Lake Erin Dam Rehabilitation

Basis of Design Report
NID ID: GA01324

City of Tucker
Dekalb County, Georgia

Project number: 60727041

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1. Introduction

1.1 Scope of the Report

This report documents the basis of design for repairs to the Lake Erin Dam. The basis of design includes the supporting analyses, engineering assumptions, input data, modeling results, and design calculations that support the design of each item in the scope of the repairs.

1.2 Statement of Limitations

Interpretation of general subsurface conditions presented herein is based on the soil, rock, and groundwater conditions encountered in the limited number of soil borings. Although representative portions of the samples taken were tested, subsurface conditions may vary outside of the exploration locations. This report does not reflect any variations that may occur across the site in areas not sampled.

This report has been prepared for the specific application to the project discussed and has been prepared in accordance reasonable and accepted engineering practices and standard of care based on the information available to AECOM Technical Services, Inc. (AECOM) at the time of performance of the work. No warranty or guarantee, either written or implied, is applicable to this work.

1.3 Site Description

The Lake Erin Dam (State ID number 044-004-00033 and National Inventory of Dams – NID identification number GA01324) is located on an unnamed tributary to the North Fork Peachtree Creek. Lake Erin Dam is an earthen dam located in Henderson Park in the City of Tucker, DeKalb County, Georgia. The dam was previously owned and maintained by the DeKalb County Department of Public Works, Roads and Drainage Division, but its ownership was recently transferred to the City of Tucker. The primary function of the dam is to serve as a recreational amenity for Henderson Park. Lake Erin Dam is regulated by the Georgia Department of Natural Resources, Environmental Protection Division, Safe Dams Program (SDP). Based on the current characteristics for Lake Erin Dam, SDP has categorized the dam as a Category I, Medium Dam. **Figure 1** below shows the location of the dam and its watershed in context of the surrounding vicinity.

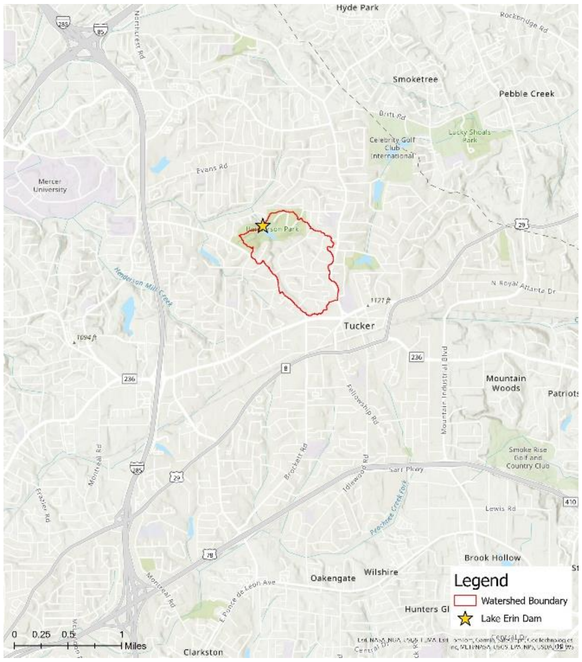

Figure 1: Location and Context Map

The dam is a 32-foot-high earth embankment dam and impounds a 5-acre reservoir. The outlet works of the dam is a principal spillway system consisting of a 30-inch Corrugated Metal Pipe (CMP) intake conduit leading to a riser structure that was constructed of a 30-inch CMP conduit. Immediately downstream of the riser structure is a 30-inch Reinforced Concrete Pipe (RCP) conduit that discharges into the auxiliary spillway channel. The outlet structure for the principal spillway conduit is a concrete end wall that is abutted by the vertical stone masonry training walls of the auxiliary spillway. The auxiliary spillway is an open channel spillway with vertical stone masonry training walls. The auxiliary spillway is founded on bedrock and has a bottom width of approximately 12 feet. There is currently no low-level outlet system at Lake Erin Dam. There is a 9-inch steel standpipe located on the upstream slope that is assumed to be the historic dewatering system but no longer functions. According to the NID, the dam was constructed in 1960. However, the lake appears on the 1956 USGS Stone Mountain Quadrangle sheet, indicating the dam was built before the aerial photograph was taken in 1955.

Table 1, provided on the following page, presents pertinent information about the dam.

Table 1. Dam and Reservoir Data

Description	Value
	General Data
Year of Original Construction	Before 1956 (estimated)
Purpose	Recreation
Current Hazard Classification	Category I
Drainage Area	0.5 square miles
Dam Height at Maximum Section	32 feet
Crest Length	360 feet
Crest Width	Varies from 13 to 20 feet
Upstream Slope	2.5H:1V
Downstream Slope	3H:1V
	Critical Elevations[1]
Normal Pool (Service Spillway Invert)	955.7 feet
Dam Crest	Varies from 968.5 to 969.4 feet
Intake Riser Invert	962.1 feet
Auxiliary Spillway Invert	967.0 feet
Natural Channel Bottom Elevation	936.2 feet
	Storage Capacities
Normal Pool (Elevation 955.7 feet)	30 acre-feet
Max Storage (Elevation 968.5 feet)	159 acre-feet
	Pool Surface Areas
Normal Pool (Elevation 955.7 feet)	5 acres
Dam Crest (Elevation 968.5 feet)	15 acres
	Outlet Features
Principal Spillway Intake Conduit	30-inch CMP conduit
Principal Spillway Intake Structure	30-inch CMP riser structure

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[1] All elevations are reported in the North American Vertical Datum of 1988 (NAVD88).

Description	Value
Principal Spillway	30-inch RCP conduit
Principal Spillway Outlet Structure	Concrete Endwall
Auxiliary Spillway	12-foot-wide masonry spillway founded on bedrock

1.4 Previous Work

Prior to the development of design of repairs to the dam, several efforts were completed to investigate the issues related to the Lake Erin Dam. These efforts are described in this section. Previous work completed by AECOM for the rehabilitation of Lake Erin Dam is included in **Appendix A** of this report.

1.4.1 Lake Erin Dam Hydrologic and Hydraulic Analysis (AECOM, 2021)

AECOM completed a hydrologic and hydraulic (H&H) analysis for the Lake Erin Dam and compared the results against the requirements set by SDP for this category of dam for compliance. Recommendations were also given in this report to correct any non-compliance issues. This work yielded the following findings:

- The design storm for Lake Erin Dam (Criterion 1.1.4.2: Medium Dam) is 33.3% of the Probable Maximum Precipitation (PMP) event.
- No energy dissipator exists at the downstream end of the principal and auxiliary spillway outlets.
- The auxiliary channel is not activated more frequently than during the 2% annual exceedance probability (AEP - 50-year) storm event.
- A freeboard analysis considering the maximum wave height was prepared and indicated a recommended freeboard value of 0.8 feet.
- There is no low-level outlet system at the reservoir; therefore, there is no way to lower the reservoir below the normal pool elevation of 955.7 feet.
- The low point of the crest (elevation 968.5 feet) is overtopped by 0.7 feet during the Spillway Design Flood (SDF) event.

The general recommendations made in this report to address the non-compliance deficiencies identified are:

- Criterion 5.3.4 – Design a sufficient energy dissipation device at the outlet of the spillway(s) such that it is capable of sufficiently dissipating energy from the design peak discharge to meet this criterion.
- Criterion 5.5 – Modify the dam, outlet works, or spillways to increase freeboard to meet the required amount. This could be done by increasing spillway capacity to lower peak water surface elevation for the SDF, raising the crest of the dam, or combinations thereof. A spillway erosion analysis using the SITES software application may be required if an earth-cut auxiliary spillway is pursued as an alternative.
- Criterion 5.7 – Design a gated pipe structure or siphon system to drain two-thirds of the volume at normal pool of the reservoir within ten days or an alternative time frame if approved by SDP.

Three potential rehabilitation alternatives were developed to address the deficiencies identified in this report:

- Rehabilitation Alternative 1 – Widen and lower the auxiliary spillway at the left abutment, raise dam crest low point to elevation 969.0 feet, install a low-level outlet siphon system, and install an energy dissipator at the outlet of the existing masonry spillway.
- Rehabilitation Alternative 2 – Construct an earth-cut auxiliary spillway located at the right abutment, block off the crest of the existing auxiliary spillway at the left abutment, install a low-level outlet siphon system, and install an energy dissipator at the outlet of the existing masonry spillway and new auxiliary spillway at the right abutment.

- Rehabilitation Alternative 3 – Abandon the existing stone masonry spillway, remove the existing principal spillway pipe, construct a new riser structure with controls for a gated low-level outlet and twin 48-inch pipes, and an energy dissipator at the outlet of the pipes.

1.4.2 Spillway Alternatives Analysis (AECOM, 2022a)

AECOM completed a comparative analysis of the three alternative rehabilitation designs presented in the report described in Section 1.4.1 above. This report describes the non-compliance deficiencies previously identified and goes into further detail of the alternatives originally described in the H&H report. The recommendations in the Spillway Alternatives Analysis memo were also updated with minor revisions from the original alternatives, including a fourth alternative which was a variation of the original Rehabilitation Alternative 3.

- Rehabilitation Alternative 3A – A less expensive medication of Alternative 3 would be to re-locate the new principal spillway near the left abutment immediately right of the existing spillway conduit. The relocation would allow the spillway conduit to be shorted and located higher up in the embankment thus reducing costs.

The client selected Alternative 3 as the preferred solution.

1.4.3 Lake Erin Dam Geotechnical Analyses (AECOM, 2022b)

AECOM completed a review of historic geotechnical information and performed two-dimensional slope stability and seepage analyses of the embankment for Lake Erin Dam. The geotechnical analysis Technical Memo (AECOM, 2022) presented the analysis and parameters used to conduct modeling of the existing conditions at Lake Erin Dam. The cross section analyzed was located along the centerline of the existing 6-inch toe drain outlet, which is at the approximate maximum height section of the dam. The subsurface investigation performed by Accura Engineering & Consulting, Inc (Accura) in 2021 was used to determine soil and rock engineering properties for the slope stability and seepage analyses. There were also borings drilled in 2013 by Willmer Engineering Inc. (Wilmer) that were used to estimate engineering parameters of soils.

The results of the slope stability and seepage analyses showed that the existing embankment does not meet the minimum required factor of safety required by Georgia Rules and Regulations 391-3-8 Rules for Dam Safety (Rule 391-3-8-0.9) or U.S. Army Corps of Engineers (USACE) guidelines for upstream rapid drawdown and seismic loading conditions. Analysis with the reservoir pool at the approximate top of dam (elevation 968) indicated that the downstream slope does not meet the required factor of safety at that pool level.

2. Scope of Repairs

The scope of the repairs to the dam and its appurtenant works includes the following measures:

- Demolish and remove existing features as identified on the Drawings;
- Install a new principal spillway system that includes an intake riser, a low-level and principal spillway conduit, and an impact basin;
- Raise the crest elevation to provide sufficient freeboard above the peak water surface elevation for the spillway design flood event that includes wave action and runup for a fetch length and minimum wind velocity of 50 miles per hour (MPH);
- Flatten the upstream slope to 3H:1V for stability and maintenance;
- Regrade and armor approximately 50 linear feet (LF) of the outlet channel downstream of the impact basin;
- Armor the upstream embankment slope from elevation 952.0 to 958.0 feet.
- Install a filter diaphragm around the principal spillway conduit;
- Install a drain at the downstream toe of the embankment, between the principal spillway conduit and the right abutment;
- Install a filter drain on the bedrock of the existing auxiliary spillway along the left abutment of the downstream face of the embankment to collect any seepage along the auxiliary spillway base and backfill spillway; and
- Install a new row of four (4) open well piezometers to monitor and measure the performance of the dam.
- Repair, by regrading and armoring, the downstream section of an eroded lateral channel that ties into the outlet channel of the principal spillway system.

The basis of design for each measure and its resulting effects on previously identified dam safety concerns are documented in subsequent sections of this report. Related calculations, model results, and other supporting data, where applicable, are provided in the appendices to this report. **Table 2** below shows the proposed conditions for Lake Erin Dam.

Table 2: Proposed Conditions

Description	Value
	General Data
Year of Original Construction	Before 1956 (estimated)
Purpose	Recreation
Current Hazard Classification	Category I
Drainage Area	0.5 square miles
Dam Height at Maximum Section	34 feet
Crest Length	360 feet
Crest Width	15 feet
Upstream Slope	3H:1V
Downstream Slope	3H:1V
	Critical Elevations
Normal Pool	955.7 feet

Description	Value
Dam Crest	970.0 feet
Intake Tower Low Stage Weir	955.7 feet
Intake Tower High Stage Weir	961.0 feet
Low Level Outlet Invert	941.0 feet
Natural Channel Bottom Elevation	936.33 feet
Storage Capacities	
Normal Pool (Elevation 955.7 feet)	30 acre-feet
Max Storage (Elevation 970.0 feet)	181 acre-feet
Pool Surface Areas	
Normal Pool (Elevation 955.7 feet)	5 acres
Dam Crest (Elevation 970.0 feet)	16 acres
Outlet Features	
Principal Spillway Intake Structure	Reinforced Concrete Intake Tower
Principal Spillway Conduit	48-inch RCP
Principal Spillway Outlet Structure	USBR Type VI Impact Basin
Low-Level Outlet Intake Structure	GDOT Standard 1001-B, U Type Wing Headwall
Low-Level Outlet Intake Conduit	15-inch RCP

2.1 Demolish and Remove Existing Features

There are existing features at Lake Erin Dam that must be removed to install the proposed features. The existing features to be demolished includes trees, vertical stone masonry training walls of the auxiliary spillway, principal spillway system (intake conduit, intake riser, spillway conduit, concrete endwall), standpipe, toe drains, piezometers, etc. The demolished features will need to be disposed of off-site in accordance with local, state, and federal regulations and laws.

2.2 Install New Principal Spillway

To replace the demolished existing principal and auxiliary spillway, a new principal spillway system will be installed. The principal spillway system will include an intake riser structure, a conduit, and an impact basin at the downstream end. The principal spillway conduit will be a 48-inch diameter ASTM C361 Reinforced Concrete Pipe that consists of a cage or cages of steel reinforcing bars or wire; an encasing wall of concrete; and a preformed gasket of rubber to provide the joint seal between adjacent pipes. To provide a stable foundation for the principal spillway conduit, the conduit will be installed in a reinforced concrete encasement. The principal spillway conduit will run at a slope of approximately 0.77% and outlet directly to a United States Bureau of Reclamation (USBR) Type VI impact basin. A rating curve relating water surface elevation in the reservoir to discharge through the spillway system was developed for the riser structure. The capacity of the system at the proposed dam crest elevation of 970.0 feet is approximately 330 cubic feet per second (cfs). The intake structure for the system will be a similar design to a standard Natural Resources Conservation Service (NRCS) style riser structure that is constructed of reinforced concrete. This riser structure will have two weir elevations, one to maintain the normal pool elevation and discharge flows for flood events up to the 50-year frequency event, and one to discharge the SDF flows. The low-stage weir will be placed at the existing normal pool elevation of 955.7 feet to maintain the existing pool level. The high-stage weir will be placed at elevation 961.0 feet. During the SDF event, the intake riser will be submerged, which is accommodated by the NRCS design.

The low-level outlet system will consist of a 15-inch diameter ASTM C361 Reinforced Concrete Pipe that consists of the same material specifications as identified above for the principal spillway conduit. To provide a stable foundation for the low-level conduit, the conduit will be installed on a reinforced concrete cradle up to the springline. The low-level outlet system will be controlled by a gate in the intake structure that will be operated from the top of the riser structure. The gate will have a hand activated actuator that only needs one person to operate. At the upstream end of the low-level outlet system, a reinforced concrete headwall will be installed. The headwall will be a modification of the Georgia Department of Transportation (GDOT) Number 1001-B, *Standard Pipe Culvert Concrete Headwall* U Type Wing for 15-inch circular conduits. The modification specifically is to expand the footing of the wall upstream to the ends of the wings to accommodate a trash rack. The trash rack will be installed over the openings to the conduits for two reasons:

- To prevent debris from clogging the conduits; and
- To prevent unauthorized access to the upstream end of the conduits.

The USBR Type VI impact basin is designed for the principal spillway conduit's size and hydraulics anticipated during the SDF event. Due to the velocities (26 feet per second) anticipated from this spillway system, other energy dissipation structures such as scour pools and stilling basins were ruled out as insufficient to adequately dissipate the energy from the spillway discharge to non-erosive levels. The USBR Type VI impact basin's minimum geometry was designed in accordance with guidance provided in the FHWA's Hydraulic Engineering Circular Number 14, *Hydraulic Design of Energy Dissipators for Culverts and Channels*, 3rd Edition (FHWA, 2006) and is primarily based on the Froude number for the design discharge entering the basin. The minimum geometry was augmented to ensure structural stability and the ability to retain the surrounding embankment fill, resulting in a basin measuring approximately 25 feet long by 20 feet wide by 15 feet high. Approximately 50 feet of riprap (D_{50} = 12 inches) will be placed in the stream channel immediately downstream of the impact basin's end sill to further dissipate energy.

A safety fence will be installed around the impact basin on the upstream end and sides of the structure to prevent unauthorized access to the structure.

2.3 Raise Embankment Crest

The crest of the embankment will be raised to establish a consistent elevation of 970.0 feet along the downstream top of crest. A two percent cross slope will be provided along the crest from the downstream to upstream, draining runoff towards the reservoir. New earth fill will be placed and compacted in lifts using material excavated from other work in the dam embankment or imported from an approved source as necessary. The purpose of this improvement is to ensure that, in the unlikely event of dam overtopping, water flows evenly over the dam at a consistent depth without concentrating in low areas. This helps reduce the risk of dam failure as it lessens the erosive forces of the overtopping discharge. The proposed conditions hydrologic and hydraulic modeling indicate that a crest at this elevation will provide approximately 1.1 feet of freeboard above the peak water surface elevation (968.9 feet) for the SDF event. This exceeds the recommended freeboard that was determined in the previously completed H&H report (AECOM, 2021).

2.4 Flatten the Upstream Slope

As part of the crest raise work described above, the upstream slope will be flattened to a consistent 3H:1V. New earth fill will be placed and compacted in lifts using material excavated from other work in the dam embankment. The purpose of this repair is to address slope stability and maintenance concerns. The existing typical upstream slope is approximately 2.5H:1V. The upstream slope stability for existing conditions does not meet the required factor of safety. It is also difficult to maintain the vegetation on the upstream slope due to the steep slope. Flattening the slope would extend the toe of the upstream slope approximately 40 feet upstream of its current location. To protect the upstream slope from erosion due to wave action, riprap slope protection (D_{50} = 12 inches) will be installed along the upstream slope from elevation 952.0 feet to elevation 958.0 feet. The riprap will be founded on a layer of bedding stone and sand.

2.5 Regrade and Armor Outlet Channel Downstream

The existing outlet channel will be regraded to allow for the installation of the proposed principal spillway system and convey reservoir discharges with an alignment that is hydraulically efficient. The proposed channel grade will tie into existing ground and the channel will be armored with riprap (D_{50} = 12 inches) for approximately 50 feet downstream of the impact basin. The purpose of this repair is to provide a well-defined channel downstream of the impact basin with a consistent slope that will reduce the flow to a non-erodible velocity downstream of the impact basin. There is an existing localized high point downstream of the toe of the dam that would impede the flow discharging from the impact basin. This high point will be graded out and removed. The outlet channel will have varying channel sections to maximize the space available to convey the SDF at a non-erosive velocity and to limit the amount of fill that is placed within the existing stream channel. At the outlet of the impact basin, there will be a well-defined channel that has a bottom width of 6 feet, a depth of 6 feet, a channel slope of 0.35%, and 2H:1V side slopes. Approximately 12 feet downstream of the initial channel section, the left overbank widens out approximately 40 feet towards the invert of the old auxiliary spillway while the right side slope is maintained at 2H:1V. The channel slope will be consistent throughout both of the channel sections described above. After the widening, the flow will constrict back to an existing well-defined channel that continues beyond the site. Using the sizing parameters in HEC-14 (FHWA, 2006), a riprap apron that is approximately 50 feet long will reduce the velocity to non-erosive conditions.

2.6 Filter Drain, Toe Drain, and Filter Diaphragm

Given the existing conditions and proposed installation of a principal spillway conduit, the following drainage systems are proposed:

- Filter drain (West Toe Drain) along the bottom of the downstream left abutment at the existing stone masonry auxiliary spillway
- Filter Diaphragm around the principal spillway conduit
- Toe Drain along the right downstream groin of the embankment (East Toe Drain)

The proposed drainage systems are designed to capture flow within and adjacent to the embankment and convey flow downstream via drainpipe conduits. The drainage systems will be constructed with coarse aggregate, which primarily acts as a drain material to convey seepage to the drainpipes, and a fine aggregate, which primarily functions as a filter to prevent particle movement. This system will provide filtered seepage collection and discharge to reduce the internal erosion of the embankment and natural soils.

Several benefits of the drainage systems include:

- The filtering capability of the material in the filter diaphragm will work to prevent preferential seepage paths within the embankment around the principal spillway conduit from carrying fine soil particles out of the dam and will reduce the potential for internal erosion issues;
- Collecting seepage will result in a lower phreatic surface in the embankment, decreasing the porewater pressure in the embankment and improving the slope stability; and
- The drain conduit outlets will provide a way to measure and monitor seepage from the embankment, creating an early detection system for potential future seepage issues.

2.6.1 Filter Drain (West Toe Drain)

As part of the proposed rehabilitation for Lake Erin Dam, the existing channel lining of the stone masonry auxiliary spillway is to be abandoned-in-place and backfilled. A filter drain, hereby referred to as the West Toe Drain and comprised of a 6-inch diameter High-Density Polyethylene (HDPE) drainage pipe, is proposed to be placed near the bottom of the existing auxiliary spillway to capture any flow along the base or through fractures of the bedrock. The West Toe Drain filter will extend the width of the existing auxiliary spillway and the 6-inch HDPE drainpipe will outlet through a precast concrete endwall to the riprap lined outlet channel downstream.

2.6.2 East Toe Drain

Along the toe of the downstream embankment slope is an existing toe drain; the precise location of which is unknown at this time. This drainpipe will be removed during rehabilitation and a new 6-inch HDPE drainpipe, embedded in filter materials, is proposed to be installed. This design prevents sediment transport or blockage of the new 6-inch HDPR drainpipe, facilitates the collection and transport of seepage along the toe to the downstream to the outlet channel, and limits the potential for surficial seepage at or immediately downstream of the toe of the dam. The East Toe Drain will be installed along the right toe and the 6-inch HDPE drainpipe will outlet through a precast concrete endwall to the riprap lined outlet channel downstream.

2.6.3 Filter Diaphragm

Current standards of practice recommend that a filter diaphragm or chimney filter be installed on embankment dams. Therefore, a filter diaphragm for the 48-inch RCP principal conduit is designed to meet the requirement of the National Engineering Handbook, Part 628, Dams, Chapter 45, Filter Diaphragms (NRCS, 2007). The filter diagram will intercept water that can flow through cracks in the surrounding fill or along the interface of the conduit, and prevent sediment transport along the conduits, which could lead to internal erosion.

2.7 Piezometers

A row of four open well piezometers will be installed parallel and to the right of the proposed principal spillway. One piezometer will be installed just upstream of the embankment centerline. The second piezometer will be placed approximately mid-slope on the downstream embankment. The third piezometer will be placed near the toe of the embankment, approximately in line with the impact basin end sill. The fourth piezometer will be installed beyond the toe of the dam between the spillway outlet channel and the lateral drainage swale along the right groin. The piezometers will be installed using 2-inch diameter schedule 40 Polyvinyl Chloride (PVC) pipe. All piezometers will have a well cap encased in concrete to protect from damage due vandalism or maintenance. The vaults should be installed flush with the proposed grade so that regular mowing activities do not damage the vault or piezometer. For the two piezometers installed on the downstream embankment, the vaults will need to be installed flush to the 3H:1V slope.

3. Proposed Conditions Geotechnical Analysis

3.1 Filter Diaphragm Design

As discussed in Section 2.6, a filter diaphragm was designed for the 48-inch RCP principal conduit to meet the requirements of chapter 45, Part 628 of the National Engineering Handbook (NRCS, 2007) to prevent particle movement adjacent to the principal spillway conduit, which could initiate internal erosion.

The filter diaphragm is comprised of fine aggregate meeting either ASTM International Standard C-33 Fine Aggregate or Georgia Department of Transportation (GDOT) 10 NS sand. The required thickness of the filter diaphragm is 3.0 feet times the height of the conduit in the horizontal and vertical direction, and a minimum of 2.0 feet below the bottom of the conduit (NRCS 2007). The filter diaphragm will contain a drainage layer of coarse aggregate and a slotted 6-inch HDPE pipe convey the collected seepage to the toe drain. Design calculations for the filter and drain features are provided in **Appendix B** of this report.

A primary drainage system was designed to outlet flow from the filter diaphragm to the downstream embankment toe. The drainage system consists of a slotted 6-inch HDPE pipe within the filter diaphragm and embedded in ASTM #8 or similar coarse aggregate, transitioning to a solid wall 6-inch HDPE drainpipe running adjacent to the RCP principal spillway conduit along the right side. The 6-inch HDPE pipe will be the primary drain of the filter diaphragm and will discharge adjacent to the impact basin, which will enable the measurement of seepage collected by the filter diaphragm.

On the left side of the RCP principal spillway conduit, a two-stage strip drain consisting of ASTM C33 fine aggregate and ASTM #8 coarse aggregate was designed as a secondary drainage system. The design of the strip drain system was based on NRCS guidelines (NRCS, 2007). The base (fine aggregate section) of the strip drain is set at 9-inches above the invert of the filter diaphragm outlet drainpipe so that accumulated seepage in the filter diaphragm will pass through the drainpipe during normal operation of the dam. The strip drain is designed as a secondary system and to activate only if the 6-inch HDPE drainpipe became unable to function properly and hydrostatic pressure were to build up within the filter diaphragm. Results from the geotechnical seepage model and NRCS (2007) design guidelines were used to estimate the required discharge capacity of the filter diaphragm and drain conduits.

Capacity calculations of the 6-inch slotted filter diagram drainpipe sizing showed that the proposed 6-inch diameter drain conduit is sufficient to convey the estimated seepage discharge from the filter diaphragm, which was calculated assuming an embankment fill permeability 100 times the estimated design permeability value. The 6-inch filter diagram pipe slots were designed to retain the adjacent coarse drain materials. Pipe stiffness was also checked to ensure that the proposed pipe will withstand overburden stresses and construction loads. Calculations for the filter diaphragm, strip drain design, and pipe design are provided in **Appendix B** of this report.

3.1.1 Filter Compatibility

Filter compatibility analysis was performed based on USACE methodology detailed in EM 1110-2-2300 General Design and Construction Considerations for Earth and Rock-Fill Dams (2004). Filter compatibility analysis is based on grain size distribution of a base and filter material to determine if a candidate filter material is of sufficient size to adequately allow seepage to free flow while maintaining the integrity of the base soil. The proposed filter material is ASTM C33 Fine Aggregate which will be utilized to filter seepage from Embankment Fill and Alluvial Soils. Analysis was also performed to evaluate compatibility between ASTM C33 Fine Aggregate and ASTM No. 8 Coarse Aggregate. ASTM No. 8 Coarse Aggregate will be utilized as a drain material to transmit captured seepage once passed through the filter material. In addition, Georgia department of Transportation 10 NS Sand was analyzed as an alternate for the ASTM C33 Fine Aggregate. For this analysis, United States Army Corps of Engineers methodology was used as detailed in EM 1110-2-2300 (2004).

Filter compatibility for the riprap base layer was analyzed using ASTM No. 3 coarse aggregate. This material will be utilized as a transition material between the embankment and foundations soils and the riprap.

Gradations for the Embankment Fill were obtained from laboratory testing on samples collected from subsurface investigation performed in 2021 and detailed in the Geotechnical Data Report for Erin Lake Dam (2021). Results from borings AB-2 and AB-3 were utilized as they are representative of the soils adjacent to the filters.

Results of the compatibility analyses are summarized in **Table 3**.

Table 3. Drain Filter Compatibility Analysis Results

Dam Material	Compatible For:	
	Filtration	Drainage
Embankment Fill (base) to ASTM C33 Fine Aggregate (filter)	Yes	Yes
Alluvium (base) to ASTMC33 Fine Aggregate (filter)	Yes	Yes
ASTM C33 Sand (base) to ASTM C33 No. 8 Stone (filter)	Yes	Yes
Embankment Fill (base) to GDOT 10 NS Sand (filter)	Yes	Yes
Alluvium (base) to GDOT 10 NS Sand (filter)	Yes	Yes
ASTM C33 No. 8 Stone (base) ASTM No. 3 Coarse Aggregate (filter)	Yes	Yes

The results of the analysis show the ASTM C33 Fine Aggregate and GDOT NS 10 Sand are compatible with the existing Embankment Fill and Alluvial soil at Erin Lake Dam. In addition, the ASTM C33 Fine Aggregate is compatible with ASTM No. 8 Coarse Aggregate while the ASTM No. 8 coarse aggregate is compatible with ASTM No. 3 coarse aggregate.

3.1.2 Proposed Conditions Slope Stability Analysis and Seepage Modeling

Slope stability analysis and seepage modeling for proposed conditions were performed in a similar manner for existing conditions using GeoStudio 2020 SEEP/W and SLOPE/W computer modelling software. The seepage analyses were performed in general accordance with USACE EM 1110-2-1901 Engineering and Design “Seepage Analysis and Control for Dams” (USACE, 1993).

The stability evaluation utilizes section geometry with input parameters including soil and rock parameters and loading conditions. The slope stability analyses of the embankment were performed using Spencer’s method of slices. Spencer’s method of slices satisfies all conditions of static equilibrium for horizontal and vertical force equilibrium and moment equilibrium. Factors of safety for slope stability analysis were analyzed using a minimum slip surface depth of two feet. Optimization was utilized for all slope stability analyses.

The embankment slope stability analysis was conducted using required minimum factors of safety from the Georgia Rules and Regulations 391-3-8 Rules for Dam Safety (Rule 391-3-8-.09) for the stability of earth embankment structures and USACE EM 110-2-1902 Slope Stability (USACE, 2003). These two guidance documents provide minimum factor of safety values for steady state seepage (normal and maximum storage pool), maximum surcharge pool, rapid drawdown (upstream), earthquake (seismic) loading, and end-of-construction conditions.

End-of-construction conditions were analyzed using drained strength parameters in free-draining materials and undrained strength parameters for materials that assumed to drain slowly. The end-of-construction case evaluates the condition that non-free draining materials may not drain sufficiently as loading conditions are applied, such as during or directly following placement of fill material. The end-of construction loading condition was analyzed with the reservoir at drawn down and normal pool conditions. For this analysis, the Embankment Fill, Alluvium, and Residual soils were analyzed with undrained material strength parameters for the maximum height cross section. The offset cross section was not analyzed for this condition as it is outside the excavated cut slope for the principal spillway installation and therefore not subject to end of construction conditions.

In addition to end of construction, a stability analysis was performed on the excavated slope section for the principal spillway construction. While the construction is anticipated to be performed under drained conditions, both effective and total strength conditions were analyzed. The excavated slope section was analyzed along the crest of the dam which is the maximum height of the excavation. The maximum allowable excavation slopes are 2H:1V. As both excavated slopes are similar, the interior excavated slope was analyzed.

Georgia Department of Natural Resources “Engineer Guidelines” (2015) details that in limited instances, rapid drawdown due to submergence of the downstream toe may be a consideration. Rapid drawdown on the downstream toe of the dam was not analyzed as the downstream toe is not anticipated to become submerged beyond the designed riprap protection at the principal spillway outlet.

Based on the proposed design, the crest of the dam will be increased to 970.0 feet. The normal pool elevation was analyzed to be 955.7 feet with maximum surcharge pool elevation at 969.0 feet.

Slope stability analysis and seepage modeling for proposed conditions were analyzed for two cross-sections: 1) adjacent to the proposed principal spillway conduit which represents the maximum embankment section of the dam outside of the principal spillway conduit profile, and 2) an offset cross section approximately 100 feet to the right of the principal spillway conduit which is outside of the proposed excavated section. The geometry of the maximum embankment section contains the filter diaphragm with drainpipe to represent phreatic surface conditions. The locations of the cross-sections analyzed are shown in **Figure 2** below.

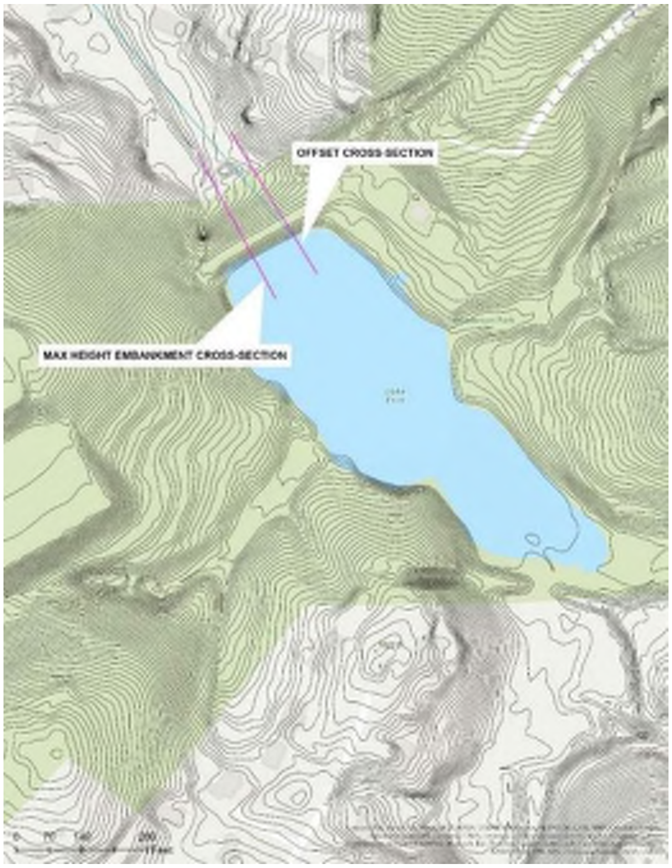

Figure 2: Cross section locations for seepage and stability analyses

Soil material properties for alluvium, residual soil, bedrock, and existing embankment fill used in the analysis were the same as the evaluation for existing conditions detailed in the Lake Erin Dam Geotechnical

Analyses (AECOM, 2022b). Soil strength properties for the Proposed Embankment Fill were based on empirical values obtained from literature and engineering judgement. The Proposed Embankment Fill is anticipated to have increased strength properties from the existing embankment material when compacted during construction, given the low blow counts observed during subsurface investigation of the existing embankment.

Filter drain hydraulic conductivity was estimated using Hazen's equation, which is based on D10 (10 percent size) gradation. Soil strength material properties for the filter material were estimated based on USACE ERDC/GSL TR-0802 Mechanical and Physical Properties of ASTM C33 Sand (2008). Material properties used in the analysis are provided in **Table 4** and **Table 5**.

Table 4: Soil Hydraulic Conductivity and Anisotropy Material Properties

Material Description	Range of Typical Hydraulic Conductivity Values (cm/sec) (NRCS)		Laboratory Tested Values (k_v) (cm/sec)	Selected Horizontal Hydraulic Conductivity, kh (cm/sec)	Selected Anisotropy (k_v/k_h)
	maximum	minimum			
Embankment Fill	1.00E-03 (SM) 1.00E-06 (SC)	1.00E-06 (SM) 1.00E-08 (SC)	2.7E-07	1.08E-06	0.25
Alluvium	1.00E-03 (SM) 1.00E-06 (SC)	1.00E-06 (SM) 1.00E-08 (SC)	-	1.10E-06	0.67
Residual Soil	1.00E-03 (SM)	1.00E-06 (SM)	-	2.50E-06	0.5
Bedrock	1.2E-08 unfractured igneous and metamorphic rock	1.2E-12 unfractured igneous and metamorphic rock	-	1.00E-09	1
Proposed Embankment Fill	-	-	-	1.08E-06	0.25
Drain Fill	-	-	-	3.5E-02	1

Table 5: Soil Strength Material Properties

Material Description	Saturated Unit Weight (pcf)	Shear Strength Parameters			
		Total Stress		Effective Stress	
		Φ (deg)	c (psf)	Φ' (deg)	c' (psf)
Embankment Fill	125	15	301	28	33
Alluvium	129	15	252	27	11
Residual Soil	132	18	250	31	0
Bedrock	165	0	225,000	0	225,000
Proposed Embankment Fill	128	0	1000	31	33
Drain Fill	130	35	0	35	0

An schematic of the maximum cross section and offset cross section are shown in **Figure 3** and **Figure 4**.

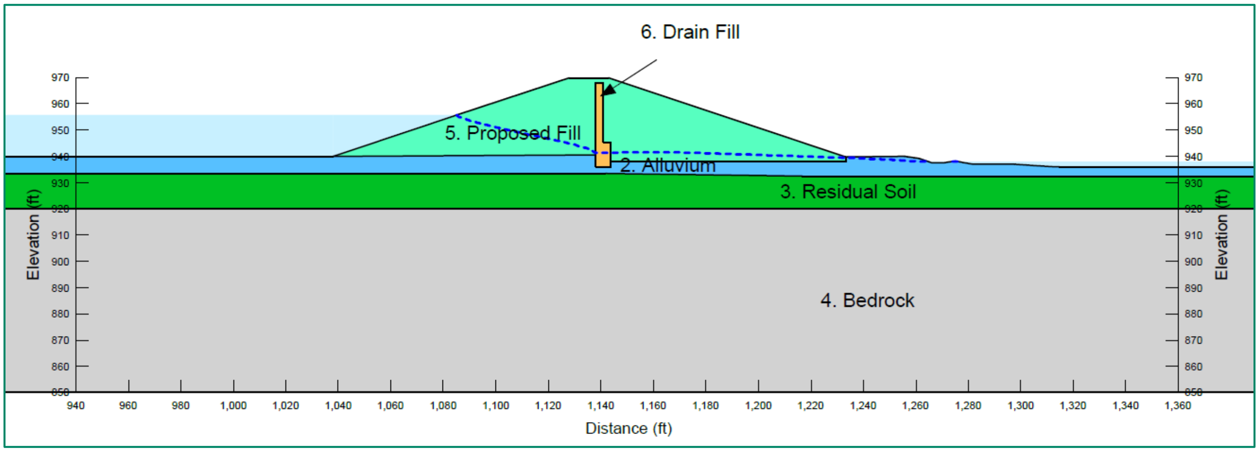

Figure 3: Maximum Embankment Cross Section

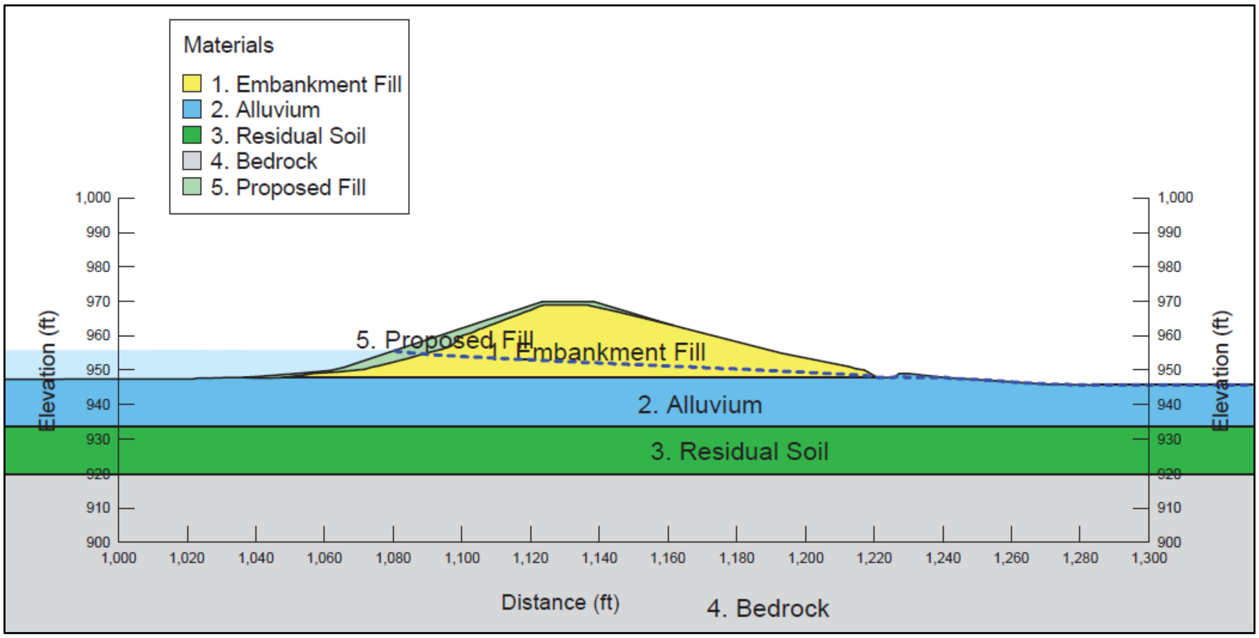

Figure 4: 100ft Offset Cross Section

For the modeled proposed embankment cross sections, the critical (lowest) slope stability factors of safety and the respective USACE, and Georgia State regulations minimum required factors of safety are presented in **Table 6** for the cross section adjacent to the principal spillway conduit and in **Table 7** for the offset cross section. The results of the seepage and slope stability analyses are provided in **Appendix B** of this report.

Table 6. Proposed Conditions Critical Slope Stability Factors of Safety Deep Cross Section

Analysis Condition	Required Minimum Factor of Safety (USACE, 2003)	Required Minimum Factor of Safety (Georgia State Regulations)	Calculated Minimum Factor of Safety (Deep)
Steady State Seepage – Normal Pool	1.5	1.5	1.8
Steady State Seepage – Maximum Surcharge Pool	1.4	N/A	1.7
Steady State Seepage with Seismic Loading – Normal Pool	N/A[1]	1.1	1.1
Rapid Drawdown (Upstream, Normal Pool to (942 ft)	1.3	1.3	1.4
Rapid Drawdown (Upstream, Maximum Pool to Normal Pool)	1.1	-	1.6
End of Construction	1.3	1.3	1.7

No required value provided by USACE EM 1110-2-1902 *Slope Stability,* 2003

Table 7. Proposed Conditions Critical Slope Stability Factors of Safety Offset Cross Section

Analysis Condition	Required Minimum Factor of Safety (USACE, 2003)	Required Minimum Factor of Safety (Georgia State Regulations)	Calculated Minimum Factor of Safety (Offset)
Steady State Seepage – Normal Pool	1.5	1.5	2.0
Steady State Seepage – Maximum Surcharge Pool	1.4	N/A	1.5
Steady State Seepage with Seismic Loading – Normal Pool	1.1[1]	1.1	1.1
Rapid Drawdown (Upstream, Normal Pool to (942 ft)	1.3	1.3	1.6
Rapid Drawdown (Upstream, Maximum Pool to Normal Pool)	1.1	-	1.5

Table 8. Cut Slope Analysis

Analysis Condition	Required Minimum Factor of Safety (USACE, 2003)	Required Minimum Factor of Safety (Georgia State Regulations)	Calculated Minimum Factor of Safety (Offset)
During Construction	1.3	1.3*	1.3

*Assumed based on end of construction requirements

The results presented above indicate that the proposed rehabilitation design will meet the minimum required factors of safety for the analyzed load conditions.

3.2 Bearing Capacity

The allowable bearing capacity for the intake riser and impact basin was estimated based on guidance from EM 1110-1-1905 *Bearing Capacity of Soils* (USACE, 1992) and Principles of Geotechnical Engineering, 5th ed (Das, 2003). Based on previous subsurface investigation, the intake riser and impact basin will be founded on Alluvium or weathered rock/Residual Soil. For conservatism, bearing capacity analysis assumed Alluvial materials. Based on reference guidance and engineering judgement, the recommended bearing capacity for the intake riser is 3924 psf and 5390 psf for the impact basin. Analysis of the foundation materials for bearing capacity are provided in **Appendix B** of this report.

3.3 Settlement Analysis

At Erin Lake Dam, material is to be excavated along the alignment of the 48” principal spillway conduit. Once construction of the conduit is complete, the embankment will be reconstructed, raised to a crest elevation of 970 ft, and the slopes regraded. Settlement analysis was therefore performed to determine the anticipated total settlement above the excavated area. Analysis was performed using Rocscience Settle3 software (Version 5.022) which analyzes immediate settlement, primary consolidation, and secondary settlement.

The results of the analysis show an expected total settlement of less than 1-inch. Given the sandy nature of the existing foundation material, the majority of the settlement is expected to occur during construction, Therefore, it is recommended to overbuild the crest of the dam by 1-inch to ensure the crest maintains a minimum of 970 ft elevation. The results of the Settlement analysis are provided in **Appendix B** of this report.

4. Proposed Conditions Hydrologic and Hydraulic Analysis

To confirm that the proposed repairs address the dam safety issues relating to the inadequate hydrologic and hydraulic performance of the dam, a hydrologic and hydraulic model of the proposed conditions of the dam was developed. Along with the proposed conditions model, calculations were performed for the sizing of the low-level outlet, principal spillway, impact basin, and the riprap outlet protection.

4.1 Proposed Conditions Model

A proposed conditions model was developed to represent the proposed conditions for this rehabilitation project. The model developed in the previous H&H analysis (AECOM, 2021) was used as a starting point. The elevation discharge curve developed for the proposed intake structure was input into the hydrologic model. This rating curve was developed empirically using equations for the prevailing hydraulic regimes that are referenced in **Appendix C** of this report. During the SDF event, the 48-inch principal spillway conduit will control the discharge from the reservoir and the riser structure will be submerged. The hydrologic inputs and the precipitation inputs are not expected to change and therefore, were not revised. The elevation storage rating for the reservoir was updated to reflect the proposed conditions. The elevation storage rating for proposed conditions did not impact the results of the SDF event but did have a small impact on the resulting peak water surface elevation for 4%, 2%, and 1% AEP events. The proposed stage storage rating for the reservoir is included in **Appendix C** of this report. **Figure 5** shows stage-storage rating for the proposed reservoir conditions. There was one error found with the previous H&H analysis where the curve number for the land cover types of developed (medium intensity) and forest were swapped. Since the Lake Erin watershed has a larger area of developed (medium intensity) than forested area, this error resulted in the curve number being underestimated. The calculations were revised to assign the correct curve number to both land cover types, which resulted in the composite curve number increasing by 3. The revised curve number was used in the existing and proposed conditions model. **Figure 6** shows a comparison of the elevation-discharge ratings for the existing and new principal spillways.

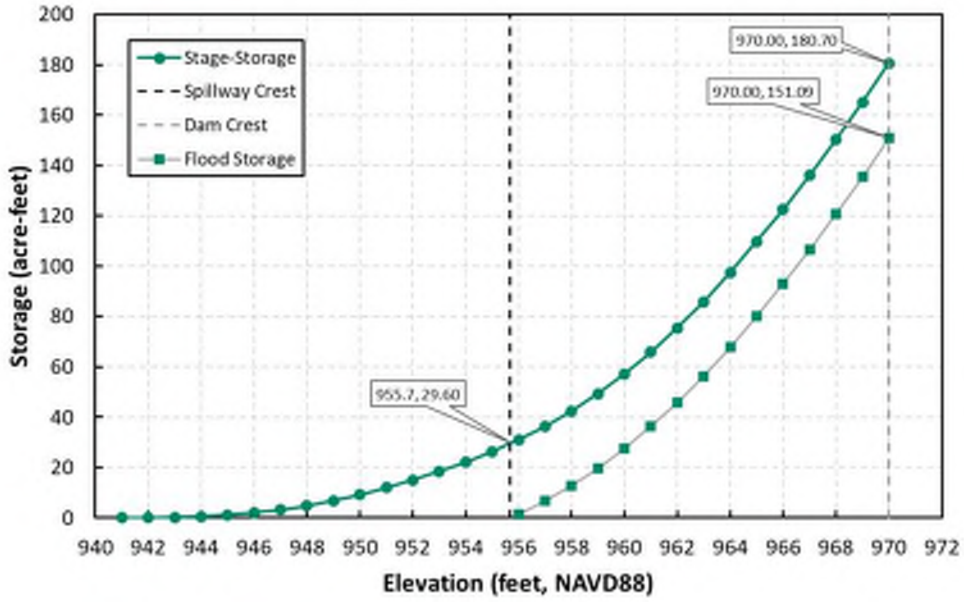

Figure 5: Elevation-Storage Rating for Proposed Conditions

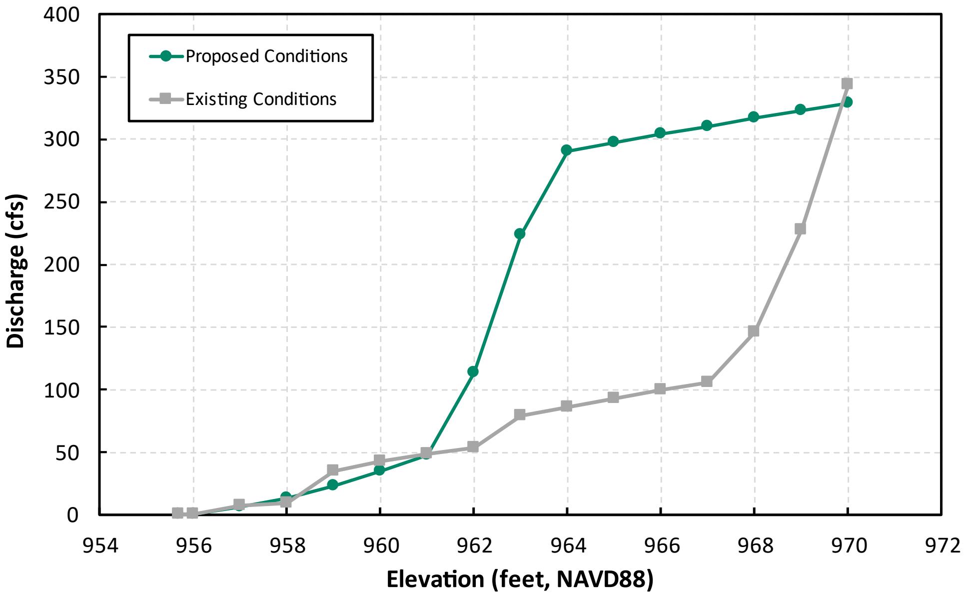

Figure 6: Elevation-Discharge Ratings for Existing and Proposed Principal Spillway

The proposed conditions were modeled using the same hydrologic model that was prepared for the existing conditions, with the revisions noted above. The model was run using the USACE Hydrologic Engineering Center, Hydrologic Modeling System (HEC-HMS), version 4.11 software application. The 4%, 2%, and 1% AEP storm events were modeled along with the SDF. Under proposed conditions, the dam provides adequate freeboard during the modeled storm events. **Table 9** compares results for each of the modeled events under existing and proposed conditions. Supporting model output is provided in **Appendix C** of this report.

Table 9. Comparative HEC-HMS Model Results

		Existing Conditions			Proposed Conditions		
Flood Event	Peak Inflow (cfs)	Peak Reservoir Elevation (feet)	Peak Outflow (cfs)	Freeboard (feet)	Peak Reservoir Elevation (feet)	Peak Outflow (cfs)	Freeboard (feet)
4% AEP (25-Year)	240	959.5	34	9.0	959.7	30	10.3
2% AEP (50-Year)	314	960.3	44	8.2	960.6	40	9.4
1% AEP (100-Year)	397	961.3	50	7.2	961.4	64	8.6
SDF (1/3 PMP)	1,588	969.8	320	-1.3	968.9	322	1.1

4.2 Low-Level Outlet Sizing

Georgia Safe Dams requires that a dam has sufficient capacity to drain two thirds of the normal pool volume in 10 days. At the normal pool elevation of 955.7 feet, the total volume in the reservoir is 30 acre-feet. This means that the proposed low-level outlet system will need to be required to discharge 20 acre-feet, or 864,409 cubic feet, of water in 10 days. The target elevation for this drawdown is approximately 950.2 feet.

The proposed low-level outlet conduit is a 15-inch ASTM C361 Reinforced Concrete Pipe conduit that is approximately 20 feet long and embedded within a concrete cradle. A discharge rating curve was developed for the low-level outlet system and was developed using the guidance from United Stated Bureau of

Reclamation Design of Small Dams (USBR, 1987) Chapter 10 Outlet Works. Based on the guidance in Chapter 10, it is estimated that the proposed low-level outlet system could meet the drawdown requirement in just under 10 hours if fully operated. The entire reservoir can be emptied from normal pool in just under 17 hours if fully operated. It is important to note that AECOM recommends the maximum drawdown rate not exceed 1 foot per day, which would completely drain the reservoir in approximately 5.5 days. See **Figure 7** below for the reservoir drawdown curve under proposed conditions.

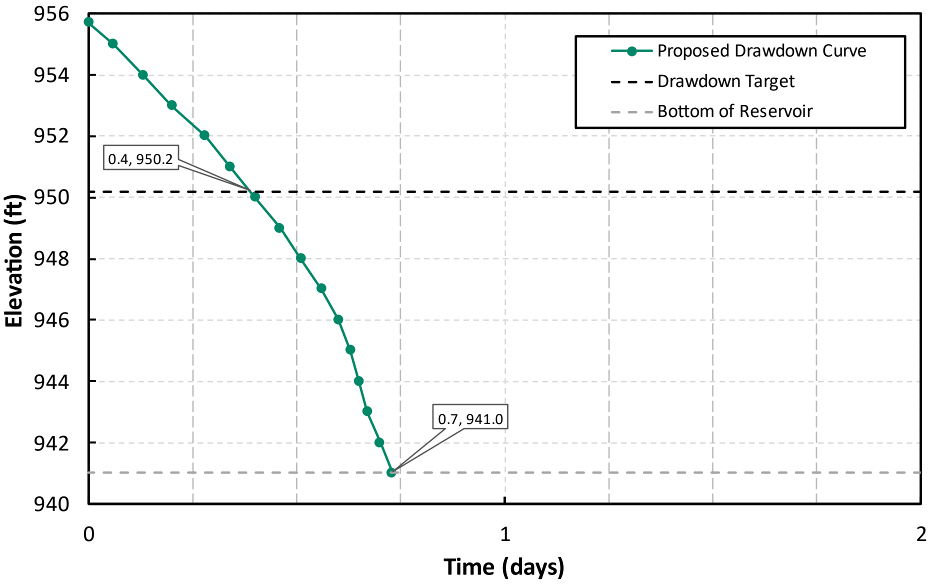

Figure 7: Proposed Drawdown Rating Curve

See **Appendix C** of this report for low-level outlet sizing calculations.

4.3 Impact Basin Sizing

The proposed spillway conduit discharges 322 cubic feet per second at a velocity of 25.7 feet per second at the peak of the SDF event. An energy dissipator is required at the spillway terminus, and therefore, a USBR Type VI impact basin was selected for the proposed design. The impact basin sizing calculations follow the recommended design procedure described in Section 9.4 of HEC-14 (FHWA, 2006). The initial dimensions of the impact basin were selected using Table 9.2 from HEC-14 (FHWA, 2006) but were slightly modified to meet proposed site conditions. The velocity exiting the impact basin was also estimated using the guidance in HEC-14 (FHWA, 2006). It was estimated that the velocity exiting the impact basin was approximately 18 feet per second. When the velocity exiting the impact basin has not been reduced sufficiently, HEC-14 recommends additional energy dissipation measures. To dissipate the velocity further, a riprap lined outlet channel at the end of the impact basin is proposed to convey flow from the impact basin to the downstream reach in a safe and non-erosive manner. Design of that channel is discussed in the following section.

See **Appendix C** of this report for impact basin sizing calculations.

4.4 Outlet Channel and Riprap Apron

As identified in the previous section, the impact basin alone is not sufficient to dissipate the energy to a non-erosive condition. To confirm that the velocity of the water exiting the impact basin returns to a velocity that is non-erodible and matches the existing conditions, a riprap lined channel that extends 50 feet beyond the impact basin is proposed. This riprap apron was sized using HEC-14 (FHWA, 2006) guidance for riprap aprons after energy dissipators in Section 10.3. The velocity exiting the impact basin is estimated to be approximately 18.3 feet per second. The Georgia Department of Natural Resources Flood Map Viewer allows the download of existing hydraulic models for river systems. A USACE Hydrologic Engineering

Center, River Analysis System (HEC-RAS) model was downloaded from this site that included the Lake Erin watershed. This model was slightly edited to have a boundary condition equal to the peak discharge during the SDF event. The resulting channel velocity for this HEC-RAS simulation was approximately 8 to 9 feet per second. The target velocity downstream of the proposed energy dissipation system shall be 8 feet per second as to not increase the velocity in the channel for the proposed conditions. The estimated required length for the riprap apron to decrease the velocity to 8 to 9 feet per second was 50 feet. Based on the results, a conservative D_{50} of 12 inches was selected for the outlet channel, which was larger than the minimum D_{50} required estimated by HEC-14. To confirm that this riprap apron dissipates the energy appropriately, a two-dimensional HEC-RAS model was developed for the outlet system under proposed conditions. The results of the two-dimensional model estimate that the velocity exiting the site is just under 5 feet per second, which is less than the target velocity.

See **Appendix C** of this report for riprap apron sizing calculations.

4.5 Wave Height Analysis

In the previous H&H analysis (AECOM, 2021), a wave height analysis was performed using methodologies described in NRCS Technical Release 56 (NRCS, 2014). SDP requires a minimum overland wind velocity of 50 miles per hours (mph) when calculating wave height. The overland wind velocity used in the 2021 analysis was 85 mph which resulted in an estimated significant wave height of 0.8 feet. After performing the wave height calculations with two additional trial locations, the significant wave height was determined to be 0.9 feet. This wave height is added to the maximum reservoir surface elevation (Elevation 968.9) during the design storm event to determine the minimum top of dam. Based on this analysis, the minimum top of dam elevation would be Elevation 969.8. Under proposed conditions, the dam crest is at Elevation 970.0 feet which provides a freeboard of 1.1 feet during the SDF.

The limits of the upstream riprap protection were determined based on the guidance presented in this document which aim to protect the area of the upstream slope that will be impacted by the design wave throughout the dam’s service life. The upper and lower limits of the upstream riprap protection are generally a function of the over water wind velocity (87 mph) and the significant wave height (0.9 feet). A conservative lower elevation of protection was selected to account for periods of draught that could cause the normal pool to be lowered for an extended period of time. The upstream riprap protection was designed with a lower elevation of protection equal to Elevation 952.0 and the upper elevation of protection equal to Elevation 958.0.

See **Appendix C** of this report for the wave height analysis calculations which include the three trial fetch locations and the determination of riprap protection limits.

5. Proposed Conditions Structural Analysis

5.1 Overview

This section describes the general technical approach and design assumptions for the structural design of the Lake Erin Dam Rehabilitation project structural features, including the new intake tower, conduits, and energy dissipation structure. The structural analyses were performed to design the structures to meet strength and stability requirements in accordance with the applicable industry standards and guidelines.

Supporting calculations for the design of the structural features are provided in **Appendix D** of this report.

5.2 Industry Codes and Standards References

The structural design of the reinforced concrete structures was performed in general accordance with the ACI 350-20, “*Code Requirements for Environmental Engineering Concrete Structures*” (ACI, 2020). The external stability of the structures was designed in accordance with criteria from the USACE, EM 1110-2-2100 “*Stability Analysis of Concrete Structures”* (USACE, 2005). Additional design references that were used to supplement the design include:

- EM 1110-2-2400 "*Structural Design and Evaluation of Outlet Works*" (2003)
- ASCE 7-22 "*Minimum Design Loads for Buildings and Other Structures*" (2022)

5.3 Material Properties

The material properties used in the stability and strength computations for the reinforced concrete structures are shown in **Table 10**. The concrete material properties were specified based on the durability requirements for water retaining structures in accordance with ACI 350. The structures are expected to be founded on weathered rock or soil (subgrade to be approved in field by Geotechnical Engineer) and the backfill placed on the earth retaining side of the structures will also be embankment material. Refer to Section 3.0 for detail descriptions of the soil properties and foundation conditions.

Table 10. Summary of Material Properties for Structural Analysis

Properties	Values
Concrete	
Unconfined Compressive Strength	5,000 lb/in^2
Unit Weight	150 lb/ft^3
Steel Reinforcement	
Yield Strength	60,000 lb/in^2
Water	
Unit Weight	62.4 lb/ft^3
Embankment/Structural Fill	
Moist Soil Unit Weight	123 lb/ft^3
Saturated Soil Unit Weight	128 lb/ft^3
Internal Friction Angle	31 Degrees
Foundation (Soil)	
Allowable Bearing Strength – Tower	3,500 lb/ft^2
Allowable Bearing Strength – Basin	5,000 lb/ft^2
Internal Friction Angle	31 degrees
Saturated Soil Unit Weight	128 lb/ft^3

5.4 Loads and Load Combinations

The structures were evaluated for the usual, unusual, and extreme loading conditions. The usual loading condition included gravity, backfill material, reservoir at normal water surface (including corresponding phreatic surface/groundwater and tailwater), uplift loads, and live load (where applicable). The unusual loading condition included gravity, backfill material, reservoir at the spillway design flood (including elevated phreatic surface/groundwater or tailwater), and uplift loads. The extreme load condition includes the usual loading conditions and the effects of ground motions due to an earthquake. The individual loads included in the load combinations are given in **Table 11** below.

The maximum design earthquake (MDE) for the structural analysis is the earthquake that has a 2-percent chance of exceedance in 50 years (also referred to as the 2,475-return period) in accordance with Georgia Rul 391-3-8.09 Standards for the Design of Dams. The peak ground accelerations used in the analysis were obtained from the ASCE Hazard Tool. A site-specific seismic hazard analysis was not performed nor warranted for this site due to the relatively low peak ground accelerations.

Table 11. Summary of Individual Loads

Load	Description
Gravity/Dead	Vertical Loads from the self-weight of concrete, soil, and reservoir water were based on material unit weights.
Live	A uniformly distributed live load of 250 psf was assumed for the intake tower service deck only.
Water	Lateral hydrostatic loads from the reservoir, phreatic surface/groundwater, and tailwater, were estimated assuming standard triangular pressure distributions.
Uplift	Hydrostatic water pressure applied to the bottom of the structure to simulate the hydrostatic (uplift) pressures.
Earth Pressures	Lateral earth pressure corresponding to the material unit weight with an “at rest” pressure state based on Rankine’s approach. Dynamic earth pressures were in accordance due to seismic ground accelerations with Mononobe-Okabe pseudo static approach as described in USACE guidelines.
Earthquake	Inertia loads due to the maximum design earthquake, a 2,475-year event with a PGA of 0.12g (ASCE Hazard Tool, Soil Class D, Risk Category IV).
Dam/Reservoir Dynamic Interaction	Added pressure to simulate the dam/reservoir interaction during the earthquake based on USACE EM 1110-2-2400.
Wind	Winds loads on the face of the structure that produce that greatest force (ASCE 7-22). Wind loads are only evaluated for the construction load condition and not expected to control for the post-construction load conditions.

5.5 Load Conditions

The load conditions that spillway structures may encounter through service life are grouped into major categories of Usual, Unusual, and Extreme load combinations according to USACE’s EM 1110-2-2100 (USACE, 2005). Certain load combinations apply to stability evaluation only; others to structural design only; while some apply to both stability and strength design, as described below.

Intake Tower (labels based on USACE 2003, Section 3-4):

- **Usual Loading Condition (U1)** – Normal pool (EL 955.7), uplift, dead load, water surface inside structure at normal pool, deck slab live load of 250 psf.
- **Usual Loading Condition (U3)** – Normal pool (EL 955.7), uplift, dead load, no water inside structure, deck slab live load of 250 psf.
- **Unusual Loading Condition (UN4)** – Construction loading condition with reservoir empty, dead load, wind load.
- **Extreme Loading Condition (ED1)** – Seismic loading condition with U3 plus MDE.
- **Extreme Loading Condition (ED2A)** – Flood loading condition with reservoir at 1/3 PMF elevation (EL 968.9), dead load, uplift, water surface inside structure to top of slab.
- **Extreme Loading Condition (ED2B)** – Flood loading condition with reservoir at 1/3 PMF elevation (EL 968.9), dead load, uplift, no water inside structure.

Impact Basin:

- **Unusual Loading Condition (1/3 PMF)** - Maximum tailwater (EL 940.1), uplift, dead load, backfill, maximum discharge (thrust).
- **Unusual Loading Condition (Floodwater Recedes)** - Normal tailwater (EL 939.0), uplift, dead load, fully saturated backfill.

5.6 Evaluation Criteria

Two-dimensional stability and strength evaluations were performed for the different structural features of this project. The structural stability evaluations are based on USACE guidelines and are shown in **Table 12**.

Table 12. Summary of Stability Criteria for Sliding, Overturning, Flotation, and Bearing

Load Combination	Sliding Criteria (Factor of Safety)	Overturning Criteria (Resultant Location)	Flotation Criteria (Factor of Safety)	Bearing Criteria (Allowable)
Usual	1.5	Middle 1/3	1.3	≤ allowable
Unusual	1.3	Middle 1/2	1.2	≤ 1.15 * allowable
Extreme	1.1	Within Base	1.1	≤ 1.5 * allowable

Source: EM 1110-2-2100, Sections 3.7-3.10

The load factors that will be used in the strength design are based on ACI 350-20 and are shown in **Table 13**.

Table 13. LRFD Load and Reduction Factors

Load Combination	Load Factors	Flexural Reduction Factor	Shear Reduction Factor
Usual	1.2(D+F) + 1.6(L+H)	0.9	0.75
Unusual	1.2(D+F) + 1.6(L+H)	0.9	0.75
Extreme	1.2(D+F) + 1.6H + 1.0E	0.9	0.75

Source: ACI 350-20, Section 9.2.1

5.7 Method of Analysis

Each of the reinforced concrete structures were analyzed using basic structural analysis methods and equivalent static loads. The limit equilibrium analysis was to evaluate the external stability of the intake tower and impact basin. The reinforced concrete sections were evaluated for shear and flexural capacity assuming a unit width (1-foot) strip of the walls and slab to determine the required thickness and reinforcement required in accordance with ACI 350-20. A summary of the specific analysis methods for each of the components of the structures are below:

Intake Tower

The intake tower structure is a reinforced concrete free-standing tower that is the spillway, a drop structure, and houses the gate for the low-level lake drain.

- The external stability of the intake tower was analyzed using basic structural analysis methods and equivalent static loads.
- For the extreme load combination, the intake tower was analyzed using linear-elastic response spectra modal using the two-mode approximate method in accordance with USACE 1110-2-2400 Appendix C.

- The intake tower horizontal reinforcement in the walls was designed to resist the usual and unusual load conditions. The intake tower vertical reinforcement in the walls was designed to resist vertical bending from the overturning moment during the extreme (earthquake) load conditions.
- The intake tower service deck (top slab) was designed to withstand a 250 psf live load, and the gate operating loads from the sluice gate operator. The top slab was designed as a simply-supported one-way slab.

Impact Basin

- The external stability of the impact basin was evaluated using basic structural analysis methods and equivalent static loads.
- The reinforced concrete strength was evaluated on an individual component basis:
  - Headwall vertical reinforcement was analyzed with the assumption that the wall behaves as a cantilevered wall. Horizontal reinforcement was analyzed with the assumption that the wall behaves as a simply-supported beam spanning between the sidewalls.
  - Sidewall reinforcement was analyzed with the assumption that the wall behaves as a cantilevered wall.
  - Baffle horizontal reinforcement was analyzed with the assumption that the baffle behaves as a fixed-fixed beam spanning between the sidewalls and the water force was conservatively approximated as a point load. Vertical reinforcement was analyzed with the assumption that the baffle behaves as a fixed cantilever and the water force is a partially distributed load.
  - The base slab was analyzed as a one-way slab spanning between the sidewalls.
  - The wingwalls were analyzed as cantilevered walls.

Conduit Encasement

- The bearing stability of the conduit encasement was evaluated; however, sliding and overturning of the concrete encasement is not a viable failure mode due to the conduit being buried in the embankment.
- The entire vertical load of the embankment above the conduit assuming saturated fill was used to calculate the maximum moment and shear within the structure based on Beggs Deformeter Stress Analysis, Shape D.

5.8 Analysis Results

The results of the impact basin and intake tower stability analysis as presented below in **Table 14** and **Table 15**, and **Table 16** and **Table 17**, respectively. The USACE requirements for stability are met for both structures for all the load combinations. The bearing pressures of the structures on its supporting soil foundations are within allowable values. The results of the reinforced concrete strength for all components of all structures met all ACI 350-20 requirements for shear and moment strength for all load combinations.

Table 14. Impact Basin Stability Analysis Results – Sliding and Flotation

Load Combination	Sliding Criteria (FOS)		Flotation Criteria (FOS)	
	Required	Analysis	Required	Analysis
Unusual – 1/3 PMF	1.3	3.0	1.2	5.0
Unusual – Floodwater Recedes	1.3	1.4	1.2	2.2

Source: EM 1110-2-2100, Sections 3.7-3.10

Table 15. Impact Basin Stability Analysis Results – Overturning and Bearing

Load Combination	Overturning Criteria (Resultant Location)		Bearing Criteria (psf)	
	Required	Analysis	Required	Analysis
Unusual – 1/3 PMF	Middle 1/3	Middle 1/3	5,750	1,076
Unusual – Floodwater Recedes	Middle 1/3	Middle 1/3	5,750	1,096

Source: EM 1110-2-2100, Sections 3.7-3.10

Table 16. Intake Tower Stability Analysis Results – Sliding and Flotation

Load Combination	Sliding Criteria (FOS)		Flotation Criteria (FOS)	
	Required	Analysis	Required	Analysis
Usual – Water Inside (U1)	1.7	10.6	1.3	3.5
Usual – No Water Inside (U3)	1.7	9.3	1.3	3.1
Unusual – Construction (UN4)	1.3	11.9	N/A	N/A
Extreme – Seismic (ED1)	1.1	8.8	1.1	3.1
Extreme – Flood (ED2A)	1.1	8.3	1.1	2.2
Extreme – Flood (ED2B)	1.1	6.2	1.1	1.8

Source: EM 1110-2-2100, Sections 3.7-3.10

Table 17. Intake Tower Stability Analysis Results – Overturning and Bearing

Load Combination	Overturning Criteria (Resultant Location)		Bearing Criteria (psf)	
	Required	Analysis	Required	Analysis
Usual – Water Inside (U1)	Middle 1/3	Middle 1/3	3,500	1,564
Usual – No Water Inside (U3)	Middle 1/3	Middle 1/3	3,500	1,368
Unusual – Construction (UN4)	Middle 1/4	Middle 1/3	4,025	1,683
Extreme – Seismic (ED1)	Within Base	Middle 1/3	5,250	1,559
Extreme – Flood (ED2A)	Within Base	Middle 1/3	5,250	1,219
Extreme – Flood (ED2B)	Within Base	Middle 1/3	5,250	919

Source: EM 1110-2-2100, Sections 3.7-3.10

6. Construction Considerations

Construction of the repairs will require considerations to manage surface and ground water, avoid disruption to utility services, maintain continued operation of Henderson Park, and maintain the integrity of the dam during the work. The following sections describe these considerations.

6.1 Water Management

During construction, both surface and groundwater sources will need to be managed by the contractor to keep the work area dry and conducive for construction of the repairs. The primary source of surface water is the baseflow entering the reservoir from the surrounding watershed. During precipitation events, surface water flows would increase. Therefore, a cofferdam system and temporary bypass conduits will be installed to control water throughout construction. The reservoir should be fully dewatered to elevation 941.0 feet and maintained in a dewatered condition throughout construction. The proposed cofferdam system is 10 feet high and has a crest elevation of 951.0 feet.

The proposed temporary bypass system will involve of three phases which are incorporated into the overall phasing of the project. The first phase (starting in Phase 2 of the overall project) utilizes two 48-inch HDPE-S conduits to discharge flood events up to the 1% AEP event. The Phase 2 configuration will be the primary bypass system used during construction. The second phase (Phase 3A) consists of one 48-inch HPDE-S conduit that is connected the newly constructed intake tower. The third phase (Phase 3B) consists of one 15-inch HDPE-S conduits that is connected to the newly constructed low-level outlet conduit. Phases 3A and 3B will take place after most of the proposed features are installed during the time when the embankment excavation is being backfilled.

Assuming a maximum dry-weather conditions pool of elevation 941.0 feet, during Phase 2 the peak water surface elevation for the 1% AEP, 6-hour duration event is estimated to be 950.1 feet. Therefore, during the Phase 2 construction conditions, the existing reservoir is anticipated to be able to fully capture and release the runoff from the 1% AEP, 6-hour duration event with the proposed Phase 2 bypass system. At this estimated peak water surface elevation, the reservoir would not encroach on the proposed limits of disturbance. However, the contractor would be expected to take measures (operating a siphon, using pumps, or other means as outlined in their reservoir control plan to be approved by the engineer of record) to dewater the reservoir if the reservoir was expected to rise above the cofferdam system. Under the same dry conditions, the Phase 3A peak water surface elevation for the 4% AEP, 6-hour duration event is estimated to be 950.2 feet. Therefore, during the Phase 3A construction conditions, the existing reservoir is anticipated to be able to fully capture and release the runoff from the 4% AEP, 6-hour duration event with the proposed Phase 3A bypass system. The Phase 3B temporary bypass system does not provide capacity to pass the 4% AEP, 6-hour duration event. During Phase 3B while the embankment is being backfilled, the contractor will be responsible for providing any additional bypass or dewatering capacity required.

A hydrologic model was developed to route the 4% AEP (25-year), 2% AEP (50-year), and the 1% AEP (100-year) storm events through each proposed temporary bypass phase. See **Appendix C** of this report for temporary by-pass conduit sizing calculations.

If during the excavation into the embankment, groundwater, both from the foundation of the dam and from seepage through the dam is encountered, dewatering will be provided as needed using a pump and sediment filter tank. If this is deemed insufficient to dewater the incoming flows, a dewatering system will need to be designed and installed by the contractor as approved by the engineer of record to adequately dewater work. Work areas will be dewatered to an elevation two feet below the lowest excavated elevation or until suitable subgrade conditions for performing contract work are achieved.

6.2 Dam Safety

The repairs proposed herein are intended to address deficiencies that affect the safety of the dam. However, it is important to point out that a dam is at highest risk of failure during construction of those repairs. Therefore, plans and specifications for the repairs will be designed in a way that minimizes potential failure risk during and following construction. Examples of this include but are not limited to:

- Prohibition of the use of trench boxes and other vertical excavation methods and requirement of 2-horizontal-to-1-vertical slope stepped laybacks on all excavations to allow for key-in of backfill material;
- Requirement of controlled fill with compaction testing of all lifts of all backfill materials;
- Continued maintenance of the reservoir at Elevation 941.0 feet until the dam repairs are completed;
- Use of construction dewatering methods to control/eliminate groundwater infiltration into critical excavations at the toe of the dam;
- Oversight of the work by the engineer of record as well as technical staff working under his or her responsible charge to confirm the repairs are installed in general accordance with the plans and specifications.

6.3 Property Impacts

In order to construct and maintain the proposed rehabilitation of Lake Erin Dam, one private property impact is anticipated requiring both a temporary and permanent easement. The temporary and permanent easements are identified for the Hathaway property, Parcel ID 18 251 04 024. A temporary construction easement, approximately 3,905 square feet, is required to access and complete the proposed grading at the downstream toe of the dam and at the proposed outlet channel to establish tie-ins with existing ground. The permanent easement is necessary for the installation and maintenance of both the impact basin and the riprap lined channel immediately downstream of the impact basin. However, the proposed impact basin structure shall be located entirely on City property. The permanent easement is approximately 4,500 square feet. The City of Tucker is procuring the easement documentation required to construct the proposed rehabilitation measures for Lake Erin Dam.

6.4 Underground Utility Impacts

No underground utility impacts are anticipated with the construction of the proposed improvements of Lake Erin Dam. However, there is an existing sanitary sewer line at the northeast end of the project site, which runs parallel to the length of the reservoir. No excavation is anticipated within the limits of the utility. The existing ground above the utility will need to be traversed to complete the construction of the proposed improvements. The contractor shall be responsible for protecting all underground utilities on site.

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Appendix A - Previous AECOM Investigations

Erin Lake Dam

State ID #044-004-00033
NID #GA01324

Hydrologic and Hydraulic Analysis Report

Prepared for:

DeKalb County, Georgia
Department of Public Works
Roads and Drainage Division
729 Camp Road
Decatur, Georgia 30032

Prepared by:

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1360 Peachtree Street, NE
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May 2021

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1. Introduction and Objectives

1.1 Project Introduction

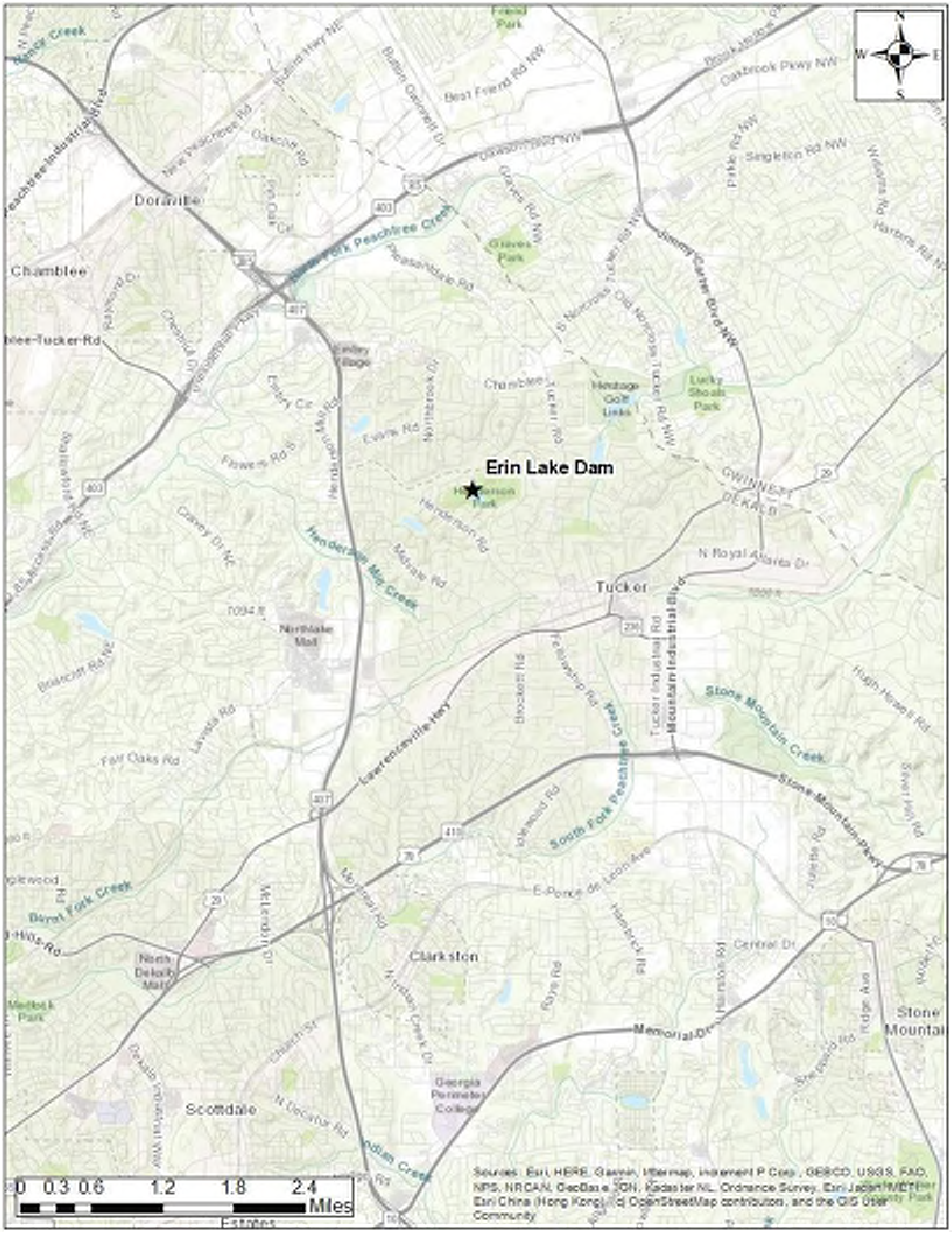
Erin Lake Dam is an earthen dam located in Henderson Park at DeKalb County, Georgia on a tributary to North Fork Peachtree Creek. The location of the dam is shown in Figure 1. The dam is currently maintained by the DeKalb County Department of Public Works, Roads and Drainage Division and its primary function is to serve for recreation purposes. An aerial photography dating back to 1955 confirms that the structure is at least 65 years old.

Erin Lake Dam is regulated by the Georgia Department of Natural Resources, Environmental Protection Division, Safe Dams Program (SDP). The structure is 27.5 feet high and 357.7 feet long with an approximately 19 feet wide crest. Based on the characteristics of the dam, SDP has categorized it as a Category 1, Medium Dam. A Category 1 dam is classified as a high hazard dam, the improper operation or failure would result in a probable loss of human life. A Medium Dam is classified as a dam that impounds more than 500 acre-feet but not more than 1,000 acre-feet, or has a height exceeding 25 feet but not exceeding 35 feet.

The dam crest has an uneven profile with its elevation ranging from approximately 968.5 feet to 969.4 feet, measured with reference to North American Vertical Datum of 1988 (NAVD 88). All elevations presented in this report are with reference to NAVD 88. The dam has a principal spillway and an auxiliary spillway. Both spillways are located at the left abutment of the dam. The principal spillway consists of a 30-inch diameter corrugated metal pipe (CMP) with an invert elevation of 955.70 feet which sets the normal pool elevation, a 30-inch diameter CMP riser structure with a crest elevation of 962.12 feet, and a 30-inch diameter reinforced concrete pipe (RCP) that drains into the masonry-lined auxiliary spillway channel at an invert elevation of 954.19 feet. The total length of the RCP and the CMP which comprise the principal spillway is 94.95 feet. The auxiliary spillway is an open channel spillway with a vertical training wall on the right side (looking downstream) and an earthen side slope on the left side with an approximate slope of 2.2H:1V. The channel is constructed from stone masonry and has a bedrock base. Its bottom width and level section length are approximately equal to 12.0 feet. The spillway crest elevation is approximately elevation 967.0 feet.

1.2 Objectives

The objectives of this report are to document the results of Hydrologic and Hydraulic (H&H) analysis for Erin Lake Dam and compare the results against SDP Category I criteria. The SDP Category I dam criteria as published in Engineer Guidelines, Version 4.0 (Georgia Department of Natural Resources, 2015) are used to evaluate the existing dam conditions and make recommendations for improvements to correct any non-compliance issues.



Sources: Esri, HERE, Garmin, Intermap, increment P Corp., GEBCO, USGS, FAO, NPS, NRCAN, GeoBase, IGN, Kadaster NL, Ordnance Survey, Esri Japan, METI, Esri China (Hong Kong), (c) OpenStreetMap contributors, and the GIS User Community

Figure 1. Vicinity map.

2. Hydrologic Analysis

2.1 Precipitation Analysis

A precipitation analysis was completed to define the required precipitation events against which Erin Lake Dam should be evaluated. Two events were analyzed: the 50-year event and spillway design flood (SDF). First, the dam must be able to pass a 50-year recurrence interval event of 6-hour duration through the principal spillway without activating the auxiliary spillway. Second, the dam must be able to safely pass the design storm or spillway design flood (SDF) commensurate with its classification with the required freeboard. For Erin Lake Dam, this event is defined as 33.3 percent of the Probable Maximum Precipitation (PMP). Both the 6- and 24-hour duration PMP events were evaluated to determine which duration generated the critical SDF. The 24-hour duration generated higher peak inflow and outflow discharges, larger runoff volumes, and a higher resultant peak water surface elevation in the reservoir and was therefore selected as the duration for the SDF.

The 50-year event was determined using the National Oceanic and Atmospheric Administration's (NOAA) Precipitation Frequency Data Server which provides Atlas 14 precipitation data to determine the precipitation depth for a six-hour duration event. Guidance for precipitation distribution was obtained from the Natural Resources Conservation Service (NRCS) who has developed regional precipitation distributions compatible with Atlas 14 precipitation data (Merkel and Moody, 2015). The recommended distribution for DeKalb County, Georgia used for this event is the MSE5 distribution.

PMP estimates represent an upper limit to the level of precipitation that the atmosphere can produce at a particular geographic location during a certain time of the year. A hydrologic model is used to simulate the PMP over the study watershed and transform the excess precipitation into the surface runoff that must be safely conveyed through a dam per the SDP standards. PMP estimates for the Erin Lake Dam watershed were obtained from the National Oceanic and Atmospheric Administration's (NOAA) Hydrometeorological Report No. 51: "*Probable Maximum Precipitation Estimates, United States East of the 105th Meridian*" (HMR51 – NOAA, 1978). Using precipitation maps for storm areas of 10 square miles for the DeKalb County area, the 24-hour duration PMP precipitation depth was determined to be 41.86 inches. Typically, distributions of PMP are developed using NOAA's Hydrometeorological Report No. 52: "*Application of Probable Maximum Precipitation Estimates East of the 105th Meridian*" (HMR52 – NOAA, 1982). However, due to the size of the watershed for this dam (0.51 square miles) and the fact that the SDF is 33.3% of the PMP, it was determined that a formal HMR52 analysis was not appropriate for a watershed of such limited size relative to the datasets and procedures outlined in HMR51 and HMR52. However, a suitable precipitation distribution was required that reflected similar rainfall intensities to those developed using the HMR52 procedure. The NRCS MSE5, NRCS Dimensionless Freeboard Storm Hydrograph (NRCS, 2019), and the NRCS Type II (NRCS, 1973) distributions were analyzed. The NRCS Type II distribution was selected as it most closely matched the HMR52 precipitation distribution pattern and peak one-hour storm intensity.

A summary of the precipitation events modeled are provided in Table 1. Calculations and figures related to development of the precipitation events are in Appendix B.

Table 1. Precipitation Event Information

Event	Duration (Hours)	Precipitation Depth (in.)	Precipitation Distribution
50-year	6	4.57	NRCS MSE5
33.3% PMP	24	13.95	NRCS Type II

2.2 Hydrologic Parameter Development

Hydrologic model parameters were developed for the Erin Lake Dam watershed. These parameters included the watershed area, loss parameters, the unit hydrograph, and the transform parameter.

The boundary for the Erin Lake Dam watershed was delineated via the United States Geological Survey (USGS) StreamStats tool. The watershed area was estimated to be approximately 324.9 acres (0.51 square miles). Since the drainage patterns and land uses within the watershed are generally consistent throughout, the watershed was not subdivided.

For the Erin Lake Dam hydrologic model, rainfall losses were computed using NRCS' Runoff Curve Number method. The Runoff Curve Number (CN) is an empirical parameter that provides an indication of storm runoff potential over an area based on land cover, underlying soil type, and hydrologic condition. Higher CN values indicate a quicker watershed response time and an increase in runoff. In addition to the CN, the Runoff Curve Number method uses an initial abstraction (I_a) value to represent the total rainfall lost before runoff initiates, including losses from interception, initial infiltration, surface depression storage, and evapotranspiration. The I_a was calculated using Formula 1 found in NRCS methodology:

$$I_a = 0.2\left(\frac{1000}{CN} - 10\right) \qquad (1)$$

Development of the CN parameter requires assessment of the land use and hydrologic soil group distribution of a watershed. Land use data for this watershed was acquired from LandPRO 2010 which was created by the Research and Analytics Division of the Atlanta Regional Commission. The land use data from 2010 was assumed to be accurate because of the assumption that the land use change since 2010 has been minimal which would not induce significant changes in stormwater runoff. The data shows that the majority of the watershed is low and medium density residential usage. This land use map can be seen in Appendix C.

Soils information used to determine the hydrologic soil group (HSG) distribution in the watershed was acquired from WebSoil Survey which was created by the United States Department of Agriculture (USDA) using SSURGO (Soil Survey Geographic Database) soils data sets. This soils data was last revised in 2019.

The CNs were developed from hydrologic soil group and land use maps using the following methodology:

1. Land use from the LandPro 2010 data set was classified using NRCS guidance.
2. The HSG for each soil type was extracted from the soils dataset.
3. The soil and land use data were combined within ArcGIS and the resulting dataset clipped to the watershed.
4. A custom CN lookup table was developed by determining a relationship between the land use data and the HSG using values published by the NRCS for antecedent moisture condition (AMC) II to assign curve number values. This table can be found in Appendix C.
5. An area-weighted CN was calculated for the watershed by cross-referencing the soil and land use data with the lookup table values.

A composite CN of 66.3 was estimated for the entire watershed.

To convert excess precipitation into surface runoff, the NRCS, formerly Soil Conservation Service (SCS) Unit Hydrograph Transform Method was employed within the watershed model. The inputs for this method include graph type and a lag time. The Standard graph type with peak rate factor of 484 was selected for this analysis based on the prevailing topography of the watershed and surrounding lands. Lag is the delay between the time runoff from a rainfall event over a watershed begins and the time at which runoff reaches its maximum peak (NRCS, 2010). Graphically, lag is represented by the time differential between the center of mass of the excess rainfall and the peak of the resulting runoff hydrograph.

The lag time for the watershed was calculated using the SCS method represented by the following formula:

$$L = \frac{l^{0.8}(S+1)^{0.7}}{1900Y^{0.5}} \qquad (2)$$

where:

L = Lag time (minutes)
l = flow length (feet)
Y = average watershed slope (%)
S = maximum potential retention (inches) defined by (1000/CN) – 10

Using the values determined for the watershed and defined in Table 2, the lag time is estimated to be 47 minutes. The watershed map showing the drainage area boundary and the longest flow path can be found in Appendix C.

Table 2. Watershed Lag Time

Longest Flow Path - l (feet.)	Maximum Potential Retention Rate – S (in.)	Average Watershed Slope – Y (%)	Lag Time - *L* (min.)
6,055	5.08	6.39	47

2.3 Reservoir Ratings

Stage-storage and stage-discharge ratings were developed for Erin Lake Dam using a topographic and bathymetric survey of the dam and reservoir completed by Accura Engineering and Consulting Services in March 2021. A topographic and bathymetric survey map is provided in Appendix A.

The bathymetric survey aided the development of the stage storage curve for Erin Lake Dam. The contours from the survey as well as those contained in the 2011 LiDAR data for DeKalb County were used to develop the stage-storage rating provided in Figure 2 below. Related calculations and tabular ratings can be found in Appendix D.

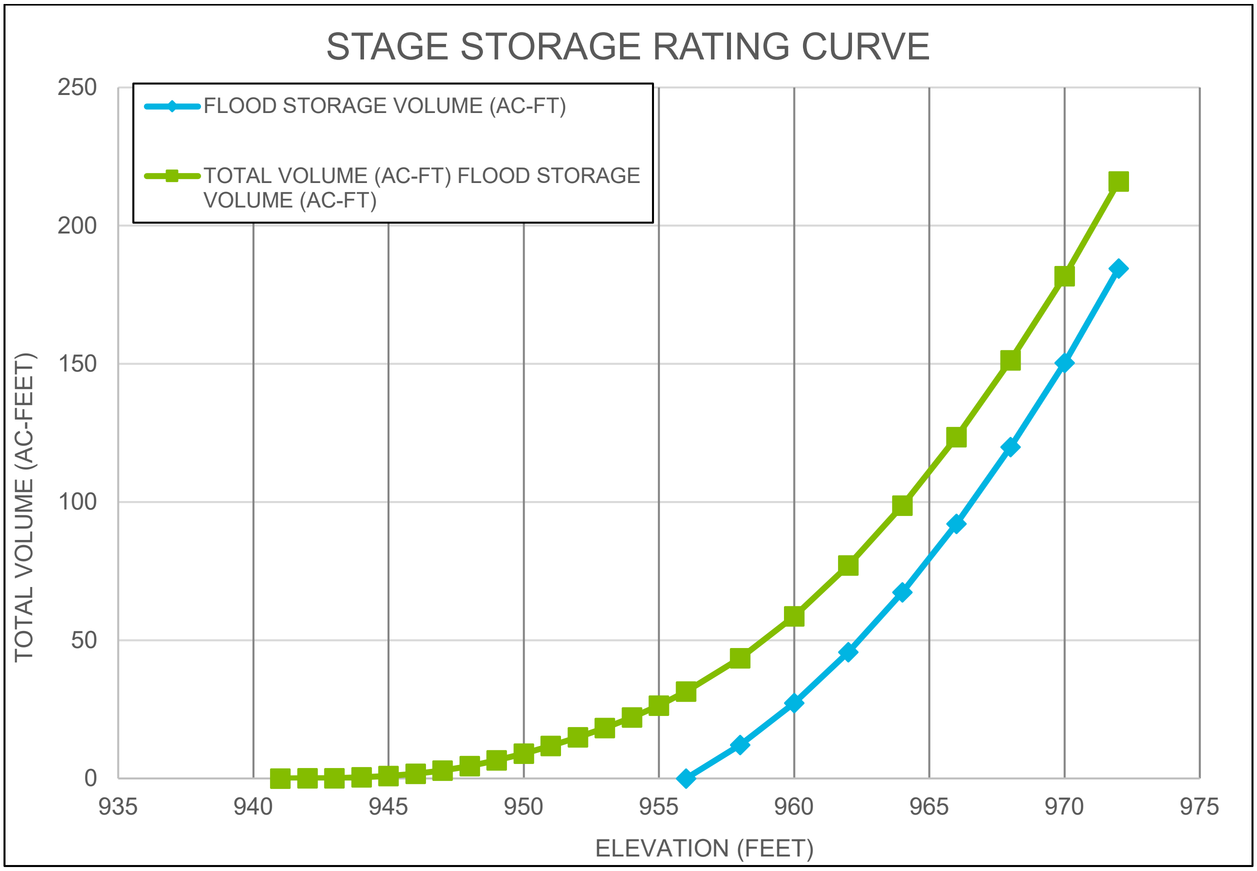


Figure 2. Stage-storage rating curve.

The stage discharge rating curve was developed by comparing a series of ratings for each of the dam's spillway components. The principal spillway was evaluated for low flow orifice, free weir, submerged weir, weir as orifice, and barrel/conduit control to determine a composite rating. The composite rating indicates that the principal spillway is controlled by its inlet at elevation (EL.) 955.70 feet and operates in conduit control. It operates in low flow orifice control between EL. 958.00 feet and EL. 962.12 feet (the crest of the riser pipe) before transitioning to conduit control again. The auxiliary spillway discharge rating was developed by modeling the spillway as an open channel using gradually varied flow regime modeling procedures defined in *Open Channel Hydraulics* (Chow, 1959). The principal and auxiliary spillway ratings were added together to create the overall stage-discharge rating for the dam. The total discharge and individual discharge components are shown in Figures 3 and 4. Related calculations and tabular ratings can be found in Appendix D.

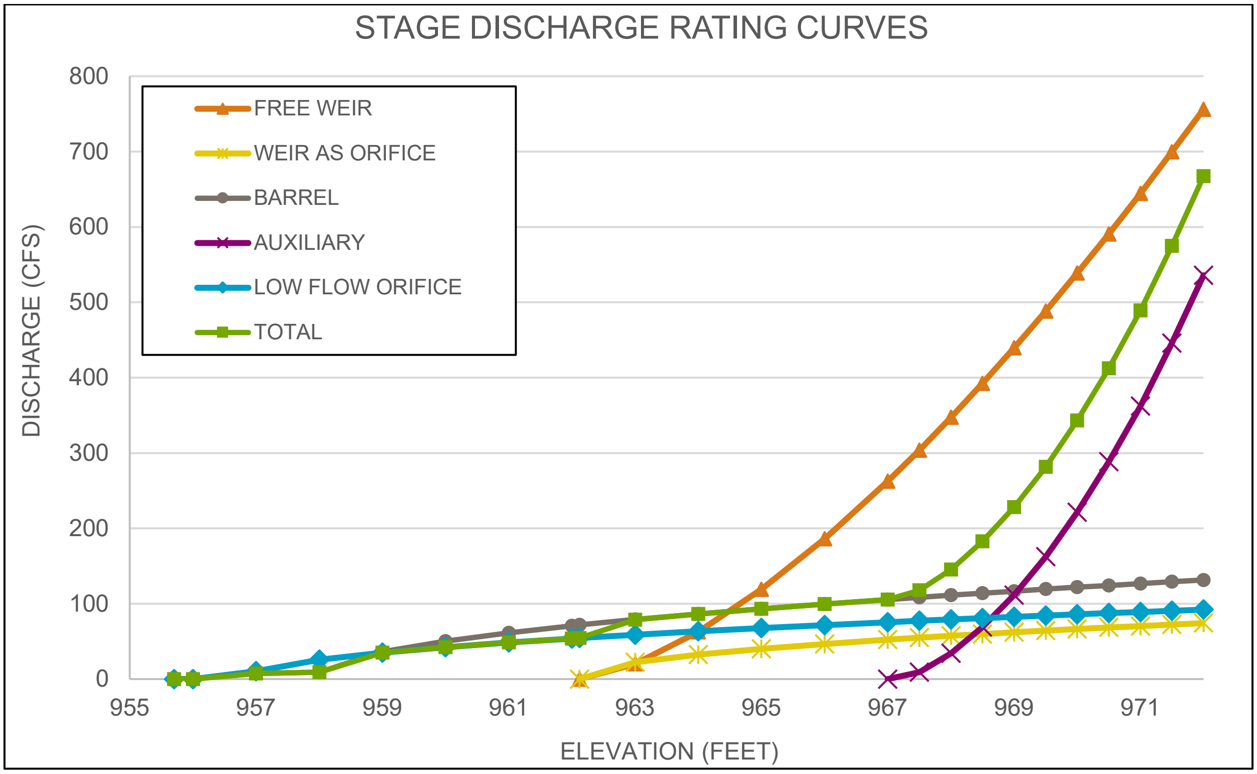


Figure 3. Stage-discharge rating curves.

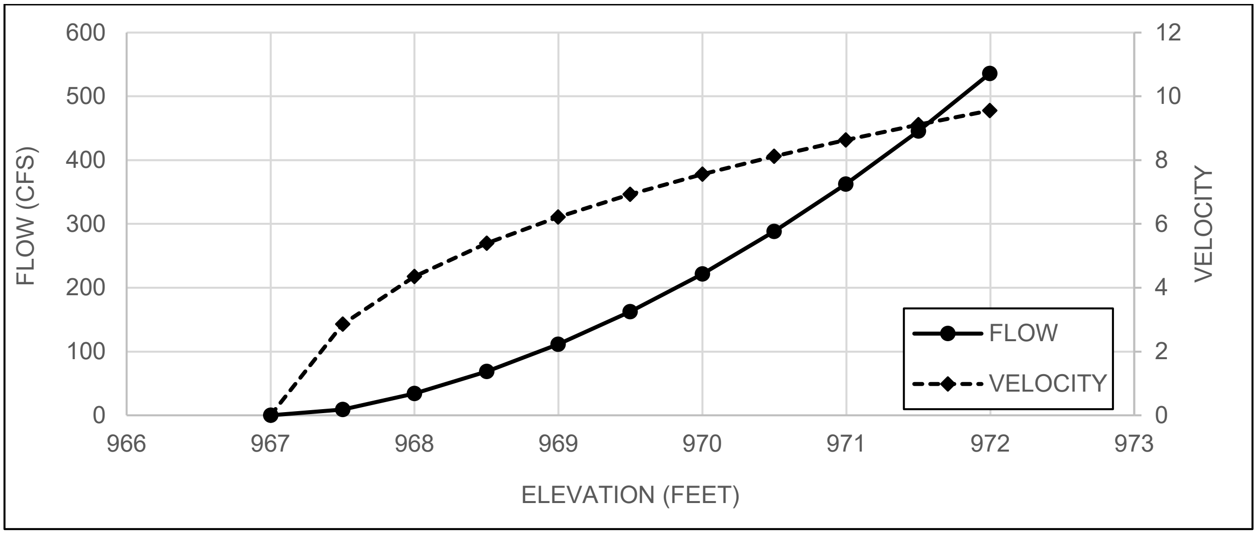
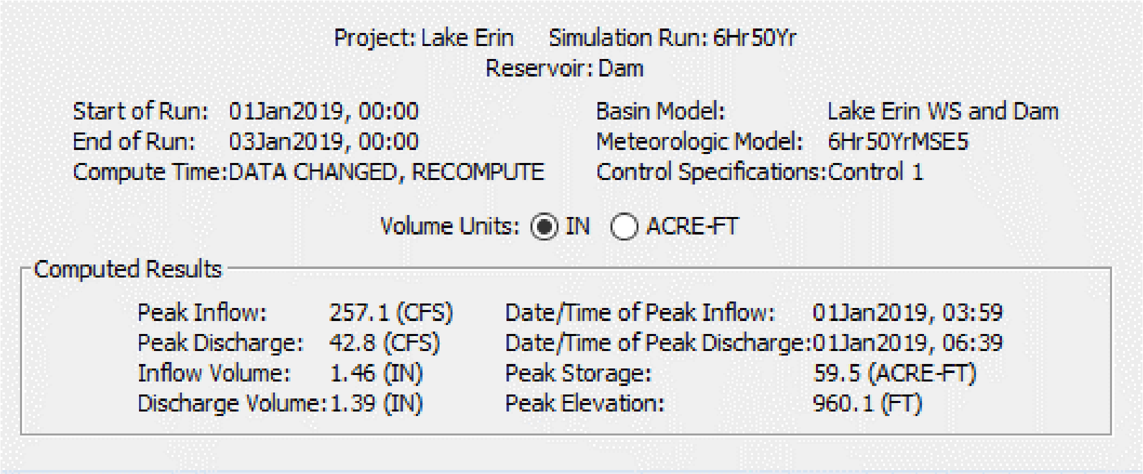


Figure 4. Auxiliary spillway stage-discharge rating curve.

2.4 Flood Routing Results

The two precipitation events defined in Section 2.1 were modeled using U.S. Army Corps of Engineers' (USACE) HEC-HMS version 4.7.1. Both storm events had the same hydrologic model parameters applied to them. The results show that the 50-year event has a peak water surface elevation of EL. 960.1 feet which indicates that the auxiliary spillway won't be activated during this event. The SDF event has a peak water surface elevation of EL. 969.2 feet which exceeds the dam crest elevation of EL. 968.5 feet and doesn't provide any freeboard. The results are displayed in Figures 5 and 6 below.



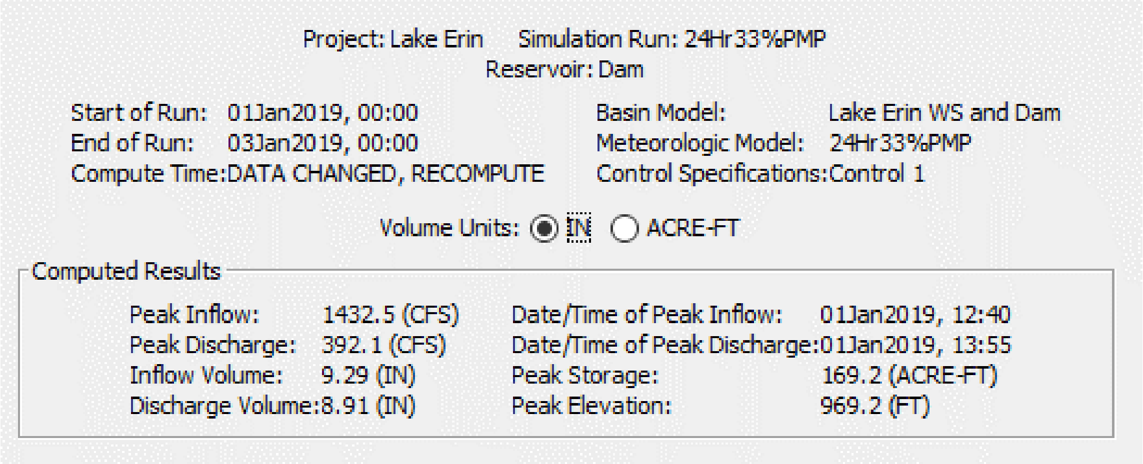
```
            Project: Lake Erin    Simulation Run: 6Hr50Yr
                          Reservoir: Dam

Start of Run:   01Jan2019, 00:00          Basin Model:           Lake Erin WS and Dam
End of Run:     03Jan2019, 00:00          Meteorologic Model:    6Hr50YrMSE5
Compute Time:DATA CHANGED, RECOMPUTE      Control Specifications:Control 1

                  Volume Units: ● IN  ○ ACRE-FT

Computed Results
      Peak Inflow:        257.1 (CFS)   Date/Time of Peak Inflow:     01Jan2019, 03:59
      Peak Discharge:     42.8 (CFS)    Date/Time of Peak Discharge:01Jan2019, 06:39
      Inflow Volume:      1.46 (IN)     Peak Storage:                 59.5 (ACRE-FT)
      Discharge Volume:1.39 (IN)        Peak Elevation:               960.1 (FT)
```

Figure 5. 50-year flood routing results.



```
          Project: Lake Erin    Simulation Run: 24Hr33%PMP
                          Reservoir: Dam

Start of Run:   01Jan2019, 00:00          Basin Model:           Lake Erin WS and Dam
End of Run:     03Jan2019, 00:00          Meteorologic Model:    24Hr33%PMP
Compute Time:DATA CHANGED, RECOMPUTE      Control Specifications:Control 1

                  Volume Units: ● IN  ○ ACRE-FT

Computed Results
      Peak Inflow:        1432.5 (CFS)   Date/Time of Peak Inflow:     01Jan2019, 12:40
      Peak Discharge:     392.1 (CFS)    Date/Time of Peak Discharge:01Jan2019, 13:55
      Inflow Volume:      9.29 (IN)      Peak Storage:                 169.2 (ACRE-FT)
      Discharge Volume:8.91 (IN)         Peak Elevation:               969.2 (FT)
```

Figure 6. SDF flood routing results.

3. Dam Breach Analysis

3.1 Overview

A dam breach analysis was prepared as an update to a previously completed analysis provided by DeKalb County. The modeling was performed using the USACE's HEC-RAS v5.07 program using the one-dimensional and unsteady modeling modes. The analysis was performed as a dry weather (Sunny Day) breach assuming a brim-full reservoir with no outflow.

3.2 Erin Lake Dam Breach Calculations

Breach calculations for Erin Lake Dam were developed using the 2008 Froehlich equations (Froehlich, 2008). These equations were developed using multi-variate regression techniques to identify significant input variables to determining breach shape and timing parameters as well as formulate appropriate equations for determining these parameters. The estimated breach for Erin Lake Dam contemplates an average breach width of 82.5 feet and a formation time of 0.30 hours consistent with the SDP guidelines.

3.3 Model Description

The model consisted of 61 user-defined cross sections, three inline structures (culverts at Allsborough Drive, Interstate 285, and Henderson Mill Road) and one in-line dam structure (Erin Lake Dam).The user-defined cross sections were supplemented with interpolated cross sections spaced at 25 feet to improve model stability and reduce computational errors. Manning's roughness coefficients were developed in the Federal Emergency Management Agency (FEMA) model and cross checked using aerial imagery for reasonableness. Contraction and expansion coefficients were defined as 0.1 and 0.3, respectively but increased to 0.3 and 0.5 in accordance with HEC-RAS guidance adjacent to bridges, dams, and other in-line structures where rapid contraction and expansion of flow was expected. Three downstream in-line structures (culverts at Allsborough Drive, Interstate 285, and Henderson Mill Road) were included in the model based on the geometry previously defined in the FEMA model.

Boundary conditions representing full-brim conditions were defined at the upstream end of the model (immediately downstream of the dam and along Peachtree Branch). To improve the model stability, a moderate baseflow of 50 cfs was initiated immediately downstream of the dam for the first five minutes of run time. Similarly, a moderate baseflow of 20 cfs was initiated for the reach northeast of the dam (Peachtree Branch). The breach hydrograph was developed in HEC-HMS with the initial elevation of the dam set to EL. 968.5 feet (dam crest low elevation) and the hydrograph was input into the HEC-RAS model. A normal depth flow with a friction slope of 0.4% was defined for the downstream limit of the model. The model was run for 1 hour and 45 minutes using a mixed flow regime which allowed for the breach to fully develop and be routed through the entire model. The computation interval was 6 seconds.

3.4 Model Results

The model yielded an estimated breach flood inundation area downstream of Erin Lake Dam between the dam and a location approximately 120 feet upstream of the confluence of Henderson Mill Creek and North Fork Peachtree Creek. A comparison of the inundation map with existing structure locations indicates that up to 29 structures may be impacted should the Erin Lake Dam fail.

The breach calculations for Erin Lake Dam, HEC-RAS output, breach inundation map and list of impacted structures are provided in Appendix E.

4. Additional Analyses

4.1 Freeboard Analysis

A freeboard analysis was performed for Erin Lake Dam using methodologies described in NRCS Technical Release 56 (NRCS, 2014). The effective fetch was evaluated at a location as shown in Figure 7 on the dam and determined to be 500 feet or approximately 0.095 miles.

Figure 7. Effective fetch calculation lengths.

The overland wind velocity was determined using the guidance to be approximately 85 miles per hour (mph). It should be noted that SDP requires a minimum overland wind velocity of 50 mph. Based on the fetch, the over water wind velocity was determined to be approximately 87 mph. The estimated significant wave height (and recommended freeboard height) based on the over water wind velocity and effective fetch is 0.8 feet. It is noted that if the SDP recommended overland wind velocity of 50 mph is considered, the recommended freeboard height would be approximately 0.5 feet. Freeboard calculations can be found in Appendix F.

4.2 Reservoir Drawdown

No low-level outlet exists at Erin Lake Dam, and therefore, there is not currently a way to drawdown the reservoir below the normal pool elevation of EL. 955.70 feet.

4.3 Spillway Attack

The Erin Lake Dam auxiliary spillway is a stone masonry open channel spillway. Hence, it is not necessary to evaluate the spillway for its erosion resistance capacity as stone masonry spillways are less prone to erosion than earthen spillways and are outside the capabilities of the NRCS SITES model.

4.4 Spillway Energy Dissipation

Recent inspections and survey data show that no energy dissipation is provided at the downstream end of the principal and auxiliary spillway outlet. It is recommended that an energy dissipation analysis be performed as part of the design of rehabilitation measures for this dam to ensure that a proposed method of dissipating energy (plunge pool, impact basin, riprap apron, etc.) is adequately sized.

5. Georgia Safe Dams Criteria Review

The following section reviews applicable criteria from the Georgia SDP Engineering Guidelines and determines the level of compliance of Erin Lake Dam has with the SDP criteria based on the results of the previously described analyses.

Criterion 1.1.4.2: Medium Dam – This is a dam that impounds more than 500 acre-feet but no more than 1,000 acre-feet, or has a height exceeding 25 feet but not exceeding 35 feet. The design storm is 33.3% PMP.

The primary design storm used throughout the entire analysis was the 33.3% PMP event. Therefore, the design storm is compliant with the SDP criteria.

Criterion 5.2.2: Design Rainfall Events – The 6-hour rainfall event as defined in HMR 52 is the minimum storm event for the design of primary and emergency spillway flows. Longer rainfall events should be used if the size of the watershed dictates. The length of storm should follow the guidelines for the generation of the storm event.

The duration of the 33.3% PMP event was 24 hours which means the design rainfall duration is compliant with the SDP criteria.

Criterion 5.2.3: Time of Concentration and/or Lag Time - The method used to determine the watershed basin time of concentration should be indicated. Parameter limitations of each method should be followed. The following are a list of methods acceptable to the SDP:

-Natural Resources Conservation Service (NRCS formerly SCS) Curve Number method for drainage basins less than 2000 acres
- Combination Overland Flow and Full Channel Flow
- NRCS Technical Release (TR) No. 55
- Watershed lag techniques based on analysis of gauged watershed similar to the study watershed may also be used. Acceptable references for those procedures include Engineering Manual EM 1110-2-1417, Flood Runoff Analysis (Corps of Engineers) and Flood Hydrology Manual (Bureau of Reclamation).

The method used during this analysis is the NRCS Method which is acceptable per the list of suggested lag time methods. Therefore, the lag time development method is compliant with the SDP standard.

Criterion 5.2.5.1: Curve Number Calculations – The curve number should be adjusted for the antecedent moisture conditions (AMC).

Since the curve numbers already represent AMC II, as described in NRCS National Engineering Handbook (NEH), Part 630, Chapter 10, Table 10-1, the curve number development is compliant with the SDP standard.

Criterion 5.2.6: Development of Design Storm Event – Use HMR No. 51 and No. 52 to develop a precipitation pattern.

The PMP depth estimates were taken from the nomographs in HMR 51 and the NRCS type II storm distribution matches the behavior of the one-hour peak intensity of the HMR-52 generated storm, the design storm is compliant with SDP standard.

<u>Criterion 5.3.3:</u> Tailwater Rating Curve - The outlet rating curve for any type of low-level discharge conduits will be affected by the tailwater downstream of the dam and it should be factored into their design.

The principal spillway outlets to approximately the mid-point of the stone masonry channel which is 17-feet above the downstream toe of the dam. The slope of the stone masonry outlet channel is 3H:1V. Therefore, it was assumed that tailwater would not influence the capacity of the principal spillway in its current configuration. Therefore, this analysis complies with Criterion 5.3.3.

<u>Criterion 5.3.4:</u> Energy Dissipation Design - Each spillway design involves the passing of water. This water is accelerated during design flows and will cause considerable damage if not controlled. The energy in the water must be dissipated using an approved methodology. Design calculations must be provided for each type of energy dissipater.

No energy dissipater exists at the downstream end of the principal and auxiliary spillway outlet. Therefore, no energy dissipation calculations were performed and at the time of this report, and the dam does not comply with Criterion 5.3.4. It is recommended that an energy dissipation analysis be performed as part of the detailed design of rehabilitation measures for this dam to ensure that a proposed method of dissipating energy (plunge pool, stilling/impact basin, riprap apron, etc.) is adequately sized.

<u>Criterion 5.4:</u> Earth Spillway Attack Calculations - Earth spillways are subject to erosion and possible failure during the design storm. Earth spillways must not activate until after the 50-year storm. Adequate resistance must be provided for the dam to perform safely.

This criterion is for earthen spillway and doesn't apply to Erin Lake Dam because it has a stone masonry spillway channel. The stone masonry channel is not activated more frequently that the 50-year storm.

<u>Criterion 5.5:</u> Appropriate freeboard for wave action and runup during the design storm shall be provided. The wave height should be calculated using fetch length and a minimum 50 MPH wind velocity. This height is added to the maximum reservoir surface elevation during the design storm event to determine the minimum top of dam elevation.

A freeboard analysis was prepared and indicated a recommended freeboard value of 0.8 feet. Currently, the SDF water surface elevation exceeds the low point elevation of the dam crest by 0.7 feet. Therefore, the dam is not compliant with Criterion 5.5.

<u>Criterion 5.7:</u> Time to Drain Reservoir - The gated pipe structure or other system shall be designed to drain two-thirds of the volume at normal pool of the reservoir within ten days unless an alternative time frame is approved by the SDP. Calculations showing the rate and volume of drainage need to be provided. Downstream water usage and possible downstream flooding conditions before draining the reservoir should be considered.

No low-level outlet exists at Erin Lake Dam, and there is not currently a way to drawdown the reservoir below the normal pool elevation of EL. 955.70 feet. Therefore, this analysis is not compliant with Criterion 5.7.

6. Proposed Alternatives

Preliminary proposed rehabilitation alternatives were developed for Erin Lake Dam to address the deficiencies outlined in Section 5 of this report. The three SDP criteria that should be addressed as part of a dam rehabilitation include:

1. Criterion 5.3.4 – Satisfactory performance of the energy dissipation at the auxiliary spillway outlet is currently unknown.
2. Criterion 5.5 – The dam is overtopped by approximately 0.7 feet during the SDF and therefore, does not provide the required freeboard over the peak SDF water surface elevation.
3. Criterion 5.7 – The reservoir cannot be drained below its normal pool elevation.

Three rehabilitation alternatives were developed to address these deficiencies which are described below. It should be noted that the rehabilitation alternatives described only address the H&H related deficiencies at the dam. Additional work related to geotechnical, structural or operations and maintenance issues may also be required. A preliminary sketch of each alternative is presented in Appendix H.

6.1 Rehabilitation Alternative 1

One potential alternative to increase spillway capacity is to widen the crest width and/or lower the crest elevation of the existing stone masonry auxiliary spillway located at the left abutment. This would require demolishing a portion of the existing right training wall and either constructing a new training wall(s) or grading the side slopes of the auxiliary spillway. Preliminary reservoir routings show that if the auxiliary spillway crest was widened to 25-feet and lowered to elevation 965.0 feet, the resulting maximum water surface elevation during the SDF would be 968.0 feet. To provide the minimum required freeboard (0.8 feet), the existing low point along the dam crest would also need to be raised. The majority of the dam crest is already at or above elevation 969.0 feet, so it is recommended to raise and level the dam crest to this elevation, thus providing the minimum required freeboard.

In additional to increasing the spillway capacity of the dam other modifications would also be required. To satisfy SDP Criterion 5.7 and provide a means for draining the reservoir, a siphon system could be pursued. To drain two-thirds of the volume at normal pool of the reservoir within ten days, the siphon would need to be able to lower the reservoir elevation from 955.70 feet to 950.4 feet. This equates to a volume of approximately 20 acre-feet over a period of ten days or an average flow rate of approximately 1.0 cfs. An energy dissipation structure would also need to be design and constructed at the outlet of the stone masonry channel. This alternative would require hydraulic and structural analyses of the current stone masonry spillway to ensure that it could safely convey the additional flows being routed.

6.2 Rehabilitation Alternative 2

Another potential alternative to increase spillway capacity is construct an earth-cut auxiliary spillway located at the right abutment. This would likely involve blocking off the crest of the existing auxiliary spillway at the left abutment so that auxiliary spillway flows would not be on both sides of the dam. The same preliminary reservoir routings completed for Alternative 1 apply to Alternative 2. The auxiliary spillway at the right abutment could be 25-feet wide at elevation 965.0 feet, and the resulting maximum water surface elevation during the SDF would be 968.0 feet. Other combinations of widening and lowering the spillway could be pursued as long as the spillway is not lowered such at that it is activated more frequently than the 50-year storm. It is also recommended to raise and level the dam crest to elevation 969.0 for this alternative to provide the minimum required freeboard.

Similar to alternative 1, a siphon system could be pursued to drain two-thirds of the volume at normal pool of the reservoir within ten days, and an energy dissipation structure would be required at the outlet of the stone masonry channel since the principal spillway flows would still be routed at the same location. Some means of energy dissipation would also be needed at the outlet of the new auxiliary spillway at the right abutment. Some considerations if this alternative is pursued include obtaining property rights to construct the spillway, impacts to a sanitary sewer line running through the potential location of the auxiliary spillway, and potentially blocking access to the dam in the event of an emergency if the auxiliary spillway is flowing at the right abutment. Additionally, this alternative would require that a spillway integrity analysis be performed on the proposed auxiliary spillway.

6.3 Rehabilitation Alternative 3

Rather than constructing a new auxiliary spillway at either abutment, a new principal spillway riser structure and conduits could be constructed. This alternative would involve abandoning the existing stone masonry spillway, removing the existing principal spillway pipe, constructing a new concrete riser structure with controls for a gated a low-level outlet and twin pipes sized to convey the SDF. Preliminary reservoir routings show that a 16-foot crest length on the principal spillway riser and twin 48-inch pipes could safely convey the SDF while providing the minimum required freeboard. An impact basin or other means for dissipating energy would be required at the outlet of the pipes. This alternative would address the spillway capacity, drawdown and energy dissipation deficiencies through the construction of a new principal spillway system.

7. Summary and Recommendations

A hydrologic and hydraulic (H&H) analysis of the Erin Lake Dam was completed to determine compliance with the SDP H&H criteria for a Category I dam. The results of the analysis indicate that the dam complies with many of the criteria in the guidance. However, there are three areas in which the dam is not compliant:

1. Criterion 5.3.4 – Satisfactory performance of the energy dissipation at the auxiliary spillway outlet is currently unknown.
2. Criterion 5.5 – The dam is overtopped by approximately 0.7 feet during the SDF and therefore, does not provide the required freeboard over the peak SDF water surface elevation.
3. Criterion 5.7 – The reservoir cannot be drained below its normal pool elevation.

As a result, the following recommendations are made to bring the dam into compliance with SDP criteria:

1. Criterion 5.3.4 – design a sufficient energy dissipation device (plunge pool, stilling/impact basin, riprap apron, etc.) at the outlet of the spillway(s) such that it is capable of sufficiently dissipating energy from the design peak discharge to meet this criterion.
2. Criterion 5.5 – modify the dam, outlet works, or spillways to increase freeboard to meet the required amount. This could be done by increasing spillway capacity to lower the peak water surface elevation for the SDF, raising the crest of the dam, or combinations thereof. A SITES analysis may be required if an earth-cut auxiliary spillway is pursued as an alternative.
3. Criterion 5.7 – design a gated pipe structure or siphon system to drain two-thirds of the volume at normal pool of the reservoir within ten days or an alternative time frame if approved by SDP.

Three potential rehabilitation alternatives were developed to address the deficiencies identified in this report. Those alternatives are summarized below:

- Rehabilitation Alternative 1 – widen and lower the auxiliary spillway at the left abutment, raise dam crest low point to elevation 969.0 feet, install a low-level outlet siphon system, and install an energy dissipater at the outlet of the existing masonry spillway.
- Rehabilitation Alternative 2 – construct an earth-cut auxiliary spillway located at the right abutment, block off the crest of the existing auxiliary spillway at the left abutment, install a low-level outlet siphon system, and install an energy dissipater at the outlet of the existing masonry spillway and new auxiliary spillway at the right abutment.
- Rehabilitation Alternative 3 – abandon the existing stone masonry spillway, remove the existing principal spillway pipe, construct a new concrete riser structure with controls for a gated a low-level outlet and twin 48-inch pipes, and an energy dissipator at the outlet of the pipes.

Modifications to the dam may result in changes to the H&H analysis. As such, revisions to the hydrologic and/or dam breach modeling and/or report should be made to reflect these modifications once they are implemented.

8. References

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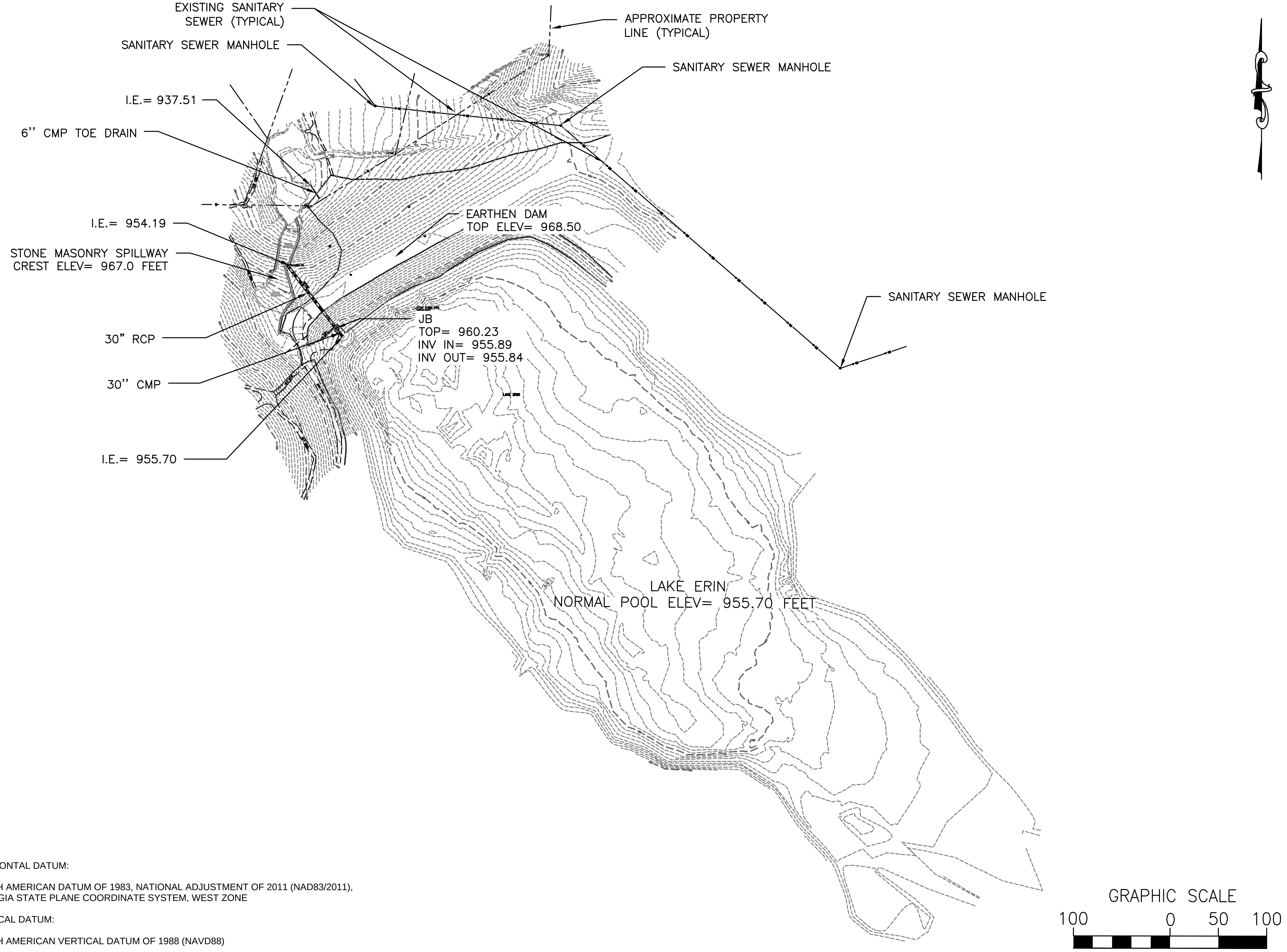
Appendix A – Topographic and Bathymetric Survey Map



HORIZONTAL DATUM:

NORTH AMERICAN DATUM OF 1983, NATIONAL ADJUSTMENT OF 2011 (NAD83/2011), GEORGIA STATE PLANE COORDINATE SYSTEM, WEST ZONE

VERTICAL DATUM:

NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD88)

AECOM

PROJECT

ERIN LAKE DAM

OWNER/CLIENT

DEKALB COUNTY, GEORGIA
PW - ROADS & DRAINAGE C/M
729 CAMP ROAD
DECATUR, GA 30032

CONSULTANT

AECOM TECHNICAL SERVICES, INC.
1360 PEACHTREE STREET, NE
SUITE 500
ATLANTA, GA 30309
PHONE: (404) 965 9600

PROJECT NUMBER

60618914

Designed By:	
Drawn By:	S. THAPA
Dept Check:	W. HOLLENBACH
Proj Check:	
Date:	5/12/2021
Scale:	1" = 100'

DISCIPLINE

WATER

SHEET TITLE

TOPOGRAPHIC AND BATHYMETRIC SURVEY PLAN

SHEET NUMBER

FIGURE 1

Appendix B - Precipitation Analysis Supporting Information

NOAA Atlas 14, Volume 9, Version 2
Location name: Tucker, Georgia, USA*
Latitude: 33.8674°, Longitude: -84.2302°
Elevation: 966.05 ft**
* source: ESRI Maps
** source: USGS

POINT PRECIPITATION FREQUENCY ESTIMATES

Sanja Perica, Deborah Martin, Sandra Pavlovic, Ishani Roy, Michael St. Laurent, Carl Trypaluk, Dale Unruh, Michael Yekta, Geoffery Bonnin

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	**0.405** (0.322-0.516)	**0.466** (0.371-0.594)	**0.568** (0.451-0.725)	**0.655** (0.518-0.837)	**0.777** (0.601-1.01)	**0.874** (0.665-1.15)	**0.974** (0.723-1.29)	**1.08** (0.776-1.44)	**1.22** (0.852-1.65)	**1.32** (0.909-1.81)
10-min	**0.593** (0.472-0.755)	**0.682** (0.543-0.870)	**0.832** (0.660-1.06)	**0.959** (0.758-1.23)	**1.14** (0.881-1.48)	**1.28** (0.974-1.68)	**1.43** (1.06-1.89)	**1.58** (1.14-2.11)	**1.78** (1.25-2.42)	**1.94** (1.33-2.65)
15-min	**0.723** (0.576-0.921)	**0.832** (0.662-1.06)	**1.01** (0.805-1.29)	**1.17** (0.924-1.50)	**1.39** (1.07-1.81)	**1.56** (1.19-2.04)	**1.74** (1.29-2.30)	**1.92** (1.39-2.58)	**2.17** (1.52-2.95)	**2.36** (1.62-3.23)
30-min	**1.02** (0.814-1.30)	**1.18** (0.937-1.50)	**1.44** (1.14-1.84)	**1.66** (1.31-2.12)	**1.97** (1.52-2.57)	**2.22** (1.69-2.90)	**2.47** (1.83-3.27)	**2.73** (1.97-3.66)	**3.08** (2.16-4.19)	**3.36** (2.31-4.59)
60-min	**1.31** (1.04-1.67)	**1.50** (1.20-1.91)	**1.83** (1.45-2.33)	**2.11** (1.67-2.70)	**2.52** (1.95-3.29)	**2.85** (2.17-3.74)	**3.19** (2.37-4.23)	**3.55** (2.56-4.77)	**4.04** (2.84-5.50)	**4.43** (3.04-6.06)
2-hr	**1.60** (1.29-2.00)	**1.83** (1.47-2.29)	**2.22** (1.78-2.79)	**2.57** (2.05-3.23)	**3.07** (2.42-3.96)	**3.48** (2.69-4.51)	**3.91** (2.95-5.13)	**4.37** (3.20-5.80)	**5.00** (3.56-6.73)	**5.50** (3.84-7.43)
3-hr	**1.78** (1.45-2.22)	**2.03** (1.64-2.52)	**2.45** (1.99-3.06)	**2.83** (2.28-3.54)	**3.39** (2.70-4.35)	**3.86** (3.01-4.96)	**4.35** (3.31-5.66)	**4.87** (3.60-6.43)	**5.61** (4.04-7.50)	**6.20** (4.36-8.30)
6-hr	**2.19** (1.80-2.68)	**2.46** (2.02-3.01)	**2.94** (2.41-3.61)	**3.37** (2.76-4.15)	**4.02** (3.25-5.09)	**4.57** (3.62-5.80)	**5.15** (3.99-6.62)	**5.78** (4.35-7.53)	**6.67** (4.88-8.81)	**7.39** (5.28-9.77)
12-hr	**2.72** (2.27-3.29)	**3.03** (2.53-3.66)	**3.59** (2.98-4.34)	**4.08** (3.38-4.95)	**4.82** (3.94-6.00)	**5.44** (4.37-6.80)	**6.10** (4.78-7.72)	**6.81** (5.19-8.73)	**7.80** (5.79-10.1)	**8.61** (6.25-11.2)
24-hr	**3.27** (2.77-3.89)	**3.68** (3.11-4.38)	**4.39** (3.70-5.23)	**5.00** (4.20-5.97)	**5.88** (4.86-7.18)	**6.60** (5.36-8.10)	**7.35** (5.83-9.13)	**8.13** (6.28-10.3)	**9.22** (6.94-11.8)	**10.1** (7.43-13.0)
2-day	**3.79** (3.25-4.43)	**4.34** (3.71-5.08)	**5.25** (4.49-6.16)	**6.03** (5.13-7.09)	**7.12** (5.95-8.54)	**7.99** (6.57-9.63)	**8.87** (7.13-10.8)	**9.78** (7.66-12.1)	**11.0** (8.41-13.9)	**12.0** (8.98-15.2)
3-day	**4.18** (3.61-4.85)	**4.74** (4.10-5.51)	**5.70** (4.91-6.63)	**6.53** (5.60-7.60)	**7.71** (6.51-9.18)	**8.67** (7.20-10.4)	**9.65** (7.84-11.7)	**10.7** (8.46-13.2)	**12.1** (9.34-15.1)	**13.2** (10.0-16.6)
4-day	**4.52** (3.93-5.21)	**5.08** (4.42-5.86)	**6.06** (5.25-7.00)	**6.92** (5.97-8.01)	**8.17** (6.96-9.69)	**9.20** (7.70-11.0)	**10.3** (8.41-12.4)	**11.4** (9.10-14.0)	**13.0** (10.1-16.2)	**14.3** (10.9-17.8)
7-day	**5.37** (4.72-6.11)	**5.98** (5.25-6.80)	**7.06** (6.18-8.04)	**8.03** (7.01-9.17)	**9.48** (8.18-11.1)	**10.7** (9.06-12.6)	**12.0** (9.94-14.3)	**13.3** (10.8-16.2)	**15.3** (12.1-18.8)	**16.9** (13.0-20.8)
10-day	**6.09** (5.39-6.87)	**6.76** (5.98-7.63)	**7.94** (7.01-8.98)	**9.01** (7.92-10.2)	**10.6** (9.22-12.3)	**11.9** (10.2-14.0)	**13.3** (11.2-15.8)	**14.9** (12.1-17.9)	**17.0** (13.5-20.8)	**18.7** (14.6-23.0)
20-day	**8.16** (7.33-9.06)	**9.01** (8.08-10.0)	**10.5** (9.37-11.6)	**11.7** (10.5-13.1)	**13.6** (12.0-15.5)	**15.1** (13.1-17.4)	**16.7** (14.2-19.5)	**18.4** (15.2-21.8)	**20.7** (16.8-24.9)	**22.6** (17.9-27.3)
30-day	**10.0** (9.07-11.0)	**11.0** (9.98-12.1)	**12.7** (11.5-14.0)	**14.2** (12.8-15.6)	**16.2** (14.3-18.2)	**17.8** (15.5-20.2)	**19.5** (16.6-22.4)	**21.1** (17.6-24.7)	**23.4** (19.1-27.8)	**25.2** (20.2-30.2)
45-day	**12.5** (11.4-13.6)	**13.8** (12.6-15.0)	**15.8** (14.4-17.3)	**17.5** (15.9-19.1)	**19.7** (17.5-21.9)	**21.4** (18.8-23.9)	**23.1** (19.8-26.1)	**24.7** (20.7-28.5)	**26.8** (22.0-31.4)	**28.3** (22.9-33.6)
60-day	**14.8** (13.6-16.0)	**16.3** (15.0-17.6)	**18.6** (17.1-20.2)	**20.5** (18.7-22.2)	**22.9** (20.4-25.1)	**24.6** (21.7-27.3)	**26.2** (22.7-29.5)	**27.8** (23.4-31.7)	**29.7** (24.5-34.4)	**30.9** (25.2-36.5)

[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.

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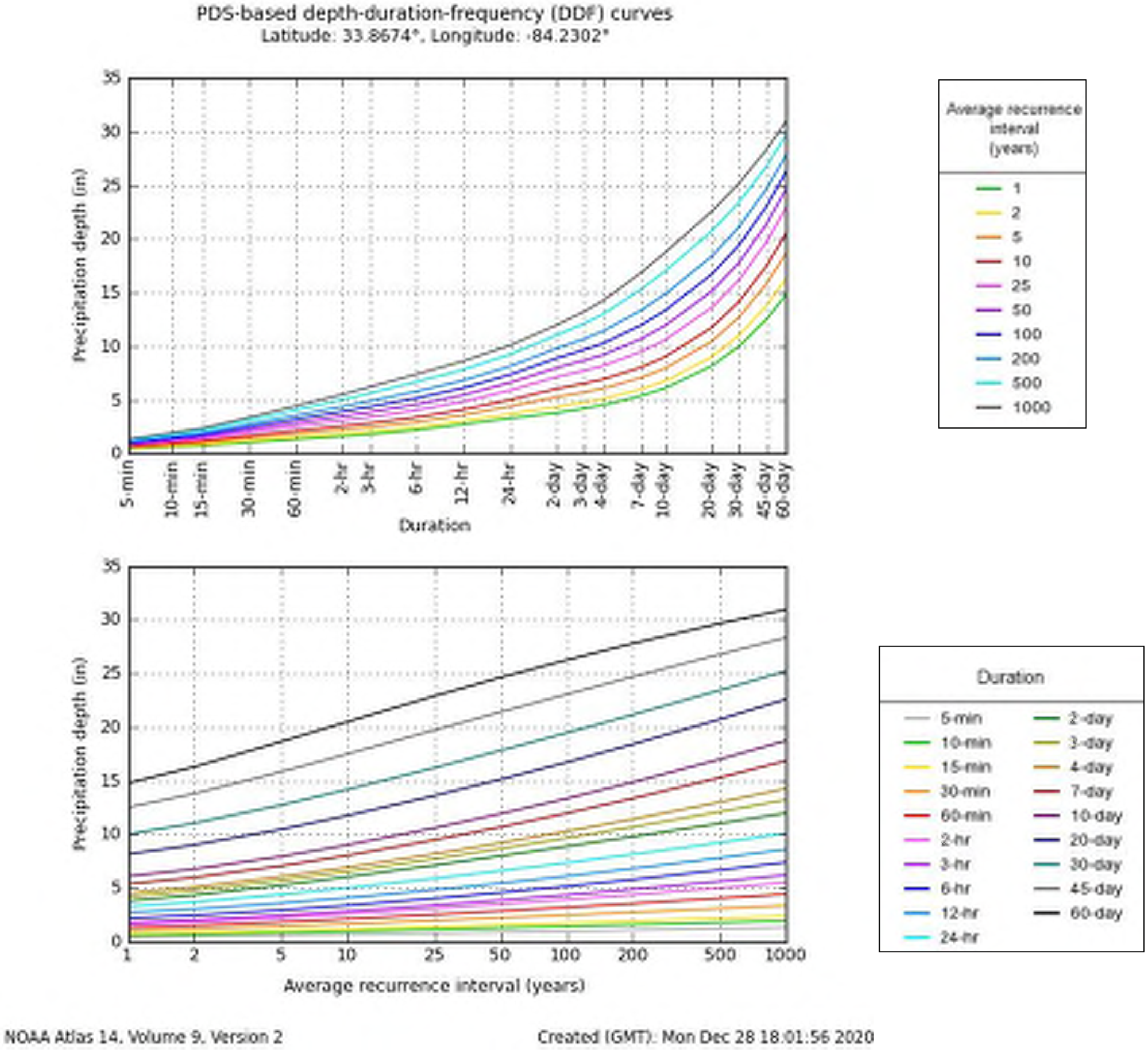
PF graphical



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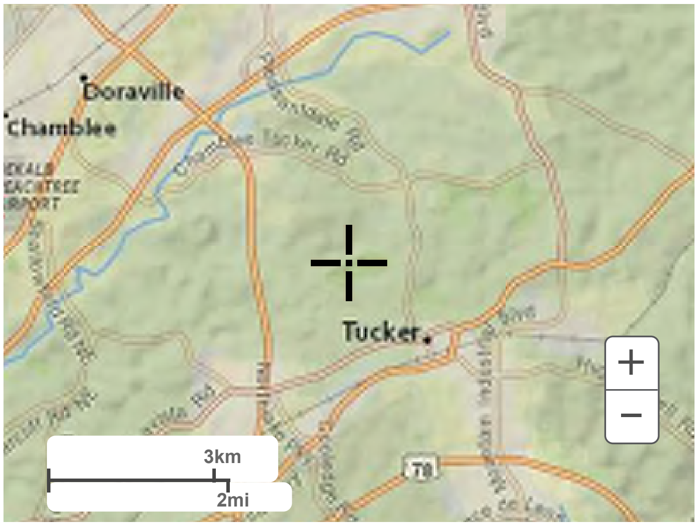
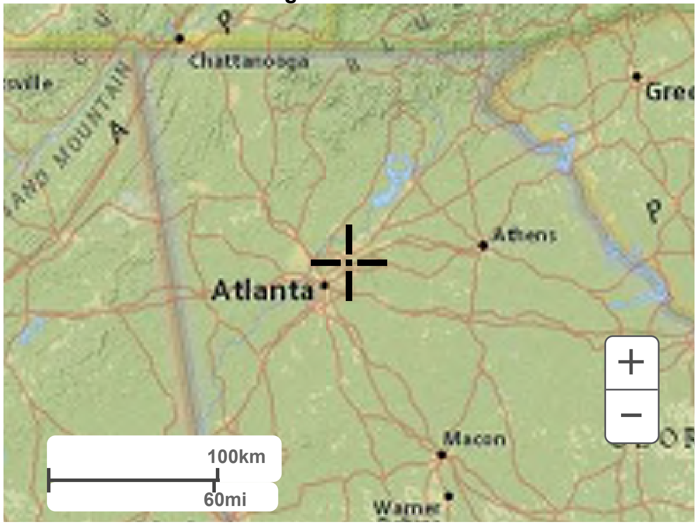
Maps & aerials

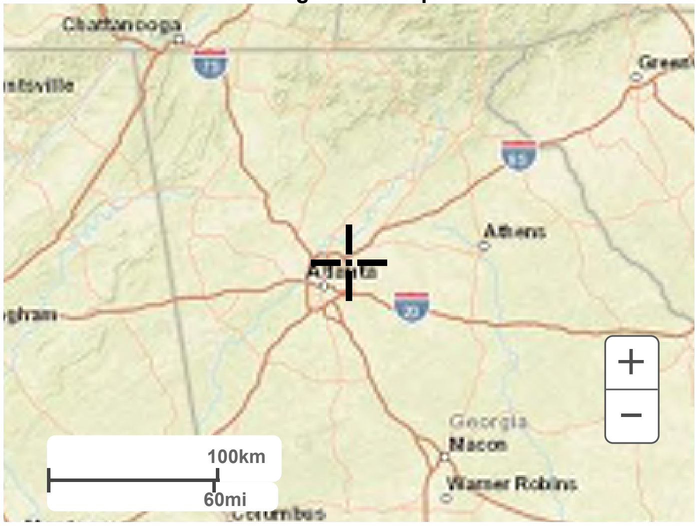
Small scale terrain



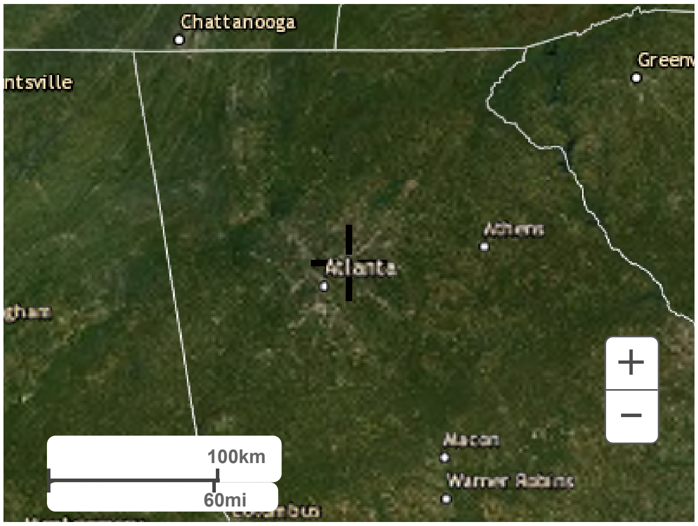
Large scale terrain



Large scale map



Large scale aerial



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US Department of Commerce
National Oceanic and Atmospheric Administration
National Weather Service
National Water Center
1325 East West Highway
Silver Spring, MD 20910
Questions?: HDSC.Questions@noaa.gov

Disclaimer

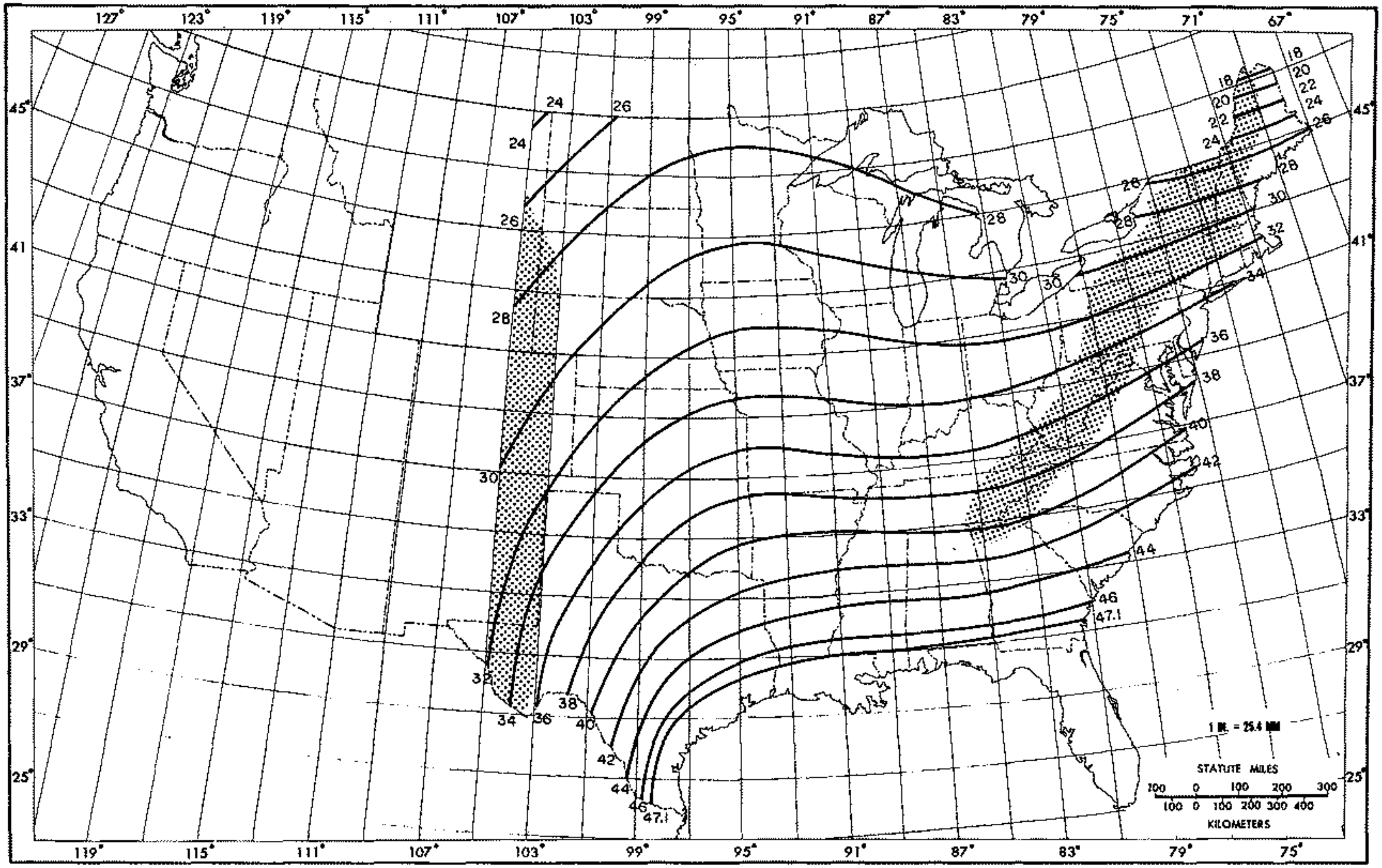


Figure 20.--All-season PMP (in.) for 24 hr 10 mi^2 (26 km^2).

Time (hours)	Distributions (cumulative inches)				
	24-Hour MSE5 Dimensionless	6-Hour MSE5 Dimensionless	50-year 6-hour MSE5	24-Hour Type II Dimensionless	33.3%PMP 24-Hour Type II
0.0	0.0000			0.000	0.0000
0.1	0.0005			0.001	0.0140
0.2	0.0011			0.002	0.0280
0.3	0.0017			0.003	0.0420
0.4	0.0023			0.004	0.0574
0.5	0.0030			0.005	0.0714
0.6	0.0036			0.006	0.0868
0.7	0.0043			0.007	0.1008
0.8	0.0050			0.008	0.1162
0.9	0.0057			0.009	0.1316
1.0	0.0065			0.011	0.1470
1.1	0.0073			0.012	0.1624
1.2	0.0080			0.013	0.1778
1.3	0.0089			0.014	0.1932
1.4	0.0097			0.015	0.2100
1.5	0.0106			0.016	0.2254
1.6	0.0115			0.017	0.2422
1.7	0.0124			0.018	0.2576
1.8	0.0133			0.020	0.2744
1.9	0.0143			0.021	0.2912
2.0	0.0152			0.022	0.3080
2.1	0.0162			0.023	0.3248
2.2	0.0173			0.024	0.3416
2.3	0.0183			0.026	0.3598
2.4	0.0194			0.027	0.3766
2.5	0.0205			0.028	0.3934
2.6	0.0216			0.029	0.4116
2.7	0.0227			0.031	0.4284
2.8	0.0239			0.032	0.4466
2.9	0.0251			0.033	0.4648
3.0	0.0263			0.035	0.4830
3.1	0.0275			0.036	0.5012
3.2	0.0288			0.037	0.5194
3.3	0.0301			0.038	0.5376
3.4	0.0314			0.040	0.5572
3.5	0.0327			0.041	0.5754
3.6	0.0340			0.043	0.5950
3.7	0.0354			0.044	0.6146
3.8	0.0368			0.045	0.6328
3.9	0.0382			0.047	0.6524
4.0	0.0397			0.048	0.6720

4.1	0.0411			0.049	0.6916
4.2	0.0426			0.051	0.7112
4.3	0.0441			0.052	0.7322
4.4	0.0456			0.054	0.7532
4.5	0.0472			0.055	0.7742
4.6	0.0488			0.057	0.7952
4.7	0.0504			0.058	0.8162
4.8	0.0520			0.060	0.8372
4.9	0.0536			0.061	0.8596
5.0	0.0553			0.063	0.8820
5.1	0.0570			0.065	0.9044
5.2	0.0587			0.066	0.9268
5.3	0.0604			0.068	0.9506
5.4	0.0622			0.070	0.9744
5.5	0.0640			0.071	0.9968
5.6	0.0658			0.073	1.0220
5.7	0.0676			0.075	1.0458
5.8	0.0695			0.076	1.0696
5.9	0.0713			0.078	1.0948
6.0	0.0732			0.080	1.1200
6.1	0.0751			0.082	1.1452
6.2	0.0771			0.084	1.1704
6.3	0.0791			0.086	1.1970
6.4	0.0810			0.087	1.2236
6.5	0.0830			0.089	1.2488
6.6	0.0851			0.091	1.2768
6.7	0.0871			0.093	1.3034
6.8	0.0892			0.095	1.3300
6.9	0.0913			0.097	1.3580
7.0	0.0934			0.099	1.3860
7.1	0.0956			0.101	1.4140
7.2	0.0978			0.103	1.4420
7.3	0.1000			0.105	1.4714
7.4	0.1022			0.107	1.5008
7.5	0.1044			0.109	1.5302
7.6	0.1067			0.111	1.5596
7.7	0.1090			0.114	1.5890
7.8	0.1113			0.116	1.6184
7.9	0.1136			0.118	1.6492
8.0	0.1160			0.120	1.6800

8.1	0.1183			0.122	1.7108
8.2	0.1207			0.125	1.7444
8.3	0.1232			0.127	1.7780
8.4	0.1256			0.130	1.8144
8.5	0.1281			0.132	1.8508
8.6	0.1306			0.135	1.8900
8.7	0.1331			0.138	1.9306
8.8	0.1356			0.141	1.9712
8.9	0.1382			0.144	2.0132
9.0	0.1408	0.0000	0.0000	0.147	2.0580
9.1	0.1446	0.0053	0.0244	0.150	2.1028
9.2	0.1485	0.0107	0.0490	0.153	2.1476
9.3	0.1524	0.0162	0.0740	0.157	2.1924
9.4	0.1564	0.0217	0.0992	0.160	2.2372
9.5	0.1604	0.0273	0.1248	0.163	2.2820
9.6	0.1645	0.0330	0.1507	0.166	2.3282
9.7	0.1686	0.0387	0.1770	0.170	2.3758
9.8	0.1728	0.0445	0.2035	0.173	2.4262
9.9	0.1770	0.0504	0.2304	0.177	2.4794
10.0	0.1813	0.0564	0.2575	0.181	2.5340
10.1	0.1856	0.0624	0.2850	0.185	2.5914
10.2	0.1899	0.0685	0.3128	0.190	2.6530
10.3	0.1944	0.0746	0.3409	0.194	2.7174
10.4	0.1988	0.0808	0.3694	0.199	2.7846
10.5	0.2034	0.0871	0.3981	0.204	2.8560
10.6	0.2097	0.0960	0.4388	0.209	2.9316
10.7	0.2168	0.1058	0.4837	0.215	3.0128
10.8	0.2245	0.1166	0.5329	0.221	3.0996
10.9	0.2330	0.1283	0.5864	0.228	3.1920
11.0	0.2420	0.1409	0.6441	0.235	3.2900
11.1	0.2518	0.1545	0.7061	0.243	3.3978
11.2	0.2622	0.1690	0.7724	0.251	3.5182
11.3	0.2733	0.1845	0.8430	0.261	3.6526
11.4	0.2851	0.2008	0.9178	0.272	3.8010
11.5	0.2975	0.2182	0.9970	0.283	3.9620
11.6	0.3142	0.2414	1.1031	0.307	4.2952
11.7	0.3363	0.2721	1.2435	0.354	4.9616
11.8	0.3658	0.3132	1.4314	0.431	6.0312
11.9	0.4057	0.3687	1.6849	0.568	7.9506
12.0	0.4756	0.4660	2.1298	0.663	9.2820

12.1	0.5944	0.6313	2.8851	0.682	9.5480
12.2	0.6342	0.6868	3.1386	0.699	9.7804
12.3	0.6637	0.7279	3.3265	0.713	9.9820
12.4	0.6858	0.7586	3.4669	0.725	10.1528
12.5	0.7025	0.7818	3.5730	0.735	10.2900
12.6	0.7149	0.7992	3.6522	0.743	10.4076
12.7	0.7267	0.8155	3.7270	0.751	10.5196
12.8	0.7378	0.8310	3.7976	0.759	10.6232
12.9	0.7482	0.8455	3.8639	0.766	10.7184
13.0	0.7580	0.8591	3.9259	0.772	10.8080
13.1	0.7671	0.8717	3.9836	0.778	10.8920
13.2	0.7755	0.8834	4.0371	0.784	10.9704
13.3	0.7832	0.8942	4.0863	0.789	11.0460
13.4	0.7903	0.9040	4.1312	0.794	11.1188
13.5	0.7967	0.9129	4.1719	0.799	11.1860
13.6	0.8012	0.9192	4.2006	0.804	11.2504
13.7	0.8056	0.9254	4.2291	0.808	11.3120
13.8	0.8101	0.9315	4.2572	0.812	11.3708
13.9	0.8144	0.9376	4.2850	0.816	11.4268
14.0	0.8188	0.9436	4.3125	0.820	11.4800
14.1	0.8230	0.9496	4.3396	0.824	11.5318
14.2	0.8273	0.9555	4.3665	0.827	11.5822
14.3	0.8314	0.9613	4.3930	0.831	11.6312
14.4	0.8355	0.9670	4.4193	0.834	11.6788
14.5	0.8396	0.9727	4.4452	0.838	11.7264
14.6	0.8436	0.9783	4.4708	0.841	11.7726
14.7	0.8476	0.9838	4.4960	0.844	11.8188
14.8	0.8515	0.9893	4.5210	0.847	11.8636
14.9	0.8554	0.9947	4.5456	0.851	11.9112
15.0	0.8592	1.0000	4.5700	0.854	11.9490
15.1	0.8618			0.857	11.9910
15.2	0.8644			0.859	12.0316
15.3	0.8669			0.862	12.0708
15.4	0.8694			0.865	12.1086
15.5	0.8719			0.868	12.1464
15.6	0.8744			0.870	12.1828
15.7	0.8768			0.873	12.2192
15.8	0.8793			0.875	12.2542
15.9	0.8817			0.878	12.2878
16.0	0.8840			0.880	12.3200

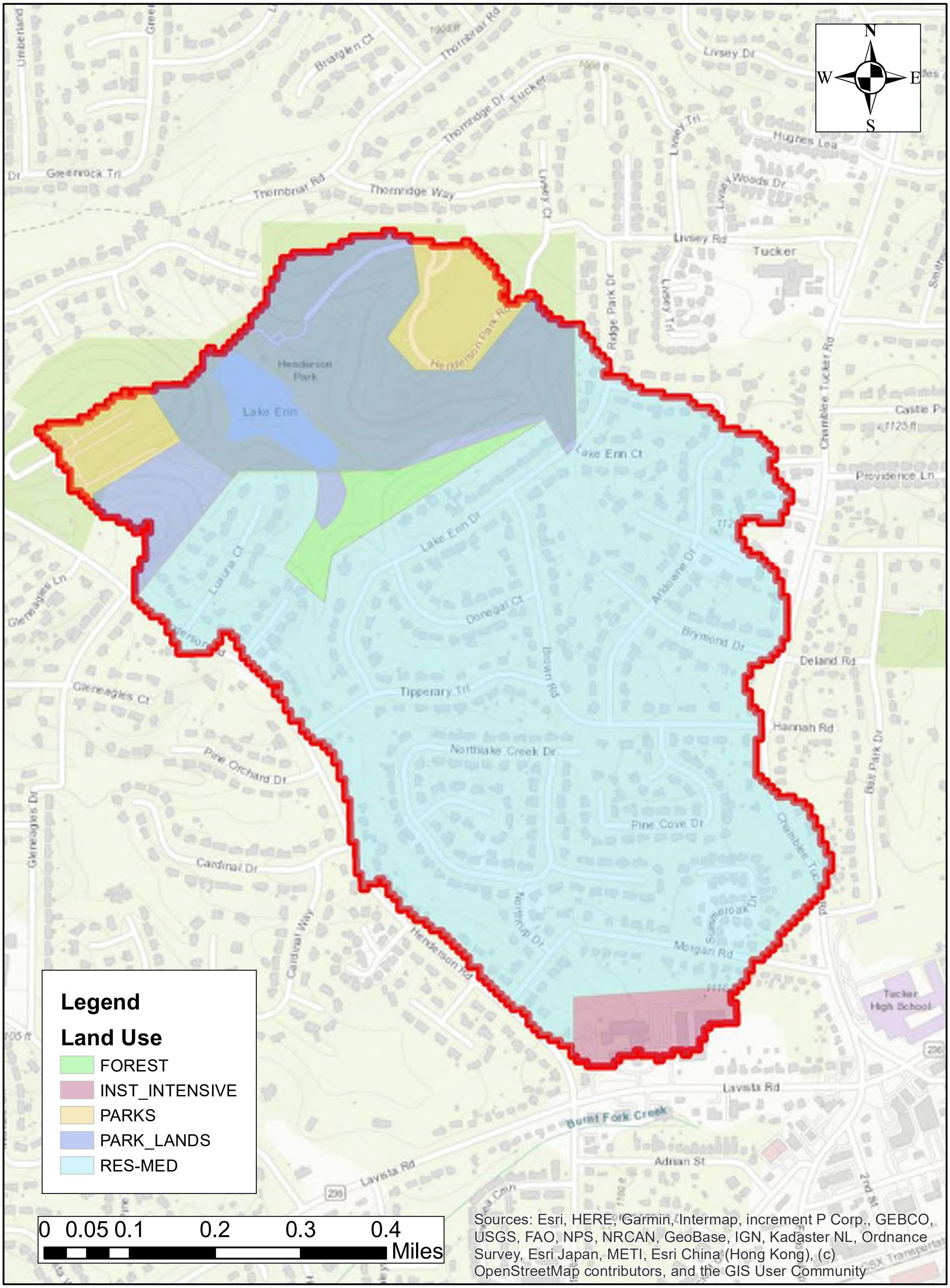
16.1	0.8864			0.882	12.3522
16.2	0.8887			0.885	12.3830
16.3	0.8910			0.887	12.4152
16.4	0.8933			0.889	12.4460
16.5	0.8956			0.891	12.4768
16.6	0.8978			0.893	12.5076
16.7	0.9000			0.896	12.5370
16.8	0.9022			0.898	12.5664
16.9	0.9044			0.900	12.5958
17.0	0.9066			0.902	12.6252
17.1	0.9087			0.904	12.6532
17.2	0.9108			0.906	12.6812
17.3	0.9129			0.908	12.7092
17.4	0.9149			0.910	12.7358
17.5	0.9170			0.912	12.7638
17.6	0.9190			0.914	12.7904
17.7	0.9210			0.916	12.8170
17.8	0.9229			0.917	12.8422
17.9	0.9249			0.919	12.8688
18.0	0.9268			0.921	12.8940
18.1	0.9287			0.923	12.9192
18.2	0.9306			0.925	12.9430
18.3	0.9324			0.926	12.9682
18.4	0.9342			0.928	12.9920
18.5	0.9360			0.930	13.0158
18.6	0.9378			0.931	13.0382
18.7	0.9396			0.933	13.0620
18.8	0.9413			0.935	13.0844
18.9	0.9430			0.936	13.1068
19.0	0.9447			0.938	13.1278
19.1	0.9464			0.939	13.1502
19.2	0.9480			0.941	13.1712
19.3	0.9496			0.942	13.1922
19.4	0.9512			0.944	13.2132
19.5	0.9528			0.945	13.2328
19.6	0.9544			0.947	13.2524
19.7	0.9559			0.948	13.2720
19.8	0.9574			0.949	13.2902
19.9	0.9589			0.951	13.3098
20.0	0.9604			0.952	13.3280

20.1	0.9618			0.953	13.3462
20.2	0.9632			0.955	13.3644
20.3	0.9646			0.956	13.3826
20.4	0.9660			0.957	13.4008
20.5	0.9673			0.958	13.4176
20.6	0.9686			0.960	13.4358
20.7	0.9699			0.961	13.4540
20.8	0.9712			0.962	13.4708
20.9	0.9725			0.964	13.4890
21.0	0.9737			0.965	13.5058
21.1	0.9749			0.966	13.5240
21.2	0.9761			0.967	13.5408
21.3	0.9773			0.969	13.5590
21.4	0.9784			0.970	13.5758
21.5	0.9795			0.971	13.5926
21.6	0.9806			0.972	13.6108
21.7	0.9817			0.973	13.6276
21.8	0.9827			0.975	13.6444
21.9	0.9838			0.976	13.6612
22.0	0.9848			0.977	13.6780
22.1	0.9857			0.978	13.6948
22.2	0.9867			0.979	13.7116
22.3	0.9876			0.981	13.7284
22.4	0.9885			0.982	13.7452
22.5	0.9894			0.983	13.7606
22.6	0.9903			0.984	13.7774
22.7	0.9911			0.985	13.7942
22.8	0.9920			0.986	13.8096
22.9	0.9928			0.988	13.8264
23.0	0.9935			0.989	13.8418
23.1	0.9943			0.990	13.8586
23.2	0.9950			0.991	13.8740
23.3	0.9957			0.992	13.8908
23.4	0.9964			0.993	13.9062
23.5	0.9971			0.994	13.9216
23.6	0.9977			0.996	13.9384
23.7	0.9983			0.997	13.9538
23.8	0.9989			0.998	13.9692
23.9	0.9995			0.999	13.9846
24.0	1.0000			1.000	14.0000

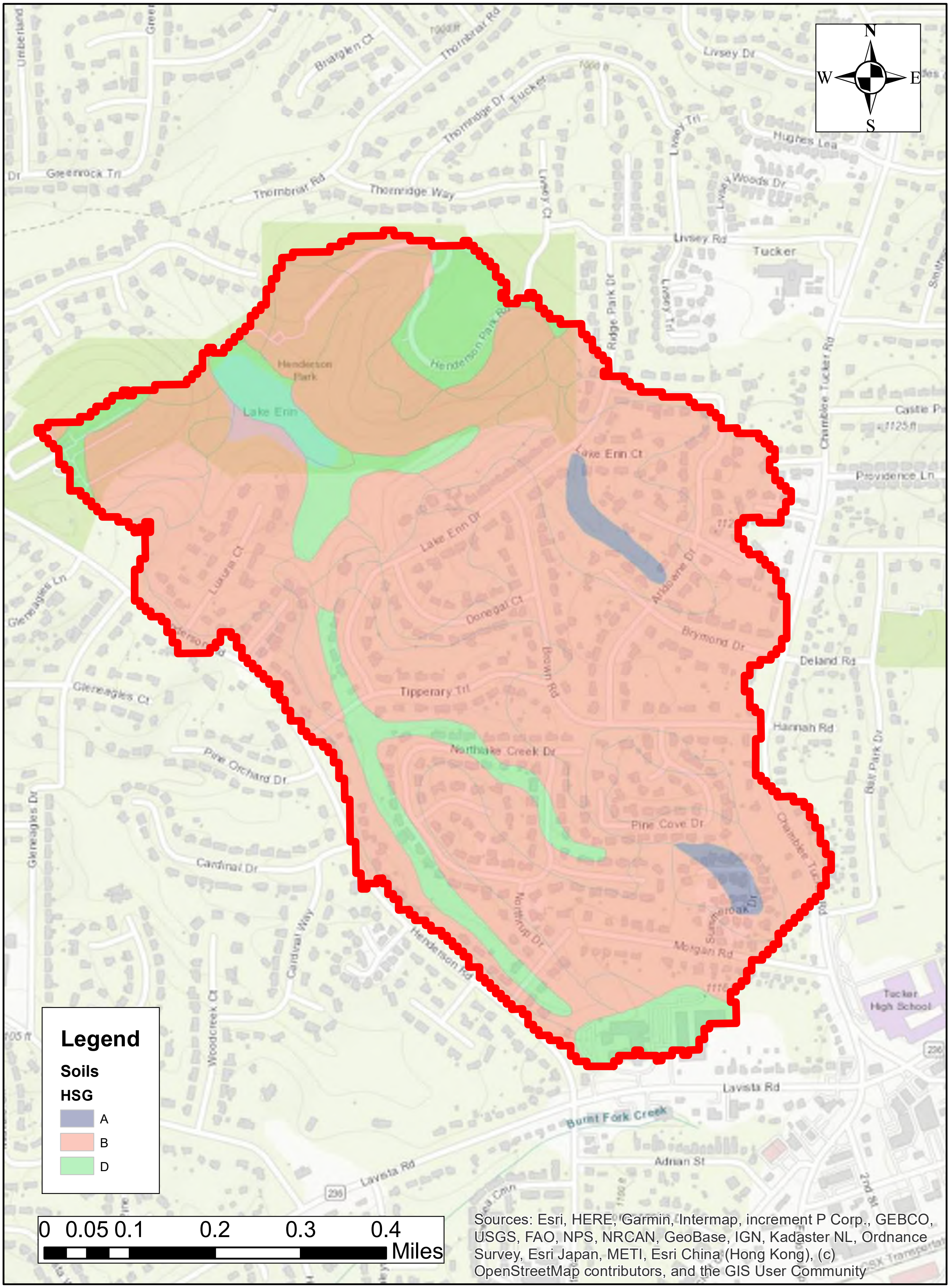
Appendix C – Hydrologic Parameter Supporting Information

Watershed Map

Sources: Esri, HERE, Garmin, Intermap, increment P Corp., GEBCO, USGS, FAO, NPS, NRCAN, GeoBase, IGN, Kadaster NL, Ordnance Survey, Esri Japan, METI, Esri China (Hong Kong), (c) OpenStreetMap contributors, and the GIS User Community



Sources: Esri, HERE, Garmin, Intermap, increment P Corp., GEBCO, USGS, FAO, NPS, NRCAN, GeoBase, IGN, Kadaster NL, Ordnance Survey, Esri Japan, METI, Esri China (Hong Kong), (c) OpenStreetMap contributors, and the GIS User Community



Sources: Esri, HERE, Garmin, Intermap, increment P Corp., GEBCO, USGS, FAO, NPS, NRCAN, GeoBase, IGN, Kadaster NL, Ordnance Survey, Esri Japan, METI, Esri China (Hong Kong), (c) OpenStreetMap contributors, and the GIS User Community

Project: Erin Lake Dam
By: ST
Date: 1/28/2021

Location: Dekalb County, Georgia
Checked: WCH
Date: 1/29/2021

Drainage Area = 324.9 acres = 0.51 mi^2

Hydrologic Soil Group	LandPRO 2010 Land Cover Description	Curve Number	Area (acres)	% of Area	CN x Area
A	40. Forest	30	5.31	1.63%	159
B	53. Reservoirs, Lakes, and Ponds	98	56.91	17.52%	5,577
B	112. Medium Density Single Family Residential	70	0.48	0.15%	34
B	40. Forest	55	211.05	64.96%	11,608
B	121. Intensive Institutional	88	1.88	0.58%	165
B	173. Parks	61	5.88	1.81%	358
D	53. Reservoirs, Lakes, and Ponds	98	4.80	1.48%	470
D	112. Medium Density Single Family Residential	85	6.85	2.11%	582
D	40. Forest	77	13.92	4.28%	1,072
D	121. Intensive Institutional	93	6.52	2.01%	606
D	173. Parks	80	11.31	3.48%	905
	Totals		324.9	100.0%	21537

Weighted CN = 66.3

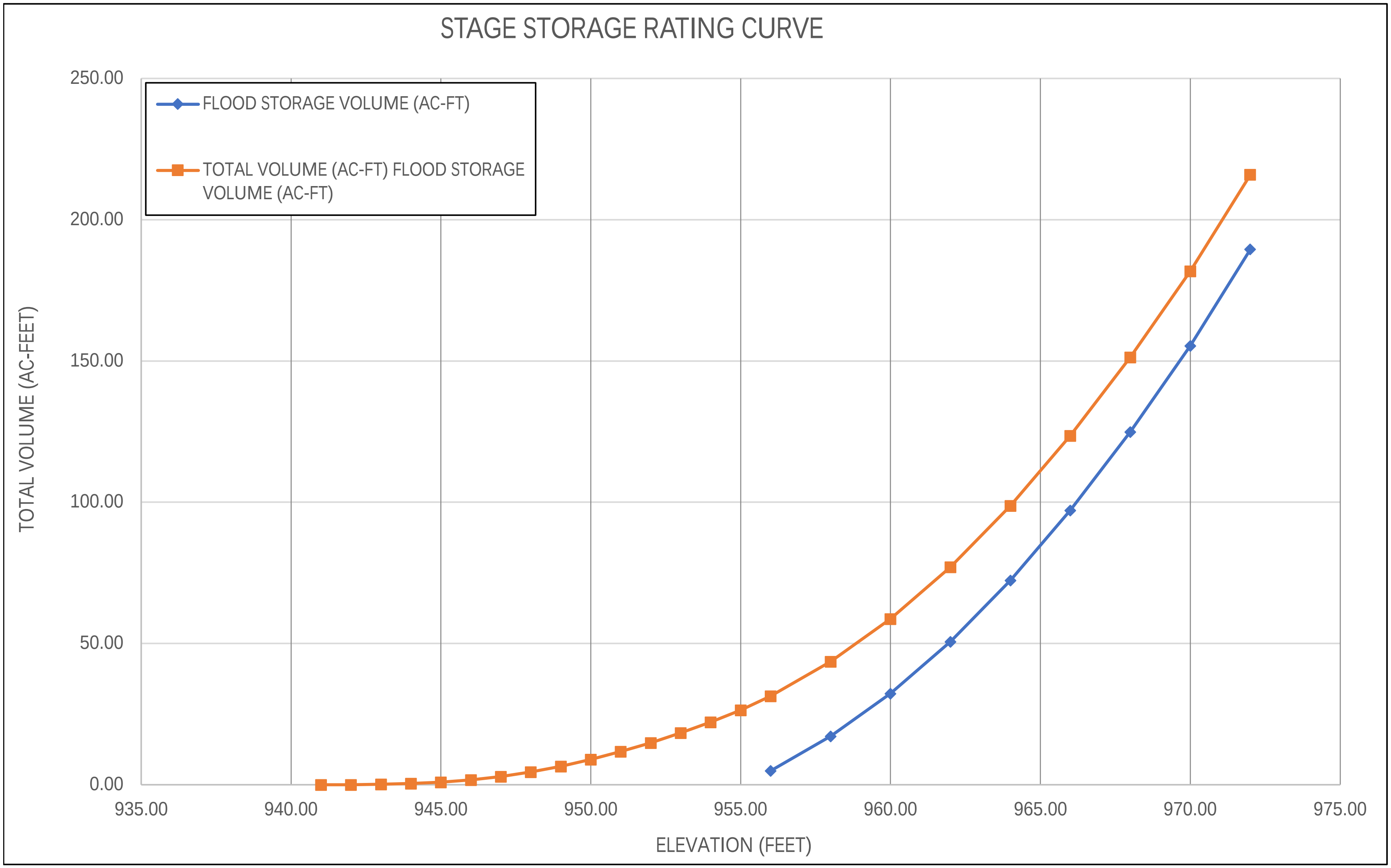
Appendix D - Reservoir Routing Supporting Information

DESIGN ST DATE 24-Mar-2021
CHECK WCH DATE 29-Mar-2021
PROJECT: Erin Lake Dam

STAGE-STORAGE RATING TABLE CALCULATIONS

DATA SOURCE	RATING ELEVATIONS		
		START	941.00'
		END	970.00'
		POOL	955.70'

DATA SOURCE	ELEVATION (FEET)	AREA (AC)	AVERAGE AREA (AC-FT)	INCREMENTAL DEPTH (FT)	INCREMENTAL VOLUME (AC-FT)	TOTAL VOLUME (AC-FT)	FLOOD STORAGE VOLUME (AC-FT)
Accura Survey (March,2021)	941.00	0.01	0.01	0.00	0.00	0.00	
	942.00	0.05	0.03	1.00	0.03	0.03	
	943.00	0.17	0.11	1.00	0.11	0.14	
	944.00	0.37	0.27	1.00	0.27	0.41	
	945.00	0.61	0.49	1.00	0.49	0.90	
	946.00	0.94	0.77	1.00	0.77	1.67	
	947.00	1.41	1.18	1.00	1.18	2.85	
	948.00	1.85	1.63	1.00	1.63	4.48	
	949.00	2.24	2.04	1.00	2.04	6.52	
	950.00	2.61	2.43	1.00	2.43	8.95	
	951.00	2.96	2.79	1.00	2.79	11.74	
	952.00	3.27	3.12	1.00	3.12	14.86	
	953.00	3.62	3.45	1.00	3.45	18.31	
	954.00	4.01	3.82	1.00	3.82	22.12	
	955.00	4.57	4.29	1.00	4.29	26.42	
LiDAR (DeKalb County, February 2011)	956.00	5.42	5.00	1.00	5.00	31.41	5.00
	958.00	6.71	6.07	2.00	12.13	43.55	17.13
	960.00	8.39	7.55	2.00	15.11	58.66	32.24
	962.00	10.04	9.22	2.00	18.43	77.09	50.67
	964.00	11.62	10.83	2.00	21.66	98.75	72.33
	966.00	13.19	12.40	2.00	24.80	123.55	97.14
	968.00	14.56	13.87	2.00	27.75	151.30	124.88
	970.00	15.90	15.23	2.00	30.47	181.77	155.35
	972.00	18.28	17.09	2.00	34.18	215.95	189.53


STAGE STORAGE RATING CURVE
FLOOD STORAGE VOLUME (AC-FT)
TOTAL VOLUME (AC-FT) FLOOD STORAGE VOLUME (AC-FT)
TOTAL VOLUME (AC-FEET)
250.00
200.00
150.00
100.00
50.00
0.00
935.00
940.00
945.00
950.00
955.00
960.00
965.00
970.00
975.00
ELEVATION (FEET)

Erin Lake Dam
Tucker, GA

DESIGN ST DATE 11-Feb-2021
CHECK WCH DATE 12-Feb-2021
PROJECT: Erin Lake Dam

RATING ELEVATIONS

RATING ELEVATIONS		LOW FLOW ORIFICE		SECONDARY ORIFICE		RISER (CORRUGATED METAL PIPE)				BARREL				AUXILIARY SPILLWAY		TOTAL
START	955.70'	HEIGHT	26.59 INCHES	HEIGHT		LENGTH	2.22'	AREA	4.91 SF	RISE	30 INCHES	INV. (IN)	955.70'	CREST ELEV	967.00'	
END	970.00'	WIDTH	26.59 INCHES	WIDTH		WIDTH	2.22'	CREST ELEV	962.12'	SPAN	30 INCHES	INV. (OUT)	954.19'	WIDTH	12.00'	
		INVERT	955.70'	INVERT		CREST LENGTH	7.85'			QUANTITY	1	LENGTH	94.95'	LENGTH	12.00'	
										TW ELEV.	940.00'	MANNINGS	0.015	MANNINGS	0.040	

	LOW FLOW ORIFICE		SECONDARY ORIFICE		FREE WEIR		SUBMERGED WEIR		WEIR AS ORIFICE							
ELEVATION (FEET)	H_O	(1) Q_O	H_O	(1A) Q_O	$H_W (H_1)$	(2) Q_W	$H_{SB}(H_2)$	(3) Q_{SB}	(4) Q_{WO}	H_{PS}	(5) Q_{BIC}	(6) Q_{BOC}	Q_B	H_{AS}	(7) Q_{AS}	(8) Q_{TOTAL}
955.70	0.00	0.0		0.0						0.00	0.0	95.7	0.0			0.0
956.00	0.00	0.0		0.0						0.30	0.5	96.6	0.5			0.0
957.00	0.19	10.4		0.0						1.30	7.4	99.6	7.4			7.4
958.00	1.19	25.8		0.0						2.30	9.3	102.5	9.3			9.3
959.00	2.19	35.0		0.0						3.30	36.3	105.3	36.3			35.0
960.00	3.19	42.2		0.0						4.30	50.4	108.0	50.4			42.2
961.00	4.19	48.4		0.0						5.30	61.4	110.7	61.4			48.4
962.00	5.19	53.9		0.0						6.30	70.7	113.3	70.7			53.9
962.12	5.31	54.5		0.0	0.00	0.0			0.0	6.42	71.8	113.6	71.8			54.5
963.00	6.19	58.8		0.0	0.88	20.1			22.2	7.30	78.9	115.8	78.9			78.9
964.00	7.19	63.4		0.0	1.88	62.8			32.4	8.30	86.4	118.3	86.4			86.4
965.00	8.19	67.6		0.0	2.88	119.0			40.1	9.30	93.2	120.8	93.2			93.2
966.00	9.19	71.7		0.0	3.88	186.1			46.6	10.30	99.6	123.2	99.6			99.6
967.00	10.19	75.5		0.0	4.88	262.5			52.2	11.30	105.6	125.5	105.6	0.00	0.0	105.6
967.50	10.69	77.3		0.0	5.38	303.8			54.8	11.80	108.5	126.7	108.5	0.50	9.1	117.6
968.00	11.19	79.1		0.0	5.88	347.2			57.3	12.30	111.3	127.8	111.3	1.00	34.2	145.5
968.50	11.69	80.8		0.0	6.38	392.4			59.7	12.80	114.0	129.0	114.0	1.50	68.8	182.8
969.00	12.19	82.5		0.0	6.88	439.4			62.0	13.30	116.7	130.1	116.7	2.00	111.3	227.9
969.50	12.69	84.2		0.0	7.38	488.1			64.2	13.80	119.3	131.2	119.3	2.50	162.2	281.5
970.00	13.19	85.8		0.0	7.88	538.6			66.3	14.30	121.8	132.3	121.8	3.00	221.5	343.3
970.50	13.69	87.5		0.0	8.38	590.6			68.4	14.80	124.3	133.4	124.3	3.50	288.1	412.5
971.00	14.19	89.0		0.0	8.88	644.3			70.4	15.30	126.8	134.5	126.8	4.00	362.3	489.0
971.50	14.69	90.6		0.0	9.38	699.5	0.07	699.3	72.4	15.80	129.2	135.6	129.2	4.50	445.8	574.9
972.00	15.19	92.1		0.0	9.88	756.1	0.30	754.6	74.3	16.30	131.5	136.6	131.5	5.00	535.8	667.4

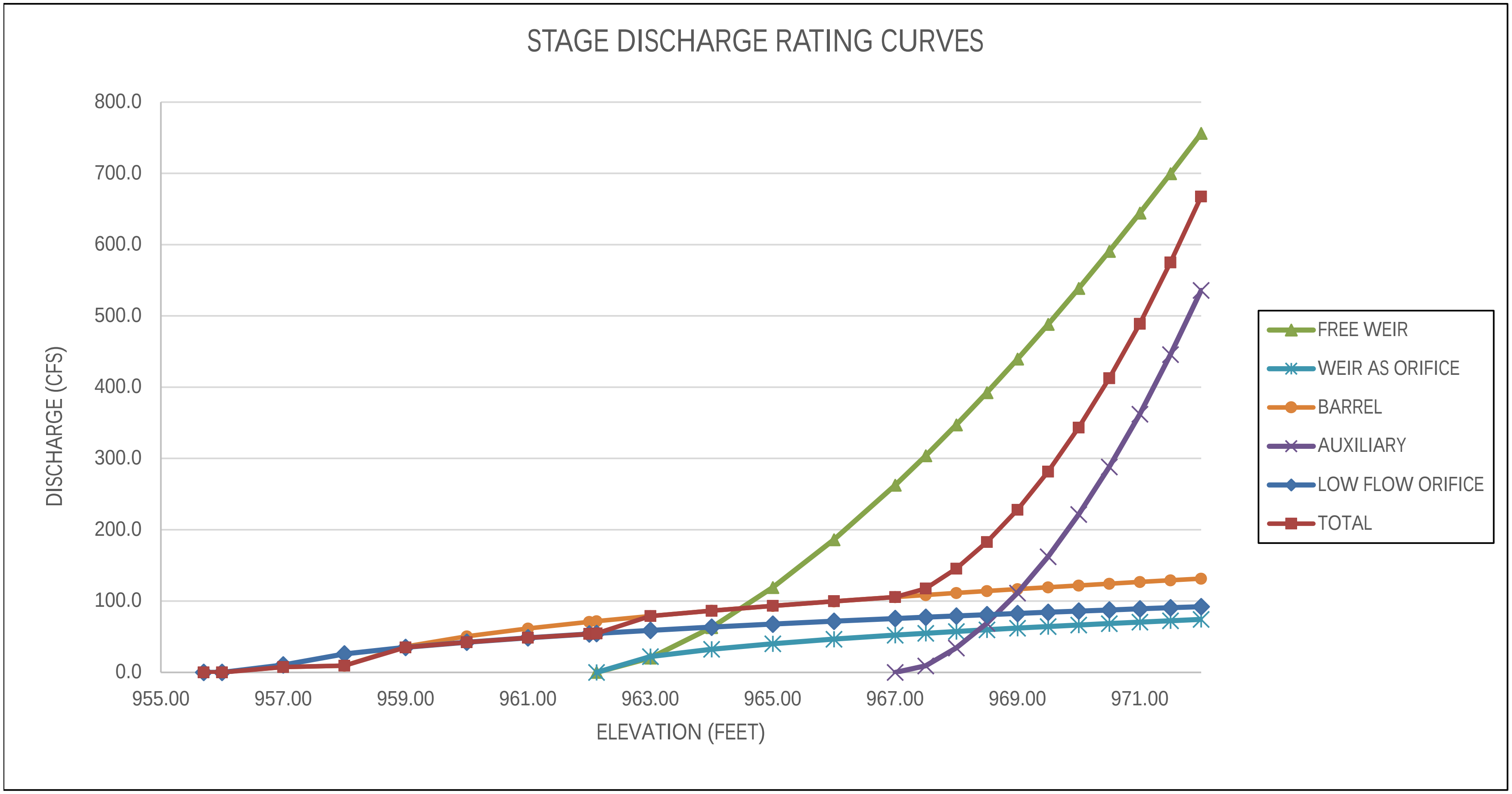
Equations Used:

(1)/(1A) Low Flow Orifice: $Q_O = C_O A \sqrt{2gH_O}$	where:	$C_O = 0.6$, $g = 32.2\ FT/S^2$
(2) Weir (Free Flow): $Q_W = C_W L_W H_W^{3/2}$	where:	$C_W = 3.1$
*(3) Weir (Submerged Flow): $Q_{SB} = Q_W * [1-(H_2/H_1)^{3/2}]^{0.385}$	where:	H_2 = TW over weir (inside of riser), H_1 = HW over weir (U/S of riser) (Brater, "Handbook of Hydraulics, 7th Ed")
(4) Riser (acting as a horizontal orifice): $Q_{WO} = C_O A \sqrt{2gH_O}$	where:	$C_O = 0.6$, $g = 32.2\ FT/S^2$, for $H_W > 0.08$ Max (Riser Length/Width) + 0.35'
(5) Barrel (Unsubmerged Inlet Control): $Q_{BIC} = A \sqrt{(D)}*[(H_O/DK)^{1/M}]$	where:	K = 0.534, M = 0.555 (FHWA, "Hydraulic Design of Highway Culverts, 3rd Ed")
(5) Barrel (Submerged Inlet Control): $Q_{BIC} = A \sqrt{[(H_O - YD - K_S SD)/C]}$	where:	Y = 0.90, C = 0.0196, $K_S = -0.5$ (FHWA, "Hydraulic Design of Highway Culverts, 3rd Ed")
(6) Barrel (Outlet Control): $Q_{BOC} = A \sqrt{[\{2g(H_W - T_W)\}]/[1+K_E+(K_U n^2 L/R^{4/3})]\}}$	where:	$g = 32.2\ FT/S^2$, $K_E = 0.5$, $K_U = 29$, R = Hyd. Radius (FHWA, "Hydraulic Deisgn of Highway Culverts, 3rd Ed")

(7) Auxiliary Spillway: Refer to attached TR-2 spreadsheet if required for model.

