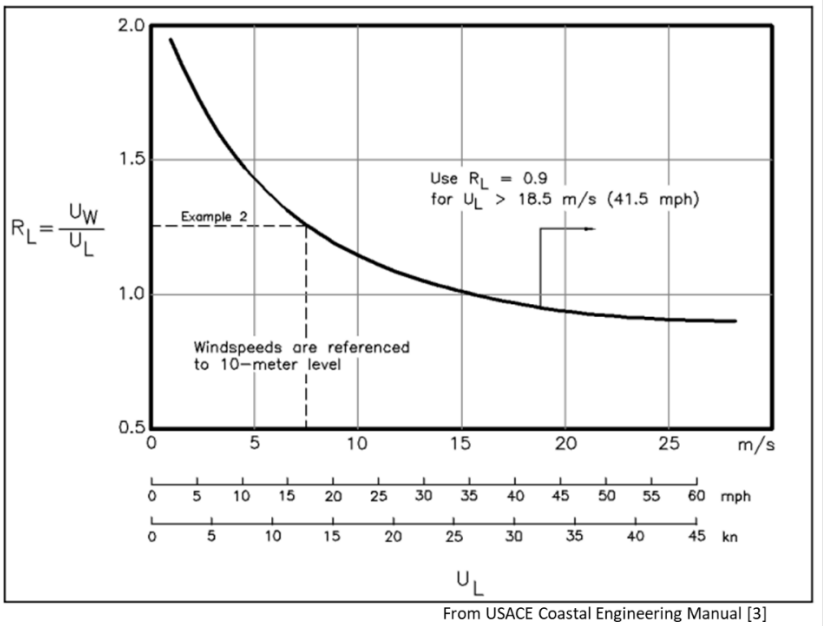


Figure 3. Ratio of Over Water to Overland Windspeed as a Function of Overland Windspeed.

C.Uncertainties

Uncertainties will vary due to the method used for modeling wind setup and wave growth (i.e. empirical equations or numerical models) and how well the adjusted wind represents conditions at the site. Both wind setup and wave growth are sensitive to the wind speed and the shear stress exerted by the wind on the water surface. Some uncertainties related to these and how they are modeled are discussed below.

1) Wind Speed Uncertainties

Adjustments described above to the elevation of the wind measurement to a standard 10-meter height and overland/overwater adjustments recognize the importance of the shape of the wind profile within the constant shear layer of the atmospheric boundary layer on the energy transfer between the wind and the water.

As seen in Figure 3, wind speeds greater than 41.5 mph are assumed to decrease by 10 percent over a distance of about 10 miles. For distances less than 10 miles, the wind profile is considered to be in a transition zone and simplified guidance from [3] is to increase the wind speed by 20 percent. It is unclear how well this speed increase represents the actual conditions for high winds in a typical reservoir.

It is also expected that guidance for these wind adjustments did not envision a water body that is elevated above ground level. The wind speed 10 meters above the water surface will be at a higher elevation than the 10-meter elevation at which the land-based wind is characterized. Just as the wind transitions from its overland profile to an overwater profile, there is likely a zone over which it takes the wind to transition from a 10-meter profile at ground level to a similar profile at the elevated level of the reservoir.

How well the transition of the wind from overland to over the water of an elevated reservoir is represented is a source of uncertainty that can significantly affect subsequent wave growth estimates.

2) Wind/Water Interface Shear Stress

Other uncertainties are related to how well the model represents the transfer of energy from the wind to the waves. Shear stresses between the wind and water surface are typically characterized based on the 10-meter wind speed and an appropriate drag coefficient, C_d. The drag coefficient is a function of the roughness of the water surface. The roughness of the water surface will change due to creation and growth of waves under the influence of wind and, for low to moderate windspeeds, C_d increases with an increase in wind speed.

It is noted that, with the exception of the formulation presented in the USACE Coastal Engineering Manual (CEM), the empirical equations used to generate Figure 1 do not explicitly calculate a drag coefficient. The CEM equation includes a linear variation of C_d with wind speed. The drag coefficient is implicit to the other equations.

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Figure 4, reproduced from [4], presents drag coefficients proposed by various researchers from 1958 to 2015 for a range of windspeeds.

Proposed approaches have varied widely from early proposals of constant drag coefficients, particularly at high wind speeds, to drag coefficients that linearly vary with wind speed. Many proposals provide different calculations dependent on the range in which the wind speed falls. Many researchers concluded that the drag coefficient plateaus at higher wind speeds

Data used for evaluating drag coefficients were generally limited to wind speeds below about 20 to 30 m/s (45 to 67 mph) prior to 1997 due to a lack of data for higher wind speeds.

Beginning in the late 1990s, the United States Tropical Prediction Center (currently the US National Hurricane Center [NHC]) began deploying GPS-based dropwindsondes, which allowed collection of vertical profiles of temperature, pressure, humidity, wind speed and wind direction in the vicinity of hurricanes. These provided datasets that could be used to estimate drag coefficients for significantly higher windspeeds than were previously available.

Evidence from these datasets suggest that not only does the drag coefficient level off at higher winds but may decrease under extreme wind conditions as indicated in Figure 4. The data also suggest that drag coefficients vary within a hurricane depending on which radial sector is considered, and that cross swells can have a significant effect on the drag coefficient [5]. It is noted that cross swell is not an issue within reservoirs and the highest proposed drag coefficient formulations in Figure 4 should not be considered relevant to conditions in reservoirs.

Based on review of aerial films from a collection of hurricanes, [5] attributed extremely low drag coefficients, indicative of a smooth surface, to foam and spray generated from wave breaking under extreme wind conditions. This foam and spray may interfere with shear stress transfer and therefore limit wave growth during extreme wind conditions.

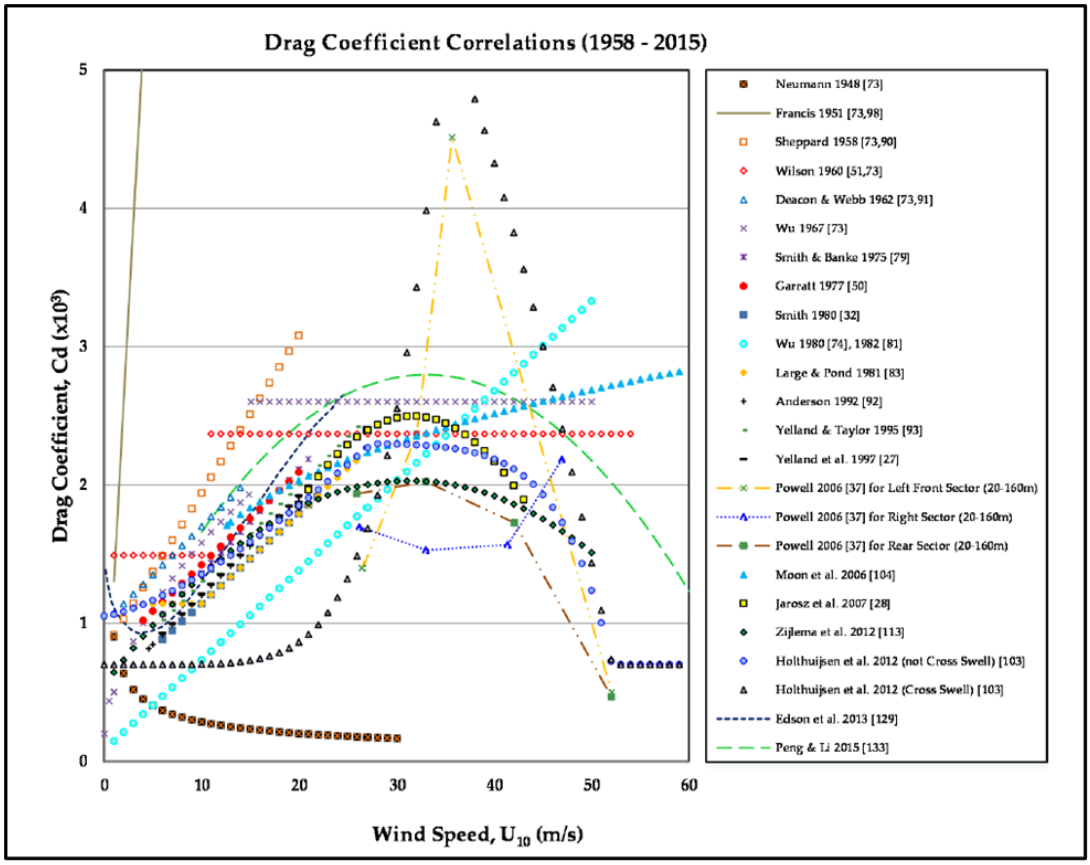


Figure 4. Drag Coefficients Proposed by Various Researchers for a range of Windspeeds (from [4])

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III.PROPOSED MODELING APPROACH TO REDUCE UNCERTAINTIES

The representation of the wind over the reservoir and how the drag coefficient between the wind and the water is represented in a wave growth model are two sources of uncertainty that can have a significant impact on the height and period resulting design waves that are used to establish freeboard requirements and therefore, on the size of the embankment required.

The approach presented in this paper addresses these uncertainties through modeling the wind transition from overland to over the surface of the reservoir using Computational Fluid Dynamics (CFD) to generate a wind map of wind speeds over the reservoir and modeling wave growth in the reservoir with a cap on the drag coefficient between the wind and waves.

This approach is described as it is applied to a generic reservoir (based on a reservoir from a previous project) subject to an overland 10-meter windspeed of 79.4 miles per hour (35.5 meters per second). This equivalent to a 120 mile per hour (53.6 meters per second) 3-second gust adjusted to a one-hour wind speed consistent with the duration required for modeling waves in the reservoir.

The reservoir is approximately 4.4 miles along its longest fetch which is aligned in a direction of about 330 degrees true north. The top of the embankment is approximately 33 feet above the average ground level at the site. The reservoir bathymetry has some variation in the bottom depths with deeper water along the southern end of the reservoir.

A.Wind Modeling

An Atmospheric Boundary Layer (ABL) model was used to model the wind as it flows from overland over the embankment and across the reservoir surface. The ABL model is a three-dimensional (3D) steady-state CFD model that models the air as an incompressible fluid using the simpleFoam solver from openFoam.

The model mesh was defined with 50 cells in the vertical direction distributed along a total height of 500 meters (1640 feet), with the cells at the ground level with 1 meter (3.3 feet) thick, with the thickness increasing with elevation to 40 meters (131 feet) at the top of the model. A constant shear stress, τ, was applied at the top boundary following Guidance from [6].

Following [7], the ground's roughness height, z_0, around the reservoir was defined as 0.03m, which is the value suggested for fallow fields. The roughness height was defined a z_0 of 0.01m for the embankment.

The shear stress between the air and water surface, which transfers momentum from the wind field into waves in the reservoir, is calculated as $\tau = C_D U^2$, in which $C_D$is the coefficient of drag. The coefficient of drag was set to 0.0026, based on findings of [8] and a review of proposed drag coefficient correlations in [4] and is consistent with the value used in the wave modeling described below.

A wind map was generated by converting the shear stress of the wind on the water surface predicted by the model into equivalent 10-meter winds across the reservoir surface for use in the wave generation model. The resulting wind map, shown in Figure 5, resulted in about a 3 percent increase in wind speed above the overland wind speed along the northern edge of the reservoir, building to an increase of about 12 percent at the southeast end of the reservoir and an average increase of about 11 percent. This wind map is considered to be representative of the wind as it transitions from overland to overwater and also as it transitions to a reservoir surface that is perched 33 feet above the ground level.

B.Wave and Wind Setup Modeling

The DHI models MIKE21-SW and MIKE21 HD were used to model waves and wind setup in the reservoir, respectively, using the bathymetry of the generic reservoir and wind map shown in Figure 5 as input.

The drag coefficient internal to MIKE21 increases linearly with wind speed until it reaches a value of 0.0036, and is then capped at that value for larger wind speeds. For the purposes of this paper, the drag coefficient was capped at 0.0026 based on data from [4] shown in Figure 4 and recent findings of [8]. Figure 6 shows the MIKE21 formulation along with the cap used for this paper superimposed on top of the data presented in [4].

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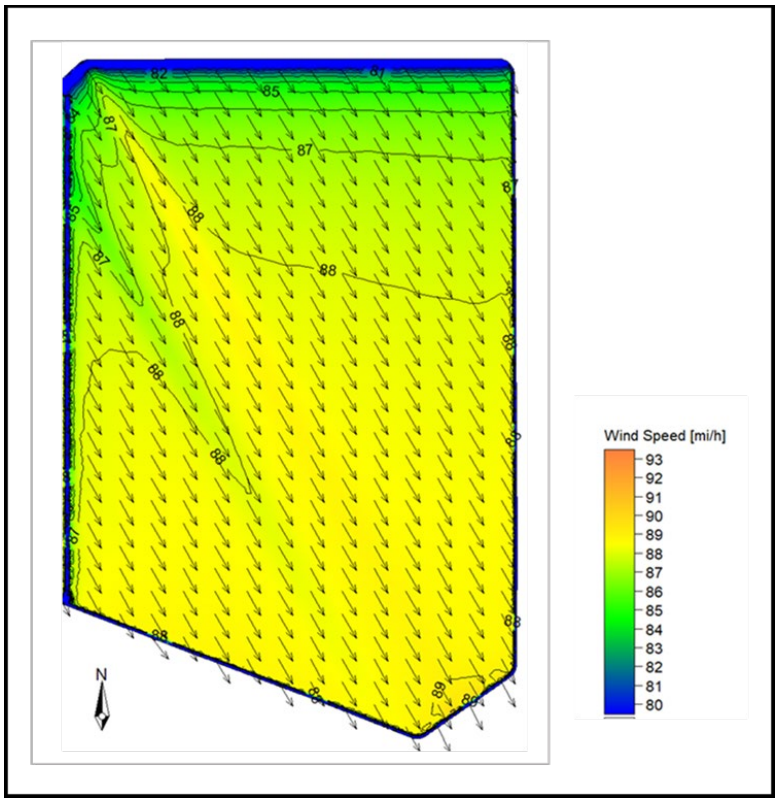


Figure 5. Wind Map

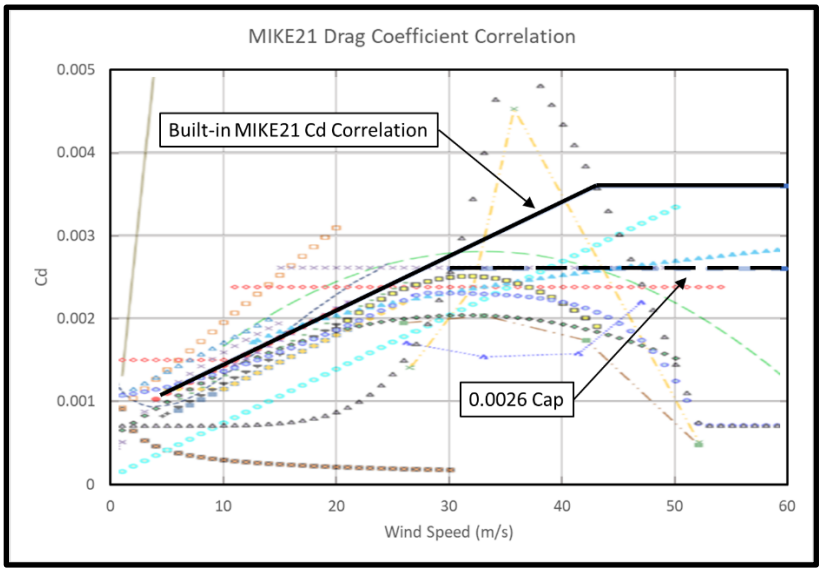


Figure 6. MIKE21 Drag Coefficient Correlation Compared with [4]

Figures 7 and 8 present plots showing results of wave growth and wind setup modeling, respectively for wind blowing from 330 degrees true north. Model results predicted a wind setup of 0.9 feet and waves with a significant wave height of 8.2 feet and a peak period of 4.8 seconds at the southeast corner of the reservoir.

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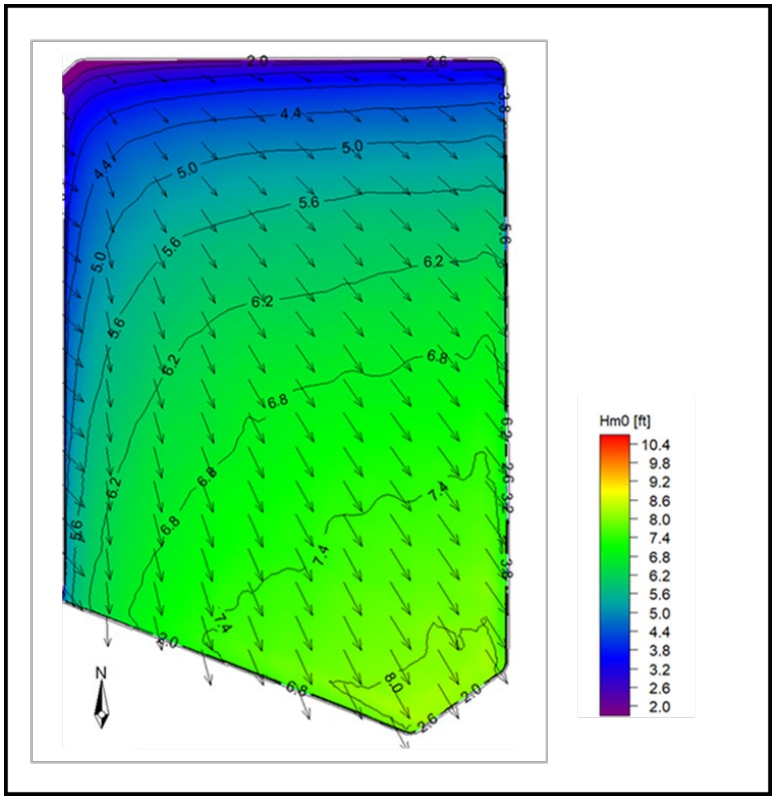


Figure 7. Wave Height Model Results

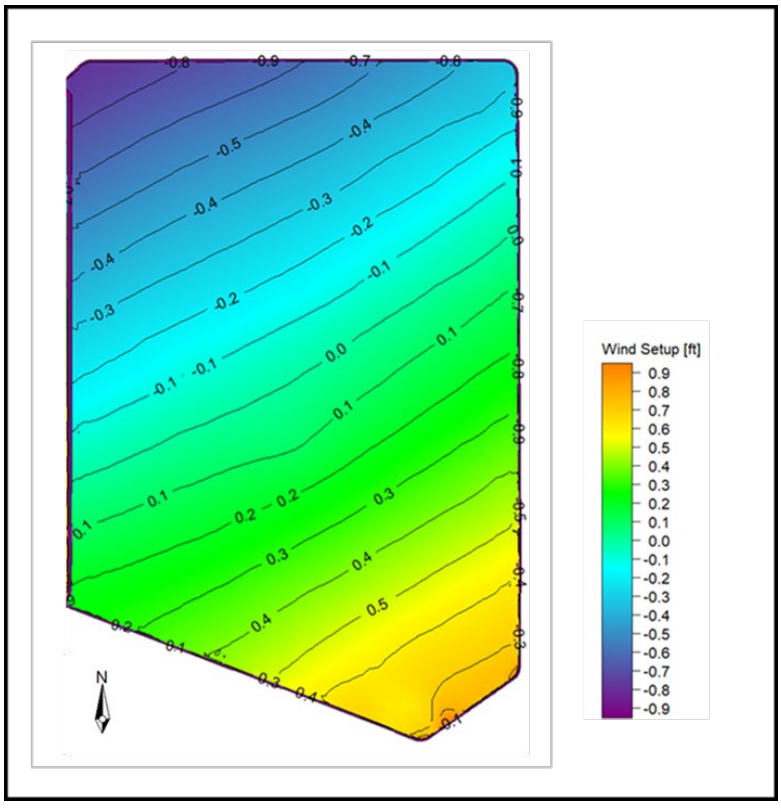


Figure 8. Wind Setup Model Results

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IV.DISCUSSION OF RESULTS

A second analysis was performed using a more typical approach, namely:

- Using an overwater wind speed equal to 1.2 times the overland wind
- Wind setup calculated using the Zieder Zee Equation
- Modeling wave growth using MIKE21 SW with the default formulation for Cd.

The model results are compared in Table I. Results show that the proposed method resulted in a 20 percent reduction in wave height, a 9 percent reduction in wave period and a 25 percent increase in wind setup compared to the more typical approach.

Reductions in wave predictions using the proposed method are due to an approximately 8 percent reduction in the overwater wind speed used and a lower Cd, resulting in less momentum transfer between the wind and waves.

The difference between the two methods compared in Table I would translate into an increase in freeboard of up to 4 feet compared to that required for the proposed method and a significant increase in cost of the reservoir. For example, assuming a 13 mile long, 30-foot-tall perimeter embankment with 3H:1V side slopes and a 20-foot crest width, an extra 4 feet of freeboard would add about 2 million cubic yards of fill to the project. Assuming $40 per cubic yard for in-place, engineered structural fill, the additional cost could run as much as $80 million.

Freeboard determination can be sensitive to the characterization of wind that acts on the reservoir and the characterization of the interaction between the wind and the water surface that is internal to the specific model that is used.

Guidance for simplified adjustments that are commonly applied to wind used in wave growth estimates for a wide variety water bodies likely over-estimates the wind speeds a perched reservoir will see, resulting in a layer of conservativism built into the determination of design waves used to establish the freeboard requirements of the reservoir. CFD modeling of winds as they transition over the surface of the reservoir can provide a more accurate characterization of the wind field that that is used to determine wind setup and wave growth through the reservoir.

Empirical wave growth equations should be used with caution for characterizing wave conditions within a reservoir. Wave predictions can vary widely depending on the equation used. Extrapolating empirical equations for wave growth and wind setup using extreme wind speeds that are outside of the data sets that they were based on can result in additional uncertainties in the results, potentially leading the engineer to err on the conservative side, increasing the costs of the reservoir, or conversely, design a reservoir with too little freeboard, increasing the risks of damage or failure of the reservoir during extreme events.

Physics-based numerical wave growth models provide a means of reducing these uncertainties and a more robust means of simulating unique conditions for each reservoir. These models are sensitive to how the shear stress between the wind and water surface is characterized and standard drag coefficient correlations built into these models may over-predict the energy transfer between wind and waves for extreme hurricane force winds.

TABLE I

Comparison of Traditional vs Proposed Wave Growth Approaches

	Traditional	Proposed	% Difference
Description	MIKE21 SW wave model. Use of Zeiderzee Equation for wind setup. Wind adjustments per USACE Coastal Engineering Manual	MIKE21 SW wave model with cap on Cd. MIKE21 HD for wind setup. CFD modeling to generate a wind map for input to the MIKE21 models.	
Wind Setup (feet)	1.2	0.9	-25%
Wave Height (feet)	10.2	8.2	-20%
Wave Period (sec)	5.25	4.8	-9%

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Breach Evaluation of a Complex 11-Dam System Using Updated PMP with Rain-On-Mesh Hydrology in HEC-RAS

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ABSTRACT

Recent failures of large tailings ponds around the world have resulted in destruction of property and many deaths in recent years. Consequently, one of FTN's clients asked FTN to perform breach analyses for 11 earthen levees/dams in series under both "sunny day" (SD) breach or "Probable Maximum Precipitation" (PMP) conditions. The analysis was complicated by the interaction between the dams under different breach conditions.

FTN used new methodologies to give more accurate runoff results including using updated PMP data with Rain-On-Mesh two-dimensional (2D) modeling afforded by the US Army Corps of Engineers' HEC-RAS modeling software.

Results show that a cascade of failures did not occur as anticipated as a result of upstream dam failures. We show that the impact of the dam failures during the PMP did not increase peak flood elevations downstream and present how this was determined using GIS techniques over the 2D mesh.

I.INTRODUCTION

Several high-profile failures of tailings pond impounded waters at various sites around the world have resulted in destruction of property and many deaths in recent years. Following these events, an FTN client decided to examine the risk of operating and maintaining 20 impoundments at one of their facilities in Arkansas. Consequently, FTN was asked to perform dam breach analyses of a system with 11 earthen levees/dams in series located within the site should one or more of these breach under either "sunny day" (SD) breach or "Probable Maximum Precipitation" (PMP) conditions.

Conventionally, hydrometeorological reports (HMRs) published by NOAA helped engineers determine the PMP based on the climatological regions within the United States. However, conventional PMP estimates do not account for the long-term climatic trends in the precipitation data, and they are based on outdated precipitation records ([1] Abbs 1999). Following recent regime shifts in the climatic records, multiple state agencies have updated the PMP estimation methods considering recent changes in the climate system. In a similar quest, the states of Arkansas, Louisiana, and Mississippi teamed up to update the PMP evaluation method for the region.

The new PMP tool was developed by Applied Weather Associates (AWA) for the Arkansas Department of Natural Resources, Louisiana Department of Transportation & Development, Mississippi Department of Environmental Quality, and Oklahoma Water Resources Board. The PMP tool developed by AWA is hereafter referred to as the "new PMP tool." The new PMP tool is the storm-based approach similar to conventional HMRs. The new PMP tool does not consider any future climate trend as the effect of climate trends on the storm dynamics

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are still not evident to the scientific communities. Climate research is still not clear if the climate trends can potentially change a PMP level storms ([2] Herath et al., 2018). However, the new PMP tool utilizes the revised and updated set of historic extreme rainfall records and is spatially focused over the aforementioned states with unique hydroclimatic geographic properties. These new features prioritize the use of the new PMP tool over other methods to evaluate the dam breach, spillway adequacies, and probable maximation flood.

The Hydrologic Engineering Center's River Analysis System (HEC-RAS) ([3] Brunner 2002) has been consistently upgraded with new capabilities by the US Army Corps of Engineers (USACE) and is widely used in the engineering community for floodplain and dam breach analyses. In addition to the dam breach modelling, HEC-RAS Version 6.0 can perform the hydraulic routing with a more robust finite volume numerical scheme as well as simultaneously calculating the hydrologic losses for two-dimensional (2D) computation cells based on the spatially varying land cover and soil type data. The simultaneous hydrologic and hydraulic analysis is termed as "rain on grid" in the HEC-RAS documentation but that is a misnomer because "grid" implies a system of regular uniform sized cells, yet irregular-sized cells can and are usually used in HEC-RAS producing a "mesh;" consequently, the terminology "rain on mesh (ROM)" is used throughout this document. ROM considers the interactions between rainfall, initial abstractions to rainfall, and the physical features within the model domain, resulting in more accurate inflows to collector and main channels.

We present herein a case study of the application of the ROM method with the new PMP tool within HEC-RAS's 2D modeling capabilities to evaluate breach scenarios in a complex system of interconnected dams.

II.METHODOLOGY

A. Study Area

The client's wastewater treatment system (WWTS) has a collection of 11 interconnected dams that impound water in pits, lakes, reservoirs, and ponds (Figure 1). All the dams are located on the client's property except for Dam 3 which was included in this analysis since it is located in the wastewater treatment system's watershed. Of the 11 different impoundments, there is only one dam that is listed on the USACE National Inventory of Dams (NID) which for the ease of identification is named "Dam 10". None of the 11 dams are in the Arkansas Natural Resources Division Dam Safety registry. All the dams within the property are earthen embankments. These dams are primarily used for stormwater collection and wastewater treatment. (Note that although the Dam 2 impounds water, it does not have an embankment since it is a pit; hence no breach analysis was performed for it).

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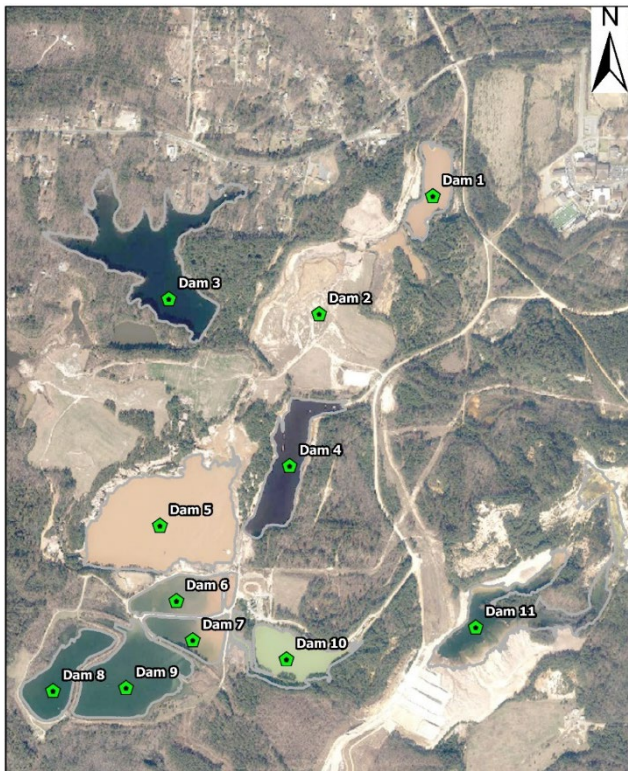


Figure 1. Location of lakes in the system.

B. Data Assimilation

Information about the construction, dam height, and storage volume of the lakes is limited, so assumptions were made during this analysis based on the best available data. The following data sources were used in this analysis: 2017 Ouachita FEMA Region 6 LiDAR terrain data, historical topographic maps, client-provided bathymetry data, pit elevation monitoring data as of 2021 provided by the client, and current and historic aerial imagery.

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As a part of this analysis, storage volume-elevation curves were developed using these sources. To do this, an approximate lakebed terrain surface was developed using a combination of the referenced historical contour maps, latest site survey data, and the LiDAR terrain data. Lakebed elevations for Dam 4 and Dam 10 were assumed to be 10 feet deeper than the LiDAR-based lake surface elevation and assuming 10 percent side slopes since no other bathymetry data were available. (All assumptions were vetted with the client before the analysis began.) The final terrain surfaces with the incorporated lakebed contours included, is shown on Figure 2. The storage-elevation curves for dams were developed using the lakebed-enforced terrain. The top of dam elevations were determined using the 2017 Ouachita FEMA Region 6 LiDAR terrain data.

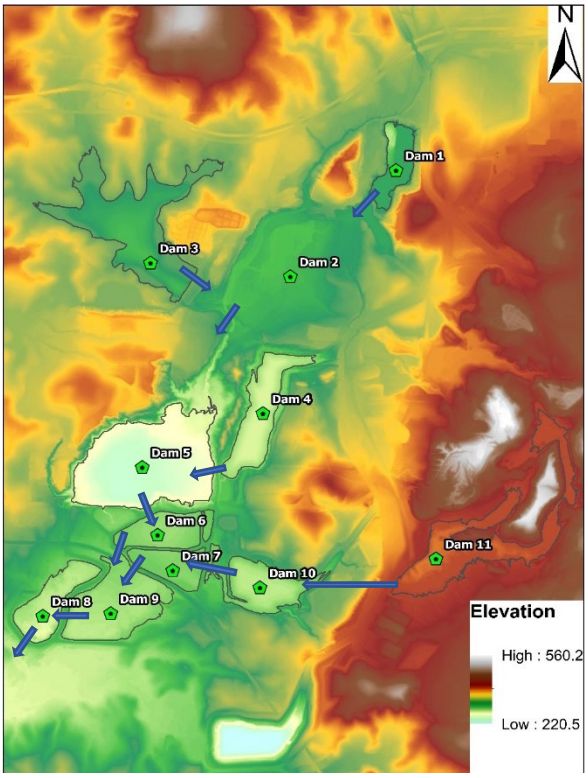


Figure 2. Lakebed terrain surface.

III.HYDROLOGIC ANALYSES

The ROM distributed runoff from rainfall excess over a 2D computational mesh within HEC-RAS was used for this analysis instead of the less accurate and more commonly used lumped (basin-averaged) hydrologic methods. This function applies the precipitation (in this case the PMP), as a time series of rainfall depths to each mesh cell individually, computes abstractions to the rainfall, and then hydraulically routes runoff through the 8.15 sq. mi. watershed and into the impoundments. The 2D unsteady flow routing uses the assigned elevations from the terrain and roughness values that represent the topographic surface and overland flow friction, respectively. The base topographic data consisted of 2017 Ouachita FEMA Region 6 LiDAR, which was used to created 1-meter Digital Elevation Models (DEMs). The DEMs were then used to delineate the watershed basin boundary.

The PMP storm was developed using the precipitation values obtained from the new PMP tool that provides basin-specific critically-stacked temporal accumulation patterns for a specified series of storm durations (e.g., 6-hrs and 72-hrs). The PMP tool produces output for three different types of PMP storms (1) Local - convective afternoon heating showers; (2) Tropical - storms emanating from the Gulf of Mexico; and (3) General - extra-tropical storms associated with the passage of storm fronts.

For this study, a 6-hour duration, 5-minute time interval, critically-stacked temporal rainfall distribution was used to create a precipitation hyetograph for the Local PMP Storm. For the General and Tropical PMP Storms, 72-hour duration, 15-minute time interval critically-stacked temporal rainfall distributions were used to create precipitation hyetographs. Critically stacked/alternating block approximation for the rainfall distribution was used because it

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imbeds the smaller duration precipitation depths within the longer duration precipitation depths, making it a most conservative approximation. To check the basin response while maximizing the rainfall intensities for a shorter duration storm, the most intense 24 hours were extracted and used for the 24-hour General and Tropical PMP storms. The precipitation hyetographs were then used as an input to the two-dimensional (2D) HEC-RAS model. Figure 3 shows the resulting Tropical PMP 24-hour storm distribution which produced the worst-case flooding scenario in this analysis.

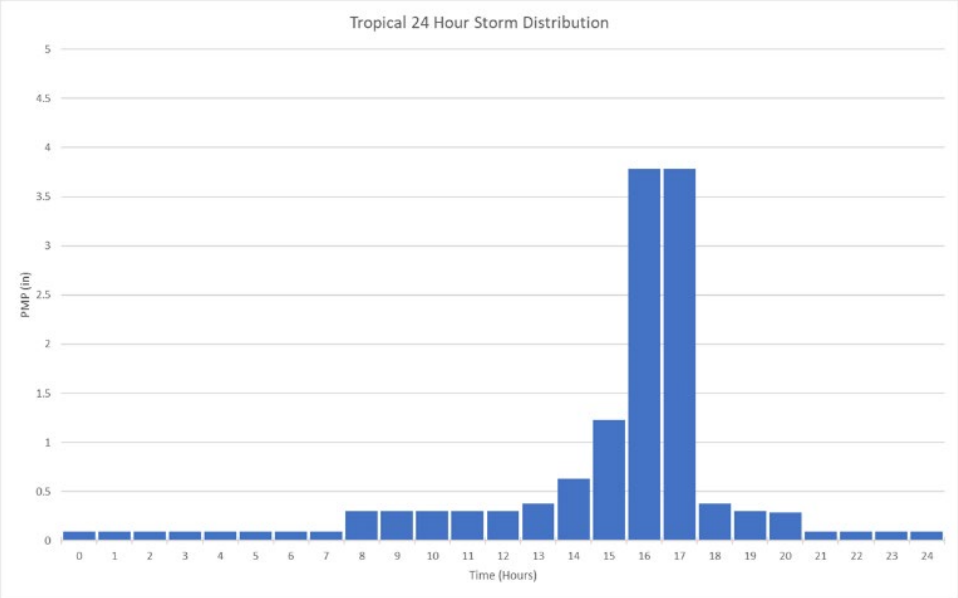


Figure 3. PMP Temporal Distribution.

Since HEC-RAS can now utilize spatially varying land cover and soils data, FTN obtained land use data from the 2016 USGS National Land Cover Database (NLCD) to create a land cover layer with Manning's "n" values, based upon aerial imagery and engineering judgment. Manning's "n" roughness coefficients values used in this analysis are presented in Table I.

Additionally, soils data from the NRCS's Soil Survey Geographic (SSURGO) database were combined with the previously-mentioned land use data to develop an infiltration layer for input to HEC-RAS. For this infiltration layer, FTN utilized the SCS Curve Number (CN) methodology. For this study, Antecedent Moisture Condition Type II (average moisture condition) was assumed. To utilize the CN Method, the Curve Number and Abstraction Ratio were defined in the newly created infiltration layer, which is simply an intersection of the land use and soils layers previously discussed. For this study, an abstraction ratio of 0.2 was used based upon SCS TR-55 guidance.

Table I. Manning's "n" Roughness Coefficients.

Manning's "n" Roughness Coefficient	Description
0.03	Undeveloped, barren land
0.04	Developed, open space
0.05	Undeveloped, grassland
0.06	Open pasture
0.08	Low intensity development
0.1	Medium intensity forested area and medium density development
0.12	Heavy forested area and high-density buildings

IV. HYDRAULIC ANALYSES

A breach analysis was performed to establish the extent of the inundated area should a dam failure occur. HEC-RAS (version 6.0) was used to simulate the breach. This program simulates a breach of the dam and uses 2D unsteady flow principals to route the outflow through the downstream valley via a computational mesh representing the downstream terrain. Since HEC-RAS uses a finite volume solution technique, the elevation-volume

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characteristics of each cell are based on the underlying terrain data for the entire cell, not just the elevations at the cell vertices, as in many other commonly used 2D models (e.g., FLO-2D, RMA-2, TABS-MD, Mike-21). This yields more accurate water surface elevations if the underlying terrain data is sufficiently detailed - as it is with the 2017 Ouachita FEMA Region 6 LiDAR terrain data that was used in this study.

For the dam breach analysis, two separate but connected 2D meshes were used. The upstream mesh was comprised of the watershed upstream of the most downstream WWTS embankment (herein referred to as the "Lakes Basin" mesh). The downstream mesh represents the area downstream of the downstream-most WWTS embankment; the boundary was extended to capture flows from the overflow areas on the north and northwest sides of the treatment system (the "Breach Basin" mesh). The elevation at the downstream end of the 2D meshes (i.e., the downstream boundary condition) was calculated to be the normal depth based on a given slope. Figure 4 depicts the perimeter of both 2D meshes (i.e., the modeling domain) used in this analysis.

The resulting 2D mesh cell sizes were set to be 100 ft by 100 ft with refinement of the mesh (i.e., smaller cell sizes) near breaklines and near breach locations resulting in the lake mesh having 9,964 cells and the downstream mesh having 12,503 cells for a total of 22,467 cells.

All simulations were run for a 96-hour period with a 5-second base timestep with Courant Condition override with a maximum Courant number allowed at any 2D cell set to 1.0 and the minimum set to 0.3. Maximum execution times were 3.9 hours for PMP runs and 1.2 hours for SD runs on a workstation with an AMD Ryzen 9 3900 (3.9 GHz) 12-Core 24-thread Processor with 32 Gb RAM and a solid-state drive for writing results.

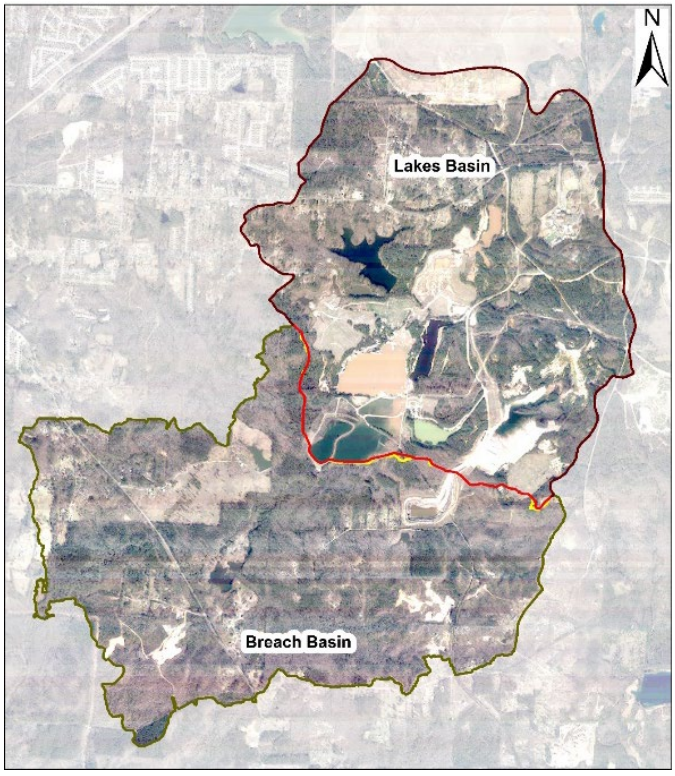


Figure 4. 2D Mesh Perimeters.

It was assumed, for the purpose of this study, that the embankment for Dam 8 and the portion extending eastward to Dam 9 are one entity and were treated as a single embankment structure. The embankment between these two dams was modeled as a connecting weir which does not breach. Embankment structure data for all other dams were also based on the 2017 Ouachita FEMA Region 6 LiDAR terrain data. It was determined that the location of the dam breach would occur at the lowest embankment point on each dam.

For the breach analyses, it was assumed that the impoundment water surface elevations would be at each of their normal pool levels for initial conditions. The water levels of some of these impoundments are primarily controlled by a system of pipes throughout the client's property while others are not controlled in any way. The normal pool elevations were based upon either the maximum elevation found in the impoundment elevation monitoring data provided by the client or to just below the lowest embankment point, whichever was the lowest

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elevation. For dams without observed elevation data, normal pool elevations were assumed to be the elevation captured by the LiDAR data.

A. PMP Breach Analyses

At the start of the simulation, the time series of PMP precipitation was applied to both meshes using the Rain-on-Mesh method in HEC-RAS 6.0. The client's guidelines mandated that the rainfall must be applied to the entire watershed, not just the watershed of the dams in question. The rainfall depths were applied to each mesh cell for each computational timestep and routed through the entire watershed using 2D unsteady flow techniques (i.e., numerical solutions of the 2D St. Venant equations).

To simulate the PMP breach scenarios, all dams were modeled as an overtopping failure when the water surface elevation reached 0.1 ft above the lowest point of the respective dam embankments. Since the 24-hour Tropical storm event produced the worst-case flooding scenario for the PMP event, it is the only PMP event discussed below.

The magnitude of flow from upstream during any PMP event is much larger than that from the dam failures so it is almost impossible to distinguish the effect of the impoundment breaching on downstream flood elevations except near Dam 8 and Dam 9 breach locations. The PMP breach parameters used in this analysis are summarized in Table II and were calculated using the equations developed by [4] Froehlich (2008).

B. Sunny Day Breach Analyses

The Sunny Day Breach Analyses consisted of a simultaneous SD breach and individual SD breaches. For the Simultaneous Sunny Day (SD) breach, all dams were modeled as piping failures that occurred simultaneously 2 hours after the beginning of the simulation. For all scenarios, it was assumed that the starting elevation of each lake was at normal pool elevation as noted above and no other inflows were added in the model. Additionally, for the individual SD breach, dams were individually modeled as piping failures 2 hours after the beginning of the simulation. The individual SD breach was modeled with the assumption that when one dam breaches, no other dam fails unless the outflow from an upstream dam failure causes it to overtop. The SD breach parameters used in this analysis are summarized in Table II and were calculated using the equations developed by [4] Froehlich (2008).

Table II. Dam breach parameters.

Dam Breached	Failure Mechanism	Final Bottom Width (feet)	Final Bottom Elevation, ft (NAVD 88)	Pool Elevation at Breach, ft (NAVD 88)	Lake Volume at Breach (acre-foot)	Side Slopes	Breach Formation Time (hours)
Sunny Day Breach							
Dam 1	Piping	11	316.2	316.6	3.33	0.7	2.36
Dam 3	Piping	41	319.0	326.4	185.8	0.7	0.96
Dam 11	Piping	26	385.0	400.7	86.3	0.7	0.32
Dam 4	Piping	34	296.0	301.4	108.8	0.7	0.75
Dam 5	Piping	18	296.0	296.4	23.7	0.7	0.34
Dam 10	Piping	32	296.0	304.1	112.2	0.7	0.52
Dam 6	Piping	29	296.0	303.4	76.4	0.7	0.54
Dam 7	Piping	19	296.0	300.3	18.6	0.7	0.49
Dam 8	Piping	38	290.0	304.8	186.2	0.7	0.57
Dam 9	Piping	47	287.0	304.8	346.6	0.7	0.65
PMP Breach							
Dam 1	Overtopping	17	316.2	316.8	5.0	1	2.89
Dam 3	Overtopping	60	319.0	328.3	253.9	1	1.12
Dam 11	Overtopping	45	385.0	403.7	184.0	1	0.47
Dam 4	Overtopping	54	296.0	305.1	193.3	1	1.00
Dam 5	Overtopping	81	296.0	305.4	594.9	1	1.69
Dam 10	Overtopping	53	296.0	309.3	213.3	1	0.71
Dam 6	Overtopping	43	296.0	306.5	114.0	1	0.66
Dam 7	Overtopping	29	296.0	301.8	31.1	1	0.63
Dam 8 and Dam 9	Overtopping	62	287.0	305.7	374.5	1	0.67

* Lake volume at breach is the volume of water at breach time i.e., difference in lake volume between Pool Elevation at Failure and Breach Bottom Elevation.

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V. Results

The results of the downstream flood routing were used to establish the limits and determine the areas inundated by the dam breach outflow for emergency conditions associated with the dam breach scenarios. The map shown on Figure 5 depicts the areas that would be flooded for the three SD scenarios (Dam 8, Dam 9, and Simultaneous SD). Only the plans that yielded maximum flooded areas were included for the results. The aim of demonstrating only the SD failures of Dam 8 and 9 is because they lie at the outlet end of the dam system where all the accumulated flow will pass through it, including the flow through the two dams themselves. The simultaneous SD breach was largely governed by the severity of the breach event and the piping failure of the dams at the boundary of the breach basin and the lakes basin. The PMP breach was included as it is the only event with regards to the precipitation and yields a significant flooded area compared to any other events.

During the PMP breach scenario, different dams breached at different times, but two significant flood waves were observed in the simulation. The first flood wave occurred when Dam 11 breached approximately 13.1 hours after starting the simulation. Its breaching caused a cascading series of dam breaches downstream of it: Dam 10 breached approximately 36 minutes after Dam 11 breached, and finally breaching Dam 9 approximately 15 minutes following that. The resulting first flood wave reaches the downstream limit of the model approximately 112 minutes after the breaching of Dam 11. The second flood wave occurred as the result of runoff from PMP rainfall in all parts of the basin. This flood wave reached the downstream limit of the model approximately 17 hours after starting the simulation (60 minutes after the peak of the PMP rainfall).

No individual dam SD breach outflows passed the Dam 8 & Dam 9 embankments except for the Dam 8 & Dam 9 breach scenarios. The SD simultaneous failure of all dams' results in the largest inundation area, making it the worst-case scenario of the SD breaches. Inundation caused by the SD event is within the 100yr FEMA floodplain.

Since the PMP event in the valley below the impoundments causes so much flooding on its own, it is difficult to see the impact of the breaches on water surface elevations there. In order to show the increases due to just the impoundment breaches during the PMP event, the differences in water surface elevations between With Dam Breach and Without Dam Breach scenarios were mapped for each cell in the mesh (Figure 6). This required that the simulation of the PMP Without Dam Breach scenario be done.

The PMP inundation areas for the worst-case PMP event with Dam Breach can be seen on Figure 7. The SD Simultaneous breach inundation areas, water depth, and water surface elevations are presented in Figures 8-10.

The net inundation effect due to the PMP breach was minimal since the volume of water leaving the dams caused little change in water surface elevation over the downstream area already flooded by upstream flows and local runoff. The only event that produced a pronounced effect downstream was the simultaneous SD breach. The simultaneous SD breach reported a maximum depth of 5.4 feet at 800 feet downstream of Dam 9, where the initial flood wave reaches 11 minutes after the breach and peaks 41 minutes after the time of breach. Likewise, at 2,000 feet downstream of Dam 9, the maximum flood depth reported is 5.6 feet with the initial flood wave arrival time at 23 minutes and peaks 50 minutes after the time of breach.

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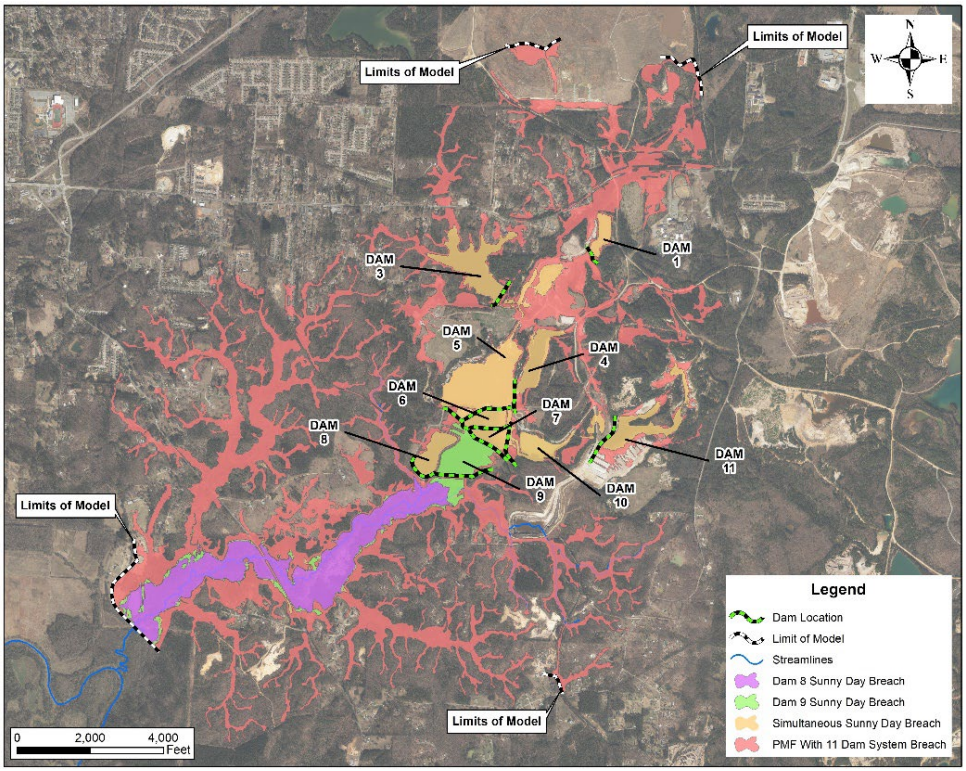


Figure 5. Flood inundation map for all the scenarios. (Note: “PMF” is the Probable Maximum Flood resulting from the PMP.).

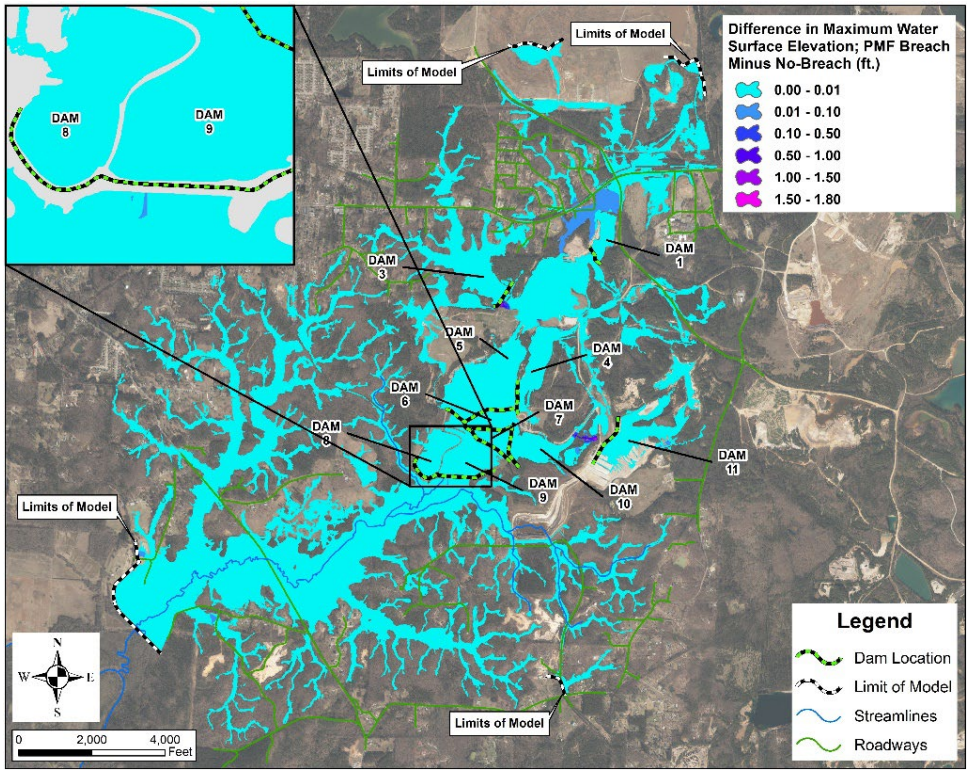


Figure 6. Difference in maximum water surface elevations between With and Without breach scenarios.

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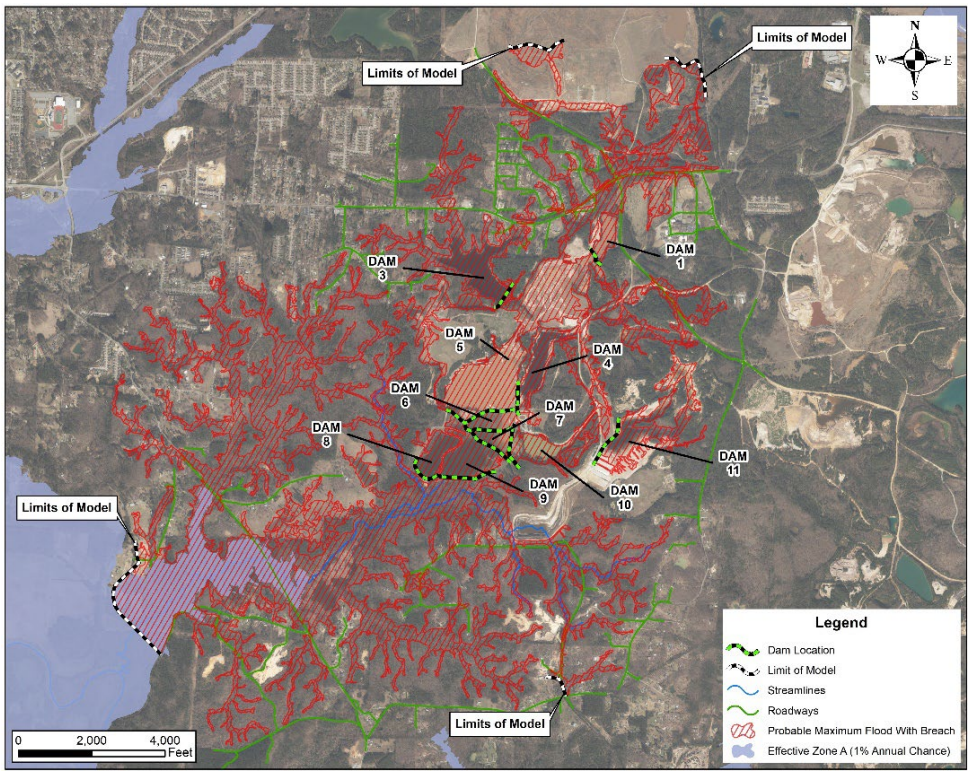


Figure 7. PMP inundation areas for the worst-case PMP event with Dam Breach.

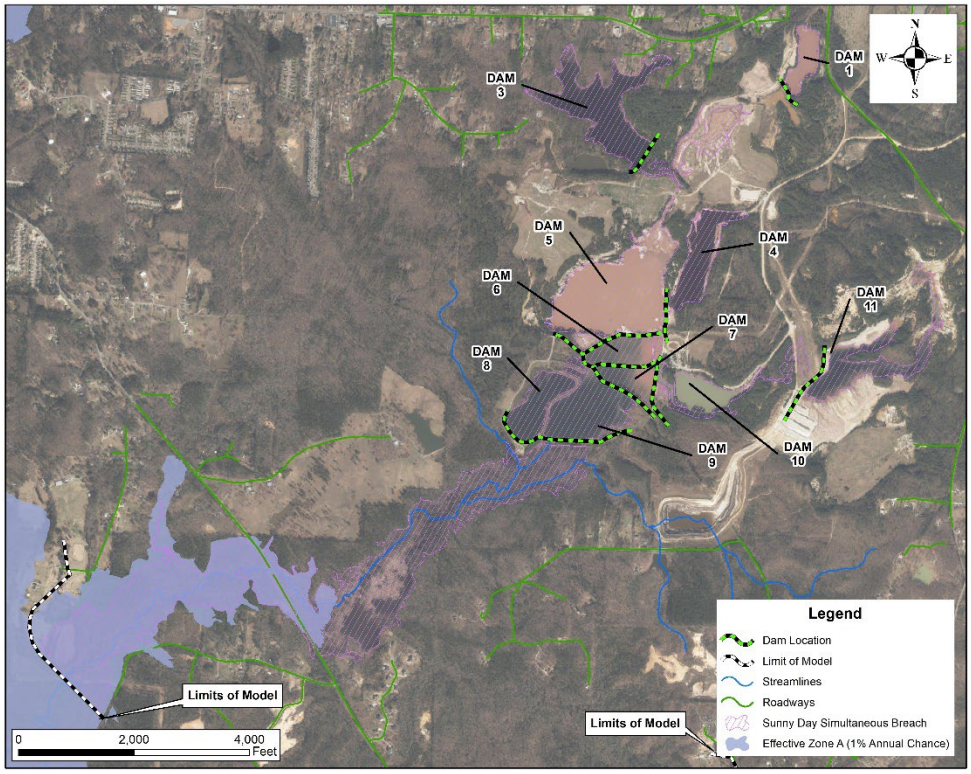


Figure 8. Inundation areas for the simultaneous sunny day breach.

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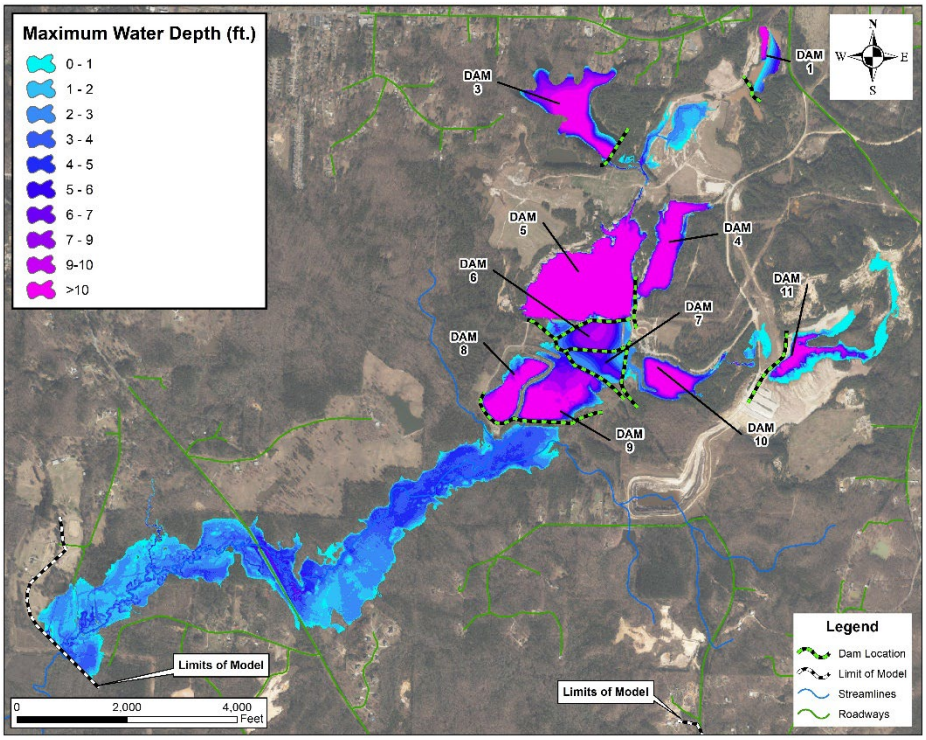


Figure 9. Simultaneous Sunny Day Breach Depth.

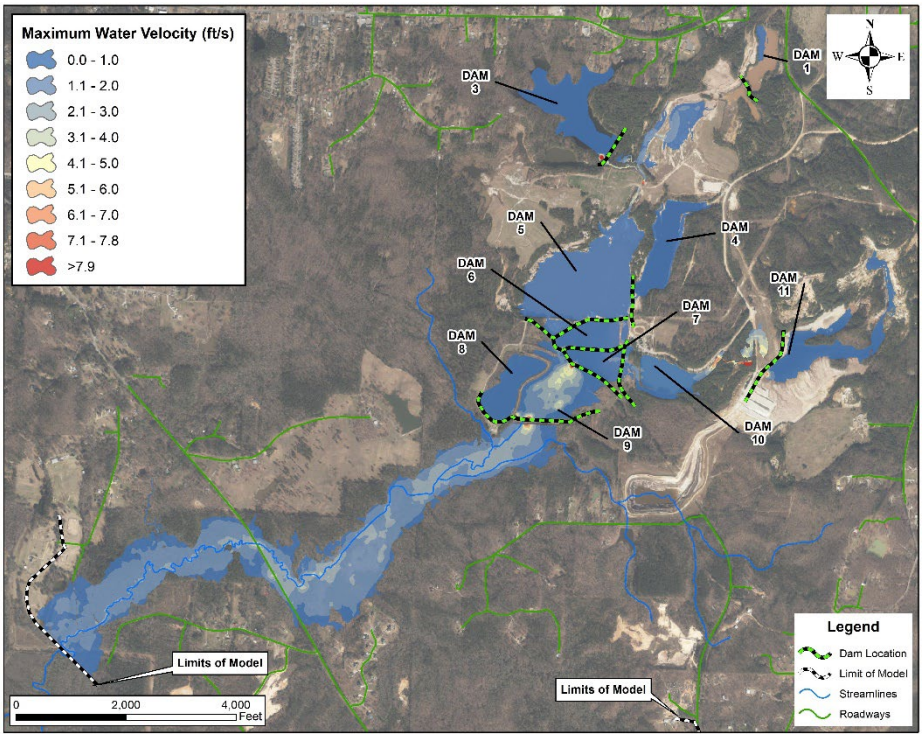


Figure 10. Simultaneous Sunny Day Breach Velocities.

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5 Summary

The complex dam system consisting of 11 dams showed a negligible change in flood depth, flood velocity and extent should the breach occur during a PMP event. Few structures downstream are affected by the dam breach in the WWTS - they are flooded during a SD breach event and are already in a FEMA 100-year floodplain. There are no structures affected by any PMP breach that are not already flooded by the PMP flood itself. The key components of the current study are as follows:

1. The dam breach analysis of a complex 11 dam system was performed. The cascade failures were conceptually modeled with the combination of different simulation plans.
2. The dam breach caused by the piping failure with the ponding at normal water level was modeled as SD breach simulation plans, while the dam breach caused by the PMP events were modeled as an overtopping breach simulation plans.
3. The simultaneous hydrologic and hydraulic computations were performed using 2D ROM feature of the HEC-RAS to get more accurate results by considering the hydraulics of the physical features at the scale of 2D computation cell compared to the typical hydrologic methods that use basin-averaged parameters.
4. Spatially varying land cover and soil type gridded data helped in modeling the hydrologic losses for each 2D computation cells unlike a lumped hydrologic model.
5. The use of the new PMP tool helped in accounting for recent extreme rainfall records within the climate region while performing the hydrometeorological computations.
6. All three possible storm configurations (i) Local storms caused by convectional heating; (ii) Tropical storms emanating from the Gulf of Mexico; and (iii) general - extratropical storms associated with the passage of storm fronts were considered within the PMP computation. Further, stacked/alternating block approximation for the synthetic rainfall distribution helped in considering the worst scenario for the possible dam breach.
7. The client was relieved to learn that none of the breach scenarios affected any homes or other structures and the impacts were minimal downstream.

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AR, KY, LA, MS, OK, SC, and TN. He is currently assisting the Arkansas Dam Safety program with their mission.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

Mississippi Rehabilitation Grant Efforts

WILLIAM MCKERCHER AND MATT IBURG, MS DEPARTMENT OF ENVIRONMENTAL QUALITY

ABSTRACT

The State of Mississippi received a FEMA HHPD Assessment grant in 2019 and a Rehabilitation Construction grant in 2020. In addition, the Mississippi Legislature created the Mississippi Dam Safety Fund in 2020, a state funded grant program to assist high hazard dam owners in Mississippi with the cost of rehabilitation to ensure compliance with state regulations. To date, the State of Mississippi has allocated $4,000,000.00 in funding through this program to High hazard dam owners to assist in rehabilitation efforts. As part of this presentation, Mississippi Dam Safety staff will describe the following:- the framework for the project selection through the FEMA HHPD program - the framework for how our agency handles the Administration of the FEMA HHPD program and the Mississippi Dam Safety Fund- present case studies for the FEMA HHPD projects addressed - 2019 Ross Barnett Reservoir Dam and Black Creek Cooling Water Lake Dam Assessments and the 2020 Rehabilitation of Lynn Cartlage Lake Dam- present case studies for a select number of the 28 dams addressed through the Mississippi Dam Safety FundThis session will provide information to the participants regarding the implementation of the two programs and provide useful information and resources for regulatory staff that may be tasked with establishing similar programs in their own respective states.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

Oklahoma's Innovative Solutions to Funding Dam Safety Projects

EMMA MORADI AND NATALIE ORBESEN, OKLAHOMA WATER RESOURCES BOARD

ABSTRACT

According to the Association of State Dam Safety Officials the average age of dams in the United Sates is 61 years and the estimated total cost to rehabilitate the nation's non-federal dams exceeds $65 billion. As dams age beyond their intended design life, the need for dam rehabilitation projects continues to increase along with the costs. Meanwhile, the availability and access to funding for these projects poses many challenges for non-federal dam owners throughout the United States.The Oklahoma Water Resources Board (OWRB) dam safety program and financial assistance program have successfully paired several different State and Federal funding sources to complete dam rehabilitation projects. Over $30 million in loans and grants have been awarded to qualified dam owners through a combination of funding sources including the Environmental Protection Agency's Clean Water State Revolving Fund; the Federal Emergency Management Agency's High Hazard Potential Dam grant program; and OWRB's Financial Assistance loan program, Rural Economic Action Plan grant program, and Emergency grant program. The Oklahoma legislature also appropriated OWRB $10 million through the US Treasury's American Rescue Plan Act, to allocate to dam owners for rehabilitation of poor and unsatisfactory condition high hazard potential dams. These available funds have eased dam owners' struggle to improve and maintain the safety of their dams. Maintaining dam infrastructure is essential for public health and welfare. The collaborative effort and non-traditional approach by OWRB to identify funding opportunities for dam rehabilitation projects has improved the safety of the dams in Oklahoma while protecting lives and property. The OWRB encourages other state dam safety programs to promote the establishment of such innovative practices and hopes sharing their experience will facilitate programs to identify similar possibilities and creative approaches.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

The Corps Water Infrastructure Financing Program - A Solution to Making Water Infrastructure Affordable

KEVIN MCDONALD, US ARMY CORPS OF ENGINEERS

ABSTRACT

The US Army Corps of Engineers is offering a new Federal loan program for non-federal dam owners. The Corps Water Infrastructure Financing Program (CWIFP) is a Federal loan program administered by the Corps that provides low-cost, long-term loans for maintaining, upgrading and repairing dams for any non-Federal borrowers. The program is open to projects, or bundles of projects that have a total cost of $20M or more. The program will be overseen by a new organization within the Corps that is under the direction of Corps headquarters. $7.5B+ is available and the program's first call for funding requests from prospective applicants is expected in Spring 2023. Borrowers can realize savings of more than 20% over other financing options. Here are some of the benefits of using CWIFP over other financing options:• Low Interest• Consolidation of multiple projects into one loan• Financing previously incurred costs for planning and design • Long-term amortization (up to 35 years following construction)• Flexible repayment terms with the opportunity for capitalized interest• Sculpted principal repayment

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

Owner's Perspective: Addressing Seepage issues during Dams Normal Operation

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ABSTRACT

Eagle Mountain Lake, a major water supply reservoir for Fort Worth, Texas had experienced chronic seepage issues for decades. A five-year preconstruction study collected site data through geotechnical borings, lab testing, topographical surveys and field instrumentation. Four years of monitoring the site helped with understanding the groundwater regimen in relation to lake level variations along with a comprehensive alternative analysis determined a Cutter Soil Mixing (CSM) Wall would be the preferred seepage cutoff system. The 250-ft wall alignment and technical specifications were developed and the contract was awarded in February 2022. This paper will focus on the challenges encountered during construction, including conflicts with existing instrumentation, narrow windows for navigation, steady QA control and presence of existing fill material. This project's lessons in addressing long-developing problems through careful coordination can be applied to dam repairs large and small.

I. INTRODUCTION

Eagle Mountain Lake is located on the West Fork of the Trinity River, about 12 miles northwest of downtown Fort Worth, Texas and owned by Tarrant Regional Water District (TRWD). The original project elements were designed in the late 1920s and the dam was constructed from 1931 to 1932. Since the 1950s, a wet area has been noted downstream of the embankment, which has created safety and maintenance concerns. Figure 1 shows the general location of the area of concern. Seepage through the embankment was noticed in 1970 during construction of the side channel spillway and recently as shown in Figure 2 and Figure 3, respectively.

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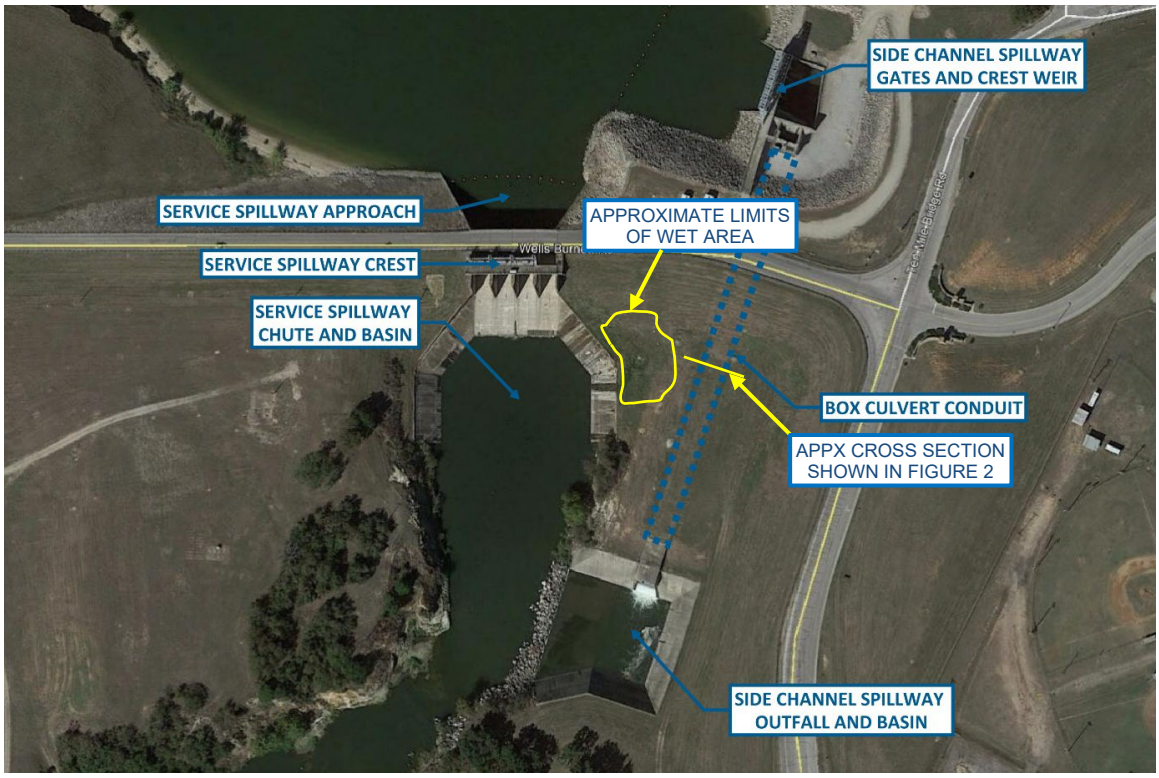


Figure 1: Embankment and Spillway Layout



Figure 2: Seepage from Holes Upper Packsand - Looking Upstream (1970)

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Figure 3: Seepage through the Embankment, Existing Downstream Face

Previous studies also identified potential safety concerns related to internal erosion and overall slope stability. A larger concern was the presence of an unfiltered seepage exit where the wet area is blanketed with a thin layer of clay near the base of the slope. Damage or cracking of this clay layer could allow for internal erosion/piping to initiate and adds credibility to a piping failure mode. This is evidenced by the formation of a boil during a test pit excavation on a 2017 study performed by Freese and Nichols, Inc. (FNI).

FNI came up with a phased approach to comprehensively study this concern and determine a process to address the issue. FNI’s study efforts began in 2012 and continued through 2021. To develop the design, a combination of field investigation, instrumentation and laboratory testing was performed and data was collected. In addition to processing the collected data, FNI developed seepage models and conducted a comparative analysis to determine the most efficient system to control seepage at the previously described location, the Eagle Mountain Dam. The multi-phase engineering and construction effort was divided into three phases:

• Phase 1: Initial Study & Alternatives Analysis (2018-Early 2020) consisted of site data collection, surveying, field instrumentation and modeling to understand groundwater variations, soil conditions and causes of seepage.

• Phase 2: Design (End of 2020-Fall 2021) included comparative analysis of several design options for the preferred seepage cutoff and ultimate selection of a Cutter Soil Mixing (CSM) wall system as the design solution.

• Phase 3: Construction (March-June 2022). The bidding and contract award processes were held in Fall 2021 and the contract was awarded to Bauer Foundation (BFC) Inc in November 2021. The construction entailed drilling and installation of 45 panels to comprise the approximate 260 linear feet (LF) CSM wall system as the seepage solution, plus onsite resident representative services. The project substantial completion date was scheduled on June 14, 2022 and actual completion date was June 22, 2022. (Bauer was granted an 11 work-day extension due to cement shortage created by supply chain issues)

The first two phases of this project were extensively discussed in a paper submitted to the United States Society on Dams 2022 conference (Verreault, et al., 2022). This paper focuses on challenges encountered during construction and the approaches taken to address issues.

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II. CONSTRUCTION SEQUENCE

A CSM wall is an in-situ soil improvement technique in which two sets of counter rotating, vertically mounted cutter wheels mix the soil with a cement slurry to form soil cement panels. The additives are used mainly for stabilizing soft or loose soils or environmental containment. The cutter head mixes soil with additives during descent and ascent from the desired depth, forming one of the 45 panels of the CSM wall for a total of 260 LF.

CSM walls are formed through a series of overlapping primary and secondary panels, as shown in Figure 4. First, Bauer drilled a primary panel, then moved the rig by at least one panel length to construct a second primary panel, continuing the procedure for approximately four primary panels to create stability before constructing secondary panels that overlap primary panels by at least 8 inches, creating a continuous wall.

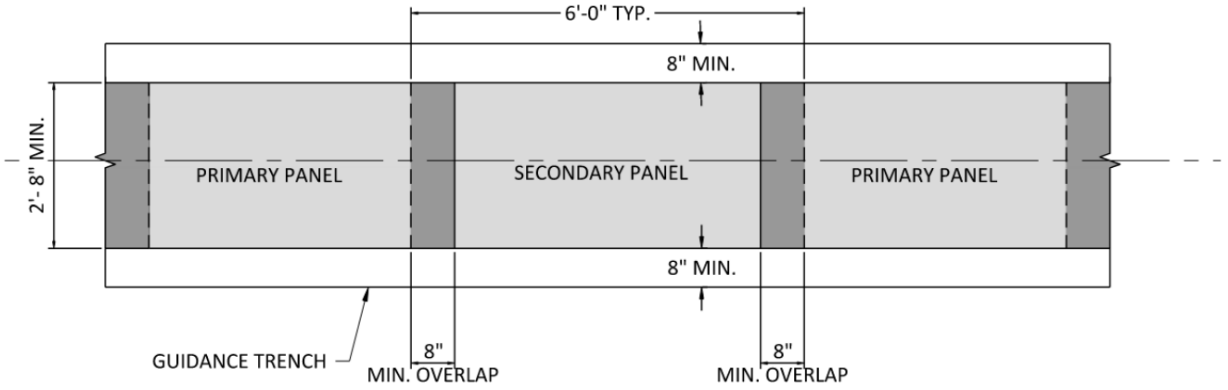


Figure 4: Panel positioning for installation

III. IDENTIFYING CONSTRUCTION ISSUES

Numerous construction considerations led to the project's successful completion. The effort started with studying possible issues to address prior to construction, defining dynamic roles during construction, setting up communication corridors as well as flexibility and innovative approaches to address unpredicted issues. This task started even during development of the drawings and carried through the completion of the construction. Some of the issues identified, encountered and addressed are discussed below.

1) Determining alignment

The alignment needed to be defined in an east-west direction, on either north or south of Wells Burnett Road which forms the crest of the dam. The area to the south of the road slopes down with a steepness of 3H:1V, which is unsafe for the heavy equipment to work on; therefore, the alignment had to remain north of the road. The north face of the embankment was populated with many existing conduits, instruments and buildings that needed protecting. The available above grade space to work was limited and heavy equipment were not allowed to be present on certain parts due to safety concerns. The effort to find a solution started even before the contract was awarded. After multiple site visits and coordination, the following factors were identified and showed in Figure 5:

1. Limits of riprap were too steep to approach. A buffer zone of 5 feet (ft) was defined as a precaution
2. Limits of side channel spillway were identified per as built drawings; a buffer zone of 5 ft was determined as a precaution as to not damage the structural integrity of the conduit. The side channel spillway was equipped with drains alongside the walls, so any seepage occurring within the buffer zone would be collected.
3. Wells Burnet Road right of way. Early in design, the team was aware that a main factor later in construction would be roadway access above the dam and adjacent to the alignment for first responders and school buses in neighboring communities. This constraint is discussed in more detail, but ultimately the road needed to stay open with a minimum of one lane open at all times. This decision also meant traffic control plans had to be coordinated and approved by the county.
4. Instrumentation sheds and vaults. The two sheds above ground and a shed below ground housed many communication and safety wiring and had to be protected from damages. In addition, the vault and the sheds created a very narrow pathway for the alignment with a width of approximately 13 ft at the narrowest

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section. The vault was located very close to the road right of way with a distance of approximately 10 ft, preventing the alignment to be placed between the vault and the road.

5. Desire to maintain functionality of existing piezometers during construction in order to monitor groundwater levels real time.

   Various lines (marked as orange lines) helped with the dam's main operations, such as gate operations, security cameras, electrical, fiber optics and communication lines. Considering all the previously mentioned restrains, the lines would inevitably interfere with the only possible location for the alignment. Therefore, new conduits were placed prior to the construction phase and new electrical and communication lines were rerouted south of the instrumentation vault. The new conduit was staked and marked post completion to indicate its presence and for the construction equipment to retain a safe distance. The final alignment is presented in Figure 5.

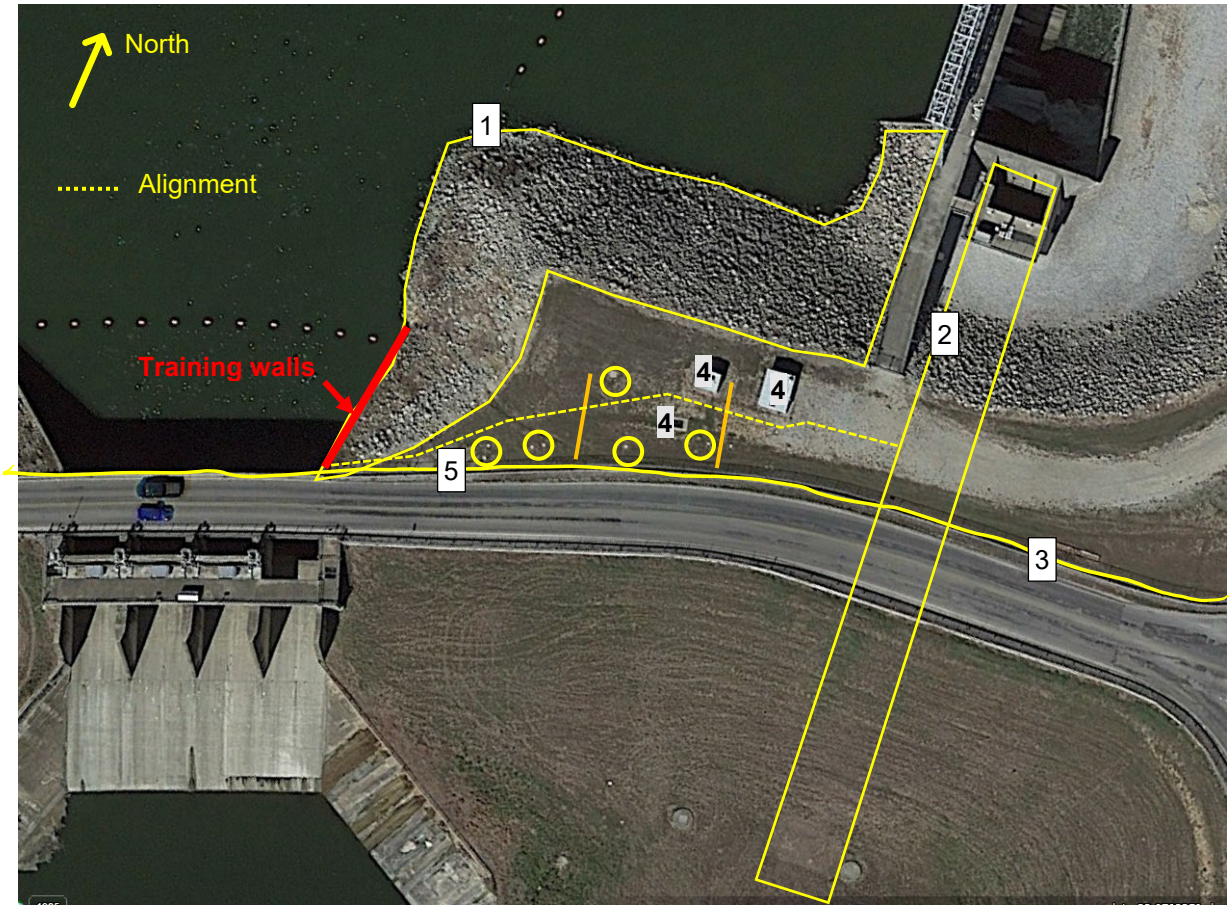


Figure 5: Present Site Limitations and the Finalized Alignment

2) Deposition and staging areas

One of the factors considered in selecting the appropriate seepage mitigation methods was minimum generation of spoils. CSM technology uses in-situ material for construction, ultimately resulting in minimum spoils. However, some spoils are inevitable. In addition to an area needed for storing spoils, a staging area was needed for the contractor's various uses, including a field office, temporary storage of construction materials, assembly of various construction equipment, vehicle parking, etc. While this area needs to be closer to the construction site, the spoil depot area can be some distance away. In case of Eagle Mountain Lake, an area immediately north of the side channel spillway was provided to the contractor for staging. Another area to the south of the channel was specified for deposal of spoils generated during construction.

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3) Road closure

Wells Burnet Road crosses the Eagle Mountain Dam and serves as the only access way connecting two major thoroughfares, Boat Club Road on the east and HW199 on the west. In order to avoid disturbing daily traffic for residents and provide proper access to the site for first responders and the construction team, it was decided that the road should remain with at least one lane open at all times. This worked for most of the alignment process, but challenges were encountered towards the end of the project where narrow spacing between the riprap and the road fence prevented the CSM equipment from being positioned outside the road's right of way. Several options were considered to address the situation, including alignment revisions and changing the construction for a short distance to a method needing smaller equipment. The solution ended up being a combination of these two. For continuity purposes, CSM remained as the method of choice, but the alignment had to be moved toward the road in order to avoid working near the steep riprap slope. This meant a lane of the road had to be closed, which if done during the day, needed a comprehensive traffic control plan and coordination with the County. Therefore, the work schedule was changed to night shift for a total of three (3) nights to complete the panels with minimal disruption to traffic. This change in traffic schedule was announced to the public and then carried out according to the schedule. Figure 6 shows the work being performed during nighttime.



Figure 6: Night Shifts

4) Material shortage

The construction team faced severe material shortages caused by global supply chain issues in April 2022. This challenged the team in obtaining cement and bentonite powder in a timely manner. During the shortage, the construction equipment were idle on site and since the crew were based in Florida and had flown to Texas, meant additional expenses imposed on the Contractor. The team took numerous steps to successfully mitigate and control the issue and to continue with the original schedule as closely as possible. These steps included:

- Requisition of material from different suppliers
- Advanced payments from the Contractor to suppliers to guarantee material delivery
- Re-evaluating mix design to use a slightly different cement (IL type), which was available locally.

The team also scheduled personnel rotations and adjusted panel construction sequence based on available material. Continuous communication between TRWD`, FNI and Bauer was key to successful project completion in terms of safety, quality, logistics, subcontractor coordination and planning.

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5) Existing fill material

A good portion of the Eagle Mountain Dam was originally constructed by raising the grade from the existing grade at the time. Although FNI had some borings and plans from dam modifications in 1970s, as well as original drawings from 1923, the team still was not sure what they would encounter during installation. This is more pronounced when the construction is happening within uncontrolled fill material, especially older fills placed without any engineering supervision. In this case, the construction team encountered prior construction debris or buried riprap within the original embankment fill, which made above ground alignment adjustments a key factor for the team to keep in mind. Not only these materials were hard to process and stabilize, they also caused the hydraulic engine to lose pressure and ultimately stop working. This was a dynamic issue and had to be monitored and addressed continuously.

During construction of one CSM panel, old construction debris was encountered 10-15 feet above the target depth and prevented advancement of the panel to its terminal depth, as shown in Figure 7. The alignment at these locations was slightly moved to work around the obstruction. During this alignment change, the minimum overlap between the panels was still maintained to block seepage effectively.



Figure 7. Historic construction debris (left) and wood obstruction (right)

6) Developing and maintaining QA/QC program

To ensure proper construction, the specifications were developed through research and site specific evaluations. The items mainly considered included:

1. Position tolerance: The following tolerances applied to the CSM panel dimensions and construction.

   Alignment: ± 2 inches of design location measured at the top of the panel

   Verticality: 0.67% of vertical or 150V:1H measured both parallel and normal to centerline

2. Sampling (wet grab and coring): the Contractor was required to grab samples immediately post-placement to be tested in the lab, mainly for permeability. The wet-grab samples were required in at least once every five panels at varying depths. Once the panels were set, continuous cores for testing were required in a minimum of three locations along the CSM alignment. The cores were also sent to the lab for analysis. The permeability of the samples had to remain less than 10^{-7} cm/s. An image of a core taken from the completed panels is shown in Figure 8.

3. Independent lab testing: the samples had to be tested in an independent laboratory complying with the American Council of Independent Laboratories (ACIL).

4. Visual imaging and documentation: Following completion of the coring, the Contractor was required to use an Optical Borehole Imager to inspect the side walls of the core hole. Prior to imaging, the sides of the core hole had to be flushed with clear water. The images and recordings were submitted to the Engineer for their records and evaluation.

5. Presence of a FNI representative on site to observe, document and report construction progress and ensure its adherence to the specifications

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6. Daily report requirements from both contractor team as well as FNI representative, weekly coordination meetings between the Owner, Contractor and Consultant as well as weekly meetings between the Owner and Consultant to streamline efforts



Figure 8: Core taken from Panel S-1068

7) Communication

TRWD met weekly onsite throughout construction to discuss progress with FNI (design engineer) and Bauer (contractor). The team also used TRWD's Project Portal as a construction management software to document submittals, photos, construction progress, schedule and other items. In addition, TRWD led public involvement and outreach to communicate with the surrounding community. Public information Open House Sessions took place before project construction for TRWD to notify local communities of the public impact of work and upcoming lane closures.

8) Monitoring

The main task to be achieved by this seepage cutoff wall was to control passage of water through the embankment. Therefore, it was only logical that the water table be monitored prior, during and post construction to ensure the proposed seepage cutoff wall would perform to the levels anticipated. To do so, a series of piezometers were installed, which were utilized to determine the groundwater regiment. With the rerouting of cables explained in Section 1, the piezometers remained functional during construction and the groundwater was constantly monitored. Prior to and during most of the construction phase, piezometric levels remained constant. However, following installation of the last two panels to complete the positive cutoff, the project team noted a drastic – and much improved – change, as water levels dropped five feet or more despite an associated rise in lake level (Figure 9). This timing and noticeable change in piezometric levels did not occur until installation of the last panels, which confirmed that the continuous positive cutoff technique was – in fact – the correct rehabilitation technique. Post-construction inspection also indicated that the wet area previously noted on the downstream slope had begun drying out. This provided on-the-ground visual proof that corroborated the drop in piezometric levels.

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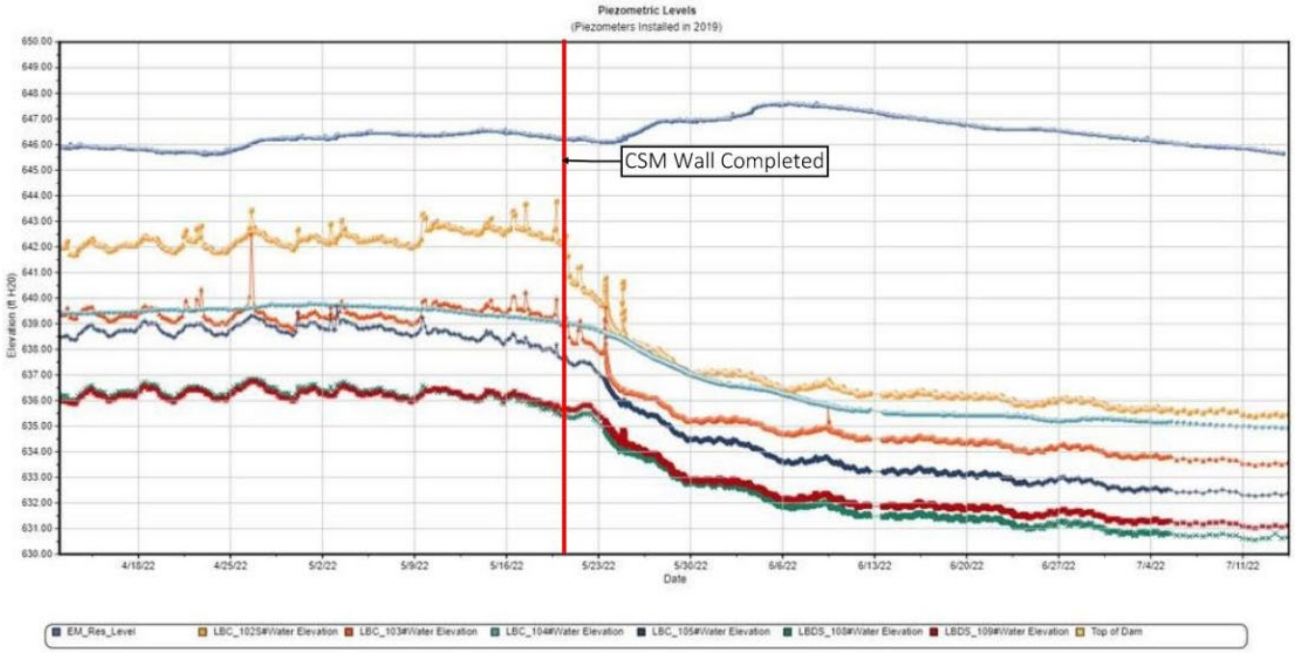


Figure 9: Groundwater levels timeline. Notice the drop in groundwater elevation immediately after completion of CSM wall

9) Gap closure between the proposed alignment and existing training walls

Towards the westernmost panels of the alignment, the CSM wall would approach the existing spillway training walls (see Figure 1 for location of training walls). The subsurface material at this location consisted of existing uncontrolled fill used to construct the embankment in the early 1930s. The fill material frequently included unknown obstruction causing construction issues and at the immediate vicinity of the training walls, it was not clear if the obstruction is a local barrier such as boulders, or an actual structural element associated with the training walls. In order to save the integrity of the training wall, it was decided that the method of construction would need to change. A method was needed that not only is able to work around any obstruction, but also effectively block the seepage. To do so, a series of grout holes were installed and the western 10 ft of the alignment was constructed as a grout curtain. The grouting injection continued to the proposed termination depth of the CSM wall, and continued until the grout pressure within the hole would start increasing, indicating the surrounding area was full with no more room for grout to penetrate.



Figure 10: Grouting the closure section of the alignment

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IV.SAFETY PERFORMANCE

To ensure safety of the personnel and public during construction, a project-Specific Accident Prevention Plan was crafted. The plan aimed at preventing accidents and incidents that could lead to occupational injury and illness by anticipating, recognizing, evaluating, and controlling Health, Safety and Environment (HSE) hazards. The plan established site-specific instructions and processes that were implemented in the event of an emergency. Also, a Site-Specific Hazard Communication Plan was crafted which would inform employees of the contents of the OSHA Hazard Communications Standard, the hazardous properties of chemicals with which they work, safe handling procedures, and measures to take to protect themselves from these chemicals

The District's superintendent conducted daily pre-job briefings to review planned work activities and steps to mitigate potential hazards. From the 4,500 total man-hours performed on this project, no accidents, first aids needed or health loss occurred, adding another successful branch to the overall project. Best Practices that created safety awareness on the Eagle Mountain Dam Seepage Remediation Project included:

- Safety board to post emergency procedures, planning, best practices, toolbox talks, and Job Safety Analyses (JSAs)
- Daily toolbox talks and safety topic discussions with the crew
- Jobsite walk-through with site supervision to identify hazards
- Coordination with TRWD and City representatives to mobilize and demobilize heavy haul equipment and road closures
- Crane lift pre-planning and planning discussions and meetings with the crew before lifting activities
- Contractor behavior-based observation card program
- Stop Work authority program

V.ENVIRONMENTAL AWARENESS

Although the project did not require environmental permitting or environmental services, the team instituted a Stormwater Pollution Prevention Plan (SWPPP) with silt fences and standard approaches for construction containment. One environmental benefit of the CSM wall design solution was less excess spoils during drilling. To specifically deal with spoils during grout mixing, the team designed a widened trench at the existing ground surface for containment of excess grout and materials that were returned to the surface during the installation process.

VI.CLOSURE

The team's multi-phase approach took time to gather data and determine the seepage location. Following selection of a CSM design solution and successful construction, TRWD is seeing positive results in the seepage location. Post-completion instrumentation readings and visual monitoring continue to indicate that the system is moving toward a new balance, with water levels stabilizing at lower and safer piezometric levels and downstream seepage also significantly reduced. The details of this project, documented in two separate papers, highlight the level of effort involved through the process and the lessons learned especially during construction, adds considerable insight and experience to be used in similar projects nationwide.

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Large Scale Peer Audit of a Dam Instrumentation and Monitoring Program

DOUGLAS S. ROY, P.E., AND DAVID J. BARILE, P.E., GZA GEOENVIRONMENTAL

ABSTRACT

The development of a new hydroelectric dam in northwestern Colombia, South America is nearing its final phase of construction. The 225 m (738 ft) high earthen embankment spans the 550 m (1,804 ft) wide valley of the Cauca River. In 2018, a portion of the unlined rock auxiliary diversion tunnel collapsed unexpectedly, blocking the flow/diversion of water. The powerhouse, still under construction, was flooded, causing significant construction delays and increased project costs.

As a result of the collapse and subsequent project re-insurance requirements, GZA GeoEnvironmental, Inc. (GZA) was retained by the Dam Owner to perform a large-scale peer audit of the existing geotechnical instrumentation and monitoring systems of the dam, powerhouse, and associated facilities. The audit primarily focused on the instrument selection based on subsurface conditions, installation methods, and range of sensors, with a particular focus on data collection, data reduction, quality control, and reporting/alerts. Multiple instrument types were installed across the project site to gauge and measure different properties of the work. GZA's audit primarily focused on settlement cells, load cells, pressure cells, vertical magnetic extensometers, multipoint extensometers, vibrating wire piezometers, inclinometers, and open tube piezometers with an overview of manual and automated deformation monitoring using optical survey methods. Over 3,400 instruments had been installed on the project site prior to GZA beginning the audit, with millions of manually and automatically collected data points. A detailed step-by-step audit—from installation, maintenance, data collection, and data reduction—was performed on each instrument.

A detailed Audit Report presenting our findings, recommendations, and suggested improvements was provided to the Dam Owner and re-insurance team. After reviewing the report, the design team instituted changes to the instrumentation and data collection systems, where feasible, to conform with US and European industry standards, allowing the instrumentation to be a vital part of the design team's ability to evaluate the facility's performance.

This paper includes: a case study; discussion of South American instrumentation practices which likely also impact other developing countries; and recommendations for instrumentation audits or reviews that might be valuable to both the geotechnical and dam safety industries.

INTRODUCTION AND BACKGROUND

The development of a new hydroelectric dam in northwestern Colombia, South America is nearing its final phase of construction. The 225 m (738 ft) high earthen embankment spans the 550 m (1804 ft) wide valley of the Cauca River. When complete, the planned eight-turbine facility will generate 2,400 Megawatts of electric power annually.

Construction of the project began in 2010, with the major civil works nearly complete as of 2023. Current ongoing construction at the site consists of mainly of repairs, sanitation, below grade powerhouse expansion works, and mechanical works to install power generation equipment.

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In early 2018, a portion of the unlined rock auxiliary diversion tunnel collapsed unexpectedly, blocking the flow of water. The powerhouse, still under construction, was flooded, causing significant construction delays and increased project costs. The tunnel collapse, associated actions, remediations, and intensive peer review—referred to as the Contingency—are well documented; the information is available through various media sources but will not be discussed here.

As a result of the collapse and subsequent project re-insurance carrier requirements, GZA was retained by the Dam Owner to perform a large-scale peer audit of select existing geotechnical instrumentation and the monitoring system for the dam, powerhouse, slopes, and associated facilities. The audit primarily focused on the instrument selection based on subsurface conditions, installation methods, and range of sensors, with a particular focus on data collection, data reduction, quality control, and reporting/alerts. Long-term, post completion, and operational dam instrumentation were outside the contracted audit scope.

Multiple project teams were involved with the instrumentation selection, procurement, design, installation, data collection, data reduction, and evaluation at the project. A brief description of each role is described below:

- Dam Owner—oversees all work, coordinates between teams, reviews results of the instrumentation program, and manages selected monitoring systems.
- Contractor–general contractor handling construction, procurement of instrumentation, technical advice, instrumentation installation, repair/replacement of instrumentation, and specified instrument automation.
- Consultancy—responsible for design of instrumentation, preparation of technical specification, preparation of instrumentation drawings, data collection and reduction, preparation of the Bi-weekly Reports, selection of instrumentation to automate, design approval, and evaluation of the instrumentation data compared to the engineered design.
- Supervision—responsible for quality control, supervision, validation of the instrumentation installations, and evaluation and approval of Contractor-selected instrumentation purchases.

All contract design documents and communications were provided in Spanish, the national language of Colombia.

Geotechnical instrumentation has been used since the inception of the project to evaluate and verify the design and engineering properties of the work. Many geotechnical instruments within the below-grade powerhouse expansion works were destroyed or damaged by flooding and rockslides during the Contingency. Selected instruments were replaced; other instruments were repaired and re-baselined; others were removed due to construction work sequencing. After the Contingency, the Dam Owner, and the Consultancy redesigned portions of the instrumentation system by adding sensors, and they adopted a more robust system to address the shortcomings of the pre-Contingency monitoring system.

When GZA became involved in the project in early 2022, approximately 97% of the planned instrumentation had been installed. The Consultancy continues to augment the evolving instrumentation program where areas of concern are identified and/or at critical areas.

The project site had over 3,400 instruments installed prior to GZA beginning the audit, with millions of manually and automatically collected data points. The scope of our audit focused primarily on settlement cells, load cells, pressure cells, vertical magnetic extensometers, multipoint extensometers, vibrating wire piezometers, inclinometers, and open tube piezometers with an overview of manual and automated deformation monitoring using optical survey methods.

Prior to the Contingency a limited number of instruments had been installed at the site; they were not monitored frequently. The limited, pre-Contingency documentation and information hindered GZA's ability to evaluate the overall efficacy of the system. After the Contingency, many instruments were added to the monitoring program by the Consultancy. A more standardized methodology was adopted for procurement, installation, documentation, monitoring, and data reduction. GZA's audit primarily focussed on the post-Contingency aspects since the project team could not produce sufficient data or records from prior to the event for review.

The data management system in place when GZA became involved was presented on multiple Microsoft SharePoint web-based collaborative platforms; no commercially based data management software was being used throughout construction. The organization of the data was divided into Fronts and Sub-Fronts. The folder

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was further divided into types of instruments and then into monitoring data and information regarding installation/repairs.

The collection and processing methods utilized a Microsoft Visual Basic Macro Scripts embedded in Excel sheets. The field data was manually transferred into Excel sheets based on instrument type. The calibration factor, baseline reading, and other pertinent information to reduce the raw readings was saved in a primary reference file. The data was reduced into engineering units and plotted. An overview of the automated data processing is shown in Figure 1. Notes regarding issues obtaining readings, damaged equipment, etc., were entered into a separate tracking sheet. This methodology led to issues tracking the instrumentation status, specifically for a project of this substantial scale. A small number of the Consultancy staff were tasked with updating and maintaining the SharePoint websites, including updating the processing scripts, calibration values, etc. The mechanism for collecting and processing the data and reporting utilized multiple Excel sheets.

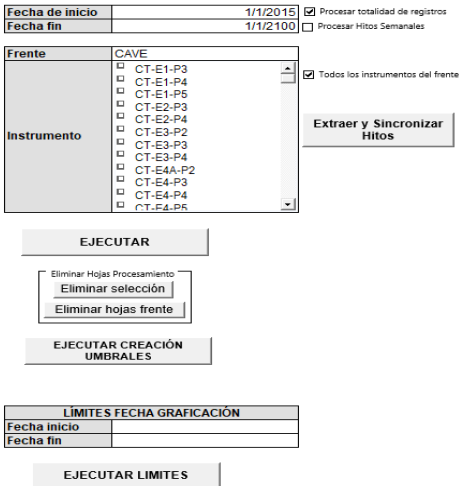


Figure 1: Example of Automated Processing Data Excel Sheet

Bi-weekly Reports were issued to the project team summarizing the data collected for the current period. These reports contained hundreds of figures and plots spaced across multiple PDF files, making it very time-consuming and challenging to review. The annexes of the Bi-weekly Reports included lists of alert limit exceedances, which were based on standard deviations rather than the engineering properties of the work.

After the Contingency, additional automated dataloggers were incorporated to monitor critical Fronts and Sub-Fronts at the site. The project team is currently developing the SADIGH data hosting platform for long-term monitoring of critical project performance instrumentation that will provide data collection, data reduction, visualization, and alerting features.

AUDIT SCOPE AND APPROACH

GZA's scope of services, generated jointly with the Dam Owner, included a detailed review of selected geotechnical instrumentation, design drawings, project specification, instrumentation submittals, and project documentation including installation, calibration, data acquisition, processing, and interpretation of results. GZA's audit intent was not just to critique the methodologies and systems in place but to identify improvements that would deliver increased, consistently reliable, accurate, and timely results.

GZA developed a detailed step-by-step audit process to evaluate the installation, maintenance, data collection, and data reduction which were performed on each installed instrument. These factors were deemed critical to operations, monitoring, and data reduction to the sensors to confirm functionality and accuracy.

This step-by-step audit process:

- Verified that the geotechnical instrumentation installed at the site operated in optimal conditions. Failing that, identified which instruments did not work properly, to define intervention and improvement actions to achieve optimum operation or conceptualize possible replacement.
- Validated that all installation, calibration, procurement procedures of data, processing and interpretation implemented were appropriate; if they were not, identified deviations.
- Reviewed the criteria of accuracy, error, uncertainty, and reliability of the instruments.
- Reviewed and conceptualized the installation processes of the instruments (e.g., borehole size, orientation, annular space, grouting, and orientation of the housing for instruments). When appropriate, defined plans and improvement actions for the installation of new instruments or reinstallation of those already available.
- Identified potential problems derived from the terrain conditions, which may have influenced the operation of the instruments.
- Verified and reviewed the installation, maintenance, and operation process of connections, reduction, and data transmission.

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- Verified and reviewed instrument calibration processes, the definition of baseline and initial readings, and readjustment of baseline where appropriate.
- Identified the reasons for changes in accuracy, precision, error, and uncertainty of the instruments installed.
- Verified and reviewed field processes related to compliance with field methods, verifications, and maintenance in accordance with the recommendations of the instrument manufacturers.
- Verified and reviewed the data processing with the specifications and procedures of the manufacturer and the project.
- Verified that the accuracy of the processed data matched the alert and threshold levels of the instruments.
- Validated the presentation of the data for its application or use during the construction stage of the work.
- Defined procedures for identifying and classifying unreliable instruments and data.
- Identified improvement actions to verify that the system delivered reliable results in a timely manner.
- Validated the processes that had been used to merge data in cases where an instrument had been changed, repaired, and/or modified.

FIELD VISIT

An integral part of our instrumentation audit was visiting the project site to review the procedures, site layout, and installations. GZA's three-person field team conducted two site visits with the Dam Owner. This aided our overall understanding of the project and allowed us to meet with the project team to discuss documents reviewed, our findings, and recommendations. We also clarified some of the technical terms which were not directly translating between English and Spanish.

During the initial site visit, GZA met with the project teams, reviewed the instrumentation construction, observed limited instrument installation and instrument calibration, and observed the data collection and processing methods. The project teams were very well prepared to answer GZA's questions and made many informative presentations. This increased our understanding of the project and the processes that were in place at the time of our site visit.



Figure 2: Collection of Manual Readings

The GZA team's hands-on review informed our knowledge of the project site and scope of monitoring, which ultimately increased the effectiveness and accuracy of the overall instrumentation audit. During the site visit we observed installation and inspected the wiring and construction of various instrumentation types. GZA accompanied the project personnel in charge of the readings to observe readings being collected and recorded. This included the onsite calibration of the manual tape extensometers. An overview of manual data collection and calibration of the tape extensometers are shown in Figure 2 and 3.

The second site visit was conducted after GZA's Audit Report had been issued, facilitating discussion with the entire project team and on-site review of GZA's recommended improvements to the instrumentation monitoring system. GZA provided the Dam Owner with the comprehensive Audit Report summarizing the findings and recommendations compiled over the course of a year.



Figure 3: On-site Calibration of Tape Extensometers

AUDIT RESULTS AND DOCUMENTATION

GZA developed an innovative approach to review the vast amount of data, while comparing it to industry standards and drawing upon GZA's expertise developed through previous experience with similar projects. The sheer number of documents as well as the poor and/or inconsistent record-keeping practices prior to the Contingency slowed our review process. GZA streamlined the review and recommendation process by breaking each instrument into four basic categories which included installation, maintenance, data collection, and reporting/alerting sections. These categories were deemed critical to ensuring accuracy and functionality of the monitoring system.

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The completed Audit Report summarized GZA's assessment of the installation and accuracy of the geotechnical instrumentation utilized at the site. We concluded that since the Contingency, the instrumentation system's operation had been updated, but there was still room for improvement in the installation, documentation, protocols, procedures, and reporting. The project team had standardized instrumentation procedures and adopted alternative means and methods to improve and streamline data processing and collection. The methodology adopted still had restrictions on the threshold exceedance and reporting of sensor data, which would be key to identifying potential problems. The advanced automated monitoring and reporting should have been deployed across the entire site and in critical areas during construction. The integration of automated data collection systems would have vastly expanded the project team's overall understanding of the data and provided thresholds if an issue arose.

GZA identified and made recommendations for several improvements related to the installation, tracking/documentation, monitoring, sensor type, data reduction, and alerting related to the instrumentation system. The recommendations were intended to improve the functionality of the instrumentation system to provided more accurate data and timely notification to the engineering team. On a project of this scale, it is critical to identify any movement or changed conditions which deviate from those in the design to safeguard both construction workers and the downstream communities from the dam.

Prior to the Contingency there were scant documentation or standardized procedures for the instrumentation at the site. Post-Contingency, the team adapted new methods. This included integration of a tablet system for field personnel to document the readings, compared to the handwritten worksheets used previously.

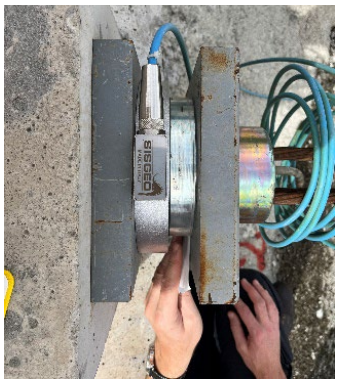
Much of the instrumentation on-site was manually read by a team of 96 technical staff on a weekly basis (monitoring was undertaken seven days a week). Only selected instruments—less than 20%—in critical infrastructure were connected to dataloggers and automatically monitored. The manually monitored instruments were read using manual readouts, and readings were recorded on a handheld tablet. As previously discussed, the instrumentation data was managed using Excel processing sheets with Microsoft Visual Basic scripts. The scripts automatically loaded data, performed calculations (where required), and graphed the data. We noted that the data was slow to load; it would be challenging to review the data between the Bi-weekly Reports. Automation would be specified to be implemented during construction, when the risk of unforeseen field conditions or changed conditions are present.

Using the manual data collection system, there was no way to accurately predict outliers in the datasets, which could have been incorrectly entered by the field personnel collecting the readings. We recommended that previous readings be shown on the tablet to confirm the accuracy and prevent manual user error.



Figure 4: Typical construction of convergence points

We also recommended some changes to the construction and on-site monitoring of the convergence points. The points were manufactured on-site from rebar with welded eye loops as shown in Figure 4. We recommended that any new convergence points be constructed using commercially manufactured, one-piece points for more consistency. The existing convergence monitoring points protruded from the wall, making them likely to get struck or damaged during mining operations. We recommended recessing them in the rock face to protect the instruments from damage and reduce the length of the extension, which may be subject to bending. A stand-alone white paper was also presented on collection of convergence point readings with electronic distance measurements instead of a steel tape; the electronic instrument is easier to operate and does not require a physical connection between the points (Mining Engineering, 2023).



Additionally, we recommended some change to the bearing plate utilized in construction of the load cells. The sensors selected by the Contractor did not have individual leads to collect readings from the sensors to verify eccentricities in the loading. As shown in Figure 5, during our site visit we were able to pass a piece of paper between the sensor and bearing plate.

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We recommended a conical bearing plate be installed to redistribute the loads.

Overall, other physical instrument improvements were minor and would have a negligible effect on the monitoring data, but we strongly recommended automating or streamlining the processing of data to prevent inaccuracies and to expedite turnaround of documents.

Figure 5: Review of Construction of Installation of Load Cell

We recommended slight changes to the equations used to convert instrument readings to engineering units and minor considerations of external factors affecting the data which needed to be addressed. These improvements would refine the data, which had an error range from 1% to 10% of the total values being processed. Significant increases or spikes in the data needed to be filtered out, especially when related to damaged equipment, faulty calibration, or nearby construction activities that may have impacted the data.

INSTRUMENTATION IN SOUTH AMERICA

Several papers have been prepared on the geotechnical instrumentation practice limitations in South America. It is likely that some of these limitations also exist in other developing countries. Specifically, papers by Silva (2022) detail various factors that limit instrumentation use—specifically the use of automation in Brazil, which borders Colombia. Generally, these factors included language barriers (reference documents and instrumentation manufacturer web sites were generally available in English or the manufacturer's home language); cost of procuring sensors/automation systems, including the VAT import tax and customs imposed; exchange rates; lead times; Contractor procurement methods; performance specifications; and, most importantly, the cost of labor required for manually monitoring vs. automated system.

Several of these factors also impacted the Colombia project, which had limited or poorly procured sensors prior to the Contingency. Additionally, the initial project had vague specifications, which left most design and implementation up to the Contractor, who looked to cut costs. The instrumentation team on the project was also selected partially based on the locality versus expertise. Given the general lack of instrumentation utilized in Colombia, it would be difficult to find the experienced personnel required, which potentially led to assumptions being made in developing and implementing the instrumentation system. As has been demonstrated (Choquet et. al 2016), automation of sensors could be justified on a cost basis given the long construction schedule. As we noted, the Colombia project employed over 96 full time staff, and with some working seven days per week because of local low labor costs and political pressures, it was considered by some as more cost effective than purchasing automation equipment. Nonetheless, the automated sensors would have facilitated more informed and timely decisions by the project team if movement or assumptions in the geotechnical design differed from what was utilized in the design phase.

Fortunately, after the events of the Fundão Dam in 2015 and Dam 1 of the Corrego do Feijão Mine in 2019, as noted by Silva (2022), Brazil promulgated regulations to maintain automated instrumentation and/or video monitoring systems, appropriate to the complexity of the structure, with real-time monitoring, following the criteria defined by the designer. Although too late for the construction portion of the Colombia project, we can hope that the lessons learned recently in Brazil help to change the instrumentation systems and designs at similar large dam projects around the world.

DISCUSSION AND NEED FOR INSTRUMENTATION AUDITS

Geotechnical instrumentation has been employed on large and small infrastructure projects for over 100 years to evaluate and verify the assumptions in the design and engineering properties of work. Monitoring is vital to assess the integrity of both the construction and existing conditions of dams, to prevent future damage and mitigate risk, as discussed by Dunnicliff (1988).

Based on the authors' experience and informal peer discussions at a recent international instrumentation conference, there are no guidance or standardized documents available to evaluate or audit the performance or suitability of installed dam-based instrumentation. The most supportive documents we found were of failure analysis of dams and similar structures. Generally, new standards for monitoring dams and critical infrastructure are only implemented as technical procedure updates once a failure or problem is encountered. Instrumentation industry publications are also limited: the former Geotechnical News, by BiTech Publishers, with the quarterly column prepared and edited by John Dunnicliff, still offers value and relevance. A review of the article and responses by Mirghasemi (2011) regarding the Karkheh Dam instrumentation program in Iran are valuable to understand the varied opinions regarding dam instrumentation installation and monitoring.

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The US Army Corp of Engineers (USACE) document EM 1110-2-1908, “Instrumentation of Embankment Dams and Levees” dated November 30, 2020, was developed to recommend the minimum number of sensors for dams based on type and construction. This was one of the first USACE-published attempts to standardize monitoring nationally within the United States.

Overall, there are little to no industry standards for installation procedures or verification of the performance of structures utilizing monitoring data. This is due, in part, to regional regulations based on the construction and operation of dam structures; most localities and federal agencies do not require instrumentation reports to be submitted and do not specify any monitoring requirements.

Third-party or peer review of instrumentation systems should be required to safeguard infrastructure. It may be required on existing dams and should be undertaken during the construction and reservoir filling phases which pose the highest risk associated with failures. A third-party review of overall instrumentation programs is not conducted during construction of new dams and maintenance/monitoring of existing data.

It would be a challenging and immense undertaking to create a strict instrumentation requirement for dams, as there are so many variations in geology, construction type, location, and design assumptions. Additionally, with environmental regulations, the development of new, large-scale dams in the United States is rare. Therefore, most dams requiring instrumentation and review are historic dams with limited design or construction records. We recommend that a minimum number of instruments be specified for each dam. As a methodology of the value engineering process, the instrumentation specialist may recommend retrofitting the existing systems on-site or, to conserve costs, using alternative design methods. This could include installation of instruments for which the sensors could be easily removed or replaced or into which newer automated loggers could be integrated. Newer technologies which have emerged on the market within the past three years require more simplistic programming and no extended runs of signal cables. Additionally, the dam's instrumentation data can be integrated onto a cloud-based website that enables remote access and automated alert notifications. The ever-advancing technology and available sensor types have changed the landscape, offering easy and cost-effective monitoring solutions.

The required peer review process would consist of evaluating the instrumentation installation, data collection processes, and data reduction into engineering units. This is critical to ensure the instrumentation functions as intended and that correct results are produced and displayed for review by the project team or ownership. We anticipate that the level of evaluation of the instrumentation system would be based on the type of construction and associated risk of the dam (i.e., age, height, proximity to other critical infrastructure).

On the design side, the instrumentation strategy should consider the planned location and reading for expected versus what is being recorded by the sensors. The instrumentation should be utilized from project conception, and, ideally, the dam owner should evaluate automation to alert the teams if an issue is encountered during construction. As noted the most risk is anticipated during the construction of these dams; monitoring the dam during operation is typically done less frequently than during the construction phase.

During GZA’s review of many dams within the northeast, several dams were observed to have instrumentation systems to monitor phreatic surface, seepage, and deformation. However, these instruments were not being actively monitored by the dam owners, as there are currently no local and national reporting requirements for this infrastructure. Governing agencies should require reporting to ensure that the dams are safeguarded, and no remedial repairs are necessary. In many instances, instrumentation data is locally hosted on a server at the dam which is rarely reviewed by the dam operations team.

We recommend that audits be conducted on dam infrastructure to determine the accuracy and functionality of the instrumentation system. It is important to verify that the geotechnical instrumentation is installed, functioning correctly, and that data reduction is being carried out in a way to accurately evaluate the engineering properties. In many instances these values are utilized by the design team to evaluate the performance of a structure; incorrect data reporting or lack of instrumentation could lead to failure.

CONCLUSION

GZA developed a detailed methodology to review and assess the instrumentation system utilized at the Colombia project site, identifying issues that affected the data. The instrumentation system we reviewed had been primarily constructed and implemented by the project teams and did not consist of a commercially procured system. There were a few shortcomings on data collection, reduction, and alerting.

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Overall, we suggest there is a need in the U.S. for a more formalized standard for review of instrumentation data, installation practices, and dam monitoring—during and after construction. This standard would be driven by local governing agencies and the manufacturers of the equipment, and they should consider the use of a peer review to obtain the best results. Currently, the existence of these standards is sporadic and is based on failures which have occurred; no single document encompasses the range of structures and recommended monitoring requirements.

ACKNOWLEDGMENTS

Thanks to the GZA team members for all their hard work in reviewing the documentation and preparing the Audit Report. This includes Susan Benyaminov, Allan Amador, Alexander Perez, Helen Fallon, and John DeLano.

The Colombian ownership, people and construction teams were integral in assisting us with our data review. Their presentations helped us understand their processes and they were always available to meet to discuss the audit findings. It is evident that many individuals associated with the project are working hard to move the project to completion.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

Century Old, Run of the River Dam, Inspected for the First Time In 70 Years

JERI TIMM, PUBLIC UTILITY NO. 1 OF OKANOGAN COUNTY; AND CATRIN BRYAN, M.SC., P.E., MCMILLEN INC.

ABSTRACT

The Public Utility District No. 1 of Okanogan County (District) owns and operates Enloe Dam on the Similkameen River, in Washington. Construction of the 57-foot high, 276-foot long concrete arch dam was completed in 1920 and generation ceased in the late 1950's. In the absence of functioning diversions around the dam, the river has passed freely over the dam crest since then. Though the dam was designed to overflow and serve as a spillway, the absence of diversion structures prevented inspection of the dam face, abutments, and tailrace, leaving the condition of the dam largely unknown. In 2019, the District committed to constructing facilities to inspect the dam. This presentation describes the efforts taken to inspect the dam. To dewater the dam for inspection, the District invested $10 million over two years, replacing the original gates and wood stave pipelines with vertical 72-inch diameter gates and two 200-foot-long sections of steel pipe. The diversion structures were designed to route late summer flows around the dam and the tailwater. Though construction was not without challenges, the work was completed in compliance with applicable permits and the diversion structures performed as intended for the late summer inspection.In September 2022, the District, led by McMillen Inc., successfully and safely conducted the dewatered inspection and condition assessment. The multi-disciplinary team used unmanned aerial vehicles to capture photogrammetry, underwater side-scan and blue beam sonar for bathymetric surveys and traditional survey and rope access to complete geologic mapping. Field surveys seamlessly stitched the elevation data together. The work yielded dimensional drawings of the dam and appurtenant structures, flaw characterization, bathymetry and geologic maps of the abutments and immediate area. Installation of survey pins on the dam crest, and survey information, confirmed resolution of photogrammetry, and ensured accurate stitching of above-water and submerged maps. Three-dimensional imaginary with millimeter accuracy will be used for future planning and monitoring purposes.This inspection is an example of a dam owner, taking initiative, investing, and committing to meeting dam safety objectives, on a unique facility, with multiple data collection techniques. The main findings from this inspection are that the dam is practically unchanged since 1950, and no emergency measures are needed to ensure continued safe operations in the near term. The District remains committed to dam safety at Enloe Dam.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

Alternative Delivery Approaches to Constructing Dams

DEL SHANNON, P.E., CHIEF DAM ENGINEER, KIEWIT ENGINEERING GROUP INC.; SCOTT JONES, P.E., PH.D., PRINCIPAL STRUCTURAL ENGINEER - DAMS, KIEWIT ENGINEERING GROUP INC.; AND VINCENT MERCIER, ING. / P.ENG., PMP, DEPARTMENT MANAGER – HYDROPOWER, DAMS AND HYDRAULIC STRUCTURES, KIEWIT ENGINEERING GROUP INC.

ABSTRACT

Over the last several decades, the dominant delivery model for dam projects has been conventional Design-Bid-Build (DBB) approach, in which the Owner hires a Design Engineer to design the dam and assist with solicitation of evaluation of bids, and then selects a Contractor to construct the design with support from the Design Engineer. This approach often results in long durations from concept design to completion of construction to allow for lengthy reviews associated with environmental permitting and/or dam safety regulations and can disincentivize early involvement by the Contractor because of the long lead times and uncertain outcome regarding the award of the construction contract. In recent years, several projects have utilized Alternative Delivery (AD) approaches such as Construction Manager/General Contractor (CM/GC), Design Build (DB) or Progressive Design Build (PDB), and Public-Private Partnership (P3) or Alliance to streamline the process and attempt to control cost and long schedule duration commonly associated with the DBB approach.

This paper provides definitions and examples of various contract delivery approaches to constructing new dams or modifications to existing dams. The paper also includes several case studies where Early Contractor Involvement (ECI) resulted in clear improvements in the design and savings to the Owner.

I. INTRODUCTION

Civil projects, especially those involving water and power, are large and complex projects that are mostly unseen by the public. Yet they are integral to our modern society. The rapid population rise in the last 200 years has dramatically increased the need for civil planning, permitting, design and construction projects.

Generally speaking, 150 years ago and later there was essentially no distinction between an architect and engineer and the term was used to describe the individual who planned, designed and constructed a large structure or project. In the early 1800's the term 'Civil Engineer' was used to distinguish the engineer who worked on non-military projects. These professionals worked closely with the building trades during all phases of the project to ensure they were ultimately successful.

In the last 150 years the industrial revolution, advances in medical sciences, and other dramatic leaps of knowledge, technology, and understanding of our world (and the coinciding population boom that accompanied these changes) required the architect/engineer model to change. The generalist architect/engineer subdivided itself into a wide range of specialists where they become experts within a small field of a broad profession. The complexity of the projects these new specialists were designing also forced the building trades to subdivide as well. Within the last 50 years design and construction specifications transformed from somewhat standardized documents to extremely complex documents; further reflecting the finer and finer point designers continue to put on their specific areas of expertise.

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Within the last 30 years, demanding regulations and permitting requirements have added additional complexity. Depending on the location of a project, rules and regulations dictate allowable areas of disturbance, specific construction seasons, work times, wildlife management, allowable impact to adjacent residents, water quality requirements, and dozens of other areas requiring strict adherence to established rules. As with the previous changes to our profession, these regulations are dramatically altering the civil engineer's approach (and associated costs) to civil projects, and often drive a design-bid-build approach to work through the regulatory approval process before proceeding with construction.

In more recent years, with evolving societal needs, with climate change and with a large proportion of existing assets reaching 75+ years in age, more and more projects consist in modifying existing facilities rather than building new ones. This adds to the complexity of construction methods and increases the risk of existing conditions being different from what the engineer assumed in her/his design.

Given these changes, it is little wonder that dam owners, and those involved in the planning, permitting, design and construction of civil projects often struggle to complete the projects on schedule and on budget. The recent period of high inflation has also made it very difficult for engineers to properly estimate projects, something that contractors are traditionally better qualified to do. However, there are some trends in the industry that may give owners options to consider alternate delivery methods where the contractor is involved earlier in the project to influence innovation in design, constructability, and ultimately help reduce costs and time required to complete these large projects.

II. OVERVIEW OF DELIVERY METHODS

Various delivery methods for completing design and construction of heavy civil projects like dams have been defined by different agencies [1] [2]. Some of the more common definitions of these delivery methods are summarized below.

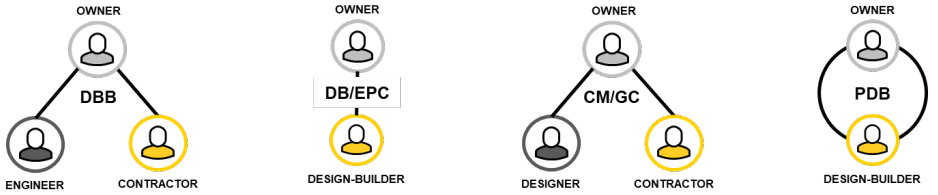
- Design-Bid-Build (DBB): The owner issues multiple contracts for the design and construction phases of the work. The owner will typically maintain separate contracts with the engineer and contractor, and there may be a separate contract for a construction or program manager for larger projects. The contractor typically is not involved until after being awarded to construct the approved design.
- Design Build (DB) or Engineer-Procure-Construct (EPC): The owner issues a single contract per work package (although the entire project is often contracted to a single EPC proponent) to include all engineering and construction services required to construct a facility to the stated project goals. This approach typically involves setting a firm price based on early design concepts and performance specifications, with the contractor responsible for delivering the final design and construction of the project for the agreed upon price and schedule. A Private Public Partnership (P3) is a common variation on the DB approach where private investors provide capital for completion of the project for a public owner.
- Construction Manager/General Contractor (CM/GC) or Construction Manager at Risk (CMAR): The owner issues separate contracts for design and construction as in the DBB process, except the contractor is awarded a contract at a preliminary design level (typically 30 to 60 percent) to review, provide input on constructability of the design, and develop an estimate to complete construction. After developing the estimate, the contractor offers a guaranteed maximum price (GMP) that serves as the basis for the construction contract.
- Integrated Project Delivery (IPD) or Alliance: The owner, engineer, and contractor enter into a partnership agreement or multi-party contracts at the inception of design and develop terms to share in the risk and reward for issues affecting project schedule and cost based on criteria established prior to undertaking the design.
- Progressive Design Build (PDB): The owner engages the contractor at the early stages of design as in the DB approach, but with an agreement to carry the design to a specified level prior to agreeing to a GMP for completion of the construction. This approach combines some of the advantages of the other methods to better share the risks associated with entering into a large project with significant uncertainty.

There are endless possibilities of variations on the above. One of the often-used variations is replacing fixed or GMP with other payment models, such as T&M or target pricing.

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Figure 1 shows a comparison of the DBB, DB, CM/GC, and PDB methods that are discussed in this paper. Note that the main advantage of PDB over DB, in the view of the authors, is that the owner turns over all decision making to the contractor in the DB approach but remains involved in the decision-making process throughout the PDB approach, similar to how they would engage in an IPD or Alliance approach. For the purposes of this paper, IPD or Alliance methods are not further discussed because it is still only rarely used (if ever in the dam market).

Depending on the owner's sources of funding and schedule demands, the various methods may provide different advantages. Figure 2 illustrates how the various methods can allow for acceleration of the schedule by providing the contractor the ability to help influence innovation, cost, and schedule by working more closely with the owner through the project life-cycle, especially in the early phases of design to provide input on constructability that could have significant implications on cost and schedule.



COLLABORATION	None	Limited	None to High[1]	High
TRANSPARENCY	None	Limited	Limited	High
OWNER'S INFLUENCE	High	Limited	High	High
OWNER'S RISK	High	Limited, *if well-defined scope*	High	Low/Moderate
RISK - Cost	Client owns	Contractor owns	Shared	Shared
RISK - Schedule	Shared	Contractor owns	Shared	Contractor owns
RISK - Performance	Client owns	Contractor owns	Client owns	Shared
DESIGN MANAGEMENT	Client owns	Contractor owns	Client owns	Collaborative
SOW REQUIREMENTS	Engineering complete as possible to avoid changes	Well-defined scope in contract	Engineering complete as possible to avoid changes	Collaboratively develop scope of work

1 – The level of collaboration under the CM/GC approach depends on the owner and the definition of the roles of the designer and contractor.

Figure 1. Delivery Model Comparisons.

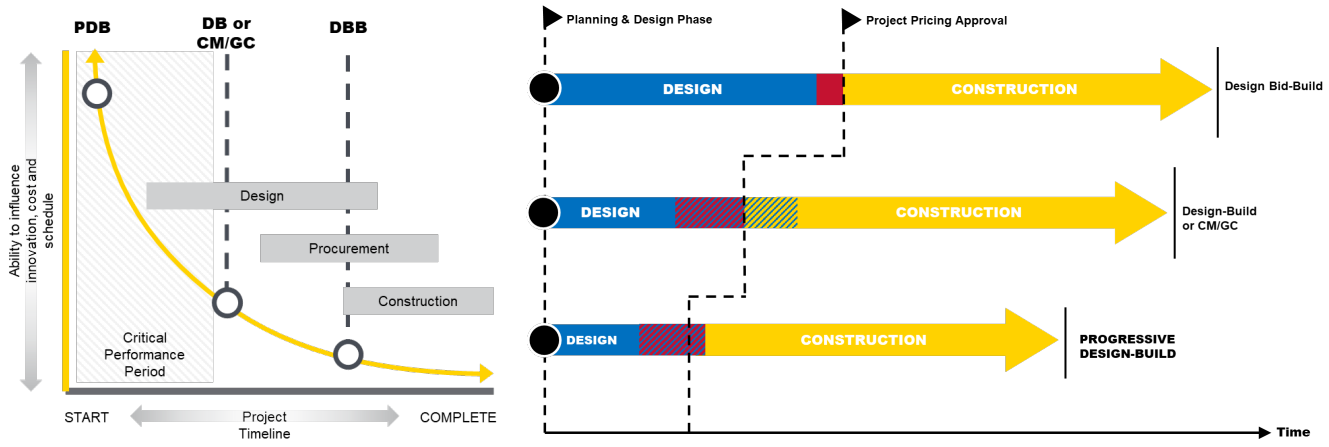


Figure 2. Potential Cost and Schedule Benefits of Delivery Models.

Figure 3 illustrates a PDB approach. Note that the preconstruction process includes refining the scope of work and engaging the required subcontractors in parallel with advancing the design to a 60 to 90 percent level of completion. This design would then be used to establish a GMP or target price to complete the work. At preset milestone dates during the preconstruction phase or development of the price, the owner has off-ramps to allow for completion of the current phase of the design and termination of the contract should they deem the direction of the contract does not suit their needs. Also note that, though the use of target pricing payment model, the potential for shared savings and costs can continue through the construction phase.

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Based on the above comparisons, the PDB approach can offer some distinct advantages to an owner who wants to engage through the process, but still wishes to leave most of the risk regarding the schedule with the contractor. In this approach, the owner is less dependent on having a well-defined scope heading into the design process than they would be with a DB approach. In the DB approach, issues with the initial scope of work can result in significant change orders over the course of the project, whereas the contractor helps define the scope during the preliminary design phase in the PDB approach. The PDB approach is also very effective in cases where the construction method and/or logistics could influence the concept, layout and/or design of the project itself. This is often true of brownfield projects. In those cases, the design can be driven by constructability from a very early stage and prevent later changes and delays.

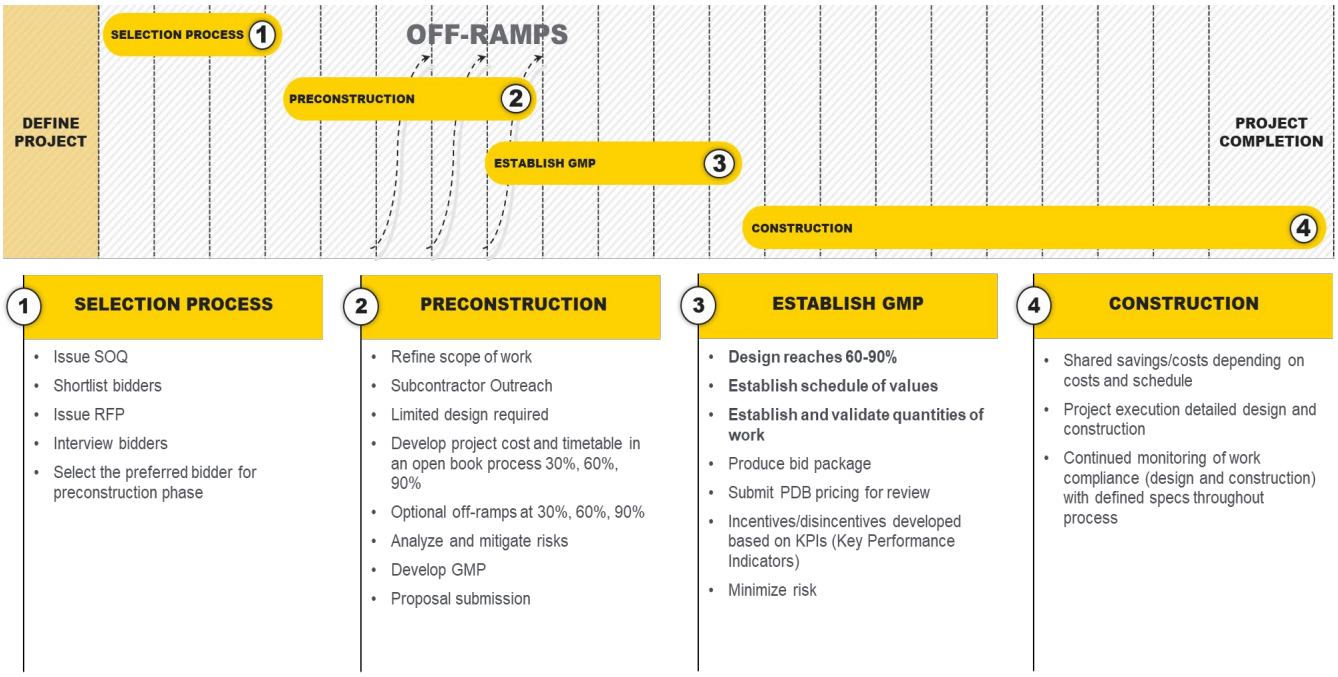


Figure 3. Example of PDB Process.

III.CASE STUDIES

Case studies for Pine Brook Dam, Genesee Dam, and Little Long Dam are presented below. Pine Brook Dam and Genesee Dam are very similar roller compacted concrete (RCC) dam projects using different contract models (Pine Brook was DB and Genesee was DBB) that are presented to illustrate the potential advantages of a DB or PDB approach. A comparison of the key aspects of these dams is presented in Table 1.

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TABLE 1

Comparison of Pine Brook Dam and Genesee Dam

	Pine Brook Dam	Genesee Dam
Year Completed	2006	2009
RCC Volume	34,140 cubic yards	34,300 cubic yards
Conventional Concrete Volume	4,300 cubic yards	2,900 cubic yards
Reservoir Volume	101 acre-feet	101 acre-feet
Principal Spillway Capacity	500 cubic feet per second	550 cubic feet per second
Emergency Spillway Capacity	4,100 cubic feet per second	4,200 cubic feet per second
Upstream Facing	Conventional Concrete	Conventional Concrete
Downstream Facing	Unformed RCC with Soil Cover	Unformed RCC with Soil Cover

Little Long Dam is presented to illustrate the advantages of employing a PDB approach on a larger construction project.

1) Pine Brook Dam

The Pine Brook Dam and Reservoir is a new dam built to store raw water for treatment and subsequent municipal use by the 400 customers of the Pine Brook Water District, located two miles northwest of Boulder, Colorado. A photograph of the completed dam is shown on Figure 4. The dam was proposed as a long-term solution to severe drought conditions that have existed in Colorado since 2002.



Figure 4. Photograph of Pine Brook Dam.

The district, which transfers water from Four Mile Creek several miles west of their service area, had only a three-week reserve of water, and several times since the drought of 2002 no surface water was available for transfer from this creek. Through a self-imposed tax that tripled existing property taxes, the customers of the Pine Brook Water District authorized $5 million to design and build a new 90-foot-high roller compacted concrete (RCC) dam and reservoir; however, it was quickly realized that these funds, which were raised based on an overly optimistic

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feasibility study, were insufficient to fund a traditional design/bid/build approach. The small size of the water district made it virtually impossible to significantly increase funding for the project and the owner made the difficult decision to structure the contract as a design/build project and the project moved forward.

The DB team, which included the owner, general contractor and engineer, began working together at the earliest phases of the project in an approach that may more closely mimic the PDB approach described above. By working as an integrated team, the group was able to quickly address design issues, minimize costly studies and evaluation of multiple alternatives, and deliver the project within the owners fixed budget.

The DB process allowed for many advances, and their associated cost savings, to occur. Some of these advances are described below.

- *Principal and Auxiliary Spillway Configuration:* A principal and auxiliary spillway configuration was determined to offer the maximum project benefit for the lowest overall costs. A smaller principal spillway was designed to safely pass flood event up to one quarter of the probable maximum flood, while the entire dam would be overtopped during flood events larger than this. It was understood by the owner that the PMF event would damage, but not fail, the Pine Brook Dam, but the owner accepted this risk as it was deemed to be relatively small.
- *Soil Cover on the Downstream Face:* Traditional RCC dam design includes a conventional concrete facing on the downstream side of the dam to protect the RCC from freeze/thaw damage and from flood flows that overtop the dam. By covering the downstream face of the dam with soil to protect the unformed RCC significant cost savings were realized. It was understood that the soil covering the downstream face of the dam would wash away for floods in excess of one quarter of the PMF, but this was deemed to be an acceptable risk given the low probability of this event.
- *Removal of Gallery and Grout Curtain:* Packer permeability tests at the site indicated the bedrock at the Pine Brook Dam had bedrock of sufficient competency to remove the seepage gallery and grout curtain from the design. To address the added uplift pressure the footprint of the dam was widened and toe drains were installed at the downstream toe of the dam. Piezometers were installed at multiple locations under the dam to ensure seepage pressures remain within acceptable levels. Toe drains currently produce less than one gallon per minute of water, indicating seepage issues at the dam are well within acceptable conditions.
- *Schedule Compression:* Arguably the single most beneficial aspect of the design/build process was the compression of the design and construction schedule. By approaching the project as a single team, construction was begun only eight and a half months after initiation of the design process. The project was completed 10 months later when the outlet valve was closed and the reservoir began storing water only 18 months after the start of design.

Through the above measures, and by incorporating many more, it was estimated the costs to design and construct Pine Brook Dam was reduced by approximately 40% when compared to traditional design/bid/build approaches. These innovations resulted in the Pine Brook Dam receiving the 2007 Award of Excellence in the Constructed Project from the US Society on Dams.

2) Genesee Dam

Completed three years after the Pine Brook Dam project, the Genesee Dam is perhaps Pine Brook's twin. A photograph of the completed Genesee Dam is shown on Figure 5. Located less than 30 miles from the Pine Brook Dam, it was originally conceived as a DBB project to provide water for the 1,500 customers of the Genesee Water District. The Genesee Dam is very similar in height, construction type (RCC) and volume, and storage capacity to the Pine Brook Dam and the design engineer estimated the project would require similar cost

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Figure 5. Photograph of Genesee Dam.

to construction, or approximately $6.5 million, which had not considered the inclusion of many project features that were part of the Pine Brook Dam. However, the low bid for the project of $12.8 million was 80% over the engineers estimate and well beyond the available finances of the Water District.

After the bidding process was completed, the apparent low bidder (the same contractor that completed the Pine Brook Dam), approached the engineer and owner and offered to apply many of the lessons learned at Pine Brook to the Genesee Dam and participate in its redesign. As with the Pine Brook Dam, the owner, engineer and contractor formed a team to evaluate ways to reduce the overall costs of the project and found many areas where modifications could be implemented and costs reduced.

Applying many of the same lessons learned at Pine Brook, the dam configuration was dramatically changed. Design modifications to the downstream facing and auxiliary and principal spillway configuration were areas where most of the savings were achieved. The removal of the downstream conventional concrete facing and creating a drop inlet principal spillway to safely pass approximately 1,000 cfs (roughly one quarter of the PMF) reduced the overall project costs by several million dollars.

The team also recognized the outlet works could be significantly redesigned. The original design included an intake tower with multiple gates, each 36 inches square, which was well in excess of the actual water release required by the owner. By reducing the size and location of these gates significant cost savings of the outlet works were achieved.

These and other modifications reduced the project cost by $5.5 million to a total of $7.3 million, or a 40% total project cost reduction. While still larger than the original budget, the owner was able to create additional financing instruments and the project moved forward.

3) Little Long Dam

The Little Long Dam was built in the mid-1960's on the Mattagami River in Northern Ontario, together with the nearby generating station of the same name, to create the headpond for the station. A photograph of Little Long Dam during recent construction is shown on Figure 6. The dam includes a 2-gate spillway next to the powerhouse, as well as an 8-gate spillway on a nearby stream called Adam's Creek, allowing for extreme flood flows to safely bypass the 3 hydropower generating stations downstream of Little Long and thus reduce the size of the spilling facilities there.

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Figure 6. Photograph of Little Long Dam during Construction.

The plant operator, Ontario Power Generation (OPG), initiated a project in 2018 to increase the throughput capacity of the Adam's Creek Spillway by approximately 50% and replace all existing gates systems with new equipment. This was to be done while maintaining the headpond levels allowing for the nearby Little Long generating station to remain in operation. A key project requirement was also to provide the operator control over gate operation during flood season.

OPG asked their Owner's Engineer (OE) to develop concepts (<10% maturity of design) to achieve the stated objectives of the project, then directly issued a Request for Proposals in a PDB and target price model, a process they had used for other hydropower projects in the past, with good results. As a bidder, Kiewit recognized that the project engineering and even the original concepts were heavily influenced by the project logistics (isolated region, camp job, etc.) and the constructability (methods, temporary structures, etc.). In order to optimally leverage the advantages of the PDB approach, Kiewit included in their proposal alternative concepts that built on the original concepts proposed by the OE by integrating the logistics and constructability features they developed.

Kiewit was awarded the first step of the project by OPG to progress the engineering enough to allow developing a target price for the work. That first step is called in OPG vernacular the FEED, or Front End Engineering and Design. During the FEED' Kiewit and it's engineering subconsultant advanced the design to a maturity of approximately 30% in close collaboration with the OE (who ensured conformance to specification) and the various Client teams, including the Operations team (who provided valuable feedback as to the operational and maintenance of the future facility). Throughout the FEED, OPG and Kiewit negotiated the parameters of the subsequent EPC contract. The FEED ended with Kiewit submitting an open-book target price estimate as well as a construction plan to OPG, the former accompanied with a quantified risk matrix for further discussion of assignment between the parties. This milestone constituted an off-ramp opportunity for OPG, should they have deemed the project parameters proposed by Kiewit as not satisfying their objectives and plans.

