Potential Failure Mode Analysis and Risk Assessment Report

INDIAN BROOK RESERVOIR DAM
NATDAM NO. VT000055

PFMA Workshop Date: June 12, 2023
Risk Assessment Workshop Date: September 27, 2023
GZA File No. 01.0175988.00

PREPARED FOR:
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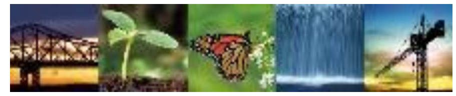

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Vermont Dam Safety Program
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Montpelier, Vermont 05620-3510

Attn: Mr. Benjamin Green, P.E. | Section Chief – Dam Safety Engineer
Mr. Andrew Sampsell, P.E. | Dam Safety Engineer

Re: Potential Failure Modes Analysis and Risk Assessment Report
Indian Brook Reservoir Dam – VT000055

Dear Mr. Green and Mr. Sampsell:

GZA GeoEnvironmental, Inc. (GZA) is pleased to present this Potential Failure Modes Analysis (PFMA) and Risk Assessment Report for the Indian Brook Reservoir Dam (VT000055) in Essex, Vermont. This report summarizes our PFMA Workshop conducted on June 12, 2023, as well as the follow up Risk Assessment conducted on September 27, 2023 as part of the High Hazard Potential Dam Risk Assessment program.

We appreciate the opportunity to provide the Vermont Department of Environmental Conservation with dam engineering consulting services. Please do not hesitate to contact us if you have any questions or would like to discuss this report or the assignment in general.

Sincerely,
GZA GeoEnvironmental, Inc.

Kevin F. Finn, P.E.
PFMA and Risk Facilitator

Derek J. Schipper, P.E.
Project Manager

James P. Guarente, P.E. (MA)
Principal-in-Charge

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1.0 INTRODUCTION & BACKGROUND

1.1 PURPOSE

GZA GeoEnvironmental, Inc. facilitated and performed a Potential Failure Mode Analysis (PFMA) Workshop and Risk Assessment for the Indian Brook Reservoir Dam (VT000055) on the behalf of the Vermont Department of Environmental Conservation – Water Investment Division (VTDEC), as part of an overall program to better understand the dam safety risk profile for this high hazard potential dam under the jurisdiction of the VTDEC.

Many agencies which oversee and regulate dams and water retaining structures have published guidelines for performing PFMA and Dam Safety Risk Assessments. Although the terminology employed by the different regulations may vary slightly, the overall process, documentation and interpretation of the potential failure modes, and their ensuing risks imposed on dam safety, are generally similar. For this exercise, the PFMA Workshop and the Risk Assessment were performed in general accordance with Chapter 17 (Potential Failure Mode Analysis) and Chapter 18 (Level 2 Risk Analysis) of the Federal Energy Regulatory Commission's Engineering Guidelines for the Evaluation of Hydropower Projects (FERC, 2021). It is important to note that although the FERC document references "hydropower" projects, the guideline is equally applicable to non-powered dams. The documents are available free of charge from the FERC's website, and as such have not been attached to this report.

1.2 KEY REFERENCE DOCUMENTS

The following key references were made available by the VTDEC and provided to the PFMA Workshop attendees prior to the workshop, and served as critical references during the PFMA Workshop and Risk Assessment:

1. Indian Brook Dam Site Location, Plan, and Sections.
2. (DuBois & King 2012) Dubois and King, Inc. October 18, 2012. Technical Memorandum Indian Brook Reservoir Dam Post Earthquake Inspection.
3. (FERC 2021) Federal Energy Regulatory Commission. 2021. Engineering Guidelines for the Evaluation of Hydropower Projects, Chapter 17 – Potential Failure Mode Analysis, December 2021.
4. (G & Underwood 1988) G & Underwood engineers, Inc. September 1988. Indian Brook Dam Engineering Report.
5. (GZA 2023) GZA GeoEnvironmental, Inc. May 9, 2023. Visual Inspection Report East Long Pond Dam (See Appendix B).
6. (Parks & Recreation Department 2011) Parks and Recreation Department of Essex Junction. January 2011. Emergency Action Plan (EAP) East Long Pond Dam.
7. (USACE 1980) United States Army Corps of Engineers New England Division. June 1980. Richelieu River Basin Essex Junction, VT Indian Brook Dam VT00055.
8. (VTDEC 2022) Vermont Department of Environmental Conservation. 2022. Indian Brook Dam Safety Inspection Report.

The following additional key references were made available following the PFMA Workshop, and served as critical additional reference during the Risk Assessment:

9. (GZA 2023) GZA GeoEnvironmental, Inc. December 15, 2023. Hydrologic and Hydraulic Analysis (including dam breach inundation maps) (See Appendix C).
10. (GZA 2023) GZA GeoEnvironmental, Inc. June 29, 2023. Stability Evaluation, auxiliary and primary spillway (See Appendix D).

Documentation of prior PFMA sessions or Risk Assessments, if any had occurred, were not discovered at the time of this report.

2.0 DESCRIPTION OF DAM AND OTHER KEY FEATURES

2.1 NARRATIVE

The Indian Brook Reservoir Dam was constructed in 1957 based on a design by Whitman and Howard Engineering. The dam impounds the Indian Brook Reservoir, which was originally used for water supply purposes; currently, the Reservoir is used strictly for recreation. The Town of Essex owns and maintains the dam. The dam is accessible off Indian Brook Road in Essex, Vermont.

The Dam is a 238-foot-long concrete gravity dam with a structural height of 31 feet. From left (looking downstream) to right, the Indian Brook Dam consists of the following water retaining structures:

- A 43-foot-long concrete principal spillway, with a crest elevation of 530 feet. The spillway has a vertical upstream face, approximate 4-foot-wide crest, and a 3H:4V sloping downstream face. A steel pedestrian bridge spans over this section, from the left abutment to the gate chamber, with a reported low-chord elevation of 533.0 feet.
- A 10-foot-long by 9-foot-wide gate chamber, cast integrally with the spillway. The gate chamber houses three 12-inch diameter inlet ports (elevations 505, 510 and 520) regulated by slide gates and protected by bar screens. The original design included two outlet ports; however, it is reported that the gates have not been operated for many years, and the outlet pipe is not visible at the toe of the spillway.
- A 70-foot-long concrete ogee-style principal spillway, with a crest elevation of 530 feet. Similar to the first spillway section, this section has a vertical upstream face, approximate 4- foot-wide crest, and a 3H:4V sloping downstream face.
- A 125-foot-long auxiliary spillway. The auxiliary spillway is similar in design to the two sections of the primary spillway, with a crest elevation 0.5 foot higher (approximate elevation 530.5 feet).

The Indian Brook Reservoir Dam remains true to its original structure; however, it has had some surficial concrete repairs along horizontal joints in the downstream spillway face in 2003 to address on-going spalling and concrete deterioration problems. These 2003 repairs also included grouting the leaking joints and bedrock foundation and the installation of reinforcing bars in the drill holes used to inject the grout. These historical issues at the site have continued to redevelop since the 2003 repairs with extensive spalling along the lift joints of the downstream principal and auxiliary spillway faces. Vegetation present within these larger spalls indicate that seepage through both spillway faces may have redeveloped.

The dam is currently classified as having a <u>HIGH</u> hazard potential by the Vermont DEC, as failure or misoperation is likely to cause loss of human life. Hazard Potential is not a reflection of the integrity of the Project, but an indication of the consequences of a hypothetical failure of a portion of the water retaining structures.

The dam is currently rated as being in <u>POOR</u> condition. This is defined by the Vermont DEC as either (a) "a dam safety deficiency is recognized for loading conditions which may realistically occur; remedial action is necessary" or (b) uncertainties exist as to critical analysis parameters which identify a potential dam safety deficiency. As such, further investigations and studies are necessary. The results of the PFMA and Risk Assessment workshops presented herein identify the necessary investigations and studies as risk reduction measures.

2.2 KEY PROJECT DATA

Table 2-1 presents a list of selected data regarding key elevations and other key features of the Project. The data was referenced by the group when developing and categorizing the PFMs. The elevations and dimensions listed in ***Table 2-1*** were obtained primarily from the 2022 VTDEC Dam Safety Inspection Report.

The generally used project vertical datum is the National Geodetic Vertical Datum of 1929 (NGVD29), which is used in this report unless otherwise noted. Some documents reference the North American Vertical Datum of 1988 (NAVD88), which is a more current vertical datum.

Datum conversion: NAVD88 = NGVD29 – 0.4 feet.

Table 2-1 – Key Project Data

Overview		
Dam Name	Indian Brook Dam	
Owner	Town of Essex	
Primary Construction	Concrete Gravity Dam	
Original Construction (year)	1957	
Primary Use	Recreation (former water supply)	
Length (ft)	238	
Maximum Height (ft)	31	
Last VTDEC-DS Inspection Date	9/8/2022	
Last VTDEC-DS Inspection Condition Rating	Poor	
Hazard Potential Classification	High	
Construction History	Repair work along the horizontal joints in the downstream face in 2003 which has continued to deteriorate	
Breach/Overtop	None	
Hydraulic Design		
Elevation Datum	NGVD29	
Streambed at Toe of Dam Elevation (ft)	504	
Normal Pool Elevation (ft)	530	
Normal Winter Pool (if different) Elevation (ft)	Unknown	
100-Year Flood	530.7 ft	(see GZA H&H Analysis, Appendix C)
Inflow Design Flood (IDF)	Probable Maximum Flood, PMF	(see GZA H&H analysis, Appendix C)
IDF Elevation (ft)	532.5 ft	(see GZA H&H Analysis, Appendix C)
IDF Inflow (cfs)	3,100 cfs	(see GZA H&H Analysis, Appendix C)
IDF Outflow (cfs)	3,040 cfs	(see GZA H&H Analysis, Appendix C)
Spillway	**Principal Spillway**	**Auxiliary Spillway**
Construction	Concrete Weir (ogee)	Concrete Weir (ogee)
Length (ft)	113	125
Slope (ft:ft)	3H:4V	3H:4V
Crest Elevation (ft, NGVD29)	530	530.5
Height of Flashboards ("0" if none)	0	0
Normal Pool Freeboard (ft)	0	0.5
IDF Pool Overtopping (ft)	2.5	2.0

Table 2-1 – Key Project Data (continued)

Low Level Outlets	
Description	Three 12-inch diameter inlets regulated by slide gate, with 2-inch clear opening bar screens. Two gated outlets with 4-inch mesh screens.
Operator Location	At the concrete platform on the primary spillway, accessed via the pedestrian bridge.
Closure Mechanism	Hand-wheel and stem
High-Level Inlet Gate Centerline Elevation (ft)	520 (not operatable)
Mid-Level Inlet Gate Centerline Elevation (ft)	510 (not operatable)
Low-Level Inlet Gate Centerline Elevation (ft)	505 (not operatable)
Outlet Pond Drain	A 12-inch-diameter cast-iron pipe that enlarges into a 15-inch-diameter pipe and discharges into the downstream channel. The valve is not operable, and the discharge point is not visible.
Outlet to Water Treatment Plant	A 12-inch-diameter cast-iron pipe (discharge valve to the defunct water treatment plant; stem is cut flush with the floor of outlet structure and the gate stand has been removed).
Outlet Invert Elevation (ft)	504.5
Storage	
Drainage Area	787.2 acres, 1.23 square miles
Reservoir Area Normal Pool (acres)	48
Normal Storage Capacity (acre-ft)	1,084
Maximum Storage Capacity (acre-ft)	1,157

3.0 POTENTIAL FAILURE MODE ANALYSIS

3.1 PROCESS

The virtual PFMA Workshop was performed in general conformance with Chapter 17 of the FERC Engineering Guidelines. The Workshop was facilitated by Kevin F. Finn, P.E. of GZA with representatives of the Town of Essex, State of Vermont Dam Safety Program, and GZA. The members of the 2023 PFMA review team are listed in **Table 3-1** below:

Table 3-1 – Participant List for June 12, 2023 PFMA Review Meeting

Participant	Representing	PFMA Role	Title
Aaron Martin	Town of Essex	Participant / Owner	Public Works Director
Allyson Vile	Town of Essex	Participant / Owner	Parks & Recreation Director
Ben Green, P.E.	Vermont DSP	Participant / Regulator	Section Chief
Kevin F. Finn, P.E.	GZA	Facilitator	Senior Project Manager
Derek J. Schipper, P.E.	GZA	Subject Matter Expert	Senior Consultant
John DeLano, P.E.^{MA}	GZA	Subject Matter Expert	Senior Technical Specialist
Seth D. Krause, P.E.^{MA}	GZA	Notetaker	Project Manager

The objectives of the PFMA Workshop were as follows:

- Discuss the adequacy of Project Documentation.
- Lead a group discussion to identify, discuss, and develop Potential Failure Modes (PFMs) for the Project structures.
- Review possible risk reduction/instrumentation/surveillance opportunities for the identified PFMs.
- Propose categorizations for the PFMs identified during the Workshop.

The PFMA review session began with a review of the PFMA process and key PFM-related terminology. Of critical importance was establishing the definition for “failure”:

Failure:	Uncontrolled release of the reservoir, in whole or in part; or Inability of project features or components to perform their intended function; or Project features or components performing in an impaired or compromised fashion.	Which results in an adverse consequence.

After a review of the key project information, the workshop transitioned into a brainstorming session to develop a list of candidate PFMs. During the workshop, participants presented available and discussed Project documentation, and discussed updates on the status of various studies, analyses, or other action items that are in progress or have recently been completed. A postulated failure mode was considered to be viable, and advanced for further discussion, if a clear chain of events could be established to advance the PFM from a postulated flaw or defect through a successive chain of events leading to failure or breach of the dam.

The PFMA sought to identify the predominant means by which the project could suffer a "failure," either through degradation of the structures, extreme loading conditions, or human and environmental factors.

After the initial brainstormed list of PFMs was developed, the Facilitator led the PFMA Team in performing a preliminary grouping by defined "Disposition" categories. The Disposition provides a common terminology to summarize the severity of a PFM, to determine which PFMs should be highlighted from a life-safety perspective (Urgent and Credible), which are focused on financial impacts of a failure (Financial/Damage State and Asset Management), which are plausible, but determined to be very unlikely (Clearly Negligible and Ruled Out), and which require further study to accurately classify (Insufficient Information). A high-level definition for each Disposition is provided in **Table 3-2**.

The facilitator lead the PFMA Team in formally documenting and further developing the team's understanding for select PFMs, by recording adverse factors (studies, inspection observations, anecdotal evidence, and available historic records) which suggest failure to be more likely, positive factors which suggest failure to be less likely, and currently-implemented surveillance and monitoring provisions in place to observe for the theorized PFM. The team then documented potential risk-reduction measures, ranging from enhanced visual inspection to additional engineering studies and remediation design, which could be implemented to lower the risk associated with the PFM. GZA further developed the remaining PFMs following the Workshop. The completed summary tables are attached as **Table 2 – 2023 PFMA and Risk Summary Tables,** which is included in this report following Section 6.0.

3.2 IDENTIFIED PFMS AND DISPOSITIONS

During the 2023 PFMA Review Session, a total of 23 PFMs were considered and categorized. The identified PFMs were sorted, and assigned a numbering system of the format **IBR-SSS-##-L-DIS**, where …

IBR	Indian Brook Reservoir Dam
SSS	Three letter abbreviation for the primary affected structure
	SPL = Principal Spillway
	AUX = Auxiliary Spillway
	INT = Intake
	GNR = General / multiple structures
##	Sequential number when multiple PFMs were identified for a primary affected structure
L	Loading condition considered
	N = Normal Reservoir conditions
	F = Flood conditions, up to and including the PMF
	I = Winter loading conditions, including ice load
	E = Post-earthquake conditions
	S = Special loading conditions
DIS	Three letter abbreviation for the assigned Disposition, as defined in ***Table 3-2***.

3.3 SUMMARY OF PFMS

Table 1 – Summary of PFMs and Risk Driver Designations included after Section 6.0 of this report, provides a tabulation of the PFMs developed during the PFMA Workshop, and subsequently expanded upon during the Risk Assessment. Detailed tables documenting the development of each PFM are included in **Table 2 – 2023 PFMA and Risk Summary** Tables**.**

Table 3-2 – Potential Failure Mode Disposition Descriptions

Disposition		PFM Category Description	Assigned PFMs
URG	Urgent	Failure mode is physically possible and visual observation, data records, stability calculations or other information suggests that failure is either imminent or currently in progress.	*none*
CRE	Credible	Failure mode is physically possible, however visual observation, data records, stability calculations or other information do not suggest that failure is imminent or currently in progress.	IBR-SPL-01-IE-CRE IBR-SPL-03-F-CRE IBR-SPL-04-N-CRE IBR-AUX-01-I-CRE IBR-AUX-04-N-CRE IBR-INT-01-IE-CRE
FDS	Financial/ Damage State	Failure mode is physically possible and would not cause loss of life. The consequences are limited to the Owner's property and are projected to exceed $2 million in damages.	*none*
ASM	Asset Management	Failure mode is physically possible and would not cause loss of life. The consequences are limited to the Owner's property and are projected to be limited to less than $2 million in damages.	IBR-SPL-02-A-ASM IBR-SPL-06-F-ASM IBR-AUX-02-N-ASM IBR-INT-08-A-ASM
INS	Insufficient Information	Insufficient information is available to determine if the failure mode is physically possible or determine if an uncontrolled release of the reservoir and associated adverse conditions will develop. Additional studies are required.	IBR-INT-09-A-INS
NEG	Clearly Negligible	Failure mode is physically possible. However, failure mode is so remote as to be considered clearly negligible.	IBR-SPL-05-N-NEG IBR-AUX-03-F-NEG IBR-AUX-05-N-NEG IBR-INT-03-IE-NEG
R/O	Ruled Out	Failure mode determined not to be physically possible.	IBR-SPL-07-F-R/O IBR-INT-02-IE-R/O IBR-INT-04-N-R/O IBR-INT-05-A-R/O IBR-INT-06-A-R/O IBR-INT-07-A-R/O IBR-GNR-01-A-R/O IBR-GNR-02-A-R/O

4.0 RISK ASSESSMENT

4.1 PROCESS

The virtual Risk Assessment was performed in general conformance with Chapter 18 of the FERC Engineering Guidelines. The Workshop was facilitated by Kevin F. Finn, P.E. of GZA with representatives of the State of Vermont Dam Safety Program and GZA serving in the capacity of Risk Estimators. The members of the 2023 Risk Assessment team are listed in **Table 4-1** below:

Table 4-1 – Participant List for September 27, 2023 Risk Assessment Workshop

Participant	Representing	Risk Assessment Role	Technical Area
Ben Green, P.E.	Vermont DSP	Risk Estimator	Dam Safety
Andrew Sampsell, P.E.	Vermont DSP	Risk Estimator	Dam Safety
Kevin F. Finn, P.E.	GZA	Facilitator	Structural
Matthew A. Taylor, P.E. MA	GZA	Co-Facilitator	Geotechnical
Derek J. Schipper, P.E.	GZA	Risk Estimator	Geotechnical
John DeLano, P.E. MA	GZA	Risk Estimator	Structural / Geotechnical

The objectives of the Risk Assessment were as follows:

- Further group the PFMs to determine which PFMs are "potentially significant" (i.e. Risk Drivers), and which are important to retain from an overall Potential Failure Mode perspective, but do not significantly impact the overall dam safety risk for the facility.
- For the Risk Drivers, systematically evaluate the likelihood that the PFM would occur, and the consequence (life safety or economic), were the PFM to occur.
- After categorizing the group of Risk Drivers, summarize which PFMs have a risk profile that are generally above industry-acceptable limits, and as such are a key candidate for future mitigation.

Risk Screening

Prior to the Risk Assessment, the Facilitator reviewed the list of identified PFMs with dispositions of Credible and Financial Damage State (there were no PFMs identified as Urgent) and screened them into two categories: "Risk Driver PFMs" and "Non-Risk Driver PFMs." The screening was discussed with, and concurred by, the Risk Assessment Team at the beginning of the workshop.

In general, only the previously identified "Credible" PFMs were identified as "Risk Driver PFMs" and were advanced for Risk Assessment. However, a few "Credible" PFMs were screened out and categorized as "Non-Risk Driver PFMs," typically due to overlapping failure progressions with other Credible "Risk Driver PFMs." The list of PFMs advanced for risk analysis is shown in **Table 4-2** below. The results of the screening for all of the PFMs identified for the project are included in **Table 1 – Summary of PFMs and Risk Driver Designations** attached herein following Section 6.0.

Table 4-2 – List of Risk Driver PFMs

PFM No.	Controlling Loading Condition	PFM Description
IBR-SPL-01-IE-CR	Ice, Seismic	Sliding/overturning of the principal spillway along the foundation
IBR-SPL-03-F-CRE	Flood	Erosion/scour of the downstream toe of the principal spillway
IBR-SPL-04-N-CRE	Normal	Deterioration of the foundation of the principal spillway
IBR-AUX-01-I-CRE	Ice	Sliding/overturning of the auxiliary spillway along the foundation
IBR-AUX-04-N-CRE	Normal	Deterioration of the foundation of the auxiliary spillway
IBR-INT-01-IE-CRE	Ice, Seismic	Sliding/overturning of the intake along the foundation

Risk Estimation

Following PFM screening, the Risk Assessment Workshop transitioned into a systematic review of the selected Risk Driver PFMs. The Workshop attendees reviewed the PFM development, description, and evaluation factors for a particular PFM.

The Risk Estimators assigned a Failure Likelihood Category (FL1 as most remote, FL7 as nearly certain) to the PFMs. There are many methods that can be utilized to estimate a failure likelihood; the Risk Estimators typically utilized an approach of assigning a nodal likelihood to each step through the progression of the PFM and comparing the ensuing cumulative probability to the "descriptor/evidence" field in **Table 4-3**. To assist in this evaluation, Hydraulic Hazard Curves (presenting the annual exceedance probability for reservoir levels up to and beyond the IDF) and Seismic Hazard Curves (presenting the annual exceedance probability for seismic ground motions up to and beyond the peak ground acceleration) were developed; those curves are included in **Appendix A - Risk Workshop Handouts**.

Similarly, the Estimators assigned a Consequence Category (LS0 as no life loss, LS6 as life loss in excess of 10,000) to the PFMs. Refer to "descriptor/evidence" field in **Table 4-4** below for respective Consequence Category information. This estimation considered the incremental life loss that would be projected to occur in the event of the postulated dam failure, versus the baseline conditions. For example, if the expected life loss during the PMF with no dam failure is 10 and the expected life loss during the PMF with dam failure is 14, the incremental life loss considered by this Risk Assessment is 4 (LS2 category). To assist in this evaluation, a Population at Risk (PAR) and Loss of Life (LoL) evaluation was completed, utilizing the USACE's LifeSim software. Additional information regarding the development of this mode, and the tabulated PAR and LoL values, are included in **Appendix A - Risk Workshop Handouts**.

Table 4-3 – Failure Likelihood Categories

Cat.	Descriptor/Evidence	Magnitude Range
FL7	There is direct evidence to suggest it is certain to nearly certain that failure is imminent or extremely likely in the next few years.	Greater than 1 in 10
FL6	There is direct evidence or substantial indirect evidence to suggest that failure has initiated or is very likely to occur during the life of the structure.	1 in 10 to 1 in 100
FL5	There is direct evidence or substantial indirect evidence to suggest that failure has initiated or is likely to initiate.	1 in 100 to 1 in 1,000
FL4	The fundamental condition or defect is known to exist; indirect evidence suggests it is plausible; and key evidence is weighted more heavily toward "more likely" than "less likely."	1 in 1,000 to 1 in 10,000
FL3	The fundamental condition or defect is known to exist; indirect evidence suggests it is plausible; and key evidence is weighted more heavily toward "less likely" than "more likely."	1 in 10,000 to 1 in 100,000
FL2	The possibility cannot be ruled out, but there is no compelling evidence to suggest it has occurred or that a condition or flaw exists that could lead to initiation.	1 in 100,000 to 1 in 1,000,000
FL1	Several events must occur concurrently or in series to cause failure, and most, if not all, have negligible likelihood such that the failure likelihood is negligible.	Less than 1 in 1,000,000

Table 4-4 – Failure Consequence Categories

Cat.	Descriptor/Evidence	Incremental. Life Loss
LS6	Catastrophic direct loss of life can be expected due to little to no warning for very large population centers and/or limited evacuation routes	Greater than 10,000
LS5	Extremely high direct loss of life can be expected due to limited warning for very large population centers and/or limited evacuation routes	1,000 to 10,000
LS4	Extensive direct loss of life can be expected due to limited warning for large population centers and/or limited evacuation routes	100 to 1,000
LS3	Large direct loss of life is likely, related primarily to difficulties in warning and evacuating smaller population centers, or difficulties evacuating large population centers with significant warning time	10 to 100
LS2	Some direct loss of life is likely, related primarily to difficulties in warning and evacuating small population centers	1 to 10
LS1	Although life-threatening releases occur, direct loss of life is unlikely due to severity or location of the flooding, or effective detection and evacuation	Less than 1
LS0	No significant impacts to the downstream population other than temporary minor flooding of roads or land adjacent to the river	None expected

During the Risk Assessment workshop, the Facilitator first instructed the Risk Estimators to determine a Failure Likelihood and Consequence Category for the PFM independently of the rest of the group. After the preliminary results were tabulated, the Facilitator led a discussion to help the Estimators arrive at a consensus ranking.

Two of the Risk Driver PFMs were developed in this manner during the virtual workshop; the Estimators completed their assessment of the 4 remaining Risk Driver PFMs following the workshop. The Facilitator reviewed the results from this independent exercise, and determined that the responses had achieved reasonable consensus.

4.2 SUMMARY OF RISK ASSESSMENT

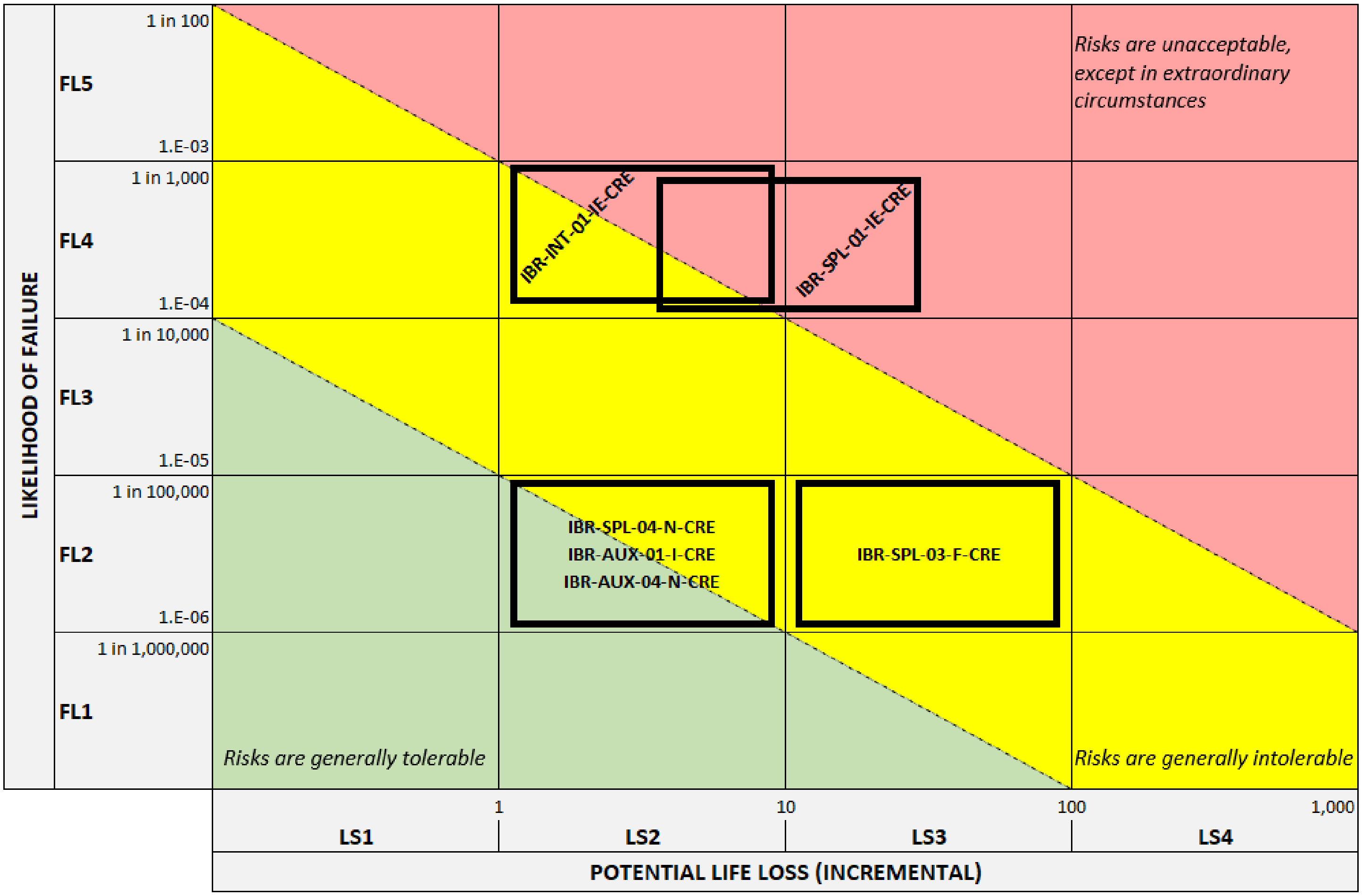
Table 1 – Summary of PFMs and Risk Driver Designations included after Section 6.0 of this report, provides a discussion of the Risk Assessment results for the identified Risk Driver PFMs, and a rationale for determining that the remaining PFMs are not considered to be Risk Drivers. Detailed tables documenting the development of each PFM are included in **Table 2 – 2023 PFMA and Risk Summary** Tables**.**

4.3 RESULTS OF RISK ASSESSMENT

A total of 6 PFMs were determined to be Risk Drivers. Refer to **Table 4-5** on the following page for the results of the Risk Assessment**.** A discussion of the estimated intensity of the risks is as follows:

- One of the Risk Driver PFMs plotted primarily in the “Risks are unacceptable” (Red) of the risk assessment matrix:
  - IBR-SPL-01-IE-CRE, Sliding/overturning of the principal spillway along the foundation, under ice or seismic loading.
- One Risk Driver PFM plotted between the “Risks are unacceptable” (Red) and the “Risks are generally intolerable (Yellow) zones of the risk assessment matrix:
  - IBR-INT-01-IE-CRE, Sliding/overturning of the intake along the foundation, under ice or seismic loading.
- One Risk Driver PFM plotted within the “Risks are generally intolerable” (Yellow) zone of the risk assessment matrix:
  - IBR-SPL-03-F-CRE, Erosion/scour of the downstream toe of the principal spillway under flood loading.
- Three Risk Driver PFMs plotted between the “Risks are generally intolerable” (Yellow) zone and the “Risks are generally tolerable” (Green) zone:
  - IBR-SPL-04-N-CRE, Deterioration of the foundation of the principal spillway under normal reservoir loading.
  - IBR-AUX-01-I-CRE, Sliding/overturning of the auxiliary spillway along the foundation under ice loading.
  - IBR-AUX-04-N-CRE, Deterioration of the foundation of the auxiliary spillway under normal loading.

Table 4-5 – Risk Assessment Matrix



5.0 MAJOR FINDINGS AND UNDERSTANDINGS

1. Indian Brook Reservoir Dam is considered a High Hazard potential dam, as failure is likely to result in loss of life.

2. The three PFMs classified as "unacceptable" or "generally intolerable" are generally associated with sliding stability of the principal spillway and intake structure. The primary observation made by the Estimators was associated with the uncertainty surrounding the rock-to-concrete sliding friction angle which was a conservatively assumed value in the stability calculations (GZA, 2023). Performing site specific investigations (i.e., subsurface explorations and laboratory testing) could support the use of a higher friction angle, which would improve the calculated sliding Factors of Safety, and thus justify reducing the likelihood of failure.

3. There is similar uncertainty regarding the base friction angle for the auxiliary spillway structure; further understanding the sliding friction angle will also afford risk mitigation for this structure.

4. There has been some inconsistency in past reporting regarding the lengths of the Principal Spillway (top elevation 530.0 feet) and the Auxiliary Spillway (top elevation 530.5 ft). After review of historic reports and drawings, and with confirmation from the Owner, it is noted that the Principal Spillway has a total length of 113 feet, with a 43-foot-long section to the left of the Intake and 70-foot-long section to the right of the intake. The Auxiliary Spillway is 125 feet long. The total spillway length is 238 feet.

6.0 CONCLUSIONS AND RECOMMENDATIONS

GZA offers the following conclusions and recommendations:

1. As part of this evaluation, 6 of the identified 23 Potential Failure Modes were determined to be potentially significant, were assigned the title of "Risk Driver" PFM, and further evaluated to assign a Failure Likelihood and Consequence. This should not be construed to suggest that the other 17 identified Potential Failure Modes are not important, or do not need to be considered. GZA recommends that the VTDEC, and the owner of the project, consider the identified Risk Reduction Measures for all 23 of the identified PFMs. Many of the identified Risk Reduction Measures, such as additional focus on features of note during future routine inspections, can be implemented with relatively minimal financial impact, and have a positive benefit to the overall safety of the project.

2. This risk evaluation does not replace or supersede regulatory requirements imposed by the VTDEC, does not replace or supersede the current dam safety performance monitoring program at the project, and does not provide an all-encompassing list of short term and long term dam safety rehabilitation and maintenance activities. The recommendations included herein are made specifically related to PFMs ("Risk Drivers") that were determined, based upon GZA's understanding of the project as of December 2023, to be critical to the overall Risk profile for the project.

3. <u>There were no Urgent PFMs identified by the Workshop</u>. The PFMs that were identified are considered typical for the age and construction methodology.

4. Two Risk Drivers PFM (IBR-SPL-01-IE-CR and IBR-INT-01-IE-CRE) plotted partially within the area of the Risk Matrix that is associated with risks that are "unacceptable except in extraordinary circumstances" and "generally intolerable, and one Risk Driver PFM (IBR-SPL-03-F-CRE) plotted wholly to the area associated with risks that are "generally intolerable." It is recommended that the Owner consider implementing Risk Reduction measures for each of these failure modes, to reduce the Likelihood that these failure modes develop.

   Table 6-1 presents a summary of the considered Interim Risk Reduction Measures (IRRMs) associated with these three target PFMs. The table also includes other PFMs that also have similar IRRMs. Therefore, implementing any of these measures to mitigate the risk associated with the highlighted PFMs will also reduce the risk associated with related PFMs. Refer to **Table 2 – 2023 PFMA and Risk Summary Tables** herein for more information.

   Of the tabulated RRMs, RRM-3 and RRM-4 are passive measures to observe for changing conditions, which may indicate that the PFMs are developing. RRM- 2 is also a passive measure, and in keeping with good dam safety practice, but will not significantly reduce the risk associated with the highlighted PFMS.

   GZA identifies RRM-5 as a potential candidate for refining our preliminary stability calculations completed under this assignment, which assumed conservative foundation properties based upon a lack of field and laboratory testing. Field studies to refine the stability assumptions, and consequent updated stability computations, would address each of the three highlighted PFMs, as well as mitigating the uncertainty surrounding several additional PFMs.

Table 6-1 – Select Interim Risk Reduction Mitigation Measures for Highlighted PFMs

Category	Measure	Affected PFMs	
		Risk Drivers[1]	Non-Risk Drivers
Risk Reduction Measures	1. Install bubblers during the winter months to prevent ice buildup against the dam	**IBR-SPL-01-IE-CRE** IBR-AUX-01-I-CRE **IBR-INT-01-IE-CRE**	-
EAP	2. Perform regular tabletop exercises and keep the EAP up to date	**IBR-SPL-01-IE-CRE** **IBR-SPL-03-F-CRE** IBR-SPL-04-N-CRE IBR-AUX-01-I-CRE IBR-AUX-04-N-CRE **IBR-INT-01-IE-CRE**	IBR-SPL-02-A-ASM IBR-SPL-05-N-NEG IBR-SPL-06-F-ASM IBR-AUX-02-N-ASM IBR-AUX-05-N-NEG
Inspections and Actions	3. Keep with formal inspection yearly basis, special event follow up inspections. In particular, note for visual sign of displacement of crest of spillway	**IBR-SPL-01-IE-CRE** **IBR-SPL-03-F-CRE** IBR-SPL-04-N-CRE IBR-AUX-01-I-CRE IBR-AUX-04-N-CRE **IBR-INT-01-IE-CRE**	IBR-SPL-02-A-ASM IBR-SPL-05-N-NEG IBR-SPL-06-F-ASM IBR-AUX-02-N-ASM IBR-AUX-05-N-NEG
	4. Perform a targeted visual inspection of the area downstream during periods of no flow over the spillway to look for signs of erosion/scour.	**IBR-SPL-03-F-CRE**	-
Follow Up Studies	5. Refined geotechnical field studies (such as borings or bedrock joint mapping downstream of section) could justify increased friction factors, which would benefit stability evaluation.	**IBR-SPL-01-IE-CRE** **IBR-SPL-03-F-CRE** IBR-SPL-04-N-CRE IBR-AUX-01-I-CRE IBR-AUX-04-N-CRE **IBR-INT-01-IE-CRE**	-
	6. Collect core samples at the toe to attempt to recover samples with intact concrete to rock bond for laboratory verification of cohesion.	**IBR-SPL-01-IE-CRE** IBR-AUX-01-I-CRE **IBR-INT-01-IE-CRE**	-
	7. Perform 3d stability evaluation.	**IBR-SPL-01-IE-CRE** IBR-AUX-01-I-CRE **IBR-INT-01-IE-CRE**	-

[1] "Risk Drivers" column: **Bold** PFMs are those which plot to the "risks are unacceptable" or "risks are generally intolerable" region of the Risk Matrix. Unbolded PFMs are other Risk Drivers that plot to the "generally tolerable" region of the Risk Matrix.

TABLE 1 – SUMMARY OF PFMS AND RISK DRIVER DESIGNATIONS

Table 1 – Summary of PFMs and Risk Driver Designations

#	PFM No.	Structure/ Feature	Controlling Loading Condition	Failure Mechanism	PFM Description	Disposition PFMA Workshop	Risk Category Risk Assessment
1	**IBR-SPL-01-IE-CRE**	**Principal Spillway**	**Ice, Earthquake**	**Stability**	**Sliding/Overturning of the Principal Spillway along the Foundation**	**Credible**	**Unacceptable / Intolerable**
2	IBR-SPL-02-A-ASM	Principal Spillway	Any	Stability	Sliding/Overturning of the Principal Spillway at a Construction Joint	Asset Management	Not a risk driver.
3	**IBR-SPL-03-F-CRE**	**Principal Spillway**	**Flood**	**Erosion/Scour**	**Erosion/Scour of the Downstream Toe of the Principal Spillway**	**Credible**	**Generally Intolerable**
4	**IBR-SPL-04-N-CRE**	**Principal Spillway**	**Normal**	**Erosion/Scour**	**Deterioration of the Foundation of the Principal Spillway**	**Credible**	**Generally Intolerable / Tolerable**
5	IBR-SPL-05-N-NEG	Principal Spillway	Normal	Erosion/Scour	Seepage Through the Left Abutment of the Principal Spillway	Clearly Negligible	Not a risk driver.
6	IBR-SPL-06-F-ASM	Principal Spillway	Flood	Stability	Debris Buildup at the Spillway Raises the Reservoir	Asset Management	Not a risk driver.
7	IBR-SPL-07-F-R/O	Principal Spillway	Flood	Structural	Structural Failure of Service Bridge During Flood	Ruled Out	Not a risk driver.
8	**IBR-AUX-01-I-CRE**	**Auxiliary Spillway**	**Ice**	**Stability**	**Sliding/Overturning of the Auxiliary Spillway along the Foundation**	**Credible**	**Generally Intolerable / Tolerable**
9	IBR-AUX-02-N-ASM	Auxiliary Spillway	Normal	Stability	Sliding/Overturning of the Auxiliary Spillway at a Construction Joint	Asset Management	Not a risk driver.
10	IBR-AUX-03-F-NEG	Auxiliary Spillway	Flood	Erosion/Scour	Erosion/Scour of the Downstream Toe of the Auxiliary Spillway	Clearly Negligible	Not a risk driver.
11	**IBR-AUX-04-N-CRE**	**Auxiliary Spillway**	**Normal**	**Erosion/Scour**	**Deterioration of the Foundation of the Auxiliary Spillway**	**Credible**	**Generally Intolerable / Tolerable**
12	IBR-AUX-05-N-NEG	Auxiliary Spillway	Normal	Erosion/Scour	Seepage Through the Right Abutment of the Auxiliary Spillway	Clearly Negligible	Not a risk driver.
13	**IBR-INT-01-IE-CRE**	**Intake**	**Ice, Earthquake**	**Stability**	**Sliding/Overturning of the Intake along the Foundation**	**Credible**	**Unacceptable / Generally Intolerable**
14	IBR-INT-02-IE-R/O	Intake	Ice, Earthquake	Stability	Separation/Rotation of the Intake at a Construction Joint	Ruled Out	Not a risk driver.
15	IBR-INT-03-IE-NEG	Intake	Ice, Earthquake	Stability	Structural Failure of the Intake Headwall	Clearly Negligible	Not a risk driver.
16	IBR-INT-04-N-R/O	Intake	Normal	Gates and Piping	Failure of the CMP at the Toe of the Intake During Release from Low Level Outlet	Ruled Out	Not a risk driver.
17	IBR-INT-05-A-R/O	Intake	Any	Gates and Piping	Failure of One of the Three Intake Gates, Closed Position	Ruled Out	Not a risk driver.
18	IBR-INT-06-A-R/O	Intake	Any	Gates and Piping	Failure of One of the Three Intake Gates, Open Position	Ruled Out	Not a risk driver.
19	IBR-INT-07-A-R/O	Intake	Any	Gates and Piping	Failure of One of the Outlet Gates, Closed Position	Ruled Out	Not a risk driver.
20	IBR-INT-08-A-ASM	Intake	Any	Gates and Piping	Failure of the Low-Level Outlet Gate, Open Position	Asset Management	Not a risk driver.
21	IBR-INT-09-A-INS	Intake	Any	Gates and Piping	Failure of the Water Treatment Facility Outlet Gate, Open Position	Insufficient Information	Not a risk driver.
22	IBR-GNR-01-A-R/O	General	Any	Overtopping	Rogue wave overtopping	Ruled Out	Not a risk driver.
23	IBR-GNR-02-A-R/O	General	Any	Overtopping	Failure of the Upstream Beaver Dam Causes Reservoir Surcharge	Ruled Out	Not a risk driver.

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RISK-DRIVING FAILURE MODES

IBR-SPL-01-IE-CRE SLIDING/OVERTURNING OF THE PRINCIPAL SPILLWAY ALONG THE FOUNDATION

PFM Information					
Structure	Principal Spillway	PFM Failure Type	Stability	PFM Source	New
Loading Condition	Ice, Earthquake	Location(s)	Principal Spillway	PFM Source Date	6/12/2023

PFM Description

Flaw:
- Insufficient cross section (geometry and mass distribution) at the Principal Spillway causes hydrostatic destabilizing loads to overcome the resisting loads, causing the Principal Spillway to slide downstream.

Continuation/Progression:
- Under the theorized loading conditions [See Note 1 under Justification].
- The force acting on the structure in a downstream direction exceeds the concrete-foundation interface strength.

Intervention:
- Intervention is not successful.

Failure:
- A section of the spillway slides downstream.
- A gap is formed in the dam.
- An uncontrolled release of the reservoir occurs.

PFM Disposition

☐ Ruled Out ☐ Clearly Negligible ☐ Asset Management
☐ Insufficient Info ☐ Financial/Damage State ☒ **Credible** ☐ Urgent

Classification Justification

This potential failure mode is considered credible.

Note 1. GZA performed a stability evaluation for a representative section of the Principal Spillway, with assumed parameters (40 degree friction angle, 0 psi cohesion) for the concrete-foundation interface, following the USACE guidelines for gravity dams. The results of this study are provided below:

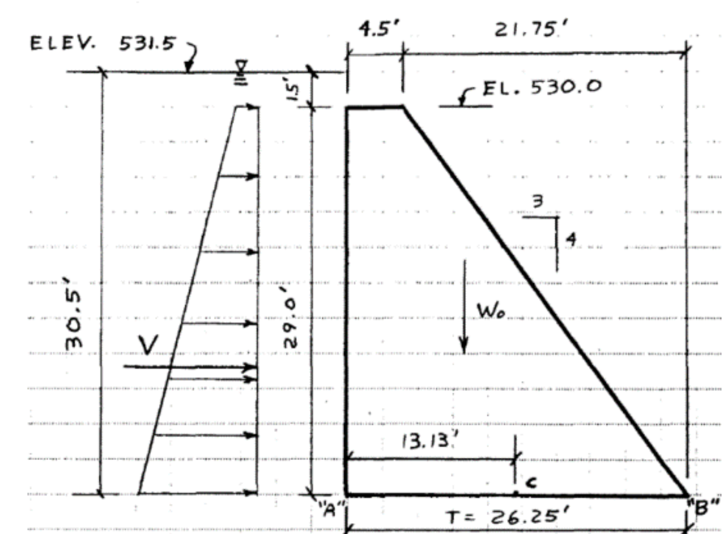
Cross-section	Water Level	Sliding			
		Minimum Required FS	Calculated FS	Base Cohesion Req'd for Min FS (psi)	Cracked Base Analysis
B-B On the Left of Gate Structure Base EL.501.6	Normal	2.0	1.4	4.0	NOT REQUIRED
	Flood (PMF)	1.1	1.1	0.3	NOT REQUIRED
	Normal + Ice	2.0	1.1	7.6	REQUIRED
	Normal + Earthquake	1.3	0.8	5.7	NOT REQUIRED

The evaluation suggests that the spillway is stable (sliding factor of safety > 1.0) under normal, flood (PMF) and normal plus ice loading conditions. However, the calculated factors of safety are lower than the USACE-recommended minimum values, unless a nominal (less than 10 psi) cohesion is assumed to exist at the interface. For static loading, the normal plus ice loading (5,000 lb ice load applied 0.5 feet below the spillway crest) had the lowest calculated factor of safety, of 1.1, versus a recommended minimum of 2.0. As this loading condition is deemed to be more likely to develop than the PMF, the ice loading condition is considered the controlling static load case.

The GZA stability evaluation also suggests that the Principal Spillway is unstable (factor of safety of 0.8) during the 2,475 year seismic event (2% in 50 year event), when analyzed via pseudostatic methods. This type of analysis is considered a preliminary estimate, with more refined dynamic modeling typically performed.

PFM Sketches

Downstream face of the Principal Spillway at the rock contact (GZA, 2022)



Free body diagram, overtopping (1988 G & Underwood Engineering Report)

Background Information			
Additional Supporting Information	• Stability evaluations performed by GZA, 2023. • 1980 USACE Phase I Inspection Report.	Performance Monitoring Information	n/a

Evaluation Factors			
Step/Node	Adverse (More Likely)	Favorable (Less Likely)	Surveillance & Monitoring Provisions
Flaw/Initiation	• Ice thickness at the dam can reach 12 inches, which is a typical assumed thickness in the absence of site-specific data.	• Ice melt is reported to typically be slow / gradual during the spring, with no large ice flows typically identified at the dam.	• n/a
Continuation/Progression	• Preliminary sliding stability factors of safety for normal, flood and normal plus ice loading conditions are less than the USACE-recommended minimum values. • Vertical upstream face of spillway allows full ice load to be transmitted to structure. • Lack of construction information, including unknown reinforcing across lift joints, unknown monolith to monolith reinforcing, and unknown foundation treatment. • Concrete deterioration noted at the vertical joints between monoliths and at horizontal construction joints; 2003 repairs failing/failed in some places. • Depth of sedimentation upstream of dam is unknown, although sedimentation has been reported to not be a concern at the site.	• Preliminary sliding stability factors of safety exceed 1.0 for normal, flood and normal plus ice loading conditions. • No known construction problems or conditions are known to have been encountered according (per USACE Phase I report). • A post-earthquake inspection (October 17, 2012) was conducted with no visible/noticeable changes. • Section is founded on bedrock, with sound rock exposed along the toe. Toe is free of debris, sediment, soils and vegetation, allowing for inspection to observe for signs of sliding or undermining erosion. • No visible misalignment at construction joints (vertical and horizontal).	• Visual inspections conducted every year by VTDSP. • Formal visual inspection by Owner in conjunction with VTDSP inspection, and inspection following a significant event (earthquake, microburst). • Last underwater upstream inspection circa 2011.
Intervention	• Minimal ability to draw down the reservoir in the event that a section of the Spillway is observed to have displaced.	• n/a	• Emergency Action Plan (2011).
Failure/Breach	• n/a	• No visual indication of downstream sliding to date (noted that past performance is not promise of future behavior). • 3D effects: would need to slide across the full monolith to destabilize. • Failure routes directly to the natural downstream channel.	• n/a

Risk Assessment	Failure Likelihood	
Category	☐ FL7 1/1 to 1/10 ☐ FL6 1/10 to 1/100 ☐ FL5 1/100 to 1/1,000 ☒ **FL4 1/1,000 to 1/10,000** ☐ FL3 1/10,000 to 1/100,000 ☐ FL2 1/100,000 to 1/1,000,000 ☐ FL1 < 1/1,000,000	
Discussion	The Risk Estimators agreed that of the two loading conditions postulated for this PFM, the earthquake loading condition controlled from a risk-driving perspective. The structure has a projected factor of safety, based upon the GZA 2022 stability evaluation, of less than 1.0 for the design earthquake, whereas the factor of safety under the full design ice load (1.1) is below the industry-recommended minimum value, but still above 1.0. The leading component of the likelihood determination was the likelihood of an earthquake of sufficient magnitude occurring, which would destabilize the structure. The stability calculation determined a factor of safety of 0.8 under an earthquake with a recurrence frequency of 1 in 2,475 years, which is well within the bracketed magnitudes for FL4. Considering that the other steps in the failure mode progression are fairly likely to occur, there was consensus that this PFM should be considered to have a FL4 likelihood.	
Confidence	☐ High ☒ **Medium** ☐ Low	Predominantly due to the unknowns associated with the foundation properties and rock to concrete friction angles, and the impact that value has on the overall stability assessment. The current stability assessment makes conservative assumptions for the friction angle.

Risk Assessment	Life Safety Consequences
Category	☐ LS0 None ☐ LS1 0-1 ☒ **LS2 1-10** ☒ **LS3 10-100** ☐ LS4 100-1,000 ☐ LS5 1,000-10,0000 ☐ LS6 >10,000
General Discussion	The dam breach inundation model (GZA 2023) extends 10.8 miles downstream from Indian Brook to Mallett's Bay. The model includes 15 roads that cross the downstream flow path, private residences, and light commercial/industrial development within the floodplain.
PAR and LoL	With the reservoir at the normal pool elevation, there are an estimated 31 structures, with a total PAR of 712, that would be within the limits of the flooded area when the dam breaches. Based upon the LifeSim modeling, the estimated loss of life is 3 people.

	The Risk Estimators were equally divided between LS2 (1-10 life loss) and LS3 (10-100 life loss). Although the estimated loss of life for a sunny day breach of the dam is 3, the Estimators discussed that an earthquake-induced failure would have limited pre-warning, which may impact the ability of the PAR to mobilize away from the flooded area. As such, this PFM was assigned as a split LS2/LS3 life safety consequence.		
Confidence	☐ High ☒ **Medium** ☐ Low	Due to the screening level PAR and LoL estimations.	
Potential Interim Risk Reduction Measures/Potential Dam Safety Management Actions			
Risk Reduction Measures	• Install bubblers during the winter months to prevent ice buildup against the dam.	EAP	• Perform regular tabletop exercises and keep the EAP up to date.
Inspections and Actions	• Keep with formal inspection yearly basis, special event follow up inspections (particularly post-earthquake). In particular, note for visual sign of displacement of crest of spillway.	Follow Up Studies	• Refined geotechnical field studies (such as borings or bedrock joint mapping downstream of section) could justify increased friction factors, which would benefit stability evaluation. • Coring through the dam to obtain intact concrete to rock samples for laboratory cohesion testing is projected to be expensive, and may not be successful (particularly accounting for the low cohesion values required to obtain acceptable stability factors of safety). • Collect core samples at the toe to attempt to recover samples with intact concrete to rock bond for laboratory verification of cohesion. • Perform 3d stability evaluation. • Cross hole seismic and NDT testing may not conclusively identify the presence of potential shear keys between concrete monoliths and is not recommended at this time. Due to the massive nature of the concrete, this testing may not pick up shear keys unless there is an air gap.
Surveillance and Monitoring	• n/a	Others	• n/a

IBR-SPL-03-F-CRE EROSION/SCOUR OF THE DOWNSTREAM TOE OF THE PRINCIPAL SPILLWAY

PFM Information					
Structure	Principal Spillway	PFM Failure Type	Erosion/Scour	PFM Source	New
Loading Condition	Flood	Location(s)	Toe of the Principal Spillway	PFM Source Date	6/12/2023

PFM Description

Flaw:	• Overtopping of the principal spillway erodes the downstream toe of the Principal Spillway, destabilizing the structure.
Continuation/Progression:	• There is a storm event up to and including the PMF (532.5 ft, NGVD29). • As flow continues over the spillway (crest 530 ft), the downstream channel begins to scour and erode. • The erosion progresses during subsequent floods until it reaches a critical distance beneath the spillway.
Intervention:	• The erosion goes unnoticed.
Failure:	• A section of the dam slides downstream. • A gap is formed in the dam. • An uncontrolled release of the reservoir occurs.
PFM Disposition	☐ Ruled Out ☐ Clearly Negligible ☐ Asset Management ☐ Insufficient Info ☐ Financial/Damage State ☒ **Credible** ☐ Urgent
Classification Justification	This potential failure mode is considered credible, for many of the similar reasons as IBR-SPL-01-IE. The results of GZA's 2023 stability study are provided below: (see table below) GZA's stability evaluation did not include embedment of the section into bedrock (based on lack of available information). However, potential loss of foundation material at the toe will have a detrimental effect on the stability of the section.

Cross-section	Water Level	Sliding			
		Minimum Required FS	Calculated FS	Base Cohesion Req'd for Min FS (psi)	Cracked Base Analysis
B-B On the Left of Gate Structure Base EL.501.6	Normal	2.0	1.4	4.0	NOT REQUIRED
	Flood (PMF)	1.1	1.1	0.3	NOT REQUIRED
	Normal + Ice	2.0	1.1	7.6	REQUIRED
	Normal + Earthquake	1.3	0.8	5.7	NOT REQUIRED

PFM Sketch(es)

Downstream face of the Principal Spillway at the rock contact (GZA, 2022)

Background Information			
Additional Supporting Information	• Stability evaluations performed by GZA, 2023. • 1980 USACE Phase I Inspection Report.	Performance Monitoring Information	n/a

Evaluation Factors			
Step/Node	Adverse (More Likely)	Favorable (Less Likely)	Surveillance & Monitoring Provisions
Flaw/Initiation	• Spillway overtops during the considered load case, up to 2.5 feet by the PMF.	• The spillway is designed to overtop. • Minimal overtopping, 0.7-ft, during the 100 year flood.	• n/a
Continuation/Progression	• Preliminary sliding stability factors of safety (without considering the stabilizing effects of embedment into bedrock) are less than the USACE-recommended minimum values. • Lack of construction information regarding foundation treatment	• Preliminary sliding stability factors of safety (without considering the stabilizing effects of embedment into bedrock) exceed 1.0 for normal, flood and normal plus ice loading conditions. • No known construction problems or conditions are known to have been encountered according (per USACE Phase I report). • Section is founded on bedrock, with sound rock exposed along the toe. Toe is free of debris, sediment, soils and vegetation, allowing for inspection to observe for signs of sliding or undermining erosion. • No visible misalignment at construction joints (vertical and horizontal). • Limited duration of discharge not likely to significantly scour the rock. • Last 17 years, maximum overtopping ~ 6 inches.	• Visual inspections conducted every two years by VTDEC. • A post-earthquake inspection (October 17, 2012) was conducted with no visible/noticeable changes. • No unknown underwater inspections have been conducted.
Intervention	• Minimal ability to draw down the reservoir in the event that a section of the Spillway is observed to have displaced.	• Slow progressing failure mode – allows for time to intervene.	• Emergency Action Plan (2011).
Failure/Breach	• n/a	• No visual indication of downstream sliding to date (noted that past performance is not promise of future behavior). • 3D effects: would need to slide across the full monolith to destabilize. • Failure routes directly to the natural downstream channel.	• n/a

Risk Assessment	**Failure Likelihood**	
Category	☐ FL7 1/1 to 1/10 ☐ FL6 1/10 to 1/100 ☐ FL5 1/100 to 1/1,000 ☐ FL4 1/1,000 to 1/10,000 ☐ FL3 1/10,000 to 1/100,000 ☒ **FL2 1/100,000 to 1/1,000,000** ☐ FL1 < 1/1,000,000	
Discussion	The Risk Estimators noted that multiple sizeable flood events would be required to cause sufficient scour to undermine the toe and cause potential stability issues. The PMF does also have a limited depth of overtopping. However, this is an untested loading condition, so although the bedrock is hard granite, significant overtopping has likely not occurred in the past. There was concurrence that this failure mode be categorized as FL2.	
Confidence	☐ High ☒ **Medium** ☐ Low	Due to the untested nature of this failure mode, and undocumented bedrock resistance to erosion.
Risk Assessment	**Life Safety Consequences**	
Category	☐ LS0 None ☐ LS1 0-1 ☐ LS2 1-10 ☒ **LS3 10-100** ☐ LS4 100-1,000 ☐ LS5 1,000-10,0000 ☐ LS6 >10,000	
General Discussion	The dam breach inundation model (GZA 2023) extends 10.8 miles downstream from Indian Brook to Mallett's Bay. The model includes 15 roads that cross the downstream flow path, private residences, and light commercial/industrial development within the floodplain.	
PAR and LoL	With the reservoir at the PMF pool elevation, there are an estimated 14 structures that would be impacted by a dam failure that would not be impacted if the dam did not fail (incremental impact). The incremental PAR is 52, and the incremental Loss of Life is 18. The Risk Estimators generally agreed with LS3 (10-100 life loss). It was noted that this may be a slower developing failure mode that requires multiple significant overtopping events to initiate, and as such intermediate intervention may be possible.	
Confidence	☐ High ☒ **Medium** ☐ Low	Due to the screening level PAR and LoL estimations.

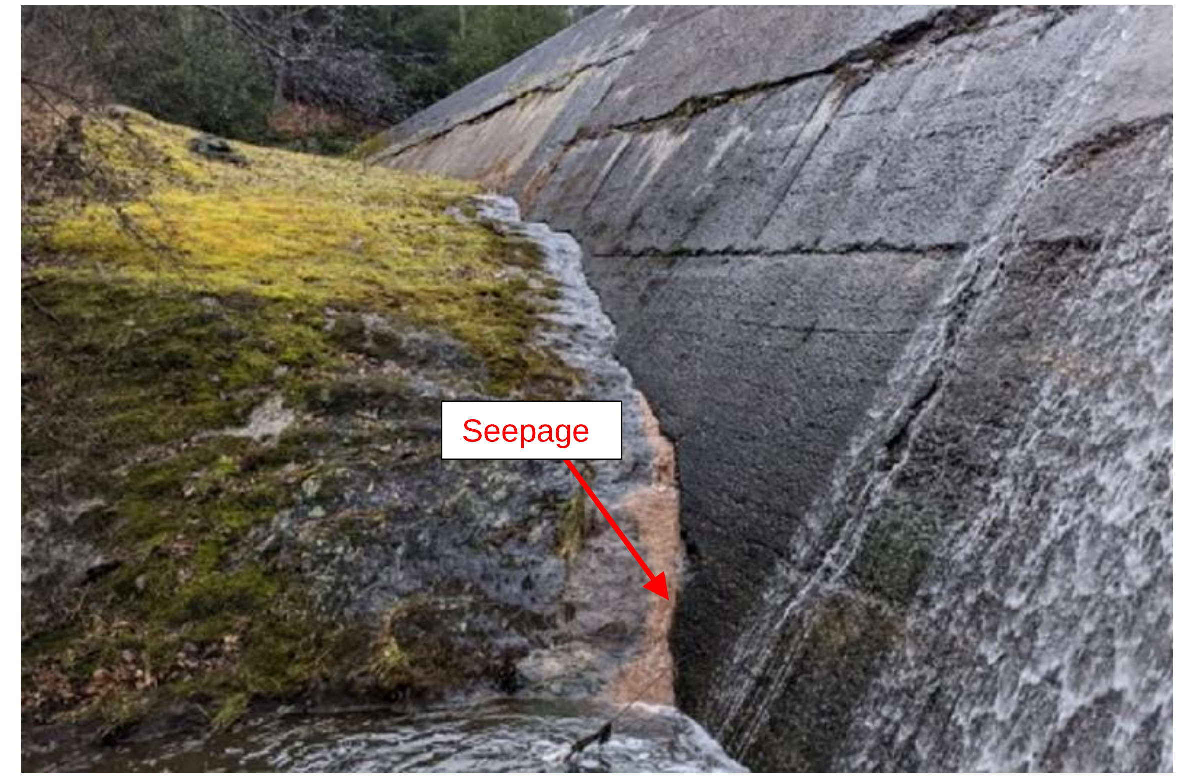
Potential Interim Risk Reduction Measures/Potential Dam Safety Management Actions			
Risk Reduction Measures	• n/a	EAP	• Perform regular tabletop exercises and keep the EAP up to date.
Inspections and Actions	• Keep with formal inspection yearly basis, special event follow up inspections. In particular, note for visual sign of displacement of crest of spillway. • Perform a targeted visual inspection of the area downstream during periods of no flow over the spillway to look for signs of erosion/scour.	Follow Up Studies	• Refined geotechnical field studies (such as borings or bedrock joint mapping downstream of section) could justify increased friction factors, which would benefit stability evaluation. • Coring through the dam to obtain intact concrete to rock samples for laboratory cohesion testing is projected to be expensive, and may not be successful (particularly accounting for the low cohesion values required to obtain acceptable stability factors of safety).
Surveillance and Monitoring	• n/a	Others	• n/a

IBR-SPL-04-N-CRE DETERIORATION OF THE FOUNDATION OF THE PRINCIPAL SPILLWAY

PFM Information					
Structure	Principal Spillway	PFM Failure Type	Erosion/Scour	PFM Source	New
Loading Condition	Normal	Location(s)	Principal Spillway	PFM Source Date	6/12/2023

PFM Description	
Flaw:	• Construction defect exists at the contact with the foundation, due to inadequate foundation preparation. Defect allows preferential seepage along the contact, increasing the destabilizing uplift pressures.
Continuation/Progression:	• The reservoir is at normal reservoir pool. • A preferential seepage path exists through a zone of inadequate foundation preparation (such as soil-infilled rock seam) which originates below the Auxiliary Spillway and daylights downstream of the embankment. • Continued seepage along the path causes the joint to open, increasing uplift pressure on the dam. • The force acting on the structure in a downstream direction exceeds the concrete-foundation interface strength.
Intervention:	• Increasing seepage volume goes unnoticed.
Failure:	• A section of the dam slides downstream. • A gap is formed in the dam. • An uncontrolled release of the reservoir occurs.
PFM Disposition	☐ Ruled Out ☐ Clearly Negligible ☐ Asset Management ☐ Insufficient Info ☐ Financial/Damage State ☒ **Credible** ☐ Urgent
Classification Justification	This potential failure mode is considered credible. It is similar to IBR-SPL-03-F; however this PFM relates to a foundation defect under the dam, which, over time, allows for uplift pressures to increase and destabilize the dam. GZA performed a stability evaluation for the deepest section of the Principal Spillway, with assumed parameters (40 degree friction angle, 0 psi cohesion) for the concrete-foundation interface, following the USACE guidelines for gravity dams. The results of the stability study are provided to the right. The evaluation suggests that the spillway is stable (sliding factor of safety > 1.0) under normal, flood (PMF) and normal plus ice loading conditions. However, the calculated factors of safety are lower than the USACE-recommended minimum values for all cases except the PMF, unless a nominal (less than 10 psi) cohesion is assumed to exist at the interface. As such, increasing uplift pressures in excess of the traditionally assumed straight line profile from headwater to tailwater may have a detrimental effect on the stability of the section.

PFM Sketch(es)



Downstream face of the Principal Spillway at the rock contact, where the Principal Spillway transitions upslope towards the Auxiliary Spillway (GZA, 2022)

Cross-section	Water Level	Sliding			
		Minimum Required FS	Calculated FS	Base Cohesion Req'd for Min FS (psi)	Cracked Base Analysis
B-B On the Left of Gate Structure Base EL.501.6	Normal	2.0	1.4	4.0	NOT REQUIRED
	Flood (PMF)	1.1	1.1	0.3	NOT REQUIRED
	Normal + Ice	2.0	1.1	7.6	REQUIRED
	Normal + Earthquake	1.3	0.8	5.7	NOT REQUIRED

GZA 2023 stability evaluation results.

Background Information			
Additional Supporting Information	• Stability evaluations performed by GZA, 2023. • 1980 USACE Phase I Inspection Report.	Performance Monitoring Information	n/a

Evaluation Factors

Step/Node	Adverse (More Likely)	Favorable (Less Likely)	Surveillance & Monitoring Provisions
Flaw/Initiation	• Seepage and iron staining observed at the downstream toe during the 2022 GZA inspection (see photograph on previous page).	• Dam founded on bedrock.	• n/a
Continuation/Progression	• Preliminary sliding stability factors of safety for normal, flood and normal plus ice loading conditions are less than the USACE-recommended minimum values. • Lack of construction information regarding foundation treatment. • There are no available boring logs or in-situ testing available to validate assumed foundation properties.	• Preliminary sliding stability factors of safety exceed 1.0 for normal, flood and normal plus ice loading conditions. • No known construction problems or conditions are known to have been encountered according (per USACE Phase I report). • Section is founded on bedrock, with sound rock exposed along the toe. Toe is free of debris, sediment, soils and vegetation, allowing for inspection to observe for signs of sliding or undermining erosion. • No visible misalignment at construction joints (vertical and horizontal).	• Visual inspections conducted every two years by VTDEC. • A post-earthquake inspection (October 17, 2012) was conducted with no visible/noticeable changes. • No unknown underwater inspections have been conducted.
Intervention	• Minimal ability to draw down the reservoir in the event that a section of the Spillway is observed to have displaced.	• Slow progressing failure mode – allows for time to intervene.	• Emergency Action Plan (2011).
Failure/Breach	• n/a	• No visual indication of downstream sliding to date (noted that past performance is not promise of future behavior). • 3D effects: would need to affect uplift over a significant portion of the base to destabilize. • Failure routes directly to the natural downstream channel.	• n/a

Risk Assessment	**Failure Likelihood**	
Category	☐ FL7 1/1 to 1/10 ☐ FL6 1/10 to 1/100 ☐ FL5 1/100 to 1/1,000 ☐ FL4 1/1,000 to 1/10,000 ☐ FL3 1/10,000 to 1/100,000 ☒ **FL2 1/100,000 to 1/1,000,000** ☐ FL1 < 1/1,000,000	
Discussion	The Risk Estimators discussed that seepage has historically been observed at the concrete to rock contact along the exposed bedrock slope, although there is not an indication that this seepage originates at the reservoir. Due to the massive nature of the spillway, it is likely that multiple opened bedrock joints would be required to generate sufficient uplift pressures to destabilize the dam, and the bedrock at the toe is generally observed to be sound and intact. The Risk Estimators were in concurrence with a FL2 likelihood.	
Confidence	☐ High ☒ **Medium** ☐ Low	Due to the observed seepage at the concrete and bedrock interface.
Risk Assessment	**Life Safety Consequences**	
Category	☐ LS0 None ☐ LS1 0-1 ☒ **LS2 1-10** ☐ LS3 10-100 ☐ LS4 100-1,000 ☐ LS5 1,000-10,0000 ☐ LS6 >10,000	
General Discussion	The Risk Estimators dam breach inundation model (GZA 2023) extends 10.8 miles downstream from Indian Brook to Mallett's Bay. The model includes 15 roads that cross the downstream flow path, private residences, and light commercial/industrial development within the floodplain.	
PAR and LoL	With the reservoir at the normal pool elevation, there are an estimated 31 structures, with a total PAR of 712, that would be within the limits of the flooded area when the dam breaches. Based upon the LifeSim modeling, the estimated loss of life is 3 people. The Risk Estimators concurred on LS2 (1-10 life loss) as the estimated loss of life for a sunny day breach of the dam is 3.	
Confidence	☐ High ☒ **Medium** ☐ Low	Due to the screening level PAR and LoL estimations.

Potential Interim Risk Reduction Measures/Potential Dam Safety Management Actions			
Risk Reduction Measures	• Dye testing to identify source of seepage, check for connection. • Water chemistry and temperature check to evaluate for connection.	EAP	• Perform regular tabletop exercises and keep the EAP up to date.
Inspections and Actions	• Keep with formal inspection yearly basis, special event follow up inspections. In particular, note for visual sign of displacement of crest of spillway.	Follow Up Studies	• Refined geotechnical field studies (such as borings or bedrock joint mapping downstream of section) could justify increased friction factors, which would benefit stability evaluation. • Coring through the dam to obtain intact concrete to rock samples for laboratory cohesion testing is projected to be expensive, and may not be successful (particularly accounting for the low cohesion values required to obtain acceptable stability factors of safety).
Surveillance and Monitoring	• Photograph the seepage regularly (start with quarterly) from same vantage point / setting baseline for flow. Note areas of observed seepage, estimate/quantify amount of seepage, color, etc. • Installing seepage collection point (such as a v-notch weir or “c” channel) to quantify seepage flow observed along the interface not practical; would be damaged/destroyed by overtopping flows.	Others	• n/a

IBR-INT-01-IE-CRE SLIDING/OVERTURNING OF THE INTAKE ALONG THE FOUNDATION

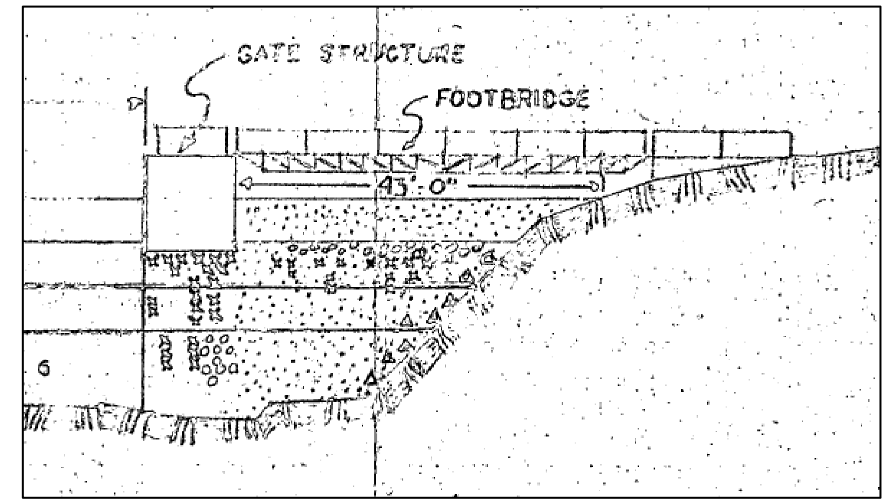
PFM Information					
Structure	Intake	PFM Failure Type	Stability	PFM Source	New
Loading Condition	Ice, Earthquake	Location(s)	Intake	PFM Source Date	6/12/2023

PFM Description

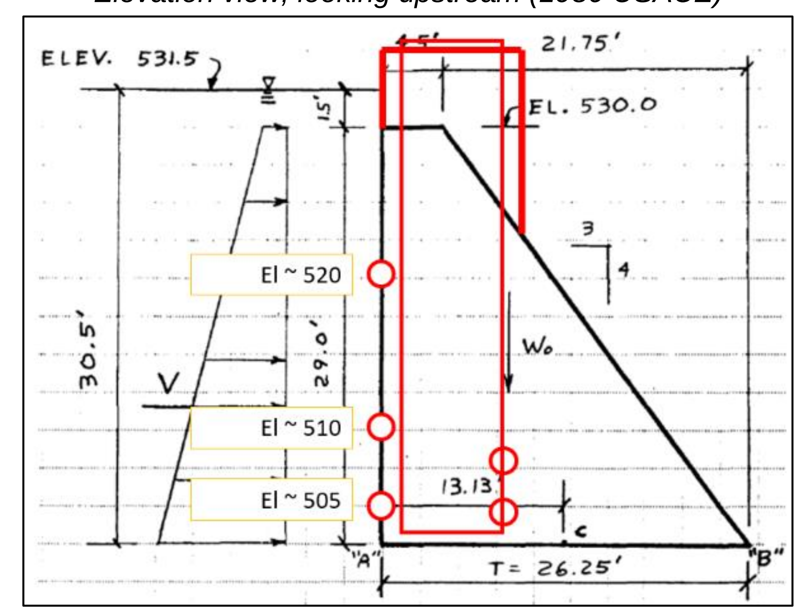
Flaw:	• Insufficient cross section (geometry and mass distribution) at the Intake causes hydrostatic destabilizing loads to overcome the resisting loads, causing the Principal Spillway to slide downstream.
Continuation/Progression:	• Under the theorized loading conditions [See Note 1 under Justification]. • The force acting on the structure in a downstream direction exceeds the concrete-foundation interface strength.
Intervention:	• Intervention is not successful.
Failure:	• A section of the spillway slides downstream. • A gap is formed in the dam. • An uncontrolled release of the reservoir occurs.
PFM Disposition	☐ Ruled Out ☐ Clearly Negligible ☐ Asset Management ☐ Insufficient Info ☐ Financial/Damage State ☒ **Credible** ☐ Urgent
Classification Justification	This potential failure mode is considered credible. The Intake section is integral with the Principal Spillway; as shown in the top figure to the right, the Intake is just to the left of a vertical construction joint. Specific construction records are not available; however it is expected that the vault chamber that forms the intake was formed during the placement of the Principal Spillway. Stability for the Intake structure itself has not been performed. Refer to IBR-SPL-01-IE for a summary of stability evaluations performed for the Principal Spillway, and a discussion establishing Ice and Earthquake as the controlling loading conditions. A cross section through the Intake (lower figure to the right) will have less mass than the general Principal Spillway (due to the open valve chamber, when the water level is near to the bottom of the chamber). The results of the stability study for the Principal Spillway are provided below:

Cross-section	Water Level	Sliding: Minimum Required FS	Sliding: Calculated FS	Sliding: Base Cohesion Req'd for Min FS (psi)	Sliding: Cracked Base Analysis
B-B *On the Left of Gate Structure Base EL.501.6*	Normal	2.0	1.4	4.0	NOT REQUIRED
	Flood (PMF)	1.1	1.1	0.3	NOT REQUIRED
	Normal + Ice	2.0	1.1	7.6	REQUIRED
	Normal + Earthquake	1.3	0.8	5.7	NOT REQUIRED

PFM Sketch(es)



Elevation view, looking upstream (1980 USACE)



Cross section, 1988 G & Underwood Engineering Report with annotations by GZA to depict Intake (three inlets, chamber and two outlets)

Background Information			
Additional Supporting Information	• Stability evaluations performed by GZA, 2023. • 1980 USACE Phase I Inspection Report.	Performance Monitoring Information	n/a

Evaluation Factors

Step/Node	Adverse (More Likely)	Favorable (Less Likely)	Surveillance & Monitoring Provisions
Flaw/Initiation	• Ice thickness at the dam can reach 12 inches, which is a typical assumed thickness in the absence of site-specific data. • There is less vertical weight associated with a cross section through the Intake, due to the valve chamber that was water only near the bottom of the chamber.	• Ice melt is reported to typically be slow / gradual during the spring, with no large ice flows typically identified at the dam. • There are significant 3D effects associated with the concrete monolith that includes the Intake. Although the Intake is 10 feet long (in the direction perpendicular to flow), the remainder of the approximately 53 foot long monolith includes approximately 20 feet of similar height, intact cross section, and a 23 foot long section that transitions up along the abutment.	• n/a
Continuation/Progression	• Preliminary sliding stability factors of safety for normal, flood and normal plus ice loading conditions, are less than the USACE-recommended minimum values. • Lack of construction information, including unknown reinforcing across lift joints, unknown monolith to monolith reinforcing, and unknown foundation treatment. • Concrete deterioration noted at the vertical joints between monoliths and at horizontal construction joints; 2003 repairs failing/failed in some places. • Depth of sedimentation upstream of dam is unknown, although sedimentation has been reported to not be a concern at the site.	• Preliminary sliding stability factors of safety exceed 1.0 for normal, flood and normal plus ice loading conditions. • No known construction problems or conditions are known to have been encountered according (per USACE Phase I report). • A post-earthquake inspection (October 17, 2012) was conducted with no visible/noticeable changes. • Section is founded on bedrock, with sound rock exposed along the toe. Toe is free of debris, sediment, soils and vegetation, allowing for inspection to observe for signs of sliding or undermining erosion. • No visible misalignment at construction joints (vertical and horizontal).	• Visual inspections conducted every year by VTDEC. • Formal visual inspection with VTDEC on a yearly basis, and inspection following a significant event (earthquake, microburst). • Last underwater upstream inspection circa 2011.
Intervention	• Minimal ability to draw down the reservoir in the event that a section of the Spillway is observed to have displaced.	• n/a	• Emergency Action Plan (2011).
Failure/Breach	• n/a	• No visual indication of downstream sliding to date (noted that past performance is not promise of future behavior). • Failure routes directly to the natural downstream channel.	• n/a

Risk Assessment	**Failure Likelihood**	
Category	☐ FL7 1/1 to 1/10 ☐ FL6 1/10 to 1/100 ☐ FL5 1/100 to 1/1,000 ☒ **FL4 1/1,000 to 1/10,000** ☐ FL3 1/10,000 to 1/100,000 ☐ FL2 1/100,000 to 1/1,000,000 ☐ FL1 < 1/1,000,000	
Discussion	The Risk Estimators agreed that based upon the available information, of the two loading conditions postulated for this PFM, the earthquake loading condition controlled from a risk-driving perspective. The similarly constructed Spillway structure has a projected factor of safety, based upon the GZA 2022 stability evaluation, of less than 1.0 for the design earthquake, whereas the factor of safety under the full design ice load (1.1) is below the industry-recommended minimum value, but still above 1.0. The leading component of the likelihood determination was the likelihood of an earthquake of sufficient magnitude occurring, which would destabilize the structure. The stability calculation for the Spillway determined a factor of safety of 0.8 under an earthquake with a recurrence frequency of 1 in 2,475 years, which is well within the bracketed magnitudes for FL4. Considering that the other steps in the failure mode progression are fairly likely to occur, there was consensus that this PFM should be considered to have a FL4 likelihood. It was discussed that while the Intake has less concrete volume than the Spillway, there are also significant 3D effects that would help to restrict downstream movement of the section.	
Confidence	☐ High ☐ Medium ☒ **Low**	Low confidence in the supporting stability information. While the comparable Spillway section stability showed factors of safety greater than 1.0 for all conditions except seismic, some of the factors of safety were close to one. Without performing formal calculation, the Estimators did not have a high confidence that the Earthquake loading would control; for example, if the Ice condition were shown to have a factor of safety less than 1, this could become the controlling loading condition for this PFM.
Risk Assessment	**Life Safety Consequences**	
Category	☐ LS0 None ☐ LS1 0-1 ☒ **LS2 1-10** ☐ LS3 10-100 ☐ LS4 100-1,000 ☐ LS5 1,000-10,0000 ☐ LS6 >10,000	
General Discussion	The dam breach inundation model (GZA 2023) extends 10.8 miles downstream from Indian Brook to Mallett's Bay. The model includes 15 roads that cross the downstream flow path, private residences, and light commercial/industrial development within the floodplain.	

