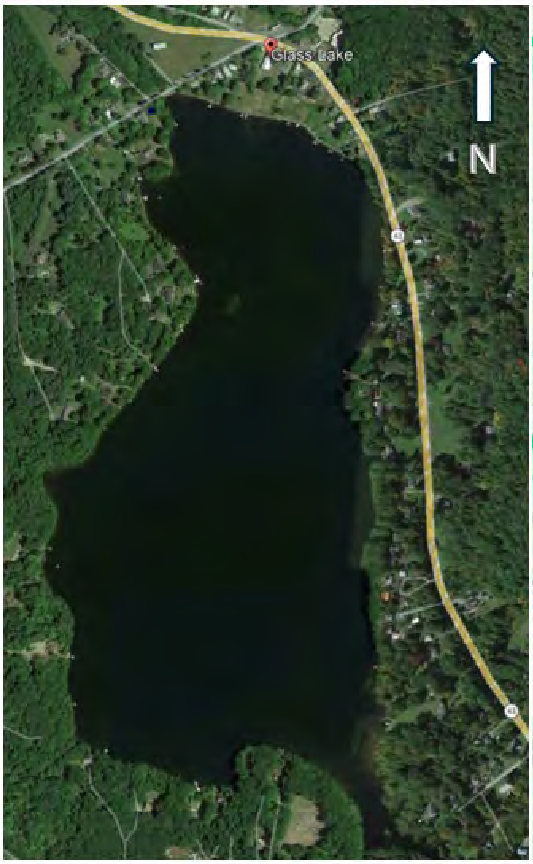
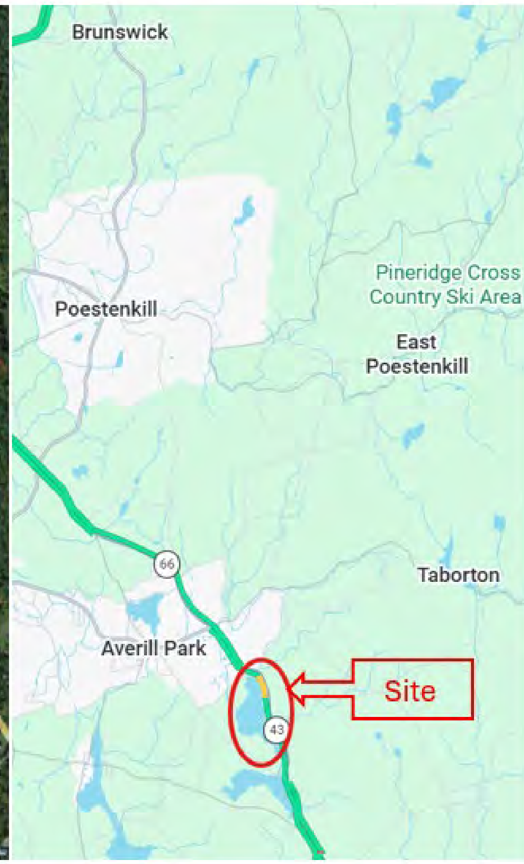
ENGINEERING ASSESSMENT

Glass Lake Dam

NYSDEC ID# 226-1344

Town of Sand Lake, Rensselaer County, NY





FINAL REPORT

August 2026

Prepared for

GLASS LAKE PRESERVATION CORPORATION

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PREFACE

The assessment of the general condition of this dam is based upon available data and visual inspection. Detailed investigations and analysis involving topographic mapping, subsurface exploration, testing and detailed computational evaluations are beyond the scope of this assessment.

In reviewing this assessment, it should be realized that the reported condition of the dam is based on observations of field conditions at the time of inspection, along with data available to the inspection team.

It is critical to note that the condition of the 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 care and inspection can there be any chance that unsafe conditions be detected.

This assessment was approved by Jennifer Wesolowski, P.E. of Gomez and Sullivan Engineers, D.P.C. (Gomez and Sullivan) and has been completed in accordance with the Environmental Conservation Law (ECL) Title 6 NYCRR Part 673 (effective August 19, 2009) and the Guidance for Dam Engineering Assessment Reports (DOW TOGS 3.1.4) by New York State Department of Environmental Conservation, March 2011.

In this assessment report, the term "safety" is interpreted to be restricted specifically to major structural and control features in regard to their adequacy against catastrophic failure due to natural or operational events. No consideration is given herein to public safety aspects related to voluntary occupancy or use of project features in any manner other than its intended use. The contents of this document are presented to satisfy the requirements of the Environmental Conservation Law (ECL) Title 6 NYCRR 673. Reuse of this document for any other purposes, in part or in whole, is at the sole risk of the user.

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1 General Description of the Dam

Glass Lake Dam (NYSDEC Dam ID No. 128-0782) is located in the Town of Sand Lake, Rensselaer County, New York. Glass Lake Dam, a masonry dam/earth dike structure, contains a 108-acre impoundment completed in 1916 (per NYSDEC), and is situated along the Wynants Kill. The Wynants Kill was an important source of waterpower for various mills and industries in the area, including those in Averill Park and Troy. It is currently used only for recreational purposes.

Per the Town of Sand Lake Comprehensive Plan, the Wynants Kill is the primary drainageway in the Town. This stream drains the entire central and northwestern portions of the Town, and it is the outlet for the principal lakes/reservoirs – Burden Lake (Class B dam), Crooked Lake (tributary to Glass Lake), Crystal Lake (Class A dam), Glass Lake (Class B dam), and Reichard's Lake. The eastern end of the Town is drained by the Poestenkill and Tsatsawassa Creeks. The southwest section is drained by tributaries of the Moordener Kill. The waters of the Wynants Kill have a long history of facilitating water powered industry, which has been important to the region and the Town. The headwaters of the Wynants Kill are at Crooked Lake. The creek meanders through the Town and into the Towns of Poestenkill and North Greenbush, descends over several waterfalls, and finally ends its 14-mile journey at the Hudson River in the City of Troy.

Glass Lake Preservation Corporation (GLPC) owns and is responsible for the upkeep, maintenance, and inspections of the dam which is currently used for recreational purposes. The dam structure is composed of a masonry section (main dam) approximately 18 feet high and 34 feet wide. The dam's spillway (broad crested) is composed of two bays separated by a center pier. The spillway has a total width of approximately 21.2 feet and is topped by the Glass Lake Road Bridge owned and operated by Town of Sand Lake, New York. Glass Lake Dam has a 30-inch inside diameter cast iron low-level outlet conduit with a gate valve at the upstream end. Additionally, there is an 1150-foot-long earthen embankment (referred to as the dike) to the right of the masonry dam and a short earthen abutment section to the left.

2 Hazard Classification Evaluation

Glass Lake Dam is currently classified as a Class B "Intermediate Hazard" Dam. A review of ground surface elevations indicates that failure of the dike located east of the masonry dam section due to elevated Glass Lake water levels may result in the inundation of several residence. However, the hydrologic and hydraulic analyses note that water levels behind the dike would likely already elevated due to inadequate drainage through the culvert under Glass Lake Road of storm runoff entering behind the dike. The United States Army Corps of Engineers (USACE) Dam Screening Tool (DST) webtool was used to evaluate a breach of the masonry dam section under the Spillway Design Flood (SDF) conditions. The model extended from the dam along Wynants Kill approximately 15 miles to the Hudson River. Inputs to the webtool are provided in Appendix A of this report. The webtool indicated that no life loss is expected due to a failure of Glass Lake Dam. While the inundation extents provided by the webtool indicate that various roads and driveways would be overtopped leading to some homes being isolated, no Principal Arterials would be inundated. Further, the webtool indicates that less than 100 structures would be inundated. These results confirm that the current Class B "Intermediate Hazard" classification is appropriate. Calculations to support this evaluation are presented in Appendix A of this report.

3 Dam Safety Inspection

An inspection of the dam was performed on November 5, 2025, by Elizabeth Posson, P.E., and Rachel Stofanak, E.I.T., of Gomez and Sullivan. Weather conditions at the time of the inspections were cloudy with a temperature of 36 degrees Fahrenheit. At the time of the inspection, the reservoir level was approximately at the spillway crest elevation of 826.1 feet, North American Vertical Datum of 1988 (NAVD88) with minimal flow over the spillway.

The project was inspected for signs that could predict a future safety issue with the water retaining structures, as is standard during a dam safety inspection. These signs include, but are not limited to, settlement, movement, erosion, seepage/leakage, cracking, deterioration, foundation drain/relief well operation, evidence of high artesian or uplift pressures, observations of sediment transport and observations of seeps, wet areas and springs. However, the inspection was limited due to the extensive amount of vegetation present on the structure. Any such noted items that could be investigated are included in the corresponding section below. Photographs taken during the onsite inspection are presented in Appendix B of this report.

3.1 Left Earthen Embankment

The left earthen embankment was inspected from Glass Lake Road, the upstream slope, and the downstream area and is generally in good condition. Portions of the upstream slope contain riprap, though leaf litter limited full visibility. Brushy vegetation, a tree, and two stumps were present on the upstream slope, with additional brush along the upstream aluminum fence. The roadway over the crest shows no signs of settlement or deterioration. A dry-stone boulder wall on the downstream side is mostly stable, though voids between stones should be periodically monitored for animal activity or seepage (Photos 1 and 2). A corrugated plastic stormwater culvert passes through the embankment and outlets just downstream of the wall; no flow was observed during the inspection.

3.2 Masonry Dam and Spillway

The spillway and adjacent structures were inspected from multiple vantage points and are generally in serviceable condition, though several deficiencies were noted. Concrete columns at the upstream end and the spillway crest appear intact (Photo 3), though leaf litter limited full observation. Minor erosion was identified on the center pier. The underside of the bridge deck exhibits significant concrete spalling and exposed rebar, and the bridge should be formally inspected and repaired (Photo 4).

The valve for the low-level outlet is located at the center of the spillway and is reportedly operated on a regular basis. A staff gauge is installed on the left side of the crest. The upstream boat launch consists of a gravel access path with handrails and riprap along both banks; some debris and brush were present. A dry fire hydrant provides fire department access. Fencing restricts public access to the dike and nearby residences.

Downstream, the vertical masonry face of the dam and spillway is generally in good condition, though wood planks over the spillway crest show moderate deterioration (Photo 5). Seepage was observed at several locations—primarily near the base left of the spillway and beneath the concrete pad—but flows were clear with no sediment. These seeps should continue to be monitored for changes in flow or turbidity (Photo 6).

Multiple stormwater pipes pass through the embankment. A corrugated metal pipe on the right side conveyed a small flow and is in good condition. A corrugated plastic pipe on the left showed no flow and is also in good condition. A PVC pipe through the right abutment is deteriorated. The 30-inch cast iron low-level outlet pipe was mostly submerged, preventing flow observation (Photo 7). The downstream channel remains stable with no signs of erosion.

3.3 Earthen Dike

The earthen dike along the north side of the reservoir was inspected from the crest, upstream slope, and downstream slope. The crest is grass-covered with some bare areas and an uneven surface likely caused by recreational use. Rodent burrows, minor sloughing, and localized erosion were observed along the embankment (Photo 8). The upstream slope includes partial riprap near the waterline, with brushy overgrowth and bare areas where riprap or vegetation is sparse; the upper slope is grass-covered. Various recreational structures—such as stairs and docks—are present on the upstream slope (Photo 9).

The downstream slope is generally in good condition, grass-covered with minor localized erosion. Additional recreational structures, including stairs and small bridges, cross the stormwater ditch at the toe. The ditch alternates between open grass- or rock-lined sections and culverts; while most sections are in good condition, some culverts show minor deterioration. The ditch ultimately discharges through a culvert beneath Glass Lake Road downstream of the spillway (Photos 10 and 11).

A rock-lined tributary enters the reservoir through the dike approximately 400 feet from the embankment, with a footbridge crossing it along the crest. The stormwater ditch passes beneath this tributary through a culvert (Photo 12).

4 Evaluation of Dam Spillway Capacity

The NYSDEC Guidelines for Design of Dams requires that the dam should:

- have sufficient spillway discharge capacity to safely pass the spillway design flood assuming routing of the flood hydrograph through the reservoir,
- have sufficient spillway discharge capacity to evacuate 75% of the storage between the maximum design high water and the spillway crest within 48 hours assuming no inflow, and
- have an energy dissipator at its terminus

4.1 SDF Capacity

A hydrologic analysis was completed in 2016 to estimate inflow to Glass Lake Dam for 150% of the 100-year flood event using Version 3.5 of the United States Army Corps of Engineers (USACE) Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS) computer software. The 4.3 square miles of drainage area to Glass Lake Dam was divided into six subbasins which used the Soil Conservation Service (SCS) curve number and unit hydrograph methods for the loss and transform methods, respectively. An additional two subbasins were used to evaluate precipitation directly onto Glass Lake and Crooked Lake. The model also includes three storage areas (i.e., Glass Lake, Crooked Lake, Low-lying area behind Glass Lake Dike) with stage-storage-discharge information. A Precipitation depth of 6.61 inches for the 24-hour duration storm with a 100-year average recurrence interval storm was obtained from the National Oceanic and Atmospheric Agency (NOAA) Atlas 14 publication. An SCS Type II storm pattern was utilized for the temporal distribution of this rainfall. Based on the information documented in the report attached as Appendix D, the model appears to be appropriate for estimating inflow to Glass Lake Dam.

The results of the HEC-HMS model were used as inputs to a two-dimensional hydraulic model of Glass Lake Dam which used Version 5.0.7 USACE Hydrologic Engineering Center River Analysis System (HEC-

RAS) to evaluate the depth and duration of overtopping for 150% of the 100-year flood event. The model utilized a digital elevation model with a 1-meter horizontal resolution and survey data of Glass Lake Road, which runs along the crest of Glass Lake Dam. In addition to inflows from the HEC-HMS model, the HEC-RAS model used the Glass Lake Dam spillway rating curve and reservoir storage information from the previously developed HEC-HMS model. The model produced a peak water level in Glass Lake of Approximately 831.6 feet, North American Vertical Datum of 1988 (NAVD88). Based on the information documented in the report attached as Appendix D, the model appears to be appropriate for routing flow at Glass Lake Dam. Based on the results, the spillway section and western abutment of Glass Lake Dam does not overtop during the SDF. However, the dike and Glass Lake Road east of the Dam experience overtopping. As previously discussed, the dike east of the dam would likely already be overtopped due to inadequate drainage of storm runoff behind the dike. However, a sensitivity analysis of the HEC-RAS model shows that the dike east of the dam would likely be overtopped even if there were no storm runoff entering the area behind the dike. As such, the spillway at Glass Lake Dam is considered to have insufficient capacity to pass the SDF.

4.2 Flood Evacuation

Computations provided in Appendix A show that the spillway can evacuate 75% of the storage between the maximum design high water (831.6 feet, NAVD88) and the spillway crest (826.11 feet, NAVD88) in approximately 29 hours assuming no inflow. Therefore, the spillway meets the NYSDEC requirement for drawdown capacity (i.e., sufficient spillway discharge capacity to evacuate 75% of the storage between the maximum design high water and the spillway crest within 48 hours assuming no inflow).

4.3 Energy Dissipator

Flow from the spillway discharges onto a rocky stream bed, forming a natural plunge zone that effectively dissipates hydraulic energy before the flow enters the downstream channel. The plunge area appears stable, with no evidence of scour, undermining, or displacement of the existing rock substrate. The downstream channel conveys the flow in and is generally in good condition, exhibiting no signs of erosion, bank sloughing, or sediment deposition that would indicate developing instability.

5 Evaluation of Dam Structural Stability

Stability analyses of the masonry spillway and earthen dike was performed as part of the 2016 EA by WSP. Both analyses were reviewed as part of this preparation of this EA.

5.1 Stability Analyses of the Masonry Dam and Spillway

The main section of the Glass Lake Dam consists of a stone masonry spillway and abutting non-overflow sections. The 2016 stability analyses evaluated these sections assuming the structure was constructed entirely of stone masonry. However, in March 2026, a portion of the downstream stone façade failed revealing earthfill behind the façade. This indicates the structure is constructed primarily of a granular fill with stone façades on the upstream and downstream faces.

Given the unit weight of soil is generally less than the assumed unit weight of the stone masonry (155 pcf), a revised stability analysis was performed as part of this EA to account for the lower assumed weight of the structure. The recent stability analysis was performed for the overflow and non-overflow sections assuming the structure consists of earthfill placed on a concrete slab with a stone masonry façade on both the upstream and downstream sides. There is no available geotechnical data for the earthfill or phreatic levels within either dam section. The earthfill was assumed to be granular and to have a unit weight of 120 pcf, based on site observations and engineering judgement.

The overflow and non-overflow sections of the spillway were evaluated using a conventional gravity analysis with a representative section on a per foot basis. Dimensions of the structures were based upon WSP's 2016 stability analysis and site observations. Conservatively, the unit weight of the stone façade and concrete pad were neglected in the analysis. Loading conditions assumed for the recent analysis are included in **Table 5.1-1**.

Table 5.1-1 Load Cases Assumed for 2026 Analysis

Load Case	Description	Headwater Elevation	Tailwater Elevation
I	Normal Pool	827.4	815.4
II	Normal Pool + Ice[1]	827.4	815.4
III	SDF[2]	831.6	816.7
IV	Earthquake[3]	827.4	815.4

1. *Ice loading assumed 5000lb/ft acting 0.5 ft below the spillway crest, representing a 1 ft thick ice sheet.*
2. *SDF elevations from Section 4.1.*
3. *Seismic analysis assumed a seismic coefficient of 0.114g.*

Silt was assumed to act on the upstream face of the dam from the crest of the spillway at El. 827.4. A unit weight of 120 pcf was assumed for the silt material, along with an at rest pressure coefficient of 0.5. Uplift pressures were assumed to act along the base of the structure varying linearly from full headwater to full tailwater.

The 2016 analysis assumed the spillway structure is founded on bedrock and there is a friction angle of 29 degrees between the concrete pad and bedrock. However, previous reports indicate the structure may in fact be founded on a gravel soil. The 29 degree friction angle is still appropriate for the gravel foundation and a friction factor of $\mu = 0.55$ was assumed for the recent stability analysis.

A seismic analysis was performed using site-specific seismic coefficients calculated using USGS 2023 National Seismic Hazard Model (NSHM) ground motions and the Risk Targeted Ground Motion (RTGM) tool in accordance with ASCE 7-22 guidelines. The site-specific design response was determined assuming 5% damping. Based upon USGS static seismic hazard maps (2023.R2 US NSHM) and an assumed site classification of B/C (soil and bedrock), the peak ground acceleration (PGA) for the site was determined to be 0.171g for a 1% probability of occurrence in 50 years. This was reduced by 2/3 in accordance with ASCE 7-22 guidelines such that the effective peak ground acceleration, s_a, used in the stability analysis was determined to be 0.114g. The seismic hazard results are included in Appendix C.

The material properties assumed are included in **Table 5.1-4**.

Table 5.1-4 Material Properties Assumed for 2026 Analysis

Property	Assumed Value
Unit weight of gravel fill	120 pcf
Unit weight of silt	120 pcf
At rest pressure coefficient of silt	0.5
Friction factor between concrete pad and soil foundation	$0.55 = \tan(29^{o})$

Property	Assumed Value
Seismic coefficient, s_a	0.114g

Results from the stability analysis are summarized in **Tables 5.1-5** below. The stability analyses are included in Appendix C.

Table 5.1-5 2026 Non-Overflow Section Stability Analysis Results

Load Case	Sliding FOS	Required FOS	Heel Pressure (ksf)	Toe Pressure (ksf)	Resultant Location (ft) (1)	Required Resultant Locations
Non-Overflow Section						
Normal Pool	5.1	1.5	1.3	2.3	1.6	Middle third
Normal Pool + Ice	2.9	1.3	1.0	2.6	2.6	Middle half
SDF	3.0	1.3	0.9	2.4	2.6	Middle half
Earthquake	1.9	1.0	0.8	2.7	3.0	Within base
Overflow Section						
Normal Pool	3.1	1.5	0.6	1.6	2.7	Middle third
Normal Pool + Ice	1.7	1.3	0.3	1.9	4.3	Middle half
SDF	1.7	1.3	0.2	1.6	4.4	Middle half
Earthquake	1.4	1.0	0.3	1.8	4.0	Within base

Notes: 1. Resultant location is distance from center of base which is located 17 ft from the toe of the structure.

The computed sliding factors of safety and bearing pressures meet stability criteria for all sections analyzed.

5.1.1 Evaluation of Non-Overflow Section after Façade failure

As discussed in the previous section, in March 2026, a portion of the stone façade on the downstream face of the non-overflow section failed, exposing the granular fill material within the dam. As of the preparation of this report, repairs are currently in design with construction to occur in the near future. Gomez and Sullivan was requested by the dam owner to evaluate the stability of the dam and estimate how much material could be lost from the downstream section.

For this analysis, an excel spreadsheet was utilized to calculate how much material could be lost from the downstream side of the non-overflow section before the recommended minimum factor of safety against sliding was met. This analysis only considered normal and normal and ice loading as it was assumed repairs would be made to the structure prior to a significant flood event and/or a seismic event. Fill material was assumed to fall away from the structure and leave a vertical face. This is generally unrealistic as the soil would likely fall to some angle leaving a sloped face. However, the vertical face is more conservative (it assumes more material is removed from the structure than if the soil slopes back) and is generally easier to analyze. Additionally, any stabilizing effects from the concrete pad below the earthen fill was ignored.

A cracked base analysis was also performed. When tension was calculated at the heel, the analysis assumed a theoretical crack with uplift pressure in the assumed crack length equal to 100% of the head water level. If the new analyses results indicated the tension zone exceeded the assumed crack length, the crack length was incrementally increased, the uplift pressure diagram modified, and a new analysis performed. This procedure of increasing the crack length and uplift pressure continued until the crack length stabilized.

Material properties and loading conditions were kept the same as discussed in Section 5.1. Results of the analysis are summarized in Table **5.1.1-1**. The stability analysis calculations are included in Appendix C.

Table 5.1.1-1 Results of the façade failure stability analysis for the non-overflow section

Load Case	Length of Material Removed from Downstream Face (ft)	Length of Dam After Failure (ft)	Crack Length (ft)	Calculated FOS	Required FOS
Normal Pool	24.0	10.0	0.4	1.5	1.5
Normal Pool plus Ice	19.4	14.6	3.3	1.25	1.25

Based on this analysis, it is recommended that no more 24 feet of material be removed from the dam face either due to the continued removal of material on the downstream face or during construction.

5.2 Slope Stability Analyses of the Dike

A slope stability analysis was performed in 2016 by WSP for a "conservative, composite, critical section" of the dike using GeoStudio SEEP/W and SLOPE/W software programs. The Morgenstern-Price method was used to analyze the embankment stability.

The geometry of the embankment was assumed from several previous reports and bathymetric and lidar surveys. The dike crest was assumed at El. 830.5. WSP noted that the dike was widened to its current configuration some time in the late 1800s; it was assumed that the placed fill is similar to the original dike material. A composite section of the dike was developed for the analysis using components from three separate dike sections. The composite section consisted of the steepest slopes for the upstream and downstream slopes and the narrowest crest.

There are no soils investigations for the site. According to the WSP report, material properties were assumed from general soil descriptions provided in previous reports and USGS maps. The dike embankment was assumed to consist of "*a mixture of sand, gravel, and silt, likely classified as SM or GM*". The foundation was assumed to consist of a dense glacial soil. Material properties were assumed based on engineering judgement. WSPs report noted that a "nominal cohesion" was assumed for the granular fill and foundation materials for drained loading conditions to remove shallower slope failures from consideration in the slope stability analysis.

Assumed material properties used in the stability analysis are summarized in **Table 5.2-1**.

Table 5.2-1 Material Properties Assumed for 2016 Slope Stability Analysis of the Earthen Dike

Material	Unit Weight (pcf)	Undrained Strength		Drained Strength		Permeability (cm/sec)
		Cohesion (psf)	Friction Angle (deg)	Cohesion (psf)	Friction Angle (deg)	
Embankment Fill (GM, SM)	120	1500	0	50	30	10^{-4}
Foundation Soil (GM, SM)	120	2000	0	50	30	10^{-4}

Loading conditions assumed for the 2016 slope stability analysis are summarized in **Table 5.2-2**.

Table 5.2-2 Summary of loading conditions for 2016 slope stability analysis

Load Case	HW Elevation	TW Elevation(1)	Slope	Required minimum factor of safety
Steady state seepage under normal loading	El. 827.4	El. 820.5	Downstream	1.5
Steady state seepage under design loading (SDF Pool)(2)	El. 830.5	El. 823.0	Downstream	1.4
Seismic Loading (3)	El. 827.4	El. 820.5	Downstream	>1.0
			Upstream	>1.0
Rapid drawdown from normal pool(4)	El. 827.4 to empty pool	El. 820.5	Downstream(5)	1.1
			Upstream	1.1
Rapid drawdown from design (SDF) pool	El. 830.5 to normal pool at El. 827.4	El. 820.5	Downstream(5)	1.3

Notes: 1. Tailwater assumed to be the invert of the drainage ditch at El. 820.5 for normal pool conditions and the top of the drainage ditch at El. 823 for design loading conditions.
2. The SDF calculated for the 2016 EA would overtopped the embankment dike. Therefore, the headwater under design loading was assumed at the dike crest at El. 830.5.
3. A seismic acceleration coefficient equal to 0.134g was assumed.
4. Rapid drawdown from normal pool was assumed to happen instantaneously from steady state conditions at normal pool to an empty pool condition.
5. During a flood event, the downstream side of the dike is anticipated to be flooded and tailwater is expected to reach the dike crest. A rapid drawdown case was evaluated assuming a flood occurs, saturating soils on the downstream side of the dike and then the water recedes rapidly to normal tailwater conditions.

The seismic design parameters were assumed using the USGS website. From USGS, the PGA for the site was calculated as equal to 0.084g for a site class D (stiff soil). The seismic acceleration coefficient A_s of 0.134g was calculated assuming a F_{pga} factor equal to 1.6 multiplied by the PGA. The A_s = 0.134g was used in the slope stability analysis to calculate the factor of safety under seismic loading.

The results of the slope stability analyses are summarized in **Table 5.2-3** below.

Table 5.2-3 2016 Dike Slope Stability Analysis Results

Load Case	Slope	FOS	Required FOS
Steady state seepage under normal loading	Downstream	1.37	1.5
Steady state seepage under design loading (SDF Pool) (2)	Downstream	1.07	1.4
Seismic Loading (3)	Downstream	1.04	1.0
	Upstream	1.12	1.0
Rapid drawdown from normal pool (4)	Downstream	1.24	1.1
	Upstream	0.83	1.1
Rapid drawdown from design (SDF) pool (5)	Downstream	0.92	1.3

Per the 2016 slope stability analysis, the dike does not meet the minimum required factor of safety under normal, design, rapid drawdown from normal pool (upstream slope) and rapid drawdown from design loading conditions (downstream slope). Based on our review of the 2016 analysis, we have several recommendations for the embankment dike:

- No subsurface explorations have been performed on the embankment dike. It is recommended that a boring program be developed for the site to verify the soil properties assumed as part of the 2016 stability analyses.
- The PGA of the project site should be confirmed via current USGS maps when a revised analysis is performed.
- A revised slope stability analysis should be performed using updated surveys of the embankment dike.
- Based on the recent evaluation of the spillway design capacity, the embankment dike will be overtopped during an SDF event (150% of the 100 year flood). The recent SDF analysis discussed in Section 4.1 states that headwater under this event will reach approximately El. 831.6 and therefore the dike will be overtopped by approximately 1.1 feet. It is recommended that the dike be raised to achieve sufficient freeboard under flood loading in accordance with NYSDEC Guidelines. A revised slope stability analysis should be performed as part of the design to raise the dike crest.

6 Evaluation of Dam Outlet Works (Hydraulic) Capacity

Computations provided in Appendix A show that the low-level outlet can evacuate 90% of the storage between the spillway crest (826.11 feet, NAVD88) and the bottom of the dam (814.11 feet, NAVD88) in approximately 12 days assuming no inflow. Therefore, the low-level outlet conduit meets the NYSDEC requirement for drawdown capacity (i.e., low level outlets should have the ability to drawdown 90% of the storage capacity of impounding lakes in 14 days).