Kiewit was ultimately awarded the EPC phase of the work by OPG, with engineering work starting late in 2019. Construction is now underway and expected to be substantially completed by the end of 2023.

Given the nature and scale of the changes introduced to the concept at an early stage of project development in order to satisfy logistical and constructability features, it is assumed that the project would have been significantly longer in its implementation should a more traditional DBB model have been used.

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IV. CONCLUSIONS

Over the past 30 plus years, most dam construction has been done using a design-bid-build approach to accommodate regulatory approvals prior to moving forward with construction. This approach has tended to inhibit early contractor involvement, which can help with innovation and cost and schedule control. Several alternatives to the traditional design-bid-build approach, including design-build, CM/GC, and progressive design build, were presented. These approaches can provide significant benefits to innovation, cost, and schedule by engaging the contractor to work through constructability and other aspects of the design prior to regulatory approval for construction. Two similar projects, Pine Brook Dam and Genesee Dam, were presented to illustrate the potential cost and schedule benefits that can be realized with using a design-build approach with owner involvement instead of the more traditional design-bid-build approach. In the case of Pine Brook Dam, the process more closely mimicked the progressive design build approach and resulted in significantly lower cost and time to complete than Genesee Dam. Finally, Little Long Dam was presented to illustrate some of the benefits realized on a larger dam rehabilitation project with significant risks resulting from work on an active spillway over multiple seasons.

As described above, delivery models where the contractor is involved early allows that contractor to influence the concepts and the design at an early stage. That includes an involvement of the contractor before and during the regulatory approval process. The early involvement of the contractor streamlines regulatory approval process as well as its aftermath, as this delivery model ensures that the initial design submitted to the regulatory agencies already includes constructability aspects, that feedback sessions with the regulatory agency will include constructability aspects, and that any subsequent adjustments or changes to the design, made to align with comments from the regulatory agencies, will also consider constructability. This approach minimizes delays, repricing and redesign efforts that are often needed in more traditional delivery models when the design output from the regulatory agency process has not included constructability inputs.

V. REFERENCES

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North Michigan Creek Dam: Breathing Life into Aging Infrastructure Through Alternative Project Delivery

NICOLE YOUNG, P.E., BUREAU OF LAND MANAGEMENT; AND CARI BEENENGA, P.E., GANNETT FLEMING, INC.

ABSTRACT

Since 2016, North Michigan Creek Dam (NMCD) had been operated under a storage restriction due to unfiltered seepage emanating on the right abutment. This presentation includes a case study documenting the lessons learned while both exploring alternatives to address dam deficiencies, and implementing a new Alternative Project Delivery Method called Construction Manager/General Contractor (CM/GC). Among the project risks included a remote location, high altitude, challenging site conditions, and a spillway that activated each spring. The CM/GC Method was utilized to balance cost with overall Risk reduction of the facility. In addition, through the use of early contractor involvement afforded by the CM/GC Method, the owner was able to minimize construction risk for itself and the construction contractor. The CM/GC Method included the development of an Integrated Project Team (IPT) comprised of the owner, design engineer, contractor, independent cost estimator, and state dam safety regulator. Each entity in the IPT worked together in its respective role to execute a successful delivery that was both on time and under budget. The construction completed in 2022, and was a FIRST for Colorado Department of Natural Resources delivered through the CM/GC Method. The project also received ENR's 2022 Water/Environment Project of the Year Award for the Mountain States Region. The project removed the existing concrete spillway, non-functioning seepage system, and the low level intake structure, and replaced these features with structural improvements designed to meet current dam safety standards and state-of-practice. A filtered chimney drain was designed and installed using an "innovation" that was proposed during the CM/GC predesign phase to save time and cost, and to minimize construction risk. The filtered seepage collection system continued under the newly installed concrete chute spillway. The existing low-level CMP outlet was lined with a cured-in-place pipe, and included the installation of a filter diaphragm through trenching to limit the required excavation. The project also included a new intake structure and concrete grade beam to protect against the site's heavy ice loading. Reservoir instrumentation provides an early warning system. The CM/GC Method is estimated to have saved one year and approximately $2.5 million.NMCD is a 53-ft-high earth embankment dam located at elevation 8933 feet. It has a normal storage capacity of 1,267 acre-feet. NMCD is owned and operated by Colorado Division of Parks and Wildlife for recreational use and fishing. The dam is located in State Forest State Park along the Medicine Bow Mountains in Colorado.

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DAM SAFETY PALM SPRINGS
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Partnering – More than meetings. A philosophy for successful construction projects.

MICHAEL GENDUSO, PE, CDT, GEOTECHNICAL ENGINEER, HDR, INC

ABSTRACT

In recent years, a growing emphasis has been placed on partnering. Partnering on a construction project is a commitment to work together from all key project stakeholders to foster a collaborative and cooperative environment for the greater good of the project. The term partnering is coming up more frequently in the construction industry and contractual requirements to participate in it is now commonplace. So, what is partnering, and what is it not?

Partnering can be a contractual requirement but is not in and of itself a contractually binding relationship. Partnering is not ignoring or relaxing contract terms or circumventing processes. It's not an excuse for poor performance or a cure-all for issues that arise during construction. Partnering is not easy to achieve and not simply attending meetings once or twice a year. Partnering is working together and following the Contract. It's a process for building relationships through a philosophy of teamwork and understanding. It's dedication to open communication and cooperation. It's an attitude of trust. Partnering is fairness and acting in good faith. It's a win-win risk-sharing (and reduction) philosophy. Partnering is empowering everyone working on the project to communicate directly and solve problems at the lowest levels.

I.INTRODUCTION

From an outsider's perspective, partnering might look like quarterly or semi-annual meetings held at the project office or another location. One might think that high level management and executives get together behind closed doors at regular intervals at partnering meetings simply to talk money. If there's partnering meetings, there's an expectation that everyone will work together as a team. However, interactions and attitudes in the field and field offices don't always quite feel aligned. There may also be the perception that partnering is this new age philosophy of holding hands and singing kumbaya together around the fire. A properly implemented partnering program is not just semi-regular executive meetings and it's not a fantasy of peace and harmony always. It's not just high-level management and ownership negotiating potential monetary disputes and changes behind closed doors. Partnering is a philosophy of collaboration. Partnering is a commitment to communication and collaboration. Partnering seeks to create and maintain a collaborative project approach through a commitment to communication. In a contractual context, partnering could also be regarded as simply a new name for "trust" and "cooperation." But it's not really new or that foreign of a concept. Formalized partnering programs began nearly four decades ago and the basic tenants they're built on, good faith and fair dealing, have long been fundamental to contractual interpretation in the United States. Because of its benefits, partnering is still in practice over four decades later and gaining popularity.

II.ORIGINS AND DEFINITIONS

Partnering's origins can be traced back to the United States Army Corps of Engineers in the late 1980's. The formal concept originates in the United States, where principles of fair dealing and good faith are integral to contractual analysis. In clear and simple technical terms, partnering can be described as a "procedural overlay to a contractual relationship" (Heal, 2009, p. 1). Although participation in partnering can be a contractual requirement,

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partnering itself is not contractually binding and neither alters the contract documents nor does it change legal relationships between parties. It is an overlying philosophy and way of being on top of the underlying contract.

The Washington State Department of Transportation (WSDOT) describes the partnering relationship as follows (Washington State Department of Transportation, 2020, p. 5):

"Partnering is a relationship in which:

- Trust and open communications are encouraged and expected from all participants.
- All parties address and resolve issues and problems promptly and at the lowest possible level. They strive to develop solutions that are agreeable and meet the needs of everyone involved (win-win approach).
- All parties have identified common goals for the partnership and at the same time are aware of and respect each other's goals and values.
- Partners seek input from each other to find better solutions to the problems and issues at hand. This creates synergy in the relationship that fosters cooperation and improves the productivity of the partnership."

At its core, the idea of partnering is quite basic. However, despite its ultimate simplicity, partnering might be the most efficient form of alternative dispute resolution, serving as effective dispute prevention. As the adage says, the best cure is prevention; or as Benjamin Franklin said, "an ounce of prevention is worth a pound of cure." Preventing disputes from escalating to the point of litigation is far less costly and time consuming than legal reconciliation should litigation become necessary. As such, partnering has increased in popularity because of frustrations with the costs and delays associated with traditional methods of dispute resolution.

To expound on WSDOT's definition, in addition to respecting each other's goals and values, establishing and nurturing common goals and values is a critical part of building and maintaining strong partnered relationships. From this aligned position, the project team can work together to improve quality and reduce overall risk through risk sharing philosophies and risk mitigation strategies.

III.STEREOTYPES VS. REALITY

There are numerous project delivery methods becoming more commonplace in recent years, with methods such as design-build and Construction Manager at Risk (CMAR) becoming increasingly popular. For the purposes of this short paper, presentation will generally be in the context of the traditional design-bid-build project delivery method. The players in a typical design-bid-build construction contract is a trifecta of the contractor, the owner, and designer (typically the owner's consulting engineer); noting the designer and contractor are typically bound to the owner by separate contracts, and that the designer was responsible for creating a significant portion of the contract documents that bind the contractor to the owner (the design). Throughout recent history contractors, designers (engineers), and owners have each been portrayed consistently in a stereotypically negative fashion by one another.

Contractors must remain profitable to stay in business. Being profitable dictates charging accordingly for services while making efficient use of labor, equipment, and material resources. Reducing labor and material costs along with minimizing expenditures on costly equipment, overhead, and administration all contribute to economic success. Efficient sequencing, elimination of redundant efforts, and implementation of practical solutions to complex technical problems all help the contractor remain profitable. Because they value their reputation and take pride in their work, most contractors strive to deliver a quality product. Production of a quality product, on-time, and on-budget in accordance with their contract helps further their chances of selection on future projects. Keeping their books in the black guarantees survival until that next potential job arrives. By fulfilling their very purpose, contractors can get a reputation for being technically inept, incompetent, untrustworthy, money-hungry, known for taking shortcuts, and wanting to do as little work as possible while making as much money as they can.

As the customer, owners solicit a contractor's services to fulfill some specific need for infrastructure or a facility. Owners, like any other individual or entity, have budgetary constraints that influence their project decisions. As an educated consumer, the owner seeks to get the highest quality product possible to meet their needs at a fair price within their budget. As the owner seeks to maximize quality and value for their expenditures, they must navigate often unfamiliar technical issues and make decisions often well outside their normal areas of expertise. Owners rely on designers or consultants, often more than one, for technical advice and must decipher the information they're provided to make informed decisions about complex project issues. As the owner struggles to manage their project

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and budget, contractors often experience them as unreasonable, shifty, cheap, demanding, perpetually changing their minds, and always wanting additional work above and beyond the scope of the contract at no additional cost.

The designer, generally a consultant (engineer) to the owner in a typical design-bid-build project delivery method uses technical knowledge, calculations, and analysis to design the project for the owner. Once construction begins, the designer or engineer is often tasked in some manner with monitoring the work for conformity with their design. As the work progresses, they're also called upon to provide their client with information and professional advice about potential risks, benefits, and possible solutions to project challenges as they arise. They must be confident and assertive as they offer advice to the owner and interact with the contractor on the owner's behalf. Although they design the projects, they don't directly participate in the construction process. As such, they may be perceived as armchair experts. To the contractor, the engineer may seem arrogant, egotistical, inflexible, overly technical, excessively conservative to a fault, impractical, and lacking understanding of constructability issues. Contractors may also experience engineers as seeking to enforce unnecessary and/or inapplicable requirements and completely unwilling to ever admit they might have been wrong.

Are these stereotypes really the case? Generally not. In reality, some contractors are more highly skilled and technically capable; some owners are easier to work with than others; and some engineers are more humble, innovative, and flexible. Despite the negative portrayals of the characters involved, everyone has something to contribute to the project; experience, knowledge, and/or wisdom.

IV.DIFFERENCES IN PHILOSOPHY

It could be said that professional relationships are either adversarial or collaborative. As the old adage goes, if you're not with me, you're against me. Stereotypically, relationships between the contractor, owner, and owner's consultants are viewed and even accepted as naturally being adversarial. With an adversarial philosophy, the key belief is that winning comes from control, power, and leverage. It's a dog-eat-dog mindset, a winner-take-all mentality, kill or be killed, survival of the fittest, king of the jungle thinking. By these rules you either win or lose and if you're not winning, you're losing. In the context of contractual relationships between contractor and owner groups, adversarial behavior involves working at arm's length, posturing, shifting risk instead of sharing it, and withholding of information which includes selective sharing.

As we move from an adversarial mindset towards a collaborative one, we pass through the realm of transactional thinking. It's easy to talk about being a team player, the win-win philosophy, and working towards a common goal. However, in the context of construction management and contract administration, there is often gravitation towards transactional thinking despite previous commitments to collaboration. Oftentimes, parties believe they are collaborating when they're not truly in partnership. An attitude develops where the commitment to partnering becomes caveated. The mantra becomes, "I will play nice but at the end of the day I will protect myself first." This is not untrue in that partnering does not promote reckless abandon or throwing caution to the wind in the conduct of our contractual obligations in order to satisfy the other party. However, when we move away from open sharing of information and collaborative problem solving, we begin withholding information and strategizing in ways incongruent with our commitment to our partners. When we begin to leverage our partners and start trading or bargaining, the relationship becomes transactional. In a transactional environment, collaboration is not open and exchanges are conditional. Everything is a negotiation and pressure is applied to our partners to leverage what we want and shift risks. In this bargaining, parties are still pitted against one another; not partnered in collaboration. It's still adversarial and we ultimately often have unresolved issues. Sometimes issues resolve themselves as situations play out; however, if unresolved issues remain, there is likely a "winner" and a "loser" determined by the terms of our transactional agreements. Conflict derived from transactional interactions is more passive-aggressive than direct; however, it is conflict regardless. This transactional environment can sometimes feel like collaboration but is not. In our minds we can convince ourselves we're being team players while we are careful not to make ourselves vulnerable. But is it really collaboration if there is withholding or leveraging the other party? In this transactional realm, oftentimes resolution isn't achieved, and an adversarial space prevails.

Stereotypes portray contractual relationships between owners and contractors as adversarial. At best, aging traditional views dictate formation of a transactional relationship. Traditional views also suggest that design professionals ought to reduce or even eliminate any and all direct interaction with the contractor. Partnering transcends those typical norms. In contrast, partnering is a commitment to a truly collaborative relationship through free and open communication between all parties. Partnering emphasizes open communication at all levels and seeks to solve problems at the lowest possible tier in lieu of immediate and rapid escalation. Efforts are focused to maximize what's in the best interest of the project, not any one of its players. In collaboration, the objective is close

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integration between the contractor, suppliers, owner, and consultants. Parties strive for win-win outcomes and achievement of shared goals through cocreation. This is done by identifying, defining, and sharing a unified purpose; a single common goal both parties are inspired to attain and working towards in unison.

V.GENERAL BENEFITS OF PARTNERING

Challenges arise on every project during construction. Partnering meets those challenges by encouraging fair, good faith solutions to those issues. This is achieved through a philosophy of open communication, risk sharing, and early conflict identification. These philosophies facilitate early avoidance and/or resolution of conflicts as well as seek to reduce risks, increase efficiency, increase quality, and ultimately reduce costs.

It can be commonplace to delay resolving conflicts as parties position themselves against each other, especially in non-partnered arrangements. When resolutions are delayed, a strategic posturing can ensue. As such, open communication breaks down and information may be guarded. Documentation may be collected and withheld for days, weeks, months, or years even; gathered and compiled in preparation for leverage in potential negotiation. Parties may write letters “reserving their right” to seek compensation for perceived additional work or threaten to withhold payment as credit for work not performed or work not in compliance with the contract. All the while parties are not communicating openly with each other, nor are they attempting to resolve the issue or deescalate the situation. As these disputes escalate, defensive attitudes prevail, resulting in a further lack of coordination and cooperation. As collaborative attitudes dissolve, open communication deteriorates further, and parties’ efforts are working against instead of with each other. Efforts are directed at preparation for contract negotiations and/or possible litigation and not towards successful execution of the project. Through preestablished issue resolution protocols, partnering facilitates fast resolution of challenges at the lowest possible levels before they balloon into costly disputes.

Without partnering, as adversarial attitudes escalate, owners and contractors tend to engage in a battle to transfer risk to each other instead of fair risk sharing. In a shared risk relationship, the project team works together to share and alleviate perceived risks. Risk is reduced through collaborative risk identification and mitigation strategies; establishment, adoption, and implementation of good practices; and effective QA/QC protocols. Utilization of third-party inspection and testing, manufacturer’s verifications or certifications, and effective utilization of subconsultants also help reduce direct risk to the primary project team and distribute it equitably amongst the appropriate parties. Collaborative efforts by the owner, contractor, and consultants facilitate identification of risks and promote equitable sharing amongst themselves as well as subconsultants on the job.

It is extremely beneficial to openly discuss and implement strategies for risk reduction together. A strong understanding and adoption of this concept lends itself to increases in overall quality while generally benefiting all involved. In distribution of risk, outside consultants are sometimes overlooked and may not be held fully accountable as they should. For example, proper documentation of changes, clarifications, and opinions from outside consultants may not be obtained as warranted. When the appropriate level of documentation prudent and necessary is not obtained from the responsible party, unnecessary risks appropriately owned by others may be acquired by the primary project team. Collaboration can help identify such risks and develop strategies for mitigation.

Although we speak of open sharing of information as a partnering strategy, in many cases the contract requires full transparency, from the contractor in particular. It is not uncommon for construction contracts to contain language, typically in the Division 00 – Procurement and Contracting Requirements, that indicate all information, drawings, reports, and data collected by a contractor during work under the contract are property of the owner. Even in the absence of such blanket language, it’s common for submittal registers in the specification to provide an exhaustive list of contractually required information owed by the contractor throughout the course of the project. This information is often a significant and valuable component of the work. Despite such language and/or clear requirements set forth by the submittal register and in the specifications, contractors may be hesitant to openly share information for fear it will be leveraged against them, as it sometimes is. As such, information may be provided in a disjointed or purposefully vague manner to confuse or hide facts that may be perceived as potentially damaging to the supplying party. Litigation will almost certainly result in the full disclosure of existing project information should disputes escalate to the point of preparation for legal proceedings. At the point lawyers are involved, the cost of that information is exponentially higher than when the associated issue first surfaced. Open sharing of information up front facilitates conflict resolution at a substantially reduced monetary (and schedule) cost. This open real-time sharing also lends itself to ensuring quality is maintained.

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Open sharing of information results in similar benefits through other contractual processes like submittal review and resolution of comments. Admittedly, there is a balance between open communication and contractor reliance on the owner team to compile, edit, and bring submissions into compliance. To be fair, sometimes the contractor may feel that the owner cannot be satisfied and finds it easier to succumb to seeking direction with respect to certain administrative requirements. When contractor submissions repeatedly fail to gain acceptance from the owner, the contractor may resort to a "tell me exactly what you want" approach. On the other side of the coin, owners may waste valuable time by rejecting contractor submittals, yet not providing clear, concise comments to communicate the exact nature of the deficiencies. Owners and their consultants may withhold detailed comments with the reasoning that the contractor should be intimately familiar with the contract requirements. Withholding detailed feedback is justified with the sentiment that providing highly detailed comments is effectively performing their work. The result can be a never-ending exchange of resubmittals without owner acceptance. Establishment of trust through mutual commitment to the partnering process and open communication can help streamline submittal and review processes while increasing both the quality and timeliness of those submissions. The same collaborative approach can be implemented for the identification of non-conforming work and subsequent issuance, evaluation, and resolution of Non-Conformance Reports (NCRs). Similar gains can be realized with a collaborative approach to RFIs as well. Preliminary discussions and comment resolution meetings can minimize multiple resubmission of submittals, NCRs, RFIs, and other official correspondences critical to the success of the project.

VI.TOOLS FOR ESTABLISHING AND MAINTAINING PARTNERING RELATIONSHIPS

Due to the sensitive nature of contractual relations and the potential for litigation during and after project completion, the example comments and information presented herein is general in nature and anonymous. Examples of surveys and graphical representations have been taken from public sources and cited or created for the purposes of demonstration and are not specifically associated with, or representative of any past or current projects. Some techniques and methods available to assess the condition of the partnering relationship throughout the progression of a contract and associated partnering relationships are briefly outlined below.

During initial partnering meetings, workshops are used to familiarize team members with each other. During these initial meetings, the team creates project charter documents which help define the partnering relationship and establish guidelines and procedures for interaction. Internal processes for issue resolution are also established. Within the project charter documents, the team shall identify and develop project goals and objectives, milestones, success criteria, critical success factors, risks, stakeholder management, organizational structure, lines of communication, and communication protocols. Establishment of these details in project charter documents is critical to setting the tone and acts as a guide for conduct throughout the project. The charter also serves to keep the team focused on their mutual objectives and maintain a collaborative environment.

Between periodic partnering workshops, health check surveys serve as a valuable tool to measure and evaluate the health of team dynamic on the project. Such surveys may ask participants from both the owner and contractor organizations to rank aspects of the partnership on a graduated scale. In the health check survey below, available online from The Partnering Initiative (Figure 1), respondents indicate their position by choosing one of the following: Green – no concerns, Amber – some concerns, or Red – serious concern. The number of green, amber, and red responses are tallied and evaluated.

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Partnership health indicators

1. FUNDAMENTALS
There is a compelling shared vision, mission and objectives fully bought-into by all partners
Partnership has clearly identified collaborative advantages, is able to create added value, deliver more than the sum of its parts
The partnership has been set up to, and is delivering, net value to all partners
Partners are sufficiently empowered and enabled to be able to contribute to the partnership
The partnership is able to include all key stakeholders holding essential resources
The partnership has been set up to deliver net value to all partners
2. PARTNERSHIP RELATIONSHIP
Partners are demonstrating collective leadership of the partnership
Partners are transparent about their assumptions, goals, needs, drivers and constraints
There is a high level of trust among the partners
Partners are empowered and there is clear equity and balance among the partners in decision-making
Partners are accountable to each other for delivering on their commitments
Challenges, problems and tensions are openly brought up and dealt with respectfully and collectively
Partners are jointly accountable for partnership delivery and will help out other partners to deliver
3. STRUCTURING AND SET-UP
The partnering agreement clearly sets out the fundamentals of the partnership (including the vision and objectives, why each partner is involved, the intended value creation, overall approach; commitments, resources, roles and responsibilities of each partner)
There is a clear theory of change (or theory of transformation) for the partnership, along with a measurement framework to be able to demonstrate progress and success
The fiduciary / legal structure for the partnership is fit for purpose
The governance structure for the partnership is fit for purpose
The management structure for the partnership is fit for purpose
RESOURCES
External (non-partner) individuals are supporting / championing the partnership
Personnel are available
Finance is available
Knowledge and data are available

RESOURCES continued
Important networks or spheres of influence are leveraged
Partnership facilitation / troubleshooting / brokering is available
Other necessary resources are available
4. MANAGEMENT
Iterative approach to project management, focused on value creation
Communication of all kinds is sufficiently frequent
Roles and responsibilities are always clear
Deliverables and timeframes are always clear
Financial management, including process for receiving/distributing funding, is effective
Information sharing is effective
The partnership vision remains compelling and relevant to the context
The partnership iterates and adjusts its approach based on experiences to date
Cultural differences between organisations are well managed and clashes avoided where possible
Partners remain fully committed to the partnership
The partnership has been institutionalized into each partner organisation (e.g. engaged key staff, built into organisational planning and budgets etc.)
MEETINGS AND WORK PROCESSES
Meetings happen with appropriate frequency
Setting of agendas and arrangement of meeting logistics ensures inclusivity of all partners
Meetings are documented appropriately and minutes circulated
Conflicts of interest are effectively managed
Partners are consistently present at meetings and represented by appropriately senior level
Decisions are made in a timely and efficient way
5. BROADER CONTEXT / ENABLING ENVIRONMENT
Partners have reviewed and strengthened their organisational capacity to partner
The partnership is connected to similar partnerships and peer learning / influencing takes place
The partnership receives ongoing support from platforms and other mechanisms, as required

Figure 1: Health Check Survey from The Partnering Initiative

https://tpiglobal.org/wp-content/uploads/2022/08/07-Partnership-health-check.pdf

Alternatively, surveys may be formatted with questions scored on a discrete scale of 1 through 5, for example. The owner and owner's representatives are asked to evaluate a series of statements and rank their agreement for each statement on a scale of 1 to 5. In this system, 1 is strongly disagree and 5 is strongly agree. Statements are categorized to represent different aspects relevant to the state of the partnering relationship. Categories might include schedule, cost, flow of information, dispute resolution, trust, and collaboration. The numerical scores are tallied and plotted on a radar chart (Figure 2). The graphical representation provides a clear visual representation of the states of team dynamic. Owner and contractor scores plotted against each other on the same chart also shows the degree of alignment on each subject. Through continued evaluation, progress of the relationship can be easily tracked and evaluated. Measures can be implemented to improve areas as needed.

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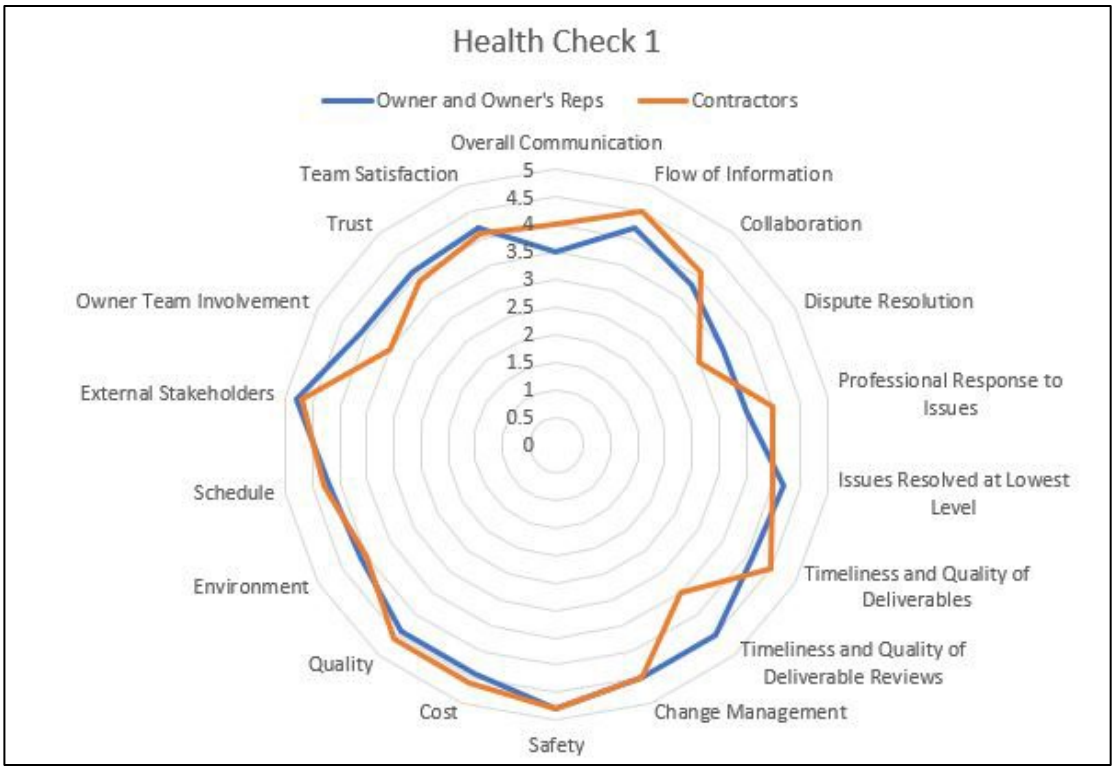


Figure 2: Example radar chart of discrete health survey scores by category.

Open questions for comment can be included in surveys to obtain more precise feedback to further identify specific items that need addressing within general topics.

Examples of positive comments received on partnering health survey checks during the course of a project could be as follows:

- Contractor has not been combative. They have been extremely cooperative.
- Safety, quality of work, cost and schedule management have been good so far.
- Project is currently on schedule and budget. Work is organized and efficient.
- Return of owner's responses to submittals has been timely with few exceptions.

Comments indicating room for improvements could be like those below:

- Work challenges in the field are dealt with hastily without proper communication to the owner for involvement.
- Differences between owner and contractor exist regarding what constitutes a minor field change vs. when formal documentation of change is required.
- (Submittal) Comments are provided on resubmittals requiring changes to information previously provided and not commented on.
- Review comments on submittals are unrelated to the specification requirements.

Through continued periodic surveys administered and evaluated by an agreed upon neutral facilitator, feedback is gathered semi-anonymously and shared in a safe environment for the betterment of the project. It may be evident which comments may have come from owners and which from contractors; however, names aren't named and the open sharing of professional feedback in this way helps smooth friction and correct off kilter trajectories before further deterioration of communications and relationships occurs. Through early identification and open communications intent on collaborative resolution, issues can be deescalated before they become or contribute to future disputes.

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Despite best efforts, issues may not be resolved in the field at the lowest levels. As such, internal issue resolution procedures are established as part of the partnering process. Resolution of issues at the lowest possible levels is the goal of the partnering model. The figure below illustrates issue resolution procedures beginning at the working level and progressing through the hierarchy of leadership (Figure 3).

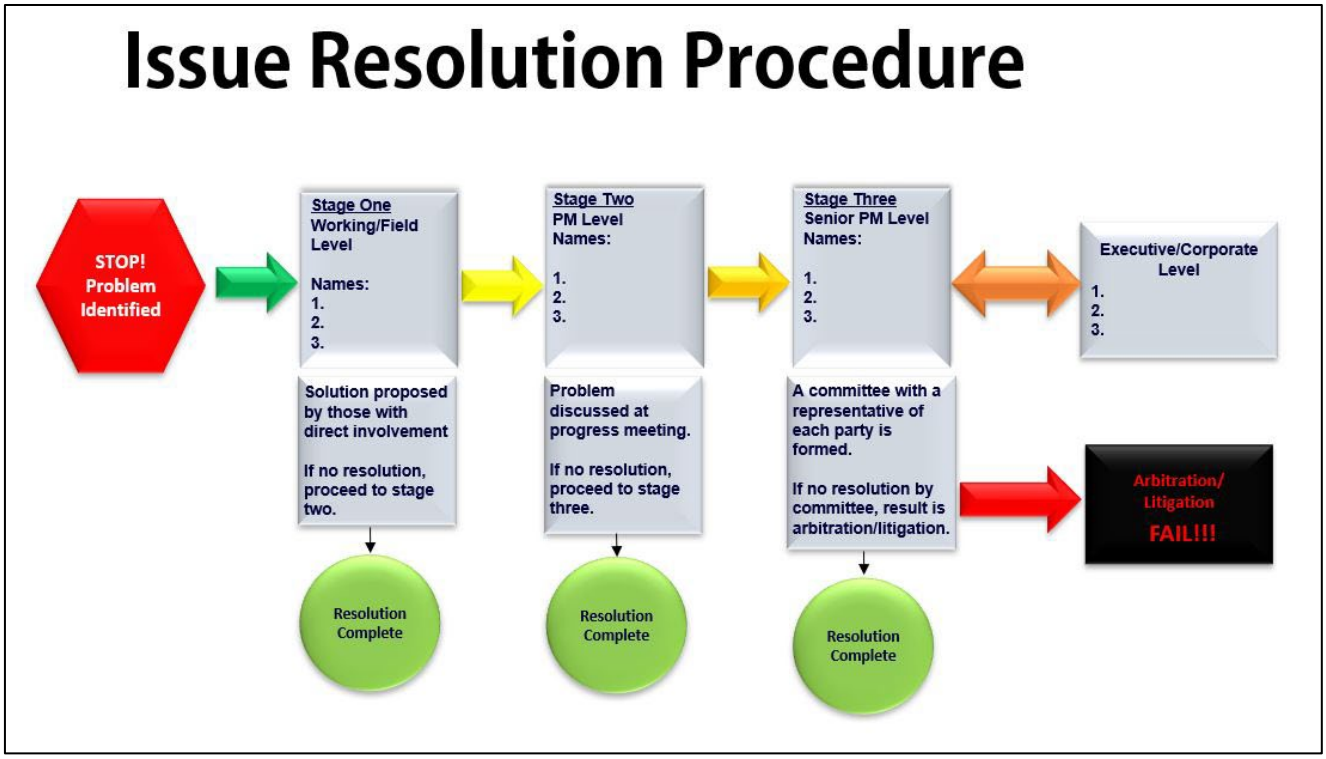


Figure 3: Example issue resolution procedural flow chart.

An issue resolution procedure should begin at the lowest level (field or working level). The next step is escalation to the PM level if a solution cannot be reached in the field at working level. Finally, issues are brought to Senior Project Management and the corporate level if PM level resolution is unsuccessful. If issues cannot be resolved internally at the highest levels, arbitration or litigation results and the process has failed. This procedure may not eliminate arbitration or litigation in all instances but is a valuable tool for reducing the number of issues that become unresolvable conflicts or disputes needing legal intervention.

VII.COST BENEFIT ANALYSIS

"Various research and surveys estimate a partnering cost in the range of 0.25 percent to 2.0 percent of total project costs, with most of the responses indicating partnering cost less than 0.25 percent" (Sander and Thorne as cited by Heal). This estimate is broad and it must be noted that project construction costs themselves can vary greatly from hundreds of thousands of dollars for small projects to hundreds of millions or more on medium to large contracts. Multiple cost proposals received in 2021 for partnering program services on a recent US project showed approximate annual costs on the order of $25,000 for the partnering facilitator's services only. These cost proposals were based on, or normalized to reflect twice a year in person workshops and quarterly surveys. Over a four-year project duration, total base cost was on the order of $100,000, not including participant costs associated with participant work hours and possible associated participant travel and incidental costs. A more robust partnering program with quarterly in person workshops and monthly health surveys might be double to triple this cost, perhaps on the order of $250,000. Considering the project itself had an approximate $250M construction cost, even a 0.25 percent budget equates to $625,000 or over six times the average base cost proposal received for bi-annual in person workshops and quarterly health check survey analysis. Even an expenditure of $250,000 for a more robust

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partnering program on a $250M project only equates to 0.1% of construction costs. Consequently, a partnering program cost of $100,000 as 0.25 percent of construction costs relates to a $40M project. From this, one could conclude that on medium to large size construction projects in excess of $50M or more, even with durations up to 2 years, a partnering program could be implemented at a cost of less than 0.25 percent of the construction budget.

According to Arcadis in their 2022 Global Construction Disputes Report, “the average value of disputes rose globally from $30.7 million in 2019 to $54.26 million in 2020, while the length of disputes fell from 15 months in 2019 to 13.4 months.” Regionally, the Arcadis report goes on to say, “In North America, construction dispute value rose from $18.8 million in 2019 to $37.9 million in 2020, while the length of disputes shortened from 17.6 to 14.2 months.” There are many factors when considering the potential risk for claims on a project including the size and complexity of the project. However, it is clear from available data that a monetary investment in partnering efforts on the order of hundreds of thousands of dollars is minimal relative to the cost of claims which could be in the tens of millions. Avoidance of a single claim attributable to the implementation of the partnering process may be worth the expenditure in legal fees along, regardless of the outcome of litigation.

It must also be noted that conditions for cost sharing are commonly included in specification language for contractually required partnering programs. It is common for owner and contractor to equally share partnering costs in this arrangement.

VIII.CONCLUSIONS

Numerous specific benefits result from implementation of a successful partnering program. The commitment to communication and collaboration, the basis for a sound partnering relationship, inherently leads to a unified team process. Through successful partnering efforts, project teams (both owner and contractor) can realize a reduction of project related claims through implementation of partnering as a dispute prevention mechanism. Collaborative team philosophies encourage innovative solutions to challenges and can yield measurable improvements in quality, efficiency, schedule, and value. Through increased efficiencies, contractors maximize profitability while owners can may also realize schedule and cost benefits. Increased quality not only benefits the end user but also fortifies the contractor’s reputation, contributing to their brand image and success in securing future work. Secondary benefits realized include creation of a positive work environment, reduced personal stress for individual employees and teams, and an overall more enjoyable and gratifying project experience. The upfront efforts required to implement and maintain partnering relationships pale in comparison to the work needed to prepare for and execute messy litigation down the road. The financial commitment to partnering programs is often a fraction of a percent of construction costs and is commonly shared. Furthermore, the investment in partnering can help avoid claims and costly legal fees orders of magnitude greater than the upfront cost of the program itself. If implemented correctly with commitment from all involved, the return on investment for partnering programs makes excellent financial sense. The secondary benefits to the quality of the project in terms of the finished product as well as the overall experience are invaluable.

IX.ACKNOWLEDGMENT

I would like to gratefully acknowledge Richard LeBlanc, Patrick White, and Jose Rodriguez for their support, encouragement, and mentorship throughout my tenure at HDR. Special thanks to my family, friends, and especially my partner Liz Watkins for their continued support, especially over the past couple of years. Thank you for your love and encouragement through the hard work, long and unpredictable hours, distant travel, and extended time far away from home. I miss you all. Thank you.

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Michael has a BS and MS in Civil Engineering from Florida Atlantic University with emphasis in Geotechnical Engineering. He is a Professional Engineer licensed in Florida and Washington and Construction Specifications Institute (CSI) Construction Documents Technology (CDT) certified. Michael has nearly 20 years’ experience providing Project Management, Engineering During Construction (EDC), and Contract Administration services on a variety of large infrastructure projects across the U.S., with focus on dams and levees over the past decade. Recent past projects include new reservoir construction as well as extensive dam rehabilitation, with individual construction costs in excess of $200M. He has expertise in borrow development and management, construction of earthen embankments and associated drains/filters, as well as soil cement armor. He also has a strong background and knowledge of concrete and other construction materials. Michael’s project involvements span complete project life cycles from design through project close out and commissioning. With a strong technical background in Geotechnical Engineering and construction materials, his practical experience makes him extremely knowledgeable on constructability issues, QA/QC practices, risk management, contract interpretation/administration, and change management. Michael’s blend of technical expertise and construction experience, with an effective communication style and problem-solving ability has fostered trusting relationships with owners/clients and effective working relationships with contractors, contributing to the successful completion of complex and challenging construction projects.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

WHAT WE HAVE HERE IS A FAILURE TO COMMUNICATE:

THE SECRET TO DAM MODELING SUCCESS - LET'S DISCUSS!

PRESENTERS/PANELISTS

MICHELE LEMIEUX PE, SUPERVISOR, MONTANA DAM SAFETY PROGRAM

MATTHEW LINDON PE, WATER RESOURCE ENGINEER, LOUGHLIN WATER ASSOCIATES

DEVON MCLAY PE, SENIOR GEOTECHNICAL ENGINEER, GEOENGINEERS, INC.

SETH THOMPSON PE, DAMS & HYDRAULICS ENGINEER, GANNETT FLEMING

MODERATED BY

NATHANIEL GEE, PE, MANAGER, DAM SAFETY GOVERNANCE AND OVERSIGHT, TENNESSEE VALLEY AUTHORITY

EXTENDED ABSTRACT

Let's be honest, dam computer models are fun to work on. They are like video games with dirt and water, math and science. So, there is a tendency to jump in before thoroughly planning a strategy.... both a modeling strategy AND a communication strategy. It is Ready, Fire, Aim–resulting in an expensive modeling effort that often misses the targets.

A good *modeling* strategy should paint a clear picture of the path forward, lay out the loading conditions, calibration approach, obstacles, considerations, data sources, etc. A good *communication* strategy must tell the same story, clearly and without jargon. Even when dealing with knowledgeable regulators and owners, the successful modeler needs to justify the reasons for undertaking an often expensive and time-consuming endeavor.... in advance. This all seems like good common sense. However, this is not always being done in the dam safety community. Talk to any dam safety regulator and they will tell stories of the poorly strategized and communicated model that leaves them with more questions than answers.

In this 90-minute Soap Box session, we first hear from "Seasoned Professionals" that will clearly lay out what they see as the problems with dam modeling in the industry. Next, we hear from Young but Professional and experienced modelers with their innovative solutions for advancing communication of modeling strategies. Key items that must be determined (and communicated) prior to modeling, depend on the engineering/modeling discipline.

This discussion will be followed by 20 minutes of "Audience Reverse Q & A" (i.e., questions for the audience and panel responses). A written summary of ideas, approaches and next steps will be the deliverable from the soap box for further consideration by ASDSO. It is our hope that this session will reinvigorate the conversation around best practices for dam modeling and inspire the industry with practical solutions to model and communicate more effectively.

Read on to hear Modeling Communication Perspectives from panelists and questions from the moderator:

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M. LEMIEUX – BEGIN WITH THE END IN MIND – COMMUNICATING TO DAM OWNERS AND REGULATORS



Dam computer modeling should always follow famous author Stephen Covey's Effective Habit #2: ***Begin with the End in Mind***. In other words, think about how you would like something to turn out – before getting started. Too often, this important habit is not followed. One means of adhering to this habit is for the engineer to assure they can communicate the following four items *in advance* to dam owners and regulatory agencies:

1. Model Purpose: Provide a detailed description of **why** a model is needed. Include a Problem Statement and the Justification. For example (actual dam in Southwest Montana):

Problem Statement: The toe drain is fractured 40 feet into the embankment and sediment is present whenever the drain flows. It is not feasible to repair the drainpipe; the drainpipe must be abandoned, and a replacement drainage system constructed closer to the toe of the dam. The replacement drainage system must properly drain the embankment and not negatively impact water moving through the foundation when the reservoir is full. The reservoir is currently restricted by 30 feet by order of the state so existing piezometers are of limited value in designing a repair as they don't reflect full pool. Unfortunately, measurements taken before the restriction have problems, limiting usefulness.

Justification A seepage model can be used to simulate the impact of full reservoir pool on various replacement drain alternatives and help choose the best one. A model will allow evaluation of the drain in different locations and at different depths and will also allow evaluation of a stability berm at the toe. Without a model, there is an increased likelihood that the replacement drainage system could be ineffective.

The problem statement and justification should be light on jargon, describe the problem, anticipate questions of the reader and most importantly, be specific. Always include consequences without the modeling effort.

2. Model Assumptions: Communicate big picture assumptions that:

- ✓ Are important to the regulator.
- ✓ Could invalidate the results if incorrect.
- ✓ Have a significant cost component.

For each assumption that meets these criteria, include:

A. An *Explanation* of the assumption, limiting jargon and acronyms.
B. The *Reason*, it is necessary to make this assumption.
C. *Consequences*, if assumption is incorrect.

3. Possible Outcomes: It is important to consider all realistic outcomes. Speculate, in advance, what you think you may (or may not) find.

Highest Probability Outcome: *What does your "spidey sense" tell you to expect? Be completely honest with yourself and client.*

Other Possible Outcomes *Consider other outcomes, even if less desirable.*

Important! Almost all modeling efforts have benefits, even non-working models. For each identified outcome, the modeler must speculate and communicate these benefits.

4. Anticipated Level of Effort: This is important to remain within the budget and keep expectations realistic. Think of communicating anticipated level effort with an "If Then Else" statement: *IF we can quickly calibrate the model and make reasonable model runs, THEN the level of effort will be X, ELSE, should the lack of data be an obstacle and the model not calibrate, we may need to drill additional wells with level of effort Y.*

***To Summarize**: Effective model communication must be crafted towards the target audience. The modeler must anticipate the questions and needs of that audience.*

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MATT LINDON – APPARENT VERACITY, MODELS LIE AND LIARS MODEL, KNOW YOUR AUDIENCE, SIMPLIFY



For engineers, getting the right answer is not the problem, communicating it is.

Take Global Warming or Y2K for example of bad communication. Using the right tools, models and equations, assumptions and data is key. Communicating, teaching and selling the proper problem and solution to the various clients, regulators and political powers-that-be is possibly the most important part of the process. This includes the media, lawyers, politicians and the public that think we fake elections and moon landings.

We have all heard the medical doctor learning process for procedures. Learn it, do it, teach it. I contend that our learning process should consist of learning models, doing models, teaching models and finally defending a model in a court of law where everyone throws stones at you to create reasonable doubt and throw your entire model out. I had such an opportunity.

I worked on a dam failure trial for 5 years, because of Covid, and learned some good lessons. After my client's failure I did the field forensics and had a fair idea of what had happened but convinced myself and the client that I had to do a lot of expensive HEHMS and RAS 2D modeling to prove it. They agreed to an unlimited budget (since they all think they will win, and the other side will have to pay for it). Not only would we prove that this flood was an Act of God or a 'Force Majeure' – something unavoidable - we would also satisfy the 'but – for' defense that the results would have happened 'but – for' (with or without) or our contribution.

I was stoked and modeled my face off and did some of the best work of my career modeling and calibrating the model and communicating my results to our legal team. It was like playing with a slot machine or Turbo tax where the numbers go up and down all day long. I can almost hear the bells ringing. It is my concept of Apparent Computer Veracity where everything looks true on computers. I can make a model say anything. Liars model and models lie.

Sometimes we approach models with a preconceived notion of what we want to prove and justify with Apparent Computer Veracity. Sometimes we can back into the preconceived solutions that we want. Sometimes we adjust reality to fit the model. Everything must be tied to reality. Regulators are savvy and resistant to this backing-in process and owners usually think it is too fancy and will cost more money. What we must strive for is the truth, no matter how surprising or inconvenient. Garbage in equals garbage out.

I spent years recreating the storm and rain-on-snowpack event in data free NE Nevada. Big country, no precipitation or flow gages. I calibrated everything to hypothetical and to real events, data, measurements. I used contiguous and equivalent data employing normalized regressions, considering proximity, area, altitude, elevation angle and aspect. I had 1000 photos with circles and arrows and a paragraph on the back of each one explaining where it is at.

I even learned and used the latest versions of the models since my lawyers said my 1974 HEC-I punch card version would be laughed out of court, even if the algorithms were the same. I surveyed cross sections and highwater marks and calibrated my 50 miles of routing through canyons and across an alluvial plane and over several dams and bridge crossings. I balanced grid size and time intervals for reasonable convergence and results over a million acres and ten days. I ran sensitivity analysis and a Monte Carlo distribution of solutions to land on the one most probable, most likely and the most defendable.

When I finished, I noticed that everything in the system worked well but the answer always depended on one undersized culvert under a highway that obviously backed up enough attenuated dam breach water to cause a second, larger breach that was responsible for the damage downstream. The answer was simple and elegant, but the robust modeling led me there and helped me communicate the results to our esteemed team of lawyers. I could not help but wonder if the solution could have been more easily formulated with better engineering and imagination. Did I model it just because I can? As a farmer rancher once cautioned me about overthinking a project – "We ain't building pianos here." Simplify.

When we presented all the technical results at a mock trial and watched the resulting jury deliberations, we realized that we had missed the mark. Even as I patiently and respectfully, but not condescendingly explained units of CFS and ACFT, they were more concerned with my tie and haircut than they were about my infiltration rates or Manning Coefficients. I felt like screaming "You can't handle the math' but my team told me not to get mad or try

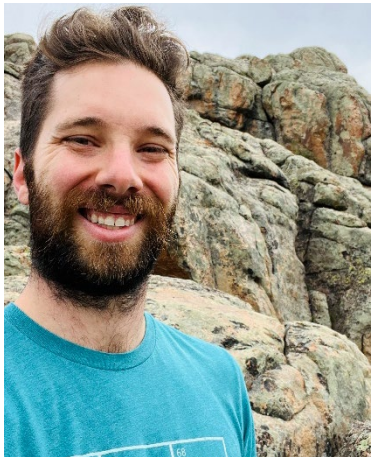
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to be funny. That would happen naturally. They threw 99% of the technical models out the door and said it was a big flood and no one was responsible. It was an Act of God. Force Majeure. It would have happened anyway. They were not wrong.

When the trial came, we threw all the technical stuff out the window and after getting the jury to like me, (they have to like you) I looked them in the eye and told them, " You live here and were here for this storm and you know that was not a 20 year storm as proposed by the plaintiff, or a 100 year storm as required by the state, it was an Act of God." They liked my casual Bola tie and my professorial ponytail and agreed with me. After two hours of deliberation, they completely exonerated the defense of any wrongdoing. The other side had to pay the legal fees.

The moral of the story is consistent communication and calibration of your client and your audience as well as your data and model is the key to elegant yet robust problem solving. Start with what answer you want to find and who and how you want to communicate it. Then back up and perform a minimal tool analysis, like they do for dam work in wilderness areas. Sometimes pack-horses are more efficient than helicopters when fixing dams in high places. What is the least complicated way of arriving at the desired solution. What are the requirements, what are the limitations, what are the desired results. Often the most elegant solution is the simplest – Occam's Razor. Robust models have their place, but make sure it is necessary, fitting and proper. Don't use a bazooka to kill a mosquito, even if you want to, and even if you can.

DEVON MCLAY - SLOPE STABILITY AND SEEPAGE MODELING



I think the purpose of a dam seepage and/or stability model should be not only to provide support to your opinions, but to illuminate the extremes and what-if cases. Depending on the complexity of a model, there can be many parameters critical to deciding whether or not the factor of safety or seepage flow rate is acceptable. These critical parameters, e.g., soil strength and permeability, are often estimated based on existing data without the opportunity for an involved field exploration and laboratory testing program. Even with a rigorous set of data, plenty of uncertainty and questions remain. We apply factors of safety (FoS) in limit equilibrium modeling to account for uncertainty. But do we check to see if that tells the whole story? What if I run the parameters to the low end of the (reasonable) range, does it drop the FoS to 1.1 or less than 1? We can use the models to check if the existing or proposed explorations are sufficient.

Modelers should be asked and be able to respond clearly to questions like: How will you account for spatial variability, such as potential anomalies in the foundation or dam construction? Is the site reasonably represented by a 2D limit equilibrium model or is more complexity required to capture the nuance of the real thing?

I suggest that modelers should take the opportunity to meet with the owners and regulators at the outset of the modeling process to communicate how they will address the uncertainty of the data gaps, why their decisions for simplification or increasing complexity are appropriate for the project. And then upon delivering the results, communicate not only the 'answer' but the things that were illuminated along the way to the engineers, regulators, and owners that review it.

SETH THOMPSON - HYDROLOGIC & HYDRAULICS MODELING



As with most numerical engineering models, Hydrologic and Hydraulic (H&H) models—inclusive of watershed modeling, (1D and 2D) hydraulic models and three-dimensional (3D) computational fluid dynamics (CFD) models—inherently contain uncertainty. It is because of this uncertainty that modelers need to communicate expectations, anticipated assumptions and parameters, quality control procedures, and project goals with the client and regulator at the beginning of a modeling effort and throughout the duration of the project.

In hydrologic modeling, we often use simplified estimates to approximate complex physical processes. Examples of this include estimates for infiltration, unit hydrograph development, and temporal rainfall distributions. Similarly, in hydraulic dam breach modeling we use estimates for surface roughness (i.e.,

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Manning's n), and assume breach parameters using empirical methods and published guidelines; further, loss of life (LOL) estimates are made using an estimated population at risk (PAR) and an informed warning time assumption.

Once an initial model is developed, my group has had success holding a progress meeting with regulators to discuss the modeling approach, key assumptions, and preliminary results. This practice allows the regulator to provide feedback at an early stage of the project, and for the modeler/engineer to revise the model prior to using its results for the next phase of the project, when applicable.

For example, a spillway assessment may consist of a spillway capacity estimate coupled with a hydraulic dam breach model and a hydrologic model of the watershed to verify/determine the hazard potential classification and develop the inflow design flood (IDF). Certain parameters used in the dam breach model, such as breach width or formation time, to name a few, can greatly impact the hydraulics and timing of the flood wave, impacting the PAR estimate, which in turn may change the hazard potential classification.

Hydrologic modeling parameters, such as initial losses, infiltration rates and unit hydrograph developments can have a similar impact by either increasing or decreasing the magnitude of the IDF. These assumptions have great economic implications, especially when considering rehabilitation alternatives, should the spillway be deemed to have inadequate capacity. Discussing the modeling approach, key parameter assumptions, and preliminary results prior to using the results for design prevents headaches, builds confidence, and saves time and money.

NATHANIEL GEE – QUESTIONS FOR DAM SAFETY COMMUNITY (TO BE DISCUSSED IN SESSION)

Questions for Consultants & Regulators:

1. How does your organization QA/QC models given that many require specialized experience the owner/regulator does not have in house? For example, is it becoming harder to verify if all you are getting is garbage in/garbage out as the specific modeling areas become more specialized (If so, how do we combat this)?



Questions for Dam Owners

1. Also how do you go about selecting an organization to conduct modeling when the owner/regulator does not have in house staff who have that specialization? Basically how do you ensure a consultant has needed expertise and the model can deliver when there is no in house expertise to validate this?
2. What criteria do you use to select a modeling software? For example, do you predefine based on what you are familiar with or have worked with in the past, or do you define a final product and allow consultants to propose various software's to get that final product?

Questions for Everyone

1. How comfortable are you with increased use of Artificial intelligence in modeling? Do you think we should be actively pursuing more use? Less? Do you think that there needs to be more validation from the research/academic side before it is used in practice?
2. How do we best communicate with stakeholders that have varying levels of expertise and interest?

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Michele is a geotechnical engineer, that often gets dragged kicking and screaming into hydrology. She is the longtime manager of Montana's Dam Safety Program. She is active in ASDSO: a past Board member, frequent presenter and active on committees. Always an advocate of seeing things from the often misunderstood "regulator's perspective" and using risk in decision making. Prior to her move to Montana, she worked in the Utah State Engineers office. She is a graduate of the University of Utah with a master's degree in civil engineering. Known to be a bit obsessed with skiing, she also likes to mountain bike and garden.

Matthew Charles Lindon, PE
Guru-Hydrologist, Temporary Teacher, Aspiring Writer



Matt was born under an Eisenhower sky, near a Kerouac stream. He was raised and prepped in New York, educated at Notre Dame and has lived in Park City Utah for 45 years. He started as an Engineer building Deer Valley Ski Resort and perfecting the Park City lifestyle. He worked as Assistant State Engineer and Hydrologist for the State of Utah on dams and rivers, snowpack-runoff and groundwater sustainability, special investigations and technical publications. He currently consults at Loughlin Water Associates and Otis Bay Ecology as a hydrologist, field engineer and expert witness.

Matt teaches Sustainable Water Resources at the University of Utah Department of Civil Engineering. He also teaches all grades regionally about water and snow, droughts and floods, climate and weather. His other passion is writing and blogging, and he has contributed extensively to the local newspaper, The Park Record, and other regional publications such as The Wasatch Sports Guide, Olympic Literature Review, High Country News and Notre Dame Technical Review. He loves skiing and biking, backpacking and hiking, camping and canoeing. He lives with his wife Tracey and super-frisbee dog Eva. He can be found at his blog at waterandwatever.blogspot.com

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Devon is a geotechnical engineer focused on dam safety projects. He has about 10 years of experience working primarily in the pacific northwest and mountain west regions. Devon has developed seepage and stability models for embankment dam projects to evaluate existing performance, the performance of proposed design modifications and new dam concepts. He has also worked with owners and regulators to define potential failure modes, evaluate risk, develop emergency action plans, exploration programs, and instrumentation systems. Devon lives in Lander, Wyoming where he tries to climb rocks year-round.

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Seth Thompson, P.E. is a dams and hydraulics engineer working with Gannett Fleming specializing in dam design and rehabilitation, and hydraulic and hydrologic design and analysis for design and assessment projects of dams and hydraulic structures (spillways, outlet works, and stilling basins) in relation to storage, flood control, and hydropower reservoirs. Technical specialties include general, and labyrinth and piano key spillway design, hydraulic spillway assessments, 3D computational fluid dynamics (CFD) modeling, 2D hydraulic modeling, detailed watershed analyses, and dam breach analyses. Seth is proficient in the use of Flow-3D, the U.S. Army Corps of Engineers' (USACE) HEC-RAS and HEC-HMS software, Esri's ArcGIS suite, and AutoCAD Civil 3D. He received a bachelor's and master's degree in civil engineering from Utah State University (USU) and completed his M.S. thesis at USU's Utah Water Research Laboratory studying arced labyrinth weir hydraulic performance while also managing the data collection and reporting efforts for multiple physical hydraulic model studies.

Nathaniel Gee

Nathaniel Gee is a professional civil engineer and serves as the Manager of the Dam Safety Oversight and Governance Group for the Tennessee Valley Authority helping guide policy for the TVA's Dam Safety Program. Prior to this he served as Regional Engineer for the LC Region of the Bureau of Reclamation helping maintain some of the largest dams in the United States. He has performed various inspections including rope access and dive inspections of some of nation's largest and oldest dams including Roosevelt and Horse Mesa dams on the Salt River in Arizona, Parker Dam spanning the Colorado River near Lake Havasu City, and the majestic Hoover Dam. Nathaniel received a bachelor's degree in Civil Engineering from Brigham Young University, and a master's degree, also in Civil Engineering, from the University of Nevada - Las Vegas. In his spare time he loves to write and recently has his second book published, ***Of Pigs and Priests***, a romantic comedy. He lives in Signal Mountain Tennessee with his wonderful wife, Jeanine, and nine energetic children.

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DAM SAFETY PALM SPRINGS
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Effort Put on the Right Path: Safety Evaluation Risk Assessments & Risk-Informed Decision Making for Maple Grove Dam in Colorado

HUI-MING (MAX) SHIH, PH.D., P.E., CFM, CDM SMITH; JAROD ROBERTS, THE CONSOLIDATED MUTUAL WATER COMPANY; JOSEPH MACHALA, P.E., CDM SMITH; CARLOS YVELLEZ, EIT, CDM SMITH

ABSTRACT

Maple Grove Dam is located in Lakewood, Colorado and a high hazard potential dam residing in a highly urbanized area, surrounded by residential and commercial development. The dam owner, The Consolidated Mutual Water company (CMWC)and Colorado State Engineer Office (CO SEO) have worked together to review the dam failure risk and evaluate a path forward for both parties. The existing dam spillway contains two crest gates that regulate reservoir levels and is classified as a High Hazard Classification Dam due to the likely loss of life from a dam breach flood hazard. Since the released discharges from the dam are primarily controlled and limited by the crest gate spillway, the overtopping breach failure has been raised as a potential dam flood hazard resulted from the extreme flood events or the crest gate failure or mis-operation. In general, a dam rehabilitation will require significant funding, construction, and such as resources. The goal of this effort was to perform a risk assessment coordinated with transparent risk-informed decision to show the dam's dam breach consequences are associated with a low-risk designation and avoid unnecessary, substantial efforts related to dam rehabilitation, developing dam construction plans, and other associated actions. This spillway deficiency led to the development of a risk-based assessment (considering only a dam overtopping failure) using the existing condition of the dam and spillway gate operation.

The risk assessment that was conducted involved utilization of new extreme storm hydrology, existing reservoir routing via the current spillway gates operation, a risk analysis workshop, an incremental consequences analysis, the development of a risk matrix, and documenting results. A critical hydrologic hazard curve was first developed by considering spillway gates operation executed normally and applied with additional hypothetical spillway gate failure scenarios. Each failure scenario was simulated within a developed HEC-RAS 2D model with existing gate standard operation procedures (SOPs) coded within the model for each scenario. A risk analysis workshop was then conducted involving eleven experienced engineers and experts to discuss potential failure initiation, progression, and each associated uncertainty, ultimately leading to the development of a failure event tree relating to an overtopping dam failure.

Based on the evaluated probabilities and estimated consequences, the risk level associated with the Maple Grove Dam (due to an overtopping failure) has been reduced from high to low. Instead of performing significant structural modifications to convey the regulatory design flood per the old deterministic State Dam Safety regulations, CMWC will pursue other improvements, such as optimizing the operational procedures for the crest gates and improving the existing flood warning system.

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Key Words: Risk Assessments, SQRA, Dam Breach, Decision Making, HEC-RAS

INTRODUCTION and POTENTIAL FAILURE MODEL MECHANISM:

Maple Grove Dam is located in Lakewood, Colorado and owned and operated by is The Consolidated Mutual Water Company (CMWC). It is a water supply dam and classified as a High Hazard Classification dam, which, by SEO policy, indicates expected life loss consequences, by deterministic depth and velocity criteria, resulting from a sunny-day dam breach. By old State Dam Safety deterministic criteria, gate spillways were not permitted, and a High Hazard Classification Dam had to safely pass the Probable Maximum Flood (PMF), regardless of risk. Under new SEO Dam Safety regulations and guidance, risk analysis can be used as part of SEO Risk Informed Decision Making to evaluate the safety of existing dams and allows mechanical spillway rates to be evaluated in terms of risk and reliability. In accordance with the Maple Grove Dam Hydrologic Analysis Report (CDM Smith, 2022), the PMF inflow hydrograph to the Maple Grove Reservoir has a peak flow of about 40,000 cfs. However, the existing crest gates spillway can only convey a maximum discharge of about 5,400 cfs without overtopping the dam. The maximum spillway capacity is about 14% of the PMF. And the flood attenuation at the existing reservoir is limited during the PMF event. In addition, the existing crest gate spillway capacity can be moderated or reduced due to crest gate operation failure, misoperation, or failure of power and telecommunication equipment. Main dam embankment materials would be erodible under sustained overtopping flows through the dam crest. Therefore, overtopping potential failure mode (PFM) for the dam embankment was evaluated under hydrologic loading conditions, recommended by the Colorado Office of the State Engineer.

EXISTING CREST GATE OPERATION AND HYDROLOGIC HAZARD CURVES:

In this project, a HEC-RAS 2D model was created to simulate the reservoir routing complying with the standard operating procedure (SOP) of the existing two crest gates (**Figure 1**) for all 24 inflow hydrographs from all three Colorado Regional Extreme Precipitation Study (REPS) storm types with 7 Annual Exceedance Probabilities (AEP) (1x10e-1 through 1x10e-7) and the three REPS deterministic Probable Maximum Precipitation (PMP) storms with different durations. The resulting probabilistic hydrologic hazard curve (probabilistic reservoir flood loading) was determined for Maple Grove Dam.



Figure 1. Maple Grove Dam Spillway, Looking north toward the upstream side of the Gate #2 (10’ Height x 40’ Width) (left) and the Gate #1 (6’x30’) steel crest gates (right)

Reservoir Routing based on the Existing Crest Gate standard operating procedure.

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In according with the Maple Grove Dam Spillway Gate Operations Plan Revisions (CWCB 2018), the current spillway gate operations are described below:

A. Set for All Times

1. Gate #1 (6'x30') is maintained at elevation, 5528.1 (NAVD 88) (5526 Local). And Gate #2 (10'x40') is maintained at elevation, 5532.1 (NAVD 88) (5528 Local).
2. Normal reservoir water level = 5527.1 (NAVD 88) (5525 Local).

B. During Storm Event

1. When reservoir level rise to 5533.6 (NAVD 88) (5531.5 Local), initiate lowering of Gate #2 (10'x40').
2. Check reservoir level 10 minutes after gate lowering is initiated (both gate crests should be at elevation 5528.1 (NAVD 88) (5526 Local))
3. If the reservoir level is below 5534.1(NAVD 88) (5532 Local), stop lowering Gate (10'x40')
4. If the reservoir level is above 5534.1 (NAVD 88) (5522 Local), initiate lowering of Gate (6x30) and continue lowering Gate (10x40) in tandem.
5. Check reservoir level 5 minutes after tandem lowering begins (both gate crests should be at elevation 5527.1 (NAVD 88))
6. If the reservoir level is below 5534.6, stop lowering both gates.
7. If the reservoir level is above 5534.6, continue tandem lowering both gates.
8. Check reservoir level in 5 minutes (both gate crests should be at elevation 5526.1 (NAVD 88))
9. If the reservoir level is below 5535.1(NAVD 88), stop lowering both gates.
10. If the reservoir level is above 5535.1(NAVD 88), continue tandem lowering both gates to spillway crest 5522.1 (NAVD 88).

A USACE HEC-RAS v6.2 unsteady flow model was created that includes the inflow hydrographs of the PMP and AEP storms, the Maple Grove Reservoir stage-volume storage, and the crest gate SOP. The Maple Grove Reservoir element was set to an initial elevation of 5,527.1 feet (NAVD 88), the normal operating level of the reservoir. The various spillway gate scenarios were modeled using the HEC-RAS "Rules" and "T.S. Gate Openings" functions for the Maple Grove Dam SA/2D connection.

Storm Hydrologic Hazard Curves

The critical storm hydrologic hazard curve was utilized to calculate dam overtopping probabilities for the Maple Grove Dam overtopping failure analysis. In addition to the normal spillway gates operations, the critical storm hydrologic hazard curve for each spillway gate failure scenario were also determined. Consequently, a total of four gate conditions scenarios were developed in the HEC-RAS model:

Scenario 1) Both Gates Operate at Normal SOP

Scenario 2) Crest Gate #1 (6-foot x 30-foot gate) Fails, Crest Gate #2 (10-foot x 40-foot gate) Operates Normally

Scenario 3) Crest Gate #2 Fails, Crest Gate #1 Operates Normally

Scenario 4) Both Gates Fail

These were developed to simulate the conditions of structural failures of the gates, or failure to operate as intended. The critical storm hydrologic hazard curves with all gate conditions were determined and are illustrated in **Figure 2**. Next, these critical hydrologic hazard curves were used to calculate dam overtopping probabilities for the Maple Grove Dam failure event tree.

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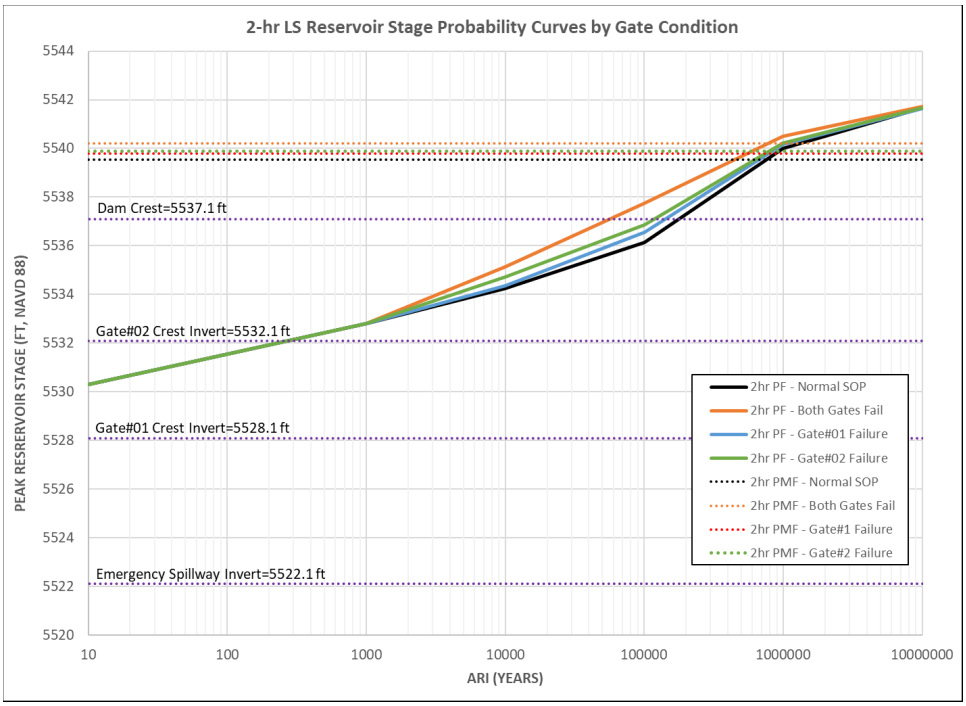


Figure 2. Critical Storm Hydrologic Hazard Curves by Gate Conditions

DAM OVERTOPPING BREACH AND CONSEQUENCES ESTIMATION:

Dam Overtopping Breach

The potential failure mode (PFM) caused by dam overtopping was simulated and evaluated. The CMWC and CDM Smith discussed the proposed PFM initiation, continuation, and intervention with CO SEO on June 10, 2022, and October 10, 2022. Importantly, at the PFM workshop, we walked through spillway gate operation, electrical, mechanical, systems, and human steps in a fault analysis framework. The team then made expert elicitation on the likelihood of spillway gate failure scenarios to be used in the overtopping PFM risk event tree. We were also informed by published literature on gate failure probabilities. The expert's best estimate of gate failure probabilities by scenario was prepared and included in the join probability of the flood induced overtopping dam failure. Incidentally, the workshop greatly improved all parties understanding of the risk and reliability of the spillway gate system, beyond what previous deterministic regulation had provided, and we were able to identify a number of low-cost improvements.

In general, participants gained confidence that the Maple Grove Dam spillway gates are a reliable system with redundancies at each step of the fault analysis. The dam failure probabilities associated with an overtopping flood scenario, initial spillway gate conditions, and programmed gate operations were estimated by considering two overtopping flow conditions involving both the overtopping flow depth threshold (initiation) and the duration of overtopping (continuation) at this set overtopping flow threshold. Singh and Snorrason (1982) found that the maximum overtopping flood depth before failure for overtopping breaches ranged between 0.5 and 2 feet. They also observed that the time taken for complete failure of dam was in general 15 minutes to one hour. These principles are included in the federal dam breach guidance by FEMA (2013), USBR (1998), and USNRC (2013). Therefore, the Maple Grove Dam breach was assumed to initiate at an overtopping depth of one foot above the dam crest and with a duration of 15 minutes. It was assumed that there will be no intervention or emergency repairing while an overtopping breach is initiated within 15 minutes under extreme flood conditions. Once these two conditions have occurred, the dam overtopping breach is considered to be triggered. The conditional

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probability of the dam crest with overtopping by at least one foot with a minimum 15-minute duration was then determined for each gate condition.

The Froehlich (2008) empirical dam breach parameter equations were used for breach parameter estimation for an overtopping failure. A 2D HEC-RAS version 6.3.0 hydraulic model was created for Maple Grove Dam and the downstream area to delineate inundation extents from the base (non-breach) flood scenario, which is associated with the critical hydrologic hazard curve, and the overtopping breach scenario (dam breach). Using these flood hydraulic routing results, the Population at Risk (PAR) and incremental loss of life (overtopping dam failure life loss minus base flood life loss) were estimated for the area downstream of the Maple Grove Dam

Each flood hydraulic routing begins at Maple Grove Dam, flows northeast to the downstream end of the model domain, near the intersection of Clear Creek and N Pecos St. A constant, peak 100-year inflow (Q = 13,470 cubic feet per second, per the FEMA Flood Insurance Study for Jefferson Country, CO) was added as an inflow boundary condition at Clear Creak to consider as the initial conditions of Clear Creek while the base flood event is occurring in Lena Gulch catchment. The downstream end of the flood hydraulic routing was selected as the downstream end since it was considered there would be no additional incremental loss of life downstream of this location based on the arrival time of the breach and non-breach floods at this location (**Figure 3**).

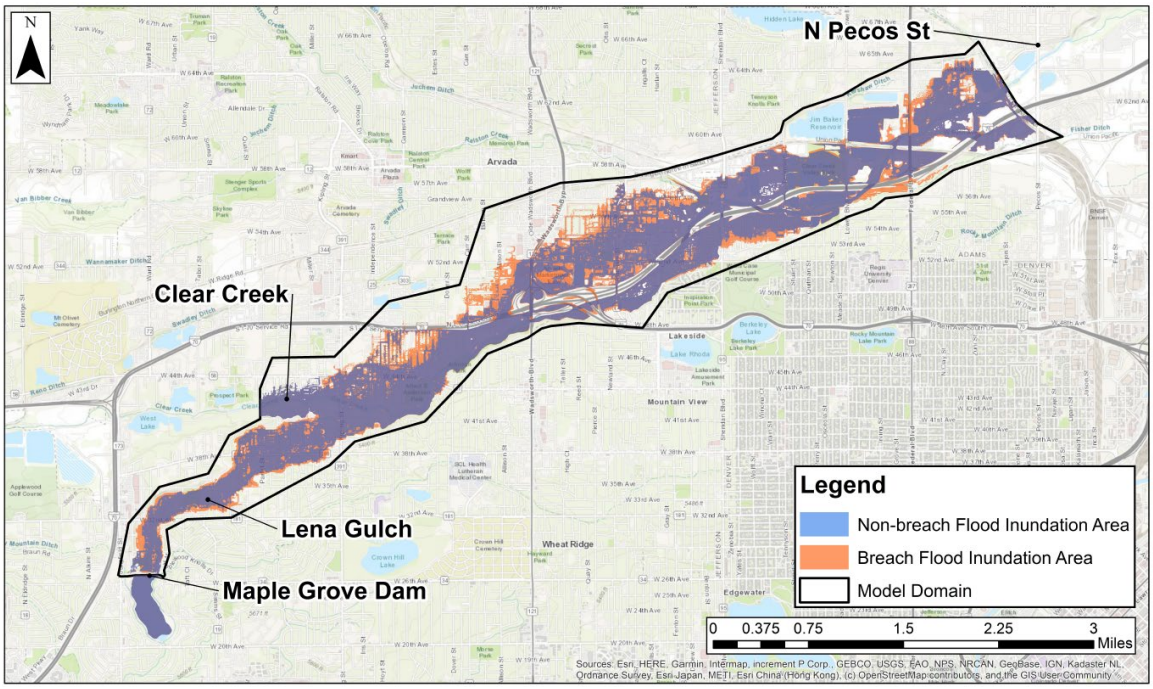


Figure 3. Maple Grove Dam Breach & Non-breach Flood Inundation Areas

Incremental Life Loss Estimation by Embankment Overtopping PFM

The population at risk (PAR) from a dam flood hazard, which consists of both residents staying in buildings and those moving through the area in a vehicle, by foot, or recreationally, was determined based on the approach outlined in the Guidelines for Hazard Classification and Guidelines for Hydrologic Hazard Analysis developed by the State of Colorado Dam Safety Branch. This approach was used to determine the likely dam flood hazard consequences downstream of Maple Grove Dam using the simulated flood hydraulics results from the overtopping breach and non-breach HEC-RAS results in conjunction with GIS tools.

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The United States Army Corps of Engineers (USACE) National Structure Inventory (NSI) database, a point-based structure inventory with population data for each structure, was used to estimate the PAR for each at-risk structure. The USACE NSI database was developed to facilitate the assessment and analysis of natural hazards. The structure inventories include, but are not limited to, day and night population estimates, over and under 65 years of age population estimates, and structure occupancy type data. Per the Guidelines for Hydrologic Hazard Analysis (CO SEO, 2020), the USBR Reclamation Consequences Estimating Methodology (RCEM) was used to evaluate adequate warning time and the fatality rate to be used to determine the incremental life loss consequences estimation. As part of the PFM workshop, RCEM "warning adequacy" associated with a hypothetical hydrologic failure of Maple Grove Dam was evaluated. The dam is part of a regional flood control and warning system, operated by the City of Lakewood and Mile High Flood District. Details of the warning system are shown in the far right column in **Table 1** below. Also, CDM Smith also did a thorough review of dam failure case histories in the USBR RCEM manual in order to make a best estimate of fatality rates. In accordance with the USBR RCEM guidance and the discussions with the CMWC and CO SEO about the existing flood monitoring and warning systems for Maple Grove Dam, the fatality rate curve graph for "adequate warning" was utilized to compute the loss of life consequences. Per the USBR's fatality rate curves, the fatality rates were selected from the upper bound of the suggested curves to estimate a conservative loss of life. The estimated loss of life and incremental consequence level are shown in Table 1 below.

Table 1. Estimated Life Loss Potential from Maple Grove Dam Overtopping Breach

Life Loss Potential	Estimated Infrastructure Impacts	Discussion of Warning Time
Life Loss at Day Time: 2.21 (Non-Breach/Natural Flood) 4.75 (Overtopping Breach)	Day Time No Breach/Natural Flood: 485 buildings* Overtopping Breach: 1701 buildings*	1. Per the USBR Reclamation Consequences Estimating Methodology (RCEM) guidance and the discussions with Consolidated Mutual Water Company (CMWC) and CO SEO about the existing flood monitoring and warning systems for Maple Grove Dam, the fatality rate curve graph for adequate warning was utilized to compute the loss of life consequences. 2. Warning Adequacy to use for USBR RCEM analysis to estimate Life Loss from dam failure: - Reservoir is closely monitored: 2 sensors + CCTV at WTP - 24/7 WTP staff monitor reservoir - City of Lakewood operator warning sirens - Current EAP (2021) and dam breach mapping (2012) - City of Lakewood conducts regular flooding emergency exercises - PFM workshop expert agreement to use ADEQUATE WARNING. 3. CDM Smith suggested using upper Suggested Limit curve on RCEM Adequate Warning graph to be conservative. SEO agreed.
Life Loss at Night Time: 1.08 (Non-Breach/Natural Flood) 2.18 (Overtopping Breach)	Night Time No Breach/Natural Flood: 559 buildings* Overtopping Breach: 1834 buildings*	

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SPECIFIC DAM RISK CONCLUSIONS

For Maple Grove Dam, the Embankment Overtopping potential failure mode (PFM) is in relation to hydrologic loading and the existing spillway crest gates conditions and operations and was carried forward for detailed analysis based on the recent hydrologic study report (CDM Smith 2022) and the spillway crest gates standard operation procedures (SOP). The embankment overtopping PFM was thoroughly evaluated by a team including three dam safety engineers from Colorado State Engineer's Office, Dam Safety (CO SEO, Dam Safety), an engineer from Mile High Flood Control District, a team of five engineers and scientists from CDM Smith and two representatives from the dam owner, The Consolidated Mutual Water Company. The group workshop contains the PFM brainstorm and PFM worksheets, which summarize the adverse and positive factors that were considered for the evaluated PFM. Consequently, multiple event trees were developed to evaluate the potential failure likelihood and its uncertainty ranges. **Figure 3** shows one event tree of a total three scenarios developed in this study to compute the probabilities of embankment overtopping failure.

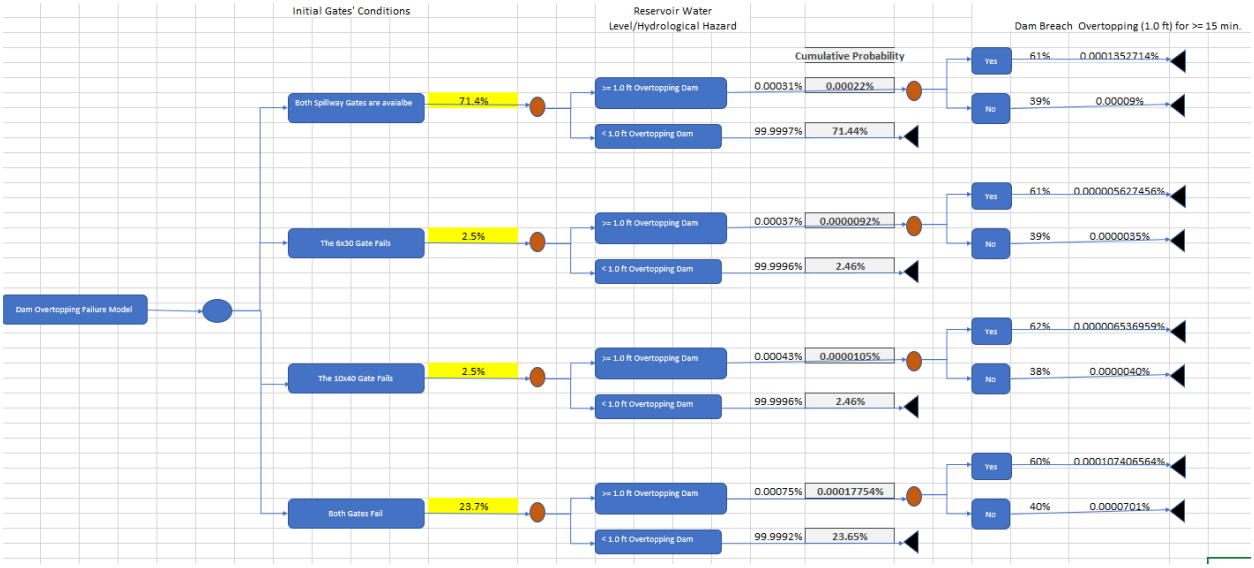


Figure 3. Event Tree of Predicting Probabilities of Embankment Overtopping Failure due to Spillway Gate Failure Scenarios

Last, per the evaluation results, referring to the chart (**Figure 4**) below, the Embankment Overtopping PFM was judged to have a LOW failure likelihood that will require increased monitoring and maintenance activities by the owner moving forward, but no structural modifications to the dam. Old State Dam Safety deterministic hydrologic criteria would have required significant structural modifications to the dam and spillway. Consequence Level was evaluated to be Level 2, which indicates an expected incremental loss of life between 1 and 10 due to the embankment overtopping breach.

Further, CDM Smith performed a rigorous sensitivity analysis on both the dam failure probability ("Y-axis" of the risk matrix) and on the failure consequences estimate ("X-axis" of the risk matrix). In terms of the sensitivity of the likelihood estimate, CDM Smith considered uncertainty in the gate failure probabilities (previous estimated by expert elicitation) and in the assumed dam overtopping failure criteria (depth and duration of overtopping). As a worst-case upper uncertainty bound, they considered the case where spillway gate failure is 100% (i.e., the gates fail in every flood scenario), and the dam embankment would fail at incipient overtopping. Likewise, CDM Smith did sensitivity analysis on the consequences estimate, considering upper and lower overall limit curves from the USBR RCEM adequate warning fatality rate vs

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Depth x Velocity (DV) plot. The recommended results are shown in the chart below and plot within the green category, which has a LOW failure likelihood and a Consequence Level 2. Even at the worst extremes of the sensitivity analysis, the hydrologic risk was shown to lie below the industry's societally acceptable risk line on the risk matrix.

After all, based on the existing Maple Grove Dam conditions, CO SEO accepted that the dam's hydrologic risk is low and satisfactory. As identified during this thorough review of the dam's hydrologic risk and reliability, CMWC will pursue non-structural improvements, such as optimizing the operational procedures for the spillway crest gates and improving the existing flood warning system; however, the risk analysis showed that major structural modifications to the dam and spillway are not warranted from a risk perspective. The benefits to the SEO's risk analysis process here were that it greatly improved all parties' understanding of the system operations, risk and reliability, and that it accurately assigned hydrologic risk, so that limited dam safety resources can be spent on risk priorities.

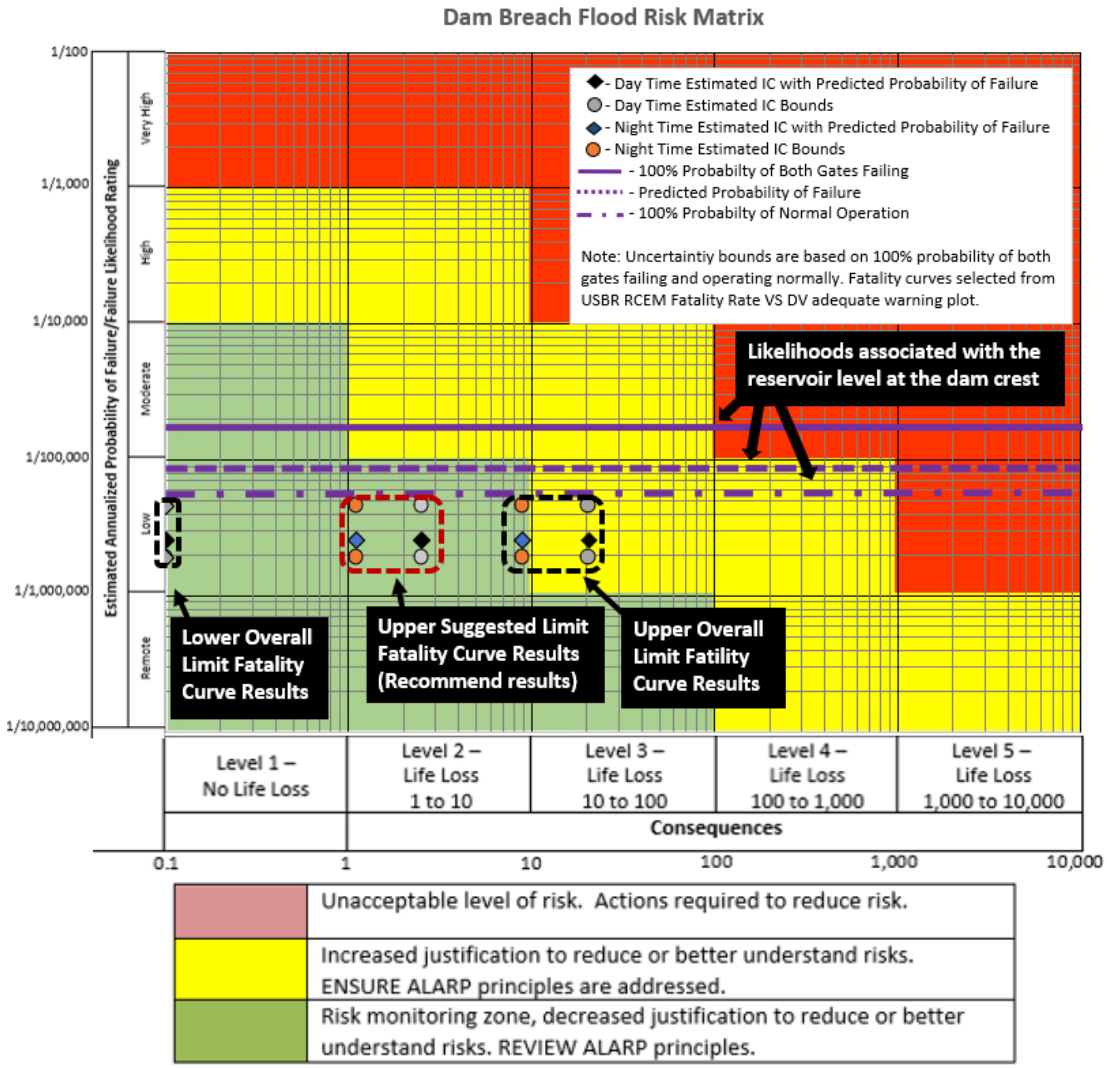


Figure 4. Risk Matrix Results of Maple Grove Dam Embankment Overtopping Failure due to Spillway Gate Failure

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

Assessing Uplift at Existing Spillway Chutes in a Post Oroville World

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ABSTRACT

The failure of the principal spillway concrete chute at Oroville Dam due to uplift from various causes has triggered many dam owners to examine their spillway chutes for the potential of an uplift failure. While there is a wealth of information on design for a new spillway chute to address uplift, there is little guidance on assessing the uplift potential of an existing spillway concrete chute. What causes uplift underneath the spillway chute? How does one assess uplift if some of the recommended defensive design measures for uplift are not in place?Current design of spillway chutes for uplift assumes a saturated subgrade with a conservative pressure distribution applied over the entire slab. Some load cases allow for a reduction in uplift for an assumed drain effectiveness. Issues of groundwater infiltration, soil or rock porosity, extent and sizing of the underdrain system, if present, and slab jointing and integrity are not part of the assessment of uplift on the slab. This paper will examine the various forms of uplift – localized, slab and systemic uplift. The cause for each of these various forms of uplift will also be discussed along with the impact of defensive measures. These causes for uplift include stagnation pressure, hydrodynamic pressure, inadequate underdrain spacing, capacity and porosity; and underdrain redundancies or lack thereof. Key takeaways from this paper are understanding of slab uplift processes, ability to assess potential failure modes for uplift at existing spillways, and to incorporate good design practices to mitigate uplift for new spillway concrete chutes.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

SQRA Methods Bringing Clarity to a Spillway Design Flow Decision

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ABSTRACT

The Minnesota Department of Natural Resources owns, operates, and regulates the Lake Bronson Dam in northwestern Minnesota. This facility is estimated to have inadequate spillway capacity, and the dam owner has decided to address this issue by replacing the existing gated primary spillway with a new labyrinth weir structure. We teamed with Freese and Nichols to design a new labyrinth spillway structure, with Barr focusing on the hydrologic and hydraulic modeling, and the determination of the design flow. After developing a 2D HEC-RAS model, we modeled a range of flood events, both with and without the dam breaching, and estimated consequences. By integrating the consequences over the range of modeled flood and failure probabilities for a set of increased conveyance capacities to consider for the new primary spillway, our team semi-quantitatively estimated risks for both the non-breach and the breach condition. The results highlighted a clear design point for the primary spillway where the incremental risks were not further reduced substantially by building a larger spillway and the overall risk magnitudes were in the acceptable range.

I.INTRODUCTION AND SETTING

Lake Bronson is located about 1.5 miles east of the City of Lake Bronson, Minnesota in Kittson County, in the northwestern part of the state. The dam that forms this lake was constructed across the South Branch Two Rivers by the Works Progress Administration (WPA) in 1937 and is owned and operated by the Minnesota Department of Natural Resources (MnDNR). The dam facility has a primary spillway and two emergency spillways. The primary spillway is located near the middle section of the dam embankment and consists of a three-bay ogee spillway structure with 4-foot-tall electrically actuated roller gates that control the flow through each 20-foot-wide bay, for a total crest length of 60 feet. The emergency spillways are represented by low sections of County State-Aid Highway (CSAH) 28 at the left and right abutments of the dam, extending for several hundred feet of crest length. The dam also has a low-level outlet.

The dam requires rehabilitation to address dam safety issues related to excessive seepage and inadequate spillway capacity that currently threaten the long-term use of the structure. Barr Engineering Co. (Barr) was retained by the MnDNR to evaluate alternatives and design the necessary upgrades such that current state dam safety rules are met, applicable industry standards and relevant design guidelines are considered, and above all, the current risk of failure and potential for loss of lives is significantly reduced. One of the upgrades will be the replacement of the existing primary spillway with a new structure, and a key question to inform the sizing of this structure is what should be the design flow rate. The design expectation is that the existing primary spillway structure will be removed and replaced with new embankment, and the new primary spillway structure will be in a new location along the embankment, slightly to the south.

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II.SUMMARY OF HYDROLOGIC MODELING

The Lake Bronson hydrologic watershed is 547-square miles, which was divided in the model into 67 subwatersheds at branches to the main drainage channels, at agricultural drainage ditches, and at major storage areas. Half of the watershed consists of agricultural land use, a quarter of the watershed consists of herbaceous land cover. Approximately 80% of the watershed corresponds to hydrologic type D soils. As shown as a dashed red line in Figure 1, a 5-foot-high levee crosses through the middle of the watershed, running from the northeast to the southwest. The levee directs all upstream runoff to flow through three release points: one at the southwest end, and two at the northeast end. The levee attenuates flows from this relatively large area into Lake Bronson, as it represents approximately 58% of the catchment area draining to the Lake Bronson Dam.

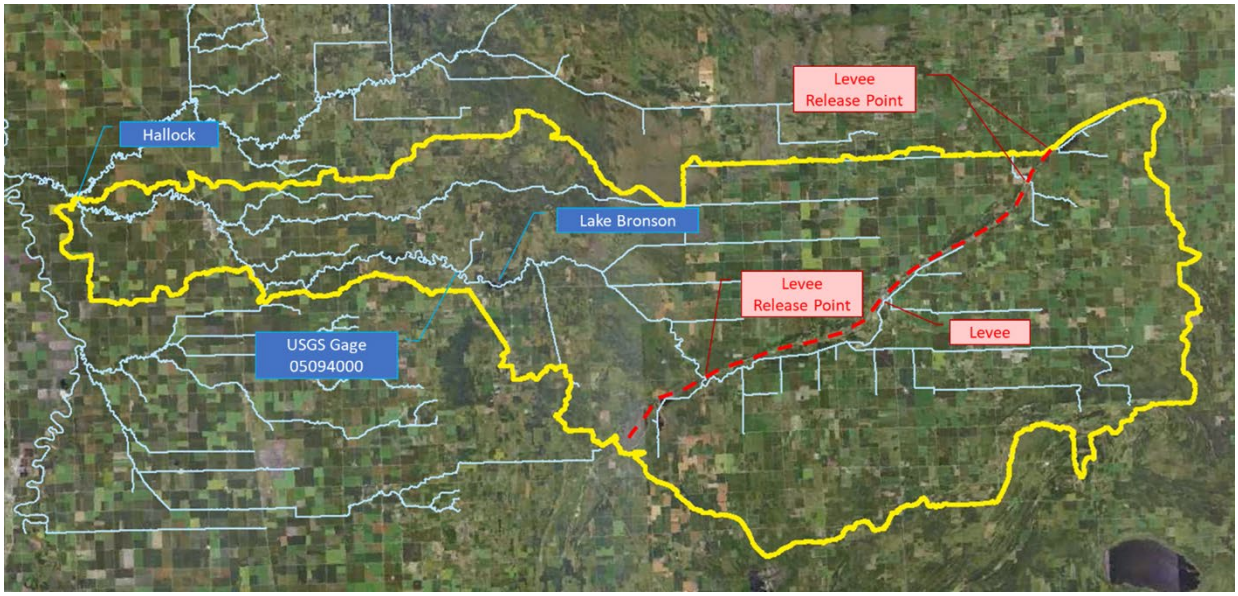


Figure 1. Lake Bronson and South Branch Two Rivers Watershed

A hydrologic model, developed in HEC-HMS by the MnDNR in 2010, was used and updated for this study. Updates to the model included: a new stage storage curve based on more recent LiDAR data and bathymetry data, a better representation of the Lake Bronson dam spillways rating curve, and updates to initial and constant loss parameters in the subwatersheds.

Review of the 94 years of flow data at USGS gage 05094000 located downstream of Lake Bronson Dam shows that the majority of peak flows correspond to events during the spring, including those resulting from snowmelt as well as contributions from rain on snow, with 16 of the top 20 peak flows in this category. One of these snowmelt events took place in 2006, the second largest peak flow of record and a significant flooding event over a large part of the Red River of the North basin. The advantage of using this event for model calibration is the availability of high-quality precipitation information as well as accurate data on Lake Bronson water levels and outflows. The other event selected for model calibration was a recent storm event in 2020, which may not have resulted in one the largest outflows, yet it showed a significant rise in the Lake Bronson water levels, hence it depicts an interesting event to characterize the watershed runoff response to a storm event that was as large as an approximately 50-year return period in some parts of the upstream watershed. Rainfall depths, event distributions, and initial snow-water-equivalent information were provided by Applied Weather Associates (AWA) for the two calibration events.

The HEC-HMS model developed for the Lake Bronson watershed was calibrated to the observed water levels in the reservoir and flows recorded at the USGS gage 05094000 that resulted from the spring event in early April 2006 (Figure 2) and the storm event on June 6-9, 2020. The observed water levels in the reservoir for both events were provided by the MnDNR in the form of manually logged lake level. The gate operations were also included in the lake level log data. While model validation is a typical practice, a validation step was not explicitly performed because our focus was on the largest storm events for which high-quality data was available.

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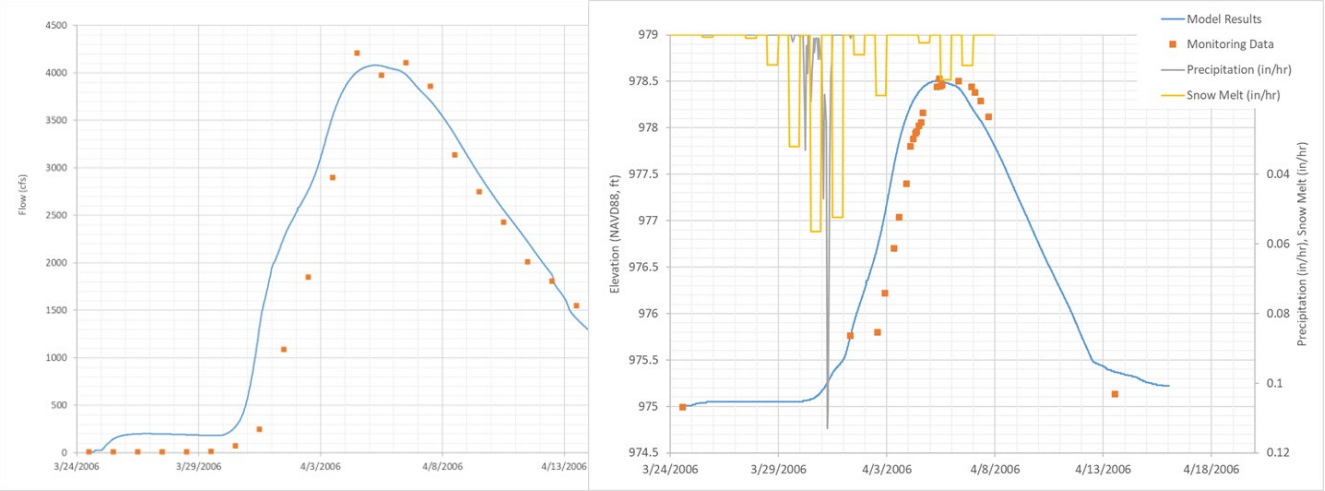


Figure 2. Calibration results of the HEC-HMS model to the April 2006 rain-on-snowmelt event

Once the model was considered calibrated to a large cool-season event, the model was used to run a set of synthetic events which were combinations of melting snowpack of varying magnitudes and 72-hour rainfall of varying magnitudes. Modeling of the watershed suggested that long-duration spring rainfall combined with snowmelt was most critical for resulting in highest peak flow rates. The AEP of each synthetic rain-on-snowmelt event as shown in equation (1) was calculated as the probability of two independent events occurring together (large rain *and* snowmelt), where *P(rain and snowmelt)* is the probability of the two events together, p_{rain} is the annual exceedance probability of the rainfall and $p_{snowmelt}$ is the annual exceedance probability of the snow water equivalent at the start of the melt.

$$P(rain\ and\ snowmelt) = 1 - (1 - p_{rain}) * (1 - p_{snowmelt}) \quad (1)$$

For each of the modeling runs, a conservatively warm temperature series was used to melt the snow, consistent with the warm series used in the modeling of the Probable Maximum Flood (PMF). The peak flows from these combinations of 72-hour rain and snowmelt resulted in the flood frequency relationship shown in Figure 3. This figure shows that the modeled peak flows of synthetic combinations of rainfall and snowmelt are a good extension of the Bulletin 17B analysis of gaged flows which are dominated by spring events. This flood-frequency curve is a critical component of the overall risk analysis for the project.

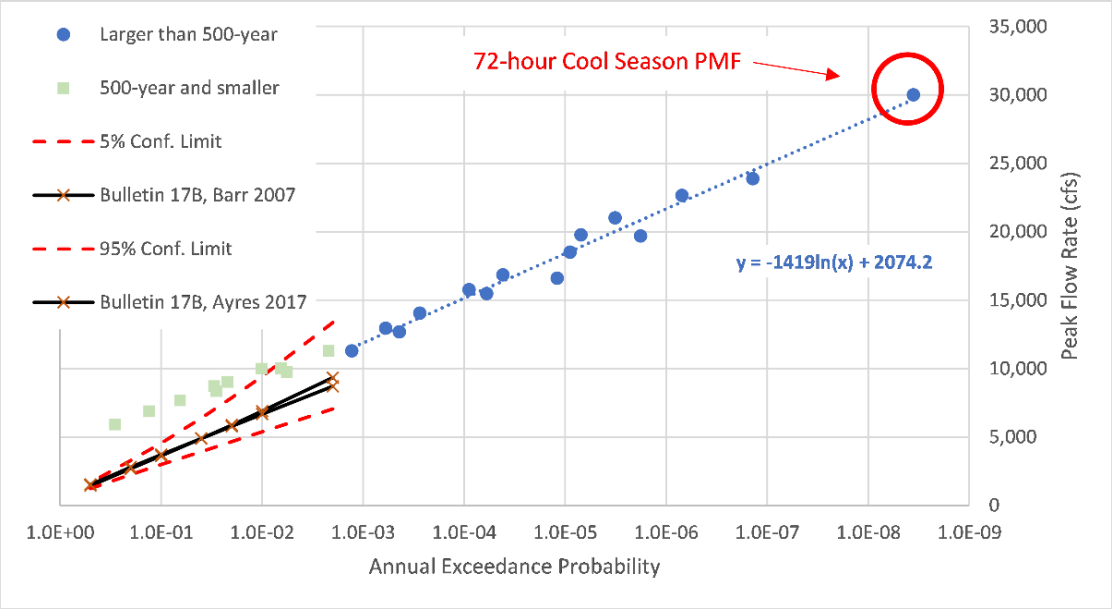


Figure 3. Adopted flood-frequency relationship for the Lake Bronson Dam

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III.SUMMARY OF HYDRAULIC AND DAM BREACH MODELING

A two-dimensional (2D) hydraulic model was developed in HEC-RAS version 6.2. The 2D model domain extends from upstream of the Lake Bronson reservoir to just downstream of Hallock, MN (Figure 4). The model focuses on the South Branch Two Rivers, but also extends sufficiently north and south to include areas that may be affected in the case of a breach at Lake Bronson Dam. The model terrain layer was developed using the MnDNR 2008 LiDAR layer with Lake Bronson bathymetry burned into the surface to model reservoir routing, particularly in the case of a dam breach. The model terrain transitions from a relatively well-defined drainage network over the eastern two-thirds of the model domain, with slopes in the order of 12.5 feet per mile, to a very flat area over the rest of the watershed where more extended breakouts and flooding can be expected. In the absence of detailed topographic information about the main stem bathymetry that was not captured by the LiDAR, a trapezoidal cross section was assumed for the channel using geomorphological relationships (i.e., bank-forming discharge is approximately the 1.5- to 2-year flow). The simplified representation of the channel cross section should have minimal impact on the results given the magnitude of the flows to be modeled, in particular those corresponding to the breach condition.

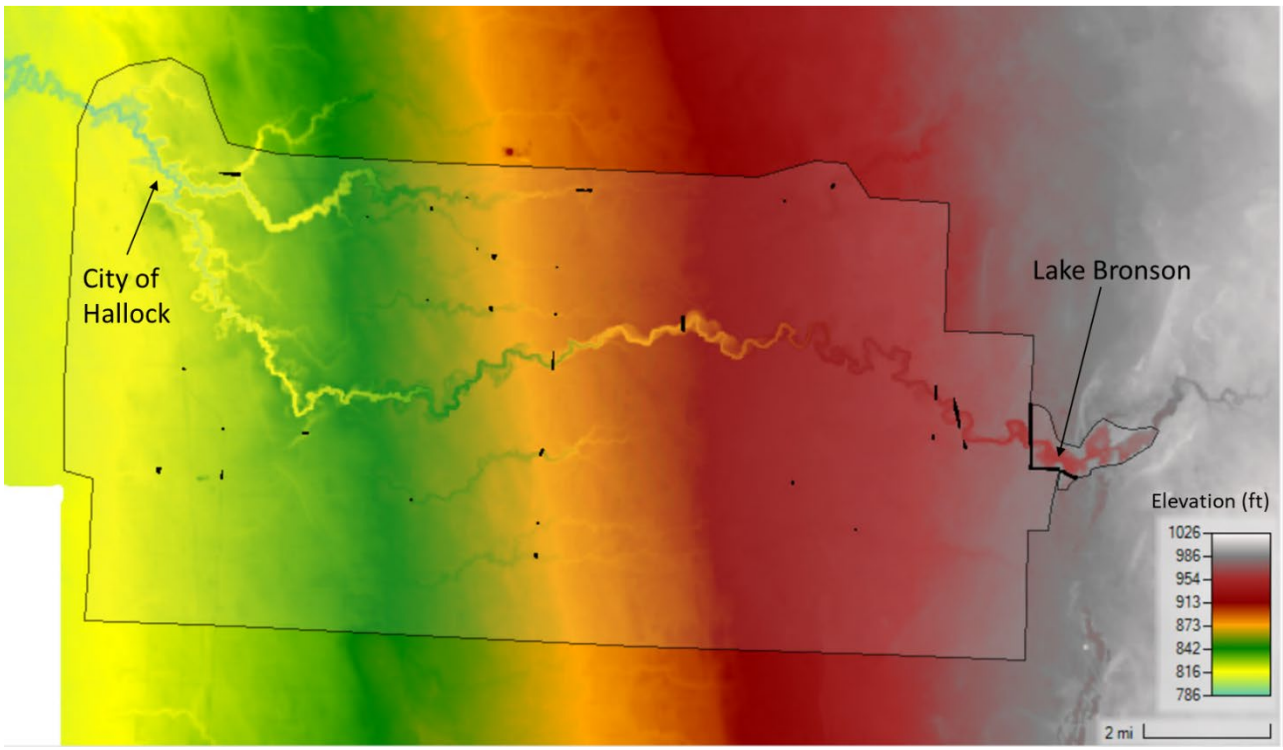


Figure 4. HEC-RAS model domain

2D Connections are placed throughout the model to incorporate important hydraulic features such as the proposed spillway as well as culverts and bridges located downstream. Boundary conditions at upstream portions of the model were defined based on the hydrographs from the synthetic rain-on-snowmelt events described previously in the section on hydrologic modeling. Five events were selected for modeling in HEC-RAS ranging from about 6,900 cfs peak flow into Lake Bronson (approximately the 75-year event) to 31,000 cfs peak flow into Lake Bronson (PMF).

The rating curves for a set of five different configurations of the new primary spillway (labyrinth weir structures), each corresponding to a different conveyance capacity, were developed by Freese and Nichols, and were included in the model, and the geometry was saved for each of the proposed spillway sizes. The spillway sizes are defined by the flow capacity of the proposed spillway when the reservoir level is at the emergency spillway crest. Note in Figure 5 where each of the spillway rating curves crosses the blue-dashed line denoting the crest of the emergency spillway. Proposed primary spillway sizes ranged from 10,000 cfs to 25,000 cfs.

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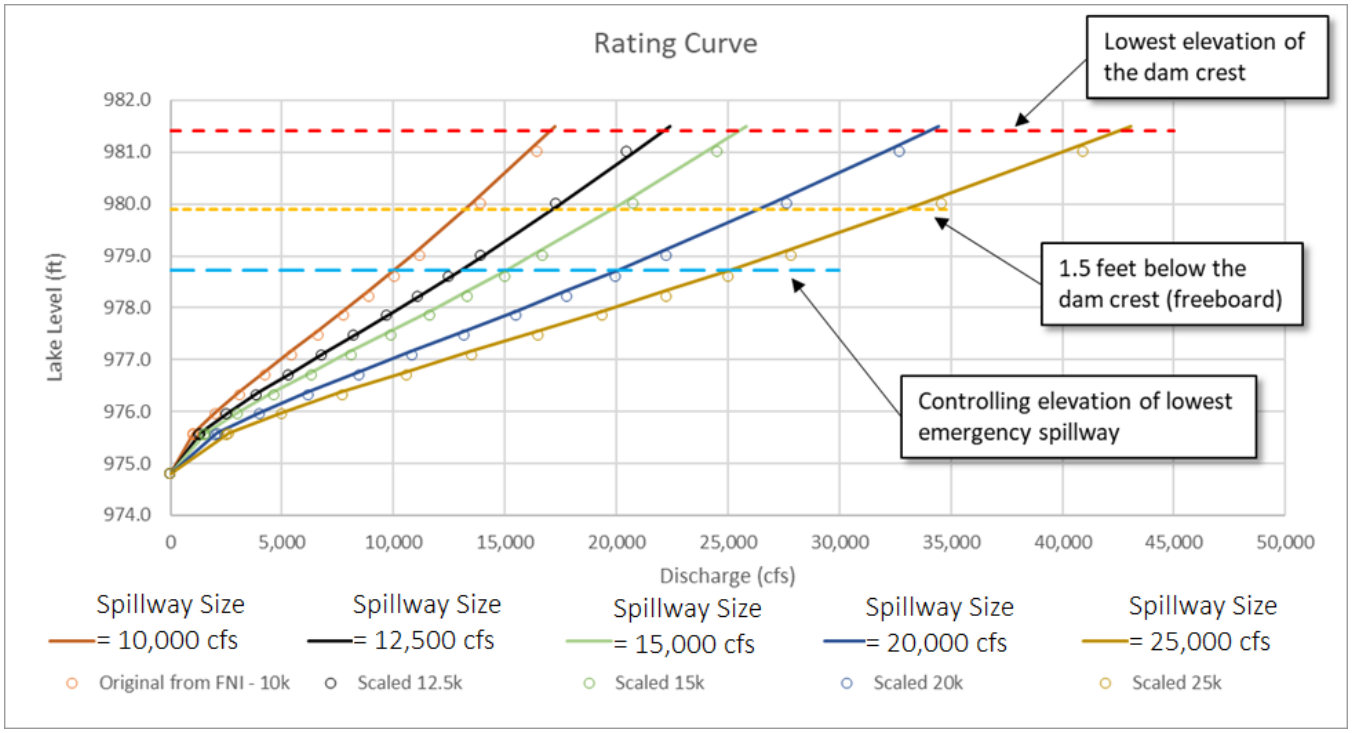


Figure 5. Primary spillway rating curves of the proposed structures

Dam breach modeling was performed for the proposed conditions to assess incremental consequences and downstream risk of the various proposed spillway sizes. As shown in Table I, two failure modes were adopted: piping through the embankment during flood events that do not overtop the embankment, and overtopping and erosion of the embankment for events where the reservoir level would rise above the flood maximum level (freeboard). Conservatively, it was assumed that if the reservoir rose into the freeboard level, it was conceivable that wind-driven waves could wash over the embankment and initiate an overtopping failure.

TABLE 1

Summary of combinations of spillway sizes and flood events with failure mode identified

Event Peak Flow at Lake Bronson	Primary spillway capacity before activation of emergency spillways				
	10,000 cfs	12,500 cfs	15,000 cfs	20,000 cfs	25,000 cfs
6,871 cfs (1.0 x10-2 AEP)	Piping	Piping	Piping	Piping	Piping
9,737 cfs (1.8 x10-3 AEP)	Piping	Piping	Piping	Piping	Piping
15,474 cfs (7.8 x10-5 AEP)	Overtopping	Piping	Piping	Piping	Piping
21,007 cfs (1.6 x10-6 AEP)	Overtopping	Overtopping	Overtopping	Piping	Piping
31,000 cfs (3.6 x10-9 AEP)	Overtopping	Overtopping	Overtopping	Overtopping	Piping

Five unsteady flow files were created to represent a range of flood events into Lake Bronson, including the PMF as the upper bound, with hydrographs input directly from the HEC-HMS model results that correspond to annual exceedance probabilities ranging from about $1.0x10^{-2}$ to $3.6x10^{-9}$. All five of the flood events were run with each of the five geometry files (representing five different primary spillway sizes) to produce a set of 25 model runs for a scenario of no dam breach, which sets the baseline condition. The same 25 model runs were repeated, but with the dam allowed to breach at the moment when the reservoir was at the peak water level during the event. The breach was chosen to occur in the location of the existing primary spillway because that is where the expected future dam embankment is the tallest, and therefore the breach would be the largest and most impactful downstream.

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IV.ESTIMATING CONSEQUENCES

A total of 50 model runs were completed to account for the combination of five primary spillway sizes (before activation of the emergency spillways), five flood events, and no breach versus breach conditions as outlined above in Table 1. For each of the 50 model runs, consequences were estimated downstream by comparing hydraulic output files from HEC-RAS to a GIS layer of building footprints. Consequences were focused on residential structures throughout the entire model domain. Various types of consequences were considered as proxies for "loss of life", including 1 foot of inundation or more, 2 feet of inundation or more, a high flood severity (hazard) category or greater, 2 feet of incremental rise from the breach, and flood wave arrival time of less than 30 minutes among some of the proxies. Having said this, the main purpose of the modeling work and analysis was to determine a spillway size for which the incremental risk in a dam breach scenario was relatively small to negligible, and as such, the overall magnitude of the consequence for a given proxy is not necessarily critical, but the incremental difference between no breach and breach conditions for that proxy is what matters.

Python scripts were developed to process the results so that, given any one of the 50 model runs, at every identified residential structure, the consequences were all calculated for that structure. For example, at structure A, the script would calculate the inundation depth and then populate a true or false value in the consequence fields associated with whether the home was inundated with at least 1 foot of inundation or at least 2 feet of inundation. Similarly, flood severity rasters were developed from the HEC-RAS results and flood severity was estimated at structure A, and the consequence field was populated with a true or false value depending on whether the severity was greater than or equal to "high". All of the consequence fields were populated for structure A, and then the code moved on to structure B, and so on. The flow chart of the python script is included as Figure 6 on the following page.

Once the script was completed and the entire database of consequences was populated for all 50 model runs (5 flood events times 5 spillway sizes, for both breach and non-breach conditions), the data was ready for the risk calculations.

V.RISK CALCULATIONS

Risk takes into account a negative consequence and the probability of that consequence occurring. Within the context of dams, risk focuses on loss of life as the consequence of interest. In the analysis presented here, the hydraulic characterization at the residential structures (e.g., flood severity hazard category, 2 feet of incremental rise from the breach) is used as a proxy for loss of life, based on the premise that these homes are where people will typically be present in the event of a flood or dam breach. The results of the dam breach analysis were thus qualified by the corresponding probabilities of the flood events simulated and likelihood of dam breaching to provide a quantification of the risk associated with each of the five proposed primary spillway sizes. We estimated the risk as the product of consequences, likelihood of the flood event, and likelihood of failure given that event (in the failure cases), and then compared that risk for the "no breach" versus the "breach" (i.e., dam failure) condition. This analysis thus focused on identifying a size of the primary spillway that would result in a relatively small to negligible incremental increase in risk.

Prior to this risk analysis, a Potential Failure Modes Analysis (PFMA) was completed where a group of subject matter experts convened and discussed the conceivable failure modes. During this process, the group collaborated on estimating probabilities of failure given a variety of conditions. The estimated conditional failure probabilities were then used in this analysis for consistency. For flood events where the rise in the lake did not reach the emergency spillways (i.e., at or less than the modeled design flow of the primary spillway), the probability of failure was defined as 0.01%, consistent with the values that resulted from the PFMA and SQRA process that was completed in parallel to this incremental risk analysis. For flood events where the rise in the lake was above the emergency spillway, but was below the recommended freeboard elevation (i.e., lowest dam crest elevation minus the appropriate freeboard amount), the probability of failure was defined as 2%, which is still small because the mechanism would most likely correspond to a piping failure. Finally, for flood events where the rise in the lake was above the maximum freeboard level (i.e., within 1.5 feet of the dam crest), the probability of failure was defined as 80% reflecting potential for overtopping of the dam, consistent again with the results of the PFMA and SQRA process. In essence this is a fragility curve for the dam embankment, capturing the low probability of failure when the emergency spillways are not activated, a higher probability of failure as the emergency spillways are activated, and a much higher probability of failure when the reservoir level is above the maximum level or over the embankment crest.

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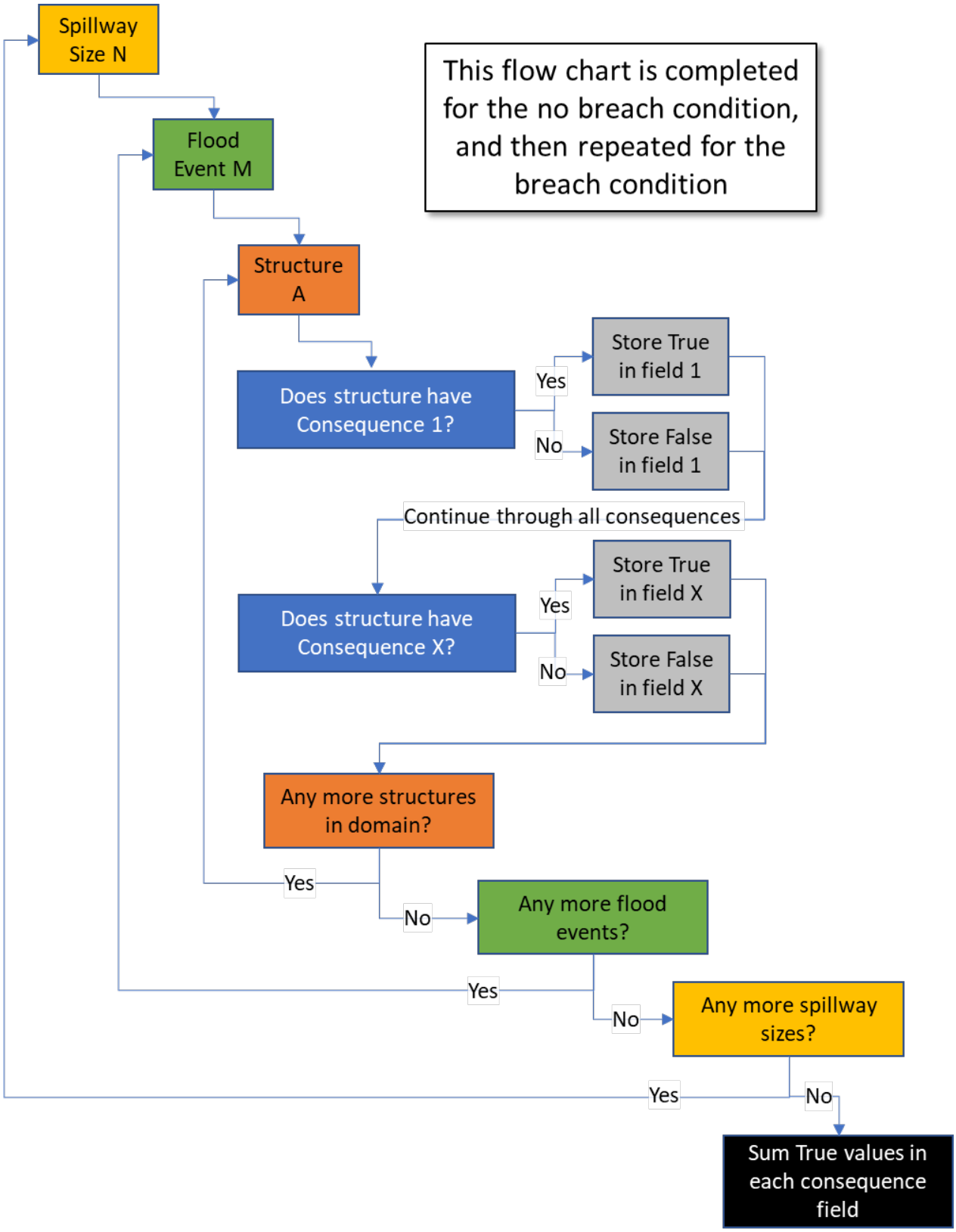


Figure 6. Flow chart for populating the consequences database in preparation for the risk calculations

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The incremental risk analysis described here results in a single quantified value of risk for each primary spillway size that accounts for consequences aggregated over a range of potential flood events that are weighted by the probability of occurrence of such events, and distinguishing the no breach condition from the breach condition. If the risk of a given primary spillway size under the breach condition is very different from the risk of the same primary spillway size under the no breach condition, the conclusion is that there is a significant incremental risk because the primary spillway size may be too small. As the primary spillway size increases, the risks of the no breach and breach condition tend to converge and at some point, the two can be considered not significantly different. Following this rationale, it could be argued that an adequate design size corresponds to the spillway capacity for which the incremental risk no longer decreases significantly as the spillway capacity gets larger and larger. In other words, designing a larger spillway would not reduce the incremental risk.

More specifically, for the no breach condition, the risk is defined as an integration (summation) of the consequences over the probability of occurrence. In equation (2), $R_{i,nb}$ is the risk for the ith spillway size under the no breach condition, $C_{i,nb}$ is the total consequences for the ith spillway size under the no breach condition that is a function of AEP, AEP is the annual exceedance probability, and m and n are the bounds on the range of AEP considered.

$$R_{i,nb} = \int_m^n (C_{i,nb})\, dAEP \quad (2)$$

For the breach condition, the risk incorporates the probability of failure on the incremental (additional) consequences. In equation (3), $R_{i,b}$ is the risk for the ith spillway size under the breach condition, $p_{i,f}$ is the probability of the dam breaching given the ith spillway size that is a function of AEP, and $C_{i,b}$ is the total consequences for the ith spillway size under the breach condition that is also a function of AEP.

$$R_{i,b} = R_{i,nb} + \int_m^n \left(p_{i,f} * \left(C_{i,b} - C_{i,nb}\right)\right) dAEP \quad (3)$$

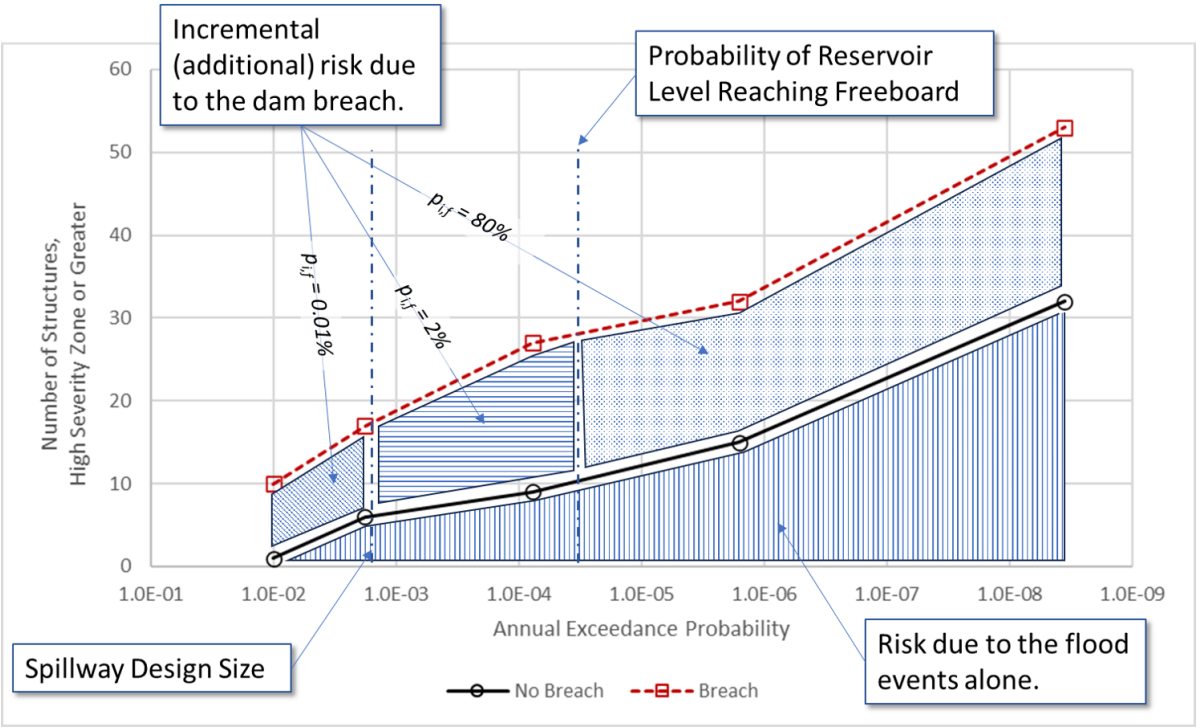


Figure 7. Diagram of the integration method to determine the risk of the non-breach ($R_{i,nb}$) and breach ($R_{i,b}$) conditions for the ith proposed spillway size; in this case, the 10,000 cfs spillway size

Figure 8 is a summary plot of the quantitative risks, R, for the breach (green bars) and no breach (blue bars) conditions for each of the five originally modeled primary spillway sizes, plus the addition of a size of 7,000 cfs per request from the MnDNR. In this figure, the consequence is the number of homes that are in a zone of high flood severity or greater. On the left end of the spectrum with a primary spillway size of 7,000 cfs, the risk associated with a dam breach during flood events is noticeably larger than the risk associated with the flood events alone, i.e., the

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no breach condition. Put simpler, the impact of a breach on structures downstream of the dam if the primary spillway size is 7,000 cfs is noticeably larger than the inundation that would happen if the dam would not fail. On the right end of the spectrum with a primary spillway size of 25,000 cfs (nearly the PMF), the risk associated with a dam breach during flood events is nearly identical to the risk associated with the flood events alone. This graph makes it clear that somewhere between a spillway design capacity of 7,000 cfs and 25,000 cfs, the risks associated with a dam breach and the risks associated with flood events alone converge, and that point appears to be somewhere between 10,000 cfs and 12,500 cfs.

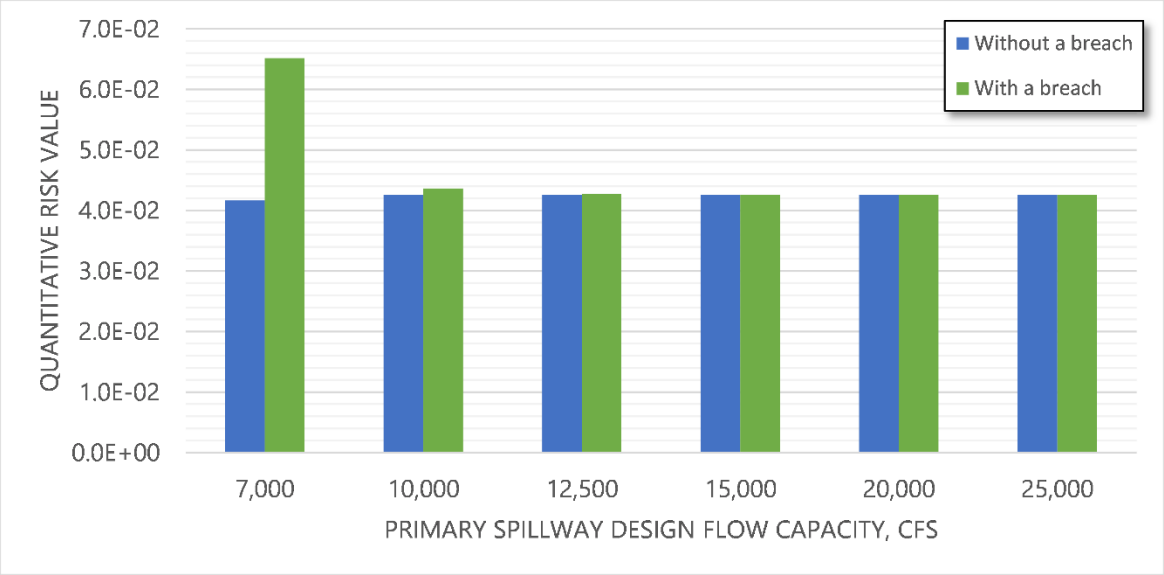


Figure 8. Risk for the range of proposed primary spillway design flows, comparing the breach and no-breach conditions, with consequence being homes in a flood severity zone of “high” or greater

Another consequence that was considered in some detail was homes inundated by 2 feet or more of water, as presented in Figure 9. The trend in the results is the same in both Figure 8 and Figure 9, suggesting that regardless of the specific definition of the consequences, the graphs consistently point to a spillway size where the incremental risk associated with a dam breach becomes relatively small to negligible.

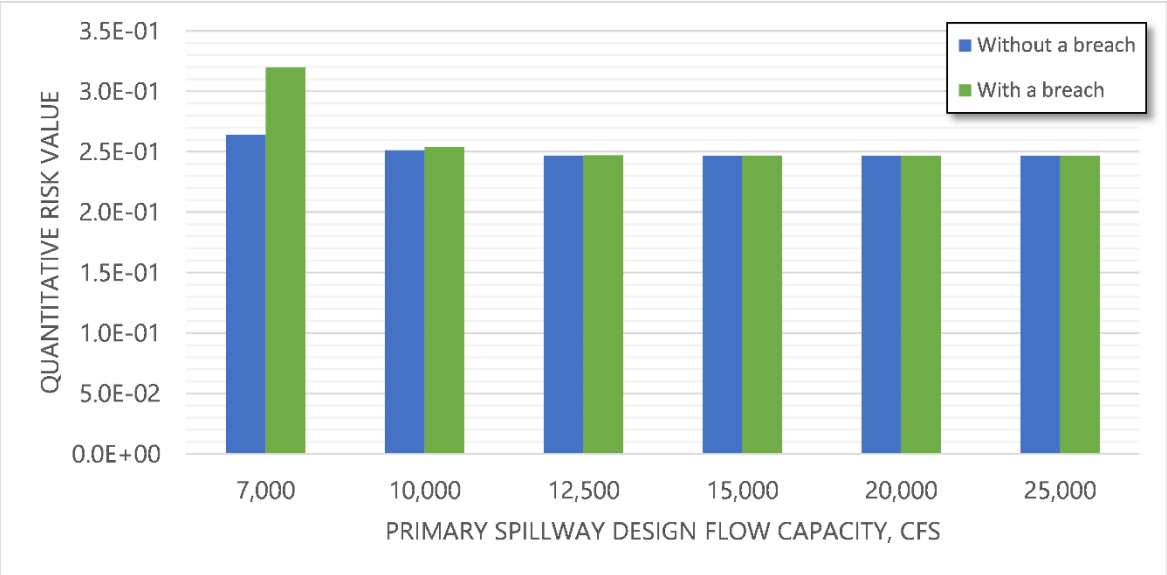


Figure 9. Risk for the range of proposed primary spillway design flows, comparing the breach and no-breach conditions, with consequence being homes inundated by 2 feet or more of water

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In parallel to this analysis was a Semi-Quantitative Risk Assessment process (SQRA) that directly compared loss of life potential to probability, and determining whether the anticipated risk to loss of life is unacceptable, broadly acceptable, or within the realm where reasonably practicable measures should be taken to reduce risk. The SQRA ultimately concluded that a proposed spillway design size of 10,000 cfs put the facility into the broadly acceptable range for failure modes from flood events. The incremental risk analysis described in this paper and the SQRA were used together to determine the appropriate design flow for the new Lake Bronson Dam primary spillway.

VI.CONCLUSIONS

Based on the analysis presented above, Barr recommended to the MnDNR to design the primary spillway to be able to pass 10,000 cfs through the spillway before the emergency spillways were activated. The incremental consequences taper off significantly at that spillway size. Our work found that most of the consequences occur during the more common events such as the 1% or 0.1% AEP events. The additional consequences that occur as events are larger do not compensate for the decrease in annual probability. In other words, for an event that is 10 times as unlikely to occur, the consequences are not 10 times greater. Therefore, the additional risk for rarer events is very small. Designing the primary spillway for events that large did not reduce the incremental risk significantly, and is therefore not warranted.

Additionally, the parallel SQRA process found that designing the primary spillway to pass 10,000 cfs before the emergency spillways are activated moves the potential failure of a breach during a flood event from a broadly unacceptable level to a broadly acceptable level. Given these two separate, parallel risk analyses that both concluded the same outcome provided confidence that this was a proper design flow for the spillway.

The method is relatively straightforward and useful for other dam facilities to base design flows on risk rather than deterministic approaches based purely on the hazard classification. This is particularly useful for high hazard dams where the deterministic approach would have required designing the Lake Bronson spillways to pass over 30,000 cfs. With the recommendation to pass 10,000 cfs through the primary spillway before the emergency spillways are activated, the combined discharge of the primary and emergency spillways is about half of the PMF at the maximum flood reservoir level (i.e., freeboard level). If designing to pass the PMF, the proposed spillway would be much larger, costing millions of dollars more to construct. In that way, the analysis presented here showed its value by properly justifying an adequate spillway size to improve dam safety.

VII.ACKNOWLEDGMENT

The authors gratefully acknowledge the contributions of Jason Boyle and his team at the Minnesota Department of Natural Resources in the development and review of this work to advance a design for the Lake Bronson Dam.

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DAM SAFETY PALM SPRINGS
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Deep Foundations Institute Guidelines for Selecting Cutoff Wall Systems

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ABSTRACT

Over the past five decades, the industry has developed innovative systems, equipment, tools, and materials that provide designers, contractors, and owners with a variety of solutions for seepage cutoff and environmental containment. Many practical, technical, operational, and logistical factors must be evaluated to select the appropriate cutoff wall and barrier system.

Deep Foundation Institute's *Guidelines for Selecting Cutoff Wall Systems* (Deep Foundations Institute, 2021) provides fundamental background information and a selection process for identifying appropriate methods for constructing vertical cutoff walls and seepage barriers to reduce water flow through embankments, dams, levees, and deep excavations.

Three broad categories of cutoff and barrier methods are presented:

- Excavated cutoff systems where in-situ embankment and/or foundation materials are excavated and fully replaced with engineered backfill material. These can be constructed using a series of individual elements (rectangular, circular, or a combination of the two) or continuous trenches.
- In situ techniques where existing materials are improved in place, either through addition of stabilizing agents and binders (deep mixing, jet grouting, and grouting), or ground freezing.
- Driven sheet pile cutoff walls.

The guidelines present relevant terminology, basic cutoff and barrier applications, a procedure and criteria for selection, details of cutoff methods (relevant evolution of method, general construction process, typical sizes, depths, widths, or shapes of walls, materials used, special considerations, and performance expectations).

The guidelines emphasize that cutoff wall designers should consider three important components when generating a design: 1) the construction risk factors (risk registry or risk profile) and concerns of the owner; 2) the material properties and nature and variability of subsurface soils and rock, groundwater flows, aquifer characteristics, and site geology; and 3) constructability, site logistic requirements, and the full cost-benefit considerations for the different systems. It is particularly recommended to involve experienced construction personnel to assist with the selection process by providing detailed construction and economic analyses of the project conditions, which in turn can influence the type of wall selected and the cutoff wall design. Additional resources and references are included.

REFERENCE

Deep Foundations Institute. (2021). "Guidelines for Selecting Cutoff Wall Systems," https://www.dfi.org/publications/single-pub?pubid=22653&bundle=1, Hawthorne, NJ, 67 p.

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ACKNOWLEDGEMENTS

Deep Foundations Institute's technical committee on Structural Slurry Walls and Seepage Control developed the Guidelines for Selecting Cutoff Wall Systems. The committee members include practicing engineers, construction contractors and specialists, material and equipment suppliers, researchers, and owners committed to advancing the state-of-practice for slurry and diaphragm walls and seepage control barriers. The Committee's webpage may be found here: https://www.dfi.org/communities/applications/structural-slurry-wall-and-seepage-control/.

The Committee dedicated this guidance document to the late Dr. Arturo Ressi de Cervia, "The Sage of Slurry Walls", for without his guidance, drive, and overall contributions, this document would not have been possible.

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DAM SAFETY PALM SPRINGS
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Development of a Definition of Plastic Concrete for Cutoff Walls

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ABSTRACT

Currently, the ICOLD Embankment Dams Committee is updating ICOLD Bulletin No. 51 – "Filling Materials for Watertight Cutoff Walls" first published in 1985. Plastic concrete is a term that has been used to describe various mixtures of cement, bentonite, aggregates, water, and various additives. Various forms of plastic concrete have been used in dam and levee construction; predominantly in foundation treatments and seepage barriers (cutoff walls). The intent of adding bentonite to the concrete mix is to increase the ductility and reduce the stiffness of the product to match the properties of the surrounding soil and/or rock. However, in addition to the changes in ductility and elasticity, the addition of bentonite has been observed to reduce the strength of mix and may also reduce the mix's resistance to internal erosion. Furthermore, while the term "plastic concrete" has been used extensively in the past to describe material used in dam and levee construction, there does not appear to be a consistent definition of what the properties of these materials are expected to be. Also, there does not appear to be a recognized standard testing procedure to measure and evaluate the properties of plastic concrete that are pertinent to dam and levee construction and performance. Based on the 1991 Kahl et. al study, the U.S. Army Corps of Engineers (USACE) developed a design procedure for designing a workable (8-inch slump) mix to produce plastic concrete targeting values of compressive strength and unconfined elastic modulus. Deformation due to hydraulic loading, changes in the seepage regime, and modifications to the embankment can result in lateral movements within the embankment. It is the goal of the ICOLD Committee updating Bulletin 51 to put forth a "definition" for plastic concrete for the geotechnical engineering profession.

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Seismic Acoustic Resonance (SAR) - A complimentary tool for mapping seepage through, beneath and around dams

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ABSTRACT

Willowstick has successfully employed a new Seismic Acoustic Resonance (SAR) method to improve seepage characterization through, beneath, and around dams. SAR is a passive seismic method that detects zones of higher permeability in the foundation and fill materials of a dam. Just as the ocean has tides, the earth's crust experiences daily "Earth Tides" and is constantly in motion. Continual motion produces resonance signals that can be measured, interpreted, and modeled to identify zones of higher permeability within a dam and its foundation. The SAR results are analyzed in conjunction with the traditional Willowstick Method to better characterize seepage flow paths and patterns trough, beneath, or around a dam in three dimensions. When both methodologies, which use entirely different scientific principles, identify the same anomalous feature, a higher degree of confidence is gained in the results of the investigation. Willowstick has effectively utilized the SAR method on many successful dam seepage investigations. The two geophysical techniques when used in parallel have yielded complimentary results, with both methods providing beneficial information about seepage locations. This paper will detail seepage flow paths identified using both Willowstick and SAR methods for two case study tailings dams in Canada and the United States.

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DAM SAFETY PALM SPRINGS
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Anderson Dam Tunnel Project - Diversion System

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ABSTRACT

The Anderson Dam Seismic Retrofit Project (ADSRP), located near Morgan Hill, California, is being undertaken by the Santa Clara Valley Water District (District) to address several important deficiencies in the existing dam and appurtenant facilities. These include predicted seismic deformations of the embankment that could lead to overtopping and failure, inadequate spillway capacity to pass the probable maximum flood, and a risk of fault rupture that could sever the existing outlet conduit beneath the dam. To bring the reservoir and its appurtenant facilities to modern standards, the retrofit project will include removal and reconstruction of most of the dam embankment, replacement and extension of the existing spillway chute, and construction of a new permanent outlet works capable of remaining in service following the design fault offset. Because of the downstream hazard, the District has also initiated an interim risk reduction project to drain the reservoir and maintain it at the dead pool elevation until the dam retrofit can be completed. The risk reduction work is being conducted separately as the Anderson Dam Tunnel Project (ADTP), which includes construction of a new diversion tunnel through the right abutment of the dam.

The ADTP includes approximately 1,400 ft of conventionally mined tunnel, extending from the downstream toe of the dam to a location upstream of the dam axis. An underground operation chamber will be constructed at that location, from which a microtunnel boring machine (MTBM) will be launched for a 300-foot drive culminating with a wet-recovery lake tap into the reservoir.

Once these features are constructed, the main tunnel will be lined with reinforced concrete and steel pipe sections, connecting to a diversion outlet control structure constructed at the downstream portal. The outlet controls will consist of two 11-foot diameter fixed cone valves with sufficient capacity to prevent reservoir level rises during all but the most severe inflow events. The new diversion tunnel constructed during the ADTP overlaps with and will later be incorporated into the permanent new outlet works that will be constructed during the ADSRP. Coordinating the design between the two projects will help reduce the overall construction duration and facilitate an earlier return to service for this important water storage reservoir.

This paper will provide an overview of the project design, including the seismic shaking and fault rupture design criteria. Key design aspects and construction sequencing considerations for the outlet system will also be presented, including the tunnel lining design for both the interim diversion (ADTP) and permanent use (ADSRP). The paper will also provide an update on the ADTP tunnel construction status, including some of the design and construction challenges of open face tunnel excavation through the sheared and weak intermediate geomaterials present at the site.

I. INTRODUCTION

The Santa Clara Valley Water District is undertaking the Anderson Dam Tunnel Project (ADTP) in response to a directive from the Federal Energy Regulatory Commission (FERC) to drain the reservoir and maintain it at the dead pool elevation in advance of the Anderson Dam Seismic Retrofit Project (ADSRP). The ADSRP will correct

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dam safety deficiencies that have been identified in the existing dam and appurtenant facilities, including seismic deformations that could lead to dam failure, inadequate spillway capacity to pass the current maximum design flood, and potential loss of the outlet works due to fault rupture on mapped traces crossing the existing outlet conduit. The ADSRP will include removal and replacement of the existing dam and spillway and construction of a new permanent outlet works. The diversion outlet system constructed during ADTP will enable the District to maintain the reservoir at the restricted level. Control of the reservoir level may not be possible with the existing outlet works in the event of damage during a large earthquake.

During the ADSRP construction, the low-level diversion outlet system constructed during ADTP will be used to keep the reservoir empty. It will be adapted into a Stage 2 Diversion System during the ADSRP and then will ultimately be incorporated into the permanent ADSRP outlet works facilities, which will include a low-level outlet works (LLOW) for normal operations and emergency evacuation, and a high-level outlet works (HLOW) to improve high-level evacuation capabilities and to help meet current reservoir drawdown criteria established by the California Division of Safety of Dams (DSOD). More details on the ADTP project elements are provided below.

II. ADTP PROJECT ELEMENTS

The main elements of the ADTP include a new low-level diversion outlet system and improvements to Coyote Creek immediately downstream to handle the diversion flow releases. Other project elements include road improvements, pipeline realignments, electrical components, and Supervisory Control and Data Acquisition (SCADA) improvements associated with the new facilities.

The general arrangement of the ADTP low-level outlet system is shown on Figure 1. From upstream to downstream, the diversion system includes the following components:

1. Diversion Intake Structure and trash rack.
2. Lake Tap consisting of minimum 8 foot-diameter steel pipe installed using a micro-tunnel boring machine (MTBM). The microtunneled conduit will serve as a diversion outlet until the ADSRP is completed.
3. MTBM Operation Chamber used for jacking/driving the MTBM. Once MTBM works is complete, the chamber will be final-lined with a 19-foot horseshoe-shaped reinforced concrete lining.
4. Low-Level Outlet Tunnel (LLOT) – 925 feet of 24-foot (excavated diameter) tunnel with a 19-foot horseshoe-shaped reinforced concrete final lining.
5. Widened Transition Section at the intersection between the LLOT and the Diversion Tunnel.
6. Diversion Tunnel – 375 foot long 19-foot (excavated diameter) horseshoe-shaped tunnel, with a 13 foot-diameter steel pipe lining backfilled with low density cellular concrete.
7. Diversion Outlet Structure (DOS) – Located at the downstream tunnel portal, this reinforced concrete energy dissipation structure will house two 11-foot fixed cone valves (FCVs), connected via a bifurcation to the concrete-encased 13 foot-diameter steel pipe coming out of the tunnel.
8. Releases from the DOS will discharge into Coyote Creek through a riprap-lined discharge channel, leading into the original Coyote Creek channel on the north and a southern channel that runs through the Live Oak picnic area. The northern channel was blocked during construction of the existing dam but will be re-opened as part of the ADTP.

The downstream leg of the LLOT and the HLOW Drop Shaft will also be constructed as part of the ADTP. Although these features are not necessary for diversion, both are included in the ADTP to avoid logistical construction challenges during the ADSRP, when the diversion system will be in operation.

The ADTP low-level diversion outlet system is temporary, intended to only operate until the reservoir is fully dewatered during the spring of the second year of ADSRP construction. It will then be reused as the part of the ADSRP Stage 1 diversion system until completion of the Stage 2 diversion system. Releases from the low-level outlet system will be controlled at the DOS so that the combined flow of the existing outlet works and the new low-level diversion outlet system is no greater than 2,500 cfs to minimize the potential for downstream impacts. The diversion outlet control comprises two electrically actuated 11-foot FCVs housed in a reinforced-concrete energy dissipation structure. Electrically actuated, 11-foot-diameter, triple offset butterfly guard valves will be located upstream of each FCV to provide for isolation and redundancy.