(8) Total Flow: Q = MIN (1 + MIN (2, 3, 4), MIN (5, 6)) + 7

*Note: Submerged Weir flow is only calculated and report when there is a tailwater over the riser weir (H_2>0) and when the system is in weir control (B/C denotes barrel control).



AECOM

Erin Lake Dam
Tucker, GA

DESIGN ST DATE 24-Mar-2021
CHECK WCH DATE 29-Mar-2021
PROJECT: Erin Lake Dam

OPEN CHANNEL SPILLWAY COMPUTATIONS

INPUT

BOTTOM WIDTH (b):	12	FEET
SIDE SLOPE (Z):	1.1H:1V	FEET/FEET
LEVEL SECTION LENGTH (L):	12	FEET
MANNING'S COEFFICIENT (n):	0.040	(from *Open Channel Hydraulics* - Chow, 1959)
SPILLWAY CREST ELEVATION:	967.00	FEET

Y_C	A_C	T	Q_C	V_C	H_{EC}	a	H_P	R	S_C	ELEV
FEET	SF	FEET	CFS	FPS	FEET		FEET	FEET	FT/FT	FEET
0.00	0.00	0.00	0.0	0.00	0.00	0.00000	0.00	0.00	0.0000	967.00
0.26	3.19	12.57	9.1	2.86	0.39	0.02451	0.50	0.25	0.0376	967.50
0.62	7.86	13.36	34.2	4.35	0.91	0.00779	1.00	0.57	0.0291	968.00
0.98	12.76	14.15	68.8	5.39	1.43	0.00430	1.50	0.86	0.0258	968.50
1.33	17.91	14.93	111.3	6.22	1.93	0.00288	2.00	1.12	0.0240	969.00
1.69	23.42	15.72	162.2	6.93	2.44	0.00211	2.50	1.38	0.0227	969.50
2.06	29.31	16.52	221.5	7.56	2.94	0.00164	3.00	1.62	0.0218	970.00
2.42	35.48	17.32	288.1	8.12	3.44	0.00133	3.50	1.85	0.0210	970.50
2.79	41.96	18.13	362.3	8.63	3.94	0.00111	4.00	2.07	0.0205	971.00
3.16	48.90	18.95	445.8	9.12	4.45	0.00094	4.50	2.29	0.0200	971.50
3.53	56.07	19.77	535.8	9.56	4.95	0.00082	5.00	2.49	0.0196	972.00

where:

(1) $Q_C = \sqrt{(gA^3/T)}$

(2) $A_C = (b + ZY_C)Y_C$

(3) $V_C = \sqrt{(gA/T)}$

(4) $T = b + 2ZY_C$

(5) $H_{EC} = Y_C + V_C^2/2g$

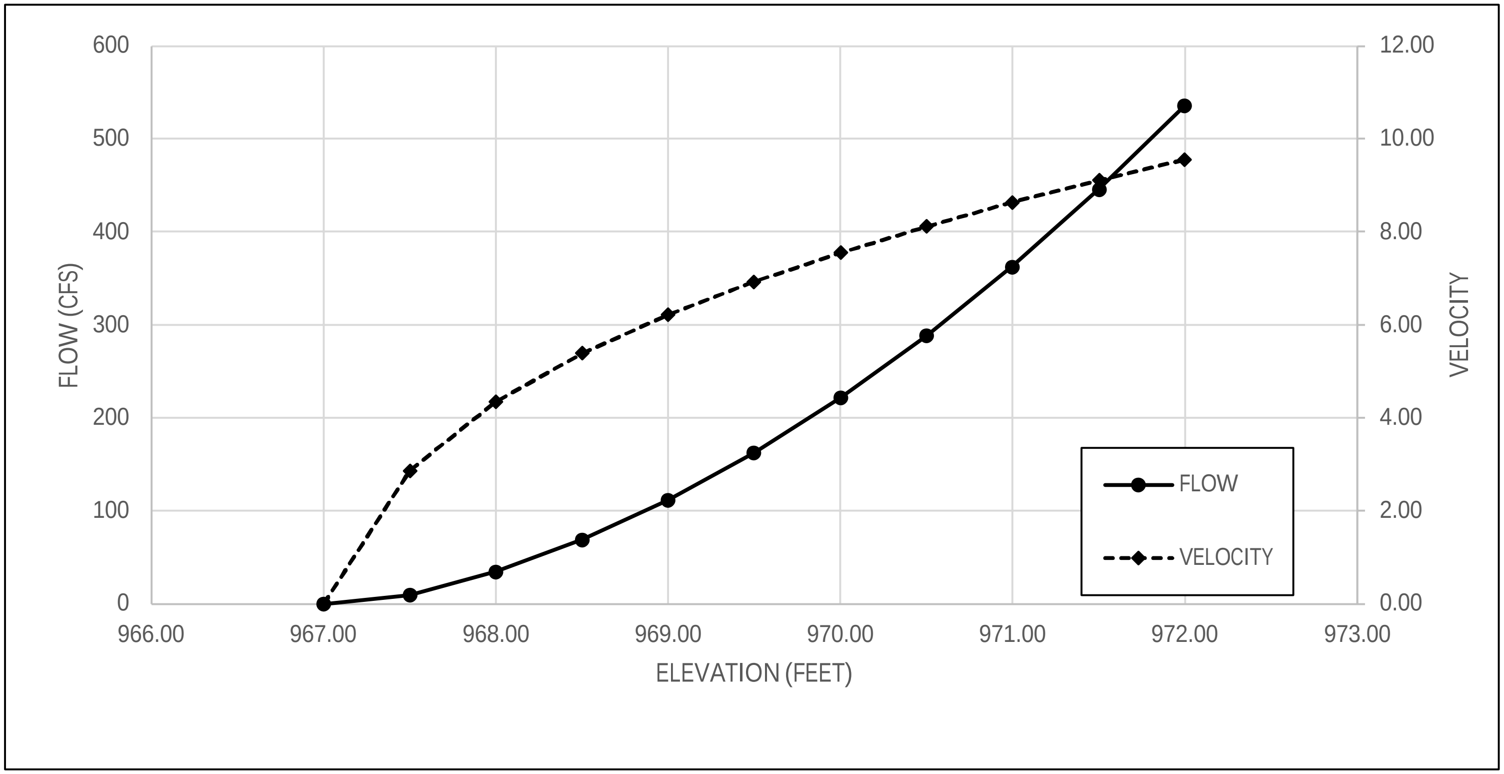
(6) $H_P = H_{EC}(1 + aL)$

(7) $a = (4.32n^2)/H_{EC}^{4/3}$

(8) $S_C = 14.56n^2A_C/(R^{4/3})T$

(9) $R = (b + ZY_C)Y_C/(b + 2Y_C\sqrt{(1 + Z^2)})$

Obtained from TR-2 (SCS, 1956) & Handbook of Hydraulics 7th Ed. (Brater, King, 1996).


600
500
400
300
200
100
0
FLOW (CFS)
12.00
10.00
8.00
6.00
4.00
2.00
0.00
VELOCITY
966.00
967.00
968.00
969.00
970.00
971.00
972.00
973.00
ELEVATION (FEET)
FLOW
VELOCITY

Appendix E - Dam Breach Analysis Supporting Information

ESTIMATION OF DAM BREACH PARAMETERS
USING THE FROEHLICH 2008 METHOD

PROJECT: **Erin Lake**

BREACH INPUT PARAMETERS:

Select Failure Mode From Drop-Down Menu:	**OVERTOPPING**	
Height of water over base elevation of breach (H_w) =	**27.5**	**Feet**
Volume of water in the reservoir at the time of failure (V_w) =	**158.9**	**Acre-Feet**
Reservoir Surface Area at Hw (A_s) =	**14.9**	**Acres**
Height of breach (H_b) =	**27.5**	**Feet**
Failure Mode Factor (K_o) =	**1.3**	
Breach Side-Slope Ratio (Z_b) =	**1**	**Z(H):1(V)**
Dam Size Class:	**Small**	**Assumes Full Reservoir At Time of Breach.**

CALCULATED BREACH CHARACTERISTICS:

Average Breach Width (B_{avg}) =	**61.9**	**Feet**
Bottom Width of Breach (B_b) =	**34.4**	**Feet**
Breach Formation Time (T_f) =	**0.30**	**Hours**
Storage Intensity (SI) =	**5.8**	**Acre Feet/Foot**
Predicted Peak Flow (Q_p) =	**13334**	**Cubic Feet per Second**

RESULTS CHECK:

Average Breach Width Divided by Height of Breach (B_{avg}/H_b) =	**2.25**	**If (B_{avg}/H_b) > 0.6, Full Breach Devlopment is Anticipated**
Erosion Rate (ER), Calculated as (B_{avg}/T_f) =	**209.2**	
Erosion Rate Divided by Height of Water Over Base of Breach (ER/H_w) =	**7.6**	**If 1.6 < (ER/H_w) < 21, Erosion Rate is Assumed Reasonable**

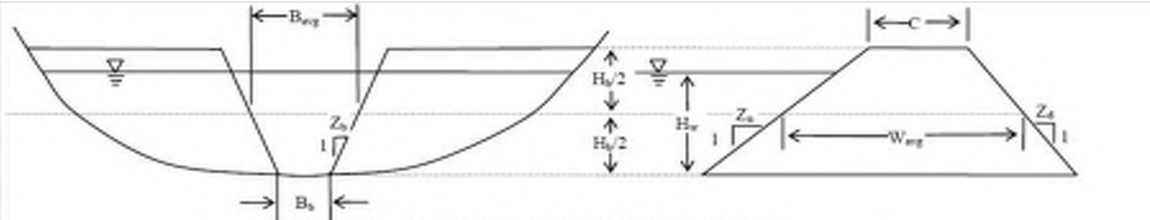
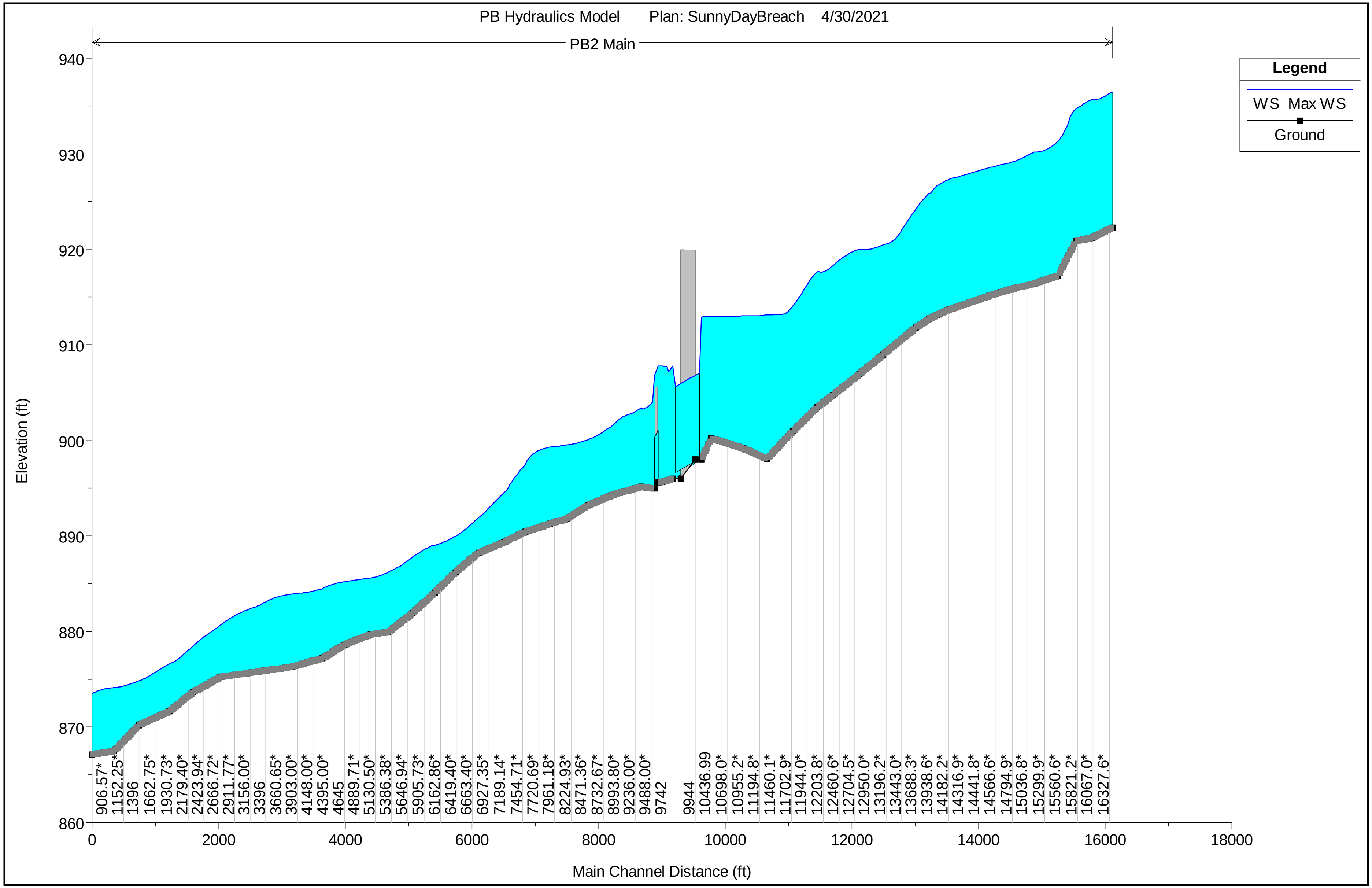


Figure 1- Breach Variable Definition Sketch

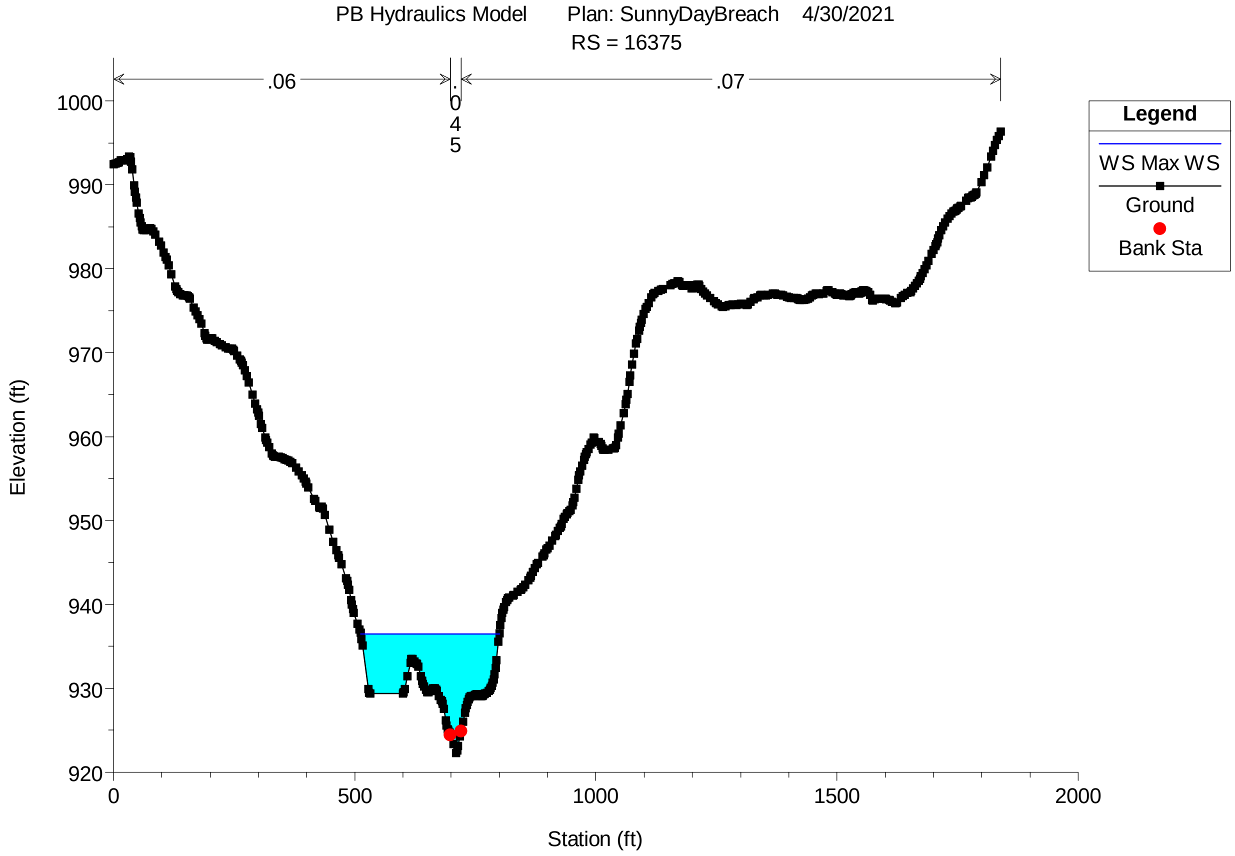

PB Hydraulics Model Plan: SunnyDayBreach 4/30/2021
PB2 Main
Legend
WS Max WS
Ground
Elevation (ft)
940
930
920
910
900
890
880
870
860
Main Channel Distance (ft)
0
2000
4000
6000
8000
10000
12000
14000
16000
18000
906.57*
1152.25*
1396
1662.75*
1930.73*
2179.40*
2423.94*
2666.72*
2911.77*
3156.00*
3396
3660.65*
3903.00*
4148.00*
4395.00*
4645
4889.71*
5130.50*
5386.38*
5646.94*
5905.73*
6162.86*
6419.40*
6663.40*
6927.35*
7189.14*
7454.71*
7720.69*
7961.18*
8224.93*
8471.36*
8732.67*
8993.80*
9236.00*
9488.00*
9742
9944
10436.99
10698.0*
10955.2*
11194.8*
11460.1*
11702.9*
11944.0*
12203.8*
12460.6*
12704.5*
12950.0*
13196.2*
13443.0*
13688.3*
13938.6*
14182.2*
14316.9*
14441.8*
14566.6*
14794.9*
15036.8*
15299.9*
15560.6*
15821.2*
16067.0*
16327.6*

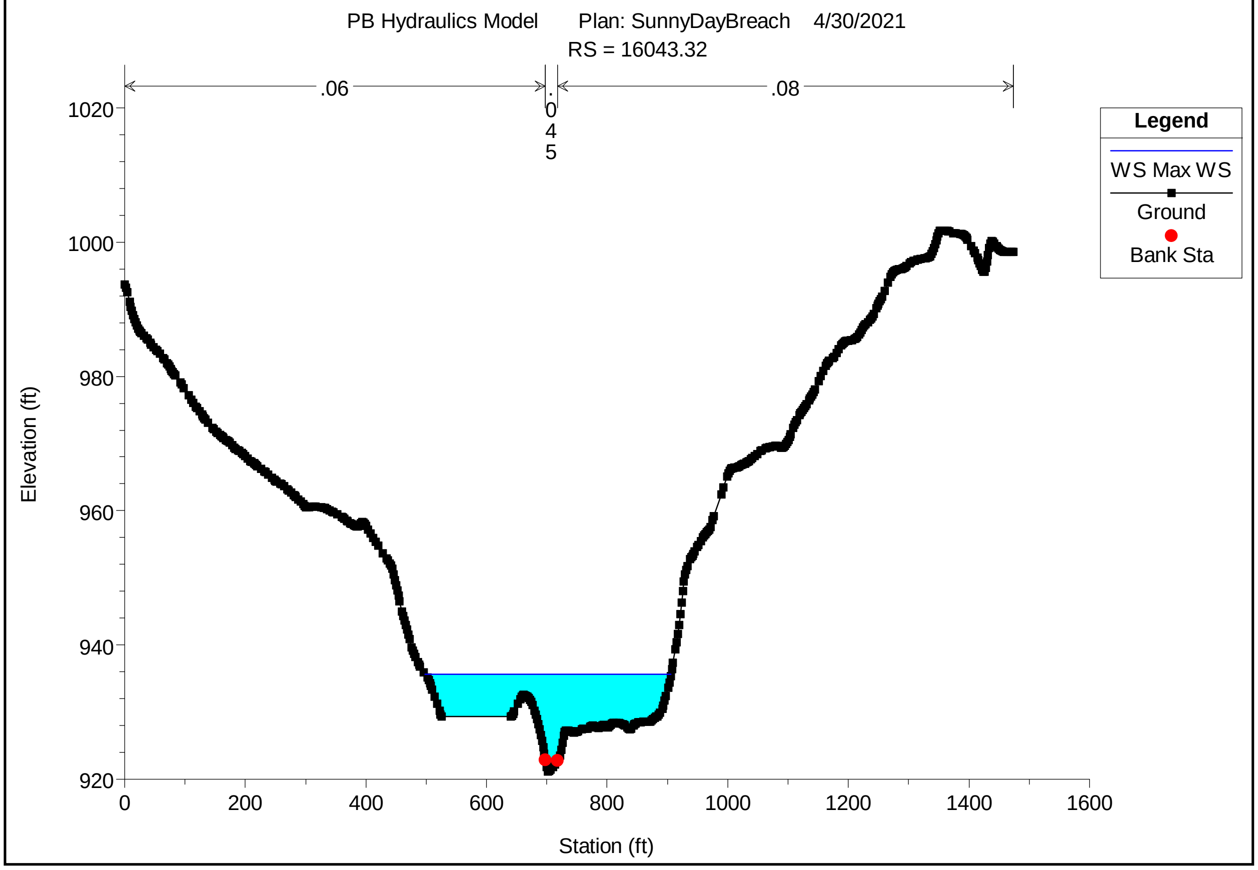
HEC-RAS Plan: SunnyDay River: PB2 Reach: Main Profile: Max WS

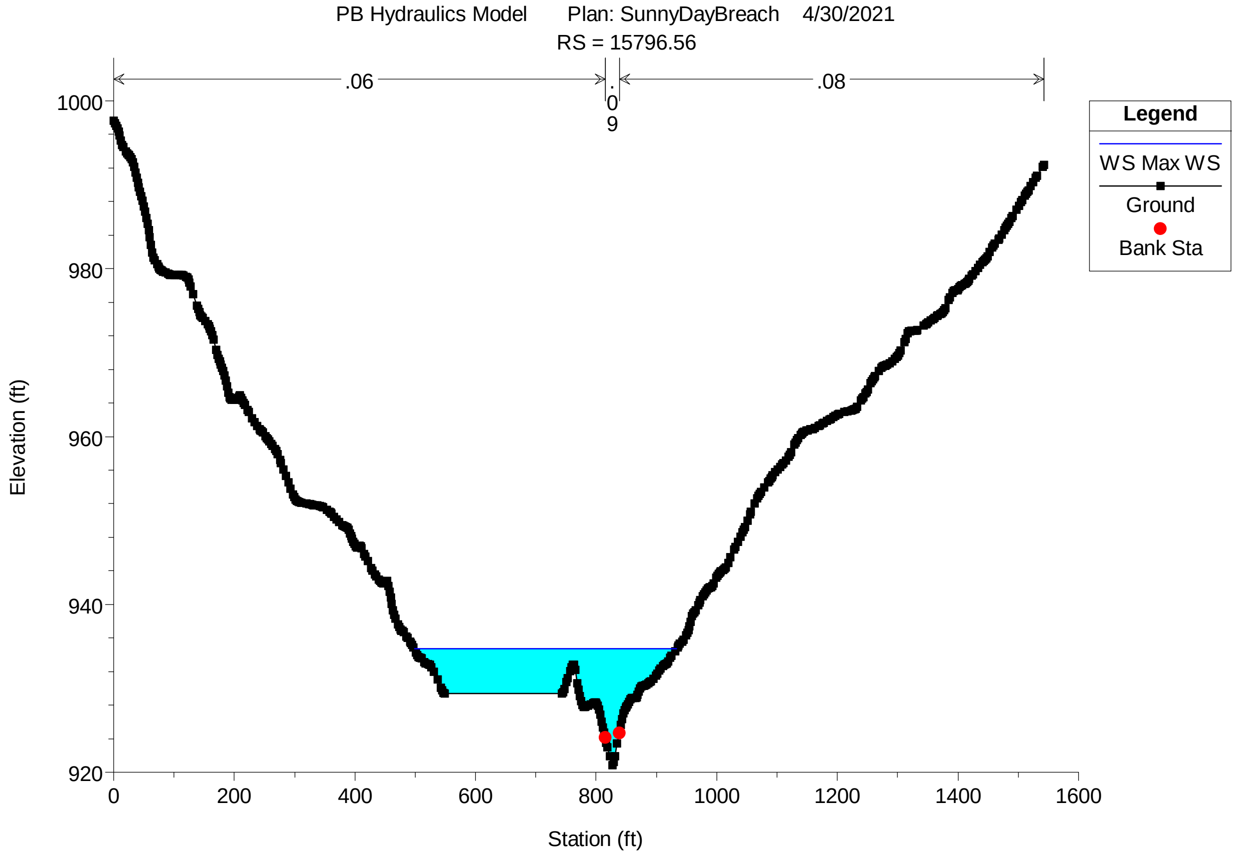
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Main	16375	Max WS	10761.39	922.28	936.48		937.12	0.003039	9.94	2036.45	287.58	0.48
Main	16043.32	Max WS	10579.53	921.18	935.66		936.02	0.001977	8.32	2805.29	407.91	0.40
Main	15796.56	Max WS	10409.67	920.87	934.68		935.04	0.004595	5.74	2231.26	434.92	0.29
Main	15513.48	Max WS	10121.38	917.21	931.32		932.91	0.007075	14.04	1436.50	243.26	0.71
Main	15133.91	Max WS	9521.73	916.40	930.17		930.62	0.002407	8.46	2115.80	316.55	0.42
Main	14842.71	Max WS	8775.52	915.93	929.23		929.78	0.003105	9.09	1817.17	287.58	0.47
Main	14604.07	Max WS	8171.63	915.52	928.86		929.14	0.001848	6.89	2314.97	380.62	0.36
Main	14204.66	Max WS	7130.78	913.72	927.37		927.71	0.001948	7.21	1909.26	293.58	0.37
Main	14002.74	Max WS	7064.64	913.15	926.67		927.30	0.002776	9.06	1662.16	323.30	0.46
Main	13874.59	Max WS	7046.54	912.72	925.84		927.24	0.004548	11.70	1035.19	197.69	0.59
Main	13665.05	Max WS	7004.22	911.79	924.20		926.17	0.009776	15.18	950.66	206.37	0.82
Main	13146.93	Max WS	6017.07	908.95	920.48		921.11	0.003561	9.01	1332.50	270.56	0.50
Main	12777.75	Max WS	5809.15	906.95	919.98		920.11	0.000704	4.51	2312.96	347.79	0.23
Main	12363.13	Max WS	5536.72	904.75	918.31		919.01	0.005225	7.80	852.26	137.44	0.40
Main	12112.81	Max WS	5472.80	903.48	917.65		917.95	0.001576	6.97	1970.73	330.83	0.34
Main	11727.17	Max WS	5405.68	900.96	914.07		916.65	0.007892	13.91	532.42	81.70	0.74
Main	11314.57	Max WS	3041.72	898.07	913.13		913.48	0.000996	5.52	875.60	112.22	0.27
Main	10931.33	Max WS	2973.77	899.20	913.03		913.12	0.000454	3.62	1584.76	252.15	0.18
Main	10628.11	Max WS	2953.02	899.81	912.96		912.99	0.000211	2.43	2721.76	411.72	0.12
Main	10436.99	Max WS	2950.97	900.22	912.96		912.96	0.000041	1.09	4592.66	595.62	0.06
Main	10282 D	Max WS	2950.16	897.99	912.89		912.99	0.000303	3.24	1371.65	468.82	0.15
Main	9944 Interstate 285		Culvert									
Main	9829	Max WS	2948.48	896.02	907.78		908.75	0.003163	8.04	419.29	59.69	0.45
Main	9742	Max WS	2948.16	895.80	907.69		908.31	0.002742	7.52	705.21	167.05	0.41
Main	9666	Max WS	2948.01	895.67	907.77		908.02	0.001317	5.37	1183.58	250.96	0.29
Main	9620 C	Max WS	2946.59	895.62	907.81		907.91	0.000375	3.02	1659.25	323.41	0.16
Main	9567 Henderson Mill R		Culvert									
Main	9510	Max WS	2944.53	894.98	903.99		905.22	0.005624	8.93	343.76	59.78	0.58
Main	9334	Max WS	2936.02	895.14	903.43		903.97	0.004218	7.36	736.73	202.84	0.50
Main	9089	Max WS	2930.76	894.70	902.63		902.99	0.002718	5.98	884.73	209.51	0.41
Main	8851	Max WS	2920.37	894.21	901.41		902.16	0.006399	8.47	614.51	170.05	0.61
Main	8496	Max WS	2779.19	893.21	900.09		900.30	0.003051	5.48	993.73	320.40	0.41
Main	8151	Max WS	2714.24	891.79	899.52		899.58	0.001020	3.40	1738.36	534.93	0.24
Main	7890 B	Max WS	2707.37	891.27	899.30		899.35	0.000749	3.04	1562.06	379.88	0.21
Main	7503	Max WS	2704.91	890.45	897.53		898.42	0.010034	9.77	480.84	141.46	0.73
Main	7165	Max WS	2702.12	889.35	894.53		894.77	0.006981	6.93	939.04	437.93	0.57
Main	6761	Max WS	2694.70	888.22	891.90		892.09	0.006104	5.71	971.38	438.76	0.55
Main	6395	Max WS	2667.91	886.19	889.98		890.08	0.004038	4.64	1214.15	523.30	0.45
Main	6070	Max WS	2646.62	884.04	889.05		889.13	0.001849	3.71	1468.49	544.64	0.31
Main	5718	Max WS	2629.83	881.92	887.78		887.96	0.005055	4.71	841.85	434.68	0.39
Main	5339	Max WS	2504.18	879.94	886.21		886.46	0.003988	5.30	1105.66	551.66	0.46
Main	5061	Max WS	2300.39	879.70	885.60		885.67	0.001456	3.31	1755.88	699.79	0.28
Main	4645	Max WS	2208.85	878.62	885.20		885.25	0.000811	2.89	1751.80	652.60	0.22

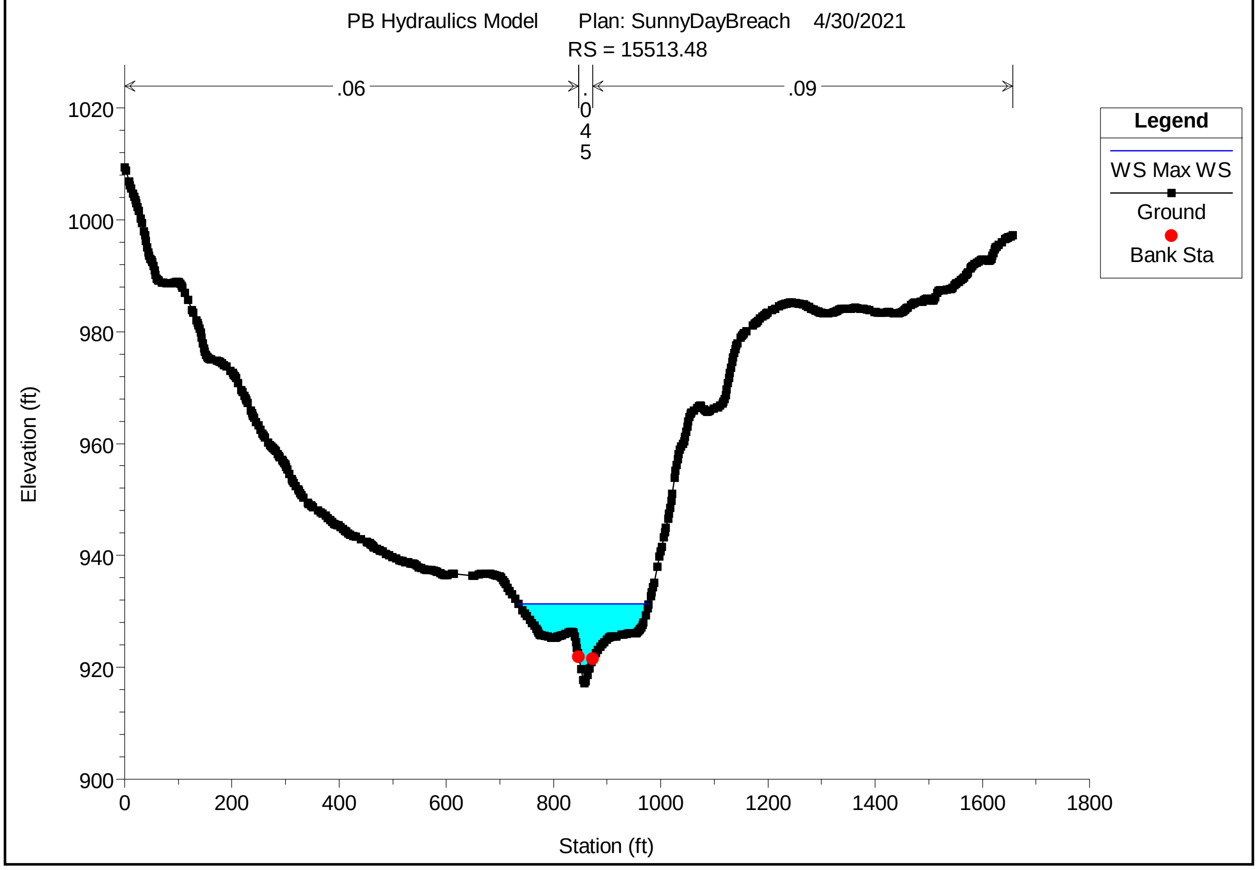
HEC-RAS Plan: SunnyDay River: PB2 Reach: Main Profile: Max WS (Continued)

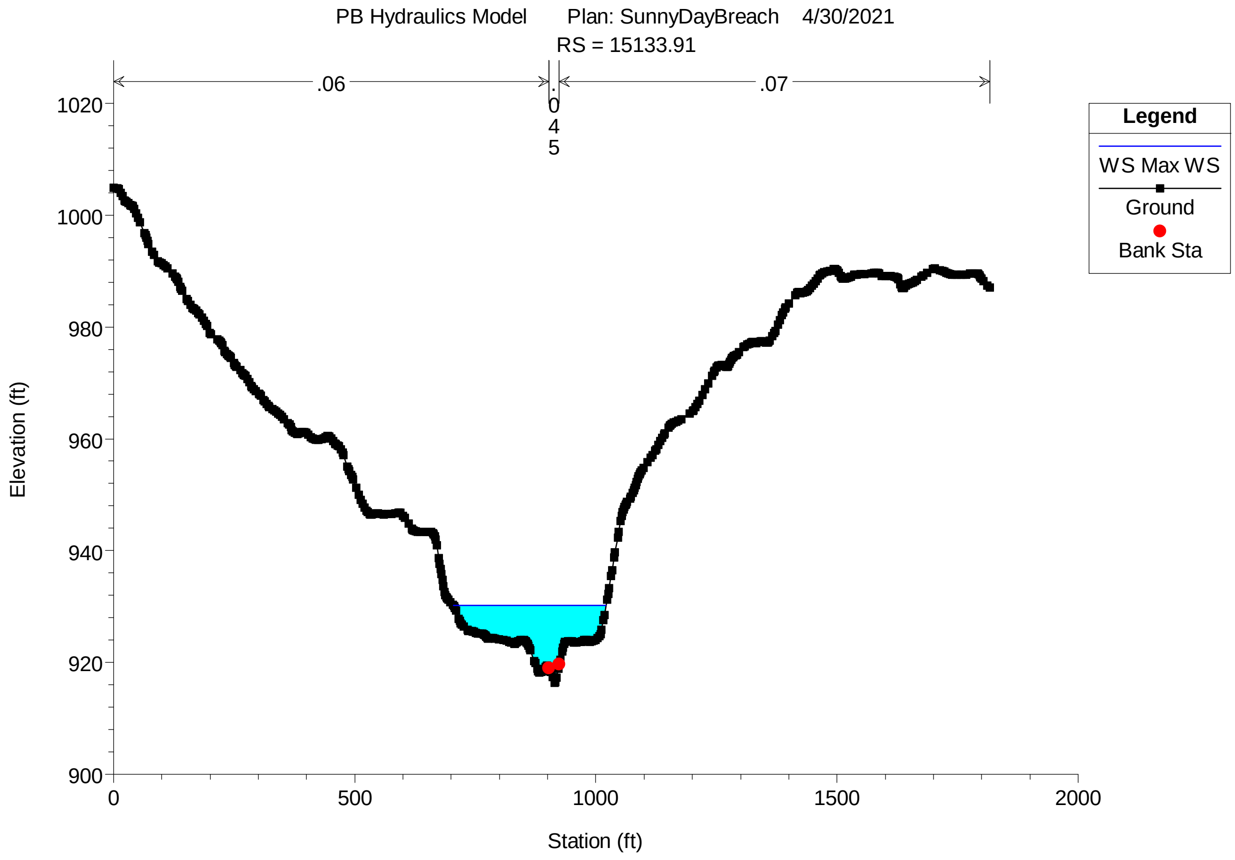
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Main	4295	Max WS	2142.35	877.19	884.47		884.77	0.002392	5.64	772.64	217.22	0.39
Main	3805	Max WS	2110.30	876.29	883.91		883.98	0.000746	2.81	1519.99	389.95	0.21
Main	3396 A	Max WS	2100.37	875.91	883.10		883.39	0.003348	6.21	764.22	205.21	0.42
Main	3108	Max WS	2095.18	875.64	882.26		882.51	0.002460	4.89	715.18	198.12	0.37
Main	2691	Max WS	2091.90	875.25	880.67		880.95	0.005649	6.33	639.26	197.37	0.53
Main	2254	Max WS	2088.72	873.65	878.49		878.80	0.006553	6.85	760.63	440.50	0.57
Main	1881	Max WS	2078.98	871.64	876.61		876.78	0.003759	4.74	958.29	559.56	0.44
Main	1396	Max WS	1985.74	870.19	874.82		874.94	0.002685	4.12	1009.01	455.23	0.37
Main	1006	Max WS	1919.37	867.48	874.13		874.18	0.000707	2.64	1994.08	931.28	0.20
Main	658	Max WS	1901.12	867.14	873.48	872.27	873.75	0.004088	5.85	820.13	515.96	0.47

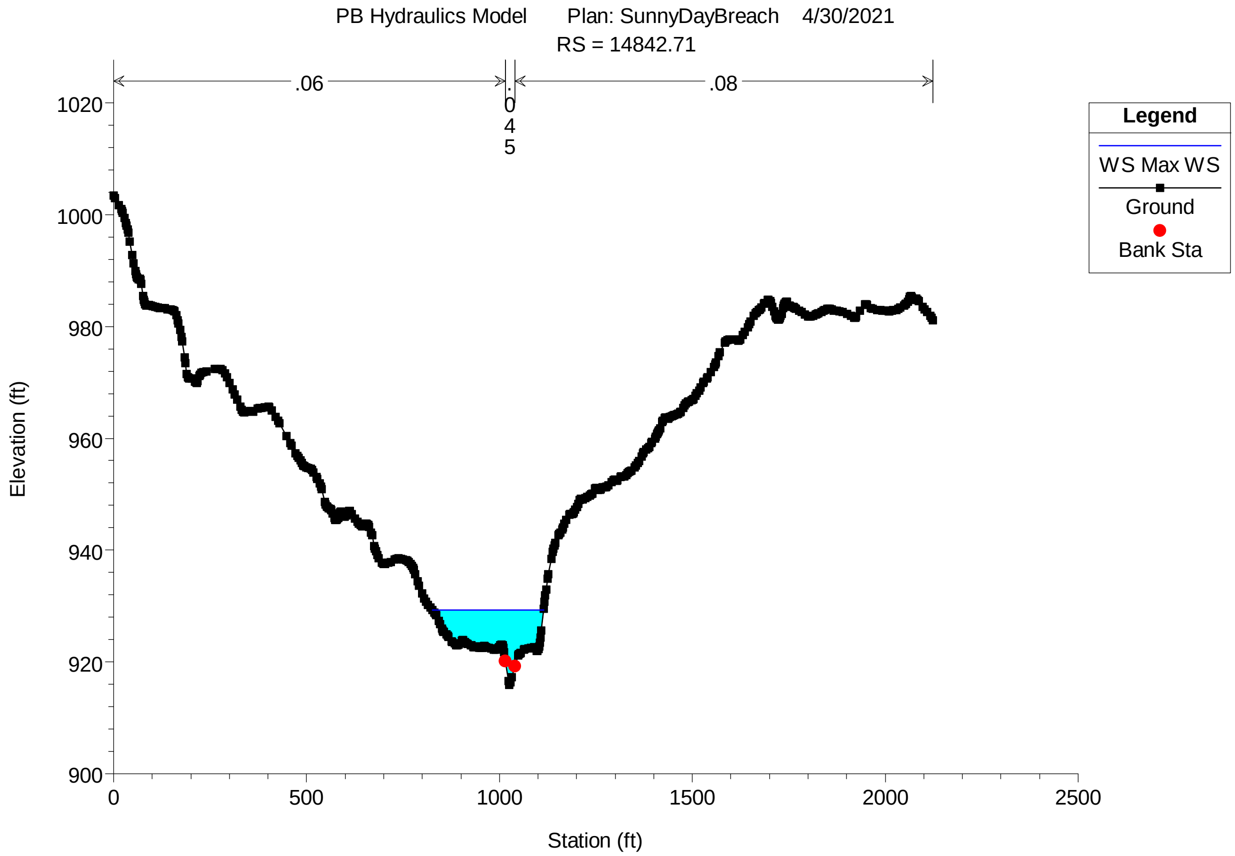

PB Hydraulics Model Plan: SunnyDayBreach 4/30/2021
RS = 16375
.06
.045
.07
Elevation (ft)
1000
990
980
970
960
950
940
930
920
0
500
1000
1500
2000
Station (ft)
Legend
WS Max WS
Ground
Bank Sta

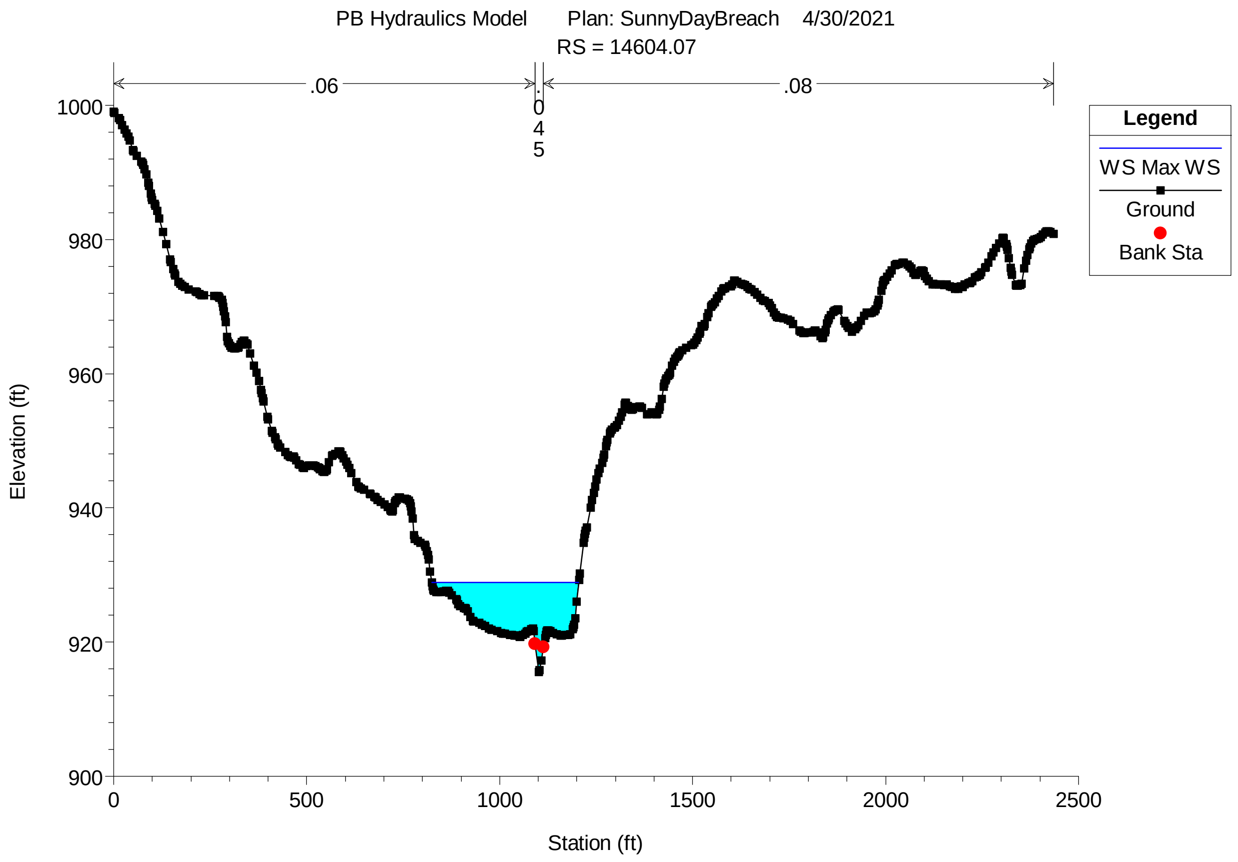

PB Hydraulics Model Plan: SunnyDayBreach 4/30/2021
RS = 16043.32
.06
.045
.08
Elevation (ft)
1020
1000
980
960
940
920
0
200
400
600
800
1000
1200
1400
1600
Station (ft)
Legend
WS Max WS
Ground
Bank Sta


PB Hydraulics Model Plan: SunnyDayBreach 4/30/2021
RS = 15796.56

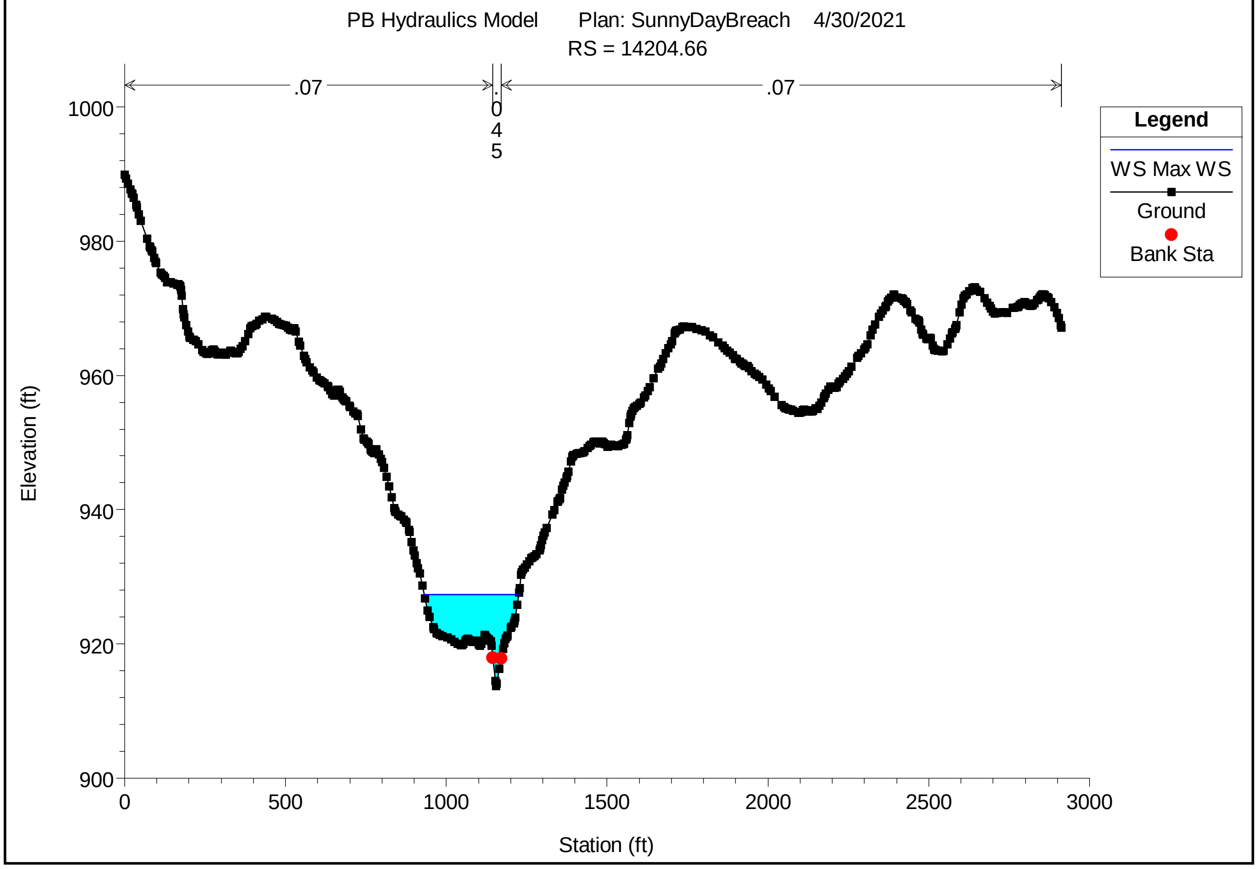

PB Hydraulics Model Plan: SunnyDayBreach 4/30/2021
RS = 15513.48


PB Hydraulics Model Plan: SunnyDayBreach 4/30/2021
RS = 15133.91
.06
.045
.07
Legend
WS Max WS
Ground
Bank Sta
Elevation (ft)
1020
1000
980
960
940
920
900
0
500
1000
1500
2000
Station (ft)

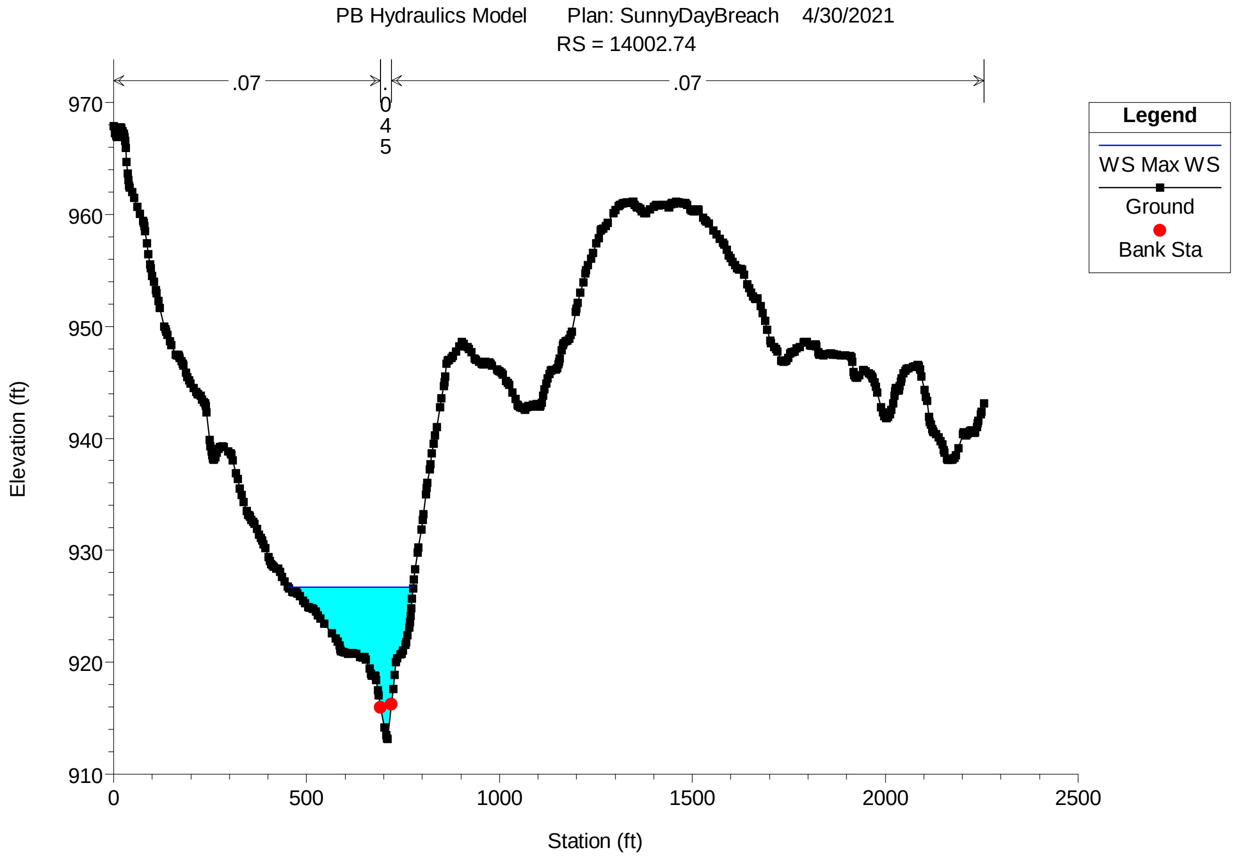

PB Hydraulics Model Plan: SunnyDayBreach 4/30/2021
RS = 14842.71
.06
.045
.08
Legend
WS Max WS
Ground
Bank Sta
Elevation (ft)
1020
1000
980
960
940
920
900
0
500
1000
1500
2000
2500
Station (ft)

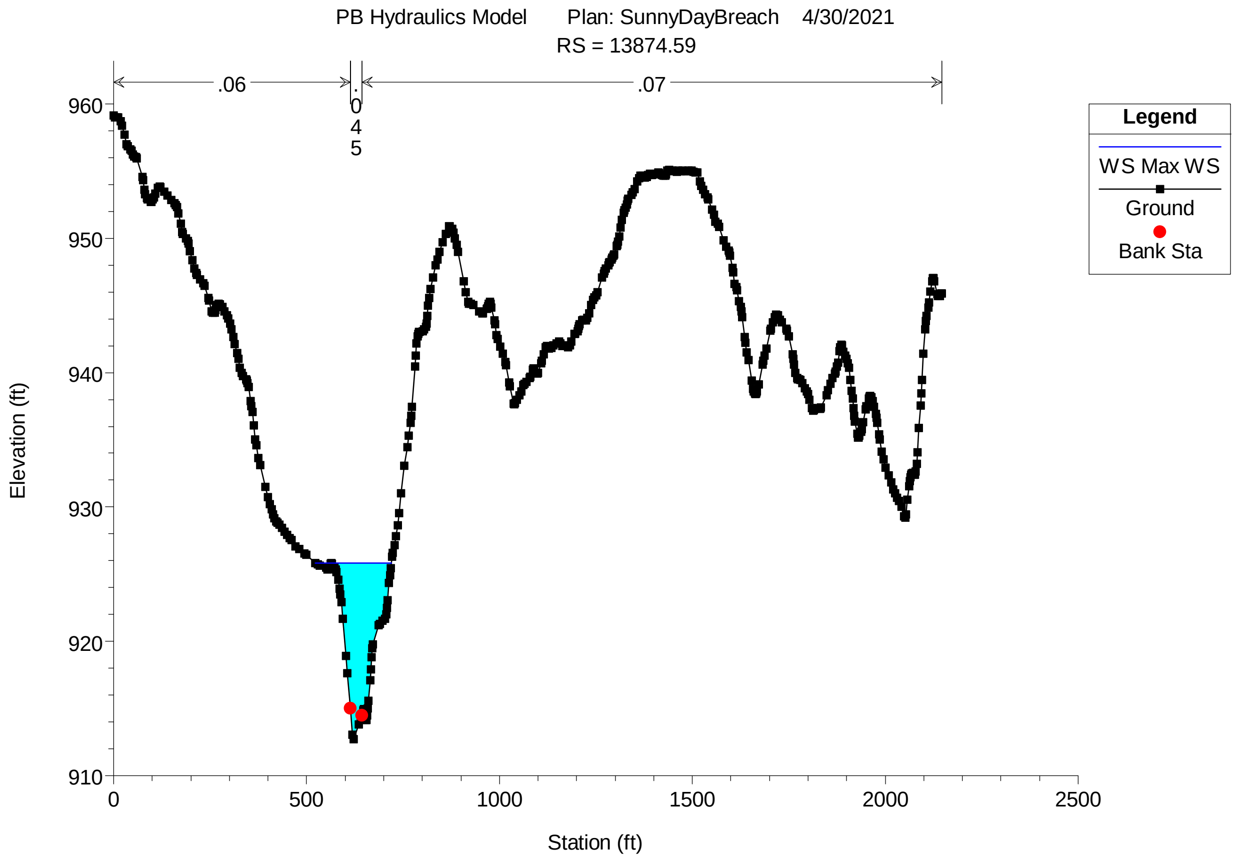


PB Hydraulics Model Plan: SunnyDayBreach 4/30/2021
RS = 14604.07
.06
.
0
4
5
.08
Legend
WS Max WS
Ground
Bank Sta
Elevation (ft)
1000
980
960
940
920
900
0
500
1000
1500
2000
2500
Station (ft)

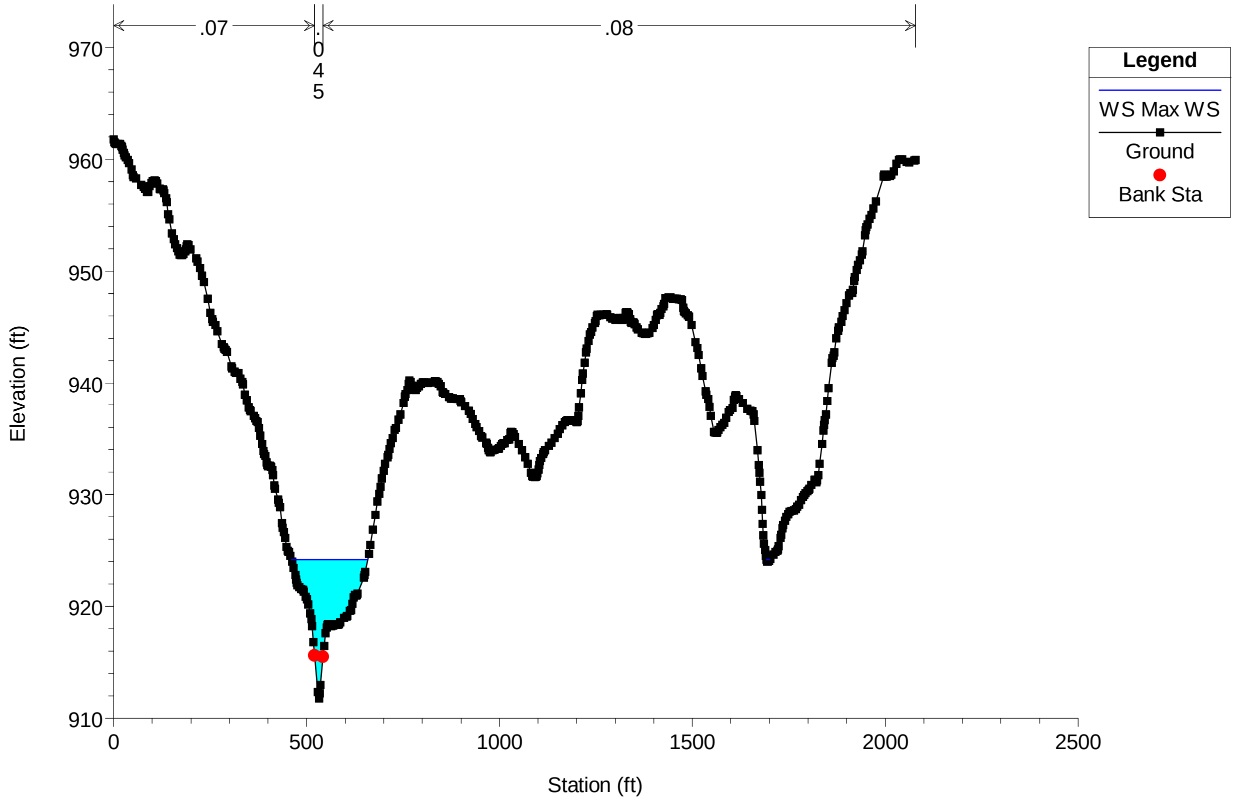


PB Hydraulics Model Plan: SunnyDayBreach 4/30/2021
RS = 14204.66
.07
.
0
4
5
.07
Legend
WS Max WS
Ground
Bank Sta
Elevation (ft)
1000
980
960
940
920
900
0
500
1000
1500
2000
2500
3000
Station (ft)

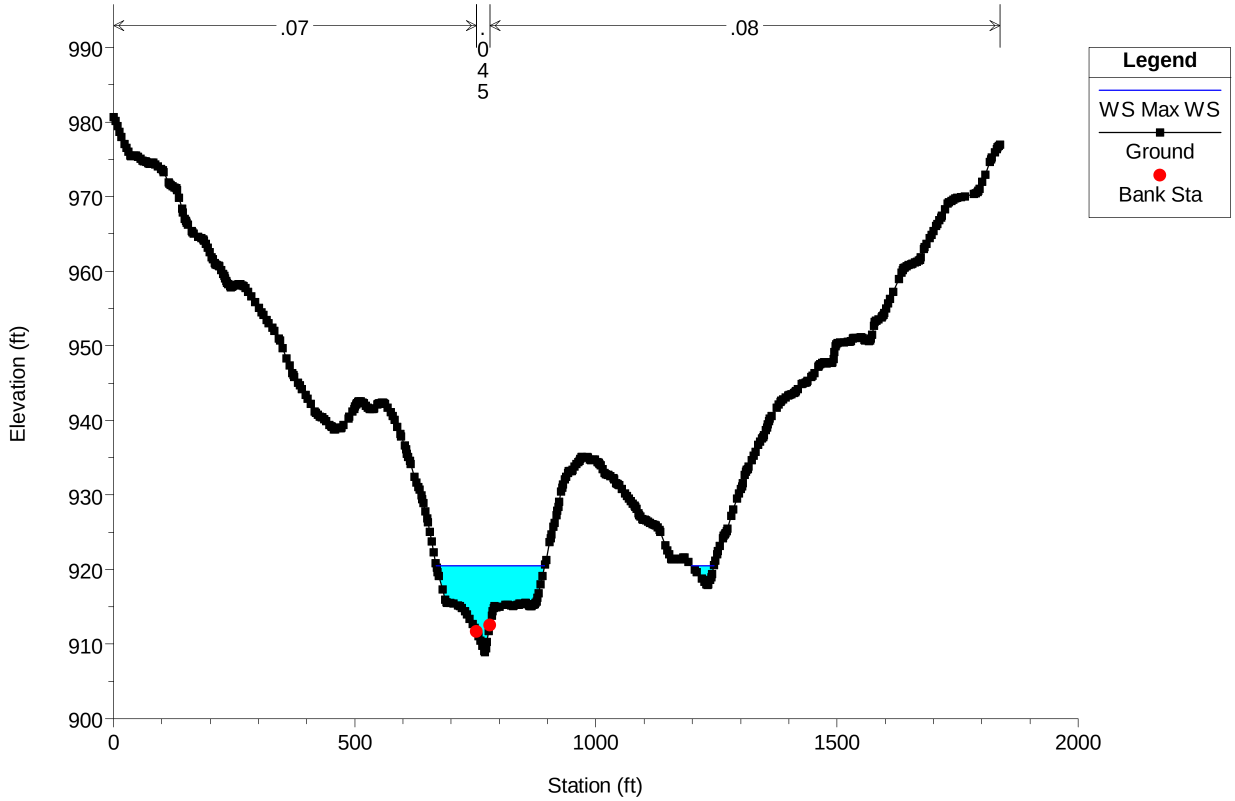

PB Hydraulics Model Plan: SunnyDayBreach 4/30/2021
RS = 14002.74
.07
.045
.07
Legend
WS Max WS
Ground
Bank Sta
Elevation (ft)
970
960
950
940
930
920
910
0
500
1000
1500
2000
2500
Station (ft)

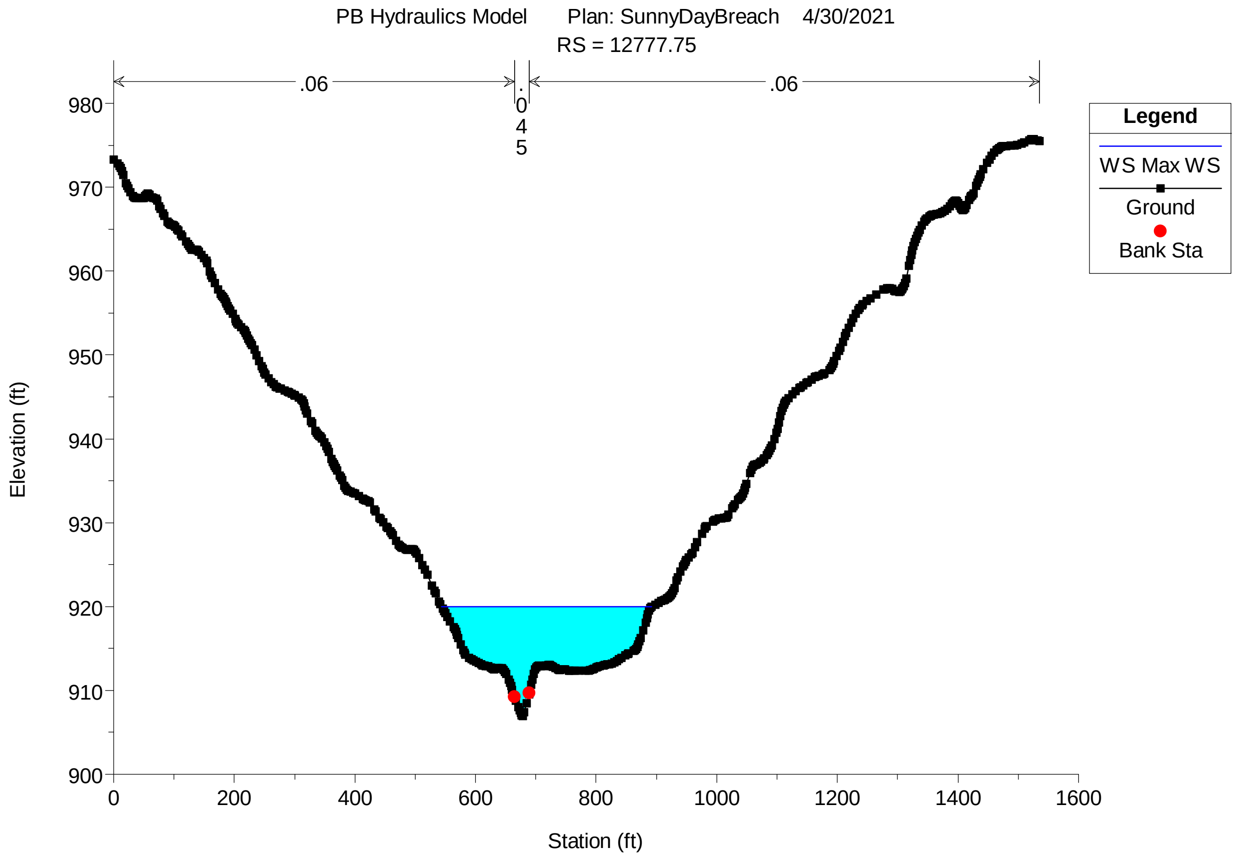

PB Hydraulics Model Plan: SunnyDayBreach 4/30/2021
RS = 13874.59
.06
.045
.07
Legend
WS Max WS
Ground
Bank Sta
Elevation (ft)
960
950
940
930
920
910
0
500
1000
1500
2000
2500
Station (ft)

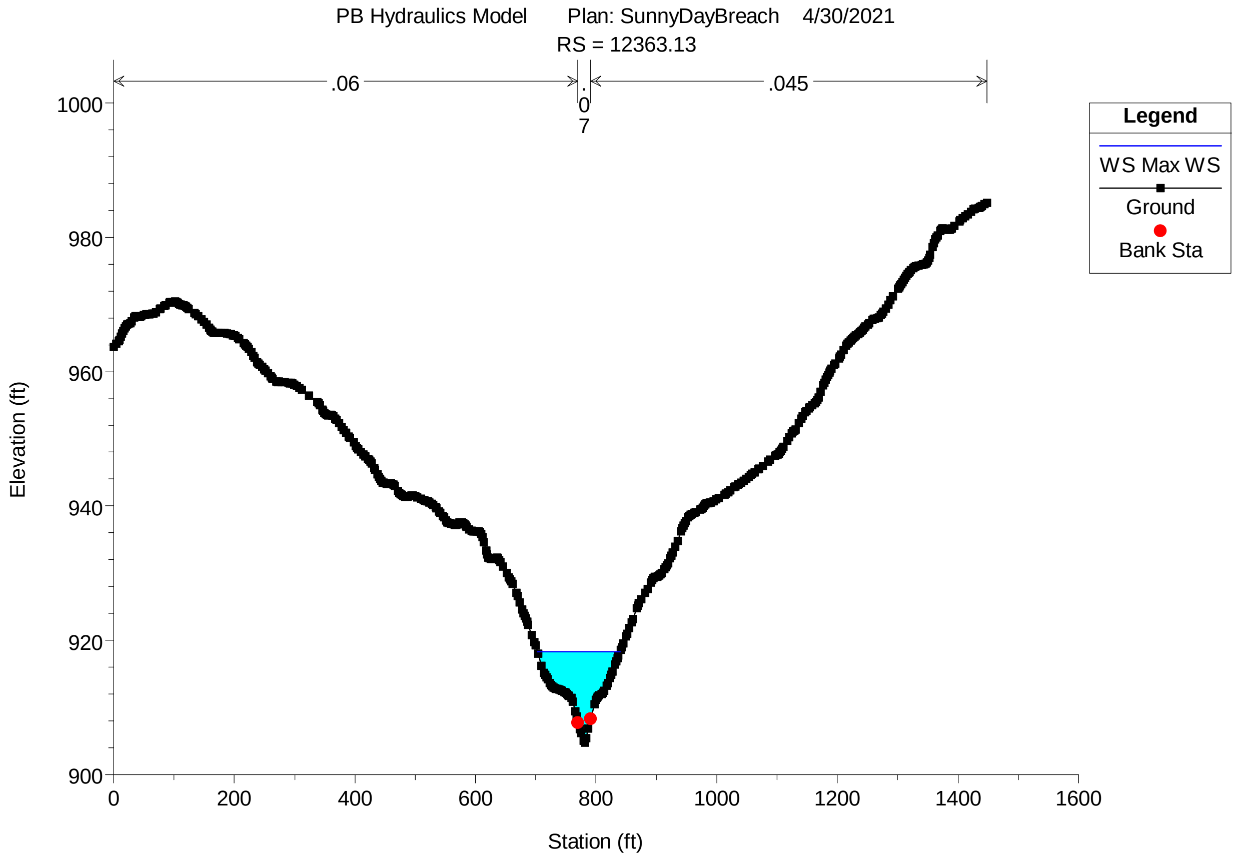
PB Hydraulics Model   Plan: SunnyDayBreach   4/30/2021
RS = 13665.05

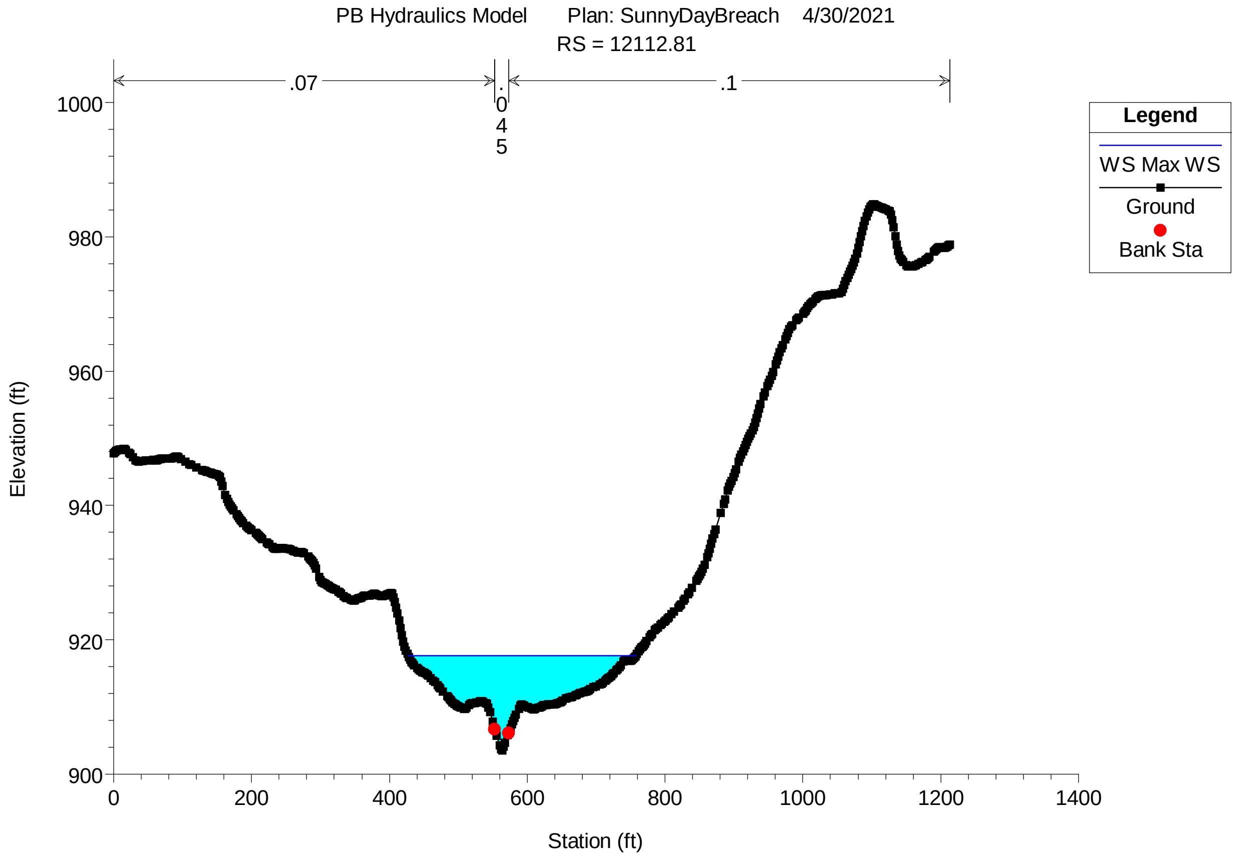


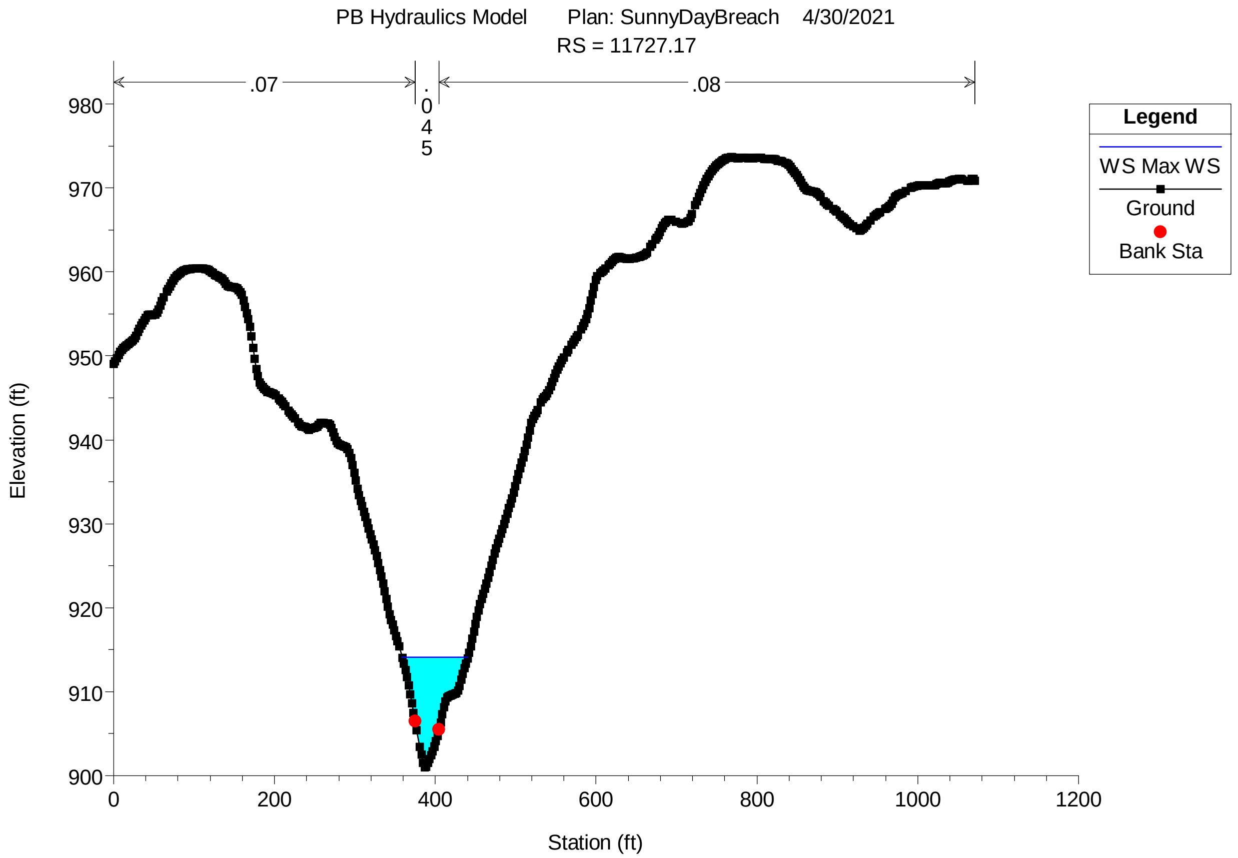
PB Hydraulics Model   Plan: SunnyDayBreach   4/30/2021
RS = 13146.93

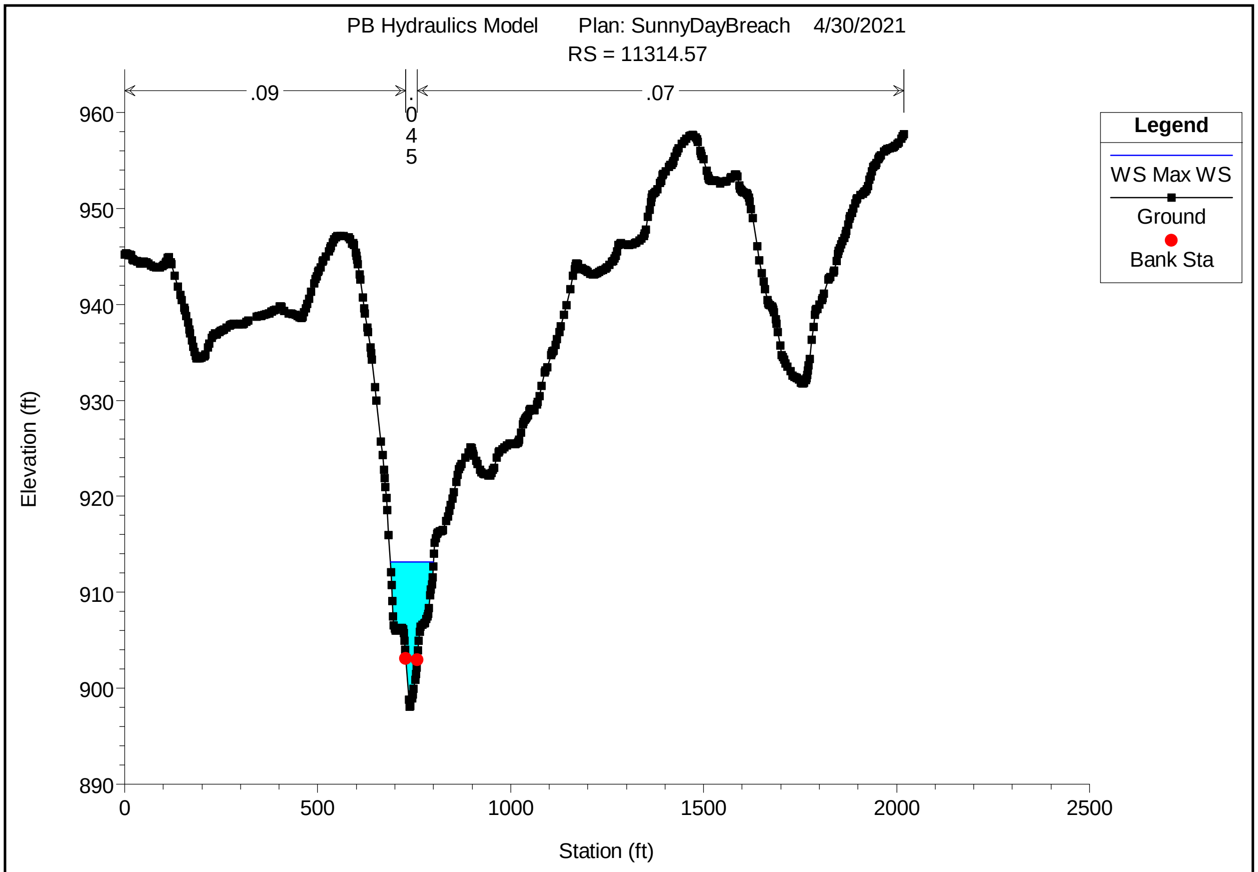


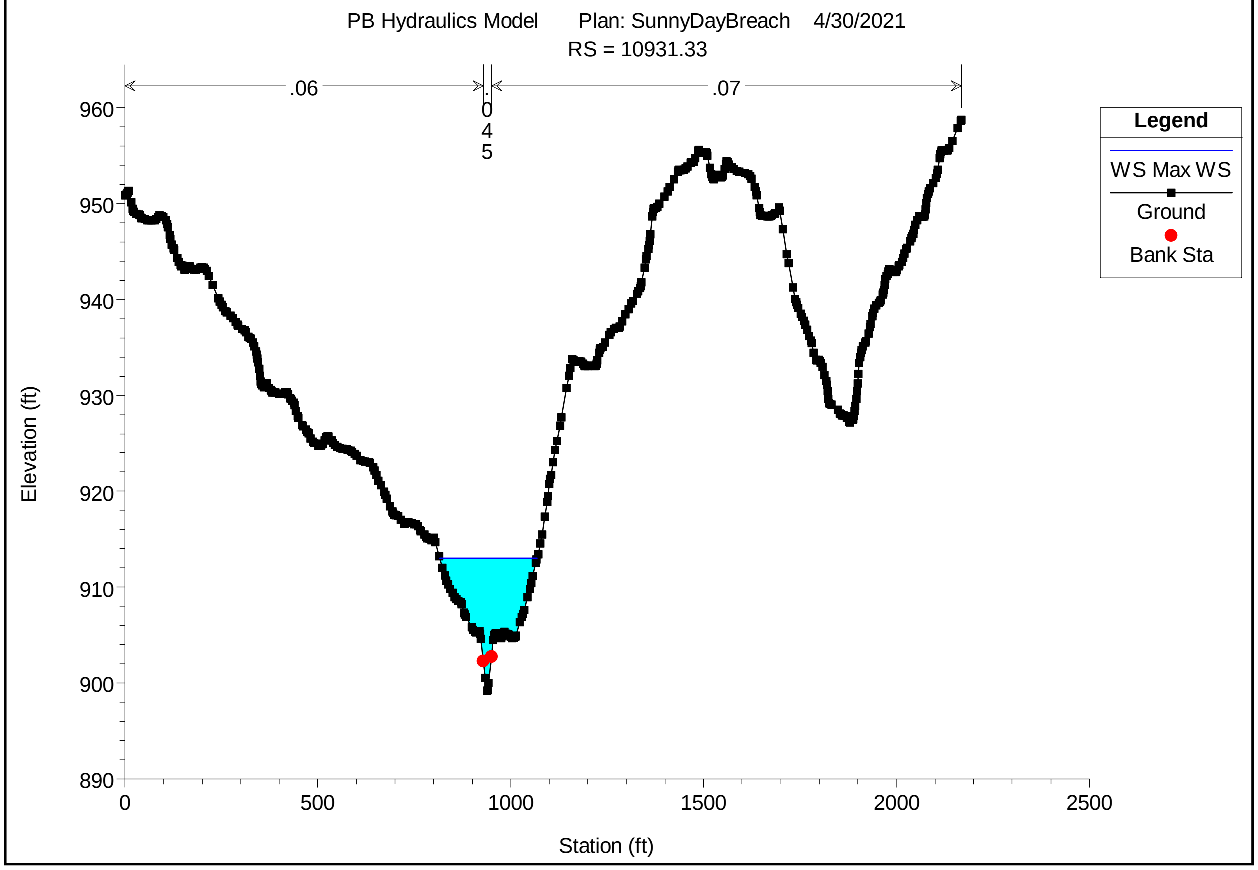

PB Hydraulics Model Plan: SunnyDayBreach 4/30/2021
RS = 12777.75
.06
.045
.06
Elevation (ft)
980
970
960
950
940
930
920
910
900
0
200
400
600
800
1000
1200
1400
1600
Station (ft)
Legend
WS Max WS
Ground
Bank Sta

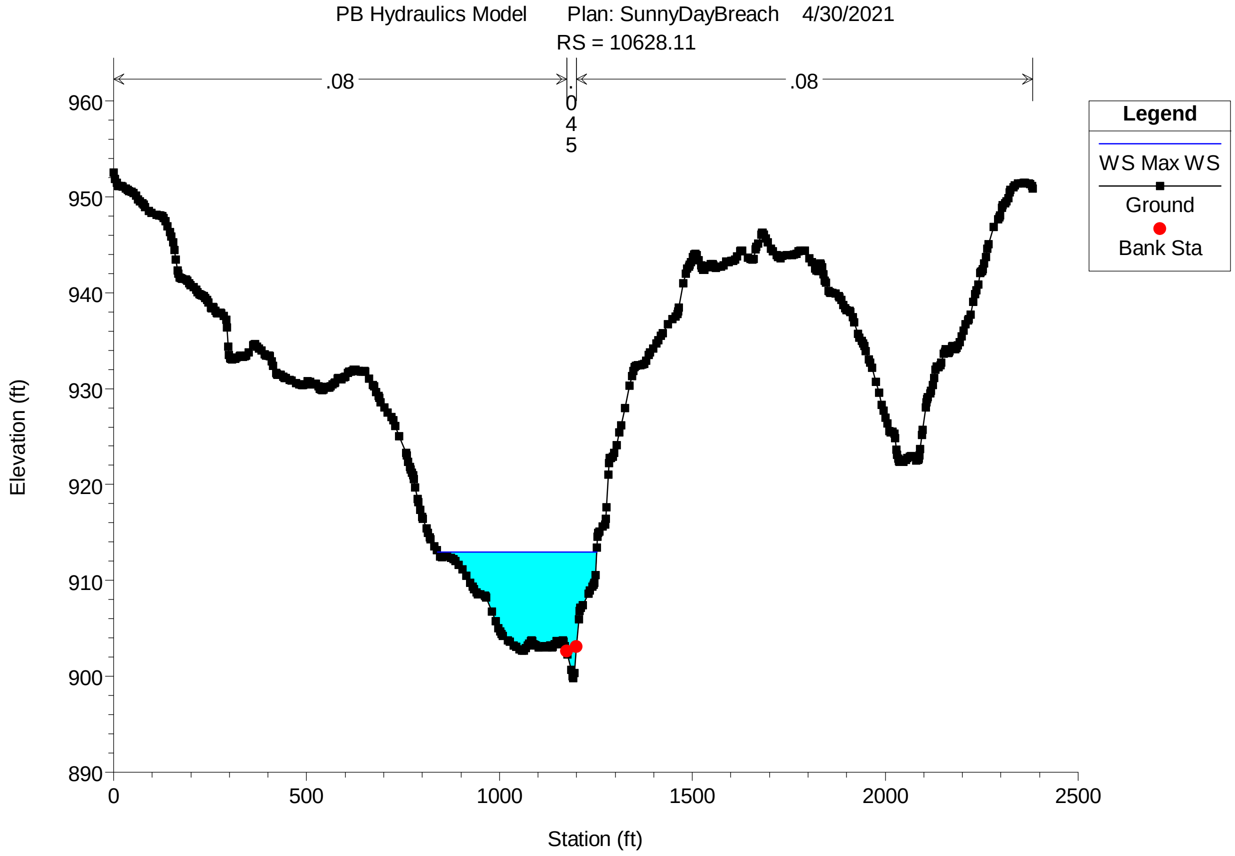

PB Hydraulics Model Plan: SunnyDayBreach 4/30/2021
RS = 12363.13
.06
.07
.045
Elevation (ft)
1000
980
960
940
920
900
0
200
400
600
800
1000
1200
1400
1600
Station (ft)
Legend
WS Max WS
Ground
Bank Sta

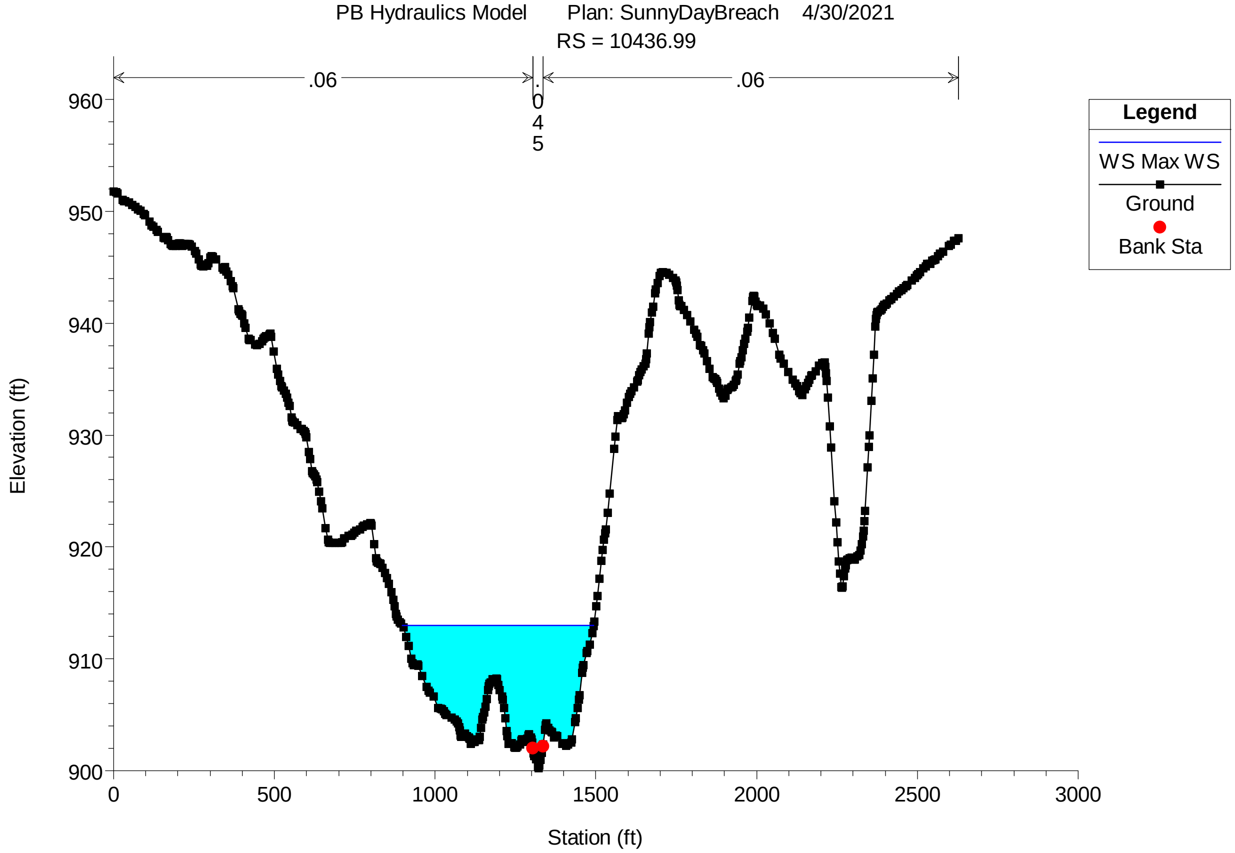

PB Hydraulics Model Plan: SunnyDayBreach 4/30/2021
RS = 12112.81
.07
.045
.1
Legend
WS Max WS
Ground
Bank Sta
Elevation (ft)
1000
980
960
940
920
900
0
200
400
600
800
1000
1200
1400
Station (ft)

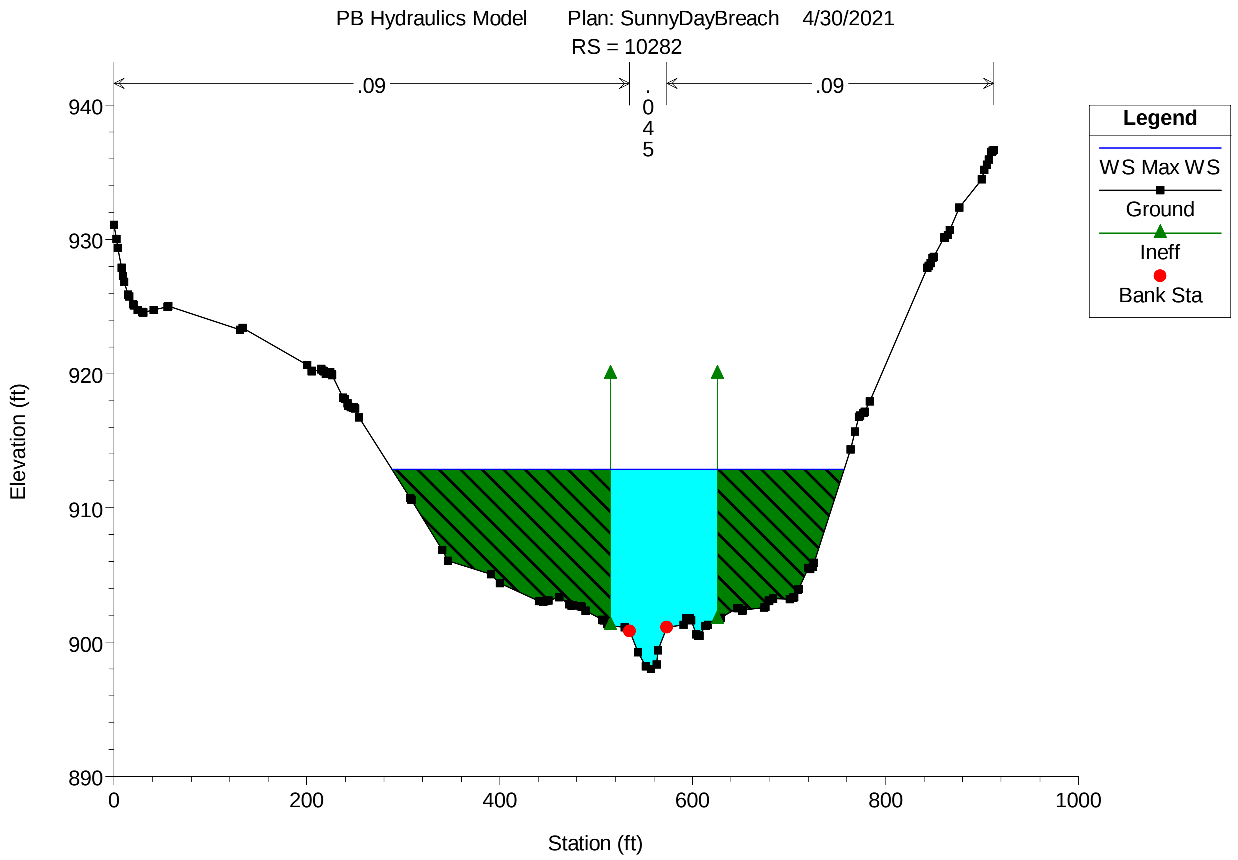

PB Hydraulics Model Plan: SunnyDayBreach 4/30/2021
RS = 11727.17
.07
.045
.08
Legend
WS Max WS
Ground
Bank Sta
Elevation (ft)
980
970
960
950
940
930
920
910
900
0
200
400
600
800
1000
1200
Station (ft)

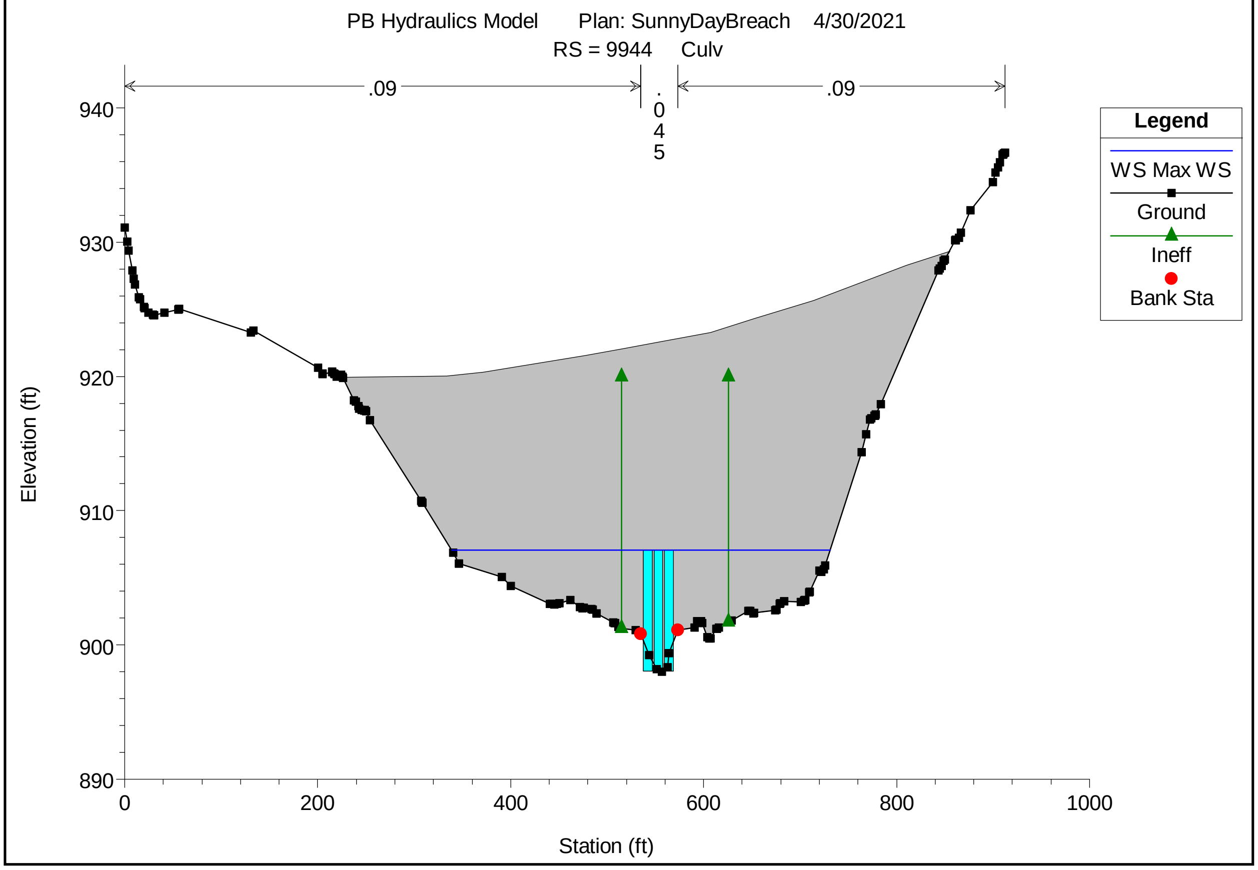

PB Hydraulics Model Plan: SunnyDayBreach 4/30/2021
RS = 11314.57
.09
.045
.07
Legend
WS Max WS
Ground
Bank Sta
Elevation (ft)
960
950
940
930
920
910
900
890
0
500
1000
1500
2000
2500
Station (ft)

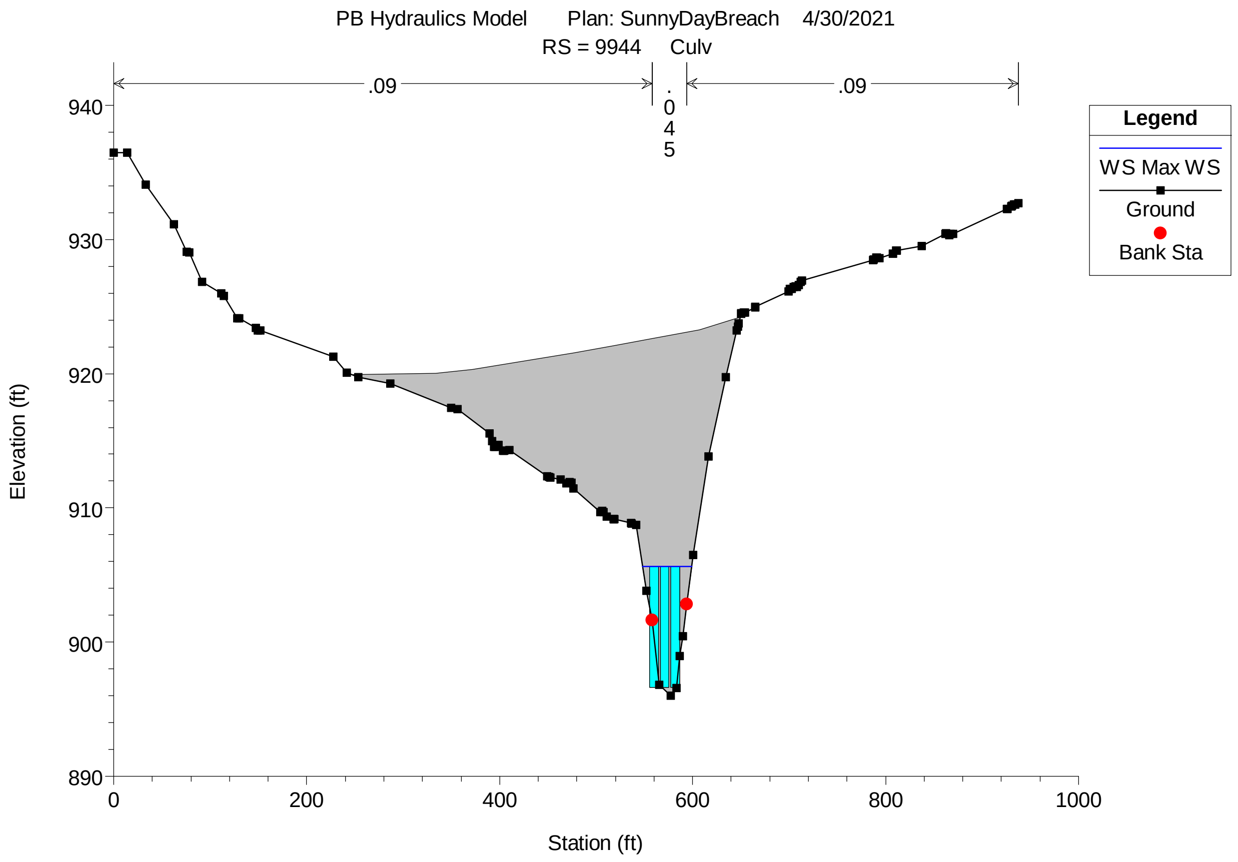

PB Hydraulics Model Plan: SunnyDayBreach 4/30/2021
RS = 10931.33
.06
.045
.07
Legend
WS Max WS
Ground
Bank Sta
Elevation (ft)
960
950
940
930
920
910
900
890
0
500
1000
1500
2000
2500
Station (ft)


PB Hydraulics Model Plan: SunnyDayBreach 4/30/2021
RS = 10628.11

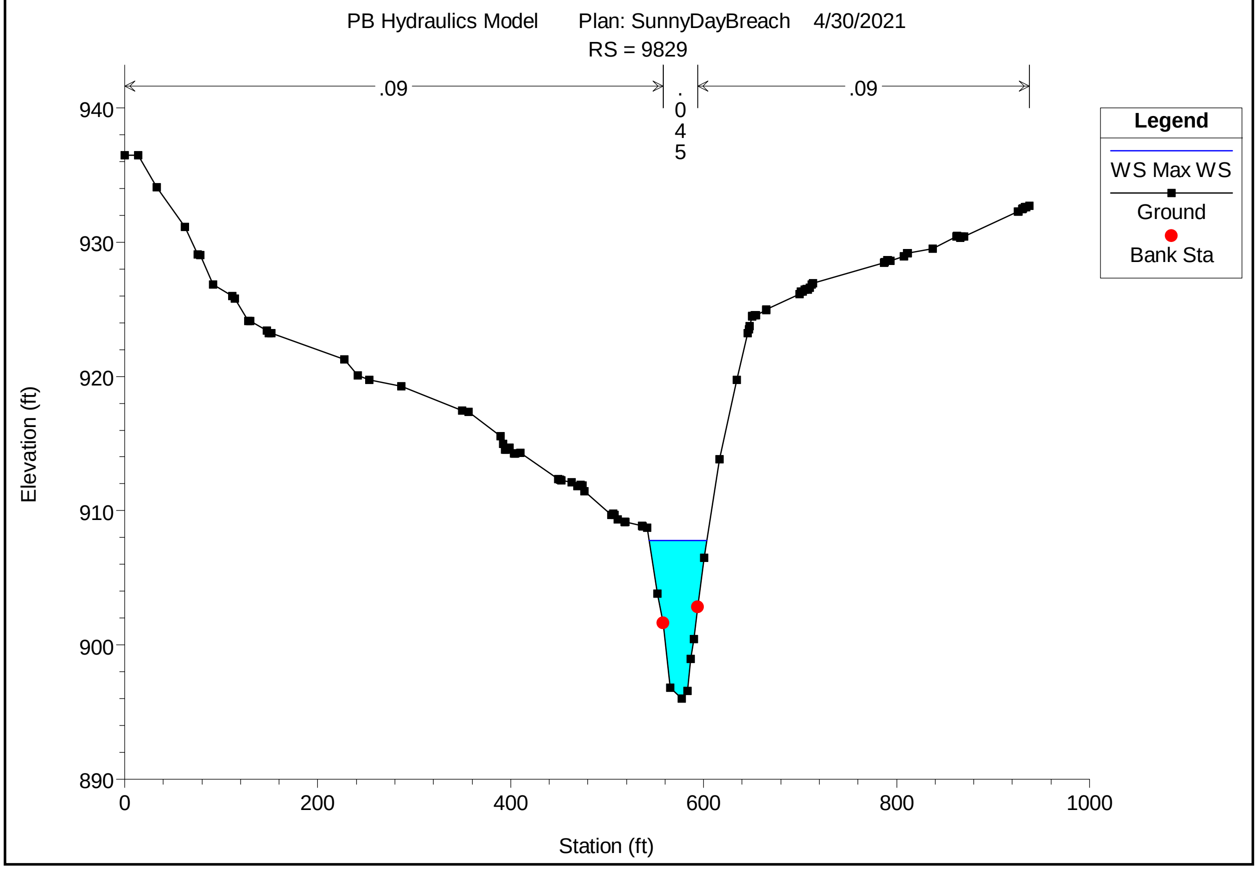

PB Hydraulics Model Plan: SunnyDayBreach 4/30/2021
RS = 10436.99


PB Hydraulics Model Plan: SunnyDayBreach 4/30/2021
RS = 10282
.09
.045
.09
Elevation (ft)
940
930
920
910
900
890
0
200
400
600
800
1000
Station (ft)
Legend
WS Max WS
Ground
Ineff
Bank Sta

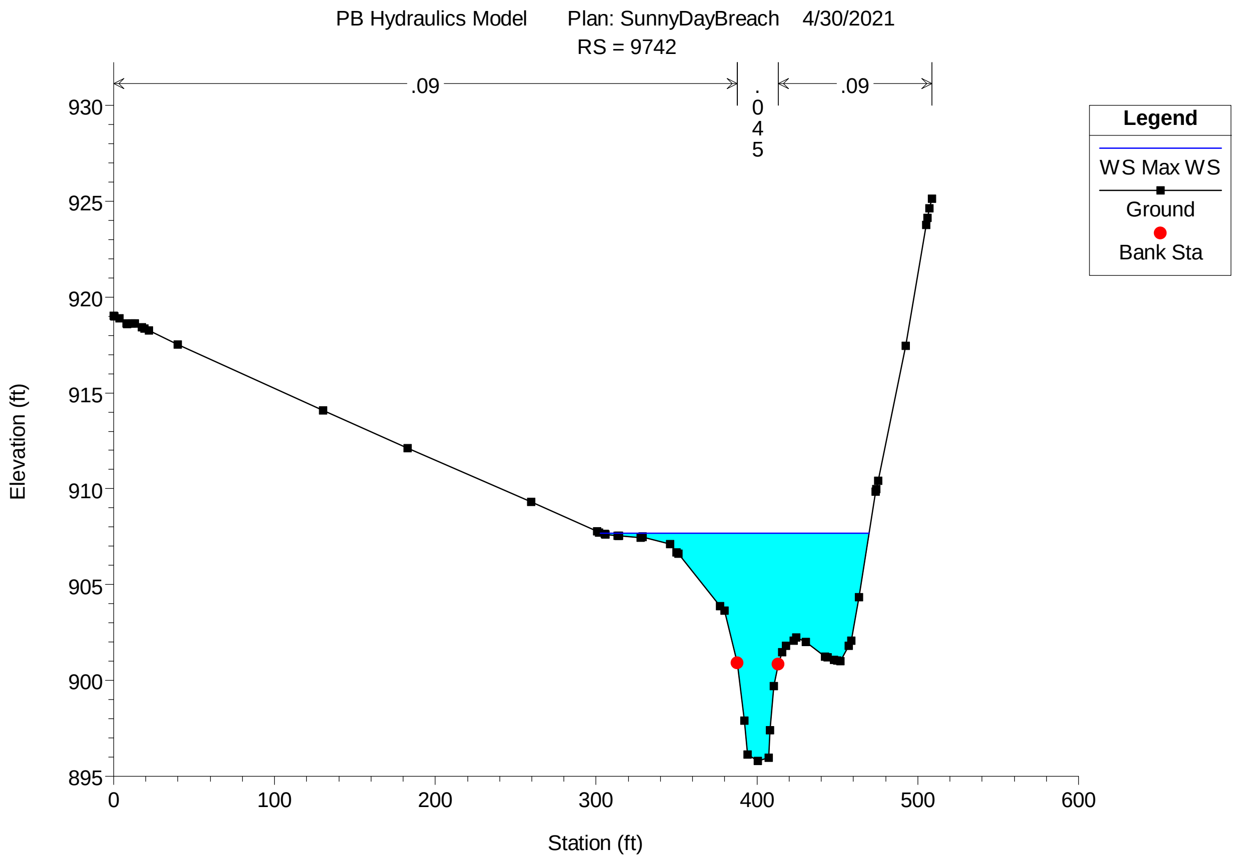

PB Hydraulics Model Plan: SunnyDayBreach 4/30/2021
RS = 9944 Culv
.09
.045
.09
Elevation (ft)
940
930
920
910
900
890
0
200
400
600
800
1000
Station (ft)
Legend
WS Max WS
Ground
Ineff
Bank Sta

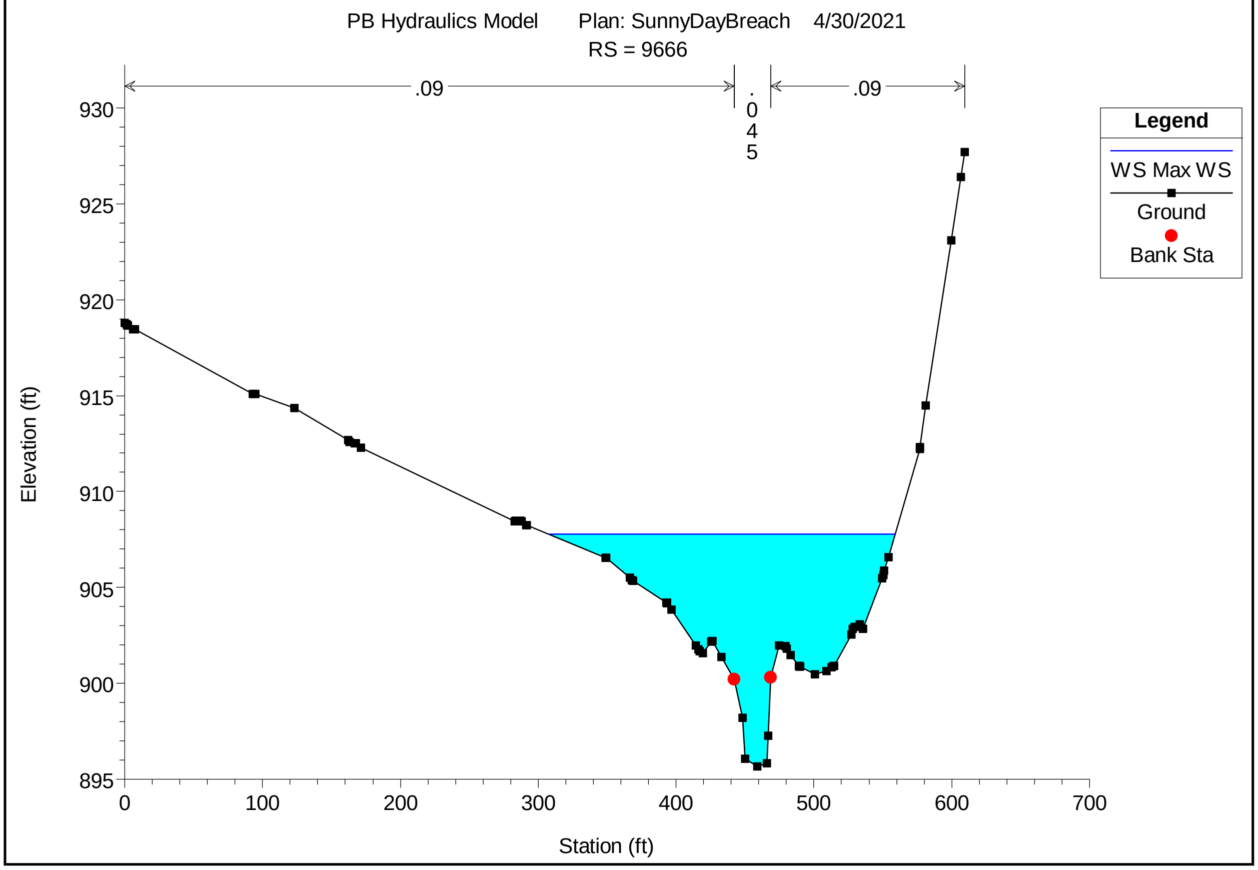


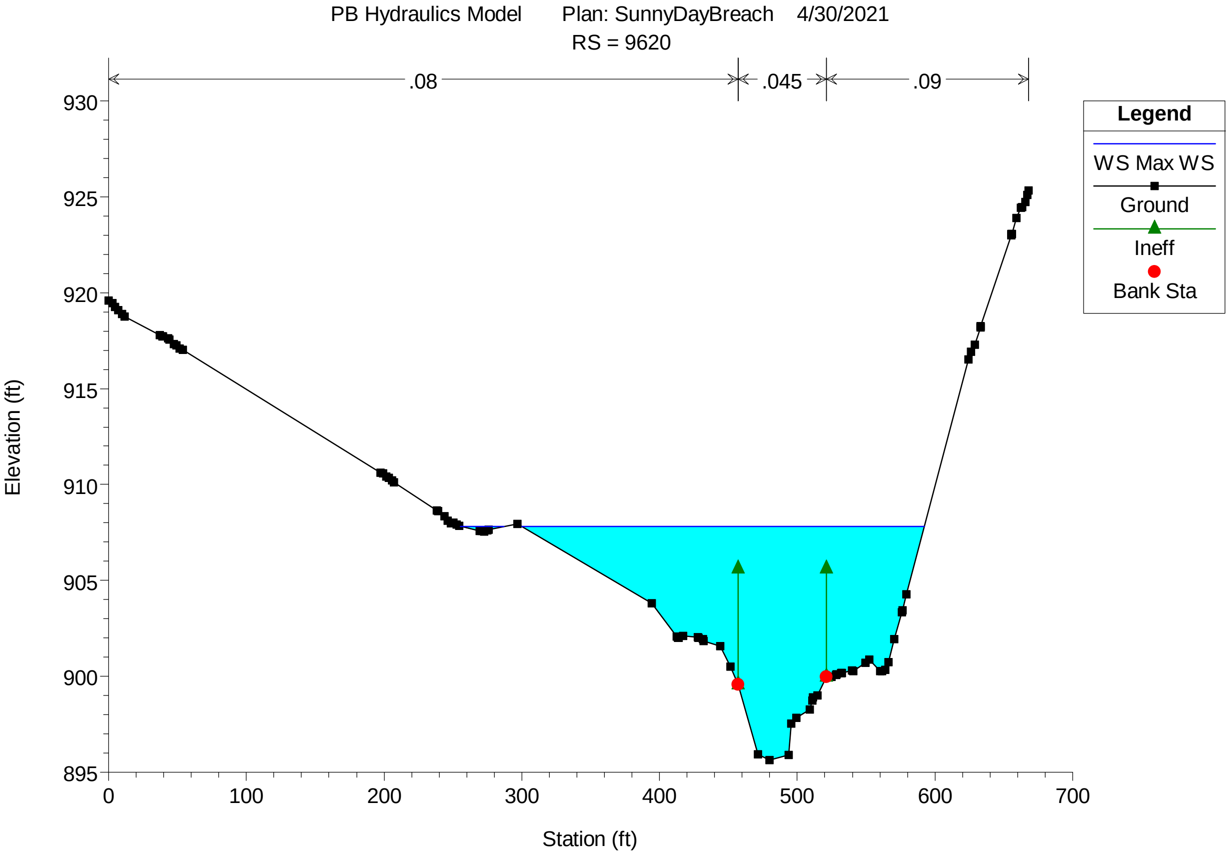
PB Hydraulics Model Plan: SunnyDayBreach 4/30/2021
RS = 9944 Culv



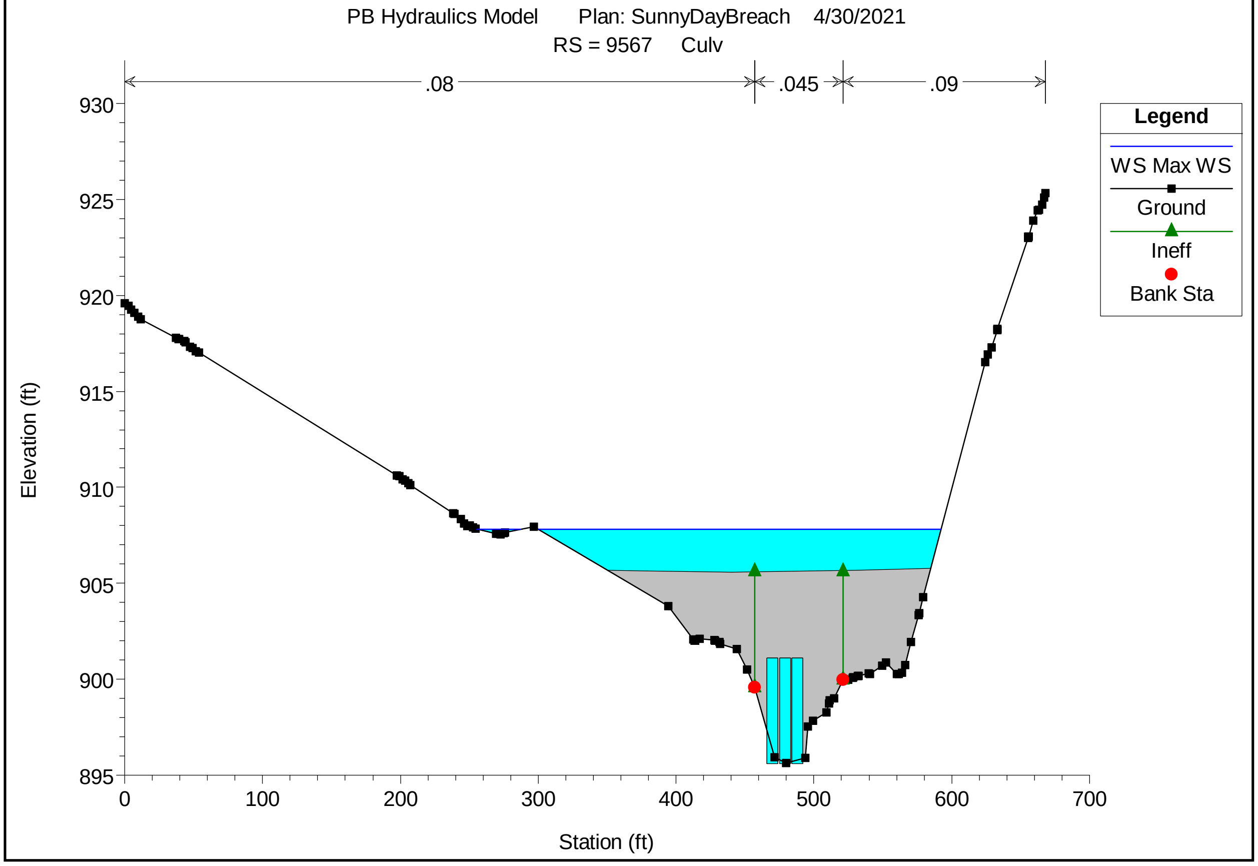
PB Hydraulics Model Plan: SunnyDayBreach 4/30/2021
RS = 9829


PB Hydraulics Model Plan: SunnyDayBreach 4/30/2021
RS = 9742
.09
.045
.09
Legend
WS Max WS
Ground
Bank Sta
Elevation (ft)
930
925
920
915
910
905
900
895
0
100
200
300
400
500
600
Station (ft)

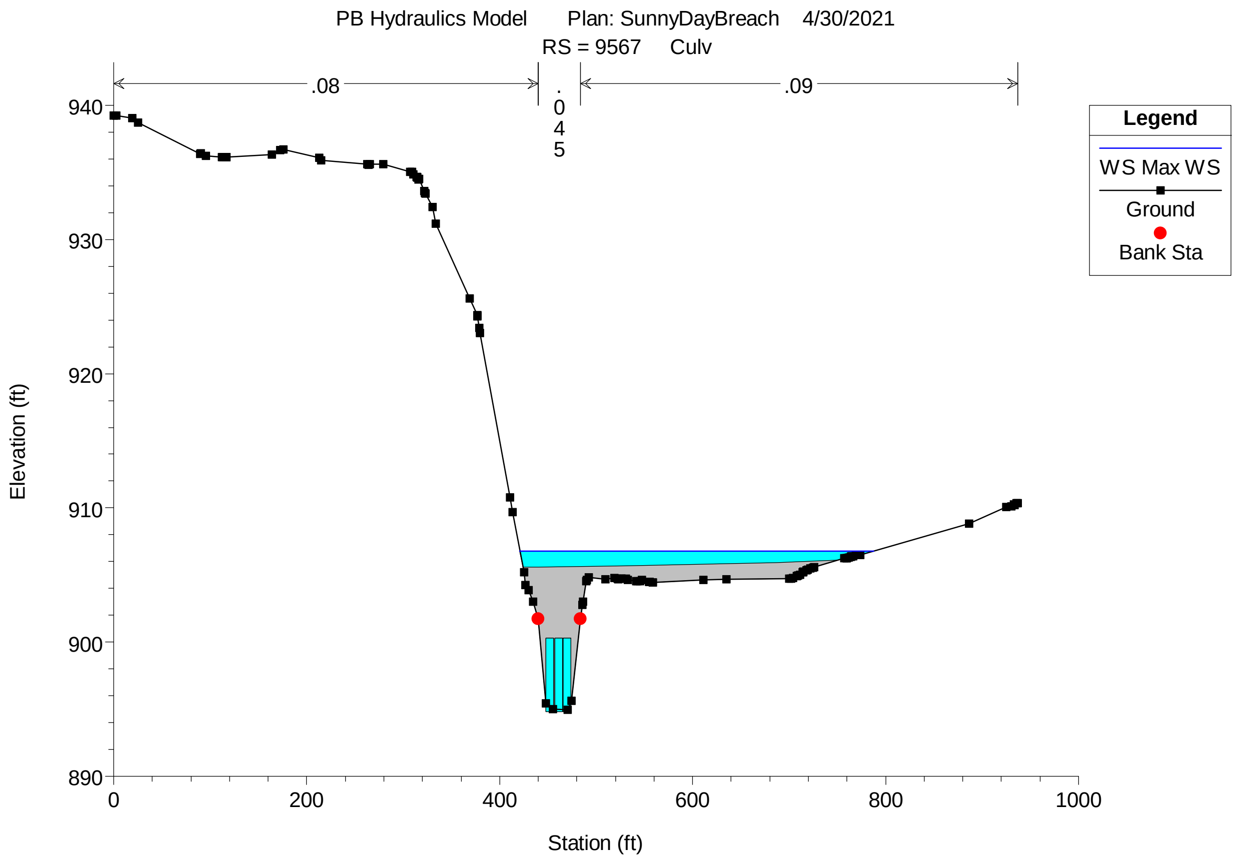

PB Hydraulics Model Plan: SunnyDayBreach 4/30/2021
RS = 9666
.09
.045
.09
Legend
WS Max WS
Ground
Bank Sta
Elevation (ft)
930
925
920
915
910
905
900
895
0
100
200
300
400
500
600
700
Station (ft)



PB Hydraulics Model Plan: SunnyDayBreach 4/30/2021
RS = 9620
.08
.045
.09
930
925
920
915
910
905
900
895
Elevation (ft)
0
100
200
300
400
500
600
700
Station (ft)
Legend
WS Max WS
Ground
Ineff
Bank Sta

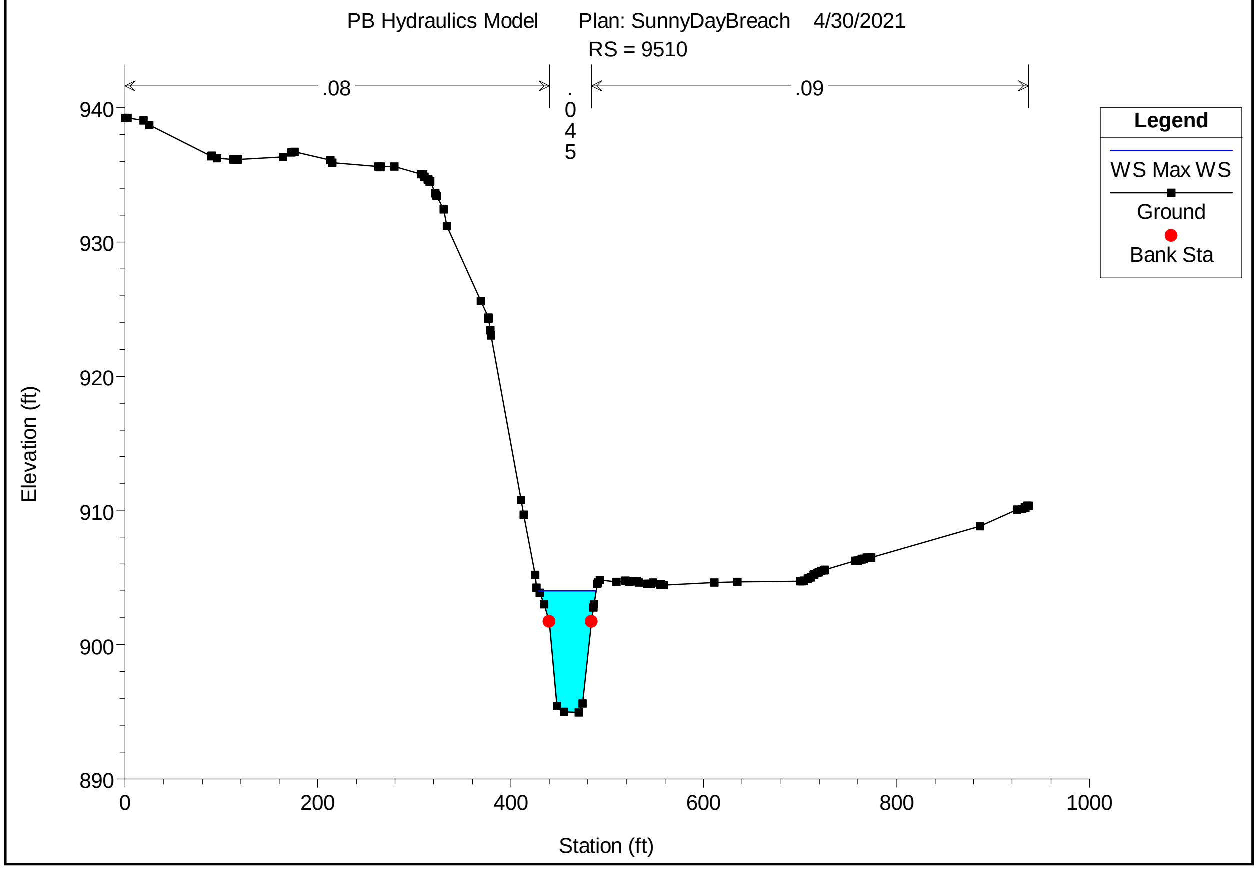


PB Hydraulics Model Plan: SunnyDayBreach 4/30/2021
RS = 9567 Culv
.08
.045
.09
930
925
920
915
910
905
900
895
Elevation (ft)
0
100
200
300
400
500
600
700
Station (ft)
Legend
WS Max WS
Ground
Ineff
Bank Sta



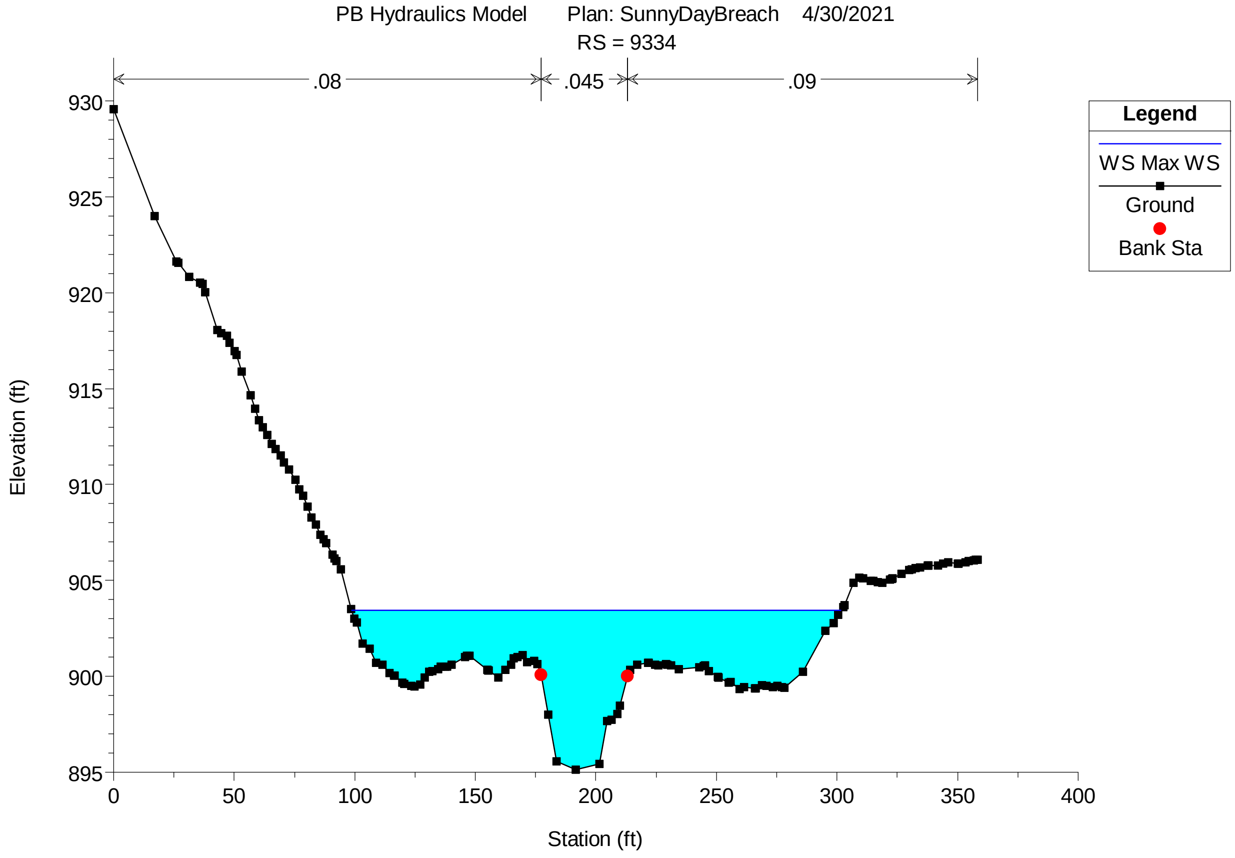
PB Hydraulics Model Plan: SunnyDayBreach 4/30/2021

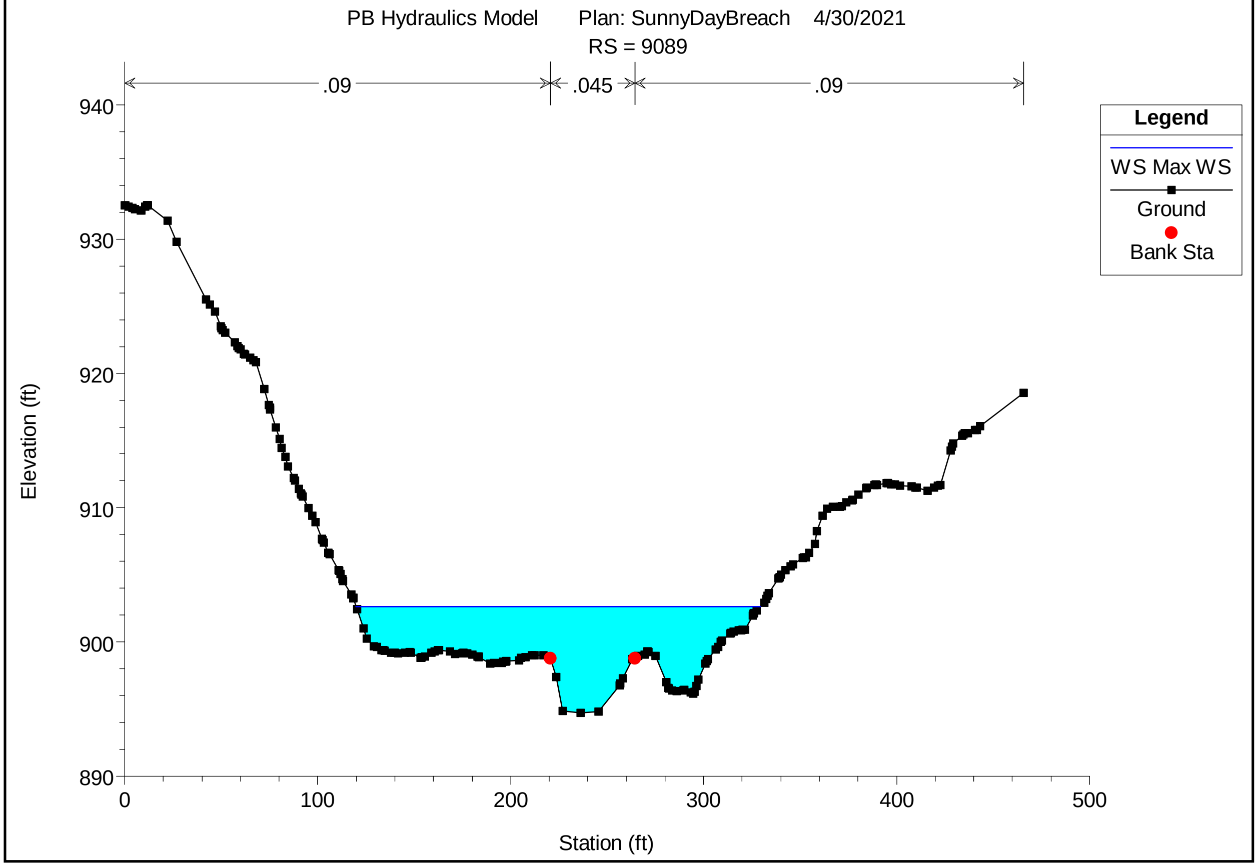
RS = 9567 Culv

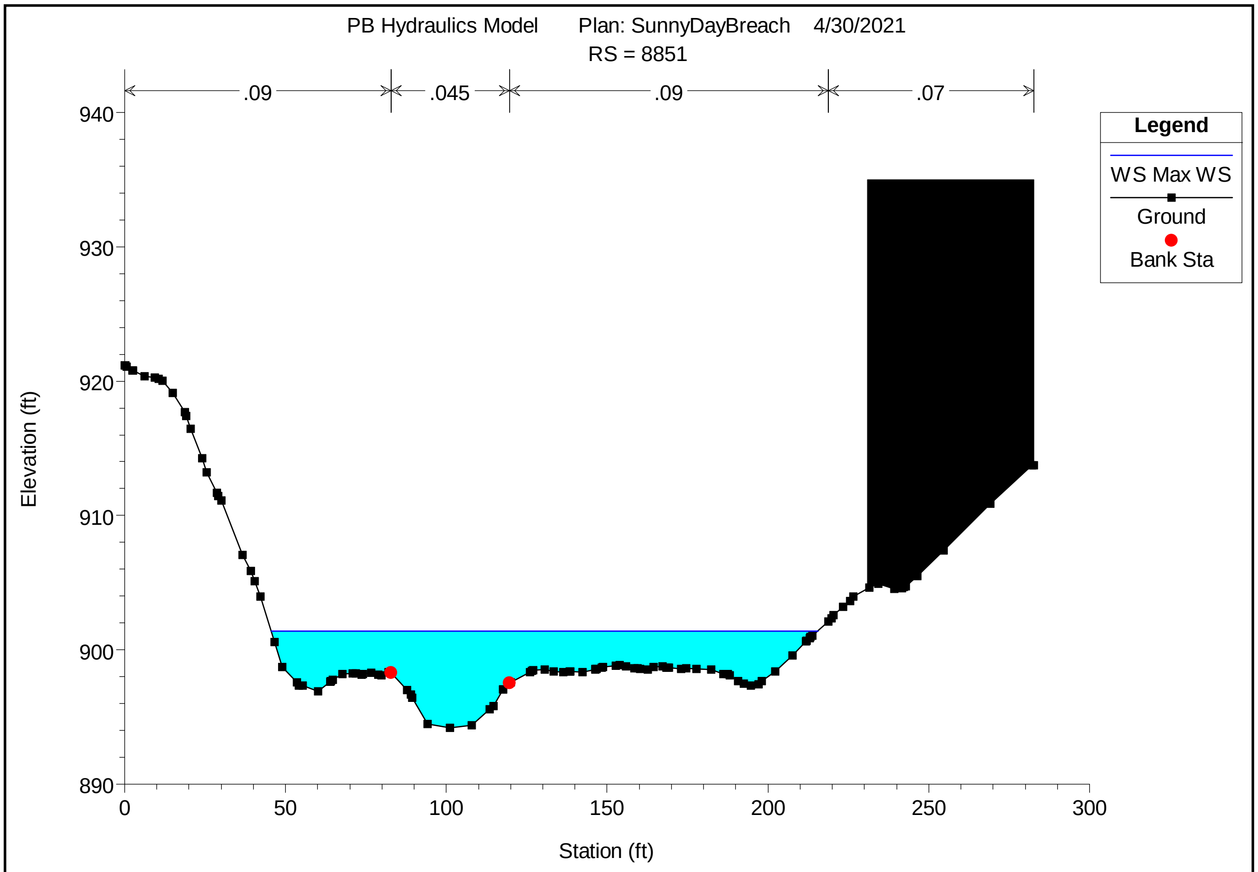


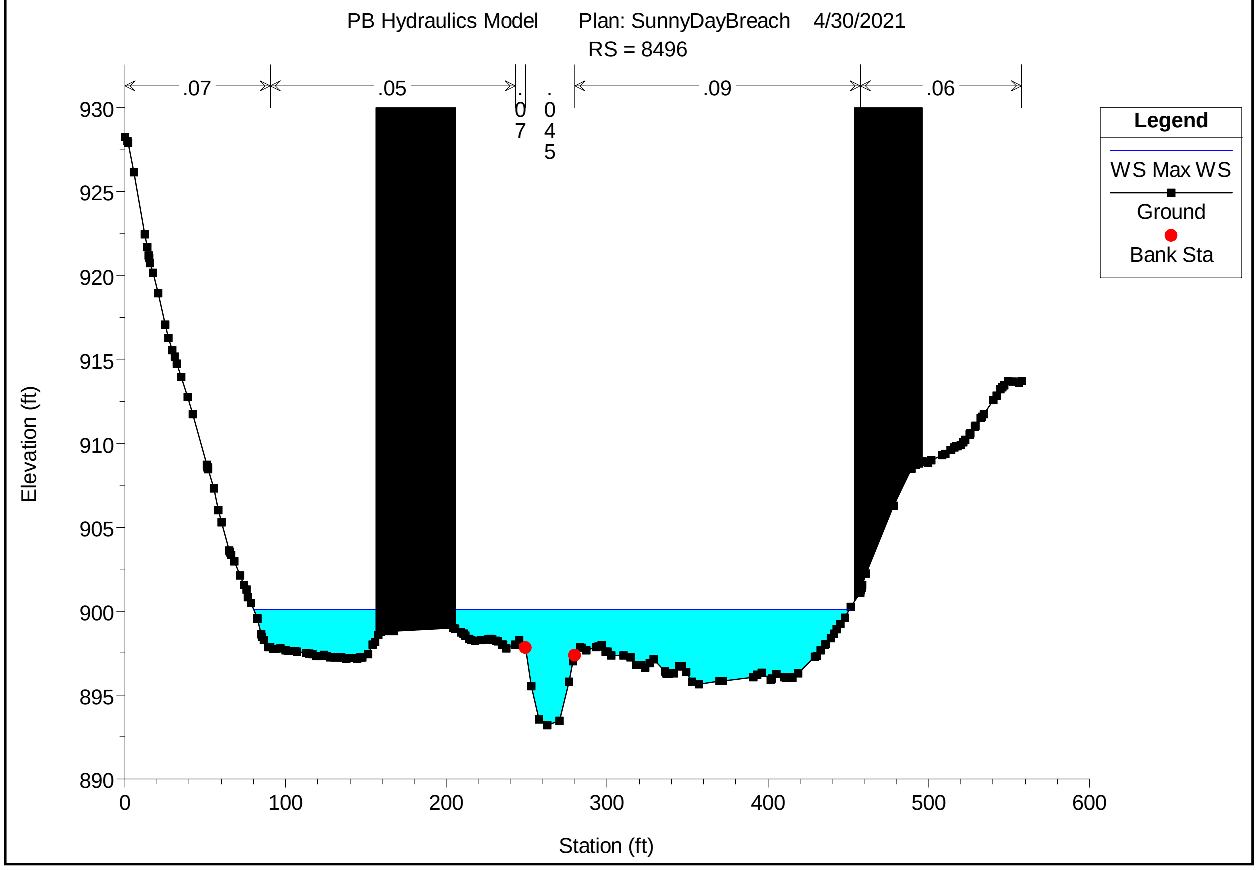
PB Hydraulics Model Plan: SunnyDayBreach 4/30/2021

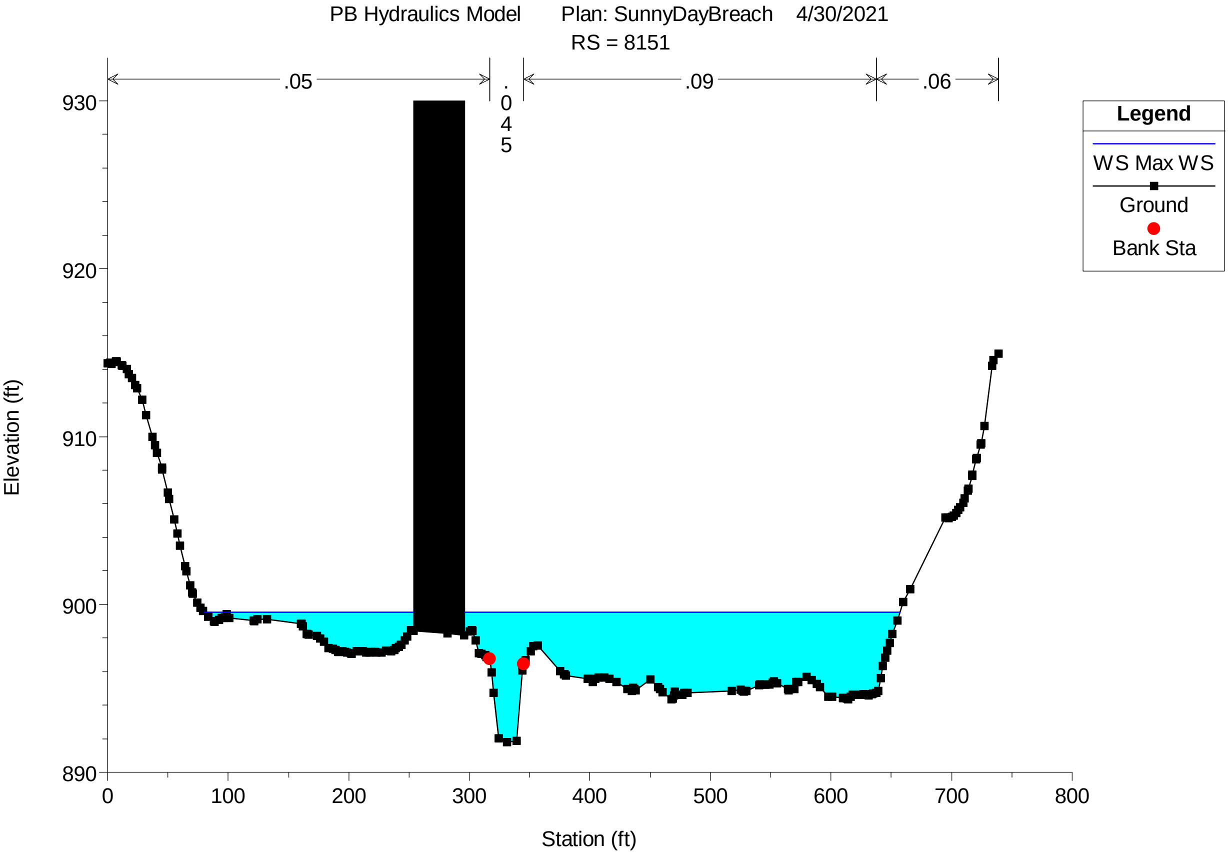
RS = 9510

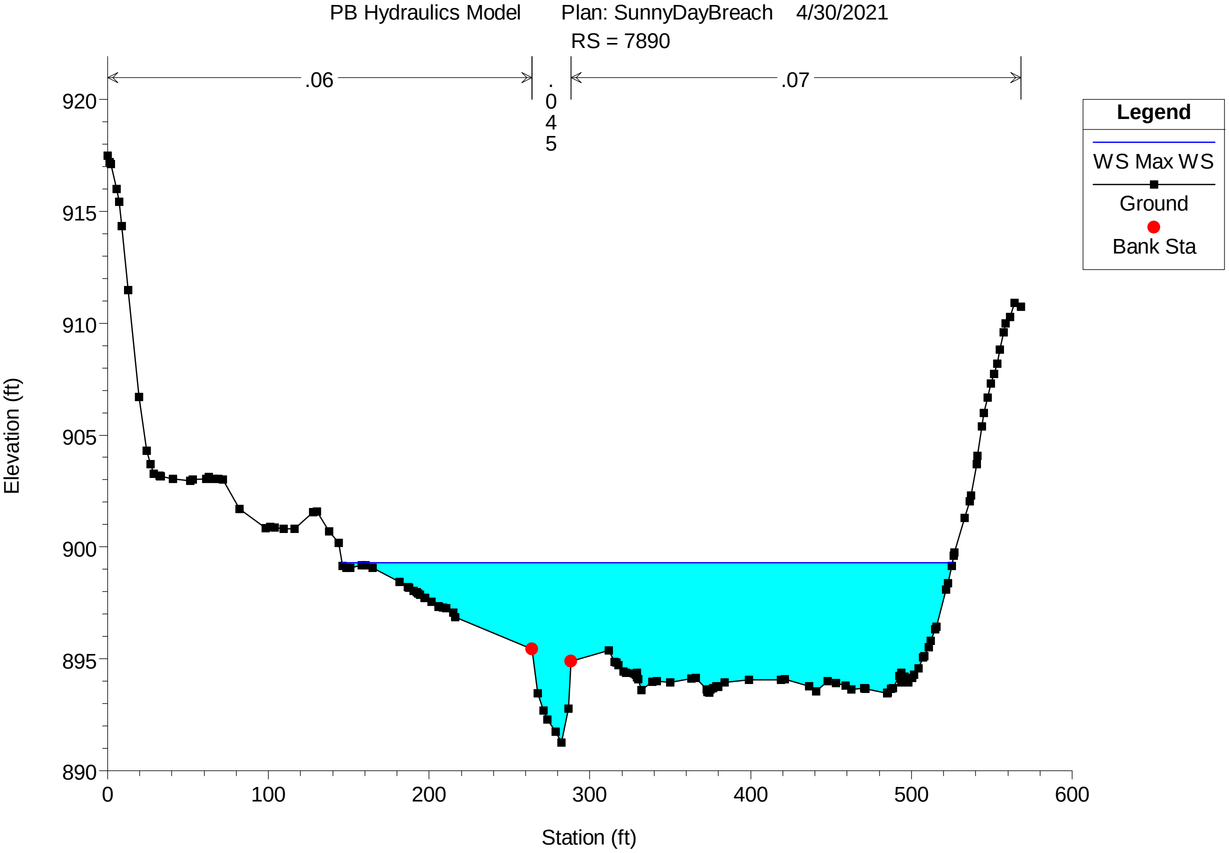

PB Hydraulics Model Plan: SunnyDayBreach 4/30/2021
RS = 9334
.08
.045
.09
930
925
920
915
910
905
900
895
Elevation (ft)
0
50
100
150
200
250
300
350
400
Station (ft)
Legend
WS Max WS
Ground
Bank Sta

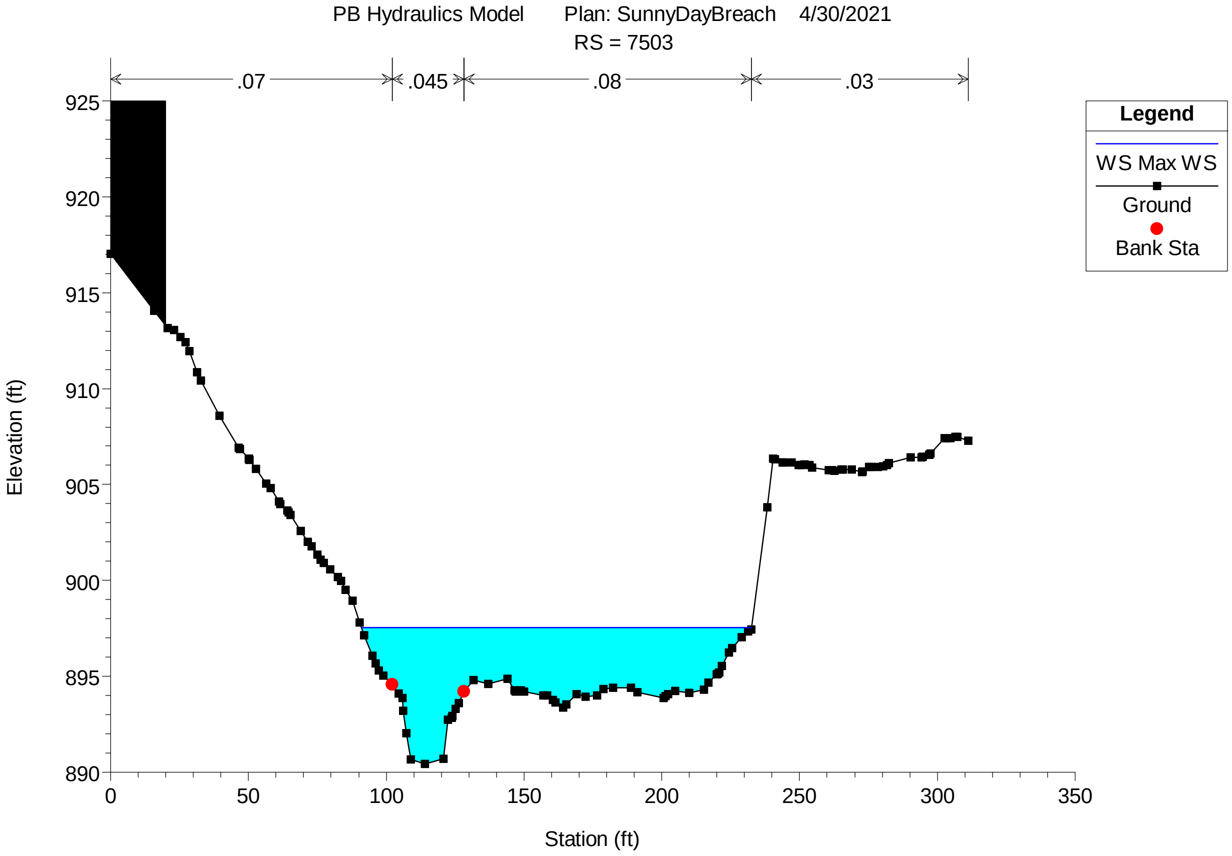

PB Hydraulics Model Plan: SunnyDayBreach 4/30/2021
RS = 9089
.09
.045
.09
940
930
920
910
900
890
Elevation (ft)
0
100
200
300
400
500
Station (ft)
Legend
WS Max WS
Ground
Bank Sta

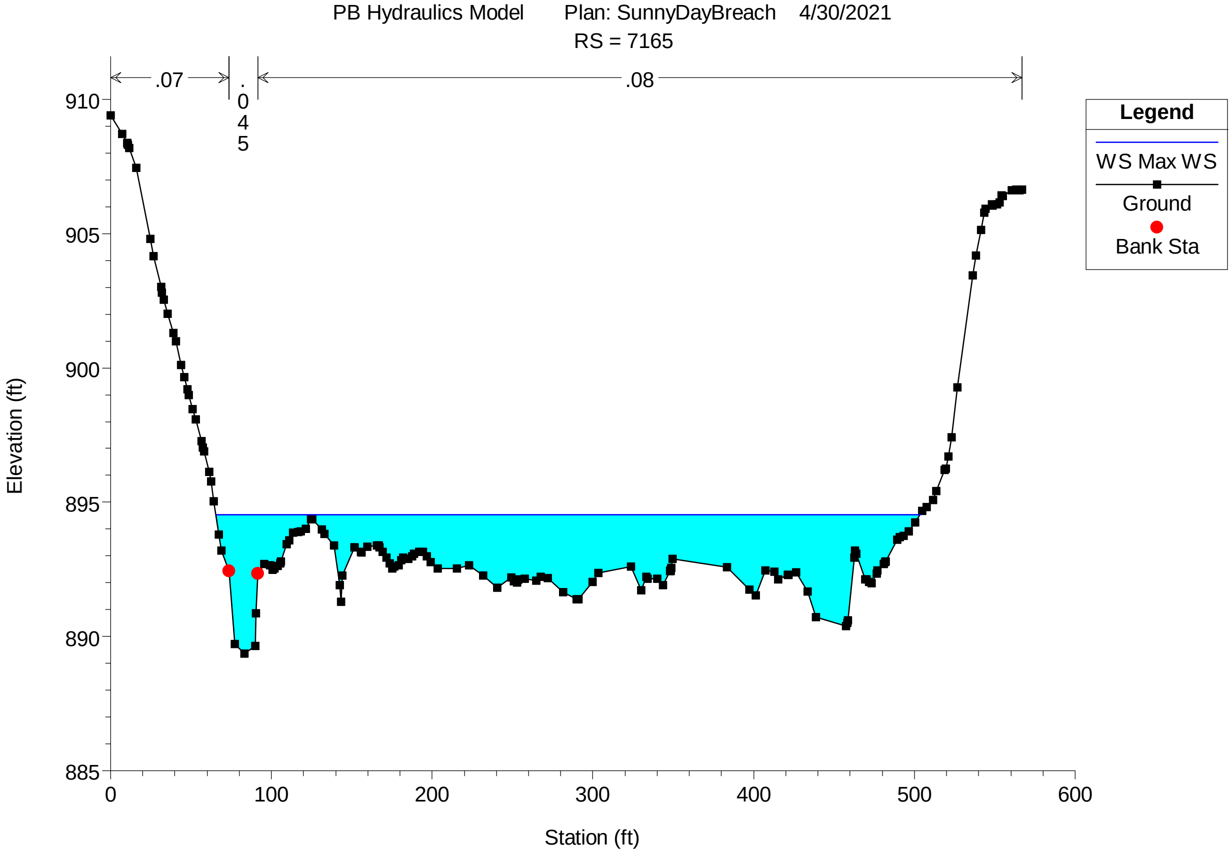

PB Hydraulics Model Plan: SunnyDayBreach 4/30/2021
RS = 8851
.09
.045
.09
.07
Legend
WS Max WS
Ground
Bank Sta
Elevation (ft)
940
930
920
910
900
890
0
50
100
150
200
250
300
Station (ft)