7 Review of the Emergency Action Plan

While the approach used to develop EAP the inundation maps for Glass Lake Dam is generally acceptable for Class B dams according to NYSDEC guidelines, it is difficult to discern the estimated inundation extents, residences, and other pertinent features on the black and white EAP inundation map provided for review. It is recommended that the inundation maps be revised to more clearly show the areas downstream subject to inundation due to a dam breach.

8 Conclusions and Recommendations

The following recommendations are being made with regard to observations made during the site inspection and the findings of the engineering analyses discussed above.

A. Evaluate options to increase the spillway capacity to safely pass the SDF.
B. Subsurface explorations have not been performed on the embankment dike. To confirm the soil properties used in the 2016 stability analyses, a subsurface investigation consisting of a boring program is recommended.

C. It is recommended that a revised slope stability analysis be performed utilizing updated surveys of the embankment dike.
D. To achieve the minimum freeboard requirements under flood loading specified by NYSDEC guidelines, the dike should be raised. A revised slope stability analysis should be completed as part of the design process to assess the stability of the modified embankment.
E. Revise EAP inundation mapping to improve the identification and delineation of downstream areas potentially impacted by a dam breach.
F. Ensure the Emergency Action Plan (EAP) —with particular attention to the Notification Flowchart contacts—is verified and updated at least annually to maintain accuracy and operational readiness.
G. Continue to perform routine maintenance as outlined in the Inspection and Maintenance (I&M) Plan.

Appendix A – Hazard and Hydraulic Evaluations

Hazard Classification Supporting Information

The USACE DST webtool was used to evaluate an appropriate hazard classification for Glass Lake Dam. Unless otherwise noted, default parameters were selected for inputs to the webtool.

Elevation-Storage

Developed from the Glass Lake elevation-area information presented in Appendix C of the May 2015 report titled "Hydrologic and Hydraulic Assessment for Glass Lake Dam".

Elevation (ft)	Storage (ac-ft)
766.1	0.0
768.7	16.3
773.7	90.5
778.7	189.0
783.7	313.5
788.7	465.0
793.7	643.0
798.7	847.5
803.7	1080.3
808.7	1342.5
813.7	1637.0
818.7	1998.0
823.7	2450.5
826.1	2699.0
826.7	2769.2
827.7	2896.6
828.7	3027.8
829.7	3161.9
830.7	3298.4
831.7	3437.3
832.7	3578.4
834.7	3867.0

Inflow

Using a 0.1-hour timestep, the inflow hydrograph was set to 13 cfs for first 180 hours, with the remaining 120 hours using the sum of the Detention Pond and Glass Lake inflow hydrographs from the HEC-RAS model described in the July 27, 2020 memo titled "Glass Lake Dam Hydrologic and Hydraulic Analysis for Spillway Design Flood".

Elevation-Outflow

Developed from the Glass Lake water level and total flow from the Glass Lake Dam and Glass Lake Road features in the HEC-RAS model described in the July 27, 2020 memo titled "Glass Lake Dam Hydrologic and Hydraulic Analysis for Spillway Design Flood".

Pool Elevation (ft)	Outflow (cfs)
826.10	0
826.52	14
827.50	105
827.77	270
828.76	584
829.20	678
829.66	736
830.47	801
830.80	945
831.55	1745
833.00	3292

Breach

<u>Breach Method</u>: User-Defined

<u>Trigger</u>: Time (194.58, consistent with the timing of the peak water impoundment level without a breach)

<u>Breach Width (feet)</u>: 34

<u>Breach Bottom Elevation (feet)</u>: 815.4

<u>Side Slopes (H:V)</u>: 0:1

<u>Breach Development Time (hours)</u>: 0.2

LifeSim Compute

<u>Hazard Occurrence</u>: 12/9/2099 02:35:00

(consistent with the breach trigger time)

Optional Parameters

<u>Computational Timestep</u>: 1 second

Flood Evacuation Supporting Information

This analysis used the same elevation-storage information presented with the "Hazard Classification Supporting Information". Additionally, the analysis used the Glass Lake Dam outlet rating curve in the HEC-RAS model described in the July 27, 2020 memo titled "Glass Lake Dam Hydrologic and Hydraulic Analysis for Spillway Design Flood".

Elevation (ft, NAVD88)	Storage (ft3)	Notes
831.60	149,056,254	SDF Peak Water Level
827.58	125,441,583	Target Evacuation Level
826.11	117,570,026	Spillway Crest Elevation

Time (hrs)	Stage (ft, NAVD88)	Storage (ft^3)	Discharge (cfs)
0	831.6	149,056,254	632
1	831.22	146,782,386	580
2	830.88	144,694,122	522
3	830.57	142,815,655	471
4	830.28	141,119,334	421
5	830.03	139,602,924	355
6	829.81	138,324,880	324
7	829.61	137,158,819	296
8	829.43	136,092,957	271
9	829.26	135,116,941	249
10	829.11	134,220,959	229
11	828.97	133,396,595	211
12	828.84	132,635,637	195
13	828.72	131,931,846	181
14	828.60	131,279,887	168
15	828.50	130,674,766	156
16	828.40	130,112,764	146
17	828.31	129,588,854	136
18	828.22	129,100,414	127
19	828.14	128,643,003	119
20	828.07	128,214,650	112
21	828.00	127,812,366	105
22	827.93	127,433,725	99
23	827.87	127,077,336	94
24	827.81	126,740,666	89
25	827.75	126,422,022	84
26	827.70	126,120,439	79
27	827.65	125,834,785	75
28	827.60	125,563,094	72
29	827.55	125,304,683	68

Outlet Works Hydraulic Capacity Supporting Information

This analysis used the same elevation-storage information presented with the "Hazard Classification Supporting Information". Per the 2016 Engineering Assessment, the outlet works consists of a 2-foot diameter cast iron conduit that is 70 feet long. Submerged outlet culvert conditions (i.e., Type 4 culvert flow) were assumed, and the following equation was used to compute discharge from the outlet works.

$$Q = C_d A_o \sqrt{2g\left(\frac{h_1 - h_4}{1 + \frac{29 C_d^2 n^2 L}{R^{4/3}}}\right)}$$

Where,

Q = discharge (cubic feet per second)

C_d = Orifice Coefficient (dimensionless), assumed to be 0.6 for this analysis

A_o = Area of orifice (square feet), estimated to be approximately 3.14 square feet

g = gravitational constant (32.2 feet/second^2)

h_1= distance from headwater elevation to downstream pipe invert (feet), varies throughout the analysis

h_4 = distance from tailwater elevation to downstream pipe invert (feet), conservatively assumed to be 812.5 feet, NAVD88 for this analysis based on water levels immediately downstream of the dam associated with a flow of approximately 60 cfs from the HEC-RAS model described in the July 27, 2020 memo titled "Glass Lake Dam Hydrologic and Hydraulic Analysis for Spillway Design Flood"

n = Manning's roughness coefficient (seconds per feet$^{1/3}$), assumed to be 0.014 for this analysis

L = conduit length (feet), 70 feet

R = hydraulic radius (feet), 1 feet

Elevation (ft, NAVD88)	Storage (ft3)	Notes
826.11	117,570,026	Spillway Crest Elevation
818.86	77,063,332	Target Evacuation Level
814.11	72,562,588	Base Elevation of Dam

Time (hrs)	Stage (ft, NAVD88)	Storage (ft^3)	Discharge (cfs)
0	826.11	117,570,026	52.2
12	825.61	115,315,326	51.2
24	825.12	113,102,397	50.3
36	824.64	110,931,248	49.3
48	824.17	108,801,887	48.3
60	823.70	106,714,320	47.4
72	823.18	104,668,643	46.2
84	822.68	102,670,903	45.1
96	822.18	100,721,113	44.0
108	821.70	98,819,290	42.9
120	821.23	96,965,447	41.8
132	820.77	95,159,600	40.7
144	820.33	93,401,767	39.6
156	819.89	91,691,966	38.5
168	819.47	90,030,216	37.4
180	819.06	88,416,536	36.2
192	818.65	86,850,948	35.1
204	818.17	85,334,916	33.7
216	817.71	83,879,477	32.3
228	817.26	82,484,686	30.9
240	816.84	81,150,599	29.5
252	816.44	79,877,279	28.1
264	816.05	78,664,795	26.7
276	815.68	77,513,225	25.2
288	815.34	76,422,654	23.8
300	815.01	75,393,178	22.4
312	814.70	74,424,904	21.0
324	814.41	73,517,955	19.6
336	814.14	72,672,473	18.1

Appendix B – Photograph Log


Photo 1: Rock wall on downstream side of right embankment.


Photo 2: Voids in rock wall on downstream side of right embankment.

Photo 3: Spillway crest.

Photo 4: Bridge and abutment at spillway crest. Note erosion and exposed rebar under bridge deck.

Photo 5: Downstream face of dam. Note wooden planks.

Photo 6: Location of seepage at base of dam.

Photo 7: Low-level outlet at base of spillway.

Photo 8: Animal burrow on crest of dike.

Photo 9: Upstream slope of dike from west end looking east.

Photo 10: Stormwater ditch at toe of dike. Note recreational features on dike.

Photo 11: Deteriorating culvert section in stormwater ditch.

Photo 12: Tributary through dike. Note footbridge.

Appendix C – Stability Analysis

PROJECT NAME	Glass Lake	PROJECT NO.		2704	
SUBJECT	Site-specific Seismic Study	BY	TMD	DATE	7/14/2026

Design Task

Determine the site-specific MCE_R response spectrum for 5% damping using USGS 2023 NSHM ground motions

Determine the site-specific design response spectrum for 5% damping

Determine the pseudo-accelerations for the pseudo-dynamic method from given damping ratios and periods per the USACE method

USGS/ASCE Parameters

The following inputs were used for the online USGS Risk-Targeted Ground Motion Tool

Code	ASCE 7
Risk	1% PoC in 50 yr
Type	Static Hazard
Model	2023.R2 US NSHM
Latitude	42.629
Longitude	-73.533
Site Class	B/C
V_{s30}	760

Site-Specific MCE_R and Design Response Spectra

Steps

1. Obtain probabilistic (MCE_R) ground motions from USGS Risk Targeted Ground Motion tool (listed below as RTGM) — *ASCE 7 (2022) 21.2.1*
2. Obtain deterministic (MCE_R) ground motions, not needed if all values of probabilistic ground motions at every period are lower than lower limits from Table 21.2-1 (listed below). — *ASCE 7 (2022) 21.2.2*
3. The site-specific (MCE_R) response spectrum is the lesser of the probabilistic and deterministic spectra (listed below as S_{aM}) — *ASCE 7 (2022) 21.2.3*
4. The design response spectrum is taken as 2/3 of the site-specific response spectrum (listed below as S_a) — *ASCE 7 (2022) 21.3*
5. Determine design acceleration parameters — *ASCE 7 (2022) 21.4*

MCE_R Parameters

PGA_M	0.171	g
S_{MS}	0.252	g
S_{M1}	0.075	g·s

Design Parameters

S_a @ T=0	0.114	g	Effective peak acceleration
S_{DS}	0.168	g	
S_{D1}	0.050	g·s	

21.4 DESIGN ACCELERATION PARAMETERS

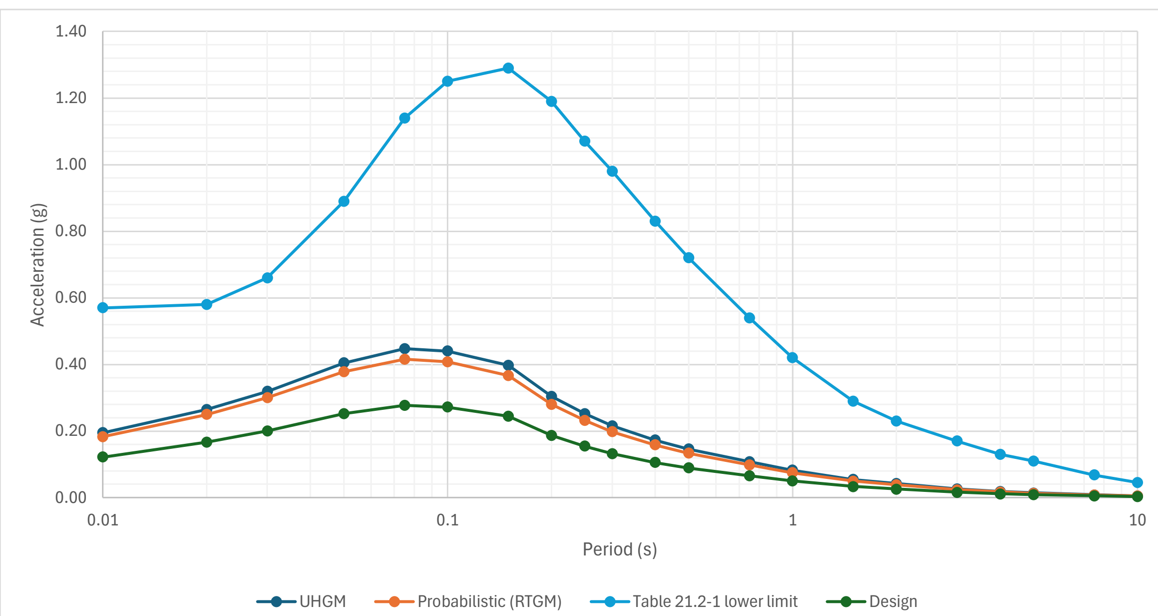
Where the site-specific procedure is used to determine the design ground motion, in accordance with Section 21.3, the parameter S_{DS} shall be taken as 90% of the maximum spectral acceleration, S_a, obtained from the site-specific spectrum at any period within the range from 0.2 to 5 s, inclusive. The parameter S_{D1} shall be taken as 90% of the maximum value of the product, TS_a, for periods from 1 to 2 s for sites with $\bar{v}_s >$ 1,450 ft/s ($\bar{v}_s >$ 442 m/s) and for periods from 1 to 5 s for sites with $\bar{v}_s \leq$ 1,450 ft/s ($\bar{v}_s \leq$ 442 m/s), but not less than 100% of the value of S_a at 1 s. The parameters S_{MS} and S_{M1} shall be taken as 1.5 times S_{DS} and S_{D1}, respectively.

PROJECT NAME	Glass Lake	PROJECT NO.	2704
SUBJECT	Site-specific Seismic Study	BY TMD	DATE 7/14/2026

Spectra

Is a deterministic MCE_R Spectrum needed according to ASCE 7 (2022) 21.2.2? NO

Period (s)	UHGM (g)	Prob. MCE_R RTGM (g)	T 21.2-1 (g)	Prob. MCE_R < T 21.2-1?	Site-Sp. MCE_R S_{aM} (g)	Design S_a (g)	$T \cdot S_a$ (g·s)
PGA	0.1831	0.1714	0.57	lower	0.171	0.114	
0.01	0.1949	0.1829	0.57	lower	0.183	0.122	
0.02	0.2649	0.2497	0.58	lower	0.250	0.166	
0.03	0.3190	0.3006	0.66	lower	0.301	0.200	
0.05	0.4045	0.3782	0.89	lower	0.378	0.252	
0.075	0.4471	0.4158	1.14	lower	0.416	0.277	
0.1	0.4403	0.4079	1.25	lower	0.408	0.272	
0.15	0.3973	0.3665	1.29	lower	0.367	0.244	
0.2	0.3042	0.2800	1.19	lower	0.280	0.187	
0.25	0.2524	0.2321	1.07	lower	0.232	0.155	
0.3	0.2157	0.1979	0.98	lower	0.198	0.132	
0.4	0.1728	0.1585	0.83	lower	0.159	0.106	
0.5	0.1462	0.1336	0.72	lower	0.134	0.089	
0.75	0.1081	0.0985	0.54	lower	0.099	0.066	
1	0.0825	0.0750	0.42	lower	0.075	0.050	0.050
1.5	0.0546	0.0495	0.29	lower	0.050	0.033	0.050
2	0.0420	0.0381	0.23	lower	0.038	0.025	0.051
3	0.0260	0.0236	0.17	lower	0.024	0.016	0.047
4	0.0182	0.0165	0.13	lower	0.017	0.011	0.044
5	0.0137	0.0125	0.11	lower	0.013	0.008	0.042
7.5	0.0083	0.0075	0.068	lower	0.008	0.005	
10	0.0052	0.0047	0.045	lower	0.005	0.003	

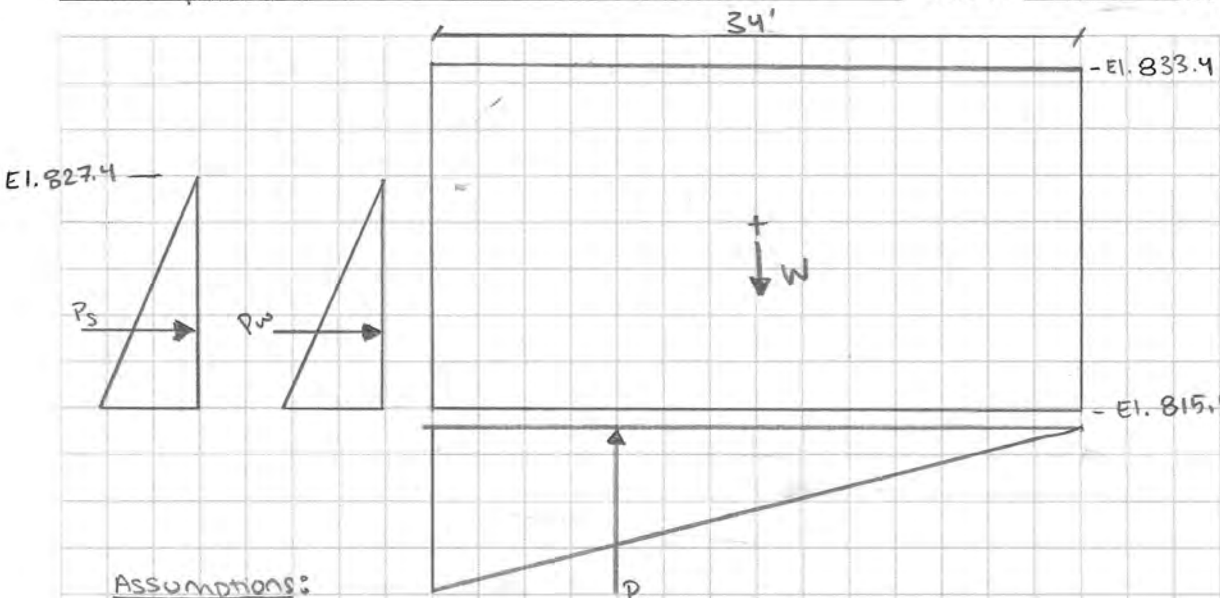


GOMEZ AND SULLIVAN ENGINEERS, D.P.C.
Engineers and Environmental Scientists

DESIGN BRIEF

PROJECT NAME Glass Lake Dam
SUBJECT Stability Analysis
Geometry of non-overflow section, material properties, assumptions

SHEET 1 of 11
PROJECT NO. 2704
BY EAP DATE 5/6/26
CHK'D TMD DATE 7/14/26



Assumptions:

+ Non-overflow section consists of earthfill placed on concrete slab with stone masonry facade on US & DS sides. This differs from previous analysis which assumed entire structure was stone masonry with $\gamma = 155$ pcf. Failure of stone facade in March, 2026 indicates the previous assumption is incorrect.
+ No DS tailwater. Tailwater assumed at toe at El. 815.4
+ Assume silt at crest of spillway at El. 827.4. This assumption is conservative as no silt information is currently known.
+ Uplift varies linearly from full headwater to 0 at toe, or TW (for SDF)
+ Assume earthfill consists of gravelly sand. $\gamma = 120$ pcf. Will neglect weight of concrete pad and stone facade.
+ Foundation consists of gravel, per 1916 inspection report. Assume $\mu = 0.55$
+ Structural dimensions assumed from previous analysis

Material properties

Unit weight structure, $\gamma_s = 120$ pcf
water, $\gamma_w = 62.4$ pcf

unit weight, silt $\gamma_{silt} = 120$ pcf.
assume at rest pressure coefficient $K_{o,silt} = 0.5$
ϕ concrete/gravel interface $= 29° = \tan\phi = 0.55$
(Note, same as WSP 2015 analysis which assumes a bedrock foundation.)

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Engineers and Environmental Scientists

DESIGN BRIEF

PROJECT NAME Glass Lake Dam
SUBJECT Stability Analysis
Loading conditions

PROJECT NO. 2904
BY EAP DATE 5/7/26
CHK'D TMD DATE 7/14/26

LOADING CONDITIONS

CASE	LOAD CASE	HW	TW	$FS_{req.}$	Loc. of Resultant
I	NORMAL POOL	El. 827.4	El. 815.4	1.5	middle third
II	NORMAL + ICE (1)	El. 827.4	El. 815.4	1.25	middle half
III	SDF (2)	El. 831.6	El. 816.7	1.25	middle half
IV	EARTHQUAKE (3)	El. 827.4	El. 815.4	1.0	w/in base

Notes 1. Ice assumed 5000 lb/ft acting 0.5 ft below spillway crest
2. SDF elevation from SDF capacity evaluation by GSE described in Section 4.1 of EA (2026)
3. Seismic evaluation performed assuming $S_a = 0.114g$ site specific seismic evaluation included Attachment A.

Calculations

Case I - Normal Pool @ El. 827.4

+ Structure Weight, $W = (833.4 - 815.4)(34')(120\text{ pcf}) = 73.44$ kip/ft

arm = 17 ft

$M_{weight} = 1248.48$ kip ft/ft

+ Water pressure, $P_W = (827.4 - 815.4)^2(1/2)(62.4\text{ pcf}) = 4.49$ kip/ft

arm = 4 ft

$M_{water} = 17.9$ kip ft/ft

+ Silt pressure, $P_S = (12\text{ ft})^2(1/2)(120 - 62.4\text{ pcf})(0.5) = 2.08$ kip/ft

arm = 4 ft

$M_{silt} = 8.32$ kip ft/ft.

+ Uplift, $P_u = (827.4 - 815.4)(1/2)(34')(62.4\text{ pcf}) = 12.73$ kip/ft

arm = $34'(2/3) = 22.67$ ft

$M_u = 288.59$ kip ft/ft.

Force	Horizontal (kip)	Vertical (kip)	Moment (kip ft)
weight, W		73.44 ↓	1248.48 ↺
water, P_W	4.49 →		17.90 ↻
Silt, P_S	2.08 →		8.32 ↻
uplift, P_u		12.73 ↑	288.59 ↻
	$\Sigma H = 6.57$	$\Sigma V = 60.71$	$\Sigma M_o = 314.81$ kip ft

$\Sigma M_r = 1248.48$ kip ft

$$FS_{sliding} = \frac{60.71 \tan(29°)}{6.57} = 5.12 \checkmark$$

$$FS_{overturning} = \frac{1248.48}{314.81} = 3.97 \checkmark$$

$$e = \frac{B}{2} - \frac{\Sigma M}{\Sigma V}$$

$$\text{eccentricity, } e = \frac{34'}{2} - \frac{(1248.48 - 314.81)}{60.71} = 1.62$$ within middle third.

$$\sigma_{heel} = \frac{60.71\,kip}{34'}\left(1 - \frac{6(1.62)}{34'}\right) = 1.27\,ksf \checkmark$$

$$\sigma_{toe} = \frac{60.71\,kip}{34'}\left(1 + \frac{6(1.62)}{34'}\right) = 2.30\,ksf. \checkmark$$

Case II Normal pool plus Ice

+ P_{ice} = 5000 lb/ft at 0.5ft from NP (assumed 1ft thick)

arm = 11.5 ft.

M_{ice} = 57.5k ft/ft

ΣH = 11.57 kip
ΣV = 60.71 kip
ΣM_o = 372.31 kip ft/ft
ΣM_r = 1248.48 kip ft/ft

$$FS_{sliding} = \frac{33.65\,kip}{11.57\,kip} = 2.91 \checkmark$$

$$FS_{overturning} = \frac{1248.48\,kip\,ft}{372.31\,kip\,ft} = 3.35 \checkmark$$

$$e = \frac{34}{2} - \left(\frac{1248.48 - 372.31}{60.71}\right) = 2.57 \checkmark$$

$$\sigma_{heel} = \frac{60.71}{34}\left(1 - \frac{6(2.57)}{34}\right) = 0.98 \checkmark$$

$$\sigma_{toe} = \frac{60.71}{34}\left(1 - \frac{6(2.57)}{34}\right) = 2.60 \checkmark$$

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Engineers and Environmental Scientists

DESIGN BRIEF

PROJECT NAME Glass Lake Dam
SUBJECT Non-overflow section

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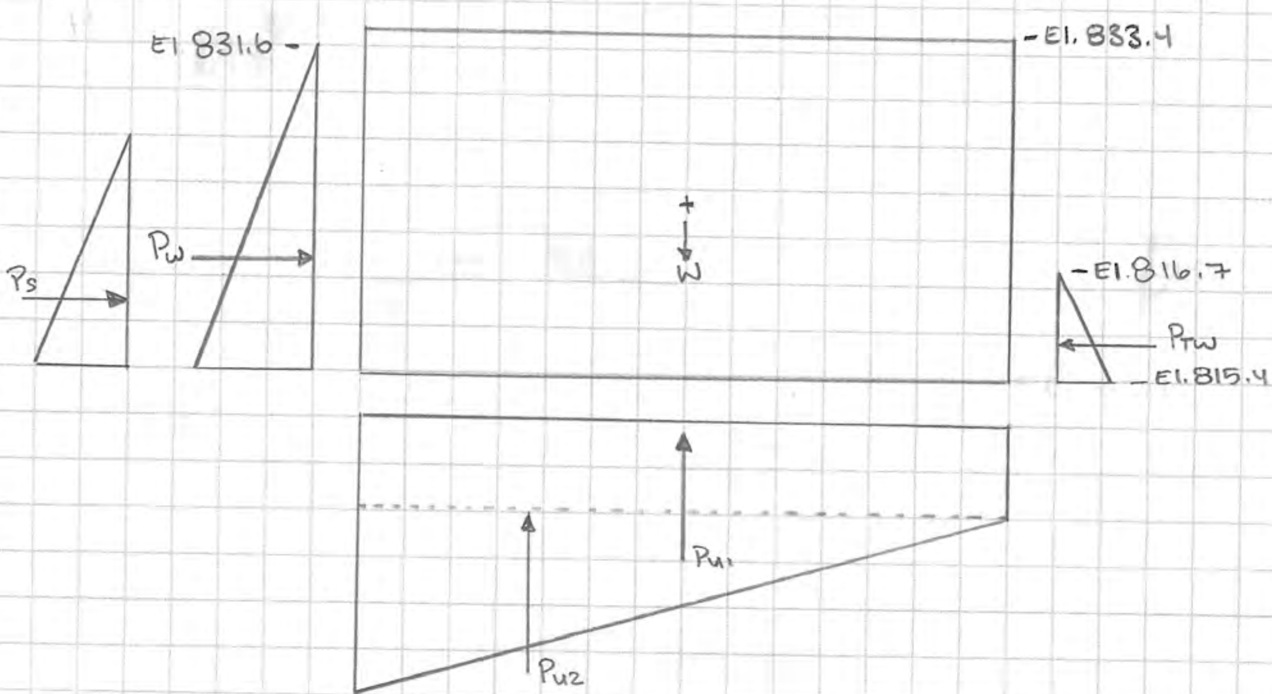
CASE III - SDF @ El. 831.6, TW @ El. 816.7 (per GSE 2026)

Note, DS TW pressure neglected



+ Water Pressure, $P_w = (831.6 - 815.4)^2 (1/2)(62.4) = 8.18$ kip
  arm = 5.4 ft
  $M_w = 44.17$ kip ft

+ Uplift, $P_{u1} = (816.7 - 815.4)(34)(62.4) = 2.76$ kip
  arm = 17 ft
  $M_{u1} = 46.89$ kip ft

  $P_{u2} = (831.6 - 816.7)(1/2)(34)(62.4) = 15.81$ kip
  arm = 22.67
  $M_{u2} = 358.32$ kip ft

Force	Horizontal (kip)	Vertical (kip)	Moment (kip ft)
Weight, W		73.44 ↓	1248.48 ↺
Water, P_w	8.18 →		44.17 ↻
Silt, P_s	2.08 →		8.32 ↻
Uplift P_{u1}		2.764	46.89 ↻
P_{u2}		15.814	358.32 ↻
	$\Sigma H = 10.26$	$\Sigma V = 54.87$	$\Sigma M_o = 457.7$
			$\Sigma M_r = 1248.48$

$$FS_{sliding} = \frac{54.87 \tan(29)}{10.26} = 2.96 \checkmark$$

$$FS_{overturning} = \frac{1248.48}{457.7} = 2.73 \checkmark$$

$$e = \frac{34}{2} - \left(\frac{1248.48 - 457.7}{54.87}\right) = 2.59 \checkmark$$

$$\sigma_{heel} = \frac{54.87}{34}\left(1 - \frac{6(2.59)}{34}\right) = 0.88 \text{ ksf} \qquad \sigma_{toe} = \frac{54.87}{34}\left(1 + \frac{6(2.59)}{34}\right) = 2.35 \text{ ksf}$$

CASE IV – Seismic PGA = 0.114g (see attachment A)

Westergaard Impulse of Water $P_{ew} = 2/3\, C_e (\alpha) h^2$

$$\text{where } C_e = \frac{51}{\sqrt{1 - 0.72\left(\frac{h}{1000 t_e}\right)^2}} = \frac{51}{\sqrt{1 - 0.72\left(\frac{12}{1000(1sec)}\right)^2}} = 51$$

$P_{ew} = 2/3\,(51)(0.114)(12)^2 = 0.558$ kip

arm = 0.4H = 4.8

$M_{ew} = 2.68$ kip ft/ft

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Engineers and Environmental Scientists

DESIGN BRIEF

PROJECT NAME Glass Lake Dam
SUBJECT Non-overflow section

SHEET 7 of 11
PROJECT NO. 2704
BY EAP DATE 6/19/26
CHK'D TMD DATE 7/14/26

Inertial load Dam $= W k_h = 73.44(0.114g) = 8.37$ kip/ft.

arm = 9 ft.