PAR and LoL	With the reservoir at the normal pool elevation, there are an estimated 31 structures, with a total PAR of 712, that would be within the limits of the flooded area when the dam breaches. Based upon the LifeSim modeling, the estimated loss of life is 3 people. The Risk Estimators were 3 in favor of LS2 (1-10 life loss) and 1 in favor of LS3 (10-100 life loss). Although the estimated loss of life for a sunny day breach of the dam is 3, the Estimators discussed that an earthquake-induced failure would have limited pre-warning, which may impact the ability of the PAR to mobilize away from the flooded area. However, with the Intake being of reduced width compared to the Spillway, the breach wave would be of smaller initial magnitude. As such, this PFM was assigned as LS2.		
Confidence	☐ High ☒ **Medium** ☐ Low	Screening level PAR and LoL estimation.	
Potential Interim Risk Reduction Measures/Potential Dam Safety Management Actions			
Risk Reduction Measures	• Install bubblers during the winter months to prevent ice buildup against the dam.	EAP	• Perform regular tabletop exercises and keep the EAP up to date.
Inspections and Actions	• Keep with formal inspection yearly basis, special event follow up inspections.	Follow Up Studies	• Refined geotechnical field studies (such as borings or bedrock joint mapping downstream of section) could justify increased friction factors, which would benefit stability evaluation. • Coring through the dam to obtain intact concrete to rock samples for laboratory cohesion testing is projected to be expensive, and may not be successful (particularly accounting for the low cohesion values required to obtain acceptable stability factors of safety). • Collect core samples at the toe to attempt to recover samples with intact concrete to rock bond for laboratory verification of cohesion. • Perform 3d stability evaluation. • Cross hole seismic and NDT testing may not conclusively identify the presence of potential shear keys between concrete monoliths, and is not recommended at this time. Due to the massive nature of the concrete, this testing may not pick up shear keys unless there is an air gap.
Surveillance and Monitoring	• n/a	Others	• n/a

IBR-AUX-01-I-CRE SLIDING/OVERTURNING OF THE AUXILIARY SPILLWAY ALONG THE FOUNDATION

PFM Information					
Structure	Auxiliary Spillway	PFM Failure Type	Stability	PFM Source	New
Loading Condition	Ice	Location(s)	Auxiliary Spillway	PFM Source Date	6/12/2023

PFM Description

Flaw:	• Insufficient cross section (geometry and mass distribution) at the Auxiliary Spillway causes hydrostatic destabilizing loads to overcome the resisting loads, causing the Auxiliary Spillway to slide downstream.
Continuation/Progression:	• The reservoir is at normal pool (to the top of the Principal Spillway), with 12 inches of ice depth in front of the dam. [See Note 1 under Justification]. • The force acting on the structure in a downstream direction exceeds the concrete-foundation interface strength.
Intervention:	• Intervention is not successful.
Failure:	• A section of the spillway slides downstream. • A gap is formed in the dam. • An uncontrolled release of the reservoir occurs.
PFM Disposition	☐ Ruled Out ☐ Clearly Negligible ☐ Asset Management ☐ Insufficient Info ☐ Financial/Damage State ☒ **Credible** ☐ Urgent
Classification Justification	This potential failure mode is considered credible. Note 1. GZA performed a stability evaluation for the Auxiliary Spillway, with assumed parameters (40 degree friction angle, 0 psi cohesion) for the concrete-foundation interface, following the USACE guidelines for gravity dams. The results of the stability study are provided to the right. The evaluation suggests that the spillway is stable (sliding FS > 1.0) under normal, flood (PMF), and normal plus ice loading conditions. The calculated factors of safety are lower than the USACE-recommended minimum values for all cases except the PMF, unless a nominal (less than 5 psi) cohesion is assumed to exist at the interface. The normal plus ice loading (5,000 lb ice load applied 0.5 feet below the spillway crest) condition had the lowest calculated factor of safety of the static loading conditions. As such, this PFM is tailored toward the ice loading condition. The GZA stability evaluation also suggests that the Auxiliary Spillway is stable (factor of safety of 1.0) during the 2,475 year seismic event (2% in 50 year event), when analyzed via pseudodynamic methods. This type of analysis is considered a preliminary estimate, with more refined dynamic modeling typically performed.

PFM Sketch(es)

Downstream face of the Auxiliary Spillway at the rock contact (GZA, 2022)

Cross-section	Water Level	Sliding: Minimum Required FS	Sliding: Calculated FS	Sliding: Base Cohesion Req'd for Min FS (psi)	Sliding: Cracked Base Analysis
A-A On the Right of Gate Structure Base EL.514.1	Normal	2.0	1.9	0.2	NOT REQUIRED
	Flood (PMF)	1.1	1.4	N/A	NOT REQUIRED
	Normal + Ice	2.0	1.3	3.9	REQUIRED
	Normal + Earthquake	1.3	1.0	1.6	NOT REQUIRED

GZA 2023 stability evaluation results.

Background Information			
Additional Supporting Information	• Stability evaluations performed by GZA, 2023. • 1980 USACE Phase I Inspection Report.	Performance Monitoring Information	n/a

Evaluation Factors

IBR-AUX-01-I-CRE Sliding/Overturning of the Auxiliary Spillway along the Foundation

Step/Node	Adverse (More Likely)	Favorable (Less Likely)	Surveillance & Monitoring Provisions
Flaw/Initiation	• Ice thickness at the dam can reach 12 inches, which is a typical assumed thickness in the absence of site-specific data.	• Ice melt is reported to typically be slow / gradual during the spring, with no large ice flows typically identified at the dam.	• n/a
Continuation/Progression	• Preliminary sliding stability factors of safety for normal, flood and normal plus ice loading conditions are less than the USACE-recommended minimum values. • Lack of construction information, including unknown reinforcing across lift joints, unknown monolith to monolith reinforcing, and unknown foundation treatment. • Concrete deterioration noted at the vertical joints between monoliths and at horizontal construction joints; 2003 repairs failing/failed in some places. • Depth of sedimentation upstream of dam is unknown, although sedimentation has been reported to not be a concern at the site. • Standing water has historically been noted at the toe, which travels to the left along the toe before cascading down slope towards the Principal Spillway.	• Preliminary sliding stability factors of safety exceed 1.0 for normal, flood and normal plus ice loading conditions. • No known construction problems or conditions are known to have been encountered according (per USACE Phase I report). • A post-earthquake inspection (October 17, 2012) was conducted with no visible/noticeable changes. • Section is assumed to be founded on bedrock. Area directly downstream of the section is expected to be rock with a thin soil cover. Toe is free of debris, sediment, soils and deleterious vegetation, allowing for inspection to observe for signs of sliding or undermining erosion. • No visible misalignment at construction joints (vertical and horizontal). • Auxiliary Spillway overtops infrequently (believed to have not overtopped during last 17 years).	• Visual inspections conducted every year by VTDEC. • Formal visual inspection with VTDEC on a yearly basis, and inspection following a significant event (earthquake, microburst). • Last underwater upstream inspection circa 2011.
Intervention	• Minimal ability to draw down the reservoir in the event that a section of the Spillway is observed to have displaced.	• n/a	• Emergency Action Plan (2011).
Failure/Breach	• n/a	• No visual indication of downstream sliding to date (noted that past performance is not promise of future behavior). • 3D effects: would need to slide across the full monolith to destabilize.	• n/a

Risk Assessment	**Failure Likelihood**	
Category	☐ FL7 1/1 to 1/10 ☐ FL6 1/10 to 1/100 ☐ FL5 1/100 to 1/1,000 ☐ FL4 1/1,000 to 1/10,000 ☐ FL3 1/10,000 to 1/100,000 ☒ **FL2 1/100,000 to 1/1,000,000** ☐ FL1 < 1/1,000,000	
Discussion	The leading component of the likelihood that a significant-enough thickness of ice could develop upstream of the auxiliary spillway, to cause the section to destabilize. The Estimators noted that the factor of safety against ice loading exceeded 1.0, and that the project is routinely subjected to the development of ice up to 12 inches thick. As such, there was agreement that this PFM should be considered to have a FL2 likelihood.	
Confidence	☐ High ☒ **Medium** ☒ Low	The Estimators noted a MEDIUM confidence level in the assigned failure likelihood, predominantly due to the unknowns associated with the foundation properties and rock to concrete friction angles, and the impact that value has on the overall stability assessment. The current stability assessment makes conservative assumptions for the friction angles, and as such refining the input parameters may justify the use of FL1.
Risk Assessment	**Life Safety Consequences**	
Category	☐ LS0 None ☐ LS1 0-1 ☒ **LS2 1-10** ☐ LS3 10-100 ☐ LS4 100-1,000 ☐ LS5 1,000-10,0000 ☐ LS6 >10,000	
General Discussion	The dam breach inundation model (GZA 2023) extends 10.8 miles downstream from Indian Brook to Mallett's Bay. The model includes 15 roads that cross the downstream flow path, private residences, and light commercial/industrial development within the floodplain.	
PAR and LoL	With the reservoir at the normal pool elevation, there are an estimated 31 structures, with a total PAR of 712, that would be within the limits of the flooded area when the dam breaches. Based upon the LifeSim modeling, the estimated loss of life is 3 people. The Risk Estimators were 3 in favor of LS2 (1-10 life loss) and 1 in favor of LS3 (10-100 life loss). The Estimators discussed that an ice loading event in excess of 12 inches could be identified and acted upon prior to failure of the section, and that the Auxiliary Spillway is a shorter section than the Principal Spillway, and as such the immediate flood wave from a breach would be reduced. As such, this PFM was assigned as LS2.	
Confidence	☐ High ☒ **Medium** ☐ Low	Screening level PAR and LoL estimation
Potential Interim Risk Reduction Measures/Potential Dam Safety Management Actions		

Risk Reduction Measures	• Install bubblers during the winter months to prevent ice buildup against the dam.	EAP	• Perform regular tabletop exercises and keep the EAP up to date.
Inspections and Actions	• Keep with formal inspection yearly basis, special event follow up inspections. In particular, note for visual sign of displacement of crest of spillway.	Follow Up Studies	• Refined geotechnical field studies (such as borings or bedrock joint mapping downstream of section) could justify increased friction factors, which would benefit stability evaluation. • Coring through the dam to obtain intact concrete to rock samples for laboratory cohesion testing is projected to be expensive, and may not be successful (particularly accounting for the low cohesion values required to obtain acceptable stability factors of safety). • Collect core samples at the toe to attempt to recover samples with intact concrete to rock bond for laboratory verification of cohesion. • Perform 3d stability evaluation.
Surveillance and Monitoring	• n/a	Others	• n/a

IBR-AUX-04-N-CRE DETERIORATION OF THE FOUNDATION OF THE AUXILIARY SPILLWAY

PFM Information					
Structure	Auxiliary Spillway	PFM Failure Type	Erosion/Scour	PFM Source	New
Loading Condition	Normal	Location(s)	Auxiliary Spillway	PFM Source Date	6/12/2023

PFM Description

Flaw:	• Construction defect exists at the contact with the foundation, due to inadequate foundation preparation. Defect allows preferential seepage along the contact, increasing the destabilizing uplift pressures.
Continuation/Progression:	• The reservoir is at normal reservoir pool. • A preferential seepage path exists through a zone of inadequate foundation preparation (such as soil-infilled rock seam) which originates below the Auxiliary Spillway and daylights downstream of the embankment. • Continued seepage along the path causes the joint to open, increasing uplift pressure on the dam. • The force acting on the structure in a downstream direction exceeds the concrete-foundation interface strength.
Intervention:	• Increasing seepage volume goes unnoticed.
Failure:	• A section of the dam slides downstream. • A gap is formed in the dam. • An uncontrolled release of the reservoir occurs.
PFM Disposition	☐ Ruled Out ☐ Clearly Negligible ☐ Asset Management ☐ Insufficient Info ☐ Financial/Damage State ☒ **Credible** ☐ Urgent
Classification Justification	This potential failure mode is considered credible. This PFM relates to a foundation defect under the dam, which, over time, allows for uplift pressures to increase and destabilize the dam. GZA performed a stability evaluation for the Auxiliary Spillway, with assumed parameters (40 degree friction angle, 0 psi cohesion) for the concrete-foundation interface, following the USACE guidelines for gravity dams. The results of the stability study are provided to the right: The evaluation suggests that the spillway is stable (sliding factor of safety > 1.0) under normal, flood (PMF), normal plus ice and earthquake loading conditions. However, the calculated factors of safety are lower than the USACE-recommended minimum values for all cases except the PMF, unless a nominal (less than 5 psi) cohesion is assumed to exist at the interface. As such, increasing uplift pressures in excess of the traditionally assumed straight line profile from headwater to tailwater may have a detrimental effect on the stability of the section.

PFM Sketch(es)

Downstream toe of the Auxiliary Spillway (GZA, 2022)

Cross-section	Water Level	Sliding			Cracked Base Analysis
		Minimum Required FS	Calculated FS	Base Cohesion Req'd for Min FS (psi)	
A-A *On the Right of Gate Structure Base EL.514.1*	Normal	2.0	1.9	0.2	NOT REQUIRED
	Flood (PMF)	1.1	1.4	N/A	NOT REQUIRED
	Normal + Ice	2.0	1.3	3.9	REQUIRED
	Normal + Earthquake	1.3	1.0	1.6	NOT REQUIRED

GZA 2023 stability evaluation results.

Background Information

Additional Supporting Information	• Stability evaluations performed by GZA, 2023. • 1980 USACE Phase I Inspection Report.	Performance Monitoring Information	n/a

Evaluation Factors

Step/Node	Adverse (More Likely)	Favorable (Less Likely)	Surveillance & Monitoring Provisions
Flaw/Initiation	• Seepage has been historically observed at the downstream toe.	• Dam founded on bedrock.	• n/a
Continuation/Progression	• Preliminary sliding stability factors of safety for normal, flood and normal plus ice loading conditions are less than the USACE-recommended minimum values. • Lack of construction information regarding foundation treatment. • There are no available boring logs or in-situ testing available to validate assumed foundation properties.	• Preliminary sliding stability factors of safety exceed 1.0 for normal, flood and normal plus ice loading conditions. • No known construction problems or conditions are known to have been encountered according (per USACE Phase I report). • Section is assumed to be founded on bedrock, with sound rock exposed along the toe. Toe is free of debris, sediment, soils and vegetation, allowing for inspection to observe for signs of sliding or undermining erosion. • No visible misalignment at construction joints (vertical and horizontal).	• Visual inspections conducted every two years by VTDEC. • A post-earthquake inspection (October 17, 2012) was conducted with no visible/noticeable changes. • No unknown underwater inspections have been conducted.
Intervention	• Minimal ability to draw down the reservoir in the event that a section of the Spillway is observed to have displaced.	• Slow progressing failure mode – allows for time to intervene.	• Emergency Action Plan (2011).
Failure/Breach	• n/a	• n/a	• n/a

Risk Assessment	**Failure Likelihood**	
Category	☐ FL7 1/1 to 1/10 ☐ FL6 1/10 to 1/100 ☐ FL5 1/100 to 1/1,000 ☐ FL4 1/1,000 to 1/10,000 ☐ FL3 1/10,000 to 1/100,000 ☒ **FL2 1/100,000 to 1/1,000,000** ☐ FL1 < 1/1,000,000	
Discussion	The Risk Estimators discussed that seepage has historically been observed at the concrete to rock contact along the exposed bedrock slope, although there is not an indication that this seepage originates at the reservoir. Due to the massive nature of the spillway, it is likely that multiple opened bedrock joints would be required to generate sufficient uplift pressures to destabilize the dam, and the bedrock at the toe is generally observed to be sound and intact. The bedrock at the toe is less exposed at this section than at the Main Spillway; however, the overall height of this section is shorter, and thus there is a lower driving head for potential seepage flows. The Risk Estimators were in concurrence with a FL2 likelihood.	
Confidence	☐ High ☒ **Medium** ☐ Low	Due to the observed seepage at the concrete and bedrock interface.
Risk Assessment	**Life Safety Consequences**	
Category	☐ LS0 None ☐ LS1 0-1 ☒ **LS2 1-10** ☐ LS3 10-100 ☐ LS4 100-1,000 ☐ LS5 1,000-10,0000 ☐ LS6 >10,000	
General Discussion	The dam breach inundation model (GZA 2023) extends 10.8 miles downstream from Indian Brook to Mallett's Bay. The model includes 15 roads that cross the downstream flow path, private residences, and light commercial/industrial development within the floodplain.	
PAR and LoL	With the reservoir at the normal pool elevation, there are an estimated 31 structures, with a total PAR of 712, that would be within the limits of the flooded area when the dam breaches. Based upon the LifeSim modeling, the estimated loss of life is 3 people. The Risk Estimators ultimately concurred on LS2 (1-10 life loss) as the estimated loss of life for a sunny day breach of the dam is 3. It is noted that this PFM is expected to have a lower potential PAR and LOL than the similar failure mode for deterioration of the foundation at the Main Spillway, due to the shorter height of the structure.	
Confidence	☐ High ☒ **Medium** ☐ Low	Due to the screening level PAR and LoL estimations.

Potential Interim Risk Reduction Measures/Potential Dam Safety Management Actions			
Risk Reduction Measures	• n/a	EAP	• Perform regular tabletop exercises and keep the EAP up to date.
Inspections and Actions	• Keep with formal inspection yearly basis, special event follow up inspections.	Follow Up Studies	• Refined geotechnical field studies (such as borings or bedrock joint mapping downstream of section) could justify increased friction factors, which would benefit stability evaluation. • Coring through the dam to obtain intact concrete to rock samples for laboratory cohesion testing is projected to be expensive, and may not be successful (particularly accounting for the low cohesion values required to obtain acceptable stability factors of safety).
Surveillance and Monitoring	• Photograph the seepage regularly (start with quarterly) from same vantage point / setting baseline for flow. Note areas of observed seepage, estimate/quantify amount of seepage, color, etc.	Others	• n/a

	• Installing seepage collection point (such as a v-notch weir or “c” channel) to quantify seepage flow observed along the interface not practical; would be damaged/destroyed by overtopping flows.		

NON RISK-DRIVING FAILURE MODES

IBR-SPL-02-A-ASM SLIDING/OVERTURNING OF THE PRINCIPAL SPILLWAY AT A CONSTRUCTION JOINT

PFM Information					
Structure	Principal Spillway	PFM Failure Type	Stability	PFM Source	New
Loading Condition	Any	Location(s)	Principal Spillway	PFM Source Date	6/12/2023

PFM Description		PFM Sketch(es)
Flaw:	• Insufficient cross section (geometry and mass distribution) at the Auxiliary Spillway at the prominent mid-height construction joint causes hydrostatic destabilizing loads to overcome the resisting loads, causing the Auxiliary Spillway to slide downstream.	
Continuation/Progression:	• Under the theorized loading conditions [See Note 1 under Justification]. • The force acting on the structure in a downstream direction exceeds the concrete-foundation interface strength.	
Intervention:	• Intervention is not successful.	
Failure:	• A section of the spillway slides downstream. • A gap is formed in the dam. • An uncontrolled release of the reservoir occurs.	
PFM Disposition	☐ Ruled Out ☐ Clearly Negligible ☒ Asset Management ☐ Insufficient Info ☐ Financial/Damage State ☐ Credible ☐ Urgent	
Classification Justification	This potential failure mode is considered to have an Asset Management disposition. Note 1. GZA performed a stability evaluation for the concrete mass above the prominent horizontal construction joint at approximate elevation 514.1 feet, with assumed parameters (50 psi shear strength). The evaluation suggests that the spillway is stable (sliding factor of safety > 1.0) and has stability factors of safety well in excess of the USACE-recommended minimum values under normal, flood (PMF) normal plus ice and earthquake loading conditions. Although the section has demonstrated acceptable factors of safety, it is still prudent, in the opinion of GZA, to patch and repair the noted deteriorated horizontal and vertical construction joints observed at the downstream face. Continued seasonal freeze-thaw cycles will cause the condition of the joint to continue to deteriorate.	

Downstream face of the Principal Spillway, highlighting assumed horizontal construction joint weak plane (GZA, 2022)

Risk Assessment	Failure Likelihood & Life Safety Consequences
General Discussion	The Risk Assessment did not include this PFM. Based upon its “Asset Management” disposition, it is not considered a Risk Driver.

Potential Interim Risk Reduction Measures/Potential Dam Safety Management Actions			
Risk Reduction Measures	• Install an upstream liner to prevent leakage through the horizontal joints. -OR- Patch the upstream horizontal joints with SplashZone / underwater joint packing. • Repair spalled and deteriorated construction joints on downstream face.	EAP	• Perform regular tabletop exercises and keep the EAP up to date.
Inspections and Actions	• Keep with formal inspection yearly basis, special event follow up inspections. Perform underwater inspection if the seepage/deterioration changes significantly.	Follow Up Studies	• n/a
Surveillance and Monitoring	• Photograph the joints regularly (start with quarterly) from same vantage point / setting baseline for deterioration.	Others	• Develop schedule for repairing downstream spalled and deteriorated concrete joints.

IBR-SPL-05-N-NEG SEEPAGE THROUGH THE LEFT ABUTMENT OF THE PRINCIPAL SPILLWAY

PFM Information					
Structure	Principal Spillway	PFM Failure Type	Erosion/Scour	PFM Source	New
Loading Condition	Normal	Location(s)	Principal Spillway	PFM Source Date	6/12/2023

PFM Description		PFM Sketch(es)	
Flaw:	• Construction defect exists at the left abutment (rock hillside) contact with the dam, due to inadequate foundation preparation. Defect allows preferential seepage along the contact, increasing the destabilizing uplift pressures.		 *Downstream face of the Principal Spillway at the rock contact, where the Principal Spillway transitions upslope towards the Auxiliary Spillway (GZA, 2022)*
Continuation/Progression:	• The reservoir is at normal reservoir pool. • A preferential seepage path exists through a zone of inadequate foundation preparation (such as soil-infilled rock seam) which originates in the abutment and daylights downstream of the embankment. • Continued seepage along the path causes the joint to open, increasing seepage volume through the abutment. • The force acting on the structure in a downstream direction exceeds the resisting load of a rock mass along the abutment.		
Intervention:	• Increasing seepage volume goes unnoticed.		
Failure:	• A section of the abutment rock shifts downstream. • A gap is formed bypassing the dam. • An uncontrolled release of the reservoir occurs.		
PFM Disposition	☐ Ruled Out ☒ Clearly Negligible ☐ Asset Management ☐ Insufficient Info ☐ Financial/Damage State ☐ Credible ☐ Urgent		
Classification Justification	This potential failure mode is considered Clearly Negligible. Historically, seepage has been noted at the contact between the dam and the rock abutment. It is possible that increasing seepage flows could cause rock jointing in the abutment to open, destabilizing the abutment and causing isolated rock blocks to fail. However, there is currently no indication that the integrity of the hillside has been compromised. Additionally, concrete is by its nature generally resistant to erosion by low volume / low pressure water flow.		

Risk Assessment	Failure Likelihood & Life Safety Consequences
General Discussion	The Risk Assessment did not include this PFM. Based upon its "Clearly Negligible" disposition, it is not considered a Risk Driver.

Potential Interim Risk Reduction Measures/Potential Dam Safety Management Actions			
Risk Reduction Measures	• n/a	EAP	• Perform regular tabletop exercises and keep the EAP up to date.
Inspections and Actions	• Keep with formal inspection yearly basis, special event follow up inspections.	Follow Up Studies	• n/a
Surveillance and Monitoring	• Photograph the seepage regularly (start with quarterly) from same vantage point / setting baseline for flow. Note areas of observed seepage, estimate/quantify amount of seepage, color, etc. • Installing seepage collection point (such as a v-notch weir or "c" channel) to quantify seepage flow observed along the interface not practical; would be damaged/destroyed by overtopping flows.	Others	• n/a

IBR-SPL-06-F-ASM DEBRIS BUILDUP AT THE SPILLWAY RAISES THE RESERVOIR

PFM Information					
Structure	Principal Spillway	PFM Failure Type	Stability	PFM Source	New
Loading Condition	Flood	Location(s)	Principal Spillway	PFM Source Date	6/12/2023

PFM Description		PFM Sketch(es)
Flaw:	• A buildup of debris collects along the upstream face of the spillway, between the abutment and the intake, which hangs up on the underside of the service bridge. This throttles the discharge capacity of the spillway, causing the reservoir to rise above the PMF elevation.	
Continuation/Progression:	• There is a flood up to the PMF. • A debris field (trash, trees and other river borne debris) grows and becomes lodged upstream of the spillway, impeding flow over the spillway. • Due to the lost spillway capacity, the reservoir rises above the PMF elevation. • The force acting on the structure in a downstream direction exceeds the concrete-foundation interface strength at the foundation.	
Intervention:	• Intervention to break up the debris field to allow it to pass over the Principal Spillway is unsuccessful.	
Failure:	• A section of the dam slides downstream. • A gap is formed in the dam. • An uncontrolled release of the reservoir occurs.	
PFM Disposition	☐ Ruled Out ☐ Clearly Negligible ☒ Asset Management ☐ Insufficient Info ☐ Financial/Damage State ☐ Credible ☐ Urgent	
Classification Justification	This potential failure mode is considered to have an Asset Management disposition. There may be structural damage to the Service Bridge in the event of formation of a significant debris mat (or impact from debris passing over the spillway), which would require restoration to allow safe access to the Intake. Per the Owner, debris buildup on the reservoir, and at the dam, is limited, with most debris washed over the spillway during overtopping events. Were a debris mat to form to the left of the intake, assisted by debris (such as trees) hung up against the underside of the service bridge, 43 feet of the spillway could be blocked, with another 195 feet (Principal and Auxiliary) available for passing flow. There is also several feet of clearance between the top of the spillway and the underside of the Service Bridge, making it less likely that debris would impinge and be hung up below the bridge.	

Potential critical area for debris mat formation (GZA, 2022)

Risk Assessment	Failure Likelihood & Life Safety Consequences
General Discussion	The Risk Assessment did not include this PFM. Based upon its "Asset Management" disposition, it is not considered a Risk Driver.

Potential Interim Risk Reduction Measures/Potential Dam Safety Management Actions			
Risk Reduction Measures	• Implement a debris prevention/removal program to prevent the buildup of significant debris.	EAP	• Perform regular tabletop exercises and keep the EAP up to date.
Inspections and Actions	• Keep with formal inspection yearly basis, special event follow up inspections.	Follow Up Studies	• n/a
Surveillance and Monitoring	• n/a	Others	• n/a

IBR-SPL-07-F-R/O STRUCTURAL FAILURE OF SERVICE BRIDGE DURING FLOOD

PFM Information					
Structure	Principal Spillway	PFM Failure Type	Stability	PFM Source	New
Loading Condition	Flood	Location(s)	Principal Spillway	PFM Source Date	6/12/2023

PFM Description		
Flaw:	• The Service Bridge fails during a flood in such a manner that flow over the Principal Spillway is impeded. This throttles the discharge capacity of the spillway, causing the reservoir to rise above the PMF elevation.	PFM Sketch(es)
Continuation/Progression:	• There is a flood up to the PMF. • The Service Bridge fails structurally, dropping vertical downwards and resting high on the downstream face of the Principal Spillway. • Due to the lost spillway capacity, the reservoir rises above the PMF elevation. • The force acting on the structure in a downstream direction exceeds the concrete-foundation interface strength at the foundation.	
Intervention:	• Intervention to dislodge the Service Bridge to allow it to fall off the face of the Spillway is unsafe to perform, or unsuccessful.	
Failure:	• A section of the dam slides downstream. • A gap is formed in the dam. • An uncontrolled release of the reservoir occurs.	
PFM Disposition	☒ Ruled Out ☐ Clearly Negligible ☐ Asset Management ☐ Insufficient Info ☐ Financial/Damage State ☐ Credible ☐ Urgent	
Classification Justification	This potential failure mode is considered to be Ruled Out. If the Service Bridge to fail, it would require replacement to allow safe access to the Intake. Based upon the inspection completed in 2022, there were no obvious signs of overstress, no signs of significant steel deterioration, no areas of concern for significant rusting, and no obvious signs of unsafe performance of the Service Bridge. As can be seen in the photograph to the right, where the Service Bridge to fail at its supports and drop vertically downwards, its center of gravity is in line with the downstream face of the Spillway; this would encourage the Service Bridge to roll down off of the face and towards the toe. The PFMA Team could not postulate a mechanism in which failure of the Service Bridge would lead to failure of the dam. It is noted that although access to the Intake is not required for flood operations, it is recommended that the Owner continue to maintain and repair the Service Bridge, as necessary, to maintain its continued safe performance.	

Service Bridge, looking left to right along the Principal Spillway (GZA, 2022)

Risk Assessment	**Failure Likelihood & Life Safety Consequences**
General Discussion	The Risk Assessment did not include this PFM. Based upon its “Ruled Out” disposition, it is not considered a Risk Driver.

IBR-AUX-02-N-ASM SLIDING/OVERTURNING OF THE AUXILIARY SPILLWAY AT A CONSTRUCTION JOINT

PFM Information					
Structure	Auxiliary Spillway	PFM Failure Type	Stability	PFM Source	New
Loading Condition	Normal	Location(s)	Auxiliary Spillway	PFM Source Date	6/12/2023

PFM Description	
Flaw:	• Ongoing deterioration of the concrete at the construction joint.
Continuation/Progression:	• The reservoir is at normal pool. • Additional concrete loss at horizontal construction joints due to overtopping and seasonal freeze/thaw action. • Leakage begins along the joints. • The leakage causes the existing spalls to widen/deepen at the joints. • Increase of uplift along the construction joint.
Intervention:	• Concrete deterioration is noted, but repairs are not successful.
Failure:	• The force acting on the structure in a downstream direction exceeds the concrete-concrete interface strength. • A section of the dam slides downstream. • A gap is formed in the dam. • An uncontrolled release of the reservoir to the elevation of the joint occurs.
PFM Disposition	☐ Ruled Out ☐ Clearly Negligible ☒ Asset Management ☐ Insufficient Info ☐ Financial/Damage State ☐ Credible ☐ Urgent
Classification Justification	This potential failure mode is considered to have an Asset Management disposition. The Auxiliary Spillway is a relatively short height spillway section, with a prominent horizontal construction joint located approximately 5 feet below the ogee crest. The surface of this joint has noted spalling, and was previously patched in 2003. The Auxiliary Spillway was apparently constructed with three monoliths of approximately similar lengths; were the Auxiliary Spillway to fail at the construction joint, a flow path approximately 42 feet wide by 5 feet deep could be opened. The ensuing discharge would quickly route back to the downstream river channel, and cause the reservoir to lower about 4.5 feet from the normal pool to the top of the construction joint. It is not anticipated that this limited breach would cause life loss downstream of the dam, but repairs would be required to restore the failed concrete section.

PFM Sketch(es)

Downstream face of the Auxiliary Spillway, showing prominent horizontal construction joint (GZA, 2022)

Risk Assessment	Failure Likelihood & Life Safety Consequences
General Discussion	The Risk Assessment did not include this PFM. Based upon its “Asset Management” disposition, it is not considered a Risk Driver.

Potential Interim Risk Reduction Measures/Potential Dam Safety Management Actions			
Risk Reduction Measures	• Install an upstream liner to prevent leakage through the horizontal joints. -OR- Patch the upstream horizontal joints with SplashZone / underwater joint packing.	EAP	• Perform regular tabletop exercises and keep the EAP up to date.
Inspections and Actions	• Keep with formal inspection yearly basis, special event follow up inspections. • Perform underwater inspection if the seepage/deterioration changes significantly.	Follow Up Studies	• n/a
Surveillance and Monitoring	• Photograph the joints regularly (start with quarterly) from same vantage point / setting baseline for deterioration.	Others	• n/a

IBR-AUX-03-F-NEG EROSION/SCOUR OF THE DOWNSTREAM TOE OF THE AUXILIARY SPILLWAY

PFM Information					
Structure	Auxiliary Spillway	PFM Failure Type	Erosion/Scour	PFM Source	New
Loading Condition	Flood	Location(s)	Toe of the Auxiliary Spillway	PFM Source Date	6/12/2023

PFM Description		PFM Sketch(es)
Flaw:	• Overtopping of the principal spillway erodes the downstream toe of the Auxiliary Spillway, destabilizing the structure.	 Downstream toe of the Auxiliary Spillway (GZA, 2022)
Continuation/Progression:	• There is a storm event up to and including the PMF. • As flow continues over the spillway, the downstream channel begins to scour and erode. • The erosion progresses during subsequent floods until it reaches a critical distance beneath the spillway.	
Intervention:	• The erosion goes unnoticed.	
Failure:	• A section of the dam slides downstream. • A gap is formed in the dam. • An uncontrolled release of the reservoir occurs.	
PFM Disposition	☐ Ruled Out ☒ Clearly Negligible ☐ Asset Management ☐ Insufficient Info ☐ Financial/Damage State ☐ Credible ☐ Urgent	
Classification Justification	This potential failure mode is considered Clearly Negligible. The Auxiliary Spillway is founded on bedrock, with expected shallow bedrock covered by a thin soil layer at the toe. Based up historic hydraulic evaluation, the Auxiliary Spillway is predicted to overtop by 0.1 feet during the 1.5 x 100 year event (although it is noted that the overtopping during the PMF is expected to be more significant). It is not expected that this limited overtopping jet would have sufficient impact force to cause toe erosion/scour.	

Risk Assessment	Failure Likelihood & Life Safety Consequences
General Discussion	The Risk Assessment did not include this PFM. Based upon its "Clearly Negligible" disposition, it is not considered a Risk Driver.

IBR-AUX-05-N-NEG SEEPAGE THROUGH THE RIGHT ABUTMENT OF THE AUXILIARY SPILLWAY

PFM Information					
Structure	Auxiliary Spillway	PFM Failure Type	Erosion/Scour	PFM Source	New
Loading Condition	Normal	Location(s)	Auxiliary Spillway	PFM Source Date	6/12/2023

PFM Description		
Flaw:	• Construction defect exists at the right abutment (hillside) contact with the dam, due to inadequate foundation preparation. Defect allows preferential seepage along the contact, increasing the destabilizing uplift pressures.	PFM Sketch(es)
Continuation/Progression:	• The reservoir is at normal reservoir pool. • A preferential seepage path exists through a zone of inadequate foundation preparation (such as soil-infilled rock seam) which originates in the abutment and daylights downstream of the embankment. • Continued seepage along the path causes the joint to open, increasing seepage volume through the abutment. • The force acting on the structure in a downstream direction exceeds the resisting load of a rock mass along the abutment.	
Intervention:	• Increasing seepage volume goes unnoticed.	
Failure:	• A section of the abutment rock shifts downstream. • A gap is formed bypassing the dam. • An uncontrolled release of the reservoir occurs.	
PFM Disposition	☐ Ruled Out ☒ Clearly Negligible ☐ Asset Management ☐ Insufficient Info ☐ Financial/Damage State ☐ Credible ☐ Urgent	
Classification Justification	This potential failure mode is considered Clearly Negligible. The Auxiliary Spillway has a tapering height, and is relatively short (less than 4 feet tall) at the Abutment. Historically, noticeable seepage has not been noted at the contact between the dam and the rock abutment. It is possible that increasing seepage flows could cause rock jointing in the abutment to open, destabilizing the abutment and causing isolated rock blocks to fail. However, there is currently no indication that the integrity of the hillside has been compromised. Additionally, concrete is by its nature generally resistant to erosion by low volume / low pressure water flow.	

Termination of the Auxiliary Spillway at the Right Abutment (GZA, 2022)

Risk Assessment	**Failure Likelihood & Life Safety Consequences**
General Discussion	The Risk Assessment did not include this PFM. Based upon its “Clearly Negligible” disposition, it is not considered a Risk Driver.

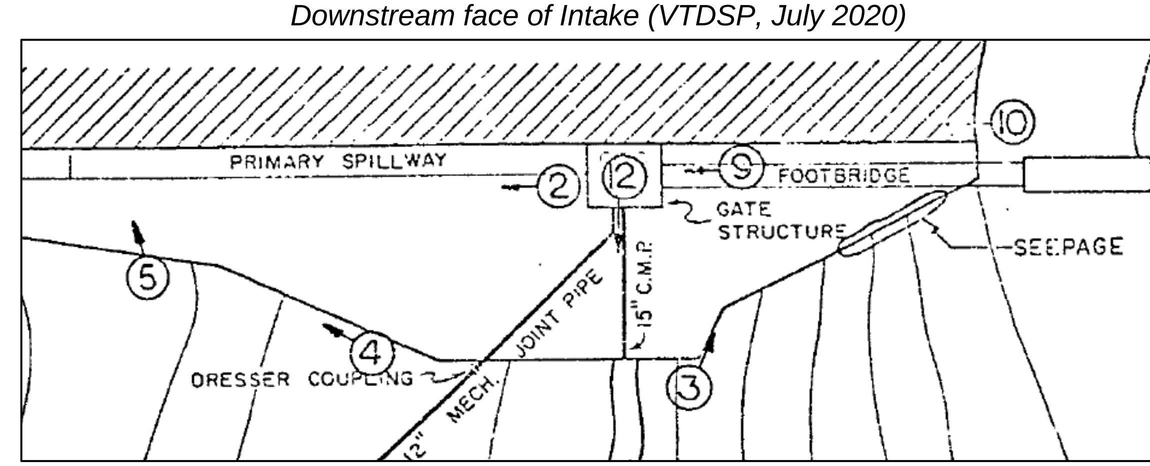
Potential Interim Risk Reduction Measures/Potential Dam Safety Management Actions			
Risk Reduction Measures	• n/a	EAP	• Perform regular tabletop exercises and keep the EAP up to date.
Inspections and Actions	• Keep with formal inspection yearly basis, special event follow up inspections.	Follow Up Studies	• n/a
Surveillance and Monitoring	• Photograph the abutment contact regularly (start with quarterly) from same vantage point / setting baseline for flow. Note areas of observed seepage, estimate/quantify amount of seepage, color, etc.	Others	• n/a

IBR-INT-02-IE-R/O Separation/Rotation of the Intake at a Construction Joint

PFM Information					
Structure	Intake	PFM Failure Type	Stability	PFM Source	New
Loading Condition	Ice, Earthquake	Location(s)	Intake	PFM Source Date	6/12/2023

PFM Description		PFM Sketch(es)
Flaw:	• The Intake is essentially an empty vertical chamber in the Principal Spillway, at a construction joint. Due to the reduced vertical weight at the Intake, the concrete monolith has a reduced lateral load resistance capacity, causing the monolith to rotate downstream.	
Continuation/Progression:	• Under the theorized loading conditions [See Note 1 under Justification]. • The force acting on the structure in a downstream direction exceeds the concrete-foundation interface strength locally at the location of the Intake.	
Intervention:	• Intervention is not successful.	
Failure:	• A section of the spillway rotates downstream. • A gap is formed in the dam. • An uncontrolled release of the reservoir occurs.	
PFM Disposition	☒ Ruled Out ☐ Clearly Negligible ☐ Asset Management ☐ Insufficient Info ☐ Financial/Damage State ☐ Credible ☐ Urgent	
Classification Justification	This potential failure mode is considered to be Ruled Out. It is known that the vertical weight (resisting force) of a representative cross section through the Intake is less than the adjacent portions of the Main Spillway. However, rotation and sliding of the Spillway would need to occur for a full concrete monolith. The Intake is located near to the end of the monolith (photograph to the right). For the monolith to rotate downstream, it would need to rotate about a fixed point near the abutment, overcome the shear resistance at the vertical construction joint, and essentially shear through or form a vertical full height crack along the vertical construction joint. By inspection, this is determined to not be a possible failure mechanism, and as such this PFM has been assigned a Ruled Out disposition.	

Downstream face of Intake (VTDSP, July 2020)



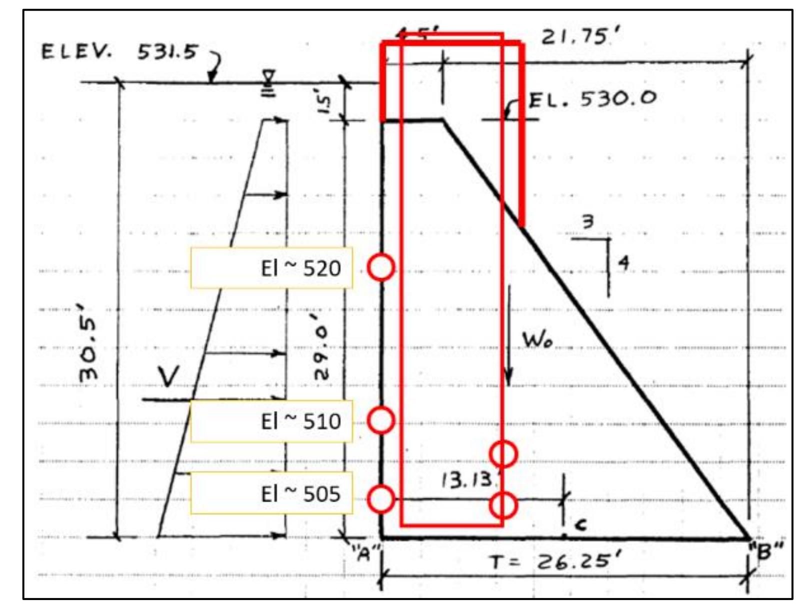
Plan view of Indian Brook Reservoir Dam, near Intake (USACE 1980)

Risk Assessment	Failure Likelihood & Life Safety Consequences
General Discussion	The Risk Assessment did not include this PFM. Based upon its "Ruled Out" disposition, it is not considered a Risk Driver.

IBR-INT-03-IE-NEG Structural Failure of the Intake Headwall

PFM Information					
Structure	Intake	PFM Failure Type	Structural Failure	PFM Source	New
Loading Condition	Flood	Location(s)	Intake	PFM Source Date	6/12/2023

PFM Description		PFM Sketch(es)
Flaw:	• During an extreme loading event, the hydrostatic pressure on the Intake structure headwall (upstream face of the Intake) exceeds the structural capacity of the concrete. The headwall fails structurally.	See sketch below
Continuation/Progression:	• There is a flood up to the PMF. • The hydrostatic loading on the headwall (upstream face) of the intake exceeds the concrete structural capacity, and the upstream concrete wall fails. • The Intake gate chamber fills with water. • The downstream face of the intake additionally fails.	
Intervention:	• Intervention is not successful.	
Failure:	• A limited breach of the spillway occurs, with a top elevation of approximately 520 to 522 feet. • An uncontrolled release of the reservoir occurs.	
PFM Disposition	☐ Ruled Out ☒ Clearly Negligible ☐ Asset Management ☐ Insufficient Info ☐ Financial/Damage State ☐ **Credible** ☐ Urgent	
Classification Justification	This potential failure mode is considered to be Clearly Negligible. There is no indication that the Intake headwall is or has bene overstressed, and the water level in the Intake chamber is near to the bottom of the chamber, suggesting that there is no concrete jointing allowing significant water inflow through the concrete. <u>As a discussion on consequences</u>: Were the headwall to fail and the Intake chamber to fill with water, the sliding stability of the structure would likely not change significantly. With the chamber full of water, there would be additional vertical dead load, to help to stabilize the structure. There would also not be an uncontrolled release of water under this scenario. Were the downstream face of the Intake to also fail (due to the water pressure associated with the flood), a flow path approximately 10 feet wide to elevation 520 to 522 feet would develop. This would be flow in addition to the overtopping flow from the PMF.	



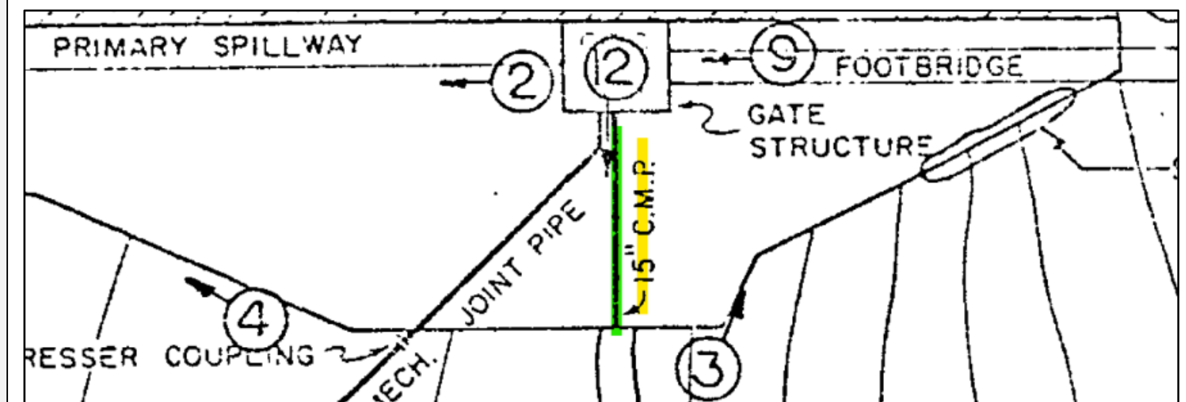
Cross section, 1988 G & Underwood Engineering Report with annotations by GZA to depict Intake (three inlets, chamber and two outlets)

Risk Assessment	Failure Likelihood & Life Safety Consequences
General Discussion	The Risk Assessment did not include this PFM. Based upon its "Clearly Negligible" disposition, it is not considered a Risk Driver.

IBR-INT-04-N-R/O FAILURE OF THE CMP AT THE TOE OF THE INTAKE DURING RELEASE FROM LOW LEVEL OUTLET

PFM Information					
Structure	Intake	PFM Failure Type	Erosion/Scour	PFM Source	New
Loading Condition	Normal	Location(s)	CMP at the toe of the Intake	PFM Source Date	6/12/2023

PFM Description		PFM Sketch(es)
Flaw:	• The Low-Level Outlet gate is opened, to lower the reservoir through the 15" diameter corrugated metal pipe (CMP) that daylights beyond the toe of the dam. The CMP fails just downstream of the toe, causing the Low-Level Outlet flow to impinge directly on the bedrock at the toe.	
Continuation/Progression:	• The reservoir is at normal pool. • The Low-Level Outlet gate is opened, and the 15" CMP pipe flows full. • The CMP is in an aged/rusted condition, and fails under the applied water pressure at the point of exit from the Intake. • Jet force from 15" pipe flowing full impinges on the thin soil cover and bedrock directly at the toe. • Erosion/scour goes unnoticed and progresses and the intake is undermined. • The force acting on the structure in a downstream direction exceeds the concrete-foundation interface strength at the foundation.	
Intervention:	• Likely that the Low-Level Outlet will be opened for a reason, so may not be able to close the gate.	
Failure:	• A section of the dam slides downstream. • A gap is formed in the dam. • An uncontrolled release of the reservoir occurs.	
PFM Disposition	☒ Ruled Out ☐ Clearly Negligible ☐ Asset Management ☐ Insufficient Info ☐ Financial/Damage State ☐ Credible ☐ Urgent	
Classification Justification	This potential failure mode is considered to be Ruled Out as a mechanism for failure of the dam. As can be seen in the photograph to the right, the interior of the CMP is rusted, and could theoretically fail when the Low-Level Outlet is opened fully. However, the PFM Team was unable to postulate a scenario where failure of the CMP, either within the limits of the Intake or immediately at the toe, would potentially cause the dam to destabilize and fail downstream. The jet force from a 15" diameter pipe flowing full for a short duration is not expected to cause significant deterioration to the concrete (where the CMP to fail inside the Intake) or cause significant enough scour at the toe to undermine the Intake. It is noted that the Low-Level Outlet has been operated in the past, without noted scour of the river channel bedrock.	



Plan view of Indian Brook Reservoir Dam, near Intake, highlighting the Low-Level Outlet and CMP pipe (USACE 1980)

Interior of CMP, at the downstream end outside of the limits of the Intake (VTDSP, July 2007)

Risk Assessment	Failure Likelihood & Life Safety Consequences
General Discussion	The Risk Assessment did not include this PFM. Based upon its "Ruled Out" disposition, it is not considered a Risk Driver.

Potential Interim Risk Reduction Measures/Potential Dam Safety Management Actions			
Risk Reduction Measures	• Rehab/replace the exposed section of the CMP (slipline, remove/replace with concrete, etc.).	EAP	• Consider pre-alerting EAP contact list if the Low-Level Outlet is going to be operated.
Surveillance and Monitoring	• Look for signs of concrete being washed out of the Low-Level Outlet when the gate is opened.	Others	• Perform a camera inspection through the CMP. May involve jetting/clearing prior to inspection.

IBR-INT-05-A-R/O FAILURE OF ONE OF THE THREE INTAKE GATES, CLOSED POSITION

PFM Information					
Structure	Intake	PFM Failure Type	Operational/Gate Failure	PFM Source	New
Loading Condition	Any	Location(s)	Intake	PFM Source Date	6/12/2023

PFM Description			
Flaw:	• One (or more) of the three intake gates fail in the closed position, either through binding in the slot, misalignment, or failure of the gate stem.	PFM Sketch(es)	*n/a*
PFM Disposition	☒ Ruled Out ☐ Clearly Negligible ☐ Asset Management ☐ Insufficient Info ☐ Financial/Damage State ☐ Credible ☐ Urgent		
Classification Justification	This potential failure mode was discussed during Brainstorming, and the PFM Team agreed that it should be considered Ruled Out as a stand-alone failure mode. The various gates and discharges at the Intake are not operated as part of the Flood Operations to release water in anticipation of a major storm event; at present, no human interaction is required at the site to accommodate flood discharges. Failure of an intake gate may restrict routine dam Operations, however there is redundancy where all three gates service one singular chamber. The PFM Team was unable to advance this failure mode to failure of the dam.		

Risk Assessment	**Failure Likelihood & Life Safety Consequences**
General Discussion	The Risk Assessment did not include this PFM. Based upon its “Ruled Out” disposition, it is not considered a Risk Driver.

IBR-INT-06-A-R/O FAILURE OF ONE OF THE THREE INTAKE GATES, OPEN POSITION

PFM Information					
Structure	Intake	PFM Failure Type	Operational/Gate Failure	PFM Source	New
Loading Condition	Any	Location(s)	Intake	PFM Source Date	6/12/2023

PFM Description			
Flaw:	• One (or more) of the three intake gates fail in the open position, either through binding in the slot or complete structural failure of the gate.	PFM Sketch(es)	*n/a*
PFM Disposition	☒ Ruled Out ☐ Clearly Negligible ☐ Asset Management ☐ Insufficient Info ☐ Financial/Damage State ☐ Credible ☐ Urgent		
Classification Justification	This potential failure mode was discussed during Brainstorming, and the PFM Team agreed that it should be considered Ruled Out as a stand-alone failure mode. Were one of the three intake gates to fail in a fully or partially opened position, the Intake gate chamber would fill with water to the elevation of the reservoir. This would essentially stabilize the intake, by adding a vertical dead load on the structure. The various gates and discharges at the Intake are not operated as part of the Flood Operations to release water in anticipation of a major storm event; at present, no human The PFM Team was unable to advance this failure mode to failure of the dam.		

Risk Assessment	Failure Likelihood & Life Safety Consequences
General Discussion	The Risk Assessment did not include this PFM. Based upon its "Ruled Out" disposition, it is not considered a Risk Driver.

IBR-INT-07-A-R/O FAILURE OF ONE OF THE OUTLET GATES, CLOSED POSITION

PFM Information					
Structure	Intake	PFM Failure Type	Operational/Gate Failure	PFM Source	New
Loading Condition	Any	Location(s)	Intake	PFM Source Date	6/12/2023

PFM Description			
Flaw:	• One (or more) of the two outlet gates fail in the closed position, either through binding in the slot, misalignment, or failure of the gate stem.	PFM Sketch(es)	*n/a*
PFM Disposition	☒ Ruled Out ☐ Clearly Negligible ☐ Asset Management ☐ Insufficient Info ☐ Financial/Damage State ☐ Credible ☐ Urgent		
Classification Justification	This potential failure mode was discussed during Brainstorming, and the PFM Team agreed that it should be considered Ruled Out as a stand-alone failure mode. The various gates and discharges at the Intake are not operated as part of the Flood Operations to release water in anticipation of a major storm event; at present, no human interaction is required at the site to accommodate flood discharges. Failure of an outlet gate in the closed position may restrict routine dam Operations, but the PFM Team was unable to advance this failure mode to failure of the dam.		

Risk Assessment	Failure Likelihood & Life Safety Consequences
General Discussion	The Risk Assessment did not include this PFM. Based upon its “Ruled Out” disposition, it is not considered a Risk Driver.

IBR-INT-08-A-ASM FAILURE OF THE LOW-LEVEL OUTLET GATE, OPEN POSITION

PFM Information					
Structure	Intake	PFM Failure Type	Operational/Gate Failure	PFM Source	New
Loading Condition	Any	Location(s)	Intake	PFM Source Date	6/12/2023

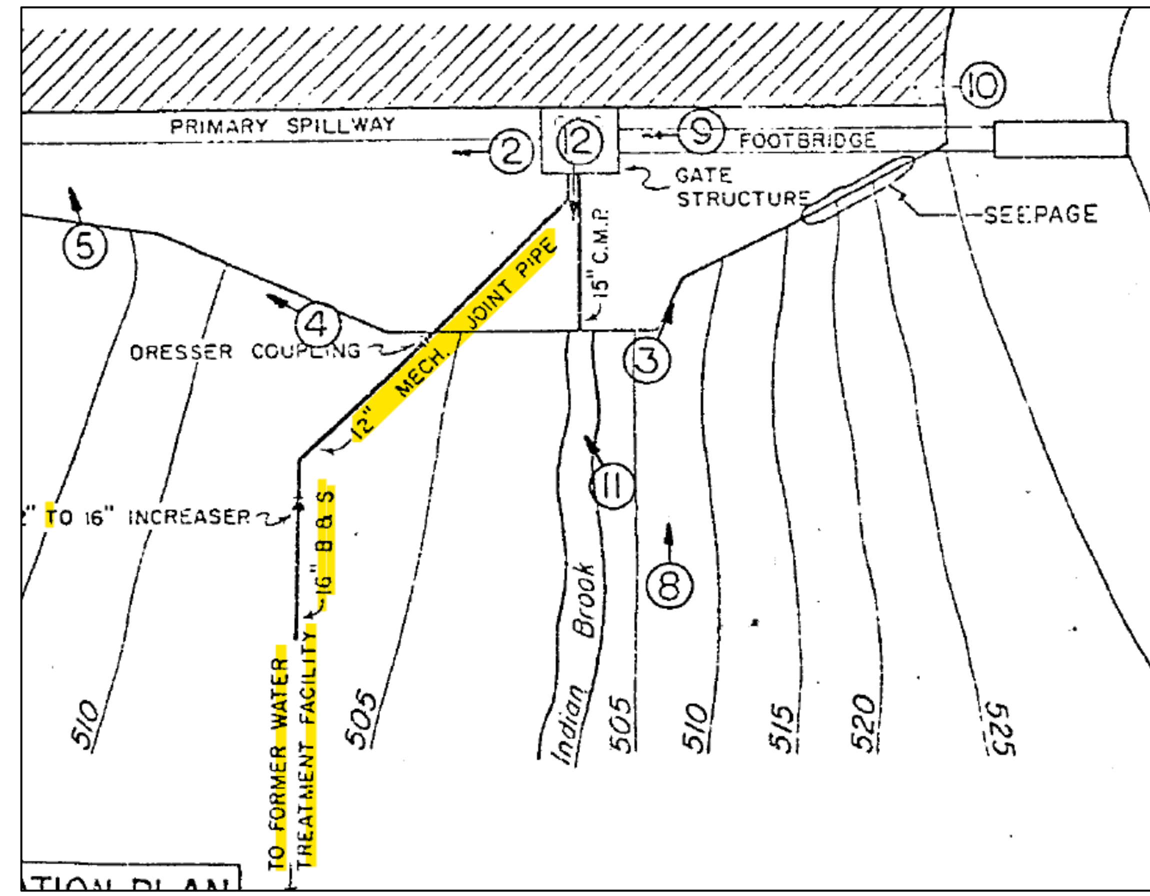
PFM Description		PFM Sketch(es)	
Flaw:	• The Low-Level Outlet gate fails in the open position, either through binding in the slot or complete structural failure of the gate.	PFM Sketch(es)	*n/a*
PFM Disposition	☐ Ruled Out ☐ Clearly Negligible ☒ Asset Management ☐ Insufficient Info ☐ Financial/Damage State ☐ Credible ☐ Urgent		
Classification Justification	This potential failure mode was discussed during Brainstorming, and the PFM Team agreed that it would not contribute to potential loss of life. Were the Low-Level Outlet gate to fail in the open position, any water introduced to the Intake gate chamber would drain through the 15” CMP and to the natural downstream river channel. In the unlikely event that one of the headgates was also failed in an open position, there may not be a way to prevent draining the reservoir to the invert elevation of the failed intake gate. As such, it is recommended by GZA that this failure mode be considered to have an Asset Management disposition, to highlight the sensitivity of maintaining operable intake gates to ensure that there is a way to prevent draining the reservoir in the event that the Low-Level Outlet gate fails in an open position.		

Risk Assessment	Failure Likelihood & Life Safety Consequences
General Discussion	The Risk Assessment did not include this PFM. Based upon its “Asset Management” disposition, it is not considered a Risk Driver.

IBR-INT-09-A-INS FAILURE OF THE WATER TREATMENT FACILITY OUTLET GATE, OPEN POSITION

PFM Information					
Structure	Intake	PFM Failure Type	Operational/Gate Failure	PFM Source	New
Loading Condition	Any	Location(s)	Intake	PFM Source Date	6/12/2023

PFM Description		PFM Sketch(es)
Flaw:	• The gate controlling flow to the former Water Treatment Plant fails in the open position, either through binding in the slot or complete structural failure of the gate.	
PFM Disposition	☐ Ruled Out ☐ Clearly Negligible ☐ Asset Management ☒ Insufficient Info ☐ Financial/Damage State ☐ Credible ☐ Urgent	
Classification Justification	According to the drawings included in the USACE 1980 Phase 1 report, a 12" diameter pipe exited from the Intake gate chamber, daylighted at the toe, and continued downstream to a former water treatment facility. Presently, there is no indication of the 12" diameter pipe, or the apparent increase to a 16" diameter pipe, above ground level. The location where this pipe daylights at the toe has also been sought during previous inspections, and has not been identified. It is unknown therefore if, or how, this pipe was terminated or the current condition of this pipe. It is also unknown if or how the gate operator in the Intake gate chamber was disconnected or plugged. GZA proposes to assign this failure mode an Insufficient Information disposition. The rows below identify Risk Reduction Measures that can be employed to resolve the unknown information, and redispose this failure mode to either "Ruled out" (most likely) or "Financial/Damage State."	



Plan view of Indian Brook Reservoir Dam, near Intake, highlighting the bypass pipe to the former water treatment facility (USACE 1980)

Risk Assessment	Failure Likelihood & Life Safety Consequences
General Discussion	The Risk Assessment did not include this PFM, despite the PFM having an "Insufficient Information" disposition. The Risk Facilitator and Estimators discussed that if an uncontrolled release were to occur through the water treatment facility outlet line, the magnitude of the flow would not be significant enough to pose potential life safety risk downstream. The pipe is embedded in mass concrete and the bedrock at the toe of the spillway is sound, and as such erosion spalling or scour was not deemed to be likely. It was agreed that this PFM is not a Risk Driver.

Potential Interim Risk Reduction Measures/Potential Dam Safety Management Actions			
Risk Reduction Measures	• n/a	EAP	• n/a
Inspections and Actions	• Perform non-invasive geotechnical exploration techniques (such as ground penetrating radar or electrical resistivity testing) to potentially locate the buried 12" diameter pipe, if the proposed Follow Up Studies are nonconclusive.	Follow Up Studies	• Dewater the Intake chamber (open the Low-Level Outlet gate with all three Intake gates closed) to allow for inspection entry to the Intake chamber. Inspect the Water Treatment Facility Outlet gate and operator, to determine if that pipe has been sealed. • If the Intake chamber cannot be dewatered, perform a camera inspection Water Treatment Facility Outlet gate and operator.
Surveillance and Monitoring	• n/a	Others	• n/a

IBR-GNR-01-A-R/O ROGUE WAVE OVERTOPPING

PFM Information					
Structure	General	PFM Failure Type	Overtopping	PFM Source	New
Loading Condition	Normal, Earthquake	Location(s)	Main and Auxiliary Spillways	PFM Source Date	6/12/2023

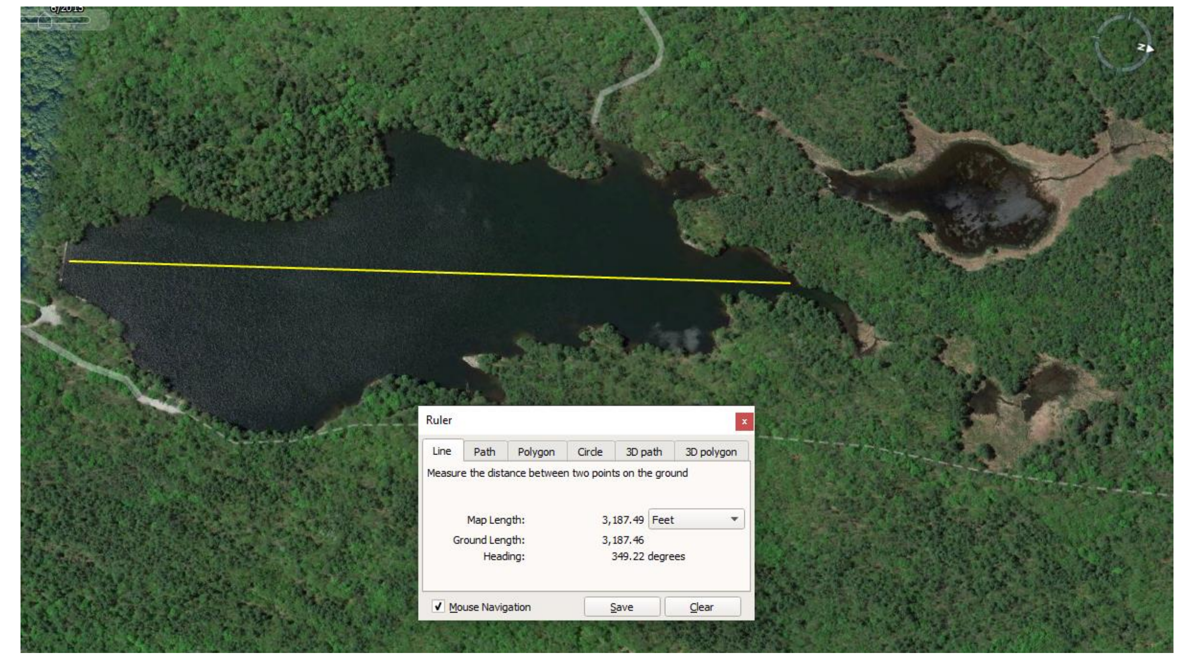
PFM Description			
Flaw:	• Large rogue wave develops on the reservoir (from landslide, seismic event, wind gust) and overtops the Spillway.	PFM Sketch(es)	*n/a*
Continuation/Progression:	• A single, short duration wave overtops the Principal and Auxiliary Spillway.		
Intervention:	• n/a.		
Failure:	• The PFM Team could not advance this failure mode to failure.		
PFM Disposition	☒ Ruled Out ☐ Clearly Negligible ☐ Asset Management ☐ Insufficient Info ☐ Financial/Damage State ☐ **Credible** ☐ Urgent		
Classification Justification	This potential failure mode was discussed during Brainstorming, and the PFM Team agreed that it should be considered Ruled Out. The water retaining barrier at the dam is concrete, which is not susceptible to damage from a single, isolated rogue wave. The PFM Team could not develop this failure mode to dam failure.		

Risk Assessment	**Failure Likelihood & Life Safety Consequences**
General Discussion	The Risk Assessment did not include this PFM. Based upon its "Ruled Out" disposition, it is not considered a Risk Driver.

IBR-GNR-02-A-R/O FAILURE OF THE UPSTREAM BEAVER DAM CAUSES RESERVOIR SURCHARGE

PFM Information					
Structure	General	PFM Failure Type	Overtopping	PFM Source	New
Loading Condition	Normal, Earthquake	Location(s)	Main and Auxiliary Spillways	PFM Source Date	6/12/2023

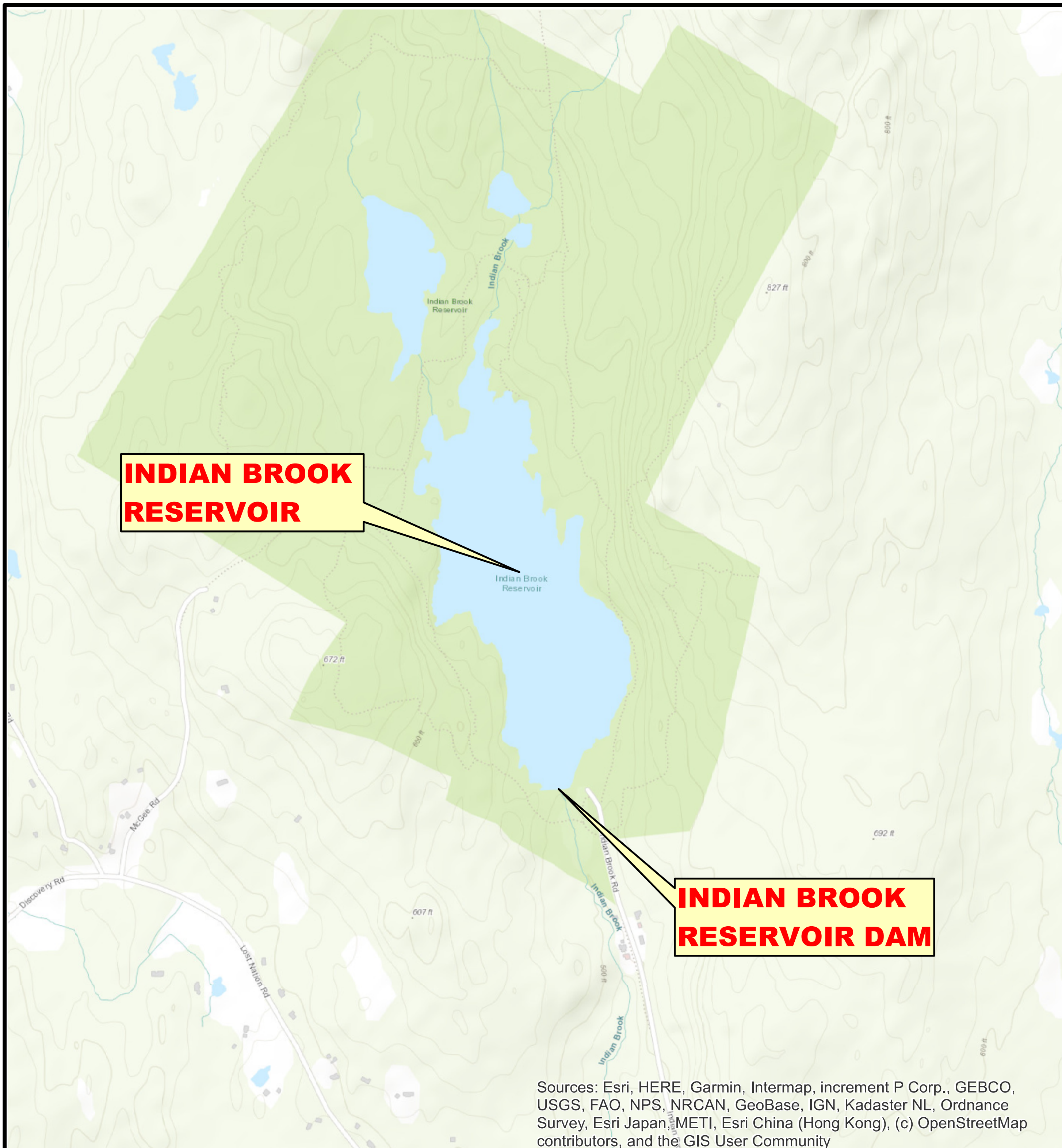
PFM Description		
Flaw:	• Beaver dam on the upstream end of the reservoir releases, allowing the impounded water to enter the Indian Brook Reservoir. • Large rogue wave develops on the reservoir (from landslide, seismic event, wind gust) and overtops the Spillway.	PFM Sketch(es)
Continuation/Progression:	• A single, short duration wave overtops the Principal and Auxiliary Spillway.	
Intervention:	• n/a.	
Failure:	• The PFM Team could not advance this failure mode to failure.	
PFM Disposition	☒ Ruled Out ☐ Clearly Negligible ☐ Asset Management ☐ Insufficient Info ☐ Financial/Damage State ☐ **Credible** ☐ Urgent	
Classification Justification	This potential failure mode was discussed during Brainstorming, and the PFM Team agreed that it should be considered Ruled Out. There are a few small ponds at the northern end of Indian Brook Reservoir, roughly 3000 feet north of the dam. Historic beaver dam activity has created small impoundments, which are either removed by people or naturally fail. Due to the limited impoundment size, and the resiliency of concrete structures to survive single, isolated rogue waves, the PFM Team could not develop this failure mode to dam failure.	

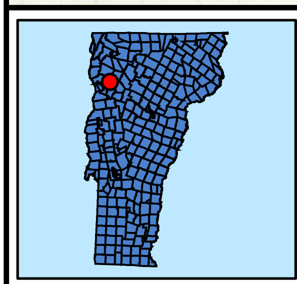


Indian Brook Reservoir, flow from right to left. (Google Maps)

Risk Assessment	**Failure Likelihood & Life Safety Consequences**
General Discussion	The Risk Assessment did not include this PFM. Based upon its "Ruled Out" disposition, it is not considered a Risk Driver.