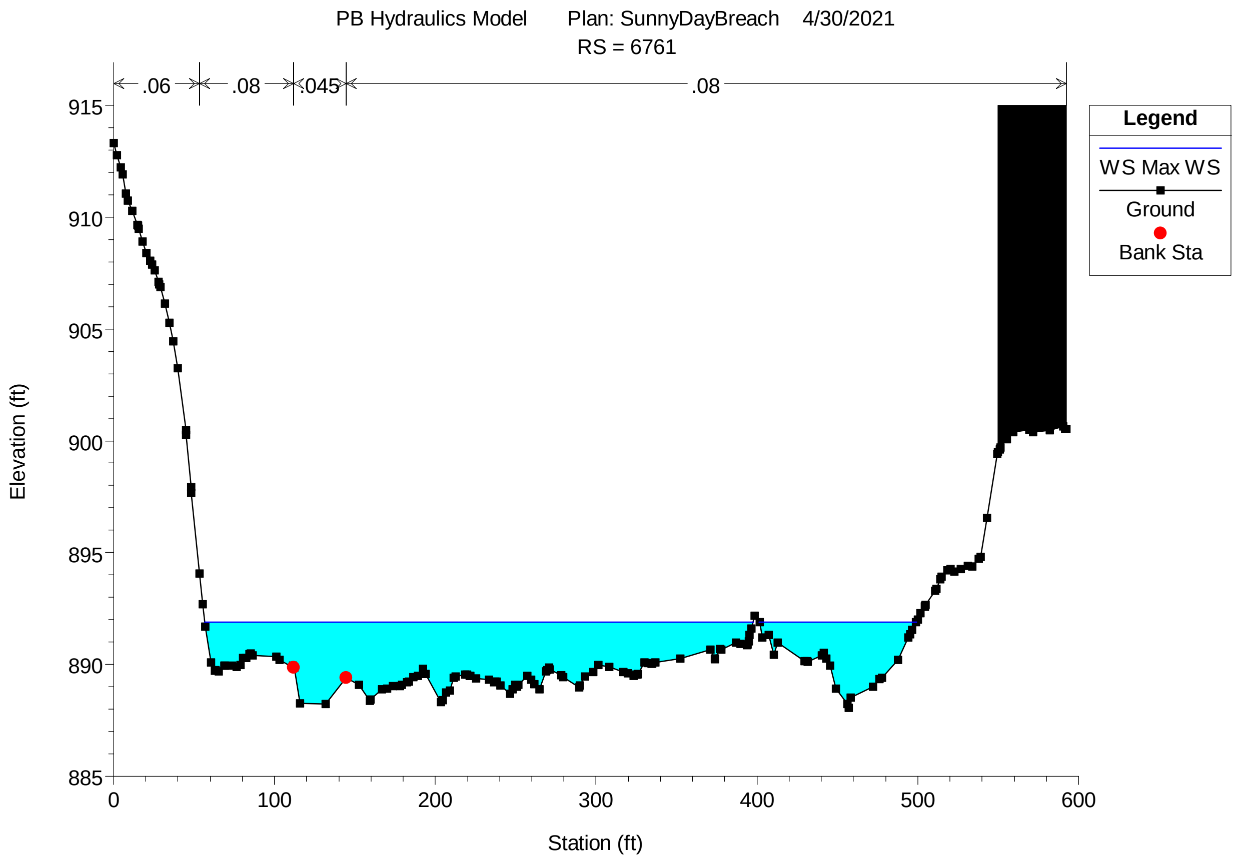

PB Hydraulics Model Plan: SunnyDayBreach 4/30/2021
RS = 8496
.07
.05
.07
.045
.09
.06
Legend
WS Max WS
Ground
Bank Sta
Elevation (ft)
930
925
920
915
910
905
900
895
890
0
100
200
300
400
500
600
Station (ft)

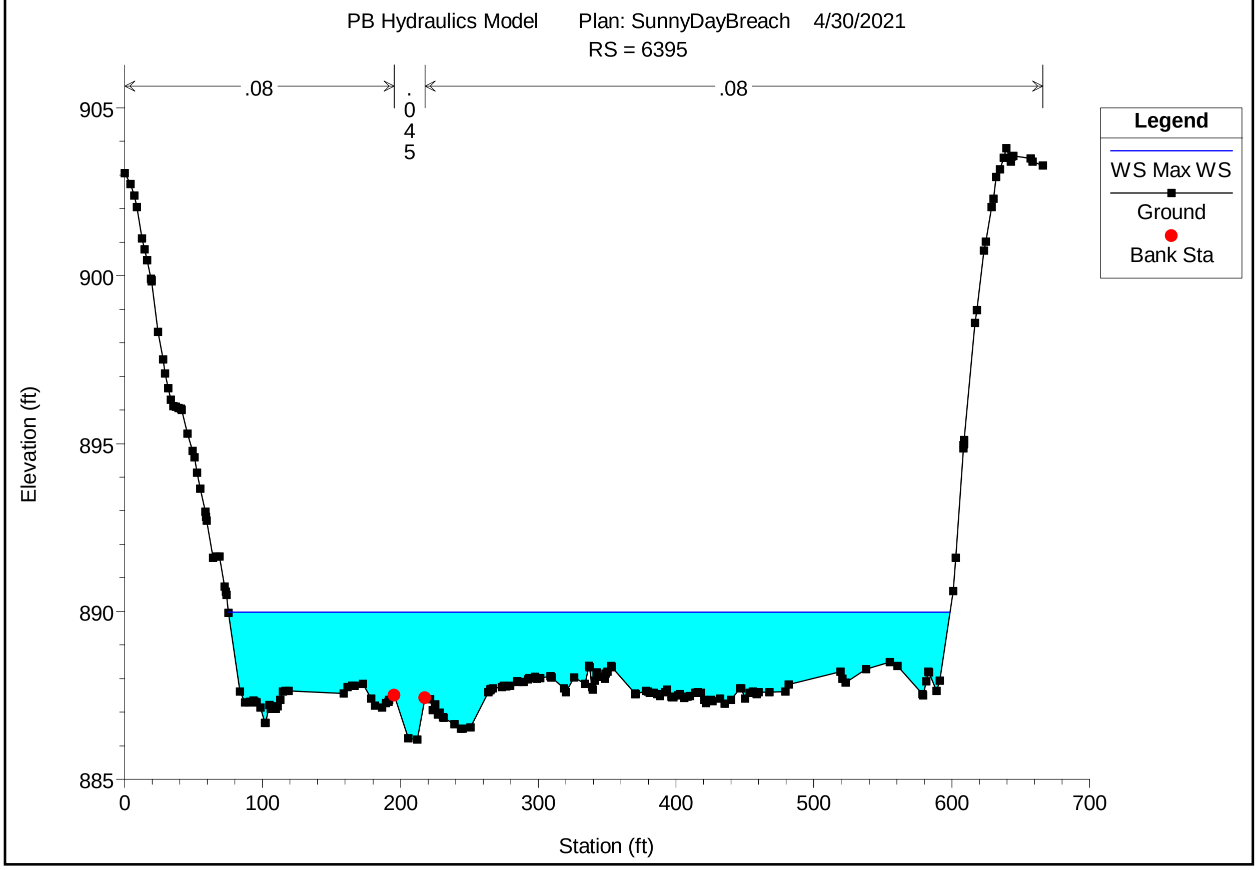

PB Hydraulics Model Plan: SunnyDayBreach 4/30/2021
RS = 8151
.05
.045
.09
.06
Legend
WS Max WS
Ground
Bank Sta
Elevation (ft)
930
920
910
900
890
0
100
200
300
400
500
600
700
800
Station (ft)

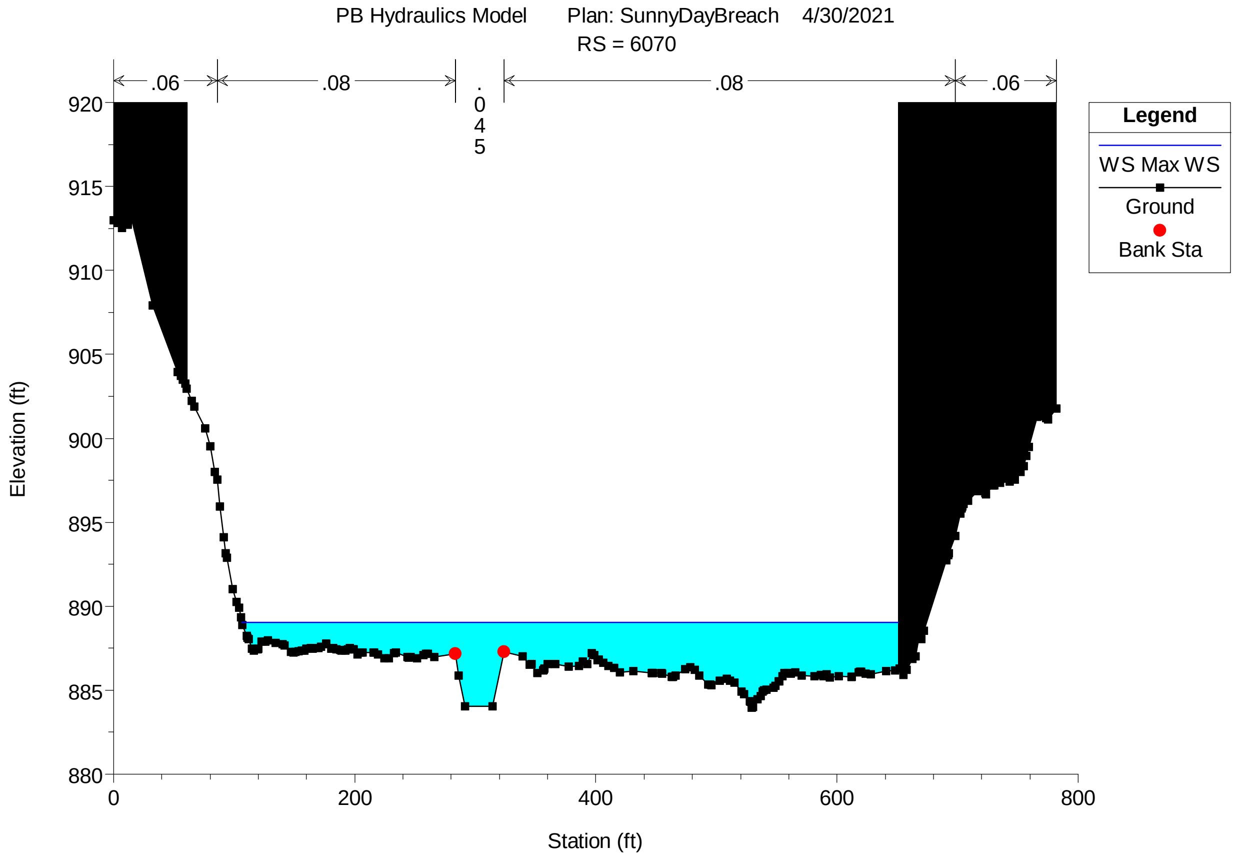

PB Hydraulics Model Plan: SunnyDayBreach 4/30/2021
RS = 7890
.06
.045
.07
Legend
WS Max WS
Ground
Bank Sta
Elevation (ft)
920
915
910
905
900
895
890
0
100
200
300
400
500
600
Station (ft)

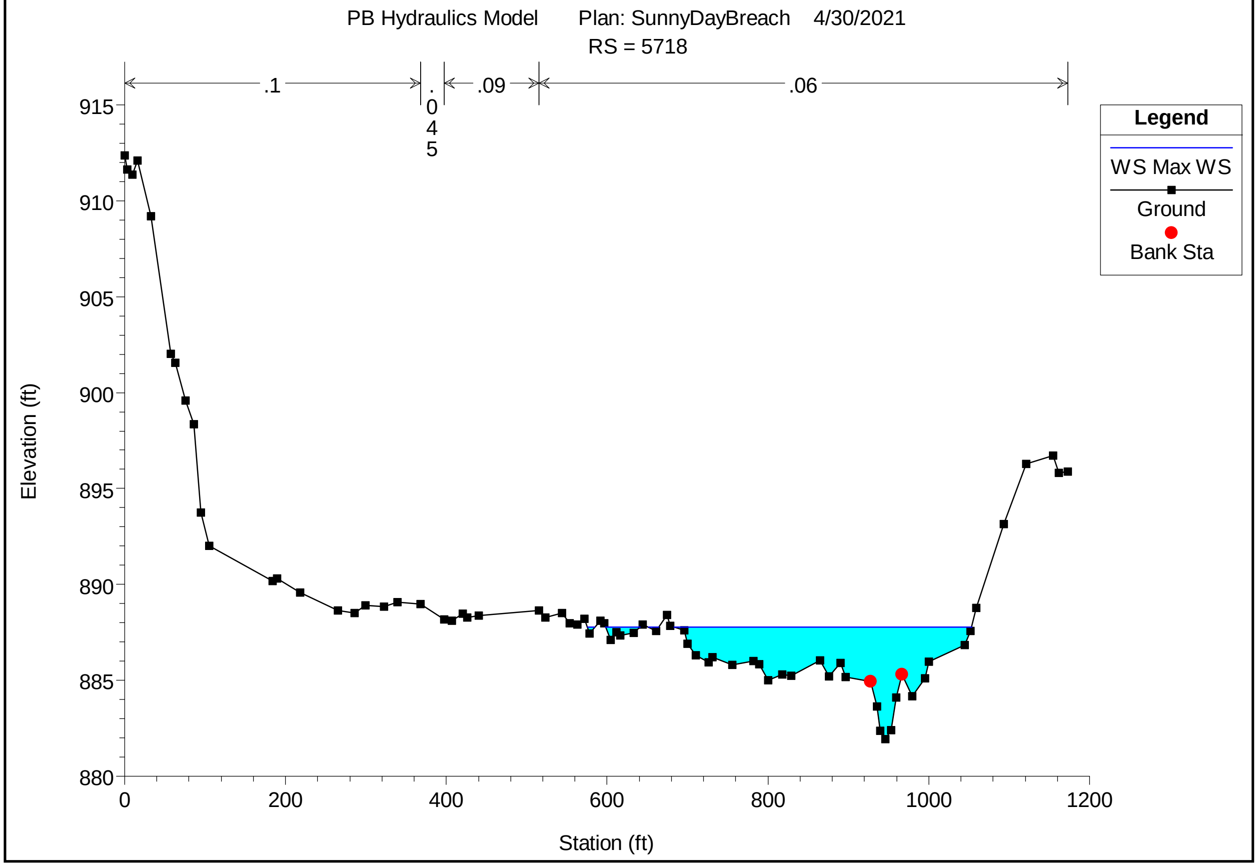

PB Hydraulics Model Plan: SunnyDayBreach 4/30/2021
RS = 7503
.07
.045
.08
.03
Legend
WS Max WS
Ground
Bank Sta
Elevation (ft)
925
920
915
910
905
900
895
890
0
50
100
150
200
250
300
350
Station (ft)

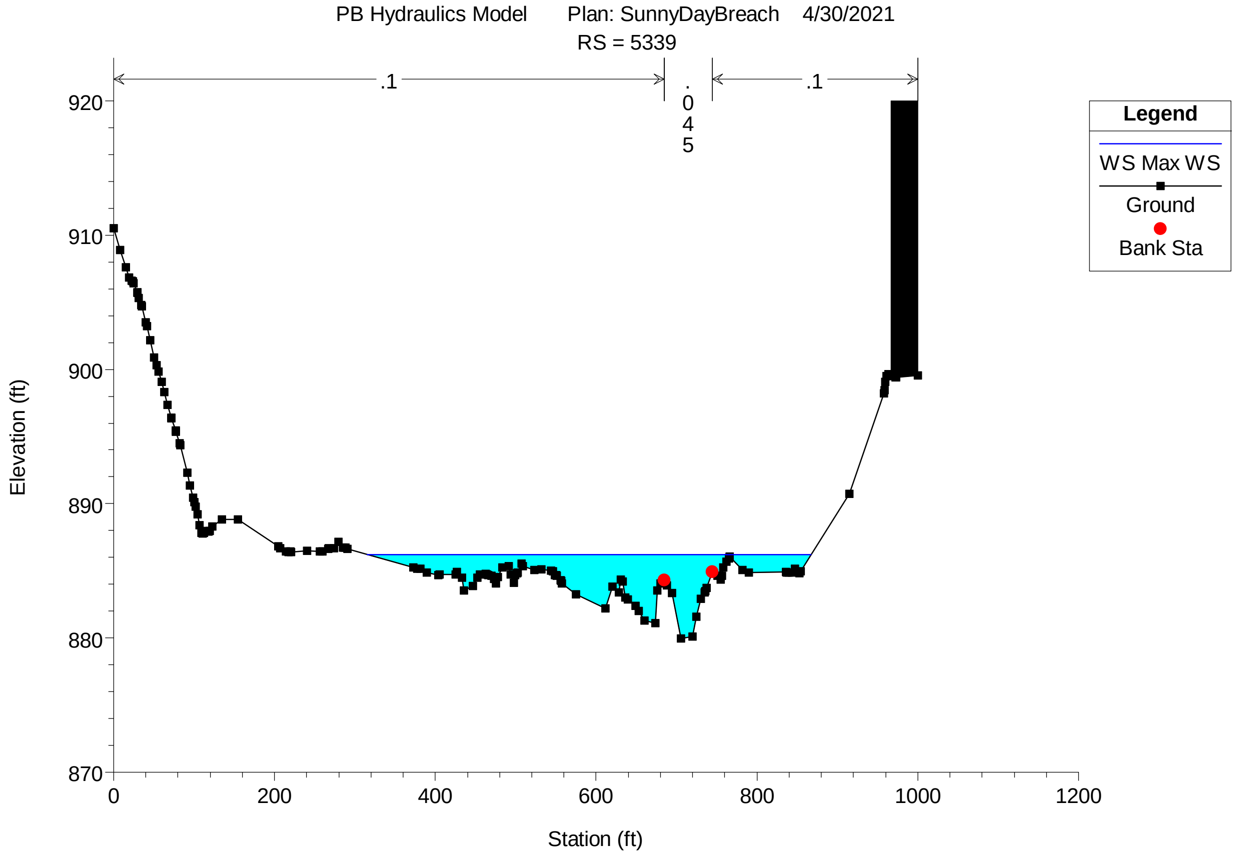

PB Hydraulics Model Plan: SunnyDayBreach 4/30/2021
RS = 7165
.07
.045
.08
Legend
WS Max WS
Ground
Bank Sta
Elevation (ft)
910
905
900
895
890
885
0
100
200
300
400
500
600
Station (ft)


PB Hydraulics Model Plan: SunnyDayBreach 4/30/2021
RS = 6761
.06
.08
.045
.08
Legend
WS Max WS
Ground
Bank Sta
Elevation (ft)
915
910
905
900
895
890
885
0
100
200
300
400
500
600
Station (ft)


PB Hydraulics Model Plan: SunnyDayBreach 4/30/2021
RS = 6395
.08
.045
.08
Legend
WS Max WS
Ground
Bank Sta
Elevation (ft)
905
900
895
890
885
0
100
200
300
400
500
600
700
Station (ft)


PB Hydraulics Model Plan: SunnyDayBreach 4/30/2021
RS = 6070
.06
.08
.045
.08
.06
Legend
WS Max WS
Ground
Bank Sta
Elevation (ft)
920
915
910
905
900
895
890
885
880
0
200
400
600
800
Station (ft)


PB Hydraulics Model Plan: SunnyDayBreach 4/30/2021
RS = 5718
.1
.045
.09
.06
Legend
WS Max WS
Ground
Bank Sta
Elevation (ft)
915
910
905
900
895
890
885
880
0
200
400
600
800
1000
1200
Station (ft)


PB Hydraulics Model Plan: SunnyDayBreach 4/30/2021
RS = 5339
.1
.045
.1
Legend
WS Max WS
Ground
Bank Sta
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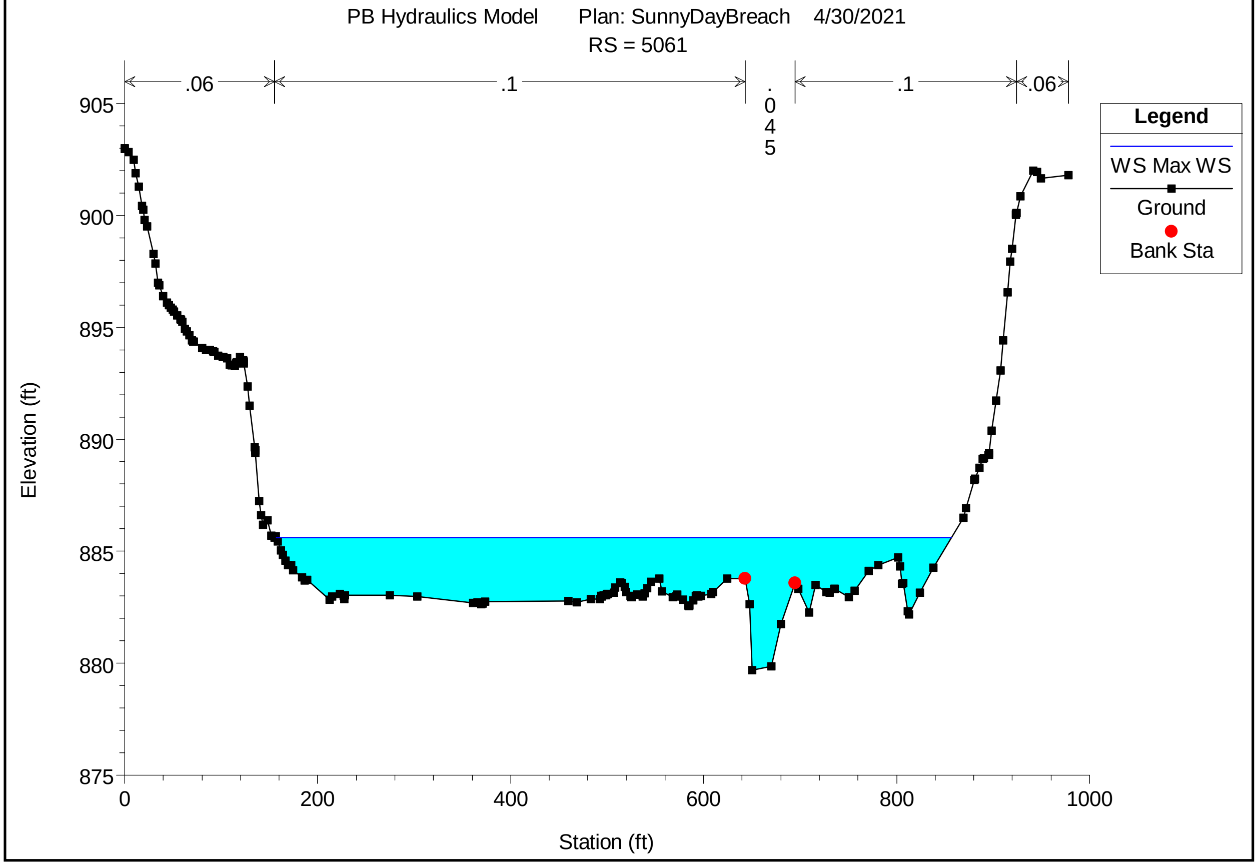
910
900
890
880
870
0
200
400
600
800
1000
1200
Station (ft)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.1
.045
.1
.06
Legend
WS Max WS
Ground
Bank Sta
Elevation (ft)
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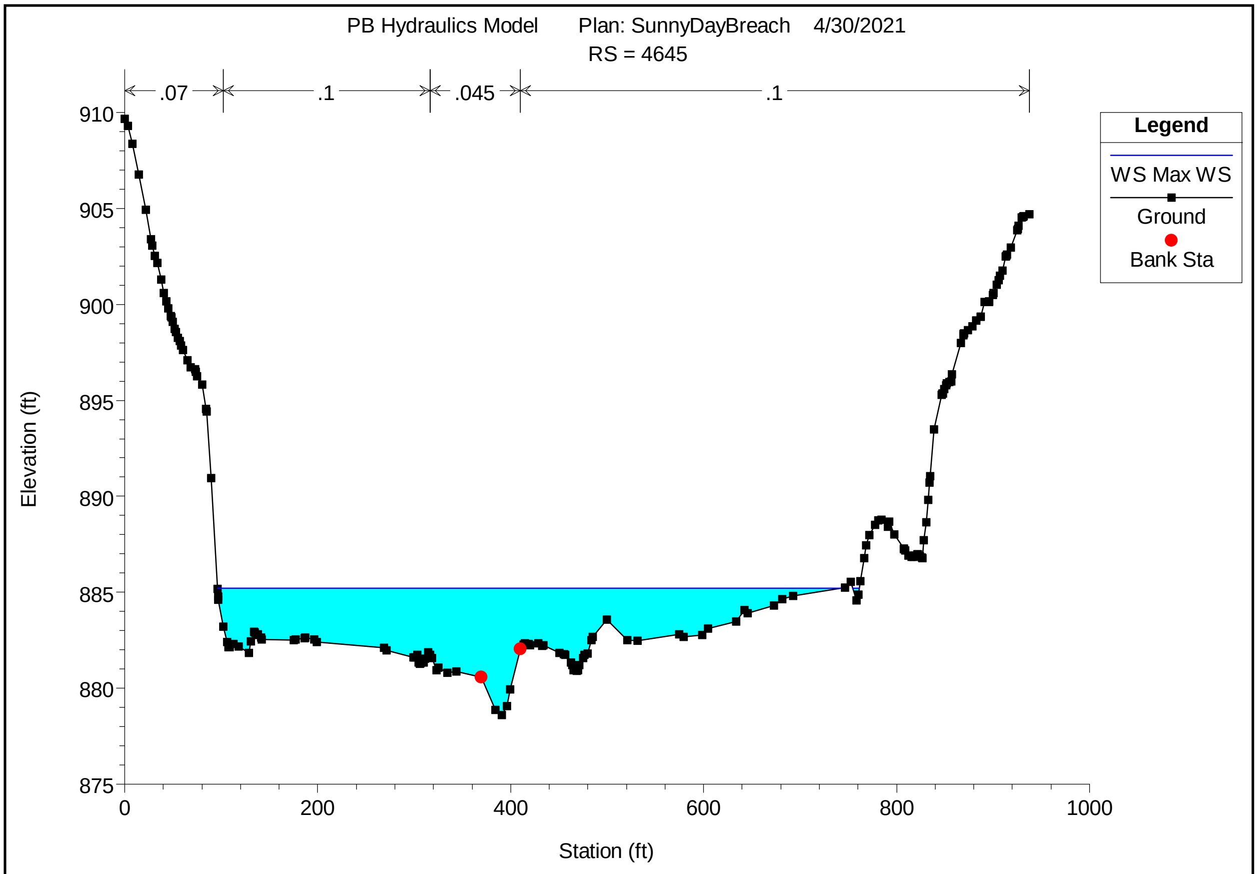
900
895
890
885
880
875
0
200
400
600
800
1000
Station (ft)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RS = 4645
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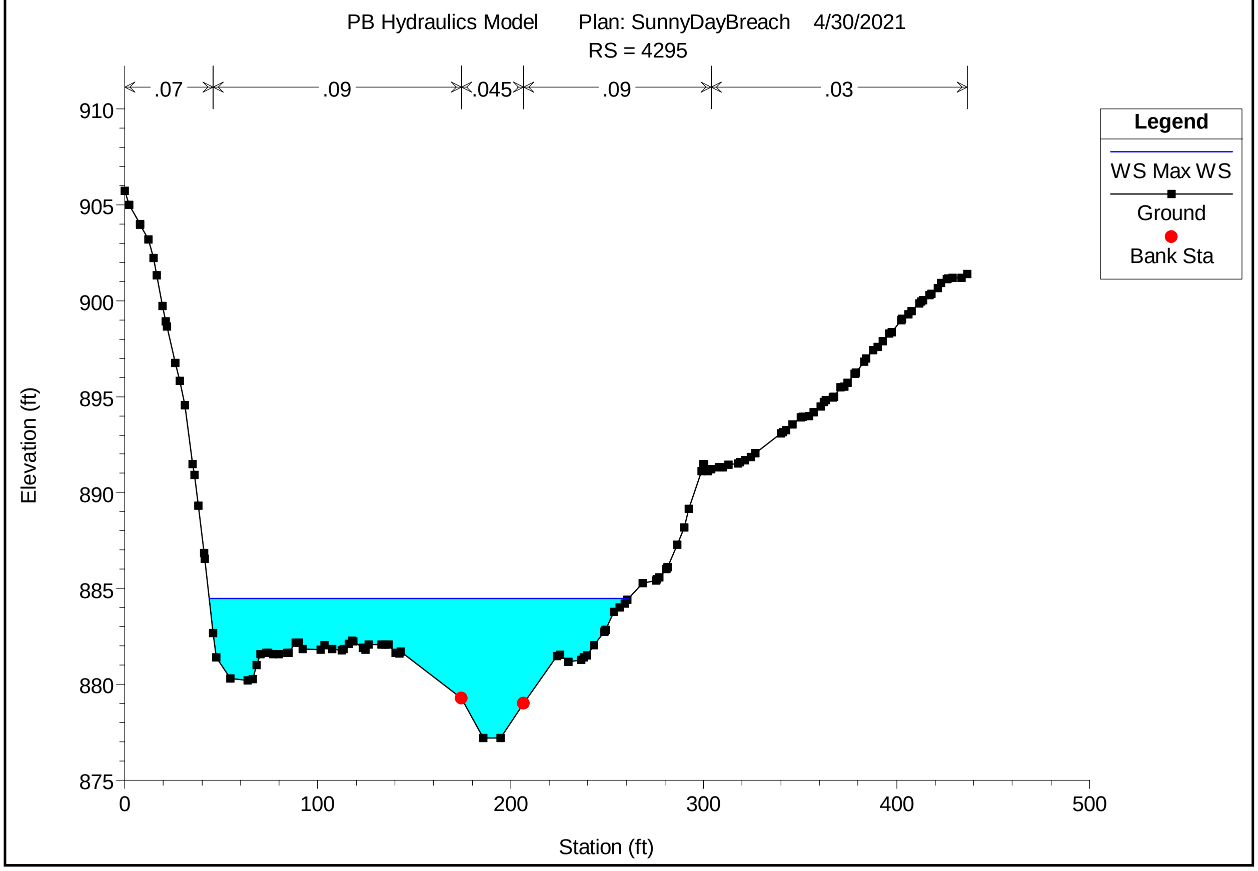
.1
.045
.1
910
905
900
895
890
885
880
875
Elevation (ft)
0
200
400
600
800
1000
Station (ft)
Legend
WS Max WS
Ground
Bank Sta


PB Hydraulics Model Plan: SunnyDayBreach 4/30/2021
RS = 4295
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.09
.045
.09
.03
910
905
900
895
890
885
880
875
Elevation (ft)
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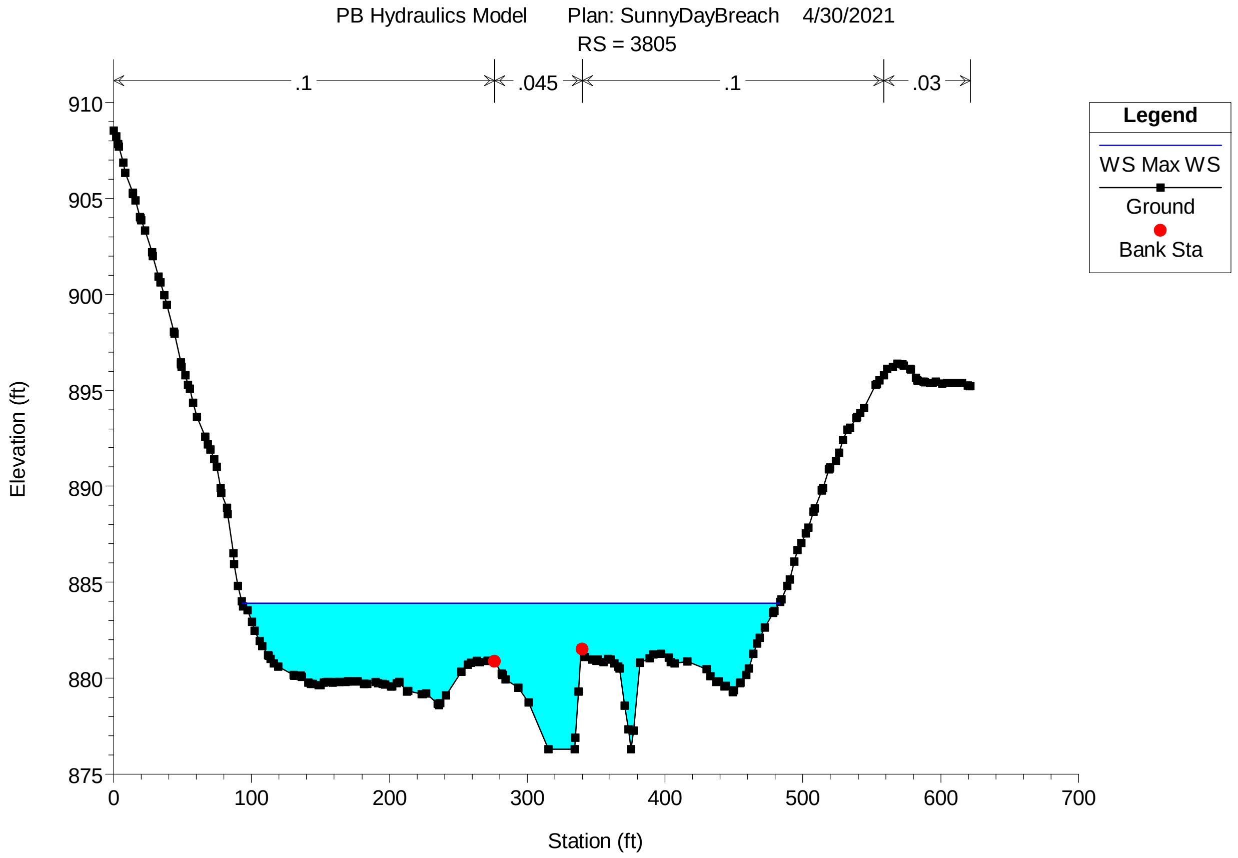
100
200
300
400
500
Station (ft)
Legend
WS Max WS
Ground
Bank Sta



PB Hydraulics Model Plan: SunnyDayBreach 4/30/2021
RS = 3805
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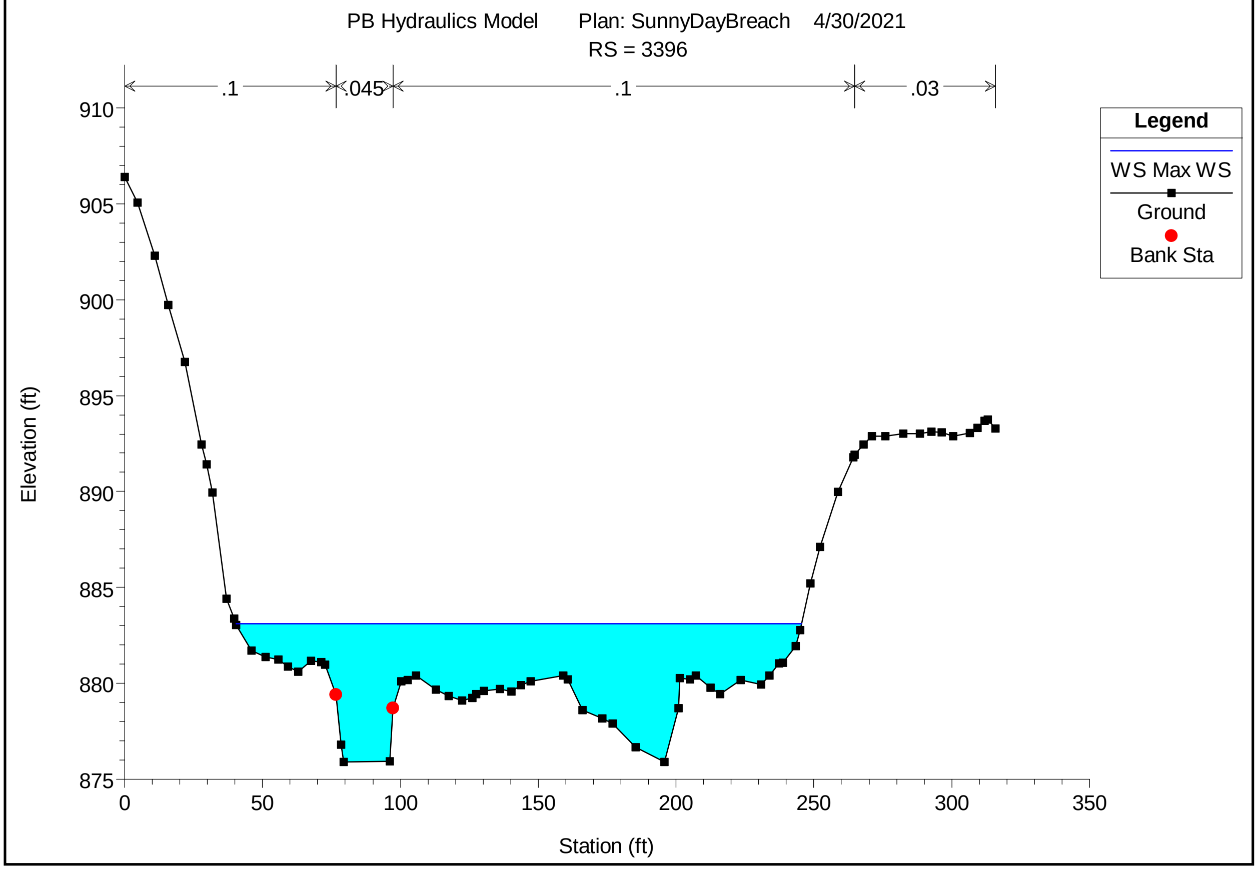
.045
.1
.03
Legend
WS Max WS
Ground
Bank Sta
Elevation (ft)
910
905
900
895
890
885
880
875
0
100
200
300
400
500
600
700
Station (ft)



PB Hydraulics Model Plan: SunnyDayBreach 4/30/2021
RS = 3396
.1
.045
.1
.03
Legend
WS Max WS
Ground
Bank Sta
Elevation (ft)
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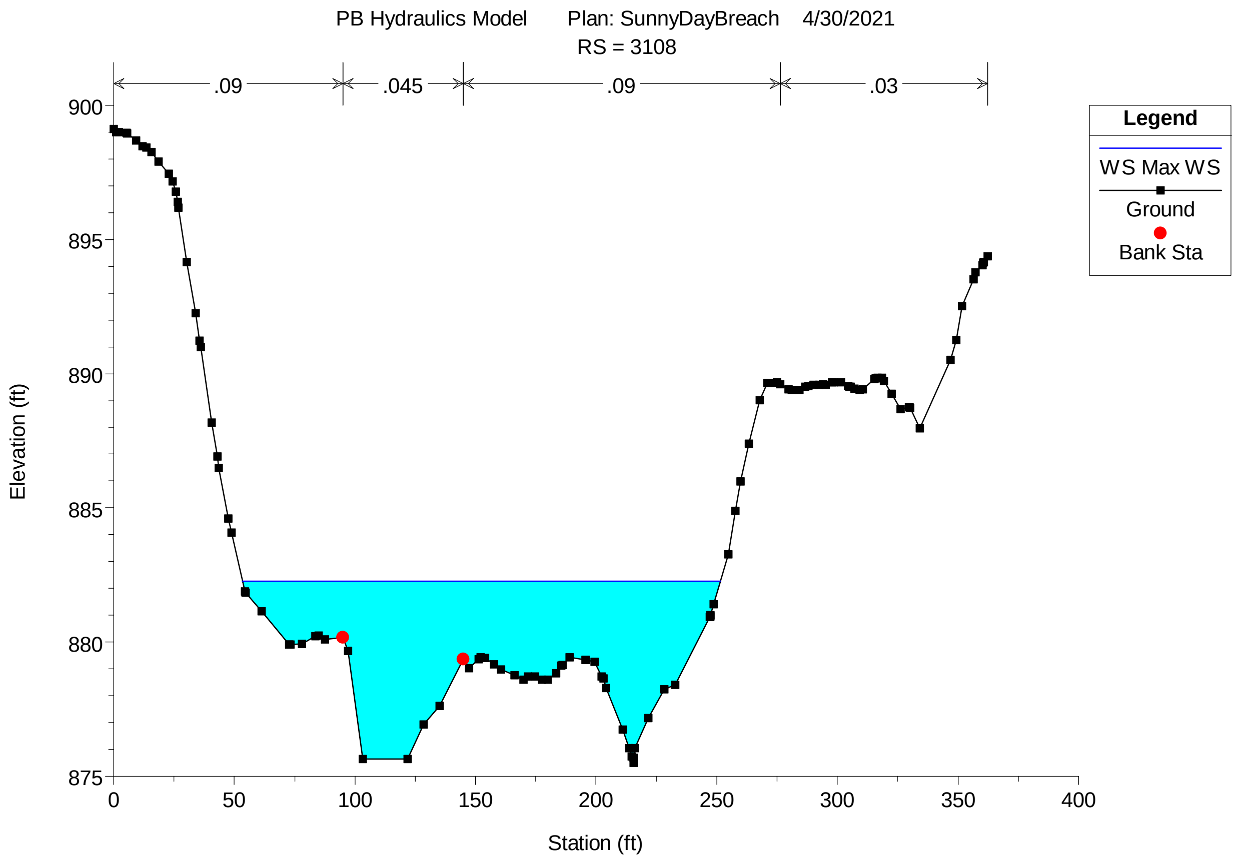
905
900
895
890
885
880
875
0
50
100
150
200
250
300
350
Station (ft)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RS = 3108
.09
.045
.09
.03
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895
890
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875
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200
250
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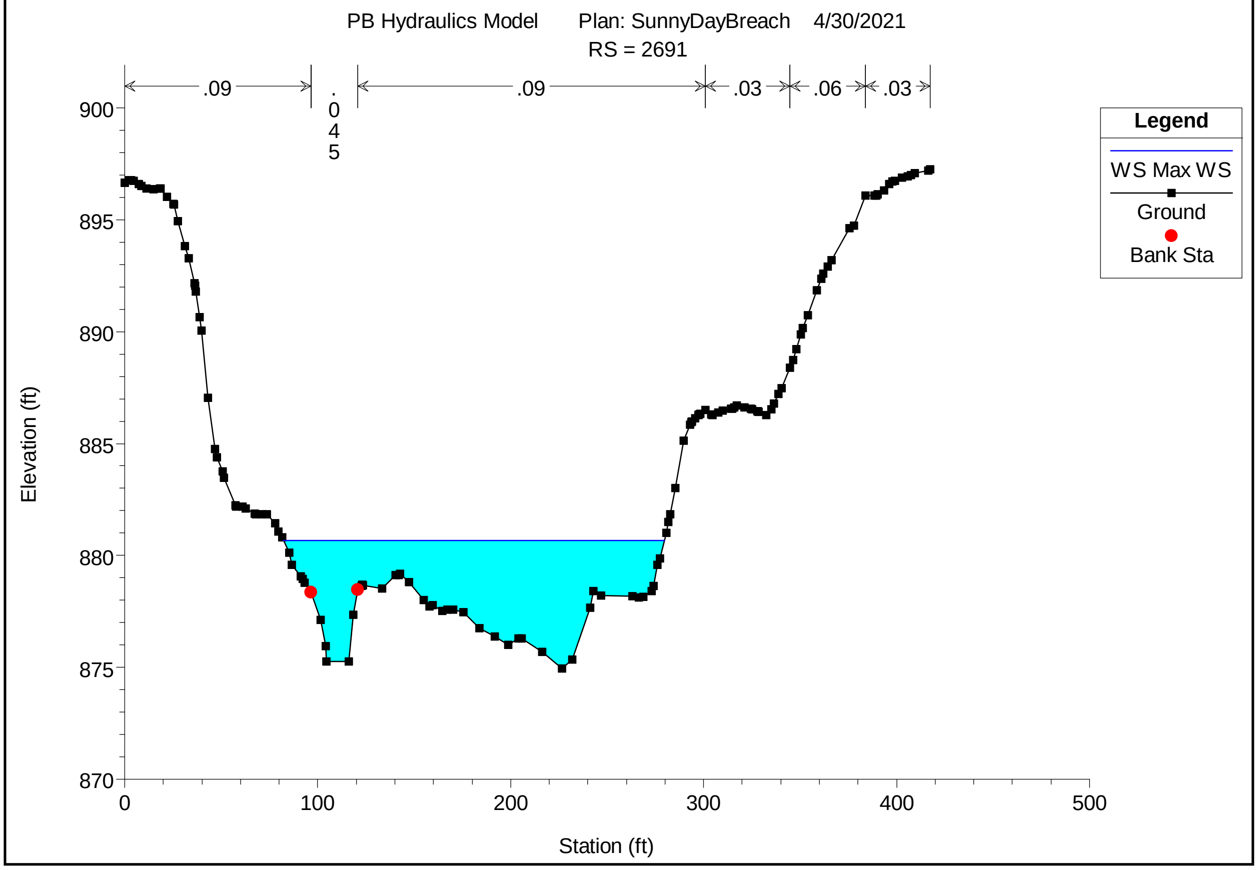
350
400
Elevation (ft)
Station (ft)
Legend
WS Max WS
Ground
Bank Sta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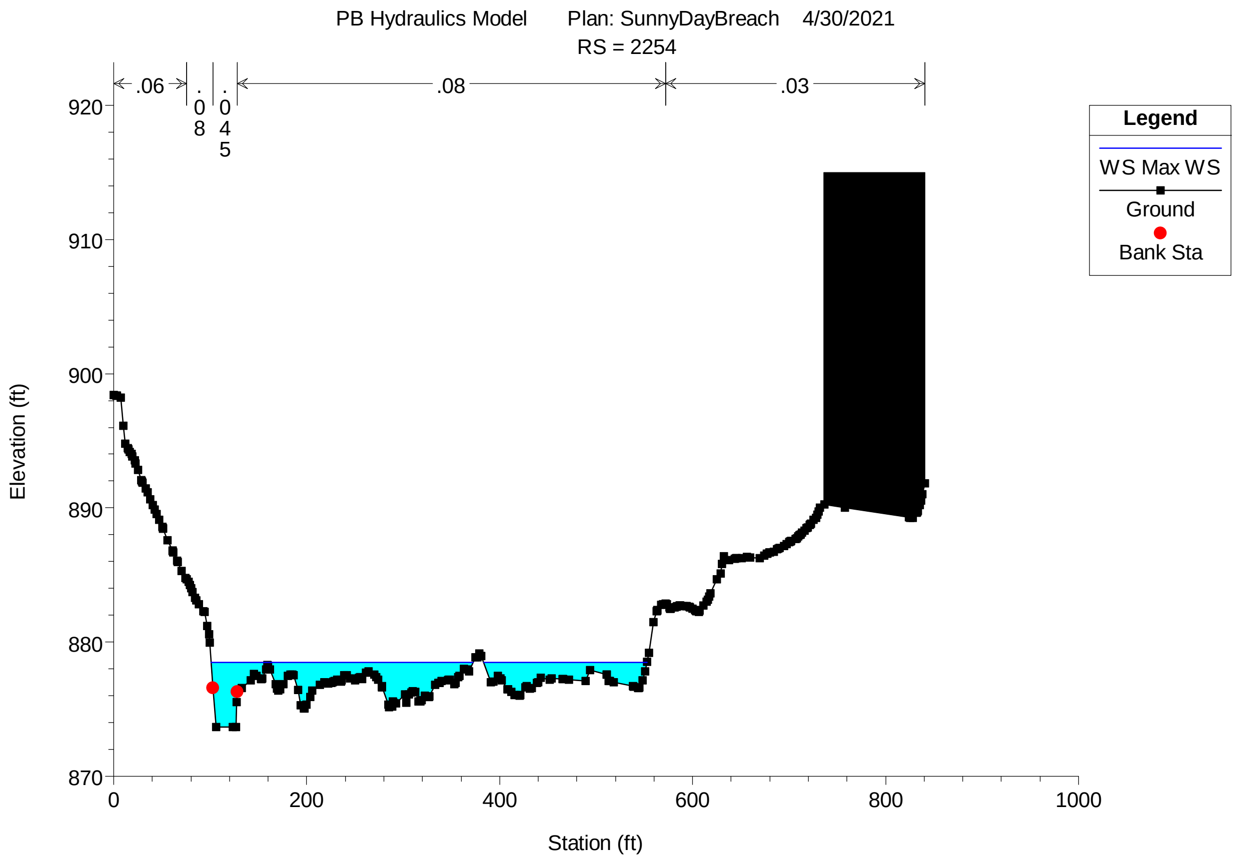
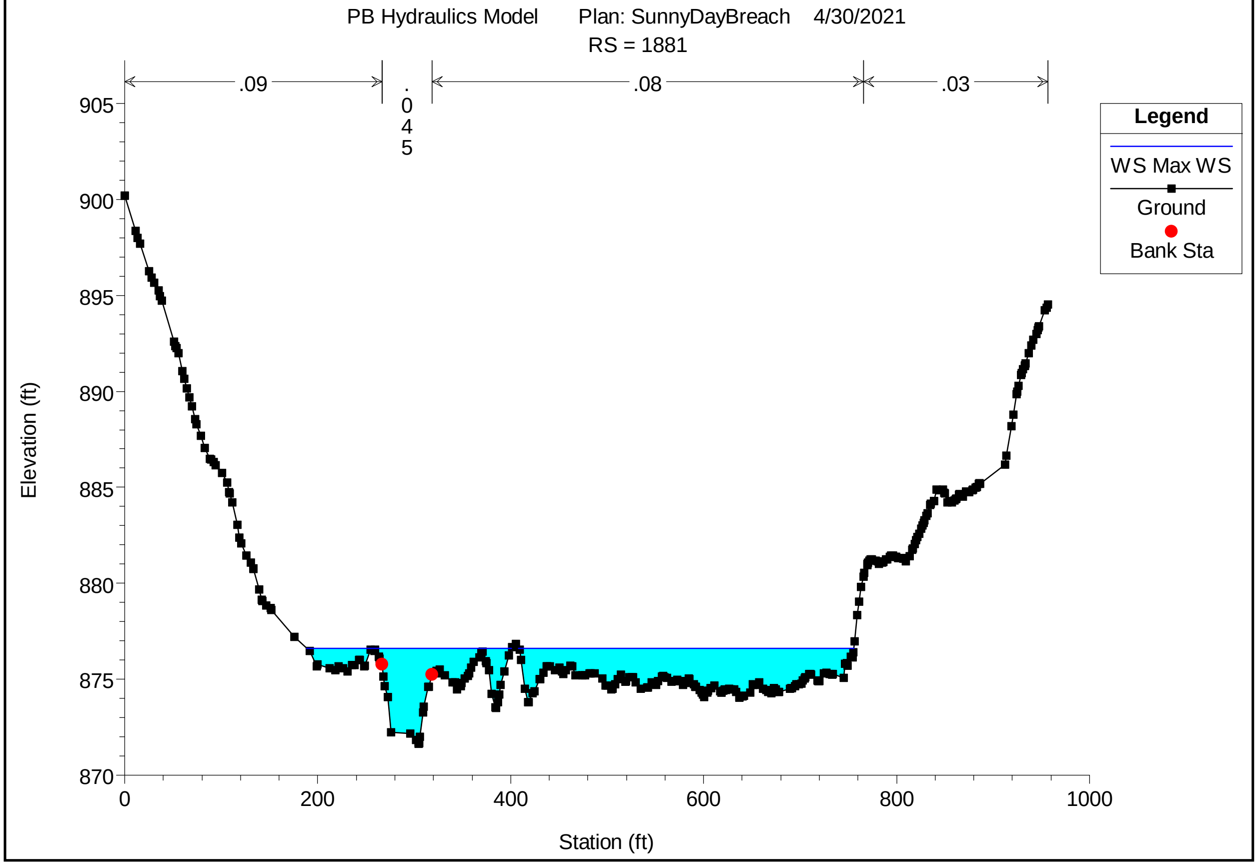
RS = 2691
.09
.045
.09
.03
.06
.03
900
895
890
885
880
875
870
0
100
200
300
400
500
Elevation (ft)
Station (ft)
Legend
WS Max WS
Ground
Bank Sta


PB Hydraulics Model Plan: SunnyDayBreach 4/30/2021
RS = 2254
.06
.08
.045
.08
.03
Legend
WS Max WS
Ground
Bank Sta
Elevation (ft)
920
910
900
890
880
870
0
200
400
600
800
1000
Station (ft)


PB Hydraulics Model Plan: SunnyDayBreach 4/30/2021
RS = 1881
.09
.045
.08
.03
Legend
WS Max WS
Ground
Bank Sta
Elevation (ft)
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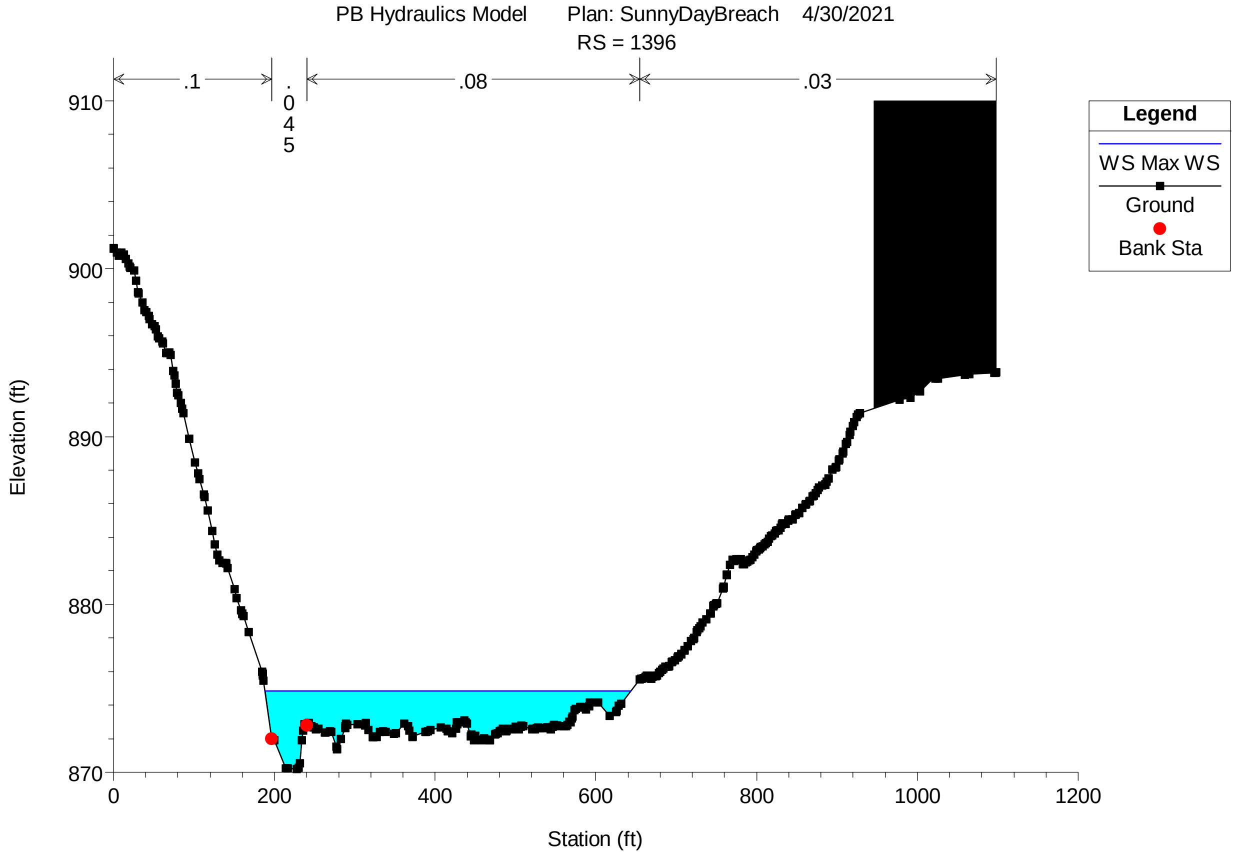
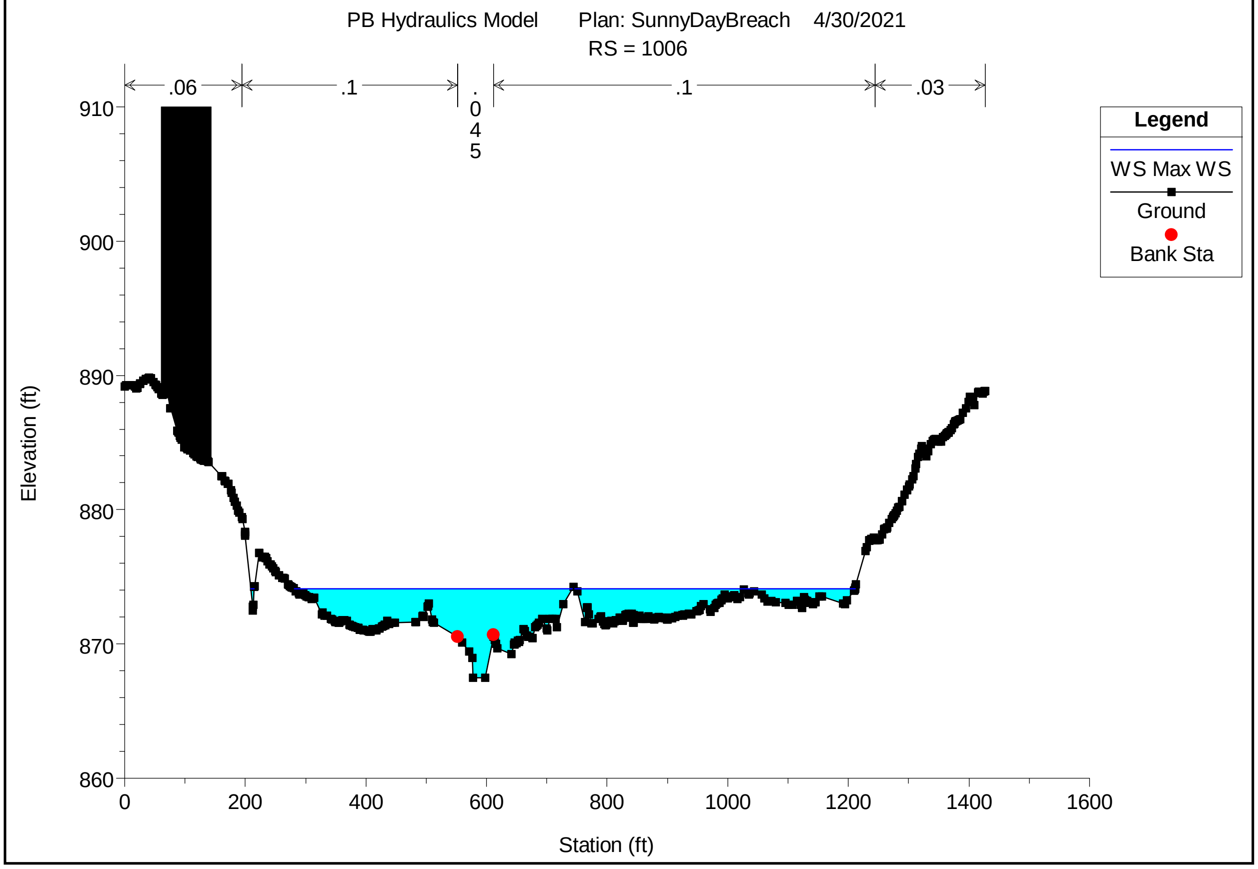
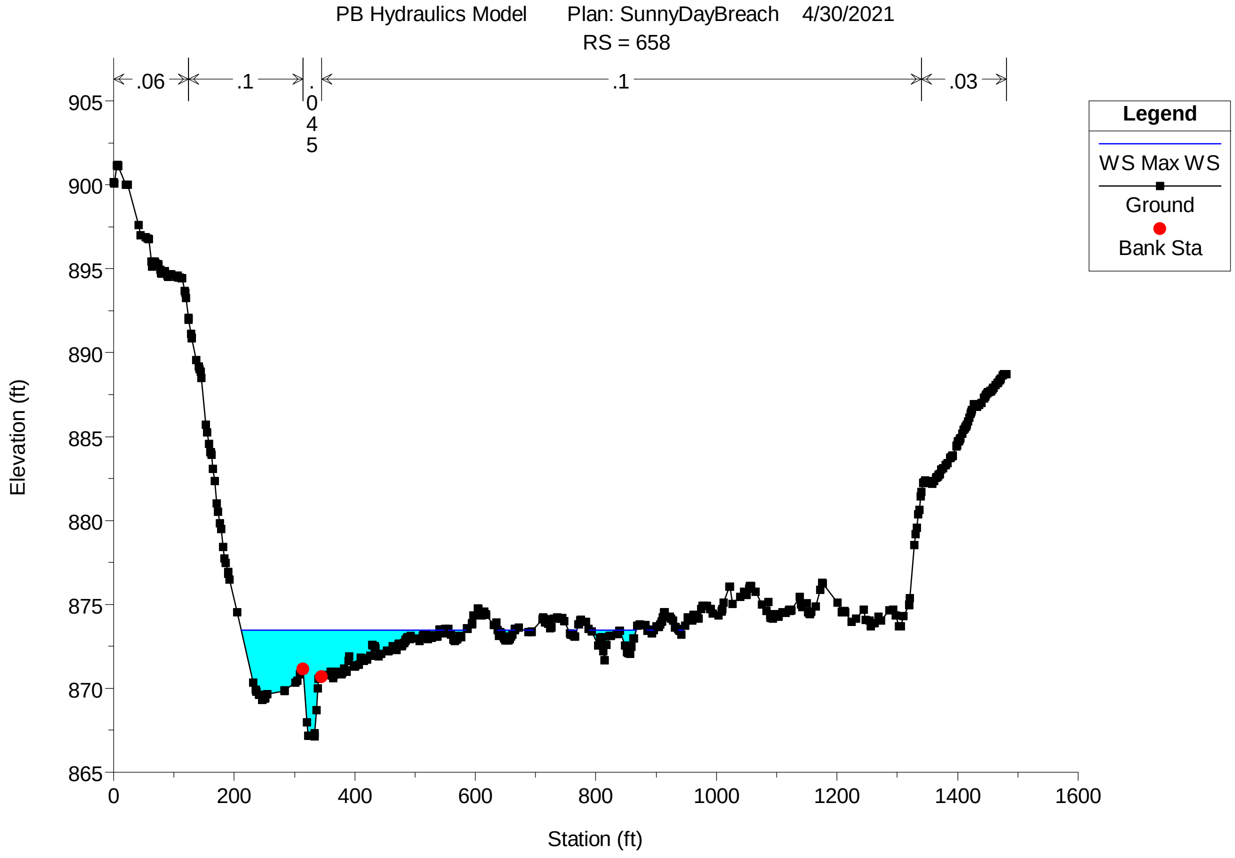
900
895
890
885
880
875
870
0
200
400
600
800
1000
Station (ft)


PB Hydraulics Model Plan: SunnyDayBreach 4/30/2021
RS = 1396
.1
.045
.08
.03
Legend
WS Max WS
Ground
Bank Sta
Elevation (ft)
910
900
890
880
870
0
200
400
600
800
1000
1200
Station (ft)


PB Hydraulics Model Plan: SunnyDayBreach 4/30/2021
RS = 1006
.06
.1
.045
.1
.03
Legend
WS Max WS
Ground
Bank Sta
Elevation (ft)
910
900
890
880
870
860
0
200
400
600
800
1000
1200
1400
1600
Station (ft)


PB Hydraulics Model Plan: SunnyDayBreach 4/30/2021
RS = 658
.06
.1
.045
.1
.03
905
900
895
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880
875
870
865
Elevation (ft)
0
200
400
600
800
1000
1200
1400
1600
Station (ft)
Legend
WS Max WS
Ground
Bank Sta

Plan: SunnyDay   PB2   Main RS: 16375   Profile: Max WS

E.G. Elev (ft)	937.12	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.64	Wt. n-Val.	0.060	0.045	0.070
W.S. Elev (ft)	936.48	Reach Len. (ft)	23.34	23.69	23.06
Crit W.S. (ft)		Flow Area (sq ft)	1201.01	285.26	550.17
E.G. Slope (ft/ft)	0.003039	Area (sq ft)	1201.01	285.26	550.17
Q Total (cfs)	10761.39	Flow (cfs)	5618.77	2834.76	2307.86
Top Width (ft)	287.58	Top Width (ft)	186.54	21.83	79.21
Vel Total (ft/s)	5.28	Avg. Vel. (ft/s)	4.68	9.94	4.19
Max Chl Dpth (ft)	14.20	Hydr. Depth (ft)	6.44	13.07	6.95
Conv. Total (cfs)	195200.3	Conv. (cfs)	101918.6	51419.6	41862.1
Length Wtd. (ft)	23.37	Wetted Per. (ft)	189.35	22.37	81.07
Min Ch El (ft)	922.28	Shear (lb/sq ft)	1.20	2.42	1.29
Alpha	1.48	Stream Power (lb/ft s)	5.63	24.05	5.40
Frctn Loss (ft)	0.07	Cum Volume (acre-ft)	183.25	88.82	183.38
C & E Loss (ft)		Cum SA (acres)	50.05	12.01	62.98

Plan: SunnyDay   PB2   Main RS: 16043.32   Profile: Max WS

E.G. Elev (ft)	936.02	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.36	Wt. n-Val.	0.060	0.045	0.080
W.S. Elev (ft)	935.66	Reach Len. (ft)	24.93	24.67	25.64
Crit W.S. (ft)		Flow Area (sq ft)	1127.97	275.25	1402.07
E.G. Slope (ft/ft)	0.001977	Area (sq ft)	1127.97	275.25	1402.07
Q Total (cfs)	10579.53	Flow (cfs)	3903.03	2290.96	4385.55
Top Width (ft)	407.91	Top Width (ft)	199.80	19.96	188.15
Vel Total (ft/s)	3.77	Avg. Vel. (ft/s)	3.46	8.32	3.13
Max Chl Dpth (ft)	14.48	Hydr. Depth (ft)	5.65	13.79	7.45
Conv. Total (cfs)	237949.6	Conv. (cfs)	87784.9	51527.0	98637.6
Length Wtd. (ft)	25.16	Wetted Per. (ft)	202.48	20.39	190.20
Min Ch El (ft)	921.18	Shear (lb/sq ft)	0.69	1.67	0.91
Alpha	1.65	Stream Power (lb/ft s)	2.38	13.87	2.85
Frctn Loss (ft)	0.05	Cum Volume (acre-ft)	174.71	86.70	177.89
C & E Loss (ft)		Cum SA (acres)	48.61	11.85	62.02

Plan: SunnyDay   PB2   Main RS: 15796.56   Profile: Max WS

E.G. Elev (ft)	935.04	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.36	Wt. n-Val.	0.060	0.090	0.080
W.S. Elev (ft)	934.68	Reach Len. (ft)	22.19	23.59	24.18
Crit W.S. (ft)		Flow Area (sq ft)	1556.43	285.00	389.83
E.G. Slope (ft/ft)	0.004595	Area (sq ft)	1556.43	285.00	389.83
Q Total (cfs)	10409.67	Flow (cfs)	7522.22	1636.69	1250.76
Top Width (ft)	434.92	Top Width (ft)	316.48	23.44	95.00
Vel Total (ft/s)	4.67	Avg. Vel. (ft/s)	4.83	5.74	3.21
Max Chl Dpth (ft)	13.81	Hydr. Depth (ft)	4.92	12.16	4.10
Conv. Total (cfs)	153559.8	Conv. (cfs)	110965.2	24143.8	18450.8
Length Wtd. (ft)	22.69	Wetted Per. (ft)	318.65	24.52	95.84
Min Ch El (ft)	920.87	Shear (lb/sq ft)	1.40	3.33	1.17
Alpha	1.07	Stream Power (lb/ft s)	6.77	19.15	3.74
Frctn Loss (ft)	0.11	Cum Volume (acre-ft)	167.38	85.11	173.23
C & E Loss (ft)		Cum SA (acres)	47.19	11.73	61.15

Plan: SunnyDay PB2 Main RS: 15513.48 Profile: Max WS

E.G. Elev (ft)	932.91	Element	Left OB	Channel	Right OB
Vel Head (ft)	1.59	Wt. n-Val.	0.060	0.045	0.090
W.S. Elev (ft)	931.32	Reach Len. (ft)	20.78	23.73	23.18
Crit W.S. (ft)		Flow Area (sq ft)	527.83	319.21	589.46
E.G. Slope (ft/ft)	0.007075	Area (sq ft)	527.83	319.21	589.46
Q Total (cfs)	10121.38	Flow (cfs)	3060.98	4482.83	2577.57
Top Width (ft)	243.26	Top Width (ft)	112.23	26.48	104.54
Vel Total (ft/s)	7.05	Avg. Vel. (ft/s)	5.80	14.04	4.37
Max Chl Dpth (ft)	14.11	Hydr. Depth (ft)	4.70	12.05	5.64
Conv. Total (cfs)	120332.9	Conv. (cfs)	36392.0	53296.3	30644.6
Length Wtd. (ft)	22.68	Wetted Per. (ft)	113.63	28.07	105.50
Min Ch El (ft)	917.21	Shear (lb/sq ft)	2.05	5.02	2.47
Alpha	2.06	Stream Power (lb/ft s)	11.90	70.53	10.79
Frctn Loss (ft)	0.16	Cum Volume (acre-ft)	162.33	83.15	169.99
C & E Loss (ft)		Cum SA (acres)	45.94	11.56	60.49

Plan: SunnyDay PB2 Main RS: 15133.91 Profile: Max WS

E.G. Elev (ft)	930.62	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.45	Wt. n-Val.	0.060	0.045	0.070
W.S. Elev (ft)	930.17	Reach Len. (ft)	24.94	24.26	23.47
Crit W.S. (ft)		Flow Area (sq ft)	1251.37	272.60	591.84
E.G. Slope (ft/ft)	0.002407	Area (sq ft)	1251.37	272.60	591.84
Q Total (cfs)	9521.73	Flow (cfs)	5183.67	2305.26	2032.81
Top Width (ft)	316.55	Top Width (ft)	197.63	21.96	96.96
Vel Total (ft/s)	4.50	Avg. Vel. (ft/s)	4.14	8.46	3.43
Max Chl Dpth (ft)	13.77	Hydr. Depth (ft)	6.33	12.41	6.10
Conv. Total (cfs)	194066.4	Conv. (cfs)	105650.5	46984.5	41431.5
Length Wtd. (ft)	24.47	Wetted Per. (ft)	198.80	22.86	98.82
Min Ch El (ft)	916.40	Shear (lb/sq ft)	0.95	1.79	0.90
Alpha	1.44	Stream Power (lb/ft s)	3.92	15.16	3.09
Frctn Loss (ft)	0.06	Cum Volume (acre-ft)	156.33	80.61	165.24
C & E Loss (ft)		Cum SA (acres)	44.77	11.35	59.65

Plan: SunnyDay PB2 Main RS: 14842.71 Profile: Max WS

E.G. Elev (ft)	929.78	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.55	Wt. n-Val.	0.060	0.045	0.080
W.S. Elev (ft)	929.23	Reach Len. (ft)	23.89	23.86	24.39
Crit W.S. (ft)		Flow Area (sq ft)	1040.66	275.85	500.66
E.G. Slope (ft/ft)	0.003105	Area (sq ft)	1040.66	275.85	500.66
Q Total (cfs)	8775.52	Flow (cfs)	4458.93	2506.16	1810.43
Top Width (ft)	287.58	Top Width (ft)	189.15	23.92	74.51
Vel Total (ft/s)	4.83	Avg. Vel. (ft/s)	4.28	9.09	3.62
Max Chl Dpth (ft)	13.30	Hydr. Depth (ft)	5.50	11.53	6.72
Conv. Total (cfs)	157497.8	Conv. (cfs)	80026.3	44979.0	32492.5
Length Wtd. (ft)	23.99	Wetted Per. (ft)	190.19	25.14	76.66
Min Ch El (ft)	915.93	Shear (lb/sq ft)	1.06	2.13	1.27
Alpha	1.53	Stream Power (lb/ft s)	4.54	19.32	4.58
Frctn Loss (ft)	0.07	Cum Volume (acre-ft)	148.49	78.78	162.00
C & E Loss (ft)		Cum SA (acres)	43.42	11.20	59.11

Plan: SunnyDay PB2 Main RS: 14604.07 Profile: Max WS

E.G. Elev (ft)	929.14	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.27	Wt. n-Val.	0.060	0.045	0.080
W.S. Elev (ft)	928.86	Reach Len. (ft)	24.52	24.97	24.90
Crit W.S. (ft)		Flow Area (sq ft)	1426.80	245.21	642.97
E.G. Slope (ft/ft)	0.001848	Area (sq ft)	1426.80	245.21	642.97
Q Total (cfs)	8171.63	Flow (cfs)	4619.56	1689.62	1862.46
Top Width (ft)	380.62	Top Width (ft)	268.22	21.42	90.98
Vel Total (ft/s)	3.53	Avg. Vel. (ft/s)	3.24	6.89	2.90
Max Chl Dpth (ft)	13.34	Hydr. Depth (ft)	5.32	11.45	7.07
Conv. Total (cfs)	190075.0	Conv. (cfs)	107452.6	39301.1	43321.4
Length Wtd. (ft)	24.70	Wetted Per. (ft)	269.06	22.93	93.06
Min Ch El (ft)	915.52	Shear (lb/sq ft)	0.61	1.23	0.80
Alpha	1.42	Stream Power (lb/ft s)	1.98	8.50	2.31
Frctn Loss (ft)	0.05	Cum Volume (acre-ft)	141.90	77.35	158.81
C & E Loss (ft)		Cum SA (acres)	42.21	11.08	58.65

Plan: SunnyDay PB2 Main RS: 14204.66 Profile: Max WS

E.G. Elev (ft)	927.71	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.35	Wt. n-Val.	0.070	0.045	0.070
W.S. Elev (ft)	927.37	Reach Len. (ft)	37.46	22.43	15.55
Crit W.S. (ft)		Flow Area (sq ft)	1329.24	298.26	281.76
E.G. Slope (ft/ft)	0.001948	Area (sq ft)	1329.24	298.26	281.76
Q Total (cfs)	7130.78	Flow (cfs)	4199.95	2150.93	779.90
Top Width (ft)	293.58	Top Width (ft)	213.45	25.69	54.44
Vel Total (ft/s)	3.73	Avg. Vel. (ft/s)	3.16	7.21	2.77
Max Chl Dpth (ft)	13.65	Hydr. Depth (ft)	6.23	11.61	5.18
Conv. Total (cfs)	161562.2	Conv. (cfs)	95158.4	48733.6	17670.3
Length Wtd. (ft)	30.41	Wetted Per. (ft)	214.63	27.10	55.49
Min Ch El (ft)	913.72	Shear (lb/sq ft)	0.75	1.34	0.62
Alpha	1.61	Stream Power (lb/ft s)	2.38	9.65	1.71
Frctn Loss (ft)	0.06	Cum Volume (acre-ft)	117.67	72.37	151.22
C & E Loss (ft)		Cum SA (acres)	38.16	10.64	57.19

Plan: SunnyDay PB2 Main RS: 14002.74 Profile: Max WS

E.G. Elev (ft)	927.30	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.63	Wt. n-Val.	0.070	0.045	0.070
W.S. Elev (ft)	926.67	Reach Len. (ft)	12.83	21.36	13.10
Crit W.S. (ft)		Flow Area (sq ft)	1021.19	332.59	308.39
E.G. Slope (ft/ft)	0.002776	Area (sq ft)	1021.19	332.59	308.39
Q Total (cfs)	7064.64	Flow (cfs)	2994.08	3011.94	1058.62
Top Width (ft)	323.30	Top Width (ft)	240.07	27.29	55.94
Vel Total (ft/s)	4.25	Avg. Vel. (ft/s)	2.93	9.06	3.43
Max Chl Dpth (ft)	13.52	Hydr. Depth (ft)	4.25	12.19	5.51
Conv. Total (cfs)	134078.1	Conv. (cfs)	56823.9	57162.9	20091.3
Length Wtd. (ft)	16.66	Wetted Per. (ft)	240.61	28.01	57.35
Min Ch El (ft)	913.15	Shear (lb/sq ft)	0.74	2.06	0.93
Alpha	2.23	Stream Power (lb/ft s)	2.16	18.64	3.20
Frctn Loss (ft)	0.05	Cum Volume (acre-ft)	109.05	70.91	150.27
C & E Loss (ft)		Cum SA (acres)	36.65	10.52	57.01

Plan: SunnyDay PB2 Main RS: 13874.59 Profile: Max WS

E.G. Elev (ft)	927.24	Element	Left OB	Channel	Right OB
Vel Head (ft)	1.40	Wt. n-Val.	0.060	0.045	0.070
W.S. Elev (ft)	925.84	Reach Len. (ft)	24.63	23.28	16.99
Crit W.S. (ft)		Flow Area (sq ft)	196.25	361.11	477.83
E.G. Slope (ft/ft)	0.004548	Area (sq ft)	196.25	361.11	477.83
Q Total (cfs)	7046.54	Flow (cfs)	539.60	4225.86	2281.08
Top Width (ft)	197.69	Top Width (ft)	91.35	29.56	76.78
Vel Total (ft/s)	6.81	Avg. Vel. (ft/s)	2.75	11.70	4.77
Max Chl Dpth (ft)	13.12	Hydr. Depth (ft)	2.15	12.22	6.22
Conv. Total (cfs)	104489.2	Conv. (cfs)	8001.5	62662.9	33824.9
Length Wtd. (ft)	21.35	Wetted Per. (ft)	92.90	29.98	78.47
Min Ch El (ft)	912.72	Shear (lb/sq ft)	0.60	3.42	1.73
Alpha	1.94	Stream Power (lb/ft s)	1.65	40.03	8.25
Frctn Loss (ft)	0.10	Cum Volume (acre-ft)	108.08	69.89	149.58
C & E Loss (ft)		Cum SA (acres)	36.35	10.44	56.89

Plan: SunnyDay PB2 Main RS: 13665.05 Profile: Max WS

E.G. Elev (ft)	926.17	Element	Left OB	Channel	Right OB
Vel Head (ft)	1.97	Wt. n-Val.	0.070	0.045	0.080
W.S. Elev (ft)	924.20	Reach Len. (ft)	20.53	24.67	26.58
Crit W.S. (ft)		Flow Area (sq ft)	197.61	230.34	522.71
E.G. Slope (ft/ft)	0.009776	Area (sq ft)	197.61	230.34	522.71
Q Total (cfs)	7004.22	Flow (cfs)	909.30	3497.71	2597.22
Top Width (ft)	206.37	Top Width (ft)	59.94	21.68	124.75
Vel Total (ft/s)	7.37	Avg. Vel. (ft/s)	4.60	15.18	4.97
Max Chl Dpth (ft)	12.41	Hydr. Depth (ft)	3.30	10.62	4.19
Conv. Total (cfs)	70841.1	Conv. (cfs)	9196.7	35376.0	26268.4
Length Wtd. (ft)	24.82	Wetted Per. (ft)	60.87	22.96	125.48
Min Ch El (ft)	911.79	Shear (lb/sq ft)	1.98	6.12	2.54
Alpha	2.34	Stream Power (lb/ft s)	9.12	92.96	12.63
Frctn Loss (ft)	0.24	Cum Volume (acre-ft)	107.06	68.48	147.88
C & E Loss (ft)		Cum SA (acres)	35.99	10.31	56.56

Plan: SunnyDay PB2 Main RS: 13146.93 Profile: Max WS

E.G. Elev (ft)	921.11	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.63	Wt. n-Val.	0.070	0.045	0.080
W.S. Elev (ft)	920.48	Reach Len. (ft)	24.94	24.61	26.27
Crit W.S. (ft)		Flow Area (sq ft)	429.39	281.41	621.70
E.G. Slope (ft/ft)	0.003561	Area (sq ft)	429.39	281.41	621.70
Q Total (cfs)	6017.07	Flow (cfs)	1620.32	2534.61	1862.13
Top Width (ft)	270.56	Top Width (ft)	82.75	27.97	159.84
Vel Total (ft/s)	4.52	Avg. Vel. (ft/s)	3.77	9.01	3.00
Max Chl Dpth (ft)	11.53	Hydr. Depth (ft)	5.19	10.06	3.89
Conv. Total (cfs)	100835.1	Conv. (cfs)	27153.7	42475.5	31205.9
Length Wtd. (ft)	25.21	Wetted Per. (ft)	83.51	28.79	161.29
Min Ch El (ft)	908.95	Shear (lb/sq ft)	1.14	2.17	0.86
Alpha	2.00	Stream Power (lb/ft s)	4.31	19.57	2.57
Frctn Loss (ft)	0.09	Cum Volume (acre-ft)	104.35	65.56	141.79
C & E Loss (ft)		Cum SA (acres)	35.30	10.02	55.05

Plan: SunnyDay PB2 Main RS: 12777.75 Profile: Max WS

E.G. Elev (ft)	920.11	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.13	Wt. n-Val.	0.060	0.045	0.060
W.S. Elev (ft)	919.98	Reach Len. (ft)	24.29	24.39	23.11
Crit W.S. (ft)		Flow Area (sq ft)	703.29	288.82	1320.86
E.G. Slope (ft/ft)	0.000704	Area (sq ft)	703.29	288.82	1320.86
Q Total (cfs)	5809.15	Flow (cfs)	1489.34	1301.94	3017.87
Top Width (ft)	347.79	Top Width (ft)	120.82	24.20	202.77
Vel Total (ft/s)	2.51	Avg. Vel. (ft/s)	2.12	4.51	2.28
Max Chl Dpth (ft)	13.03	Hydr. Depth (ft)	5.82	11.93	6.51
Conv. Total (cfs)	218892.3	Conv. (cfs)	56119.2	49058.0	113715.1
Length Wtd. (ft)	23.71	Wetted Per. (ft)	121.60	24.76	203.79
Min Ch El (ft)	906.95	Shear (lb/sq ft)	0.25	0.51	0.28
Alpha	1.33	Stream Power (lb/ft s)	0.54	2.31	0.65
Frctn Loss (ft)	0.02	Cum Volume (acre-ft)	99.87	63.16	134.96
C & E Loss (ft)		Cum SA (acres)	34.49	9.80	53.71

Plan: SunnyDay PB2 Main RS: 12363.13 Profile: Max WS

E.G. Elev (ft)	919.01	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.70	Wt. n-Val.	0.060	0.070	0.045
W.S. Elev (ft)	918.31	Reach Len. (ft)	24.20	22.76	20.65
Crit W.S. (ft)		Flow Area (sq ft)	361.07	251.90	239.29
E.G. Slope (ft/ft)	0.005225	Area (sq ft)	361.07	251.90	239.29
Q Total (cfs)	5536.72	Flow (cfs)	1945.70	1964.40	1626.62
Top Width (ft)	137.44	Top Width (ft)	67.89	20.95	48.60
Vel Total (ft/s)	6.50	Avg. Vel. (ft/s)	5.39	7.80	6.80
Max Chl Dpth (ft)	13.56	Hydr. Depth (ft)	5.32	12.02	4.92
Conv. Total (cfs)	76594.4	Conv. (cfs)	26916.6	27175.3	22502.5
Length Wtd. (ft)	22.66	Wetted Per. (ft)	69.14	21.99	49.79
Min Ch El (ft)	904.75	Shear (lb/sq ft)	1.70	3.74	1.57
Alpha	1.07	Stream Power (lb/ft s)	9.18	29.15	10.66
Frctn Loss (ft)	0.12	Cum Volume (acre-ft)	95.22	60.55	129.87
C & E Loss (ft)		Cum SA (acres)	33.66	9.58	52.70

Plan: SunnyDay PB2 Main RS: 12112.81 Profile: Max WS

E.G. Elev (ft)	917.95	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.30	Wt. n-Val.	0.070	0.045	0.100
W.S. Elev (ft)	917.65	Reach Len. (ft)	22.76	24.10	21.23
Crit W.S. (ft)		Flow Area (sq ft)	692.17	256.23	1022.33
E.G. Slope (ft/ft)	0.001576	Area (sq ft)	692.17	256.23	1022.33
Q Total (cfs)	5472.80	Flow (cfs)	1805.83	1786.43	1880.53
Top Width (ft)	330.83	Top Width (ft)	125.96	20.01	184.85
Vel Total (ft/s)	2.78	Avg. Vel. (ft/s)	2.61	6.97	1.84
Max Chl Dpth (ft)	14.17	Hydr. Depth (ft)	5.50	12.80	5.53
Conv. Total (cfs)	137841.3	Conv. (cfs)	45482.8	44994.3	47364.3
Length Wtd. (ft)	22.75	Wetted Per. (ft)	127.09	20.89	185.69
Min Ch El (ft)	903.48	Shear (lb/sq ft)	0.54	1.21	0.54
Alpha	2.50	Stream Power (lb/ft s)	1.40	8.41	1.00
Frctn Loss (ft)	0.04	Cum Volume (acre-ft)	92.54	59.11	127.57
C & E Loss (ft)		Cum SA (acres)	33.14	9.47	52.22

Plan: SunnyDay   PB2   Main RS: 11727.17   Profile: Max WS

E.G. Elev (ft)	916.65	Element	Left OB	Channel	Right OB
Vel Head (ft)	2.58	Wt. n-Val.	0.070	0.045	0.080
W.S. Elev (ft)	914.07	Reach Len. (ft)	19.45	24.27	31.66
Crit W.S. (ft)		Flow Area (sq ft)	56.21	327.84	148.37
E.G. Slope (ft/ft)	0.007892	Area (sq ft)	56.21	327.84	148.37
Q Total (cfs)	5405.68	Flow (cfs)	228.00	4561.41	616.27
Top Width (ft)	81.70	Top Width (ft)	16.07	29.90	35.73
Vel Total (ft/s)	10.15	Avg. Vel. (ft/s)	4.06	13.91	4.15
Max Chl Dpth (ft)	13.11	Hydr. Depth (ft)	3.50	10.96	4.15
Conv. Total (cfs)	60849.7	Conv. (cfs)	2566.5	51346.1	6937.1
Length Wtd. (ft)	24.90	Wetted Per. (ft)	17.82	31.74	37.15
Min Ch El (ft)	900.96	Shear (lb/sq ft)	1.55	5.09	1.97
Alpha	1.61	Stream Power (lb/ft s)	6.30	70.81	8.17
Frctn Loss (ft)	0.20	Cum Volume (acre-ft)	90.58	56.48	124.85
C & E Loss (ft)		Cum SA (acres)	32.69	9.24	51.66

Plan: SunnyDay   PB2   Main RS: 11314.57   Profile: Max WS

E.G. Elev (ft)	913.48	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.35	Wt. n-Val.	0.090	0.045	0.070
W.S. Elev (ft)	913.13	Reach Len. (ft)	21.30	23.95	21.75
Crit W.S. (ft)		Flow Area (sq ft)	258.19	385.19	232.23
E.G. Slope (ft/ft)	0.000996	Area (sq ft)	258.19	385.19	232.23
Q Total (cfs)	3041.72	Flow (cfs)	442.58	2126.83	472.32
Top Width (ft)	112.22	Top Width (ft)	40.47	29.71	42.04
Vel Total (ft/s)	3.47	Avg. Vel. (ft/s)	1.71	5.52	2.03
Max Chl Dpth (ft)	15.06	Hydr. Depth (ft)	6.38	12.96	5.52
Conv. Total (cfs)	96375.5	Conv. (cfs)	14022.8	67387.5	14965.2
Length Wtd. (ft)	23.21	Wetted Per. (ft)	43.27	31.59	43.90
Min Ch El (ft)	898.07	Shear (lb/sq ft)	0.37	0.76	0.33
Alpha	1.86	Stream Power (lb/ft s)	0.64	4.19	0.67
Frctn Loss (ft)	0.02	Cum Volume (acre-ft)	89.55	53.19	122.71
C & E Loss (ft)		Cum SA (acres)	32.47	8.96	51.19

Plan: SunnyDay   PB2   Main RS: 10931.33   Profile: Max WS

E.G. Elev (ft)	913.12	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.09	Wt. n-Val.	0.060	0.045	0.070
W.S. Elev (ft)	913.03	Reach Len. (ft)	23.40	23.33	25.11
Crit W.S. (ft)		Flow Area (sq ft)	579.36	249.01	756.40
E.G. Slope (ft/ft)	0.000454	Area (sq ft)	579.36	249.01	756.40
Q Total (cfs)	2973.77	Flow (cfs)	902.44	901.46	1169.87
Top Width (ft)	252.15	Top Width (ft)	113.38	20.20	118.58
Vel Total (ft/s)	1.88	Avg. Vel. (ft/s)	1.56	3.62	1.55
Max Chl Dpth (ft)	13.83	Hydr. Depth (ft)	5.11	12.33	6.38
Conv. Total (cfs)	139556.4	Conv. (cfs)	42350.7	42304.6	54901.1
Length Wtd. (ft)	24.01	Wetted Per. (ft)	114.25	21.34	119.63
Min Ch El (ft)	899.20	Shear (lb/sq ft)	0.14	0.33	0.18
Alpha	1.60	Stream Power (lb/ft s)	0.22	1.20	0.28
Frctn Loss (ft)	0.01	Cum Volume (acre-ft)	86.81	50.42	119.44
C & E Loss (ft)		Cum SA (acres)	31.99	8.74	50.55

Plan: SunnyDay PB2 Main RS: 10628.11 Profile: Max WS

E.G. Elev (ft)	912.99	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.03	Wt. n-Val.	0.080	0.045	0.080
W.S. Elev (ft)	912.96	Reach Len. (ft)	28.00	23.89	20.64
Crit W.S. (ft)		Flow Area (sq ft)	2169.51	298.52	253.73
E.G. Slope (ft/ft)	0.000211	Area (sq ft)	2169.51	298.52	253.73
Q Total (cfs)	2953.02	Flow (cfs)	2037.02	724.04	191.96
Top Width (ft)	411.72	Top Width (ft)	334.14	25.40	52.19
Vel Total (ft/s)	1.08	Avg. Vel. (ft/s)	0.94	2.43	0.76
Max Chl Dpth (ft)	13.15	Hydr. Depth (ft)	6.49	11.75	4.86
Conv. Total (cfs)	203100.4	Conv. (cfs)	140100.5	49797.8	13202.2
Length Wtd. (ft)	26.50	Wetted Per. (ft)	334.66	26.29	54.12
Min Ch El (ft)	899.81	Shear (lb/sq ft)	0.09	0.15	0.06
Alpha	1.77	Stream Power (lb/ft s)	0.08	0.36	0.05
Frctn Loss (ft)	0.01	Cum Volume (acre-ft)	78.33	48.51	116.16
C & E Loss (ft)		Cum SA (acres)	30.51	8.58	49.94

Plan: SunnyDay PB2 Main RS: 10436.99 Profile: Max WS

E.G. Elev (ft)	912.96	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.01	Wt. n-Val.	0.060	0.045	0.060
W.S. Elev (ft)	912.96	Reach Len. (ft)	23.57	22.14	20.71
Crit W.S. (ft)		Flow Area (sq ft)	3008.36	367.86	1216.44
E.G. Slope (ft/ft)	0.000041	Area (sq ft)	3008.36	367.86	1216.44
Q Total (cfs)	2950.97	Flow (cfs)	1803.02	402.19	745.76
Top Width (ft)	595.62	Top Width (ft)	406.39	30.91	158.33
Vel Total (ft/s)	0.64	Avg. Vel. (ft/s)	0.60	1.09	0.61
Max Chl Dpth (ft)	12.74	Hydr. Depth (ft)	7.40	11.90	7.68
Conv. Total (cfs)	461941.8	Conv. (cfs)	282243.4	62958.4	116739.9
Length Wtd. (ft)	22.64	Wetted Per. (ft)	407.99	31.18	159.46
Min Ch El (ft)	900.22	Shear (lb/sq ft)	0.02	0.03	0.02
Alpha	1.16	Stream Power (lb/ft s)	0.01	0.03	0.01
Frctn Loss (ft)	0.00	Cum Volume (acre-ft)	65.11	47.05	113.92
C & E Loss (ft)		Cum SA (acres)	28.60	8.46	49.57

Plan: SunnyDay PB2 Main RS: 10282 Profile: Max WS

E.G. Elev (ft)	912.99	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.11	Wt. n-Val.	0.090	0.045	0.090
W.S. Elev (ft)	912.89	Reach Len. (ft)	496.48	453.40	419.71
Crit W.S. (ft)		Flow Area (sq ft)	236.40	524.31	610.93
E.G. Slope (ft/ft)	0.000303	Area (sq ft)	1971.24	524.31	1699.33
Q Total (cfs)	2950.16	Flow (cfs)	351.21	1700.34	898.62
Top Width (ft)	468.82	Top Width (ft)	246.95	38.40	183.47
Vel Total (ft/s)	2.15	Avg. Vel. (ft/s)	1.49	3.24	1.47
Max Chl Dpth (ft)	14.90	Hydr. Depth (ft)	11.77	13.65	11.63
Conv. Total (cfs)	169575.9	Conv. (cfs)	20187.4	97735.8	51652.8
Length Wtd. (ft)	453.40	Wetted Per. (ft)	20.10	39.09	52.72
Min Ch El (ft)	897.99	Shear (lb/sq ft)	0.22	0.25	0.22
Alpha	1.51	Stream Power (lb/ft s)	0.33	0.82	0.32
Frctn Loss (ft)		Cum Volume (acre-ft)	55.47	45.47	109.90
C & E Loss (ft)		Cum SA (acres)	27.33	8.34	49.05

Plan: SunnyDay   PB2   Main RS: 9829   Profile: Max WS

E.G. Elev (ft)	908.75	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.97	Wt. n-Val.	0.090	0.045	0.090
W.S. Elev (ft)	907.78	Reach Len. (ft)	20.95	21.69	21.59
Crit W.S. (ft)		Flow Area (sq ft)	44.48	351.81	22.99
E.G. Slope (ft/ft)	0.003163	Area (sq ft)	44.48	351.81	22.99
Q Total (cfs)	2948.48	Flow (cfs)	83.56	2829.59	35.33
Top Width (ft)	59.69	Top Width (ft)	14.18	35.91	9.60
Vel Total (ft/s)	7.03	Avg. Vel. (ft/s)	1.88	8.04	1.54
Max Chl Dpth (ft)	11.76	Hydr. Depth (ft)	3.14	9.80	2.40
Conv. Total (cfs)	52424.5	Conv. (cfs)	1485.7	50310.5	628.2
Length Wtd. (ft)	21.67	Wetted Per. (ft)	15.46	39.04	10.80
Min Ch El (ft)	896.02	Shear (lb/sq ft)	0.57	1.78	0.42
Alpha	1.26	Stream Power (lb/ft s)	1.07	14.31	0.65
Frctn Loss (ft)	0.07	Cum Volume (acre-ft)	55.47	41.62	109.90
C & E Loss (ft)		Cum SA (acres)	25.84	7.95	48.12

Plan: SunnyDay   PB2   Main RS: 9742   Profile: Max WS

E.G. Elev (ft)	908.31	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.62	Wt. n-Val.	0.090	0.045	0.090
W.S. Elev (ft)	907.69	Reach Len. (ft)	16.09	19.07	19.38
Crit W.S. (ft)		Flow Area (sq ft)	134.19	263.97	307.05
E.G. Slope (ft/ft)	0.002742	Area (sq ft)	134.19	263.97	307.05
Q Total (cfs)	2948.16	Flow (cfs)	156.24	1984.49	807.42
Top Width (ft)	167.05	Top Width (ft)	85.22	25.53	56.30
Vel Total (ft/s)	4.18	Avg. Vel. (ft/s)	1.16	7.52	2.63
Max Chl Dpth (ft)	11.89	Hydr. Depth (ft)	1.57	10.34	5.45
Conv. Total (cfs)	56301.6	Conv. (cfs)	2983.8	37898.3	15419.6
Length Wtd. (ft)	18.96	Wetted Per. (ft)	85.86	29.12	57.88
Min Ch El (ft)	895.80	Shear (lb/sq ft)	0.27	1.55	0.91
Alpha	2.29	Stream Power (lb/ft s)	0.31	11.67	2.39
Frctn Loss (ft)	0.05	Cum Volume (acre-ft)	55.34	41.02	109.76
C & E Loss (ft)		Cum SA (acres)	25.77	7.89	48.08

Plan: SunnyDay   PB2   Main RS: 9666   Profile: Max WS

E.G. Elev (ft)	908.02	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.25	Wt. n-Val.	0.090	0.045	0.090
W.S. Elev (ft)	907.77	Reach Len. (ft)	13.88	22.80	37.31
Crit W.S. (ft)		Flow Area (sq ft)	408.70	286.77	488.11
E.G. Slope (ft/ft)	0.001317	Area (sq ft)	408.70	286.77	488.11
Q Total (cfs)	2948.01	Flow (cfs)	513.00	1539.92	895.09
Top Width (ft)	250.96	Top Width (ft)	134.38	26.44	90.14
Vel Total (ft/s)	2.49	Avg. Vel. (ft/s)	1.26	5.37	1.83
Max Chl Dpth (ft)	12.10	Hydr. Depth (ft)	3.04	10.85	5.42
Conv. Total (cfs)	81245.7	Conv. (cfs)	14138.0	42439.4	24668.3
Length Wtd. (ft)	24.93	Wetted Per. (ft)	134.76	30.23	91.14
Min Ch El (ft)	895.67	Shear (lb/sq ft)	0.25	0.78	0.44
Alpha	2.64	Stream Power (lb/ft s)	0.31	4.19	0.81
Frctn Loss (ft)	0.02	Cum Volume (acre-ft)	54.95	40.54	109.05
C & E Loss (ft)		Cum SA (acres)	25.61	7.84	47.95

Plan: SunnyDay   PB2   Main RS: 9620   Profile: Max WS

E.G. Elev (ft)	907.91	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.10	Wt. n-Val.	0.080	0.045	0.090
W.S. Elev (ft)	907.81	Reach Len. (ft)	135.43	110.47	91.07
Crit W.S. (ft)		Flow Area (sq ft)	558.33	673.22	427.69
E.G. Slope (ft/ft)	0.000375	Area (sq ft)	558.33	673.22	427.69
Q Total (cfs)	2946.59	Flow (cfs)	462.78	2035.70	448.11
Top Width (ft)	323.41	Top Width (ft)	188.03	64.25	71.13
Vel Total (ft/s)	1.78	Avg. Vel. (ft/s)	0.83	3.02	1.05
Max Chl Dpth (ft)	12.19	Hydr. Depth (ft)	2.97	10.48	6.01
Conv. Total (cfs)	152089.5	Conv. (cfs)	23886.6	105073.5	23129.4
Length Wtd. (ft)	110.47	Wetted Per. (ft)	188.38	65.51	72.15
Min Ch El (ft)	895.62	Shear (lb/sq ft)	0.07	0.24	0.14
Alpha	2.09	Stream Power (lb/ft s)	0.06	0.73	0.15
Frctn Loss (ft)		Cum Volume (acre-ft)	54.65	40.03	108.28
C & E Loss (ft)		Cum SA (acres)	25.51	7.80	47.81

Plan: SunnyDay   PB2   Main RS: 9510   Profile: Max WS

E.G. Elev (ft)	905.22	Element	Left OB	Channel	Right OB
Vel Head (ft)	1.23	Wt. n-Val.	0.080	0.045	0.090
W.S. Elev (ft)	903.99	Reach Len. (ft)	18.76	21.99	22.03
Crit W.S. (ft)		Flow Area (sq ft)	10.77	327.34	5.66
E.G. Slope (ft/ft)	0.005624	Area (sq ft)	10.77	327.34	5.66
Q Total (cfs)	2944.53	Flow (cfs)	14.80	2922.77	6.95
Top Width (ft)	59.78	Top Width (ft)	10.73	43.82	5.23
Vel Total (ft/s)	8.57	Avg. Vel. (ft/s)	1.37	8.93	1.23
Max Chl Dpth (ft)	9.01	Hydr. Depth (ft)	1.00	7.47	1.08
Conv. Total (cfs)	39264.2	Conv. (cfs)	197.4	38974.1	92.7
Length Wtd. (ft)	21.97	Wetted Per. (ft)	10.98	47.81	5.72
Min Ch El (ft)	894.98	Shear (lb/sq ft)	0.34	2.40	0.35
Alpha	1.08	Stream Power (lb/ft s)	0.47	21.46	0.43
Frctn Loss (ft)	0.13	Cum Volume (acre-ft)	54.65	38.49	108.28
C & E Loss (ft)		Cum SA (acres)	25.20	7.66	47.73

Plan: SunnyDay   PB2   Main RS: 9334   Profile: Max WS

E.G. Elev (ft)	903.97	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.55	Wt. n-Val.	0.080	0.045	0.090
W.S. Elev (ft)	903.43	Reach Len. (ft)	19.98	24.48	28.58
Crit W.S. (ft)		Flow Area (sq ft)	223.37	243.82	269.54
E.G. Slope (ft/ft)	0.004218	Area (sq ft)	223.37	243.82	269.54
Q Total (cfs)	2936.02	Flow (cfs)	536.15	1794.61	605.26
Top Width (ft)	202.84	Top Width (ft)	78.71	35.73	88.41
Vel Total (ft/s)	3.99	Avg. Vel. (ft/s)	2.40	7.36	2.25
Max Chl Dpth (ft)	8.29	Hydr. Depth (ft)	2.84	6.82	3.05
Conv. Total (cfs)	45205.7	Conv. (cfs)	8255.1	27631.4	9319.2
Length Wtd. (ft)	24.44	Wetted Per. (ft)	79.58	38.35	88.94
Min Ch El (ft)	895.14	Shear (lb/sq ft)	0.74	1.67	0.80
Alpha	2.22	Stream Power (lb/ft s)	1.77	12.32	1.79
Frctn Loss (ft)	0.10	Cum Volume (acre-ft)	54.54	37.37	107.67
C & E Loss (ft)		Cum SA (acres)	25.14	7.50	47.25

Plan: SunnyDay   PB2   Main RS: 9089   Profile: Max WS

E.G. Elev (ft)	902.99	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.36	Wt. n-Val.	0.090	0.045	0.090
W.S. Elev (ft)	902.63	Reach Len. (ft)	22.10	23.81	21.88
Crit W.S. (ft)		Flow Area (sq ft)	348.68	292.29	243.76
E.G. Slope (ft/ft)	0.002718	Area (sq ft)	348.68	292.29	243.76
Q Total (cfs)	2930.76	Flow (cfs)	683.23	1748.34	499.19
Top Width (ft)	209.51	Top Width (ft)	100.86	43.52	65.13
Vel Total (ft/s)	3.31	Avg. Vel. (ft/s)	1.96	5.98	2.05
Max Chl Dpth (ft)	7.93	Hydr. Depth (ft)	3.46	6.72	3.74
Conv. Total (cfs)	56214.9	Conv. (cfs)	13105.1	33535.0	9574.9
Length Wtd. (ft)	23.09	Wetted Per. (ft)	101.52	45.13	66.42
Min Ch El (ft)	894.70	Shear (lb/sq ft)	0.58	1.10	0.62
Alpha	2.09	Stream Power (lb/ft s)	1.14	6.57	1.28
Frctn Loss (ft)	0.06	Cum Volume (acre-ft)	53.27	35.88	106.22
C & E Loss (ft)		Cum SA (acres)	24.73	7.28	46.76

Plan: SunnyDay   PB2   Main RS: 8851   Profile: Max WS

E.G. Elev (ft)	902.16	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.75	Wt. n-Val.	0.090	0.045	0.090
W.S. Elev (ft)	901.41	Reach Len. (ft)	24.13	23.68	23.08
Crit W.S. (ft)		Flow Area (sq ft)	124.53	217.66	272.32
E.G. Slope (ft/ft)	0.006399	Area (sq ft)	124.53	217.66	272.32
Q Total (cfs)	2920.37	Flow (cfs)	358.89	1843.25	718.23
Top Width (ft)	170.05	Top Width (ft)	37.39	36.78	95.88
Vel Total (ft/s)	4.75	Avg. Vel. (ft/s)	2.88	8.47	2.64
Max Chl Dpth (ft)	7.20	Hydr. Depth (ft)	3.33	5.92	2.84
Conv. Total (cfs)	36507.8	Conv. (cfs)	4486.5	23042.6	8978.6
Length Wtd. (ft)	23.60	Wetted Per. (ft)	38.63	37.92	96.49
Min Ch El (ft)	894.21	Shear (lb/sq ft)	1.29	2.29	1.13
Alpha	2.13	Stream Power (lb/ft s)	3.71	19.42	2.97
Frctn Loss (ft)	0.14	Cum Volume (acre-ft)	52.05	34.48	105.05
C & E Loss (ft)		Cum SA (acres)	24.38	7.06	46.37

Plan: SunnyDay   PB2   Main RS: 8496   Profile: Max WS

E.G. Elev (ft)	900.30	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.21	Wt. n-Val.	0.051	0.045	0.090
W.S. Elev (ft)	900.09	Reach Len. (ft)	26.74	24.61	23.09
Crit W.S. (ft)		Flow Area (sq ft)	266.69	169.20	557.85
E.G. Slope (ft/ft)	0.003051	Area (sq ft)	266.69	169.20	557.85
Q Total (cfs)	2779.19	Flow (cfs)	734.15	926.56	1118.49
Top Width (ft)	320.40	Top Width (ft)	119.23	30.65	170.53
Vel Total (ft/s)	2.80	Avg. Vel. (ft/s)	2.75	5.48	2.01
Max Chl Dpth (ft)	6.88	Hydr. Depth (ft)	2.24	5.52	3.27
Conv. Total (cfs)	50315.5	Conv. (cfs)	13291.3	16774.7	20249.5
Length Wtd. (ft)	24.57	Wetted Per. (ft)	122.73	32.52	171.12
Min Ch El (ft)	893.21	Shear (lb/sq ft)	0.41	0.99	0.62
Alpha	1.74	Stream Power (lb/ft s)	1.14	5.43	1.24
Frctn Loss (ft)	0.07	Cum Volume (acre-ft)	50.43	32.91	101.86
C & E Loss (ft)		Cum SA (acres)	23.63	6.78	45.31

Plan: SunnyDay PB2 Main RS: 8151 Profile: Max WS

E.G. Elev (ft)	899.58	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.06	Wt. n-Val.	0.050	0.045	0.089
W.S. Elev (ft)	899.52	Reach Len. (ft)	25.84	23.74	22.39
Crit W.S. (ft)		Flow Area (sq ft)	260.76	181.00	1296.60
E.G. Slope (ft/ft)	0.001020	Area (sq ft)	260.76	181.00	1296.60
Q Total (cfs)	2714.24	Flow (cfs)	300.31	614.70	1799.24
Top Width (ft)	534.93	Top Width (ft)	194.77	28.25	311.91
Vel Total (ft/s)	1.56	Avg. Vel. (ft/s)	1.15	3.40	1.39
Max Chl Dpth (ft)	7.73	Hydr. Depth (ft)	1.34	6.41	4.16
Conv. Total (cfs)	84965.9	Conv. (cfs)	9400.7	19242.5	56322.8
Length Wtd. (ft)	23.08	Wetted Per. (ft)	197.58	31.33	313.04
Min Ch El (ft)	891.79	Shear (lb/sq ft)	0.08	0.37	0.26
Alpha	1.66	Stream Power (lb/ft s)	0.10	1.25	0.37
Frctn Loss (ft)	0.02	Cum Volume (acre-ft)	47.79	31.54	95.66
C & E Loss (ft)		Cum SA (acres)	21.98	6.55	43.55

Plan: SunnyDay PB2 Main RS: 7890 Profile: Max WS

E.G. Elev (ft)	899.35	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.06	Wt. n-Val.	0.060	0.045	0.070
W.S. Elev (ft)	899.30	Reach Len. (ft)	25.20	24.19	22.85
Crit W.S. (ft)		Flow Area (sq ft)	218.69	160.91	1182.46
E.G. Slope (ft/ft)	0.000749	Area (sq ft)	218.69	160.91	1182.46
Q Total (cfs)	2707.37	Flow (cfs)	222.99	488.75	1995.62
Top Width (ft)	379.88	Top Width (ft)	118.36	24.05	237.47
Vel Total (ft/s)	1.73	Avg. Vel. (ft/s)	1.02	3.04	1.69
Max Chl Dpth (ft)	8.03	Hydr. Depth (ft)	1.85	6.69	4.98
Conv. Total (cfs)	98945.0	Conv. (cfs)	8149.6	17862.2	72933.2
Length Wtd. (ft)	23.30	Wetted Per. (ft)	118.48	26.11	238.74
Min Ch El (ft)	891.27	Shear (lb/sq ft)	0.09	0.29	0.23
Alpha	1.28	Stream Power (lb/ft s)	0.09	0.88	0.39
Frctn Loss (ft)	0.02	Cum Volume (acre-ft)	46.20	30.51	88.60
C & E Loss (ft)		Cum SA (acres)	21.00	6.39	41.99

Plan: SunnyDay PB2 Main RS: 7503 Profile: Max WS

E.G. Elev (ft)	898.42	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.89	Wt. n-Val.	0.070	0.045	0.080
W.S. Elev (ft)	897.53	Reach Len. (ft)	23.51	24.17	27.41
Crit W.S. (ft)		Flow Area (sq ft)	19.87	145.35	315.62
E.G. Slope (ft/ft)	0.010034	Area (sq ft)	19.87	145.35	315.62
Q Total (cfs)	2704.91	Flow (cfs)	60.60	1419.81	1224.49
Top Width (ft)	141.46	Top Width (ft)	11.13	26.04	104.29
Vel Total (ft/s)	5.63	Avg. Vel. (ft/s)	3.05	9.77	3.88
Max Chl Dpth (ft)	7.08	Hydr. Depth (ft)	1.79	5.58	3.03
Conv. Total (cfs)	27002.6	Conv. (cfs)	605.0	14173.8	12223.9
Length Wtd. (ft)	25.67	Wetted Per. (ft)	11.57	28.64	104.94
Min Ch El (ft)	890.45	Shear (lb/sq ft)	1.08	3.18	1.88
Alpha	1.80	Stream Power (lb/ft s)	3.28	31.06	7.31
Frctn Loss (ft)	0.25	Cum Volume (acre-ft)	45.35	29.07	82.98
C & E Loss (ft)		Cum SA (acres)	20.58	6.17	40.63

Plan: SunnyDay PB2 Main RS: 7165 Profile: Max WS

E.G. Elev (ft)	894.77	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.23	Wt. n-Val.	0.070	0.045	0.080
W.S. Elev (ft)	894.53	Reach Len. (ft)	22.95	23.72	22.92
Crit W.S. (ft)		Flow Area (sq ft)	10.24	81.35	847.45
E.G. Slope (ft/ft)	0.006981	Area (sq ft)	10.24	81.35	847.45
Q Total (cfs)	2702.12	Flow (cfs)	20.78	564.12	2117.21
Top Width (ft)	437.93	Top Width (ft)	8.06	17.89	411.99
Vel Total (ft/s)	2.88	Avg. Vel. (ft/s)	2.03	6.93	2.50
Max Chl Dpth (ft)	5.18	Hydr. Depth (ft)	1.27	4.55	2.06
Conv. Total (cfs)	32340.3	Conv. (cfs)	248.7	6751.7	25339.9
Length Wtd. (ft)	23.09	Wetted Per. (ft)	8.37	20.41	414.90
Min Ch El (ft)	889.35	Shear (lb/sq ft)	0.53	1.74	0.89
Alpha	1.81	Stream Power (lb/ft s)	1.08	12.04	2.22
Frctn Loss (ft)	0.16	Cum Volume (acre-ft)	45.24	28.20	78.65
C & E Loss (ft)		Cum SA (acres)	20.51	6.00	38.72

Plan: SunnyDay PB2 Main RS: 6761 Profile: Max WS

E.G. Elev (ft)	892.09	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.19	Wt. n-Val.	0.080	0.045	0.080
W.S. Elev (ft)	891.90	Reach Len. (ft)	24.17	24.40	23.57
Crit W.S. (ft)		Flow Area (sq ft)	95.04	108.17	768.16
E.G. Slope (ft/ft)	0.006104	Area (sq ft)	95.04	108.17	768.16
Q Total (cfs)	2694.70	Flow (cfs)	196.31	618.05	1880.34
Top Width (ft)	438.76	Top Width (ft)	55.43	32.42	350.92
Vel Total (ft/s)	2.77	Avg. Vel. (ft/s)	2.07	5.71	2.45
Max Chl Dpth (ft)	3.83	Hydr. Depth (ft)	1.71	3.34	2.19
Conv. Total (cfs)	34491.0	Conv. (cfs)	2512.7	7910.7	24067.6
Length Wtd. (ft)	23.80	Wetted Per. (ft)	55.96	32.82	352.55
Min Ch El (ft)	888.22	Shear (lb/sq ft)	0.65	1.26	0.83
Alpha	1.56	Stream Power (lb/ft s)	1.34	7.18	2.03
Frctn Loss (ft)	0.15	Cum Volume (acre-ft)	44.99	27.30	71.58
C & E Loss (ft)		Cum SA (acres)	20.29	5.77	35.32

Plan: SunnyDay PB2 Main RS: 6395 Profile: Max WS

E.G. Elev (ft)	890.08	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.10	Wt. n-Val.	0.080	0.045	0.080
W.S. Elev (ft)	889.98	Reach Len. (ft)	22.39	23.24	23.43
Crit W.S. (ft)		Flow Area (sq ft)	291.67	73.57	848.91
E.G. Slope (ft/ft)	0.004038	Area (sq ft)	291.67	73.57	848.91
Q Total (cfs)	2667.91	Flow (cfs)	619.20	341.46	1707.25
Top Width (ft)	523.30	Top Width (ft)	120.40	22.15	380.75
Vel Total (ft/s)	2.20	Avg. Vel. (ft/s)	2.12	4.64	2.01
Max Chl Dpth (ft)	3.79	Hydr. Depth (ft)	2.42	3.32	2.23
Conv. Total (cfs)	41985.0	Conv. (cfs)	9744.4	5373.5	26867.1
Length Wtd. (ft)	23.15	Wetted Per. (ft)	120.91	22.36	381.67
Min Ch El (ft)	886.19	Shear (lb/sq ft)	0.61	0.83	0.56
Alpha	1.32	Stream Power (lb/ft s)	1.29	3.85	1.13
Frctn Loss (ft)	0.09	Cum Volume (acre-ft)	43.60	26.55	65.30
C & E Loss (ft)		Cum SA (acres)	19.59	5.54	32.32

Plan: SunnyDay PB2 Main RS: 6070 Profile: Max WS

E.G. Elev (ft)	889.13	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.08	Wt. n-Val.	0.080	0.045	0.080
W.S. Elev (ft)	889.05	Reach Len. (ft)	22.45	23.48	20.20
Crit W.S. (ft)		Flow Area (sq ft)	297.34	176.76	994.39
E.G. Slope (ft/ft)	0.001849	Area (sq ft)	297.34	176.76	994.39
Q Total (cfs)	2646.62	Flow (cfs)	335.35	655.04	1656.23
Top Width (ft)	544.64	Top Width (ft)	176.91	40.81	326.92
Vel Total (ft/s)	1.80	Avg. Vel. (ft/s)	1.13	3.71	1.67
Max Chl Dpth (ft)	5.10	Hydr. Depth (ft)	1.68	4.33	3.04
Conv. Total (cfs)	61546.4	Conv. (cfs)	7798.5	15232.7	38515.2
Length Wtd. (ft)	21.36	Wetted Per. (ft)	177.20	41.92	330.22
Min Ch El (ft)	884.04	Shear (lb/sq ft)	0.19	0.49	0.35
Alpha	1.63	Stream Power (lb/ft s)	0.22	1.80	0.58
Frctn Loss (ft)	0.04	Cum Volume (acre-ft)	41.50	25.66	58.54
C & E Loss (ft)		Cum SA (acres)	18.53	5.30	29.62

Plan: SunnyDay PB2 Main RS: 5718 Profile: Max WS

E.G. Elev (ft)	887.96	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.19	Wt. n-Val.	0.060	0.060	0.060
W.S. Elev (ft)	887.78	Reach Len. (ft)	23.82	23.67	22.00
Crit W.S. (ft)		Flow Area (sq ft)	502.97	173.89	164.99
E.G. Slope (ft/ft)	0.005055	Area (sq ft)	502.97	173.89	164.99
Q Total (cfs)	2629.83	Flow (cfs)	1365.61	819.22	445.00
Top Width (ft)	434.68	Top Width (ft)	308.77	39.09	86.82
Vel Total (ft/s)	3.12	Avg. Vel. (ft/s)	2.72	4.71	2.70
Max Chl Dpth (ft)	5.86	Hydr. Depth (ft)	1.63	4.45	1.90
Conv. Total (cfs)	36987.8	Conv. (cfs)	19206.9	11522.2	6258.7
Length Wtd. (ft)	23.47	Wetted Per. (ft)	309.10	39.73	87.03
Min Ch El (ft)	881.92	Shear (lb/sq ft)	0.51	1.38	0.60
Alpha	1.23	Stream Power (lb/ft s)	1.39	6.51	1.61
Frctn Loss (ft)	0.12	Cum Volume (acre-ft)	38.05	24.22	55.32
C & E Loss (ft)		Cum SA (acres)	16.08	4.98	28.22

Plan: SunnyDay PB2 Main RS: 5339 Profile: Max WS

E.G. Elev (ft)	886.46	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.24	Wt. n-Val.	0.100	0.045	0.100
W.S. Elev (ft)	886.21	Reach Len. (ft)	24.42	23.18	20.67
Crit W.S. (ft)		Flow Area (sq ft)	721.06	248.01	136.60
E.G. Slope (ft/ft)	0.003988	Area (sq ft)	721.06	248.01	136.60
Q Total (cfs)	2504.18	Flow (cfs)	1053.27	1313.32	137.58
Top Width (ft)	551.66	Top Width (ft)	368.88	60.22	122.55
Vel Total (ft/s)	2.26	Avg. Vel. (ft/s)	1.46	5.30	1.01
Max Chl Dpth (ft)	6.27	Hydr. Depth (ft)	1.95	4.12	1.11
Conv. Total (cfs)	39652.2	Conv. (cfs)	16678.0	20795.7	2178.5
Length Wtd. (ft)	23.58	Wetted Per. (ft)	371.28	61.29	122.86
Min Ch El (ft)	879.94	Shear (lb/sq ft)	0.48	1.01	0.28
Alpha	3.05	Stream Power (lb/ft s)	0.71	5.34	0.28
Frctn Loss (ft)	0.09	Cum Volume (acre-ft)	32.93	22.41	54.15
C & E Loss (ft)		Cum SA (acres)	12.93	4.55	27.39

Plan: SunnyDay   PB2   Main RS: 5061   Profile: Max WS

E.G. Elev (ft)	885.67	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.07	Wt. n-Val.	0.100	0.045	0.100
W.S. Elev (ft)	885.60	Reach Len. (ft)	18.07	24.47	22.16
Crit W.S. (ft)		Flow Area (sq ft)	1212.64	229.91	313.33
E.G. Slope (ft/ft)	0.001456	Area (sq ft)	1212.64	229.91	313.33
Q Total (cfs)	2300.39	Flow (cfs)	1265.07	760.29	275.03
Top Width (ft)	699.79	Top Width (ft)	485.62	52.15	162.02
Vel Total (ft/s)	1.31	Avg. Vel. (ft/s)	1.04	3.31	0.88
Max Chl Dpth (ft)	5.90	Hydr. Depth (ft)	2.50	4.41	1.93
Conv. Total (cfs)	60276.6	Conv. (cfs)	33148.3	19921.8	7206.5
Length Wtd. (ft)	20.52	Wetted Per. (ft)	485.99	54.09	162.71
Min Ch El (ft)	879.70	Shear (lb/sq ft)	0.23	0.39	0.18
Alpha	2.51	Stream Power (lb/ft s)	0.24	1.28	0.15
Frctn Loss (ft)	0.03	Cum Volume (acre-ft)	26.70	20.91	52.95
C & E Loss (ft)		Cum SA (acres)	9.79	4.19	26.59

Plan: SunnyDay   PB2   Main RS: 4645   Profile: Max WS

E.G. Elev (ft)	885.25	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.05	Wt. n-Val.	0.072	0.045	0.100
W.S. Elev (ft)	885.20	Reach Len. (ft)	24.26	24.97	25.07
Crit W.S. (ft)		Flow Area (sq ft)	870.89	221.23	659.67
E.G. Slope (ft/ft)	0.000811	Area (sq ft)	870.89	221.23	659.67
Q Total (cfs)	2208.85	Flow (cfs)	1129.54	640.08	439.22
Top Width (ft)	652.60	Top Width (ft)	273.49	40.56	338.55
Vel Total (ft/s)	1.26	Avg. Vel. (ft/s)	1.30	2.89	0.67
Max Chl Dpth (ft)	6.58	Hydr. Depth (ft)	3.18	5.45	1.95
Conv. Total (cfs)	77549.3	Conv. (cfs)	39656.5	22472.4	15420.3
Length Wtd. (ft)	24.60	Wetted Per. (ft)	274.29	41.00	338.98
Min Ch El (ft)	878.62	Shear (lb/sq ft)	0.16	0.27	0.10
Alpha	2.12	Stream Power (lb/ft s)	0.21	0.79	0.07
Frctn Loss (ft)	0.02	Cum Volume (acre-ft)	19.20	18.80	48.79
C & E Loss (ft)		Cum SA (acres)	7.13	3.75	24.51

Plan: SunnyDay   PB2   Main RS: 4295   Profile: Max WS

E.G. Elev (ft)	884.77	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.30	Wt. n-Val.	0.090	0.045	0.090
W.S. Elev (ft)	884.47	Reach Len. (ft)	23.50	24.54	23.16
Crit W.S. (ft)		Flow Area (sq ft)	408.76	211.78	152.09
E.G. Slope (ft/ft)	0.002392	Area (sq ft)	408.76	211.78	152.09
Q Total (cfs)	2142.35	Flow (cfs)	706.18	1194.04	242.13
Top Width (ft)	217.22	Top Width (ft)	130.57	32.14	54.51
Vel Total (ft/s)	2.77	Avg. Vel. (ft/s)	1.73	5.64	1.59
Max Chl Dpth (ft)	7.28	Hydr. Depth (ft)	3.13	6.59	2.79
Conv. Total (cfs)	43801.4	Conv. (cfs)	14438.2	24412.7	4950.5
Length Wtd. (ft)	24.04	Wetted Per. (ft)	132.27	32.47	54.95
Min Ch El (ft)	877.19	Shear (lb/sq ft)	0.46	0.97	0.41
Alpha	2.47	Stream Power (lb/ft s)	0.80	5.49	0.66
Frctn Loss (ft)	0.06	Cum Volume (acre-ft)	14.09	17.03	46.16
C & E Loss (ft)		Cum SA (acres)	5.55	3.45	23.35

Plan: SunnyDay PB2 Main RS: 3805 Profile: Max WS

E.G. Elev (ft)	883.98	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.07	Wt. n-Val.	0.100	0.045	0.100
W.S. Elev (ft)	883.91	Reach Len. (ft)	24.29	24.03	21.93
Crit W.S. (ft)		Flow Area (sq ft)	688.94	364.52	466.53
E.G. Slope (ft/ft)	0.000746	Area (sq ft)	688.94	364.52	466.53
Q Total (cfs)	2110.30	Flow (cfs)	674.78	1024.91	410.61
Top Width (ft)	389.95	Top Width (ft)	183.34	63.46	143.15
Vel Total (ft/s)	1.39	Avg. Vel. (ft/s)	0.98	2.81	0.88
Max Chl Dpth (ft)	7.62	Hydr. Depth (ft)	3.76	5.74	3.26
Conv. Total (cfs)	77258.2	Conv. (cfs)	24703.6	37522.0	15032.6
Length Wtd. (ft)	23.69	Wetted Per. (ft)	183.79	66.23	146.09
Min Ch El (ft)	876.29	Shear (lb/sq ft)	0.17	0.26	0.15
Alpha	2.23	Stream Power (lb/ft s)	0.17	0.72	0.13
Frctn Loss (ft)	0.02	Cum Volume (acre-ft)	8.45	13.78	43.52
C & E Loss (ft)		Cum SA (acres)	3.85	2.91	22.34

Plan: SunnyDay PB2 Main RS: 3396 Profile: Max WS

E.G. Elev (ft)	883.39	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.29	Wt. n-Val.	0.100	0.045	0.100
W.S. Elev (ft)	883.10	Reach Len. (ft)	23.84	23.99	24.32
Crit W.S. (ft)		Flow Area (sq ft)	68.12	142.07	554.03
E.G. Slope (ft/ft)	0.003348	Area (sq ft)	68.12	142.07	554.03
Q Total (cfs)	2100.37	Flow (cfs)	88.12	881.80	1130.44
Top Width (ft)	205.21	Top Width (ft)	36.36	20.60	148.25
Vel Total (ft/s)	2.75	Avg. Vel. (ft/s)	1.29	6.21	2.04
Max Chl Dpth (ft)	7.21	Hydr. Depth (ft)	1.87	6.90	3.74
Conv. Total (cfs)	36299.8	Conv. (cfs)	1523.0	15239.8	19537.0
Length Wtd. (ft)	24.16	Wetted Per. (ft)	36.90	24.26	151.55
Min Ch El (ft)	875.91	Shear (lb/sq ft)	0.39	1.22	0.76
Alpha	2.45	Stream Power (lb/ft s)	0.50	7.60	1.56
Frctn Loss (ft)	0.08	Cum Volume (acre-ft)	5.50	11.40	39.21
C & E Loss (ft)		Cum SA (acres)	2.89	2.52	21.09

Plan: SunnyDay PB2 Main RS: 3108 Profile: Max WS

E.G. Elev (ft)	882.51	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.25	Wt. n-Val.	0.090	0.045	0.090
W.S. Elev (ft)	882.26	Reach Len. (ft)	24.50	24.53	24.39
Crit W.S. (ft)		Flow Area (sq ft)	73.55	265.92	375.70
E.G. Slope (ft/ft)	0.002460	Area (sq ft)	73.55	265.92	375.70
Q Total (cfs)	2095.18	Flow (cfs)	87.90	1301.17	706.11
Top Width (ft)	198.12	Top Width (ft)	41.53	49.94	106.66
Vel Total (ft/s)	2.93	Avg. Vel. (ft/s)	1.20	4.89	1.88
Max Chl Dpth (ft)	6.76	Hydr. Depth (ft)	1.77	5.32	3.52
Conv. Total (cfs)	42246.3	Conv. (cfs)	1772.4	26236.2	14237.7
Length Wtd. (ft)	24.48	Wetted Per. (ft)	41.71	51.49	108.04
Min Ch El (ft)	875.64	Shear (lb/sq ft)	0.27	0.79	0.53
Alpha	1.88	Stream Power (lb/ft s)	0.32	3.88	1.00
Frctn Loss (ft)	0.06	Cum Volume (acre-ft)	5.04	10.02	36.36
C & E Loss (ft)		Cum SA (acres)	2.64	2.29	20.28

Plan: SunnyDay PB2 Main RS: 2691 Profile: Max WS

E.G. Elev (ft)	880.95	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.28	Wt. n-Val.	0.090	0.045	0.090
W.S. Elev (ft)	880.67	Reach Len. (ft)	23.53	24.30	23.62
Crit W.S. (ft)		Flow Area (sq ft)	18.54	107.07	513.65
E.G. Slope (ft/ft)	0.005649	Area (sq ft)	18.54	107.07	513.65
Q Total (cfs)	2091.90	Flow (cfs)	27.35	677.55	1387.00
Top Width (ft)	197.37	Top Width (ft)	14.06	24.37	158.94
Vel Total (ft/s)	3.27	Avg. Vel. (ft/s)	1.48	6.33	2.70
Max Chl Dpth (ft)	5.71	Hydr. Depth (ft)	1.32	4.39	3.23
Conv. Total (cfs)	27833.6	Conv. (cfs)	363.9	9015.1	18454.6
Length Wtd. (ft)	23.83	Wetted Per. (ft)	14.30	26.29	160.01
Min Ch El (ft)	875.25	Shear (lb/sq ft)	0.46	1.44	1.13
Alpha	1.67	Stream Power (lb/ft s)	0.67	9.09	3.06
Frctn Loss (ft)	0.13	Cum Volume (acre-ft)	4.65	8.29	32.04
C & E Loss (ft)		Cum SA (acres)	2.34	1.93	19.02

Plan: SunnyDay PB2 Main RS: 2254 Profile: Max WS

E.G. Elev (ft)	878.80	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.31	Wt. n-Val.	0.080	0.045	0.080
W.S. Elev (ft)	878.49	Reach Len. (ft)	22.03	24.84	23.76
Crit W.S. (ft)		Flow Area (sq ft)	1.52	114.87	644.24
E.G. Slope (ft/ft)	0.006553	Area (sq ft)	1.52	114.87	644.24
Q Total (cfs)	2088.72	Flow (cfs)	1.65	786.68	1300.38
Top Width (ft)	440.50	Top Width (ft)	1.61	25.27	413.62
Vel Total (ft/s)	2.75	Avg. Vel. (ft/s)	1.08	6.85	2.02
Max Chl Dpth (ft)	4.84	Hydr. Depth (ft)	0.95	4.55	1.56
Conv. Total (cfs)	25801.6	Conv. (cfs)	20.4	9717.8	16063.4
Length Wtd. (ft)	24.17	Wetted Per. (ft)	2.49	28.01	416.33
Min Ch El (ft)	873.65	Shear (lb/sq ft)	0.25	1.68	0.63
Alpha	2.68	Stream Power (lb/ft s)	0.27	11.49	1.28
Frctn Loss (ft)	0.16	Cum Volume (acre-ft)	4.59	7.21	25.83
C & E Loss (ft)		Cum SA (acres)	2.29	1.68	16.25

Plan: SunnyDay PB2 Main RS: 1881 Profile: Max WS

E.G. Elev (ft)	876.78	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.17	Wt. n-Val.	0.090	0.045	0.080
W.S. Elev (ft)	876.61	Reach Len. (ft)	23.77	24.26	24.31
Crit W.S. (ft)		Flow Area (sq ft)	57.73	189.42	711.14
E.G. Slope (ft/ft)	0.003759	Area (sq ft)	57.73	189.42	711.14
Q Total (cfs)	2078.98	Flow (cfs)	47.67	897.55	1133.76
Top Width (ft)	559.56	Top Width (ft)	78.16	51.60	429.80
Vel Total (ft/s)	2.17	Avg. Vel. (ft/s)	0.83	4.74	1.59
Max Chl Dpth (ft)	4.97	Hydr. Depth (ft)	0.74	3.67	1.65
Conv. Total (cfs)	33908.1	Conv. (cfs)	777.4	14639.1	18491.6
Length Wtd. (ft)	24.28	Wetted Per. (ft)	78.36	52.90	432.09
Min Ch El (ft)	871.64	Shear (lb/sq ft)	0.17	0.84	0.39
Alpha	2.36	Stream Power (lb/ft s)	0.14	3.98	0.62
Frctn Loss (ft)	0.08	Cum Volume (acre-ft)	4.55	5.91	20.82
C & E Loss (ft)		Cum SA (acres)	2.20	1.35	12.86

Plan: SunnyDay PB2 Main RS: 1396 Profile: Max WS

E.G. Elev (ft)	874.94	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.11	Wt. n-Val.	0.100	0.045	0.080
W.S. Elev (ft)	874.82	Reach Len. (ft)	24.57	24.41	23.54
Crit W.S. (ft)		Flow Area (sq ft)	12.24	164.60	832.16
E.G. Slope (ft/ft)	0.002685	Area (sq ft)	12.24	164.60	832.16
Q Total (cfs)	1985.74	Flow (cfs)	11.54	677.62	1296.58
Top Width (ft)	455.23	Top Width (ft)	8.58	43.50	403.16
Vel Total (ft/s)	1.97	Avg. Vel. (ft/s)	0.94	4.12	1.56
Max Chl Dpth (ft)	4.63	Hydr. Depth (ft)	1.43	3.78	2.06
Conv. Total (cfs)	38324.4	Conv. (cfs)	222.7	13078.0	25023.7
Length Wtd. (ft)	23.85	Wetted Per. (ft)	9.04	44.10	403.97
Min Ch El (ft)	870.19	Shear (lb/sq ft)	0.23	0.63	0.35
Alpha	1.90	Stream Power (lb/ft s)	0.21	2.58	0.54
Frctn Loss (ft)	0.06	Cum Volume (acre-ft)	4.40	3.96	13.22
C & E Loss (ft)		Cum SA (acres)	1.94	0.82	8.32

Plan: SunnyDay PB2 Main RS: 1006 Profile: Max WS

E.G. Elev (ft)	874.18	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.05	Wt. n-Val.	0.100	0.045	0.100
W.S. Elev (ft)	874.13	Reach Len. (ft)	22.45	24.82	25.54
Crit W.S. (ft)		Flow Area (sq ft)	626.79	317.23	1050.05
E.G. Slope (ft/ft)	0.000707	Area (sq ft)	626.79	317.23	1050.05
Q Total (cfs)	1919.37	Flow (cfs)	429.00	836.81	653.55
Top Width (ft)	931.28	Top Width (ft)	276.13	59.75	595.40
Vel Total (ft/s)	0.96	Avg. Vel. (ft/s)	0.68	2.64	0.62
Max Chl Dpth (ft)	6.65	Hydr. Depth (ft)	2.27	5.31	1.76
Conv. Total (cfs)	72189.1	Conv. (cfs)	16135.1	31473.3	24580.7
Length Wtd. (ft)	24.52	Wetted Per. (ft)	277.69	60.91	597.56
Min Ch El (ft)	867.48	Shear (lb/sq ft)	0.10	0.23	0.08
Alpha	3.53	Stream Power (lb/ft s)	0.07	0.61	0.05
Frctn Loss (ft)	0.02	Cum Volume (acre-ft)	3.30	1.89	5.94
C & E Loss (ft)		Cum SA (acres)	1.22	0.36	4.62

Plan: SunnyDay PB2 Main RS: 658 Profile: Max WS

E.G. Elev (ft)	873.75	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.27	Wt. n-Val.	0.100	0.045	0.100
W.S. Elev (ft)	873.48	Reach Len. (ft)			
Crit W.S. (ft)	872.27	Flow Area (sq ft)	323.12	149.82	347.20
E.G. Slope (ft/ft)	0.004088	Area (sq ft)	323.12	149.82	347.20
Q Total (cfs)	1901.12	Flow (cfs)	659.48	875.75	365.88
Top Width (ft)	515.96	Top Width (ft)	102.29	30.89	382.79
Vel Total (ft/s)	2.32	Avg. Vel. (ft/s)	2.04	5.85	1.05
Max Chl Dpth (ft)	6.34	Hydr. Depth (ft)	3.16	4.85	0.91
Conv. Total (cfs)	29732.3	Conv. (cfs)	10313.9	13696.2	5722.2
Length Wtd. (ft)		Wetted Per. (ft)	102.62	32.52	384.07
Min Ch El (ft)	867.14	Shear (lb/sq ft)	0.80	1.18	0.23
Alpha	3.24	Stream Power (lb/ft s)	1.64	6.87	0.24
Frctn Loss (ft)		Cum Volume (acre-ft)			
C & E Loss (ft)		Cum SA (acres)			

List of Parcels with Potentially Impacted Houses

PARCEL ID	ADDRESS NUMBER	FULL STREET	CITY	STATE	ZIP	ZIP EXTENSION
18 250 05 008	3572	HERSHEY LANE	TUCKER	GA	30084	2305
18 263 16 064	2811	GREENROCK TRAIL	DORAVILLE	GA	30340	5013
18 251 01 037	2745	HENDERSON COURT	TUCKER	GA	30084	2339
18 264 02 024	2811	TOWNLEY CIRCLE	DORAVILLE	GA	30340	4825
18 251 06 003	2950	HENDERSON ROAD	TUCKER	GA	30084	2341
18 250 04 003	2772	CARAWAY DRIVE	TUCKER	GA	30084	2335
18 250 04 005	2790	CARAWAY DRIVE	TUCKER	GA	30084	2335
18 251 01 039	2963	HENDERSON ROAD	TUCKER	GA	30084	2342
18 263 16 066	2827	GREENROCK TRAIL	DORAVILLE	GA	30340	5013
18 263 16 101	3852	ALLSBOROUGH DRIVE	TUCKER	GA	30084	2405
18 251 01 019	2798	EVANS DALE CIRCLE	DORAVILLE	GA	30340	4808
18 263 16 067	2831	GREENROCK TRAIL	DORAVILLE	GA	30340	5013
18 250 05 006	3584	HERSHEY LANE	TUCKER	GA	30084	2305
18 264 02 008	2810	CARAWAY DRIVE	TUCKER	GA	30084	2337
18 250 05 007	3578	HERSHEY LANE	TUCKER	GA	30084	2305
18 264 02 007	2804	CARAWAY DRIVE	TUCKER	GA	30084	2337
18 264 02 006	2798	CARAWAY DRIVE	TUCKER	GA	30084	2335
18 264 02 025	2805	TOWNLEY CIRCLE	DORAVILLE	GA	30340	4825
18 251 04 020	3837	ALLSBOROUGH DRIVE	TUCKER	GA	30084	2406
18 250 04 004	2782	CARAWAY DRIVE	TUCKER	GA	30084	2335
18 263 16 065	2819	GREENROCK TRAIL	DORAVILLE	GA	30340	5013
18 251 01 020	2788	EVANS DALE CIRCLE	DORAVILLE	GA	30340	4808
18 263 16 099	3836	ALLSBOROUGH DRIVE	TUCKER	GA	30084	2405
18 251 01 036	2753	HENDERSON COURT	TUCKER	GA	30084	2339
18 264 02 009	2818	CARAWAY DRIVE	TUCKER	GA	30084	2337
18 265 03 050	3340	LANSBURY VILLAGE DRIVE	CHAMBLEE	GA	30341	5752
18 264 02 010	2836	CARAWAY DRIVE	TUCKER	GA	30084	2337
18 264 02 026	2797	TOWNLEY CIRCLE	DORAVILLE	GA	30340	4800
18 250 04 002	2760	CARAWAY DRIVE	TUCKER	GA	30084	2335

Appendix F – Freeboard Calculations

AECOM

DESIGN: ST
CHECK: WCH
DATE: 26-Apr-2021
PROJECT: Erin Lake Dam
DESCRIPTION: Freeboard Computations

Freeboard Calculations (per *A Guide for Design and Layout of Vegetated Wave Protection for Earthen Embankments and Shorelines, Technical Release 56 (NRCS, 2014)*)

Step 1: Determine Effective Fetch Length (maximum of three trial locations).

Trial Location 1

Angle to Normal (degrees)	Cosine of Angle	Length (feet)	Length* Cosine (feet)
0	1.000	878	878
6	0.995	878	874
6	0.995	854	850
12	0.978	505	494
12	0.978	610	597
18	0.951	472	449
18	0.951	547	520
24	0.914	329	301
24	0.914	458	419
30	0.866	259	224
30	0.866	376	326
36	0.809	223	180
36	0.809	319	258
42	0.743	204	152
42	0.743	307	228
	13.512		6750
Effective Fetch Length (feet)			500

Step 2: Estimate 50-year recurrence overland wind velocity from NRCS TR-56 (Fig. 4).

V_L (mph) =	85

Figure 4 Maximum wind velocity over land (mph), 50-year recurrence

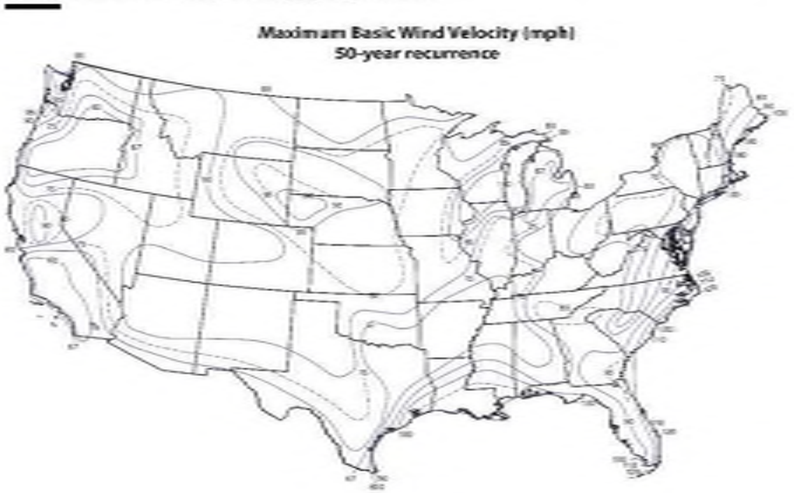
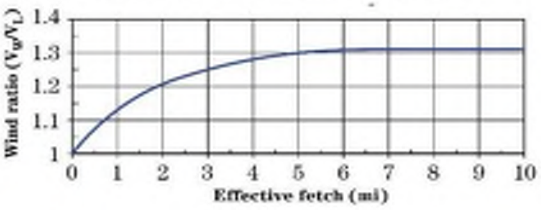
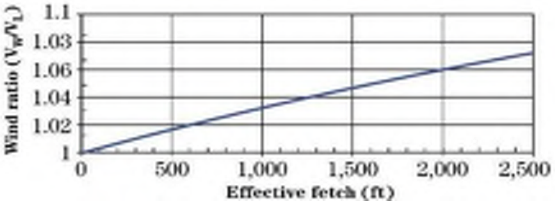


Figure 5 Wind ratio

(a) Wind relationship overwater to overland (miles)



(b) Wind relationship overwater to overland (feet)



Step 3: Determine the over water wind velocity from NRCS TR-56 (Fig. 5).

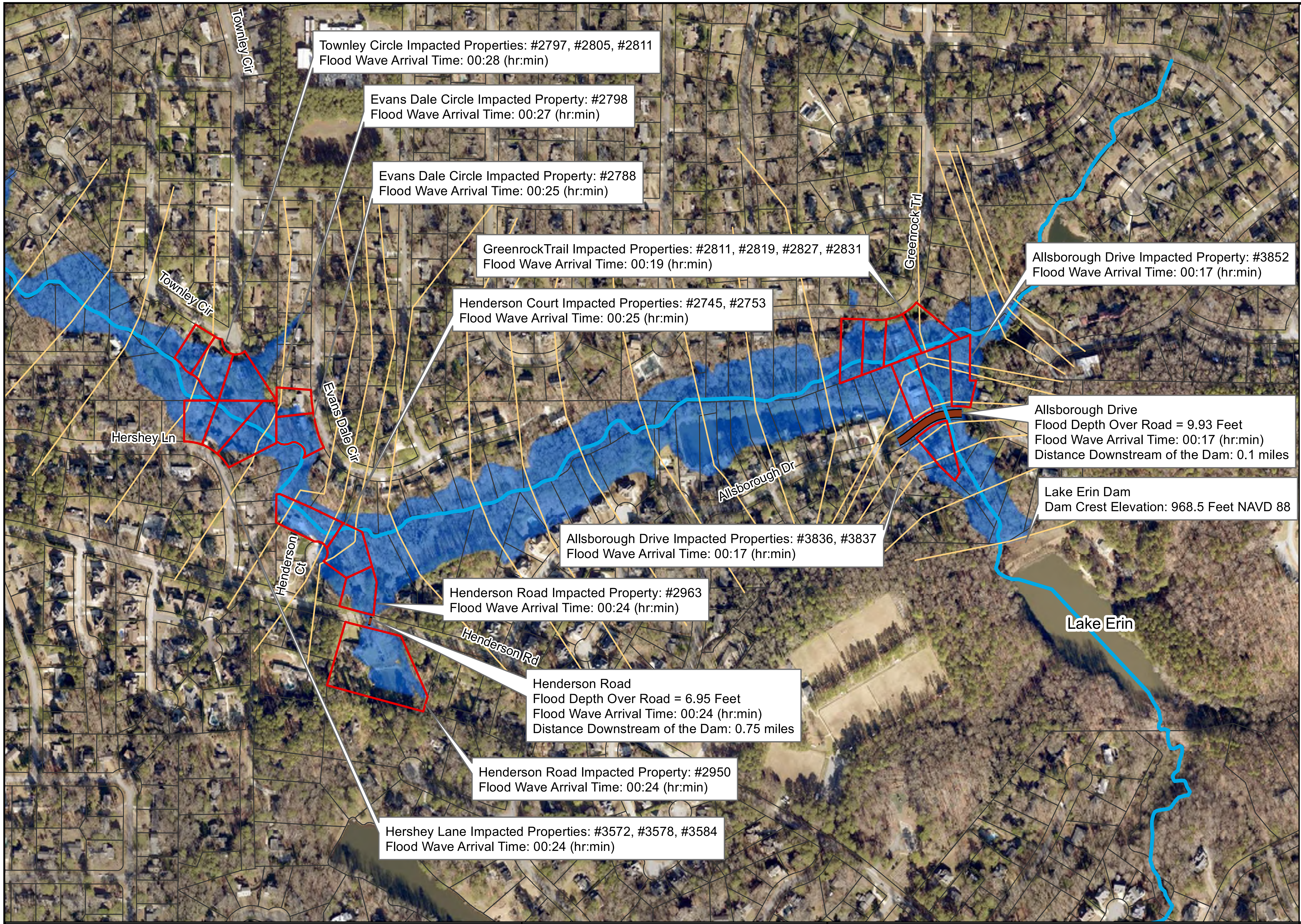
V_L (mph) =	85
V_W/V_L =	1.019
V_W (mph) =	86.6

Step 4: Determine the significant wave height (freeboard).

$$H_S = 0.0232V_W^{1.06}F_e^{0.47}$$

V_W (mph) =	86.6
F_e (miles) =	0.09
H_S (feet) =	0.8

Appendix G – Inundation Maps



Townley Circle Impacted Properties: #2797, #2805, #2811
Flood Wave Arrival Time: 00:28 (hr:min)

Evans Dale Circle Impacted Property: #2798
Flood Wave Arrival Time: 00:27 (hr:min)

Evans Dale Circle Impacted Property: #2788
Flood Wave Arrival Time: 00:25 (hr:min)

GreenrockTrail Impacted Properties: #2811, #2819, #2827, #2831
Flood Wave Arrival Time: 00:19 (hr:min)

Henderson Court Impacted Properties: #2745, #2753
Flood Wave Arrival Time: 00:25 (hr:min)

Allsborough Drive Impacted Property: #3852
Flood Wave Arrival Time: 00:17 (hr:min)

Allsborough Drive
Flood Depth Over Road = 9.93 Feet
Flood Wave Arrival Time: 00:17 (hr:min)
Distance Downstream of the Dam: 0.1 miles

Lake Erin Dam
Dam Crest Elevation: 968.5 Feet NAVD 88

Allsborough Drive Impacted Properties: #3836, #3837
Flood Wave Arrival Time: 00:17 (hr:min)

Henderson Road Impacted Property: #2963
Flood Wave Arrival Time: 00:24 (hr:min)

Henderson Road
Flood Depth Over Road = 6.95 Feet
Flood Wave Arrival Time: 00:24 (hr:min)
Distance Downstream of the Dam: 0.75 miles

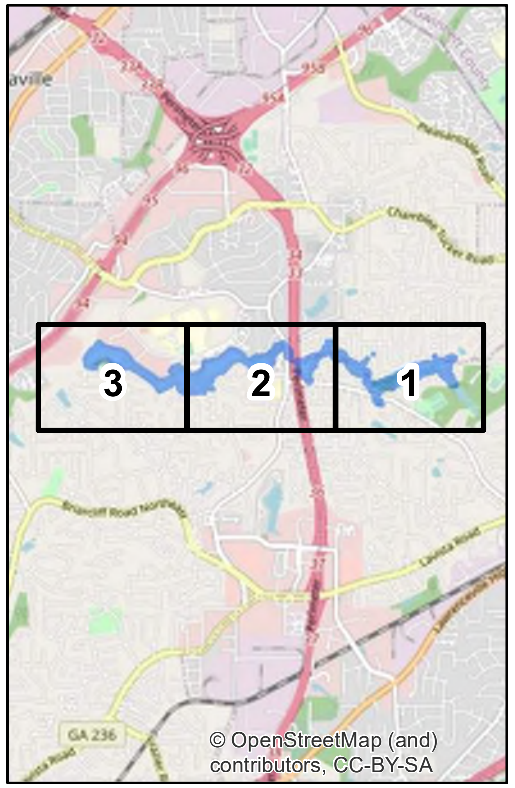
Henderson Road Impacted Property: #2950
Flood Wave Arrival Time: 00:24 (hr:min)

Hershey Lane Impacted Properties: #3572, #3578, #3584
Flood Wave Arrival Time: 00:24 (hr:min)

Legend

—	Stream Centerline
—	Cross Sections
▬	Road within Inundation Limits
▢	Parcels with Impacted Houses
▢	Parcels
🟦	Dam Breach Inundation Limits

Index Map



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April 2021
Sheet 1 of 3

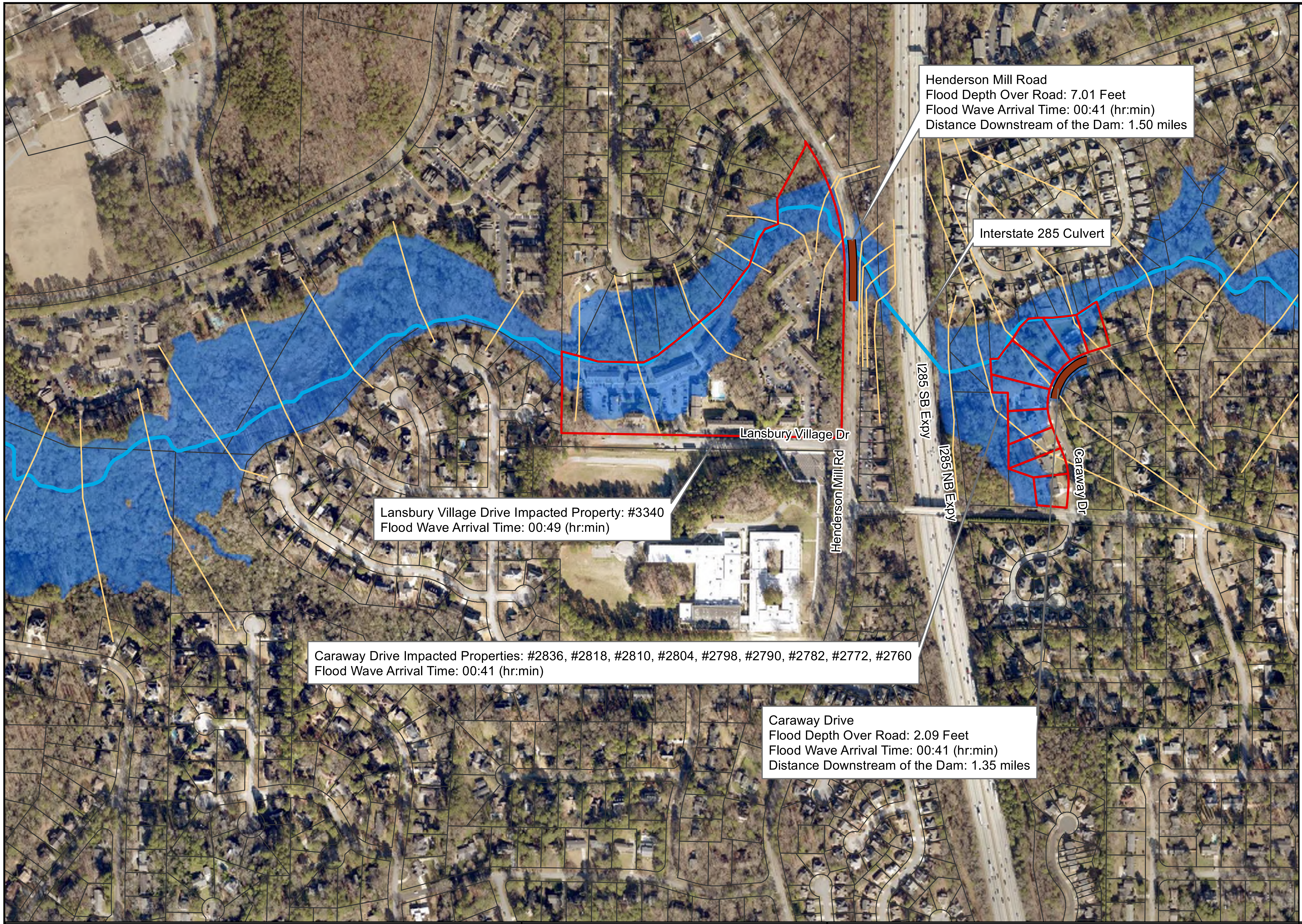
0 200 400 800 1,200 1,600
Feet

N
W E
S

Erin Lake Dam
Dam Breach Inundation Map

NOTES: The information contained on this map is prepared for use in the notification of downstream property owners by emergency management personnel. Timing and extent of actual inundation may differ from the inundation presented on this map. Times in this map are calculated from the dam breach initiation time.

https://aecom.sharepoint.com/sites/ErinLake-DeKalbCounty/Shared%20Documents/General/400-Technical/H&H/Inundation%20Maps



Henderson Mill Road
Flood Depth Over Road: 7.01 Feet
Flood Wave Arrival Time: 00:41 (hr:min)
Distance Downstream of the Dam: 1.50 miles

Interstate 285 Culvert

Lansbury Village Drive Impacted Property: #3340
Flood Wave Arrival Time: 00:49 (hr:min)

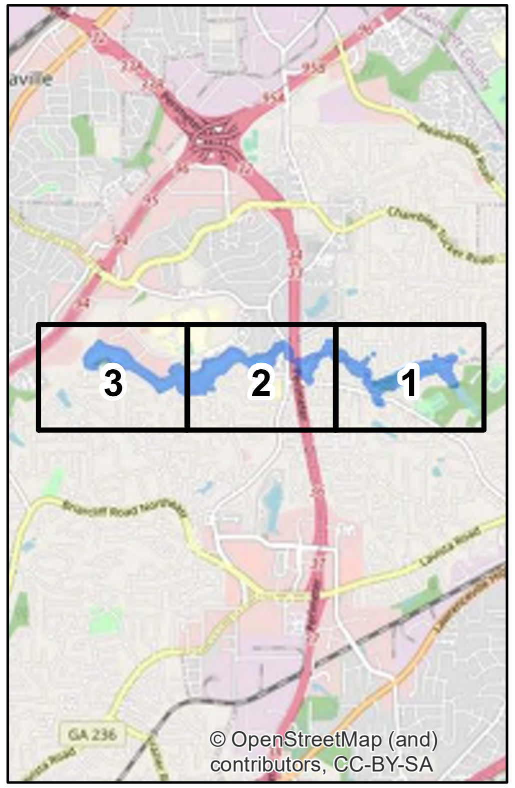
Caraway Drive Impacted Properties: #2836, #2818, #2810, #2804, #2798, #2790, #2782, #2772, #2760
Flood Wave Arrival Time: 00:41 (hr:min)

Caraway Drive
Flood Depth Over Road: 2.09 Feet
Flood Wave Arrival Time: 00:41 (hr:min)
Distance Downstream of the Dam: 1.35 miles

Legend

▬	Stream Centerline
▬	Cross Sections
▬	Road within Inundation Limits
▢	Parcels with Impacted Houses
▢	Parcels
■	Dam Breach Inundation Limits

Index Map



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April 2021
Sheet 2 of 3

0 200 400 800 1,200 1,600
Feet

Erin Lake Dam
Dam Breach Inundation Map

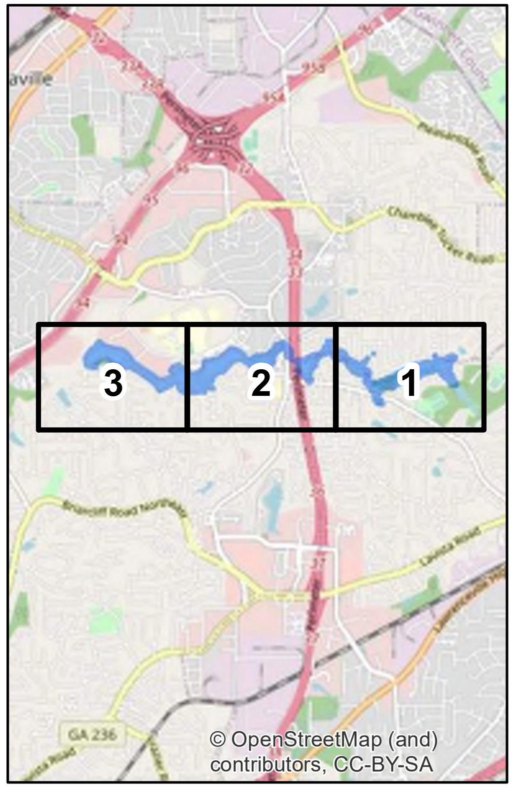
NOTES: The information contained on this map is prepared for use in the notification of downstream property owners by emergency management personnel. Timing and extent of actual inundation may differ from the inundation presented on this map. Times in this map are calculated from the dam breach initiation time.

https://aecom.sharepoint.com/sites/ErinLake-DeKalbCounty/Shared%20Documents/General/400-Technical/H&H/Inundation%20Maps

Legend

- Stream Centerline
- Cross Sections
- ▭ Parcels
- 🟦 Dam Breach Inundation Limits

Index Map



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April 2021
Sheet 3 of 3

Erin Lake Dam
Dam Breach Inundation Map

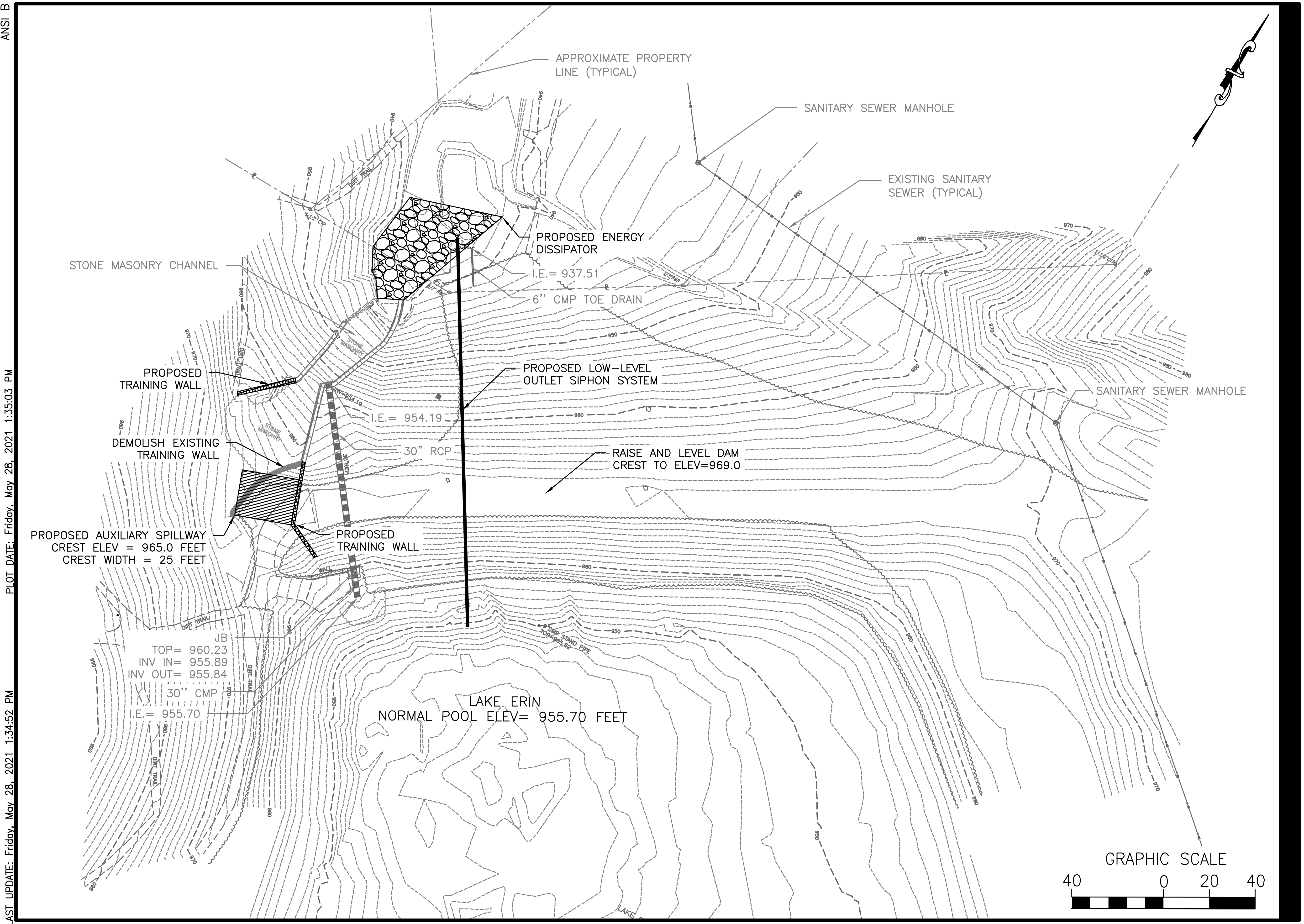
NOTES: The information contained on this map is prepared for use in the notification of downstream property owners by emergency management personnel. Timing and extent of actual inundation may differ from the inundation presented on this map. Times in this map are calculated from the dam breach initiation time.

https://aecom.sharepoint.com/sites/ErinLake-DeKalbCounty/Shared%20Documents/General/400-Technical/H&H/Inundation%20Maps

Appendix H – Proposed Alternatives Preliminary Sketches

ANSI B

PATH/FILENAME: C:\USERS\MADISON.WOELTJE\ONEDRIVE – AECOM DIRECTORY\DESKTOP\GEORGIA PROJECTS\ERIN LAKE DAM\ALTERNATIVE SKETCHS_ERIN LAKE DAM.DWG
PLOT DATE: Friday, May 28, 2021 1:35:03 PM
LAST UPDATE: Friday, May 28, 2021 1:34:52 PM



AECOM

PROJECT
ERIN LAKE DAM

OWNER/CLIENT
DEKALB COUNTY, GEORGIA
PW - ROADS & DRAINAGE C/M
729 CAMP ROAD
DECATUR, GA 30032

CONSULTANT
AECOM TECHNICAL SERVICES, INC.
1360 PEACHTREE STREET, NE
SUITE 500
ATLANTA, GA 30309
PHONE: (404) 965 9600

PROJECT NUMBER
60618914

Designed By:	
Drawn By:	MKW
Dept Check:	WCH
Proj Check:	
Date:	5/28/2021
Scale:	1" = 40'

DISCIPLINE
WATER

SHEET TITLE
PROPOSED ALTERNATIVE #1
PRELIMINARY SKETCH

SHEET NUMBER
1

ANSI B

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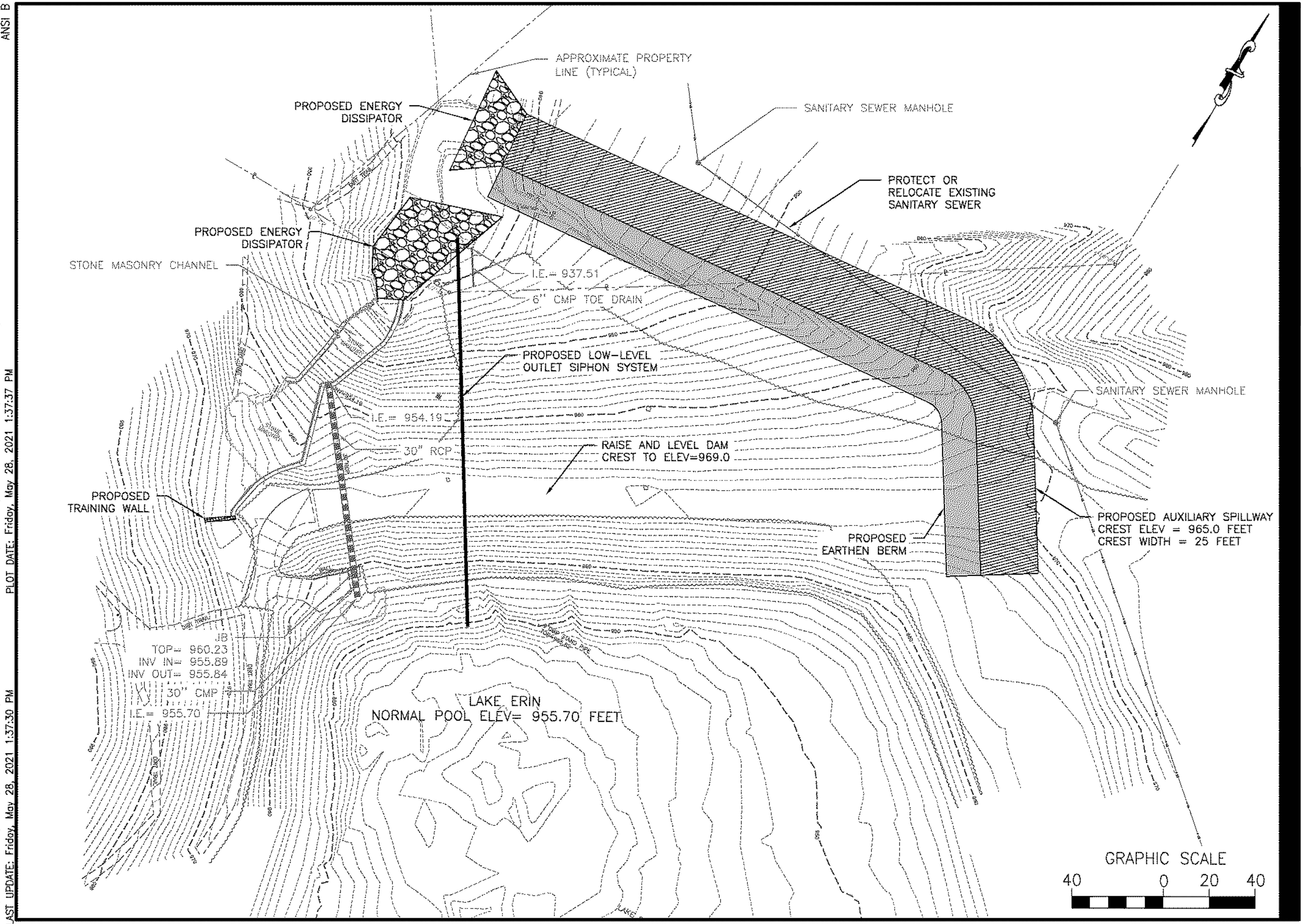

AECOM

PROJECT
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DEKALB COUNTY, GEORGIA
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Date:	5/28/2021
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DISCIPLINE
WATER

SHEET TITLE
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PRELIMINARY SKETCH

SHEET NUMBER
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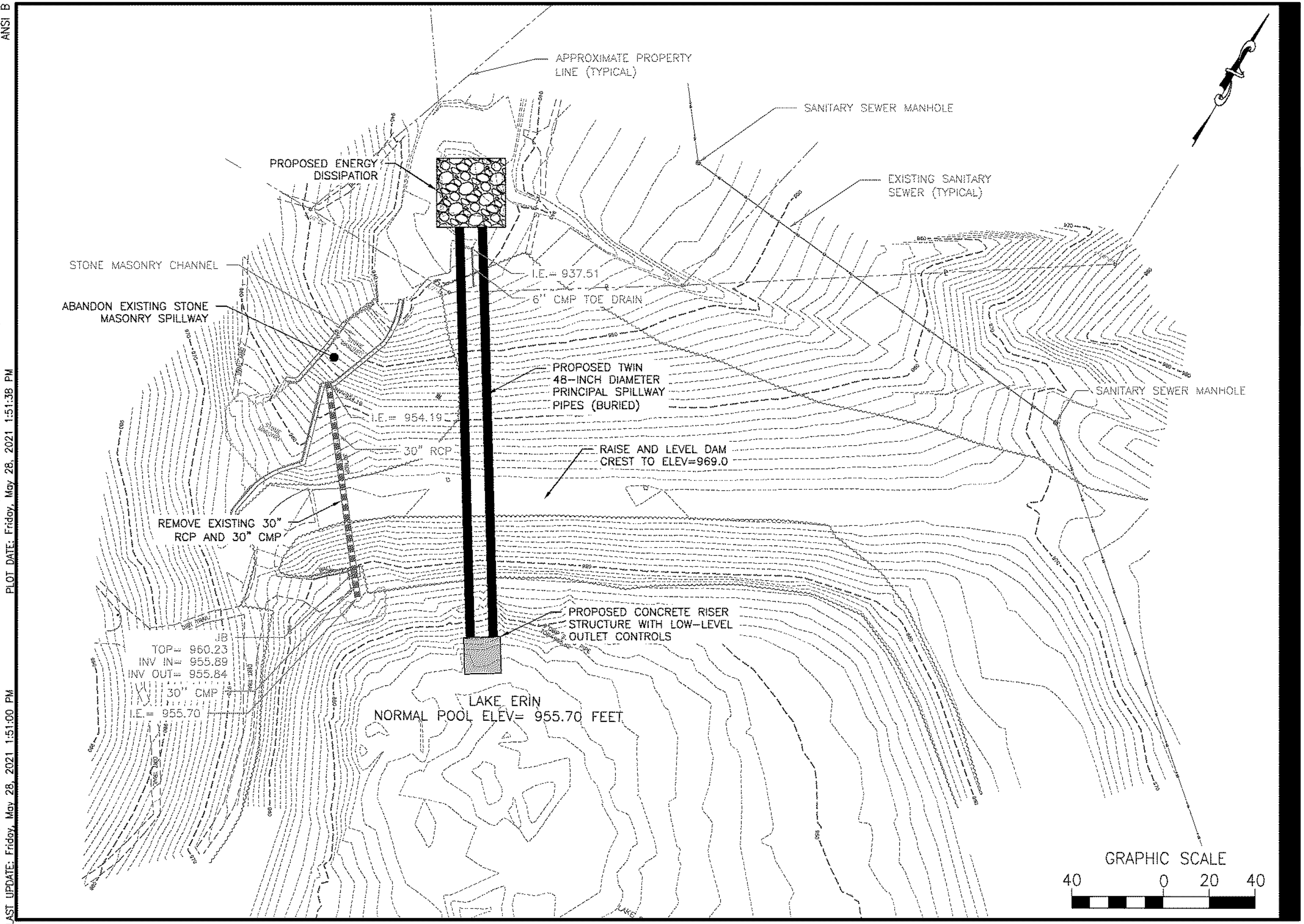

AECOM

PROJECT

ERIN LAKE DAM

OWNER/CLIENT

DEKALB COUNTY, GEORGIA
PW - ROADS & DRAINAGE C/M
729 CAMP ROAD
DECATUR, GA 30032

CONSULTANT

AECOM TECHNICAL SERVICES, INC.
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ATLANTA, GA 30309
PHONE: (404) 965 9600

PROJECT NUMBER

60618914

Designed By:	
Drawn By:	MKW
Dept Check:	WCH
Proj Check:	
Date:	5/28/2021
Scale:	1" = 40'

DISCIPLINE

WATER

SHEET TITLE

PROPOSED ALTERNATIVE #3
PRELIMINARY SKETCH

SHEET NUMBER

3

aecom.com

AECOM
1360 Peachtree Street
Atlanta, GA 30309
aecom.com

Project name:
Erin Lake Dam

Project ref:
WA36 Task #1

From:
Jeff Blass, P.E.