$M_e = 75.33$ kipft/ft

Silt, liquefied

$P_{es} = 2.08 \text{ kip/ft} \times \left(\frac{1}{0.5}\right) = 4.16$ kip/ft

arm = 4ft

$M_{es} = 16.64$ kip ft/ft.

Force	Horizontal (kip)	Vertical (kip)	Moment (kip ft)
Weight, W		73.44 ↓	1248.48 ↺
Water, P_w	4.49 →		17.90 ↻
Uplift, P_u,		12.73 ↑	288.59 ↻
Silt, P_s	4.16 →		16.64 ↻
Seismic, water P_{ew}	0.558 →		2.68 ↻
weight W_s	8.37 →		75.33 ↻
	$\Sigma H = 17.578$	$\Sigma V = 60.71$	$\Sigma M_o = 401.14$
			$\Sigma M_r = 1248.48$

$FS_{sliding} = \frac{60.71 \tan(29°)}{17.578} = 1.91$ ✓

$FS_{overturning} = \frac{1248.48}{401.14} = 3.11$

$e = \frac{34}{2} - \frac{(1248.48 - 401.14)}{60.71} = 3.04$ ✓

$\sigma_{heel} = \frac{60.71}{34}\left(1 - \frac{6(3.04)}{34}\right) = 0.83$ ✓

$\sigma_{toe} = \frac{60.71}{34}\left(1 + \frac{6(3.04)}{34}\right) = 2.74$ ✓

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Engineers and Environmental Scientists

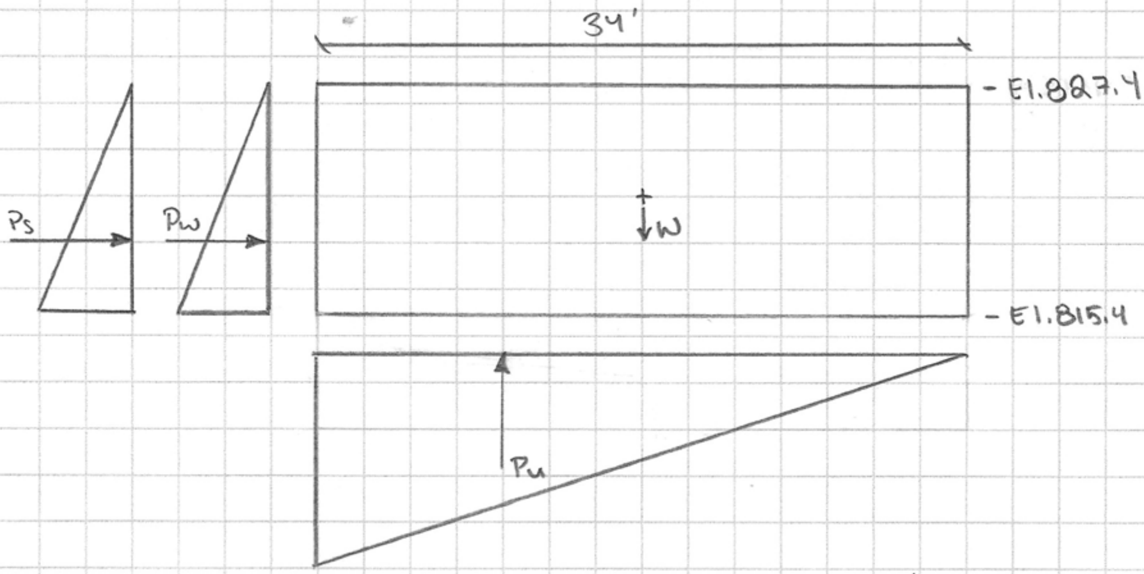
DESIGN BRIEF

PROJECT NAME Glass Lake Dam
SUBJECT Stability Analysis – overflow section

SHEET 8 of 11
PROJECT NO. 2704
BY EAP DATE 5/7/26
CHK'D TMD DATE 7/14/26

Overflow Section

CASE I - NORMAL POOL



+Structure weight, $W = (827.4 - 815.4)(34')(120\,pcf) = 48.96$ kip

arm = 17 ft $M_{weight} = 832.32$ kip ft

P_w, P_s, P_u same as non-overflow section

$\Sigma H = 6.57$ kip

$\Sigma V = 36.23$ kip

$\Sigma M_o = 314.81$ kip

$\Sigma M_r = 832.32$ kipft

$FS_{sliding} = \dfrac{36.23(\tan 29)}{6.57} = 3.06$ ✓ $FS_{overturning} = \dfrac{832.32}{314.81} = 2.64$ ✓

$e = 17 - \left(\dfrac{832.32 - 314.81}{36.23}\right) = 2.72$ ft ✓

$\sigma_{heel} = \dfrac{36.23}{34}\left(1 - \dfrac{6(2.72)}{34}\right) = 0.55$ ksf ✓ $\sigma_{toe} = 1.58$ ksf ✓

CASE II - NORMAL + ICE

P_{ice} = same as non-overflow

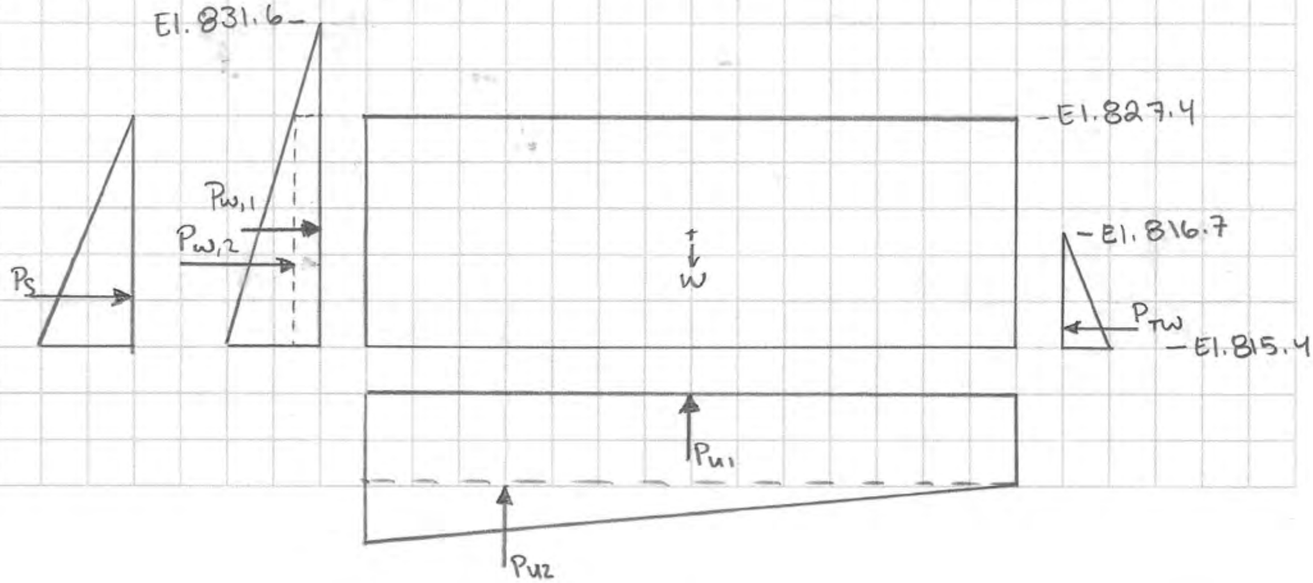
$\Sigma H = 11.57$ kip
$\Sigma V = 36.23$ kip
$\Sigma M_o = 372.31$ kip ft
$\Sigma M_r = 832.32$ kip ft

$FS_{sliding} = \frac{36.23 \tan(29)}{11.57} = 1.74$ ✓ $FS_{overturning} = \frac{832.32}{372.31} = 2.24$ ✓

$e = 17 - \frac{832.32 - 372.31}{36.23} = 4.30 \text{ ft}$ ✓

$\sigma_{heel} = \frac{36.23}{34}\left(1 - \frac{6(4.30)}{34}\right) = 0.26 \text{ ksf}$ $\sigma_{toe} = 1.87$ ksf

CASE III - SDF



GOMEZ AND SULLIVAN ENGINEERS, D.P.C.
Engineers and Environmental Scientists

DESIGN BRIEF

PROJECT NAME Glass Lake Dam
SUBJECT Stability Analysis
overflow section

SHEET 10 of 11
PROJECT NO. 2704
BY GAP DATE 5/7/26
CHK'D TMD DATE 7/14/26

neglecting overtopping water forces.

+ Water Pressure, $P_{W1} = (831.6 - 827.4)(62.4)(827.4 - 815.4) = 3.1$ kips/ft
arm = 6ft
$M_{W1} = 18.6$ kip ft/ft

$P_{W2} = (827.4 - 815.4)^2 (1/2)(62.4) = 4.5$ kips
arm = 4ft
$M_{W2} = 17.97$ kip ft/ft

Force	Horizontal (kips)	Vertical (kips)	Moment (kip ft)
Weight, W		48.96 ↓	832.32 ↺
Water, P_{W1}	3.1 →		18.6 ↻
P_{W2}	4.5 →		17.97 ↻
Silt, P_S	2.08 →		8.32 ↻
Uplift, P_{U1}		2.76 ↑	46.89 ↻
P_{U2}		15.81 ↑	358.32 ↻
	ΣH: 9.68 kips	ΣV: 30.39 kips	ΣM_o = 450.1 kip ft

$\Sigma M_r = 832.32$ kip ft
$\Sigma M_{net} = 382.22$ kip ft

$FS_{sliding} = \frac{30.39 \tan(29^\circ)}{9.68} = 1.74$ ✓ $\quad FS_{overturning} = \frac{832.32}{450.1} = 1.85$ ✓

$e = 17 - \left(\frac{382.22}{30.39}\right) = 4.42$ ✓

$\sigma_{heel} = \frac{30.39}{34}\left(1 - \frac{6(4.42)}{34}\right) = 0.2$ ✓ $\quad \sigma_{toe} = \frac{30.39}{34}\left(1 + \frac{6(4.42)}{34}\right) = 1.59$ ✓

CASE IV - SEISMIC

P_{ew} = same as non-overflow section = 0.558 kip/ft M_{ew} = 2.68 kip ft/ft
P_{silt} = same as non-overflow section = 4.16 kip/ft M_{es} = 16.64 kip ft/ft

$W k_h = 48.96\,(0.114g) = 5.58$ kip/ft
arm = 6 ft
$M_e = 33.48$ kip ft/ft

Force	Horiz (kip)	Vert (kip)	Moment (kip ft)
W		48.96 ↓	832.32 ↺
P_w	4.49 →		17.9 ↻
P_u		12.73 ↑	288.59 ↻
P_{silt}	4.16 →		16.64 ↻
P_{ew}	0.558 →		2.68 ↻
Wk_h	5.58 →		33.48 ↻
	$\Sigma H = 14.79$	$\Sigma V = 36.32$	$\Sigma M_o = 359.29$
			$\Sigma M_r = 832.32$

$$FS_{sliding} = \frac{36.23\tan(29)}{14.79} = 1.36 \checkmark \qquad FS_{overturning} = \frac{832.32}{359.29} = 2.32 \checkmark$$

$$e = 17 - \left(\frac{832.32 - 359.29}{36.23}\right) = 3.98 \checkmark$$

$$\sigma_{heel} = \frac{36.23}{34}\left(1 - \frac{6(3.98)}{34}\right) = 0.32 \checkmark \qquad \sigma_{toe} = \frac{36.23}{34}\left(1 + \frac{6(3.98)}{34}\right) = 1.82 \checkmark$$

PROJECT NAME	Glass Lake Dam	PROJECT NO.			2704
SUBJECT	Stability Analysis - Dam Failure Check	BY	EAP	DATE	6/19/2026
		CHK'D	TMD	DATE	7/14/2026

The stone façade on the downstream face of the non-overflow section failed in March 2026. Further failure of the dam face is a concern. Additionally, excavation of portions of the existing dam may be required as part of the repair efforts. An analysis was performed to calculate how much material can be removed from the face of the non-overflow section before the FS against sliding reaches the minimum recommended per NYS Guidelines.

LOADING CONDITIONS

Load Case	Description	Headwater Elevation	Tailwater Elevation	Notes
1	Normal Pool	827.40 ft	815.40 ft	
2	Normal + Ice	827.40 ft	815.40 ft	5000 plf ice load

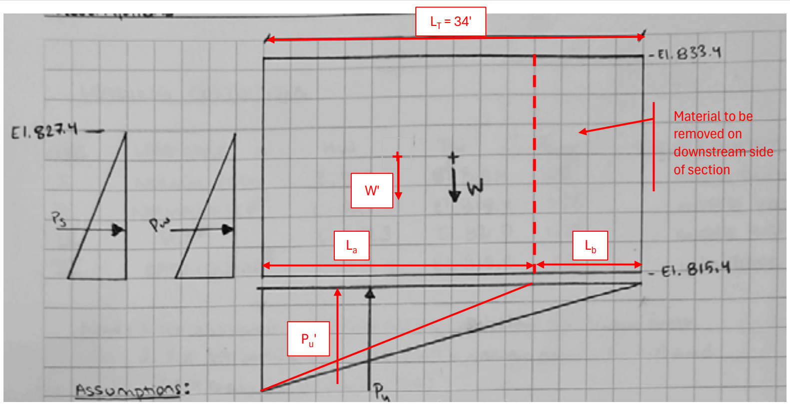
Note: Does not consider flood or seismic loading

SUMMARY OF ANALYSIS RESULTS

* Sliding FOS criteria taken from NYSDEC Guidelines for Design of Dams (1989), Section 10.6.4
* Overturning criteria taken from NYSDEC Guidelines for Design of Dams (1989), Section 10.6.2

		FOS sliding (w/o cohesion)			OT or Resultant Location		Base Pressures (psi)			
Load Case	Description	Criteria	Actual	Acceptable?	Criteria	Acceptable?	Minimum	Maximum	Allowable	Acceptable?
1	Normal Pool	1.5	1.50	YES	middle 1/3	YES	0.0	25.9	30	YES

DRAWINGS & PHOTOGRAPHS



$L_a = L_b = L_T$

L_a = length of section to remain

L_b = length of section removed

W' and P_u' = Weight and uplift forces revised for geometry of dam section to remain.

ASSUMPTIONS

* Non-overflow section consists of earthfill placed on concrete slab with stone masonry façade on US and DS sides.
* No DS tailwater applied
* Assume silt at crest of spillway at El. 827.4
* Uplift varies linearly from full HW to 0 at toe
* Assume earthfill consists of gravelly sand, γ = 120 pcf. Neglects weight of concrete pad and stone façade.
* Foundation (below concrete pad) consists of gravel, per 1916 inspection report. Assume μ=0.55.
* Structural dimensions assumed from previous analysis
* Ice load of 5 kip/ft applied 0.5-foot below spillway crest (assumes 1 foot thickness)
* Assume sliding along base of structure

PROJECT NAME	Glass Lake Dam	PROJECT NO.			2704	
SUBJECT	Stability Analysis - Dam Failure Check	BY	EAP	DATE	6/19/2026	
		CHK'D	TMD	DATE	7/14/2026	

The stone façade on the downstream face of the non-overflow section failed in March 2026. Further failure of the dam face is a concern. Additionally, excavation of portions of the existing dam may be required as part of the repair efforts. An analysis was performed to calculate how much material can be removed from the face of the non-overflow section before the FS against sliding reaches the minimum recommended per NYS Guidelines.

GEOMETRY & MATERIALS

Elevations			
Permanent spillway crest elevation	827.40	ft	Normal pool at spillway crest
Ice elevation	826.90	ft	Assume 1' thick ice sheet
U/S sediment elevation	827.40	ft	Silt at top of weir is critical condition versus no silt
Top of structure	833.4	ft	
Base elevation	815.40	ft	Assumed sliding plane
Dimensions			
Width of structure	9.95	ft	U/S - D/S direction
Width of original structure	34.0	ft	
Dam length	1.0	ft	along axis of dam. Assume unit width
Materials			
Fill behind masonry façade			
unit weight	120	pcf	
Water, unit weight	62.4	pcf	
Ice load	5	kip/ft	NYSDEC Guidelines for Design of Dams (1989), Section 10.5
Base/Foundation (native soil)			
unit weight	120	pcf	Assumed
friction angle	29	°	Assumed
cohesion	0	psf	Assumed
allowable bearing capacity	30	psi	Assumed
U/S Sediment			
unit weight	120	pcf	assumed
internal friction angle	30	°	assumed
at-rest earth pressure coefficient	0.50		

GRAVITY FORCES

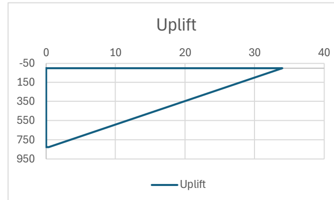
* Cross section approximated from dimensions on WSP calculations and site observations.

Part	Height (ft)	Width (ft)	Length (ft)	Unit Weight (pcf)	W (kip)	x (ft)	W*x (k-ft)	Notes
Non-overflow Section	18.0	9.95	1.0	120	21.5	4.98	107.0	
					21.5	4.98	107.0	

PROJECT NAME	Glass Lake Dam	PROJECT NO.			2704
SUBJECT	Stability Analysis - Dam Failure Check	BY	EAP	DATE	6/19/2026
		CHK'D	TMD	DATE	7/14/2026

The stone façade on the downstream face of the non-overflow section failed in March 2026. Further failure of the dam face is a concern. Additionally, excavation of portions of the existing dam may be required as part of the repair efforts. An analysis was performed to calculate how much material can be removed from the face of the non-overflow section before the FS against sliding reaches the minimum recommended per NYS Guidelines.

LOAD CASE 1 - NORMAL POOL



Elevations

HW	827.40	ft
TW	815.40	ft
Crest	833.40	ft
U/S sediment	827.40	ft
Base	815.40	ft

Crack Length 0.42 ft

Uplift Pressure Calculations

Point		Height	Width (US to DS)	Length	Unit Weight	Fv	Fh	Arm	Moment
		(ft)	(ft)	(ft)	(pcf)	(kips)	(kips)	(ft)	(k-ft)
U1	Δ	12.00	9.5	1.0	62.40	3.57		6.36	22.7
U2	☐	12.00	0.4	1.0	62.40	0.16		9.74	1.52
						3.73		6.50	24.2

Stability Calcs

Force	Shape	Height	Width	Length	Unit Weight	Fv (+ DN)	Fh (+ U/S)	Arm	Moment (+ Righting)	Notes
		(ft)	(ft)	(ft)	(pcf)	(kips)	(kips)	(ft)	(k-ft)	
DL	↓					21.5		4.98	107	Dead load of dam, see gravity forces calculation above
HW	Δ	12.00	12.00	1.0	62.4		-4.5	4.00	-18	Headwater against upstream face
SED	Δ	12.00	12.00	1.0	28.8		-2.1	4.00	-8	Sediment against U/S face (at-rest pressures)
UP	↑					-4		6.50	-24	Uplift, see Uplift Pressure Calculations above
						18	-7	3.18	56	

Cracked Base Analysis

Crack length 0.42 ft

Summation of Pressures

Horizontal	ΣH	7	kip
Vertical	ΣV	18	kip
Overturning	ΣMo	50	kipft
Resistance to Overturning	ΣMr	107	kipft

Base Pressures

Base in Compression	9.5	ft		
Area in Compression	96%			
Section Modulus	15	ft^3		
Eccentricity (cracked base)	1.59	ft		
Pressure at heel	0.00	ksf =	0.00	psi
Pressure at toe	3.73	ksf =	25.88	psi

Overturning or Resultant Location Analysis

Resultant within middle 1/3? YES

FS overturning 2.12

Sliding Analysis

Sliding driving force 6.6 kips

Sliding Plane at Base of Slab

Sliding resistance (friction) 9.8 kips

FS sliding 1.5

PROJECT NAME	Glass Lake Dam	PROJECT NO.			2704	
SUBJECT	Stability Analysis - Dam Failure Check	BY	EAP	DATE	6/19/2026	
		CHK'D	TMD	DATE	7/14/2026	

LOADING CONDITIONS

Load Case	Description	Headwater Elevation	Tailwater Elevation	Notes
1	Normal Pool	827.40 ft	815.40 ft	
2	Normal + Ice	827.40 ft	815.40 ft	5000 plf ice load

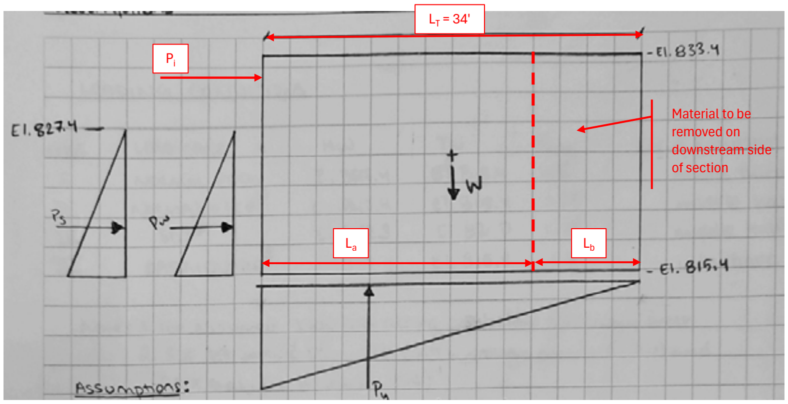
Note: Does not consider flood or seismic loading

SUMMARY OF ANALYSIS RESULTS

* Sliding FOS criteria taken from NYSDEC Guidelines for Design of Dams (1989), Section 10.6.4
* Overturning criteria taken from NYSDEC Guidelines for Design of Dams (1989), Section 10.6.2

		FOS sliding (w/o cohesion)			OT or Resultant Location		Base Pressures (psi)			
Load Case	Description	Criteria	Actual	Acceptable?	Criteria	Acceptable?	Minimum	Maximum	Allowable	Acceptable?
2	Normal Pool + Ice	1.5	1.25	NO	middle 1/3	YES	0.0	31.9	30	NO

DRAWINGS & PHOTOGRAPHS



ASSUMPTIONS

* Non-overflow section consists of earthfill placed on concrete slab with stone masonry façade on US and DS sides.
* No DS tailwater applied
* Assume silt at crest of spillway at El. 827.4
* Uplift varies linearly from full HW to 0 at toe
* Assume earthfill consists of gravelly sand, γ = 120 pcf. Neglects weight of concrete pad and stone façade.
* Foundation (below concrete pad) consists of gravel, per 1916 inspection report. Assume μ=0.55.
* Structural dimensions assumed from previous analysis
* Ice load of 5 kip/ft applied 0.5-foot below spillway crest (assumes 1 foot thickness)
* Assume sliding along base of structure

PROJECT NAME	Glass Lake Dam	PROJECT NO.			2704	
SUBJECT	Stability Analysis - Dam Failure Check	BY	EAP	DATE	6/19/2026	
		CHK'D	TMD	DATE	7/14/2026	

GEOMETRY & MATERIALS

* Dimensions are based on values given in project drawings

Elevations			
Permanent spillway crest elevation	827.40	ft	
Ice elevation	826.90	ft	Assume 1' thick ice sheet
U/S sediment elevation	827.40	ft	Silt at top of weir is critical condition versus no silt
Top of structure	833.4	ft	
Base elevation	815.40	ft	Assumed sliding plane
Dimensions			
Width of structure	14.6	ft	U/S - D/S direction
Width of original structure	34.0	ft	
Dam length	1.0	ft	along axis of dam. Assume unit width
Materials			
Fill behind masonry façade			
unit weight	120	pcf	
Water, unit weight	62.4	pcf	
Ice load	5	kip/ft	NYSDEC Guidelines for Design of Dams (1989), Section 10.5
Base/Foundation (native soil)			
unit weight	120	pcf	Assumed
friction angle	29	°	Assumed
cohesion	0	psf	Assumed
allowable bearing capacity	30	psi	Assumed
U/S Sediment			
unit weight	120	pcf	assumed
internal friction angle	30	°	assumed
at-rest earth pressure coefficient	0.50		

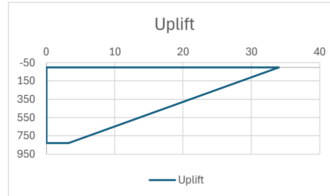
GRAVITY FORCES

* Cross section approximated from dimensions on WSP calculations and site observations.

Part	Height (ft)	Width (ft)	Length (ft)	Unit Weight (pcf)	W (kip)	x (ft)	W*x (k-ft)	Notes
Non-overflow Section	18.0	14.61	1.0	120	31.6	7.30	230.4	
					31.6	7.30	230	

PROJECT NAME	Glass Lake Dam	PROJECT NO.			2704		
SUBJECT	Stability Analysis - Dam Failure Check	BY	EAP	DATE	6/19/2026		
		CHK'D	TMD	DATE	7/14/2026		

LOAD CASE 2 - NORMAL POOL PLUS ICE



Elevations

HW	827.40	ft
TW	815.40	ft
Crest	833.40	ft
U/S sediment	827.40	ft
Base	815.40	ft

Crack Length 3.26 ft

Uplift Pressure Calculations

Point		Height	Width (US to DS)	Length	Unit Weight	Fv	Fh	Arm	Moment
		(ft)	(ft)	(ft)	(pcf)	(kips)	(kips)	(ft)	(k-ft)
U1	Δ	12.00	11.4	1.0	62.40	4.25		7.57	32.2
U2	☐	12.00	3.3	1.0	62.40	1.22		12.98	15.8
						5.47		8.77	48.0

Stability Calcs

Force	Shape	Height	Width	Length	Unit Weight	(+ DN) Fv	(+ U/S) Fh	Arm	(+ Righting) Moment	Notes
		(ft)	(ft)	(ft)	(pcf)	(kips)	(kips)	(ft)	(k-ft)	
DL	↓					31.6		7.30	230	Dead load of dam, see gravity forces calculation above
HW	Δ	12.00	12.00	1.0	62.4		-4.5	4.00	-18	Headwater against upstream face
SED	Δ	12.00	12.00	1.0	28.8		-2.1	4.00	-8	Sediment against U/S face (at-rest pressures)
ICE	→						-5.0	11.50	-58	
UP	↑					-5.5		8.77	-48	Uplift, see Uplift Pressure Calculations above
						26	-12	3.78	99	

Cracked Base Analysis

Crack length 3.26 ft

Summation of Pressures

Horizontal	ΣH	12	kip
Vertical	ΣV	26	kip
Overturning	ΣMo	132	kipft
Resistance to Overturning	ΣMr	230	kipft

Base Pressures

Base in Compression	11.4	ft		
Area in Compression	78%			
Section Modulus	21	ft^3		
Eccentricity (cracked base)	1.89	ft		
Pressure at heel	0.00	ksf =	0.00	psi
Pressure at toe	4.60	ksf =	31.91	psi

Overturning or Resultant Location Analysis

Resultant within middle 1/3? YES

FS overturning 1.75

Sliding Analysis

Sliding driving force 11.6 kips

Sliding Plane at Base of Slab

Sliding resistance (friction) 14.5 kips

FS sliding 1.25

Appendix D – Previous Analyses

HYDROLOGIC AND HYDRAULIC ASSESSMENT FOR GLASS LAKE DAM

DAM STATE ID: #226-1344

SUBMITTED TO: GLASS LAKE PRESERVATION CORPORATION

MAY 2016

HYDROLOGIC AND HYDRAULIC ASSESSMENT FOR GLASS LAKE DAM

DAM STATE ID: #226-1344

Project Location: Averill Park, NY
Rensselaer County

Date: May 2016

WSP Global Inc.
555 Pleasantville Road, South Bldg.
Briarcliff Manor, NY 10510

Phone: (914) 747-1120
www.wspgroup.com

QUALITY MANAGEMENT

ISSUE/REVISION	FIRST ISSUE	REVISION 1	REVISION 2	REVISION 3
Date	January 21, 2016			
Prepared by	Gregory Shaffer, PE			
Signature	G Shaffer			
Checked by	Allan Estivalet, PE			
Signature	Allan Estivalet			
P.E. License #:	094796			
Project number	I188096A			

PRODUCTION TEAM

CLIENT

Glass Lake Preservation Corporation	Peter T. Burkart, President GLPC

WSP GLOBAL INC. (WSP)

Project Manager	Allan Estivalet, PE
GIS Manager	Joshua Persson, CP, PPS, GISP
Engineers	Gregory Shaffer PE, Bruna Paranhos

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TABLES

FIGURES

APPENDICES

1 STATEMENT OF PURPOSE

Engineering assessments for dams in New York State include a complete hydrologic and hydraulic evaluation. The purpose of this analysis is to evaluate the Glass Lake Dam (State ID: 226-1344)'s spillway and outlet works capacities and review the current hazard classification of the structure. Glass Lake Dam is located in Averill Park, Rensselaer County, New York. In addition, this analysis will provide input water levels for the structural stability analyses to be performed for the Dam.