KEY FIGURES





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N W E S

Data Supplied by : esri

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SCALE IN FEET

GZA	PROJ. MGR.: DJS DESIGNED BY: LTP REVIEWED BY: DJS OPERATOR: LTP DATE: 12/28/2022	SITE LOCATION PLAN INDIAN BROOK RESERVOIR DAM	JOB NO. 01.0175988.00
		VERMONT DEPARTMENT OF ENVIRONMENTAL CONSERVATION VERMONT DAM SAFETY PROGRAM ESSEX, VERMONT	FIGURE NO. 1

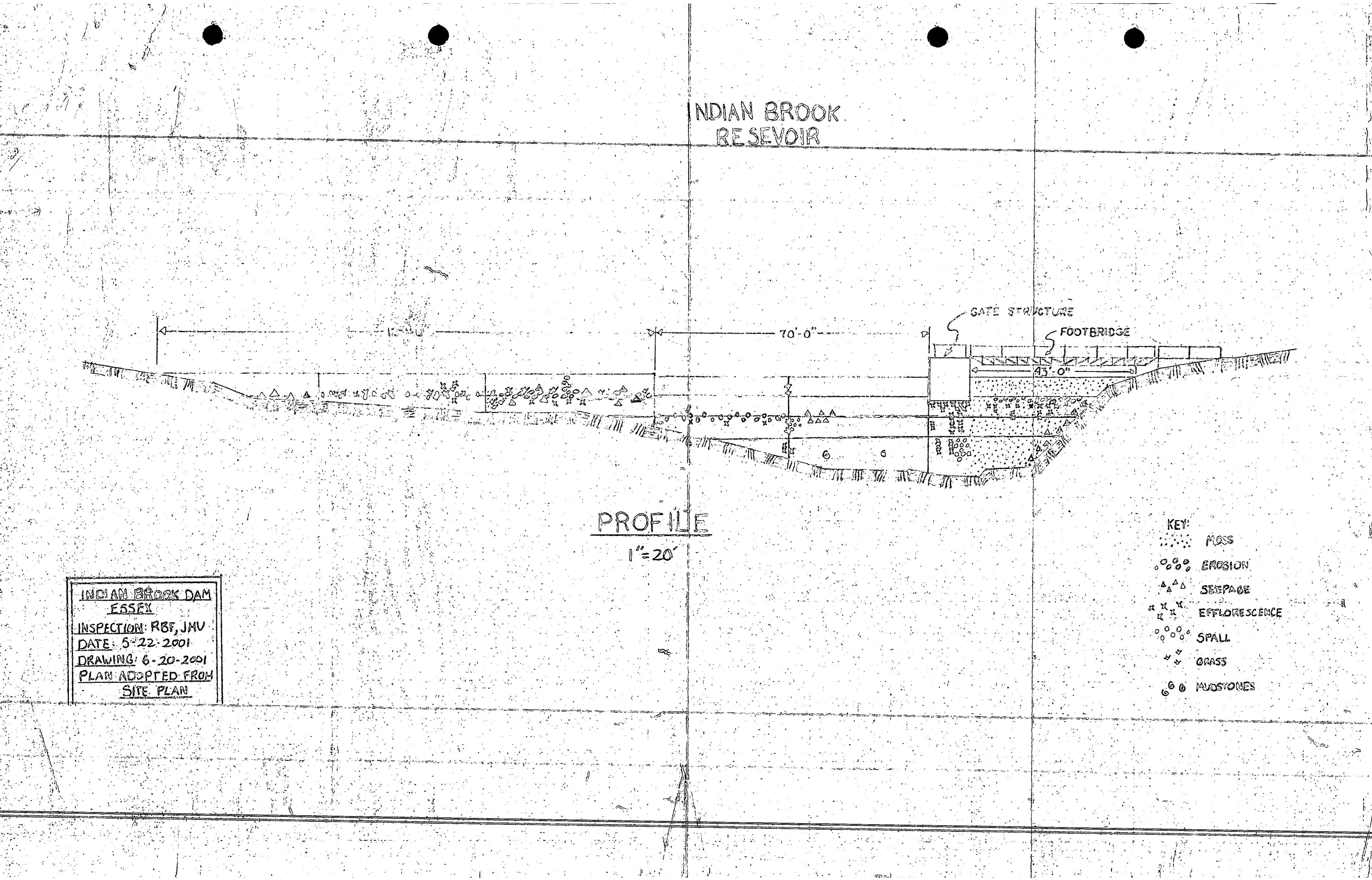


Figure 2. Section Sketch 2001

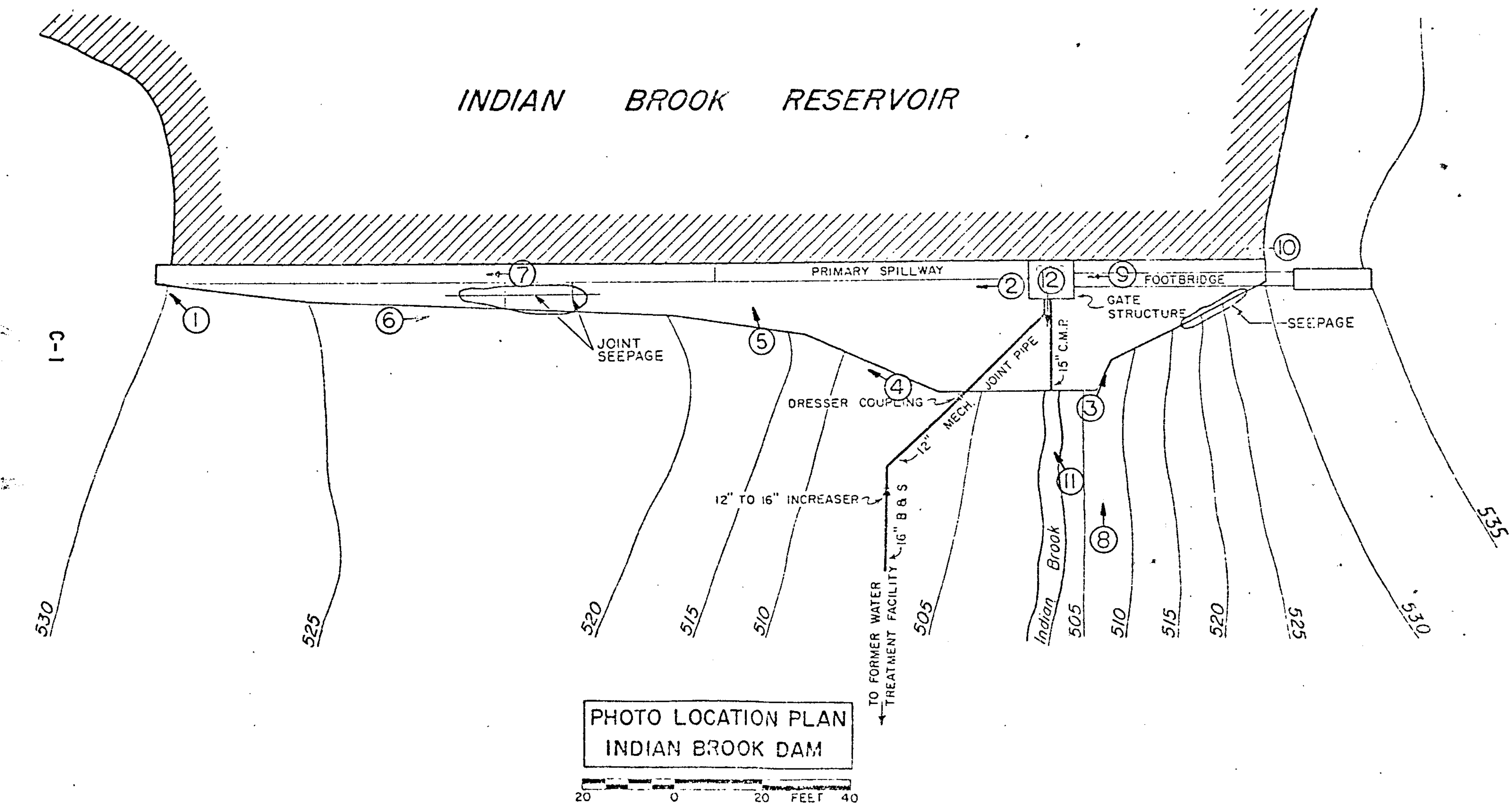


Figure 3. Plan Sketch

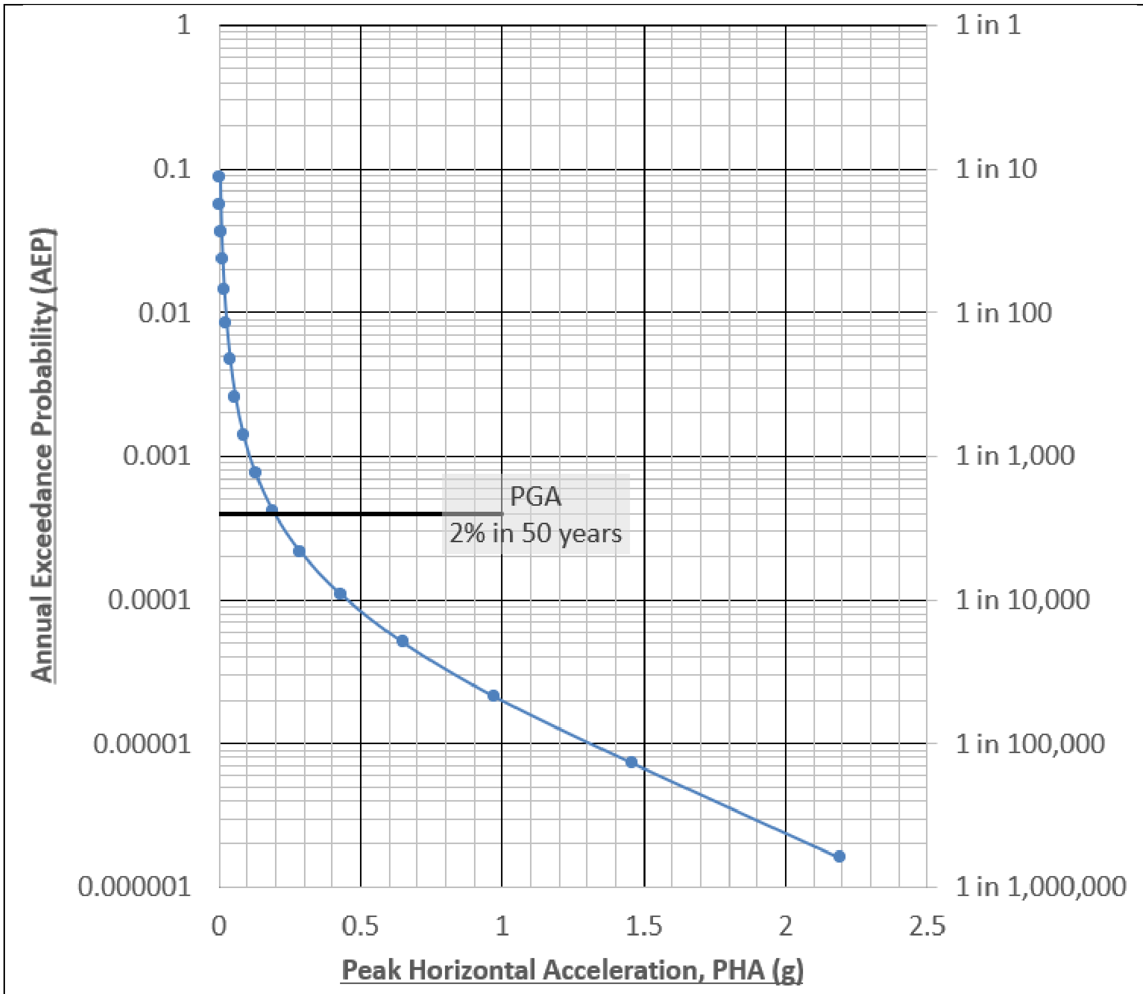
APPENDIX A - RISK WORKSHOP HANDOUTS

A1. SEISMIC HAZARD

Figure A1 presents the Peak Horizontal Seismic Acceleration at the project as a function of annual exceedance probability (AEP), with critical acceleration values tabulated to the right of the figure. The curve was developed utilizing the online USGS Unified Hazard Tool (https://earthquake.usgs.gov/hazards/interactive/) with the following direct inputs:

- Location: 44.5308 N, 73.097412 W
- Site Class B/C, based on limited available subsurface information. Founded on rock.

Figure A1 – Seismic Hazard Curve



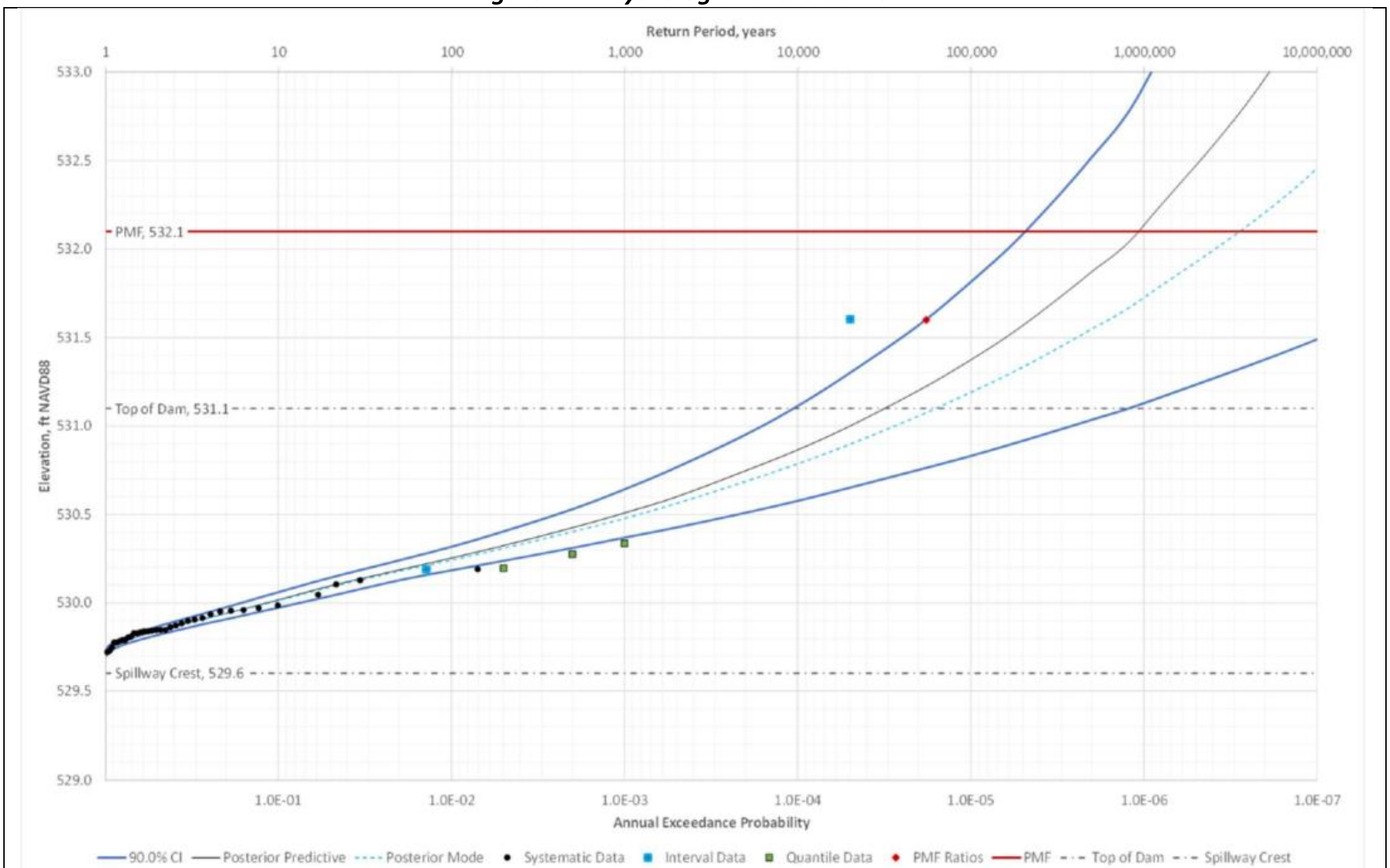
Embankment yield acceleration	
n/a AEP	**n/a** PHA
Peak Ground Acceleration (PGA)	
1 in 2,475 **4×10^{-4}** AEP	**0.198 g** PHA

A2. <u>HYDROLOGIC HAZARD</u>

Figure A2 presents the Hydrologic Hazard Curve for the project, which is a graph of the maximum reservoir level as a function of annual exceedance probability (AEP). The curve is a predictive model, based upon available USGS StreamStats statistics, historical flood estimates based on the 1927 flood, regional Paleo flood information from Ball Mountain Dam and probabilistic modeling based upon the developed HEC-HMS model.

The solid black dashed line ("Posterior Predictive") is the most frequent result derived from Monte Carlo simulation; this line can be considered to represent the AEP versus Headwater Elevation relationship. The blue dashed line ("Posterior Mode") is the statistical mode from the Monte Carlo simulation. The solid blue lines provide a 90% confidence interval band for the results. Key headwater elevation and AEP pairs are tabulated below the figure.

Figure A2 – Hydrologic Hazard Curve



	Reservoir Level		**AEP**		
	NGVD29	NAVD88			
Crest of principal spillway	530	529.6	1×10^{0}	1	Normal reservoir.
Crest of auxiliary spillway	530.5	530.1	5×10^{-2}	0.05	1 in 20 years
100-year flood	530.7	530.3	1×10^{-2}	0.01	1 in 100 years
PMF	532.5	532.1	1×10^{-6}	0.000001	1 in 1,000,000 years

A3. DETERMINATION OF POPULATION AT RISK

GZA used the United States Army Corps of Engineers (USACE) Life Loss Estimation software (LifeSim) which is a spatially distributed dynamic simulation modeling system capable of estimating Population At Risk. Simulations are performed using hydraulic data (i.e., depth grids and velocity grids) directly from HEC-RAS. In addition to providing the time series of depth and velocity (i.e., grid data), a hydrograph representing the hydraulic event is also provided for performing life loss/PAR calculations. The representative hydrograph is critical in developing the warning issuance timing relative to a specific condition (e.g., dam breach). The hazard occurrence for each simulation and representative hydrograph is set to be at the time of the initiation of the breach formation.

Structure data in the study area below the dam was imported from that National Structure Inventory (NSI) using the built in import features; simulations were run using the default occupancy type and building stability criteria. Emergency planning data was assigned to a single emergency planning zone representing the full extent of the study area. Preparedness for the Warning and Protective Action Data (i.e., Warning issuance delay, first alert, protective action initiation, and issuance to initiation) were all set to the default values of "unknown," which is a "worst case" conservative assumption. No traffic analysis was performed during these simulations. Agricultural data was imported directly from the U.S. Department of Agriculture (USDA) National Agricultural Statistics Service (NASS) Cropland Data Layer (CDL) database for the most recent year available (i.e., 2016).

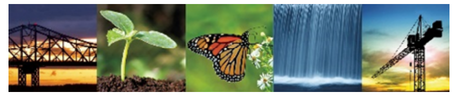
Simulations were setup assuming hazard occurrence happening at 12 different times throughout the day (e.g., 12 am, 2 am, 4 am, etc.) with a total number of 100 iterations which take into consideration the confidence limits of the Warning and Protective Action Data but do not impact PAR estimation. Full details from the evaluation can be found in report "Hydrologic and Hydraulic Analyses" prepared by GZA (see Appendix C).

A summary of information pertinent to the Risk Assessment is as follows:

Figure A3– Summary of Population at Risk

Simulation	**Structures Inundated**	**Total PAR**	**Estimated Life Loss**
No failure during Fair Weather	0	0	0
Fair Weather Failure	31	712	3
Fair Weather Failure (Incremental Increase)	31	712	3
No failure during PMF	*29*	*708*	*0*
Failure during PMF	*43*	*760*	*18*
PMF Failure (Incremental Increase)	14	52	18

APPENDIX B – GZA VISUAL INSPECTION REPORT



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May 9, 2023
File No. 01.0175988.00

Vermont Department of Environmental Conservation – Water Investment Division
Vermont Dam Safety Program
Davis Building – 3rd Floor
One National Life Drive
Montpelier, Vermont 05620-3510

Attn: Mr. Benjamin Green, P.E. | Section Chief - Dam Safety Engineer
Mr. Andrew Sampsell, P.E. | Dam Safety Engineer

Re: Visual Inspection Report
Indian Brook Reservoir – VT000055

Dear Mr. Green:

In accordance with our agreement dated December 1, 2022, GZA GeoEnvironmental, Inc. (GZA) is pleased to submit to the Vermont Dam Safety Program (VTDSP) – Vermont Department of Environmental Conservation (VTDEC), the following dam safety inspection for Indian Brook Reservoir Dam in Essex, Vermont. Our services and report are subject to the Limitations found in **Appendix A**.

BACKGROUND

The Indian Brook Reservoir Dam was constructed in 1957 based on a design by Whitman and Howard Engineering. The dam impounds the Indian Brook Reservoir, which was originally filled for water supply purposes; currently, the Reservoir is used strictly for recreation. The Town of Essex owns and maintains the dam. The dam is accessible off Indian Brook Road in Essex, Vermont. A site locus plan is provided in **Figure 1**.

The Dam is a 238-foot-long concrete gravity dam with a structural height of 31 feet. From left (looking downstream) to right, the Indian Brook Dam consists of the following primary water retaining structures:

- A 43-foot-long concrete ogee-style principal spillway, with a crest elevation of 530 feet. The spillway has a vertical upstream face, approximate 4 foot wide crest, and a 3H:4V sloping downstream face. A steel pedestrian bridge spans over this section, from the left abutment to the gate chamber, with a reported low-chord elevation of 533.0 feet.
- A 10-foot-long by 9-foot-wide gate chamber, cast integrally with the spillway. The gate chamber houses three 12-inch diameter inlet ports (elevations 505, 510 and 520) regulated by slide gates and protected by bar screens. The original design included two outlet ports; however, it is reported that the gates have not been operated for many years, and the outlet pipe is not visible at the toe of the spillway.
- A 70-foot-long concrete ogee-style principal spillway, with a crest elevation of 530 feet. Similar to the first spillway section, this section has a vertical upstream face, approximate 4- foot-wide crest, and a 3H:4V sloping downstream face.

- A 125-foot-long auxiliary spillway. The auxiliary spillway is similar in design to the two sections of the primary spillway, with a crest elevation 0.5 foot higher (approximate elevation 530.5 feet).

Historical site plans are provided in **Appendix D**.

The Indian Brook Reservoir Dam remains true to its original structure; however, it has had some surficial concrete repairs along horizontal joints in the downstream spillway face in 2003 to address on-going spalling and concrete deterioration problems. These 2003 repairs also included grouting the leaking joints and bedrock foundation and the installation of reinforcing bars in the drill holes used to inject the grout. These historical issues at the site have continued to redevelop since the 2003 repairs with extensive spalling along the lift joints of the downstream principal and auxiliary spillway faces. Vegetation present within these larger spalls indicate that seepage through both spillway faces may have redeveloped.

The dam is currently classified as a HIGH hazard potential.

VISUAL INSPECTION

On December 6, 2022, GZA engineers Derek J. Schipper, P.E., Kevin F. Finn, P.E., Lexus T. Pattershall, E.I.T., in the company of VTDSP engineer Andrew Sampsell, P.E., performed a visual inspection of Indian Brook Reservoir Dam. Representatives from the Town of Essex (Ann Costandi, P.E., Ally Vile, and Aaron Martin) were also present. The temperature was 43°F with light showers at the time of inspection. The water level was approximately 0.25 to 0.5 of an inch above the principal spillway crest; there was no overtopping at the auxiliary spillway. Photographs were taken and are provided in **Appendix B** with approximate locations in **Figure 2.** Underwater areas were not inspected. An inspection checklist developed by GZA is provided in **Appendix C.**

SUMMARY OF DAM DEFFICIENCIES

The following dam deficiencies were noted during the visual dam inspection:

Pedestrian Bridge

1. Minor surface spalling of the concrete platform at the left abutment (See Photo No. 17).
2. Steel walkway generally in adequate condition, with some minor staining/discoloration and coating loss, particularly on horizontally-oriented members (See Photo No. 4).

Principal Spillway

3. Concrete deterioration along the downstream face of the horizontal lift joint and the spillway crest (See Photo No. 4).
4. Minor missing grout between the downstream spillway face joint to the low-level outlet structure (See Photo No. 5).
5. Spalling at the horizontal construction lift joints of the downstream spillway face (See Photo No. 6).
6. Spalling present with iron-stained seepage at the rock contact with the toe of the principal spillway (See Photo No. 7).
7. Prominent horizontal construction joint with spalled joint faces, approximately 15 feet below the crest; the deteriorated joint extends the majority of the length of the principal spillway (See Photo No. 7 and 8).

8. Vegetation growth present throughout the spillway downstream face and crests.

Low-Level Outlet

9. Intake screens are warped and bulging (See Photo No. 16).
10. The three-slide gate-controlled outlets are reportedly inoperable for the low-level outlet.

Auxiliary Spillway

11. Seepage along the toe (See Photo No. 11). Seepage follows the elevation contour down the toe towards the tailrace.
12. Top of rebar exposed on spillway crest (See Photo No. 12).
13. Flowing water at the toe of the auxiliary spillway (See Photo No. 21).
14. Vegetation growth present throughout the spillway downstream faces and crests.

These observations are consistent with the findings noted in the VTDSP 2022 Periodic Inspection.

Should you have any questions, please do not hesitate to contact the undersigned.

Very truly yours,
GZA GEOENVIRONMENTAL, INC.

Derek J. Schipper, P.E.
Senior Consultant

Kevin F. Finn, P.E. (MA)
Consultant/Reviewer

James P. Guarente, P.E. (MA)
Principal-in Charge

Attachments:

Figure 1 – Site Locus

Figure 2 – Photo Location Plan

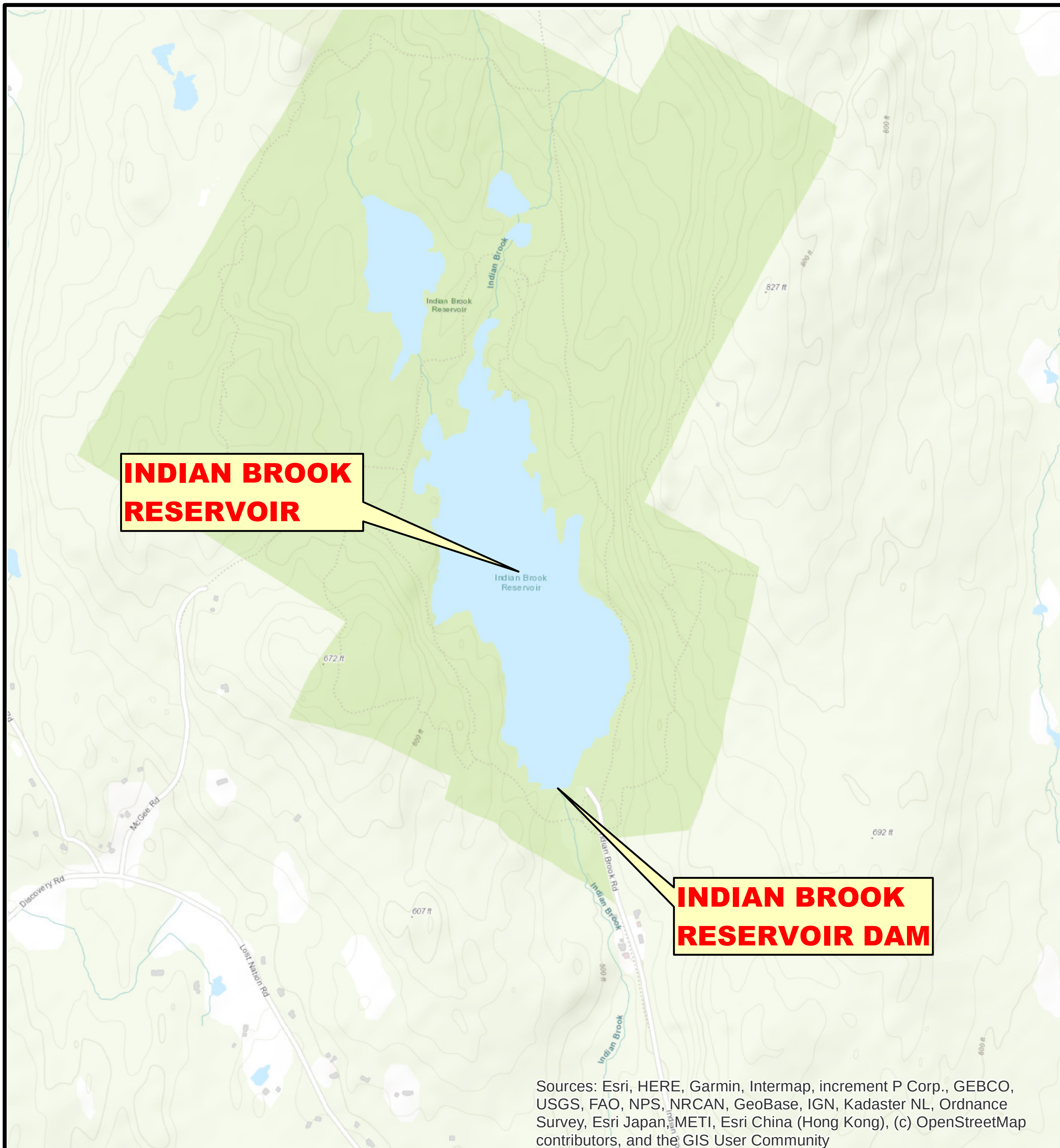
Appendix A – Limitations

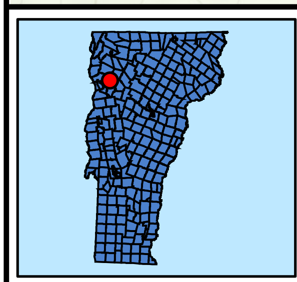
Appendix B – Inspection Photographs

Appendix C - Inspection Checklist

Appendix D – Historic Site Plans

FIGURES





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N W E S

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SCALE IN FEET

GZA

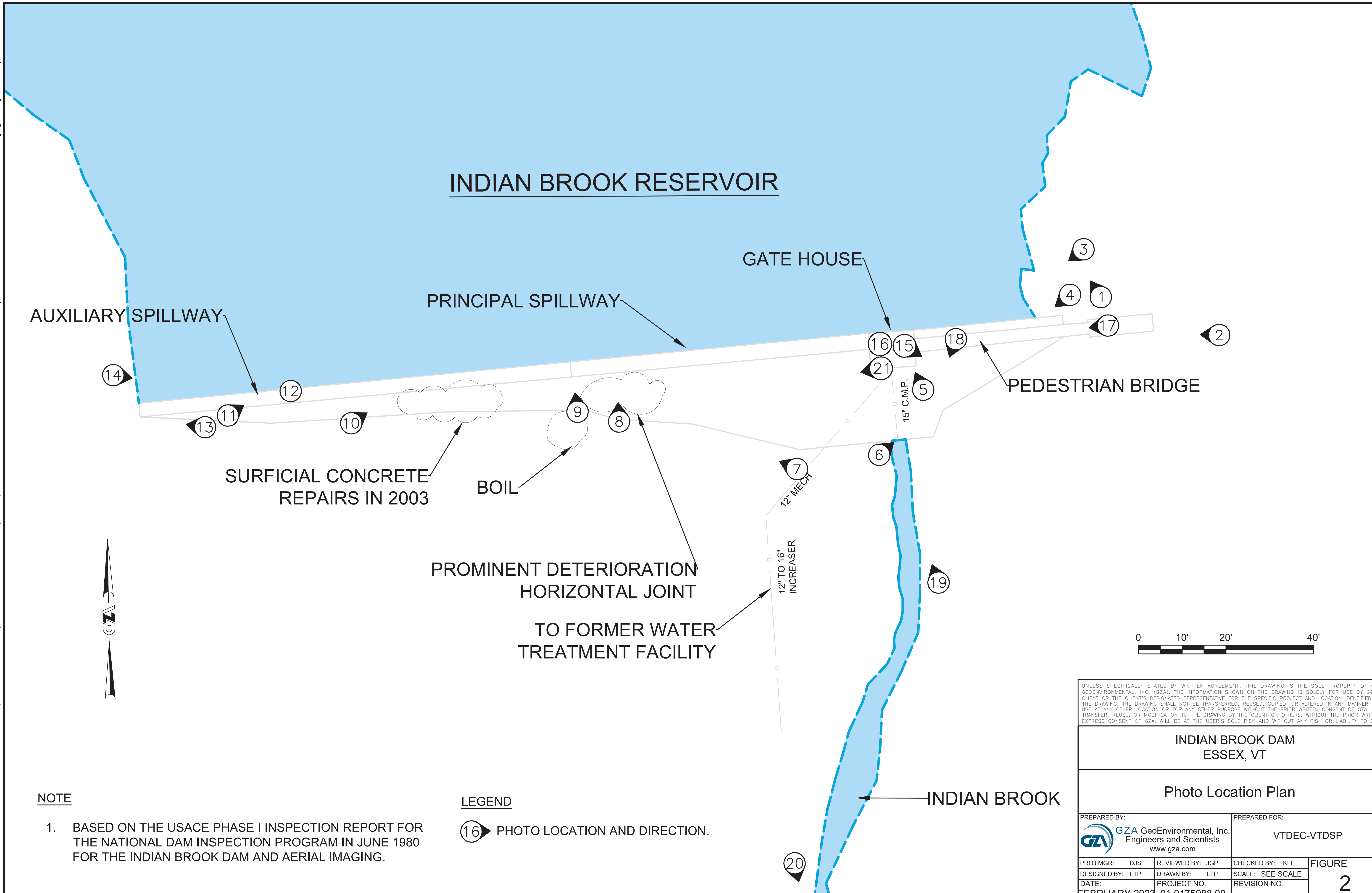
PROJ. MGR.: DJS
DESIGNED BY: LTP
REVIEWED BY: DJS
OPERATOR: LTP
DATE: 12/28/2022

SITE LOCATION PLAN
INDIAN BROOK RESERVOIR DAM

VERMONT DEPARTMENT OF ENVIRONMENTAL CONSERVATION
VERMONT DAM SAFETY PROGRAM
ESSEX, VERMONT

JOB NO.
01.0175988.00

FIGURE NO.
1



NOTE

1. BASED ON THE USACE PHASE I INSPECTION REPORT FOR THE NATIONAL DAM INSPECTION PROGRAM IN JUNE 1980 FOR THE INDIAN BROOK DAM AND AERIAL IMAGING.

LEGEND

(16) PHOTO LOCATION AND DIRECTION.

INDIAN BROOK DAM
ESSEX, VT

Photo Location Plan

PREPARED BY: GZA GeoEnvironmental, Inc. Engineers and Scientists www.gza.com		PREPARED FOR: VTDEC-VTDSP	
PROJ MGR: DJS	REVIEWED BY: JGP	CHECKED BY: KFF	FIGURE 2
DESIGNED BY: LTP	DRAWN BY: LTP	SCALE: SEE SCALE	
DATE: FEBRUARY 2023	PROJECT NO. 01.0175988.00	REVISION NO.	

Appendix A - Limitations

USE OF REPORT

1. GZA GeoEnvironmental, Inc. (GZA) prepared this report on behalf of, and for the exclusive use of The State of Vermont (Client) for the stated purpose(s) and location(s) identified in the Report. Use of this report, in whole or in part, at other locations, or for other purposes, may lead to inappropriate conclusions; and we do not accept any responsibility for the consequences of such use(s). Further, reliance by any party not identified in the agreement, for any use, without our prior written permission, shall be at that party's sole risk, and without any liability to GZA.

STANDARD OF CARE

2. Our findings and conclusions are based on the work conducted as part of the Scope of Services set forth in the Report and/or proposal, and reflect our professional judgment. These findings and conclusions must be considered not as scientific or engineering certainties, but rather as our professional opinions concerning the limited data gathered during the course of our work. Conditions other than described in this report may be found at the subject location(s).

3. Our services were performed using the degree of skill and care ordinarily exercised by qualified professionals performing the same type of services at the same time, under similar conditions, at the same or a similar property. No warranty, expressed or implied, is made.

SUBSURFACE CONDITIONS

4. If presented, the generalized soil profile(s) and description, along with the conclusions and recommendations provided in our Report, are based in part on widely-spaced subsurface explorations by GZA and/or others, with a limited number of soil and/or rock samples and groundwater /piezometers data and are intended only to convey trends in subsurface conditions. The boundaries between strata are approximate and idealized, and were based on our assessment of subsurface conditions. The composition of strata, and the transitions between strata, may be more variable and more complex than indicated. For more specific information on soil conditions at a specific location refer to the exploration logs. The nature and extent of variations between these explorations may not become evident until further exploration or construction. If variations or other latent conditions then appear evident, it will be necessary to reevaluate the conclusions and recommendations of this report.

5. Water level readings have been made in test holes (as described in the Report), monitoring wells and piezometers, at the specified times and under the stated conditions. These data have been reviewed and interpretations have been made in this Report. Fluctuations in the groundwater and piezometer levels, however, occur due to temporal or spatial variations in areal recharge rates, soil heterogeneities, reservoir and tailwater levels, the presence of subsurface utilities, and/or natural or artificially induced perturbations.

GENERAL

6. The observations described in this report were made under the conditions stated therein. The conclusions presented were based solely upon the services described therein, and not on scientific tasks or procedures beyond the scope of described services or the time and budgetary constraints imposed by the Client.

7. In preparing this report, GZA relied on certain information provided by the Client, state and local officials, and other parties referenced therein available to GZA at the time of the evaluation. GZA did not attempt to independently verify the accuracy or completeness of all information reviewed or received during the course of this evaluation.

8. Any GZA hydrologic analysis presented herein is for the rainfall volumes and distributions stated herein. For storm conditions other than those analyzed, the response of the site's spillway, impoundment, and drainage network has not been evaluated.

9. Observations were made of the site and of structures on the site as indicated within the report. Where access to portions of the structure or site, or to structures on the site was unavailable or limited, GZA renders no opinion as to the condition of that portion of the site or structure. In particular, it is noted that water levels in the impoundment and elsewhere and/or flow over the spillway may have limited GZA's ability to make observations of underwater portions of the structure. Excessive vegetation, when present, also inhibits observations.

10. In reviewing this Report, it should be realized that the reported condition of the dam is based on observations of field conditions during the course of this study along with data made available to GZA. It is important to note that the condition of a dam depends on numerous and constantly changing internal and external conditions, and is evolutionary in nature. It would be incorrect to assume that the present condition of the dam will continue to represent the condition of the dam at some point in the future. Only through continued inspection and care can there be any chance that unsafe conditions be detected.

COMPLIANCE WITH CODES AND REGULATIONS

11. We used reasonable care in identifying and interpreting applicable codes and regulations. These codes and regulations are subject to various, and possibly contradictory, interpretations. Compliance with codes and regulations by other parties is beyond our control.

12. This scope of work does not include an assessment of the need for fences, gates, no-trespassing signs, repairs to existing fences and railings and other items which may be needed to minimize trespass and provide greater security for the facility and safety to the public. An evaluation of the project for compliance with OSHA rules and regulations is also excluded.

COST ESTIMATES

13. Unless otherwise stated, our cost estimates are for comparative, or general planning purposes. These estimates may involve approximate quantity evaluations and may not be sufficiently accurate to develop construction bids, or to predict the actual cost of work addressed in this Report. Further, since we have no control over the labor and material costs required to plan and execute the anticipated work, our estimates were made using our experience and readily available information. Actual costs may vary over time and could be significantly more, or less, than stated in the Report.

ADDITIONAL SERVICES

14. It is recommended that GZA be retained to provide services during any future: site observations, explorations, evaluations, design, implementation activities, construction and/or implementation of remedial measures recommended in this Report. This will allow us the opportunity to: i) observe conditions and compliance with our design concepts and opinions; ii) allow for changes in the event that conditions are other than anticipated; iii) provide modifications to our design; and iv) assess the consequences of changes in technologies and/or regulations.

Appendix B – Inspection Photos

Photo 1: View of the upstream left embankment. Note anchors for buoys (red arrow) – reportedly removed from the reservoir prior to cold weather months.

Photo 2: View of the top of the dam from the left abutment. Note warning signage present.

Photo 3: View of the spillway crest, auxiliary crest, and pedestrian bridge from upstream.

Photo 4: Close up view of the spillway crest from the top of the dam. Note the leaf litter and concrete deterioration of the spillway crest (red circle).

Photo 5: Minor missing grout of the construction joint of the gate chamber structure and downstream principal spillway face.

Photo 6: Note the spalling at the construction joints and efflorescence of the spillway face.

Photo 7: Principal spillway downstream, where toe bedrock elevation rises from the tailrace (foreground). Note the spalling present at the lift joints and toe with iron staining and seeping at toe.

Photo 8: Prominent deterioration and joint face spalling at horizontal construction joint of the principal spillway face. Note vegetation growth in construction joint.

Photo 9: Close-up view of the downstream face where the dam transitions from the principal (left) to the auxiliary (right) spillway. Horizontal patchwork from 2003 concrete joint repairs (red arrow).

Photo 10: Auxiliary spillway downstream face with horizontal cracking.

Photo 11: Moss/vegetation growth along the crest and downstream face of the auxiliary spillway. Note the seepage at the toe.

Photo 12: View of the auxiliary spillway crest with exposed rebar.

Photo 13: Right abutment and downstream face of the auxiliary spillway.

Photo 14: View of the auxiliary spillway crest and approach area. Good alignment of spillway crest.

Photo 15: View of a typical gate stem for the low-level outlet gate chamber, at the end of the pedestrian bridge.

Photo 16: View inside the gate chamber. Note warping and bulging of the lowest section of intake screen.

Photo 17: Spalling of concrete at the entrance of the pedestrian bridge.

Photo 18: View of the downstream area from the pedestrian bridge.

Photo 19: View of the downstream area, looking upstream towards the principal spillway.

Photo 20: View of the downstream.

Photo 21: Approximate location of flowing water at toe of auxiliary spillway.

Appendix C – Inspection Checklist

Name of Dam: Indian Brook Reservoir Dam   NID ID: VT00055   Inspection Date: 12/6/2022

DOWNSTREAM AREA AND MISC.
1 of 1

AREA INSPECTED	ITEM NO.	CONDITION	OBSERVATIONS	MONITOR	INVESTIGATE	REPAIR
DOWNSTREAM AREA	1	Abutment Leakage	None.			
	2	Foundation Seepage	Flowing water at the toe of the auxiliary spillway.	X		
	3	Slide, Slough, Scarp	None.			
	4	Toe Drain Manhole	N/A			
	5	Toe Drain Headwall/Cleanout	N/A			
	6	Toe Drains	N/A			
		Flow Measurements:	N/A			
	7	Downstream Hazard Description	Residential housing and park facilities.			
	8	Date of Last Update EAP	2011			
MISCELLANEOUS	9	Reservoir Slopes	No noted reservoir rim slope instabilities.			
	10	Access Roads	Maintained gravel road.			
	11	Security Lock	Pedestrian bridge above the principal spillway is locked including the (11)			
	12	Other Security Devices	None.			
	13	Warning Signs	Boat warning buoys are deployed upstream of the dam; reportedly (13)			
	14					
	15					

ADDITIONAL COMMENTS: REFER TO ITEM NO. IF APPLICABLE

(11) covers for the gate stems at the gate chamber.

(13) removed in cold weather months following recreation season.

Name of Dam: Indian Brook Reservoir Dam NID ID: VT00055 Inspection Date: 12/6/2022

AREA INSPECTED	PRINCIPAL SPILLWAY CHANNEL 1 of 1			CHECK ACTION NEEDED		
	ITEM NO.	CONDITION	OBSERVATIONS	MONITOR	INVESTIGATE	REPAIR
ERODIBLE DOWNSTREAM CHANNEL	16	Slide, Slough, Scarp	None.			
	17	Erosion	None.			
	18	Vegetation Condition	Maintained grass with wooded area.			
	19	Debris	Leaf litter and minor wood debris.	X		
	20	Seepage	Not observed.			
	21					
NON-ERODIBLE SPILLWAY CHANNEL	22	Sidewall	N/A			
	23	Channel Floor	Natural stream channel with boulders.			
	24	Unusual Movement	None.			
	25	Approach Area	None.			
	26	Weir or Control	N/A			
	27	Discharge Area	Iron-staining along the toe of the right end of the principal spillway.	X		
	28	Floor Joints	N/A			
	29	Rip Rap	Some larger rocks.			
INLET AREA	30	Intake Structure	At the end of the pedestrian bridge is the inlet structure for the (30)			X
	31	Wet Well Cover Plates	N/A			
	32	Trashrack	Trash rack warping and bulging of the screen.			X
	33	Stilling Basin	N/A			
	34	Type	Concrete Ogee Weir			

ADDITIONAL COMMENTS: REFER TO ITEM NO. IF APPLICABLE

(30) low-level outlet. Surface spalling along the inlet structure.

Name of Dam: Indian Brook Reservoir Dam NID ID: VT00055 Inspection Date: 12/6/2022

AREA INSPECTED	ITEM NO.	CONDITION	OBSERVATIONS	MONITOR	INVESTIGATE	REPAIR
		OUTLET WORKS 1 of 1		CHECK ACTION NEEDED		
OUTLET WORKS	35	Primary Closure	Three slide gates at different invert elevations; approximately 48-feet (35)			
	36	Secondary Closure	N/A			
	37	Control Mechanisms	Slide gate stems extend to the deck level of the gate chamber; (37)			X
	38	Outlet Pipe	Outlet pipe exits from downstream face; not observed/not located.			
	39	Wet Well Structure Interior	N/A			
	40	Erosion Along Dam Toe	N/A			
	41	Seepage	None.			
	42	Unusual Movement	None.			
	43					
	44					
	45					
	46					

ADDITIONAL COMMENTS: REFER TO ITEM NO. IF APPLICABLE

(35) from the left side of the principal spillway.

(37) covered in locking deck boxes. Reportedly inoperable/not routinely exercised.

Name of Dam: Indian Brook Reservoir Dam NID ID: VT00055 Inspection Date: 12/6/2022

PRINCIPAL SPILLWAY

AREA INSPECTED	ITEM NO.	CONDITION	OBSERVATIONS	MONITOR	INVESTIGATE	REPAIR
UPSTREAM FACE	47	Surface Conditions	High reservoir levels prevented observation.			
	48	Condition of Slope	High reservoir levels prevented observation.			
	49	Unusual Movement	High reservoir levels prevented observation.			
	50	Abutment -Dam Contacts	High reservoir levels prevented observation.			
	51	Leakage	High reservoir levels prevented observation.			
	52	Other	Good alignment along length of principal spillway.			
DOWNSTREAM FACE	53	Surface Conditions	Fair condition; Erosion along the horizontal construction joints (53)			X
	54	Condition of Slope	Efflorescence and vegetation growing out of the large spalling sections.			X
	55	Unusual Movement	None.			
	56	Toe	Spalling along the bedrock contact with the toe.			X
	57	Seepage	Observation hinder due to raining conditions and partial overtopping.			
	58	Other	Partial flow over the principal spillway hinder the observation of the (58)	X		
	59					
	60					
CREST	61	Surface Conditions	Minor woody debris and dead leaves.			X
	62	Horizontal Alignment	Good alignment.			
	63	Vertical Alignment	Possible minor misalignment since there is only partial flow over the crest.		X	
	64	Condition of Slope	Concrete deterioration with erosion along the joints.			X
	65	Unusual Movement	None.			
	66					
	67					

ADDITIONAL COMMENTS: REFER TO ITEM NO. IF APPLICABLE

(53) with large spalling sections.

(58) entire spillway face. Exposed construction filler in large spalling section on the right end.

Name of Dam: Indian Brook Reservoir Dam   NID ID: VT00055   Inspection Date: 12/6/2022

AREA INSPECTED	AUXILIARY SPILLWAY CHANNEL 1 of 1			CHECK ACTION NEEDED		
	ITEM NO.	CONDITION	OBSERVATIONS	MONITOR	INVESTIGATE	REPAIR
ERODIBLE DOWNSTREAM CHANNEL	68	Slide, Slough, Scarp	N/A			
	69	Erosion	N/A			
	70	Vegetation Condition	Efflorescence on top of bedrock with recently cut grass.			
	71	Debris	Minor leaf litter and woody debris.			
	72	Seepage	Flowing water along the auxiliary spillway toe of the downstream (72)	X		
	73	Channel Floor	Bedrock and grass lined.			
NON-ERODIBLE SPILLWAY CHANNEL	74	Sidewalls	N/A			
	75	Channel Floor	N/A			
	76	Unusual Movement	None.			
	77	Approach Area	None.			
	78	Weir or Control	Concrete deterioration.			X
	79	Discharge Area	N/A			
	80	Floor Joints	N/A			
	81	Rip Rap	N/A			
INLET AREA	82	Intake Structure	N/A			
	83	Wet Well Cover Plates	N/A			
	84	Trashrack	N/A			
	85	Stilling Basin	N/A			
	86	Type	Concrete Ogee Weir			

ADDITIONAL COMMENTS: REFER TO ITEM NO. IF APPLICABLE

(72) channel. Seeping along the toe.

Name of Dam: Indian Brook Reservoir Dam NID ID: VT00055 Inspection Date: 12/6/2022

AREA INSPECTED	ITEM NO.	CONDITION	OBSERVATIONS	MONITOR	INVESTIGATE	REPAIR
			AUXILIARY SPILLWAY 1 of 1	CHECK ACTION NEEDED		
UPSTREAM FACE	87	Surface Conditions	High reservoir levels prevented observation.			
	88	Condition of Slope	High reservoir levels prevented observation.			
	89	Unusual Movement	High reservoir levels prevented observation.			
	90	Abutment -Dam Contacts	High reservoir levels prevented observation.			
	91	Leakage	High reservoir levels prevented observation.			
	92	Other				
DOWNSTREAM FACE	93	Surface Conditions	Poor condition; erosion along the horizontal construction joints (93)			X
	94	Condition of Slope	Efflorescence and vegetation growing out of the large spalling sections.			X
	95	Unusual Movement	None.			
	96	Abutment -Dam Contacts	Good alignment.			
	97	Leakage	Observation hindered due to raining conditions. Previously observed (97)	X		
	98	Other				
	99					
	100					
CREST	101	Surface Conditions	Minor woody debris and dead leaves.			X
	102	Horizontal Alignment	Good alignment.			
	103	Vertical Alignment	Good alignment.			
	104	Condition of Slope	Concrete deterioration with erosion along the joints. Exposed rebar.			X
	105	Unusual Movement	None.			
	106					
	107					

ADDITIONAL COMMENTS: REFER TO ITEM NO. IF APPLICABLE

(93) with large horizontal cracking.

(97) seeping through the face.

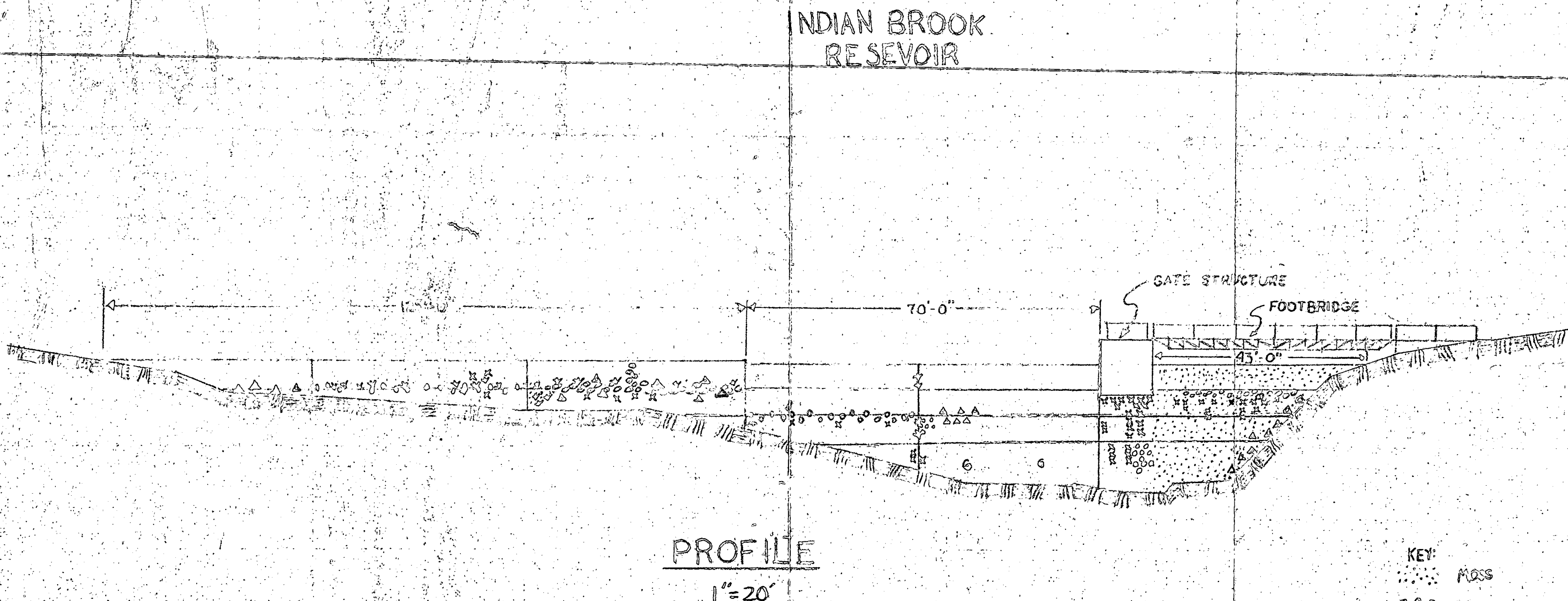
Name of Dam: Indian Brook Reservoir Dam NID ID: VT00055 Inspection Date: 12/6/2022

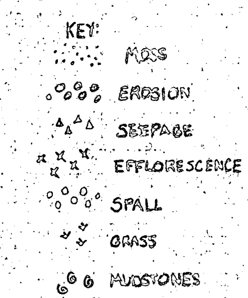
AREA INSPECTED	ITEM NO.	CONDITION	OBSERVATIONS	MONITOR	INVESTIGATE	REPAIR
		PEDESTRIAN BRIDGE 1 of 1		CHECK ACTION NEEDED		
BRIDGE PIERS AND DECK	108	Surface Conditions	Good condition. Coating is generally in good condition, with (108)			
	109	Condition of Concrete Joints	Good.			
	110	Unusual Movement	None.			
	111	Contacts	N/A			
	112	Condition of Support Beams	Good condition.			
	113	Condition of Deck/Railings	Good.			
	114	Vertical Alignment	Good alignment.			
	115	Horizontal Alignment	Good alignment.			
APPROACH SLABS	116	Surface Conditions	Minor surface spalling/delamination at left abutment landing.			
	117	Condition of Joints	Good condition.			
	118	Unusual Movement	None.			
	119	Contacts	Good alignment.			
	120	Drains or Leakage	N/A			
	121	Vertical Alignment	Good alignment.			
	122	Horizontal Alignment	Good alignment.			
	123					

ADDITIONAL COMMENTS: REFER TO ITEM NO. IF APPLICABLE

(108) some discoloration/deterioration and coating lost; predominantly along the horizontal surface below the walkway level.

Appendix D – Historic Site Plan





INDIAN BROOK DAM
ESSEX
INSPECTION: RBF, JMU
DATE: 5-22-2001
DRAWING: 6-20-2001
PLAN ADOPTED FROM
SITE PLAN

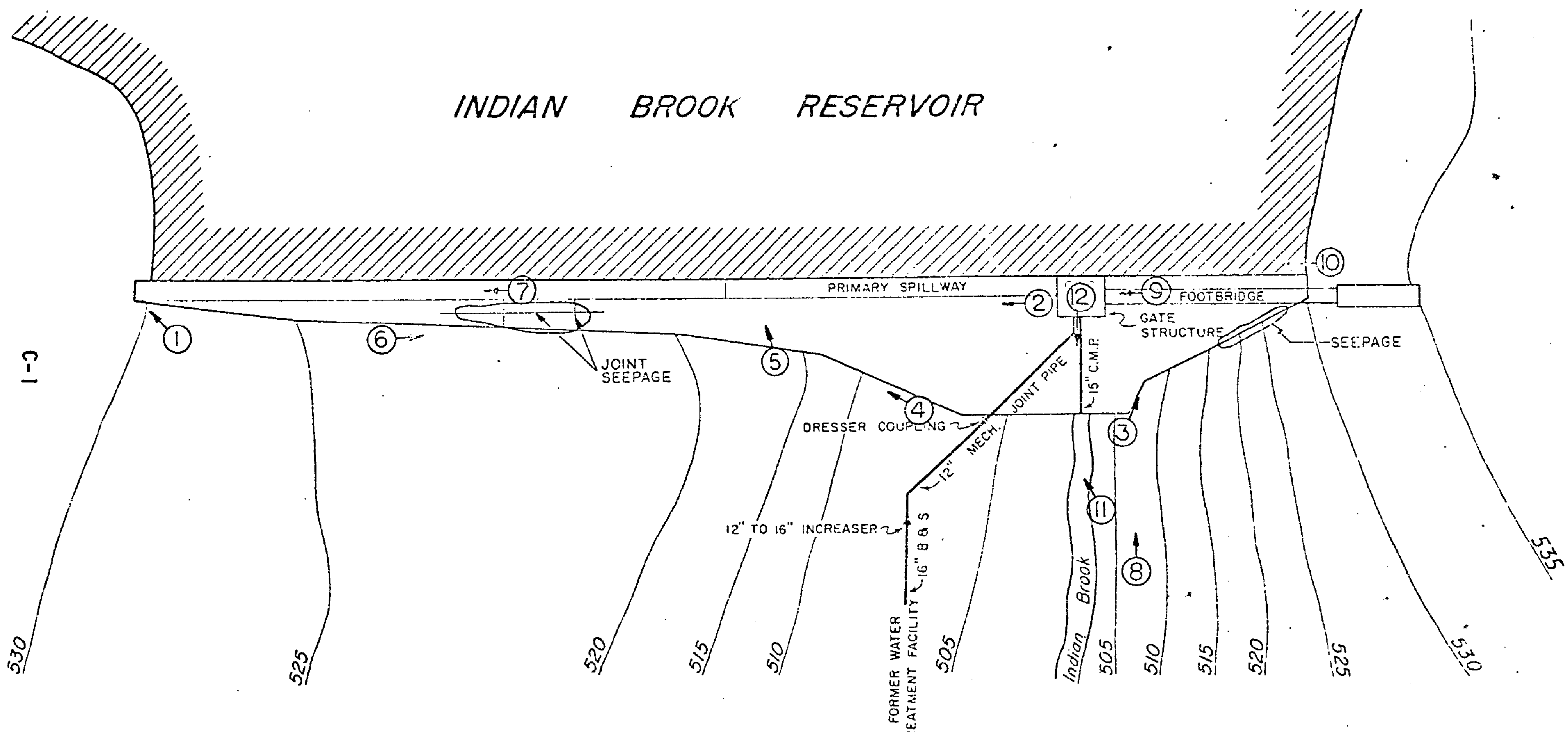


PHOTO LOCATION PLAN
INDIAN BROOK DAM

20 0 20 FEET 40

APPENDIX C – GZA HYDROLOGIC AND HYDRAULIC ANALYSIS MEMORANDUM

Known for excellence.
Built on trust.

GEOTECHNICAL
ENVIRONMENTAL
ECOLOGICAL
WATER
CONSTRUCTION MANAGEMENT

December 15, 2023
File No. 01.0175988.00

Vermont Department of Environmental Conservation – Water Investment Division
Vermont Dam Safety Program
Davis Building – 3rd Floor
One National Life Drive
Montpelier, Vermont 05620-3510

Attn: Mr. Benjamin Green, P.E. | Section Chief - Dam Safety Engineer
Mr. Andrew Sampsell, P.E. | Dam Safety Engineer

Re: Dam Engineering Services
Hydrologic and Hydraulic Analysis
Indian Brook Reservoir Dam (VT00055)

Dear Mr. Green,

In accordance with our contract signed December 1, 2022, GZA GeoEnvironmental, Inc. (GZA) is providing this memorandum to the Vermont Department of Conservation (DEC)- Agency of Natural Resources (ANR) (Client) to summarize our hydrologic and hydraulic (H&H) analysis of Indian Brook Reservoir Dam. This memorandum is subject to the attached Limitations in **Appendix A**.

The purpose of this work was to evaluate failure modes (static, hydrologic, and seismic) to inform the portfolio risk analysis for Indian Brook Reservoir Dam. This analysis estimated the spillway design flood (SDF), i.e., the Probable Maximum Flood (PMF), inflow to estimate the spillway adequacy of Indian Brook Reservoir Dam. Dam breach modeling for each of the failure modes was conducted and associated inundation mapping was used to estimate the consequences of dam failure in terms of Population at Risk (PAR).

PROJECT BACKGROUND

Indian Brook Reservoir Dam, National Inventory of Dams No. VT00055, is located in Essex, Vermont. The dam's location is presented in **Figure 1**.

Indian Brook Reservoir Dam is a concrete gravity dam, approximately 238 feet (ft) long and 31 feet high, with a concrete ogee-style spillway. The spillway is a 43-foot-long section and a 70-foot-long section of ogee-style spillway, separated by a 10-foot-long gate chamber, cast integrally with the spillway. The spillway structure has a vertical upstream face, approximate 4-foot-wide crest, and a 3H:4V sloping downstream face and discharges into Indian Brook.

The dam impounds about 1,084 acre-feet of water at normal pool (529.6 ft) with a maximum storage capacity of 1,157 acre-feet at top of dam (530.1 ft). The contributory drainage area to the dam is approximately 1.23 square miles. The primary use of the dam and its impoundment is recreation. Indian Brook Reservoir Dam is classified by VT DEC as a **<u>High</u>** hazard structure.

DATUM

The elevation datum referenced throughout this document is the North American Vertical Datum of 1988 (NAVD88).

HYDROLOGIC ANALYSIS

HYDROLOGIC ANALYSIS METHODOLOGY

The hydrologic analysis methodology consisted of the following steps:

1. Delineate the watershed based on available GIS and topographic information[1].
2. USGS StreamStats Review: The Indian Brook Reservoir Dam watershed is ungaged (i.e., there is no long-term, reliable streamflow data used to calibrate and verify H&H analyses). Therefore, GZA reviewed the United States Geologic Survey's (USGS) StreamStats application to establish peak discharge estimates at Indian Brook Reservoir Dam.
3. Application of the Unit Hydrograph Method: Rainfall / runoff processes were modeled by GZA utilizing the Snyder Unit Hydrograph (UH) processes within the US Army Corps of Engineers' (USACE) HEC-HMS (version 4.10) computer program, in combination with estimated initial and constant losses. The Snyder method was selected because it is appropriate for the size of watershed, and it was originally developed for analysis of ungaged watersheds in the Appalachian Highlands. Snyder Unit Hydrograph uses the Snyder's standard lag (t_p) along with the Peak Flow Coefficient (C_p) as rainfall to runoff translation parameters. The method uses Basin coefficients to describe regional unit hydrograph parameters that can be determined using observed data and the following relationships:

$$t_p = C_t * (L * L_c)^{0.3}$$

$$C_p = \frac{(Q_p * t_p)}{(640 * A)}$$

$$C_t = \frac{t_p}{(L * L_c)^{0.3}}$$

Where:

t_p = Time to peak measured from the onset of precipitation excess (hours) (hr)

L = Length of the main watercourse (miles) (mi)

L_C= Length along the main watercourse measured upstream to the point opposite the centroid of the Basin (miles)

Q_P = Peak discharge rate (cubic feet per second) (cfs)

A = Drainage area of the watershed (square miles) (mi^2)

C_t and C_p are empirical constants that vary based on catchment storage, basin slopes, stream patterns, basin shape, and other watershed properties accounting for flood wave storage conditions. Typical values for C_t and C_p range

[1] Light Detection and Ranging (Lidar) terrain and elevation Data, USGS 3DEP, December 2021, data acquired December 2022. TNM Download v2 (nationalmap.gov)

between 1.8 to 2.2 and 0.4 to 0.8, respectively (Bedient and Huber 1992)[2]. L and L_c values were estimated using the DEM in ArcGIS and C_p and C_t values were adjusted within the typical ranges in order to reproduce peak flows that closely resembled the observed values.

4. Estimate constant and initial soil losses: Constant loss is estimated using published [USDA 1955[3]] infiltration rates of the hydraulic soil groups within the watershed. The initial loss is estimated from the watershed's antecedent conditions prior to each storm. Initial losses for the SDF are assumed to be zero.

5. Compare the model to published data: GZA checked the model results by simulating the 100-year flood and comparing it to published flood frequency data. This methodology provides increased confidence in the model results and reduces the potential for grossly overestimating the inflow design flood.

   The simulated 100-year flood hydrograph was compared to the USGS StreamStats flood peak, and key hydrologic input parameters were adjusted to approximately match published data. This process was considered successful when an acceptable comparison of simulated versus published flood flow data was achieved.

   a. Obtain published flood information inflow to Indian Brook Reservoir Dam.

   b. Import these data into HEC-HMS and iteratively adjust the Snyder UH parameter (i.e., standard lag t_p) and the precipitation loss parameters (i.e., initial and constant loss,) for the dam watershed until the simulated runoff response reflects the observed response to the published flood data. Each parameter is adjusted, within the general bounds of their expected values based on published guidance, initial calculated values, and engineering judgment.

6. Apply the final Snyder United Hydrograph parameter (t_p) and loss rates in the HEC-HMS model based on the process above.

7. Per standard practice, estimate Probable Maximum Precipitation (PMP) distribution using the Hydrometeorological Report (HMR)-52 storm tool in HEC-HMS. The PMP was developed based on National Oceanic and Atmospheric Administration (NOAA) HMR Nos. 51 and 52 procedures[4]. Note that HMR-51 and HMR-52 reports represent the latest PMP guidance from NOAA, while NOAA Atlas 14 applies to lesser magnitude storms (i.e., 1,000-year return period and below).

8. Use the developed parameters and the Probable Maximum Precipitation, i.e., PMP hyetograph, to simulate the watershed response.

A detailed description of the hydrological modeling and PMF simulation are presented below.

MODEL INPUTS

Watershed Delineation

GZA estimated the total drainage area to the reservoir (presented in **Figure 1**) using topographic obtained from terrain data from the USGS 3DEP dataset. The digital elevation model (DEM) was then used to delineate the watershed in HEC-HMS. The key data for the watershed are presented in **Table 1**.

---

[2] Bedient, P.B., and Huber, W.C. (1992). Hydrology and floodplain analysis. Addison-Wesley, New York, NY.

[3] USDA 1955 Yearbook of Agriculture. Chapter 2- Hydrologic Soil Group. United States Department of Agriculture, 1955.

[4] NOAA HYDROMETEOROLOGICAL REPORT NOs. 51 and 52, Probable Maximum Precipitation Estimates -United States East of the 105th Meridian, U.S. Department of Commerce National Oceanic and Atmospheric Administration, U.S. Department of The Army Corps of Engineers, 1978 and 1982, respectively.

Table 1: Watershed Area

Watershed	Watershed Area (mi^2)
Indian Brook Reservoir Dam	1.23

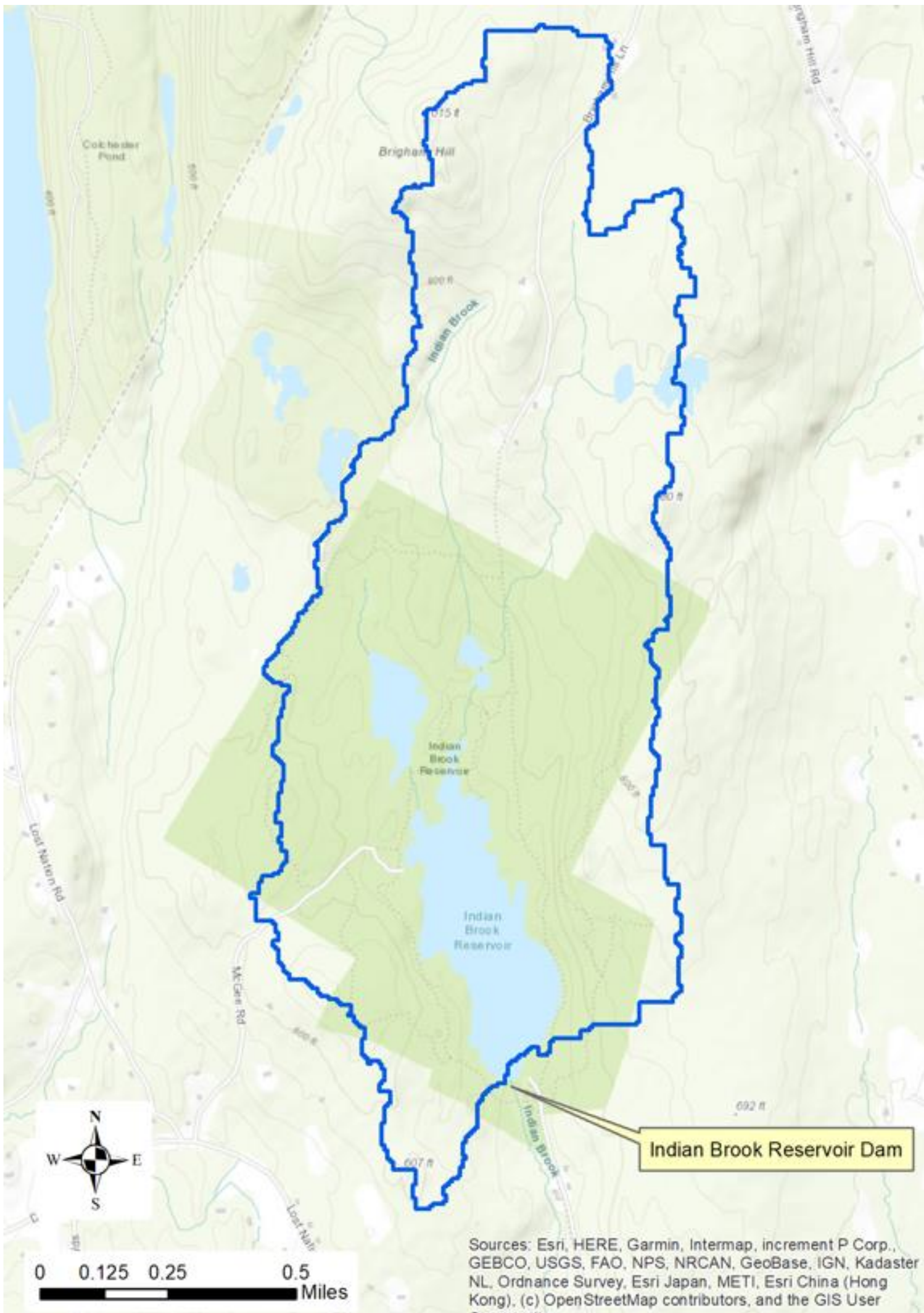


Figure 1: Indian Brook Reservoir Dam Drainage Area

USGS StreamStats

GZA used the USGS StreamStats application for peak flood flow estimates at the dam. **Table 2** presents the peak flood flow values for Indian Brook at Indian Brook Reservoir Dam.

Table 2: USGS StreamStats Peak Flood Flows- Indian Brook at Indian Brook Reservoir Dam

Average Recurrence Interval (years)	10	50	100	500
Peak Discharge (cfs)	63.2	102	122	175

Precipitation Data

Point precipitation frequency estimates from NOAA Atlas 14, Volume 10, Version 3 were used for rainfall data. The 24-hr 100-yr recurrence interval precipitation estimates (**Table 3**) is used as a model input to estimate the 100-yr flood flows. The precipitation is temporally distributed in half-hour increments using the NOAA Atlas 14 distribution: 50% of occurrence, cumulative percentages of total precipitation for all cases.

Table 3: Point Precipitation Estimates with 90% Confidence Intervals (inches)

Duration	Average Recurrence Interval (years)
	100
24-hr	5.21 (3.84-7.09)

Probable Maximum Precipitation

GZA derived the 72-hour PMP for the Indian Brook Reservoir Dam watershed model using NOAA Hydrometeorological Report No. 52 procedures in the HEC-HMS program (**Table 4**).

Table 4: HMR 51/52 Derived PMP Depths for PMF Model Elements

Basin Area (mi^2)	Storm Area (mi^2)	X Coordinate	Y Coordinate	Orientation (Degrees)	Peak Intensity	1 to 6 Ratio	72-hour Precipitation Depth (inches)
1.23	10	1484452.3	745455.8	205	Hours 42 to 48	0.71	30.55

The PMF inflows are developed by applying the PMP to the drainage basin and estimating the peak flows.

HEC-HMS MODEL CHECK

As discussed above USGS streamflow estimates were used to inform GZA's model development.

In GZA's opinion, the analysis parameters, as summarized herein, resulted in a reasonable agreement of the 100-year flood scenario within the Indian Brook Reservoir Dam basin with published data, as shown in **Table 5**. Graphical results are provided in **Appendix B**. Using the input parameters shown, GZA simulated the runoff process for the 100-year rainfall event. GZA compared the simulated results of the 100-year peak flood flow (130 cfs) to the published USGS peak flood flow (122 cfs), and the results are very similar.

Table 5: Summary of 100-year flood check

Standard Lag (hr)	Peaking Coefficient	Initial Loss (inches)	Constant Loss Rate (inches/hr)	Percent Impervious	USGS 100-yr Peak Flood Flow (cfs)	Simulated 100-yr Peak Flood Flow (cfs)
2.503	0.4	0	0.28	1.79	122	130

GZA applied a Snyder Unit Hydrograph lag t_p of 2.503 hours, which is a function of basin characteristics and length of the watershed. The initial loss rate of zero was taken to be representative of initial watershed conditions for a severe design flood. A constant loss rate of 0.02 inch per hour was calculated as a function of the hydrologic soil groups in the watershed. This value was adjusted to 0.28 inch per hour to closely match the USGS peak flow Based on GZA's work within the region, this loss rate is judged to be a plausible value, in our opinion.

Hydrology prepared for this study was performed specifically for the purpose of screening and relative risk assessment to provide for consistent comparison (e.g., the selection of hydrologic parameters) between each of the analyzed structures. Results presented herein should be used for this purpose only. As part of the hydrologic assessment, the coefficient, Cp (i.e., Unit hydrograph peaking coefficient), was increased from the initially estimated value of 0.4 to 0.6, which results in a higher peak inflow that is more consistent with previously reported peak flows at several of the dams. Future design considerations should include a more detailed assessment of the PMF.

RESULTS

The HEC-HMS analysis resulted in an estimated peak inflow from the Indian Brook Reservoir Dam watershed. The PMF hydrograph is presented in **Appendix B**.