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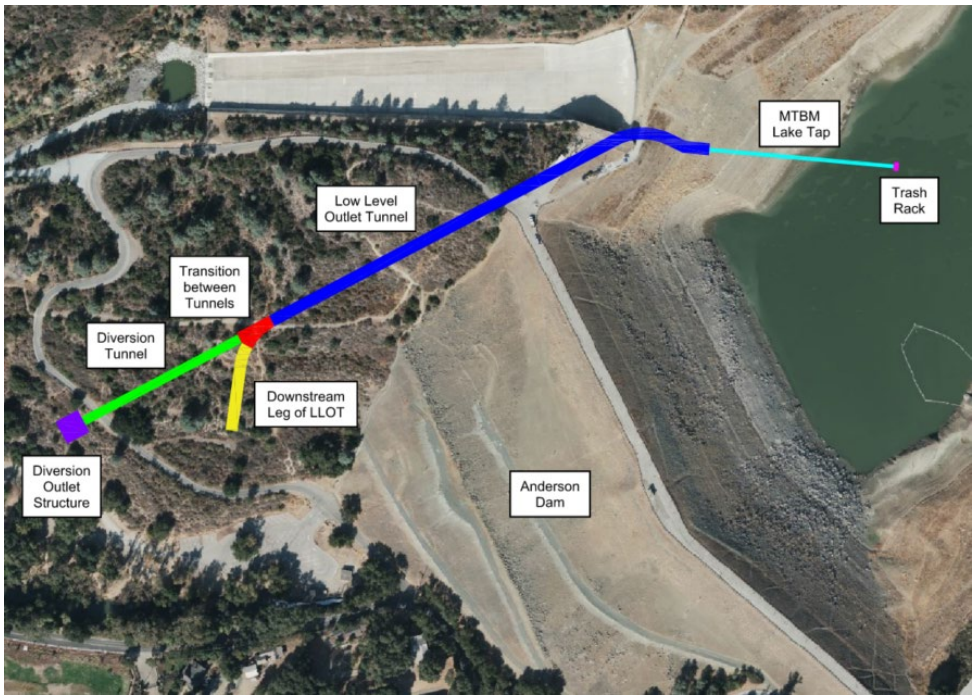


Figure 1 - ADTP General Arrangement



Figure 2 – Existing Anderson Dam and ADTP During Construction

III. DESIGN CRITERIA

The low-level outlet system was designed for the following criteria:

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Hydraulic: Capable of meeting the DSOD evacuation criterion which require lowering the top 10 percent of the design head (20 feet, from El. 627.9 to El. 607.9) in 7 days, assuming the existing outlet has been rendered inoperable. A maximum discharge rate of about 2,000 cubic feet per second (cfs) at reservoir level 627.9 is required to meet this evacuation criterion.

Operational: To maintain operability following the design earthquake with fault rupture, the permanent LLOT lining was designed to prevent collapse and loss of surrounding ground into the tunnel. The lining was also designed to maintain access into the tunnel for maintenance personnel and to prevent impact damage to the permanent outlet pipe inside the tunnel (to be installed during ADSRP). The LLOT design also included features to accommodate inspection and maintenance activities that may occur in the tunnel after completion of ADSRP.

Seismic Criteria: The LLOT final lining was designed to tolerate damage from potential fault displacement without collapse, preventing damage to the outlet pipe that could compromise its flow carrying capacity. The LLOT design dimensions included sufficient annular space between the permanent steel outlet conduit and the concrete tunnel lining to accommodate fault offset and allow access for future routine maintenance and repair.

IV. GEOLOGY

Anderson Dam is located on the southwestern flank of the Diablo Range at the eastern margin of southern Santa Clara Valley, at the western edge of the Silver Creek structural block. In the dam's vicinity, bedrock in the Diablo Range is composed of Mesozoic rocks of the Franciscan Complex (KJf). The Franciscan rocks include greenstone, basalt, chert, graywacke sandstone, serpentinite, and shale, and are highly deformed by past geologic deformation. Plio-Pleistocene Santa Clara Formation (QTsc) clastic deposits unconformably overlie the Franciscan bedrock and are in turn overlain by younger Quaternary alluvial, colluvial, fill, and landslide deposits. The two main geologic units at the site are each described in more detail below.

Santa Clara Formation

The Santa Clara Formation consists of non-marine clastic deposits Pliocene to Pleistocene to Holocene in age. The Santa Clara Formation deposits mostly consist of fine to medium grained, semi-consolidated sandstone, conglomerates, and discontinuous clay-rich strata, frequently weathered to fine to medium grained residual soils, completely altered and friable sandstone, or sandy to silty gravel. This material has almost no visible jointing. Locally, younger Quaternary to Holocene age alluvium overlies the Santa Clara Formation. These deposits are generally similar to weathered Santa Clara Formation soils. The Santa Clara Formation deposits unconformably mantle the Franciscan Complex rocks.

Franciscan Complex

The Franciscan Complex bedrock is of Jurassic to Cretaceous age. Franciscan Complex lithologies at the site include serpentinite, graywacke, greenstone (including basalt and metavolcanics), and mélange. The mélange is characterized by a pervasively sheared fabric consisting of extremely weak to weak, dark gray to black, clayey shale matrix with variably sized blocks of shale, graywacke, serpentinite, greenstone, and minor chert ranging from gravel-size to several feet. The greenstone and graywacke rocks are typically strong to occasionally very strong. The shale blocks range in strength from weak to moderately strong. The greenstone blocks range in strength from extremely weak to strong; and graywacke blocks are extremely weak to very strong. The serpentinite's strength has a wide range from weak to very weak, with medium stiff clayey zones, and also includes occasional moderately strong to extremely strong blocks.

Franciscan Complex rocks are generally sheared due to the tectonic forces that led to accretion of oceanic crust material onto the edge of the North American continental plate. At the project site, the Franciscan mélange has also undergone more recent fault-related shearing along the east-dipping reverse faults of the Coyote Creek–Range Front (CCRF) system. The internal structure of the mélange is chaotic and variable in the degree and intensity of localized shearing and the size and distribution of the harder block inclusions. Bedrock strength is correspondingly variable but is generally low in areas that display the greatest degree of shearing and/or a higher proportion of matrix material. The matrix ranges from weathered and sheared, very weak serpentinite and shale to fresh, but tectonically deformed, and intensely folded graywacke and shale.

The degree of weathering of the near surface rock and alteration (natural processes that change the mineral composition) at depth in the Franciscan Complex rock is highly variable and depends mainly on the rock type. Lithologies such as graywacke and greenstone typically have thinner weathering profiles. Shale and serpentinite are typically highly to moderately weathered and are often altered.

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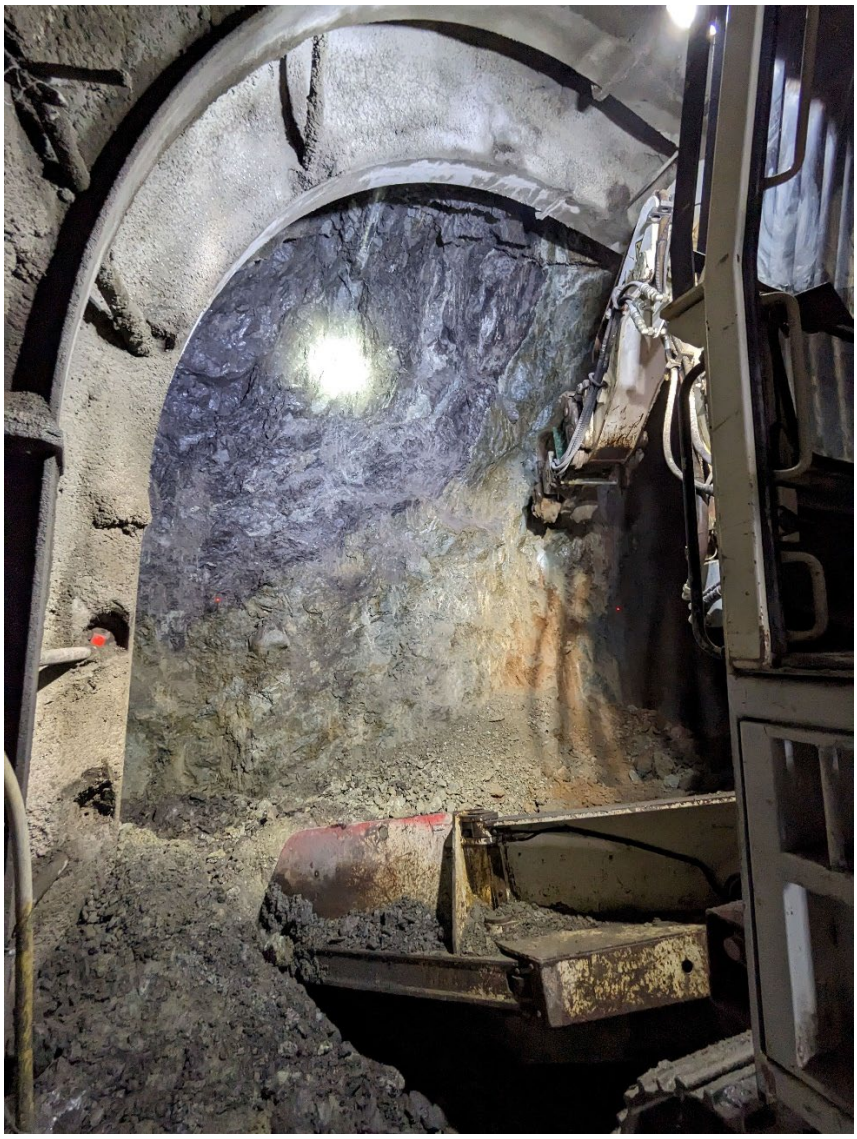


Figure 3 – Franciscan Complex Overlying Santa Clara Formation

V. DESIGN EARTHQUAKE GROUND MOTIONS AND FAULT DISPLACEMENT

The ADTP and ADSRP designs include both strong ground shaking and fault displacement scenarios. For ground shaking, two levels were used in the seismic analyses. The Maximum Credible Earthquake (MCE) was used for evaluation of permanent structures, and a construction-phase earthquake equivalent to a 100 year event was used for evaluation of temporary structures and cut slopes. Only the MCE event is discussed in this paper. The fault displacement design criteria included up to 4 feet of displacement on any of the active Coyote Creek – Range Front Fault traces crossing the outlet tunnel alignment. More details on each seismic design criteria are provided below.

Maximum Credible Earthquake

The MCE design earthquake ground motions for the ADSRP and ADTP were developed using a deterministic ground motion analysis, as normally required by the DSOD. Three sets of acceleration time histories were selected to represent the design ground motions for each of two scenario MCEs, resulting in a total of six sets of acceleration time histories for design purposes. The key parameters for the two MCEs are as follows:

Calaveras Fault

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Magnitude: Mw 7¼ strike-slip event
Distance: 2 kilometers from dam site
Peak Ground Acceleration (PGA): 0.85g

Coyote Creek – Range Front (CCRF) Faults
Magnitude: Mw 6.6 reverse-oblique event
Distance: 0 kilometers (fault traces pass through site)
Peak Ground Acceleration: 1.14g.

Using parameters developed primarily during a seismic source characterization, bedrock acceleration response spectra representative of the MCE events were estimated for several potentially controlling seismic sources. The response spectra were calculated at the 84th percentile level, in accordance with DSOD criteria (Fraser and Howard 2002), using an estimated shear wave velocity of 2,100 feet per second for the upper 100 feet of foundation bedrock beneath the dam. The geometric mean values of the spectral ordinates calculated using four of the five Next Generation of Attenuation (NGA) relationships were used to develop the best-estimated spectra. Near-fault effects were accounted for using the relationships developed by Spudich and Chiou (2008), Somerville et al. (1997), and Abrahamson (2000).

Based on a comparison of the response spectra calculated for MCE events along the Coyote Creek, Calaveras, and San Andreas Faults, and with consideration of the differences in MCE magnitude between the Calaveras and San Andreas Faults, previous investigators concluded that MCE events along the Coyote Creek and Calaveras Faults should be considered for the development of design ground motions. A summary of the key parameters for both scenario events is presented in below in Table 1. Target spectra were developed for the fault normal (FN) and fault parallel (FP) components for both scenario events.

Table 1 - Summary of Controlling Seismic Events

Fault	Fault Type	MCE Magnitude	Rupture Distance, Rrup (km)	Average 84th-Percentile PGA (g)	84th Percentile Arias Intensity (m/s)
Coyote Creek	Reverse	6.6	0	1.14	7.24
Calaveras	Strike-slip	7.25	2	0.85	8.45

Using these parameters, three sets of recorded ground motions for the two controlling seismic events were selected from the Pacific Earthquake Engineering Research Center (PEER)-NGA database. Each set contained the two horizontal components of the recorded motion. The selected motions were rotated into FN and FP orientations.

These seed motions were then scaled linearly so that their PGAs matched the PGAs of the corresponding target spectrum. The scaled motions were then modified using the time-domain spectral matching procedure proposed by Lilhanand and Tseng (1988) and modified by Abrahamson (1992) to match the corresponding target spectrum (i.e., FN and FP seed motions were modified to match the FN and FP target spectra for each scenario) within a tolerance of about 3 to 5 percent. The resulting time histories were then used in the design seismic analyses for both the ADTP and ADSRP project components.

Fault Displacement

Based on prior studies and analysis of the fault zones by others, the design criteria include 4 feet of discrete reverse-fault offset, occurring anywhere along the LLOT alignment where faulting is not ruled out by geologic observations and mapping in the tunnel. The reverse fault displacement is characterized as southwest-vergent reverse-oblique dextral slip with a rake of up to 45 degrees on northeast-dipping fault strands. As a separate design criterion, 2 feet of strike-slip displacement was also considered due to the potential for lateral movement from surface fault rupture.

Full design fault displacement could be distributed across the entire CCRF fault system or could be limited to a narrower zone of discrete shearing within one or more of the more broadly defined fault zones. The width of the fault zones likely to accommodate displacement varies from narrow (5-10 feet) to wide (greater than 30 feet). Displacement on the Northeast and Central strands of the CCFZ (CCFZ-NE and CCFZ-C) could be distributed over 30- to greater than 50-foot-wide zones, likely concentrated along discrete fault splays and more prominent shears within these zones. Some strands of the CCFZ-C have moderate widths of 15 to 25 feet over which fault displacement is likely. The Range Front Fault strands (RFFs) are more likely to have discrete displacement distributed on fewer shears within a narrower zone of 5-10 feet.

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Based on the observed fault conditions and the high degree of fracturing and sheared conditions of the rock adjacent to the fault zones, three feet is considered a lower limit for the width of a fault zone that could experience maximum displacement of the CCRF fault system, and it is judged to be more likely that the maximum displacement would be distributed across a minimum 5- to 10-foot zone in cases where the maximum displacement occurs along a single splay of the CCRF fault system. The observed fault zone and adjacent rock conditions are judged to be inconsistent with so called 'knife-edge' displacement in which the maximum amount of displacement occurs along a single plane, and so this scenario was not considered in the design analysis.

VI. OUTLET STRUCTURE

The Diversion Outlet Structure (DOS) is located at the downstream end of the low-level outlet works and houses twin parallel 132-inch diameter fixed cone valves, each protected by a 132-inch diameter butterfly valve, each discharging into its own chamber of the energy dissipation structure. Following completion of the ADTP, the DOS will also be used as part of the Stage 2 diversion system during ADSRP construction. After completion of the ADSRP, the DOS will become the permanent outlet structure for the HLOW system.

The DOS is designed to withstand strong ground motions caused by the design MCE events, and is also designed considering the potential for fault rupture that could occur along the Southwest Strand of the RFF that may be located in the vicinity. The design intent of the DOS is to survive potential offset of the RFF without collapse. As described above, potential fault offset includes up to 4 feet of reverse thrust offset, or 2 feet of strike-slip offset.

The DOS includes a 6- to 8-foot thick reinforced concrete foundation slab, designed to bridge displacements due to fault offset, reduce relative movement between the structure, piping and valves, and to maintain the ability to release approximately 375 cfs during an emergency evacuation of the reservoir. The design objective is reduce the damage that could occur to the DOS structure in the event of fault offset. The design also includes a relatively thick layer of Controlled Low-Strength Material (CLSM) backfill between the Santa Clara Formation and the reinforced concrete structure above. To provide additional stability and resistance to seismic loading, the reinforced concrete DOS structure is also tied into the foundation using numerous rock anchors.



Figure 4 - Diversion Outlet Structure Under Construction

VII. DIVERSION PORTAL SLOPES

An approximately 300-foot-long by 270-foot-wide area on the northern bank of the Coyote Creek was excavated to develop a foundation for the new DOS and the downstream portal of the Diversion Tunnel. The Diversion Tunnel Portal served as the main point of access and egress during the construction of the tunnel. The proposed toe of the portal is at approximately El. 415 and the highest point of the cut slope is El. 537. This portal is a permanent earth structure and the cut will not be backfilled. Because of the very low strength of the Santa Clara Formation materials,

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it was necessary for long term stability to flatten the slope angle and to reinforce the slope by installation of a pattern of deep soil nail anchors and a reinforced shotcrete facing.

VIII. TUNNEL DESIGN

For the ADTP, the tunnel starts at the Diversion Portal, and proceeds upstream in the 18.5-foot Diversion Tunnel to meet the HLOW Drop Shaft. Upstream from the shaft, the tunnel extends through the Transition Zone and then continues as the 24-foot LLOT to the MTBM Operation Chamber, sized to accommodate the MTBM subcontractor's requirements. From there, the MTBM microtunnel (minimum 8 foot diameter; final size to be determined by the contractor) extends to the wet recovery lake tap in the reservoir.

The ADTP low-level outlet system design includes a 13-foot internal diameter (ID) steel-lined Diversion Tunnel, a 19-foot reinforced concrete lined portion of the LLOT, a 19-foot reinforced concrete lined MTBM Operation Chamber, and the steel pipe installed by microtunneling.

Excavation Methods and Initial Support

The ADTP tunnels were expected to encounter variable ground conditions, ranging from strong, massive rock to very weak and intensely fractured sheared rock, including fault gouge. The tunnels ground conditions were expected to include raveling, running, caving, and squeezing ground. Based on the geotechnical site explorations, conventional tunneling methods, including roadheaders, drill-and-blast techniques, and hydraulic excavators, were expected to be adaptable to the anticipated variable ground conditions. The use of these types of conventional tunneling methods was anticipated, along with the use of dual drifts and/or other sequential methods for mining the tunnel using partial face openings, for more effective ground control in areas with poor ground conditions and short standup times.

Initial Support measures for the Diversion Tunnel and LLOT were designed to withstand ground loads associated with the variable ground conditions. The design primarily included horseshoe steel sets at several different spacings with steel fiber reinforced shotcrete in different thicknesses. Due to the anticipated variability in ground conditions, the ground loads and conditions expected along the tunnel alignment were divided into five ground classes based on the rock mass rating (RMR) system in accordance with Bieniawski (1989). This system of rock mass classification is based on the physical characteristics of the ground and its anticipated behavior during the tunnel excavation. The ground assigned to a particular class is expected to perform similarly in the tunnel excavation, and to require similar support methods. Each ground classification was referred to as a "Ground Condition Class" (GCC) because some of the rock encountered during the subsurface investigations had soil-like characteristics.

A total of five GCC's were identified, but the subsurface materials were classified into three Excavation and Support (E&S) classes, corresponding to the most prevalent GCCs expected based on the subsurface investigations data:

- GCC I = E&S III - Massive to Slightly Fractured Rock
- GCC II = E&S III - Moderately Blocky Rock
- GCC III = E&S III - Moderately to Highly Fractured Rock
- GCC IV = E&S IV - Highly Fractured Rock, Rock Degraded to Soil-like Material
- GCC V = E&S V - Heavily Sheared/Faulted Rock with Clay Gouge/Infilling Materials

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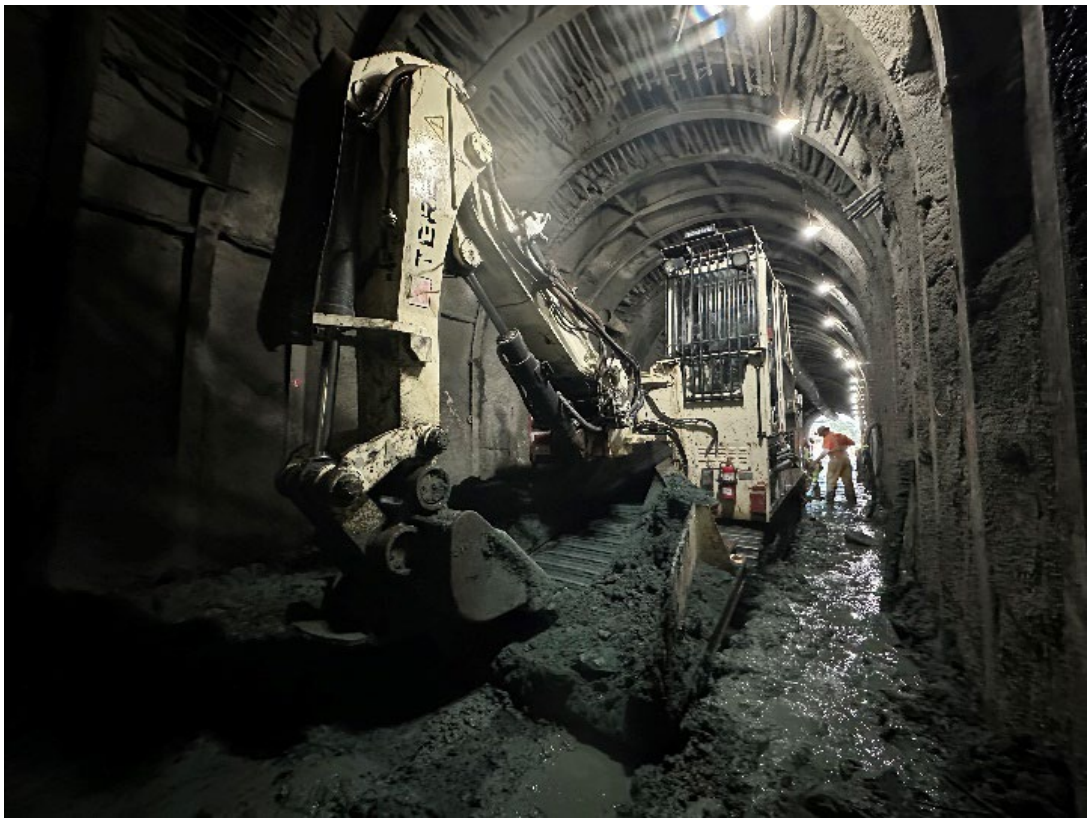


Figure 4 – Tunnel Excavation and Initial Support in the Diversion Tunnel

Final Lining – Non-Offset Zones

The final lining for the LLOT consists of reinforced concrete and provides a structural support system capable of withstanding the anticipated ground loads, internal water pressures (during diversion), and external ground and groundwater loads for the design life of the project. The analysis for the final lining was performed using a two-dimensional, plane-strain computer program FLAC (Itasca, 2018a). The calculated lining forces and deformations of the tunnel lining system were then compared to the load carrying capacity of the tunnel lining system. The permanent hydrostatic load was conservatively assumed to be equal to a reservoir elevation of 627.9 (the spillway crest elevation). The tunnel lining design for non-offset zones considered inertial loads from dynamic shaking but did not consider fault offset.

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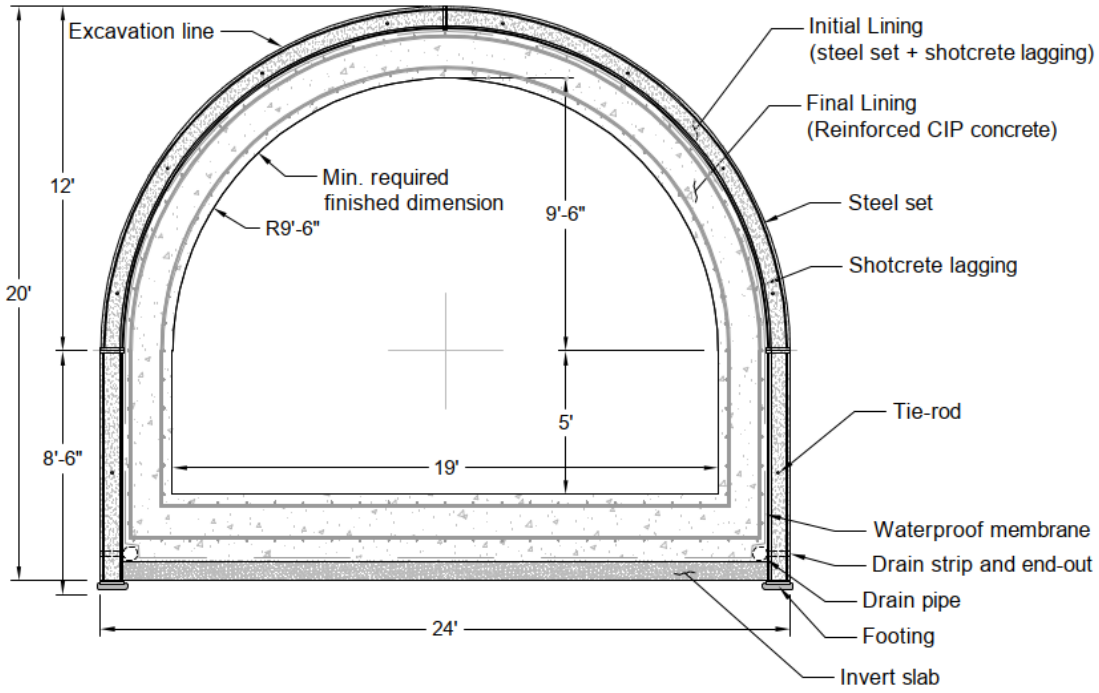


Figure 5 – Initial and Final Lining for the LLOT

Final Lining – Seismic Offset Zones

For the LLOT, a segmental tunnel lining concept was chosen based on the size of the tunnel and the anticipated magnitude of offset. The segmental tunnel lining concept allows the tunnel lining geometry and initial size of the excavation to remain the same for the fault zones and non-fault zones. This is important because it may not be possible to identify the full extent of each fault zone during tunnel excavation and changing the excavated size within a fault zone would be difficult to accommodate.

As a part of the future ADSRP construction, a permanent 78-inch outlet pipe will be installed inside the LLOT. The final dimensions of the LLOT are designed to provide sufficient clearance for both dynamic shaking and fault offset effects on the 78-inch pipe during the maximum credible earthquake (MCE) without the pipe impacting the tunnel walls. A dynamic shaking analysis and a fault offset analysis were performed to estimate the movement of the pipe inside the tunnel. As a result of these analyses, finished internal dimensions of the LLOT were selected. The selected 19-foot-wide by 14.5-foot-high horseshoe cross section will provide sufficient clearance for ventilation, lighting, other electrical and mechanical features, and will accommodate inspection and maintenance activities.

The adopted fault crossing design concept is shown in Figure 7. The final lining will consist of a series of short cast-in-place concrete segments (articulated segments) that form structurally discontinuous rings along the tunnel alignment through each fault zone. The articulated joints will separate segments of the final lining at each fault crossing. The segments will be constructed through the interpreted fault zone width and 20 feet (approximately one tunnel diameter) beyond the limits of each fault zone, based on interpretation of geologic mapping within the tunnel during excavation. The segments will be either 4 feet or 9 feet long, depending on the interpreted width of the fault zone, and will be each separated by 1-ft wide joint openings filled with crushable material.

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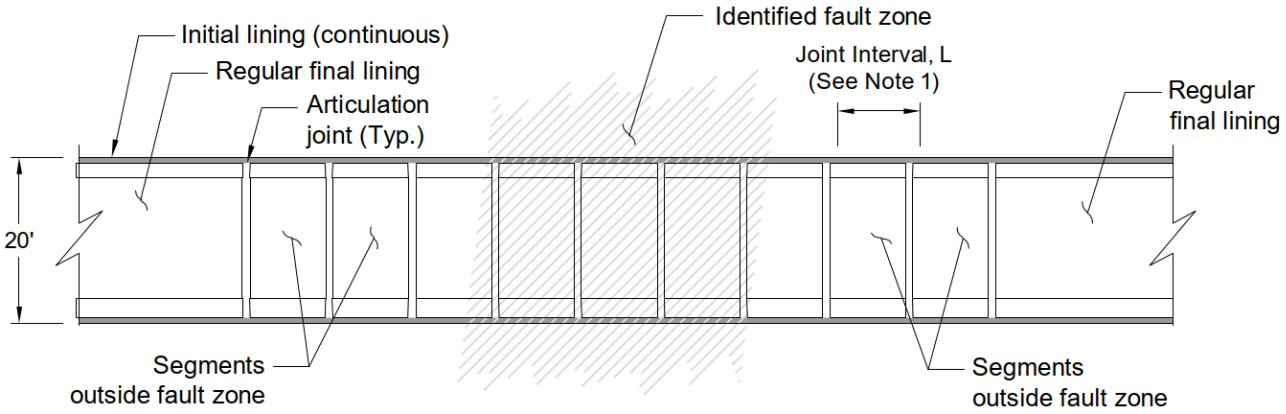


Figure 6 - Fault Crossing Concept

Fault rupture will result in longitudinal tunnel lining displacement (compression or extension of the tunnel lining) in addition to lateral displacement. To accommodate the displacements, articulation joint with a minimum 10-inch-wide inter-segment gap will be installed separating concrete segments. The gap will be permanently filled with crushable lightweight cellular concrete with 200 psi compressive strength. During the diversion period, when the LLOT will contain flowing water, the gaps will be temporarily filled with 2,500 psi concrete. The temporary gap filling concrete will be pigmented for better visibility during demolition, and it will be replaced by the crushable lightweight cellular concrete at all intersegment gaps prior to completion of ADSRP construction.

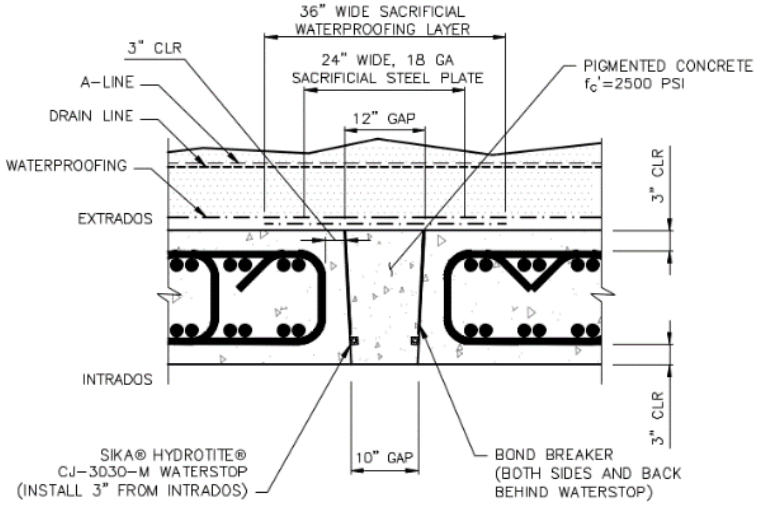


Figure 7 - Articulation Joint Details (for service during Diversion Period)

Numerical analyses were conducted using FLAC3D (Itasca, 2018) to evaluate the effects of potential fault offset on the articulated segments. The additional lining forces induced by the fault displacements were calculated and compared with the load carrying capacity of the tunnel lining. Models with different dips and dip directions were created considering the anticipated variability of fault orientation along the tunnel alignment. The analysis confirmed the viability of the articulated segment concept to survive fault offset. The results showed that segment lengths of 5 feet and 10 feet (depending on the fault zone width and angle of intersection) would distribute the displacements gradually across several segments, while meeting the allowable demand to capacity ratios. These allowable ratios were established for the corresponding damage control performance objective for the design event. The articulated segment system design is therefore judged capable of withstanding the loads and deformations due to the design fault offset without tunnel collapse. More details on the segmental lining design and analysis are provided in Kim, et al, 2022.

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IX. CONSTRUCTION UPDATE

The project has been in construction for two years and several elements have been completed or are on-going in parallel. The Diversion Portal has been substantially completed and the DOS foundation has been excavated and backfilled with CLSM. The DOS construction is in progress with concrete and rebar being placed sequentially due its large size. The Diversion Tunnel and HLOW Drop Shaft has been excavated and the Contractor continues to make progress through the transition area and the LLOT. The Lake Tap Portal Excavation silt curtain has been installed, marine equipment has been mobilized, and earthwork has begun.

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Time-history Analysis of Pyramid Dam Outlet Works Intake Structure as a Basis for the Future Risk Assessments

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In this paper, the seismic analysis of the Pyramid Dam Outlet Works Intake Structure (OWIS) is presented. The purpose of the analysis is to evaluate various Potential Failure Modes (PFMs) for the structure as well as to estimate the threshold earthquake beyond which the structure exhibits a nonlinear behavior. Pyramid Dam is part of California's State Water Project which is operated and maintained by the Department of Water Resources. Pyramid OWIS is a fully submerged reinforced concrete tower located near the right abutment of the Pyramid Dam in Los Angeles County, California. It is 119-feet tall and has an internal diameter of 15-feet. The intake structure is a critical appurtenant structure for the management of the reservoir levels during flood events and for reservoir drawdown in the event of damage to the dam due to a large seismic event. Given that the current codes and guidelines have more stringent requirements related to the ductility of intake tower structures, most of the 1960's vintage intake towers like that of Pyramid Dam are lightly reinforced, i.e., they fail in a non-ductile manner. Further, considering the significant increase in the seismic forces compared to those used for the original design of the old intake towers, these structures do not meet the performance requirements of the current codes and guidelines for the seismic loading. Per the recent Level 2 Risk Assessment (L2RA) for Pyramid Dam, failure of the submerged OWIS due to an earthquake was estimated to be a "damage state", and not lead to direct loss of life. In this work, the seismic performance of the Pyramid OWIS is checked against the requirements of US Army Corps of Engineers guidelines and American Concrete Institute code, the results of which will be used by DWR for their future risk assessments and potential dam safety risk reduction measures for this facility.

I.INTRODUCTION

The intake structure is a fully submerged reinforced concrete tower located near the right abutment of the Pyramid Dam. The intake structure is a critical appurtenant structure for the drawdown of the reservoir. It is 119-feet tall and has an internal diameter of 15-feet. A 30-foot radius elbow at the base of the intake structure connects to a 40-foot-long cut-and-cover conduit connection to the outlet tunnel. The elbow and the conduit cover are also 15-feet in diameter. A steel trashrack cover is located on the top of the structure, and vertical reinforced concrete columns support vertical steel trashrack panels around the sides of the structure. The reservoir normal water surface is at El. 2578 ft, which is 220 ft. above the roof of the intake structure. Per Federal Energy Regulation Commission (FERC) guidelines, Pyramid Dam is classified as a "High Hazard Potential" structure per Federal Energy Regulatory Committee (FERC). The profile view of the intake structure is presented in Figure 1.

II.ANALYSIS APPROACH

We developed a 3D finite element (FE) model based on the as-built drawings and other available data provided by DWR to perform the linear elastic time-history analyses of the Pyramid Dam intake structure. The analyses

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included five sets of spectrally matched acceleration time-histories provided by DWR. The spectrum compatible time histories were developed based on results of the site-specific probabilistic and deterministic seismic hazard analysis (PSHA and DSHA). The analyses were performed in general compliance with the guidelines provided in the United States Army Corps of Engineers (USACE) Engineering Manuals (EMs) (USACE, 2003a, USACE, 2016, USACE, 2005, USACE, 2003b, USACE, 2007), American Concrete Institute (ACI) code (ACI, 2014), and Goyal and Chopra's report (Goyal, 1989).

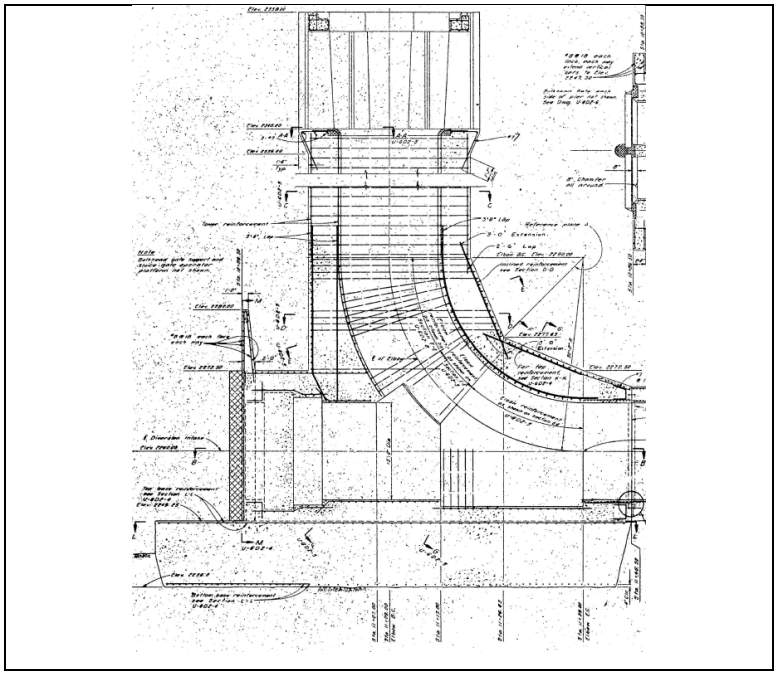
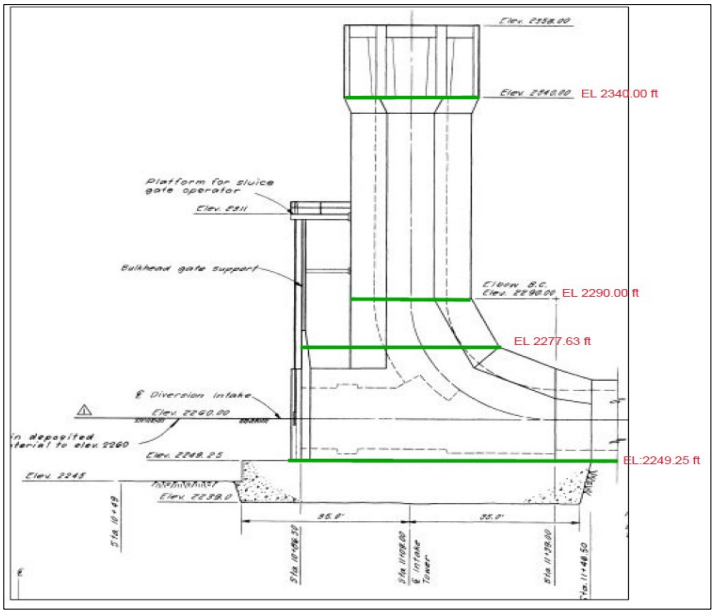


Figure 1. Plan and Profile Views

III.ACCEPTANCE CRITERIA

Several potential failure modes (PFMs) were investigated for the structure, which are described herein (USACE, 2003a). The demand forces were calculated at the critical locations along the height of the structure, which are shown in Figure 2.



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Figure 2. Critical Sections for the Analysis

A. *Moment Failure (Flexure)*

The structure is deemed adequate for flexural forces if the following requirements are met if:

- $$\frac{1.1M_u}{2.0} \leq \phi M_n \qquad (1)$$

  Where, M_u is the demand flexural moment obtained from the FE model, ϕ is the strength reduction factor per (ACI, 2014), and M_n is the nominal flexural capacity of the reinforced concrete section.

- Cumulative duration of bending-Moment excursions above DCR ratios of 1 to 2 falls below the acceptance curve provided in Figure 3a.
- The extent of yielding along the height of tower (i.e., plastic hinge length for DCR ratios of 1 to 2) is limited and falls below the acceptance curve provided in Figure 3b.

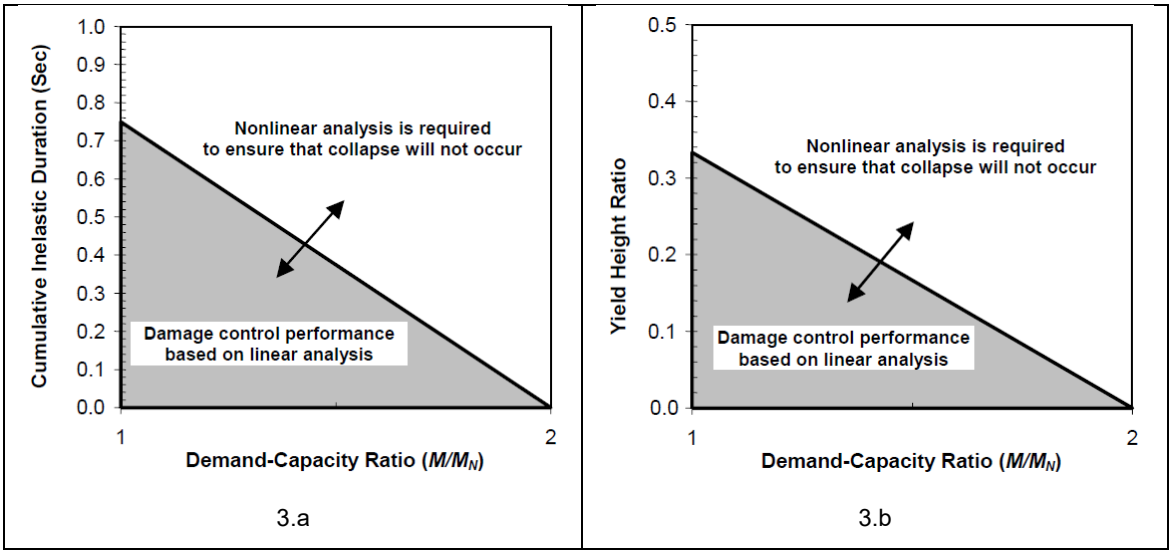


Figure 3. Acceptance Criteria for Freestanding Intake Towers (USACE2007)

The approach provided in USACE's document recommends changing the damping ratio from 5% to 10% depending on the calculated DCR values. Increasing the damping ratio in the linear model is interpreted as occurrence of damage within the structure, the extent of which could not be estimated in a linear model. If the damage in the structure is local (local cracking of concrete due to tensile stresses), increasing the damping ratio from 5% to 10% would reduce the energy that is absorbed by the entire structure, and consequently would cause the demand stresses to be underestimated in the linear time-history analysis. Thus, we kept the damping ratio to be 5% during the linear time-history analysis of the intake structure.

B. *Shear Failure (Diagonal Tension)*

Shear failure of the intake tower is a brittle failure mode, which will cause the abrupt collapse of the intake structure if the demand shear forces exceed the shear capacity of the intake structure. The shear capacity of the intake structure at each section should be greater than lesser of:

a - V_u (demand shear force at the concrete cross-section, obtained from linear TH analysis (2)

b - $\frac{1.5V_u}{\frac{M_u}{M_n}}$ (3)

The shear capacity of the reinforced concrete is calculated using the following equations:

$$\emptyset V_n = 0.85(V_c + V_{st}) \qquad (4)$$

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With:

$$V_c = 2\left[0.5 + \frac{P}{2000A_g}\right]\sqrt{(0.15 * f_C')} * (0.8 * A_g) \quad (5)$$

V_{st}, for a circular cross section:

$$V_{st} = \frac{\pi A_h f_y (0.8 * d)}{2s} \quad (6)$$

Where:

f_C' = Compressive strength of concrete

A_h= Horizontal reinforcement cross-section area

d = Outside diameter of the tower

f_y= Stress in the transverse reinforcing due to unfactored hydrostatic loading and other non-earthquake loads

s = Spacing of reinforcement

V_{st}, for a rectangular cross section:

$$V_{st} = \frac{A_h f_y (0.8 * d)}{s} \quad (7)$$

Where:

d = Dimension in the direction of the seismic shear force

s = Spacing of shear reinforcement

C. Sliding Shear Failure (Shear Friction)

Sliding shear failure is also categorized as a brittle failure mode. The nominal shear strength will be determined by the following equation:

$$V_{sl} = \mu_{SF}(Pu + 0.25 f_y A_{vf}) \quad (8)$$

Where:

μ_{SF} = Sliding shear coefficient of friction, per Table 22.9.4.2 in ACI (ACI, 2014)

Pu = Axial load on section

f_y= Yield strength of reinforcement

A_{vf} = Area of shear friction reinforcement (area of vertical reinforcement crossing the horizontal failure plane)

D. Fracture of Tensile Reinforcement (Minimum Tensile Reinforcement Requirements)

The following requirement should be met to avoid the fracture of tensile reinforcement:

$$M_n \geq 1.20 M_{cr} \quad (9)$$

If the above requirement is not met, the intake tower is categorized as a lightly-reinforced structure; and will have limited-ductile behavior based on (USACE, 2007).

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IV.MATERIAL PROPERTIES

Table 1 summarizes the material properties for concrete, rock foundation, and the reinforcing steel.

TABLE 1 - MATERIAL PROPERTIES

Properties	Values
Concrete	
28-day Comp. Strength (psi)	3,000
Density (pcf)	150
Dynamic Modulus of Elasticity (psi)	3,100,000
Poisson's Ratio	0.15
Tensile Strength (psi)	350
Rock	
Density (pcf)	165
Rock Mass Modulus of Deformation (psi)	1,500,000
Dynamic Shear Modulus (psi)	2,280,000
Static Elastic Modulus (psi)	4,400,000[(2)]
Static Poisson's ratio	0.33
Dynamic Poisson's ratio	0.24
Reinforcing Steel	
Yield Strength, f_y (psi)	40,000
Ultimate Strength, F_u (psi)	70,000
Elastic Modulus, E (psi)	29,000,000
Poisson Ratio, υ	0.3

V.ANALYSIS LOADS

The following loads were considered for the dynamic analysis of the intake structure:

- Gravity Load
- Hydrostatic and Uplift Load
- Silt Load
- Backfill Load
- Hydrodynamic Load, which was calculated using the Goyal and Chopra's method (UASCE, 2003a).
- Seismic Loads: Figure 4 shows the 5% damped 5000-year return period target spectra for the project.

Five sets of spectrally matched ground motions were developed for the time-history analyses (RSN5656, RSN983, RSN1004, RSN1549, RSN6893). Each set of time histories consisted of three components (two horizontal and one vertical) and were used as input for the 3D time-history analyses of the Pyramid Dam intake structure. Figure 5 shows the acceleration time-histories for two representative (and governing) ground motions that were used in this paper.

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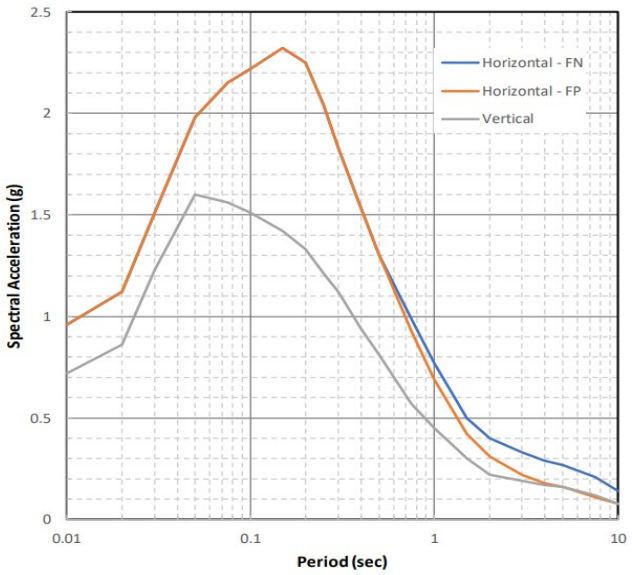


Figure 4. Target Spectra for Hard Rock Level at Pyramid Dam

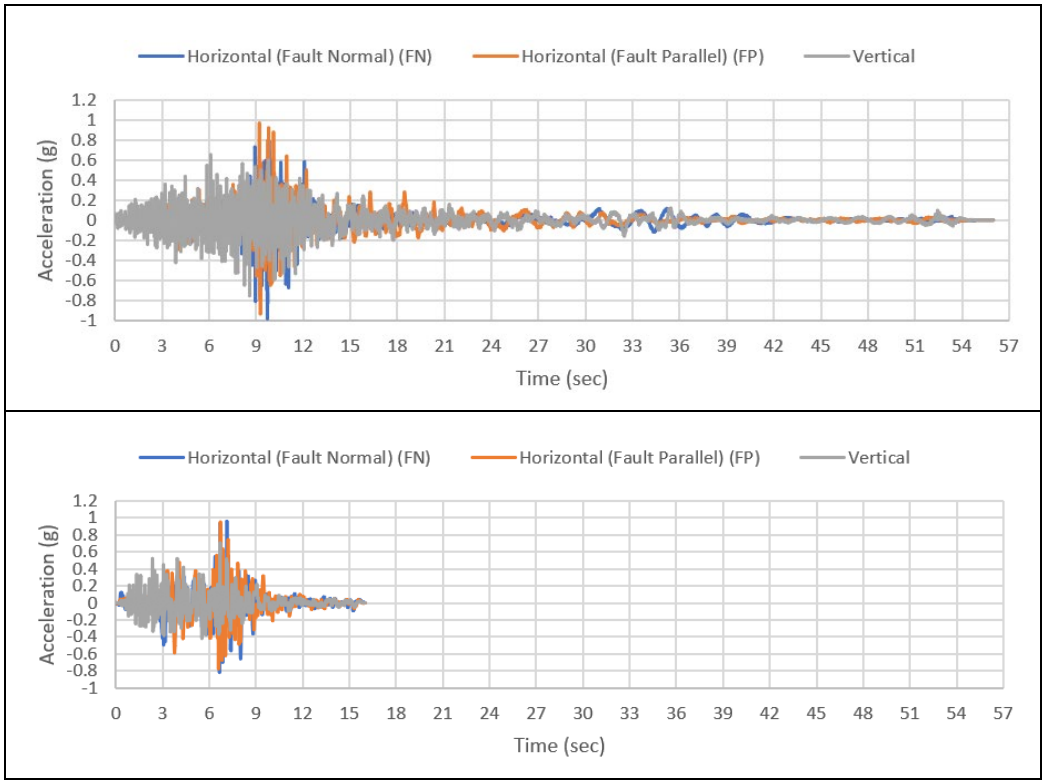


Figure 5. Acceleration Time-Histories - Top: RSN5656, Bottom: RSN983

VI.FINITE ELEMENT MODEL

The 3D linear time-history finite element analyses of the intake structure were performed using the computer program ANSYS WORKBENCH Version 2019 R1 (ANSYS, 2019). The 3D geometry was first developed in INVENTOR (AutoDesk) 3D modeling software for additional geometry processing and then imported into ANSYS, which was subsequently used as the basis for creating the FEM mesh of the intake structure.

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Distinct components of the intake structure (intake tower, elbow, tunnel, etc.) were imported as separate bodies, meshed separately, and fully bonded together by using contact elements defined in ANSYS. The material properties for the concrete and rock were assumed to remain in the linear regime throughout all the analyses. The intake structure foundation (rock) is 300 feet in the upstream-downstream and cross-canyon directions; and 140 feet below the base of the structure at El. 2249.25 ft. The circular section of the intake structure consisted of minimum of 6 elements across the thickness. The trashrack supports (columns) were modeled using finer mesh arrangements due to their complex geometries. The FE model consisted of 127,732 solid elements (SOLID-185) and 137,732 nodes. A total of 24,206 mass elements (MASS-21) were included to apply the hydrodynamic forces (inside and outside the intake structure), the backfill and silt dynamic loads. A total of 10,608 CONTACT-174 elements were used to model the contacts at interfaces among different bodies in the FE model. The model geometry is shown in Figure 6.

Rayleigh damping was used in the linear time-history analysis of the intake structure. The Rayleigh damping matrix consists of mass-proportional and stiffness-proportional parts, which was determined following the work of Professor John F. Hall (Hall, 2005).

The rock foundation was considered “massless” during the linear time-history analysis in ANSYS. The massless foundation serves only as an elastic support for the structure and neglects the radiation damping in the rock, thus, the demand stresses within the structure, obtained from the analysis are expected to be relatively higher than those obtained from an analysis where the foundation mass is included in the model.

We used the acceleration time-histories to calculate the velocity time-histories, which were subsequently applied to the rock boundaries of the FE model. Two horizontal velocity time histories were applied, one in the upstream-downstream direction and the other in the cross-canyon direction, while the third velocity time history was applied in the vertical direction. The “direct integration” method was used for solving the dynamic equations. The solution outputs were used to calculate the demand moment, axial and shear forces at the critical sections of the intake structure to compare them with the capacity of the structure.

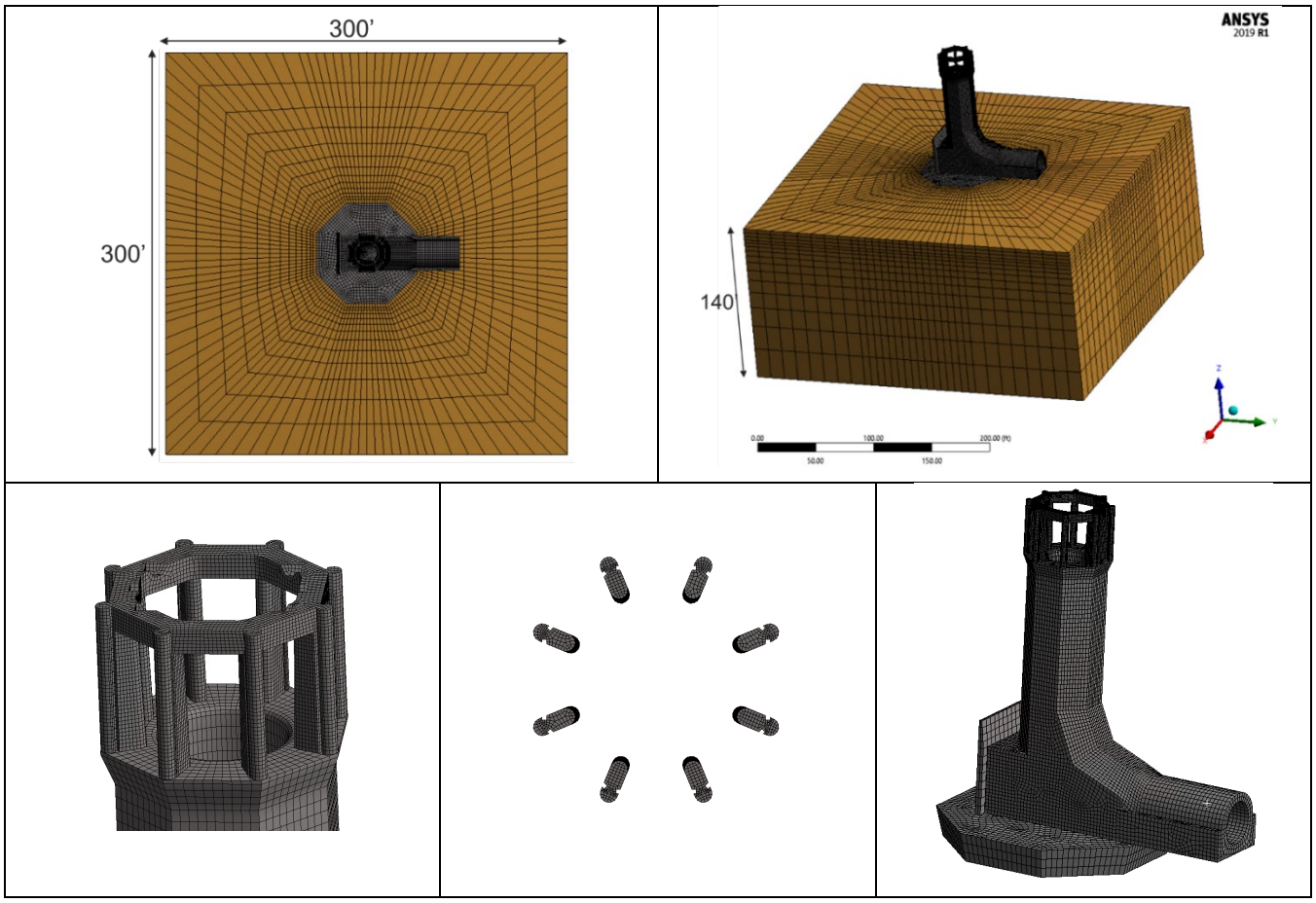


Figure 6. Finite Element Model

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VII.ANALYSIS RESULTS

Selected results from two time-history analyses (RSN5656 and RSN983) are presented in this section.

A. Modal Analysis

Modal analysis was conducted to estimate the modal frequencies of the structure to be used to estimate the Reighley damping coefficients for the models. Table 2 summarizes the significant modes of the structure.

TABLE 2 – SIGNIFICANT MODES OF THE STRUCTURE

Mode No.	Period (sec)	Cumulative Mass participation-X (Cross-Canyon)	Cumulative Mass participation-Y (Canyon)	Cumulative Mass participation-Z (Vertical)
1	0.337	37.2%	7.3e-7%	1.04e-9%
2	0.279	37.2%	23.9%	0.012%
6	0.130	59.6%	29.2%	0.012%
9	0.086	41.7%	41.8%	0.036%
14	0.065	99.9%	89.5%	0.046%

Figure 7 illustrates the comparisons between the response spectra for ground motion RSN5656, and the period range of the structure. The graph indicates that the intake structure will experience horizontal spectral accelerations ranging from 1.7g to 2.4g.

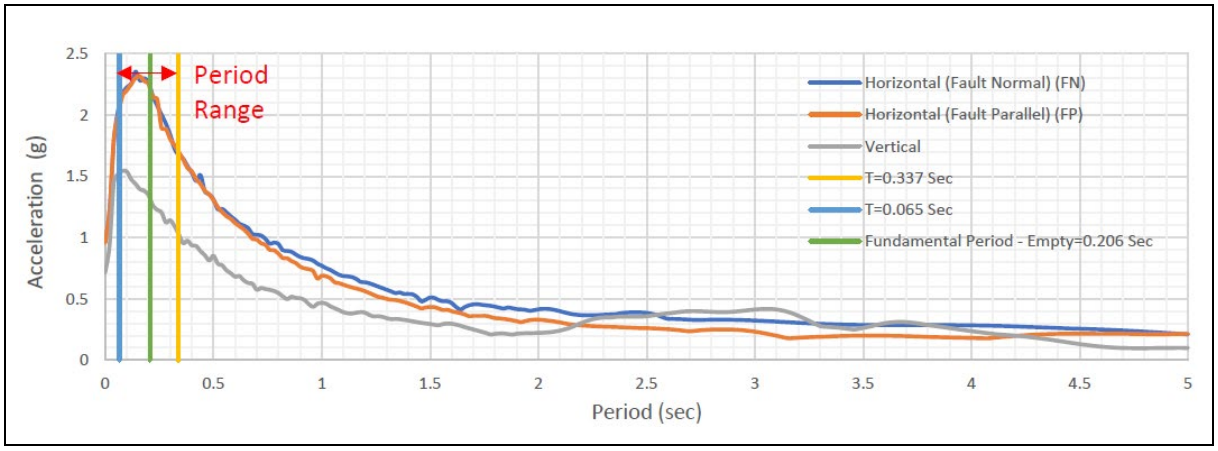


Figure 7. Response Spectrum for RSN5656 Vs. Period Range of the Structure

B. Time-History Analysis Results

The evaluations pertaining to all potential failure modes are presented in the following subsections. It is noted that sample results are provided in this paper.

1) Moment Failure

The summary of the flexural strength checks at El. 2290 (above the elbow) are provided in Table 3.

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TABLE 3 - DEMAND MOMENT VS. CAPACITY-SECTIONS AT EL. 2290 FT ABOVE THE ELBOW

Ground Motion	Critical Section (EL.)	Time (sec)	Demand Moment M_u* (Kip-ft.)	$1.1M_u$ (Kip-ft.)	Pu** (Kips)	$0.1f'_cA_g$ (Kips)	ϕM_n*** (Kip-ft.)	DCR $1.1M_u/\phi M_n$<2
RSN5656	2290	12.13	645,124	709,637	123	-14,782	54,174	**13.10**
RSN983	2290	7.97	649,995	714,994	-539	-14,782	54,174	**13.19**

* Demand moment is calculated as $M_u = \sqrt{M_{ux}^2 + M_{uy}^2}$.
** Negative sign indicates compression.
*** M_n has been calculated using SAP2000. Per (USACE, 2003a), $\phi = 0.9$ if $P < 0.1f'_cA_g$.

Considering RSN5656, the demand moment time-history is plotted in Figure 8 along with the capacity of the section at Elevations 2290 ft. The plot indicates that there are several excursions above the allowable bending moment (ϕM_n) and $(2\phi M_n)$. The curves for the "cumulative duration of bending-moment excursions above DCR ratios of 1 to 2" (performance curves) for RSN5656 are provided in Figure 9. The curves are plotted at several elevations along the height of the tower (between Elevations 2277.63 ft. and 2336 ft.). The results show that all the curves along the entire height of the tower from Elevations 2277.63 ft. to 2336 ft. fall above the inelastic limit line. The "yield-height ratio" versus the "DCR ratios of "1 to 2" curve for RSN5656 is plotted in Figure 10. The plot indicates that the entire height of the tower will yield.

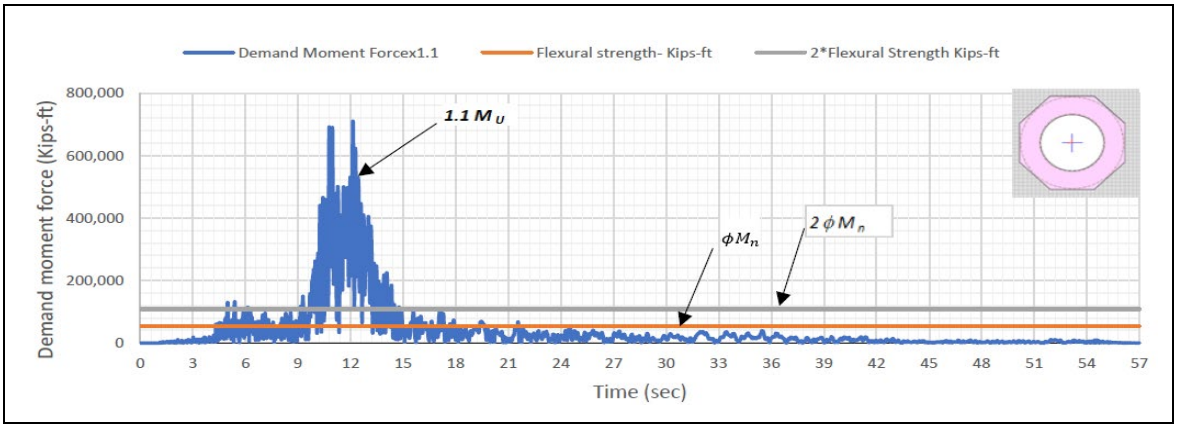


Figure 8. Demand Moment Force Vs. Capacity at EL 2290 Above the Elbow (RSN5656)

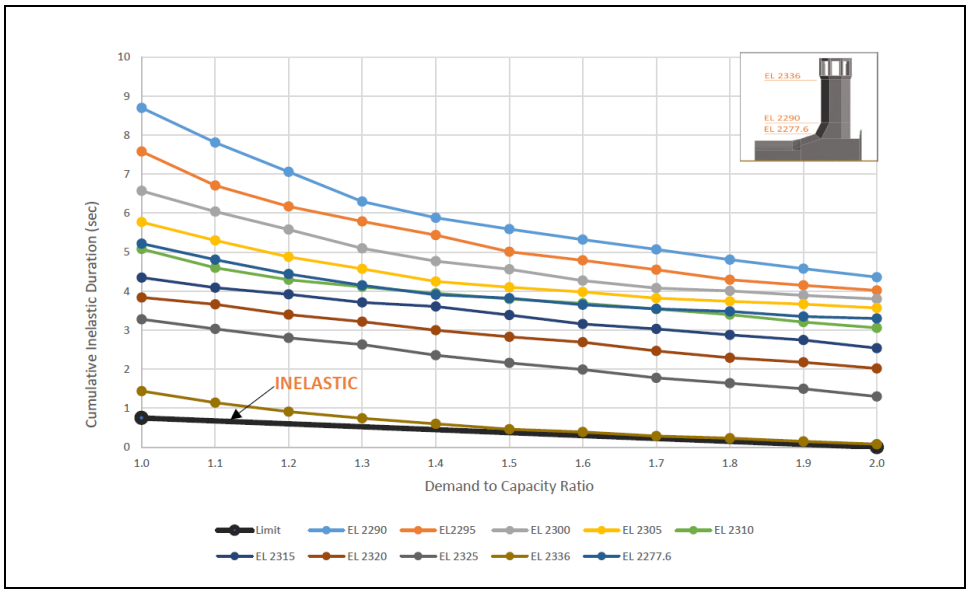


Figure 9. Cumulative Inelastic Duration Vs. DCR- Riser (RSN5656)

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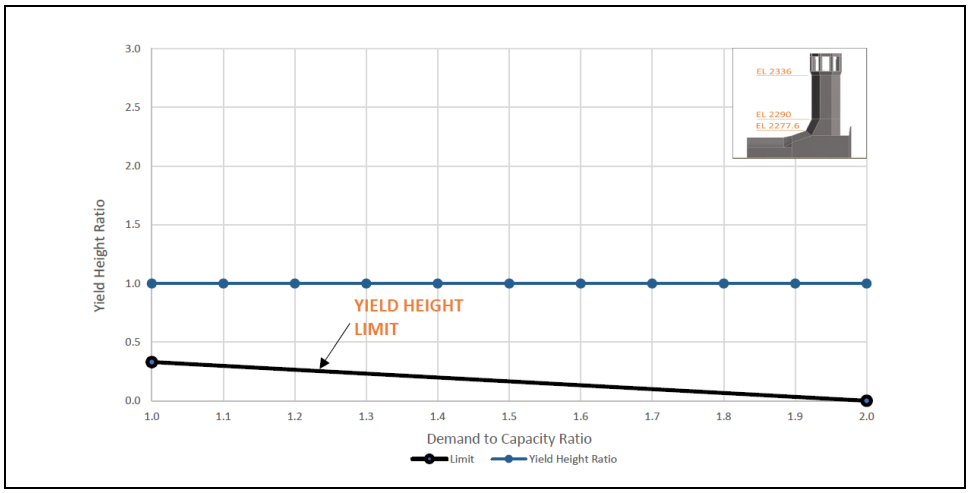


Figure 10. Yield Height Vs. DCR- Riser (RSN5656)

2) Shear Failure (Diagonal Tension)

The comparisons between the demand shear forces and the shear capacity for the cross-section at El. 2290 ft. for cases RSN5656 and RSN983 are summarized in Table 4.

Table 4 - Demand Shear (Diagonal Tension) vs. Capacity – at El. 2290 ft.

Ground Motion	Critical Section (EL.)	Time (sec)	Demand Shear $1.1V_u$* (Kips)	$1.1M_u$ ** (Kip-ft.)	R=$\frac{1.1M_u}{M_n}$	$V_{uMax} = 1.5V_u/R$ *** (Kips)	Shear Capacity ϕV_n (Kips)	DCR using $1.1V_u/\phi V_n$ ****
RSN5656	2290	11.21	7,406	73,114	1.34	8,231	3,608	**2.05**
RSN983	2290	7.74	8,127	96,603	1.60	7,596	3,852	**1.97**

The time-history of the demand “shear” force vs. the shear capacity (considering both plain and reinforced concrete cross-sections) for the section at Elevations 2290 ft., considering RSN656, are presented in Figure 11. The plots indicate that there are several excursions of the demand shear forces above the shear capacities of the section.

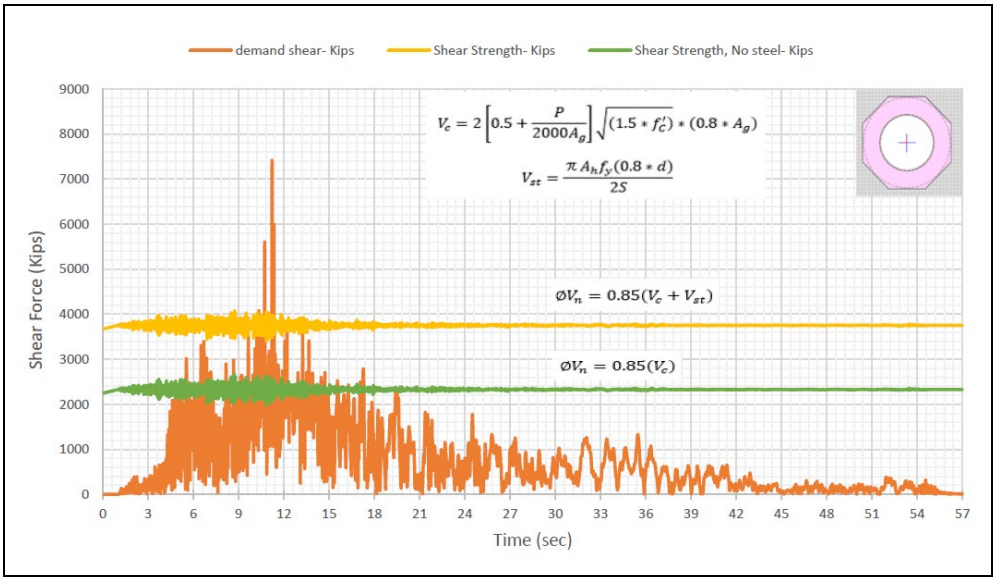


Figure 11. Demand Shear Force Vs. Shear Capacity Time-history at El. 2290' (RSN5656)

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3) Sliding Shear Failure (Shear Friction)

Considering RSN5656 and RSN983, Table 5 summarizes the comparisons between the demand sliding shear forces and the shear capacity values at the cross-section at El. 2290 ft. Based on the analysis results, the intake structure is not adequate against demand sliding shear forces at section at Elevation 2290 ft.

Table 5 - Demand Sliding Shear vs. Capacity – at El. 2290 ft.

Ground Motion	Section (EL.)	Time (sec)	Shear Demand $1.1V_u$* (Kips)	Axial Load P (Kips)	Nominal Shear Capacity V_{sl} (Kips)	DCR using $1.1V_u/\phi V_n$ **
RSN5656	2290	10.94	18,707	4,968 (tension)	0***	**N/A*** **
RSN983	2290	7.95	20,487	-792 compression	1,283	**15.96**

* Demand shear is calculated as $V_u = \sqrt{V_x^2 + V_y^2}$.
** The demand shear forces exceeded the capacity of the cross section at multiple time steps.
*** Per Equation $V_{sl} = \mu_{SF}(Pu + 0.25 f_y A_{vf})$, if $|Pu\ (tensile)| > 0.25 f_y A_{vf}$, the shear capacity is mathematically zero.

The time-histories of the demand sliding shear forces vs. the corresponding sliding shear capacities at the section at El. 2290 ft., considering RSN5656, are presented in Figure 12. The plots indicate that there are several excursions of the demand sliding shear forces above the sliding shear capacity of the section.

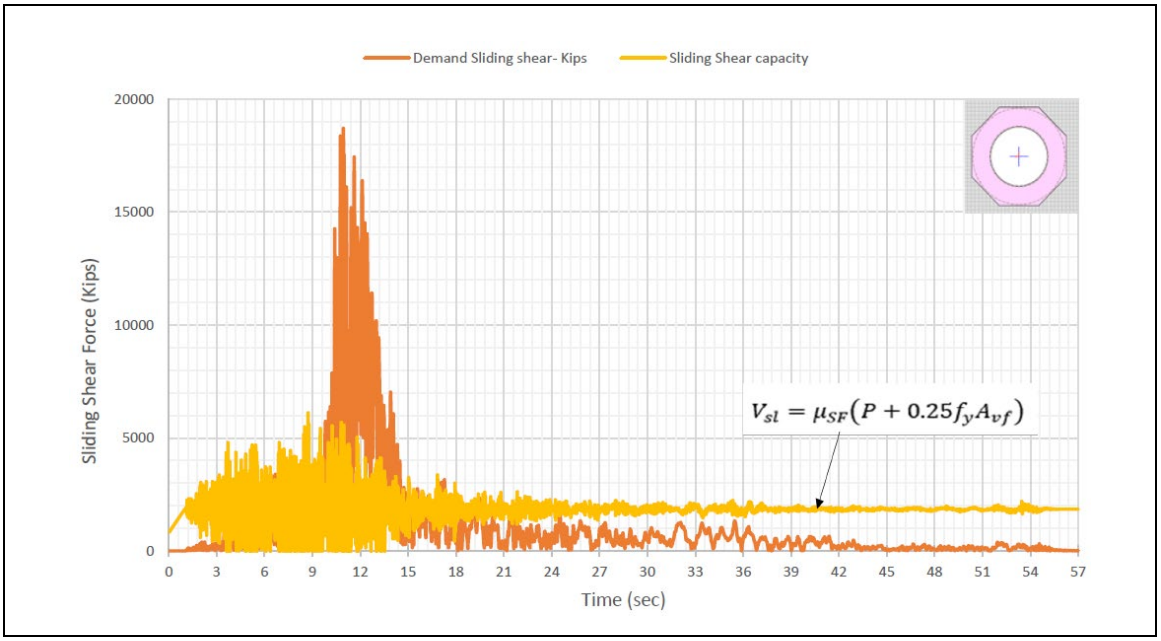


Figure 12. Demand Shear Force Vs. Shear Capacity Time-history at El. 2290' (RSN5656)

4) Fracture of Tensile Reinforcement Check

The Nominal Moment capacity (M_n) values were obtained using "section builder" feature in SAP2000 software. The geometry and reinforcement arrangement for the concrete cross section was obtained from available drawings. The "cracking moment" (M_{cr}) was calculated using Equation 4-9 of USACE (USACE, 2003a). Considering RSN5656, the time-history of the $\frac{M_n}{M_{cr}}$ at the section at El. 2290 ft. is plotted in Figure 13. The figure indicates that the tensile reinforcement at section at Elevations 2290 ft. will undergo sudden fracture. It is noted that inequality $M_n > 1.20M_{cr}$ is to check if the section is lightly reinforced and it will have a brittle failure. This requirement is different from the comparison between the demand moment versus the capacity of the section. For example, in low seismic areas, many of the vintage structures (especially spillway piers that were built in 60s), have enough flexural capacity,

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but they are lightly reinforced i.e., M_n<1.20M_{cr}.

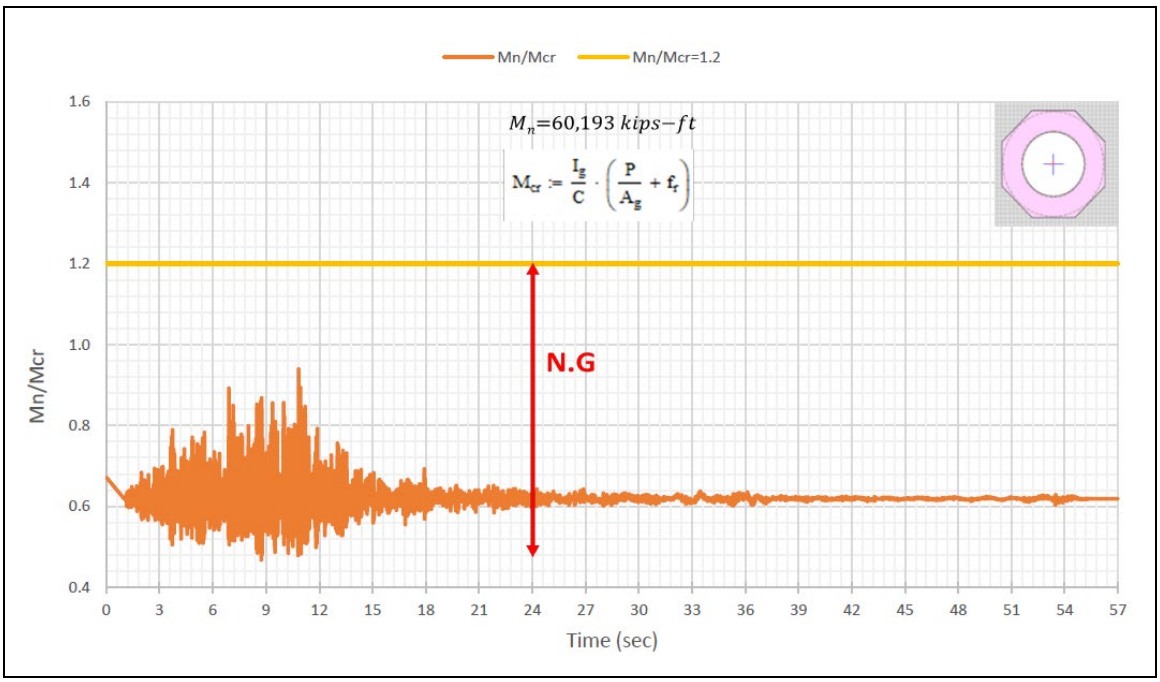


Figure 13. M_n/M_{cr} Ratio Time-History at El. 2290' (RSN5656)

VIII.DEVELOPMENT OF THRESHOLD GROUND MOTION

The "*Threshold Ground Motion*" is the scaled-down ground motion that will cause the demand forces within the critical section to reach the capacity of the section (i.e., DCR=1.0). Among all the failure modes investigated for the intake structure, the flexural failure mode appeared to be governing. The following are the steps that were taken to identify the threshold ground motion for the Pyramid Dam intake structure:

- Ran the LETHA using the five-ground motion time-histories.
- Selected the governing time-history which yielded the maximum DCR values. The analysis results showed that RSN5656 (longest time-history) would yield the maximum DCR values amongst the five time-histories.
- Selected the sections of the tower with the maximum DCR values. Section at Elevation 2290 ft. would experience the maximum DCR values.
- Performed Incremental Dynamic Analyses (IDA) using the governing time-history to scale down the acceleration time-history until the demand moment at the critical section reached the flexural capacity of the section (until DCR=1.0 was reached).
- Developed the corresponding acceleration response spectrum curve for the scaled-down acceleration time-history.
- Compared the scaled-down response spectrum to the PSHA results to estimate the corresponding return period for the scaled-down earthquake.

The incremental dynamic analysis results indicate that the maximum flexural moment at section at Elevation 2290 ft. will reach the capacity of the section (DCR=1.0), when the intake structure is subjected to ground motion RSN5656 which has been scaled down by a factor of 12.5 (RSN5656/12.5). Further IDAs were performed on the intake structure to identify the ground motion beyond which a nonlinear analysis would be required for evaluation of the seismic performance of the intake structure. The analyses indicate that, considering RSN5656/8.35, the damage control performance of the structure based on linear analysis is acceptable, and no nonlinear analysis is required to ensure that collapse will not occur.

Figure 14 shows the response spectra for earthquakes with 25 and 50 years return periods along with the response spectra for ground motions RSN5656/8.35 and RSN5656/12.5.

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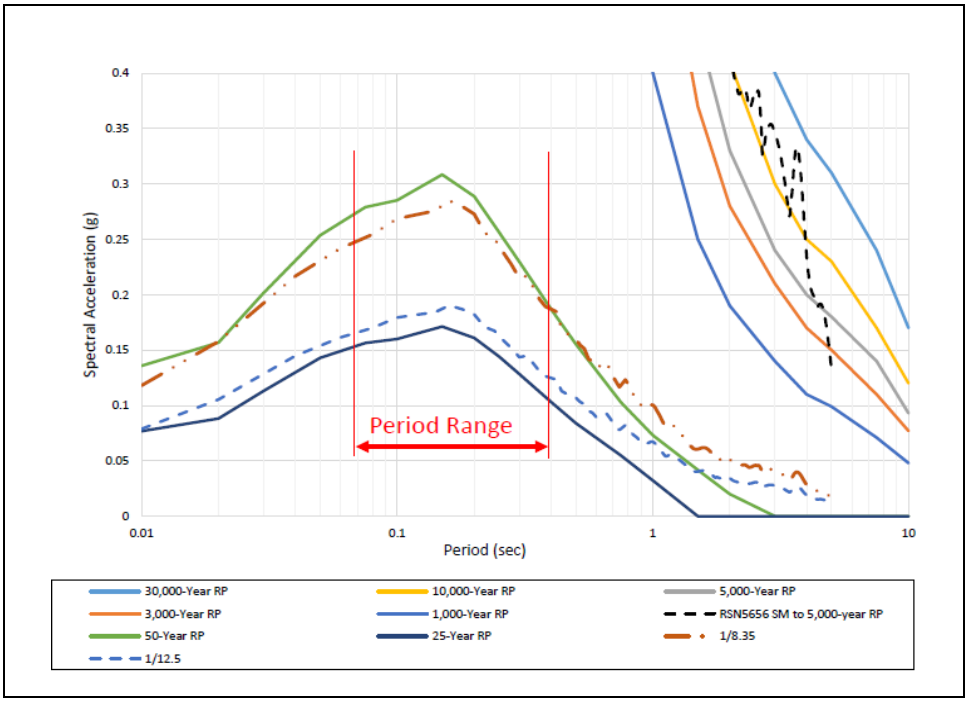


Figure 14. M_n/M_{cr} Ratio Time-History at El. 2290' (RSN5656)

Comparing the response spectra, it is concluded that RSN56568/8.35 and RSN5656/12.5 correspond to the earthquakes with return periods of 50 and 30 years, respectively. The analysis indicates that RSN5656/12.5 with approximate return period of 30 years is the threshold ground motion beyond which the demand moment forces at section at Elevation 2290 ft. will exceed the flexural capacity of the cross-section (DCR=1.0). It is noted that, the indicated earthquake with 30 years return period will not cause the intake structure to collapse or experience excessive deformation or damage. When the demand flexural forces gradually exceed the capacity of the section, the tensile reinforcing steel will yield, and the tensile cracks and compressive crushing will develop in the concrete (i.e., a plastic hinge develops at the section). As the demand flexural forces increase, the reinforcement bars will exhibit kinematic-hardening behavior, the joints will undergo incipient opening; and the stresses will redistribute along the structure. The indicated mechanism continues until the tensile reinforcement ruptures and the lateral deformation of the tower become excessive, i.e., the intake structure collapses.

IX.CONCLUSIONS

Several PFMs related to free-standing intake structures were investigated in this paper. The analyses show that due to high seismicity of the site, and the fact that the structure is lightly- reinforced, the demand flexural and shear forces exceed the capacity of the reinforced concrete against the flexure and shear at the critical locations along the height of the intake structure. Per the analysis results, the structure will fail first at El. 2290 ft. (above the elbow). Further, given that the structure is lightly reinforced, it will experience sudden fracture (brittle failure) during the seismic event, which is typical of many similar vintage intake structures, which do not have sufficient reinforcement.

The additional sensitivity analyses show that variation of concrete's compressive strength from 3,000 psi (base case) to 5,000 psi (upper-bound value) changes the DCR values; however, the conclusions related to the inadequacy of the intake structure during the seismic event do not change.

The intake structure will undergo flexural yielding (DCR≥1.0) at Elevation 2290 (right above elbow) by earthquakes with return periods larger than 30 years. It is noteworthy that the term "yielding" should be distinguished from "collapse". Once the demand flexural forces gradually exceed the flexural capacity of the section (1.0≤DCR≤2.0), the section will experience some level of damage in the form of yielding of tensile steel and

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cracking/crushing of concrete; and a plastic hinge will develop. The collapse occurs when the reinforcement ruptures, and the structure exhibits excessive deformation. The level of damage within the intake structure can be approximated by the linear time-history analysis if it undergoes earthquakes with return periods smaller than 50 years. i.e., there is no need to perform any nonlinear analysis to evaluate the performance of the tower against earthquakes with return periods smaller than 50 years.

The linear time-history analyses indicate that the intake structure will undergo extensive damage during an MCE. All the analyses were performed by assuming linear elastic material for the concrete, presence of reinforcement, effects of the joints, and increase in damping forces due to cracks and nonlinear response of the intake structure during the analyses. Nevertheless, considering the high DCR values obtained from the linear time-history analysis, it is expected that the nonlinear time-history analysis results (including material plasticity, smeared cracking, rupture of reinforcing rebar, etc.) will provide the same conclusions related to the structural performance of the intake structure. The advantage of the nonlinear analysis is that it will provide more detailed understanding of the tower's dynamic behavior and understanding of the progression of damage as compared to a linear analysis, which will better inform determining levels of risk for selected earthquakes as well as associated development of retrofit alternatives.

Given that the current codes and guidelines have more stringent requirements related to the ductility of intake tower structures, most of the 1960's vintage intake towers like that of Pyramid Dam are lightly reinforced, i.e., they fail in a non-ductile manner. Further, considering the significant increase in the seismic forces compared to those used for the original design of the old intake towers, these structures do not meet the performance requirements of the current codes and guidelines for the seismic loading. Per the recent Level 2 Risk Assessment (L2RA) for Pyramid Dam, failure of the submerged intake tower due to an earthquake was estimated to be a "damage state", and not lead to direct loss of life. In this work, the seismic performance of the Pyramid intake structure is checked against the requirements of US Army Corps of Engineers guidelines and American Concrete Institute code, the results of which will be used by DWR for their future risk assessments and potential dam safety risk reduction measures for this facility.

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DAM SAFETY PALM SPRINGS
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Replacing a Siphon with a Low-Level Delivery Conduit on a Soft Compressible Foundation

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ABSTRACT

The Weber Basin Water Conservancy District (WBWCD) operates a siphon near the southwest corner of the A.V. Watkins Dam, which is 14.5-mile long earth embankment, with a maximum height of 36 ft, that impounds the 250,000 ac-ft Willard Bay Reservoir. The dam was constructed near the eastern shore of the Great Salt Lake on soft to firm lacustrine clay soils interbedded with alluvial and beach sand deposits.

A new low-level conduit was designed to be constructed while retaining approximately 95,000 ac-ft of water in the reservoir. Chimney filters and drains, a butt-welded 1" thick steel conduit encased in reinforced concrete, and an LLDPE liner were included in the project design to reduce risks under static and seismic loading conditions. Risk analyses performed by the Bureau of Reclamation demonstrated that the risks associated with the new low-level conduit are less than the risks associated with the existing siphon.

I.INTRODUCTION

A.V. Watkins Dam impounds Willard Bay Reservoir, and is located about 6 miles southwest of Willard, Utah at the location shown in Figure 1. A.V. Watkins Dam is 14.5 miles long and has a maximum height 36 ft (11.0 m) above the original ground surface. Willard Bay Reservoir has a surface area of 10,000 acres (4,047 hectares) and storage capacity of 250,000 ac-ft (30,800 hectare-m). The dam is owned and regulated by the United States Department of Interior Bureau of Reclamation (Reclamation) and operated by the Weber Basin Water Conservancy District (WBWCD). The dam crest elevation is at 4237.0 ft (1,291.4 m)[1] and the maximum normal reservoir elevation is at 4228.0 ft (1,288.7 m).

WBWCD operates an existing siphon at the location shown in Figure 2. The siphon has two pipelines with intake elevations at about 4209 ft (1,283 m). From the intake, the siphon pipelines slope upwards to an elevation of about 4225 ft (1,288 m) beneath the crest of the dam embankment, then slope downwards to about elevation 4203 ft (1,281 m) at the outlets. WBWCD normally uses the siphon to provide 10 to 20 cfs (0.28 to 0.57 cms) of water for municipal and irrigation use. The water is discharged at the siphon outlet into the Great Salt Lake Canal, then flows south and west within the open channel canal to the water users.

The existing siphon does not have adequate hydraulic capacity to satisfy water delivery demand when the elevation of Willard Bay Reservoir is lower than about 4214 ft (1,284 m). WBWCD uses portable pumps to meet

[1] Elevations shown are Above Mean Sea Level.

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water delivery commitments during periods when the reservoir is low, which results in significant operating costs. After preliminary project studies which evaluated operational requirements and the condition of the existing pipelines, WBWCD made the decision to replace the existing siphon. Consideration was given to constructing either 1) a new siphon with greater hydraulic capacity and/or a lower crest elevation to meet water delivery commitments during periods when the reservoir is low, or 2) a low-level pipeline.



Figure 1. Vicinity Map [1]

II.SITE CONDITIONS

A.V. Watkins Dam is near the eastern shore of the Great Salt Lake. The Great Salt Lake is a remnant of the ancient Lake Bonneville, which covered most of northwestern Utah between about 30,000 and 13,000 years ago [2]. Lacustrine sediments thousands of feet thick were deposited within northern Utah during Lake Bonneville times [3]. During and following regression of Lake Bonneville, lacustrine, marsh, and alluvial materials have been deposited within the area of A.V. Watkins Dam [4].

The native soil within the dam foundation consolidates when loaded. Loads applied to the subgrade by the existing dam have resulted in as much as 18 ft (5.5 m) of settlement. The native clay soils beneath the western portion of the dam embankment are generally more compressible than the soils beneath the eastern portion of the reservoir, where there are more sand deposits within the upper 30 ft (9 m).

Sandy deposits are interbedded with clayey soil deposits beneath the entire dam alignment. Many of the sand layers are susceptible to liquefaction during an earthquake, even at peak ground accelerations as low as 0.12g. This acceleration has a probabilistic recurrence interval of about 150 years at the site. The estimated 1,000-yr recurrence acceleration is 0.43g. The calculated lateral spread displacement that could occur due to liquefaction is 8 ft.

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Figure 2. Site Plan [1]

A.V. Watkins Dam was originally constructed in three stages as a zoned earthfill embankment. The staged construction was planned due to the expected settlement at the site and potential for slope instability. This approach resulted in a large percentage of the settlement occurring during construction and an increased foundation strength prior to fully loading the soils. The existing dam was constructed using materials available at the project site, except that riprap, riprap bedding, and the crest surface course were imported. The dam was constructed without a filtered internal drainage system. The dam embankment has a 2.5H:1V (Horizontal:Vertical) downstream slope. The upper 18 ft (5.5 m) of the upstream embankment slope is 2.5H:1V, and the lower portion of the upstream slope is 10H:1V, resulting in an embankment base width of about 290 ft (88 m).

A Cement-Bentonite (CB) wall was installed between Stations 470+00 and 733+00 (see Figure 2) in 2008 after backwards erosion piping nearly resulted in failure of the dam [5]. The presence of the CB wall was a major consideration when evaluating potential locations for the new water delivery conduit.

The eastern portion of A.V. Watkins Dam is adjacent to agricultural land with a few residential structures, and the southwest portions of the dam are adjacent to the Great Salt Lake. Due to downstream conditions and relative proximity to existing developments and infrastructure, the eastern portion of the dam is classified as High hazard by Reclamation, and the southwest and western portions are classified as Low hazard.

III.PROJECT DESIGN

Replacing the siphon with a new more hydraulically efficient siphon or with a low-level delivery conduit was considered. A siphon would require vertical bends in the conduit, and the bends would be subject to stress concentrations induced by foundation soil consolidation, seismic ground shaking, liquefaction related settlement, and lateral spread. Considering these disadvantages of the siphon option, a low-level delivery conduit was selected and designed to avoid vertical or horizontal bends beneath the dam embankment. WBWCD requested a design intake elevation of the delivery system of 4200 ft.

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A 5,450-ft (1,662-m) long segment of the dam between Sta 398+50 and 453+00 was evaluated to determine the location for the new delivery conduit. Investigations performed during preliminary project design indicated that foundation conditions were not significantly different at the east end of the study area compared to the west end. Sta 404+00 was selected for the conduit for several reasons, including 1) the intake could be constructed with a flowline at elevation 4200.0 ft (1,280.5 m), providing access to a greater volume of water, 2) the existing siphon could be left in service during construction of the new delivery conduit, 3) the conduit would be located within the Low hazard portion of the dam embankment, 4) construction at the selected location would not interfere with the existing CB wall, and 5) this location's relative proximity to the Great Salt Lake Canal.

The project was designed to be constructed with the reservoir water surface at or below elevation 4,216 ft (1,285 m). The design includes a cofferdam with a length of about 800 ft (244 m) and maximum height of about 23 ft (7 m), which will allow the reservoir to retain up to 95,000 ac-ft (11,700 hectare-m) of water during construction. The cofferdam will be U-shaped and extend about 350 ft (110 m) upstream of the existing dam crest. The cofferdam was designed to allow construction of an intake structure at the upstream toe of the 10H:1V embankment. The cofferdam design requires creation of a working pad by pushing fill into the reservoir, installation of vinyl sheet-piles through the fill pad and into the underlying clayey soil to cut off seepage, and compaction of fill materials to the crest elevation. A plan view of the proposed cofferdam is shown in Figure 3.

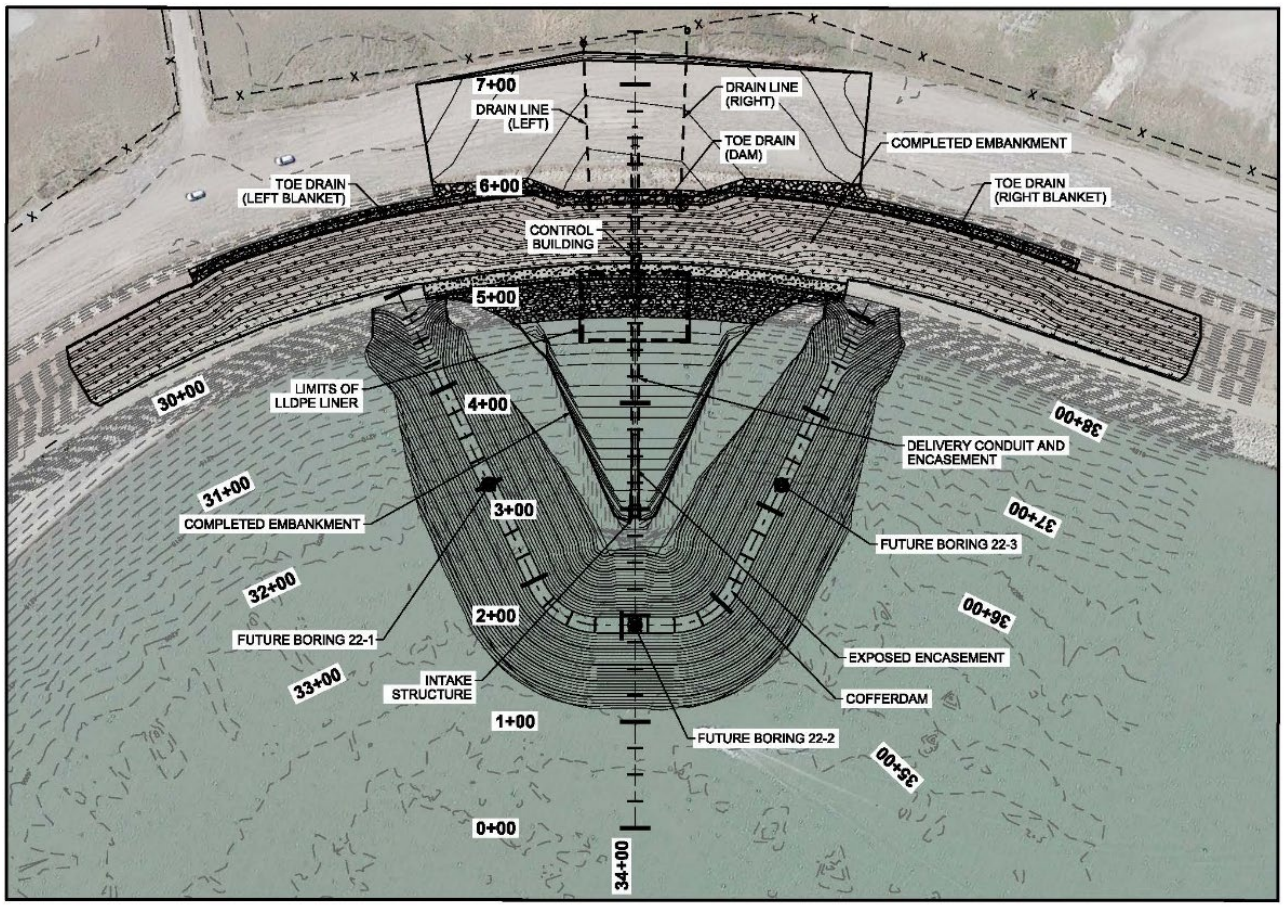


Figure 3. Cofferdam and excavation alignments [1]

Installation of the low-level delivery conduit requires excavation of the existing dam embankment and upper portion of the foundation, dewatering, installation of a welded steel pipe conduit encased in reinforced concrete, and reconstruction of the embankment. The design includes an internal filter and drain zones meeting current design requirements within the reconstructed portion of the embankment.

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Settlement at the project site was analyzed using information from laboratory testing and refined by calibrating the model to settlements measured during and following previous embankment placement. Approximately 7.6 ft (2.3 m) of settlement has occurred at the location of the proposed delivery conduit since construction of the Stage 1 embankment began in 1962. The existing dam was constructed in three stages to Elevation 4235 ft (1290.8 m) to maximize consolidation of the clay foundation materials during construction. About 3.5 ft (1.1 m) of the settlement occurred within the first three years after embankment construction began, and about 3.5 ft occurred between 1965 and 1990. The semi-logarithmic rate of settlement appeared to decrease substantially after about 1990, and design analyses for the current project estimated that future settlement at the delivery conduit location would be less than or equal to 0.3 ft (0.09 m) in 50 years.

Installation of the cofferdam embankment has the potential to cause settlement since stresses within native soil deposits will increase compared to existing conditions. A three-dimensional analysis was performed to estimate settlements which will occur due to construction of the cofferdam. The three-dimensional analysis was selected for project evaluation to better model stresses in native soil deposits within the area of the U-shaped cofferdam. The estimated construction schedule and the time rate of consolidation were included in the cofferdam settlement analyses. It was estimated that up to 3 ft (0.9 m) of cofferdam settlement will occur during construction of the project. It was also estimated that the existing dam will settle as much as 10 inches (4 cm) at the transition between the 2.5H:1V and 10H:1V upstream slopes, but less than 1 inch (2.5 cm) at the dam crest, while the cofferdam is in place.

IV.Risk Analysis

A Reclamation requirement for the siphon replacement project is that an analysis demonstrate that the risk for failure of the dam after modification be equal to or less than the risk for failure of the existing facility. A preliminary risk assessment was performed by Reclamation, who invited the participation of WBWCD and the design team. The risk assessment was performed by quantifying the risks for the existing siphon and comparing the risk estimates to those developed for the proposed low-level conduit. Because the existing siphon crest is only 3.5 ft (1.1 m) below the maximum normal operating elevation of the reservoir, the risk team estimated relatively low risk of failure for the existing facility. The risk assessment team considered the following conditions to be factors in initially assigning elevated risk of the proposed project compared to the existing siphon:

- Installation of a low-level conduit results in a risk that a void will develop beneath the conduit as the foundation soils continue to consolidate, which could result in internal erosion along the conduit.
- Settlement induced by installation of the cofferdam could result in cracking and internal erosion through the existing embankment.

The preliminary risk evaluations were used to inform the design process, and the following project features were included in the design to address the risks identified:

- Reinforced concrete keyways were added to the conduit encasement to interrupt voids that could develop beneath the conduit,
- The chimney filter and drain zones were extended about 100 ft beyond the planned cofferdam abutments to prevent internal erosion if the existing embankment cracks,
- An LLDPE liner was added to the upstream slope of the reconstructed embankment in the vicinity of the delivery conduit to prevent concentrated seepage if an embankment crack develops.

The Reclamation risk evaluation team re-assessed the project and estimated that the Annualized Failure Probability (AFP) for the updated design is approximately one order of magnitude lower than the AFP for the existing conditions. Based upon the results of the risk analysis, WBWCD was authorized to complete final design and proceed with construction of the low-level delivery conduit. A cross section of the designed low-level delivery conduit is shown in Figure 4.

The project was scheduled to be constructed in 2023 but was delayed due to the above average winter snowfall, which gave WBWCD the opportunity to fill Willard Bay Reservoir above the maximum construction pool elevation.

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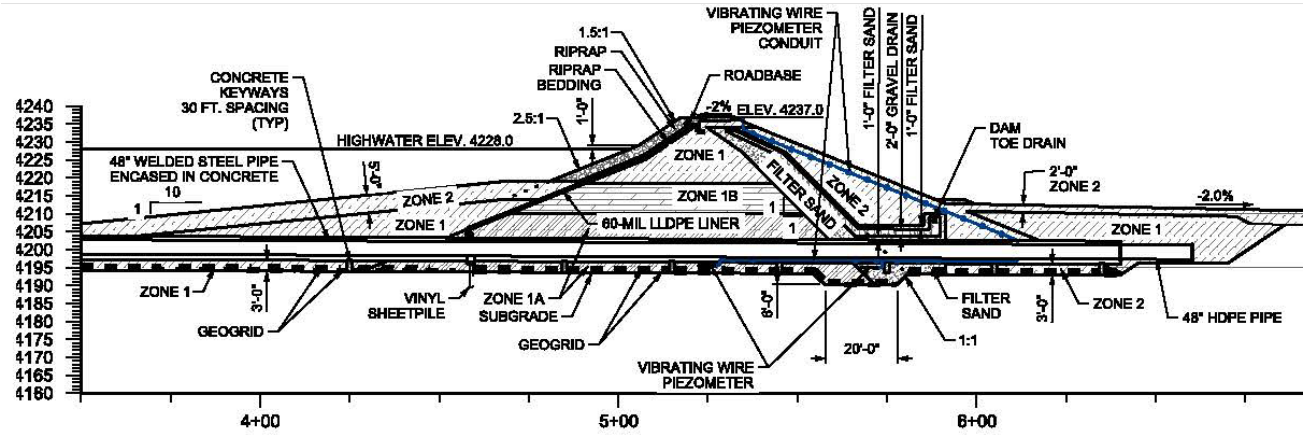


Figure 4. Low-level delivery conduit cross section [1]

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

NDSP BIL Implementation Progress/Updates

JENA'I JORDAN, FEDERAL EMERGENCY MANAGEMENT AGENCY

ABSTRACT

This presentation will review the latest updates regarding FEMA's NDSP BIL Implementation to include grants, the Planning for Dam Emergencies Collaborative Technical Assistance Program, training, and the 2023 Research Summit Report findings.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

Levees? Why Should States Care? Don't They Have Enough DAM Problems?

MICHAEL L. BACHAND AND JENNIFER LAUX, US ARMY CORPS OF ENGINEERS

ABSTRACT

The impending roll out of the National Levee Safety Program can seem overwhelming, especially for states who are being asked to play a key role in the implementation of the program. Consider this – it was over 40 years ago that a similar national-level program was rolled out for dams, and states are still struggling to implement that program due to a variety of reasons including being asked to do more (e.g., inspections, risk assessments and risk communication) with less resources. So why should states care about a new national program focused on state-led levee safety activities and programs when they already have enough challenges managing the nearly 65,000 state-regulated dams across the country? A Model State Levee Safety Program Guide is being developed under the National Levee Safety Program that includes a step-wise or scalable framework for utilizing the National Levee Safety Guidelines and implementing activities that are necessary to manage levees in a consistent manner (e.g., data management, inspection, assessment, communication and outreach, etc.). The National Levee Safety Program intends to collaborate with each state to work through the step-wise approach using a decision-making framework so they can decide what scale of levee safety activities and level of program (if any) is right for them. In addition to helping states understand why they should care about levee safety activities and/or programs and walking through the scalable approach, this presentation will focus on progress made to date on the overall National Levee Safety Program. There will also be opportunities to discuss how states can participate in the one-time levee assessment process and how the U.S. Army Corps of Engineers and the Federal Emergency Management Agency, both co-leads of the National Levee Safety Program, can further support states in levee safety activities or setting up their own levee safety program.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

Implementing Risk-Informed Decision Making in a Regulatory Dam Safety Program

DOUGLAS BOYER, RISK-INFORMED DECISION MAKING BRANCH CHIEF, FERC;
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ABSTRACT

Over the years, many papers and presentations have addressed the use of risk analyses and risk assessments in informing dam safety decisions. There have also been several papers espousing the benefits (and limitations) of risk analyses. Many of these papers have been written from a dam owners perspective illustrating the use of risk analysis in dam safety decisions, prioritizing potential dam safety actions and activities, and resource and budget allocation. However, not much has been written about the use of dam safety risk analyses and risk assessment from a regulatory perspective, and even less regarding implementation of risk-informed decision making in regulatory agencies and organizations charged with dam safety responsibilities.

Since 2009, the Federal Energy Regulatory Commission (FERC) Division of Dam Safety and Inspections (D2SI) has been contemplating, evaluating, and more recently, implementing risk concepts in our dam safety program. This paper will provide a summary of the evolution and development of risk within the D2SI regulatory program, the recent implementation of risk analyses in the periodic review of projects, and a discussion of the challenges and limitations faced and overcome in implementing risk in a regulatory dam safety program.

I. STANDARDS AND RISK

Dam safety programs, whether a dam owner's internal program or a regulatory program, are designed to assure the safe and reliable operation of dams. Traditionally, a standards-based approach (SBA) has been employed to dam engineering and dam safety. In this approach, safety is assessed by following established rules for design events and loads, structural capacity, safety coefficients, and defensive design measures. The SBA developed as a codification of the design practices of major dam owner agencies for design of new dams. Over many years, the traditional SBA has developed from recognized good practice that has been gained from basic theoretical considerations and empirical evidence. This recognized good practice, combined with experienced judgment, has generally served the objective of dam safety well, and is still a necessary component of dam safety management. However, there are some important limitations to an SBA:

- The frequency of deterministic loads such as a probable maximum flood (PMF), maximum credible earthquake (MCE), and other deterministic loads are not explicitly considered and are non-equal among projects. For example, an annualized frequency of the PMF at Dam A might be 1:500,000 while at Dam B it might be 1:12,000 or the annualized frequency of the PMF at Dam C might be 1:500,000 while the annualized frequency of the MCE at the same dam might be 1:900.
- It is difficult to compare dam safety issues or concerns at a single dam consistently and transparently. For example, how does one compare or prioritize seismic liquefaction vs. dam crest overtopping due to flooding vs. internal erosion vs. spillway ogee crest sliding stability vs. emergency spillway channel erosion concerns at a single dam or similar concerns at multiple dams.

Other SBA limitations include:

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- Typically, the SBA looks only at specific technical aspects of a dam, and not necessarily the sequences of events, conditions, operations, history, or temporal aspects that could cause a dam to fail.
- It does not provide nor is it applied equitably between practitioners, projects, or regions.
- It is not well suited for evaluating some important dam safety issues such as internal erosion, spillway gate reliability, human factors and operational issues, and others as no established guideline or criteria exists for these types of concerns.
- It focuses on the infrastructure and typically does not include or consider potential societal impacts.
- Practitioners and owners don't really know if they are solving the right problem or reducing the impacts of the identified deficiency.
- It can often lead to a "check-the-box" mentality, meaning meeting standards equates to safety, acceptance, and no acknowledge uncertainty.
- Relies on "best engineering judgment" that can vary greatly from project to project.

It has been demonstrated that some dams have a significant risk even when all criteria included in a traditional standards-based dam safety program are met. Due to the success of standards-based safety evaluations, dams seldom fail from sliding or overturning under normal, probable maximum flood, or seismic loadings. However, dams still fail. The most common failure modes for dams in the United States are internal erosion/piping and overtopping at floods less than the extreme flood (Von Thun, 1985). Traditional standards-based safety evaluations are unable to fully evaluate potential failure modes such as internal erosion, human factors, spillway gate reliability issues, and control systems. In addition, standards-based safety assessments do not assess the probability of failure and only indirectly account for the downstream consequences of failure, typically through varying the required factors of safety based on downstream hazard potential classification (Regan and Boyer, 2008). These are the gray areas of dam safety assessment.

The use of SBA also raises (or at least should raise) several important questions:

- Does meeting a guideline mean the dam is safe?
- When is a structural modification 'good enough'?
- What if you can't meet all the guidelines?
- What if the guidelines mean repairs are too expensive to make?
- What if the guidelines don't go far enough?
- Should all high hazard dams be treated equally?
- Are the risks posed by each high hazard dam equally tolerable to society?

Unfortunately, at times there's been a misconception in our industry regarding SBA. Meeting regulatory or industry standards was never intended to lead to the conclusion that the dam is 'safe' (i.e., no possibility of failure) or that performance of the structure or feature was guaranteed to be acceptable, or that there would be no uncertainty, or that no more thinking (critical thinking) would be required. SBA was never intended to just be a check the box approach!

Many industries have been confronted with this same issue. Like them, we must ask ourselves some hard questions:

1. Do we really recognize or acknowledge the limitations of the SBA to dam safety evaluations?
2. Should we consider different approaches if it improved our understanding and decision-making ability?
3. Should we let our reliance on using current approaches and challenges to overcome understanding a new approach and limited resources hinder us from improving our dam safety decision making?

Many industries and some dam safety organizations and agencies have recognized that there are other approaches that can address many of the SBA limitations. Risk concepts in dam safety are currently being used by several Federal agencies, some dam owners, and in international practice. The Federal Guidelines for Dam Safety (FEMA, 1979) predicted the increased use of risk approaches more than four decades ago when they stated, ". . . risk-based analyses will probably gain wider acceptance in the engineering profession and realize potential as a major aid to decision-making in the interest of public safety".

The RIDM approach is the process of making safety decisions by evaluating if existing risks are tolerable and present risk measures are adequate, and if not, whether alternative risk reduction measures are justified (Boyer and Allerton, 2016). For entities that own or regulate dams, various decisions are made regarding an individual structure or an inventory of structures, including decisions about the safety of a structure, necessary actions to reduce risks, and prioritization of actions for an inventory of structures.

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The many benefits of the RIDM approach are well documented (HSE, 2001; ANCOLD, 2003; Bowles, 1998; Regan and Boyer, 2009). Examples of some of the purposes risk has been used to inform dam safety decisions include (modified from Environment Agency, 2009):

- To systematically identify and better understand potential failure modes.
- To identify, justify and prioritize investigations and analyses to reduce uncertainties in risk estimates for individual dams and an inventory of dams.
- To strengthen the formulation, justification and prioritization of risk reduction measures for individual dams and an inventory of dams.
- To justify decisions on reservoir operating restrictions.
- To identify ways to improve dam safety through changes in reservoir operation, monitoring and surveillance, safety management systems, staff training, emergency action planning, and business decisions related to dam safety.
- To identify opportunities to improve the effectiveness of warning and evacuation plans.
- To identify cost-effective options for more rapidly achieving reduced dam safety risks.
- To justify expenditures on dam safety improvements to owners and economic regulators.
- To identify and understand those risks that exist through normal operation (non-failure risk) of the project.
- To provide a framework for quantifying engineering judgment and communicating technical issues with dam owners in a more open and transparent manner.
- To facilitate the evaluation of dam safety risks to the public in a manner that allows comparison with other infrastructure and technological hazards.
- To provide a non-technical basis for communicating dam safety risks to the public.
- To provide a basis for development of a safety case or safety demonstration for owners and regulators.
- To assess the adequacy of insurance coverage.
- To strengthen the basis for corporate governance related to dam safety risks.
- To strengthen the exercise of the owner's duty of care, due diligence and legal defensibility with respect to dam safety incidents or dam failure.

A risk-informed approach to dam safety acknowledges that:

- A reliance on standards alone is not enough.
- There are no simple numerical solutions – That dam safety decisions are informed by and not based on the results of risk analyses and risk assessments.
- Risk compliments but does not replace traditional engineering standards or experience.
- Risk provides a credible means to capture and treat uncertainty in decision making.
- Risk is a periodic and continuing process and not a one-and-done approach.
- Risk becomes an integral approach to all aspects of the dam safety program and that it is not simply a “bolt on” approach.
- Risk is a tool that can help deepen our understanding of the fundamental dam safety issues and concerns at a dam and within a safety program in a meaningful way that results in more efficient use of limited resources and more effective dam safety decisions resulting in reduced risks to the public and the dam owner.

II. FERC RIDM APPROACH

There were several possible pathways the FERC considered in developing a RIDM approach (FERC, 2016). Those included:

1. Using RIDM as sole method of making dam safety decisions.
2. Using RIDM as an alternative method.
3. Using RIDM to enhance the SBA.

The FERC supported using RIDM to enhance the SBA for the following reasons:

- The long-term standing of the traditional SBA.
- The need to gain wider acceptance in the community of tolerable life safety guidelines and the use of as-low-as-reasonably-practicable (ALARP) concepts.
- The need to develop improved methods for estimating the uncertainties in risks.
- The limited expertise in the dam safety profession in risk analysis and risk assessment processes.

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So, from the FERC perspective, RIDM provides an enhancement to the SBA (Boyer and Allerton, 2016). Satisfying the SBA does not guarantee that tolerable risk will be achieved, though that may often be the case. In general, the SBA will likely be met if tolerable risk is achieved, although there may be exceptions, which should be carefully justified. The systematic steps of potential failure mode analysis (PFMA) and ALARP evaluation in the RIDM approach will often result in dam safety issues being identified and addressed which may not have been either identified or addressed using the SBA. For these reasons, dam owners are encouraged to use the RIDM approach to enhance the understanding gained from the SBA. Doing so will assist the defensibility of decision making.

There was previously a view by some that risk assessment was a means to justify less costly safety upgrades of dams than those required by the SBA. It is now recognized that such a view seriously misunderstands the true aim of risk assessment, which is more informed decision making than would be possible from reliance on the SBA alone. It may be that the additional understanding that comes from the risk assessment process will reveal that a less costly solution to a dam safety problem could be justified, though a decision that way should be made with great care and having regard to all the community risk and business risk considerations. But it could as easily be the case that risk assessment shows that a more stringent safety level, and thus a more costly solution, ought to be implemented.

Inherent in most risk-informed dam safety programs is the concept that traditional guidelines and risk assessment methodologies are not mutually exclusive but are in fact both key components of a thorough dam safety program.

As a regulator FERC does not perform risk analyses. FERC established a risk analysis process and guidance for performing the risk analyses, identified the risk measures required to be provided, established documentation requirements, and defined qualifications for the risk analysis team member. FERC's role during the risk analysis is to provide oversight of the process and review of the risk analysis reports and documentation. FERC reviews risk analysis results, the evaluation of the risks, performed by either the consultant or dam owner (licensee), and the plan and schedules for completing the recommendations.

III.EVOLUTION OF RIDM IN FERC

In the late 1990's FERC began to internally discuss the possibility of incorporating risk principles into the dam safety program. At that time the Bureau of Reclamation was regularly engaged in performing risk analyses and using RIDM in their dam safety program to help prioritize dam safety actions and to help make dam safety decisions. As a regulator, FERC was looking for improved methods and tools that could be used to better understand dam safety issues and to help make better decisions. Rather than taking on the effort of developing a larger and more comprehensive risk-informed program, the decision was made to take smaller, incremental steps to slowly introduce and trial risk-informed approaches and to evaluate the benefits and value these newer approaches. Those initial risk discussions lead to the initial development of draft guidance in 2001 that eventually became the Potential Failure Mode Analysis (PFMA) part of Chapter 14 of the FERC Engineering Guidelines in 2003. Over the following years, PFMAs became an integral element of FERC's dam safety program, being routinely performed as part of Part 12 Independent Consultant Inspection Reports, as well as design and construction evaluations.

As FERC dam safety leadership recognized the value of PFMAs and as other Federal agencies, including the U.S. Army Corps of Engineers, communicated the benefits they were seeing by incorporating risk into their dam safety program, FERC began to explore other risk tools and looked to potentially broaden the application of risk within the program. In 2008 FERC began developing a risk portfolio prioritization tool to be used internally to provide another approach to better understand risk analysis concepts and to better understand how risk might be used within the dam safety program. At that same time, a dam safety risk framework was included in the 2009 five-year FERC strategic plan (FERC, 2009). That framework provided a series of steps FERC was planning to undertake in the development and implementation of risk into the dam safety program.

Per the FERC's Strategic Plan (2014), "Risk-informed decision making (RIDM) provides the capability to assess nontraditional failure modes, . . . focus inspections and surveillance on the specific potential failure modes and monitoring programs at projects, and guide remediation projects to provide an overall reduced level of risk to the public. FERC will continue to develop the necessary risk assessment guidelines, procedures and policies, and to train Commission staff, dam owners, and consultants in risk assessment procedures, methodologies and tools" (FERC, 2014).

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In 2010 a small ad hoc group was formed within D2SI to begin to develop additional portfolio risk analysis methodologies and to work with industry in developing risk analysis guidance documents. In 2015 the Risk-Informed Decision Making Branch of D2SI was created and in 2016 Interim RIDM Guidelines were published (FERC 2016). In establishing risk guidance, and from a regulatory perspective, FERC identified three distinct components of RIDM – risk analysis, risk assessment, and risk management and communication – to develop policies and procedures. This framework followed the RIDM framework laid out in the Federal Guidelines for Dam Safety Risk Management, published one year earlier, and shown in Figure 1. These new interim risk policies and procedures established a scalable approach to conducting and implementing risk analyses including establishing side boards for performing various levels of risk analyses, defining minimum qualifications in terms of knowledge, training, and experience for those involved in performing risk analyses, establishing minimum documentation standards for risk analyses, establishing tolerable risk guidelines and measures to evaluate ALARP, and outlining risk management and risk communication concepts in the management and prioritization of RIDM actions.

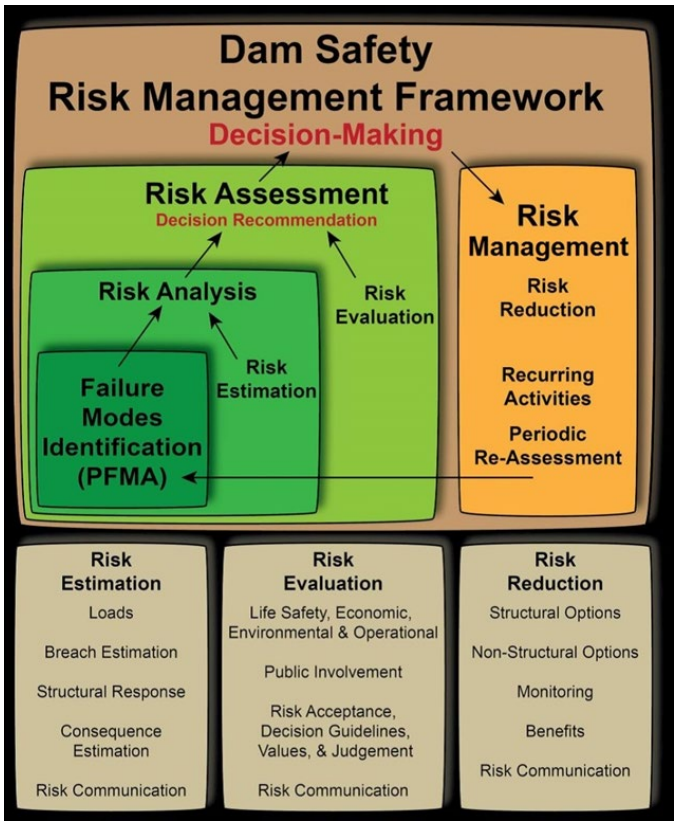


Figure 1. Relationship Between Risk Analysis, Risk Assessment, and Risk Management (adapted from FEMA, 2015).

With the publishing of the risk guidelines, risk analysis pilot studies were solicited with the intent to:

1. evaluate the RIDM policy and processes, including identifying potential shortcomings in the interim guidelines;
2. evaluate the risk methodologies, including looking for holes or gaps in the methodology and guidance, identifying potential challenges, and identifying potential tools that would be beneficial for future risk analyses;
3. identify future training needs for risk implementation; and
4. obtain input from licensees and consultants on the overall process and methodologies. The 2017 Oroville spillways incident precipitated many advancements in FERC's dam safety program. Some of these included improvements to the PFMA process and inclusion of a risk analysis as an added element to the Part 12 process.

In 2018, the US Congress directed the execution of a risk analysis for Oroville Dam using draft Level 2 Risk Analysis (L2RA) available at that time.

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In December 2021 FERC published revised guidance for PFMAs and new L2RA guidance (FERC, 2021a and FERC 2021b). Implementation of that guidance began in 2022.

Since 2016, forty-one risk analysis pilot studies have been performed or are in the process of being completed. These studies have covered the full range of risk analysis complexity from screening level to quantitative risk analysis. Most of these pilot studies have been performed using semi-quantitative approaches.

IV.BENEFITS OF RIDM FROM A REGULATORY PERSPECTIVE

Unlike Reclamation or the USACE, the FERC, like state regulatory agencies, is not a dam owner. The liability of FERC regulated dams is clearly stated in the Federal Power Act as belonging with the owners of the dam (FPA, 1920). The FERC's Division of Dam Safety and Inspections (D2SI) regulates approximately 2,520 dams owned by more than 800 different owners. The dams include 833 classified as high hazard potential and 216 classified as significant hazard potential with heights ranging from 770 feet to less than four feet.

In its role as a regulator the FERC does not prioritize remediation across its entire portfolio of regulated dams. Therefore, the common use of risk assessment to conduct a portfolio risk assessment is not germane to the FERC. Similarly, the FERC does not distinguish between an immediate need for remediation and a longer term need for remediation. This distinction is only valid when resources within a dam owning organization are limited. Instead, the FERC's application of risk informed decision-making is focused on assessing the safety of individual structures in the context of safeguarding life, health and property (Regan and Boyer, 2009).

Beyond utilizing risk assessment methodologies in a RIDM process focused on individual dams, the FERC also believes risk assessment methodologies can assist the FERC:

- Identify FERC-D2SI's highest risk dams
- Prepare for and conduct focused inspections of all dams
- Identify dams which may require team inspections
- Identify dams which may require special instrumentation inspections
- Identify dams which may require more, or less, frequent inspections
- Provide a tool to allocate resources commensurate with the risk associated with individual projects.
- Communicate the risk associated with dams in a consistent manner among Federal agencies, state dam safety regulators, dam owners, and the public.

FERC recognizes that risk assessment methodologies are advisory and not used as a prescriptive tool to judge the safety of a dam. Risk assessment methodologies, although quantitative, do not provide precise numerical results. Rather, site-specific considerations, good judgment, and all other relevant factors should be applied along with traditional analyses and risk assessment methodologies in dam safety decision-making.

We also believe that risk assessment methodologies can provide benefits to owners' dam safety programs, including:

- Addressing regulatory challenges
- Decision making using deterministic vs probabilistic guidelines and criteria
- Understanding the frequency of extreme loading
- Providing a framework where all high hazard dams are treated equally
- Providing a transparent and consistent process for prioritization of dam safety activities (e.g., not all previously identified Category II potential failure modes (PFMs) are equivalent. The loading conditions vary (seismic, flood, normal, operational aspects, etc.) and it's not easy to judge and defend priorities, particularly when those priorities can, and often do, change with different perspectives and individuals.)
- A consistent means to identify and defend the urgency of dam safety actions

Risk analysis included a more transparent approach to 'how good is good enough' through the incorporation of ALARP.

V.DAM OWNER RESPONSIBILITIES

The dam owner and operator are in the legal position of being responsible for the safety of their dam, its operation, and the consequences of a failure should one ever occur. All dam owners should fully understand and appreciate their legal, regulatory, moral, and social obligations of owning a dam. Without a deliberate effort to

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understand the risks that a dam imposes on its surroundings, in both the magnitude and frequency of the hazards and magnitude of potential consequences, including impacts to life, health, and property, an owner cannot fulfill these obligations (FERC, 2016).

In addition to community interests, such as risks to life, third-party property (economic), and the environment, the owner needs to consider the financial risk relevant to business and asset risk management. This consideration should include business responsibilities to consumers and the community, credibility and political issues, and potential financial and legal liabilities arising from a dam failure or misoperation. Such considerations could warrant a higher level of safety than indicated by the assessed risk to life, economic, and environmental risks.

Dam owners, in setting their own tolerable risk policies, need to have regard to FERC's RIDM guidelines, legal and political constraints within which they operate, the legitimate interests of society, and to recognize good practice.

Finally, the owner has a duty and responsibility to communicate information to the community on the risks associated with dam failure, operation, and flooding.

VI. CHALLENGES AND LIMITATIONS

Is a risk-informed approach perfect? By no means. There are limitations to this approach as well. Some of these limitations include:

- Lots of technical and professionally derived judgment is needed (just like standards, but more transparent)
- Risk concepts are poorly understood (assigning probabilities to conditions/events is not always intuitive)
- Results of risk analysis are not necessarily intuitive
- Risk analysis results and information can be difficult to communicate (particularly by engineers)
- Risk analysis participants need practice and experience to gain confidence and consistency in the process

Several challenges to implementing RIDM approaches in dam safety regulatory agencies remain:

- As a relatively new approach to evaluating dam safety in a regulatory environment, the RIDM process and understanding is mostly new and untested in the United States. Dam owners, consultants, and regulatory agencies and staffs are not familiar with some aspects of this new approach and procedures. There remains a general lack of understanding of the benefits and application of risk in the industry. These changes will take time to gain the knowledge and the experience in the process before they reach a broader acceptance and confidence is achieved in using this approach.
- There currently exists a relatively small number of individuals who have prior knowledge and experience in risk analysis, risk assessment, and risk management. This lack of qualified individuals limits the number of risk analyses that can be performed until additional individuals can be trained. Focused training and on-the-job experience will be critical to the development of a sufficient pool of individuals to perform risk analyses and assessments.
- There exists an already heavy workload for dam safety regulatory agencies. Adding more work with a limited ability to add staff or convert existing personnel is often difficult to impossible. On top of that it is difficult to find individuals that have the knowledge and experience in RIDM policies, procedures, and practices to help develop internal RIDM procedures.

As mentioned above, many challenges exist to integrating and implementing RIDM into the FERC dam safety program. As a regulatory agency in dam safety, FERC has made significant strides in evaluating how RIDM approaches can be applied in a regulatory setting. Key successes in developing and implementing RIDM within the FERC dam safety program include:

- Do your homework. Seek out those in the industry that have embarked on the RIDM journey and have applied RIDM concepts and practices in their dam safety programs. In the late 1990's FERC sought out individuals and agencies with prior risk knowledge and experience to help lay a foundational understanding of how risk could be used in a regulatory setting. This proved extremely valuable in setting a twenty-year pathway toward implementing risk.
- Start simple. Consider developing a portfolio risk analysis of dams within your jurisdiction. Perhaps start with high hazard dams. This information can be helpful in developing an understanding of risk

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terminology and concepts by hands-on application with existing staff resources. This can also be helpful in providing a better understanding of the dam safety risks within an inventory of dams, including what failure modes are contributing most significantly to the risk within the inventory. Ultimately this can be used to help identify potential dam safety activities and priorities, training topics, and potentially future hiring. Lastly, this information can also be helpful in identifying dam safety inspection priorities, surveillance and monitoring improvements, and emergency action plan (EAP) enhancements.

- Manage expectations. Eat the elephant one bite at a time. Don't try to do too much too quickly. Develop an overall vision, achievable goals, and clearly defined tasks for how to implement RIDM approaches into the dam safety program over time. Develop foundational understandings and an initial simple plan that addresses the early steps in the process. Don't get caught up or overwhelmed about details that may exist many steps or years in the future. Remember, you must crawl first before you learn to walk. It's only after you've learned to walk that you can then run. And when you run, you should seek to fly. But all in good time.
- Cultivate and train internal staff. Seek out opportunities to find training opportunities and hands-on applications in daily tasks to slowly integrate RIDM concepts and approaches into routine activities such as dam safety inspections, prioritizing routine dam safety activities, reviews of EAPs, and reviews of engineering analyses and designs.

VII. FUTURE

So, where to from here?

Within the FERC dam safety program, we see the potential for several opportunities for the continued growth and implementation of RIDM within the dam safety program, some of which may take many more years to develop and implement. Some of these include:

- Continue to train existing D2SI staff in RIDM. Like for most of our industry, RIDM experience is still rather limited in our staff. The implementation of L2RAs in 2022 as part of the Part 12 process will go a long way in assisting D2SI and industry with this effort. As we gain more experience through the L2RAs in the early years of implementation, we anticipate additional internal and industry training will be required – both in procedures and practices and in risk methodologies and risk evaluations. The development of supplemental guidance and risk analysis tools is also anticipated.
- Expand the use of quantitative risk analysis methods for the evaluation of specific potential failure modes. As dam owners and consultants gain experience in risk through the performance of L2RAs, we expect to see an increase in performing more detailed quantitative risk analyses for the evaluation of specific potential failure modes. This will require the development of a thorough understanding of tolerable risk concepts and application of ALARP. Again, supplemental guidance and tools and additional internal and industry training are anticipated.
- Continue to develop a deeper understanding of making risk-informed decisions. Again, as industry develops more understanding, experience, and confidence in RIDM concepts, the need for using risk concepts to aid in decision making is expected to grow and broaden. Some dam safety decisions are hard and will require expert understanding in RIDM concepts.
- Application of risk-informed design. Risk-informed design approaches practices are beginning to be implemented in some Federal agencies. It is expected that that trend will continue in the broader dam safety industry as more experience is gained applying these principles and concepts. And like above, supplemental guidance and tools and additional internal and industry training are anticipated.

Risk is no longer that four-letter word that was quietly muttered in the dark halls of dam safety conferences and workshops as it was in the 1990's. It has been demonstrated that RIDM is a useful approach and tool and does improve our dam safety decision making ability. In the past only practiced by large dam-owning Federal agencies, RIDM is now being implemented within regulatory dam safety programs, dam owners, and in the dam industry and will likely continue to see increased application in the dam safety community.

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Climate Vulnerability Assessment for Possible Hot-Dry and Warm-Wet Future Climates for Dams in California

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ABSTRACT

An evaluation of the change in hydrologic hazards under future climate conditions was completed to assess potential climate vulnerabilities. Hydrometeorological data were adjusted based on global climate model output to quantify the effects of climate change for four climate scenarios. Predicted changes in temperature and precipitation were used to represent hot-dry (hotter temperatures with diminished precipitation magnitudes) and warm-wet (warmer temperatures with enhanced precipitation magnitudes) for the time horizons 2050 and 2070.

The Stochastic Event Flood Model (SEFM) was used to develop climate-adjusted hydrologic hazards: peak magnitude-frequency relationships for reservoir inflow and outflow rates, and inflow and outflow volume, as well as water surface elevation.

Changes in precipitation were greatest in winter months for synoptic-scale storms. Increases in temperatures were greatest in the summer months. Hydrologic loading curves increased from baseline for the warm-wet scenarios and decreased for the hot-dry scenarios. Local storm hydrologic loading curves decreased for both scenarios.

I.INTRODUCTION

The purpose of this study was to develop hydrologic hazard curves (HHCs) for two possible future time horizons (2050 and 2070) for a number of Southern California Edison (SCE) dams on the western slope of the Sierra Nevada Mountains. Global climate model projections of changes in temperature and precipitation were used to adjust the hydrometeorological inputs to the Stochastic Event Flood Model (SEFM) to develop climate-adjusted HHCs.

The analysis included two storm types: mid-latitude cyclones (MLC) which are characterized by long-duration storms (72-hours) that originate over the Pacific Ocean typically in winter months and have large areal coverage; and local storms (LS), which are typically isolated short duration (2-hour) convection events that occur during the warm season. Separate HHCs were developed for each storm type using SEFM.

The basic concept employed in the stochastic approach is the simulation of multi-thousand years of flood annual maxima. This is accomplished by utilizing a deterministic watershed model along with rainfall-runoff and snowmelt computation modules and treating the hydrometeorological inputs as variables instead of fixed values. Each flood simulation contains a set of climatic and storm-related inputs. Monte Carlo sampling procedures are used to allow the climatic and storm-related hydrometeorological inputs to vary in accordance with that observed in nature while maintaining dependencies between them. The result is a multi-thousand-year series of annual maxima flood hydrographs. HHCs for annual maxima flood characteristics are developed as probability-plots for selected flood

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characteristics for each reservoir. HHCs of interest typically include maximum reservoir inflow rate; reservoir inflow volume; maximum water reservoir elevation; depth and duration of reservoir level above user-specified elevations; maximum reservoir outflow rate; and reservoir outflow volume.

II.FUTURE CLIMATE HYDROMETEOROLOGICAL INPUTS

Downscaled, bias-corrected gridded Global Climate Model (GCM) data from CMIP5 (https://cal-adapt.org/) were used to evaluate changes in precipitation-frequency (PF) between the baseline (historical) period (1986-2005) and two projected future periods: 2050 (2041-2060) and 2070 (2061-2080). The primary statistics used in this study were the average monthly number of days exceeding precipitation thresholds, average monthly total precipitation, and average monthly maximum/minimum temperature. For each period, the statistics were calculated for each month and then averaged over the respective 20-year period. To minimize noise and spatial uncertainty of GCM data, the data were averaged over a contiguous, climatically-similar region for application in the study.

Two scenarios, hot-dry and warm-wet, were identified to represent upper and lower bounds of the potential future changes. The climate scenarios were based on the following set of criteria:

- Hot-dry: 10th percentile change in precipitation, 90th percentile change in temperature. This scenario consists of diminished precipitation magnitudes with much hotter temperatures.
- Warm-wet: 90th percentile change in precipitation, 10th percentile change in temperature. This scenario consists of enhanced precipitation magnitudes with somewhat warmer temperatures.

The incremental change (i.e., delta) in future precipitation relative to the historical period is shown in Figure 1 for 2050 and 2070, respectively. The hot-dry scenario results in an overall decrease in precipitation, especially in the cool season months from November-December. Conversely, the results for the cool season period from November-February show in an increase in precipitation in the warm-wet scenario. Both scenarios show smaller absolute changes in the amount of monthly precipitation during the warm season relative to the cool season. Most precipitation in this region occurs during the cool season from MLC storms. Precipitation in the warm season typically occurs in the form of local convective events that are smaller in both magnitude and areal extent.

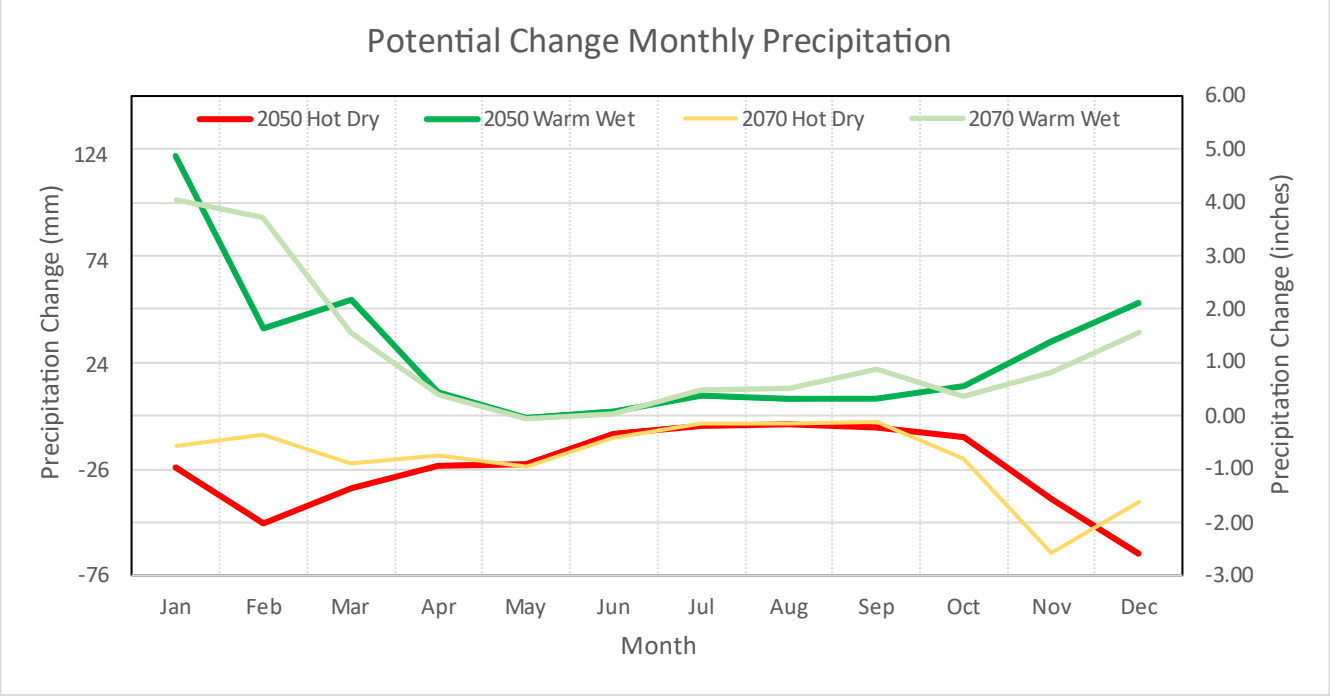


Figure 1. Potential changes between monthly historical and future monthly precipitation in 2050 and 2070 for both scenarios in inches and millimeters.

Future changes in monthly precipitation can be the result of changes in the number of storms that occur in a month in addition to changes in precipitation magnitude for individual storms. A peak-over-threshold (POT) statistic was used to better apply the monthly delta information at a daily timescale. The POT statistic represents the average

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number of days per month when an MLC or Local storm occurred. The threshold was 0.50 inches for MLC events and 0.07 inches for Local storms. Events exceeding these thresholds were counted and averaged per month over the relevant periods. For each month, the ratios of future count to baseline count were used to adjust individual annual maximum events according to changes in the both the POT statistic and monthly total delta.

Future changes in precipitation were applied to annual maximum series (AMS) data at representative index stations to evaluate changes in PF. An L-moment regional analysis methodology was used to conduct point PF analyses for baseline conditions. These findings were used to develop areal-average watershed PF relationships and 90% uncertainty bounds for baseline conditions at the watersheds for the two storm types.

A watershed PF relationship is an input to the SEFM and the magnitude of basin-average watershed precipitation (storm volume) is a dominant factor in the magnitude of resultant floods. The 4-parameter Kappa distribution was used to describe the watershed PF relationships for both the baseline and future periods (the results for an example watershed is shown in Figure 2 and Figure 3 for MLC storms and Local storms, respectively).

The changes in watershed-specific PF estimates between baseline and future periods were computed based on changes in L-moment statistics for the collection of index stations and using previously established PF areal reduction factors (ARFs) which convert precipitation magnitudes from point to areal precipitation. The LS PF-ARF represents a notable reduction in precipitation magnitude reflecting the relatively small areal scale of convective thunderstorms in the inter-mountain areas west of the crest of the Sierra Mountains. Whereas a minimal difference between point and areal precipitation for the MLC storm type is due to the very large areal extent of MLC storms relative to the smaller size of the watershed.

The resulting L-moment statistics were used as inputs to an uncertainty analysis. The outputs from the uncertainty analysis yielded mean and median watershed PF relationships, 90% uncertainty bounds, and associated distribution parameters for the 4-parameter Kappa distribution for the various climate scenarios.

It should be recognized that the sample L-moment statistics computed from the climate-adjusted precipitation AMS are imperfect surrogates for changes in L-moment statistics for projected future climates. Sample statistics for skewness and kurtosis have inherently high sampling variability and computed changes in L-moment statistics for the higher order L-moment ratios, L-skewness and L-kurtosis, were not considered reliable for making inferences regarding possible changes in the regional probability distribution. Accordingly, the L-moment statistics for L-skewness and L-kurtosis for the baseline climate conditions and the associated Kappa distribution parameters were retained for use with the future climate scenarios.

The following conclusions were made after review of the group behavior of changes in L-moment statistics for the two climate scenarios, two future years (2050 and 2070) and two storm types:

- For Hot-Dry scenarios, the watershed mean was considered to be non-stationary.
- For Warm-Wet scenarios, the watershed mean and variance were considered to be non-stationary as indicated by changes in the L-moments for the watershed mean and L-Cv.

A review of Figure 2 and Figure 3 show that the watershed PF relationships for hot-dry future climate conditions (red/orange curves) are similar in magnitude to current baseline conditions (blue curve). For warm-wet future climate conditions (green curves), precipitation magnitudes increase above the current baseline conditions with larger increases for rarer AEPs. Limitations regarding the interpretation of these results includes the inability of GCMs to adequately model short duration, local thunderstorms and other additional epistemic uncertainties in PF estimation of future climate projections.

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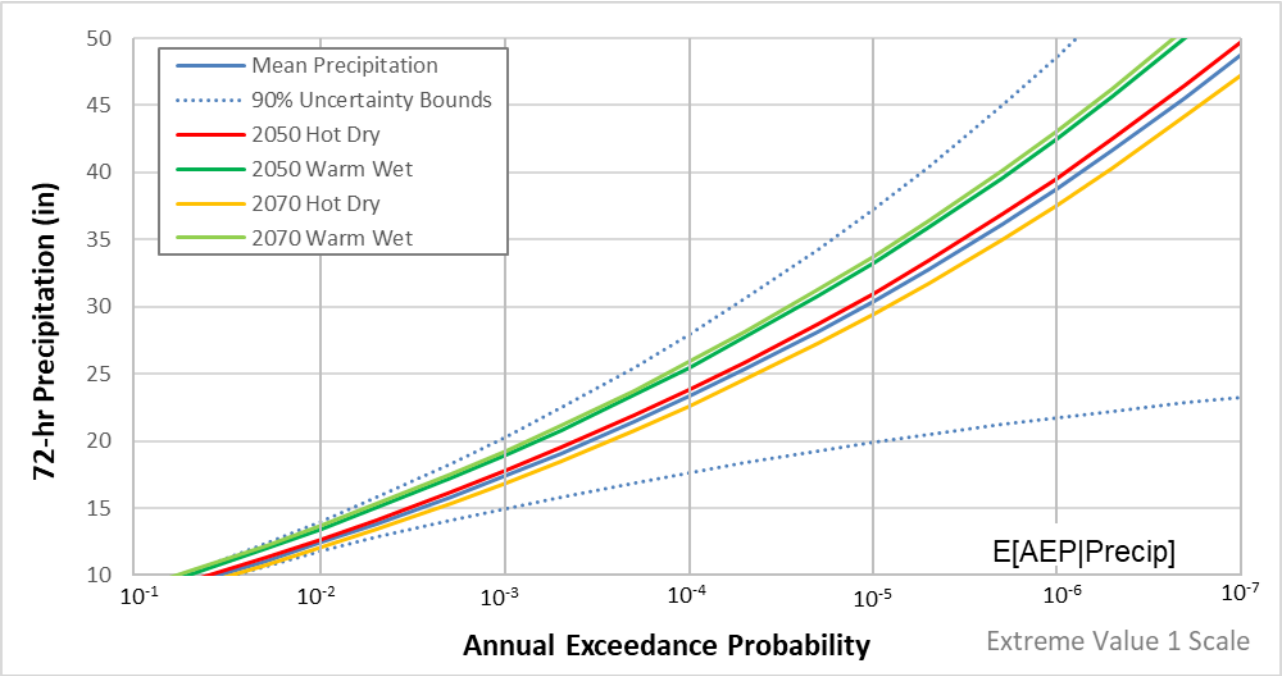


Figure 2. Watershed precipitation-frequency relationships for an example watershed for 72-hour MLC storms for baseline and future climate conditions.