The Dam, regulated by the New York State Department of Environmental Conservation (NYSDEC), is privately owned by the Glass Lake Preservation Corporation. According to the NYSDEC Dam Hazard Classification System, Glass Lake Dam is listed as a Class B - Intermediate Hazard Dam.

2 PROJECT DESCRIPTION AND SITE INSPECTIONS

2.1 PROJECT DESCRIPTION

The Glass Lake Dam holds back water from the Wynants Kill, a small tributary of the Hudson River. The Dam, constructed circa 1866, is composed of a masonry section and an embankment section (called the dike) which maintains the reservoir at its normal level and prevents flooding in adjacent low lying areas.

The Dam's watershed is predominantly forested and moderately developed. The overall watershed is divided into six subbasins and has a combined drainage area of approximately 3.98 square miles. The Glass Lake Dam Watershed encompasses Glass Lake and Crooked Lake, which are hydraulically connected by a corrugated metal culvert underneath Route 49A.

2.2 SITE INSPECTION

WSP performed two site inspections of the Glass Lake Dam and its vicinity. The first site inspection was completed on September 4, 2015. During this inspection, our team performed a visual inspection of the masonry and embankment sections and collected basic construction details and dimensions of the Dam. A second inspection was done on November 20, 2015, focusing on the area downstream of the dam up to the Faith Mills Lower Dam (State ID# 226-0874).

3 APPROACH AND METHODOLOGY

NYSDEC uses a dam hazard classification system that dictates hydrologic criteria to be used for dam design purposes. Per NYSDEC Regulations (*Reference 14*), existing dams should have adequate spillway capacity to pass the following Spillway Design Floods (SDFs) without overtopping:

TABLE 2-1: EXISTING DAMS – REQUIRED SPILLWAY DESIGN FLOODS

Hazard Classification	Required Spillway Design Flood (SDF)
A	100 year
B	150% of 100 year
C	50% of the PMF

In addition, assuming no inflow, the service spillway should have sufficient capacity to evacuate 75% of the storage between the maximum design high water and the spillway crest within 48 hours.

The NYSDEC Dam Safety regulations also require that a dam's low-level outlet has sufficient capacity to draw down 90% of the dam impoundment's storage below the lowest spillway crest within 14 days, assuming no inflow into the Lake.

To determine the Glass Lake Dam's SDF, and to assess the capacities of the Glass Lake Dam spillway and low level outlet pipe, WSP has performed a comprehensive hydrologic and hydraulic analysis of the Dam Watershed, further described below.

Version 3.5 of the Hydrologic Engineering Center – Hydrologic Modeling System (HEC-HMS) software, developed by the United States Army Corps of Engineer (USACE), is used to simulate the precipitation-runoff processes through the watershed. The geographic information system software ArcGIS is used to manipulate, edit, and analyze spatial data necessary to perform the hydrologic analysis.

Precipitation events are determined over the entire watershed for this Project. The National Oceanic and Atmospheric Agency (NOAA) Atlas 14 contains precipitation frequency estimates for selected storm durations in New York State and is used for this analysis.

Infiltration losses are determined using the averaged National Resources Conservation Service (NRCS) Curve Numbers (CN) method. NRCS's Curve Number Method is used to translate excess precipitation into a simulated discharge hydrograph at the outlet of the watershed. The lakes included in the watershed are modeled into HEC-HMS using different sub-basins. Flood routing through these lakes is performed using the level pool routing method within HEC-HMS. This method allows us to take into account storage in the lakes, translating into peak outflow attenuation within Glass Lake and providing more accurate flow hydrographs.

The Glass Lake Dam's Spillway Rating Curve is developed based on the field measurements collected during our site inspections and discharge coefficients determined using empirical relationships developed by the U.S.

Geological Survey - USGS (*Reference X*). In addition to the dam service spillway composed of two bays underneath Glass Lake Road, two additional broad-crested spillways are modelled. These spillways simulate the overtopping that may occur over Glass Lake Road and the dike during large flood events.

Finally, the Glass Lake Dam's outlet pipe is assessed using our in-house Excel spreadsheet to model the lake drawdown. The calculation solves the energy equation for steady, incompressible, non-viscous flow. Friction loss in the conduits will be modeled using the Hazen-Williams friction loss equation.

4 ASSUMPTIONS AND JUSTIFICATION

- Elevations are referenced to North American Vertical Datum of 1988 (NAVD 88). At the Glass Lake Dam Site (Lat: 42˚ 37' 45.0'' N/Long: 73˚ 32' 0.0'' W), the difference between NAVD-88 and the National Geodetic Vertical Datum of 1929 (NGVD-29) is approximately - 0.577 feet.

- The 100-year storm (6.61 inches of rain in 24 hours) occurs uniformly over the Glass Lake Dam Watershed. The temporal precipitation distribution is a Type II distribution.

- The hydraulic system, composed of the corrugated metal pipe underneath Route 49A, the cast iron pipe underneath the garage which drains the water from the ditch to the Wynant kills, and the service spillway, is assumed to perform as intended during flooding conditions.

- The dimensions of Glass Lake Dam and its appurtenant structures are determined based on available topographic surveys (*Reference 12*) and the measurements taken during the site visits.

5 HYDROLOGIC AND HYDRAULIC ANALYSES

5.1 HYDROLOGIC ANALYSIS

To determine the Dam's SDF hydrograph, a hydrologic analysis of the Glass Lake Dam Watershed is performed. As previously mentioned, the dam's watershed is approximately 3.98 square miles. The majority of the drainage basin is undeveloped, although portions of the basin contain residential developments.

As explained in Section 3, HEC-HMS 3.5 is used to calculate the SDF hydrograph at Glass Lake Dam. HEC-HMS is a generalized modeling system capable of representing many different watersheds. A model of the watershed is constructed by separating the hydrologic model into a physical model and meteorological model. Sub-basins are constructed within the overall watershed based on the drainage patterns observed within the overall watershed. The two main pieces of the model, further described below, are:

- Basin Model
- Meteorological Model

The complete HEC-HMS model is presented in *Appendix D,* and all the pertinent model data are placed on CD and are provided in *Appendix G.*

Glass Lake Dam Basin Model

Watershed Physical Model

The Glass Lake Dam Watershed is delineated using the software ArcGIS, produced by ESRI. The watershed characteristics are determined using GIS data. ArcGIS is used to analyse the Digital Elevation Model (DEM), Land Use and Land Cover (LULC) data, and Soil Survey Geographic (SSURGO) data. Figures displaying this information are presented in *Appendix A*. Based on the LULC and soil data, watershed curve numbers are determined to evaluate the runoff processes. The curve numbers obtained for the six sub-basins are summarized in Table 5-1.

TABLE 5-1: WEIGHTED SCS CURVE NUMBER

Sub-basin	Weighted SCS Curve Number
1	77
2	76.9
3	76.4
4	76.3
5	76.6
6	77

For the purpose of this analysis, the watershed is divided into 6 subbasins (See *Appendix A* – Figure A-1). Two lakes (Crooked Lake and Glass Lake) are included in HEC-HMS to account for the hydrologic routing effects. Elevation-area functions are developed using DEM data to determine the elevation-area relationships for the lakes. An elevation-area function is also developed for the “bathtub area” forming directly downstream from the dike during significant flood events. The elevation-area functions developed for this project are provided in *Appendix C*.

Runoff Method

Runoff is simulated in HEC-HMS using mathematical methods describing the rainfall-runoff relationships for a drainage basin. The meteorological data taken into account in this analysis is described in this section.

The runoff for the watershed is estimated using the Natural Resources Conservation Service (NRCS) Runoff CN method. CNs are calculated based on watershed’s hydrologic soil groups (*Appendix A* – Figure A-4), and land use and land cover (*Appendix A* – Figure A-3). The CN method determines the quantity of rainfall that is abstracted from the precipitation via infiltration, evapotranspiration, localized detention, and other means. The difference between the precipitation and the abstracted rainfall is the excess precipitation which becomes runoff. CN Numbers for the watershed are presented in *Appendix C* - Figure A-5.

SCS Dimensionless Unit Hydrograph Method

The SCS Unit Hydrograph (UH) method is used to transform excess precipitation into a simulated discharge hydrograph at the outlet of each sub-basin. The SCS UH Method develops a synthetic hydrograph using the calculated basin lag time. The lag time is taken to be 60% of the time of concentration, which is determined using the velocity method.

The velocity method breaks the flow path into segments based on three flow regimes: sheet flow, shallow concentrated flow, and channel flow. Sheet flow occurs first before transitioning into shallow concentrated flow. Due to the small size of sub-basins 1, 5, and 6 and the lack of developed channels, channel flow was not considered for these sub-basins. Sheet flow was assumed to be limited to 50 ft. before transitioning into shallow concentrated flow. For sub-basins 1, 2, and 4, sheet flow was limited to 25 ft. before transitioning to shallow concentrated flow. The transition between shallow concentrated flow and channel flow corresponds to the position where the defined channel starts to be visible on orthoimagery.

The computed time of concentration and lag time for each sub-basin are presented in Table 5-1. Lag time is equal to 60% of the time of concentration. The calculations for the lag times are provided in *Appendix D*.

Table 5-2: SCS UNITS HYDROGRAPH METHOD PARAMETERS

Sub-basin	Time of Concentration (minutes)	Lag Time (minutes)
1	73.8	44.3
2	133.8	80.3
3	20.8	12.5
4	69.8	41.9
5	11.2	6.7
6	23.2	13.9

Routing Method through Reservoirs

As previously mentioned, two lakes are included in the HEC-HMS model to account for hydrologic routing effects through the water bodies. Hydrologic reservoir routing is the physical translation of an inflow hydrograph through an impoundment to the downstream reach. The outflow hydrograph is delayed in time and reduced in peak magnitude relative to the inflow hydrograph. The flood routing through the reservoir is performed using the level pool routing method in HEC-HMS.

Inflow hydrograph I(t) and storage outflow characteristics O(t) are input and lake water levels are assumed uniform throughout. Reservoir outflow is determined from a discretized form of the continuity equation shown below (*Equation 1*):

$$\frac{dS(t)}{dt} = I(t) - O(t)$$

<u>Glass Lake Dam Watershed Meteorological Model</u>

The cumulative depths of the analyzed rainfall events are determined using NOAA Atlas 14 (*Reference 11*). The Glass Lake Dam's SDF is computed using the 100-year precipitation event over a 24-hour period at the dam site. The temporal rainfall distribution is assigned based on the geographic location of the watershed within the United States. The NRCS developed four synthetic 24-hour rainfall distributions (I, IA, II and III) from available National Weather Service (NWS) duration-frequency precipitation, each relevant for a specific region of the country (*Reference 7*). A type II distribution is used for this analysis, which corresponds to the location of the Glass Lake watershed.

To determine the 150% of the 100-year flood, the percentage is applied to the outflow hydrographs and not to the rainfall value according to Guidance for Design of Dams (*Reference 5*).

Low-lying area directly downstream from the Glass Lake Dike

Glass Lake sub-basin 1 drains directly behind the Glass Lake Dike through a small channel which releases water into Glass Lake through the Dike (Figure 5-1). Some of the water from sub-basin 1 is also collected in a small ditch, which runs at the Dike toe, and conveys water into a two feet diameter pipe underneath Glass Lake Road to the Wynant Kill.

FIGURE 5-1: DITCH DRAINING SUB-BASIN NO. 1 AND DISCHARGING INTO GLASS LIKE THROUGH GLASS LAKE DIKE

Channel draining Sub-basin 1

Channel outlet into Glass Lake through the Glass Lake Dike

During significant flood events, water accumulates behind the dike prior to eventually releasing into the Wynants Kill. This phenomenon is due to the relatively flat slope of the ditch as well as the limited capacity of the pipe underneath Glass Lake Road. During Hurricane Irene, in August 2011, the water built up only inches behind the dike's crest to an elevation significantly superior to the water elevation observed in Glass Lake. The hydraulic model developed for this project simulates this situation. Results are presented into the subsequent sections.

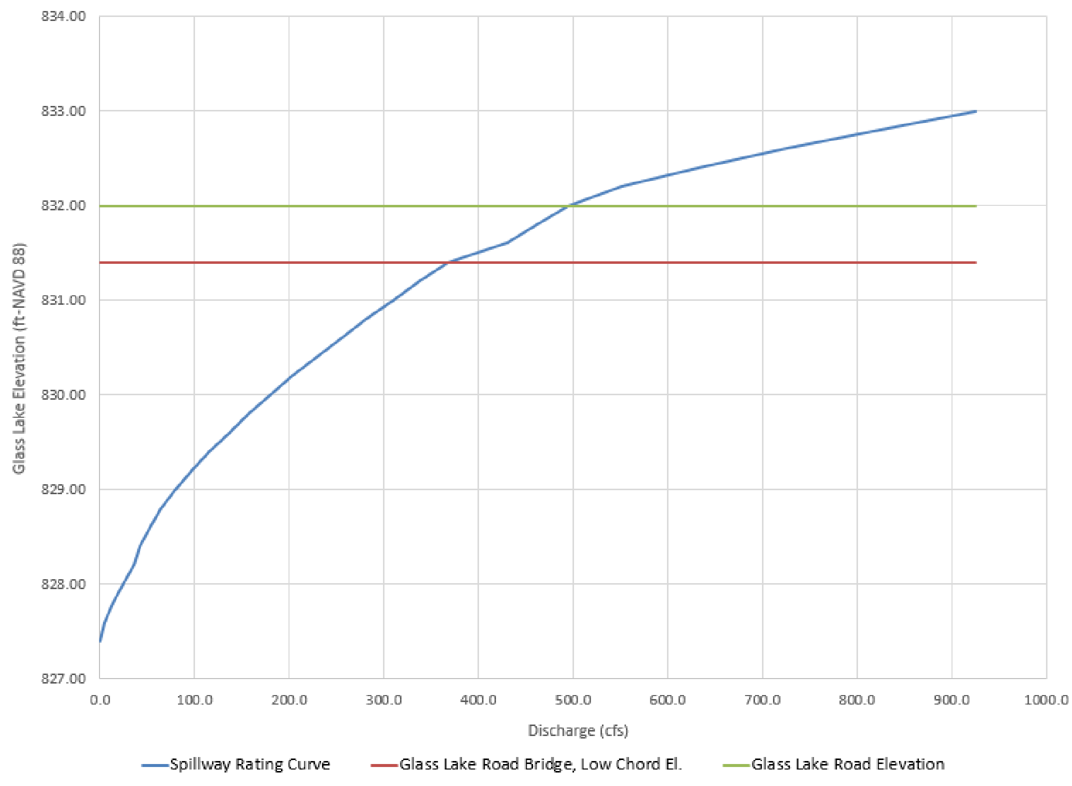
5.2 HYDRAULIC ANALYSIS

Spillway Capacity Evaluation

The Glass Lake Dam's spillway rating curve is developed based on the field measurements collected during our site inspections and discharge coefficients determined using empirical relationships developed by the U.S. Geological Survey - USGS (*Reference X*).

Figure 5-2 presents the computed Spillway Rating Curve for Glass Lake Dam. The spillway crest is at El. 827.4 ft. The dam service spillway, composed of two bays underneath Glass Lake Road, functions as a broad-crest spillway up to El. 831.4 ft. which represents the low chord El. of the bridge over the spillway. When the water reaches El. 831.4 ft., the flow transitions into pressurized flow. The spillway capacity is evaluated under the SDF scenario to determine if the spillway has enough capacity to keep the water surface elevation in the reservoir under control.

FIGURE 5-2: GLASS LAKE DAM SERVICE SPILLWAY RATING CURVE



The HEC-HMS Model is also used to determine whether the service spillway has sufficient capacity to evacuate 75% of the storage between the maximum design high water and the spillway crest within 48 hours.

Outlet Works Capacity Evaluation

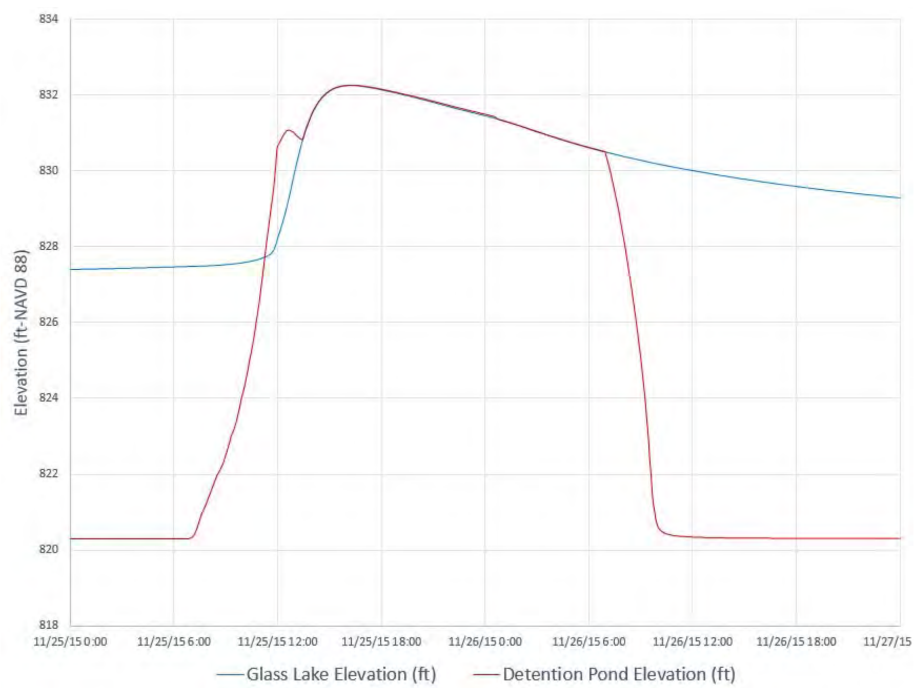
Glass Lake Dam has a 30-inch diameter cast iron low-level outlet conduit. NYSDEC dam safety regulations require outlet works to have sufficient capacity to evacuate 90% of the reservoir storage volume within 14 days, assuming no inflow. The 30-inch low-level outlet has been evaluated based on this criterion to determine if the pipe has the adequate capacity to lower Glass Lake.

The hydraulic evaluation is performed by applying the conservation of energy principle for a steady incompressible fluid. The time required to release 90% of the Glass Lake Volume above El. 815.34 (natural ground elevation) is calculated at one-foot intervals until approximately 90% of the storage is evacuated. The calculations are completed in a Microsoft Excel spreadsheet. The detailed calculation is included in *Appendix F.*

6 SUMMARY OF RESULTS AND CONCLUSIONS

The SDF, as defined by the NYSDEC, is the largest flood that a given spillway is designed to pass safely. For a Class 'B' Intermediate Hazard Dam, this corresponds to a flood event equal to 150% of the 100 year flood. This flood scenario is simulated using the developed HEC-HMS model. The fluctuation of the water elevations in Glass Lake and within the Low-lying area ("detention pond") is presented on Figure 6.1.

FIGURE 6-1: GLASS LAKE AND DETENTION POND'S WATER SURFACE ELEVATIONS DURING THE SDF



During the SDF storm event, the hydraulic analysis demonstrates that the water level in the detention pond rises quicker than in Glass Lake and that the Dike eventually overtops when the water elevation reaches El. 830.5 ft. At this point, water flows from the detention pond into Glass Lake overtopping the Dike crest.

Subsequently, the water level in Glass Lake continues to rise due to discharge from its other upstream watersheds. The water level rises above the Dike crest elevation (830.5 ft.) and flows back into the detention pond. The water level in Glass Lake and the detention pond continue to rise until they peak at 832.3 ft. at which point the water surface elevation beings to fall. As the water level falls, the detention pond and Glass Lake are continually connected until the water surface elevation hits 830.5 ft. and the detention pond is cut off from Glass Lake by the Dike. Once the detention pond is isolated from Glass Lake, the water surface elevation in the detention pond drops much faster.

Since the Dike is made of earthen material, it is likely to erode and would probably breach during the SDF event. However, the detention pond is already flooded by discharge from the upstream subbasin. There are no houses within the flooded area aside from one house that would be subject to flooding of its basement.

The masonry dam and Glass Lake Road overtop as well, by about 0.3 ft. for a limited period of time.

FIGURE 6-2: LOW-LYING AREA DOWNSTREAM OF GLASS LAKE DAM WITH WATER EL. AT 832.3

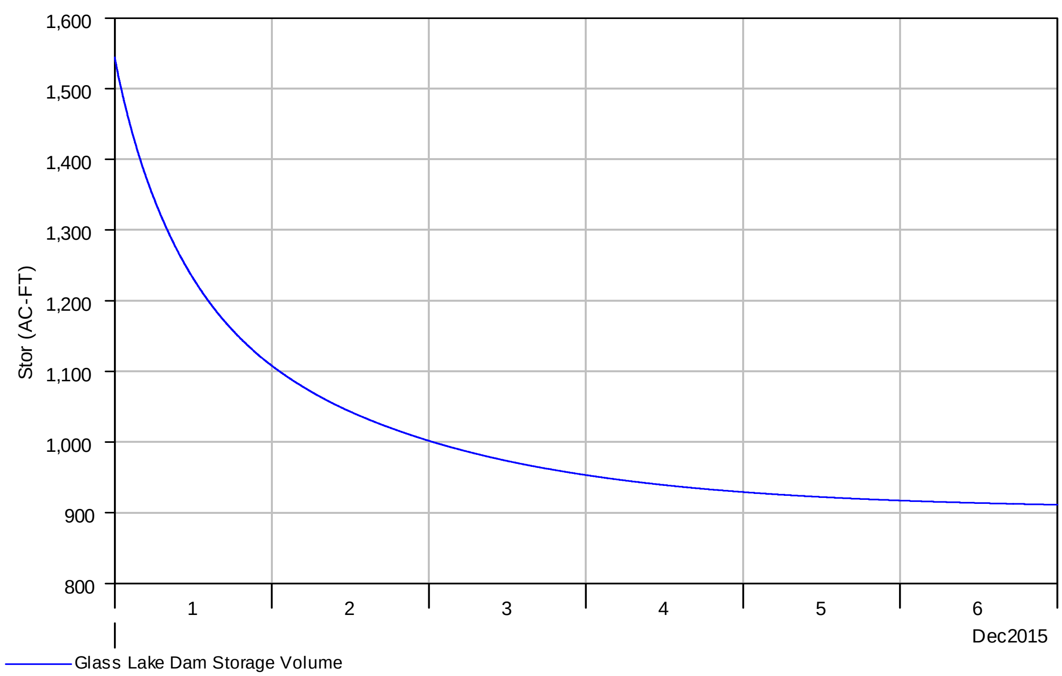
Complete results including the flood hydrograph, lake storage, and elevation fluctuations during the hydrologic events analysed are provided in *Appendix E*.

Service Spillway Capacity

The hydrologic and hydraulic analysis performed for the Glass Lake Dam shows that the Dam's spillway does not have a sufficient capacity to pass the SDF without overtopping. More specifically, the analysis results show that the Dike overtops of 1.8 ft. and the Glass Lake Road overtops by 0.3 ft.

The peak water surface elevation during the SDF is 832.3 ft., corresponding to a peak storage volume in the reservoir of 1544 ac-ft. The normal storage behind the dam when the water surface elevation is at the spillway invert (827.4 ft.) is 900 ac-ft. Therefore, the surplus storage in the reservoir during the SDF is 644 ac-ft. To be compliant with the NYSDEC Dam Safety Regulations, the service spillway must be able to lower the reservoir volume from 1544 ac-ft. to 1061 ac-ft. within 48 hrs. As shown on Figure 6-3, the service spillway reduces the storage volume in the reservoir to 1000 ac-ft. within 48 hrs, and therefore meets the regulatory requirement.

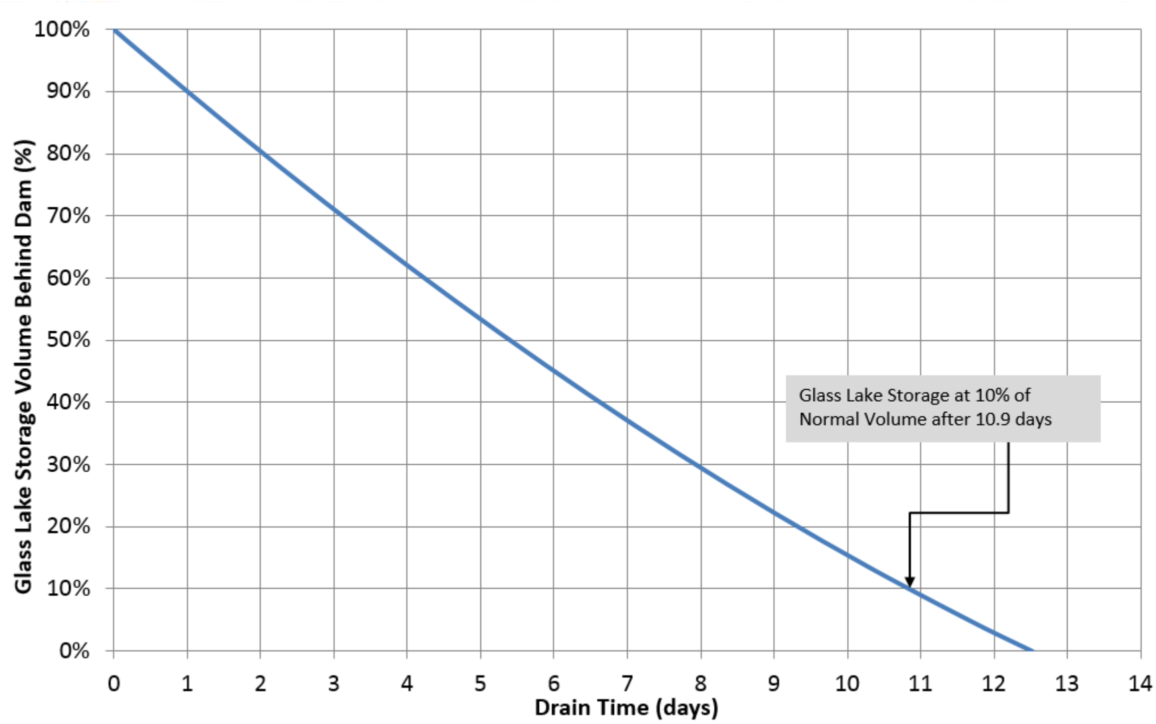
FIGURE 6-3: STORAGE DRAWDOWN VIA SERVICE SPILLWAY FOR GLASS LAKE DAM



Outlet works capacity

Based on the calculation, the low level outlet at Glass Lake Dam can evacuate approximately 90 percent of the storage capacity of Glass Lake in approximately 10.9 days. The calculation assumes that the gate valve is fully open and accounts for a reduction of the conduits hydraulic capacity due to possible corrosion and debris. Therefore, the low-level outlet conduit meets the NYSDEC requirement for drawdown capacity. The detailed calculation is provided in *Appendix F*.