Indian Brook Reservoir Dam was modeled in HEC-HMS with a 70-foot-long primary spillway at elevation 529.6 ft and a weir coefficient of 3.95. The auxiliary spillway was modeled as 168-feet-long at elevation 530.1 ft and a weir coefficient of 3.95. Results from the SDF routing are summarized in **Table 6**. These results show that there is overtopping of non-overtopping structures indicating that there is insufficient spillway capacity to safely pass the SDF.

Table 6: SDF Routing Results

Flood	Rainfall Depth (inches)	Peak inflow (cfs)	Peak outflow (cfs)	Peak Stage (ft, NAVD88)	Top of Dam Elevation (ft, NAVD88)	Embankment Overtopping (ft)
PMF (SDF)	30.55	3,100	3,040	532.1	531.1	1.0

DAM BREACH MODELING METHODOLOGY AND RESULTS

The objective of the Indian Brook Reservoir Dam breach modeling was to estimate the downstream inundation areas and the corresponding time to flooding, thus identifying critical downstream areas that would be affected by a dam failure (i.e., sudden, uncontrolled release of water). GZA simulated the Fair weather and SDF failures of Indian Brook Reservoir Dam using two-dimensional (2-D) hydraulic computer modeling with the U.S. Army Corps of Engineers software program HEC-RAS, version 6.3.1. The hydraulic model was developed using the following steps:

1. Develop model terrain using topographic data;
2. Develop model geometry by defining 2-D flow area grid and adding in structures and storage areas;

3. Establish roughness zones using Manning's n values estimated from land cover data;
4. Define model boundary conditions;
5. Estimate dam breach parameters using published guidance[5].

The HEC-RAS input parameters are summarized in **Table 7**.

Table 7: HEC-RAS Model Input Parameters

Model Element	Input Parameters
Model Extents and Layout	The model extends from 3,200 feet upstream of Indian Brook Reservoir Dam to 10.8 miles downstream of Indian Brook Reservoir Dam, until Mallett's Bay. The upstream impoundment is represented with a Storage Area. The downstream area is represented with a 2D Flow Area with a mesh resolution of 100 feet. The Storage Area and 2D Flow Area are connected with an SA/2D Connector representing Indian Brook Reservoir Dam.
Terrain Data	Terrain data was downloaded from the USGS 3DEP dataset. Data were in 1/3 arc second resolution, with data collected in 2018 and published on 4-16-2019. Breaklines were added to align grid cells with topographic high points.
Indian Brook Reservoir Storage	Elevation-storage curve was estimated using normal and maximum pool volumes as well as the terrain data.
Indian Brook Reservoir Dam	Spillway is modeled as a 113-foot-long weir at elevation 529.6 ft with a weir coefficient of 3.95. Auxiliary spillway is modeled as a 125-foot-long weir at elevation 530.1 ft with a weir coefficient of 3.95.
Downstream Road Crossings	The model includes 15 roads that cross the downstream flow path. In order from upstream to downstream, the road crossings are: Lost Nation Road, SR 289, Upper Main Street, Hubbells Falls Road, Brickyard Road, Main Street, Drury Drive, Old Colchester Road, SR 2A, Pinecrest Drive, Susie Wilson Road, Susie Wilson Bypass, Mill Pond Road, US RT 2, and Creek Farm Road. Road crossings were modeled using a break line along the top of road. The following terrain modifications were included because the terrain did not reflect a bridge opening: U.S. Route 2: 20-ft-wide rectangular notch Hubbells Falls Drive: 20-ft-wide rectangular notch Lincoln Street: 20-ft-wide rectangular notch Pinecrest Drive: 15-ft-wide rectangular notch Susie Wilson Bypass: 12-ft-wide rectangular notch
Downstream Dams	No downstream dams are in the model extents.

[5] FERC, Engineering Guidelines for the Evaluation of Hydropower Projects, Chapter 2, August 2015.

Model Element	**Input Parameters**
Manning's n (i.e., roughness)	National Land Cover Database (NLCD) land cover was used to establish roughness zones within the model with the following estimated Manning's n values: Barren Land = 0.03 Developed Open Space = 0.035 Open Water = 0.035 Grassland – Herbaceous = 0.04 Emergent Herbaceous Wetland = 0.045 Pasture – Hay = 0.045 Cultivated Crops = 0.05 Scrub-Shrub = 0.05 Woody Wetlands = 0.07 Developed – Low Intensity = 0.08 Deciduous Forest = 0.1 Developed – Medium Intensity = 0.12 Mixed Forest = 0.12 Developed – High Intensity = 0.15 Evergreen Forest = 0.15 Manning's n values were assigned based on "Manning's n Values for Various Land Covers To Use for Dam Breach Analyses by NRCS in Kansas" and the HEC-RAS Mapper User's Manual.
Downstream Boundary Condition	Normal depth with slope of 0.0002.
Upstream Inflows	Fair Weather: No streamflow was added, assuming the channel topography included normal baseflow. SDF: Inflow hydrograph from HEC-HMS. Indian Brook Reservoir peak water surface is 532.0 ft.
Lateral Inflows	No lateral inflows were added to the model, as the flood travels on the main stem of Indian Brook from the Indian Brook Reservoir Dam until Mallett's Bay.
Dam Breach Parameters	Construction Type: Concrete Gravity Failure Mode: Overtopping Side Slopes: 0H:1V (vertical) Bottom Elevation: 500.1 ft Bottom Opening Width: 119 ft (0.5 W) Failure Development Time: 0.2 hour Fair weather Trigger Time: start of simulation at time 01:00 SDF Trigger Time: Time of peak water surface elevation The selected breach dimension parameters are uniform between each failure case. The Fair weather failure is assumed to be equivalent to a seismic failure.

The peak outflow from the Fair Weather dam failure was estimated as 31,430 cfs. The peak outflow from the SDF dam failure was estimated as 37,740 cfs. See **Appendix C** for inundation mapping.

POPULATION AT RISK METHODOLOGY AND RESULTS

GZA used the United States Army Corps of Engineers (USACE) Life Loss Estimation software (LifeSim) which is a spatially distributed dynamic simulation modeling system capable of estimating PAR. Simulations are performed using hydraulic data (i.e., depth grids and velocity grids) directly from HEC-RAS. In addition to providing the time series of depth and velocity (i.e., grid data), a hydrograph representing the hydraulic event is also provided for performing life loss/PAR calculations. The representative hydrograph is critical in developing the warning issuance timing relative to a specific condition (e.g., dam breach). The hazard occurrence for each simulation and representative hydrograph is set to be at the time of the initiation of the breach formation.

Structure data in the study area below the dam is then imported to the LifeSim model from that National Structure Inventory (NSI) using the built in import features. The NSI is a database of structure locations and attributes and contains information on structure occupancy types, population, structure values, foundation height, build type, and other information. The NSI was developed using information from the HAZUS database and other sources including the Longitudinal Employer-Household Dynamics database. Simulations were run using the default occupancy type and building stability criteria.

Emergency planning data was assigned to a single emergency planning zone representing the full extent of the study area (i.e., from the Indian Brook Reservoir Dam to Mallett's Bay). The area included in the PAR analysis (i.e., full HEC-RAS model extent) extends beyond the limits of inundation mapping and includes structures where the incremental rise is less than 2 feet. Preparedness for the Warning and Protective Action Data (i.e., Warning issuance delay, first alert, protective action initiation, and issuance to initiation) were all set to the default values of "unknown". It should be noted that these simulation settings do not impact the total number of structures inundated or PAR and only impact parameters associated with emergency actions (e.g., PAR warned, PAR mobilized, loss of life, etc.).

No traffic analysis was performed during these simulations.

Agricultural data was imported directly from the U.S. Department of Agriculture (USDA) National Agricultural Statistics Service (NASS) Cropland Data Layer (CDL) database for the most recent year available (i.e., 2016). Default variable monthly costs and duration percent-damage relationships were used with an assumed event date of August 1st.

Simulations were setup assuming hazard occurrence happening at 12 different times throughout the day (e.g., 12 am, 2 am, 4 am, etc.) with a total number of 100 iterations which take into consideration the confidence limits of the Warning and Protective Action Data but do not impact PAR estimation. LifeSim output in the form of "Maximum PAR" (as well as population distribution under and over the age of 65) as well as the total number of inundated structures and inundation statistics (e.g., minimum depth, maximum depth, etc.) for the Fair weather and SDF failure simulations are provided in **Table 8** and **Table 9**. **Table 9** includes PAR estimates with and without failure to demonstrate the incremental risk from dam failure. Economic losses associated with each simulation are provided in **Tables 10** and **11**. Note that a time of day of 0 hours corresponds to midnight (00:00).

Table 8: Population at Risk Estimate- Indian Brook Reservoir Dam Fair Weather Failure

Time of Day, hr	Structures Inundated	Average Structure Depth, ft	Minimum Structure Depth, ft	Maximum Structure Depth, ft	Total Pop Under 65	Total Pop Over 65	PAR Under 65	PAR Over 65	Total PAR
10	31	3.0	0.0	12.9	9,261	1,626	637	75	712

Table 9: Population at Risk Estimate- Indian Brook Reservoir Dam Spillway Design Flood Failure

Scenario	Time of Day, hr	Structures Inundated	Average Structure Depth, ft	Minimum Structure Depth, ft	Maximum Structure Depth, ft	Total Pop Under 65	Total Pop Over 65	PAR Under 65	PAR Over 65	Total PAR
No Failure	10	29	2.8	0.3	13.3	9,261	1,626	635	73	708
Failure	10	43	4.6	0.1	18.3	9,261	1,626	673	87	760
Incremental Increase	0	14	1.8	-0.2	4.9	0	0	38	14	52

Table 10: Economic Impacts Estimate – Indian Brook Reservoir Dam Fair Weather Failure

Structure, Content, and Vehicular Damage	Agricultural Damage	Total Damage
$8,180,000	$5,000	$8,185,000

Table 11: Economic Impacts Estimate- Indian Brook Reservoir Dam Spillway Design Flood Failure

Scenario	Structure, Content, and Vehicular Damage	Agricultural Damage	Total Damage
No Failure	$7,800,000	$7,000	$7,807,000
Failure	$22,840,000	$10,000	$22,850,000
Incremental Increase	$15,040,000	$3,000	$15,043,000

CONCLUSION

GZA performed updated PMF, dam failure analyses, and prepared updated inundation mapping for Indian Brook Reservoir Dam. The purpose of our analyses was to establish the PMF and dam failure analysis specifically for the purpose of screening and relative risk assessment to provide for consistent comparison (e.g., the selection of hydrologic parameters) between each of the analyzed structures and to provide updated dam failure analysis and inundation mapping to assess consequences of dam failure. GZA's updated analysis uses the latest version of HEC-RAS employing a detailed two-dimensional modeling approach. Updated mapping is shown in **Appendix B**. The results of this analysis indicated that overtopping of non-overtopping structures is anticipated, indicating that there is insufficient spillway capacity to safely pass the SDF. Additionally, the results of the dam failure simulations were used to perform PAR estimation using USACE software, LifeSim. PAR results are provided above in **Tables 8 through 11**.

GZA appreciates the opportunity to provide dam engineering services to Vermont DCE ANR. Please contact Derek Schipper at derek.schipper@gza.com if you have any questions or concerns.

Sincerely,
GZA GEOENVIRONMENTAL, INC.

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Water Resources Engineer

Derek Schipper, P.E.[VT, CT, MA, NH]
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Principal

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Consultant / Reviewer

Appendices

Appendix A: Engineering Report Limitations

Appendix B: HEC-HMS Results

Appendix C: Inundation Maps

APPENDIX A - LIMITATIONS

USE OF REPORT

1. GZA GeoEnvironmental, Inc. (GZA) prepared this report on behalf of, and for the exclusive use of the Vermont Agency of Natural Resources - Department of Environmental Conservation (Client) for the stated purpose(s) and location(s) identified in the Report. Use of this report, in whole or in part, at other locations, or for other purposes, may lead to inappropriate conclusions; and we do not accept any responsibility for the consequences of such use(s). Further, reliance by any party not identified in the agreement, for any use, without our prior written permission, shall be at that party's sole risk, and without any liability to GZA.

STANDARD OF CARE

2. Our findings and conclusions are based on the work conducted as part of the Scope of Services set forth in the Report and/or proposal, and reflect our professional judgment. These findings and conclusions must be considered not as scientific or engineering certainties, but rather as our professional opinions concerning the limited data gathered during the course of our work. Conditions other than described in this report may be found at the subject location(s).

3. GZA's flood evaluation was performed in accordance with generally accepted practices of qualified professionals performing the same type of services at the same time, under similar conditions, at the same or a similar property. No warranty, expressed or implied, is made. The findings of the risk characterization are dependent on numerous assumptions and uncertainties inherent in the risk assessment process. The findings of the flood evaluation are not an absolute characterization of actual risks, but rather serve to highlight potential sources of risk at the site(s).

4. Unless specifically stated otherwise, the flood evaluations performed by GZA and associated results and conclusions are based upon evaluation of historic data, trends, references, and guidance with respect to the current climate and sea level conditions. Future climate change may result in alterations to inputs which influence flooding at the site (*e.g.*, rainfall totals, storm intensities, mean sea level, *etc*.). Such changes may have implications on the estimated flood elevations, wave heights, flood frequencies and/or other parameters contained in this report.

GENERAL

5. The observations described in this report were made under the conditions stated therein. The conclusions presented were based solely upon the services described therein, and not on scientific tasks or procedures beyond the scope of described services or the time and budgetary constraints imposed by the Client.

6. In preparing this report, GZA relied on certain information provided by the Client, state and local officials, and other parties referenced therein available to GZA at the time of the evaluation. GZA did not attempt to independently verify the accuracy or completeness of all information reviewed or received during the course of this evaluation.

7. Any GZA hydrologic analysis presented herein is for the rainfall volumes and distributions stated herein. For storm conditions other than those analyzed, the response of the site's spillway, impoundment, and drainage network has not been evaluated.

COMPLIANCE WITH CODES AND REGULATIONS

8. We used reasonable care in identifying and interpreting applicable codes and regulations. These codes and regulations are subject to various, and possibly contradictory, interpretations. Compliance with codes and regulations by other parties is beyond our control.

9. This scope of work does not include an assessment of the need for fences, gates, no-trespassing signs, repairs to existing fences and railings and other items which may be needed to minimize trespass and provide greater security for the facility and safety to the public. An evaluation of the project for compliance with OSHA rules and regulations is also excluded.

ADDITIONAL INFORMATION

10. In the event that the Client or others authorized to use this report obtain information on conditions at the site(s) not contained in this report, such information shall be brought to GZA's attention forthwith. GZA will evaluate such information and, on the basis of this evaluation, may modify the opinions stated in this report.

ADDITIONAL SERVICES

11. It is recommended that GZA be retained to provide services during any future: site observations, explorations, evaluations, design, implementation activities, construction and/or implementation of remedial measures recommended in this Report. This will allow us the opportunity to: i) observe conditions and compliance with our design concepts and opinions; ii) allow for changes in the event that conditions are other than anticipated; iii) provide modifications to our design; and iv) assess the consequences of changes in technologies and/or regulations.

APPENDIX B - HEC-HMS RESULTS

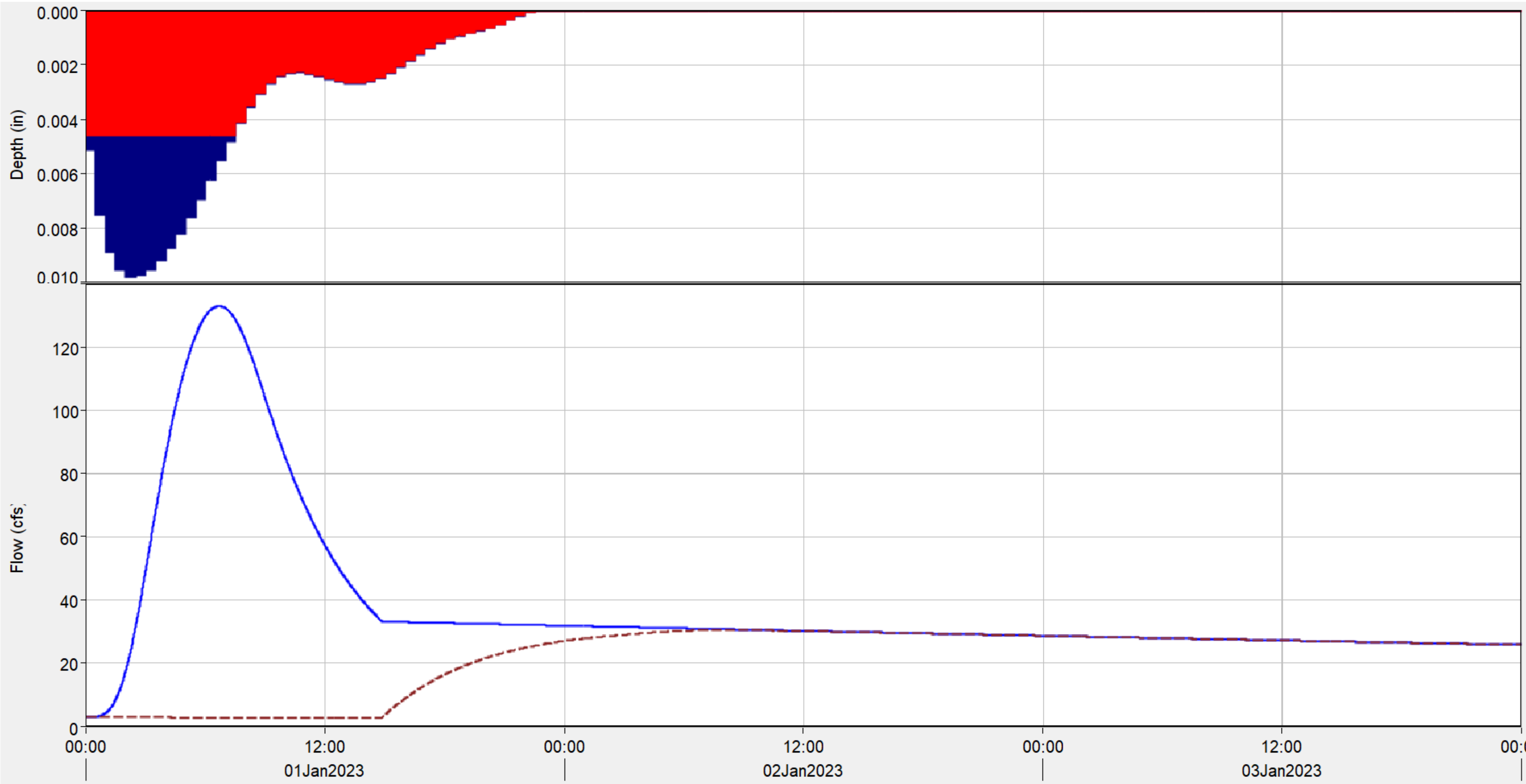


Figure B-1: Indian Brook Reservoir Dam 100-year Hydrograph

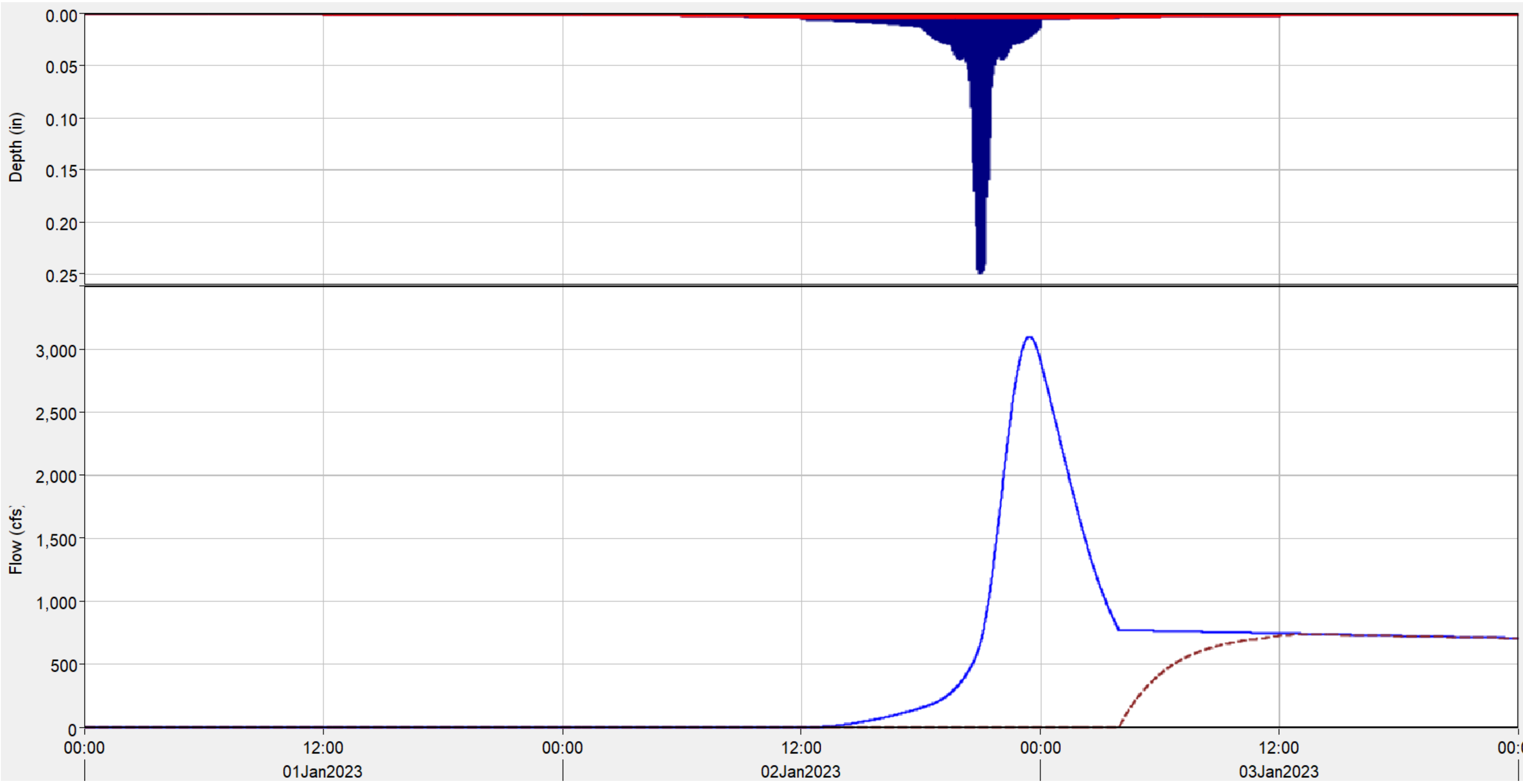
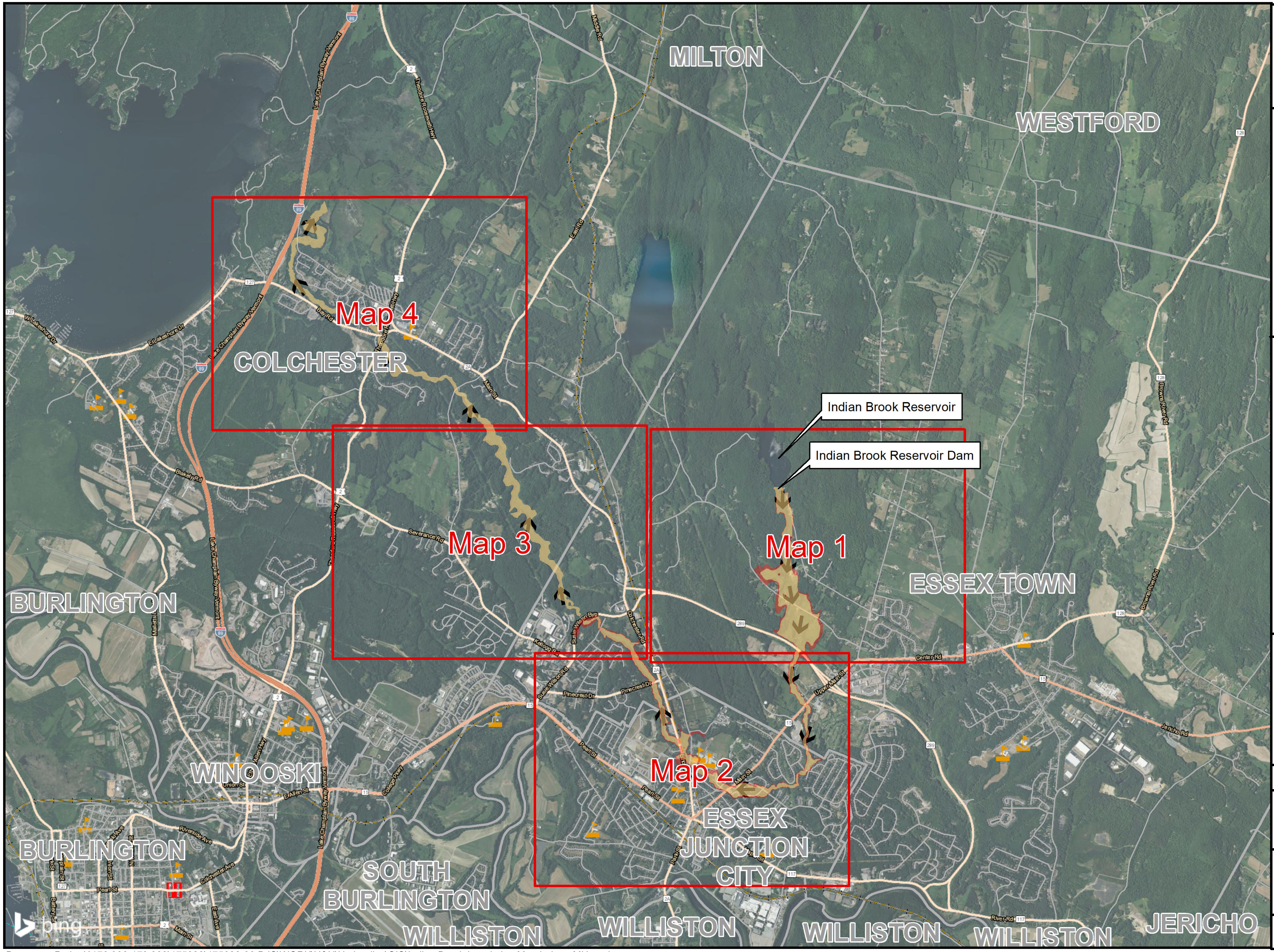


Figure B-2: Indian Brook Reservoir Dam PMF Hydrograph

APPENDIX C - INUNDATION MAPS



Legend

- AREA FLOODED BY DAM FAILURE DURING FAIR WEATHER
- AREA FLOODED BY DAM FAILURE DURING SDF
- FLOW DIRECTION
- HOSPITALS
- SCHOOLS
- RAIL LINES
- TOWN LINE

NOTES:

1. AERIAL PHOTO OBTAINED FROM BING MAPS.
2. ROADS FROM WORLD TRANSPORTATION.
3. VERTICAL DATUM: NAVD 88.
4. SCHOOL LOCATIONS FROM HIFLD (MAY - DEC 2022).
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9. BUILDINGS FROM VT E911 (JAN 2023).
10. TOWN BOUNDARIES FROM VT E911 (JAN 2023).
11. THE INUNDATION AREAS SHOWN ON THIS MAP REFLECT EVENTS OF AN EXTREMELY REMOTE NATURE. THESE RESULTS ARE NOT IN ANY WAY INTENDED TO REFLECT UPON THE INTEGRITY OF INDIAN BROOK RESERVOIR DAM.
12. THE INUNDATION AREA SHOWN IS APPROXIMATE AND SHOULD BE USED AS A GUIDELINE FOR ESTABLISHING EVACUATION ZONES.
13. ACTUAL INUNDATION AREA WILL DEPEND ON ACTUAL FAILURE CONDITIONS AND MAY DIFFER FROM THIS MAP.
14. INUNDATION AREA WAS CALCULATED BY SIMULATING DAM FAILURE WITH HEC-RAS v.6.3.1 SOFTWARE.
15. DAM FAILURE DURING FAIR WEATHER WAS SIMULATED WITH INDIAN BROOK RESERVOIR AT NORMAL POOL ELEVATION (531.1 FT NAVD88) AND WITH NORMAL BASEFLOWS.
16. DAM FAILURE DURING SDF WAS SIMULATED WITH INDIAN BROOK RESERVOIR AT SDF POOL ELEVATION (531.99 FT NAVD88).

INUNDATION MAP

INDIAN BROOK RESERVOIR DAM (VT00055)

ESSEX, VERMONT

INDEX MAP

Prepared For:
VERMONT DEPARTMENT OF ENVIRONMENTAL CONSERVATION
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Job No.: 01.0175988.00

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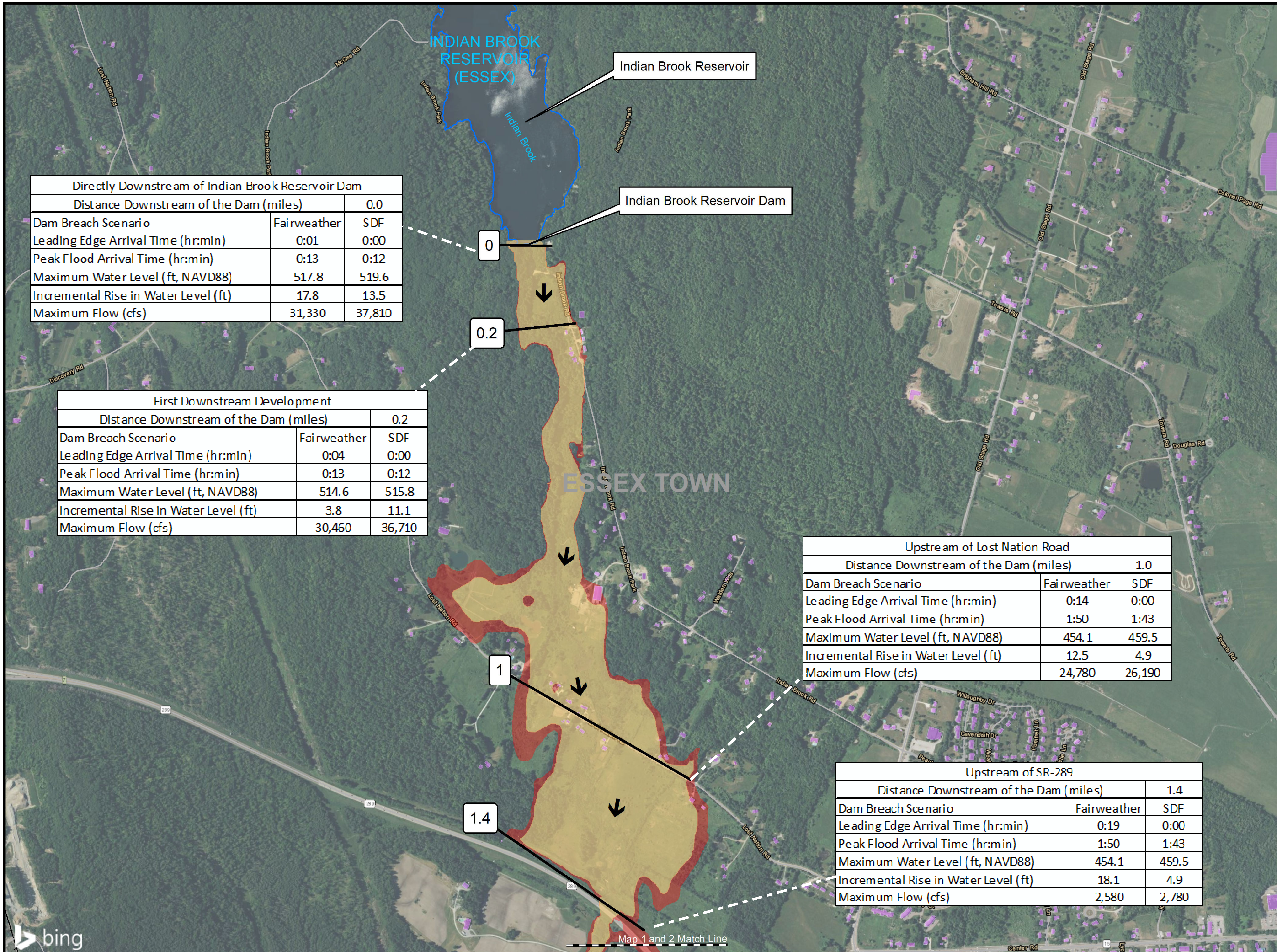

Directly Downstream of Indian Brook Reservoir Dam		
Distance Downstream of the Dam (miles)		0.0
Dam Breach Scenario	Fairweather	SDF
Leading Edge Arrival Time (hr:min)	0:01	0:00
Peak Flood Arrival Time (hr:min)	0:13	0:12
Maximum Water Level (ft, NAVD88)	517.8	519.6
Incremental Rise in Water Level (ft)	17.8	13.5
Maximum Flow (cfs)	31,330	37,810

First Downstream Development		
Distance Downstream of the Dam (miles)		0.2
Dam Breach Scenario	Fairweather	SDF
Leading Edge Arrival Time (hr:min)	0:04	0:00
Peak Flood Arrival Time (hr:min)	0:13	0:12
Maximum Water Level (ft, NAVD88)	514.6	515.8
Incremental Rise in Water Level (ft)	3.8	11.1
Maximum Flow (cfs)	30,460	36,710

Upstream of Lost Nation Road		
Distance Downstream of the Dam (miles)		1.0
Dam Breach Scenario	Fairweather	SDF
Leading Edge Arrival Time (hr:min)	0:14	0:00
Peak Flood Arrival Time (hr:min)	1:50	1:43
Maximum Water Level (ft, NAVD88)	454.1	459.5
Incremental Rise in Water Level (ft)	12.5	4.9
Maximum Flow (cfs)	24,780	26,190

Upstream of SR-289		
Distance Downstream of the Dam (miles)		1.4
Dam Breach Scenario	Fairweather	SDF
Leading Edge Arrival Time (hr:min)	0:19	0:00
Peak Flood Arrival Time (hr:min)	1:50	1:43
Maximum Water Level (ft, NAVD88)	454.1	459.5
Incremental Rise in Water Level (ft)	18.1	4.9
Maximum Flow (cfs)	2,580	2,780

N

0 500 1,000 2,000 Feet

Legend

- AREA FLOODED BY DAM FAILURE DURING FAIR WEATHER
- AREA FLOODED BY DAM FAILURE DURING SDF
- FLOW DIRECTION
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- OBSERVATION LINES
- MATCH LINES
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- HOSPITALS
- RAIL LINES
- BUILDINGS

NOTES:

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INUNDATION MAP

INDIAN BROOK RESERVOIR DAM (VT00055)

ESSEX, VERMONT

MAP 1 OF 4

Prepared For:
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Reviewed By: JPG
Operator: AMR

Dwg. Date: 8/21/2023

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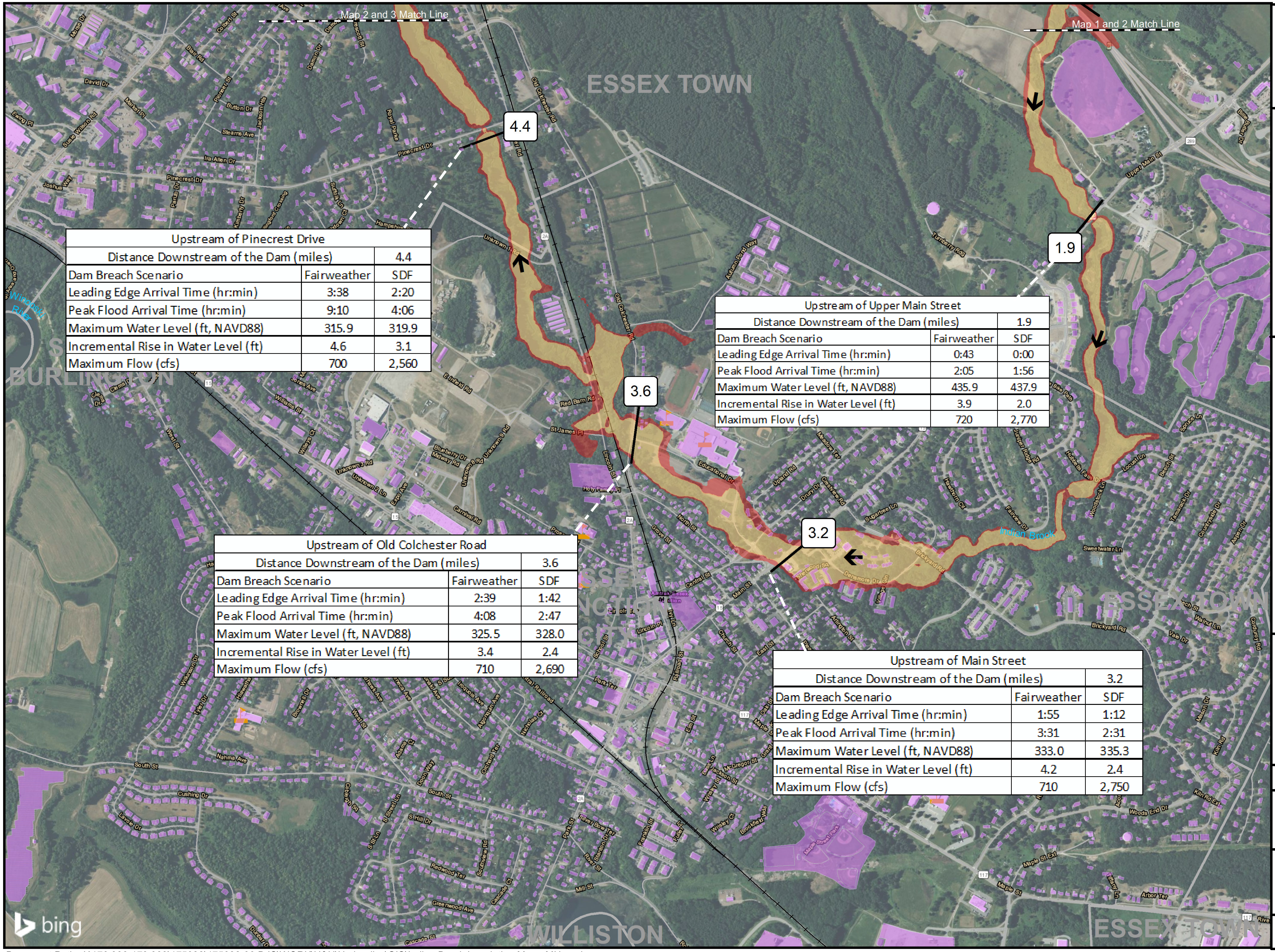

Upstream of Pinecrest Drive		
Distance Downstream of the Dam (miles)		4.4
Dam Breach Scenario	Fairweather	SDF
Leading Edge Arrival Time (hr:min)	3:38	2:20
Peak Flood Arrival Time (hr:min)	9:10	4:06
Maximum Water Level (ft, NAVD88)	315.9	319.9
Incremental Rise in Water Level (ft)	4.6	3.1
Maximum Flow (cfs)	700	2,560

Upstream of Upper Main Street		
Distance Downstream of the Dam (miles)		1.9
Dam Breach Scenario	Fairweather	SDF
Leading Edge Arrival Time (hr:min)	0:43	0:00
Peak Flood Arrival Time (hr:min)	2:05	1:56
Maximum Water Level (ft, NAVD88)	435.9	437.9
Incremental Rise in Water Level (ft)	3.9	2.0
Maximum Flow (cfs)	720	2,770

Upstream of Old Colchester Road		
Distance Downstream of the Dam (miles)		3.6
Dam Breach Scenario	Fairweather	SDF
Leading Edge Arrival Time (hr:min)	2:39	1:42
Peak Flood Arrival Time (hr:min)	4:08	2:47
Maximum Water Level (ft, NAVD88)	325.5	328.0
Incremental Rise in Water Level (ft)	3.4	2.4
Maximum Flow (cfs)	710	2,690

Upstream of Main Street		
Distance Downstream of the Dam (miles)		3.2
Dam Breach Scenario	Fairweather	SDF
Leading Edge Arrival Time (hr:min)	1:55	1:12
Peak Flood Arrival Time (hr:min)	3:31	2:31
Maximum Water Level (ft, NAVD88)	333.0	335.3
Incremental Rise in Water Level (ft)	4.2	2.4
Maximum Flow (cfs)	710	2,750

N

0 500 1,000 2,000 Feet

Legend

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- AREA FLOODED BY DAM FAILURE DURING SDF
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3. VERTICAL DATUM: NAVD 88.
4. SCHOOL LOCATIONS FROM HIFLD (MAY - DEC 2022).
5. HOSPITAL LOCATIONS FROM THE VT OPEN GEODATA (OCT 2022).
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11. THE INUNDATION AREAS SHOWN ON THIS MAP REFLECT EVENTS OF AN EXTREMELY REMOTE NATURE. THESE RESULTS ARE NOT IN ANY WAY INTENDED TO REFLECT UPON THE INTEGRITY OF INDIAN BROOK RESERVOIR DAM.
12. THE INUNDATION AREA SHOWN IS APPROXIMATE AND SHOULD BE USED AS A GUIDELINE FOR ESTABLISHING EVACUATION ZONES.
13. ACTUAL INUNDATION AREA WILL DEPEND ON ACTUAL FAILURE CONDITIONS AND MAY DIFFER FROM THIS MAP.
14. INUNDATION AREA WAS CALCULATED BY SIMULATING DAM FAILURE WITH HEC-RAS v.6.3.1 SOFTWARE.
15. DAM FAILURE DURING FAIR WEATHER WAS SIMULATED WITH INDIAN BROOK RESERVOIR AT NORMAL POOL ELEVATION (531.1 FT NAVD88) AND WITH NORMAL BASEFLOWS.
16. DAM FAILURE DURING SDF WAS SIMULATED WITH INDIAN BROOK RESERVOIR AT SDF POOL ELEVATION (531.99 FT NAVD88).

INUNDATION MAP

INDIAN BROOK RESERVOIR DAM (VT00055)

ESSEX, VERMONT

MAP 2 OF 4

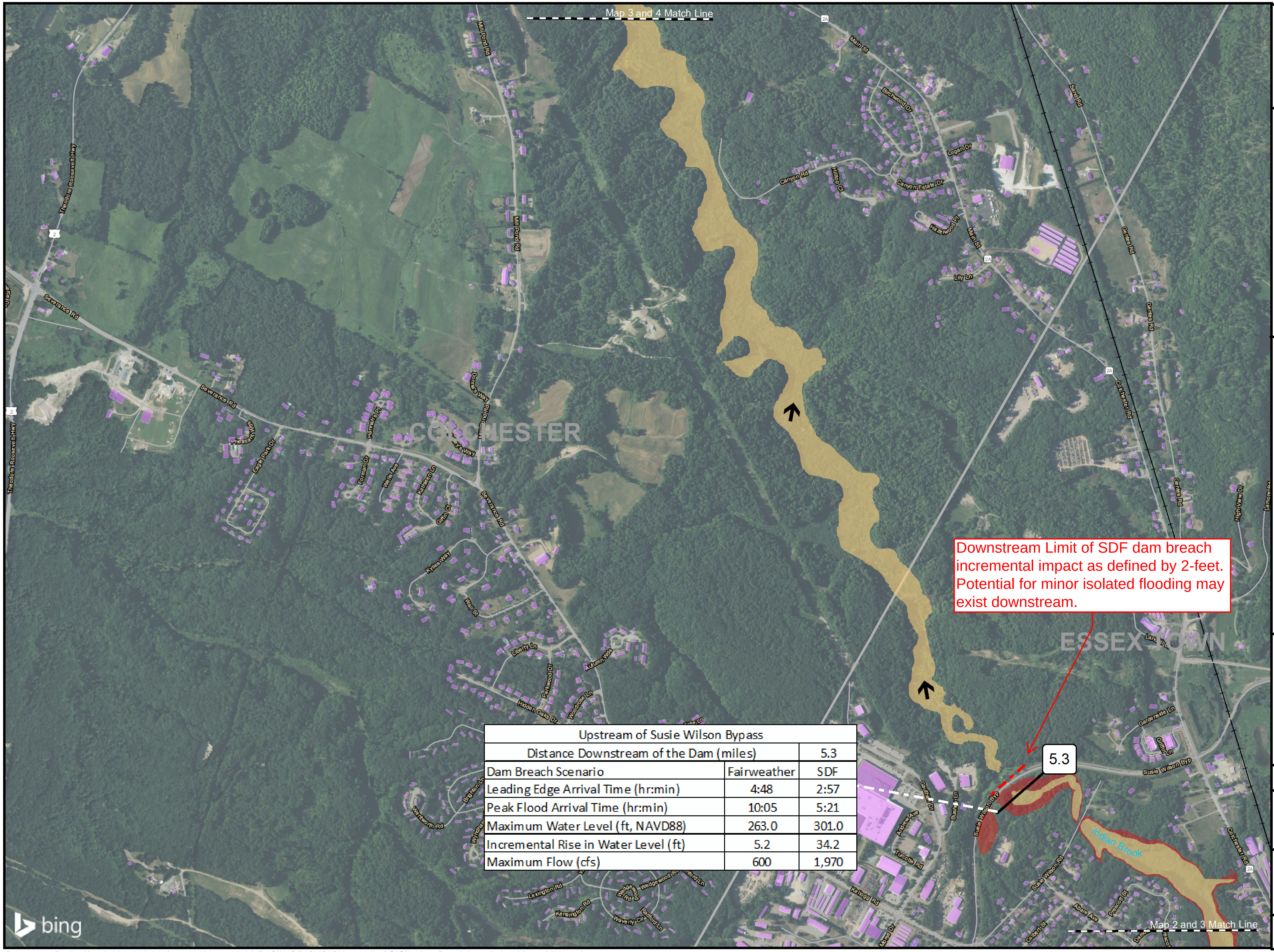
Prepared For:
VERMONT DEPARTMENT OF ENVIRONMENTAL CONSERVATION
One National Life Drive
Montpelier, Vermont 05620-3510

Prepared By:
GZA GeoEnvironmental, Inc.
249 Vanderbilt Ave
Norwood, MA 02062
Phone: (781) 278-3700 Fax: (781) 278-5701

Proj. Mgr.: DJS
Designed By: AMR
Reviewed By: JPG
Operator: AMR

Dwg. Date: 8/21/2023
Job No.: 01.0175988.00

Document Path: J:\170,000-179,999\175988\175988-00.DJS\WORK\H&H\Hydraulics\GIS\Indian Brook Inundation Map - MH.mxd



Upstream of Susie Wilson Bypass		
Distance Downstream of the Dam (miles)		5.3
Dam Breach Scenario	Fairweather	SDF
Leading Edge Arrival Time (hr:min)	4:48	2:57
Peak Flood Arrival Time (hr:min)	10:05	5:21
Maximum Water Level (ft, NAVD88)	263.0	301.0
Incremental Rise in Water Level (ft)	5.2	34.2
Maximum Flow (cfs)	600	1,970

N

0 500 1,000 2,000 Feet

Legend

- AREA FLOODED BY DAM FAILURE DURING FAIR WEATHER
- AREA FLOODED BY DAM FAILURE DURING SDF
- FLOW DIRECTION
- MILES DOWNSTREAM FROM INDIAN BROOK DAM
- OBSERVATION LINES
- MATCH LINES
- TOWN LINE
- SCHOOLS
- HOSPITALS
- RAIL LINES
- BUILDINGS

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INUNDATION MAP

INDIAN BROOK RESERVOIR DAM (VT00055)

ESSEX, VERMONT

MAP 3 OF 4

Prepared For:
VERMONT DEPARTMENT OF ENVIRONMENTAL CONSERVATION
One National Life Drive
Montpelier, Vermont 05620-3510

Prepared By:
GZA GeoEnvironmental, Inc.
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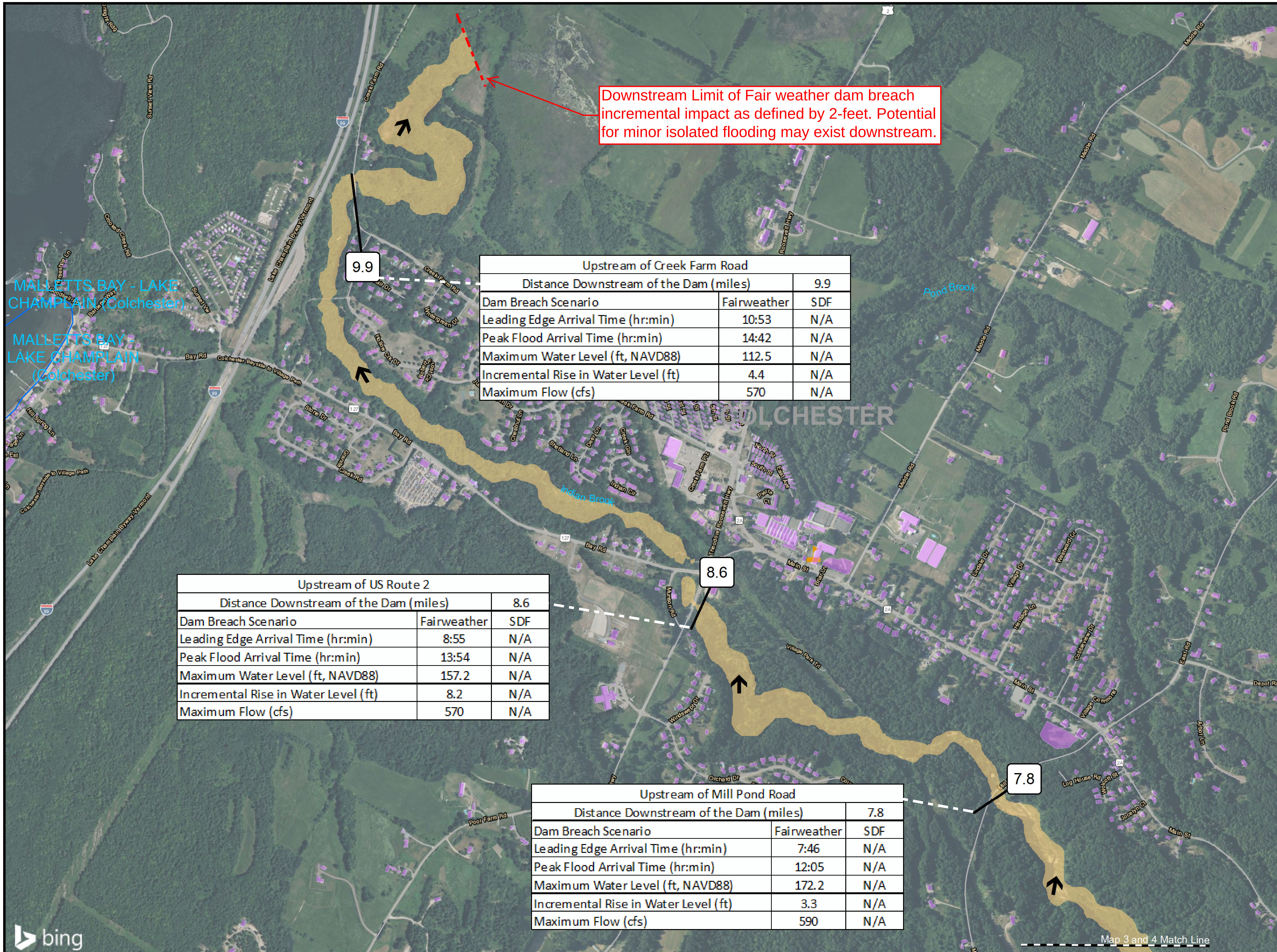
Norwood, MA 02062
Phone: (781) 278-3700 Fax: (781) 278-5701

Proj. Mgr.: DJS
Designed By: AMR
Reviewed By: JPG
Operator: AMR

Dwg. Date: 8/21/2023
Job No.: 01.0175988.00

Document Path: J:\170,000-179,999\175988\175988-00.DJS\WORK\H&H\Hydraulics\GIS\Indian Brook Inundation Map - MH.mxd



Upstream of Creek Farm Road		
Distance Downstream of the Dam (miles)		9.9
Dam Breach Scenario	Fairweather	SDF
Leading Edge Arrival Time (hr:min)	10:53	N/A
Peak Flood Arrival Time (hr:min)	14:42	N/A
Maximum Water Level (ft, NAVD88)	112.5	N/A
Incremental Rise in Water Level (ft)	4.4	N/A
Maximum Flow (cfs)	570	N/A

Upstream of US Route 2		
Distance Downstream of the Dam (miles)		8.6
Dam Breach Scenario	Fairweather	SDF
Leading Edge Arrival Time (hr:min)	8:55	N/A
Peak Flood Arrival Time (hr:min)	13:54	N/A
Maximum Water Level (ft, NAVD88)	157.2	N/A
Incremental Rise in Water Level (ft)	8.2	N/A
Maximum Flow (cfs)	570	N/A

Upstream of Mill Pond Road		
Distance Downstream of the Dam (miles)		7.8
Dam Breach Scenario	Fairweather	SDF
Leading Edge Arrival Time (hr:min)	7:46	N/A
Peak Flood Arrival Time (hr:min)	12:05	N/A
Maximum Water Level (ft, NAVD88)	172.2	N/A
Incremental Rise in Water Level (ft)	3.3	N/A
Maximum Flow (cfs)	590	N/A

N

0 500 1,000 2,000 Feet

Legend

- AREA FLOODED BY DAM FAILURE DURING FAIR WEATHER
- AREA FLOODED BY DAM FAILURE DURING SDF
- FLOW DIRECTION
- MILES DOWNSTREAM FROM INDIAN BROOK DAM
- OBSERVATION LINES
- MATCH LINES
- TOWN LINE
- SCHOOLS
- HOSPITALS
- RAIL LINES
- BUILDINGS

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INUNDATION MAP

INDIAN BROOK RESERVOIR DAM (VT00055)

ESSEX, VERMONT

MAP 4 OF 4

Prepared For:
VERMONT DEPARTMENT OF ENVIRONMENTAL CONSERVATION
One National Life Drive
Montpelier, Vermont 05620-3510

Prepared By:
GZA GeoEnvironmental, Inc.
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Proj. Mgr.: DJS
Designed By: AMR
Reviewed By: JPG
Operator: AMR

Dwg. Date: 8/21/2023
Job No.: 01.0175988.00

APPENDIX D – GZA STABILITY EVALUATION CALCULATIONS

GZA GeoEnvironmental
Engineers and Scientists
249 Vanderbilt Ave
Norwood, Massachusetts, 02062
844-342-5492
FAX 781-278-5701
http://www.gza.com

JOB	01.0175988.00		
	Indian Brook Dam, Essex, VT		
SHEET NO.	1	OF	2
CALCULATED BY	MZ	DATE	6/26/2023
CHECKED BY	JGD	DATE	6/29/2023
SCALE	N/A		

Objective:

To perform a stability analysis of the Indian Brook Reservoir Dam, Essex, VT, at Cross-section A-A (auxiliary spillway), Cross-section B-B (primary spillway), and an intermidiate failure plane (along the lift line, use the same geometry of A-A).

Methodology:

1) Develop stability using convertional equilibrium analyses and limit state theory, and back-calculate the required cohesion to for cases where minimum factor of safety is not achieved.
2) Calculate base pressures with and without considering uplift effects.
3) Determine non-compression zone, where a cracked section is assumed to have developed and is assumed to be subjected to full headwater pressure.
4) Uplift pressure profile is revised and a cracked length is obtained using an iterative solution per USACE methodology.

Loading Conditions:

- Case #1 - Normal Water levels
- Case #2 - Flood (SDF) Water Levels
- Case #3- Normal Water levels + Ice
- Case #4- Normal Water Levels + Earthquake

Subsurface Information:

- No test borings available.
- The datum used for the elevations presented herein include the conversion of NGVD29 - 0.4' = NAVD88.

Basic Parameters and Assumptions:

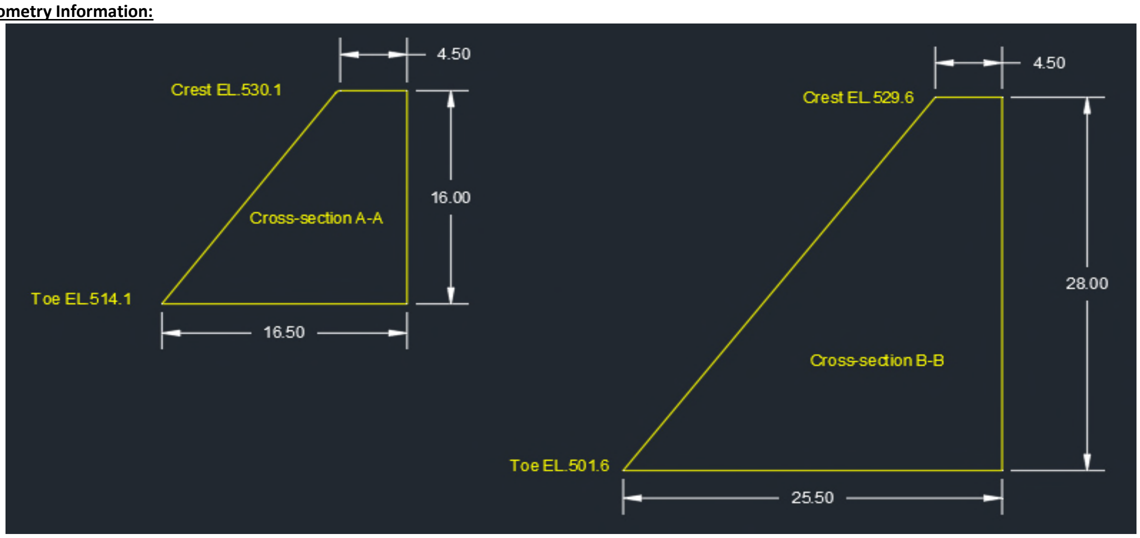
- Assumed no positive "key" extends into the bedrock or that any rock anchors exist.
- The flood pool elevation is 531.8 based on the H&H simulation performed by GZA in 2023.
- Assumed the Normal Pool elevation is at EL.529.6.
- Assumed no tailwater for both A-A (Auxiliary Spillway) and B-B (Primary Spillway) cross-section.
- Horizontal seismic coefficient λ = 0.198g, for bedrock and a 2500-year return period per 2018 USGS data.
- Assumed the interface friction angle φ.dam = 40°, for concrete on clean sound rock.
- Assumed the allowable foundation bearing capacity if 15 ksf.
- Assumed the allowable compressive strength of dam is 3000 psi.
- No upstream or downstream fill.
- Ice thickness: 1 foot (a "unusual" case per guidance by the USACE Engineer Manual 1110-2-2200 Table 4-1).
- Ice Pressure: 5 kips/ft (5,000 pounds per linear foot).

- Refer to **Appendix A** for Dam information.

Key Inputs:

- Calculation conservatively assumes no structure embedment into bedrock, and conservatively neglects cohesion present between base of dam and foundation for Cross-sections A-A and B-B.
- Calculation assumes a cohesion of 50 psi at failure plane at elevation 514.1'.
- Assumed interface friction angle may need to be revised depending on the orientation of bedrock bedding planes or foliation.

Geometry Information:



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JOB	01.0175988.00		
	Indian Brook Dam, Essex, VT		
SHEET NO.	2	OF	2
CALCULATED BY	MZ	DATE	6/26/2023
CHECKED BY	JGD	DATE	6/29/2023
SCALE	N/A		

STABILITY ANALYSIS RESULTS - EXISTING CONDITIONS

Cross-Section	Water Level	Sliding				Overturning			
		Minimum Required FS	Calculated FS	Base Cohesion Req'd for Min FS (psi)	Cracked Base Analysis	Calculated Resultant Location	Calculated Base Pressure at Toe (ksf)	Bearing Capacity OK?	Dam Compressive Strength OK?
A-A *On the Right of Gate Structure Base EL.514.1*	Normal	2.0	*1.9*	0.2	NOT REQUIRED	Within Middle 1/3	1.0	OK	OK
	Flood (PMF)	1.1	1.4	N/A	NOT REQUIRED	Within Base	1.4	OK	OK
	Normal + Ice	2.0	*1.3*	3.9	REQUIRED	*Outside Middle 1/2*	3.0	OK	OK
	Normal + Earthquake	1.3	*1.0*	1.6	NOT REQUIRED	Within Base	2.0	OK	OK
B-B *On the Left of Gate Structure Base EL.501.6*	Normal	2.0	*1.4*	4.0	NOT REQUIRED	Within Middle 1/3	2.2	OK	OK
	Flood (PMF)	1.1	*1.1*	0.3	NOT REQUIRED	Within Base	2.5	OK	OK
	Normal + Ice	2.0	*1.1*	7.6	REQUIRED	Within Middle 1/2	3.6	OK	OK
	Normal + Earthquake	1.3	*0.8*	5.7	NOT REQUIRED	Within Base	4.0	OK	OK
Intermidiate Failure Plane *On the Right of Gate Structure Base EL.514.1*	Normal	2.0	15.8	N/A	NOT REQUIRED	Within Middle 1/3	1.0	OK	OK
	Flood (PMF)	1.1	11.9	N/A	NOT REQUIRED	Within Base	1.4	OK	OK
	Normal + Ice	2.0	9.0	N/A	REQUIRED	*Outside Middle 1/2*	3.0	OK	OK
	Normal + Earthquake	1.3	8.4	N/A	NOT REQUIRED	Within Base	2.0	OK	OK

Notes: - Calculated factor of safety less than minimum values are in red italic.
- The resultant location is relative to the overall base width.
- Refer to **Appendix B** for MathCad calculation sheets.

References:

- "Evaluation of Concrete Dam Safety" by ASDSO. Northeast Regional Technical Seminar
- "Gravity Dam Design" by USACE EM1110-2-2200, Jun 1995
- "Stability Analysis of Concrete Structures" by USACE EM1110-2-2100, Dec 2005
- "Design of Small Dams" by US Bureau of Reclamation, 1977
- "Evaluation and comparison of stability analysis and uplift criteria for concrete gravity dams by three federal agencies by USACE ERDC/ITL TR-00-1, Jun 2000

APPENDIX A

Dam Information

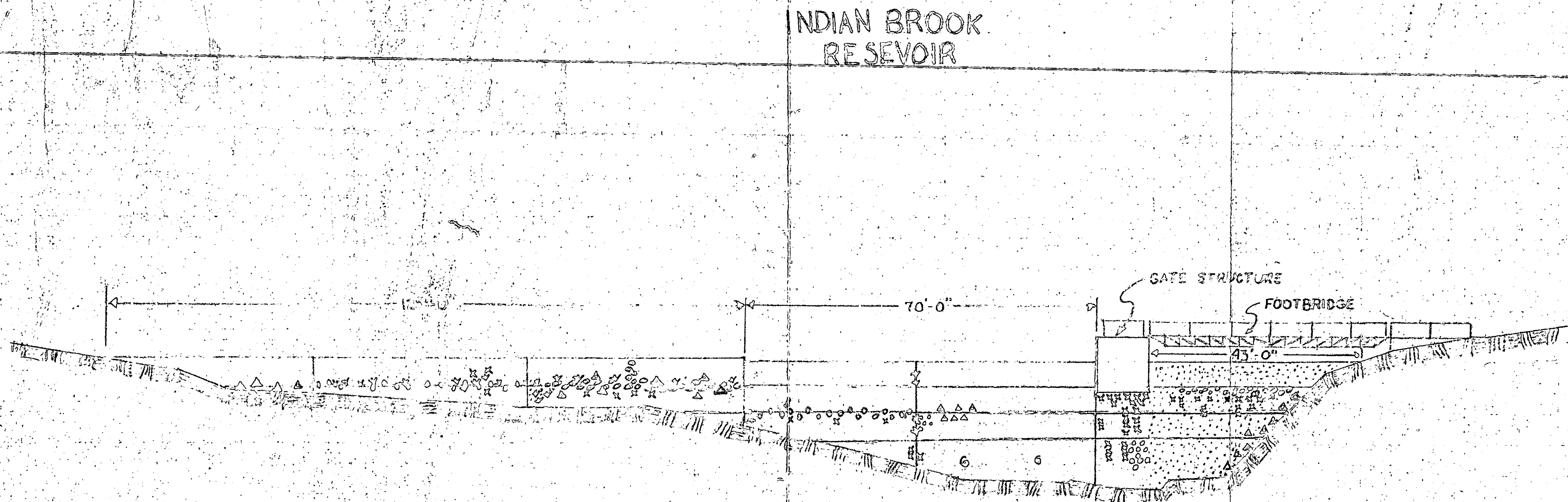
III INTRODUCTION

The Town of Essex is located in Chittenden County, State of Vermont, just east of the City of Burlington. Indian Brook Dam is located at the headwaters of Indian Brook in the north end of the Town of Essex. Indian Brook flows southerly to the Village of Essex Junction, then northwesterly to Lake Champlain.

Indian Brook Dam was designed in 1955 by Whitman and Howard Engineers, Inc. of Boston, Massachusetts, to create a water supply (Indian Brook Reservoir) for the Village of Essex Junction. Construction on the dam was completed in 1957. The dam was owned and operated by the Village until 1978, when it was sold to Mr. Steve G. Phillips of Waitsfield, Vermont. The Town of Essex purchased the dam and the surrounding property from Mr. Phillips on December 31, 1986. The dam is currently being operated by the Town of Essex Parks and Recreation Department and Indian Brook Reservoir is currently used for recreation purposes.

Indian Brook Dam is a concrete gravity structure, founded on bedrock, approximately 248 feet long and 26.5 feet high. A steel footbridge extends 48 feet from the left abutment to the concrete gate structure, housing three intake gates and two outlet gates. The dam spillways are ogee sections, approximately 4 feet wide at the crest. The right 125 feet of crest has an elevation of 530.5. The remainder of the crest is at elevation 530.0.

Under the National Dam Inspection Program, a Phase I Inspection Report was prepared by James W. Sewall Company for the Army Corps of Engineers, dated June 1980. The dam was considered to be in fair condition at that time.



PROFILE

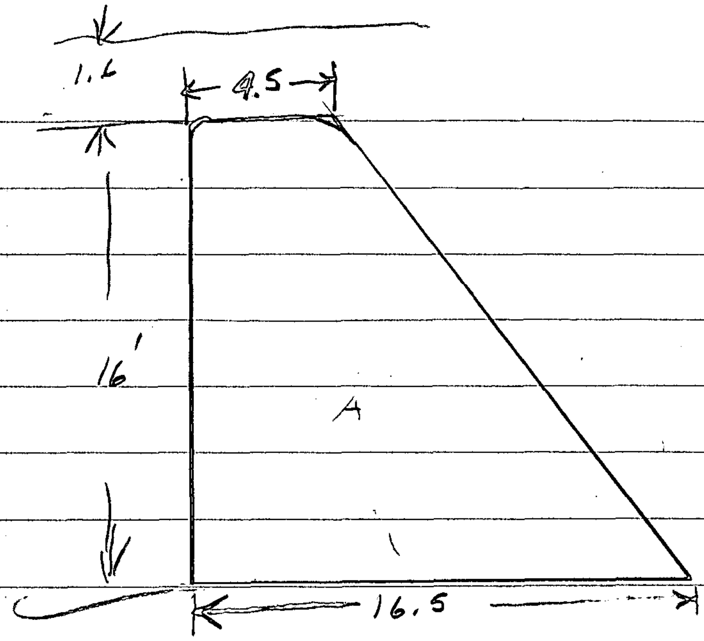
1" = 20'

KEY:
- MOSS
- EROSION
- SEEPAGE
- EFFLORESCENCE
- SPALL
- GRASS
- MUDSTONES

INDIAN BROOK DAM
ESSEX

INSPECTION: RBF, JMV
DATE: 5-22-2001
DRAWING: 6-20-2001
PLAN ADOPTED FROM
SITE PLAN

Essex dam



1.	Portion A	+ 150 (168) = 25,200	3.7	93,240
11.	uplift	− 62.5 (17.6)(.5)(8) = −4400	5.5	− 24,200

ΣW 20,800

13	$+\frac{62.5(17.6)^2}{2} = 9,800$	+ 8.8	+ 86,240
13 A	$-\frac{62.5(1.6)^2}{2} = -78$	17.6 − 1.1 = 16.5	− 1,287

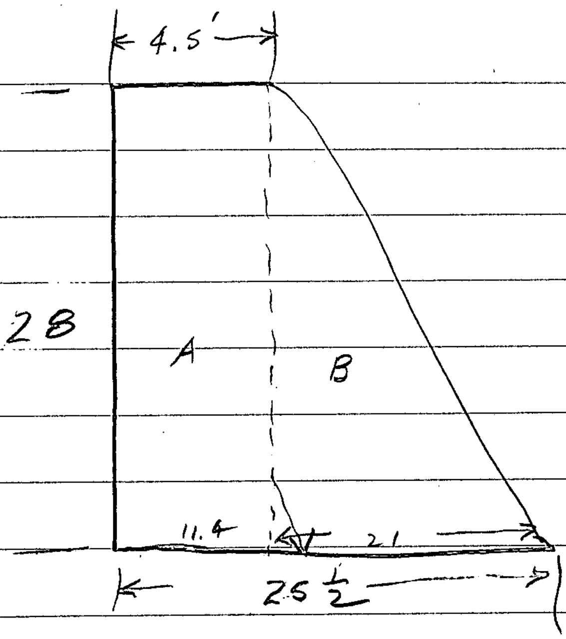
ΣP 9,700 ΣM 164,000

$\frac{9,700}{20,800} < f'$ $f' = .75$

.47 < .75 OK for sliding

OK for stability

Essex Dam



1. Portion A	+150 (126) = +18,900	2.25	+42,500
" B	+150 (294) = +44,100	11.5	+507,150
11 Uplift	$-62.5(.5)(31.6)\left(\frac{25.5}{2}\right) = -12{,}620$	+8.5	−107,270

ΣW 63,000

13	$\frac{62.5(29.6)^2}{2} = +28{,}000$	$+\frac{29.6}{3} = 9.9$	+280,000
13 A	$-62.5 \times \frac{(1.6)^2}{2} = -80$	+28.6	−2,300

ΣP 27,000 ΣM 720,000

$\frac{27000}{63000} < f'$ $f' = .75$

.43 < .75 OK Sliding

$\frac{720,000}{63,000} = 11.4$ OK Stability

VI METHODS AND PROCEDURES

A number of information sources were researched for this report. The Vermont Department of Water Resources maintains a file of dams in the state. The Indian Brook Dam file contains the construction drawings, specifications, pictures, and inspection report data. Construction records and shop drawings from Whitman and Howard Engineers could not be located. Files from the Town of Essex, Mr. Phillips, and the Champlain Water District were reviewed for operation and maintenance records, but no records were located.

As recommended in the Phase I Report and by G & Underwood Engineers, the Town contracted to recondition the three intake gates and one outlet gate to allow draining of the reservoir. The right outlet gate is connected to a plugged/abandoned pipe and did not warrant repairs. The work was performed in May and June 1988 by Reliance Steel, Inc. of Colchester, Vermont. The work involved cleaning of the gates, replacement of rusted nuts, bolts and pins with stainless steel parts, reinstalling the stem guides with expansion anchors, installing new stainless steel stems, and providing new operators with locking covers. The gates were lubricated and the wedges adjusted to assure a good seal when closed. Other work at the site included clearing debris from the control tower and the 15 inch outlet, reinstalling grating on the footbridge, grouting of abandoned bolt holes, and painting the footbridge and handrails.

The reservoir was drawn down 63 inches below the dam crest to elevation 524.8 between July 11, 1988 and August 1, 1988. The middle intake gate was used in order to minimize siltation of Indian Brook from reservoir bottom sediments. Reliance Steel, Inc. had a representative on-site at all times when the gates were opened to assure the public's safety.

VIII STRUCTURAL STABILITY EVALUATION

The Army Corps of Engineers Phase I Report did not include an in-depth stability analysis because of a lack of detailed engineering data for the dam. However, based on the limited information available including record drawings and conversations with people involved with the dam construction, a stability analysis of overturning and sliding was prepared for this report. Calculations are presented in Appendix II.

Full uplift pressure is assumed to act on the dam. It is assumed that no positive "key" extends into the bedrock or that any rock anchors exist. Sliding calculations were performed both with and without cohesion forces. The water level was assumed to be at the elevation of the test flood surcharge elevation of 531.5 feet, as calculated in the Phase I Report. Since the dam is within a Zone 2 seismic area, a seismic investigation is not warranted.

The stability calculations indicate that positive vertical pressure is sustained along the base of the dam during test flood. Therefore, the dam is stable in regards to the overturning forces of upstream water pressure and uplift. Using a conservative assumption of 15 pounds per square inch for cohesion, a 3.1 factor of safety against sliding is calculated. If it is assumed that no cohesion exists, which is highly unlikely, the factor of safety is calculated at 1.2 against sliding.