Date:
May 06, 2022

To:
Ms. Peggy Allen
Roads and Drainage Division
Department of Public Works
DeKalb County, Georgia
727 Camp Road
Decatur, Georgia 30032

CC:
AECOM - Malavika Tripathi, P.E., Bob Pinciotti, P.E.,
FILE

Memorandum

Subject: Spillway Alternatives Analysis

This memorandum is provided to present a comparative analysis of the three alternative modification designs presented in the *Erin Lake Dam Hydrologic and Hydraulic Analysis Report* (AECOM, 2021) to address spillway capacity at Erin Lake Dam in Tucker, Georgia. The memorandum discusses the background for this work, the scope of work, description of alternatives, rough-order-of-magnitude cost estimates for each alternative and a discussion of the significant drivers of those cost estimates.

1. Background

Erin Lake Dam is a Category I dam located in the City of Tucker, Georgia. However, the dam is understood to be operated and maintained by DeKalb County, Georgia. In 2021, AECOM Technical Services, Inc. (AECOM) prepared a hydrologic and hydraulic (H&H) analysis report which assessed the dam against pertinent criteria described in *Engineer Guidelines Version 4.0* (Georgia Department of Natural Resources, 2015) which are required for all Category I dams in the state of Georgia. The analysis report noted the following deficiencies:

- Criterion 5.3.4 – Satisfactory performance of the energy dissipation at the auxiliary spillway outlet is currently unknown.
- Criterion 5.5 – The dam is overtopped by approximately 0.7 feet during the SDF and therefore, does not provide the required freeboard over the peak SDF water surface elevation.
- Criterion 5.7 – The reservoir cannot be drained to the required elevation reflective of two-thirds of the normal pool storage volume.

AECOM developed three design alternatives to address these criteria and described them in Section 6 of the H&H report. DeKalb County subsequently requested that AECOM prepare budget-level cost estimates to support the County in its deliberations over which alternative to select for implementation. These three proposed design alternatives only address the H&H-related deficiencies described above and do not address geotechnical and or other issues associated with the dam. Similarly, the alternative costs listed in this memorandum only address the relative cost of the described alternative and do not address all of the costs that will be necessary to complete the full remediation of the dam to meet the requirements for Category I dams.

2. Scope of Work

This memorandum is prepared as a required deliverable for Task #1 of Work Authorization 36 in accordance with Agreement No. 1039255 between the DeKalb County Department of Public Works (DeKalb County) and AECOM Technical Services, Inc. (AECOM). The scope of work for this Task Order includes the following activities:

- Make minor updates to the sketches of each alternative presented in the H&H report (up to three total)
- Prepare rough-order-of-magnitude cost estimates for each alternative presented in the H&H report (up to three total)
- Prepare a memorandum describing the three alternatives and the costs thereof including discussion of the significant drivers of cost.

3. Alternatives Description

This section describes each of the proposed alternatives as originally described in the H&H report and updated with minor revisions as a result of completing this scope of work. Sketches are provided at attachments to this memorandum.

3.1 Alternative 1 – Left Abutment Auxiliary Spillway/Siphon Drawdown System

Through Alternative 1, AECOM proposes to increase spillway capacity by modifying the low area in the left abutment of the dam in the location of the masonry training walls to formalize an auxiliary spillway in the left abutment. This would require demolishing a portion of the existing right training wall and either constructing a new cast-in-place concrete training wall(s) or grading the side slopes of the auxiliary spillway. Preliminary reservoir routings show that if the auxiliary spillway crest was widened to 25-feet and lowered to elevation 965.0 feet, the resulting maximum water surface elevation during the spillway design flood (SDF) event would be 968.0 feet. To provide the minimum required freeboard (0.8 feet), the existing low point along the dam crest would also need to be raised. The majority of the dam crest is already at or above elevation 969.0 feet, so AECOM proposes to raise and level the dam crest to this elevation, thus providing the minimum required freeboard.

In additional to increasing the spillway capacity of the dam other modifications would also be required. To satisfy SDP Criterion 5.7 and provide a means for draining the reservoir, AECOM proposes installing a siphon system to be installed below grade in the embankment. To drain two-thirds of the volume at normal pool of the reservoir within ten days, the siphon would need to be able to lower the reservoir elevation from 955.7 feet to 950.4 feet. This equates to a volume of approximately 20 acre-feet over a period of ten days or an average flow rate of approximately 1.0 cfs. Riprap would also be installed at the outlet of the stone masonry channel.

3.2 Alternative 2 – Right Abutment Auxiliary Spillway/Siphon Drawdown System

In Alternative 2, AECOM proposed to increase spillway capacity by constructing an auxiliary spillway in the right abutment lined with articulated concrete block mats. This alternative would require installing a small cast-in-place concrete wall to blocking off the crest of the existing low area at the left abutment so that new auxiliary would carry the flow not handled by the principal spillway. The same preliminary reservoir routings completed for Alternative 1 apply to Alternative 2. The auxiliary spillway at the right abutment would be 25-feet wide at elevation 965.0 feet, and the resulting maximum water surface elevation during the SDF would be 968.0 feet. AECOM also proposes to raise and level the dam crest to elevation 969.0 feet for this alternative to provide the minimum required freeboard. Similar to Alternative 1, a siphon system would be installed to drain two-thirds of the volume at normal pool of the reservoir within ten days, and riprap would be required at the outlet of the stone masonry channel since the principal spillway flows would still be routed at the same location. Riprap would also be installed at the outlet of the new auxiliary spillway at the right abutment.

3.3 Alternative 3 – Principal Spillway Replacement

In lieu of constructing an auxiliary spillway in either of the dam's abutments, AECOM proposes in Alternative 3 to replace the existing principal spillway with a new principal spillway including a new riser structure and conduit. This alternative would involve abandoning the existing stone masonry structures on the left abutment of the dam as well as the existing principal spillway (which would be removed). A new concrete riser structure with a gated drain conduit and concrete pipe conduit sized to convey the SDF would be installed. The gated drain conduit would be sized and located to drain two-thirds of the volume at normal pool of the reservoir within the required ten days and would also allow the reservoir to be fully drained without the use of pumps. The riser would be accessible from the dam via a fenced and gated bridge from the dam crest to the riser top where the drain gate could be operated. Preliminary reservoir routings show that an 18-foot crest length on the principal spillway riser and 48-inch concrete pipe conduit could safely convey the SDF while providing the minimum required freeboard. AECOM also proposes to raise and level the dam crest to elevation 969.0 feet for this alternative to provide the minimum required freeboard. An impact basin and riprap apron would be constructed at the outlet of the principal spillway. This alternative would address the spillway capacity, drawdown and energy dissipation deficiencies through the construction of a new principal spillway system.

3.3.1 Alternative 3A – Principal Spillway Replacement

A less expensive modification of Alternative 3 would be to re-locate the new principal spillway near the left abutment immediately right of the existing principal spillway conduit. The relocation would allow the spillway conduit to be shorter and located higher up in the embankment thus reducing costs.

4. Rough-Order-of-Magnitude Cost Estimates

Based on the information available and the design progress of each alternative at the time of this memo, AECOM has prepared rough-order-of-magnitude cost estimates for each alternative. These estimates are representative of an approximately five percent design level and do not reflect input from any project stakeholders outside of AECOM. Project scope and associated costs are subject to change by incorporation of input from other project stakeholders, discovery of new information, and through the course of the design and construction document development progress. As such, AECOM has included contingencies in the amount of 25% of the estimated project costs (construction items and soft costs) in the cost estimates to reflect these factors.

Table 1 provides a summary of each estimate. More details can be found in the estimate documents attached to this memorandum.

Table 1. Rough-Order-of-Magnitude Cost Estimate Summary

Item	Alternative 1	Alternative 2	Alternative 3	Alternative 3A
Construction Item Costs	$715,200	$1,476,660	$1,570,890	$1,232,976
Soft Costs[1]	$286,373	$591,910	$614,903	$493,191
Contingency	$250,393	$517,143	$546,448	$431,542
Total (rounded to the nearest $10,000)	$1,250,000	$2,590,000	$2,730,000	$2,160,000

5. Cost Drivers

Within each estimate, there are several items that have a significant influence on the total cost estimate value. Table 2 below provides a summary of these line items which were selected if they exceeded five percent of the construction line item subtotal in the estimate.

Table 2. Cost Drivers

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[1] Soft costs include items such as engineering, permitting, bonds, insurance, etc. and are estimated at approximately 40 percent of the construction item costs

Item	Value	Percent of Construction Line Item Subtotal	Comment
Alternative 1			
Dewatering and Water Management	$220,000	18%	For installation of siphon system
Temporary Access	$160,000	13%	Limited access and on site laydown space for construction work
Alternative 2			
Articulated Concrete Block Mats	$648,000	25%	For lining of auxiliary spillway
Dewatering and Water Management	$220,000	9%	For installation of siphon system
Temporary Access	$160,000	6%	Limited access and on site laydown space for construction work
Alternative 3			
Dewatering	$260,000	10%	As part of coffer dam installation
Temporary Access	$200,000	8%	Limited access and on site laydown space for construction work
Coffer Dam	$185,976	7%	For principal spillway installation
Reinforced Encasement of Concrete Spillway	$157,249	6%	Around principal spillway conduit
Alternative 3A			
Dewatering	$260,000	12%	As part of coffer dam installation
Temporary Access	$200,000	9%	Limited access and on site laydown space for construction work
Reinforced Encasement of Concrete Spillway	$114,363	5%	Around principal spillway conduit
Riprap Outlet Protection	$117,984	5%	At outlet of principal spillway

For all alternatives, the nature of the work around a full reservoir is a primary driver of cost. To facilitate work in the reservoir and/or on the upstream slope of the embankment, the entire pool must be drawn down, or as proposed for Alternative 3, the pool partially drawn down and a coffer dam installed. Managing the water during both dry and wet events without the aid of passive drawdown devices such as drains requires active practices such as pumping and cofferdam-ing which, in turn, drive costs. In addition, the cost of temporary construction access and laydown of material and equipment for construction of the improvements is extremely limited and thus a higher cost is assigned due to the potential need for smaller and more frequent deliveries and associated trucking costs as well as additional construction costs to install suitable access ways to and within the site.

For Alternative 2, the primary cost driver is the articulated concrete block mats which are required to line the auxiliary spillway to prevent erosion. These mats are relatively simple to install but have a high material cost due to the intensive factory fabrication required.

For Alternative 3, the primary cost drivers beyond dewatering and access are the coffer dam required to replace the principal spillway and the concrete encasement of the spillway conduit which is required to reduce the impacts of conduit settlement should it occur and to provide a suitable surface geometry around the circular conduit against which to compact the backfill material. Alternative 3A has similar cost drivers to Alternative 3 except the coffer dam is not as significant of a cost while and riprap protection at the principal spillway outlet to reduce downstream erosion from spillway discharges is significant.

6. Non-Financial Considerations

Beyond financial considerations, a project owner should consider non-financial aspects such as operation, inspection, and maintenance of the alternatives when selecting the preferred alternative. Table 2 presents a summary of non-financial considerations and evaluation of each alternative against them.

Consideration	Alternative 1	Alternative 2	Alternative 3	Alternative 3A
Low Level Outlet Operational Complexity	Complex (requires multiple staff and equipment to start up and shut down siphon safely)	See Alternative 1	Simple (requires one staff turning one valve)	See Alternative 3
Personal Safety Risks for Operation	Minimal – no work at heights or over water.	See Alternative 1 - Low Level Outlet and dam crest may be inaccessible during flood event if spillway has activated.	Requires traversing a catwalk over water to the riser to turn gate operator.	See Alternative 3
Embankment Risk	Encased pipe placed on embankment fill can lead to differential settlement which can cause voids, cracking, conduit misalignment, seepage, piping, etc. Settlement must be analyzed during final design. Embankment risks associated with existing principal spillway will remain	See Alternative 1	A pipe with a cradle placed on residual soil or rock presents less risk than a siphon system. Settlement must be analyzed during final design.	Although construction is the same, the location near the abutment presents a greater opportunity to place the conduit on a good foundation and subjects it to less hydrostatic head due to its higher location in the embankment.
Inspection Complexity	Low Level Outlet – Remotely operated vehicle (ROV) camera inspection through access hatch. Inlet must be bulkheaded by diver to inspect conduit below the reservoir water surface. Spillway inspection will include concrete inspection of the training walls. Principal spillway conduit will also need to be inspected via ROV.	See Alternative 1	Simple ROV or visual inspection downstream of riser structure and the structure itself. Inlet must be bulkheaded by diver to inspect conduit upstream of the riser structure.	See Alternative 3
Construction Complexity	Low Level Outlet - More pipes and fittings = more things that can go wrong during construction. Requires experienced contractor and thorough construction oversight. Left abutment spillway will require more demolition and intensive concrete work.	Low Level Outlet - more pipes and fittings = more things that can go wrong during construction. Requires experienced contractor and thorough construction oversight. Right abutment spillway will require significant earthwork and installation of articulated concrete block (ACB)'s and ACB bedding.	Construction of a conduit through an embankment and two major structures represents a significant effort and complexity of construction.	Construction of a conduit through an embankment and two major structures represents a significant effort and complexity of construction. Effort will be less than for Alternative 3 due to smaller footprint and less excavation.

Consideration	Alternative 1	Alternative 2	Alternative 3	Alternative 3A
Maintenance Requirements	Low Level Outlet - Biannual test, lubrication of all valves, Spillway - concrete repairs as needed. System has many failure locations, and one failure will fail the entire system. Trash and debris accumulation may continue to persist around existing principal spillway riser.	See Alternative 1. Additional Maintenance includes maintenance of ACB mats and turfgrass maintenance of spillway.	Low-Level Outlet - Biannual test, lubrication of the gate, concrete repairs as needed. System only has a few failure points.	See Alternative 3

7. Summary and Conclusions

Pursuant to the scope of work defined for Task #1 of Work Authorization 36, AECOM prepared rough-order-of-magnitude cost estimates for four spillway design alternatives based on sketches prepared for those alternatives under a previous task and updated with minor revisions under this task. The estimates were prepared for the purposes of comparison and selection of a preferred alternative by DeKalb County.

Attachments: Alternative Sketches

Rough-Order-of-Magnitude Cost Estimates

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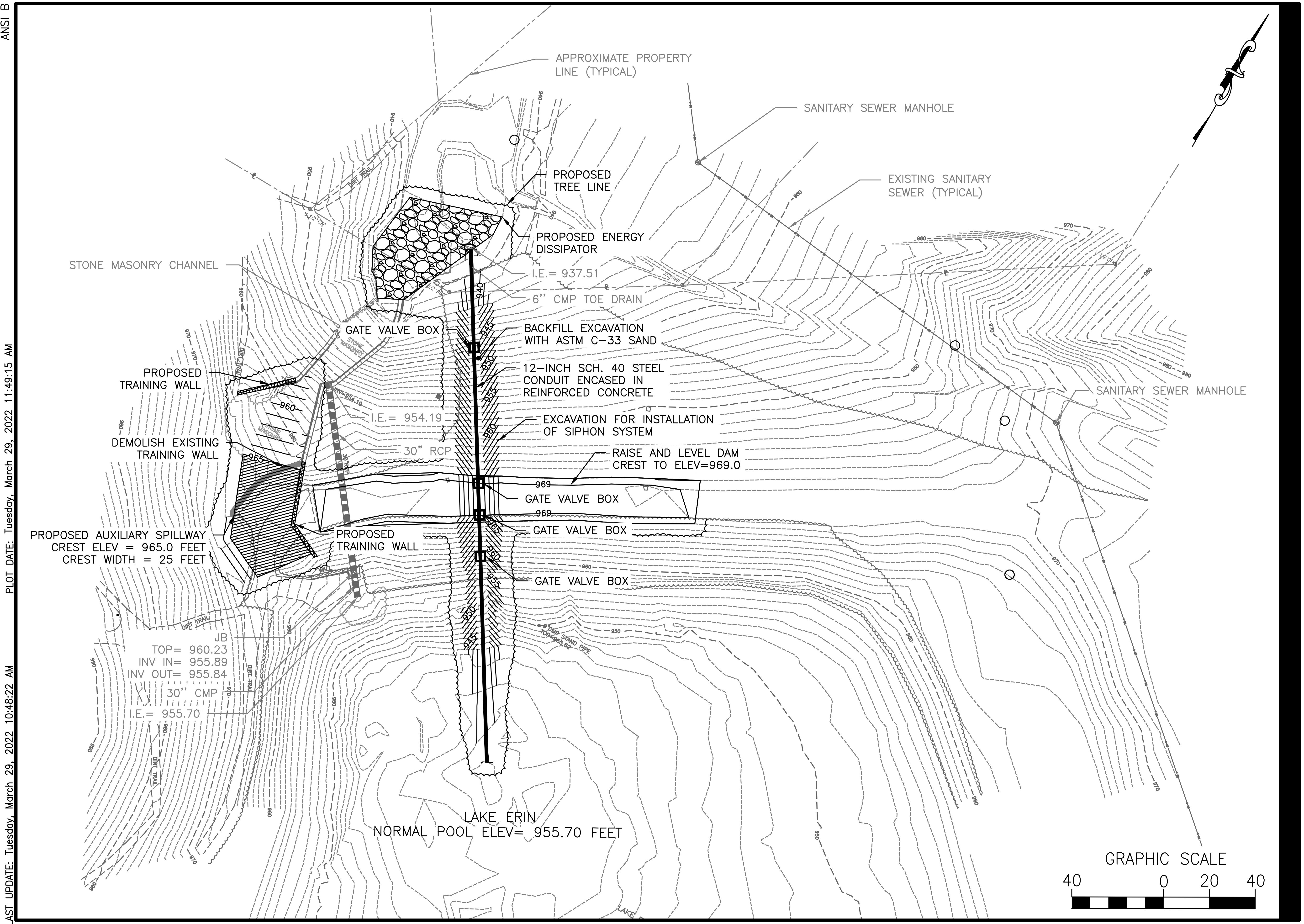


AECOM

PROJECT

ERIN LAKE DAM

OWNER/CLIENT

DEKALB COUNTY, GEORGIA
PW - ROADS & DRAINAGE C/M
729 CAMP ROAD
DECATUR, GA 30032

CONSULTANT

AECOM TECHNICAL SERVICES, INC.
1360 PEACHTREE STREET, NE
SUITE 500
ATLANTA, GA 30309
PHONE: (404) 965 9600

PROJECT NUMBER

60618914

Designed By:	
Drawn By:	MKW
Dept Check:	WCH
Proj Check:	
Date:	3/29/2022
Scale:	1" = 40'

DISCIPLINE

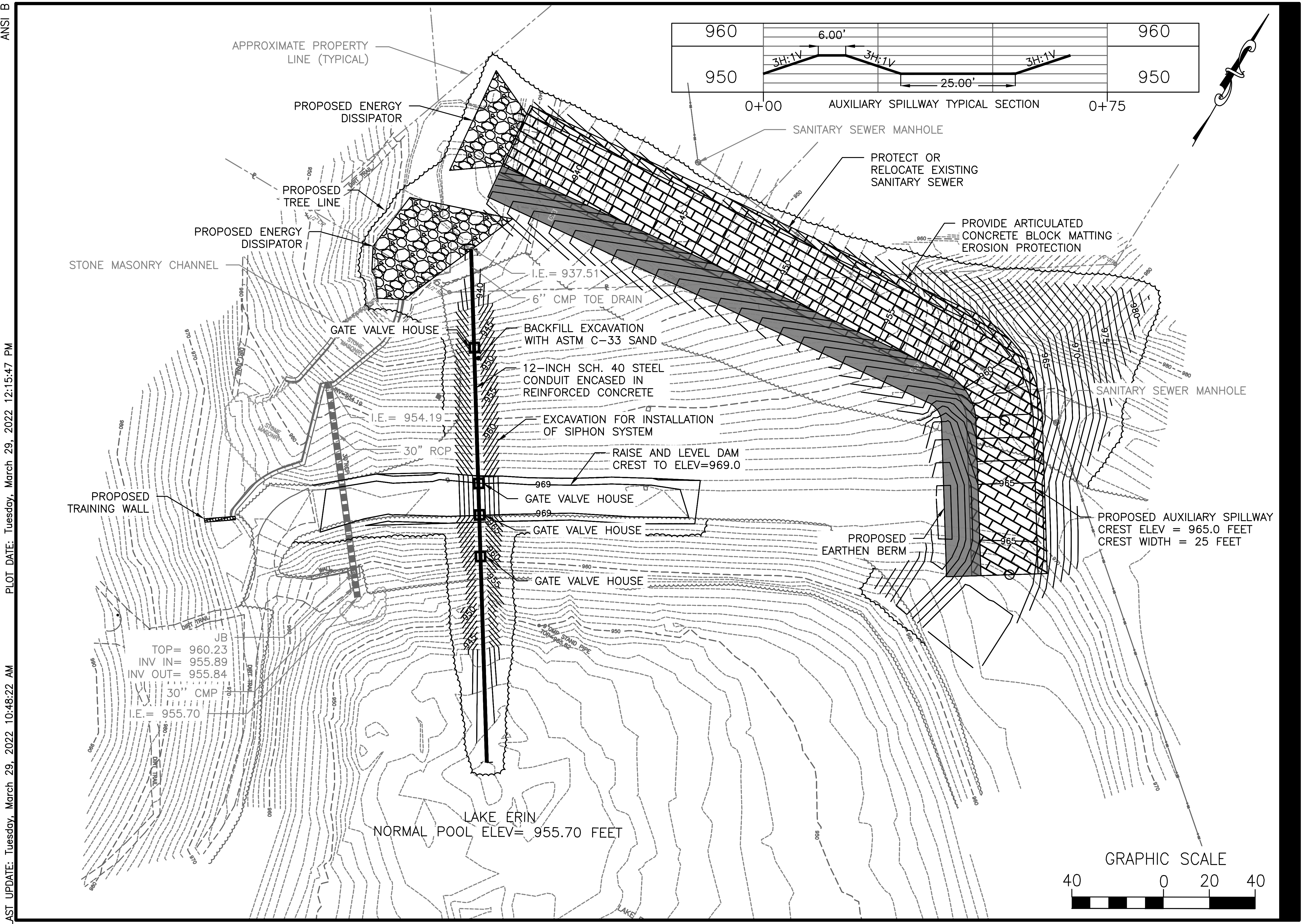
WATER

SHEET TITLE

PROPOSED ALTERNATIVE #1
PRELIMINARY SKETCH

SHEET NUMBER

1



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LAST UPDATE: Tuesday, March 29, 2022 10:48:22 AM PLOT DATE: Tuesday, March 29, 2022 12:15:47 PM

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PROJECT

ERIN LAKE DAM

OWNER/CLIENT

DEKALB COUNTY, GEORGIA
PW - ROADS & DRAINAGE C/M
729 CAMP ROAD
DECATUR, GA 30032

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Drawn By:	MKW
Dept Check:	WCH
Proj Check:	
Date:	3/29/2022
Scale:	1" = 40'

DISCIPLINE

WATER

SHEET TITLE

PROPOSED ALTERNATIVE #2
PRELIMINARY SKETCH

SHEET NUMBER

2

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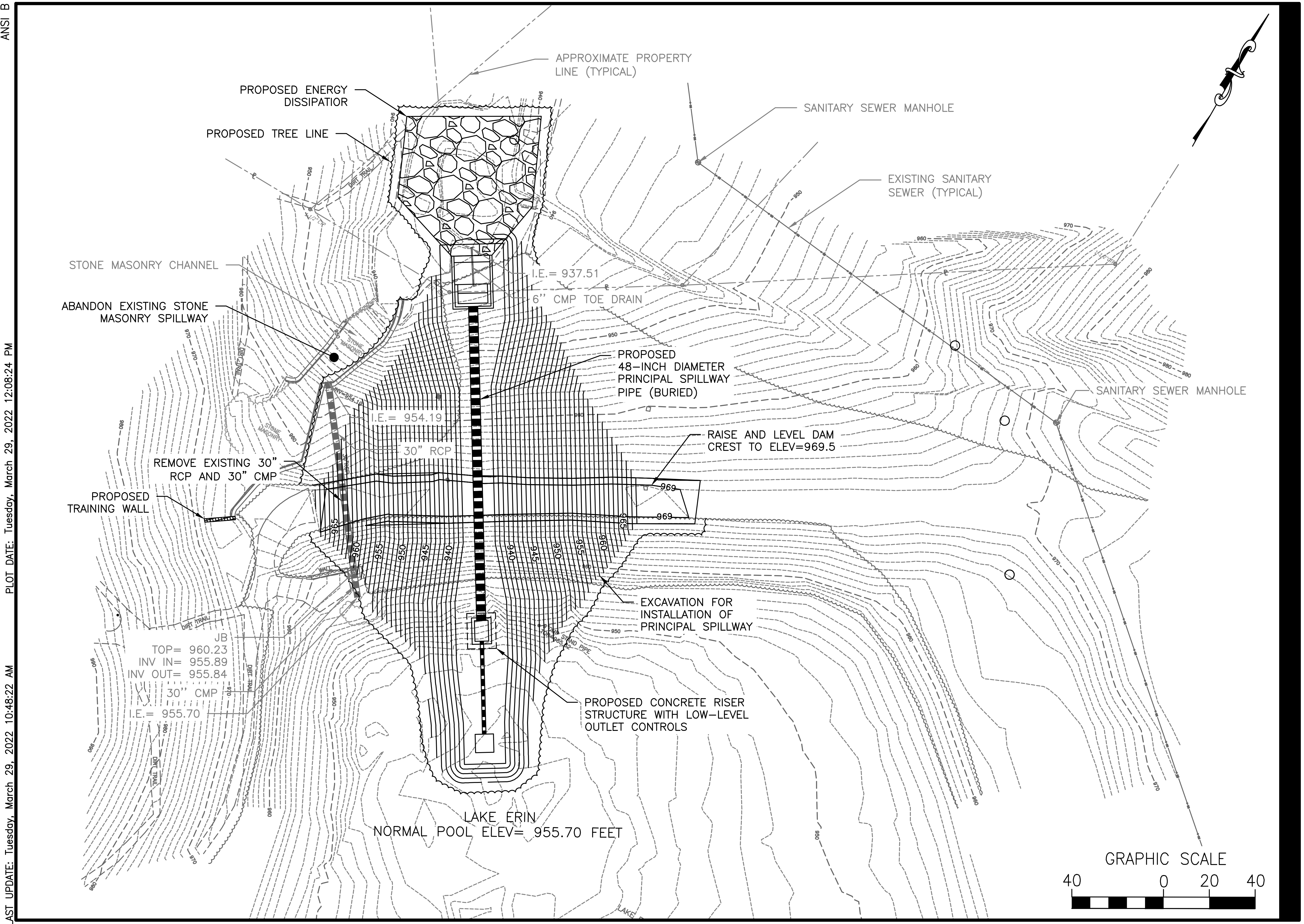

AECOM

PROJECT
ERIN LAKE DAM

OWNER/CLIENT
DEKALB COUNTY, GEORGIA
PW - ROADS & DRAINAGE C/M
729 CAMP ROAD
DECATUR, GA 30032

CONSULTANT
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1360 PEACHTREE STREET, NE
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ATLANTA, GA 30309
PHONE: (404) 965 9600

PROJECT NUMBER
60618914

Designed By:	
Drawn By:	MKW
Dept Check:	WCH
Proj Check:	
Date:	3/29/2022
Scale:	1" = 40'

DISCIPLINE
WATER

SHEET TITLE
PROPOSED ALTERNATIVE #3
PRELIMINARY SKETCH

SHEET NUMBER
3

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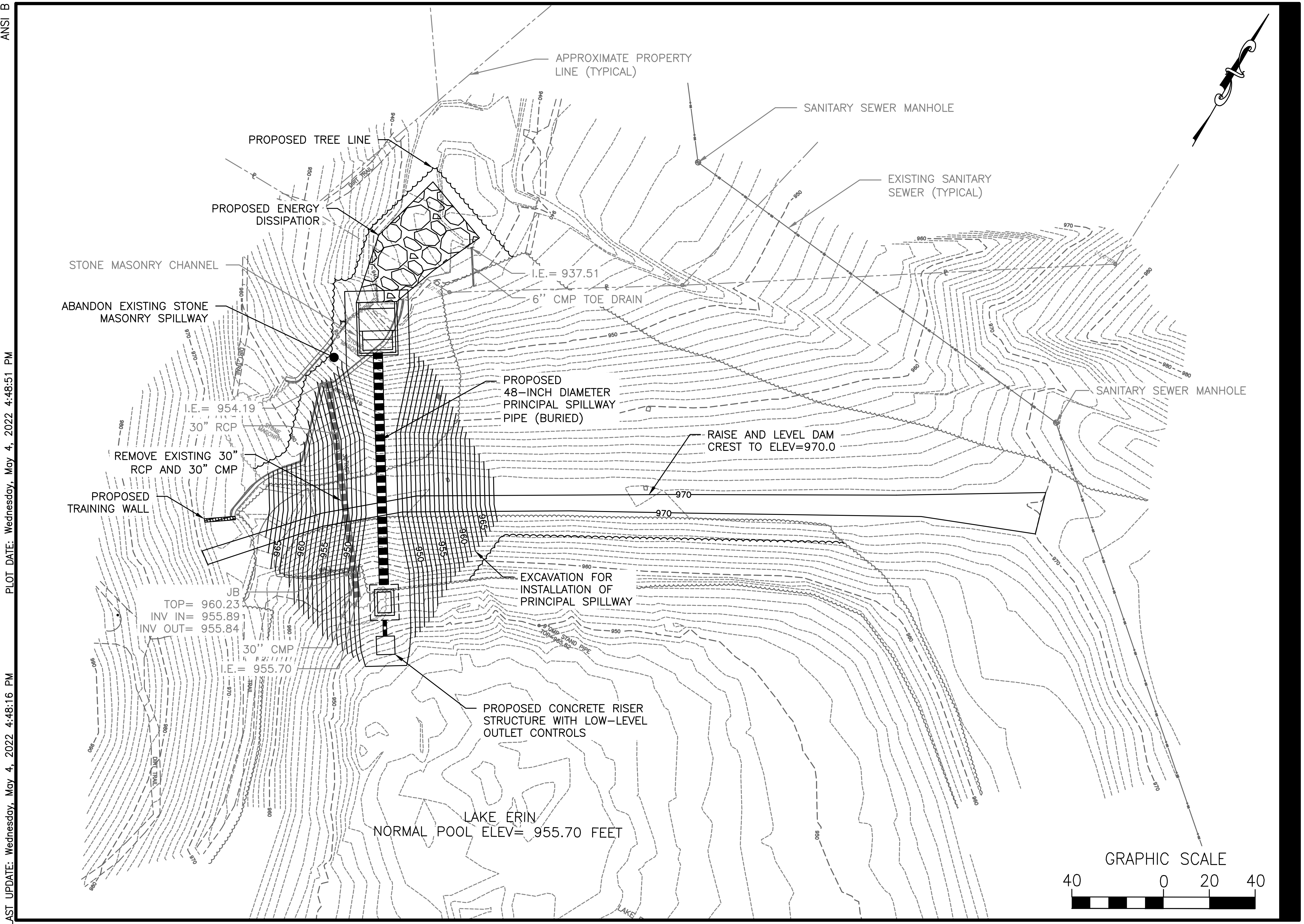

AECOM

PROJECT

ERIN LAKE DAM

OWNER/CLIENT

DEKALB COUNTY, GEORGIA
PW - ROADS & DRAINAGE C/M
729 CAMP ROAD
DECATUR, GA 30032

CONSULTANT

AECOM TECHNICAL SERVICES, INC.
1360 PEACHTREE STREET, NE
SUITE 500
ATLANTA, GA 30309
PHONE: (404) 965 9600

PROJECT NUMBER

60618914

Designed By:	
Drawn By:	MKW
Dept Check:	WCH
Proj Check:	
Date:	5/3/2022
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DISCIPLINE

WATER

SHEET TITLE

PROPOSED
ALTERNATIVE #3A
PRELIMINARY SKETCH

SHEET NUMBER

4

12240 Milestone Center Drive, Suite 150
Germantown, Maryland 20876

DESIGN: JBB DATE: 22-Mar-2022
CHECK: MKW DATE: 6-May-2022
PROJECT: Erin Lake Dam Spillway Alternative 1

ENGINEER'S ESTIMATE

ID	DESCRIPTION	UNIT	UNIT PRICE	QUANTITY	TOTAL PRICE
1	Clearing and Grubbing for Spillway Modifications	AC	$ 10,472.00	0.25	$ 2,618
2	Dewatering & Watering Management	LS	$ 220,000.00	1.00	$ 220,000
3	Temporary Access	LS	$ 160,000.00	1.00	$ 160,000
4	Siphon Conduit - 12-inch diameter Schedule 40 Steel Conduit	LF	$ 224.36	230	$ 51,603
5	Reinforced Concrete Encasement of Steel Conduit	CY	$ 571.82	70	$ 40,027
6	12-inch Non-Rising Stem Gate Valve	EA	$ 10,764.85	4	$ 43,059
7	Valve Boxes (4'L x 4'W x 4'D interior, 12-inch thick walls and slabs)	EA	$ 4,871.41	4	$ 19,486
8	Excavation for Conduit and Boxes (dispose of on site)	CY	$ 9.46	230	$ 2,177
9	Backfill for Conduit and Boxes (import ASTM C-33 Sand)	CY	$ 24.45	230	$ 5,623
10	Dam Crest Backfill to Grade (using excavated material)	CY	$ 17.34	70	$ 1,214
11	Demolish Existing Masonry Wall (5-foot high)	CY	$ 385.46	9	$ 3,469
12	Demolish Existing Masonry Slab (12-inches thick)	CY	$ 385.46	42	$ 16,189
13	Concrete Training Walls (75 LF total x 4'H x 1.5't, assume equiv. ftg)	CY	$ 817.16	40	$ 32,686
14	Spillway Grading (10 CY Cut/50 CY Fill all to or from on-site)	SY	$ 16.56	300	$ 4,968
15	Concrete Slab (12-inches thick)	SY	$ 685.04	42	$ 28,772
16	Riprap Outlet Protection (1400 SF x 36-inches thick)	TON	$ 157.31	350	$ 55,059
17	Topsoil/Seed/Mulch	SY	$ 17.66	1600	$ 28,250
Construction Line Item Subtotal					$ 715,200
Mobilization					$ 133,591
Construction Subtotal					$ 848,791
Engineering and Permitting (18% of Construction Subtotal)					$ 152,782
Project Subtotal					$ 1,001,573
Contingency (25% of Project Subtotal)					$ 250,393
Project Total (to the nearest $10,000)					**$ 1,250,000**

AECOM

12240 Milestone Center Drive, Suite 150
Germantown, Maryland 20876

DESIGN: JBB DATE: 22-Mar-2022
CHECK: MKW DATE: 6-May-2022
PROJECT: Erin Lake Dam Spillway Alternative 2

ENGINEER'S ESTIMATE

ID	DESCRIPTION	UNIT	UNIT PRICE	QUANTITY	TOTAL PRICE
1	Clearing and Grubbing for Spillway Modifications	AC	$ 10,472.00	0.50	$ 5,236
2	Dewatering & Watering Management	LS	$ 220,000.00	1.00	$ 220,000
3	Temporary Access	LS	$ 160,000.00	1.00	$ 160,000
4	Siphon Conduit - 12-inch diameter Schedule 40 Steel Conduit	LF	$ 224.36	230	$ 51,603
5	Reinforced Concrete Encasement of Steel Conduit	CY	$ 571.82	70	$ 40,027
6	12-inch Non-Rising Stem Gate Valve	EA	$ 10,764.85	4	$ 43,059
7	Valve Boxes (4'L x 4'W x 4'D interior, 12-inch thick walls and slabs)	EA	$ 4,871.41	4	$ 19,486
8	Excavation for Conduit and Boxes (dispose of on site)	CY	$ 9.46	230	$ 2,177
9	Backfill for Conduit and Boxes (import ASTM C-33 Sand)	CY	$ 24.45	230	$ 5,623
10	Dam Crest Backfill to Grade (using excavated material)	CY	$ 17.34	70	$ 1,214
11	Auxiliary Spillway Excavation (dispose of on site)	CY	$ 9.46	2,500	$ 23,660
12	Auxiliary Spillway Fill (fill from on site)	CY	$ 17.34	500	$ 8,668
13	Articulated Concrete Block Mats for Auxiliary Spillway	SY	$ 360.00	1,800	$ 648,000
14	Aggregate Bedding for ACBs (12-inches thick)	TON	$ 102.40	1,200	$ 122,880
15	Concrete Training Wall (15 LF total x 4'H x 1.5't, assume equiv. ftg)	CY	$ 817.16	10	$ 8,172
16	Riprap Outlet Protection (1400 SF x 36-inches thick)	TON	$ 157.31	350	$ 55,059
17	Topsoil/Seed/Mulch	SY	$ 17.66	3500	$ 61,796
Construction Line Item Subtotal					$ 1,476,660
Mobilization					$ 276,365
Construction Subtotal					$ 1,753,025
Engineering and Permitting (18% of Construction Subtotal)					$ 315,545
Project Subtotal					$ 2,068,570
Contingency (25% of Project Subtotal)					$ 517,143
Project Total (to the nearest $10,000)					**$ 2,590,000**

12240 Milestone Center Drive, Suite 150
Germantown, Maryland 20876

DESIGN: JBB
DATE: 25-Mar-2022
CHECK: MKW
DATE: 6-May-2022
PROJECT: Erin Lake Dam Spillway Alternative 3

ENGINEER'S ESTIMATE

ID	DESCRIPTION	UNIT	UNIT PRICE	QUANTITY	TOTAL PRICE
1	Clearing and Grubbing for Spillway Modifications	AC	$ 10,472.00	0.25	$ 2,618
2	Dewatering & Watering Management	LS	$ 260,000.00	1.00	$ 260,000
3	Temporary Access	LS	$ 200,000.00	1.00	$ 200,000
4	Coffer Dam (Sheet Pile 287' Long X 30' Sheet Length)	SF	$ 21.60	8,610	$ 185,976
5	Demolish Existing 30" RCP and Riser Structure	LF	$ 658.80	90	$ 59,292
6	Spillway Conduit (48-inch AWWA C-300)	LF	$ 366.18	150	$ 54,928
7	Reinforced Concrete Encasement of Spillway Conduit	CY	$ 571.82	275	$ 157,249
8	Excavation for Principal Spillway	CY	$ 9.46	9,400	$ 88,962
9	Backfill for Principal Spillway (Fill from Excavation)	CY	$ 17.34	9,400	$ 162,958
10	Filter Diaphragm Material (ASTM C-33 Sand)	CY	$ 24.45	200	$ 4,890
11	Concrete Riser Structure	CY	$ 1,142.40	70	$ 79,968
12	Concrete Riser Trash Racks	EA	$ 6,000.00	2	$ 12,000
13	Drain Conduit (12-inch AWWA C-302)	LF	$ 152.00	60	$ 9,120
14	16" x 16" Sluice Gate, Stem Extension with guides, hand operator	EA	$ 47,046.85	1	$ 47,047
15	Catwalk Bridge from Dam to Riser (3 feet wide)	SF	$ 168.69	135	$ 22,773
16	Concrete Type VI Impact Basin	CY	$ 1,222.40	60	$ 73,344
17	Riprap Outlet Protection (3100 SF x 36-inches thick)	TON	$ 157.31	750	$ 117,984
18	Topsoil/Seed/Mulch	SY	$ 17.66	1800	$ 31,781
Construction Line Item Subtotal					$ 1,570,890
Mobilization					$ 281,477
Construction Subtotal					$ 1,852,367
Engineering and Permitting (18% of Construction Subtotal)					$ 333,426
Project Subtotal					$ 2,185,793
Contingency (25% of Project Subtotal)					$ 546,448
Project Total (to the nearest $10,000)					**$ 2,730,000**

AECOM

12240 Milestone Center Drive, Suite 150
Germantown, Maryland 20876

DESIGN: JBB DATE: 25-Mar-2022
CHECK: MKW DATE: 6-May-2022
PROJECT: Erin Lake Dam Spillway Alternative 3A

ENGINEER'S ESTIMATE

ID	DESCRIPTION	UNIT	UNIT PRICE	QUANTITY	TOTAL PRICE
1	Clearing and Grubbing for Spillway Modifications	AC	$ 10,472.00	0.25	$ 2,618
2	Dewatering & Watering Management	LS	$ 260,000.00	1.00	$ 260,000
3	Temporary Access	LS	$ 200,000.00	1.00	$ 200,000
4	Coffer Dam (Sheet Pile 115' Long X 30' Sheet Length)	SF	$ 21.60	3,450	$ 74,520
5	Demolish Existing 30" RCP and Riser Structure	LF	$ 658.80	90	$ 59,292
6	Spillway Conduit (48-inch AWWA C-300)	LF	$ 366.18	110	$ 40,280
7	Reinforced Concrete Encasement of Spillway Conduit	CY	$ 571.82	200	$ 114,363
8	Excavation for Principal Spillway	CY	$ 9.46	3,750	$ 35,490
9	Backfill for Principal Spillway (Fill from Excavation)	CY	$ 17.34	3,750	$ 65,010
10	Filter Diaphragm Material (ASTM C-33 Sand)	CY	$ 24.45	200	$ 4,890
11	Concrete Riser Structure	CY	$ 1,142.40	60	$ 68,544
12	Concrete Riser Trash Racks	EA	$ 6,000.00	2	$ 12,000
13	Drain Conduit (12-inch AWWA C-302)	LF	$ 152.00	20	$ 3,040
14	16" x 16" Sluice Gate, Stem Extension with guides, hand operator	EA	$ 47,046.85	1	$ 47,047
15	Catwalk Bridge from Dam to Riser (3 feet wide)	SF	$ 168.69	135	$ 22,773
16	Concrete Type VI Impact Basin	CY	$ 1,222.40	60	$ 73,344
17	Riprap Outlet Protection (3100 SF x 36-inches thick)	TON	$ 157.31	750	$ 117,984
18	Topsoil/Seed/Mulch	SY	$ 17.66	1800	$ 31,781
Construction Line Item Subtotal					$ 1,232,976
Mobilization					$ 229,877
Construction Subtotal					$ 1,462,853
Engineering and Permitting (18% of Construction Subtotal)					$ 263,314
Project Subtotal					$ 1,726,167
Contingency (25% of Project Subtotal)					$ 431,542
Project Total (to the nearest $10,000)					**$ 2,160,000**

AECOM
12420 Milestone Center Drive, Suite 150
Germantown, Maryland 20876
aecom.com

Project Name:
Lake Erin Dam

Project Reference
Slope Stability and Seepage Analysis
Existing Conditions

From:
Bob Pinciotti, P.E.
Thomas Wachtel, Ph.D., P.E.

Date:
May 10, 2022

To:
Ms. Peggy Allen
Roads and Drainage Division
Department of Public Works
DeKalb County, Georgia
727 Camp Road
Decatur, Georgia 30032

CC:
AECOM - Malavika Tripathi, P.E.
FILE

Technical Memo – Geotechnical Analyses

Geotechnical Analyses Background

This memo presents the analysis and parameters used to conduct modeling of the existing conditions at Erin Lake Dam. The scope of work for Erin Lake Dam included reviewing available historical project data and information, and performing two-dimensional slope stability and seepage analyses of the embankment. The analysis of the embankment was conducted in general accordance with the requirements in Georgia Department of Natural Resources *Engineer Guidelines 2015 Edition Version 4.0* (GADEP, 2015) and Rules and Regulations of the State of Georgia Chapter 391-3-8. *Rules for Dam Safety* (State of Georgia, 2020).

Overview

One cross section of the Erin Lake Dam embankment was evaluated for seepage and slope stability. The cross section analyzed is located along the centerline of the 6-inch toe drain outlet, along the approximate maximum height of the dam. The location of the cross section is shown in **Figure 1**. The cross section was developed using historical drawings, topographic and bathymetric survey data, and data obtained from recent and historical subsurface investigation borings. Downstream subsurface drain elevation was estimated from test pit excavations performed by Accura in 2021.

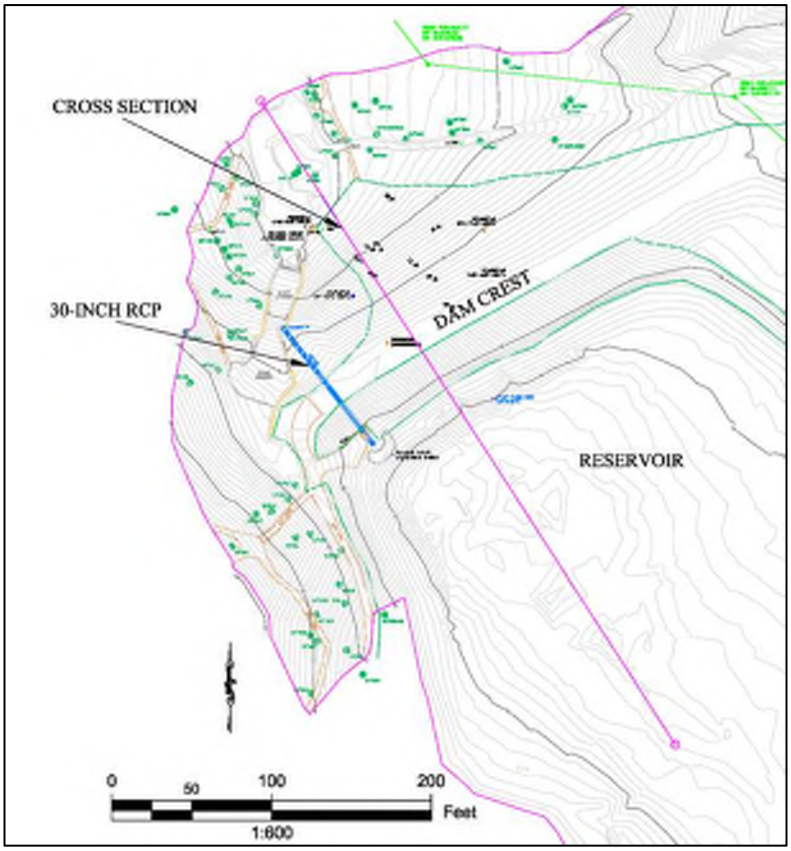

Figure 1: Cross Section Location

Selection of Engineering Properties

AECOM reviewed subsurface exploration data to determine soil and rock engineering properties for slope stability and seepage analyses. Elevations for the material layers were estimated within the embankment beneath the crest of the dam and at the downstream toe. No borings were drilled on the upstream section of the dam. Soil and rock material properties at Erin Lake Dam were categorized into four general layers based on soil index properties, regional geology, depth, and location. These layers are described below.

Geotechnical investigation indicates that the embankment and underlying geology consist of Embankment Fill, underlain by Alluvium, Residual Soil, and bedrock. The Alluvium and Residual Soil layer thickness and elevations were based on results of subsurface investigation performed by Accura in 2021, which is presented in their Geotechnical Data report (GDR), and compared with historical documentation. A rehabilitation engineering report by Willmer/Kimley Horn was provided with borings drilled and piezometers installed by Willmer in 2013. The modeled existing embankment section is shown in **Figure 2** below.

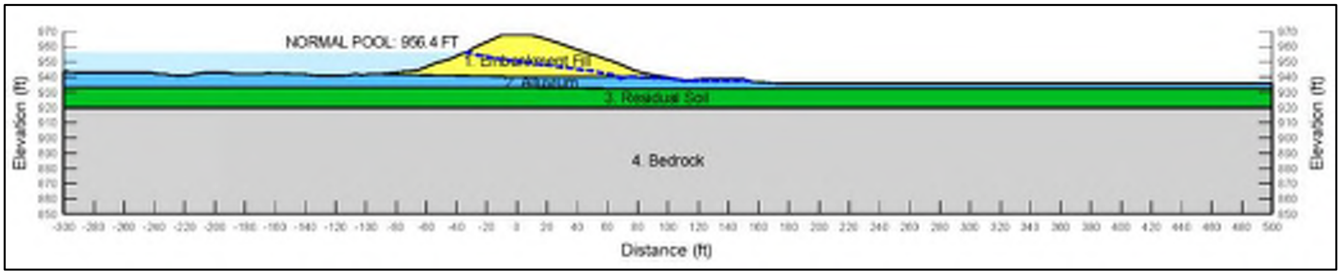

Figure 2: Modeled Existing Embankment Section

Embankment Fill

Embankment Fill was encountered within Boring AB-2 at the dam crest Boring AB-3 at the dam toe. Fill material was also encountered along the left abutment in boring AB-1 and the right abutment in borings AB-4 and AB-5. However, these borings were not utilized for Embankment Fill in the model as they are located outside of the embankment.

The stratum was visually described as reddish yellow and dark grey silty SAND, greenish grey clayey SAND, yellowish red sandy SILT and grey to dark reddish brown sandy CLAY. Five jar samples of Embankment Fill material from Boring AB-2 and one sample from Boring AB-3 were obtained during subsurface investigation. Five of these sample classified as silty SAND (AB-2 between 2 to 18 feet depth and AB-3, 0-2 feet depth), one as CLAY (AB-2, 20-22 feet depth), and one as clayey SAND (AB-2, 26-28 feet depth). The natural moisture content of these samples ranged from 19.5% to 25.4% with an average of 22.1%. Fines content ranged from 38.8% to 53.1% with an average of 43.6%. Samples AB-2 (20-22 feet) and AB-2 (26-28 feet) were tested to have Plasticity Indexes (PI) of 16 and 8, respectively. The remained Embankment Fill samples were determined to be non-plastic except for AB-2 (6-8 feet) which was not tested for plasticity.

One hydraulic conductivity test was performed on a relatively undisturbed sample of material obtained from offset Boring AB-2A between depths of 10 feet to 12 feet. The hydraulic conductivity of the sample was determined by falling head test to be 2.7 x 10^{-7} cm/sec.

One Consolidated Isotropic Undrained triaxial test (CIU), with Pore Water Pressure measurements was performed on a relatively undisturbed Shelby tube sample obtained in Boring AB-2A at 22 feet to 24 feet depth. Laboratory test results indicate the Embankment Fill has an effective friction angle of 28.2 degrees and effective cohesion of 0.232 psi (33.4 psf). Total friction angle was tested to be of 14.9 degrees with a total cohesion of 2.09 psi (301psf).

Alluvium

Alluvium was encountered within Boring AB-2 on the dam crest Boring AB-3 at the downstream toe of the embankment based on boring logs and blow counts. The stratum was visually described as grey silty SAND and clayey SAND. One jar sample from Boring AB-2 and two from Boring AB-3 were obtained during subsurface investigation. Two of these samples were USCS classified as silty SAND (AB-2, 30-32 feet and AB-3, 12-14 feet) and one as clayey SAND (AB-3, 6-8 feet). The natural moisture content of these samples ranged from 21.8% to 49.5% with an average of 33.3%. Fines content ranged from 22.6% to 47.6% with an average of 34.0%. The clayey SAND sample from boring AB-3 (6-8 feet) had a plasticity index of 10, while the silty SAND sample from AB-3 (12-14 feet) was determined to be nonplastic. The sample from Boring AB-2 (30-32 feet, SM) was not tested for plasticity.

One Consolidated Undrained with Pore Water Pressure (CU) test was performed on a relatively undisturbed Shelby tube sample obtained in Boring AB-3A at 7 feet to 9 feet depth. This material was estimated to be Alluvium given the depth and location of the sample taken. Laboratory test results indicate the Alluvium has an effective friction angle of 26.6 degrees and effective cohesion of 0.0784 psi (11.3 psf). Total friction angle was tested to be of 14.6 degrees with a total cohesion of 1.75 psi (252 psf).

Residual Soil

Residual Soil was encountered beneath the Alluvium soils in all borings drilled. For this analysis, the location of the Residual Soil was estimated based on boring logs, blow counts, identification of Saprolite (weathered or decomposed rock), and historical documentation The Residual Soil was visually classified as black, brown, and white silty SAND and sandy SILT. Two jar samples were tested for Residual Soil; one in Boring AB-4 and one in Boring AB-5. Both samples classified as silty SAND. Natural moisture content ranged from 27.6% to 33.2% with an average of 30.4%. Fines content ranged from 25.6% to 31.3% with an average of 28.5%. Both samples tested as nonplastic.

Bedrock

According to the Geotechnical Data Report (Accura, 2021) the project site is underlain by the Granite Gneiss/Amphibolite formation in the Piedmont region. Bedrock was encountered in Boring AB-1 along the left abutment at an approximate depth of 8.5 feet. The bedrock was cored a total of 10 feet (to 18.5 ft depth). For each 5 feet core run, the recovery and RQD of the rock was 100%, indicating excellent rock quality.

One unconfined compressive strength test was performed on a cored sample of the bedrock. The laboratory test resulted in an unconfined compressive strength of 12,570 psi (86.7 MPa). Based on Natural Resources Conservation Service (NRCS) National Engineering Handbook Part 631 Geology Chapter 4 Engineering Classification of Rock Materials (2012), the bedrock encountered at Erin Lake Dam is considered “hard rock”.

Bedrock was not encountered beneath the embankment prior to termination in either Boring AB-2 on the crest of the dam or Boring AB-3 on at the downstream toe. However, the top of the bedrock was modelled to be at 920 ft EL based on weathered rock encountered in Boring AB-3 from subsurface investigation performed in 2021. Previous subsurface investigation encountered auger refusal west of the modelled cross section at the location of piezometers P-4 and P-6 at approximate elevations 935 ft and 927.5 feet, respectively. Therefore, 920 ft is considered a conservative elevation for top of bedrock.

Parameter Selection for Seepage Analysis

Hydraulic conductivities for soil and rock were estimated using laboratory test results, local knowledge, model calibration, and engineering judgement. A laboratory permeability test was performed on one (1) undisturbed sample in the Embankment Fill stratum (Boring AB-2A, 10-12 feet depth) in general accordance with ASTM D5084, with a result of 2.7×10^{-7} cm/sec. Details of the hydraulic conductivity testing can be found in the referenced Geotechnical Data Report prepared by Accura (2021).

Selected hydraulic conductivity values for soil materials were compared against typical values referenced in NRCS National Engineering Handbook Part 631 Chapter 3 Engineering Classification of Earth Materials (2012). Bedrock hydraulic conductivity was compared against typical values referenced in NRCS National Engineering Handbook Part 631 Geology Chapter 4 Engineering Classification of Rock Materials (2012)

Calibration of the seepage material properties was performed by adjusting the hydraulic conductivity parameters of the seepage model until the results closely met previously measured groundwater levels obtained in the open well piezometers location along the crest and downstream slope of the dam.

The anisotropic relationship between horizontal and vertical hydraulic conductivity for the soil layers was estimated based on engineering judgement and model calibration. A summary table of the hydraulic conductivity and anisotropy values are presented in **Table 1**.

Table 1: Summary of Seepage Analysis Input Parameters

Material Description	Range of Typical Hydraulic Conductivity Values (cm/sec) (NRCS)		Laboratory Tested Values (k_v) (cm/sec)	Selected Vertical Hydraulic Conductivity, k_v (cm/sec)	Selected Anisotropy (k_v/k_h)
	maximum	minimum			
Embankment Fill	1.00E-03 (SM) 1.00E-06 (SC)	1.00E-06 (SM) 1.00E-08 (SC)	2.7E-07	2.7E-07	0.25
Alluvium	1.00E-03 (SM) 1.00E-06 (SC)	1.00E-06 (SM) 1.00E-08 (SC)	-	1.10E-06	0.67
Residual Soil	1.00E-03 (SM)	1.00E-06 (SM)	-	2.50E-06	0.5
Bedrock	1.2E-08 unfractured igneous and metamorphic rock	1.2E-12 unfractured igneous and metamorphic rock	-	1.00E-09	1

Parameter Selection for Slope Stability Analysis

Soil strength material properties for the modeled Embankment Fill, Alluvium, Residual Soil, and Bedrock layers were selected based on laboratory testing, SPT results, empirical values, and engineering judgement. Empirical values used for soil material property selection were obtained from US Bureau of Reclamation “Earth Manual” Part 1, 3rd edition (1998) and NRCS National Engineering Handbook Part 631 Chapter 3 Engineering Classification of Earth Materials (2012).

Effective and total stress shear strength parameters utilized in the analysis for Embankment Fill were based on the CIU triaxial test performed on a relatively undisturbed Shelby tube sample taken in the Embankment Fill from Boring AB-2A. For this analysis, an effective friction angle of 28 degrees and effective cohesion of 33 psf was used. Total friction angle of 15 degrees with a total cohesion of 300 psf was used for total strength parameters.

Alluvium soil strength parameters were selected based on the CIU triaxial test performed on a relatively undisturbed Shelby tube sample taken from Boring AB-3A as it was determined to be within the Alluvium layer based on observation and historical documentation. The selected soil strength parameters for Alluvium used in the analysis were an effective friction angle of 27 degrees, effective cohesion of 11 psf, total friction angle of 15 degrees and total cohesion of 252 psf.

Residual soil effective strength material properties were estimated based on SPT blow counts from the subsurface investigation performed by Accura. Effective friction angle used for the analysis was 31 degrees. Effective cohesion was conservatively assumed to be 0 psf. Total strength material properties were estimated based on the Alluvium layer given similarities between the layers. A total friction angle of 18 degrees was selected as the subsurface investigation indicated the Residual Soil to have a resistance to penetration and therefore higher strength than the Alluvium.

Bedrock cohesion was estimated to be one-half of the unconfined compressive strength. Cohesion equaling one-half compressive strength is based on a zero-degree friction angle and cohesion equal to one-half the difference between major and minor principal stresses using Mohr’s circle. As the compressive strength is unconfined, the minor principal stress is 0 psi. Therefore, the Mohr’s circle radius is equal to one half of the major principal stress, which is the resultant compressive strength of rock. Material strength for rock was based on one unconfined compressive strength test performed by Accura which was 12,570 psi (1.81E-06 psf). However, fractures within bedrock can reduce overall compressive strength. Therefore, a cohesion of 225,000 psf (one-quarter of the determined cohesion based on laboratory test results) and effective friction angle of 0 degrees were conservatively selected for Bedrock material properties.

Saturated unit weight for the Embankment Fill and Alluvium were based on measured densities from the hydraulic conductivity and CIU triaxial test results provided in the Geotechnical Data Report (Accura, 2021). Saturated unit weight for Residual Soil was based on USBR Earth Manual Part 1, 3E (1998), NRCS National Engineering Handbook Part 631 Chapter 3 Engineering Classification of Earth Materials (2012) and laboratory test data. Saturated unit weight of the Bedrock was estimated to be 165 pcf.

Soil parameters selected for the stability analysis are summarized in Table 2 below.

Table 2: Summary of Stability Analysis Selected Soil Parameters

Material Description	Saturated Unit Weight (pcf)	Shear Strength Parameters			
		Total Stress		Effective Stress	
		Φ (deg)	c (psf)	Φ’ (deg)	c’ (psf)
Embankment Fill	124.8	15	301	28	33
Alluvium	128.6	15	252	27	11
Residual Soil	132	18	250	31	0
Bedrock	165	0	225,000	0	225,000

Seepage and Slope Stability Analysis

Seepage Analysis

The current standard of engineering practice is to perform seepage analysis by means of a simplified two-dimensional analysis. For this site, a two-dimensional finite element seepage analysis was performed to estimate the phreatic surface and porewater pressure distribution in the dam and foundation for use in the slope stability analysis under steady-state conditions.

The seepage analyses were performed using GeoStudio 2020 SEEP/W computer modelling software in general accordance with USACE EM 1110-2-1901 Engineering and Design “Seepage Analysis and Control for Dams” (1993). Calibration of the seepage models was performed using pool conditions with known surface elevations and from piezometer groundwater measurements. During calibration of the model, water levels from the piezometers located along the crest and downstream slope of the dam were compared with the modelled phreatic surface within the embankment. The modelled hydraulic conductivities of the materials were adjusted until they closely matched the measured water elevations.

Boundary conditions were set within SEEP/W to simulate observed conditions at the dam. At the reservoir, a boundary condition for the head elevation of the pool level (normal pool or maximum pool) was used in each model. Normal pool elevation was determined from historical measurements and topographical survey performed by Accura in 2021. Hydrologic and hydraulic analysis shows Erin Lake Dam would overtop in the event of a 1/3 24-hr probable Maximum Precipitation (PMP) event. Maximum pool elevation was therefore set at the approximate top of dam (968.0 ft) for reference to show the response of the dam at the existing maximum potential retention elevation. Tailwater elevation for maximum pool was set at 940.0 ft based on H&H analysis.

Erin Lake Dam incorporates an internal drainage system consisting of 8-inch corrugated pipe. These pipes drain along the right and left downstream groin, intersecting at a “T” junction before outletting at the downstream toe. For this analysis, a drain was modeled in the seepage analysis at the approximate location of the “T” connection.

The boundary conditions used for seepage analysis are:

- Normal Pool Elevation: 956.4 ft
- Normal Pool Tailwater Elevation: 937.51 ft (invert elevation of toe drain)
- “Submerged Toe” Tailwater Elevation: 940.0 ft
- Maximum Surcharge Pool Elevation: 968.0 ft (Approximate top of dam)
- Maximum Surcharge Pool Tailwater Elevation: 940.0 ft

Figure 3 below shows typical porewater pressure contours used to model the seepage flow through the embankment. Results of the seepage analyses are provided in **Attachment A**.

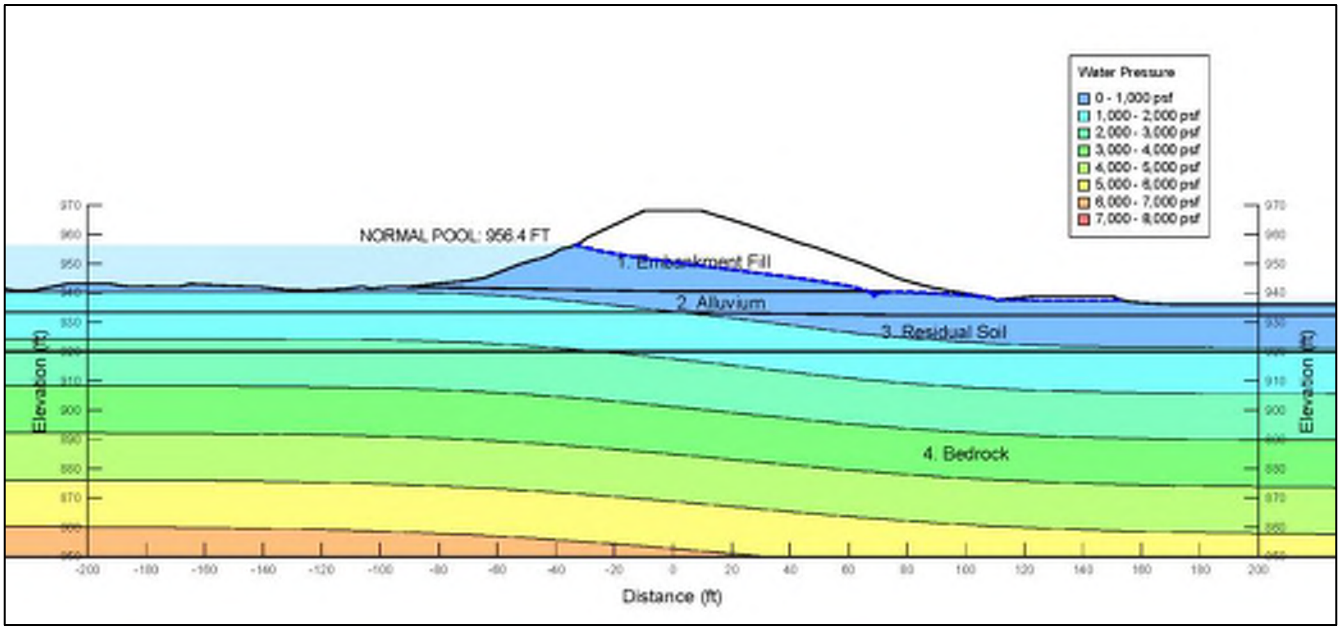

Figure 3: Example Porewater Pressure Distribution

Slope Stability Analyses

The stability evaluation was performed utilizing GeoStudio 2020 SLOPE/W computer modeling software package. The analysis utilizes section geometry with input parameters including soil parameters and loading conditions. The slope stability analyses of the embankment were performed using Spencer's method of slices. Spencer's method of slices satisfies all conditions of static equilibrium for horizontal and vertical force equilibrium and moment equilibrium. Factors of safety for slope stability analysis were analyzed using a minimum slip surface depth of two feet.

The embankment slope stability analysis was conducted using required minimum factors of safety from the Georgia Rules and Regulations 391-3-8 *Rules for Dam Safety* (Rule 391-3-8-.09) for the stability of earth embankment structures and USACE EM 110-2-1902 *Slope Stability* (2003). This guidance provides minimum factor safety values for steady state seepage (normal and maximum storage pool), maximum surcharge pool, rapid drawdown (upstream), submerged toe with rapid drawdown (downstream) and earthquake loading conditions. The minimum required factors of safety for the loading conditions are presented in **Table 3** below.

Table 3: USACE and Georgia Rules for Dam Safety Slope Stability Required Factors of Safety

Minimum Required Factors of Safety			
Analysis Condition	**Required Minimum Factor of Safety (USACE)**	**Required Minimum Factor of Safety (Georgia State Regulations)**	**Slope**
Steady State Seepage – Maximum Storage Pool (Normal Pool)	1.5	1.5	Downstream
Steady State Seepage - Maximum Surcharge Pool	1.4	N/A	Downstream
Steady State Seepage with Earthquake Loading (Normal Pool)	N/A	1.1	Downstream
Rapid Drawdown (Upstream)	1.3	1.3	Upstream
Submerged Toe with Rapid Drawdown (Downstream)	N/A	1.3	Downstream

Slope stability analysis was performed on Erin Lake Dam for existing conditions to evaluate the embankment and determine if it meets the minimum required factors of safety for specified loading conditions.

Loading Conditions

The Erin Lake Dam existing embankment was analyzed for stability under steady state seepage, rapid drawdown, submerged toe with rapid drawdown and earthquake loading conditions. Each of these conditions are described below.

Long-term Steady-State Seepage Conditions

Steady-State conditions were considered long-term and pore pressures within the embankment were modelled to reach steady state conditions at normal pool (storage pool) elevation. This analysis uses effective stress parameters for soil and rock material strength. While not required by Georgia State Regulations, the embankment was also analyzed for steady state conditions at the maximum designed pool level (1/3 24-hr PMP). As H&H analysis shows the existing dam will slightly overtop during a 1/3 24-hr PMP event, the maximum surcharge pool used in the analysis was set at the approximate top of dam elevation (968.0 ft).

Upstream Rapid Drawdown

Rapid drawdown slope stability analysis for the upstream embankment slope was performed using the Slope Stability for Rapid Drawdown Method developed by Duncan, Wright, and Wong (1990) and detailed in USACE EM 1110-2-1902 (2003). Duncan's method utilizes both effective and total strength parameters in its analysis. This is a conservative method as it models instantaneous drawdown of the reservoir. This method uses two phreatic surfaces.

For Rapid Drawdown, the initial phreatic surface is developed for steady-state conditions at the maximum storage pool (normal pool) elevation. The pool level is then modelled to rapidly drawdown to the lowest gated or ungated outlet elevation. For Erin Lake Dam, the bottom of lake elevation was selected as the low elevation for drawdown. The slope stability is then analyzed at the upstream slope.

Submerged Toe with Rapid Drawdown

For Submerged Toe with Rapid Drawdown at Erin Lake Dam, the tailwater level is modelled to have risen on the downstream embankment slope due to outlet flow from a significant precipitation event. The tailwater is then modelled to rapidly drawdown to its normal observed elevation. The slope stability is then analyzed at the downstream toe.

For this analysis, transient conditions were analyzed as the drawdown rates of the tailwater is different than the reservoir pool drawdown. The reservoir was conservatively modelled to initiate at maximum surcharge pool level. The tailwater head was estimated based on HEC-RAS analysis, with a maximum tailwater elevation of 940.0 ft.

Under transient conditions, the reservoir pool was modelled to drawdown from maximum surcharge (968.0 ft) pool elevation. Headwater and tailwater rates were estimated based on HEC-RAS modeling. The total time for drawdown of the tailwater was estimated to dissipate over a 39.5-hr period. Slope stability on the downstream slope was analyzed at the 39.5-hr period. As the estimated hydraulic conductivity of the soils are less than 10^{-4} cm/sec (2.83E-04 ft/day), undrained material properties are used in the analyses. However, for this condition, effective strength material properties were also analyzed for the embankment fill material and the more conservative factor of safety is presented.

Earthquake Loading

Earthquake loading conditions were analyzed based on USACE ER 1110-2-1806 Engineering and Design "Earthquake Design and Evaluation for Civil Works Projects" (2016). The earthquake loading case considers the stability of the embankments during potential seismic events by applying a pseudo-static (horizontal inertial force) coefficient to simulate earthquake loading. Vertical seismic coefficients have little impact on the resulting factors of safety and were ignored. For seismic analyses, drained and undrained conditions are often analyzed due to suspected partial drainage during seismic loading. It was determined that the undrained conditions (total stress parameters) controlled in this analysis.

A peak ground acceleration (PGA) was determined based on USACE ER 1110-2-1806 (2016). Erin Lake Dam is a High Hazard potential dam, which is a determining factor in PGA return period selection. For this site, a return period of 2475 years (2% in 50 years) was selected as there is potential for loss of life from failure at normal pool levels. A shear wave velocity of 260 m/sec was selected as lithography at Erin Lake Dam was estimated to be Class D “Medium dense sand or stiff clay” site classification from ASCE Standard 7-22 Minimum Design Loads and Associated Criteria for Buildings and Other Structures (2022). Based on the 2018 National Seismic Hazard Model for the conterminous United States (USGS), the peak horizontal acceleration was estimated to be 0.14. The peak horizontal acceleration is presented in **Attachment B**.

Slope Stability Analyses Results

Slope stability of the embankment slopes were analyzed for loading conditions detailed in USACE EM 1110-2-1902 (2003) and Georgia Rules and Regulations 391-3-8 Rules for Dam Safety (Rule 391-3-8-.09) guidelines. These results are presented in **Table 4**. The factors of safety for the analyzed existing embankment section are provided in **Attachment C**.

The results of the analyses show that the existing conditions at Erin Lake Dam do not meet the requirements for slope stability for maximum surcharge pool and Rapid Drawdown on the upstream slope.

Table 4: Existing Critical Slope Stability Factors of Safety

Analysis Condition	Required Minimum Factor of Safety (USACE)	Required Minimum Factor of Safety (GA RR)	Calculated Minimum Factor of Safety (Min depth 2.0 ft)
Steady State Seepage - Maximum Storage Pool (Normal Pool)	1.5	1.5	1.6
Steady State Seepage - Maximum Surcharge Pool	1.4	N/A	1.3*
Steady State Seepage with Earthquake Loading (Normal Pool)	N/A	1.1	1.0
Rapid Drawdown (Upstream)	1.3	1.3	1.0
Submerged Toe with Rapid Drawdown (Downstream)	N/A	1.3	1.3

*With maximum storage pool at approximate top of dam elevation (968.0 ft). H&H analysis shows dam will overtop during a 1/3 24-hr PMP event

Conclusions and Recommendations

The results of the analyses show the existing embankment does not meet the minimum required factor of safety required by Georgia Rules and Regulations 391-3-8 *Rules for Dam Safety* (Rule 391-3-8-.09) or USACE guidelines for upstream rapid drawdown and seismic analysis. In addition, H&H analysis show that the dam would slightly overtop during a 1/3 PMD precipitation event. Analysis with reservoir pool at the approximate top of dam shows the downstream slope would not meet the required minimum factor of safety at that pool elevation. Based on geotechnical investigation for Erin Lake Dam, proposed rehabilitation is recommended on the embankment to address existing deficiencies on the dam.

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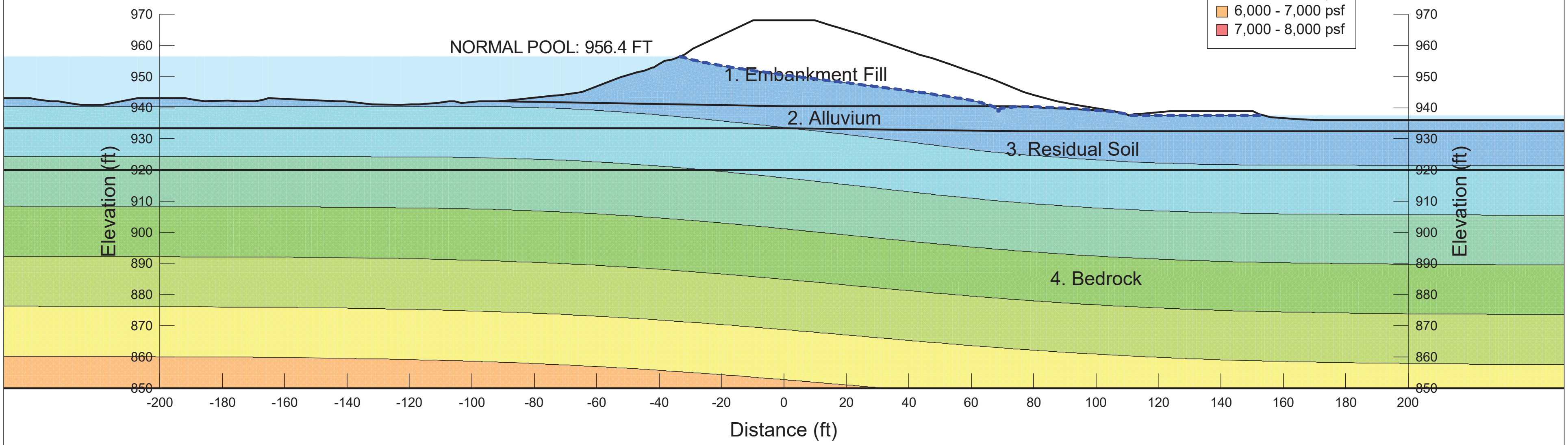
Attachment A: Existing Conditions Seepage Analysis Results

ERIN LAKE DAM
NORMAL POOL CONDITIONS
NORMAL POOL ELEVATION: 956.4 FT
TAILWATER ELEVATION: 937.51 FT
STEADY SEEPAGE

Name	**Sat Kv (cm/sec)**	**Kv'/Kh' Ratio**
1. Embankment Fill	2.70E-07	0.25
2. Alluvium	1.10E-06	0.67
3. Residual Soil	2.50E-06	0.5
4. Bedrock	1.00E-09	1

Water Pressure
- 🟦 0 - 1,000 psf
- 🟦 1,000 - 2,000 psf
- 🟩 2,000 - 3,000 psf
- 🟩 3,000 - 4,000 psf
- 🟩 4,000 - 5,000 psf
- 🟨 5,000 - 6,000 psf
- 🟧 6,000 - 7,000 psf
- 🟥 7,000 - 8,000 psf



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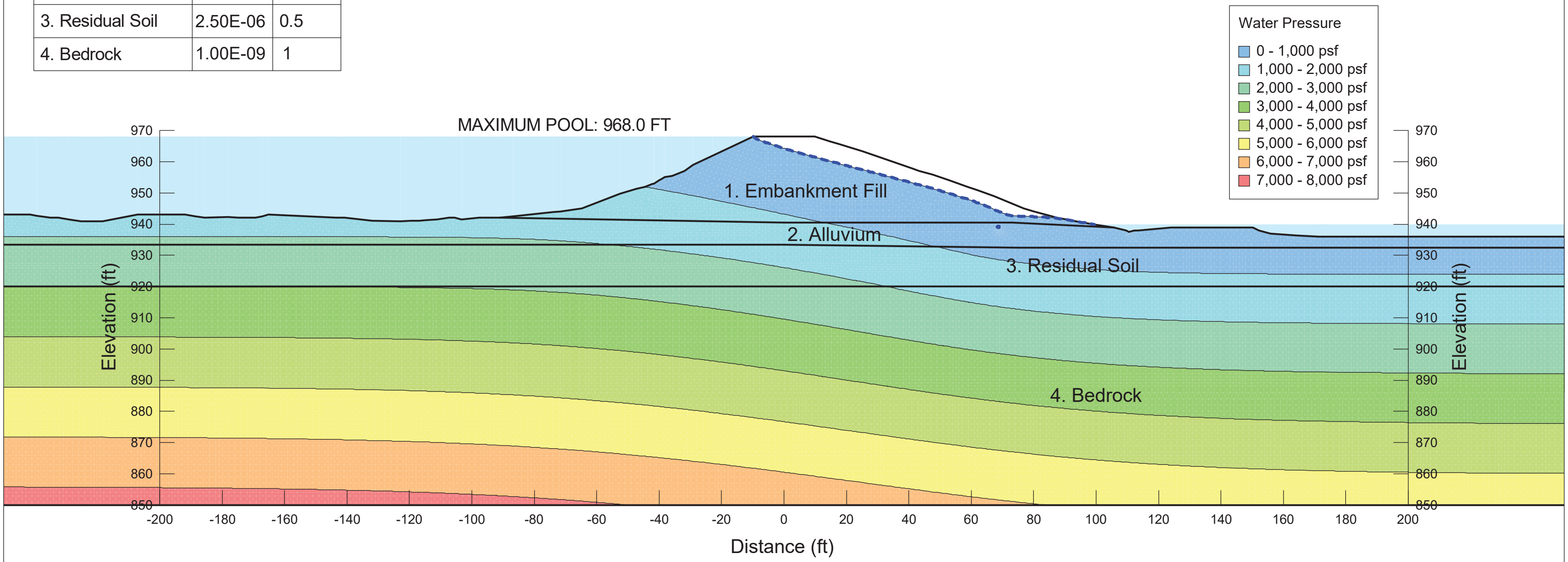
Tucker, DeKalb County, Georgia

Seepage Analysis - Existing Conditions

Date: May 2022	Project No.: 60656765	Checked By: NS

ERIN LAKE DAM
MAXIMUM POOL CONDITIONS
MAXIMUM POOL ELEVATION: 968.0 FT
TAILWATER ELEVATION: 940.0 FT
STEADY SEEPAGE

Name	**Sat Kv (cm/sec)**	**Kv'/Kh' Ratio**
1. Embankment Fill	2.70E-07	0.25
2. Alluvium	1.10E-06	0.67
3. Residual Soil	2.50E-06	0.5
4. Bedrock	1.00E-09	1



***MAXIMUM POOL OVERTOPS DAM.
ASSUMED TOP OF DAM FOR
RESERVOIR ELEVATION TO USE
FOR REFERENCE ONLY***

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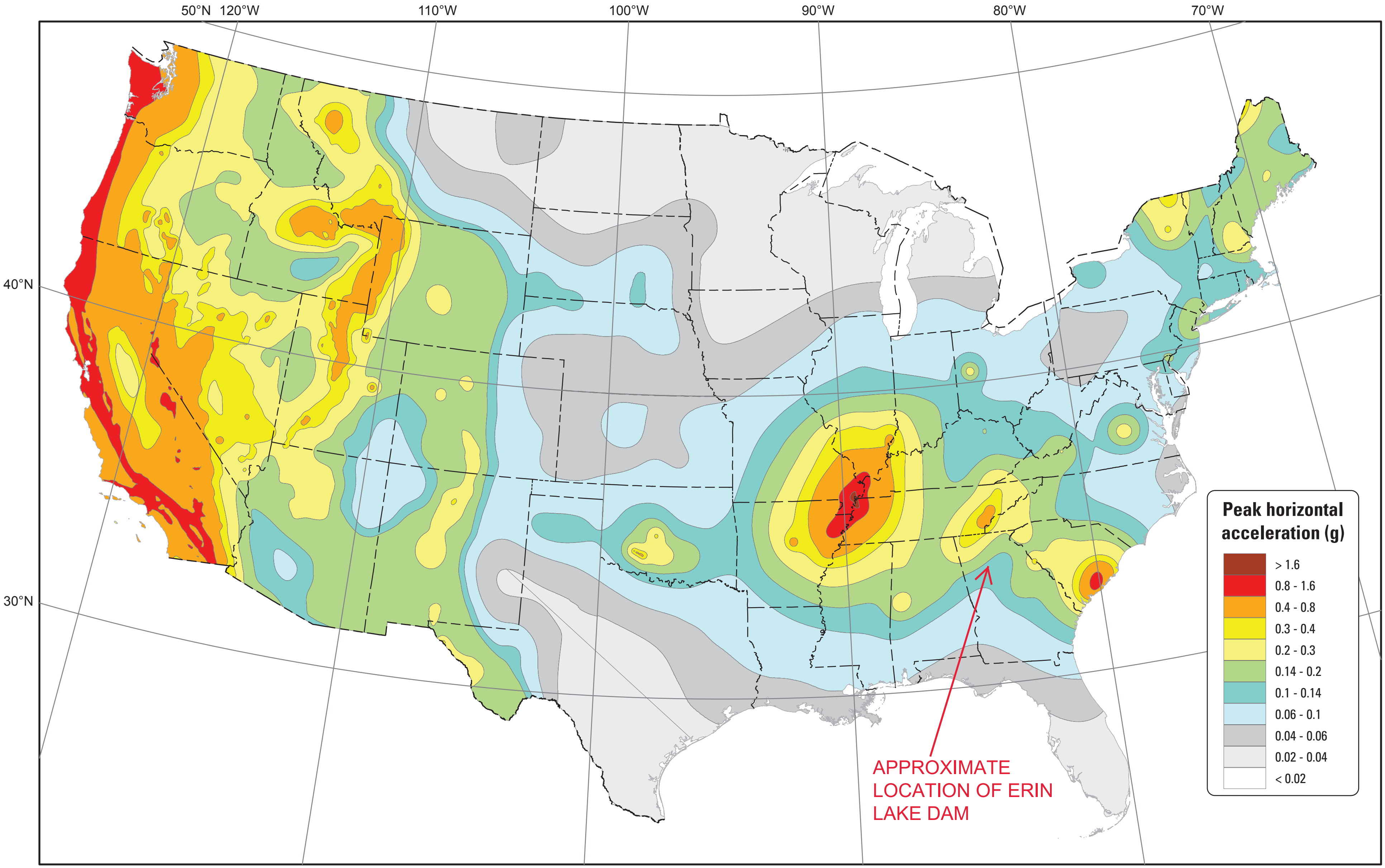
Seepage Analysis - Existing Conditions

Date: May 2022	Project No.: 60656765	Checked By: NS

Attachment B: 2018 National Seismic Hazard Model Peak Ground Acceleration



APPROXIMATE
LOCATION OF ERIN
LAKE DAM

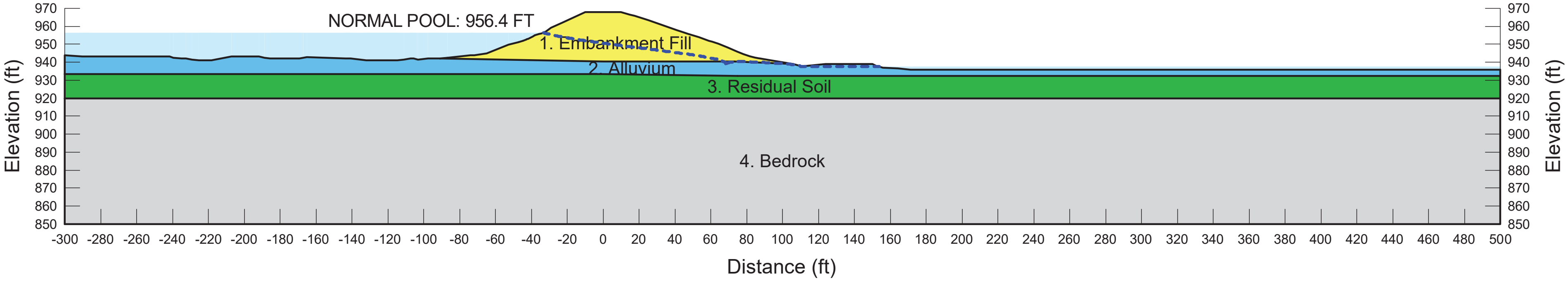
2018 National Seismic Hazard Model for the conterminous United States
Peak horizontal acceleration
with a 2% probability of exceedance in 50 years
NEHRP site class D (V_{s30} = 260 m/s)

REFERENCE: https://www.sciencebase.gov/catalog/item/5d5597d0e4b01d82ce8e3ff1 (accessed May 2022)

Attachment C: Existing Conditions Slope Stability Analysis Results

ERIN LAKE DAM
NORMAL POOL CONDITIONS
NORMAL POOL ELEVATION: 956.4 FT
TAILWATER ELEVATION: 937.51 FT
STEADY SEEPAGE
OVERALL MODEL

Color	Name
🟨	1. Embankment Fill
🟦	2. Alluvium
🟩	3. Residual Soil
⬜	4. Bedrock



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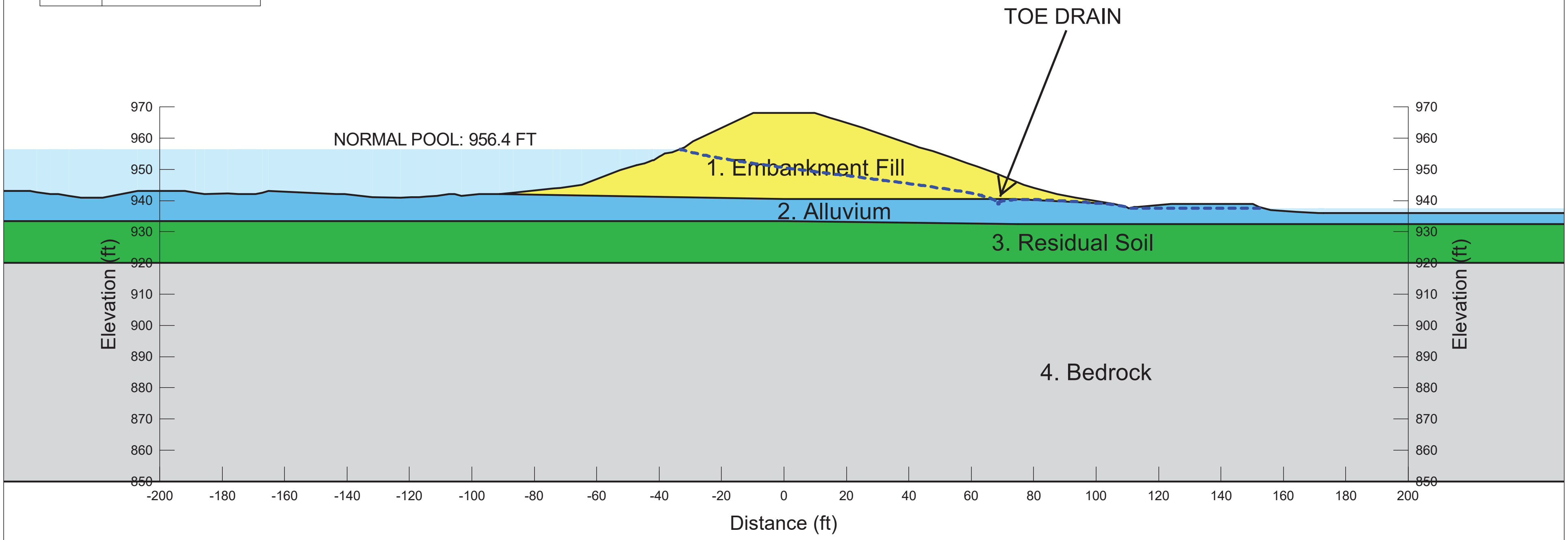
Erin Lake Dam Project
Tucker, DeKalb County, Georgia

Slope Stability - Existing Conditions

Date: May 2022	Project No.: 60656765	Checked By: NS

ERIN LAKE DAM
NORMAL POOL CONDITIONS
NORMAL POOL ELEVATION: 956.4 FT
TAILWATER ELEVATION: 937.51 FT
STEADY SEEPAGE
TOE DRAIN LOCATION

Color	Name
🟨	1. Embankment Fill
🟦	2. Alluvium
🟩	3. Residual Soil
⬜	4. Bedrock



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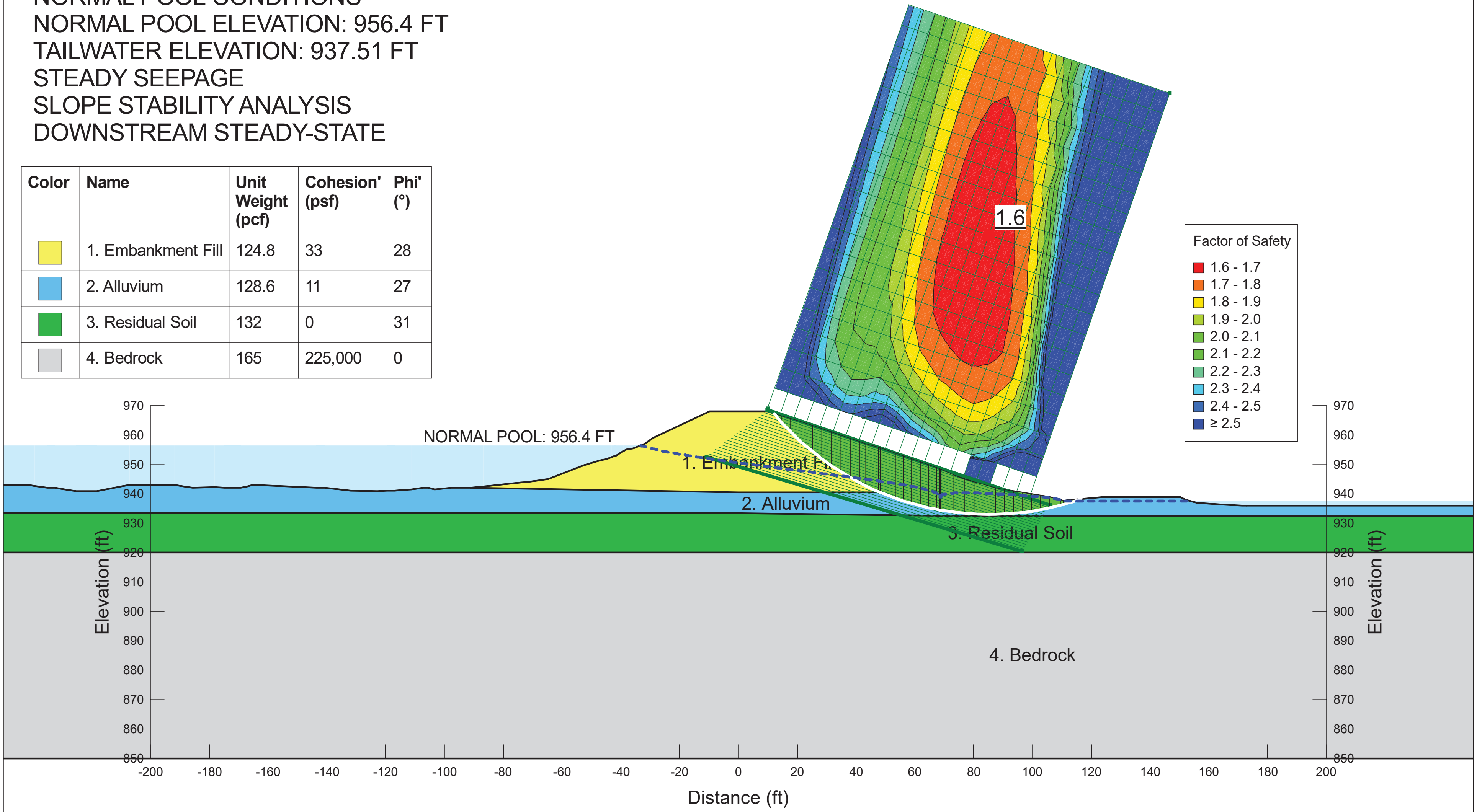
Slope Stability - Existing Conditions

Date: May 2022	Project No.: 60656765	Checked By: NS

ERIN LAKE DAM
NORMAL POOL CONDITIONS
NORMAL POOL ELEVATION: 956.4 FT
TAILWATER ELEVATION: 937.51 FT
STEADY SEEPAGE
SLOPE STABILITY ANALYSIS
DOWNSTREAM STEADY-STATE

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟨	1. Embankment Fill	124.8	33	28
🟦	2. Alluvium	128.6	11	27
🟩	3. Residual Soil	132	0	31
⬜	4. Bedrock	165	225,000	0



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Erin Lake Dam Project
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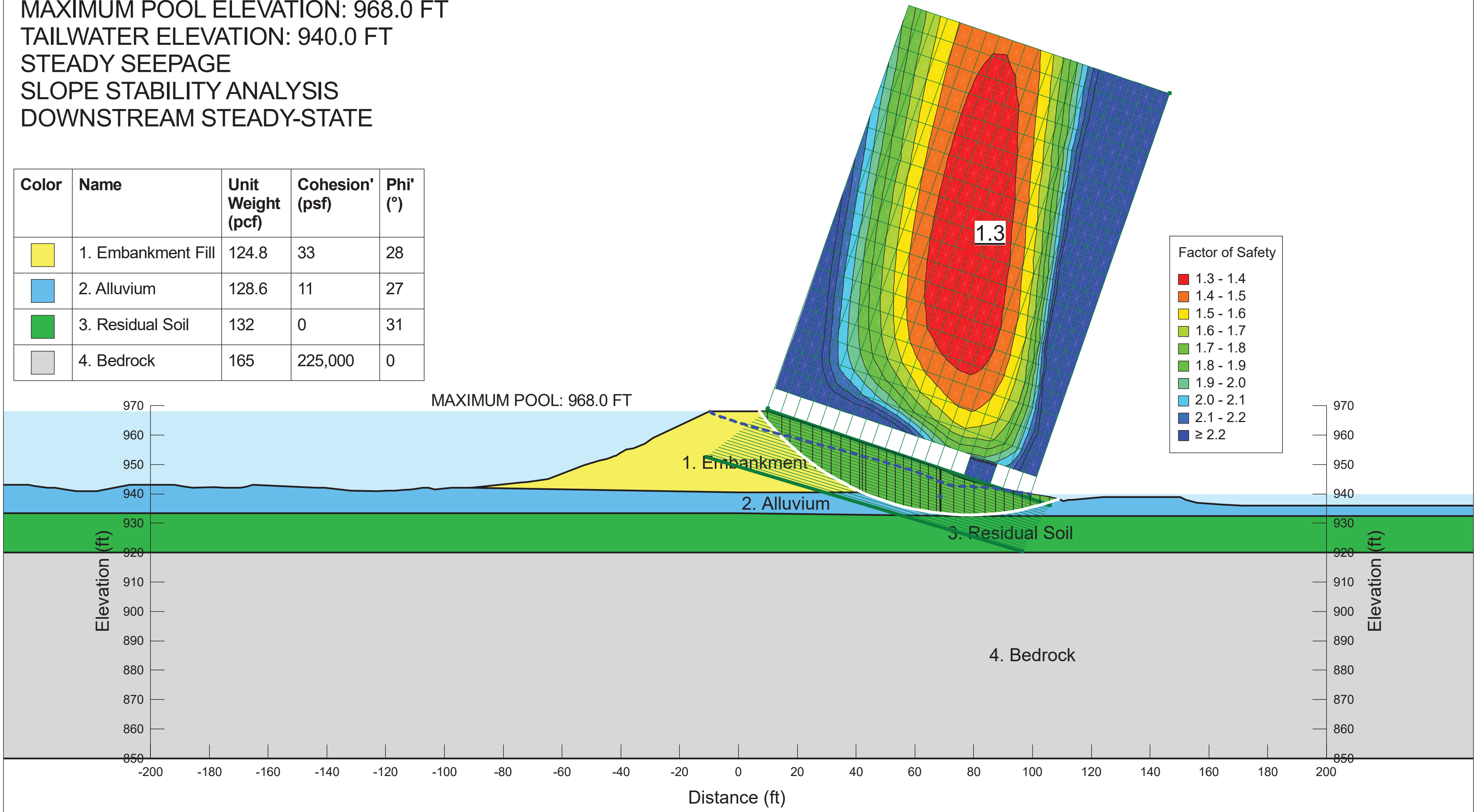
Slope Stability - Existing Conditions

Date: May 2022	Project No.: 60656765	Checked By: NS

ERIN LAKE DAM
MAXIMUM POOL CONDITIONS
MAXIMUM POOL ELEVATION: 968.0 FT
TAILWATER ELEVATION: 940.0 FT
STEADY SEEPAGE
SLOPE STABILITY ANALYSIS
DOWNSTREAM STEADY-STATE

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟨	1. Embankment Fill	124.8	33	28
🟦	2. Alluvium	128.6	11	27
🟩	3. Residual Soil	132	0	31
⬜	4. Bedrock	165	225,000	0



MAXIMUM POOL MODELLED TO TOP OF DAM

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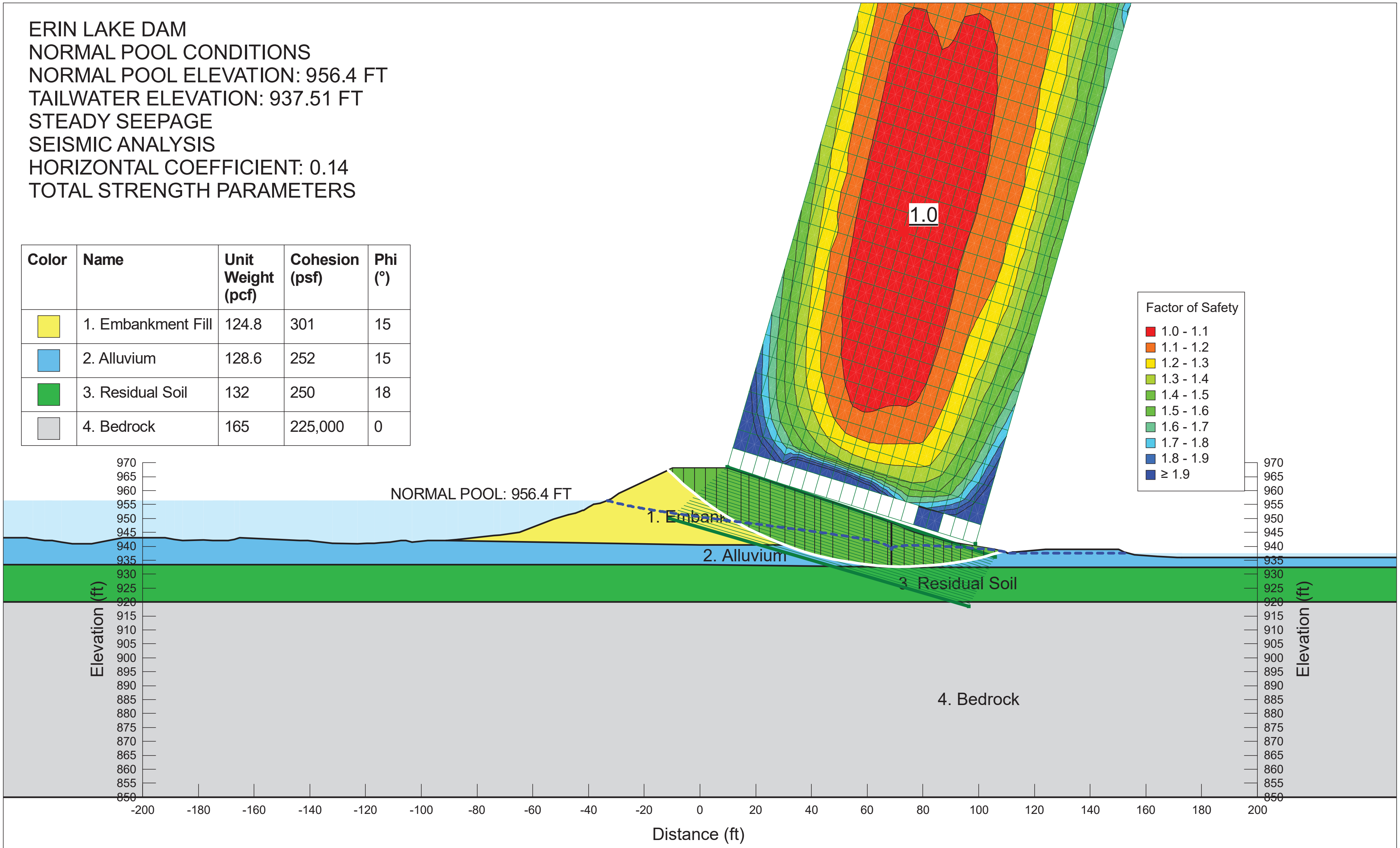
Erin Lake Dam Project
Tucker, DeKalb County, Georgia

Slope Stability - Existing Conditions

Date: May 2022	Project No.: 60656765	Checked By: NS

ERIN LAKE DAM
NORMAL POOL CONDITIONS
NORMAL POOL ELEVATION: 956.4 FT
TAILWATER ELEVATION: 937.51 FT
STEADY SEEPAGE
SEISMIC ANALYSIS
HORIZONTAL COEFFICIENT: 0.14
TOTAL STRENGTH PARAMETERS

Color	Name	Unit Weight (pcf)	Cohesion (psf)	Phi (°)
🟨	1. Embankment Fill	124.8	301	15
🟦	2. Alluvium	128.6	252	15
🟩	3. Residual Soil	132	250	18
⬜	4. Bedrock	165	225,000	0



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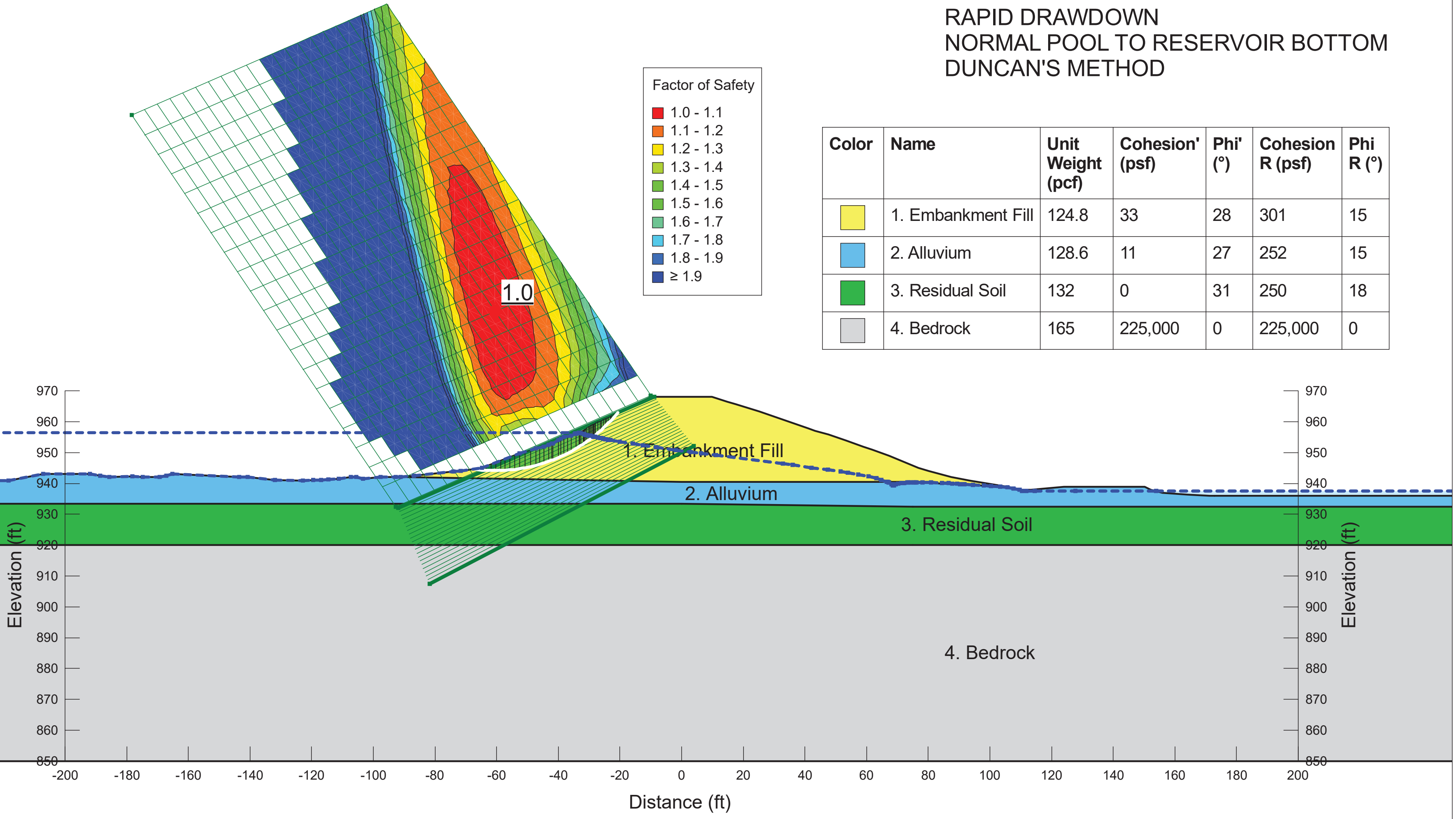
Erin Lake Dam Project
Tucker, DeKalb County, Georgia

Slope Stability - Existing Conditions

Date: May 2022	Project No.: 60656765	Checked By: NS

ERIN LAKE DAM
NORMAL POOL ELEVATION: 956.4 FT
TAILWATER ELEVATION: 937.51 FT
RAPID DRAWDOWN
NORMAL POOL TO RESERVOIR BOTTOM
DUNCAN'S METHOD

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Cohesion R (psf)	Phi R (°)
🟨	1. Embankment Fill	124.8	33	28	301	15
🟦	2. Alluvium	128.6	11	27	252	15
🟩	3. Residual Soil	132	0	31	250	18
⬜	4. Bedrock	165	225,000	0	225,000	0



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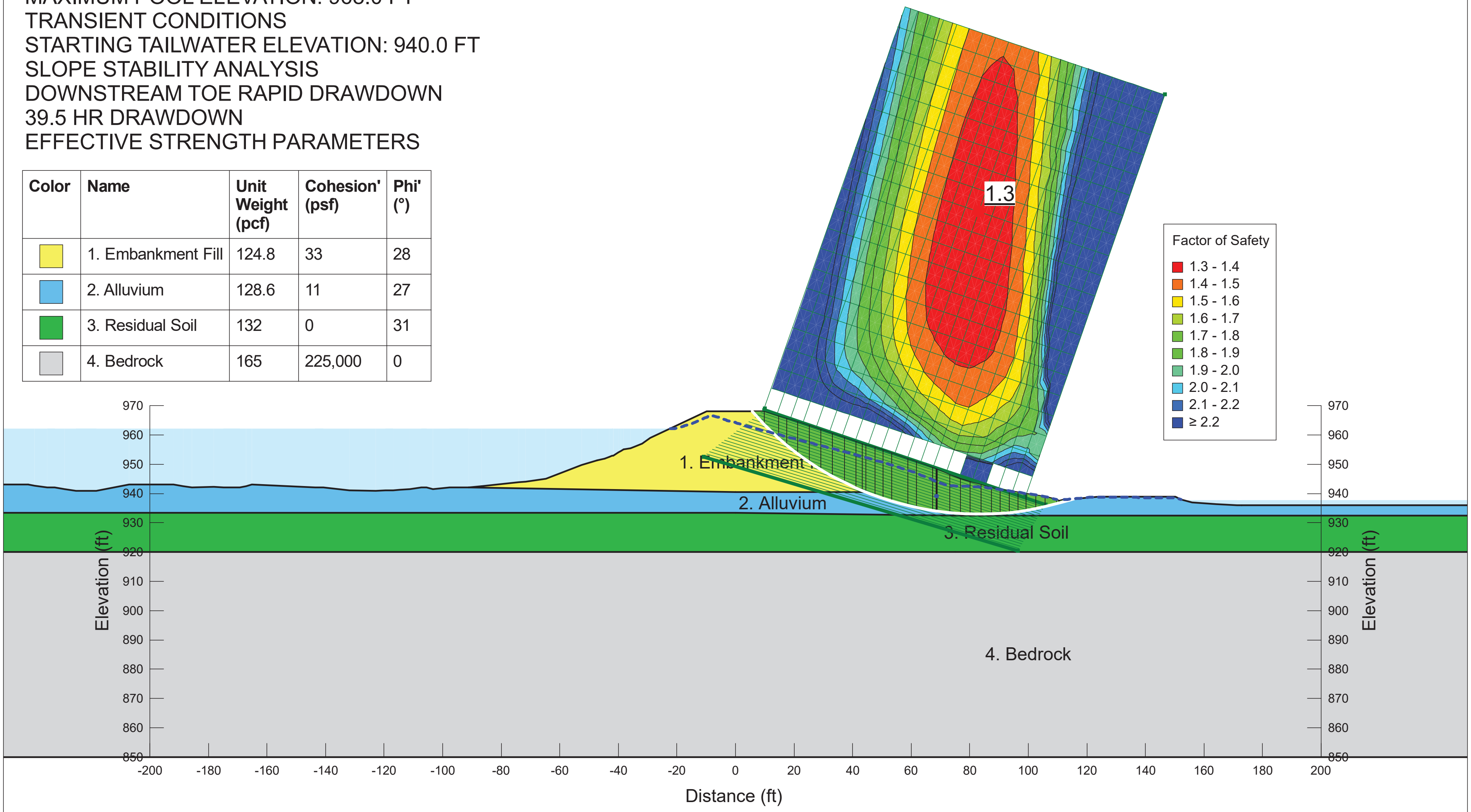
Erin Lake Dam Project
Tucker, DeKalb County, Georgia

Slope Stability - Existing Conditions

Date: May 2022
Project No.: 60656765
Checked By: NS

ERIN LAKE DAM
MAXIMUM POOL CONDITIONS
MAXIMUM POOL ELEVATION: 968.0 FT
TRANSIENT CONDITIONS
STARTING TAILWATER ELEVATION: 940.0 FT
SLOPE STABILITY ANALYSIS
DOWNSTREAM TOE RAPID DRAWDOWN
39.5 HR DRAWDOWN
EFFECTIVE STRENGTH PARAMETERS

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟨	1. Embankment Fill	124.8	33	28
🟦	2. Alluvium	128.6	11	27
🟩	3. Residual Soil	132	0	31
⬜	4. Bedrock	165	225,000	0



***MAXIMUM POOL MODELED TO
TOP OF DAM***

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Erin Lake Dam Project
Tucker, DeKalb County, Georgia

Slope Stability - Existing Conditions

Date: May 2022	Project No.: 60656765	Checked By: NS

Appendix B – Geotechnical Analysis Calculations

Filter Diaphragm Design

Project: Erin Lake Dam Originator: TKW Date: 6/18/2024
Reviewed by: LD Date: 6/19/2024

CALCULATION PACKAGE: Filter Diaphragm Sizing

Erin Lake Dam Filter Diaphragm Seepage Analysis Calculations

Objective: To determine the appropriate sizing of a filter diaphragm around the Principal Spillway Conduit (PSW) and strip drain outlet.

Step 1: Determine Minimum Size of Filter Diaphragm (From NRCS NEH Part 628 Dams: Chapter 45 Filter Diaphragms, 2007)

Horizontal Extents: 3 times the horizontal dimension of the box conduit (NRCS NEH Chapter 45, Filter Diaphragms, Figure 45A-2, 2007)

Vertical Upward extents: 3 times the vertical dimension of the box conduit (NRCS NEH Chapter 45, Filter Diaphragms, Figure 45A-4, 2007)

Vertical Extents below the conduit: Based on foundation type. Assume settlement ratio of 0.7 for ordinary soil. Extents are the greater of 2 feet or 1-foot below the bottom of the trench excavation made to install the conduit. (NRCS NEH Chapter 45, Filter Diaphragms, Figure 45A-6, 2007)

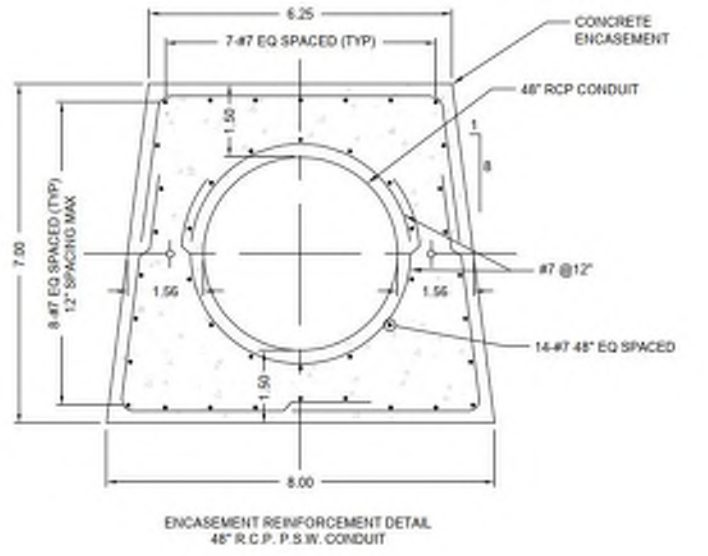


Figure 1: PSW Conduit Dimensions

The base of the PSW conduit at EL 938 ft at design location of filter diaphragm.

Vertical Height of the conduit: 7 feet

The top of the conduit is 945 ft (938 ft + 7 feet).

Project: Erin Lake Dam Originator: TKW Date: 6/18/2024
Reviewed by: LD Date: 6/19/2024

CALCULATION PACKAGE: Filter Diaphragm Sizing

Minimum horizontal and vertical upward extents: 7 ft x 3 = 21 feet. For conservativism, extend diaphragm vertically to 968 ft (2 feet below embankment crest)

$$968\,ft - 945\,ft = 23\,ft$$

Minimum vertical extent below the conduit: 2 feet

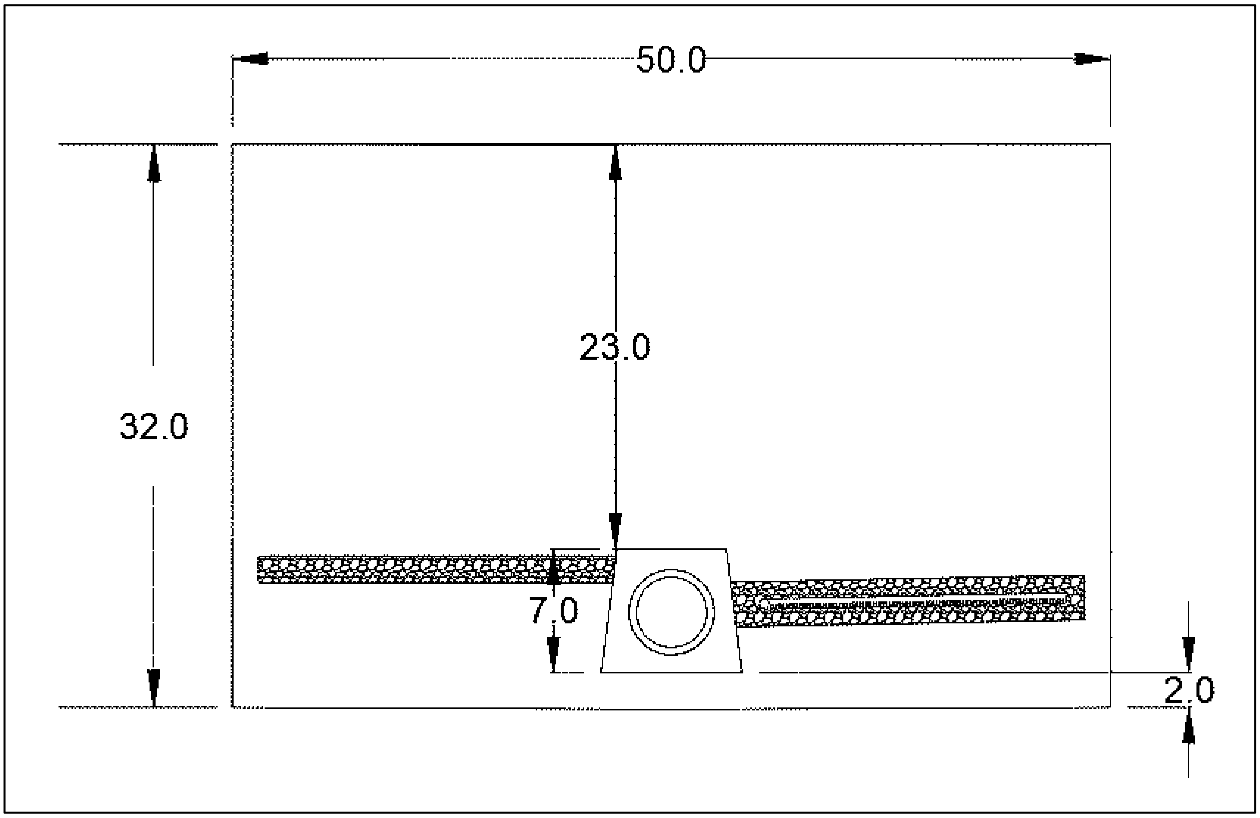


Figure 2: Filter Diaphragm Dimensions

Minimum Design Thickness: 3.0 ft. (NRCS NEH Chapter 45, Filter Diaphragms, Figure 45A-11, 2007). At base of filter diaphragm where coarse filter is applied, filter minimum thickness is 2.0 feet ((NRCS NEH Chapter 45, Filter Diaphragms, Figure 45A-12, 2007).

Use method described in NRCS NEH Chapter 45, Appendix C, to calculate the design flow:

Area Contributing Flow: Area of diaphragm

Area of filter diaphragm extents minus area of conduit

CALCULATION PACKAGE: Filter Diaphragm Sizing

$$A_{fd} = (23\ ft + 7\ ft + 2\ ft)x(8\ ft + 21\ ft + 21\ ft) - (6.25\ ft * 7\ ft + 0.5 * .875\ ft * 7\ ft * 2) =$$

$$A_{fd} = 32\ ft * 50\ ft - 49.875\ ft^2 = \ 1550\ ft^2$$

Step 2: Compute Hydraulic Gradient

Analyzed based on Maximum Pool reservoir level (969 ft). Assume change in head is to approximate elevation 959 ft (10 ft head loss) based on Geostudio Analysis.

Area of filter diaphragm below 960 ft =

$$A_{fd} = 1550\ ft^2 - 10\ ft * 50\ ft = 1050\ ft^2$$

Step 2: Compute Hydraulic Gradient

Distance from upstream toe: 100.06 ft

Change in head from maximum pool to top of filter diaphragm: 10 ft.

$i = \frac{10\ ft}{100.06\ ft} = 0.1$

Step 3: Assume Embankment Fill hydraulic conductivity is 100 times higher than estimated to provide a safety factor for uncertainties involved as recommended in NRCS (NRCS NEH Chapter 45, Filter Diaphragms, Page 45C-1, 2007).

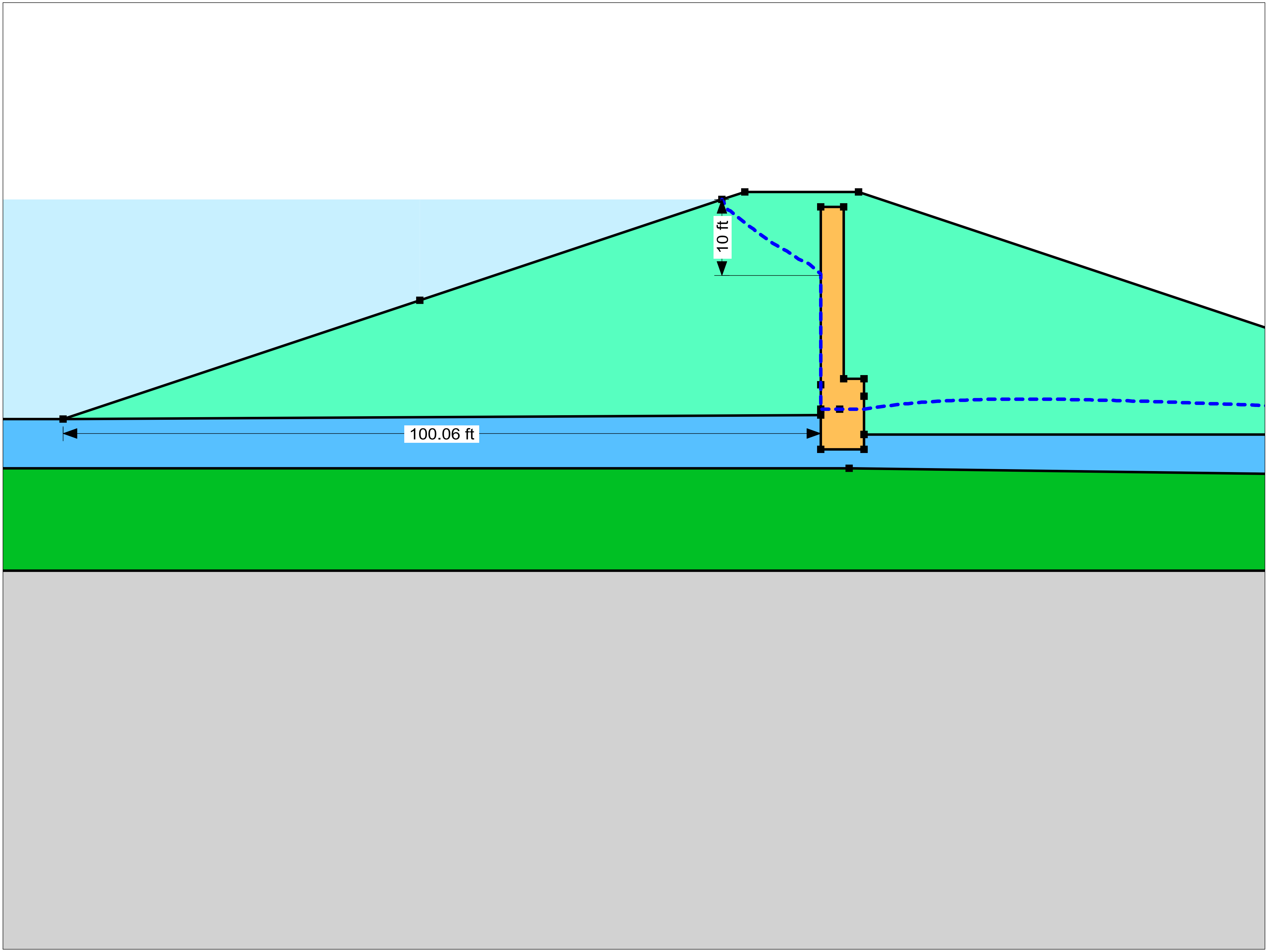
3.06E-03 ft/day x 100 = 0.31 ft/day

Step 4: Compute Q based on Darcy's equation:

Q = KiA = .31 ft/day x 0.1 ft/ft x 1050 ft^2 = 33 ft^3/day

Assume two times the flow to account for potential flow from outside the diaphragm.

Filter Diaphragm drain pipe required to transport **66 ft^3/day**.


10 ft
100.06 ft

Outlet Strip Drain (For Filter Diaphragm) Based on NRCS Chapter 45		
Required flow capacity (Using Fs = 100)	6.60E+01	ft^3/day
ASTM #8 Coarse Aggregate	6.25E+00	cm/sec
ASTM #8 Coarse Aggregate	1.77E+04	ft/day

Δh, ft	i	A (ft^2)	d (ft)	y_d
0.2	0.002778	1.34E+00	4.47E-01	0.65
0.4	0.005556	6.71E-01	2.24E-01	0.62
0.6	0.008333	4.47E-01	1.49E-01	0.75
0.8	0.011111	3.35E-01	1.12E-01	0.91
1	0.013889	2.68E-01	8.94E-02	1.09
1.2	0.016667	2.24E-01	7.45E-02	1.27
1.4	0.019444	1.92E-01	6.39E-02	1.46
1.6	0.022222	1.68E-01	5.59E-02	1.66
2	0.027778	1.34E-01	4.47E-02	2.04
2.4	0.033333	1.12E-01	3.73E-02	2.44
2.6	0.036111	1.03E-01	3.44E-02	2.634388

Length of pipe to outlet is 72 ft
Design Base Elevation of Gravel strip drain at Filter Diaphragm: 942.9 ft
Design outlet elevation of strip drain: 942.2 ft
Assume base width of gravel section of strip drain is 3 ft
Change in head is limited to <1 ft due to strip drain outlet.

Based on analysis, the minimum depth required is 0.62 ft. Designed coarse aggregate section of strip drain 1.5 ft depth

Project: Lake Erin Dam

Originator: TKW

Date: 5/8/2024

Reviewed by: LD

Date:5/9/2024

CALCULATION PACKAGE: Drain Pipe Flow Calculations: Manning's Equation

6 inch, HDPE Pipe Flow Calculation-Filter Diaphragm

OBJECTIVE:

Determine the maximum flow rate and flow velocity for a 6-inch HDPE drain pipe to ensure it meets minimum flow requirements.

CALCULATION:

Manning's Equation (SI units) $Q = \frac{1.49}{n} A R^{2/3} S_0^{1/2}$

For Velocity $V = \frac{1.49}{n} R^{2/3} S_0^{1/2}$

$$Area\ (A) = \frac{1}{8}\left(2\pi - \frac{\theta\pi}{180} + \sin\theta\right) d_i^2$$

$$Wetted\ Perimeter = \frac{\pi d_i (360 - \theta)}{360}$$

$$Hydraulic\ Radius\ (R) = \frac{A}{P}$$

$$Top\ Width\ (B) = \left[\sin\left(\frac{\theta}{2}\right)\right] d_i$$

Outside diameter of 6-inch PE pipe (per ATSM F714) equals 6.9 inches. The minimum wall thickness of 0.511 inches. Assume interior HDPE pipe diameter equals 5.878 inches.

Assume head in pipe 75% of diameter (4.4 inches)

Ө = 120 degrees (degrees)

d_i =5.878 inches (0.49 feet)

$$Area\ (A) = \frac{1}{8}\left(2\pi - \frac{120\pi}{180} + \sin 120\right)(0.49\ ft)^2$$

$$Wetted\ Perimeter\ (P) = \frac{\pi (0.49\ ft)(360 - 120)}{360}$$

Project: Lake Erin Dam

Originator: TKW

Date: 5/8/2024

Reviewed by: LD

Date: 5/9/2024

CALCULATION PACKAGE: Drain Pipe Flow Calculations: Manning’s Equation

$$Hydraulic\ Radius\ (R) = \frac{A}{P}$$

$$Top\ Width\ (B) = \left[\sin\frac{120}{2}\right] 0.49\ feet$$

Area = 0.15 ft^2

Wetted Perimeter = 1.02 ft

Hydraulic Radius = 0.15 ft

Top Width = 0.42 ft

Slope (S)= 0.01 ft/ft (1%)

Manning’s Coefficient (n)= 0.012

$$Q = \frac{1.49}{0.012}(0.15\ ft^2)(0.15\ ft)^{2/3}(0.01\frac{ft}{ft})^{1/2}$$

Q_{max} = 0.53 ft^3/sec

$$V = \frac{1.49}{0.012}(0.15\ ft)^{2/3}(0.01\frac{ft}{ft})^{1/2}$$

V_{max}= 3.5 ft/sec

0.53 ft^3/sec = 45792 ft^3/day

From filter diaphragm design with Fs of 100, required flow rate is **66 ft^3/day**

Okay

TABLE 2 Outside Diameters and Tolerances-DIPS Sizing System

Nominal Size	Outside Diameter, in (mm)	Minimum Outside Diameter, in. (mm)	Maximum Outside Diameter, in. (mm)
3	3.960 (100.58)	3.942 (100.13)	3.976 (100.99)
4	4.800 (121.92)	4.778 (121.37)	4.822 (122.48)
6	6.900 (175.26)	6.869 (174.47)	6.931 (176.05)
8	9.050 (229.87)	9.009 (228.84)	9.091 (230.91)
10	11.100 (281.94)	11.050 (280.67)	11.150 (283.21)
12	13.200 (335.28)	13.141 (333.77)	13.259 (336.78)
14	15.300 (388.62)	15.231 (386.87)	15.369 (390.37)
16	17.400 (441.96)	17.322 (439.97)	17.478 (443.94)
18	19.500 (495.30)	19.412 (493.07)	19.588 (497.54)
20	21.600 (548.64)	21.503 (546.17)	21.697 (551.10)
24	25.800 (655.32)	25.684 (652.37)	25.916 (658.27)
30	32.000 (815.80)	31.856 (809.14)	32.144 (816.46)
36	38.300 (972.82)	38.128 (968.44)	38.472 (977.19)
42	44.500 (1130.30)	44.300 (1125.21)	44.700 (1135.38)
48	50.800 (1290.32)	50.571 (1284.51)	51.029 (1296.14)
54	57.560 (1462.3)	57.301 (1455.72)	57.819 (1468.88)
60	61.610 (1564.9)	61.333(1557.86)	61.887 (1571.94)

PE2708 or PE3608 or PE4608 or PE4710, and shall meet Table 1 requirements for PE2708 or PE3608 or PE4608 or PE4710, and shall meet thermal stability, brittleness temperature and elongation at break requirements in accordance with Specification D3350.

4.1.1 *Color and Ultraviolet (UV) Stabilization*—Per Table 1, polyethylene compounds shall meet Specification D3350 code C or E. In addition, Code C polyethylene compounds shall have 2 to 3 percent carbon black, and Code E polyethylene compounds shall have sufficient UV stabilizer to protect pipe from deleterious UV exposure effects during unprotected outdoor shipping and storage for at least eighteen (18) months.

4.1.2 *Colors for solid color, a color shell layer, or color stripes used to identify pipe service or pipe DR*—In accordance with the APWA Uniform Color Code, blue shall identify potable water service; green shall identify sewer service; purple (lavender) shall identify reclaimed water service. Yellow that identifies gas service shall not be used. Colors used to identify DR may be used in accordance with end user specifications.

4.2 *Health Effects Requirements*—Products intended for contact with potable water or when otherwise required, shall be certified for conformance with NSF/ANSI/CAN Standard No. 61 or the health effects portion of NSF/ANSI Standard No. 14 by an acceptable certifying organization.

4.3 *Oxidative Resistance*—For pipe that is intended for use in the transport of potable water containing disinfectants, or where required by the application, customer or regulatory authority having jurisdiction, the PE compound shall have an oxidative resistance classification of CC2 or CC3 in accordance with Specification D3350.

NOTE 4—See PPI TN-44 or www.plasticpipe.org for further information on potable water disinfectants.

4.4 *Rework Material*—Clean polyethylene compound from the manufacturer's own pipe production that met 4.1 through 4.3 as new compound is suitable for reextrusion into pipe, when blended with new compound of the same thermoplastic pipe material designation code and the same or greater oxidative resistance classification. Pipe containing rework material shall meet the requirements of this specification.

5. Requirements

5.1 *Workmanship*—The pipe shall be homogeneous throughout and essentially uniform in color, opacity, density, and other properties. The inside and outside surfaces shall be semimatte or glossy in appearance (depending on the PE compound) and free of chalking, sticky, or tacky material. The surfaces shall be free of excessive bloom, that is, slight bloom is acceptable. The pipe walls shall be free of cracks, holes, blisters, voids, foreign inclusion, or other defects that are visible to the naked eye and that may affect the wall integrity. Holes deliberately placed in perforated pipe are acceptable. Bloom or chalking may develop in pipe exposed to direct rays of the sun (ultraviolet radiant energy) for extended periods and, consequently, these requirements do not apply to pipe after extended exposure to direct rays of the sun.

5.2 *Dimensions and Tolerances:*

5.2.1 *Outside Diameters*—These shall be in accordance with Table 2, Table 4, or Table 6 when measured in accordance with Test Method D2122 at any point not closer than 300 mm (11.8 in.) to the cut end of a length of pipe. Conditioning to standard temperature without regard to relative humidity is required.

5.2.2 *Wall Thicknesses*—The minimum thicknesses shall be in accordance with Table 3, Table 5, or Table 7 when measured in accordance with Test Method D2122. Conditioning to standard temperature without regard to relative humidity is required.

5.2.3 *Eccentricity*—The wall thickness variability as measured and calculated in accordance with Test Method D2122 in any diametrical cross section of the pipe shall not exceed 12 %.

5.2.4 *Toe-In*—When measured in accordance with 5.2.1, the outside diameter at the cut end of the pipe shall not be more than 1.5 % smaller than the undistorted outside diameter. Measurement of the undistorted outside diameter shall be made no closer than 1.5 pipe diameters or 11.8 in. (300 mm), whichever distance is less, from the cut end of the pipe. Undistorted outside diameter shall meet specifications in Table 2, Table 4, or Table 6.

5.2.5 *Special Sizes*—Where existing system conditions or special local requirements make other diameters or dimension ratios necessary, other sizes or dimension ratios, or both, shall be acceptable for engineered applications when mutually agreed upon by the customer and the manufacturer, if the pipe is manufactured from plastic compounds meeting the material requirements of this specification, and the strength and design requirements are calculated on the same basis as those used in this specification. For diameters not shown in Table 2, Table 4, or Table 6, the tolerance shall be the same percentage as that used in the corresponding table for the next smaller listed size. Minimum wall thicknesses for DRs not shown in Table 3, Table 5, or Table 7 or shall be determined by dividing the average outside diameter by the DR and rounding to three decimal places for inch sized pipes or two decimal places for metric sized pipes, and the tolerance shall comply with 5.2.3.

TABLE 3 Minimum Wall Thickness DIPS Sizing System, in.

		PE4710^A								PE3608^A			
		PR100^B	PR125^B	PR160^B	PR200^B	PR250^B	PR335^B	PR100^B	PR150^B	PR200^B	PR250^B	PR300^B	PR350^B
Nominal Size	Outside Diameter in. (mm)^C	100 psi (690 kPa)^D	125 psi (860 kPa)^D	160 psi (1100 kPa)^D	200 psi (1380 kPa)^D	250 psi (1725 kPa)^D	335 psi (2310 kPa)^D	100 psi (690 kPa)^D	150 psi (1035 kPa)^D	200 psi (1380 kPa)^D	250 psi (1725 kPa)^D	300 psi (2070 kPa)^D	350 psi (2415 kPa)^D
		DR 21	DR 17	DR 13.5	DR 11	DR 9	DR 7	DR 17	DR 11.7	DR 9	DR 7.4	DR 6.3	DR 5.6
3	3.960 (100.58)	0.189 (4.80)	0.233 (5.92)	0.293 (7.53)	0.360 (9.14)	0.440 (11.18)	0.605 (14.00)	0.233 (5.92)	0.338 (8.59)	0.440 (11.18)	0.535 (13.59)	0.629 (15.97)	0.707 (17.96)
4	4.800 (121.92)	0.229 (5.82)	0.282 (7.16)	0.356 (9.04)	0.436 (11.07)	0.533 (13.54)	0.686 (17.42)	0.282 (7.16)	0.410 (10.41)	0.533 (13.54)	0.649 (16.48)	0.762 (19.35)	0.857 (21.77)
6	6.900 (175.26)	0.329 (8.36)	0.406 (10.31)	0.511 (12.98)	0.627 (15.93)	0.767 (19.48)	0.986 (25.04)	0.406 (10.31)	0.590 (14.99)	0.767 (19.48)	0.932 (23.67)	1.095 (27.82)	1.232 (31.30)
8	9.050 (229.87)	0.431 (10.95)	0.532 (13.51)	0.670 (17.02)	0.823 (20.90)	1.006 (25.55)	1.293 (32.84)	0.532 (13.51)	0.774 (19.66)	1.006 (25.55)	1.223 (31.06)	1.437 (36.49)	1.616 (41.05)
10	11.100 (281.94)	0.529 (13.44)	0.653 (16.59)	0.978 (24.84)	1.009 (25.63)	1.233 (31.32)	1.586 (40.28)	0.653 (16.59)	0.949 (24.10)	1.233 (31.32)	1.500 (38.10)	1.762 (44.75)	1.982 (50.35)
12	13.200 (335.28)	0.629 (15.98)	0.776 (19.71)	0.978 (24.84)	1.200 (30.48)	1.467 (37.26)	1.886 (47.90)	0.776 (19.71)	1.128 (28.65)	1.467 (37.26)	1.784 (45.31)	2.095 (53.22)	2.357 (59.87)
14	15.300 (388.62)	0.729 (18.52)	0.900 (22.86)	1.133 (28.78)	1.391 (35.33)	1.700 (43.18)	2.186 (55.52)	0.900 (22.86)	1.308 (33.22)	1.700 (43.18)	2.068 (52.53)	2.429 (61.69)	2.732 (69.40)
16	17.400 (441.96)	0.829 (21.06)	1.024 (26.01)	1.289 (32.74)	1.582 (39.67)	1.933 (49.10)	2.486 (63.14)	1.024 (26.01)	1.487 (37.77)	1.933 (49.10)	2.351 (59.72)	2.762 (70.15)	3.107 (78.92)
18	19.500 (495.30)	0.929 (23.60)	1.147 (29.13)	1.444 (36.68)	1.773 (45.03)	2.167 (55.04)	2.789 (70.76)	1.147 (29.13)	1.667 (42.34)	2.167 (55.04)	2.635 (66.93)	3.095 (78.62)	3.482 (88.45)
20	21.600 (548.64)	1.029 (26.14)	1.271 (32.28)	1.600 (40.64)	1.964 (49.89)	2.400 (60.96)	3.086 (78.38)	1.271 (32.28)	1.846 (46.89)	2.400 (60.96)	2.919 (74.14)	3.429 (87.09)	...
24	25.800 (655.32)	1.229 (31.22)	1.518 (38.56)	1.911 (48.54)	2.345 (59.56)	2.867 (72.82)	3.686 (93.62)	1.518 (38.56)	2.205 (56.01)	2.867 (72.82)	3.486 (88.54)	...	...
30	32.000 (815.80)	1.524 (38.71)	1.882 (47.80)	2.370 (60.20)	2.909 (73.89)	3.556 (90.32)	...	1.882 (47.80)	2.735 (69.47)	3.556 (90.32)	...	...	...
36	38.300 (972.82)	1.824 (46.33)	2.253 (57.23)	2.837 (72.06)	3.482 (88.44)	...	...	2.253 (57.23)	3.274 (83.16)	...	...	...	...
42	44.500 (1130.30)	2.119 (53.82)	2.618 (66.50)	3.296 (83.72)	...	...	...	2.618 (66.50)	...	...	...	...	...
48	50.800 (1290.32)	2.419 (61.44)	2.988 (75.90)	3.763 (95.58)	...	...	...	2.988 (75.90)	...	...	...	...	...
54	57.560 (1462.3)	2.741 (69.620)	...	...	...	...	...	...	...	...	...	...	...
60	61.610 (1564.9)	2.934 (74.520)	...	...	..	...	...	...	...	...	...	...	...

[A] Thermoplastic material designation code per 4.1.1.

[B] See 9.1.7.

[C] Per Table 2

[D] Per 3.2.1. Values rounded to the nearest 5 kPa.

Pipe Calculations

AECOM

Project: Lake Erin Dam Originator: TKW Date: 5/24/2024
Reviewed by: LD Date: 6/14/2024

CALCULATION PACKAGE: Drain Pipe Load on Pipe

Loads on Pipe (Reference NRCS National Engineering Handbook, Part 636.5203)

OBJECTIVE:

Verify the designed internal drainpipe is suitable for expected loading conditions at Lake Erin Dam. The filter diaphragm is analyzed as it will have the highest loading pressure, given the location beneath the crest of the dam.

REFERENCES:

NRCS. (2005). National Engineering Handbook, Part 636 Structural Engineering, Chapter 52 “Structural Design of Flexible Conduits”. United States Department of Agriculture, Natural Resources Conservation Service.

ASTM. (2022). F714-22 “Standard Specification for Polyethylene (PE) Plastic Pipe (DR-PR) Based on Outside Diameter”. ASTM International. West Conshohocken, PA.

ASTM. (2014). D3350 “Standard Specification for Polyethylene Plastics Pipe and Fittings Materials”. ASTM International. West Conshohocken, PA.

FEMA. (2007). Technical Manual: Plastic Pipe Used in Embankment Dams

CALCULATIONS:

6-inch HDPE Specifications:

Outside Diameter (D_o) = 6.9 inches (Based on DIPS Sizing, ASTM F714, Table 2, 2022) (Reference 1)

Minimum wall thickness (t) = 0.406 inches (ASTM F714, Table 3, 2022) (Reference 1)

Calculate loads on pipe

Soil Pressure

The most interior approach into embankment is within the filter diaphragm, located beneath the crest of the dam with the upstream face located 2 feet downstream of the centerline of the crest. Analysis was performed at the downstream edge filter diaphragm base, which is at the approximate downstream edge of crest. At this location, the overburden is primarily embankment fill, which will have a higher unit weight than the drain fill after compaction. The top of drainpipe is designed at 941.9 ft. Based on proposed surface elevation of the crest at 970 ft, the approximate depth to drainpipe is 28.1 feet (970 ft - 941.9 ft).

CALCULATION PACKAGE: Drain Pipe Load on Pipe

$[P_s = \gamma_s * h]$ (NRCS 2005, equation 52-17)

P_s = Pressure due to weight of soil at depth of h, lb/ft^2
Υ_s = unit weight of soil, lb/ft^3
h = height of ground surface above top of pipe, ft

h = 28.1 feet
Assume overburden (Embankment Fill) is 128 lb/ft^3 unit weight
Conservatively assume saturated unit weight for overburden with no buoyancy reduction (groundwater elevation at or below top of pipe).

$$P_s = (28.1\ feet)\left(128\frac{lb}{ft^3}\right) = 3596.8\frac{lb}{ft^2}$$

Soil load per foot length of pipe

$\left[W_s = P_s * \frac{D_o}{12}\right]$ (NRCS 2005, equation 52-18)

W_s = Soil load per linear foot of pipe, lb/ft
P_s = Pressure due to weight of soil at depth of h, lb/ft^2
D_o = outside diameter of pipe, in

$$\left[W_s = 3596.8\frac{lb}{ft^2} * \frac{6.9\ in}{12}\right] = 2068.2\ \frac{lb}{ft}$$

Wheel Loading

Assume Load Class = field equipment: P_L = 10,000 lb
When depth of fill is 2 feet or more, wheel loads may be considered as uniformly distributed over a square with sides equal to 1 ¾ times the depth of fill (NRCS, 2005).

$P_w = \frac{P_L}{(1.75h)^2}$ (NRCS 2005, equation 52-22)

P_w, pressure on pipe from wheel load, lb/ft^2
D_o = outside diameter of pipe, in

$$P_w = \frac{10{,}000\ lb}{(1.75(28.1\ ft))^2} = 4.1\frac{lb}{ft^2}$$

P_w = 4.1 lb/ft^2

Vacuum Pressure

Assume no vacuum load or hydrostatic pressure as the conduit is slotted within the filter diaphragm and open at outlet. Groundwater designed to be at below top of pipe at downstream face of filter diaphragm.

<u>Section 636.5204, Buried Pipe Design</u>

The typical modes of failure of buries flexible pipe include wall crushing, local buckling, or excessive deflection.

Wall Crushing

$$P = P_s + P_w + P_v \qquad \text{(NRCS 2005, equation 52-25)}$$

P = Pressure on pipe, lb/ft^2
P_s = Pressure due to weight of soil at depth of h, lb/ft^2
P_w = pressure on pipe from wheel load, lb/ft^2
P_v = internal vacuum pressure, lb/ft^2
D_o = outside diameter of pipe, in

$$P = 3596.8\ \frac{lb}{ft^2} + 4.1\ \frac{lb}{ft^2} + 0\ \frac{lb}{ft^2} = 3600.9\ \frac{lb}{ft^2}$$

Thrust

$$T_{pw} = \frac{P * \frac{D_0}{12}}{2} \qquad \text{(NRCS 2005, equation 52-26)}$$

T_{pw} = thrust in pipe wall, lb/ft
P = Pressure on pipe, lb/ft^2
D_o = outside pipe diameter, in

$$T_{pw} = \frac{3600.9\,\frac{lb}{ft^2} * \frac{6.9\,in}{12}}{2} = 1035.3\,\frac{lb}{ft}$$

$$A_{pw} = \frac{\frac{T_{pw}}{12}}{\sigma}$$ (NRCS 2005, equation 52-27)

A_{pw} = required wall area, in^2/in,

T_{pw} = thrust in pipe wall, lb/ft

σ = allowable long term compressive stress, lb/in^2,

From FEMA “Technical Manual: Plastic Pipe Used in Embankment Dams (November 2007), Section 3.1.1, the allowable long term compressive strength of pipe is equal to one-half the hydrostatic design basis of pipe (HDB) (Reference 2). From ASTM D3350, the HDB for Class 4 is 1600 psi (Reference 3). Therefore, the allowable long term compressive strength is assumed 800 (lb/in2).

$$A_{pw} = \frac{\frac{1035.3\,\frac{lb}{ft}}{12}}{800\,\frac{lb}{in^2}} = 0.108\,\frac{in^2}{in}$$

t = 0.406 in^2/in > 0.108 in^2/in **Okay**

Deflection

For drain in embankment dam, deflection should be less than 7.5 percent for drains in embankment dams (NRCS 2005, pg 52-11)

$$\frac{\%\Delta X}{\mathrm{D}} = \frac{(D_L P_S + P_W + P_V)\left(\frac{1}{144}\right)\mathrm{K}(100)}{\left[\left(\frac{2E}{3(SDR-1)^3} + 0.061E'\right)\right]}$$ (NRCS 2005, Equation 52-29)

Where:

$\frac{\%\Delta X}{D}$ = percent deflection

D_L = deflection lag factor, (1.0 to 1.5), assume 1.5 from Section 636.5205 (NRCS, 2005, page 52-9)

K = bedding constant (0.1)

Ps = pressure on pipe from soil, lb/ft^2

Pw = pressure on pipe from wheel load, lb/ft^2

Pv = internal vacuum pressure, lb/ft^2, assume 0 lb/ft^2

E = modulus of elasticity of pipe material, 110,000 lb/in^2 for Polyethylene, (from NRCS 2005, page 52-11)

SDR = Do dimension ratio (Do/t) = 17, from design

E' = modulus of soil reaction, lb/in^2, 2000 lb/in^2 from Table 52-2 of NRCS, 2005. From laboratory testing, the embankment fill is represented as coarse-grained soil with more than 12% fines. Assumed high density for degree of compaction of bedding based on design specifications of 98% compaction.

$$\frac{\%\Delta X}{\text{D}} = \frac{\left((1.5)(3596.8) + 4.1 + 0\right)\left(\frac{1}{144}\right)(0.1)(100)}{\left[\left(\frac{2(110,000)}{3(17-1)^3} + 0.061(2000)\right)\right]} = \frac{374.9}{139.9} = 2.68\%$$

$\frac{\%\Delta X}{D}$ = percent deflection = 2.68% < 7.5% **Okay**

Wall Buckling

Allowable buckling pressure

$$q_a = \frac{1}{FS}\left(32R_w B'E'\frac{E_{long}I_{pw}}{D_o^3}\right)^{1/2}$$ (NRCS 2005, equation 52-33)

q_a = allowable buckling pressure, lb/in^2

FS = design factor of safety, 2.5 for $(h/(D_o/12))>2$

Rw = water buoyancy factor, (assume 1, no water above pipe)

B' = empirical coefficient of plastic support

$$B' = \frac{4\left(h^2 + \left(\frac{D_o}{12}\right)h\right)}{1.5\left(2h + \left(\frac{D_0}{12}\right)\right)^2} = \frac{4\left(28.1^2 + \left(\frac{6.9}{12}\right)(28.1)\right)}{1.5\left(2(28.1) + \left(\frac{6.9}{12}\right)\right)^2} = \frac{3223.1}{4835.1} = 0.67$$

B' = 0.67

E_{long} = long term modulus of elasticity, lb/in^2; 22,000 lb/in^2 for PE (From NRCS 2005, Page 52-11)

E' = modulus of soil reaction, lb/in^2; 2,000 lb/in^2

I_{pw}, pipe wall moment of inertia; t^3/12, in^4/in = 0.00557 in^4/in

Do = outside diameter

$$q_a = \frac{1}{2.5}\left(32(1)(0.67)(2000)\frac{(22,000)(0.00557)}{328.51}\right)^{1/2}$$

$$q_a = \frac{1}{2.5}((42880)(0.373))^{1/2}$$

q_a = 50.59 lb/in^2 → 7285.0lb/ft^2 > 3600.9 lb/ft^2 **Okay**

Correction Factor

$$C = \frac{1-\frac{\%\Delta X}{D}\frac{1}{100}}{1+\frac{\%\Delta X}{D}\frac{1}{100}}$$ (NRCS 2005, equation 52-34)

$\frac{\%\Delta X}{D}$ = percent deflection = 2.68 %

$C = \frac{1-\frac{2.68}{100}}{1+\frac{2.68}{100}} = \frac{0.9732}{1.0268}$ = 0.948

qa = 50.59 lb/in^2 *0.948 → 6906.1 lb/ft^2 > 3600.9 lb/ft^2

However, for all types of solid-wall plastic pipe (PVC, HDPE, etc), slots will reduce the load-carrying capacity (loss in strength proportional to slot percent open area) (FEMA Technical Manual: Plastic Pipe Used in Embankment Dams, page 103 (November 2007) (Reference 3).

From slot size calculations using 2 row, 1/8” slot perforations (assume 60 degree slots, 3.62 inch outside length).

Perimeter of circle = $2\pi r$

r = 6.9 in /2 = 3.45 in

Percent of openings at slots = $\frac{60}{360} = 0.167$

$$Length\ of\ opening\ for\ each\ slot = 0.1672\pi(3.45\text{ in}) = 3.62\ in$$

Percent open area per unit length (1-ft) is:

$$area\ of\ pipe = 2\pi r * 1\ ft = 2\pi(3.45\ in) * 12\ in = 260.1\ in^2$$

Area of opening per ft

$$3.62\ in\ \left(\frac{1}{8} in\ width\right) * 2 * 9\ (slot\ sections\ per\ foot) = 8.1\ in^2$$

$$percent\ area\ of\ pipe = \frac{8.1\ in^2}{260.1\ in^2} = 3\%$$

Conservatively assume 10%.

qa = .9 * 6906.1 lb/ft^2 = 6215.5 lb/ft^2> 3600.9 lb/ft^2

Strain

$\varepsilon_h = \frac{\frac{P}{144} D_m}{2tE}$ (NRCS 2005, equation 52-36)

Where:
εh=maximum strain in pipe wall because of ring bending, in/in
P= pressure on/in pipe, lb/ft2
DM=mean pipe diameter, in
E= modulus of elasticity of the pipe material, lb/in2
t =pipe wall thickness, in

$$\varepsilon_h = \frac{\frac{3600.9\ lb/ft^2}{144}(6.9 - .406)\ in}{2(0.406 in)(22000\ lb/in^2)} = \frac{162.4\ in}{17864\ in}$$

ε_h= 9.09E-03 in/in

$$\varepsilon_f = \frac{t}{D_m}\left(\frac{\frac{3\Delta Y}{D_m}}{1-2\frac{\Delta Y}{D_m}}\right) \qquad \text{(NRCS 2005, equation 52-37)}$$

$$\varepsilon_f = \frac{0.406\ in}{6.494\ in}\left(\frac{3 \times 0.0178}{1 - 2(0.0178)}\right)$$

$$\varepsilon_f = 3.46 \times 10^{-3}\ in/in$$

$$\varepsilon = \varepsilon_f + \varepsilon_h = 3.46 \times 10^{-3}\frac{in}{in} + 9.09 \times 10^{-3}\frac{in}{in} += 1.26 \times 10^{-2}\frac{in}{in}$$

$$\varepsilon \leq \varepsilon_{all} \leq 5\% = 0.05$$ **Okay**

CONLCUSION:

6-inch HDPE pipe is suitable for expected loading conditions at Lake Erin Dam based on crushing, buckling, deflection and strain.

(Reference 1): ASTM. (2022). F714-22 "Standard Specification for Polyethylene (PE) Plastic Pipe (DR-PR) Based on Outside Diameter". ASTM International. West Conshohocken, PA.

TABLE 3 Minimum Wall Thickness DIPS Sizing System, in.

		PE4710[A]						PE3608[A]					
		PR100[B]	PR125[B]	PR160[B]	PR200[B]	PR250[B]	PR335[B]	PR100[B]	PR150[B]	PR200[B]	PR250[B]	PR300[B]	PR350[B]
Nominal Size	Outside Diameter in. (mm)[C]	100 psi (690 kPa)[D]	125 psi (860 kPa)[D]	160 psi (1100 kPa)[D]	200 psi (1380 kPa)[D]	250 psi (1725 kPa)[D]	335 psi (2310 kPa)[D]	100 psi (690 kPa)[D]	150 psi (1035 kPa)[D]	200 psi (1380 kPa)[D]	250 psi (1725 kPa)[D]	300 psi (2070 kPa)[D]	350 psi (2415 kPa)[D]
		DR 21	DR 17	DR 13.5	DR 11	DR 9	DR 7	DR 17	DR 11.7	DR 9	DR 7.4	DR 6.3	DR 5.6
3	3.960	0.189	0.233	0.293	0.360	0.440	0.605	0.233	0.338	0.440	0.535	0.629	0.707
	(100.58)	(4.80)	(5.92)	(7.53)	(9.14)	(11.18)	(14.00)	(5.92)	(8.59)	(11.18)	(13.59)	(15.97)	(17.96)
4	4.800	0.229	0.282	0.356	0.436	0.533	0.686	0.282	0.410	0.533	0.649	0.762	0.857
	(121.92)	(5.82)	(7.16)	(9.04)	(11.07)	(13.54)	(17.42)	(7.16)	(10.41)	(13.54)	(16.48)	(19.35)	(21.77)
6	6.900	0.329	0.406	0.511	0.627	0.767	0.986	0.406	0.590	0.767	0.932	1.095	1.232
	(175.26)	(8.36)	(10.31)	(12.98)	(15.93)	(19.48)	(25.04)	(10.31)	(14.99)	(19.48)	(23.67)	(27.82)	(31.30)
8	9.050	0.431	0.532	0.670	0.823	1.006	1.293	0.532	0.774	1.006	1.223	1.437	1.616
	(229.87)	(10.95)	(13.51)	(17.02)	(20.90)	(25.55)	(32.84)	(13.51)	(19.66)	(25.55)	(31.06)	(36.49)	(41.05)
10	11.100	0.529	0.653	0.978	1.009	1.233	1.586	0.653	0.949	1.233	1.500	1.762	1.982
	(281.94)	(13.44)	(16.59)	(24.84)	(25.63)	(31.32)	(40.28)	(16.59)	(24.10)	(31.32)	(38.10)	(44.75)	(50.35)
12	13.200	0.629	0.776	0.978	1.200	1.467	1.886	0.776	1.128	1.467	1.784	2.095	2.357
	(335.28)	(15.98)	(19.71)	(24.84)	(30.48)	(37.26)	(47.90)	(19.71)	(28.65)	(37.26)	(45.31)	(53.22)	(59.87)
14	15.300	0.729	0.900	1.133	1.391	1.700	2.186	0.900	1.308	1.700	2.068	2.429	2.732
	(388.62)	(18.52)	(22.86)	(28.78)	(35.33)	(43.18)	(55.52)	(22.86)	(33.22)	(43.18)	(52.53)	(61.69)	(69.40)
16	17.400	0.829	1.024	1.289	1.582	1.933	2.486	1.024	1.487	1.933	2.351	2.762	3.107
	(441.96)	(21.06)	(26.01)	(32.74)	(39.67)	(49.10)	(63.14)	(26.01)	(37.77)	(49.10)	(59.72)	(70.15)	(78.92)
18	19.500	0.929	1.147	1.444	1.773	2.167	2.789	1.147	1.667	2.167	2.635	3.095	3.482
	(495.30)	(23.60)	(29.13)	(36.68)	(45.03)	(55.04)	(70.76)	(29.13)	(42.34)	(55.04)	(66.93)	(78.62)	(88.45)
20	21.600	1.029	1.271	1.600	1.964	2.400	3.086	1.271	1.846	2.400	2.919	3.429	...
	(548.64)	(26.14)	(32.28)	(40.64)	(49.89)	(60.96)	(78.38)	(32.28)	(46.89)	(60.96)	(74.14)	(87.09)	
24	25.800	1.229	1.518	1.911	2.345	2.867	3.686	1.518	2.205	2.867	3.486	...	...
	(655.32)	(31.22)	(38.56)	(48.54)	(59.56)	(72.82)	(93.62)	(38.56)	(56.01)	(72.82)	(88.54)		
30	32.000	1.524	1.882	2.370	2.909	3.556	...	1.882	2.735	3.556	...	...	...
	(815.80)	(38.71)	(47.80)	(60.20)	(73.89)	(90.32)		(47.80)	(69.47)	(90.32)			
36	38.300	1.824	2.253	2.837	3.482	...	...	2.253	3.274	...	...	...	...
	(972.82)	(46.33)	(57.23)	(72.06)	(88.44)			(57.23)	(83.16)				
42	44.500	2.119	2.618	3.296	...	...	...	2.618	...	...	...	...	...
	(1130.30)	(53.82)	(66.50)	(83.72)				(66.50)					
48	50.800	2.419	2.988	3.763	...	...	...	2.988	...	...	...	...	...
	(1290.32)	(61.44)	(75.90)	(95.58)				(75.90)					
54	57.560	2.741	...	...	...	...	...	...	...	...	...	...	...
	(1462.3)	(69.620)											
60	61.610	2.934	...	...	..	...	...	...	...	...	...	...	...
	(1564.9)	(74.520)											

[A] Thermoplastic material designation code per 4.1.1.
[B] See 9.1.7.
[C] Per Table 2
[D] Per 3.2.1. Values rounded to the nearest 5 kPa.

(Reference 3): ASTM D3350 “Standard Specification for Polyethylene Plastics Pipe and Fittings Materials”. ASTM International. West Conshohocken, PA.

TABLE 1 Primary Properties[A] —Cell Classification Limits

Property	Test Method	0	1	2	3	4	5	6	7	8
1. Density, g/cm^3	D1505	Unspecified	0.925 or lower	>0.925-0.940	>0.940-0.947	>0.947-0.955	>0.955	. . .	Specify Value	
2. Melt index	D1238	Unspecified	>1.0	1.0 to 0.4	<0.4 to 0.15	<0.15[B]	[C]		Specify Value	
3. Flexural modulus, MPa (psi)	D790	Unspecified	<138 (<20 000)	138-<276 (20 000 to <40 000)	276-<552 (40 000 to 80 000)	552-<758 (80 000 to 110 000)	758-<1103 (110 000 to <160 000)	>1103 (>160 000)	Specify Value	
4. Tensile strength at yield, MPa (psi)	D638	Unspecified	<15 (<2200)	15-<18 (2200-<2600)	18-<21 (2600-<3000)	21-<24 (3000-<3500)	24-<28 (3500-<4000)	>28 (>4000)	Specify Value	
5. Slow Crack Growth Resistance										
I. ESCR	D1693	Unspecified								
a. Test condition (100% Igepal.)[D]			A	B	C	C	. . .	. . .	. . .	Specify Value
b. Test duration, h			48	24	192	600				
c. Failure, max, %		Unspecified	50	50	20	20				
II. PENT (hours)	F1473									
Molded plaque, 80°C, 2.4 MPa		Unspecified	. . .	. . .	. . .	10	30	100	500	Specify Value
Notch depth, F1473, Table 1		Unspecified								
6. Hydrostatic Strength Classification										
I. Hydrostatic design basis, MPa (psi), (23°C)	D2837	NPR[E]	5.52 (800)	6.89 (1000)	8.62 (1250)	11.03 (1600)	. . .	. . .		
II. Minimum required strength, MPa (psi), (20°C)	ISO 12162	. . .	. . .	. . .	. . .	. . .	8 (1160)	10 (1450)		

[A] Compliance with physical properties in accordance with Section 8 is required including requirements for cell classification, color, and ultraviolet (UV) stabilizer, thermal stability, brittleness temperature, density, tensile strength at yield, and elongation at break.

[B] Refer to 10.1.4.1.

[C] Refer to 10.1.4.2.

[D] There are environmental concerns regarding the disposal of Nonylphenoxy poly(ethyleneoxy) ethanol (CAS 68412-54-4) for example, Igepal CO-630. Users are advised to consult their supplier or local environmental office and follow the guidelines provided for the proper disposal of this chemical.

[E] NPR = Not Pressure Rated.

6. Physical Properties

6.1 *Cell Classification*—Test values for specimens of the PE material prepared as specified in Section 9 and tested in accordance with Section 10 shall conform to the requirements given in Table 1. A typical property value for a PE material is to be the average value from testing numerous lots or batches and determines the cell number. When, due to manufacturing tolerances and testing bias, individual lot or batch values fall into the adjoining cell, the individual value shall not be considered acceptable unless the user, or both the user and the producer, determine that the individual lot or batch is suitable for its intended purpose.

6.2 *Color and Ultraviolet (UV) Stabilizer*—The color and UV stabilization shall be indicated at the end of the cell classification by means of a letter designation in accordance with the following code:

Code Letter	Color and UV Stabilizer
A	Natural
B	Colored
C	Black with a carbon black in the range as noted in 6.2.1 and 6.2.2
D	Natural with UV stabilizer
E	Colored with UV stabilizer

6.2.1 For PE compounds with a hydrostatic strength classification cell class 0 (not pressure-rated), the carbon black content shall be in the range of 2.0 % to 4.0 %.

6.2.2 For PE compounds with a hydrostatic strength classification other than cell class 0, the carbon black content shall be in the range of 2.0 % to 3.0 %.

6.3 *Thermal Stability*—The PE material shall contain sufficient antioxidant so that the minimum induction temperature shall be 220°C when tested in accordance with 10.1.9.

6.4 *Brittleness Temperature*—The brittleness temperature shall not be warmer than −60°C when tested in accordance with Test Method D746.

6.5 *Density*—The density used to classify the material shall be the density of the PE base resin (uncolored PE) determined in accordance with 10.1.3. When the average density of any lot or shipment falls within ±0.002 g/cm^3 of the nominal value, it shall be considered as conforming to the nominal value and to all classifications based on the nominal value.

6.5.1 For black compounds, containing carbon black, determine the density, Dp, and calculate the resin density, Dr, as follows:

(Reference 3): FEMA Technical Manual: Plastic Pipe Used in Embankment Dams (November 2007)

Typical failure modes of flexible pipes are shown in figure 43. Flexible pipe design of buried plastic pipe includes analyses of the wall crushing, buckling resistance, allowable long-term deflection, and allowable strain. Deflection and buckling most often control the design of flexible pipe. Table 9 in section 3.5.6 provides the appropriate method of determining the soil load based on soil type and type of conduit.

3.1.1 Wall crushing

Wall crushing in plastic pipe is characterized by localized yielding when the in-wall stress reaches the yield stress of the pipe material (Moser, 2001, p. 499). Wall crushing typically occurs at the 3 and 9 o'clock positions as illustrated in figure 43a. Figure 44 shows an example of wall crushing. This localized yielding can occur in improperly designed stiff flexible pipes installed in deep, highly compacted fill. Less stiff flexible pipe more frequently fails from wall buckling, as discussed in section 3.1.2.

Resistance to wall crushing of plastic pipe is evaluated by:

$$T_{pw} = \frac{PD_O}{2} \tag{3-1}$$

where:

T_{pw} = thrust in pipe wall, lb/in
D_O = outside diameter of the pipe, in
P = design pressure ($P_S+P_V+P_W$), lb/in^2 (see equations 2-6, 2-15, and 2-17)

The required wall cross-sectional area is determined by:

$$A_{pw} = \frac{T_{pw}}{\sigma} \tag{3-2}$$

where:

A_{pw} = area of the pipe wall, in^2/in of pipe length
T_{pw} = thrust in pipe wall, lb/in
σ = allowable long-term compressive stress, lb/in^2
= $HDB/2$
HDB = hydrostatic design basis of the pipe, lb/in^2

The actual area for a solid wall pipe wall may be computed as:

$$A_{pw} = \frac{(D_O - D_i)}{2} \text{ or } t \tag{3-3}$$

(Reference 3): FEMA Technical Manual: Plastic Pipe Used in Embankment Dams (November 2007)

Figure 53.—Installation of profile wall corrugated pipe for a drainpipe replacement during a modification of an embankment dam.

Table 11.—Drainpipe diameter based on dam size and foundation type

	Foundation type	
Dam height (ft)	Pervious < 15% fines (SP, GP)	Semipervious or impervious >15% fines (SM, GM, ML, CL, SC, GC)
< 30	min. 12 in	min. 8 in
30-100	12-18 in	min. 12 in
> 100	18-24 in	min. 12 in

While the load-carrying capacity of nonperforated pipe is well documented, the strength of perforated pipe is less commonly addressed. Since the corrugations carry the majority of the load for both single-wall and profile-wall HDPE pipe, perforations through the corrugation valley have negligible effect on pipe strength (less than 1 percent). However, for all types of solid-wall plastic pipe (PVC, HDPE, etc), perforations will reduce the load-carrying capacity (loss in strength proportional to perforation percent open area). Additional research (PM-3) is needed as proposed in chapter 8.

Solid and profile wall corrugated pipe have the additional benefits of a smooth interior, which increases flow capacity, and no interior corrugations to collect and trap soil particles (which should be trapped at the measurement point sediment trap). Joints for corrugated pipe are typically bell and spigot with a gasket. Solid wall

Drain Pipe Slot Sizing

Determine cross-sectional area of perforations

NRCS NEH 210-633-H, Chapter 26 Gradation Design of Sand and Gravel Filters (2017) (Reference 1)

Perforations shall be the smaller of:

$$\frac{\text{Minimum 50 percent size of filter material}}{\text{hole diameter or slot width}} \geq 1$$

or

$$\frac{\text{One-half Minimum 85 percent size of filter material}}{\text{hole diameter or slot width}} \geq 1$$

Enter filter size, $d_{50,}$ and d_{85} of the fine side of the filter					
Min D_{50}	ASTM #8	5.7	mm	0.22	in
1/2 Min D_{85}	ASTM #8	4.1	mm	0.16	in

*Use minimum gradation to limit potential particle infiltration into pipe

Similar Commmercially Available Perforation Diameter or Slot Width		
or	0.1875	in
	0.125	in

Recommend a maximum perforation size of 0.125 inch

Slot sizes commercially available from 0.008 inches (1/12 inch)

Size		
Inch	Inch	mm
1/8	0.125	3.18
3/16	0.1875	4.76
1/4	0.25	6.35
5/16	0.3125	7.94
3/8	0.375	9.53
7/16	0.4375	11.11
1/2	0.5	12.70
9/16	0.5625	14.29
5/8	0.625	15.88

Use 6-inch pipe minimum for cleanout and ROV Inspection

From NRCS Material Specification 547-Plastic Pipe (2009)-Slot Perforations-6 Inch Pipe (Table 547-1) (Reference 2)		
Minimum # of rows	2	
Minimum Center-Center distance between	11d	
minimum required opening/foot	0.44	in^2

Symmetrically located in two rows, one on each side of pipe centerline

Slot perforations shall be located within lower quadrants of the pipe with slots no wider than 1/8 inch and spaced not to exceed 11 times perforation width

For Area of Slots	Use 60° Slot	
I.D. of 6 inch Pipe	6.088	in
perimeter of 6 inch pipe	19.13	in
degrees in circle	360	
degrees of slot	60	
percent degrees of slot	16.67%	
length of slot	3.19	in

for 1/8" in width Slots:	Use 60° Slot	
Area =	0.40	in^2
11d =	1.375	in
# columns per foot	8.73	
# of Rows	2	
# slots per foot	17.5	
Area opening per foot	6.95	in^2
Area opening per foot	0.05	ft^2

Drainpipe Flow Calculations

	Slotted	
From Filter Drain Design, flow rate (Q) is:		
66	ft^3/day	
With fs	=	20
Design Discharge =	1.32E+03	ft^3/day

From: NRCS Soil Mechanics Note No. 3, Appendix A (1971) (Reference 3)

Using NRCS Material Specification 547 for Area

Description	Symbol	Value	Unit		
discharge in cfs, per foot length of pipe	q	5.51E-02	ft^3/sec	4759.559	ft^3/day
cross-sectional area of orifice per foot	A	0.05	ft^2	60 degree slots	
Effective cross-sectional Area of orifice per foot	A_e	0.029	ft^2		
head on center of orifice in ft	h	0.125	ft	Assume 1.5-inch	
acceleration of gravity in ft/sec^2	g	32.2	ft/sec^2		
coefficient of discharge	C	0.67	---		

$$q = CA_e(2gh)^{0.5}$$

For 6 inch slotted pipe		
Slotted	2	rows
Approximate cfd per unit foot	4759.6	ft^3/day
cfd required for drain	1320	ft^3/day
Minimum slotted pipe length	1	ft

Conclusion:
flow through the designed slots per only 1 foot of the pipe > design discharge

(Reference 1): NRCS NEH 210-633-H, Chapter 26 Gradation Design of Sand and Gravel Filters (2017)

size is then five times the minimum D_{60} size. See figure 26–1, points 3 and 4.

To prevent gap graded filters
Both sides of the design filter band will have a CU defined as coefficient of uniformity = $D_{60} \div D_{10}$, equal to or less than 6. Initial design filter bands by this step will have CU values of 6. For final design, filter bands may be adjusted to a steeper configuration, with CU values less than 6, if needed. This is acceptable as long as other filter and permeability criteria are satisfied. Filters should not be designed with a CU value less than 2, as this would be a very poorly graded filter that could be subject to bulking, difficult to obtain, and difficult to compact. Initial bands are often steepened to accommodate the use of a standard commercially available gradation. Appendix 26A, 26A–12 has extensive additional descriptions of this step in the design of filters.

Step 8 The maximum particle size allowed is 2 inches and the maximum percentage passing the No. 200 sieve is 5 percent. Refer to appendix 26A, 26A–8 for additional guidance.

Step 9 To ensure that the filter cannot easily segregate during construction, the filter must not be overly broad in gradation. The relationship between the maximum D_{90} and the minimum D_{10} of the filter is important. Calculate a preliminary minimum D_{10} size by dividing the minimum D_{15} size by 1.2. (This factor of 1.2 is based on the assumption that the slope of the line connecting D_{15} and D_{10} should be on a coefficient of uniformity of about 6.) Determine the maximum D_{90}. The coarse side of the design band must be finer than the maximum $D_{90.}$ (See point 5 on fig. 26–1. See app. 26A, 26A–9 for the description.)

Step 10 Connect the minimum $D_{5,}$ D_{15}, and D_{60} sizes with a smooth curve to begin forming the fine side of the design band. Then, extrapolate the curve upwards smoothly, with a slightly convex shape to the D_{100} size. Connect the coarse control points, which are the maximum D_{15} and D_{60} control points, with a smooth curve. Extrapolate the curve upwards to an even D_{100} size that is equal to or smaller than the established maximum D_{100} size from step 8. Extrapolate the curve downwards from the maximum D_{15} size to the zero percent passing axis, intercepting the axis at a sieve size that will be used in writing specifications. Ensure that the curve is finer than the maximum D_{90} size established in step 9. For purposes of writing specifications, select appropriate sieves and corresponding percent finer values that best reconstruct the design band and tabulate the values. See appendix 26A, 26A–10 for an illustration.