FIGURE 6-4: GLASS LAKE DAM'S LOW LEVEL OUTLET DRAWDOWN CURVE



7 REFERENCES

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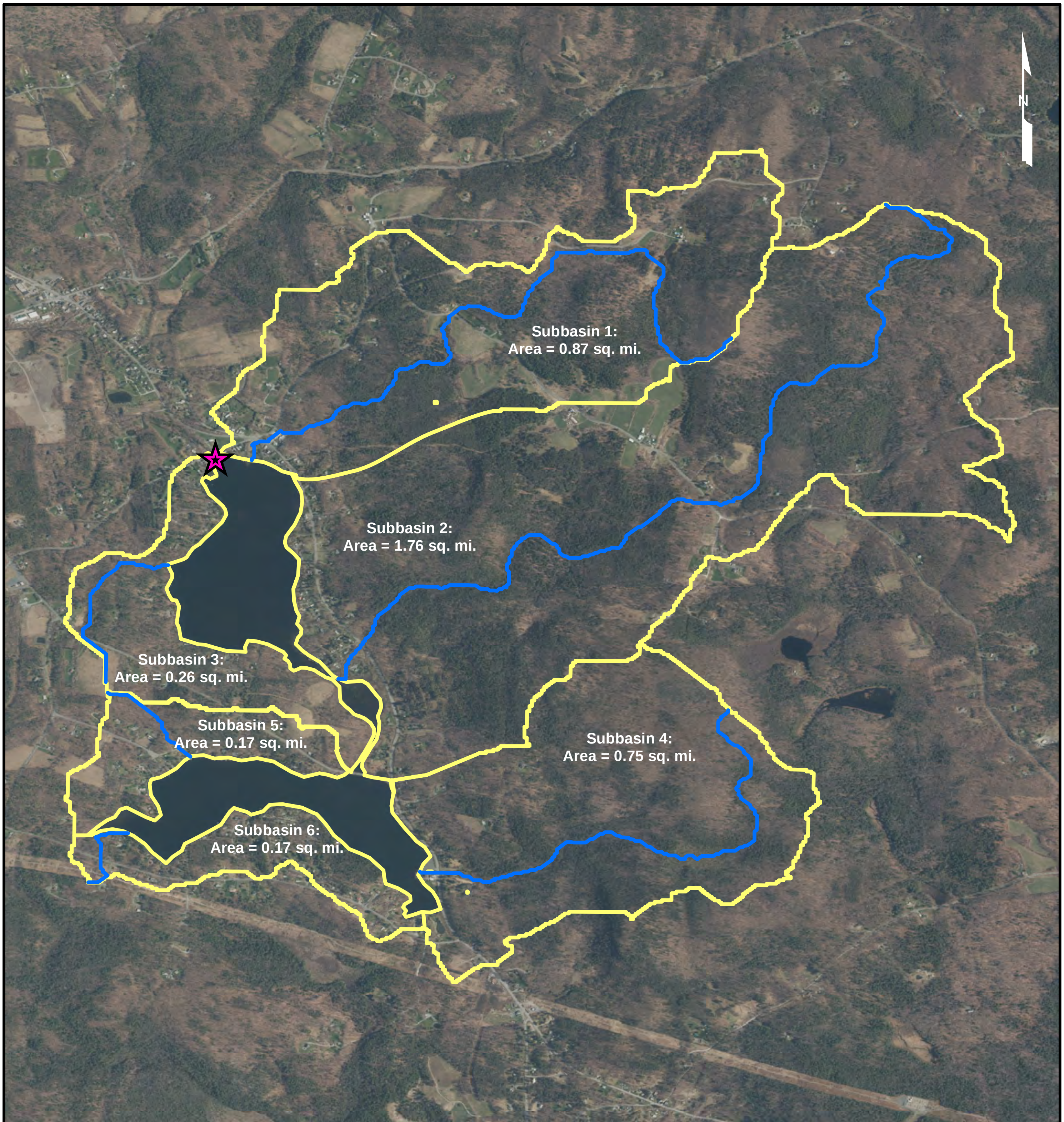
APPENDICES

APPENDIX A

FIGURES



Legend

Glass Lake Dam

Longest Flow Path

Subbasin Boundaries

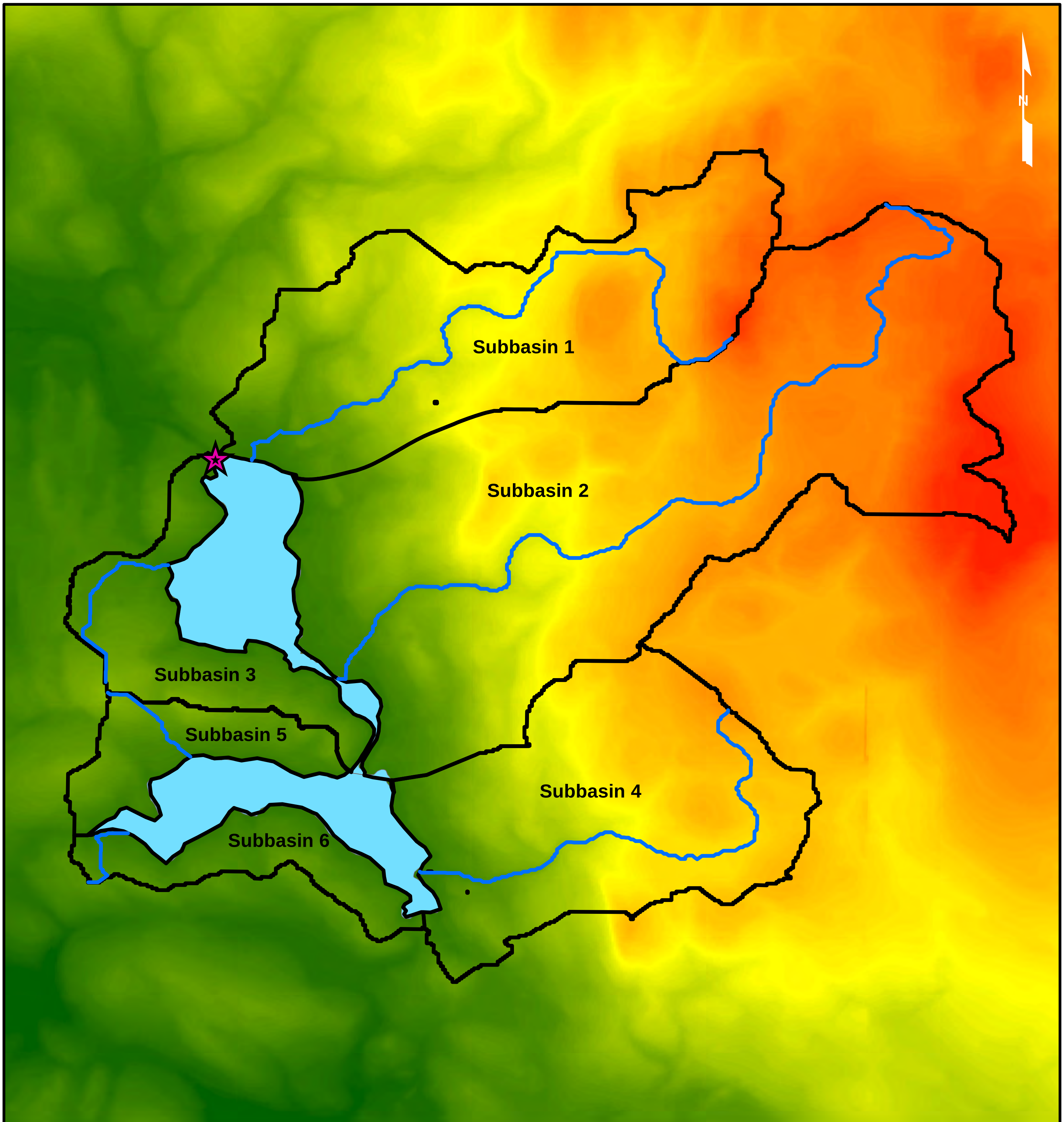
DATUM: NAD 1983 StatePlane New York East FIPS 3101 Feet
PROJECTION: Transverse Mercator
ORTHOPHOTOGRAPHY: NY GIS Clearinghouse 2014

Figure A-1

Watershed Delineation

Project: Glass Lake Dam Engineering Assessment
Prepared for: Glass Lake Preservation Corporation
Averill Park, NY

WSP



Legend

Glass Lake Dam

Longest Flow Path

Subbasin Boundaries

Elevation (NAVD 88 Ft.)

High : 508.4

Low : 209.3

2,000 1,000 0 2,000 4,000 Feet

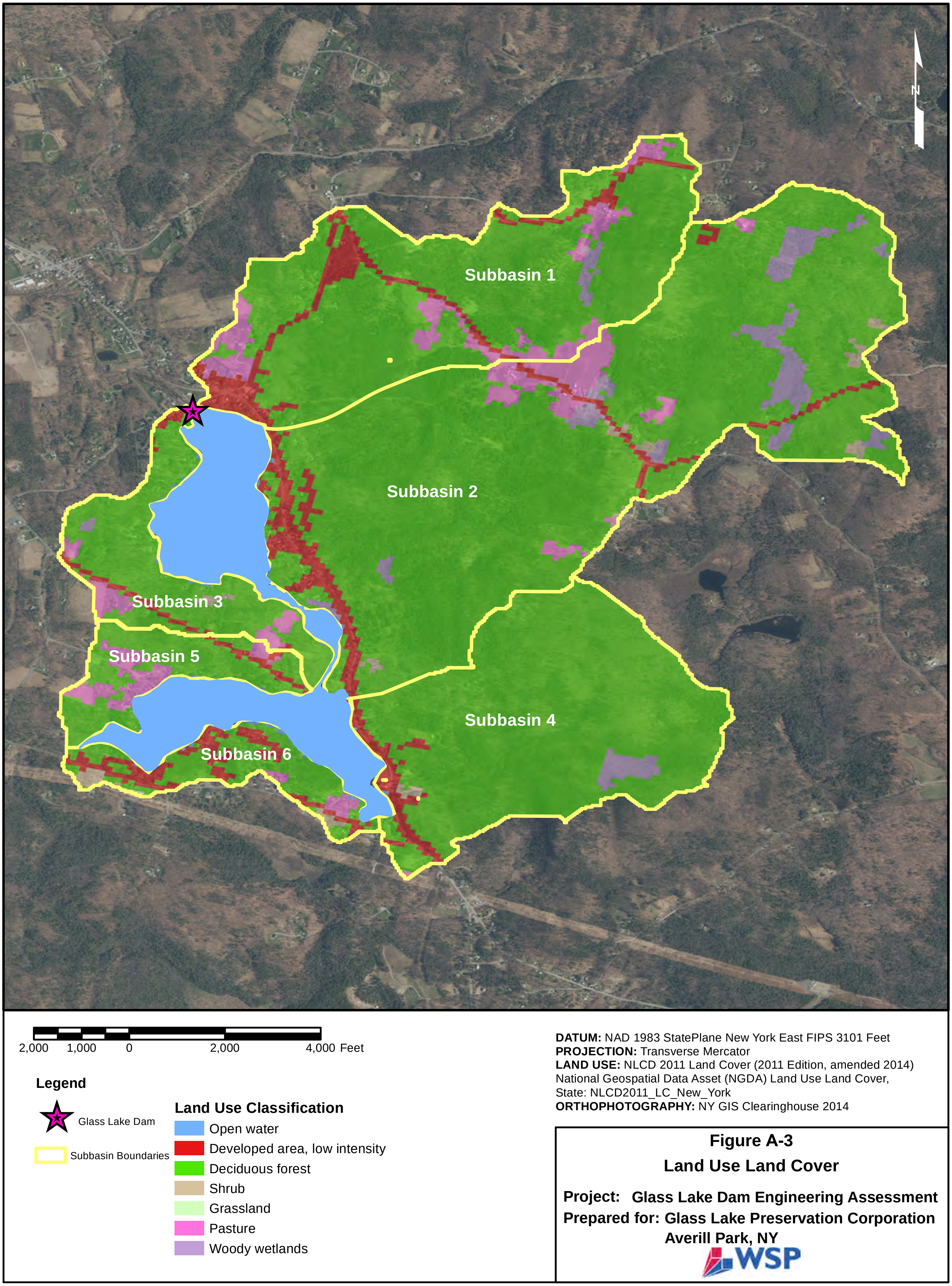
DATUM: NAD 1983 StatePlane New York East FIPS 3101 Feet
PROJECTION: Transverse Mercator
DEM: USGS NED 1/3 arc-second n42w074 1 x 1 degree ArcGrid 2015

Figure A-2

Digital Elevation Model

Project: Glass Lake Dam Engineering Assessment
Prepared for: Glass Lake Preservation Corporation
Averill Park, NY

WSP

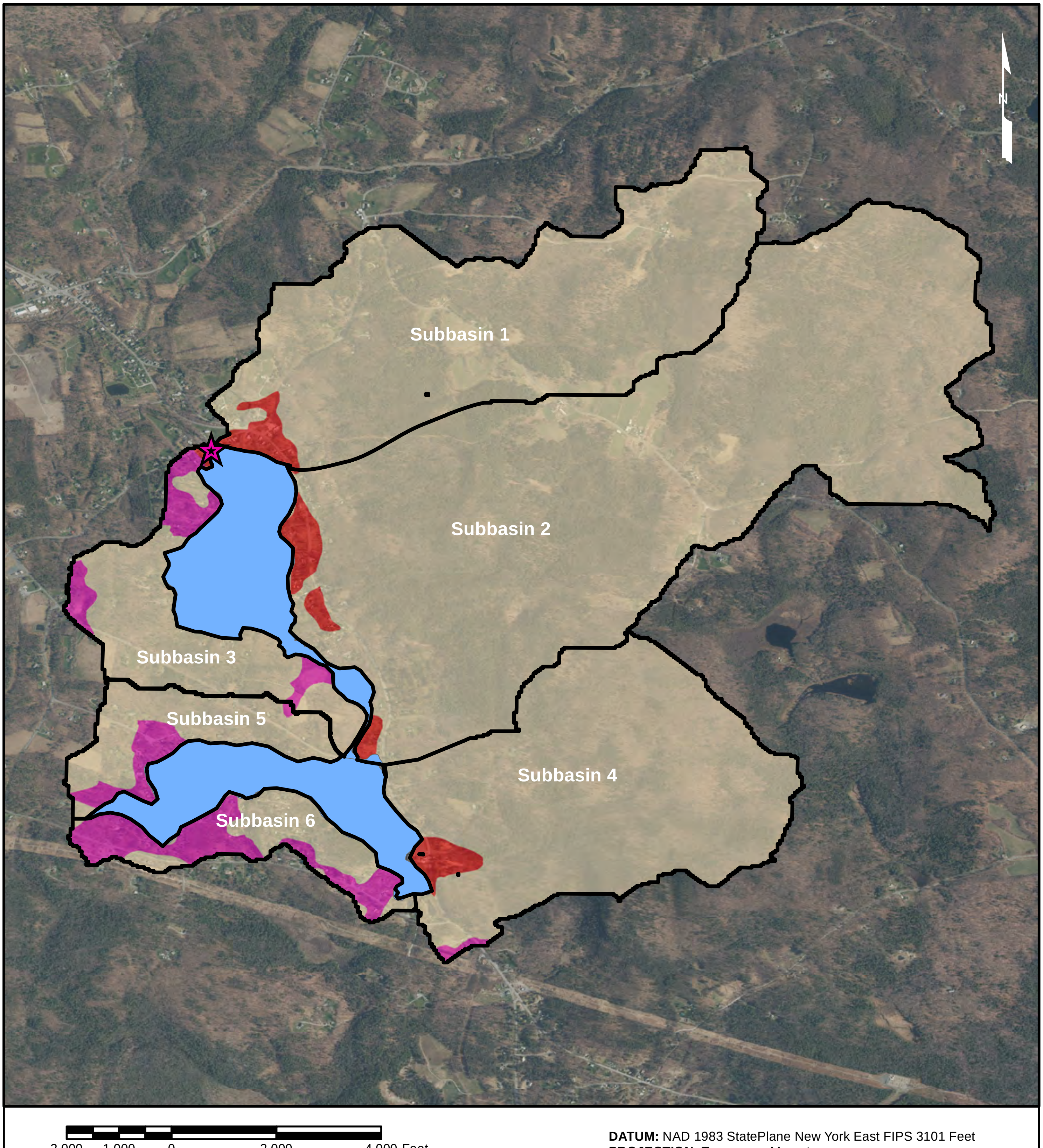


DATUM: NAD 1983 StatePlane New York East FIPS 3101 Feet
PROJECTION: Transverse Mercator
LAND USE: NLCD 2011 Land Cover (2011 Edition, amended 2014) National Geospatial Data Asset (NGDA) Land Use Land Cover, State: NLCD2011_LC_New_York
ORTHOPHOTOGRAPHY: NY GIS Clearinghouse 2014

Figure A-3
Land Use Land Cover

Project: Glass Lake Dam Engineering Assessment
Prepared for: Glass Lake Preservation Corporation
Averill Park, NY

WSP



Legend

Glass Lake Dam

Subbasin Boundaries

Soil Type

Gravelly Sandy Loam / Hydro Soil Group A

Gravelly Silt Loam / Hydro Soil Group C

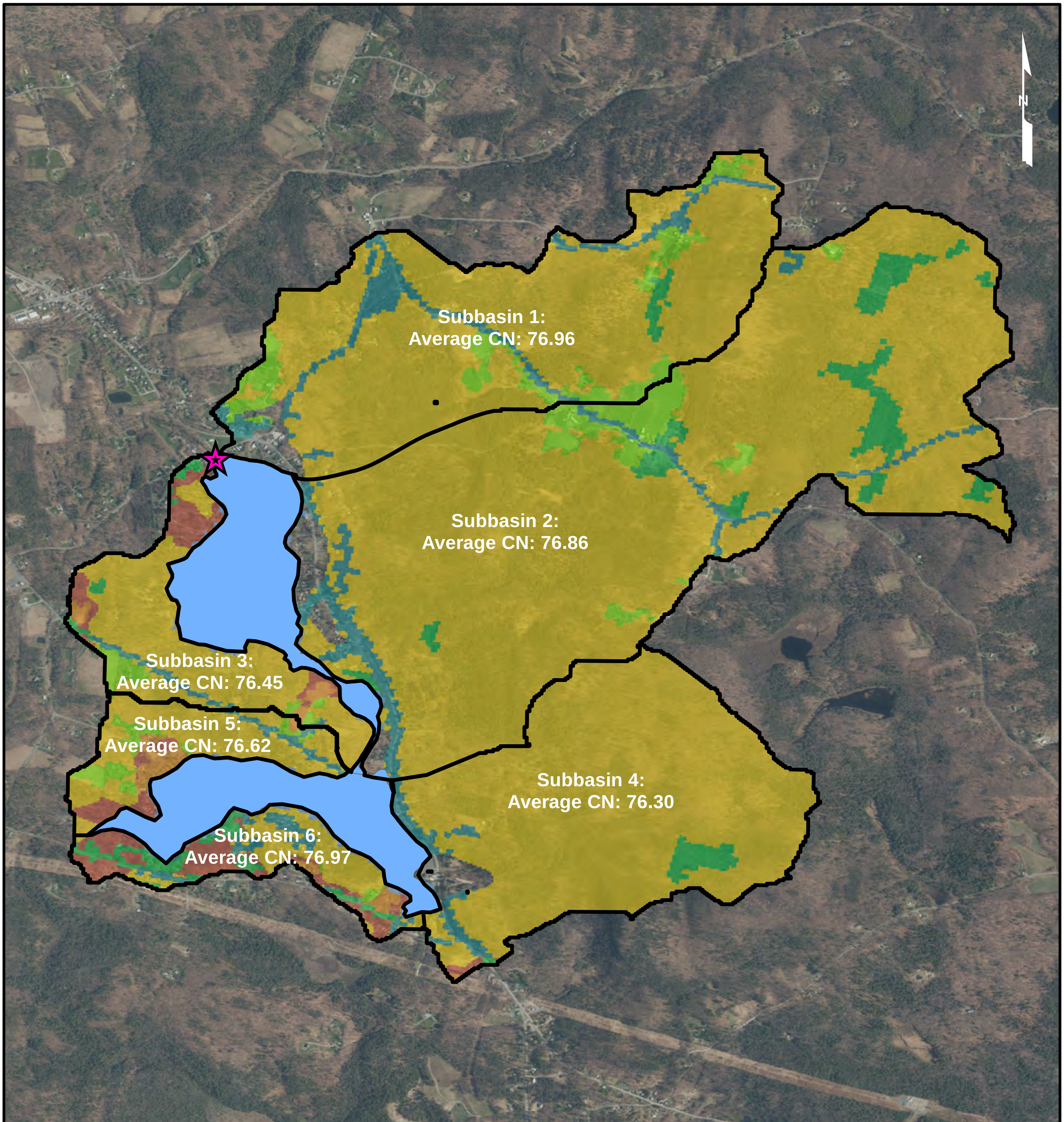
Very Stony Loam / Hydro Soil Group D

DATUM: NAD 1983 StatePlane New York East FIPS 3101 Feet
PROJECTION: Transverse Mercator
SOIL: U.S. Department of Agriculture, Natural Resources Conservation Service Soil Survey Geographic (SSURGO) database for Dutchess County, New York
ORTHOPHOTOGRAPHY: NY GIS Clearinghouse 2014

Figure A-4
Soil Classification

Project: Glass Lake Dam Engineering Assessment
Prepared for: Glass Lake Preservation Corporation
Averill Park, NY

WSP



Legend

Glass Lake Dam

Subbasin Boundaries

Curve Number

Low: 70
71
77
78
80
85
High: 100

2,000 1,000 0 2,000 4,000 Feet

DATUM: NAD 1983 StatePlane New York East FIPS 3101 Feet
PROJECTION: Transverse Mercator
ORTHOPHOTOGRAPHY: NY GIS Clearinghouse 2014

Figure A-5
Curve Numbers

Project: Glass Lake Dam Engineering Assessment
Prepared for: Glass Lake Preservation Corporation
Averill Park, NY

WSP

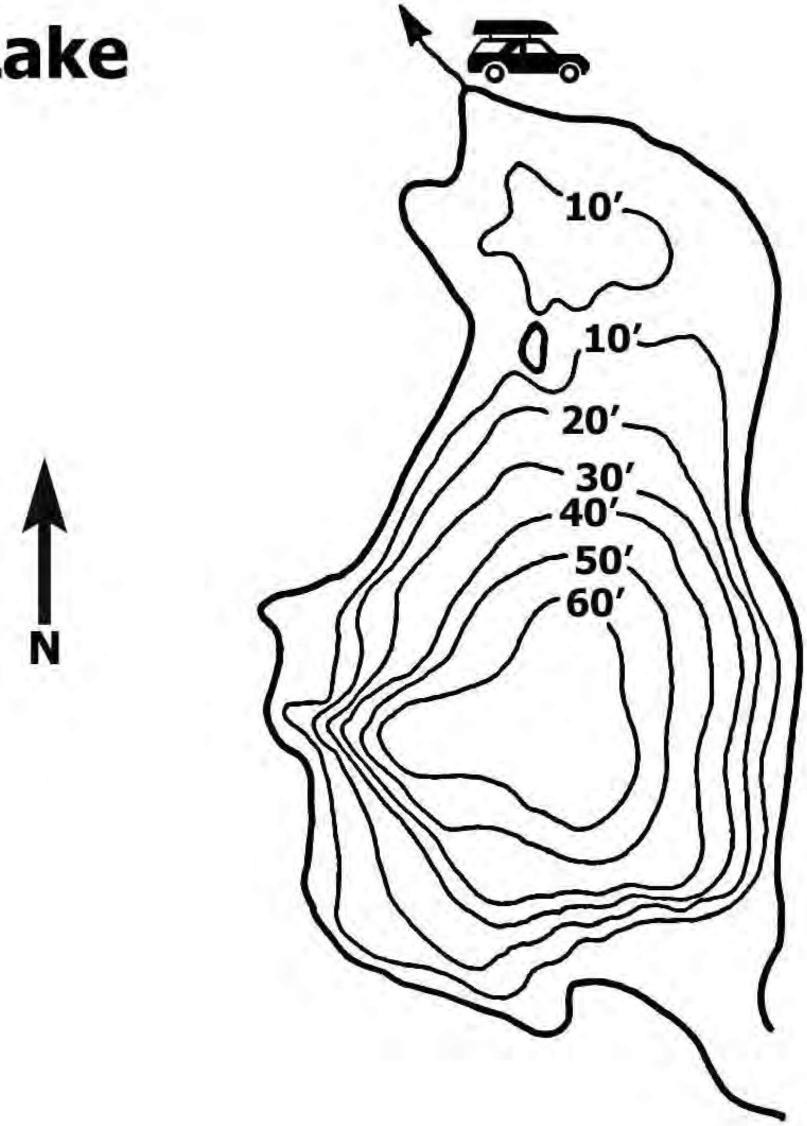
APPENDIX B

GLASS LAKE DAM BATHYMETRIC SURVEY DATA

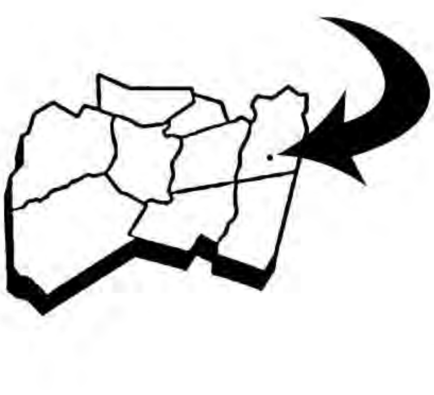
New York State Department of Environmental Conservation
Division of Fish, Wildlife and Marine Resources
Lake Map Series

Region 4

Glass Lake



Not For Use in Navigation



Glass Lake

County: Rensselaer **Town: Sand Lake**

Surface Area: 126 Acres **Mean Depth: 30ft**

Fish Species Present: Brown Bullhead, Black Crappie, Kokanee Salmon, Largemouth Bass, Chain Pickerel, Pumpkinseed, Rainbow Trout Smallmouth Bass, Yellow Perch

Scale: 0 735 ft

APPENDIX C

ELEVATION-AREA RELATONSHIPS FOR GLASS LAKE AND CROOKED LAKE

Glass Lake		Crooked Lake		Low-lying Area behind the Glass Lake Dike	
(Dam State ID: 226-1344)		(Dam State ID: N/A)		(Dam State ID: N/A)	
Elevation	Area	Elevation	Area	Elevation	Area
[ft-NAVD88]	[acres]	[ft-NAVD88]	[acres]	[ft-NAVD88]	[acres]
767.4	0	827.9	103	820	0.02
770	12.5	828.5	112.64	821	0.05
775	17.2	829	114.49	822	0.1
780	22.2	829.5	116.33	823	0.3
785	27.6	830	119.26	824	0.4
790	33	830.5	121.39	825	0.6
795	38.2	831	123.2	826	0.8
800	43.6	831.5	124.82	827	1.0
805	49.5	832	126.32	828	1.2
810	55.4	832.5	127.75	829	1.5
815	62.4	833	128.61	830	1.8
820	82			830.5	2.2
825	99			831	2.6
827.4	108.2			832	3.5
828	125.2			833	4.7
829	129.6				
830	132.8				
831	135.4				
832	137.7				
833	140				
834	142.2				
836	146.4				