9. Total project discharge at test flood el. 531.5: 1220 cfs

*Dam is designed to overflow its entire length. Elevation 533 is the low steel of the outlet works access bridge.

c. Elevation (Feet NGVD)

1.	Streambed at toe of dam	504±
2.	Bottom of cutoff	N/A
3.	Maximum tailwater	N/A
4.	Recreation pool	530±
5.	Full flood control pool	N/A
6.	Spillway crest (ungated)	530
7.	Design surcharge (original design)	N/A
8.	Top of dam	530.5
9.	Test flood surcharge	531.5

d. Reservoir

1.	Length of normal pool	3150 ft
2.	Length of flood control pool	N/A
3.	Length of spillway crest pool	3150 ft
4.	Length of pool at top of dam	3150 ft
5.	Length of test flood pool	3150 ft

e. Storage

1.	Normal pool	1084 acre-ft
2.	Flood control pool	N/A
3.	Spillway crest pool	1084 acre-ft
4.	Top of dam	1084 acre-ft
5.	Test flood pool	1157 acre-ft

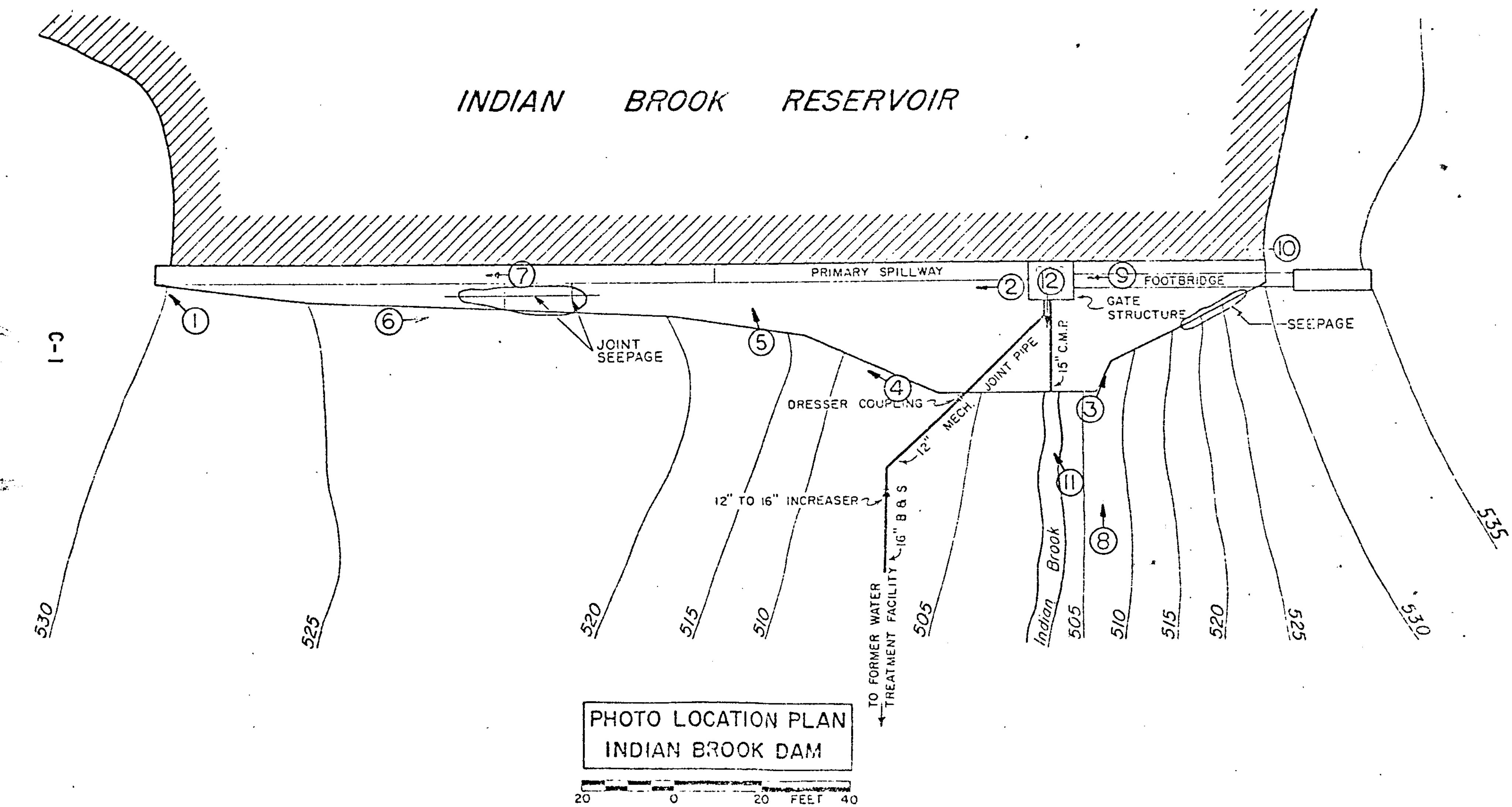


PHOTO LOCATION PLAN
INDIAN BROOK DAM

C-1

APPENDIX B

Gravity Structure Stability Calculation

Cross-Section A-A

(Unit Definition - Click Arrow to Expand)

Objective:

To perform a stability analysis of the **Indian Brook Reservoir Dam**, **A-A cross section** using assumption of cracked base where applicable, to calculate factors of safety against sliding and to evaluate overturning stability.

Design Methodology:

- Evaluate stability using conventional equilibrium analyses and limit state theory
- Calculate base pressures with and without considering uplift effects
- Determine non-compression zone, where a cracked section is assumed to have developed and is assumed to be subjected to full headwater pressure.
- Uplift pressure profile is revised and a cracked length is obtained using an iterative solution per USACE methodology

References:

- "Evaluation of Concrete Dam Safety" by ASDSO. Northeast Regional Technical Seminar
- "Gravity Dam Design" by USACE EM1110-2-2200, Jun 1995
- "Stability Analysis of Concrete Structures" by USACE EM1110-2-2100, Dec 2005
- "Design of Small Dams" by US Bureau of Reclamation, 1977
- "Evaluation and comparison of stability analysis and uplift criteria for concrete gravity dams by three federal agencies" by USACE ERDC/ITL TR-00-1, Jun 2000

Case Descriptions - Loading Conditions:

(Per US Army Corps of Engineers)

Case #1: Normal water levels
Case #2: Flood (SDF) water levels
Case #3: Normal water levels + ice
Case #4: Normal water levels + earthquake

Assumptions:

- Full upstream hydraulic head applied to cracked length and is linearly interpolated to downstream hydraulic head over uncracked length (depending on efficiency and location of relief wells)
- Pseudostatic method for seismic analysis (apply horizontal acceleration as a % of g)
- Summation of moments about the centerline of the base of the dam
- Plane of analysis at dam/foundation interface (EL. 514.1)

Notes for MathCAD User (No Calculations)

Input Parameters

1) Dam Geometry:

Total Base width	$B := 16.5ft$	
Toe base width	$B_1 := 12ft$	
Heel width	$B_2 := 0ft$	
Crest width	$B_c := B - B_1 - B_2 = 4.5\,ft$	
Height of heel slope	$H_2 := 16ft$	
Analysis length	$LF := 1ft$	
Dam base elevation	$EL_b := 514.1ft$	
Dam crest elevation	$EL_c := 530.1ft$	
Dam height	$H_{dam} := EL_c - EL_b = 16\,ft$	
Upstream batter angle (from vertical)	$\theta_u := atan\left(\frac{B_2}{H_2}\right) = 0\cdot°$	
Downstream batter angle (from vertical)	$\theta_d := atan\left(\frac{B_1}{H_{dam}}\right) = 36.87\cdot°$	
Upstream fill elevation	$EL_{fillus} := EL_b = 514.1\,ft$	*(No upstream fill)*
Inclination angle of base	$\beta := 0deg$	
Sliding direction (upslope / downslope to DS)	$f_\beta := 1$	(+1 sloping down to DS; and -1 sloping up to DS)
Drainage Gallery base elevation	$EL_{dg} := EL_b = 514.1\,ft$	*(No drainage gallery in this section)*
Drain Effectiveness	$E_{dr} := 0\%$	(fully effective = 100%; ineffective = 0%)
Distance of drain to heel	$d_{dr} := 0ft$	*(set to zero if no drain installed)*
Shear Key Area	$A_{shear} := 0ft^2$	*(set to zero if no shear keys present)*
Shear Key Cohesion	$c_{shear} := 0psf$	*(set to zero if no shear keys present)*

1) Dam Geometry (continued): *Refer to Figure #1*

Assume coordinates of toe (0,0) *(Geometry based on CAD)*

W_c: Concrete cross-section

$X_{Centroid.c} := 10.68ft$ $Y_{Centroid.c} := 6.48ft$

$Area_c := 168ft^2$

$F_{uwa.y}$ dimensions: headwater above heel

$X_{Centroid.uwa.y} := 0ft$ $Y_{Centroid.uwa.y} := 0ft$

$Area_{uwa.y} := 0ft^2$

$F_{fillus.y}$ dimensions: fill above toe (upstream side)

$X_{Centroid.fillus.y} := 0ft$ $Y_{Centroid.fillus.y} := 0ft$

$Area_{fillus.y} := 0ft^2$

▶ N/A Fields

2) General Design Elevations

Flood pool elevation $EL_{fw} := 532.1ft$

Normal pool elevation $EL_{nw} := 529.6ft$ *(Assumed at the crest elevation of primary spillway)*

Assume NO silt $H_{silt} := 0ft$ *(No silt)*

Silt/sediment surface level $EL_s := 0ft$

Silt surface slope angle $\alpha := 0°$

3) Case Specific Loads, Dimensions and Elevations

Tailwater Elevation
$$EL_{dw} := \begin{pmatrix} 514.1ft \\ 514.1ft \\ 514.1ft \\ 514.1ft \end{pmatrix} \begin{matrix} \text{Case \#1} \\ \text{Case \#2} \\ \text{Case \#3} \\ \text{Case \#4} \end{matrix}$$

Tailwater Height
$$H_{dw} := EL_{dw} - EL_b \quad H_{dw} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} ft \quad \begin{matrix} \text{Case \#1} \\ \text{Case \#2} \\ \text{Case \#3} \\ \text{Case \#4} \end{matrix}$$

Headwater Elevation
$$EL_{uw} := \begin{pmatrix} EL_{nw} \\ EL_{fw} \\ EL_{nw} \\ EL_{nw} \end{pmatrix} \begin{matrix} \text{Case \#1} \\ \text{Case \#2} \\ \text{Case \#3} \\ \text{Case \#4} \end{matrix}$$

Headwater Height
$$H_{uw} := EL_{uw} - EL_b \quad H_{uw} = \begin{pmatrix} 15.5 \\ 18 \\ 15.5 \\ 15.5 \end{pmatrix} ft \quad \begin{matrix} \text{Case \#1} \\ \text{Case \#2} \\ \text{Case \#3} \\ \text{Case \#4} \end{matrix}$$

Ice thickness $H_{ice} := 1ft$ *(initial assumption)*

Ice pressure $f_{ice} := 5ksf$ *(per USACE)*

Horizontal Seismic Coefficient $\lambda := 0.198$ % acceleration *(Class B PGA for bedrock and a 2500-year return period per 2018 USGS data)*

4) Basic Material Properties

Cohesion between dam/foundation $c_0 := 0psi$ *(Initial assumption)*

Cohesion between dam/foundation $c_{dam} := c_0$ *(Initial assumption)*

Interface friction angle

Concrete on clean sound rock $\phi_{dam} := 40°$ *(Initial assumption)*

Allowable foundation bearing capacity $BC := 15ksf$ *(Initial assumption)*

Allowable compressive strength of dam: $Cu_{dam} := 3000psi$ *(Initial assumption)*

Unit weight of dam material $\gamma_{concrete} := 150pcf$ *(Based on analysis parameter from other project)*

Unit weight of Fill material $\gamma_{fillus} := 0pcf$ *(No Upstream or Downstream Fill)*

$\gamma_{fillds} := 0pcf$

Fill internal frictional angle $\phi_{fillus} := 0°$

$\phi_{fillds} := 0°$

5) Design Factor of Safety (FS) against sliding

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Case		**Loading Conditions**	**Required Factor of Safety**	
			Ordinary Site Info	Well Defined Site Info
1	Normal Pool	Usual	2	1.7
2	PMF	Extreme	1.1	1.1
3	Normal Pool + Ice	Unusual	2	1.7
4	Normal Pool + Seismic	Extreme	1.3	1.1

Factors of Safety Used in Analysis (Ordinary)

$$FS\_SLIDING_{min} := \begin{pmatrix} 2.0 \\ 1.1 \\ 2.0 \\ 1.3 \end{pmatrix}$$

Case #1
Case #2 - See Notes
Case #3
Case #4

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Factor of Safety Notes

- PMF is considered "Extreme" due to return period of > 300 yr, in accodcance with EM 1110-2-2100.

▶ Misc, Input Parameter Notes (No Calculations)

LOAD CASE #1 - Normal Pool

I. INITIAL VERTICAL LOADS AND MOMENTS:

A. Dam self-weights - (Refer to FBD for dam geometry, variable notation, and sign convention)

Weight of individual Dam Sections

$$W_c := -Area_c \cdot \gamma_{concrete} \cdot LF = -25.2 \cdot kip$$

Moment arms about Center of Base:

$$D_c := X_{Centroid.c} - \frac{B}{2} = 2.43\,ft$$

Dam Weight Moment about centerline of dam:

$$M_c := W_c \cdot D_c = -61.236 \cdot kip \cdot ft$$

B. Headwater: (Vertical Component)

Conditional statement using variable H_water_over_crest
(*determines whether headwater higher than dam crest)*

$$H_{water\_over\_crest} := H_{uw_1} - H_{dam} = -0.5\,ft$$

Height of water above crest

$$H_{ov}(H_{water\_over\_crest}) := \begin{cases} H_{water\_over\_crest} & \text{if } H_{water\_over\_crest} > 0 \cdot ft \\ 0 & \text{otherwise} \end{cases}$$

$$H_{ov}(H_{water\_over\_crest}) = 0\,ft$$

Headwater weight on dam (if crest not overtopped, areas of b and c= 0)
a, b, c correspond to various areas of water over the dam, refer to FBD

Headwater Area A $\quad F_{uwa.y} := -\gamma_w \cdot Area_{uwa.y} \cdot LF = 0$

Headwater Area B $\quad F_{uwb.y} := 0kip$

Headwater Area C $\quad F_{uwc.y} := 0kip$

Σ Vertical components of Headwater over the Upstream face of the Dam

$$F_{uw.y} := F_{uwa.y} + F_{uwb.y} + F_{uwc.y} = 0$$

Moment arms of a, b & c about the Center of Base

Headwater Area A $\quad arm_{uwa.x} := X_{Centroid.uwa.y} - \frac{1}{2}B = -8.25\,ft \quad$ Above heel

Headwater Area B $\quad arm_{uwb.x} := 0 \quad$ Above crest

Headwater Area C $\quad arm_{uwc.x} := 0$

Moments of vertical headwater forces a, b & c forces about centerline:

Headwater Area A $M_{uwa.y} := F_{uwa.y} \cdot arm_{uwa.x} = 0 \cdot kip \cdot ft$

Headwater Area B $M_{uwb.y} := F_{uwb.y} \cdot arm_{uwb.x} = 0 \cdot kip \cdot ft$

Headwater Area C $M_{uwc.y} := F_{uwc.y} \cdot arm_{uwc.x} = 0 \cdot kip \cdot ft$

Σ Moments from the vertical component of Headwater on U/S face of the Dam

$$M_{uw.y} := M_{uwa.y} + M_{uwb.y} + M_{uwc.y} = 0 \cdot kip \cdot ft$$

C. Tailwater: (Vertical Component)

No vertical tailwater loads appiled.

D. Soil Loads: (Vertical Component due to Weight)

No upstream or downstream fill.

E. Uplift Pressure:

Drainage Gallery related calculation - Click to expand *(No drainage gallery)*

Head at heel $H_{heel} := H_{uw_1} = 15.5\,ft$

Head at toe $H_{toe} := H_{dw_1} = 0\,ft$ *(Ignore the tailwater here)*

Uplift Pressure at Heel $u_{up\_us} := (H_{heel}) \cdot \gamma_w = 0.967 \cdot ksf$

Uplift Pressure at Toe $u_{up\_ds} := H_{toe} \cdot \gamma_w = 0 \cdot ksf$

Uplift Pressure below Drainage Gallery $u_{up\_dg} := H_{dr} \cdot \gamma_w = 0.967 \cdot ksf$

Uplift forces below dam base:
Refer to FBD for notation:

$$U_1 := u_{up\_ds} \cdot B \cdot LF = 0 \cdot kip$$

$$U_2 := \frac{1}{2}(u_{up\_dg} - u_{up\_ds}) \cdot B \cdot LF = 7.979 \cdot kip$$

Σ Uplift Forces = $U := U_1 + U_2 = 7.979 \cdot kip$

Moment arms of Uplift Forces

$$d_{up1} := \frac{1}{2}B - \frac{1}{2}B = 0\,ft$$

$$d_{up2} := \frac{2}{3}B - \frac{1}{2}B = 2.75\,ft$$

Moments due to Uplift Components $M_{up1} := U_1 \cdot d_{up1} = 0 \cdot kip \cdot ft$

$M_{up2} := U_2 \cdot d_{up2} = 21.943 \cdot kip \cdot ft$

Σ **Uplift Moments =** $M_{up} := M_{up1} + M_{up2} = 21.943 \cdot kip \cdot ft$

II. INITIAL HORIZONTAL FORCES AND MOMENTS:

A. Headwater: (Horizontal Component)

Horizontal Component of Headwater on Dam

$$F_{uwa.x} := \begin{cases} \gamma_w \cdot \left(H_{uw_1} - H_{dam}\right) \cdot H_{dam} \cdot LF & \text{if } H_{ov}\left(H_{water\_over\_crest}\right) > 0 \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kip$$

$$F_{uwb.x} := \begin{cases} \frac{1}{2} \gamma_w \cdot H_{dam}^{\ 2} \cdot LF & \text{if } H_{ov}\left(H_{water\_over\_crest}\right) > 0 \\ \frac{1}{2} \gamma_w \cdot \left(H_{heel}\right)^2 \cdot LF & \text{otherwise} \end{cases} = 7.496 \cdot kip$$

Σ **Horizontal Forces by Headwater** $F_{uw.x} := F_{uwa.x} + F_{uwb.x} = 7.496\,kip$

Moment arms of Headwater on Dam

$$arm_{uwa.y} := \begin{cases} \frac{H_{dam}}{2} & \text{if } H_{ov}\left(H_{water\_over\_crest}\right) > 0 \\ 0 & \text{otherwise} \end{cases} = 0 \cdot ft$$

$$arm_{uwb.y} := \begin{cases} \frac{1}{3} H_{dam} & \text{if } H_{ov}\left(H_{water\_over\_crest}\right) > 0 \\ \frac{H_{heel}}{3} & \text{otherwise} \end{cases} = 5.167 \cdot ft$$

Moment due to Headwater on Dam

$M_{uwa.x} := F_{uwa.x} \cdot arm_{uwa.y} = 0 \cdot kip \cdot ft$

$M_{uwb.x} := F_{uwb.x} \cdot arm_{uwb.y} = 38.728 \cdot kip \cdot ft$

Σ **Moments due to Headwater** $M_{uw.x} := M_{uwa.x} + M_{uwb.x} = 38.728 \cdot kip \cdot ft$

B. Tailwater: (Horizontal Component)

Horizontal Force due to Tailwater $F_{dw.x} := \frac{-1}{2} \gamma_w \cdot \left(H_{dw_1}\right)^2 \cdot LF = 0\,kip$

Moment arm of Tailwater $arm_{dw.y} := \frac{1}{3} H_{dw_1} = 0\,ft$

Moment due to Tailwater $M_{dw.x} := F_{dw.x} \cdot arm_{dw.y} = 0\,ft \cdot kip$

C. Soil horizontal loading on upstream side of dam:

Earth Pressure Coefficients,
At Rest Condition

Angle of upstream embankment slope $\beta_{soil} := 0°$

$$K_{0fillus} := \left(1 - \sin\left(\phi_{fillus}\right)\right)\cdot\left(1 + \sin\left(\beta_{soil}\right)\right) = 1.00$$

Horizontal Force due to upstream fill

$$F_{fillus.x} := \frac{1}{2}\cdot K_{0fillus}\cdot\left(\gamma_{fillus} - \gamma_{w}\right)\cdot\left(EL_{fillus} - EL_{b}\right)^{2}\cdot LF = 0\ \text{kip}$$

Moment arm of fill

$$arm_{fillus.y} := \left(\frac{1}{3}\right)\left(EL_{fillus} - EL_{b}\right) = 0\ \text{ft}$$

Moment due to fill

$$M_{fillus.x} := F_{fillus.x}\cdot arm_{fillus.y} = 0\ \text{ft}\cdot\text{kip}$$

D. Soil horizontal loading on downstream side of dam:

No horizontail fill load applied on downsitream side

▸ N/A Fields

▸ (Summary of Vertical Forces Raw Data - Click to expand)

III. SUMMARY OF INITIAL LOADS AND MOMENTS - CASE #1

SUMMARY OF VERTICAL FORCES/MOMENTS

Component	Acting			Resisting		
	Force (kip)	Arm (ft)	Moment (kip-ft)	Force (kip)	Arm (ft)	Moment (kip-ft)
Weight of Concrete	-	-	-	-25.2	2.4	-61.2
Uplift Pressure #1	0.0	0.0	0.0	-	-	-
Uplift Pressure #2	8.0	2.8	21.9	-	-	-
Totals	**8.0**	**-**	**21.9**	**-25.2**	**-**	**-61.2**

Σ **Vertical Forces w/ uplift** $FV_{tot} := (F_{vr} + F_{va}) \cdot kips$ $FV_{tot} = -17.221 \cdot kips$

Σ **Vertical Moments w/ uplift** $MV_{tot} := (M_{vr} + M_{va}) \cdot kips \cdot ft$ $MV_{tot} = -39.3 \cdot kips \cdot ft$

(Summary of Horizontal Forces Raw Data - Click to expand)

SUMMARY OF HORIZONTAL FORCES/MOMENTS

Component	Acting			Resisting		
	Force (kip)	Arm (ft)	Moment (kip-ft)	Force (kip)	Arm (ft)	Moment (kip-ft)
Headwater on Dam, Fuwa.x	0.0	0.0	0.0	-	-	-
Headwater on Dam, Fuwb.x	7.5	5.2	38.7	-	-	-
Tailwater on Dam, Fdw.x	-	-	-	0.0	0.0	0.0
Totals	**7.5**	**-**	**38.7**	**0.0**	**-**	**0.0**

Σ **Horizontal Forces** $FH_{tot} := (F_{hr} + F_{ha}) \cdot kips$ $FH_{tot} = 7.496 \cdot kips$

Σ **Horizontal Moments** $MH_{tot} := (M_{hr} + M_{ha}) \cdot kips \cdot ft$ $MH_{tot} = 38.7 \cdot kips \cdot ft$

Σ **Moments (w/ uplift)** $M_{tot} := (M_{ha} + M_{hr} + M_{va} + M_{vr}) \cdot kips \cdot ft$ $M_{tot} = -0.6 \cdot kips \cdot ft$

IV. EVALUATE OVERTURNING AND BASE PRESSURES

- Check Resultant Location - COE EM1110-2-2200 Overturning Stability Criteria

- **Usual Conditions = Within middle third of the base**
- Unusual Conditions = Within middle half of the base
- Extreme Conditions = Within base

A. Calculate Eccentricity and Base Pressures

- Resultant and Eccentricity

- Eccentricity (from centroid of Base Area)

$$e_o := \frac{M_{tot}}{-FV_{tot}} = -0.033\,ft$$

(+) = D/S of Centroid
(-) = U/S of Centroid

- Resultant Location (from toe)

$$R_o := \frac{1}{2}B - e_o = 8.283\,ft$$

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- Evaluate Overturning using Resultant Location

$$\text{Location}_{Ro_1} := \begin{cases} \text{"WITHIN MIDDLE 1/3"} & \text{if } \left(R_o \geq \frac{B}{3}\right) \wedge \left(R_o \leq \frac{2B}{3}\right) \\ \text{"OUTSIDE MIDDLE 1/3"} & \text{otherwise} \end{cases}$$

$\text{Location}_{Ro_1} = \text{"WITHIN MIDDLE 1/3"}$

- Base Pressures (includes Uplift)

Base Pressure at Heel:

$$P_{us\_o} := \frac{-FV_{tot}}{B \cdot LF} \cdot \left(1 - \frac{6 \cdot e_o}{B}\right) = 1.056\,\text{ksf}$$

(-) = tension
(+) = compression

Base Pressure at Toe:

$$P_{ds\_o} := \frac{-FV_{tot}}{B \cdot LF} \cdot \left(1 + \frac{6 \cdot e_o}{B}\right) = 1.031\,\text{ksf}$$

(-) = tension
(+) = compression

C. Check if Cracked Base Analysis (CBA) is Required

(CBA is required when base pressure with uplift are negative, i.e. base in tension)

$$\text{CBA}_1 := \begin{cases} \text{"REQUIRED"} & \text{if } \left(P_{us\_o} < 0\right) \vee \left(P_{ds\_o} < 0\right) \\ \text{"NOT REQUIRED"} & \text{otherwise} \end{cases}$$

$\text{CBA}_1 = \text{"NOT REQUIRED"}$

(Note: if CBA not needed, do not edit Cracked Base Analysis Region

Cracked Base Analysis - Case #1 (Click to Expand, if Required)

Summary of Vertical forces - Click to expand

Revised summary table - Click to expand

Revised Parameters (Click to Expand)

Evaluate Sliding Stability (shear friction factor):

Base Inclination Angle $\beta = 0$

Base Area for Analysis $A_0 := B_{unc} \cdot LF = 16.5\,ft^2$

With no cohesion

$$FS\_SLIDING_1 := \frac{\left(-F_V \cdot \cos(\beta) - F_U + F_H \cdot \sin(\beta) \cdot f_\beta\right) \cdot \tan\left(\phi_{dam}\right) + \left(A_0 - A_{shear}\right) \cdot c_0 + A_{shear} \cdot c_{shear}}{F_H \cdot \cos(\beta) - F_V \cdot \sin(\beta) \cdot f_\beta} = 1.928$$

Calculated $FS_{sliding}$ $FS\_SLIDING_1 = 1.9$

Required Factor of Safety (from Page 5): $FS\_SLIDING_{min_1} = 2.0$

$$\text{Sliding\_Stability} := \begin{cases} \text{"SLIDING OK"} & \text{if } FS\_SLIDING_1 \geq FS\_SLIDING_{min_1} \\ \text{"NOT ADEQUATE"} & \text{otherwise} \end{cases}$$

$\text{Sliding\_Stability} = \text{"NOT ADEQUATE"}$

Calculate base cohesion necessary to achieve FS = 2.0

$$c_{req_1} := \frac{\left[2.0\cdot\left(F_H\cdot\cos(\beta) - F_V\cdot\sin(\beta)\cdot f_\beta\right)\right] - \left(-F_V\cdot\cos(\beta) - F_U + F_H\cdot\sin(\beta)\cdot f_\beta\right)\cdot\tan\left(\phi_{dam}\right) - A_{shear}\cdot c_{shear}}{A_0 - A_{shear}} = 0.2\cdot psi$$

Re-Evaluate Overturning and Base Pressures

Final Resultant Location:

$$R_{rev} := \frac{1}{2}\cdot B_{unc} - e_{rev} = 8.283\,ft$$

(+) = U/S of toe
(-) = D/S of toe

$$Location_{Rrev_1} := \begin{cases} \text{"OK"} & \text{if } \left(R_{rev} \ge \frac{B_{unc}}{3}\right) \wedge \left(R_{rev} \le \frac{2B_{unc}}{3}\right) \\ \text{"NG"} & \text{otherwise} \end{cases}$$

$$Location_{Rrev_1} = \text{"OK"}$$

- Base Pressure at Toe (includes Uplift)

$$p_{toe_1} := \begin{cases} P_{ds\_o} & \text{if } CBA_1 = \text{"NOT REQUIRED"} \\ \frac{-\left(F_V + F_U\right)}{B_{unc}\cdot LF}\cdot\left[1 + 6\cdot\left(\frac{e_{rev}}{B_{unc}}\right)\right] & \text{otherwise} \end{cases}$$

$$p_{toe_1} = 1.0\cdot ksf$$

Check to see if pressure exceeds foundation bearing capacity:

$$BC_1 := \begin{cases} \text{"OK"} & \text{if } p_{toe_1} < BC \\ \text{"EXCEEDED"} & \text{otherwise} \end{cases}$$

$$BC_1 = \text{"OK"}$$

Check to see if pressure exceeds dam compressive strength:

$$Cu\_dam_1 := \begin{cases} \text{"OK"} & \text{if } p_{toe_1} < 0.3\cdot Cu_{dam} \\ \text{"EXCEEDED"} & \text{otherwise} \end{cases}$$

$$Cu\_dam_1 = \text{"OK"}$$

Check flotation:

$$FS\_FLOT_1 := \frac{-F_V}{F_U} = 3.158$$

Required Factor of Safety against Flotation (from USACE EM 1110-2-2100, Table 3-4):

$$FS\_FLOT_{min_1} := 1.3$$

$$Flotation_1 := \begin{cases} \text{"OK"} & \text{if } FS\_FLOT_1 \ge FS\_FLOT_{min_1} \\ \text{"NOT ADEQUATE"} & \text{otherwise} \end{cases}$$

$$Flotation_1 = \text{"OK"}$$

END OF LOAD CASE #1 ANALYSIS

LOAD CASE #2 - Flood Pool

I. INITIAL VERTICAL LOADS AND MOMENTS:

A. Dam self-weights - (Same as Load Case #1)

B. Headwater: (Vertical Component)

Conditional statement using variable H_water_over_crest
(*determines whether headwater higher than dam crest)*

$$H_{water\_over\_crest} := H_{uw_2} - H_{dam} = 2\ ft$$

Height of water above crest

$$H_{ov}(H_{water\_over\_crest}) := \begin{cases} H_{water\_over\_crest} & \text{if } H_{water\_over\_crest} > 0 \cdot ft \\ 0 & \text{otherwise} \end{cases}$$

$$H_{ov}(H_{water\_over\_crest}) = 2\ ft$$

Headwater weight on dam (if crest not overtopped, areas of a, b, and c = 0)
a, b, c correspond to various areas of water over the dam, refer to FBD

$F_{uwb.y}$ dimensions:

$X_{Centroid.uwb.y} := 0ft$ $\quad Y_{Centroid.uwb.y} := 0ft$

$Area_{uwb.y} := 0ft^2$

$F_{uwc.y}$ dimensions:

$X_{Centroid.uwc.y} := 15ft$ $\quad Y_{Centroid.uwc.y} := 16.66ft$

$Area_{uwc.y} := 4.47ft^2$

Headwater weight on dam

Headwater Area A $\quad F_{uwa.y.f} := -Area_{uwa.y} \cdot \gamma_w \cdot LF = 0$ $\quad$ Above heel

Headwater Area B $\quad F_{uwb.y.f} := -Area_{uwb.y} \cdot \gamma_w \cdot LF = 0$ $\quad$ Overtopping crest

Headwater Area C $\quad F_{uwc.y.f} := -Area_{uwc.y} \cdot \gamma_w \cdot LF = -0.279\ kip$

Σ Vertical components of Headwater over the Upstream face of the Dam

$$F_{uw.y.f} := F_{uwa.y.f} + F_{uwb.y.f} + F_{uwc.y.f} = -0.279\ kip$$

Moment arms of a, b & c about the Center of Base

Headwater Area A $\quad arm_{uwa.x.f} := X_{Centroid.uwa.y} - \frac{1}{2}B = -8.25\ ft$ $\quad$ Above heel

Headwater Area B $arm_{uwb.x.f} := X_{Centroid.uwb.y} - \frac{1}{2}B = -8.25\,ft$ Overtopping crest

Headwater Area C $arm_{uwc.x.f} := X_{Centroid.uwc.y} - \frac{1}{2}B = 6.75\,ft$

Moments of vertical headwater forces a, b & c forces about centerline:

Headwater Area A $M_{uwa.y.f} := F_{uwa.y.f} \cdot arm_{uwa.x.f} = 0 \cdot kip \cdot ft$

Headwater Area B $M_{uwb.y.f} := F_{uwb.y.f} \cdot arm_{uwb.x.f} = 0 \cdot kip \cdot ft$

Headwater Area C $M_{uwc.y.f} := F_{uwc.y.f} \cdot arm_{uwc.x.f} = -1.883 \cdot kip \cdot ft$

Σ Moments from the vertical component of Headwater on U/S face of the Dam

$$M_{uw.y.f} := M_{uwa.y.f} + M_{uwb.y.f} + M_{uwc.y.f} = -1.883 \cdot kip \cdot ft$$

C. Tailwater: (Vertical Component)

Tailwater load $F_{dw.y.f}$ dimensions:

$X_{Centroid.dw.y.f} := 0ft$ $Y_{Centroid.dw.y.f} := 0ft$

$Area_{dw.y.f} := 0ft^2$

Vertical component of Tailwater $F_{dw.y.f} := -0.6 Area_{dw.y.f} \cdot \gamma_w \cdot LF = 0 \cdot kip$ *(Use 60% of tailwater force per USACE EM 1110-2-2200, Section 3-3.c.(3).(b))*

Moment arm of Tailwater weight about centerline of Base $arm_{dw.x.f} := X_{Centroid.dw.y.f} - \frac{B}{2} = -8.25\,ft$

Moment due to Weight of Tailwater about centerline of Base $M_{dw.y.f} := F_{dw.y.f} \cdot \left(arm_{dw.x.f}\right) = 0 \cdot kip \cdot ft$

D. Soil Loads: (Vertical Components) **-** Same as Load Case #1

E. Uplift Pressure:

Height of drainage gallery to plane of analysis $H_{dg} := EL_{dg} - EL_b = 0\,ft$

Effective Hydraulic Head at Drainage Gallery, H_{dr}

$$H_{dr} := \begin{cases} \left(1 - E_{dr}\right) \cdot \left[\left(H_{uw_2} - H_{dw_2}\right) \frac{B - d_{dr}}{B} + H_{dw_2} - H_{dg}\right] + H_{dg} & \text{if } H_{dg} \geq H_{dw_2} \\ \left(1 - E_{dr}\right) \cdot \left(H_{uw_2} - H_{dw_2}\right) \cdot \frac{B - d_{dr}}{B} + H_{dw_2} & \text{otherwise} \end{cases} = 18 \cdot ft$$

based on drain effectiveness $E_{dr} = 0 \cdot \%$

Head at heel $H_{heel} := H_{uw_2} = 18\,ft$

Head at toe $H_{toe} := H_{dw_2} = 0\,ft$

Uplift Pressure at Heel $u_{up\_us} := H_{heel} \cdot \gamma_w = 1.123 \cdot ksf$

Uplift Pressure at Toe $u_{up\_ds} := H_{toe} \cdot \gamma_w = 0 \cdot ksf$

Uplift Pressure below Drainage Gallery $u_{up\_dg} := H_{dr} \cdot \gamma_w = 1.123 \cdot ksf$

Uplift forces below dam base:
Refer to FBD for notation:

$U_1 := u_{up\_ds} \cdot B \cdot LF = 0 \cdot kip$

$U_2 := \frac{1}{2}\left(u_{up\_dg} - u_{up\_ds}\right) \cdot B \cdot LF = 9.266 \cdot kip$

Σ **Uplift Forces =** $U := U_1 + U_2 = 9.266 \cdot kip$

Moment arms of Uplift Forces

$d_{up1} := \frac{1}{2}B - \frac{1}{2}B = 0\,ft$

$d_{up2} := \frac{2}{3}B - \frac{1}{2}B = 2.75\,ft$

Moments due to Uplift Components

$M_{up1} := U_1 \cdot d_{up1} = 0 \cdot kip \cdot ft$

$M_{up2} := U_2 \cdot d_{up2} = 25.483 \cdot kip \cdot ft$

Σ **Uplift Moments =** $M_{up} := M_{up1} + M_{up2} = 25.483 \cdot kip \cdot ft$

II. INITIAL HORIZONTAL FORCES AND MOMENTS:

A. Headwater: (Horizontal Component)

Horizontal Component of Headwater on Dam

$$F_{uwa.x} := \begin{cases} \gamma_w \cdot \left(H_{uw_2} - H_{dam}\right) \cdot H_{dam} \cdot LF & \text{if } H_{ov}\left(H_{water\_over\_crest}\right) > 0 \\ 0 & \text{otherwise} \end{cases} = 1.997 \cdot kips$$

$$F_{uwb.x} := \begin{cases} \frac{1}{2}\gamma_w \cdot H_{dam}^{\ 2} \cdot LF & \text{if } H_{ov}\left(H_{water\_over\_crest}\right) > 0 \\ \frac{1}{2}\gamma_w \cdot \left(H_{dam}\right)^2 \cdot LF & \text{otherwise} \end{cases} = 7.987 \cdot kips$$

Σ **Horizontal Forces by Headwater** $F_{uw.x} := F_{uwa.x} + F_{uwb.x} = 9.984\,kip$

Moment arms of Headwater on Dam

$$arm_{uwa.y} := \begin{cases} \frac{H_{dam}}{2} & \text{if } H_{ov}\left(H_{water\_over\_crest}\right) > 0 \\ 0 & \text{otherwise} \end{cases} = 8 \cdot ft$$

$$arm_{uwb.y} := \begin{cases} \frac{1}{3}H_{dam} & \text{if } H_{ov}(H_{water\_over\_crest}) > 0 \\ \frac{1}{3}H_{uw_2} & \text{otherwise} \end{cases} = 5.333 \cdot ft$$

Moment of Headwater on Dam

$$M_{uwa.x} := F_{uwa.x} \cdot arm_{uwa.y} = 15.974 \cdot kip \cdot ft$$

$$M_{uwb.x} := F_{uwb.x} \cdot arm_{uwb.y} = 42.598 \cdot kip \cdot ft$$

Σ Moments due to Headwater

$$M_{uw.x} := M_{uwa.x} + M_{uwb.x} = 58.573 \cdot kip \cdot ft$$

B. Tailwater: (Horizontal Component)

Horizontal Force due to Tailwater

$$F_{dw.x.f} := \frac{-1}{2} 0.6 \gamma_w \cdot \left(H_{dw_2}\right)^2 \cdot LF = 0\,kip$$

(Use 60% of tailwater force per USACE EM 1110-2-2200, Section 3-3.c.(3).(b))

Moment arm of Tailwater

$$arm_{dw.y.f} := \frac{1}{3} H_{dw_2} = 0\,ft$$

Moment due to Tailwater

$$M_{dw.x.f} := F_{dw.x.f} \cdot arm_{dw.y.f} = 0\,ft \cdot kip$$

C. Soil horizontal loading on upstream side of dam:- Same as Load Case #1

D. Soil horizontal loading on downstream side of dam: - Same as Load Case #1

(Summary of Vertical Forces Raw Data - Click to expand)

III. SUMMARY OF INITIAL LOADS AND MOMENTS - CASE #2

SUMMARY OF VERTICAL FORCES/MOMENTS

Component	Acting			Resisting		
	Force (kip)	Arm (ft)	Moment (kip-ft)	Force (kip)	Arm (ft)	Moment (kip-ft)
Weight of Concrete	-	-	-	-25.2	2.4	-61.2
Headwater over Dam, Fuwa.y	-	-	-	0.0	-8.3	0.0
Headwater over Dam, Fuwb.y	-	-	-	0.0	-8.3	0.0
Headwater over Dam, Fuwc.y	-	-	-	-0.3	6.8	-1.9
Uplift Pressure, Area #1	0.0	0.0	0.0			
Uplift Pressure, Area #2	9.3	2.8	25.5			
Totals	**9.3**	**-**	**25.5**	**-25.5**	**-**	**-63.1**

Σ **Vertical Forces w/ uplift** $FV_{tot} := (F_{vr} + F_{va}) \cdot kips$ $FV_{tot} = -16.213 \cdot kips$

Σ **Vertical Moments w/ uplift** $MV_{tot} := (M_{vr} + M_{va}) \cdot kips \cdot ft$ $MV_{tot} = -37.6 \cdot kips \cdot ft$

(Summary of Horizontal Forces Raw Data - Click to expand)

SUMMARY OF HORIZONTAL FORCES/MOMENTS

Component	Acting			Resisting		
	Force (kip)	Arm (ft)	Moment (kip-ft)	Force (kip)	Arm (ft)	Moment (kip-ft)
Headwater on Dam, Fuwa.x	2.0	8.0	16.0	-	-	-
Headwater on Dam, Fuwb.x	8.0	5.3	42.6	-	-	-
Tailwater on Dam, Fdw.x	-	-	-	0.0	0.0	0.0
Totals	**10.0**	**-**	**58.6**	**0.0**	**-**	**0.0**

Σ **Horizontal Forces** $FH_{tot} := (F_{hr} + F_{ha}) \cdot kips$ $FH_{tot} = 9.984 \cdot kips$

Σ **Horizontal Moments** $MH_{tot} := (M_{hr} + M_{ha}) \cdot kips \cdot ft$ $MH_{tot} = 58.6 \cdot kips \cdot ft$

Σ **Moments (w/ uplift)** $M_{tot} := (M_{ha} + M_{hr} + M_{va} + M_{vr}) \cdot kips \cdot ft$ $M_{tot} = 20.9 \cdot kips \cdot ft$

IV. EVALUATE OVERTURNING AND BASE PRESSURES

- Check Resultant Location - COE EM1110-2-2200 Overturning Stability Criteria

- Usual Conditions = Within middle third of the base
- Unusual Conditions = Within middle half of the base
- **Extreme Conditions = Within base**

A. Calculate Eccentricity and Base Pressures

- Resultant and Eccentricity

- Eccentricity (from centroid of Base Area)

$$e_o := \frac{M_{tot}}{-FV_{tot}} = 1.291\,ft$$

(+) = D/S of Centroid
(-) = U/S of Centroid

- Resultant Location (from toe)

$$R_o := \frac{1}{2}B - e_o = 6.959\,ft$$

- Evaluate Overturning using Resultant Location

$$Location_{Ro_2} := \begin{cases} \text{"WITHIN BASE"} & \text{if } (R_o \geq 0) \wedge (R_o \leq B) \\ \text{"OUTSIDE BASE"} & \text{otherwise} \end{cases}$$

$Location_{Ro_2}$ = "WITHIN BASE"

- Base Pressures (includes Uplift)

Base Pressure at Heel:

$$P_{us\_o} := \frac{-FV_{tot}}{B \cdot LF} \cdot \left(1 - \frac{6 \cdot e_o}{B}\right) = 0.521\,ksf$$

(-) = tension
(+) = compression

Base Pressure at Toe:

$$P_{ds\_o} := \frac{-FV_{tot}}{B \cdot LF} \cdot \left(1 + \frac{6 \cdot e_o}{B}\right) = 1.444\,ksf$$

(-) = tension
(+) = compression

C. Check if Cracked Base Analysis (CBA) is Required

(CBA is required when base pressure with uplift are negative, i.e. base in tension)

$$CBA_2 := \begin{cases} \text{"REQUIRED"} & \text{if } (P_{us\_o} < 0) \vee (P_{ds\_o} < 0) \\ \text{"NOT REQUIRED"} & \text{otherwise} \end{cases}$$

CBA_2 = "NOT REQUIRED"

(Note: if CBA not needed, skip cracked base analysis section and move on to Factor of Safety calculation)

▶ Cracked Base Analysis - Case #2 (Click to Expand, if Required)

▶ Summary of Vertical Forces - Click to Expand

▶ Revised Summary Table - Click to Expand

▶ Revised Parameters (Click to Expand)

Evaluate Sliding Stability (shear friction factor):

Base Inclination Angle $\beta = 0$

Base Area for Analysis $A_0 := B_{unc} \cdot LF = 16.5\,ft^2$

With no cohesion

$$FS\_SLIDING_2 := \frac{(-F_V \cdot \cos(\beta) - F_U + F_H \cdot \sin(\beta) \cdot f_\beta) \cdot \tan(\phi_{dam}) + (A_0 - A_{shear}) \cdot c_0 + A_{shear} \cdot c_{shear}}{F_H \cdot \cos(\beta) - F_V \cdot \sin(\beta) \cdot f_\beta} = 1.363$$

Calculated $FS_{sliding}$ $FS\_SLIDING_2 = 1.4$

Required Factor of Safety (from Page 5): $FS\_SLIDING_{min_2} = 1.1$

$$Sliding\_Stability := \begin{cases} \text{"SLIDING OK"} & \text{if } FS\_SLIDING_2 \geq FS\_SLIDING_{min_2} \\ \text{"NOT ADEQUATE"} & \text{otherwise} \end{cases}$$

$Sliding\_Stability$ = "SLIDING OK"

Re-Evaluate Overturning and Base Pressures

Final Resultant Location:

$$R_{rev} := \frac{1}{2} \cdot B_{unc} - e_{rev} = 6.959\,ft$$

(+) = U/S of toe
(-) = D/S of toe

$$\textbf{Location}_{\textbf{Rrev}_2} := \begin{cases} \text{"OK"} & \text{if } (R_{rev} \geq 0) \wedge (R_{rev} \leq B_{unc}) \\ \text{"NG"} & \text{otherwise} \end{cases}$$

$$\textbf{Location}_{\textbf{Rrev}_2} = \text{"OK"}$$

- Base Pressure at Toe (includes Uplift)

$$\textbf{p}_{\textbf{toe}_2} := \begin{cases} P_{ds\_o} & \text{if } \textbf{CBA}_2 = \text{"NOT REQUIRED"} \\ \frac{-(F_V + F_U)}{B_{unc} \cdot LF} \cdot \left[1 + 6 \cdot \left(\frac{e_{rev}}{B_{unc}}\right)\right] & \text{otherwise} \end{cases}$$

$$\textbf{p}_{\textbf{toe}_2} = 1.4 \cdot ksf$$

Check to see if pressure exceeds foundation bearing capacity:

$$\textbf{BC}_2 := \begin{cases} \text{"OK"} & \text{if } \textbf{p}_{\textbf{toe}_2} < 1.33BC \\ \text{"EXCEEDED"} & \text{otherwise} \end{cases}$$

$$\textbf{BC}_2 = \text{"OK"}$$

Check to see if pressure exceeds dam compressive strength:

$$\textbf{Cu\_dam}_2 := \begin{cases} \text{"OK"} & \text{if } \textbf{p}_{\textbf{toe}_2} < 0.9 \cdot Cu_{dam} \\ \text{"EXCEEDED"} & \text{otherwise} \end{cases}$$

$$\textbf{Cu\_dam}_2 = \text{"OK"}$$

Check flotation:

$$FS\_FLOT_2 := \frac{-F_V}{F_U} = 2.75$$

Required Factor of Safety against Flotation (from USACE EM 1110-2-2100, Table 3-4):

$$FS\_FLOT_{min_2} := 1.1$$

$$\textbf{Flotation}_2 := \begin{cases} \text{"OK"} & \text{if } FS\_FLOT_2 \geq FS\_FLOT_{min_2} \\ \text{"NOT ADEQUATE"} & \text{otherwise} \end{cases}$$

$$\textbf{Flotation}_2 = \text{"OK"}$$

END OF LOAD CASE #2 ANALYSIS

LOAD CASE #3 - Normal Pool + Ice

I. INITIAL VERTICAL LOADS AND MOMENTS:

A. Dam self-weights - Same as Load Case #1

B. Headwater: (Vertical Component) - Same as Load Case #1

Conditional statement using variable H_water_over_crest
(*determines whether headwater higher than dam crest)*

$$H_{water\_over\_crest} := H_{uw_3} - H_{dam} = -0.5\,ft$$

Height of water above crest

$$H_{ov}(H_{water\_over\_crest}) := \begin{cases} H_{water\_over\_crest} & \text{if } H_{water\_over\_crest} > 0 \cdot ft \\ 0 & \text{otherwise} \end{cases}$$

$$H_{ov}(H_{water\_over\_crest}) = 0\,ft$$

C. Tailwater: (Vertical Component) - Same as Load Case #1

D. Soil Loads: (Vertical Components) - Same as Load Case #1

E. Uplift Pressure:

Height of drainage gallery to plane of analysis

$$H_{dg} := EL_{dg} - EL_b = 0\,ft$$

Effective Hydraulic Head at Drainage Gallery, H_{dr}

$$H_{dr} := \begin{cases} (1 - E_{dr}) \cdot \left[(H_{uw_3} - H_{dw_3}) \dfrac{B - d_{dr}}{B} + H_{dw_3} - H_{dg} \right] + H_{dg} & \text{if } H_{dg} \ge H_{dw_3} \\ (1 - E_{dr}) \cdot (H_{uw_3} - H_{dw_3}) \cdot \dfrac{B - d_{dr}}{B} + H_{dw_3} & \text{otherwise} \end{cases} = 15.5 \cdot ft$$

based on drain effectiveness $E_{dr} = 0 \cdot \%$

Head at heel $H_{heel} := H_{uw_3} = 15.5\,ft$

Head at toe $H_{toe} := H_{dw_3} = 0\,ft$

Uplift Pressure at Heel $u_{up\_us} := H_{uw_3} \cdot \gamma_w = 0.967 \cdot ksf$

Uplift Pressure at Toe $u_{up\_ds} := H_{toe} \cdot \gamma_w = 0 \cdot ksf$

Uplift Pressure below Drainage Gallery $u_{up\_dg} := H_{dr} \cdot \gamma_w = 0.967 \cdot ksf$

Uplift forces below dam base:
Refer to FBD for notation:

$$U_1 := u_{up\_ds} \cdot B \cdot LF = 0 \cdot kip$$

$$U_2 := \frac{1}{2}(u_{up\_dg} - u_{up\_ds}) \cdot B \cdot LF = 7.979 \cdot kip$$

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Engineers and Scientists

Σ Uplift Forces = $U := U_1 + U_2 = 7.979 \cdot kip$

Moment arms of Uplift Forces

$$d_{up1} := \frac{1}{2}B - \frac{1}{2}B = 0\ ft$$

$$d_{up2} := \frac{2}{3}B - \frac{1}{2}B = 2.75\ ft$$

Moments due to Uplift Components

$$M_{up1} := U_1 \cdot d_{up1} = 0 \cdot kip \cdot ft$$

$$M_{up2} := U_2 \cdot d_{up2} = 21.943 \cdot kip \cdot ft$$

Σ Uplift Moments = $M_{up} := M_{up1} + M_{up2} = 21.943 \cdot kip \cdot ft$

II. INITIAL HORIZONTAL FORCES AND MOMENTS:

A. Headwater: (Horizontal Component)

Horizontal Component of Headwater on Dam

$$F_{uwa.x} := \begin{cases} \gamma_w \cdot \left(H_{uw_3} - H_{dam}\right) \cdot H_{dam} \cdot LF & \text{if } H_{ov}\left(H_{water\_over\_crest}\right) > 0 \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kips$$

$$F_{uwb.x} := \begin{cases} \frac{1}{2}\gamma_w \cdot H_{dam}^{\ 2} \cdot LF & \text{if } H_{ov}\left(H_{water\_over\_crest}\right) > 0 \\ \frac{1}{2}\gamma_w \cdot \left(H_{uw_3}\right)^2 \cdot LF & \text{otherwise} \end{cases} = 7.496 \cdot kips$$

Σ Horizontal Forces by Headwater $F_{uw.x} := F_{uwa.x} + F_{uwb.x} = 7.496\ kip$

Moment arms of Headwater on Dam

$$arm_{uwa.y} := \begin{cases} \frac{H_{dam}}{2} & \text{if } H_{ov}\left(H_{water\_over\_crest}\right) > 0 \\ 0 & \text{otherwise} \end{cases} = 0 \cdot ft$$

$$arm_{uwb.y} := \begin{cases} \frac{1}{3}H_{dam} & \text{if } H_{ov}\left(H_{water\_over\_crest}\right) > 0 \\ \frac{1}{3}H_{uw_3} & \text{otherwise} \end{cases} = 5.167 \cdot ft$$

Moment of Headwater on Dam

$$M_{uwa.x} := F_{uwa.x} \cdot arm_{uwa.y} = 0 \cdot kip \cdot ft$$

$$M_{uwb.x} := F_{uwb.x} \cdot arm_{uwb.y} = 38.728 \cdot kip \cdot ft$$

Σ Moments due to Headwater $M_{uw.x} := M_{uwa.x} + M_{uwb.x} = 38.728 \cdot kip \cdot ft$

B. Tailwater: (Horizontal Component) - Same as Load Case #1

C. Soil horizontal loading on upstream side of dam: - Same as Load Case #1

D. Soil horizontal loading on downstream side of dam: - Same as Load Case #1

E. Ice Loading

Horizontal Ice Force $F_{ice.x} := H_{ice} \cdot (f_{ice}) \cdot LF = 5\,kip$

Moment Arm of Ice Force $arm_{ice.y} := (H_{uw_3} - 0.5 \cdot H_{ice}) = 15\,ft$

Moment due to Ice Force $M_{ice.x} := F_{ice.x} \cdot arm_{ice.y} = 75\,ft \cdot kip$

(Summary of Vertical Forces Raw Data - Click to expand)

III. SUMMARY OF INITIAL LOADS AND MOMENTS - CASE #3

SUMMARY OF VERTICAL FORCES/MOMENTS

Component	Acting			Resisting		
	Force (kip)	Arm (ft)	Moment (kip-ft)	Force (kip)	Arm (ft)	Moment (kip-ft)
Weight of Concrete	-	-	-	-25.2	2.4	-61.2
Uplift Pressure #1	0.0	0.0	0.0			
Uplift Pressure #2	8.0	2.8	21.9	-	-	-
Totals	**8.0**	**-**	**21.9**	**-25.2**	**-**	**-61.2**

Σ Vertical Forces w/ uplift $FV_{tot} := (F_{vr} + F_{va}) \cdot kips$ $FV_{tot} = -17.221 \cdot kips$

Σ Vertical Moments w/ uplift $MV_{tot} := (M_{vr} + M_{va}) \cdot kips \cdot ft$ $MV_{tot} = -39.3 \cdot kips \cdot ft$

(Summary of Horizontal Forces Raw Data - Click to expand)

SUMMARY OF HORIZONTAL FORCES/MOMENTS

Component	Acting			Resisting		
	Force (kip)	Arm (ft)	Moment (kip-ft)	Force (kip)	Arm (ft)	Moment (kip-ft)
Headwater on Dam, Fuwa.x	0.0	0.0	0.0	-	-	-
Headwater on Dam, Fuwb.x	7.5	5.2	38.7	-	-	-
Ice Force	5.0	15.0	75.0	-	-	-
Tailwater on Dam, Fdw.x	-	-	-	0.0	0.0	0.0
Totals	**12.5**	**-**	**113.7**	**0.0**	**-**	**0.0**

Σ Horizontal Forces $FH_{tot} := (F_{hr} + F_{ha}) \cdot kips$ $FH_{tot} = 12.496 \cdot kips$

Σ Horizontal Moments $MH_{tot} := (M_{hr} + M_{ha}) \cdot kips \cdot ft$ $MH_{tot} = 113.7 \cdot kips \cdot ft$

Σ Moments (w/ uplift) $M_{tot} := (M_{ha} + M_{hr} + M_{va} + M_{vr}) \cdot kips \cdot ft$ $M_{tot} = 74.4 \cdot kips \cdot ft$

IV. EVALUATE OVERTURNING AND BASE PRESSURES

- Check Resultant Location - COE EM1110-2-2200 Overturning Stability Criteria

- Usual Conditions = Within middle third of the base
- **Unusual Conditions = Within middle half of the base**
- Extreme Conditions = Within base

A. Calculate Eccentricity and Base Pressures

- Resultant and Eccentricity

- Eccentricity
(from centroid of Base Area)

$$e_o := \frac{M_{tot}}{-FV_{tot}} = 4.322\,ft$$

(+) = D/S of Centroid
(-) = U/S of Centroid

- Resultant Location (from toe)

$$R_o := \frac{1}{2} B - e_o = 3.928\,ft$$

- Evaluate Overturning using Resultant Location

$$\text{Location}_{Ro_3} := \begin{cases} \text{"WITHIN MIDDLE 1/2"} & \text{if } \left(R_o \geq \frac{B}{4}\right) \wedge \left(R_o \leq \frac{3}{4}B\right) \\ \text{"OUTSIDE MIDDLE 1/2"} & \text{otherwise} \end{cases}$$

$$\text{Location}_{Ro_3} = \text{"OUTSIDE MIDDLE 1/2"}$$

- Base Pressures (includes Uplift)

Base Pressure at Heel:

$$P_{us\_o} := \frac{-FV_{tot}}{B \cdot LF} \cdot \left(1 - \frac{6 \cdot e_o}{B}\right) = -0.597\,\text{ksf}$$

(-) = tension
(+) = compression

Base Pressure at Toe:

$$P_{ds\_o} := \frac{-FV_{tot}}{B \cdot LF} \cdot \left(1 + \frac{6 \cdot e_o}{B}\right) = 2.684\,\text{ksf}$$

(-) = tension
(+) = compression

C. Check if Cracked Base Analysis (CBA) is Required

(CBA is required when base pressure with uplift are negative, i.e. base in tension)

$$\text{CBA}_3 := \begin{cases} \text{"REQUIRED"} & \text{if } \left(P_{us\_o} < 0\right) \vee \left(P_{ds\_o} < 0\right) \\ \text{"NOT REQUIRED"} & \text{otherwise} \end{cases}$$

$$\text{CBA}_3 = \text{"REQUIRED"}$$

(Note: if CBA not needed, do not edit Cracked Base Analysis Region

Cracked Base Analysis - Case #3 (Click to Expand, if Required)

Cracked Base Analysis - Case #3

Estimate Initial Trial Uncracked Base Length

$$x_{0\_tr} := \frac{-B^2}{12e_o} = -5.249\,\text{ft} \qquad B_{NEW} := \frac{1}{2} \cdot B - x_{0\_tr} = 13.499\,\text{ft}$$

****** ****** ****** ****** ****** ****** ****** ****** ****** Begin Iteration ****** ****** ****** ****** ****** ****** ****** ****** ******

(Manually change uncracked length B_{tr} to achieve D <=0.5%, use B_{NEW}, above, as initial input)

Trial uncracked length: $B_{tr} := 12.55\,\text{ft}$

Corresponding cracked length: $T_{cr} := B - B_{tr} = 3.95\,\text{ft}$

Re-evaluate Vertical Moments based on CBA

Per USACE ERDC/ITL TR-00-1

A. Dam self-weights - (Refer to FBD for dam geometry)

Moment arms about Center of Uncracked Base:

$$D_{1.xcr} := X_{Centroid.c} - \frac{B_{tr}}{2} = 4.405\,\text{ft}$$

Moments of Dam Weights about Center of Uncracked Base:

$$M_{W1.ycr} := W_c \cdot D_{1.xcr} = -111.006 \cdot \text{kip} \cdot \text{ft}$$

Σ Dam Weight Moments about Center of Uncracked Base

$$M_{damCR} := M_{W1.ycr} = -111.006 \cdot kip \cdot ft$$

B. Headwater: (Vertical Component)

Moment arms of a, b & c about the Center of Uncracked Base

$$arm_{uwa.xcr} := X_{Centroid.uwa.y} - \frac{1}{2}B = -8.25\,ft$$

$$arm_{uwb.xcr} := X_{Centroid.uwb.y} - \frac{1}{2}B = -8.25\,ft$$

$$arm_{uwc.xcr} := X_{Centroid.uwc.y} - \frac{1}{2}B = 6.75\,ft$$

Moments of vertical forces from a, b & c

$$M_{uwa.ycr} := F_{uwa.y} \cdot arm_{uwa.xcr} = 0 \cdot kip \cdot ft$$

$$M_{uwb.ycr} := F_{uwb.y} \cdot arm_{uwb.xcr} = 0 \cdot kip \cdot ft$$

$$M_{uwc.ycr} := F_{uwc.y} \cdot arm_{uwc.xcr} = 0 \cdot kip \cdot ft$$

Σ Moments from the vertical component of Headwater on U/S face of the Dam

$$M_{uw.yCR} := M_{uwa.ycr} + M_{uwb.ycr} + M_{uwc.ycr} = 0 \cdot kip \cdot ft$$

C. Tailwater: (Vertical Component)

D. Soil Loads: (Vertical Component due to Weight)

E. Re-evaluate Uplift Profile based on Trial Crack Length

Trial Crack Length $T_{cr} = 3.95\,ft$

Distance from Heel to Drain Gallery $d_{dr} = 0$

Revise H_{dr_1} based on trial uncracked base length

$$H_{dr\_1} := \begin{cases} (1 - E_{dr}) \cdot \left[\left(H_{uw_1} - H_{dw_1} \right) \dfrac{B - d_{dr}}{B - T_{cr}} + H_{dw_1} - H_{dg} \right] + H_{dg} & \text{if } H_{dg} > H_{dw_1} \\ (1 - E_{dr}) \cdot \left(H_{uw_1} - H_{dw_1} \right) \cdot \dfrac{B - d_{dr}}{B - T_{cr}} + H_{dw_1} & \text{otherwise} \end{cases} = 20.378 \cdot ft$$

Revise H_{dr_1} based on relationship between crack length and distance from heel to drains

$$H_{dr\_1} := \begin{cases} H_{uw_1} & \text{if } T_{cr} > d_{dr} \\ H_{dr\_1} & \text{otherwise} \end{cases} = 15.5 \cdot ft$$

Revised Uplift Pressure at the Drainage Gallery

$$u_{up\_dg\_1} := H_{dr\_1} \cdot \gamma_w = 0.967\,\text{ksf}$$

Revised Uplift Forces Based on Trial Cracked Length:

$$U_{tr} := \begin{bmatrix} \begin{cases} u_{up\_ds} \cdot (B - d_{dr}) \cdot LF & \text{if } d_{dr} > T_{cr} \\ u_{up\_ds} \cdot B_{tr} \cdot LF & \text{otherwise} \end{cases} \\ \begin{cases} \frac{1}{2}(u_{up\_dg\_1} - u_{up\_ds}) \cdot (B - d_{dr}) \cdot LF & \text{if } d_{dr} > T_{cr} \\ \frac{1}{2}(u_{up\_us} - u_{up\_ds}) \cdot B_{tr} \cdot LF & \text{otherwise} \end{cases} \\ \begin{cases} (u_{up\_dg\_1}) \cdot (d_{dr} - T_{cr}) \cdot LF & \text{if } d_{dr} > T_{cr} \\ 0 & \text{otherwise} \end{cases} \\ \begin{cases} \frac{1}{2}(u_{up\_us} - u_{up\_dg\_1}) \cdot (d_{dr} - T_{cr}) \cdot LF & \text{if } d_{dr} > T_{cr} \\ 0 & \text{otherwise} \end{cases} \\ u_{up\_us} \cdot T_{cr} \cdot LF \end{bmatrix}$$

$$U_{tr} = \begin{pmatrix} 0 \\ 6.069 \\ 0 \\ 0 \\ 3.82 \end{pmatrix} \cdot \text{kips}$$

$$U_{TR} := \sum_{n=1}^{5} U_{tr_n} = 9.89\,\text{kip}$$

Arms of Revised Uplift Forces about Center of Uncracked Base:

$$D_{tr} := \begin{bmatrix} \begin{cases} \frac{1}{2} \cdot (B - d_{dr}) - \frac{B_{tr}}{2} & \text{if } d_{dr} > T_{cr} \\ \frac{1}{2} \cdot (B - T_{cr}) - \frac{B_{tr}}{2} & \text{otherwise} \end{cases} \\ \begin{cases} \frac{2}{3} \cdot (B - d_{dr}) - \frac{B_{tr}}{2} & \text{if } d_{dr} > T_{cr} \\ \frac{2}{3} \cdot (B - T_{cr}) - \frac{B_{tr}}{2} & \text{otherwise} \end{cases} \\ \begin{cases} \frac{B_{tr}}{2} - \frac{(d_{dr} - T_{cr})}{2} & \text{if } d_{dr} > T_{cr} \\ 0 & \text{otherwise} \end{cases} \\ \begin{cases} \frac{B_{tr}}{2} - \frac{(d_{dr} - T_{cr})}{3} & \text{if } d_{dr} > T_{cr} \\ 0 & \text{otherwise} \end{cases} \\ \frac{B_{tr}}{2} + \frac{T_{cr}}{2} \end{bmatrix}$$

$$D_{tr} = \begin{pmatrix} 0.00 \\ 2.09 \\ 0.00 \\ 0.00 \\ 8.25 \end{pmatrix} \cdot \text{ft}$$

Sum Moments using Revised Uplift Forces and Arms about Center of Base

$$M_{up\_tr} := \begin{pmatrix} U_{tr_1} \cdot D_{tr_1} \\ U_{tr_2} \cdot D_{tr_2} \\ U_{tr_3} \cdot D_{tr_3} \\ U_{tr_4} \cdot D_{tr_4} \\ U_{tr_5} \cdot D_{tr_5} \end{pmatrix} \qquad M_{up\_tr} = \begin{pmatrix} 0 \\ 12.695 \\ 0 \\ 0 \\ 31.519 \end{pmatrix} \cdot kips \cdot ft \qquad M_{up\_TR} := \sum_{n=1}^{5} \left(M_{up\_tr}\right)_n = 44.213\, ft \cdot kip$$

Evaluate the revised eccentricity using the results of the CBA

$$M_{tot\_CR} := M_{damCR} + M_{dw.ycr} + MH_{tot} = 2.722\, ft \cdot kip$$

$$M_{up\_TR} = 44.213\, ft \cdot kip$$

$$e_{tr} := \frac{M_{tot\_CR} + M_{up\_TR}}{-\left(F_{vr} \cdot kip + U_{TR}\right)} = 3.066\, ft$$

$$M_{tot\_CR} + M_{up\_TR} = 46.936\, ft \cdot kip$$

$$F_{vr} \cdot kip + U_{TR} = -15.31\, kip$$

<u>**Trial Crack Length based on Revised Eccentricity**</u>

$$x_{1\_tr} := \frac{-B_{tr}^{\,2}}{12 e_{tr}} = -4.281\, ft \qquad B_{unc} := \frac{1}{2} \cdot B - x_{1\_tr} = 12.531\, ft$$

Check for convergence between Trial Uncracked Length, B_{tr} and Recalculated Uncracked Length, $B_{uncracked}$

Difference (target<=0.5%)

$$\Delta := \frac{\left(B_{unc} - B_{tr}\right)}{B_{tr}} = -0.148 \cdot \%$$

$$Action := \begin{cases} \text{"OK"} & \text{if } \Delta \le 0.5\% \wedge \Delta \ge -0.5\% \\ \text{"CHANGE B\_tr"} & \text{otherwise} \end{cases}$$

Action = "OK"

- If "OK"

<u>Final Result of Cracked Base Analysis</u> --->

$$B_{unc} = 12.5\, ft$$

$$T_{cracked} := B - B_{unc} = 4 \cdot ft$$

Cracked Base Analysis - Case #3 (Click to Expand, if Required)

Summary of Vertical Forces - Click to Expand

Revised Summary Table

III. SUMMARY OF REVISED CRACKED BASE LOADS AND MOMENTS - CASE #3

SUMMARY OF VERTICAL FORCES/MOMENTS

Component	Acting			Resisting		
	Force (kip)	Arm (ft)	Moment (kip-ft)	Force (kip)	Arm (ft)	Moment (kip-ft)
Weight of Concrete	-	-	-	-25.2	4.4	-111.0
Uplift Pressure, Area #1	0.0	0.0	0.0	-		
Uplift Pressure, Area #2	6.1	2.1	12.7	-	-	-
Uplift Pressure, Area #3	0.0	0.0	0.0	-	-	-
Uplift Pressure, Area #4	0.0	0.0	0.0	-	-	-
Uplift Pressure, Area #5	3.8	8.3	31.5	-	-	-
Totals	**9.9**	**-**	**44.2**	**-25.2**	**-**	**-111.0**

Σ Vertical Forces w/ uplift $FV_{cb\_tot} := (F_{cb\_vr} + F_{cb\_va}) \cdot kips$ $FV_{cb\_tot} = -15.31 \cdot kips$

Σ Vertical Moments w/ uplift $MV_{cb\_tot} := (M_{cb\_vr} + M_{cb\_va}) \cdot kips \cdot ft$ $MV_{cb\_tot} = -66.8 \cdot kips \cdot ft$

SUMMARY OF HORIZONTAL FORCES/MOMENTS

Component	Acting			Resisting		
	Force (kip)	Arm (ft)	Moment (kip-ft)	Force (kip)	Arm (ft)	Moment (kip-ft)
Headwater on Dam, Fuwa.x	0.0	0.0	0.0	-	-	-
Headwater on Dam, Fuwb.x	7.5	5.2	38.7	-	-	-
Ice Force	5.0	15.0	75.0	-	-	-
Tailwater on Dam, Fdw.x	-	-	-	0.0	0.0	0.0
Totals	**12.5**	**-**	**113.7**	**0.0**	**-**	**0.0**

S Horizontal Forces $FH_{tot} := (F_{hr} + F_{ha}) \cdot kips$ $FH_{tot} = 12.496 \cdot kips$

S Horizontal Moments $MH_{tot} := (M_{hr} + M_{ha}) \cdot kips \cdot ft$ $MH_{tot} = 113.7 \cdot kips \cdot ft$

S Moments (w/ uplift) $M_{tot} := (M_{ha} + M_{hr} + M_{va} + M_{vr}) \cdot kips \cdot ft$ $M_{tot} = 74.4 \cdot kips \cdot ft$

Revised Summary Table

Revised Parameters (Click to Expand)

Evaluate Sliding Stability (shear friction factor):

Base Inclination Angle $\beta = 0$

Base Area for Analysis $A_0 := B_{unc} \cdot LF = 12.531\,ft^2$

$$FS\_SLIDING_3 := \frac{(-F_V \cdot \cos(\beta) - F_U + F_H \cdot \sin(\beta) \cdot f_\beta) \cdot \tan(\phi_{dam}) + (A_0 - A_{shear}) \cdot c_0 + A_{shear} \cdot c_{shear}}{F_H \cdot \cos(\beta) - F_V \cdot \sin(\beta) \cdot f_\beta} = 1.287$$

Calculated $FS_{sliding}$ $FS\_SLIDING_3 = 1.3$ Required Factor of Safety (from Page 5): $FS\_SLIDING_{min_3} = 2.0$

Sliding_Stability := | "SLIDING OK" if $FS\_SLIDING_3 \geq FS\_SLIDING_{min_3}$
| "NOT ADEQUATE" otherwise

Sliding_Stability = "NOT ADEQUATE"

Calculate base cohesion necessary to achieve FS = 2.0

$$c_{req_3} := \frac{\left[2.0\cdot\left(F_H\cdot\cos(\beta) - F_V\cdot\sin(\beta)\cdot f_\beta\right)\right] - \left(-F_V\cdot\cos(\beta) - F_U + F_H\cdot\sin(\beta)\cdot f_\beta\right)\cdot\tan\left(\phi_{dam}\right) - A_{shear}\cdot c_{shear}}{A_0 - A_{shear}} = 3.9\cdot psi$$

Re-Evaluate Overturning and Base Pressures

Final Resultant Location:

$$R_{rev} := \frac{1}{2}\cdot B_{unc} - e_{rev} = 3.2\,ft$$

(+) = U/S of toe
(-) = D/S of toe

Location$_{Rrev_3}$:= | "OK" if $\left(R_{rev} \geq \frac{B_{unc}}{4}\right) \wedge \left(R_{rev} \leq 3\,\frac{B_{unc}}{4}\right)$
| "NG" otherwise

Location$_{Rrev_3}$ = "OK"

- Base Pressure at Toe (includes Uplift)

p_{toe_3} := | P_{ds_o} if **CBA**$_3$ = "NOT REQUIRED"
| $\frac{-\left(F_V + F_U\right)}{B_{unc}\cdot LF}\cdot\left[1 + 6\cdot\left(\frac{e_{rev}}{B_{unc}}\right)\right]$ otherwise

p_{toe_3} = 3.0·ksf

Check to see if pressure exceeds foundation bearing capacity:

BC$_3$:= | "OK" if $p_{toe_3} < 1.33BC$
| "EXCEEDED" otherwise

BC$_3$ = "OK"

Check to see if pressure exceeds dam compressive strength:

Cu_dam$_3$:= | "OK" if $p_{toe_3} < 0.9\cdot Cu_{dam}$
| "EXCEEDED" otherwise

Cu_dam$_3$ = "OK"

Check flotation:

$$FS\_FLOT_3 := \frac{-F_V}{F_U} = 2.548$$

Required Factor of Safety against Flotation (from USACE EM 1110-2-2100, Table 3-4):

$FS\_FLOT_{min_3} := 1.1$

Flotation$_3$:= | "OK" if $FS\_FLOT_3 \geq FS\_FLOT_{min_3}$
| "NOT ADEQUATE" otherwise

Flotation$_3$ = "OK"

END OF LOAD CASE #3 ANALYSIS

LOAD CASE #4 - Normal Pool + Earthquake

I. INITIAL VERTICAL LOADS AND MOMENTS:

A. Dam self-weights - Same as Load Case #1

B. Headwater: (Vertical Component) - Same as Load Case #1

Conditional statement using variable H_water_over_crest
(*determines whether headwater higher than dam crest)*

$$H_{water\_over\_crest} := H_{uw_4} - H_{dam} = -0.5\,ft$$

Height of water above crest

$$H_{ov}(H_{water\_over\_crest}) := \begin{cases} H_{water\_over\_crest} & \text{if } H_{water\_over\_crest} > 0 \cdot ft \\ 0 & \text{otherwise} \end{cases}$$

$$H_{ov}(H_{water\_over\_crest}) = 0\,ft$$

C. Tailwater: (Vertical Component) - Same as Load Case #1

D. Soil Loads: (Vertical Components) - Same as Load Case #1

E. Uplift Pressure:

Height of drainage gallery to plane of analysis

$$H_{dg} := EL_{dg} - EL_b = 0\,ft$$

Effective Hydraulic Head at Drainage Gallery, H_{dr}

$$H_{dr} := \begin{cases} (1 - E_{dr}) \cdot \left[\left(H_{uw_4} - H_{dw_4}\right) \dfrac{B - d_{dr}}{B} + H_{dw_4} - H_{dg} \right] + H_{dg} & \text{if } H_{dg} \ge H_{dw_4} \\ (1 - E_{dr}) \cdot \left(H_{uw_4} - H_{dw_4}\right) \cdot \dfrac{B - d_{dr}}{B} + H_{dw_4} & \text{otherwise} \end{cases} = 15.5 \cdot ft$$

based on drain effectiveness $E_{dr} = 0 \cdot \%$

Head at heel $H_{heel} := H_{uw_4} = 15.5\,ft$

Head at toe $H_{toe} := H_{dw_4} = 0\,ft$

Uplift Pressure at Heel $u_{up\_us} := H_{uw_4} \cdot \gamma_w = 0.967 \cdot ksf$

Uplift Pressure at Toe $u_{up\_ds} := H_{toe} \cdot \gamma_w = 0 \cdot ksf$

Uplift Pressure below Drainage Gallery $u_{up\_dg} := H_{dr} \cdot \gamma_w = 0.967 \cdot ksf$

Uplift forces below dam base:
Refer to FBD for notation:

$$U_1 := u_{up\_ds} \cdot B \cdot LF = 0 \cdot kip$$

$$U_2 := \frac{1}{2}\left(u_{up\_dg} - u_{up\_ds}\right) \cdot B \cdot LF = 7.979 \cdot kip$$

Σ **Uplift Forces =** $U := U_1 + U_2 = 7.979 \cdot kip$

Moment arms of Uplift Forces $d_{up1} := \frac{1}{2}B - \frac{1}{2}B = 0\,ft$

$d_{up2} := \frac{2}{3}B - \frac{1}{2}B = 2.75\,ft$

Moments due to Uplift Components $M_{up1} := U_1 \cdot d_{up1} = 0 \cdot kip \cdot ft$

$M_{up2} := U_2 \cdot d_{up2} = 21.943 \cdot kip \cdot ft$

Σ **Uplift Moments =** $M_{up} := M_{up1} + M_{up2} = 21.943 \cdot kip \cdot ft$

II. INITIAL HORIZONTAL FORCES AND MOMENTS:

A. Headwater: (Horizontal Component)

Horizontal Component of Headwater on Dam

$$F_{uwa.x} := \begin{cases} \gamma_w \cdot \left(H_{uw_4} - H_{dam}\right) \cdot H_{dam} \cdot LF & \text{if } H_{ov}\left(H_{water\_over\_crest}\right) > 0 \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kips$$

$$F_{uwb.x} := \begin{cases} \frac{1}{2}\gamma_w \cdot H_{dam}^{2} \cdot LF & \text{if } H_{ov}\left(H_{water\_over\_crest}\right) > 0 \\ \frac{1}{2}\gamma_w \cdot \left(H_{heel}\right)^2 \cdot LF & \text{otherwise} \end{cases} = 7.496 \cdot kips$$

Σ **Horizontal Forces by Headwater** $F_{uw.x} := F_{uwa.x} + F_{uwb.x} = 7.496\,kip$

Moment arms of Headwater on Dam

$$arm_{uwa.y} := \begin{cases} \frac{H_{dam}}{2} & \text{if } H_{ov}\left(H_{water\_over\_crest}\right) > 0 \\ 0 & \text{otherwise} \end{cases} = 0 \cdot ft$$

$$arm_{uwb.y} := \begin{cases} \frac{1}{3}H_{dam} & \text{if } H_{ov}\left(H_{water\_over\_crest}\right) > 0 \\ \frac{H_{heel}}{3} & \text{otherwise} \end{cases} = 5.167 \cdot ft$$

Moment of Headwater on Dam $M_{uwa.x} := F_{uwa.x} \cdot arm_{uwa.y} = 0 \cdot kip \cdot ft$

$M_{uwb.x} := F_{uwb.x} \cdot arm_{uwb.y} = 38.728 \cdot kip \cdot ft$

Σ **Moments due to Headwater** $M_{uw.x} := M_{uwa.x} + M_{uwb.x} = 38.728 \cdot kip \cdot ft$

B. Tailwater: (Horizontal Component)

(Typically neglect stabilizing force from tailwater during seismic event, unless additional capacity is needed)

$TW := 0$

Horizontal Force due to Tailwater

$$F_{dw.x} := \begin{cases} -\frac{1}{2}\gamma_w \cdot (H_{dw_1})^2 \cdot LF & \text{if } TW = 1 \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kips$$

Moment arm of Tailwater $\quad arm_{dw.y} := \frac{1}{3} H_{dw_3} = 0\ ft$

Moment due to Tailwater $\quad M_{dw.x} := F_{dw.x} \cdot arm_{dw.y} = 0\ ft \cdot kip$

C. Soil horizontal loading on upstream side of dam: - Same as Load Case #1

D. Earthquake Loading

1. Add'l Horizontal Forces and Moments for Concrete due to Earthquake:

Additional horizontal forces: $\quad F_{W1q.x} := -\lambda \cdot W_c = 4.99\ kip$

Σ Additional Masonry Forces Due to Earthquake $\quad F_{damq.x} := F_{W1q.x} = 4.99\ kip$

Moment arms about Toe: $\quad arm_{W1q.y} := Y_{Centroid.c} = 6.48\ ft$

Additional moments: $\quad M_{W1q.x} := F_{W1q.x} \cdot arm_{W1q.y} = 32.333\ ft \cdot kip$

Σ Additional Masonry Moments Due to Earthquake

$$M_{damq.x} := M_{W1q.x} = 32.333\ ft \cdot kip$$

2. Additional Horizontal Forces and Moments from Soil due to Earthquake:

Earthquake Forces from Upstream and Downstream Soil (See Appendix G of EM 1110-2-2100)

Peak Ground Acceleration: $\quad PGA := \lambda = 0.198\ g$

- Assume $k_v := 0$ and neglect effect of soil friction on dam. Vertical face, therefore use EQ G-5 and G-6

Upstream (active) Embankment Angle: $\quad \beta_{us} := 0 \quad \beta_{us} = 0 \cdot °$

Passive Side Embankment Angle: $\quad \beta_{ds} := 0 \quad \beta_{ds} = 0 \cdot °$

Seismic Inertia Angle:

$$\psi := atan\left(\frac{\frac{2}{3}PGA}{1 - k_v}\right) = 0.131 \qquad \psi = 7.52 \cdot °$$

Active seismic soil pressure coefficient (fill):

$$K_{AE\_fill} := \frac{\cos(\phi_{fillus} - \psi)^2}{\cos(\psi)^2 \cdot \left(1 + \sqrt{\frac{\sin(\phi_{fillus})\sin(\phi_{fillus} - \psi - \beta_{us})}{\cos(\beta_{us})\cdot\cos(\psi)}}\right)^2} = 1$$

Passive seismic soil pressure coefficient (fill):

Analyses of previous load cases conservatively used K_0 rather than K_P. Since $K_{PE} >> K_0$, assume seismic force will negate any stabilizing force of the downstream soil.