Step 11 The D_{50} of the surrounding filter must be larger than the perforation diameters or slot widths in a collector pipe installed in the filter. Perforations or slots should not be smaller than a quarter inch unless the pipe is surrounded with a gravel filter or a well-screen-type pipe is used with a slot size smaller than the criterion specified. See appendix 26A, 26A–11 for more detail.

Criteria for filters used adjacent to perforated collector pipe
Perforations or slots in pipes placed in the designed filter zone should be no larger than the smaller of the following:

- Half the d_{85} of the fine side of the filter
- The D_{50} size of the fine side of the filter

Step 12 The design band obtained in these steps is satisfactory to meet all the established filter and permeability requirements for a filter. However, in some cases, adjustments to the preliminary design band are made to accommodate standard readily available gradations. Appendix 26A, 26A–12 has additional information on adjusting the preliminary design band obtained in these steps to accommodate standard readily available gradations.

Table 26–3 Segregation criteria

Base soil category	**If D_{10} is: (mm)**	**Then, maximum D_{90} is: (mm)**
ALL categories	< 0.5	20
	0.5-1.0	25
	1.0-2.0	30
	2.0-5.0	40
	5.0-10	50
	> 10	60

(Reference 2): NRCS Material Specification 547 - Plastic Pipe (2009)

Material Specification 547—Plastic Pipe

1. Scope

This specification covers the quality of Poly Vinyl Chloride (PVC), Polyethylene (PE), High Density Polyethylene (HDPE), and Acrylonitrile-Butadiene-Styrene (ABS) plastic pipe, fittings, and joint materials.

2. Material

Pipe—The pipe shall be as uniform as commercially practicable in color, opaqueness, density, and other specified physical properties. It shall be free from visible cracks, holes, foreign inclusions, or other defects. The dimensions of the pipe shall be measured as prescribed in ASTM D 2122.

Unless otherwise specified, the pipe shall conform to the requirements listed in this specification and the applicable reference specifications in table 547–2, the requirements specified in Construction Specification 45, Plastic Pipe, and the requirements shown on the drawings.

Fittings and joints—Fittings and joints shall be of a schedule, SDR or DR, pressure class, external load carrying capacity, or pipe stiffness that equals or exceeds that of the plastic pipe. The dimensions of fittings and joints shall be compatible with the pipe and measured in accordance with ASTM D 2122. Joint and fitting material shall be compatible with the pipe material. The joints and fittings shall be as uniform as commercially practicable in color, opaqueness, density, and other specified physical properties. It shall be free from visible cracks, holes, foreign inclusions, or other defects.

Fittings and joints shall conform to the requirements listed in this specification, the requirements of the applicable specification referenced in the ASTM or AWWA specification for the pipe, the requirements specified in Construction Specification 45, and the requirements shown on the drawings.

Solvents—Solvents for solvent welded pipe joints shall be compatible with the plastic pipe used and shall conform to the requirements of the applicable specification referenced in the ASTM or AWWA specification for the pipe, fitting, or joint.

Gaskets—Rubber gaskets for pipe joints shall conform to the requirements of ASTM F 477, Elastomeric Seals (Gaskets) for Jointing Plastic Pipe.

3. Perforations

When perforated pipe is specified, perforations shall conform to the following requirements unless otherwise specified in Construction Specification 45 or shown on the drawings:

a. Perforations shall be either circular or slots.

b. Circular perforations shall be 1/4 ± 1/16-inch diameter holes arranged in rows parallel to the axis of the pipe. Perforations shall be evenly spaced along each row such that the center-to-center distance between perforations is not less than eight times the perforation diameter. Perforations may appear at the ends of short and random lengths. The minimum perforation opening per foot of pipe shall be as shown in table 547–1.

Table 547–1 Perforations

Nominal pipe size (in)	Minimum number of rows		Minimum opening/foot (in^2)
	circular	slot	
4	2	2	0.22
6	4	2	0.44
8	4	2	0.44
10	4	2	0.44
12	6	2	0.66

(Reference 3): NRCS Soil Mechanics Note No. 3, Appendix A (1971)

Appendix A

Pipes in Drains

This appendix contains an approach for sizing of pipes installed in drains. Two flow conditions are considered: (1) flow through openings into the pipe is based on orifice flow with an area reduction to account for blockage by particles and (2) flow in the pipe is based on open channel flow.

Review of many papers dealing with pipes in drains yielded only a small amount of data on head-discharge relationships for perforations or slots. Information from three studies is plotted on Figure A-1, Head-discharge relationship for pipe perforations. Gradation of the drain material surrounding these pipes is shown on Figure A-2. Comments on the head-discharge curves are as follows:

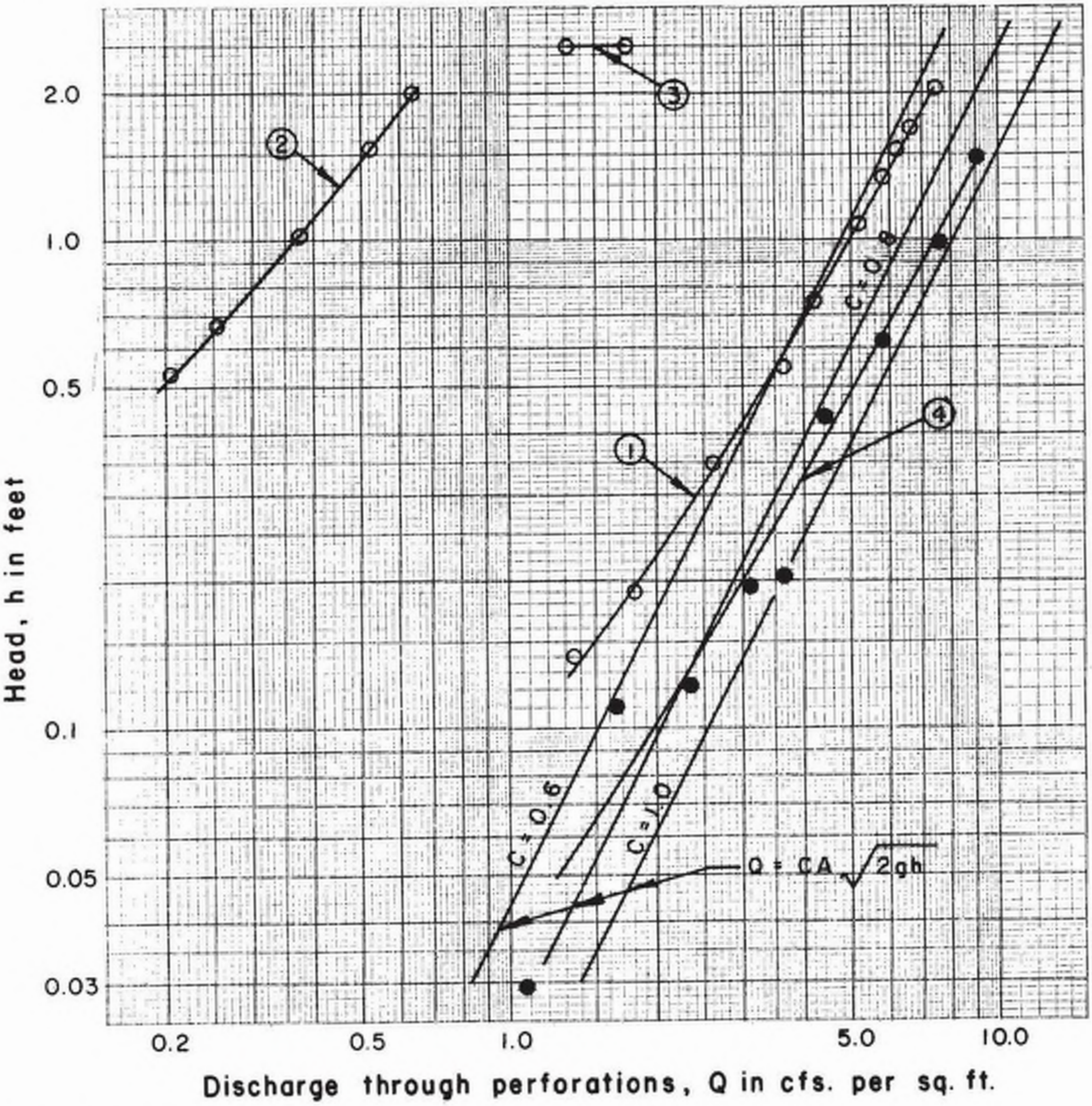
1. A comparison of the curve for $Q = CA\sqrt{2gh}$, $C = 0.6$, and curve No. 1, pipe in water only, indicates that a coefficient of discharge of 0.6 is reasonable for the perforations in this uncoated corrugated metal pipe.

2. With the uncoated corrugated metal pipe imbedded in medium SP, curve No. 2, discharge through the perforations is about 10% of the discharge without sand around the pipe.

3. The range represented by No. 3 shows that discharge through perforations of this coated corrugated metal pipe placed in coarse SP is about 20% of the discharge with the pipe in water only and assuming that $C = 0.6$.

4. Curve No. 4 is for flow into clay pipe with a wall thickness of 5/8 in. The perforation length to diameter ratio is 2.5, in the range of short tubes, where a discharge coefficient of 0.8 is normal. Flow through joints could not be separated from flow through perforations and it is not known how this would affect the discharge coefficient. Even assuming that $C = 1.0$, discharge through openings in this pipe placed in GP is greater than 80% of that without restriction.

In a recent study, "Laboratory Tests of Relief Well Filters", Report No. 1, MP S-68-4, Waterways Experiment Station, Corps of Engineers, two clean sands and a fine gravel were placed around a wood screen having 3/16 in. (4.76 mm) slots. Discharges were measured before and after surging and the unclogged slot area was determined by observation after testing. The D_{50} size of the finer sand was 2.7 mm. and the unclogged slot area was 20% of the total. The coarser sand had a D_{50} size of 3.6 mm. and an unclogged slot area of 50%. The gravel had a D_{50} size of 4.7 mm. (about the slot width) and an unclogged slot area of 70%.



① From First Progress Report on Performance of Filter Materials, J. C. Gillou, Univ. of Ill., 1960. 8 in. dia. cmp., 5/16 in. perforations, assuming 16 per foot. Pipe in water only.

② Same as ① but with pipe in medium SP, gradation 1, Fig. A-2.

③ From WES TM 183-1, 1941. 6 in. dia. coated cmp., effective perforation dia. 3/16 in., 40 per foot. Pipe in coarse SP, gradation 2, Fig. A-2.

④ From Spindletop Research Report 580, 1967. 6 in. dia. clay pipe, 1/4 in. perforations, 44 per 3 ft. length, wall thickness 5/8 in. Pipe in GP equivalent to Indiana No. 7 Stone, gradation 3, Fig. A-2. Joints considered as perforations for curve.

Figure A-1. Head-discharge relationship for pipe perforations.

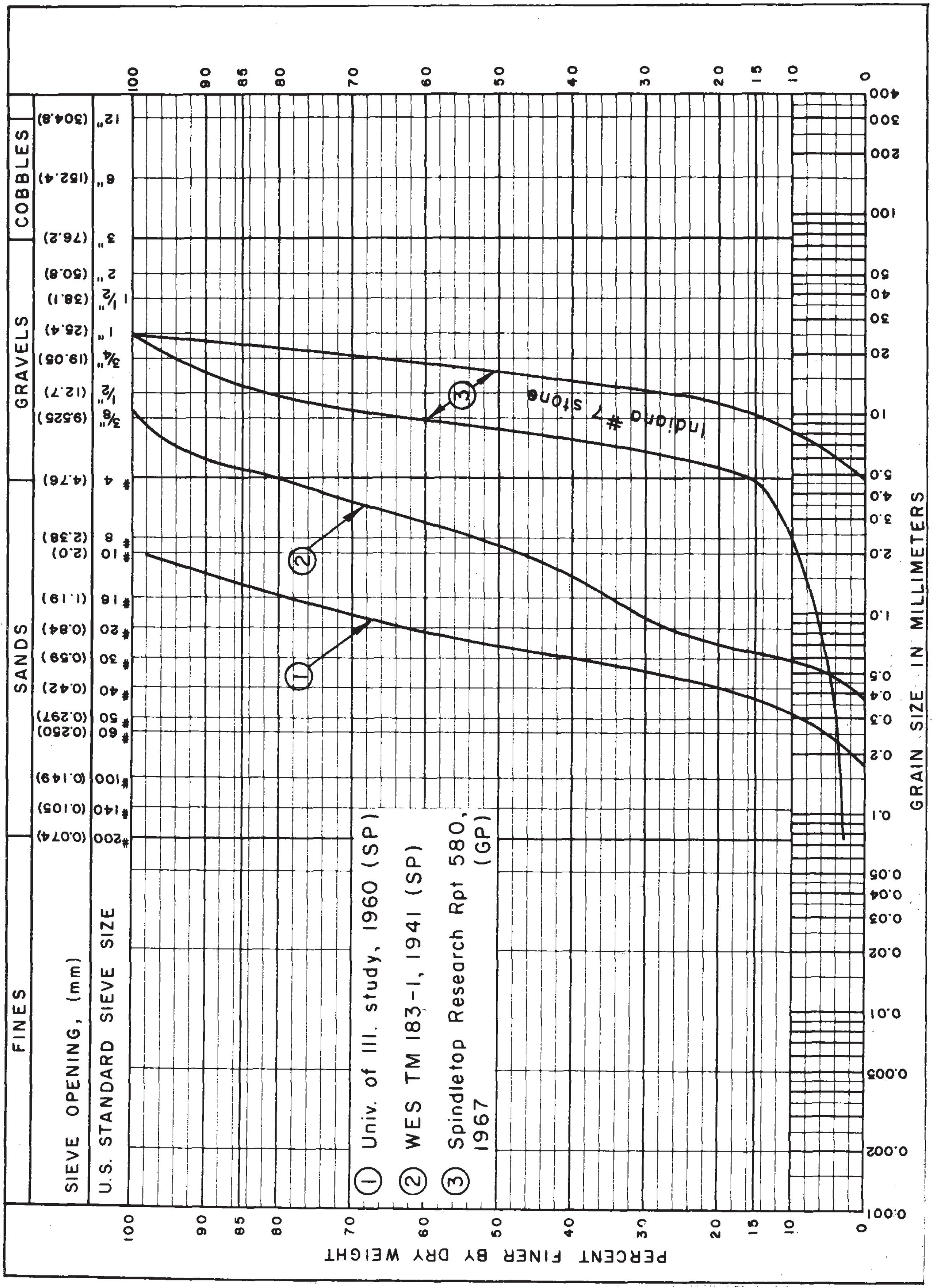


Figure A-2. Gradation of drain material used in the studies from which Fig. A-1 curves were developed.

Considering that only a few studies are available for this type of review and that these are not complete in every aspect, any procedure developed for estimating discharge into perforated pipe must necessarily be conservative. The above studies show that sands are more restrictive to flow through small openings than gravels. Therefore, the development that follows is limited to pipes placed in gravel drain material meeting the requirements that: (1) it will be virtually clean, (2) it will have a coefficient of uniformity less than 3, and (3) it will have a median or D_{50} size equal to or greater than the perforation diameter or slot width. Area or discharge reductions are made for conservatism: 70% for circular perforations and 40% for rectangular slots.

The area (A) per foot of pipe is given in Figure A-3 for 1/4 in., 5/16 in., and 3/8 in. diameter perforations. Flow quantity (q in cfs.) per foot of pipe can be estimated from Figure A-4 for circular perforations and from Figure A-5 for rectangular slots. The maximum orifice head considered is 2.0 feet since it is preferred that the water surface be maintained within the gravel drain material.

The flow equation for Figures A-3 and A-4 is:

$$q = CA_e\ (2gh)^{1/2} \quad \text{where}$$

q = discharge in cfs. per foot length of pipe

C = orifice coefficient (0.6 for circular perforations and 0.67 for rectangular slots).

A_e = effective area of openings per foot length of pipe (0.3A for circular perforations and 0.6A for rectangular slots, A being the non-restricted area). This correction is to account for blockage of openings by sand and gravel particles.

Note: Computations for discharge quantity curves included a conversion from square inches to square feet.

h = head over the orifice in feet.

ES-97 of NEH Section 5, Hydraulics, is recommended for estimating flow conditions within the pipe.

When high design discharges are involved and multiple outlets are not practical, more than one perforated pipe may be used to satisfy either the inflow (orifice) condition or the pipe flow condition.

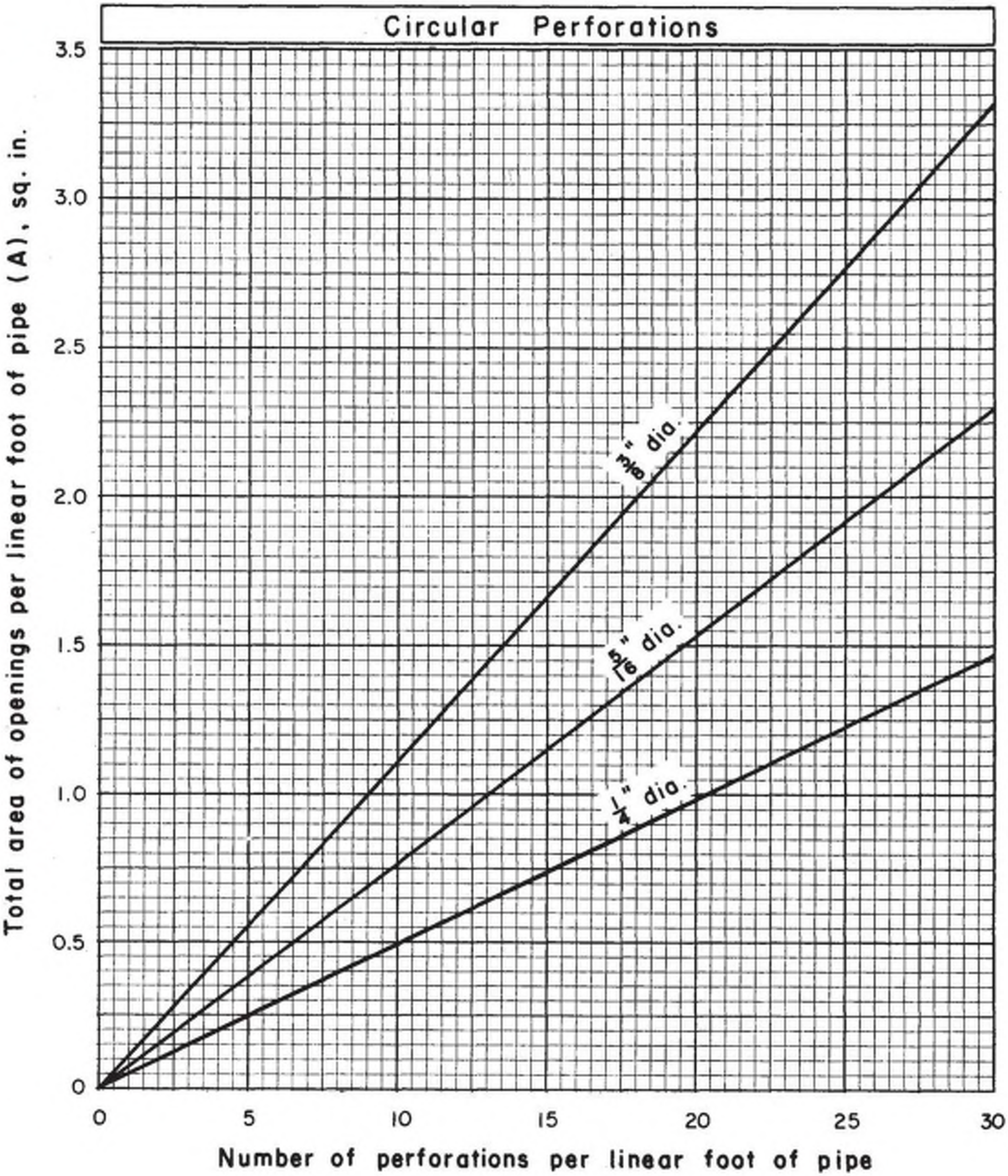


Figure A-3: Total area of circular perforations per foot length of pipe.

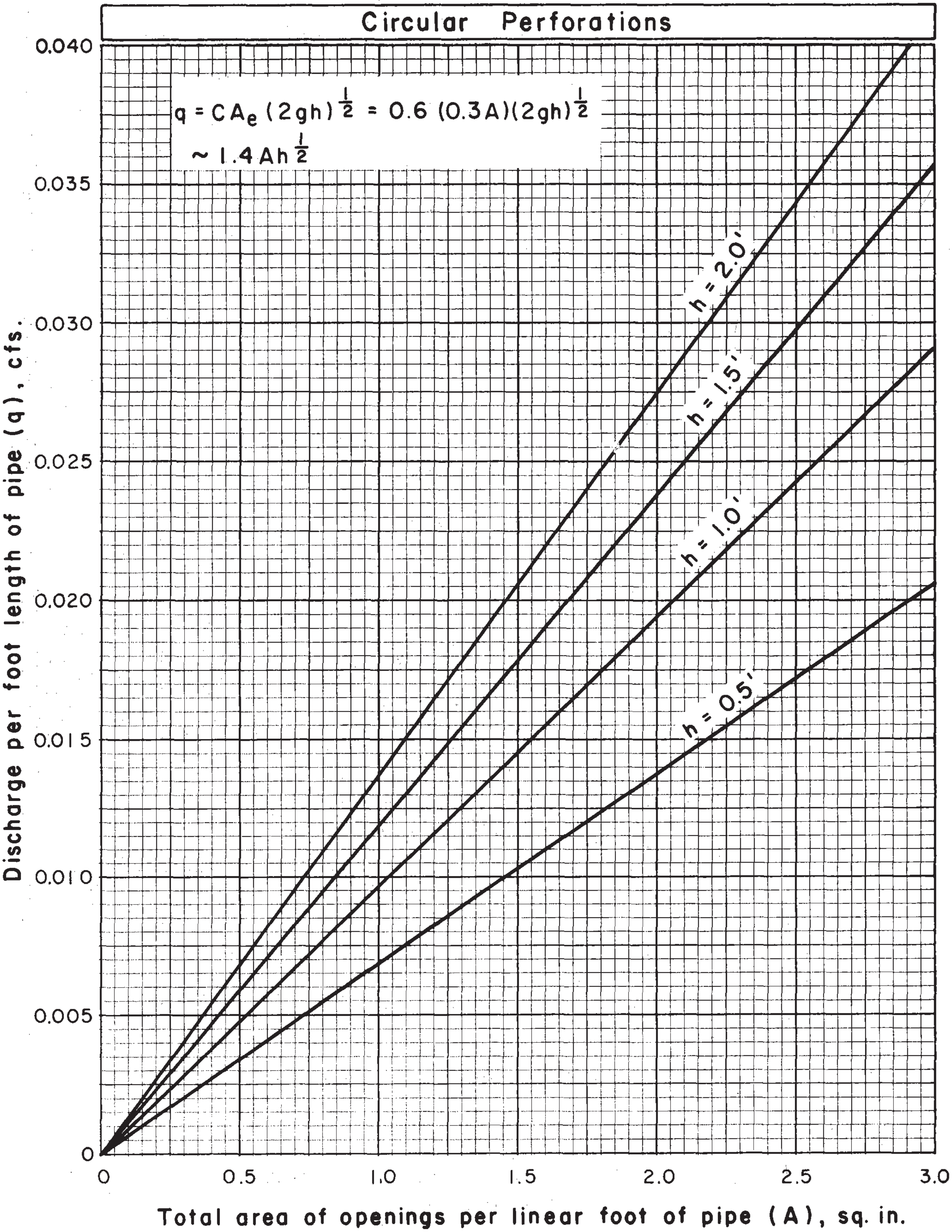


Figure A-4: Flow into pipe with circular perforations.

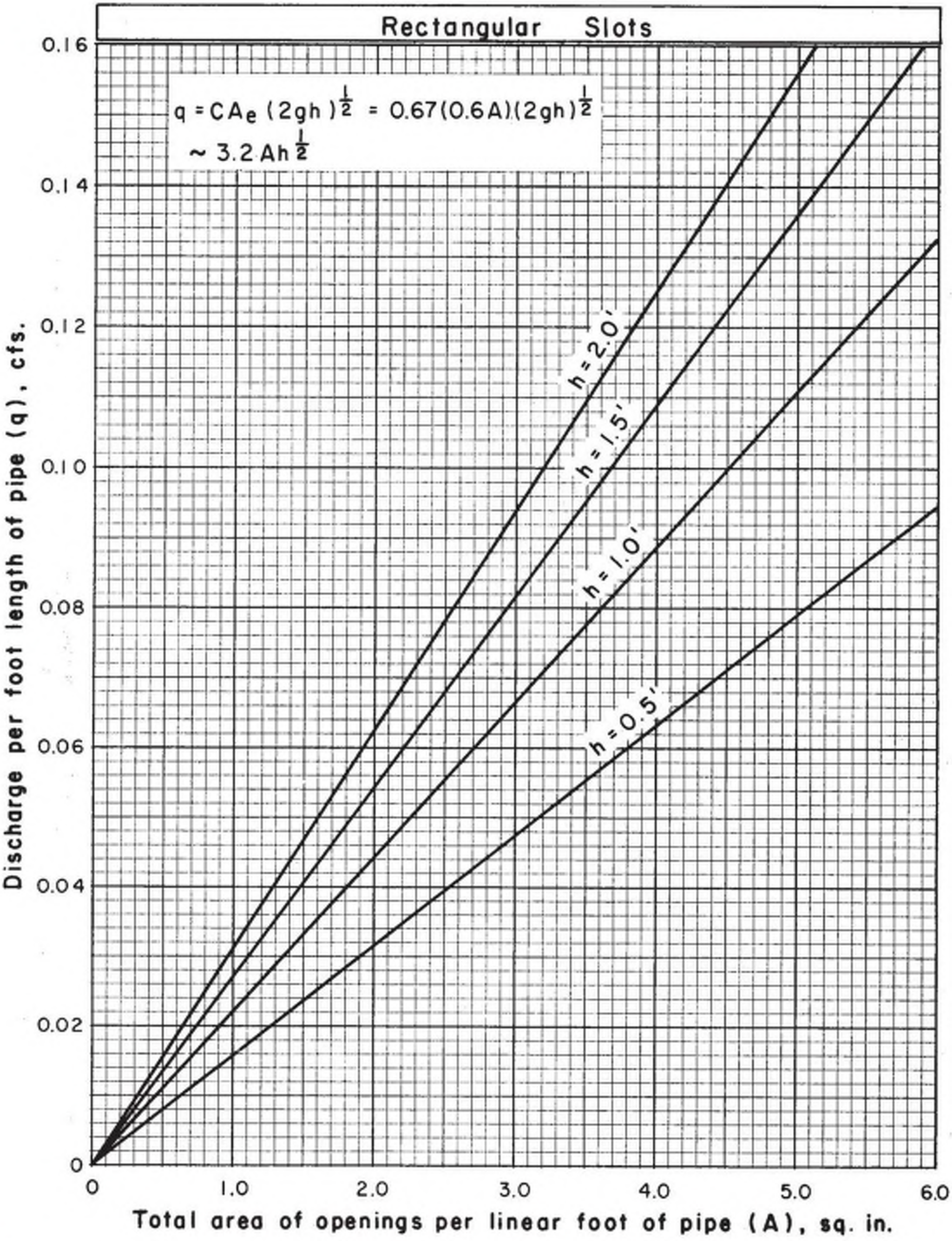


Figure A-5: Flow into pipe with rectangular slots.

Filter Compatibility Analysis

Project: Erin Lake Dam
Originator: TKW
Date: 5/8/2024
Reviewed by: LD
Date: 5/9/2024

CALCULATION PACKAGE: Filter Compatibility

Filter Compatibility

OBJECTIVE:

Determine the filter compatibility of proposed drain fill material with embankment and foundation soils.

REFERENCES:

1. ASTM C33/C33M Standard Specification for Concrete Aggregates (2018)
2. Georgia Department of Transportation, Standard Specifications Construction of Transportation Systems (2021)
3. USACE, EM 1110-2-2300 General Design and Construction Considerations for Earth and Rock-Fill Dams (2004)
4. ACCURA Geotechnical Data Report, Erin Lake Dam (2021).

METHOD:

Evaluate filter compatibility of the proposed drain fill material for use a seepage control measure with adjacent Embankment Fill and Alluvium soils at Erin Lake Dam. The purpose of the filter is to allow free movement of water with sufficient discharge capacity while retaining the protected materials.

Filter compatibility analysis is based on grain size distribution of a base and filter material to determine if a candidate filter material is of sufficient size to adequately allow seepage to free flow while maintaining the integrity of the base soil. The proposed filter material is ASTM C33 Fine Aggregate which will be utilized to filter seepage from Embankment Fill and Alluvial Soils. Analysis was also performed to evaluate compatibility between ASTM C33 Fine Aggregate and ASTM No. 8 Coarse Aggregate. ASTM No. 8 Coarse Aggregate will be utilized as a drain material to transmit captured seepage once passed through the filter material. In addition, Georgia department of Transportation 10 NS Sand was analyzed as an alternate for the ASTM C33 Fine Aggregate. For this analysis, United States Army Corps of Engineers methodology was used as detailed in EM 1110-2-2300 (2004).

Gradations for the Embankment Fill were obtained from laboratory testing on samples collected from subsurface investigation performed in 2021 and detailed in the Geotechnical Data Report for Erin Lake Dam (2021). Results from borings AB-2 and AB-3 were utilized as they are representative of the soils adjacent to the filters.

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For filter analysis, the particle diameter for 15 percent passing is required. Given the percent of fines in the materials, estimations were required for the finer gradation sizes for the Embankment Fill and Alluvial Soil. The estimations were based on the laboratory gradations and engineering judgement and are identified in the following attached tables.

The following compatibility calculations were performed:

- Embankment Fill (Base) with ASTM C33 Fine Aggregate (filter)
- Alluvium (Base) with ASTM C33 Fine Aggregate (filter)
- ASTM C33 Fine Aggregate (filter) with ASTM No. 8 Coarse Aggregate
- Embankment Fill (Base) with GDOT 10 NS Sand
- Alluvium (Base) with GDOT 10 NS Sand

The gradations and analyses are provided in the following attached tables and spreadsheets.

RESULTS:

The results of the analysis shows the ASTM C33 Fine Aggregate and GDOT NS 10 Sand are compatible with the existing Embankment Fill and Alluvial soil at Erin Lake Dam. In addition, the ASTM C33 Fine Aggregate is compatible with ASTM No. 8 Coarse Aggregate.

Dam Material	Compatible For:	
	Filtration	Drainage
Embankment Fill to ASTM C33 Fine Aggregate	Yes	Yes
Alluvium to ASTMC33 Fine Aggregate	Yes	Yes
ASTM C33 Sand to ASTM C33 No. 8 Stone	Yes	Yes
Embankment Fill to GDOT 10 NS Sand	Yes	Yes
Alluvium to GDOT 10 NS Sand	Yes	Yes

Estimate the minimum and maximum percent passing from embankment fill gradations for use in the compatibility analysis. Use borings AB-2 through AB-3 (representative of the soil adjacent to the filter). Assume soil is not dispersive.

Erin Lake Dam-Filter Compatibility

diameter		**Embankment Fill Gradation Borings**						diameter		Minimum Percent Passing	Maximum Percent Passing
		AB-2	AB-2	AB-2	AB-2	AB-2	AB-3				
		2-4 FT	6-8 FT	16-18 FT	20-22 FT	26-28 FT	0-2 FT				
mm	inch							mm	inch		
75	3"	100	100	100	100	100	100	75	3"	100	100
50.8	2"	100	100	100	100	100	100	50.8	2"	100	100
37.5	1.5"	100	100	100	100	100	100	37.5	1.5"	100	100
25.4	1"	100	100	100	100	100	100	25.4	1"	100	100
19	3/4"	100	100	100	100	100	100	19	3/4"	100	100
12.7	1/2"	100	100	100	100	100	100	12.7	1/2"	100	100
9.51	3/8"	100	100	96.6	100	97.5	100	9.51	3/8"	96.6	100
4.75	#4	100	99.9	90.2	100	97.2	96.8	4.75	#4	90.2	100
2	#10	100	99	87.2	99.9	97.1	96.1	2	#10	87.2	100
	#20	96.7	93.7	80.9	96.5	93	91.9		#20	80.9	96.7
0.42	#40	83.9	79.1	68.5	83.9	77.3	79.3	0.42	#40	68.5	83.9
0.25	#60	70.6	65.8	58.1	71.9	62.8	67.4	0.25	#60	58.1	71.9
0.147	#100	55.7	52.3	49.8	61.9	50.3	56.1	0.147	#100	49.8	61.9
	#140	48.1	44.9	45.8	57	44.4	49.9		#140	44.4	57
0.074	#200	42.1	38.8	42.6	53.1	39.7	45.1	0.074	#200	38.8	53.1
0.02	N/A	27		28	35	26		0.02	N/A	26	35
0.01	N/A	21		26	30	23		0.01	N/A	21	30
0.002	N/A	11		19	22	14		0.002	N/A	11	22
0.001	N/A							0.001	N/A	8*	17*
0.0001	N/A							0.0001	N/A		5*

*Estimated

Base and Filter Material Project: Erin Lake Dam-Embankment Fill vs ASTM C33 Fine Aggregate Name: TKW Date: 5/9/24 12:09 PM

Base Material

Determine the gradation curves of the base soil. Use enough samples to define the range of grain size for the base soil. Design the filter gradation based on the base soil that requires the smallest D_{15F} size. If soil has particles larger than the #4 sieve, an adjusted gradation is calculated. Input values below for the base soil (original) gradation (in red):

Particle size (mm)	Sieve #	Base soil (original), % passing		Adjusted coarse boundary	Adjusted fine boundary
		(coarse boundary)	(fine boundary)		
75	-				
37.5	-			*(No adjustment needed)*	
12.7	-	100.0%			
9.5	-	96.6%			
4.75	4	90.2%	101.0%		
4.00	5				
3.35	6				
2.80	7				
2.36	8				
2.00	10	87.2%	100.0%		
1.70	12				
1.40	14				
1.18	16				
1.00	18				
0.850	20	80.9%	96.7%		
0.710	25				
0.600	30				
0.500	35				
0.425	40	68.5%	83.9%		
0.300	50				
0.250	60	58.1%	71.9%		
0.212	70				
0.180	80				
0.150	100	49.8%	61.9%		
0.125	120				
0.106	140	44.4%	57.0%		
0.090	170				
0.075	200	38.8%	53.1%		
0.053	270				
0.02	-	26.0%	35.0%		
0.01	-	21.0%	30.0%		
0.002	-	11.0%	22.0%		
0.001	-	8.0%	17.0%		
0.0005	-				
0.0004	-				
0.0001	-		5.0%		

Maximum % passing #200 after regrading (if any) = A = 53.1%

Candidate Filter

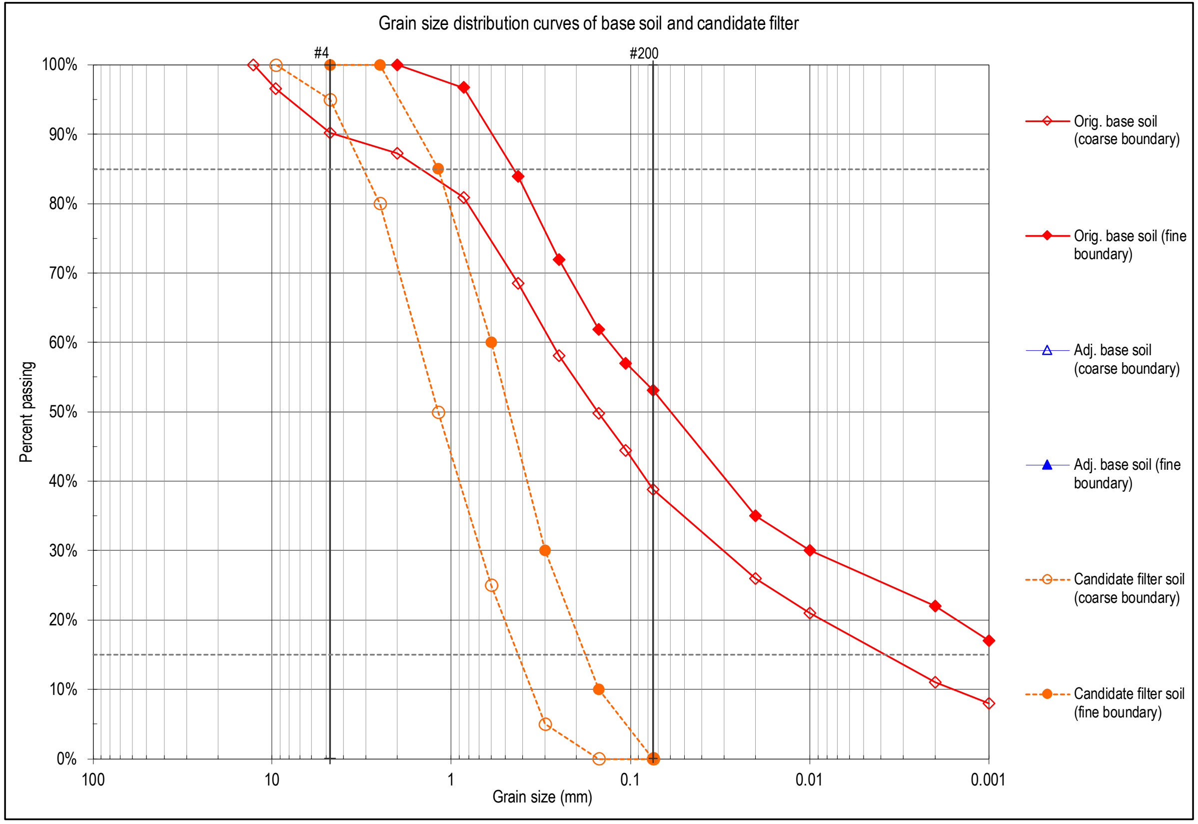
Candidate filter soil gradation. Values shown in red in the left column, and all values in the two right columns, can be changed.

Particle size *mm*	Sieve #	% Passing (coarse boundary)	% Passing (fine boundary)
150.0	-		
100.0	-		
90.0	-		
75.0	-		
63.0	-		
50.0	-		
37.5	-		
25.0	-		
19.0	-		
12.5	-		
9.5	-	100.0%	
4.75	4	95.0%	100.0%
3.35	6		
2.50	8	80.0%	100.0%
2.00	10		
1.70	12		
1.40	14		
1.18	16	50.0%	85.0%
0.850	20		
0.600	30	25.0%	60.0%
0.425	40		
0.300	50	5.0%	30.0%
0.250	60		
0.212	70		
0.180	80		
0.150	100	0.0%	10.0%
0.125	120		
0.106	140		
0.090	170		
0.075	200	(0.0%)	(0.0%)
0.053	270		
0.037	-		
0.019	-		
0.009	-		
0.005	-		
Candidate Filter Gradation	ASTM C33 - 02a Section 6.1, Fine Aggregate		

*Required entry values for base soil & candidate filter gradations:

1. Particle size for 100% passing.
2. % Passing the #4 sieve.
3. % Passing the #200 sieve.
4. Enough points to accurately represent the grain size distribution curve as straight lines between points. D_{85} and D_{15} sizes are interpolated from points on a log linear scale.
5. A zero % passing.
6. No duplicate entries; if D100<#4, enter 101% for #4 and 100% for appropriate size.

Gradation Curves



Properties of base soil	Coarse boundary	Fine Boundary
D_{85B} of original base soil =	1.483 mm	0.451 mm
D_{85B} of adjusted base soil =	--- mm	--- mm
D_{15B} of original base soil =	0.0038 mm	0.0007 mm
D_{15B} of adjusted base soil =	--- mm	--- mm
D_{60B} of original base soil =	0.275 mm	0.131 mm
D_{10B} of original base soil =	0.0016 mm	0.0003 mm
C_u of original base soil =	173.5	502.3

Properties of candidate filter soil (CF). D sizes are in mm:

	D_{85CF}	D_{15CF}	D_{60CF}	D_{10CF}	D_{30CF}	D_{90CF}	C_u	C_C
coarse boundary	3.10	0.42	1.52	0.36	0.69	3.84	4.25	0.87
fine boundary	1.18	0.18	0.60	0.15	0.30	1.52	4.00	1.00

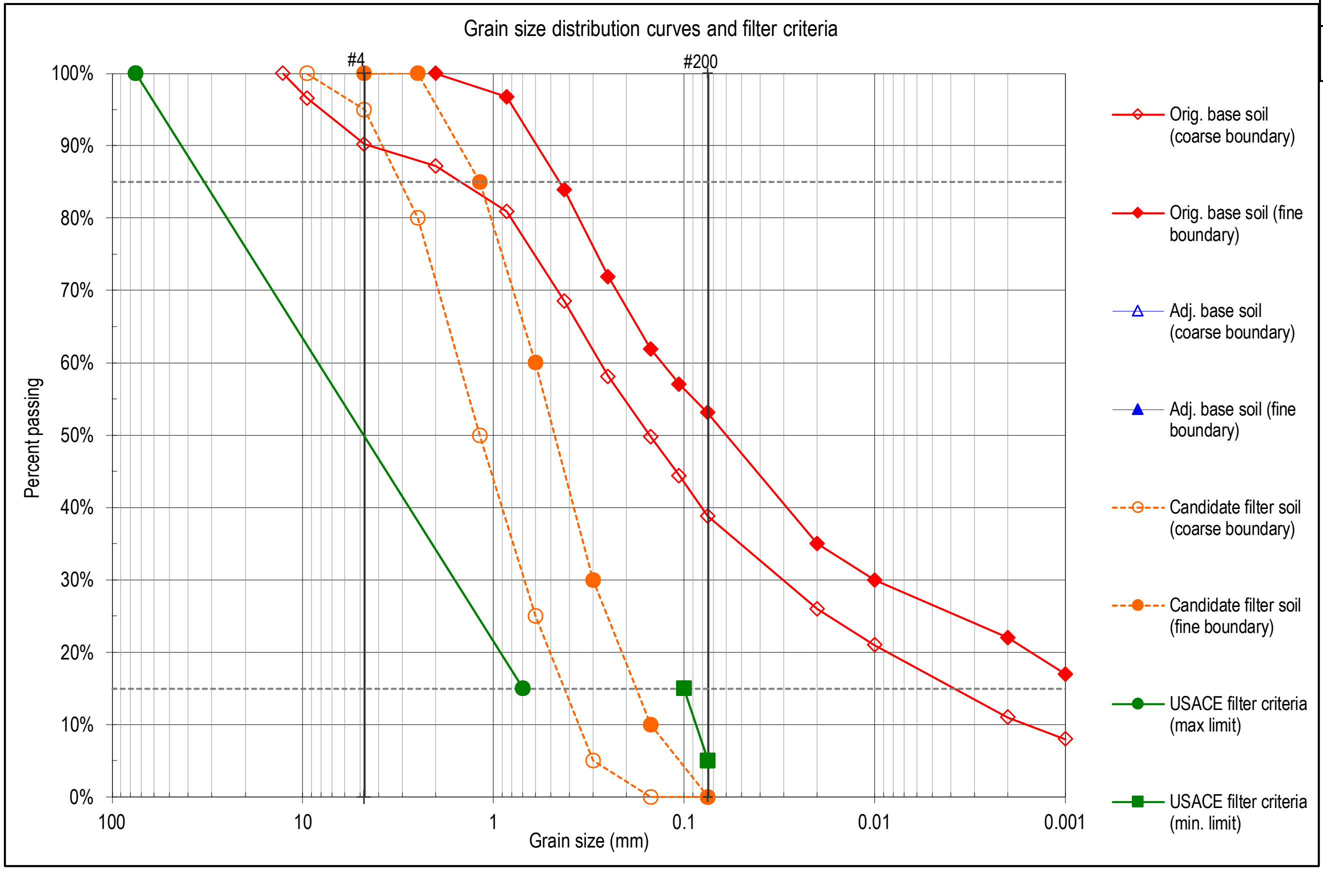
USACE Filter Criteria

Project: Erin Lake Dam-Embankment Fill vs ASTM C33 Fine Aggregate

Name: TKW

Date: 5/9/24 12:09 PM

Candidate Filter Gradation
ASTM C33 - 02a
Section 6.1, Fine Aggregate



Properties of base soil		Coarse boundary	Fine boundary
D_{85B}	of original base soil =	1.483 mm	0.451 mm
D_{85B}	of adjusted base soil =	--- mm	--- mm
D_{15B}	of original base soil =	0.0038 mm	0.0007 mm
D_{15B}	of adjusted base soil =	--- mm	--- mm
D_{60B}	of original base soil =	0.2754 mm	0.1311 mm
D_{10B}	of original base soil =	0.0016 mm	0.0003 mm
C_u	of original base soil =	173.5	502.3

USACE Filter Material

Filter criteria required by the US Army Corps of Engineers as published in EM 1110-2-2300 (31 Jul 94):

D_{85B} used in filter design	0.451
Maximum Passing #200 sieve of base soil	53.1%
Base soil category	2
Filter criteria (mm)	Maximum: $D_{15F} \le$ 0.70 to 0.70 To ensure sufficient permeability: Minimum: $D_{15F} \ge$ 0.10 to 0.10
Maximum particle size of filter (mm)	75
Maximum % passing # 200 sieve	5%
PI of material passing #40	0 when tested in accordance with EM 1110-2-1906

**If the base soil is in category 4, use the lower of the two 'max. D_{15F}' values when the filter is beneath riprap subject to wave action or beneath drains which may be subject to violent surging and/or vibration.

USACE filter gradation limits:

Maximum limit

Grain size (mm)	% Passing
75.00	100.0%
0.70	15.0%
0.70	15.0%

Minimum limit

Grain size (mm)	% Passing
0.10	15.0%
0.10	15.0%
0.075	5.0%

Acceptability of candidate filter (CF) soil:

USACE criteria	Coarse boundary	Fine boundary
Max % passing #200:	OK	OK
Max particle size (mm):	OK	OK
Maximum D_{15CF}:	OK	OK
Minimum D_{15CF} ($3 \times D_{15B}$):	OK	OK
Minimum D_{15CF} ($5 \times D_{15B}$):	OK	OK
To minimize segregation (from *Table B-3*)***		
Max allowable D_{90CF} =	20	OK
Max D_{90CF} =	3.84	

Filters should be relatively uniform (see the C_U value of the candidate filter soil). Also, filters should not be gap-graded.

*** Generally, this requirement is only necessary for coarse filters and gravel zones that serve as both filters and drains. For sand filters with D_{90} < ~20mm, these limitations are usually not necessary.

Estimate the minimum and maximum percent passing from embankment fill gradations for use in the compatibility analysis. Use borings AB-2 through AB-3 (representative of the soil adjacent to the filter). Assume soil is not dispersive.

Lake Erin Dam-Filter Compatibility								
diameter		**Alluvium Gradation Borings**			diameter		Minimum Percent Passing	Maximum Percent Passing
		AB-2	AB-3					
		30-32 FT	6-8 FT					
mm	inch				mm	inch		
75	3"	100	100		75	3"	100	100
50.8	2"	100	100		50.8	2"	100	100
37.5	1.5"	100	100		37.5	1.5"	100	100
25.4	1"	100	100		25.4	1"	100	100
19	3/4"	100	100		19	3/4"	100	100
12.7	1/2"	100	100		12.7	1/2"	100	100
9.51	3/8"	100	100		9.51	3/8"	100	100
4.75	#4	100	100		4.75	#4	100	100
2	#10	99.6	100		2	#10	99.6	100
	#20	97.2	98.1			#20	97.2	98.1
0.42	#40	89	88.4		0.42	#40	88.4	89
0.25	#60	68.8	76		0.25	#60	68.8	76
0.147	#100	42.8	62.1		0.147	#100	42.8	62.1
	#140	30.3	53.6			#140	30.3	53.6
0.074	#200	22.6	47.6		0.074	#200	22.6	47.6
0.02	N/A		33		0.02	N/A		33
0.01	N/A		27		0.01	N/A	13*	27
0.002	N/A		16.5		0.002	N/A		16.5
0.001	N/A				0.001	N/A	10*	14*
0.0001	N/A				0.0001	N/A		8*

*Estimated

Base and Filter Material Project: Erin Lake Dam-Alluvium vs ASTM C33 Fine Aggregate Name: TKW Date: 5/9/24 11:26 AM

Base Material

Determine the gradation curves of the base soil. Use enough samples to define the range of grain size for the base soil. Design the filter gradation based on the base soil that requires the smallest D_{15F} size. If soil has particles larger than the #4 sieve, an adjusted gradation is calculated. Input values below for the base soil (original) gradation (in red):

Particle size (mm)	Sieve #	Base soil (original), % passing (coarse boundary)	Base soil (original), % passing (fine boundary)	Adjusted coarse boundary	Adjusted fine boundary
75	-				
37.5	-			*(No adjustment needed)*	
12.7	-				
9.5	-				
4.75	4	100.0%	101.0%		
4.00	5				
3.35	6				
2.80	7				
2.36	8				
2.00	10	99.6%	100.0%		
1.70	12				
1.40	14				
1.18	16				
1.00	18				
0.850	20	97.2%	98.1%		
0.710	25				
0.600	30				
0.500	35				
0.425	40	88.4%	89.0%		
0.300	50				
0.250	60	68.8%	76.0%		
0.212	70				
0.180	80				
0.150	100	42.8%	62.1%		
0.125	120				
0.106	140	30.3%	53.6%		
0.090	170				
0.075	200	22.6%	47.6%		
0.053	270				
0.02	-		33.0%		
0.01	-	13.0%	27.0%		
0.002	-		16.5%		
0.001	-	10.0%	14.0%		
0.0004	-				
0.0003	-				
0.0001	-		8.0%		

Maximum % passing #200 after regrading (if any) = A = 47.6%

Candidate Filter

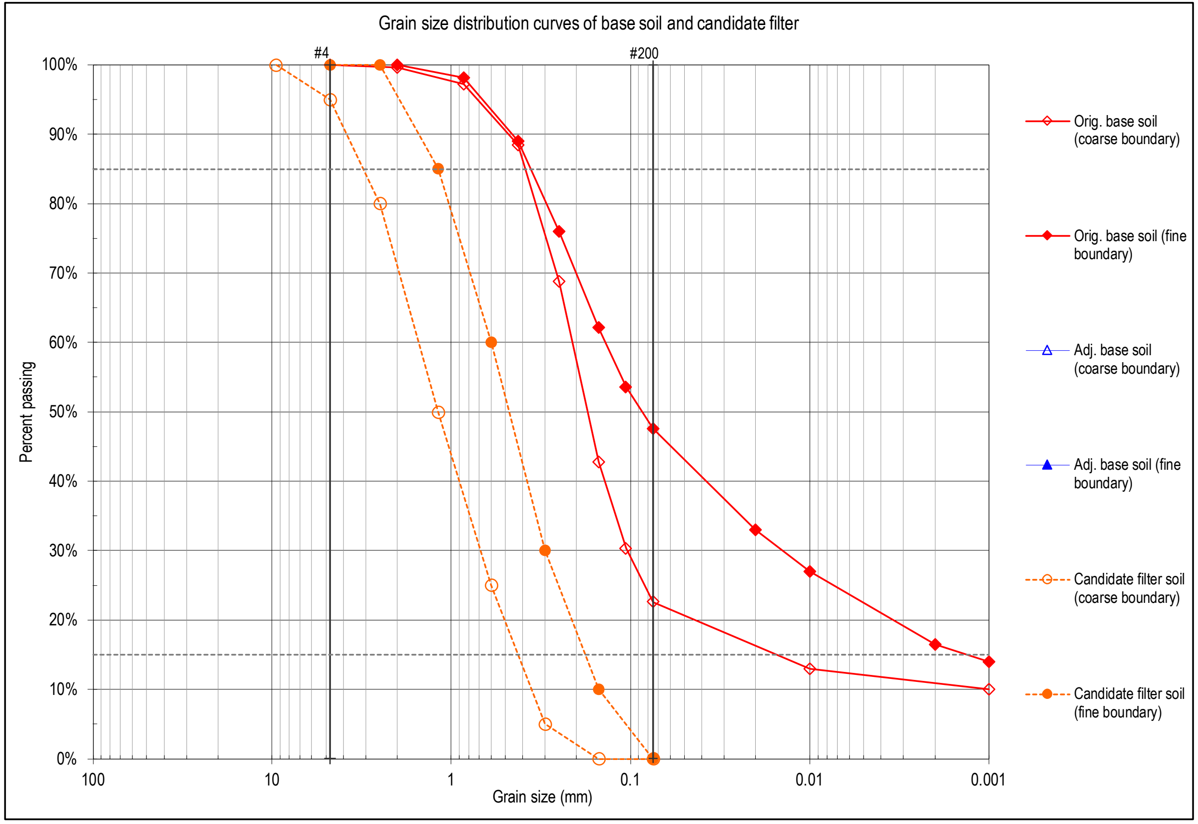
Candidate filter soil gradation. Values shown in red in the left column, and all values in the two right columns, can be changed.

Particle size *mm*	Sieve #	% Passing (coarse boundary)	% Passing (fine boundary)
150.0	-		
100.0	-		
90.0	-		
75.0	-		
63.0	-		
50.0	-		
37.5	-		
25.0	-		
19.0	-		
12.5	-		
9.5	-	100.0%	
4.75	4	95.0%	100.0%
3.35	6		
2.50	8	80.0%	100.0%
2.00	10		
1.70	12		
1.40	14		
1.18	16	50.0%	85.0%
0.850	20		
0.600	30	25.0%	60.0%
0.425	40		
0.300	50	5.0%	30.0%
0.250	60		
0.212	70		
0.180	80		
0.150	100	0.0%	10.0%
0.125	120		
0.106	140		
0.090	170		
0.075	200	(0.0%)	(0.0%)
0.053	270		
0.037	-		
0.019	-		
0.009	-		
0.005	-		
Candidate Filter Gradation	ASTM C33 - 02a Section 6.1, Fine Aggregate		

*Required entry values for base soil & candidate filter gradations:

1. Particle size for 100% passing.
2. % Passing the #4 sieve.
3. % Passing the #200 sieve.
4. Enough points to accurately represent the grain size distribution curve as straight lines between points. D_{85} and D_{15} sizes are interpolated from points on a log linear scale.
5. A zero % passing.
6. No duplicate entries; if D100<#4, enter 101% for #4 and 100% for appropriate size

Gradation Curves



Properties of base soil	Coarse boundary	Fine Boundary
D_{85B} of original base soil =	0.388 mm	0.361 mm
D_{85B} of adjusted base soil =	--- mm	--- mm
D_{15B} of original base soil =	0.0152 mm	0.0013 mm
D_{15B} of adjusted base soil =	--- mm	--- mm
D_{60B} of original base soil =	0.210 mm	0.138 mm
D_{10B} of original base soil =	0.0010 mm	0.0002 mm
C_u of original base soil =	210.3	639.0

Properties of candidate filter soil (CF). D sizes are in mm:

	D_{85CF}	D_{15CF}	D_{60CF}	D_{10CF}	D_{30CF}	D_{90CF}	C_u	C_C
coarse boundary	3.10	0.42	1.52	0.36	0.69	3.84	4.25	0.87
fine boundary	1.18	0.18	0.60	0.15	0.30	1.52	4.00	1.00

USACE Filter Criteria

Project: Erin Lake Dam-Alluvium vs ASTM C33 Fine Aggregate

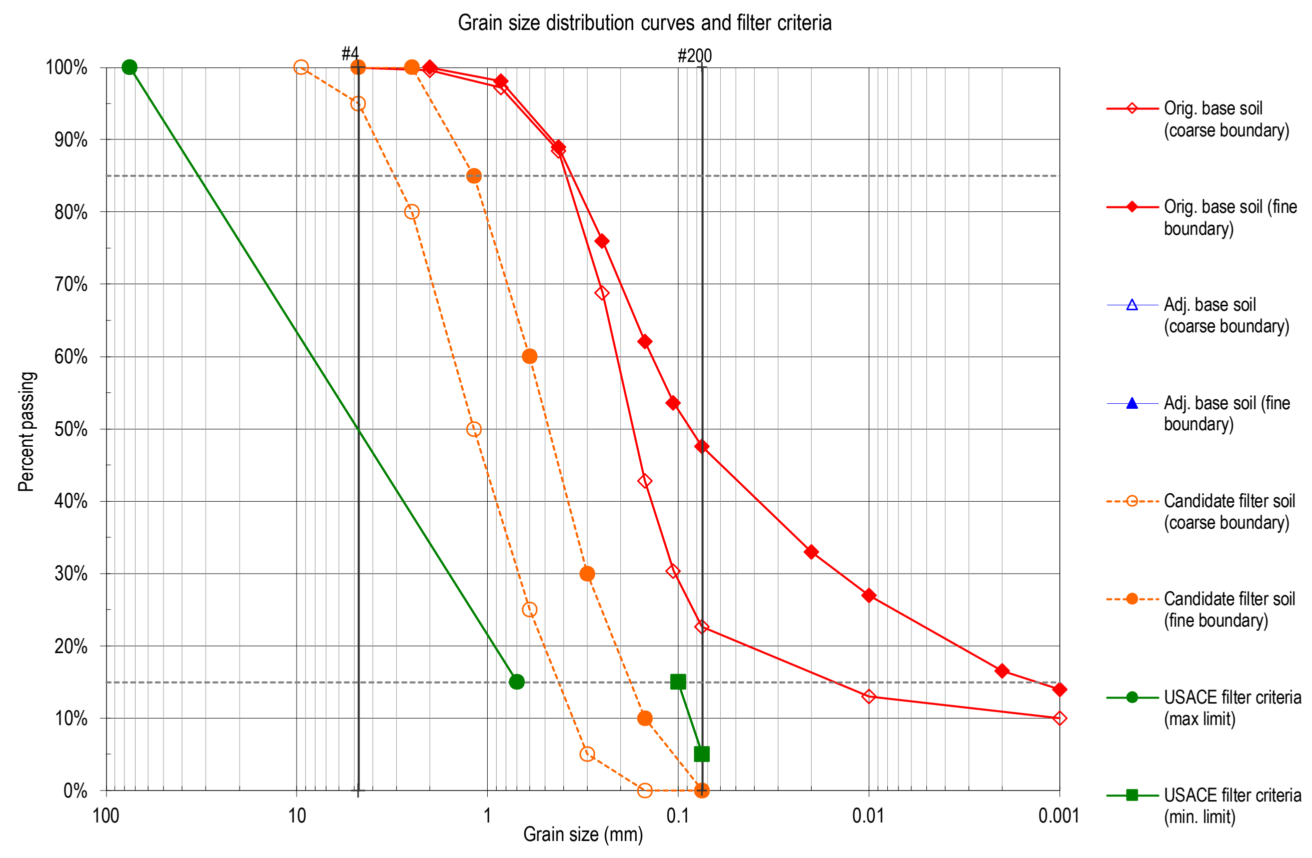
Name: TKW

Date: 5/9/24 11:24 AM

Candidate Filter Gradation

ASTM C33 - 02a

Section 6.1, Fine Aggregate



Properties of base soil		Coarse boundary	Fine boundary
D_{85B}	of original base soil =	0.388 mm	0.361 mm
D_{85B}	of adjusted base soil =	--- mm	--- mm
D_{15B}	of original base soil =	0.0152 mm	0.0013 mm
D_{15B}	of adjusted base soil =	--- mm	--- mm
D_{60B}	of original base soil =	0.2103 mm	0.1377 mm
D_{10B}	of original base soil =	0.0010 mm	0.0002 mm
C_u	of original base soil =	210.3	639.0

USACE Filter Material

Filter criteria required by the US Army Corps of Engineers as published in EM 1110-2-2300 (31 Jul 94):

D_{85B} used in filter design	0.361
Maximum Passing #200 sieve of base soil	47.6%
Base soil category	2
Filter criteria (mm)	Maximum: $D_{15F} \leq$ 0.70 to 0.70 To ensure sufficient permeability: Minimum: $D_{15F} \geq$ 0.10 to 0.10
Maximum particle size of filter (mm)	75
Maximum % passing # 200 sieve	5%
PI of material passing #40	0 when tested in accordance with EM 1110-2-1906

**If the base soil is in category 4, use the lower of the two 'max. D_{15F}' values when the filter is beneath riprap subject to wave action or beneath drains which may be subject to violent surging and/or vibration.

USACE filter gradation limits:

Maximum limit

Grain size (mm)	% Passing
75.00	100.0%
0.70	15.0%
0.70	15.0%

Minimum limit

Grain size (mm)	% Passing
0.10	15.0%
0.10	15.0%
0.075	5.0%

Acceptability of candidate filter (CF) soil:

USACE criteria	Coarse boundary	Fine boundary
Max % passing #200:	OK	OK
Max particle size (mm):	OK	OK
Maximum D_{15CF}:	OK	OK
Minimum D_{15CF} ($3 \times D_{15B}$):	OK	OK
Minimum D_{15CF} ($5 \times D_{15B}$):	OK	OK
To minimize segregation (from *Table B-3*)***		
Max allowable D_{90CF} =	20	OK
Max D_{90CF} =	3.84	

Filters should be relatively uniform (see the C_U value of the candidate filter soil). Also, filters should not be gap-graded.

*** Generally, this requirement is only necessary for coarse filters and gravel zones that serve as both filters and drains. For sand filters with D_{90} < ~20mm, these limitations are usually not necessary.

Base and Filter Material Project: Erin Lake Dam-ASTM C33 Fine Aggregate vs ASTM #8 Coarse Aggregate Name: TKW Date: 5/7/24 10:50 AM

Base Material

Determine the gradation curves of the base soil. Use enough samples to define the range of grain size for the base soil. Design the filter gradation based on the base soil that requires the smallest D_{15F} size. If soil has particles larger than the #4 sieve, an adjusted gradation is calculated. Input values below for the base soil (original) gradation (in red):

Particle size (mm)	Sieve #	Base soil (original), % passing (coarse boundary)	Base soil (original), % passing (fine boundary)	Adjusted coarse boundary	Adjusted fine boundary
75	-				
37.5	-			*(No adjustment needed)*	
12.7	-				
9.5	-	100.0%			
4.75	4	95.0%	100.0%		
4.00	5				
3.35	6				
2.80	7				
2.36	8	89.0%	100.0%		
2.00	10				
1.70	12				
1.40	14				
1.18	16	50.0%	85.0%		
1.00	18				
0.850	20				
0.710	25				
0.600	30	25.0%	60.0%		
0.500	35				
0.425	40				
0.300	50	5.0%	30.0%		
0.250	60				
0.212	70				
0.180	80				
0.150	100	0.0%	10.0%		
0.125	120				
0.106	140				
0.090	170				
0.075	200	0.0%	0.0%		
0.053	270				
0.004	-				
0.002	-				
0.002	-				
0.005	-				
0.003	-				
0.002	-				
0.001	-				

Maximum % passing #200 after regrading (if any) = A = 0.0%

Candidate Filter

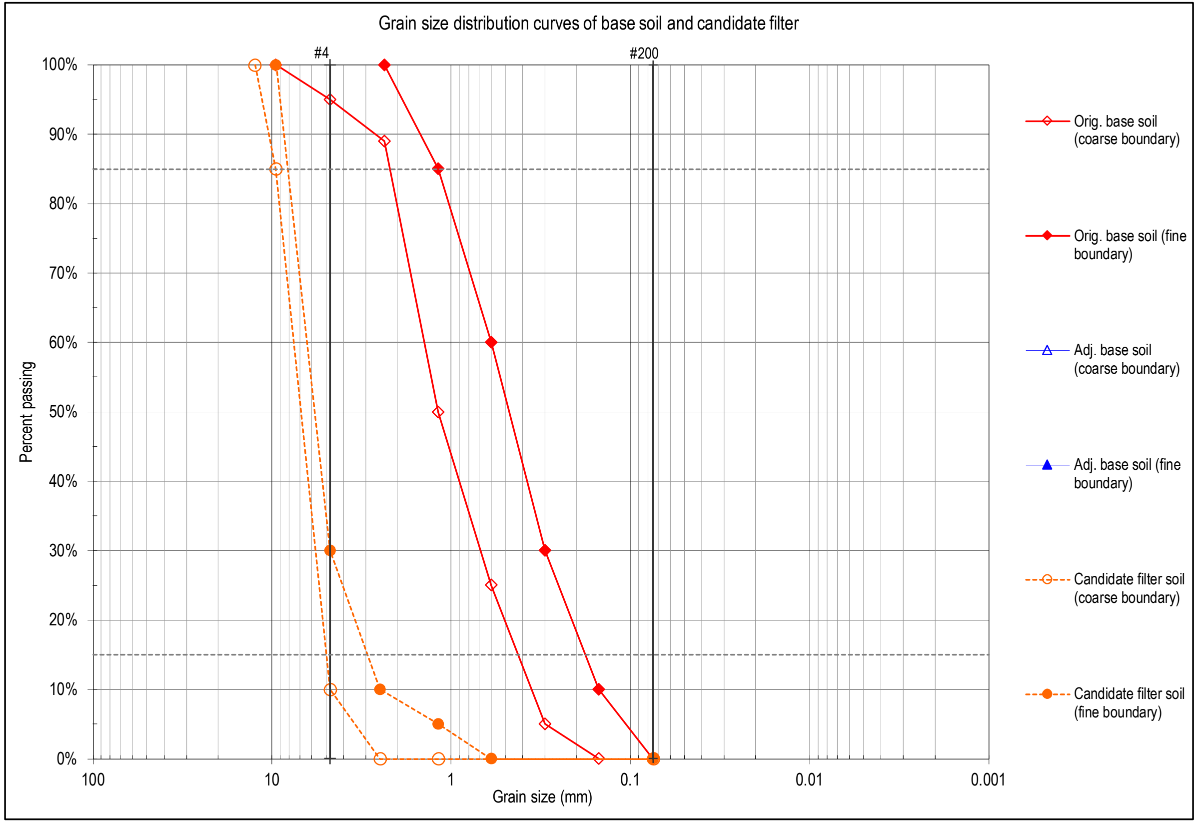
Candidate filter soil gradation. Values shown in red in the left column, and all values in the two right columns, can be changed.

Particle size *mm*	Sieve #	% Passing (coarse boundary)	% Passing (fine boundary)
150.0	-		
100.0	-		
90.0	-		
75.0	-		
63.0	-		
50.0	-		
37.5	-		
25.0	-		
19.0	-		
12.5	-	100.0%	
9.5	-	85.0%	100.0%
4.75	4	10.0%	30.0%
3.35	6		
2.50	8	0.0%	10.0%
2.00	10		
1.70	12		
1.40	14		
1.18	16	0.0%	5.0%
0.850	20		
0.600	30		(0.0%)
0.425	40		
0.300	50		
0.250	60		
0.212	70		
0.180	80		
0.150	100		
0.125	120		
0.106	140		
0.090	170		
0.075	200	(0.0%)	(0.0%)
0.053	270		
0.037	-		
0.019	-		
0.009	-		
0.005	-		
Candidate Filter Gradation	ASTM C33 - 02a Table 2: Size # 8		

*Required entry values for base soil & candidate filter gradations:

1. Particle size for 100% passing.
2. % Passing the #4 sieve.
3. % Passing the #200 sieve.
4. Enough points to accurately represent the grain size distribution curve as straight lines between points. D_{85} and D_{15} sizes are interpolated from points on a log linear scale.
5. A zero % passing.
6. No duplicate entries; if D100<#4, enter 101% for #4 and 100% for appropriate size.

Gradation Curves



Properties of base soil	Coarse boundary	Fine Boundary
D_{85B} of original base soil =	2.198 mm	1.180 mm
D_{85B} of adjusted base soil =	--- mm	--- mm
D_{15B} of original base soil =	0.4243 mm	0.1784 mm
D_{15B} of adjusted base soil =	--- mm	--- mm
D_{60B} of original base soil =	1.410 mm	0.600 mm
D_{10B} of original base soil =	0.3568 mm	0.1500 mm
C_u of original base soil =	4.0	4.0

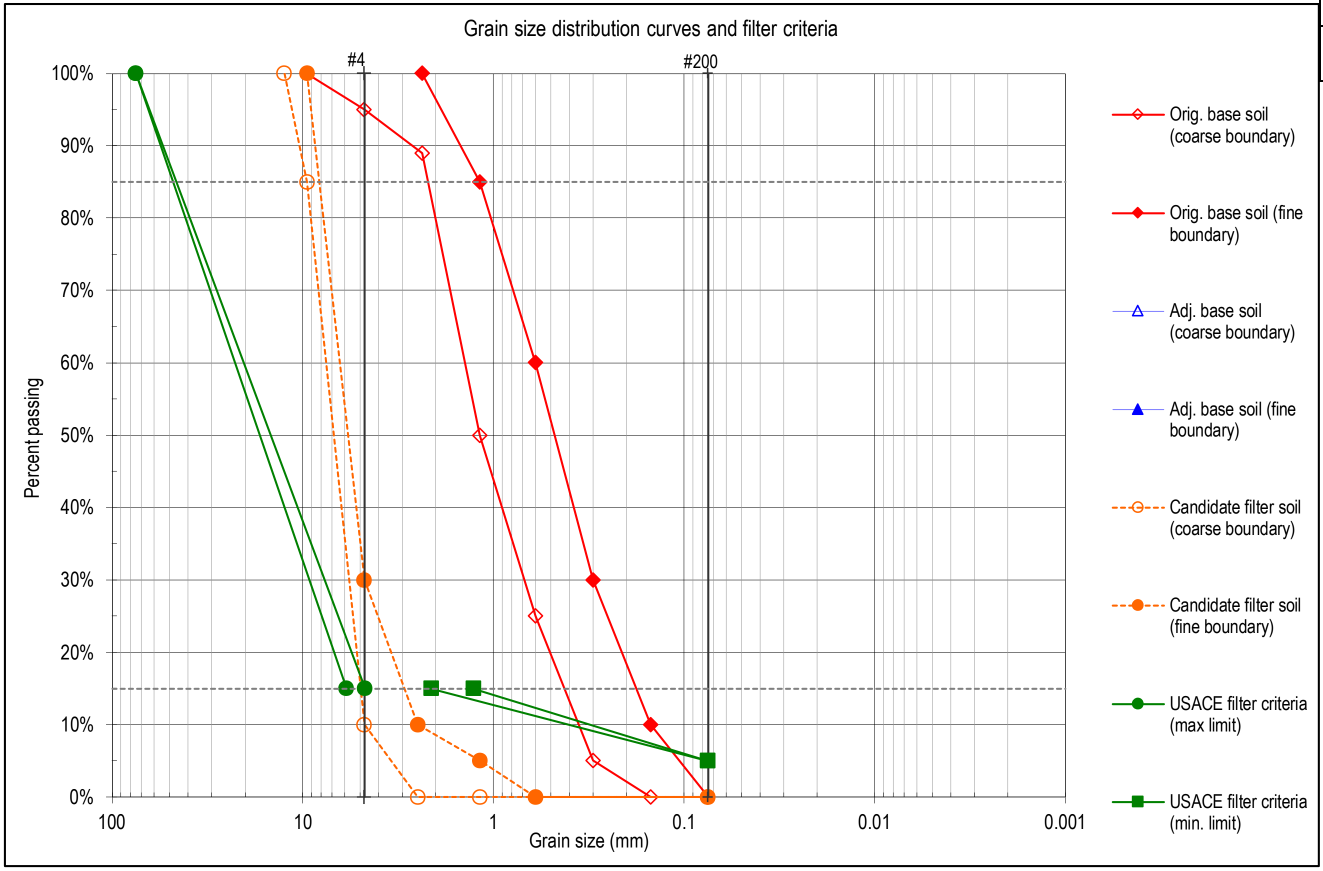
Properties of candidate filter soil (CF). D sizes are in mm:

	D_{85CF}	D_{15CF}	D_{60CF}	D_{10CF}	D_{30CF}	D_{90CF}	C_u	C_C
coarse boundary	9.50	4.97	7.54	4.75	5.71	10.41	1.59	0.91
fine boundary	8.19	2.94	6.39	2.50	4.75	8.60	2.56	1.41

USACE Filter Criteria

Project: Erin Lake Dam-ASTM C33 Fine Aggregate vs ASTM #8 Coarse Aggregate
Name: TKW
Date: 5/7/24 10:50 AM

Candidate Filter Gradation
ASTM C33 - 02a
Table 2: Size # 8



Properties of base soil		Coarse boundary	Fine boundary
D_{85B}	of original base soil =	2.198 mm	1.180 mm
D_{85B}	of adjusted base soil =	--- mm	--- mm
D_{15B}	of original base soil =	0.4243 mm	0.1784 mm
D_{15B}	of adjusted base soil =	--- mm	--- mm
D_{60B}	of original base soil =	1.4095 mm	0.6000 mm
D_{10B}	of original base soil =	0.3568 mm	0.1500 mm
C_u	of original base soil =	4.0	4.0

USACE Filter Material

Filter criteria required by the US Army Corps of Engineers as published in EM 1110-2-2300 (31 Jul 94):

D_{85B} used in filter design	1.180
Maximum Passing #200 sieve of base soil	0.0%
Base soil category	4**
Filter criteria (mm)	Maximum: $D_{15F} \leq$ 4.72 to 5.90 To ensure sufficient permeability: Minimum: $D_{15F} \geq$ 1.27 to 2.12
Maximum particle size of filter (mm)	75
Maximum % passing # 200 sieve	5%
PI of material passing #40	0 when tested in accordance with EM 1110-2-1906

**If the base soil is in category 4, use the lower of the two 'max. D_{15F}' values when the filter is beneath riprap subject to wave action or beneath drains which may be subject to violent surging and/or vibration.

USACE filter gradation limits:

Maximum limit	
Grain size (mm)	% Passing
75.00	100.0%
5.90	15.0%
4.72	15.0%

Minimum limit	
Grain size (mm)	% Passing
1.27	15.0%
2.12	15.0%
0.075	5.0%

Acceptability of candidate filter (CF) soil:

USACE criteria	Coarse boundary	Fine boundary
Max % passing #200:	OK	OK
Max particle size (mm):	OK	OK
Maximum D_{15CF}:	OK	OK
Minimum D_{15CF} ($3 \times D_{15B}$):	OK	OK
Minimum D_{15CF} ($5 \times D_{15B}$):	OK	OK
To minimize segregation (from *Table B-3*)***		
Max allowable D_{90CF} =	40	OK
Max D_{90CF} =	10.41	

Filters should be relatively uniform (see the C_U value of the candidate filter soil). Also, filters should not be gap-graded.

*** Generally, this requirement is only necessary for coarse filters and gravel zones that serve as both filters and drains. For sand filters with D_{90} < ~20mm, these limitations are usually not necessary.

Base and Filter Material Project: Erin Lake Dam-Embankment Fill vs GDOT 10 NS Sand Name: TKW Date: 5/9/24 2:06 PM

Base Material

Determine the gradation curves of the base soil. Use enough samples to define the range of grain size for the base soil. Design the filter gradation based on the base soil that requires the smallest D_{15F} size. If soil has particles larger than the #4 sieve, an adjusted gradation is calculated. Input values below for the base soil (original) gradation (in red):

Particle size (mm)	Sieve #	Base soil (original), % passing (coarse boundary)	(fine boundary)	Adjusted coarse boundary	Adjusted fine boundary
75	-				
37.5	-			*(No adjustment needed)*	
12.7	-	100.0%			
9.5	-	96.6%			
4.75	4	90.2%	101.0%		
4.00	5				
3.35	6				
2.80	7				
2.36	8				
2.00	10	87.2%	100.0%		
1.70	12				
1.40	14				
1.18	16				
1.00	18				
0.850	20	80.9%	96.7%		
0.710	25				
0.600	30				
0.500	35				
0.425	40	68.5%	83.9%		
0.300	50				
0.250	60	58.1%	71.9%		
0.212	70				
0.180	80				
0.150	100	49.8%	61.9%		
0.125	120				
0.106	140	44.4%	57.0%		
0.090	170				
0.075	200	38.8%	53.1%		
0.053	270				
0.02	-	26.0%	35.0%		
0.01	-	21.0%	30.0%		
0.002	-	11.0%	22.0%		
0.001	-	8.0%	17.0%		
0.0005	-				
0.0004	-				
0.0001	-		5.0%		

Maximum % passing #200 after regrading (if any) = A = 53.1%

Candidate Filter

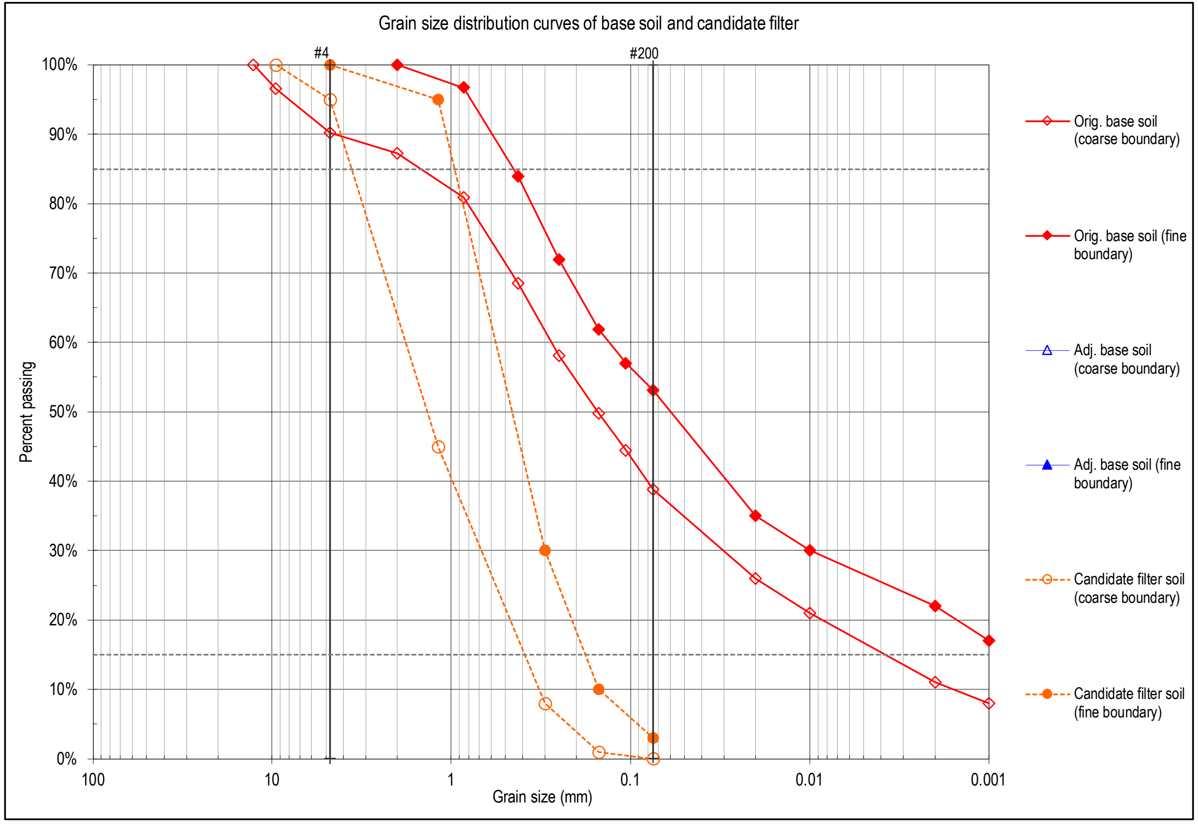
Candidate filter soil gradation. Values shown in red in the left column, and all values in the two right columns, can be changed.

Particle size *mm*	Sieve #	% Passing (coarse boundary)	% Passing (fine boundary)
150.0	-		
100.0	-		
90.0	-		
75.0	-		
63.0	-		
50.0	-		
37.5	-		
25.0	-		
19.0	-		
12.5	-		
9.5	-	100.0%	
4.75	4	95.0%	100.0%
3.35	6		
2.50	8		
2.00	10		
1.70	12		
1.40	14		
1.18	16	45.0%	95.0%
0.850	20		
0.600	30		
0.425	40		
0.300	50	8.0%	30.0%
0.250	60		
0.212	70		
0.180	80		
0.150	100	1.0%	10.0%
0.125	120		
0.106	140		
0.090	170		
0.075	200	0.0%	3.0%
0.053	270		
0.037	-		
0.019	-		
0.009	-		
0.005	-		
Candidate Filter Gradation	User Defined Gradation		

*Required entry values for base soil & candidate filter gradations:

1. Particle size for 100% passing.
2. % Passing the #4 sieve.
3. % Passing the #200 sieve.
4. Enough points to accurately represent the grain size distribution curve as straight lines between points. D_{85} and D_{15} sizes are interpolated from points on a log linear scale.
5. A zero % passing.
6. No duplicate entries; if D100<#4, enter 101% for #4 and 100% for appropriate size.

Gradation Curves



Properties of base soil	Coarse boundary	Fine Boundary
D_{85B} of original base soil =	1.483 mm	0.451 mm
D_{85B} of adjusted base soil =	--- mm	--- mm
D_{15B} of original base soil =	0.0038 mm	0.0007 mm
D_{15B} of adjusted base soil =	--- mm	--- mm
D_{60B} of original base soil =	0.275 mm	0.131 mm
D_{10B} of original base soil =	0.0016 mm	0.0003 mm
C_u of original base soil =	173.5	502.3

Properties of candidate filter soil (CF). D sizes are in mm:

	D_{85CF}	D_{15CF}	D_{60CF}	D_{10CF}	D_{30CF}	D_{90CF}	C_u	C_C
coarse boundary	3.60	0.39	1.79	0.32	0.68	4.13	5.55	0.79
fine boundary	0.96	0.18	0.56	0.15	0.30	1.06	3.76	1.06

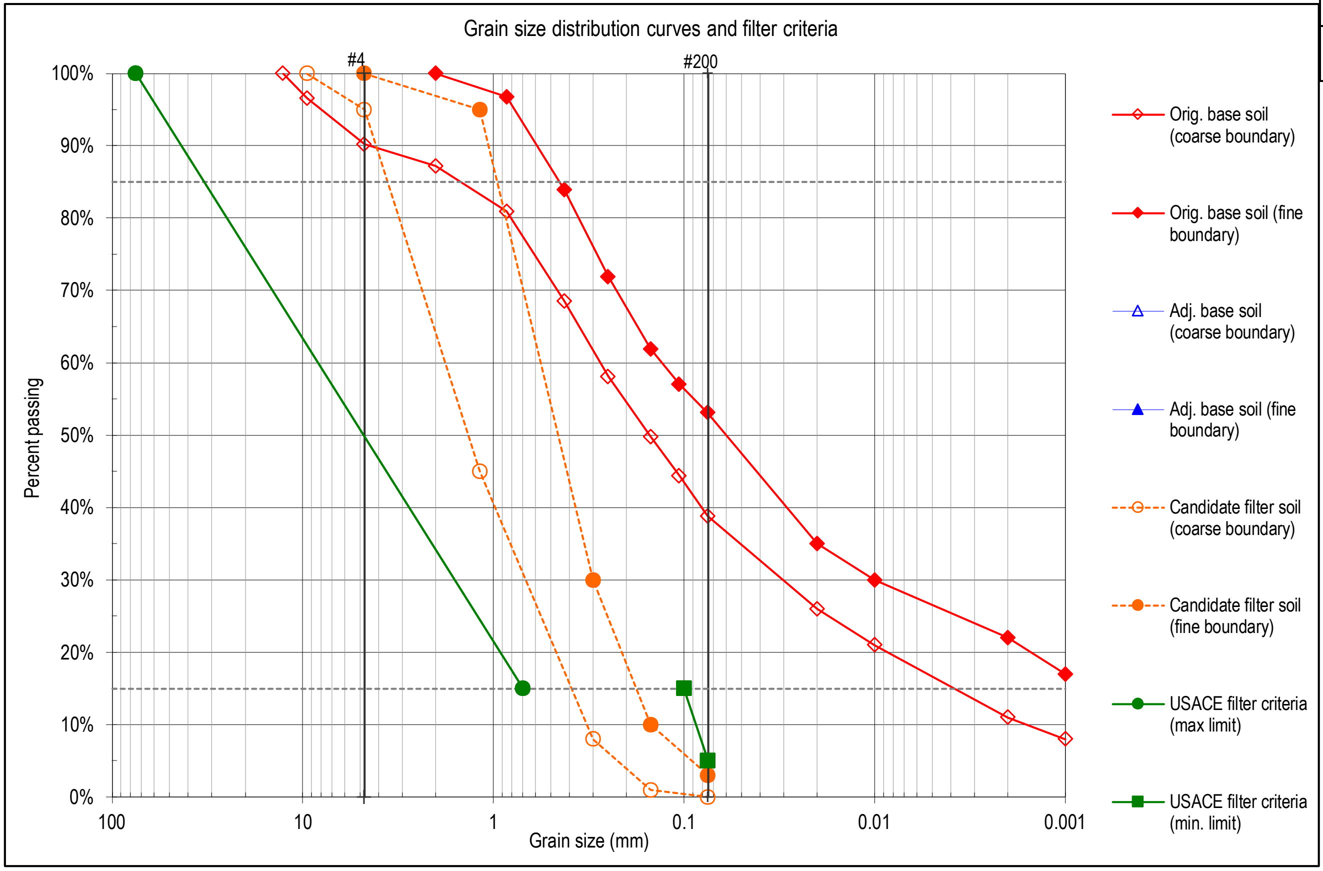
USACE Filter Criteria

Project: Erin Lake Dam-Embankment Fill vs GDOT 10 NS Sand

Name: TKW

Date: 5/9/24 2:06 PM

Candidate Filter Gradation
User Defined Gradation



Properties of base soil		Coarse boundary	Fine boundary
D_{85B}	of original base soil =	1.483 mm	0.451 mm
D_{85B}	of adjusted base soil =	--- mm	--- mm
D_{15B}	of original base soil =	0.0038 mm	0.0007 mm
D_{15B}	of adjusted base soil =	--- mm	--- mm
D_{60B}	of original base soil =	0.2754 mm	0.1311 mm
D_{10B}	of original base soil =	0.0016 mm	0.0003 mm
C_u	of original base soil =	173.5	502.3

USACE Filter Material

Filter criteria required by the US Army Corps of Engineers as published in EM 1110-2-2300 (31 Jul 94):

D_{85B} used in filter design	0.451
Maximum Passing #200 sieve of base soil	53.1%
Base soil category	2
Filter criteria (mm)	Maximum: $D_{15F} \leq$ 0.70 to 0.70 To ensure sufficient permeability: Minimum: $D_{15F} \geq$ 0.10 to 0.10
Maximum particle size of filter (mm)	75
Maximum % passing # 200 sieve	5%
PI of material passing #40	0 when tested in accordance with EM 1110-2-1906

**If the base soil is in category 4, use the lower of the two 'max. D_{15F}' values when the filter is beneath riprap subject to wave action or beneath drains which may be subject to violent surging and/or vibration.

USACE filter gradation limits:

Maximum limit	
Grain size (mm)	% Passing
75.00	100.0%
0.70	15.0%
0.70	15.0%

Minimum limit	
Grain size (mm)	% Passing
0.10	15.0%
0.10	15.0%
0.075	5.0%

Acceptability of candidate filter (CF) soil:

USACE criteria	Coarse boundary	Fine boundary
Max % passing #200:	OK	OK
Max particle size (mm):	OK	OK
Maximum D_{15CF}:	OK	OK
Minimum D_{15CF} ($3 \times D_{15B}$):	OK	OK
Minimum D_{15CF} ($5 \times D_{15B}$):	OK	OK
To minimize segregation (from *Table B-3*)***		
Max allowable D_{90CF} =	20	OK
Max D_{90CF} =	4.13	

Filters should be relatively uniform (see the C_U value of the candidate filter soil). Also, filters should not be gap-graded.

*** Generally, this requirement is only necessary for coarse filters and gravel zones that serve as both filters and drains. For sand filters with D_{90} < ~20mm, these limitations are usually not necessary.

Base and Filter Material Project: Erin Lake Dam-Alluvium vs GDOT 10 NS Sand Name: TKW Date: 5/9/24 2:09 PM

Base Material

Determine the gradation curves of the base soil. Use enough samples to define the range of grain size for the base soil. Design the filter gradation based on the base soil that requires the smallest D_{15F} size. If soil has particles larger than the #4 sieve, an adjusted gradation is calculated. Input values below for the base soil (original) gradation (in red):

Particle size (mm)	Sieve #	Base soil (original), % passing (coarse boundary)	Base soil (original), % passing (fine boundary)	Adjusted coarse boundary	Adjusted fine boundary
75	-				
37.5	-			*(No adjustment needed)*	
12.7	-				
9.5	-				
4.75	4	100.0%	101.0%		
4.00	5				
3.35	6				
2.80	7				
2.36	8				
2.00	10	99.6%	100.0%		
1.70	12				
1.40	14				
1.18	16				
1.00	18				
0.850	20	97.2%	98.1%		
0.710	25				
0.600	30				
0.500	35				
0.425	40	88.4%	89.0%		
0.300	50				
0.250	60	68.8%	76.0%		
0.212	70				
0.180	80				
0.150	100	42.8%	62.1%		
0.125	120				
0.106	140	30.3%	53.6%		
0.090	170				
0.075	200	22.6%	47.6%		
0.053	270				
0.02	-		33.0%		
0.01	-	13.0%	27.0%		
0.002	-		16.5%		
0.001	-	10.0%	14.0%		
0.0004	-				
0.0003	-				
0.0001	-		8.0%		

Maximum % passing #200 after regrading (if any) = A = 47.6%

Candidate Filter

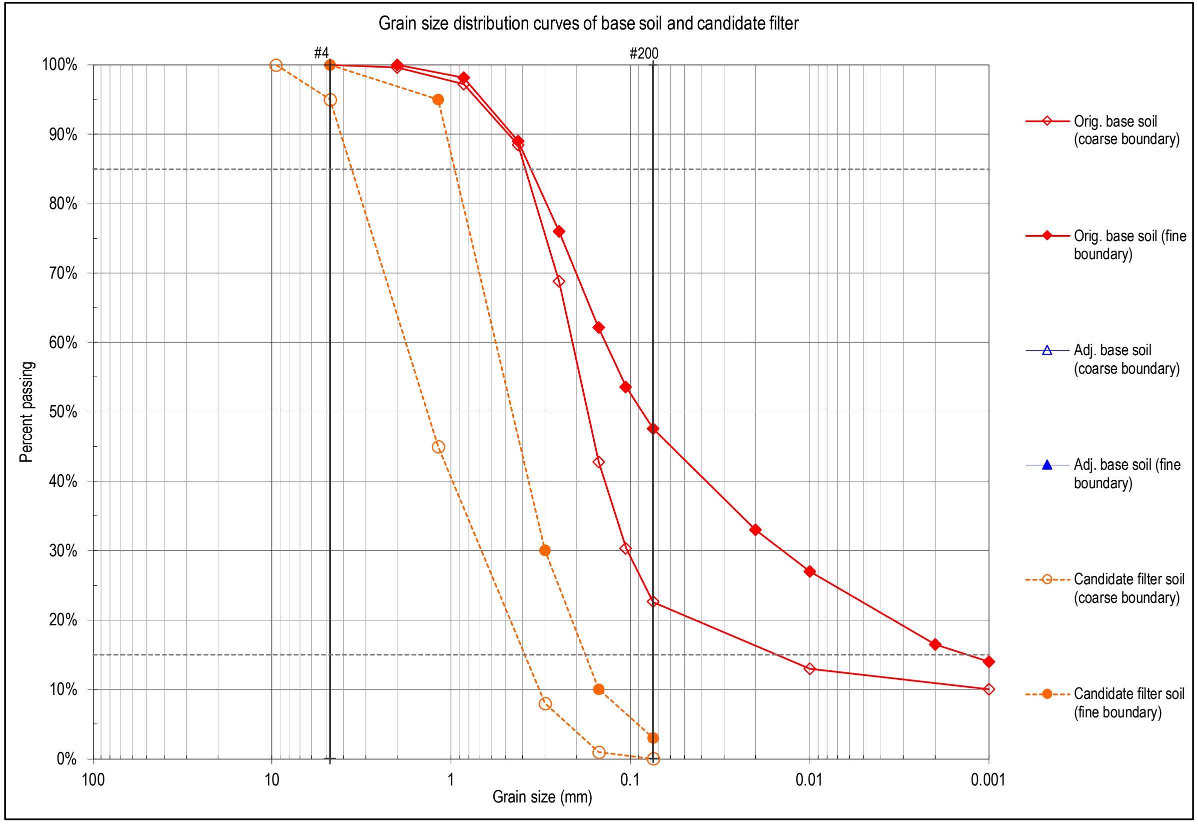
Candidate filter soil gradation. Values shown in red in the left column, and all values in the two right columns, can be changed.

Particle size *mm*	Sieve #	% Passing (coarse boundary)	% Passing (fine boundary)
150.0	-		
100.0	-		
90.0	-		
75.0	-		
63.0	-		
50.0	-		
37.5	-		
25.0	-		
19.0	-		
12.5	-		
9.5	-	100.0%	
4.75	4	95.0%	100.0%
3.35	6		
2.50	8		
2.00	10		
1.70	12		
1.40	14		
1.18	16	45.0%	95.0%
0.850	20		
0.600	30		
0.425	40		
0.300	50	8.0%	30.0%
0.250	60		
0.212	70		
0.180	80		
0.150	100	1.0%	10.0%
0.125	120		
0.106	140		
0.090	170		
0.075	200	0.0%	3.0%
0.053	270		
0.037	-		
0.019	-		
0.009	-		
0.005	-		
Candidate Filter Gradation	User Defined Gradation		

*Required entry values for base soil & candidate filter gradations:

1. Particle size for 100% passing.
2. % Passing the #4 sieve.
3. % Passing the #200 sieve.
4. Enough points to accurately represent the grain size distribution curve as straight lines between points. D_{85} and D_{15} sizes are interpolated from points on a log linear scale.
5. A zero % passing.
6. No duplicate entries; if D100<#4, enter 101% for #4 and 100% for appropriate size.

Gradation Curves



Properties of base soil	Coarse boundary	Fine Boundary
D_{85B} of original base soil =	0.388 mm	0.361 mm
D_{85B} of adjusted base soil =	--- mm	--- mm
D_{15B} of original base soil =	0.0152 mm	0.0013 mm
D_{15B} of adjusted base soil =	--- mm	--- mm
D_{60B} of original base soil =	0.210 mm	0.138 mm
D_{10B} of original base soil =	0.0010 mm	0.0002 mm
C_u of original base soil =	210.3	639.0

Properties of candidate filter soil (CF). D sizes are in mm:

	D_{85CF}	D_{15CF}	D_{60CF}	D_{10CF}	D_{30CF}	D_{90CF}	C_u	C_C
coarse boundary	3.60	0.39	1.79	0.32	0.68	4.13	5.55	0.79
fine boundary	0.96	0.18	0.56	0.15	0.30	1.06	3.76	1.06

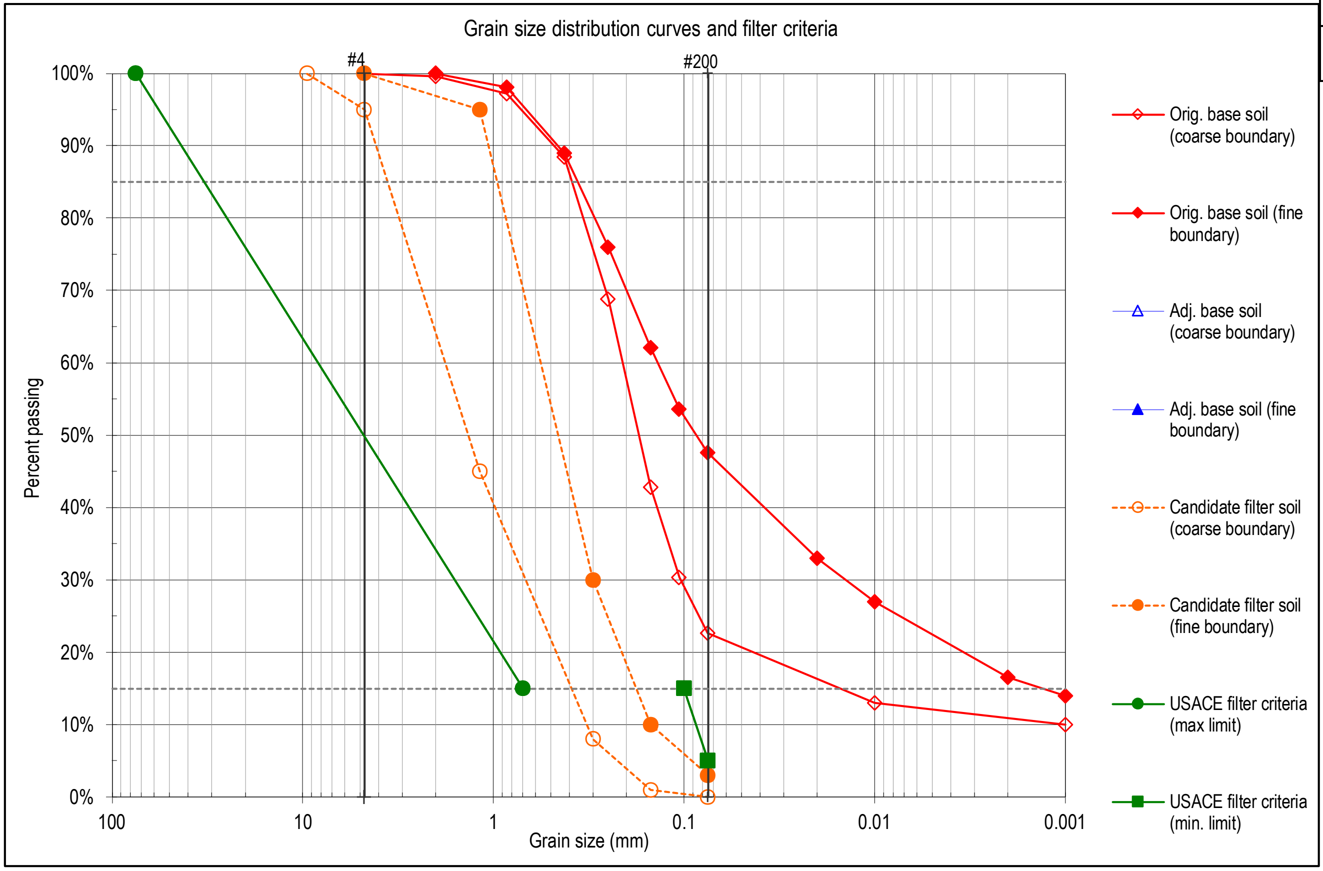
USACE Filter Criteria

Project: Erin Lake Dam-Alluvium vs GDOT 10 NS Sand

Name: TKW

Date: 5/9/24 2:09 PM

Candidate Filter Gradation
User Defined Gradation



Properties of base soil		Coarse boundary	Fine boundary
D_{85B}	of original base soil =	0.388 mm	0.361 mm
D_{85B}	of adjusted base soil =	--- mm	--- mm
D_{15B}	of original base soil =	0.0152 mm	0.0013 mm
D_{15B}	of adjusted base soil =	--- mm	--- mm
D_{60B}	of original base soil =	0.2103 mm	0.1377 mm
D_{10B}	of original base soil =	0.0010 mm	0.0002 mm
C_u	of original base soil =	210.3	639.0

USACE Filter Material

Filter criteria required by the US Army Corps of Engineers as published in EM 1110-2-2300 (31 Jul 94):

D_{85B} used in filter design	0.361
Maximum Passing #200 sieve of base soil	47.6%
Base soil category	2
Filter criteria (mm)	Maximum: $D_{15F} \le$ 0.70 to 0.70 To ensure sufficient permeability: Minimum: $D_{15F} \ge$ 0.10 to 0.10
Maximum particle size of filter (mm)	75
Maximum % passing # 200 sieve	5%
PI of material passing #40	0 when tested in accordance with EM 1110-2-1906

**If the base soil is in category 4, use the lower of the two 'max. D_{15F}' values when the filter is beneath riprap subject to wave action or beneath drains which may be subject to violent surging and/or vibration.

USACE filter gradation limits:

Maximum limit	
Grain size (mm)	% Passing
75.00	100.0%
0.70	15.0%
0.70	15.0%

Minimum limit	
Grain size (mm)	% Passing
0.10	15.0%
0.10	15.0%
0.075	5.0%

Acceptability of candidate filter (CF) soil:

USACE criteria	Coarse boundary	Fine boundary
Max % passing #200:	OK	OK
Max particle size (mm):	OK	OK
Maximum D_{15CF}:	OK	OK
Minimum D_{15CF} ($3 \times D_{15B}$):	OK	OK
Minimum D_{15CF} ($5 \times D_{15B}$):	OK	OK
To minimize segregation (from *Table B-3*)***		
Max allowable D_{90CF} =	20	OK
Max D_{90CF} =	4.13	

Filters should be relatively uniform (see the C_U value of the candidate filter soil). Also, filters should not be gap-graded.

*** Generally, this requirement is only necessary for coarse filters and gravel zones that serve as both filters and drains. For sand filters with D_{90} < ~20mm, these limitations are usually not necessary.

Project: Erin Lake Dam | Originator: TKW | Date: 6/24/2024
Reviewed by: ld | Date: 06/27/2024

Erin Lake Dam Base Material for Riprap at outlet channel of dam

Objective: To evaluate the compatibility of ASTM C33 coarse aggregate to be used as a base soil for designed riprap at outlet channel on Erin Lake Dam.

References:

ASTM. (2018) ASTM C33 Standard Specification for Concrete Aggregates.

USACE. (2004). General Design and Construction Considerations for Earth and Rock-Fill Dams

Georgia Department of Transportation. (2021) “Standard Specifications Construction of Transportation Systems.

Method: Evaluate the compatibility of ASTM #3 coarse aggregate and riprap based on general filter gradation from USACE General Design and Construction Considerations for Earth and Rock-Fill Dams (2004).

Filter material is evaluated to transition from the embankment fill and foundation soil to the riprap at the downstream outlet channel. Evaluation of filter compatibility for the filter drains detailed that ASTM C33 fine aggregate (sand) and ASTM #8 coarse aggregate are compatible for filters of the embankment and foundation soils. These two filter materials will be used as filter layers beneath the riprap. However, an additional filter layer consisting of ASTM #3 coarse aggregate is proposed to transition between ASTM #8 and the designed riprap. The designed filter layers from the fill and foundation soils to the riprap are as follows:

Embankment Fill or Foundation > ASTM C33 Fine Aggregate > ASTM #8 Coarse Aggregate > ASTM #3 Coarse Aggregate > riprap

Results of the compatibility analysis (attached) shows the ASTM #3 coarse aggregate is compatible with the ASTM #8 coarse aggregate for filtration and permeability.

Based on proposed design of the riprap, the D_{50} (filter) of the riprap is 1.0 ft (12 inches).

While USACE filter compatibility analyses generally have a design maximum aggregate diameter size of 3.0 inches, the filter criteria method was utilized for the proposed riprap using the ASTM #3 aggregate as the base and riprap for the filter.

The maximum and minimum D_{15} for candidate filter soils (riprap) are based on specific particle sizes of the base soil (ASTM #3). From USACE EM 1110-2-2300 (2004) Table B-2 for sands and gravels with less than 15% fines, “the D_{15} (filter) $\leq 4 \times d_{85}$ (base) criterion should be used in the case of filters beneath riprap subject to wave action and drains which may be subject to violent surging and/or vibration. Therefore, based on USACE Table B-2, for base soil Type 4, The D_{15} of the filter (riprap) should be less than or equal to 4 times the d_{85} of the base soil (ASTM #3). Compatibility from ASTM #3 coarse aggregate (base) to riprap (filter) was assumed based on these correlations.

Table B-2 (USACE, 2004). Filter Criteria

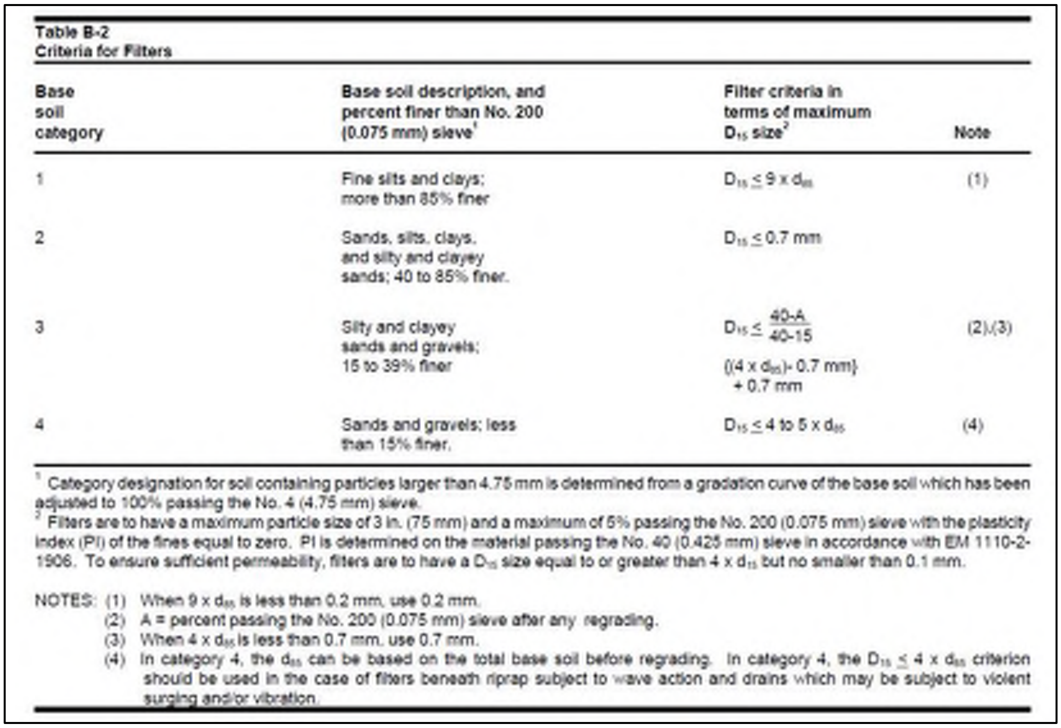


Table B-2
Criteria for Filters

Base soil category	Base soil description, and percent finer than No. 200 (0.075 mm) sieve[1]	Filter criteria in terms of maximum D_{15} size[2]	Note
1	Fine silts and clays; more than 85% finer	$D_{15} \leq 9 \times d_{85}$	(1)
2	Sands, silts, clays, and silty and clayey sands; 40 to 85% finer.	$D_{15} \leq 0.7$ mm	
3	Silty and clayey sands and gravels; 15 to 39% finer	$D_{15} \leq \frac{40\text{-}A}{40\text{-}15}$ $((4 \times d_{85})\text{- } 0.7\text{ mm})$ $+ 0.7$ mm	(2),(3)
4	Sands and gravels; less than 15% finer.	$D_{15} \leq 4$ to $5 \times d_{85}$	(4)

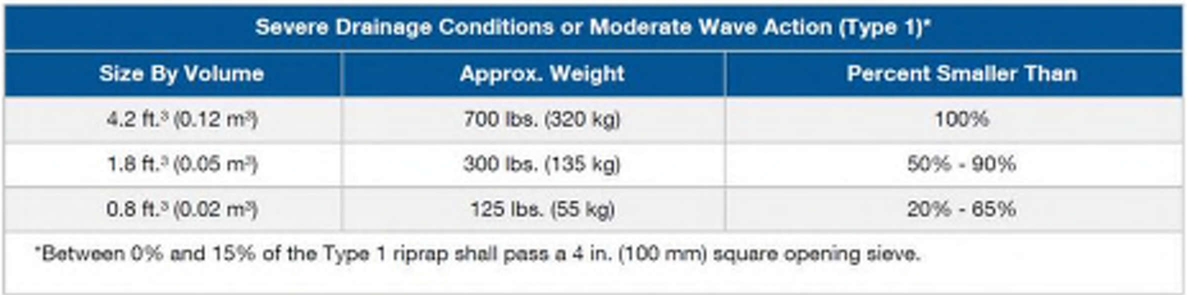
[1] Category designation for soil containing particles larger than 4.75 mm is determined from a gradation curve of the base soil which has been adjusted to 100% passing the No. 4 (4.75 mm) sieve.

[2] Filters are to have a maximum particle size of 3 in. (75 mm) and a maximum of 5% passing the No. 200 (0.075 mm) sieve with the plasticity index (PI) of the fines equal to zero. PI is determined on the material passing the No. 40 (0.425 mm) sieve in accordance with EM 1110-2-1906. To ensure sufficient permeability, filters are to have a D_{15} size equal to or greater than $4 \times d_{15}$ but no smaller than 0.1 mm.

NOTES:
(1) When $9 \times d_{85}$ is less than 0.2 mm, use 0.2 mm.
(2) A = percent passing the No. 200 (0.075 mm) sieve after any regrading.
(3) When $4 \times d_{85}$ is less than 0.7 mm, use 0.7 mm.
(4) In category 4, the d_{85} can be based on the total base soil before regrading. In category 4, the $D_{15} \leq 4 \times d_{85}$ criterion should be used in the case of filters beneath riprap subject to wave action and drains which may be subject to violent surging and/or vibration.

Based on USACE EM 1110-2-2300 (2004) the maximum candidate filter soil D_{15} (riprap) allowable is 4 times the base soil d_{85} (ASTM #3). The fine band d_{85} for ASTM #3 coarse aggregate is approximately 43.3 mm (1.7 inches). The maximum allowable candidate D_{15} (4 times 1.7 inch) is therefore 6.7 inches. The design D_{50} of the riprap is 1 ft and between 0 % and 15% of GDOT Type 1 riprap may pass 4 inches (GDOT Standard Specifications Construction of Transportation Systems Section 805.2.01.A.2 “riprap”, 2021).

GDOT Type 1 riprap sizing (GADOT Standard Specifications Construction of Transportation Systems, 2021)



Severe Drainage Conditions or Moderate Wave Action (Type 1)*

Size By Volume	Approx. Weight	Percent Smaller Than
4.2 ft.^{3} (0.12 m^3)	700 lbs. (320 kg)	100%
1.8 ft.^{3} (0.05 m^3)	300 lbs. (135 kg)	50% - 90%
0.8 ft.^{3} (0.02 m^3)	125 lbs. (55 kg)	20% - 65%

*Between 0% and 15% of the Type 1 riprap shall pass a 4 in. (100 mm) square opening sieve.

From interpolation, the coarse D_{15} (riprap) of the riprap was estimated to be 6.4 inches.

Interpolating for D_{15} based on D_{50} = 12 inches (304.8 mm) and 0% passing 4 inches (100 mm).

D_{15}=((Size of D_{50}-Dize of D_0)/(Percent difference)) *15 (percent) + 100 mm (4 inches) =

D_{15}=((304.8mm-100mm)/(50-0))*15+100 mm=161.44 mm*1in/(25.4 mm)= 6.4 inches

Therefore, ASTM #3 coarse aggregate is acceptable for use as a base for the designed riprap to be utilized for Erin Lake Dam.

Reference 1: ASTM C33 Standard Specification for Concrete Aggregates (2018).

TABLE 3 Grading Requirements for Coarse Aggregates

Size Number	Nominal Size (Sieves with Square Openings)	Amounts Finer than Each Laboratory Sieve (Square-Openings), Mass Percent													
		100 mm (4 in.)	90 mm (3½ in.)	75 mm (3 in.)	63 mm (2½ in.)	50 mm (2 in.)	37.5 mm (1½ in.)	25.0 mm (1 in.)	19.0 mm (¾ in.)	12.5 mm (½ in.)	9.5 mm (⅜ in.)	4.75 mm (No. 4)	2.36 mm (No. 8)	1.18 mm (No. 16)	300 µm (No.50)
1	90 to 37.5 mm (3½ to 1½ in.)	100	90 to 100	...	25 to 60	...	0 to 15	...	0 to 5	...	...	...	...	...	...
2	63 to 37.5 mm (2½ to 1½ in.)	...	...	100	90 to 100	35 to 70	0 to 15	...	0 to 5	...	...	...	...	...	...
3	50 to 25.0 mm (2 to 1 in.)	...	...	...	100	90 to 100	35 to 70	0 to 15	...	0 to 5	...	...	...	...	...
357	50 to 4.75 mm (2 in. to No. 4)	...	...	...	100	95 to 100	...	35 to 70	...	10 to 30	...	0 to 5	...	...	...
4	37.5 to 19.0 mm (1½ to ¾ in.)	...	...	...	...	100	90 to 100	20 to 55	0 to 15	...	0 to 5	...	...	...	...
467	37.5 to 4.75 mm (1½ in. to No. 4)	...	...	...	...	100	95 to 100	...	35 to 70	...	10 to 30	0 to 5	...	...	...
5	25.0 to 12.5 mm (1 to ½ in.)	...	...	...	...	...	100	90 to 100	20 to 55	0 to 10	0 to 5	...	...	...	...
56	25.0 to 9.5 mm (1 to ⅜ in.)	...	...	...	...	...	100	90 to 100	40 to 85	10 to 40	0 to 15	0 to 5	...	...	...
57	25.0 to 4.75 mm (1 in. to No. 4)	...	...	...	...	...	100	95 to 100	...	25 to 60	...	0 to 10	0 to 5	...	...
6	19.0 to 9.5 mm (¾ to ⅜ in.)	...	...	...	...	...	...	100	90 to 100	20 to 55	0 to 15	0 to 5	...	...	...
67	19.0 to 4.75 mm (¾ in. to No. 4)	...	...	...	...	...	...	100	90 to 100	...	20 to 55	0 to 10	0 to 5	...	...
7	12.5 to 4.75 mm (½ in. to No. 4)	...	...	...	...	...	...	...	100	90 to 100	40 to 70	0 to 15	0 to 5	...	...
8	9.5 to 2.36 mm (⅜ in. to No. 8)	...	...	...	...	...	...	...	...	100	85 to 100	10 to 30	0 to 10	0 to 5	...
89	9.5 to 1.18 mm (⅜ in. to No. 16)	...	...	...	...	...	...	...	...	100	90 to 100	20 to 55	5 to 30	0 to 10	0 to 5
9[A]	4.75 to 1.18 mm (No. 4 to No. 16)	...	...	...	...	...	...	...	...	...	100	85 to 100	10 to 40	0 to 10	0 to 5

[A] Size number 9 aggregate is defined in Terminology C125 as a fine aggregate. It is included as a coarse aggregate when it is combined with a size number 8 material to create a size number 89, which is a coarse aggregate as defined by Terminology C125.

Base and Filter Material Project: Erin Lake Dam- ASTM #8 Coarse Aggregate vs ASTM #3 Coarse Aggregate Name: TKW Date: 6/24/24 11:13 AM

Base Material

Determine the gradation curves of the base soil. Use enough samples to define the range of grain size for the base soil. Design the filter gradation based on the base soil that requires the smallest D_{15F} size. If soil has particles larger than the #4 sieve, an adjusted gradation is calculated. Input values below for the base soil (original) gradation (in red):

Particle size (mm)	Sieve #	Base soil (original), % passing (coarse boundary)	Base soil (original), % passing (fine boundary)	Adjusted coarse boundary	Adjusted fine boundary
75	-				
37.5	-				
12.5	-	100.0%			
9.5	-	85.0%	101.0%		
4.75	4	10.0%	30.0%	100.0%	100.0%
4.00	5				
3.35	6				
2.80	7				
2.36	8	0.0%	10.0%	0.0%	33.3%
2.00	10				
1.70	12				
1.40	14				
1.18	16	0.0%	5.0%	0.0%	16.7%
1.00	18				
0.850	20				
0.710	25				
0.600	30		0.0%		0.0%
0.500	35				
0.425	40				
0.300	50				
0.250	60				
0.212	70				
0.180	80				
0.150	100				
0.125	120				
0.106	140				
0.090	170				
0.075	200	0.0%	0.0%	0.0%	0.0%
0.053	270				
0.004	-				
0.002	-				
0.002	-				
0.005	-				
0.003	-				
0.002	-				
0.001	-				

Maximum % passing #200 after regrading (if any) = A = 0.0%

Candidate Filter

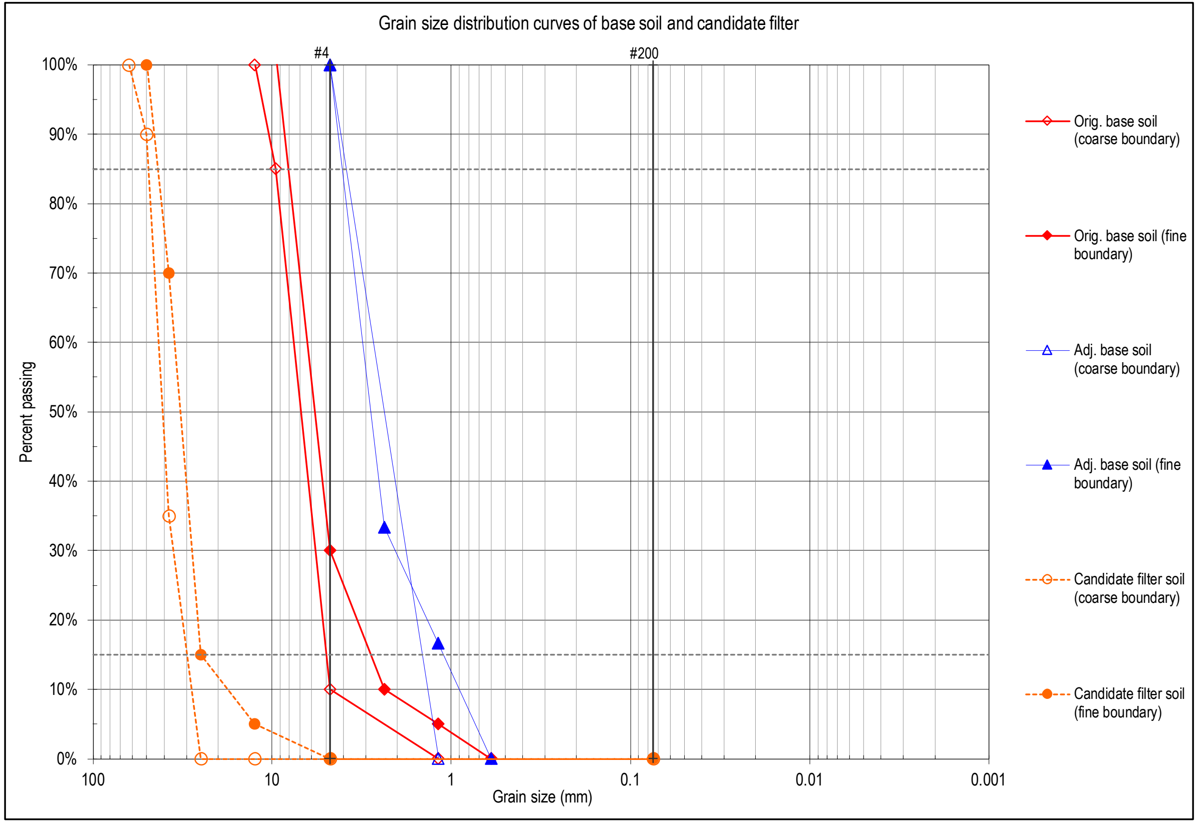
Candidate filter soil gradation. Values shown in red in the left column, and all values in the two right columns, can be changed.

Particle size *mm*	Sieve #	% Passing (coarse boundary)	% Passing (fine boundary)
150.0	-		
100.0	-		
90.0	-		
75.0	-		
63.0	-	100.0%	
50.0	-	90.0%	100.0%
37.5	-	35.0%	70.0%
25.0	-	0.0%	15.0%
19.0	-		
12.5	-	0.0%	5.0%
9.5	-		
4.75	4	(0.0%)	(0.0%)
3.35	6		
2.50	8		
2.00	10		
1.70	12		
1.40	14		
1.18	16		
0.850	20		
0.600	30		
0.425	40		
0.300	50		
0.250	60		
0.212	70		
0.180	80		
0.150	100		
0.125	120		
0.106	140		
0.090	170		
0.075	200	(0.0%)	(0.0%)
0.053	270		
0.037	-		
0.019	-		
0.009	-		
0.005	-		
Candidate Filter Gradation	ASTM C33 - 02a Table 2: Size # 3		

*Required entry values for base soil & candidate filter gradations:

1. Particle size for 100% passing.
2. % Passing the #4 sieve.
3. % Passing the #200 sieve.
4. Enough points to accurately represent the grain size distribution curve as straight lines between points. D_{85} and D_{15} sizes are interpolated from points on a log linear scale.
5. A zero % passing.
6. No duplicate entries; if D100<#4, enter 101% for #4 and 100% for appropriate size

Gradation Curves



Properties of base soil	Coarse boundary	Fine Boundary
D_{85B} of original base soil =	9.500 mm	8.126 mm
D_{85B} of adjusted base soil =	3.855 mm	4.058 mm
D_{15B} of original base soil =	4.9746 mm	2.8110 mm
D_{15B} of adjusted base soil =	1.4541 mm	1.1028 mm
D_{60B} of adjusted base soil =	2.721 mm	3.122 mm
D_{10B} of adjusted base soil =	1.3563 mm	0.9003 mm
C_u of adjusted base soil =	2.0	3.5

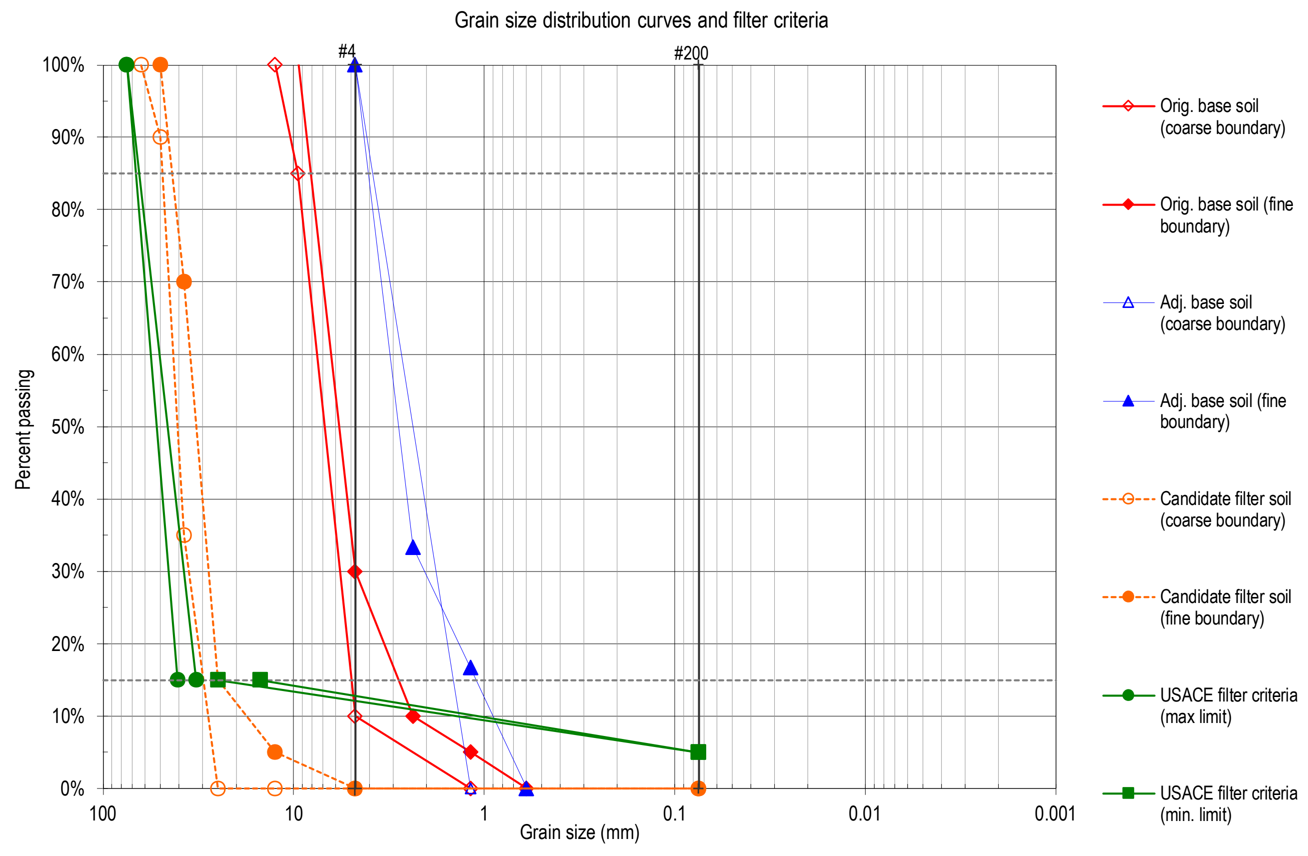
Properties of candidate filter soil (CF). D sizes are in mm:

	D_{85CF}	D_{15CF}	D_{60CF}	D_{10CF}	D_{30CF}	D_{90CF}	C_u	C_C
coarse boundary	48.71	29.74	42.74	28.07	35.39	50.00	1.52	1.04
fine boundary	43.30	25.00	34.83	17.68	27.92	45.43	1.97	1.27

USACE Filter Criteria

Project: Erin Lake Dam- ASTM #8 Coarse Aggregate vs ASTM #3 Coarse Aggregate
Name: TKW
Date: 6/24/24 11:13 AM

Candidate Filter Gradation
ASTM C33 - 02a
Table 2: Size # 3



Properties of base soil		Coarse boundary	Fine boundary
D_{85B}	of original base soil =	9.500 mm	8.126 mm
D_{85B}	of adjusted base soil =	3.855 mm	4.058 mm
D_{15B}	of original base soil =	4.9746 mm	2.8110 mm
D_{15B}	of adjusted base soil =	1.4541 mm	1.1028 mm
D_{60B}	of adjusted base soil =	2.7213 mm	3.1219 mm
D_{10B}	of adjusted base soil =	1.3563 mm	0.9003 mm
C_u	of adjusted base soil =	2.0	3.5

USACE Filter Material

Filter criteria required by the US Army Corps of Engineers as published in EM 1110-2-2300 (31 Jul 94):

D_{85B} used in filter design	8.126
Maximum Passing #200 sieve of base soil (from adjusted gradation)	0.0%
Base soil category	4**
Filter criteria (mm)	Maximum: $D_{15F} \leq$ 32.50 to 40.63 To ensure sufficient permeability: Minimum: $D_{15F} \geq$ 14.92 to 24.87
Maximum particle size of filter (mm)	75
Maximum % passing # 200 sieve	5%
PI of material passing #40	0 when tested in accordance with EM 1110-2-1906

**If the base soil is in category 4, use the lower of the two 'max. D_{15F}' values when the filter is beneath riprap subject to wave action or beneath drains which may be subject to violent surging and/or vibration.

USACE filter gradation limits:

Maximum limit	
Grain size (mm)	% Passing
75.00	100.0%
40.63	15.0%
32.50	15.0%

Minimum limit	
Grain size (mm)	% Passing
14.92	15.0%
24.87	15.0%
0.075	5.0%

Acceptability of candidate filter (CF) soil:

USACE criteria	Coarse boundary	Fine boundary
Max % passing #200:	OK	OK
Max particle size (mm):	OK	OK
Maximum D_{15CF}:	OK	OK
Minimum D_{15CF} ($3 \times D_{15B}$):	OK	OK
Minimum D_{15CF} ($5 \times D_{15B}$):	OK	OK
To minimize segregation (from *Table B-3*)***		
Max allowable D_{90CF} =	60	OK
Max D_{90CF} =	50.00	

Filters should be relatively uniform (see the C_U value of the candidate filter soil). Also, filters should not be gap-graded.

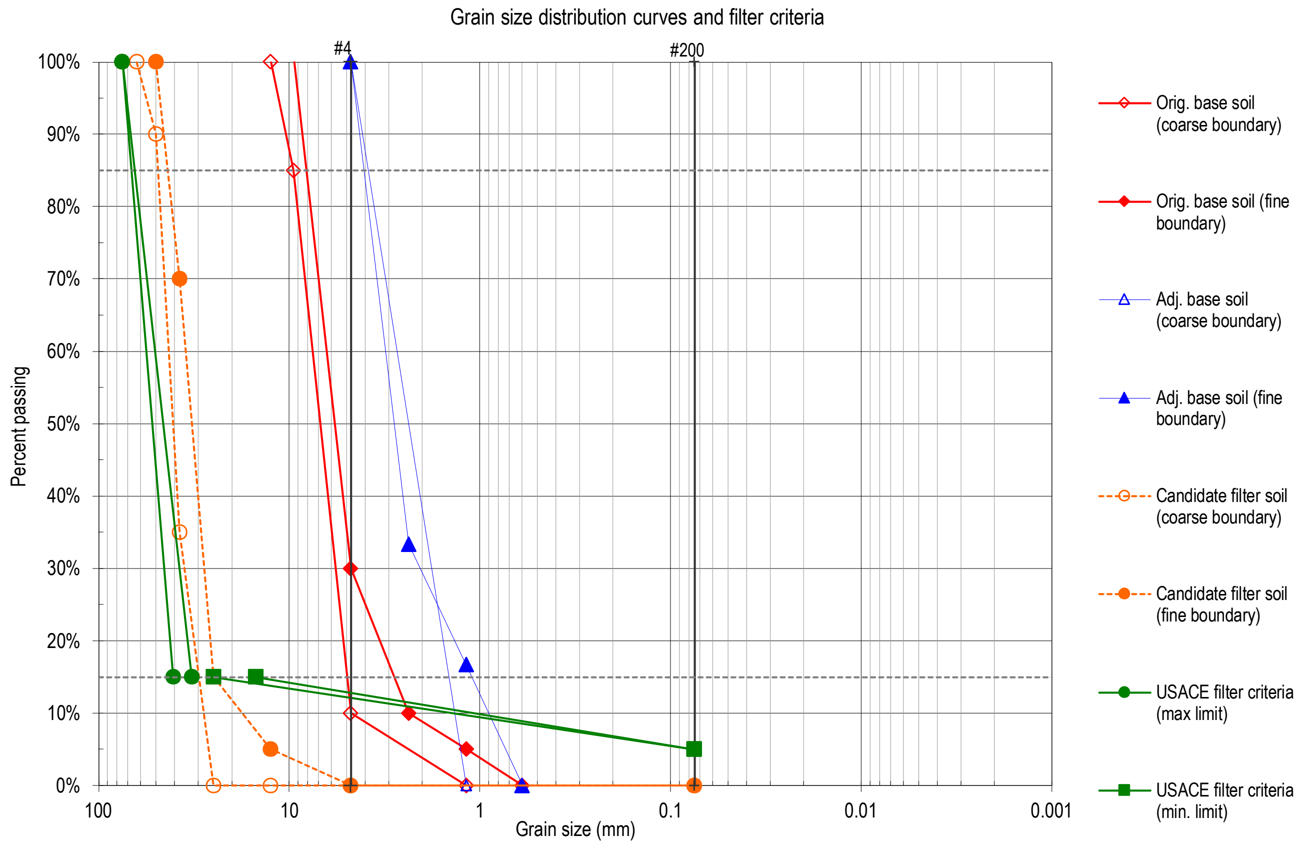
*** Generally, this requirement is only necessary for coarse filters and gravel zones that serve as both filters and drains. For sand filters with D_{90} < ~20mm, these limitations are usually not necessary.

USACE Filter Criteria

Project: Erin Lake Dam- ASTM #8 Coarse Aggregate vs ASTM #3 Coarse Aggregate   Name: TKW   Date: 6/24/24 11:13 AM

Candidate Filter Gradation
ASTM C33 - 02a
Table 2: Size # 3

Grain size distribution curves and filter criteria



Properties of base soil		Coarse boundary	Fine boundary
D_{85B}	of original base soil =	9.500 mm	8.126 mm
D_{85B}	of adjusted base soil =	3.855 mm	4.058 mm
D_{15B}	of original base soil =	4.9746 mm	2.8110 mm
D_{15B}	of adjusted base soil =	1.4541 mm	1.1028 mm
D_{60B}	of adjusted base soil =	2.7213 mm	3.1219 mm
D_{10B}	of adjusted base soil =	1.3563 mm	0.9003 mm
C_u	of adjusted base soil =	2.0	3.5

USACE Filter Material

Filter criteria required by the US Army Corps of Engineers as published in EM 1110-2-2300 (31 Jul 94):

D_{85B} used in filter design	8.126
Maximum Passing #200 sieve of base soil (from adjusted gradation)	0.0%
Base soil category	4**
Filter criteria (mm)	Maximum: $D_{15F} \leq$ 32.50 to 40.63 To ensure sufficient permeability: Minimum: $D_{15F} \geq$ 14.92 to 24.87
Maximum particle size of filter (mm)	75
Maximum % passing # 200 sieve	5%
PI of material passing #40	0 when tested in accordance with EM 1110-2-1906

**If the base soil is in category 4, use the lower of the two 'max. D_{15F}' values when the filter is beneath riprap subject to wave action or beneath drains which may be subject to violent surging and/or vibration.

USACE filter gradation limits:

Maximum limit

Grain size (mm)	% Passing
75.00	100.0%
40.63	15.0%
32.50	15.0%

Minimum limit

Grain size (mm)	% Passing
14.92	15.0%
24.87	15.0%
0.075	5.0%

Acceptability of candidate filter (CF) soil:

USACE criteria	Coarse boundary	Fine boundary
Max % passing #200:	OK	OK
Max particle size (mm):	OK	OK
Maximum D_{15CF}:	OK	OK
Minimum D_{15CF} ($3{\times}D_{15B}$):	OK	OK
Minimum D_{15CF} ($5{\times}D_{15B}$):	OK	OK
To minimize segregation (from *Table B-3*)***		
Max allowable D_{90CF} =	60	OK
Max D_{90CF} =	50.00	

Filters should be relatively uniform (see the C_U value of the candidate filter soil). Also, filters should not be gap-graded.

*** Generally, this requirement is only necessary for coarse filters and gravel zones that serve as both filters and drains. For sand filters with D_{90} < ~20mm, these limitations are usually not necessary.

REFERENCE 2: USACE. (2004). General Design and Construction Considerations for Earth and Rock-Fill Dams

(2) Multiply the percentage passing each sieve size of the base soil smaller than No. 4 (4.75 mm) by the correction factor from step *c*(1).

(3) Plot these adjusted percentages to obtain a new gradation curve.

(4) Use the adjusted curve to determine the percent passing the No. 200 (0.075 mm) sieve in step *d*.

d. Place the base soil in a category based on the percent passing the No. 200 (0.075 mm) sieve in accordance with Table B-1.

Table B-1
Categories of Base Soil Materials

Category	Percent finer than the No. 200 (0.075 mm) sieve
1	85
2	40-85
3	15-39
4	15

e. Determine the maximum D_{15} size for the filter in accordance with Table B-2. Note that the maximum D_{15} is not required to be smaller than 0.20 mm.

Table B-2
Criteria for Filters

Base soil category	Base soil description, and percent finer than No. 200 (0.075 mm) sieve[1]	Filter criteria in terms of maximum D_{15} size[2]	Note
1	Fine silts and clays; more than 85% finer	$D_{15} \leq 9 \times d_{85}$	(1)
2	Sands, silts, clays, and silty and clayey sands; 40 to 85% finer.	$D_{15} \leq 0.7$ mm	
3	Silty and clayey sands and gravels; 15 to 39% finer	$D_{15} \leq \frac{40-A}{40-15}$ $\{(4 \times d_{85}) - 0.7 \text{ mm}\} + 0.7 \text{ mm}$	(2),(3)
4	Sands and gravels; less than 15% finer.	$D_{15} \leq 4$ to $5 \times d_{85}$	(4)

[1] Category designation for soil containing particles larger than 4.75 mm is determined from a gradation curve of the base soil which has been adjusted to 100% passing the No. 4 (4.75 mm) sieve.

[2] Filters are to have a maximum particle size of 3 in. (75 mm) and a maximum of 5% passing the No. 200 (0.075 mm) sieve with the plasticity index (PI) of the fines equal to zero. PI is determined on the material passing the No. 40 (0.425 mm) sieve in accordance with EM 1110-2-1906. To ensure sufficient permeability, filters are to have a D_{15} size equal to or greater than $4 \times d_{15}$ but no smaller than 0.1 mm.

NOTES:
(1) When $9 \times d_{85}$ is less than 0.2 mm, use 0.2 mm.
(2) A = percent passing the No. 200 (0.075 mm) sieve after any regrading.
(3) When $4 \times d_{85}$ is less than 0.7 mm, use 0.7 mm.
(4) In category 4, the d_{85} can be based on the total base soil before regrading. In category 4, the $D_{15} \leq 4 \times d_{85}$ criterion should be used in the case of filters beneath riprap subject to wave action and drains which may be subject to violent surging and/or vibration.

Reference 3: Georgia Department of Transportation. (2021) “Standard Specifications Construction of Transportation Systems.

Section 805—Riprap and Curbing Stone

805.1 General Description

This section includes the requirements for riprap and curbing stone. Construction and material will be covered under the Special Provisions.

805.1.01 Related References

A. Standard Specifications

General Provisions 101 through 150.

B. Referenced Documents

AASHTO T 96
AASHTO T 104
ASTM C 295
ASTM D 5519

805.2 Materials

805.2.01 Riprap

A. Requirements

1. Aggregate Quality

All riprap stone shall be made of sound, durable rock pieces that meet these requirements:

Aggregate Quality	Maximum Percent
Abrasion loss “B” grading	65
Soundness loss	15
Flat and slabby pieces (length five times more than the average thickness)	5
Weathered and/or decomposed pieces and shale	5

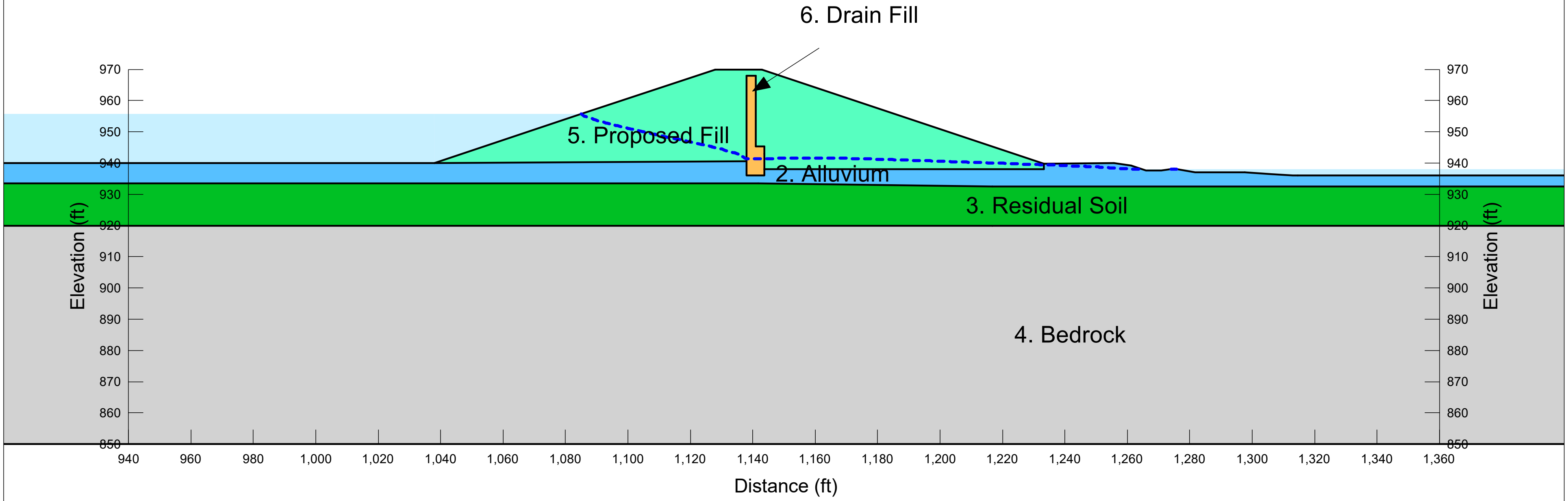
2. Gradation for Stone-Dumped riprap Type 1 and Type 3:

Severe Drainage Conditions or Moderate Wave Action (Type 1)*		
Size By Volume	**Approx. Weight**	**Percent Smaller Than**
4.2 ft.^{3} (0.12 m^3)	700 lbs. (320 kg)	100%
1.8 ft.^{3} (0.05 m^3)	300 lbs. (135 kg)	50% - 90%
0.8 ft.^{3} (0.02 m^3)	125 lbs. (55 kg)	20% - 65%
*Between 0% and 15% of the Type 1 riprap shall pass a 4 in. (100 mm) square opening sieve.		

Seepage and Slope Stability Analysis

Seepage and Slope Stability Analysis
At Principal Spillway Conduit
Cross-Section

ERIN LAKE DAM- PROPOSED CONDITIONS
NORMAL POOL ELEVATION: 955.7 FT
TAILWATER ELEVATION: 938 FT
MATERIAL PROPERTIES



AECOM

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Lake Erin Dam
DeKalb County, GA

Material Properties

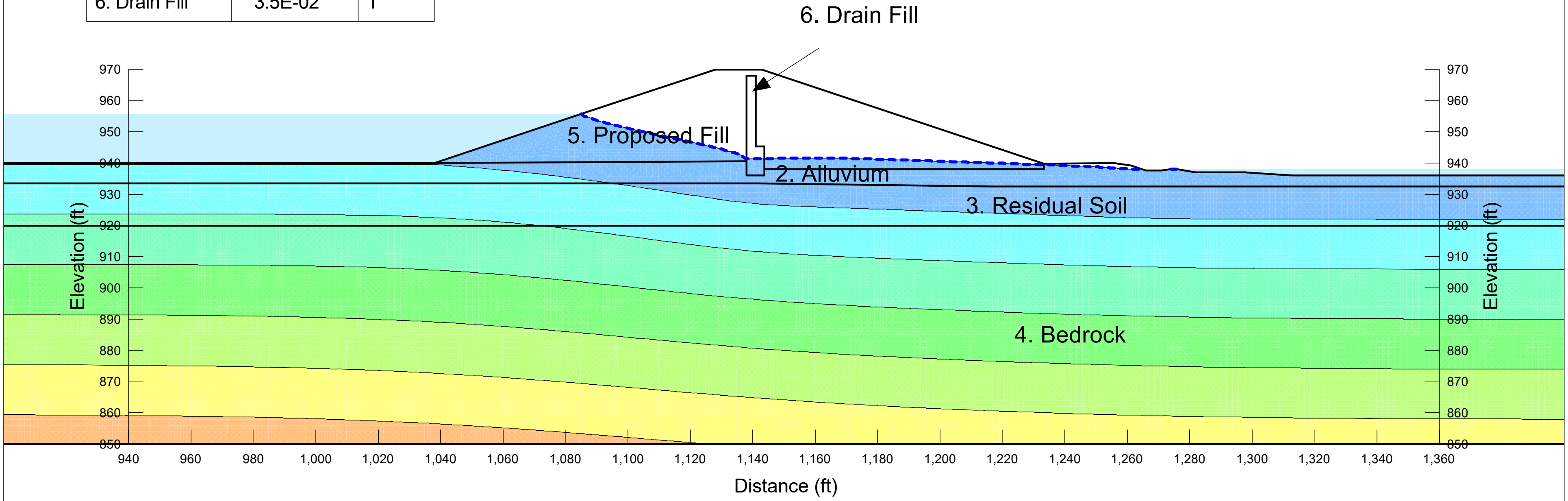
Date: June 2024 | Project No. 60727041 | Draft

ERIN LAKE DAM- PROPOSED CONDITIONS
NORMAL POOL ELEVATION: 955.7 FT
TAILWATER ELEVATION: 938 FT
STEADY SEEPAGE

Name	**Kx cm/sec**	**Ky'/Kx' Ratio**
2. Alluvium	1.08E-06	0.67
3. Residual Soil	2.50E-06	0.5
4. Bedrock	1.00E-09	1
5. Proposed Fill	1.08E-06	0.25
6. Drain Fill	3.5E-02	1

Water Pressure

- 🟦 0 - 1,000 psf
- 🟦 1,000 - 2,000 psf
- 🟩 2,000 - 3,000 psf
- 🟩 3,000 - 4,000 psf
- 🟩 4,000 - 5,000 psf
- 🟨 5,000 - 6,000 psf
- 🟧 6,000 - 7,000 psf
- 🟥 7,000 - 8,000 psf



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DeKalb County, GA

Seepage Analysis

Date: June 2024

Project No. 60727041

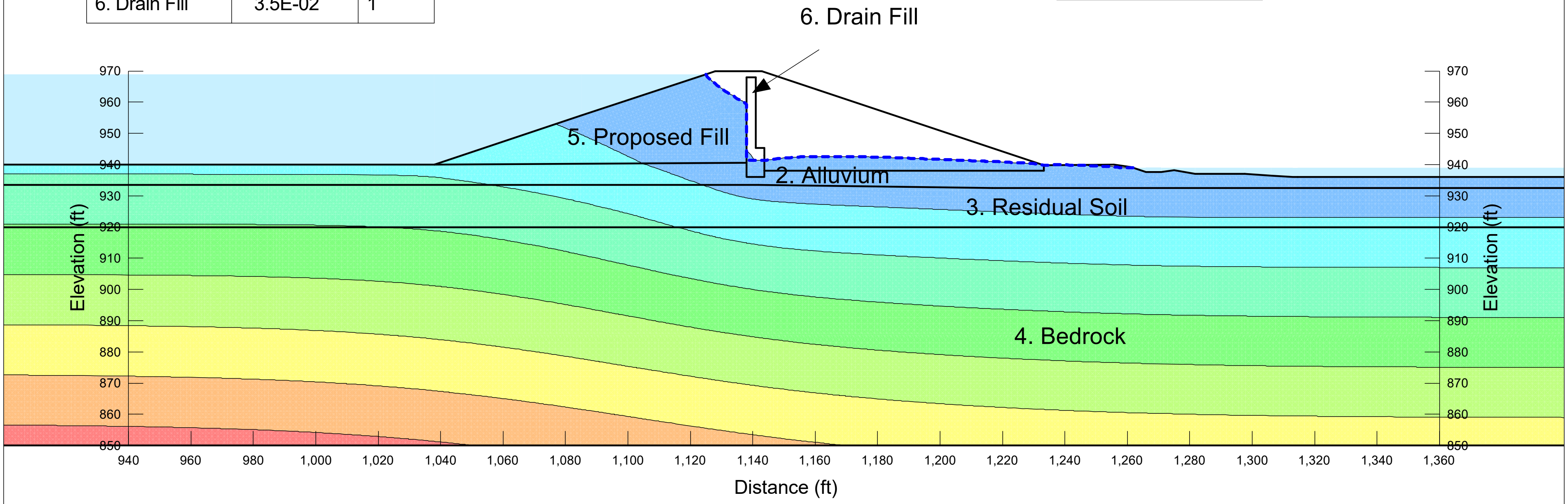
Draft

ERIN LAKE DAM-PROPOSED CONDITIONS
MAXIMUM POOL CONDITIONS
MAXIMUM POOL ELEVATION: 969.0 FT
TAILWATER ELEVATION: 939 FT
STEADY SEEPAGE

Name	**Kx cm/sec**	**Ky'/Kx' Ratio**
2. Alluvium	1.08E-06	0.67
3. Residual Soil	2.50E-06	0.5
4. Bedrock	1.00E-09	1
5. Proposed Fill	1.08E-06	0.25
6. Drain Fill	3.5E-02	1

Water Pressure

- 🟦 0 - 1,000 psf
- 🟦 1,000 - 2,000 psf
- 🟩 2,000 - 3,000 psf
- 🟩 3,000 - 4,000 psf
- 🟩 4,000 - 5,000 psf
- 🟨 5,000 - 6,000 psf
- 🟧 6,000 - 7,000 psf
- 🟥 7,000 - 8,000 psf



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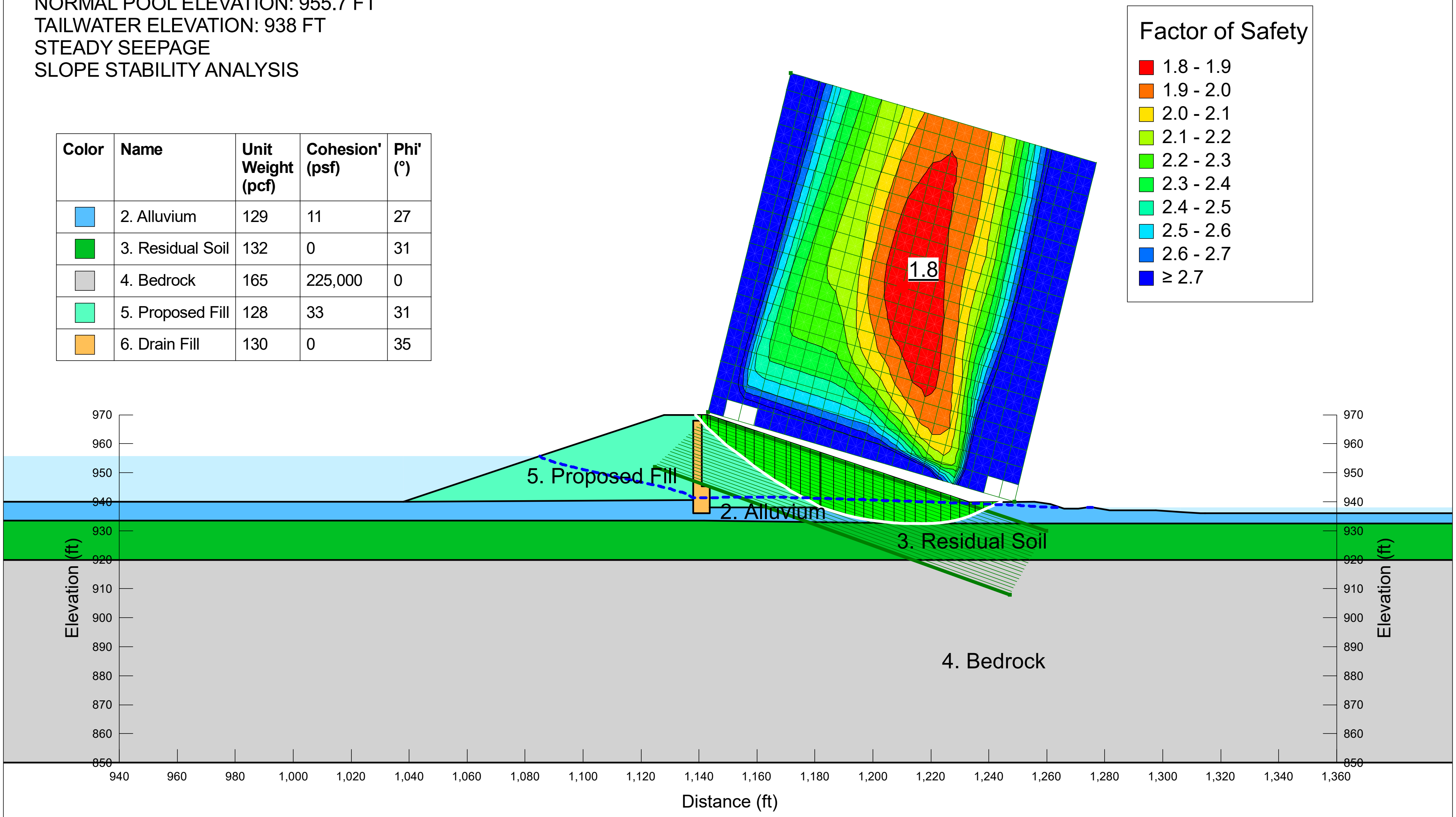
Lake Erin Dam
DeKalb County, GA

Seepage Analysis

Date: June 2024
Project No. 60727041
Draft

ERIN LAKE DAM- PROPOSED CONDITIONS
NORMAL POOL CONDITIONS
NORMAL POOL ELEVATION: 955.7 FT
TAILWATER ELEVATION: 938 FT
STEADY SEEPAGE
SLOPE STABILITY ANALYSIS

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟦	2. Alluvium	129	11	27
🟩	3. Residual Soil	132	0	31
⬜	4. Bedrock	165	225,000	0
🟦	5. Proposed Fill	128	33	31
🟧	6. Drain Fill	130	0	35



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Lake Erin Dam
DeKalb County, GA

Slope Stability Analysis

Date: June 2024
Project No. 60727041
Draft

ERIN LAKE DAM-PROPOSED CONDITIONS
MAXIMUM POOL CONDITIONS
MAXIMUM POOL ELEVATION: 969.0 FT
TAILWATER ELEVATION: 939 FT
STEADY SEEPAGE
SLOPE STABILITY ANALYSIS

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
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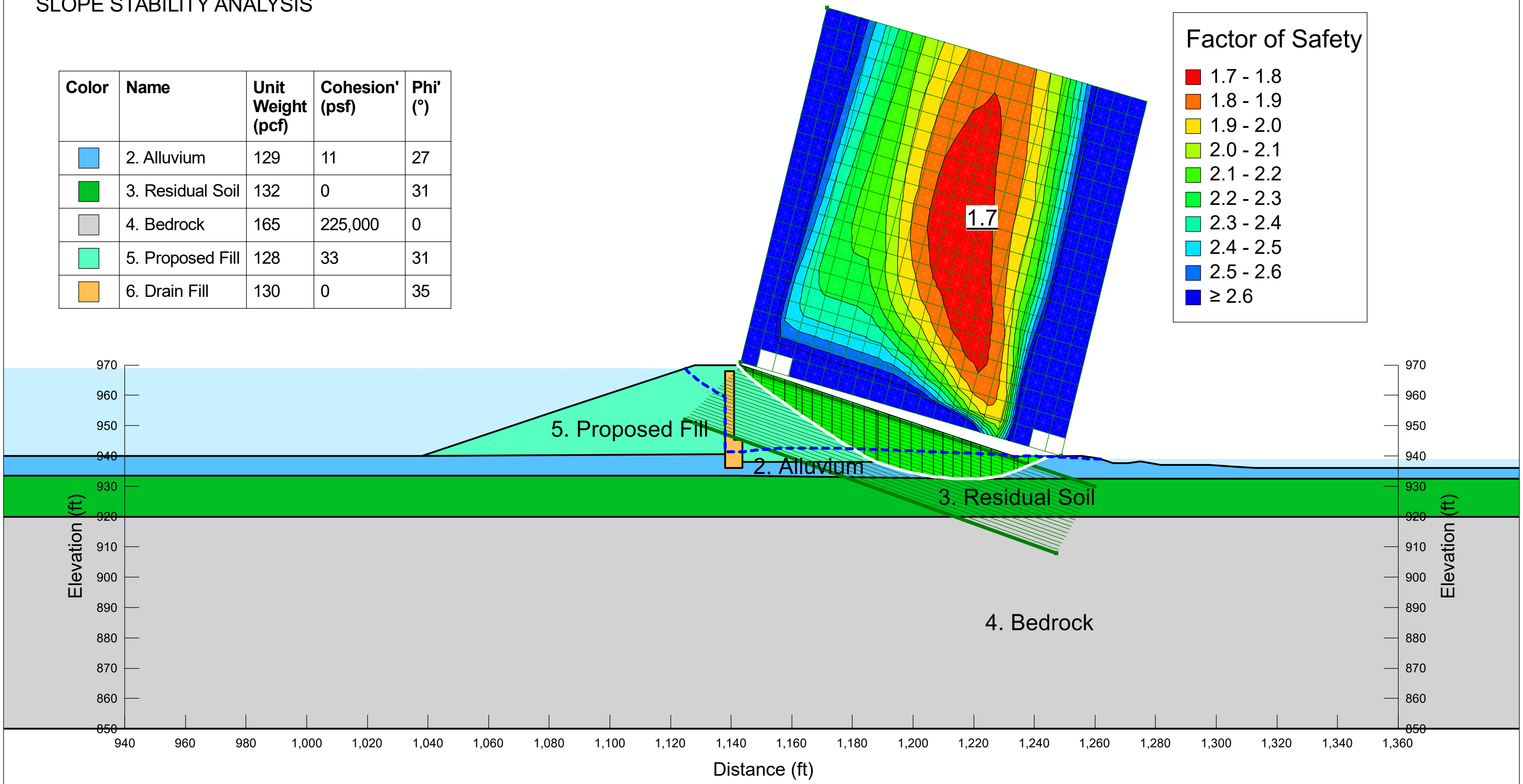
🟩	3. Residual Soil	132	0	31
⬜	4. Bedrock	165	225,000	0
🟩	5. Proposed Fill	128	33	31
🟧	6. Drain Fill	130	0	35



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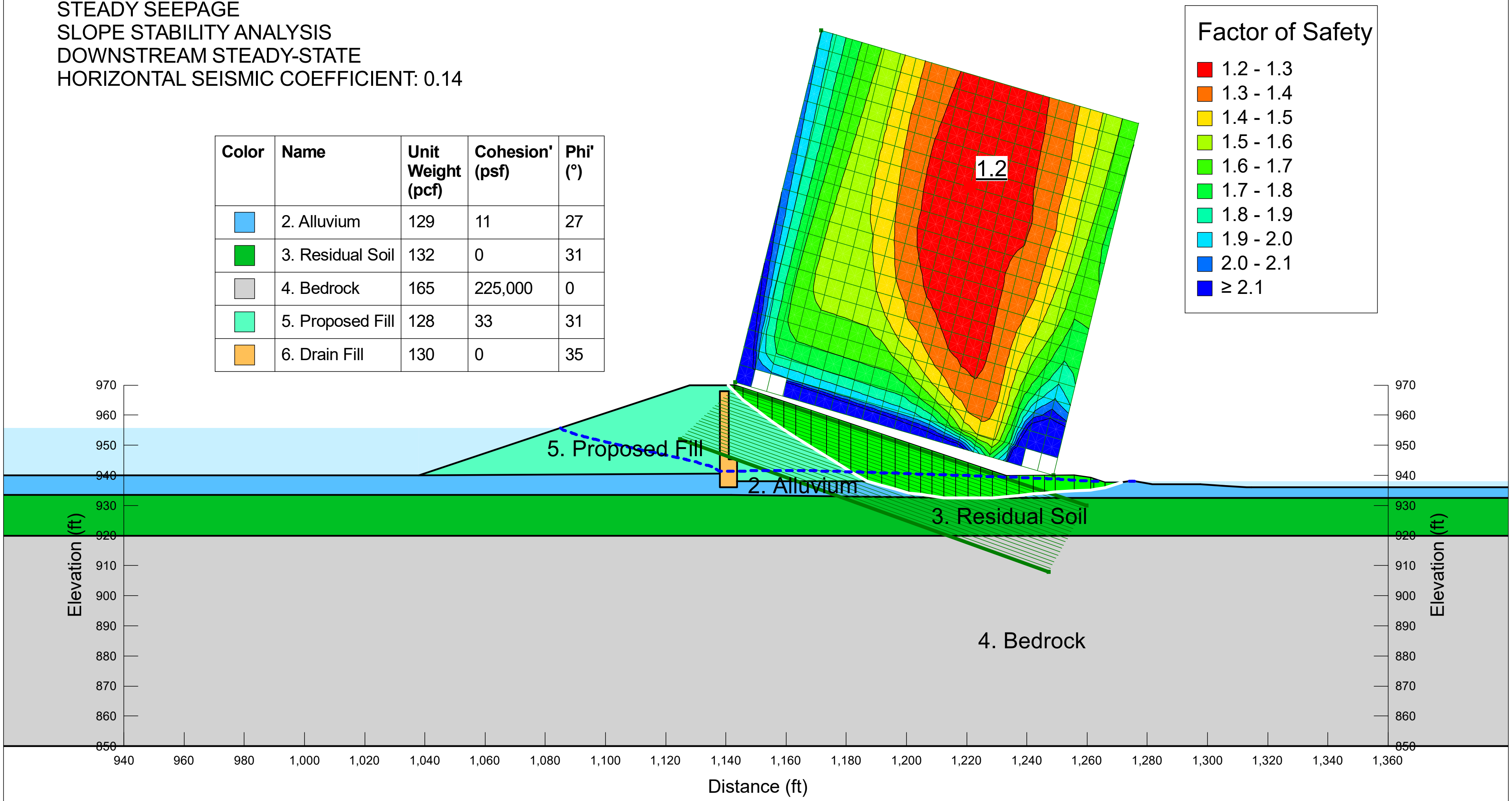
Lake Erin Dam
DeKalb County, GA

Slope Stability Analysis

Date: June 2024 | Project No. 60727041 | Draft

ERIN LAKE DAM-PROPOSED CONDITIONS
SEISMIC ANALYSIS-EFFECTIVE STRENGTH PARAMETERS
NORMAL POOL ELEVATION: 955.7 FT
TAILWATER ELEVATION: 938 FT
STEADY SEEPAGE
SLOPE STABILITY ANALYSIS
DOWNSTREAM STEADY-STATE
HORIZONTAL SEISMIC COEFFICIENT: 0.14

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟦	2. Alluvium	129	11	27
🟩	3. Residual Soil	132	0	31
⬜	4. Bedrock	165	225,000	0
🟩	5. Proposed Fill	128	33	31
🟧	6. Drain Fill	130	0	35



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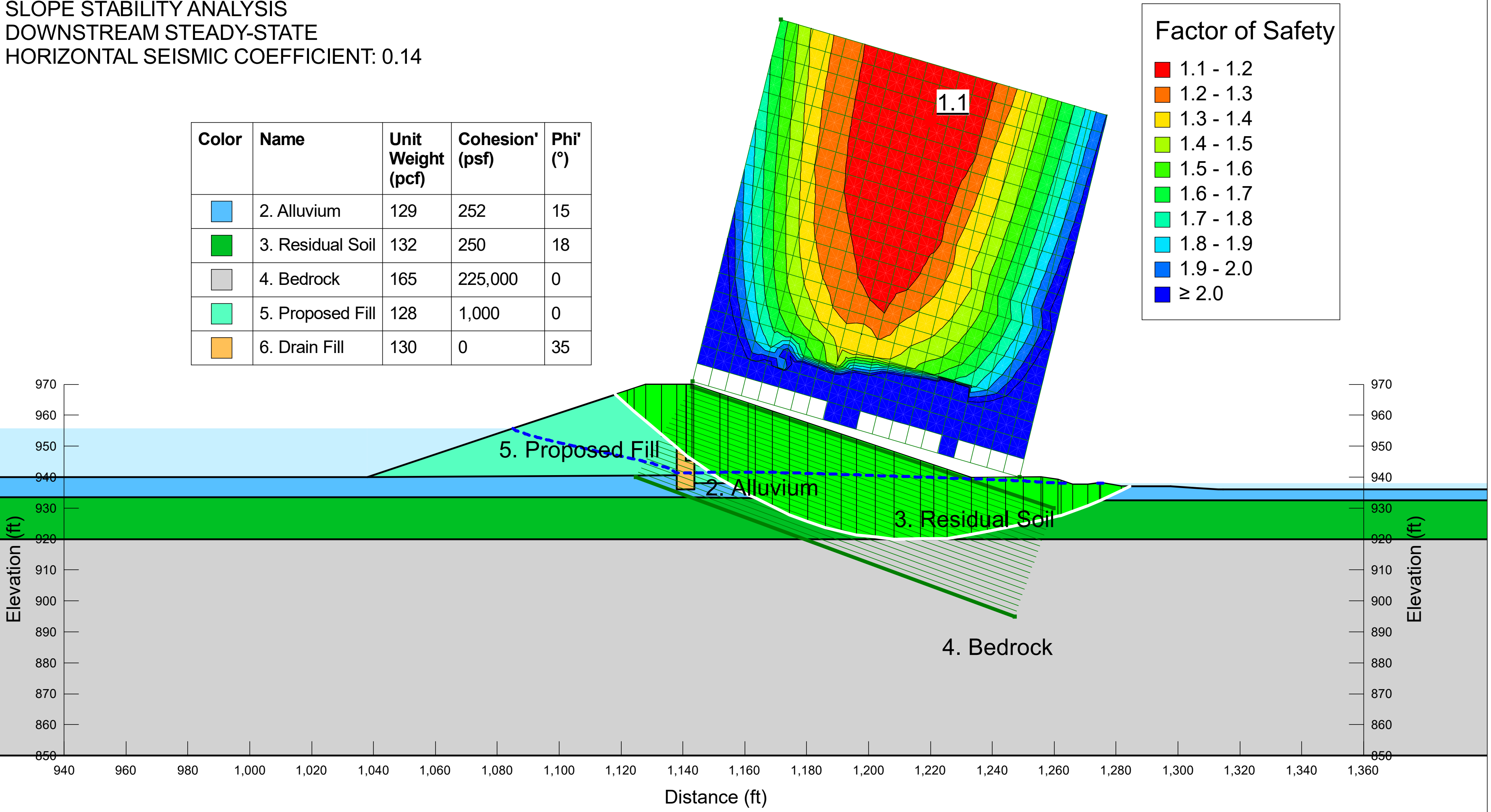
ERIN LAKE DAM-PROPOSED CONDITIONS
SEISMIC ANALYSIS-TOTAL STRENGTH PARAMETERS
NORMAL POOL ELEVATION: 955.7 FT
TAILWATER ELEVATION: 938 FT
STEADY SEEPAGE
SLOPE STABILITY ANALYSIS
DOWNSTREAM STEADY-STATE
HORIZONTAL SEISMIC COEFFICIENT: 0.14

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟦	2. Alluvium	129	252	15
🟩	3. Residual Soil	132	250	18
⬜	4. Bedrock	165	225,000	0
🟩	5. Proposed Fill	128	1,000	0
🟧	6. Drain Fill	130	0	35



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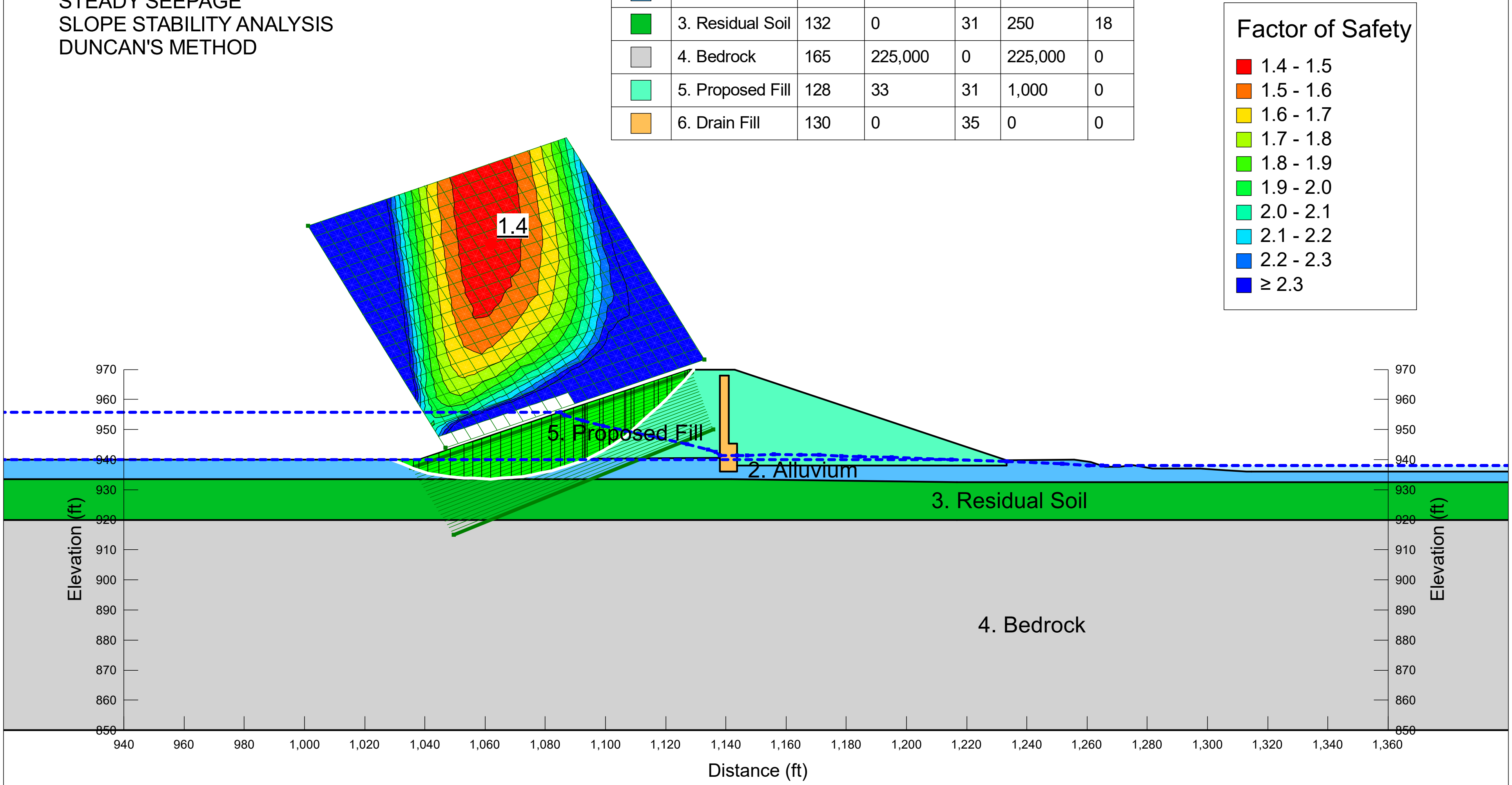
Draft

ERIN LAKE DAM-PROPOSED CONDITIONS
RAPID DRAWDOWN FROM NORMAL POOL TO EL. 940 FT
NORMAL POOL ELEVATION: 955.7 FT
TAILWATER ELEVATION: 938 FT
STEADY SEEPAGE
SLOPE STABILITY ANALYSIS
DUNCAN'S METHOD

Color	**Name**	**Unit Weight (pcf)**	**Cohesion' (psf)**	**Phi' (°)**	**Cohesion R (psf)**	**Phi R (°)**
🟦	2. Alluvium	129	11	27	252	15
🟩	3. Residual Soil	132	0	31	250	18
⬜	4. Bedrock	165	225,000	0	225,000	0
🟩	5. Proposed Fill	128	33	31	1,000	0
🟧	6. Drain Fill	130	0	35	0	0