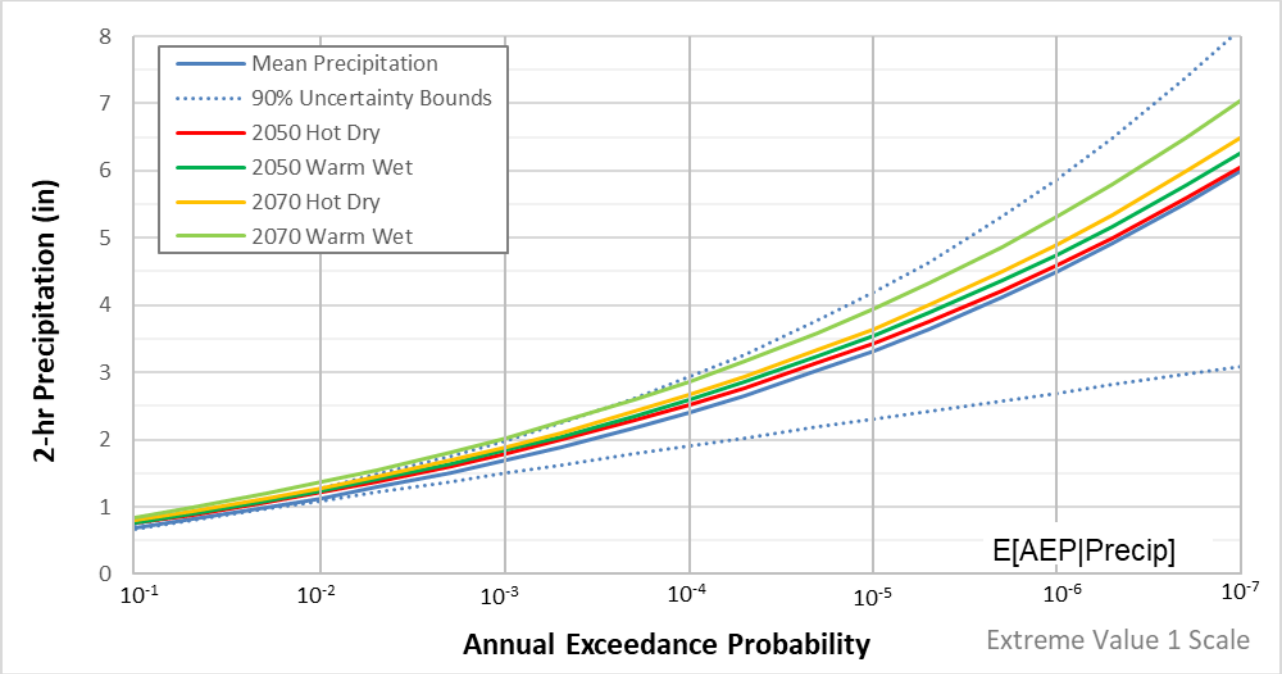


Figure 3. Watershed precipitation-frequency relationships for the example watershed for 2-hour Local storms for baseline and future climate conditions.

III.FUTURE TEMPERATURES AND FREEZING LEVEL ELEVATIONS

Changes in temperature were comparatively straight forward. Average monthly minimum and maximum temperatures were estimated for all GCM models in the hot-dry and warm-wet scenarios and then averaged for

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both the historical (baseline) and future periods. These temperature deltas were applied to the historical daily temperature time series and storm reconstructions for hydrologic modeling of individual storms. Daily temperatures affect seasonal evapotranspiration, soil moisture, and seasonal snowpack accumulation and melt-out. Temperatures during storm events affect snowmelt contributions for rain-on-snow flood events.

Monthly 1,000-mb dewpoint temperatures with an exceedance probability of 1:100 are inputs to an algorithm in SEFM for stochastically generating 1,000-mb air temperature time series for modeling snowmelt runoff. The stochastically generated 1,000-mb air temperature along with a stochastically sampled air-temperature lapse rate are used to compute an indexing value of freezing level. The indexing values of 1,000-mb air temperature and freezing level are used within SEFM for scaling hourly time-series patterns of 1,000-mb air temperature and freezing level observed in historical storms. This procedure provides for scaling of air-temperature time-series for elevation bands in a watershed for use in determining the phase of precipitation (rainfall or snowfall) and to compute snowmelt during rainy periods for rain-on-snow flood simulations.

A warming climate would be expected to produce higher 1,000-mb dewpoint temperatures and higher freezing levels compared to baseline conditions. Outputs available from the GCM models did not provide direct information on future changes in 1,000-mb dewpoint temperatures. Alternatively, changes in dewpoint temperatures were inferred from the changes in precipitation magnitude. Using a relationship between precipitable water and dewpoint temperatures, changes in 1,000-mb dewpoint temperatures were back-calculated from changes in precipitation assuming a 1 °F increase in temperature per 5% increase in precipitation for a given storm efficiency for the range of 1,000-mb dewpoint temperatures (nominal 60 °F) for very rare to extreme fall-winter storms in south-central California. This is based on several assumptions and considerations including that warmer temperatures and greater moisture atmospheric moisture holding capacity leads to greater precipitation magnitudes.

IV.HYDROLOGIC WATERSHED MODEL

The Stochastic Event Flood Model (SEFM; Schaefer and Barker 2021) was used to develop HHCs for both the current and future climate scenarios. Stochastic simulation of floods is accomplished utilizing the SEFM watershed model along with the SACSMA-Holtan rainfall-runoff model and UBC snowpack and snowmelt algorithm and treating the hydrometeorological inputs as variables. Monte Carlo sampling procedures are used to allow the climatic and storm-related inputs for each flood simulation to vary in accordance with that observed in nature while maintaining dependencies between hydrometeorological variables. Execution of the SEFM watershed model, SACSMA-Holtan runoff module, UBC snowpack and snowmelt module and reservoir routing of the inflow floods results in a multi-thousand-year series of annual maxima flood hydrographs.

The stochastic simulation procedure can be grouped into five steps.

Step 1: Select date of storm occurrence based on historical seasonality of storm occurrences.
Step 2: Select all hydrometeorological inputs associated with occurrence of the storm event: watershed PF, storm template for spatial and temporal distribution of precipitation, index value of 1000-mb air temperature, air temperature lapse-rate.
Step 3: Select watershed initial states from long term continuous watershed simulation based on storm date.
Step 4: Run rainfall-runoff model using SACSMA-Holtan and snowmelt modeling using the UBC snowpack and snowmelt module. Use the SEFM watershed model to compute streamflows and route flood hydrographs to individual reservoirs. Reservoir inflow hydrographs are saved for each simulation.
Step 5: Route flood hydrographs through each reservoir and spillway system using level pool routing techniques and reservoir operating rules.

This process is repeated thousands of times. The results are then used to calculate flood frequencies for the hydrologic parameters of interest and assemble HHCs. Table 1 lists the hydrometeorological inputs to the stochastic model and the dependencies that exist in the stochastic simulation of a particular hydrometeorological input.

Table 1. Listing of Hydrometeorological Inputs to Stochastic Flood Model and Dependencies that Exist in Simulation of the Hydrometeorological Inputs

Model Input	Dependencies	Probability Model	Comments
Storm Seasonality	Independent	Histogram	Storm date sampled on daily time step

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Watershed Precipitation Frequency	Independent	4-Parameter Kappa Distribution	Developed from regional precipitation analyses and application of PF-areal reduction factor
Storm Templates: Temporal and Spatial Distribution of Storms	Independent	Resampling from historical storms.	Storm templates obtained from largest historical storms observed by rain gages and radar data.
Temperature Temporal Pattern	Temporal patterns are matched one-to-one to Storm Templates.	Resampling from historical storms	Pattern indexed to 1000-mb Temperature and Freezing Level for day of maximum 24-hour precipitation
1000-mb Air Temperature	Precipitation Magnitude	Physically based stochastic model	For maximum 12-hour average 1000-mb air temperature for day of maximum 24-hour precipitation in storm
Air Temperature Lapse-Rate	Independent	Normal Distribution	For day of maximum 24-hour precipitation in storm
Freezing-Level Height	1000-mb Temperature, Temperature Lapse-Rate and Precipitation Magnitude	Physically based stochastic model	For maximum 12-hour average Freezing Level Height for day of maximum 24-hour precipitation in storm
Watershed Initial State Variables	Storm Seasonality	Resampling of historical conditions	Sampled from continuous simulation timeseries, stored on a daily time step. Sampled year is independent, sampled month and day corresponds to storm day.
Initial Reservoir Level	Seasonality of Storm and Watershed Initial State Variables	Resampling of historical conditions	Sampled from recorded reservoir level data. Sampled year has similar antecedent precipitation as year sampled Watershed Initial State Variables. Sampled month and day corresponds to selected storm day.

For development of HHCs under future climate scenarios, model inputs were adjusted for changes in temperature and precipitation as described above.

V.HYDROLOGIC HAZARD CURVES

Hydrologic loading curves comparing future climate scenarios with current conditions are summarized in Figure 4 and Figure 5. 90% uncertainty bounds on the current climate HHCs were quantified using a simplified approach which addresses the uncertainty in the watershed PF relationship which is the dominant factor for the simulated floods. The hydrologic loading curves were increased (more severe, more common) for the warm-wet scenarios. The hot-dry scenario loading curves were decreased (less severe, less common). The future climate conditions scenarios are contained within the 90% uncertainty bounds for the current climate conditions for the MLC storm type for exceedance probabilities more rare than 10^{-5} and for the LS storm type for AEPs more rare than 10^{-4}.

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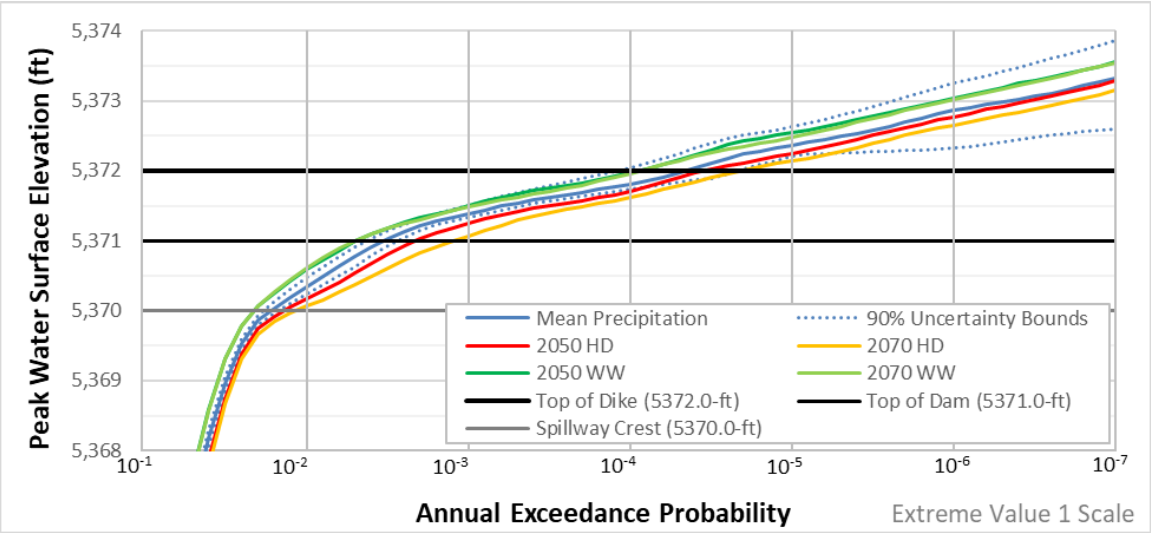


Figure 4. Hydrologic Hazard Curve for Maximum Reservoir Elevation for the MLC storm type for Current and Future Climatic Conditions.

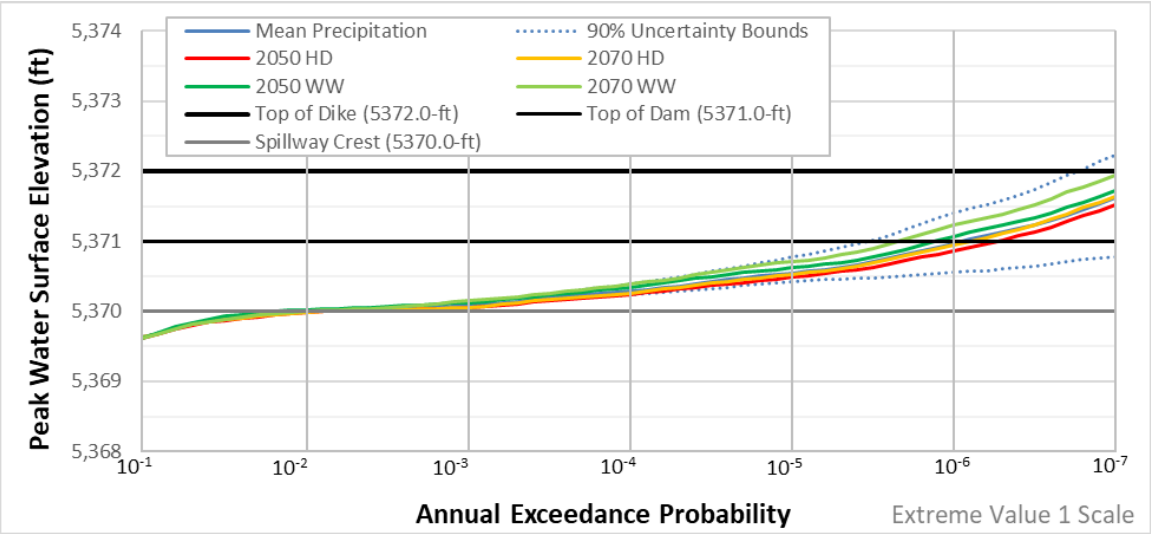


Figure 5. Hydrologic Hazard Curve for Maximum Reservoir Elevation for the Local storm type for Current and Future Climatic Conditions.

VI.ACKNOWLEDGMENT

The authors gratefully acknowledge Southern California Edison for their funding and contribution to the contents of this document.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

Development of Precipitation Frequency Areal Reduction Factors for Dams in British Columbia

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ABSTRACT

An accurate estimate of watershed precipitation frequency (PF) is crucially important for computing flood-frequency for assessing spillway performance and conducting other dam safety studies that aim to protect critical infrastructure. While British Columbia (BC) has recently updated point-PF estimates, a missing piece of updated information is how to translate the point-based findings into PF information applicable to a specific watershed. Spatial analyses of storm reconstructions in BC have been completed previously as part of probable maximum precipitation (PMP) studies which provide storm-focused areal reduction factors (ARFs). However, PF areal reduction factors (PF-ARFs) are needed for geographically fixed areas (watersheds) for developing watershed PF relationships for modeling watershed flood-frequency. For any given average recurrence interval (ARI) of precipitation, the ratio between the areal-average watershed PF curve and the areal-average point-PF curve is the PF-ARF. In particular, the PF-ARF values for a given watershed are not fixed values but vary with ARI. In this study, PF-ARFs are developed for the BC region in two ways. First, a watershed-specific approach is used in which a stochastic storm generation algorithm is used to develop a watershed-PF curve for a given watershed. This is accomplished by sampling historical storms to re-create spatial patterns over the watershed and producing a simulated watershed-average annual maxima series which is then used to develop the watershed PF relationship. This method is conceptually correct, but computationally intensive, and provides a PF-ARF for only a single watershed area and duration, across multiple ARIs.Therefore, an approach is needed that can take advantage of available spatial storm analyses to develop PF-ARFs applicable for watersheds across the entire BC region. A simplified approach is developed that utilizes significant historical storms of various durations and computes a storm-focused ARF as the ratio between the maximum point precipitation and the areal-averaged precipitation for various sizes of domains. By repeating this process and synthesizing results across multiple storms, durations, and area sizes, a statistically robust relationship for storm-focused ARFs as a function of watershed area can be developed. These results are then compared with the findings of stochastically generated watershed PF relationships as a calibration mechanism to identify adjustments needed to convert storm-focused ARFs to PF-ARFs for watersheds throughout British Columbia.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

Reanalysis of Record-Breaking Storm in the Colorado Rockies- Combining Meteorology with Hydrology to Find the Answer

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ABSTRACT

A localized thunderstorm produced extreme rainfall and flooding in July 1999 at a high elevation, interior location of Colorado, near the town of Saguache. This storm was utilzied for PMP development during the Colorado-New Mexico Regional Extreme Precipitation Study (CO-NM REPS) and was controlling of PMP depths for the local storm type in mountainous regions in that study. A partnership between Colorado State University (CSU), Colorado Department of Water Resources (DWR) and Applied Weather Associates (AWA) was formed to investigate high-elevation extreme precipitation and runoff events with the goal of updating hydrologic guidelines in Colorado. AWA's Storm Precipitation Analysis System (SPAS) rainfall data were used as input for flood runoff reconstruction to determine if the rainfall as originally analyzed would produce the flooding observed in the location. For the Saguache July 1999 storm, the result of this investigation demonstrated that the original total storm rainfall amounts were likely accurate, however, the exact location, spatial extents, and timing needed to be updated. Based on initial modeling, a detailed investigation took place to more accurately quantify the rainfall and flooding associated with this storm. This was completed by performing bucket survey type analyses, talking with witnesses of the event, and investigating radar scan data to derive additional information to update the original storm analysis. These updates were applied to the hydrologic model for sensitivity analysis and ultimately were applied to the CO-NM REPS study and PMP outputs. This presentation will detail the on the ground "bucket survey" that was completed in cooperation with CSU and DWR to derive new data, how those additional data were applied to update the original storm analysis, how the updated storm analysis was applied to the CO-NM REPS tool, and the results of those updates on PMP and flood runoff investigation.

BACKGROUND AND NEED TO REVISIT SAGUACHE, COLORADO, JULY 24-27, 1999 STORM ANALYSIS

DISCREPANCY BETWEEN RADAR STORM ANALYSIS, STREAM GAGE HYDROGRAPH AND RAINFALL-RUNOFF MODEL

In 2001, the Colorado Water Conservation Board (CWCB) sponsored a simple-level effort to document several extreme local thunderstorms and flash floods that occurred in the mountains of Colorado in 1999, including the Saguache storm, a July 25, 1999, storm in the Jacks Creek and Middle Creek basins, located about 13 miles

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northwest of Saguache, Colorado . It was described by Henz (2001) as “an intense stationary thunderstorm complex that formed along the mountains to the northwest of Saguache, Colorado. This storm was but one of three impressive flash flooding events that occurred over western Colorado between July 25th and July 31st [1999]”.

Documenting these storms at this time proved to be very valuable because otherwise these small-scale severe storms and their important contributions to Probable Maximum Precipitation (PMP) and hydrologic analysis would be lost. However, due to its cursory nature, the report left questions, and detailed data analysis went unanswered. The CWCB report (Henz, 2001) estimated a rainfall volume of 13,242 acre-feet from the July 25, 1999 storm over a roughly 64 square mile area, based on his analysis of radar data (pg. 12). However, later (pg. 23) Henz stated that the storm covered an area of roughly 72 square miles with a basin average rainfall of 1.84 inches, and that a smaller area of average rainfall of 3.5 inches covered an area of roughly 30 square miles, which yield rainfall volumes of 7,066 acre-feet and 5,600 acre-feet, respectively.

As part of Colorado and New Mexico’s Regional Extreme Precipitation Study (REPS) (CO DWR and NM OSE, 2018), Applied Weather Associates provided hourly subbasin-average rainfall for the July 25, 1999 storm, which they derived from their SPAS gridded rainfall analysis. They estimated basin average rainfall of 0.763 inches over the 517.7 square mile drainage area above the SAGSAGCO stream gage, which yields rainfall volume of 21,070 acre-feet.

The storm occurred on side tributaries just upstream from Colorado Division of Water Resources’ Saguache Creek near Saguache (SAGSAGCO) stream gage. As a cross-check, we calculated that the runoff volume from the July 25-26, 1999 SAGSAGCO stream gage was 380 acre-feet, only 1.8% of AWA’s rainfall volume. Other historical Colorado mountain floods modeled by Colorado State University as part of our concurrent mountain hydrology research study yielded runoff percentages ranging from 23% to 69% (note: these were from long duration, lower intensity general storm events). The Saguache Creek July 25, 1999 storm was an intense convective storm from which we would expect a large amount of infiltration-excess runoff. The REPS Volume II Appendix F depth-area-duration table for the July 1999 storm lists peak 1-hour intensities of 4.9 in/hr over 10 square miles and 2.6 in/hr over 100 square miles. NRCS Web Soil Survey lists basin soils as gravely loam, loam, and clay loam within the top 12-inches of the soil profile with saturated hydraulic conductivities ranging from 0.2 to 6 in/hr. Even if we assume runoff only occurred from the most intense 10 square mile, 1 hour storm center, the associated storm center rainfall volume would be 2,613 acre-feet and the SAGSAGCO stream gage hydrograph runoff is still only 15% of the rainfall volume.

The REPS study (DWR, 2018) reports a maximum observed rainfall of 5.0 inches and a maximum radar grid amount of 6.68 inches, which fell in about 2 hours from July 25, 1999 3:00 pm to 5:00 pm local time. For comparison, REPS precipitation frequency 10e-6 and 10e-7 annual exceedance probability (AEP) 2-hour Local Storm point precipitation frequency estimates for the area are 4.84 inches and 6.09 inches, respectively. Therefore, the REPS-reported July 25, 1999 maximum radar amount of 6.68 inches exceeds the 10e-7 AEP point precipitation frequency estimate; very extreme indeed.

BUCKET SURVEY INVESTIGATION

The extreme magnitude of the reported Saguache July 25, 1999 rainfall tends to control REPS 1 and 2-hour Local Storm probable maximum precipitation (PMP) estimates in the mountains above 7,500 feet elevation, which typically controls spillway sizing for mountain reservoirs, most of which have small drainage areas. Therefore, it was important to Colorado Dam Safety that the storm analysis was as accurate as possible and that our hydrology methods are able to reasonably reconstruct the observed Saguache flood. We began investigations of the aforementioned rainfall and runoff volume discrepancies by performing a bucket survey-type investigation with local residents. Conducting a bucket survey more than 20 years after-the-fact is obviously less than desirable, but we found that residents seemed to have vivid recollections of this extreme event.

First Colorado Dam Safety contacted the local *Saguache Crescent* newspaper. Although the *Crescent* did not have records of the storm or flooding, they provided contact information from local ranchers. The first rancher we contacted was a now-75 year old woman who ranched on Jacks Creek. She had a very clear recollection of the storm, as if it made a lasting impression. She recounted how she was running cattle at the time, with her mother. The blackest cloud she ever saw formed to the west of her ranch house on Jacks Creek (Figure 1). The first thing

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she heard was loud hail to the west. Then came the hardest rain she has ever seen. It rained 7 inches, measured at a rain gage on her fence post at the ranch house. Then the flood waters started coming up on both sides of the house. She said the storm lasted about an hour and a half, then it was calm. A then-88 year old rancher nearby told her that it was the hardest rain he had ever seen, and he had spent his life in those mountains on horseback. She said Jacks Creek, Cross Creek and Middle Creek flooded. The storm hit high-up on Cross Creek, up in the meadows where they had cows. The ranch house is at 8,500 feet; the storm was above that. It rolled up the grass like carpet and knocked over fences. They lost one cow there. On Middle Creek the flood was more like mud. The land and vegetation were different there. The ranch house at the bottom of Middle Creek filled with mud. She said the flood went down to the highway [CO 114] and knocked down the fences there and flooded the road. The highway crew came out the next day to fix the fences, but they were so full of rattlesnakes and tumbleweeds that it took some time. The creeks were still running the next morning. She guessed the storm probably went about a mile and a half north and south and 3 miles east; very localized – the next ranch down the hill barely got any rain.

Next we called that rancher, located about a mile down the mountain, along Middle Creek. His house is on west side of Middle Creek, immediately north of Hwy 114. He confirmed that it did not rain at his house. In fact, he was getting ready to put up hay that afternoon of July 25th, when a wall of mud and water came down Middle Creek. Someone called him to try to warn him it was coming. He ended up with chest-deep water at his gate. He said Jacks Creek flooded first at the highway, but it is separated from his house by high ground. Then Middle Creek flooded and got his house. He said water went all around the Saguache Creek stream gage, out-of-bank on all sides. His dad's house, about 2 miles downstream on CR CC40 had about 6-inch deep flood waters at their shop.

Another resident, whose property was to the northeast of the storm, at the confluence of Bear Creek and Middle Creek, remembered that there was only moderate to light rain there, no flooding. He said there was a clear north-side dividing line that demarcated the flash flooding, near the Section 22-Section 27 border on Middle Creek. South of that line (i.e., south of Section 22) there was severe flood damage, north of it there was no flooding.

We called a half-dozen other residents, all of whom provided information that helped corroborate and put bounds on the aerial extent, magnitude, and timing of the storm and flooding. These reports, even 20 years after-the-fact, were invaluable, as we attempted to ground-truth the radar data from the previous storm analysis.

STREAM GAGE RATING EXTENSION STUDY AND FLOOD HYDROGRAPH RECONSTRUCTION

In terms of flood runoff from the July 25, 1999 storm, tributary flooding on Jacks Creek and Middle Creek was investigated and documented by Robert Jarrett of the USGS, as noted in Henz (2001). As part of the current study, Dr. Jarrett provided his unpublished analysis to us via personal communication, and he assisted with review of our work. Dr. Jarrett had estimated peak flows from high water lines and channel geometry surveys in the area of the most extreme flood runoff and erosion -- Jacks Creek, Cross Creek, Cabin Draw, and Middle Creek.

Also, the Colorado Division of Water Resources (DWR) operates the Saguache Creek near Saguache (SAGSAGCO) stream gage on Saguache Creek, immediately downstream of the side tributaries of Jacks Creek and Middle Creek where the July 25, 1999 storm and flood occurred. The stream gage is at latitude 38°09'48", longitude 106°17'24" NAD27, elevation 8030 feet, and has a drainage area of 595 square miles[1] The stream gage period of record is July 1910 through present. Gage height is measured at a stilling well with an electronic, automated stage recorder at 15-minute intervals and is converted to discharge using an empirical, measurement-derived discharge rating. Figure 1 shows the location of the SAGSAGCO gage. No previous reanalysis of the July 25, 1999, flood at the stream gage had been done to our knowledge. At the SAGSAGCO stream gage, the peak 15-minute gage height of 5.53 feet occurred at 6:15 pm, local time, on July 25, 1999. The DWR-reported peak discharge of about 1,225 cfs was estimated by extrapolating the measurement-derived discharge rating in use at that time. However, the maximum empirical measurement of 452 cfs was only 37% of the estimated July 25, 1999 peak flow, so the 1,225 cfs extrapolated peak was not considered to be reliable. Therefore, as part of the current study, Colorado Dam Safety and DWR hydrographers conducted a channel survey and rating extension analysis in order to develop a high flow rating extension and reanalyze the July 1999 flood hydrograph.

Staff from DWR hydrography, Colorado Dam Safety, and Colorado State University made a site visit to the SAGSAGCO stream gage on April 15, 2021, in order to perform a channel survey for a HEC-RAS hydraulic model.

[1] https://dwr.state.co.us/Tools/Stations/SAGSAGCO?params=DISCHRG

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First, an assessment was made of the general channel and overbank flow patterns. The channel exhibits severe meandering. At the gage location there is a high left bank terrace. Approximately 200 feet downstream the bank height drops about 4 feet where the channel leaves the left terrace, which heads east, as the present channel makes a sharp bend to the south. The maximum July 25, 1999, flood stage was below the top of the left bank at the gage but would likely have spilled into the left overbank at the terrace drop. The right overbank is about 500 feet wide and relatively low but is obstructed by dense willows. It slopes gently upward to the south until it reaches a steep canyon wall. As the main channel turns and meanders south (right), it cuts into the right overbank. Approximately 500 feet downstream from the gage (measured along the main channel) the right overbank rises steeply to the canyon wall and right overbank flows would be funneled back into the main channel. At approximately 750 feet downstream from the gage (measured along the main channel) flood flows would be in the left overbank and main channel and generally aligned with the valley in a southeasterly flow direction.

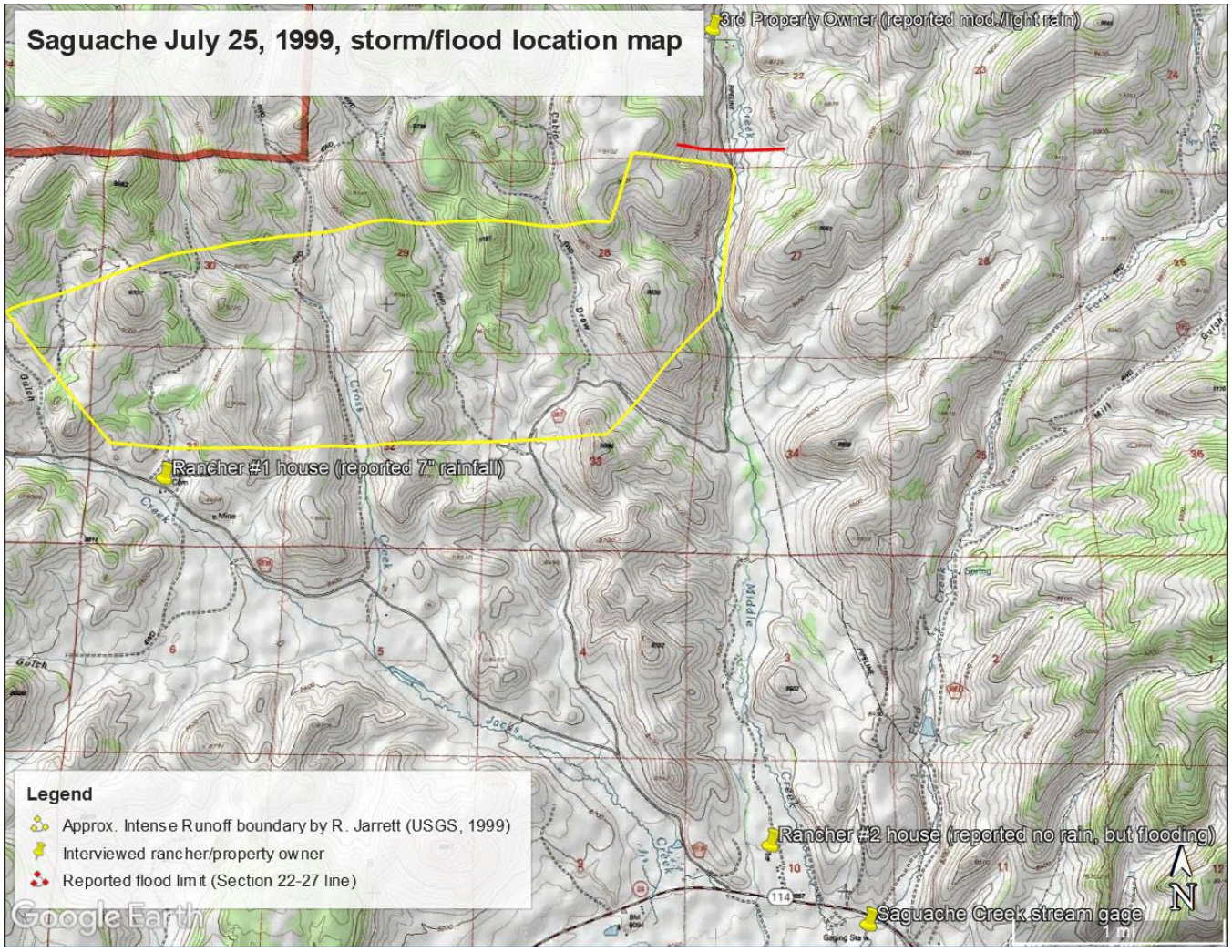


Figure 1: Location of the SAGSAGCO gage.

A 1-dimensional open channel hydraulic model was developed using HEC-RAS 5.0.7, using cross-sections from our channel survey. Downstream distances between sections were assigned by section subdivision (i.e., left overbank, main channel and right overbank). Distances were estimated from the plotted survey sections and using Google Earth. Sections have irregular bends and alignments to match expected flow directions throughout the section subdivisions, meaning they are not parallel. Therefore, downstream distances vary along the sections, but the selected values are intended to approximate subdivision centroid locations at high flows of interest. Manning's n roughness coefficients were assigned using various recommendations in Jarrett (1985). He reports that

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roughness can be correlated to channel slope. As slope increases roughness coefficient increases linearly (Jarrett, 1985, Figure 12). The average channel slope for a mile above and below the SAGSAGCO gage is about 0.004 ft/ft, as determined from USGS quad map topography. Comparing the SAGSAGCO slope to streamflow measurements sites by Jarrett (1985), White River above Coal Creek near Meeker, Yampa River at Steamboat Springs, and Yampa River near Oak Creek have similar slopes at about 0.002 – 0.006 ft/ft. Jarrett (1985) back-calculated Manning's roughness coefficients of 0.034 to 0.047. Further, Jarrett (1985) provides base estimates of Manning's n for natural channels by bed material, with n of 0.02 (Chow) to 0.032 (Benson & Dalrymple) for firm earth. An adjustment factor of 1.3 is to be applied for severe meandering. Finally HEC-RAS recommends Manning's n of 0.04 for natural streams, main channels – clean, winding with some pools and shoals. A main channel Manning's n of 0.04 was selected for all cross-sections for our model.

For overbanks, Jarrett (1985) states that more judgement is involved because there is little data on out-of-bank flood flows and associated overbank roughness. He provides estimates from Chow: 0.025 – 0.035 for short grass pasture, 0.07 to 0.16 for medium to dense overbank brush in summer (Jarrett, 1985, Table 6), but he says that Petryk and Bosmajian reported n values as high as 0.40 in heavily vegetated floodplains. For our model, left overbank Manning's n of 0.08 was used to represent the mix of short grass pasture and clumps of willows; right overbank n of 0.2 was used for the dense willows there. Again, uniform left overbank (0.08), right overbank (0.2) and main channel (0.04) Manning's n values were used for all cross-sections in our RAS model. As the immediate purpose of this study was to reconstruct the July 25, 1999 flood discharge hydrograph, we compared Google Earth satellite images from October 9, 1999, and October 13, 2015 (most recent) and could not visually detect significant differences in overbank vegetation.

Finally, our HEC-RAS model was run allowing mixed supercritical and subcritical flow regimes. A normal depth downstream boundary condition of 0.004 ft/ft friction slope was used, based on USGS topography; critical depth was used for the upstream boundary condition (NOTE: model results do not indicate supercritical flows, which is consistent with findings of Jarrett (1985)).

In our HEC-RAS model, we ran a range of flood profiles from 1 cfs though 7000 cfs. The reported July 25, 1999, peak gage height of 5.54 feet corresponded to 2150 cfs in our model. The DWR chief hydrographer checked our new HEC-RAS discharge rating by combining it with their empirical discharge rating in the Aquarius Rating Development Toolbox and felt the RAS rating extension provided reasonable agreement.

Due to uncertainty in HEC-RAS parameter selection, a sensitivity analysis was performed. First the downstream boundary conditions were varied: normal depth, S=0.025 ft/ft (calculated from our channel survey sections), normal depth S=0.004 ft/ft (base case, from USGS topographic mapping), and critical depth. Results at the stream gage were found to be insensitive to the downstream boundary condition, which is desirable. Also the upstream boundary condition was irrelevant since flow is not supercritical.

Next, Manning's roughness coefficients were varied for the main channel and right overbank. First the main channel n was lowered to 0.02 and the right overbank n was increased to 0.3. This resulted in a discharge rating that significantly exceeds the measurement-based rating at its upper end. Next both the main channel and right overbank n values were increased (to 0.06 and 0.3 respectively), which resulted in a rating curve that underestimates discharge compared to the measurement-based rating. The base case provided the best fit to the measurement-based rating curve at the upper (high flow) end. Low end discharges (<100 cfs) were not particularly relevant to the high flow rating extension.

Next, Colorado Dam Safety used the recorded SAGSAGCO 15-minute stage data from the July 25, 1999, flood and recomputed the flood discharge hydrograph based on our new HEC-RAS discharge rating for discharges above 200 cfs. Below 200 cfs DWR's previous discharge estimated were maintained. The following information was taken from our new reconstructed July 25, 1999, flood hydrograph:

Instantaneous Peak flow:	2,400 cfs (estimated, between recorded 15-min gage heights)
Peak flow from 15-min GH:	2,147 cfs
Time of Peak (15-min GH):	July 25, 1999, 6:15 pm, local time
Assumed baseflow:	141 cfs (last discharge before rising limb of flood)
Runoff volume:	380 ac-ft, July 25 4pm – July 26 7:15am

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The new peak flow estimate (2,400 cfs) is larger than DWR's previous reported peak (1,225 cfs). It is also larger than the 0.2% AEP flood frequency estimate of 1,700 cfs by the USGS.

As a last sensitivity analysis, we lowered Manning's n values in our HEC-RAS model as much as was considered reasonable in order to see the effect on runoff volume, since it was obviously much lower than reported rainfall volumes: 0.04 for left overbank, 0.01 for main channel, and 0.1 for right overbank. A new rating was developed, which yielded a 15-minute peak flow of 2,339 cfs and a runoff volume of 581 acre-feet. Further, beyond a reasonable parameter range, Manning's n of 0.013 (equivalent to troweled concrete) for the left overbank, main channel, and right overbank yielded a peak flow of 9,500 cfs and runoff volume of 1,399 acre-feet. Clearly our stream gage hydrograph reanalysis showed that gaged runoff was a small fraction of previously reported rainfall volumes for the July 25, 1999, Saguache event.

RAINFALL-RUNOFF MODEL

Concurrently Colorado State University was doing a rainfall-runoff modeling research study for Colorado Dam Safety in order to develop better methods to estimate extreme floods in ungaged mountain basins. Historically extreme storms and floods in Colorado's mountains above 7,500 feet elevation had been significantly over-estimated in comparison with the observed flood and paleo-flood record (Jarrett, 1989). The objective of this research was to develop and test parameter estimation methods for soil moisture accounting-based HEC-HMS rainfall-runoff models to accurately simulated extreme floods in Colorado's mountains.

CSU's HEC-HMS SMA parameter estimates methods used physically-based basin characteristics to the extent possible, so that they can be used on ungaged basins for Colorado Dam Safety hydrology analyses. The parameter methods were developed by CSU using calibrated reconstructions of historical extreme floods on gaged basins in Colorado's mountains. In the first phase of study, six historical extreme floods in Colorado's Front Range mountains were reconstructed (Woolridge et al, 2020). Then in the second phase, six more extreme floods that occurred in southwestern Colorado's San Juan mountain range (Irvin et al, 2022) were reconstructed. The Saguache Creek July 25, 1999 flood was one of the 12 historical extreme floods used by CSU for model calibration. Using the original REPS storm analysis as precipitation input to their HEC-HMS rainfall-runoff model, CSU found that their Saguache Creek rainfall-runoff model significantly overestimated the observed flood volume compared to the DWR stream gage.

The discrepancies of rainfall volumes and aerial extents, stream gage volumes, and rainfall-runoff modeling volumes, suggested a need to reanalyze the Saguache July 25, 1999 rainfall, using the better information that we had obtained through our bucket survey, channel study, and rainfall-runoff modeling. More information, when it is available, can almost always lead to more accurate storm reconstructions. Here, there was a likely source of rainfall error in that the storm was originally reconstructed from first generation radar, from Pueblo, Colorado, located nearly 100 miles away. With possible errors by radar scatter, hail contamination, and interference from intervening mountains, Applied Weather Associates got to work reanalyzing the storm.

SPAS REANALYSIS OF SAGUACHE, CO, JULY 24-27, 1999 STORM

AWA was tasked to re-investigative the Saguache Colorado storm of July 24-27, 1999 (SPAS 1662). Interviews collected on the extreme storm timing, magnitude, duration, and area was collected by Mark Perry of Colorado Dam Safety (2021-10-06 Saguache eyewitness storm docuementation_MPERRY.pdf). This additional information and bucket survey type of analysis provided new insight to the storm location, information of the storm spatial patterns, and confirmation of the storm magnitude.

SPAS 1662 REANALYSIS

AWA used information collected from the eyewitness interviews and discussions provided by Mark Perry, information from the HDR HMS-Hydro report (June 2001), and data from Dr. Bob Jarrett's storm report along with the previously analyzed SPAS 1662 storm data (rain gauge data, radar data, basemap climatologies). The analysis was based on 47 hourly stations, daily data, and supplemental station data and NEXRAD Radar. Several additional data points (Table 1) were used to aid in the SPAS 1662 reanalysis.

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Table 1: Additional data points investigate to constrain the precipitation pattern.

n	Notes	utm_X	utm_Y	Lat	Lon	Ppt (in)
1	Flood boundary on Middle Creek (no flood north of line, severe flash flood south)	386171	4231121	38.2209	-106.3	
2	Ken Williams' house on Middle Creek – moderate rainfall, not severe	385912	4232085	38.2295	-106.304	
3	Corey Hill house – no rain	386194	4225339	38.1688	-106.299	0.0
4	Elder Hill property (Corey's father) in Saguache Creek floodplain, 6" flood depth in shop	389701	4221131	38.1313	-106.259	
5	Sutherland Ranch house on Jacks Creek – gaged 7-inches rain in 1-1/2 hours	381392	4228278	38.1946	-106.355	7.0
6	Robert Jarrett intense runoff boundary -- approximate					
7	Best guess area of storm center by Mark Perry based on phone interviews – approximate					

Specific notes on the SPAS 1662 reanalysis are provided below.

Notes on this re-run

1. The radar data (raw reflectivity) placed the storm center/event just to the west of the Bob Jarret and eyewitness accounts like the initial SPAS analysis with a slightly larger footprint (Bob Jarrett report, Mark Perry's notes). The location and area-size/footprint of the storm center is likely an issue with radar over shooting and geo-referencing for precipitation at high elevations.

2. Based on radar limitations, AWA focused the re-analysis on a non-radar basemap SPAS approach. A basemap climatology was developed based on Bob Jarrett's isohyets, Mark Perry's notes, and shifted radar reflectivity (shifted radar reflectivity grids to location of observed 7-inch location and Bob Jarrett 7-inch contour).

3. We also modified the native resolution from 0.01 decimal degrees (~ 1/3rd-mi^2) to 0.005 decimal degrees to aid in interpolating the tight precipitation gradient.

SPAS 1662 REANALYSIS RESULTS

The reanalysis resulted in an increase in small-area precipitation (<1-mi^2; 0.91-inches) which is attributed to the finer resolution (0.005 vs 0.01), the created basemap was based on shifted radar data, Bob Jarrett isohyets, and Mark Perry's eyewitness field notes. The finer resolution, spatial location of observed values, geo-referenced isohyets, and reflectivity generated a gradient/increase in precipitation from observed locations for a higher storm center value (Figure 2 and Figure 3).

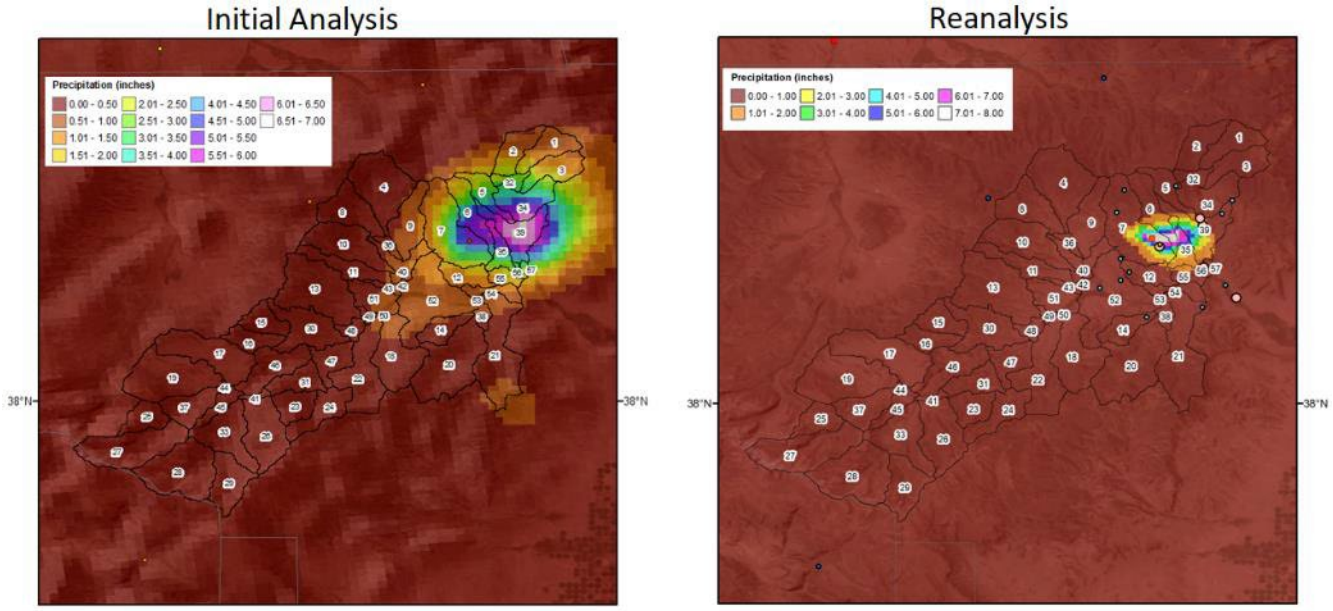


Figure 2: Total storm for July 25- 26, 1999: a) initial analysis and b) updated reanalysis. Black sub-basin outlines represent the basins used as part of the hydrologic model analysis for the storm event.

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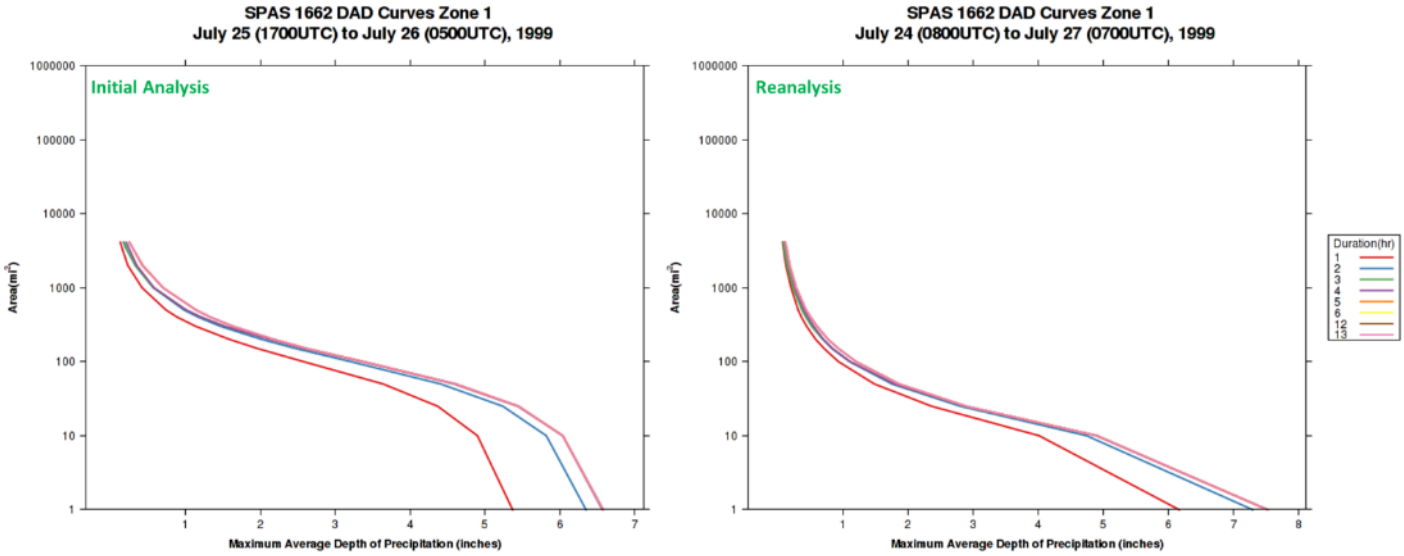


Figure 3: Results of the July 25- 26, 1999 storm depth-area-duration: a) initial analysis and b) updated reanalysis.

For the CO-NM REPS study, the PMP tool was updated to include the updated depth-area-duration (DAD) results for the SPAS 1662 reanalysis. The updated analysis provides a more refined SPAS analysis that had better overall goodness-of-fit measures when compared to the hydrologic model analysis.

GIS PMP TOOL UPDATE

The current release of the CO-NM REPS GIS PMP Evaluation Tool (version 2.0) reflects the Saguache, CO 1999 storm SPAS 1662 reanalysis. The update includes the reanalyzed DAD table and updated storm adjustment factors using the revised storm center location. In addition, several edits and improvements to the script and GIS tool are implemented in the current version.

STORM DATABASE UPDATES

The Saguache, CO, 1999 storm (SPAS 1662) is the only event updated in the current release. Both the DAD table and the Storm Adjustment Factor feature class elements of the storm database are updated for the SPAS 1662 event.

The revised DAD table provides depth at all durations, reflecting the SPAS reanalysis. The largest reduction, ~2.8” occurs at the 50 square-mile area, with lesser reductions occurring with increasing area-size. The reanalyzed depths are only larger, approximately 1” larger, at very small areas (~1-mi^2) due to the increased spatial resolution of the SPAS reanalysis.

The reanalyzed SPAS 1266 storm center is located at 38.2025° N, 106.3575° W, 3.5 miles west of the previously analyzed storm center location. The in-place maximization factor (IPMF) for the reanalyzed storm center is 1.203. This is a slight reduction compared to the previously analyzed IPMF of 1.207 due to the somewhat lower storm center elevation.

The updated storm center location also impacts the geographic transposition factor (GTF) for SPAS 1266 based on the 100-year 6-hour climatological rainfall determined at the storm center. The 100-year 6-hour rainfall depth at the updated storm center location is 2.15”. The value at the previous storm center is 2.28”. SPAS 1662 is transposed to zones 5, 6, and 17, covering the Rocky Mountains east of the Continental Divide. The lower 100-year rainfall at the new storm center results in an average GTF increase of 0.06 over the three target zones.

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The SPAS 1662 event is classified as a local storm in the PMP tool database, therefore only local storm PMP over transposition zones 5, 6, and 17 has the potential to change with the version 2.0 update. The updated DAD table, the updated adjustment factors, and how the adjusted rainfall depths compare to the other contributing storm events combine to determine if the local storm PMP changes from the previous version 1.10 depths (CO-NM REPS 2018). In general, there is little to no change over zone 5, except for the 1-hour duration for very small basins (<10-square miles) where PMP is ~20% higher. For zones 6 and 17, PMP is also ~20% higher for basins less than the 10-square mile size over zones 6 and 17 for all durations. For larger basins over zones 6 and 17, PMP decreases from previous versions as SPAS 1662 is no longer the controlling event, with the largest decrease of ~30% for basins around 50-square miles. Appendix A of this report provides local storm PMP maps over the REPS domain as an update to the 2018 Volume II Appendix A maps. Appendix B provides percent change from PMP version 1.10 (2018) over the relevant transposition zones 5, 6, and 17.

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AUTHOR BIOGRAPHIES

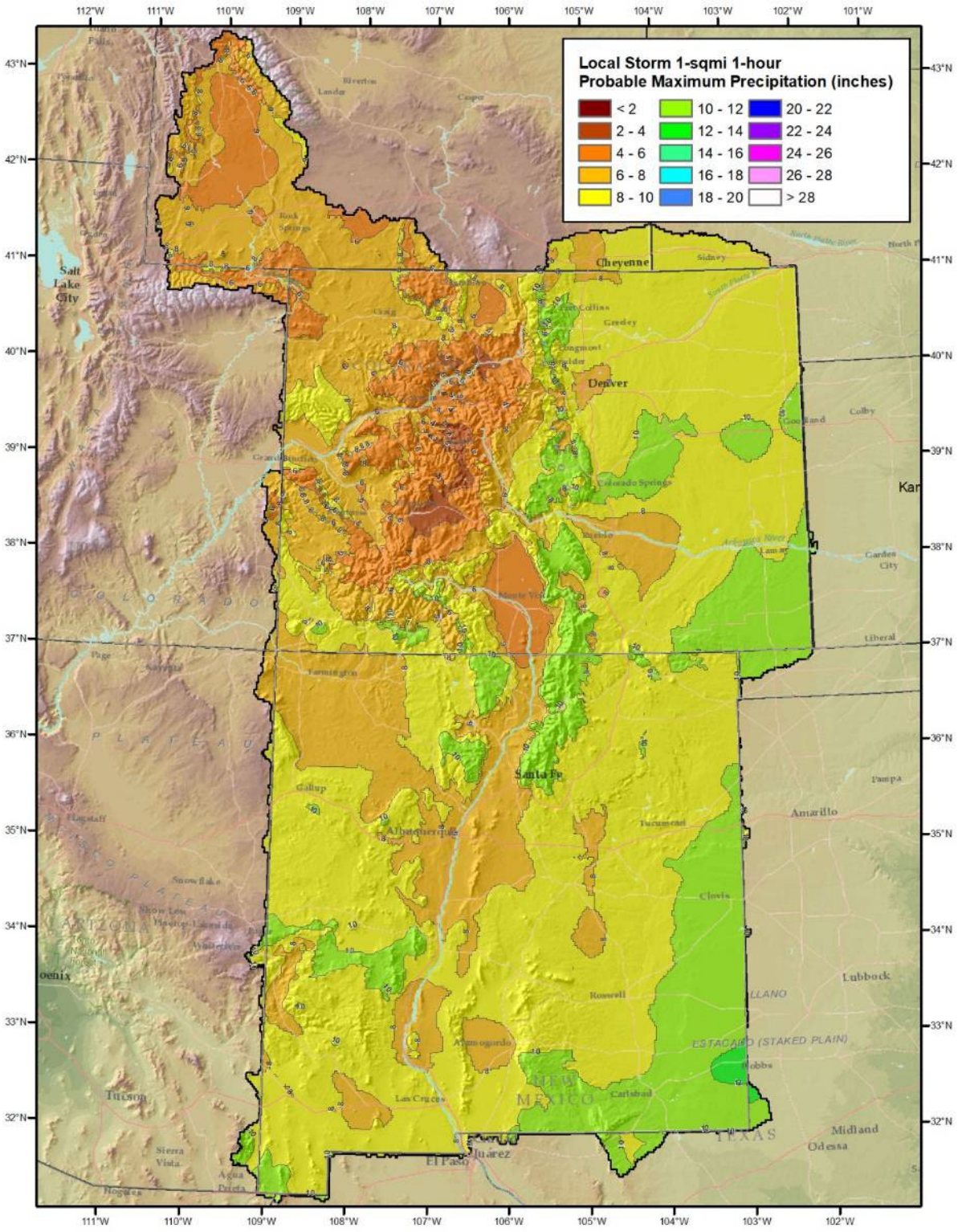
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Mr. Kappel is President and Chief Meteorologist of Applied Weather Associates (AWA). Mr. Kappel has been the project manager/technical lead for all aspects of AWA work ranging from PMP development to climate change analyses to risk quantification since joining AWA in 2003. Prior to joining AWA, he served as an on-air meteorologist at various television stations across the country. Mr. Kappel has been a guest instructor at the University of Colorado in Colorado Springs, guest lecturer for the Tailings Center and Mine Water Waste and Closure Course, has published numerous peer reviewed articles on extreme rainfall, and has provided presentations at numerous conferences on various meteorological topics.

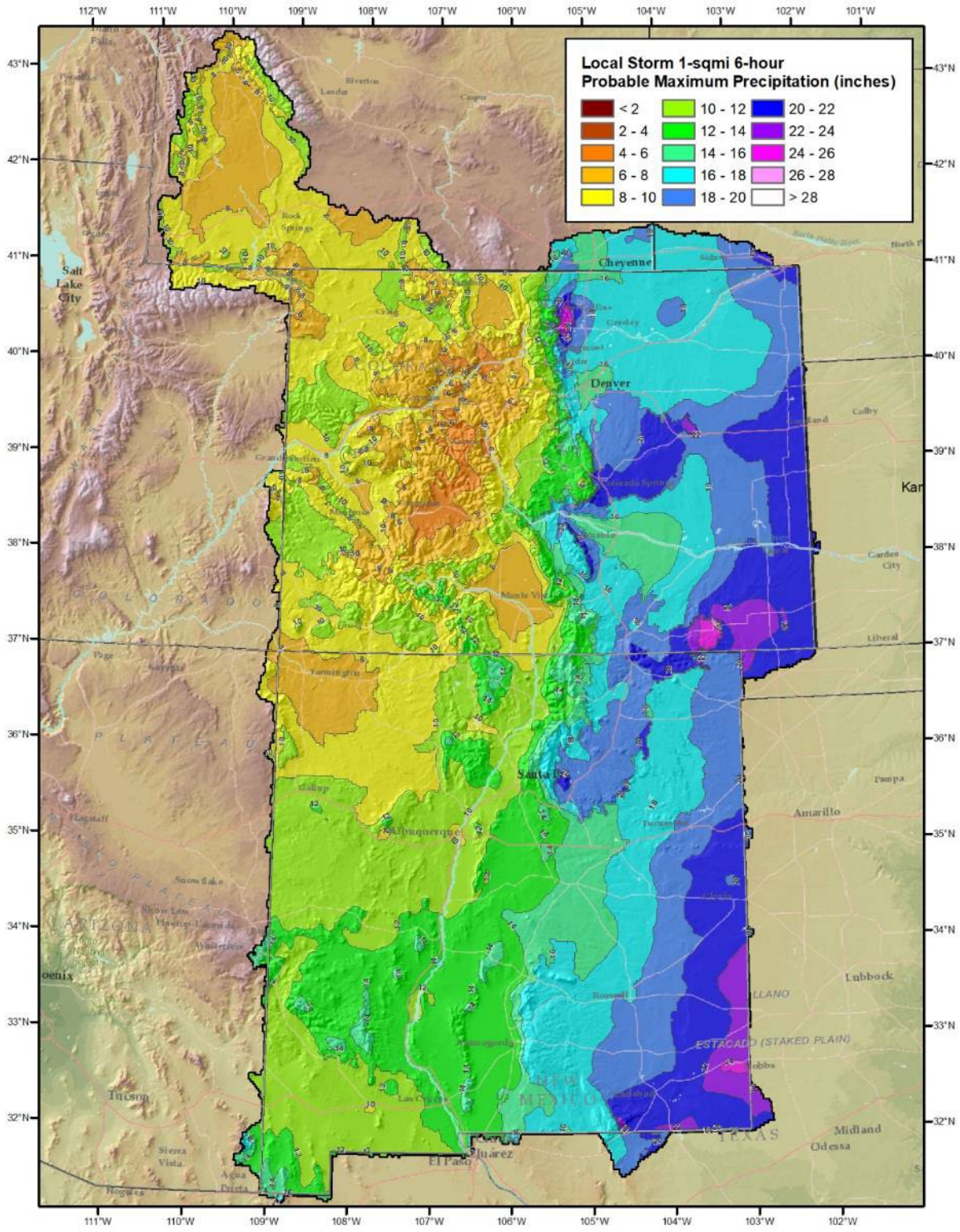
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Appendix A LOCAL STORM PMP MAPS

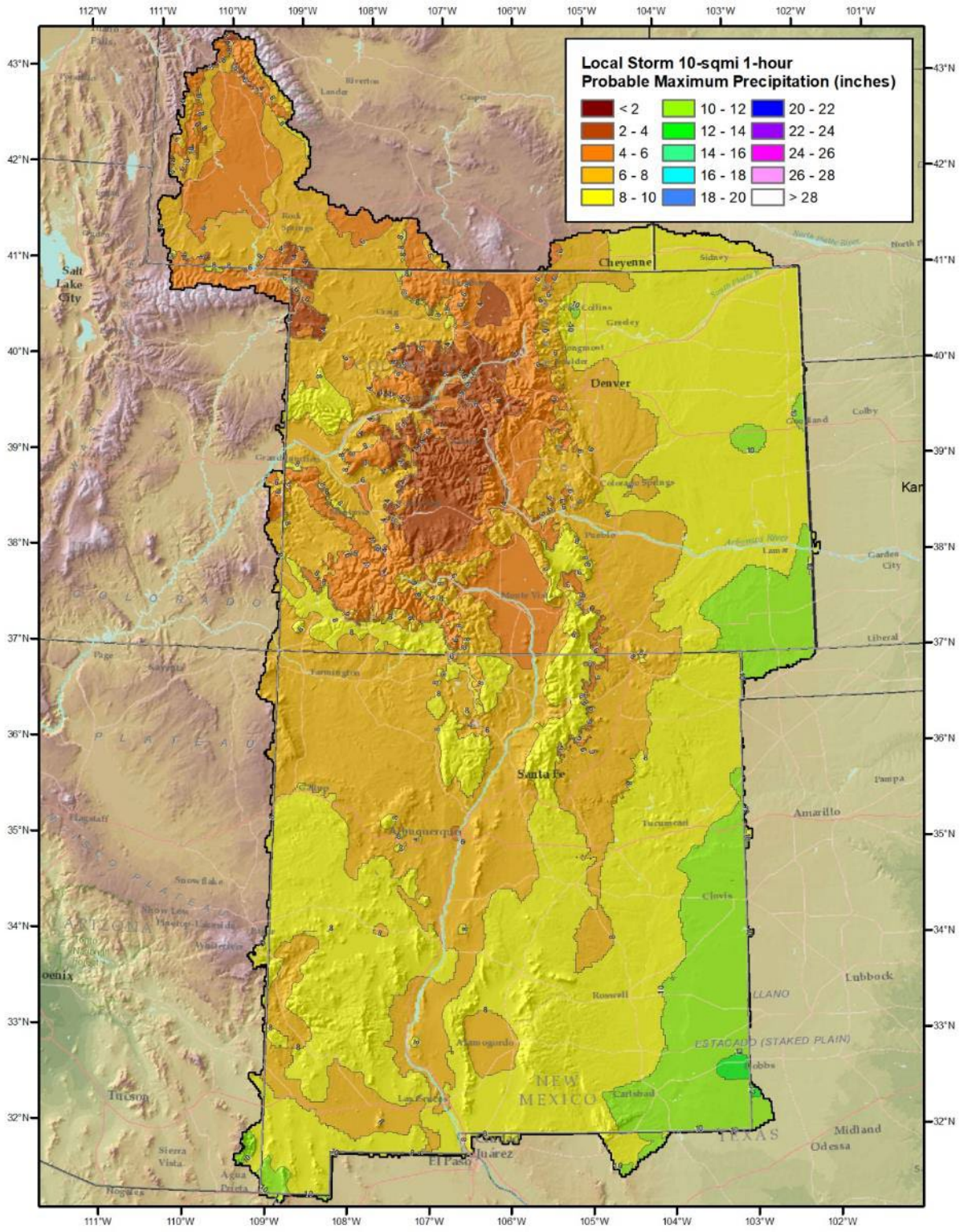
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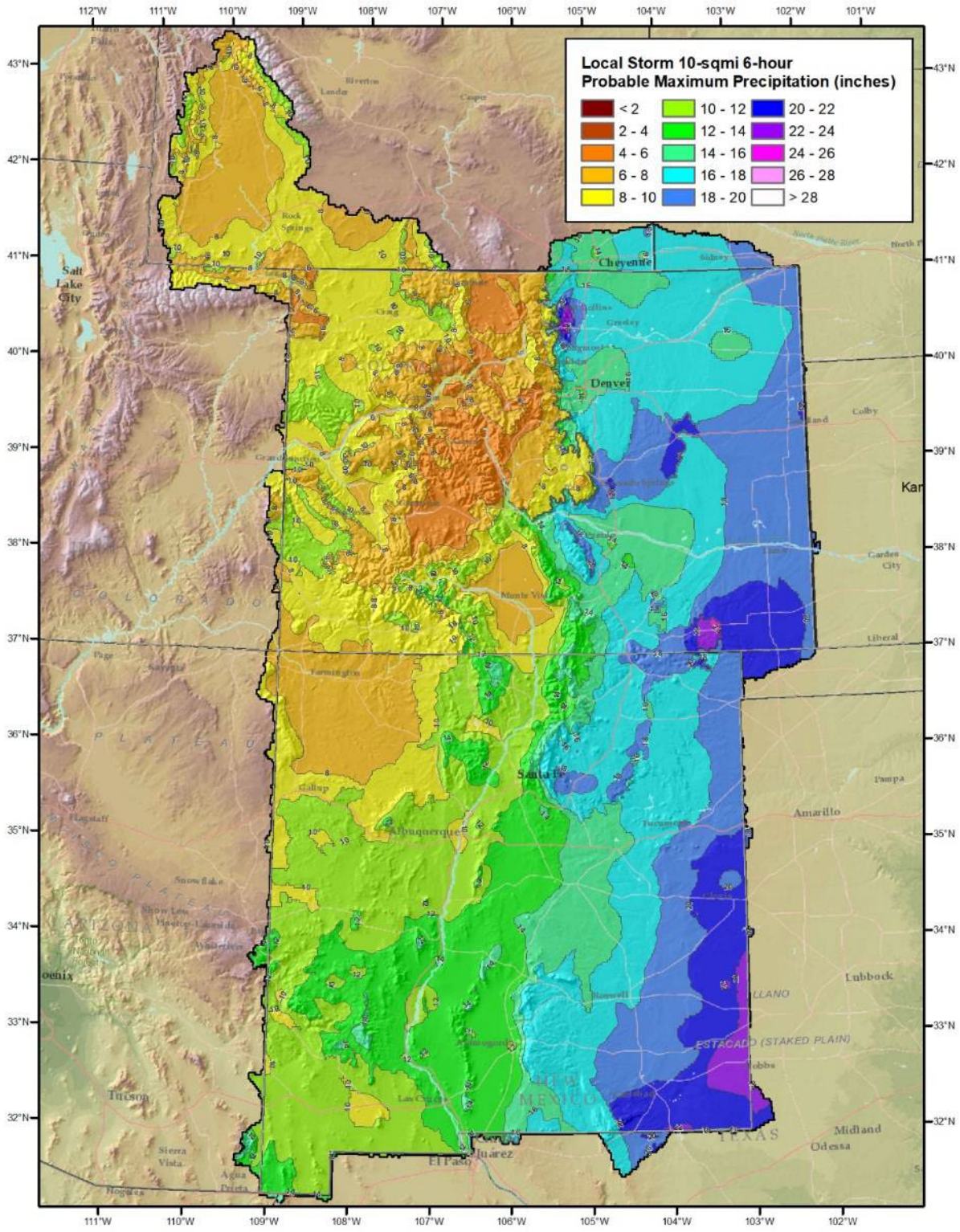
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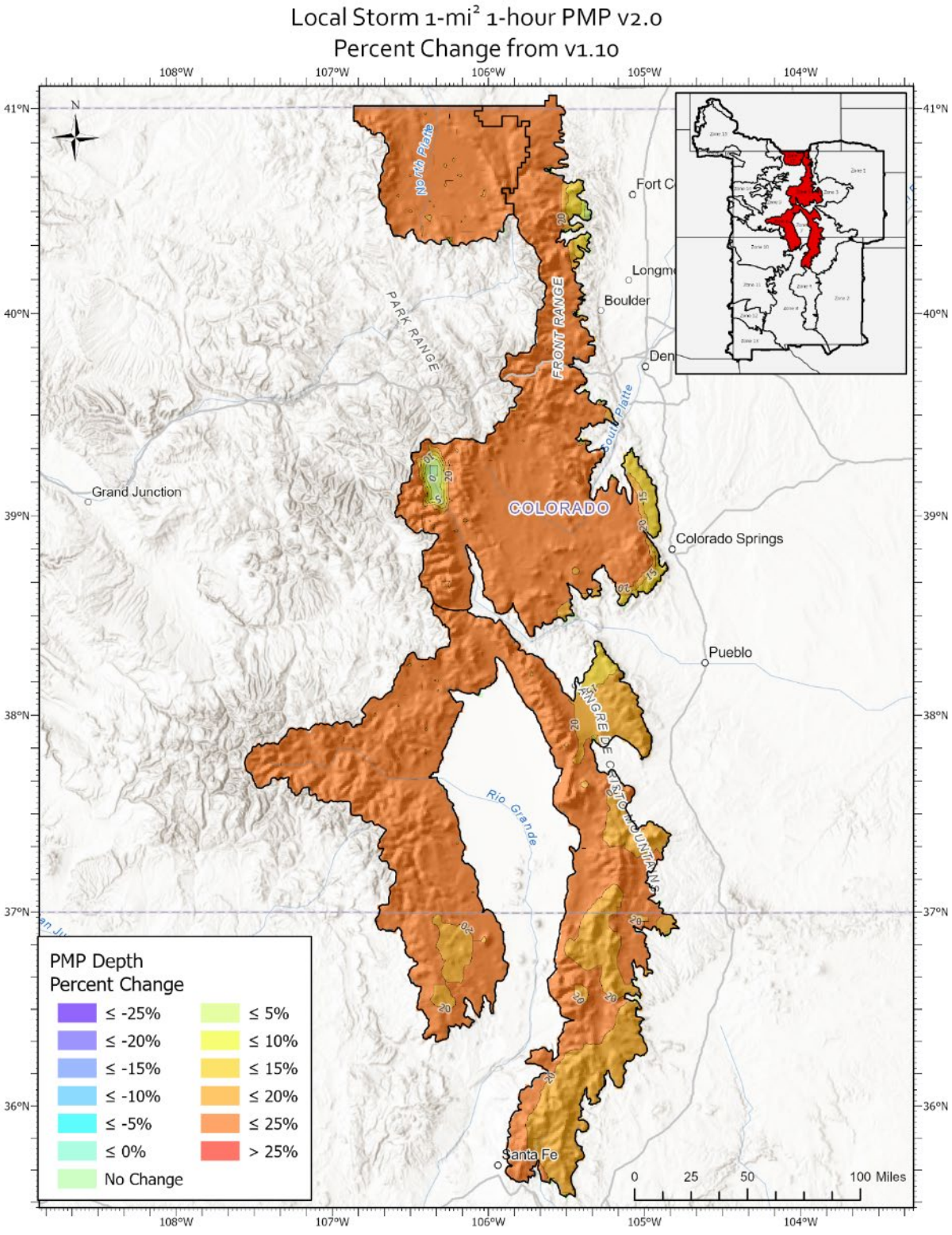
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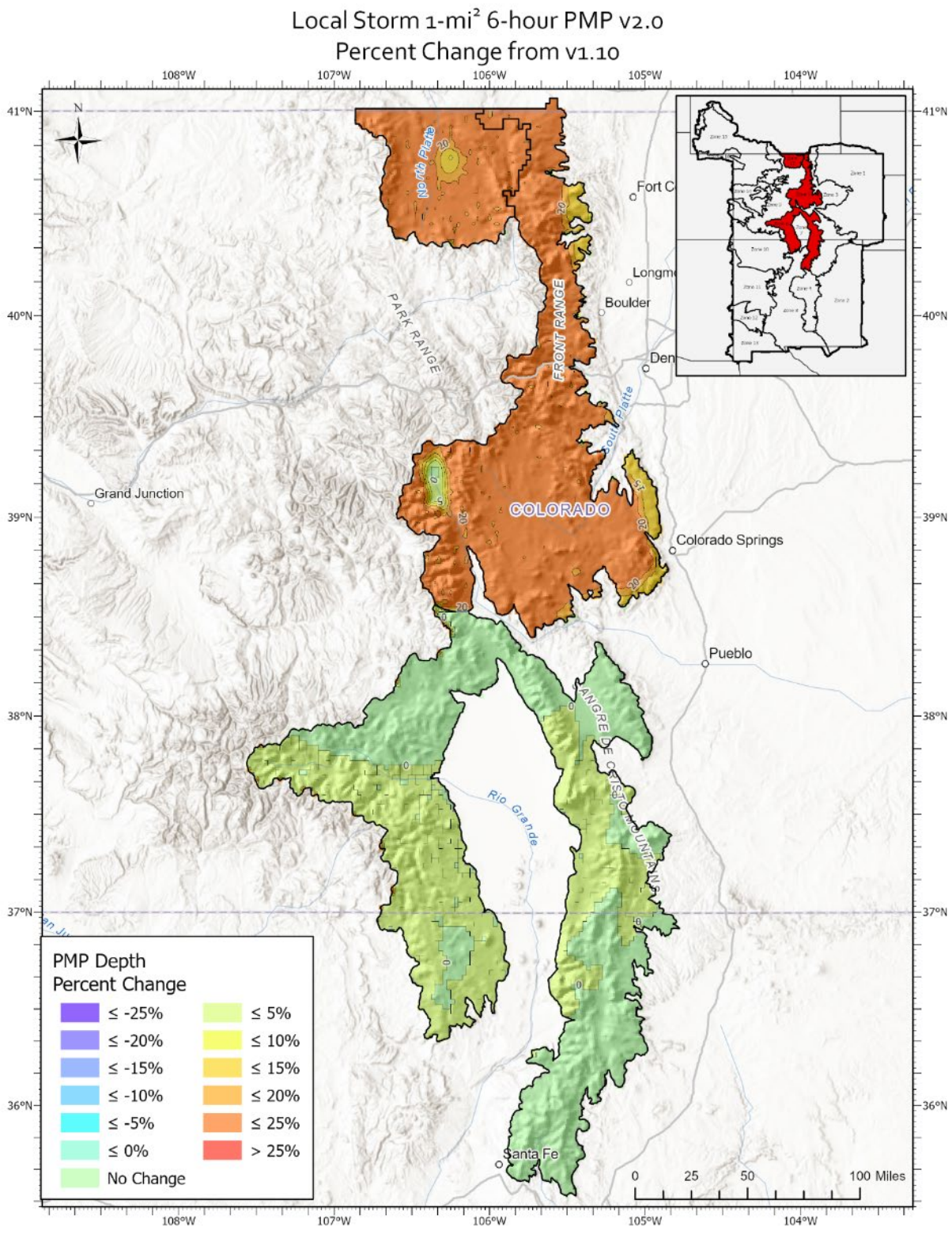
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Appendix B PMP Percent Change Maps

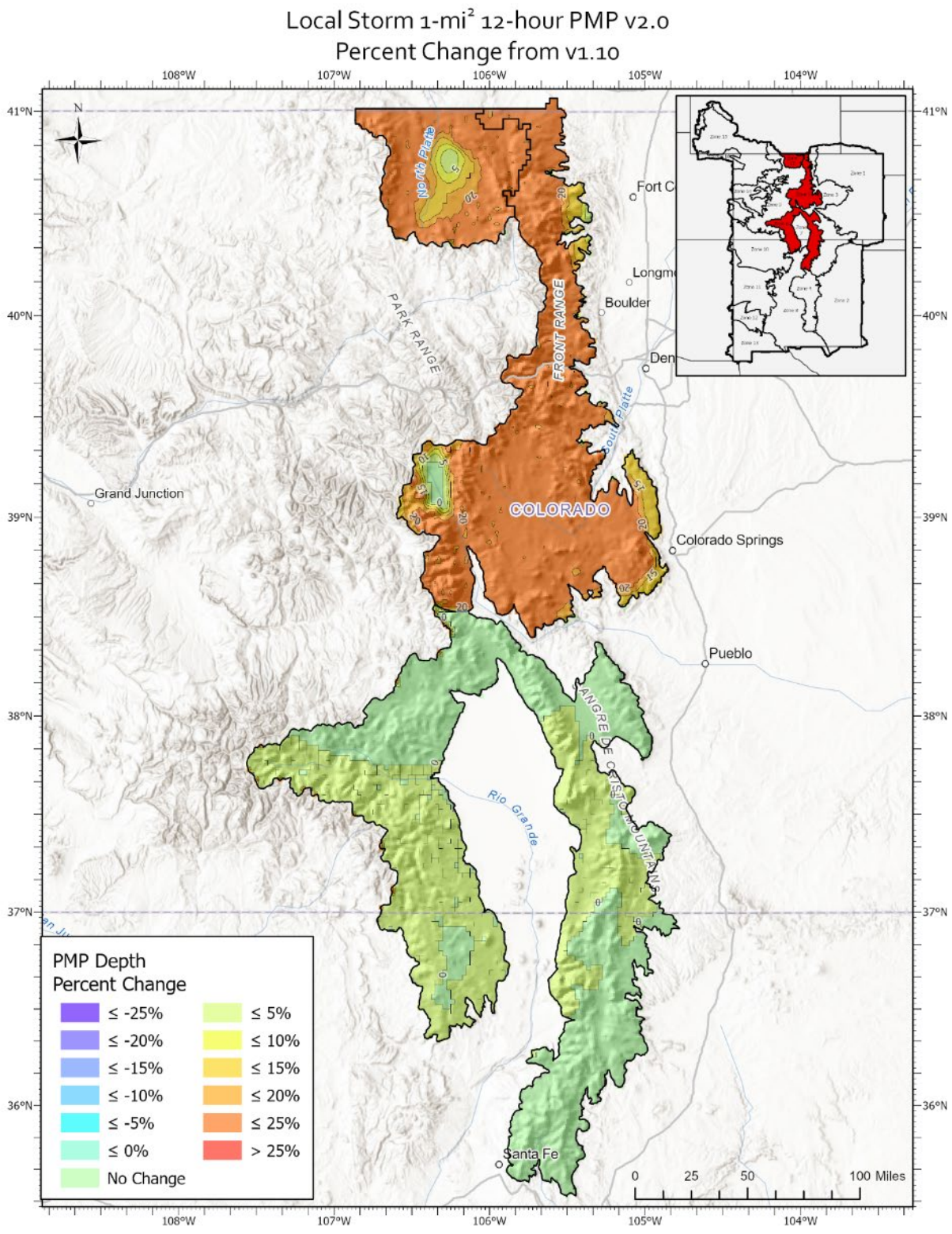
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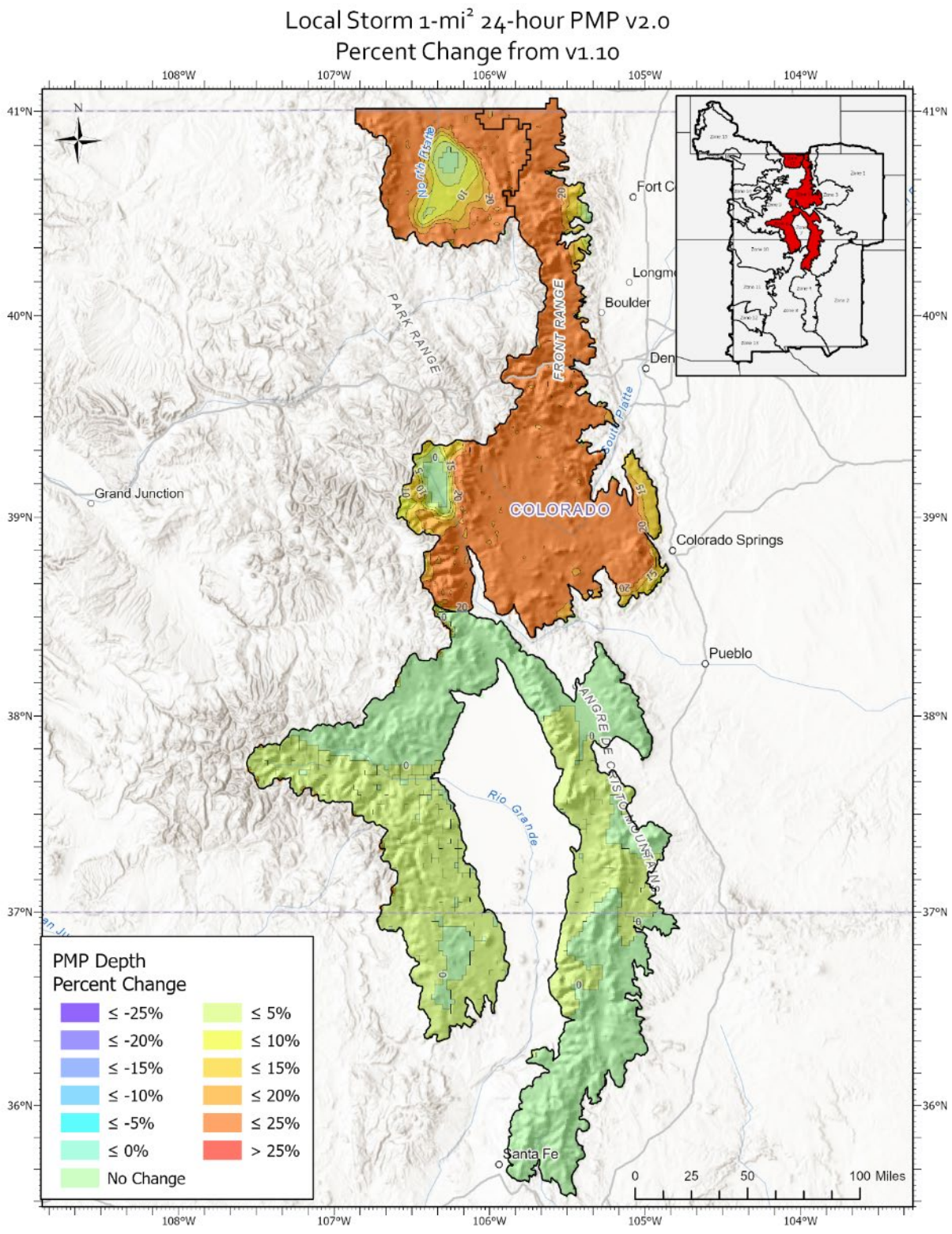
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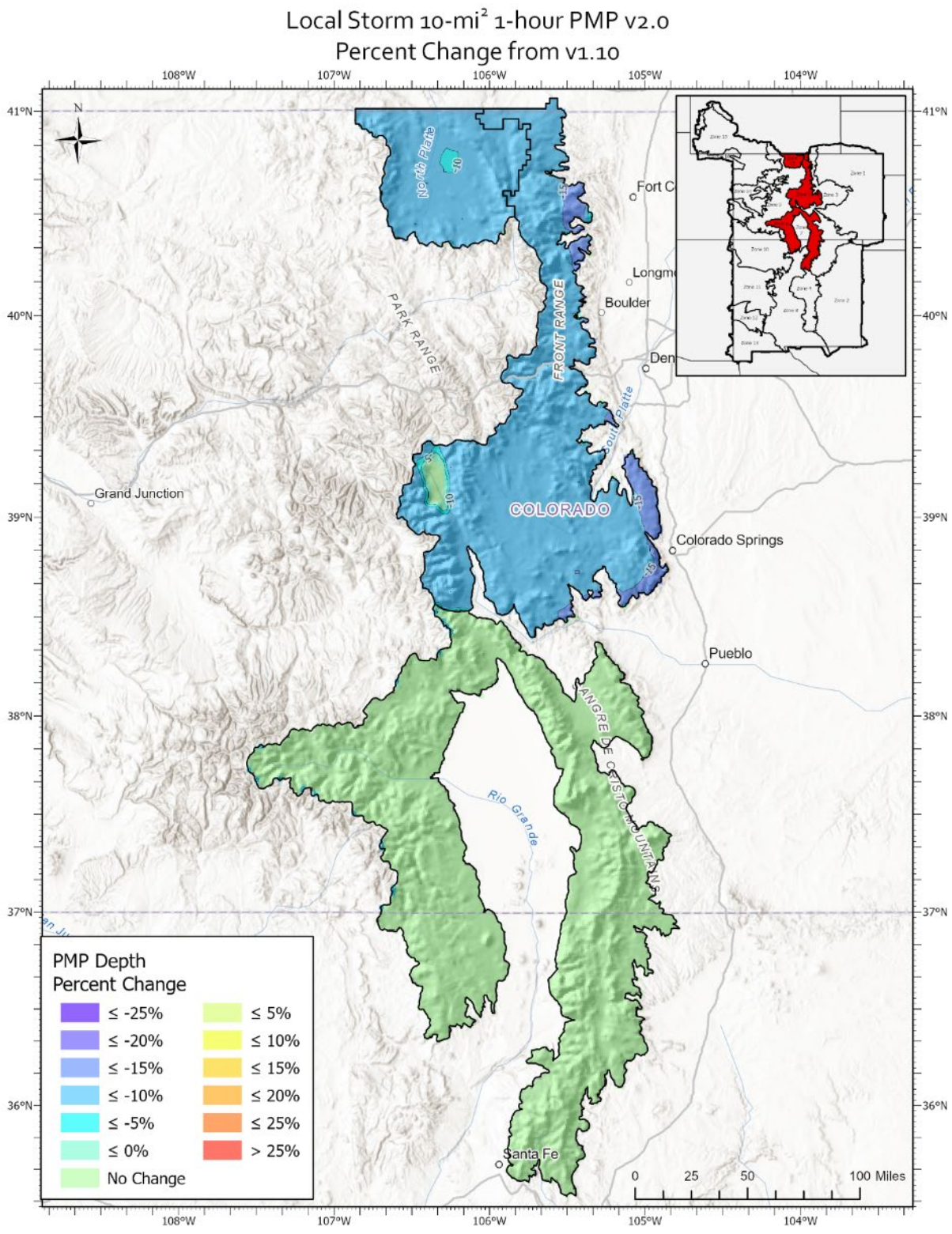
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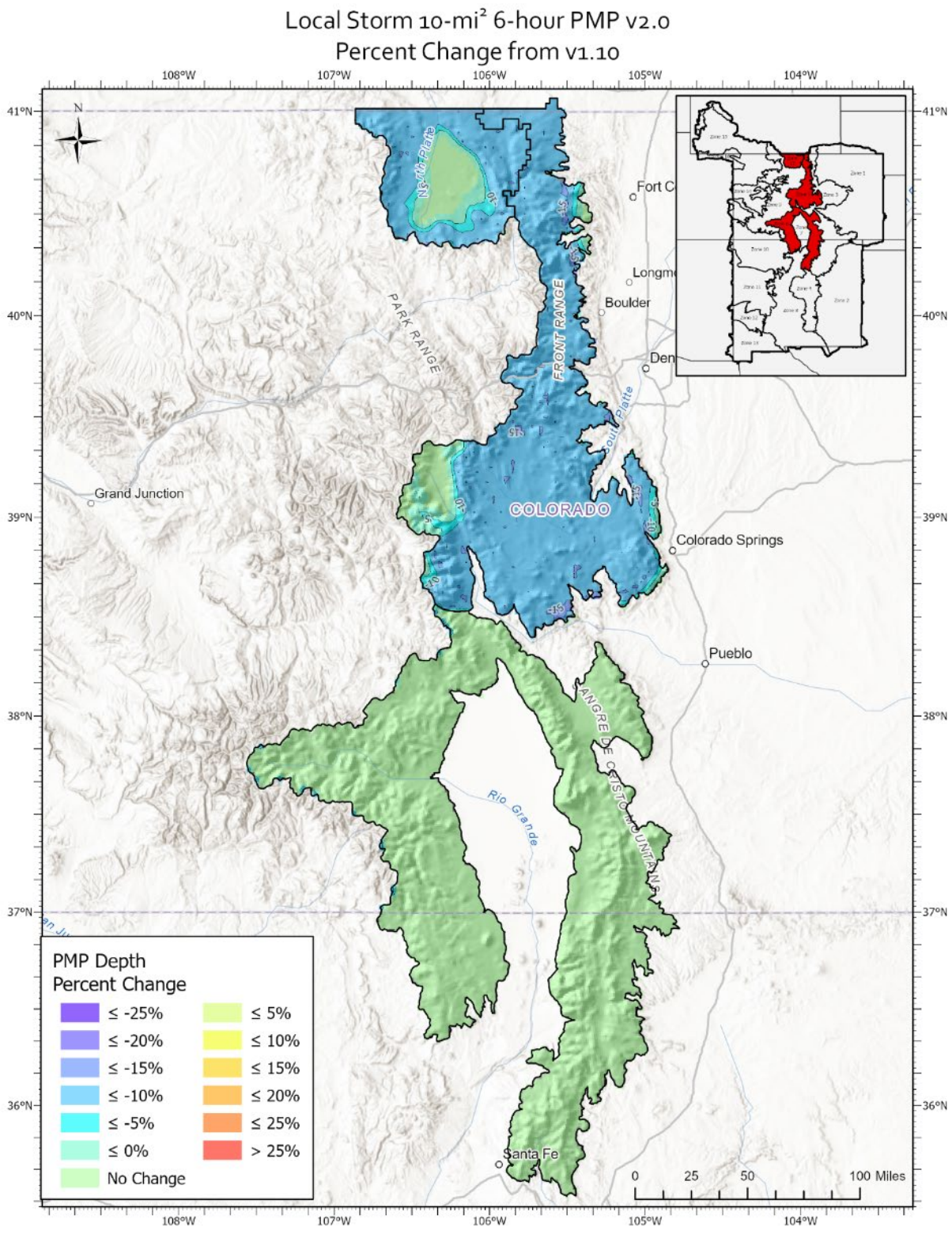
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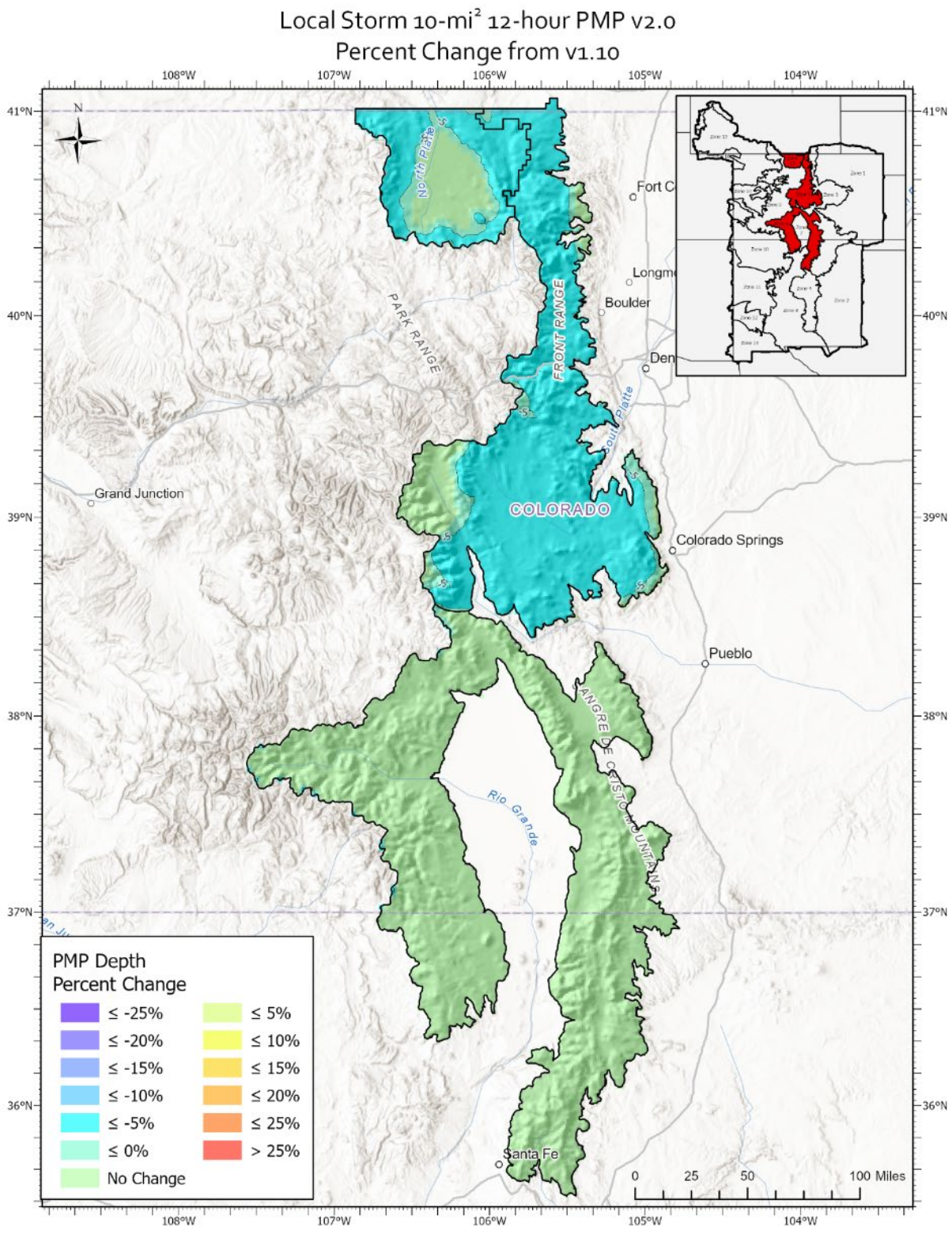
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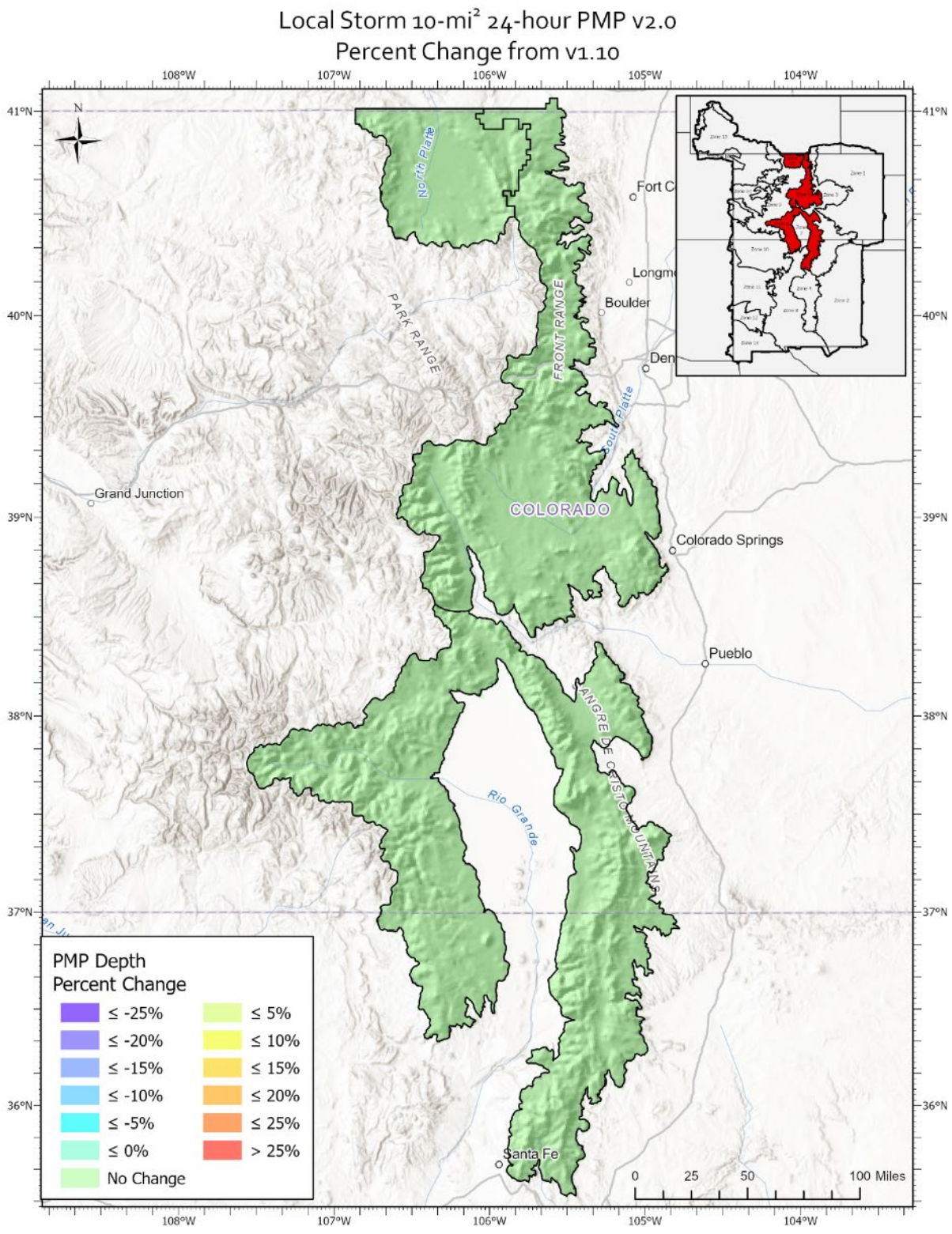
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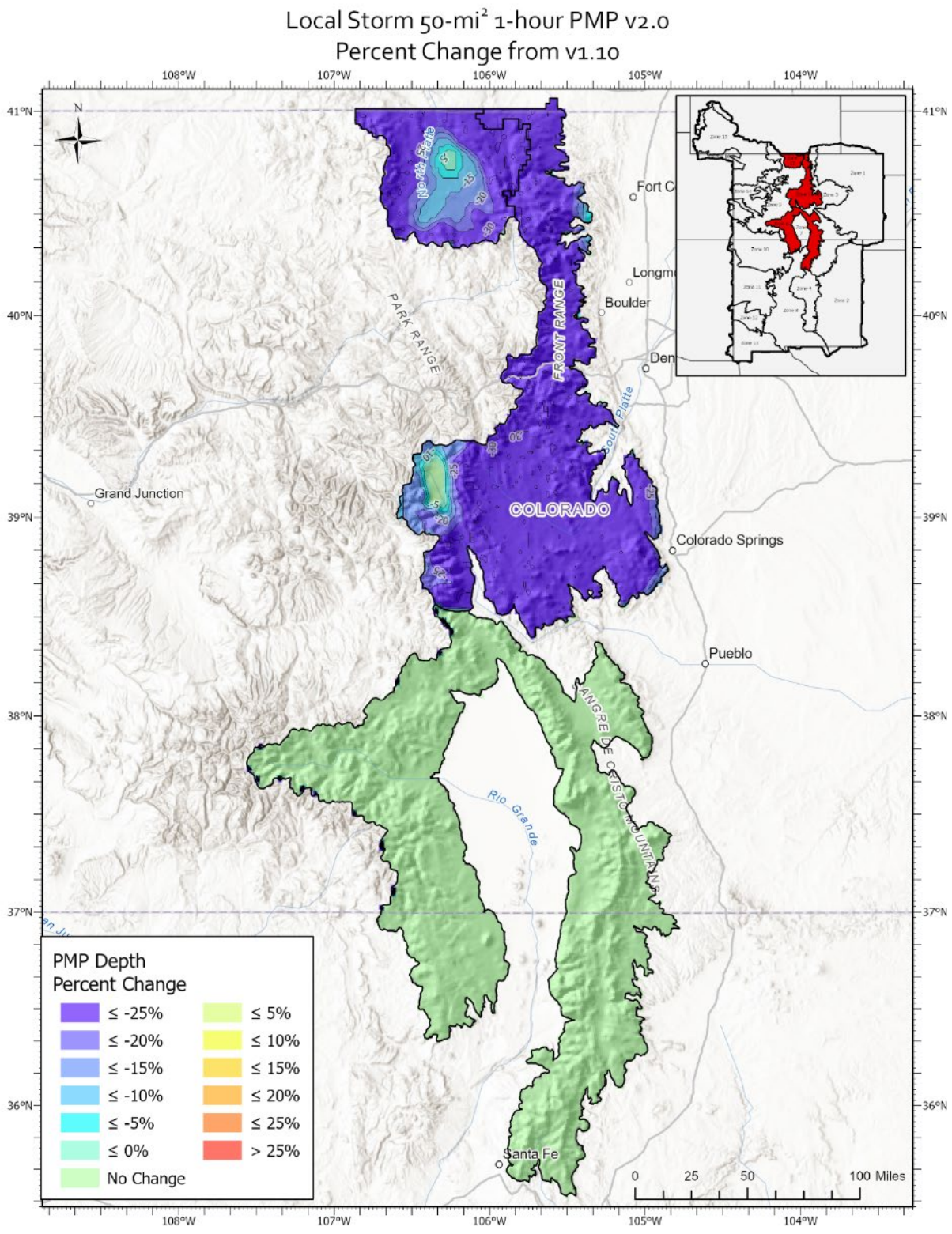
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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

Lessons Learned - Artesian Head at Earth Dams in Glacial Country

DONALD R. GREEN, P.E., MICHAEL BAKER INTERNATIONAL, INC.

ABSTRACT

Management of seepage to mitigate artesian head is a critical factor to assess and maintain the stability of earthen embankment dams. This is particularly important at dams that are constructed on glacial soils. Too often there is a fundamental lack of a thorough understanding of the glacial geology and a lack of vision about the possible variation of subsurface conditions from that depicted in a limited number of test borings, and how that variance could require a shift in design approach to contend with the presence of artesian head. A discussion of practical Lessons Learned will be presented that are based on several past dam rehabilitation projects that were completed. The discussion will focus on both what was done during the initial site characterization, followed by the development of mitigation details and design strategy, and measures taken to adapt implementation based on actual conditions encountered during construction. One of the example projects will highlight lessons learned to address dam rehabilitation at an earthen embankment where an artesian head was encountered that rose above the dam crest elevation at a test boring that was drilled and sampled near one of the dam abutments.The goal of the presentation is to equip the audience with Lessons Learned to address situations where an artesian head is encountered to rehabilitate earthen dam embankments.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

South Fork Dam: Understanding and Mitigating Complex Geologic and Seepage Issues

AUSTIN MARTIN, P.E., NV DIVISION OF WATER RESOURCES; E.J. BARBEN, P.E., AND CARI BEENENGA, P.E., GANNETT FLEMING, INC.

ABSTRACT

This paper presents a case study documenting the lessons learned while exploring alternatives to address seepage issues at South Fork Reservoir, the only dam currently owned by the State of Nevada Division of Water Resources. South Fork Dam is an 85-foot high, zoned earth embankment dam situated on an extremely complex geological formation. The site was once considered for development by the U.S. Army Corps of Engineers, however no work supported by the Corps was ever completed. The reason the site was passed over by the Corps is unknown, but in the mid-1980s, the Elko County Recreation board experienced firsthand how complex geological and seepage issues can complicate dam construction. The intersection of three, variably weathered geologic formations beneath the dam create conditions that require multiple means of seepage mitigation, including a multi-line grout curtain, multiple foundation aggregate drains, and relief wells. Additionally, a very flat floodplain prohibits the conveyance of collected seepage downstream from the dam. When ownership of the dam was transitioned to the State of Nevada shortly after completion of construction, establishing a new state park, much of the information documenting the subsurface conditions was lost. Despite the existing seepage mitigation systems, the State has battled a clogged toe drain, relief well inefficiency, and occurrences of uncontrolled, unfiltered seepage. This presentation will explain the importance of understanding a dam's seepage regime and mitigation systems. The method used to evaluate the seepage regime and develop supplemental mitigation will be discussed. The effectiveness of this approach will be illustrated by the South Fork Dam case study. Lessons learned from this project will be shared so that other engineers that encounter similar conditions may benefit.

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DAM SAFETY PALM SPRINGS
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Grindstone Canyon Dam Seepage Gallery Drain Cleaning

GRESHAM D. ECKRICH, PE, PG, CEG, SENIOR GEOLOGICAL ENGINEER, YEH AND ASSOCIATES

ABSTRACT

Grindstone Canyon Dam is a 123-foot-high gravity roller-compacted concrete (RCC) structure in the Village of Ruidoso, New Mexico. Grindstone Reservoir is impounded by the dam and stores up to 1,635 acre-feet of water. Yeh and Associates provided engineering services for the Grindstone Canyon Dam Drain Cleaning project that was completed in May 2022. The primary purpose of the project was to restore the functionality of the dam's internal seepage gallery drains and mitigate the potential for excess uplift pressure that could destabilize the dam.

Lessons from the drain cleaning project that may apply to future dam projects include construction sequencing relative to grouting and the drilling of seepage relief drains, the need to case seepage relief drains, and the potential for hydraulic connection (via rock discontinuities) between grout holes and seepage relief drains.

I. INTRODUCTION

Grindstone Canyon Dam is a 123-foot-high gravity roller-compacted concrete (RCC) structure in the Village of Ruidoso, New Mexico. The dam impounds Grindstone Reservoir, located on the western side of the dam. The reservoir holds up to 1,635 acre-feet of water and collects runoff from the surrounding approximately 742-acre watershed and diverted water from the Rio Ruidoso. The reservoir is an off-stream storage site for the Village of Ruidoso's (Village) drinking water system. The dam was constructed with approximately 120,000 cubic yards of RCC between September 1985 and July 1986 and approved for impoundment by the New Mexico Office of the State Engineer (NMOSE) in early 1988. Grindstone Canyon Dam was the fifth major water retention structure in the US to be completed using RCC. The total construction cost was approximately $7.5 million, funded entirely by the Village.

The dam is approximately 1,304 feet long and is oriented close to north/south. The dam crest elevation is approximately 6,923 feet and the maximum dam height is approximately 115 feet. The dam has a maximum base width of approximately 102 feet and crest width of approximately 12 feet. The upstream slope of the dam is vertical. The inclination of the downstream face is approximately 0.75h:1v. The principal spillway is a 100-foot long ogee weir that spills to a 100-foot wide by 30-foot long by 3-foot deep concrete lined stilling basin located at the downstream toe of the dam. The stilling basin continues downstream of the basin as a 100-foot-wide rectangular channel with vertical concrete walls. Grindstone Canyon Dam is classified as a high-hazard dam because failure or mis-operation of the dam would probably cause loss of human life (NMOSE 2010).

NMOSE (2016, 2018, 2019) Inspection Memoranda rated the Grindstone Canyon Dam in "fair" condition due, in part, to maintenance issues related to the condition of the drainage gallery drains at the base of the dam, noting that many of the gallery drains were clogged due to mineralization (e.g., calcification). The memoranda stated that the cleaning of the drains must be performed regularly so that the drains are able to alleviate excess hydrostatic pressure on the dam and ensure safe operation of the dam.

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II. GEOLOGIC CONDITIONS

The site is located on the eastern border of the Basin and Range physiographic province, on the eastern slope of the north-northeasterly trending Sierra Blanca structural basin. The Grindstone Creek watershed is bounded by mountains composed of Precambrian rocks and igneous intrusions into gently dipping sedimentary rocks. Bedrock mapped at the dam site by Sergent, Hauskins & Beckwith (SH&B) following excavation of the dam foundation in March 1986 consists of interbedded Cretaceous age sandstone, siltstone, and shale of the Mesaverde Group and Tertiary age intrusive volcanic dikes.

The sedimentary units of the Mesaverde Group were characterized by SH&B (Boyle 1987) as moderately hard to hard sandstone, moderately soft to hard siltstone, and soft to moderately soft shale. Joints encountered within the sedimentary units varied from very close to open. Joints were generally open near the ground surface and closed with depth. Similarly, weathering of the rocks generally increased with depth. SH&B (1983) noted weathering induced by adjacent igneous intrusions and clay-filled fractures within the weathered profile with rare calcite. The SH&B map noted that the “shale slakes readily on exposure to air and/or water.”

The intrusive volcanic dikes vary in composition from andecite to dacite and were characterized by SH&B (Boyle 1987) as moderately hard to very hard, except in weathered zones where the dikes are completed altered to clay minerals. The igneous intrusions generally dip vertical to near-vertical and are likely associated with the Three Rivers Stock, which forms the core of Sierra Blanca peak (elevation 11,981 feet), located approximately 8 miles northwest of the site.

SH&B (Boyle 1987) mapped natural springs and faults within the limits of the foundation excavation. SH&B measured flows of less than approximately 1 gallon per minute (gpm) from the springs. The fault ages were not estimated, but the faults did not appear to offset Quaternary alluvium mapped in the area, and the fault surfaces lacked brecciated or gouge zones. Therefore, the faults were inferred to be pre-Quaternary age. The estimated displacement from faulting typically varied from several inches to 3 feet, although SH&B estimated displacements up to 15 feet.

III. SEEPAGE DRAINAGE GALLERY

The project Engineer’s Completion Report (Boyle 1988a) noted a design change during construction that added an internal seepage gallery to which both the foundation and internal drains discharged. The initial design planned for the drains to be piped to the toe of the dam and discharged to the stream below. The design change was the result of concerns by the general contractor on construction sequencing and was approved by the project engineer and NMOSE as an alternative to provide drainage for the anticipated seepage. The date of the design change was not included in the report.

The seepage drainage gallery is accessed from an entrance (adit) on the right side of the dam’s stilling basin. The length of the gallery is approximately 588 feet and includes staircases to the south and north of the adit. The elevation of the gallery floor at the entrance is approximately 6,812 feet, and the elevation at the top of the gallery staircases is approximately 6,851 feet. Seepage that collects in the gallery drains to the entrance, which drains to the stilling basin at the base of the dam’s principal spillway.

The Grindstone Dam Record Drawings (Boyle 1987) show a total of 104 drains connected to the seepage drainage gallery, including 59 vertical foundation drains, 36 vertical ceiling drains, 5 horizontal drains at the right abutment, and 4 horizontal drains at the left abutment. Figure 1 presents a typical section from the record drawings that shows the foundation drains were drilled at 10 feet on-center and ceiling drains were drilled at 15 feet on-center. Figure 1 also shows a section of the drainage gallery profile and horizontal drain locations at the right abutment.

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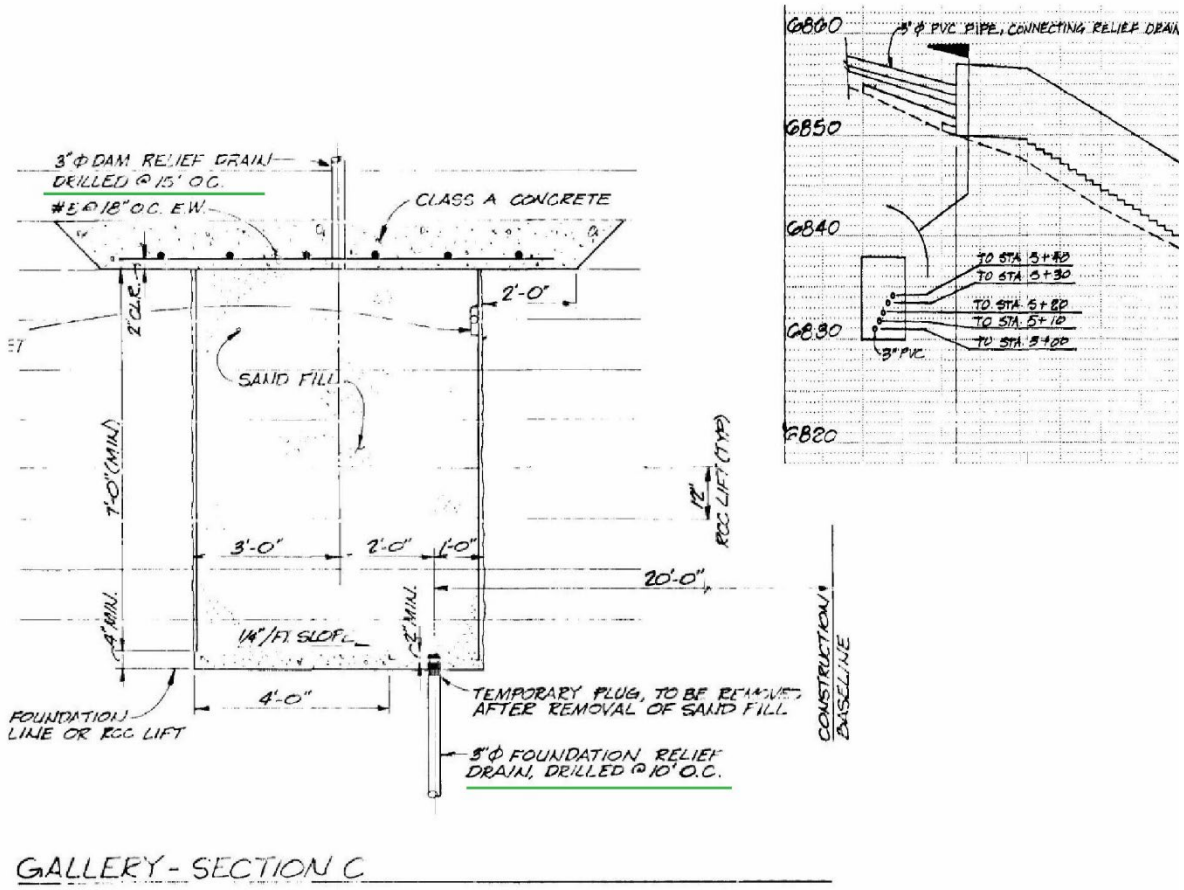


Figure 1: Typical Drainage Gallery Section with ceiling [aka relief] drain and foundation drain callouts underlined in green (left); section of drainage gallery profile and as-built horizontal right abutment drain locations (upper right)

The foundation drains were drilled in 1986 and the as-built depth of the foundation drains was shown as 50 feet. Construction photos presented in the Engineers Completion Report (Boyle 1988b) show construction of the gallery and drilling equipment presumably used to drill the foundation drains (Figure 2). The vertical ceiling drain holes were drilled upward through overlying RCC lifts, and the as-built length of the ceiling drains was shown as 40 feet.

The vertical foundation drain holes were drilled downward through the previously placed RCC lifts and into the foundation bedrock. The record drawings show the foundation drains located 20 feet east (or downstream) of the construction baseline (shown in Figure 1), which was the upstream face of the dam. Drilling records for the foundation drains were not included in the record drawings, so the rock conditions encountered at each drain are unknown.

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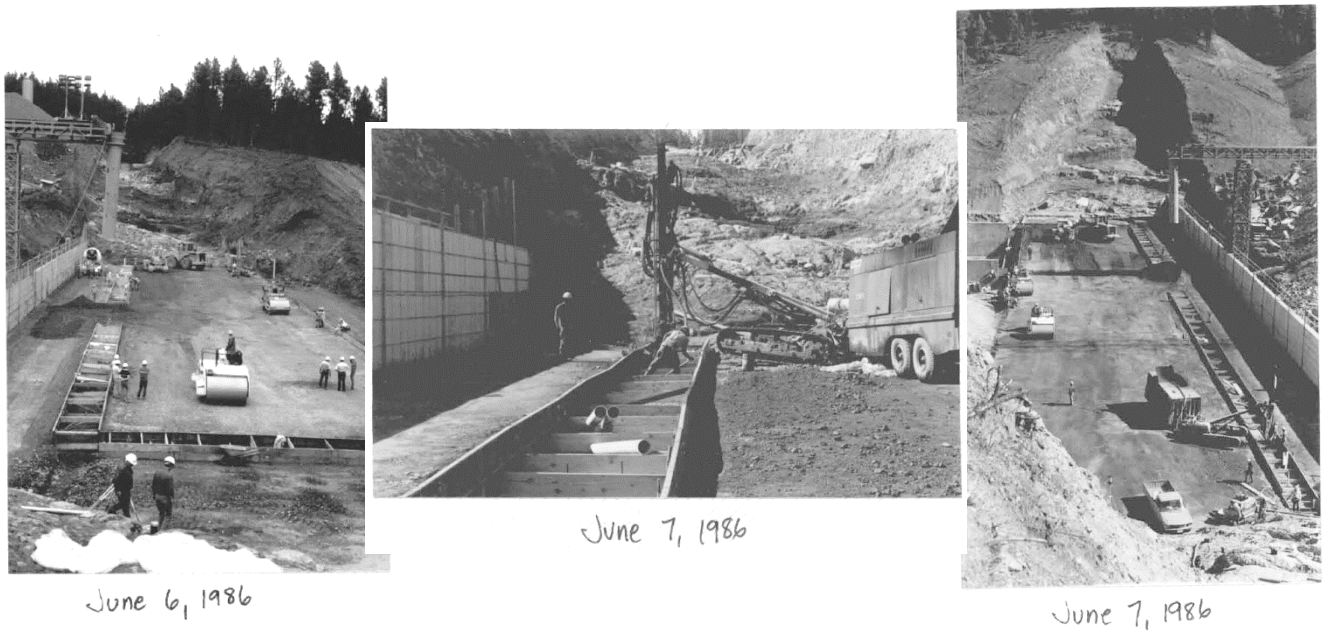


Figure 2: Dam construction photos showing formwork for seepage drainage gallery; vertical drain drilling equipment visible in background of center photo and foreground of right photo. (Photos provided by Village of Ruidoso)

IV. FOUNDATION GROUTING AND DAM SEEPAGE

The SH&B (1983) geotechnical report noted that open or clay-filled bedrock joints were anticipated at the dam foundation level and recommended a dental grout treatment to provide a competent bond between the dam structure and rock surrounding the discontinuities, and to provide a uniform surface for concrete compaction. Boyle's (1988a) Completion Report stated that dental grouting was performed following completion of the foundation excavation, and that "soft shale pockets" were removed and replaced with concrete. The description of soft shale pockets is consistent with SH&B's characterization of the shale's slake potential.

The SH&B (1983) report also recommended a low permeability curtain below the heel of the dam, formed by pressure grouting, to reduce seepage losses through relatively permeable zones associated with jointed and fractured bedrock.

The initial foundation grouting (grout curtain) was completed in May 1986. The Boyle (1987) record drawings include a Drilling and Instrumentation Layout sheet that shows a total of 349 grouting locations. Primary grout holes are shown 2 feet east (or downstream) of the construction baseline (upstream dam face). The record layout sheet includes a note that "Secondary and Tertiary Holes at Lesser Depths May be Required." Grout holes are shown on the layout sheet legend as "Hole Number (P)" for primary grout holes. Although not shown in the legend, "(S)", "(T)", and "(Q)" labels next to hole numbers are inferred to represent secondary (S), tertiary (T), and quaternary (Q) grout holes. Holes labeled (P) appear to be deeper than (S) holes, and the (S) holes deeper than (T) holes. The (S), (T), and (Q) grout holes appear to be located on the primary grout curtain line, although one note on the layout sheet indicates 4 grout holes were located 3 to 5 feet east (or downstream) of the grout curtain line. The orientation of the grout holes relative to the construction baseline (i.e., inclined downstream or upstream) is not shown on the layout sheet, but the SH&B (1983) report recommends that the bid documents for grouting include costs for grout holes up to 45 degrees off vertical.

As noted above, the dam was approved for impoundment by NMOSE in early 1988, after testing to complete the project record on concrete core strengths and testing to characterize seepage water (Boyle 1988a). That seepage water was presumably associated with watershed runoff that pooled behind the dam.

Historical seepage observed in the drainage gallery has been primarily attributed to expansion and contraction of vertical construction joints on the upstream face of the dam, and secondarily attributed to the foundation and abutment contacts. NMOSE Inspection Memoranda (2016, 2018, 2019) stated that higher seepage rates generally

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correlated with higher reservoir water levels, with fairly consistent lag times, but seepage is correlated more directly with the time of the year. NMOSE hypothesized that seepage increased in colder temperatures as a result of RCC contraction opening vertical construction joints and seepage decreased in warmer temperatures as a result of RCC expansion. The memoranda also indicated that seepage through the dam significantly increased when reservoir elevations were sustained above 6,900 feet to 6,910 feet.

Wilson & Company (2002) prepared a report with recommendations for repairs to joints on the upstream face that included a history of seepage since the dam was first filled. Initial seepage rates, presumably measured near the gallery entrance, were up to almost 1,500 gallons per minute (gpm), or 2 million gallons per day (MGD). Due to the high seepage rates, NMOSE requested that the Village reduce the seepage, which was accomplished by draining the reservoir and sealing both the upstream construction joints and foundation contact area in 1989.

The left, undated photograph in Figure 3 appears to show an excavation in the upstream foundation contact area, indicating the photo was likely taken in 1989. The two right, undated photographs in Figure 3 appear to show grouting pipes in the vicinity of the gallery stairs that were likely installed around the same time. The layout sheet from the Boyle (1987) record drawings includes a circled location on the south drainage gallery steps between approximately Stations 6+20 and 6+50 that is noted as an “Additional Foundation Drilling and Grouting Area,” and a circled location on the north gallery steps between approximately Stations 9+70 and 10+20 that is noted as a “Possible Area of Additional Foundation Drilling and Grouting.” Those post-construction grouting locations are presumably the same as the grouting locations shown in the two, right photos of Figure 3; however, additional grouting details, such as grout locations, hole logs, and grout take, were not found in the available as-built information. The photos indicate the additional grout holes were inclined downstream and/or upstream from vertical.



Figure 3: Undated photos of an excavation in the upstream foundation contact area (left), foundation drilling and grouting equipment in the seepage drainage gallery (center and right); note seepage draining from drilled grout pipes in right photo, presumably prior to grouting. (Photos provided by Village of Ruidoso)

A URS (2015) report for a 2015 seepage mitigation project stated that information provided by the Village indicated repair projects intended to mitigate seepage through the dam were executed in 1989, 1996, 2002, and 2009 with limited effectiveness. The Wilson & Company (2002) report indicates the repairs between 1989 and 2002 were primarily focused on sealing the vertical construction joints on the upstream face of the dam, and the highest measured seepage rate during that span was 646 gpm. The NMOSE (2016, 2018, 2019) Inspection Memoranda noted a historical maximum seepage rate of 715 gpm in December 2006. AMEC (2013) performed a stability analysis of the dam that considered hydrostatic pressures. AMEC recommended a liner on the upstream dam face and the liner provide at least 49-percent seepage reduction in order for the dam to meet typical design criteria for sliding and overturning. The 2015 project involved placing a Carpi liner on the upstream face of the dam, epoxy grout injections on the upstream face, and excavation and backfilling with bentonite clay along the upstream groins

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and portions of the upstream toe. The 2021 NMOSE Inspection Memorandum stated that seepage rates measured by the Village appeared to have been significantly reduced as a result of the 2015 seepage mitigation project but recommended that the Village refine its measurement methods for consistency between measurements. The memorandum did not include an estimate of the seepage reduction percentage.

The Village installed a Parshall flume in 2017 to measure discharge from the principal spillway stilling basin, which collects water from the seepage gallery and other sources, including three drains that discharge water collected behind the Carpi liner due to condensation and typical leakage.

V. DRAIN CLEANING PROJECT

Yeh and Associates (Yeh) provided engineering services for the Grindstone Canyon Dam Drain Cleaning project that was completed in May 2022. The primary purpose of the project was to restore the functionality of the dam's internal seepage gallery drains and mitigate the potential for excess hydrostatic pressure that could destabilize the dam. Secondary purposes of the 2022 Drain Cleaning project were to generally improve access to the drainage gallery and install a removable weir at the gallery entrance to isolate the gallery seepage measurements from the other locations where water drains to the stilling basin (and is measured by the Parshall flume).

During a December 2019 tour of the gallery, Yeh observed that mineral precipitates from the seepage water had accumulated around the surface of drain holes and on the gallery floor, resulting in an uneven surface that typically ponded water between floor high points. The ponded water depth was variable and up to approximately 3 feet. Vertical foundation drains were generally difficult to locate due to the ponded water. The composition of the mineral precipitates has apparently not been identified by laboratory testing but generally appeared to be white to red calcite and dark brown to black manganese.

The drain-cleaning contractor, Beartooth Water Jetting and Video Inspection (Beartooth) initially removed mineralization on the gallery floor, exposing the original concrete floor and an approximately 2-inch deep drainage trench to convey seepage water to the gallery entrance. Beartooth then installed a removable weir plate at the gallery entrance to measure pre- and post-cleaning seepage rates. The Village collected weir measurements for two weeks prior to the drain cleaning to establish a seepage rate baseline. Beartooth located and cleaned 55 vertical foundation drains, 35 vertical ceiling drains, 6 horizontal drains on the left abutment gallery wall, and 7 horizontal drains on the right abutment gallery wall. Beartooth performed pre- and post-cleaning video surveys of the drains and measured pre- and post-cleaning flow rates for each of the drains.

The equipment that Beartooth used for drain cleaning consisted of a high-pressure (approximately 4,000 to 12,000 pounds per square inch) hose and cleaning heads, and chipping power tools for removing mineral deposits near the surface of drains. The three types of cleaning heads consisted of a universal head, unblocking head, and a polishing head.

Seepage rates measured at many individual drains and the gallery entrance increased from the start to the end of the project, even as the reservoir’s water surface elevation was lowered. The effectiveness of the drain cleaning project was documented by collecting seepage measurements at individual drains and the gallery entrance before and after the project. Measured pre-cleaning flow rates from individual drains ranged from approximately 0 to 4.5 gpm; the measured flow of all drains combined was approximately 14 gpm prior to drain cleaning. Measured post-cleaning flow rates from individual drains ranged from approximately 0 to 5 gpm; the measured flow of all drains combined was approximately 35 gpm after drain cleaning, so the measured flow from all drains increased by approximately 21 gpm following the drain cleaning.

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VI. DRAIN BLOCKAGES AND POSSIBLE EXPLANATIONS

As noted above, the as-built depth of the foundation drains was shown as 50 feet (Boyle 1987). Beartooth encountered unanticipated blockages at lesser depths in many of the vertical foundation drains. Beartooth also cleaned the vertical ceiling drains and horizontal drains at the right and left abutments but did not encounter blockages in those drains. The blockages encountered in foundation drains generally occupied the full area of the drains and could not be cleared with high-pressure cleaning tools, indicating the blockages are likely not composed of mineral deposits. The composition and thickness of the blockages was difficult to discern based on video footage. Images of blockages taken from post-cleaning videos of two drains are shown in Figure 4.

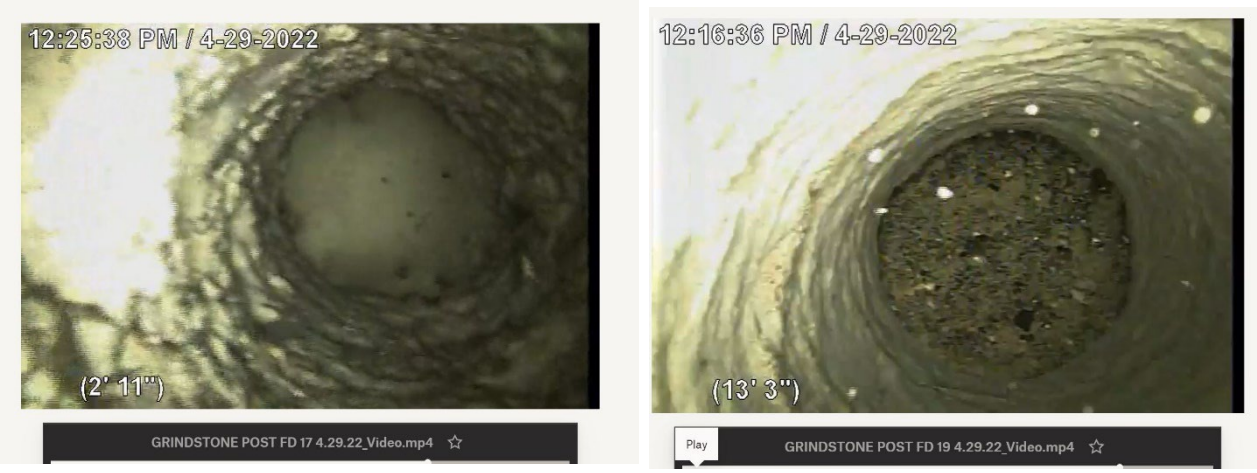


Figure 4: Blockages at 4 feet depth in Foundation Drain 17 (left) and 15 feet depth in Foundation Drain 19 (right). (Photos provided by Village of Ruidoso)

The number of foundation drains cleaned by Beartooth, and range of approximate cleaned lengths are summarized in Table 1.

TABLE 1: CLEANED LENGTHS OF FOUNDATION DRAINS (AS-BUILT LENGTH = 50 FEET)

Approximate Cleaned Length (ft)	Number of Foundation Drains	Percentage of Total Number of Foundation Drains (%)
50	6	11
40 to 50	10	18
30 to 40	3	5
20 to 30	6	11
10 to 20	6	11
< 10	24	44

The historical documentation described above provides some possible explanations for the blockages, which may consist of intact (undrilled) rock, fragments of rock collapsed from the drilled hole, or grout placed after the drain holes were drilled. Figure 5 shows the as-built geologic map of the dam foundation (Boyle 1987), project stationing, approximate limits of the drainage gallery, pertinent geologic features mapped by SH&B, approximate limits of post-construction grouting, and drain locations where blockages were encountered at depths of 10 feet or less (red bullseye) and where measured post-cleaning flow rates exceeded approximately 0.5 gpm (green bullseye).

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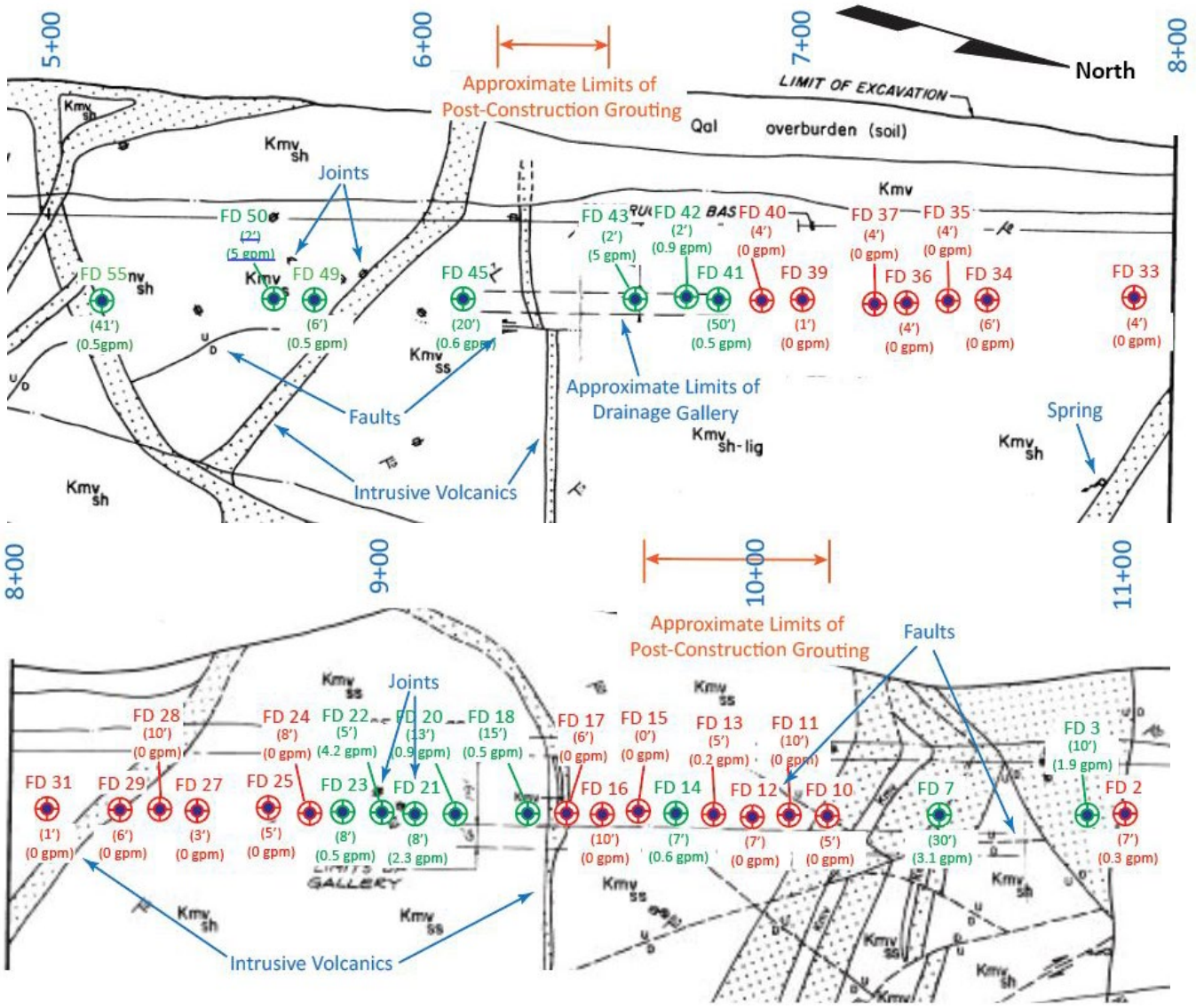


Figure 5: Boyle (1987) geologic map of foundation excavation from Station 5+00 to 8+00 (top) and from Station 8+00 to 11+00 (bottom) with drain locations shown where blockages were encountered at depths of 10 feet or less (red bullseye) and where measured post-cleaning flow rates exceeded approximately 0.5 gpm (green bullseye); note intrusive volcanics denoted with stippled pattern, Mesaverde Group sedimentary rocks (sandstone, siltstone and shale) denoted with no (or white) pattern.

The SH&B (1983) geotechnical report recommended the foundation drains should be drilled to a depth of 50 feet and slotted PVC casing should be installed in each drilled hole to maintain borehole stability. The foundation drain lengths observed in the pre- and post-cleaning video footage were uncased, and the jointing described on the as-built geologic map was apparent in the video footage. The diameters of jointed zones in drain holes generally appeared to be greater than the diameters of unfractured zones, indicating fragments of fractured rock may have detached from or slaked off the drain walls and collapsed into the drain holes. Slotted PVC casing would have likely prevented the collapse of rock fragments into the drilled hole and may have mitigated the potential for grout intrusion by post-construction grouting.

As shown in Figure 5, the blockages are predominantly located where SH&B (Boyle 1987) mapped Mesaverde Group sedimentary rocks that were characterized as moderately hard to hard sandstone, moderately soft to hard siltstone, and soft to moderately soft shale. Few of the blockages are located within the intrusive volcanics characterized as hard to very hard rock, which is typically associated with difficult drilling conditions and sometimes drilling refusal. The presence of intact (undrilled) rock in the foundation drains at depths less than the planned depth

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of 50 feet would likely be attributable to drilling refusal. However, the locations of blockages relative to the hard to very hard intrusive volcanics indicate that the blockages are likely not attributable to drilling refusal when the drains were installed.

The record drawings indicate that post-construction grouting was performed between approximately Stations 6+20 and 6+50 and between approximately Stations 9+70 and 10+20. The exact locations of post-construction grouting holes are not shown on the record drawings. Seven of the drain blockages shown in Figure 5 are between approximately Stations 9+70 and 10+20, but the drains between approximately Stations 6+20 and 6+50 were typically unblocked to depths greater than 10 feet, or flow rates exceeded 0.5 gpm. Grouting is inexact in practice, and the injected grout may preferentially flow along rock discontinuities (e.g., faults, joints, bedding) to locations far from the source location. Therefore, it is possible that grouting between approximately Stations 6+20 and 6+50 could have resulted in drain blockages beyond those limits.

VII. SEEPAGE FLOW RATE CORRELATION TO SITE GEOLOGY

Figure 5 also indicates the seepage measurements at individual drains may reflect geologic structure, as relatively high flows were measured near bedrock faults, joints, and geologic contacts mapped by SH&B (Boyle 1987). Figure 6 shows the drainage gallery floor, boring logs, and interpreted geologic profile from SH&B (1983), and the unblocked length of the same foundation drains shown in Figure 5. Foundation drains with blockages encountered at depths of 10 feet or less (and measured flow rates less than 0.5 gpm) are shown as red sticks and drains where measured post-cleaning flow rates exceeded 0.5 gpm are shown as green sticks. It should be noted that the interpreted geologic profile was prepared by SH&B prior to the dam foundation excavation, so the geologic contacts are likely inconsistent with those shown in Figure 5.

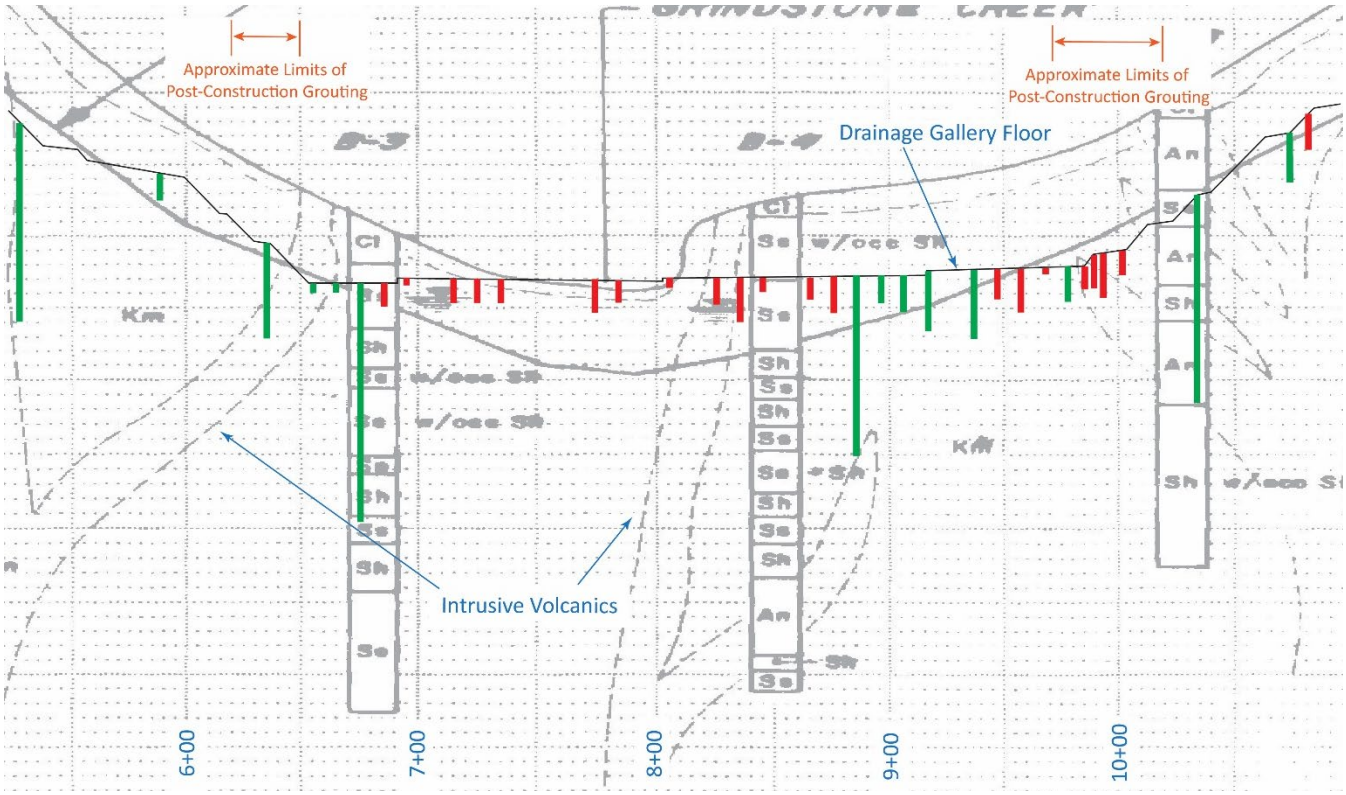


Figure 6: SH&B (1983) interpreted geologic profile along the construction baseline (upstream face of dam); foundation drains with blockages encountered at depths of 10 feet or less (and measured flow rates less than 0.5 gpm) are shown as red sticks and drains where measured post-cleaning flow rates exceeded 0.5 gpm are shown as green sticks; contacts between intrusive volcanic dikes (An, andesite) and Mesaverde Group sedimentary rocks (Km) are shown as dashed lines.

Foundation drain locations with relatively high measured flow rates are generally located near mapped bedrock faults and joints, which are shears or discontinuities in rock caused by tectonic stresses. Those fractures appear to be more permeable than the surrounding rock and are effectively subsurface conduits for relatively high seepage

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flows at the dam site. Similarly, some of the foundation drain locations with relatively high measured flow rates appear to coincide with contacts between the younger, intrusive volcanic dikes and older, Mesaverde Group sedimentary rocks in both Figures 5 and 6. As noted above, SH&B (1983) described weathered zones in the Mesaverde Group that were induced by adjacent volcanic intrusions. Those weathered zones are likely characterized by fracturing typically associated with groundwater paths (e.g., springs), and higher seepage flow rates. The spring shown in Figure 5 was mapped at one of those contacts.

VIII. LESSONS LEARNED

The drain blockages encountered during the drain cleaning project were likely the result of a combination of post-construction grouting and partially collapsed, uncased drill holes. Dam designers should perform water pressure tests after the initial grouting program to evaluate the effectiveness of grouting, which was recommended by SH&B (1983) and was likely performed during construction of Grindstone Canyon Dam. Designers should also anticipate the need for post-construction grouting, after the reservoir is filled, and include provisions to re-drill seepage relief drains after grouting.

The measured flow of foundation drains was approximately 7 gpm prior to drain cleaning and approximately 23.5 gpm following the 2022 Drain Cleaning Project, so the measured flow from foundation drains increased by approximately 16.5 gpm. The Village and NMOSE decided that re-drilling the blocked foundation drains to the as-built length was not necessary. The decision considered the effectiveness of the vertical ceiling and horizontal abutment drain cleaning, the increase in measured flow rates from the foundation drains, and the reduction in seepage following the 2015 seepage mitigation project. The decision also considered the results of the 2013 stability analyses, which assumed full uplift pressures on the dam's foundation (i.e., ineffective seepage gallery drains). The Village and NMOSE will continue to monitor seepage flow rates from the gallery and evaluate the effectiveness of the foundation drains.

Dam designers should also identify possible post-construction grout hole locations prior to drilling seepage relief drains. The design of seepage relief drains should include slotted casing to mitigate the potential for caving rock and post-construction grout intrusion. The casing slot locations should be evaluated by the project geologist or geotechnical engineer based on a review of drain hole and grout hole drilling logs that record the depths and density of foundation bedrock discontinuities. Slots in the casing should then be strategically located to both provide a path for seepage, which typically flows preferentially along rock discontinuities, and to mitigate the potential for hydraulic connection (via rock discontinuities) between drain holes and post-construction grout holes.

IX. ACKNOWLEDGMENT

The author gratefully acknowledges the support of Randy Koehn, Terry Yarborough, and the Village of Ruidoso Water Production staff during the 2022 Drain Cleaning project. The author also thanks the Village of Ruidoso for permitting the use of photos presented herein.

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XI. AUTHOR BIOGRAPHIES

Gresh Eckrich is a registered professional engineer and certified engineering geologist with over 16 years of experience in the evaluation of dams and levee systems, seepage and slope stability analyses, seismic hazard analyses, and the mapping and assessment of geologic hazards. Gresh graduated from the University of Michigan in 2003 with a B.S. in Geological Sciences. Gresh later obtained his M.S. from the University of California, Berkeley in Geological Engineering. Gresh is a Senior Geological Engineer with Yeh and Associates in Grover Beach, California. He has managed geotechnical services for high-profile onshore and nearshore infrastructure projects across the western US, including the design and construction of dam rehabilitations, flood control levees, storm damage repairs, bridges, and geologic hazard mitigation projects. He has international experience in Panama, Saudi Arabia, Abu Dhabi, Dubai, and Canada.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

The Argument for Mandatory Outlet/Drain Inspections on a 5-year Cycle.