Earthquake Horizontal Forces from Soil upstream (refer to FBD for dam geometry)

Horizontal Force

$$F_{fillus.q} := \frac{1}{2}(\gamma_{fillus} - \gamma_w)\cdot(EL_{fillus} - EL_b)^2 \cdot LF = 0\ kip$$

Moment arm of fill

$$arm_{fillus.q} := \left(\frac{1}{3}\right)(EL_{fillus} - EL_b) = 0\ ft$$

Moment due to fill

$$M_{fillus.q} := F_{fillus.x} \cdot arm_{fillus.y} = 0\ ft \cdot kip$$

▶ N/A Field; Seismic Loading due to Silt

3. Additional Horizontal Forces and Moments from Reservoir and Tailwater due to Earthquake:

Hydrodynamic Force - Upstream Side:

From Figure #10 of USBR Engineering Monograph #11

$C_e := 0.76$ for dam with a vertical upstream face/slope

The increase in water pressure due to horizontal earthquake acceleration becomes:

$$P_e := C_e \cdot PGA \cdot \gamma_w \cdot \left(EL_{uw_4} - EL_b\right) = 145.544 \cdot psf$$

The total horizontal force due to P_e is expressed analytically as:

$$F_{uwq.x} := 0.726 \cdot P_e \cdot \left(EL_{uw_4} - EL_b\right) \cdot LF = 1.638 \cdot kip$$

The total horizontal moment due to P_e is expressed analytically as:

$$M_{uwq.x} := 0.299 \cdot P_e \cdot \left(EL_{uw_4} - EL_b\right)^2 \cdot LF = 10.455\ ft \cdot kip$$

Hydrodynamic Force - Downstream Side:

Neglected tailwater force during seismic event.

▶ (Summary of Vertical Forces Raw Data - Click to expand)

III. SUMMARY OF INITIAL LOADS AND MOMENTS - CASE #4

SUMMARY OF VERTICAL FORCES/MOMENTS

Component	Acting			Resisting		
	Force (kip)	Arm (ft)	Moment (kip-ft)	Force (kip)	Arm (ft)	Moment (kip-ft)
Weight of Concrete	-	-	-	-25.2	2.4	-61.2
Uplift Pressure #1	0.0	0.0	0.0	-	-	-
Uplift Pressure #2	8.0	2.8	21.9	-	-	-
Totals	**8.0**	**-**	**21.9**	**-25.2**	**-**	**-61.2**

Σ **Vertical Forces w/ uplift** $FV_{tot} := (F_{vr} + F_{va}) \cdot kips$ $FV_{tot} = -17.221 \cdot kips$

Σ **Vertical Moments w/ uplift** $MV_{tot} := (M_{vr} + M_{va}) \cdot kips \cdot ft$ $MV_{tot} = -39.3 \cdot kips \cdot ft$

(Summary of Horizontal Forces Raw Data - Click to expand)

SUMMARY OF HORIZONTAL FORCES/MOMENTS

Component	Acting			Resisting		
	Force (kip)	Arm (ft)	Moment (kip-ft)	Force (kip)	Arm (ft)	Moment (kip-ft)
Headwater on Dam, Fuwa.x	0.0	0.0	0.0	-	-	-
Headwater on Dam, Fuwb.x	7.5	5.2	38.7	-	-	-
Seismic Force, Dam (Total)	5.0	6.5	32.3	-	-	-
Seismic Force, Headwater, Fuwq.x	1.6	0.0	10.5	-	-	-
Tailwater on Dam, Fdw.x				0.0	0.0	0.0
Totals	**14.1**	**-**	**81.5**	**0.0**	**-**	**0.0**

Σ **Horizontal Forces** $FH_{tot} := (F_{hr} + F_{ha}) \cdot kips$ $FH_{tot} = 14.123 \cdot kips$

Σ **Horizontal Moments** $MH_{tot} := (M_{hr} + M_{ha}) \cdot kips \cdot ft$ $MH_{tot} = 81.5 \cdot kips \cdot ft$

Σ **Moments (w/ uplift)** $M_{tot} := (M_{ha} + M_{hr} + M_{va} + M_{vr}) \cdot kips \cdot ft$ $M_{tot} = 42.2 \cdot kips \cdot ft$

IV. EVALUATE OVERTURNING AND BASE PRESSURES

- Check Resultant Location - COE EM1110-2-2200 Overturning Stability Criteria

- Usual Conditions = Within middle third of the base
- Unusual Conditions = Within middle half of the base
- **Extreme Conditions = Within base**

A. Calculate Eccentricity and Base Pressures

- Resultant and Eccentricity

- Eccentricity
(from centroid of Base Area)

$$e_o := \frac{M_{tot}}{-FV_{tot}} = 2.452\,ft$$

(+) = D/S of Centroid
(-) = U/S of Centroid

- Resultant Location (from toe)

$$R_o := \frac{1}{2}B - e_o = 5.798\,ft$$

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- Evaluate Overturning using Resultant Location

$$\textbf{Location}_{\textbf{Ro}_4} := \begin{cases} \text{"WITHIN BASE"} & \text{if } (R_o \geq 0) \wedge (R_o \leq B) \\ \text{"OUTSIDE BASE"} & \text{otherwise} \end{cases}$$

$$\textbf{Location}_{\textbf{Ro}_4} = \text{"WITHIN BASE"}$$

- Base Pressures (includes Uplift)

Base Pressure at Heel:

$$P_{us\_o} := \frac{-FV_{tot}}{B \cdot LF} \cdot \left(1 - \frac{6 \cdot e_o}{B}\right) = 0.113\,\text{ksf}$$

(-) = tension
(+) = compression

Base Pressure at Toe:

$$P_{ds\_o} := \frac{-FV_{tot}}{B \cdot LF} \cdot \left(1 + \frac{6 \cdot e_o}{B}\right) = 1.974\,\text{ksf}$$

(-) = tension
(+) = compression

C. Check if Cracked Base Analysis (CBA) is Required

(CBA is required when base pressure with uplift are negative, i.e. base in tension)

(Note: CBA not required for seismic, unless a crack exists under normal conditions)

$$\textbf{CBA}_4 := \textbf{CBA}_1 = \text{"NOT REQUIRED"}$$

Cracked Base Analysis - Case #4 (Click to Expand, if Required)
Summary of Vertical Forces - Click to Expand
Revised Summary Table
Revised Parameters (Click to Expand)

Evaluate Sliding Stability (shear friction factor):

Base Inclination Angle $\beta = 0$

Base Area for Analysis $A_0 := B_{unc} \cdot LF = 16.5\,\text{ft}^2$

$$FS\_SLIDING_4 := \frac{(-F_V \cdot \cos(\beta) - F_U + F_H \cdot \sin(\beta) \cdot f_\beta) \cdot \tan(\phi_{dam}) + (A_0 - A_{shear}) \cdot c_0 + A_{shear} \cdot c_{shear}}{F_H \cdot \cos(\beta) - F_V \cdot \sin(\beta) \cdot f_\beta} = 1.023$$

Calculated $FS_{sliding}$ $FS\_SLIDING_4 = 1.0$

Required Factor of Safety (from Page 5): $FS\_SLIDING_{min_4} = 1.3$

$$\textbf{Sliding\_Stability} := \begin{cases} \text{"SLIDING OK"} & \text{if } FS\_SLIDING_4 \geq FS\_SLIDING_{min_4} \\ \text{"NOT ADEQUATE"} & \text{otherwise} \end{cases}$$

$$\textbf{Sliding\_Stability} = \text{"NOT ADEQUATE"}$$

Calculate base cohesion necessary to achieve FS = 1.3

$$\textbf{c}_{\textbf{req}_4} := \frac{[1.3 \cdot (F_H \cdot \cos(\beta) - F_V \cdot \sin(\beta) \cdot f_\beta)] - (-F_V \cdot \cos(\beta) - F_U + F_H \cdot \sin(\beta) \cdot f_\beta) \cdot \tan(\phi_{dam}) - A_{shear} \cdot c_{shear}}{A_0 - A_{shear}} = 1.6 \cdot \text{psi}$$

Re-Evaluate Overturning and Base Pressures

Final Resultant Location:

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$R_{rev} := \frac{1}{2} \cdot B_{unc} - e_{rev} = 5.798\,ft$

(+) = U/S of toe
(-) = D/S of toe

$\mathbf{Location}_{\mathbf{Rrev}_4} := \begin{cases} \text{"OK"} & \text{if } (R_{rev} \geq 0) \wedge (R_{rev} \leq B_{unc}) \\ \text{"NG"} & \text{otherwise} \end{cases}$

$\mathbf{Location}_{\mathbf{Rrev}_4} = \text{"OK"}$

- Base Pressure at Toe (includes Uplift)

$\mathbf{p}_{\mathbf{toe}_4} := \begin{cases} P_{ds\_o} & \text{if } \mathbf{CBA_4} = \text{"NOT REQUIRED"} \\ \frac{-(F_V + F_U)}{B_{unc} \cdot LF} \cdot \left[1 + 6 \cdot \left(\frac{e_{rev}}{B_{unc}}\right)\right] & \text{otherwise} \end{cases}$

$\mathbf{p}_{\mathbf{toe}_4} = 2.0 \cdot ksf$

Check to see if pressure exceeds foundation bearing capacity:

$\mathbf{BC_4} := \begin{cases} \text{"OK"} & \text{if } \mathbf{p}_{\mathbf{toe}_4} < 1.33BC \\ \text{"EXCEEDED"} & \text{otherwise} \end{cases}$

$\mathbf{BC_4} = \text{"OK"}$

Check to see if pressure exceeds dam compressive strength:

$\mathbf{Cu\_dam_4} := \begin{cases} \text{"OK"} & \text{if } \mathbf{p}_{\mathbf{toe}_4} < 0.9 \cdot Cu_{dam} \\ \text{"EXCEEDED"} & \text{otherwise} \end{cases}$

$\mathbf{Cu\_dam_4} = \text{"OK"}$

Check flotation:

$FS\_FLOT_4 := \frac{-F_V}{F_U} = 3.158$

Required Factor of Safety against Flotation (from USACE EM 1110-2-2100, Table 3-4):

$FS\_FLOT_{min_4} := 1.1$

$\mathbf{Flotation_4} := \begin{cases} \text{"OK"} & \text{if } FS\_FLOT_4 \geq FS\_FLOT_{min_4} \\ \text{"NOT ADEQUATE"} & \text{otherwise} \end{cases}$

$\mathbf{Flotation_4} = \text{"OK"}$

END OF LOAD CASE #4 ANALYSIS

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SUMMARY OF STABILITY ANALYSIS RESULTS

Sliding

Case	Description	Cracked Base Analysis	Minimum Required FS	Calculated FS	Base Cohesion Req'd for Min FS (psi)
1	Normal water levels	*NOT REQUIRED*	2.0	**1.9**	0.2
2	Flood water levels	*NOT REQUIRED*	1.1	1.4	0.0
3	Normal water levels + Ice	*REQUIRED*	2.0	**1.3**	3.9
4	Normal water levels + Earthquake	*NOT REQUIRED*	1.3	**1.0**	1.6

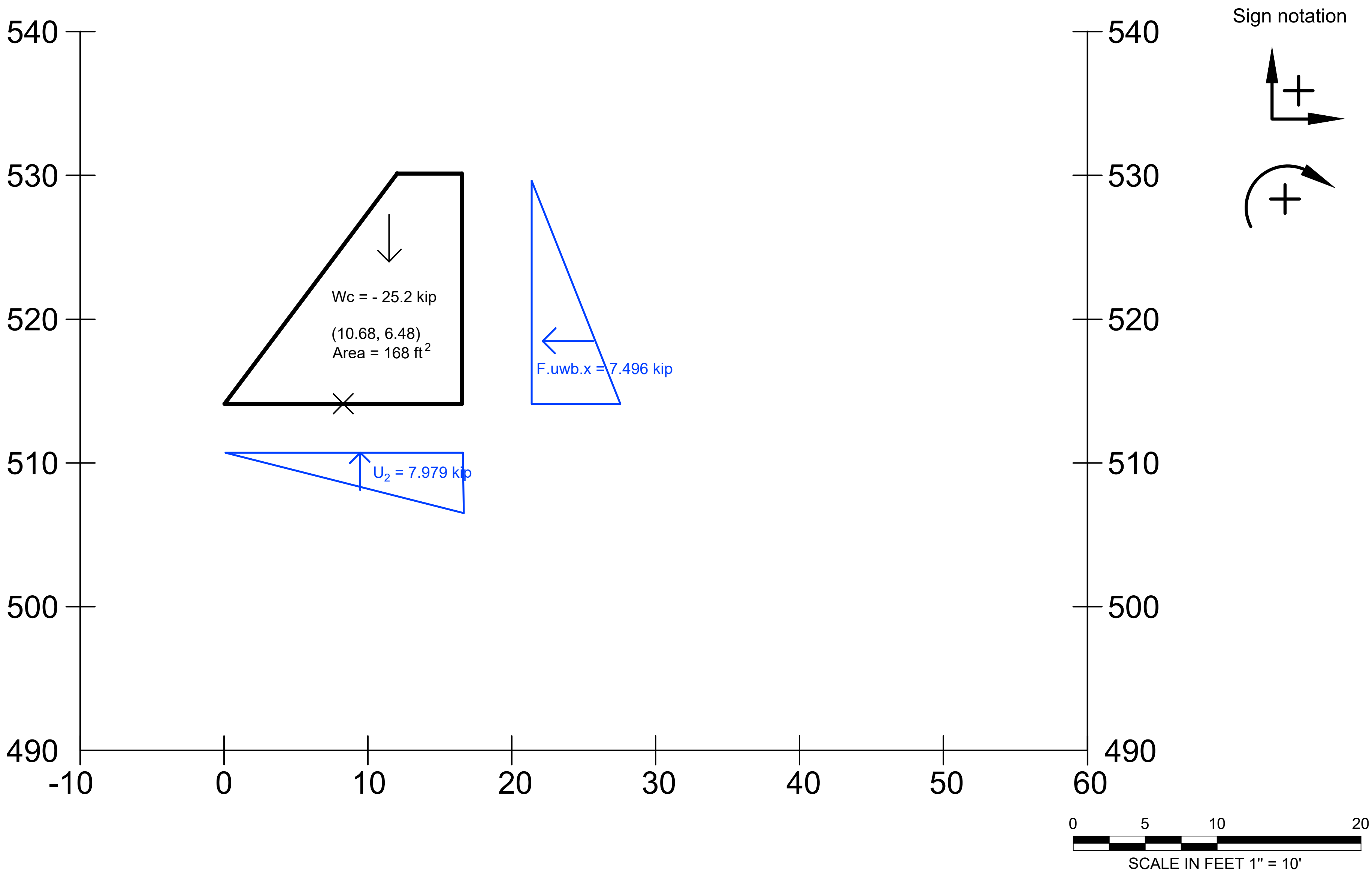
Overturning

Case	Description	Required Resultant Location	Calculated Resultant Location*	Calculated Base Pressure at Toe (ksf)	Bearing Capacity OK?	Dam Compressive Strength OK?
1	Normal water levels	WITHIN MIDDLE 1/3	WITHIN MIDDLE 1/3	1.0	OK	OK
2	Flood water levels	WITHIN BASE	WITHIN BASE	1.4	OK	OK
3	Normal water levels + Ice	WITHIN MIDDLE 1/2	OUTSIDE MIDDLE 1/2	3.0	OK	OK
4	Normal water levels + Earthquake	WITHIN BASE	WITHIN BASE	2.0	OK	OK

Flotation

Case	Description	Minimum Required FS	Calculated FS	FS Flotation OK?
1	Normal water levels	1.3	3.2	OK
2	Flood water levels	1.1	2.7	OK
3	Normal water levels + Ice	1.3	2.5	OK
4	Normal water levels + Earthquake	1.1	3.2	OK

**The resultant location is relative to the overall base width.*



NO.	ISSUE/DESCRIPTION	BY	DATE

INDIAN BROOK RESERVOIR DAM

ESSEX, VT

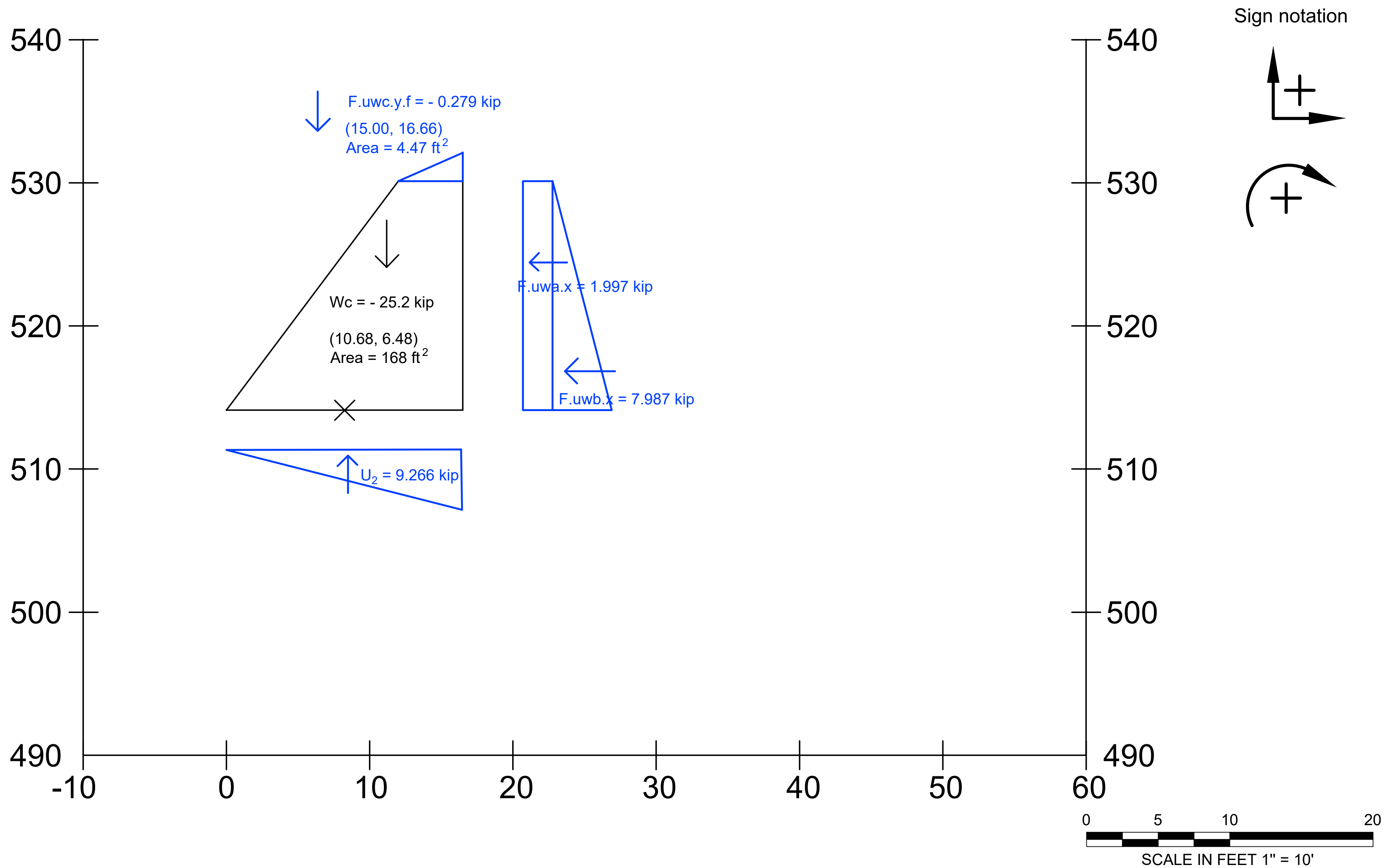
SECTION A-A NORMAL POOL

PREPARED BY: GZA GeoEnvironmental, Inc. www.gza.com

PREPARED FOR: Vermont Department of Environmental Conservation

PROJ MGR: DJS	REVIEWED BY: JGD	CHECKED BY: JGD	FIGURE
DESIGNED BY: MZ	DRAWN BY: MZ	SCALE: 1"=10'	1
DATE: 3-9-2023	PROJECT NO. 01.0175988.00	REVISION NO.	SHEET NO. 1 OF 2

GZA–\\GZANOR\JOBS\170,000–179,999\175988\175988-00.DJS\WORK\STABILITY CALCS\06 INDIAN BROOK RESERVOIR DAM\INDIAN BROOK RESERVOIR DAM_CROSS-SECTION – FMR.DWG A-A PMF NOVEMBER 22, 2023 MENG



NO.	ISSUE/DESCRIPTION	BY	DATE

INDIAN BROOK RESERVOIR DAM

ESSEX, VT

SECTION A-A FLOOD (SDF) POOL

PREPARED BY:
GZA GeoEnvironmental, Inc.
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PREPARED FOR:
Vermont Department of Environmental Conservation

PROJ MGR: DJS	REVIEWED BY: JGD	CHECKED BY: JGD	FIGURE
DESIGNED BY: MZ	DRAWN BY: MZ	SCALE: 1"=10'	2
DATE: 3-9-2023	PROJECT NO. 01.0175988.00	REVISION NO.	SHEET NO. 2 OF 2

Cross-Section B-B

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(Unit Definition - Click Arrow to Expand)

Objective:

To perform a stability analysis of the **Indian Brook Reservoir Dam**, **B-B cross section** using assumption of cracked base where applicable, to calculate factors of safety against sliding and to evaluate overturning stability.

Design Methodology:

- Evaluate stability using conventional equilibrium analyses and limit state theory
- Calculate base pressures with and without considering uplift effects
- Determine non-compression zone, where a cracked section is assumed to have developed and is assumed to be subjected to full headwater pressure.
- Uplift pressure profile is revised and a cracked length is obtained using an iterative solution per USACE methodology

References:

- "Evaluation of Concrete Dam Safety" by ASDSO. Northeast Regional Technical Seminar
- "Gravity Dam Design" by USACE EM1110-2-2200, Jun 1995
- "Stability Analysis of Concrete Structures" by USACE EM1110-2-2100, Dec 2005
- "Design of Small Dams" by US Bureau of Reclamation, 1977
- "Evaluation and comparison of stability analysis and uplift criteria for concrete gravity dams by three federal agencies" by USACE ERDC/ITL TR-00-1, Jun 2000

Case Descriptions - Loading Conditions:

(Per US Army Corps of Engineers)

Case #1: Normal water levels
Case #2: Flood (SDF) water levels
Case #3: Normal water levels + ice
Case #4: Normal water levels + earthquake

Assumptions:

- Full upstream hydraulic head applied to cracked length and is linearly interpolated to downstream hydraulic head over uncracked length (depending on efficiency and location of relief wells)
- Pseudostatic method for seismic analysis (apply horizontal acceleration as a % of g)
- Summation of moments about the centerline of the base of the dam
- Plane of analysis at dam/foundation interface (EL. 501.6)

Notes for MathCAD User (No Calculations)

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Input Parameters

1) Dam Geometry:

Total Base width	$B := 25.5ft$	
Toe base width	$B_1 := 21ft$	
Heel width	$B_2 := 0ft$	
Crest width	$B_c := B - B_1 - B_2 = 4.5\,ft$	
Height of heel slope	$H_2 := 28ft$	
Analysis length	$LF := 1ft$	
Dam base elevation	$EL_b := 501.6ft$	
Dam crest elevation	$EL_c := 529.6ft$	
Dam height	$H_{dam} := EL_c - EL_b = 28\,ft$	
Upstream batter angle (from vertical)	$\theta_u := \operatorname{atan}\left(\frac{B_2}{H_2}\right) = 0 \cdot °$	
Downstream batter angle (from vertical)	$\theta_d := \operatorname{atan}\left(\frac{B_1}{H_{dam}}\right) = 36.87 \cdot °$	
Upstream fill elevation	$EL_{fillus} := EL_b = 501.6\,ft$	*(No upstream fill)*
Inclination angle of base	$\beta := 0deg$	
Sliding direction (upslope / downslope to DS)	$f_\beta := 1$	(+1 sloping down to DS; and -1 sloping up to DS)
Drainage Gallery base elevation	$EL_{dg} := EL_b = 501.6\,ft$	*(No drainage gallery in this section)*
Drain Effectiveness	$E_{dr} := 0\%$	(fully effective = 100%; ineffective = 0%)
Distance of drain to heel	$d_{dr} := 0ft$	*(set to zero if no drain installed)*
Shear Key Area	$A_{shear} := 0ft^2$	*(set to zero if no shear keys present)*
Shear Key Cohesion	$c_{shear} := 0psf$	*(set to zero if no shear keys present)*

1) Dam Geometry (continued): *Refer to Figure #1*

Assume coordinates of toe (0,0) *(Geometry based on CAD)*

W_c: Concrete cross-section

$X_{Centroid.c} := 16.78ft$ $Y_{Centroid.c} := 10.73ft$

$Area_c := 420ft^2$

$F_{uwa.y}$ dimensions: headwater above heel

$X_{Centroid.uwa.y} := 0ft$ $Y_{Centroid.uwa.y} := 0ft$

$Area_{uwa.y} := 0ft^2$

$F_{fillus.y}$ dimensions: fill above toe (upstream side)

$X_{Centroid.fillus.y} := 0ft$ $Y_{Centroid.fillus.y} := 0ft$

$Area_{fillus.y} := 0ft^2$

▶ N/A Fields

2) General Design Elevations

Flood pool elevation $EL_{fw} := 532.1ft$

Normal pool elevation $EL_{nw} := 529.6ft$ *(Assumed at the crest elevation)*

Assume NO silt $H_{silt} := 0ft$ *(No silt)*

Silt/sediment surface level $EL_s := 0ft$

Silt surface slope angle $\alpha := 0°$

3) Case Specific Loads, Dimensions and Elevations

Tailwater Elevation $EL_{dw} := \begin{pmatrix} 501.6ft \\ 506.08ft \\ 501.6ft \\ 501.6ft \end{pmatrix}$ Case #1, Case #2, Case #3, Case #4

Tailwater Height $H_{dw} := EL_{dw} - EL_b$ $H_{dw} = \begin{pmatrix} 0 \\ 4.48 \\ 0 \\ 0 \end{pmatrix} ft$ Case #1, Case #2, Case #3, Case #4

Headwater Elevation $EL_{uw} := \begin{pmatrix} EL_{nw} \\ EL_{fw} \\ EL_{nw} \\ EL_{nw} \end{pmatrix}$ Case #1, Case #2, Case #3, Case #4

Headwater Height $H_{uw} := EL_{uw} - EL_b$ $H_{uw} = \begin{pmatrix} 28 \\ 30.5 \\ 28 \\ 28 \end{pmatrix} ft$ Case #1, Case #2, Case #3, Case #4

Ice thickness $H_{ice} := 1ft$ *(initial assumption)*

Ice pressure $f_{ice} := 5ksf$ *(per USACE)*

Horizontal Seismic Coefficient $\lambda := 0.198$ % acceleration *(Class B PGA for bedrock and a 2500-year return period per 2018 USGS data)*

4) Basic Material Properties

Cohesion between dam/foundation $c_0 := 0psi$ *(Initial assumption)*

Cohesion between dam/foundation $c_{dam} := c_0$ *(Initial assumption)*

Interface friction angle

Concrete on clean sound rock $\phi_{dam} := 40°$ *(Initial assumption)*

Allowable foundation bearing capacity $BC := 15ksf$ *(Initial assumption)*

Allowable compressive strength of dam: $Cu_{dam} := 3000psi$ *(Initial assumption)*

Unit weight of dam material $\gamma_{concrete} := 150pcf$ *(Based on analysis parameter from other project)*

Unit weight of Fill material $\gamma_{fillus} := 0pcf$ *(No Upstream or Downstream Fill)*

$\gamma_{fillds} := 0pcf$

Fill internal frictional angle $\phi_{fillus} := 0°$

$\phi_{fillds} := 0°$

5) Design Factor of Safety (FS) against sliding

US Army Corps of Engineers				
Case		**Loading Conditions**	**Required Factor of Safety**	
			Ordinary Site Info	Well Defined Site Info
1	Normal Pool	Usual	2	1.7
2	PMF	Extreme	1.1	1.1
3	Normal Pool + Ice	Unusual	2	1.7
4	Normal Pool + Seismic	Extreme	1.3	1.1

Factors of Safety Used in Analysis (Ordinary)

$$FS\_SLIDING_{min} := \begin{pmatrix} 2.0 \\ 1.1 \\ 2.0 \\ 1.3 \end{pmatrix}$$

Case #1
Case #2 - See Notes
Case #3
Case #4

Factor of Safety Notes

- PMF is considered "Extreme" due to return period of > 300 yr, in accordance with EM 1110-2-2100.

▶ Misc, Input Parameter Notes (No Calculations)

LOAD CASE #1 - Normal Pool

I. INITIAL VERTICAL LOADS AND MOMENTS:

A. Dam self-weights - (Refer to FBD for dam geometry, variable notation, and sign convention)

Weight of individual Dam Sections

$$W_c := -Area_c \cdot \gamma_{concrete} \cdot LF = -63 \cdot kip$$

Moment arms about Center of Base:

$$D_c := X_{Centroid.c} - \frac{B}{2} = 4.03\,ft$$

Dam Weight Moment about centerline of dam:

$$M_c := W_c \cdot D_c = -253.89 \cdot kip \cdot ft$$

B. Headwater: (Vertical Component)

Conditional statement using variable H_water_over_crest
(*determines whether headwater higher than dam crest)*

$$H_{water\_over\_crest} := H_{uw_1} - H_{dam} = 0\,ft$$

Height of water above crest

$$H_{ov}(H_{water\_over\_crest}) := \begin{cases} H_{water\_over\_crest} & \text{if } H_{water\_over\_crest} > 0 \cdot ft \\ 0 & \text{otherwise} \end{cases}$$

$$\boxed{H_{ov}(H_{water\_over\_crest}) = 0\,ft}$$

Headwater weight on dam (if crest not overtopped, areas of b and c= 0)
a, b, c correspond to various areas of water over the dam, refer to FBD

Headwater Area A $\quad F_{uwa.y} := -\gamma_w \cdot Area_{uwa.y} \cdot LF = 0$

Headwater Area B $\quad F_{uwb.y} := 0 kip$

Headwater Area C $\quad F_{uwc.y} := 0 kip$

Σ Vertical components of Headwater over the Upstream face of the Dam

$$F_{uw.y} := F_{uwa.y} + F_{uwb.y} + F_{uwc.y} = 0$$

Moment arms of a, b & c about the Center of Base

Headwater Area A $\quad arm_{uwa.x} := X_{Centroid.uwa.y} - \frac{1}{2}B = -12.75\,ft \quad$ Above heel

Headwater Area B $\quad arm_{uwb.x} := 0 \quad$ Above crest

Headwater Area C $\quad arm_{uwc.x} := 0$

Moments of vertical headwater forces a, b & c forces about centerline:

Headwater Area A $M_{uwa.y} := F_{uwa.y} \cdot arm_{uwa.x} = 0 \cdot kip \cdot ft$

Headwater Area B $M_{uwb.y} := F_{uwb.y} \cdot arm_{uwb.x} = 0 \cdot kip \cdot ft$

Headwater Area C $M_{uwc.y} := F_{uwc.y} \cdot arm_{uwc.x} = 0 \cdot kip \cdot ft$

Σ Moments from the vertical component of Headwater on U/S face of the Dam

$$M_{uw.y} := M_{uwa.y} + M_{uwb.y} + M_{uwc.y} = 0 \cdot kip \cdot ft$$

<u>C. Tailwater:</u> (Vertical Component)

No vertical tailwater loads appiled.

<u>D. Soil Loads:</u> (Vertical Component due to Weight)

No upstream or downstream fill.

<u>E. Uplift Pressure:</u>

▶ Drainage Gallery related calculation - Click to expand *(No drainage gallery)*

Head at heel $H_{heel} := H_{uw_1} = 28\,ft$

Head at toe $H_{toe} := H_{dw_1} = 0\,ft$ *(Ignore the tailwater here)*

Uplift Pressure at Heel $u_{up\_us} := (H_{heel}) \cdot \gamma_w = 1.747 \cdot ksf$

Uplift Pressure at Toe $u_{up\_ds} := H_{toe} \cdot \gamma_w = 0 \cdot ksf$

Uplift Pressure below Drainage Gallery $u_{up\_dg} := H_{dr} \cdot \gamma_w = 1.747 \cdot ksf$

Uplift forces below dam base:
Refer to FBD for notation:

$$U_1 := u_{up\_ds} \cdot B \cdot LF = 0 \cdot kip$$

$$U_2 := \frac{1}{2}(u_{up\_dg} - u_{up\_ds}) \cdot B \cdot LF = 22.277 \cdot kip$$

Σ Uplift Forces = $U := U_1 + U_2 = 22.277 \cdot kip$

Moment arms of Uplift Forces

$$d_{up1} := \frac{1}{2}B - \frac{1}{2}B = 0\,ft$$

$$d_{up2} := \frac{2}{3}B - \frac{1}{2}B = 4.25\,ft$$

Moments due to Uplift Components $M_{up1} := U_1 \cdot d_{up1} = 0 \cdot kip \cdot ft$

$M_{up2} := U_2 \cdot d_{up2} = 94.676 \cdot kip \cdot ft$

Σ Uplift Moments = $M_{up} := M_{up1} + M_{up2} = 94.676 \cdot kip \cdot ft$

II. INITIAL HORIZONTAL FORCES AND MOMENTS:

A. Headwater: (Horizontal Component)

Horizontal Component of Headwater on Dam

$$F_{uwa.x} := \begin{cases} \gamma_w \cdot \left(H_{uw_1} - H_{dam}\right) \cdot H_{dam} \cdot LF & \text{if } H_{ov}\left(H_{water\_over\_crest}\right) > 0 \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kip$$

$$F_{uwb.x} := \begin{cases} \frac{1}{2} \gamma_w \cdot H_{dam}^{\;2} \cdot LF & \text{if } H_{ov}\left(H_{water\_over\_crest}\right) > 0 \\ \frac{1}{2} \gamma_w \cdot \left(H_{heel}\right)^2 \cdot LF & \text{otherwise} \end{cases} = 24.461 \cdot kip$$

Σ Horizontal Forces by Headwater $F_{uw.x} := F_{uwa.x} + F_{uwb.x} = 24.461\ kip$

Moment arms of Headwater on Dam

$$arm_{uwa.y} := \begin{cases} \frac{H_{dam}}{2} & \text{if } H_{ov}\left(H_{water\_over\_crest}\right) > 0 \\ 0 & \text{otherwise} \end{cases} = 0 \cdot ft$$

$$arm_{uwb.y} := \begin{cases} \frac{1}{3} H_{dam} & \text{if } H_{ov}\left(H_{water\_over\_crest}\right) > 0 \\ \frac{H_{heel}}{3} & \text{otherwise} \end{cases} = 9.333 \cdot ft$$

Moment due to Headwater on Dam

$M_{uwa.x} := F_{uwa.x} \cdot arm_{uwa.y} = 0 \cdot kip \cdot ft$

$M_{uwb.x} := F_{uwb.x} \cdot arm_{uwb.y} = 228.301 \cdot kip \cdot ft$

Σ Moments due to Headwater $M_{uw.x} := M_{uwa.x} + M_{uwb.x} = 228.301 \cdot kip \cdot ft$

B. Tailwater: (Horizontal Component)

Horizontal Force due to Tailwater $F_{dw.x} := \frac{-1}{2} \gamma_w \cdot \left(H_{dw_1}\right)^2 \cdot LF = 0\ kip$

Moment arm of Tailwater $arm_{dw.y} := \frac{1}{3} H_{dw_1} = 0\ ft$

Moment due to Tailwater $M_{dw.x} := F_{dw.x} \cdot arm_{dw.y} = 0\ ft \cdot kip$

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C. Soil horizontal loading on upstream side of dam:

Earth Pressure Coefficients, At Rest Condition

Angle of upstream embankment slope $\beta_{soil} := 0°$

$$K_{0fillus} := \left(1 - \sin\left(\phi_{fillus}\right)\right)\cdot\left(1 + \sin\left(\beta_{soil}\right)\right) = 1.00$$

Horizontal Force due to upstream fill

$$F_{fillus.x} := \frac{1}{2}\cdot K_{0fillus}\cdot\left(\gamma_{fillus} - \gamma_w\right)\cdot\left(EL_{fillus} - EL_b\right)^2\cdot LF = 0\ kip$$

Moment arm of fill

$$arm_{fillus.y} := \left(\frac{1}{3}\right)\left(EL_{fillus} - EL_b\right) = 0\ ft$$

Moment due to fill

$$M_{fillus.x} := F_{fillus.x}\cdot arm_{fillus.y} = 0\ ft\cdot kip$$

D. Soil horizontal loading on downstream side of dam:

No horizontail fill load applied on downsitream side

▶ N/A Fields

▶ (Summary of Vertical Forces Raw Data - Click to expand)

III. SUMMARY OF INITIAL LOADS AND MOMENTS - CASE #1

SUMMARY OF VERTICAL FORCES/MOMENTS

Component	Acting			Resisting		
	Force (kip)	Arm (ft)	Moment (kip-ft)	Force (kip)	Arm (ft)	Moment (kip-ft)
Weight of Concrete	-	-	-	-63.0	4.0	-253.9
Uplift Pressure #1	0.0	0.0	0.0	-	-	-
Uplift Pressure #2	22.3	4.3	94.7	-	-	-
Totals	**22.3**	**-**	**94.7**	**-63.0**	**-**	**-253.9**

Σ **Vertical Forces w/ uplift** $FV_{tot} := (F_{vr} + F_{va}) \cdot kips$ $FV_{tot} = -40.723 \cdot kips$

Σ **Vertical Moments w/ uplift** $MV_{tot} := (M_{vr} + M_{va}) \cdot kips \cdot ft$ $MV_{tot} = -159.2 \cdot kips \cdot ft$

(Summary of Horizontal Forces Raw Data - Click to expand)

SUMMARY OF HORIZONTAL FORCES/MOMENTS

Component	Acting			Resisting		
	Force (kip)	Arm (ft)	Moment (kip-ft)	Force (kip)	Arm (ft)	Moment (kip-ft)
Headwater on Dam, Fuwa.x	0.0	0.0	0.0	-	-	-
Headwater on Dam, Fuwb.x	24.5	9.3	228.3	-	-	-
Tailwater on Dam, Fdw.x	-	-	-	0.0	0.0	0.0
Totals	**24.5**	**-**	**228.3**	**0.0**	**-**	**0.0**

Σ **Horizontal Forces** $FH_{tot} := (F_{hr} + F_{ha}) \cdot kips$ $FH_{tot} = 24.461 \cdot kips$

Σ **Horizontal Moments** $MH_{tot} := (M_{hr} + M_{ha}) \cdot kips \cdot ft$ $MH_{tot} = 228.3 \cdot kips \cdot ft$

Σ **Moments (w/ uplift)** $M_{tot} := (M_{ha} + M_{hr} + M_{va} + M_{vr}) \cdot kips \cdot ft$ $M_{tot} = 69.1 \cdot kips \cdot ft$

IV. EVALUATE OVERTURNING AND BASE PRESSURES

- Check Resultant Location - COE EM1110-2-2200 Overturning Stability Criteria

- Usual Conditions = Within middle third of the base
- Unusual Conditions = Within middle half of the base
- Extreme Conditions = Within base

A. Calculate Eccentricity and Base Pressures

- Resultant and Eccentricity

- Eccentricity (from centroid of Base Area)

$$e_o := \frac{M_{tot}}{-FV_{tot}} = 1.697\,ft$$

(+) = D/S of Centroid
(-) = U/S of Centroid

- Resultant Location (from toe)

$$R_o := \frac{1}{2}B - e_o = 11.053\,ft$$

- Evaluate Overturning using Resultant Location

$$\mathbf{Location_{Ro_1}} := \begin{cases} \text{"WITHIN MIDDLE 1/3"} & \text{if } \left(R_o \geq \frac{B}{3}\right) \wedge \left(R_o \leq \frac{2B}{3}\right) \\ \text{"OUTSIDE MIDDLE 1/3"} & \text{otherwise} \end{cases}$$

$$\mathbf{Location_{Ro_1}} = \text{"WITHIN MIDDLE 1/3"}$$

- Base Pressures (includes Uplift)

Base Pressure at Heel:

$$P_{us\_o} := \frac{-FV_{tot}}{B \cdot LF} \cdot \left(1 - \frac{6 \cdot e_o}{B}\right) = 0.96\,\text{ksf}$$

(-) = tension
(+) = compression

Base Pressure at Toe:

$$P_{ds\_o} := \frac{-FV_{tot}}{B \cdot LF} \cdot \left(1 + \frac{6 \cdot e_o}{B}\right) = 2.234\,\text{ksf}$$

(-) = tension
(+) = compression

C. Check if Cracked Base Analysis (CBA) is Required

(CBA is required when base pressure with uplift are negative, i.e. base in tension)

$$\mathbf{CBA_1} := \begin{cases} \text{"REQUIRED"} & \text{if } \left(P_{us\_o} < 0\right) \vee \left(P_{ds\_o} < 0\right) \\ \text{"NOT REQUIRED"} & \text{otherwise} \end{cases}$$

$$\mathbf{CBA_1} = \text{"NOT REQUIRED"}$$

(Note: if CBA not needed, do not edit Cracked Base Analysis Region

Cracked Base Analysis - Case #1 (Click to Expand, if Required)

Summary of Vertical forces - Click to expand

Revised summary table - Click to expand

Revised Parameters (Click to Expand)

Evaluate Sliding Stability (shear friction factor):

Base Inclination Angle $\beta = 0$

Base Area for Analysis $A_0 := B_{unc} \cdot LF = 25.5\,\text{ft}^2$

With no cohesion

$$FS\_SLIDING_1 := \frac{\left(-F_V \cdot \cos(\beta) - F_U + F_H \cdot \sin(\beta) \cdot f_\beta\right) \cdot \tan\left(\phi_{dam}\right) + \left(A_0 - A_{shear}\right) \cdot c_0 + A_{shear} \cdot c_{shear}}{F_H \cdot \cos(\beta) - F_V \cdot \sin(\beta) \cdot f_\beta} = 1.397$$

Calculated $FS_{sliding}$ $FS\_SLIDING_1 = 1.4$

Required Factor of Safety (from Page 5): $FS\_SLIDING_{min_1} = 2.0$

$$\mathbf{Sliding\_Stability} := \begin{cases} \text{"SLIDING OK"} & \text{if } FS\_SLIDING_1 \geq FS\_SLIDING_{min_1} \\ \text{"NOT ADEQUATE"} & \text{otherwise} \end{cases}$$

$$\mathbf{Sliding\_Stability} = \text{"NOT ADEQUATE"}$$

Calculate base cohesion necessary to achieve FS = 2.0

$$c_{req_1} := \frac{\left[2.0\cdot\left(F_H\cdot\cos(\beta) - F_V\cdot\sin(\beta)\cdot f_\beta\right)\right] - \left(-F_V\cdot\cos(\beta) - F_U + F_H\cdot\sin(\beta)\cdot f_\beta\right)\cdot\tan\left(\phi_{dam}\right) - A_{shear}\cdot c_{shear}}{A_0 - A_{shear}} = 4\cdot psi$$

Re-Evaluate Overturning and Base Pressures

Final Resultant Location:

$$R_{rev} := \frac{1}{2}\cdot B_{unc} - e_{rev} = 11.053\,ft$$

(+) = U/S of toe
(-) = D/S of toe

$$Location_{Rrev_1} := \begin{cases} \text{"OK"} & \text{if } \left(R_{rev} \geq \frac{B_{unc}}{3}\right) \wedge \left(R_{rev} \leq \frac{2B_{unc}}{3}\right) \\ \text{"NG"} & \text{otherwise} \end{cases}$$

$Location_{Rrev_1}$ = "OK"

- Base Pressure at Toe (includes Uplift)

$$p_{toe_1} := \begin{cases} P_{ds\_o} & \text{if } CBA_1 = \text{"NOT REQUIRED"} \\ \frac{-\left(F_V + F_U\right)}{B_{unc}\cdot LF}\cdot\left[1 + 6\cdot\left(\frac{e_{rev}}{B_{unc}}\right)\right] & \text{otherwise} \end{cases}$$

$p_{toe_1} = 2.2\cdot ksf$

Check to see if pressure exceeds foundation bearing capacity:

$$BC_1 := \begin{cases} \text{"OK"} & \text{if } p_{toe_1} < BC \\ \text{"EXCEEDED"} & \text{otherwise} \end{cases}$$

BC_1 = "OK"

Check to see if pressure exceeds dam compressive strength:

$$Cu\_dam_1 := \begin{cases} \text{"OK"} & \text{if } p_{toe_1} < 0.3\cdot Cu_{dam} \\ \text{"EXCEEDED"} & \text{otherwise} \end{cases}$$

Cu_dam_1 = "OK"

Check flotation:

$$FS\_FLOT_1 := \frac{-F_V}{F_U} = 2.828$$

Required Factor of Safety against Flotation (from USACE EM 1110-2-2100, Table 3-4):

$FS\_FLOT_{min_1} := 1.3$

$$Flotation_1 := \begin{cases} \text{"OK"} & \text{if } FS\_FLOT_1 \geq FS\_FLOT_{min_1} \\ \text{"NOT ADEQUATE"} & \text{otherwise} \end{cases}$$

$Flotation_1$ = "OK"

END OF LOAD CASE #1 ANALYSIS

LOAD CASE #2 - Flood Pool

I. INITIAL VERTICAL LOADS AND MOMENTS:

A. Dam self-weights - (Same as Load Case #1)

B. Headwater: (Vertical Component)

Conditional statement using variable H_water_over_crest
(*determines whether headwater higher than dam crest)*

$H_{water\_over\_crest} := H_{uw_2} - H_{dam} = 2.5\,ft$

Height of water above crest

$$H_{ov}(H_{water\_over\_crest}) := \begin{cases} H_{water\_over\_crest} & \text{if } H_{water\_over\_crest} > 0 \cdot ft \\ 0 & \text{otherwise} \end{cases}$$

$H_{ov}(H_{water\_over\_crest}) = 2.5\,ft$

Headwater weight on dam (if crest not overtopped, areas of a, b, and c = 0)
a, b, c correspond to various areas of water over the dam, refer to FBD

$F_{uwb.y}$ dimensions:

$X_{Centroid.uwb.y} := 0ft$ $\quad$ $Y_{Centroid.uwb.y} := 0ft$

$Area_{uwb.y} := 0ft^2$

$F_{uwc.y}$ dimensions:

$X_{Centroid.uwc.y} := 24ft$ $\quad$ $Y_{Centroid.uwc.y} := 28.85ft$

$Area_{uwc.y} := 5.76ft^2$

Headwater weight on dam

Headwater Area A $\quad F_{uwa.y.f} := -Area_{uwa.y} \cdot \gamma_w \cdot LF = 0$ $\quad$ Above heel

Headwater Area B $\quad F_{uwb.y.f} := -Area_{uwb.y} \cdot \gamma_w \cdot LF = 0$ $\quad$ Overtopping crest

Headwater Area C $\quad F_{uwc.y.f} := -Area_{uwc.y} \cdot \gamma_w \cdot LF = -0.359\,kip$

Σ Vertical components of Headwater over the Upstream face of the Dam

$$F_{uw.y.f} := F_{uwa.y.f} + F_{uwb.y.f} + F_{uwc.y.f} = -0.359\,kip$$

Moment arms of a, b & c about the Center of Base

Headwater Area A $arm_{uwa.x.f} := X_{Centroid.uwa.y} - \frac{1}{2}B = -12.75\,ft$ Above heel

Headwater Area B $arm_{uwb.x.f} := X_{Centroid.uwb.y} - \frac{1}{2}B = -12.75\,ft$ Overtopping crest

Headwater Area C $arm_{uwc.x.f} := X_{Centroid.uwc.y} - \frac{1}{2}B = 11.25\,ft$

Moments of vertical headwater forces a, b & c forces about centerline:

Headwater Area A $M_{uwa.y.f} := F_{uwa.y.f} \cdot arm_{uwa.x.f} = 0 \cdot kip \cdot ft$

Headwater Area B $M_{uwb.y.f} := F_{uwb.y.f} \cdot arm_{uwb.x.f} = 0 \cdot kip \cdot ft$

Headwater Area C $M_{uwc.y.f} := F_{uwc.y.f} \cdot arm_{uwc.x.f} = -4.044 \cdot kip \cdot ft$

Σ Moments from the vertical component of Headwater on U/S face of the Dam

$$M_{uw.y.f} := M_{uwa.y.f} + M_{uwb.y.f} + M_{uwc.y.f} = -4.044 \cdot kip \cdot ft$$

C. Tailwater: (Vertical Component)

Tailwater load $F_{dw.y.f}$ dimensions:

$X_{Centroid.dw.y.f} := 1.12ft$ $Y_{Centroid.dw.y.f} := 2.99ft$

$Area_{dw.y.f} := 7.53ft^2$

Vertical component of Tailwater $F_{dw.y.f} := -0.6 Area_{dw.y.f} \cdot \gamma_w \cdot LF = -0.282 \cdot kip$ *(Use 60% of tailwater force per USACE EM 1110-2-2200, Section 3-3.c.(3).(b))*

Moment arm of Tailwater weight about centerline of Base $arm_{dw.x.f} := X_{Centroid.dw.y.f} - \frac{B}{2} = -11.63\,ft$

Moment due to Weight of Tailwater about centerline of Base $M_{dw.y.f} := F_{dw.y.f} \cdot (arm_{dw.x.f}) = 3.279 \cdot kip \cdot ft$

D. Soil Loads: (Vertical Components) **-** Same as Load Case #1

E. Uplift Pressure:

Height of drainage gallery to plane of analysis $H_{dg} := EL_{dg} - EL_b = 0\,ft$

Effective Hydraulic Head at Drainage Gallery, H_{dr}

$$H_{dr} := \begin{cases} (1 - E_{dr}) \cdot \left[\left(H_{uw_2} - H_{dw_2} \right) \frac{B - d_{dr}}{B} + H_{dw_2} - H_{dg} \right] + H_{dg} & \text{if } H_{dg} \geq H_{dw_2} \\ (1 - E_{dr}) \cdot \left(H_{uw_2} - H_{dw_2} \right) \cdot \frac{B - d_{dr}}{B} + H_{dw_2} & \text{otherwise} \end{cases} = 30.5 \cdot ft$$

based on drain effectiveness $E_{dr} = 0 \cdot \%$

Head at heel $H_{heel} := H_{uw_2} = 30.5\ ft$

Head at toe $H_{toe} := H_{dw_2} = 4.48\ ft$

Uplift Pressure at Heel $u_{up\_us} := H_{heel} \cdot \gamma_w = 1.903 \cdot ksf$

Uplift Pressure at Toe $u_{up\_ds} := H_{toe} \cdot \gamma_w = 0.28 \cdot ksf$

Uplift Pressure below Drainage Gallery $u_{up\_dg} := H_{dr} \cdot \gamma_w = 1.903 \cdot ksf$

Uplift forces below dam base:
Refer to FBD for notation:

$U_1 := u_{up\_ds} \cdot B \cdot LF = 7.129 \cdot kip$

$U_2 := \frac{1}{2} \left(u_{up\_dg} - u_{up\_ds} \right) \cdot B \cdot LF = 20.702 \cdot kip$

Σ Uplift Forces = $U := U_1 + U_2 = 27.83 \cdot kip$

Moment arms of Uplift Forces

$d_{up1} := \frac{1}{2} B - \frac{1}{2} B = 0\ ft$

$d_{up2} := \frac{2}{3} B - \frac{1}{2} B = 4.25\ ft$

Moments due to Uplift Components

$M_{up1} := U_1 \cdot d_{up1} = 0 \cdot kip \cdot ft$

$M_{up2} := U_2 \cdot d_{up2} = 87.981 \cdot kip \cdot ft$

Σ Uplift Moments = $M_{up} := M_{up1} + M_{up2} = 87.981 \cdot kip \cdot ft$

II. INITIAL HORIZONTAL FORCES AND MOMENTS:

A. Headwater: (Horizontal Component)

Horizontal Component of Headwater on Dam

$$F_{uwa.x} := \begin{cases} \gamma_w \cdot \left(H_{uw_2} - H_{dam} \right) \cdot H_{dam} \cdot LF & \text{if } H_{ov}\left(H_{water\_over\_crest} \right) > 0 \\ 0 & \text{otherwise} \end{cases} = 4.368 \cdot kips$$

$$F_{uwb.x} := \begin{cases} \frac{1}{2} \gamma_w \cdot H_{dam}^{\ 2} \cdot LF & \text{if } H_{ov}\left(H_{water\_over\_crest} \right) > 0 \\ \frac{1}{2} \gamma_w \cdot \left(H_{dam} \right)^2 \cdot LF & \text{otherwise} \end{cases} = 24.461 \cdot kips$$

Σ Horizontal Forces by Headwater $F_{uw.x} := F_{uwa.x} + F_{uwb.x} = 28.829\,kip$

Moment arms of Headwater on Dam

$$arm_{uwa.y} := \begin{cases} \frac{H_{dam}}{2} & \text{if } H_{ov}(H_{water\_over\_crest}) > 0 \\ 0 & \text{otherwise} \end{cases} = 14 \cdot ft$$

$$arm_{uwb.y} := \begin{cases} \frac{1}{3}H_{dam} & \text{if } H_{ov}(H_{water\_over\_crest}) > 0 \\ \frac{1}{3}H_{uw_2} & \text{otherwise} \end{cases} = 9.333 \cdot ft$$

Moment of Headwater on Dam

$$M_{uwa.x} := F_{uwa.x} \cdot arm_{uwa.y} = 61.152 \cdot kip \cdot ft$$

$$M_{uwb.x} := F_{uwb.x} \cdot arm_{uwb.y} = 228.301 \cdot kip \cdot ft$$

Σ Moments due to Headwater $M_{uw.x} := M_{uwa.x} + M_{uwb.x} = 289.453 \cdot kip \cdot ft$

B. Tailwater: (Horizontal Component)

Horizontal Force due to Tailwater

$$F_{dw.x.f} := \frac{-1}{2} 0.6 \gamma_w \cdot \left(H_{dw_2}\right)^2 \cdot LF = -0.376\,kip$$

(Use 60% of tailwater force per USACE EM 1110-2-2200, Section 3-3.c.(3).(b))

Moment arm of Tailwater

$$arm_{dw.y.f} := \frac{1}{3} H_{dw_2} = 1.493\,ft$$

Moment due to Tailwater

$$M_{dw.x.f} := F_{dw.x.f} \cdot arm_{dw.y.f} = -0.561\,ft \cdot kip$$

C. Soil horizontal loading on upstream side of dam:- Same as Load Case #1

D. Soil horizontal loading on downstream side of dam: - Same as Load Case #1

(Summary of Vertical Forces Raw Data - Click to expand)

III. SUMMARY OF INITIAL LOADS AND MOMENTS - CASE #2

SUMMARY OF VERTICAL FORCES/MOMENTS

Component	Acting			Resisting		
	Force (kip)	Arm (ft)	Moment (kip-ft)	Force (kip)	Arm (ft)	Moment (kip-ft)
Weight of Concrete	-	-	-	-63.0	4.0	-253.9
Headwater over Dam, Fuwa.y	-	-	-	0.0	-12.8	0.0
Headwater over Dam, Fuwb.y	-	-	-	0.0	-12.8	0.0
Headwater over Dam, Fuwc.y	-	-	-	-0.4	11.3	-4.0
Tailwater over Dam, Fdw.y.f	-	-	-	-0.3	-11.6	3.3
Uplift Pressure, Area #1	7.1	0.0	0.0	-	-	-
Uplift Pressure, Area #2	20.7	4.3	88.0	-	-	-
Totals	**27.8**	**-**	**88.0**	**-63.6**	**-**	**-254.7**

Σ **Vertical Forces w/ uplift** $FV_{tot} := (F_{vr} + F_{va}) \cdot kips$ $FV_{tot} = -35.811 \cdot kips$

Σ **Vertical Moments w/ uplift** $MV_{tot} := (M_{vr} + M_{va}) \cdot kips \cdot ft$ $MV_{tot} = -166.7 \cdot kips \cdot ft$

(Summary of Horizontal Forces Raw Data - Click to expand)

SUMMARY OF HORIZONTAL FORCES/MOMENTS

Component	Acting			Resisting		
	Force (kip)	Arm (ft)	Moment (kip-ft)	Force (kip)	Arm (ft)	Moment (kip-ft)
Headwater on Dam, Fuwa.x	4.4	14.0	61.2	-	-	-
Headwater on Dam, Fuwb.x	24.5	9.3	228.3	-	-	-
Tailwater on Dam, Fdw.x	-	-	-	-0.4	1.5	-0.6
Totals	**28.8**	**-**	**289.5**	**-0.4**	**-**	**-0.6**

Σ **Horizontal Forces** $FH_{tot} := (F_{hr} + F_{ha}) \cdot kips$ $FH_{tot} = 28.453 \cdot kips$

Σ **Horizontal Moments** $MH_{tot} := (M_{hr} + M_{ha}) \cdot kips \cdot ft$ $MH_{tot} = 288.9 \cdot kips \cdot ft$

Σ **Moments (w/ uplift)** $M_{tot} := (M_{ha} + M_{hr} + M_{va} + M_{vr}) \cdot kips \cdot ft$ $M_{tot} = 122.2 \cdot kips \cdot ft$

IV. EVALUATE OVERTURNING AND BASE PRESSURES

- Check Resultant Location - COE EM1110-2-2200 Overturning Stability Criteria

- Usual Conditions = Within middle third of the base
- Unusual Conditions = Within middle half of the base
- **Extreme Conditions = Within base**

A. Calculate Eccentricity and Base Pressures

- Resultant and Eccentricity

- Eccentricity (from centroid of Base Area)

$$e_o := \frac{M_{tot}}{-FV_{tot}} = 3.413\,ft$$

(+) = D/S of Centroid
(-) = U/S of Centroid

- Resultant Location (from toe)

$$R_o := \frac{1}{2}B - e_o = 9.337\,ft$$

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Engineers and Scientists

- Evaluate Overturning using Resultant Location

$$\text{Location}_{Ro_2} := \begin{cases} \text{"WITHIN BASE"} & \text{if } (R_o \geq 0) \wedge (R_o \leq B) \\ \text{"OUTSIDE BASE"} & \text{otherwise} \end{cases}$$

$\text{Location}_{Ro_2} = \text{"WITHIN BASE"}$

- Base Pressures (includes Uplift)

Base Pressure at Heel:

$$P_{us\_o} := \frac{-FV_{tot}}{B \cdot LF} \cdot \left(1 - \frac{6 \cdot e_o}{B}\right) = 0.277\,\text{ksf}$$

(-) = tension
(+) = compression

Base Pressure at Toe:

$$P_{ds\_o} := \frac{-FV_{tot}}{B \cdot LF} \cdot \left(1 + \frac{6 \cdot e_o}{B}\right) = 2.532\,\text{ksf}$$

(-) = tension
(+) = compression

C. Check if Cracked Base Analysis (CBA) is Required

(CBA is required when base pressure with uplift are negative, i.e. base in tension)

$$\text{CBA}_2 := \begin{cases} \text{"REQUIRED"} & \text{if } (P_{us\_o} < 0) \vee (P_{ds\_o} < 0) \\ \text{"NOT REQUIRED"} & \text{otherwise} \end{cases}$$

$\text{CBA}_2 = \text{"NOT REQUIRED"}$

(Note: if CBA not needed, skip cracked base analysis section and move on to Factor of Safety calculation)

▶ Cracked Base Analysis - Case #2 (Click to Expand, if Required)

▶ Summary of Vertical Forces - Click to Expand

▶ Revised Summary Table - Click to Expand

▶ Revised Parameters (Click to Expand)

Evaluate Sliding Stability (shear friction factor):

Base Inclination Angle $\beta = 0$

Base Area for Analysis $A_0 := B_{unc} \cdot LF = 25.5\,\text{ft}^2$

With no cohesion

$$FS\_SLIDING_2 := \frac{\left(-F_V \cdot \cos(\beta) - F_U + F_H \cdot \sin(\beta) \cdot f_\beta\right) \cdot \tan(\phi_{dam}) + (A_0 - A_{shear}) \cdot c_0 + A_{shear} \cdot c_{shear}}{F_H \cdot \cos(\beta) - F_V \cdot \sin(\beta) \cdot f_\beta} = 1.056$$

Calculated $FS_{sliding}$ $FS\_SLIDING_2 = 1.1$

Required Factor of Safety (from Page 5): $FS\_SLIDING_{min_2} = 1.1$

$$\text{Sliding\_Stability} := \begin{cases} \text{"SLIDING OK"} & \text{if } FS\_SLIDING_2 \geq FS\_SLIDING_{min_2} \\ \text{"NOT ADEQUATE"} & \text{otherwise} \end{cases}$$

$\text{Sliding\_Stability} = \text{"NOT ADEQUATE"}$

Calculate base cohesion necessary to achieve FS = 1.1

$$c_{req_2} := \frac{\left[1.1\cdot\left(F_H\cdot\cos(\beta) - F_V\cdot\sin(\beta)\cdot f_\beta\right)\right] - \left(-F_V\cdot\cos(\beta) - F_U + F_H\cdot\sin(\beta)\cdot f_\beta\right)\cdot\tan\left(\phi_{dam}\right) - A_{shear}\cdot c_{shear}}{A_0 - A_{shear}} = 0.3\cdot psi$$

Re-Evaluate Overturning and Base Pressures

Final Resultant Location:

$$R_{rev} := \frac{1}{2}\cdot B_{unc} - e_{rev} = 9.337\,ft$$

(+) = U/S of toe
(-) = D/S of toe

$$Location_{Rrev_2} := \begin{cases} \text{"OK"} & \text{if } \left(R_{rev} \ge 0\right) \wedge \left(R_{rev} \le B_{unc}\right) \\ \text{"NG"} & \text{otherwise} \end{cases}$$

$$Location_{Rrev_2} = \text{"OK"}$$

- Base Pressure at Toe (includes Uplift)

$$p_{toe_2} := \begin{cases} P_{ds\_o} & \text{if } CBA_2 = \text{"NOT REQUIRED"} \\ \frac{-\left(F_V + F_U\right)}{B_{unc}\cdot LF}\cdot\left[1 + 6\cdot\left(\frac{e_{rev}}{B_{unc}}\right)\right] & \text{otherwise} \end{cases}$$

$$p_{toe_2} = 2.5\cdot ksf$$

Check to see if pressure exceeds foundation bearing capacity:

$$BC_2 := \begin{cases} \text{"OK"} & \text{if } p_{toe_2} < 1.33BC \\ \text{"EXCEEDED"} & \text{otherwise} \end{cases}$$

$$BC_2 = \text{"OK"}$$

Check to see if pressure exceeds dam compressive strength:

$$Cu\_dam_2 := \begin{cases} \text{"OK"} & \text{if } p_{toe_2} < 0.9\cdot Cu_{dam} \\ \text{"EXCEEDED"} & \text{otherwise} \end{cases}$$

$$Cu\_dam_2 = \text{"OK"}$$

Check flotation:

$$FS\_FLOT_2 := \frac{-F_V}{F_U} = 2.287$$

Required Factor of Safety against Flotation (from USACE EM 1110-2-2100, Table 3-4):

$$FS\_FLOT_{min_2} := 1.1$$

$$Flotation_2 := \begin{cases} \text{"OK"} & \text{if } FS\_FLOT_2 \ge FS\_FLOT_{min_2} \\ \text{"NOT ADEQUATE"} & \text{otherwise} \end{cases}$$

$$Flotation_2 = \text{"OK"}$$

END OF LOAD CASE #2 ANALYSIS

LOAD CASE #3 - Normal Pool + Ice

I. INITIAL VERTICAL LOADS AND MOMENTS:

A. Dam self-weights - Same as Load Case #1

B. Headwater: (Vertical Component) - Same as Load Case #1

Conditional statement using variable H_water_over_crest
(*determines whether headwater higher than dam crest)*

$$H_{water\_over\_crest} := H_{uw_3} - H_{dam} = 0\ ft$$

Height of water above crest

$$H_{ov}(H_{water\_over\_crest}) := \begin{cases} H_{water\_over\_crest} & \text{if } H_{water\_over\_crest} > 0 \cdot ft \\ 0 & \text{otherwise} \end{cases}$$

$$H_{ov}(H_{water\_over\_crest}) = 0\ ft$$

C. Tailwater: (Vertical Component) - Same as Load Case #1

D. Soil Loads: (Vertical Components) - Same as Load Case #1

E. Uplift Pressure:

Height of drainage gallery to plane of analysis $\quad H_{dg} := EL_{dg} - EL_b = 0\ ft$

Effective Hydraulic Head at Drainage Gallery, H_{dr}

$$H_{dr} := \begin{cases} (1 - E_{dr}) \cdot \left[(H_{uw_3} - H_{dw_3}) \dfrac{B - d_{dr}}{B} + H_{dw_3} - H_{dg} \right] + H_{dg} & \text{if } H_{dg} \ge H_{dw_3} \\ (1 - E_{dr}) \cdot (H_{uw_3} - H_{dw_3}) \cdot \dfrac{B - d_{dr}}{B} + H_{dw_3} & \text{otherwise} \end{cases} = 28 \cdot ft$$

based on drain effectiveness $\quad E_{dr} = 0 \cdot \%$

Head at heel $\quad H_{heel} := H_{uw_3} = 28\ ft$

Head at toe $\quad H_{toe} := H_{dw_3} = 0\ ft$

Uplift Pressure at Heel $\quad u_{up\_us} := H_{uw_3} \cdot \gamma_w = 1.747 \cdot ksf$

Uplift Pressure at Toe $\quad u_{up\_ds} := H_{toe} \cdot \gamma_w = 0 \cdot ksf$

Uplift Pressure below Drainage Gallery $\quad u_{up\_dg} := H_{dr} \cdot \gamma_w = 1.747 \cdot ksf$

Uplift forces below dam base:
Refer to FBD for notation:

$$U_1 := u_{up\_ds} \cdot B \cdot LF = 0 \cdot kip$$

$$U_2 := \frac{1}{2}(u_{up\_dg} - u_{up\_ds}) \cdot B \cdot LF = 22.277 \cdot kip$$

Σ Uplift Forces = $U := U_1 + U_2 = 22.277 \cdot kip$

Moment arms of Uplift Forces

$d_{up1} := \frac{1}{2}B - \frac{1}{2}B = 0\,ft$

$d_{up2} := \frac{2}{3}B - \frac{1}{2}B = 4.25\,ft$

Moments due to Uplift Components

$M_{up1} := U_1 \cdot d_{up1} = 0 \cdot kip \cdot ft$

$M_{up2} := U_2 \cdot d_{up2} = 94.676 \cdot kip \cdot ft$

Σ Uplift Moments = $M_{up} := M_{up1} + M_{up2} = 94.676 \cdot kip \cdot ft$

II. INITIAL HORIZONTAL FORCES AND MOMENTS:

A. Headwater: (Horizontal Component)

Horizontal Component of Headwater on Dam

$$F_{uwa.x} := \begin{cases} \gamma_w \cdot \left(H_{uw_3} - H_{dam}\right) \cdot H_{dam} \cdot LF & \text{if } H_{ov}\left(H_{water\_over\_crest}\right) > 0 \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kips$$

$$F_{uwb.x} := \begin{cases} \frac{1}{2}\gamma_w \cdot H_{dam}^{\;2} \cdot LF & \text{if } H_{ov}\left(H_{water\_over\_crest}\right) > 0 \\ \frac{1}{2}\gamma_w \cdot \left(H_{uw_3}\right)^2 \cdot LF & \text{otherwise} \end{cases} = 24.461 \cdot kips$$

Σ Horizontal Forces by Headwater $F_{uw.x} := F_{uwa.x} + F_{uwb.x} = 24.461\,kip$

Moment arms of Headwater on Dam

$$arm_{uwa.y} := \begin{cases} \frac{H_{dam}}{2} & \text{if } H_{ov}\left(H_{water\_over\_crest}\right) > 0 \\ 0 & \text{otherwise} \end{cases} = 0 \cdot ft$$

$$arm_{uwb.y} := \begin{cases} \frac{1}{3}H_{dam} & \text{if } H_{ov}\left(H_{water\_over\_crest}\right) > 0 \\ \frac{1}{3}H_{uw_3} & \text{otherwise} \end{cases} = 9.333 \cdot ft$$

Moment of Headwater on Dam

$M_{uwa.x} := F_{uwa.x} \cdot arm_{uwa.y} = 0 \cdot kip \cdot ft$

$M_{uwb.x} := F_{uwb.x} \cdot arm_{uwb.y} = 228.301 \cdot kip \cdot ft$

Σ Moments due to Headwater $M_{uw.x} := M_{uwa.x} + M_{uwb.x} = 228.301 \cdot kip \cdot ft$

B. Tailwater: (Horizontal Component) - Same as Load Case #1

C. Soil horizontal loading on upstream side of dam: - Same as Load Case #1

D. Soil horizontal loading on downstream side of dam: - Same as Load Case #1

E. Ice Loading

Horizontal Ice Force $F_{ice.x} := H_{ice} \cdot \left(f_{ice}\right) \cdot LF = 5\,kip$

Moment Arm of Ice Force $arm_{ice.y} := \left(H_{uw_3} - 0.5 \cdot H_{ice}\right) = 27.5\,ft$

Moment due to Ice Force $M_{ice.x} := F_{ice.x} \cdot arm_{ice.y} = 137.5\,ft \cdot kip$

(Summary of Vertical Forces Raw Data - Click to expand)

III. SUMMARY OF INITIAL LOADS AND MOMENTS - CASE #3

SUMMARY OF VERTICAL FORCES/MOMENTS

Component	Acting			Resisting		
	Force (kip)	Arm (ft)	Moment (kip-ft)	Force (kip)	Arm (ft)	Moment (kip-ft)
Weight of Concrete	-	-	-	-63.0	4.0	-253.9
Uplift Pressure #1	0.0	0.0	0.0			
Uplift Pressure #2	22.3	4.3	94.7	-	-	-
Totals	**22.3**	**-**	**94.7**	**-63.0**	**-**	**-253.9**

Σ **Vertical Forces w/ uplift** $FV_{tot} := (F_{vr} + F_{va}) \cdot kips$ $FV_{tot} = -40.723 \cdot kips$