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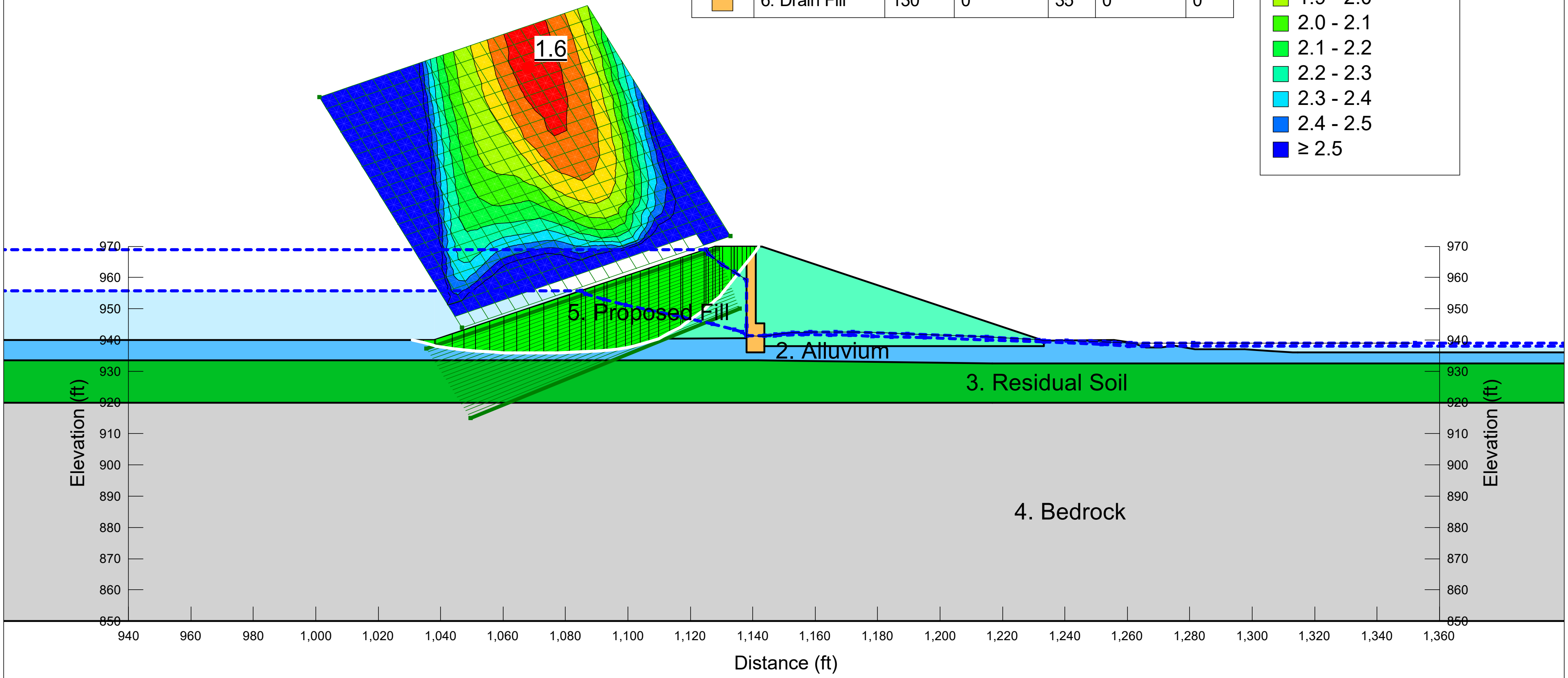
Date: June 2024	Project No. 60727041	Draft

ERIN LAKE DAM- PROPOSED CONDITIONS
RAPID DRAWDOWN - FROM MAXIMUM POOL TO NORMAL POOL
MAXIMUM POOL ELEVATION: 969 FT
NORMAL POOL ELEVATION: 955.7 FT
SLOPE STABILITY ANALYSIS
DUNCAN'S METHOD

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Cohesion R (psf)	Phi R (°)
🟦	2. Alluvium	129	11	27	252	15
🟩	3. Residual Soil	132	0	31	250	18
⬜	4. Bedrock	165	225,000	0	225,000	0
🟩	5. Proposed Fill	128	33	31	1,000	0
🟧	6. Drain Fill	130	0	35	0	0

Factor of Safety

- 🟥 1.6 - 1.7
- 🟧 1.7 - 1.8
- 🟨 1.8 - 1.9
- 🟩 1.9 - 2.0
- 🟩 2.0 - 2.1
- 🟩 2.1 - 2.2
- 🟩 2.2 - 2.3
- 🟦 2.3 - 2.4
- 🟦 2.4 - 2.5
- 🟦 ≥ 2.5



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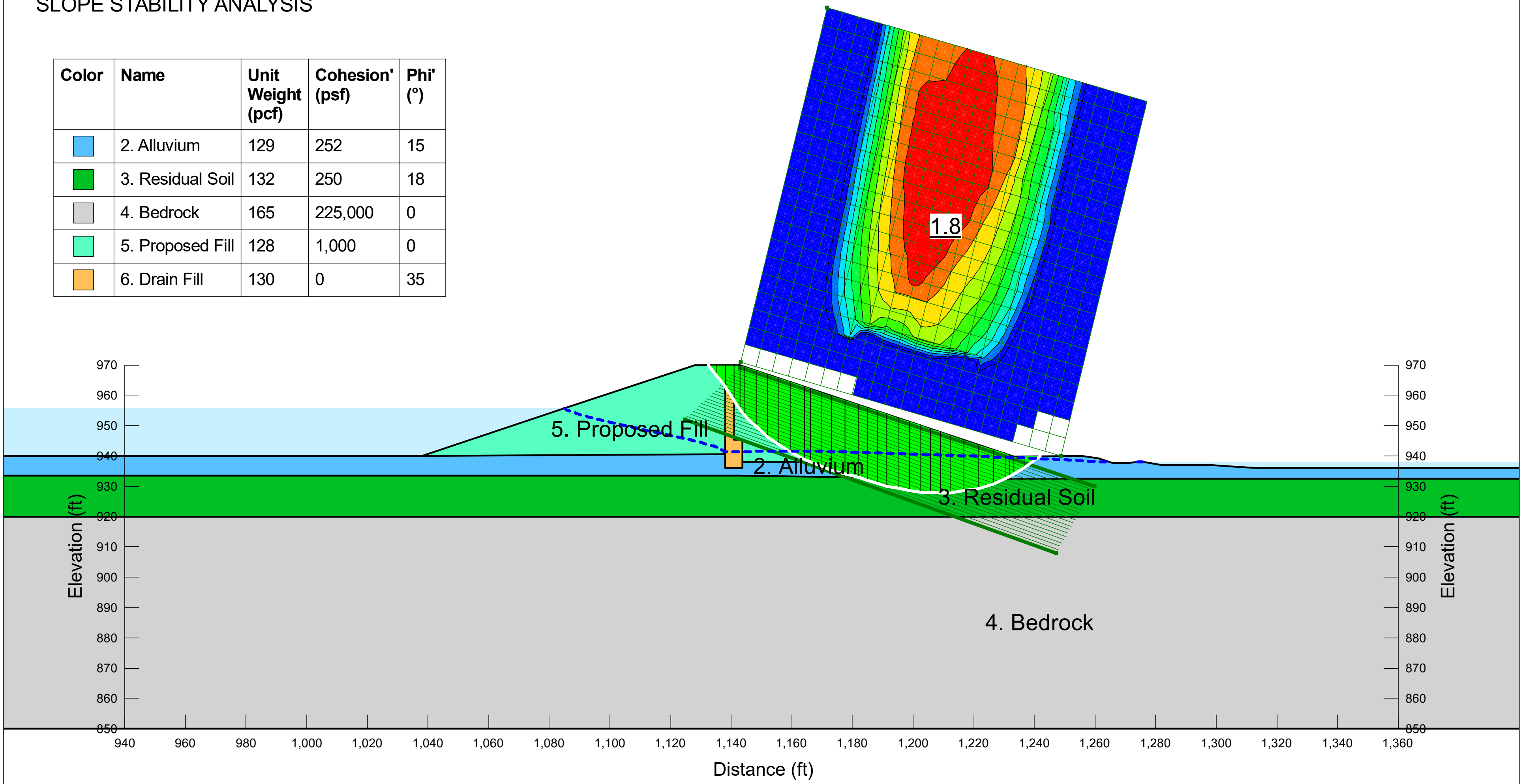
ERIN LAKE DAM- PROPOSED CONDITIONS
END OF CONSTRUCTION - TOTAL STRENGTH PARAMETERS
NORMAL POOL: 955.7 FT
STEADY SEEPAGE
SLOPE STABILITY ANALYSIS

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟦	2. Alluvium	129	252	15
🟩	3. Residual Soil	132	250	18
⬜	4. Bedrock	165	225,000	0
🟩	5. Proposed Fill	128	1,000	0
🟧	6. Drain Fill	130	0	35



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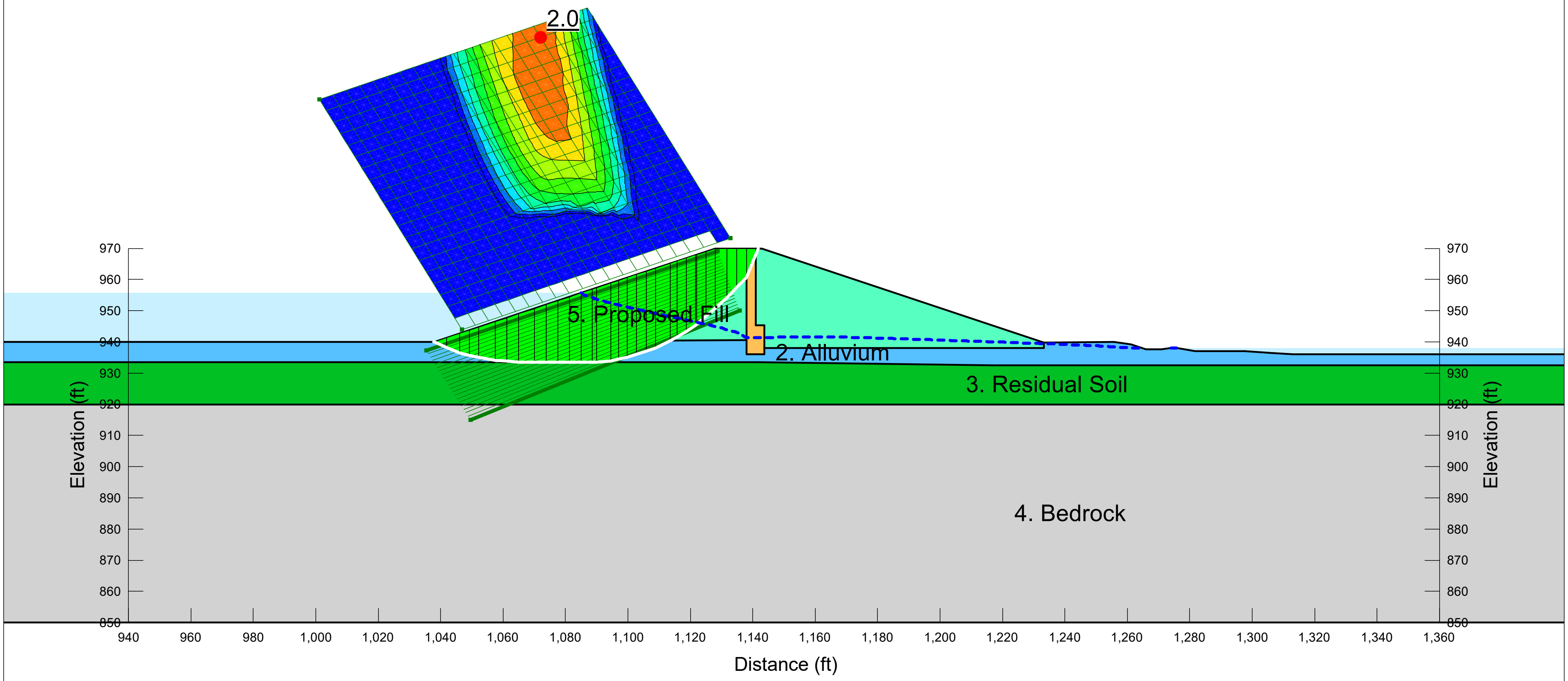
ERIN LAKE DAM- PROPOSED CONDITIONS
END OF CONSTRUCTION - TOTAL STRENGTH PARAMETERS
NORMAL POOL: 955.7 FT
STEADY SEEPAGE
SLOPE STABILITY ANALYSIS

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟦	2. Alluvium	129	252	15
🟩	3. Residual Soil	132	250	18
⬜	4. Bedrock	165	225,000	0
🟩	5. Proposed Fill	128	1,000	0
🟧	6. Drain Fill	130	0	35



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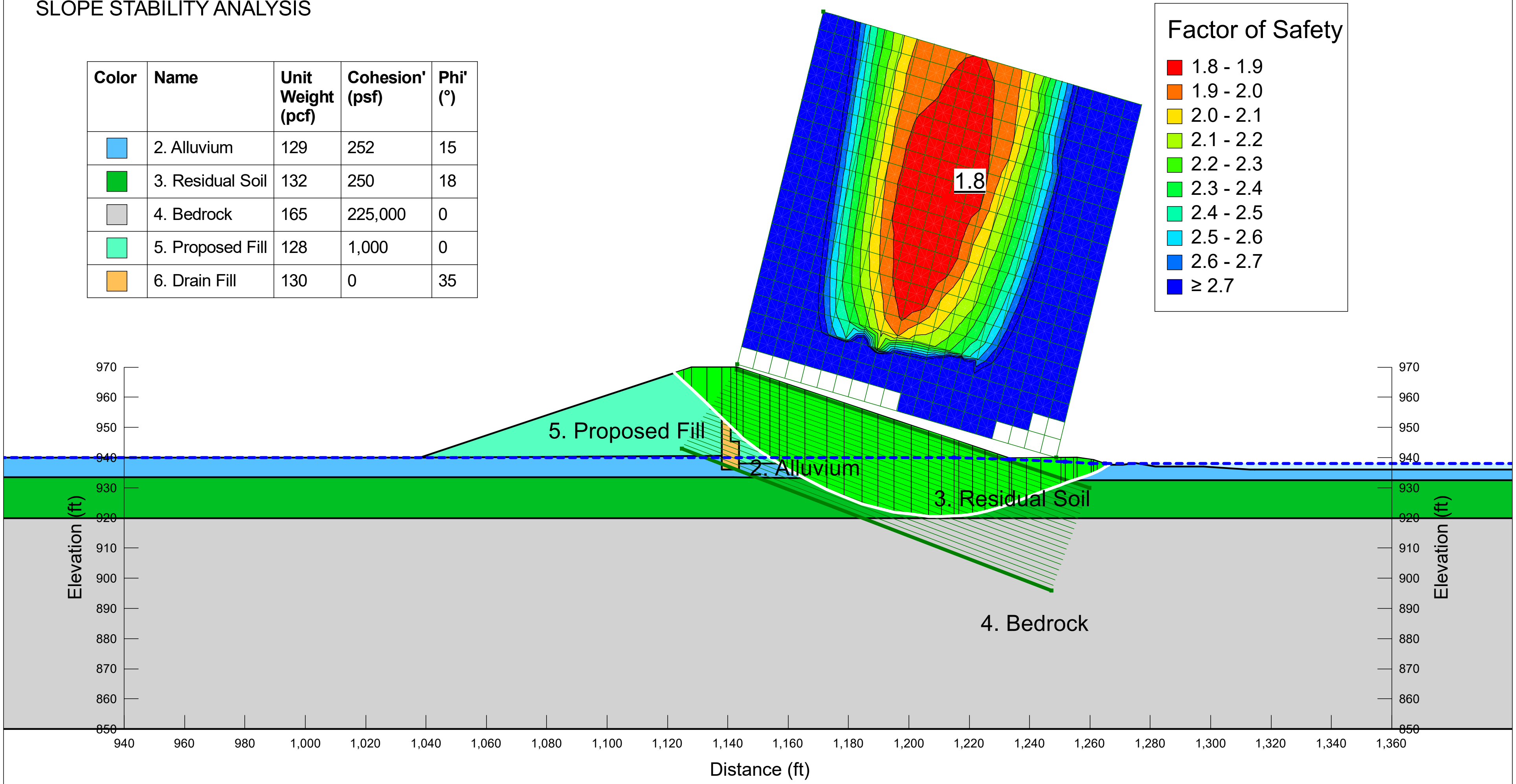
Slope Stability Analysis

Date: June 2024 | Project No. 60727041 | Draft

ERIN LAKE DAM- PROPOSED CONDITIONS
END OF CONSTRUCTION - TOTAL STRENGTH PARAMETERS
UPSTREAM GROUNDWATER SURFACE AT 942 FT
DOWNSTREAM GROUNDWATER SURFACE AT 938 FT
STEADY SEEPAGE
SLOPE STABILITY ANALYSIS

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟦	2. Alluvium	129	252	15
🟩	3. Residual Soil	132	250	18
⬜	4. Bedrock	165	225,000	0
🟩	5. Proposed Fill	128	1,000	0
🟧	6. Drain Fill	130	0	35



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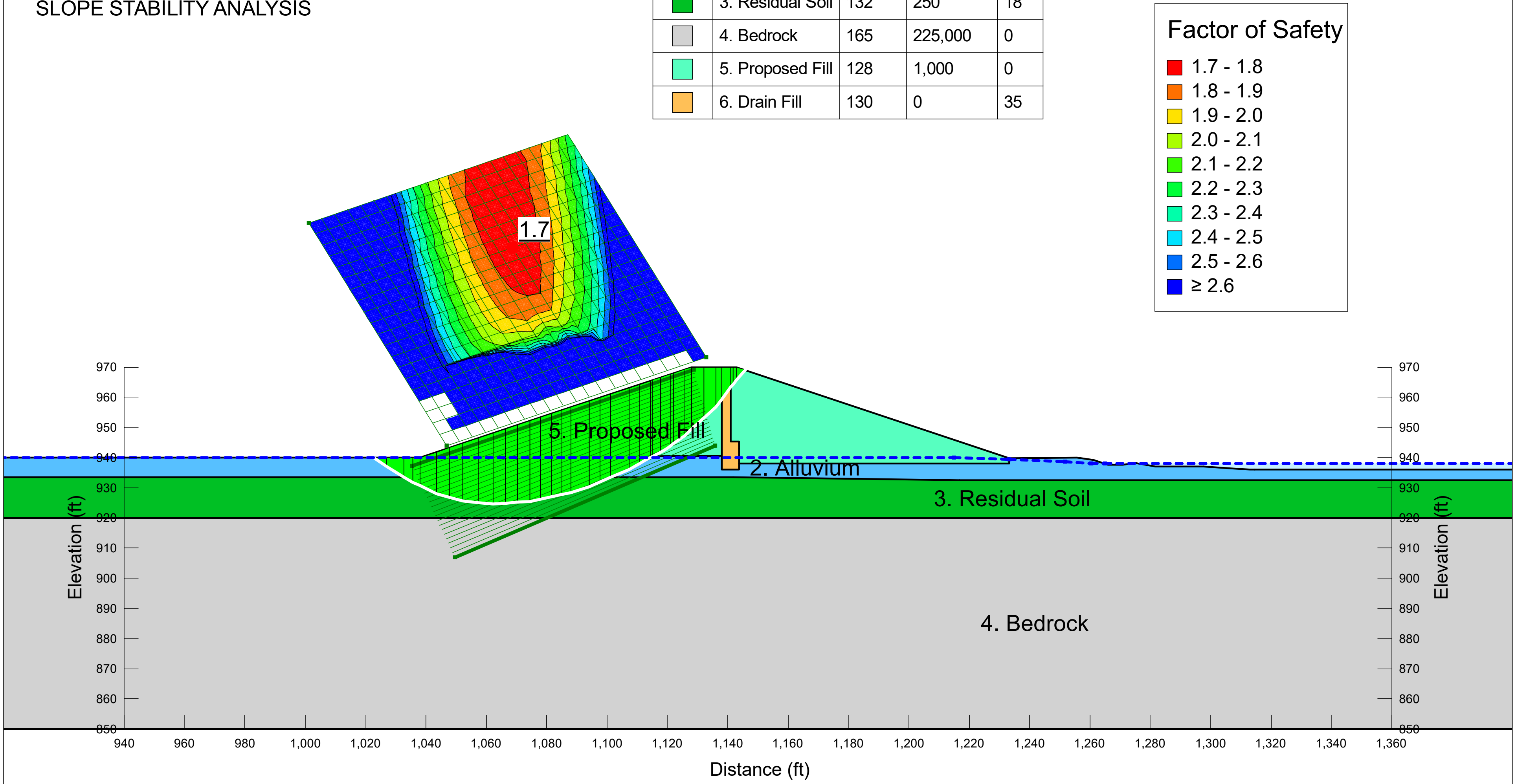
ERIN LAKE DAM- PROPOSED CONDITIONS
END OF CONSTRUCTION - TOTAL STRENGTH PARAMETERS
UPSTREAM GROUNDWATER SURFACE AT 942 FT
DOWNSTREAM GROUNDWATER SURFACE AT 938 FT
STEADY SEEPAGE
SLOPE STABILITY ANALYSIS

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟦	2. Alluvium	129	252	15
🟩	3. Residual Soil	132	250	18
⬜	4. Bedrock	165	225,000	0
🟩	5. Proposed Fill	128	1,000	0
🟧	6. Drain Fill	130	0	35



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Slope Stability Analysis

Date: June 2024

Project No. 60727041

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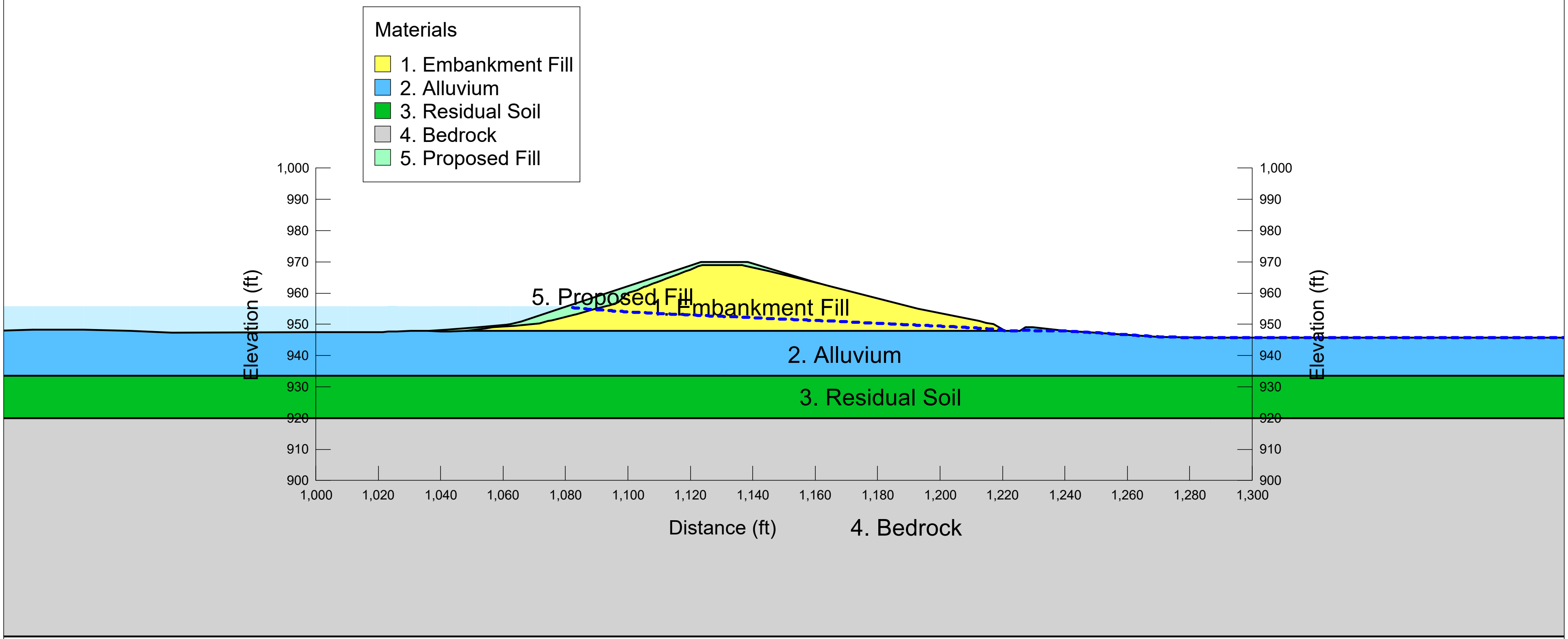
Seepage and Slope Stability Analysis

At 100 ft Offset Cross-Section

ERIN LAKE DAM-PROPOSED CONDITIONS 100 FT OFFSET FROM LOW LEVEL OUTLET
NORMAL POOL CONDITIONS
NORMAL POOL ELEVATION: 955.7 FT
MATERIAL LAYERS



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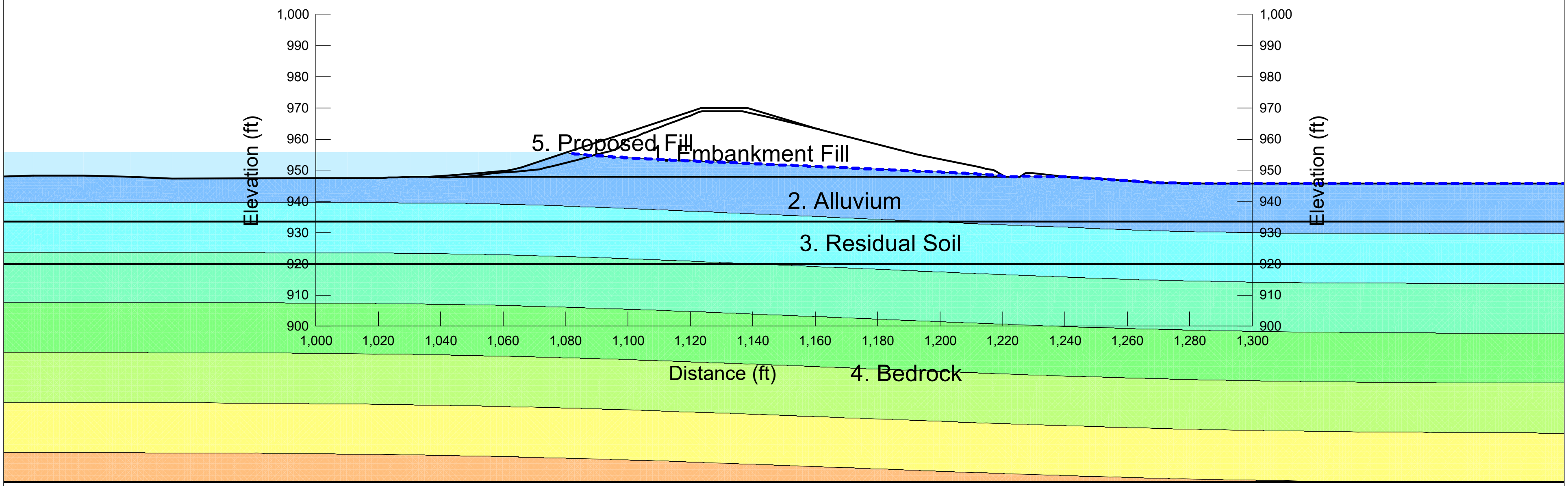
ERIN LAKE DAM-PROPOSED CONDITIONS 100 FT OFFSET FROM LOW LEVEL OUTLET
NORMAL POOL CONDITIONS
NORMAL POOL ELEVATION: 955.7 FT
STEADY SEEPAGE

Name	Kh cm/sec	Kv/Kh Ratio
1. Embankment Fill	1.08E-06	0.25
2. Alluvium	1.10E-06	0.67
3. Residual Soil	2.50E-06	0.5
4. Bedrock	1.00E-09	1
5. Proposed Fill	1.08E-06	0.25

Water Pressure

- 🟦 0 - 1,000 psf
- 🟦 1,000 - 2,000 psf
- 🟩 2,000 - 3,000 psf
- 🟩 3,000 - 4,000 psf
- 🟩 4,000 - 5,000 psf
- 🟨 5,000 - 6,000 psf
- 🟧 6,000 - 7,000 psf
- 🟥 7,000 - 8,000 psf



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Seepage Analysis

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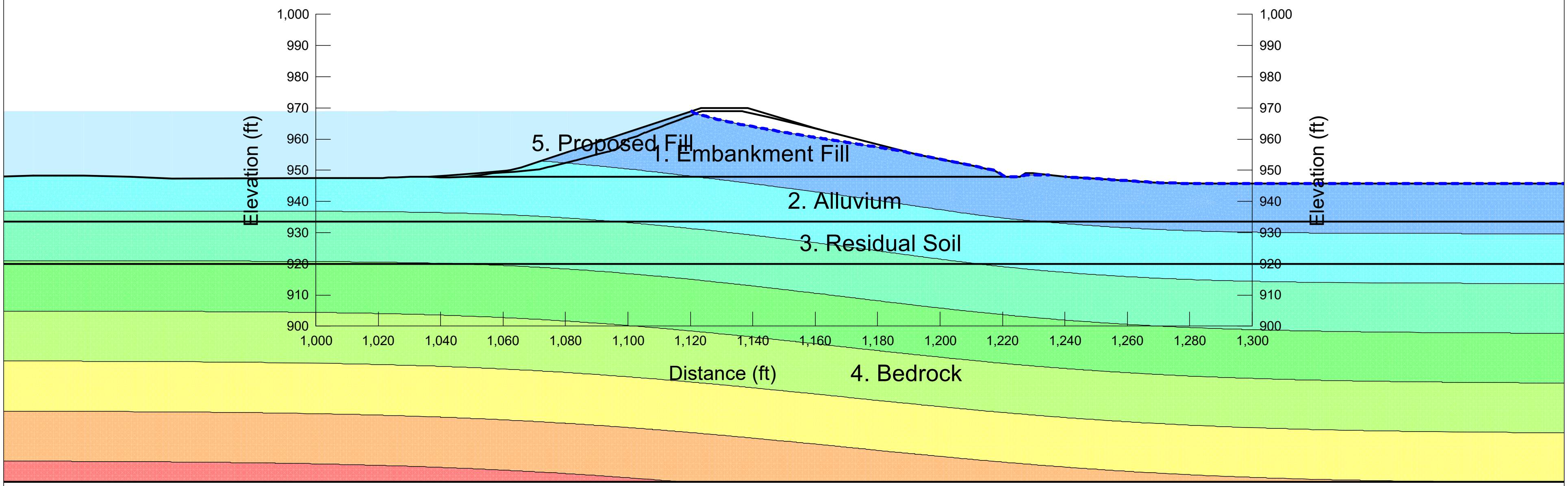
ERIN LAKE DAM-PROPOSED CONDITIONS 100 FT OFFSET FROM LOW LEVEL OUTLET
MAXIMUM POOL CONDITIONS
MAXIMUM POOL ELEVATION: 969 FT
STEADY SEEPAGE

Name	Kh cm/sec	Kv/Kh Ratio
1. Embankment Fill	1.08E-06	0.25
2. Alluvium	1.10E-06	0.67
3. Residual Soil	2.50E-06	0.5
4. Bedrock	1.00E-09	1
5. Proposed Fill	1.08E-06	0.25

Water Pressure

🟦	0 - 1,000 psf
🟦	1,000 - 2,000 psf
🟩	2,000 - 3,000 psf
🟩	3,000 - 4,000 psf
🟩	4,000 - 5,000 psf
🟨	5,000 - 6,000 psf
🟧	6,000 - 7,000 psf
🟥	7,000 - 8,000 psf



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Seepage Analysis

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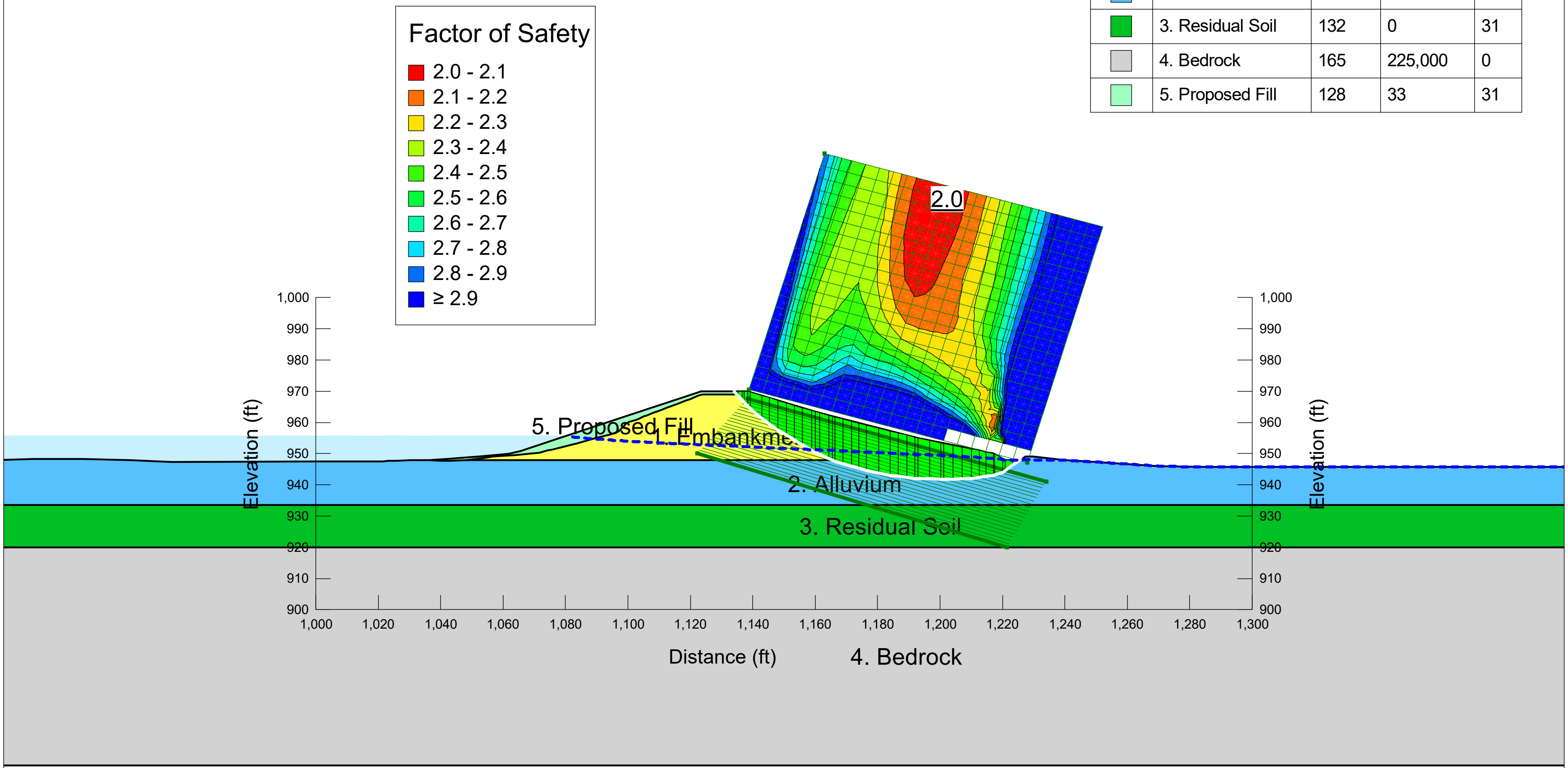
ERIN LAKE DAM-PROPOSED CONDITIONS 100 FT OFFSET FROM LOW LEVEL OUTLET
NORMAL POOL CONDITIONS
NORMAL POOL ELEVATION: 955.7 FT
STEADY SEEPAGE
SLOPE STABILITY ANALYSIS

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟨	1. Embankment Fill	125	33	28
🟦	2. Alluvium	129	11	27
🟩	3. Residual Soil	132	0	31
⬜	4. Bedrock	165	225,000	0
🟩	5. Proposed Fill	128	33	31



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Slope Stability Analysis

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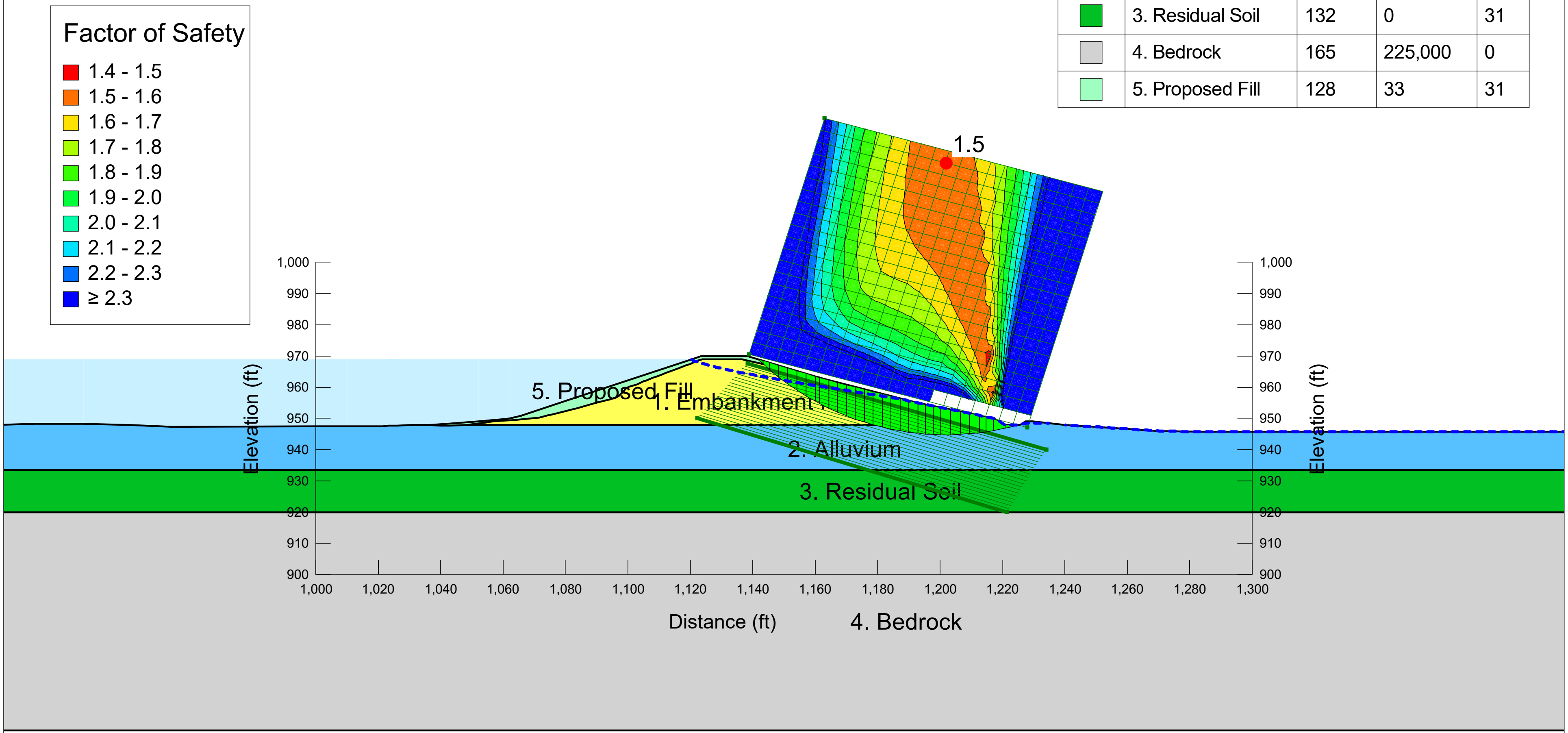
ERIN LAKE DAM-PROPOSED CONDITIONS 100 FT OFFSET FROM LOW LEVEL OUTLET
MAXIMUM POOL CONDITIONS
MAXIMUM POOL ELEVATION: 969 FT
STEADY SEEPAGE
SLOPE STABILITY ANALYSIS

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟨	1. Embankment Fill	125	33	28
🟦	2. Alluvium	129	11	27
🟩	3. Residual Soil	132	0	31
⬜	4. Bedrock	165	225,000	0
🟩	5. Proposed Fill	128	33	31



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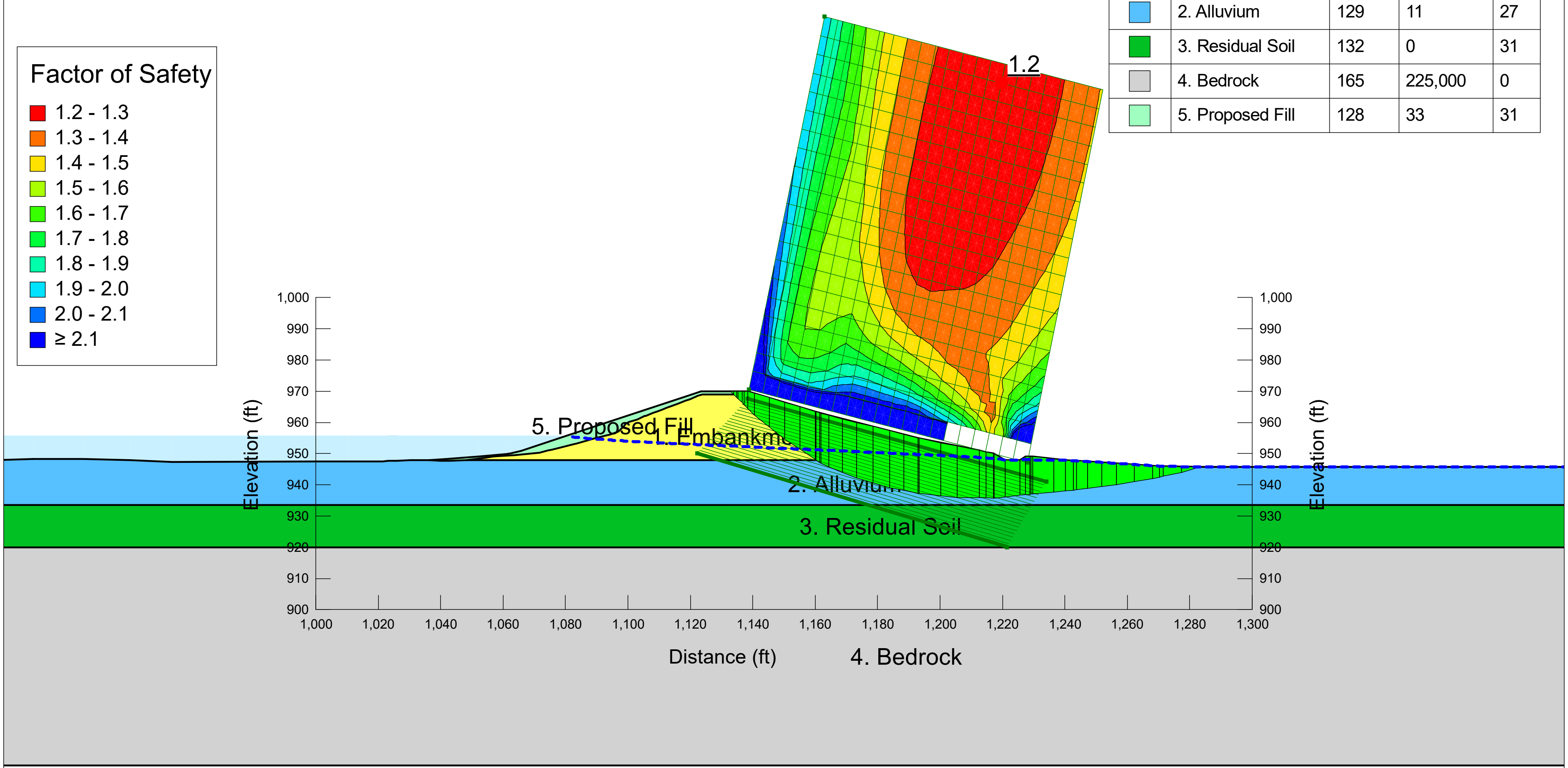
ERIN LAKE DAM-PROPOSED CONDITIONS 100 FT OFFSET FROM LOW LEVEL OUTLET
SEISMIC ANALYSIS-EFFECTIVE STRENGTH CONDITIONS
NORMAL POOL ELEVATION: 955.7 FT
STEADY SEEPAGE
SLOPE STABILITY ANALYSIS
HORIZONTAL SEISMIC COEFFICIENT: 0.14

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟨	1. Embankment Fill	125	33	28
🟦	2. Alluvium	129	11	27
🟩	3. Residual Soil	132	0	31
⬜	4. Bedrock	165	225,000	0
🟩	5. Proposed Fill	128	33	31



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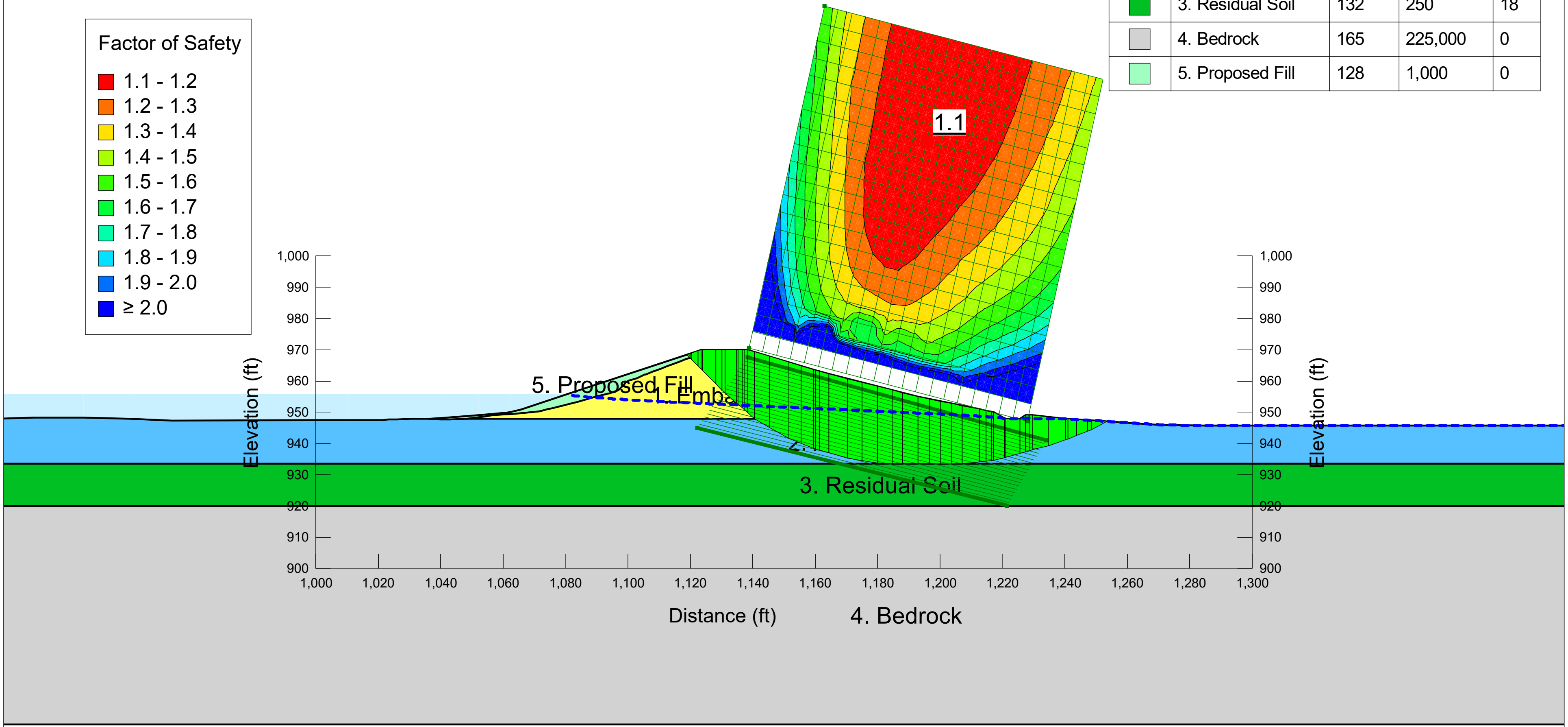
ERIN LAKE DAM-PROPOSED CONDITIONS 100 FT OFFSET FROM LOW LEVEL OUTLET
SEISMIC ANALYSIS- TOTAL STRENGTH PARAMETERS
NORMAL POOL ELEVATION: 955.7 FT
STEADY SEEPAGE
SLOPE STABILITY ANALYSIS
HORIZONTAL SIESMIC COEFFICIENT: 0.14

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟨	1. Embankment Fill	125	301	15
🟦	2. Alluvium	129	252	15
🟩	3. Residual Soil	132	250	18
⬜	4. Bedrock	165	225,000	0
🟩	5. Proposed Fill	128	1,000	0



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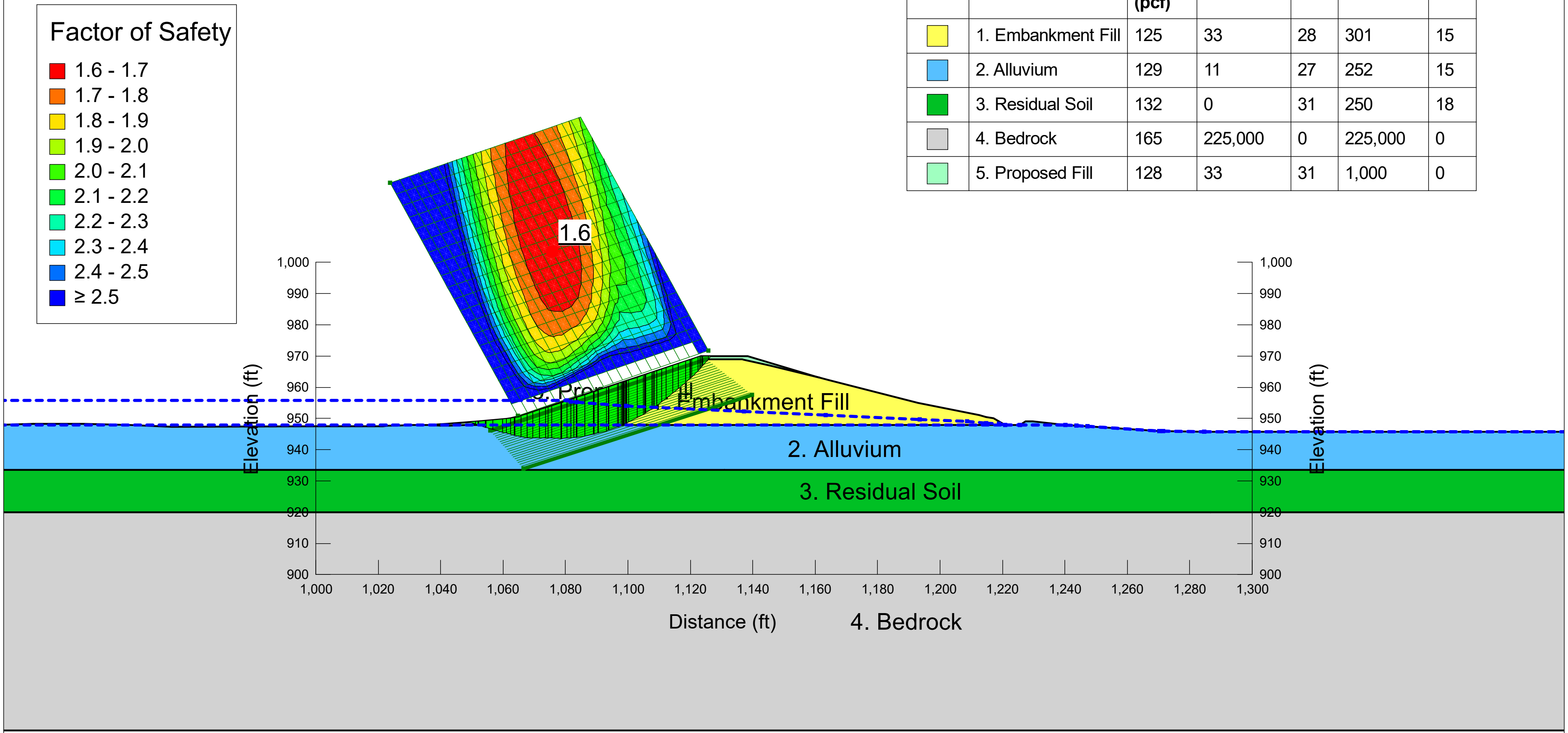
ERIN LAKE DAM-PROPOSED CONDITIONS 100 FT OFFSET FROM PRINCIPAL SPILLWAY CONDUIT
RAPID DRAWDOWN
NORMAL POOL ELEVATION: 955.7 FT DRAWN DOWN TO 948 FT
STEADY SEEPAGE
SLOPE STABILITY ANALYSIS
DUNCAN'S METHOD

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Cohesion R (psf)	Phi R (°)
🟨	1. Embankment Fill	125	33	28	301	15
🟦	2. Alluvium	129	11	27	252	15
🟩	3. Residual Soil	132	0	31	250	18
⬜	4. Bedrock	165	225,000	0	225,000	0
🟩	5. Proposed Fill	128	33	31	1,000	0



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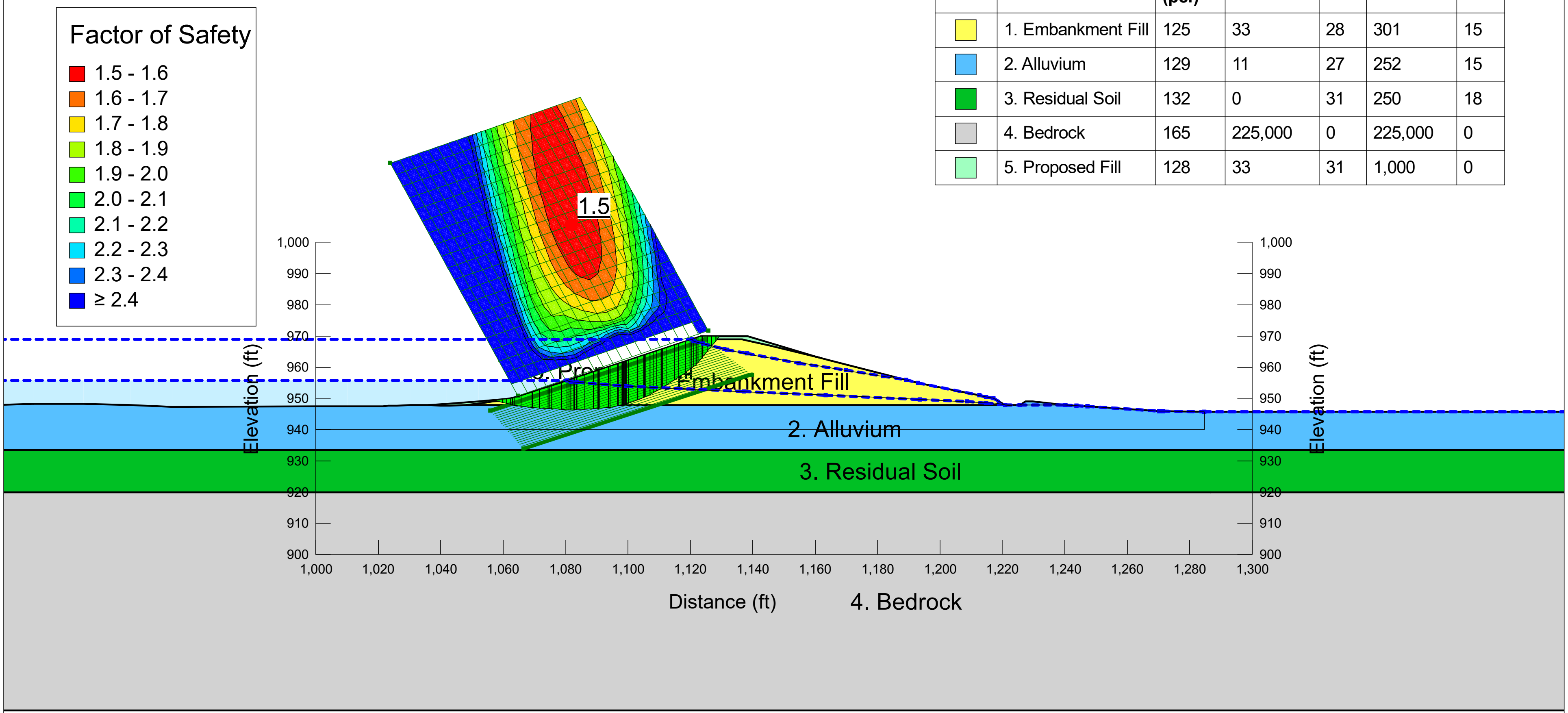
ERIN LAKE DAM-PROPOSED CONDITIONS 100 FT OFFSET FROM LOW LEVEL OUTLET
RAPID DRAWDOWN
MAXIMUM POOL ELEVATION: 969 FT DRAWN DOWN TO NORMAL POOL ELEVATION 955.7 FT
STEADY SEEPAGE
SLOPE STABILITY ANALYSIS
DUNCAN'S METHOD

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Cohesion R (psf)	Phi R (°)
🟨	1. Embankment Fill	125	33	28	301	15
🟦	2. Alluvium	129	11	27	252	15
🟩	3. Residual Soil	132	0	31	250	18
⬜	4. Bedrock	165	225,000	0	225,000	0
🟩	5. Proposed Fill	128	33	31	1,000	0



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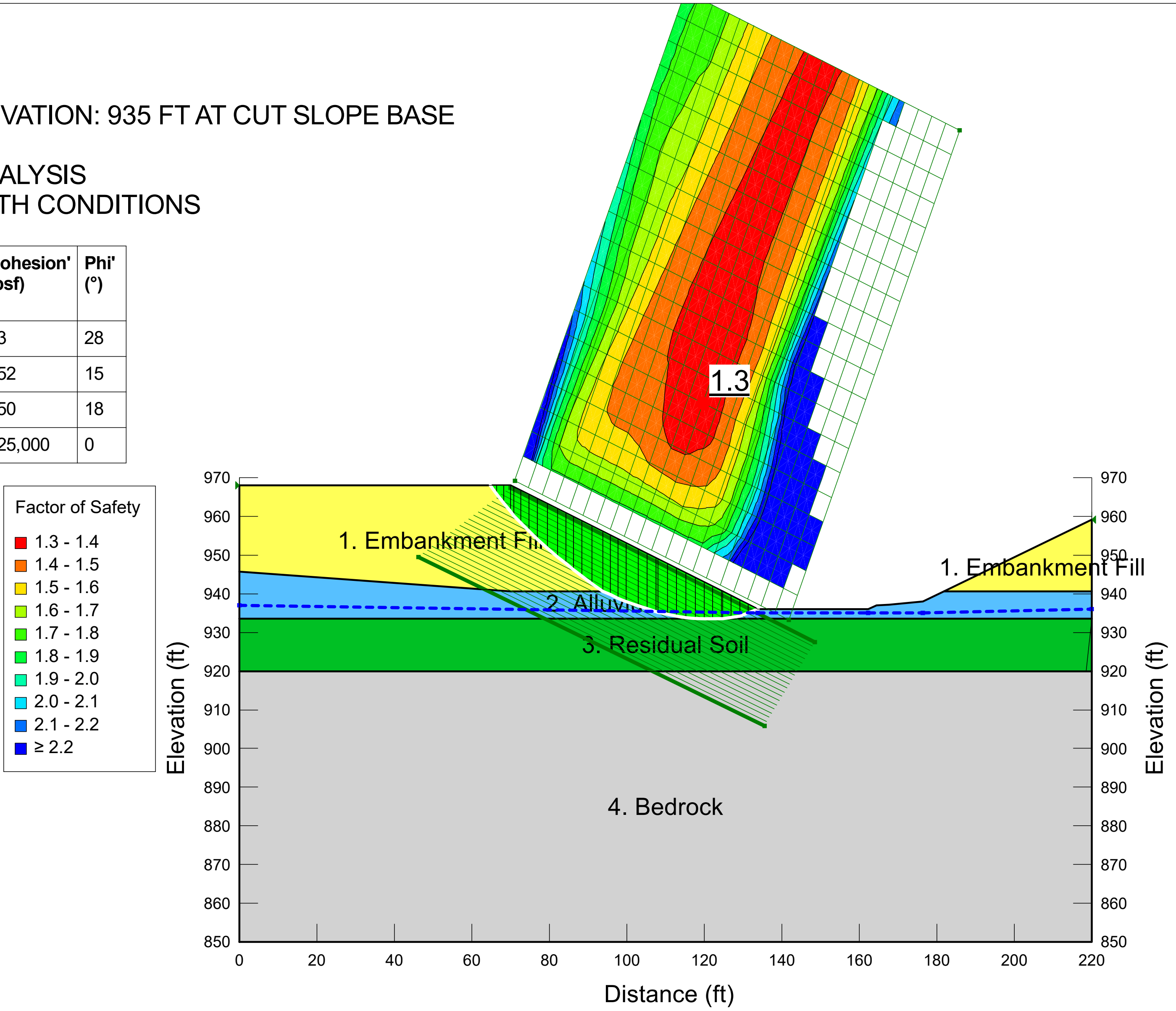
Seepage and Slope Stability Analysis

Excavated Slope Section

ERIN LAKE DAM
CUT SLOPE
GROUNDWATER ELEVATION: 935 FT AT CUT SLOPE BASE
STEADY SEEPAGE
SLOPE STABILITY ANALYSIS
EFFECTIVE STRENGTH CONDITIONS

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟨	1. Embankment Fill	124.8	33	28
🟦	2. Alluvium	128.6	252	15
🟩	3. Residual Soil	132	250	18
⬜	4. Bedrock	165	225,000	0



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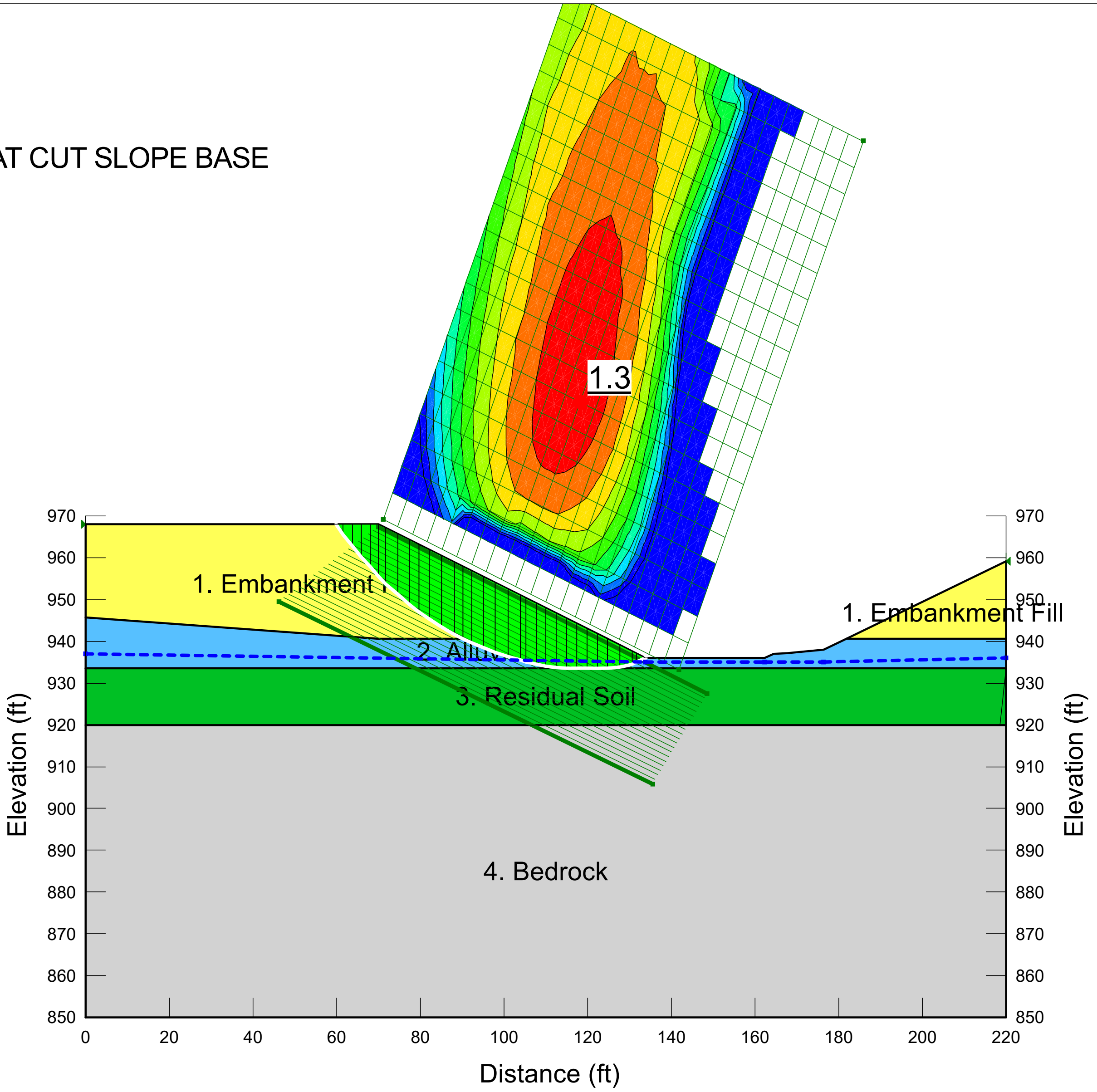
ERIN LAKE DAM
CUT SLOPE
GROUNDWATER ELEVATION: 935 FT AT CUT SLOPE BASE
STEADY SEEPAGE
SLOPE STABILITY ANALYSIS
TOTAL STRENGTH CONDITIONS

Color	**Name**	**Unit Weight (pcf)**	**Cohesion' (psf)**	**Phi' (°)**
🟨	1. Embankment Fill	124.8	301	15
🟦	2. Alluvium	128.6	252	15
🟩	3. Residual Soil	132	250	18
⬜	4. Bedrock	165	225,000	0

Factor of Safety

- 🟥 1.3 - 1.4
- 🟧 1.4 - 1.5
- 🟨 1.5 - 1.6
- 🟩 1.6 - 1.7
- 🟩 1.7 - 1.8
- 🟩 1.8 - 1.9
- 🟩 1.9 - 2.0
- 🟦 2.0 - 2.1
- 🟦 2.1 - 2.2
- 🟦 ≥ 2.2



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Slope Stability Analysis

Date: June 2024	Project No. 60727041	Draft

Bearing Capacity Analysis

Project: Erin Lake Dam Originator: TKW Date: 5/8/2024

Reviewed by: AS Date: 5/9/2024

CALCULATION PACKAGE: Bearing Capacity

Bearing Capacity

OBJECTIVE:

Determine the bearing capacity for the proposed Intake Structure and Impact Basin at Lake Erin Dam.

METHOD: Analyzed bearing capacity based on Das, B.M. “Principles of Foundation Engineering”, 5th Edition, Thomson Brooks/Cole Publication, Singapore, 2003. Mayerhof (1963) bearing capacity, shape, and inclinations factors were used in the analysis. Depth factors from Hansen (1970) were used in the analysis.

ASSUMPTIONS:

For the Intake Structure:

The foundation of the intake structure is designed as 18 ft by 15 ft. For this analysis, it was assumed the groundwater was at the ground surface. The depth of the ground surface to the base of the intake structure foundation is 6.1 ft and was conservatively assumed to be founded on Alluvium material. The material properties of the Alluvium are:

Saturated Unit weight: 128.8 pcf
Effective unit weight: 66.4 pcf
Friction Angle: 27 degrees
Cohesion: 11 psf

For the Impact Basin:

The foundation of the impact basin structure is designed as 32 ft by 24.6 ft. For this analysis, it was conservatively assumed the groundwater was at the ground surface. The depth of the ground surface to the base of the intake structure foundation is 7.5 ft and was conservatively assumed to be founded on Alluvium material. The material properties of the Alluvium are the same as described above.

RESULTS:

The calculations for the analysis are provided on the following spreadsheets. The results of the analysis shows the net allowable bearing capacity for the Intake Structure to be 3924 psf. The net allowable bearing capacity for the Impact Basin was determined to be 5390 psf.

CLIENT: DeKalb County
PROJECT: Erin Lake Dam
JOB NO: 60727041
LOCATION: DeKalb County, GA
COMP BY: TKW
DATE: 4/17/2024
CHECK BY: AS
DATE CHK: 5/3/2024

Structure Description

Name of the Surface Structure: Intake Tower
Relevent Borings: B-2, B-4

Assumptions

Maximum Anticipated Column Load:	0	kips
Relevent Borings:	B-2, B-4	

Reference:

Das, B. M., "Principles of Foundation Engineering," 5th Edition, Thomson Brooks/Cole Publication, Singapore, 2003

Key Elevations

Ground Surface Elevation:	942.1	ft		
Length of Footing (L):	18	ft		
Breadth of Footing (B):	15	ft		
Depth of the Footing (Df):	6.1	ft		
Bearing Pressure (psf):	0	psf		
Bottom of the Footing Elevation:	936	ft		
Depth to the Ground water from Ground Surface:	0	ft		
Ground Water Elevation:	942.1	ft		
Inclination of Load on Foundation with respect to vertical (β):	0	deg =	0.000	rad
Df/B =	0.4			
B/L =	0.8			

Soil Properties

Friction Angle (ϕ'):	27	deg =	0.471	rad
Undrained Cohesion (c'):	11	psf		
Moist Unit Weight (Υ_m):	114	pcf		
Saturated Unit Weight (Υ_{sat}):	128.8	pcf		
Unit Weight of Water (Υ_w):	62.4	pcf		
Submerged Unit Weight ($\Upsilon' = \Upsilon_{sat}-\Upsilon_w$):	66.4	pcf		
Effective Surcharge (q) =	405	psf		
Unit Weight of the Soil in the Bearing Stratum "ϒ" =	66.4	pcf		

Calculation

	Mayerhof (1963)		
Bearing Capacity Factors	N_c	N_q	N_γ
	23.94	13.20	9.46

	Mayerhof (1963)		
Shape Factors	Fcs	Fqs	$F_{\gamma s}$
	1.3	1.1	1.1

	Hansen (1970)		
Depth Factors	Fcd	Fqd	$F_{\gamma d}$
	1.2	1.1	1.0

	Maherhof (1963)		
Inclination Factors	Fci	Fqi	$F_{\gamma i}$
	1.0	1.0	1.0

Based on the modified bearing capacity equation by Mayerhof (1963),

$$q_u = c' * N_c * F_{cs} * F_{cd} * F_{ci} + q * N_q * F_{qs} * F_{qd} * F_{qi} + \frac{1}{2}\gamma B * N_\gamma * F_{\gamma s} * F_{\gamma d} * F_{\gamma i}$$

Ultimate Bearing Capacity (q_u) = 12467 psf

Net Ultimate Bearing Capacity ($q_{net(u)}$) = 11772 psf

Factor of Safety = 3

Allowable Bearing Capacity ($q_{(all)}$) = 4156 psf

Net Allowable Bearing Capacity ($q_{net(all)}$) = 3924 psf **OK**

CLIENT: DeKalb County
PROJECT: Erin Lake Dam
JOB NO: 60727041
LOCATION: DeKalb County, GA

COMP BY: TKW
DATE: 3/27/2024
CHECK BY: AS
DATE CHK: 5/3/2024

Structure Description

Name of the Surface Structure: Impact Basin
Relevent Borings: B-6

Assumptions

Maximum Anticipated Column Load: 0 kips
Relevent Borings: B-6

Reference:

Das, B. M., "Principles of Foundation Engineering," 5th Edition, Thomson Brooks/Cole Publication, Singapore, 2003

Key Elevations

Ground Surface Elevation:	941.8	ft		
Length of Footing (L):	32	ft		
Breadth of Footing (B):	24.6	ft		
Depth of the Footing (Df):	7.5	ft		
Bearing Pressure (psf):	0	psf		
Bottom of the Footing Elevation:	934.3	ft		
Depth to the Ground water from Ground Surface:	0	ft		
Ground Water Elevation:	941.8	ft		
Inclination of Load on Foundation with respect to vertical (β):	0	deg =	0.000	rad

Df/B = 0.3
B/L = 0.8

Soil Properties

Friction Angle (ϕ'):	27	deg =	0.471	rad
Undrained Cohesion (c'):	11	psf		
Moist Unit Weight (Υ_{m}):	114	pcf		
Saturated Unit Weight (Υ_{sat}):	128.8	pcf		
Unit Weight of Water (Υ_{w}):	62.4	pcf		
Submerged Unit Weight ($\Upsilon' = \Upsilon_{sat}-\Upsilon_{w}$):	66.4	pcf		

Effective Surcharge (q) =	498	psf
Unit Weight of the Soil in the Bearing Stratum "ϒ" =	66.4	pcf

Calculation

	Mayerhof (1963)		
Bearing Capacity Factors	N_c	N_q	N_Υ
	23.94	13.20	9.46

	Mayerhof (1963)		
Shape Factors	Fcs	Fqs	$F_{\Upsilon s}$
	1.2	1.1	1.1

	Hansen (1970)		
Depth Factors	Fcd	Fqd	$F_{\Upsilon d}$
	1.1220	1.0926	1.0000

	Maherhof (1963)		
Inclination Factors	Fci	Fqi	$F_{\Upsilon i}$
	1.0	1.0	1.0

Based on the modified bearing capacity equation by Mayerhof (1963),

$$q_u = c' * N_c * F_{cs} * F_{cd} * F_{ci} + q * N_q * F_{qs} * F_{qd} * F_{qi} + \frac{1}{2}\gamma B * N_\gamma * F_{\gamma s} * F_{\gamma d} * F_{\gamma i}$$

Ultimate Bearing Capacity (q_u) = 17025 psf

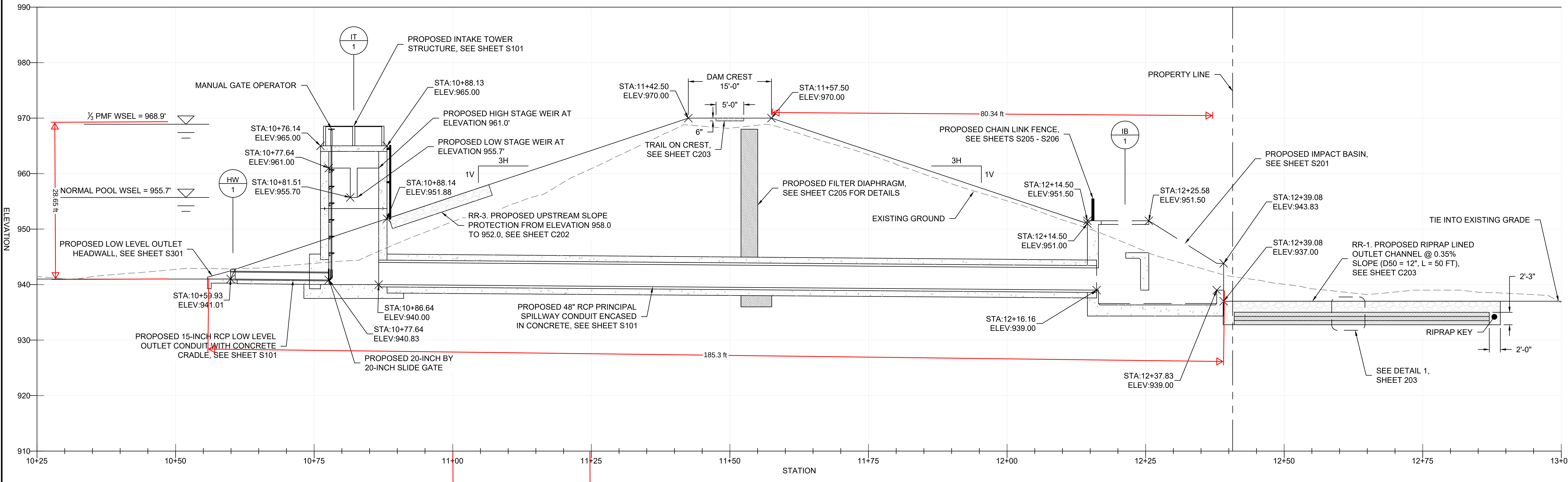
Net Ultimate Bearing Capacity ($q_{net(u)}$) = 16170 psf

Factor of Safety = 3

Allowable Bearing Capacity ($q_{(all)}$) = 5675 psf

Net Allowable Bearing Capacity ($q_{net(all)}$) = 5390 psf **OK**

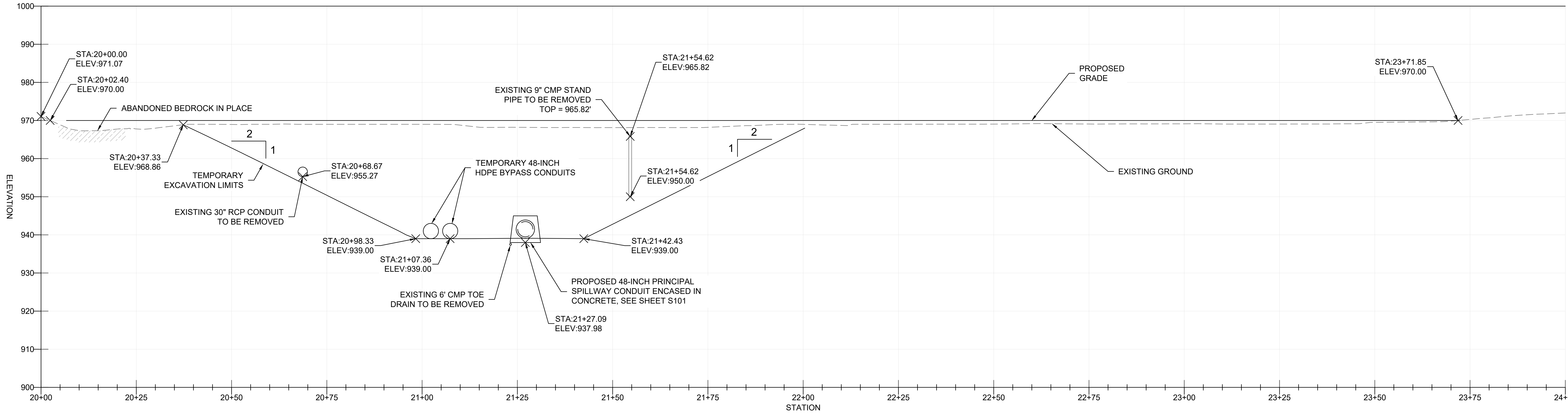
Settlement Analysis



1 C201 **PRINCIPAL SPILLWAY CENTERLINE PROFILE**

Scale 1"=10'

0 10' 20'
SCALE:1"=10'



1 C201 **EMBANKMENT CENTERLINE PROFILE**

Scale 1"=15'

0 15' 30'
SCALE:1"=15'

LAST SAVED BY: WEISA(2024-06-21) LAST PLOTTED: 2024-06-21
FILENAME: L:\DCS\PROJECTS\WTR\60727041_LAKEERINDAMENGINEERING\900_CAD_GIS\910_CAD\04_SHEETS\PP-C201-LAKEERIN.DWG

AECOM

PROJECT

LAKE ERIN DAM
REHABILITATION
DEKALB COUNTY, GEORGIA

95% DESIGN
NOT FOR CONSTRUCTION

CLIENT

CITY OF TUCKER
1975 LAKESIDE PKWY
SUITE 350,
TUCKER, GA 30084
770-865-5645 TEL
WWW.TUCKERGA.GOV

CONSULTANT

AECOM
12420 MILESTONE CENTER DRIVE
SUITE 150
GERMANTOWN, MD 20876
(301) 944-2354 TEL
WWW.AECOM.COM

REGISTRATION

ISSUED FOR BIDDING
DATE BY

ISSUED FOR CONSTRUCTION
DATE BY

REVISIONS

NO.	DATE	DESCRIPTION

AECOM PROJECT NO:	60727041
DRAWN BY:	AJW/JES
DESIGNED BY:	JCG
CHECKED BY:	JBB
APPROVED BY:	RDP
PLOT DATE:	6/21/2024
SCALE:	AS SHOWN
ACAD VER:	2021

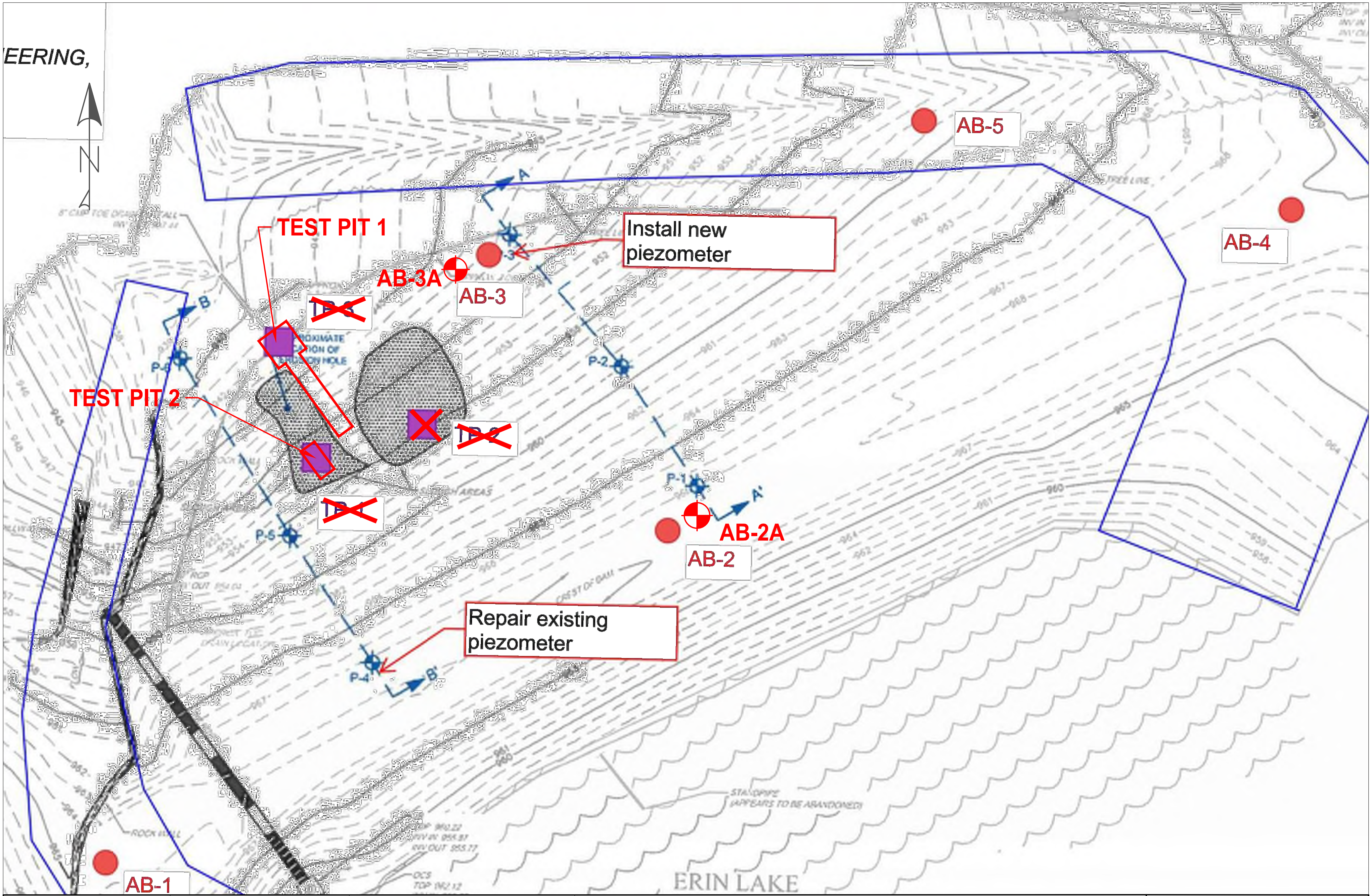
DRAWING TITLE

PRINCIPAL SPILLWAY AND
EMBANKMENT PROFILE

SHEET NUMBER

C201

SHEET 08 OF 48

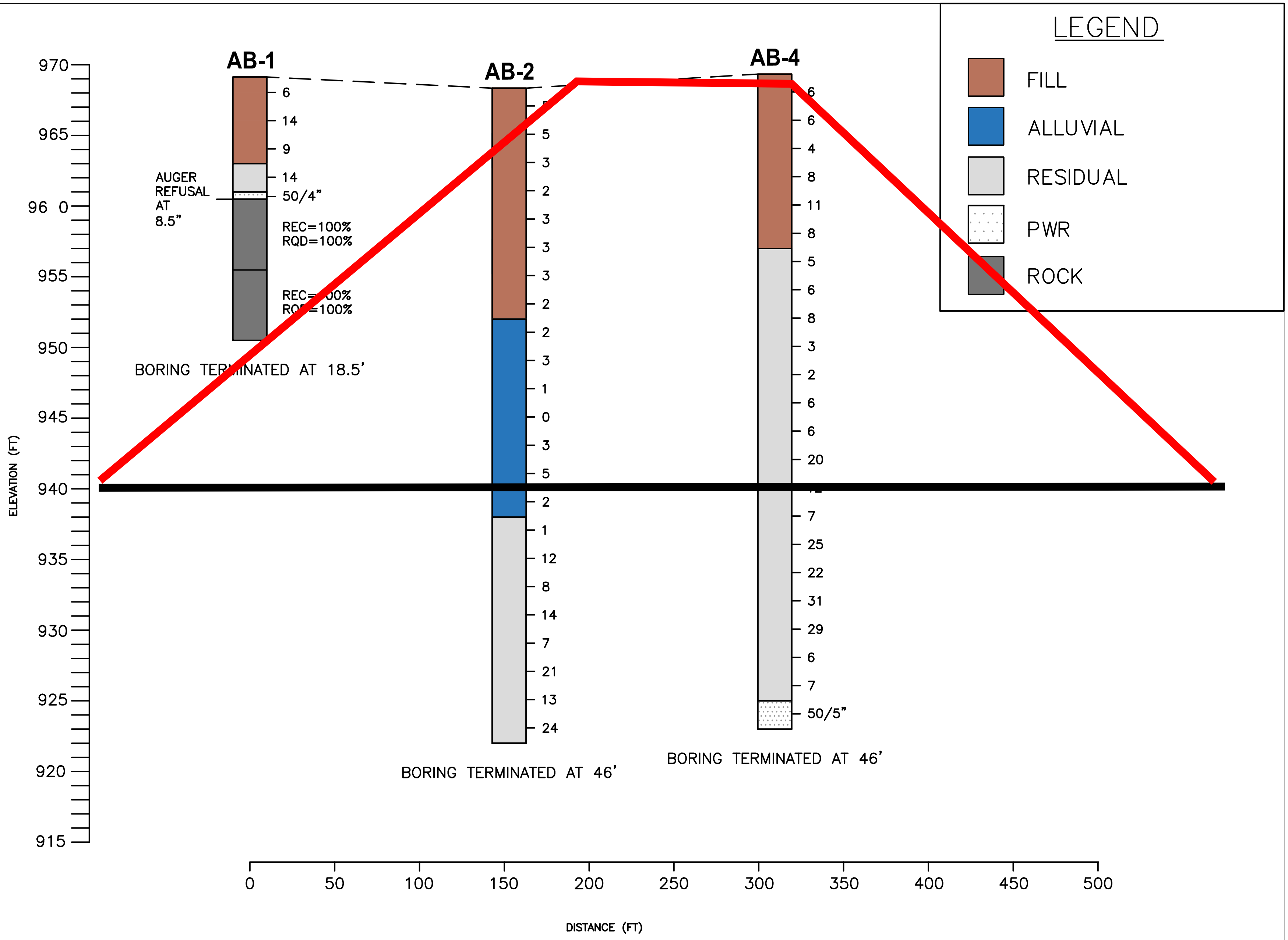


SCALE: AS SHOWN	DATE: 08/20/2021	PROJECT NO: 10205	TITLE:
PREPARED: JL	CHECKED: KK	REVISIONS:	
CLIENT: **AECOM**			

TEST LOCATION PLAN
ERIN LAKE DAM
DEKALB COUNTY, GEORGIA

ACCURA ENGINEERING AND CONSULTING SERVICES, INC.
3200 PRESIDENTIAL DR ATLANTA, GA 30340
PHONE: 404.241.8722 FAX: 404.241.4577

DRAWING NO. 02



SCALE: AS SHOWN	DATE: 08/27/2021	PROJECT NO: 10205
PREPARED: JL	CHECKED: KK	REVISIONS:

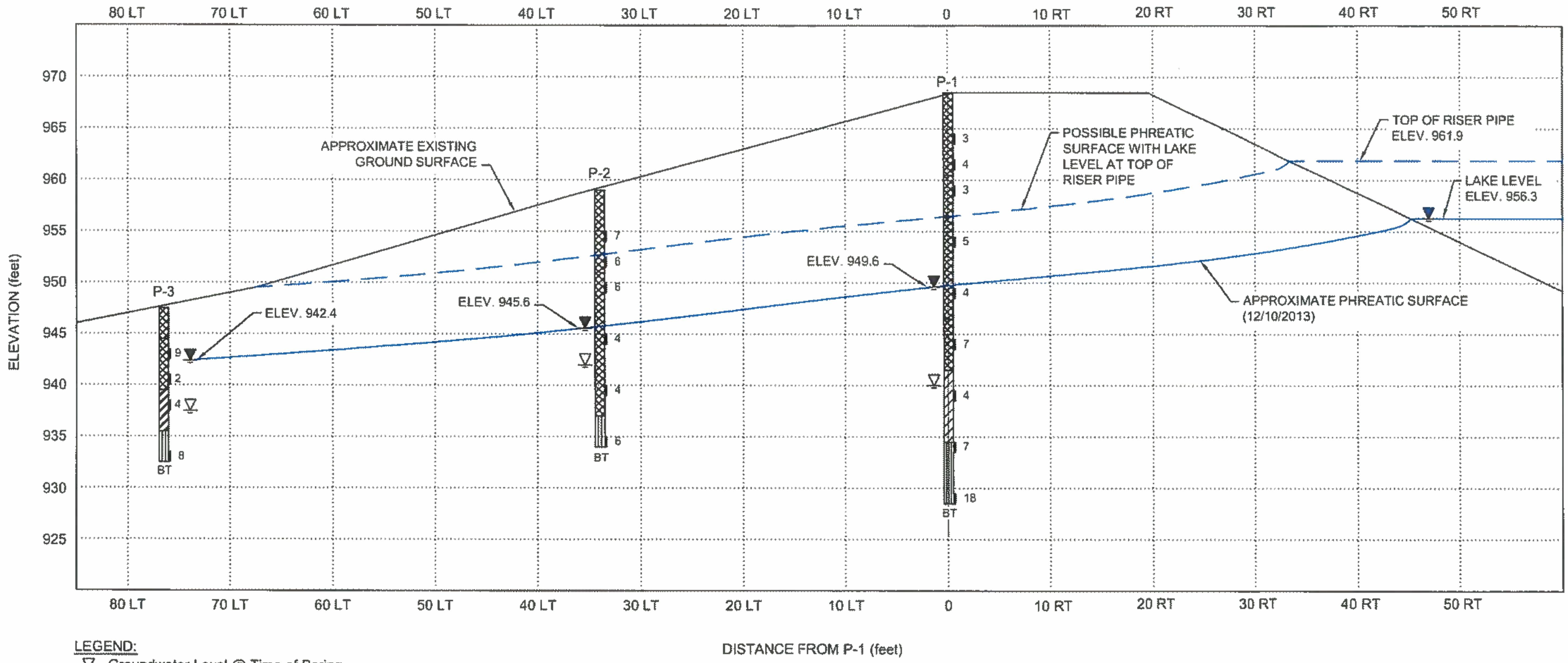
CLIENT: **AECOM**

TITLE:

SUBSURFACE PROFILE
ERIN LAKE DAM
DEKALB COUNTY, GEORGIA

ACCURA ENGINEERING AND CONSULTING SERVICES, INC.
3200 PRESIDENTIAL DR ATLANTA, GA 30340
PHONE: 404.241.8722 FAX: 404.241.4577

DRAWING NO. 03-01



LEGEND:

∇ - Groundwater Level @ Time of Boring

▼ - Groundwater Level on 12/10/2013

BT - Boring Terminated

SCALE: 1" = 10'

DATE: 2/3/2014

DRAWN BY: ZMH

REVIEWED BY: BD

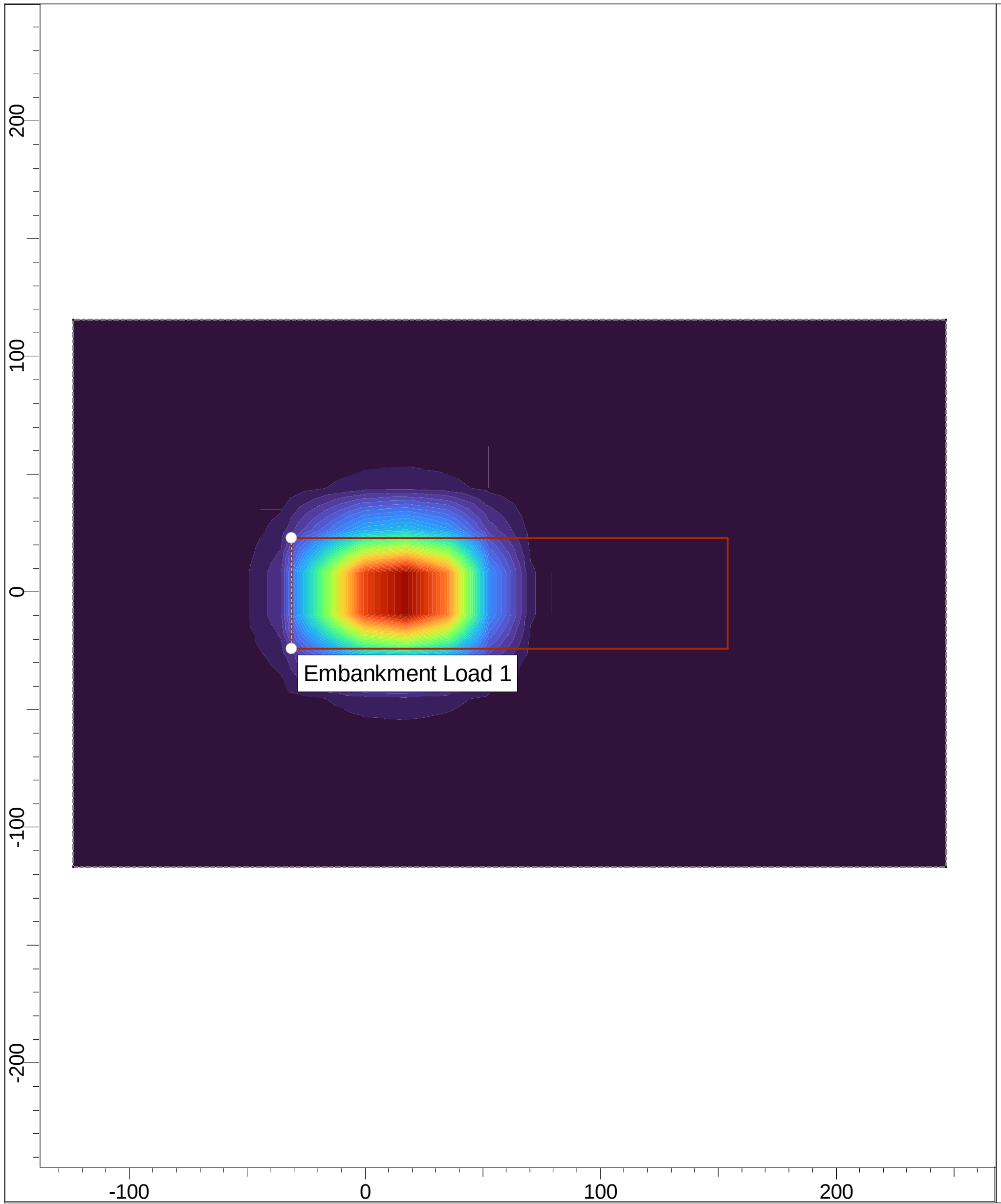
WILLMER ENGINEERING INC. WE

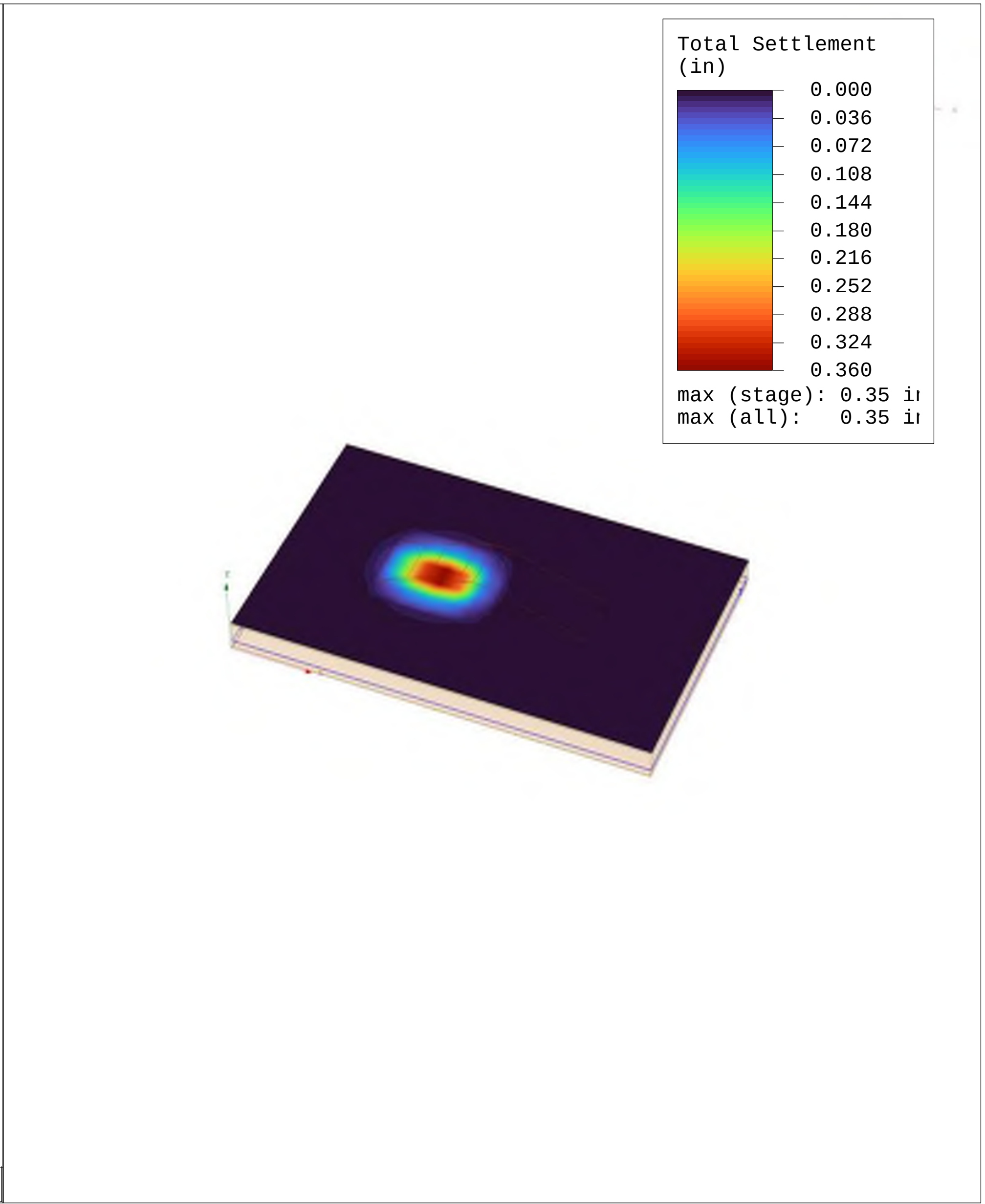
GEOTECHNICAL ENGINEERING ¤ CONSTRUCTION SERVICES
ENVIRONMENTAL SERVICES AND ENGINEERING
3772 PLEASANTDALE ROAD - SUITE 165
ATLANTA, GA 30340-4270

FIGURE 3A

GENERALIZED SUBSURFACE PROFILE A-A'
ERIN LAKE DAM REHABILITATION
DEKALB COUNTY, GEORGIA
WILLMER PROJECT No. 71.3638

P:\3638-Lake Erin-Henerdon Park Geotech Consult\CADD\Subsurface Profiles.dwg

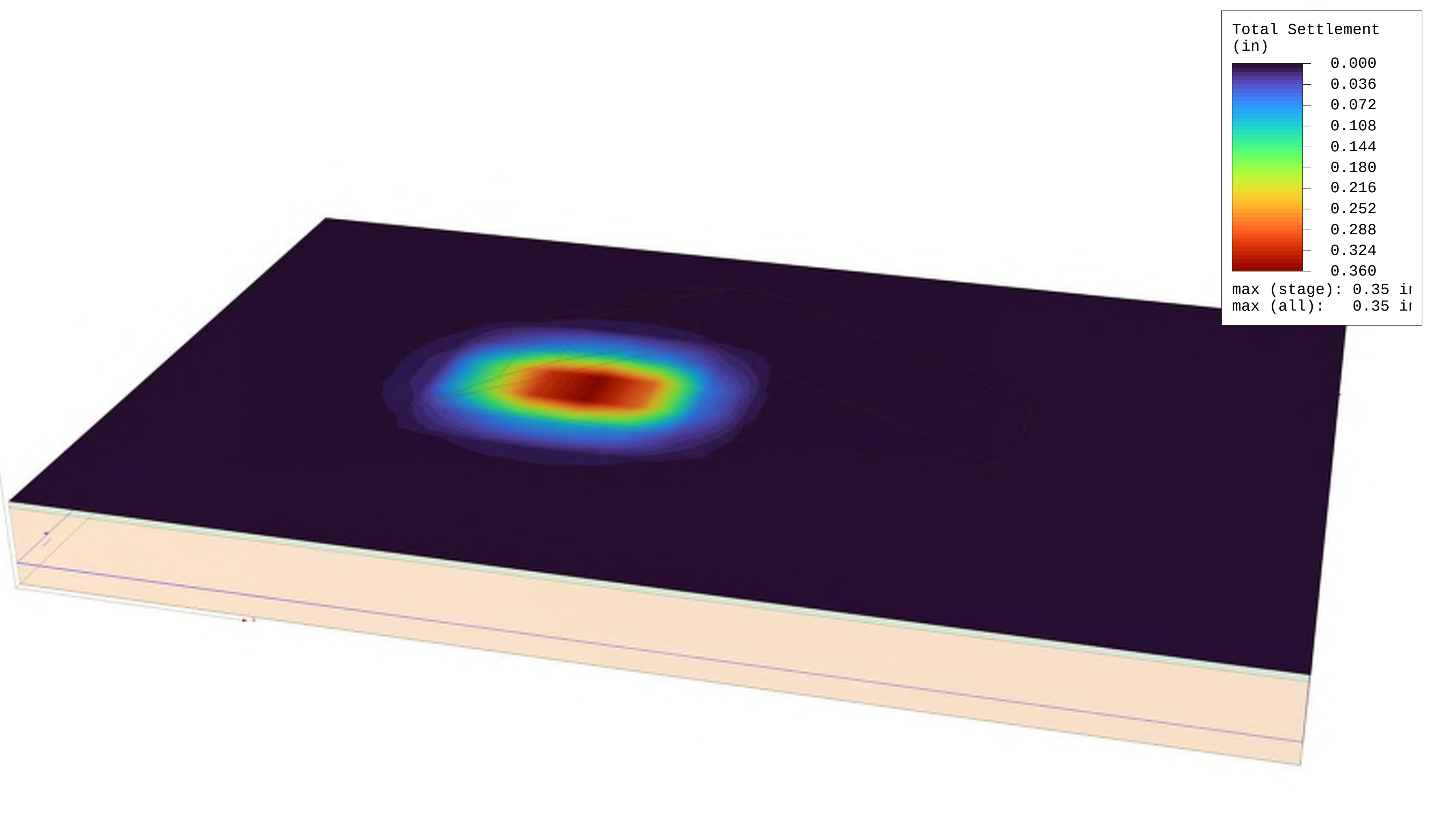




rocscience

SETTLE3 5.022

Project	Erin Lake Dam		
Analysis Description	Settlement		
Drawn By	Atul Singh	Company	AECOM
Date	09/12/2024, 9:03:44 AM	File Name	Erid Dam settlment.s3z



rocscience	Project	Erin Lake Dam	
	Analysis Description	Settlement	
	Drawn By	Atul Singh	Company AECOM
SETTLE3 5.022	Date	09/12/2024, 9:03:44 AM	File Name Erid Dam settlment.s3z

Erin Lake Dam
AECOM
Report Creation Date: 09/12/2024

Table of Contents

Project Settings

Document Name	Erid Dam settlment.s3z
Project Title	Erin Lake Dam
Analysis	Settlement
Author	Atul Singh
Company	AECOM
Date Created	7/1/2024, 9:03:44 AM
Last saved with Settle3 version	5.022
Stress Computation Method	Boussinesq
Stress Units	Imperial, stress as ksf
Settlement Units	inches
Time-dependent Consolidation Analysis	
Time Units	months
Permeability Units	feet/year

Advanced Settings

Start of secondary consolidation (% of primary)	95
Min. stress for secondary consolidation (% of initial)	1
Reset time when load changes for secondary consolidation	No
Minimum settlement ratio for subgrade modulus	0.9
Use average poisson's ratio to calculate layered stresses	
Update Cv in each time step (improves consolidation accuracy)	
Ignore negative effective stresses in settlement calculations	
Add field points to load edges	

Soil Profile

Layer Option	Horizontal Soil Layers
Vertical Axis	Elevation
Ground Elevation (ft)	0

Results (relative to Stage: Stage 1 = 0 mon)

Time taken to compute: 0.71073 seconds

Stage: Stage 1 = 0 mon

Data Type	Minimum	Maximum
Total Settlement [in]	0	0
Total Consolidation Settlement [in]	0	0
Virgin Consolidation Settlement [in]	0	0
Recompression Consolidation Settlement [in]	0	0
Immediate Settlement [in]	0	0
Secondary Settlement [in]	0	0
Loading Stress ZZ [ksf]	0	0
Loading Stress XX [ksf]	0	0
Loading Stress YY [ksf]	0	0
Effective Stress ZZ [ksf]	0	0
Effective Stress XX [ksf]	0	0
Effective Stress YY [ksf]	0	0
Total Stress ZZ [ksf]	0	0
Total Stress XX [ksf]	0	0
Total Stress YY [ksf]	0	0
Modulus of Subgrade Reaction (Total) [ksf/ft]	0	0
Modulus of Subgrade Reaction (Immediate) [ksf/ft]	0	0
Modulus of Subgrade Reaction (Consolidation) [ksf/ft]	0	0
Total Strain	0	0
Pore Water Pressure [ksf]	0	0
Excess Pore Water Pressure [ksf]	0	0
Degree of Consolidation [%]	0	0
Pre-consolidation Stress [ksf]	0	0
Over-consolidation Ratio	0	0
Void Ratio	0	0
Permeability [ft/y]	0	0
Coefficient of Consolidation [ft^2/y]	0	0
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	0
Undrained Shear Strength	0	0

Stage: Stage 2 = 2 mon

Data Type	Minimum	Maximum
Total Settlement [in]	0	0.350498
Total Consolidation Settlement [in]	0	0
Virgin Consolidation Settlement [in]	0	0
Recompression Consolidation Settlement [in]	0	0
Immediate Settlement [in]	0	0.350498
Secondary Settlement [in]	0	0
Loading Stress ZZ [ksf]	-1.33985e-10	0.512478
Loading Stress XX [ksf]	-0.206175	0.57035
Loading Stress YY [ksf]	-0.0944636	0.374141
Effective Stress ZZ [ksf]	-1.33985e-10	0.512478
Effective Stress XX [ksf]	-0.206175	0.57035
Effective Stress YY [ksf]	-0.0944636	0.374141
Total Stress ZZ [ksf]	-1.33985e-10	0.512478
Total Stress XX [ksf]	-0.206175	0.57035
Total Stress YY [ksf]	-0.0944636	0.374141
Modulus of Subgrade Reaction (Total) [ksf/ft]	0	0
Modulus of Subgrade Reaction (Immediate) [ksf/ft]	0	0
Modulus of Subgrade Reaction (Consolidation) [ksf/ft]	0	0
Total Strain	0	0.00292844
Pore Water Pressure [ksf]	0	0
Excess Pore Water Pressure [ksf]	0	0
Degree of Consolidation [%]	0	0
Pre-consolidation Stress [ksf]	0	0.512477
Over-consolidation Ratio	-0.000240669	0
Void Ratio	0	0
Permeability [ft/y]	0	0
Coefficient of Consolidation [ft^2/y]	0	0
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	0
Undrained Shear Strength	0	0.00934444

Stage: Stage 3 = 12 mon

Data Type	Minimum	Maximum
Total Settlement [in]	0	0.350498
Total Consolidation Settlement [in]	0	0
Virgin Consolidation Settlement [in]	0	0
Recompression Consolidation Settlement [in]	0	0
Immediate Settlement [in]	0	0.350498
Secondary Settlement [in]	0	0
Loading Stress ZZ [ksf]	-1.33985e-10	0.512478
Loading Stress XX [ksf]	-0.206175	0.57035
Loading Stress YY [ksf]	-0.0944636	0.374141
Effective Stress ZZ [ksf]	-1.33985e-10	0.512478
Effective Stress XX [ksf]	-0.206175	0.57035
Effective Stress YY [ksf]	-0.0944636	0.374141
Total Stress ZZ [ksf]	-1.33985e-10	0.512478
Total Stress XX [ksf]	-0.206175	0.57035
Total Stress YY [ksf]	-0.0944636	0.374141
Modulus of Subgrade Reaction (Total) [ksf/ft]	0	0
Modulus of Subgrade Reaction (Immediate) [ksf/ft]	0	0
Modulus of Subgrade Reaction (Consolidation) [ksf/ft]	0	0
Total Strain	0	0.00292844
Pore Water Pressure [ksf]	0	0
Excess Pore Water Pressure [ksf]	0	0
Degree of Consolidation [%]	0	0
Pre-consolidation Stress [ksf]	0	0.512477
Over-consolidation Ratio	-0.000240669	0
Void Ratio	0	0
Permeability [ft/y]	0	0
Coefficient of Consolidation [ft^2/y]	0	0
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	0
Undrained Shear Strength	0	0.00934444

Embankments

1. Embankment: "Embankment Load 1"

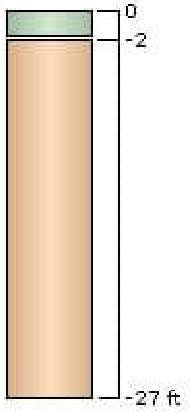
Label	Embankment Load 1
Center Line	(-31.377, -24.236) to (-31.377, 22.849)
Near End Angle	90 degrees
Far End Angle	90 degrees
Number of Layers	2

Layer	Stage	Unit Weight (kips/ft3)
1	Stage 2	0.125
2	Stage 1	0.125

Soil Layers

Ground Surface Drained: Yes

Layer #	Type	Thickness [ft]	Elevation [ft]	Drained at Bottom
1	Fill	0	0	No
2	Alluvium	2	0	No
3	Residual	25	-2	No



Soil Properties

Property	Fill	Alluvium	Residual
Color	🟨	🟩	🟧
Unit Weight [kips/ft3]	0.12	0.12	0.115
Saturated Unit Weight [kips/ft3]	0.128	0.128	0.115
K0	1	1	1
Immediate Settlement	Enabled	Enabled	Enabled
Es [ksf]	150	175	450
Esur [ksf]	208.9	208.9	208.9
B-bar	-	-	-
Undrained Su A [kips/ft2]	0	0	0
Undrained Su S	0.2	0.2	0.2
Undrained Su m	0.8	0.8	0.8
Piezo Line ID	0	1	1

Groundwater

Groundwater method	Piezometric Lines
Water Unit Weight	0.0624 kips/ft3

Piezometric Line Entities

ID	Elevation (ft)
1	-2 ft

Appendix C – Hydrologic and Hydraulic Analysis Calculations

C.1 Proposed Elevation-Storage and Elevation-Discharge Rating Curve

AECOM

12420 Milestone Center Drive, Suite 150
Germantown, Maryland 20876

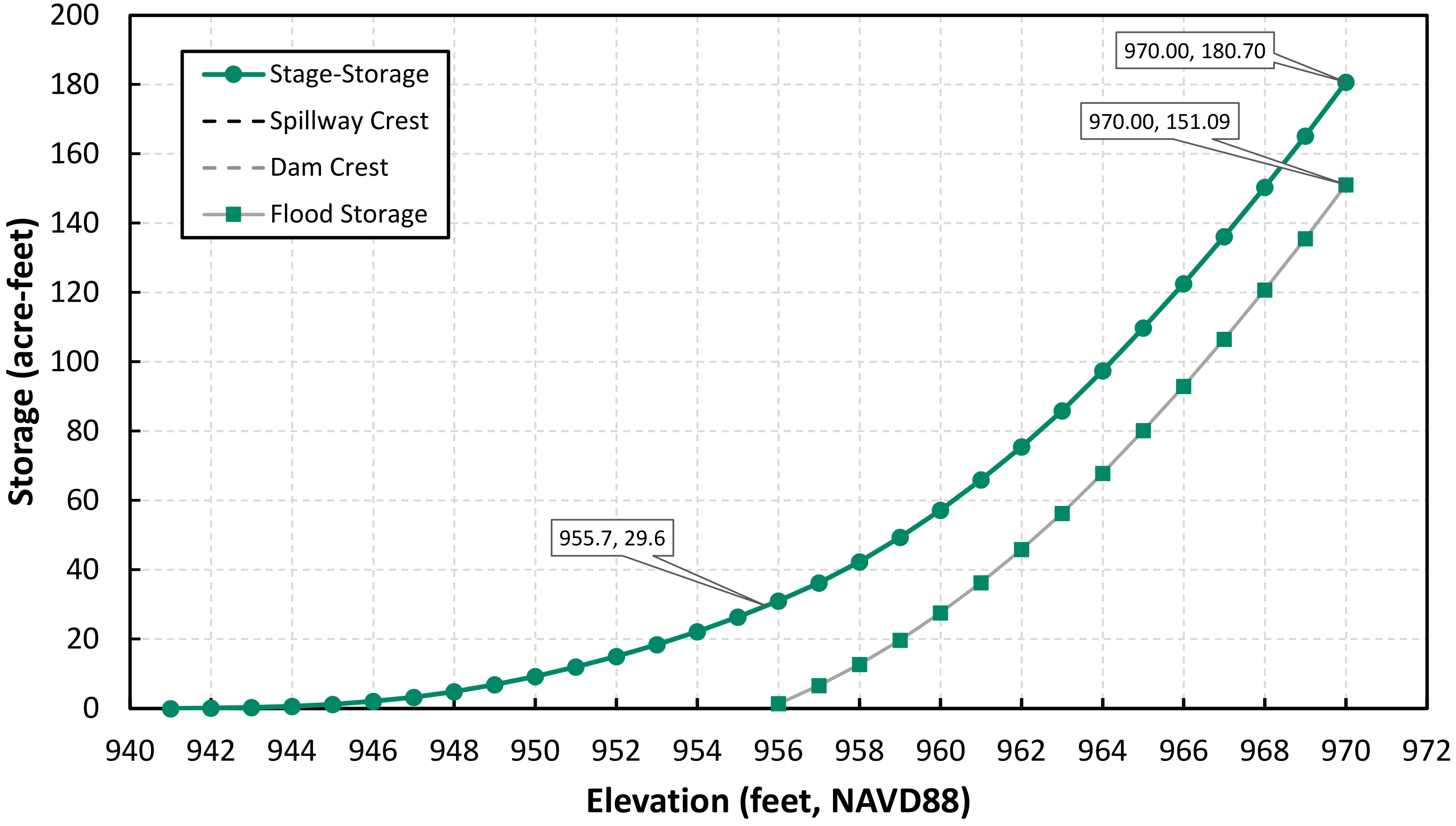
DESIGN	AJW	DATE	12-Sep-2024
CHECK	JCG	DATE	13-Sep-2024
PROJECT:	Erin Lake Dam Proposed Conditions		

ELEVATION-STORAGE RATING TABLE CALCULATIONS - PROPOSED CONDITIONS

RATING ELEVATIONS		
	START	941.00'
	END	970.00'
	POOL	955.70'

DATA SOURCE	ELEVATION (FEET)	AREA (AC)	AVERAGE AREA (AC-FT)	INCREMENTAL DEPTH (FT)	INCREMENTAL VOLUME (AC-FT)	TOTAL VOLUME (AC-FT)	FLOOD STORAGE VOLUME (AC-FT)
Proposed Surface (AECOM, 2024)	941.00	0.02	0.02	0.00	0.00	0.00	
	942.00	0.16	0.09	1.00	0.09	0.09	
	943.00	0.22	0.19	1.00	0.19	0.28	
	944.00	0.38	0.30	1.00	0.30	0.58	
	945.00	0.90	0.64	1.00	0.64	1.22	
	946.00	0.90	0.90	1.00	0.90	2.11	
	947.00	1.37	1.13	1.00	1.13	3.25	
	948.00	1.80	1.59	1.00	1.59	4.84	
	949.00	2.19	2.00	1.00	2.00	6.83	
	950.00	2.57	2.38	1.00	2.38	9.21	
	951.00	2.91	2.74	1.00	2.74	11.95	
	952.00	3.22	3.07	1.00	3.07	15.02	
	953.00	3.56	3.39	1.00	3.39	18.41	
	954.00	3.93	3.75	1.00	3.75	22.16	
	955.00	4.45	4.19	1.00	4.19	26.35	
	956.00	4.85	4.65	1.00	4.65	31.00	1.39
	957.00	5.57	5.21	1.00	5.21	36.21	6.61
	958.00	6.66	6.12	1.00	6.12	42.33	12.72
	959.00	7.38	7.02	1.00	7.02	49.35	19.74
	960.00	8.33	7.86	1.00	7.86	57.20	27.60
	961.00	9.13	8.73	1.00	8.73	65.94	36.33
	962.00	10.01	9.57	1.00	9.57	75.51	45.91
	963.00	10.78	10.40	1.00	10.40	85.91	56.30
	964.00	12.31	11.54	1.00	11.54	97.45	67.85
	965.00	12.40	12.35	1.00	12.35	109.80	80.20
	966.00	13.16	12.78	1.00	12.78	122.58	92.98
	967.00	13.89	13.53	1.00	13.53	136.11	106.51
	968.00	14.54	14.21	1.00	14.21	150.32	120.72
	969.00	15.14	14.84	1.00	14.84	165.16	135.55
	970.00	15.94	15.54	1.00	15.54	180.70	151.09

Elevation-Storage Rating for Proposed Conditions



AECOM

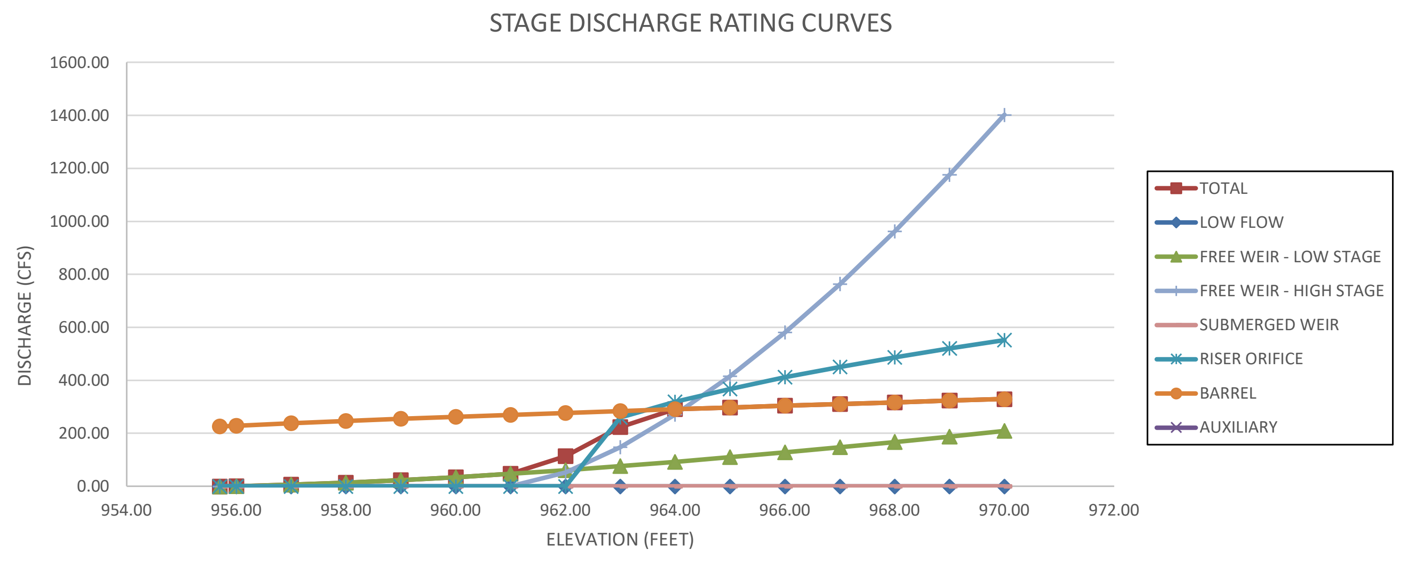
12420 Milestone Center Drive, Suite 150
Germantown, Maryland 20876

DESIGN AJW DATE 12-Sep-2024
CHECK JCG DATE 13-Sep-2024
PROJECT: Erin Lake Dam Proposed Conditions

ELEVATION-DISCHARGE RATING TABLE CALCULATIONS

RATING ELEVATIONS		LOW FLOW ORIFICE		RISER						BARREL				AUXILIARY SPILLWAY		TOTAL
START	955.70'	HEIGHT		LENGTH	9.00'	WIDTH	6.00'	AREA	54.00 SF	RISE	48 INCHES	INV. (IN)	940.00'	CREST ELEV		
END	970.00'	WIDTH		CREST LENGTH	1.25'	CREST ELEV	955.70'	LOW STAGE		SPAN	48 INCHES	INV. (OUT)	939.00'	WIDTH		
		INVERT		CREST LENGTH	16.75'	CREST ELEV	961.00'	HIGH STAGE		QUANTITY	1	LENGTH	129.53'	LENGTH		
										TW ELEV.	945.00'	MANNINGS	0.013	MANNINGS		

			LOW STAGE		HIGH STAGE		SUBMERGED		ORIFICE							
ELEVATION (FEET)	H_O	(1) Q_O	H_{WLS} (H_1)	(2) Q_{WLS}	H_{WHS}	(2A) Q_{WHS}	H_{SB} (H_2)	(3) Q_{SB}	(4) Q_{WO}	H_{PS}	(5) Q_{BIC}	(6) Q_{BOC}	Q_B	H_{AS}	(7) Q_{AS}	(8) Q_{TOTAL}
955.70			0.00	0.00						15.70	227.29	225.77	225.77			0.00
956.00			0.30	0.64						16.00	229.89	228.91	228.91			0.64
957.00			1.30	5.74						17.00	238.36	239.09	238.36			5.74
958.00			2.30	13.52						18.00	246.55	248.85	246.55			13.52
959.00			3.30	23.23						19.00	254.47	258.25	254.47			23.23
960.00			4.30	34.55						20.00	262.15	267.31	262.15			34.55
961.00			5.30	47.28	0.00	0.00				21.00	269.61	276.08	269.61			47.28
962.00			6.30	61.27	1.00	51.93				22.00	276.87	284.57	276.87			113.20
963.00			7.30	76.43	2.00	146.87			260.01	23.00	283.94	292.82	283.94			223.29
964.00			8.30	92.66	3.00	269.81	8.30	B/C	318.44	24.00	290.85	300.85	290.85			290.85
965.00			9.30	109.90	4.00	415.40	9.30	B/C	367.71	25.00	297.59	308.66	297.59			297.59
966.00			10.30	128.09	5.00	580.54	10.30	B/C	411.11	26.00	304.18	316.29	304.18			304.18
967.00			11.30	147.19	6.00	763.14	11.30	B/C	450.35	27.00	310.63	323.73	310.63			310.63
968.00			12.30	167.16	7.00	961.66	12.30	B/C	486.43	28.00	316.96	331.01	316.96			316.96
969.00			13.30	187.95	8.00	1174.93	13.30	B/C	520.02	29.00	323.16	338.12	323.16			323.16
970.00			14.30	209.54	9.00	1401.98	14.30	B/C	551.56	30.00	329.24	345.10	329.24			329.24



Equations Used:

(1) Low Flow Orifice: $Q_O = C_O A \sqrt{2gH_O}$ where: $C_O = 0.6$, $g = 32.2\ FT/S^2$

(2) Weir (Free Flow): $Q_W = C_W L_W H_W^{3/2}$ where: $C_W = 3.1$

**(3) Weir (Submerged Flow): $Q_{SB} = Q_W * [1-(H_2/H_1)^{3/2}]^{0.385}$* where: *$H_2$ = TW over weir (inside of riser), H_1 = HW over weir (U/S of riser) (Brater, "Handbook of Hydraulics, 7th Ed")*

(4) Riser (acting as a horizontal orifice): $Q_{WO} = C_O A \sqrt{2gH_O}$ where: *$C_O = 0.6$, $g = 32.2\ FT/S^2$, for $H_W > 0.08$ Max (Riser Length/Width) + 0.35'*

(5) Barrel (Submerged Inlet Control): $Q_{BIC} = [A\sqrt{D}] * \sqrt{[(H_W/DC) - (Y/C)]}$ where: *Y = 0.67, C = 0.0398 (FHWA, "Hydraulic Design of Highway Culverts, 3rd Ed")*

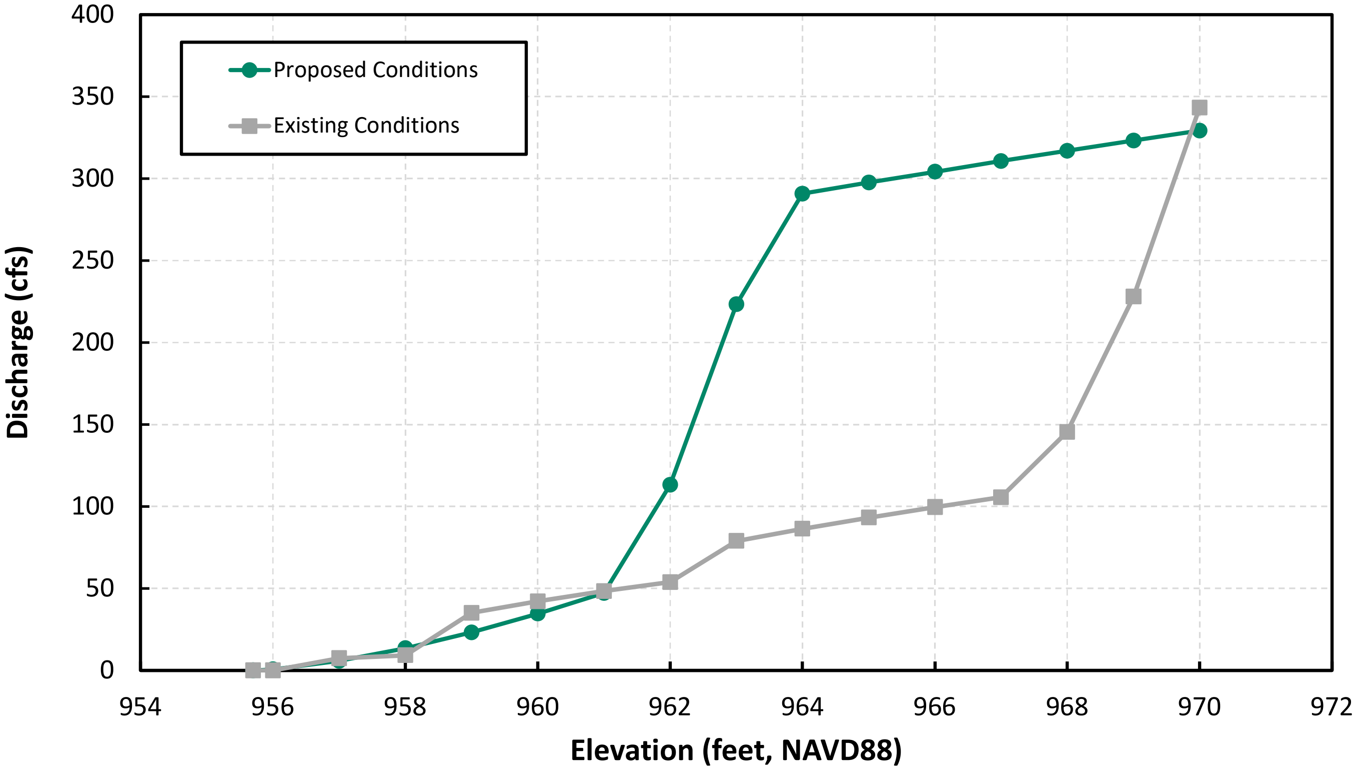
(6) Barrel (Outlet Control): $Q_{BOC} = A\sqrt{[\{2g(H_W - T_W)\}/[K_{EXT} + K_{ENT} + (K_U n^2 L/R^{4/3})]\}}$ where: *$g = 32.2\ FT/S^2$, KEXT = 1.0, $K_{ENT} = 0.5$, $K_U = 29$, R = Hyd. Radius (FHWA, "Hydraulic Deisgn of Highway Culverts, 3rd Ed")*

(7) Auxiliary Spillway: Refer to attached TR-2 spreadsheet if required for model.

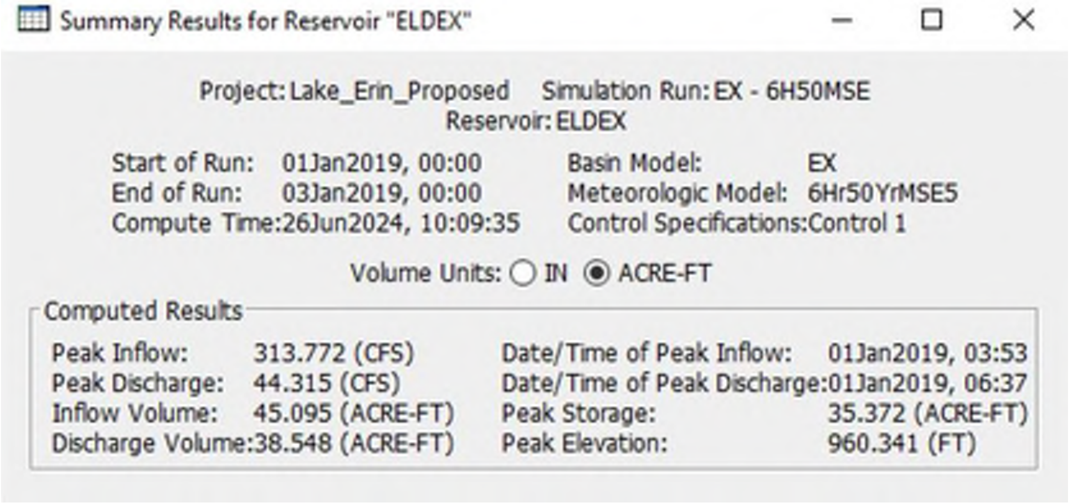
(8) Total Flow: Q = MIN (1 + MIN (2 + 2A, 3, 4), MIN (5, 6)) + 7

**Note: Submerged Weir flow is only calculated and reported when there is a tailwater over the riser weir (H_2>0) and when the system is in weir control (B/C denotes barrel control).*

Elevation-Discharge Rating for Proposed and Existing Conditions



C.2 Proposed Conditions Hydrologic Model Results



Project: Lake_Erin_Proposed Simulation Run: EX - 6H50MSE
Reservoir: ELDEX

Start of Run: 01Jan2019, 00:00
End of Run: 03Jan2019, 00:00
Compute Time: 26Jun2024, 10:09:35

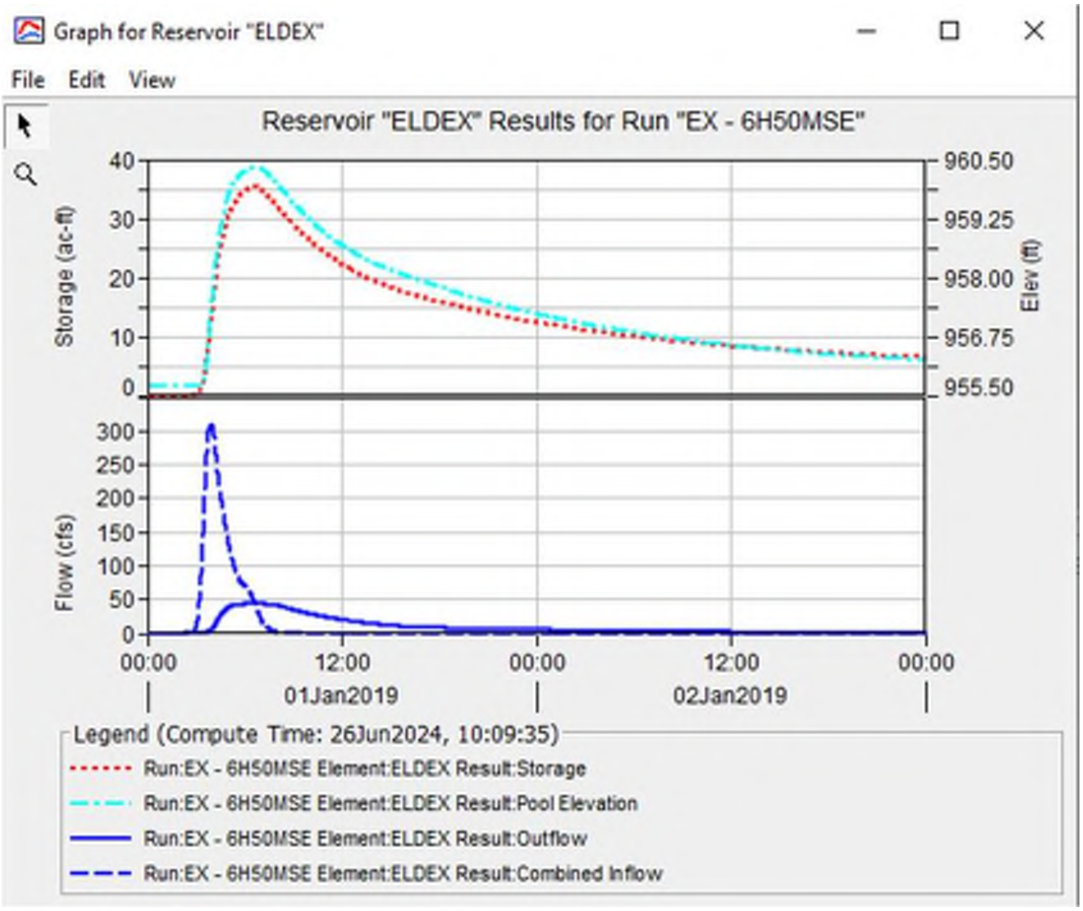
Basin Model: EX
Meteorologic Model: 6Hr50YrMSE5
Control Specifications: Control 1

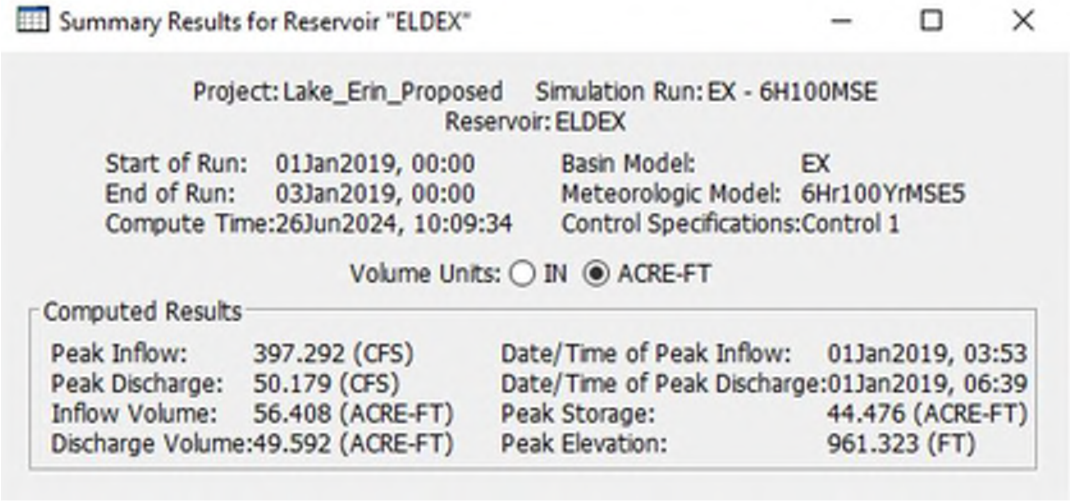
Volume Units: ○ IN ● ACRE-FT

Computed Results

Peak Inflow: 313.772 (CFS)
Peak Discharge: 44.315 (CFS)
Inflow Volume: 45.095 (ACRE-FT)
Discharge Volume: 38.548 (ACRE-FT)

Date/Time of Peak Inflow: 01Jan2019, 03:53
Date/Time of Peak Discharge: 01Jan2019, 06:37
Peak Storage: 35.372 (ACRE-FT)
Peak Elevation: 960.341 (FT)




Summary Results for Reservoir "ELDEX"

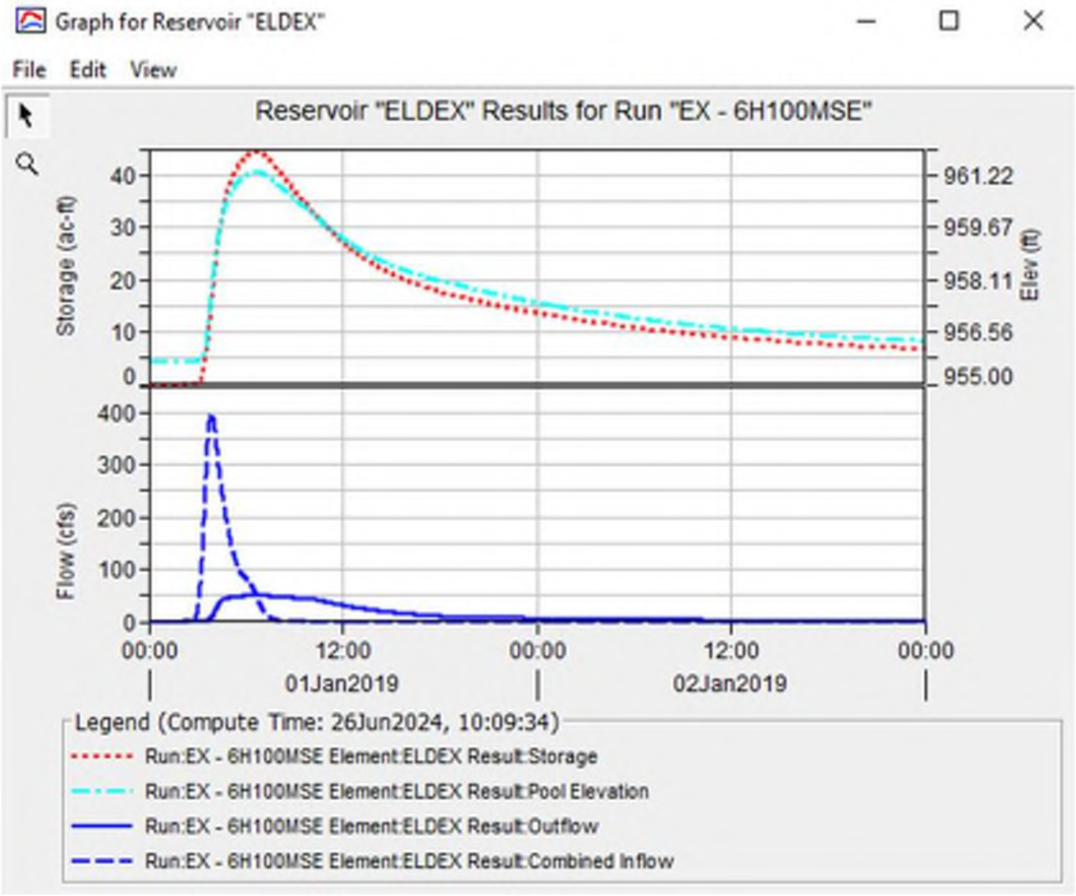
Project: Lake_Erin_Proposed   Simulation Run: EX - 6H100MSE
Reservoir: ELDEX

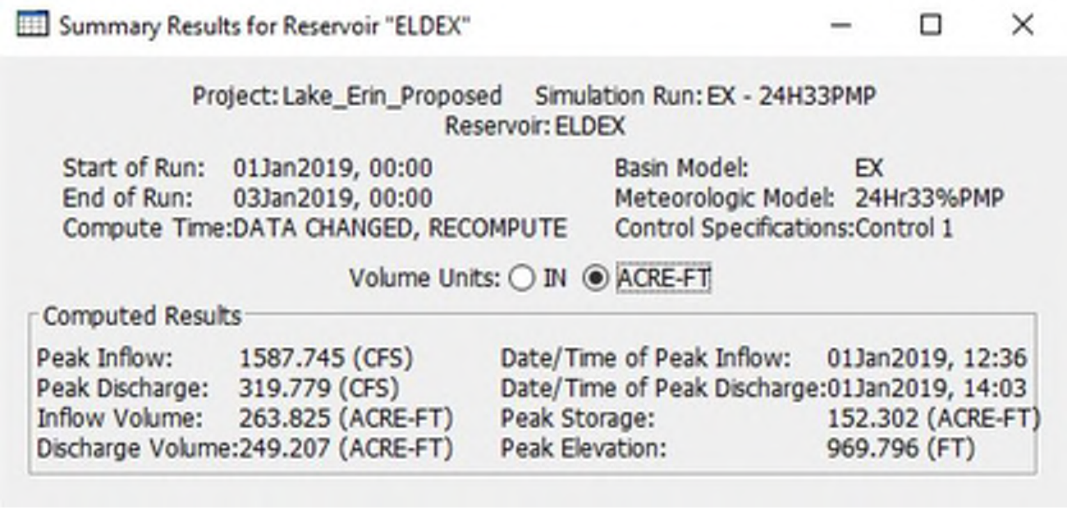
Start of Run: 01Jan2019, 00:00   Basin Model: EX
End of Run: 03Jan2019, 00:00   Meteorologic Model: 6Hr100YrMSE5
Compute Time: 26Jun2024, 10:09:34   Control Specifications: Control 1

Volume Units: ○ IN ● ACRE-FT

Computed Results

Peak Inflow:	397.292 (CFS)	Date/Time of Peak Inflow:	01Jan2019, 03:53
Peak Discharge:	50.179 (CFS)	Date/Time of Peak Discharge:	01Jan2019, 06:39
Inflow Volume:	56.408 (ACRE-FT)	Peak Storage:	44.476 (ACRE-FT)
Discharge Volume:	49.592 (ACRE-FT)	Peak Elevation:	961.323 (FT)




Summary Results for Reservoir "ELDEX"

Project: Lake_Erin_Proposed   Simulation Run: EX - 24H33PMP
Reservoir: ELDEX

Start of Run: 01Jan2019, 00:00
End of Run: 03Jan2019, 00:00
Compute Time: DATA CHANGED, RECOMPUTE

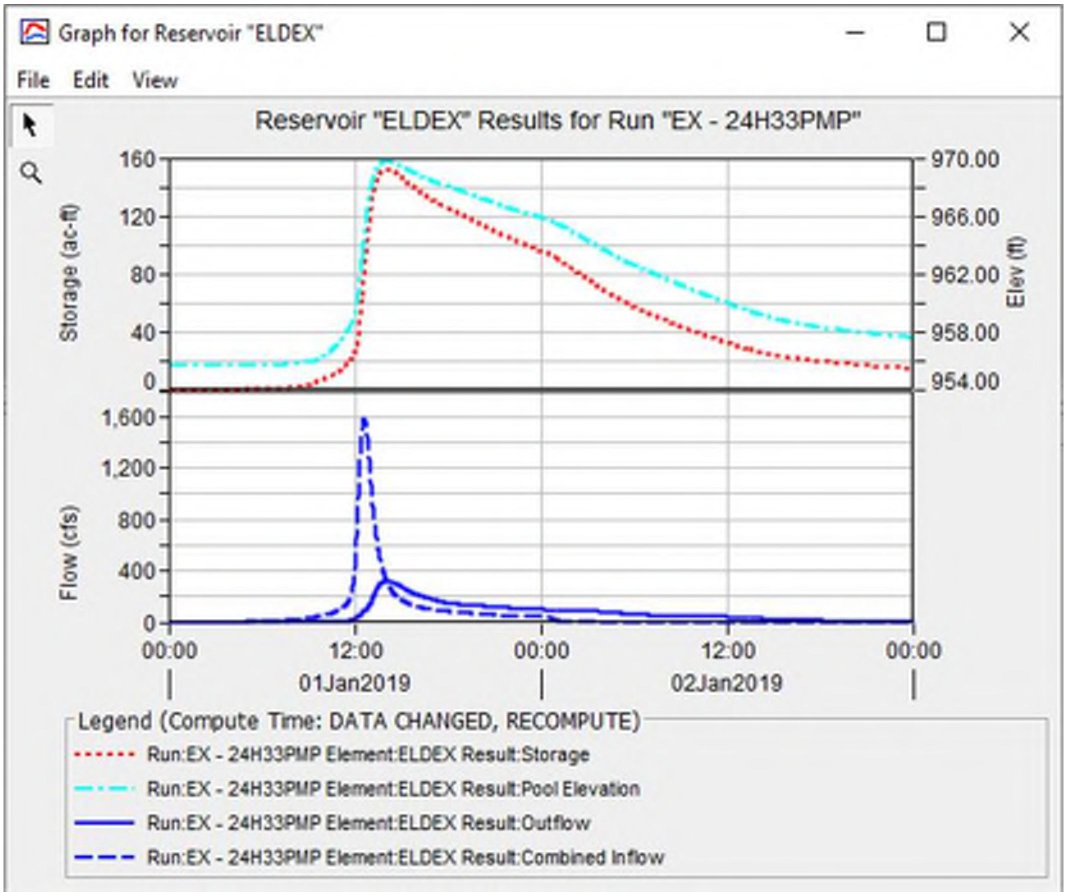
Basin Model: EX
Meteorologic Model: 24Hr33%PMP
Control Specifications: Control 1

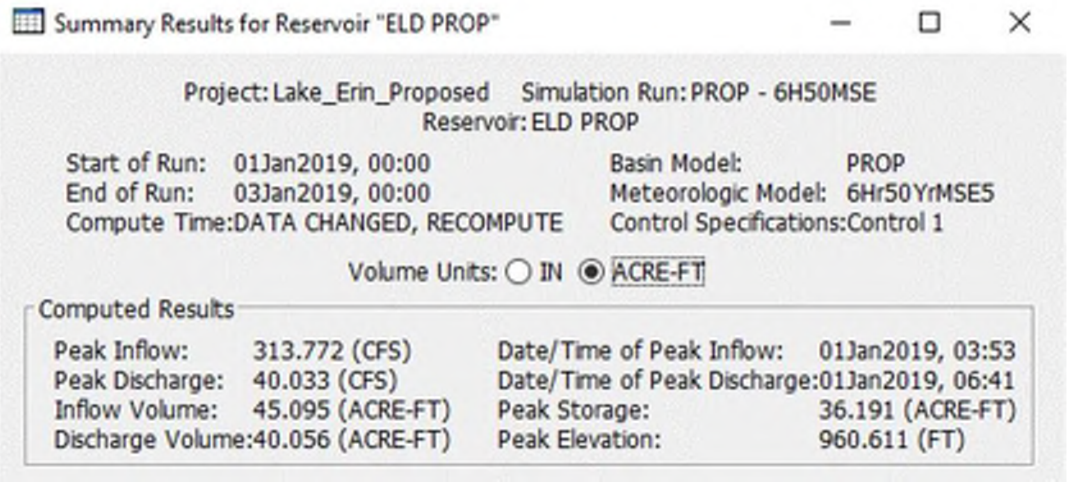
Volume Units: ○ IN ● ACRE-FT

Computed Results

Peak Inflow: 1587.745 (CFS)
Peak Discharge: 319.779 (CFS)
Inflow Volume: 263.825 (ACRE-FT)
Discharge Volume: 249.207 (ACRE-FT)

Date/Time of Peak Inflow: 01Jan2019, 12:36
Date/Time of Peak Discharge: 01Jan2019, 14:03
Peak Storage: 152.302 (ACRE-FT)
Peak Elevation: 969.796 (FT)





Summary Results for Reservoir "ELD PROP"

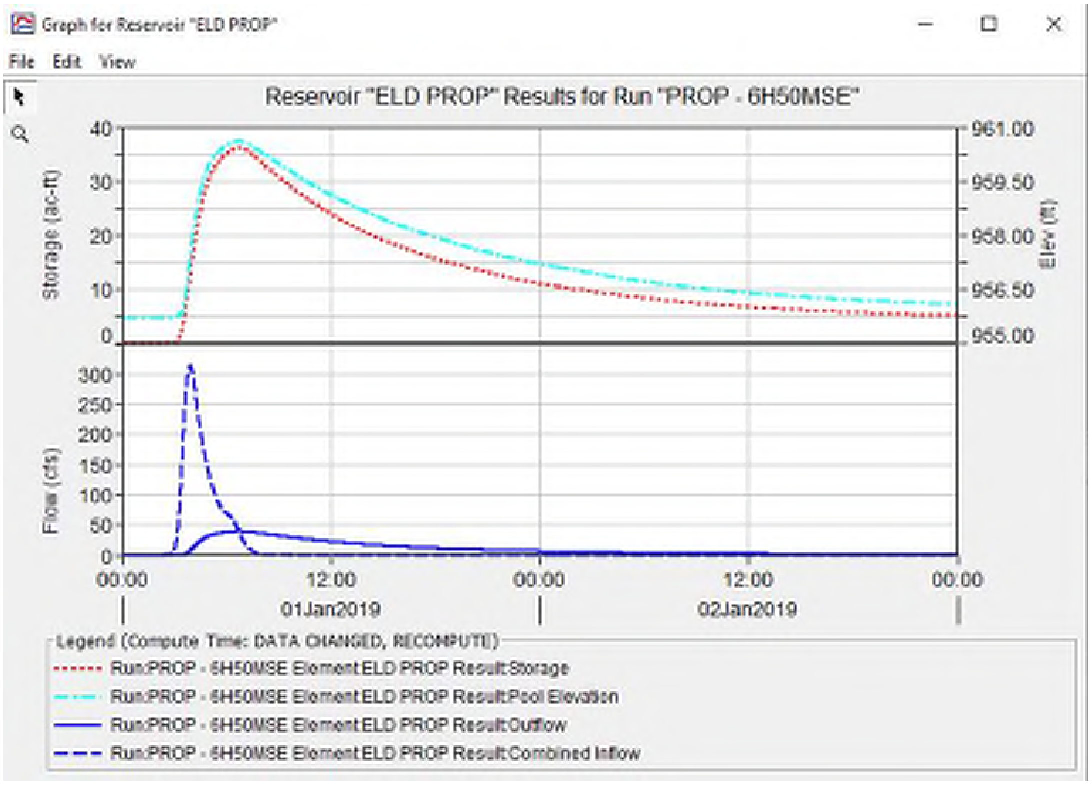
Project: Lake_Erin_Proposed   Simulation Run: PROP - 6H50MSE
Reservoir: ELD PROP

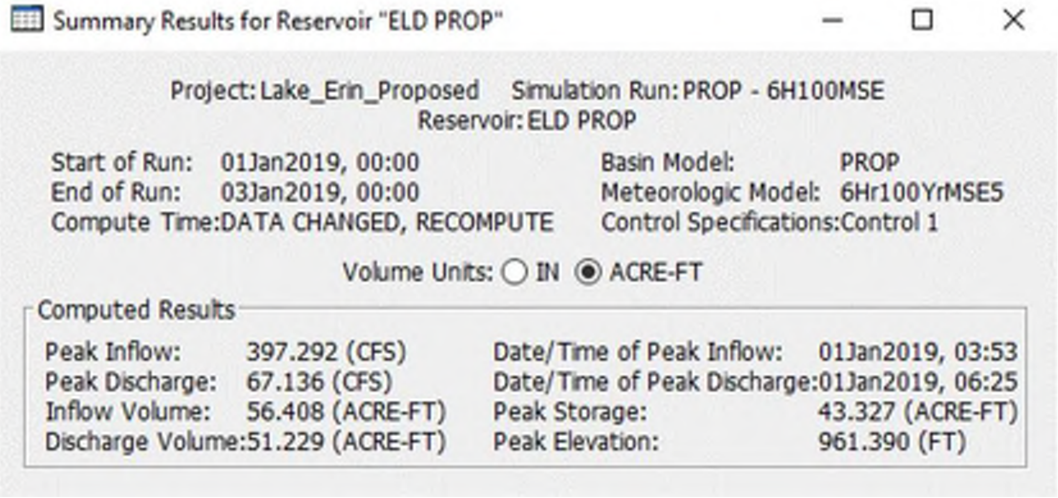
Start of Run:	01Jan2019, 00:00	Basin Model:	PROP
End of Run:	03Jan2019, 00:00	Meteorologic Model:	6Hr50YrMSE5
Compute Time:	DATA CHANGED, RECOMPUTE	Control Specifications:	Control 1

Volume Units: ○ IN ● ACRE-FT

Computed Results

Peak Inflow:	313.772 (CFS)	Date/Time of Peak Inflow:	01Jan2019, 03:53
Peak Discharge:	40.033 (CFS)	Date/Time of Peak Discharge:	01Jan2019, 06:41
Inflow Volume:	45.095 (ACRE-FT)	Peak Storage:	36.191 (ACRE-FT)
Discharge Volume:	40.056 (ACRE-FT)	Peak Elevation:	960.611 (FT)





Summary Results for Reservoir "ELD PROP"

Project: Lake_Erin_Proposed Simulation Run: PROP - 6H100MSE
Reservoir: ELD PROP

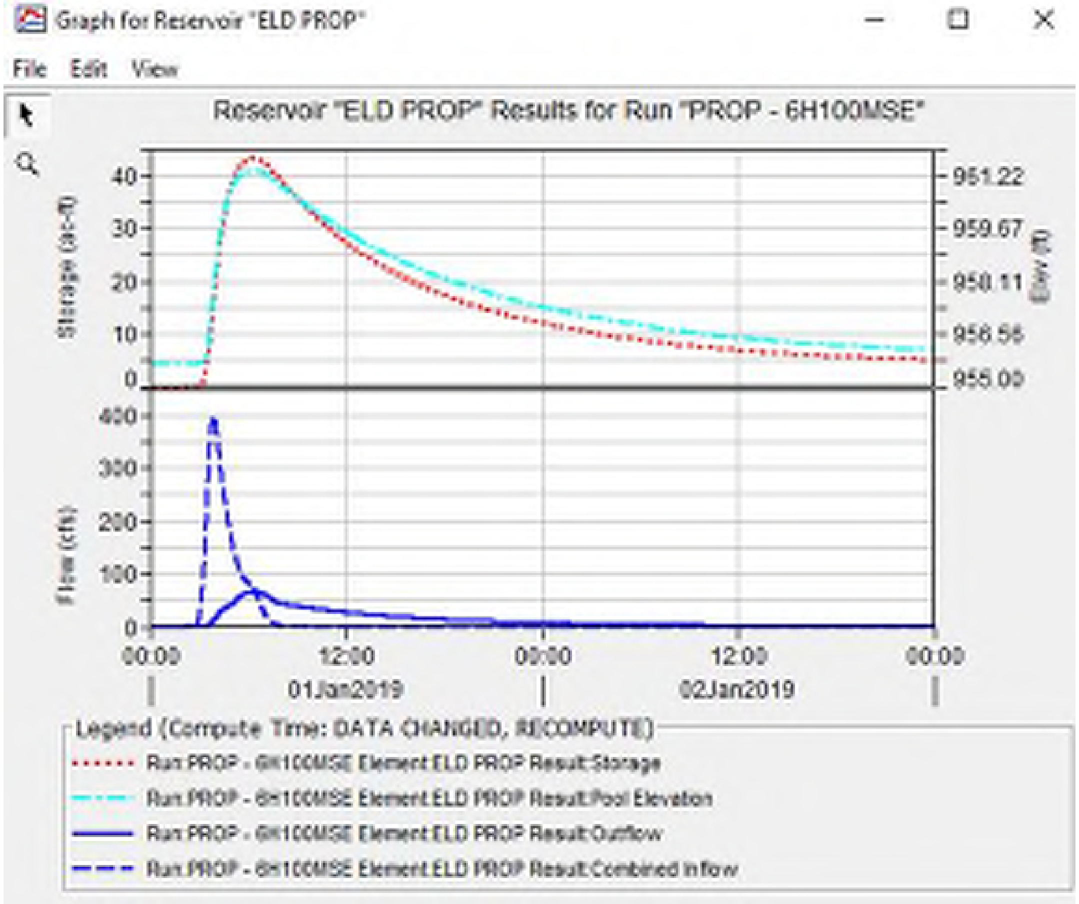
Start of Run: 01Jan2019, 00:00
End of Run: 03Jan2019, 00:00
Compute Time: DATA CHANGED, RECOMPUTE

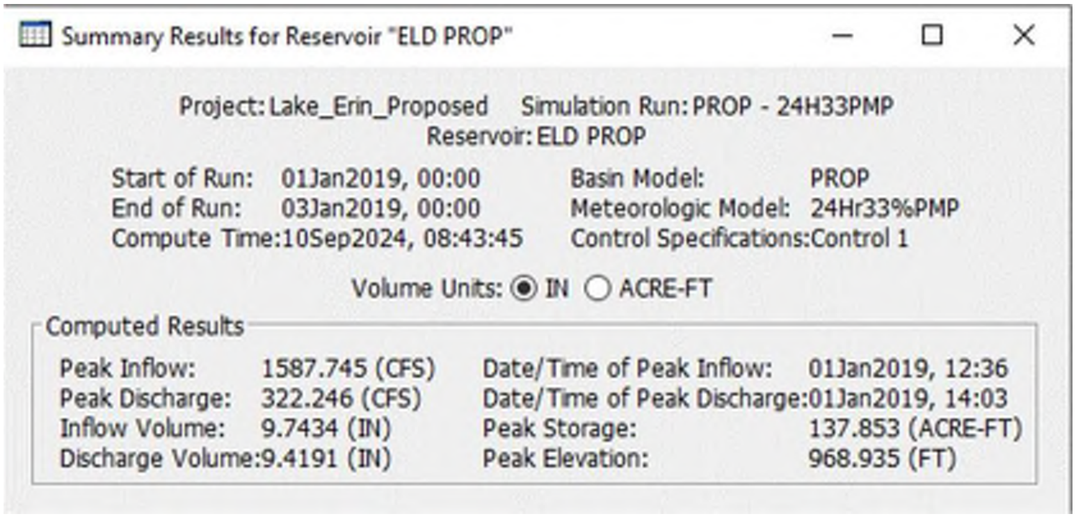
Basin Model: PROP
Meteorologic Model: 6Hr100YrMSE5
Control Specifications: Control 1

Volume Units: ○ IN ● ACRE-FT

Computed Results

Peak Inflow:	397.292 (CFS)	Date/Time of Peak Inflow:	01Jan2019, 03:53
Peak Discharge:	67.136 (CFS)	Date/Time of Peak Discharge:	01Jan2019, 06:25
Inflow Volume:	56.408 (ACRE-FT)	Peak Storage:	43.327 (ACRE-FT)
Discharge Volume:	51.229 (ACRE-FT)	Peak Elevation:	961.390 (FT)





Summary Results for Reservoir "ELD PROP"

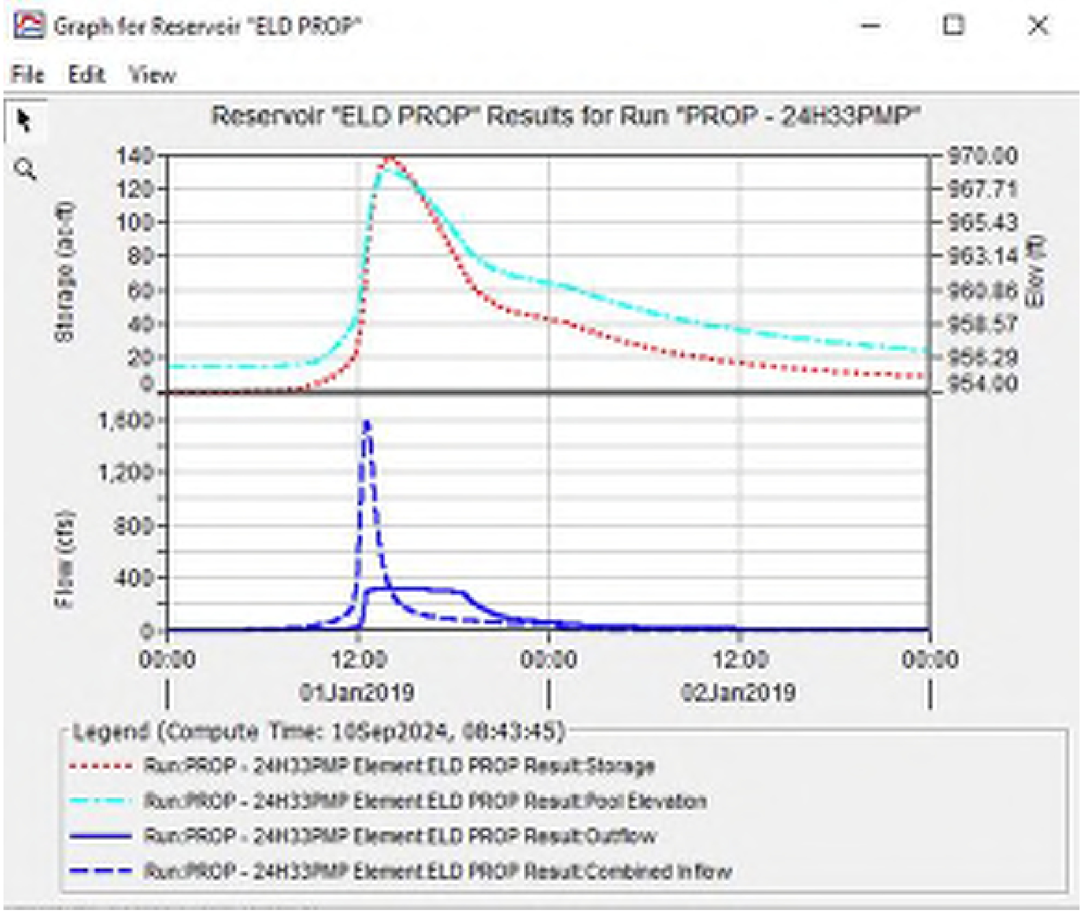
Project: Lake_Erin_Proposed Simulation Run: PROP - 24H33PMP
Reservoir: ELD PROP

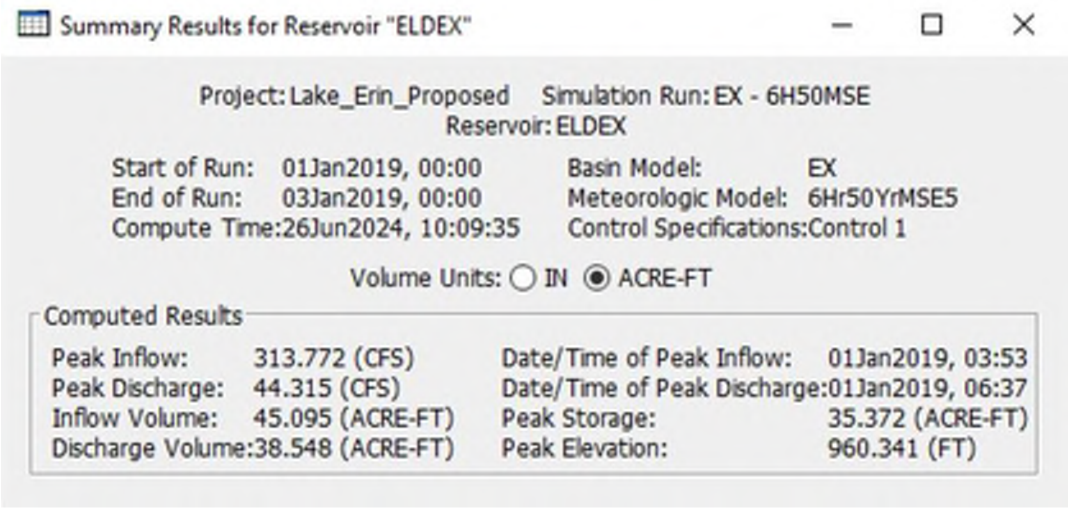
Start of Run:	01Jan2019, 00:00	Basin Model:	PROP
End of Run:	03Jan2019, 00:00	Meteorologic Model:	24Hr33%PMP
Compute Time:	10Sep2024, 08:43:45	Control Specifications:	Control 1

Volume Units: ● IN ○ ACRE-FT

Computed Results

Peak Inflow:	1587.745 (CFS)	Date/Time of Peak Inflow:	01Jan2019, 12:36
Peak Discharge:	322.246 (CFS)	Date/Time of Peak Discharge:	01Jan2019, 14:03
Inflow Volume:	9.7434 (IN)	Peak Storage:	137.853 (ACRE-FT)
Discharge Volume:	9.4191 (IN)	Peak Elevation:	968.935 (FT)





Summary Results for Reservoir "ELDEX"

Project: Lake_Erin_Proposed Simulation Run: EX - 6H50MSE
Reservoir: ELDEX

Start of Run: 01Jan2019, 00:00
End of Run: 03Jan2019, 00:00
Compute Time: 26Jun2024, 10:09:35

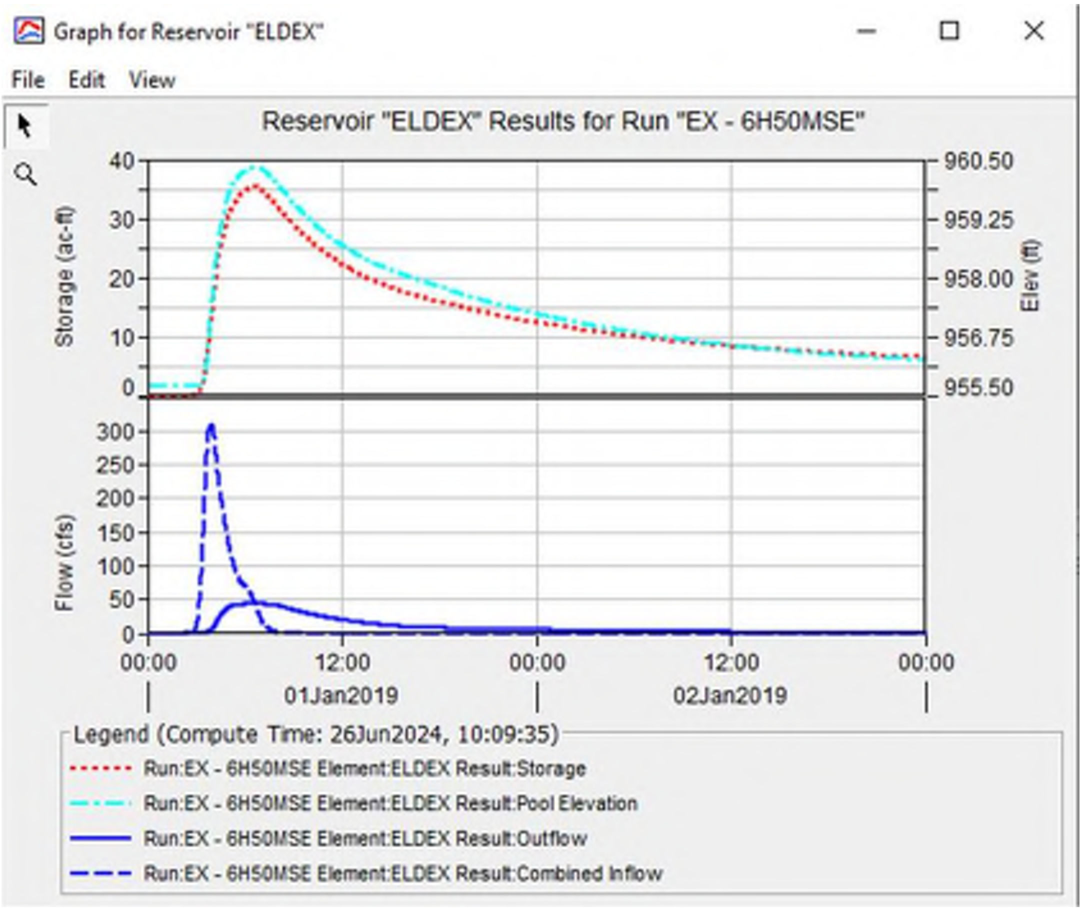
Basin Model: EX
Meteorologic Model: 6Hr50YrMSE5
Control Specifications: Control 1

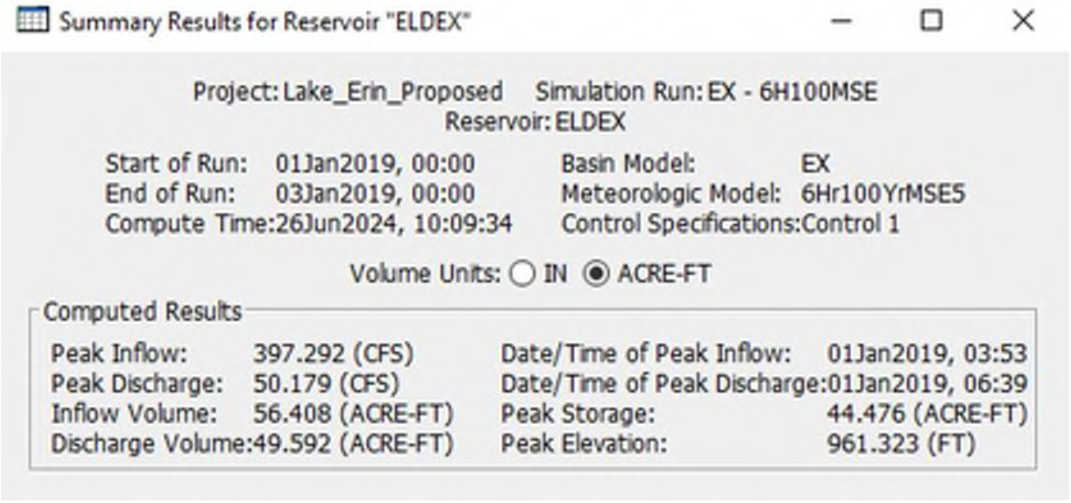
Volume Units: ○ IN ● ACRE-FT

Computed Results

Peak Inflow: 313.772 (CFS)
Peak Discharge: 44.315 (CFS)
Inflow Volume: 45.095 (ACRE-FT)
Discharge Volume: 38.548 (ACRE-FT)

Date/Time of Peak Inflow: 01Jan2019, 03:53
Date/Time of Peak Discharge: 01Jan2019, 06:37
Peak Storage: 35.372 (ACRE-FT)
Peak Elevation: 960.341 (FT)




Summary Results for Reservoir "ELDEX"