CHADRICK D. HILL, MT DEPARTMENT OF NATURAL RESOURCES AND CONSERVATION

ABSTRACT

Montana currently requires dam owners to have the outlets/drains on their dams inspected every 5 years, congruent with their 5-year dam evaluations (permit renewal).The reason for outlet/drain inspections every 5-years? While this high frequency has been questioned, there is an alarming statistic that of the 36 structures inspected since 2020, almost 40% (14) were found to be deficient. Deficiencies found include inoperable gates, deterioration of CMP, worrisome cracks in concrete, and concerning pipe deflection.One concern is that five years ago many of these structures looked okay. This is not just small dams with CMP, we are seeing a trend in larger dams too.In this presentation, I will present a clear argument for conduit inspections on at least a 5-year cycle and present multiple case studies to illustrate this position.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

Overtopping Potential for 550 Dams: Considering Storm Events with Varying Duration and Return Period

JACOB A. SMITH, NC DEPARTMENT OF ENVIRONMENTAL QUALITY

ABSTRACT

During both Hurricane Matthew and Hurricane Florence, flooding and damages in North Carolina were exacerbated by the overtopping of multiple dams in the Cape Fear, Neuse, and Lumber River Basin's. As a result, the Program initiated overtopping studies in the affected basins, which eventually grew to include most of NC's major river basins. The initial study for the affected basins was funded entirely by the State from its Disaster Relief funds and the additional basins were funded by House Bill 200. In total, 550 dams of various hazard and size classification were selected for evaluation. The results of this study were compiled into an ArcGIS Online WebApp where point features represent dams included in the study and the associated attributes contain the results. Results can be filtered by the hazard classification, predicted rainfall accumulation and amount of freeboard, or overtopping for a selected storm. This study evaluated the potential for overtopping at the selected dams during a series of 36 pre-selected storm events of varying duration and return period. For each selected dam, Hydrologic Engineering Center - Hydrologic Modeling System (HEC-HMS) models were created, and automated reports generated that would eventually be shared to the public for education and outreach. The reports summarized the Hydrologic and Hydraulic (H&H) performance of the dam and evaluated the potential for overtopping with respect to the accumulation and duration of rainfall. Both North Carolina Department of Environmental Quality (NCDEQ) and North Carolina Emergency Management (NCEM) worked together to compile the results of this study and by utilizing weather forecasts for a given hurricane or storm, we can identify dams at risk of overtopping and implement early mitigation measures in an effort to protect life and property downstream. The results of this study were already implemented during Hurricane Ian in 2022 and helped guide NC Dam Safety and the Regional Office Staff to the dams that were of the most concern based on the data models. Thanks to the Overtopping Study, NC Dam Safety was able to prepare for the incoming hurricane days in advance and notify dam owners and engineers of the increased risk, as well as provide details on what they could do in advance of the storm to mitigate the potential for overtopping.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

Rapid Overtopping Evaluation for State-Regulated Dams in North Carolina

LANA MILLARD WANEKA, P.E., AND MICHAEL CROUCH, P.E., CFM, RTI INTERNATIONAL

ABSTRACT

Research Triangle Institute International (RTI) along with the State of North Carolina developed an efficient method to assess dam risk at large scale to prioritize rehabilitation and regulation updates. Dam failures significantly exacerbate impacts to downstream communities during flood events, and it was not well understood which dams across North Carolina were at risk of overtopping. Hurricane Matthew (2016) and Hurricane Florence (2018) caused massive flood damage in the Neuse, Lumber and Cape Fear River basins in North Carolina. In response, the state launched multiple flood control initiatives, including a study for the Cape Fear basin in which RTI evaluated the potential of overtopping for 261 dams.Pertinent characteristics for each dam such as reservoir volume, embankment elevation, spillway type and dimensions were collected for all dams by RTI partners. RTI developed an automated tool which utilized survey data to calculate the elevation-discharge relationship for each dam. Hydrologic models were then produced and executed for each dam, using automated tools, to determine which dams are most likely to overtop for the 36 design storm conditions. The process automation allowed for the nearly 10,000 required model runs to be performed under a tight timeframe and for a reasonable budget. Results of this overtopping analysis are used by the state to prioritize and monitor dams before, during, and after extreme rainfall events. The state has also incorporated the data from the overtopping studies into the Natural Resources Conservation Service's DamWatch tool, which allows officials to produce an alert when a threshold is reached at a particular dam. Based on the forecast data and overtopping risk, the state can provide guidance to dam owners ahead of an extreme precipitation event such as a hurricane. RTI's dam assessment study was part of a larger NCDEQ effort that received the Southeast Regional Award from the Association of Dam Safety Officials as well as an innovation award from the NC Department of Environmental Quality.

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DAM SAFETY PALM SPRINGS
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The NRCS Model to Estimate Monetary Benefits of Watershed Dams from a Specific Storm or Series of Storms

LARRY CALDWELL P.E., WATERSHED SPECIALIST, USDA-NRCS RETIRED, STILLWATER, OKLAHOMA; AND GARY UTLEY, HYDRAULIC ENGINEER, USDA-NRCS RETIRED, STILLWATER, OKLAHOMA

ABSTRACT

Since 1948, the National Resource Conservation Service (NRCS) has assisted local communities to build nearly 12,000 dams in 47 states. NRCS planned, designed, and constructed the watershed dams through USDA Watershed programs. The projects provide flood control, erosion control, water supply, and/or recreation. This paper describes the basis of the NRCS watershed benefits model that estimates the monetary benefits provided by each watershed project in the nation each day based on the rainfall that occurred in the drainage area of each dam. The model summarizes the benefits data by various geographical units such as watershed project, county, congressional district, state, and nation. It accumulates the benefits for various time intervals (daily to year-to-date) as well annual benefits and any user-defined period back to 2005. The model archives the rainfall amounts and monetary benefits computed for each geographical area for future use. The model uses the National Weather Service (NWS) daily rainfall data, NWS precipitation depth-duration-frequency data, and the NRCS National Inventory of Dams (NID) data. The monetary benefits data comes from the NRCS watershed database that includes the results of the economic analyses completed during the original project planning. NRCS updates the monetary benefits annually to current dollars. The total average annual monetary benefits for all watershed projects with dams is over $1.4 billion. The monetary benefits consider flood damage reduction to crops, pastures, fences, farm buildings, equipment, livestock loss, residential and commercial properties, utilities, roads, railroads, and bridges, as well as non-flood benefits such as water management (irrigation), livestock water, municipal water supply, recreation, and improved fish and wildlife habitat. The information from the model is helpful for educating residents and decision makers about the value of these projects, the benefits they provide to their community, and the need to maintain this important public infrastructure so it can continue to serve generations to come. The model reports the benefits immediately after a storm occurs so that news stories can report on the damage that did not occur due to the watershed projects, during the news cycle while the rest of the media is focusing on the devastation caused by the storm. The benefits model has been incorporated into the NRCS DamWatch application so that it is readily available to all DamWatch users.

Watershed Program Background

Since 1948, the Natural Resources Conservation Service (NRCS) assisted project sponsors to implement over 2,000 watershed projects through one of the following USDA watershed program authorities: Public Law 78-534, Public Law 83-566, Pilot Watershed Program, and Resource Conservation and Development (RC&D). NRCS planned, designed, and constructed nearly 12,000 dams in 47 states and the territory of Puerto Rico in 1,271 of these watershed projects (there are no dams in watershed projects in Delaware, Rhode Island, or Alaska). These watershed dams are federally assisted, not federally owned; they are locally owned and maintained by local watershed project sponsors. Project sponsors are generally local conservation districts, special-use conservancy districts, or municipalities and usually have easements on private lands to construct, operate, and maintain the watershed dams. In most cases, sponsors have non-technical personnel in charge of the operation and

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maintenance of the dams, so many of the project sponsors rely on NRCS for technical assistance with operation and maintenance issues.

These watershed projects have been so successful at providing flood control for so many decades that many residents do not even know that the dams exist, let alone how they impact their daily lives by reducing flood risks to their homes, businesses, schools, roads and bridges. The authors and Jim Henley, Oklahoma NRCS GIS Specialist, developed a model to provide timely information on the value of the benefits of the watershed projects immediately following a storm. While the news cycle is concentrating on the devastation resulting from the storm, the monetary benefits information can be highlighted as the damages that were avoided as a result of the watershed projects. This information is useful for not only the general public, but also elected officials and other decision makers.

Watershed Dam Description

Typically, watershed dams are earthen embankments ranging from 20 to 90 feet in height with vegetated earth auxiliary spillways and concrete or metal principal spillways. NRCS planned most watershed dams to provide flood control for agricultural land in rural areas. Some dams also provide grade stabilization, water supply for municipalities, irrigation, and/or recreation a primary or secondary purpose. Figure 1 shows a schematic of a typical watershed dam. The majority of the reservoirs provide storage for the anticipated sediment accumulation over a 50-year period. The area above the principal spillway crest elevation, as illustrated in Figure 2a, provides temporary detention storage for runoff from large storms. The principal spillway conduit, as shown in Figure 2b, slowly releases the water temporarily stored in the reservoir. This controlled release reduces the peak rate of runoff downstream from the watershed, thus reducing flood damages downstream. NRCS typically designed principal spillways to control runoff from a 25 to 100-year frequency storm (4% to 1% chance of occurrence event) depending on the structure's hazard classification. Designs typically include a vegetated earthen auxiliary spillway to safely pass excessive runoff generated by less frequent larger storm events around the earthen embankment.

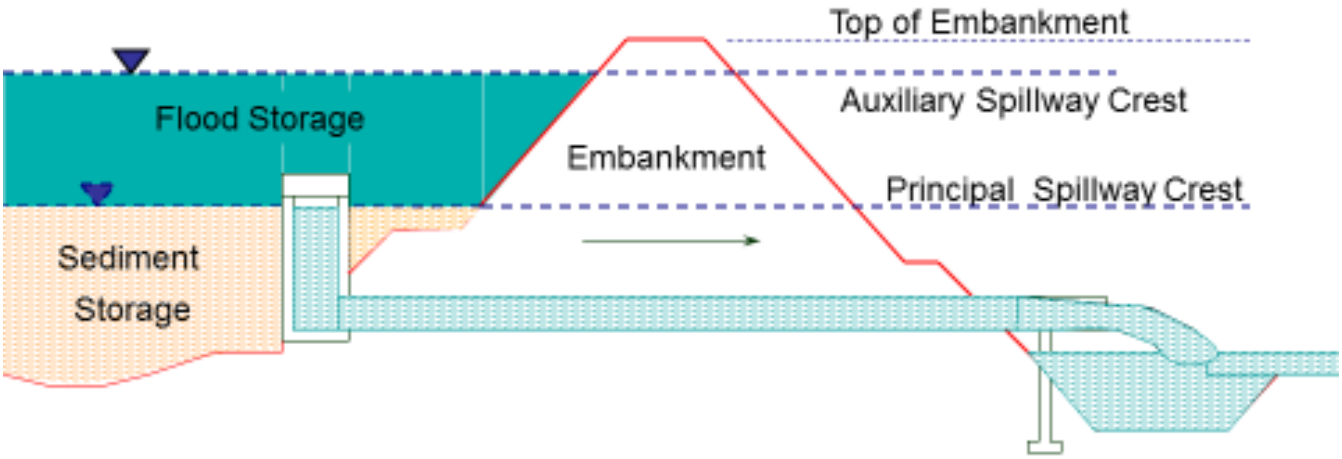


Figure 1: Section of Typical Watershed Dam

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(a)



(b)

Figure 2: (a) The detention pool (below the yellow line) temporarily stores runoff from large storms (b) The principal spillway conduit slowly releases the runoff stored in the detention pool.

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The following is a summary of the typical sizes of the 11,845 watershed dams according to the NRCS National Inventory of Dams (NID):

Dam Characteristic	Median	Maximum
Height	32 feet	144 feet
Drainage area*	1.4 sq. mi.	252 sq. mi.
Principal spillway conduit diameter	24 inches	12 feet
Auxiliary spillway width	68 feet	1,750 feet
Reservoir surface area	12 acres	3,100 acres
Sediment storage	70 ac.-ft.	18,000 ac.-ft.
Detention storage	269 ac.-ft.	59,880 ac.-ft.
Maximum storage	530 ac.-ft.	215,150 ac.-ft.
Embankment volume	50,200 cu. yds.	4,430,000 cu. yds.
Embankment length	846 feet	62,000 feet

* 93% of the dams have a drainage area less than 10 sq. mi.

Table 1: Summary of dam characteristics of watershed dams in the United States

***DamWatch*® Monitoring Application:**

NRCS partnered with USEngineering Solutions Corporation (USES) to implement their web-based application *DamWatch*® to monitor and store data for the 11,845 dams built with USDA assistance in 47 states. *DamWatch* is a patented web-based system that allows dam operators, NRCS staff, emergency managers, and state dam safety personnel to store and retrieve data, monitor storm events, and quickly respond to potentially destructive flood events. The system can be accessed in the office or through any mobile device. Only preapproved users can access the system.

The system gathers real-time meteorological and hydrological data from sources such as the National Oceanic and Atmospheric Administration (NOAA), National Weather Service (NWS), United States Geological Survey (USGS), and the NRCS Snow Telemetry (SNOTEL) system. *DamWatch* then compares meteorological data against established site-specific thresholds and alerts predetermined staff of anticipated spillway flows. Users are automatically alerted by cell phone texts and emails. Upon notification, staffs are dispatched as needed during or after a flood event to specific structures for which alerts were issued. The system also provides navigation aids to direct the user to the dams.

The *DamWatch* system also stores thousands of files of site-specific data such as as-built drawings, design data, operation and maintenance (O&M) inspection reports, emergency action plans, breach inundation maps, photos, videos, and now, watershed-benefit data.

Description of the Watershed Benefits Model:

The NRCS national watershed benefits model is based on an Oklahoma model that has been in use since 2007. In 2016, work began to expand the model to compute the monetary benefits generated by all of the watershed dams across the nation each day. The model summarizes the monetary benefits data for many geographical units such as watershed project, county, congressional district, state, NRCS Region, and nation.

The model computes daily monetary benefits for 11,779 watershed dams located within 1,265 projects in 47 states. Nineteen watershed projects cross state lines (the model considers the area in each state as a separate project). Even though the model contains all of the data needed to compute daily monetary benefits for 11,845 dams in 1,271 watershed projects, rainfall data for five projects in Hawaii (9 dams) and one project in Puerto Rico (2 dams) is not readily available. Therefore, the model computes only non-flood benefits for those six projects and 11 dams. The model does not compute benefits for 66 dams located in 13 watersheds that are inactive in DamWatch.

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The model displays the results of the daily benefits computation and archives the weighted rainfall amounts, storm frequency, and benefits computed for each geographical area for future reference. The model automatically accumulates the watershed benefits for year-to-date and annual benefits from 2005 to the present. The model can also compute the benefits for any user defined period back to January 2, 2005 (example: 21 days starting April 4, 2016, and ending April 25, 2016).

This benefits information is useful for outreach by NRCS and project sponsors as they prepare news releases, feature news stories, videos, and other media information to help promote the value of their projects and the benefits they provide to their community.

Basic Data Required for Model Development and Analysis:

1. NWS daily rainfall accumulation product called the Quantitative Precipitation Estimate (QPE) obtained for a 24-hour period ending at 1200z (6:00am CST). This precipitation data is available 8 hours after 1200z (1:00pm CST) from all states except Hawaii. NWS River Forecast Centers in the western US may not always have up-to-date data by 2:00pm CST. 1200 GMT is used as the ending time for a 24-hour total, because it is the end of the "hydrologic day by NWS. 1200 GMT coincides with 8 AM EDT.
2. NOAA Atlas 14 rainfall Depth-Duration-Frequency (DDF) data
3. Average annual benefits data updated to current year values. The monetary benefits represent the reduction in flood damages and non-flood benefits that NRCS computed during the original planning process. This data is part of a NRCS benefits database formerly named POINTS.
4. NRCS NID data for the dams, including dam ID, location (latitude and longitude), and drainage area.
5. GIS shapefiles of the drainage areas of each watershed dam developed for the updated DamWatch system.
6. Economic damage factor curve (EDF): percent of total average annual benefits for each storm frequency. The authors developed a default for each watershed project in the nation based on an analysis of economic planning data from several watershed projects in Kansas, Nebraska, and Oklahoma.

The model requires a significant amount of data gathered from various sources or computed using various assumptions. Default data is used if the required data was not readily available. States can replace default data if better data becomes available in the future.

Process used by the model to compute watershed benefits:

1. Downloads the NWS nationwide QPE rainfall grid data at about 2:00pm CST each day. The weighted average is computed by taking each 4.76 km x 4.76 km QPE grid cell and multiplying its rainfall value by the cell's overlap area within the structure drainage basin. Those values are then summed together and divided by the total drainage area of the structure. This result is the weighted average.
2. Interpolates the rainfall return frequency from Depth-Duration-Frequency Curve for that project.
3. Interpolates the EDF for the computed return frequency from the economic damage curves.
4. Calculates the percent of the total planned control of each dam by dividing the drainage area for each dam by the total drainage area in the watershed project controlled by the dams.
5. Computes the daily storm flood benefits for each dam by multiplying the EDF by the % control by the average annual flood benefits in the POINTS database.
6. Computes the daily non-flood benefits by dividing the total non-flood benefits in the POINTS database for that project by 365 days per year.
7. Accumulates and summarizes the computed benefits for standard geographical areas.
8. Archives the daily benefits data at the end of each day for future use so economic benefits reports can be generated for any time period.

Description of How Monetary Benefits for Watershed Projects were Originally Determined:

When evaluating the original watershed projects, planners computed average annual benefits for without-project conditions and for with-project conditions assuming all planned measures were installed. The difference between these two conditions is considered the planned benefits of the project. Benefits were categorized by flood damage reduction and non-flood conditions for both agriculture and non-agricultural land uses.

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Flood damage reduction benefits were generally computed by comparing the without-project and with-project damages resulting from various storm frequencies (typically 1-year, 2-year, 5-year, 10-year, 25 year, 50-year, and 100-year return intervals) for agriculture and non-agriculture conditions and then computing the statistical average annual benefits over the evaluated life of the project.

Examples of flood damage reduction benefits to agriculture include crop and pasture damage (yield reduction); damage to fences, farm buildings, and equipment; livestock loss; gully erosion, flood plain scour, and sedimentation/debris removal from flood plains and channels, etc. Examples of flood damage reduction to non-agriculture included damage reduction to urban properties (residential and commercial), utilities, roads, railroads, and bridges.

Examples of non-flood benefits to agriculture include water management, and water supply for irrigation and livestock. Non-flood benefits to non-agriculture include municipal and industrial water supply, recreation, fish and wildlife, increased property values, and reduced water treatment costs.

The total average annual benefits in the POINTS database (2022 dollars) is $2,710,861,564). Approximately half of these benefits are from 1,071 projects with dams ($1,321,905,037); the other half from 631 projects without dams ($1,388,956,527). However, seven watersheds that protect the New York City water supply skew the total monetary benefits in POINTs. These seven watersheds involve benefits of $826,558,696 (almost one-third (30.4%)) of the total monetary benefits for all projects in POINTS. Therefore, almost 60% of the monetary benefits for the projects without dams are from the seven New York City projects.

The average annual benefits data in the POINTS database is updated annually to current dollars. The following is a summary of the average annual benefits for the four categories in POINTS (2022 dollars)

Category	Benefits for all 1,683 Projects	Benefits for 1,070 Projects with Dams
Ag flood damage reduction	$ 431,556,862	$ 324,992,554
Non-ag flood benefits	$ 566,055,728	$ 452,216,171
Total flood benefits	$ 997,624,590	$ 777,208,725
Ag non-flood damage reduction	$ 540,547,729	$ 237,430,330
Non-ag non-flood benefits	$1,172,689,245	$ 307,265,982
Total non-flood benefits	$1,713,236,974	$ 544,696,312
Total benefits	$2,710,861,564	$1,321,905,037

There are 201 watershed projects with dams that do not have benefits data entered in the POINTS database. These are generally smaller pilot and RC&D projects. Based on the national median benefits per square mile of controlled data from POINTS, these projects produce approximately $37 million in flood benefits and $30 million in non-flood benefits for a total of approximately $67 million of additional average annual benefits.

The POINTS benefits data is considered the weakest link of all the data needed to compute daily benefits today. It should be noted that the economic analysis for watershed planning was completed by different people using varying criteria, base prices, and methods over forty years.. Thus, the average annual benefits per square mile of control can vary greatly from watershed to watershed depending on what was in the downstream floodplain and how the benefits were analyzed.

While far from perfect, the POINTS benefits database is the most up-to-date and complete source of the average annual (AA) benefits available for each watershed that NRCS planned and constructed. The AA benefits for each watershed reported in the POINTS database (updated to 2022 dollars) was transferred to and utilized in the national benefits model. If significant changes are made to the POINTS database in the future, the AA benefits for the

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affected watersheds can be updated in the national benefits model. The national benefits model will include the capability to update the base average annual benefits to the current year's dollars.

The POINTS flood benefits per square mile vary from $ 0/sq. mi. to $4.7 million /sq. mi. However, there are 610 dams controlling 1,752 sq. mi. with more than $50,000 benefits/sq. mi. and 415 dams controlling 3,350 sq. mi. with less than $1,000 benefits/sq. mi. Therefore, 91% of the dams controlling 88% of the total drainage area have flood benefits between $1,000 and $50,000 per square mile.

Based on 2022 data, the national weighted-average flood benefits are $19,519 per sq. mi. controlled and the median is $10,672 per sq. mi. controlled. The lower benefits per square mile are generally in the Midwest where most of the dams are located. The higher benefit values are generally in projects in states on the east and west coasts. The national weighted-average total benefits (flood and non-flood) is $33,293/sq. mi. controlled and the median is $19,309/sq. mi. controlled.

Economic Damage Factor Curves:
The Economic Damage Factor (EDF) is the percent of average annual benefits for a watershed project for a specific storm frequency. The EDF curve shows the percent of average annual benefits for a series of frequency storms. The default EDF curve in the model was developed using analyses of projects in Nebraska, Kansas, and Oklahoma. These analyses resulted in an EDF curve that is used as the default curve for all watersheds nationwide for the model.

During the review of the model by five NRCS state staffs and subsequent discussions with other state staffs, Nebraska and Kansas were determined to be the only states that had reliable data to develop EDF curves to compare with the Oklahoma benefits model default EDF curve, which has been producing reasonable results for over seven years. Nebraska staffs normally developed curves of with and without project benefits for various storm frequencies soon after a watershed project was authorized. They use these curves to report benefits from specific storms soon after they occur. Kansas staffs recently analyzed data from their projects that had significant economic data available. Table 2 and Figure 3 displays a summary of the EDF data from two Nebraska projects and two Kansas projects and the Oklahoma default damage curve.

Storm Freq. (Yrs.)	0.1	0.25	0.5	1	2	5	10	25	50	100	200	500	1,000
Swan Creek W'shed (NE)				17.3	47.2	93.2	128.7	128.4	126.9	119.0			
Wilson Creek W'shed (NE)				25.8	63.8	109.9	149.3	164.2	177.7	184.5			
Elk Creek W'shed (KS)			13.9	33.1	63.1	89.6	108.8	151.5	165.8	172.9			
South Fork W'shed (KS)			10.2	31.3	57.5	106.1	135.0	181.5	197.0	204.7			
OK Model	0	5.1	11.1	25.4	43.9	82.8	114.7	138.9	160.9	170.0	177.0	185.0	200.0
Benefit Model Default	0	5.1	12.0	26.0	55.0	90.0	125.0	150.0	170.0	180.0	188.0	195.0	200.0

Table 2: Nebraska, Kansas, and Oklahoma Economic Damage Factor (EDF) Curves:
% of Average Annual Flood Benefits for Various Storm Frequencies

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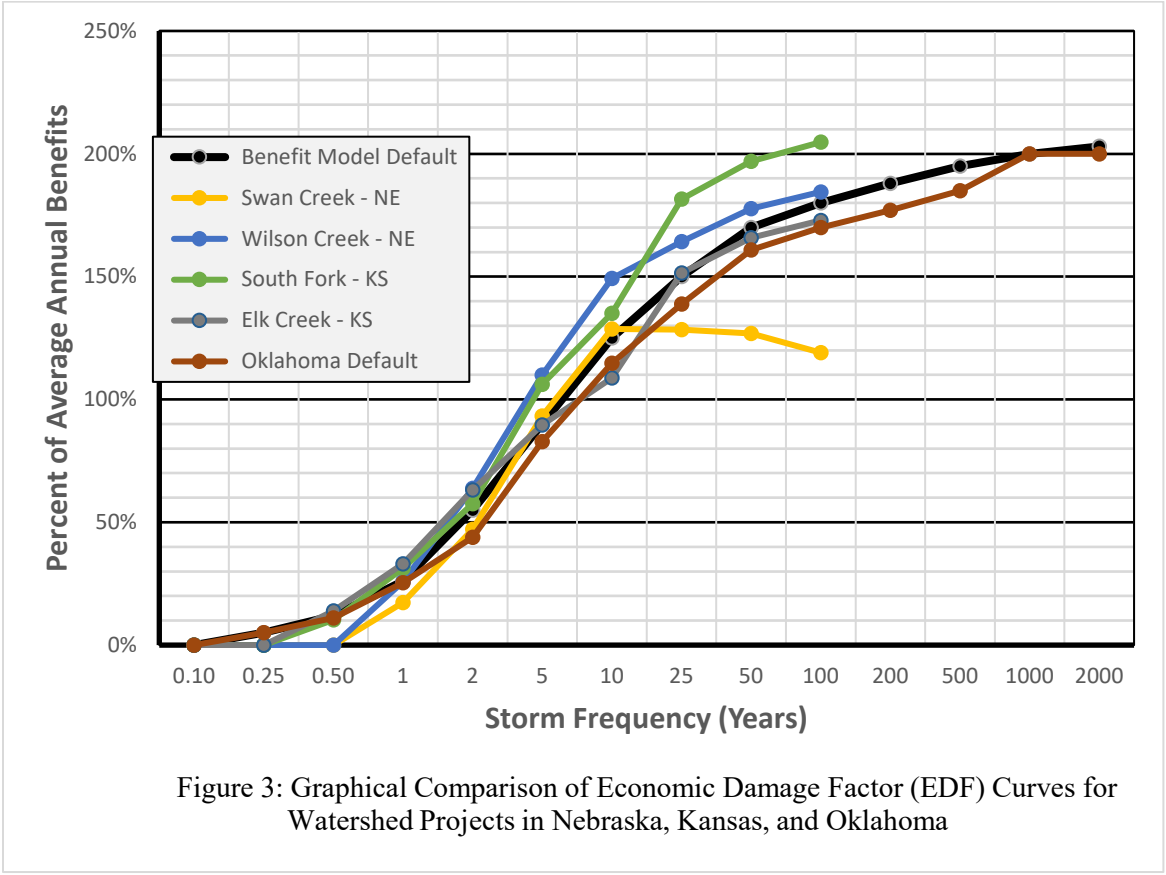


Figure 3: Graphical Comparison of Economic Damage Factor (EDF) Curves for Watershed Projects in Nebraska, Kansas, and Oklahoma

The following is the default economic damage curve data used in the model:

Storm Frequency (years)	0.1	0.25	0.5	1	2	5	10	25	50	100	200	500	1000
% of Average-Annual Benefits	0	5.1	12.0	26.0	55.0	90.0	125.0	150.0	170.0	180.0	188.0	195.0	200.0

State staffs can replace this default EDF with an EDF curve for specific watershed projects if better representative damage factor curves can be developed. However, the developers concluded that information needed for representative EDF curves is difficult to obtain for many projects because of the following factors:

a. Much of the original planning data is not available in many states.
b. Available data is not well organized making it difficult to understand.
c. Some data is for hydraulic reaches only without a composite for the entire project.
d. Much of the early watershed planning analysis was completed before computers were used so documentation is difficult to interpret.
e. Various versions of the ECONII computer program were used in the economic analysis making it difficult to interpret results. ECONII is a NRCS computer model that calculated flood damages.
f. Many projects were planned prior to the availability of the ECONII computer program so flood damage data needed to prepare a damage curve was not summarized for the earlier planned projects.
g. ECONII calculated with and without project flood damages for agriculture only. All other damages were calculated by other means. A complete economic damage curve was not a common product and was typically not included in the project documentation.

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Resolution of Precipitation and Depth-Duration-Frequency Data Issues:
The model uses NOAA Atlas 14 Depth-Duration-Frequency (DDF) precipitation data, when available, for the one through 1,000 -year storm frequencies. The model uses default data for DDF data not available through the NOAA website (see further explanation below). The default data can be replaced on a watershed basis as the data becomes available.

NOAA Atlas-14 DDF data is not available for Washington, Oregon, Nevada, Montana, and Wyoming. The NOAA Atlas-2 is the most current DDF data available for the northwest; however, Atlas-2 data is not available in a usable format. NRCS elected to also digitize and utilize TP-40 for the DDF data needed for the five northwest states.

Nationwide DDF data for less than a one-year storm frequency (0.10 year, 0.25 year and 0.50 year storm frequencies) is not available at this time. Developing this frequency data for each watershed with National Weather Bureau Technical Paper 57 (TP-57) would be extremely time consuming. Since this data has been developed for each watershed in Oklahoma, the average percentage of each frequency storm to the one-year frequency storm was used. This analysis produced the following results:

- Use 37% of the 01YR-24HR storm depth for the 0.10YR-24HR storm depth
- Use 61% of the 01YR-24HR storm depth for the 0.25YR-24HR storm depth
- Use 81% of the 01YR-24HR storm depth for the 0.50YR-24HR storm depth

A readily available source for the daily rainfall accumulation for Hawaii has not been located. Until a usable source is found, a null daily rainfall value is entered for the nine structures located in Hawaii, which will of course produce zero flood benefits. The exclusion of Hawaii allows the model to report benefits in a more timely manner since the number of time zones required is reduced from seven to five.

Jim Henley, Oklahoma NRCS GIS Specialist, developed a nationwide continental United States (CONUS) shapefile with all of the DDF data that is now available for use in this model and other purposes, for all states except Alaska.

Download of Daily NWS Precipitation Data:
NWS accumulates and reports the precipitation grid data for a 24-hour period each day from 1200z to 1200z (6:00am to 6:00am CST). The complete dataset covering the CONUS is first available by 11:30am CST. NWS updates the data several times each day as they improve the accuracy through ground-truthing, etc. In Oklahoma, reported precipitation amounts generally increase as NWS updates the data. The model uses the data version that is available at approximately 2:00pm CST and the model produces the standard reports that are usually available by 3:00pm CST. The archived precipitation data uses the latest corrected data around 9:30pm CST. Therefore, there may be a minor variation in the initial reported benefits and the archived values.

One inherent problem with the NWS precipitation data is the way it is recorded. Higher frequency storms are sometimes missed when a storm starts one day and ends the next day or the day after. The NWS QPE product records the precipitation falling each 24-hour period, starting at 6:00am CST and ending at 6:00am CST the next day. The model assumes each day's precipitation is a separate storm. For example, using a watershed project near Stillwater, Oklahoma, if a storm produces 3 inches from midnight to 6:00am one day and 3 inches from 6:00am to noon the next day, the benefits computed by the model for the two smaller storms each day are only one-third of the benefits computed for the higher frequency storm for the six-inch 24-hour event. This issue is inherent to the reporting of the rainfall.

Changes in National Weather Service Reporting of Observed Precipitation:
Prior to June 27, 2017, the NWS provided a GIS shapefile of the observed precipitation that included the CONUS, Alaska, and Puerto Rico. The DBF file for this shapefile was easily loaded into EXCEL and processed to determine the rainfall at each of the 11,836 structures (excluding 9 dams in Hawaii). On June 27, 2017, the NWS dropped production of the shapefile and replaced it with a GeoTIFF format that requires the use of GIS. This forced the use of the NetCDF format that requires considerable pre-processing before use in the benefits model. This was a significant change in the data delivery for the observed precipitation data. The NetCDF format also has individual

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files for CONUS, Alaska, and Puerto Rico, which would require processing of both the CONUS and Puerto Rico files. Since Puerto Rico only has two structures, it was decided not to include those dams in the model due to the complexity required to provide rainfall and compute flood benefits for Puerto Rico. All of the rainfall data since January 2005 has been downloaded and archived in case NWS again changes locations or eliminates stored data. This situation is a good example of the need for administrators of the model to monitor the results on a daily basis to identify changes in data that will affect the accuracy and running of the model.

Resolution of Questionable Dam Drainage Area Data:
Jim Henley, Oklahoma NRCS GIS Specialist, compared the dam drainage areas shown in the NID for each dam to the drainage areas calculated from the GIS drainage area shapefile for each dam. The results showed that the drainage areas of 1,479 dams were more than 20 percent different. Further evaluation showed that many of the issues with the NID drainage areas involved dams in series. The NID drainage area is to include the entire drainage area including any upstream dams, while the benefits model drainage area only uses the uncontrolled portion of the drainage area for dams in series. It is critical for the drainage area controlled for each dam to be as accurate as possible to calculate the correct benefits. The model uses the NID drainage area for each dam unless the NID drainage area value was zero (78 dams) or the difference between the NID and calculated GIS values was greater than 20 percent, in which case the value calculated from the GIS shapefile was used. The model enters a zero for the drainage area of all of the 66 inactive dams in DamWatch so no benefits are computed for those dams.

POINTS Benefits Data is Generally Conservative:
Most users of the POINTS data believe that the average annual benefits in POINTS today are very conservative and do not capture the present-day benefits of the watershed projects. The following are the primary reasons why POINTS benefits are conservative:

- Changes in land use in the project benefited area including increased downstream development. Sometimes land that was in agricultural crops have been converted to subdivisions with many homes, businesses, roads, etc., so the projects that were planned to protect agricultural crops now protect lives as well as millions of dollars of property.
- The original planned benefits did not capture all of the benefits at the time they were planned. Discussions with many of the original watershed project planners in several states revealed that once enough benefits were identified and documented to justify the project with a benefit cost ratio greater than 1.0, the planning was considered complete, the work plan was authorized, and they moved on to the next project.
- Property values around reservoirs have increased greatly due to the demand for lake front property.
- Many secondary benefits resulted from the increased flood protection and businesses associated with increased water-based recreation and wildlife habitat improvement created by the projects.
- The benefits of the watershed projects are continuing well past the original economic evaluation period. Most of the projects were evaluated for a 50-year period. More than half of the dams have been functioning for more than 50 years, some as long as 70 years. It is anticipated that most of the projects will continue to provide planned benefits for several more generations assuming the measures are properly maintained.
- Today's crop yields are much greater than anticipated when the projects were planned due to improvements in plant genetics, intensive use of fertilizers and pesticides, and irrigation. For example, Figure 4 displays USDA-NASS yield data that shows that since 1955, corn grain yields in the U.S. have increased at a fairly constant rate of 1.9 bushels per acre per year, increasing from a national average of approximately 40 bushels per acre in 1955 to 170 bushels per acre in 2016.

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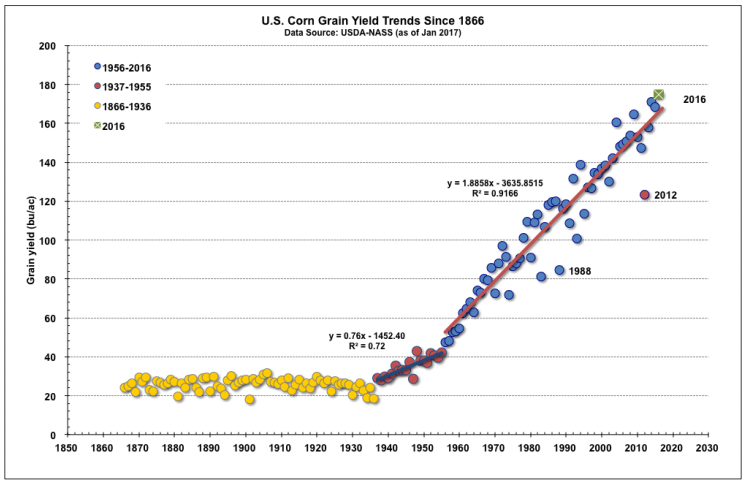


Figure 4: U.S. Corn Yield Trends since 1866
https://www.agry.purdue.edu/ext/corn/news/timeless/yieldtrends.html

- Today's cropland values are much greater than anticipated when the projects were planned. For example, Figure 5 displays land value data that shows that since 1969, land values in several Midwestern and Western states have increased from 6 to 15 times during the past 48 years, increasing from approximately $500 to $1,000 /acre in 1969 to $3,000 to 11,000 per acre in 2017. This data is taken from a study completed by the Department of Agricultural and Consumer Economics University of Illinois.

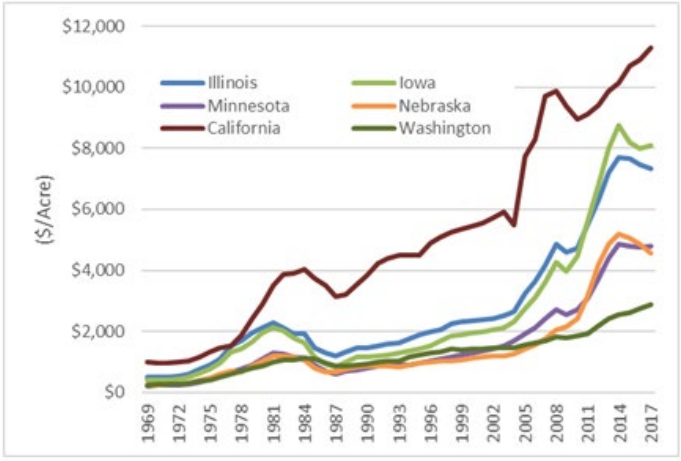


Figure 5: Cropland values since 1969 in selected states
http://farmdocdaily.illinois.edu/2017/08/updated-farmland-values-indexing-tools-2017.html

There are also many social benefits that were not captured in the original economic benefit analysis. In addition to providing significant water-based recreation and wildlife habitat improvements, watershed projects reduced the following risks that increased the quality of life within the watershed projects:

- Threat to loss of life
- Inconvenience associated with damage to roads and bridges
- Disruption of life safety services such as police, fire protection, and emergency equipment.
- Interruption of utilities
- Pollution of drinking water supplies

There are some projects where the benefits may be less than originally planned due to the following:

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- Some of the planned projects assumed that landuse would change in the with-project condition to more intense cropland with higher values. Sometimes this intensification was not realized.
- Downstream landuse changed from cropland to less intensive use like range or pastureland.
- Large reservoirs were constructed downstream on areas where flood reduction benefits were originally assumed.

However, most NRCS staffs that are knowledgeable of the watershed program history believe that re-evaluations of benefits using today's techniques and conditions would yield much greater average annual benefits than originally planned.

1987 Evaluation of the Watershed Program:
The Soil Conservation Service (now NRCS) conducted a comprehensive study of the PL-566 watershed program from October 1983 to October 1985. In July 1987, SCS published the findings in a report entitled "Evaluation of the Watershed Protection and Flood Prevention Program". The study collected an extensive set of physical and economic data for 60 projects randomly selected at the beginning of the evaluation in October 1983. The evaluation team selected these projects from the 486 projects that had been completed for at least 5 years (i.e., completed by October 1978). The evaluation team determined that 5 years was an adequate period for identifying economic benefits and other impacts of a project. The study estimated economic benefits accruing to the 486 projects from the sample data with 80 percent statistical reliability. The evaluation team did not have the resources to measure directly the impacts of land treatment installed in the watershed projects. Instead, they used data from the National Resources Inventory to estimate those impacts.

The evaluation concluded "the ratio of economic benefits to costs in fiscal year 1984 was greater than expected. Although the Federal dollars spent on these projects exceeded the planned cost by an average of 22 percent, the economic benefits accruing in 1984 exceeded by 34 percent the benefits anticipated in the project plans." It also concluded that "it is likely that the measured agricultural benefits in 1984 are conservative". The planed estimate of benefit-cost ratio for all 486 completed projects averaged about 2.0 to 1.0. The actual ratio of benefit to cost in the 1984 study, based on the sampled projects, was about 2.2 to 1.0.

The model developers and reviewers of the model from five states with remaining institutional knowledge of the watershed program, universally agree that the current POINTS benefit data needs attention. A significant effort is needed to review and update the data to be more accurate and consistent with actual current conditions. This group recommends that a comprehensive evaluation of the current watershed program benefits (similar to the benefits portion of the 1987 study) be conducted in order to be able to report more accurate benefits of the watershed program.

The Watershed Benefits Model has been Incorporated into the NRCS DamWatch Application:
DamWatch® is a patented web-based dam-monitoring software system owned by USEngineering Solutions. The application empowers users to leverage electronically stored dam safety data with real-time precipitation, seismic, and USGS gage data. The flexibility of secured remote access allows NRCS, sponsors, emergency managers, and dam safety officials to make informed decisions based on real-time data and dam information from anywhere at any time. The system can be accessed from any computer or any mobile device that has Web access. Only preapproved users with an assigned login and password can access the system.

DamWatch® utilizes Web mapping services to display the dam location and drainage area, enabling dam owners and dam safety personnel to monitor dams in real time and proactively respond to potentially hazardous and life-threatening storm events. This system can be used to collect and process real-time data at regular intervals and can then be compared against predetermined thresholds to alert essential personnel when the dams are experiencing a critical event.

Rainfall amounts at specific locations can vary significantly when a precipitation event moves through an area with numerous watershed dams. Using NWS radar data, DamWatch® determines the weighted average rainfall within their drainage basin. This provides more accurate real-time precipitation data to be used in the benefits model.

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DamWatch Displays of Benefits Data:
The model computes the benefits for all geographies each day and archives the data. Any of the following geographies can then be accessed in DamWatch (number of areas are shown in parenthesis):

1. National totals (1)
2. NRCS regions (4)
3. State/territory (48)
4. Congressional districts (210)
5. Counties (877)
6. Watershed projects (1,265)

When a specific geography is selected and the user accesses the benefits feature, the following benefits data is immediately displayed in a table for the following time periods:

1. Previous day from the current day
2. Year to date
3. Annual benefits data for each year from 2005 to the current full year (note, the benefits shown are in the dollars of that year; not updated to current dollars).
4. The average annual benefits updated to current dollars.

For each line, the following information is displayed:

1. Time period
2. Maximum rainfall
3. Weighted average rainfall over the drainage areas of the dams within the geography
4. Flood benefits
5. Non-flood benefits
6. Total benefits

Benefits can also be displayed for any multi-day period from the current date back to February 2005.

“Top Results” Reports in the DamWatch Benefits Model
For each day, the top results of most benefits and highest rainfall frequency rating for that day are displayed from highest to lowest. The rainfall frequency rating gives an indication of the storm strength for the 24-hour period (used mainly for structures and watersheds). The frequency ratings are shown to highlight the significance of the rainfall for each geography (for example a 3-inch rainfall in the Southeast may be only a 5-year, 24-hour frequency rainfall event, while a 3-inch rainfall in the Southwest may be a 100-year, 24-hour frequency rainfall event.

Each of the lines in the “Top Results” displays include the following:

1. Geography identification
2. Geography Type (such as state, county, project, etc.)
3. The number of dams within the geography area
4. The total controlled drainage area of the dams within the geography area
5. The maximum rainfall within the geography area
6. The weighted rainfall within the geography area
7. The rainfall frequency rating which is the weighted average of the return period or storm frequency for each of the dams within the geography area
8. Monetary flood benefits in millions of dollars for that geography area
9. Monetary non-flood benefits in millions of dollars for that geography area
10. Total monetary benefits in millions of dollars for that geography area

Each display can be exported in a CVS or Excel table.

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Figures 6 thru 10 provides examples of DamWatch benefits displays showing the rainfall and benefits data for selected geography areas.

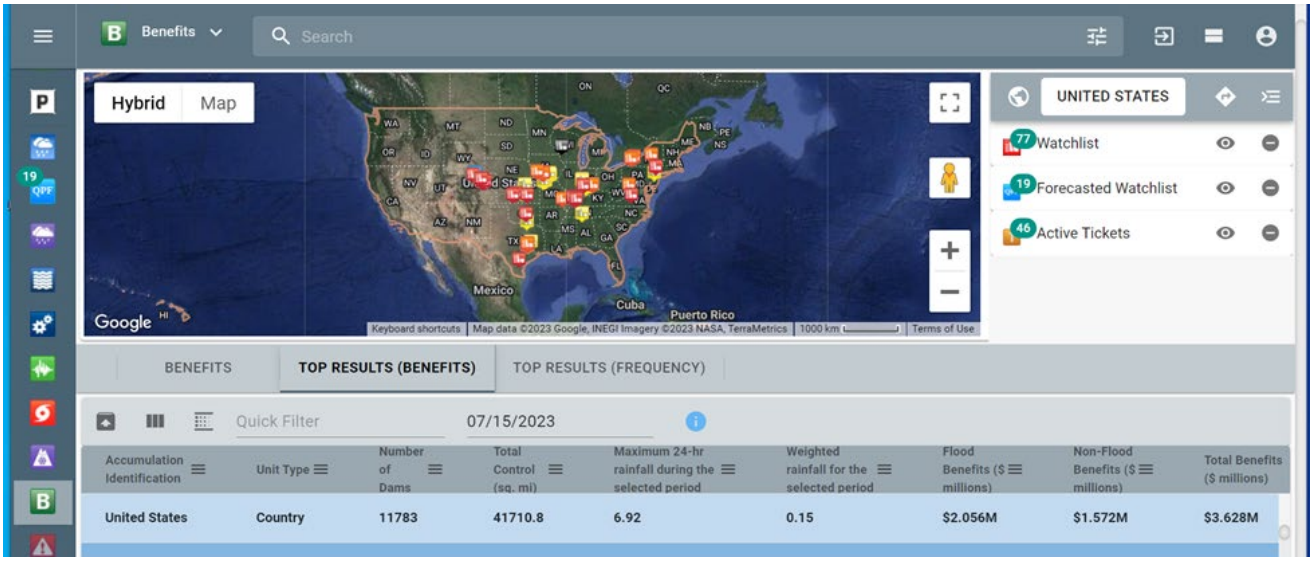


Figure 6: Display of DamWatch showing number of watershed dams, total controlled drainage area, maximum rainfall, and flood, non-flood and total monetary benefits for all US projects on July 15, 2023

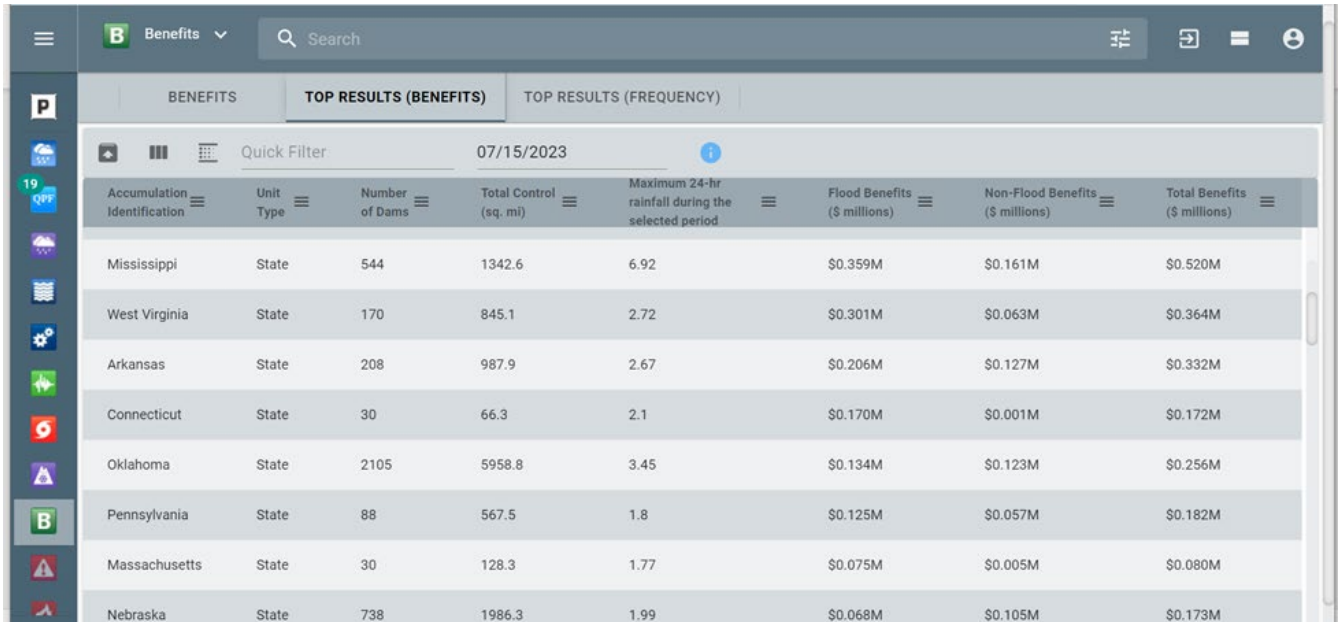
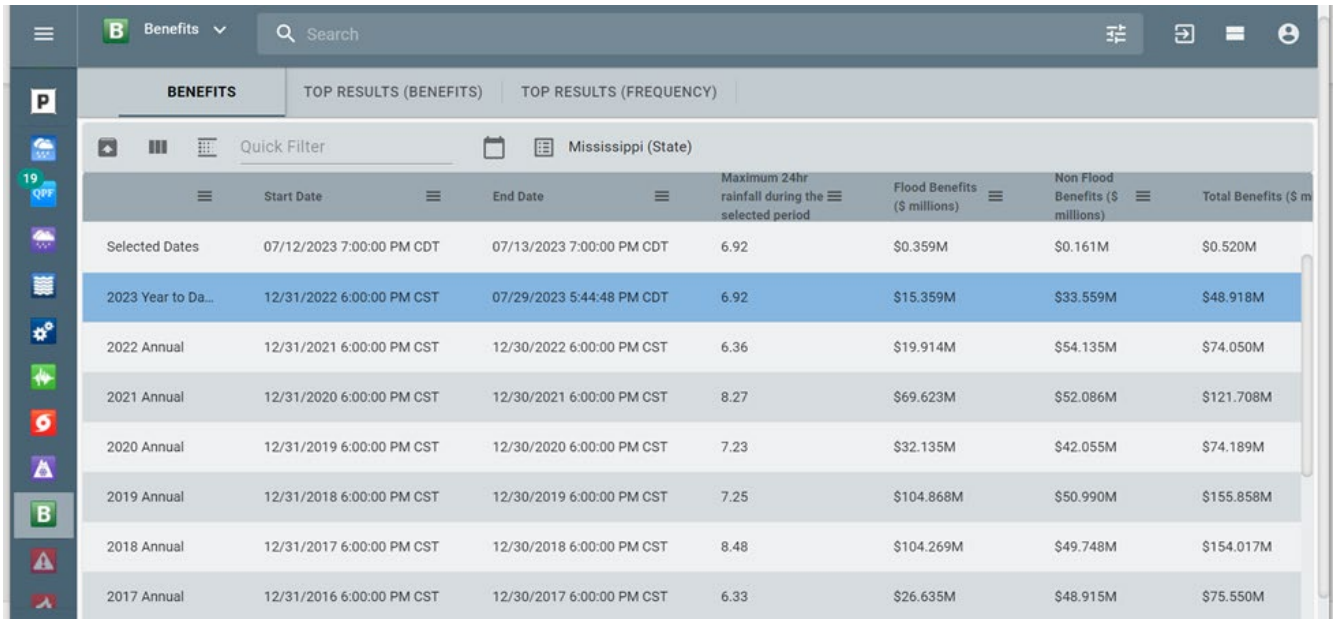


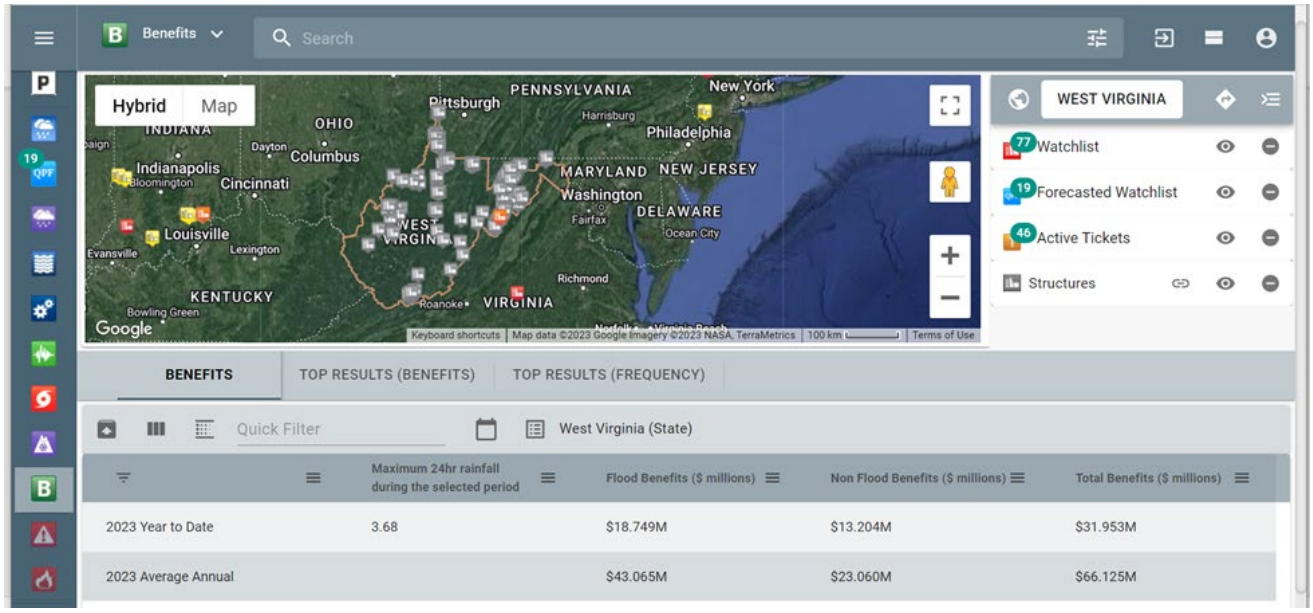
Figure 7: Display of DamWatch showing the states with the highest monetary benefits on July 15, 2023 (Note: the 544 watershed dams in Mississippi provide benefits of $520,000 for that day)

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	Start Date	End Date	Maximum 24hr rainfall during the selected period	Flood Benefits ($ millions)	Non Flood Benefits ($ millions)	Total Benefits ($ m
Selected Dates	07/12/2023 7:00:00 PM CDT	07/13/2023 7:00:00 PM CDT	6.92	$0.359M	$0.161M	$0.520M
2023 Year to Da...	12/31/2022 6:00:00 PM CST	07/29/2023 5:44:48 PM CDT	6.92	$15.359M	$33.559M	$48.918M
2022 Annual	12/31/2021 6:00:00 PM CST	12/30/2022 6:00:00 PM CST	6.36	$19.914M	$54.135M	$74.050M
2021 Annual	12/31/2020 6:00:00 PM CST	12/30/2021 6:00:00 PM CST	8.27	$69.623M	$52.086M	$121.708M
2020 Annual	12/31/2019 6:00:00 PM CST	12/30/2020 6:00:00 PM CST	7.23	$32.135M	$42.055M	$74.189M
2019 Annual	12/31/2018 6:00:00 PM CST	12/30/2019 6:00:00 PM CST	7.25	$104.868M	$50.990M	$155.858M
2018 Annual	12/31/2017 6:00:00 PM CST	12/30/2018 6:00:00 PM CST	8.48	$104.269M	$49.748M	$154.017M
2017 Annual	12/31/2016 6:00:00 PM CST	12/30/2017 6:00:00 PM CST	6.33	$26.635M	$48.915M	$75.550M

Figure 8: Display of DamWatch showing the benefits for 544 watershed dams in Mississippi for each of the past years (Note: when one scrolls down, the benefits data for each year from the present back to 2005 can be seen.)



	Maximum 24hr rainfall during the selected period	Flood Benefits ($ millions)	Non Flood Benefits ($ millions)	Total Benefits ($ millions)
2023 Year to Date	3.68	$18.749M	$13.204M	$31.953M
2023 Average Annual		$43.065M	$23.060M	$66.125M

Figure 9: Display of DamWatch showing the monetary benefits of the watershed dams in West Virginia for the year to date (July 31, 2023, compared to the average annual benefits)

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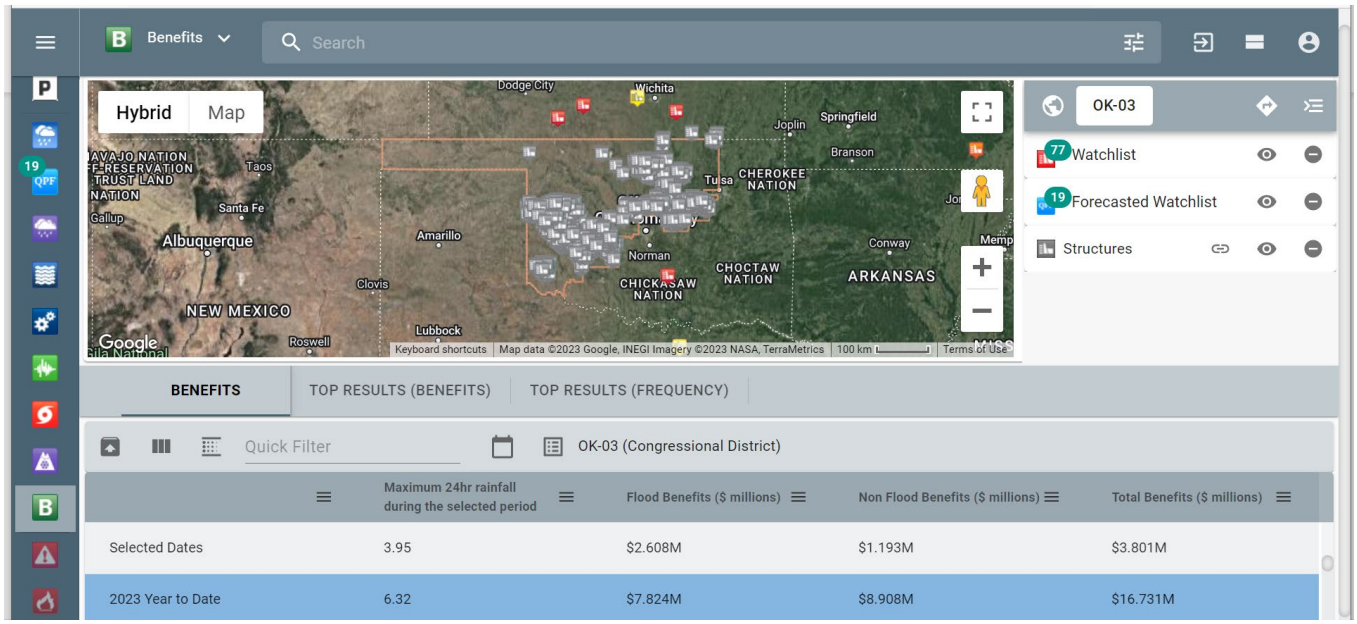


Figure 10: Display of DamWatch showing the benefits provided by the watershed dams in Oklahoma's Congressional District 3 during the month of July 2023

Conclusions:

The NRCS watershed benefits computer model that has recently been incorporated into the NRCS DamWatch application allows users to determine the estimated daily monetary benefits resulting from 1,265 projects containing 11,779 dams in 47 states. Benefits can be reported for any specified period from the previous day to any multi-day period back to 2005. The benefits computations are based on daily accumulated National Weather Service QPE rainfall data over the drainage area of each dam. The benefits data is based on the average annual benefits calculated when the projects were originally evaluated; updated annually to current dollars. The model summarizes the monetary benefits data for many geographical units such as watershed project, county, congressional district, state, NRCS Region, and nation.

After a geography area is selected, the data is displayed in a table for the selected date, as well as the year-to-date values, and annual values for each year back to 2005 and the average annual benefits for the current year. The "Top Results" tabs display the benefit values from largest to smallest and the associated rainfall and storm frequencies.

The benefits model provides timely information on the value of the benefits of the watershed projects immediately following a storm. While the news cycle is concentrating on the devastation resulting from the storm, the monetary benefits information can be highlighted as the damages that did not occur as a result of the watershed projects. This information is useful for not only the general public, but also elected officials and other decision makers.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

SREL C3 – Construction Challenges and Solutions

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ABSTRACT

This paper discusses the challenges encountered and solutions adopted during the construction of 3,876 feet of deep mixing method (DMM) cutoff wall, 7,116 feet of soil-bentonite (SB) wall, 1,100 feet of Slag-Cement-Cement-Bentonite (SCCB) wall, and earthwork during reconstruction of the levee after cutoff walls were constructed. The DMM wall encountered challenges due to use of air pressure through the drill stems, multiple instances of stuck augers in the ground, and impact on mixing index due to a broken paddle on the auger. The challenges with open trench soil-bentonite (SB) and Slag-Cement-Cement-Bentonite (SCCB) walls included confirmation of the tie-in layer, interpretation of trench sounding data, a slurry leak, and quality control issues. The paper emphasizes the importance of construction sequencing and challenges encountered at the tie-in locations of different walls. The paper also discusses the challenges to reconstruction of levee embankment and the solutions adopted to overcome the varying stratigraphic conditions at the degrade elevation and different cutoff wall types creating varying challenges to the reconstruction of the levee.

PROJECT BACKGROUND

Sacramento River East Levee (SREL) levee improvements, part of American River Common Features (ARCF) 2016 Project, are being executed by the United States Army Corps of Engineers (USACE) under four different contracts (Contracts 1 through 4). The Contract 3 Project (Contract 3), completed in 2022, involved construction of two miles of cutoff wall. The construction of Contract 3 flood protection features is intended to reduce the flooding risk to the critical infrastructure in urban areas of Sacramento, California. Joint Venture (JV) Partners Kleinfelder and Stantec supported the USACE Sacramento District (SPK) on the design of SREL Contracts 3 and 4. The team provided civil and geotechnical engineering-related design services during the design of various project features and prepared construction documents. The JV team further supported USACE by providing engineering during construction services on Contract 3.

Project Location

The project assumes significance due to its location and importance from public safety and asset protection standpoint. The of the Contract 3 footprint is along the east bank of the Sacramento River between City of Sacramento storm water pump stations Sump 70 at upstream end and Sump 132 at the downstream end, as shown in Figure 1.

Project Scope

The 2-mile-long seepage cutoff wall included construction of 3,876 feet of deep mixing method (DMM) cutoff wall, 7,116 feet of soil-bentonite (SB) wall, 1,100 feet of Slag-Cement-Cement-Bentonite (SCCB) wall, and earthwork during reconstruction of the levee after cutoff walls were constructed.

The DMM method of construction uses specialized equipment to mix in place water, cement, bentonite, and soil to construct a uniform, low permeability, soil cement bentonite (SCB) seepage barrier wall through and beneath the levee.



Figure 1: Map of SREL Contract 3

For Contract 3, the deep soil mix (DSM) method was used, which consisted of a gang of three overlapping mixing shafts/augers with a series of paddles that are three feet across at the bottom of the shafts. The cutoff wall will consist of a series of overlapping 3-foot diameter columns installed in primary and secondary panels.

SB and SCCB cutoff walls are constructed using open trenches that are excavated to the planned depth with a long-reach excavator and the excavation is stabilized with a trench slurry. For SB cutoff walls, the trench slurry is a mixture of bentonite and water, and the open trench is backfilled with a SB mixture which displaces the bentonite slurry and forms the low permeability seepage barrier. For SCCB cutoff walls, the trench slurry is a mixture of slag cement, portland cement, bentonite and water that hardens in the trench to form the low permeability seepage barrier. A SCCB cutoff wall is excavated in overlapping, 30-foot-long panels with 2 or 3 constructed per day.
Several Challenges were encountered during the construction of the cutoff wall. These Challenges can be broadly grouped into three categories associated with either the cutoff wall construction methodology or can be categorized as miscellaneous. A few of these challenges have been discussed as part of this paper under each identified category.

Category 1: DMM Method

1. Multiple instances of stuck augers in the ground
2. Use of air pressure through the DMM drill stems
3. Impact on mixing index due to a broken paddle on the auger
4. Tie-in locations of different cutoff walls

Category 2: Open Trench Methods

5. Importance of cut-off wall tie-in layer
6. Interpretation of trench sounding data
7. Slurry leak – Research on History

Category 3: Miscellaneous

8. Quality control issues – Field QA & QC Coordination & Cutoff wall removal
    a. QA SCCB Samples – Poor quality Samples
    b. QA Testing
    c. QA/QC coordination calls
9. Overcome the varying stratigraphic conditions
10. Collaborative team environment approaches

1. Multiple instances of stuck augers in the ground

During the construction of DMM wall, multiple instances of stuck augers occurred at the varying depths. Efforts to retrieve the augers were unsuccessful leading to abandoning the grouting the augers in place. An in-depth analysis was conducted to understand the reason for the stuck augers on a case-by-case basis.

Instance 1. Stratigraphic Evaluation on stuck augers

The augers were stuck during drilling at a depth of 139 feet below the degrade (Elevation -112 ft) after completion of mixing to design depth of 140 feet. After 8-hour effort the augers were raised to a depth of 123.7 feet below degrade (Elevation -96.7 ft). No additional progress could be made to retrieve the augers. The stratigraphic representation of the data is shown Figure 2.

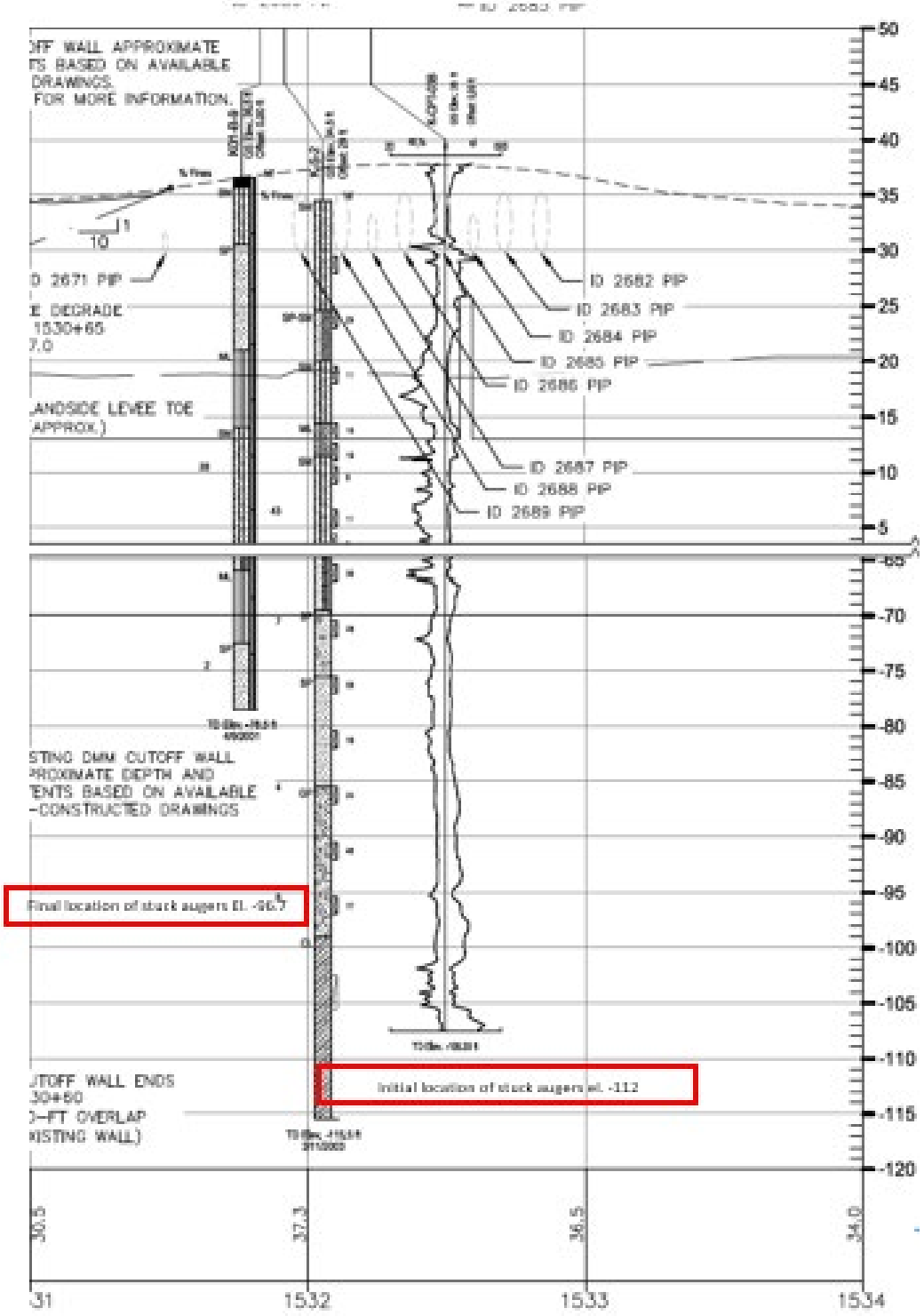


Figure 2: Stratigraphic Representation of the Instance 1

Instance 2: Stratigraphic Evaluation on stuck augers

The augers encountered resistance during drilling at a depth of 124 feet (below degrade) before reaching the target depth of 137 feet. During an attempt to restroke the element, the augers got stuck at a depth of 120 feet below the degrade (Elevation -93 ft). No additional progress was made to retrieve the augers. The stratigraphic representation of the data is shown in Figure 3.

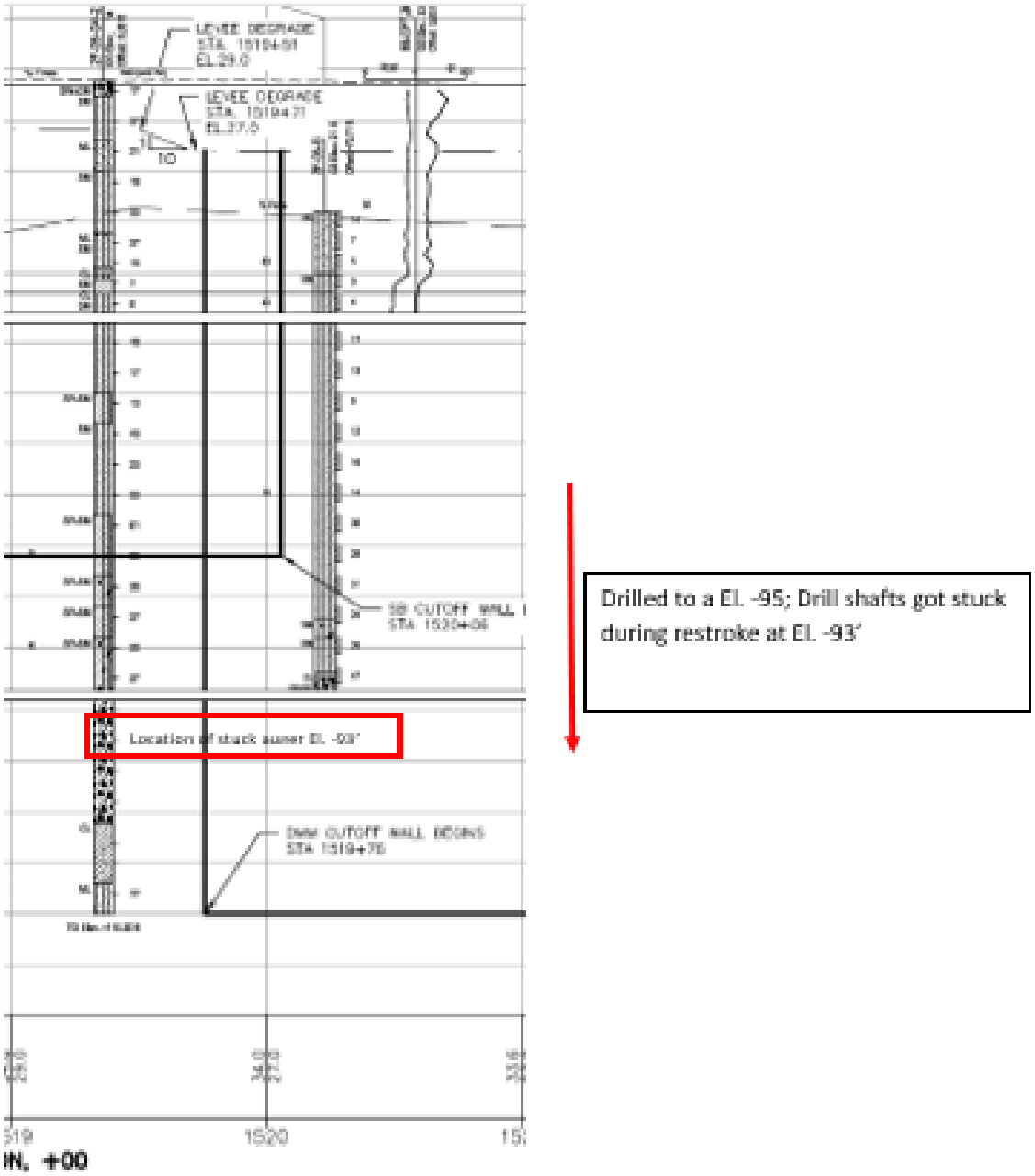


Figure 3: Stratigraphic Representation of the Instance 2

Stratigraphic evaluation of the two instances of auger stuck cases indicate that drilling through stratigraphic changes below elevation of 90 feet seems to be a major challenge. In both instances of stuck augers, the augers could not overcome the resistance and dislodge itself from the gravel interfacing layer, even after multiple restroking efforts. This could be potentially due to the accumulation and consolidation of non-plastic fines at these interfaces, with grout injection during the drilling process to the target depth.

The stuck augers needed to be abandoned in place. The ground surrounding the mixing tools were excavated to a sufficient depth to allow the auger steel to be cut below the working pad elevation. CLSM was placed below the auger stem cutoff elevation (e.g. 2 feet below the working surface platform). The remainder of the auger cutoff excavation below the working surface platform was backfilled with select levee fill. The interior of the hollow augers

was tremie grouted to the top of the auger with the DMM grout mix. To avoid additional instances of augers getting stuck, the top blade of the auger was flipped to support the cutting during withdrawal.

2. Use of air pressure through the DMM drill stems

DMM process includes the use of a cement grout along with a compressed air stream. Increased volume and air pressure with depth to keep heavier particles suspended in the mixed column of grout is common practice. However, the use of compressed air at shallower depths is usually reduced to mitigate potential risk to the levee embankment. These two streams are delivered through separate hoses that combine in a “pod” at the drill rig. Compressed air is introduced by a 375 cfm compressor with normal operating working pressures of 120 psi to 170 psi at the discharge port. Compressed air is only combined with the grout stream going to the center drill stem of the three-axis DMM tooling system. The grout-air fluid travels through a hose to the swivel at the top of the rig and down the interior of the drill stem before exiting a port at the side of the center mixing tool.

The use of air injection is not anticipated to have a direct deleterious effect on the levee since the grout-air fluid will be combined prior to delivering the fluid to the mixing tool and into the ground. As such, the risk of direct erosion of the subsurface soils through a concentrated stream of high-pressure air injection is low. The pressure of the grout-air fluid delivered to the bottom of DMM column is expected to be a function of the air pressure. As such, the air pressure should be actively regulated when mixing soil within 10 to 45 feet of the working surface, to ensure that air pressure does not exceed the recommended values provided in Table 1.

Table 1: Recommended Air Pressure Values by Depth

Depth Below Working Surface (ft)	Allowable Air Pressure at Compressor (psi)
0 to 10	Do not apply air pressure
10 to 20	75
20 to 30	115
30 to 40	145
40 to 45	160
>45	170

However, in controlling the risk for hydraulic fracturing, mitigation is closely tied to the presence of a skilled drill rig operator and experienced crew to observe, anticipate, and react to any conditions as they arise during the normal construction process and adjust equipment controls accordingly. As such, a conservative approach to suspend the usage of air pressure in the top 40 feet was adopted on the project after a minor discrepancy in the field.

3. Impact on mixing index due to a broken paddle on the auger

DMM creates a soil-cement cutoff wall by injecting cement-bentonite grout into soil and mixing it in place with rotating auger paddles. A Mixing Index equation is used for quality control, whereby a theoretical target is established during preconstruction for comparison against production data. As outlined in the project specifications, the Mixing Index for Wet Rotary Shaft (WRS) method is to be calculated as follows:

$$I_m = M\,(N_p/V_p + N_w/V_w)$$

Where M is the total number of mixing blades, N_p and N_w are the rotational speed of axis during penetration and withdrawal in rev/min, and V_p and V_w are the velocity of the mixing apparatus penetration and withdrawal.

A target Mixing Index of 156 rev/ft for the full length of every element was established.

Significant drilling resistance at element N-151 between the depth of 88 ft and 103 ft was noted. An obstruction (e.g. dense gravel, cobble) caused loud drilling noise and chattering, and resistance resulted in the breaking of an auger paddle in column 2, just above the drill head. The paddle section was replaced after completion of element N-153. Three elements were installed with the broken paddle: N-151, N-154, and N-153. After completing element N-153, the contractor replaced the mixing paddles and ended production for the day.

The established Mixing Index of 156 was not met for the full lengths of the three elements mixed with a broken paddle. Figure 4 is a plot of the contractor's calculated Mixing Index versus depth for the three elements installed with a broken center auger paddle. As Figure 4 shows, the target Mixing Index was not met in the upper twelve feet or bottom three feet of the elements.

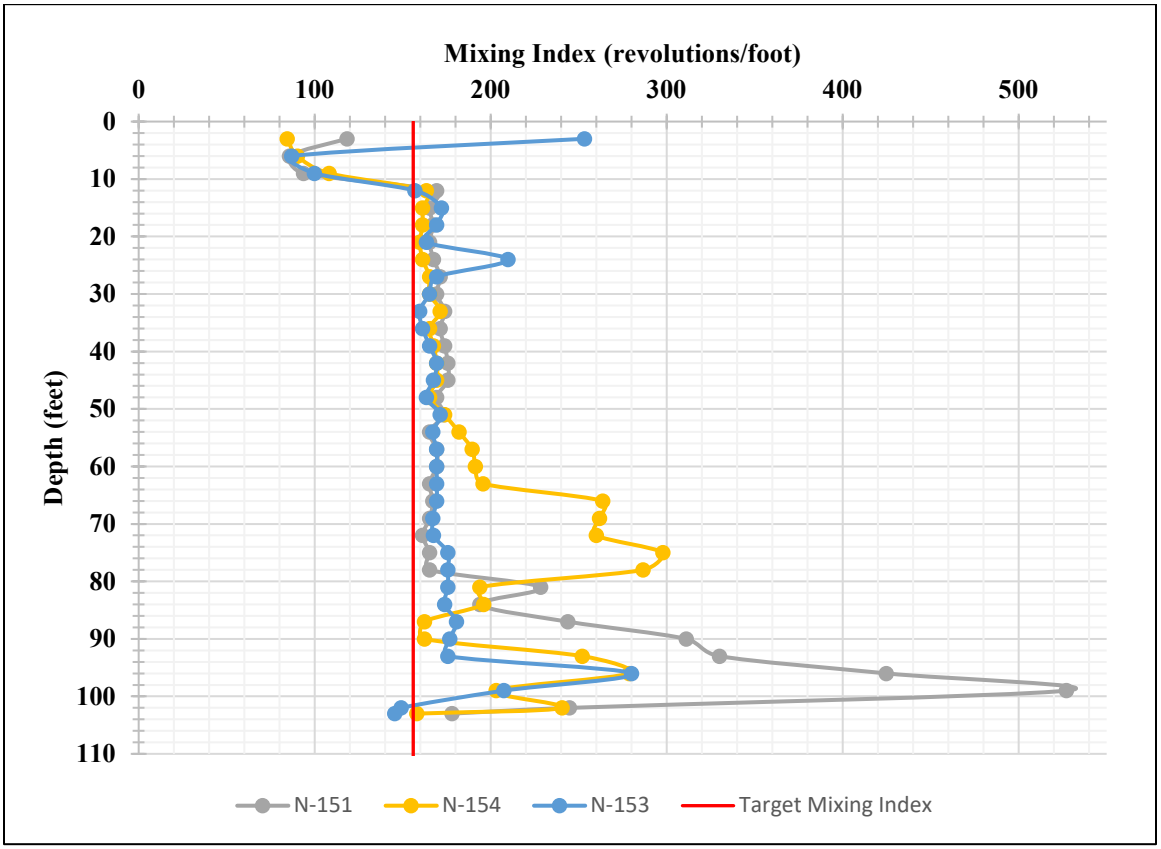


Figure 4: Broken Paddle Element Depth vs Mixing Index (Contractor Calculation)

As part of its analysis, to calculate the Mixing Index for the three elements installed with a broken paddle, USACE developed an advanced spreadsheet tool for more accurate calculations than the contractor's method. This study concluded that the mixing efforts for the three DMM-elements (N-151, N-153, and N-154) installed using damaged mixing equipment were insufficient. Figure 5 is a plot of USACE's calculated Mixing Index versus depth for the three elements installed with a broken center auger paddle.

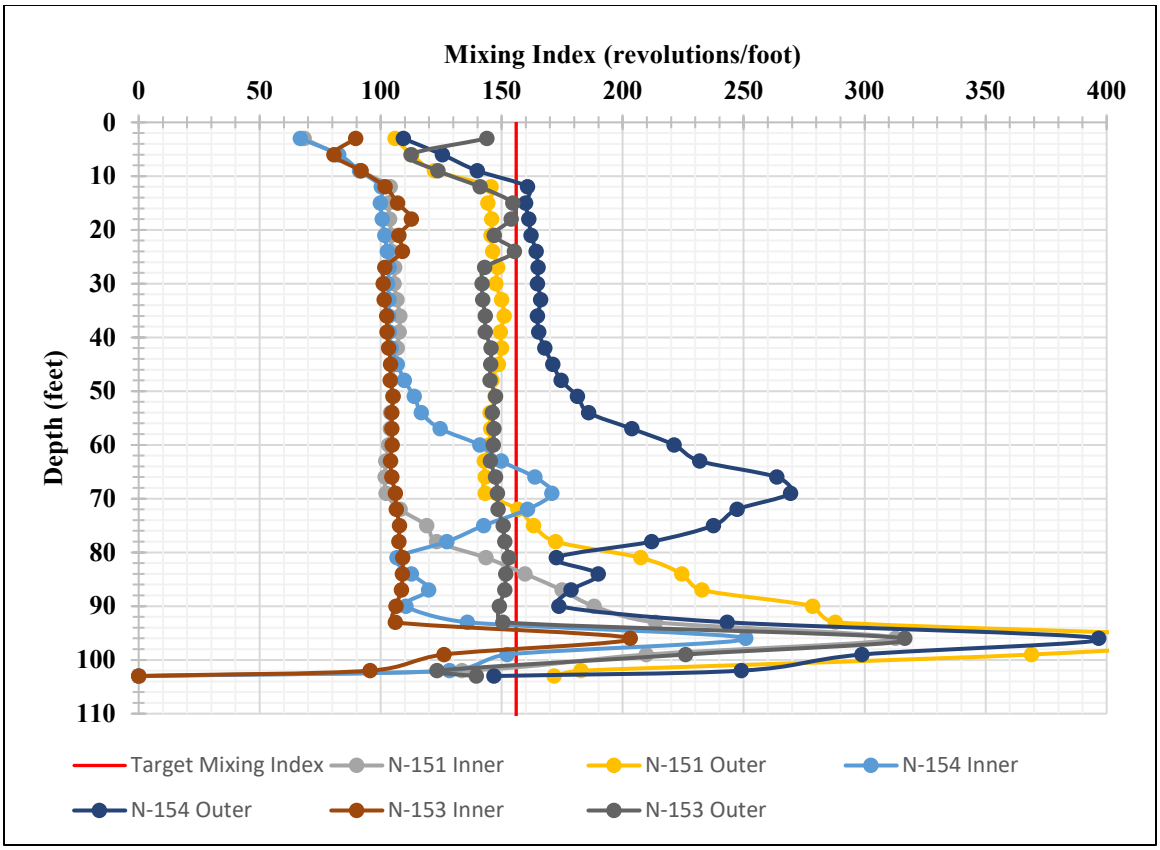


Figure 5: Broken Paddle Element Depth vs Mixing Index (USACE Calculation)

As Figure 5 shows, there were several zones of inadequate Mixing Index. Failure to meet the established Mixing Index may result in inadequate performance of the cutoff wall. Other performance criteria, such as wet bulk sample permeability and unconfined compressive strength were used as criteria for cutoff wall acceptance. The cutoff wall was accepted as is, despite there being portions of wall not meeting the established Mixing Index.

4. Tie-in locations of different cutoff walls

When constructing a new cutoff wall using different construction methods, it is important to include an appropriate overlap between the cutoff wall types. The length of overlap can vary depending on the type of overlapping walls. For example, overlaps between two SB walls would typically be about 30 feet and overlaps between cemented walls (e.g. DMM and jet grout) would be about 15 feet. When a soft wall like SB overlaps with a firm wall like DMM, the sequence of construction is an important aspect. Typical the waterside space of the cutoff wall alignment is utilized for placement of rig for firm walls, and for slurry mixing ponds in case of soft walls. The waterside slurry mixing ponds could create softer surface on the working platform. Based on our experience from SREL Contract 3 Project, it is best to have the firm wall constructed first in order to have stable foundation conditions under the rig. If the firm wall is constructed after the soft wall, the rig constructing the firm wall will have to be located on these softer soils created by mixing ponds at the tie-in locations. These conditions create a stability risk for the rig. This kind of situation occurred in SREL Contract 3 when a DMM wall overlap is being constructed into a recently constructed SB wall.

The DMM rig constructing a landside overlap to a recently installed SB wall. It was soon noted that the weight of DMM rig was causing a cutoff wall side slope to fail. This is due to the existence of soft lead-in trench of the already installed SB wall on the water side. USACE and the contractor discussed the trench cave in and identified it as a DMM overlap issue with lead-in trench for the SB Wall. USACE and Contractor agreed to realign the DMM wall to the waterside of the SB alignment, to utilize the stable landside foundation for the DMM rig. The sloughing was

approximately 6 ft parallel and 10 ft in perpendicular with the cutoff wall. This alignment change could have been avoided if the DMM wall had been constructed before the SB wall.

Other than risks of alignment change and stability concerns, it is best to have a firm wall in place before a soft wall, because having a firm wall tie into a soft wall would change the properties of the firm wall over the overlap section. For example, DMM constructed in virgin soil will be much stronger than DMM constructed in a soft SB backfill. Normally, the low strength wall would connect well to a higher strength wall (e.g. greater than 200 psi unconfined compressive strength). The properties of each wall would be separate and distinct in the overlap region. When a high strength wall is installed into a low strength wall, the overlap section would be a blended material that has a strength somewhere in the middle of what is typical for the two different wall types. This was proven in SREL Contract 3 during the testing of SB-DMM mixture. Permeability of the two material types is similar, though the blended material tended to have a slightly higher permeability. The test results are shown in Tables 2 and 3.

Table 2: Unconfined Compressive Strength test results of DMM and SB-DMM Blend samples for two DMM elements.

	Unconfined Compressive Strength (psi)			
	Element 1		**Element 2**	
Cured Age (days)	**DMM**	**SB-DMM Blend**	**DMM**	**SB-DMM Blend**
7	316.68	56.63	146.03	88.77
14	398.67	70.49	216.37	159.50
28	628.87	74.11	259.01	72.80

Table 3: Hydraulic Conductivity test results of DMM and SB-DMM Blend samples for two DMM elements.

	Hydraulic Conductivity (cm/s)			
	Element 1		**Element 2**	
Cured Age (days)	**DMM**	**SB-DMM Blend**	**DMM**	**SB-DMM Blend**
7	3.47E-08	1.31E-07	9.29E-08	2.75E-07
14	3.87E-08	5.34E-08	5.15E-08	5.32E-08
28	3.10E-08	7.17E-08	3.58E-08	9.65E-08

For projects with two new cutoff walls of differing material and construction method, it is generally preferred to construct the softer wall first. However, the softer wall may need to be constructed first for scheduling or other reasons. If this happens, the alignment may need to be changed, and the overlap material might have different properties than either of the walls. Figures 6 and 7 show how the alignment changed for SREL C3.

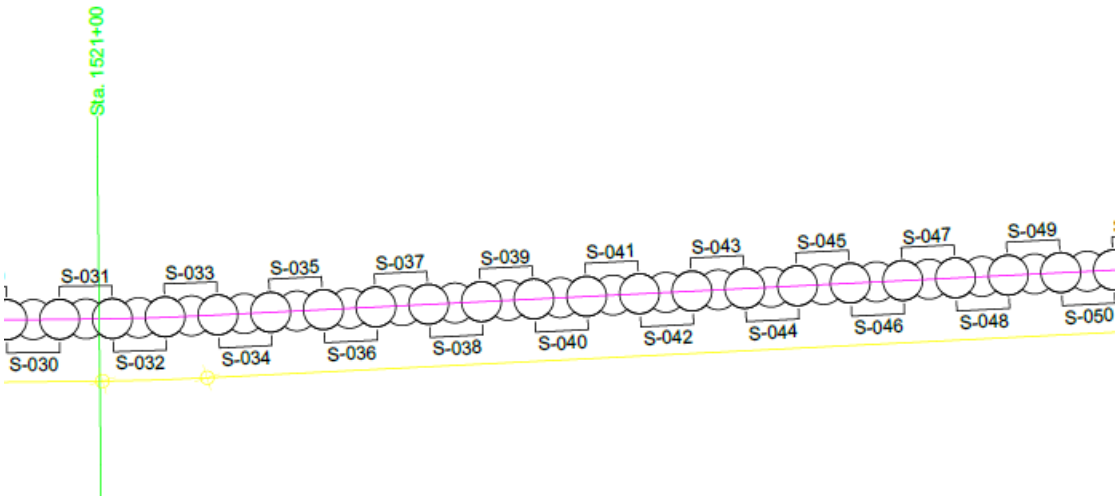


Figure 6: Initial DMM Layout

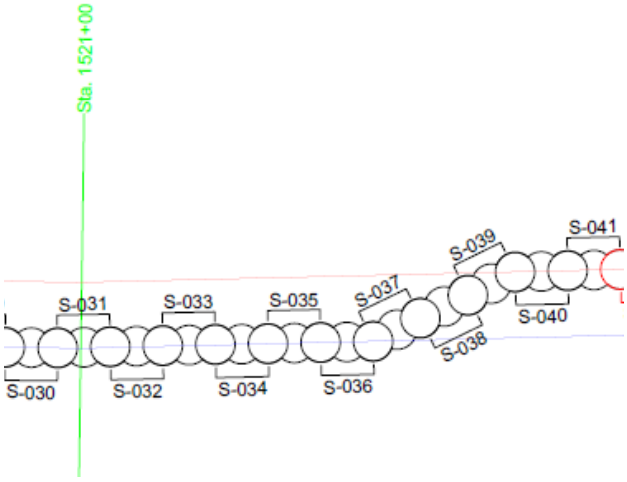


Figure 7: Adjusted DMM Layout

5. Importance of cut-off wall tie-in layer

The cutoff wall tip elevation was determined using existing explorations performed along the levee alignment during the design phase of the project. The existing explorations were spaced up to 1,000 feet apart along the levee crown. Where DMM cutoff walls were proposed, additional explorations were performed to achieve a 500-foot spacing in between explorations to confirm the fine-grained (low permeability) tie-in layer since the DMM method does not allow for the fine-grained tie-in layer depth to be confirmed. Based on the existing explorations and the explorations drilled for the DMM cutoff walls, a fine-grained tie-in layer was encountered in the northern half of the site and starting about 1,500 feet upstream from Sump 132.

Where a tie-in layer was encountered, the cutoff wall tip elevation was selected so the cutoff wall would “tie-in” to the fine-grained layer 5 feet. Cutoff walls that tie-in to a fine-grained layer is considered a “fully-penetrating” cutoff wall and essentially cuts off any flow from the waterside to the landside. Where the tie-in layer was not encountered, the cutoff wall tip elevation was selected. Cutoff walls that do not tie-in to a fine-grained layer are considered a “hanging” cutoff wall and lengthen the seepage path of flow from the waterside to the landside.

During construction of the open-trench cutoff walls (SB and SCCB), the QC trench logger and the QA trench logger logged the soil being excavated from the cutoff wall trench. Where fully penetrating open trench cutoff walls were

designed, it is important for the QC and QA logger to determine at what depth the fine-grained layer is encountered. If the fine-grained layer is opened deeper than anticipated, the 5 feet of tie-in will not be achieved. During construction, there is little time between excavating the last 5 feet of the cutoff wall and moving to the next panel of the cutoff wall. Therefore, it is important for the QA trench logger to properly log the excavated soils, note the depth of the fine-grained tie-in layer, and notify the client if the fine-grained tie-in layer was encountered deeper than expected.

6. Interpretation of trench sounding data

During construction of the SB cutoff walls, the soil-bentonite backfill slope was measured in the morning before work started and in the evening after work was completed for the day to monitor the progress of the header. The trench sounding was completed by the Contractor and noted by the QC and QA field representatives. A graphical representation of the trench sounding data would be helpful for interpretation and continuous overlay of daily data. Therefore, the trench sounding data was entered into a spreadsheet tool developed for this project to create a graphic to show the trench sounding data day after day. Figure 8 is an example of a sounding and excavation profile of a SB wall heading.

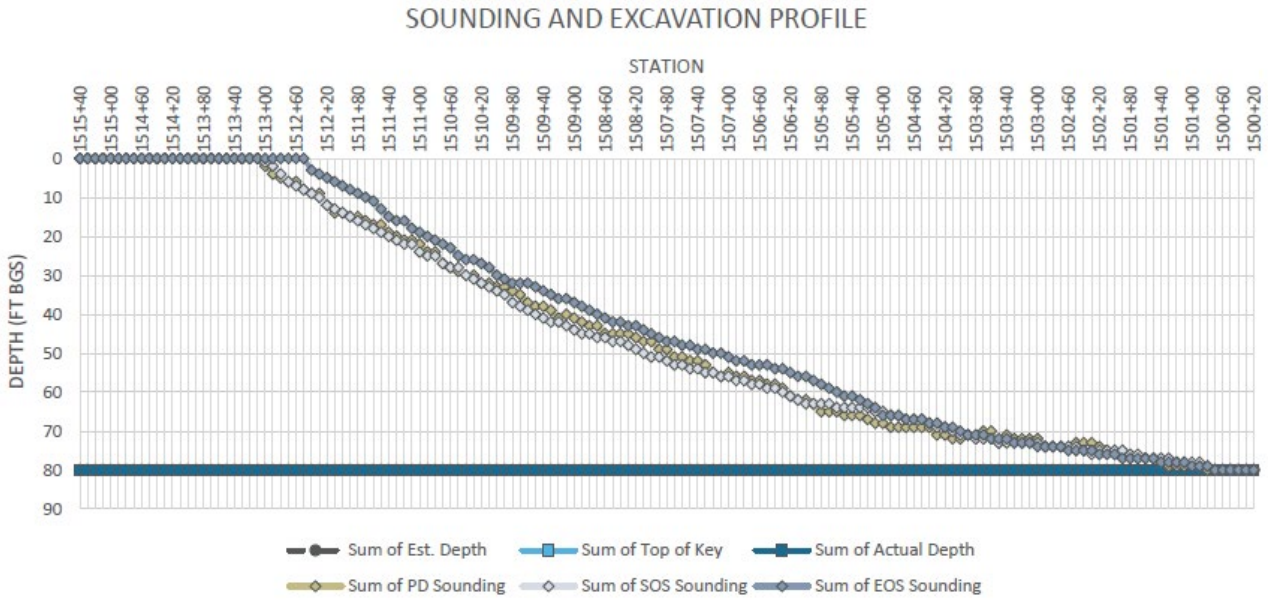


Figure 8: SB southern heading example sounding and excavation profile

Significant slope changes (i.e. substantial increase in the slope compared to previous days or spikes in the trench sounding at certain locations) were noted and can indicate a trench collapse or placement of low slump material due to inadequate mixing of the soil and bentonite in the mixing ponds. Such anomalies provide important information regarding the quality aspect of the constructed cutoff wall as shown in Figure 9.



Figure 9: Large blocks of unblended material in sample of SB backfill material

In addition, the location of the backfill slope with respect to the excavator progress can be ascertained. If the backfill slope toe gets too close to the excavator, then the excavator may “grab” the backfill slope toe.

These conditions can lead to non-uniform construction of open trench cutoff wall and therefore may not very low permeability performance criteria established in the specification.

7. Slurry leak – Research on History

A leak of bentonite slurry from the soil bentonite trench excavation occurred at Station 1507+00 during the overnight hours between June 21 and June 22, 2022 (Figure 10). When the contractor arrived on June 22, bentonite slurry was observed ponded against the silt fence and had overtopped the silt fence and flowed into the backyard of the adjacent residential property, where it had collected in landscape areas (fortunately, it did not flow into the swimming pool). When the contractor arrived to start work in the morning, it did not appear that the leak was active. Slurry cutoff wall excavation at 1507+00 occurred on June 16/17, 2022; therefore, the trench bentonite slurry was present in the trench six days prior to the leak occurring. The trench in this area was 80 feet deep with a measured depth of SB backfill at 74 feet (SB backfill depth of 6 feet) below the degrade surface. The trench bentonite slurry had a measured density of 74 to 85 pounds per cubic foot (pcf) and the SB backfill had a measured density of 114 to 115 pcf.



Figure 10: Bentonite Slurry Leak against Silt Fence and into backyard to right of pool area.

The contractor notified USACE construction personnel and QA staff and implemented their slurry leak mitigation plan. The location of the leak was identified and investigated. A vacuum truck was deployed to remove the slurry material from the backyard.

The contractor's bentonite slurry supply HPDE pipes ran through the leak area; however, they were intact and not found to be leaking bentonite slurry. Therefore, the bentonite slurry source would have to be from the open slurry trench. The area of ponded slurry was investigated for the leak source and a soft area was noted by hand probing near the levee toe at about Station 1507+10. The contractor used hand equipment to expose this area and small hole approximately 2-to 3 inches in diameter was exposed (Figure 11). When exposed, a small amount of bentonite slurry flowed from the hole; however, it quickly stopped. The leak emanated from relatively clean sand (USCS classification SP or SP-SM) that appeared to be part of the side cast material placed by the contractor during levee degrade to provide a working platform. The thickness of side cast material was unknown but was at least several feet thick. It appeared that the bentonite slurry flowing through the leak had eventually sealed it. The area was left open and a sandbag dike was placed around the excavation. It was also noted that the area was near a walnut tree and the area was littered with walnut shells.



Figure 11: Bentonite slurry leak exit

Several potential seepage sources were considered.

1. **An abandoned pipe/utility** – There were no existing pipes identified in the area during design or found during slurry trench excavation. Therefore, this potential source was discounted.

2. **Hydrofracture** – The observed leak was observed to be about 8 feet below the bentonite slurry elevation in the trench and about 35 away from the slurry trench. The levee had been degraded by 4 feet to lower the hydrostatic pressures in the open trench. At the time of the leak, multiple headings of open trench slurry walls were being constructed for the project with a similar degrade without evidence of slurry leaks. The trench in this area was 80 feet deep with a measured depth of SB backfill to at 74 feet (SB backfill depth of 6 feet). The trench bentonite slurry had a measured density of 74 to 85 pounds per cubic foot (pcf) and the SB backfill had a measured density of 114 to 115 pcf. Therefore, hydrofracture was considered a low probability of being the source of the leak.

3. **Flaw in Embankment** – The leak site is adjacent to a walnut tree, which can attract rodent activity. When researching the history of the site, a study to investigate tree roots in levee embankments in this area was found. The study was the 'California Levee Vegetation Research Program Investigation of Tree Root Penetration Into A Levee Soil-Cement-Bentonite Slurry Cutoff Wall – Part II' (Harder, et. al, 2011). The study was investigating the potential for tree roots to penetrate the existing shallow soil-cement-bentonite levee cutoff wall that is present at the site. Findings of that report as it relates to the slurry leak are summarized below.
This study excavated a trench 60 feet long and between 8 and 9½ feet deep and was located approximately 5 to 7 feet landward of the alignment of the existing SCB wall. This study identified grouted burrows, likely from ground squirrels, in the walls of the trench that were 2-1/2 to 4 inches high and 4 to 12 inched in width. The burrows had been grouted by the California Department of Water Resources Sacramento Maintenance Yard as part of their rodent mitigation program. The grout has fragments of walnut shells indicating the ground squirrels had been harvesting walnuts. The nearest walnut tree is located at the slurry leak location. The trench was then backfilled by crews of the California Department of Water Resources Sacramento Maintenance Yard.

   Figures 12 and 13 show a cross section of the slurry leak area and an aerial view of the site and tree root investigation trench.

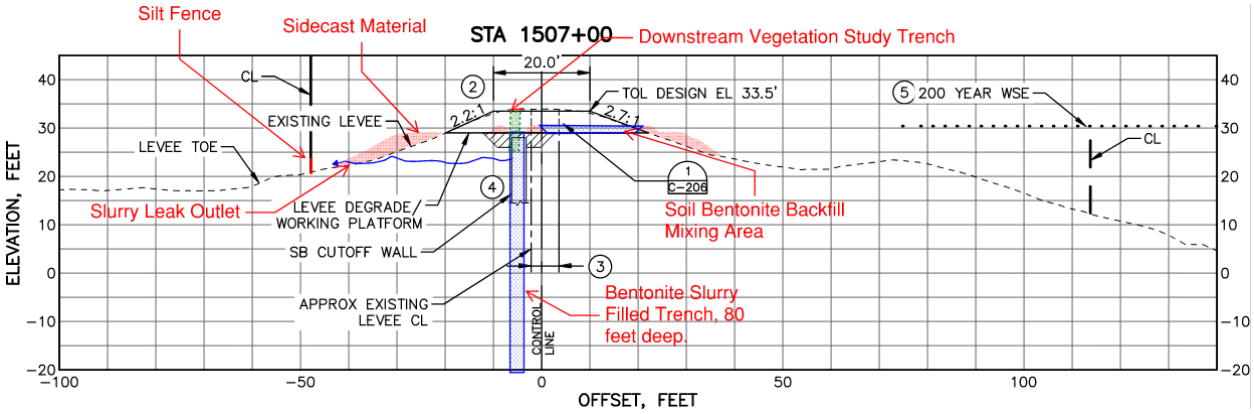


Figure 12: Section at Station 1507+00 showing site features.

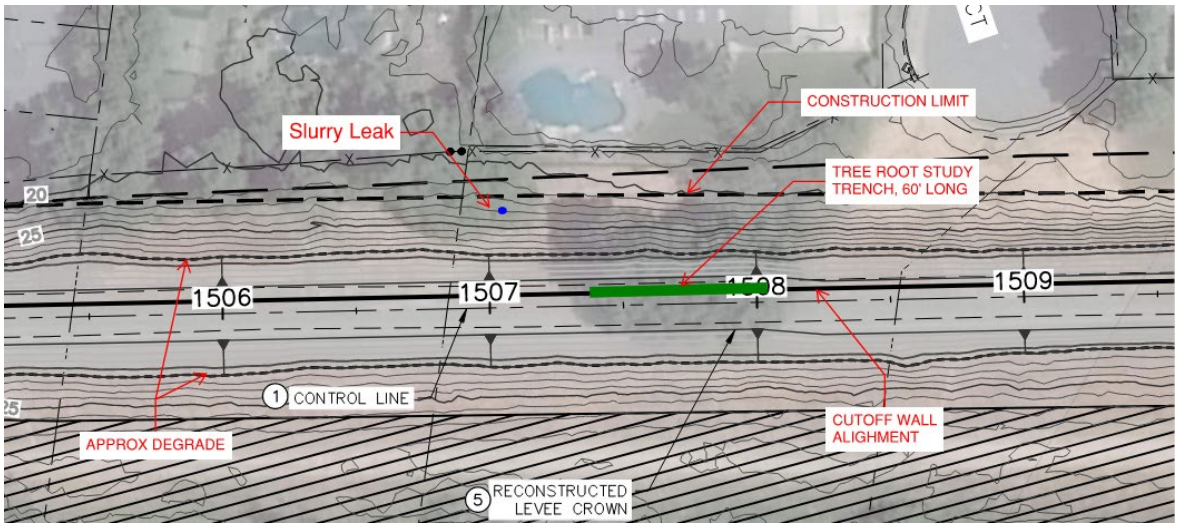


Figure 12: Slurry leak location

SUMMARY

The trench bentonite slurry was present in the trench 6 days prior to the leak occurring, and after the leak occurred, it self-sealed and did not reoccur. Since other slurry leaks did not occur at the project site, it is unlikely hydrofracture alone was the cause of the leak. As discussed above, rodent burrows are present in the levee at this location. Therefore, it is likely that the slurry leak is the result of the trench either encountering a partially grouted burrow and the leak enlarging the burrow over time until it found an outlet or burrows causing a weak area in the levee that allowed small hydrofractures to develop over time until the bentonite slurry found an undetected rodent burrow that exited near the landside toe.

Since the trench slurry leak was likely due to a flaw in the levee embankment and not hydrofracture through the competent levee, construction was allowed to continue the next day. Soil bentonite backfill reached this area on July 11, 2022. No other slurry leaks were encountered at the slurry leak area or at the project. During levee reconstruction after completion of cutoff wall construction for the project, the slurry leak area was excavated and replaced with compacted levee embankment fill.

8. Quality control issues – Field QA & QC Coordination and Cutoff wall removal

Coordination between the Contractor's quality control representatives (QC) and the Government's quality assurance representatives (QA) is important to the success of construction. QC is responsible for developing and executing a QC program that demonstrates the project features are constructed per the plans and specifications. QA is responsible for performing a percentage of the QC testing to confirm QC is performing the testing per the specifications. The SREL project team relied on weekly QA-QC coordination meetings and project feature of work preparatory meetings to discuss field and laboratory testing procedures and results. Topics discussed included:

- Coordinating sample type and size (confirmation no variation from specifications)
- Confirmation of testing methods
- QA and QC results –
    - Data trends to inform future construction
    - Tests not meeting specifications and corrective action
    - Variation in testing procedures that could produce variation between QA and QC results

Some examples of construction issues that were identified because of the QC and QA coordination are described below.

Example 1:

Samples of slag-cement-portland-cement-bentonite (SCCB) cutoff wall were casted into cylinders for hydraulic conductivity (ASTM D5084) and unconfined compressive strength (ASTM D2166) testing. During stripping the molds, QA noted voids in the samples (Figure 14). During the QA-QC coordination meeting, the voids were discussed and suggested causes included potential damage during transportation and improper sample casting.



Figure 13: Examples of poor quality SCCB sample cylinders provided to the QA for testing

Sample transportation was eliminated as a potential cause due to lack of cracking and the presence of voids being present throughout the sample. Transportation procedures by QA and QC from the site were also reviewed. Transportation of samples followed specified method ASTM D4832. Samples were transported on the fourth day after casting, which the material was set not allowing deformation to form voids. Once it was concluded transportation was not a factor the sample casting procedures were reviewed, and it was found the specified procedures were not consistently being followed. This was immediately corrected, and voids were no longer observed in subsequent samples.

Example 2:

As fill material index testing was trending towards the specification limit, the Contractor was asked to provide what steps would be taken to reverse the trend. The Contractor stated they would perform more blending at the borrow site before importing the material. USACE was able to confirm whether the Contractor's methods were successful by monitoring the laboratory test data going forward.

9. Overcome the varying stratigraphic conditions

Efforts to reconstruct the levee to project grade after completion of the 2-mile long cutoff wall encountered varying stratigraphic conditions. The conditions varied from freshly constructed cutoff wall to sandy surface conditions. Customized solutions were used to achieve the required field density on the first lift over the degrade without damaging the top of the cutoff wall. Few of these are discussed below.

Bridge Layer over Cohesionless Subgrade

On Contract 3, it has been observed that compaction of the first lift of select fill over a cohesionless subgrade is extremely difficult to achieve. A lift of select fill was required as a bridge layer over the sandy subgrade. The select levee embankment fill was placed over cohesionless soil subgrade in two 8-inch loose lifts contrary to using a foot of fill. Each loose lift was compacted with at least 6 complete passes of a tamping roller or equivalent compaction equipment. Each compacted lift of soil was probed with a soil probe upon completion of compaction using moderate pressure. If soil probe penetration is less than one and one-half inches, the layer is compacted layer was considered accepted. Otherwise, additional 4 complete passes were completed, and the compacted soil was probed again. The compacted soil was observed to make sure that no deflection under standard construction traffic wheel loads occurs following completion of compaction.

Bridge Layer over Soil-Bentonite (SB) Cutoff Wall

The initial bridge layer to be placed above the SB wall would be critical as the wall would not be set. A thicker fill layer may not achieve the required compaction while a thin layer might damage the top of the wall. Based on the condition of the SB wall, an initial 24-inch thick lift of select fill was used over the SB cutoff wall and compacted by 8 passes and then a 6-inch lift compacted by 6 passes in lieu of the two equal 15-inch thick lifts.

Bridge Layer over DMM and SCB Cutoff Wall

Initial backfill over SCCB and DMM cutoff walls was initiated after excavating the upper 1-foot of cutoff wall below the working surface to remove any possible contamination. The select fill was used to backfill the excavated depth, moisture condition and compact in lieu of placing a two feet fill to avoid any shortcomings on the required compaction effort.

10. Collaborative team environment approaches

The success of a project greatly relies on the collaboration and prompt communication of the construction team. The SREL project was a joint effort by USACE construction team members and USACE engineering. QA team supported the USACE engineering team to provide onsite observations of the construction and also included quality assurance field and laboratory testing. The QA team's understanding of the design aspects of the project helped to understand the design intent and helped to identify potential construction issues to both USACE teams promptly.

The keys to the success of the QA and USACE teams were in the following:

- Clear understanding of each team's roles and responsibilities

  A clear understanding of each team's roles and responsibilities establishes the chain of command and who to notify and how. Establishing an organization chart at the beginning of the project prior to construction beginning is critical to ensure prompt and complete notifications which issues arise and also when they are resolved.

- Weekly meetings to discuss construction issues identified in the field to confirm resolution

  Weekly meetings to discuss construction issues identified in the field to confirm resolution is important. The meetings were a way for the teams to come together to discuss onsite observations, provide an understanding of any outstanding concerns, and compile a list of follow-up action items.

- Group text message with QA, USACE Construction, and USACE Engineering to provide prompt and consistent communication of levels.

  The group text message with the QA, USACE Construction, and USACE Engineering team leads supported prompt and complete notifications generally 'in-real-time'. This ensured the key team members were notified. The best way to resolve construction issues is as soon as possible and right after they are identified.

- Follow-up documentation of issues that could not be immediately resolved on site.

  Documentation of the construction issues is important as the construction progresses to provide an understanding of when and how the issues were resolved. This documentation is a log of issues and was discussed at the weekly team meeting. The methods used by USACE and QA to collaborate as a team contribute greatly to the success of the project.

CONCLUSIONS

Challenges are a normal part of every construction project. The SREL Contract 3 project had challenges with unanticipated site conditions, equipment issues, implementation and integration of 3 different types of cutoff walls, working in an urban environment with limited access, and residential backyards that backed up to the construction limits. Additionally, the work had to fit into a very tight construction window and had to be completed before the

winter flood season. As discussed in this paper, many issues occurred during construction that required a quick response by the team. USACE, KSJV, and the contractor worked closely to analyze the issues, develop solutions, and implement them. By maintaining a collaborative work environment and close communication between the contractor, and the design, quality assurance, construction engineering, and construction oversight teams, the SREL Contract 3 project was successfully completed.

ACKNOWLEDGEMENTS

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The project could not have been completed without the hard work and effort by Maloney Odin, Joint Venture, the prime contractor, and Nordic Industries and Raito, Inc., their major subcontractors.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

Case Study: The Effects of Hot Temperatures on Deep Mix Method Seepage Barrier Construction

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ABSTRACT

This paper presents a case study of hot weather impacts on cementitious cutoff wall seepage barrier construction. For a recent Sacramento River project, Deep Soil Mixing (DSM) Deep Mix Method (DMM) was deployed which uses a grout slurry mixture of Portland cement, bentonite, and water. Slurry is injected into the ground and mixed with the surrounding soil creating a soil-cement-bentonite cutoff wall. Specifications for maintaining the grout mixture under 90°F were developed with concern for the potential negative effects on strength and permeability of the cutoff wall. Cement grout applications are typically required to adhere to American Concrete Institute (ACI) code and ASTM standards. However, there are no applicable codes that fully address the use of cement in seepage barrier cutoff walls. The data obtained from this case study does not support the need for temperature control requirements of soil-cement-bentonite grout mixtures during construction.

I.INTRODUCTION

Project technical reviewers for a Sacramento River project submitted specification modifications to require cement-bentonite grout temperatures to always remain between 35°F and 90°F, and that the grout does not begin its initial set before it is injected into the ground. The reason for this specification adjustment is for concern about the negative effects temperature may have on strength (Burg,1996) and permeability. During the construction of DMM cutoff walls in June 2022, the project quality assurance (QA) team reported grout temperatures at the mixing plant were greater than 90°F. Several DMM elements were constructed with grout that reached recorded temperatures between 91.3°F and 98.7°F. Grout temperatures were recorded by QA field engineers with a digital infrared thermometer directed from a safe distance of about two feet into the mixing tank.

A DMM cutoff wall test program was developed at locations where high temperature grout was recorded. The evaluation included literature review on the subject, review of standards related to hot weather concrete construction, performing a test program which included coring and testing of the DMM elements that did not meet temperature specification requirements, review of relevant QC/QA test results, and consideration of the Contractor's performance on similar projects.

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II.DMM INSTALLATION

DMM cutoff walls consisted of overlapping soil-cement-bentonite elements extending to depths between 90 and 137 feet below construction grade. DMM is an in-situ soil treatment in which soil is mixed with cement and bentonite grout slurry to create a series of elements that form a continuous seepage barrier. This seepage barrier required a minimum compressive strength of 200 pounds per square inch (psi) at 28 days (ASTM D2166) and bulk sample permeability of 5×10^{-7} cm/s at 28 days (ASTM D5084). The specialized three-auger equipment consists of a crawler base rig with three overlapping cutter heads and mixing paddles. The triple shafts are secured to a frame that penetrate the ground vertically (Figure 1). The cement-bentonite slurry injection and soil mixing procedure is carried out while advancing the augers down. Once the design depth is reached, the shaft is raised 10 feet, lowered again, and the augers are withdrawn in stages while rotating and re-mixing at speeds based on the approved mixing index.



Figure 1: DMM Rig Installation

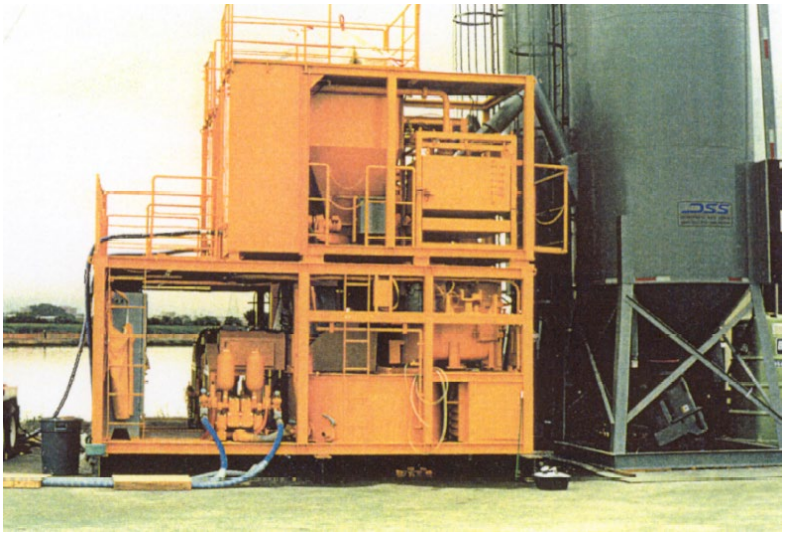


Figure 2: Batch Plant Setup

The on-site grout batch plant consists of storage silos, water tanks, batching scales, grout mixer, agitation tank, and a computer control unit (Figure 2). The grout is mixed at the batch plant and delivered through hoses to the DMM rig. The hoses are typically partially buried under loose soil to limit direct sun exposure where possible. According to the project's implementation plan, the maximum time the grout is held in the agitation tanks is approximately 3 hours. However, the Contractor indicated the batch plant only produces grout in 1,800-liter (2.4 yd^3) batches, resulting in typical holding times between batches on the order of 3 to 10 minutes to supply the rig. The grout is typically held in the hoses for up to approximately 35 minutes during the withdrawal phase, for approximately 10 to 15 minutes during the stem extension process, and 10 to 15 minutes during the movement of the rig between elements.

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III.CONCRETE CODE REQUIREMENTS FOR TEMPERATURE

American Concrete Institute (ACI) 305R (2020) defines hot weather as one or a combination of conditions that tend to impair the quality of concrete by accelerating the rate of moisture loss and rate of cement hydration. Such conditions include high ambient temperature, high concrete temperature, low relative humidity, and high wind speed.

Potential problems of placing concrete in hot weather include increased water demand, increased rate of slump loss, increased rate of setting, increased tendency of plastic shrinkage and increased thermal cracking. Potential material quality deficiencies to concrete placed in hot weather include decreased strength resulting from higher water demand, increased tendency for drying shrinkage and differential thermal cracking, decreased durability resulting from cracking and greater variability of surface appearance.

ACI 305.1 (2014) recommends 95°F as the maximum temperature for fresh concrete at the time of discharge. Temperatures greater than 95°F may be allowed but should supported by field experience under similar conditions, materials, proportions, temperatures, and delivery times. Acceptance should also be supported by preconstruction testing under similar conditions as production. ASTM C94 (2020) recommends stricter maximum temperature of 90°F which is the basis for the project specification requirement.

IV.FIELD DATA

Element T was installed on 21 June 2022 to a depth of approximately 137 feet. The QA team observed and recorded grout temperatures during installation of this element between 92.6°F and 98.7°F. The highest ambient temperature for that day was recorded at 105°F. Table 1 presents the grout temperatures measured at the batch plant, in the trench during construction, and at the sampler box. The sampler box is a specialized piece of equipment that can be lowered to desired depths to collect bulk samples of the mixed material. Temperatures were collected using an infrared thermometer since it was deemed unsafe to use a concrete temperature probe within the batch plant mixing tank and near an actively advancing DMM rig.

TABLE 1

Temperature Records for Element T Not Meeting Specification (90°F)

Installation Date	Time	Measurement Location	Grout Temperature (°F)
21 June 2022	13:30	Batch Plant	92.1°
	13:40	Batch Plant	92.8°
	15:30	Trench	96.7°
	15:30	Sample Box	98.7°
	15:30	Batch Plant	92.6°

To evaluate temperature-related negative effects on strength and permeability on the DMM cutoff wall, the following tests were performed:

- In-Situ Testing
  - Drilling and coring of one borehole in accordance with ASTM D2113 (2014)
  - Borehole caliper logging
  - Optical and acoustic televiewer survey
  - Constant head permeability test, Stage 1 (above the groundwater table)
  - Rising head recovery test, Stage 2 (below the groundwater table)
- Laboratory Testing
  - Unconfined compressive strength in accordance with ASTM D2166 (2016)
  - Hydraulic conductivity in accordance with ASTM D5084 (2016)

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The test results are presented in Table 2. In general, DMM cores are not tested for unconfined compressive strength (UCS) or permeability since coring is considered a destructive test method that could conceivably create fractures within the sample which would not be representative of the in-situ cutoff wall. In the spirit of collecting and providing as much data as reasonably possible, the test results from the cores are also provided below in Table 2.

TABLE 2

Element T In-Situ and Laboratory Test Results Summary

Sample/Test	Depth	Sample Date	Test Date	Age (days)	UCS (psi)	Permeability[1] (cm/s)
Core	9’ 10”	18 Aug 22	20 Sep 22	91	--	1.48 E-6
Core	10’ 5”	18 Aug 22	20 Sep 22	91	458	--
Core	20’ 4”	18 Aug 22	20 Sep 22	91	--	4.14 E-6
Core	20’ 9”	18 Aug 22	20 Sep 22	91	313	--
Core	40’ 10”	06 Sep 22	20 Sep 22	91	402	--
Core	63’ 9”	06 Sep 22	20 Sep 22	91	--	1.17 E-6
Bulk	90’	06 Jun 22	19 Jul 22	28	663	1.42 E-8
Constant Head Permeability Test	21’ 7”	--	20 Aug 22	60	--	5.72 E-8
Rising Head Permeability Test	128’ 11”	--	20 Aug 22	87	--	1.90 E-5

[1]Project Specifications require Bulk sample permeability to be less than or equal to 5.0E-7 cm/s and in-situ requirements less than or equal to 1.0E-6 cm/s.

V.TYPICAL CONSTRUCTION CONDITIONS

Levee improvements in the Sacramento region are typically performed outside of the flood season which is April through September. This results in most levee construction work being performed during the hottest months of the year. Ambient air temperatures in summer often reach over 100°F. The DMM Contractor had successfully completed several DMM cutoff wall projects in the Sacramento area between 2018 and 2022.

A DMM cutoff wall constructed in 2019 extended between 90 and 135 feet below construction grade. According to the End of Construction Summary Report, all bulk samples tested as part of the quality control program met the required permeability of $\leq 5\times10^{-7}$ cm/s at 28 days or sooner, one sample tested less than 200 psi at 28 day, and spare samples at later ages resulted in lower UCS values. The Contractor indicated lower UCS values were due to casting, handling, or transportation error and not indicative of material quality which was supported by strength gain over time. 40 bulk samples were cast from elements installed on days with ambient temperatures between 90°F and 109°F. However, grout temperatures for those bulk samples were not collected or recorded.

VI.CONCRETE CODE RELEVANCE

ACI specifications and recommendations may not be appropriate guides for the DMM system because of several key differences between typical concreting and DMM cutoff wall application:

- Strength requirements: structural and flatwork concrete typically require a minimum compressive strength of 2,500 psi, but the DMM cutoff walls minimum requirement is 200 psi at 28 days. This significant difference makes strength gain less critical. Also, higher compressive strength concrete requires larger quantities of cementitious materials which results in higher heat of hydration when compared to lower strength concrete.
- Exposed surface area: hot concreting can be a significant issue for concrete exposed to high winds, high temperatures, or low humidity, which can accelerate hydration rates, evaporation, and strength gain. All of

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which may result in lower ultimate compressive strengths, cracking, or decreased durability. However, the DMM cutoff walls are deep vertical elements surrounded by cooler subgrade soil and groundwater, with only a small portion of the wall near the surface exposed to ambient conditions.

- Off-site vs. on-site mixing: concreting recommendations usually assume off-site mixing conditions and delivery of concrete trucks that may need to be continuously rotated for up to 1½ hours. DMM grout is prepared on site and delivered through hoses within minutes, which is then pumped into the ground and mixed with in-situ soils.
- Water demands: some negative effects of hot concreting are related to the addition of water for workability. However, the DMM mix design is developed with the understanding the grout will mix with on-site soil and groundwater.
- Temperature differentials and Retempering: shrinkage, thermal cracking and water requirements are affected by high concrete temperatures and temperature differentials within the cross section of a concrete member. However, these issues are likely reduced for the DMM cutoff walls because mixing with in-situ soils and groundwater takes place as the augers are advanced, and mixing continues as the augers are withdrawn. The mixing method reduces temperature differential across the element and acts as retempering of the mix without needing additional water.

ACI and ASTM standards govern concrete construction but there are no state, national, or international codes for DMM cutoff walls. Therefore, engineering judgement and good understanding of the DMM system is necessary.

VII.SITE CONDITIONS AND CONSTRUCTION CONSIDERATIONS

The authors believe high ambient and grout temperatures have a greater impact on the upper portion of the DMM elements and therefore greater emphasis is placed on the potential effects on the mixed material closer to the ground surface. Site conditions and construction aspects believed to affect the upper portions of the DMM elements are described below.

The hoses from the batch plant to the DMM rig are black and remain empty for some part of the construction process. The hoses are sometimes covered with lose soil to reduce sun exposure, but during hot days, the hoses still get hot. As the grout starts flowing through the empty hoses, the heat from the hoses is transferred to the grout, increasing the temperature of the grout. As more grout is delivered, the hoses likely cool down and the effects on the grout are reduced.

Since the injection and mixing process is carried out from top to bottom, the first grout to reach the rig is mixed with soil near the ground surface. The grout, already influenced by the hot temperature in the hoses, is mixed with hot soil making the upper portion of the DMM elements especially susceptible to negative effects from high temperatures. Soils experience seasonal temperature variations, especially those closer to the ground surface. These variations are typically limited to depths of approximately 30 feet (Figure 3). In addition, groundwater at the site was measured at approximately 23 feet below the ground surface which limits possible cooling effects in the upper portions of the cutoff wall from groundwater.

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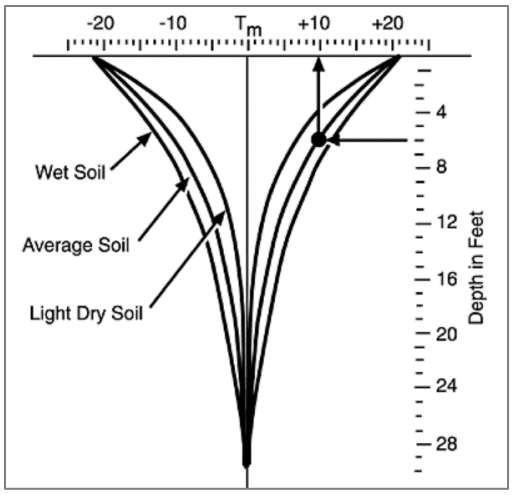


Figure 3. Seasonal soil temperature change as a function of depth below ground surface (Shi, 2020)

Flash setting is a concern especially for the upper portion of the cutoff wall. Flash setting is the rapid development of rigidity in freshly mixed concrete, typically resulting in the creation of heat. The setting cannot be reversed, and plasticity is not regained by adding water. Flash setting can result in early high strength but reduced strength gain over time. In addition to the higher temperature expected in the mixed elements closer to the ground surface as described above, these upper zones also need to remain workable for longer periods of time as the mixing start at the top and re-stroking is performed as the augers are retrieved. However, evidence of flash setting was not observed and all DMM elements were completed without early setting issues.

VIII.TEST RESULTS

UCS tests for wet bulk and core samples met the minimum acceptance requirement of 200 psi at 28 days. This includes results at depths between 10 and 90 feet below the ground surface which indicates the high temperature grout did not have a noticeable negative effect on the strength of the DMM cutoff wall.

In addition to coring, in-situ permeability tests were performed in two stages. Stage 1 is a constant head test which tested the upper 21.6 feet of the core hole. Stage 2 is a rising head test which tested the permeability of the core hole to the termination depth of 128.9 feet. Static groundwater was measured at a depth of about 23 feet. The Stage 1 constant head in-situ permeability tests also met the minimum acceptance requirements of 1×10^{-6} cm/s. Permeability tests performed on the wet bulk sample at 90 feet met the minimum acceptance requirements of 5×10^{-7} cm/s.

Three core sample permeability test results did not meet the project specification established for wet bulk samples but met requirements for in-situ permeability tests. There are no contract specification requirements for tests on core samples since coring is a destructive process and often leads to lower UCS and higher permeability as fractures propagate within the relatively fresh soil-cement-bentonite (SCB) mixture. It is also reasonable to suspect the core samples could be damaged during removal or transportation to the laboratory. DMM testing of cores is rarely performed unless there is a pointed reason or concern. QC/QA typically include bulk sample and in-situ testing, which are considered the standard of care.

The Stage 2 rising head test results did not meet the project specifications. The drillers noted difficulty coring below a depth 87 feet which is substantiated by poor or no recovery below 80 feet. The core hole bullseye graph (Figures 3 and 4) indicates the core hole migrated 1.8 feet from the initial start point near the center of the DMM element. A DMM element creates an effective 2.2-foot-wide seepage barrier using concentric 3-foot-diameter augers (Figure 5). A core hole can migrate up to 1.5 feet before “walking out” of the element, which means that the core likely penetrated the sidewall of the element and into the surrounding soil.

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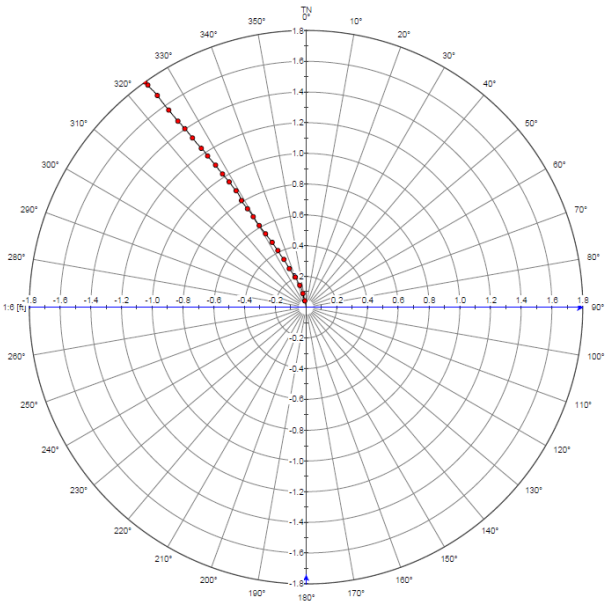


Figure 3: Core Bullseye Graph

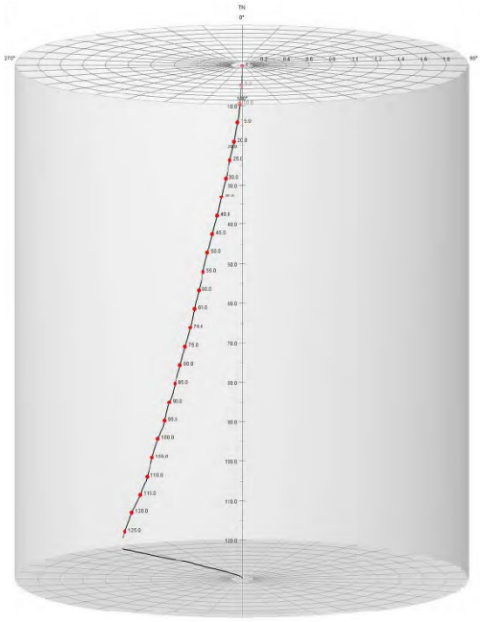


Figure 4: Core Offset vs Depth

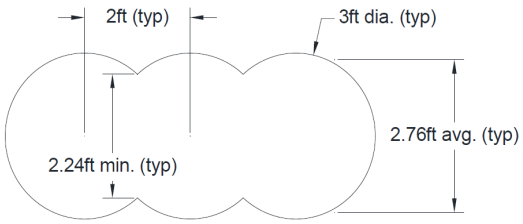


Figure 5: DMM Element Dimensions (Plan View)

Additionally, sediment was observed at the bottom of the core hole from 118 to 132 feet deep. This sediment was vacuumed out prior to performing the rising head test. These permeability results were carefully interpreted along with other data sources such as the caliper log, and the televiewer and acoustic survey reports.

The passing wet bulk sample permeability test at 90 feet conflicts with the rising head test result. We believe the wet bulk sample has a higher reliability since it is directly representative of the quality of materials within the DMM cutoff wall. In addition, the upper portion of the cutoff wall met all relevant project criteria for strength and permeability.

It is noted that limited testing was performed for this case study and only a few samples were selected for laboratory testing. Ideally, a robust sampling and testing program would provide a more complete picture of the potential negative effects of high grout temperatures on strength and permeability. Since the samples and testing were limited, this case study's statistical significance cannot be implied.

IX.CONCLUSIONS

The temperature concerns raised by technical reviewers regarding potential negative effects on strength and permeability of DMM cutoff walls due to high grout temperatures were found to be valid and worthy of additional evaluation. The primary concern was that cementitious slurry would be heated during the start of the injection phase and "flash set", only to be remixed again when the augers are withdrawn. The upper portion of the DMM SCB cutoff wall is of the highest concern for temperature sensitivity. The upper portion test results passed project requirements

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for wet bulk samples, cored samples, and in-situ constant head permeability testing. Based on the test results provided, the authors did not find strong evidence to support more rigorous temperature control requirements at the batch plant or during grout transport to the DMM rig for similar future projects.

X.RECOMMENDATIONS

Based on the evaluation of relevant literature, review of ACI and ASTM standards for hot weather construction, performing a test program that included coring and testing of the DMM element that did not meet temperature specification requirements, review of relevant QC/QA test results, and consideration of the Contractor's experience on similar projects, the following actions should be considered for similar DMM projects. These are not meant to be acceptance criteria, but rather to serve as data points for consideration in the event a project has unfavorable test results:

1. Measure and record ambient and grout temperatures as part of the quality control and quality assurance program. This will allow for a more complete evaluation of test results versus time, depth, and temperature.
2. DMM implementation plan should include consideration for casting mix design trial batches at a temperature that is expected to be the maximum experienced in the field and require the Contractor to stay below this threshold during construction.
3. As the potential effects of global warming become more obvious and weather or seasonal conditions change, the suggestions provided in this study will need to be re-evaluated.

XI.ACKNOWLEDGMENT

The authors would like to thank Hans Carota, Joe Sciandrone, Nikole May, Anthony Tran, and Erik James from USACE for supporting in-situ and laboratory testing programs and supporting initiatives to make improvements to existing specifications. The authors would like to thank Raito, Inc. for bringing Contractor's perspectives and practical experience from recent cutoff wall projects. The authors also like to thank USACE personnel, the non-federal partner agencies, consultants, and geotechnical laboratories involved in execution of projects that provided data for this study.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

Best Practices in Developing and Reviewing CFD Models for Spillways

BENJAMIN ISRAEL DEVADASON, P.E., CFM, AND PAUL SCHWEIGER, P.E., CFM. GANNETT FLEMING, INC.

ABSTRACT

Some dams have unique and complex spillway geometries that make the hydraulic design and analysis of those structures complex. For such dams, Computational Fluid Dynamics (CFD) modeling, a sophisticated approach to numerically solve hydraulics problems, is emerging as a great tool. As technology has been advancing and high performing computers are becoming more and more accessible, CFD modeling has become a powerful, economical, and therefore, popular tool in the hands of modelers and designers. While CFD usage is becoming more prevalent in the dam safety industry, many professionals without much prior experience with this tool may be tasked with developing and/or reviewing CFD models. This presentation aims to provide guidance on best practices for both parties; those developing and those reviewing CFD models. For modelers, it is important to know the significance of key model input parameters that impact the results and when sensitivity analyses to these parameters may be necessary. Such parameters, whose appropriateness is important to achieve reliable modeling, include three-dimensional model geometry, simulation duration, model boundary location, turbulence approximation, mesh cell size, and surface roughness. Modelers must also understand how to extract and report the output data necessary to verify and gain confidence in the model results. Reviewers of CFD models should have a fundamental knowledge of modeling objectives, fluid dynamics equations, turbulence models, numerical methods, mesh quality, model validation, and model parameters. To provide a thorough and informed review, it is crucial that modeling approaches, major assumptions, model set up inputs, sources of all boundary conditions data, mesh details, key geometry dimensions, design drawings, and model results are adequately documented. The authors will define and discuss each of the aforementioned aspects of CFD modeling using recent project examples from the perspective of both the modeler and the reviewer.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

Performance-based design of existing and future reservoir structures: An application of computational fluid dynamics modelling and machine learning techniques

SCOTT DUNN AND ANGELOS DIMAKOPOULOS, JACOBS ENGINEERING GROUP

ABSTRACT

Traditionally, structural design of reservoir structures, such as wave walls and crown walls, is performed by estimating return periods of the reservoir wave climate, isolating specific events of interest (say a 200-year wave event), and performing calculations of hydraulic and structural loads (forces and overtopping) by using simple empirical methods or more sophisticated methods, such as computational fluid dynamics (CFD) and/or physical modelling. This traditional approach to testing and confirming the design has been widely utilized; however, it has several drawbacks. Perhaps most important, setting the return period at the level of wave climate does not mean that the wave loadings or overtopping rates are subject to the same probability of occurrence: i.e., a 200-year-return-period wave event does not necessarily lead to a 200-year-return-period overtopping event. In addition, it is likely that a return period estimated today may be revised in the future due to climate change and/or collection of additional data. Tests or calculations that are based on isolated events may have limited future value; it is likely that these tests will have to be repeated, adding to the cost and duration of future engineering studies. This paper proposes a methodology to overcome these challenges. A workflow of CFD modelling and machine learning algorithms is used to estimate return periods for each individual hydraulic or structural load component by simulating more than 100 test cases and using these to train a machine learning application. The simulation output is presented in terms of return periods of wave overtopping and forces for various scenarios of relative sea level rise. The creation of the emulator also has the added benefit that the model can be used as a real-time forecasting system for overtopping or wave loading. The selection of the CFD model to generate the simulations database is preferred, due to the low cost of CFD compared to physical modelling and the relatively high fidelity of these models, which, when implemented carefully, may yield results of equivalent value to physical models.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

Showdown at Cowboy Dam

CHRIS THORNTON, P.E., COLORADO STATE UNIVERSITY; DAVID HUGHES, FRANK LAN, AND MICHAEL ZUSI, AECOM; AND STAN ALI, CONTRA COSTA WATER DISTRICT

ABSTRACT

The Los Vaqueros Reservoir Expansion (LVRE) Project is located on Kellogg Creek near Brentwood, California. Project objectives include raising the existing Los Vaqueros Dam by 58 feet, thereby increasing the reservoir's storage capacity approximately 70%, while utilizing the existing chute and stilling basin. In addition, a new High Level Outlet Works (sluice) is to be constructed on the left abutment of the dam and will discharge onto the existing spillway and the base of the spillway chute extension. These features, particularly the sluice exit utilizing the full width of the spillway chute, made the spillway and sluice unique, placing their design beyond the state of knowledge expressed in existing literature on dams.A physical hydraulic model, constructed at a 1:12 Froude scale, was designed and operated in conjunction with a computational fluid dynamics (CFD) model developed by AECOM to confirm and, if needed, adjust the design and hydraulic performance of the spillway and sluice conveying the probable-maximum flood (PMF) design discharge of 3,000 cubic feet per second. Moreover, a further objective of the study was to identify and verify the effects of adjustments to the design of the spillway and sluice. This presentation will summarize the system modifications and enhancements evaluated, and the benefits provided by combining a physical model with CFD simulations. Final design configuration and lesion learned with be presented.

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SEPTEMBER 17-21, 2023

Security and Resilience of Dams

KATE SCHWARTZER, PMP, CYBERSECURITY AND INFRASTRUCTURE SECURITY AGENCY

ABSTRACT

Effective security risk management to maintain operations and delivery of services depends on the ability of owners and operators to understand the risks to their facilities and operations and integrate a range of activities to manage those risks. This presentation will feature an overview of security risk, including examples of threats and adversaries, how security and safety intersect, the importance of reporting suspicious activity, and examples of ways to protect a dam against damage or loss caused by intentional acts.

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SEPTEMBER 17-21, 2023

Prioritizing a Portfolio using the Consequence-Based Top Screen

MICHELLE YEZIERSKI, P.E., AND J. SPENCER EVANS, CYBERSECURITY AND INFRASTRUCTURE SECURITY AGENCY

ABSTRACT

The Consequence-Based Top Screen (CTS) methodology identifies and prioritizes facilities within the Dams Sector that have the potential for the greatest impact to their surrounding area in the event of a failure or disruption. Originally launched in 2009 by the U.S. Department of Homeland Security, in coordination with Dams Sector partners, the Cybersecurity and Infrastructure Security Agency released a new web-based implementation tool in 2023. This presentation will feature an overview of the CTS methodology and new web-based tool for use by owners and operators.

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SEPTEMBER 17-21, 2023

Dam Security Panel Discussion

KATE SCHWARTZER, PMP, CYBERSECURITY AND INFRASTRUCTURE SECURITY AGENCY

ABSTRACT

Owners, operators, and government agencies all have an important role to play in ensuring the safe and secure operation of Dams Sector assets and the reliable continuity of their benefits. Join us for a panel discussion with subject matter experts—including State Dam Safety Offices, CISA as the Sector Risk Management Agency, ASDSO, and other organizations—to discuss lessons learned from security incidents, common security questions asked by owners and operators, and actions to take to mitigate security risk.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

Programmatic and Project-Specific Approaches to Using Dam Removal Projects as Dam Safety Solutions

MEGHAN A. QUINN, SERENA MCCLAIN, AND BRIAN GRABER, AMERICAN RIVERS

ABSTRACT

United States infrastructure is on the cusp of significant change. Expansive public infrastructure funding has been made available through the Infrastructure Investment and Jobs Act at a time when (1) the state of dam infrastructure continues to deteriorate due to an aging fleet; (2) the number of high-hazard-potential dams is doubling due to downstream development; and (3) more extreme weather patterns are emerging. Experts estimate that roughly $80 billion is needed to rehabilitate and/or remove many of the nation's non-federal dams most in need of attention.Dams, however, are not designed to last forever. A growing number of dam owners are opting to work with state dam safety offices and NGO partners to decommission and remove their dams, especially where the costs associated with liability, repair, and maintenance exceed removal costs. Once a radical notion, dam removal is now a common practice and a reliable solution for dam owners wrestling with weighing the liability and expense associated with dam ownership against the benefits provided by their dams.In this presentation, American Rivers will outline how to access funding to address aging, outdated dam infrastructure and develop multi-benefit dam removal projects that will appeal to a broader array of funding sources. We will examine how current efforts to inventory and prioritize dams based on a number of ecological and social factors can be leveraged to help connect dam owners with different funding options, as well as to groups that can assist owners in further developing a project. We will demonstrate how building collaborative partnerships with non-traditional stakeholders and exploring innovative financing options are one way to address outdated dams in this competitive funding environment.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

Wellsville Reservoir Dam Decommissioning: a HHPD Success Story

MELISSA MENEREY, CFM, AND MIA KANNIK, P.E., OH DEPARTMENT OF NATURAL RESOURCES, DAM SAFETY PROGRAM

ABSTRACT

Wellsville Reservoir Dam was built in 1926 to supply water to the Village of Wellsville which is located in eastern Ohio in the foothills of the Appalachian Mountains. Operation and maintenance responsibilities for the dam had changed several times over the years. In 2011, the Village of Wellsville found itself the owner and responsible party for a high hazard dam in poor condition. In 2016, the Village had to declare fiscal emergency due to lack of funds.Since 1973, the Ohio Department of Natural Resources, Division of Water Resources, Dam Safety Program (Division) conducted 8 dam safety inspections and the U.S. Army Corps of Engineers conducted a Phase I Inspection. These inspections repeatedly identified several serious issues with the dam: no Emergency Action Plan, insufficient discharge/storage capacity, seepage, questionable stability of the principal spillway, concrete deterioration, unlevel crest, and questionable operation of the lake drain. With prompting in 2017 from the Division, the Village was able to develop an Emergency Action Plan which showed that the Village itself would be impacted if the dam were to fail. After a dam safety inspection in 2019, the Division issued a Chief's Order to the Village, forcing the Village to implement interim risk reduction measures which include lowering the reservoir by 15 feet and filling erosion holes found in the spillway with concrete. When the NOFO for the 2020 Rehabilitation of High Hazard Potential Dam (HHPD) Grant Program included construction as an eligible activity, the Village and ODNR worked together to develop an application to render this dam nonhazardous. With funding help from FEMA and the Ohio Public Works Commission this dam was safely breached in the summer of 2022 and as a result the Village of Wellsville is much safer from a catastrophic failure of the structure. Join Division staff as they review the history of the dam and discuss utilizing the HHPD grant to successfully breach a dam.