ELEVATION – AREA RELATIONSHIPS FOR GLASS LAKE, CROOKED LAKE AND THE LOW-LYING AREA BEHIND THE GLASS LAKE DIKE

APPENDIX D

HEC-HMS INPUT

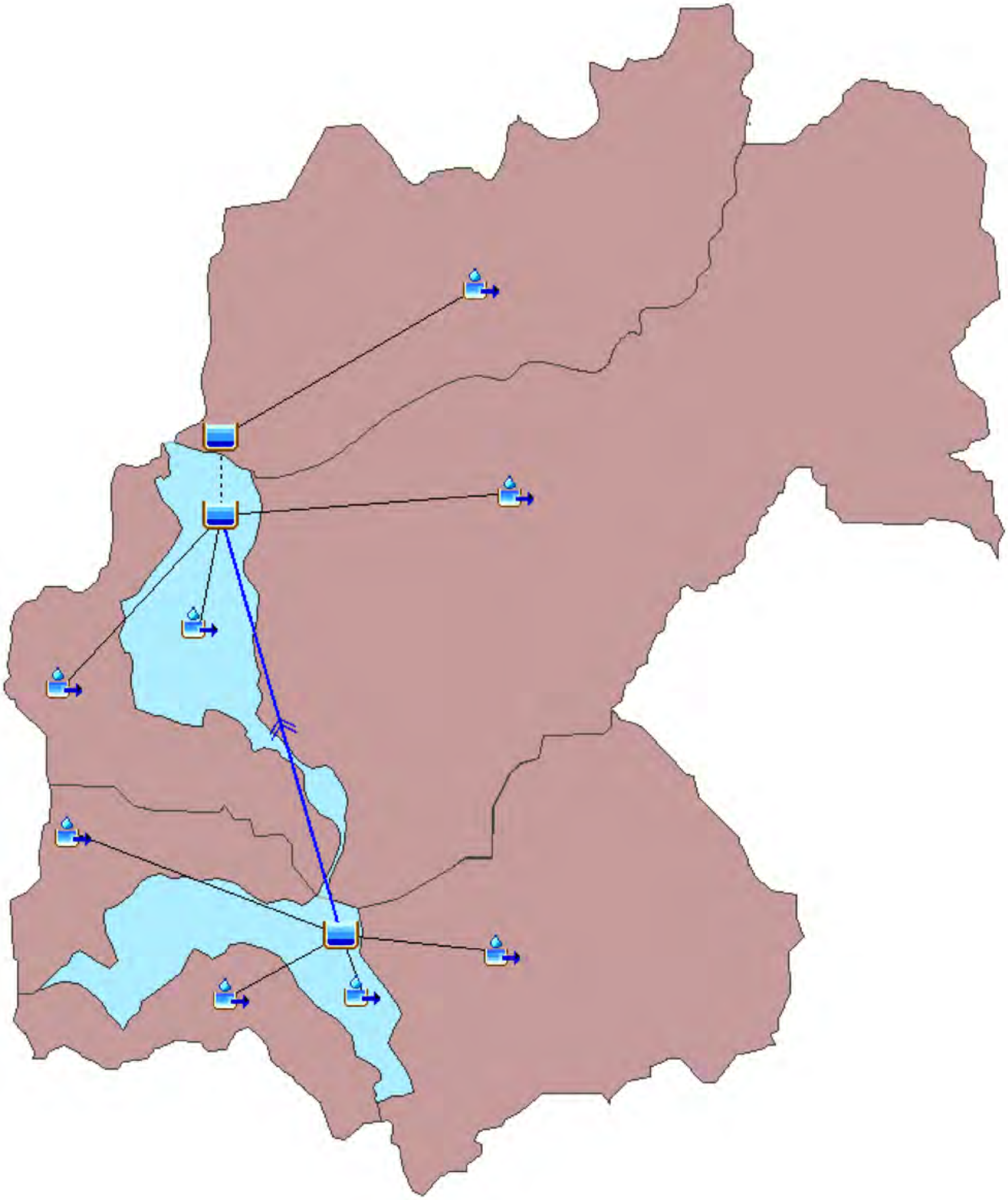


Figure D-1

Plan View of HEC-HMS Model

Project: Glass Lake Dam Engineering Assessment

Prepared for: Glass Lake Preservation Corporation
Averill Park, NY

NOAA Atlas 14, Volume 10, Version 2
Location name: Averill Park, New York, US*
Latitude: 42.6229°, Longitude: -73.5318°
Elevation: 823 ft*
* source: Google Maps

POINT PRECIPITATION FREQUENCY ESTIMATES

Sanja Perica, Sandra Pavlovic, Michael St. Laurent, Carl Trypaluk, Dale Unruh, Orlan Wilhite

NOAA, National Weather Service, Silver Spring, Maryland

PF_tabular | PF_graphical | Maps_&_aerials

PF tabular

PDS-based point precipitation frequency estimates with 90% confidence intervals (in inches)[1]

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.308 (0.234-0.404)	0.372 (0.282-0.488)	0.476 (0.360-0.627)	0.563 (0.424-0.745)	0.682 (0.499-0.937)	0.774 (0.556-1.08)	0.866 (0.607-1.25)	0.978 (0.654-1.44)	1.13 (0.727-1.71)	1.24 (0.783-1.91)
10-min	0.436 (0.331-0.572)	0.526 (0.399-0.691)	0.674 (0.510-0.888)	0.797 (0.600-1.05)	0.966 (0.707-1.33)	1.10 (0.788-1.53)	1.23 (0.860-1.77)	1.39 (0.926-2.04)	1.59 (1.03-2.42)	1.75 (1.11-2.71)
15-min	0.513 (0.390-0.673)	0.619 (0.470-0.813)	0.793 (0.600-1.04)	0.938 (0.706-1.24)	1.14 (0.832-1.56)	1.29 (0.927-1.80)	1.44 (1.01-2.08)	1.63 (1.09-2.40)	1.88 (1.21-2.85)	2.06 (1.30-3.18)
30-min	0.710 (0.539-0.931)	0.856 (0.650-1.13)	1.10 (0.830-1.44)	1.30 (0.975-1.72)	1.57 (1.15-2.16)	1.78 (1.28-2.49)	1.99 (1.40-2.88)	2.25 (1.50-3.31)	2.59 (1.67-3.93)	2.85 (1.80-4.40)
60-min	0.906 (0.689-1.19)	1.09 (0.830-1.44)	1.40 (1.06-1.84)	1.65 (1.25-2.19)	2.00 (1.47-2.75)	2.27 (1.63-3.18)	2.54 (1.78-3.67)	2.87 (1.92-4.22)	3.30 (2.13-5.01)	3.63 (2.30-5.61)
2-hr	1.14 (0.866-1.48)	1.37 (1.04-1.79)	1.75 (1.33-2.29)	2.07 (1.57-2.72)	2.51 (1.85-3.43)	2.84 (2.06-3.96)	3.18 (2.25-4.59)	3.62 (2.43-5.30)	4.21 (2.73-6.34)	4.65 (2.95-7.13)
3-hr	1.29 (0.988-1.68)	1.56 (1.19-2.02)	1.99 (1.52-2.59)	2.35 (1.78-3.08)	2.85 (2.10-3.88)	3.23 (2.35-4.49)	3.61 (2.56-5.20)	4.13 (2.77-6.02)	4.81 (3.13-7.23)	5.33 (3.39-8.14)
6-hr	1.61 (1.24-2.07)	1.94 (1.49-2.50)	2.48 (1.90-3.21)	2.92 (2.23-3.80)	3.54 (2.63-4.80)	4.01 (2.93-5.55)	4.49 (3.20-6.43)	5.14 (3.47-7.45)	6.01 (3.91-8.97)	6.67 (4.25-10.1)
12-hr	1.99 (1.54-2.55)	2.40 (1.85-3.07)	3.06 (2.36-3.94)	3.61 (2.77-4.67)	4.37 (3.26-5.88)	4.95 (3.63-6.80)	5.54 (3.97-7.88)	6.34 (4.29-9.12)	7.40 (4.83-11.0)	8.20 (5.25-12.4)
24-hr	2.38 (1.85-3.03)	2.86 (2.22-3.65)	3.65 (2.83-4.67)	4.31 (3.32-5.54)	5.22 (3.91-6.97)	5.91 (4.36-8.06)	6.61 (4.75-9.34)	7.56 (5.14-10.8)	8.81 (5.78-13.0)	9.76 (6.26-14.6)
2-day	2.74 (2.14-3.47)	3.29 (2.57-4.17)	4.20 (3.27-5.33)	4.95 (3.83-6.30)	5.97 (4.50-7.93)	6.77 (5.00-9.16)	7.56 (5.46-10.6)	8.63 (5.89-12.2)	10.0 (6.61-14.7)	11.1 (7.16-16.5)
3-day	3.02 (2.37-3.80)	3.60 (2.82-4.54)	4.55 (3.55-5.75)	5.33 (4.14-6.78)	6.42 (4.84-8.48)	7.25 (5.37-9.76)	8.09 (5.84-11.3)	9.18 (6.28-13.0)	10.6 (7.01-15.5)	11.7 (7.56-17.4)
4-day	3.27 (2.57-4.10)	3.86 (3.03-4.86)	4.84 (3.79-6.10)	5.65 (4.40-7.16)	6.77 (5.11-8.90)	7.63 (5.66-10.2)	8.49 (6.13-11.8)	9.59 (6.57-13.5)	11.0 (7.29-16.0)	12.1 (7.84-17.9)
7-day	3.93 (3.10-4.91)	4.57 (3.60-5.71)	5.61 (4.41-7.03)	6.47 (5.06-8.15)	7.66 (5.80-10.0)	8.57 (6.37-11.4)	9.49 (6.85-13.0)	10.6 (7.28-14.8)	12.0 (7.97-17.4)	13.1 (8.50-19.3)
10-day	4.59 (3.63-5.71)	5.26 (4.16-6.55)	6.35 (5.00-7.94)	7.26 (5.69-9.12)	8.52 (6.46-11.1)	9.48 (7.05-12.5)	10.4 (7.53-14.2)	11.5 (7.95-16.1)	13.0 (8.62-18.7)	14.1 (9.13-20.6)
20-day	6.63 (5.27-8.19)	7.40 (5.88-9.15)	8.66 (6.86-10.7)	9.71 (7.64-12.1)	11.1 (8.49-14.3)	12.3 (9.13-16.0)	13.4 (9.63-17.9)	14.5 (10.0-20.0)	15.9 (10.6-22.7)	17.0 (11.0-24.7)
30-day	8.34 (6.65-10.3)	9.18 (7.32-11.3)	10.6 (8.39-13.1)	11.7 (9.25-14.5)	13.3 (10.1-17.0)	14.5 (10.8-18.8)	15.7 (11.3-20.9)	16.8 (11.7-23.1)	18.2 (12.2-25.9)	19.3 (12.6-27.9)
45-day	10.4 (8.36-12.8)	11.4 (9.09-14.0)	12.9 (10.3-15.9)	14.2 (11.2-17.5)	15.9 (12.2-20.2)	17.2 (12.9-22.2)	18.6 (13.4-24.5)	19.6 (13.7-26.9)	21.0 (14.1-29.7)	22.1 (14.4-31.8)
60-day	12.2 (9.78-14.9)	13.2 (10.6-16.1)	14.8 (11.8-18.2)	16.2 (12.8-19.9)	18.0 (13.8-22.8)	19.4 (14.5-24.9)	20.9 (15.0-27.4)	21.9 (15.3-29.9)	23.2 (15.6-32.7)	24.2 (15.8-34.8)

[1] Precipitation frequency (PF) estimates in this table are based on frequency analysis of partial duration series (PDS).

Numbers in parenthesis are PF estimates at lower and upper bounds of the 90% confidence interval. The probability that precipitation frequency estimates (for a given duration and average recurrence interval) will be greater than the upper bound (or less than the lower bound) is 5%. Estimates at upper bounds are not checked against probable maximum precipitation (PMP) estimates and may be higher than currently valid PMP values.

Please refer to NOAA Atlas 14 document for more information.

Back to Top

PF graphical

	Sheet Flow					
Subbasin	n	S (ft/ft)	L (ft)	P2 (in)	Tt (hrs)	(Tt (min)
1	0.80	0.2231	25	2.86	0.083	5.0
2	0.80	0.0197	25	2.86	0.219	13.1
3	0.41	0.0919	50	2.86	0.121	7.2
4	0.80	0.0820	25	2.86	0.124	7.4
5	0.41	0.1378	50	2.86	0.102	6.1
6	0.41	0.0098	50	2.86	0.294	17.7

	Shallow Concentrated Flow				
Subbasin	L*	Land Cover	S (ft/ft)	V (ft/s)	Tt (min)
1	3500	woodlands	0.0504	1.13	51.6
2	5000	woodlands	0.0333	0.92	90.7
3	3263	pavement/gullies	0.0390	4.01	13.6
4	4500	woodlands	0.0718	1.35	55.6
5	1769	pavement/gullies	0.0813	5.80	5.1
6	1524	pavement/gullies	0.0520	4.63	5.5

Pavement/Gullies:
V = 20.328 (s)^0.5

Woodlands:
V = 5.032 (s)^0.5

	Channel Flow						Tc	
Subbasin	n	R (ft)	S (ft/ft)	V (ft/s)	L	Tt (min)	(min)	T_lag (min)
1	0.045	1.17	0.0504	8.3	8541.5	17.2	73.8	44.3
2	0.045	1.17	0.0333	6.7	12083.6	30.0	133.8	80.3
3	1.045	1.17	0.0390	0.3	0.0	0.0	20.8	12.5
4	0.045	1.17	0.0718	9.9	4009.5	6.8	69.8	41.9
5	1.045	1.17	0.0813	0.5	0.0	0.0	11.2	6.7
6	2.045	1.17	0.0520	0.2	0.0	0.0	23.2	13.9

Assumed Channel Geometry

Depth	2	ft
Side slope (H:1V)	2	H:1V
Base width	3	ft
Area	14	ft^2
Wetted Perimeter	11.9	ft
Hydraulic Radius	1.17	ft

APPENDIX E

HEC-HMS RESULTS

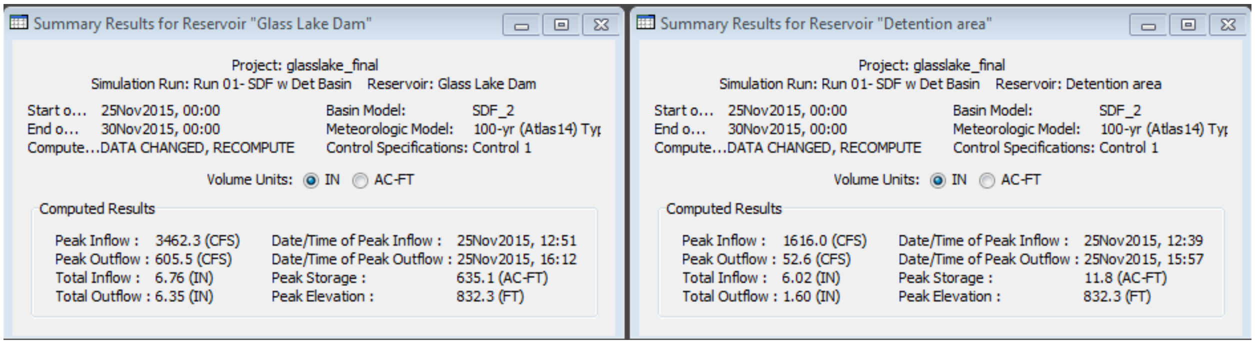


Figure 1: Results Summary for SDF Case

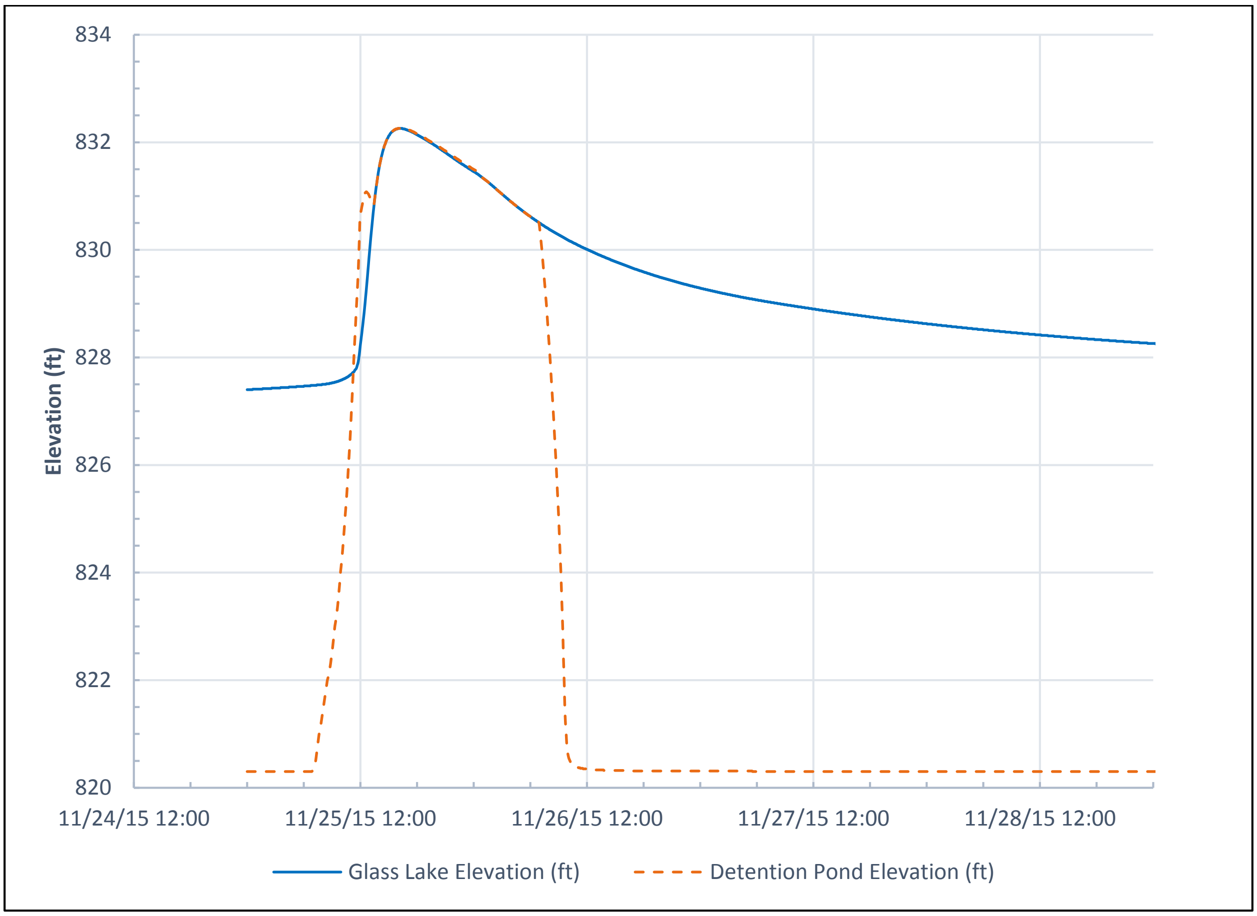


Figure 2: Stage Hydrographs for Glass Lake and the Detention Pond directly downstream from the Glass Lake Dike

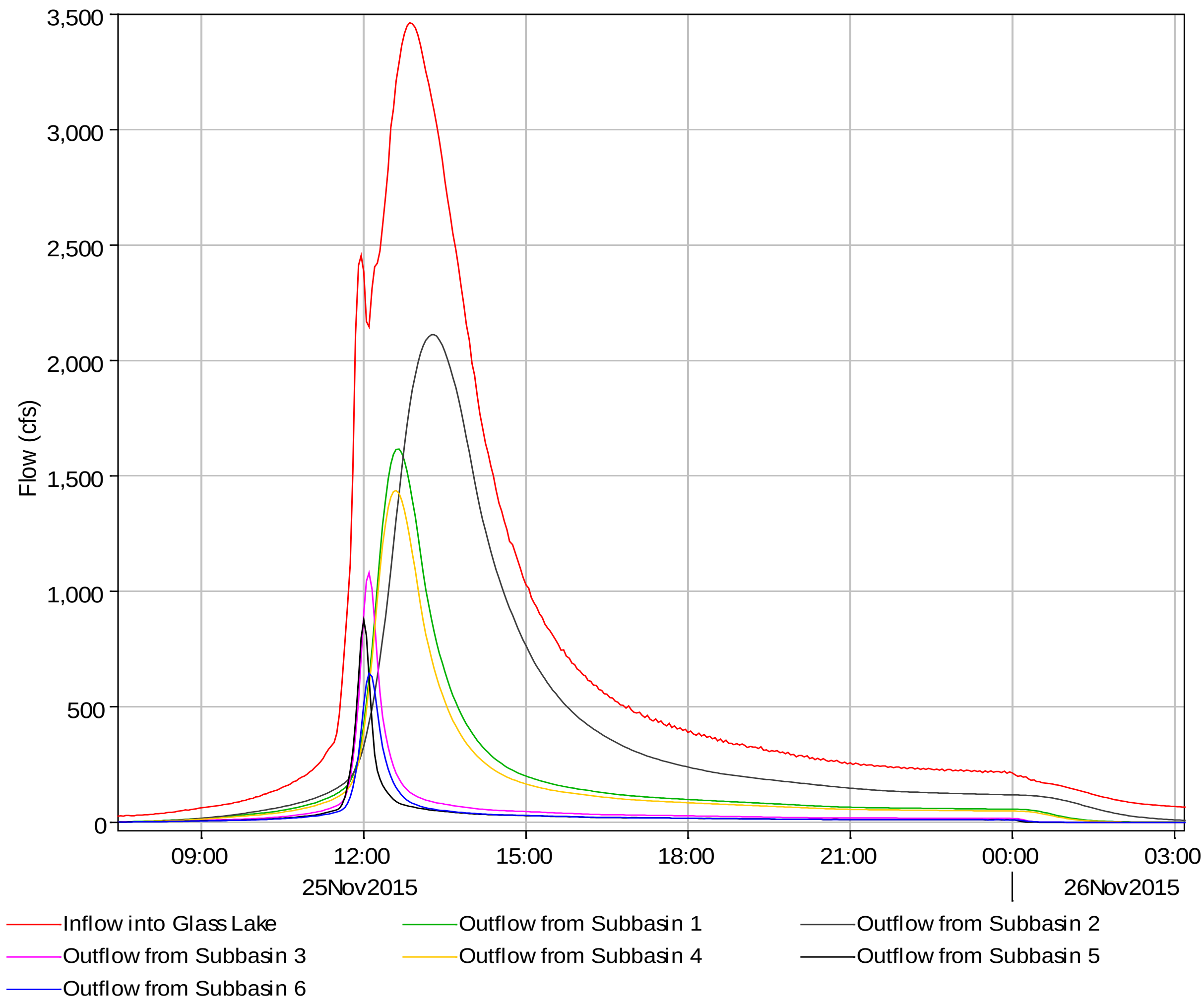


Figure 3: Flow Hydrographs

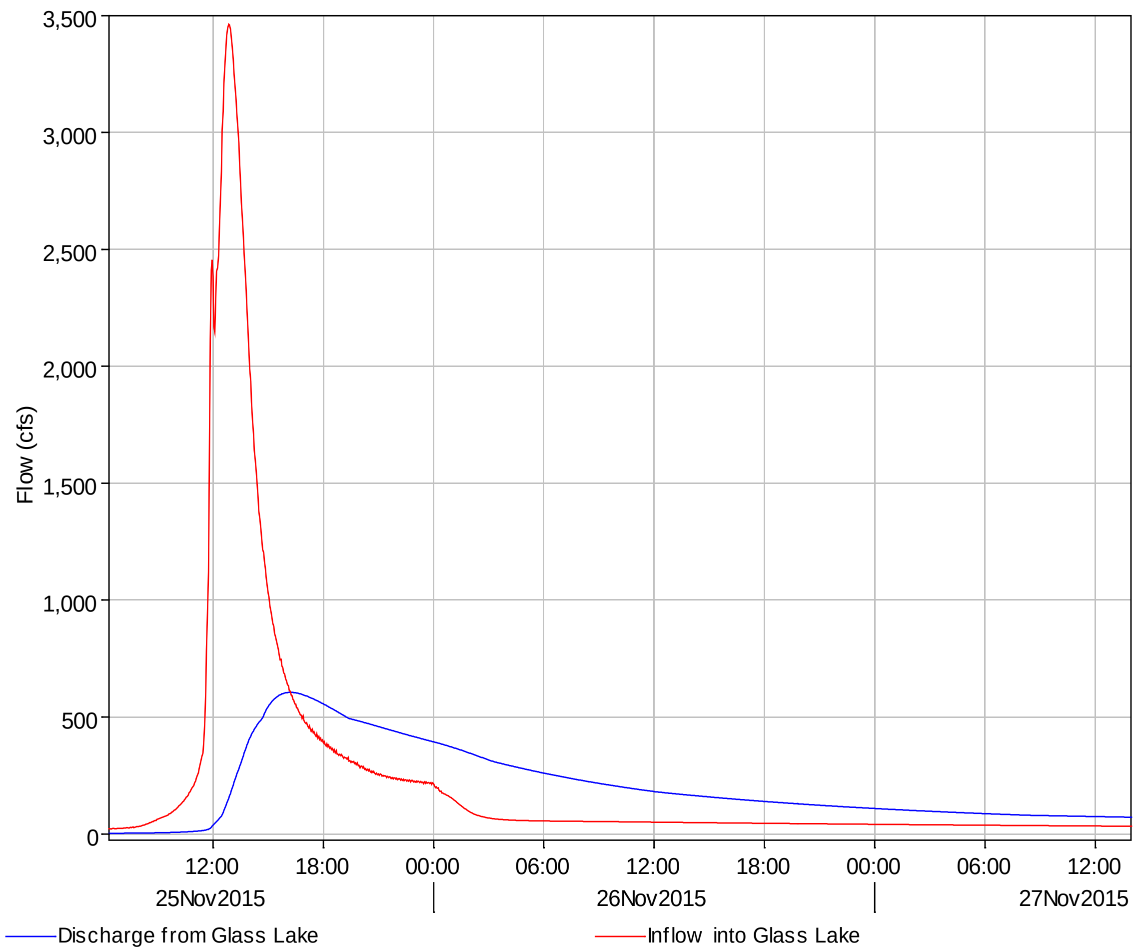


Figure 4: Inflow and Discharge Hydrographs for Glass Lake

APPENDIX F

LOW LEVEL OUTLET DRAWDOWN EVALUATION

CALCULATION SHEET

Pipe Properties		
pipe diameter (D) =	2	feet
pipe length (L) =	70	feet
tailwater elevation (z_2) =	812.75	feet
pipe material	cast iron	

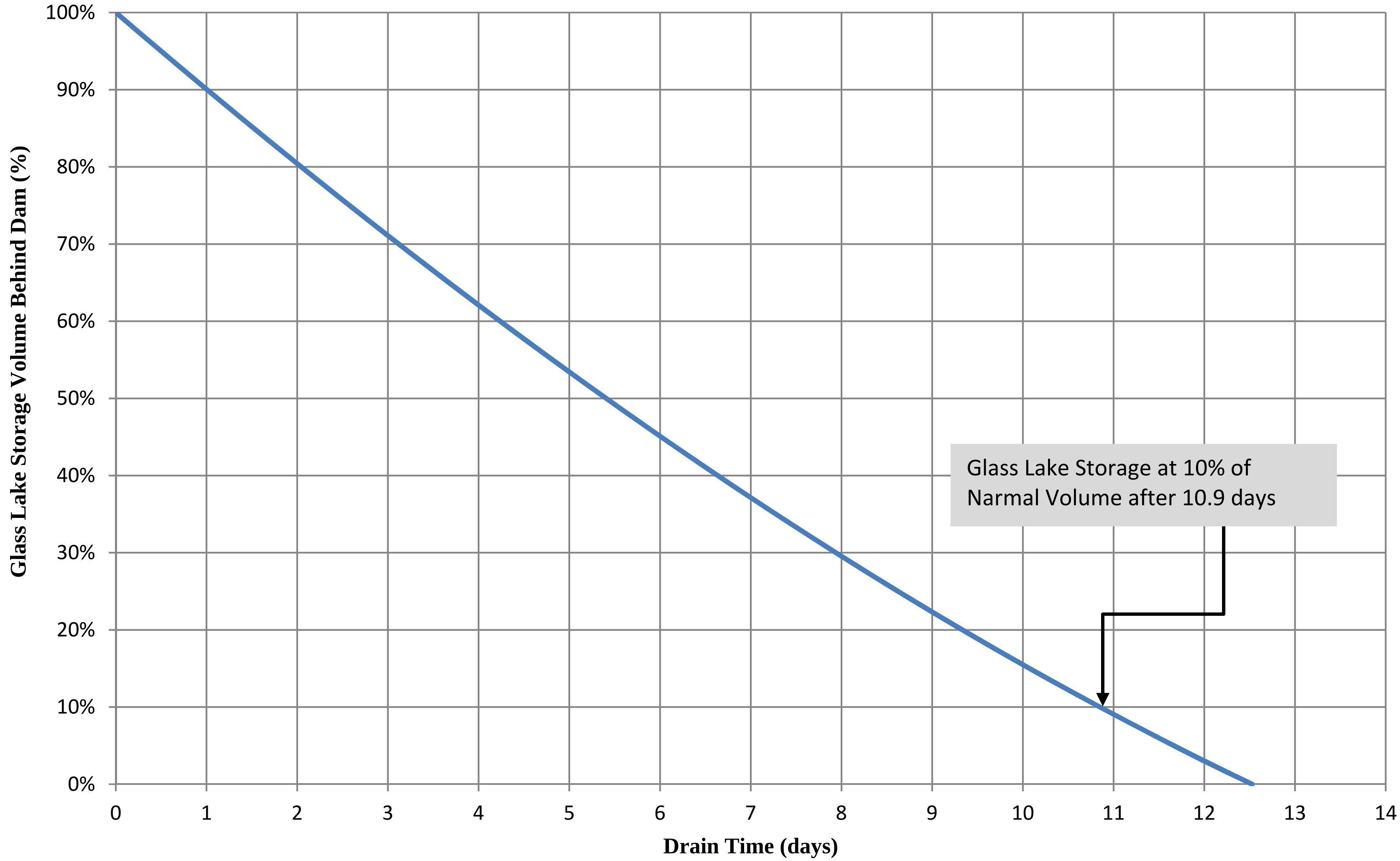
Hazen-Williams Inputs (constant)		
K_H =	1.318	conversion for English units
C =	65	pipe roughness coefficient
R = A/P =	0.500	hydraulic radius