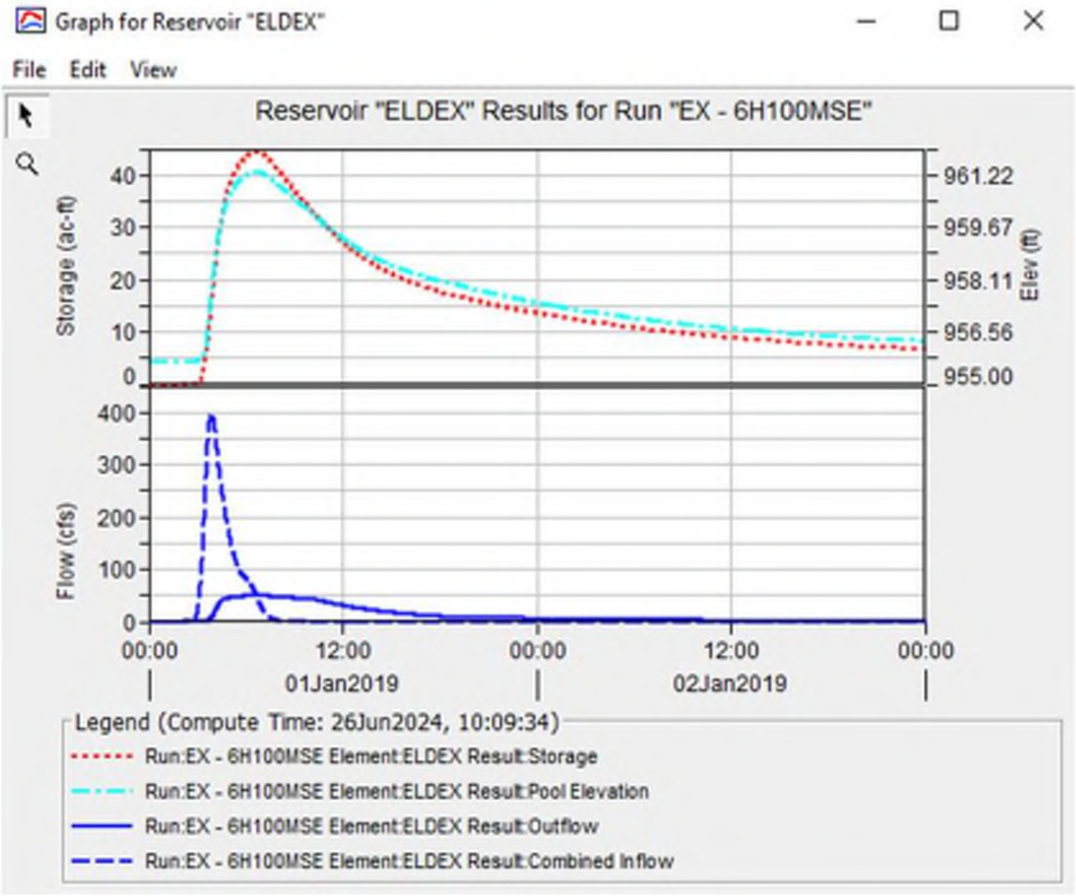
Project: Lake_Erin_Proposed Simulation Run: EX - 6H100MSE
Reservoir: ELDEX

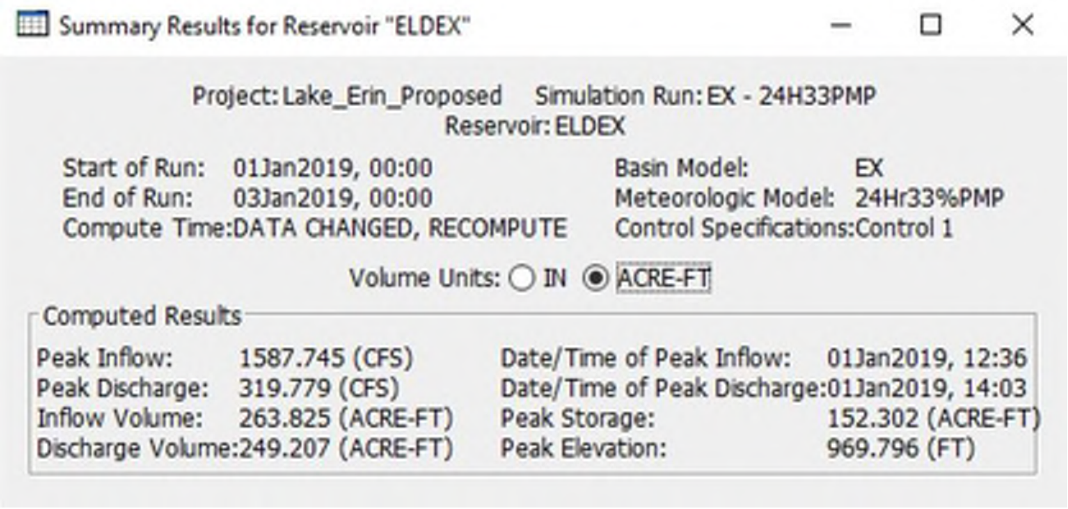
Start of Run: 01Jan2019, 00:00 Basin Model: EX
End of Run: 03Jan2019, 00:00 Meteorologic Model: 6Hr100YrMSE5
Compute Time: 26Jun2024, 10:09:34 Control Specifications: Control 1

Volume Units: ○ IN ● ACRE-FT

Computed Results

Peak Inflow: 397.292 (CFS) Date/Time of Peak Inflow: 01Jan2019, 03:53
Peak Discharge: 50.179 (CFS) Date/Time of Peak Discharge: 01Jan2019, 06:39
Inflow Volume: 56.408 (ACRE-FT) Peak Storage: 44.476 (ACRE-FT)
Discharge Volume: 49.592 (ACRE-FT) Peak Elevation: 961.323 (FT)





Summary Results for Reservoir "ELDEX"

Project: Lake_Erin_Proposed Simulation Run: EX - 24H33PMP
Reservoir: ELDEX

Start of Run: 01Jan2019, 00:00
End of Run: 03Jan2019, 00:00
Compute Time: DATA CHANGED, RECOMPUTE

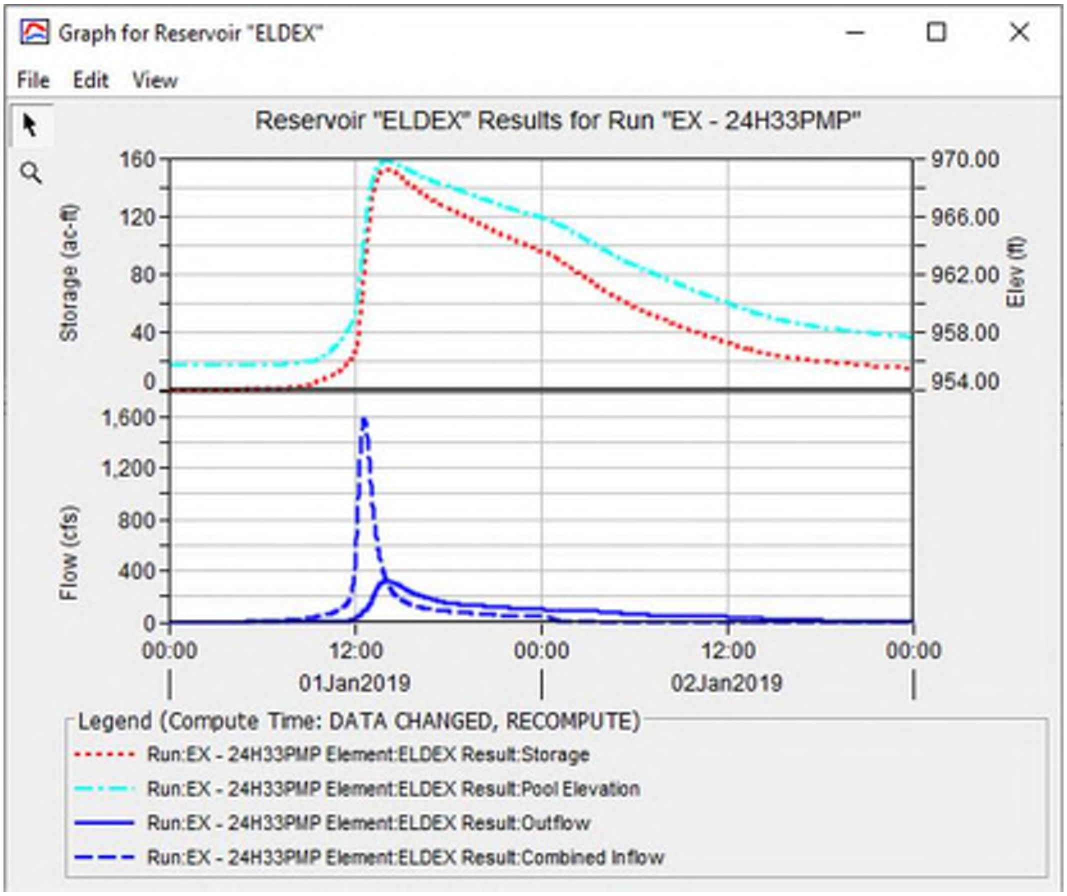
Basin Model: EX
Meteorologic Model: 24Hr33%PMP
Control Specifications: Control 1

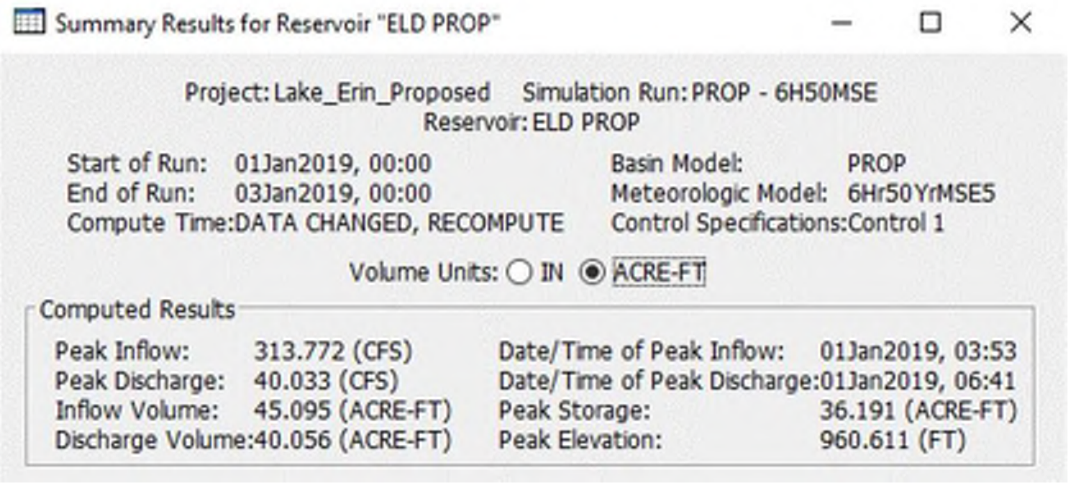
Volume Units: ○ IN ● ACRE-FT

Computed Results

Peak Inflow: 1587.745 (CFS)
Peak Discharge: 319.779 (CFS)
Inflow Volume: 263.825 (ACRE-FT)
Discharge Volume: 249.207 (ACRE-FT)

Date/Time of Peak Inflow: 01Jan2019, 12:36
Date/Time of Peak Discharge: 01Jan2019, 14:03
Peak Storage: 152.302 (ACRE-FT)
Peak Elevation: 969.796 (FT)





Summary Results for Reservoir "ELD PROP"

Project: Lake_Erin_Proposed Simulation Run: PROP - 6H50MSE
Reservoir: ELD PROP

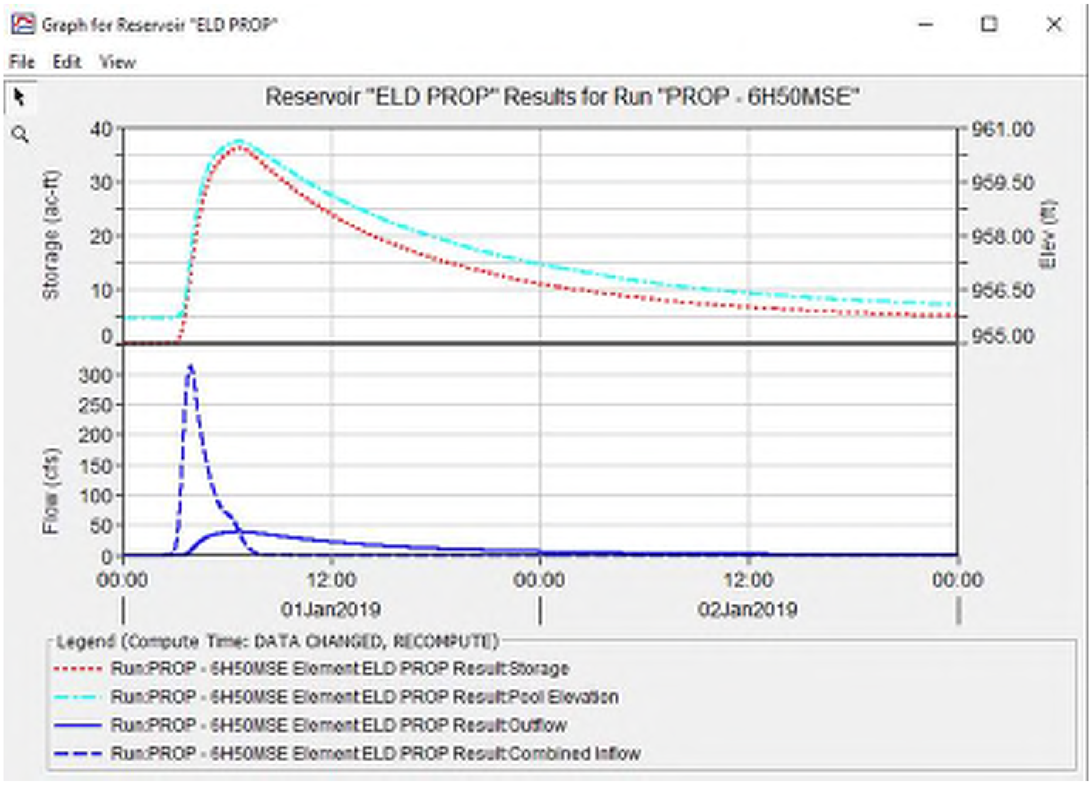
Start of Run: 01Jan2019, 00:00
End of Run: 03Jan2019, 00:00
Compute Time: DATA CHANGED, RECOMPUTE

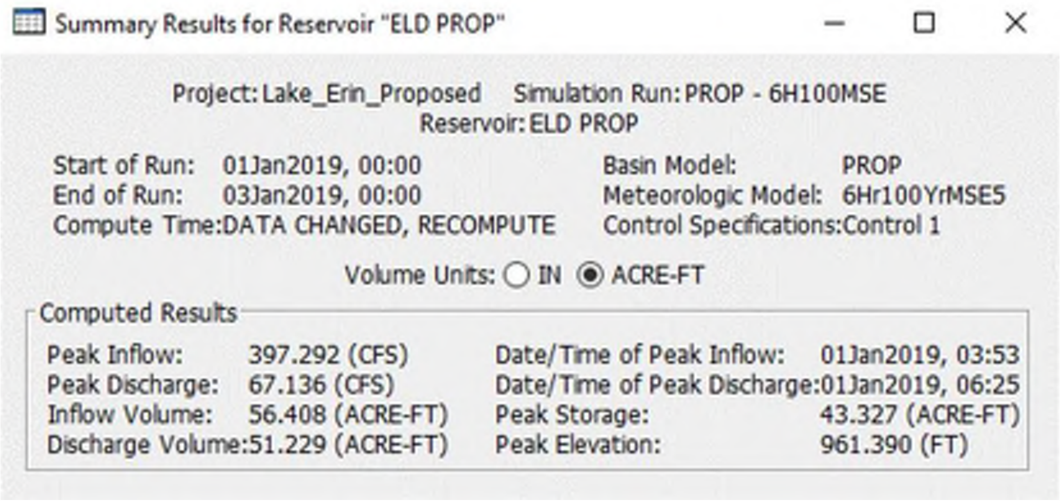
Basin Model: PROP
Meteorologic Model: 6Hr50YrMSE5
Control Specifications: Control 1

Volume Units: ○ IN ● ACRE-FT

Computed Results

Peak Inflow:	313.772 (CFS)	Date/Time of Peak Inflow:	01Jan2019, 03:53
Peak Discharge:	40.033 (CFS)	Date/Time of Peak Discharge:	01Jan2019, 06:41
Inflow Volume:	45.095 (ACRE-FT)	Peak Storage:	36.191 (ACRE-FT)
Discharge Volume:	40.056 (ACRE-FT)	Peak Elevation:	960.611 (FT)





Summary Results for Reservoir "ELD PROP"

Project: Lake_Erin_Proposed Simulation Run: PROP - 6H100MSE
Reservoir: ELD PROP

Start of Run: 01Jan2019, 00:00
End of Run: 03Jan2019, 00:00
Compute Time: DATA CHANGED, RECOMPUTE

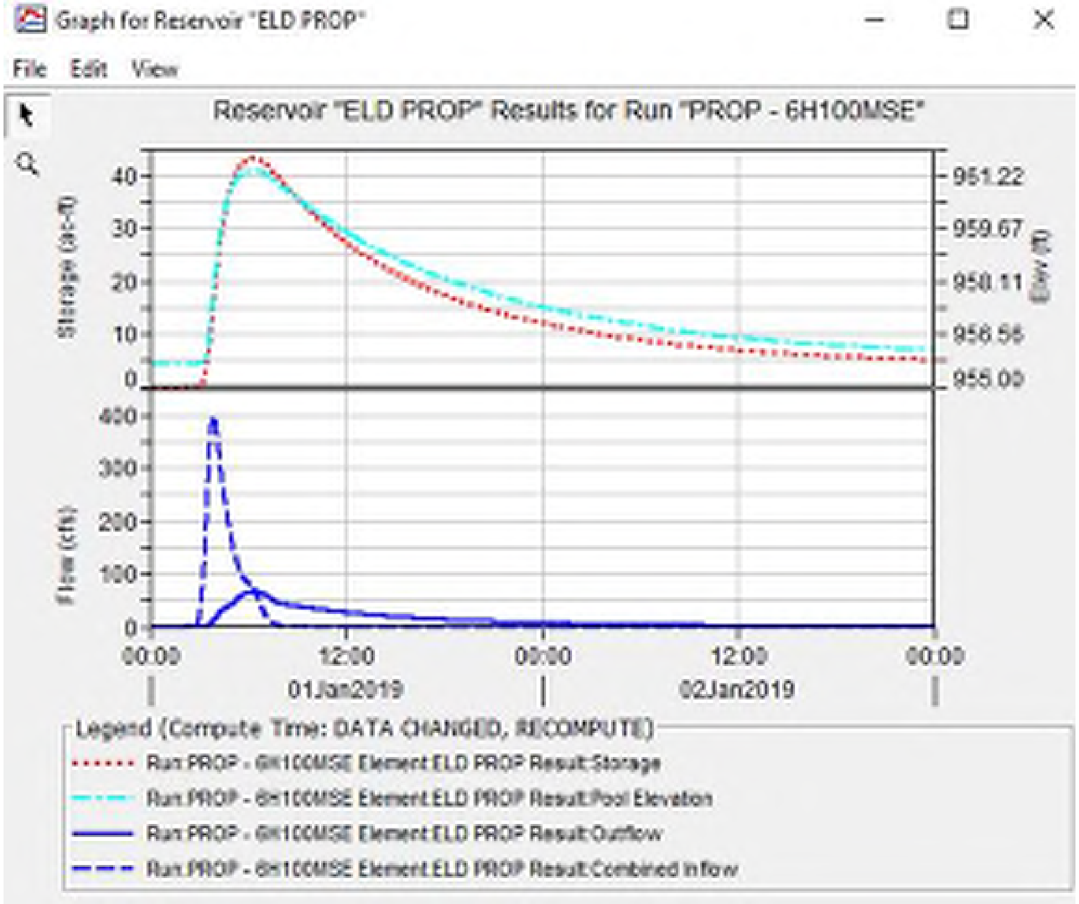
Basin Model: PROP
Meteorologic Model: 6Hr100YrMSE5
Control Specifications: Control 1

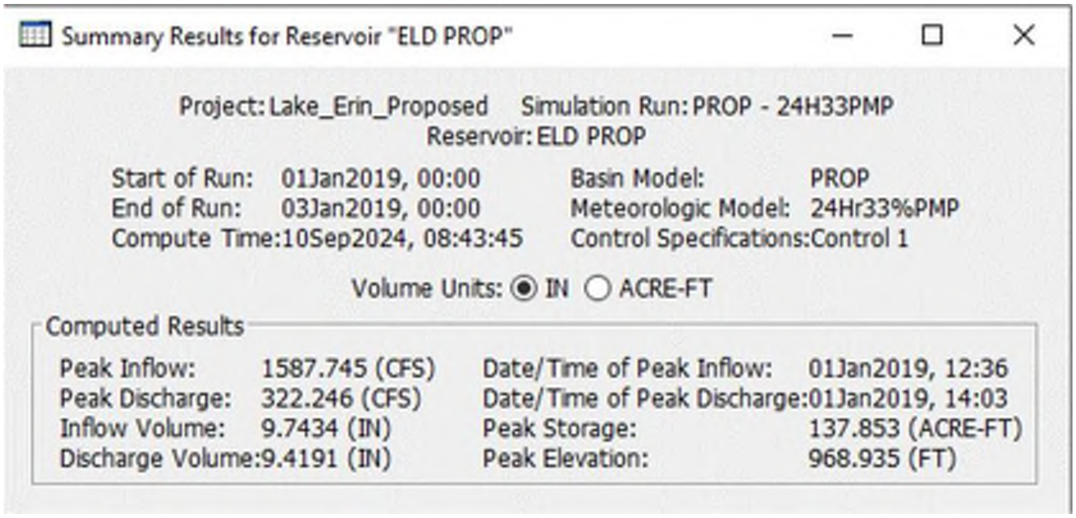
Volume Units: ○ IN ● ACRE-FT

Computed Results

Peak Inflow: 397.292 (CFS)
Peak Discharge: 67.136 (CFS)
Inflow Volume: 56.408 (ACRE-FT)
Discharge Volume: 51.229 (ACRE-FT)

Date/Time of Peak Inflow: 01Jan2019, 03:53
Date/Time of Peak Discharge: 01Jan2019, 06:25
Peak Storage: 43.327 (ACRE-FT)
Peak Elevation: 961.390 (FT)





Project: Lake_Erin_Proposed Simulation Run: PROP - 24H33PMP
Reservoir: ELD PROP

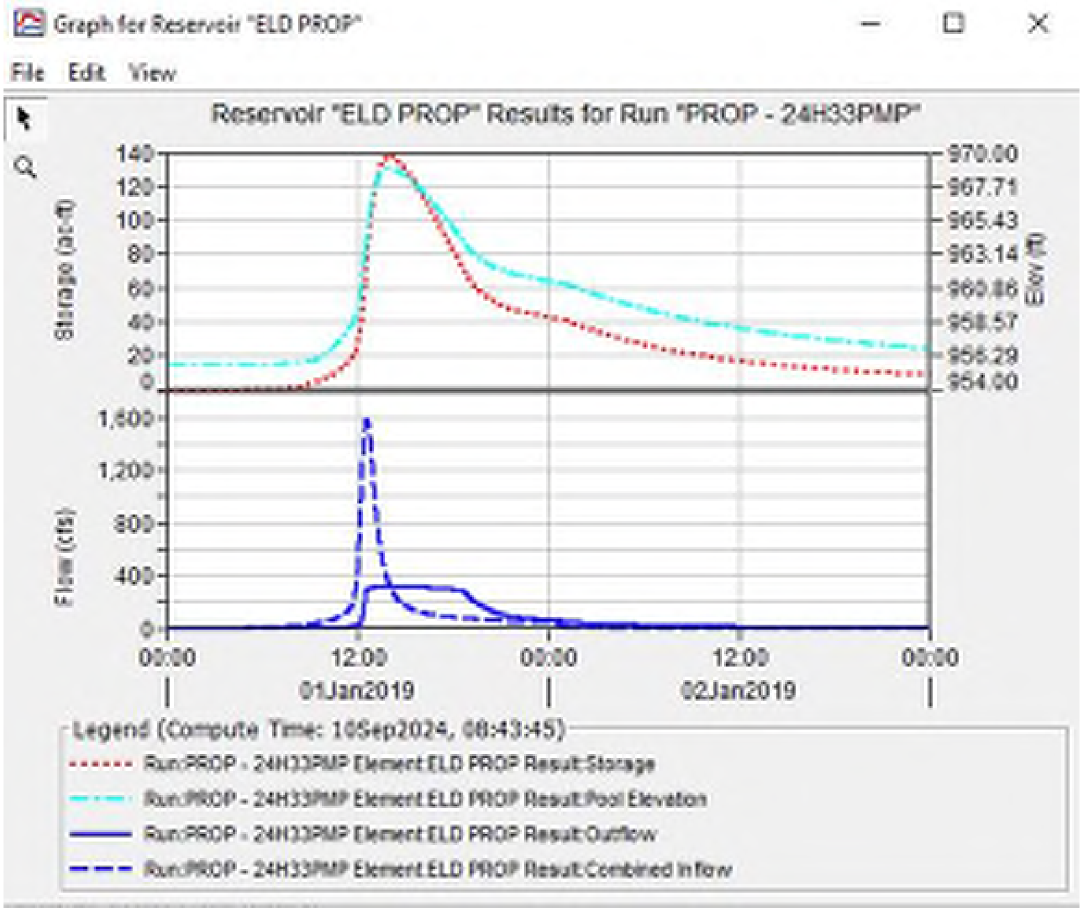
Start of Run: 01Jan2019, 00:00
End of Run: 03Jan2019, 00:00
Compute Time: 10Sep2024, 08:43:45

Basin Model: PROP
Meteorologic Model: 24Hr33%PMP
Control Specifications: Control 1

Volume Units: ● IN ○ ACRE-FT

Computed Results

Peak Inflow:	1587.745 (CFS)	Date/Time of Peak Inflow:	01Jan2019, 12:36
Peak Discharge:	322.246 (CFS)	Date/Time of Peak Discharge:	01Jan2019, 14:03
Inflow Volume:	9.7434 (IN)	Peak Storage:	137.853 (ACRE-FT)
Discharge Volume:	9.4191 (IN)	Peak Elevation:	968.935 (FT)



C.3 Low-Level Outlet Drawdown Curve

AECOM

12420 Milestone Center Drive, Suite 150
Germantown, Maryland 20876

DESIGN AJW DATE 3-Jul-2024
CHECK JG DATE 3-Jul-2024
PROJECT: Erin Lake Dam Proposed Conditions

Location	**Description**	**Area (SF)**	**Length**	**Manning's n**	**$(A1/AX)^2$**	**K**	**$K(A1/AX)^2$**	**K Source**	**Description**
Intake Structure	Trash Rack	18			0.49	0.98	0.48	2	50% Blocked Trash Rack Loss: $K = 1.45 - 0.45 (a_n/a_g) - (a_n/a_g)^2$
LLO Conduit (15")	Entrance	1.23			104.44	0.50	52.22	1	Entrance Loss (Square Edge = 0.5)
	Friction	1.23	18	0.013	104.44	0.13	13.58	1	Conduit Friction Losses: $K = 29.1n^2L/(R^{4/3})$
	Exit	1.23			104.44	1.00	104.44	1	Exit Loss
Intake Riser (6'x9')	Gate	1.23			104.44	0.00	0.00	1	Gate Loss (100% Open)
	Friction	30	9	0.013	0.18	0.14	0.03	1	Conduit Friction Losses: $K = 29.1n^2L/(R^{4/3})$
Spillway Conduit (48")	Entrance	12.57			1.00	0.50	0.50	1	Entrance Loss (Square Edge = 0.5)
	Friction	12.57	130	0.013	1.00	0.94	0.94	1	Conduit Friction Losses: $K = 29.1n^2L/(R^{4/3})$
	Exit	12.57			1.00	1.00	1.00	1	Exit Loss
Total Losses							**173.19**		
Tailwater Elevation							**942.00**		Crown of Discharge Conduit

K Sources

[1] Brater, E.F., King, H.W., Lindell, J.E., & Wei, C.Y. (1996) *Handbook of Hydraulics*, 7th Ed. McGraw-Hill Education.

[2] United States Department of the Interior, Bureau of Reclamation. (1987). *Design of small dams: A water resources technical publication*. SBS Publishers & Distributors.

AECOM

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DESIGN AJW DATE 12-Sep-2024
CHECK JCG DATE 13-Sep-2024
PROJECT: Erin Lake Dam Proposed Conditions

Drawdown Rating Calculations

WSE (feet NAVD88)	Volume (acre-feet)[1]	Proposed LLO Discharge (cfs)	Incremental Duration (hours)	Total Duration (hours)	Drawdown Rate (feet per day)	Total Duration (days)	Benchmark
955.7	29.60	28	0.0	0.0	0.0	0.0	Normal Pool
955.0	26.35	28	1.4	1.4	12.0	0.1	
954.0	22.16	27	1.8	3.2	13.3	0.1	
953.0	18.41	25	1.7	4.9	14.1	0.2	
952.0	15.02	24	1.7	6.6	14.1	0.3	
951.0	11.95	23	1.6	8.2	15.0	0.3	
950.0	9.21	22	1.5	9.7	16.0	0.4	2/3 NP Volume
949.0	6.83	20	1.4	11.1	17.1	0.5	
948.0	4.84	19	1.2	12.3	20.0	0.5	
947.0	3.25	17	1.1	13.4	21.8	0.6	
946.0	2.11	15	0.9	14.3	26.7	0.6	
945.0	1.22	13	0.8	15.1	30.0	0.6	
944.0	0.58	11	0.6	15.7	40.0	0.7	
943.0	0.28	8	0.4	16.1	60.0	0.7	
942.0	0.09	0	0.6	16.7	40.0	0.7	
941.0	0.00	0	1.3	17.4	36.9	0.7	Bottom of Reservoir

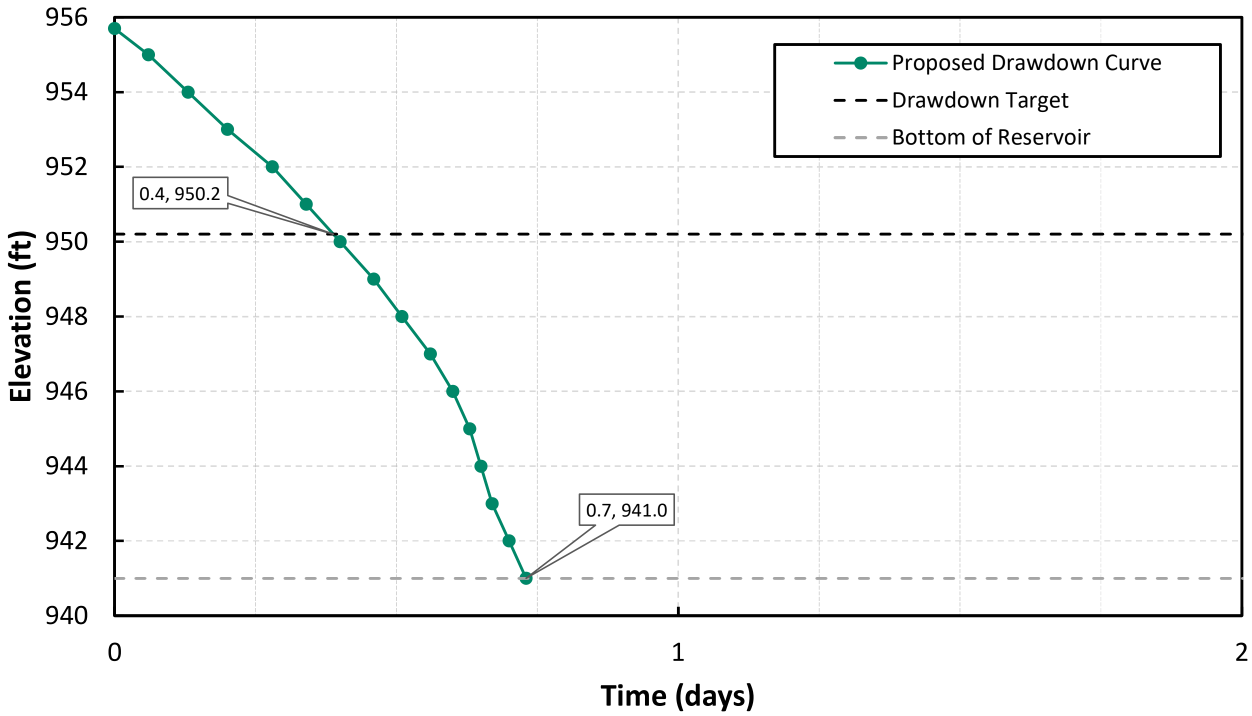
Sources

[1] AECOM (2021). *Hydrologic and Hydraulic Analysis Report Erin Lake Dam*.

[2] Georgia Department of Natural Resources Environmental Protection Divison Watershed Protection Branch Safe Dams Program (2015). *Engineering Guidelines*.

Benchmark Definitions

[A] Georgia requires dams to have sufficient capacity to drawdown the reservoir to 2/3 of the normal pool by volume within 10 days.



C.4 Impact Basin and Riprap Apron Sizing

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Lake Erin Dam Rehabilitation

Project ref:
60727041

From:
Andrew Weis

Date:
July 3, 2024

DRAFT

To:
Jonathan Genn

CC:
Bob Pinciotti
Jeff Blass

Memo

Attachment A – Riprap Apron Sizing Calculations

Subject: Energy Dissipation

The purpose of this memo is to describe the calculations performed to size the approximate dimensions of the energy dissipation features for the Lake Erin Dam Rehabilitation project. The two energy dissipation features are a United States Bureau of Reclamation (USBR) Type VI Impact Basin and a riprap basin. These two features were sized using the U.S. Department of Transportation Federal Highway Administration Hydraulic Engineering Circular No. 14, Third Edition Hydraulic Design of Energy Dissipators for Culverts and Channels (HEC-14).

Introduction

The proposed spillway conduit for Lake Erin Dam is a 48-inch RCCP conduit that is approximately 120 feet with an upstream invert at Elevation 940.0 feet and a downstream invert at Elevation 939.0 feet. During the spillway design flood (SDF), which is equal to approximately 1/3 of the probable maximum flood (PMF) for Lake Erin Dam, the maximum discharge from the proposed spillway conduit is approximately 322 cubic feet per second (cfs) and the maximum velocity is approximately 26 feet per second (ft/s). To avoid erosive flows downstream of the proposed spillway conduit, the maximum velocity must be reduced. The proposed impact basin will be sized such that the proposed system discharges flow at a velocity at or below existing conditions for the SDF event. Based on the Georgia Department of Natural Resources Flood Map Viewer allows the download of existing HEC-RAS models for river systems. A HEC-RAS model was downloaded from this site that included the Lake Erin watershed. This model was slightly edited to have a boundary condition equal to the peak discharge during the SDF event. The resulting channel velocity for this HEC-RAS simulation was approximately 8-9 ft/s. The target velocity downstream of the proposed energy dissipation system shall be 8 ft/s as to not increase the velocity in the channel for the proposed conditions.

USBR Type VI Impact Basin

Inputs

- Q = 322.3 cfs
- Vo = 25.7 ft/s

Sizing Calculations

The impact basin sizing calculations follow the recommended design procedure described in Section 9.4 USBR Type VI Impact Basin, HEC-14.

Step 1. Determine the maximum discharge, Q, and velocity, Vo. Compute the flow area at the end of the spillway conduit, Use the flow area to compute the equivalent depth, ye.

The maximum discharge and velocity at the end of the conduit for the SDF flow conditions as described in the inputs are used for the equivalent depth calculations.

$$\mathrm{A} = \frac{Q}{Vo} \quad \mathrm{A} = \frac{322.3}{25.7} \quad = \quad \text{12.54 ft^2}$$

$$\mathrm{Ye} = (\frac{A}{2})^{.5} \quad \mathrm{Ye} = \left(\frac{12.54}{2}\right)^{.5} \quad = \quad \text{2.50 ft}$$

Step 2. Compute the Froude number, Fr, and the energy at the end of the pipe, Ho.

$$\mathrm{Fr} = \frac{V}{\sqrt{g*Ye}} \quad \mathrm{Fr} = \frac{25.7}{\sqrt{32.2*2.50}} \quad = \quad 2.86$$

$$\mathrm{Ho} = Ye + \frac{V\mathrm{sdf}^2}{2*g} \quad \mathrm{Ho} = 2.50 + \frac{25.7\text{^}2}{64.4} \quad = \text{12.76 ft}$$

Step 3. Determine Ho/WB from Figure 9.14, HEC-14. Calculate the required width of basin, Wb.

$$\frac{\mathrm{Ho}}{\mathrm{Wb}} = 0.8$$

$$\mathrm{Wb} = \frac{Ho}{\frac{Ho}{Wb}} \quad \mathrm{Wb} = \frac{12.76}{0.8} \quad = \quad \text{15.95 ft = 16 ft}$$

Step 4. Obtain the remaining dimensions of the USBR Type VI impact basin from Table 9.2 (HEC-14) using Wb = 16 feet obtained from step 3. Results are summarized in the follow table:

Dimension	Value	Dimension	Value
h1	12.25	W1	1.25
h2	6.00	W2	3.00
h3	2.67	t1	0.75
h4	6.67	t2	1.00
L	21.33	t3	1.00
L1	9.08	t4	1.00
L2	12.25	t5	0.50

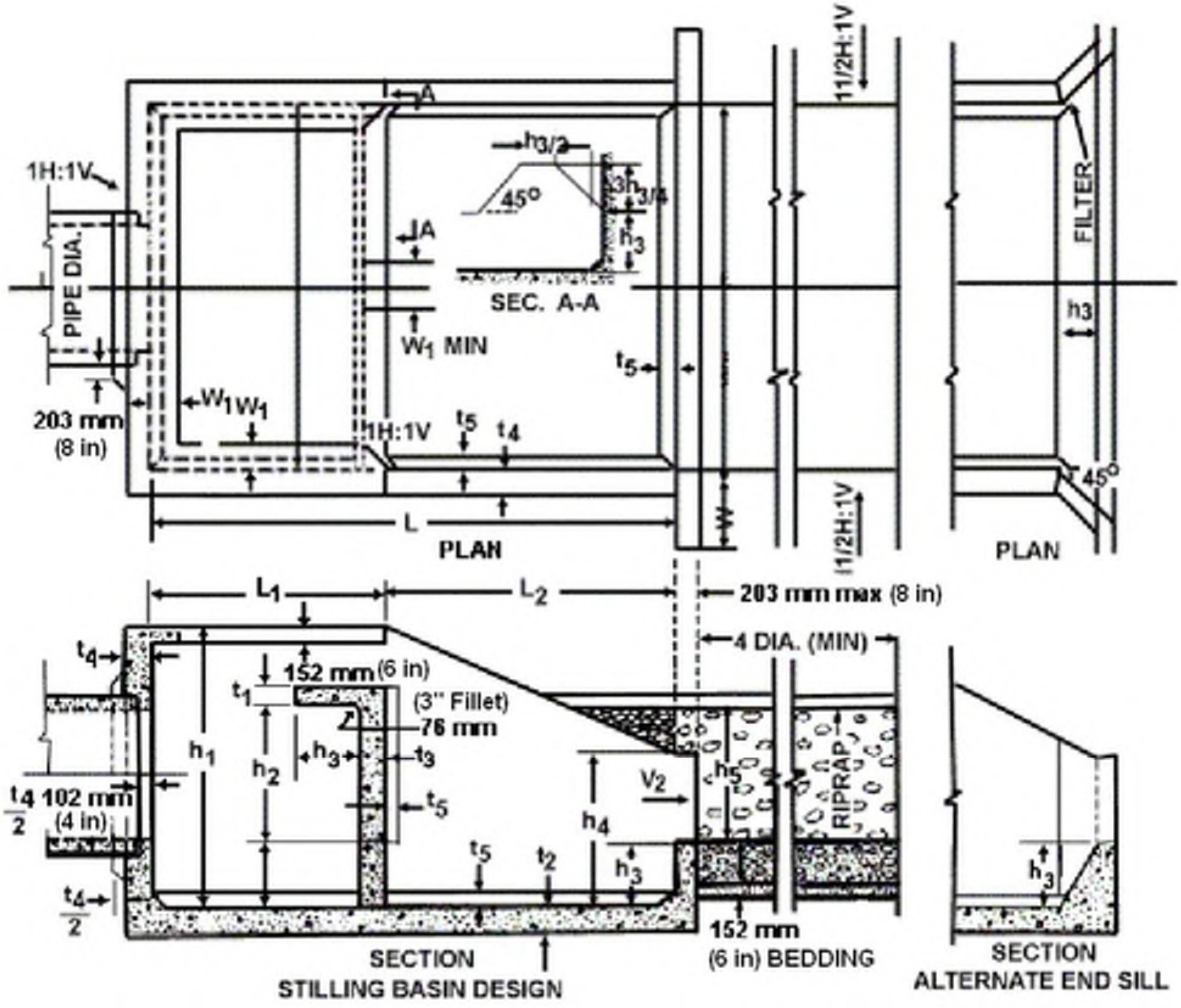

Figure 1: HEC-14 Figure 9.13. USBR Type VI Impact Basin

Step 5. Determine the exit velocity, Vb, by trial and error using an energy balance between the culvert exit and the basin exit. Determine if this velocity is acceptable and whether or not riprap protection is needed downstream. Use Figure 9.15 (HEC-14) to determine Hl/Ho.

$$\mathrm{Hb} = \frac{Q}{Wb * Vb} + \frac{Vb^2}{2g} = Ho\left(1 - \frac{Hl}{Ho}\right)$$

This equation is a cubic equation yielding 3 solutions, two positive and one negative. The negative solution is discarded. The two positive roots yielded a subcritical and supercritical solution. Where low or no tailwater exits, the supercritical solution is taken. Where sufficient tailwater existing, the subcritical solution is taken. For the SDF event, the exit velocity from the impact basin is equal to the subcritical solution.

$$\frac{322.3}{16 * Vb} + \frac{Vb^2}{64.4} = 12.76(1 - 0.51)$$

$$\frac{20.14}{Vb} + \frac{Vb^2}{64.4} = 6.25$$

$$\frac{Vb^3}{64.4} - 6.25Vb + 20.14 = 0$$

$$Vb = \ -21.5\ \mathrm{ft}/s$$

$$3.3\ \mathrm{ft}/s$$

$$18.2\ \mathrm{ft}/s$$

The subcritical solution for the cubic equation is 18.2 ft/s. This velocity is higher than the existing velocity in the channel just downstream of the dam. Thus, a riprap basin will be required to further dissipate the energy exiting the proposed spillway conduit.

Q = V* A

A = Q / V

D * w = Q / V

D = Q / (V*w) = 322 / (18.25 * 16) = 1.1 ft

Riprap Apron

When the energy exiting the proposed energy dissipation structure still needs to be reduced, HEC-14 recommends designing a riprap apron. Section 10.3 describes the process for sizing a riprap apron after an energy dissipation structure. Using Figure 10.3, a ratio between velocity entering and exiting a riprap apron based on the length of the apron and the equivalent circular diameter for the flow. Based on this figure, a 50-foot-long riprap apron of 12-inch D50 stone would reduce the velocity below the target velocity of 8 ft/s.

See riprap apron sizing calculations attached to this memo to see the exit velocity for a range of apron lengths and D50 values.

To confirm that this proposed riprap would reduce the velocity below the target, a HEC-RAS two-dimensional hydraulic model was developed. The results of this two-dimensional model confirm that the proposed riprap apron will return the spillway discharge to non-erodible levels. The two-dimensional model estimates that at the downstream limit of the project site, the velocity of the water is just under 5 ft/s. See Figure 2 below which shows velocity in the outlet channel calculated in the two-dimensional model.

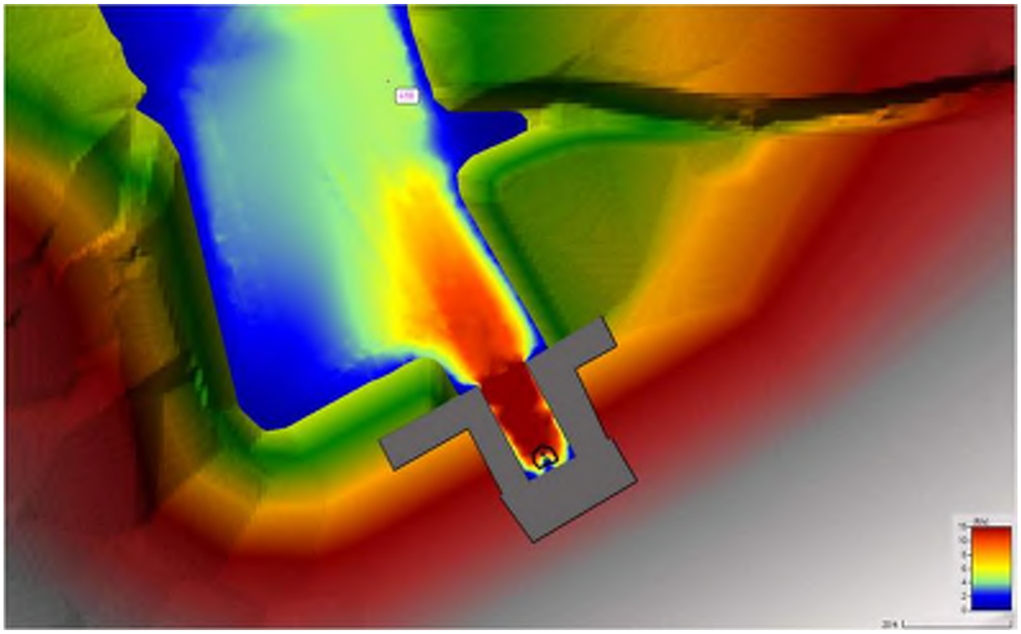

Figure 2: Proposed Conditions Two-Dimensional Model Results

References

FHWA. (2006). Hydraulic Design of Energy Dissipators for Culverts and Channels, HEC-14. United States Department of Transportation, Federal Highway Administration. Washington, D.C.

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DESIGN	AJW	DATE	3-Jul-2024
CHECK	JG	DATE	3-Jul-2024
PROJECT:	Erin Lake Dam Proposed Conditions		

Riprap Apron Design

1. Compute the culvert outlet velocity, V_o, and equivalent brink depth, y_e.

Type =	Rectangular		"Rectangular" or "Circular"		
D =	4	ft	Diameter of circular culvert or height of rectangular culvert		
B =	16	ft	Width of rectangular culvert (if applicable)		
Q =	322.34	cfs	Culvert design storm discharge	Return Period (years) =	SDF
V_n =	18.25	ft/s	Velocity exiting impact basin		
y_n =	1.1	ft	Normal depth exiting impact basin		
Fr_n =	**3.07**		Froude number for normal conditions (Subcritical: Fr < 1, Supercritical: Fr > 1)		
TW =	1.10	ft	Tailwater depth		

Compute the Froude number for brink conditions using brink depth for box culverts ($y_e = y_o$) and equivalent depth ($y_e = (A/2)^{1/2}$) for non-rectangular sections.

y_o =	**1.10**	ft	Brink depth (depth of flow at culvert outlet)
A_o =	17.60	ft^2	Brink area (area of flow at culvert outlet)
V_o =	**18.25**	ft/s	Culvert outlet velocity
y_e =	**1.10**	ft	Equivalent brink depth
Fr_o =	**3.07**		Froude number for brink conditions, $Fr_o = V_o / (gy_e)^{0.5}$

2 Assess the need for additional riprap downstream of the dissipator exit. If $TW/y_o < 0.75$, no additional riprap is needed. With high tailwater ($TW/y_o \geq 0.75$), estimate the centerline velocity at a series of downstream cross sections using Figure 10.3 to determine the size and extent of additional protection. The riprap design details should be in accordance with specifications in HEC 11 (Brown and Clyde, 1989) or similar highway department specifications.

TW/y_o = **1.00** ≥ 0.75, Additional riprap needed

D_e = **4.733816218** ft Equivalent circular diameter

Riprap must extend at minimum distance L where $V_L \leq V_{allow}$.

L (ft)	L/D_e	V_L/V_o	V_L (ft/s)	D_{50} (ft)
40	**8.45**	0.50	**9.13**	**0.54**
50	**10.56**	0.40	**7.30**	**0.35**
60	**12.67**	0.30	**5.48**	**0.20**
150	**31.69**	0.12	**2.19**	**0.03**

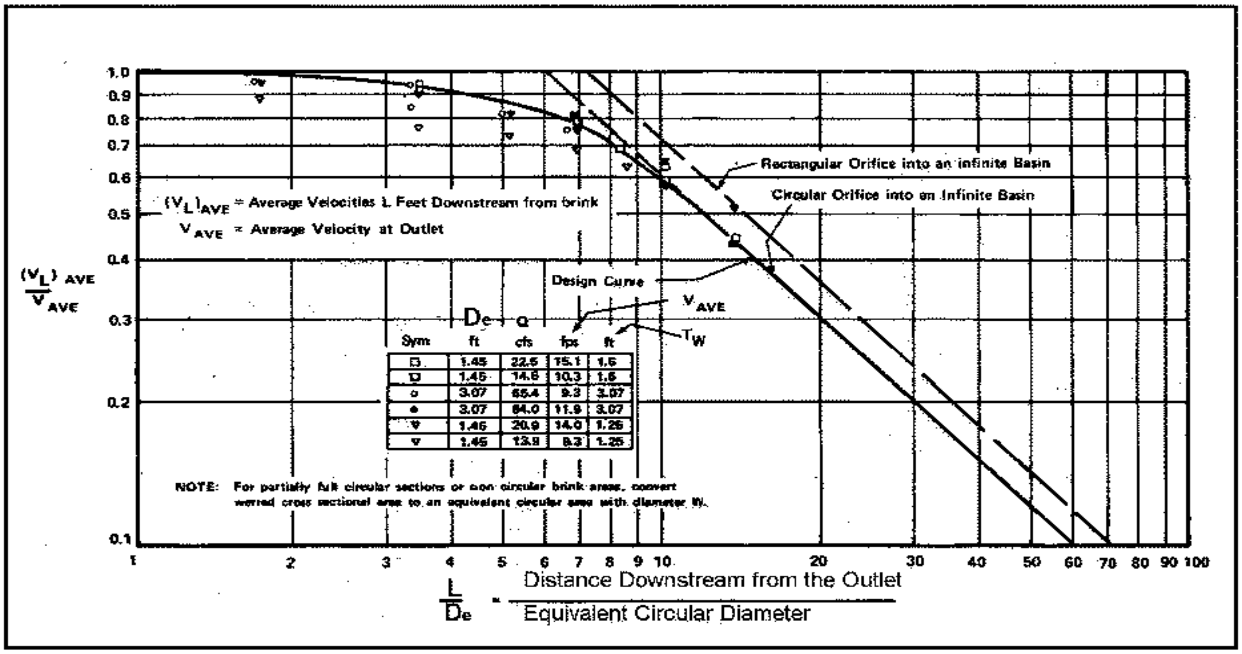
where L = Distance downstream from the outlet

V_L = Velocity L feet downstream from brink

V_L/V_o = read from Figure 10.3

D_{50} = $[0.692 / (S-1)] [V^2 / (2g)]$ - Equation 10.6

S = Riprap specific gravity = 2.65



	De	Q		
Sym	ft	cfs	fps	ft
□	1.45	22.5	15.1	1.6
□	1.45	14.6	10.3	1.6
○	3.07	65.4	9.3	3.07
●	3.07	84.0	11.9	3.07
▽	1.46	20.9	14.0	1.26
▽	1.45	13.9	8.3	1.25

Figure 10.3. Distribution of Centerline Velocity for Flow from Submerged Outlets

Methodology from: Hydraulic Engineering Circular No. 14, Third Ed. *Hydraulic Design of Energy Dissipators for Culverts and Channels* . Federal Highway Administration Publication No. FHWA-NHI-06-086, Chapter 10, pp. 10-1 through 10-6. U.S. Department of Transportation. July 2006.

C.5 Temporary Bypass System

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Project name:
Lake Erin Dam Rehabilitation

Project ref:
60727041

To:
Jonathan Genn

CC:
Bob Pinciotti
Jeff Blass

From:
Andrew Weis

Date:
July 3, 2024

DRAFT

Memo

Subject: 95% Design Temporary Bypass Conduit Sizing

The purpose of this memo is to describe the calculations to determine the approximate size of the conduit required to temporarily bypass flows during different phases of construction for the Lake Erin Dam Rehabilitation project. The range of storm events, conduits sizes, and number of conduits has been refined to reflect the current design for Lake Erin Dam.

Introduction

The proposed spillway conduit for Lake Erin Dam is a 48-inch RCCP conduit that is approximately 130 feet with an upstream invert at Elevation 940.0 feet and a downstream invert at Elevation 939.0 feet. To install the proposed spillway conduit, a large portion of the embankment will need to be excavated. While the embankment is opened and proposed features are being constructed, a temporary bypass system will need to be installed to carry stormwater discharge coming into the reservoir. To determine an appropriate bypass system, various conduit alternatives were considered given some initial assumptions. These assumptions, such as the invert / outlet elevation, conduit material, and alignment reflect the current design for the 95% submittal.

Bypass Phasing

The proposed temporary bypass system will involve of three phases. The first phase (Phase 2) utilizes two 48-inch HDPE-S conduits to discharge any storm up to the 1% AEP event. Phase 2 will be the primary bypass system throughout construction. The second phase (Phase 3A) consists of one 48-inch HPDE-S conduit that is connected the newly constructed intake tower. The third phase (Phase 3B) consists of one 15-inch HDPE-S conduits that is connected to the newly constructed low-level outlet conduit. Phases 3A and 3B will take place after most of the proposed features are installed and the embankment is being backfilled.

Model Inputs

The inlet elevation for all proposed temporary bypass conduits was assumed to be equal to the bottom of the reservoir, which is at Elevation 941.0. The two 48-inch HDPE-S conduits in Phase 2 were modeled directly in HEC-HMS using the inputs for a circular culvert outlet shown in Table 1 below. Two barrels were selected to reflect the two bypass conduits.

Table 1. Phase 2 Model Inputs

Length (ft)	Diameter (ft)	Inlet Elevation	Entrance Coefficient	Outlet Elevation (ft)	Exit Coefficient	Manning's n
286	4	941.0	0.9	937.7	1.0	0.013

A rating curve was manually calculated for the bypass conduits under Phase 3A and 3B. This was done to account for the additional losses experienced in these two phases.

To estimate how the proposed temporary bypass system for each phase would respond to a range of storm events, the 4% AEP (25-year), 2% AEP (50-year), and 1% AEP (100-year) storm events were routed through each phase.

Simulation Results

Table 2. Phase 2 [(2) 48-inch HDPE-S Bypass Conduits]

Storm Event	Maximum Water Surface Elevation (ft)	Peak Discharge (cfs)	Peak Velocity (ft/s)
100-year	950.1	281.4	11.2
50-year	948.5	247.6	9.9
25-year	947.2	213.2	8.5

Table 3: Phase 3A [(1) 48-inch HDPE-S Bypass Conduit]

Simulation	Maximum Water Surface Elevation (ft)	Peak Discharge (cfs)	Peak Velocity (ft/s)
100-year	953.9	158.1	12.6
50-year	952.0	144.1	11.5
25-year	950.2	128.8	10.2

Table 4: Phase 3B [(1) 15-inch HDPE-S Bypass Conduit]

Simulation	Maximum Water Surface Elevation (ft)	Peak Discharge (cfs)	Peak Velocity (ft/s)
100-year	958.5	29.4	2.3
50-year	956.9	27.9	2.2
25-year	955.2	26.2	2.1

C.6 Wave Height Analysis Calculations

AECOM

DESIGN: AJW
CHECK: JCG
DATE: 13-Sep-2024
PROJECT: Lake Erin Dam
DESCRIPTION: Wave Height Analysis

Freeboard Calculations (per *A Guide for Design and Layout of Vegetated Wave Protection for Earthen Embankments and Shorelines, Technical Release 56 (NRCS, 2014)*)

Step 1: Determine Effective Fetch Length (maximum of three trial locations).

Trial Location 1

Angle to Normal (degrees)	Cosine of Angle	Length (feet)	Length* Cosine (feet)
0	1.000	878	878
6	0.995	878	874
6	0.995	854	850
12	0.978	505	494
12	0.978	610	597
18	0.951	472	449
18	0.951	547	520
24	0.914	329	301
24	0.914	458	419
30	0.866	259	224
30	0.866	376	326
36	0.809	223	180
36	0.809	319	258
42	0.743	204	152
42	0.743	307	228
	13.512		6750
Effective Fetch Length (feet)			**500**

Step 2: Estimate 50-year recurrence overland wind velocity from NRCS TR-56 (Fig. 4).

V_L (mph) =	85

Figure 4 Maximum wind velocity over land (mph), 50-year recurrence

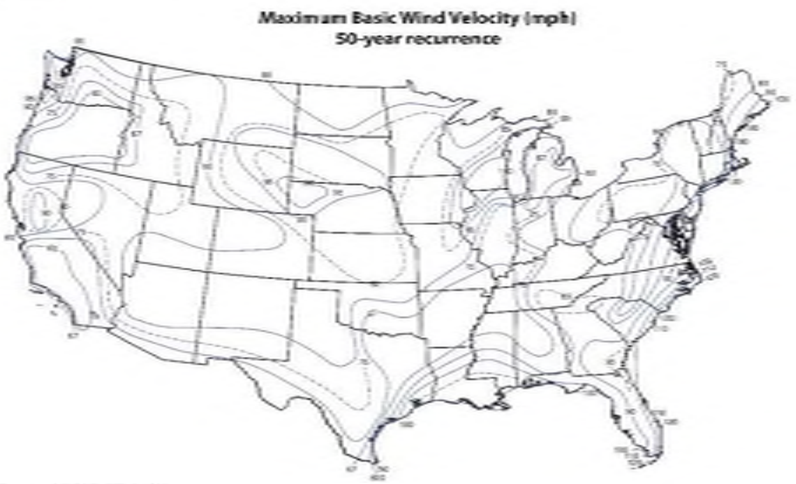
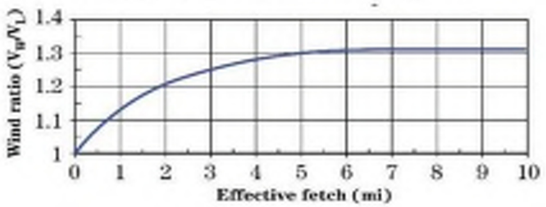
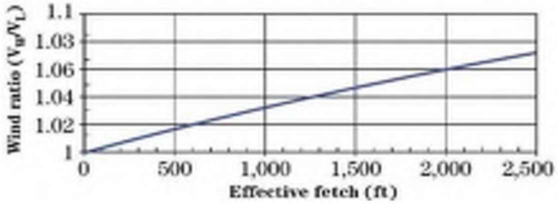


Figure 5 Wind ratio

(a) Wind relationship overwater to overland (miles)



(b) Wind relationship overwater to overland (feet)



Step 3: Determine the over water wind velocity from NRCS TR-56 (Fig. 5).

V_L (mph) =	85
V_W/V_L =	1.019
V_W (mph) =	**86.6**

Step 4: Determine the significant wave height (freeboard).

$$H_S = 0.0232V_W^{1.06}F_e^{0.47}$$

V_W (mph) =	86.6
F_e (miles) =	0.09
H_S (feet) =	**0.8**

AECOM

DESIGN: AJW
CHECK: JCG
DATE: 13-Sep-2024
PROJECT: Lake Erin Dam
DESCRIPTION: Wave Height Analysis

Freeboard Calculations (per *A Guide for Design and Layout of Vegetated Wave Protection for Earthen Embankments and Shorelines, Technical Release 56 (NRCS, 2014)*)

Step 1: Determine Effective Fetch Length (maximum of three trial locations).

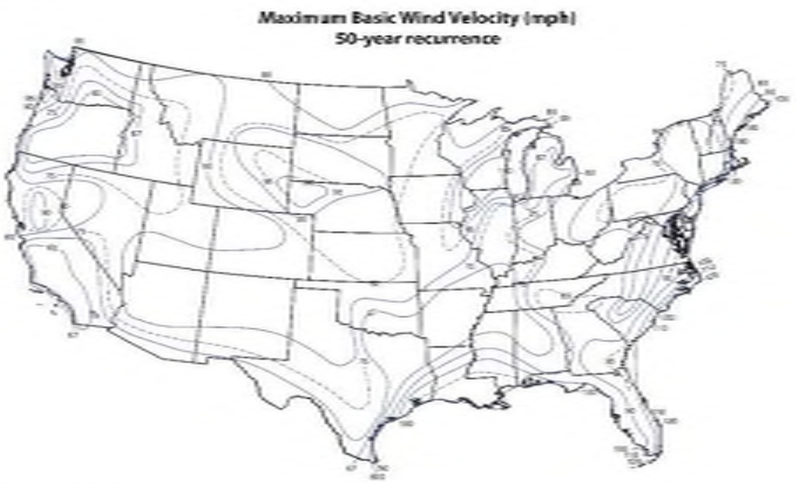
Trial Location 2

Angle to Normal (degrees)	Cosine of Angle	Length (feet)	Length* Cosine (feet)
0	1.000	883.9	884
6	0.995	863.1	859
6	0.995	623.4	620
12	0.978	792.5	775
12	0.978	575.2	563
18	0.951	486.3	462
18	0.951	502.1	477
24	0.914	475.2	434
24	0.914	401.4	367
30	0.866	284.4	246
30	0.866	326.7	283
36	0.809	249.2	202
36	0.809	312.0	252
42	0.743	203.6	151
42	0.743	292.3	217
	13.512		6792
Effective Fetch Length (feet)			**503**

Step 2: Estimate 50-year recurrence overland wind velocity from NRCS TR-56 (Fig. 4).

V_L (mph) =	85

Figure 4 Maximum wind velocity over land (mph), 50-year recurrence

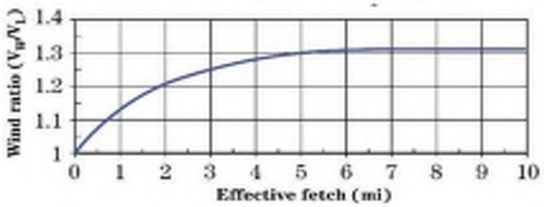


Step 3: Determine the over water wind velocity from NRCS TR-56 (Fig. 5).

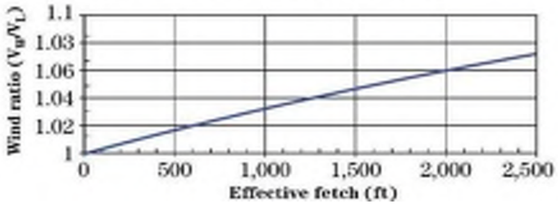
V_L (mph) =	85
V_W/V_L =	1.019
V_W (mph) =	**86.6**

Figure 5 Wind ratio

(a) Wind relationship overwater to overland (miles)



(b) Wind relationship overwater to overland (feet)



Step 4: Determine the significant wave height (freeboard).

$$H_S = 0.0232 V_W^{1.06} F_e^{0.47}$$

V_W (mph) =	86.6
F_e (miles) =	0.1
H_S (feet) =	**0.9**

AECOM

DESIGN: AJW
CHECK: JCG
DATE: 13-Sep-2024
PROJECT: Lake Erin Dam
DESCRIPTION: Wave Height Analysis

Freeboard Calculations (per *A Guide for Design and Layout of Vegetated Wave Protection for Earthen Embankments and Shorelines, Technical Release 56 (NRCS, 2014)*)

Step 1: Determine Effective Fetch Length (maximum of three trial locations).

Trial Location 3

Angle to Normal (degrees)	Cosine of Angle	Length (feet)	Length* Cosine (feet)
0	1.000	929.7	930
6	0.995	513.6	511
6	0.995	839.0	835
12	0.978	471.9	461
12	0.978	643.3	629
18	0.951	351.6	334
18	0.951	601.0	572
24	0.914	260.6	238
24	0.914	511.9	468
30	0.866	232.1	201
30	0.866	453.5	393
36	0.809	199.7	162
36	0.809	355.5	288
42	0.743	214.9	160
42	0.743	316.4	235
	13.512		6417
Effective Fetch Length (feet)			**475**

Step 2: Estimate 50-year recurrence overland wind velocity from NRCS TR-56 (Fig. 4).

V_L (mph) =	85

Figure 4 Maximum wind velocity over land (mph), 50-year recurrence

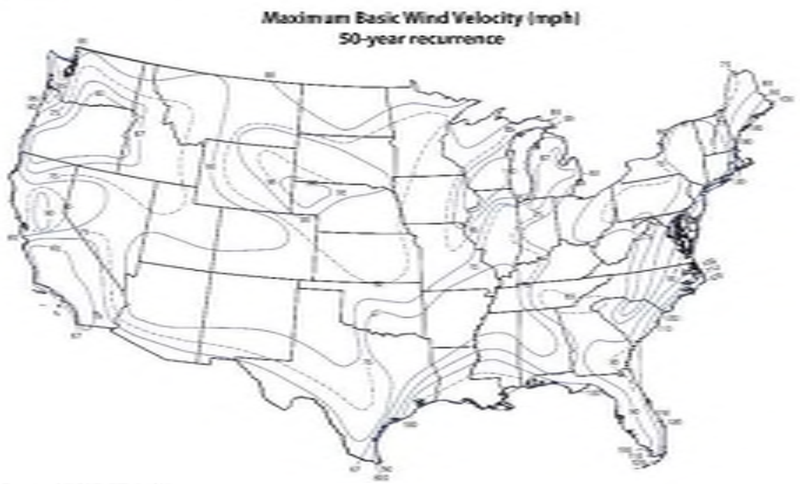
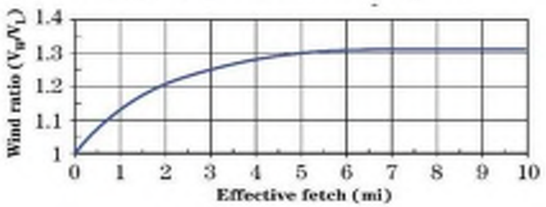
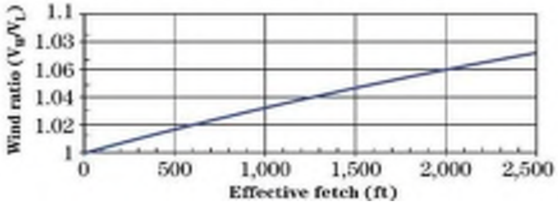


Figure 5 Wind ratio

(a) Wind relationship overwater to overland (miles)



(b) Wind relationship overwater to overland (feet)



Step 3: Determine the over water wind velocity from NRCS TR-56 (Fig. 5).

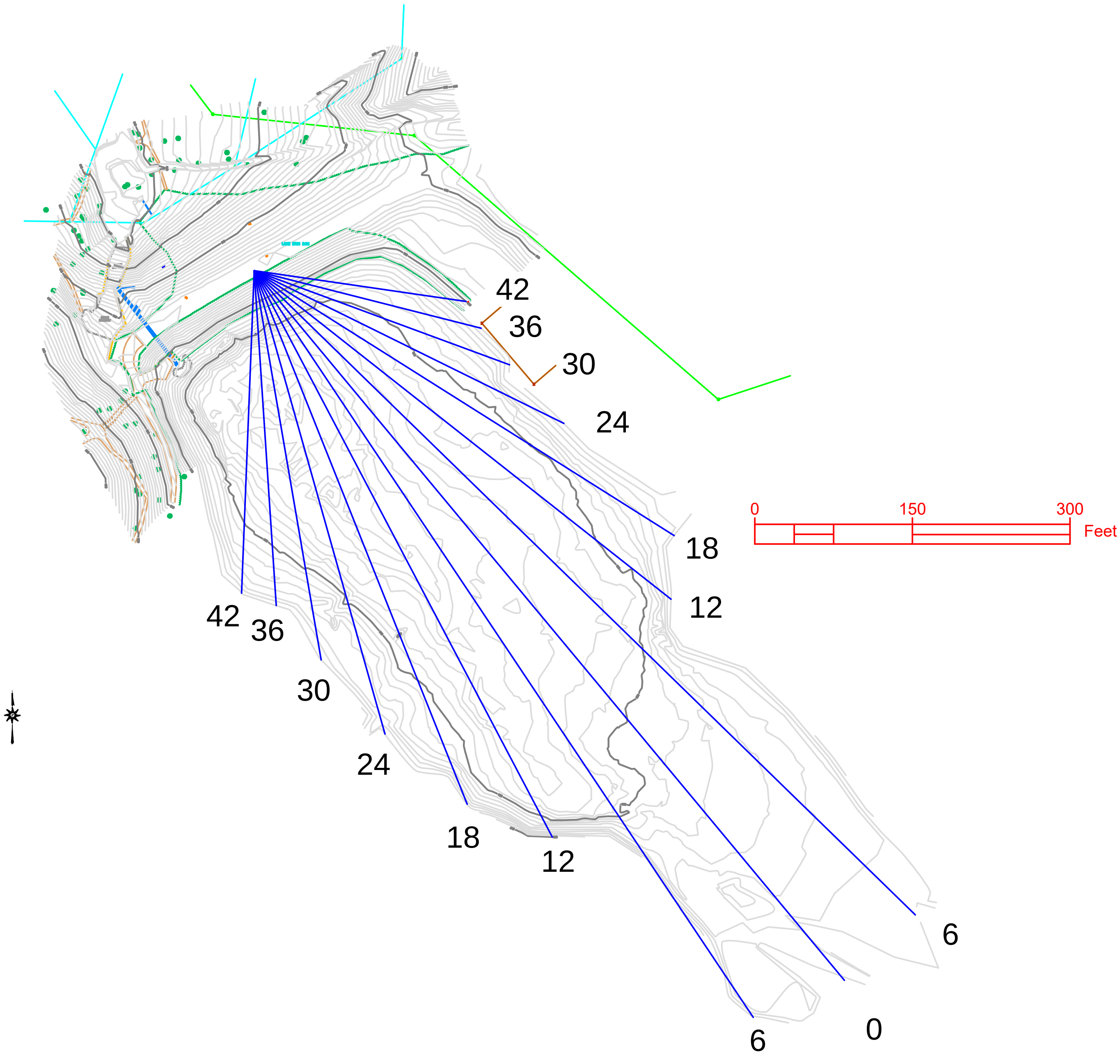
V_L (mph) =	85
V_W/V_L =	1.019
V_W (mph) =	**86.6**

Step 4: Determine the significant wave height (freeboard).

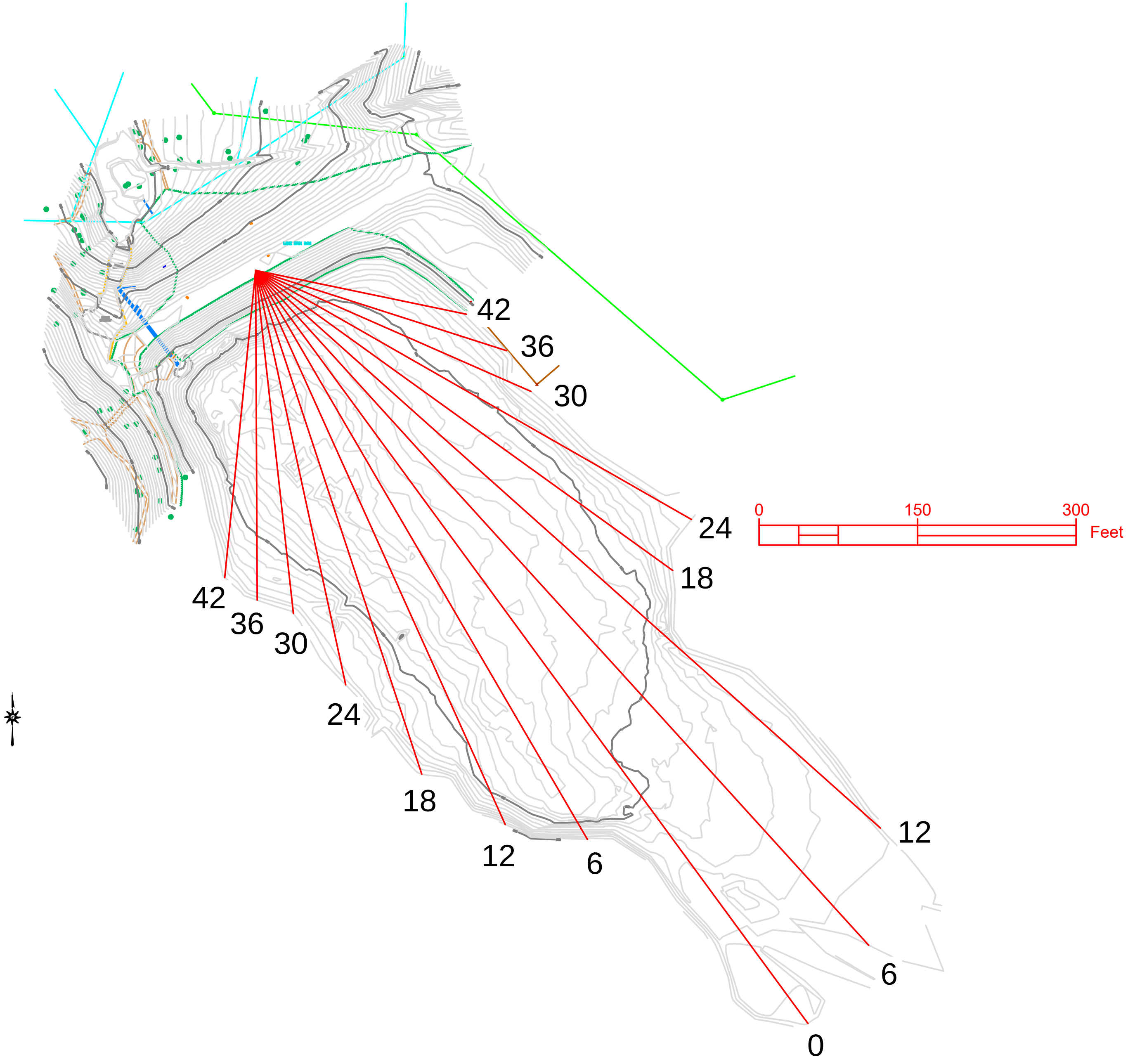
$$H_S = 0.0232V_W^{1.06}F_e^{0.47}$$

V_W (mph) =	86.6
F_e (miles) =	0.09
H_S (feet) =	**0.8**

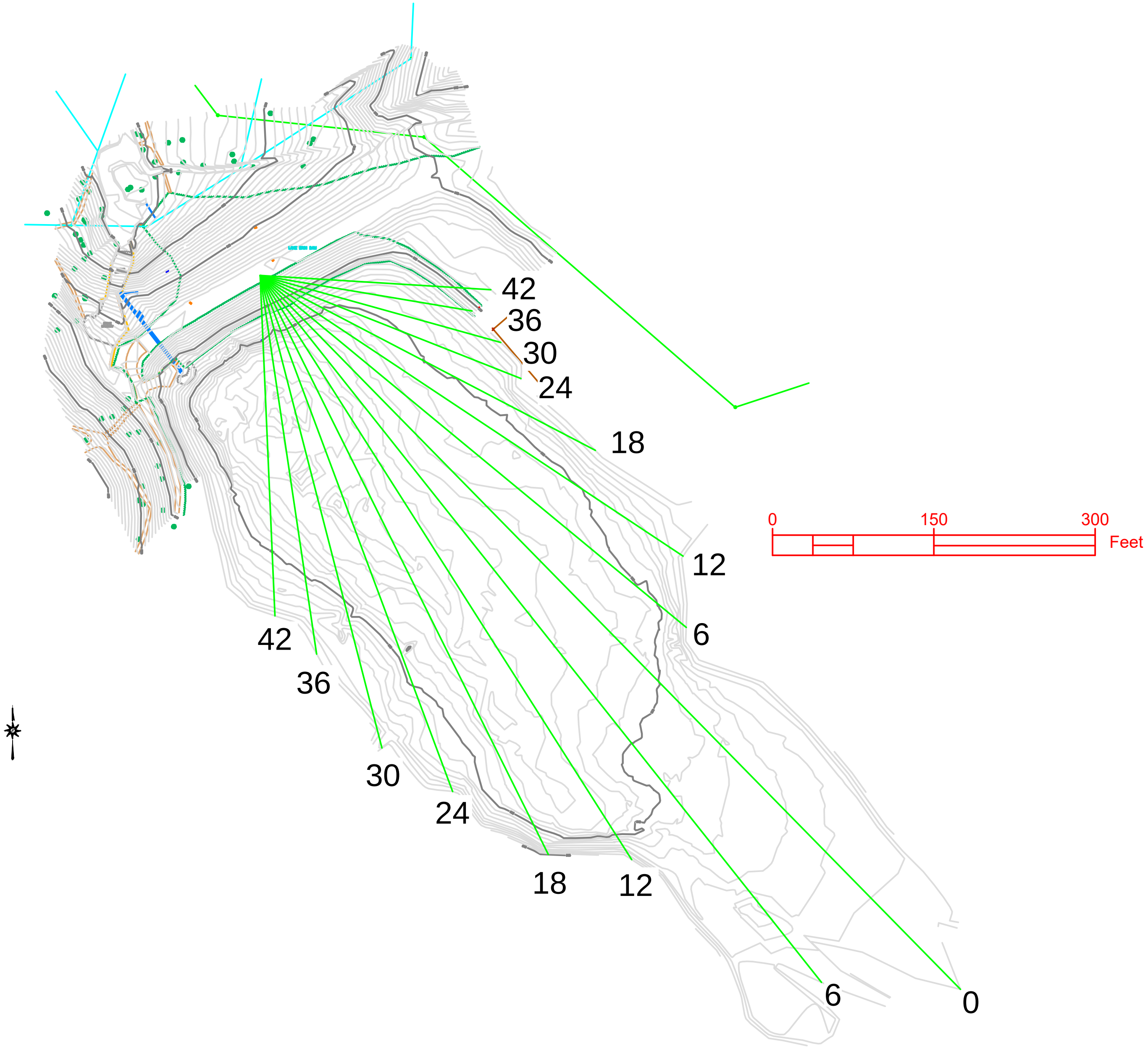
Fetch Trial 1



Fetch Trial 2



Fetch Trial 3



AECOM

DESIGN: AJW
CHECK: JCG
DATE: 13-Sep-2024
PROJECT: Lake Erin Dam
DESCRIPTION: Wave Height Analysis

Freeboard Calculations (per *A Guide for Design and Layout of Vegetated Wave Protection for Earthen Embankments and Shorelines, Technical Release 56 (NRCS, 2014)*)

Determination of wave protection limits

Step 1: Determine Lower Elevation of Protection from NRCS TR-56 (eq. 4).

$Elev_L$	955.7	ft
H_s	0.9	ft

Lower limit of wave impact = $Elev_L$ - (1.5 x H_s)   (eq. 4)

where:

$Elev_L$	is the normal water elevation for the lowest ungated opening
H_s	is the signficant wave height (ft)

Lower limit of wave impact 954.4 ft   Round down 2 whole feet to account for drought conditions

952.0 ft

Step 2: Determine Wave Runup and Wave Setup from NRCS TR-56 (eq. 6 , eq 8., eq. 9, and eq. 10)).

Vw	86.6	mph
F	0.18	miles
D	6.0	ft

S = [(Vw^2) x F] / [1400 x D]   (eq. 6)

where:

S	is the wave setup (ft)
Vw	is the over water wind velocity (mph)
F	maximum fetch distance along the wind direction (miles)
D	average water depth along the fetch line (ft)

S 0.2 ft

Vw	86.6	mph
Fe	0.10	miles

L = 1.24 x Vw^(0.88) x Fe^(0.56)   (eq. 10)

where:

L	is the deep water wave length (ft)
Vw	is the over water wind velocity (mph)
Fe	effective fetch (miles)

L 17.3 ft

Θ	18.43	degrees
Hs	0.9	ft
L	17.3	ft

δ = tan(Θ) / sqrt (Hs / L)   (eq. 9)

where:

δ	is the surf parameter defined from equation 0 from EM 1110-2-1100
Θ	is the angle of the embankment slope to the horizontal
Hs	is the signficant wave height (ft)
L	is the deep water wave length (ft)

δ 1.5

Hs	0.9	ft
A	1.286	
B	0.247	
δ	1.5	

R = Hs x ((A x δ) / (1 + B x δ)) x 1.7   (eq. 8)

where:

R	is the wave runup (ft)
Hs	is the signficant wave height (ft)
A	is a constant
B	is a constant
δ	is the surf parameter defined from equation 0 from EM 1110-2-1100

R 2.1 ft

Step 3: Determine Upper Elevation of Protection from NRCS TR-56 (eq. 5).

$Elev_{NP}$	955.7
R	2.1
S	0.2

Uppet limit of wave impact = $Elev_{NP}$ + R + S (eq. 5)

where:

$Elev_{NP}$	is the normal pool elevation (ft)
R	is the wave runup (ft)
S	is the wave setup (ft)

Lower limit of wave impact 958.0 ft

C.7 Erosion and Sediment Control Calculations

Table A-2
Atlanta

		Return Period						
		1	2	5	10	25	50	100
	n	0.75129	0.8542	0.7846	0.7768	0.7475	0.7519	0.7378
	a	35.11	66.20	62.28	69.74	72.79	83.83	87.36
	b	7	12	12	13	13	14	14
Hours	Minutes	Rainfall Intensity						
0.08	5	5.43	5.89	6.74	7.38	8.39	9.16	9.95
	6	5.11	5.61	6.45	7.08	8.06	8.81	9.58
	7	4.84	5.35	6.18	6.80	7.75	8.50	9.24
	8	4.59	5.12	5.94	6.55	7.48	8.20	8.93
	9	4.37	4.91	5.71	6.32	7.22	7.93	8.64
	10	4.18	4.72	5.51	6.10	6.99	7.68	8.38
	11	4.00	4.55	5.32	5.91	6.77	7.45	8.13
	12	3.84	4.38	5.15	5.72	6.56	7.24	7.90
	13	3.70	4.23	4.98	5.55	6.37	7.03	7.68
	14	3.57	4.09	4.83	5.39	6.20	6.84	7.48
0.25	15	3.44	3.96	4.69	5.24	6.03	6.67	7.28
	16	3.33	3.84	4.56	5.10	5.87	6.50	7.10
	17	3.23	3.73	4.44	4.97	5.73	6.34	6.93
	18	3.13	3.62	4.32	4.84	5.59	6.19	6.77
	19	3.04	3.52	4.21	4.72	5.46	6.05	6.62
	20	2.95	3.43	4.11	4.61	5.33	5.91	6.48
	21	2.87	3.34	4.01	4.51	5.22	5.79	6.34
	22	2.80	3.26	3.92	4.41	5.10	5.67	6.21
	23	2.73	3.18	3.83	4.31	5.00	5.55	6.09
	24	2.66	3.10	3.74	4.22	4.90	5.44	5.97
	25	2.60	3.03	3.66	4.13	4.80	5.33	5.85
	26	2.54	2.96	3.59	4.05	4.71	5.23	5.75
	27	2.48	2.90	3.52	3.97	4.62	5.14	5.64
	28	2.43	2.83	3.45	3.90	4.53	5.05	5.54
	29	2.38	2.77	3.38	3.82	4.45	4.96	5.45
0.50	30	2.33	2.72	3.32	3.75	4.38	4.87	5.36
	31	2.28	2.66	3.26	3.69	4.30	4.79	5.27
	32	2.24	2.61	3.20	3.62	4.23	4.71	5.18
	33	2.20	2.56	3.14	3.56	4.16	4.64	5.10
	34	2.16	2.52	3.09	3.50	4.09	4.56	5.02
	35	2.12	2.47	3.04	3.45	4.03	4.49	4.95
	36	2.08	2.43	2.99	3.39	3.97	4.43	4.87
	37	2.05	2.38	2.94	3.34	3.91	4.36	4.80
	38	2.01	2.34	2.89	3.29	3.85	4.30	4.73
	39	1.98	2.30	2.85	3.24	3.80	4.24	4.67
	40	1.95	2.27	2.81	3.19	3.74	4.18	4.60
	41	1.92	2.23	2.76	3.15	3.69	4.12	4.54
	42	1.89	2.19	2.72	3.10	3.64	4.06	4.48
	43	1.86	2.16	2.68	3.06	3.59	4.01	4.42
	44	1.83	2.13	2.65	3.02	3.54	3.96	4.37
0.75	45	1.80	2.09	2.61	2.98	3.50	3.91	4.31
	46	1.78	2.06	2.58	2.94	3.45	3.86	4.26
	47	1.75	2.03	2.54	2.90	3.41	3.81	4.21
	48	1.73	2.00	2.51	2.86	3.37	3.76	4.16
	49	1.71	1.98	2.48	2.83	3.33	3.72	4.11
	50	1.68	1.95	2.44	2.79	3.29	3.68	4.06
	51	1.66	1.92	2.41	2.76	3.25	3.63	4.02
	52	1.64	1.90	2.38	2.72	3.21	3.59	3.97
	53	1.62	1.87	2.35	2.69	3.18	3.55	3.93
	54	1.60	1.85	2.33	2.66	3.14	3.51	3.88
	55	1.58	1.82	2.30	2.63	3.11	3.47	3.84
	56	1.56	1.80	2.27	2.60	3.07	3.44	3.80
	57	1.54	1.78	2.25	2.57	3.04	3.40	3.76
	58	1.53	1.76	2.22	2.54	3.01	3.36	3.72
	59	1.51	1.74	2.20	2.52	2.98	3.33	3.69
1	60	1.49	1.72	2.17	2.49	2.95	3.30	3.65
2	120	0.96	1.14	1.40	1.58	1.84	2.02	2.21
3	180	0.68	0.81	1.01	1.14	1.32	1.46	1.61
6	360	0.39	0.48	0.60	0.69	0.80	0.90	0.97
12	720	0.23	0.28	0.36	0.41	0.47	0.53	0.58
24	1440	0.14	0.17	0.20	0.23	0.27	0.30	0.33

AECOM

Lake Erin Dam
Tucker, GA

DESIGN	JCG	DATE	10-Jun-2024
CHECK	JBB	DATE	10-Jun-2024
PROJECT:	Lake Erin Dam Rehabilitation		

EROSION AND SEDIMENT CONTROL - TEMPORARY BYPASS CONDUIT RATIONAL METHOD COMPUTATIONS

ID	Incremental Drainage Area (acres)	Total Drainage Area (acres)	"C" Factor (1)	Frequency Factor C_f (2)	Max. Modified Runoff Coefficient (3)	C X A (4)	Time of Concentration (min) (5)	Rainfall Intensity (I) (in/hr) (6)	Discharge (Q) (cfs) (7)	Pipe Diameter (in)	Pipe Slope (%)	Manning's "n" (8)	Partial Flow Pipe Velocity (fps)	Full Flow Capacity (cfs)	Pipe Length (ft)	Travel Time in Pipe (min)	Remarks
CWD-18	1.23	1.23	0.95	1.25	1.00	1.23	5.00	9.95	12.24	18	8.59	0.013	16.21	30.79	69.85	0.07	HDPE-S

Assumptions:

Maintains conservative approach assuming worst case scenario conditions:

(1) "C" Factor - Assumed Description of Area as Asphalt and Concrete or Drives, Walks, and Roofs to generate maximum runoff coefficient and produce a maximum discharge for the site. "C" values obtained from Table 2.1.4-2 "Recommended Runoff Coefficient Values" of the SWM Manual.

(2) "C_f" Frequency Factor - Utilized a frequency factor for the 100-year design storm event to generate a maximum adjusted runoff coefficient and produce a maximum discharge for the site. "C_f" values obtained from Table 2.1.4-1 "Frequency Factors for Rational Formula" of the SWM Manual.

(3) "Maximum Modified Runoff Coefficient - Per Section 2.1.4.3 of the SWM Manual, the product of C_f times C shall not exceed 1.0.

(4) C X A = Product of the maximum modified runoff coefficient and the Total Drainage Area to the system.

(5) Time of Concentration (TC) = Assumes 5 minutes or (0.8 hours) for the Time of Concentration to produce a maximum discharge for the site.

(6) Rainfall Intensity (I) - Rainfall intensity for the 100-year design storm event obtained from Appendix A Table A-2 (Atlanta Region) of the SWM Manual.

(7) Discharge (Q) - Discharge calculated using Equation 2.1.3 where Q = CfCIA per section 2.1.4.3 of the SWM Manual.

(8) Manning's "n" Value - Obtained from Table 7.4 Average Manning's n Values for Storm Sewer Pipes per section 7.3.1 of the Drainage Manual.

References:

Georgia Stormwater Management Manual Volume 2 / Technical Handbook, First Edition, August 2001 (SWM Manual)
Georgia Department of Transportation Drainage Design for Highways, Rev. 3.6, March 2023 (Drainage Manual)

Equations and Tables:

Equation 2.1.2 of the SWM Manual - $Q = CIA$
Equation 2.1.3 - of the SWM Manual $Q = C_f CIA$

where:
Q = Maximum rate of runoff (cfs)
C = Runoff coefficient representing a ratio of runoff to rainfall
I = Average rainfall intensity for a duration equal to the TC (in/hr)
A = Drainage area contributing to the design location (acres)
Cf = Frequency Factor for less frequent, higher intensity storms (greater than 10-year design storm event)

Table 2.1.4-1 Frequency Factors for Rational Formula

Recurrence Interval (Years)	C_f
10 or less	*1*
25	*1.1*
50	*1.2*
100	*1.25*

Full Flow: Single 48 Inch HDPE-S Bypass Pipe

Project Description

Friction Method	Manning Formula
Solve For	Full Flow Capacity

Input Data

Roughness Coefficient	0.013	
Channel Slope	0.0115	ft/ft
Normal Depth	4.00	ft
Diameter	4.00	ft
Discharge	154.03	cfs

Results

Discharge	154.03	cfs
Normal Depth	4.00	ft
Flow Area	12.57	ft²
Wetted Perimeter	12.57	ft
Hydraulic Radius	1.00	ft
Top Width	0.00	ft
Critical Depth	3.63	ft
Percent Full	100.0	%
Critical Slope	0.0101	ft/ft
Velocity	12.26	ft/s
Velocity Head	2.33	ft
Specific Energy	6.33	ft
Froude Number	(N/A)	
Maximum Discharge	165.69	cfs
Discharge Full	154.03	cfs
Slope Full	0.0115	ft/ft
Flow Type	Undefined	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description	N/A	
Profile Headloss	0.00	ft
Average End Depth Over Rise	0.0	%
Normal Depth Over Rise	100.0	%
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	4.00	ft
Critical Depth	3.63	ft
Channel Slope	0.0115	ft/ft
Critical Slope	0.0101	ft/ft

Full Flow: Single 18 Inch HDPE-S Bypass Pipe

Project Description

Friction Method	Manning Formula
Solve For	Full Flow Capacity

Input Data

Roughness Coefficient	0.013
Channel Slope	0.0859 ft/ft
Normal Depth	1.50 ft
Diameter	1.50 ft
Discharge	30.79 cfs

Results

Discharge	30.79 cfs
Normal Depth	1.50 ft
Flow Area	1.77 ft²
Wetted Perimeter	4.71 ft
Hydraulic Radius	0.38 ft
Top Width	0.00 ft
Critical Depth	1.49 ft
Percent Full	100.0 %
Critical Slope	0.0815 ft/ft
Velocity	17.42 ft/s
Velocity Head	4.72 ft
Specific Energy	6.22 ft
Froude Number	(N/A)
Maximum Discharge	33.12 cfs
Discharge Full	30.79 cfs
Slope Full	0.0859 ft/ft
Flow Type	Undefined

GVF Input Data

Downstream Depth	0.00 ft
Length	0.00 ft
Number Of Steps	0

GVF Output Data

Upstream Depth	0.00 ft
Profile Description	N/A
Profile Headloss	0.00 ft
Average End Depth Over Rise	0.0 %
Normal Depth Over Rise	100.0 %
Downstream Velocity	Infinity ft/s
Upstream Velocity	Infinity ft/s
Normal Depth	1.50 ft
Critical Depth	1.49 ft
Channel Slope	0.0859 ft/ft
Critical Slope	0.0815 ft/ft

100-YR: Single 18 Inch HDPE-S Bypass Pipe

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.013
Channel Slope	0.0859 ft/ft
Diameter	1.50 ft
Discharge	12.24 cfs

Results

Normal Depth	0.66 ft
Flow Area	0.75 ft²
Wetted Perimeter	2.17 ft
Hydraulic Radius	0.34 ft
Top Width	1.49 ft
Critical Depth	1.32 ft
Percent Full	43.8 %
Critical Slope	0.0122 ft/ft
Velocity	16.42 ft/s
Velocity Head	4.19 ft
Specific Energy	4.85 ft
Froude Number	4.091
Maximum Discharge	33.12 cfs
Discharge Full	30.79 cfs
Slope Full	0.0136 ft/ft
Flow Type	Supercritical

GVF Input Data

Downstream Depth	0.00 ft
Length	0.00 ft
Number Of Steps	0

GVF Output Data

Upstream Depth	0.00 ft
Profile Description	N/A
Profile Headloss	0.00 ft
Average End Depth Over Rise	0.0 %
Normal Depth Over Rise	43.8 %
Downstream Velocity	Infinity ft/s
Upstream Velocity	Infinity ft/s
Normal Depth	0.66 ft
Critical Depth	1.32 ft
Channel Slope	0.0859 ft/ft
Critical Slope	0.0122 ft/ft

100-YR: Single 48 Inch HDPE-S Bypass Pipe

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.013	
Channel Slope	0.0115	ft/ft
Diameter	4.00	ft
Discharge	140.70	cfs

Results

Normal Depth	3.00	ft
Flow Area	10.13	ft²
Wetted Perimeter	8.39	ft
Hydraulic Radius	1.21	ft
Top Width	3.46	ft
Critical Depth	3.52	ft
Percent Full	75.1	%
Critical Slope	0.0086	ft/ft
Velocity	13.90	ft/s
Velocity Head	3.00	ft
Specific Energy	6.01	ft
Froude Number	1.432	
Maximum Discharge	165.69	cfs
Discharge Full	154.03	cfs
Slope Full	0.0096	ft/ft
Flow Type	Supercritical	

GVF Input Data

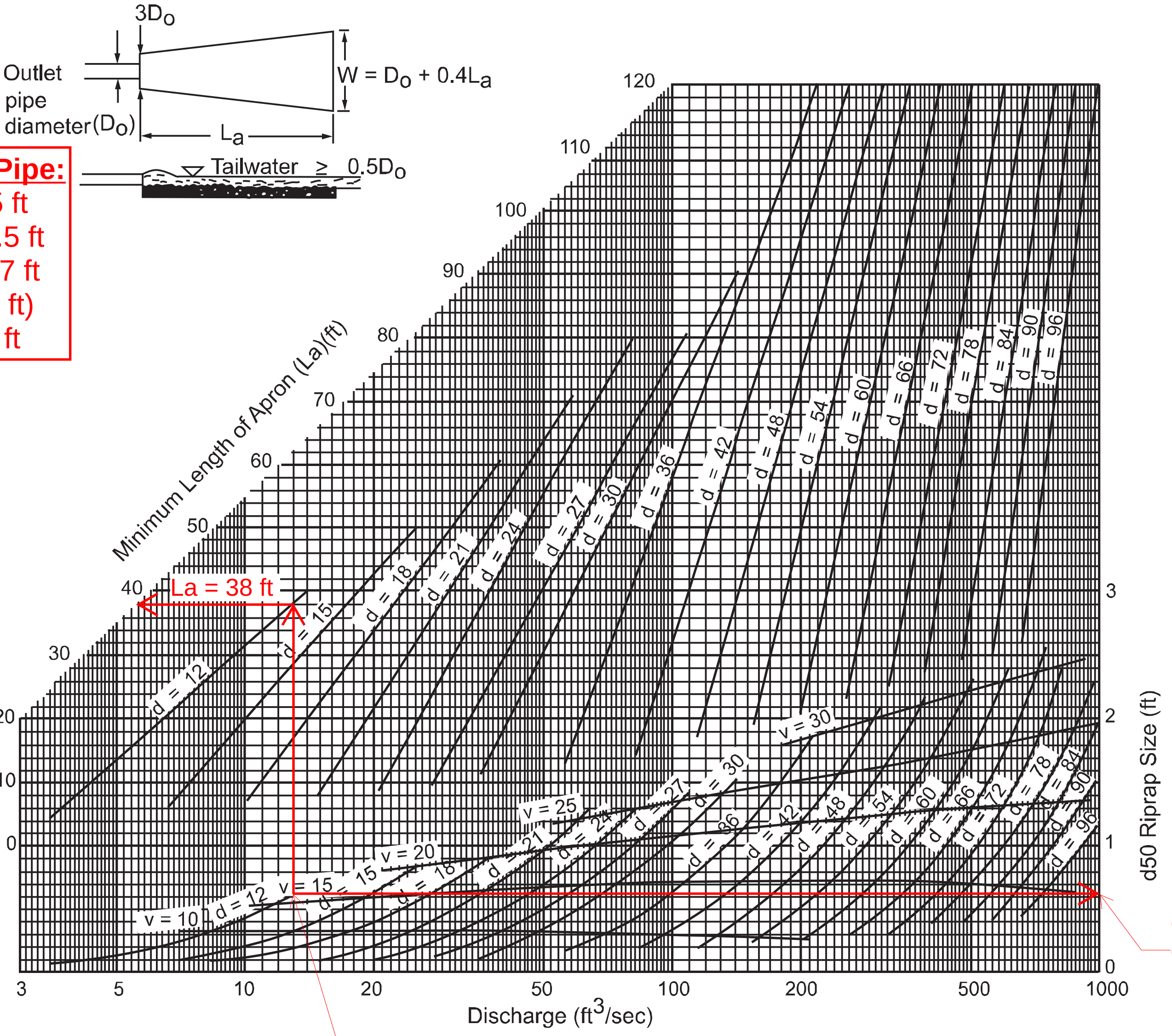
Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description	N/A	
Profile Headloss	0.00	ft
Average End Depth Over Rise	0.0	%
Normal Depth Over Rise	75.1	%
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	3.00	ft
Critical Depth	3.52	ft
Channel Slope	0.0115	ft/ft
Critical Slope	0.0086	ft/ft

18-Inch HDPE-S Clearwater Bypass Pipe [CP-1 (1)]

Single Pipe:
D_o = 1.5 ft
$3D_o$ = 4.5 ft
W = 16.7 ft
(Use 17 ft)
La = 38 ft

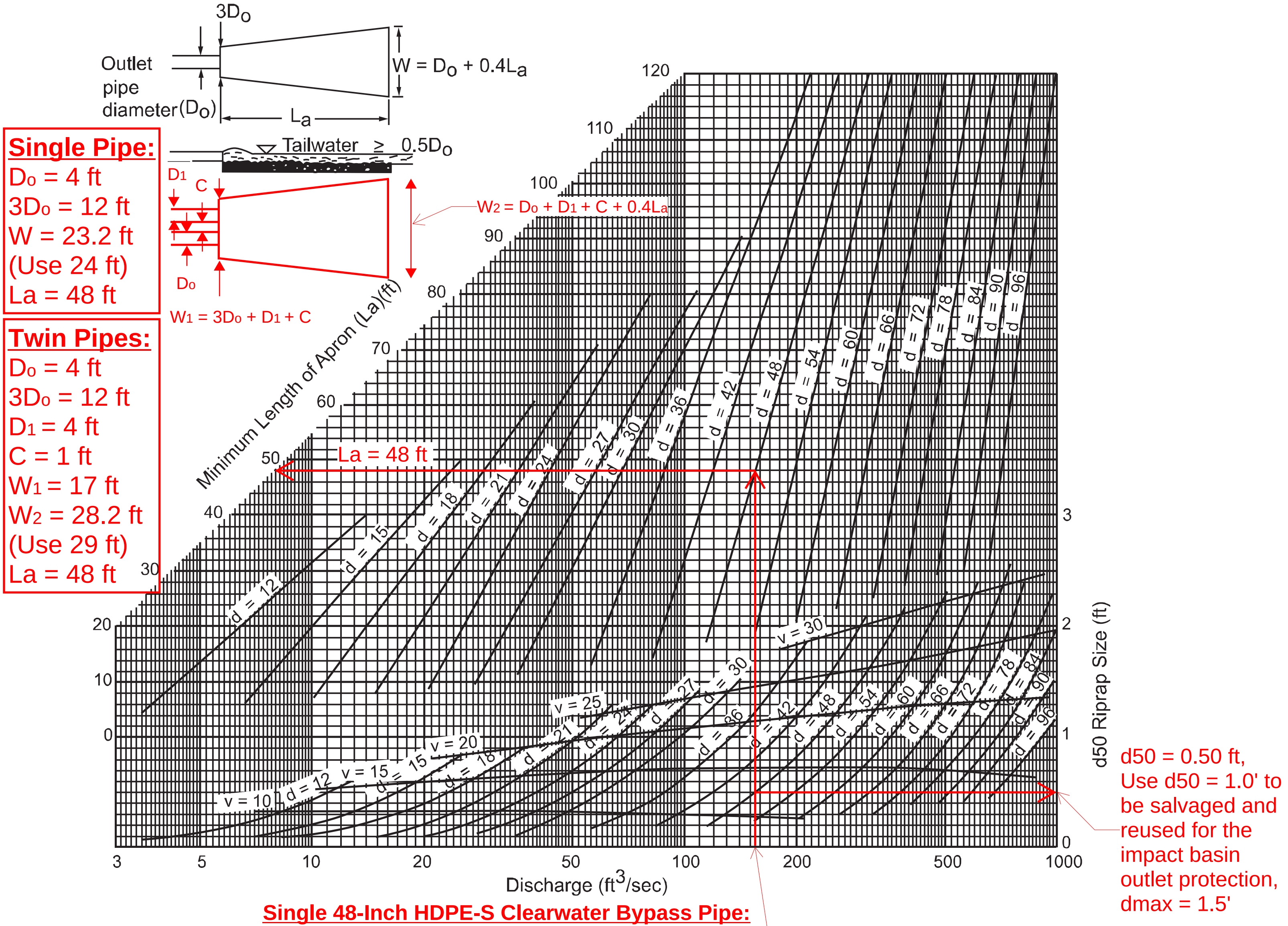


d50 = 0.65 ft
dmax = 0.98 ft
(say 1 ft)

Single 18-Inch HDPE-S Clearwater Bypass Pipe:
Full Flow Discharge = 30.79 cfs
Full Flow Velocity = 17.42 ft/s
100-Year Partial Flow Discharge = 12.24 cfs
(See Rational Method Computations)
Partial Flow Velocity = 16.42 ft/s
Partial Flow Normal Depth = 0.66 ft (say d=12)

Figure 6-34.2 - Design of Outlet Protection From a Round Pipe Flowing Full, Maximum Tailwater Condition (Tw > 0.5 Diameter)

48-Inch HDPE-S Clearwater Bypass Pipes [CP-1-(2)]



Single Pipe:
Do = 4 ft
3Do = 12 ft
W = 23.2 ft
(Use 24 ft)
La = 48 ft

Twin Pipes:
Do = 4 ft
3Do = 12 ft
D1 = 4 ft
C = 1 ft
W1 = 17 ft
W2 = 28.2 ft
(Use 29 ft)
La = 48 ft

d50 = 0.50 ft,
Use d50 = 1.0' to
be salvaged and
reused for the
impact basin
outlet protection,
dmax = 1.5'

Single 48-Inch HDPE-S Clearwater Bypass Pipe:
Full Flow Discharge = 154.03 cfs
Full Flow Velocity = 12.26 ft/s
100-YR Partial Flow Discharge (1) Conduit = 140.70 cfs
100-YR Partial Flow Discharge (2) Conduits = 281.40 cfs
100-YR Velocity = 11.20 ft/s
100-YR Normal Depth = 3.00 ft

Figure 6-34.2 - Design of Outlet Protection From a Round Pipe Flowing Full, Maximum Tailwater Condition (Tw > 0.5 Diameter)

Appendix D - Structural Calculations

Lake Erin Dam
Rehabilitation

Structural Calculations
Table of Contents

September 13, 2024

Prepared by:

AECOM – Germantown
12420 Milestone Center Drive
Germantown, MD 20876

INTAKE TOWER STABILITY ANALYSIS

12420 Milestone Ctr Drive
Germantown, MD
Telephone: (301) 820-3000
www.aecom.com

Calculated By: NRP
Date: 3/19/2024
Checked By: IML
Date: 5/3/2024
Project Number: 60727041

Project: Lake Erin Dam Rehabilitation

Task: Intake Tower Design - Stability Analysis

Description:

Check the stability of the reinforced concrete Intake Tower according to USACE EM 1110-2-2400, Structural Design and Evaluation of Outlet Works and USACE EM 1110-2-2100 Stability and Analysis of Concrete Structures.

Codes, Standards, & References:

1. USACE EM 1110-2-2400, Structural Design and Evaluation of Outlet Works
2. USACE EM 1110-2-2100, Stability and Analysis of Concrete Structures
3. ASCE 7-22, Minimum Design Loads and Associated Criteria for Buildings and Other Structures
4. AECOM Basis of Design Report

Hydrologic and Hydraulic:

Normal water level elevation is assumed to be at EL 955.7 feet and flood water elevation is assumed to be at EL 968.9 ft.

Geotechnical/Subsurface Investigation:

Geotechnical parameters were based on the subsurface investigation. The structure is conservatively assumed to be founded on soil, although weathered rock may be near the elevation of the foundation slab.

Assumptions:

- Wave load is assumed to be negligible for all load combinations.
- Wind load is calculated based on ASCE 7-22 (Chapters 26 and 27) and is assumed to apply for only the Construction Load Combination. A separate spreadsheet was used to calculate the wind loading (attached).
- Seismic load was calculated based on the Two-Mode Approximate Method (Appendix C of EM 1110-2-2400). A separate spreadsheet is attached to illustrate the calculations.
- Live load assumed to be 250 psf on top slab (conservatively excluded for flotation check).
- The point of rotation is calculated against the upstream toe.

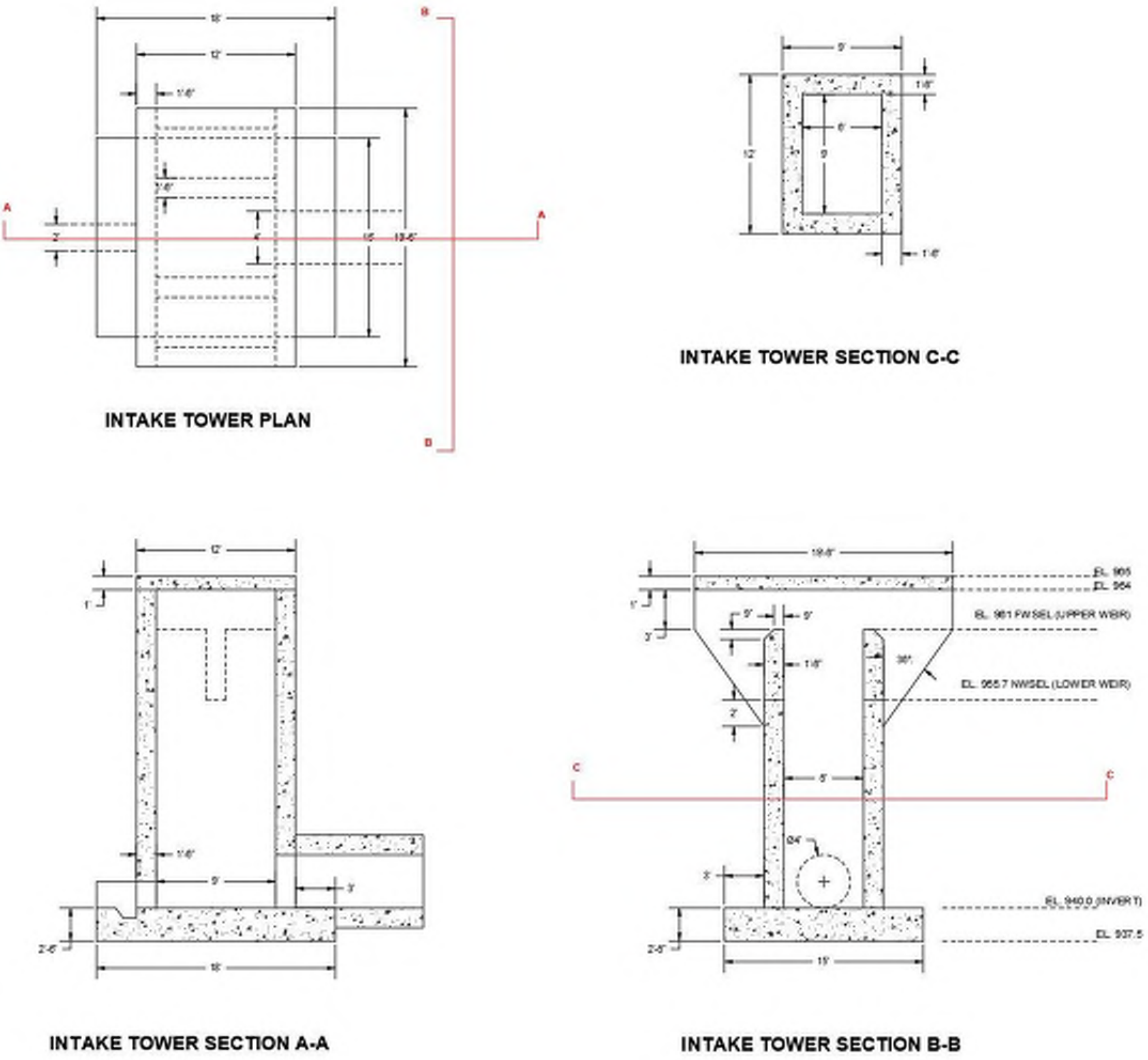


Figure 1 - Section, Plan and Elevation Views

Material Properties

$\gamma_c := 150\ \text{pcf}$	Unit Weight of Concrete
$\gamma_w := 62.4\ \text{pcf}$	Unit Weight of Water
$f'_c := 5000\ \text{psi}$	Concrete Compressive Strength
$\gamma_{moist\_fill} := 123\ \text{pcf}$	Moist Unit Weight of Backfill
$\gamma_{sat\_fill} := 128\ \text{pcf}$	Saturated Unit Weight of Backfill
$\sigma_{allow} := 3500\ \text{psf}$	Allowable Foundation Bearing Capacity
$\phi_f := 31\ \text{deg}$	Internal Friction Angle of Foundation
$\phi_{fill} := 31\ \text{deg}$	Internal Friction Angle of Backfill
$\mu_f := \tan\left(\phi_f\right) = 0.601$	Friction Coefficient for Sliding Concrete/Foundation Interface
$c := 0\ \text{psf}$	Conservatively, Neglect Cohesion at Concrete/Foundation Interface
$F_{live} := 250\ \text{psf}$	Live Load on Top Slab

Lateral Soil Coefficients:

$K_o := 1 - \sin\left(\phi_{fill}\right) = 0.485$	At-Rest Earth Pressure Coefficient (Rankine)
$K_a := \tan\left(45\ \text{deg} - \left(\dfrac{\phi_{fill}}{2}\right)\right)^2 = 0.32$	Active Earth Pressure Coefficient (Rankine)
$K_p := \tan\left(45\ \text{deg} + \left(\dfrac{\phi_{fill}}{2}\right)\right)^2 = 3.124$	Passive Earth Pressure Coefficient (Rankine)

Seismic Soil Coefficients:

$k_h := 0.12$	Peak Horizontal Ground Acceleration in G's (USGS Hazard Tool)
$k_v := 0.0$	Vertical Acceleration in G's
$\psi := \operatorname{atan}\left(\dfrac{k_h}{1 - k_v}\right) = 6.843\ \text{deg}$	Seismic Inertia Angle
$\beta := 0\ \text{deg}$	Inclination of Soil Surface
$K_{AE} := \dfrac{\cos\left(\phi_{fill} - \psi\right)^2}{\cos\left(\psi\right)^2 \cdot \left(1 + \sqrt{\dfrac{\sin\left(\phi_{fill}\right) \cdot \sin\left(\phi_{fill} - \psi - \beta\right)}{\cos\left(\beta\right) \cdot \cos\left(\psi\right)}}\right)^2} = 0.4$	Dynamic Active Earth Pressure Coefficient

$$K_{PE} := \frac{\cos\left(\phi_{fill} - \psi\right)^2}{\cos\left(\psi\right)^2 \cdot \left(1 - \sqrt{\frac{\sin\left(\phi_{fill}\right) \cdot \sin\left(\phi_{fill} - \psi + \beta\right)}{\cos\left(\beta\right) \cdot \cos\left(\psi\right)}}\right)^2} = 2.9$$ Dynamic Passive Earth Pressure Coefficient

Intake Tower Details & Geometry

$EL_{invert} := 940.0\ ft$ Elevation of Bottom of Invert

$EL_{foundation} := 937.5\ ft$ Elevation of Bottom of Foundation

$EL_{bot\_slab} := 964.0\ ft$ Elevation of Bottom of Top Slab

$EL_{top\_slab} := 965\ ft$ Elevation of Top of Top Slab

$EL_{bot\_weir} := 955.7\ ft$ Elevation of Lower Weir

$EL_{top\_weir} := 961.0\ ft$ Elevation of Higher Weir

$EL_{ds\_soil} := 950\ ft$ Elevation of Downstream Soil Elevation

$EL_{us\_soil} := 945\ ft$ Elevation of Upstream Soil Elevation

$l_i := 9\ ft$ Length of Tower - Inside

$w_i := 6\ ft$ Width of Tower - Inside

$t_{wall} := 1.5\ ft$ Thickness of Tower Wall

$l_o := l_i + 2 \cdot t_{wall} = 12\ ft$ Length of Tower - Outside

$w_o := w_i + 2 \cdot t_{wall} = 9\ ft$ Width of Tower - Outside

$t_{foundation} := EL_{invert} - EL_{foundation} = 2.5\ ft$ Thickness of Base Foundation

$l_{foundation} := 18.0\ ft$ Length of Base Foundation

$w_{foundation} := 15.0\ ft$ Width of Base Foundation

$l_{top\_slab} := l_o = 12\ ft$ Length of Top Slab

$w_{top\_slab} := 19.5\ ft$ Width of Top Slab

$t_{top\_slab} := EL_{top\_slab} - EL_{bot\_slab} = 1\ ft$ Thickness of Top Slab

$EL_{join\_wall} := EL_{bot\_weir} - 2\ ft = 953.7\ ft$ Elevation of Diagonal Portion of Wall Begins

$EL_{HW\_norm} := EL_{bot\_weir} = 955.7\ ft$ Elevation of Headwater (Normal)

$EL_{HW\_flood} := 968.9\ \text{ft}$	Elevation of Headwater (Flood)
$H_{HW\_norm} := EL_{HW\_norm} - EL_{foundation} = 18.2\ \text{ft}$	Height of Headwater (Normal)
$H_{HW\_flood} := EL_{HW\_flood} - EL_{foundation} = 31.4\ \text{ft}$	Height of Headwater (Flood)
$H_{us\_soil} := EL_{us\_soil} - EL_{foundation} = 7.5\ \text{ft}$	Height of Upstream Soil
$H_{ds\_soil} := EL_{ds\_soil} - EL_{foundation} = 12.5\ \text{ft}$	Height of Downstream Soil
$D_{intake} := 1.25\ \text{ft}$	Diameter of Intake Pipe

Calculate Weight and Section Properties of Tower

$F_{top\_slab} := l_{top\_slab} \cdot w_{top\_slab} \cdot t_{top\_slab} \cdot \gamma_c = 35.1\ \text{kip}$	Weight of Top Slab
$F_{foundation} := l_{foundation} \cdot w_{foundation} \cdot t_{foundation} \cdot \gamma_c = 101.25\ \text{kip}$	Weight of Foundation Slab
$F_{weir\_walls} := \left(l_o \cdot t_{wall}\right) \cdot \left(EL_{top\_weir} - EL_{invert}\right) \cdot \gamma_c \cdot 2 = 113.4\ \text{kip}$	Weight of Weir Walls (Short)
$F_{walls} := \left(\gamma_c \cdot 2 \cdot t_{wall}\right) \cdot \left(\begin{array}{l} \left(EL_{join\_wall} - EL_{invert}\right) \cdot w_o + \left(EL_{top\_weir} - EL_{join\_wall}\right) \cdot \frac{1}{2} \cdot \left(w_{top\_slab} + w_o\right) \hookleftarrow \\ + \left(EL_{bot\_slab} - EL_{top\_weir}\right) \cdot w_{top\_slab} \end{array}\right) = 128.621\ \text{kip}$	Weight of Other Walls (Tall)
$W_S := F_{top\_slab} + F_{foundation} + F_{walls} + F_{weir\_walls} = 378.371\ \text{kip}$	Total Weight of Structure
$x_{ws} := \frac{l_{foundation}}{2} = 9\ \text{ft}$	Centroid of Structure (x)

Loads and Loading Conditions:

Loading Conditions to be analyzed in accordance with EM 1110-2-2400:

Case No. 1: Usual Loading Condition - U1
Normal Pool Elevation = 955.7 feet, Uplift, Dead, Live, Water Surface Inside Structure at Normal Pool, Soil

Case No. 2: Usual Loading Condition - U3
Normal Pool Elevation = 955.7 feet, Uplift, Dead, Live, No Water Inside Structure, Soil

Case No. 3: Unusual Loading Condition (Construction) - UN4
Pool Elevation = 937.5 feet, Wind, Dead

Case No. 4: Extreme Loading Condition (Seismic) - ED1
Load Combination U3 plus Maximum Design Earthquake

Case No. 5: Extreme Loading Condition (Flood) - ED2A
Flood Pool Elevation = 968.9 feet (1/3 PMF), Dead, Uplift, Water Surface Inside Structure to Top Slab, Soil Conservatively Neglected (critical bearing pressure load case)

Case No. 6: Extreme Loading Condition (Flood) - ED2B
Flood Pool Elevation = 968.9 feet (1/3 PMF), Dead, Uplift, No Water Inside Structure, Soil Conservatively Neglected (critical flotation load case)

Case 1 - Usual Load Condition (Normal With Water Inside Structure - U1)

Free Body Diagram:

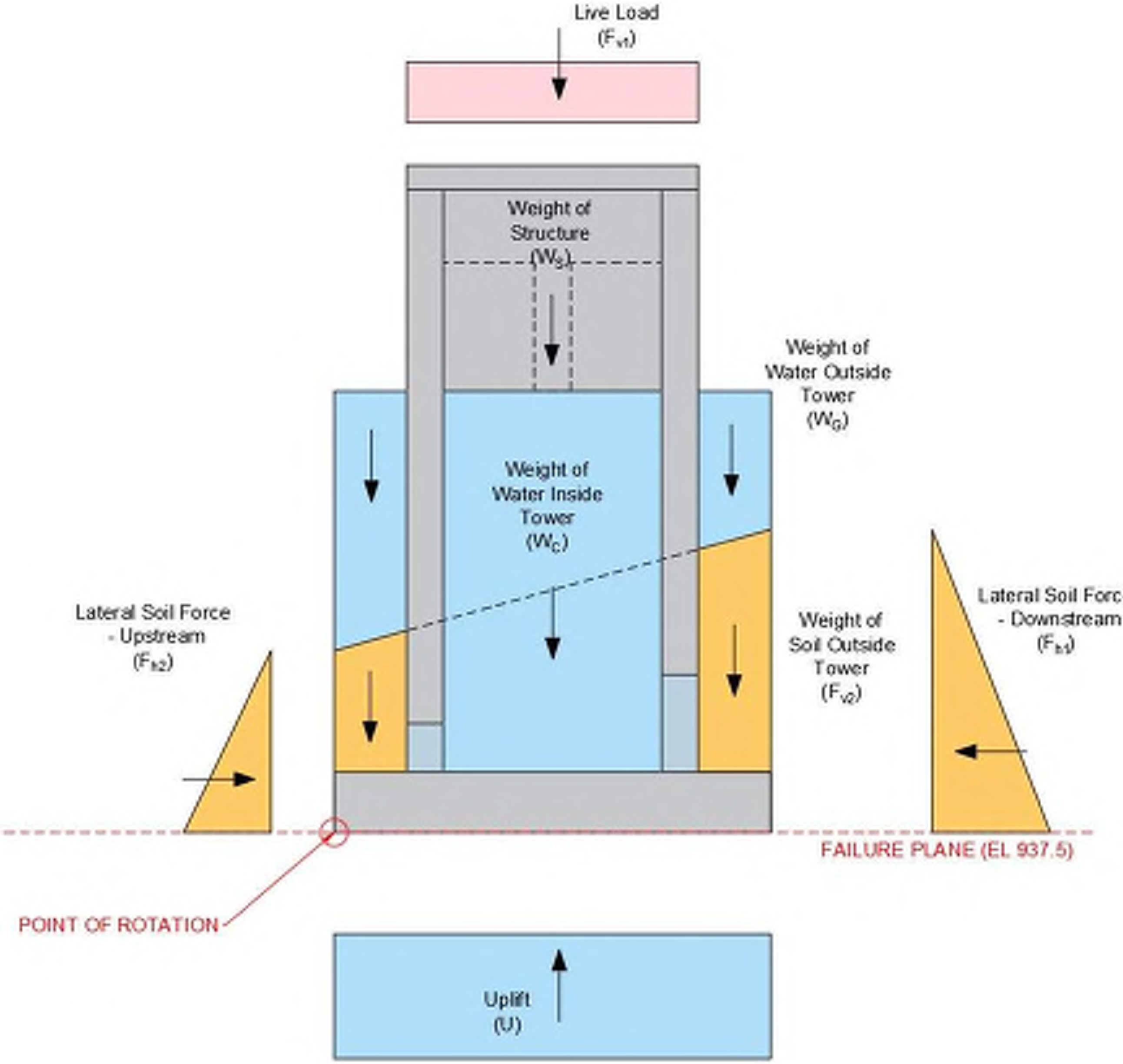


Figure 2: Free Body Diagram - Usual Loading Condition (U1)

Calculate Vertical Loads:

(A) Gravity Load of Tower

$W_S = 378.371\ \text{kip}$ Total Weight of Structure

$x_{ws} = 9\ \text{ft}$ Moment Arm

(B) Gravity Load of Water Inside Tower

$W_C := l_i \cdot w_i \cdot \left(EL_{HW\_norm} - EL_{invert}\right) \cdot \gamma_w = 52.903\ \text{kip}$ Weight of Water Inside Structure

$x_{wc} := \dfrac{l_{foundation}}{2} = 9\ \text{ft}$ Moment Arm

(C) Gravity Load of Water Above Top Surface of the Structure

$W_{G\_1} := 0\ \text{kip}$ Weight of Water Above Top of Structure

$x_{wg\_1} := \dfrac{l_{foundation}}{2} = 9\ \text{ft}$ Moment Arm

$W_{G\_2} := \left(l_{foundation} \cdot w_{foundation} - l_o \cdot w_o\right) \cdot \left(EL_{HW\_norm} - EL_{invert}\right) \cdot \gamma_w = 158.708\ \text{kip}$ Weight of Water Around Tower Foundation

$x_{wg\_2} := \dfrac{l_{foundation}}{2} = 9\ \text{ft}$ Moment Arm

$W_G := W_{G\_1} + W_{G\_2} = 158.708\ \text{kip}$ Total Weight of Water Outside Tower

$x_{wg} := \dfrac{W_{G\_1} \cdot x_{wg\_1} + W_{G\_2} \cdot x_{wg\_2}}{W_G} = 9\ \text{ft}$ Moment Arm

(D) Uplift Force at Concrete/Foundation Interface

$U := H_{HW\_norm} \cdot l_{foundation} \cdot w_{foundation} \cdot \gamma_w = 306.634\ \text{kip}$ Uplift Force Under Tower

$x_u := \dfrac{l_{foundation}}{2} = 9\ \text{ft}$ Moment Arm

(E) Live Load on Top Slab

$F_{v1} := F_{live} \cdot l_{top\_slab} \cdot w_{top\_slab} = 58.5\ \text{kip}$ Live Load on Top Slab

$x_{v1} := \dfrac{l_{foundation}}{2} = 9\ \text{ft}$ Moment Arm

(F) Gravity Load of Soil Around Structure Foundation

$$F_{v2\_1} := (\gamma_{sat\_fill} - \gamma_w) \cdot ((H_{us\_soil} + H_{ds\_soil}) \cdot 0.5 - t_{foundation}) \cdot (w_{foundation} - w_o) \cdot l_{foundation} = 53.136\ \text{kip}$$

Weight of Soil on Left/Right Sides of Tower Foundation

$$x_{v2\_1} := \frac{2 \cdot (H_{ds\_soil} - t_{foundation}) + (H_{us\_soil} - t_{foundation})}{H_{ds\_soil} + H_{us\_soil} - t_{foundation} \cdot 2} \cdot \frac{l_{foundation}}{3} = 10\ \text{ft}$$

Moment Arm

$$F_{v2\_2} := (\gamma_{sat\_fill} - \gamma_w) \cdot (H_{ds\_soil} - t_{foundation}) \cdot (l_{foundation} - l_o) \cdot w_o \cdot 0.5 = 17.712\ \text{kip}$$

Weight of Soil on Downstream Side of Tower Foundation

$$x_{v2\_2} := l_{foundation} - (l_{foundation} - l_o) \cdot 0.5 \cdot 0.5 = 16.5\ \text{ft}$$

Moment Arm

$$F_{v2\_3} := (\gamma_{sat\_fill} - \gamma_w) \cdot (H_{us\_soil} - t_{foundation}) \cdot (l_{foundation} - l_o) \cdot w_o \cdot 0.5 = 8.856\ \text{kip}$$

Weight of Soil on Upstream Side of Tower Foundation

$$x_{v2\_3} := (l_{foundation} - l_o) \cdot 0.5 \cdot 0.5 = 1.5\ \text{ft}$$

Moment Arm

$$F_{v2} := F_{v2\_1} + F_{v2\_2} + F_{v2\_3} = 79.704\ \text{kip}$$

Weight of Soil Around Tower Foundation

$$x_{v2} := \frac{F_{v2\_1} \cdot x_{v2\_1} + F_{v2\_2} \cdot x_{v2\_2} + F_{v2\_3} \cdot x_{v2\_3}}{F_{v2}} = 10.5\ \text{ft}$$

Moment Arm

Calculate Horizontal Loads:

(A) Downstream Lateral Soil Forces

$$F_{h1} := \frac{1}{2} \cdot (\gamma_{sat\_fill} - \gamma_w) \cdot H_{ds\_soil}^{2} \cdot K_o \cdot w_{foundation} = 37.281\ \text{kip}$$

Downstream Lateral Earth Pressure

$$x_{h1} := \frac{H_{ds\_soil}}{3} = 4.167\ \text{ft}$$

Moment Arm

(B) Upstream Lateral Soil Forces

$$F_{h2} := \frac{1}{2} \cdot (\gamma_{sat\_fill} - \gamma_w) \cdot H_{us\_soil}^{2} \cdot K_o \cdot w_{foundation} = 13.421\ \text{kip}$$

Upstream Lateral Earth Pressure

$$x_{h2} := \frac{H_{us\_soil}}{3} = 2.5\ \text{ft}$$

Moment Arm

Overturning Stability Check

$$M_{OT} := U \cdot x_u + F_{h1} \cdot x_{h1} = 2915.042 \ \text{kip} \cdot \text{ft}$$

Total Overturning Moment

$$M_R := W_S \cdot x_{ws} + W_C \cdot x_{wc} + W_G \cdot x_{wg} + F_{v1} \cdot x_{v1} + F_{v2} \cdot x_{v2} + F_{h2} \cdot x_{h2} = 6706.784 \ \text{kip} \cdot \text{ft}$$

Total Restoring Moment

$$FS_{Overturning} := \frac{M_R}{M_{OT}} = 2.301$$

Factor of Safety Against Overturning (Note overturning stability is evaluated based on location of resultant, see below)

Resultant Location:

$$F_N := W_S + W_C + W_G - U + F_{v1} + F_{v2} = 421.553 \ \text{kip}$$

Total Vertical Load Resisting Overturning

$$X := \frac{M_R - M_{OT}}{F_N} = 8.995 \ \text{ft}$$

Center of Total Weight from Edge of Toe

$$e_{cc} := \frac{l_{foundation}}{2} - X = 0.005 \ \text{ft}$$

Eccentricity of the Resultant

$$\frac{l_{foundation}}{6} = 3 \ \text{ft}$$

Limit for Base Being in Compression Only

```
Resultant_Location :=
    if $|e_{cc}| \le \frac{l_{foundation}}{6}$
        “100% of Base in Compression”
    else if $\frac{l_{foundation}}{6} < |e_{cc}| \le \frac{l_{foundation}}{4}$
        “75% of Base in Compression”
    else if $\frac{l_{foundation}}{4} < |e_{cc}| \le \frac{l_{foundation}}{2}$
        “Resultant Within Base”
    else
        “Unstable-Resultant Outside Base”
```

Resultant_Location = “100% of Base in Compression”

The resultant location is within the middle third of the base and thus the base is 100% in compression. The requirement for overturning for Case 1 is the resultant force should be in the middle third of the base (EM 1110-2-2100). Meets the requirements.

Check Stresses at the Base

$$q_{max} := \begin{array}{l} \text{if } |e_{cc}| \le \dfrac{l_{foundation}}{6} \\ \quad \dfrac{F_N}{w_{foundation} \cdot l_{foundation}} \cdot \left(1 + \dfrac{6 \cdot e_{cc}}{l_{foundation}}\right) \\ \text{else} \\ \quad \dfrac{4 \cdot F_N}{3 \cdot w_{foundation} \cdot \left(l_{foundation} - 2 \cdot e_{cc}\right)} \end{array}$$

$q_{max} = 10.862 \text{ psi}$ $q_{max} = 1564 \text{ psf}$

Equations 3-1 and 3-2 from EM 1110-2-2502

$$q_{min} := \begin{array}{l} \text{if } |e_{cc}| \le \dfrac{l_{foundation}}{6} \\ \quad \dfrac{F_N}{w_{foundation} \cdot l_{foundation}} \cdot \left(1 - \dfrac{6 \cdot e_{cc}}{l_{foundation}}\right) \\ \text{else} \\ \quad 0 \text{ psi} \end{array}$$

$q_{min} = 10.823 \text{ psi}$ $q_{min} = 1559 \text{ psf}$

$$Check_{bearing} := \begin{array}{l} \text{if } \sigma_{allow} > q_{max} \\ \quad \text{“PASS”} \\ \text{else} \\ \quad \text{“FAIL”} \end{array}$$

$Check_{bearing} = \text{“PASS”}$

The allowable bearing capacity of the foundation is not exceeded. Additionally, the stresses are significantly less than the assumed strength of concrete and as a result, concrete crushing is not considered to be a viable failure mechanism.

Check Flotation Stability

$$FS_{flotation} := \frac{W_S + W_C + F_{v2}}{U - W_G} = 3.45$$

Factor of Safety for Flotation (Equation 3-2 from EM 1110-2-2100)

$$FS_{usual\_flotation} := 1.3$$

Required Factor of Safety for Flotation (Table 3-4 from EM 1110-2-2100)

$$\begin{aligned} Check_{flotation} := \text{if } & FS_{flotation} \geq FS_{usual\_flotation} \\ & \Vert \text{"PASS"} \\ \text{else} & \\ & \Vert \text{"FAIL"} \end{aligned}$$

$Check_{flotation} = \text{"PASS"}$

The requirements for flotation in a Normal loading condition is a factor of safety of 1.3 (EM 1110-2-2100). The structure passes this condition.

Sliding Stability Check

$$F_N = 421.553 \text{ kip}$$

Total Vertical Loads Resisting Sliding

$$F_D := F_{h1} - F_{h2} = 23.86 \text{ kip}$$

Sum of Sliding Force

$$FS_{sliding} := \frac{c \cdot w_{foundation} \cdot l_{foundation} + F_N \cdot \mu_f}{F_D} = 10.616$$

Sliding Factor of Safety (Cohesion conservatively neglected)

$$FS_{req\_sliding\_usual} := 1.7$$

Minimum Required Sliding Factor of Safety

$$\begin{aligned} Check_{sliding} := \text{if } & FS_{sliding} > FS_{req\_sliding\_usual} \\ & \Vert \text{"PASS"} \\ \text{else} & \\ & \Vert \text{"FAIL"} \end{aligned}$$

$Check_{sliding} = \text{"PASS"}$

The requirements for sliding in an Usual loading condition is a factor of safety of 1.7 (EM 1110-2-2100). The structure passes this condition.

Case 2 - Usual Load Condition (Normal With No Water Inside Structure - U3)

Free Body Diagram:

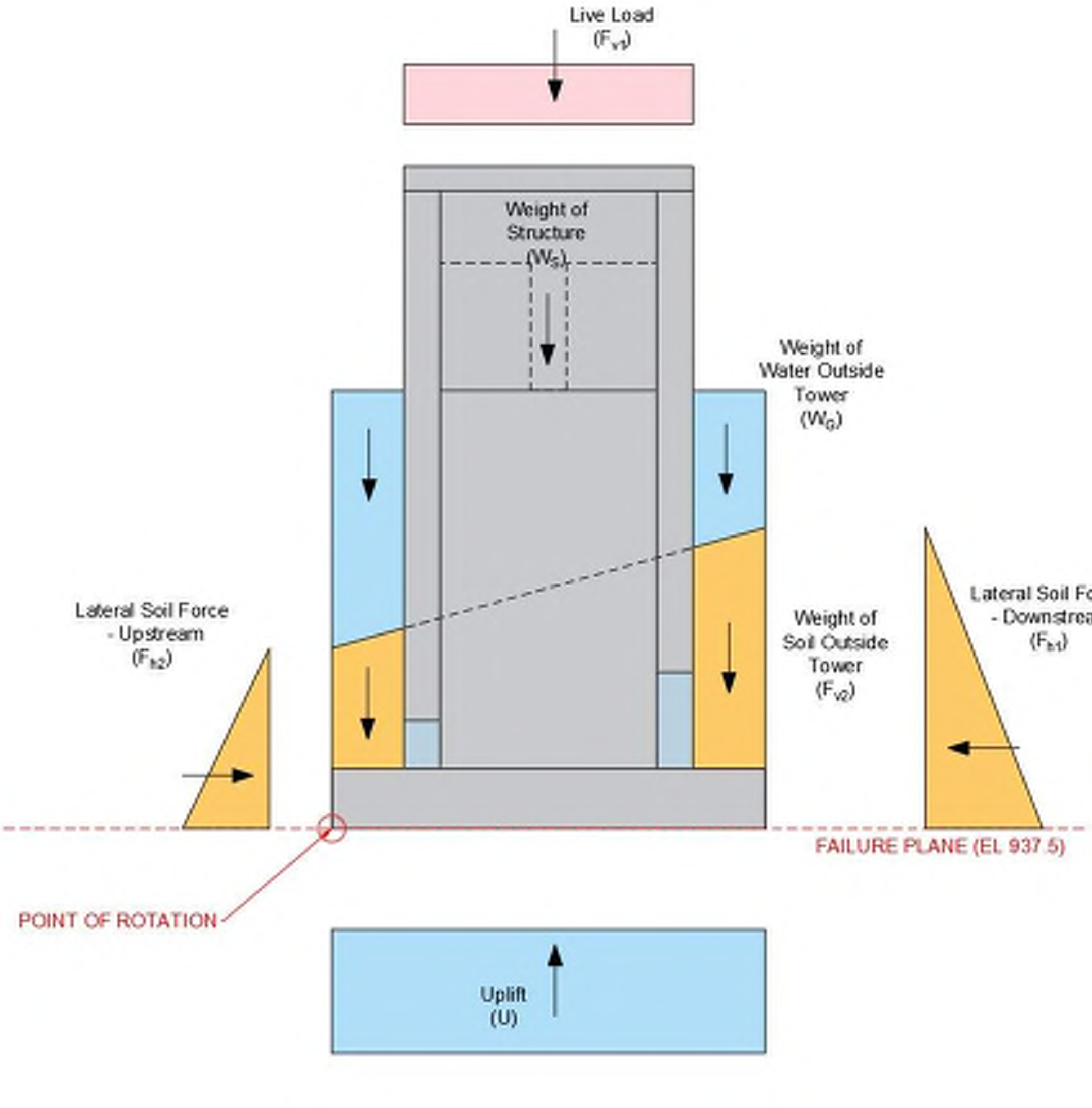


Figure 3: Free Body Diagram - Usual Loading Condition (U3)

Calculate Vertical Loads:

(A) Gravity Load of Tower

$W_S = 378.371\ \text{kip}$	Total Weight of Structure
$x_{ws} = 9\ \text{ft}$	Moment Arm

(B) Gravity Load of Water Inside Tower

$W_C := 0\ \text{kip}$	Weight of Water Inside Structure
$x_{wc} := \frac{l_{foundation}}{2} = 9\ \text{ft}$	Moment Arm

(C) Gravity Load of Water Above Top Surface of the Structure

$W_{G\_1} := 0\ \text{kip}$	Weight of Water Above Top of Structure
$x_{wg\_1} := \frac{l_{foundation}}{2} = 9\ \text{ft}$	Moment Arm
$W_{G\_2} := \left(l_{foundation} \cdot w_{foundation} - l_o \cdot w_o\right) \cdot \left(EL_{HW\_norm} - EL_{invert}\right) \cdot \gamma_w = 158.708\ \text{kip}$	Weight of Water Around Tower Foundation
$x_{wg\_2} := \frac{l_{foundation}}{2} = 9\ \text{ft}$	Moment Arm
$W_G := W_{G\_1} + W_{G\_2} = 158.708\ \text{kip}$	Total Weight of Water Outside Tower
$x_{wg} := \frac{W_{G\_1} \cdot x_{wg\_1} + W_{G\_2} \cdot x_{wg\_2}}{W_G} = 9\ \text{ft}$	Moment Arm

(D) Uplift Force at Concrete/Foundation Interface

$U := H_{HW\_norm} \cdot l_{foundation} \cdot w_{foundation} \cdot \gamma_w = 306.634\ \text{kip}$	Uplift Force Under Tower
$x_u := \frac{l_{foundation}}{2} = 9\ \text{ft}$	Moment Arm

(E) Live Load on Top Slab

$F_{v1} := F_{live} \cdot l_{top\_slab} \cdot w_{top\_slab} = 58.5\ \text{kip}$	Live Load on Top Slab
$x_{v1} := \frac{l_{foundation}}{2} = 9\ \text{ft}$	Moment Arm

(F) Gravity Load of Soil Around Structure Foundation

$$F_{v2\_1} := \left(\gamma_{sat\_fill} - \gamma_w\right) \cdot \left(\left(H_{us\_soil} + H_{ds\_soil}\right) \cdot 0.5 - t_{foundation}\right) \cdot \left(w_{foundation} - w_o\right) \cdot l_{foundation} = 53.136\ \text{kip}$$

Weight of Soil on Left/Right Sides of Tower Foundation

$$x_{v2\_1} := \frac{2 \cdot \left(H_{ds\_soil} - t_{foundation}\right) + \left(H_{us\_soil} - t_{foundation}\right)}{H_{ds\_soil} + H_{us\_soil} - t_{foundation} \cdot 2} \cdot \frac{l_{foundation}}{3} = 10\ \text{ft}$$

Moment Arm

$$F_{v2\_2} := \left(\gamma_{sat\_fill} - \gamma_w\right) \cdot \left(H_{ds\_soil} - t_{foundation}\right) \cdot \left(l_{foundation} - l_o\right) \cdot w_o \cdot 0.5 = 17.712\ \text{kip}$$

Weight of Soil on Downstream Side of Tower Foundation

$$x_{v2\_2} := l_{foundation} - \left(l_{foundation} - l_o\right) \cdot 0.5 \cdot 0.5 = 16.5\ \text{ft}$$

Moment Arm

$$F_{v2\_3} := \left(\gamma_{sat\_fill} - \gamma_w\right) \cdot \left(H_{us\_soil} - t_{foundation}\right) \cdot \left(l_{foundation} - l_o\right) \cdot w_o \cdot 0.5 = 8.856\ \text{kip}$$

Weight of Soil on Upstream Side of Tower Foundation

$$x_{v2\_3} := \left(l_{foundation} - l_o\right) \cdot 0.5 \cdot 0.5 = 1.5\ \text{ft}$$

Moment Arm

$$F_{v2} := F_{v2\_1} + F_{v2\_2} + F_{v2\_3} = 79.704\ \text{kip}$$

Weight of Soil Around Tower Foundation

$$x_{v2} := \frac{F_{v2\_1} \cdot x_{v2\_1} + F_{v2\_2} \cdot x_{v2\_2} + F_{v2\_3} \cdot x_{v2\_3}}{F_{v2}} = 10.5\ \text{ft}$$

Moment Arm

Calculate Horizontal Loads:

(A) Downstream Lateral Soil Forces

$$F_{h1} := \frac{1}{2} \cdot \left(\gamma_{sat\_fill} - \gamma_w\right) \cdot H_{ds\_soil}{}^2 \cdot K_o \cdot w_{foundation} = 37.281\ \text{kip}$$

Downstream Lateral Earth Pressure

$$x_{h1} := \frac{H_{ds\_soil}}{3} = 4.167\ \text{ft}$$

Moment Arm

(B) Upstream Lateral Soil Forces

$$F_{h2} := \frac{1}{2} \cdot \left(\gamma_{sat\_fill} - \gamma_w\right) \cdot H_{us\_soil}{}^2 \cdot K_o \cdot w_{foundation} = 13.421\ \text{kip}$$

Upstream Lateral Earth Pressure

$$x_{h2} := \frac{H_{us\_soil}}{3} = 2.5\ \text{ft}$$

Moment Arm

Overturning Stability Check

$M_{OT} := U \cdot x_u + F_{h1} \cdot x_{h1} = 2915.042 \ \text{kip} \cdot \text{ft}$

Total Overturning Moment

$M_R := W_S \cdot x_{ws} + W_C \cdot x_{wc} + W_G \cdot x_{wg} + F_{v1} \cdot x_{v1} + F_{v2} \cdot x_{v2} + F_{h2} \cdot x_{h2} = 6230.66 \ \text{kip} \cdot \text{ft}$

Total Restoring Moment

$$FS_{Overturning} := \frac{M_R}{M_{OT}} = 2.137$$

Factor of Safety Against Overturning (Note overturning stability is evaluated based on location of resultant, see below)

Resultant Location:

$F_N := W_S + W_C + W_G - U + F_{v1} + F_{v2} = 368.65 \ \text{kip}$

Total Vertical Load Resisting Overturning

$$X := \frac{M_R - M_{OT}}{F_N} = 8.994 \ \text{ft}$$

Center of Total Weight from Edge of Toe

$$e_{cc} := \frac{l_{foundation}}{2} - X = 0.006 \ \text{ft}$$

Eccentricity of the Resultant

$$\frac{l_{foundation}}{6} = 3 \ \text{ft}$$

Limit for Base Being in Compression Only

$$\text{Resultant\_Location} := \left\| \begin{array}{l} \text{if } \left|e_{cc}\right| \le \dfrac{l_{foundation}}{6} \\ \quad \left\| \text{“100\% of Base in Compression”} \right. \\ \text{else if } \dfrac{l_{foundation}}{6} < \left|e_{cc}\right| \le \dfrac{l_{foundation}}{4} \\ \quad \left\| \text{“75\% of Base in Compression”} \right. \\ \text{else if } \dfrac{l_{foundation}}{4} < \left|e_{cc}\right| \le \dfrac{l_{foundation}}{2} \\ \quad \left\| \text{“Resultant Within Base”} \right. \\ \text{else} \\ \quad \left\| \text{“Unstable-Resultant Outside Base”} \right. \end{array} \right.$$

$\text{Resultant\_Location} = \text{“100\% of Base in Compression”}$

The resultant location is within the middle third of the base and thus the base is 100% in compression. The requirement for overturning for Case 2 is the resultant force should be in the middle third of the base (EM 1110-2-2100). Meets the requirements.

Check Stresses at the Base

$$q_{max} := \begin{cases} \dfrac{F_N}{w_{foundation} \cdot l_{foundation}} \cdot \left(1 + \dfrac{6 \cdot e_{cc}}{l_{foundation}}\right) & \text{if } \left|e_{cc}\right| \le \dfrac{l_{foundation}}{6} \\ \dfrac{4 \cdot F_N}{3 \cdot w_{foundation} \cdot \left(l_{foundation} - 2 \cdot e_{cc}\right)} & \text{else} \end{cases}$$

$q_{max} = 9.501 \text{ psi}$ $q_{max} = 1368 \text{ psf}$

Equations 3-1 and 3-2 from EM 1110-2-2502

$$q_{min} := \begin{cases} \dfrac{F_N}{w_{foundation} \cdot l_{foundation}} \cdot \left(1 - \dfrac{6 \cdot e_{cc}}{l_{foundation}}\right) & \text{if } \left|e_{cc}\right| \le \dfrac{l_{foundation}}{6} \\ 0 \text{ psi} & \text{else} \end{cases}$$

$q_{min} = 9.463 \text{ psi}$ $q_{min} = 1363 \text{ psf}$

$$Check_{bearing} := \begin{cases} \text{“PASS”} & \text{if } \sigma_{allow} > q_{max} \\ \text{“FAIL”} & \text{else} \end{cases}$$

$Check_{bearing} = \text{“PASS”}$

The allowable bearing capacity of the foundation is not exceeded. Additionally, the stresses are significantly less than the assumed strength of concrete and as a result, concrete crushing is not considered to be a viable failure mechanism.

Check Flotation Stability

$$FS_{flotation} := \frac{W_S + W_C + F_{v2}}{U - W_G} = 3.1$$

Factor of Safety for Flotation (Equation 3-2 from EM 1110-2-2100)

$$FS_{usual\_flotation} := 1.3$$

Required Factor of Safety for Flotation (Table 3-4 from EM 1110-2-2100)

$$Check_{flotation} := \begin{array}{l} \text{if } FS_{flotation} \geq FS_{usual\_flotation} \\ \quad \| \text{“PASS”} \\ \text{else} \\ \quad \| \text{“FAIL”} \end{array}$$

$$Check_{flotation} = \text{“PASS”}$$

The requirements for flotation in a Normal loading condition is a factor of safety of 1.3 (EM 1110-2-2100). The structure passes this condition.

Sliding Stability Check

$$F_N = 368.65 \text{ kip}$$

Total Vertical Loads Resisting Sliding

$$F_D := F_{h1} - F_{h2} = 23.86 \text{ kip}$$

Sum of Sliding Force

$$FS_{sliding} := \frac{c \cdot w_{foundation} \cdot l_{foundation} + F_N \cdot \mu_f}{F_D} = 9.284$$

Sliding Factor of Safety (Cohesion conservatively neglected)

$$FS_{req\_sliding\_usual} := 1.7$$

Minimum Required Sliding Factor of Safety

$$Check_{sliding} := \begin{array}{l} \text{if } FS_{sliding} > FS_{req\_sliding\_usual} \\ \quad \| \text{“PASS”} \\ \text{else} \\ \quad \| \text{“FAIL”} \end{array}$$

$$Check_{sliding} = \text{“PASS”}$$

The requirements for sliding in an Usual loading condition is a factor of safety of 1.7 (EM 1110-2-2100). The structure passes this condition.

Case 3 - Unusual Load Condition (Construction - UN4)

Free Body Diagram:

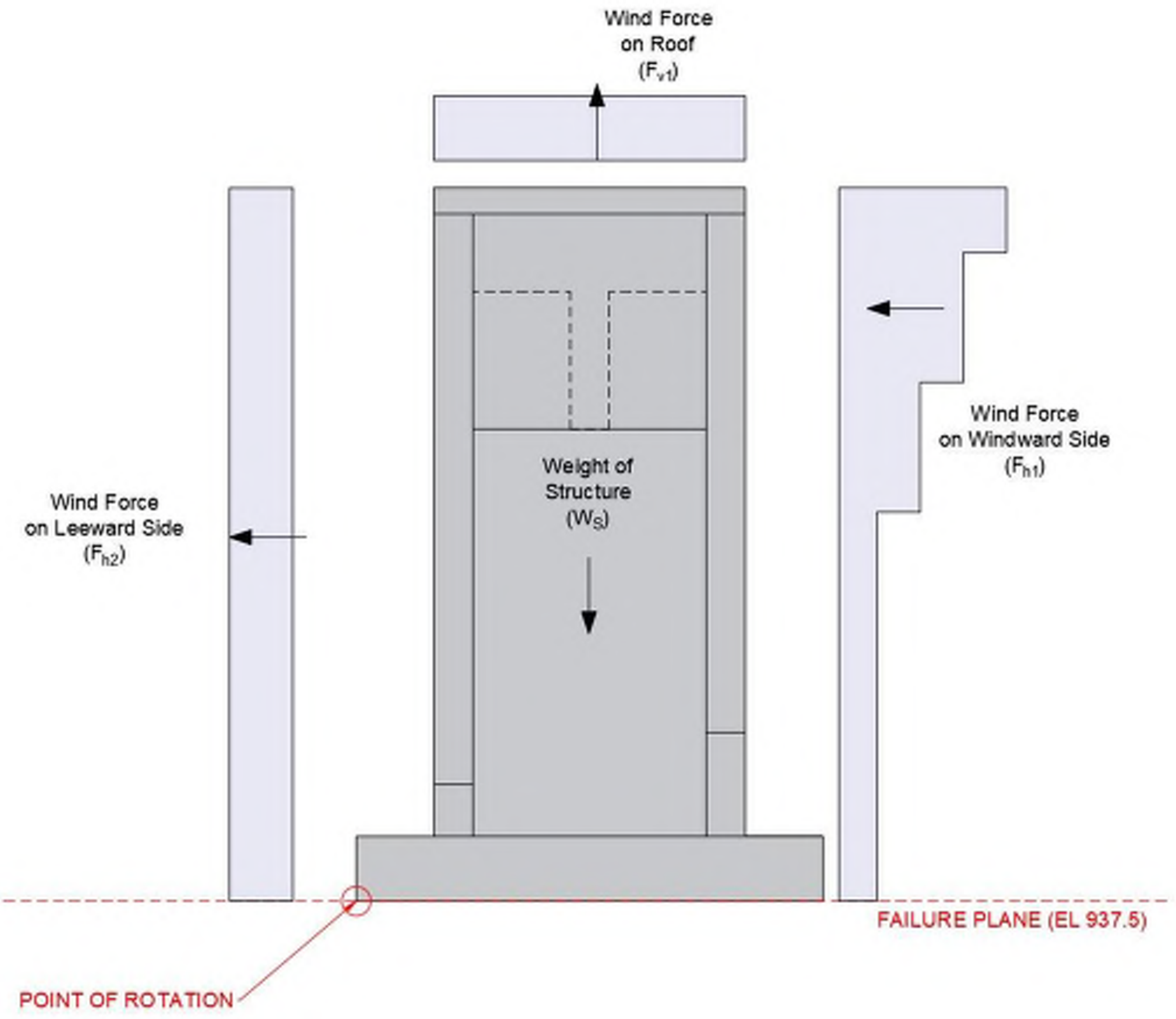


Figure 4: Free Body Diagram - Unusual (Construction) Loading Condition (U4)

Calculate Vertical Loads:

(A) Gravity Load of Tower

$W_S = 378.371\ \text{kip}$ Total Weight of Structure

$x_{ws} = 9\ \text{ft}$ Moment Arm

(B) Gravity Load of Water Inside Tower

$W_C := 0\ \text{kip}$ Weight of Water Inside Structure

$x_{wc} := \dfrac{l_{foundation}}{2} = 9\ \text{ft}$ Moment Arm

(C) Gravity Load of Water Above Top Surface of the Structure

$W_{G\_1} := 0\ \text{kip}$ Weight of Water Above Top of Structure

$x_{wg\_1} := \dfrac{l_{foundation}}{2} = 9\ \text{ft}$ Moment Arm

$W_{G\_2} := 0\ \text{kip}$ Weight of Water Around Tower Foundation

$x_{wg\_2} := \dfrac{l_{foundation}}{2} = 9\ \text{ft}$ Moment Arm

$W_G := W_{G\_1} + W_{G\_2} = 0\ \text{kip}$ Weight of Water Outside Tower

$x_{wg} := \dfrac{l_{foundation}}{2} = 9\ \text{ft}$ Moment Arm

(D) Uplift Force at Concrete/Foundation Interface

$U := 0\ \text{kip}$ Uplift Force Under Tower

$x_u := \dfrac{l_{foundation}}{2} = 9\ \text{ft}$ Moment Arm

(E) Wind Force on Roof

$F_{v1} := 12.750\ \text{kip}$ Negative Pressure Wind Force

$x_{v1} := \dfrac{l_{foundation}}{2} = 9\ \text{ft}$ Moment Arm

Calculate Horizontal Loads:

(A) Wind Force on Windward Side

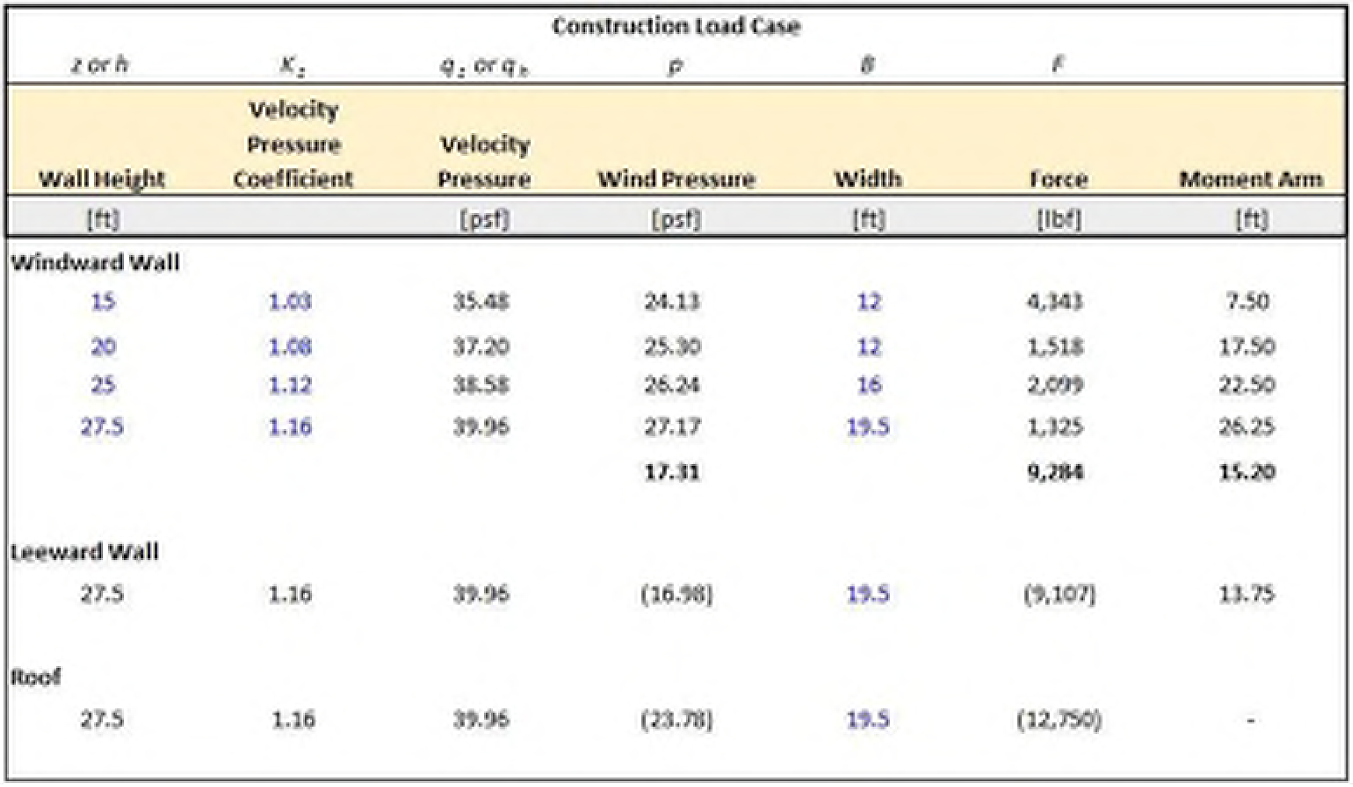
$F_{h1} := 9.284\ \text{kip}$ Wind Load on Windward Side

$x_{h1} := 15.20\ \text{ft}$ Moment Arm

(B) Wind Force on Leeward Side

$F_{h2} := 9.107\ \text{kip}$ Wind Load on Leeward Side

$x_{h2} := 13.75\ \text{ft}$ Moment Arm



Construction Load Case						
z or h	K_z	q_z or q_h	p	B	F	
Wall Height	Velocity Pressure Coefficient	Velocity Pressure	Wind Pressure	Width	Force	Moment Arm
[ft]		[psf]	[psf]	[ft]	[lbf]	[ft]
Windward Wall						
15	1.03	35.48	24.13	12	4,343	7.50
20	1.08	37.20	25.30	12	1,518	17.50
25	1.12	38.58	26.24	16	2,099	22.50
27.5	1.16	39.96	27.17	19.5	1,325	26.25
			17.31		**9,284**	**15.20**
Leeward Wall						
27.5	1.16	39.96	(16.98)	19.5	(9,107)	13.75
Roof						
27.5	1.16	39.96	(23.78)	19.5	(12,750)	-

Figure 5: Wind Load Calculations - Unusual (Construction) Loading Condition

Overturning Stability Check

$M_{OT} := F_{h1} \cdot x_{h1} + F_{h2} \cdot x_{h2} + U \cdot x_u + F_{v1} \cdot x_{v1} = 381.088\ \text{kip} \cdot \text{ft}$

Total Overturning Moment

$M_R := W_S \cdot x_{ws} + W_C \cdot x_{wc} + W_G \cdot x_{wg} = 3405.341\ \text{kip} \cdot \text{ft}$

Total Restoring Moment

$FS_{Overturning} := \frac{M_R}{M_{OT}} = 8.936$

Factor of Safety Against Overturning (Note overturning stability is evaluated based on location of resultant, see below)

Resultant Location:

$F_N := W_S + W_C + W_G - U - F_{v1} = 365.621\ \text{kip}$

Total Vertical Load Resisting Overturning

$X := \frac{M_R - M_{OT}}{F_N} = 8.272\ \text{ft}$

Center of Total Weight from Edge of Toe

$e_{cc} := \frac{l_{foundation}}{2} - X = 0.728\ \text{ft}$

Eccentricity of the Resultant

$\frac{l_{foundation}}{6} = 3\ \text{ft}$

Limit for Base Being in Compression Only

$$
\text{Resultant\_Location} := \left\| \begin{array}{l}
\text{if } \left|e_{cc}\right| \le \frac{l_{foundation}}{6} \\
\quad \left\| \text{“100\% of Base in Compression”} \right. \\
\text{else if } \frac{l_{foundation}}{6} < \left|e_{cc}\right| \le \frac{l_{foundation}}{4} \\
\quad \left\| \text{“75\% of Base in Compression”} \right. \\
\text{else if } \frac{l_{foundation}}{4} < \left|e_{cc}\right| \le \frac{l_{foundation}}{2} \\
\quad \left\| \text{“Resultant Within Base”} \right. \\
\text{else} \\
\quad \left\| \text{“Unstable-Resultant Outside Base”} \right.
\end{array} \right.
$$

$\text{Resultant\_Location} = \text{“100\% of Base in Compression”}$

The resultant location is within the middle third of the base and thus the base is 100% in compression. The requirement for overturning for Case 3 is the resultant force should be in the middle half of the base (EM 1110-2-2100). Meets the requirements.

Check Stresses at the Base

$$q_{max} := \left\| \begin{array}{l} \text{if } \left|e_{cc}\right| \le \dfrac{l_{foundation}}{6} \\ \quad \left\| \dfrac{F_N}{w_{foundation} \cdot l_{foundation}} \cdot \left(1 + \dfrac{6 \cdot e_{cc}}{l_{foundation}}\right) \right. \\ \text{else} \\ \quad \left\| \dfrac{4 \cdot F_N}{3 \cdot w_{foundation} \cdot \left(l_{foundation} - 2 \cdot e_{cc}\right)} \right. \end{array} \right.$$

$q_{max} = 11.687\ \text{psi}$ $q_{max} = 1683\ \text{psf}$

Equations 3-1 and 3-2 from EM 1110-2-2502

$$q_{min} := \left\| \begin{array}{l} \text{if } \left|e_{cc}\right| \le \dfrac{l_{foundation}}{6} \\ \quad \left\| \dfrac{F_N}{w_{foundation} \cdot l_{foundation}} \cdot \left(1 - \dfrac{6 \cdot e_{cc}}{l_{foundation}}\right) \right. \\ \text{else} \\ \quad \left\| 0\ \text{psi} \right. \end{array} \right.$$

$q_{min} = 7.12\ \text{psi}$ $q_{min} = 1025\ \text{psf}$

$$Check_{bearing} := \begin{array}{l} \text{if } \sigma_{allow} \cdot 1.15 > q_{max} \\ \quad \left\| \text{“PASS”} \right. \\ \text{else} \\ \quad \left\| \text{“FAIL”} \right. \end{array}$$

$Check_{bearing} = \text{“PASS”}$

The allowable bearing capacity of the foundation is not exceeded. Additionally, the stresses are significantly less than the assumed strength of concrete and as a result, concrete crushing is not considered to be a viable failure mechanism.

Check Flotation Stability

The requirement for flotation stability does not apply in this load combination.

Sliding Stability Check

Sliding Check

$F_N = 365.621\ \text{kip}$

Total Vertical Loads Resisting Sliding

$F_D := F_{h1} + F_{h2} = 18.391\ \text{kip}$

Sum of Sliding Force

$$FS_{sliding} := \frac{c \cdot w_{foundation} \cdot l_{foundation} + F_N \cdot \mu_f}{F_D} = 11.945$$

Sliding Factor of Safety (Cohesion conservatively neglected)

$FS_{req\_sliding\_unusual} := 1.3$

Minimum Required Sliding Factor of Safety

$$Check_{sliding} := \begin{array}{l} \text{if } FS_{sliding} > FS_{req\_sliding\_unusual} \\ \quad \| \text{“PASS”} \\ \text{else} \\ \quad \| \text{“FAIL”} \end{array}$$

$Check_{sliding} = \text{“PASS”}$

The requirements for sliding in an Unusual loading condition is a factor of safety of 1.3 (EM 1110-2-2100). The structure passes this condition.

Case 4 - Extreme Load Condition (Load Condition U3 Plus Seismic - ED1)

Free Body Diagram:

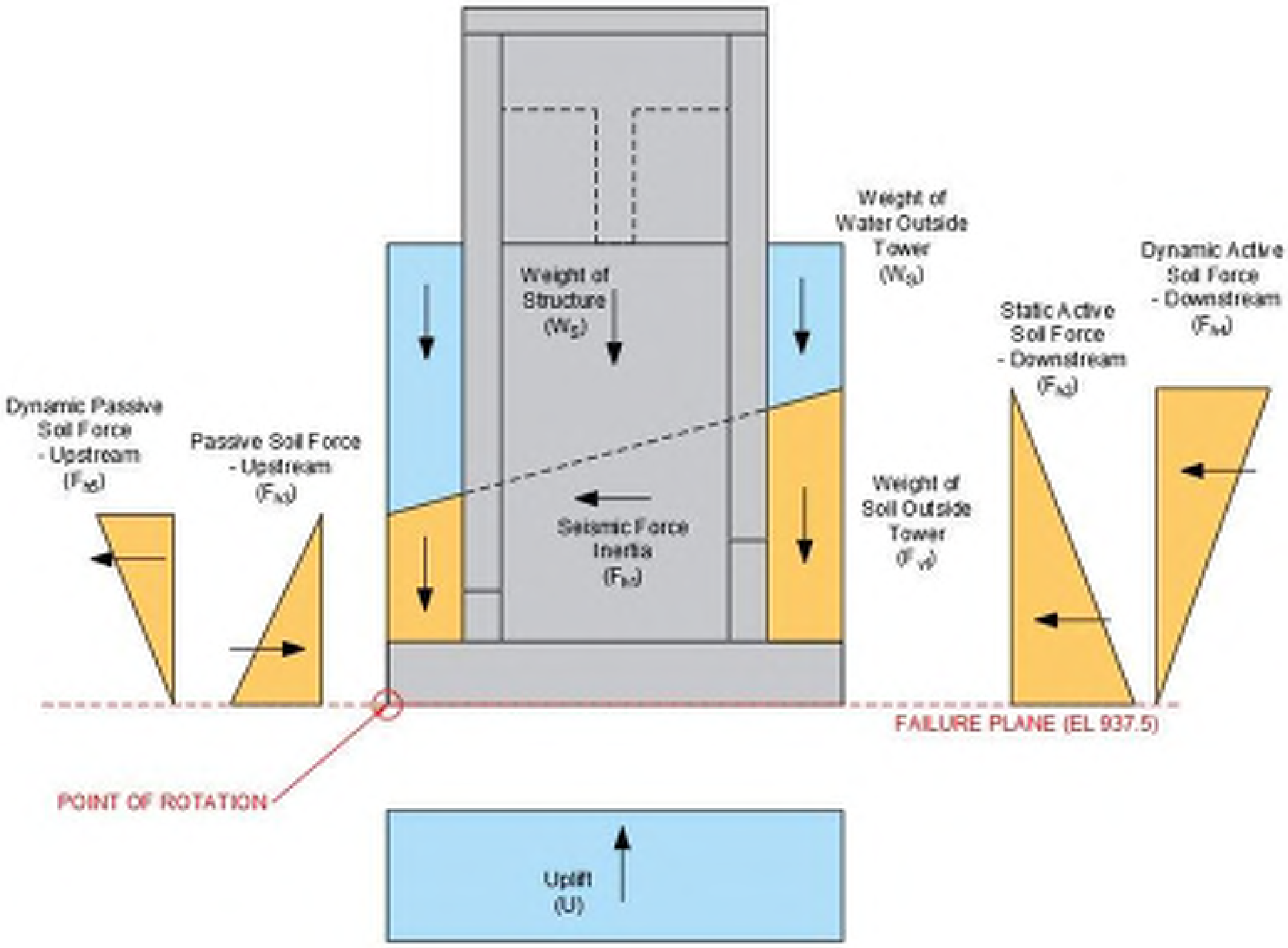


Figure 6: Free Body Diagram - Extreme (Seismic) Loading Condition (ED1)

Calculate Vertical Loads:

(A) Gravity Load of Tower

$W_S = 378.371\ \text{kip}$ Total Weight of Structure

$x_{ws} = 9\ \text{ft}$ Moment Arm

(B) Gravity Load of Water Inside Tower

$W_C := 0\ \text{kip}$ Weight of Water Inside Structure

$x_{wc} := \dfrac{l_{foundation}}{2} = 9\ \text{ft}$ Moment Arm

(C) Gravity Load of Water Above Top Surface of the Structure

$W_{G\_1} := 0\ \text{kip}$ Weight of Water Above Top of Structure

$x_{wg\_1} := \dfrac{l_{foundation}}{2} = 9\ \text{ft}$ Moment Arm

$W_{G\_2} := \left(l_{foundation} \cdot w_{foundation} - l_o \cdot w_o\right) \cdot \left(EL_{HW\_norm} - EL_{invert}\right) \cdot \gamma_w = 158.708\ \text{kip}$ Weight of Water Around Tower Foundation

$x_{wg\_2} := \dfrac{l_{foundation}}{2} = 9\ \text{ft}$ Moment Arm

$W_G := W_{G\_1} + W_{G\_2} = 158.708\ \text{kip}$ Weight of Water Outside Tower

$x_{wg} := \dfrac{W_{G\_1} \cdot x_{wg\_1} + W_{G\_2} \cdot x_{wg\_2}}{W_G} = 9\ \text{ft}$ Moment Arm

(D) Uplift Force at Concrete/Foundation Interface

$U := H_{HW\_norm} \cdot l_{foundation} \cdot w_{foundation} \cdot \gamma_w = 306.634\ \text{kip}$ Uplift Force Under Tower

$x_u := \dfrac{l_{foundation}}{2} = 9\ \text{ft}$ Moment Arm

(E) Gravity Load of Soil Around Structure Foundation

$F_{v1\_1} := \left(\gamma_{sat\_fill} - \gamma_w\right) \cdot \left(\left(H_{us\_soil} + H_{ds\_soil}\right) \cdot 0.5 - t_{foundation}\right) \cdot \left(w_{foundation} - w_o\right) \cdot l_{foundation} = 53.136\ \text{kip}$ Weight of Soil on Left/Right Sides of Tower Foundation

$x_{v1\_1} := \dfrac{2 \cdot \left(H_{ds\_soil} - t_{foundation}\right) + \left(H_{us\_soil} - t_{foundation}\right)}{H_{ds\_soil} + H_{us\_soil} - t_{foundation} \cdot 2} \cdot \dfrac{l_{foundation}}{3} = 10\ \text{ft}$ Moment Arm

$F_{v1\_2} := (\gamma_{sat\_fill} - \gamma_w) \cdot (H_{ds\_soil} - t_{foundation}) \cdot (l_{foundation} - l_o) \cdot w_o \cdot 0.5 = 17.712\ kip$

Weight of Soil on Downstream Side of Tower Foundation

$x_{v1\_2} := l_{foundation} - (l_{foundation} - l_o) \cdot 0.5 \cdot 0.5 = 16.5\ ft$

Moment Arm

$F_{v1\_3} := (\gamma_{sat\_fill} - \gamma_w) \cdot (H_{us\_soil} - t_{foundation}) \cdot (l_{foundation} - l_o) \cdot w_o \cdot 0.5 = 8.856\ kip$

Weight of Soil on Upstream Side of Tower Foundation

$x_{v1\_3} := (l_{foundation} - l_o) \cdot 0.5 \cdot 0.5 = 1.5\ ft$

Moment Arm

$F_{v1} := F_{v1\_1} + F_{v1\_2} + F_{v1\_3} = 79.704\ kip$

Weight of Soil Around Tower Foundation

$$x_{v1} := \frac{F_{v1\_1} \cdot x_{v1\_1} + F_{v1\_2} \cdot x_{v1\_2} + F_{v1\_3} \cdot x_{v1\_3}}{F_{v1}} = 10.5\ ft$$

Moment Arm

Calculate Horizontal Loads:

(A) Seismic Force (Inertia)

$F_{h1} := 28.7\ kip$

Seismic Shear Force at Base (see separate spreadsheet for calculation)

$M_{h1} := 486.5\ kip \cdot ft$

Seismic Moment at Base (see separate spreadsheet for calculation)

(B) Downstream Lateral Soil Forces (Static Active)

$$F_{h2} := \frac{1}{2} \cdot (\gamma_{sat\_fill} - \gamma_w) \cdot H_{ds\_soil}^{\ 2} \cdot K_a \cdot w_{foundation} = 24.608\ kip$$

Downstream Lateral Earth Pressure

$$x_{h2} := \frac{H_{ds\_soil}}{3} = 4.167\ ft$$

Moment Arm

(C) Upstream Lateral Soil Forces (Static Passive)

$$F_{h3} := \frac{1}{2} \cdot (\gamma_{sat\_fill} - \gamma_w) \cdot H_{us\_soil}^{\ 2} \cdot K_p \cdot w_{foundation} = 86.458\ kip$$

Downstream Lateral Earth Pressure

$$x_{h3} := \frac{H_{us\_soil}}{3} = 2.5\ ft$$

Moment Arm

(D) Downstream Lateral Soil Forces (Dynamic Active)

$F_{h4} := \frac{1}{2} \cdot \left(\gamma_{sat\_fill} - \gamma_w\right) \cdot H_{ds\_soil}^{\ 2} \cdot \left(K_{AE} - K_a\right) \cdot w_{foundation} = 5.818 \text{ kip}$ Downstream Lateral Earth Pressure

$x_{h4} := \frac{2 \cdot H_{ds\_soil}}{3} = 8.333 \text{ ft}$ Moment Arm

(E) Upstream Lateral Soil Forces (Dynamic Passive)

$F_{h5} := \frac{1}{2} \cdot \left(\gamma_{sat\_fill} - \gamma_w\right) \cdot H_{us\_soil}^{\ 2} \cdot \left(K_p - K_{PE}\right) \cdot w_{foundation} = 6.085 \text{ kip}$ Upstream Lateral Earth Pressure

$x_{h5} := \frac{2 \cdot H_{us\_soil}}{3} = 5 \text{ ft}$ Moment Arm

Overturning Stability Check

$M_{OT} := U \cdot x_u + M_{h1} + F_{h2} \cdot x_{h2} + F_{h4} \cdot x_{h4} + F_{h5} \cdot x_{h5} = 3427.645 \text{ kip} \cdot \text{ft}$ Total Overturning Moment

$M_R := W_S \cdot x_{ws} + W_C \cdot x_{wc} + W_G \cdot x_{wg} + F_{v1} \cdot x_{v1} + F_{h3} \cdot x_{h3} = 5886.751 \text{ kip} \cdot \text{ft}$ Total Restoring Moment

$FS_{Overturning} := \frac{M_R}{M_{OT}} = 1.717$ Factor of Safety Against Overturning (Note overturning stability is evaluated based on location of resultant, see below)

Resultant Location:

$F_N := W_S + W_C + W_G - U + F_{v1} = 310.15 \text{ kip}$ Total Vertical Load Resisting Overturning

$X := \frac{M_R - M_{OT}}{F_N} = 7.929 \text{ ft}$ Center of Total Weight from Edge of Toe

$e_{cc} := \frac{l_{foundation}}{2} - X = 1.071 \text{ ft}$ Eccentricity of the Resultant

$\frac{l_{foundation}}{6} = 3 \text{ ft}$ Limit for Base Being in Compression Only

$$\text{Resultant\_Location} := \left\| \begin{array}{l} \text{if } \left|e_{cc}\right| \le \frac{l_{foundation}}{6} \\ \quad \left\| \text{“100\% of Base in Compression”} \right. \\ \text{else if } \frac{l_{foundation}}{6} < \left|e_{cc}\right| \le \frac{l_{foundation}}{4} \\ \quad \left\| \text{“75\% of Base in Compression”} \right. \\ \text{else if } \frac{l_{foundation}}{4} < \left|e_{cc}\right| \le \frac{l_{foundation}}{2} \\ \quad \left\| \text{“Resultant Within Base”} \right. \\ \text{else} \\ \quad \left\| \text{“Unstable-Resultant Outside Base”} \right. \end{array} \right.$$

$\text{Resultant\_Location} = \text{“100\% of Base in Compression”}$

The resultant location is within the middle third of the base and thus the base is 100% in compression. The requirement for overturning for Case 4 is the resultant force should be within the base (EM 1110-2-2100). Meets the requirements.