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SEPTEMBER 17-21, 2023

Dam Removal: Improving Fish Passage, Removing Hazards, and Catalyzing Community Revitalization

LEE HOLT, US FISH AND WILDLIFE SERVICE

ABSTRACT

The Bipartisan Infrastructure Law appropriated over $2 billion to several federal agencies to remove barriers in rivers and streams to allow for natural fish migration, to restore ecosystems, and to improve public safety. Multiple federal agencies with various missions including dam safety, transportation, conservation, water quality and science are coordinating their activities and working with States, Tribes, associations, and communities to ensure that dam removal, culvert replacement, and other activities are beneficial to fish, and also meet other important safety and community objectives. The presentation will:• Discuss the potential public safety, economic, ecological impacts of dam removal, including how dam removal can benefit owners and play a role in revitalizing communities. • Detail the various types of funding and technical assistance available for dam removal.• Explain how interested parties, including owners and states, can find information about dam removal projects and funding, and assess whether removal is the right option for them.

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SEPTEMBER 17-21, 2023

Underpinning Prairie: Micropiles Support Prairie Du Sac Dam - Part Two

BRIAN BARKAUSKAS, NICHOLSON CONSTRUCTION COMPANY

ABSTRACT

Completed in 1914 and located a little over 1.5-miles due North of Prairie Du Sac, Wisconsin, the Prairie du Sac Hydroelectric Facility comprises Prairie Du Sac Dam ("the dam") and powerhouse. The dam impounds the Wisconsin River creating the 9,000-acre Lake Wisconsin, and the powerhouse contributes 31 MW of power to Wisconsin Power & Light's (Owner) customers in Southern and Central Wisconsin. The dam's spillway, one of the few hollow gravity structures in the United States, is approximately 1,188 lin. ft long with 41 gates, each approximately 40 ft high and 20 ft wide. The dam sits entirely upon a vast network of over 12,000 steam-driven timber piles.Most of these original timber piles have proven durable over their 105 years of service. However, deterioration of some piles did occur after piles were exposed by historic tailwater recession and associated wetting and drying cycles. The pile deterioration posed the potential for more than 5 inches of settlement (including differential settlement) and concern for spillway cracking. The Owner selected the team of Nicholson Construction Company and GEI Consultants as the Design/Build Contractor to evaluate foundation solutions, conduct design analyses, perform the pre-production test program, and execute construction. After extensive analyses, the team chose a micropile solution based on a variety of selection criteria. In 2020, Nicholson presented Part One of the project including the micropile analyses and design, the initial testing phase program, and start of construction activities.This Part Two presentation will outline the project constraints, summarize the installation process, and discuss challenges encountered during the micropile installation process. Although not a stranger to the dam foundation remediation, micropiles are not as commonly used to underpin dams as was done for Prairie du Sac Dam. Micropiles proved to be an effective way to support the existing structure while managing the goals and concerns presented by the Owner.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

Bethany Dams Restoration and Rodent Burrow Prevention Project Case History

YOUSSEF M. AWAD AND DANIEL BREMERMAN, CA DEPARTMENT OF WATER RESOURCES

ABSTRACT

The California Department of Water Resources' (DWR) Bethany Dam Complex located northwest of Tracy California consists of five earthen dams and is part of the State Water Project (SWP). The Dams have experienced rodent activities on the downstream embankments for years. Backfilling of burrows occurred early in the dam's service life but ceased in the early 2000's with the identification of California red-legged frog and California tiger salamander habitat onsite, and establishment of conservation areas adjacent to the reservoir. These protected species utilize burrows as habitat. Thereafter, DWR relied on bait stations to control ground squirrels, which ultimately were ineffective in reducing the rodent activity. Over time, the burrowing escalated to a dam safety concern that could potentially promote internal erosion, piping, and instability. A decision was made in 2018 to invest in the lengthy environmental permitting needed for adequate protection of the species while allowing for the needed improvements. DWR completed the Project in 2021 after extensive permitting to ensure that the work could be done while adequately protecting species. The Project restored the embankments and provided effective long-term burrowing prevention measures to ensure dam safety and reliability of the Reservoir operations. The Project design included two main measures: restoration and prevention measures. Restoration measures included excavation, backfill, and compaction of downstream slopes to repair burrow holes and cavities, surficial distresses, and eroded areas. The restoration measures included a contingency to grout deeper holes beyond the excavation depth if such holes were encountered. Prevention measures included armoring the dam downstream embankments with Rock Slope Protection (RSP) and steel wire mesh underlayment embedded in a bedding rock layer. Additionally, measures included constructing a burrow prevention cutoff trench at the dam toes and groins to prevent future burrowing around and under RSP.The constraints used to ensure adequate protection of species posed a significant challenge with some aspects of the project being designed in parallel with environmental permit processes. Environmental requirements prohibited measuring or invasively investigating the burrows before obtaining permits. As such, DWR designed the project based on research studies of burrow architecture and habitat of various rodents in the area and included a contingency plan to adjust the excavations and grouting extent of remnant burrows.DWR successfully completed the Project with minor adjustments during construction and set a successful model for the design of other rodent burrow remediation projects currently under design and construction for three other SWP dams.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

A case study for embankment rehabilitation under full headwater pool: Challenges, surprises, and lessons learned

ADAM J. SCHNEIDER, P.E., AND AUSTIN RIEDER, P.E., AYRES; AND RYAN GRONDIN, P.E., WEC ENERGY GROUP

ABSTRACT

If a dam owner cannot draw down an impoundment enough to eliminate high exit gradients for embankment seepage, how can we still complete embankment repairs safely? The Tomahawk Hydroelectric Project is located on the Wisconsin River two miles downstream of the City of Tomahawk in north-central Wisconsin. The project's reservoir, Lake Mohawksin, covers about 2,700 acres and has a storage volume of about 15,200 acre-feet at normal pool. One of the project's features is an 18 feet high, 1,300 feet long detached earthen dike separated to the left of the main dam by a section of high ground. A periodic safety inspection of the dam in 2016 determined that the left detached dike did not meet Federal Energy Regulatory Commission (FERC) required factors of safety for stability and embankment seepage conveyance, so the owner, Wisconsin Public Service (WPS), committed to rehabilitating the dike to comply with FERC requirements. WPS retained Ayres to design required improvements to the dike, which included a two-stage blanket filter and flattened downstream slopes. A significant complication to the overall project was that a drawdown of the upstream reservoir during construction was not economically possible due to process water needs at a large paper mill. Therefore, a staged 250-wellpoint dewatering system was installed at the top of the dike to draw down the phreatic surface and provide safe, workable conditions during construction. Construction challenges included well encounters with occasional cobbles, challenging subgrade soils, a surprise uncovering of a corduroy logging road, limited construction access, and a safety requirement to stabilize all exposed foundation soils prior to contractor leaving the site each night. Ayres provided on-call assistance, fulltime (when needed) construction observation, and risk opinions to both owner and contractor throughout the project.This presentation is focused on construction oversight during the installation of the dewatering wells, monitoring of the embankment throughout the course of construction, challenges encountered during construction, and adaptive management for rewatering of the embankment following major construction activities. Lessons learned will be presented to help other embankment owners with limited drawdown capabilities to evaluate challenges with and possible applications elsewhere for dewatering wells.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

Iowa Beef Processors Waste Pond Failure, 1993 – DFIC Decade Dam Failure Series

DOUGLAS JOHNSON, P.E., REGIONAL ENGINEER, FERC, PORTLAND, OR

I. INTRODUCTION

The Iowa Beef Processors (IBP) Waste Pond was originally constructed in 1971 and was used for the storage of wastewater from the IBP Beef Processing Plant near Wallula, about 12 miles southeast of Pasco, Washington (Figure 1). When full, the waste pond had a surface area of 37 acres and stored about 270 acre-feet of wastewater and sediment from the adjacent cattle feedlot. The pond was created by construction of a 15-foot high, 1,000-foot long earthfill dam across a natural ephemeral drainage course. The dam was not provided with any spillways, and thus could not pass any flood runoff from the 55 square mile drainage basin above the dam.

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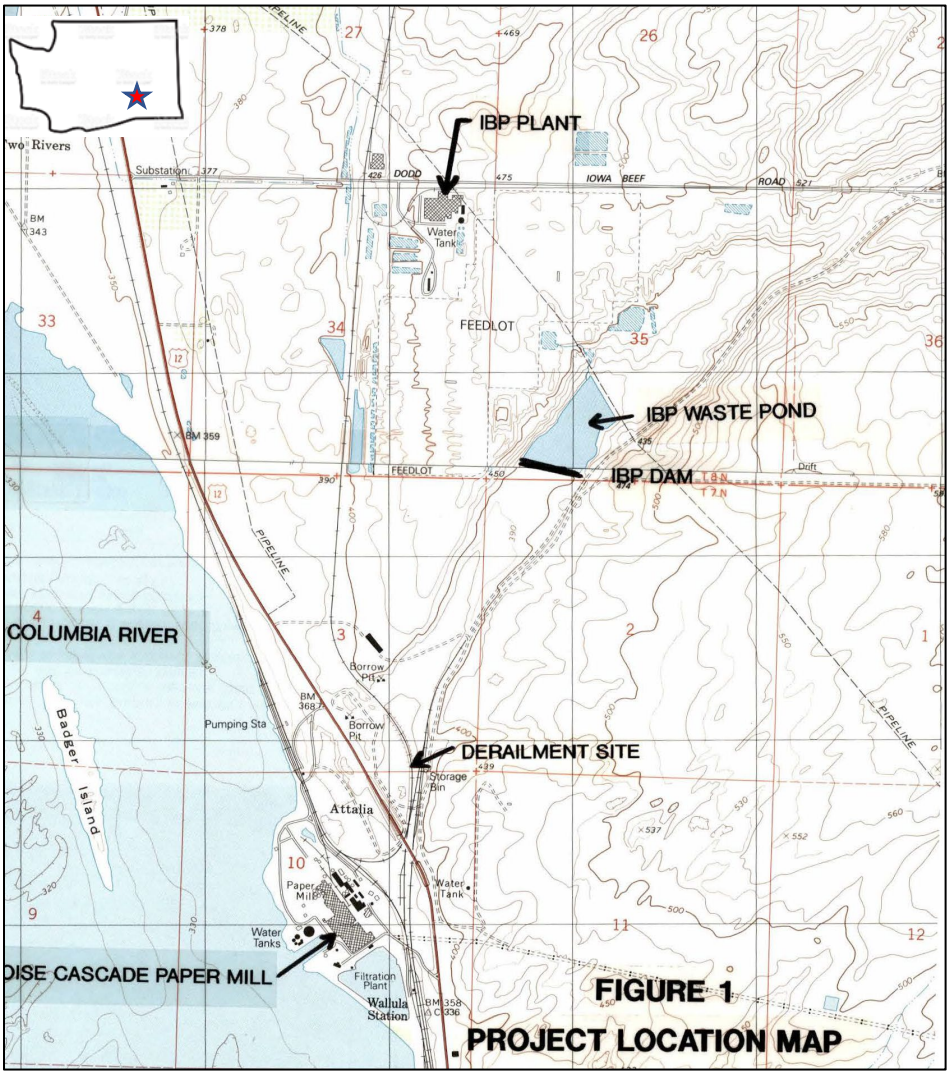


Figure 1: Project Location

The dam was inspected under the U.S. Army Corps of Engineers Phase 1 inspection program in 1981, and their report concluded that:

- The dam did not have an overflow spillway and thus could not handle any flood from the 55 square mile drainage basin above the pond.
- The embankment stability was questionable, especially during sustained high pool levels.
- The dam was riddled with animal burrows.

The report recommended that an emergency spillway capable of passing a major flood be constructed, that a geotechnical evaluation of the embankment stability be performed, and that all animal burrows on the dam be backfilled with compacted embankment material.

In 1984, the Washington Department Ecology Dam Safety Section (DSS) sent a letter to IBP reminding them that the dam had deficiencies as identified in the Phase 1 report that needed to be corrected. In response, IBP retained a consultant to perform a geotechnical investigation of the dam. Their report [1] found that the embankment was stable under static loading conditions, provided seepage remains at low levels within the embankment. The consultant also cautioned the owner that burrowing animals could initiate a piping and erosional failure if they intercepted the line of seepage within the

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embankment. They recommended that the animal burrows be filled, the animals be eliminated from the site, and that piezometers be installed in the dam to monitor seepage levels.

Following submittal of this report to the DSS, apparently no further action was taken by IBP to modify or repair the dam.

II. DAM FAILURE

According to newspaper accounts [2] of the incident, the failure of the dam occurred sometime between 11:19 p.m. on January 24th, and 3:40 a.m. on January 25th, 1993, when a Union Pacific freight train derailed on a washed-out section of track downstream from the IBP pond. The dam breach had a width of 60 feet and depth of 19 feet below the dam crest (Figure 2). The resulting dam breach flood passed down the natural drainage channel below the dam washing out the dikes on two Boise Cascade holding ponds located about 100 and 200 feet downstream from the IBP dam. The flood was then impounded behind the Union Pacific railroad fill located about 1 mile downstream from the dam (Figure 3). High water marks upstream from the railroad fill and debris lines on the railroad tracks indicated that the fill had impounded water to a depth of 15 feet. The flood water appeared to have flowed over the railroad tracks and through the ballast at the south end of the fill, significantly eroding the downstream face. Sometime after this overtopping occurred, the railroad fill failed, probably by internal seepage erosion, about 700 feet north of the overtopping area. This released the water impounded behind the railroad fill and inundated the Boise Cascade wood waste landfill east of the U.S. Highway 12 embankment.

Most of the water was then impounded in the low-lying area upstream from the highway, with only a small portion of the flood water flowing under the highway bridge which passes over the railroad tracks. A northbound freight train derailed at the location of the overtopping where the fill had been washed out (Figure 4). All 5 locomotives went off the track into the flood waters, injuring the three crewmen.

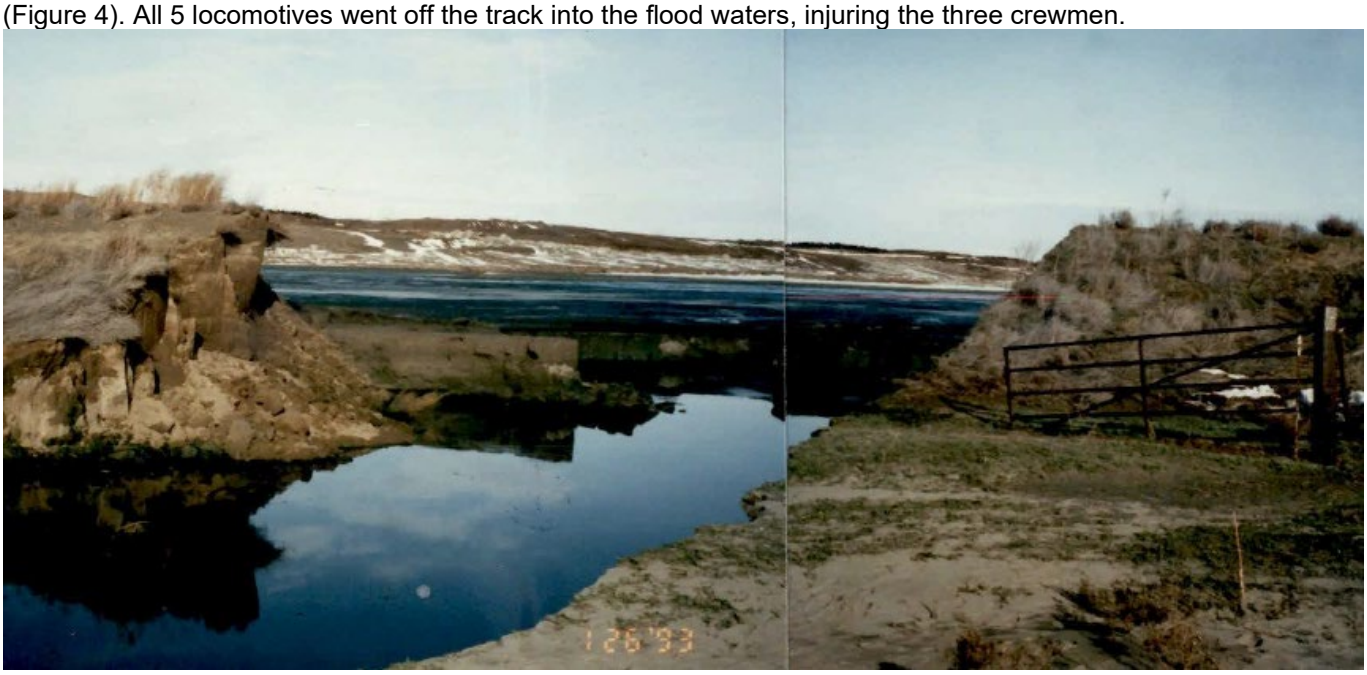


Figure 2: View through breached section of dam.
Photo Credit: 1993 Report on Failure of Iowa Beef Processors Waste Pond, WA Department of Ecology. (Public records)

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Figure 3: Aerial View of Flood Path from Failure, with derailed train in foreground. Arrow points to IBP Dam location.

Photo Credit: 1993 Aerial Failure Photos, WA Department of Ecology files. (Public records)

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Figure 4: Close up view of area where overtopping of railroad fill occurred. Water flowed over tracks and through ballast, eroding right side of embankment in photo.
Photo Credit: 1993 Report on Failure of Iowa Beef Processors Waste Pond, WA Department of Ecology. (Public records)

The estimated cost of this failure was over $10,000,000 (2023 dollars). This included the cost of the locomotives, environmental cleanup, and repair of the rail line. The cost for IBP to permit and build a new off-channel waste facility was several million more dollars.

III. Cause of Failure

Investigations performed by DSS engineers [3] attributed the failure of the IBP waste pond to three main factors:

1. The rapid melting of a record snowpack, coupled with higher-than-normal pond levels, filled the waste pond and overtopped the west end of the dam. The source of the runoff was a cattle feed lot to the west, and not from the 55 square mile drainage basin.
2. The facility did not have an emergency overflow spillway to pass flood flow out of the facility. Because of this, the reservoir level rose and flowed into animal burrows located above the normal high-water level on the upstream face of the dam.
3. The dam most likely failed directly from seepage through the animal burrows, exiting on the downstream face and causing erosion. Evidence of similar erosion was found at the west end of the dam (Figures 5 and 6). The silty soil of the embankment was highly susceptible to erosion, and it is likely that the erosion backcut rapidly toward the upstream face, eventually breaching the dam.

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Figure 5: Erosion of downstream face by flow exiting an animal burrow.
Photo Credit: 1993 Report on Failure of Iowa Beef Processors Waste Pond, WA Department of Ecology. (Public records)



Figure 6: Animal burrows (circled) exposed in west side of breached section.

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Photo Credit: 1993 Report on Failure of Iowa Beef Processors Waste Pond, WA Department of Ecology. (Public records)

IV. Conclusions

The lack of an adequate maintenance and inspection program at the dam allowed the numerous animal burrows to exist in the embankment. Animal burrows were noted in the sides of the breach and in numerous locations on the embankment. Because the facility did not have a spillway, the elevated pond level allowed water to enter animal burrows on the upstream face. Thus, the lack of a spillway and poorly maintained condition of the dam appear to have been the primary causes of this dam failure.

V. Lessons Learned

- **Reservoir Design and Location.** The siting of a wastewater pond on a natural drainage course with a 55 square mile tributary area was a poorly chosen. An appropriately designed dam would require some sort of emergency overflow spillway; however, that would run contrary to the purposes of a wastewater pond (i.e., preventing wastewater discharging to streams or groundwater). Thus, in most cases, it is desirable to construct off-channel impoundments for wastewater storage.
- **Regular Inspections.** The owner claimed they "inspected" the dam weekly, but this only involved someone driving over the dam crest and not actually walking the face and toe area. Inspections should be performed regularly and thoroughly by the owner, and if any deficiencies are found, they must be corrected immediately.
- **Safe Operation.** As in many other cases with small dams, the owner only viewed the pond was as a place to dump wastewater until spray irrigation could be resumed in the spring. No thought was given to safe levels of operation to retain the structural stability. Normal operation was to fill the pond with processing water and runoff during the winter months when spray irrigation was not possible. There were no limits placed on how much the pond could fill in order to maintain sufficient freeboard. Thus, the pond was allowed to fill to unsafe levels prior to the snowmelt event which precipitated the failure.
- **Animal Burrows.** Maintenance of animal burrows on embankment dams is critical. Burrows should be backfilled as thoroughly as possible on a regular basis, especially those near the dam crest. If at all possible, the offending animals should be removed from the area.

Disclaimer: This paper and the views expressed herein are my own and do not necessarily represent those of my Commission, the individual Commissioners, or the Federal Government.

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Note: The author personally investigated this failure in 1993 while a staff engineer at the Washington State Department of Ecology, Dam Safety Office.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

Lake Overholser Dam Failure of 1923
Devastation in Oklahoma City
[DFIC Decade Dam Failure Series]

ROBB ROY, PE, SENIOR GEOTECHNICAL ENGINEER, HATCH; BILL KUSSMANN, PE, SENIOR GEOTECHNICAL ENGINEER, HATCH; JONATHAN PITTMAN, PE, PRINCIPAL, SCHNABEL ENGINEERING

ABSTRACT

Lake Overholser Dam, located in the heart of Oklahoma City, Oklahoma along the North Canadian River, consists of a buttressed concrete pumphouse with four arched sluiceway openings and buttressed spillway bays and a gated spillway concrete gravity section tied into earthen embankments. To this day, the dam impounds a 1,500-acre lake that serves as a critical water supply source and major recreational amenity for the city. However, on October 16, 1923, the lake became a threat to the community. That year, the city had suffered severe flooding from spring rains, and it was happening again in October. These rains pummeled the watershed and caused lake levels to rise sharply and overtop and breach the earthen embankments on either end of the concrete structures. The observed initial failure point was the west embankment, causing the river downstream of the dam to rise by seven feet in approximately 30 minutes. Two more hours passed, and the east embankment was breached, causing the river to rise an additional two feet. Immediately after the failure of the west embankment, a 25-foot-high flood wave engulfed the city streets and inundated significant portions of the city. Urgent telephone warnings were made, and more than 300 members of the National Guard and American Legion volunteers worked to evacuate over 15,000 residents across 117 city blocks. These evacuations saved hundreds, if not thousands, of lives, but unfortunately at least five lives were lost and downstream damages, not only to Oklahoma City but also to several other communities as far downstream as 30 miles, exceeded $3.5M in 1923 dollars (approximately $62.5M in 2023 dollars). Recovering from the flood was a tall task that took months. As a part of the recovery and rebuilding effort, the City also worked to straighten portions of the river to help reduce future downstream flooding and the impacts associated with a dam breach. Thankfully, there has not been another dam breach and Lake Overholser continues to serve the residents of Oklahoma City, with multiple dam safety modifications and improvements currently planned or underway by the Oklahoma Water Resources Board to extend the service life of the structure.

This paper will explore the critical nature of the dam and lake to Oklahoma City, the events surrounding the failure of the dam, the major undertaking to restore the City and mitigate future flooding impacts, and the current steps to improve the safety of this essential structure.

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I. INTRODUCTION AND SITE DESCRIPTION

At the turn of the 20th century, Oklahoma City, Oklahoma was experiencing rapid growth, and the need for additional water supply was imminent. In 1916, the mayor of Oklahoma City, Ed Overholser, proposed that the city build a water supply reservoir along the North Canadian River just west of the then city limits. Shortly thereafter, the citizens agreed to a $1.5M bond (1916 dollars; approximately $42M in 2023 dollars) to fund the project, and the City engaged Niels Ambursen and the Ambursen Construction Company of New York to design and construct the dam. The construction of Lake Overholser Dam, named for the mayor, and an associated water treatment plant were completed in 1919, providing a critical water supply source and major recreational amenity for the city.

The original dam, which was about 1,100 feet long and had a maximum height of about 54 feet, consisted primarily of a reinforced concrete slab and buttress ("Ambursen-style") structure with a 126-foot-long concrete overflow spillway near the center of the dam. A river bypass section at the left (east) end of the dam, separated from the main body of the lake upstream by an earthen diversion dike, included a reinforced concrete pump house located atop four arched sluiceway openings with two buttressed overflow spillway bays on either side. A short earth embankment tie-out at the left (east) end of the dam included a concrete core wall, and a longer earth embankment section at the right (west) end of the dam reportedly included a concrete core wall as well as a sheet pile wall into the foundation. See Figures 1 and 2 below for a view of the original dam construction and a layout of the overall project, respectively.

Based on available reference information and one of the author's experiences, the geology at the site consists of sandy to clayey alluvium overlying Permian-aged shale bedrock. The concrete dam structures were reportedly founded on the shale bedrock, requiring the removal of up to 27 feet of alluvium, which presented multiple excavation support and dewatering challenges during construction. The foundation conditions at the earth embankment sections on either end of the dam are not described in the reference documents, but available drawings appear to indicate that some of the alluvial soils may have been left in-place below the embankments.

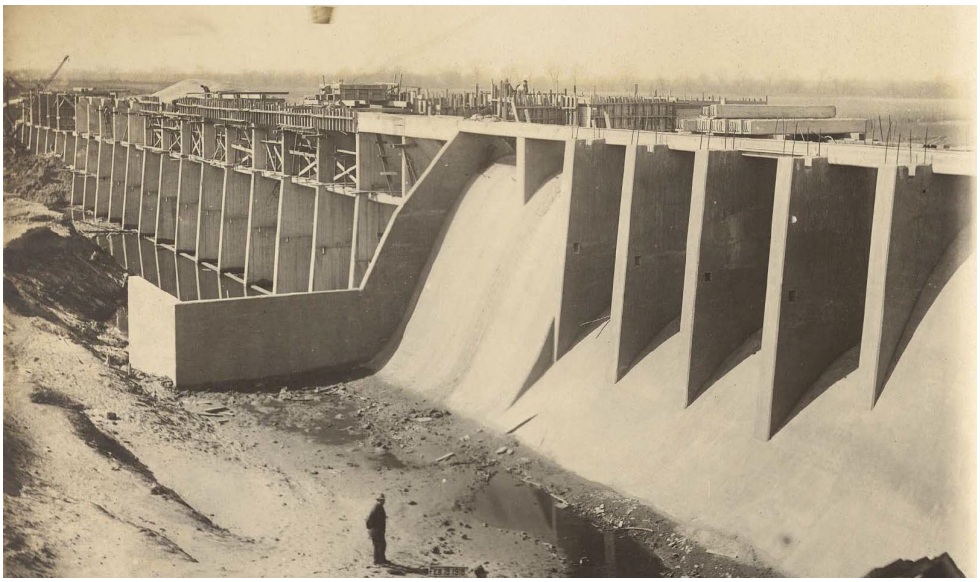


Figure 1: 1918 photograph of the original dam construction looking right [1]

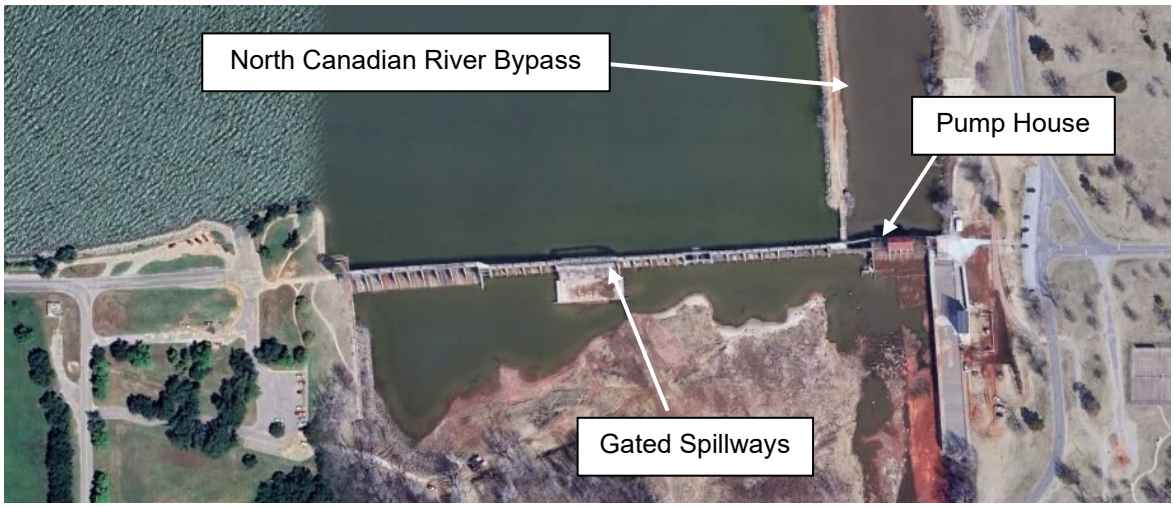


Figure 2. Current day aerial view of the Lake Overholser Dam [from Google Earth, 2023]

II. EVENTS LEADING TO FAILURE

1923 was a very wet year for Oklahoma City. In June, the North Canadian River had set a record depth of 16 feet relative to a normal depth of approximately 6 feet, and the wet conditions continued into October. By Friday, October 12, high water had cut off streetcar service to the Stockyards City neighborhood in the heart of Oklahoma City. By Sunday, October 14, flooding was occurring upstream in Woodward (116 miles northwest of Oklahoma City), and on Monday, October 15, reports indicated that water had risen 5 feet in 25 minutes in Canton (65 miles northwest of Oklahoma City). Weather officials indicated the flood crest would reach Oklahoma City on Thursday, October 18.

On Monday, October 15, a sudden rise in river levels washed out the U.S. Highway 66 Bridge and the interurban bridge just north of Lake Overholser. The debris was carried down the river and river bypass where they crashed into the dam. At approximately 9:30 PM on Monday, October 15, the dam was breached. The observed initial failure point was the west earthen embankment, of which almost 300 feet of the embankment failed suddenly due to flood waters and debris battering against the banks, causing the river downstream of the dam to rise by 7 feet in approximately 30 minutes. Two more hours passed, and the east earthen embankment was breached by the flood water, causing the river levels downstream of the dam to rise by an additional 2 feet. The reported maximum floodwater height was 25 feet at approximately 2:00 AM. Newspaper articles printed on Tuesday, October 16, the day after the breach, (see Figure 3) indicated that immediately after the failure, a 25-foot-high flood wave engulfed the city streets and inundated significant portions of the city.



THE DAILY OKLAHOMAN

117,810

CONDUIT LINE CARRYING CITY WATER SUPPLY IS THREATENED

25-FOOT FLOOD RUSHES ON CITY

STREAM OF REFUGEES HASTENING TO SAFETY IN NORTH TOWN

WESTERN AREA OF STATE MAY FACE FAMINES

Water Cuts Around Ends of City Waterworks Dam

RIVER RIPS HUGE GAP IN DAM; 15,000 FLEE TORRENT'S WRATH; FLIGHT DIRECTED BY MILITARY

Reservoir Will Be Emptied; Hospitals Filling; Hundreds of Motor Cars Carry Refugees

Flood Facts

MAN PARDONED, UNDER ARREST

Lower House Ready To File Impeaching Bill Late This Week

VOTERS NOTES ALARM SOLON

SEVEN CONFESS TO OIL FRAUDS

Figure 3. Headlines from the local newspaper the day after the dam breach. [4]

While the exact extents of the 1923 breach inundation area are not known by the authors, breach modeling performed by the USGS in 2015 provides an estimate of the inundation extents for a failure of Lake Overholser Dam during an extreme flood, showing significant impacts to Oklahoma City and areas along the river downstream (Figure 4).

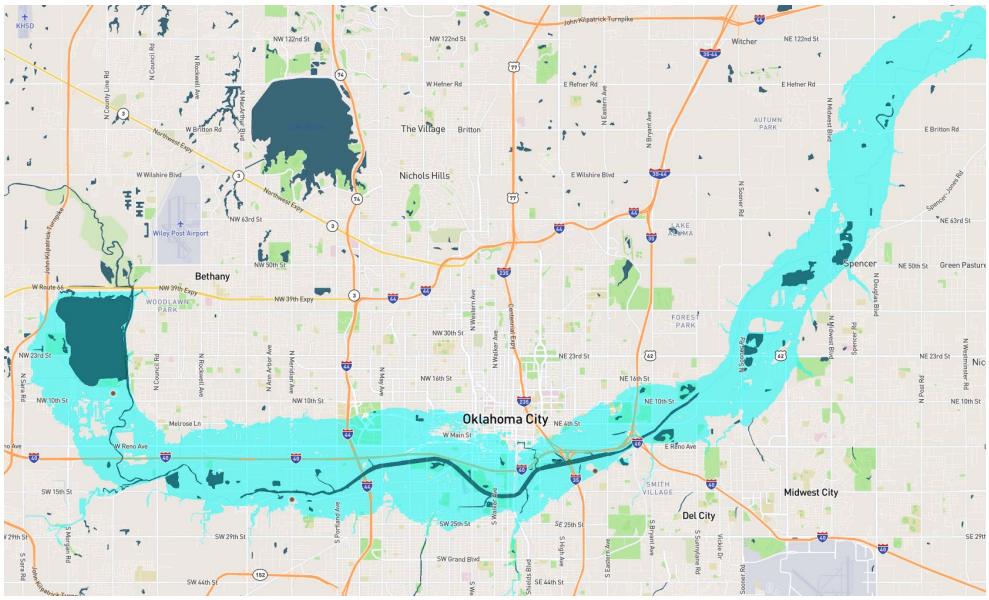


Figure 4. 2015 breach inundation map developed by the USGS showing potential extents of a breach Lake Overholser Dam during an extreme flood. Note that this is shown for reference only and does not represent the exact limits of the 1923 flood [7]

III. EMERGENCY RESPONSE AND CONSEQUENCES

Emergency procedures went into effect immediately after the initial breach of the west embankment. Urgent telephone warnings were used to notify the downstream community, and over 300 members of the National Guard and American Legion volunteers worked to evacuate more than 15,000 residents over 117 city blocks. Every available ambulance and many privately owned cars and trucks manned by volunteers were used to assist with the evacuation. In addition to the telephone notifications, whistles and sirens sounded immediately after the

breach and police fired their guns in warning. These evacuations saved hundreds, if not thousands, of lives, but unfortunately at least five lives were lost and downstream damages, not only to Oklahoma City but also to several other communities as far downstream as 30 miles, exceeded $3.5M in 1923 dollars (approximately $62.5M in 2023 dollars).



Figures 5 through 7. Photographs of the flow over the dam on either side of the pumphouse (top), debris near the pump house after the reservoir had been drained (middle), and breach of the west embankment looking downstream (bottom) [6]

As the population evacuated the city, looters quickly appeared to rob homes and businesses which were now deserted. O.A. Cargill, the mayor of Oklahoma City at the time, reportedly proclaimed to the public, as well as police and National Guard, that they had order to shoot to kill anyone discovered looting.

Two additional newspaper articles were printed on October 16, 1923, regarding more specific experiences observed during the flood. Harrowing stories included the ambulance rescue of a paralyzed woman and a woman

with a two-day old baby; the expeditious transfer of a family of five all suffering with typhoid fever to area hospitals; and the evacuation of families camping at the local circus grounds who were awoken during the night to find water around their cots and had to wade out to a nearby telephone to call for help.

In addition to human lives, there were many stories about impacts to animals. Most of the animals at the city zoo, located approximately 8 miles downstream, had been moved in the spring of 1923 to avoid the earlier floods. The remaining large animals left at the zoo, which primarily included buffalo, were released by police following the dam breach and survived the flood. However, it was estimated that approximately 600 hogs which had been feeding on garbage at the city dump, were washed away with the flood waters.

IV. RECOVERY AND RECONSTRUCTION

Recovering from the flood was a tall task that took months, though luckily the original concrete portions of the dam were not damaged during the 1923 flood event (see Figure 8). The reservoir was emptied to reconstruct the dam, with flows diverted through the bypass section. Major modifications to the dam, which were completed in 1924, included replacing the majority of the previous west earthen embankment with a cyclopean concrete gravity section with Stoney gates and converting the non-overflow slab and buttress sections into radial gated spillway sections. As a part of the recovery and rebuilding effort, the City also worked to straighten portions of the river to help reduce future downstream flooding and the potential impacts associated with a dam breach, although the North Canadian River had essentially cut itself a new channel through the western part of the city. Remnants of the old channel can still be seen today in the lagoons near the fairgrounds and in oxbow lakes near Reno and Rockwell Avenues.



Figure 8. Current day view of the downstream side of Lake Overholser Dam. [8]

V. CAUSE OF FAILURE AND LESSONS LEARNED

To the authors' knowledge, no detailed dam failure investigation was performed, or at least documented. As a result, and since the original design calculations and any monitoring data at the time are not available for review, the explicit failure mechanism(s) of the earthen embankments is unknown. The river was flowing at high levels, but whether the embankments were under designed, or the driving forces were above the original design intent is not possible to determine without the original design documents. However, there are multiple likely contributing factors that reinforce various current dam safety requirements and documented lessons learned.

Due to the apparent presence of concrete core walls extending to foundation bedrock in the earthen embankments, one could surmise that internal erosion likely did not contribute significantly to the failures. However, as previously indicated, alluvial soils may have been left in place below the earthen embankments, and these typically weak materials could have contributed to a stability and/or internal erosion failure mechanism [9]. In addition, while there is no record of embankment overtopping, it should be noted that the engineers responsible for the post-flood reconstruction of the dam believed the original dam was significantly undersized hydraulically, based on the added spillway length and controls, which is a common deficiency for dams constructed prior to the

availability of extreme rainfall data [10]. In addition, the configuration of the downstream river channel apparently contributed significantly to the degree of downstream flooding observed, which should be a recognized operational constraint in cases of significant spillway releases [11].

The Overholser Dam failure is also a prime example of how timely warning and rapid public response save lives during a dam safety emergency [12]. Although five deaths occurred, the potential loss of life could have been significantly worse if dam owners/operators had not initiated emergency procedures when they did (presumably these procedures followed the facility's Emergency Action Plan, if one existed). With the benefit of hindsight, it appears that given the flood stage at the time of the breach, failure of the upstream railroad bridge, and upstream flooding warnings, it may have been prudent to begin evacuation earlier. However, the breach occurred at night, and communication systems in 1923 were limited. In any case, given the severity of the flood, the emergency actions taken likely saved lives.

VI. ONGOING DAM SAFETY MODIFICATIONS

Thankfully, there has not been another dam breach, and Lake Overholser and its dam, which was added to the National Register of Historic Places in 2007, has continued to serve the residents of Oklahoma City, now primarily as a recreational resource and backup water supply, to this day. Like much of our nation's infrastructure, Lake Overholser Dam has required multiple upgrades and modifications over its 100+-year old service life. Current plans are underway to address deficiencies with the spillway gates, concrete structure stability concerns, and concrete structure deterioration. These improvements should be complete by 2030, further extending the life of this historic structure.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

Have Hope – The Eventful History of Hope Mills Dam, NC [DFIC Decade Dam Failure Series]

JONATHAN PITTMAN, PE, PRINCIPAL, MARK LANDIS, PG, PE, PRINCIPAL, AND ADAM PAISLEY, PE, ASSOCIATE, SCHNABEL ENGINEERING

ABSTRACT

In 1839, the Town of Hope Mills (Town) in southern North Carolina, then known as Little Rockfish Village, became a thriving location because of industry powered by a new dam in the town center, and the dam and lake have served as the centerpiece of the Town for most of the last 180+ years. Unfortunately, though, the dam has an eventful history transitioning from tragic to celebratory multiple times. This presentation will explore that history, including the multiple dam failures and reconstructions, as well as the associated lessons learned that have helped define the Town.

The first dam, a wood cribbing and rockfill structure, was built to power cotton mills, but it also formed a nearly 90-acre lake that became a key recreational amenity. However, the first heartbreak suffered by the Town occurred in 1865 when the Union Army burned the mills to the ground. Luckily, the dam was spared, and the mills were rebuilt, also spurring an official change in the name of the area to Hope Mills. That first dam was replaced by a new embankment dam in early 1924, officially becoming Town property in 1984. That second dam served the Town and multiple industrial owners well, until May 2003, when heavy rainfall resulted in overtopping and a breach of the embankment, exacerbated by a spillway gate operations failure. Emergency evacuations of 40 downstream homes and 1,600 residents were made swiftly, preventing loss of life, but the flood wave caused about $2.1M in damages (2003 dollars; approximately $3.5M in 2023 dollars). After years working to obtain the required funds and permits, construction on a new dam began in March 2007. This third dam, an embankment dam which included a labyrinth spillway and fish ladder, was completed in June 2008, and the lake was full again by August 2008. Unfortunately, tragedy struck again on June 16, 2010, when engineers noticed muddy flows downstream of the spillway, and overnight, the lake drained through voids that had formed beneath the spillway structure. Frustrations in the Town were at an all-time high and lawsuits followed. Over the next several years, the Town suffered, but Town leadership continued to push forward with plans to rebuild the structure. In March 2016, the Town was finally able to enter into a design-build contract to rebuild the structure, which continues to serve the Town well to this day.

I. THE EARLY YEARS OF THE TOWN AND LAKE (1839 TO 1923)

Hope Mills Lake, located in south central North Carolina just south of the City of Fayetteville, was originally established in 1839 when the Rockfish Creek Manufacturing Company built a rock crib dam and mill race across Little Rockfish Creek, a deep, swift-moving stream that is part of the Cape Fear River watershed. The factory built on the north banks of the mill pond spun cotton grown on nearby farms. The nearly 90-acre lake created by the dam also became a recreational centerpiece for the growing community which, by time of the Civil War, was one of North Carolina's major textile producers. Unfortunately, in 1865, the mills in the surrounding area were burned to the ground by General Sherman's advancing Union Army. Luckily, the dam was spared. By the mid-1880s, the surrounding mills had been rebuilt and the Town was flourishing, spurring an official change in the name of the area from Little Rockfish Village to Hope Mills.

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II. THE NEXT CHAPTER (1924 TO 2003)

In 1924, the original rock crib dam across Little Rockfish Creek was replaced with an embankment dam at a cost of just over $27,000 (1927 dollars; approximately $480,000 in 2023 dollars). The 33-foot-high, 550-foot-long embankment dam, known as Hope Mills Dam No. 1, included an 85-foot-wide concrete overflow spillway with three, 25-foot-wide x 10-foot-tall radial gates (Figure 1). The radial gates consisted of steel frame members with wooden boards serving as the “gate skin”. The radial gates were operated by an electrical winch crane that could be moved along the concrete service walkway above the spillway.



Figure 1. Late 1990s Aerial View of the 1924 Dam Prior to Failure. [2]

In 1984, Dixie Yarns, the Chattanooga, Tennessee-based company that owned the Hope Mills factories formally deeded Hope Mills Lake, used exclusively for recreation at that point, to the Town. While the dam served both Dixie Yarns and the Town well, it was not without issue. In 1993, Lakeview Road, a two-lane connector road, was constructed across the dam. During the construction of the road and bridge on the downstream side of the gated spillway, a significant leak in the dam was discovered, apparently near the interface of the spillway and the embankment, necessitating emergency repairs. Shortly after that, a turbine shaft from the old mill was discovered through the embankment and had to be filled with concrete to mitigate seepage concerns. Concerns were also raised about the stability of the spillway wing walls, which were buttressed with rock riprap.

Multiple reports, including the 1979 U.S. Army Corps of Engineers (USACE) Phase I Inspection Report, as well as subsequent dam safety inspection reports, indicated the dam and its spillway system were significantly undersized hydraulically, having the capacity to only pass approximately 20 percent of the required design storm (equivalent to 1/2 of the Probable Maximum Flood) prior to overtopping. These reports also highlighted the lack of a functional low-level outlet at the dam. While these significant dam safety concerns were acknowledged, the small community of Hope Mills had few resources to address them at the time.

III. DISASTER STRIKES – FAILURE OF THE 1924 DAM

On Memorial Day weekend in 2003, a series of overnight thunderstorms resulted in an eight-inch deluge that overwhelmed the watershed. Town personnel attempted to open the spillway gates, but they were unable to do so, even manually using an excavator. As a result, on the morning of Monday, May 26, 2003, the floodwaters overtopped and breached the 80-year-old embankment and washed out Lakeview Road along the crest of the dam (Figure 2). The failure of the high hazard dam triggered emergency response and the evacuation of about 40 homes downstream, temporarily displacing 1,600 people, and resulting in about $2.1M in downstream damages (2003 dollars; approximately $3.5M in 2023 dollars). Thankfully, no lives were lost.



Figure 2. Aerial View of the 2003 Dam Failure. [2]

IV. MOVING FORWARD (2004 TO 2010)

The Town spent five years working toward financing, designing, and building a third dam whose spillway design was intended to increase the dam's ability to discharge flood waters. In 2008, the new dam was completed: a $14M (2008 dollars; approximately $20M in 2023 dollars) labyrinth spillway partially founded on steel H-piles. The labyrinth consisted of two pairs of cycles that were separated by a large fish ladder. The total weir length was about 800 feet, including a 108-foot-long broad crested weir that served as an upstream entrance to the fish ladder. The outer labyrinth cycles joined to the spillway sidewalls which trained flow toward the outlet that ran under the Lakeview Road bridge spanning between two embankment sections. A pedestrian bridge spanned across the spillway outlet just upstream of the road bridge.

Although the townspeople were excited to have their lake back, many voiced their disappointment that the dam was too large – because of its size and shape, some lakeside residents dubbed the dam "the dinosaur's footprint" (Figure 3) – and at elevation 104 feet, the water level in the lake was approximately one foot lower than it had been before. Nonetheless, the lake was back.



Figure 3. Aerial View of the 2008 Dam and Labyrinth Spillway. [Google Earth, 2009]

V. DISASTER STRIKES, AGAIN – FAILURE OF THE 2008 DAM

It would not take another major storm to create chaos in the Town. On a sunny day in June 2010, less than two years after the lake reopened, the third Hope Mills Lake dam failed. Muddy water was observed flowing from beneath the downstream slabs of the large labyrinth spillway, and within 48 hours, the lake was empty again. Fortunately, the release of the reservoir, while uncontrolled, did not produce a large downstream flood wave, thus avoiding significant evacuations and major damages downstream.

Once the lake drained, one could see where the vinyl sheet piles, which were part of a construction modification to the seepage cutoff beneath the large labyrinth spillway, had separated or been broken. As a result, full head from the reservoir was applied to the foundation drain system and induced erosion of the drainage materials underlying the spillway. These materials deposited in the outlet channel, and a large void developed that exposed the steel H-piles supporting the spillway (Figures 4 and 5). This void beneath the spillway was accessible via a small boat. Failure was likely initiated during first filling, when uplift forces separated the non-pile supported slabs from the underlying drainage materials creating a gap and increasing both structural and hydrostatic forces exceeding the capacity of the vinyl sheet piles. In addition, in the pile-supported slab areas, the rigidity of the slab combined with settlement of the underlying foundation soils, may have contributed to further void development and sheet pile damage. Furthermore, during dam demolition and foundation excavation, alluvial soils and trees stumps were uncovered beneath the embankment and spillway.



Figure 4. Void beneath the 2008 Spillway [3]



Figure 5. Exposed H-Piles and Sheet Piles beneath the 2008 Spillway [3]

Shortly after failure, measures were implemented to temporarily stabilize the dam. A section of the labyrinth was saw cut and demolished to convey normal flow through the spillway. In addition, the area

immediately upstream of the spillway was backfilled to prevent access to the voids under the spillway and an earthen berm was built to channel flow through the demolished section of the spillway.

With the lake drained, the ecosystems that had surrounded it began to change. The former lakebed was rapidly overtaken by opportunistic woody and herbaceous growth, entire species of birds left the area, and the majestic cypress trees at the upper end of the lake showed alarming signs of stress. The Town leadership and citizens were devastated. Not only was their lake gone again, but the failure came at a time when the Town was planning a heritage park, a local history museum, and improvements to the historic downtown area all centered around the lake.

VI. REBUILDING AND INCORPORATING LESSONS LEARNED

Over the next several years, the Town's plans for revitalization suffered at the mercy of the funding needed to rebuild the dam and restore the lake. Four years after the dam failure, the Town was finally able to reach a $9.4M (2007 dollars; approximately $13.8M in 2023 dollars) settlement agreement with the designers and builders of the 2008 dam. Once the lawsuit was settled, the Town's Board of Commissioners worked expeditiously to select a design-build team to design and construct a new dam as well as revitalize the lakebed, all within an $8M dollar budget (2007 dollars; approximately $11.8M in 2023 dollars).

The most important goal for the Town was simply to get their beloved lake back, but they also wanted the new dam to avoid and correct the shortcomings of the most recent failed dam at a cost within the limits of the financial settlement of the 2010 dam failure. A collaborative process between the design-build team, the Town's representatives, and local stakeholders identified the top project goals. These included a smaller dam footprint, a more attractive and less obtrusive design, the mitigation of potential causes of seepage within the foundation, and more lakeside recreation areas. Due to construction and design issues that caused the 2010 dam failure, complete reconstruction of the dam was selected as the preferred alternative.

Preliminary work on the dam site began in April 2016; demolition and construction of the new dam, a zoned earthen embankment with a five-cycle labyrinth spillway design, began in August 2016. This labyrinth spillway design, which allowed the Town to keep a passive spillway system, included removal of alluvial soils prone to seepage issues. It also included a seepage cutoff with a slurry wall to intercept braided stream patterns in the foundation, which further addressed the critical foundation stability and seepage control concerns of the previous spillway. The steel H-piles supporting the former spillway were cut off two to three feet below grade and were backfilled with a compacted soil fill cushion located upstream of the new dam. The new labyrinth structure and the foundation of the core were founded on the harder, less permeable Cape Fear Formation soils, and the earthen tie-outs provided a newly built zoned embankment on each side of the spillway to remove reliance on the previous road embankment as an earth dam.



Figure 6. 2017 Photograph of the New Hope Mills Dam and Spillway during Reservoir Refilling [3]

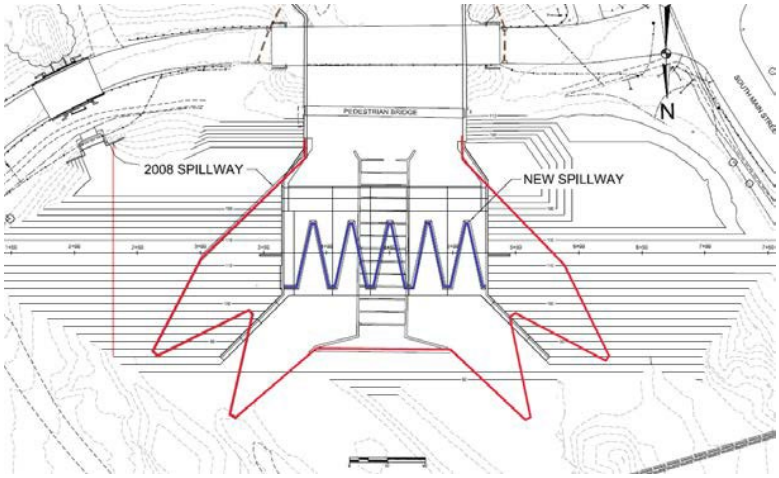


Figure 7. Comparison of the Footprints of the 2008 Spillway (red) and the New Spillway (blue) [3]

While there were several significant construction challenges, including receiving rainfall in excess of the 500-year event due to Hurricane Matthew in October 2016, the dam construction was successfully completely in October 2017 and the lake was restored by January 2018. The dam was tested shortly after completion with a thunderstorm producing rainfall in excess of a 100-year event. Nay-sayers from the media flocked to the site saying the dam would fail again, yet it performed as designed and the waters receded with no damage. The Town attracted further attention in 2018 when they were presented with the first ever Envision certification for sustainability for a dam construction project (see footnote below).

From the earliest concept meetings through the final, much-celebrated stages of construction, the community of Hope Mills had been a central participant in the restoration of their beloved lake. As one of the lakeside homeowners stated, “This community was founded on the lake and the whole community grew up around it. The lake has been the heart of the community and when the lake went down it was like the community lost a little bit of its heart.” Thankfully, that heart has been restored, and the new dam continues to perform as designed.

[1] Envision is a framework and certification process provided by the Institute for Sustainable Infrastructure to encourage and assess sustainability, resiliency, and equity in civil infrastructure. https://sustainableinfrastructure.org/envision

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

Constructability Considerations for Dams and Spillways

BRIANNA K. CROTTY, PE, SCHNABEL ENGINEERING; CHARLES M. JOHNSON, PE, SE, SCHNABEL ENGINEERING; AND MARIUS SIMA, SCHNABEL ENGINEERING

ABSTRACT

Constructability is commonly referred to as the integration of construction expertise and knowledge into the design or preconstruction phase of a project. While there are numerous available references for constructability best practices, many of these are not well-documented. The project team's goal of developing a constructable design improves project construction schedules, reduces construction costs, and provides the owner with high quality product. Although numerous dam projects are completed each year, valuable information is often communicated too late between the contractors and designers, and when potential constructability issues become difficult or costly to prevent. The sooner constructability challenges are identified, the easier and less costly these items are to address, resulting in reduced potential for schedule delays.

This paper summarizes a variety of common dam and spillway design and constructability considerations. This paper is the culmination of observations and lessons learned from construction field personnel and engineers specializing in dam design and construction. The importance, challenges, and benefits of constructability presented are applicable to a wide variety of construction projects. However, the main focus of this paper is on constructability considerations for reinforced concrete spillways constructed in embankment dams. This paper focuses on common constructability challenges, such as rebar detailing, wall and slab geometries, concrete placements, and joint details It also provides examples of specific details, such as alternative weir designs (e.g., ogee, labyrinth, and piano key weirs), as well as energy dissipators, drains, wall penetrations, backfill compaction methods, and anti-seep walls. Examples and lessons learned are provided for the identified constructability concerns.

The information presented in this paper will benefit those involved with design and construction of spillways and dams, including engineers, contractors, construction representatives, regulators, and manufacturers seeking safer, less expensive, and more durable spillways.

I.INTRODUCTION

Dam construction and rehabilitation is an ongoing need, particularly as infrastructure ages and water resources continue to grow more unpredictable. A study published by ASDSO in February of 2023 reported a need for the rehabilitation of over 100,000 non-federal dams, of which 15% are considered High Hazard [1]. Total costs to upgrade and rehabilitate the aging infrastructure are estimated to be close to $100 billion. The layout and size of features related to rehabilitation are established by hydraulic and geotechnical demands, along with structural considerations. Decisions made on these design features can have a significant impact on constructability and affect the owners' costs, schedules, and quality of their dam assets.

Constructability is defined as "the optimal use of construction knowledge and experience in planning, procurement, and field operations to achieve overall project objectives" [2]. Formal constructability programs are important to improving overall project quality, schedule, and costs. Engineers, contractors, and owners can all benefit from having dedicated constructability reviews, site visits, and modeling. According to the Construction Industry Institute (CII), an effective constructability program promotes the following benefits:

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- Reduce overall project cost by an average of 4.3%.
- Reduce overall project schedule by 7.5% on average.
- Improve project quality, Safety, security, and environmental impact.
- Reduce rework and rescheduling.
- Owner savings from constructability programs result in a ten-fold return on investment.

To gain maximum benefit from an effective constructability program, the program should be implemented early in the design phase and continue through construction. An effective constructability program includes a corporate commitment to implementation, planning for and implementing constructability, and regularly updating the corporate program [2]. Formal constructability programs typically include indoctrination of constructability as an integral focus of corporate, project, and individual mentality supported by corporate policies and checklists; contractual measures; formal reviews during planning, design, and/or construction; constructability modeling; focused training; and case study reviews [3]. For engineers, constructability implementation often includes reviews at design milestones by qualified personnel or may include hiring a contractor for external reviews, as noted in Section V of this report [4]. Reviews may utilize standard checklists tailored to the level of design, recording lessons learned, and monitoring constructability review processes. Designers and contractors should also be familiar with best constructability practices relevant to the project, such as steel [5], concrete [6], or earthwork. Based on the authors' review of various state and federal dam safety regulatory agencies, whose primary responsibility is dam safety, these agencies typically do not include constructability specifically on applications for dam construction, modification, or rehabilitation projects, defer responsibilities on constructability to engineers and contractors.

A good constructability plan considers such items as site access, safety, permitting, hauling and lifting plans, resource availability, temporal requirements (e.g., outages, drawdowns, seasonal variations), quality assurance/quality control requirements, project risk, and operation and maintenance (O&M) implications.

The authors received survey results from 36 dam owners, regulators, engineering firms, and construction companies for this paper to evaluate if these organizations had a constructability program, and if so, whether it was formal or informal. The results are included in Figure 1.

Survey results showed the following:

- Engineers consider constructability a shared responsibility of the design team, and that it should be considered throughout the design process. Of the respondancs, 75% of the firms either had staff specifically dedicated to constructability, or they sought advice from leading experts.
- Some owners have in-house constructability resources, though several respondents utilize external resources for constructability reviews, either as part of a formal process or informally when needed.
- Construction companies typically have the highest percentage of programs in place to ensure constructability. This is not surprising, given the burden is on the contractors for ultimately constructing the project.
- Dam Safety regulators' primary responsibility is dam safety, and not necessarily constructability. It is not surprising that glaring constructability issues may be questioned on design reviews, but regulators typically do not view constructability reviews as a primary responsibility.

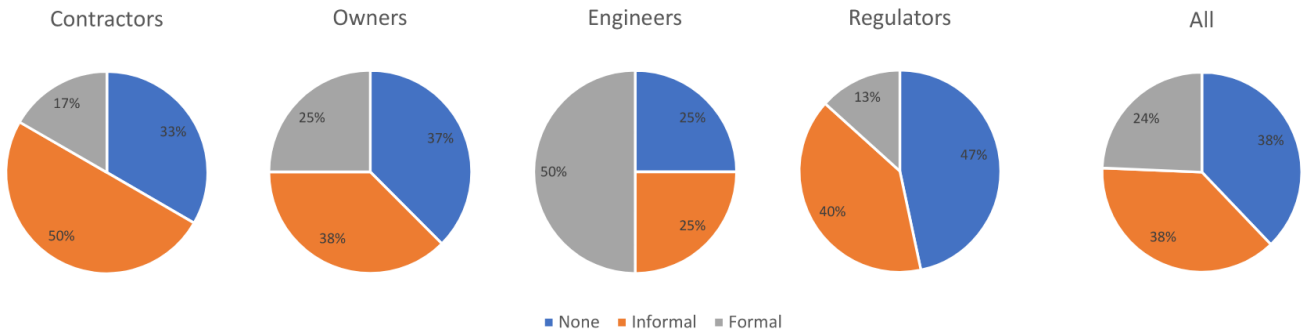


Figure 1: Level of participation in constructability programs by group (owners, regulators, engineers, and contractors)

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II.KNOWN DEFICIENCIES

Over the last 10 years, the number of high-hazard potential dams where loss of life after a dam failure is anticipated has increased by 20 percent, to include over 16,000 dams. This is largely due to a growing population, an overall increase in non-permeable terrain, and aging infrastructure. Though the number of dams in each state is on the order of hundreds and thousands, many of the dam owners cannot afford the prohibitive costs associated with traditional dam and spillway rehabilitation. According to the 2022 ASDSO Task Force, the cost of repairing and rehabilitating high hazard non-federal dams alone would cost nearly 34.1 billion dollars. The same Task Force estimated that bringing all non-federal dams up to current standards would cost approximately 157.5 billion dollars [1].

The purpose of this paper is not to define the best or most efficient ways to achieve constructable spillway designs, as this is a subjective topic that may vary, but instead explain what constructability is and how it can be considered throughout a project. This is done by highlighting a few examples of constructability challenges for dams and spillways and how they can be overcome. Examples include structural, geotechnical, and other considerations.

III.STRUCTURAL CONSIDERATIONS

Many spillways consist of reinforced concrete sidewalls, a spillway slab, a control section, and a stilling basin. The following structural constructability considerations are provided for walls, weirs, structures with difficult site access or difficult concrete placements (such as energy dissipators), slabs, footings, energy dissipators, and roller compacted concrete.

A. Spillway Walls

There are several geometric considerations for spillway wall construction that can have a significant impact on constructability. These include thickness, batter on the outside face, curvature, slopes, and joint details.

The minimum wall thickness for spillway walls is typically 12 inches to facilitate concrete placement using an elephant trunk inserted between two mats of rebar at the inside and outside faces [7, 8]. Depending on concrete cover and rebar size, the minimum wall thickness may need to be increased. While spillway wall faces exposed to flows within the spillway are preferably vertical rather than battered (tapered) to reduce unacceptable flow eddies [9], outside faces of walls that are backfilled are preferably battered with height to allow improved compaction of backfill on the soil face of spillway walls. Typical wall batters are between 1:8 to 1:12 (horizontal:vertical). Constructability of battered walls is more difficult if the wall height is not constant along the length. Changes in battered wall height within a single wall placement typically result in a uniformly varying base width to maintain the batter or holding a constant top and bottom of wall thickness, which requires that formwork is warped. Warped battered walls are perhaps more difficult for engineers and designers to detail in construction drawings than for a contractor to construct, and warped formwork has been successfully used. Yet another option is to maintain a constant batter to a set height and transition to a vertical face above the minimum wall height. The batter typically extends to or near the final grade to allow compaction, while the constant wall thickness allows a uniform appearance of the wall top. Depending on the wall batter, wall thickness, and reinforcement size and spacing, rebar congestion can be an issue at the transition from battered to vertical.

Wall vertical joints should typically be located approximately 2 feet minimum from corners. This helps relieve horizontal stresses and allows for easily constructable corner reinforcement.

With ongoing population growth and the expansion of land development, ensuring sufficient hydraulic capacity for spillways has emerged as an urgent concern. Increasing spillway capacity may require increasing wall height and thickening the walls. Making the thickened portion of wall composite with the existing wall may be performed by roughening the surface, applying a bonding agent on the existing concrete immediately prior to placement of the new concrete, or drilling a gridded arrangement of holes and installing rebar dowels with an adhesive to serve as shear-friction reinforcement. Shear-friction reinforcement requires development into the existing and new concrete placements, which may not be possible if the existing walls are not thick enough. Therefore, we recommend designing thicker walls to plan for future spillway enhancements.

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Figure 1. (Left) Formwork at full-height battered wall corner (Right) Formwork at partial height battered wall corner

Wall geometric transitions pose some important constructability challenges. Horizontal and vertical curves are used for training spillway flows or slope transitions. Although more difficult to construct than straight, flat walls, horizontally curved walls can be constructed using curved formwork. Commercially-available formwork systems are suitable for a minimum internal radius of about 3 feet [10]. For large radius walls, particularly walls not subjected to spillway flows, horizontal curvature can often be more economically approximated by a series of straight segmented walls between joints. Compound horizontal and vertical curvature of a single wall is difficult, and there may be a tendency to avoid compound curves within a single wall section; however, due to the need for custom formwork, it may be better to limit the extent of the custom formwork by locating compound and vertical curves in a single wall section, particularly for walls that are battered on one or more faces.

B. Weirs

Non-straight weirs, such as labyrinth, piano key, and ogee weirs, entail unique challenges. One of the challenges of labyrinth weir construction is the upstream and downstream apexes. Several philosophies have been applied to labyrinth weir design. One approach is to place vertical joints near the upstream and downstream apexes to relieve horizontal stresses. Another approach is to provide vertical contraction joints in the labyrinth walls only at the center of the upstream apexes, where the wall tends to be compressed due to hydrostatic pressure and halfway downstream in each cycle, where the weir walls behave most like pure cantilevers. One way to increase the amount of concrete in an individual placement while controlling shrinkage cracking is by using a shrinkage reducing admixture. This type of admixture, with the proper dosage rate, creates expansion in the concrete to slightly overcome the initial shrinkage and result in a net expansion, reducing cracking and providing better protection against water or chemical intrusion.

In order to avoid potential vibration issues due to spillway flows, labyrinth weirs typically have a half-round weir crest [11]. Transitions from battered labyrinth weir walls to a half-round crest have been successfully constructed but can usually be avoided by using vertical walls for heights 10 feet or less, without a significant impact on construction cost, due to minimum wall thickness requirements.

In Figure 2 below, the clear distance between sidewall (on the left) and labyrinth weir wall (on the right) was 9 inches, which was successfully constructed, but made construction and quality control inspections difficult. Usually sidewalls are constructed before weirs, so tying the walls together is a challenge. Use of rebar splices (form savers) and split flange water stops, which can be embedded in the first pour and then straightened and secured in the second pour, can simplify the formwork. These are described further in the following sections of this paper.

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Figure 2. Difficult access at labyrinth wall apex

Expected appearance of finished concrete walls can be defined by requiring a wall finish per ACI 347R in project specifications [12]. Exposed faces of spillway walls are typically specified to a Type “A” finish, which requires a 1/8-inch tolerance on wall surface. Caution should be taken to avoid “over-finishing” Type A finishes, which can result in spalling. Tolerances on structure dimensions that are unnecessarily strict can add cost and time to a project or result in rejection of a concrete structure that would perform very similarly to a structure with slightly more flexible tolerances. Wall footing plan dimensions and spacings between joints are typically specified to the nearest inch, preferably the nearest foot, unless there are conditions such as angles or curves where keeping angles and plan dimensions “clean” is not possible. Note that specifying dimensions more precise than tolerances per ACI 117 can lead to unnecessary rejection of concrete that would otherwise perform well, in addition to increased project costs with little to no benefit [13].

C. Difficult Site Access

Because many hydraulic structures are located in remote areas with limited or underwater access, access for concrete placement may have a significant impact on design and constructability. Concrete mix performance requirements are typically specified by the engineer, while concrete mix designs and placement plans are developed by the contractor. There are several methods used for concrete placements in such areas. Using preplaced aggregate concrete allows the rebar and coarse aggregate to be taken to the site in smaller quantities, transported using the contractor’s available means and methods, and placed first, followed by pumping in flowable grout. Alternatively, concrete can be pre-mixed and trucked to the site or mixed onsite and placed in hoppers (bins) on material barges (see Figure 3), or the concrete trucks can be driven onto barges and transported to the pour location. A volumetric concrete mixing truck can be used, which contains compartments carrying the mix materials which are batched onsite in the vicinity of the discharge location. Pumping concrete may be an option but can sometimes result in a large amount of waste for small batches with long pump lines as material is wasted in the pump hose. Underwater concrete placements can add significant flexibility to projects by avoiding reservoir drawdowns. Mix designs for concrete placed underwater need to include an anti-washout admixture to prevent washout of the cement and require placement plans that avoid creating large voids in the concrete. The contractor typically develops concrete placement plans.

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Figure 3. Concrete placement using bins transported by barge

D. Difficult Concrete Placements

Rebar congestion is sometimes an issue for hydraulic structures, such as at tall intake towers in high seismic zones. In these areas that are difficult to consolidate, external form vibrators or self-consolidating concrete (SCC) have been used. SCC has been used to place concrete in wall sections over 60 feet tall in a single lift [14]. More commonly, the maximum height for a single wall pour is about 30 feet.

SCC, which has a viscosity similar to water, can be used in areas of rebar congestion or where there are features, such as pipe penetrations, where vibratory consolidation is difficult. Figure 4 below shows two such locations. On the left is an intake structure with bundled rebar, having heavy rebar congestion. The structure on the right is a headwall structure with narrow piers between pipe penetrations. The concrete placement was achieved in both instances using self-consolidating concrete (SCC).



Figure 4. (Left) Rebar congestion on a wall pour (Right) Concrete placement around closely spaced pipes using SCC

Energy dissipators, such as chute blocks, baffle blocks, and dentated end sills, have tapered sections. If construction joint locations between the slabs and energy dissipators are not selected carefully, the energy dissipators may result in an edge that tapers to a thin, sharp edge, or “feathered edge". Feathered edges are highly susceptible to spalling. To avoid feathered edges, energy dissipators may be placed monolithically with the underlying concrete, which can be exceedingly difficult to construct as formwork has to be hung from above rather than supported off a base. Alternatively, block-outs can be provided in the slabs so the bottom of the energy dissipators may be recessed in the slabs (Figure 5), sometimes below the top mat of slab reinforcement, or formed joints can be constructed normal to the finished surface.

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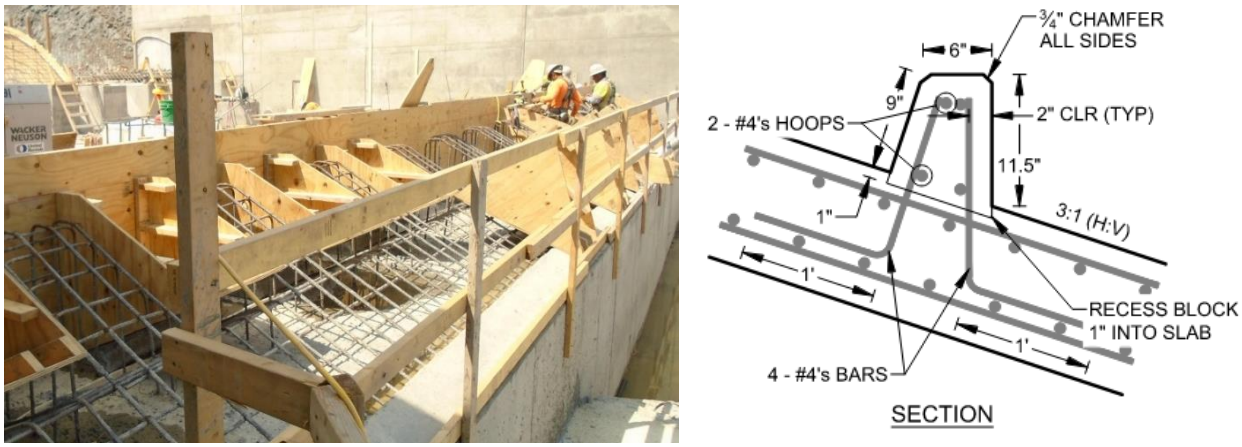


Figure 5. (Left) Inclined construction joint in end sill to avoid feathered edge (Right) Construction detail of chute block set into spillway slab.

Ogees and steep chutes present challenges for concrete placement. Using closed formwork on the top surface of ogees can make it difficult to consolidate concrete. A slot can be left open at the top of ogee formwork to allow placement. In other cases, formwork can be eliminated on the curved surface by using low slump concrete and finishing concrete to a prefabricated steel guide (or other material capable of holding its shape for reference).

Several products are available to reduce impacts on formwork that would otherwise require destructive holes or slots in formwork to accommodate materials crossing concrete joints. Some of these products include plastic inserts such as smooth dowel sleeves, rebar form savers, and split flange waterstops, which may be cast into a wall placement and allow for desired mechanical connections to a subsequent concrete placement and are shown in Figure 6. Speed dowels are plastic sleeve inserts that can be connected to formwork bulkheads and allow subsequent insertion of smooth metal dowels to create contraction joints. Rebar form savers are mechanical rebar splices consisting of threaded female couplings that are nailed to the formwork of the first pour at rebar ends and allow for threaded rebar to be inserted in a subsequent pour to form continuous rebar at construction joints. Split flange waterstops are PVC waterstops installed as a tee shape in the initial concrete placement. After form removal, the tee is straightened into an I-shape to provide equal embedment into both concrete placements at a joint, so formwork does not have to be slotted for the waterstop. Formwork is expensive, but can be moved easily, so it is advantageous to select uniform wall or slab dimensions where possible to improve constructability.



Figure 6. (Left) Smooth dowel sleeves (Center) Rebar form savers (Right) Split flange waterstop.

One responsibility of design engineers is to develop efficient designs that reduce waste and result in economical designs. However, the goal of minimizing material quantities must be balanced with other constructability considerations. For example, on tall, cantilevered retaining walls, a larger amount of rebar is required at the base of the wall compared to the top. Reducing the amount of rebar at the upper portion of the walls may save a significant amount of time and money over the course of a project. However, use of compatible rebar spacings, where all bars in a particular direction are multiples of a common spacing, is important for constructability, compared to using fully optimized rebar sizes and spacings designed to minimize the area of steel provided to meet design

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requirements, which may vary. Design focused solely on minimizing rebar area can result in rebar interferences of tied-in-place or pre-tied rebar mats and have the opposite effect of the designer's intent, ultimately increasing project complexity and costs. For example, use of rebar spacings of 6 inches at lower portions of the walls and footings and 12 inches at the upper portions of walls, where alternating bars are terminated with height, would result in a more constructable structure than if rebar spacings of 6 inches were used in the footings, 8 inches at the lower portion of the walls, and 10 inches at the upper portion of the walls. Furthermore, it is recommended to avoid using rebar with diameters less than ¼-inch different within a single element for ease of placement.

E. Slab and Footing Construction

The detailing of a slab may dictate a certain construction sequence or impose unnecessary restrictions on construction sequencing. For example, slab and footing joints like those shown in the USBR DS-14 [15] include a turndown at transverse joints where the upstream concrete placement bears on top of a seat formed in the downstream placement. This often results in one of two placement methods, both of which are shown in Figure 7: 1) downstream section is placed before the upstream section with the seat monolithic with the slab or footing, or 2) a horizontal joint is constructed between the lower portion of the seat and the upper portion. If the second option is acceptable to the designer, it is advantageous to consider this during design as reconfiguring the joint during construction may require significant rebar modifications to achieve the desired lap splices and avoid rebar conflicts with waterstops.



Figure 7. (Left) Slab turndown formwork. (Right) Construction joint for slab turndown

Steep grades can result in constructability issues, which can be addressed by tailoring construction processes and concrete mix designs to site-specific conditions. Slopes of 3:1 or steeper have been placed without forming the top, exposed surface. One way to reduce disturbance of foot traffic on steep slopes where drainage course has been placed is to use fiber reinforced concrete, which can provide shrinkage reinforcement and eliminate the need for tying rebar where construction workers would disturb the subgrade, as shown in Figure 8 (Left). Note that dosage rates for fiber reinforcement are typically limited to temperature and shrinkage reinforcement and not used where bending is a design consideration. Rotary compactors can be used to consolidate the concrete, as shown in Figure 8 (Center). Admixtures can also fulfill a key role in meeting project specific needs. For example, shrinkage-reducing admixtures can be used to increase placement sizes and reduce cracking in walls and slab overlays as shown in Figure 8 (Right).

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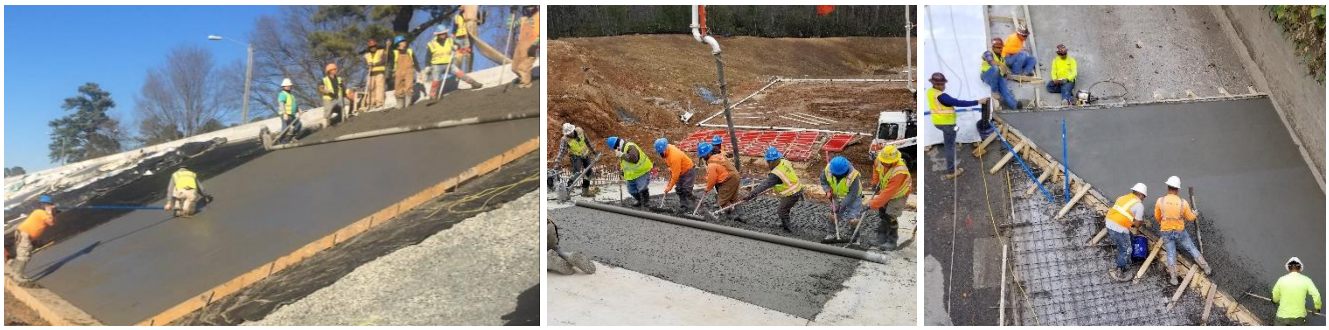


Figure 8. Concrete placement on steep slope using (Left) Fiber reinforced concrete (Center) Rotary compactor (Right) Spillway overlay with shrinkage reducing admixture.

A small but important detail of wall construction is the joint between the wall and footing, which is shown in Figure 9. For hydraulic structures, this joint typically includes a bulbed PVC waterstop. The height of the waterstop, which is often 6 inches or more, would typically interfere with the top mat of rebar or have insufficient clearance if the centerbulb is placed at the top of footing. Various approaches to this have been used, including mounding the concrete up at the waterstop, forming a trapezoidal key that sticks up into the wall pour, placing a wall stub section about 6 inches tall that is monolithic with the footing, or increasing the amount of concrete cover over the top mat of footing rebar. Depending on the engineer and contractor, opinions on constructability of the mound or key options vary from standard practice to extremely difficult. Therefore, engaging contractors, whenever possible during design development, can prevent inflated costs and help tailor the design to the contractor's preferred practices.



Figure 9. Wall-to-footing construction joint details to accommodate waterstops:
(Left) Mockup of waterstop support (Center) Top mat rebar low enough to accommodate waterstop (Right) Wall stub placed with footing

The upper portion of walls sometimes need to be widened to support other items, such as bridge seats, railing, fencing, or personnel walkways. Corbeling should generally be limited to the inside face of the spillway walls, out of the flow path. Using corbels on the outside face of walls where backfill extends above the base of the corbel should typically be avoided as it results in a location where backfill of embankment materials is extremely difficult and may result in preferential seepage paths, which pose a dam safety concern.

Concrete spillway construction on soil subgrades can result in constructability issues due to the potential for rain, which can disturb subgrade preparation. One way to reduce the potential effects is to use a mud mat, which is a thin layer of lean concrete, having a minimum thickness of 2 to 3 inches. Additional benefits of using mud mats can be providing a level base to install formwork and seal against. The soil subgrade may also be intentionally roughened to improve sliding resistance, whereby the mud mat penetrates depressions in the soil and forces a sliding failure plane through the soil, as shown in Figure 10. This may allow designers to consider the internal friction angle of the soil at the sliding failure plane rather than the interface friction angle for concrete on soil, which typically is a lower value and would generally require a larger footing design.

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Figure 10. Mud mats placed on soil subgrades to support cast-in-place concrete placement.

F. Roller Compacted Concrete

Roller compacted concrete (RCC) has been used, primarily for concrete gravity dams and overtopping protection on embankment dams. While the constructability of RCC can depend on a variety of factors, there is generally no minimum or maximum volume for it to be considered cost-effective, as placements have ranged from less than 3,000 to more than 1,000,000 cubic yards at dam projects. The major cost components are the onsite batch plant rental and transportation costs for aggregates, so the closer the quarry to the project, the more cost-effective RCC will be. RCC is often placed in horizontal lifts that are nominally one foot thick; however, RCC has been placed on slopes of 4:1 (horizontal to vertical) and possible to place on steeper slopes, such as 3:1, depending on the roller type and RCC moisture content. RCC production rates can be relatively high, typically with production rates of around 400 cubic yards per day. RCC lift thickness tolerances are typically +/-1 inch, which may create constructability challenges where the RCC is used as base for reinforced concrete, such as spillway sidewalls, particularly where there is sloped reinforcement, as shown in Figure 11 below. Increased tolerances on minimum concrete cover to rebar is likely a more economical solution than using tighter tolerances on RCC deviation from neat lines. The minimum width of a concrete placement is typically based on the rollers used for the project and is typically a minimum of 6 feet. Creating a test section is important for verifying constructability the concrete mix using project personnel and equipment and can be used to vet processes and alleviate issues before impacting project production.



Figure 11. Variation of RCC steps below reinforced concrete sidewall footings.

IV.GEOTECHNICAL CONSIDERATIONS

The interface between embankment construction and spillway walls is important to avoid the creation of problematic areas, such as preferential seepage pathways. Corbels have sometimes been used to flare out the top of walls for several reasons, such as support of guardrail or fencing, personnel access, or equipment placement. Compaction adjacent to walls is already difficult due to potential for impact by equipment or wall overloading due to

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heavy equipment, so compaction equipment within close proximity to walls (usually within about 5 feet) are limited to trench rollers, jumping jacks, or other manually operated compaction equipment, as shown in Figure 12. Corbels make compaction extremely difficult. Poor compaction can lead to large settlements of embankment material and if hydraulically connected to a source of water, can allow preferential seepage. For these reasons, corbels should be avoided on the backfill side of walls.



Figure 12. (Left to Right): Drum roller, trench roller, and handheld tamping compactor equipment used adjacent to concrete wall.

Another area that is difficult to compact around is pipes. Pipe cradles or encasements are commonly used. Cradles consist of concrete placement below the spring line and battered faces. The outer edges should be battered to improve compaction of adjacent fill material. Encasements using Flash Fill or flowable fill can be placed around the pipe. Also known as Controlled Density Fill (CDF), Flash Fill is an easily flowable cementitious material that can fill voids under pipes that otherwise would be unreachable with more common materials. In addition, it can be easily excavated should pipe maintenance be required.



Figure 13. Flash fill along length of pipe

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V.OTHER CONSIDERATIONS

Constructability is often subjective, and opinions vary among owners, engineers, designers, and contractors. However, one recurring theme in constructability is this: “the simpler, the better.” For projects where significant or non-standard constructability challenges may arise, owners and engineers without formal constructability programs should consider performing in-house constructability design reviews or consider alternative project delivery methods that veer from the traditional Design-Bid-Build process, such as Design-Build; Engineering, Procurement, and Construction (EPC); Early Contractor Involvement (ECI); Construction Management at Risk (CMAR) or other methods that solicit constructability feedback from a contractor during the design of a project.

Design-Build and EPC are both turn-key processes for the owner. The owner hires a team to design and construct the project. The primary difference between Design-Build and EPC is that the EPC contractor is also responsible for the timely performance of the project.

The primary difference between Design-Build and EPC, as compared with CMAR or ECI is that Design-Build and EPC have the contractor and engineer work as a single entity for the owner. CMAR and ECI design places the responsibility of the engineer, who is contracted directly to the owner, separately from the contractor. For CMAR projects, the construction manager (which is usually a general contractor) is hired by the owner, establishes a guaranteed maximum price, and is financially responsible for project overruns, but is able to collaborate with the design team prior to construction. The owner’s participation in constructability reviews or ability to make changes is diminished for Design-Build, CMAR, and EPC after the contract is signed because the design-builder, construction manager, or EPC contractor is responsible for controlling the project [16]. For owners who are required to publicly bid projects, an alternative way to obtain constructability feedback during design may be to hire the constructability reviewer separate from the design engineer and contractor or have the design engineer contract with the constructability reviewer as a subconsultant.

Throughout design and construction, clear and frequent communication between the owner, engineer, trusted contractor, and reviewer is beneficial to identify potential constructability issues and develop solutions to will reduce the need for requests for interpretation, schedule delays, and cost increases. Training and refresher courses are encouraged at regular intervals to ensure efficiency and consistency with best practices. Post-construction meetings are also vital to sharing lessons learns and continued improvement on future projects.

There are a variety of general resources available for constructability, but very few are tailored to dams and spillways. Within those resources, constructability is inferred in “best practices”. Engineers, owners, and contractors should maintain close professional relationships and budget for pre- and post-construction meetings with the intent to improve efficiency and safety and to develop constructability standards unique to each project.

VI.CONCLUSION

Constructability matters to all parties involved in dam and spillway projects. Opinions may justifiably vary based on a person’s previous experience, available resources, or other factors. Constructability is sometimes in the eye of the beholder, but in general, simpler is better when it comes to constructability. Engaging experienced professionals and sharing lessons learned early and often is important to the design and construction process, advancing the industry, and managing owner’s limited resources. Benefits of constructability include reduced design and construction time, reduced construction time, and improved quality of the product.

For especially complex projects, non-standard projects, or those with ambitious schedules, project teams can benefit from project delivery methods such as Design-Build, ECI, or EPC, or where owners, design professionals, and contractors can collaborate on issues during design to improve constructability.

VII.ACKNOWLEDGMENT

The authors gratefully acknowledge the respondents to our inquiries about organization constructability programs. The authors also would like to acknowledge the contributions of individuals and project teams who provided input and figures included in this paper as lessons learned for the industry and continually invest time and effort to promote industry innovations and meet the constructability challenges facing dam and spillway construction.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

Designing or Evaluating a Spillway? Chute Condition Matters!

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ABSTRACT

While designing a spillway, an understanding of the conditions at the spillway chute and their impacts to conveyance and overall hydraulic behavior is required to provide an efficient, sustainable solution. Some commonly observed undesirable conditions at chutes include changing geometry, insufficient training walls, super elevated chute floor, changing roughness, potential for air entrainment and flow bulking, structural impedances including bridge crossings and piers, ball milling, and flow anomalies caused by improper gate operation. Other not-so-common conditions such as sudden increases in stream power, stagnation pressure, and cavitation can also impact a spillway's hydraulic performance. Oftentimes well-defined relationships for these various chute conditions on spillway performance are not available, and therefore, their impacts are difficult to evaluate. Such conditions, when left unevaluated and unremedied, can result in a hydraulics-induced compromise of the chute that can ultimately lead to a failure of the spillway. This presentation will provide a perspective on such spillway chute conditions that are sometimes overlooked, but can adversely impact spillway performance. Actual spillway case studies will be used as examples. Recommendations for confirming spillway performance using CFD modeling will be discussed.

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How Low Can You Go - Whittier Narrows Erodibility Study

CHRIS THORNTON, P.E., AND JEFF ELLIS, COLORADO STATE UNIVERSITY; ETHAN THOMPSON, US ARMY CORPS OF ENGINEERS; AND DANIEL ROGERS, STANTEC CONSULTING SERVICES, INC.

ABSTRACT

Whittier Narrows Dam (WND) is an earthen dam, approximately 3 miles long, located on the Rio Hondo and San Gabriel Rivers in Los Angeles County, California. The dam is one of the main elements of the flood control system for the Los Angeles County Drainage Area, while supporting water conservation. The probable maximum flood would overtop the dam by about 4 feet. Due to the high consequences of breach resulting from flow overtopping the dam, the U.S. Army Corps of Engineers is developing plans to armor the crest and downstream face of the dam to prevent failure during overtopping. The downstream face will include a stepped, non-erodible material designed to help dissipate energy of the overtopping flow. Of prime interest was the erodibility of a proposed hardfill (soil-cement) mix used to form the steps of the downstream face. Hardfill mix design incorporates a low concentration of cement (usually ~10%), mixed with non-uniform gravel or site-specific material to a specified moisture content. Construction and placement techniques vary, however hardfill mix is typically placed in lifts to produce steps. Realizing that reducing cement content significantly benefited project costs, a prototype scale study was performed to quantify the erodibility of a hardfill mix with varying cement content. Site specific material and proposed mix components were identified and transported to the laboratory where multiple 15,000 pound specimens were constructed, cured and placed in an outdoor flume. Increasing flow rates, up to the prototype unit design discharge, were conveyed over each specimen and erosion quantified through post test LiDAR scans. Testing quickly identified flow stability concerns on the proposed step geometry and a 1:4 Froude scale sectional model was constructed and operated to confirm flow stability and tailwater requirements. This presentation will present the findings of this erodibility study as well as a lesson learned on the impact of chute slope on flow stability.

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Sites Reservoir

DEREK S. MORLEY, P.E., GEOSYNTEC CONSULTANTS

ABSTRACT

This is not an individual presentation within a Traditional Technical Session; rather, this will be a full, themed Session, consisting of multiple Traditional and/or Grab-N-Go length presentations on various aspects of the planned Sites Reservoir project. The planned Sites Reservoir will be the largest new reservoir in California in decades. The reservoir will provide 1.5 million acre-feet of off-river storage, supplied by canals routing stormwater flows from the Sacramento River. The main reservoir will be impounded by two large zoned earthen dams, each close to 300 feet in height, plus multiple zoned earthen saddle dams. The project includes new pipelines and canals, pumping and hydropower-generating facilities, power transmission facilities, and roadway rerouting. The project is situated in a highly seismic region, creating a key driver for dam design. Design for the many types of facilities comprising the project is being performed by a team of consulting firms, integrated under the Sites Project Authority. Members of the team will present on various aspects of the project:• Project Overview: A Robust Solution for a Critical Need (HDR)• The Big Dams: Design Approach for Sites Dam and Golden Gate Dam (AECOM)• Using the Site Effectively: Planning the Sites Reservoir Saddle Dams (GEI)• The Heart of the System: Developing the Regulating Reservoirs (Geosyntec)• Wise Use of Water: Evaluating Pumping, Releases, and Generating Scenarios (Jacobs)

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DAM SAFETY PALM SPRINGS
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The Engineer-of-Record's Role in Risk Informed Decision Making in Levee Design

DANIEL N. TEAK, P.E., AND KIMBERLY LUKAS, P.E., CPESC, HDR

ABSTRACT

Risk Informed Decision Making (RIDM) is becoming more common in standard levee design practice. The United States Army Corps of Engineers (USACE) is actively updating their guidance documents to highlight the use of RIDM in levee design. Historically, levee design has been primarily criteria- and regulation-based, leaving little room for deviations. This has allowed the Engineer-of-Record (EOR) to confidently stamp levee designs as they are protected by the regulations and guidelines they adhere to.In dam design, RIDM has often led to increasing design criteria to lower or meet tolerable risk as well as designing to mitigate risks not necessarily covered by criteria or regulations. When using RIDM in levee designs, it may offer a unique opportunity to change the required criteria to allow for more affordable flood control infrastructure under certain circumstances. In these situations, a Risk Assessment is conducted by an independent panel of experts, separate from the EOR and their design team. The Risk Assessment is not a product that is signed or sealed by a professional engineer; though, the EOR will need to rely on the information contained in the Risk Assessment to inform the design. In the event the Risk Assessment recommends changing the criteria for the levee design, the EOR, responsible for the standard of care for the design, may have concerns about incorporating decisions they were not involved with. This could lead to a potential unwillingness to incorporate the Risk Assessment recommendations without the professional assurance that they will not be held liable for the Risk Assessment's team decisions.With RIDM still relatively new to levee design, this is a dynamic and changing topic. In this presentation, we will be discussing several instances where certain design criteria were reduced based on the recommendations following a Risk Assessment, and the rationale behind the EOR's inclination to accommodate the lowered criteria. We will also offer best practices for EORs who will be sealing designs that incorporate RIDM.

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Risks and Approach of Private Infrastructure on Public Levee ROW

VICKI TWERDOCHLIB, PE, CFM, PROFESSIONAL ASSOCIATE, HDR; AND **GEORGE PARIZEK, PE, LEVEE SPONSOR, CITY OF OMAHA**

ABSTRACT

Levee sponsors that operate federally authorized levee systems are tasked with the job of working with USACE to manage risk to their levee systems. Levee systems frequently intersect with other public and private infrastructure that levee sponsors are not generally responsible for maintaining. Due to the intersection with the levee, the levee sponsor is now thrust into a position where they have a duty to verify compliance for the purposes of reducing risk of life safety. Private infrastructure that parallels or intersects the public levee ROW without clear divisions of responsibility can present operational and USACE PL 84-99 program compliance challenges.

In the USACE Engineering Circular (EC) 1165-2-218 *USACE Levee Safety Program*, USACE documents general expectations on levee risk communication and risk management that are accomplished by partnering with local levee sponsors. While risk communication and risk management may look different for USACE and for the local levee sponsor, local levee sponsors have best practices that have proven useful in tackling existing conditions that may introduce risk to their levee systems, such as at locations with private infrastructure intersections.

The purpose of this presentation will be to describe two examples of one levee sponsor's active risk management and risk communication with private landward property owners with private infrastructure within public levee ROW. The presentation will review these two examples and show how the communication, management of the expectations, and coordination with the property owner resulted in modifications that resulted in reduced risk for the levee sponsor as well as a better understanding of levee purpose by the landward property owners.

I. INTRODUCTION

Levee sponsors are responsible for maintaining complex levee systems that require multiple different types of operations and maintenance needs. Sponsors also maintain working relationships with a multitude of adjacent landowners and stakeholders. These relationships often require notifications of impending flooding, levee work or USACE visits, required compliance with levee programs, and any required deficiencies that these land owners may be responsible for mitigating. All of these activities are all part of a sponsor's required daily activities, regardless of whether the levee is enrolled in the USACE Rehabilitation and Inspection Program[1] (RIP) or not.

The case studies presented herein represent examples of third party infrastructure on levee right-of-way that was contributing to repeated deficiencies. The case studies herein represent two different property owners that the City coordinated with in order to rectify the deficiency and thereby reducing risk to the City's levee. The City of Omaha, as a contributor towards this article, has developed a proactive policy of coordinating with landward property owners and stakeholders and providing them with updated information with respect to levee policies. This

[1] The RIP, or Public Law (PL) 84-99 program, is managed by USACE. As a levee sponsor with active status in the program, the sponsor agrees to a set of operation and maintenance criteria that are evaluated during routine inspections. This program provides for flood fighting and flood damage repair assistance provided the levee sponsor has active status.

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coordination and relationship cultivation has led to ongoing partnerships with these stakeholders and the levee sponsor through multiple flood fights and project improvement activities along the levee system.

II. BACKGROUND

The City of Omaha Missouri River Levee (Omaha Levee) is located on the right bank of the Missouri River between approximate river mile 625 at the upstream end to river mile 611 at the downstream end. See Figure X for a figure of the levee alignment with respect to the Missouri River. The Omaha Levee is over 13 miles long and consists of approximately one mile of floodwall, with the remaining 12+ miles consisting of earthen levee. The levee system was authorized as part of the 1942 Flood Control Act, with construction predominantly being completed by 1952.

At the time that the levee was originally constructed, there were both gravity outfall structures and pump stations that were designed to drain through the levee and outfall riverward of the levee. Table 1 provides the summary of the gravity outfall structures in place at the time the levee was initially constructed by USACE. The table represents 23 unique crossing locations of either the levee system or floodwall. Since that time, the City has processed two updates to the O&M manual and are in the process of undergoing a more comprehensive update. Additional pipes through the levee have been constructed, consisting of both those constructed by the City and those constructed by landowners.

TABLE I
Summary of Gravity Outfall Structures Documented in the 1987 O&M Manual

Outfall Name	Outfall Description
Florence Lake	- Underseepage pump station
Carter Lake	- Pump station
Grace Street	- Pump station
Burt Izard	- Pump station
Chicago Street	- 54-inch diameter
North Downtown Trunk Storm Sewer	- 108-inch diameter
Douglas Street	- 36-inch diameter gravity and pump station
Farnam	- 42-inch diameter
Harney Street	- 48-inch diameter gravity and pump station
Jones Street	- 14’ x 8’4” gravity and pump station
Leavenworth	- (2) 8’4” x 10’ gravity
Mason Street	- 16-inch diameter
Poppleton Avenue	- 30-inch diameter
Pierce Street	- Pump station
Hickory Street	- Pump station
Spring Street	- Pump station
Hascall Street	- 24-inch diameter
CB&Q Railroad	- 24-inch diameter
“I” Culvert	- 24-inch diameter
“K” Culvert	- (2) 66-inch diameter
“N” Culvert	- 48-inch diameter
“P” Culvert	- 36-inch diameter
“Q” Culvert	- 24-inch diameter

See Figure 1 with the current levee and floodwall alignments with the existing levee drainage and pump station structures identified. See Figure 2 that shows the two drainage structures (Kinder Morgan and Magellan Storm Outlets), that will be the focus of this paper. The following sections will describe the two gravity outfall locations in more detail, discuss the landowners, and discuss the methodology behind how the City coordinated with both entities to make improvements to both structures that resulted in reduced risk to the levee.

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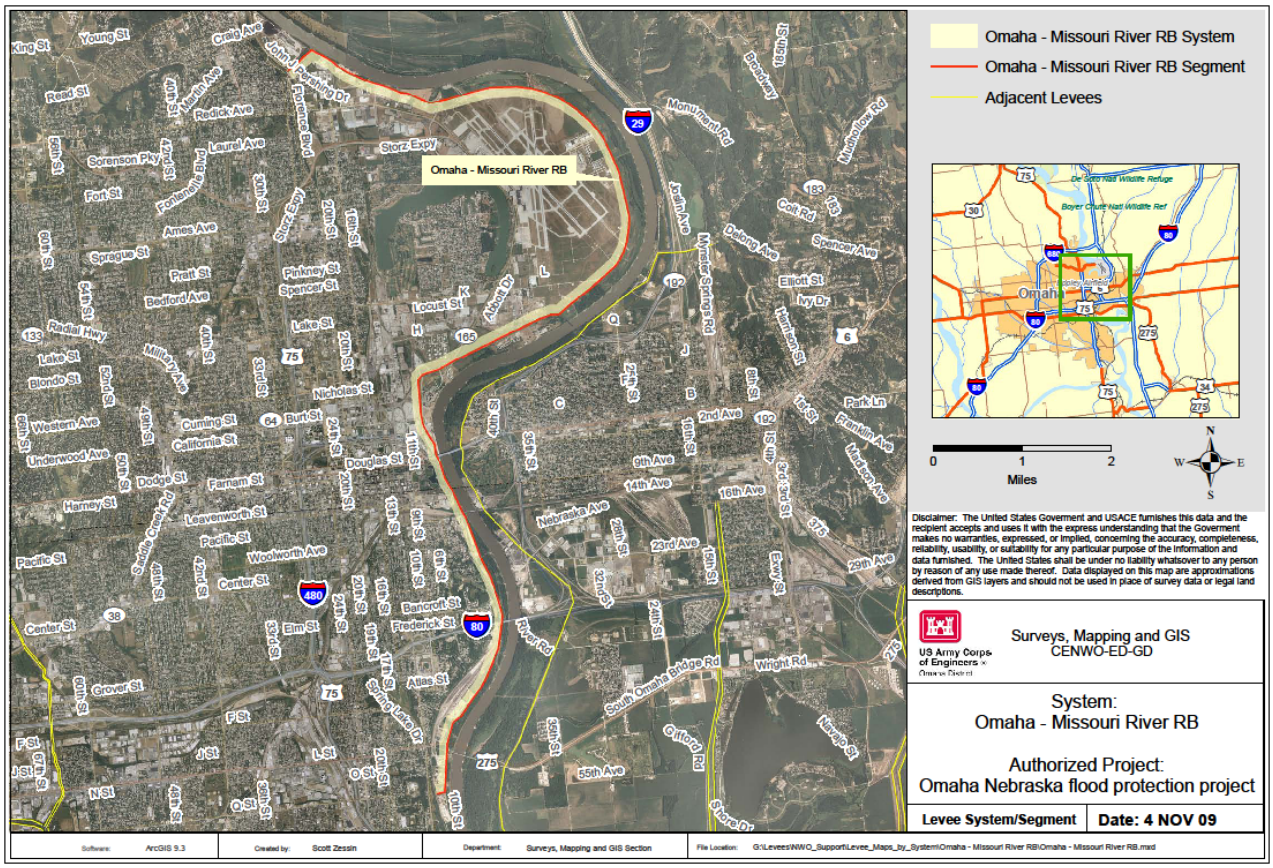


Figure 1: Current Omaha Levee and Floodwall Centerline Alignments

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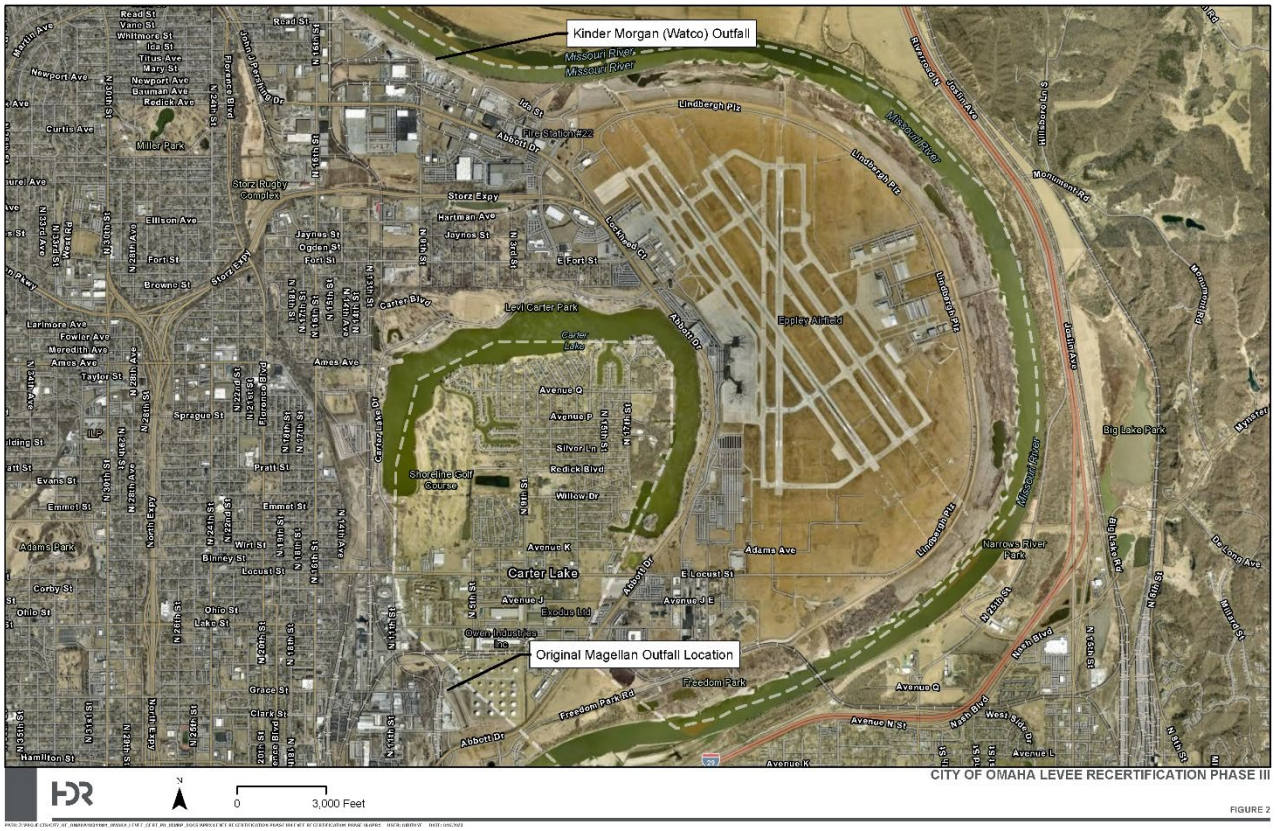


Figure 2: Aerial Figure of Kinder Morgan and Magellan Outfall Locations

III. KINDER MORGAN OUTFALL

This outfall was assigned it's name when Kinder Morgan operated the terminal. Kinder Morgan has since sold this terminal location, and Watco operates the terminal today. The outfall was constructed to provide stormwater conveyance from the terminal site into the Missouri River. See Figure 2 showing the existing site location and outfall. The following sections provide more information about the outfall itself, the circumstances that led to concerns about the stormwater conveyance arrangement, and the ultimate proposed solution.

A. Original Outfall

The original outfall consisted of 30-inch RCP outfall with a manhole on the landward side of the levee, a gatewell with a handwheel operator on the slide gate, and an outfall structure on the riverward side of the levee. See Figure 2 showing the configuration of the outfall. The upstream manhole was connected to a storm sewer conveyance network that included one RCP segment parallel to the levee on levee ROW, and additional storm sewer and manholes off levee ROW. The two manholes on levee ROW consisted of inlet tops with throats on the side of the structure rather than the top. The throats were originally located at the natural ground surface to collect sheet flow runoff on the site.

B. Kinder Morgan Outfall Concerns

During the 2011 Missouri River flood fight, underseepage was a concern for the levee in this reach. In response to increased underseepage concerns during the extended flood fight, USACE placed a rock blanket on the dry side of the levee within this reach, including on the dry side of the levee where this drainage structure was located. Following the flood fight, the City elected to keep the rock blankets in place and not remove them. The City eventually followed up with USACE to get permission to keep them as a permanent feature of the levee system.

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Due to the throats of the storm inlets being at native ground surface, seepage rock fell into the storm structures over time. The aggregate placed as part of the rock blanket eventually filled up one portion of the storm sewer on levee ROW.

Given the private nature of the storm sewer conveyance system being on levee ROW, the City had increasing concerns about the ability for the City to continue to operate, maintain, and flood fight this portion of the levee given the private storm sewer system. The City also had concerns over the continued migration of rock blanket material into the storm sewer conveyance system.

Given the continued migration of rock blanket material into the storm sewer conveyance system, Watco had increasing concerns about the ability of their site to drain and their storm sewer conveyance system to operate without obstructions.

C. Coordination Between Watco and the City

As a result of the concerns of both parties, Watco and the City conducted a series of meetings to understand the requirements on both sides and determine a path forward. The City requested that Watco relocate their storm sewer assets off levee ROW. Watco indicated they would pay for the construction of the improvements and the City committed to pay for the engineering of the revised design and the Section 408 authorization.

As a result of the coordination, Watco agreed to construct two new inlets outside of the levee ROW on Watco property. Watco agreed to construct new RCP storm sewer between these two new inlets and between one of the new inlets and the existing inlet upstream of the levee penetration.

The City of Omaha agreed to abandon the existing pipe within the levee ROW in addition to the second inlet located on levee ROW with flowable fill. See Figure 3 for the proposed layout. This described work involved the following:

1. Constructed over 160 feet 18-inch RCP with flowable fill bedding up to the spring line of the pipe
2. Constructed two new inlets
3. Connected proposed 18-inch RCP with existing inlet
4. Removed existing inlet top directly upstream of the levee penetration, removed debris and rock, and raised inlet top by 2 feet

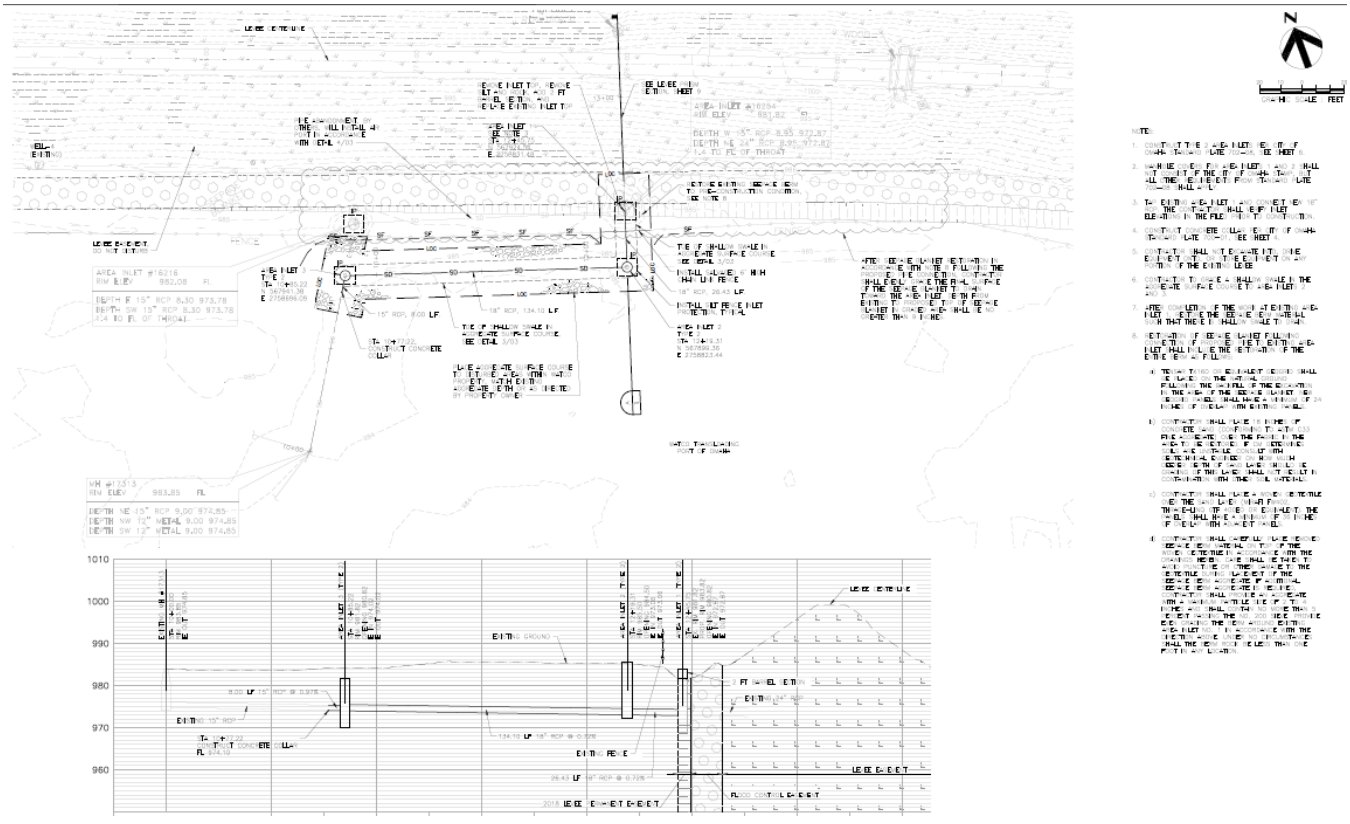


Figure 3: Modified Layout of the Kinder Morgan Stormwater Conveyance System

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The storm sewer repairs were completed by open cut excavation, laying the slopes back at a 2 foot horizontal to 1 foot vertical slope. No shoring was utilized for construction.

The contractor connected the new storm sewer to the existing inlet on levee ROW. The contractor had to remove the existing rock blanket, segregate the excavated materials, and reinstall the rock blanket in accordance with the cross section the USACE used during initial construction.

USACE Section 408 authorization was granted for this construction approach. The inlet upstream of the levee penetration was located within the projected levee prism. The City was trying to keep the construction outside of the projected levee prism, to the extent possible. See Figure 4 for the cross section of the penetration showing the upstream inlet with the proposed pipe that was being connected to it.

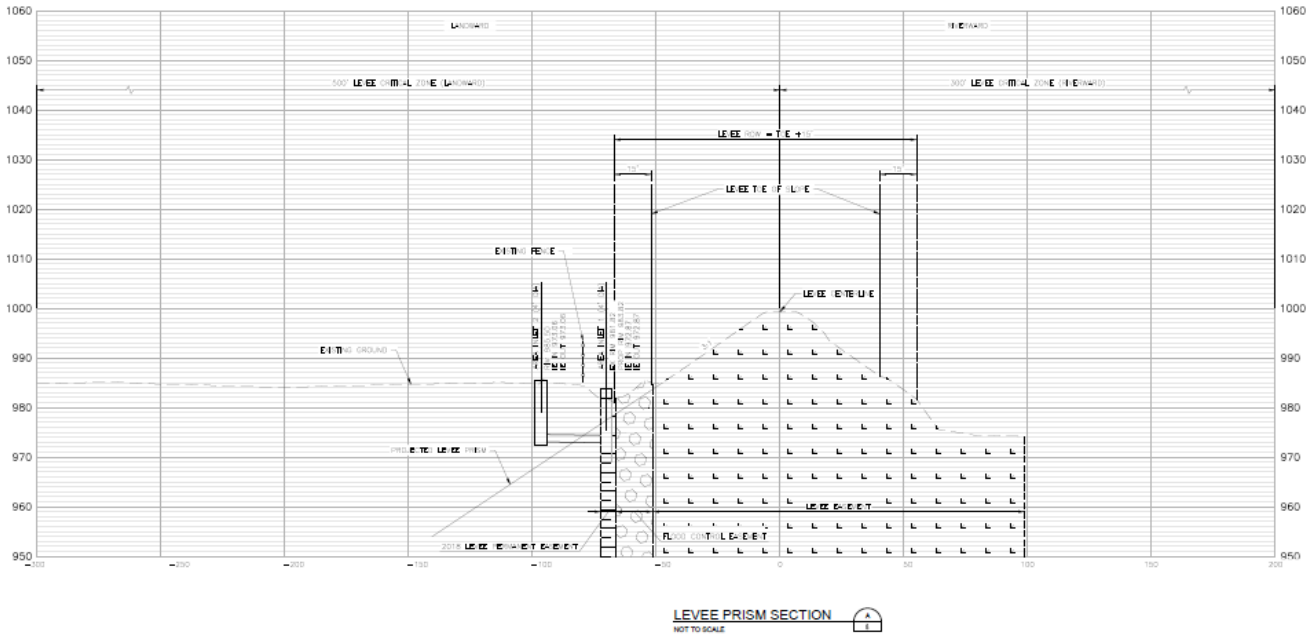


Figure 4: Cross Section Showing Landward Construction at Location of Projected Levee Prism

IV. MAGELLAN OUTFALL RELOCATION

Magellan operates what they call a refined asset terminal on the landward side of the Omaha tie-back levee along Grace Street Ditch. See Figure 2 for the overall location. As part of this site, there are several tanks with earthen secondary containment systems as well as stormwater runoff conveyance system. See the following sections for more information on this system.

A. Original Outfall

See Figures 5 and 6 for a depiction of the original outfall alignment conveying stormwater runoff into the Grace Street Ditch. The area inlet is the downstream connection point of collected stormwater runoff from the Magellan site as well as having an inlet top that collects surface water runoff from the area draining to it. The area inlet is the upstream structure for the levee penetration draining into the Grace Street Ditch.

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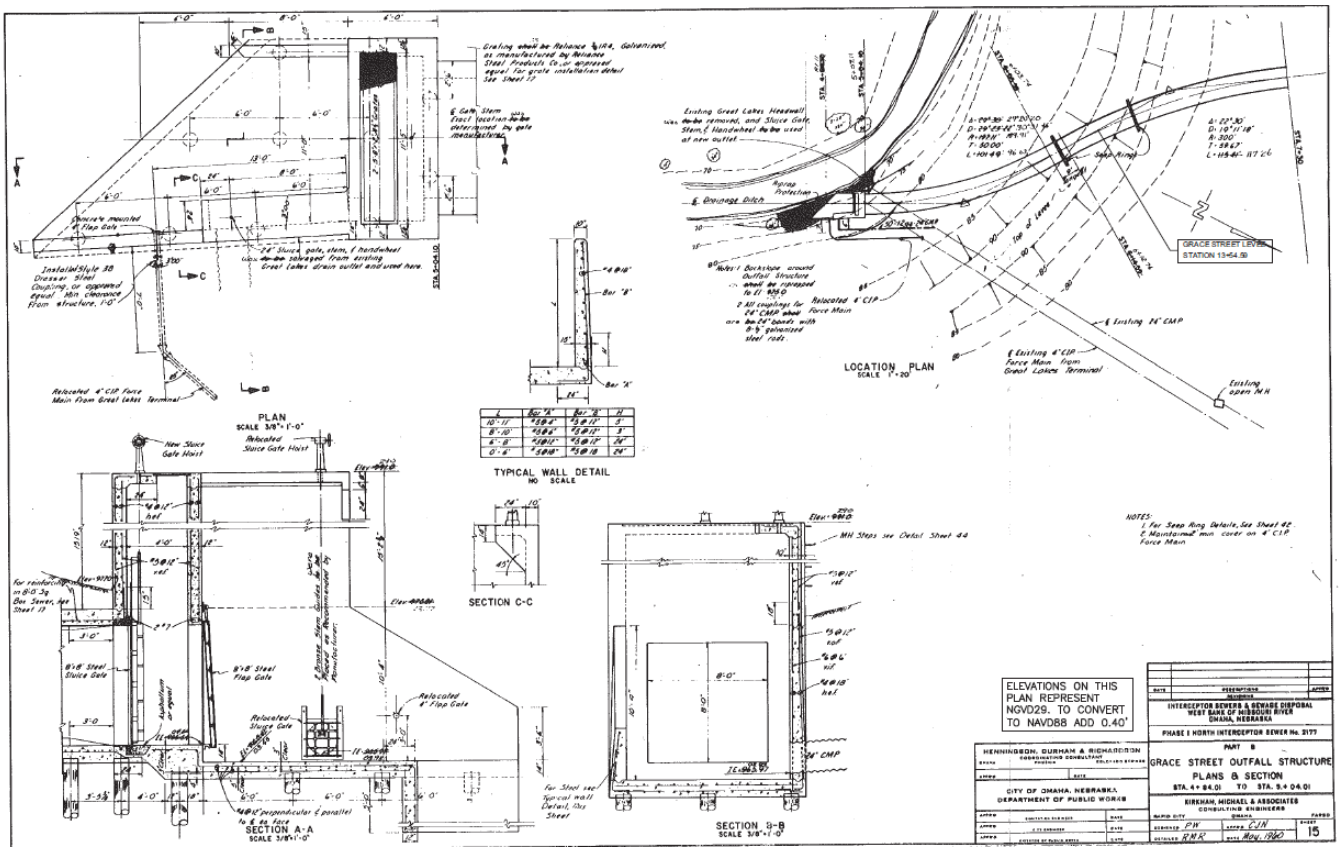


Figure 5: Original Levee As-Built Drawings Showing Original 24-Inch CMP Outfall Pipe

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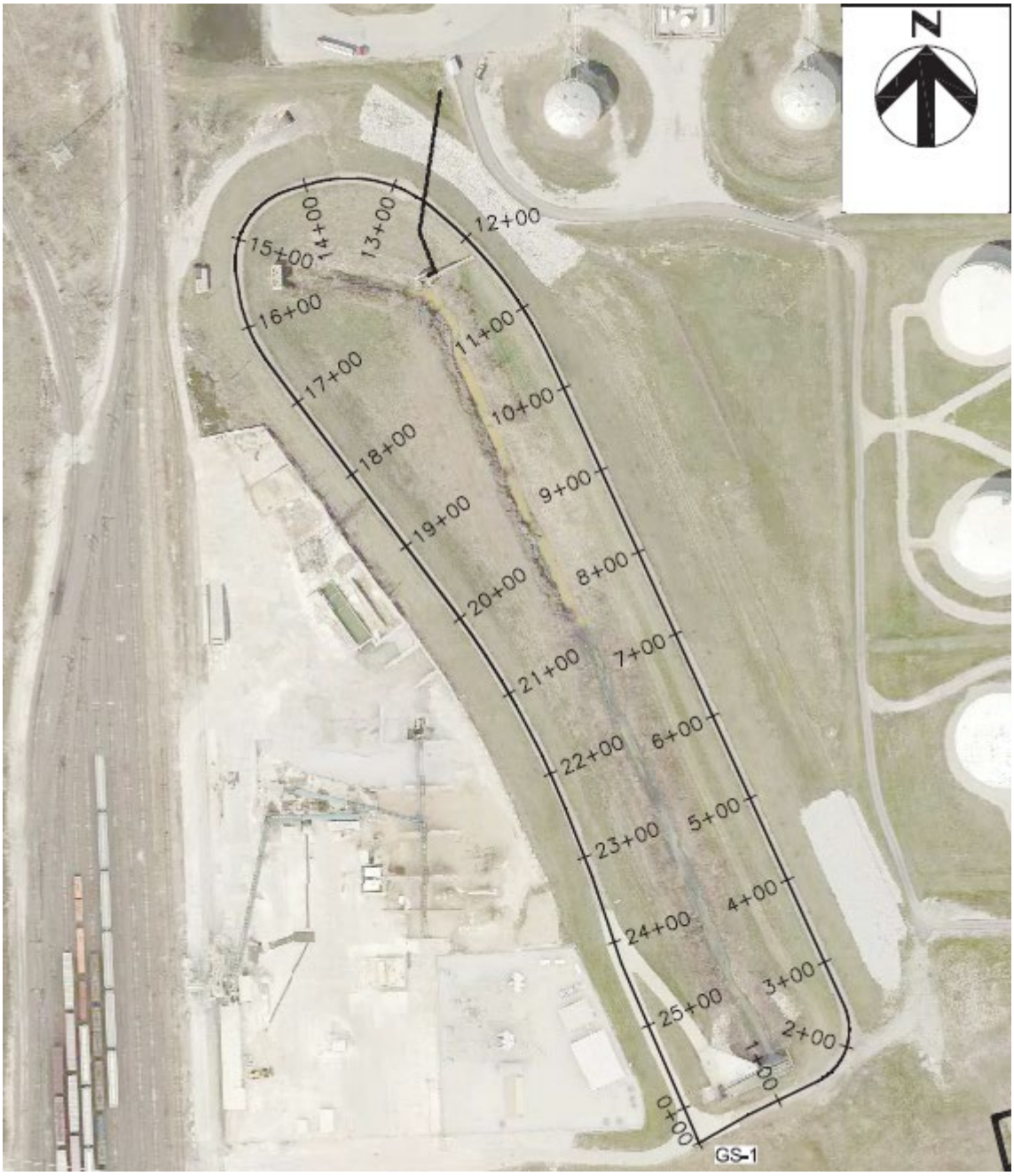


Figure 6: Aerial Plan View with Depiction of Original 24-Inch CMP Outfall Pipe

The levee penetration consisted of a 24-inch CMP pipe that discharged into the Grace Street Ditch with a flap gate connected to the outfall structure. See Figure 7 for a photo of the outfall structure. This was a unique headwall structure that served as the outfall headwall for two different levee penetrations. The gate shown on the south side of the outfall structure is the downstream discharge location for the North Interceptor levee penetration. As shown in Figures 5 and 6, the 24-inch CMP levee penetration had a bend in the pipe within the levee prism.

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Figure 7: Photograph of Original 24-Inch CMP Outfall Structure

B. Magellan Outfall Concerns

As a result of Omaha's continued levee recertification efforts and in maintaining compliance with the USACE PL 84-99 program, the City pursued condition assessment on all of their gravity lines through the levee system. The February 2020 condition assessment on this 24-inch CMP through the levee system was accomplished in accordance with the following:

1. On day one of the condition assessment, the provider attempted to access the downstream end of the 24-inch CMP. There was a rubber plug in place within the pipe upstream of the flap gate. There was no record of who placed the plug and coordination with the landward stakeholders did not identify the source of the plug, so the City authorized the condition assessment provider to remove the plug. Following the removal of the plug, the provider immediately noticed an odor consistent with oils, fuels, or gas. A gas detector was inserted and reading demonstrated that it was unsafe to further pursue cleaning and CCTV operations that day.
2. After two full days, the condition assessment team returned to the site and determined it was safe to resume cleaning and CCTV activities. The provider started cleaning the pipe from the downstream end but the jet nozzle kept getting stuck, so they abandoned further attempts at cleaning out of concern that the pipe would be further damaged.
3. The provider pivoted to a CCTV approach for condition assessment. The CCTV was originated at the upstream end of the pipe and was abandoned approximately 110 feet into the 215-foot long pipe due to the fact that the camera was about to be submerged in standing water.
4. The provider pivoted to performing CCTV from the downstream end. A smaller camera was used since there are two pipe bends that had to be navigated, which were located just upstream of the flap gate.

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Once the provider got past the two bends, the CCTV was able to continue for 60 feet due to the fact that the camera was about to be submerged in standing water.

5. Joints were in relatively good shape within the section that was available for CCTV, but in general the pipe was heavily corroded, there was pipe bottom loss, and there was concern about the condition of the pipe to continue performing condition assessment cleaning activities. As a result of the heavy corrosion present, the apparent sag within the section of the CCTV that could not be viewed due to the fact that the pipe was holding water, and the pipe limitations that forced the condition assessment provider to stop cleaning, this levee penetration was considered unacceptable as a result of the condition assessment review.

C. Coordination Between Magellan and the City

As a result of the realities of the pipe condition assessment, the City reached out to Magellan to coordinate with respect to the path forward with respect to this levee penetration. The City referenced the levee system's top priority as providing health and safety benefit to the public. The City further demonstrated that their budget limitations prohibit the City from being able to fund any repair or replacement of private infrastructure that intersects with their levee system.

As a result of the good working relationship the City has developed over several decades with Magellan and the proactive approach the City took, Magellan moved forward with hiring a consultant to assist with the design and authorization of a full abandonment of the levee penetration using flowable fill. The proposed project did not involve a new levee penetration, but rather rerouted the stormwater flows to a nearby diversion structure to take advantage of an existing outfall into the Grace Street Ditch. See Figure 8 for the revised storm sewer system alignment that has been modified. The blue linework depicts the newly constructed storm sewer alignment while the yellow and black alignment depicts the alignment that was abandoned in place, in accordance with requirements of EM 1110-2-2902.

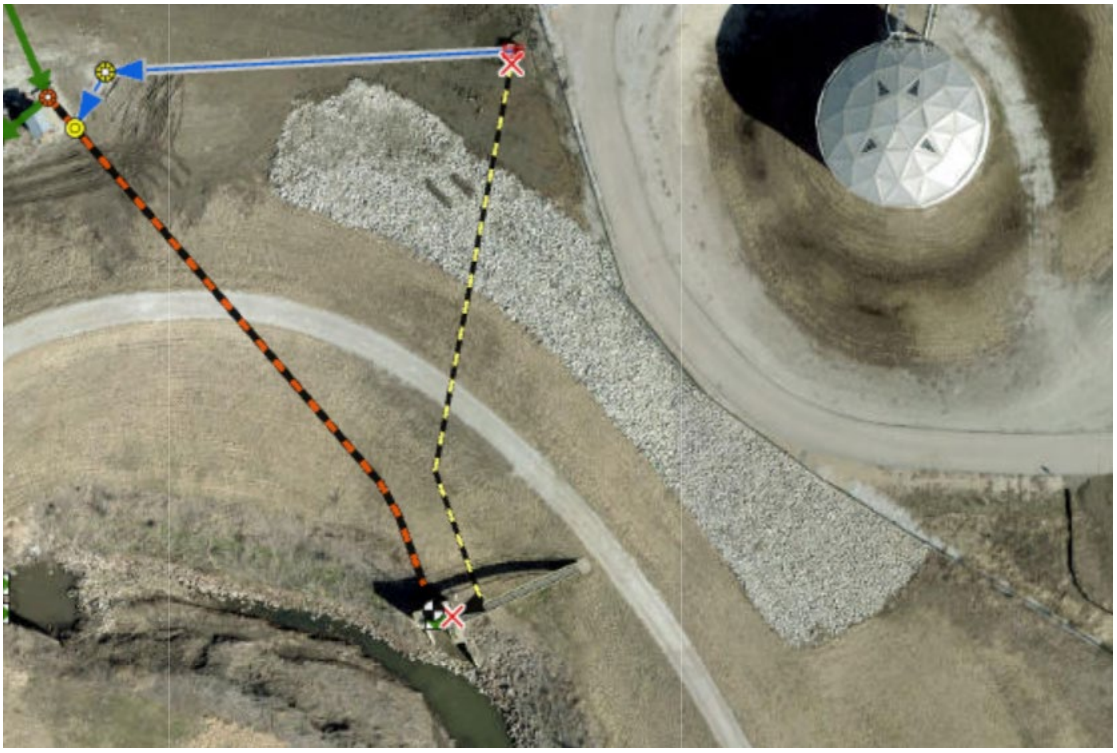


Figure 8: Current Alignment of Revised Stormwater Conveyance from Magellan's Site

This revised layout for the storm sewer pipe was designed, underwent review with the City of Omaha, received authorization from USACE as part of the Section 408 process, and proceeded through construction. Construction has been completed. Construction entailed the rerouting of stormwater flows in accordance with the proposed improvements shown on Figure 8, abandonment of the existing storm sewer aligned through the existing levee embankment to the outfall shown on Figure 7, and the revision of the City's GIS records to show the revisions to

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the proposed project. Magellan will continue to retain operation and maintenance responsibilities for the pipe conveying stormwater from their site. The revision to the pipe alignment removes an active penetration from the levee for the City to document and for USACE to inspect.

V.SUMMARY

The USACE Levee Safety Engineering Circular (EC) 1165-2-218 documents a variety of conditions when USACE and the levee sponsor are required to communicate levee performance information with those stakeholders behind the levee system. That stakeholder communication becomes even more important when stakeholders are responsible for the operations and maintenance of private infrastructure through a levee managed by a local levee sponsor.

As showcased in the examples presented here, the City of Omaha has developed and maintained good working relationships with these two specific stakeholders that operate on the landward side of the levee and manage private infrastructure going through the levee. As a result of these cultivated relationships, the City and the stakeholders were able to work together to develop and complete modifications to both of these outfalls that extended the useful life of the outfall for the stakeholder and significantly reduced risk to the levee system for the levee sponsor.

VI. REFERENCES

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VII. AUTHOR BIOGRAPHIES

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Vicki is a registered professional engineer and an experienced Project Manager with over 23 years of experience with a depth of experience in USACE levee policy, levee certification and flood control issues. Vicki has extensive experience in levee design, levee analysis and feasibility review, levee policy matters, and assisting levee sponsors with long-term levee O&M matters. In addition to being a licensed Professional Engineer, Vicki is also a Certified Floodplain Manager. This certification reflects an understanding of the National Flood Insurance Program and the various tools available. Vicki has performed evaluations, completed various levee policy analyses, provided flood fighting expertise, and performed design work on several levees across the region. Vicki currently serves on the Levees Committee for USSD and is a member of ASDSO.

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George is a registered professional engineer and a currently the Levee/ Lift Station program manager for the City of Omaha Public Works Department. George spent the first 10 of his 13 years in the industry as a project engineer, then a project manager for a private consultant specializing on municipal water and wastewater infrastructure studies and designs for linear and process projects. In 2018, George transitioned to the City of Omaha Public Works Department working in multiple capacities and has served as the levee sponsor since 2019. George works closely with the O&M superintendent to enhance the levee's condition and splits his remaining time on the Department's network of lift stations and dams.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

Risk Informed Assessment to Support a Section 408 Authorization

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ABSTRACT

This paper will summarize why this process was chosen and the process followed by the team, present findings and identified design refinements, discusses the associated potential failure modes with using existing risk assessment results, and share lessons learned from incorporating risk-informed processes into levee design and the Section 408 Permit application.

I.INTRODUCTION

The Louisville Metro Levee System reduces flood risk to the City of Louisville, Kentucky. The levee system was constructed by the U.S. Army Corps of Engineers (USACE) and is now operated and maintained by the local sponsor, the Louisville and Jefferson County Metropolitan Sewer District (MSD). The levee system has a total length of 25.9 miles consisting of concrete floodwalls and an earthen levee with sixteen flood pump stations for flood risk reduction.

MSD determined the need to upgrade the existing Paddy's Run Flood Pump Station's capacity to reduce the risk of interior flooding, due to past flooding issues caused by ponding at the pump station and backing up into upstream metropolitan roadways and structures. The age and complications associated with upgrading the existing pump station, resulted in the decision to construct a new flood pump station within the existing levee prism, while portions of the existing station will be selectively demolished. In 2022, MSD initiated a Progressive Design-Build project for the new Paddy's Run Flood Pump Station and contracted with the Progressive Design-Build team led by Ulliman Schutte Construction, LLC (US) and HDR Engineering Inc. (HDR) for the Paddy's Run Flood Pump Station Capacity Upgrade (PRFPSCU) project. The scope of the PRFPSCU project is for US and HDR to design and construct a new flood pump station with an upgraded pumping capacity of 1,900 MGD to replace the outdated existing 875 MGD flood pump station. The new flood pump station is proposed to be located at Station 571+90 along the Louisville Metro System, Segment E (NLD ID 3905000028), Louisville Kentucky shown in Figure 1. The project requires a Section 408 authorization per US Code: Title 33 from the U.S. Army Corps of Engineers.

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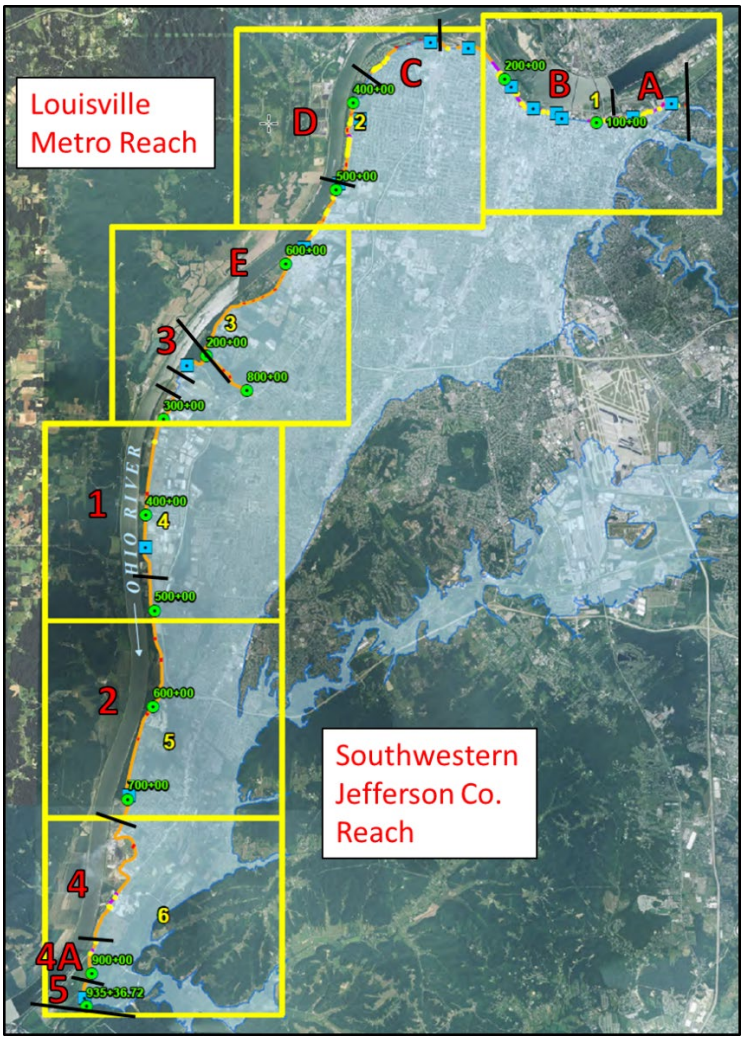


Figure 1. Louisville Metro Levee System Map

In 2019-2020 USACE had completed a Semi-Quantitative Risk Assessment (SQRA) for the Louisville Metro Levee System to assess life safety risks associated with the levee in its current conditions (USACE, RMC, 2020). The SQRA was completed in general accordance with Draft EC 1165-2-218 “Levee Safety Program – Policy and Procedure” dated March 2019. Based on the results of the SQRA, USACE has classified the levee system risk as ‘High’, meaning the likelihood of inundation due to breach and/or system malfunction in combination with loss of life, economic, or environmental consequences results in high risk. The levee risk was judged to be primarily driven by the risk of breach due to overtopping of the levee embankment, however other potential failure modes (PFMs) such as floodwall instability, pump station failure, and internal erosion contributed to the total risk of the Levee System.

II.PURPOSE (THE WHY)

During the early stages of the project as the 408 Alteration request and Review Plan was being coordinated between USACE and MSD, USACE reviewed the proposed project with members of the 2020 SQRA. The location of the Paddy’s Run Flood Pump Station was a contender for the risk driving location for multiple PFMs during the SQRA. With potential changes to the likelihood of these PFMs due to construction of the new flood pump station and changes in operations, it was requested by USACE the project to undergo a risk informed approach through a Potential Failure Modes Analysis (PFMA). This was considered to help ensure the project does not increase the risk of the levee system identified in the 2020 SQRA. With the relatively recent 2020 SQRA in hand, key ingredients to evaluate the risk of the project were already in place, such as the hydrologic loading of the levee and estimates

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of the consequences from a nearby modeled breach location. These were considered sufficient for the team to qualitatively evaluate a list of brainstormed PFMs to identify if any would be considered risk drivers. This PFMA was completed in conjunction with the Section 408 process to evaluate PFMs associated with the new flood pump station (during construction and post-construction).

The scope of this PFMA was for the project delivery team to familiarize itself with the 2020 Louisville Metro SQRA results and through a facilitated process, identify PFMs for the section of the levee system associated with the new flood pump station USACE, RMC, 2020). The PFMs would then be screened to identify risk drivers and develop a list of recommendations. PFMs of the levee are defined in the USACE Engineering Circular (EC) 1165-2-218 Levee Safety Program as a structured way to describe a chain of events that leads to a levee failure.

III.BACKGROUND

The proposed PRFPSCU project is in the western portion of Jefferson County, Kentucky, between Station 578+15.27 and Station 572+20 of the existing levee reach per as-built drawing 613-12.2/7 (USACE, 1953). The existing flood pump station and levee reach is owned, operated, and maintained by MSD. The existing levee and Paddy's Run Flood Pump Station were constructed in the 1950s by the USACE to reduce the risk of flooding to Louisville's western metropolitan area while the levee serves as flood risk reduction from the Ohio River. An outfall at this reach of levee known as Southwestern Outfall conveys water from the western metropolitan area under the levee to Gatewell Structure No. 6 and discharges by gravity into the Ohio River. Additionally, the USACE installed the Paddy's Run Flood Pump Station to pump the combined sewer water from the outfall, collected in the station's wet well and ponding area, over the levee to the Ohio River during flood events. The flood pump station is active when flood events cause elevated Ohio River levels to prevent gravity water run-off through Gatewell Structure No. 6.

For the Paddy's Run Flood Pump Station, MSD projected a 10-year, 24-hour storm event within the 20-year planning horizon (2035; 5.0 inches in 24 hours) as the design storm for the proposed pump station. For this storm, the proposed pump station's capacity must be increased to 1,900 million gallons per day (MGD). Based on this required increase in capacity and the condition of existing facilities, MSD has determined that a new flood pump station is needed.

The 2020 SQRA examined this area as the potential risk driving location for multiple failure modes. This was primarily due to the lower landside levee toe elevation present due to the ponding area of the pump station. The most significant of these was the Internal Erosion – Backward Erosion Piping PFM. While not the elicited location, the Paddy's Run ponding area was considered the next most likely risk driving location for this PFM. From the 2020 SQRA:

> *Paddy's Run original channel is located at Station 574+00 in the Louisville Metro Reach, Section E. The levee embankment crest elevation is 460, bottom of the stream channel is at EL 400, and the landside toe is at EL 415. Borings (B-3, L-1306, B-2) near the channel show there is a poorly graded sand with gravel increasing with depth. Overlying the sand is a continuous clay layer at the ground surface ranging from 30 feet riverside of the levee to 15 feet landside of the levee which extends below the bottom elevation of the channel. This location is pumped down much further than some of the other locations with pump stations downstream, since the outfall pipe here is associated with the combined sewer system. This creates a significant amount of head differential at this tall levee section. Following the SQRA, the team learned that a scour hole (10 feet deep, 40 foot diameter) from the pump discharge had formed riverside of the levee, and could contribute to a higher horizontal gradient than previously considered. The bottom of the scour hole is approximately at EL 415, 200 feet riverward of the levee axis. The thick clay layer though at the landside surface prevents seepage from readily exiting the sand layer near the levee, and considering transient conditions, is not expected to experience uplift. Relief wells in the area have not been maintained but are still expected to provide some pore pressure dissipation based on the last inspection. This location is likely to have a gradient (for horizontal progression) as high or higher than Upper Mill but is less likely to have an unfiltered exit and also less likely to have a continuous fine sand. It is considered to potentially be within the same order of magnitude for likelihood of failure to Upper Mill if the riverside scour continues and removes the clay cover and presents an entry condition. A sketch of the failure path is shown in the figure below.*

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Figure 2. Sketch of Backward Erosion Piping Potential Failure Mode Pathway

A preliminary review of the planned Flood Pump Station project identified the following items as part of the 2020 SQRA, which could affect the risk of the levee system and therefore recommended to the 408 Alteration Project Delivery Team (PDT);

1. Replacing a pump station with different pumps and capacities has the potential to alter the risk of interior flooding during a coincident rain event with a high river stage. Replacing older pumps with newer pumps is anticipated to result in a decrease in risk to interior flooding, however consideration should be given to the potential for the new equipment to not function properly, particularly at the first use.
2. Embankment penetrations pose risk to levee systems due to the potential for internal erosion failures along the penetration or in the disturbed area of excavations. The PDT will consider these potential failure modes and ensure the latest guidance and best practices in design and construction of penetrations such as pump station discharge pipes are followed.
3. During the 2020 SQRA the existing Paddy's Run ponding area was considered for a risk driving location of internal erosion through the foundation. This area has a high gradient due to the height of the embankment. The area was eventually excluded as a risk driver, with one factor being the amount of ponding depth that would lower gradients across the foundation. These ponding depths are dependent on when the gravity gates are closed, as well as the pump station on/off elevations that control the ponding depths in this area. The PDT will examine the design ponding depths to ensure they have not changed from the 2020 SQRA. In the event they have changed, the PDT will need to re-examine the resulting gradients and the current risk of internal erosion through the foundation.
4. Pump station discharge flow as well as riverine erosion could scour riverside soil and potentially result in exposing erodible sands in the foundation closer to the embankment, decreasing the seepage path and increasing gradients. This could lead to an increased risk of internal erosion through the foundation. Consideration for the potential of and design features to address riverside scour will be considered as part of the project.

IV.NEW ALTERATION

The scope of work for the construction of the new flood pump station with a capacity of 1,900 MGD located at the landside levee toe includes several features that are within the critical area of the levee and are included in the

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408 Alteration. These project features include the construction of permanent concrete diaphragm walls for foundation design, levee seepage cutoff, and support of excavation. The new flood pump station will also be equipped with new discharge pipes up and over the levee, and seepage collection for pore pressure mitigation on the land side levee toe. Project features are shown below in Figure 3.

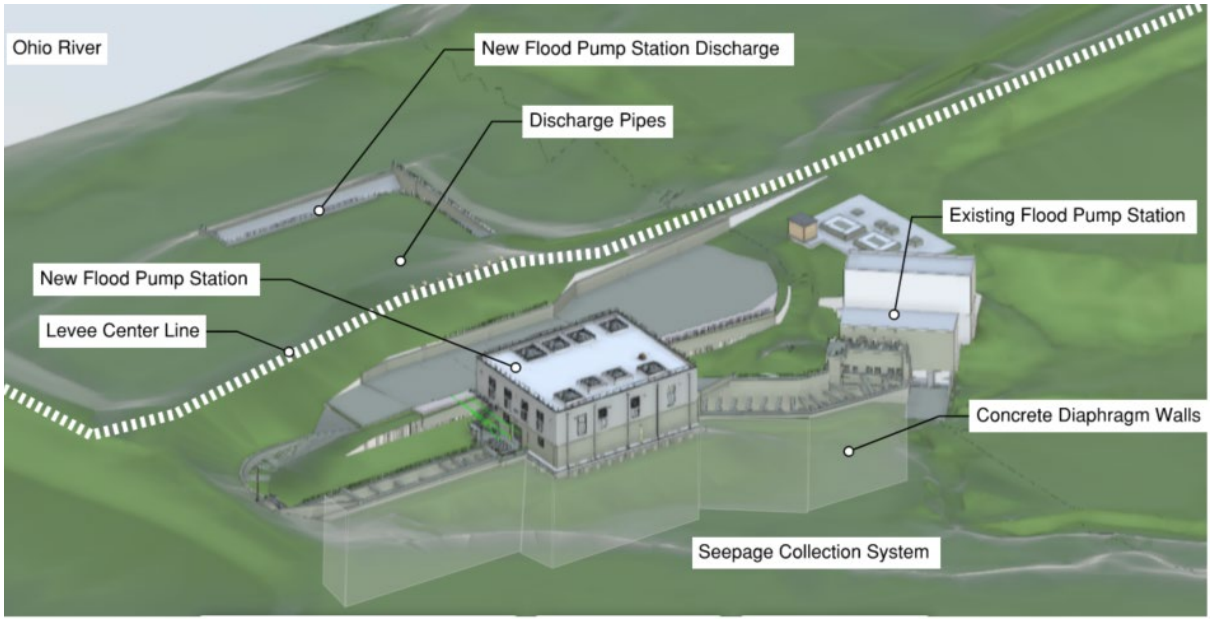


Figure 3. New Flood Pump Station Project Features

V.PFMA WORKSHOP (THE HOW)

The PFMA workshop was held on December 14 and 15, 2022 in the HDR office in Louisville, KY. The workshop was attended by representatives from US, HDR, MSD, Heritage Engineering, LLC (Heritage) and USACE. The PFMA team included a facilitator (HDR), co-facilitator (HDR), and technical subject matter experts. Each of the team members was provided with the project background information prior to the PFMA workshop, ensuring all members had familiarity with the project at the time of the workshop. PFMs were assigned into failure groups occurring during construction and in the permanent condition after completion of construction. In specific, the PFMs considered levee breach prior to overtopping or overtopping with breach, as well as pump station mis-operation or malfunction.