Hydraulic Properties (assumes full pipe flow)		
Area (A) =	3.14	sf
Wetted Perimeter (P) =	6.28	ft

Minor Head Loss Coefficients		
location	description	K
entrance	slightly rounded	0.78
trashrack	-	0.70
gate	gate	0.15
exit	free jet	0.00
	misc	0.00
Total		1.63

							Energy Equation					
Elevation	Volume	Volume	Step Number	Change in Volume	Average Upstream Water Elevation	Velocity	left (z_1 - z_2)	right $\frac{v^2}{2g}+L\left(\frac{v^2}{K_H C R^{0.63}}\right)+K\frac{v^2}{2g}$	Discharge from Low-Level Outlet Pipe	Time to Reach Next Reservoir Elevation	Cumulative Drawdown Time	% of storage capacity
z	V	V		ΔV	z_1	v			Q	t	t	
(NAVD 88)	(acre-feet)	cf		cf	(feet)	fps			cfs	(sec)	(sec)	
827.40	900.71	39,235,056										100.0
			1	1,871,587	827.20	14.50	14.45	14.45	45.54	41,097	0	
827.00	857.75	37,363,470										95.2
			2	4,559,478	826.50	14.13	13.75	13.75	44.39	102,716	1.66	
826.00	753.08	32,803,992										83.6
			3	4,393,583	825.50	13.59	12.75	12.75	42.69	102,909	2.86	
825.00	652.21	28,410,409										72.4
			4	4,233,075	824.50	13.03	11.75	11.75	40.93	103,417	4.05	
824.00	555.04	24,177,334										61.6
			5	4,077,816	823.50	12.44	10.75	10.75	39.10	104,302	5.26	
823.00	461.42	20,099,518										51.2
			6	3,927,682	822.50	11.83	9.75	9.75	37.18	105,652	6.48	
822.00	371.25	16,171,836										41.2
			7	3,782,623	821.50	11.19	8.75	8.75	35.16	107,594	7.73	
821.00	284.42	12,389,213										31.6
			8	3,642,645	820.50	10.51	7.75	7.75	33.02	110,310	9.00	
820.00	200.79	8,746,568										22.3
			9	3,507,751	819.50	9.79	6.75	6.75	30.75	114,078	10.32	
819.00	120.27	5,238,817										13.4
			10	3,377,938	818.50	9.01	5.75	5.75	28.31	119,338	11.71	
818.00	42.72	1,860,879										4.7
			11	1,860,879	817.70	8.34	4.95	4.95	26.20	71,031	12.53	
817.40	0.00	-										0.0

Low Level Outlet Rating Curve for 30-inch Cast Iron Low-Level Outlet Pipe
Glass Lake Dam



APPENDIX G

CD INCLUDING HEC-HMS MODEL & GIS DATA

MEMO

TO: Hans Hasnay, PE

FROM: Theresa Chu, EIT; Greg Shaffer, PE

SUBJECT: Glass Lake Dam Hydrologic and Hydraulic Analysis for Spillway Design Flood

DATE: July 27, 2020

Glass Lake Dam in Averill Park, NY is a Class B – Intermediate Hazard Dam which impounds Glass Lake on the north end of the lake. Glass Lake Road passes over the Glass Lake Dam spillway. The Dam also consists of an embankment section which is a dike. Directly downstream of the dike on the eastern side of the spillway is a low-lying area that is connected to Glass Lake via a channel.

Glass Lake Dam does not pass the Spillway Design Flood (SDF), which is 150% of the 100-year storm for Class B Dams. This memo summarizes subsequent hydrologic and hydraulic analyses based on new survey data of the road to determine the maximum reservoir elevation, the dam height of overtopping, and the duration of overtopping during the SDF event.

METHODOLOGY

A survey was conducted for Glass Lake Road downstream of the Glass Lake Dam. The survey data was incorporated into existing 1-meter resolution terrain data. The road profile point locations are shown in Figure 1, point elevations are shown in Figure 2, and the road profile is shown in Figure 3.

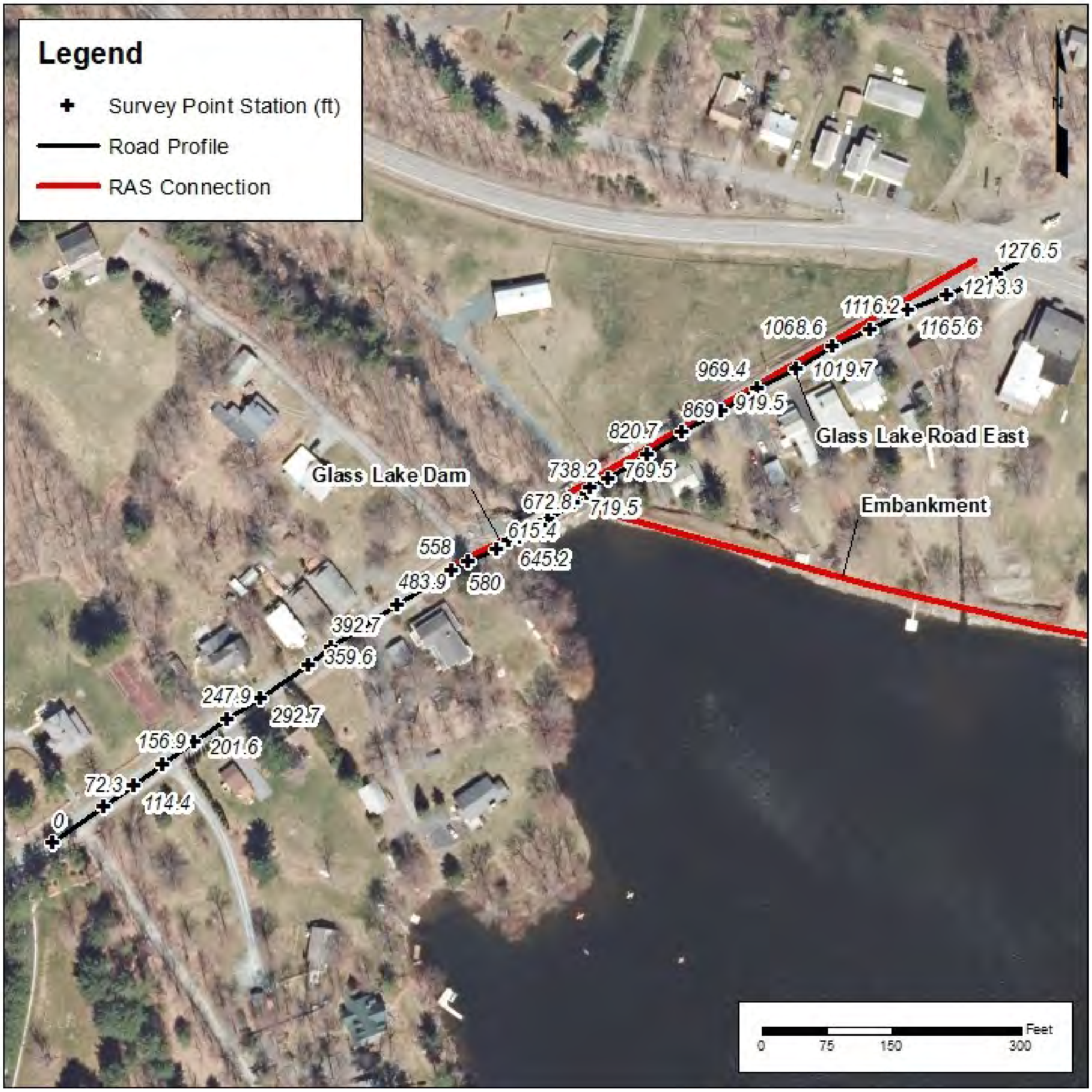


Figure 1: Glass Lake Road Survey Point Locations

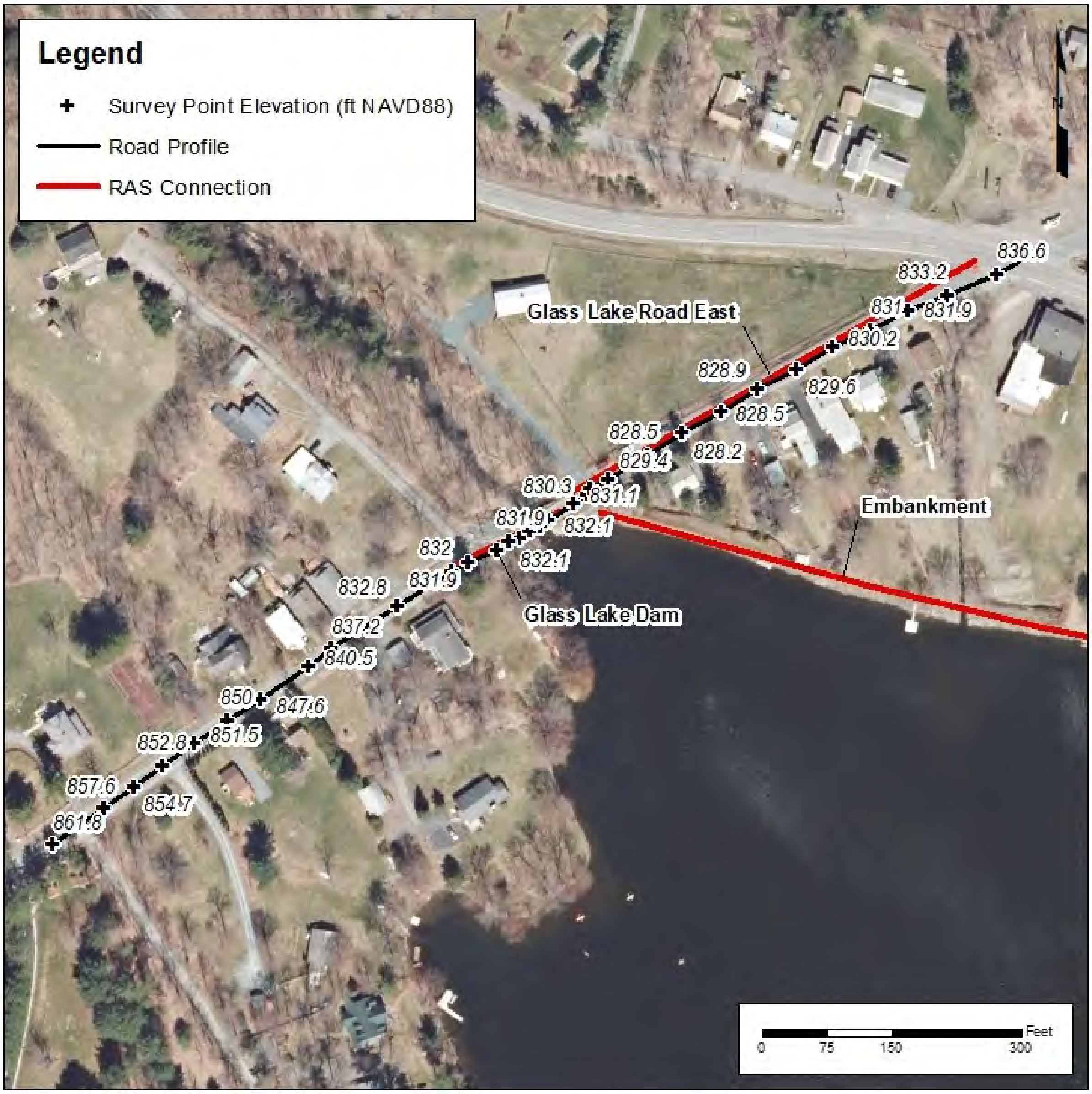


Figure 2: Glass Lake Road Survey Point Elevations

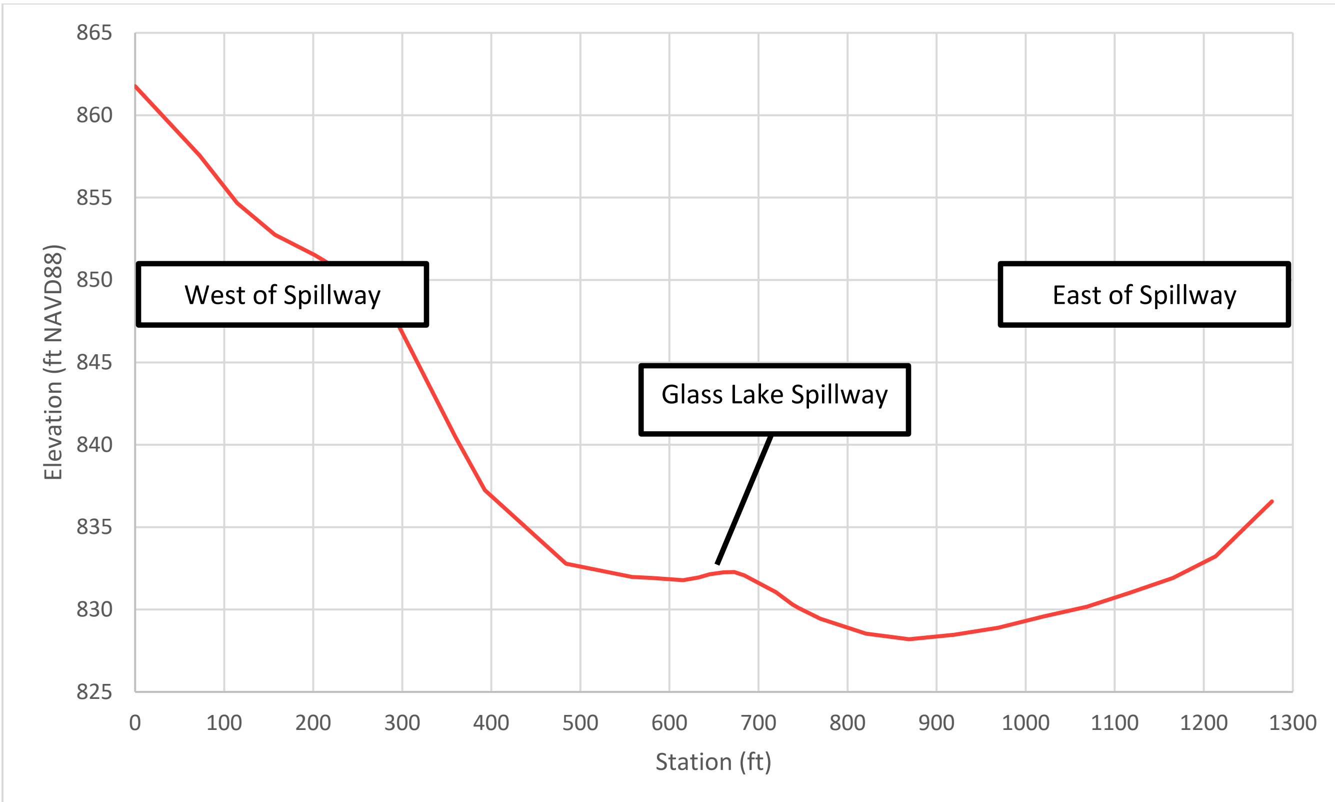


Figure 3: Glass Lake Road Profile

A hydraulic model for Glass Lake Dam was developed in HEC-RAS 5.0.7 to supplement the previously developed HEC-HMS model. The hydraulic model is necessary to determine the depth of flow over Glass Lake Road during the SDF and build upon the analysis already completed.

DEVELOPMENT OF THE HEC-RAS MODEL

A two-dimensional hydraulic model was developed using USACE's Hydrologic Engineer Center's River Analysis System (HEC-RAS). The HEC-RAS model used boundary conditions, spillway rating curves and reservoir storage information from the previously developed HEC-HMS model.

A depiction of the HEC-RAS model is seen in Figure 4.

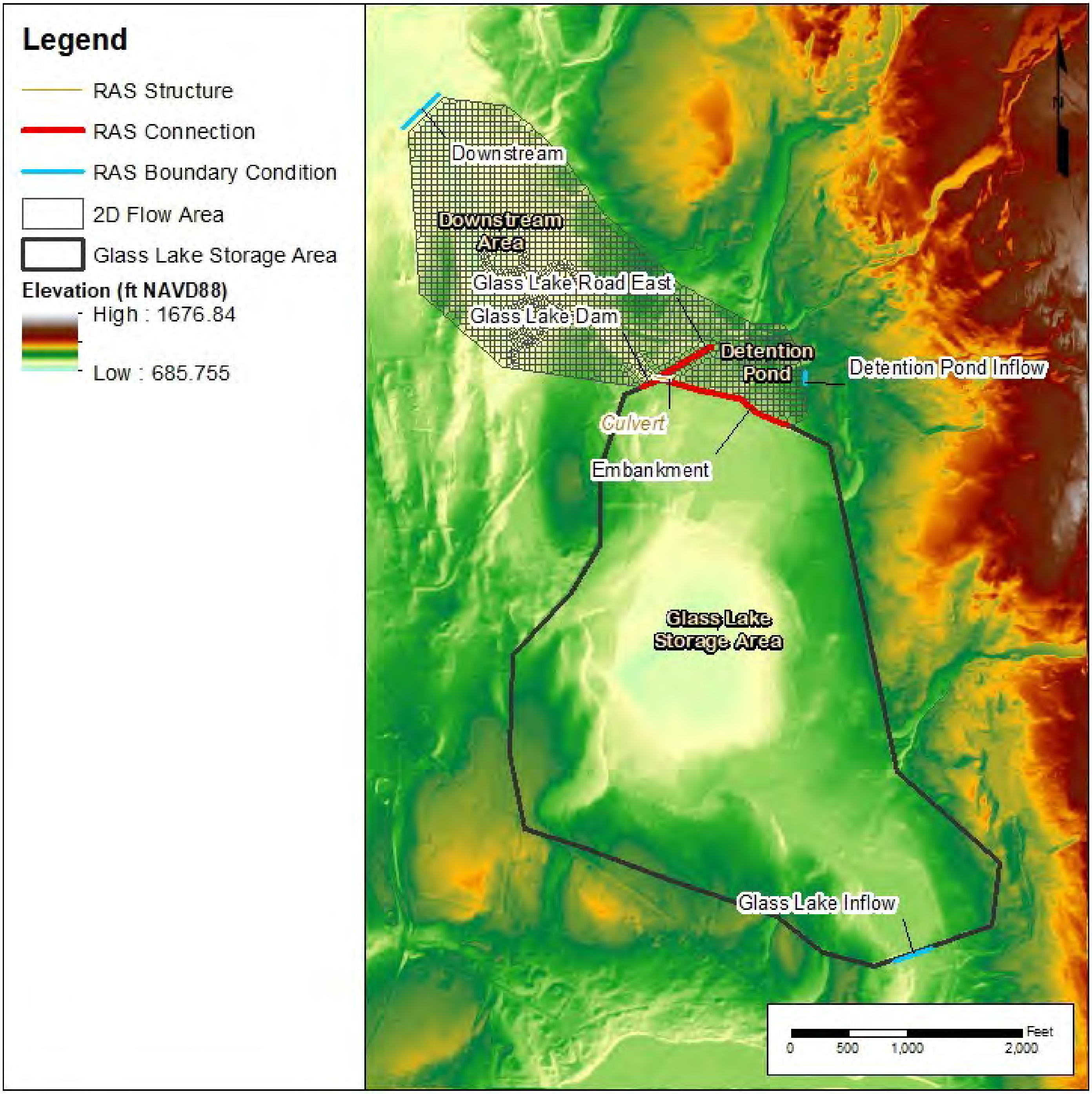


Figure 4: Glass Lake Dam and Storage System Mapped in HEC-RAS

In the HEC-HMS model, Glass Lake and the low-lying area downstream of Glass Lake are both modeled as reservoirs. This low-lying area is referred to as the Detention Pond. Glass Lake is modeled in HEC-RAS as a storage area using the elevation-volume curve converted from the corresponding elevation-area curve in the HMS model, shown in the Appendix. The Detention Pond and the rest of the area downstream of the Dam is modeled as a 2D flow area. Three different structures are modeled as Storage Area/2D Area connections:

— Glass Lake Dam, which connects Glass Lake to the Downstream Area. The spillway outflow is modeled with the same empirically calculated spillway discharge curve as the HEC-HMS model. The rating curve is shown in the Appendix.

- Glass Lake Road east of the Dam, which is an internal 2D Flow Area connection from the Detention Pond to the Downstream Area. Road profile data is incorporated from the recent survey, with the resulting profile shown in the Appendix. This connection also contains a culvert under the road, with geometry characteristics shown in the Appendix.
- Glass Lake Embankment, which connects Glass Lake Reservoir to the Detention Pond. The dike linking the Detention Pond and Glass Lake is modeled as part of the overall weir profile. The profile is shown in the Appendix.

EXTERNAL BOUNDARY CONDITIONS

The unsteady HEC-RAS model developed for this analysis uses flow hydrographs as upstream boundary conditions. These flow hydrographs are obtained from the previously developed HEC-HMS model output and used as lateral inflow hydrographs at their respective locations. Each storage area has a boundary condition modeling the combined inflow from the upstream subbasins, with the inflows shown in Figure 5. This allows the hydrographs to be routed through the reservoirs for analysis of the SDF.

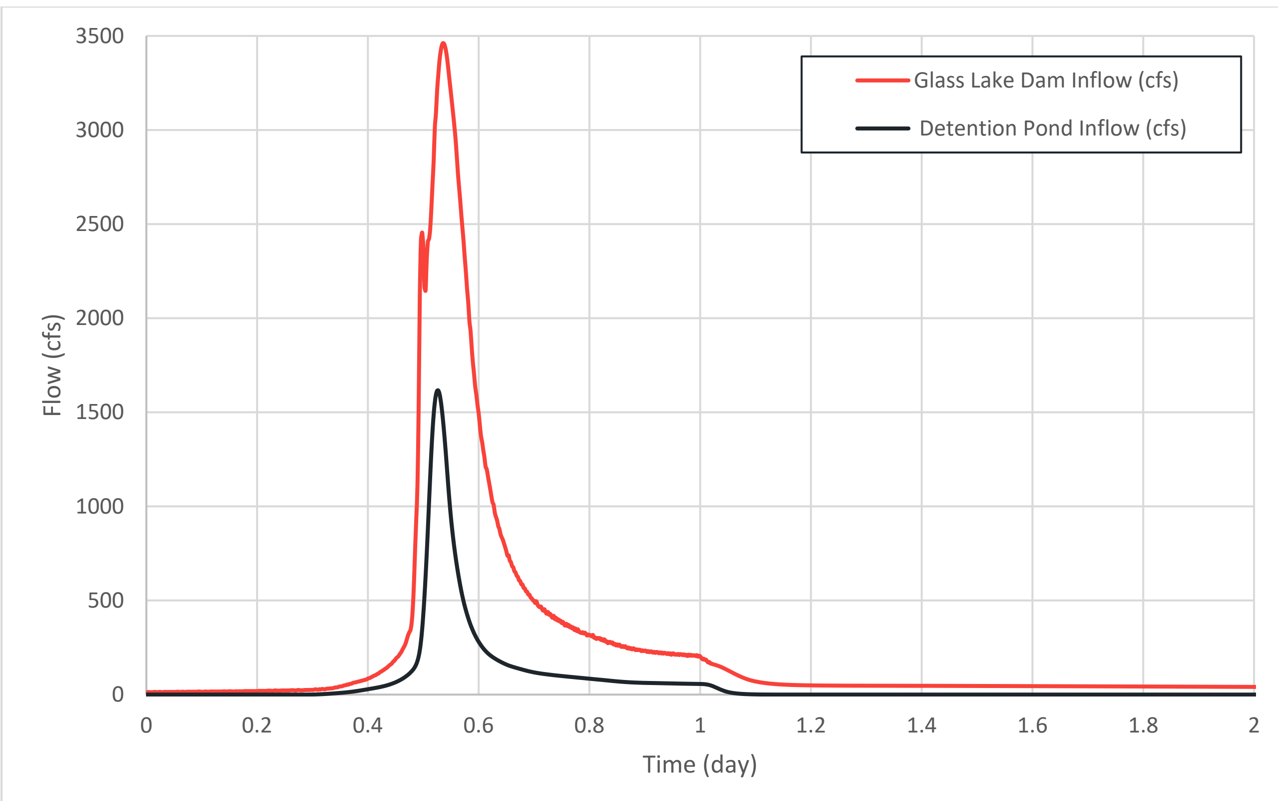


Figure 5: Inflow Hydrograph for Storage Areas

For this model, the downstream boundary condition was set to normal depth with a slope of 0.0059, which is based off the average channel bed slope downstream of the model extent.