Σ **Vertical Moments w/ uplift** $MV_{tot} := (M_{vr} + M_{va}) \cdot kips \cdot ft$ $MV_{tot} = -159.2 \cdot kips \cdot ft$

(Summary of Horizontal Forces Raw Data - Click to expand)

SUMMARY OF HORIZONTAL FORCES/MOMENTS

Component	Acting			Resisting		
	Force (kip)	Arm (ft)	Moment (kip-ft)	Force (kip)	Arm (ft)	Moment (kip-ft)
Headwater on Dam, Fuwa.x	0.0	0.0	0.0	-	-	-
Headwater on Dam, Fuwb.x	24.5	9.3	228.3	-	-	-
Ice Force	5.0	27.5	137.5	-	-	-
Tailwater on Dam, Fdw.x	-	-	-	0.0	0.0	0.0
Totals	**29.5**	**-**	**365.8**	**0.0**	**-**	**0.0**

Σ **Horizontal Forces** $FH_{tot} := (F_{hr} + F_{ha}) \cdot kips$ $FH_{tot} = 29.461 \cdot kips$

Σ **Horizontal Moments** $MH_{tot} := (M_{hr} + M_{ha}) \cdot kips \cdot ft$ $MH_{tot} = 365.8 \cdot kips \cdot ft$

Σ **Moments (w/ uplift)** $M_{tot} := (M_{ha} + M_{hr} + M_{va} + M_{vr}) \cdot kips \cdot ft$ $M_{tot} = 206.6 \cdot kips \cdot ft$

IV. EVALUATE OVERTURNING AND BASE PRESSURES

- Check Resultant Location - COE EM1110-2-2200 Overturning Stability Criteria

- Usual Conditions = Within middle third of the base
- **Unusual Conditions = Within middle half of the base**
- Extreme Conditions = Within base

A. Calculate Eccentricity and Base Pressures

- Resultant and Eccentricity

- Eccentricity
(from centroid of Base Area)

$$e_o := \frac{M_{tot}}{-FV_{tot}} = 5.073\,ft$$

(+) = D/S of Centroid
(-) = U/S of Centroid

- Resultant Location (from toe)

$$R_o := \frac{1}{2}B - e_o = 7.677\,ft$$

- Evaluate Overturning using Resultant Location

$$\text{Location}_{Ro_3} := \begin{cases} \text{"WITHIN MIDDLE 1/2"} & \text{if } \left(R_o \ge \frac{B}{4}\right) \wedge \left(R_o \le \frac{3}{4}B\right) \\ \text{"OUTSIDE MIDDLE 1/2"} & \text{otherwise} \end{cases}$$

$$\text{Location}_{Ro_3} = \text{"WITHIN MIDDLE 1/2"}$$

- Base Pressures (includes Uplift)

Base Pressure at Heel:

$$P_{us\_o} := \frac{-FV_{tot}}{B \cdot LF} \cdot \left(1 - \frac{6 \cdot e_o}{B}\right) = -0.309\,\text{ksf}$$

(-) = tension
(+) = compression

Base Pressure at Toe:

$$P_{ds\_o} := \frac{-FV_{tot}}{B \cdot LF} \cdot \left(1 + \frac{6 \cdot e_o}{B}\right) = 3.503\,\text{ksf}$$

(-) = tension
(+) = compression

C. Check if Cracked Base Analysis (CBA) is Required

(CBA is required when base pressure with uplift are negative, i.e. base in tension)

$$\text{CBA}_3 := \begin{cases} \text{"REQUIRED"} & \text{if } \left(P_{us\_o} < 0\right) \vee \left(P_{ds\_o} < 0\right) \\ \text{"NOT REQUIRED"} & \text{otherwise} \end{cases}$$

$$\text{CBA}_3 = \text{"REQUIRED"}$$

(Note: if CBA not needed, do not edit Cracked Base Analysis Region

Cracked Base Analysis - Case #3 (Click to Expand, if Required)

Cracked Base Analysis - Case #3

Estimate Initial Trial Uncracked Base Length

$$x_{0\_tr} := \frac{-B^2}{12e_o} = -10.682\,\text{ft} \qquad B_{NEW} := \frac{1}{2} \cdot B - x_{0\_tr} = 23.432\,\text{ft}$$

****** ****** ****** ****** ****** ****** ****** ****** ****** Begin Iteration ****** ****** ****** ****** ****** ****** ****** ****** ******

(Manually change uncracked length B_{tr} to achieve D <=0.5%, use B_{NEW}, above, as initial input)

Trial uncracked length: $B_{tr} := 22.7\,\text{ft}$

Corresponding cracked length: $T_{cr} := B - B_{tr} = 2.8\,\text{ft}$

Re-evaluate Vertical Moments based on CBA Per USACE ERDC/ITL TR-00-1

A. Dam self-weights - (Refer to FBD for dam geometry)

Moment arms about Center of Uncracked Base:

$$D_{1.xcr} := X_{Centroid.c} - \frac{B_{tr}}{2} = 5.43\,\text{ft}$$

Moments of Dam Weights about Center of Uncracked Base:

$$M_{W1.ycr} := W_c \cdot D_{1.xcr} = -342.09 \cdot \text{kip} \cdot \text{ft}$$

Σ Dam Weight Moments about Center of Uncracked Base

$$M_{damCR} := M_{W1.ycr} = -342.09 \cdot kip \cdot ft$$

B. Headwater: (Vertical Component)

Moment arms of a, b & c about the Center of Uncracked Base

$$arm_{uwa.xcr} := X_{Centroid.uwa.y} - \frac{1}{2}B = -12.75\,ft$$

$$arm_{uwb.xcr} := X_{Centroid.uwb.y} - \frac{1}{2}B = -12.75\,ft$$

$$arm_{uwc.xcr} := X_{Centroid.uwc.y} - \frac{1}{2}B = 11.25\,ft$$

Moments of vertical forces from a, b & c

$$M_{uwa.ycr} := F_{uwa.y} \cdot arm_{uwa.xcr} = 0 \cdot kip \cdot ft$$

$$M_{uwb.ycr} := F_{uwb.y} \cdot arm_{uwb.xcr} = 0 \cdot kip \cdot ft$$

$$M_{uwc.ycr} := F_{uwc.y} \cdot arm_{uwc.xcr} = 0 \cdot kip \cdot ft$$

Σ Moments from the vertical component of Headwater on U/S face of the Dam

$$M_{uw.yCR} := M_{uwa.ycr} + M_{uwb.ycr} + M_{uwc.ycr} = 0 \cdot kip \cdot ft$$

C. Tailwater: (Vertical Component)

D. Soil Loads: (Vertical Component due to Weight)

E. Re-evaluate Uplift Profile based on Trial Crack Length

Trial Crack Length $T_{cr} = 2.8\,ft$

Distance from Heel to Drain Gallery $d_{dr} = 0$

Revise H_{dr_1} based on trial uncracked base length

$$H_{dr\_1} := \begin{cases} (1 - E_{dr}) \cdot \left[\left(H_{uw_1} - H_{dw_1} \right) \dfrac{B - d_{dr}}{B - T_{cr}} + H_{dw_1} - H_{dg} \right] + H_{dg} & \text{if } H_{dg} > H_{dw_1} \\ (1 - E_{dr}) \cdot \left(H_{uw_1} - H_{dw_1} \right) \cdot \dfrac{B - d_{dr}}{B - T_{cr}} + H_{dw_1} & \text{otherwise} \end{cases} = 31.454 \cdot ft$$

Revise H_{dr_1} based on relationship between crack length and distance from heel to drains

$$H_{dr\_1} := \begin{cases} H_{uw_1} & \text{if } T_{cr} > d_{dr} \\ H_{dr\_1} & \text{otherwise} \end{cases} = 28 \cdot ft$$

Revised Uplift Pressure at the Drainage Gallery

$$u_{up\_dg\_1} := H_{dr\_1} \cdot \gamma_w = 1.747\,ksf$$

Revised Uplift Forces Based on Trial Cracked Length:

$$U_{tr} := \begin{bmatrix} \begin{cases} u_{up\_ds} \cdot (B - d_{dr}) \cdot LF & \text{if } d_{dr} > T_{cr} \\ u_{up\_ds} \cdot B_{tr} \cdot LF & \text{otherwise} \end{cases} \\ \begin{cases} \frac{1}{2}(u_{up\_dg\_1} - u_{up\_ds}) \cdot (B - d_{dr}) \cdot LF & \text{if } d_{dr} > T_{cr} \\ \frac{1}{2}(u_{up\_us} - u_{up\_ds}) \cdot B_{tr} \cdot LF & \text{otherwise} \end{cases} \\ \begin{cases} (u_{up\_dg\_1}) \cdot (d_{dr} - T_{cr}) \cdot LF & \text{if } d_{dr} > T_{cr} \\ 0 & \text{otherwise} \end{cases} \\ \begin{cases} \frac{1}{2}(u_{up\_us} - u_{up\_dg\_1}) \cdot (d_{dr} - T_{cr}) \cdot LF & \text{if } d_{dr} > T_{cr} \\ 0 & \text{otherwise} \end{cases} \\ u_{up\_us} \cdot T_{cr} \cdot LF \end{bmatrix}$$

$$U_{tr} = \begin{pmatrix} 0 \\ 19.831 \\ 0 \\ 0 \\ 4.892 \end{pmatrix} \cdot kips$$

$$U_{TR} := \sum_{n=1}^{5} U_{tr_n} = 24.723\,kip$$

Arms of Revised Uplift Forces about Center of Uncracked Base:

$$D_{tr} := \begin{bmatrix} \begin{cases} \frac{1}{2} \cdot (B - d_{dr}) - \frac{B_{tr}}{2} & \text{if } d_{dr} > T_{cr} \\ \frac{1}{2} \cdot (B - T_{cr}) - \frac{B_{tr}}{2} & \text{otherwise} \end{cases} \\ \begin{cases} \frac{2}{3} \cdot (B - d_{dr}) - \frac{B_{tr}}{2} & \text{if } d_{dr} > T_{cr} \\ \frac{2}{3} \cdot (B - T_{cr}) - \frac{B_{tr}}{2} & \text{otherwise} \end{cases} \\ \begin{cases} \frac{B_{tr}}{2} - \frac{(d_{dr} - T_{cr})}{2} & \text{if } d_{dr} > T_{cr} \\ 0 & \text{otherwise} \end{cases} \\ \begin{cases} \frac{B_{tr}}{2} - \frac{(d_{dr} - T_{cr})}{3} & \text{if } d_{dr} > T_{cr} \\ 0 & \text{otherwise} \end{cases} \\ \frac{B_{tr}}{2} + \frac{T_{cr}}{2} \end{bmatrix}$$

$$D_{tr} = \begin{pmatrix} 0.00 \\ 3.78 \\ 0.00 \\ 0.00 \\ 12.75 \end{pmatrix} \cdot ft$$

Sum Moments using Revised Uplift Forces and Arms about Center of Base

$$M_{up\_tr} := \begin{pmatrix} U_{tr_1} \cdot D_{tr_1} \\ U_{tr_2} \cdot D_{tr_2} \\ U_{tr_3} \cdot D_{tr_3} \\ U_{tr_4} \cdot D_{tr_4} \\ U_{tr_5} \cdot D_{tr_5} \end{pmatrix} \qquad M_{up\_tr} = \begin{pmatrix} 0 \\ 75.026 \\ 0 \\ 0 \\ 62.375 \end{pmatrix} \cdot kips \cdot ft \qquad M_{up\_TR} := \sum_{n=1}^{5} \left(M_{up\_tr}\right)_n = 137.401\,ft \cdot kip$$

Evaluate the revised eccentricity using the results of the CBA

$$M_{tot\_CR} := M_{damCR} + M_{dw.ycr} + MH_{tot} = 27.624\,ft \cdot kip$$

$$M_{up\_TR} = 137.401\,ft \cdot kip$$

$$e_{tr} := \frac{M_{tot\_CR} + M_{up\_TR}}{-\left(F_{vr} \cdot kip + U_{TR}\right)} = 4.311\,ft$$

$$M_{tot\_CR} + M_{up\_TR} = 165.025\,ft \cdot kip$$

$$F_{vr} \cdot kip + U_{TR} = -38.277\,kip$$

<u>Trial Crack Length based on Revised Eccentricity</u>

$$x_{1\_tr} := \frac{-B_{tr}^{\,2}}{12 e_{tr}} = -9.96\,ft \qquad B_{unc} := \frac{1}{2} \cdot B - x_{1\_tr} = 22.71\,ft$$

Check for convergence between Trial Uncracked Length, B_{tr} and Recalculated Uncracked Length, $B_{uncracked}$

Difference (target<=0.5%)

$$\Delta := \frac{\left(B_{unc} - B_{tr}\right)}{B_{tr}} = 0.044 \cdot \%$$

$$Action := \begin{cases} \text{"OK"} & \text{if } \Delta \le 0.5\% \wedge \Delta \ge -0.5\% \\ \text{"CHANGE B\_tr"} & \text{otherwise} \end{cases}$$

$$Action = \text{"OK"}$$

- If "OK"

<u>Final Result of Cracked Base Analysis</u> --->

$$B_{unc} = 22.7\,ft$$

$$T_{cracked} := B - B_{unc} = 2.8 \cdot ft$$

Cracked Base Analysis - Case #3 (Click to Expand, if Required)

Summary of Vertical Forces - Click to Expand

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Engineers and Scientists

Revised Summary Table

III. SUMMARY OF REVISED CRACKED BASE LOADS AND MOMENTS - CASE #3

SUMMARY OF VERTICAL FORCES/MOMENTS

Component	Acting			Resisting		
	Force (kip)	Arm (ft)	Moment (kip-ft)	Force (kip)	Arm (ft)	Moment (kip-ft)
Weight of Concrete	-	-	-	-63.0	5.4	-342.1
Uplift Pressure, Area #1	0.0	0.0	0.0	-		
Uplift Pressure, Area #2	19.8	3.8	75.0	-	-	-
Uplift Pressure, Area #3	0.0	0.0	0.0	-	-	-
Uplift Pressure, Area #4	0.0	0.0	0.0	-	-	-
Uplift Pressure, Area #5	4.9	12.8	62.4	-	-	-
Totals	**24.7**	**-**	**137.4**	**-63.0**	**-**	**-342.1**

Σ Vertical Forces w/ uplift $FV_{cb\_tot} := (F_{cb\_vr} + F_{cb\_va})\cdot kips$ $FV_{cb\_tot} = -38.277\cdot kips$

Σ Vertical Moments w/ uplift $MV_{cb\_tot} := (M_{cb\_vr} + M_{cb\_va})\cdot kips\cdot ft$ $MV_{cb\_tot} = -204.7\cdot kips\cdot ft$

SUMMARY OF HORIZONTAL FORCES/MOMENTS

Component	Acting			Resisting		
	Force (kip)	Arm (ft)	Moment (kip-ft)	Force (kip)	Arm (ft)	Moment (kip-ft)
Headwater on Dam, Fuwa.x	0.0	0.0	0.0	-	-	-
Headwater on Dam, Fuwb.x	24.5	9.3	228.3	-	-	-
Ice Force	5.0	27.5	137.5	-	-	-
Tailwater on Dam, Fdw.x	-	-	-	0.0	0.0	0.0
Totals	**29.5**	**-**	**365.8**	**0.0**	**-**	**0.0**

S Horizontal Forces $FH_{tot} := (F_{hr} + F_{ha})\cdot kips$ $FH_{tot} = 29.461\cdot kips$

S Horizontal Moments $MH_{tot} := (M_{hr} + M_{ha})\cdot kips\cdot ft$ $MH_{tot} = 365.8\cdot kips\cdot ft$

S Moments (w/ uplift) $M_{tot} := (M_{ha} + M_{hr} + M_{va} + M_{vr})\cdot kips\cdot ft$ $M_{tot} = 206.6\cdot kips\cdot ft$

Revised Summary Table

Revised Parameters (Click to Expand)

Evaluate Sliding Stability (shear friction factor):

Base Inclination Angle $\beta = 0$

Base Area for Analysis $A_0 := B_{unc}\cdot LF = 22.71\,ft^2$

$$FS\_SLIDING_3 := \frac{(-F_V\cdot\cos(\beta) - F_U + F_H\cdot\sin(\beta)\cdot f_\beta)\cdot\tan(\phi_{dam}) + (A_0 - A_{shear})\cdot c_0 + A_{shear}\cdot c_{shear}}{F_H\cdot\cos(\beta) - F_V\cdot\sin(\beta)\cdot f_\beta} = 1.129$$

Calculated $FS_{sliding}$ $FS\_SLIDING_3 = 1.1$ Required Factor of Safety (from Page 5): $FS\_SLIDING_{min_3} = 2.0$

Sliding_Stability := | "SLIDING OK" if $FS\_SLIDING_3 \geq FS\_SLIDING_{min_3}$
| "NOT ADEQUATE" otherwise

Sliding_Stability = "NOT ADEQUATE"

Calculate base cohesion necessary to achieve FS = 2.0

$$c_{req_3} := \frac{\left[2.0\cdot\left(F_H\cdot\cos(\beta) - F_V\cdot\sin(\beta)\cdot f_\beta\right)\right] - \left(-F_V\cdot\cos(\beta) - F_U + F_H\cdot\sin(\beta)\cdot f_\beta\right)\cdot\tan\left(\phi_{dam}\right) - A_{shear}\cdot c_{shear}}{A_0 - A_{shear}} = 7.6\cdot psi$$

Re-Evaluate Overturning and Base Pressures

Final Resultant Location:

$$R_{rev} := \frac{1}{2}\cdot B_{unc} - e_{rev} = 7.044\,ft$$

(+) = U/S of toe
(-) = D/S of toe

$$Location_{Rrev_3} := \begin{cases} \text{"OK"} & \text{if } \left(R_{rev} \ge \frac{B_{unc}}{4}\right) \wedge \left(R_{rev} \le 3\frac{B_{unc}}{4}\right) \\ \text{"NG"} & \text{otherwise} \end{cases}$$

$$Location_{Rrev_3} = \text{"OK"}$$

- Base Pressure at Toe (includes Uplift)

$$p_{toe_3} := \begin{cases} P_{ds\_o} & \text{if } CBA_3 = \text{"NOT REQUIRED"} \\ \frac{-\left(F_V + F_U\right)}{B_{unc}\cdot LF}\cdot\left[1 + 6\cdot\left(\frac{e_{rev}}{B_{unc}}\right)\right] & \text{otherwise} \end{cases}$$

$$p_{toe_3} = 3.6\cdot ksf$$

Check to see if pressure exceeds foundation bearing capacity:

$$BC_3 := \begin{cases} \text{"OK"} & \text{if } p_{toe_3} < 1.33BC \\ \text{"EXCEEDED"} & \text{otherwise} \end{cases}$$

$$BC_3 = \text{"OK"}$$

Check to see if pressure exceeds dam compressive strength:

$$Cu\_dam_3 := \begin{cases} \text{"OK"} & \text{if } p_{toe_3} < 0.9\cdot Cu_{dam} \\ \text{"EXCEEDED"} & \text{otherwise} \end{cases}$$

$$Cu\_dam_3 = \text{"OK"}$$

Check flotation:

$$FS\_FLOT_3 := \frac{-F_V}{F_U} = 2.548$$

Required Factor of Safety against Flotation (from USACE EM 1110-2-2100, Table 3-4):

$$FS\_FLOT_{min_3} := 1.1$$

$$Flotation_3 := \begin{cases} \text{"OK"} & \text{if } FS\_FLOT_3 \ge FS\_FLOT_{min_3} \\ \text{"NOT ADEQUATE"} & \text{otherwise} \end{cases}$$

$$Flotation_3 = \text{"OK"}$$

END OF LOAD CASE #3 ANALYSIS

LOAD CASE #4 - Normal Pool + Earthquake

I. INITIAL VERTICAL LOADS AND MOMENTS:

A. Dam self-weights - Same as Load Case #1

B. Headwater: (Vertical Component) - Same as Load Case #1

Conditional statement using variable H_water_over_crest
(*determines whether headwater higher than dam crest)*

$$H_{water\_over\_crest} := H_{uw_4} - H_{dam} = 0\,ft$$

Height of water above crest

$$H_{ov}(H_{water\_over\_crest}) := \begin{cases} H_{water\_over\_crest} & \text{if } H_{water\_over\_crest} > 0 \cdot ft \\ 0 & \text{otherwise} \end{cases}$$

$$\boxed{H_{ov}(H_{water\_over\_crest}) = 0\,ft}$$

C. Tailwater: (Vertical Component) - Same as Load Case #1

D. Soil Loads: (Vertical Components) - Same as Load Case #1

E. Uplift Pressure:

Height of drainage gallery to plane of analysis

$$H_{dg} := EL_{dg} - EL_b = 0\,ft$$

Effective Hydraulic Head at Drainage Gallery, H_{dr}

$$H_{dr} := \begin{cases} (1 - E_{dr}) \cdot \left[(H_{uw_4} - H_{dw_4}) \dfrac{B - d_{dr}}{B} + H_{dw_4} - H_{dg} \right] + H_{dg} & \text{if } H_{dg} \geq H_{dw_4} \\ (1 - E_{dr}) \cdot (H_{uw_4} - H_{dw_4}) \cdot \dfrac{B - d_{dr}}{B} + H_{dw_4} & \text{otherwise} \end{cases} = 28 \cdot ft$$

based on drain effectiveness $E_{dr} = 0 \cdot \%$

Head at heel $H_{heel} := H_{uw_4} = 28\,ft$

Head at toe $H_{toe} := H_{dw_4} = 0\,ft$

Uplift Pressure at Heel $u_{up\_us} := H_{uw_4} \cdot \gamma_w = 1.747 \cdot ksf$

Uplift Pressure at Toe $u_{up\_ds} := H_{toe} \cdot \gamma_w = 0 \cdot ksf$

Uplift Pressure below Drainage Gallery $u_{up\_dg} := H_{dr} \cdot \gamma_w = 1.747 \cdot ksf$

Uplift forces below dam base:
Refer to FBD for notation:

$$U_1 := u_{up\_ds} \cdot B \cdot LF = 0 \cdot kip$$

$$U_2 := \frac{1}{2}(u_{up\_dg} - u_{up\_ds}) \cdot B \cdot LF = 22.277 \cdot kip$$

Σ **Uplift Forces =** $U := U_1 + U_2 = 22.277 \cdot kip$

Moment arms of Uplift Forces $d_{up1} := \frac{1}{2}B - \frac{1}{2}B = 0\,ft$

$d_{up2} := \frac{2}{3}B - \frac{1}{2}B = 4.25\,ft$

Moments due to Uplift Components $M_{up1} := U_1 \cdot d_{up1} = 0 \cdot kip \cdot ft$

$M_{up2} := U_2 \cdot d_{up2} = 94.676 \cdot kip \cdot ft$

Σ **Uplift Moments =** $M_{up} := M_{up1} + M_{up2} = 94.676 \cdot kip \cdot ft$

II. INITIAL HORIZONTAL FORCES AND MOMENTS:

A. Headwater: (Horizontal Component)

Horizontal Component of Headwater on Dam

$$F_{uwa.x} := \begin{cases} \gamma_w \cdot \left(H_{uw_4} - H_{dam}\right) \cdot H_{dam} \cdot LF & \text{if } H_{ov}\left(H_{water\_over\_crest}\right) > 0 \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kips$$

$$F_{uwb.x} := \begin{cases} \frac{1}{2}\gamma_w \cdot H_{dam}^{\;2} \cdot LF & \text{if } H_{ov}\left(H_{water\_over\_crest}\right) > 0 \\ \frac{1}{2}\gamma_w \cdot \left(H_{heel}\right)^2 \cdot LF & \text{otherwise} \end{cases} = 24.461 \cdot kips$$

Σ **Horizontal Forces by Headwater** $F_{uw.x} := F_{uwa.x} + F_{uwb.x} = 24.461\,kip$

Moment arms of Headwater on Dam

$$arm_{uwa.y} := \begin{cases} \frac{H_{dam}}{2} & \text{if } H_{ov}\left(H_{water\_over\_crest}\right) > 0 \\ 0 & \text{otherwise} \end{cases} = 0 \cdot ft$$

$$arm_{uwb.y} := \begin{cases} \frac{1}{3}H_{dam} & \text{if } H_{ov}\left(H_{water\_over\_crest}\right) > 0 \\ \frac{H_{heel}}{3} & \text{otherwise} \end{cases} = 9.333 \cdot ft$$

Moment of Headwater on Dam $M_{uwa.x} := F_{uwa.x} \cdot arm_{uwa.y} = 0 \cdot kip \cdot ft$

$M_{uwb.x} := F_{uwb.x} \cdot arm_{uwb.y} = 228.301 \cdot kip \cdot ft$

Σ **Moments due to Headwater** $M_{uw.x} := M_{uwa.x} + M_{uwb.x} = 228.301 \cdot kip \cdot ft$

B. Tailwater: (Horizontal Component)

(Typically neglect stabilizing force from tailwater during seismic event, unless additional capacity is needed)

$TW := 0$

Horizontal Force due to Tailwater

$$F_{dw.x} := \begin{cases} -\frac{1}{2}\gamma_w \cdot \left(H_{dw_1}\right)^2 \cdot LF & \text{if } TW = 1 \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kips$$

Moment arm of Tailwater

$$arm_{dw.y} := \frac{1}{3} H_{dw_3} = 0\ ft$$

Moment due to Tailwater

$$M_{dw.x} := F_{dw.x} \cdot arm_{dw.y} = 0\ ft \cdot kip$$

C. Soil horizontal loading on upstream side of dam: - Same as Load Case #1

D. Earthquake Loading

1. Add'l Horizontal Forces and Moments for Concrete due to Earthquake:

Additional horizontal forces: $F_{W1q.x} := -\lambda \cdot W_c = 12.474\ kip$

Σ Additional Masonry Forces Due to Earthquake $F_{damq.x} := F_{W1q.x} = 12.474\ kip$

Moment arms about Toe: $arm_{W1q.y} := Y_{Centroid.c} = 10.73\ ft$

Additional moments: $M_{W1q.x} := F_{W1q.x} \cdot arm_{W1q.y} = 133.846\ ft \cdot kip$

Σ Additional Masonry Moments Due to Earthquake

$$M_{damq.x} := M_{W1q.x} = 133.846\ ft \cdot kip$$

2. Additional Horizontal Forces and Moments from Soil due to Earthquake:

Earthquake Forces from Upstream and Downstream Soil (See Appendix G of EM 1110-2-2100)

Peak Ground Acceleration: $PGA := \lambda = 0.198\ g$

- Assume $k_v := 0$ and neglect effect of soil friction on dam. Vertical face, therefore use EQ G-5 and G-6

Upstream (active) Embankment Angle: $\beta_{us} := 0$ $\beta_{us} = 0 \cdot °$

Passive Side Embankment Angle: $\beta_{ds} := 0$ $\beta_{ds} = 0 \cdot °$

Seismic Inertia Angle:

$$\psi := \operatorname{atan}\left(\frac{\frac{2}{3} PGA}{1 - k_v}\right) = 0.131 \qquad \psi = 7.52 \cdot °$$

Active seismic soil pressure coefficient (fill):

$$K_{AE\_fill} := \frac{\cos(\phi_{fillus} - \psi)^2}{\cos(\psi)^2 \cdot \left(1 + \sqrt{\frac{\sin(\phi_{fillus})\sin(\phi_{fillus} - \psi - \beta_{us})}{\cos(\beta_{us})\cdot\cos(\psi)}}\right)^2} = 1$$

Passive seismic soil pressure coefficient (fill):

Analyses of previous load cases conservatively used K_0 rather than K_P. Since $K_{PE} >> K_0$, assume seismic force will negate any stabilizing force of the downstream soil.

Earthquake Horizontal Forces from Soil upstream (refer to FBD for dam geometry)

Horizontal Force

$$F_{fillus.q} := \frac{1}{2}(\gamma_{fillus} - \gamma_w)\cdot(EL_{fillus} - EL_b)^2 \cdot LF = 0\ kip$$

Moment arm of fill

$$arm_{fillus.q} := \left(\frac{1}{3}\right)(EL_{fillus} - EL_b) = 0\ ft$$

Moment due to fill

$$M_{fillus.q} := F_{fillus.x} \cdot arm_{fillus.y} = 0\ ft\cdot kip$$

▶ N/A Field; Seismic Loading due to Silt

3. Additional Horizontal Forces and Moments from Reservoir and Tailwater due to Earthquake:

Hydrodynamic Force - Upstream Side:

From Figure #10 of USBR Engineering Monograph #11

$C_e := 0.76$ for dam with a vertical upstream face/slope

The increase in water pressure due to horizontal earthquake acceleration becomes:

$$P_e := C_e \cdot PGA \cdot \gamma_w \cdot \left(EL_{uw_4} - EL_b\right) = 262.919 \cdot psf$$

The total horizontal force due to P_e is expressed analytically as:

$$F_{uwq.x} := 0.726 \cdot P_e \cdot \left(EL_{uw_4} - EL_b\right) \cdot LF = 5.345 \cdot kip$$

The total horizontal moment due to P_e is expressed analytically as:

$$M_{uwq.x} := 0.299 \cdot P_e \cdot \left(EL_{uw_4} - EL_b\right)^2 \cdot LF = 61.632\ ft\cdot kip$$

Hydrodynamic Force - Downstream Side:

Neglected tailwater force during seismic event.

▶ (Summary of Vertical Forces Raw Data - Click to expand)

III. SUMMARY OF INITIAL LOADS AND MOMENTS - CASE #4

SUMMARY OF VERTICAL FORCES/MOMENTS

Component	Acting			Resisting		
	Force (kip)	Arm (ft)	Moment (kip-ft)	Force (kip)	Arm (ft)	Moment (kip-ft)
Weight of Concrete	-	-	-	-63.0	4.0	-253.9
Uplift Pressure #1	0.0	0.0	0.0	-	-	-
Uplift Pressure #2	22.3	4.3	94.7	-	-	-
Totals	**22.3**	**-**	**94.7**	**-63.0**	**-**	**-253.9**

Σ **Vertical Forces w/ uplift** $FV_{tot} := (F_{vr} + F_{va}) \cdot kips$ $FV_{tot} = -40.723 \cdot kips$

Σ **Vertical Moments w/ uplift** $MV_{tot} := (M_{vr} + M_{va}) \cdot kips \cdot ft$ $MV_{tot} = -159.2 \cdot kips \cdot ft$

(Summary of Horizontal Forces Raw Data - Click to expand)

SUMMARY OF HORIZONTAL FORCES/MOMENTS

Component	Acting			Resisting		
	Force (kip)	Arm (ft)	Moment (kip-ft)	Force (kip)	Arm (ft)	Moment (kip-ft)
Headwater on Dam, Fuwa.x	0.0	0.0	0.0	-	-	-
Headwater on Dam, Fuwb.x	24.5	9.3	228.3	-	-	-
Seismic Force, Dam (Total)	12.5	10.7	133.8	-	-	-
Seismic Force, Headwater, Fuwq.x	5.3	0.0	61.6	-	-	-
Tailwater on Dam, Fdw.x	-	-	-	0.0	0.0	0.0
Totals	**42.3**	**-**	**423.8**	**0.0**	**-**	**0.0**

Σ **Horizontal Forces** $FH_{tot} := (F_{hr} + F_{ha}) \cdot kips$ $FH_{tot} = 42.279 \cdot kips$

Σ **Horizontal Moments** $MH_{tot} := (M_{hr} + M_{ha}) \cdot kips \cdot ft$ $MH_{tot} = 423.8 \cdot kips \cdot ft$

Σ **Moments (w/ uplift)** $M_{tot} := (M_{ha} + M_{hr} + M_{va} + M_{vr}) \cdot kips \cdot ft$ $M_{tot} = 264.6 \cdot kips \cdot ft$

IV. EVALUATE OVERTURNING AND BASE PRESSURES

- Check Resultant Location - COE EM1110-2-2200 Overturning Stability Criteria

- Usual Conditions = Within middle third of the base
- Unusual Conditions = Within middle half of the base
- **Extreme Conditions = Within base**

A. Calculate Eccentricity and Base Pressures

- Resultant and Eccentricity

- Eccentricity
(from centroid of Base Area)

$$e_o := \frac{M_{tot}}{-FV_{tot}} = 6.497\,ft$$

(+) = D/S of Centroid
(-) = U/S of Centroid

- Resultant Location (from toe)

$$R_o := \frac{1}{2}B - e_o = 6.253\,ft$$

- Evaluate Overturning using Resultant Location

$$\text{Location}_{Ro_4} := \begin{cases} \text{"WITHIN BASE"} & \text{if } (R_o \geq 0) \wedge (R_o \leq B) \\ \text{"OUTSIDE BASE"} & \text{otherwise} \end{cases}$$

$\text{Location}_{Ro_4} = \text{"WITHIN BASE"}$

- Base Pressures (includes Uplift)

Base Pressure at Heel:

$$P_{us\_o} := \frac{-FV_{tot}}{B \cdot LF} \cdot \left(1 - \frac{6 \cdot e_o}{B}\right) = -0.844\,\text{ksf}$$

(-) = tension
(+) = compression

Base Pressure at Toe:

$$P_{ds\_o} := \frac{-FV_{tot}}{B \cdot LF} \cdot \left(1 + \frac{6 \cdot e_o}{B}\right) = 4.038\,\text{ksf}$$

(-) = tension
(+) = compression

C. Check if Cracked Base Analysis (CBA) is Required

(CBA is required when base pressure with uplift are negative, i.e. base in tension)

(Note: CBA not required for seismic, unless a crack exists under normal conditions)

$\text{CBA}_4 := \text{CBA}_1 = \text{"NOT REQUIRED"}$

Cracked Base Analysis - Case #4 (Click to Expand, if Required)
Summary of Vertical Forces - Click to Expand
Revised Summary Table
Revised Parameters (Click to Expand)

Evaluate Sliding Stability (shear friction factor):

Base Inclination Angle $\beta = 0$

Base Area for Analysis $A_0 := B_{unc} \cdot LF = 25.5\,\text{ft}^2$

$$FS\_SLIDING_4 := \frac{\left(-F_V \cdot \cos(\beta) - F_U + F_H \cdot \sin(\beta) \cdot f_\beta\right) \cdot \tan(\phi_{dam}) + (A_0 - A_{shear}) \cdot c_0 + A_{shear} \cdot c_{shear}}{F_H \cdot \cos(\beta) - F_V \cdot \sin(\beta) \cdot f_\beta} = 0.808$$

Calculated $FS_{sliding}$ $FS\_SLIDING_4 = 0.8$

Required Factor of Safety (from Page 5): $FS\_SLIDING_{min_4} = 1.3$

$$\text{Sliding\_Stability} := \begin{cases} \text{"SLIDING OK"} & \text{if } FS\_SLIDING_4 \geq FS\_SLIDING_{min_4} \\ \text{"NOT ADEQUATE"} & \text{otherwise} \end{cases}$$

$\text{Sliding\_Stability} = \text{"NOT ADEQUATE"}$

Calculate base cohesion necessary to achieve FS = 1.3

$$c_{req_4} := \frac{\left[1.3 \cdot \left(F_H \cdot \cos(\beta) - F_V \cdot \sin(\beta) \cdot f_\beta\right)\right] - \left(-F_V \cdot \cos(\beta) - F_U + F_H \cdot \sin(\beta) \cdot f_\beta\right) \cdot \tan(\phi_{dam}) - A_{shear} \cdot c_{shear}}{A_0 - A_{shear}} = 5.7 \cdot \text{psi}$$

Re-Evaluate Overturning and Base Pressures

Final Resultant Location:

$R_{rev} := \frac{1}{2} \cdot B_{unc} - e_{rev} = 6.253\,ft$

(+) = U/S of toe
(-) = D/S of toe

$Location_{Rrev_4} := \begin{cases} \text{"OK"} & \text{if } (R_{rev} \ge 0) \wedge (R_{rev} \le B_{unc}) \\ \text{"NG"} & \text{otherwise} \end{cases}$

$Location_{Rrev_4} = \text{"OK"}$

- Base Pressure at Toe (includes Uplift)

$p_{toe_4} := \begin{cases} P_{ds\_o} & \text{if } CBA_4 = \text{"NOT REQUIRED"} \\ \frac{-(F_V + F_U)}{B_{unc} \cdot LF} \cdot \left[1 + 6 \cdot \left(\frac{e_{rev}}{B_{unc}}\right)\right] & \text{otherwise} \end{cases}$

$p_{toe_4} = 4.0 \cdot ksf$

Check to see if pressure exceeds foundation bearing capacity:

$BC_4 := \begin{cases} \text{"OK"} & \text{if } p_{toe_4} < 1.33BC \\ \text{"EXCEEDED"} & \text{otherwise} \end{cases}$

$BC_4 = \text{"OK"}$

Check to see if pressure exceeds dam compressive strength:

$Cu\_dam_4 := \begin{cases} \text{"OK"} & \text{if } p_{toe_4} < 0.9 \cdot Cu_{dam} \\ \text{"EXCEEDED"} & \text{otherwise} \end{cases}$

$Cu\_dam_4 = \text{"OK"}$

Check flotation:

$FS\_FLOT_4 := \frac{-F_V}{F_U} = 2.828$

Required Factor of Safety against Flotation (from USACE EM 1110-2-2100, Table 3-4):

$FS\_FLOT_{min_4} := 1.1$

$Flotation_4 := \begin{cases} \text{"OK"} & \text{if } FS\_FLOT_4 \ge FS\_FLOT_{min_4} \\ \text{"NOT ADEQUATE"} & \text{otherwise} \end{cases}$

$Flotation_4 = \text{"OK"}$

END OF LOAD CASE #4 ANALYSIS

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SUMMARY OF STABILITY ANALYSIS RESULTS

Sliding

Case	Description	Cracked Base Analysis	Minimum Required FS	Calculated FS	Base Cohesion Req'd for Min FS (psi)
1	Normal water levels	*NOT REQUIRED*	2.0	**1.4**	4.0
2	Flood water levels	*NOT REQUIRED*	1.1	**1.1**	0.3
3	Normal water levels + Ice	*REQUIRED*	2.0	**1.1**	7.6
4	Normal water levels + Earthquake	*NOT REQUIRED*	1.3	**0.8**	5.7

Overturning

Case	Description	Required Resultant Location	Calculated Resultant Location*	Calculated Base Pressure at Toe (ksf)	Bearing Capacity OK?	Dam Compressive Strength OK?
1	Normal water levels	WITHIN MIDDLE 1/3	WITHIN MIDDLE 1/3	2.2	OK	OK
2	Flood water levels	WITHIN BASE	WITHIN BASE	2.5	OK	OK
3	Normal water levels + Ice	WITHIN MIDDLE 1/2	WITHIN MIDDLE 1/2	3.6	OK	OK
4	Normal water levels + Earthquake	WITHIN BASE	WITHIN BASE	4.0	OK	OK

Flotation

Case	Description	Minimum Required FS	Calculated FS	FS Flotation OK?
1	Normal water levels	1.3	2.8	OK
2	Flood water levels	1.1	2.3	OK
3	Normal water levels + Ice	1.3	2.5	OK
4	Normal water levels + Earthquake	1.1	2.8	OK

**The resultant location is relative to the overall base width.*

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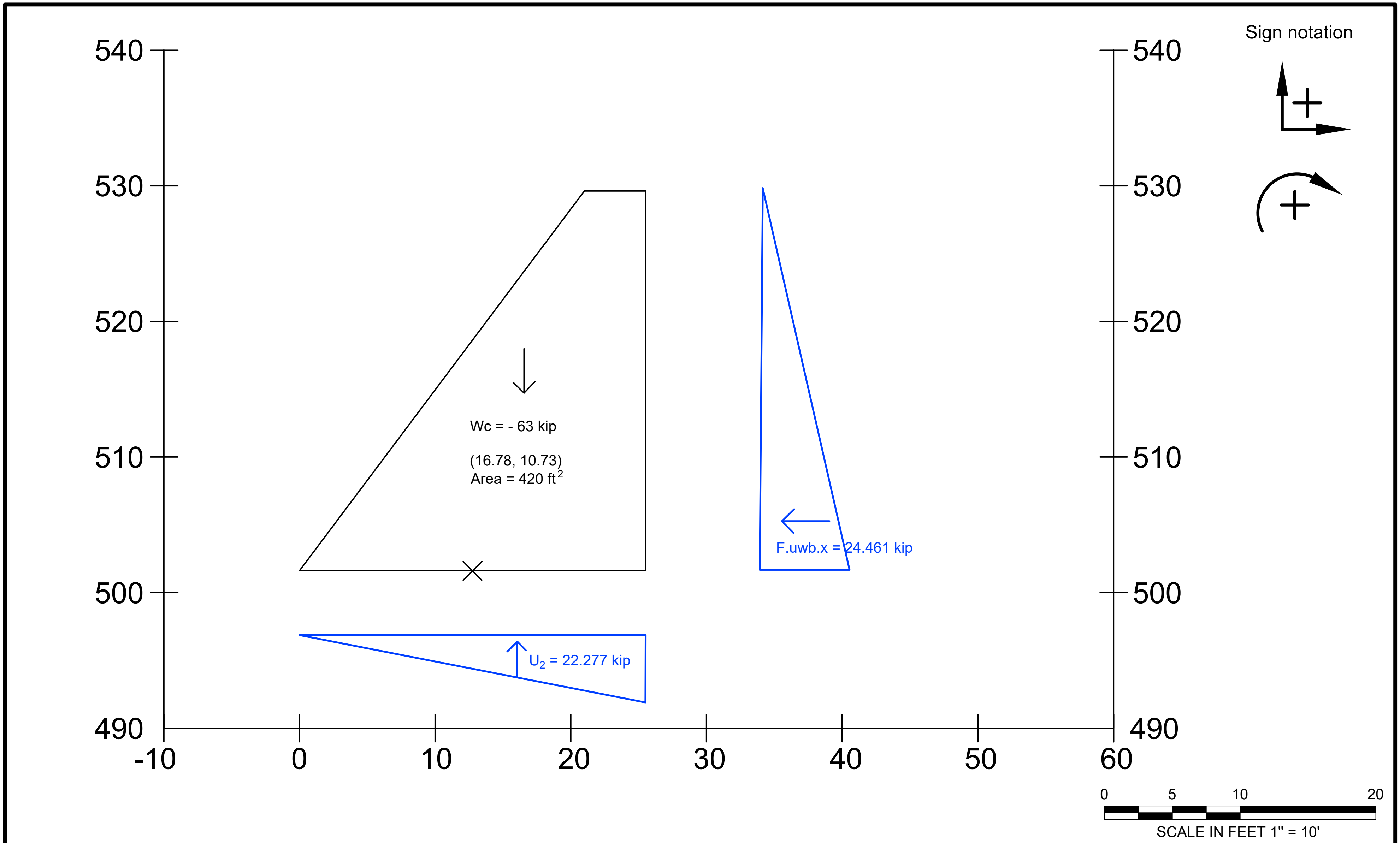

NO.	ISSUE/DESCRIPTION	BY	DATE

INDIAN BROOK RESERVOIR DAM

ESSEX, VT

SECTION B-B NORMAL POOL

PREPARED BY:
GZA GeoEnvironmental, Inc.
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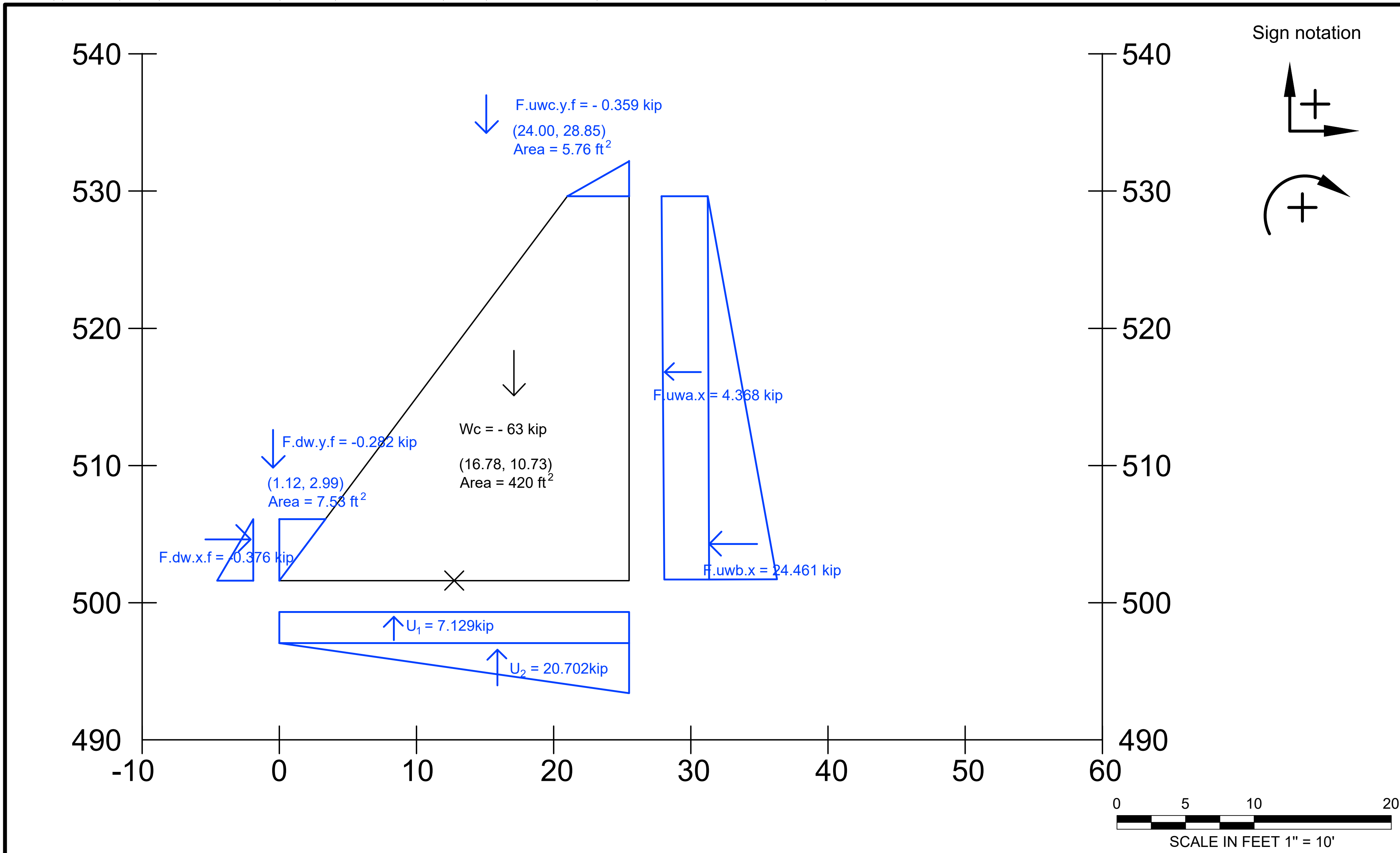
PREPARED FOR:
Vermont Department of Environmental Conservation

PROJ MGR: DJS	REVIEWED BY: JGD	CHECKED BY: JGD	FIGURE
DESIGNED BY: MZ	DRAWN BY: MZ	SCALE: 1"=10'	1
DATE: 3-9-2023	PROJECT NO. 01.0175988.00	REVISION NO.	SHEET NO. 1 OF 2

GZA–\\GZANOR\JOBS\170,000–179,999\175988\175988–00.DJS\WORK\STABILITY CALCS\06 INDIAN BROOK RESERVOIR DAM\INDIAN BROOK RESERVOIR DAM_CROSS–SECTION – FMR.DWG B–B PMF NOVEMBER 22, 2023 MENC



NO.	ISSUE/DESCRIPTION	BY	DATE

INDIAN BROOK RESERVOIR DAM

ESSEX, VT

SECTION B-B FLOOD (SDF) POOL

PREPARED BY: GZA GeoEnvironmental, Inc. www.gza.com

PREPARED FOR: Vermont Department of Environmental Conservation

PROJ MGR: DJS	REVIEWED BY: JGD	CHECKED BY: JGD	FIGURE
DESIGNED BY: MZ	DRAWN BY: MZ	SCALE: 1"=10'	2
DATE: 3-9-2023	PROJECT NO. 01.0175988.00	REVISION NO.	SHEET NO. 2 OF 2

Intermediate Failure Plane

(Unit Definition - Click Arrow to Expand)

Objective:

To perform a stability analysis of the **Indian Brook Reservoir Dam**, Intermediate failure plane at the spillway (along a lift line) using assumption of cracked base where applicable, to calculate factors of safety against sliding and to evaluate overturning stability.

Design Methodology:

- Evaluate stability using conventional equilibrium analyses and limit state theory
- Calculate base pressures with and without considering uplift effects
- Determine non-compression zone, where a cracked section is assumed to have developed and is assumed to be subjected to full headwater pressure.
- Uplift pressure profile is revised and a cracked length is obtained using an iterative solution per USACE methodology

References:

- "Evaluation of Concrete Dam Safety" by ASDSO. Northeast Regional Technical Seminar
- "Gravity Dam Design" by USACE EM1110-2-2200, Jun 1995
- "Stability Analysis of Concrete Structures" by USACE EM1110-2-2100, Dec 2005
- "Design of Small Dams" by US Bureau of Reclamation, 1977
- "Evaluation and comparison of stability analysis and uplift criteria for concrete gravity dams by three federal agencies" by USACE ERDC/ITL TR-00-1, Jun 2000

Case Descriptions - Loading Conditions:

(Per US Army Corps of Engineers)

Case #1: Normal water levels
Case #2: Flood (SDF) water levels
Case #3: Normal water levels + ice
Case #4: Normal water levels + earthquake

Assumptions:

- Full upstream hydraulic head applied to cracked length and is linearly interpolated to downstream hydraulic head over uncracked length (depending on efficiency and location of relief wells)
- Pseudostatic method for seismic analysis (apply horizontal acceleration as a % of g)
- Summation of moments about the centerline of the base of the dam
- Plane of analysis at dam/foundation interface (EL. 514.1)

Notes for MathCAD User (No Calculations)

Input Parameters

1) Dam Geometry:

Total Base width	$B := 16.5ft$	
Toe base width	$B_1 := 12ft$	
Heel width	$B_2 := 0ft$	
Crest width	$B_c := B - B_1 - B_2 = 4.5\,ft$	
Height of heel slope	$H_2 := 16ft$	
Analysis length	$LF := 1ft$	
Dam base elevation	$EL_b := 514.1ft$	
Dam crest elevation	$EL_c := 530.1ft$	
Dam height	$H_{dam} := EL_c - EL_b = 16\,ft$	
Upstream batter angle (from vertical)	$\theta_u := atan\left(\frac{B_2}{H_2}\right) = 0 \cdot °$	
Downstream batter angle (from vertical)	$\theta_d := atan\left(\frac{B_1}{H_{dam}}\right) = 36.87 \cdot °$	
Upstream fill elevation	$EL_{fillus} := EL_b = 514.1\,ft$	*(No upstream fill)*
Inclination angle of base	$\beta := 0deg$	
Sliding direction (upslope / downslope to DS)	$f_\beta := 1$	(+1 sloping down to DS; and -1 sloping up to DS)
Drainage Gallery base elevation	$EL_{dg} := EL_b = 514.1\,ft$	*(No drainage gallery in this section)*
Drain Effectiveness	$E_{dr} := 0\%$	(fully effective = 100%; ineffective = 0%)
Distance of drain to heel	$d_{dr} := 0ft$	*(set to zero if no drain installed)*
Shear Key Area	$A_{shear} := 0ft^2$	*(set to zero if no shear keys present)*
Shear Key Cohesion	$c_{shear} := 0psf$	*(set to zero if no shear keys present)*

1) Dam Geometry (continued): *Refer to Figure #1*

Assume coordinates of toe (0,0) *(Geometry based on CAD)*

W_c: Concrete cross-section

$X_{Centroid.c} := 10.68ft$ $Y_{Centroid.c} := 6.48ft$

$Area_c := 168ft^2$

$F_{uwa.y}$ dimensions: headwater above heel

$X_{Centroid.uwa.y} := 0ft$ $Y_{Centroid.uwa.y} := 0ft$

$Area_{uwa.y} := 0ft^2$

$F_{fillus.y}$ dimensions: fill above toe (upstream side)

$X_{Centroid.fillus.y} := 0ft$ $Y_{Centroid.fillus.y} := 0ft$

$Area_{fillus.y} := 0ft^2$

▶ N/A Fields

2) General Design Elevations

Flood pool elevation $EL_{fw} := 532.1ft$

Normal pool elevation $EL_{nw} := 529.6ft$ *(Assumed at the crest elevation of primary spillway)*

Assume NO silt $H_{silt} := 0ft$ *(No silt)*

Silt/sediment surface level $EL_s := 0ft$

Silt surface slope angle $\alpha := 0°$

3) Case Specific Loads, Dimensions and Elevations

Tailwater Elevation $EL_{dw} := \begin{pmatrix} 514.1ft \\ 514.1ft \\ 514.1ft \\ 514.1ft \end{pmatrix}$ Case #1, Case #2, Case #3, Case #4

Tailwater Height

$H_{dw} := EL_{dw} - EL_b$ $H_{dw} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} ft$ Case #1, Case #2, Case #3, Case #4

Headwater Elevation $EL_{uw} := \begin{pmatrix} EL_{nw} \\ EL_{fw} \\ EL_{nw} \\ EL_{nw} \end{pmatrix}$ Case #1, Case #2, Case #3, Case #4

Headwater Height

$H_{uw} := EL_{uw} - EL_b$ $H_{uw} = \begin{pmatrix} 15.5 \\ 18 \\ 15.5 \\ 15.5 \end{pmatrix} ft$ Case #1, Case #2, Case #3, Case #4

Ice thickness	$H_{ice} := 1ft$	*(initial assumption)*
Ice pressure	$f_{ice} := 5ksf$	*(per USACE)*
Horizontal Seismic Coefficient	$\lambda := 0.198$ % acceleration	*(Class B PGA for bedrock and a 2500-year return period per 2018 USGS data)*

4) Basic Material Properties

Cohesion along construction joint	$c_0 := 50psi$	*(Initial assumption)*
Interface friction angle		
Construction joint	$\phi_{dam} := 0°$	*(Initial assumption)*
Allowable foundation bearing capacity	$BC := 15ksf$	*(Initial assumption)*
Allowable compressive strength of dam:	$Cu_{dam} := 3000psi$	*(Initial assumption)*
Unit weight of dam material	$\gamma_{concrete} := 150pcf$	*(Based on analysis parameter from other project)*
Unit weight of Fill material	$\gamma_{fillus} := 0pcf$	*(No Upstream or Downstream Fill)*
	$\gamma_{fillds} := 0pcf$	
Fill internal frictional angle	$\phi_{fillus} := 0°$	
	$\phi_{fillds} := 0°$	

5) Design Factor of Safety (FS) against sliding

US Army Corps of Engineers

Case		Loading Conditions	Required Factor of Safety	
			Ordinary Site Info	Well Defined Site Info
1	Normal Pool	Usual	2	1.7
2	PMF	Extreme	1.1	1.1
3	Normal Pool + Ice	Unusual	2	1.7
4	Normal Pool + Seismic	Extreme	1.3	1.1

Factors of Safety Used in Analysis (Ordinary)

$$FS\_SLIDING_{min} := \begin{pmatrix} 2.0 \\ 1.1 \\ 2.0 \\ 1.3 \end{pmatrix}$$

Case #1
Case #2 - See Notes
Case #3
Case #4

Factor of Safety Notes
- PMF is considered "Extreme" due to return period of > 300 yr, in accordance with EM 1110-2-2100.

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▶ Misc, Input Parameter Notes (No Calculations)

LOAD CASE #1 - Normal Pool

I. INITIAL VERTICAL LOADS AND MOMENTS:

A. Dam self-weights - (Refer to FBD for dam geometry, variable notation, and sign convention)

Weight of individual Dam Sections

$$W_c := -Area_c \cdot \gamma_{concrete} \cdot LF = -25.2 \cdot kip$$

Moment arms about Center of Base:

$$D_c := X_{Centroid.c} - \frac{B}{2} = 2.43\,ft$$

Dam Weight Moment about centerline of dam:

$$M_c := W_c \cdot D_c = -61.236 \cdot kip \cdot ft$$

B. Headwater: (Vertical Component)

Conditional statement using variable H_water_over_crest
(*determines whether headwater higher than dam crest)*

$$H_{water\_over\_crest} := H_{uw_1} - H_{dam} = -0.5\,ft$$

Height of water above crest

$$H_{ov}(H_{water\_over\_crest}) := \begin{cases} H_{water\_over\_crest} & \text{if } H_{water\_over\_crest} > 0 \cdot ft \\ 0 & \text{otherwise} \end{cases}$$

$$H_{ov}(H_{water\_over\_crest}) = 0\,ft$$

Headwater weight on dam (if crest not overtopped, areas of b and c= 0)
a, b, c correspond to various areas of water over the dam, refer to FBD

Headwater Area A $\quad F_{uwa.y} := -\gamma_w \cdot Area_{uwa.y} \cdot LF = 0$

Headwater Area B $\quad F_{uwb.y} := 0kip$

Headwater Area C $\quad F_{uwc.y} := 0kip$

Σ Vertical components of Headwater over the Upstream face of the Dam

$$F_{uw.y} := F_{uwa.y} + F_{uwb.y} + F_{uwc.y} = 0$$

Moment arms of a, b & c about the Center of Base

Headwater Area A $\quad arm_{uwa.x} := X_{Centroid.uwa.y} - \frac{1}{2}B = -8.25\,ft$ $\quad$ Above heel

Headwater Area B $\quad arm_{uwb.x} := 0$ $\quad$ Above crest

Headwater Area C $\quad arm_{uwc.x} := 0$

Moments of vertical headwater forces a, b & c forces about centerline:

Headwater Area A $M_{uwa.y} := F_{uwa.y} \cdot arm_{uwa.x} = 0 \cdot kip \cdot ft$

Headwater Area B $M_{uwb.y} := F_{uwb.y} \cdot arm_{uwb.x} = 0 \cdot kip \cdot ft$

Headwater Area C $M_{uwc.y} := F_{uwc.y} \cdot arm_{uwc.x} = 0 \cdot kip \cdot ft$

Σ Moments from the vertical component of Headwater on U/S face of the Dam

$$M_{uw.y} := M_{uwa.y} + M_{uwb.y} + M_{uwc.y} = 0 \cdot kip \cdot ft$$

C. Tailwater: (Vertical Component)

No vertical tailwater loads appiled.

D. Soil Loads: (Vertical Component due to Weight)

No upstream or downstream fill.

E. Uplift Pressure:

▶ Drainage Gallery related calculation - Click to expand *(No drainage gallery)*

Head at heel $H_{heel} := H_{uw_1} = 15.5\,ft$

Head at toe $H_{toe} := H_{dw_1} = 0\,ft$ *(Ignore the tailwater here)*

Uplift Pressure at Heel $u_{up\_us} := \left(H_{heel}\right) \cdot \gamma_w = 0.967 \cdot ksf$

Uplift Pressure at Toe $u_{up\_ds} := H_{toe} \cdot \gamma_w = 0 \cdot ksf$

Uplift Pressure below Drainage Gallery $u_{up\_dg} := H_{dr} \cdot \gamma_w = 0.967 \cdot ksf$

Uplift forces below dam base:
Refer to FBD for notation:

$$U_1 := u_{up\_ds} \cdot B \cdot LF = 0 \cdot kip$$

$$U_2 := \frac{1}{2}\left(u_{up\_dg} - u_{up\_ds}\right) \cdot B \cdot LF = 7.979 \cdot kip$$

Σ Uplift Forces = $U := U_1 + U_2 = 7.979 \cdot kip$

Moment arms of Uplift Forces

$$d_{up1} := \frac{1}{2}B - \frac{1}{2}B = 0\,ft$$

$$d_{up2} := \frac{2}{3}B - \frac{1}{2}B = 2.75\,ft$$

Moments due to Uplift Components $M_{up1} := U_1 \cdot d_{up1} = 0 \cdot kip \cdot ft$

$$M_{up2} := U_2 \cdot d_{up2} = 21.943 \cdot kip \cdot ft$$

Σ **Uplift Moments =** $M_{up} := M_{up1} + M_{up2} = 21.943 \cdot kip \cdot ft$

II. INITIAL HORIZONTAL FORCES AND MOMENTS:

A. Headwater: (Horizontal Component)

Horizontal Component of Headwater on Dam

$$F_{uwa.x} := \begin{cases} \gamma_w \cdot \left(H_{uw_1} - H_{dam}\right) \cdot H_{dam} \cdot LF & \text{if } H_{ov}\left(H_{water\_over\_crest}\right) > 0 \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kip$$

$$F_{uwb.x} := \begin{cases} \frac{1}{2} \gamma_w \cdot H_{dam}{}^2 \cdot LF & \text{if } H_{ov}\left(H_{water\_over\_crest}\right) > 0 \\ \frac{1}{2} \gamma_w \cdot \left(H_{heel}\right)^2 \cdot LF & \text{otherwise} \end{cases} = 7.496 \cdot kip$$

Σ **Horizontal Forces by Headwater** $F_{uw.x} := F_{uwa.x} + F_{uwb.x} = 7.496\,kip$

Moment arms of Headwater on Dam

$$arm_{uwa.y} := \begin{cases} \frac{H_{dam}}{2} & \text{if } H_{ov}\left(H_{water\_over\_crest}\right) > 0 \\ 0 & \text{otherwise} \end{cases} = 0 \cdot ft$$

$$arm_{uwb.y} := \begin{cases} \frac{1}{3} H_{dam} & \text{if } H_{ov}\left(H_{water\_over\_crest}\right) > 0 \\ \frac{H_{heel}}{3} & \text{otherwise} \end{cases} = 5.167 \cdot ft$$

Moment due to Headwater on Dam

$$M_{uwa.x} := F_{uwa.x} \cdot arm_{uwa.y} = 0 \cdot kip \cdot ft$$

$$M_{uwb.x} := F_{uwb.x} \cdot arm_{uwb.y} = 38.728 \cdot kip \cdot ft$$

Σ **Moments due to Headwater** $M_{uw.x} := M_{uwa.x} + M_{uwb.x} = 38.728 \cdot kip \cdot ft$

B. Tailwater: (Horizontal Component)

Horizontal Force due to Tailwater $F_{dw.x} := \frac{-1}{2} \gamma_w \cdot \left(H_{dw_1}\right)^2 \cdot LF = 0\,kip$

Moment arm of Tailwater $arm_{dw.y} := \frac{1}{3} H_{dw_1} = 0\,ft$

Moment due to Tailwater $M_{dw.x} := F_{dw.x} \cdot arm_{dw.y} = 0\,ft \cdot kip$

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C. Soil horizontal loading on upstream side of dam:

Earth Pressure Coefficients, At Rest Condition

Angle of upstream embankment slope $\beta_{soil} := 0°$

$$K_{0fillus} := \left(1 - \sin\left(\phi_{fillus}\right)\right)\cdot\left(1 + \sin\left(\beta_{soil}\right)\right) = 1.00$$

Horizontal Force due to upstream fill

$$F_{fillus.x} := \frac{1}{2}\cdot K_{0fillus}\cdot\left(\gamma_{fillus} - \gamma_{w}\right)\cdot\left(EL_{fillus} - EL_{b}\right)^{2}\cdot LF = 0\ kip$$

Moment arm of fill

$$arm_{fillus.y} := \left(\frac{1}{3}\right)\left(EL_{fillus} - EL_{b}\right) = 0\ ft$$

Moment due to fill

$$M_{fillus.x} := F_{fillus.x}\cdot arm_{fillus.y} = 0\ ft\cdot kip$$

D. Soil horizontal loading on downstream side of dam:

No horizontail fill load applied on downsitream side

▶ N/A Fields

▶ (Summary of Vertical Forces Raw Data - Click to expand)

III. SUMMARY OF INITIAL LOADS AND MOMENTS - CASE #1

SUMMARY OF VERTICAL FORCES/MOMENTS

Component	Acting			Resisting		
	Force (kip)	Arm (ft)	Moment (kip-ft)	Force (kip)	Arm (ft)	Moment (kip-ft)
Weight of Concrete	-	-	-	-25.2	2.4	-61.2
Uplift Pressure #1	0.0	0.0	0.0	-	-	-
Uplift Pressure #2	8.0	2.8	21.9	-	-	-
Totals	**8.0**	**-**	**21.9**	**-25.2**	**-**	**-61.2**

Σ **Vertical Forces w/ uplift** $FV_{tot} := (F_{vr} + F_{va}) \cdot kips$ $FV_{tot} = -17.221 \cdot kips$

Σ **Vertical Moments w/ uplift** $MV_{tot} := (M_{vr} + M_{va}) \cdot kips \cdot ft$ $MV_{tot} = -39.3 \cdot kips \cdot ft$

(Summary of Horizontal Forces Raw Data - Click to expand)

SUMMARY OF HORIZONTAL FORCES/MOMENTS

Component	Acting			Resisting		
	Force (kip)	Arm (ft)	Moment (kip-ft)	Force (kip)	Arm (ft)	Moment (kip-ft)
Headwater on Dam, Fuwa.x	0.0	0.0	0.0	-	-	-
Headwater on Dam, Fuwb.x	7.5	5.2	38.7	-	-	-
Tailwater on Dam, Fdw.x	-	-	-	0.0	0.0	0.0
Totals	**7.5**	**-**	**38.7**	**0.0**	**-**	**0.0**

Σ **Horizontal Forces** $FH_{tot} := (F_{hr} + F_{ha}) \cdot kips$ $FH_{tot} = 7.496 \cdot kips$

Σ **Horizontal Moments** $MH_{tot} := (M_{hr} + M_{ha}) \cdot kips \cdot ft$ $MH_{tot} = 38.7 \cdot kips \cdot ft$

Σ **Moments (w/ uplift)** $M_{tot} := (M_{ha} + M_{hr} + M_{va} + M_{vr}) \cdot kips \cdot ft$ $M_{tot} = -0.6 \cdot kips \cdot ft$

IV. EVALUATE OVERTURNING AND BASE PRESSURES

- Check Resultant Location - COE EM1110-2-2200 Overturning Stability Criteria

- **Usual Conditions = Within middle third of the base**
- Unusual Conditions = Within middle half of the base
- Extreme Conditions = Within base

A. Calculate Eccentricity and Base Pressures

- Resultant and Eccentricity

- Eccentricity (from centroid of Base Area)

$$e_o := \frac{M_{tot}}{-FV_{tot}} = -0.033\,ft$$

(+) = D/S of Centroid
(-) = U/S of Centroid

- Resultant Location (from toe)

$$R_o := \frac{1}{2}B - e_o = 8.283\,ft$$

- Evaluate Overturning using Resultant Location

$$\textbf{Location}_{\textbf{Ro}_1} := \begin{cases} \text{"WITHIN MIDDLE 1/3"} & \text{if } \left(R_o \geq \frac{B}{3}\right) \wedge \left(R_o \leq \frac{2B}{3}\right) \\ \text{"OUTSIDE MIDDLE 1/3"} & \text{otherwise} \end{cases}$$

$\textbf{Location}_{\textbf{Ro}_1}$ = "WITHIN MIDDLE 1/3"

- Base Pressures (includes Uplift)

Base Pressure at Heel:

$$P_{us\_o} := \frac{-FV_{tot}}{B \cdot LF} \cdot \left(1 - \frac{6 \cdot e_o}{B}\right) = 1.056\,\text{ksf}$$

(-) = tension
(+) = compression

Base Pressure at Toe:

$$P_{ds\_o} := \frac{-FV_{tot}}{B \cdot LF} \cdot \left(1 + \frac{6 \cdot e_o}{B}\right) = 1.031\,\text{ksf}$$

(-) = tension
(+) = compression

C. Check if Cracked Base Analysis (CBA) is Required

(CBA is required when base pressure with uplift are negative, i.e. base in tension)

$$\textbf{CBA}_1 := \begin{cases} \text{"REQUIRED"} & \text{if } \left(P_{us\_o} < 0\right) \vee \left(P_{ds\_o} < 0\right) \\ \text{"NOT REQUIRED"} & \text{otherwise} \end{cases}$$

$\textbf{CBA}_1$ = "NOT REQUIRED"

(Note: if CBA not needed, do not edit Cracked Base Analysis Region

Cracked Base Analysis - Case #1 (Click to Expand, if Required)

Summary of Vertical forces - Click to expand

Revised summary table - Click to expand

Revised Parameters (Click to Expand)

Evaluate Sliding Stability (shear friction factor):

Base Inclination Angle $\beta = 0$

Base Area for Analysis $A_0 := B_{unc} \cdot LF = 16.5\,\text{ft}^2$

With no cohesion

$$FS\_SLIDING_1 := \frac{\left(-F_V \cdot \cos(\beta) - F_U + F_H \cdot \sin(\beta) \cdot f_\beta\right) \cdot \tan\left(\phi_{dam}\right) + \left(A_0 - A_{shear}\right) \cdot c_0 + A_{shear} \cdot c_{shear}}{F_H \cdot \cos(\beta) - F_V \cdot \sin(\beta) \cdot f_\beta} = 15.849$$

Calculated $FS_{sliding}$ $FS\_SLIDING_1 = 15.8$

Required Factor of Safety (from Page 5): $FS\_SLIDING_{min_1} = 2.0$

$$\textbf{Sliding\_Stability} := \begin{cases} \text{"SLIDING OK"} & \text{if } FS\_SLIDING_1 \geq FS\_SLIDING_{min_1} \\ \text{"NOT ADEQUATE"} & \text{otherwise} \end{cases}$$

$\textbf{Sliding\_Stability}$ = "SLIDING OK"

Re-Evaluate Overturning and Base Pressures

Final Resultant Location:

$$R_{rev} := \frac{1}{2} \cdot B_{unc} - e_{rev} = 8.283\,ft$$

(+) = U/S of toe
(-) = D/S of toe

$$\textbf{Location}_{\textbf{Rrev}_1} := \begin{cases} \text{"OK"} & \text{if } \left(R_{rev} \geq \frac{B_{unc}}{3}\right) \wedge \left(R_{rev} \leq \frac{2B_{unc}}{3}\right) \\ \text{"NG"} & \text{otherwise} \end{cases}$$

$$\textbf{Location}_{\textbf{Rrev}_1} = \text{"OK"}$$

- Base Pressure at Toe (includes Uplift)

$$\textbf{p}_{\textbf{toe}_1} := \begin{cases} P_{ds\_o} & \text{if } \textbf{CBA}_1 = \text{"NOT REQUIRED"} \\ \frac{-(F_V + F_U)}{B_{unc} \cdot LF} \cdot \left[1 + 6 \cdot \left(\frac{e_{rev}}{B_{unc}}\right)\right] & \text{otherwise} \end{cases}$$

$$\textbf{p}_{\textbf{toe}_1} = 1.0 \cdot ksf$$

Check to see if pressure exceeds foundation bearing capacity:

$$\textbf{BC}_1 := \begin{cases} \text{"OK"} & \text{if } \textbf{p}_{\textbf{toe}_1} < BC \\ \text{"EXCEEDED"} & \text{otherwise} \end{cases}$$

$$\textbf{BC}_1 = \text{"OK"}$$

Check to see if pressure exceeds dam compressive strength:

$$\textbf{Cu\_dam}_1 := \begin{cases} \text{"OK"} & \text{if } \textbf{p}_{\textbf{toe}_1} < 0.3 \cdot Cu_{dam} \\ \text{"EXCEEDED"} & \text{otherwise} \end{cases}$$

$$\textbf{Cu\_dam}_1 = \text{"OK"}$$

Check flotation:

$$FS\_FLOT_1 := \frac{-F_V}{F_U} = 3.158$$

Required Factor of Safety against Flotation (from USACE EM 1110-2-2100, Table 3-4):

$$FS\_FLOT_{min_1} := 1.3$$

$$\textbf{Flotation}_1 := \begin{cases} \text{"OK"} & \text{if } FS\_FLOT_1 \geq FS\_FLOT_{min_1} \\ \text{"NOT ADEQUATE"} & \text{otherwise} \end{cases}$$

$$\textbf{Flotation}_1 = \text{"OK"}$$

END OF LOAD CASE #1 ANALYSIS

LOAD CASE #2 - Flood Pool

I. INITIAL VERTICAL LOADS AND MOMENTS:

A. Dam self-weights - (Same as Load Case #1)

B. Headwater: (Vertical Component)

Conditional statement using variable H_water_over_crest
(*determines whether headwater higher than dam crest)*

$H_{water\_over\_crest} := H_{uw_2} - H_{dam} = 2\ ft$

Height of water above crest

$$H_{ov}(H_{water\_over\_crest}) := \begin{cases} H_{water\_over\_crest} & \text{if } H_{water\_over\_crest} > 0 \cdot ft \\ 0 & \text{otherwise} \end{cases}$$

$H_{ov}(H_{water\_over\_crest}) = 2\ ft$

Headwater weight on dam (if crest not overtopped, areas of a, b, and c = 0)
a, b, c correspond to various areas of water over the dam, refer to FBD

$F_{uwb.y}$ dimensions:

$X_{Centroid.uwb.y} := 0ft$ $\quad Y_{Centroid.uwb.y} := 0ft$

$Area_{uwb.y} := 0ft^2$

$F_{uwc.y}$ dimensions:

$X_{Centroid.uwc.y} := 15ft$ $\quad Y_{Centroid.uwc.y} := 16.66ft$

$Area_{uwc.y} := 4.47ft^2$

Headwater weight on dam

Headwater Area A $\quad F_{uwa.y.f} := -Area_{uwa.y} \cdot \gamma_w \cdot LF = 0$ $\quad$ Above heel

Headwater Area B $\quad F_{uwb.y.f} := -Area_{uwb.y} \cdot \gamma_w \cdot LF = 0$ $\quad$ Overtopping crest

Headwater Area C $\quad F_{uwc.y.f} := -Area_{uwc.y} \cdot \gamma_w \cdot LF = -0.279\ kip$

Σ Vertical components of Headwater over the Upstream face of the Dam

$$F_{uw.y.f} := F_{uwa.y.f} + F_{uwb.y.f} + F_{uwc.y.f} = -0.279\ kip$$

Moment arms of a, b & c about the Center of Base

Headwater Area A $\quad arm_{uwa.x.f} := X_{Centroid.uwa.y} - \frac{1}{2}B = -8.25\ ft$ $\quad$ Above heel

Headwater Area B $\quad arm_{uwb.x.f} := X_{Centroid.uwb.y} - \frac{1}{2}B = -8.25\,ft$ $\quad$ Overtopping crest

Headwater Area C $\quad arm_{uwc.x.f} := X_{Centroid.uwc.y} - \frac{1}{2}B = 6.75\,ft$

Moments of vertical headwater forces a, b & c forces about centerline:

Headwater Area A $\quad M_{uwa.y.f} := F_{uwa.y.f} \cdot arm_{uwa.x.f} = 0 \cdot kip \cdot ft$