VI.FINDINGS (WHAT WE LEARNED)

The following major findings and understandings about the project were identified as a result from the review of background information and PFMA workshop discussions:

a) Based on information gathered for the PRFPSCU project, the levee risk at the Paddy's Run location under existing conditions was judged to be similar or potentially higher than estimated by the 2020 SQRA. Global stability as well as seepage conditions at this location are potentially more adverse than the considered risk-driving

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locations.. However, with the new flood pump station in place the conditions will be improved, and the levee risk is expected to be lower compared to existing conditions.

b) The PRFPSCU project increases pump station capacity and significantly improves its reliability, reducing the risk of not being able to operate the pump station.

c) The design of the new pump station is generally robust with several defensive measures. The PFMA team identified 24 PFMs under permanent conditions and 8 PFMs that could occur during construction. All but two PFMs were screened out/excluded as risk drivers.

d) Design enhancements were recommended to address Backward Erosion Piping of the Foundation Material under the Levee Embankment.

e) Additional structural analyses were recommended to evaluate Internal Erosion into an Open Joint of the Existing Southwestern Outfall caused by Movement of Existing Flood Pump Station due to lack of Lateral Support.

f) While most PFMs were screened out, the team brainstormed potential design enhancements that could be incorporated to lower the risk yet further.

g) Levee risks during construction can be adequately managed by construction control provisions. No changes to design were considered warranted.

The first PFM identified as a potential risk driver was Backward Erosion Piping (BEP) of the Foundation Material. The process of BEP is identified in Figure 4. BEP is particularly a concern when there are continuous uniform fine sands present in the foundation and sufficient flow/gradients present to initiate and progress the erosion.

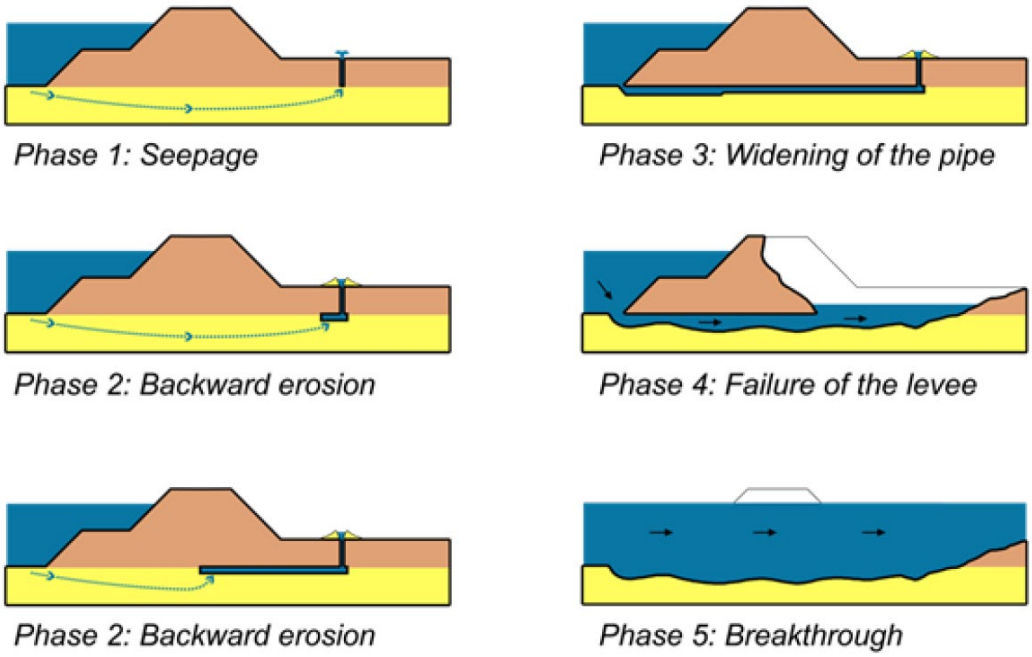


Figure 4. BEP progressing to Breach (adapted from van Beek et al. 2011)

Three primary locations were identified as critical locations for this PFM. All three locations are connected to the aquifer sands and are subjected to potential seepage. The proposed flood pump station structure consists of diaphragm walls to bedrock. The diaphragm walls would function as a cut-off wall for seepage within the aquifer from the riverside to the landside. Refer to Figure 4 below for a plan view of the project site with critical locations A, B, and C marked in red. The dashed line outlines the perimeter support of excavation (SOE) and the location of diaphragm walls. Seepage originating from the Ohio River within the aquifer sands is forced around the proposed structure which creates an end-around seepage effect that increase the hydraulic gradients around the northern and southern corner locations. It was noted that the existing riverside pump discharge area has eroded the existing clay blanket, creating a scour area that is likely exposing the aquifer sands. This location shown on Figure 5 below creates a shorter seepage to locations A & B.

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The previous 2020 SQRA considered multiple locations along the metropolitan area levee reach for this PFM and identified Upper Mill (3 miles downstream) and Paddy's Run as the two primary risk driving locations. The risk was estimated for Upper Mill, but Paddy's Run was judged to be equivalent in risk. Additional explorations since the 2020 SQRA have identified the sands to be more continuous and uniform in gradation, making them more susceptible to BEP than initially estimated. Instrumentation and seepage analyses suggests a strong potential for elevated pore pressures in the aquifer sands and high exit gradients through the clay blanket with increased Ohio River water elevations. Future operations of the new pump station will draw the ponding area water level down further than current operations, increasing the gradient through the foundation. Additionally, the noted scour was not considered in the previous risk assessment as it was identified at a later date.

With the construction of diaphragm walls for the SOE functioning as cut-off walls to bedrock, the seepage at the location of the new flood pump station would be redirected and concentrated around the northern and southern corners of the new flood pump station. The most vulnerable exit point was judged to be at the north side of the new flood pump station, where multiple exit points were identified. Refer to Location A in Figure 5 below.

With consistent uniform fine sands in the foundation and an entry/exit location with a sufficient global/horizontal gradient, the risk at this location was qualitatively judged to be higher than the PFM estimated in the 2020 SQRA at the Upper Mill location. Therefore, it was recommended future design modifications to address this PFM along the postulated path from the scour to the existing Paddy's Run Creek (Location A) be considered.

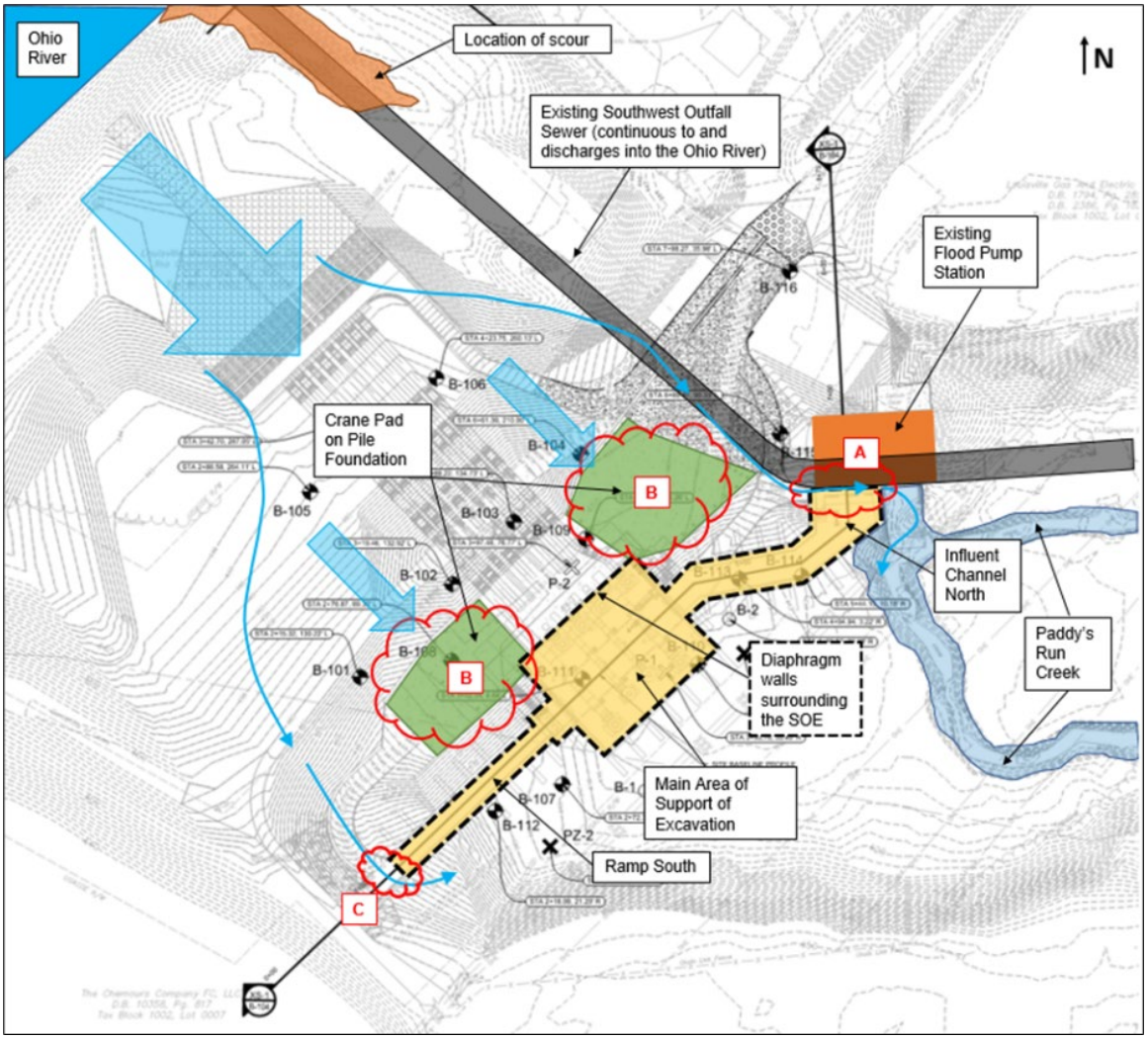


Figure 5. Sketch of Backward Erosion Piping Potential Failure Mode for the Alteration.

The second PFM the team considered a potential risk driver involves internal erosion into an open joint or defect in the existing Southwestern Outfall pipe, leading to loss of material into the pipe, and an eventual collapse of the levee crest. The earthen support for the existing pump station will be removed on the south side of the existing

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pump station structure during construction of the proposed influent channel for the new flood pump station. When removing the earthen support, lateral support on the landside of the station is reduced. During construction and in final condition, additional material is placed north of the existing flood pump station, increasing the load above the excavation area. Under these loading conditions, it was envisioned the existing flood pump station rotates or slides and causing damage to the connection between Southwestern Outfall and existing flood pump station. Potential damage around the Southwestern Outfall, such as cracks or open joints, causes seepage in or out of the sewer resulting in unraveling of surrounding soil. The unraveling continues resulting in an opening of a sinkhole of the embankment followed by breach and failure of the levee. Figure 6 below shows the potential areas that are subjected to shear stresses circled in red.

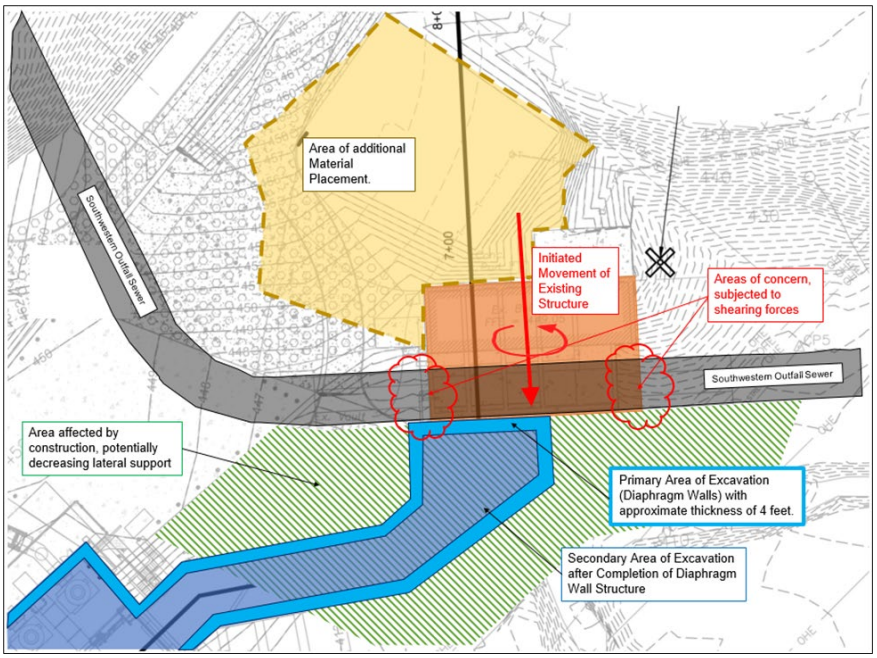


Figure 6. Sketch of Internal Erosion into an Open Joint of the Existing Southwestern Outfall caused by Movement of Existing Flood Pump Station due to lack of Lateral Support.

This PFM carried significant uncertainty during the PFMA session. This particular loading scenario had not been analyzed and the condition of the Southwestern Outfall pipe was not considered well understood.

VII.SUMMARY OF ANY DESIGN CHANGES REALIZED

As previously discussed, the PFMA workshop identified two PFMs where further design refinement was warranted for the PRFPSCU project. Both PFMs identified the connection between existing Southwestern Outfall structure and the new diaphragm walls is a critical location that the design team needs to consider further refinement as the design progress further. Particularly, the design will need to provide additional defensive measure against potential seepage exits around the diaphragm wall and the connection between the existing and new structures. Additional analyses would be needed by the design team to confirm stability of existing pump station and ability of sewer pipe to resist shear stresses. Lastly, the design team and diaphragm wall contractor should consider different approaches to establish a successful connection between the interface of the diaphragm wall and existing structure wall. The following paragraph describes the current design that has been developed between the design team and diaphragm wall contractor.

Figure 7 shows the area between the new and existing structure where jet grouting will be placed to cut-off potential seepage paths. Figure 8 shows the lateral supports within the structure to support the load from the existing pump station and a concrete slab within the creek to cutoff seepage. Not shown in Figure 8 is a series of relief wells around the corner to lower the excess pore pressure at this location as an additional measure against potential seepage.

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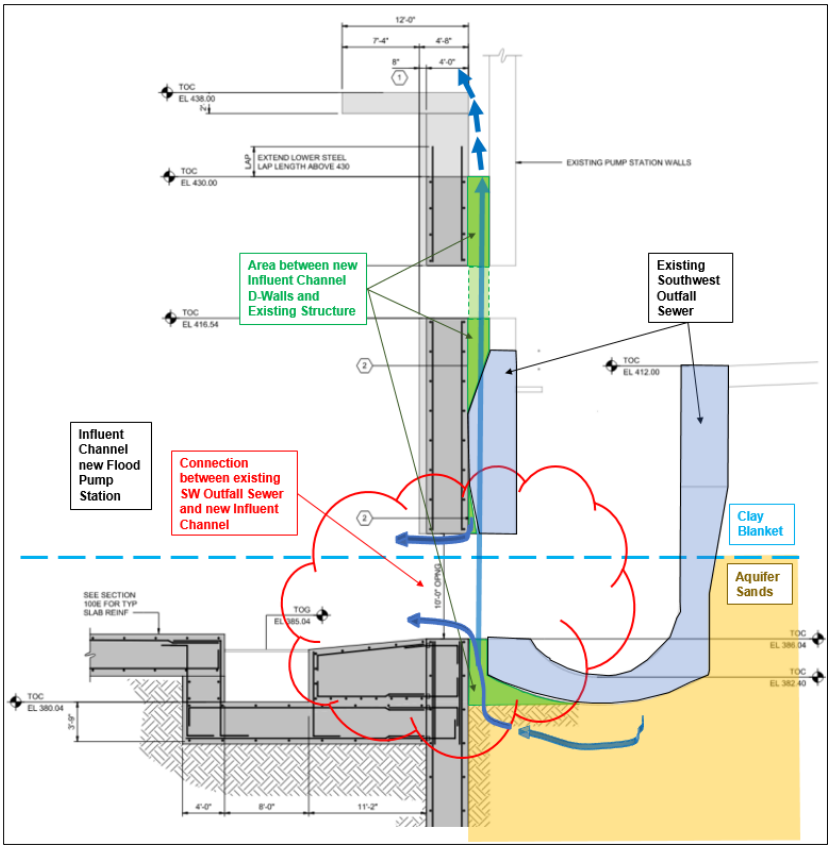


Figure 7. Sketch of Internal Erosion into the connection of the Existing Southwestern Outfall.

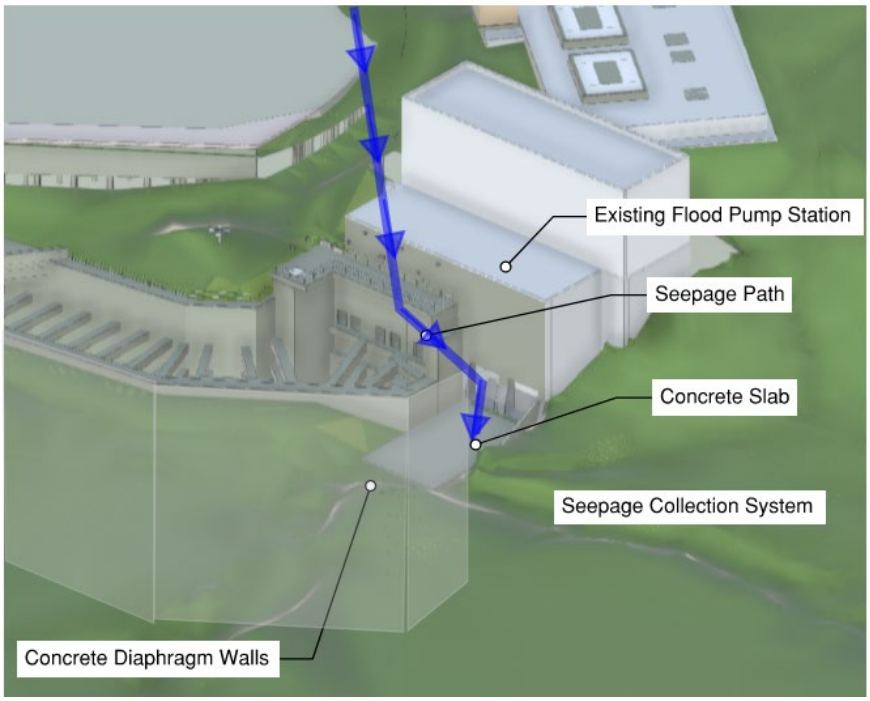


Figure 8. Design Refinements approach for BEP.

With the design refinements still on-going at the time of this paper, the PDT anticipates a secondary virtual session to discuss how the refinements would affect the likelihood of the PFMs identified. With the design

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refinements considered likely to lower the risk of the PFMs, the final project is expected not to increase (and likely lower) the overall total risk of the levee system.

VIII.ADVANTAGES AND DISADVANTAGES

The industry continues to expand towards the direction of evaluating and understanding risk related to dams and levees. This direction includes the need to understand the impacts to the risk related to modifications to a levee system or dam. In this particular case, USACE requested the PFMA as part of the 408 Alternation due to the impacts caused by this modification. This PFMA provided additional value to the design team and feedback to USACE and the owner MSD that the design was robust. However, there is opportunity to further refine the design to address the risk for the permanent design and construction process. In summary, collaboration between USACE, MSD, HDR, and US was important to the success of this PFMA. The following points were discussed as advantages and disadvantages with the process that future projects conducting a PFMA or other risk informed process for a Section 408 Authorization should consider:

a) The benefit of increased confidence in design and future performance of the levee for the Owner and USACE.

b) The negative impact of project cost and schedule if the project was not anticipating on conducting a PFMA.

c) The design team may have to address confidence concerns with their client if a major design modification is identified during the PFMA workshop. Additionally, the design team and clients will need to manage the impacts to schedule and cost depending on the impacts of the design modification.

d) The workshop attendees may be new to the PFMA process and not see the value in the process. Therefore, it is important to have a strong and experienced facilitator to conduct the workshop.

e) The Section 408 Review Plan provided by USACE noted this requirement that was not anticipated by the project team for the Section 408 Authorization. As you plan your project, USACE does not have a requirement that all Section 408 applications include a PFMA. This will be done on a case by case basis. This project was within a levee system that has an existing SQRA and was invasive to the levee system, which triggered the PFMA requirement. Future projects should coordinate with local USACE district to determine if their project will have such a requirement.

f) The existing SQRA was a significant benefit to the PFMA process including representation from individuals from the sponsor and USACE that participated in the prior SQRA. The existing SQRA contained primary information related to the hydrologic loading and consequences 4that were able to be utilized in the PFMA screening process.

g) The advantage of an experienced facilitator was realized. Although USACE did not have specific requirements on this project for the facilitator or subject matter experts, it is important to recognize having experienced individuals for those roles.

h) The benefit of focusing the design team on more significant features of the projects, as well as problems.

IX.ACKNOWLEDGMENT

The authors gratefully acknowledge the contributions of Eric Meister, PE, Vice President, Ulliman Schutte Construction, LLC; Brandon Flaherty, PE, Senior Engineer, Louisville MSD for their work on this document.

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Working in the geotechnical section of USACE Louisville District since 2007, currently serves as the Risk Cadre Lead for Louisville District. Matthew has been leading multiple USACE risk assessment projects concurrently since 2019 and working with the Risk Cadre since 2017. Technical competencies include; risk calculation software, risk facilitation, event tree development, slope stability and seepage analysis, bearing capacity calculations, settlement analysis, deep foundation design and modeling, levee design and repair, subsurface exploration and sampling, pile modeling and design, foundation design, materials and soil laboratory testing, and evaluation of measures and alternatives for feasibility studies. Matthew has a Bachelors and Masters of Science degree in Civil Engineering from the University of Kentucky.

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Matthias is a geotechnical engineer with experience working on various dams and levee projects, performing seepage, slope stability, shallow and deep foundation, and retaining wall analyses. Matthias has experience preparing geotechnical reports and technical memorandums for dams, levees, and civil works structures. He is familiar with the U.S. Army Corps of Engineer's Section 408 program and has taken part in potential failure mode (PFM) workshops. Matthias has construction infrastructure experience for a variety of structures in various geological regions in Germany. Matthias has a bachelor's degree in civil engineering from the University of Louisville and a master's degree in civil engineering from the Technical Institute of Berlin, Germany.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

Stillwater Creek Watershed Dam Monitoring and Inspection Pilot Program

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ABSTRACT

Scientists at the USDA Agricultural Research Service (ARS) Agroclimate and Hydraulic Engineering Research Unit (HERU) in Stillwater, Oklahoma in collaboration with ARS Partnership for Data Innovations and the USDA Natural Resources Conservation Service (NRCS) have initiated a pilot dam monitoring and inspection program in the Stillwater Creek Watershed. This project will focus on utilizing cloud-based technologies that consumes data from sensor networks, specialized instrumentation, and unmanned aerial systems (UAS) to name a few. These technologies are deployed across a range of dam and reservoir sizes, dam construction years, and land use (e.g., urban to rural). Data will be transmitted from these real-time, continuous monitoring systems to the USDA ARS cloud-based data repository. Additionally, real-time data will be coupled with historic observation data for the selected sites made available from federal and state agencies, academic institutions, and other collaborators. The expected outcomes of this pilot project are for the data to be used 1) to develop new or improve existing cloud-based models for forecasting water storage; runoff; reservoir sedimentation; and dam, spillway, and channel performance; 2) to achieve a better understanding of dam performance and reservoir response time to climate changes and extreme weather events; 3) to predict impacts to urban and agricultural landscapes in a more timely manner; 4) to improve flood warning systems; and 5) to create a suite of decision support tools, models, standard operating procedures, and design guidance for a variety of end users including dam operators, emergency managers, planning and design engineers, irrigation district managers, scientists, farmers, producers, agri-businesses, and water resources policy makers. These outcomes will allow end-users to make informed decisions on dam safety, water resources operation, infrastructure maintenance and rehabilitation needs; irrigation management; water allocations; zoning regulations; and lending for commercial or residential development.

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DAM SAFETY PALM SPRINGS
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An Evaluation of Uncertainty Associated with Generalized PMP Estimation Methods

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EXTENDED ABSTRACT

Three generalized methods were tested to obtain the Probable Maximum Flood (PMF) for the Island Lake Dams. The results obtained with these methods were compared, to select which one to use in defining the dam's Inflow Design Flood (IDF), and, correspondingly, to determine if the spill capacity of the dam was adequate. The study showed that the selection of the most appropriate generalized method is greatly influenced by the size of the watershed being studied.

The Island Lake watershed, with a total basin area of approximately 26.5 km^2, is located in Orangeville, Ontario. The reservoir at the Lake was created with the construction of the Island Lake Dams. This reservoir allows for recreational activities, improves aesthetics, maintains fish and wildlife habitat, and provides attenuation to spring floods.

The Island Lake Dams were assigned the maximum of the dam safety categories in Ontario: Hazard Potential Classification (HPC) of Very High, due to the potential for incremental loss of life associated with its failure. When a dam is classified as Very High, the IDF is typically in the range of extreme floods. For the Island Lake Dams, the IDF was determined to be the PMF.

The PMF is typically estimated using the Probable Maximum Precipitation (PMP), which is defined by the World Meteorological Organization (WMO) as "the greatest depth of precipitation for a given duration meteorologically possible for a design watershed or a given storm area at a particular time of year" [1]. The PMP hyetograph is subsequently input to a hydrologic model to obtain the PMF. A commonly employed approach for estimating the PMP is to utilize generalized methods that provide PMP values across wide regions. However, applying these generalized methods, as opposed to conducting a site-specific study, introduces uncertainties, which can have significant implications when assessing the adequacy of a particular dam to pass its required IDF.

The three generalized methods of PMP used in this study were the Bruce Graphical Method (1957) [2], the findings of the study entitled "PMP for Ontario" (IBI, 2006) [3], and the National Weather Service Hydrometeorological Report (HMR) 51 [4].

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The Bruce Method was developed in 1957 and it was based on storm maximization procedures. The study resulted in a graph from which point rainfall amounts can be selected. It includes thunderstorms as well as tropical storms, with durations ranging from 6 to 48 hours and for areas in the range of 25 km^2 to 12,900 km^2. PMP estimates with this method were widely used in Ontario. The IBI study was completed in 2006. It was commissioned by the Ontario Ministry of Natural Resources with the purpose of updating the Bruce Method. This method provides estimates of PMP rainfall values across Ontario, that were derived applying procedures recommended by the WMO, on rainfall events that had occurred in the period from May to October. The report includes storm durations ranging from 6 to 72 hours and for areas ranging from 25 km^2 to 50,000 km^2. HMR 51 is a generalized PMP study for areas in the eastern USA, that applied storm maximization and basic data published by the US Army Corps of Engineers and the Canada Atmospheric Environment Service. The results were graphically provided in maps for storm durations ranging from 6 to 72 hours and for areas ranging from 26 km^2 to 51,800 km^2. The HMR 51 study determines all-season PMPs. The point PMP values from these methods corresponds to an area of approximately 25 km^2 (10 mi^2), which is close to the area of the Island Lake watershed. Table 1 provides a summary of the three methods.

TABLE I

Generalized PMP Estimation Methods Used in this Study

Method	Developed by	Storm Durations	Storm Areas	Region	PMP
Bruce Method	[2]	6 to 48 hours for Thunder Storms and Tropical Storms	25 to 12,900 km^2	Southern Ontario	Summer/Fall
PMP for Ontario	[3]	6 to 72 hours	25 to 50,000 km^2	Ontario	Summer/Fall
HMR 51	[4]	6 to 72 hours	26 to 51,800 km^2	Eastern USA	All-season

The total 24-hour PMP depths for the Island Lake watershed, obtained with these methods were:

- Bruce Method: 431.5 millimetres (16.98 Inches)
- IBI (2006) Report 600 millimetres (23.6 Inches)
- HMR 51: 724 millimetres (28.5 Inches)

Given the wide range of these results it was important to investigate which was the most appropriate for the study area, with the limited information available.

To provide context to these results, reports from site-specific studies completed for other sites in Ontario, Canada, were reviewed. The three generalized methods were applied to those areas for which site-specific hydrometeorological analyses were available, and the results were compared. The correspondence of the results obtained with the generalized methods and the site-specific studies varied with the size of the watershed as follows:

- The values obtained from using IBI (2006) Report were the closest to the hydrometeorological analyses for small watersheds (areas in the range of 25 to 500 km^2). The HMR 51 approach generally resulted in the highest values of PMP for small watersheds.
- The Bruce Method resulted in the lowest PMP depth values for all watersheds.

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- The IBI (2006) Report and the HMR 51 method provided similar results for large watersheds, which were also in the same range as those obtained with the more rigorous site-specific hydrometeorological analyses.

To apply the PMP depths for estimation of the PMF, spatial and temporal distributions were required to define a hyetograph that could be input to a hydrologic model. The spatial and temporal distribution applied to the values obtained from IBI (2006) and HMR 51 was determined using the method described in the US National Oceanic and Atmospheric Administration (NOAA) HMR 52 [5]. Temporal distribution for the PMP estimated from the application of the Bruce method was obtained from the Ontario Technical Guide for Flooding Hazar Limit [6]. The PMF was assessed by simulating the runoff from the PMPs with a hydrologic model prepared with the software Visual OTTHYMO, adjusted to represent saturated antecedent soil conditions, in line with the guidelines for PMF estimation from most agencies and authorities in Canada. The peak PMF flow values corresponding to the three PMP estimation methods are provided in Table 2.

TABLE 2

Results for the PMP and PMF Values Based on Different PMP Estimation Methods (Watershed Area: 26.5 km^2)

Method	PMP Depth (10^{th} of meter)	PMF Peak (m^3/s)
Bruce Method	4.31	560
PMP for Ontario	6.00	652
HMR 51	7.24	921

The wide range of results shown in Table 2 underscores the uncertainty associated with the application of these generalized PMP estimates. The comparison of results with those obtained in site-specific studies suggests that the selection of generalized method is influenced by the size of the watershed.

The PMF estimates were compared with some other estimates for other sites by plotting them on the Creager Diagram, which displays the relationship between unit discharge and total drainage area of the basin. This Diagram is commonly used to provide a graphical comparison of PMF estimates and large floods for different areas on a common basis. The magnitude of the Creager Constant “C” can be used as a measure of the flood-producing characteristics of a region. The results obtained with the three generalized methods corresponded to relatively high C values, which is not unexpected given the relatively small size of the watershed.

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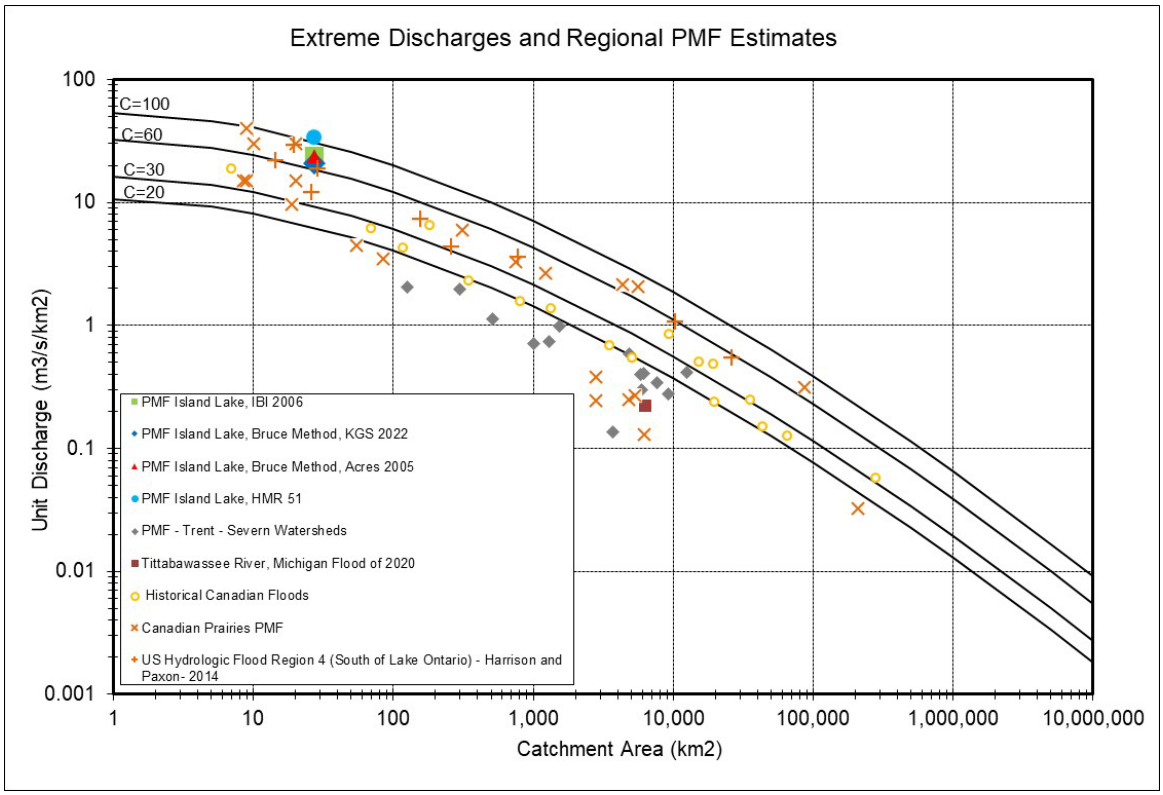


Figure 1. Island Lake Dams Creager Diagram

To estimate the required spill capacity at the Island Lake Dams, the PMF inflow hydrographs obtained with the hydrologic model were routed through the Island Lake Reservoir. A level pool routing analysis was carried out using the reservoir stage-storage relationship and the spillway stage-discharge rating curve. The analysis was carried out with the PMF obtained with both the IBI (2006) and the Bruce Method and indicated that the discharge capacity of the dams was not adequate to pass the IDF obtained with either method.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

Hydrologic Insights from 14,000 Square Miles: Unique Dynamics of the Coosa and Tallapoosa Site-Specific PMP & PMF Study

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ABSTRACT

The last comprehensive hydrometeorological studies of the Coosa and Tallapoosa River Basins were performed by Alabama Power Company (APC) over 30 years ago. In partnership with Applied Weather Associates, the FNI team developed a site-specific PMP/PMF update to modernize all aspects of the hydrometeorological and hydrologic assessments for the eleven hydropower dams in these basins. This state-of-the-practice analysis makes use of the latest hydrologic and hydraulic modeling techniques, advanced meteorologic analyses, and improved stream and reservoir monitoring data. Comprehensive calibrated hydrologic models covered the 14,000 square mile study area, and hydraulic routing was performed using a fully two-dimensional modeling approach in HEC-RAS, which more accurately captures the influence of river bends, floodplain storage, and backwater tributaries. The overall 2D model geometry was sub-divided at each dam to optimize model runtimes and consisted of over one million cells representing 2,000 square miles of riverine, reservoir, and floodplain routing environments. Gate operations were modeled within HEC-RAS utilizing detailed operational scripts, which were then validated with HEC-ResSim.One unique finding related to full-2D reservoir routing was reservoir stage sensitivity to bathymetry. Testing indicated that lack of bathymetry in the terrain led to lower water depths and higher velocities, increasing the effect of roughness and causing increased lag and attenuation. These influences were mitigated by incorporating available bathymetric surveys and estimating a bathymetric surface for reservoir areas without available data.Rigorous storm maximization helped determine the critical scenario for each dam by evaluating the entire river basin as a system, applying alternate spatial and temporal patterns, and optimizing storm area to produce the greatest runoff to each reservoir. In total, more than 400 model scenarios were analyzed to identify the controlling PMP/PMF scenario. Managing the suite of hydrologic and hydraulic models and associated data outputs for all these scenarios relied on manipulating HEC-RAS with a robust HECRASController script, which allowed the project team to batch compute large, interdependent models and evaluate multiple scenarios seamlessly and efficiently.Overall, the study identified both increases and decreases in PMF reservoir elevations relative to the previous studies. For example, notable differences in flood response were observed between the wide, dendritic reservoirs in the Upper Coosa and the more linear, flashier watersheds of the Lower Coosa. This meant that Lower Coosa dams were more sensitive to steeper temporal distributions and more intense spatial patterns and thus saw relative increases where Upper Coosa dams saw decreases.

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DAM SAFETY PALM SPRINGS
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Savage River Dam, A Wild Ride: Developing 86 Miles of Inundation Mapping and Emergency Action Planning for 10 Municipalities/Counties Downstream of Savage River Dam.

JEFFERY G. POWERS, KEVIN CHAPMAN, AND KATE NAUGHTON, HAZEN AND SAWYER

ABSTRACT

Savage River Dam is one of the tallest rockfill earth dams in the US with a 180-foot hydraulic height and impounding an approximately 6.3-billion-gallon reservoir. The dam is owned by the Upper Potomac River Commission (UPRC) and operated by the UPRC and the US Army Corp of Engineers. The Savage River Reservoir is a major water source for a prized native trout stream and the Potomac River which serves three major utilities in the greater DC area. This presentation will focus on the recent breach failure inundation analysis and emergency preparedness activities including an Emergency Action Plan (EAP) update, interagency coordination, and tabletop exercise for Savage River Dam. Past modeling processes involved orographic Probable Maximum Precipitation (PMP) determination, HEC-1 hydrologic modeling for dam runoff determination and spillway peak flow calibration, and Probable Maximum Flood (PMF) breach analysis using DAMBREAK. New hydrologic and hydraulic models were developed in HEC-HMS and two dimensional (2D) HEC-RAS software, respectively. The hydrologic model was developed using Geographical Information Systems (GIS) watershed boundary and infiltration analysis as well as hydrologic model calibration. The 2D hydraulic model totaled approximately 250,000 cells and included 30 bridge crossing structures along 86 miles of stream, 55 miles on North Branch Potomac River and 27 miles along the Potomac River. Five breach methodologies were applied to determine final overtopping and sunny day failure breach sizes. The breach failure inundation area was estimated to impact 4,600 structures in at least 25 different municipalities along with 13 roadway and 17 railroad crossings. The EAP was updated to include 38 pages of the resulting inundation extent and to meet to current Maryland Department of the Environment (MDE) Dam Safety guidelines. Updating the EAP involved coordination between UPRC and MDE to define appropriate notification flowcharts for different emergency levels. Notifications for a Level 3 (Imminent Failure) involves two states, 5 counties and 10 downstream municipalities/counties. Guided by the Federal Emergency Management Agency (FEMA) as part of the Maryland/West Virginia Collaborative Technical Assistance Program, an EAP tabletop exercise was held for all responsible parties to demonstrate emergency response procedures following a simulated failure scenario.

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DAM SAFETY PALM SPRINGS
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Cementing the Future of Dam Safety

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ABSTRACT

Long lasting, sustainable structures require durable and resilient materials and for many dam applications concrete is the material of choice. Cement and concrete are central to the economic viability and resilience of dam and water resource projects. Concrete materials are selected for the long service life they provide, which can be readily demonstrated with a life cycle analysis (LCA). The initial carbon footprint of any material or construction method is an important aspect of an LCA, but may not have the greatest contribution: maintenance considerations and overall service life often ultimately outweigh the initial footprint. Portland cement (PC) has been the default binder material in concrete specifications for water resources projects for over a century. The properties and performance of both fresh and hardened PC concrete are well documented, as is the superior durability and longevity of concrete structures that are properly designed and constructed. The environmental impact of manufacturing PC is also well documented, as regulators impose strict requirements on all cement producers to monitor and report emissions. According to the U.S. Environmental Protections Agency (EPA), the CO2 footprint of cement production in the U.S. accounts for 1.25% of the total U.S. CO2 emissions. While the cement industry has made considerable progress since the 1970's to reduce its energy demand by 40%, the Portland Cement Association (PCA) and its members recognized that more must be done to tackle the carbon footprint of the cement and concrete industry. In 2021, PCA developed a Roadmap to Carbon Neutrality to lay out specific tasks and targets to achieve its goals.Dam safety professionals have many opportunities to participate and engage with the cement and concrete industry to achieve carbon neutrality including optimization of concrete design and materials selection and facilitating a shift towards performance-based specifications. For example, replacing PC with portland-limestone cement (PLC or Type IL) in concrete for water resource projects delivers the same long-term durability and resilience with the added benefit of significantly reducing the carbon footprint. Portland-limestone cement offers up to a 10% lower carbon footprint than PC. Extensive comparison testing has demonstrated similar properties and performance of PLC when compared to PC in concrete.

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DAM SAFETY PALM SPRINGS
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Hydraulics of beveled face stepped spillways

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ABSTRACT

Rehabilitation of embankment dams has included the usage of RCC to construct stepped spillways. Significant research has been completed on the complex flows and hydraulics of stepped chutes with vertical faces (usage of formworks). However, construction techniques common to RCC often result in steps with an inclined or beveled vertical face. Currently, there is limited scientific understanding on how this step geometry modifies the flow field relative to the conventional square edge, and how any changes would affect the overall hydraulic performance of the stepped spillways. Therefore, this study has been undertaken to better understand the hydraulics of stepped chutes with inclined faces. Research is being conducted in a field-scale sectional facility at Utah Water Research Laboratory with a stepped chute having a mild slope of 3:1 (H:V), width of 1m, and total drop height of 1.83 m. A broad crested weir at the top conveys water to the stepped spillway. Tests were conducted for a step height of 0.2 m with the vertical faces sloping at 45 degrees. Air water flow properties were obtained using a dual tip phase detection conductivity probe. A critical component of this research includes a baseline study of steps with vertical faces, for direct comparison. Results showed that the length from the crest to the point of inception was reduced by approximately (20-25) % with the use of beveled face steps. Beveled faces also increase the extent of splash and spray as well as the required sidewall heights, when compared to the conventional flat steps. Likewise, the mean air concentration was higher for beveled steps, meaning they induced more air entrainment into the flow. For similar distances from the weir crest, it was seen that the interfacial velocity of the multiphase mixture was lower for the beveled face steps and hence slightly lower residual energy. It is anticipated that the results from this study will be of high interest to hydraulic engineers in dam safety.

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DAM SAFETY PALM SPRINGS
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How Do We Get to This Gate?

PETE NIX, PE, SENIOR PROGRAM MANAGER – TETRA TECH

ABSTRACT

Rocky Fork Dam is an 80-foot high, high-hazard, concrete gravity dam in Highland County that is owned and operated by the Ohio Department of Natural Resources (ODNR). The dam was constructed in the early 1950's and has had no safety issues nor significant repair work since its construction. The reservoir is controlled by a single 5'x7' sluice gate that is operated by the original gate controls installed in the 1950's.

An inspection of the dam in 2021 indicated that there was some leakage around the sluice gate and that a number of areas of exposed concrete on the dam needed patched or repaired. ODNR implemented a rehabilitation project to repair the concrete and to replace the existing sluice gate. Tetra Tech was hired to prepare the final designs for this work.

The concrete repairs were straightforward but there were several challenges with the new gate design. Unfortunately, the only access to the gate was by entering the outlet of the sluiceway, then walking upstream to the gate. There was no access to the gate from the operating gallery. Obviously, only the downstream side of the gate could be inspected this way.

The original dam designers had specified a reinforced concrete frame around the sluiceway inlet. This frame held a trash rack but was also designed to hold a bulkhead when needed. Unfortunately, there was no housing nor an operator to remove the trash rack or install the bulkhead; divers would have to be used for all of the work. In addition, the condition of the original bulkhead was suspect, so a new steel bulkhead would have to be designed.

Consequently, divers were used to first remove the existing trash rack from the frame and then measure the sluiceway inlet for the new bulkhead design. The new steel bulkhead was then designed and fabricated. In early January, the new steel bulkhead was delivered to the site and installed on the sluiceway inlet. Tetra Tech's engineers and staff from the Construction Manager on the project were able to enter the sluiceway from the downstream end and complete their inspection of the upstream face of the gate and the upstream portion of the sluiceway. Based on the results of this inspection, a new 5'x7' gate is currently being designed for the project. Once the new gate is ready, divers will be used to install the bulkhead again so that the existing gate can be replaced.

I. INTRODUCTION

Rocky Fork Lake Dam is a Class I, high-hazard level dam located in Highland County, Ohio. The dam is owned and operated by the Ohio Department of Natural Resources (ODNR), Division of Wildlife. A regional location map is presented in **Figure 1.**



Figure 1. Rocky Fork Lake Dam Location Plan

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A. Purpose and Scope

The dam was constructed in 1952 for flood control. Improvements are being made to the project including replacing the original bonnet gate, upgrading the electrical components, and clearing the abutments to allow better monitoring of the abutment conditions. The improvements will be constructed using the Construction Manager at Risk (CMAR) delivery method. Beaver Excavating is the CMAR contractor and construction is slated to begin in July 2023.

B. Description of Dam and Other Features

The principal spillway consists of a 154-foot-long concrete ogee spillway weir founded on rock, with a crest at elevation of 880.0 feet. The principal spillway discharges into a stilling basin, which is located immediately downstream of the spillway and founded on rock. The stilling basin slab design elevation is 825.0 feet. Concrete gravity training walls are located on both sides of the stilling basin. The ogee spillway training walls consist of the vertical faces of the monoliths for the non- overflow sections that are located on either side of the ogee spillway weir. The stilling basin discharges into a natural downstream channel. **Figure 2** presents the location of the primary features of the dam.

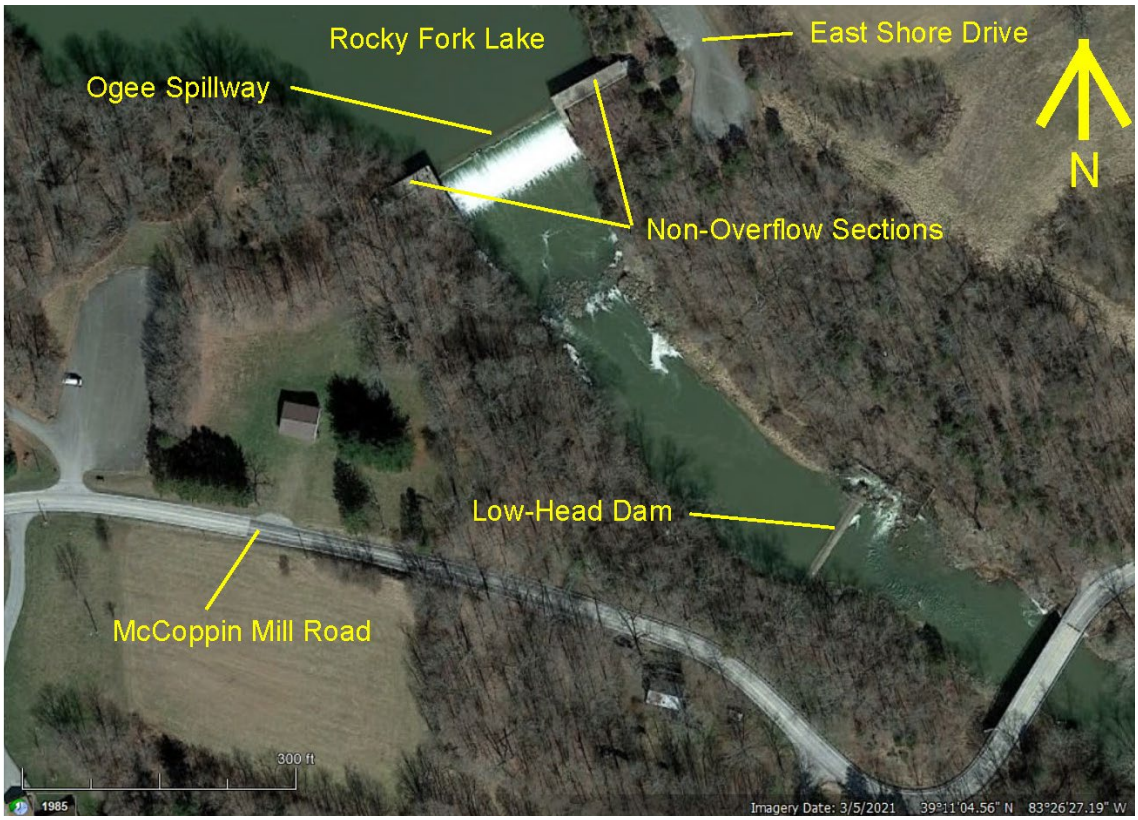


Figure 2. Rocky Fork Dam primary features

Concrete non-overflow sections are located on both sides of the ogee spillway. The design elevation for the top of the non-overflow sections is 898.0 feet. The non-overflow sections each consist of a 14-foot-high chimney section with a 7H:10V sloping downstream face. The upstream faces of the non-overflow sections have a 1H:20V slope to match the slope of the ogee spillway upstream face. The non-overflow sections are founded bedrock. Soil was placed between the bedrock and concrete on the downstream sloping portion of the non-overflow sections. The west non-overflow section consists of three monoliths and the east non-overflow section consists of four monoliths. The monoliths adjacent to the ogee spillway weir extend downstream to form a portion of the training walls for the ogee spillway weir. **Figure 3** shows a plan view of the dam and the stilling basin. **Figure 4** shows a cross-section view of the dam.

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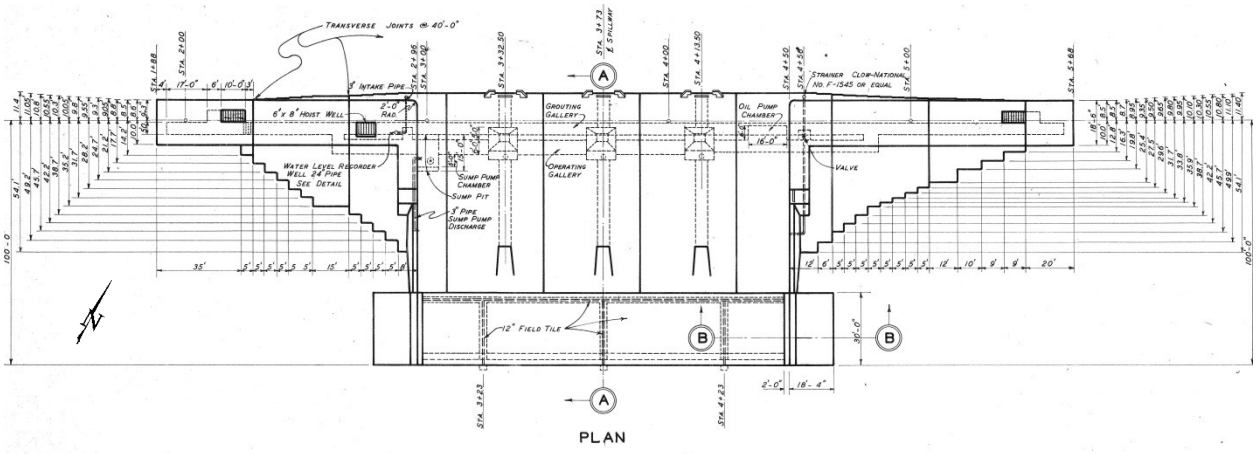


Figure 3. Rocky Fork Lake Dam Plan View (1951 Construction Drawings).

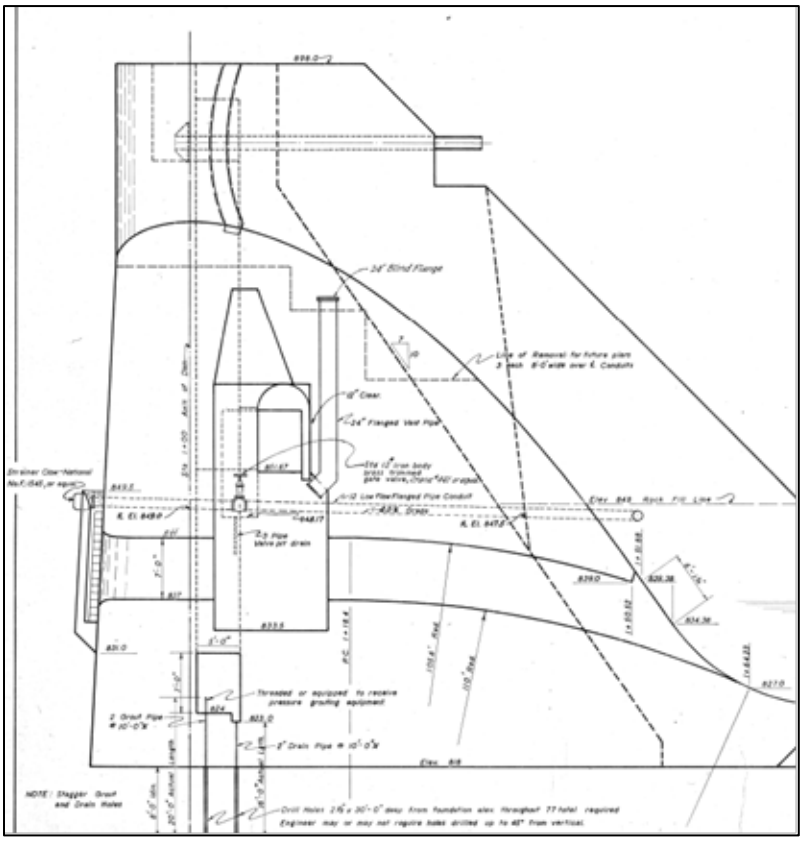


Figure 4. Section View at the Sluiceway in the Ogee Spillway (1951 Construction Drawings).

However, the original gate design resulted in several challenges. The sluiceway was beneath the spillway and not intake tower or other access to the gate was provided – the only way to access the gate was from the downstream end of the sluiceway. The upstream face of the dam and the sluiceway upstream of the gate could only be accessed when a bulkhead was placed across the sluiceway on the upstream face of the dam. Because there was no intake tower or other lifting mechanism provided for in the original design, the bulkhead installation required two divers to remove the trash rack, then set the bulkhead. Once any gate inspection was complete, the divers would be needed again to remove the bulkhead and reset the trash rack. **Table 1** on the following page presents the pertinent data for the project.

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Table 1. Rocky Fork Dam Pertinent Data

General Information			
Dam Name	Rocky Fork Lake Dam		
NID Number	OH00302		
File Number	9133-001		
Constructed	1952		
Owner	ODNR, Division of Parks and Recreation		
Hazard Class	I		
Location Information			
County	Highland		
State	Ohio		
Stream	Rocky Fork of Paint Creek		
Latitude	39°11'03"		
Longitude	83°26'22"		
Structure Information			
Purpose	Recreation, Public		
Drainage Area	114 square miles		
Type of Impound	Dam and Spillway		
Type of Structure	Concrete, Gravity		
Dam Length	380 feet		
Dam Height	73.5 feet		
Top Width	19 feet		
Upstream Slope	N/A		
Downstream Slope	N/A		
Fill Volume	29,000 cubic yards		
Lake Drain	5-foot by 7-foot sluiceway through ogee section		
Principal Spillway	154-foot ogee section in center of dam with a 12-inch diameter low flow pipe		
Emergency Spillway	None		
Maximum Discharge	42,489 cubic feet per second		
Reservoir Data			
Feature	**Elevation (feet - MSL)**	**Area (acres)**	**Storage (acre-feet)**
Top of Dam	898.5	3,925	87,050
Principal Spillway	880.5	2,100	35,299
Streambed	825	---	---
Foundation	820	---	---

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C. Gate and Controls

During construction of the original project, a set of drawings for the sluiceway and gate were developed. The original design of the sluiceway inlet is shown in **Figure 5** below.

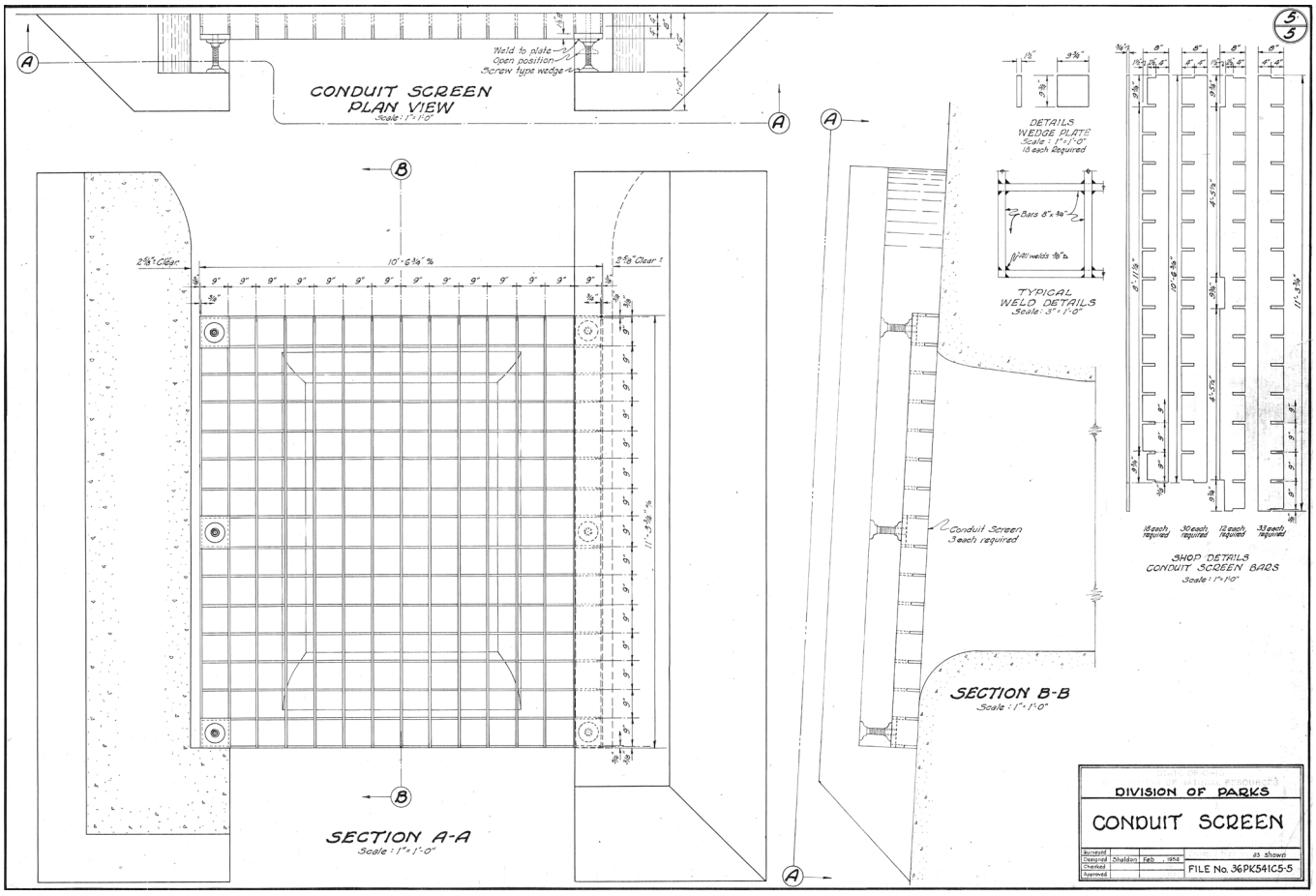


Figure 5. Sluiceway Inlet Frame and Trash rack

As can be seen on the drawing, the sluiceway has a U-shaped concrete frame constructed immediately upstream of the inlet. The underwater inspection of this feature noted no distress to this frame. The concrete frame was designed to hold a trash rack using screw-type wedges to keep the trash rack against the upstream face of the dam. The trash rack and screw-type wedges could be removed, and a bulkhead installed within the frame to isolate the sluice gate for inspection, maintenance, or repairs.

II. GATE INSPECTION

The inspection of the gate, the sluiceway, and the liner was scheduled for January 2023. The results of the inspection were to be used to determine a recommended path forward for the rehabilitation, partial replacement, or full replacement of the slide gate. In support of a potential replacement of the gate only, the inspection also included measurements of the gate guide rails.

Since the original bulkhead was 70 years old and personnel would be immediately downstream of the bulkhead inside the sluiceway, a new bulkhead was designed and fabricated for the inspection. On January 10, 2023, divers removed the trash rack from the sluiceway inlet and replaced it with the new bulkhead. In addition, ODNR lowered the pool a few inches below the crest of the ogee spillway to make it easier to access the downstream end of the sluiceway. The sluiceway was a confined space entry so confined space protocols were developed and followed for the inspection. A diver was also on hand to assist with ingress and egress from the sluiceway.

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On January 11, 2023, ODNR raised the gate so that the sluiceway was fully open. Some water was flowing from the sluiceway, but the bulkhead appeared to be working adequately.

Following an on-site safety meeting, structural engineers from Tetra Tech, as well as staff from the Construction Manager and ODNR, entered the downstream end of the sluiceway and proceeded to the gate for the inspection. Due to leakage from the new bulkhead, there was approximately 2 inches of flowing water on the sluiceway floor. The water made it difficult to take a good picture of the gate sill but did not otherwise affect the inspection. The inspection of the sluiceway and gate included the following activities:

- Inspection of the cast iron frames and liner, the vent intake, and the concrete immediately upstream and downstream of the cast iron components.
- Inspection and measurement of the gate guides and sill, and the visible parts of the gate leaf.
- Observation of the visible portions of the vent liner above the intake.
- Photos and video of the conduit interior.

On exiting the conduit, an attempt was made to determine the shape of the conduit floor at the downstream end to support the design of the bypass pipe to be used in the dewatering of the tailbay but was unable to determine anything due to the depth of the water from the leaking bulkhead at that location.

After completion of the sluiceway inspection, the gate was closed, the space downstream of the bulkhead was rewatered, and the bulkhead was removed. Once the divers removing the bulkhead were clear, an inspection was performed of the dam gallery to observe the top of the Bonnet, the Bonnet Cap, and the Operating Cylinder. In addition, measurements were made of the gallery interior and the gate and operator components. The arrangement of the slide gate, frames and liner is shown in **Figure 6** below.

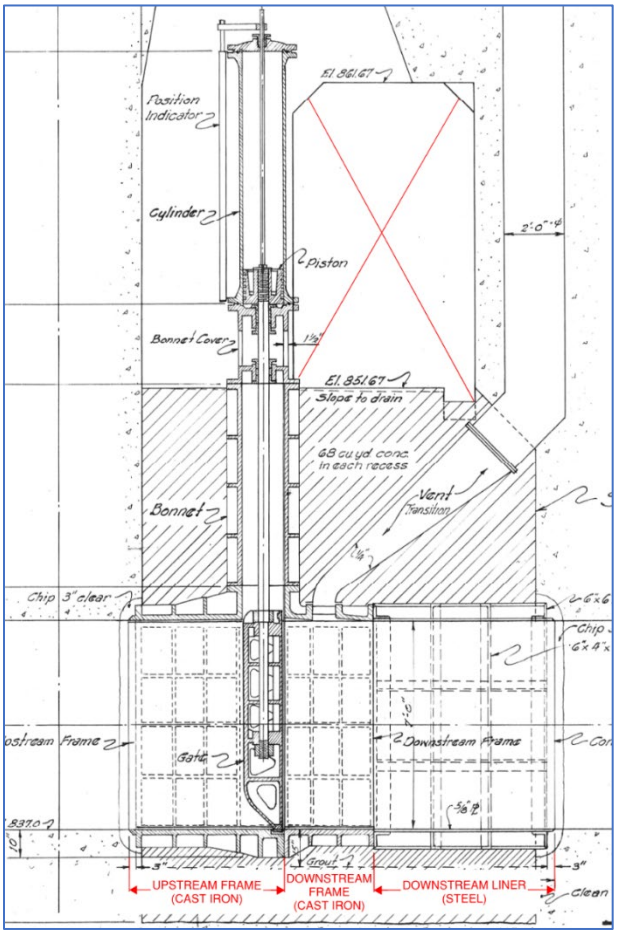


Figure 6. Slide Gate and Outlet Conduit Components

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III. INSPECTION FINDINGS

* Photos mentioned in the following sections are provided below in section **"*D. Photographs"***

A. Outlet Conduit

The concrete upstream and downstream of the gate was generally in very good condition. There was one shallow flat-bottomed delamination approximately 2 inches by 3 inches across and 1/4-inch deep near the top of the left wall just downstream of the liner **(Photo 1).** The delamination had the appearance of a poorly adhered surface layer, but there were no other similar defects.

The upstream and downstream frames and the downstream liner all had scaling rust, with the worst condition on the downstream frame just downstream of the gate. The deepest areas of corrosion losses were approximately 1/2-inch. Most areas with significant losses on the frames were in the range of 1/4-inch to 3/8-inch **(Photo 2).** Most areas of the frames and liners showed pitting corrosion with losses ranging from around 1/4-inch to 1/16-inch **(Photo 3)**. Corrosion losses inside the vent liner appeared similar to typical areas of scaling corrosion on the downstream liner **(Photo 4)**. Only the sill of the gate was visible during the inspection. Most of the sill was in good condition with minor losses, but at both corners, there was pitting corrosion with the deepest pits around 1/4-inch deep **(Photo 5).** The gate guides and sill were generally in good condition. **(Photos 5, 6 and 7).**

B. Gallery and Gate Operator

Overall, the Gallery is in good condition and dry. The exception is a leak in the gallery roof above the Operating Cylinder that has led to corrosion on the head of the Cylinder, on areas of the Bonnet Cover and around the parts of the Bonnet that protrude above the gallery floor. The base flange of the Bonnet Cover, the top flange of the Bonnet, and the bolts connecting them show complete coating loss and significant corrosion. **(Photo 8)** The skin and stiffener plates at the top of the Bonnet below the flange were hidden by standing water, grime, and the small size of the gap, but can be expected to have similar or worse corrosion. The rest of the Bonnet Cover shows isolated areas of coating failure and rust staining.

The base flange of the Cylinder and the top flange of the Bonnet Cover have some areas of total coating loss and pitting corrosion, but not as severe as the base of the cover **(Photo 9).** The top cover of the Cylinder has similar, but more extensive coating loss and corrosion to the bottom flange **(Photo 10).**

The drainage channel for the Gallery was cut under the downstream wall at the bay that contains the Operating Cylinder on the upstream side, and because of the this, the top surface of the vent is exposed in the bottom corner of the channel. The exposed portion of the vent has suffered complete coating loss and has corroded through, leaving a hole approximately 2 inches by 3/4 inch **(Photo 11).**

C. Conclusions

Overall, the sluiceway and gate for Rocky Fork Dam are in moderate condition. With proper rehabilitation and maintenance, the existing equipment could function adequately for some time. However, it is our understanding that given the current need for work on the spillway and tail bay, ODNR prefers to take this opportunity to undertake all repairs and replacements required so that the dam will need no future work other than routine maintenance for many years. Therefore, Tetra Tech recommended that the existing slide gate and all of its operating machinery be removed and replaced with a new stainless steel bonneted gate. This will entail removing most of the surrounding second-stage concrete as well. While replacing the gate, attempts should be made to seal the leak above the gallery in order to limit the potential for future damage.

While there is no specific need to remove the vent transition since the steel was only necessary as a form liner, fitting the vent outlet of a new gate to the irregular shape of the existing transition will be difficult. In addition, replacing the vent transition can also address the existing hole into the gallery. Therefore, we recommend removing the existing liner to a point above the hole where the vent section is a round pipe and making the connection to the new gate vent there. We also recommend rerouting the drainage channel in the floor slightly upstream so that there is at least two inches of concrete cover over the vent and applying a sealant over the channel floor in that area to prevent water migration to the vent through cracks. The recommended extent of demolition is shown in **Figure 7** below. Depending on the dimensions of the new gate, it is possible that some additional concrete demolition will be needed. However, we recommend that to the extent possible, the demolition be limited to the second-stage concrete above the conduit and to the grout surrounding the frames and liner.

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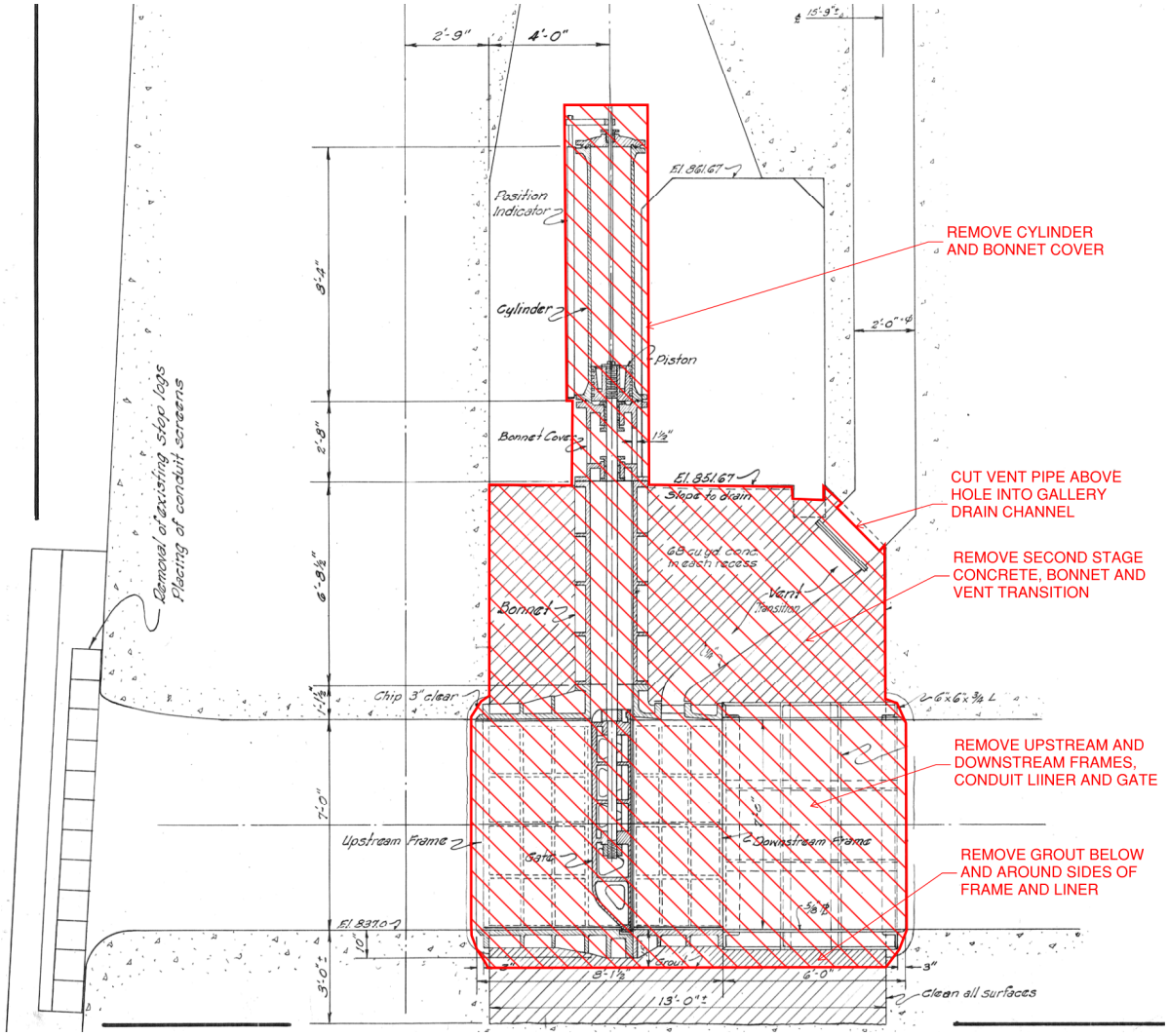


Figure 7. Recommended demolition for installation of new gate and vent

The original designers of the dam made no provisions for accessing the sluiceway and the gate unless the pool was lowered below the inlet level or divers were used to set a bulkhead. Removal of the gate and installing a replacement gate will also be challenging. There is a hoist well in the right non-overtopping section, but this is the opposite side of the dam from the gate and sluiceway. The Construction Manager is also not confident the new gate can be maneuvered from the hoist well into the operating gallery as a single piece and the new gate will likely have to be lowered into the dam in two sections, then reconnected to set it into place.

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D. Photographs



Photo 1: Surface concrete delamination. End of Liner is just to the left

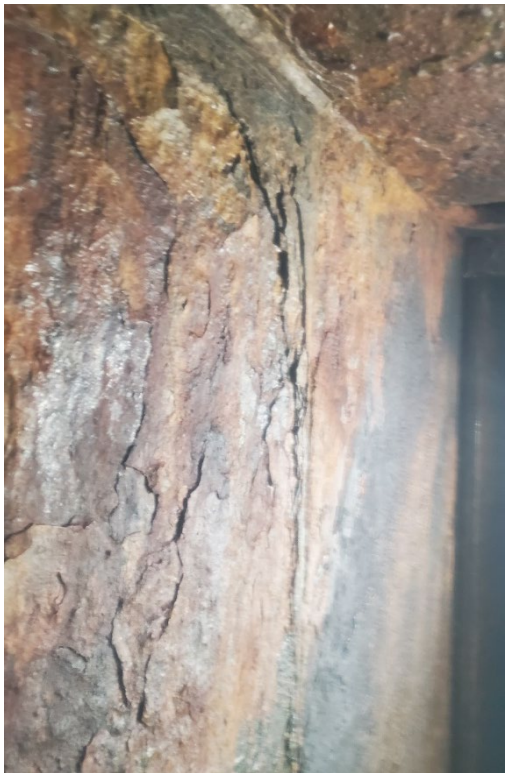


Photo 2: Typical scaling rust on Downstream Frame

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Photo 3: Corrosion on Downstream Frame (Foreground) and Liner (Midground)



Photo 4: Corrosion inside vent liner

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Photo 5: Pitting corrosion at lower left corner of gate. Right corner similar.



Photo 6: Gate Guides

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Photo 7: Gate guide and sill



Photo 8: Corrosion on Bonnet Cover and Bonnet flange



Photo 9: Top of Bonnet Cover and base of Cylinder showing coating loss and corrosion

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Photo 10: Coating loss and corrosion at top of Cylinder



Photo 11: Drainage channel and exposed surface of vent liner with hole

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

Impact of Groundwater Observations on Dam Design

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ABSTRACT

The Eagle Creek Flood Basin was designed to reduce the risk of flooding in Findlay, Ohio. A field exploration was completed consisting of over 100 geotechnical borings and 29 open-standpipe piezometer installations along the proposed embankment alignment (3.7 miles) and within the basin footprint (765 acres). Early design phases included a desktop study and limited soil resistivity testing to consider corrosivity to proposed dam elements such as cutoff walls and concrete spillway foundations. These results indicated that the soils could be corrosive to steel. During piezometer data collection, field team members documented degradation of the stainless-steel cables used to suspend electronic transducers and noted a black substance that had accumulated within the open standpipes. The engineering design team engaged with groundwater professionals to perform environmental site assessments (ESA) to evaluate groundwater conditions that may have led to the observations within the piezometers. The initial ESA focused on groundwater sampling and testing for total petroleum hydrocarbons that may have been related to the historical oil production in the area. It was thought that the observed black substance in the piezometers could be degraded petroleum. The initial ESA results indicated that the groundwater did not contain petroleum hydrocarbons. However, during the initial groundwater sampling event, the field crew noted strong odors commonly associated with hydrogen sulfide produced by sulfur-reducing bacteria. Additionally, groundwater geochemical testing results for oxidation-reduction potential indicated highly reducing conditions, which creates an environment conducive to the sulfur-reducing bacteria. Metals can corrode by hydrogen sulfide, and iron in the groundwater can form a black precipitate (iron sulfide). Based on field observations and geochemical testing results, it was believed that the site contained naturally occurring hydrogen sulfide. A second ESA was conducted and confirmed the presence of hydrogen sulfide. Because this condition was identified, the engineering team was able to specify construction materials and concrete additives during design so that cutoff walls and concrete foundations would be more resilient to the corrosive conditions within the groundwater. Identification during design reduced the risk for potential future dam safety issues and costly repairs due to the corrosive conditions.

I. INTRODUCTION

Hancock County and the City of Findlay, Ohio experience frequent and significant flooding from the Blanchard River and its major tributaries, often flooding agricultural land and the City's streets, homes, and businesses within the floodplain. As a result, the Maumee Watershed Conservancy District (MWCD) developed the Hancock County Flood Risk Reduction (HCFRR) Program. Stantec Consulting Services Inc. (Stantec) was contracted by the MWCD to develop potential alternative solutions to reduce the risk of flooding.

The Eagle Creek Flood Basin is a flood-risk reduction project that is part of the HCFRR Program [1]. It is designed to have storage capacity for excess flows from the 1% Annual Chance Exceedance (ACE) (100-year) storm event on Eagle Creek and to safely pass flood events of greater magnitude. A vicinity map of the project area is shown in Figure 1.

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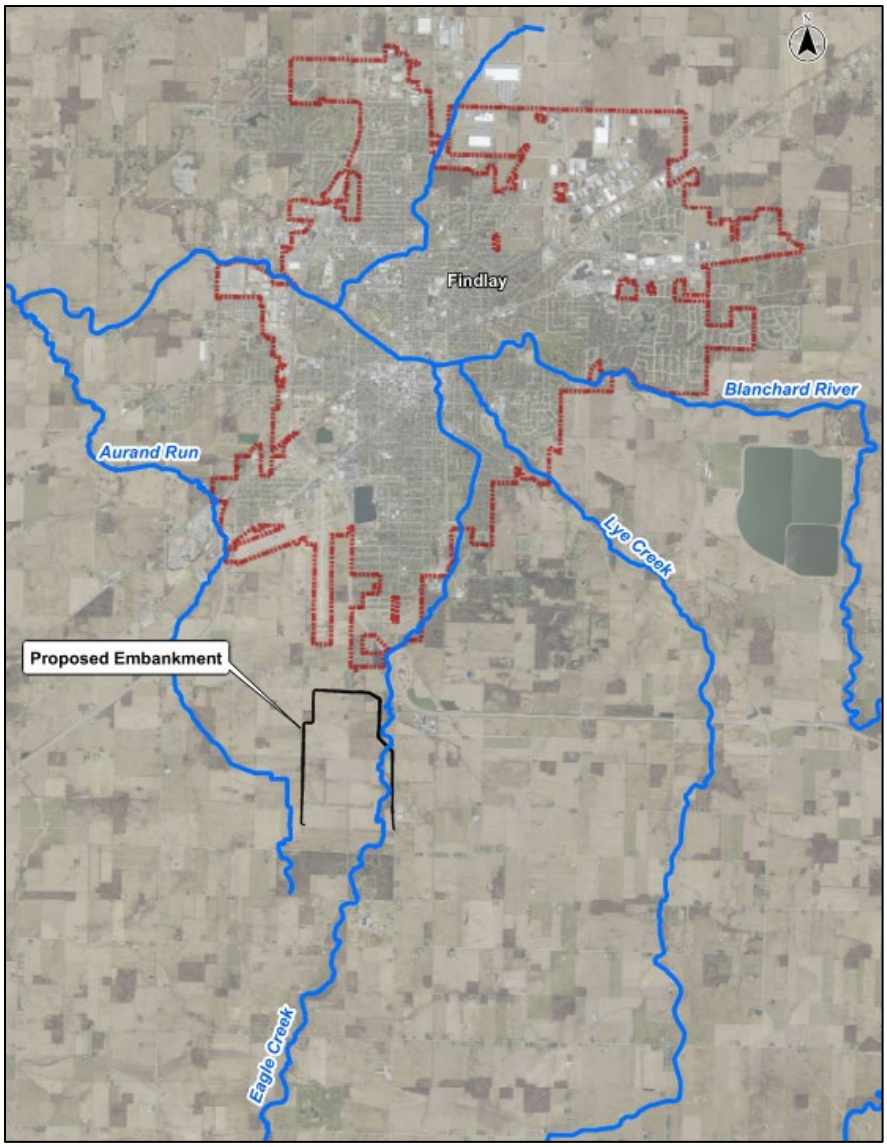


Figure 1. Eagle Creek Flood Basin Vicinity Map.

The basin will remain dry during normal flows (up to approximately 650 cubic feet per second (cfs) along Eagle Creek). The basin will begin to store flood waters during larger rain events. The purpose of the project is to reduce the peak flow rate in Eagle Creek and the Blanchard River during these large storm events, thereby reducing the downstream water surface elevations and associated flood risk. The primary project goals include the following:

- Reduce water surface elevations during flooding events along Eagle Creek and the Blanchard River; and
- Reduce flood risk of potential structural, social, and environmental damages.

A secondary project goal is to improve water quality by creating wetlands, native habitats, and riparian corridors within the inundation area. A tertiary goal is to provide passive recreation opportunities on the project site. The Eagle Creek Flood Basin project site is located in Eagle Township in Hancock County, Ohio, approximately four miles south of the City of Findlay's downtown area.

The primary flood risk reduction element for the project is an earthen embankment dam forming an approximately 3.7 mile long, in-line, dry-storage basin, with a maximum height of about 30 feet. Other primary components consist of a combined concrete principal and auxiliary spillway structure, exterior drainage features, and interior drainage improvements and land use design. Seepage cutoffs are included below the combined principal and auxiliary spillway structure, and at select locations below the embankment dam. The impoundment area of the basin within the dam alignment will be approximately 765 acres. The site layout is shown in Figure 2, and a three-dimensional rendering of the proposed project is shown in Figure 3.

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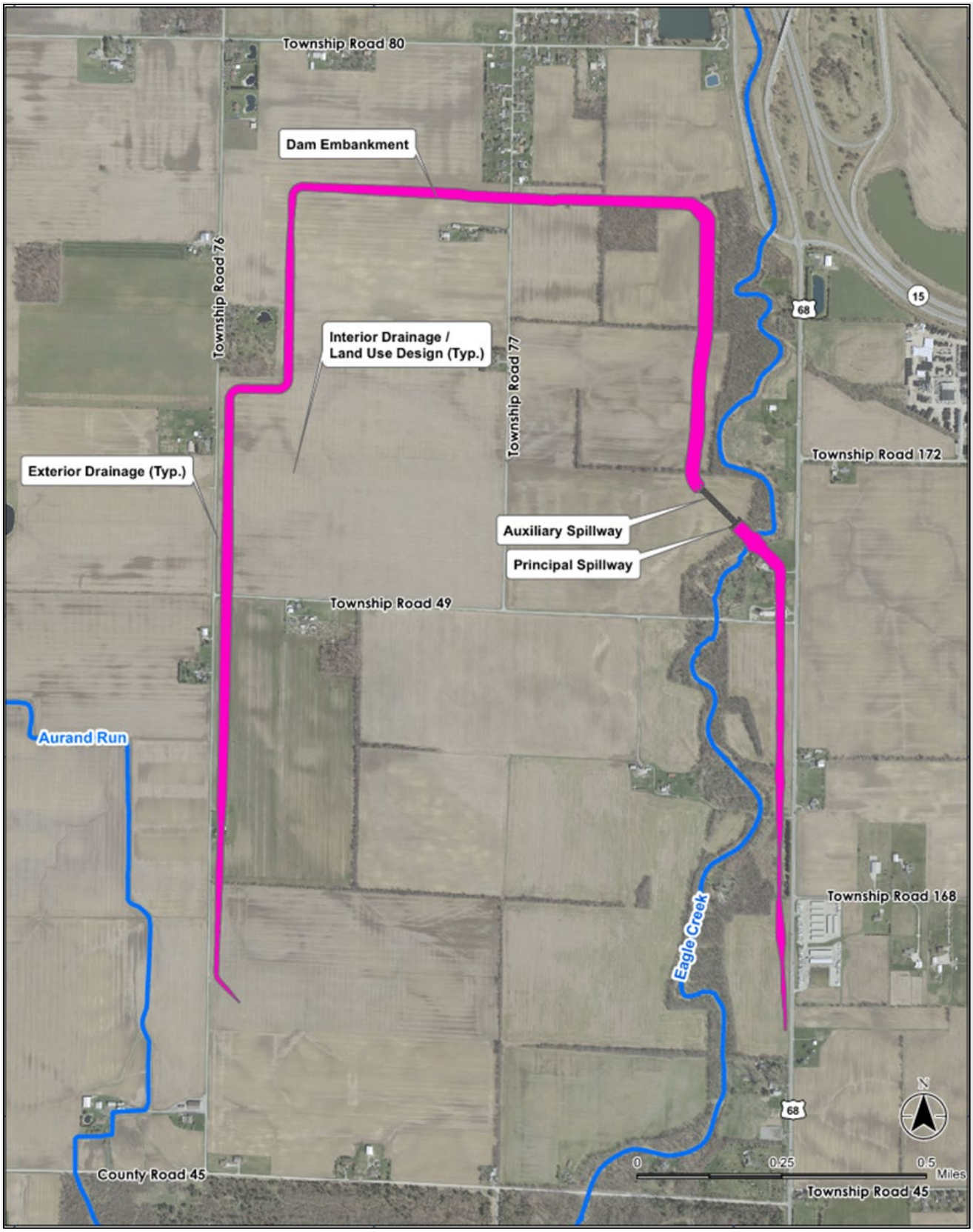


Figure 2. Eagle Creek Flood Basin Site Layout.

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Figure 3. Three-Dimensional Rendering of Project – Embankment and Combined Spillway Structure

Model results show that the Eagle Creek Flood Basin project results in a peak flow reduction of about 2,550 cfs (16% decrease) on the Blanchard River during the 1% ACE event. This flow reduction translates to an approximately 2.2 feet lower base flood elevation near the confluence with Eagle Creek. The reduction in water surface elevations along Eagle Creek and the Blanchard River is estimated to remove approximately 1,830 parcels and 1,400 acres from the regulatory floodplain.

To date, the project has progressed through Final Design and construction is expected to begin in early 2024. The construction duration is estimated to be approximately 18 to 24 months.

II. SUBSURFACE EXPLORATION AND INSTRUMENT INSTALLATION

Stantec advanced 120 borings and excavated six (6) test pits during the design to obtain geotechnical data for the proposed flood basin. Seventy-two (72) borings were located along the proposed embankment alignment and 15 borings at the proposed spillway structure. Ten (10) borings were located at regular intervals approximately 400 feet upstream from the toe of the proposed embankment to evaluate the thickness and continuity of a low-permeability upper fine-grained soil layer. Twenty-three (23) borings and six (6) test pits were located within the interior of the basin to classify and quantify potential borrow materials. Water pressure testing was performed in fifteen (15) borings to provide data for estimating permeability and flow regimes through the bedrock.

Twenty-nine (29) open standpipe piezometers were installed, and twenty-five (25) were outfitted with water level transducers to collect pressure and temperature readings. Transducers were suspended within the open standpipes using stainless steel cables. The piezometer data was collected at regular intervals to evaluate piezometric conditions and trends for consideration during design.

The geotechnical data obtained from the explorations were used to select representative soil layers for foundation soil characterization and assess the needs for seepage cutoffs or other design features. Soil deposits were generally around 20 feet thick, overlying dolomite bedrock. Groundwater was typically observed at depths ranging from 3 to 10 feet. Four soil layers were identified overlying bedrock: Upper Fine-Grained material, Upper Coarse-Grained material, Lower Fine-Grained material, and Lower Coarse-Grained material. The subsurface soil layers are shown graphically in Figure 4.

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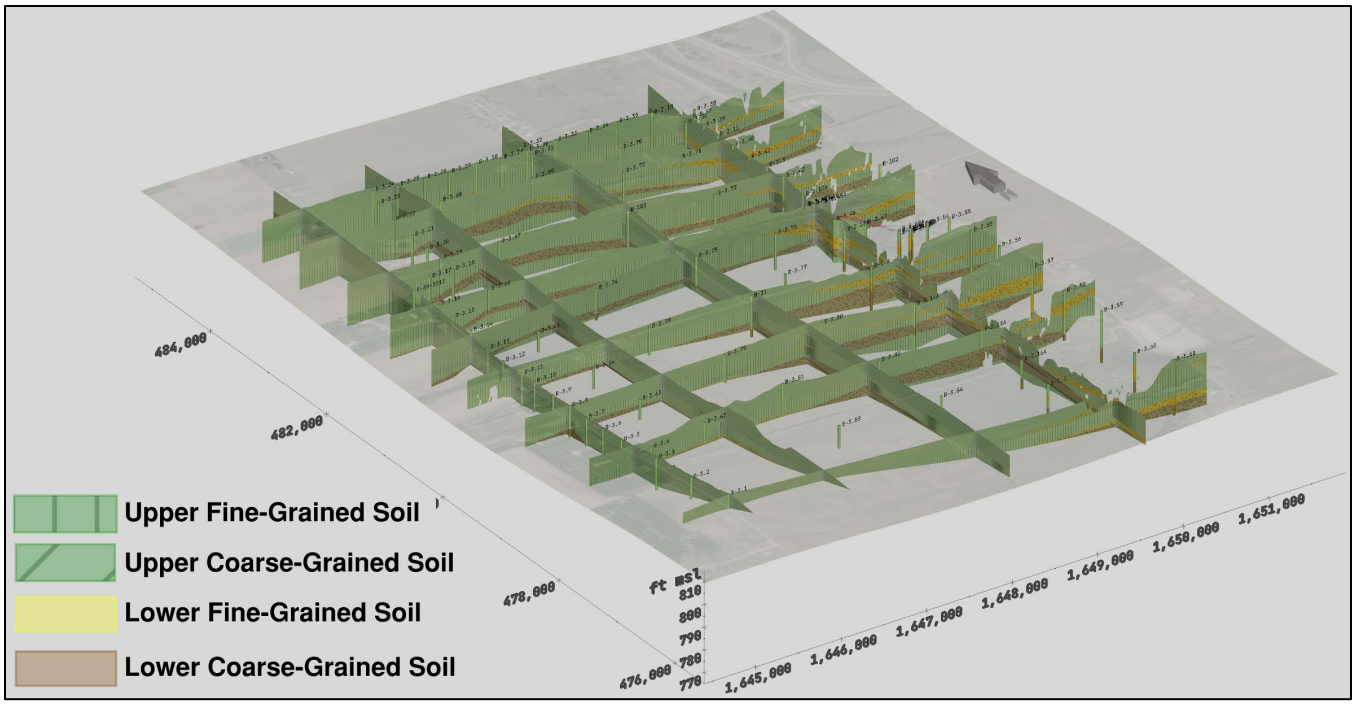


Figure 4. Subsurface Soil Layers.

The Upper Fine-Grained material consists of glacial and alluvial (near Eagle Creek) deposits and was encountered in each of the borings advanced at the site. The thickness of this layer ranged from 1.5 to 12.5 feet, averaging 6.6 feet. This soil typically classified as a lean clay (CL) with varying amounts of sand, and an average plasticity index of 17. This soil was typically described as brown to orange-brown.

The Upper Coarse-Grained material consists of alluvial deposits and was typically encountered in the central and eastern portions of the proposed basin. The presence and thickness of this material generally increased in borings located closer to Eagle Creek. In the laboratory, samples typically classified as a silty sand (SM), clayey sand (SC), or a silty, clayey sand (SC-SM), with about 40 percent of the samples classifying as non-plastic. Samples that did exhibit plasticity had an average plasticity index of 8. This soil was typically described as gray, although it was common for the soil to be described as brown at higher elevations. The Upper Coarse-Grained material was typically not encountered in the borings advanced along the western dam alignment and was interlayered with the Lower Fine-Grained material when it was encountered.

The Lower Fine-Grained material consists of glacial till deposits that exhibited increased sand and gravel content, less plasticity, and higher blow counts when compared to the Upper Fine-Grained material. This soil typically classified as a lean clay (CL) with varying amounts of sand and gravel, with an average plasticity index of 11. Soils in this layer were typically described as gray. This layer was encountered below the Upper Fine-Grained material and was interlayered with the Upper Coarse-Grained material.

The Lower Coarse-Grained material was typically encountered in the central and eastern portions of the proposed basin. The presence and thickness of this material generally increased in borings located closer to Eagle Creek. Where present, this layer was typically encountered directly above the top of bedrock; however, there were several instances where fine-grained material was observed between this layer and the top of rock. The Lower Coarse-Grained material was typically coarser than the Upper Coarse-Grained material, with visual descriptions and laboratory classifications indicating increased gravel content compared to the Upper Coarse-Grained material. Samples from this layer typically classified as non-plastic silty sand (SM), well-graded sand with silt (SW-SM), or silty, clayey sand (SC-SM). Approximately two-thirds of the samples classified as non-plastic. Samples that did exhibit plasticity had an average plasticity index of 5. This soil was typically described as gray or gray and black. Similar to the Upper Coarse-Grained material, the Lower Coarse-Grained material was typically not encountered in the borings advanced along the western dam alignment.

The bedrock was described as gray dolomite, slightly weathered, fractured to highly fractured, slightly rough, and thin to medium bedded. Fractured zones and water loss were noted in the bedrock until the termination depths.

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III. CORROSIVITY OBSERVATIONS

The preliminary design phase included a desktop study and limited soil resistivity testing to consider corrosivity to proposed project elements such as cutoff walls and concrete foundations. Eight (8) soil resistivity tests were performed on bulk and undisturbed samples obtained from the subsurface exploration program. The soil samples were generally obtained from the Upper Fine-Grained material, although samples were also obtained from the Upper Coarse-Grained and Lower Fine-Grained materials. The results indicated soil resistivity between 1,236 and 2,706 ohm-cm.

Initial corrosivity potential of the soil to ductile-iron pipes was estimated using methods as outlined by the American Water Works Association (AWWA) [2]. This standard uses a point system to evaluate the soil corrosivity potential to ductile-iron pipes. Points are summed based on results of resistivity, pH, redox potential, and sulfide laboratory testing, with additional points added for the drainage condition. A result of 10 points or more indicates that the soil is corrosive to ductile-iron pipe and that protection is needed. The standard assigns soil with resistivity values less than 1,500 ohm-cm 10 points, indicating that the soil is corrosive to ductile-iron pipe. Of the eight (8) soil resistivity tests performed, four (4) resulted in resistivity values less than 1,500 ohm-cm.

Based on the results of the laboratory resistivity testing and according to AWWA [2], the soil was expected to be corrosive to ductile-iron pipe. Additionally, characterization provided by the Natural Resources Conservation Service (NRCS) Web Soil Survey [3] indicated that soils at the site were rated as having high potential for corrosion of steel. Therefore, the designers were aware that final design of project elements would require coatings or protection from corrosive conditions.

Stantec also conducted an instrumentation monitoring program to obtain periodic water level readings following piezometer installation and continuing through final design. Readings were collected on 15-minute intervals using electronic transducers and began after installation in May 2021. In August 2021, eight (8) of the piezometers were observed to be damaged or were not properly recording data. Observed damages included transducers that had been disconnected from the stainless-steel suspension cables and lost down the standpipes, as well as some broken or bent standpipes at the surface. Other transducers did not collect data as intended. Following the site visit in August 2021, six (6) of the eight damaged piezometer transducers were repaired or reset to properly record data. The remaining two (2) transducers had sustained irreparable damage and were determined to be out of service. At the time, the damaged and malfunctioning instruments were assumed to be due to installation errors (poor connections to the stainless-steel suspension cables, incorrect software settings) or the standpipes being struck by agricultural equipment.

A field visit was conducted in November 2021 to obtain additional piezometer data. Manual piezometer readings were collected from the instruments. Transducer data was obtained from nineteen (19) of the instruments. Three (3) instruments experienced software malfunctions and one (1) additional instrument was determined to be out of service, totaling three (3) instruments out of service during this site visit. One (1) of the previously out of service instruments was replaced and set to record data.

Another field visit was conducted in June 2022 to obtain additional piezometer data. Manual piezometer readings were collected from the instruments. Four (4) additional transducers were determined to be out of service, totaling six (6) instruments out of service. Sixteen (16) transducers experienced software malfunctions and were inactive (did not record data) between November 2021 and June 2022. The inactive transducers were re-activated so they would collect data prior to the next field visit.

A final data-collection field visit was made in September 2022. Transducer data was obtained from eleven (11) instruments. Six (6) additional instruments were determined to be out of service, totaling twelve (12) instruments out of service. Three (3) instruments experienced software malfunctions and did not record data. Of the twelve (12) instruments that were out of service, eight (8) had visible evidence of corroded suspension cables and had been lost down the open standpipes.

The observations made throughout the course of the piezometer monitoring and data collection included corrosion of the stainless-steel suspension cables and a black residue within the open standpipes and on the transducer equipment and suspension cables. A summary of the instruments exhibiting corrosion is shown in Figure 5. Photographs of the initial suspension cable condition and the corroded condition with black residue are shown in Figures 6 and 7, respectively.

Observations of the multiple damaged transducers, corroded stainless-steel suspension cables, and black residue accumulation indicated a possible pattern at the site that required evaluation during the final design phase.

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A field program was initiated to test groundwater samples retrieved from the installed piezometers. Because of the observed corrosion of the stainless-steel cables, it was important to evaluate potential causes that could result in similar impacts to the proposed spillway structures and seepage cutoffs.

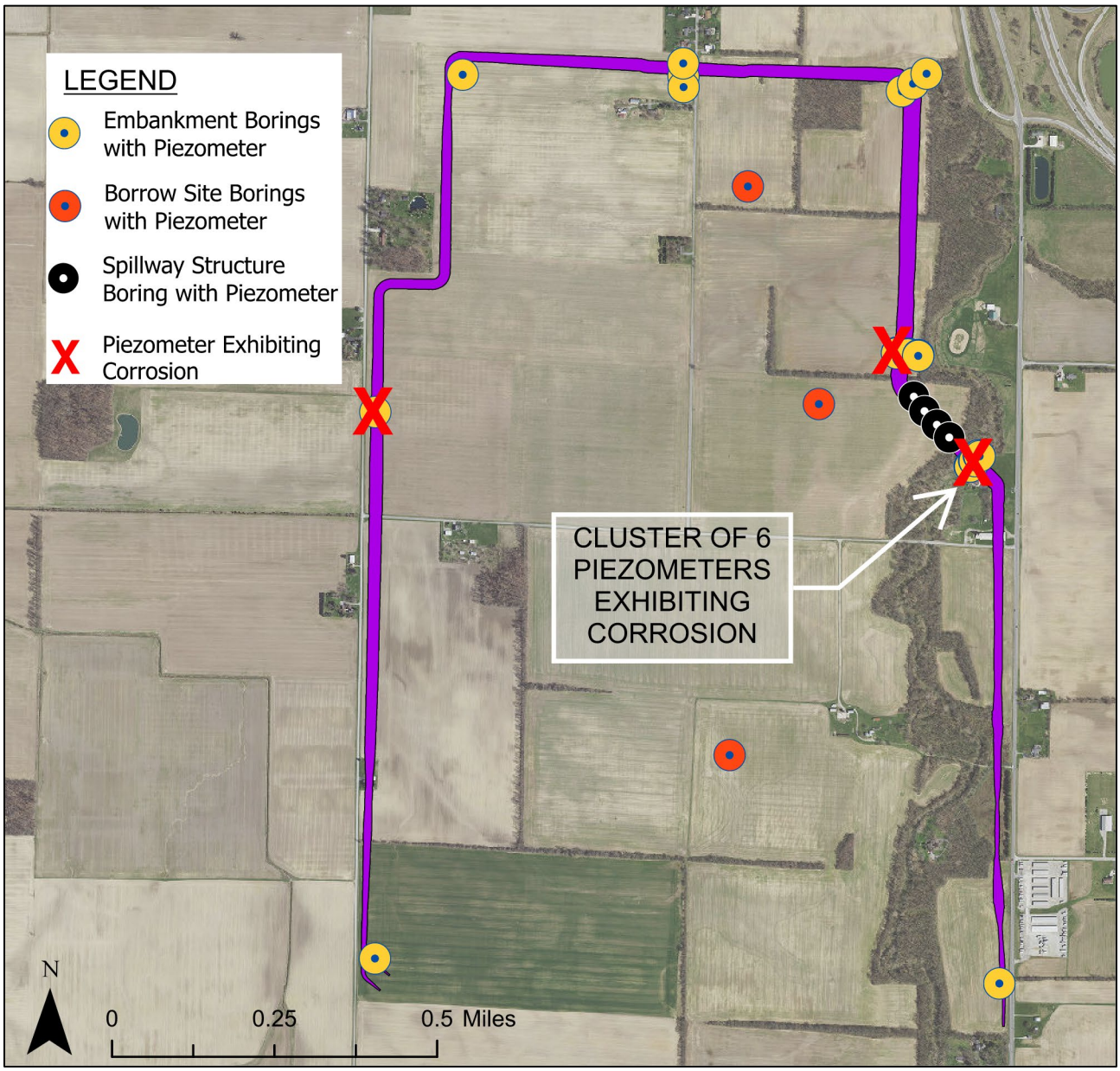


Figure 5. Summary of Instruments Exhibiting Corrosion

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Figure 6. Initial Condition of Stainless-Steel Transducer Suspension Cable



Figure 7. Corroded Condition of Stainless-Steel Transducer Suspension Cable

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IV. ENVIRONMENTAL SITE ASSESSMENTS

The Stantec engineering design team collaborated with Stantec groundwater professionals to discuss potential causes of the field observations and planned an ESA to evaluate groundwater conditions. The initial discussions focused on the observed residue in the standpipes, boring logs that documented hydrocarbon odors in samples from near the top of bedrock, and a regional history of oil and gas well operations. It was hypothesized that the residue could have been degraded petroleum that infiltrated the open standpipes through the piezometer screens. It was determined that targeted groundwater sampling was necessary to evaluate the observed conditions and attempt to correlate any findings to specific geologic strata or isolated locations within the project area.

An initial groundwater sampling and testing program was conducted to determine if the black residue observed on the piezometer transducers, as well as the petroleum hydrocarbon odors noted during the geotechnical exploration, were naturally occurring. Groundwater samples were collected from select standpipes and analyzed for the following constituents:

- Total Petroleum Hydrocarbon (TPH),
- Middle and Heavy Distillates/Diesel Range,
- Extended Range Organic, and
- TPH Fingerprinting.

Groundwater analytical results indicated that the observed residues in the piezometers were not a result of naturally occurring hydrocarbons in the groundwater. However, during the sampling event, the field geologist noticed a strong rotten egg smell (commonly associated with sulfur-reducing bacteria) and significant deterioration of the metal cables used to suspend the transducers in the open standpipes. Groundwater geochemical results for Oxidation Reduction Potential (ORP) also indicated highly reducing conditions in the sampled piezometers.

According to the Ohio Watershed Network [4], hydrogen sulfide gas can occur naturally in groundwater. Metals can corrode by hydrogen sulfide, and iron in the groundwater can form a black precipitate (iron sulfide). The presence of hydrogen sulfide can be a result of several sources, such as: decomposing deposits of organic matter; wells drilled in shale, sandstone, or near coal or peat deposits; or sulfur-reducing bacteria that feed on naturally occurring sulfates in water, producing the hydrogen sulfide gas. Significant deposits of organic matter were not observed during the subsurface exploration for the project. However, there is a regional history of oil and gas well production, and there was field evidence of sulfur-reducing bacteria.

Based on the ORP results and the on-site observations, corrosion of the transducer cables and the black residue in the piezometers was likely caused by naturally occurring hydrogen sulfide. Further groundwater sampling and testing was warranted to support this hypothesis. Additional constituents of interest in groundwater included sulfates, sulfides, and hydrogen sulfide in groundwater.

Considering the potential corrosive nature of hydrogen sulfide, a second ESA sampling event was conducted to further characterize surface and groundwater conditions, to support material selection for the seepage cutoffs and spillway foundations. Groundwater samples were collected and analyzed for the following constituents:

- hydrogen sulfide,
- sulfate,
- sulfide,
- dissolved gases (ethane, ethene, methane),
- ferrous iron,
- iron,
- manganese, and
- total organic carbon (TOC).

Results from the second ESA sampling event confirmed the presence of hydrogen sulfide in groundwater with the highest concentrations occurring in the area of the proposed seepage cutoff wall below the spillway structures. However, hydrogen sulfide was not detected in surface water samples. Groundwater geochemical results for ORP indicated highly reducing groundwater conditions (i.e., anaerobic). Anaerobic conditions along with the presence of organic compounds provide a favorable environment for sulfur reducing bacteria. Concentrations of sulfate were detected in each of the sampled standpipes, which has the potential of being converted to hydrogen sulfide. The occurrence of hydrogen sulfide did not appear to correlate to a given geologic material in which the piezometers

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were screened. Attempts were made to correlate the occurrence of hydrogen sulfide to geologic strata (i.e., upper soils, lower coarse grain material, or dolomite) but no strong correlation was observed.

V. CONCLUSIONS

Based on the results from the second ESA, alternatives to metal structures (e.g., PVC (vinyl) sheet piles) were recommended for seepage cutoffs. Stantec geochemists as well as sheet pile manufacturers advised that the vinyl material would be resilient to hydrogen sulfide in the groundwater. The vinyl sheet piles were specified to be driven to the top of bedrock using a mandrel to facilitate installation.

According to the American Concrete Institute (ACI) [5], hydrogen sulfide can cause slow disintegration of concrete in moist, oxidizing environments where it converts to sulfurous acid. Stantec structural engineers specified that crystalline additives be included in the concrete mix to reduce permeability of the concrete. The crystalline additives were preferred to protective coatings because of the simplicity of adding the protection into the concrete mix. Other protective coatings are attached or painted on and require special care to avoid damaging the coating during construction activities.

Because the presence of hydrogen sulfide was identified, the engineering team was able to specify construction materials and concrete additives during design so that cutoff walls and concrete foundations would be more resilient to the corrosive conditions within the groundwater. Identification during design reduced the risk for potential future dam safety issues and costly repairs due to the corrosive conditions.

VI. ACKNOWLEDGMENT

The authors gratefully acknowledge the Maumee Watershed Conservancy District in advancing projects to enhance their watershed and the communities within them. Particular recognition goes to the Board of Directors and Wendy Yunker and S. C. Wilson for their contributions.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

Coal mining tailings dam closure

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ABSTRACT

The mining closure of tailings dams has gained evidence in nowadays due to the several dam failures with severe consequences to the population living downstream and to the environment. Proper closure design dam and monitoring can minimize the risk of failures. This study presents the study of case of one method to de-characterize a coal mining tailings dam, in south of Brazil. This paper demonstrating the process of the tailings dam adaptation and the monitoring needed for the final closure. Based on the knowledge of the geotechnical properties, geophysical surveys, SPT and CPTU tests, the environmental and structural constraints of the tailings dam, the project to adapt the tailings dam to a tailings landfill was developed. After the conclusion of the process of the tailing dam de-characterization a monitoring program was established based on the measurement of slope displacement, water level and water pressure at the landfill. The results show that the tailings landfill is consolidating, stable and adapting to the topography and landscaping conditions of the region.

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DAM SAFETY PALM SPRINGS
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Design of a New Riparian Conduit through an Active Embankment Using Innovative Techniques to Mitigate Risks

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ABSTRACT

The Deloraine Dam, owned by Manitoba Transportation and Infrastructure (MTI), was constructed in 1962 by the Prairie Farm Rehabilitation Administration (PFRA). The dam is located approximately 10 kilometers (6 miles) southeast of the community of Deloraine in southwestern Manitoba. The facility consists of a 300 m long earthfill embankment dam with a riparian outlet structure, and an adjacent fixed crest concrete ogee spillway that transitions to a timber chute and ends with a 9 m vertical drop to a plunge pool. The riparian outlet structure consists of a 610 mm (24 in) diameter corrugated metal pipe culvert that runs through the base of the embankment structure and outlets to a concrete outfall structure. The gate of the riparian structure has been set in the closed position for at least 10 years. Attempts to raise the gate found that it was inoperable.There are near-term plans to replace the existing spillway structure. As a first phase, the Owner has completed the detailed design for a new riparian conduit to be located through the Right Abutment of the dam. The new 1650 mm diameter riparian outlet has been designed to be capable of passing the construction design flood (CDF) of 14 m3/s. During the detailed design phase, several risks were identified including deep, open trench excavations with high groundwater levels, long-term piping risks, and open channel flow at the downstream toe of the embankment. A trenchless option to construct the new riparian outlet structure was needed to avoid adverse soil and groundwater conditions along with dam safety risks otherwise associated with an open trench excavation. This presentation presents details of the risks associated with the installation of a new riparian conduit through an active embankment dam and the innovative trenchless technologies and design methods that were considered to mitigate the risks.

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DAM SAFETY PALM SPRINGS
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How to Exercise your Emergency Action Plans to be both impactful to dam owners and your downstream communities

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ABSTRACT

Exercising your Emergency Action Plans (EAPs) is an important way to increase preparedness for dam-related hazards. By running a Tabletop Exercise (TTX), which involves simulating emergency scenarios, both dam owners and communities can better understand their roles in the event of a dam incident.When designing a TTX, it's essential to consider the different members of an exercise design team. This typically includes dam safety engineering professionals, emergency management professionals and other technical experts who may have knowledge related to possible impacts of an incident on the local community and environment. Once the exercise design team is assembled, objectives should be compiled for each participant to ensure that everyone understands their responsibilities during the exercise. These objectives might include identifying gaps in existing plans, understanding communication procedures and exploring ways to collaborate with other stakeholders.The exercise design team should also develop scenario(s) to challenge all participants. The scenarios should reflect potential risks related to a dam incident and be realistic enough so that everyone involved can gain valuable insights into areas where they need improved preparedness and increased awareness of emergency situations.Finally, the evaluation elements of the TTX are critical to ensure that it's successful. Evaluation criteria should measure how well each participant handled their responsibilities during the exercise and provide feedback on any changes or improvements needed for increased safety when an emergency situation occurs. Additionally, participants should have ample opportunities to practice responding to various emergency scenarios throughout the TTX.

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DAM SAFETY PALM SPRINGS
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Was FEMA assistance equitable after the 2015 dam failures in Columbia, SC?

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ABSTRACT

FEMA does not consider dam breach inundations when it creates its flood risk maps, potentially leaving additional flood-prone areas overlooked. FEMA has also been shown to unfairly allocate more payouts to claims filed in predominately White and wealthy communities. Furthermore, loss of life and damage costs tend to be the only indicators used for measuring a dam failure's impact, leaving demographic disparities unexplored. Acknowledging these oversights, it is necessary to determine if FEMA assistance is equitable after a dam failure disaster and ensure that those impacted and socially marginalized can recover effectively. Using the 2015 dam failure disaster in Columbia, SC as a case study, this research assessed if the distribution of FEMA claim payouts was equitable when regarding race and class. Relevant FEMA claim data for the disaster was obtained and filtered using R and ArcGIS Pro. The claim data was then categorized by census tract and linked to corresponding race and median household income data. After conducting a comparative analysis, results revealed that predominantly Black and lower-income census tracts had higher percentages of claims unpaid. This disparity indicates that FEMA's assistance was inequitable and thus could have excluded those who would need assistance the most due to social barriers. This research not only introduces an alternative way to assess the impact of dam failures, but also provides findings that support the need to consider additional and more detailed impacts that tend to be overlooked.

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DAM SAFETY PALM SPRINGS
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Sensitivity analysis of input parameters to flood characteristics for historical dam failures using DSS-WISE Lite web-based system

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ABSTRACT

Accurately predicting flood characteristics, such as flow depth, arrival time, and inundation extent, is of importance in dam failure studies. These flow characteristics are influenced by various factors, including breach geometry, breach formation time, and simulation cell size. Conducting sensitivity analysis can provide valuable insights into the significance of these input parameters on downstream flow characteristics. In this study, we utilize the DSS-WISE Lite web-based system to perform sensitivity analysis on average breach width, breach formation time, and simulation cell size for two historical dam failure events: the 2005 Taum Sauk Upper Dam failure and the 2020 Edenville Dam and Sanford Dam failure. Within specific ranges, we vary the average breach widths, breach formation times, and simulation cell sizes. The simulated results, including breach and downstream hydrographs, peak discharge, arrival time, maximum depth, and inundation extent, are compared among different scenarios and with actual observations. The outcomes of this sensitivity analysis can assist modelers in evaluating parameters for predicting dam failure in cases where the values are unknown.

1. Introduction

Dam failures can have catastrophic consequences, resulting in the loss of lives, widespread property damage, and negative environmental impacts (Altinakar et al., 2009). Several dam failure events have occurred, including the Taum Sauk Dam and the Edenville and Sanford Dam incidents. The Taum Sauk Pump Storage Powerplant, owned by Ameren UE, was constructed in the 1960s in Missouri. It consisted of an upper reservoir formed by a rock-fill dike and a lower reservoir. On December 14, 2005, the upper reservoir was overtopped, causing erosion, and breaching the embankment. Within 20 minutes, a breach approximately 656 feet wide at the top and 496 feet wide at the base formed, releasing a peak discharge of around 273,000 cubic feet per second. The water caused significant damage to Johnson's Shut-Ins State Park, Highway N, the park, campground, and surrounding areas. Field crews assessed the damage and water levels along the East Fork Black River. The Edenville and Sanford Dams, owned by Boyce Hydro, were built in the 1920s along the Tittabawassee River in central Michigan. During a three-day rain event on May 19, 2020, the eastern side of the Edenville Dam collapsed as Wixom Lake experienced a record-high water level. This led to a significant drainage of the lake into the Tittabawassee River, which then flowed into Sanford Lake. The Sanford Dam's fuse plug spillway began eroding, and by 8:20 p.m., the dam experienced an overtopping breach failure. The combined outflows from both dams and runoff inundated downstream areas, causing widespread damage to buildings, homes, roads, and infrastructure.

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Approximately 11,000 residents were successfully evacuated with no serious injuries or loss of life reported. More than 4,000 structures were reportedly impacted by the floodwaters, with estimated losses of roughly $245 million.

Accurately predicting and assessing flood characteristics resulting from dam failures are important for effective emergency preparedness, risk management, and mitigation strategies. However, during the occurrence of a failure event, the input parameters for prediction are often unknown. Therefore, sensitivity analysis plays a crucial role in evaluating the influence of input parameters on dam failure simulations. This paper aims to investigate the sensitivity of flood characteristics to average breach width, breach formation time, and simulation cell size for the failures of Taum Sauk Dam, as well as the Edenville and Sanford Dams. By varying these parameters and studying their effects, the objective is to gain an understanding of their significance and impact on simulation results.

2. Materials and methods

The failure event of the Taum Sauk Dam and the Edenville and Sanford Dams was modeled using the DSS-WISE Lite web-based system. Data sets used by the system include terrain data from USGS 3D Elevation Program (3DEP), levee data from the USACE National Levee Database (NLD), and land cover data is from the National Land Cover Database (NLCD), with Manning's for different land covers defined. The model setup for the Taum Sauk Dam failure used a reservoir-type simulation, where reservoir bathymetry is estimated (if not detected in the DEM) and the breach hydrograph is calculated according to the outflow of the reservoir through the breach whose parameters are provided by the user. The Edenville and Sanford Dam failure, characterized by the cascading failure of two dams, is currently not supported by the DSS-WISE Lite web-based version due to its restriction to breaching only one structure per simulation. To address this limitation, two approaches were employed to study the failure of these dams. The first approach utilized the DSS-WISE Lite web-based version in conjunction with a combined hydrograph technique. This technique employed a hydrograph-type simulation, where the input hydrograph was a combination of contributions from (i) the Edenville Dam failure (without breaching the Sanford Dam), (ii) the Sanford Dam failure (without breaching the Edenville Dam), (iii) spillway gate discharge, and (iv) streamflow from tributaries. The second method involved utilizing the full version of the DSS-WISE solver, which permits multiple breaches but is not publicly accessible. Within this method, a reservoir-type simulation was conducted, assuming breaches of both the Edenville and Sanford Dams, while also incorporating spillway gate discharge and streamflow from tributaries. Additional details about the setup of these DSS-WISE simulations and field case studies can be found in Pophet et al. (2023). Additionally, details about the DSS-WISE modeling system and its validation in other case studies can be found in Altinakar et al. (2012, 2017).

Table 1 provides the input parameters used for the simulations, with sensitivity analysis conducted by varying one parameter at a time while keeping other parameters the same. In the case of the Taum Sauk Dam failure, simulations were performed using different cell sizes (1 m, 3 m, 6 m, and 10 m), designated as TS_BL (baseline simulation), TS_CS3, TS_CS6, and TS_CS10, respectively. For the Edenville and Sanford Dam failure, the following scenarios were set up: ES_BL (baseline simulation), ES_CS15, and ES_CS30, representing simulations using the input parameters from Table 1 with cell sizes of 6 m, 15 m, and 30 m, respectively. ES_BW800 involved a simulation with a cell size of 6 m and an average breach width of 800 ft at Sanford Dam. As for ES_EdenBFT0 and ES_SanBFT0, both simulations had a cell size of 6 m and a breach formation time of 0 hours at Edenville and Sanford Dams, respectively. The input parameters for ES_BW800, ES_EdenBFT0, and ES_SanBFT0, apart from previously mentioned, were taken from Table 1.

Table 1: Input parameters for the Taum Sauk Upper Reservoir dam failure and the Edenville and Sanford dam failure.

Reservoir Details	Taum Sauk Upper Reservoir dam failure		Edenville and Sanford dam failure	
	Upper Reservoir	Lower Reservoir	Wixom Lake	Sanford Lake
Normal Pool Elevation (ft NAVD88)	1560	742	675.2	630.2
Normal Pool Volume (ac-ft)	2500	2880	36000	15000

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Maximum Pool Elevation (ft NAVD88)	1597	750	681.5	636.2
Maximum Pool Volume (ac-ft)	4366	5190	66200	60000
Failure Pool Elevation (ft NAVD88)	1597	736	681.5	630.2
Failure Pool Volume (ac-ft)	436	1570	66200	15000
Dam Details	**Upper Reservoir Dam**	**Lower Reservoir Dam**	**Edenville Dam**	**Sanford Dam**
Crest Elevation (ft NAVD88)	1597	750	681.5	636.2
Hydraulic Height (ft)	92	55	54	36
Structure Type	Embankment	Embankment	Embankment	Embankment
Breach Parameters				
Failure Type	Partial Breach Failure	-	Partial Breach Failure	Partial Breach Failure
Breach Invert Elevation (ft NAVD88)	1505	-	627.5	600.2
Breach Width (ft)	576	-	481	1200
Breach Formation Time (hour)	0.18	-	0.17	0.17

3. Results

This section presents the simulation results for inundation extent, arrival time, and maximum flood depth. The deviation of the simulated inundation extent from observation is quantified using a measure of fit. Furthermore, breach hydrograph and downstream flood hydrograph are plotted for different scenarios.

3.1 Taum Sauk Dam failure

The comparison between the simulated and observed inundation extents in Figure 1 (a) reveals a generally good agreement, with some deviations observed at specific locations. The overestimation at the northern side of the confluence and the eastern side of the lower reservoir may be due to limited field observations in those areas. The overestimation and underestimation downstream of the breach can be attributed to local effects that dominate the flow, such as 3D effects or large amounts of sediment transport. These effects were not considered in the simulations. Figure 1 (b) shows that as the cell size increases, the overestimation becomes more pronounced. Nevertheless, despite the overestimation near the breach location, the simulated inundation extent downstream aligns well with the observed data, as shown in Figure 1(c)-(h).

The deviation of simulated results from observations can be quantified using a measure of fit known as the F statistic, as presented by Bates & De Roo, 2000:

$$F^{(2)} = \frac{(A^{obs} \cap A^{sim})}{(A^{obs} \cup A^{sim})} \times 100$$

where, A^{obs} and A^{sim} represent the observed and simulated inundation extents, respectively. The value of F varies between 0 for a model with no overlap between observed and predicted inundation areas and 100 for a model where these areas coincide perfectly. Table 2 summarizes the F statistic comparisons, simulation times, and speedups for simulations with different cell sizes for the Taum Sauk Dam failure. For the TS_BL baseline simulation, the fit is 84.1% compared to the observation. For the other simulations, as the cell size increases, the F statistic decreases, indicating a reduction in fit compared to the observation and to the TS_BL simulation. Nonetheless, all cases still maintain F statistics above 80% when compared to the observation. The speedups are also shown, indicating the improvement in simulation times relative to the TS_BL baseline simulation. For example, the TS_CS10 simulation with a cell size of 10 m has a speedup of 66.8 times compared to the TS_BL baseline simulation with a cell size of 1 m. Overall, as the cell size increases, the simulation times decrease, but there is a trade-off with reduced accuracy as indicated by lower F statistics.

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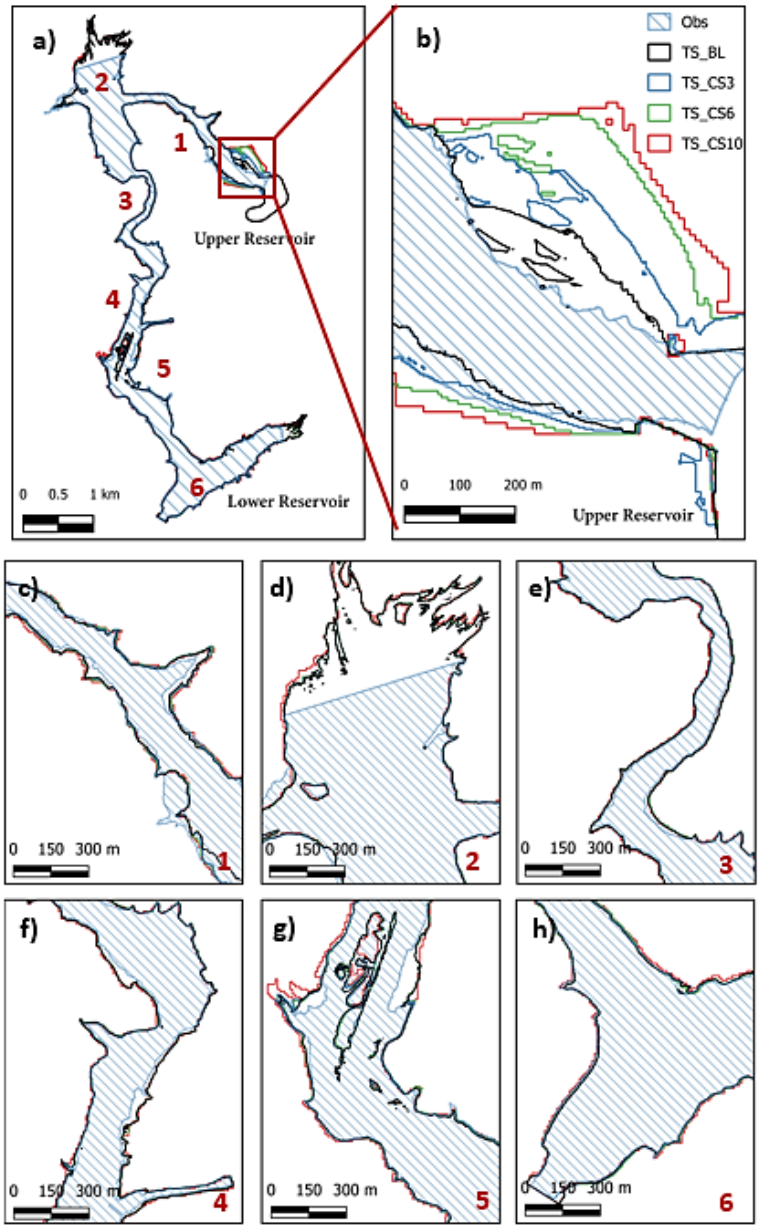


Figure 1: Comparison of simulated inundation extents for different cell sizes with observation ((Team, FERC Taum Sauk Investigation, 2006)) for the Taum Sauk Dam failure. (a) simulated and observed inundation extents for the entire simulation domain, (b) zoomed-in view downstream of the breach, and (c) – (h) zoomed-in views at downstream locations 1 to 6.

Table 2: F statistic comparisons, simulation times and speedups for simulations with different cell sizes for Taum Sauk Dam failure.

Simulation names	Cell sizes (m)	F(%) compared to the observation	F (%) compared to simulation TS_BL	Simulation times (minute)	Speedups
TS_BL	1	84.1	100.0	60.1	1
TS_CS3	3	83.5	98.8	24.2	2.5
TS_CS6	6	82.2	95.8	3.1	19.4

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TS_CS10	10	80.7	92.8	0.9	66.8

Figure 2 shows observed and simulated breach hydrographs for the Taum Sauk Dam failure with different cell sizes. The simulated breach hydrograph had a peak discharge of 272,887 cfs at 5:25 AM, closely aligned with the observed peak discharge of 273,000 cfs at 5:23 AM as reported by the Team, FERC Taum Sauk Investigation, 2006. No significant variation is observed among simulations with different cell sizes.

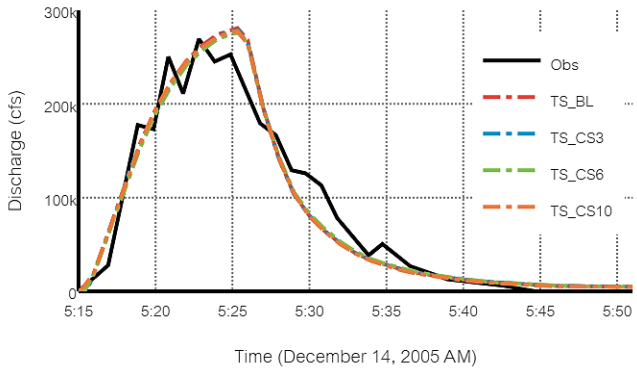


Figure 2: Observed and simulated breach hydrographs for simulation with different cell sizes for Taum Sauk Dam failure.

3.2 Edenville and Sanford Dam failure

Figure 3 demonstrates the agreement between the simulated inundation extent of the ES_BL baseline simulation and the observed extent (DFO, 2020) for the Edenville and Sanford Dam failure, covering the area from downstream of Sanford Dam to the exit at Saginaw Bay. The simulation captures the overall patterns of inundation, although some discrepancies are observed in specific locations. Table 3 presents F statistical analyses for simulations of the Edenville and Sanford Dam failure, encompassing different input parameters and cell sizes. The ES_BL baseline simulation, utilizing parameters from Table 1 with a 6m cell size, achieved an F statistic of 47.1% in comparison to observations. Despite variations in cell sizes or input parameters, the F statistics remained above 42% compared to observations, with values exceeding 90% relative to the ES_BL baseline simulation. It should be noted that the observed inundation extent was derived from satellite data, and the accuracy of flood extent mapping using satellite imagery can be influenced by various factors, including cloud cover, shadows, and land cover types. Additionally, the presence of dense vegetation can hinder the satellite's ability to detect floodwater. It is worth noting that using coarser cell sizes significantly reduces simulation time, with the ES_CS15 and ES_CS30 simulations running 13 and 155 times faster than the ES_BL baseline simulation. Since the cell sizes for the ES_BW800, ES_EdenBFT0, and ES_SanBFT0 simulations remain the same as the ES_BL baseline simulation, the simulation times were not expected to differ significantly.

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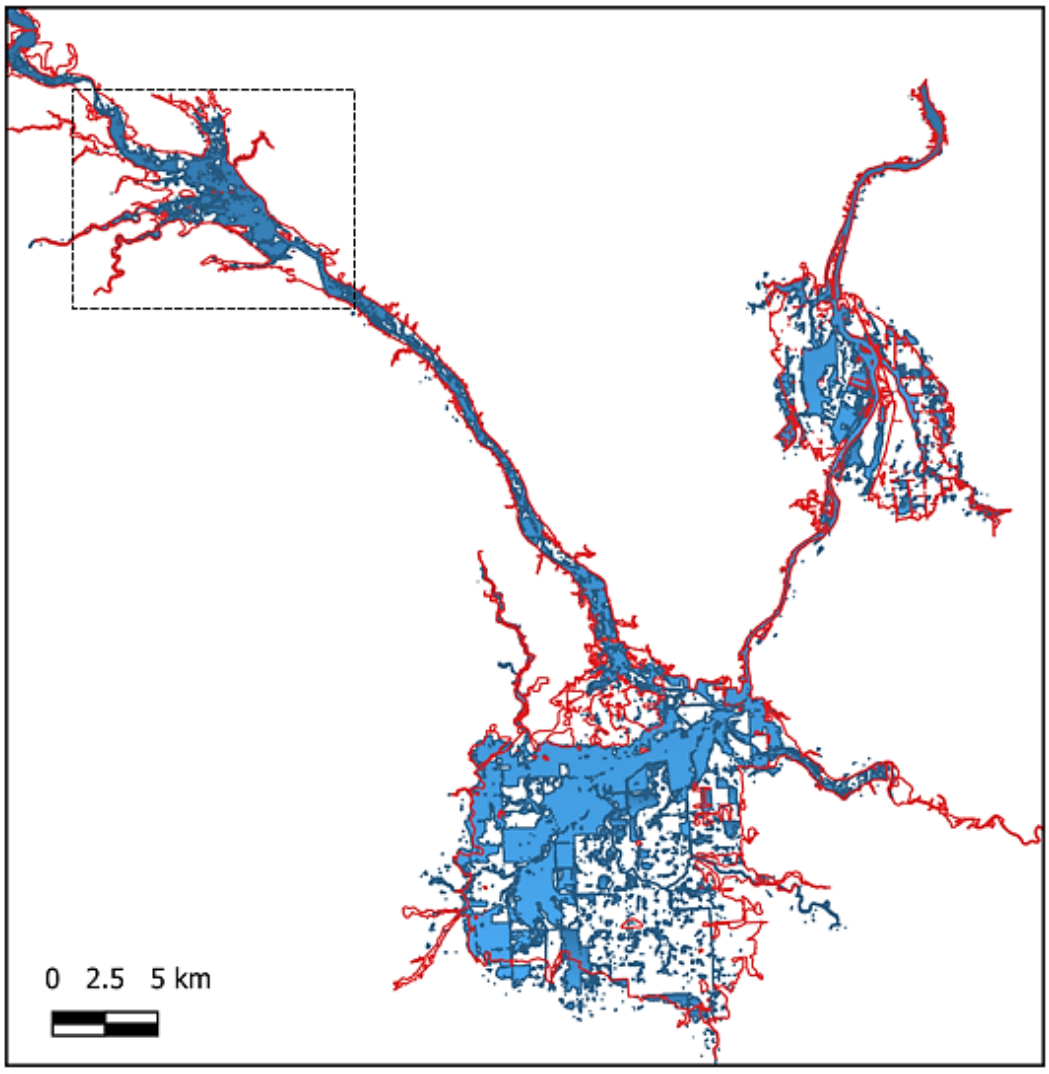


Figure 3: Comparison of simulated (red, ES_BL baseline simulation) and observed inundation (filled blue, Dartmouth Flood Observatory (DFO, 2020)) extents for the Edenville and Sanford failure.

Table 3: F statistic comparisons for simulations with different input parameters and cell sizes for the Edenville and Sanford Dam failure.

Simulation names	Scenarios		F(%) compared to the observation	F (%) compared to the simulation ES_BL
	Input parameters different from Table 1	Cell sizes (m)		
ES_BL	-	6	47.1	100.0
ES_CS15	-	15	43.0	96.9
ES_CS30	-	30	42.4	90.9
ES_BW800	Average breach width at Sanford Dam 800 ft	6	43.1	97.8
ES_EdenBFT0	Breach formation time at Edenville Dam 0 hr	6	43.1	99.9

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ES_SanBFT0	Breach formation time at Sanford Dam 0 hr	6	43.1	99.9

In Figure 4 and Figure 5, the simulated arrival times and maximum flood depths are presented for ES_BL and ES_CS30 at a location near Midland, approximately 10 miles downstream of Sanford Dam. There is no significant difference observed in maximum flood depths and arrival times between these two simulations. It should be noted that the authors were unable to obtain an accurate spatial distribution of observations for arrival times and flood depths. As a result, direct comparisons between the simulation and actual data could not be made. Additionally, no discernible differences were observed in inundation extents, arrival times, and maximum flood depths between ES_CS15, ES_BW800, ES_EdenBFT0, and ES_SanBFT0 scenarios in comparison to ES_BL baseline simulation. Therefore, these scenarios were not depicted in the figures.

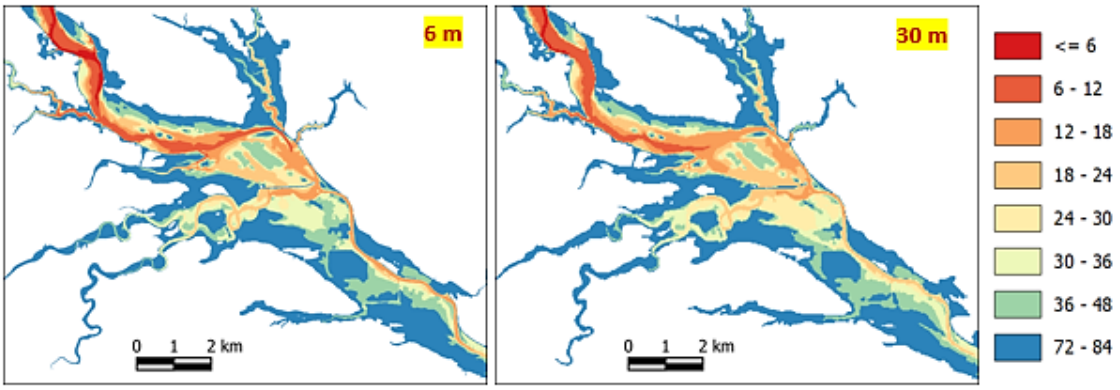


Figure 4: Arrival times in hours for simulations with 6 m (left, ES_BL baseline simulation) and 30 m (right, ES_CS30) cell sizes for Edenville and Sanford Dam failure at the location near Midland (black dashed rectangle in Figure 3), about 10 miles downstream of Sanford Dam.

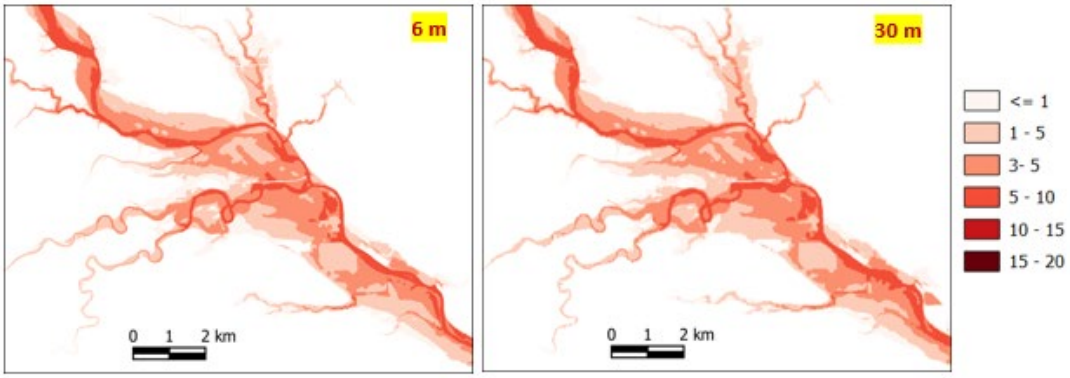


Figure 5: Maximum flood depths for simulations with 6 m (left, ES_BL baseline simulation) and 30 m (right, ES_CS30) cell sizes for Edenville and Sanford Dam failure at the location near Midland (black dashed rectangle in Figure 3), about 10 miles downstream of Sanford Dam.

Figure 6 presents the observed flood hydrograph at the USGS gage station 04156000, alongside simulated flood hydrographs for various scenarios involving different average breach widths, breach formation times, and cell sizes for the Edenville and Sanford Dam failure. The recorded peak discharge occurred on May 20th at 11:00 AM, measuring 51,800 cfs, while the simulated peak discharge was 51,583 cfs on May 20th at 07:12 AM. As anticipated, the simulated peak values are lower than the observed peaks, as these simulations do not account for the influence of river flow or inflows from the watershed. Furthermore, the simulated peak times preceded the observed peak times. Nevertheless, the hydrographs for different average breach widths, breach formation times, and cell sizes exhibit similar pattern.

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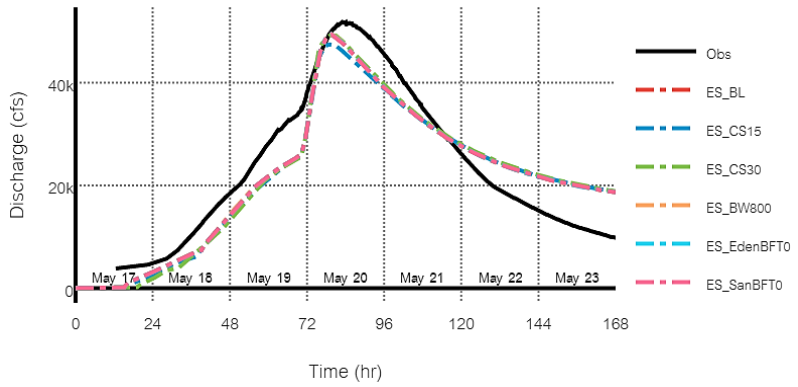


Figure 6: Observed flood hydrograph at the USGS gage station 04156000 and simulated flood hydrographs for simulations with different average breach widths, breach formation times, and cell sizes for the Edenville and Sanford Dam failure.

4. Conclusion

In conclusion, the simulations exhibit agreement with observed inundation extents for the Taum Sauk Dam failure, yielding an F statistic value of 84.1%. However, for the Edenville and Sanford Dam failure, the F statistic value is lower at 47.1%. This discrepancy may arise from uncertainties inherent in satellite-derived flood extent data. Sensitivity analysis unveiled a trend where increasing cell size reduces simulation times, yet this reduction comes at the expense of accuracy, as indicated by lower F statistics. Varying average breach width and breach formation time within the studied ranges demonstrated minimal impact when compared to the baseline simulation, which maintains an F statistic value exceeding 97.8%. In the context of a dam break emergency, where rapid simulation results are essential for response preparation and input parameters are unknown, this sensitivity study serves as a valuable resource. It offers modelers insight into the potential effects of larger cell sizes or estimated input parameters on simulation outcomes, aiding in quick decision-making and response planning.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

Digital Twins are Twice the Fun

BRIAN SCHROEDER, P.E., COLLINS ENGINEERS, INC.

Photogrammetry is the science and technology of obtaining reliable information about physical objects and the environment through the process of recording, measuring, and interpreting photographic images. The first photograph was taken in 1827, the field of photogrammetry has made significant advancements. In 1934, the American Society for Photogrammetry and Remote Sensing (ASPRS) was founded with the mission to ethically utilize active and passive sensors, photogrammetry, remote sensing, geographic information systems, and other geospatial technologies. Various Industries have embraced the use of photogrammetry and associated sciences to enhance efficiency in data collection, historic documentation, and condition assessment of assets.

Collaboration becomes more effective with the integration of advanced technologies. Just as phone calls have evolved into video calls, the traditional 2-dimensional plan sets and reports can be upgraded to interactive 3-dimensional digital twins. Rather than relying on one picture, an entire asset can be viewed in real life detail. These digital twins can be collectively presented to the entire project team or explored at the desired pace of individual team members, regardless of their location. Moreover, digital twins offer multi-faceted views and the ability to tag data to specific locations on the asset, enhancing collaboration and understanding.

The control of dimensions within digital twins is key for their usability. For instance, a digital twin of a bridge is more valuable when it accurately represents its geographic location. By linking other data sources to the twin, such as sensors or environmental information, a comprehensive view can be achieved. Dimensions of the asset itself (length, width, height, thickness) are critical for leveraging the digital twin effectively. For example, a room's digital twin can aid in optimizing space utilization or determining the compatibility of additional assets within its dimensions. Dimensional control plays a key role in the twin's performance.

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Both creating and using digital twins is a fun and enjoyable process. The initial model creation involves capturing a significant amount of data through the collection of numerous photos and data using photogrammetry or other methods such as side-scan sonar and LiDAR mapping. The more data collected, the better the quality of the resulting digital twin. Images can be obtained from any image recording device like handheld digital cameras (phones) or those carried onboard a drone. With the advancements in drone technology, images can be captured autonomously, making the process faster and more efficient to reliably create a digital twin. These images are then stitched together using 3-dimensional rendering software, to form the digital twin. The twin is then ready to accept sketches of proposed changes, point specific data, or simply for distribution to view. Additionally, subsequent initiatives are exploring artificial intelligence (AI) to detect deficiencies and compare subsequent digital twins over time.

The first aspect of the "twice the fun" experience lies in the initial model creation. The comprehensive data collection effort, involving thousands of photos capturing the entire asset from various angles, ensures the creation of a viable reality model. Each photo contributes to the value of the digital twin, making the effort invested in the data gathering and construction of the model worthwhile and enjoyable.

The second aspect of the "twice the fun" experience is utilizing the model. The digital twin allows for the visualization of the asset's condition at the time the photos were taken. Further additions, such as notes describing defects or annotations highlighting the extent of a defect, enhance the usability of the twin, or an annotation could simply point to a particular issue or nuance of the asset to better help users of the digital twin navigate the model. With the baseline digital twin established, subsequent data collections enable the creation of additional digital twins, which can be compared to identify changes in the asset or its surroundings. The second twin offers the opportunity for another moment-in-time capture and facilitates a comparative analysis with the first twin.

Just as photography has evolved, the creation and utilization of digital twins have undergone significant advancements. Understanding the equipment required for creating reality models, visualizing the quality of twins, and appreciating the historical context of photogrammetry can provide insights into how digital twins can be leveraged throughout the lifecycle of physical assets, from conception to design, construction, and maintenance. Embracing digital twins offers an engaging and productive experience for participants involved in various industries.

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Estimated Rates of Failure for Dams in the United States

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ABSTRACT

Annual rate of failure for dams in the United States were estimated by examining the National Inventory of Dams and the Worldwide Historical Dam Failures Database. Based on information from these databases, there has been a total of 5,628,516 dam-years, 2,694 dam failures, and thus 0.00048 dam failures per dam-year. Concrete and earthfill dams both have less than 0.0005 failures per dam-year; masonry and rockfill dams have more than 0.001 failures per dam-year and timber dams have more than 0.0035 failures per dam-year. For all dam types, failures per dam-year are greatest in the first five years after construction. For all types of dams, except for earthfill and rockfill, data shows a steady decline in failure rates as dams age. However, earthfill and rockfill dams reach the lowest rates of failures per dam-year when they are between 20-50 years old with an increase in failure rate, especially for earthfill dams, after 50 years after construction. Results from this study are predicated by the available data, which likely does not include information on all failed dams. In addition, these databases do not include certain types of information, e.g., construction activities and contractor performance.