RESULTS

The flow and stage hydrographs for Glass Lake Dam are shown in Figure 6 and Figure 7, respectively

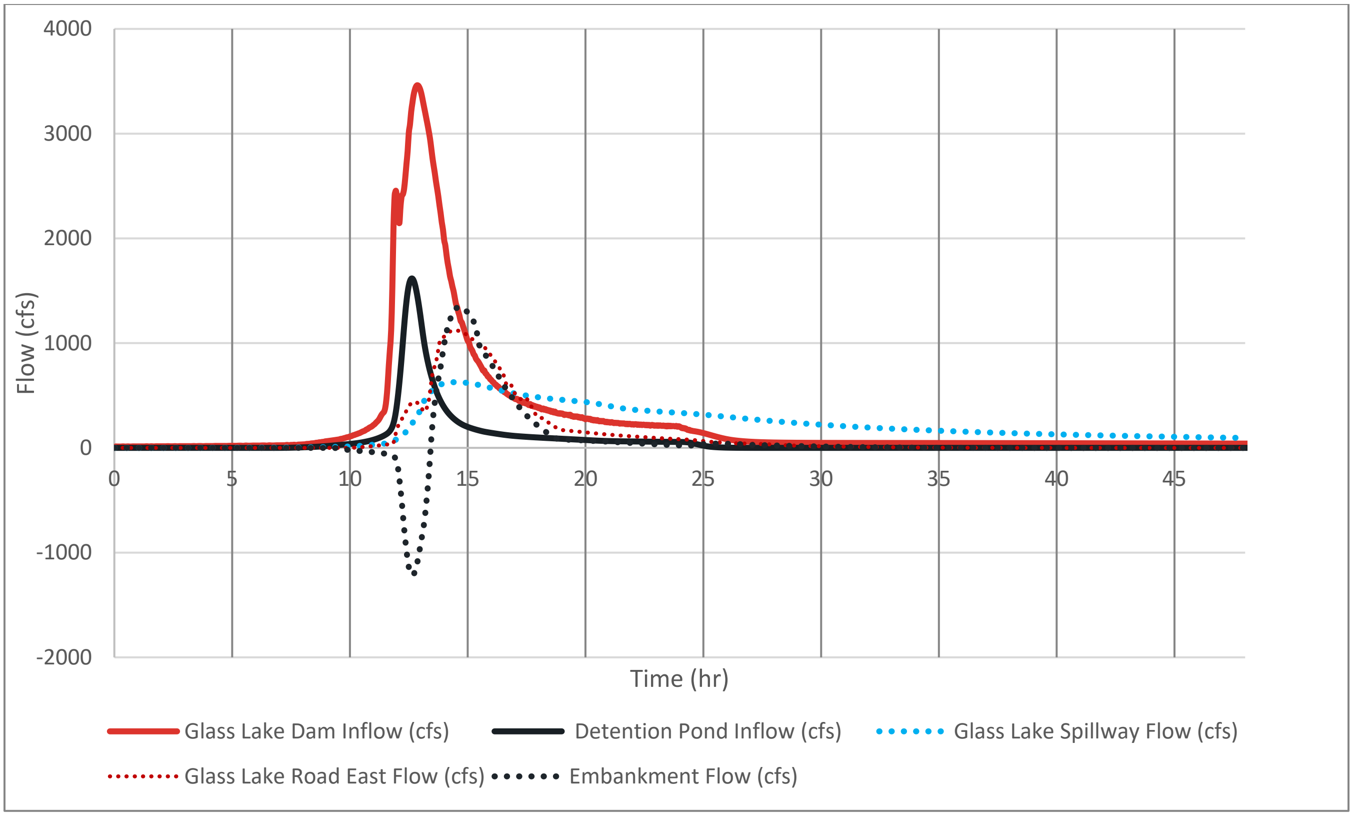


Figure 6: Inflow and Flow Hydrographs for Glass Lake Dam, Glass Lake Road, and Embankment

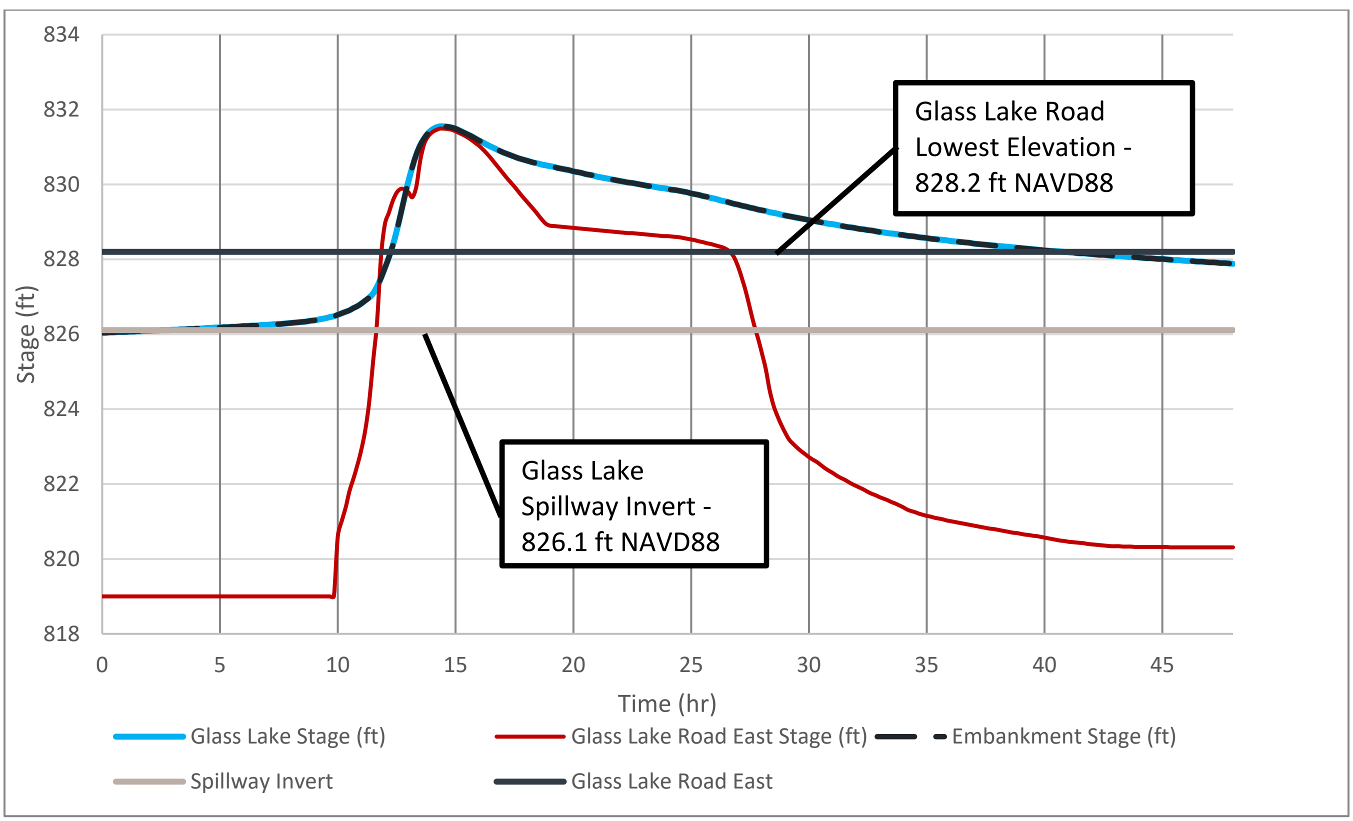


Figure 7: Stage Hydrographs for Glass Lake Dam, Glass Lake Road, and Embankment

The peak stage elevations and weir flows are shown in Table 1.

Table 1: Peak Stage and Flows for Hydraulic Connections

Connection	Peak Elevation (ft NAVD88)	Peak Weir Flow (cfs)	Lowest Weir Elevation (ft NAVD88)	Depth of Overtopping (ft)	Duration of Overtopping (hr)
Glass Lake Dam	831.6	627.2	832.2[a]	-	-
Glass Lake Road East[b]	831.5	1122.0	828.2	3.3	14.5
Glass Lake Embankment	831.6	1349.1	830.5	1.1	5.5

a) This represents the lowest dam crest elevation. The spillway invert elevation is 826.11 ft.

b) An additional culvert modeled through the connection has a peak flow of 50.6 cfs.

The peak water surface elevation at Glass Lake Road East is plotted with the road surface profile in Figure 8.

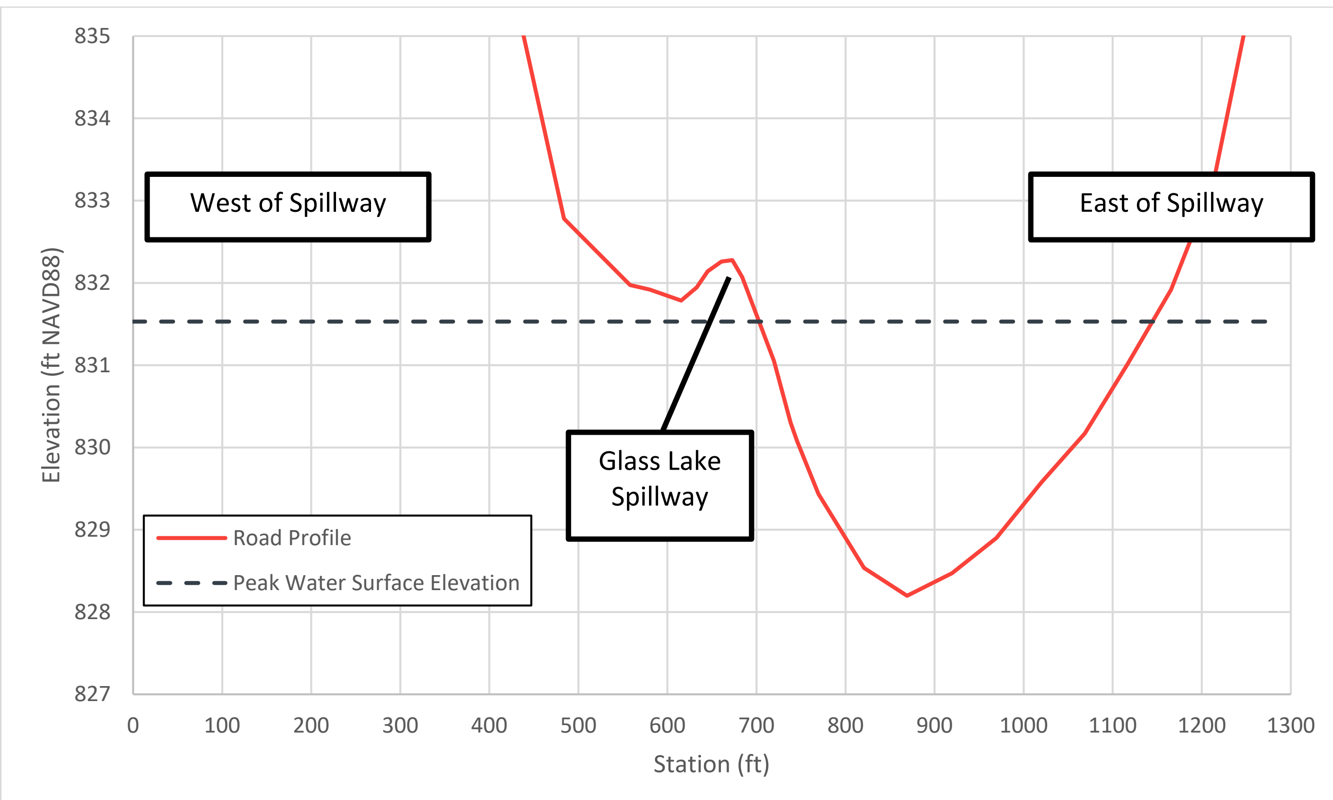


Figure 8: Glass Lake Road Profile at Peak Water Surface Elevation

Based on the results, the spillway section and western abutment of Glass Lake Dam does not overtop during the SDF. However, the embankment section of the Dam dividing the Detention Pond and Glass Lake overtops by 1.1 ft, which rapidly fills the Detention Pond. With the Detention Pond full, Glass Lake Road east of the Dam overtops by up to 3.3 ft at the lowest road crest elevation. Overall overtopping across Glass Lake Road occurs for 14.5 hours.

The flow depth over time for the lowest spot on Glass Lake Road is plotted in Figure 9. The peak flow velocity across Glass Lake Road is approximately 1.9 ft/s.

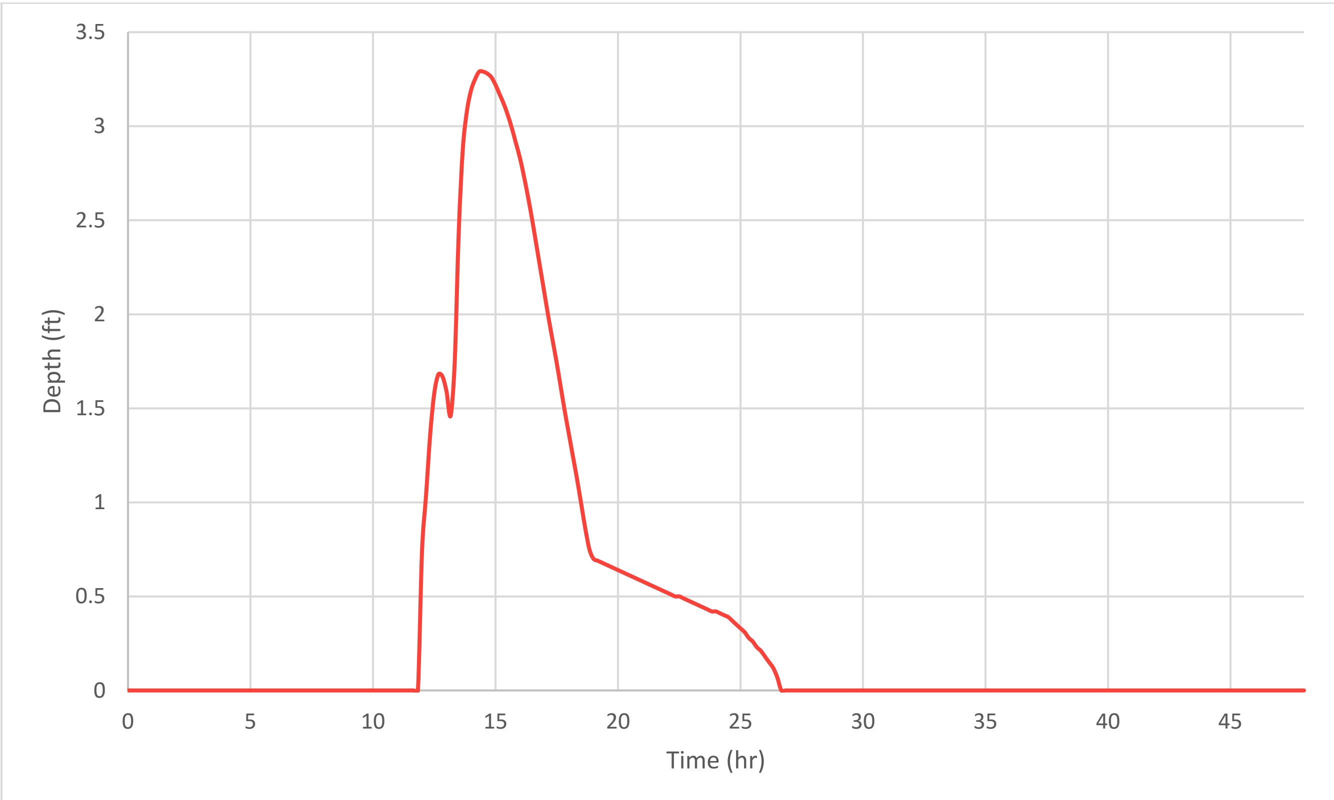


Figure 9: Depth of Flow Over Glass Lake Road East at Point of Lowest Elevation

Downstream of Glass Lake Road, overtopping flow quickly reenters the channel downstream of the spillway. The maximum depths and flood extent are shown in Figure 10.

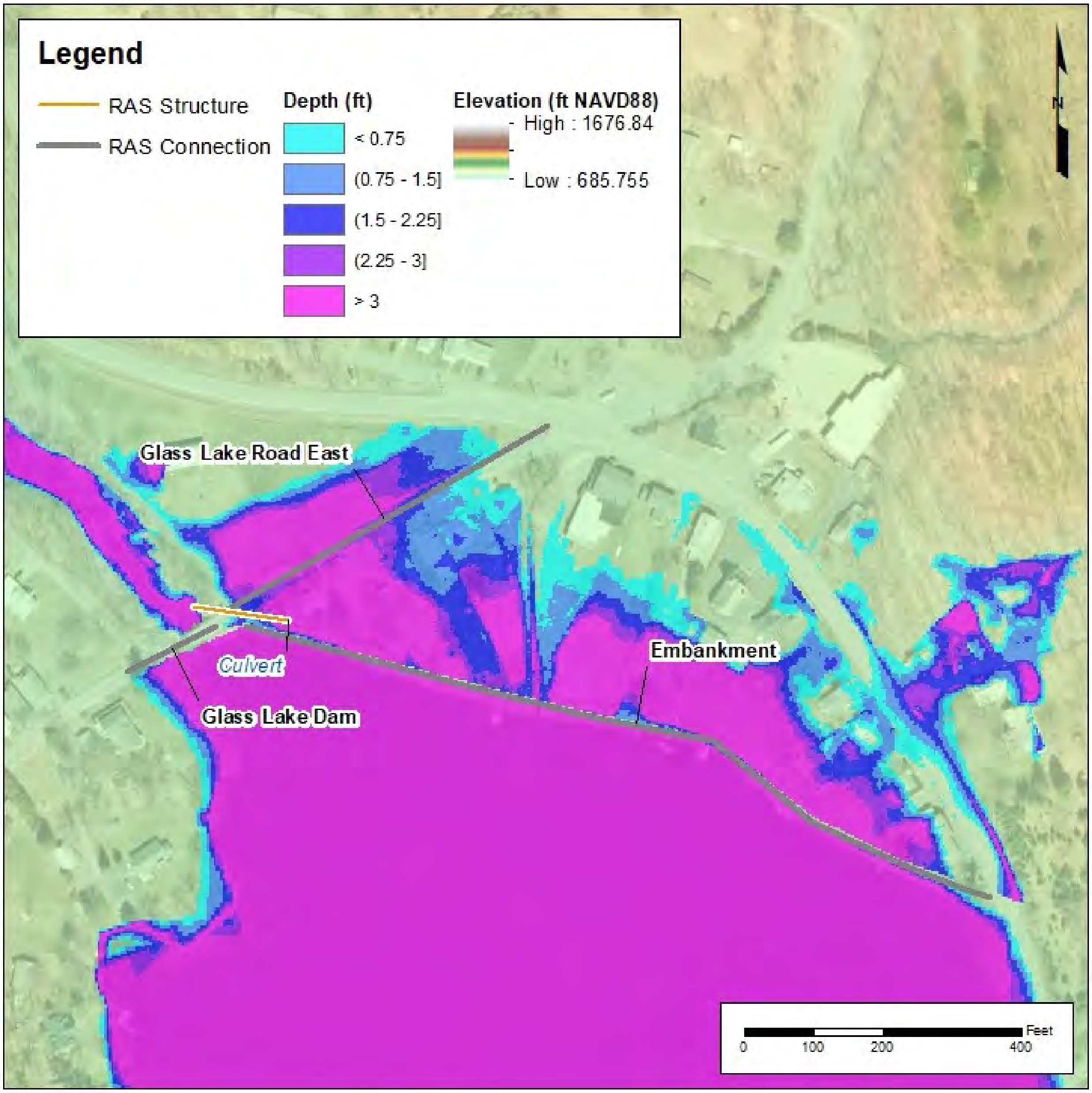


Figure 10: Maximum Depth and Flood Inundation Extents of the SDF at Glass Lake Road

Greg Shaffer, PE, CFM
Lead Engineer
License: NY-093781

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1. WSP, Site Inspection and Survey of Glass Lake Dam, June 2020.
2. WSP, Hydrologic and Hydraulic Assessment for Glass Lake Dam, January 2016.
3. New York State GIS Clearinghouse, 2015, "New York Office of Information Technology Services Classified LiDAR Tiles," Albany, NY, http://gis.ny.gov/elevation/metadata/NY16-Columbia-Rensselaer.xml

APPENDIX

Table 1: Glass Lake Storage Elevation Curve

Elevation (ft NAVD88)	Area (ac)	Volume (ac-ft)
767.40	0	0
827.399	108.2	2707.09
828	125.2	2777.24
829	129.6	2904.66
830	132.8	3035.86
831	135.4	3169.95
832	137.7	3306.53
833	140.0	3445.40

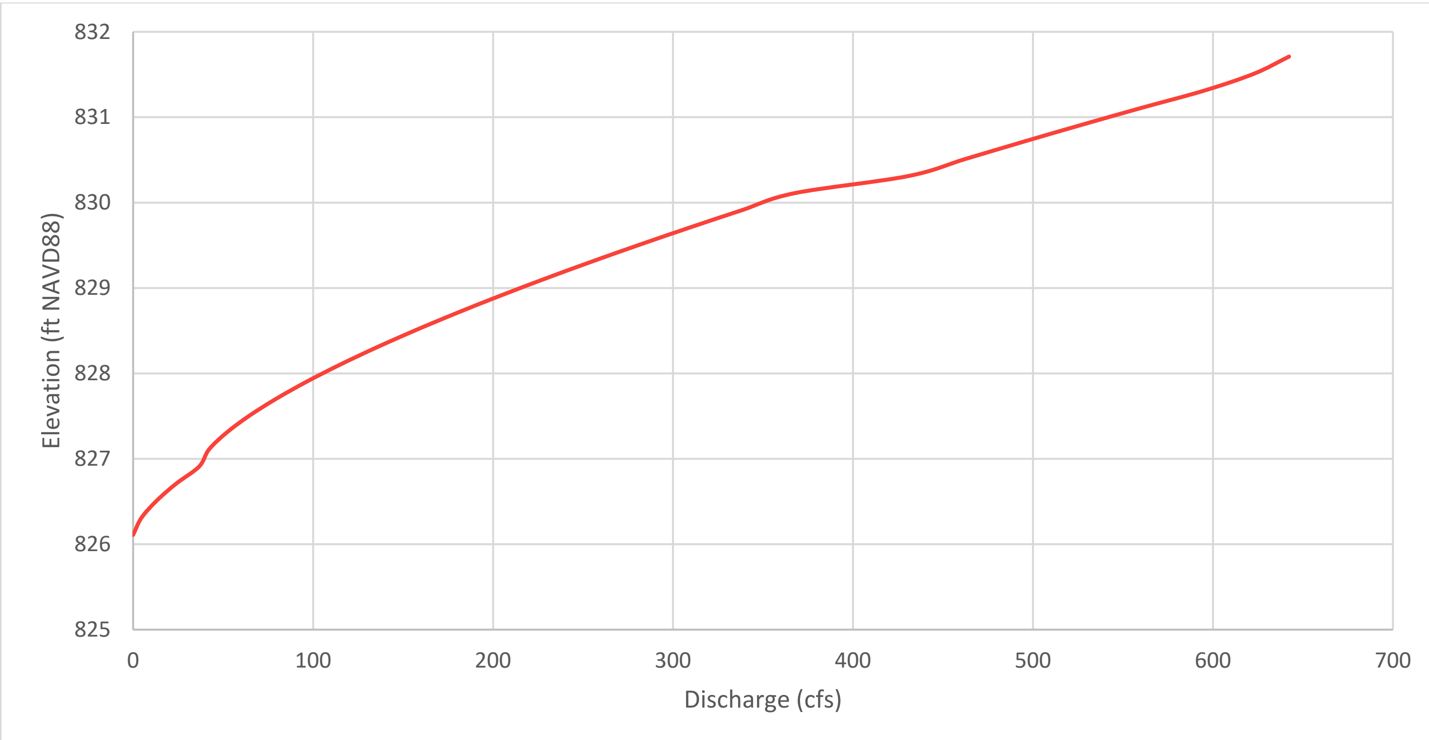

Figure 1: Glass Lake Dam Spillway Rating Curve

Table 2: Glass Lake Dam Spillway Rating Curve Values

Elevation (ft NAVD88)	Discharge (cfs)	Elevation (ft NAVD88)	Discharge (cfs)
826.11	0.00	829.71	309.52
826.31	4.65	829.91	338.06
826.51	13.09	830.11	367.37
826.71	23.94	830.31	430.50
826.91	36.69	830.51	462.30
827.11	42.17	830.71	494.20
827.31	52.12	830.91	526.90
827.51	64.94	831.11	560.40
827.71	80.01	831.31	594.90
827.91	97.01	831.51	622.90
828.11	115.67	831.71	642.30
828.31	135.84		
828.51	157.36		
828.71	180.12		
828.91	204.02		
829.11	228.98		
829.31	254.92		
829.51	281.79		

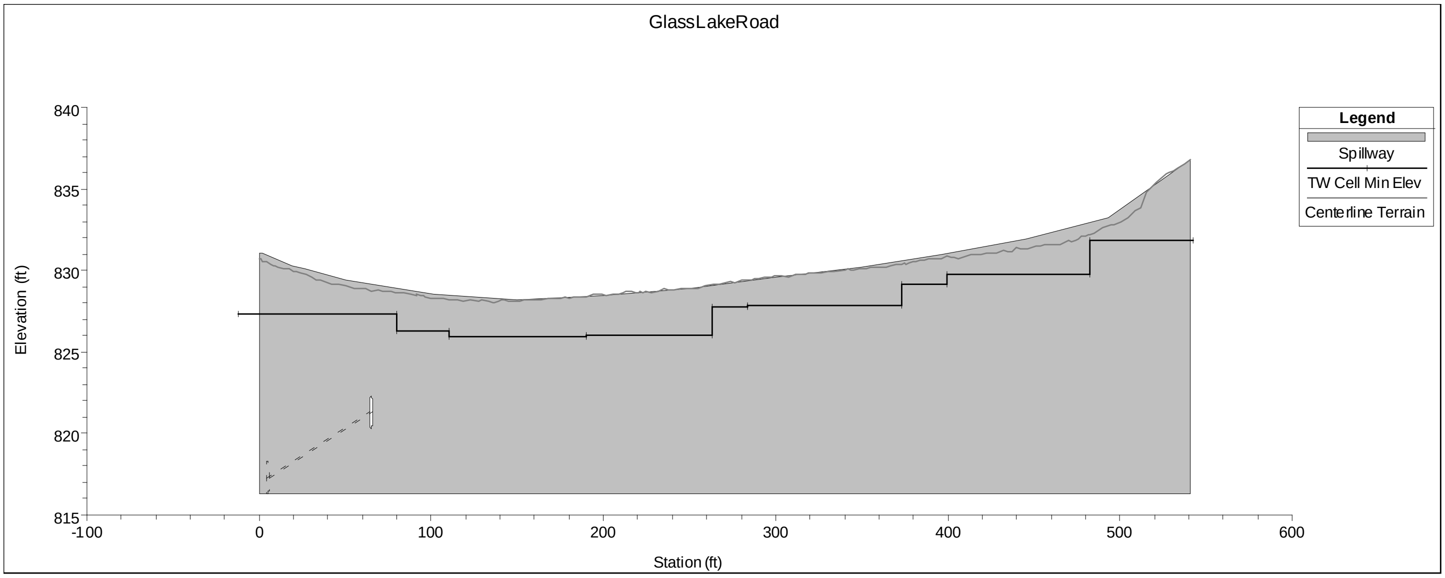

Figure 2: Glass Lake Road Profile

Table 3: Glass Lake Road Culvert Characteristics

Parameter	Value
Culvert Length (ft)	100
Entrance Loss Coefficient	0.5
Exit Loss Coefficient	1
Manning's n for Top	0.013
Manning's n for Bottom	0.013
Upstream Invert Elevation (ft NAVD88)	820.3
Downstream Invert Elevation (ft NAVD88)	816.3

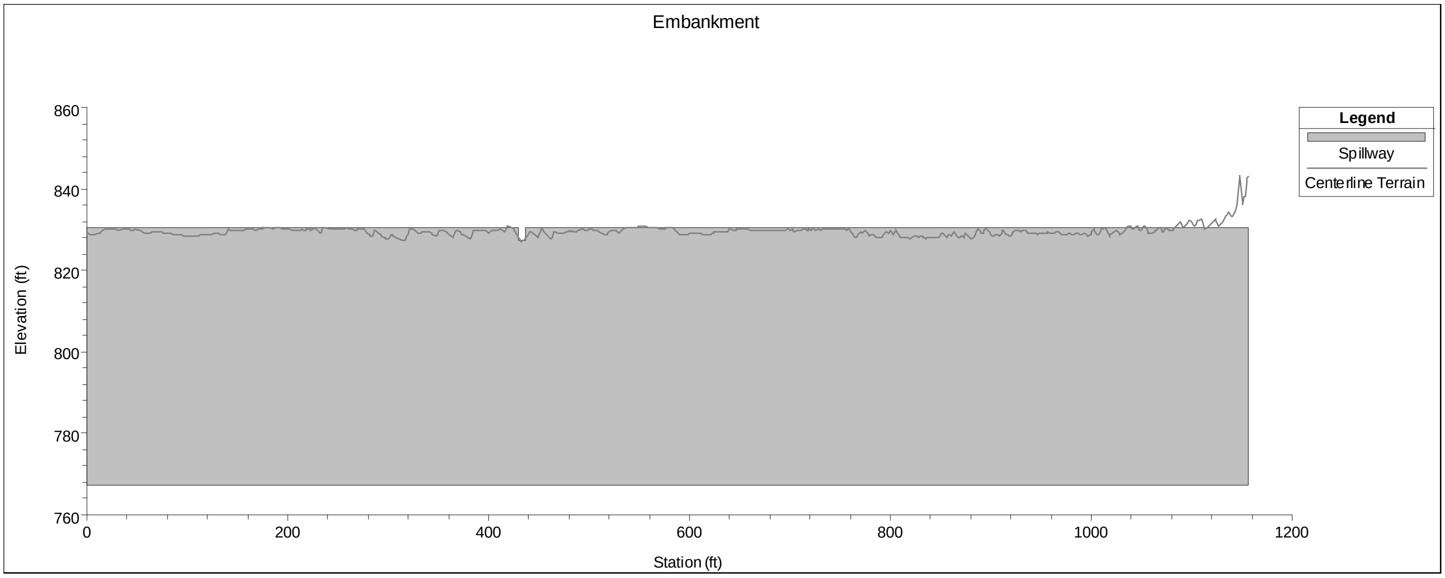

Figure 3: Glass Lake Embankment