Headwater Area B $\quad M_{uwb.y.f} := F_{uwb.y.f} \cdot arm_{uwb.x.f} = 0 \cdot kip \cdot ft$

Headwater Area C $\quad M_{uwc.y.f} := F_{uwc.y.f} \cdot arm_{uwc.x.f} = -1.883 \cdot kip \cdot ft$

Σ Moments from the vertical component of Headwater on U/S face of the Dam

$$M_{uw.y.f} := M_{uwa.y.f} + M_{uwb.y.f} + M_{uwc.y.f} = -1.883 \cdot kip \cdot ft$$

<u>C. Tailwater:</u> (Vertical Component)

Tailwater load $F_{dw.y.f}$ dimensions:

$X_{Centroid.dw.y.f} := 0ft \qquad Y_{Centroid.dw.y.f} := 0ft$

$Area_{dw.y.f} := 0ft^2$

Vertical component of Tailwater $\quad F_{dw.y.f} := -0.6 Area_{dw.y.f} \cdot \gamma_w \cdot LF = 0 \cdot kip$ $\quad$ *(Use 60% of tailwater force per USACE EM 1110-2-2200, Section 3-3.c.(3).(b))*

Moment arm of Tailwater weight about centerline of Base $\quad arm_{dw.x.f} := X_{Centroid.dw.y.f} - \frac{B}{2} = -8.25\,ft$

Moment due to Weight of Tailwater about centerline of Base $\quad M_{dw.y.f} := F_{dw.y.f} \cdot \left(arm_{dw.x.f}\right) = 0 \cdot kip \cdot ft$

<u>D. Soil Loads:</u> (Vertical Components) **-** Same as Load Case #1

<u>E. Uplift Pressure:</u>

Height of drainage gallery to plane of analysis $\quad H_{dg} := EL_{dg} - EL_b = 0\,ft$

Effective Hydraulic Head at Drainage Gallery, H_{dr}

$$H_{dr} := \begin{cases} (1 - E_{dr}) \cdot \left[\left(H_{uw_2} - H_{dw_2} \right) \frac{B - d_{dr}}{B} + H_{dw_2} - H_{dg} \right] + H_{dg} & \text{if } H_{dg} \ge H_{dw_2} \\ (1 - E_{dr}) \cdot \left(H_{uw_2} - H_{dw_2} \right) \cdot \frac{B - d_{dr}}{B} + H_{dw_2} & \text{otherwise} \end{cases} = 18 \cdot ft$$

based on drain effectiveness $\quad E_{dr} = 0 \cdot \%$

Head at heel $H_{heel} := H_{uw_2} = 18\,ft$

Head at toe $H_{toe} := H_{dw_2} = 0\,ft$

Uplift Pressure at Heel $u_{up\_us} := H_{heel} \cdot \gamma_w = 1.123 \cdot ksf$

Uplift Pressure at Toe $u_{up\_ds} := H_{toe} \cdot \gamma_w = 0 \cdot ksf$

Uplift Pressure below Drainage Gallery $u_{up\_dg} := H_{dr} \cdot \gamma_w = 1.123 \cdot ksf$

Uplift forces below dam base:
Refer to FBD for notation:

$U_1 := u_{up\_ds} \cdot B \cdot LF = 0 \cdot kip$

$U_2 := \frac{1}{2}\left(u_{up\_dg} - u_{up\_ds}\right) \cdot B \cdot LF = 9.266 \cdot kip$

Σ **Uplift Forces =** $U := U_1 + U_2 = 9.266 \cdot kip$

Moment arms of Uplift Forces

$d_{up1} := \frac{1}{2}B - \frac{1}{2}B = 0\,ft$

$d_{up2} := \frac{2}{3}B - \frac{1}{2}B = 2.75\,ft$

Moments due to Uplift Components

$M_{up1} := U_1 \cdot d_{up1} = 0 \cdot kip \cdot ft$

$M_{up2} := U_2 \cdot d_{up2} = 25.483 \cdot kip \cdot ft$

Σ **Uplift Moments =** $M_{up} := M_{up1} + M_{up2} = 25.483 \cdot kip \cdot ft$

II. INITIAL HORIZONTAL FORCES AND MOMENTS:

A. Headwater: (Horizontal Component)

Horizontal Component of Headwater on Dam

$$F_{uwa.x} := \begin{cases} \gamma_w \cdot \left(H_{uw_2} - H_{dam}\right) \cdot H_{dam} \cdot LF & \text{if } H_{ov}\left(H_{water\_over\_crest}\right) > 0 \\ 0 & \text{otherwise} \end{cases} = 1.997 \cdot kips$$

$$F_{uwb.x} := \begin{cases} \frac{1}{2}\gamma_w \cdot H_{dam}^{\ 2} \cdot LF & \text{if } H_{ov}\left(H_{water\_over\_crest}\right) > 0 \\ \frac{1}{2}\gamma_w \cdot \left(H_{dam}\right)^2 \cdot LF & \text{otherwise} \end{cases} = 7.987 \cdot kips$$

Σ **Horizontal Forces by Headwater** $F_{uw.x} := F_{uwa.x} + F_{uwb.x} = 9.984\,kip$

Moment arms of Headwater on Dam

$$arm_{uwa.y} := \begin{cases} \frac{H_{dam}}{2} & \text{if } H_{ov}\left(H_{water\_over\_crest}\right) > 0 \\ 0 & \text{otherwise} \end{cases} = 8 \cdot ft$$

$$arm_{uwb.y} := \begin{cases} \frac{1}{3}H_{dam} & \text{if } H_{ov}(H_{water\_over\_crest}) > 0 \\ \frac{1}{3}H_{uw_2} & \text{otherwise} \end{cases} = 5.333 \cdot ft$$

Moment of Headwater on Dam

$$M_{uwa.x} := F_{uwa.x} \cdot arm_{uwa.y} = 15.974 \cdot kip \cdot ft$$

$$M_{uwb.x} := F_{uwb.x} \cdot arm_{uwb.y} = 42.598 \cdot kip \cdot ft$$

Σ Moments due to Headwater

$$M_{uw.x} := M_{uwa.x} + M_{uwb.x} = 58.573 \cdot kip \cdot ft$$

B. Tailwater: (Horizontal Component)

Horizontal Force due to Tailwater

$$F_{dw.x.f} := \frac{-1}{2} 0.6 \gamma_w \cdot \left(H_{dw_2}\right)^2 \cdot LF = 0\ kip$$

(Use 60% of tailwater force per USACE EM 1110-2-2200, Section 3-3.c.(3).(b))

Moment arm of Tailwater

$$arm_{dw.y.f} := \frac{1}{3} H_{dw_2} = 0\ ft$$

Moment due to Tailwater

$$M_{dw.x.f} := F_{dw.x.f} \cdot arm_{dw.y.f} = 0\ ft \cdot kip$$

C. Soil horizontal loading on upstream side of dam:- Same as Load Case #1

D. Soil horizontal loading on downstream side of dam: - Same as Load Case #1

▶ (Summary of Vertical Forces Raw Data - Click to expand)

III. SUMMARY OF INITIAL LOADS AND MOMENTS - CASE #2

SUMMARY OF VERTICAL FORCES/MOMENTS

Component	Acting			Resisting		
	Force (kip)	Arm (ft)	Moment (kip-ft)	Force (kip)	Arm (ft)	Moment (kip-ft)
Weight of Concrete	-	-	-	-25.2	2.4	-61.2
Headwater over Dam, Fuwa.y	-	-	-	0.0	-8.3	0.0
Headwater over Dam, Fuwb.y	-	-	-	0.0	-8.3	0.0
Headwater over Dam, Fuwc.y	-	-	-	-0.3	6.8	-1.9
Uplift Pressure, Area #1	0.0	0.0	0.0			
Uplift Pressure, Area #2	9.3	2.8	25.5			
Totals	**9.3**	**-**	**25.5**	**-25.5**	**-**	**-63.1**

Σ **Vertical Forces w/ uplift** $FV_{tot} := (F_{vr} + F_{va}) \cdot kips$ $FV_{tot} = -16.213 \cdot kips$

Σ **Vertical Moments w/ uplift** $MV_{tot} := (M_{vr} + M_{va}) \cdot kips \cdot ft$ $MV_{tot} = -37.6 \cdot kips \cdot ft$

(Summary of Horizontal Forces Raw Data - Click to expand)

SUMMARY OF HORIZONTAL FORCES/MOMENTS

Component	Acting			Resisting		
	Force (kip)	Arm (ft)	Moment (kip-ft)	Force (kip)	Arm (ft)	Moment (kip-ft)
Headwater on Dam, Fuwa.x	2.0	8.0	16.0	-	-	-
Headwater on Dam, Fuwb.x	8.0	5.3	42.6	-	-	-
Tailwater on Dam, Fdw.x	-	-	-	0.0	0.0	0.0
Totals	**10.0**	**-**	**58.6**	**0.0**	**-**	**0.0**

Σ **Horizontal Forces** $FH_{tot} := (F_{hr} + F_{ha}) \cdot kips$ $FH_{tot} = 9.984 \cdot kips$

Σ **Horizontal Moments** $MH_{tot} := (M_{hr} + M_{ha}) \cdot kips \cdot ft$ $MH_{tot} = 58.6 \cdot kips \cdot ft$

Σ **Moments (w/ uplift)** $M_{tot} := (M_{ha} + M_{hr} + M_{va} + M_{vr}) \cdot kips \cdot ft$ $M_{tot} = 20.9 \cdot kips \cdot ft$

IV. EVALUATE OVERTURNING AND BASE PRESSURES

- Check Resultant Location - COE EM1110-2-2200 Overturning Stability Criteria

- Usual Conditions = Within middle third of the base
- Unusual Conditions = Within middle half of the base
- **Extreme Conditions = Within base**

A. Calculate Eccentricity and Base Pressures

- Resultant and Eccentricity

- Eccentricity (from centroid of Base Area)

$$e_o := \frac{M_{tot}}{-FV_{tot}} = 1.291\,ft$$

(+) = D/S of Centroid
(-) = U/S of Centroid

- Resultant Location (from toe)

$$R_o := \frac{1}{2}B - e_o = 6.959\,ft$$

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Engineers and Scientists

- Evaluate Overturning using Resultant Location

$$\text{Location}_{Ro_2} := \begin{cases} \text{"WITHIN BASE"} & \text{if } (R_o \ge 0) \wedge (R_o \le B) \\ \text{"OUTSIDE BASE"} & \text{otherwise} \end{cases}$$

Location_{Ro_2} = "WITHIN BASE"

- Base Pressures (includes Uplift)

Base Pressure at Heel:

$$P_{us\_o} := \frac{-FV_{tot}}{B \cdot LF} \cdot \left(1 - \frac{6 \cdot e_o}{B}\right) = 0.521\,\text{ksf}$$

(-) = tension
(+) = compression

Base Pressure at Toe:

$$P_{ds\_o} := \frac{-FV_{tot}}{B \cdot LF} \cdot \left(1 + \frac{6 \cdot e_o}{B}\right) = 1.444\,\text{ksf}$$

(-) = tension
(+) = compression

C. Check if Cracked Base Analysis (CBA) is Required

(CBA is required when base pressure with uplift are negative, i.e. base in tension)

$$\text{CBA}_2 := \begin{cases} \text{"REQUIRED"} & \text{if } (P_{us\_o} < 0) \vee (P_{ds\_o} < 0) \\ \text{"NOT REQUIRED"} & \text{otherwise} \end{cases}$$

CBA_2 = "NOT REQUIRED"

(Note: if CBA not needed, skip cracked base analysis section and move on to Factor of Safety calculation)

Cracked Base Analysis - Case #2 (Click to Expand, if Required)

Summary of Vertical Forces - Click to Expand

Revised Summary Table - Click to Expand

Revised Parameters (Click to Expand)

Evaluate Sliding Stability (shear friction factor):

Base Inclination Angle $\beta = 0$

Base Area for Analysis $A_0 := B_{unc} \cdot LF = 16.5\,\text{ft}^2$

With no cohesion

$$FS\_SLIDING_2 := \frac{(-F_V \cdot \cos(\beta) - F_U + F_H \cdot \sin(\beta) \cdot f_\beta) \cdot \tan(\phi_{dam}) + (A_0 - A_{shear}) \cdot c_0 + A_{shear} \cdot c_{shear}}{F_H \cdot \cos(\beta) - F_V \cdot \sin(\beta) \cdot f_\beta} = 11.899$$

Calculated $FS_{sliding}$ $FS\_SLIDING_2 = 11.9$

Required Factor of Safety (from Page 5): $FS\_SLIDING_{min_2} = 1.1$

$$\text{Sliding\_Stability} := \begin{cases} \text{"SLIDING OK"} & \text{if } FS\_SLIDING_2 \ge FS\_SLIDING_{min_2} \\ \text{"NOT ADEQUATE"} & \text{otherwise} \end{cases}$$

Sliding_Stability = "SLIDING OK"

Re-Evaluate Overturning and Base Pressures

Final Resultant Location:

$$R_{rev} := \frac{1}{2} \cdot B_{unc} - e_{rev} = 6.959\,ft$$

(+) = U/S of toe
(-) = D/S of toe

$$Location_{Rrev_2} := \begin{cases} \text{"OK"} & \text{if } (R_{rev} \geq 0) \wedge (R_{rev} \leq B_{unc}) \\ \text{"NG"} & \text{otherwise} \end{cases}$$

$$Location_{Rrev_2} = \text{"OK"}$$

- Base Pressure at Toe (includes Uplift)

$$p_{toe_2} := \begin{cases} P_{ds\_o} & \text{if } CBA_2 = \text{"NOT REQUIRED"} \\ \frac{-(F_V + F_U)}{B_{unc} \cdot LF} \cdot \left[1 + 6 \cdot \left(\frac{e_{rev}}{B_{unc}}\right)\right] & \text{otherwise} \end{cases}$$

$$p_{toe_2} = 1.4 \cdot ksf$$

Check to see if pressure exceeds foundation bearing capacity:

$$BC_2 := \begin{cases} \text{"OK"} & \text{if } p_{toe_2} < 1.33BC \\ \text{"EXCEEDED"} & \text{otherwise} \end{cases}$$

$$BC_2 = \text{"OK"}$$

Check to see if pressure exceeds dam compressive strength:

$$Cu\_dam_2 := \begin{cases} \text{"OK"} & \text{if } p_{toe_2} < 0.9 \cdot Cu_{dam} \\ \text{"EXCEEDED"} & \text{otherwise} \end{cases}$$

$$Cu\_dam_2 = \text{"OK"}$$

Check flotation:

$$FS\_FLOT_2 := \frac{-F_V}{F_U} = 2.75$$

Required Factor of Safety against Flotation (from USACE EM 1110-2-2100, Table 3-4):

$$FS\_FLOT_{min_2} := 1.1$$

$$Flotation_2 := \begin{cases} \text{"OK"} & \text{if } FS\_FLOT_2 \geq FS\_FLOT_{min_2} \\ \text{"NOT ADEQUATE"} & \text{otherwise} \end{cases}$$

$$Flotation_2 = \text{"OK"}$$

END OF LOAD CASE #2 ANALYSIS

LOAD CASE #3 - Normal Pool + Ice

I. INITIAL VERTICAL LOADS AND MOMENTS:

A. Dam self-weights - Same as Load Case #1

B. Headwater: (Vertical Component) - Same as Load Case #1

Conditional statement using variable H_water_over_crest (*determines whether headwater higher than dam crest)*

$$H_{water\_over\_crest} := H_{uw_3} - H_{dam} = -0.5\,ft$$

Height of water above crest

$$H_{ov}(H_{water\_over\_crest}) := \begin{cases} H_{water\_over\_crest} & \text{if } H_{water\_over\_crest} > 0 \cdot ft \\ 0 & \text{otherwise} \end{cases}$$

$$H_{ov}(H_{water\_over\_crest}) = 0\,ft$$

C. Tailwater: (Vertical Component) - Same as Load Case #1

D. Soil Loads: (Vertical Components) - Same as Load Case #1

E. Uplift Pressure:

Height of drainage gallery to plane of analysis $H_{dg} := EL_{dg} - EL_b = 0\,ft$

Effective Hydraulic Head at Drainage Gallery, H_{dr}

$$H_{dr} := \begin{cases} (1 - E_{dr}) \cdot \left[\left(H_{uw_3} - H_{dw_3}\right) \dfrac{B - d_{dr}}{B} + H_{dw_3} - H_{dg} \right] + H_{dg} & \text{if } H_{dg} \ge H_{dw_3} \\ (1 - E_{dr}) \cdot \left(H_{uw_3} - H_{dw_3}\right) \cdot \dfrac{B - d_{dr}}{B} + H_{dw_3} & \text{otherwise} \end{cases} = 15.5 \cdot ft$$

based on drain effectiveness $E_{dr} = 0 \cdot \%$

Head at heel $H_{heel} := H_{uw_3} = 15.5\,ft$

Head at toe $H_{toe} := H_{dw_3} = 0\,ft$

Uplift Pressure at Heel $u_{up\_us} := H_{uw_3} \cdot \gamma_w = 0.967 \cdot ksf$

Uplift Pressure at Toe $u_{up\_ds} := H_{toe} \cdot \gamma_w = 0 \cdot ksf$

Uplift Pressure below Drainage Gallery $u_{up\_dg} := H_{dr} \cdot \gamma_w = 0.967 \cdot ksf$

Uplift forces below dam base:
Refer to FBD for notation:

$$U_1 := u_{up\_ds} \cdot B \cdot LF = 0 \cdot kip$$

$$U_2 := \frac{1}{2}\left(u_{up\_dg} - u_{up\_ds}\right) \cdot B \cdot LF = 7.979 \cdot kip$$

Σ Uplift Forces = $U := U_1 + U_2 = 7.979 \cdot kip$

Moment arms of Uplift Forces

$$d_{up1} := \frac{1}{2}B - \frac{1}{2}B = 0\,ft$$

$$d_{up2} := \frac{2}{3}B - \frac{1}{2}B = 2.75\,ft$$

Moments due to Uplift Components

$$M_{up1} := U_1 \cdot d_{up1} = 0 \cdot kip \cdot ft$$

$$M_{up2} := U_2 \cdot d_{up2} = 21.943 \cdot kip \cdot ft$$

Σ Uplift Moments = $M_{up} := M_{up1} + M_{up2} = 21.943 \cdot kip \cdot ft$

II. INITIAL HORIZONTAL FORCES AND MOMENTS:

A. Headwater: (Horizontal Component)

Horizontal Component of Headwater on Dam

$$F_{uwa.x} := \begin{cases} \gamma_w \cdot \left(H_{uw_3} - H_{dam}\right) \cdot H_{dam} \cdot LF & \text{if } H_{ov}\left(H_{water\_over\_crest}\right) > 0 \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kips$$

$$F_{uwb.x} := \begin{cases} \frac{1}{2}\gamma_w \cdot H_{dam}^{\ 2} \cdot LF & \text{if } H_{ov}\left(H_{water\_over\_crest}\right) > 0 \\ \frac{1}{2}\gamma_w \cdot \left(H_{uw_3}\right)^2 \cdot LF & \text{otherwise} \end{cases} = 7.496 \cdot kips$$

Σ Horizontal Forces by Headwater $F_{uw.x} := F_{uwa.x} + F_{uwb.x} = 7.496\,kip$

Moment arms of Headwater on Dam

$$arm_{uwa.y} := \begin{cases} \frac{H_{dam}}{2} & \text{if } H_{ov}\left(H_{water\_over\_crest}\right) > 0 \\ 0 & \text{otherwise} \end{cases} = 0 \cdot ft$$

$$arm_{uwb.y} := \begin{cases} \frac{1}{3}H_{dam} & \text{if } H_{ov}\left(H_{water\_over\_crest}\right) > 0 \\ \frac{1}{3}H_{uw_3} & \text{otherwise} \end{cases} = 5.167 \cdot ft$$

Moment of Headwater on Dam

$$M_{uwa.x} := F_{uwa.x} \cdot arm_{uwa.y} = 0 \cdot kip \cdot ft$$

$$M_{uwb.x} := F_{uwb.x} \cdot arm_{uwb.y} = 38.728 \cdot kip \cdot ft$$

Σ Moments due to Headwater $M_{uw.x} := M_{uwa.x} + M_{uwb.x} = 38.728 \cdot kip \cdot ft$

B. Tailwater: (Horizontal Component) - Same as Load Case #1

C. Soil horizontal loading on upstream side of dam: - Same as Load Case #1

D. Soil horizontal loading on downstream side of dam: - Same as Load Case #1

E. Ice Loading

Horizontal Ice Force $F_{ice.x} := H_{ice} \cdot (f_{ice}) \cdot LF = 5\,kip$

Moment Arm of Ice Force $arm_{ice.y} := (H_{uw_3} - 0.5 \cdot H_{ice}) = 15\,ft$

Moment due to Ice Force $M_{ice.x} := F_{ice.x} \cdot arm_{ice.y} = 75\,ft \cdot kip$

(Summary of Vertical Forces Raw Data - Click to expand)

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III. SUMMARY OF INITIAL LOADS AND MOMENTS - CASE #3

SUMMARY OF VERTICAL FORCES/MOMENTS

Component	Acting			Resisting		
	Force (kip)	Arm (ft)	Moment (kip-ft)	Force (kip)	Arm (ft)	Moment (kip-ft)
Weight of Concrete	-	-	-	-25.2	2.4	-61.2
Uplift Pressure #1	0.0	0.0	0.0			
Uplift Pressure #2	8.0	2.8	21.9	-	-	-
Totals	**8.0**	**-**	**21.9**	**-25.2**	**-**	**-61.2**

Σ **Vertical Forces w/ uplift** $FV_{tot} := (F_{vr} + F_{va}) \cdot kips$ $FV_{tot} = -17.221 \cdot kips$

Σ **Vertical Moments w/ uplift** $MV_{tot} := (M_{vr} + M_{va}) \cdot kips \cdot ft$ $MV_{tot} = -39.3 \cdot kips \cdot ft$

(Summary of Horizontal Forces Raw Data - Click to expand)

SUMMARY OF HORIZONTAL FORCES/MOMENTS

Component	Acting			Resisting		
	Force (kip)	Arm (ft)	Moment (kip-ft)	Force (kip)	Arm (ft)	Moment (kip-ft)
Headwater on Dam, Fuwa.x	0.0	0.0	0.0	-	-	-
Headwater on Dam, Fuwb.x	7.5	5.2	38.7	-	-	-
Ice Force	5.0	15.0	75.0	-	-	-
Tailwater on Dam, Fdw.x	-	-	-	0.0	0.0	0.0
Totals	**12.5**	**-**	**113.7**	**0.0**	**-**	**0.0**

Σ **Horizontal Forces** $FH_{tot} := (F_{hr} + F_{ha}) \cdot kips$ $FH_{tot} = 12.496 \cdot kips$

Σ **Horizontal Moments** $MH_{tot} := (M_{hr} + M_{ha}) \cdot kips \cdot ft$ $MH_{tot} = 113.7 \cdot kips \cdot ft$

Σ **Moments (w/ uplift)** $M_{tot} := (M_{ha} + M_{hr} + M_{va} + M_{vr}) \cdot kips \cdot ft$ $M_{tot} = 74.4 \cdot kips \cdot ft$

IV. EVALUATE OVERTURNING AND BASE PRESSURES

- Check Resultant Location - COE EM1110-2-2200 Overturning Stability Criteria

- Usual Conditions = Within middle third of the base
- **Unusual Conditions = Within middle half of the base**
- Extreme Conditions = Within base

A. Calculate Eccentricity and Base Pressures

- Resultant and Eccentricity

- Eccentricity
(from centroid of Base Area)

$$e_o := \frac{M_{tot}}{-FV_{tot}} = 4.322\,ft$$

(+) = D/S of Centroid
(-) = U/S of Centroid

- Resultant Location (from toe)

$$R_o := \frac{1}{2}B - e_o = 3.928\,ft$$

- Evaluate Overturning using Resultant Location

$$\text{Location}_{Ro_3} := \begin{cases} \text{"WITHIN MIDDLE 1/2"} & \text{if } \left(R_o \geq \frac{B}{4}\right) \wedge \left(R_o \leq \frac{3}{4}B\right) \\ \text{"OUTSIDE MIDDLE 1/2"} & \text{otherwise} \end{cases}$$

Location_{Ro_3} = "OUTSIDE MIDDLE 1/2"

- Base Pressures (includes Uplift)

Base Pressure at Heel:

$$P_{us\_o} := \frac{-FV_{tot}}{B \cdot LF} \cdot \left(1 - \frac{6 \cdot e_o}{B}\right) = -0.597\,\text{ksf}$$

(-) = tension
(+) = compression

Base Pressure at Toe:

$$P_{ds\_o} := \frac{-FV_{tot}}{B \cdot LF} \cdot \left(1 + \frac{6 \cdot e_o}{B}\right) = 2.684\,\text{ksf}$$

(-) = tension
(+) = compression

C. Check if Cracked Base Analysis (CBA) is Required

(CBA is required when base pressure with uplift are negative, i.e. base in tension)

$$\text{CBA}_3 := \begin{cases} \text{"REQUIRED"} & \text{if } \left(P_{us\_o} < 0\right) \vee \left(P_{ds\_o} < 0\right) \\ \text{"NOT REQUIRED"} & \text{otherwise} \end{cases}$$

CBA_3 = "REQUIRED"

(Note: if CBA not needed, do not edit Cracked Base Analysis Region

Cracked Base Analysis - Case #3 (Click to Expand, if Required)

Cracked Base Analysis - Case #3

Estimate Initial Trial Uncracked Base Length

$$x_{0\_tr} := \frac{-B^2}{12e_o} = -5.249\,\text{ft} \qquad B_{NEW} := \frac{1}{2} \cdot B - x_{0\_tr} = 13.499\,\text{ft}$$

****** ****** ****** ****** ****** ****** ****** ****** ****** Begin Iteration ****** ****** ****** ****** ****** ****** ****** ****** ******

(Manually change uncracked length B_{tr} to achieve D <=0.5%, use B_{NEW}, above, as initial input)

Trial uncracked length: $B_{tr} := 12.55\,\text{ft}$

Corresponding cracked length: $T_{cr} := B - B_{tr} = 3.95\,\text{ft}$

Re-evaluate Vertical Moments based on CBA Per USACE ERDC/ITL TR-00-1

A. Dam self-weights - (Refer to FBD for dam geometry)

Moment arms about Center of Uncracked Base:

$$D_{1.xcr} := X_{Centroid.c} - \frac{B_{tr}}{2} = 4.405\,\text{ft}$$

Moments of Dam Weights about Center of Uncracked Base:

$$M_{W1.ycr} := W_c \cdot D_{1.xcr} = -111.006 \cdot \text{kip} \cdot \text{ft}$$

Σ Dam Weight Moments about Center of Uncracked Base

$$M_{damCR} := M_{W1.ycr} = -111.006 \cdot kip \cdot ft$$

B. Headwater: (Vertical Component)

Moment arms of a, b & c about the Center of Uncracked Base

$$arm_{uwa.xcr} := X_{Centroid.uwa.y} - \frac{1}{2}B = -8.25\,ft$$

$$arm_{uwb.xcr} := X_{Centroid.uwb.y} - \frac{1}{2}B = -8.25\,ft$$

$$arm_{uwc.xcr} := X_{Centroid.uwc.y} - \frac{1}{2}B = 6.75\,ft$$

Moments of vertical forces from a, b & c

$$M_{uwa.ycr} := F_{uwa.y} \cdot arm_{uwa.xcr} = 0 \cdot kip \cdot ft$$

$$M_{uwb.ycr} := F_{uwb.y} \cdot arm_{uwb.xcr} = 0 \cdot kip \cdot ft$$

$$M_{uwc.ycr} := F_{uwc.y} \cdot arm_{uwc.xcr} = 0 \cdot kip \cdot ft$$

Σ Moments from the vertical component of Headwater on U/S face of the Dam

$$M_{uw.yCR} := M_{uwa.ycr} + M_{uwb.ycr} + M_{uwc.ycr} = 0 \cdot kip \cdot ft$$

C. Tailwater: (Vertical Component)

D. Soil Loads: (Vertical Component due to Weight)

E. Re-evaluate Uplift Profile based on Trial Crack Length

Trial Crack Length $T_{cr} = 3.95\,ft$

Distance from Heel to Drain Gallery $d_{dr} = 0$

Revise H_{dr_1} based on trial uncracked base length

$$H_{dr\_1} := \begin{cases} (1 - E_{dr}) \cdot \left[\left(H_{uw_1} - H_{dw_1}\right) \dfrac{B - d_{dr}}{B - T_{cr}} + H_{dw_1} - H_{dg} \right] + H_{dg} & \text{if } H_{dg} > H_{dw_1} \\ (1 - E_{dr}) \cdot \left(H_{uw_1} - H_{dw_1}\right) \cdot \dfrac{B - d_{dr}}{B - T_{cr}} + H_{dw_1} & \text{otherwise} \end{cases} = 20.378 \cdot ft$$

Revise H_{dr_1} based on relationship between crack length and distance from heel to drains

$$H_{dr\_1} := \begin{cases} H_{uw_1} & \text{if } T_{cr} > d_{dr} \\ H_{dr\_1} & \text{otherwise} \end{cases} = 15.5 \cdot ft$$

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Revised Uplift Pressure at the Drainage Gallery

$$u_{up\_dg\_1} := H_{dr\_1} \cdot \gamma_w = 0.967\,\text{ksf}$$

Revised Uplift Forces Based on Trial Cracked Length:

$$U_{tr} := \begin{bmatrix} \begin{cases} u_{up\_ds} \cdot (B - d_{dr}) \cdot LF & \text{if } d_{dr} > T_{cr} \\ u_{up\_ds} \cdot B_{tr} \cdot LF & \text{otherwise} \end{cases} \\ \begin{cases} \frac{1}{2}(u_{up\_dg\_1} - u_{up\_ds}) \cdot (B - d_{dr}) \cdot LF & \text{if } d_{dr} > T_{cr} \\ \frac{1}{2}(u_{up\_us} - u_{up\_ds}) \cdot B_{tr} \cdot LF & \text{otherwise} \end{cases} \\ \begin{cases} (u_{up\_dg\_1}) \cdot (d_{dr} - T_{cr}) \cdot LF & \text{if } d_{dr} > T_{cr} \\ 0 & \text{otherwise} \end{cases} \\ \begin{cases} \frac{1}{2}(u_{up\_us} - u_{up\_dg\_1}) \cdot (d_{dr} - T_{cr}) \cdot LF & \text{if } d_{dr} > T_{cr} \\ 0 & \text{otherwise} \end{cases} \\ u_{up\_us} \cdot T_{cr} \cdot LF \end{bmatrix}$$

$$U_{tr} = \begin{pmatrix} 0 \\ 6.069 \\ 0 \\ 0 \\ 3.82 \end{pmatrix} \cdot \text{kips}$$

$$U_{TR} := \sum_{n=1}^{5} U_{tr_n} = 9.89\,\text{kip}$$

Arms of Revised Uplift Forces about Center of Uncracked Base:

$$D_{tr} := \begin{bmatrix} \begin{cases} \frac{1}{2} \cdot (B - d_{dr}) - \frac{B_{tr}}{2} & \text{if } d_{dr} > T_{cr} \\ \frac{1}{2} \cdot (B - T_{cr}) - \frac{B_{tr}}{2} & \text{otherwise} \end{cases} \\ \begin{cases} \frac{2}{3} \cdot (B - d_{dr}) - \frac{B_{tr}}{2} & \text{if } d_{dr} > T_{cr} \\ \frac{2}{3} \cdot (B - T_{cr}) - \frac{B_{tr}}{2} & \text{otherwise} \end{cases} \\ \begin{cases} \frac{B_{tr}}{2} - \frac{(d_{dr} - T_{cr})}{2} & \text{if } d_{dr} > T_{cr} \\ 0 & \text{otherwise} \end{cases} \\ \begin{cases} \frac{B_{tr}}{2} - \frac{(d_{dr} - T_{cr})}{3} & \text{if } d_{dr} > T_{cr} \\ 0 & \text{otherwise} \end{cases} \\ \frac{B_{tr}}{2} + \frac{T_{cr}}{2} \end{bmatrix}$$

$$D_{tr} = \begin{pmatrix} 0.00 \\ 2.09 \\ 0.00 \\ 0.00 \\ 8.25 \end{pmatrix} \cdot \text{ft}$$

Sum Moments using Revised Uplift Forces and Arms about Center of Base

$$M_{up\_tr} := \begin{pmatrix} U_{tr_1} \cdot D_{tr_1} \\ U_{tr_2} \cdot D_{tr_2} \\ U_{tr_3} \cdot D_{tr_3} \\ U_{tr_4} \cdot D_{tr_4} \\ U_{tr_5} \cdot D_{tr_5} \end{pmatrix} \qquad M_{up\_tr} = \begin{pmatrix} 0 \\ 12.695 \\ 0 \\ 0 \\ 31.519 \end{pmatrix} \cdot kips \cdot ft \qquad M_{up\_TR} := \sum_{n=1}^{5} \left(M_{up\_tr}\right)_n = 44.213\, ft \cdot kip$$

Evaluate the revised eccentricity using the results of the CBA

$$M_{tot\_CR} := M_{damCR} + M_{dw.ycr} + MH_{tot} = 2.722\, ft \cdot kip$$

$$M_{up\_TR} = 44.213\, ft \cdot kip$$

$$e_{tr} := \frac{M_{tot\_CR} + M_{up\_TR}}{-\left(F_{vr} \cdot kip + U_{TR}\right)} = 3.066\, ft$$

$$M_{tot\_CR} + M_{up\_TR} = 46.936\, ft \cdot kip$$

$$F_{vr} \cdot kip + U_{TR} = -15.31\, kip$$

Trial Crack Length based on Revised Eccentricity

$$x_{1\_tr} := \frac{-B_{tr}^{\;2}}{12 e_{tr}} = -4.281\, ft \qquad B_{unc} := \frac{1}{2} \cdot B - x_{1\_tr} = 12.531\, ft$$

Check for convergence between Trial Uncracked Length, B_{tr} and Recalculated Uncracked Length, $B_{uncracked}$

Difference (target<=0.5%)

$$\Delta := \frac{\left(B_{unc} - B_{tr}\right)}{B_{tr}} = -0.148 \cdot \%$$

$$Action := \begin{cases} \text{"OK"} & \text{if } \Delta \le 0.5\% \wedge \Delta \ge -0.5\% \\ \text{"CHANGE B\_tr"} & \text{otherwise} \end{cases}$$

Action = "OK"

- If "OK"

Final Result of Cracked Base Analysis --->

$$B_{unc} = 12.5\, ft$$

$$T_{cracked} := B - B_{unc} = 4 \cdot ft$$

Cracked Base Analysis - Case #3 (Click to Expand, if Required)

Summary of Vertical Forces - Click to Expand

Revised Summary Table

III. SUMMARY OF REVISED CRACKED BASE LOADS AND MOMENTS - CASE #3

SUMMARY OF VERTICAL FORCES/MOMENTS

Component	Acting			Resisting		
	Force (kip)	Arm (ft)	Moment (kip-ft)	Force (kip)	Arm (ft)	Moment (kip-ft)
Weight of Concrete	-	-	-	-25.2	4.4	-111.0
Uplift Pressure, Area #1	0.0	0.0	0.0	-		
Uplift Pressure, Area #2	6.1	2.1	12.7	-	-	-
Uplift Pressure, Area #3	0.0	0.0	0.0	-	-	-
Uplift Pressure, Area #4	0.0	0.0	0.0	-	-	-
Uplift Pressure, Area #5	3.8	8.3	31.5	-	-	-
Totals	**9.9**	**-**	**44.2**	**-25.2**	**-**	**-111.0**

Σ Vertical Forces w/ uplift $FV_{cb\_tot} := (F_{cb\_vr} + F_{cb\_va})\cdot kips$ $FV_{cb\_tot} = -15.31\cdot kips$

Σ Vertical Moments w/ uplift $MV_{cb\_tot} := (M_{cb\_vr} + M_{cb\_va})\cdot kips\cdot ft$ $MV_{cb\_tot} = -66.8\cdot kips\cdot ft$

SUMMARY OF HORIZONTAL FORCES/MOMENTS

Component	Acting			Resisting		
	Force (kip)	Arm (ft)	Moment (kip-ft)	Force (kip)	Arm (ft)	Moment (kip-ft)
Headwater on Dam, Fuwa.x	0.0	0.0	0.0	-	-	-
Headwater on Dam, Fuwb.x	7.5	5.2	38.7	-	-	-
Ice Force	5.0	15.0	75.0	-	-	-
Tailwater on Dam, Fdw.x	-	-	-	0.0	0.0	0.0
Totals	**12.5**	**-**	**113.7**	**0.0**	**-**	**0.0**

S Horizontal Forces $FH_{tot} := (F_{hr} + F_{ha})\cdot kips$ $FH_{tot} = 12.496\cdot kips$

S Horizontal Moments $MH_{tot} := (M_{hr} + M_{ha})\cdot kips\cdot ft$ $MH_{tot} = 113.7\cdot kips\cdot ft$

S Moments (w/ uplift) $M_{tot} := (M_{ha} + M_{hr} + M_{va} + M_{vr})\cdot kips\cdot ft$ $M_{tot} = 74.4\cdot kips\cdot ft$

Revised Summary Table

Revised Parameters (Click to Expand)

Evaluate Sliding Stability (shear friction factor):

Base Inclination Angle $\beta = 0$

Base Area for Analysis $A_0 := B_{unc}\cdot LF = 12.531\ ft^2$

$$FS\_SLIDING_3 := \frac{(-F_V\cdot\cos(\beta) - F_U + F_H\cdot\sin(\beta)\cdot f_\beta)\cdot\tan(\phi_{dam}) + (A_0 - A_{shear})\cdot c_0 + A_{shear}\cdot c_{shear}}{F_H\cdot\cos(\beta) - F_V\cdot\sin(\beta)\cdot f_\beta} = 9.037$$

Calculated $FS_{sliding}$ $FS\_SLIDING_3 = 9.0$

Required Factor of Safety (from Page 5): $FS\_SLIDING_{min_3} = 2.0$

$$\textbf{Sliding\_Stability} := \begin{cases} \text{"SLIDING OK"} & \text{if } FS\_SLIDING_3 \geq FS\_SLIDING_{min_3} \\ \text{"NOT ADEQUATE"} & \text{otherwise} \end{cases}$$

Sliding_Stability = "SLIDING OK"

Re-Evaluate Overturning and Base Pressures

Final Resultant Location:

$$R_{rev} := \frac{1}{2} \cdot B_{unc} - e_{rev} = 3.2\,ft$$

(+) = U/S of toe
(-) = D/S of toe

$$\textbf{Location}_{\textbf{Rrev}_3} := \begin{cases} \text{"OK"} & \text{if } \left(R_{rev} \geq \frac{B_{unc}}{4}\right) \wedge \left(R_{rev} \leq 3\,\frac{B_{unc}}{4}\right) \\ \text{"NG"} & \text{otherwise} \end{cases}$$

$\textbf{Location}_{\textbf{Rrev}_3}$ = "OK"

- Base Pressure at Toe (includes Uplift)

$$\textbf{p}_{\textbf{toe}_3} := \begin{cases} P_{ds\_o} & \text{if } \textbf{CBA}_3 = \text{"NOT REQUIRED"} \\ \frac{-(F_V + F_U)}{B_{unc} \cdot LF} \cdot \left[1 + 6 \cdot \left(\frac{e_{rev}}{B_{unc}}\right)\right] & \text{otherwise} \end{cases}$$

$\textbf{p}_{\textbf{toe}_3} = 3.0 \cdot ksf$

Check to see if pressure exceeds foundation bearing capacity:

$$\textbf{BC}_3 := \begin{cases} \text{"OK"} & \text{if } \textbf{p}_{\textbf{toe}_3} < 1.33BC \\ \text{"EXCEEDED"} & \text{otherwise} \end{cases}$$

$\textbf{BC}_3$ = "OK"

Check to see if pressure exceeds dam compressive strength:

$$\textbf{Cu\_dam}_3 := \begin{cases} \text{"OK"} & \text{if } \textbf{p}_{\textbf{toe}_3} < 0.9 \cdot Cu_{dam} \\ \text{"EXCEEDED"} & \text{otherwise} \end{cases}$$

$\textbf{Cu\_dam}_3$ = "OK"

Check flotation:

$$FS\_FLOT_3 := \frac{-F_V}{F_U} = 2.548$$

Required Factor of Safety against Flotation (from USACE EM 1110-2-2100, Table 3-4):

$$FS\_FLOT_{min_3} := 1.1$$

$$\textbf{Flotation}_3 := \begin{cases} \text{"OK"} & \text{if } FS\_FLOT_3 \geq FS\_FLOT_{min_3} \\ \text{"NOT ADEQUATE"} & \text{otherwise} \end{cases}$$

$\textbf{Flotation}_3$ = "OK"

END OF LOAD CASE #3 ANALYSIS

LOAD CASE #4 - Normal Pool + Earthquake

I. INITIAL VERTICAL LOADS AND MOMENTS:

A. Dam self-weights - Same as Load Case #1

B. Headwater: (Vertical Component) - Same as Load Case #1

Conditional statement using variable H_water_over_crest
(*determines whether headwater higher than dam crest)*

$H_{water\_over\_crest} := H_{uw_4} - H_{dam} = -0.5\,ft$

Height of water above crest

$$H_{ov}(H_{water\_over\_crest}) := \begin{cases} H_{water\_over\_crest} & \text{if } H_{water\_over\_crest} > 0 \cdot ft \\ 0 & \text{otherwise} \end{cases}$$

$H_{ov}(H_{water\_over\_crest}) = 0\,ft$

C. Tailwater: (Vertical Component) - Same as Load Case #1

D. Soil Loads: (Vertical Components) - Same as Load Case #1

E. Uplift Pressure:

Height of drainage gallery to plane of analysis $H_{dg} := EL_{dg} - EL_b = 0\,ft$

Effective Hydraulic Head at Drainage Gallery, H_{dr}

$$H_{dr} := \begin{cases} (1 - E_{dr}) \cdot \left[(H_{uw_4} - H_{dw_4}) \dfrac{B - d_{dr}}{B} + H_{dw_4} - H_{dg} \right] + H_{dg} & \text{if } H_{dg} \ge H_{dw_4} \\ (1 - E_{dr}) \cdot (H_{uw_4} - H_{dw_4}) \cdot \dfrac{B - d_{dr}}{B} + H_{dw_4} & \text{otherwise} \end{cases} = 15.5 \cdot ft$$

based on drain effectiveness $E_{dr} = 0 \cdot \%$

Head at heel $H_{heel} := H_{uw_4} = 15.5\,ft$

Head at toe $H_{toe} := H_{dw_4} = 0\,ft$

Uplift Pressure at Heel $u_{up\_us} := H_{uw_4} \cdot \gamma_w = 0.967 \cdot ksf$

Uplift Pressure at Toe $u_{up\_ds} := H_{toe} \cdot \gamma_w = 0 \cdot ksf$

Uplift Pressure below Drainage Gallery $u_{up\_dg} := H_{dr} \cdot \gamma_w = 0.967 \cdot ksf$

Uplift forces below dam base:
Refer to FBD for notation:

$U_1 := u_{up\_ds} \cdot B \cdot LF = 0 \cdot kip$

$U_2 := \dfrac{1}{2}(u_{up\_dg} - u_{up\_ds}) \cdot B \cdot LF = 7.979 \cdot kip$

Σ **Uplift Forces =** $U := U_1 + U_2 = 7.979 \cdot kip$

Moment arms of Uplift Forces

$$d_{up1} := \frac{1}{2}B - \frac{1}{2}B = 0\,ft$$

$$d_{up2} := \frac{2}{3}B - \frac{1}{2}B = 2.75\,ft$$

Moments due to Uplift Components

$$M_{up1} := U_1 \cdot d_{up1} = 0 \cdot kip \cdot ft$$

$$M_{up2} := U_2 \cdot d_{up2} = 21.943 \cdot kip \cdot ft$$

Σ **Uplift Moments =** $M_{up} := M_{up1} + M_{up2} = 21.943 \cdot kip \cdot ft$

II. INITIAL HORIZONTAL FORCES AND MOMENTS:

A. Headwater: (Horizontal Component)

Horizontal Component of Headwater on Dam

$$F_{uwa.x} := \begin{cases} \gamma_w \cdot \left(H_{uw_4} - H_{dam}\right) \cdot H_{dam} \cdot LF & \text{if } H_{ov}\left(H_{water\_over\_crest}\right) > 0 \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kips$$

$$F_{uwb.x} := \begin{cases} \frac{1}{2}\gamma_w \cdot H_{dam}^{2} \cdot LF & \text{if } H_{ov}\left(H_{water\_over\_crest}\right) > 0 \\ \frac{1}{2}\gamma_w \cdot \left(H_{heel}\right)^2 \cdot LF & \text{otherwise} \end{cases} = 7.496 \cdot kips$$

Σ **Horizontal Forces by Headwater** $F_{uw.x} := F_{uwa.x} + F_{uwb.x} = 7.496\,kip$

Moment arms of Headwater on Dam

$$arm_{uwa.y} := \begin{cases} \frac{H_{dam}}{2} & \text{if } H_{ov}\left(H_{water\_over\_crest}\right) > 0 \\ 0 & \text{otherwise} \end{cases} = 0 \cdot ft$$

$$arm_{uwb.y} := \begin{cases} \frac{1}{3}H_{dam} & \text{if } H_{ov}\left(H_{water\_over\_crest}\right) > 0 \\ \frac{H_{heel}}{3} & \text{otherwise} \end{cases} = 5.167 \cdot ft$$

Moment of Headwater on Dam

$$M_{uwa.x} := F_{uwa.x} \cdot arm_{uwa.y} = 0 \cdot kip \cdot ft$$

$$M_{uwb.x} := F_{uwb.x} \cdot arm_{uwb.y} = 38.728 \cdot kip \cdot ft$$

Σ **Moments due to Headwater** $M_{uw.x} := M_{uwa.x} + M_{uwb.x} = 38.728 \cdot kip \cdot ft$

B. Tailwater: (Horizontal Component)

(Typically neglect stabilizing force from tailwater during seismic event, unless additional capacity is needed)

$TW := 0$

Horizontal Force due to Tailwater

$$F_{dw.x} := \begin{cases} -\frac{1}{2}\gamma_w \cdot \left(H_{dw_1}\right)^2 \cdot LF & \text{if } TW = 1 \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kips$$

Moment arm of Tailwater $arm_{dw.y} := \frac{1}{3} H_{dw_3} = 0\ ft$

Moment due to Tailwater $M_{dw.x} := F_{dw.x} \cdot arm_{dw.y} = 0\ ft \cdot kip$

C. Soil horizontal loading on upstream side of dam: - Same as Load Case #1

D. Earthquake Loading

1. Add'l Horizontal Forces and Moments for Concrete due to Earthquake:

Additional horizontal forces: $F_{W1q.x} := -\lambda \cdot W_c = 4.99\ kip$

Σ Additional Masonry Forces Due to Earthquake $F_{damq.x} := F_{W1q.x} = 4.99\ kip$

Moment arms about Toe: $arm_{W1q.y} := Y_{Centroid.c} = 6.48\ ft$

Additional moments: $M_{W1q.x} := F_{W1q.x} \cdot arm_{W1q.y} = 32.333\ ft \cdot kip$

Σ Additional Masonry Moments Due to Earthquake

$$M_{damq.x} := M_{W1q.x} = 32.333\ ft \cdot kip$$

2. Additional Horizontal Forces and Moments from Soil due to Earthquake:

Earthquake Forces from Upstream and Downstream Soil (See Appendix G of EM 1110-2-2100)

Peak Ground Acceleration: $PGA := \lambda = 0.198\ g$

- Assume $k_v := 0$ and neglect effect of soil friction on dam. Vertical face, therefore use EQ G-5 and G-6

Upstream (active) Embankment Angle: $\beta_{us} := 0$ $\beta_{us} = 0 \cdot °$

Passive Side Embankment Angle: $\beta_{ds} := 0$ $\beta_{ds} = 0 \cdot °$

Seismic Inertia Angle:

$$\psi := \operatorname{atan}\left(\frac{\frac{2}{3}PGA}{1 - k_v}\right) = 0.131 \qquad \psi = 7.52 \cdot °$$

Active seismic soil pressure coefficient (fill):

$$K_{AE\_fill} := \frac{\cos(\phi_{fillus} - \psi)^2}{\cos(\psi)^2 \cdot \left(1 + \sqrt{\frac{\sin(\phi_{fillus}) \sin(\phi_{fillus} - \psi - \beta_{us})}{\cos(\beta_{us}) \cdot \cos(\psi)}}\right)^2} = 1$$

Passive seismic soil pressure coefficient (fill):

Analyses of previous load cases conservatively used K_0 rather than K_P. Since $K_{PE} >> K_0$, assume seismic force will negate any stabilizing force of the downstream soil.

Earthquake Horizontal Forces from Soil upstream (refer to FBD for dam geometry)

Horizontal Force

$$F_{fillus.q} := \frac{1}{2}(\gamma_{fillus} - \gamma_w) \cdot (EL_{fillus} - EL_b)^2 \cdot LF = 0\ kip$$

Moment arm of fill

$$arm_{fillus.q} := \left(\frac{1}{3}\right)(EL_{fillus} - EL_b) = 0\ ft$$

Moment due to fill

$$M_{fillus.q} := F_{fillus.x} \cdot arm_{fillus.y} = 0\ ft \cdot kip$$

▶ N/A Field; Seismic Loading due to Silt

3. Additional Horizontal Forces and Moments from Reservoir and Tailwater due to Earthquake:

Hydrodynamic Force - Upstream Side:

From Figure #10 of USBR Engineering Monograph #11

$C_e := 0.76$ for dam with a vertical upstream face/slope

The increase in water pressure due to horizontal earthquake acceleration becomes:

$$P_e := C_e \cdot PGA \cdot \gamma_w \cdot \left(EL_{uw_4} - EL_b\right) = 145.544 \cdot psf$$

The total horizontal force due to P_e is expressed analytically as:

$$F_{uwq.x} := 0.726 \cdot P_e \cdot \left(EL_{uw_4} - EL_b\right) \cdot LF = 1.638 \cdot kip$$

The total horizontal moment due to P_e is expressed analytically as:

$$M_{uwq.x} := 0.299 \cdot P_e \cdot \left(EL_{uw_4} - EL_b\right)^2 \cdot LF = 10.455\ ft \cdot kip$$

Hydrodynamic Force - Downstream Side:

Neglected tailwater force during seismic event.

▶ (Summary of Vertical Forces Raw Data - Click to expand)

III. SUMMARY OF INITIAL LOADS AND MOMENTS - CASE #4

SUMMARY OF VERTICAL FORCES/MOMENTS

Component	Acting			Resisting		
	Force (kip)	Arm (ft)	Moment (kip-ft)	Force (kip)	Arm (ft)	Moment (kip-ft)
Weight of Concrete	-	-	-	-25.2	2.4	-61.2
Uplift Pressure #1	0.0	0.0	0.0	-	-	-
Uplift Pressure #2	8.0	2.8	21.9	-	-	-
Totals	**8.0**	**-**	**21.9**	**-25.2**	**-**	**-61.2**

Σ **Vertical Forces w/ uplift** $FV_{tot} := (F_{vr} + F_{va}) \cdot kips$ $FV_{tot} = -17.221 \cdot kips$

Σ **Vertical Moments w/ uplift** $MV_{tot} := (M_{vr} + M_{va}) \cdot kips \cdot ft$ $MV_{tot} = -39.3 \cdot kips \cdot ft$

(Summary of Horizontal Forces Raw Data - Click to expand)

SUMMARY OF HORIZONTAL FORCES/MOMENTS

Component	Acting			Resisting		
	Force (kip)	Arm (ft)	Moment (kip-ft)	Force (kip)	Arm (ft)	Moment (kip-ft)
Headwater on Dam, Fuwa.x	0.0	0.0	0.0	-	-	-
Headwater on Dam, Fuwb.x	7.5	5.2	38.7	-	-	-
Seismic Force, Dam (Total)	5.0	6.5	32.3	-	-	-
Seismic Force, Headwater, Fuwq.x	1.6	0.0	10.5	-	-	-
Tailwater on Dam, Fdw.x				0.0	0.0	0.0
Totals	**14.1**	**-**	**81.5**	**0.0**	**-**	**0.0**

Σ **Horizontal Forces** $FH_{tot} := (F_{hr} + F_{ha}) \cdot kips$ $FH_{tot} = 14.123 \cdot kips$

Σ **Horizontal Moments** $MH_{tot} := (M_{hr} + M_{ha}) \cdot kips \cdot ft$ $MH_{tot} = 81.5 \cdot kips \cdot ft$

Σ **Moments (w/ uplift)** $M_{tot} := (M_{ha} + M_{hr} + M_{va} + M_{vr}) \cdot kips \cdot ft$ $M_{tot} = 42.2 \cdot kips \cdot ft$

IV. EVALUATE OVERTURNING AND BASE PRESSURES

- Check Resultant Location - COE EM1110-2-2200 Overturning Stability Criteria

- Usual Conditions = Within middle third of the base
- Unusual Conditions = Within middle half of the base
- **Extreme Conditions = Within base**

A. Calculate Eccentricity and Base Pressures

- Resultant and Eccentricity

- Eccentricity
(from centroid of Base Area)

$$e_o := \frac{M_{tot}}{-FV_{tot}} = 2.452\,ft$$

(+) = D/S of Centroid
(-) = U/S of Centroid

- Resultant Location (from toe)

$$R_o := \frac{1}{2} B - e_o = 5.798\,ft$$

- Evaluate Overturning using Resultant Location

$$\text{Location}_{Ro_4} := \begin{cases} \text{"WITHIN BASE"} & \text{if } (R_o \geq 0) \wedge (R_o \leq B) \\ \text{"OUTSIDE BASE"} & \text{otherwise} \end{cases}$$

$\text{Location}_{Ro_4} = \text{"WITHIN BASE"}$

- Base Pressures (includes Uplift)

Base Pressure at Heel:

$$P_{us\_o} := \frac{-FV_{tot}}{B \cdot LF} \cdot \left(1 - \frac{6 \cdot e_o}{B}\right) = 0.113\,\text{ksf}$$

(-) = tension
(+) = compression

Base Pressure at Toe:

$$P_{ds\_o} := \frac{-FV_{tot}}{B \cdot LF} \cdot \left(1 + \frac{6 \cdot e_o}{B}\right) = 1.974\,\text{ksf}$$

(-) = tension
(+) = compression

C. Check if Cracked Base Analysis (CBA) is Required

(CBA is required when base pressure with uplift are negative, i.e. base in tension)

(Note: CBA not required for seismic, unless a crack exists under normal conditions)

$CBA_4 := CBA_1 = \text{"NOT REQUIRED"}$

Cracked Base Analysis - Case #4 (Click to Expand, if Required)
Summary of Vertical Forces - Click to Expand
Revised Summary Table
Revised Parameters (Click to Expand)

Evaluate Sliding Stability (shear friction factor):

Base Inclination Angle $\beta = 0$

Base Area for Analysis $A_0 := B_{unc} \cdot LF = 16.5\,\text{ft}^2$

$$FS\_SLIDING_4 := \frac{\left(-F_V \cdot \cos(\beta) - F_U + F_H \cdot \sin(\beta) \cdot f_\beta\right) \cdot \tan\left(\phi_{dam}\right) + \left(A_0 - A_{shear}\right) \cdot c_0 + A_{shear} \cdot c_{shear}}{F_H \cdot \cos(\beta) - F_V \cdot \sin(\beta) \cdot f_\beta} = 8.412$$

Calculated $FS_{sliding}$ $FS\_SLIDING_4 = 8.4$ Required Factor of Safety (from Page 5): $FS\_SLIDING_{min_4} = 1.3$

$$\text{Sliding\_Stability} := \begin{cases} \text{"SLIDING OK"} & \text{if } FS\_SLIDING_4 \geq FS\_SLIDING_{min_4} \\ \text{"NOT ADEQUATE"} & \text{otherwise} \end{cases}$$

$\text{Sliding\_Stability} = \text{"SLIDING OK"}$

Re-Evaluate Overturning and Base Pressures

Final Resultant Location:

$$R_{rev} := \frac{1}{2} \cdot B_{unc} - e_{rev} = 5.798\,\text{ft}$$

(+) = U/S of toe
(-) = D/S of toe

$$\mathbf{Location_{Rrev_4}} := \begin{cases} \text{"OK"} & \text{if } (R_{rev} \geq 0) \wedge (R_{rev} \leq B_{unc}) \\ \text{"NG"} & \text{otherwise} \end{cases}$$

$\mathbf{Location_{Rrev_4}} = \text{"OK"}$

- Base Pressure at Toe (includes Uplift)

$$\mathbf{p_{toe_4}} := \begin{cases} P_{ds\_o} & \text{if } \mathbf{CBA_4} = \text{"NOT REQUIRED"} \\ \dfrac{-(F_V + F_U)}{B_{unc} \cdot LF} \cdot \left[1 + 6 \cdot \left(\dfrac{e_{rev}}{B_{unc}}\right)\right] & \text{otherwise} \end{cases}$$

$\mathbf{p_{toe_4}} = 2.0 \cdot ksf$

Check to see if pressure exceeds foundation bearing capacity:

$$\mathbf{BC_4} := \begin{cases} \text{"OK"} & \text{if } \mathbf{p_{toe_4}} < 1.33BC \\ \text{"EXCEEDED"} & \text{otherwise} \end{cases}$$

$\mathbf{BC_4} = \text{"OK"}$

Check to see if pressure exceeds dam compressive strength:

$$\mathbf{Cu\_dam_4} := \begin{cases} \text{"OK"} & \text{if } \mathbf{p_{toe_4}} < 0.9 \cdot Cu_{dam} \\ \text{"EXCEEDED"} & \text{otherwise} \end{cases}$$

$\mathbf{Cu\_dam_4} = \text{"OK"}$

Check flotation:

$$FS\_FLOT_4 := \frac{-F_V}{F_U} = 3.158$$

Required Factor of Safety against Flotation (from USACE EM 1110-2-2100, Table 3-4): $FS\_FLOT_{min_4} := 1.1$

$$\mathbf{Flotation_4} := \begin{cases} \text{"OK"} & \text{if } FS\_FLOT_4 \geq FS\_FLOT_{min_4} \\ \text{"NOT ADEQUATE"} & \text{otherwise} \end{cases}$$

$\mathbf{Flotation_4} = \text{"OK"}$

END OF LOAD CASE #4 ANALYSIS

SUMMARY OF STABILITY ANALYSIS RESULTS

Sliding

Case	Description	Cracked Base Analysis	Minimum Required FS	Calculated FS	Base Cohesion Req'd for Min FS (psi)
1	Normal water levels	*NOT REQUIRED*	2.0	15.8	N/A
2	Flood water levels	*NOT REQUIRED*	1.1	11.9	N/A
3	Normal water levels + Ice	*REQUIRED*	2.0	9.0	N/A
4	Normal water levels + Earthquake	*NOT REQUIRED*	1.3	8.4	N/A

Overturning

Case	Description	Required Resultant Location	Calculated Resultant Location*	Calculated Base Pressure at Toe (ksf)	Bearing Capacity OK?	Dam Compressive Strength OK?
1	Normal water levels	WITHIN MIDDLE 1/3	WITHIN MIDDLE 1/3	1.0	OK	OK
2	Flood water levels	WITHIN BASE	WITHIN BASE	1.4	OK	OK
3	Normal water levels + Ice	WITHIN MIDDLE 1/2	OUTSIDE MIDDLE 1/2	3.0	OK	OK
4	Normal water levels + Earthquake	WITHIN BASE	WITHIN BASE	2.0	OK	OK

Flotation

Case	Description	Minimum Required FS	Calculated FS	FS Flotation OK?
1	Normal water levels	1.3	3.2	OK
2	Flood water levels	1.1	2.7	OK
3	Normal water levels + Ice	1.3	2.5	OK
4	Normal water levels + Earthquake	1.1	3.2	OK

**The resultant location is relative to the overall base width.*

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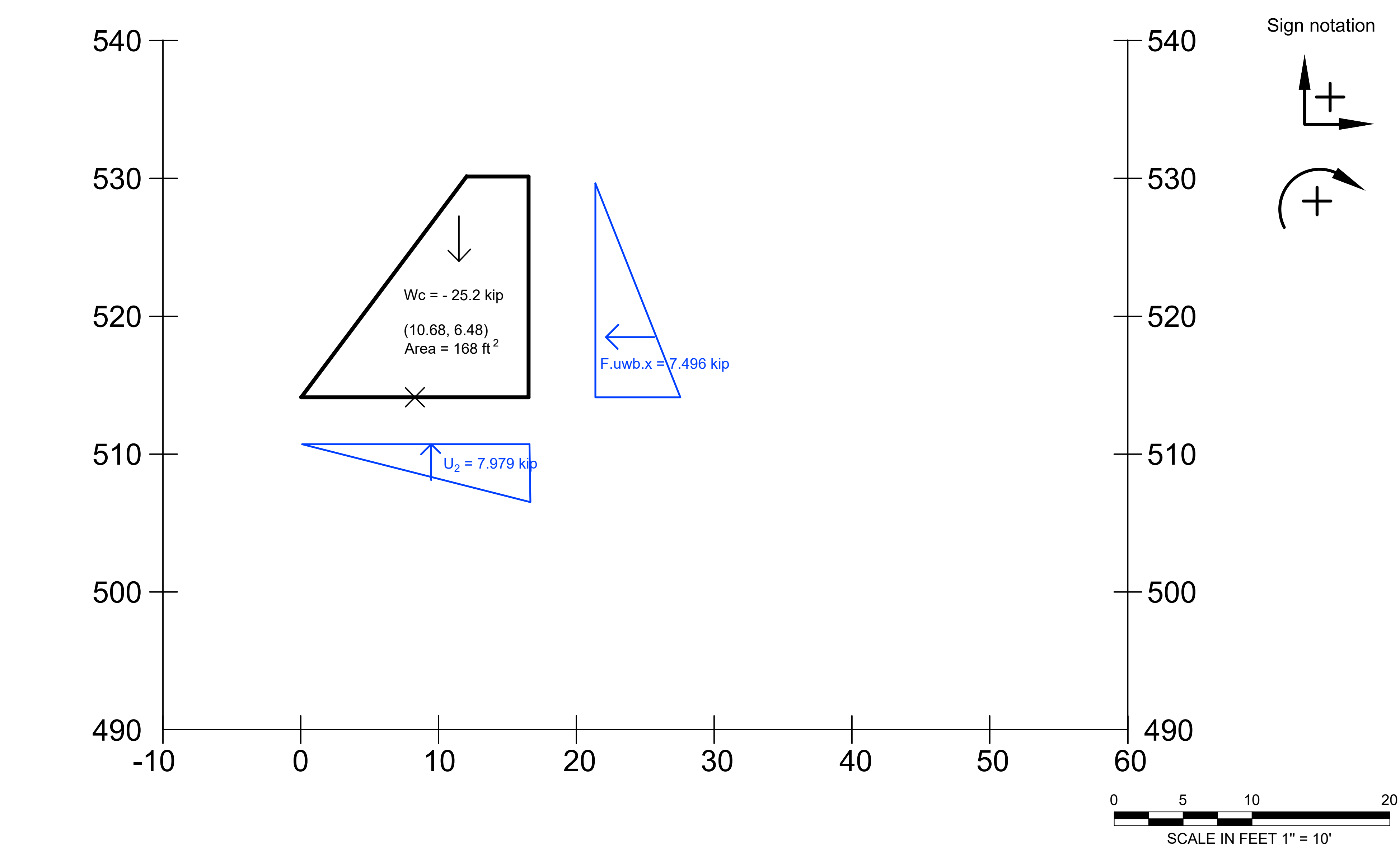

NO.	ISSUE/DESCRIPTION	BY	DATE

INDIAN BROOK RESERVOIR DAM

ESSEX, VT

INTERMEDIATE FAILURE PLANE NORMAL POOL

PREPARED BY: GZA GeoEnvironmental, Inc. www.gza.com

PREPARED FOR: Vermont Department of Environmental Conservation

PROJ MGR: DJS	REVIEWED BY: JGD	CHECKED BY: JGD	FIGURE
DESIGNED BY: MZ	DRAWN BY: MZ	SCALE: 1"=10'	1
DATE: 3-9-2023	PROJECT NO. 01.0175988.00	REVISION NO.	SHEET NO. 1 OF 2

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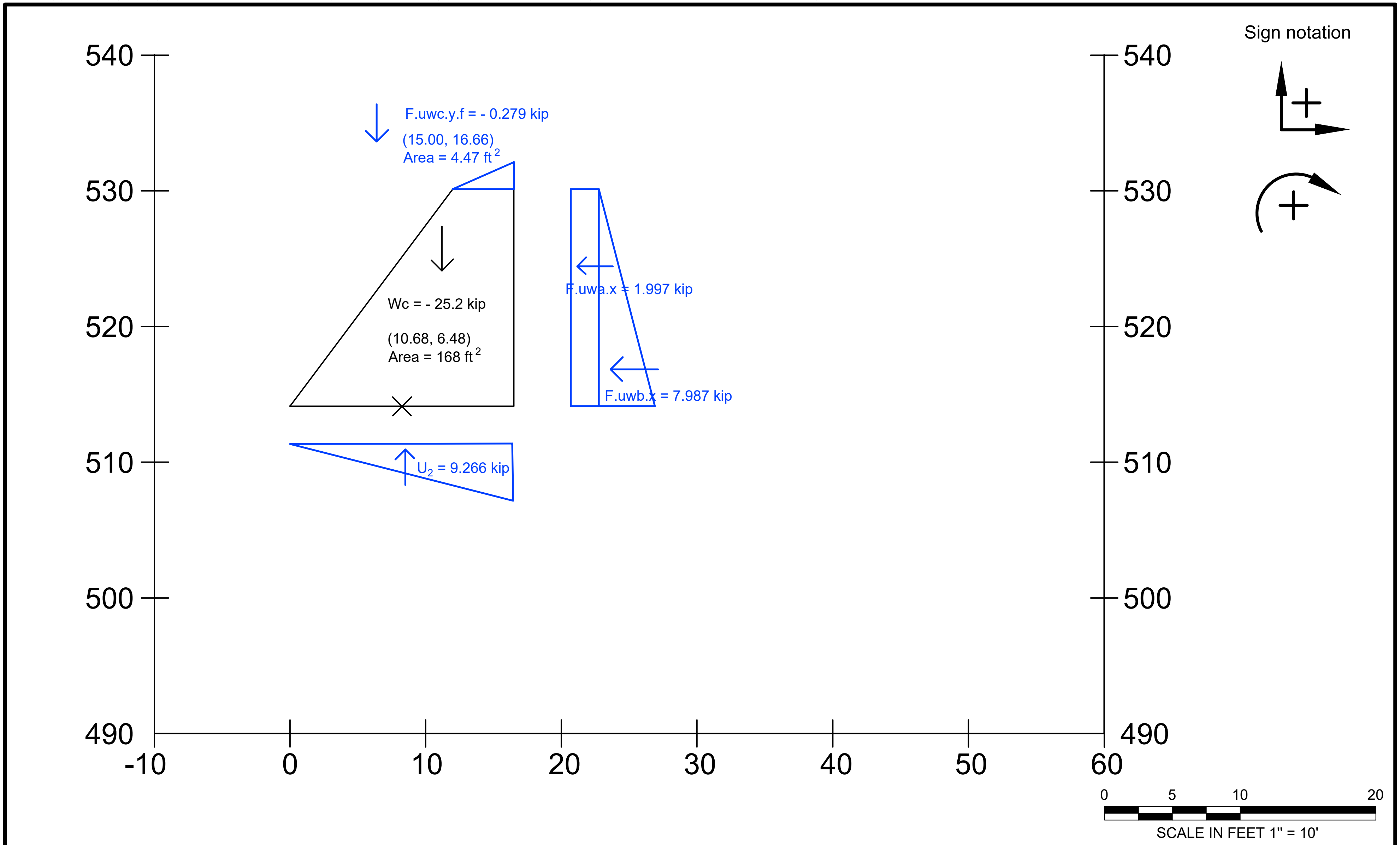
GZA–\\GZANOR\JOBS\170,000–179,999\175988\175988-00.DJS\WORK\STABILITY CALCS\06 INDIAN BROOK RESERVOIR DAM\INDIAN BROOK RESERVOIR DAM_CROSS-SECTION – FMR.DWG A-A PMF NOVEMBER 22, 2023 MENG



NO.	ISSUE/DESCRIPTION	BY	DATE

INDIAN BROOK RESERVOIR DAM

ESSEX, VT

INTERMEDIATE FAILURE PLANE FLOOD (SDF) POOL

PREPARED BY:
GZA GeoEnvironmental, Inc.
www.gza.com

PREPARED FOR:
Vermont Department of Environmental Conservation

PROJ MGR: DJS	REVIEWED BY: JGD	CHECKED BY: JGD	FIGURE
DESIGNED BY: MZ	DRAWN BY: MZ	SCALE: 1"=10'	2
DATE: 3-9-2023	PROJECT NO. 01.0175988.00	REVISION NO.	SHEET NO. 2 OF 2

APPENDIX E - LIMITATIONS

USE OF REPORT

1. GZA GeoEnvironmental, Inc. (GZA) prepared this report on behalf of, and for the exclusive use of Vermont Department of Environmental Conservation – Water Investment Division (Client) for the stated purpose(s) and location(s) identified in the Report. Use of this report, in whole or in part, at other locations, or for other purposes, may lead to inappropriate conclusions; and we do not accept any responsibility for the consequences of such use(s). Further, reliance by any party not identified in the agreement, for any use, without our prior written permission, shall be at that party's sole risk, and without any liability to GZA.

STANDARD OF CARE

2. Our findings and conclusions are based on the work conducted as part of the Scope of Services set forth in the Report and/or proposal, and reflect our professional judgment. These findings and conclusions must be considered not as scientific or engineering certainties, but rather as our professional opinions concerning the limited data gathered during the course of our work. Conditions other than described in this report may be found at the subject location(s).

3. Our services were performed using the degree of skill and care ordinarily exercised by qualified professionals performing the same type of services at the same time, under similar conditions, at the same or a similar property. No warranty, expressed or implied, is made.

SUBSURFACE CONDITIONS

4. If presented, the generalized soil profile(s) and description, along with the conclusions and recommendations provided in our Report, are based in part on widely-spaced subsurface explorations by GZA and/or others, with a limited number of soil and/or rock samples and groundwater /piezometers data and are intended only to convey trends in subsurface conditions. The boundaries between strata are approximate and idealized, and were based on our assessment of subsurface conditions. The composition of strata, and the transitions between strata, may be more variable and more complex than indicated. For more specific information on soil conditions at a specific location refer to the exploration logs. The nature and extent of variations between these explorations may not become evident until further exploration or construction. If variations or other latent conditions then appear evident, it will be necessary to reevaluate the conclusions and recommendations of this report.

5. Water level readings have been made in test holes (as described in the Report), monitoring wells and piezometers, at the specified times and under the stated conditions. These data have been reviewed and interpretations have been made in this Report. Fluctuations in the groundwater and piezometer levels, however, occur due to temporal or spatial variations in areal recharge rates, soil heterogeneities, reservoir and tailwater levels, the presence of subsurface utilities, and/or natural or artificially induced perturbations.

GENERAL

6. The observations described in this report were made under the conditions stated therein. The conclusions presented were based solely upon the services described therein, and not on scientific tasks or procedures beyond the scope of described services or the time and budgetary constraints imposed by the Client.

7. In preparing this report, GZA relied on certain information provided by the Client, state and local officials, and other parties referenced therein available to GZA at the time of the evaluation. GZA did not attempt to independently verify the accuracy or completeness of all information reviewed or received during the course of this evaluation.

8. Any GZA hydrologic analysis presented herein is for the rainfall volumes and distributions stated herein. For storm conditions other than those analyzed, the response of the site's spillway, impoundment, and drainage network has not been evaluated.

9. Observations were made of the site and of structures on the site as indicated within the report. Where access to portions of the structure or site, or to structures on the site was unavailable or limited, GZA renders no opinion as to the condition of that portion of the site or structure. In particular, it is noted that water levels in the impoundment and elsewhere and/or flow over the spillway may have limited GZA's ability to make observations of underwater portions of the structure. Excessive vegetation, when present, also inhibits observations.

10. In reviewing this Report, it should be realized that the reported condition of the dam is based on observations of field conditions during the course of this study along with data made available to GZA. It is important to note that the condition of a dam depends on numerous and constantly changing internal and external conditions, and is evolutionary in nature. It would be incorrect to assume that the present condition of the dam will continue to represent the condition of the dam at some point in the future. Only through continued inspection and care can there be any chance that unsafe conditions be detected.

COMPLIANCE WITH CODES AND REGULATIONS

11. We used reasonable care in identifying and interpreting applicable codes and regulations. These codes and regulations are subject to various, and possibly contradictory, interpretations. Compliance with codes and regulations by other parties is beyond our control.

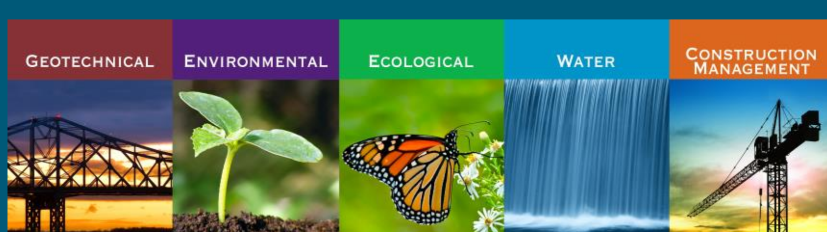
12. This scope of work does not include an assessment of the need for fences, gates, no-trespassing signs, repairs to existing fences and railings and other items which may be needed to minimize trespass and provide greater security for the facility and safety to the public. An evaluation of the project for compliance with OSHA rules and regulations is also excluded.

COST ESTIMATES

13. Unless otherwise stated, our cost estimates are for comparative, or general planning purposes. These estimates may involve approximate quantity evaluations and may not be sufficiently accurate to develop construction bids, or to predict the actual cost of work addressed in this Report. Further, since we have no control over the labor and material costs required to plan and execute the anticipated work, our estimates were made using our experience and readily available information. Actual costs may vary over time and could be significantly more, or less, than stated in the Report.

ADDITIONAL SERVICES

14. It is recommended that GZA be retained to provide services during any future: site observations, explorations, evaluations, design, implementation activities, construction and/or implementation of remedial measures recommended in this Report. This will allow us the opportunity to: i) observe conditions and compliance with our design concepts and opinions; ii) allow for changes in the event that conditions are other than anticipated; iii) provide modifications to our design; and iv) assess the consequences of changes in technologies and/or regulations.



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