I.INTRODUCTION

Dam safety programs in the United States, particularly those for hydropower dams regulated by the Federal Energy Regulatory Commission, are transitioning to a risk-informed approach to dam safety assessments. This transition has been motivated in part by expert recommendations following reviews of recent high-profile dam safety incidents, most notably the service and auxiliary spillway failures at Oroville Dam in 2017 [4]. Risk-informed dam safety assessments are based on the explicit goal of reducing the societal risk to *As Low As Reasonably Practicable* (ALARP) with the implicit understanding that some amount of risk must be tolerated. A fundamental component of risk-informed dam safety assessments is estimation of annual probability of failure for various failures modes identified in a potential failure modes analysis. Such estimates are difficult to make for complex dam systems, typically resulting in large uncertainties, and there is little data to validate the estimates. Therefore, historical rates of failures for dams of various types, construction eras, and ages may be used as a component to complement risk-informed dam safety processes, which seek to estimate annual probability of failure for dams. Previous studies have estimated the rate of failures of dams e.g., Foster et al [3], and Ingles [6]. Engemoen [2] completed a similar study which investigated the historical rates of failures for Bureau of Reclamation embankment dams and dikes and considered the effect of dam age on failure rates and failure type. This study attempts to supplement such work by providing additional estimates, which consider the effect of dam type, construction era, and dam age.

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II.BACKGROUND

We use two datasets for this analysis:

- National Inventory of Dams [7]
- Worldwide Historical Dam Failures Database [1]

The National Inventory of Dams contains data for over 90,000 dams in the United States; the Worldwide Historical Dam Failures Database contains data for 3,861 dam failures from around the world but is filtered to only include dam failures in the United States, resulting in 2,543 recorded dam failures. The focus of this paper is on permanent, engineered water dams, so all data for landslide dams, test dams, cofferdams, and tailings dams are removed.

These datasets contain measures of various dam characteristics such as dam height, dam length, reservoir volume, etc. The Worldwide Historical Dam Failures Database also contains information on failure consequences: e.g., fatalities and economic impact; however, this information is incomplete, therefore, it is not considered for this study except to complete a catalogue correction. Similarly, we do not attempt to match dam failures from the Worldwide Historical Dam Failures Database to dams in the National Inventory of Dams. Instead, this study uses categorical data attributes, which were available for each dam and each dam failure: dam type, construction era, and dam age.

The failures database includes recorded instances of dam failures from 1848 to present. These failures vary significantly in their consequences. A variety of dam safety incidents are included in the failures database: disasters, internal erosion, overtopping, quality problems, and only a subset of these resulted in fatalities, N. It should be noted that in this study all types of failures are given equal weight regardless of their severity. This approach is generally consistent with that of the Federal Energy Regulatory Commission in that it includes the recorded uncontrolled release. However, for risk-informed analyses, catastrophic failure is the primary consideration, and thus the definition of failure differs from that in this study because there is an opportunity for intervention to prevent catastrophic failure once uncontrolled release is observed.

III. DATA COMPLETENESS, CATALOGUE CORRECTION, AND IMPUTATION

This study does not differentiate between these failures based on their economic or human costs. However, it is likely that some older failures that resulted in no fatalities (N = 0) were not recorded. This is analogous to the catalogue completeness issue in earthquake ground motion databases for which a magnitude of completeness, M_c, is used to express the minimum magnitude for which all earthquakes are recorded [8]. In this study, we adopt a similar approach and assume a fatality of completeness, N_c, where the failures database is assumed complete for all failures which resulted in at least one fatality. We experimented with using various fatality thresholds for which the catalogue is assumed complete, e.g., N_c = 1, 10, and 100, and did not find significant sensitivity. The data in this study is limited, therefore we did not explore geographical variation of catalogue completeness and assumed that geographical effects are negligible.

The assumption of a constant background rate, as is done for seismicity, is not valid for dam failures due to changes in dam safety and dam populations over time. We assumed a self-similarity in the consequence profile (F-N plot shape) over time, which is analogous to a constant Gutenberg-Richter curve, and that the ratio of failures which cause fatalities (i.e., lethality) is constant over time. Therefore, we used the rate of dam failures in 2022 determined from a first-order ordinary least squares regression to back-project a corrected failure rate curve based on the trend of failures with lives lost, and then compute a corresponding transfer function, as illustrated in Figure 1. The transfer function, which is dependent on failure year, is applied to all failures in the failures database to scale up the number of failures to current levels of failure reporting and therefore address the catalogue completeness issue. The effect of this catalogue correction was investigated; it was found to increase the total number of failures by 6%, from 2,543 to 2,694 failures.

In addition to the catalogue completeness correction, missing categorical data in the two datasets is imputed based on the ratios of available data. This is done to mitigate bias in the analysis due to missing data when the effects of multiple categorical characteristics are examined.

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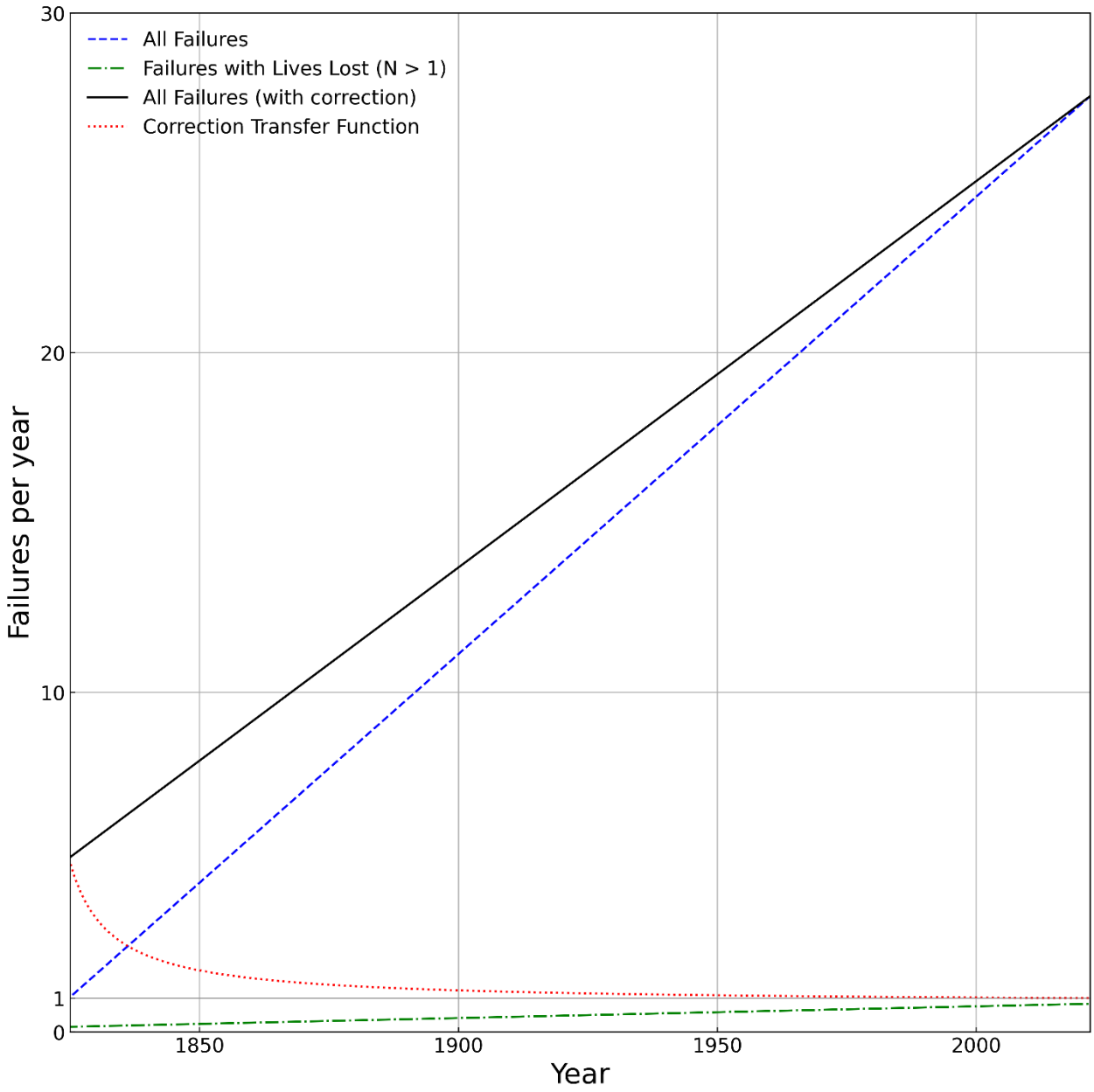


Figure 1. First-order ordinary least square regression showing dam failures per year as a function of year for all failures (uncorrected), failures resulting in fatalities (N > 1), the corrected back projection for all failures based on the N > 1 failure curve, and the resulting transfer function used to correct the catalogue for missing historical failures.

IV.METHODS

Both datasets contain categorical labels of the type of dams, which were harmonized into the following dam types: concrete (including gravity, roller-compacted, buttress, multiple-arch, and arch), earthfill, masonry, rockfill, timber, and other (including stone, steel, and others). We consider five construction eras, roughly delineated by transitions in dam design and construction practice: Practice-Based (pre-1920), Standards-Based (1920-1950), Improved Standards-Based (1950-1970), Criteria-Based (1970-1990), and Risk-Informed (post-1990) [5]. Dam ages are grouped into stages of dam life: 0-5 years, 5-10 years, 10-20 years, 20-50 years, 50-100 years, and 100-200 years. Dams in all 50 states as well as Guam and Puerto Rico are included in the analysis.

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We group each dataset into categories and sub-categories. From the National Inventory of Dams, we count the cumulative number of dam-years in each category, where dam-years are the product of the number of dams and their respective ages (i.e., 10 dams constructed in 1923 would constitute 1,000 dam-years by 2023). From the Worldwide Historical Dam Failures Database, we count the number of dam failures in each category, and we then compute the historical rates of failures in units of failures per dam-year in each category.

V.DATA SUMMARY

In aggregate, there has been a total of 5,628,516 dam-years, 2,694 dam failures, and thus 0.00048 dam failures per dam-year in the history of dams in the United States (once catalogue completeness has been adjusted for and missing data has been imputed). Table I through Table III provide summaries of the combined dam inventory and dam failures dataset partitioned by dam type, construction era, and dam age; the totals from each table differ slightly due to the application of the catalogue correction, imputation of missing data, and rounding errors. For reference, the number of failures without the catalogue correction applied are provided in parentheses.

As shown in Table I, historical failure rates range between 0.00031 and 0.00392 failures per dam-year when aggregated by the type of dam. Dam types with the highest failure rates (i.e., timber, rockfill, and masonry) appear to be those which were common in the earliest construction eras, which tended to have higher rates of failures as shown in Table II. A portion of the higher relative failure rates for early-era timber, rockfill and masonry dams appears attributable to both the era of construction, and to these dam types being inherently more failure prone.

TABLE I

Summary of dam failures by dam types, failures without the catalogue correction applied are shown in parentheses

Dam Type	**Dam-years**	**Failures**	**Failures per dam-year**
Concrete	530,391	187 (174)	0.00035
Earthfill	4,857,697	2,196 (2,086)	0.00045
Masonry	91,994	115 (103)	0.00126
Rockfill	77,274	111 (104)	0.00144
Timber	16,163	63 (57)	0.00392
Other	53,765	16 (16)	0.00031

TABLE II

Summary of dam failures by construction era, failures without the catalogue correction applied are shown in parentheses

Construction Era	**Dam-years**	**Failures**	**Failures per dam-year**
Practice-Based (pre-1920)	1,018,081	1,223 (1,065)	0.00120
Standards-Based (1920-1950)	1,166,655	508 (476)	0.00044
Improved Standards-Based (1950-1970)	2,291,867	440 (423)	0.00019
Criteria-Based (1970-1990)	916,653	254 (246)	0.00028
Risk-Informed (post-1990)	235,258	337 (331)	0.00144

The high number of failures per dam-year for dams constructed in the Risk-Informed (post-1990) era (Table II) is initially surprising. We believe this finding reflects changes in reporting through the history of dams in the United States, with much more diligent documentation of dam safety failures in the modern eras relative to eras prior to 1970.

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As shown in Table III, the dams demonstrate the highest rates of failures when they are in the earliest stages of their life cycles, especially in the first 5 years after initial filling of the reservoir. The rate of failures steadily declines up until 20-50 years of age with a very slight increase in failure rate for very old dams (i.e., 50-100 years and 100-200 years). As shown in Figure 2, the slight increase in failure rate for old dams does not appear to occur for all dam types; only rockfill and earthfill dams demonstrate increased failure rates while all other dam types exhibit steadily declining rates of failures as they age beyond 50 years. Concrete dams appear to age particularly well.

TABLE III

Summary of dam failures by dam age, failures without the catalogue correction applied are shown in parentheses

Dam Age	Dam-years	Failures	Failures per dam-year
0-5 years	452,584	829 (756)	0.00183
5-10 years	451,014	310 (282)	0.00069
10-20 years	886,601	316 (286)	0.00036
20-50 years	2,327,418	613 (559)	0.00026
50-100 years	1,286,753	548 (525)	0.00043
100-200 years	224,142	136 (132)	0.00061

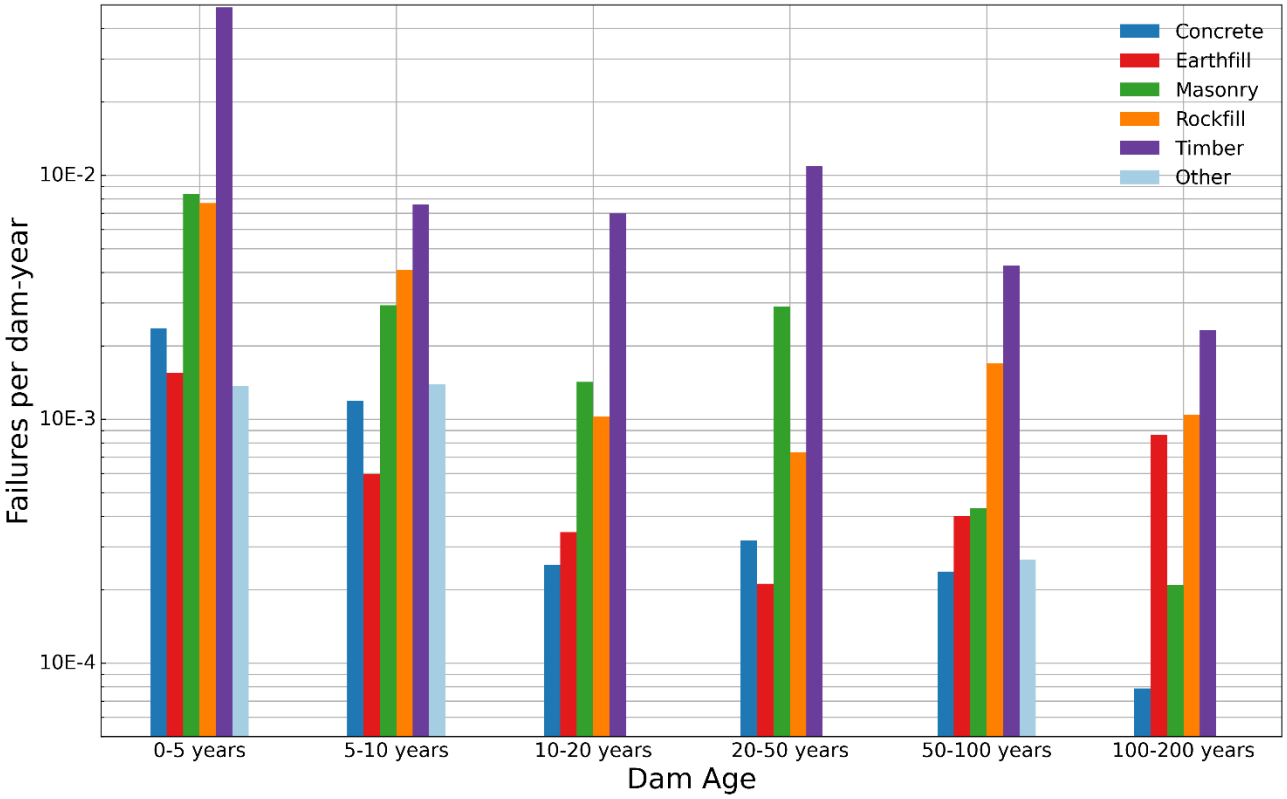


Figure 2. Failures per dam-year grouped by dam age for various types of dams.

The decline in failure rates as dams age appears to be consistent for all construction eras, as shown in Figure 3, with very significant reductions in failure rates occurring for dams constructed in modern eras between 1970-1990 and post-1990. High failure rates observed for dams constructed post-1990 (Table II and Figure 3) are initially surprising; however, the higher failure rate may be attributed to more rigorous reporting in recent years.

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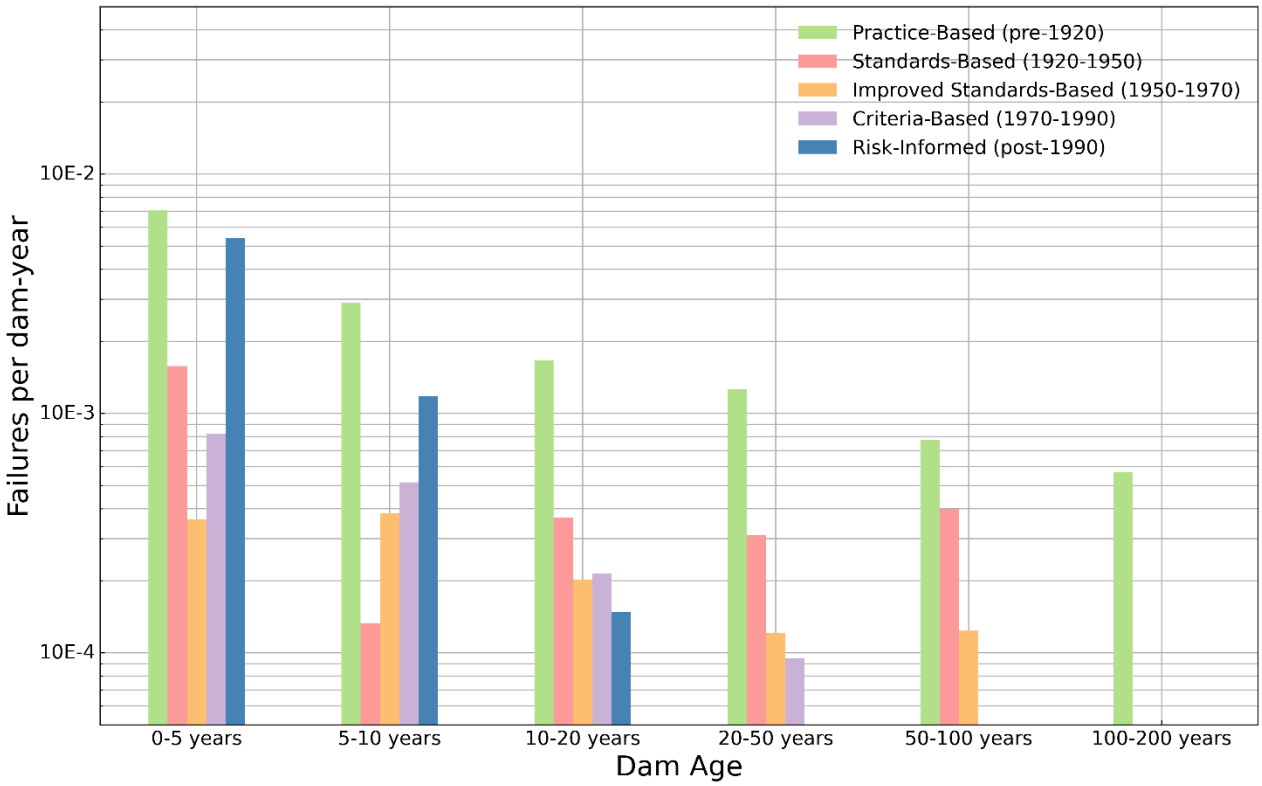


Figure 3. Failures per dam-year grouped by dam age for various construction eras.

Figure 4 and Figure 5 show the total failures, failures per dam-year, and total dam-years in each dam type-construction era and dam type-dam age subcategory, respectively. Masonry, rockfill, and timber dams have demonstrated the highest historical rates of failures; however, most historical failures are earthfill dams. When partitioned by dam type and construction era, the subgroups have a median of 0.00061 failures per dam-year with a standard deviation of 0.00576. When partitioned by dam type and dam age, the subgroups have a median of 0.00112 failures per dam-year with a standard deviation of 0.00814.

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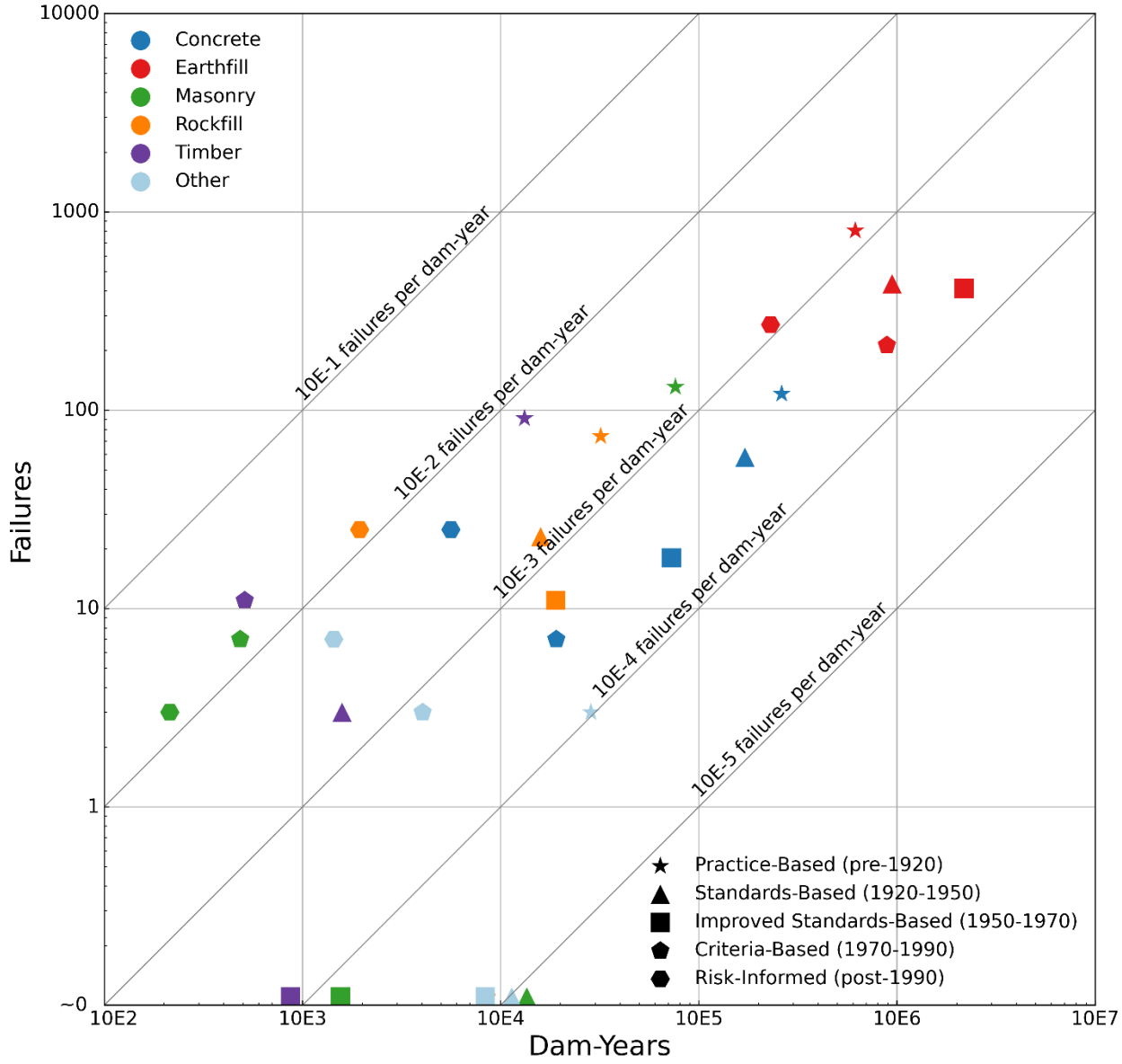


Figure 4. Failures, failures per dam-year, and dam-years for various dam types and construction eras.

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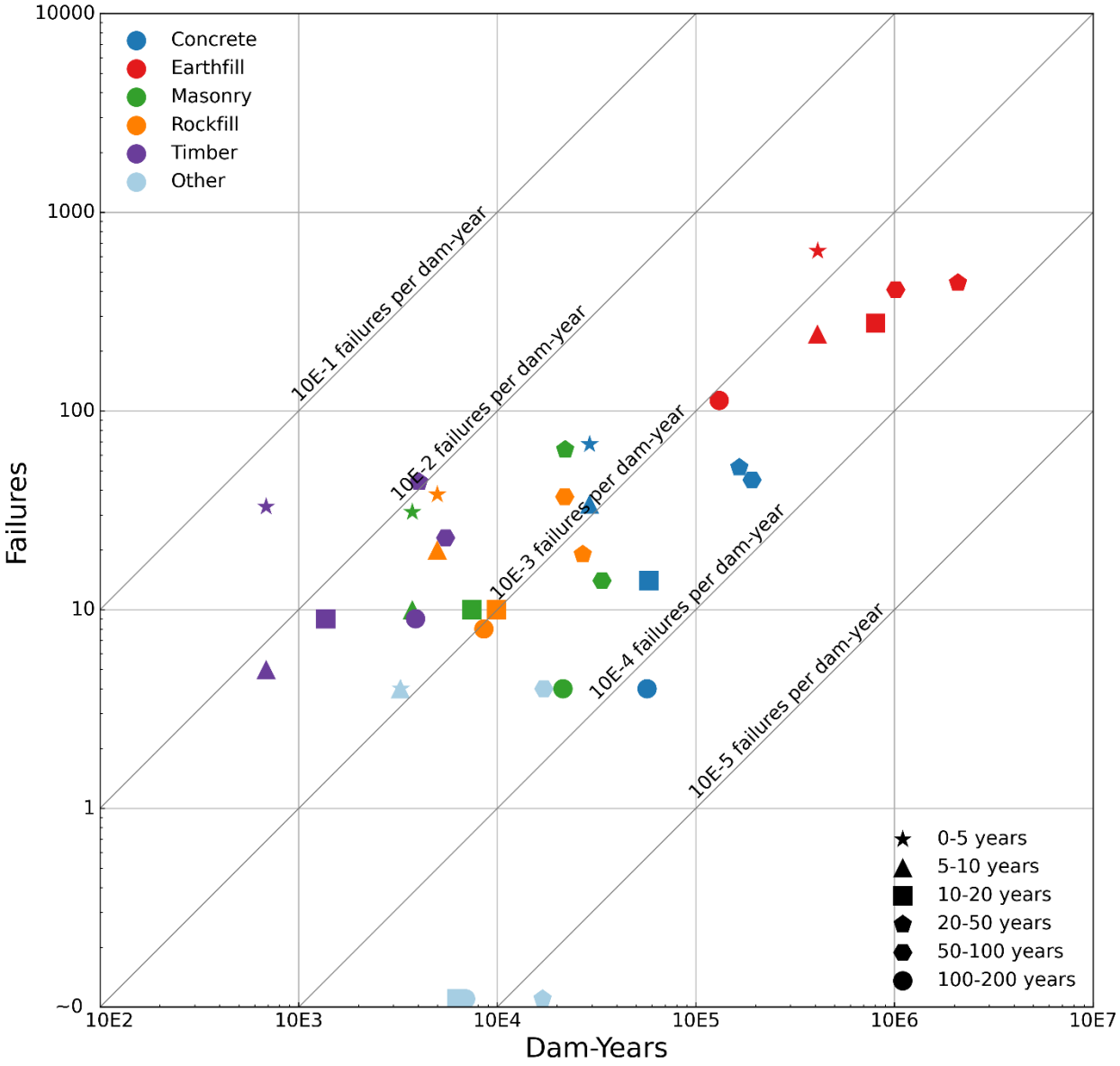


Figure 5. Failures, failures per dam-year, and dam-years for various dam types and dam ages.

VI.REGRESSION ANALYSIS

Dummy variables, with values of either 0 or 1, were used to replace each of the categorical variables for dam type, construction era, and dam age. Z-1 dummy variables were required for each; all dummy variables were set to 0 to indicate the Zth category. Ordinary least squares linear regression analysis was done in log-space with each subcategory constituting a single data point in the regression. Two different versions of this regression analysis were done, including different independent categorical variables: (i) considering dam type, construction era, and dam age as shown in Equation 1 and Table IV; and (ii) considering dam type and dam age only, as shown in Equation 2 and Table V.

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$$Annual\ probability\ of\ Failure = e^{a+b+c+y_0} \quad (1)$$

TABLE IV

Coefficients and standard errors for the ordinary least squares linear regression including the effects of dam type, construction era, and dam age.

Dam Type Parameter, a	**Value**	**Standard error**
Concrete	-0.73	0.61
Earthfill	-1.16	0.59
Masonry	0.73	0.63
Rockfill	0.82	0.62
Timber	2.14	0.65
Other	0	0.62
Construction Era Parameter, b	**Value**	**Standard error**
Practice-Based (pre-1920)	-0.82	0.37
Standards-Based (1920-1950)	-1.28	0.44
Improved Standards-Based (1950-1970)	-1.67	0.47
Criteria-Based (1970-1990)	-0.41	0.44
Risk-Informed (post-1990)	0	0.43
Dam Age Parameter, c	**Value**	**Standard error**
0-5 years	3.09	0.51
5-10 years	2.08	0.51
10-20 years	1.31	0.52
20-50 years	1.35	0.51
50-100 years	0.97	0.52
100-200 years	0	0.51
Constant, y_0	**Value**	**Standard error**
All dams	-8.76	0.76

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$$Annual\ probability\ of\ Failure = e^{a+c+y_0} \quad (2)$$

TABLE V

Coefficients and standard errors for the ordinary least squares linear regression including the effects of dam type and dam age only.

Dam Type Parameter, a	**Value**	**Standard error**
Concrete	-1.16	0.65
Earthfill	-1.70	0.63
Masonry	0.48	0.69
Rockfill	0.41	0.67
Timber	1.79	0.70
Other	0	0.67
Dam Age Parameter, c	**Value**	**Standard error**
0-5 years	3.30	0.53
5-10 years	2.15	0.53
10-20 years	1.47	0.55
20-50 years	1.21	0.53
50-100 years	0.74	0.56
100-200 years	0	0.53
Constant, y_0	**Value**	**Standard error**
All dams	-9.19	0.76

The results of the regression analysis indicate that dam age is the most important determinant of annual rate of failure, with generally declining rates of failure (in aggregate) as dams age. Earthfill and rockfill dams exhibit increased rates of failure for increased dam age; however, these dam-type specific effects were not handled by the ordinary least square regression. Timber dams appear to have high associated rates of failure, while concrete and earthfill dams exhibit the lowest rates of failure. Rockfill, masonry, and other types of dams have intermediate rates of failure.

The regression results for construction era are counterintuitive (Equation 1; Table IV), but are consistent with earlier observations for Table II, and show an apparently increased annual rate of failure for dams constructed after 1970, especially for dams constructed after 1990. We believe this is due to two main factors: (i) relatively diligent reporting in modern eras, especially post-1990; and (ii) records of historical dam failures being lost to history, which indicates that the catalogue completeness correction may not have fully addressed reporting biases throughout the history of dams in the United States. It is this finding that motivated the completion of a second ordinary least squares linear regression with the construction era parameter, b, removed (Equation 2; Table V). We recommend considering estimations from both regression equations. It should be noted that all parameters must be included for each equation, i.e., a, b, c, and y_0 for Equation 1 and a, c, and y_0 for Equation 2 because omission is equivalent to a value of zero which corresponds to the Zth category.

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VII.LIMITATIONS

The analysis approach we use has several significant limitations; therefore, it is important that the historical rates of failures which we compute be interpreted as only rough order-of-magnitude approximations. Despite the catalogue correction and data imputation, the main limitation in this study is completeness, or lack thereof, of the two datasets considered. The catalogue completeness correction only addresses missing failures and does not address the possibility of missing dam inventory. It assumes a constant F-N curve throughout the history of dams in the United States and that the ratio of failures with fatalities to all failures has been constant over time. This assumption is dubious; modern dam inspection and reporting practices differ significantly from historical norms, and it is likely the portion of recorded failures which resulted in fatalities has decreased over time. We do not account for changes in reporting practices over time.

A further limitation is that no distinction between the types, causes, and consequences of dam failures is made in this study, except in determining the correction for catalogue completeness.

VIII.CONCLUSIONS AND RECOMMENDATIONS

We analyze two datasets containing data on dams and dam failures in the United States to calculate the historical rates of failures and make inferences on the estimated annual probability of failure for dams of various construction types, construction eras, and dam ages. We compute that there has been a total of 5,628,516 dam-years, 2,694 dam failures, and thus 0.00048 dam failures per dam-year in the history of dams in the United States (once a catalogue completeness correction is applied and incomplete data has been imputed).

Examination of variation in the rates of historical dam failures in the United States reveals various insights. There are significant differences in the historical rate of failures for different types of dams; timber, masonry, and rockfill dams have very high rates of failures while concrete and earthfill dams fare much better. The most risk-prone stage in a dam's life is during the initial 5 years, which is followed by a steady decline in rates of failure for all dam types except for earthfill and rockfill dams, which demonstrate a resurgence in failure rates as they age beyond 50 years. Dams constructed in more modern eras demonstrate approximately an order of magnitude reduction in rates of failures compared with those constructed prior to 1920. High rates of failures are observed for dams constructed after 1990, however this may be attributed to higher reporting for such dams during their most risk prone years rather than a decline in the state of the dam engineering practice.

This study does not differentiate between the types, causes, and consequences of failures in the Worldwide Historical Dam Failures Database, i.e., every failure is treated the same although their consequences may vary greatly. Future work could further investigate the variation of these failures and the consequences of failure by producing F-N curves or through other methods.

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DAM SAFETY PALM SPRINGS
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3D Detailing for Hydraulic Structures: Capabilities and Deficiencies

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ABSTRACT

Starting with hand drafting and evolving with the increased use of computers and computer aided design and drafting (CADD) in the early 1980s, production of construction drawings for hydraulic structures has typically been created as two-dimensional (2D) representations of three-dimensional (3D) objects. However, for dams, spillways, gates, and other hydraulic structures that may have complex geometries, 2D CADD has its limitations. Since the late 2000s, building information modeling (BIM) has become heavily used. BIM programs are tailored towards efficient modeling and development of construction drawings for building structures. 2D CADD models can be difficult to produce complex structures. Designs often progress rapidly throughout a project. Changes, especially late in the design or construction process, can lead to redrawing elements numerous times on multiple different sheets in 2D CADD. 3D modeling can better illustrate more complex or irregular geometries while also improving production of less complex structures. Another benefit of 3D modeling is updating an object one time can apply that change to all occurrences in a drawing set. Modeling structures in 3D accurately improves visualization and communication with owners, reviewers, and contractors. BIM is a great tool for modeling, visualization, and it creates quality details, but it does have its challenges. 3D object-based modeling changes the workflow for drawing production. Producing detail views such as sections and isometrics may only require a few mouse clicks, but annotation poses challenges not observed in 2D CADD. Some issues have workarounds within the software while others do not, which could result in increased production time or adjustments to one's drafting standards.Presents the capabilities and challenges of producing construction documents using BIM software, based on examples created using Revit software that include a labyrinth weir, converging chute, stilling basin, and other structures. Lessons learned for addressing design changes, complex geometries, object modeling, annotation issues are presented for the betterment of the dams industry. Better and more efficient designs can lead to safer dams, better clarification of design intent, and overall higher productivity during design and construction. This paper will benefit engineers, CADD technicians, contractors, owners who operate and maintain their dams, and regulators who review design drawings, whether produced in 2D or 3D, and presents selection considerations. Although this paper focuses on more commonly utilized aspects of 3D versus 2D modeling, capabilities of up to seven dimensions are summarized, adding the dimensions of time, money, sustainability, and facility management.

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Building a Digital Defect Timeline for Concrete Dams and Spillways – A Pathway to Predictive Monitoring for Dam Safety Engineers

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ABSTRACT

This study presents an innovative method for concrete dam health monitoring that utilizes a defect detection and quantification software coupled with photogrammetry to create mm-scale digital defect baselines of a critical concrete dam and spillway. The software utilizes optical imagery, thermal imagery, and acoustic sounding points from a novel Unmanned Aerial Vehicle (UAV)-based data collection technique and Machine Learning based image processing to detect and quantify surface and subsurface defects such as delamination, voids, and cracking. In this case study, Niricson partners with a US dam owner to collect data on a run-of-river concrete dam to construct a high-resolution digital baseline model with mm-scale surficial defect maps of exposed concrete. The data processing resulted in a 3D model for the structure along with quantified defect maps at 1mm pixel resolution. Cracks, spalls, delamination along with efflorescence are viewed and analyzed in a georeferenced GIS environment. Quantified defects at high resolution allow for advanced analytics such as spatial density mapping, analysis of defect spatial correlations, along with change detection through time. Results of these analyses provide advanced insight for inspection, monitoring, repair and maintenance decision making with the opportunity to use defect growth trends for predictive modeling. Furthermore, the condition-based life cycle assessment on a timescale allows owners to make cost-efficient business decisions to plan and execute the maintenance repair schedule across all assets, using the risk modelling methods to extend the lifespan of the structure rather than replacing the entire structure. This approach grants an accurate, date-driven environment for building predictive deterioration models using the historical performance which can be utilized for complete asset management.

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Using the Simplified Stochastic Event Flood Modeling Approach to Support the Risk Informed Decision Making Method to Evaluate Dam Safety

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ABSTRACT

One of the objectives and supporting strategies in FERC's Strategic Plan for fiscal years 2014-2018 is to minimize risk to the public by using Risk-Informed Decision-Making (RIDM) for evaluating dam safety in parallel to traditional dam safety methods. Resulting risk estimates can be used, along with standards-based analyses, to decide if dam safety investments are justified. Consumers Energy Company (CEC) identified a concern at their Alcona Dam in Michigan regarding potential erosion of the unlined, earthen auxiliary spillway, and the potential subsequent failure of the dam during flood events more frequent than the inflow design flood (the PMF). However, given the possible consequences downstream, the estimated dam fragility, and the proposed and completed risk reduction measures, the risk may be low enough such that modifications to the auxiliary spillway are not warranted. Therefore, in 2017 CEC began a RIDM study of the Alcona Dam auxiliary spillway for submission to FERC.RIDM requires a set of hydrologic hazard curves (HHCs) to estimate the overall risk. A Simplified Stochastic Event Flood Modeling (SSEFM) approach was used to develop the HHCs for Alcona Dam. The SSEFM method is a compromise between a purely deterministic approach which tends to be conservative and a fully stochastic, Monte-Carlo approach. The resulting HHCs are estimates of peak inflow rates for a range of annual exceedance probabilities (AEPs) from 0.01 to less than 1x10 7 for both cool-season (rain on snow) and warm-season (rain only) events. The SSEFM approach is the cornerstone of this RIDM study, allowing all other aspects of the study to relate important loading characteristics (peak water level, hydrostatic pressure, auxiliary spillway flow duration, etc.) to AEPs and therefore, a proper estimate of the risk. We will present the steps of the approach, data sources, the results of the SSEFM, subsequent RIDM steps using these results, and the lessons learned throughout the process.

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Informed Decision Making with Automated Instrumentation Data Reports - A Case History in Dam Safety Monitoring

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ABSTRACT

A slope failure below a series of recently constructed and historical retaining walls threatened the construction, schedule, and budget of the Santa Anita Dam modification project in Los Angeles, California. The series of retaining walls above the slope failure supports the only ingress and egress for dam access and modification construction. To restore the dam modification construction, the project team designed and deployed a real-time passive monitoring system with scheduled and on-demand instrumentation data reports. The real-time instrumentation and automated data reports provided the confidence and assurance needed to facilitate the completion of the Santa Anita Dam modification project. The monitoring methods used at Santa Anita were subsequently implemented at Big Tujunga dam to replace the existing obsolete Dams Automated Data Acquisition System (DADAS) and similarly improve data access through automated reporting of instrumentation data.

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Assessing Dam Risk in Iowa: An Analysis of the Iowa Department of Natural Resources Dam Inventory

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ABSTRACT

Dams play a vital role in protecting communities and infrastructure from the impacts of flooding, but they also pose potential risks if they fail. In this study, we used data from the Iowa Online Dam Inventory to assess the risk of dam failure in the state of Iowa. We analyzed the relationships between hazard classification, age, and condition of dams, and we explored how these variables influence the risk of a dam failure.To assess the risk of dam failure, we used K-means clustering to group dams into different categories based on their hazard classification, age, and condition. We found that a five-cluster solution provided the best balance between simplicity and fit. We then used statistical tests, such as ANOVA and chi-squared tests, to assess the relationships between the variables and the clusters.Our results suggest that older dams and those in poor condition are more likely to fail, and that dams with higher hazard classifications are associated with greater potential consequences in the event of a failure. These findings highlight the importance of ongoing maintenance and inspection of dams to ensure their safety and reliability. Our study has important implications for dam safety and risk management in Iowa, and it suggests that more resources should be allocated to the maintenance and inspection of older dams and those in poor condition.

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Numerical Evaluation of Anti-Liquefaction Performance of Deep Soil Mixing Method under Embankment Dams

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ABSTRACT

Liquefaction-induced failure in structures like embankment dams is recognized as one of the most destructive outcomes resulting from earthquakes. When the foundation experiences liquefaction, excess pore water pressure (EPWP) is generated in the liquefied area and spreads within and underlying the structures. Addressing this issue requires the development of appropriate measures to counteract liquefaction. This numerical study examined the effectiveness of the deep soil mixing (DSM) method beneath embankment dams using Flac3D. The model results have been compared to the centrifuge experiments conducted as part of the VELACS (VErification of Liquefaction Analysis by Centrifuge Studies) project to validate the model. Based on the validated model, the anti-liquefaction performance of DSM columns installed beneath embankment dams in reducing EPWP was assessed. The findings indicate that DSM columns can effectively mitigate the detrimental effects of liquefaction beneath embankment dams by considering proper column diameter and a spacing-to-diameter ratio. These results have practical implications in engineering applications and offer valuable insights into the anti-liquefaction capabilities of the DSM method, specifically when applied to embankment dams.

I.INTRODUCTION

The United States relies on over 75,000 dams for crucial purposes such as water management [1]. The failure of a dam can have far-reaching consequences, impacting the lives of thousands of people and incurring significant financial costs. Consequently, ensuring the safety of these structures is of paramount importance. Geo-structures like embankment dams, river dikes, and highway embankments have experienced substantial damage in past major earthquakes [1-3]. Extensive studies have shown that the most severe damages occur when the underlying saturated granular soils undergo liquefaction, leading to embankment cracking, settlement, lateral spreading, and slumping. Notable failures in U.S. Dam Engineering history include the Sheffield Dam failure during the 1925 Santa Barbara earthquake and the Lower San Fernando Dam failure during the February 1971 earthquake [1–2]. Therefore, it is crucial to thoroughly investigate the potential impact of foundation liquefaction on earth embankments. Developing a deeper understanding of the dynamic response and deformations of embankment dams due to liquefaction would greatly enhance our ability to assess liquefaction risks, determine the need for mitigation measures, and design more effective and cost-efficient remedial procedures. These advancements could save hundreds of millions of dollars by preventing or mitigating dam failures.

Several techniques have been developed and implemented to address the remediation of liquefiable soils beneath embankment dams. These techniques include densification [1], the use of stone columns, and grouting. Among these methods, deep soil mixing (DSM) has emerged as a relatively new approach for improving the seismic stability of embankment dams. DSM involves the in-situ mechanical cutting and mixing of soil combined with low-pressure injection of cement slurry, resulting in the creation of uniform-diameter columns that match the dimensions of the mixing auger [4]. The effectiveness of DSM in mitigating liquefaction has been demonstrated at various geotechnical sites. While the reconstruction of Jackson Lake Dam in Wyoming during the late 1980s marked a significant application of DSM for dam construction in the United States, its specific application for mitigating

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liquefaction under embankment dams has not received sufficient attention [4]. This research aims to address this gap. In this paper, the effectiveness of the DSM method in mitigating liquefaction under embankment dams is evaluated numerically in FLAC^{3D}. This study focuses on the application of group DSM columns beneath embankment dams and assesses their performance in terms of excess pore water pressure (EPWP) generation, a key indicator of liquefaction.

II.NUMERICAL MODELING

This research involves two main components in its 3D dynamic modeling. The initial part focuses on modeling and verifying soil liquefaction, while the second part involves modeling an embankment dam over a liquefiable soil improved by installing DSM columns.

1) Soil Liquefaction Modeling and Verification

The accuracy and capability of a 3D dynamic finite difference model developed in FLAC^{3D} to examine liquefaction in loose sandy soil were validated through a comparison of the simulation results with data obtained from Test No.1 of the Verification of Liquefaction Analysis by Centrifuge Studies (VELACS) project, confirming its ability to predict EPWP during liquefaction. To simulate the behavior of liquefiable loose sandy soils in FLAC^{3D}, the Finn constitutive model was utilized. The laminar box in Test No.1 of the VELACS project consisted of a horizontal layer of uniform Nevada No. 120 sand (properties are given in Table 1). This experimental setup aimed to simulate a 10 m soil layer in the prototype. The numerical model was selected to have dimensions of 12×16 m at a depth of 10 m and the boundary conditions in the numerical model were fixed on all four sides and rigid at the bottom to avoid seismic waves reflection.

TABLE 1. NEVADA SAND PROPERTIES [5]

Characteristics	Dry Density	Void ratio	Permeability	Poisson ratio	Shear Modulus	Friction Angle	Cohesion	$(N_1)_{60}$
Unit	kg/m^3	-	m/s	-	MPa	Degree	kPa	Blows
Nevada Sand	1500	0.73	6×10^{-5}	0.3	3.85	30	0	7

The acceleration time history applied at the base of the box, illustrated in Figure 1, comprised 20 cycles of a sinusoidal input with a frequency of 100 Hz [5]. For the dynamic loading in the model, an acceleration time history was applied at the lowest point with a frequency of 2 Hz and an acceleration of 0.235 g.

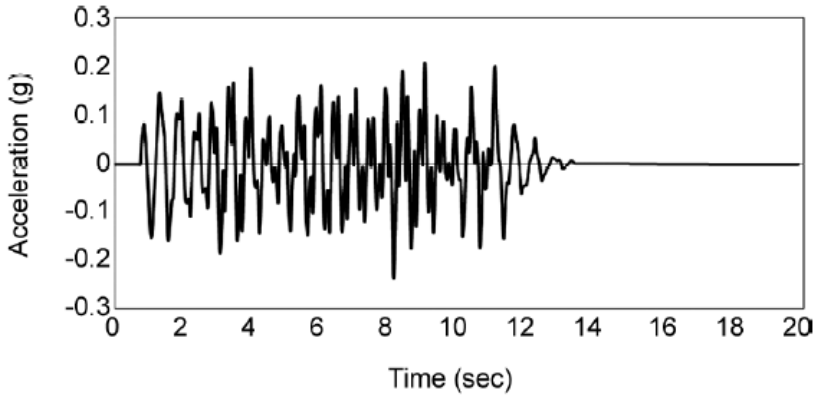


Figure 1. Horizontal acceleration time history for VELACS project (The graph source: [5])

In order to analyze the generation of dynamic EPWP, the Finn constitutive model [6] is incorporated into FLAC^{3D}. This model, which connects equations (1) and (2) to the plastic Mohr-Coulomb model [6-8], considers

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specific boundary conditions and coefficients to accurately determine the changes in fluid pressure within a porous medium. The Finn constitutive model is capable of conducting the coupled dynamic-groundwater flow calculations and simulating the effects of liquefaction.

$$\Delta\varepsilon_{vd} = C_1.(\gamma - C_2.\varepsilon_{vd}) + \frac{C_3.\varepsilon_{vd}^2}{\gamma + C_4.\varepsilon_{vd}} \quad (1)$$

$$\frac{\Delta\varepsilon_{vd}}{\gamma} = C_1.exp\left(-C_2.\frac{\varepsilon_{vd}}{\gamma}\right) \quad (2)$$

Where C_1, C_2 , C_3 , and C_4 denote constants that are linked as follows: $C_1C_2C_4 = C_3$. $\Delta\varepsilon_{vd}$ shows the soil volume decrease, γ denotes the size of periodic shear strains, and ε_{vd} denotes the accumulated volumetric strain from previous cycles in percent. The amount of C_1 coefficient depends on the sand relative density and C_2 is also a constant fraction of C_1 and can be expressed as $C_2 = \frac{0.4}{C_1}$. Considering Rayleigh damping, the natural frequency of the model is 2 Hz in this study and the damping coefficient applied was 5% of critical damping [9-11].

The comparison between EPWP at a depth of 1.25 m for numerical modeling and the centrifuge test No. 1 in the VELACS project in Figure 2 reveals a relatively good agreement between the numerical results and the results of the model test No. 1 in the VELACS project. liquefaction occurs when excess pore water pressure increases due to cyclic loading during seismic activity, leading to a decrease in effective stress (Liquefaction likely occurs when EPWP > effective stress). The reduction in effective stress weakens the soil, causing it to behave like a liquid and lose its shear strength. A similar agreement was also obtained at depth of 2.5 and 3.5 m. Similar approaches and results in the comparison of numerical results and the results of the model test No. 1 in the VELACS project have been successfully achieved by the author in the previous studies on soil liquefaction, such as [9-11].

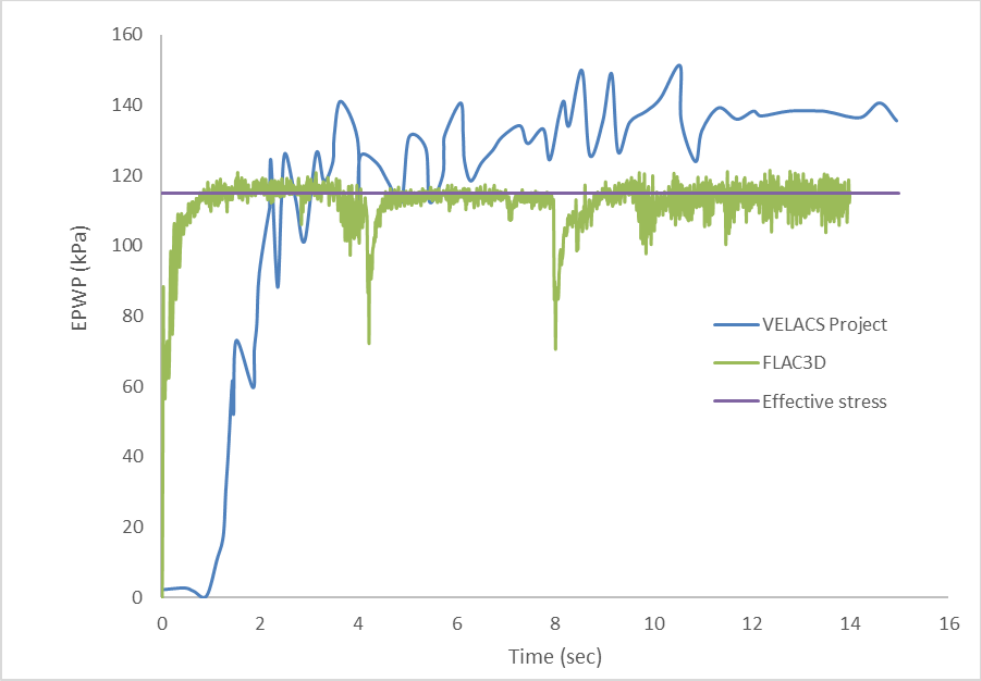


Figure 2. EPWP values at a depth of 1.25 m with finite difference model and test No. 1 of VELACS project

2) Modeling and verification of an Embankment Dam on the group column type of DSM columns

Once the validation of the liquefaction simulation model was completed, a 3D dynamic finite difference model was created to represent an embankment dam supported by a square pattern of group columns using the DSM method. The model considered a 10-meter thick liquefiable layer composed of loose sandy soil beneath the

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embankment dam, with properties similar to Nevada No. 120 sand. The characteristics of the DSM columns utilized in this numerical study are presented in Table 2.

TABLE 2. DSM COLUMNS PROPERTIES [12]

Characteristics	Dry Density	Void ratio	Permeability	Poisson ratio	Shear Modulus	Friction Angle	Cohesion	Bulk Modulus	Young Modulus
Unit	kg/m^3	-	m/s	-	MPa	Degree	kPa	MPa	MPa
DSM Columns	2100	0.45	10^{-1}	0.3	173	33	2800	375	450

In this study, a typical embankment dam with a 34-meter (m) height constructed on 20 m of alluvium which includes a 10-m thick loose sandy soil susceptible to liquefaction over a 10-m thick clay layer. In this study, for simplicity, the weight of the embankment dam has been applied on a square shape pattern over group type DSM columns. The weight of an embankment dam depends on various factors, including its dimensions, geometry, and the materials used for construction. Embankment dams are typically constructed using compacted earth materials. The specific gravity and moisture content of the materials also influence the weight of the dam. To estimate the weight of an embankment dam, the cross-sectional area of the dam and the average unit weight of the materials used were considered. In Table 3, the geotechnical characteristics of a typical embankment dam used in the U.S and other countries and adopted herein for the numerical modeling have been presented.

TABLE 3. CHARACTERISTICS OF AN EMBANKMENT DAM [13]

Characteristics	Wet Density	Saturated Density	Permeability	Bulk Modulus	Shear Modulus	Friction Angle	Cohesion	$(N_1)_{60}$
Unit	kN/m^3	kN/m^3	cm/s	MPa	MPa	Degree	kPa	Blows
Dam Core	20.5	21.5	10^{-6}	600	120	25	39.2	-
Dam Shell	22	23	10^{-3}	450	210	38	9.8	50

The seismogram recorded by University of California – Santa Barbara (UCSB) during the 1978 Santa Barbara earthquake is shown in Figure 3. The seismogram was been used as an input for horizontal input motion at the bottom of the model. This seismogram shows the acceleration of the ground during the earthquake as a percentage of gravity. Figure 4 depicts the finite difference model for the embankment dam load applied over group type DSM columns in Flac^{3D}. The finite difference model shown in Figure 4, used for simulating DSM column behavior, has been previously employed by the author to study soil liquefaction beneath shallow foundations [9-11].

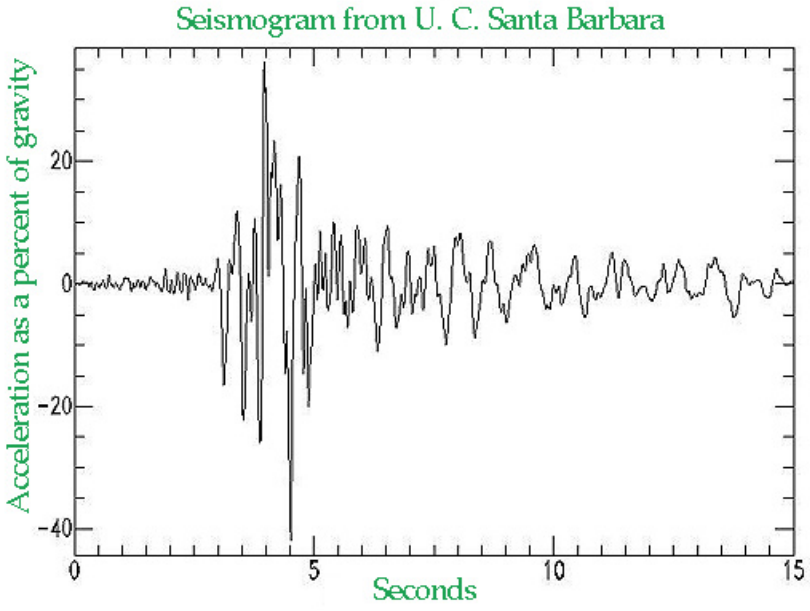


Figure 3. Seismogram from UCSB recorded during the 1978 earthquake (The graph source: [14])

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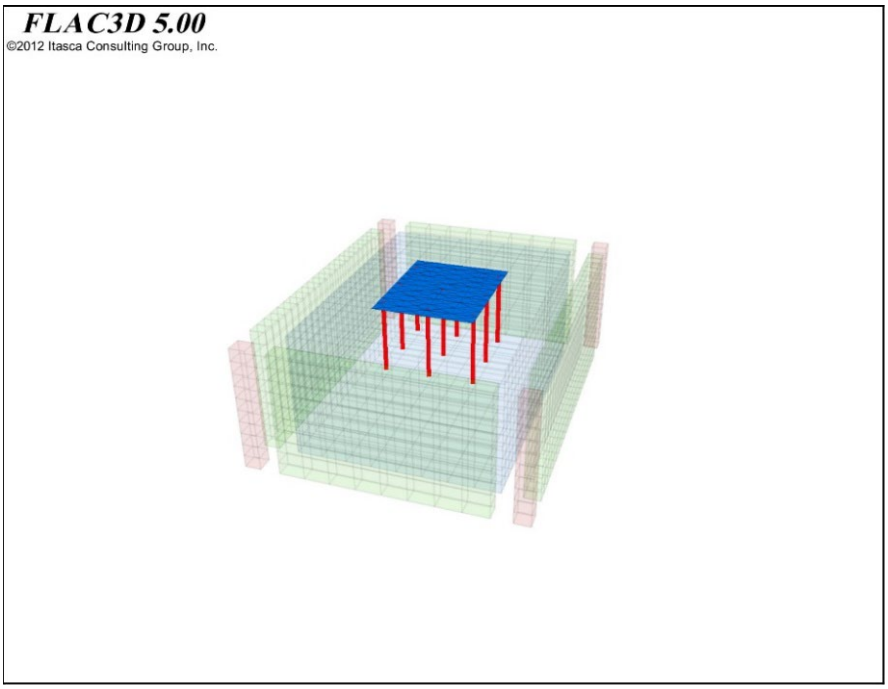


Figure 4. Finite difference model for the embankment dam load applying over group type of DSM columns

III.RESULTS AND DISCUSSION

The numerical findings are presented for a square pattern of DSM columns, which are commonly used in geotechnical projects, with diameters of 100 centimeters (cm), 120 cm, and 150 cm. These columns are subjected to the load of an embankment dam, and the spacing between the columns is determined by the ratio of column spacing to diameter $\left(\frac{s}{d} = 2,3,4\right)$. Figures 5-7 illustrate the results of the dynamic analysis, specifically focusing on the changes in EPWP for the group DSM at a depth of 10 meters. Models with $\left(\frac{s}{d} = 5\ or\ greater\right)$are not shown in the graphs as they all experienced complete liquefaction. The EPWP values increase when the spacing between the DSM columns is increased regardless of their column diameters. The findings indicate that as the diameter of DSM columns increases, excess pore water pressure decreases and the risk of liquefaction decreases. In addition, DSM columns with a diameter of 120 and 150 cm showed satisfactory results in mitigating liquefaction. The DSM columns for s/d=2 with a diameter of 150 cm showed the best performance. The results depicted in Figures 5-7 demonstrate that, regardless of the column diameter, an increase in the spacing between the DSM columns leads to higher EPWP values. This indicates that larger column spacing may result in reduced effectiveness in mitigating liquefaction. The results suggest that implementing a group DSM configuration with larger column diameters and reduced spacing can significantly improve the resistance of embankment dams to liquefaction.

According to Figure 5, for the square pattern of group-type DSM columns with diameters of 100 cm, as EPWP is greater than the effective stress, liquefaction will be anticipated for that column diameter. According to the results presented in Figures 6-7, the performance of the square pattern of group-type DSM columns with diameters of 120 cm, and 150 cm demonstrates its effectiveness in mitigating liquefaction for a duration of approximately up to 5 seconds. During this critical time frame, the DSM columns successfully reduce the EPWP generated by liquefaction, thus enhancing the stability and resilience of the embankment dams. The significance of this observation lies in the fact that the initial moments following an earthquake are often the most critical for the structural integrity of earth structures. Within this 5-second duration, the group DSM configuration effectively controls and reduces the EPWP, minimizing the potential for liquefaction-induced failure and its devastating consequences.

These findings contribute to the understanding of the time-dependent behavior of the DSM method under seismic loading. By identifying the optimal performance window, engineers and geotechnical professionals can design and implement the appropriate group DSM configurations to effectively counteract liquefaction within this critical timeframe. It is important to note that the specific duration of 5 seconds mentioned here may vary depending on the seismic characteristics and soil conditions of the site. Although, according to the results presented in Figures

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6-7, the performance of the square pattern of group-type DSM columns with diameters of 120 cm and 150 cm demonstrates its effectiveness in mitigating liquefaction for durations exceeding 5 seconds (as EPWP is less than the effective stress), further research and analysis are needed to investigate the behavior of group DSM configurations over extended time periods and under various seismic scenarios. Nonetheless, the results presented in this study offer valuable insights into the performance of DSM columns within the initial seconds of seismic activity, providing a foundation for further investigations and practical applications in earthquake-resistant design.

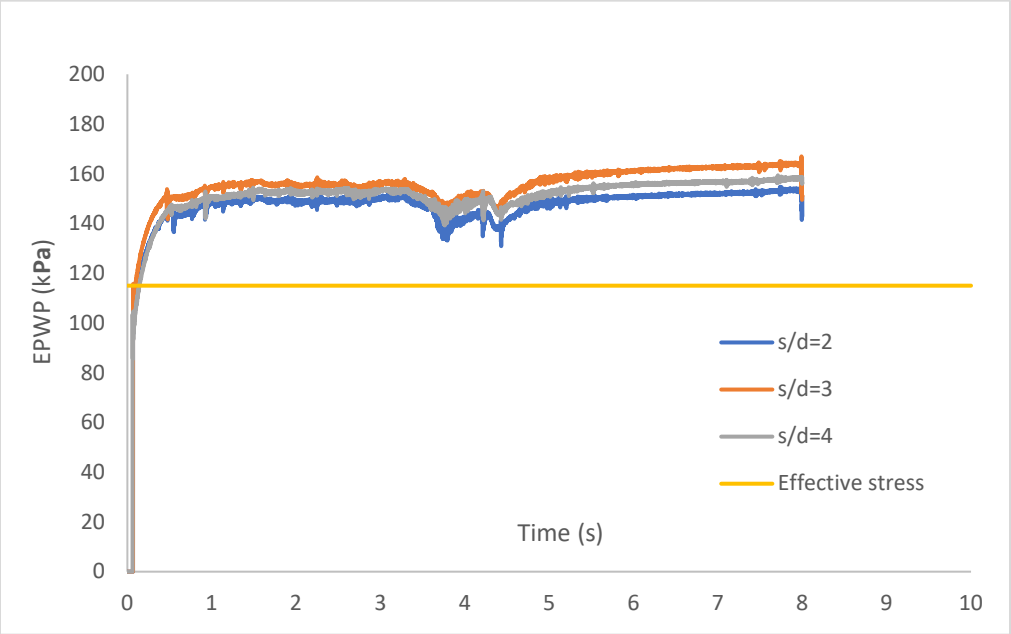


Figure 5. EPWP variation beneath the embankment dam for a diameter of 100 cm DSM columns and a ratio of $\left(\frac{s}{d} = 2,3,4\right)$

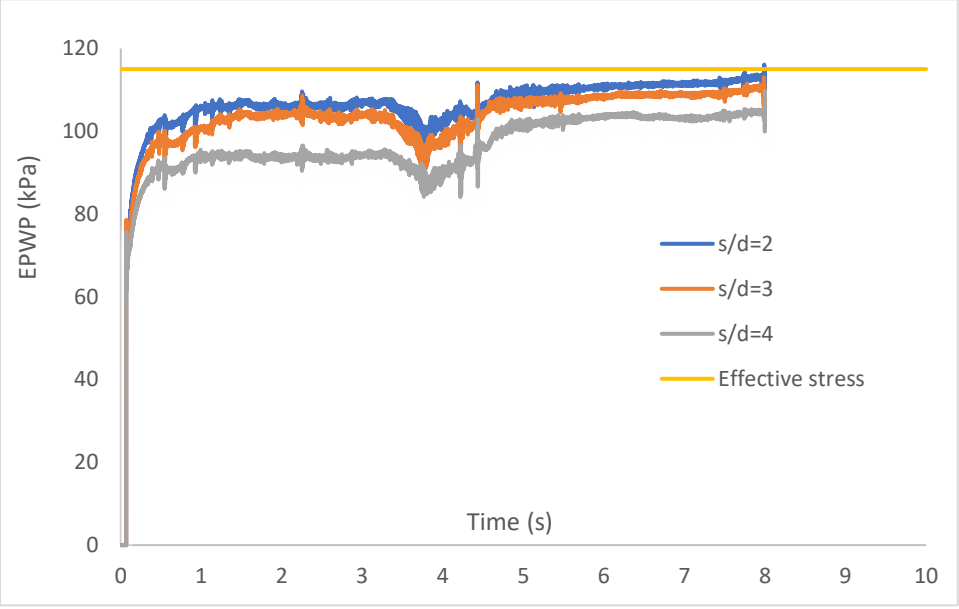


Figure 6. EPWP variation beneath the embankment dam for a diameter of 120 cm DSM columns and a ratio of $\left(\frac{s}{d} = 2,3,4\right)$

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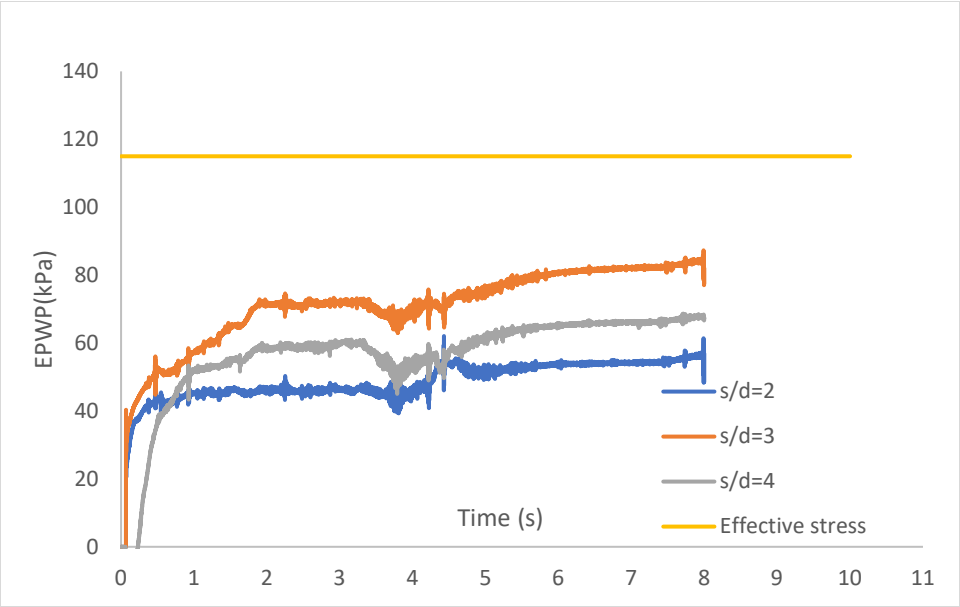


Figure 7. EPWP variation beneath the embankment dam for a diameter of 150 cm DSM columns and a ratio of $\left(\frac{s}{d} = 2,3,4\right)$

IV.CONCLUSIONS

This paper investigated the influence of using DSM columns for liquefaction mitigation for an embankment dam founded on liquefiable soils. The main findings of this study are as follows:

- The application of the group column DSM method has been identified as an effective approach for mitigating liquefaction caused by earthquakes beneath embankment dams.
- The results highlight the importance of column spacing and diameter in mitigating liquefaction-induced failure of embankment dams. The findings demonstrate that increasing the spacing between DSM columns leads to higher EPWP values, indicating reduced effectiveness in mitigating liquefaction. Conversely, increasing the diameter of DSM columns decreases the risk of liquefaction, with DSM columns of 120 cm and 150 cm diameters exhibiting satisfactory performance in reducing liquefaction effects.
- Among the group type DSM columns evaluated at a depth of 10 m with different s/d and diameter, the DSM columns with a diameter of 150 cm and a s/d ratio of 2 demonstrated the most effective performance in mitigating the risk of liquefaction beneath the embankment dam.
- The observed performance of the DSM method is much more effective for a duration of approximately up to 5 seconds. Further research is necessary to investigate its behavior over longer durations and under different seismic scenarios, accounting for site-specific soil conditions and seismic characteristics.
- The developed model and results can be used in the design and implementation of effective group DSM configurations, reducing the vulnerability of embankment dams to liquefaction-induced failure during seismic events.

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As a civil/geotechnical engineer, Araz Hasheminezhad has over 6 years of experience working on civil engineering projects dealing with geotechnical problems related to problematic soils. His work focuses on soil improvement techniques and how to effectively utilize them for improving the characteristics of problematic soils and decreasing the probable problems resulting from them. His efforts in academia and initiated several collaborations, leading to multiple papers in the field of geotechnical engineering, focusing on soil improvement techniques.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

Fracture Networks in Mississippian Carbonate Rocks: Investigation of Dam Bedrock Foundations in Northeastern Oklahoma

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ABSTRACT

This study focuses on three dam facilities in NE Oklahoma where the foundations' underlying carbonate bedrock experiences post-flooding seepage events, undermining the foundations and possibly diminishing their containment efficacy. The internal erosion related to natural discontinuities, such as joints and faults, threatens the safety of the dams, as they indicate a potential for changes in the structural conditions of the foundation. In carbonate rocks, these discontinuities often consist of complex fracture networks due to their heterogenous origin, evolution, and geometry. As a result, these networks often employ controls on fluid flow, where the fractures can act as conduits. From the geologic perspective, the three facilities have minimal historical information; hence, characterizing the bedrock at each facility is essential. This study aims to provide new geologic data and identify the characteristics of the fracture networks present and the role of those fractures in fluid migration within the bedrock foundations of the dams.New photogrammetry surveys of each facility were conducted to create high-resolution digital elevation models and orthophotos. These models are used as base maps in a geographic information system to map the geology and structures at each facility. Detailed structural mapping using the augmented circular scanline method, a fracture sampling method, was applied to several exposures chosen based on the best representation of fractures available in outcrop.The formations exposed at all three facilities comprise of the Mississippian-aged Keokuk and Reeds Spring Formations and consist primarily of interbedded limestone and chert. The geologic and structural mapping revealed that joints along bedding planes, systematic joint sets, fractured chert beds, and dissolution surfaces contribute the most to the permeability of the rocks. Detailed structural mapping was performed in the lower portion of the Reeds Spring Fm., and it was found that the fracture intensity varies depending on lithology. The chert beds have a higher fracture intensity, with an average of 26 m/m2, while the limestone beds have an average fracture intensity of 14 m/m2. Six fracture networks are present: oriented N-S, E-W, WNW-ESE, NW-SE, NNE-SSW, and ENE-WSW. The orientation of the fractures depends on the lithology of the bedding surface measured, with the joints in the limestone-dominated bed predominately oriented in the ENE-WSW direction. These structures intersect at high angles to form fracture networks that allow fluid passage.Future monitoring of structural conditions is being developed through geologic maps, core logging, cross-sections, downhole geophysical logging, and additional geophysical surveys.

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DAM SAFETY PALM SPRINGS
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Escape Training Monotony by Hosting an Escape Room

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ABSTRACT

U.S. Army Corps of Engineers requires Operations personnel to periodically attend Dam Safety Training so that they are prepared to prevent or respond to a dam safety emergency. It is at times difficult to keep their attention and help them to retain the information presented. While planning the 2022 Dam Safety Training sessions for the Wilmington District, I considered my audience's potential learning styles, the training topics for that year, and cost-effective ways to make the training memorable and easily adaptable to each of my projects. I found an escape room (a series of puzzles fit to a storyline that are solved by a team to reach a goal) complemented a combination of traditional classroom and hands-on training and improved student engagement. To prepare for the escape room training activity, I conducted trial sessions with coworker volunteers. After observing their attempts at the puzzles, I revised the activities to ensure dam safety knowledge and not puzzle skills were being tested. I also made adjustments between training sessions to accommodate various training locations and class sizes (ranging from 7 to 27 students). I conducted the training with four classes, reusing props and puzzles but tailoring the information each time to match the uniqueness of the six different dams cared for by the various student groups. All classes provided positive feedback. The escape room activity not only showed retention of information, but also demonstrated that students could apply that knowledge in a practical way. Dam Safety Training providers can easily adapt the escape room activity to suit their own projects and training goals while reducing the monotony often experienced with routine training.

I.INTRODUCTION

The Wilmington District of the U.S. Army Corps of Engineers (USACE) owns and operates nine dams. The Dam Safety Program Manager (DSPM) performs annual Dam Safety Training for the district's dam-assigned Operations staff. Over the course of five years, it is expected that all employees will have received at least six hours of training as required in Engineering Regulation 1110-2-1156 [1] even if they miss a class or two during that time. Topics for the annual training are not the same each year and do not cover the entirety of topics that are required in the five-year period, but over the course of the five years, all required topics are covered multiple times. Teaching and testing methods are also varied to keep participants interested and engaged. The training sessions are provided on-site at each dam and are tailored with site-specific information for that dam.

In 2022, the training consisted of a combination of classroom and hands-on training prior to a test. The topics covered in this training were Dam Safety Action Classification (DSAC), types of dams, seepage mitigation measures, the use of sand in filters, distress indicators, common deficiencies, dam features and terminology, instrumentation (including what it tells us and how we monitor it), and Emergency Action Plan (EAP) contents. The test was done with an escape room type activity that tested the participants' knowledge retention and also their ability to put that knowledge to practical use by applying the information they had learned earlier in the day. The training and activity took approximately three and a half hours to complete. This paper will explain the considerations that went into planning the escape room activity, provide detail on each of the puzzles that made up the activity,

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and state lessons learned during the 2022 undertaking. It is the author's intent that other trainers will be able to use the detailed descriptions of puzzles in this paper to put together their own escape room activities to reinforce Dam Safety Training topics with their own students.

II.ESCAPE ROOM ACTIVITY CONSIDERATIONS AND PLANNING

An escape room is a series of puzzles fit to a storyline that are solved by a team to reach a goal. Some students may not have experience with escape rooms and the concept will need to be explained. Other students may be very experienced with escape rooms and the differences between a commercial escape room and this Dam Safety Training activity will need to be explained. Try to get a sense of level of escape room experience by polling the group and use this information when forming the teams. Attempt to have an even level of experience across all teams. The goal is not to test the participants' ability to solve puzzles, but to use the puzzles to test their knowledge and understanding in a new and interesting way. Participants less familiar with escape rooms may need the trainer to step in and help if they get stuck on a puzzle mechanic and the trainer should be prepared to do so.

Some ways the 2022 activity puzzles described in this paper were not like a commercial escape room:

1. Puzzle materials were obvious to the players. All "hidden" materials were marked with a USACE Dam Safety Beaver image. This helped ensure players knew they had found the right object.
2. Props were not permanent fixtures of the facility. To provide some traditional escape room puzzle mechanics, while not permanently securing things to the floor, props that were not meant to be moved were labeled as such and these signs were explained to participants before the activity began.
3. Locking props were not tamper proof. Many of the locks used in this activity were able to be forced open without the correct key. Participants were instructed that if it looks like it is supposed to be locked shut until the correct combination or key opens the lock, play along with that understanding and do not forcibly remove any lock. This also applied to locks on doors. For safety reasons, no doors were actually locked. If a door was meant to be "locked" until opened, a locked external lock was added to the handle or knob. Participants understood that the door was not allowed to be opened unless the lock was also opened by legitimate means.
4. All players were instructed to bring a writing utensil to the activity.
5. Rather than a set number of hints, the trainer(s) occupied the play area with the players and monitored the teams' activities. When a team was stuck, the trainer would offer hints to either move the activity along or ensure that Dam Safety was what the players were being tested on, not puzzle mechanics.
6. Multiple teams playing concurrently: Due to the design that these puzzles would be done by multiple teams in a single session, a number of instructions were given to the participants to help facilitate this while easing effort on the trainer. Teams were instructed to reset all puzzles once solved so that they would be "locked" again when the next team arrived. Teams were also instructed to only take one key if multiple keys were found in a single place. The other keys would be for the teams that had not yet solved that puzzle. Due to the fact that so many of the puzzles relied on access to an EAP, multiple copies were brought to the sites where normally only one would be located. They were still located in the normal place so that teams were tested in knowing where to go to get the EAP, but there were always enough for each team to grab their own copy.
7. Rather than the goal being to escape from a room, the goal was explained when the scene was set. Getting to the goal involved either escaping from or breaking into a room or building to complete the various puzzles.

To set the scene for the storyline of the 2022 Dam Safety Training escape room activity, the first task the participants did after listening to the rules was watch a short video (Figure 1) in a space separate from the puzzle stations that will be described later. The video was of a puppet beaver, speaking in a slightly alien-like voice. The beaver said:

> *You are from the future. In your time, there are widespread blackouts and famine due to lack of energy to satisfy demand. Technological development has come to a halt and in many places, regressed. You are part of a team that is trying to find a solution to the energy crisis before it's too late. You have been told that an ancient alien energy source, great enough to provide all the world's populations' energy needs, was in the hands of the US Government years ago but due to Records Management mishandling, the box that stored this technology was forgotten and lost. Through your research, you have determined that that box was in this office in the year 2022. Your research has also informed you that the alien civilization responsible for the energy source was made up of dam safety obsessed creatures that spoke*

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in rhyme, and that they had also found the lost source was in this location in 2022. They appear to have been testing the humans at the time to see if they were worthy of rediscovering the power source. Your counterparts have used almost the last of their energy reserves to send you back in time to find this source and return it to your own time. You have been time transported into the bodies of U.S. Army Corps of Engineers staff who have just completed Dam Safety Training. Tap into their knowledge to remove the barriers that prevent you from identifying the power source. These bodies may not be completely reliable, and certain functions such as speaking or walking may fail to function when you want them to. If that happens, you will need to rely on the other members of your team to keep working at the issue at hand. If your team is unable to find the energy source, you will be stuck in the past forever, as your teammates left in the future do not have enough energy on their side alone to bring you back. Good Luck.



Figure 1. Introduction video to set the scene. (Photo by Megan Garrett depicting video by Megan Garrett)

Built into this transcript is the control used to keep one person from doing all the work on the team. If the trainer thought some team members were doing a disproportionate amount of the work, the trainer would instruct those players that they were not allowed to speak or use some other body part for some period of time. This allowed/forced the other team members to participate more and also gave the overachieving team members the clue that they should allow or encourage more team effort once the trainer allowed them to be fully functional again. This control ensured that everyone was participating, even in larger teams.

Any storyline is fine to set the scene as long as it sets the purpose for why the participants are solving puzzles and for what goal. A video is not necessary. The storyline could be explained with a readahead or just explained by the trainer.

The number of participants should be considered when determining how many teams will play at once. Limiting the teams to no more than six members is a good rule of thumb to ensure everyone can participate in the puzzles. More teams will require more puzzles or more geographic space between puzzles to keep the groups separated but still actively engaged the entire time (if teams are playing concurrently, as the puzzles in this paper are designed). More teams and puzzles will also require more trainers overseeing the activity.

Site-specific and environmental considerations will play a role in where the puzzle stations will be setup. A puzzle consisting of card or paper puzzle pieces, for example, will not fare well outside on a windy day. Some sites have easy access to instrumentation while others do not. Considering all the various site-specific and environmental factors prior to planning the puzzle logistics and finding something that works for all the sites where the training will take place will save effort on having to tailor or change some puzzles between sites for nontechnical reasons.

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Making props so that they can be reused at multiple projects will reduce preparatory time for each training. While some puzzles will work universally for most or all of the dams in a portfolio, others are meant to be tailored to the technical site-specifics. These changes should be made to the puzzles between projects. The puzzles described in this paper are easily adaptable by changing out individual dam facts and details to match the dam being trained.

Once the escape room activity is fully planned, it is important to test the puzzles on some participants who were not involved in creating the puzzles. Observing these subjects interact with and attempt to solve the puzzles will offer insight into what tweaks need to be made to wording or props to again ensure that Dam Safety knowledge is being tested, not puzzle mechanic savvy. This test will also provide a frame of reference for how long it will take a team to complete a puzzle and whether the activity appears too challenging or not challenging enough.

Trainers should come to the activity with cheat sheets that contain all puzzle answers and lock combinations. This will expedite assisting struggling teams and also assist with set-up and take-down.

III.PUZZLE STATIONS

The following subsections detail the puzzles and puzzle stations used in one session of the 2022 escape room activity. This session included the most students; 27 students on five teams. The puzzles are nonlinear and can be at least partially done in any order. Each team was given a map (Figure 2) which indicated their starting station (either 1, 2, 3, 5, or 6). After this starting station, teams were allowed to go to any unoccupied station. Question marked locations shown in the corner of the map represented locations that numbered stations would bring them to if completed successfully. In the puzzle descriptions following, these locations are labeled with letters. Teams were instructed to check off the numbered stations as they were completed and told not to return to the Powerhouse (where earlier training had taken place and where the final mystery station was located) until all numbered stations were completed.



Figure 2. Map sheet provided to Green Team. (Sheet developed by Megan Garrett using Google Maps Imagery ©2023 CNES / Airbus, Commonwealth of Virginia, Maxar Technologies, Map data ©2023)

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A. Station 1

Concepts Tested: Common deficiencies (animal burrows), terminology, project-specific features

Materials Required: Locking briefcase, mirror with a Scrabble tile affixed to the frame, mirrored riddle printed on a piece of paper taped to briefcase, bag of scrabble tiles, maze puzzle sheets

Prerequisite for Success: None

Resulting Action or Product: Answer to maze puzzle leads team to Station E

Puzzle Presentation: See Figure 3 for main features of Station 1.

Team is presented with a black locking briefcase, a mirror, and a bag of scrabble tiles. The briefcase has a piece of paper affixed to one side on which the following riddle is printed in mirrored text:

Humans of Earth
How smart you be?
Responsible dam owners?
We shall see.
To be worthy of the power,
Our tests you must pass.
Show off your dam safety knowledge,
And the treasure will be within your grasp!
(Besides vegetation)
□□□□□□□□□□□ s are often to blame,
For recurring deficiencies.
Keep an eye out on your inspections,
Else one may bring you to your knees.
They shorten seepage paths,
And provide starting points for erosion.
Keep the critters at bay and fill them in,
A granular fill or other soil would be well-chosen (not cement!).

The team needs to hold up the mirror to the case to read the riddle (Figure 4). The answer to this riddle is "burrow," which when spelled out in the provided Scrabble tiles, the point values of the letters shown on the tiles produce the numerical string 311114. This code unlocks the briefcase.

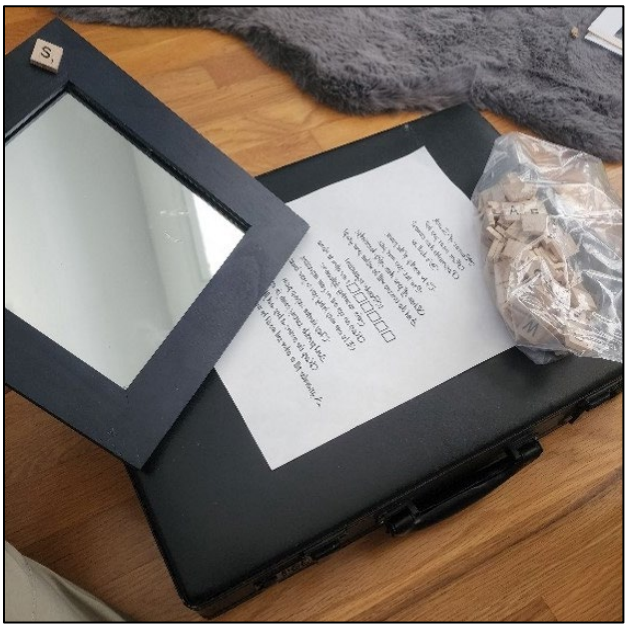


Figure 3. Mirror, briefcase, mirrored riddle, and bag of Scrabble tiles for Station 1. (Photo by Megan Garrett)



Figure 4. Team using mirror to read riddle at Station 1. (Photo by Megan Garrett)

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The briefcase contains a maze puzzle. The maze is full of seemingly random words and letters. The team needs to solve the maze to read the words and letters in the correct order to reveal the following clue:

On Left Wing Dike, 5 Piezometers in a line. At the one nearest the crest, your next clue you'll find.

The team needs to know where the Left Wing Dike is at the project, what a piezometer is, and where the crest is, in order to follow the clue to arrive at Station E.

Puzzle Preparation: The text for the mirrored riddle was printed in Vivaldi as this would be a font not easily read when mirrored without the aid of a mirror. The text was converted to an image and flipped in Word to create the mirroring. A Scrabble tile "S" was attached to the frame of the mirror with double-sided tape to give a clue to the team that the bag of tiles was related to the mirrored riddle. Some of the standard Scrabble game tiles not used in "BURROW" were omitted from the bag, shortening the teams' time to search for letters and solve the briefcase combination. Reference [2] provides the website used to generate the maze puzzle. Enough maze sheets were printed and staged in the briefcase so that each player on each team had access to their own copy.

B. Station E

Concepts Tested: Ability to read instrumentation, project-specific features

Materials Required: Piezometer, water level indicator, printed riddle

Prerequisite for Success: Complete Station 1

Resulting Action or Product: Three-digit code for half of metal briefcase at Station 4 and second part of riddle leads team to Station F

Puzzle Presentation: The team is presented with an unlocked open-tube piezometer, a water level indicator, and a riddle printed on a piece of paper which reads as follows:

PZ-20, you have found.
Find the depth to water,
then up to whole number do round.
Take this number,
and with the PZ ID do addition.
Round to nearest 10 and put a 1 in front
to complete this mini-mission.
Before you leave, one thing more:
There is a place near here, where water does pour
Out of a pipe, that is part of a toe drain.
Check it out, to continue our game.

The team needs to use the water level indicator to find the depth to water in the piezometer and do the simple math in the riddle to produce the number 160. The team does not know what this number will be used for yet unless they have visited Station 4 and must retain it for later. The second part of the riddle leads the team to the nearby toe drain outfall and weir.

Puzzle Preparation: Only one printed riddle is needed if affixed to the water level indicator, or piezometer, or if proctored. The resulting number needs to be three digits for the metal briefcase at Station 4. Since monitoring history indicated the water level at this location would reliably be between 36 and 41 feet, simple math was selected that would result in 160 as long as the measurement was within this range. For other training sessions, the same methodology was used to select the necessary math to result in a three-digit code for the lock. The lock's combination was reset for each unique session's code.

C. Station F

Concept Tested: Emergency Action Plan (EAP) location and use

Materials Required: Binder or folder, Riddle printed on sheets of paper, EAP (not located at Station F)

Prerequisite for Success: Complete Stations E and 2

Resulting Action or Product: Three-digit code for half of metal briefcase at Station 4

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Puzzle Presentation: Team is presented with a binder on the weir headwall. Inside the binder are sheets of paper printed with the following riddle:

Through this weir, seepage must pass.
We hope the water is clear as glass.
Cloudy water could cause alarm
And indicate a sign of harm.
Consult the EAP.
If five inches of water were passing,
how much flow would there be?
Now return!
Resume the chase!
Take these numbers you've learned;
Open the metal case!

The team must know that the EAP is located in the Visitor Center. If the team had not yet completed Station 2 at this point, they needed to do that to access the Visitor Center. Once access was gained, the team needed to know where in the Visitor Center the EAP was located. The team must retrieve the plan and locate the section that has a table to convert water height to weir flow. The answer is 123 gallons per minute. This three-digit code will open the one side of the metal briefcase located at Station 4.

Puzzle Preparation: A binder was used to hold the riddle sheets so that they did not blow away while unattended. Enough riddles were printed so that each team could take one with them and instructions were written inside the binder to take one copy of the riddle per team. A small weir diagram was printed at the top of the riddle sheet. This image is the standard one used in our EAP just before the flow conversion table. This image was also attached near the lock for which the code "123" was applicable on the metal briefcase at Station 4.

D. Station 2

Concepts Tested: Uses of sand and sandbag rings, Surveillance and Monitoring Plan

Materials Required: Small can, printed riddle attached to can, 26 popsicle sticks (labeled), 4 unmarked popsicle sticks, zippered mesh bag, printed riddle attached to mesh bag, combination (numbers) padlock, combination (letters) cable lock

Prerequisite for Success: None

Resulting Action or Product: Access to Visitor Center

Puzzle Presentation: See Figure 5 as an example of materials supplied. Team is presented with a can containing four unmarked popsicle sticks, and is wrapped in the following riddle:

This material is used in filters and drains.
There is a stockpile of it,
And every year we check
How much of it remains.
You may fill bags with this,
and make a ring,
if a new boil,
from the ground does spring.
Pack it tight,
But don't forget,
To leave a notch,
the flow to let.

Team is also presented with a small mesh bag locked closed by a combination lock through the double zipper. The bag has a printed riddle attached which reads:

Every Month, things at Island Creek we monitor.
But if this elevation is passed,
Daily we may saunter!
Through this mesh, feel free to take a peek.
Enter the elevation, If access you seek.

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Through the mesh, the team can see popsicle sticks inside the bag.

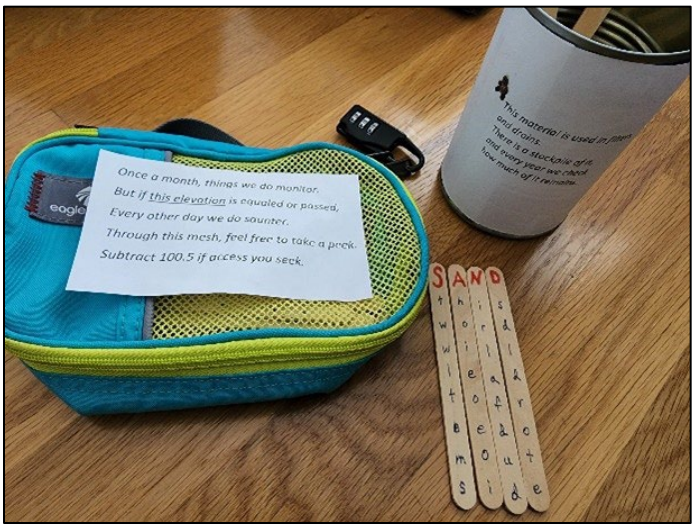


Figure 5. Equipment present at Station 2; shown with popsicle puzzle solved for escape room setup where participants start inside a conference room and need to gain access out of the room. (Photo by Megan Garrett)

Station 2 is located near to the entrance to the Visitor Center where a combination cable lock (letter combination) is wrapped around the door handle to indicate the door cannot be opened until the lock is opened.

The team needs to solve the riddle on the mesh bag to open the combination lock on the zipper with the resulting elevation of 310 feet. This lake elevation is documented in the Surveillance and Monitoring Plan for when daily monitoring takes place. Once unlocked, the team needs to extract and examine the marked popsicle sticks. Each one has a red letter at the top and seemingly random black letters down the stick. Using the riddle on the can, the team needs to determine that the answer is “sand” and spell out SAND with the red letters on the popsicle sticks. When these sticks are lined up, the following will be revealed in the black letters:

This word will open the visitor center

The team should now realize that it is the four-letter combination “SAND” that will open the combination cable lock on the Visitor Center entrance door, allowing access to the Visitor Center.

Puzzle Preparation: Popsicle sticks were labeled with sharpies. Each red letter appears only once. Only the sticks that spell out “sand” contain a hidden message. All other black lettering is random. For sessions where the teams began inside a conference room, the four SAND sticks were replaced with ones that had the appropriate message for the locked location.

E. Station 4

Concept Tested: EAP use

Materials Required: Locking briefcase, posterboard printed with EAP notification chart and cut into puzzle shapes, an approximately 3 ft^2 (1 m^2) plexiglass sheet with some puzzle pieces attached with double sticky tape

Prerequisite for Success: Stations E, F, and 2

Resulting Action or Product: Location of next clue (Station B)

Puzzle Presentation: Team is presented with a metal briefcase and sheet of plexiglass (see Figure 6). The metal briefcase has two small images affixed near the locks. One image is of a water level indicator, such as the one used at Station E. The other image is the same as the weir diagram used at the top of the riddle at Station F. The codes from each of these other stations are used to open the respective locks on the case. Inside the case are cardstock puzzle pieces with an enlarged image of an EAP notification chart printed on them. A sheet of plexiglass is also presented with several cardstock puzzle pieces already affixed to the glass.

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Figure 6. Metal briefcase at Station 4 containing EAP notification chart puzzle pieces and plexiglass sheet with starter puzzle pieces affixed. (Photos by Megan Garrett)

The team must complete the puzzle (Figure 7), starting with the affixed pieces on the plexiglass, so that the plexiglass serves as a base for the puzzle. The team should realize, shortly after revealing the pieces, that they are pieces of an emergency notification chart. They will need to reference the notification chart in the EAP to complete the puzzle on the plexiglass. Once the puzzle is complete, the team needs to lift the plexiglass to read the clue that was written on the backs of the puzzle pieces. This clue reads “FLAGPOLE,” indicating to the team that they must go to the flagpole (Station B) to find their next clue.



Figure 7. Team using the EAP to assemble the puzzle at Station 4. (Photo by Megan Garrett)

Puzzle Preparation: The enlarged notification chart was printed on several sheets of paper and pasted onto the posterboard. Prior to cutting into shapes, the word “FLAGPOLE” was written with several markers together at one time so that parts of the words would not be decipherable on individual pieces. Every training session location had access to a flagpole. Each dam in Wilmington District has different notification charts, however only one enlarged

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chart puzzle was made and reused at all projects. Just prior to the training activity, the actual notification chart from which the enlarged print was made was inserted into the EAP at the project. This allowed the prop to be reused while still allowing the teams to find and use the relevant section of their EAP to complete the task.

F. Station B

Concept Tested: EAP use

Materials Required: Box locked by combination, riddles printed on small paper sheets

Prerequisite for Success: Stations 2, 4, 5, and 6

Resulting Action or Product: Clue to be used in conjunction with results from Station 6 to reach the final station of the activity

Puzzle Presentation: Team is presented with a combination lock box such as shown in Figure 8.

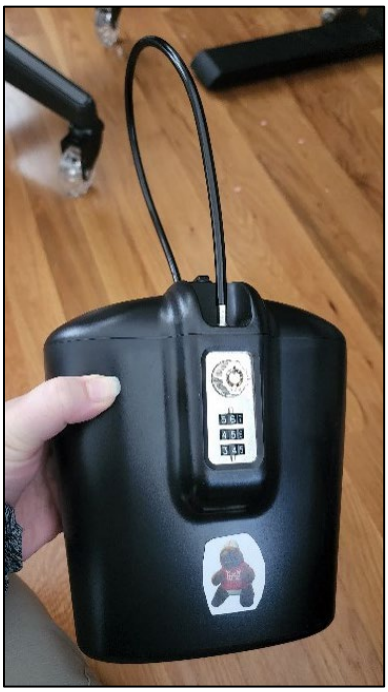


Figure 8. Locked box found at Station B (Flagpole). (Photo by Megan Garrett)

If the team has completed Station 5, they will be able to open the locked box. Inside the locked box are several slips of paper printed with the following riddle:

If the EAP is right, What kind of Small Hand Tools
Does Island Creek Dam have on site?

The team needs to know to look in the EAP appendix for materials and equipment available at the project to find that the answer is “Miscellaneous.” If the team has completed Station 6, they will recognize that this is the word that was written across the cypher they discovered. The team must hold up the cypher to the page in the EAP where “Miscellaneous” was found to reveal the clue, “Desk of Tyler” and lead the team to Tyler’s desk (Station D), which is located in the Powerhouse.

Puzzle Preparation: Riddles were revised to be site-specific for each session but always asked about something on the EAP page that lists materials and equipment (e.g., “How many sandbags” instead of “What kind of Small Hand Tools”). Enough slips of paper were prepared so that each team could take one from the box. Prior to the start of the activity, the page in the EAP that has the materials and equipment information on it was replaced with a sheet containing identical information, but which was printed on the same printer as that used to make the cypher. This ensures the holes in the cypher will line up with the letters on the sheet as expected.

G. Station 3

Concepts Tested: Types of dams, terminology, emergency measures, dam features, Dam Safety Program

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Materials Required: Bubble questionnaire printed on sheets of paper, box with key-based padlock, colored cardstock skeleton keys, padlock keys (located at Location A)

Prerequisite for Success: None

Resulting Action or Product: Colored cardstock key that is needed for final station of the activity

Puzzle Presentation: Team is presented with a small box with a padlock (Figure 9) and paper sheets printed with the following questions:

"Arch" and "gravity" are both this type of dam: __ __ (O) __ __ __ __ __
"Right" and "left" are always referred to as if we are looking __ (O) __ __ __ __ __ __ __ __.
We like to have stockpiles of gravel and sand so that we have material in case we need to build an emergency __ (O) __ __ __ __ __.
This feature lets water flow safety, instead of over the non-overflow parts of the dam: __ __ (O) __ __ __ __ __
Foundation drains relieve this under the dam: __ __ __ (O) __ __ __ __
The part of a concrete gravity dam farthest upstream, at the bottom, is called the: (O) __ __ __.
Every 5 years we do a Periodic (O) __ __ __ __ __ __ __ __ __.
This type of passageway is used to access piezometers and foundation drains inside concrete dams: (O) __ __ __ __ __ __

The bottom of the sheet indicates that a key can be found near a sign that says the phrase created by the circled letters in the answers. As the answers to the questions are: concrete, downstream, filter, spillway, pressure, heel, inspection, and gallery, the bubbles spell out "No Fishing." This leads the team to the dock (Location A) where they will find a key ring with keys to fit the small padlock at Station 3. Once the box is opened, the team will find cardstock skeleton keys and they will take one. The team will not know what to do with this key until the final station of the activity.



Figure 9. Box containing cardstock skeleton keys of various colors at Station 3. (Photo by Megan Garrett)

Puzzle Preparation: The order of questions and answer bubble placement were revised for each session so that the key location was site-specific. In multiple sessions, identical questionnaires were able to be used with the key location at a rain gauge. Enough bubble questionnaires were printed so that each member of each team could have their own. The key ring had a magnet on it, allowing it to stick to the no fishing sign (or rain gauge or other locations for other sessions). A generic and universal padlock was used on the box so that many keys were available to put on the key ring for each team to take one, removing the need to reset the key in the hiding place for each new team.

H. Station 5

Concept Tested: Dam Safety Action Classification (DSAC), seepage mitigation construction measures, signs of distress, terminology, project-specific features

Materials Required: Printed multiple choice questionnaires (Figure 10), printed matrix sheets (Figure 11), iPad mini, tube-like transparent container, paper "keys" on a metal keyring, magnet on a string (located at Location C)

Prerequisite for Success: Station 2

Resulting Action or Product: Riddle in the paper key provides code to open locked box at Station B

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Puzzle Presentation: The team is presented with a tall transparent tube labeled with “Do not touch me,” an iPad mini which is locked by a four-digit passcode, and sheets of paper printed with multiple choice questions and a matrix of images. Paper strips with images of keys can be seen within the transparent tube on a metal keyring. The tube is too narrow and tall to reach into to retrieve the keyring and the team knows they are not permitted to pick up the tube to dump out the ring because of the sign.

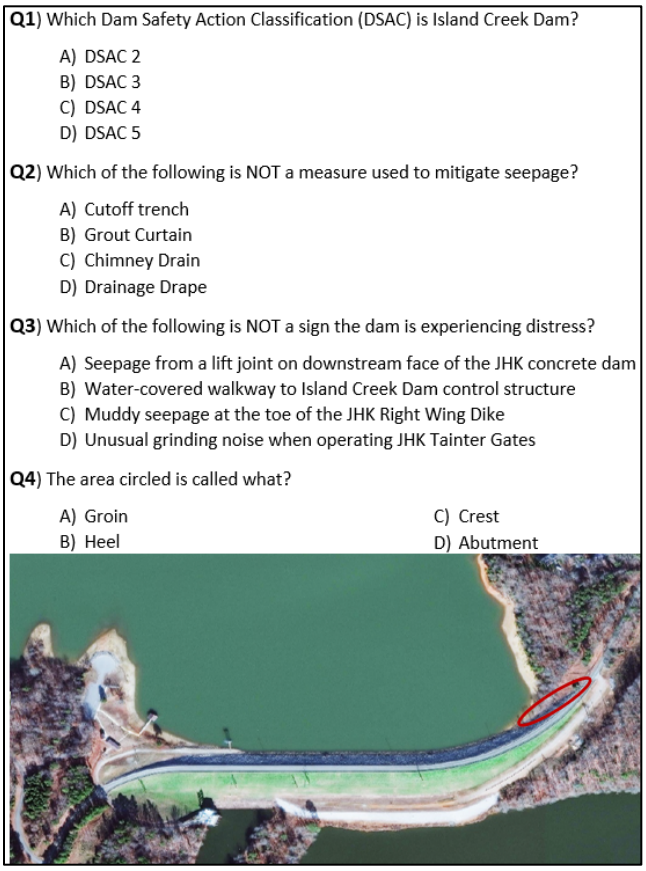


Figure 10. Station 5 questionnaire. (Sheet composed by Megan Garrett including Imagery © MAXAR 2019)

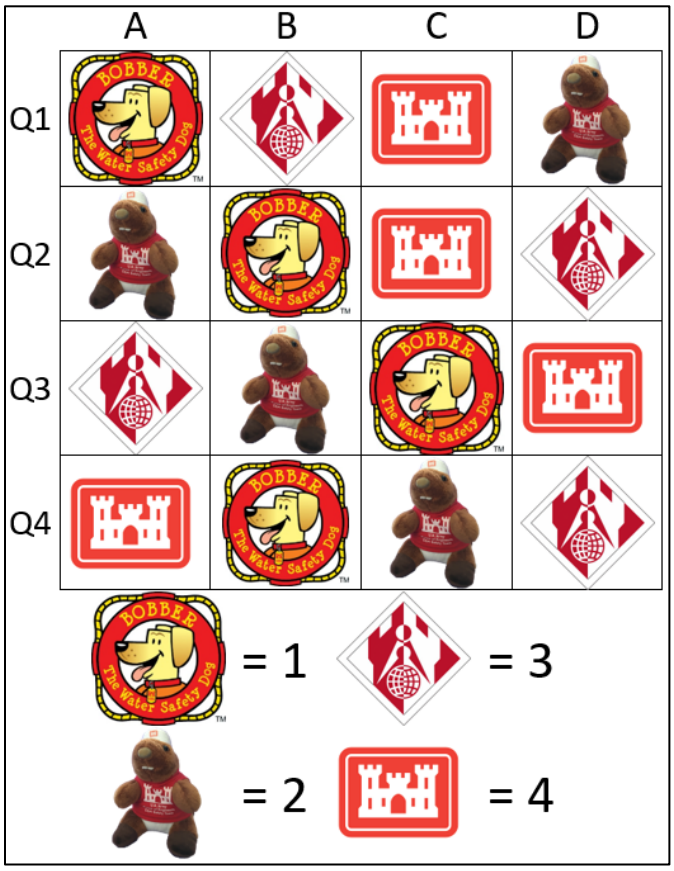


Figure 11. Station 5 puzzle matrix sheet. (Sheet composed by Megan Garrett. Army images courtesy of Army.)

The questions on the questionnaire are labeled Q1, Q2, Q3, and Q4. The matrix has corresponding row labels. The answer choices to the questions are labeled A, B, C, and D and the matrix has corresponding column labels. The team must answer the questions on the questionnaire and find the corresponding images in the matrix. The correct answers are: Q1-C (rounded rectangle castle), Q2-D (diamond castle), Q3-B (beaver), and Q4-A (rounded rectangle castle). Using the key below the matrix, these images translate into 4324. This code unlocks the iPad. Once unlocked, the iPad displays a photograph of an easily identifiable location in the Visitor Center. There is a magnet on a string in the photo with a tag identifying it as part of the activity, indicating that the team needs to retrieve the magnet from this location (Figure 12). If the team has completed Station 2, they will retrieve the magnet from the location in the Visitor Center (Location C) and return with it to Station 5. The magnet will then be lowered into the tube by the string to retrieve the keyring (Figure 13). The team will remove one of the paper strips identified as keys by the image of a key drawn on it and replace the keyring for the next team. Unfolding the paper strip will reveal a riddle which reads:

What is the number of Saddle Dikes, Masonry Dams, and DSAC at John H. Kerr? Dial in those numbers in the cable-locked container, and you may go far. #, #, #?

The answer to this riddle is 4,0,4 and this three-digit code will unlock the box located at Station B. If the team has not gotten to Station B yet, they will carry the code with them until they find the right place to use it.

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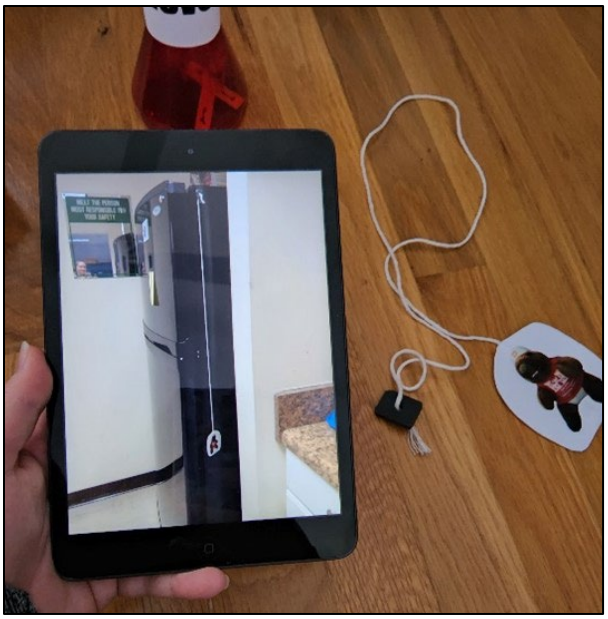


Figure 12. Unlocked iPad showing photo of magnet in Visitor Center, magnet on string, and base of tube from Station 5. (Photo by Megan Garrett)



Figure 13. Team retrieving keyring from tube at Station 5. (Photo by Megan Garrett)

Puzzle Preparation: Questions and answers were revised for each session to be site-specific, but the correct answers always remained in the same place on the sheet so that the pin for the iPad was the same for all sessions. Questionnaire and matrix sheets were printed so that there was one for every team member of each team. After staging the magnet/string, a photo was taken on the iPad. This was done for each session as each location's staging location was unique. The paper keys were strips of paper printed with the riddle on one side. The strips were then folded up to be small and pierced with the keyring. Keys were then drawn on the exposed sides of the paper slips. For sessions with less teams, actual keys were used, and the riddle was not needed. The lock box used at Station B had the flexibility to be locked with either key or code and allowed this session-specific tailoring.

I. Station 6

Concept Tested: Use of National Inventory of Dams (NID)

Materials Required: Zippered folio, numerical combination cable lock, riddle printed on paper taped to folio, cardstock cypher, access to internet capable device

Prerequisite for Success: None to get the cypher but all except Stations 3 and D to use the cypher

Resulting Action or Product: Cypher to be used at Station B

Puzzle Presentation: Team is presented with a zippered folio wrapped in a locked combination cable lock (Figure 14). After turning the folio over, the team will find a riddle attached to the folio which reads:

How many Nighttime People at Risk for John H. Kerr Dam would there be,
In a Normal High Pool Breach scenario?
Use the NID.

The team must access the NID through a smartphone or a computer to access this risk information. The answer is 4,309. This four-digit code unlocks the cable lock on the folio. Inside the folio is a sheet of cardstock with small rectangular holes cut out and the word "Miscellaneous" written in large letters across the page. The three cut out rectangles in the one corner are covered with transparent tape and the letters E, A, and P are written in each tape window. If the team has completed Station B, they will understand that this is a cypher that must be held up to the page in the EAP that the riddle at Station B directs them to which results in the word "Miscellaneous". The E, A, and P, line up with the header of the page and other letters line up with the rectangular cutouts to spell out, "Desk of Tyler" and lead the team to Tyler's (one of the supervisors') desk (Station D), which is located in the Powerhouse.

Puzzle Preparation: The cypher was created by printing the page of the EAP which contained the answer to Station B, laying that sheet on top of a sheet of cardstock, and cutting out the required letters (cutting through both

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the paper and underlying cardstock) to form the desired message. In addition to those letters, the E, A, and P in the header were also cut out. These holes were then covered with transparent tape, filled in with E, A, and P in pen on the sticky side of the tape, and then covered over with another strip of tape (sticky sides together). The word “Miscellaneous” was then written in marker in open space. Each session had a site-specific cypher and the variations in the site-specific nature of the riddle in Station B required the clue written in marker to be unique to that session as well (Figure 15).



Figure 14. Station 6 folio and cable lock. (Photo by Megan Garrett)



Figure 15. Cypher from another session where 35,000 was the answer to the riddle at Station B and the revealed clue was “Asst. OPM desk.” (Photo by Megan Garrett)

J. Station D

Concept Tested: None (culmination of all tasks)

Materials Required: Treasure box with padlock, candy or another prize that can be shared

Prerequisite for Success: All Stations

Resulting Action or Product: Win the escape room activity

Puzzle Presentation: Teams are presented with a locked box (Figure 16) with the following instruction printed on it:

Are you worthy? We shall see.
To open this box,
Give the DSPM green, blue, pink, beige, & orange keys.



Figure 16. Final locked box of the activity. Instruction for turning in colored keys was for two teams in this photo (different session than main example). (Photo by Megan Garrett)

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The DSPM exchanges all the colored cardstock keys for one metal key that will open the box. The participants can decide if the have a most valuable player to be the one to open the box. Inside the box, the participants will find candy (or whatever prize was left for them) as well as a sheet of paper that says the following:

Congratulations! You've found the power source! You can't see or feel it. You have machines in the future that can sense it though and you must bring it home. You'll be going in 3… 2… 1…

At this point, the participants have finished the escape room activity and are invited to choose from a selection of photobooth-type signs for a group photo (Figure 17).



Figure 17. Group photo after a job well done. This was the smallest class of seven students, all on one team. (Photo by Megan Garrett)

Puzzle Preparation: The instruction for turning in colored keys was adjusted to be session-specific based on the number of teams participating. Photobooth signs were reused for each session.

IV.CONCLUSIONS

The 2022 training was well received by the students and garnered numerous positive comments during and well after the events. Students were engaged and successfully demonstrated retention and application of the training through the escape room type activity.

This type of activity can be done by tailoring the readymade riddles and puzzles described here and would be a valuable addition to any Dam Safety Training Program where student engagement is a concern or priority.

V.REFERENCES

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Megan Garrett is a registered professional geologist who has worked for USACE for almost 18 years and as a Dam Safety Program Manager for the last 9 years. She currently manages the dam safety program for 9 dams in North Carolina and Virginia within the Wilmington District's boundaries. Megan provides dam safety training for USACE personnel and has taught 2 courses for troops at Fort Bragg. Megan has a Bachelor of Science degree in geology from Edinboro University of Pennsylvania and a Masters of Science degree in environmental pollution and protection from Lancaster University, UK.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

How the City of Austin uses the ESRI Survey123 Connect App

SERGIO MENDOZA, P.E., CFM, CESCP, CITY OF AUSTIN - WATERSHED PROTECTION DEPARTMENT

ABSTRACT

During the Dam Safety 2022 Conference in Baltimore, MD, I was introduced to the ESRI Survey123 and Survey123 Connect apps. The presentation was a great introduction into the benefits of these two related apps. My presentation will use the 2022 Conference presentation as a springboard. I propose to present a tutorial, based on how we, at the City of Austin Dam and Pond Safety Program (DPSP) have created our survey. The DPSP inspects 33 dams on an annual basis, as part of our responsibilities to our state's regulatory agency: the Texas Commission on Environmental Quality (TCEQ). We moved away from a printed form to an electronic form. I will discuss the steps I took to create the electronic survey. This will include an explanation of the scripts I used in the Excel XML form generated by ESRI Survey123.After we tested our survey, I found the need to modify it. The original form was linear, but we discovered that our inspections did not follow a linear path. For example, we might start an inspection at the outfall of dam one but start another inspection at the top of dam two. I needed to produce a survey that allowed us to select the element we wanted to inspect at any time.With the survey completed, I then created a form that we could send to TCEQ. In the presentation, I will explain how I created the form. I will include the steps that I took to format the MS Word document in order to publish it as a PDF.

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DAM SAFETY PALM SPRINGS
SEPTEMBER 17-21, 2023

Closing the Gap: Combining numerical and analytical methods to predict gate closures

ADRIAN STRAIN AND BRENNAN SMITH, HDR

ABSTRACT

Gates play a critical role in the operation and maintenance of Hydropower facilities. A recent potential failure mode analysis identified runaway turbine operation as a credible potential failure mode. Therefore, demonstrating adequate gate closure during this potential failure is critical to the turbine shutdown process. Seating the gate requires the overcoming multiple forces, which may vary by orders of magnitude during the closure. Analytical methods and Computational Fluid Dynamics (CFD) were combined to determine the forces the gates and crane system would experience during a potential runaway turbine failure mode. The CFD model supplied the hydrodynamic forces the system would need to overcome during closure and the analytical methods showed the potential failure points of the gates. A repeatable analysis process was developed using previous physical and numerical models allowing the multiple unique turbine and intake configurations to be evaluated. The portfolio analysis allowed the utility to take a systems approach, understand various risk scenarios, and prioritize actions to address the areas of highest risk.

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Levee Performance during Floods: A Finite Element Method Study of Transient Hydrodynamic Conditions

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ABSTRACT

The study focuses on targeting the Finite Element Method (FEM) analysis of a levee section under transient condition simulation. The primary objective is to investigate the behavior of the levee section during different stages of a flood hydrograph. To achieve this, the study considers different stages of the hydrograph, including rising, peak and recession stages. The FEM analysis is used to model the levee section and simulate the transient condition under each stage of the flood hydrograph. The analysis results are then used to evaluate the performance of the levee section, including the changes in deformation, pore water pressure and stability during the flood event. The study findings will help improve the understanding of levee behavior under different stages of a flood hydrograph, which can contribute to the development of more effective flood protection strategies.

I.INTRODUCTION

Levees are important flood protection structures that have been providing a safe shelter for decades. While levees are effective flood protection structures, it is important to note that they are not immune to failure. They can be vulnerable to overtopping, erosion, and breaches during extreme flood events or when not properly designed, constructed, or maintained (1). Without regular inspection, levees can bring about unanticipated catastrophes and damage to local communities. Effective levee health monitoring is vital for the early detection of potential issues to reduce or stop damage caused by levee failure (2). Finite Element Methods (FEM) have emerged as invaluable tools in the field of levee prediction, offering a comprehensive and sophisticated approach to modeling and analyzing the complex behaviors and interactions that occur within and around levee systems. Finite Element Methods involve breaking down a complex problem into smaller, more manageable elements or regions and then applying mathematical equations to model the behavior of each element. This allows for a detailed analysis of stress, strain, deformation, and other critical factors affecting the levee's stability and performance. By continuously assessing factors such as seepage rates, soil moisture, subsurface movements, and slope stability, engineers and levee operators can assess the potential risks and vulnerabilities, identify critical areas, and prioritize maintenance or remedial actions. Levee health monitoring aids in the development of effective maintenance and maintenance planning strategies. By monitoring changes in levee conditions over time, authorities can establish maintenance schedules, prioritize repair or reinforcement efforts, and allocate resources efficiently.

Transient hydrodynamic condition in levees refers to a temporary or short-term state in which there are dynamic changes in water flow patterns and hydraulic conditions within the levee system. These changes can occur due to factors such as fluctuating water levels, varying flow velocities, wave action, or other transient hydrological phenomena. During transient conditions, the levee may experience increased hydraulic pressure, wave action, or overtopping, which can pose challenges to its integrity and flood protection capabilities. In our study, levee health monitoring during different stages of flood hydrograph is analyzed to compare the deformation, pore water pressure,

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stability and factor of safety. Finite Element Method (FEM) analysis was used for our study. The FEM is well-suited for the analysis of transient hydrodynamic conditions in levees (3). Monitoring and understanding transient hydrodynamic conditions are crucial for assessing the performance and response of levees to dynamic water flow and for implementing appropriate design, maintenance, and risk mitigation strategies.

II.STUDY AREA

The area selected for this study is located on a reservoir of the Pearl River between Madison and Rankin counties in Mississippi. The presence of expansive soil in Mississippi has significant impacts on the development and construction practices. Expansive soil undergoes notable volume changes in response to variations in moisture content. As a response to moisture content variation, expansive soil shows shrink-swell behavior. This poses challenges for construction. The selected levee for this study is surrounded by the Ross R Bernett reservoir, which gives the soil underneath it a constant supply of water (4). Expansive soil characteristically undergoes significant volume changes and can result in deformation of the levee structure. The soil's expansion and contraction cycles induce movements and strains within the levee, causing deformations such as settlement, heave, or lateral movement. These deformations can lead to changes in the levee's geometry, alignment, and cross-section, affecting its overall stability and flood protection capabilities (5).



Figure 1: Study Area of Levee Performance analysis during Floods

It is important to note that the study area connects Madison and Rankin Counties with US Highway Road 43. The volumetric changes associated with expansive soil can induce cracking and erosion within the levee. Swelling during wet periods can lead to internal pressure buildup and subsequent cracking of the soil can provide pathways for water infiltration (6). Infiltration can exacerbate soil erosion and weaken the levee's structural integrity, potentially

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leading to breaches or seepage. It may develop cracks and erosion on the road of US 43, which is an important connecting road used every day by a significant number of people. The cross-section of the road was taken from Google Earth Pro and then analyzed in Plaxis 2D software using the flood height of the flood that occurred on Aug 26, 2022, to understand its impact on the levee.

III.METHODOLOGY

(i) Finite Element Method (FEM): FEM is a powerful numerical technique widely used in engineering and scientific applications for its ability to handle complex geometries (7). In our study, FEM was used to analyze the effect of water level on levee slope deformation, soil suction and stability. In PLAXIS, deformation refers to the changes in shape, position, or dimensions of soil and structures under the influence of applied loads, boundary conditions, and other factors.

In Plaxis 2D, the geometry of the profile, including its shape, dimensions and layers of soil is added. The slope of the dam structure is then divided into smaller elements or nodes, creating a mesh. Material properties, boundary conditions, and the water level are applied as inputs in the model. From different analysis types, static analysis is typically used for slope deformation, soil suction and stability, where the equilibrium state of the slope under the applied loads is determined. PLAXIS uses the principles of finite element analysis to solve the governing equations of soil mechanics. The software iteratively calculates the deformation of each element based on the applied loads and material properties. The methodology can be described in two sections. A flood hydrograph was generated in the first section, and in the second section, a FEM model was developed to numerically analyze different parameters.

Soil deformation refers to the changes in shape, position, or volume of soil masses in response to external forces, loading, and other factors. Deformation analysis in PLAXIS involves studying how different components of a geotechnical system, such as soil layers, retaining walls, foundations, or slopes, respond to external forces and changes in the surrounding environment. The FEM considers vertical, horizontal, shear, rotational and volumetric deformation and calculates how each element deforms in response to applied loads and boundary conditions. Soil suction, also known as matric suction or pore water suction, is a measure of the tension or negative pressure within the pore spaces of a soil mass due to the presence of water. It represents the force by which water is held in soil against the influence of gravity and can significantly affect the mechanical behavior of unsaturated soils. The SWCC is a fundamental relationship that describes how the volumetric water content of a soil changes with respect to soil suction (8). PLAXIS allows users to input or define the SWCC for the specific soil being analyzed. The SWCC provides critical information about how the soil's water retention and drainage properties change as the soil becomes more or less saturated.

(ii) Flood Hydrograph Generation:

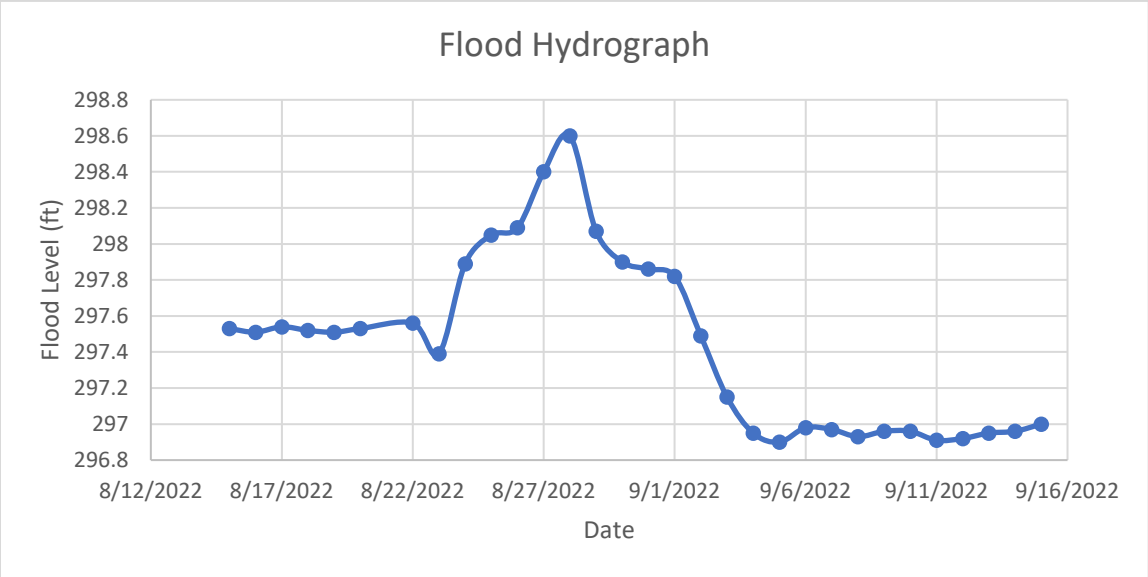


Figure 2: Flood Hydrograph of Ross R Bernett Reservoir (from 8/15/2022 to 9/15/2022)

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A flood hydrograph is a graphical representation of the flow rate or discharge of a river or streams over time during a rainfall event or flood. It serves as a valuable tool for analyzing and predicting flood events, providing insights into their characteristics and behavior (9). For the study area, the flood hydrograph was generated using open-source data provided by the US Army Corps of Engineers (USACE). The data encompassed a month-long period from 15 August 2022 to 15 September 2022. This dataset allowed for the examination of the flow rate and discharge patterns during that specific time frame. Notably, a significant flood event occurred on 26 August 2022. During this event, the water level rose by approximately 1 foot, indicating a substantial increase in the flow rate and discharge of the river or stream. The flood hydrograph captured the temporal variations in the water level and discharge throughout the entire flood event.

To ensure accurate analysis using real data, the flood hydrograph data was subsequently incorporated into the FEM model. The FEM model utilized the actual flood hydrograph to simulate and predict the behavior of the study area during the flood event. This integration of the flood hydrograph data with the FEM model facilitated a more precise analysis and evaluation of the flood's impact on the area, enabling insights into potential flood risks, hydraulic performance, and floodplain management.

(iii) Model Development: The FEM program Plaxis 2D was used to conduct the flow analysis and stability of the slope (10). A 15-node triangular element was used. The soil parameters presented in Table 1 were used to perform the numerical analysis using Plaxis 2D.

TABLE 1

Soil Parameters

Parameter	Symbol	Unit	Unweathered Yazoo Clay	Weathered Yazoo Clay
Bulk Unit Weight	γ_{unsat}	lbf/ft^3 (pcf)	126.6	128.5
Saturated Unit Weight	γ_{sat}	pcf	126.6	134.3
Modulus of Elasticity	E	lbf/ft^2 (psf)	198.4E+03	148E+03
Poissons Ratio	N	-	0.25	0.3
Cohesion	c	psf	100	30
Friction Angle	Φ'	degree	15	8
Permeability	k_x=k_y	ft/day	8.60E-03	8.60E-03
Residual Saturation	S_{res}	-	0.1283	0.1283
Van Genuchten Parameters	g_a	1/ft	0.01494	0.01494
	g_n	-	1.08	1.08
	g_l	-	0.074	0.074

The levee section was determined using Google Earth Pro. The slope of the levee was considered to be 10H:1V. Initial water level was considered to be 297.53 ft according to the USACE website. Stage flood hydrograph from 15 August 2022 to 15 September 2022 was imported in water levels segment of attributes library as a time dependent function. In the stage construction section, each phase was analyzed at a 2-day interval. The boundary condition of the slope was considered as a mixture of seepage and closed behavior.

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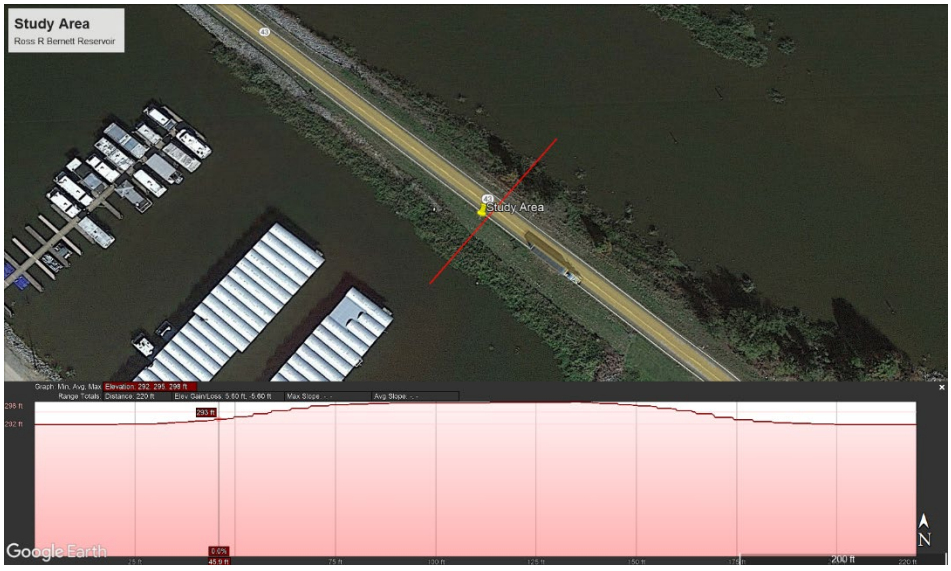


Figure 3: Elevation profile of the Study Area from Google Earth Pro

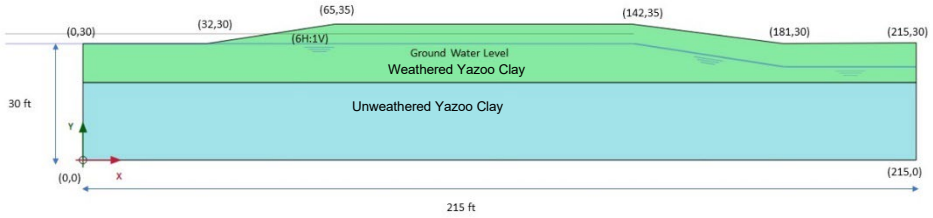


Figure 4: Elevation Profile used in Plaxis 2D Model.

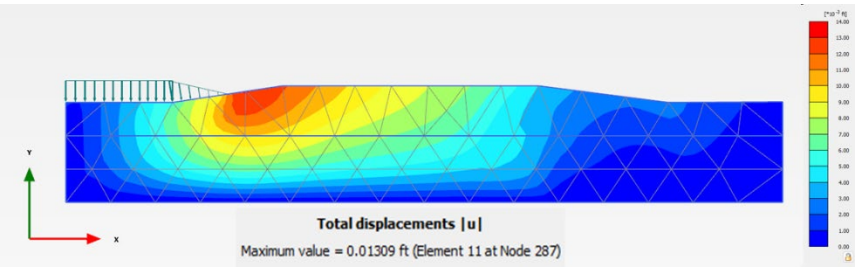
The length of the dam section was taken as 215 ft and the height of the total section was taken to be 35 ft. The slope was 6H:1V . Upper layer of the soil was considered unweathered Yazoo clay and lower section of the layer was considered as weathered Yazoo clay . The ground water level was taken 5 ft below the surface level.

IV. RESULT

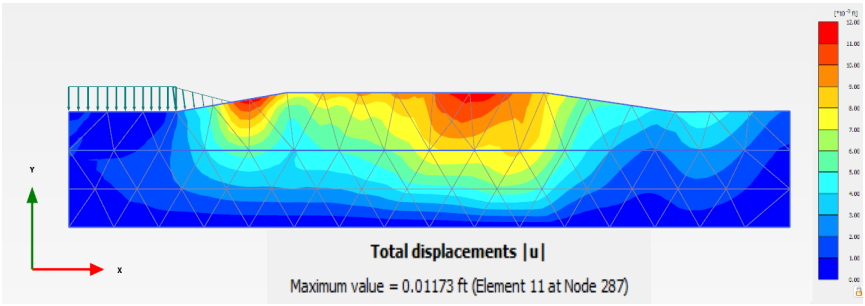
The flood level of the reservoir was at its maximum level of 298.6 ft for two days after the flood began on 28 August 2022. the minimum flood level was observed on 5 September 2022 with a flood level of 296.9 ft. The result is compared for four scenarios: (i) initial state (15 August 2022) (ii) maximum flood level state (28 August 2022) (iii) minimum flood level state (5 September 2022) and (iv) final state (15 September 2022). Initial state and final state are considered as the start and end of the analysis, respectively.

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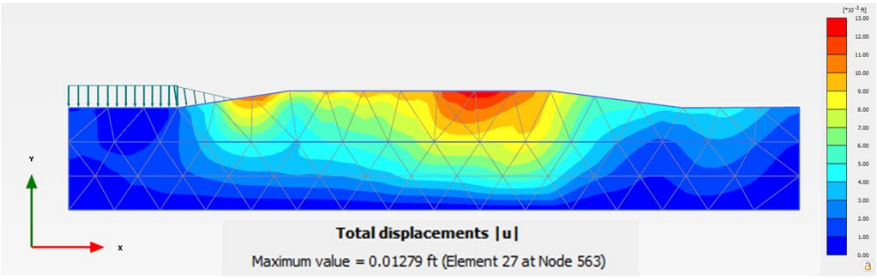
Deformation: The flood level increased with the precipitation and reached maximum on August 28, 2022, and then reached a minimum on September 5, 2022. Deformation in the initial stage was higher than deformation in the maximum flood level state. After the maximum flood level state, deformation level started to increase. Deformation in the four scenario stages is provided below.



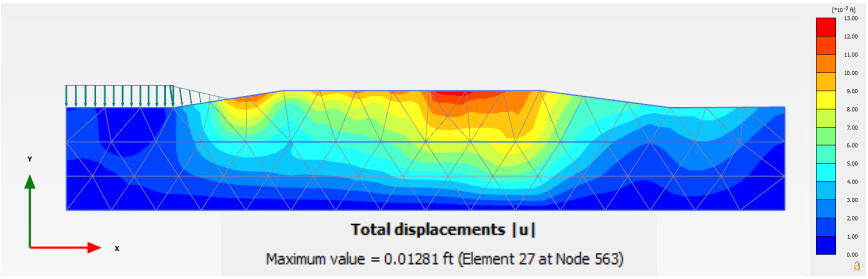
(a)



(b)



(c)



(d)

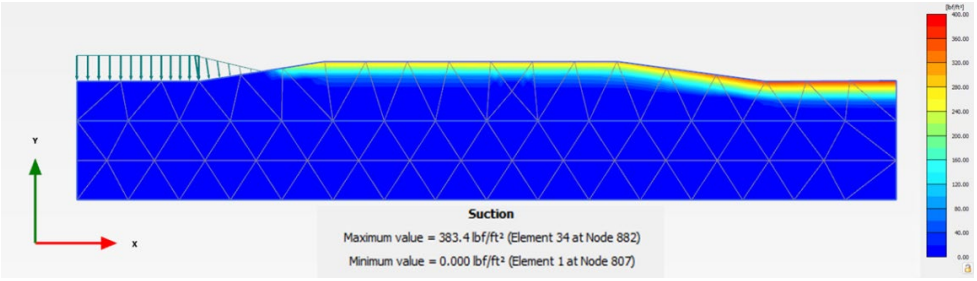
Figure 6: Deformation in (a) Initial Phase (b) Maximum Flood Level Stage (c) Minimum Flood Level Stage (d) Final Stage

As the flood level reaches its maximum, the deformation decreases which seems to be contrary to what might be expected. But initially, when rainfall infiltrates the soil, it may increase the pore water pressure and reduce effective stress, which can lead to increased displacement (11). However, as the soil becomes fully saturated, its ability to hold additional water diminishes. This reduction in pore space for water to occupy can lead to an increase in effective stress over time, potentially resulting in a decrease in soil displacement. But as the flood level reaches its minimum, the water pressure acting on the soil diminishes. This reduction in water pressure leads to the dissipation of excess pore water pressure within the soil. As the effective stress begins to increase, the soil starts

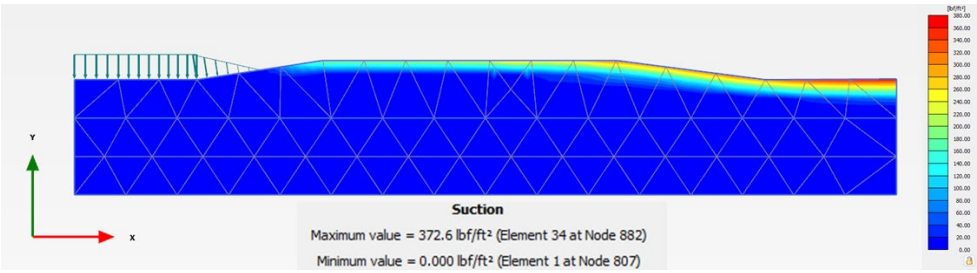
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to regain its strength and stiffness. As the floodwater recedes, the soil undergoes drainage and drying. This drainage process leads to an increase in effective stress as the excess water is removed from the soil pores. However, the drying process also led to soil shrinkage and settlement, resulting in additional deformation (12).

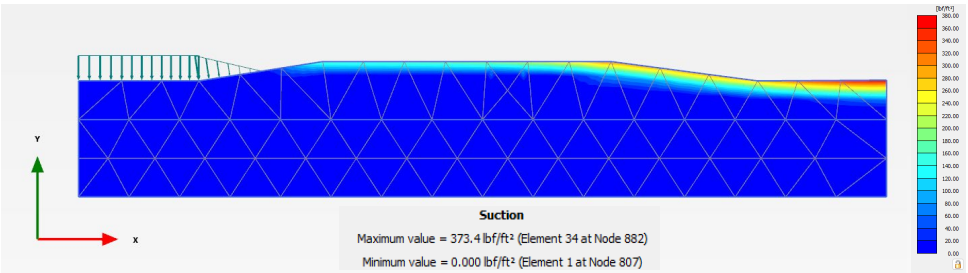
Suction: Suction, in the context of slope stability, refers to the negative pore water pressure or tension developed within unsaturated soils. Suction reduces the pore water pressure within the soil matrix. As a result, the effective stress acting on the soil particles increases. Higher effective stress enhances the shear strength and resistance of the soil, making it less susceptible to failure. However, as the water level rises, the reduction in suction weakens the levee's stability, potentially increasing its susceptibility to slope failure.



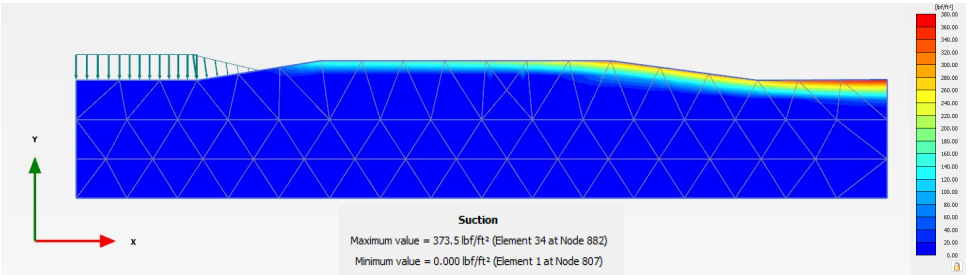
(a)



(b)



(c)



(d)

Figure 5: Suction in (a) initial state, (b) maximum flood level, (c) minimum flood level, (d) final state

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In the initial case, the suction was the highest 383.4 lb/ft2. During maximum flood level it reduced to 372.6 lb/ft2 and then increased again as the effect of flood started reducing. So, the levee was in the most vulnerable position during the flood.

Factor of Safety: Factor of Safety is a measure of how close a slope is to its failure condition and is calculated by dividing the available shear strength of the soil by the mobilized shear stress acting on the slope. In the analysis, the factor of safety drastically reduced after the flood. In theory, if the factor of safety is higher than 1, it means that the available shear strength is greater than the mobilized shear strength acting on the slope. Slope performance is decreasing with the event of flooding.

TABLE 2

Analysis Results

State	Total Displacement (ft)	Suction (psf)	Factor of Safety
Initial State	0.01309	383.4	4.749
Maximum Flood Level State	0.01173	372.6	4.632
Minimum Flood Level State	0.01279	373.4	4.596
Final State	0.01281	373.5	4.581

From the overall analysis it can be seen that flooding makes the slope vulnerable to failure. Deformation, suction, factor of safety all of the parameters get affected by the flood. Deformation seems to decrease in maximum flood level state because of the development of pore water pressure and it increases gradually with time again. Suction has an inverse relationship with flood level. Suction decreases with increasing flood level and vice versa. One noticeable fact in this analysis is that the factor of safety gradually decreases irrespective of flood level. This variation in suction and displacement affects soil stability. The soil's shrink-swell characteristics introduce fluctuations in the volume of voids within the soil matrix. Soils with higher void ratios exhibit reduced strength compared to those with lower void ratios. This ongoing alteration in void spaces leads to a gradual decline in soil strength, which accumulates progressively over time. In the context of changing climate patterns, elevated precipitation levels contribute to intensified impacts on the soil (13). Each instance of heavy precipitation contributes to a gradual reduction in the soil's strength, causing a gradual deterioration of its factor of safety. This cumulative effect leaves levees in a precarious state of vulnerability. As rainfall increases due to climate change, the soil undergoes repeated cycles of strength reduction, amplifying the risk associated with slope stability. The ongoing loss of strength undermines the structural integrity of the soil, magnifying concerns about the safety and stability of levees.

V. LIMITATIONS

For conducting a thorough analysis of levee performance, accurate cross-section data is crucial. In this study, the cross-section of the levee was obtained from Google Earth Pro software, which provides a convenient way to visualize the topography. However, it is important to note that using the actual cross-section would have yielded more accurate results. The availability of precise measurements and field data would have enhanced the reliability and precision of the analysis. Future studies should consider incorporating more precise cross-sectional data to improve the accuracy of the findings.

While the analysis in this study focused on a one-month period, it is essential to consider the long-term loading of water on the levee and the effects of consecutive flooding events. By incorporating historical flooding data and analyzing the levee's response to increasing water levels, a more comprehensive understanding of its stability can be obtained. Examining the maximum water level that could potentially cause the levee to fail provides valuable

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insights into its performance and vulnerability. To ensure a complete assessment, it is recommended to include all previous flooding events and continuously monitor the levee's condition during subsequent flood events (14).

During the analysis, the factor of safety (FoS) dropped as low as 0.3 during a specific flooding event. This observation highlights the significance of the FoS as a critical parameter in assessing levee stability. A FoS below the acceptable threshold indicates potential instability and raises concerns about the levee's capacity to withstand the hydraulic forces acting upon it. The findings emphasize the need for remedial measures or structural enhancements to improve the levee's stability and overall performance.

To gain further insights into the levee's behavior and potential risks, it is recommended to conduct an electrical resistivity imaging (ERI) test. This test provides valuable information about the position and characteristics of the perched water zone within the slope. Understanding the distribution of water within the levee can help identify potential areas of saturation, seepage, or localized instability. Incorporating ERI test results into the analysis enhances the accuracy of the assessment and enables a more informed decision-making process regarding slope stability and mitigation measures.

VI.CONCLUSION

Levee performance evaluation is a comprehensive process that may involve multiple analyses, including static and dynamic conditions, long-term performance assessments, and consideration of environmental and climate change factors. The FoS is a key factor in this evaluation, providing a quantitative measure of levee stability and aiding in decision-making for levee management and potential risk reduction measures.

In summary, this study highlights the importance of a comprehensive approach to levee performance analysis, encompassing accurate data, long-term loading considerations, continuous monitoring, and the incorporation of multiple flooding events. The flood height for this analysis was 3 ft and this caused the FoS to reduce by 0.3. if the levee is not maintained these small floods will keep reducing the factor of safety and the slope will fail. By incorporating these factors into the assessment, a more robust evaluation of levee performance can be achieved, leading to improved decision-making processes, enhanced risk management, and the development of effective measures to ensure the stability and resilience of levee structures.

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Dr. Khan is a technical member of ASCE GI Unsaturated Soil, ASCE GI Shallow Foundation, AKG-20 TRB Standing Committee on Soil and Rock Properties and Site Characterization, and AKG-40 TRB Standing Committee on Mechanics and Drainage of Saturated and Unsaturated Geomaterials.

Dr. Khan received the 2018 Engineer of the Year award from MS ASCE Section, and 2019 Engineer of the Year award from ASCE Region 5. He was awarded the Career Award from the National Science Foundation in 2021 on “Climate Resilient Landslide Repair on Expansive Soil Using Vetiver Grass”. Dr. Khan is a licensed engineer in the State of Texas and the State of Mississippi.

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DAM SAFETY PALM SPRINGS
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Using State-of-the Art Geophysical Technologies to Assess Earthen Embankments

LINCOLN STEELE, MAJID MIRZANEJAD, AND VICKI SCHARNHORST, TETRA TECH INC.

ABSTRACT

The current U.S. dam and levee inventory which includes over 14,000 high hazard non-federal dams, averages over 70 years of age but providing an accurate assessment of aging infrastructure can prove difficult. Structure location, site access, geologic setting, and size often presents complications for conducting accurate condition assessments or geohazard characterizations. Recently developed geophysical methods, including Semi-Airborne Electromagnetics, 3D Seismic SPT, Full Waveform Tomography, and Fiber Optic Sensing Technologies, along with revised applications of standard geophysical methods, provides cost-effective, accurate approaches for levee and dam geo-characterization. Phasing various geophysical methods (types, scale of survey and specific applications) can reduce uncertainty in the characterization process and focus existing and more traditional methods on targeted problem areas. Further, the use of Unmanned Aerial Vehicles (UAVs) to support survey and geophysical instrumentation can overcome several difficulties in dam and levee characterization including ground access and ensuring consistent instrument height over rough or steep terrain. This targeted land-based geophysical surveys can then the specific and critical data required for proper engineering assessments. New geophysical methods take advantage of standard geotechnical data and methods to maximize the information gained over the life of the project. These new geophysical applications and technologies are cost effective, reduce uncertainty and risk in dam and levee characterizations, and allow prioritization of the survey effort to take advantage of what information is being learned as the work progresses.

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