Check Stresses at the Base

$$q_{max} := \begin{array}{|l} \text{if } \left|e_{cc}\right| \le \dfrac{l_{foundation}}{6} \\ \quad \left\| \dfrac{F_N}{w_{foundation} \cdot l_{foundation}} \cdot \left(1 + \dfrac{6 \cdot e_{cc}}{l_{foundation}}\right) \right. \\ \text{else} \\ \quad \left\| \dfrac{4 \cdot F_N}{3 \cdot w_{foundation} \cdot \left(l_{foundation} - 2 \cdot e_{cc}\right)} \right. \end{array}$$

$q_{max} = 10.826 \text{ psi}$ $q_{max} = 1559 \text{ psf}$

Equations 3-1 and 3-2 from EM 1110-2-2502

$$q_{min} := \begin{array}{|l} \text{if } \left|e_{cc}\right| \le \dfrac{l_{foundation}}{6} \\ \quad \left\| \dfrac{F_N}{w_{foundation} \cdot l_{foundation}} \cdot \left(1 - \dfrac{6 \cdot e_{cc}}{l_{foundation}}\right) \right. \\ \text{else} \\ \quad \left\| 0 \text{ psi} \right. \end{array}$$

$q_{min} = 5.129 \text{ psi}$ $q_{min} = 739 \text{ psf}$

$$Check_{bearing} := \begin{array}{|l} \text{if } \sigma_{allow} \cdot 1.5 > q_{max} \\ \quad \left\| \text{“PASS”} \right. \\ \text{else} \\ \quad \left\| \text{“FAIL”} \right. \end{array}$$

$Check_{bearing} = \text{“PASS”}$

The allowable bearing capacity of the foundation is not exceeded. Additionally, the stresses are significantly less than the assumed strength of concrete and as a result, concrete crushing is not considered to be a viable failure mechanism.

Check Flotation Stability

$FS_{flotation} := \frac{W_S + W_C + F_{v1}}{U - W_G} = 3.1$

Factor of Safety for Flotation (Equation 3-2 from EM 1110-2-2100)

$FS_{extreme\_flotation} := 1.1$

Required Factor of Safety for Flotation (Table 3-4 from EM 1110-2-2100)

```
Check_flotation := if FS_flotation ≥ FS_extreme_flotation
                       ‖ "PASS"
                   else
                       ‖ "FAIL"
```

$Check_{flotation} = \text{"PASS"}$

The requirements for flotation in an Extreme loading condition is a factor of safety of 1.1 (EM 1110-2-2100). The structure passes this condition.

Sliding Stability Check

Sliding Check

$F_N = 310.15\ \text{kip}$

Total Vertical Loads Resisting Sliding

$F_D := F_{h1} + F_{h2} + F_{h4} + F_{h5} - F_{h3} = -21.247\ \text{kip}$

Sum of Sliding Force

$FS_{sliding} := \left| \frac{c \cdot w_{foundation} \cdot l_{foundation} + F_N \cdot \mu_f}{F_D} \right| = 8.771$

Sliding Factor of Safety (Cohesion conservatively neglected)

$FS_{req\_sliding\_extreme} := 1.1$

Minimum Required Sliding Factor of Safety

```
Check_sliding := if FS_sliding > FS_req_sliding_extreme
                     ‖ "PASS"
                 else
                     ‖ "FAIL"
```

$Check_{sliding} = \text{"PASS"}$

The requirements for sliding in an Extreme loading condition is a factor of safety of 1.1 (EM 1110-2-2100). The structure passes this condition.

Case 5 - Unusual Load Condition (Flood With Water Inside Structure - ED2A)

Free Body Diagram:

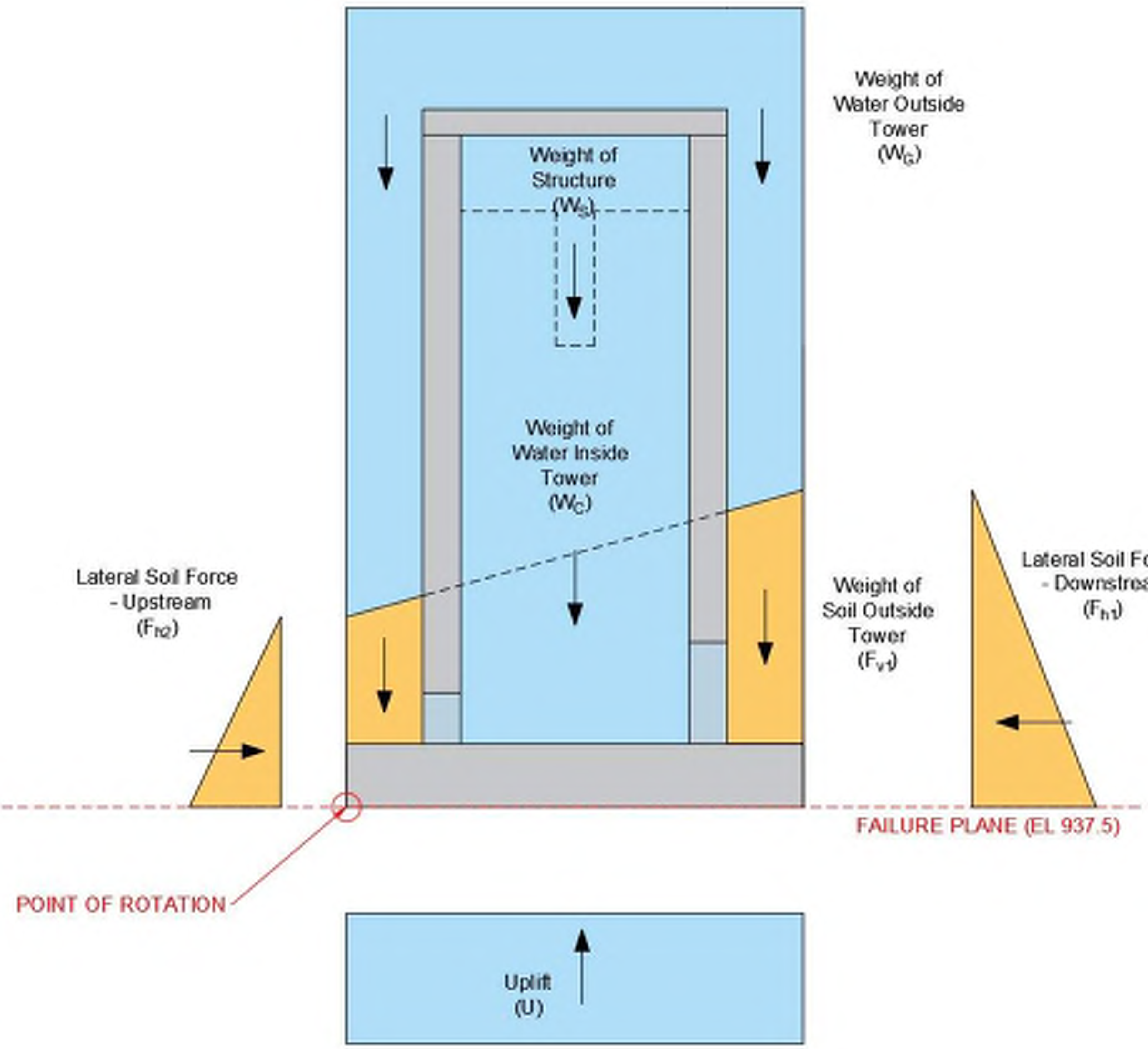


Figure 7: Free Body Diagram - Extreme (Flood) Loading Condition (ED2A)

Calculate Vertical Loads:

(A) Gravity Load of Tower

$W_S = 378.371\ \text{kip}$ Total Weight of Structure

$x_{ws} = 9\ \text{ft}$ Moment Arm

(B) Gravity Load of Water Inside Tower

$W_C := l_i \cdot w_i \cdot \left(EL_{bot\_slab} - EL_{invert}\right) \cdot \gamma_w = 80.87\ \text{kip}$ Weight of Water Inside Structure

$x_{wc} := \dfrac{l_{foundation}}{2} = 9\ \text{ft}$ Moment Arm

(C) Gravity Load of Water Above Top Surface of the Structure

$W_{G\_1} := l_o \cdot w_o \cdot \left(EL_{HW\_flood} - EL_{top\_slab}\right) \cdot \gamma_w = 26.283\ \text{kip}$ Weight of Water Above Top of Structure

$x_{wg\_1} := \dfrac{l_{foundation}}{2} = 9\ \text{ft}$ Moment Arm

$W_{G\_2} := \left(l_{foundation} \cdot w_{foundation} - l_o \cdot w_o\right) \cdot \left(EL_{HW\_flood} - EL_{invert}\right) \cdot \gamma_w = 292.144\ \text{kip}$ Weight of Water Around Tower Foundation

$x_{wg\_2} := \dfrac{l_{foundation}}{2} = 9\ \text{ft}$ Moment Arm

$W_G := W_{G\_1} + W_{G\_2} = 318.427\ \text{kip}$ Weight of Water Outside Tower

$x_{wg} := \dfrac{W_{G\_1} \cdot x_{wg\_1} + W_{G\_2} \cdot x_{wg\_2}}{W_G} = 9\ \text{ft}$ Moment Arm

(D) Uplift Force at Concrete/Foundation Interface

$U := H_{HW\_flood} \cdot l_{foundation} \cdot w_{foundation} \cdot \gamma_w = 529.027\ \text{kip}$ Uplift Force Under Tower

$x_u := \dfrac{l_{foundation}}{2} = 9\ \text{ft}$ Moment Arm

(E) Gravity Load of Soil Around Structure Foundation

$F_{v1\_1} := \left(\gamma_{sat\_fill} - \gamma_w\right) \cdot \left(\left(H_{us\_soil} + H_{ds\_soil}\right) \cdot 0.5 - t_{foundation}\right) \cdot \left(w_{foundation} - w_o\right) \cdot l_{foundation} = 53.136\ \text{kip}$ Weight of Soil on Left/Right Sides of Tower Foundation

$x_{v1\_1} := \dfrac{2 \cdot \left(H_{ds\_soil} - t_{foundation}\right) + \left(H_{us\_soil} - t_{foundation}\right)}{H_{ds\_soil} + H_{us\_soil} - t_{foundation} \cdot 2} \cdot \dfrac{l_{foundation}}{3} = 10\ \text{ft}$ Moment Arm

$$F_{v1\_2} := \left(\gamma_{sat\_fill} - \gamma_w\right) \cdot \left(H_{ds\_soil} - t_{foundation}\right) \cdot \left(l_{foundation} - l_o\right) \cdot w_o \cdot 0.5 = 17.712\ \text{kip}$$

Weight of Soil on Downstream Side of Tower Foundation

$$x_{v1\_2} := l_{foundation} - \left(l_{foundation} - l_o\right) \cdot 0.5 \cdot 0.5 = 16.5\ \text{ft}$$

Moment Arm

$$F_{v1\_3} := \left(\gamma_{sat\_fill} - \gamma_w\right) \cdot \left(H_{us\_soil} - t_{foundation}\right) \cdot \left(l_{foundation} - l_o\right) \cdot w_o \cdot 0.5 = 8.856\ \text{kip}$$

Weight of Soil on Upstream Side of Tower Foundation

$$x_{v1\_3} := \left(l_{foundation} - l_o\right) \cdot 0.5 \cdot 0.5 = 1.5\ \text{ft}$$

Moment Arm

$$F_{v1} := F_{v1\_1} + F_{v1\_2} + F_{v1\_3} = 79.704\ \text{kip}$$

Weight of Soil Around Tower Foundation

$$x_{v1} := \frac{F_{v1\_1} \cdot x_{v1\_1} + F_{v1\_2} \cdot x_{v1\_2} + F_{v1\_3} \cdot x_{v1\_3}}{F_{v1}} = 10.5\ \text{ft}$$

Moment Arm

Calculate Horizontal Loads:

(A) Downstream Lateral Soil Forces

$$F_{h1} := \frac{1}{2} \cdot \left(\gamma_{sat\_fill} - \gamma_w\right) \cdot H_{ds\_soil}^{\ 2} \cdot K_o \cdot w_{foundation} = 37.281\ \text{kip}$$

Downstream Lateral Earth Pressure

$$x_{h1} := \frac{H_{ds\_soil}}{3} = 4.167\ \text{ft}$$

Moment Arm

(B) Upstream Lateral Soil Forces

$$F_{h2} := \frac{1}{2} \cdot \left(\gamma_{sat\_fill} - \gamma_w\right) \cdot H_{us\_soil}^{\ 2} \cdot K_o \cdot w_{foundation} = 13.421\ \text{kip}$$

Upstream Lateral Earth Pressure

$$x_{h2} := \frac{H_{us\_soil}}{3} = 2.5\ \text{ft}$$

Moment Arm

Overturning Stability Check

$$M_{OT} := U \cdot x_u + F_{h1} \cdot x_{h1} = 4916.584 \text{ kip} \cdot \text{ft}$$

Total Overturning Moment

$$M_R := W_S \cdot x_{ws} + W_C \cdot x_{wc} + W_G \cdot x_{wg} + F_{v1} \cdot x_{v1} + F_{h2} \cdot x_{h2} = 7869.465 \text{ kip} \cdot \text{ft}$$

Total Restoring Moment

$$FS_{Overturning} := \frac{M_R}{M_{OT}} = 1.601$$

Factor of Safety Against Overturning (Note overturning stability is evaluated based on location of resultant, see below)

Resultant Location:

$$F_N := W_S + W_C + W_G - U + F_{v1} = 328.346 \text{ kip}$$

Total Vertical Load Resisting Overturning

$$X := \frac{M_R - M_{OT}}{F_N} = 8.993 \text{ ft}$$

Center of Total Weight from Edge of Toe

$$e_{cc} := \frac{l_{foundation}}{2} - X = 0.007 \text{ ft}$$

Eccentricity of the Resultant

$$\frac{l_{foundation}}{6} = 3 \text{ ft}$$

Limit for Base Being in Compression Only

$$Resultant\_Location := \left\| \begin{array}{l} \text{if } \left|e_{cc}\right| \le \dfrac{l_{foundation}}{6} \\ \quad \left\| \text{“100\% of Base in Compression”} \right. \\ \text{else if } \dfrac{l_{foundation}}{6} < \left|e_{cc}\right| \le \dfrac{l_{foundation}}{4} \\ \quad \left\| \text{“75\% of Base in Compression”} \right. \\ \text{else if } \dfrac{l_{foundation}}{4} < \left|e_{cc}\right| \le \dfrac{l_{foundation}}{2} \\ \quad \left\| \text{“Resultant Within Base”} \right. \\ \text{else} \\ \quad \left\| \text{“Unstable-Resultant Outside Base”} \right. \end{array} \right.$$

$$Resultant\_Location = \text{“100\% of Base in Compression”}$$

The resultant location is within the middle third of the base and thus the base is 100% in compression. The requirement for overturning for Case 5 is the resultant force should within the base (EM 1110-2-2100). Meets the requirements.

Check Stresses at the Base

$$q_{max} := \begin{cases} \dfrac{F_N}{w_{foundation} \cdot l_{foundation}} \cdot \left(1 + \dfrac{6 \cdot e_{cc}}{l_{foundation}}\right) & \text{if } \left|e_{cc}\right| \le \dfrac{l_{foundation}}{6} \\ \dfrac{4 \cdot F_N}{3 \cdot w_{foundation} \cdot \left(l_{foundation} - 2 \cdot e_{cc}\right)} & \text{else} \end{cases}$$

$q_{max} = 8.464 \text{ psi}$ $q_{max} = 1219 \text{ psf}$

Equations 3-1 and 3-2 from EM 1110-2-2502

$$q_{min} := \begin{cases} \dfrac{F_N}{w_{foundation} \cdot l_{foundation}} \cdot \left(1 - \dfrac{6 \cdot e_{cc}}{l_{foundation}}\right) & \text{if } \left|e_{cc}\right| \le \dfrac{l_{foundation}}{6} \\ 0 \text{ psi} & \text{else} \end{cases}$$

$q_{min} = 8.426 \text{ psi}$ $q_{min} = 1213 \text{ psf}$

$$Check_{bearing} := \begin{cases} \text{"PASS"} & \text{if } \sigma_{allow} > q_{max} \\ \text{"FAIL"} & \text{else} \end{cases}$$

$Check_{bearing} = \text{"PASS"}$

The allowable bearing capacity of the foundation is not exceeded. Additionally, the stresses are significantly less than the assumed strength of concrete and as a result, concrete crushing is not considered to be a viable failure mechanism.

Check Flotation Stability

$$FS_{flotation} := \frac{W_S + W_C}{U - W_G} = 2.18$$

Factor of Safety for Flotation (Equation 3-2 from EM 1110-2-2100)

$$FS_{extreme\_flotation} := 1.1$$

Required Factor of Safety for Flotation (Table 3-4 from EM 1110-2-2100)

$$Check_{flotation} := \left| \begin{array}{l} \text{if } FS_{flotation} \geq FS_{extreme\_flotation} \\ \quad \Vert \text{“PASS”} \\ \text{else} \\ \quad \Vert \text{“FAIL”} \end{array} \right.$$

$Check_{flotation} = \text{“PASS”}$

The requirements for flotation in an Extreme loading condition is a factor of safety of 1.1 (EM 1110-2-2100). The structure passes this condition.

Sliding Stability Check

$F_N = 328.346\ \text{kip}$

Total Vertical Loads Resisting Sliding

$F_D := F_{h1} - F_{h2} = 23.86\ \text{kip}$

Sum of Sliding Force

$$FS_{sliding} := \frac{c \cdot w_{foundation} \cdot l_{foundation} + F_N \cdot \mu_f}{F_D} = 8.269$$

Sliding Factor of Safety (Cohesion conservatively neglected)

$FS_{req\_sliding\_extreme} := 1.1$

Minimum Required Sliding Factor of Safety

$$Check_{sliding} := \left| \begin{array}{l} \text{if } FS_{sliding} > FS_{req\_sliding\_extreme} \\ \quad \Vert \text{“PASS”} \\ \text{else} \\ \quad \Vert \text{“FAIL”} \end{array} \right.$$

$Check_{sliding} = \text{“PASS”}$

The requirements for sliding in an Extreme loading condition is a factor of safety of 1.1 (EM 1110-2-2100). The structure passes this condition.

Case 6 - Unusual Load Condition (Flood With No Water Inside Structure - ED2B)

Free Body Diagram:

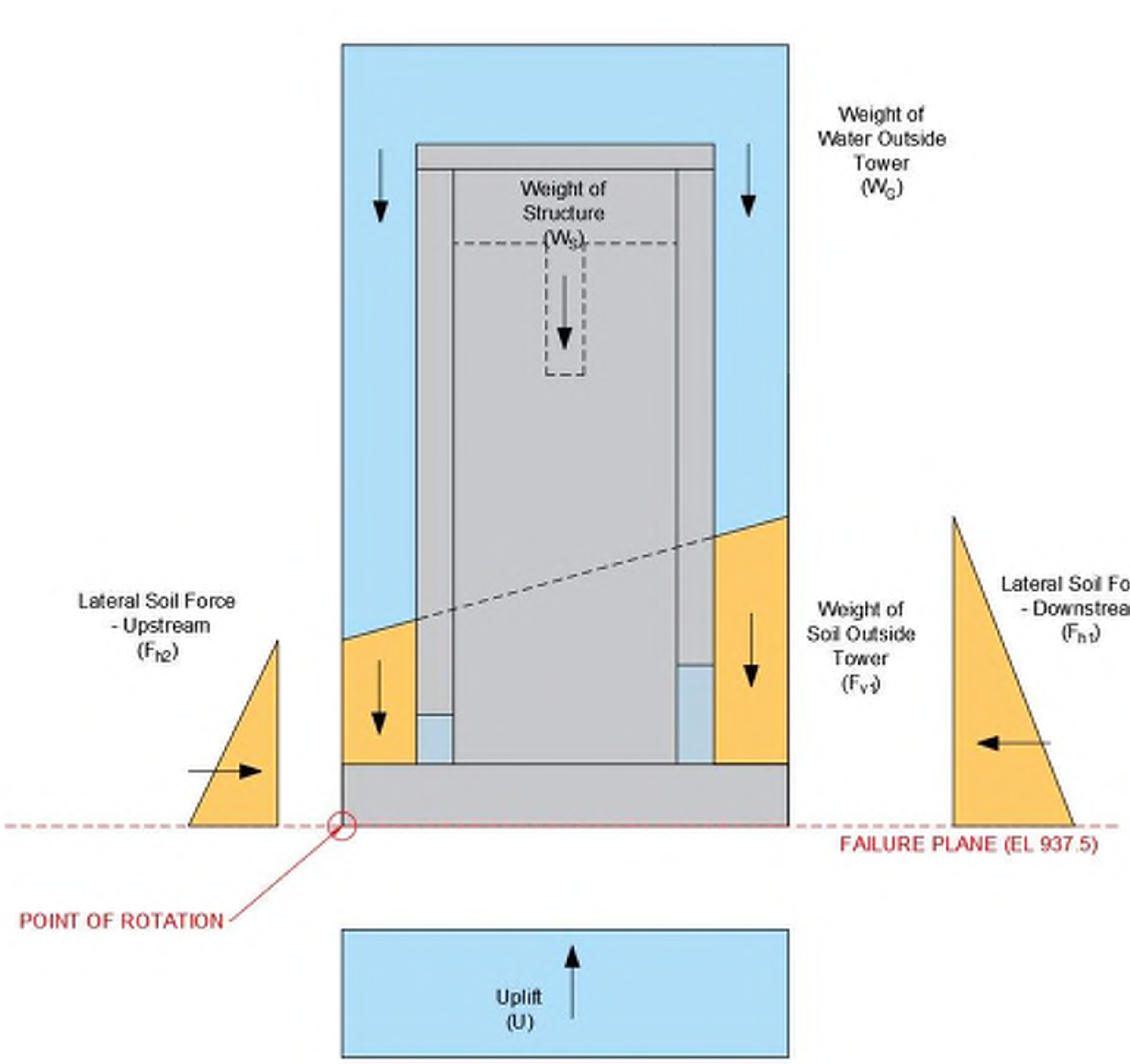


Figure 8: Free Body Diagram - Extreme (Flood) Loading Condition (ED2B)

Calculate Vertical Loads:

(A) Gravity Load of Tower

$W_S = 378.371 \text{ kip}$ Total Weight of Structure

$x_{ws} = 9 \text{ ft}$ Moment Arm

(B) Gravity Load of Water Inside Tower

$W_C := 0 \text{ kip}$ Weight of Water Inside Structure

$x_{wc} := \dfrac{l_{foundation}}{2} = 9 \text{ ft}$ Moment Arm

(C) Gravity Load of Water Above Top Surface of the Structure

$W_{G\_1} := l_o \cdot w_o \cdot \left(EL_{HW\_flood} - EL_{top\_slab}\right) \cdot \gamma_w = 26.283 \text{ kip}$ Weight of Water Above Top of Structure

$x_{wg\_1} := \dfrac{l_{foundation}}{2} = 9 \text{ ft}$ Moment Arm

$W_{G\_2} := \left(l_{foundation} \cdot w_{foundation} - l_o \cdot w_o\right) \cdot \left(EL_{HW\_flood} - EL_{invert}\right) \cdot \gamma_w = 292.144 \text{ kip}$ Weight of Water Around Tower Foundation

$x_{wg\_2} := \dfrac{l_{foundation}}{2} = 9 \text{ ft}$ Moment Arm

$W_G := W_{G\_1} + W_{G\_2} = 318.427 \text{ kip}$ Weight of Water Outside Tower

$x_{wg} := \dfrac{W_{G\_1} \cdot x_{wg\_1} + W_{G\_2} \cdot x_{wg\_2}}{W_G} = 9 \text{ ft}$ Moment Arm

(D) Uplift Force at Concrete/Foundation Interface

$U := H_{HW\_flood} \cdot l_{foundation} \cdot w_{foundation} \cdot \gamma_w = 529.027 \text{ kip}$ Uplift Force Under Tower

$x_u := \dfrac{l_{foundation}}{2} = 9 \text{ ft}$ Moment Arm

(E) Gravity Load of Soil Around Structure Foundation

$F_{v1\_1} := \left(\gamma_{sat\_fill} - \gamma_w\right) \cdot \left(\left(H_{us\_soil} + H_{ds\_soil}\right) \cdot 0.5 - t_{foundation}\right) \cdot \left(w_{foundation} - w_o\right) \cdot l_{foundation} = 53.136 \text{ kip}$ Weight of Soil on Left/Right Sides of Tower Foundation

$x_{v1\_1} := \dfrac{2 \cdot \left(H_{ds\_soil} - t_{foundation}\right) + \left(H_{us\_soil} - t_{foundation}\right)}{H_{ds\_soil} + H_{us\_soil} - t_{foundation} \cdot 2} \cdot \dfrac{l_{foundation}}{3} = 10 \text{ ft}$ Moment Arm

$F_{v1\_2} := (\gamma_{sat\_fill} - \gamma_w) \cdot (H_{ds\_soil} - t_{foundation}) \cdot (l_{foundation} - l_o) \cdot w_o \cdot 0.5 = 17.712\ \text{kip}$ Weight of Soil on Downstream Side of Tower Foundation

$x_{v1\_2} := l_{foundation} - (l_{foundation} - l_o) \cdot 0.5 \cdot 0.5 = 16.5\ \text{ft}$ Moment Arm

$F_{v1\_3} := (\gamma_{sat\_fill} - \gamma_w) \cdot (H_{us\_soil} - t_{foundation}) \cdot (l_{foundation} - l_o) \cdot w_o \cdot 0.5 = 8.856\ \text{kip}$ Weight of Soil on Upstream Side of Tower Foundation

$x_{v1\_3} := (l_{foundation} - l_o) \cdot 0.5 \cdot 0.5 = 1.5\ \text{ft}$ Moment Arm

$F_{v1} := F_{v1\_1} + F_{v1\_2} + F_{v1\_3} = 79.704\ \text{kip}$ Weight of Soil Around Tower Foundation

$x_{v1} := \frac{F_{v1\_1} \cdot x_{v1\_1} + F_{v1\_2} \cdot x_{v1\_2} + F_{v1\_3} \cdot x_{v1\_3}}{F_{v1}} = 10.5\ \text{ft}$ Moment Arm

Calculate Horizontal Loads:

(A) Downstream Lateral Soil Forces

$F_{h1} := \frac{1}{2} \cdot (\gamma_{sat\_fill} - \gamma_w) \cdot {H_{ds\_soil}}^2 \cdot K_o \cdot w_{foundation} = 37.281\ \text{kip}$ Downstream Lateral Earth Pressure

$x_{h1} := \frac{H_{ds\_soil}}{3} = 4.167\ \text{ft}$ Moment Arm

(B) Upstream Lateral Soil Forces

$F_{h2} := \frac{1}{2} \cdot (\gamma_{sat\_fill} - \gamma_w) \cdot {H_{us\_soil}}^2 \cdot K_o \cdot w_{foundation} = 13.421\ \text{kip}$ Upstream Lateral Earth Pressure

$x_{h2} := \frac{H_{us\_soil}}{3} = 2.5\ \text{ft}$ Moment Arm

Overturning Stability Check

$M_{OT} := U \cdot x_u + F_{h1} \cdot x_{h1} = 4916.584 \text{ kip} \cdot \text{ft}$

Total Overturning Moment

$M_R := W_S \cdot x_{ws} + W_C \cdot x_{wc} + W_G \cdot x_{wg} + F_{v1} \cdot x_{v1} + F_{h2} \cdot x_{h2} = 7141.631 \text{ kip} \cdot \text{ft}$

Total Restoring Moment

$FS_{Overturning} := \frac{M_R}{M_{OT}} = 1.453$

Factor of Safety Against Overturning (Note overturning stability is evaluated based on location of resultant, see below)

Resultant Location:

$F_N := W_S + W_C + W_G - U + F_{v1} = 247.475 \text{ kip}$

Total Vertical Load Resisting Overturning

$X := \frac{M_R - M_{OT}}{F_N} = 8.991 \text{ ft}$

Center of Total Weight from Edge of Toe

$e_{cc} := \frac{l_{foundation}}{2} - X = 0.009 \text{ ft}$

Eccentricity of the Resultant

$\frac{l_{foundation}}{6} = 3 \text{ ft}$

Limit for Base Being in Compression Only

```
Resultant_Location := | if |e_cc| ≤ l_foundation/6
                      |    || "100% of Base in Compression"
                      | else if l_foundation/6 < |e_cc| ≤ l_foundation/4
                      |    || "75% of Base in Compression"
                      | else if l_foundation/4 < |e_cc| ≤ l_foundation/2
                      |    || "Resultant Within Base"
                      | else
                      |    || "Unstable-Resultant Outside Base"
```

Resultant_Location = “100% of Base in Compression”

The resultant location is within the middle third of the base and thus the base is 100% in compression. The requirement for overturning for Case 6 is the resultant force should within the base (EM 1110-2-2100). Meets the requirements.

Check Stresses at the Base

$$q_{max} := \begin{array}{l} \text{if } \left|e_{cc}\right| \le \dfrac{l_{foundation}}{6} \\ \quad \left\| \dfrac{F_N}{w_{foundation} \cdot l_{foundation}} \cdot \left(1 + \dfrac{6 \cdot e_{cc}}{l_{foundation}}\right) \right. \\ \text{else} \\ \quad \left\| \dfrac{4 \cdot F_N}{3 \cdot w_{foundation} \cdot \left(l_{foundation} - 2 \cdot e_{cc}\right)} \right. \end{array}$$

$q_{max} = 6.384 \text{ psi}$ $q_{max} = 919 \text{ psf}$

Equations 3-1 and 3-2 from EM 1110-2-2502

$$q_{min} := \begin{array}{l} \text{if } \left|e_{cc}\right| \le \dfrac{l_{foundation}}{6} \\ \quad \left\| \dfrac{F_N}{w_{foundation} \cdot l_{foundation}} \cdot \left(1 - \dfrac{6 \cdot e_{cc}}{l_{foundation}}\right) \right. \\ \text{else} \\ \quad \left\| 0 \text{ psi} \right. \end{array}$$

$q_{min} = 6.346 \text{ psi}$ $q_{min} = 914 \text{ psf}$

$$Check_{bearing} := \begin{array}{l} \text{if } \sigma_{allow} > q_{max} \\ \quad \| \text{“PASS”} \\ \text{else} \\ \quad \| \text{“FAIL”} \end{array}$$

$Check_{bearing} = \text{“PASS”}$

The allowable bearing capacity of the foundation is not exceeded. Additionally, the stresses are significantly less than the assumed strength of concrete and as a result, concrete crushing is not considered to be a viable failure mechanism.

Check Flotation Stability

$$FS_{flotation} := \frac{W_S + W_C}{U - W_G} = 1.8$$

Factor of Safety for Flotation (Equation 3-2 from EM 1110-2-2100)

$$FS_{extreme\_flotation} := 1.1$$

Required Factor of Safety for Flotation (Table 3-4 from EM 1110-2-2100)

$$Check_{flotation} := \text{if } FS_{flotation} \geq FS_{extreme\_flotation} \;\; \| \text{"PASS"} \;\; \text{else} \;\; \| \text{"FAIL"}$$

$Check_{flotation} = \text{"PASS"}$

The requirements for flotation in an Extreme loading condition is a factor of safety of 1.1 (EM 1110-2-2100). The structure passes this condition.

Sliding Stability Check

$F_N = 247.475 \text{ kip}$

Total Vertical Loads Resisting Sliding

$F_D := F_{h1} - F_{h2} = 23.86 \text{ kip}$

Sum of Sliding Force

$$FS_{sliding} := \frac{c \cdot w_{foundation} \cdot l_{foundation} + F_N \cdot \mu_f}{F_D} = 6.232$$

Sliding Factor of Safety (Cohesion conservatively neglected)

$FS_{req\_sliding\_extreme} := 1.1$

Minimum Required Sliding Factor of Safety

$$Check_{sliding} := \text{if } FS_{sliding} > FS_{req\_sliding\_extreme} \;\; \| \text{"PASS"} \;\; \text{else} \;\; \| \text{"FAIL"}$$

$Check_{sliding} = \text{"PASS"}$

The requirements for sliding in an Extreme loading condition is a factor of safety of 1.1 (EM 1110-2-2100). The structure passes this condition.

INTAKE TOWER SEISMIC LOADING

Preliminary Seismic Design - Lake Erin Intake Tower

Reference: USACE EM 1110-2-2400 (Structural Design and Evaluation of Outlet Works, Appendix C)

Calculated by:	NRP
Date:	4/3/2024
Checked by:	IML
Date:	5/3/2024

VARIABLES

Name	Value	Unit	Variable	Notes
Water Mass Density	1.94	slug/ft^3	ρ_w	
Unit Weight of Concrete	150.0	lb/ft^3	γ_c	
Acceleration due to Gravity	32.2	ft/s^2	g	
Probable Ground Acceleration	0.120	g	PGA	
Submerged Outside Height	18.2	ft	H_o	
Submerged Inside Height	15.7	ft	H_i	**Load Case Assumes No Water Inside**
Outside Length of Average Section	12.0	ft	a_o	
Outside Width of Average Section	9.0	ft	b_o	
Outside Area of Average Section	108.00	ft^2	A_o	
Inside Length of Average Section	9.00	ft	a_i	
Inside Width of Average Section	6.00	ft	b_i	
Inside Area of Average Section	54.00	ft^2	A_i	
Length of Base Section	18.0	ft		
Width of Base Section	15.0	ft		
Total Height of Tower	27.5	ft	L	
Modulus of Elasticity of Concrete	449,568.0	ksf	E	

Tower Section Properties and Mass (Table C-5)

Node #	Distance from Base z [ft]	Elem #	Length [ft]	Section Area [ft^2]	Ixx [ft^4]	Iyy [ft^4]	Distributed Mass due to Self-Weight [$k\text{-}s^2/ft^2$]	Mass due to Self-Weight [$k\text{-}s^2/ft^2$]
10	27.5							0.25
		9	1	108.00	1,296.00	729.00	0.50	
9	26.5							0.63
		8	3	54.00	931.50	567.00	0.25	
8	23.5							0.75
		7	3	54.00	931.50	567.00	0.25	
7	20.5							0.67
		6	2.3	54.00	931.50	567.00	0.25	
6	18.2							0.75
		5	3.7	54.00	931.50	567.00	0.25	
5	14.5							0.97
		4	4	54.00	931.50	567.00	0.25	
4	10.5							1.01
		3	4	54.00	931.50	567.00	0.25	
3	6.5							1.01
		2	4	54.00	931.50	567.00	0.25	
2	2.5							2.08
		1	2.5	270.00	7,290.00	5,062.50	1.26	
1	0							1.57

TRANSVERSE DIRECTION

Approximate Added Hydrodynamic Mass Calculations, Outside Water, Transverse Direction (Table C-1)

Node #	Distance from Base z [ft]	z / H_o	Elem #	Depth a_0 [ft]	Width b_o [ft]	a_o/b_o	r_o [ft]	r_o/H_o	m^0_a / m^a_∞	$\rho_w A_o$ [$k\text{-}s^2/ft^2$]	$m^a_\infty / \rho_w A_o$	m^a_∞ [$k\text{-}s^2/ft^2$]	length [ft]	m^0_a Distributed Hydrodynamic Added Mass [$k\text{-}s^2/ft^2$]	Lumped Hydrodynamic Added Mass [$k\text{-}s^2/ft^2$]
10	27.5														
9	26.5														
8	23.5														
7	20.5														
6	18.2	1.00													0.36
			5	12.0	9.0	1.33	5.86	0.32	0.61	0.21	1.53	0.32	3.70	0.20	
5	14.5	0.80													0.88
			4	12.0	9.0	1.33	5.86	0.32	0.80	0.21	1.53	0.32	4.00	0.26	
4	10.5	0.58													1.08
			3	12.0	9.0	1.33	5.86	0.32	0.88	0.21	1.53	0.32	4.00	0.28	
3	6.5	0.36													1.17
			2	12.0	9.0	1.33	5.86	0.32	0.95	0.21	1.53	0.32	4.00	0.30	
2	2.5	0.14													1.00
			1	12.0	9.0	1.33	5.86	0.32	0.98	0.21	1.53	0.32	2.50	0.31	
1	0	0.00													0.39

Approximate Added Hydrodynamic Mass Calculations, Inside Water, Transverse Direction (Table C-2)

Node #	Distance from Base z	z / H_i	Elem #	Depth a_i	Width b_i	a_i/b_i	r_i	r_i/H_i	m^i_a / m^i_∞	$\rho_w A_i$	$m^i_\infty / \rho_w A_i$	m^i_∞	length	m^i_a Distributed Hydrodynamic Added Mass	Lumped Hydrodynamic Added Mass
	[ft]			[ft]	[ft]		[ft]			[k-s^2/ft^2]		[k-s^2/ft^2]	[ft]	[k-s^2/ft^2]	[k-s^2/ft^2]
10	27.5														
9	26.5														
8	23.5														
7	20.5														
6	18.2	1.16													0.20
			5	9.0	6.0	1.50	4.15	0.26	0.61	0.10	1.69	0.18	3.70	0.11	
5	14.5	0.92													0.48
			4	9.0	6.0	1.50	4.15	0.26	0.80	0.10	1.69	0.18	4.00	0.14	
4	10.5	0.67													0.60
			3	9.0	6.0	1.50	4.15	0.26	0.88	0.10	1.69	0.18	4.00	0.16	
3	6.5	0.41													0.65
			2	9.0	6.0	1.50	4.15	0.26	0.95	0.10	1.69	0.18	4.00	0.17	
2	2.5	0.16													0.34
			1	0.0	0.0	1.00	0.00	0.00	0.98	0.00	1.20	0.00	2.50	0.00	
1	0	0.00													0.00

Total Mass for Earthquake Motions in Transverse Direction (Table C-7)

Node #	Distance from Base z	Mass due to Self-Weight	Inside Hydrodynamic Added Mass	Outside Hydrodynamic Added Mass	Total Lumped Mass
	[ft]	[k-s^2/ft^2]	[k-s^2/ft^2]	[k-s^2/ft^2]	[k-s^2/ft^2]
10	27.5	0.25			0.25
9	26.5	0.63			0.63
8	23.5	0.75			0.75
7	20.5	0.67			0.67
6	18.2	0.75	0.00	0.36	1.12
5	14.5	0.97	0.00	0.88	1.84
4	10.5	1.01	0.00	1.08	2.08
3	6.5	1.01	0.00	1.17	2.18
2	2.5	2.08	0.00	1.00	3.08
1	0.0	1.57	0.00	0.39	1.97

STEP 1: Determine First and Second Shape Functions

I_{base} =	567.00	**1.00**	<---Ratio
I_{top} =	567.00		Use this ratio to determine first and second shape functions from Tables B-1 and B-2

STEP 2: Determine First and Second Natural Periods of Vibration

k* =	3.09	Stiffness coefficient for 1st shape function
k* =	121.40	Stiffness coefficient for 2nd shape function
k* =	30,309	Stiffness for 1st shape function (kip/ft)
k* =	1,190,389	Stiffness for 2nd shape function (kip/ft)

STEP 4: Shears and Moments

Forces, Shears, and Moments for Transverse Excitation, First Shape Function

	h_j		m_j	L_n	m*	Y_n	F_n	V_n		M_n
Node #	**Height**	**Mode Shape**	**Lumped Mass**	**ϕ_{j1} x m_j**	**ϕ_{j1}^2 x m_j**	**Lateral Displacement**	**Elastic Force**	**Shear**	**h_j x f_{j1}**	**Moment**
	[ft]					[in]	[kips]	[kips]	[kip-ft]	[kip-ft]
10	27.5	1.00	0.25	0.25	0.25	0.0059	1.65	1.65	45.31	0.00
9	26.5	0.95	0.63	0.60	0.57	0.0056	3.91	5.56	103.68	1.65
8	23.5	0.80	0.75	0.60	0.48	0.0047	3.95	9.51	92.89	18.33
7	20.5	0.65	0.67	0.43	0.28	0.0039	2.85	12.36	58.35	46.87
6	18.2	0.54	1.12	0.60	0.33	0.0032	3.96	16.32	72.08	75.29
5	14.5	0.37	1.84	0.69	0.26	0.0022	4.51	20.83	65.33	135.67
4	10.5	0.21	2.08	0.44	0.09	0.0013	2.91	23.73	30.53	218.97
3	6.5	0.09	2.18	0.20	0.02	0.0005	1.29	25.02	8.37	313.90
2	2.5	0.02	3.08	0.05	0.00	0.0001	0.31	25.33	0.78	413.99
1	0.0	0.00	1.97	0.00	0.00	0.0000	0.00	25.33	0.00	477.32
			Σ	**3.87**	**2.28**					

L_1 = 3.87 **Normalization Factor**
M_1 = 2.28 **Effective Mass**
L_1 / M_1 = 1.70 **Normalization Ratio**

T_1 = 0.05 **Natural Period** s
ω_1 = 115.25 **Natural Frequency** rad/s
STEP 3 S_A = 3.86 **Spectral Acceleration** ft/s^2
S_d = 0.0003 **Pseudo-Displacement**

STEP 4: Shears and Moments at each lumped mass

Forces, Shears, and Moments for Transverse Excitation, Second Shape Function

	h_j		m_j	L_n	m*	Y_n	F_n	V_n		M_n
Node #	**Height**	**Mode Shape**	**Lumped Mass**	**ϕ_{j2} x m_j**	**ϕ_{j2}^2 x m_j**	**Lateral Displacement**	**Elastic Force**	**Shear**	**h_j x f_{j1}**	**Moment**
	[ft]					[in]	[kips]	[kips]	[kip-ft]	[kip-ft]
10	27.5	1.00	0.25	0.25	0.25	-0.0001	-1.04	-1.04	-28.57	0.00
9	26.5	0.83	0.63	0.52	0.43	-0.0001	-2.15	-3.19	-56.92	-1.04
8	23.5	0.32	0.75	0.24	0.08	0.0000	-0.99	-4.18	-23.27	-10.60
7	20.5	(0.14)	0.67	-0.09	0.01	0.0000	0.39	-3.79	7.96	-23.13
6	18.2	(0.42)	1.12	-0.47	0.20	0.0001	1.94	-1.85	35.33	-31.84
5	14.5	(0.68)	1.84	-1.25	0.85	0.0001	5.18	3.33	75.10	-38.68
4	10.5	(0.65)	2.08	-1.36	0.89	0.0001	5.64	8.97	59.17	-25.35
3	6.5	(0.38)	2.18	-0.83	0.32	0.0000	3.45	12.42	22.42	10.52
2	2.5	(0.08)	3.08	-0.26	0.02	0.0000	1.07	13.49	2.69	60.19
1	0.0	0.00	1.97	0.00	0.00	0.0000	0.00	13.49	0.00	93.92
			Σ	**-3.27**	**3.06**					

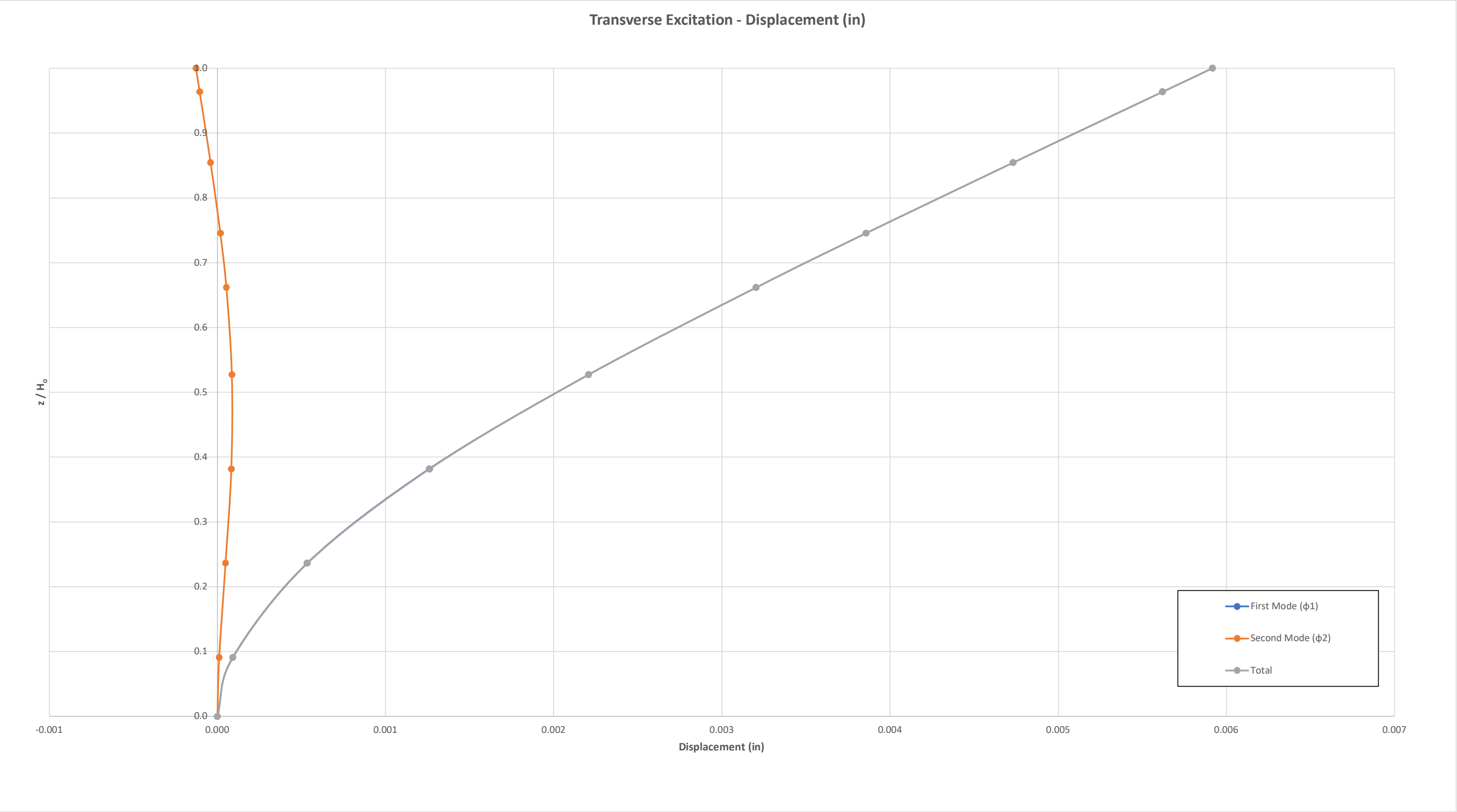
L_1 = (3.27) **Normalization Factor**
M_1 = 3.06 **Effective Mass**
L_1 / M_1 = (1.07) **Normalization Ratio**

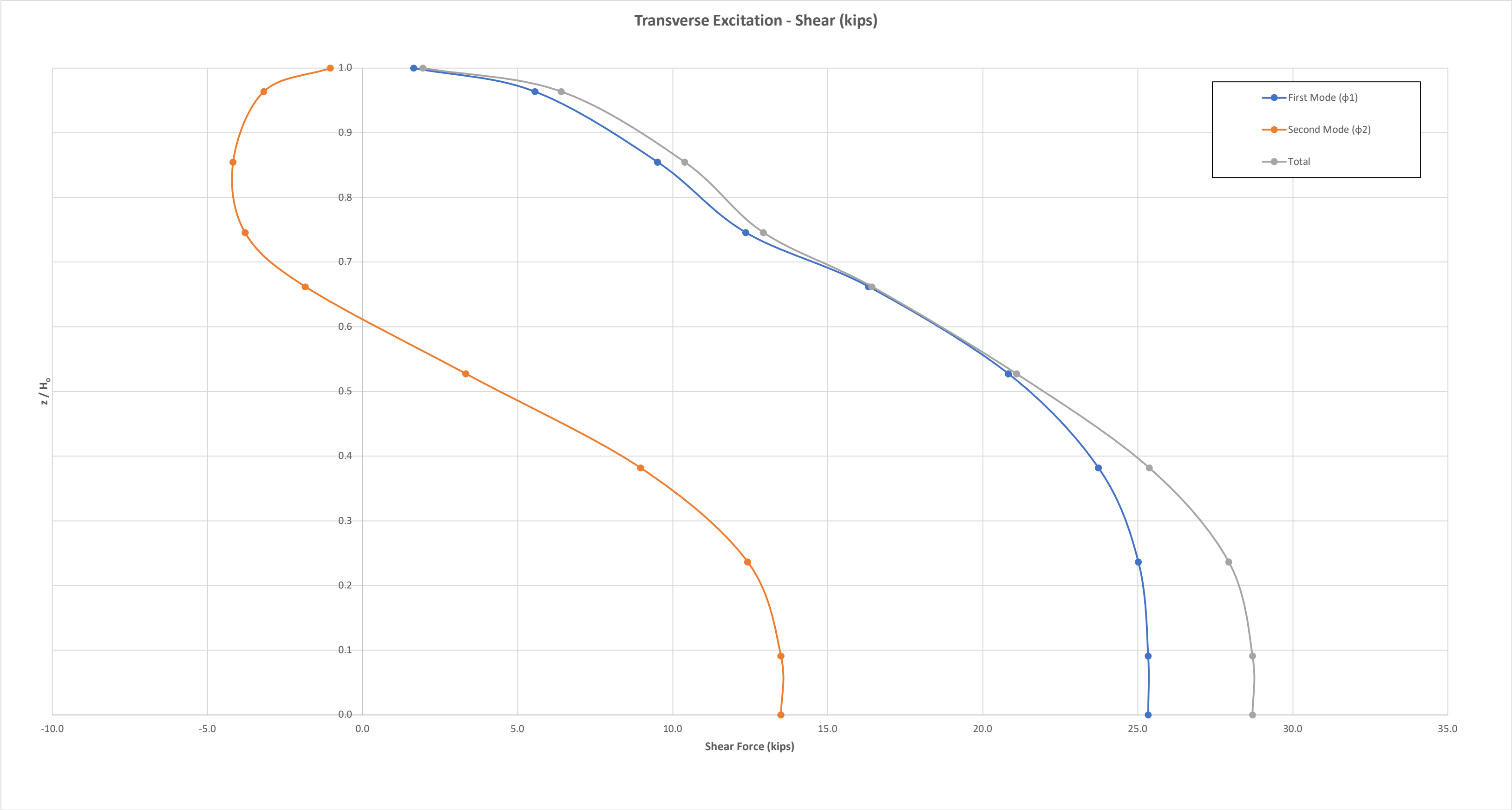
T_1 = 0.01 **Natural Period** s
ω_1 = 624.12 **Natural Frequency** rad/s
STEP 3 S_A = 3.86 **Spectral Acceleration** ft/s^2
S_d = 0.0000 **Pseudo-Displacement**

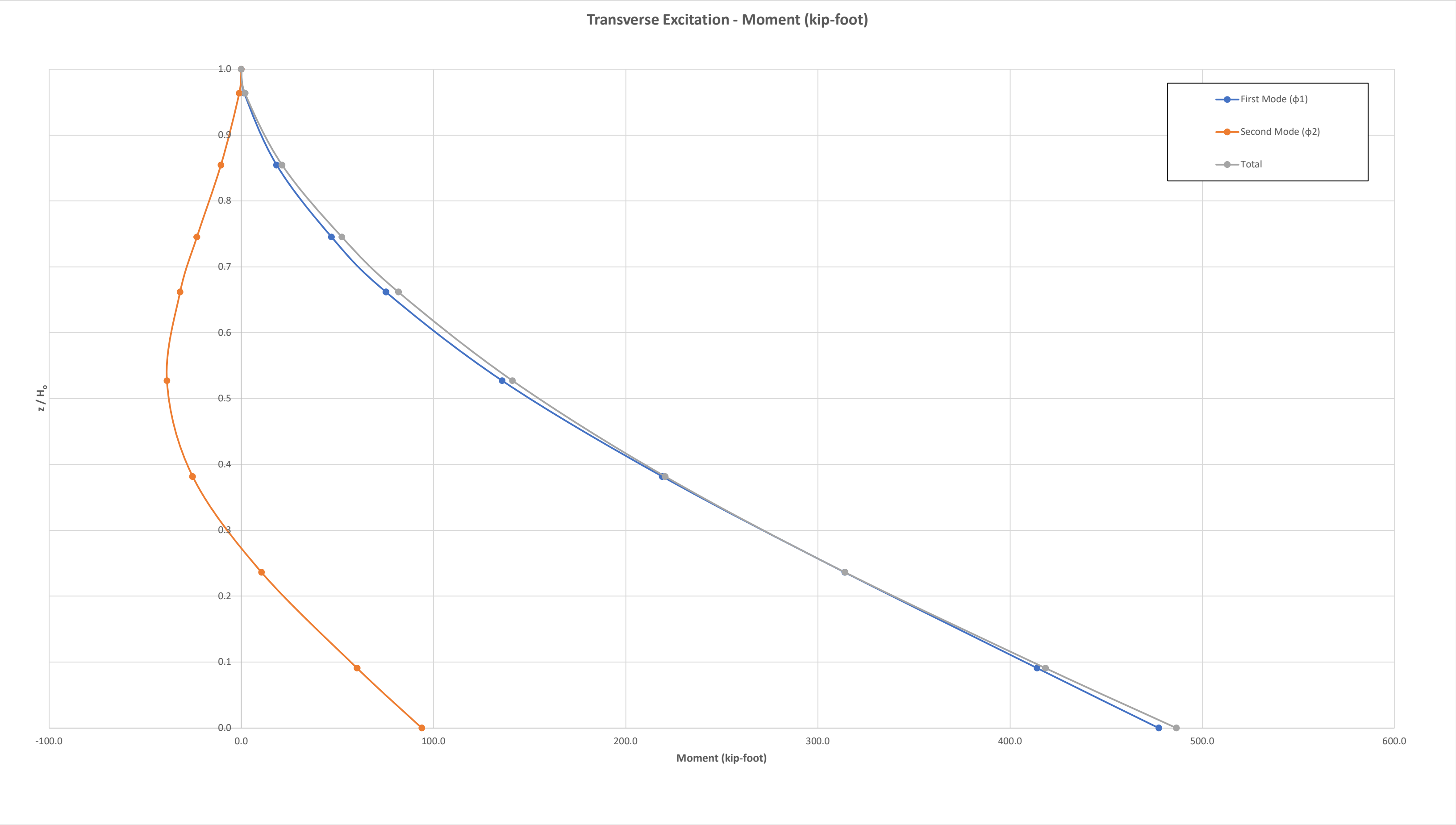
STEP 5: Find Base Shears, Base Moments, and Displacement of Top Mass using Square Root Sum of Squares (SRSS)

Δ_{top} =	**0.0059**	inches
V_b =	**28.7**	kips
M_b =	**486.5**	kip-feet

	z		Y_n	V_n	M_n
Node #	**Distance from Base**	**z / H_o**	**Lateral Displacement**	**Shear**	**Moment**
	[ft]		[in]	[kips]	[kip-ft]
10	27.5	1.00	**0.0059**	1.9	0.0
9	26.5	0.96	0.0056	6.4	1.9
8	23.5	0.85	0.0047	10.4	21.2
7	20.5	0.75	0.0039	12.9	52.3
6	18.2	0.66	0.0032	16.4	81.7
5	14.5	0.53	0.0022	21.1	141.1
4	10.5	0.38	0.0013	25.4	220.4
3	6.5	0.24	0.0005	27.9	314.1
2	2.5	0.09	0.0001	28.7	418.3
1	0.0	0.00	0.0000	**28.7**	**486.5**


Transverse Excitation - Displacement (in)
z / H$_o$
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.0
-0.001
0.000
0.001
0.002
0.003
0.004
0.005
0.006
0.007
Displacement (in)
First Mode (ϕ1)
Second Mode (ϕ2)
Total


Transverse Excitation - Shear (kips)
First Mode (ϕ1)
Second Mode (ϕ2)
Total
z / H$_o$
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.0
-10.0
-5.0
0.0
5.0
10.0
15.0
20.0
25.0
30.0
35.0
Shear Force (kips)


Transverse Excitation - Moment (kip-foot)
z / H_o
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.0
-100.0
0.0
100.0
200.0
300.0
400.0
500.0
600.0
Moment (kip-foot)
First Mode (ϕ1)
Second Mode (ϕ2)
Total

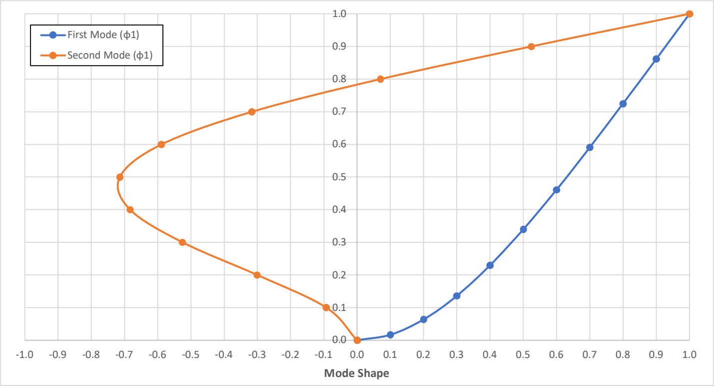
k* (1st) 3.091
k* (2nd) 121.4

Longtitudinal Direction				
Node	Distance from Base	Z	ϕ_1	ϕ_2
10	27.5	1.00	1.000	1.000
9	26.5	0.96	0.950	0.827
8	23.5	0.85	0.800	0.318
7	20.5	0.75	0.652	-0.141
6	18.2	0.66	0.541	-0.421
5	14.5	0.53	0.373	-0.680
4	10.5	0.38	0.213	-0.654
3	6.5	0.24	0.090	-0.383
2	2.5	0.09	0.015	-0.085
1	0.0	0.00	0.000	0.000

From Tables B-1 and B-2

I_{base} / I_{top} = 1

Z	First Mode (ϕ_1)	Second Mode (ϕ_1)
1.00	1.000	1.000
0.90	0.862	0.524
0.80	0.725	0.070
0.70	0.591	-0.317
0.60	0.461	-0.589
0.50	0.340	-0.714
0.40	0.230	-0.683
0.30	0.136	-0.526
0.20	0.064	-0.301
0.10	0.017	-0.093
0.00	0.000	0.000



k* (1st) 3.091
k* (2nd) 121.4

Transverse Direction				
Node	Distance from Base	Z	ϕ_1	ϕ_2
10	27.5	1.00	1.000	1.000
9	26.5	0.96	0.950	0.827
8	23.5	0.85	0.800	0.318
7	20.5	0.75	0.652	-0.141
6	18.2	0.66	0.541	-0.421
5	14.5	0.53	0.373	-0.680
4	10.5	0.38	0.213	-0.654
3	6.5	0.24	0.090	-0.383
2	2.5	0.09	0.015	-0.085
1	0.0	0.00	0.000	0.000

INTAKE TOWER WIND LOADING

Wind Load Calculator for Proposed Lake Erin Intake Tower

Reference: ASCE 7-22 Chapter 26, 27 (Wind Load)

VARIABLES

	Step	Variable	Value	Unit	Notes
Step 1)	Risk Category of Structure		IV		Table 1.5-1
Step 2)	Basic Wind Speed	V	116	mph	Figure 26.5-1D
Step 3)	Wind Directionality Factor	K_d	0.85		Section 26.6
	Exposure Category		D		Section 26.7
	Topographic Factor	K_{zt}	1.0		Section 26.8
	Ground Elevation Factor	K_e	1.0		Section 26.9
	Gust-Effect Factor	G	0.85		Section 26.11
	Enclosure Classification		Enclosed		Section 26.12
	Internal Pressure Coefficient	GC_{pi}	0.18		Section 26.13
Step 4)	Determine Velocity Pressure Coefficient (See Table to the Right)	K_z			Table 26.10-1
Step 5)	Determine Velocity Pressure (See Table to the Right)	q_z			Equation 26.10-1
Step 6)	Determine External Pressure Coefficients	C_p	0.8		Figure 27.3-1 (Windward Wall)
		C_p	(0.5)		Figure 27.3-1 (Leeward Wall)
		C_p	(0.7)		Figure 27.3-1 (Roof)
Step 7)	Calculate Wind Pressure (See Table to Right)	p		psf	Equation 28.3-1

Construction Load Case

z or h	*K_z*	*q_z or q_h*	*p*	*B*	*F*	
Wall Height	**Velocity Pressure Coefficient**	**Velocity Pressure**	**Wind Pressure**	**Width**	**Force**	**Moment Arm**
[ft]		[psf]	[psf]	[ft]	[lbf]	[ft]
Windward Wall						
15	1.03	35.48	24.13	12	4,343	7.50
20	1.08	37.20	25.30	12	1,518	17.50
25	1.12	38.58	26.24	16	2,099	22.50
27.5	1.16	39.96	27.17	19.5	1,325	26.25
			17.31		**9,284**	**15.20**
Leeward Wall						
27.5	1.16	39.96	(16.98)	19.5	(9,107)	13.75
Roof						
27.5	1.16	39.96	(23.78)	19.5	(12,750)	-

Normal Load Combination

z or h	*K_z*	*q_z or q_h*	*p*	*B*	*F*	
Wall Height	**Velocity Pressure Coefficient**	**Velocity Pressure**	**Wind Pressure**	**Width**	**Force**	**Moment Arm**
[ft]		[psf]	[psf]	[ft]	[lbf]	[ft]
Windward Wall						
11	1.03	35.48	24.13	19.5	5,175	5.50
Leeward Wall						
11	1.03	35.48	(15.08)	19.5	(3,235)	5.50
Roof						
11	1.03	35.48	(21.11)	19.5	(4,528)	5.50

INTAKE TOWER TOP SLAB

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Calculated By: NRP
Date: 5/13/2024
Checked By: IML
Date: 5/29/2024
Project Number: 60727041

Project: Lake Erin Dam Rehabilitation

Task: Top Slab Analysis - Intake Tower

Description:

Design the Intake Tower top slab in accordance with ACI 350-20, Code Requirements for Environmental Structures.

Codes, Standards, & References:

1. ACI 350-20, Code Requirements for Environmental Structures
2. ACI 318-19, Building Code Requirements for Structural Concrete
3. AECOM Basis of Design Report
4. USACE EM 1110-2-2400, Structural Design and Evaluation of Outlet Works

Assumptions:

- Conservatively design the base slab as a one-way slab spanning between the sidewalls.
- Design service deck for uniform live load of 250 psf per USACE 1110-2-2400, Section 6-3.
- Check for punching shear capacity from sluice gate operator in accordance with ACI 318-19, Section 22.
- Reinforcement assumed to be #6 bars spaced 12" OC.

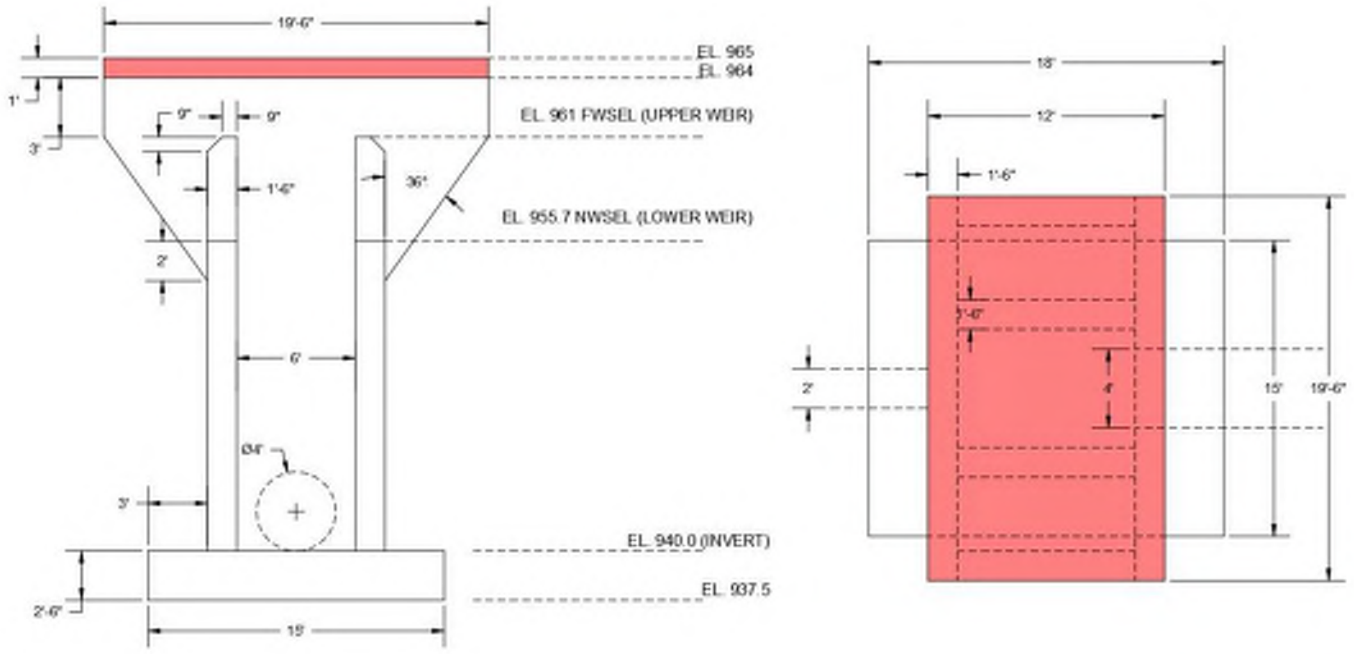

Figure 1 - Intake Tower Plan/Elevation Sections and Top Slab (In Red)

Material Properties

$\gamma_c := 150\ \text{pcf}$	Unit Weight of Concrete
$f'_c := 5000\ \text{psi}$	Concrete Compressive Strength
$f_y := 60000\ \text{psi}$	Yield Strength of Reinforcement
$\gamma_w := 62.4\ \text{pcf}$	Unit Weight of Water

Top Slab Details & Geometry

$h := 12\ \text{in}$	Thickness of Top Slab
$b := 12\ \text{in}$	Unit Width of Top Slab
$L := 12\ \text{ft}$	Length of Top Slab
$t := 18\ \text{in}$	Thickness of Side Walls
$L_b := L - t = 10.5\ \text{ft}$	Effective Length of Top Slab

Check for Punching Shear from Sluice Gate Stem

$H := 25\ \text{ft}$	Head from Top of Slab to Invert
$L_{gate} := 20\ \text{in}$	Gate Opening Size (Square Sluice Gate)
$F := 0.35 \cdot H \cdot \gamma_w \cdot L_{gate}{}^2 = 1.517\ \text{kip}$	Force Required to Overcome Friction Component of Water Load (Rodney Hunt)
$c := 10\ \text{in}$	Flood Stand Dimension (Rodney Hunt - S-5002.5)
$\alpha_s := 40$	Value for Interior Columns (ACI 318-19, 22.6.5.3)
$\lambda_s := \max\left(1, \sqrt{\dfrac{2}{1+\dfrac{h}{10\ \text{in}}}}\right) = 1$	Size Effect Modification Factor (ACI 318-19, 22.5.5.1.3)
$\beta := 1$	Ratio of Sides of Reaction Area (Assume Square Base Plate)
$b_o := 2 \cdot (c+h) + 2 \cdot (c+h) = 7.333\ \text{ft}$	Perimeter of Critical Section
$V_{c1} := 4 \cdot \lambda_s \cdot \sqrt{f'_c \cdot \text{psi}} \cdot b_o \cdot h = 298.682\ \text{kip}$	Punching Shear Capacity, Equation A (ACI 318-19, Table 22.6.5.2)

$V_{c2} := \left(2 + \frac{4}{\beta}\right) \cdot \lambda_s \cdot \sqrt{f'_c \cdot psi} \cdot b_o \cdot h = 448.023 \text{ kip}$

Punching Shear Capacity, Equation B (ACI 318-19, Table 22.6.5.2)

$V_{c3} := \left(2 + \frac{\alpha_s \cdot h}{b_o}\right) \cdot \lambda_s \cdot \sqrt{f'_c \cdot psi} \cdot b_o \cdot h = 556.634 \text{ kip}$

Punching Shear Capacity, Equation C (ACI 318-19, Table 22.6.5.2)

$V_c := min\left(V_{c1}, V_{c2}, V_{c3}\right) = 298.682 \text{ kip}$

Punching Shear Capacity

$Check_{punch} :=$ if $1.6 \cdot F \le 0.75 \cdot V_c$

“PASS”

else

“FAIL”

$Check_{punch} =$ “PASS”

Determine Reactions on Base Slab

Load Combination: U = 1.2D + 1.6L (LRFD) (ACI 350-20 Section 9.2.1, Equation 9-2)

Calculate Dead Load:

$\phi_{DL} := 1.2$

Dead Load Factor

$w_{u1} := \gamma_c \cdot b \cdot h = 0.15 \text{ klf}$

Dead Load of Top Slab

$\phi_{DL} \cdot w_{u1} = 0.18 \text{ klf}$

Factored Dead Load

Calculate Live Load:

$LL := 250 \text{ psf}$

Assumed Live Load

$\phi_{LL} := 1.6$

Live Load Factor

$w_{u2} := LL \cdot b = 0.25 \text{ klf}$

Live Load on Top Slab

$\phi_{LL} \cdot w_{u2} = 0.4 \text{ klf}$

Factored Live Load

$w_u := \phi_{DL} \cdot w_{u1} + \phi_{LL} \cdot w_{u2} = 0.58 \text{ klf}$

Total Load Combination

Calculate Maximum Moment/Shear:

$M := \frac{w_{u1} \cdot L^2}{8} + \frac{w_{u2} \cdot L^2}{8} = 7.2\ \text{kip} \cdot \text{ft}$ Service Moment

$M_u := \frac{w_u \cdot L^2}{8} = 10.44\ \text{kip} \cdot \text{ft}$ Factored Moment

$V_u := \frac{w_u \cdot L}{2} = 3.48\ \text{kip}$ Factored Shear

Intakee Tower Top Slab Concrete Reinforcement Design Per ACI 350-20

Rectangular Section in Flexure:

$h = 1\ \text{ft}$ Depth of Section

$d_b := 0.75\ \text{in}$ No. 6 Bar Diameter (Assumed Bar Size)

$A_b := \frac{\pi \cdot {d_b}^2}{4} = 0.442\ \text{in}^2$ Area of #6 Bar

$c_c := 2\ \text{in}$ Cover to Reinforcement

$s := 12\ \text{in}$ Bar Spacing

$d := h - c_c - \frac{d_b}{2} = 9.625\ \text{in}$ Depth to Reinforcement

$A_{s\_prov} := A_b \cdot \frac{b}{s} = 0.442\ \text{in}^2$ Total Area of Reinforcement Provided

$a := \frac{A_{s\_prov} \cdot f_y}{0.85 \cdot f'_c \cdot b} = 0.52\ \text{in}$ Depth of Equivalent Stress Block

Calculate Net Tensile Strain:

$\beta_1 := 0.85 - .00005 \cdot \frac{\text{in}^2}{\text{lbf}} \cdot \left(f'_c - 4000\ \text{psi}\right) = 0.8$ Equivalent Depth Factor

$\varepsilon_t := \frac{0.003 \cdot \left(\beta_1 \cdot d - a\right)}{a} = 0.041$ Net Tensile Strain

$$\phi_{RF} := \left. \begin{array}{l} \text{if } \varepsilon_t \geq 0.005 \\ \quad \| \; 0.9 \\ \text{else} \\ \quad \| \; 0.65 \end{array} \right| = 0.9$$

Strength Reduction Factor

$\varepsilon_t \geq 0.005$ Tension-Controlled Section

Check Flexural Strength:

$$M_n := A_{s\_prov} \cdot f_y \cdot \left(d - \frac{a}{2} \right) = 20.687 \text{ kip} \cdot \text{ft}$$

Nominal Flexural Strength

$$\phi_{RF} \cdot M_n = 18.618 \text{ kip} \cdot \text{ft}$$

Design Flexural Strength

$$\frac{M_u}{\phi_{RF} \cdot M_n} = 0.561$$

Demand Capacity Ratio

$$Check_{Flex} := \left. \begin{array}{l} \text{if } M_u \leq \phi_{RF} \cdot M_n \\ \quad \| \; \text{“PASS”} \\ \text{else} \\ \quad \| \; \text{“FAIL”} \end{array} \right|$$

$Check_{Flex} = \text{“PASS”}$

Check Minimum Area of Reinforcement Required:

$$A_{s\_prov} = 0.442 \text{ in}^2$$

Area of Reinforcement Provided

$$A_{smin} := \max \left(\frac{3 \cdot \sqrt{\frac{f'_c}{psi}} \cdot psi}{f_y} \cdot b \cdot d , \frac{200 \text{ psi} \cdot b \cdot d}{f_y} \right) = 0.408 \text{ in}^2$$

Minimum Area of Reinforcement Required (Section 10.5.1)

$$Check_{Reinforcement} := \left. \begin{array}{l} \text{if } A_{smin} \leq A_{s\_prov} \\ \quad \| \; \text{“PASS”} \\ \text{else} \\ \quad \| \; \text{“FAIL”} \end{array} \right|$$

$Check_{Reinforcement} = \text{“PASS”}$

Rectangular Section in Shear (Per ACI 350-20 Section 11.1):

$\lambda := 1$

Modification Factor for Normalweight Concrete

$\phi_s := 0.75$

Shear Reduction Factor

$V_u = 3.48\ \text{kip}$

Factored Shear Force

$V_c := 2\ \text{psi} \cdot \lambda \cdot \sqrt{\dfrac{f'_c}{\text{psi}}} \cdot b \cdot d = 16.334\ \text{kip}$

Nominal Shear Strength Provided by Concrete

Demand Capacity Ratio

$\dfrac{V_u}{\phi_s \cdot V_c} = 0.284$

```
Check_Shear := if V_u ≤ φ_s • V_c
                 ‖ "PASS"
               else
                 ‖ "FAIL"
```

$\text{Check}_{\text{Shear}} = \text{“PASS”}$

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Date: 5/29/2024
Project Number: 60727041

Project: Lake Erin Dam Rehabilitation

Task: Intake Tower - Design the Wall Reinforcement

Description:

Design the intake structure vertical and horizontal wall reinforcement in accordance with ACI 350-20, Code Requirements for Environmental Structures.

Codes, Standards, & References:

1. ACI 350-20, Code Requirements for Environmental Structures
2. AECOM Basis of Design Report

Assumptions:

- Design the Intake Tower horizontal reinforcement with the assumption that the wall behaves as a simply-supported beam spanning between the sidewalls. Assume flood load case and fully saturated backfill.
- Horizontal reinforcement is #8 bars spaced minimum 12 inches.
- Design the Intake Tower vertical reinforcement on the assumption that the wall behaves as a cantilever wall. This calculation is done in a separate spreadsheet and the relevant forces are calculated in this document.
- Vertical reinforcement is #6 bars spaced minimum 12 inches.

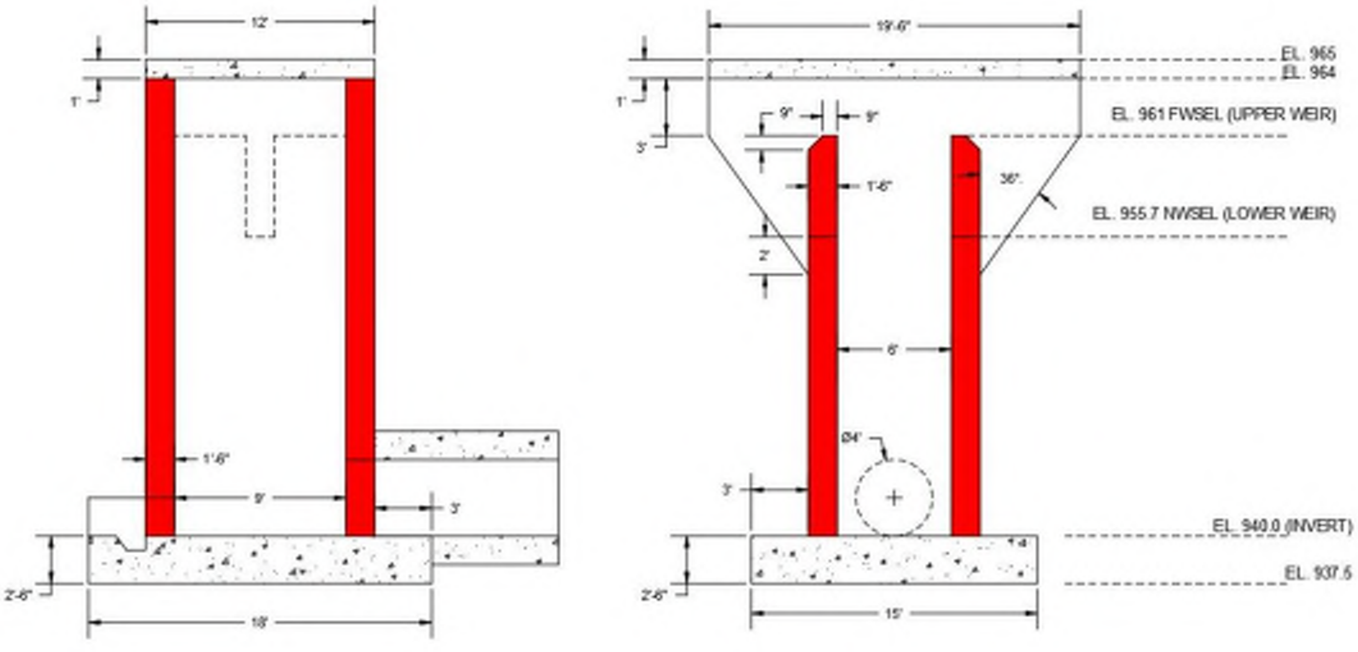


Figure 1 - Section Cross Section and Plan View (Vertical Walls in Red)

Material Properties

** Parameter were provided by Geotechnical Engineers*

$\gamma_c := 150 \text{ pcf}$	Unit Weight of Concrete
$\gamma_w := 62.4 \text{ pcf}$	Unit Weight of Water
$f'_c := 5000 \text{ psi}$	Concrete Compressive Strength
$f_y := 60000 \text{ psi}$	Yield Strength of Reinforcement
$\gamma_{sat\_fill} := 128 \text{ pcf}$	Saturated Unit Weight of Backfill Soil*
$\phi := 31 \text{ deg}$	Effective Friction Angle of Backfill Soil*
$K_o := 1 - \sin(\phi) = 0.485$	At-Rest Earth Pressure Coefficient*

Hydrologic and Hydraulic Parameters

$EL_{HW\_flood} := 969.0 \text{ ft}$	Elevation of Flood Water Elevation
$EL_{HW\_norm} := 955.7 \text{ ft}$	Elevation of Normal Water Elevation

Wall Details & Geometry

$EL_{invert} := 940.0 \text{ ft}$	Elevation of Invert (Top of Foundation Slab)
$EL_{soil} := 950.0 \text{ ft}$	Elevation of Soil
$H_{HW\_flood} := EL_{HW\_flood} - EL_{invert} = 29 \text{ ft}$	Height of Flood Water Above Invert
$H_{HW\_norm} := EL_{HW\_norm} - EL_{invert} = 15.7 \text{ ft}$	Height of Flood Water Above Invert
$H_{soil} := EL_{soil} - EL_{invert} = 10 \text{ ft}$	Height of Soil
$t_{wall} := 18 \text{ in}$	Thickness of Wall
$L_o := 12.0 \text{ ft}$	Outside Length of Wall Face
$L := L_o - t_{wall} = 10.5 \text{ ft}$	Effective Span Length
$b := 12 \text{ in}$	Unit Width

Design Horizontal Reinforcement

Calculate Forces Acting on the Wall:

Conservatively analyze the wall with the assumption that it behaves as a fixed-fixed beam spanning from the corners.

$\phi_{FL} := 1.2$	Fluid Load Factor (ACI 350-20 Section 9.2.1)
$w_{FL} := \gamma_w \cdot H_{HW\_flood} \cdot b = 1.81\ \text{klf}$	Distributed Load on Wall from Water
$M_{FL} := \frac{w_{FL} \cdot L^2}{12} = 16.626\ \text{kip} \cdot \text{ft}$	Service Moment from Water (Max Negative at Ends)
$V_{FL} := w_{FL} \cdot \frac{L}{2} = 9.5\ \text{kip}$	Service Shear from Water
$\phi_S := 1.6$	Soil Load Factor (ACI 350-20 Section 9.2.1)
$w_S := K_o \cdot \left(\gamma_{sat\_fill} - \gamma_w\right) \cdot H_{soil} \cdot b = 0.318\ \text{klf}$	Distributed Load on Wall from Soil
$M_S := \frac{w_S \cdot L^2}{12} = 2.923\ \text{kip} \cdot \text{ft}$	Service Moment from Soil (Max Negative at Ends)
$V_S := w_S \cdot \frac{L}{2} = 1.67\ \text{kip}$	Service Shear from Soil
$M_u := \phi_{FL} \cdot M_{FL} + \phi_S \cdot M_S = 24.627\ \text{kip} \cdot \text{ft}$	Factored Moment
$V_u := \phi_{FL} \cdot V_{FL} + \phi_S \cdot V_S = 14.073\ \text{kip}$	Factored Shear Force

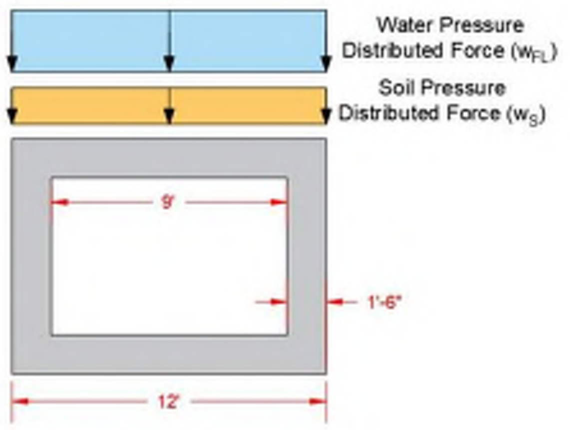


Figure 2 - Horizontal Reinforcement Analysis Free Body Diagram

Horizontal Reinforcement Design Per ACI 350-20

Rectangular Section in Flexure:

$b = 1\ \text{ft}$	Unit Width
$d_b := 1.0\ \text{in}$	No. 6 Bar Diameter (Assumed Bar Size)
$A_b := \frac{\pi \cdot {d_b}^2}{4} = 0.785\ \text{in}^2$	Area of #6 Bar
$h := t_{wall} = 1.5\ \text{ft}$	Depth of Section
$c_c := 2\ \text{in}$	Cover to Reinforcement
$s := 12\ \text{in}$	Bar Spacing
$d := h - c_c - \frac{d_b}{2} = 15.5\ \text{in}$	Depth to Reinforcement
$A_{s\_prov} := \frac{b}{s} \cdot A_b = 0.785\ \text{in}^2$	Total Area of Reinforcement Provided
$a := \frac{A_{s\_prov} \cdot f_y}{0.85 \cdot f'_c \cdot b} = 0.924\ \text{in}$	Depth of Equivalent Stress Block

Calculate Net Tensile Strain:

$\beta_1 := 0.85 - .00005 \cdot \frac{\text{in}^2}{\text{lbf}} \cdot \left(f'_c - 4000\ \text{psi}\right) = 0.8$	Equivalent Depth Factor
$\varepsilon_t := \frac{0.003 \cdot \left(\beta_1 \cdot d - a\right)}{a} = 0.037$	Net Tensile Strain
$\phi_{RF} := \left. \begin{array}{l} \text{if } \varepsilon_t \geq 0.005 \\ \quad \| \ 0.9 \\ \text{else} \\ \quad \| \ 0.65 \end{array} \right\vert = 0.9$	Strength Reduction Factor

$\varepsilon_t \geq 0.005$ Tension-Controlled Section

Calculate Environmental Durability Factor:

$$\beta := \text{if } h \geq 16\ \text{in} \;\middle\|\; 1.2 \;\text{else}\; \middle\|\; 1.35 = 1.2$$

Strain Gradient Factor

$$\gamma := \frac{M_u}{M_{FL} + M_S} = 1.26$$

Combined Load Factor (Section 9.2.6)

$$f_{smax} := \frac{320\ \frac{kip}{in}}{\beta \cdot \sqrt{s^2 + 4 \cdot \left(2\ in + \frac{d_b}{2}\right)^2}} = 20.513\ ksi$$

Permissible Stress in Reinforcement for Normal Environmental Exposure (Section 10.6.4.5)

$$S_d := \max\left(\frac{\phi_{RF} \cdot f_y}{\gamma \cdot f_{smax}}, 1.0\right) = 2.09$$

Environmental Durability Factor (Section 9.2.6)

$$M_{u1} := S_d \cdot M_u = 51.462\ kip \cdot ft$$

Required Moment Strength

Check Flexural Strength:

$$M_n := A_{s\_prov} \cdot f_y \cdot \left(d - \frac{a}{2}\right) = 59.054\ kip \cdot ft$$

Nominal Flexural Strength

$$\phi_{RF} \cdot M_n = 53.149\ kip \cdot ft$$

Design Flexural Strength

$$\frac{M_{u1}}{\phi_{RF} \cdot M_n} = 0.968$$

Demand Capacity Ratio

$$Check_{Flex} := \text{if } M_{u1} \leq \phi_{RF} \cdot M_n \;\middle\|\; \text{“PASS”} \;\text{else}\; \middle\|\; \text{“FAIL”}$$

$Check_{Flex} = \text{“PASS”}$

Check Minimum Area of Reinforcement Required:

$A_{s\_prov} = 0.785\ in^2$

Area of Reinforcement Provided

$$A_{smin} := \max\left(\frac{3 \cdot \sqrt{\frac{f'_c}{psi}} \cdot psi}{f_y} \cdot b \cdot d, \frac{200\ psi \cdot b \cdot d}{f_y}\right) = 0.658\ in^2$$

Minimum Area of Reinforcement Required (Section 10.5.1)

$$\begin{aligned} Check_{Reinforcement} := &\text{ if } A_{smin} \le A_{s\_prov} \\ &\quad \| \text{"PASS"} \\ &\text{ else} \\ &\quad \| \text{"FAIL"} \end{aligned}$$

$Check_{Reinforcement} = \text{"PASS"}$

Rectangular Section in Shear (Per ACI 350-20 Section 11.1):

$\lambda := 1$

Modification Factor for Normalweight Concrete

$\phi_s := 0.75$

Shear Reduction Factor

$V_u = 14.073\ kip$

Factored Shear Force

$$V_c := 2\ psi \cdot \lambda \cdot \sqrt{\frac{f'_c}{psi}} \cdot b \cdot d = 26.304\ kip$$

Nominal Shear Strength Provided by Concrete

Demand Capacity Ratio

$$\frac{V_u}{\phi_s \cdot V_c} = 0.713$$

$$\begin{aligned} Check_{Shear} := &\text{ if } V_u \le \phi_s \cdot V_c \\ &\quad \| \text{"PASS"} \\ &\text{ else} \\ &\quad \| \text{"FAIL"} \end{aligned}$$

$Check_{Shear} = \text{"PASS"}$

Design Vertical Reinforcement

Calculate Forces Acting on the Wall (To be used in separate spreadsheet):

Load Case = 1.2(fluid) + 1.6(soil) + 1.0(earthquake)

$\phi_{FL} := 1.2$ Fluid Load Factor (ACI 350-20 Section 9.2.1)

$V_{FL} := \frac{1}{2} \cdot \gamma_w \cdot H_{HW\_norm}^{\ 2} \cdot L_o = 92.286\ \text{kip}$ Service Shear from Lateral Water Pressure

$M_{FL} := V_{FL} \cdot \frac{H_{HW\_norm}}{3} = 482.963\ \text{kip} \cdot \text{ft}$ Service Moment from Lateral Water Pressure

$\phi_S := 1.6$ Soil Load Factor (ACI 350-20 Section 9.2.1)

$V_S := \frac{1}{2} \cdot \left(\gamma_{sat\_fill} - \gamma_w\right) \cdot H_{soil}^{\ 2} \cdot L_o = 39.36\ \text{kip}$ Service Shear from Soil

$M_S := V_S \cdot \frac{H_{soil}}{3} = 131.2\ \text{kip} \cdot \text{ft}$ Service Moment from Soil

$\phi_E := 1.0$ Earthquake Load Factor (ACI 350-20 Section 9.2.1)

$V_E := 28.7\ \text{kip}$ Seismic Shear Force (Inertia, from Step 5 of Seismic Spreadsheet Calculator)

$M_E := 486.5\ \text{kip} \cdot \text{ft}$ Seismic Moment (Inertia, from Step 5 of Seismic Spreadsheet Calculator)

$M_u := \phi_{FL} \cdot M_{FL} + \phi_S \cdot M_S + \phi_E \cdot M_E = 1275.975\ \text{kip} \cdot \text{ft}$ Factored Moment

$V_u := \phi_{FL} \cdot V_{FL} + \phi_S \cdot V_S + \phi_E \cdot V_E = 202.419\ \text{kip}$ Factored Shear Force

INTAKE TOWER WALL REINFORCEMENT - VERTICAL

Moment Capacity Calculator (LRFD Design)

Lake Erin Dam Rehabilitation

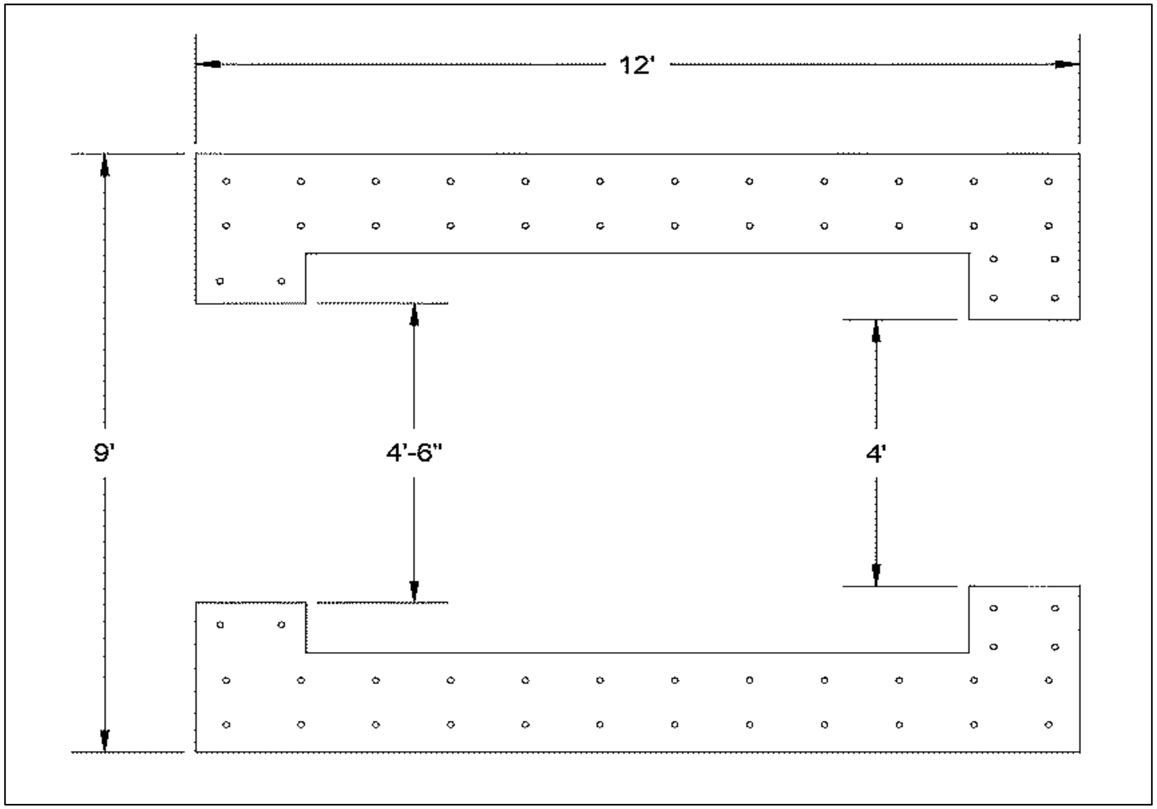
Reference: ACI 318-19 and ACI 350-20

Section Property Calculator

Dimensions of Tower Walls					
X-direction			*Y-direction*		
Outside Width =	12.0	ft	Outside Width =	9.0	ft
Inside Width =	9.0	ft	Inside Width =	6.0	ft
Wall Thickness =	1.5	ft	Wall Thickness =	1.5	ft

Dimensions of Gate Openings											
Left Side			*Right Side*			*Top*			*Bottom*		
Width =	4.5	ft	Width =	4.0	ft	Width =	0.0	ft	Width =	0.0	ft

Section Properties		
Moment of Inertia		
Hollow Rectangle	567.0	ft^4
Left Opening	11.4	ft^4
Right Opening	32.0	ft^4
Total	**523.6**	ft^4
	10,857,564.0	in^4
Area		
Hollow Rectangle	54.0	ft^2
Left Opening	6.8	ft^2
Right Opening	6.0	ft^2
Total	**41.3**	ft^2
	5,940.0	in^2



Moment Capacity Calculator (LRFD Design)

Lake Erin Dam Rehabilitation

Reference: ACI 318-19 and ACI 350-20

Calculated by:	NRP
Date:	4/25/2024
Checked by:	IML
Date:	5/29/2024

Material Properties			Axial Force and Moment Capacity					
Variable	**Value**	**Unit**	**Variable**	**Value**	**Unit**	**Variable**	**Value**	**Unit**
Concrete Compressive Strength (f'_c)	5,000	psi	Neutral Axis (NA)	125.139883	in	Center of Gravity	72	in
Concrete Modulus of Elasticity (E_c)	4,030,509	psi	Factored Moment (M_u)	1,276	kip-ft	Axial Dead Load (P)	0	kips
Concrete Modulus of Rupture (f_r)	530.33	psi		*15,311,700*	lb-in	Nominal Axial Capacity (P_n)	21,457	kips
Reinforcement Yield Strength (f_y)	60,000	psi	Nominal Moment Strength (M_n)	6,515	kip-ft	Reduced Axial Capacity (ϕ P_n)	13,947	kips
Steel Modulus of Elasticity (E_s)	29,000,000	psi		*78,181,233*	lb-in	Moment of Inertia (I_g)	10,857,564	in^4
Yield Strain (ε_y)	0.00207	in/in	Reduced Moment Capacity (ϕ M_n)	5,864	kip-ft	Cracked Moment Capacity (M_{cr})	6,664	kip-ft
Reinforcement Diameter (d_b)	0.75	in		*70,363,110*	lb-in		*79,973,512*	lb-in
Reinforcement Area (A_b)	0.442	in^2				Reduced Cracked Moment (ϕ M_{cr})	5,998	kip-ft
			Moment Check	OK			*71,976,161*	lb-in

Curvature = 1.65E-05

Distance from Bottom Fiber	Width	Number of Bars	Area of One Bar	Area of Bars	Area of Concrete	Location of CG from Bottom	Distance of CG from NA	Area of Concrete in Compression	Strain	Concrete Stress	Steel Stress	Force	Moment
[in]	[in]		[in^2]	[in^2]	[in^2]	[in]	[in]	[in^2]	[in/in]	[psi]	[psi]	[lb]	[lb-in]
144	60	-	0.44	-	60.00	143.625	18.49	60.00	0.000306	1,231.80	8,862.94	73,907.89	1,366,196.00
143	60	-	0.44	-	60.00	142.625	17.49	60.00	0.000289	1,165.16	8,383.47	69,909.65	1,222,378.45
142	60	-	0.44	-	60.00	141.625	16.49	60.00	0.000273	1,098.52	7,904.01	65,911.42	1,086,557.38
141	60	-	0.44	-	59.12	140.625	15.49	59.12	0.000256	1,031.89	7,424.55	61,001.43	944,614.28
140	60	4	0.44	1.77	58.23	139.625	14.49	58.23	0.000239	965.25	6,945.08	68,482.18	991,972.38
139	60	4	0.44	1.77	59.12	138.625	13.49	59.12	0.000223	898.61	6,465.62	64,548.41	870,442.82
138	60	-	0.44	-	60.00	137.625	12.49	60.00	0.000206	831.97	5,986.16	49,918.46	623,237.86
137	60	-	0.44	-	60.00	136.625	11.49	60.00	0.000190	765.34	5,506.69	45,920.23	527,399.17
136	60	-	0.44	-	60.00	135.625	10.49	60.00	0.000173	698.70	5,027.23	41,921.99	439,556.95
135	60	-	0.44	-	60.00	134.625	9.49	60.00	0.000157	632.06	4,547.77	37,923.75	359,711.21
134	60	-	0.44	-	60.00	133.625	8.49	60.00	0.000140	565.43	4,068.30	33,925.51	287,861.94
133	60	-	0.44	-	59.12	132.625	7.49	59.12	0.000124	498.79	3,588.84	29,486.56	220,710.35
132	60	4	0.44	1.77	59.12	131.625	6.49	59.12	0.000107	432.15	3,109.38	31,041.92	201,310.50
131	60	-	0.44	-	60.00	130.625	5.49	60.00	0.000091	365.51	2,629.91	21,930.80	120,293.00
130	60	-	0.44	-	60.00	129.625	4.49	60.00	0.000074	298.88	2,150.45	17,932.56	80,429.64
129	60	-	0.44	-	60.00	128.625	3.49	60.00	0.000058	232.24	1,670.99	13,934.33	48,562.75
128	60	-	0.44	-	59.12	127.625	2.49	59.12	0.000041	165.60	1,191.52	9,789.77	24,328.71
127	60	4	0.44	1.77	59.12	126.625	1.49	59.12	0.000025	98.96	712.06	7,108.72	10,557.28
126	36	-	0.44	-	36.00	125.625	0.49	36.00	0.000008	32.33	232.60	1,163.77	564.56
125	36	-	0.44	-	36.00	124.625	-0.51	0.00	-0.000009	0.00	(246.87)	0.00	0.00
124	36	-	0.44	-	36.00	123.625	-1.51	0.00	-0.000025	0.00	(726.33)	0.00	0.00
123	36	-	0.44	-	36.00	122.625	-2.51	0.00	-0.000042	0.00	(1,205.79)	0.00	0.00
122	36	-	0.44	-	36.00	121.625	-3.51	0.00	-0.000058	0.00	(1,685.26)	0.00	0.00
121	36	-	0.44	-	36.00	120.625	-4.51	0.00	-0.000075	0.00	(2,164.72)	0.00	0.00
120	36	-	0.44	-	36.00	119.625	-5.51	0.00	-0.000091	0.00	(2,644.18)	0.00	0.00
119	36	-	0.44	-	36.00	118.625	-6.51	0.00	-0.000108	0.00	(3,123.65)	0.00	0.00
118	36	-	0.44	-	36.00	117.625	-7.51	0.00	-0.000124	0.00	(3,603.11)	0.00	0.00
117	36	-	0.44	-	36.00	116.625	-8.51	0.00	-0.000141	0.00	(4,082.58)	0.00	0.00
116	36	-	0.44	-	35.12	115.625	-9.51	0.00	-0.000157	0.00	(4,562.04)	0.00	0.00
115	36	4	0.44	1.77	35.12	114.625	-10.51	0.00	-0.000174	0.00	(5,041.50)	(8,909.07)	93,677.83
114	36	-	0.44	-	36.00	113.625	-11.51	0.00	-0.000190	0.00	(5,520.97)	0.00	0.00
113	36	-	0.44	-	36.00	112.625	-12.51	0.00	-0.000207	0.00	(6,000.43)	0.00	0.00
112	36	-	0.44	-	36.00	111.625	-13.51	0.00	-0.000223	0.00	(6,479.89)	0.00	0.00
111	36	-	0.44	-	36.00	110.625	-14.51	0.00	-0.000240	0.00	(6,959.36)	0.00	0.00
110	36	-	0.44	-	36.00	109.625	-15.51	0.00	-0.000257	0.00	(7,438.82)	0.00	0.00
109	36	-	0.44	-	36.00	108.625	-16.51	0.00	-0.000273	0.00	(7,918.28)	0.00	0.00
108	36	-	0.44	-	36.00	107.625	-17.51	0.00	-0.000290	0.00	(8,397.75)	0.00	0.00
107	36	-	0.44	-	36.00	106.625	-18.51	0.00	-0.000306	0.00	(8,877.21)	0.00	0.00
106	36	-	0.44	-	36.00	105.625	-19.51	0.00	-0.000323	0.00	(9,356.67)	0.00	0.00
105	36	-	0.44	-	36.00	104.625	-20.51	0.00	-0.000339	0.00	(9,836.14)	0.00	0.00
104	36	-	0.44	-	35.12	103.625	-21.51	0.00	-0.000356	0.00	(10,315.60)	0.00	0.00
103	36	4	0.44	1.77	35.12	102.625	-22.51	0.00	-0.000372	0.00	(10,795.06)	(19,076.45)	429,504.09
102	36	-	0.44	-	36.00	101.625	-23.51	0.00	-0.000389	0.00	(11,274.53)	0.00	0.00
101	36	-	0.44	-	36.00	100.625	-24.51	0.00	-0.000405	0.00	(11,753.99)	0.00	0.00
100	36	-	0.44	-	36.00	99.625	-25.51	0.00	-0.000422	0.00	(12,233.45)	0.00	0.00
99	36	-	0.44	-	36.00	98.625	-26.51	0.00	-0.000438	0.00	(12,712.92)	0.00	0.00
98	36	-	0.44	-	36.00	97.625	-27.51	0.00	-0.000455	0.00	(13,192.38)	0.00	0.00
97	36	-	0.44	-	36.00	96.625	-28.51	0.00	-0.000471	0.00	(13,671.84)	0.00	0.00
96	36	-	0.44	-	36.00	95.625	-29.51	0.00	-0.000488	0.00	(14,151.31)	0.00	0.00
95	36	-	0.44	-	36.00	94.625	-30.51	0.00	-0.000505	0.00	(14,630.77)	0.00	0.00
94	36	-	0.44	-	36.00	93.625	-31.51	0.00	-0.000521	0.00	(15,110.23)	0.00	0.00
93	36	-	0.44	-	36.00	92.625	-32.51	0.00	-0.000538	0.00	(15,589.70)	0.00	0.00
92	36	-	0.44	-	35.12	91.625	-33.51	0.00	-0.000554	0.00	(16,069.16)	0.00	0.00
91	36	4	0.44	1.77	35.12	90.625	-34.51	0.00	-0.000571	0.00	(16,548.62)	(29,243.83)	1,009,347.53
90	36	-	0.44	-	36.00	89.625	-35.51	0.00	-0.000587	0.00	(17,028.09)	0.00	0.00
89	36	-	0.44	-	36.00	88.625	-36.51	0.00	-0.000604	0.00	(17,507.55)	0.00	0.00
88	36	-	0.44	-	36.00	87.625	-37.51	0.00	-0.000620	0.00	(17,987.02)	0.00	0.00
87	36	-	0.44	-	36.00	86.625	-38.51	0.00	-0.000637	0.00	(18,466.48)	0.00	0.00
86	36	-	0.44	-	36.00	85.625	-39.51	0.00	-0.000653	0.00	(18,945.94)	0.00	0.00
85	36	-	0.44	-	36.00	84.625	-40.51	0.00	-0.000670	0.00	(19,425.41)	0.00	0.00
84	36	-	0.44	-	36.00	83.625	-41.51	0.00	-0.000686	0.00	(19,904.87)	0.00	0.00
83	36	-	0.44	-	36.00	82.625	-42.51	0.00	-0.000703	0.00	(20,384.33)	0.00	0.00
82	36	-	0.44	-	36.00	81.625	-43.51	0.00	-0.000719	0.00	(20,863.80)	0.00	0.00
81	36	-	0.44	-	36.00	80.625	-44.51	0.00	-0.000736	0.00	(21,343.26)	0.00	0.00
80	36	-	0.44	-	35.12	79.625	-45.51	0.00	-0.000753	0.00	(21,822.72)	0.00	0.00
79	36	4	0.44	1.77	35.12	78.625	-46.51	0.00	-0.000769	0.00	(22,302.19)	(39,411.22)	1,833,208.14
78	36	-	0.44	-	36.00	77.625	-47.51	0.00	-0.000786	0.00	(22,781.65)	0.00	0.00
77	36	-	0.44	-	36.00	76.625	-48.51	0.00	-0.000802	0.00	(23,261.11)	0.00	0.00
76	36	-	0.44	-	36.00	75.625	-49.51	0.00	-0.000819	0.00	(23,740.58)	0.00	0.00
75	36	-	0.44	-	36.00	74.625	-50.51	0.00	-0.000835	0.00	(24,220.04)	0.00	0.00
74	36	-	0.44	-	36.00	73.625	-51.51	0.00	-0.000852	0.00	(24,699.50)	0.00	0.00
73	36	-	0.44	-	36.00	72.625	-52.51	0.00	-0.000868	0.00	(25,178.97)	0.00	0.00
72	36	-	0.44	-	36.00	71.625	-53.51	0.00	-0.000885	0.00	(25,658.43)	0.00	0.00
71	36	-	0.44	-	36.00	70.625	-54.51	0.00	-0.000901	0.00	(26,137.89)	0.00	0.00
70	36	-	0.44	-	36.00	69.625	-55.51	0.00	-0.000918	0.00	(26,617.36)	0.00	0.00
69	36	-	0.44	-	36.00	68.625	-56.51	0.00	-0.000934	0.00	(27,096.82)	0.00	0.00
68	36	-	0.44	-	35.12	67.625	-57.51	0.00	-0.000951	0.00	(27,576.28)	0.00	0.00
67	36	4	0.44	1.77	35.12	66.625	-58.51	0.00	-0.000967	0.00	(28,055.75)	(49,578.60)	2,901,085.92
66	36	-	0.44	-	36.00	65.625	-59.51	0.00	-0.000984	0.00	(28,535.21)	0.00	0.00
65	36	-	0.44	-	36.00	64.625	-60.51	0.00	-0.001001	0.00	(29,014.67)	0.00	0.00
64	36	-	0.44	-	36.00	63.625	-61.51	0.00	-0.001017	0.00	(29,494.14)	0.00	0.00
63	36	-	0.44	-	36.00	62.625	-62.51	0.00	-0.001034	0.00	(29,973.60)	0.00	0.00
62	36	-	0.44	-	36.00	61.625	-63.51	0.00	-0.001050	0.00	(30,453.06)	0.00	0.00
61	36	-	0.44	-	36.00	60.625	-64.51	0.00	-0.001067	0.00	(30,932.53)	0.00	0.00
60	36	-	0.44	-	36.00	59.625	-65.51	0.00	-0.001083	0.00	(31,411.99)	0.00	0.00
59	36	-	0.44	-	36.00	58.625	-66.51	0.00	-0.001100	0.00	(31,891.46)	0.00	0.00
58	36	-	0.44	-	36.00	57.625	-67.51	0.00	-0.001116	0.00	(32,370.92)	0.00	0.00
57	36	-	0.44	-	36.00	56.625	-68.51	0.00	-0.001133	0.00	(32,850.38)	0.00	0.00

Moment Capacity Calculator (LRFD Design)

Lake Erin Dam Rehabilitation

Reference: ACI 318-19 and ACI 350-20

Calculated by:	NRP
Date:	4/25/2024
Checked by:	IML
Date:	5/29/2024

Material Properties

Variable	Value	Unit
Concrete Compressive Strength (f'_c)	5,000	psi
Concrete Modulus of Elasticity (E_c)	4,030,509	psi
Concrete Modulus of Rupture (f_r)	530.33	psi
Reinforcement Yield Strength (f_y)	60,000	psi
Steel Modulus of Elasticity (E_s)	29,000,000	psi
Yield Strain (ε_y)	0.00207	in/in
Reinforcement Diameter (d_b)	0.75	in
Reinforcement Area (A_b)	0.442	in^2

Axial Force and Moment Capacity

Variable	Value	Unit	Variable	Value	Unit
Neutral Axis (NA)	125.139883	in	Center of Gravity	72	in
Factored Moment (M_u)	1,276	kip-ft	Axial Dead Load (P)	0	kips
	15,311,700	lb-in	Nominal Axial Capacity (P_n)	21,457	kips
Nominal Moment Strength (M_n)	6,515	kip-ft	Reduced Axial Capacity (ϕ P_n)	13,947	kips
	78,181,233	lb-in	Moment of Inertia (I_g)	10,857,564	in^4
Reduced Moment Capacity (ϕ M_n)	5,864	kip-ft	Cracked Moment Capacity (M_{cr})	6,664	kip-ft
	70,363,110	lb-in		*79,973,512*	lb-in
			Reduced Cracked Moment (ϕ M_{cr})	5,998	kip-ft
Moment Check	OK			*71,976,161*	lb-in

Curvature = 1.65E-05

Distance from Bottom Fiber	Width	Number of Bars	Area of One Bar	Area of Bars	Area of Concrete	Location of CG from Bottom	Distance of CG from NA	Area of Concrete in Compression	Strain	Concrete Stress	Steel Stress	Force	Moment
[in]	[in]		[in^2]	[in^2]	[in^2]	[in]	[in]	[in^2]	[in/in]	[psi]	[psi]	[lb]	[lb-in]
56	36	-	0.44	-	35.12	55.625	-69.51	0.00	-0.001149	0.00	(33,329.85)	0.00	0.00
55	36	4	0.44	1.77	35.12	54.625	-70.51	0.00	-0.001166	0.00	(33,809.31)	(59,745.98)	4,212,980.87
54	36	-	0.44	-	36.00	53.625	-71.51	0.00	-0.001182	0.00	(34,288.77)	0.00	0.00
53	36	-	0.44	-	36.00	52.625	-72.51	0.00	-0.001199	0.00	(34,768.24)	0.00	0.00
52	36	-	0.44	-	36.00	51.625	-73.51	0.00	-0.001215	0.00	(35,247.70)	0.00	0.00
51	36	-	0.44	-	36.00	50.625	-74.51	0.00	-0.001232	0.00	(35,727.16)	0.00	0.00
50	36	-	0.44	-	36.00	49.625	-75.51	0.00	-0.001249	0.00	(36,206.63)	0.00	0.00
49	36	-	0.44	-	36.00	48.625	-76.51	0.00	-0.001265	0.00	(36,686.09)	0.00	0.00
48	36	-	0.44	-	36.00	47.625	-77.51	0.00	-0.001282	0.00	(37,165.55)	0.00	0.00
47	36	-	0.44	-	36.00	46.625	-78.51	0.00	-0.001298	0.00	(37,645.02)	0.00	0.00
46	36	-	0.44	-	36.00	45.625	-79.51	0.00	-0.001315	0.00	(38,124.48)	0.00	0.00
45	36	-	0.44	-	36.00	44.625	-80.51	0.00	-0.001331	0.00	(38,603.94)	0.00	0.00
44	36	-	0.44	-	35.12	43.625	-81.51	0.00	-0.001348	0.00	(39,083.41)	0.00	0.00
43	36	4	0.44	1.77	35.12	42.625	-82.51	0.00	-0.001364	0.00	(39,562.87)	(69,913.36)	5,768,893.00
42	36	-	0.44	-	36.00	41.625	-83.51	0.00	-0.001381	0.00	(40,042.33)	0.00	0.00
41	36	-	0.44	-	36.00	40.625	-84.51	0.00	-0.001397	0.00	(40,521.80)	0.00	0.00
40	36	-	0.44	-	36.00	39.625	-85.51	0.00	-0.001414	0.00	(41,001.26)	0.00	0.00
39	36	-	0.44	-	36.00	38.625	-86.51	0.00	-0.001430	0.00	(41,480.72)	0.00	0.00
38	36	-	0.44	-	36.00	37.625	-87.51	0.00	-0.001447	0.00	(41,960.19)	0.00	0.00
37	36	-	0.44	-	36.00	36.625	-88.51	0.00	-0.001463	0.00	(42,439.65)	0.00	0.00
36	36	-	0.44	-	36.00	35.625	-89.51	0.00	-0.001480	0.00	(42,919.11)	0.00	0.00
35	36	-	0.44	-	36.00	34.625	-90.51	0.00	-0.001497	0.00	(43,398.58)	0.00	0.00
34	36	-	0.44	-	36.00	33.625	-91.51	0.00	-0.001513	0.00	(43,878.04)	0.00	0.00
33	36	-	0.44	-	36.00	32.625	-92.51	0.00	-0.001530	0.00	(44,357.50)	0.00	0.00
32	36	-	0.44	-	35.12	31.625	-93.51	0.00	-0.001546	0.00	(44,836.97)	0.00	0.00
31	36	4	0.44	1.77	35.12	30.625	-94.51	0.00	-0.001563	0.00	(45,316.43)	(80,080.75)	7,568,822.30
30	36	-	0.44	-	36.00	29.625	-95.51	0.00	-0.001579	0.00	(45,795.90)	0.00	0.00
29	36	-	0.44	-	36.00	28.625	-96.51	0.00	-0.001596	0.00	(46,275.36)	0.00	0.00
28	36	-	0.44	-	36.00	27.625	-97.51	0.00	-0.001612	0.00	(46,754.82)	0.00	0.00
27	36	-	0.44	-	36.00	26.625	-98.51	0.00	-0.001629	0.00	(47,234.29)	0.00	0.00
26	36	-	0.44	-	36.00	25.625	-99.51	0.00	-0.001645	0.00	(47,713.75)	0.00	0.00
25	36	-	0.44	-	36.00	24.625	-100.51	0.00	-0.001662	0.00	(48,193.21)	0.00	0.00
24	36	-	0.44	-	36.00	23.625	-101.51	0.00	-0.001678	0.00	(48,672.68)	0.00	0.00
23	36	-	0.44	-	36.00	22.625	-102.51	0.00	-0.001695	0.00	(49,152.14)	0.00	0.00
22	36	-	0.44	-	36.00	21.625	-103.51	0.00	-0.001711	0.00	(49,631.60)	0.00	0.00
21	36	-	0.44	-	36.00	20.625	-104.51	0.00	-0.001728	0.00	(50,111.07)	0.00	0.00
20	36	-	0.44	-	36.00	19.625	-105.51	0.00	-0.001745	0.00	(50,590.53)	0.00	0.00
19	36	-	0.44	-	36.00	18.625	-106.51	0.00	-0.001761	0.00	(51,069.99)	0.00	0.00
18	84	-	0.44	-	83.12	17.625	-107.51	0.00	-0.001778	0.00	(51,549.46)	0.00	0.00
17	84	4	0.44	1.77	83.12	16.625	-108.51	0.00	-0.001794	0.00	(52,028.92)	(91,942.69)	9,977,150.41
16	84	-	0.44	-	83.12	15.625	-109.51	0.00	-0.001811	0.00	(52,508.38)	0.00	0.00
15	84	4	0.44	1.77	83.12	14.625	-110.51	0.00	-0.001827	0.00	(52,987.85)	(93,637.25)	10,348,310.31
14	84	-	0.44	-	84.00	13.625	-111.51	0.00	-0.001844	0.00	(53,467.31)	0.00	0.00
13	84	-	0.44	-	84.00	12.625	-112.51	0.00	-0.001860	0.00	(53,946.77)	0.00	0.00
12	84	-	0.44	-	84.00	11.625	-113.51	0.00	-0.001877	0.00	(54,426.24)	0.00	0.00
11	84	-	0.44	-	84.00	10.625	-114.51	0.00	-0.001893	0.00	(54,905.70)	0.00	0.00
10	84	-	0.44	-	84.00	9.625	-115.51	0.00	-0.001910	0.00	(55,385.16)	0.00	0.00
9	84	-	0.44	-	84.00	8.625	-116.51	0.00	-0.001926	0.00	(55,864.63)	0.00	0.00
8	84	-	0.44	-	84.00	7.625	-117.51	0.00	-0.001943	0.00	(56,344.09)	0.00	0.00
7	84	-	0.44	-	84.00	6.625	-118.51	0.00	-0.001959	0.00	(56,823.55)	0.00	0.00
6	84	-	0.44	-	82.23	5.625	-119.51	0.00	-0.001976	0.00	(57,303.02)	0.00	0.00
5	84	8	0.44	3.53	82.23	4.625	-120.51	0.00	-0.001992	0.00	(57,782.48)	(204,220.15)	24,611,567.18
4	84	-	0.44	-	84.00	3.625	-121.51	0.00	-0.002009	0.00	(58,261.94)	0.00	0.00
3	84	-	0.44	-	84.00	2.625	-122.51	0.00	-0.002026	0.00	(58,741.41)	0.00	0.00
2	84	-	0.44	-	84.00	1.625	-123.51	0.00	-0.002042	0.00	(59,220.87)	0.00	0.00
1	84	-	0.44	-	84.00	0.625	-124.51	0.00	-0.002059	0.00	(59,700.34)	0.00	0.00
0	84	-	0.44	-	0.00	0	-125.14	0.00	-0.002069	0.00	(60,000.00)	0.00	0.00
Sum Σ		**64**		**28.27**								**0.00**	**78,181,232.80**

IMPACT BASIN STABILITY ANALYSIS

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Calculated By: NRP
Date: 3/11/2024
Checked By: IML
Date: 4/22/2024
Project Number: 60727041

Project: Lake Erin Dam Rehabilitation

Task: Structural Stability Analysis - Impact Basin

Description:

Check the external global stability (sliding, flotation, overturning, and bearing) of the USBR Type VI Impact Basin structure in accordance with USACE EM 1110-2-2100. Stability analysis is based on the conceptual design of the impact basin.

Codes, Standards, & References:

1. USACE EM 1110-2-2100, Stability Analysis of Concrete Structures (2005)
2. Civil Engineering Reference Manual 13th Edition
3. AECOM Basis of Design Report

Hydrologic and Hydraulic:

Maximum conduit discharge and tailwater elevation is based on AECOM's hydraulic analysis. The impact basin is designed based on the discharge from the spillway design flood (1/3 PMF). The maximum discharge out of the pipe is 322.3 cfs with a maximum velocity of 25.7 feet/s. The impact basin geometry was determined as part of the hydraulic analysis based on the conduit size and hydraulics using FHWA Circular 14, Hydraulic Design of Energy Dissipators for Culverts and Channels, 3rd Edition.

Geotechnical/Subsurface Investigation:

Geotechnical parameters were based on the subsurface investigation. The structure is conservatively assumed to be founded on residual soil, although weathered rock may be near the elevation of the foundation slab.

Assumptions:

- External stability of the impact basin was evaluated assuming the structure as one monolithic unit.
- Discharge against the impact baffle is evaluated as an impact from a jet on a flat surface.
- Conservatively ignore passive pressure resistance from shear key.
- Uplift acts throughout the entire foundation base including the foundation ledges.
- The sliding failure plane is at the foundation/soil interface and the point of rotation is at the downstream point of the sill.
- Seismic loads are negligible. The PGA for this site is 0.12g (2,475-year event).
- Two critical (unusual) load cases are considered: 1) During the 1/3 PMF event which includes thrust from the pipe flow and maximum water in the basin, and 2) After the 1/3 PMF event when the upstream soil is saturated.
- Factors of safety are based on a "normal" structure and "ordinary" site information (EM 1110-2-2100, Chapter 3).

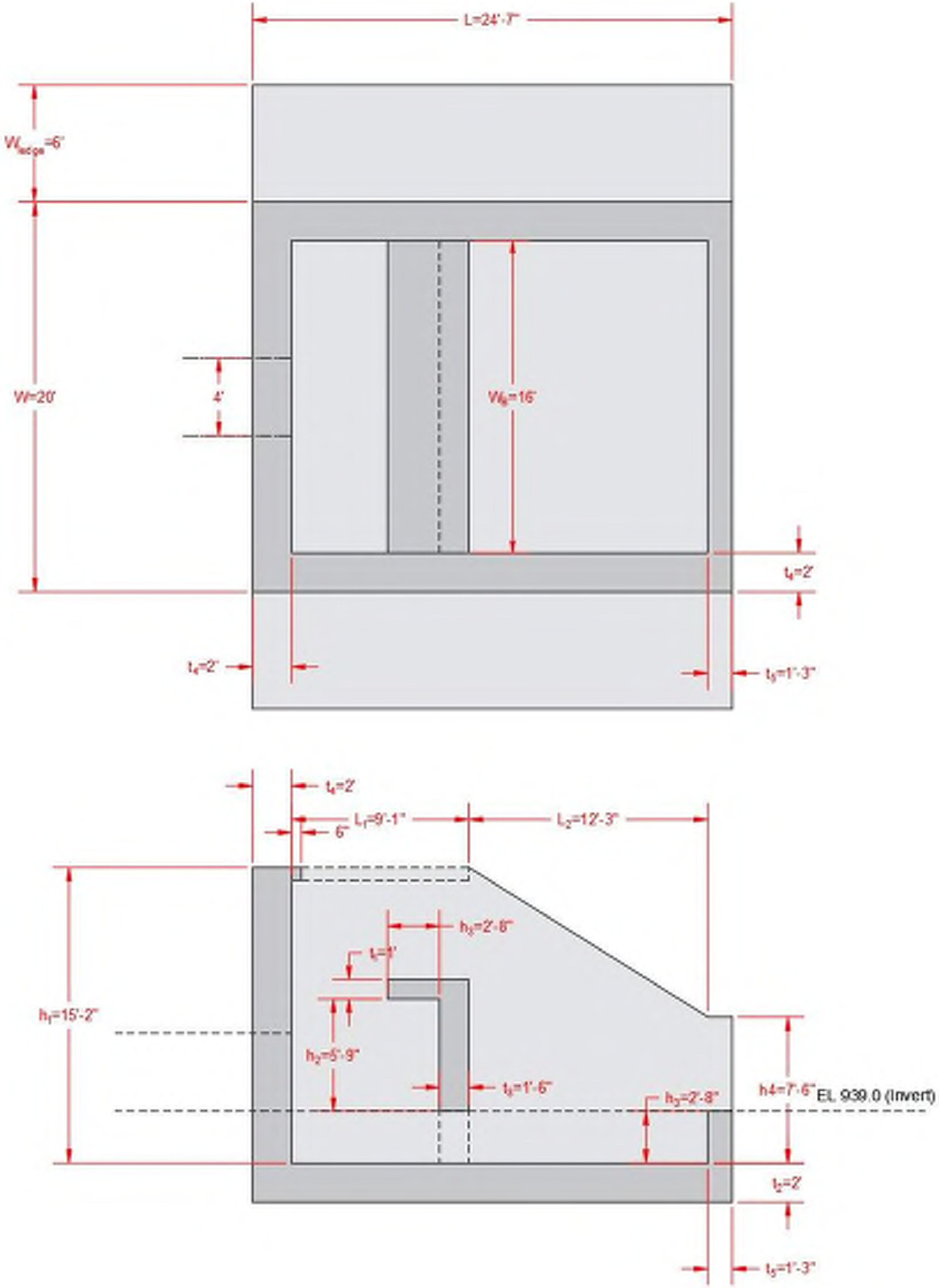


Figure 1 - Section Plan and Elevation View

Material Properties

$\gamma_c := 150\ \text{pcf}$	Unit Weight of Concrete
$\gamma_w := 62.4\ \text{pcf}$	Unit Weight of Water
$f'_c := 5000\ \text{psi}$	Concrete Compressive Strength
$f_y := 60000\ \text{psi}$	Yield Strength of Reinforcement

Hydrologic and Hydraulic Parameters (*AECOM Basis of Design Report - Appendix B*)

$Q_{max} := 322.3\ \frac{ft^3}{s}$	Design Discharge
$v_{max} := 25.66\ \frac{ft}{s}$	Maximum Velocity
$EL_{tw} := 939.0\ ft$	Elevation of Tailwater (Normal)
$EL_{tw\_flood} := 940.1\ ft$	Elevation of Tailwater (Flood)
$EL_{base} := 934.333\ ft$	Elevation of Base
$h_{tw} := EL_{tw} - EL_{base} = 4.667\ ft$	Tailwater Head on Impact Basin (Normal)
$h_{tw\_flood} := EL_{tw\_flood} - EL_{base} = 5.767\ ft$	Tailwater Head on Impact Basin (Flood)

Geotechnical Parameters *(AECOM Basis of Design Report - Appendix A)*

$\gamma_{s\_moist} := 123\ \text{pcf}$	Moist Unit Weight of Backfill
$\gamma_{s\_sat} := 128\ \text{pcf}$	Saturated Unit Weight of Backfill
$\phi_{fill} := 31\ \text{deg}$	Friction Angle (Backfill)
$\phi := 31\ \text{deg}$	Friction Angle (Foundation)
$\mu_f := \tan(\phi) = 0.601$	Coefficient of Friction
$\sigma_{all} := 5000\ \text{psf}$	Allowable Bearing Capacity
$K_o := 1 - \sin(\phi_{fill}) = 0.485$	At-Rest Earth Coefficient
$EL_{us\_soil} := 951.0\ ft$	Elevation of Upstream Soil
$h_{us\_soil} := EL_{us\_soil} - EL_{base} = 16.667\ ft$	Height of Upstream Soil
$EL_{ds\_soil} := 943.33\ ft$	Elevation of Downstream Soil
$h_{ds\_soil} := EL_{ds\_soil} - EL_{base} = 8.997\ ft$	Height of Downstream Soil

Impact Basin Dimensions

Geometry of Impact Basin based on hydraulic evaluation. See AECOM Basis of Design Report - Appendix B.

$W_B := 16\ \text{ft}$	Width of Baffle (Wb)
$h_{headwall} := 15.166\ \text{ft}$	Height of Headwall (h1)
$t_{headwall} := 24\ \text{in}$	Thickness of Headwall (t4)
$h_{sidewall1} := 15.166\ \text{ft}$	Height of Sidewall 1 (Upstream Side) (h1)
$h_{sidewall2} := 7.5\ \text{ft}$	Height of Sidewall 2 (Downstream Side) (h4)
$t_{sidewall} := 24\ \text{in}$	Thickness of Sidewalls (t4)
$W := W_B + t_{sidewall} \cdot 2 = 20\ \text{ft}$	Width of Impact Basin
$h_{sill} := 2.67\ \text{ft}$	Height of Downstream Sill (h3)
$t_{sill} := 1.25\ \text{ft}$	Thickness of Downstream Sill (t5)
$h_{baffle} := 6.75\ \text{ft}$	Height of Baffle (h2+t1)
$l_{baffle} := 4.17\ \text{ft}$	Length of Baffle (h3+t3)
$t_{baffle1} := 18\ \text{in}$	Thickness of Baffle Vertical Segment (t3)
$t_{baffle2} := 12\ \text{in}$	Thickness of Baffle Horizontal Segment (t1)
$L_1 := 9.08\ \text{ft}$	Length of Upstream Sidewall (L1)
$L_2 := 12.25\ \text{ft}$	Length of Downstream Sidewall (tapered) (L2)
$L := L_1 + L_2 + t_{headwall} + t_{sill} = 24.58\ \text{ft}$	Total Length of Impact Basin (L1+L2+t4+t5)
$t_{overhang} := 8\ \text{in}$	Thickness of Top Slab Overhang
$L_{overhang} := 6\ \text{in}$	Length of Overhang (Cantilever)
$t_{base} := 24\ \text{in}$	Thickness of Base Slab (t2)
$D_{opening} := 4\ \text{ft}$	Diameter of Conduit
$A_{opening} := \frac{\pi}{4} \cdot {D_{opening}}^2 = 12.566\ \text{ft}^2$	Area of Conduit Opening
$W_{ledge} := 6.0\ \text{ft}$	Width of Foundation Ledge
$h_{wingwall} := h_{sidewall2} = 7.5\ \text{ft}$	Height of Downstream Wingwall
$t_{wingwall} := t_{sill} = 1.25\ \text{ft}$	Thickness of Downstream Wingwall

Calculate Weight of Impact Basin

$$F_{base} := L \cdot \left(W + W_{ledge} \cdot 2\right) \cdot t_{base} \cdot \gamma_c = 235.968\ \text{kip}$$

Weight of Base Slab

$$x_{base} := \frac{L}{2} = 12.29\ \text{ft}$$

Centroid of Base Slab

$$F_{headwall} := t_{headwall} \cdot \left(h_{headwall} \cdot W - 2 \cdot A_{opening}\right) \cdot \gamma_c = 83.456\ \text{kip}$$

Weight of Headwall Stem

$$x_{headwall} := L - \frac{t_{headwall}}{2} = 23.58\ \text{ft}$$

Centroid of Headwall Stem

$$F_{sidewall1} := 2 \cdot t_{sidewall} \cdot h_{sidewall1} \cdot L_1 \cdot \gamma_c = 82.624\ \text{kip}$$

Weight of Sidewall Stem 1

$$x_{sidewall1} := L - t_{headwall} - 0.5 \cdot L_1 = 18.04\ \text{ft}$$

Centroid of Sidewall Stem 1

$$F_{sidewall2} := 2 \cdot \frac{h_{sidewall1} + h_{sidewall2}}{2} \cdot L_2 \cdot t_{sidewall} \cdot \gamma_c = 83.298\ \text{kip}$$

Weight of Sidewall Stem 2

$$x_{sidewall2} := L - t_{headwall} - L_1 - \frac{L_2 \cdot \left(2 \cdot h_{sidewall2} + h_{sidewall1}\right)}{3 \cdot \left(h_{sidewall1} + h_{sidewall2}\right)} = 8.066\ \text{ft}$$

Centroid of Sidewall Stem 2

$$F_{sill} := t_{sill} \cdot h_{sill} \cdot W \cdot \gamma_c = 10.013\ \text{kip}$$

Weight of Sill Stem

$$x_{sill} := \frac{t_{sill}}{2} = 0.625\ \text{ft}$$

Centroid of Sill Stem

$$F_{baffle1} := t_{baffle1} \cdot h_{baffle} \cdot W_B \cdot \gamma_c = 24.3\ \text{kip}$$

Weight of Impact Baffle 1

$$x_{baffle1} := t_{sill} + L_2 + 0.5 \cdot t_{baffle1} = 14.25\ \text{ft}$$

Centroid of Impact Baffle 1

$$F_{baffle2} := \left(l_{baffle} - t_{baffle1}\right) \cdot t_{baffle2} \cdot W_B \cdot \gamma_c = 6.408\ \text{kip}$$

Weight of Impact Baffle 2

$$x_{baffle2} := t_{sill} + L_2 + t_{baffle1} + 0.5 \cdot \left(l_{baffle} - t_{baffle1}\right) = 16.335\ \text{ft}$$

Centroid of Impact Baffle 2

$F_{overhang1} := L_{overhang} \cdot t_{overhang} \cdot W_B \cdot \gamma_c = 0.8\ \text{kip}$ Weight of Overhang 1

$x_{overhang1} := L - t_{headwall} - 0.5 \cdot L_{overhang} = 22.33\ \text{ft}$ Centroid of Overhang 1

$F_{overhang2} := 2 \cdot L_{overhang} \cdot t_{overhang} \cdot L_1 \cdot \gamma_c = 0.908\ \text{kip}$ Weight of Overhang 2

$x_{overhang2} := L - t_{headwall} - \frac{L_1}{2} = 18.04\ \text{ft}$ Centroid of Overhang 2

$F_{wingwall} := 2 \cdot W_{ledge} \cdot h_{wingwall} \cdot t_{wingwall} \cdot \gamma_c = 16.875\ \text{kip}$ Weight of Downstream Wingwalls

$x_{wingwall} := \frac{t_{wingwall}}{2} = 0.625\ \text{ft}$ Centroid of Downstream Wingwalls

$P := F_{base} + F_{headwall} + F_{sidewall1} + F_{sidewall2} + F_{sill} + F_{baffle1} + F_{baffle2} + F_{overhang1} + F_{overhang2} + F_{wingwall} = 544.65\ \text{kip}$ Total Vertical Load

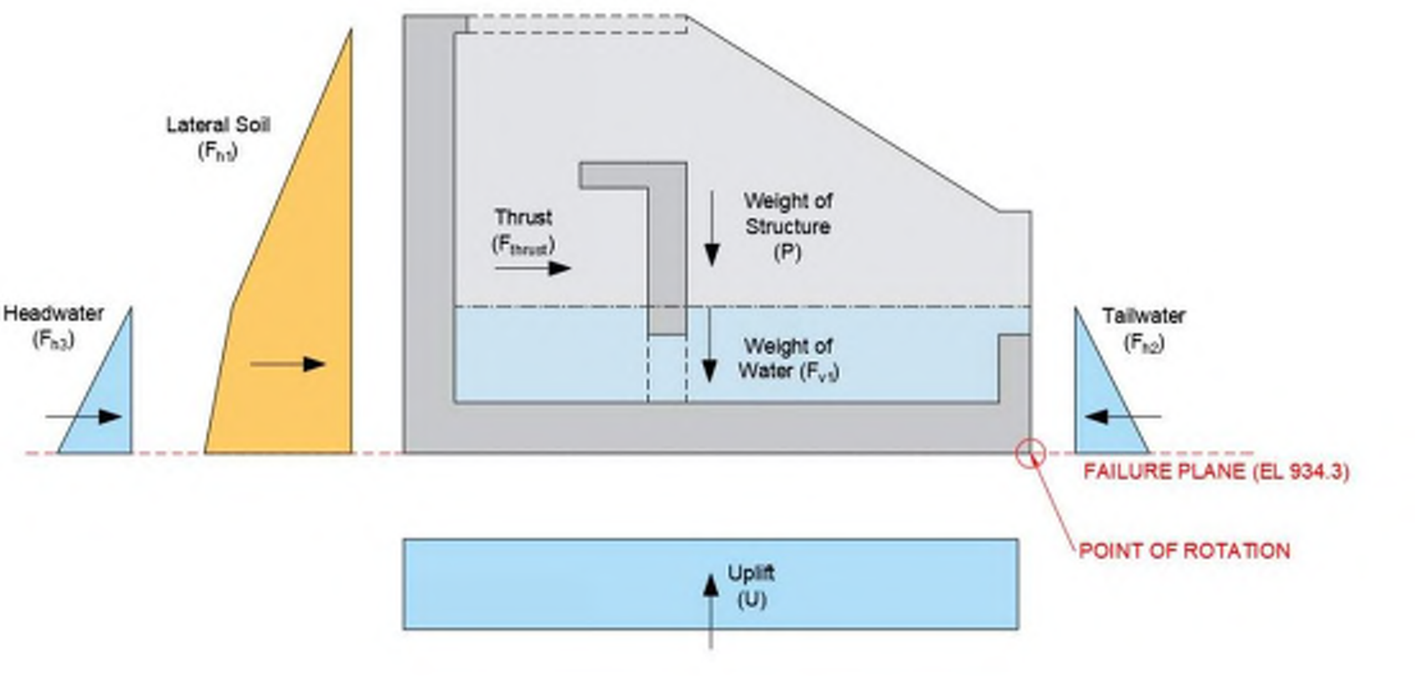
$$\Sigma M := F_{base} \cdot x_{base} + F_{headwall} \cdot x_{headwall} + F_{sidewall1} \cdot x_{sidewall1} + F_{sidewall2} \cdot x_{sidewall2} + F_{sill} \cdot x_{sill} \hookleftarrow \\ + F_{baffle1} \cdot x_{baffle1} + F_{baffle2} \cdot x_{baffle2} + F_{overhang1} \cdot x_{overhang1} + F_{overhang2} \cdot x_{overhang2} + F_{wingwall} \cdot x_{wingwall} = 7532.324\ \text{kip} \cdot \text{ft}$$

Sum of Moments

$x := \frac{\Sigma M}{P} = 13.83\ \text{ft}$ Centroid

Case 1 - Unusual Load Condition (1/3 PMF Event)

Free Body Diagram:



Calculate Vertical Forces

A) Uplift Force at Foundation/Concrete Interface

$U := \gamma_w \cdot h_{tw\_flood} \cdot (W + 2 \cdot W_{ledge}) \cdot L = 283.052 \text{ kip}$ Uplift Force

$x_u := \frac{L}{2} = 12.29 \text{ ft}$ Moment Arm

B) Weight of Water in Impact Basin

$F_{v1} := \gamma_w \cdot W_B \cdot (L_1 + L_2) \cdot (h_{tw\_flood} - t_{base}) = 80.222 \text{ kip}$ Weight of Water in Impact Basin

$x_{v1} := \frac{1}{2} \cdot (L_1 + L_2) + t_{sill} = 11.915 \text{ ft}$ Moment Arm

C) Weight of Soil on Ledges

$F_{v2} := \frac{1}{2} \cdot (h_{us\_soil} + h_{ds\_soil}) \cdot 2 \cdot W_{ledge} \cdot L \cdot \gamma_{s\_moist} = 465.546 \text{ kip}$ Weight of Soil Over Ledges

$x_{v2} := \frac{2 \cdot h_{us\_soil} + h_{ds\_soil}}{h_{us\_soil} + h_{ds\_soil}} \frac{L}{3} = 13.514 \text{ ft}$ Moment Arm

Calculate Horizontal Forces

A) Lateral Soil Force Upstream of Basin

$$F_{h1\_1} := \frac{1}{2} \cdot K_o \cdot \gamma_{s\_moist} \cdot W \cdot \left(h_{us\_soil} - h_{tw\_flood}\right)^2 = 70.871\ \text{kip}$$

Lateral Soil Force (At-Rest) from Dry Soil

$$x_{h1\_1} := h_{tw\_flood} + \left(h_{us\_soil} - h_{tw\_flood}\right) = 16.667\ \text{ft}$$

Moment Arm for Component 1

$$F_{h1\_2} := \frac{1}{2} \cdot K_o \cdot \gamma_{s\_moist} \cdot W \cdot \left(h_{us\_soil} - h_{tw\_flood}\right) \cdot h_{tw\_flood} = 37.496\ \text{kip}$$

Lateral Soil Force (At-Rest) from Dry Soil Above

$$x_{h1\_2} := h_{tw\_flood} \cdot .5 = 2.883\ \text{ft}$$

Moment Arm for Component 2

$$F_{h1\_3} := \frac{1}{2} \cdot K_o \cdot \left(\gamma_{s\_sat} - \gamma_w\right) \cdot W \cdot h_{tw\_flood}{}^2 = 10.581\ \text{kip}$$

Lateral Soil Force (At-Rest) from Submerged Soil

$$x_{h1\_3} := \frac{h_{tw\_flood}}{3} = 1.922\ \text{ft}$$

Moment Arm for Component 3

$$F_{h1} := F_{h1\_1} + F_{h1\_2} + F_{h1\_3} = 118.948\ \text{kip}$$

Lateral Soil Force (At-Rest)

$$x_{h1} := \frac{F_{h1\_1} \cdot x_{h1\_1} + F_{h1\_2} \cdot x_{h1\_2} + F_{h1\_3} \cdot x_{h1\_3}}{F_{h1}} = 11.01\ \text{ft}$$

Moment Arm

B) Thrust Force on Impact Baffle


Jet Force on an angled plate ($\theta = 180^{\circ}$)

$u_1 = V$ and $u_2 = V \cos\theta$ therefore

$F = Q\rho V\,(1 - \cos 180^{\circ}) = 2Q\rho V = 2\rho A V^2$

Figure 4 - Jet Propulsion on Impact Baffle [https://roymech.org/Related/Fluids/Fluids_Jets.html]

$$\rho := \frac{\gamma_w}{g}$$

Mass Density of Water

$$F_{thrust} := 2 \cdot Q_{max} \cdot \rho \cdot v_{max} = 32.079\ \text{kip}$$

Impact Force (Conservatively Assume Jet Force on Angled Plate)

$$x_{thrust} := t_{base} + h_{sill} + \frac{D_{opening}}{2} = 6.67\ \text{ft}$$

Moment Arm

C) Horizontal Water Forces (Upstream and Downstream)

$$F_{h2} := 0.6 \cdot \frac{1}{2} \cdot \gamma_w \cdot {h_{tw\_flood}}^2 \cdot W = 12.452\ \text{kip}$$

Lateral Tailwater Force (60% Reduction for Tailwater Retrogression)

$$x_{h2} := \frac{h_{tw\_flood}}{3} = 1.922\ \text{ft}$$

Moment Arm

$$F_{h3} := \frac{1}{2} \cdot \gamma_w \cdot {h_{tw\_flood}}^2 \cdot W = 20.753\ \text{kip}$$

Lateral Headwater Force

$$x_{h3} := \frac{h_{tw\_flood}}{3} = 1.922\ \text{ft}$$

Moment Arm

Check Sliding Stability

$$F_D := F_{thrust} + F_{h1} + F_{h3} - F_{h2} = 159.328\ \text{kip}$$

Driving Force

$$F_R := \left(P - U + F_{v1} + F_{v2}\right) \cdot \mu_f = 485.114\ \text{kip}$$

Resisting Force

$$FS_{Sliding} := \frac{F_R}{F_D} = 3.045$$

Factor of Safety for Sliding

```
Sliding_Stability := if $FS_{Sliding} \geq 1.3$
                         "PASS"
                     else
                         "FAIL"
```

Sliding_Stability = “PASS”

Check Flotation Stability

$$FS_{Flotation} := \frac{P + F_{v2}}{U - F_{v1}} = 4.98$$

Factor of Safety for Flotation

```
Flotation_Stability := if $FS_{Flotation} \geq 1.2$
                           "PASS"
                       else
                           "FAIL"
```

Flotation_Stability = “PASS”

Check Overturning Stability

$M_{OT} := U \cdot x_u + F_{h1} \cdot x_{h1} + F_{thrust} \cdot x_{thrust} + F_{h3} \cdot x_{h3} = 5042.2\ \text{kip} \cdot \text{ft}$

Total Overturning Moment

$M_R := P \cdot x + F_{h2} \cdot x_{h2} + F_{v1} \cdot x_{v1} + F_{v2} \cdot x_{v2} = 14803.6\ \text{kip} \cdot \text{ft}$

Total Restoring Moment

$FS_{Overturning} := \dfrac{M_R}{M_{OT}} = 2.936$

Factor of Safety Against Overturning (Note overturning stability is evaluated based on location of resultant, see below)

Resultant Location:

$F_N := P - U + F_{v1} + F_{v2} = 807.365\ \text{kip}$

Total Vertical Load Resisting Overturning

$X := \dfrac{M_R - M_{OT}}{F_N} = 12.09\ \text{ft}$

Center of Total Weight from Edge of Toe

$e_{cc} := \dfrac{L}{2} - X = 0.2\ \text{ft}$

Eccentricity of the Resultant

$\dfrac{L}{6} = 4.097\ \text{ft}$

Limit for Base Being in Compression Only

```
Resultant_Location := if |e_cc| ≤ L/6
                         "100% of Base in Compression"
                      else if L/6 < |e_cc| ≤ L/4
                         "75% of Base in Compression"
                      else if L/4 < |e_cc| ≤ L/2
                         "Resultant Within Base"
                      else
                         "Unstable-Resultant Outside Base"
```

Resultant_Location = "100% of Base in Compression"

The resultant location is within the middle third of the base and thus the base is 100% in compression. The requirement for overturning for Case 1 is the resultant force should be in the middle half of the base (EM 1110-2-2100). Meets the requirements.

Check Bearing Stability

$$q_{max} := \begin{array}{l} \text{if } \left|e_{cc}\right| \le \frac{L}{6} \\ \quad \frac{F_N}{L \cdot \left(W + 2 \cdot W_{ledge}\right)} \cdot \left(1 + \frac{6 \cdot e_{cc}}{L}\right) \\ \text{else} \\ \quad \frac{4 \cdot F_N}{3 \cdot \left(W + 2 \cdot W_{ledge}\right) \cdot \left(L - 2 \cdot e_{cc}\right)} \end{array}$$

$q_{max} = 7.475 \text{ psi}$ $q_{max} = 1076 \text{ psf}$

Equations 3-1 and 3-2 from EM 1110-2-2502

$$q_{min} := \begin{array}{l} \text{if } \left|e_{cc}\right| \le \frac{L}{6} \\ \quad \frac{F_N}{L \cdot \left(W + 2 \cdot W_{ledge}\right)} \cdot \left(1 - \frac{6 \cdot e_{cc}}{L}\right) \\ \text{else} \\ \quad 0 \text{ psi} \end{array}$$

$q_{min} = 6.781 \text{ psi}$ $q_{min} = 976 \text{ psf}$

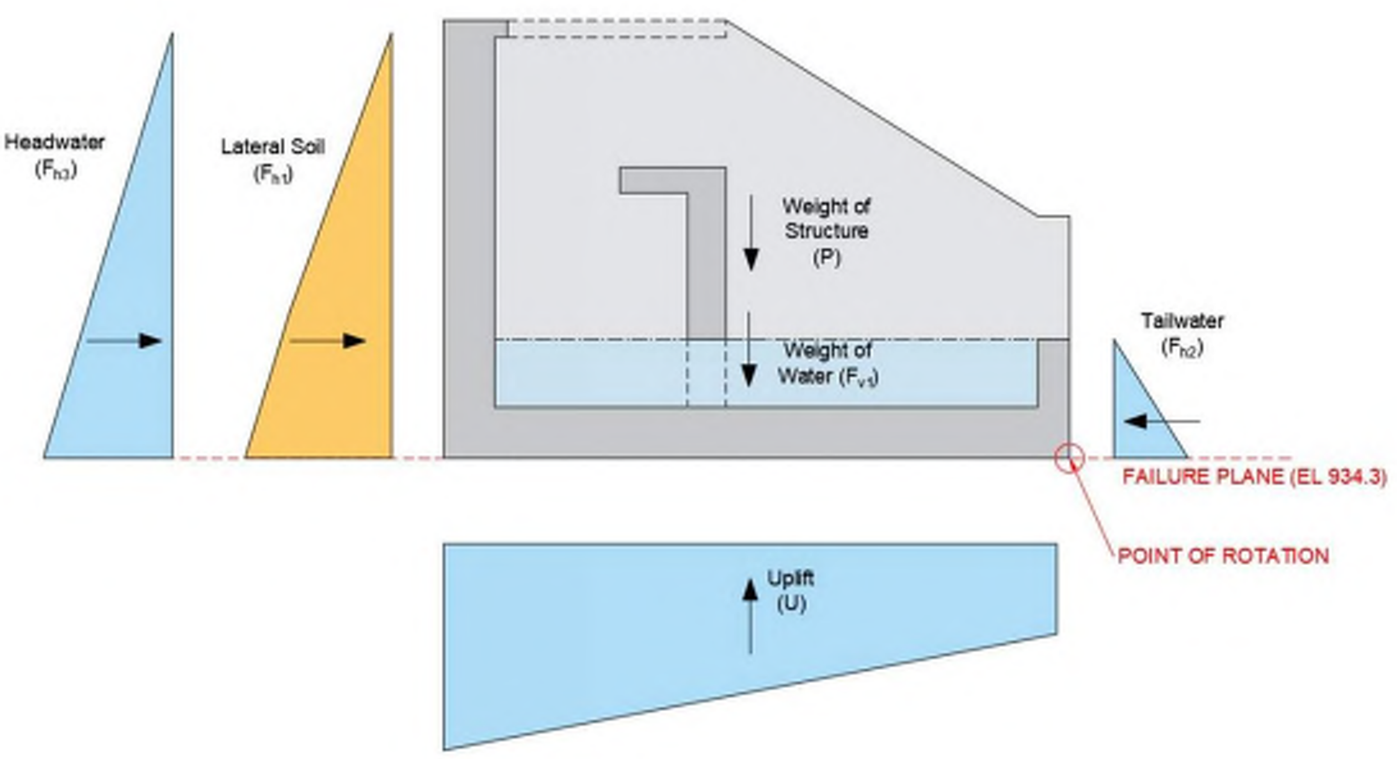
$$\text{Bearing\_Stability} := \begin{array}{l} \text{if } 1.15 \cdot \sigma_{all} > q_{max} \\ \quad \text{“OK”} \\ \text{else} \\ \quad \text{“NO GOOD”} \end{array}$$

$\text{Bearing\_Stability} = \text{“OK”}$

The allowable bearing capacity of the foundation is not exceeded. Additionally, the stresses are significantly less than the assumed strength of concrete and as a result, concrete crushing is not considered to be a viable failure mechanism.

Case 2 - Unusual Load Condition (Floodwater Recedes)

Free Body Diagram:



Calculate Vertical Forces

A) Uplift Force at Foundation/Concrete Interface

$$U := \gamma_w \cdot 0.5 \cdot \left(h_{tw} + h_{us\_soil}\right) \cdot \left(W + 2 \cdot W_{ledge}\right) \cdot L = 523.551 \ \text{kip}$$

Uplift Force

$$x_u := \frac{2 \cdot h_{us\_soil} + h_{tw}}{h_{us\_soil} + h_{tw}} \cdot \frac{L}{3} = 14.594 \ \text{ft}$$

Moment Arm

B) Weight of Water in Impact Basin

$$F_{v1} := \gamma_w \cdot W_B \cdot \left(L_1 + L_2\right) \cdot \left(h_{tw} - t_{base}\right) = 56.796 \ \text{kip}$$

Weight of Water in Impact Basin

$$x_{v1} := \frac{1}{2} \cdot \left(L_1 + L_2\right) + t_{sill} = 11.915 \ \text{ft}$$

Moment Arm

C) Weight of Soil on Ledges

$$F_{v2} := \frac{1}{2} \cdot \left(h_{us\_soil} + h_{ds\_soil}\right) \cdot 2 \cdot W_{ledge} \cdot L \cdot \gamma_{s\_sat} = 484.471 \ \text{kip}$$

Weight of Soil Over Ledges

$$x_{v2} := \frac{2 \cdot h_{us\_soil} + h_{ds\_soil}}{h_{us\_soil} + h_{ds\_soil}} \cdot \frac{L}{3} = 13.514 \ \text{ft}$$

Moment Arm

Calculate Horizontal Forces

A) Lateral Soil Force Upstream of Basin

$$F_{h1} := \frac{1}{2} \cdot K_o \cdot \left(\gamma_{s\_sat} - \gamma_w\right) \cdot W \cdot \left(h_{us\_soil}\right)^2 = 88.374 \text{ kip}$$

Lateral Soil Force (At-Rest) from Saturated Soil

$$x_{h1} := \frac{h_{us\_soil}}{3} = 5.556 \text{ ft}$$

Moment Arm

B) Horizontal Water Forces (Upstream and Downstream)

$$F_{h2} := \frac{1}{2} \cdot \gamma_w \cdot {h_{tw}}^2 \cdot W = 13.591 \text{ kip}$$

Lateral Tailwater Force

$$x_{h2} := \frac{h_{tw}}{3} = 1.556 \text{ ft}$$

Moment Arm

$$F_{h3} := \frac{1}{2} \cdot \gamma_w \cdot {h_{us\_soil}}^2 \cdot W = 173.34 \text{ kip}$$

Lateral Headwater Force

$$x_{h3} := \frac{h_{us\_soil}}{3} = 5.556 \text{ ft}$$

Moment Arm

Check Sliding Stability

$F_D := F_{h1} + F_{h3} - F_{h2} = 248.123\ \text{kip}$

Driving Force

$F_R := \left(P - U + F_{v1} + F_{v2}\right) \cdot \mu_f = 337.903\ \text{kip}$

Resisting Force

$$FS_{Sliding} := \frac{F_R}{F_D} = 1.362$$

Factor of Safety for Sliding

$$\begin{array}{l} \text{Sliding\_Stability} := \text{if } FS_{Sliding} \geq 1.3 \\ \qquad \Vert \text{“PASS”} \\ \quad \text{else} \\ \qquad \Vert \text{“FAIL”} \end{array}$$

Sliding_Stability = “PASS”

Check Flotation Stability

$$FS_{Flotation} := \frac{P + F_{v2}}{U - F_{v1}} = 2.205$$

Factor of Safety for Flotation

$$\begin{array}{l} \text{Flotation\_Stability} := \text{if } FS_{Flotation} \geq 1.2 \\ \qquad \Vert \text{“PASS”} \\ \quad \text{else} \\ \qquad \Vert \text{“FAIL”} \end{array}$$

Flotation_Stability = “PASS”

Check Overturning Stability

$$M_{OT} := U \cdot x_u + F_{h1} \cdot x_{h1} + F_{h3} \cdot x_{h3} = 9094.9 \text{ kip} \cdot \text{ft}$$

Total Overturning Moment

$$M_R := P \cdot x + F_{h2} \cdot x_{h2} + F_{v1} \cdot x_{v1} + F_{v2} \cdot x_{v2} = 14777.5 \text{ kip} \cdot \text{ft}$$

Total Restoring Moment

$$FS_{Overturning} := \frac{M_R}{M_{OT}} = 1.625$$

Factor of Safety Against Overturning (Note overturning stability is evaluated based on location of resultant, see below)

Resultant Location:

$$F_N := P - U + F_{v1} + F_{v2} = 562.366 \text{ kip}$$

Total Vertical Load Resisting Overturning

$$X := \frac{M_R - M_{OT}}{F_N} = 10.105 \text{ ft}$$

Center of Total Weight from Edge of Toe

$$e_{cc} := \frac{L}{2} - X = 2.185 \text{ ft}$$

Eccentricity of the Resultant

$$\frac{L}{6} = 4.097 \text{ ft}$$

Limit for Base Being in Compression Only

```
Resultant_Location := if |e_cc| ≤ L/6
                          "100% of Base in Compression"
                      else if L/6 < |e_cc| ≤ L/4
                          "75% of Base in Compression"
                      else if L/4 < |e_cc| ≤ L/2
                          "Resultant Within Base"
                      else
                          "Unstable-Resultant Outside Base"
```

Resultant_Location = “100% of Base in Compression”

The resultant location is within the middle third of the base and thus the base is 100% in compression. The requirement for overturning for Case 2 is the resultant force should be in the middle half of the base (EM 1110-2-2100). Meets the requirements.

Check Bearing Stability

$$q_{max} := \begin{cases} \text{if } |e_{cc}| \le \dfrac{L}{6} \\ \quad \dfrac{F_N}{L \cdot (W + 2 \cdot W_{ledge})} \cdot \left(1 + \dfrac{6 \cdot e_{cc}}{L}\right) \\ \text{else} \\ \quad \dfrac{4 \cdot F_N}{3 \cdot (W + 2 \cdot W_{ledge}) \cdot (L - 2 \cdot e_{cc})} \end{cases}$$

$q_{max} = 7.613 \text{ psi}$ $q_{max} = 1096 \text{ psf}$

Equations 3-1 and 3-2 from EM 1110-2-2502

$$q_{min} := \begin{cases} \text{if } |e_{cc}| \le \dfrac{L}{6} \\ \quad \dfrac{F_N}{L \cdot (W + 2 \cdot W_{ledge})} \cdot \left(1 - \dfrac{6 \cdot e_{cc}}{L}\right) \\ \text{else} \\ \quad 0 \text{ psi} \end{cases}$$

$q_{min} = 2.317 \text{ psi}$ $q_{min} = 334 \text{ psf}$

$$\text{Bearing\_Stability} := \begin{cases} \text{if } 1.15 \cdot \sigma_{all} > q_{max} \\ \quad \text{“OK”} \\ \text{else} \\ \quad \text{“NO GOOD”} \end{cases}$$

$\text{Bearing\_Stability} = \text{“OK”}$

The allowable bearing capacity of the foundation is not exceeded. Additionally, the stresses are significantly less than the assumed strength of concrete and as a result, concrete crushing is not considered to be a viable failure mechanism.

IMPACT BASIN BAFFLE

Project: Lake Erin Dam Rehabilitation

Task: Impact Basin Baffle Analysis - Horizontal and Vertical Bending

Description:

Design concrete USBR Type VI Impact Basin baffle structure in accordance with ACI 350-20, Code Requirements for Environmental Structures.

Codes, Standards, & References:

1. ACI 350-20, Code Requirements for Environmental Structures
2. Civil Engineering Reference Manual 13th Edition
3. AECOM Basis of Design Report

Hydrologic and Hydraulic:

Maximum conduit discharge and tailwater elevation is based on AECOM's hydraulic analysis. The maximum discharge out of the pipe is 322 cfs with a maximum velocity of 25.7 ft/s, which is based on the spillway design flood (1/3 PMF).

Assumptions:

- Design the horizontal reinforcement with the assumption that the baffle behaves as a fixed-fixed beam spanning between the sidewalls and the water force can be conservatively approximated as a point load.
- Horizontal reinforcement is #6 bars spaced 6 inches.
- Design the vertical reinforcement with the assumption that the baffle behaves as a fixed cantilever and the water force is a partially distributed load.
- Vertical reinforcement is #6 bars spaced 6 inches.

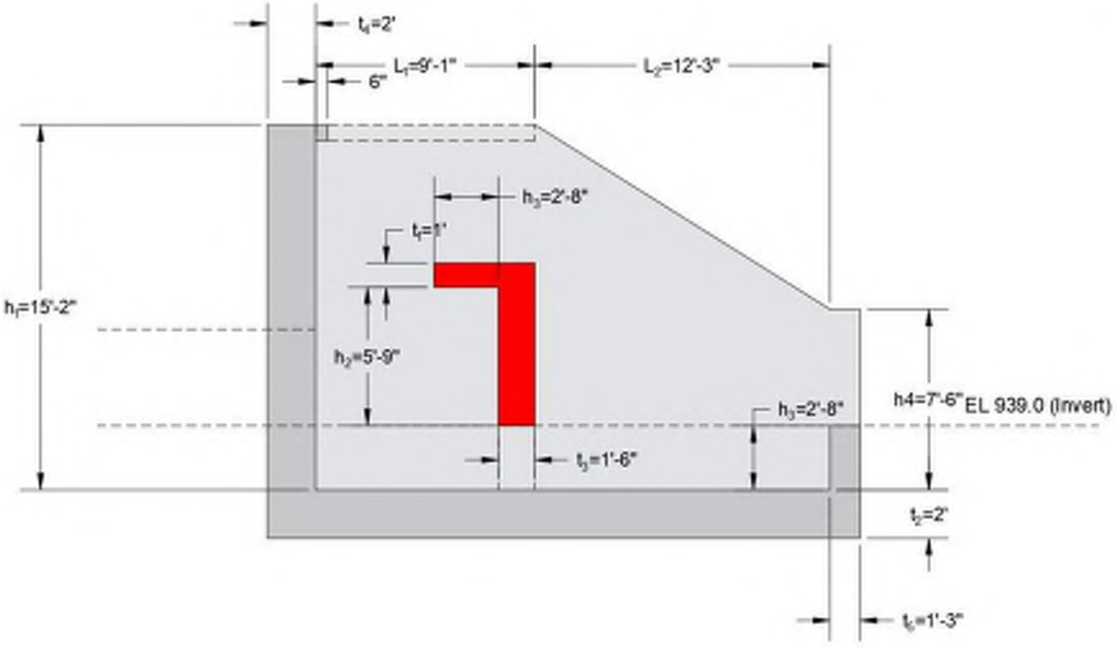


Figure 1 - Impact Basin Cross Section and Impact Baffle (In Red)

Material Properties

$\gamma_c := 150\ \text{pcf}$	Unit Weight of Concrete
$\gamma_w := 62.4\ \text{pcf}$	Unit Weight of Water
$f'_c := 5000\ \text{psi}$	Concrete Compressive Strength
$f_y := 60000\ \text{psi}$	Yield Strength of Reinforcement

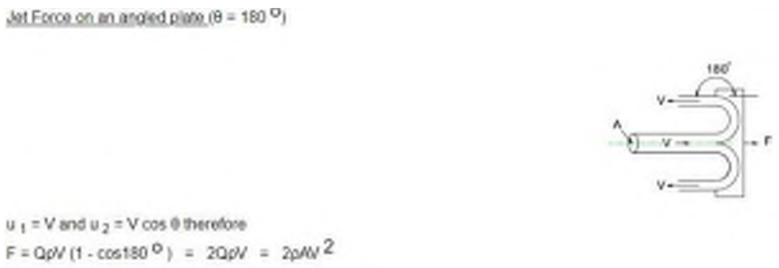
Hydrologic and Hydraulic Parameters

$Q_{max} := 322.3\ \dfrac{ft^3}{s}$	Design Discharge
$v_{max} := 25.66\ \dfrac{ft}{s}$	Maximum Velocity

Baffle Details & Geometry

$EL_{baffle} := 945.75\ ft$	Top of Baffle Elevation
$EL_{bot} := 939.0\ ft$	Bottom of Baffle Elevation
$H_{baffle} := EL_{baffle} - EL_{bot} = 6.75\ ft$	Height of Baffle
$t_{baffle\_h} := 12\ in$	Thickness of Baffle (Horizontal Segment)
$t_{baffle\_v} := 18\ in$	Thickness of Baffle (Vertical Segment)
$L_{baffle} := H_{baffle} - t_{baffle\_h} = 5.75\ ft$	Length of Face of Baffle
$b := 12\ in$	Unit Width
$W_b := 16\ ft$	Width of Baffle Block
$D_{inside} := 4\ ft$	Conduit Inside Diameter
$A_{inside} := \dfrac{D_{inside}^{\ 2} \cdot \pi}{4} = 12.566\ ft^2$	Conduit Inside Area

Jet Force on Impact Baffle



Jet Force on an angled plate ($\theta = 180^{\circ}$)

$u_1 = V$ and $u_2 = V \cos\theta$ therefore

$F = Q\rho V(1 - \cos 180^{\circ}) = 2Q\rho V = 2\rho AV^2$

Figure 2 - Jet Propulsion on Impact Baffle

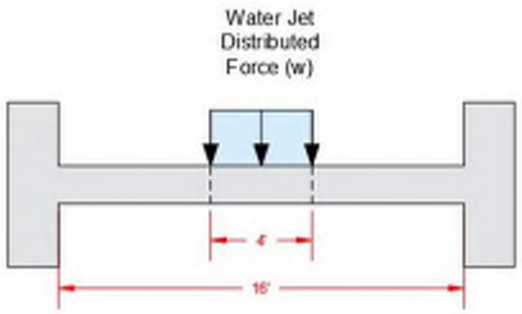
$\rho := \frac{\gamma_w}{g}$ Mass Density of Water

$F_{jet} := 2 \cdot Q_{max} \cdot \rho \cdot v_{max} = 32.079 \text{ kip}$ Impact Force (Conservatively Assume Jet Force on Angled Plate)

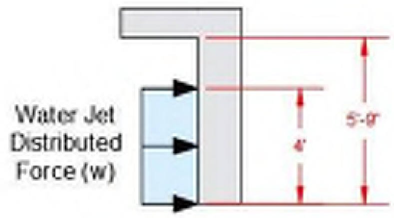
$w := \frac{F_{jet}}{A_{inside}} \cdot b = 2.553 \frac{\text{kip}}{\text{ft}}$ Distributed Load

Free Body Diagrams

Case 1: Horizontal Loading (Conservatively Assumed as Point Load at Center of Fixed-Fixed Beam)



Case 2: Vertical Loading (Distributed Load on Fixed Cantilever Beam)



Check Horizontal Loading

Determine Reactions on Impact Baffle

Analyze the impact baffle with the assumption that the baffle behaves as a fixed-fixed beam spanning from the sidewalls and the jet force is a point load at the center.

$\phi_{FL} := 1.4$	Fluid Load Factor Per ACI 350-20 Section 9.2.1
$M := \dfrac{w \cdot D_{inside} \cdot W_b}{8} = 20.422\ kip \cdot ft$	Service Moment
$M_u := \phi_{FL} \cdot M = 28.591\ kip \cdot ft$	Factored Moment
$V := \dfrac{w \cdot D_{inside}}{2} = 5.106\ kip$	Service Shear Force
$V_u := \phi_{FL} \cdot V = 7.148\ kip$	Factored Shear Force

Impact Baffle Concrete Reinforcement Design Per ACI 350-20

Rectangular Section in Flexure:

$d_b := 0.75\ in$	No. 6 Bar Diameter (Assumed Bar Size)
$A_b := \dfrac{\pi \cdot {d_b}^2}{4} = 0.442\ in^2$	Area of #6 Bar
$h := t_{baffle\_v} = 1.5\ ft$	Depth of Section
$c_c := 2\ in$	Cover to Reinforcement
$s := 6\ in$	Bar Spacing
$d := h - c_c - \dfrac{d_b}{2} = 15.625\ in$	Depth to Reinforcement
$A_{s\_prov} := \dfrac{b}{s} \cdot A_b = 0.884\ in^2$	Total Area of Reinforcement Provided
$a := \dfrac{A_{s\_prov} \cdot f_y}{0.85 \cdot f'_c \cdot b} = 1.039\ in$	Depth of Equivalent Stress Block

Calculate Net Tensile Strain:

$$\beta_1 := 0.85 - .00005 \cdot \frac{in^2}{lbf} \cdot \left(f'_c - 4000\ psi\right) = 0.8$$

Equivalent Depth Factor

$$\varepsilon_t := \frac{0.003 \cdot \left(\beta_1 \cdot d - a\right)}{a} = 0.033$$

Net Tensile Strain

$$\phi_{RF} := \left.\begin{array}{l} \text{if } \varepsilon_t \geq 0.005 \\ \quad \| \ 0.9 \\ \text{else} \\ \quad \| \ 0.65 \end{array}\right| = 0.9$$

Strength Reduction Factor

$\varepsilon_t \geq 0.005$ Tension-Controlled Section

Calculate Environmental Durability Factor:

$$\beta := \left.\begin{array}{l} \text{if } h \geq 16\ in \\ \quad \| \ 1.2 \\ \text{else} \\ \quad \| \ 1.35 \end{array}\right| = 1.2$$

Strain Gradient Factor

$$\gamma := \frac{M_u}{M} = 1.4$$

Combined Load Factor (Section 9.2.6)

$$f_{smax} := \frac{320\ \frac{kip}{in}}{\beta \cdot \sqrt{s^2 + 4 \cdot \left(2\ in + \frac{d_b}{2}\right)^2}} = 34.846\ ksi$$

Permissible Stress in Reinforcement for Normal Environmental Exposure (Section 10.6.4.5)

$$S_d := \max\left(\frac{\phi_{RF} \cdot f_y}{\gamma \cdot f_{smax}}, 1.0\right) = 1.107$$

Environmental Durability Factor (Section 9.2.6)

$$M_{u1} := S_d \cdot M_u = 31.648\ kip \cdot ft$$

Required Moment Strength

Check Flexural Strength:

$$M_n := A_{s\_prov} \cdot f_y \cdot \left(d - \frac{a}{2}\right) = 66.733\ kip \cdot ft$$

Nominal Flexural Strength

$$\phi_{RF} \cdot M_n = 60.06\ kip \cdot ft$$

Design Flexural Strength

$$\frac{M_{u1}}{\phi_{RF} \cdot M_n} = 0.527$$

Demand Capacity Ratio

$$\text{Check}_{\text{Flex}} := \text{if } M_{u1} \le \phi_{RF} \cdot M_n \;\Big\|\; \text{“PASS”} \;\; \text{else} \;\Big\|\; \text{“FAIL”}$$

$\text{Check}_{\text{Flex}} = \text{“PASS”}$

Check Minimum Area of Reinforcement Required:

$A_{s\_prov} = 0.884 \text{ in}^2$

Area of Reinforcement Provided

$$A_{smin} := \max\left(\frac{3 \cdot \sqrt{\frac{f'_c}{psi}} \cdot psi}{f_y} \cdot b \cdot d, \frac{200 \text{ psi} \cdot b \cdot d}{f_y}\right) = 0.663 \text{ in}^2$$

Minimum Area of Reinforcement Required (Section 10.5.1)

$$\text{Check}_{\text{Reinforcement}} := \text{if } A_{smin} \le A_{s\_prov} \;\Big\|\; \text{“PASS”} \;\; \text{else} \;\Big\|\; \text{“FAIL”}$$

$\text{Check}_{\text{Reinforcement}} = \text{“PASS”}$

Rectangular Section in Shear (Per ACI 350-20 Section 11.1):

$\lambda := 1$

Normalweight Concrete

$\phi_s := 0.75$

Shear Reduction Factor

$V_u = 7.148 \text{ kip}$

Factored Shear Force

$$V_c := 2 \text{ psi} \cdot \lambda \cdot \sqrt{\frac{f'_c}{psi}} \cdot b \cdot d = 26.517 \text{ kip}$$

Nominal Shear Strength Provided by Concrete

Demand Capacity Ratio

$$\frac{V_u}{\phi_s \cdot V_c} = 0.359$$

$$\text{Check}_{\text{Shear}} := \text{if } V_u \le \phi_s \cdot V_c \;\Big\|\; \text{“PASS”} \;\; \text{else} \;\Big\|\; \text{“FAIL”}$$

$\text{Check}_{\text{Shear}} = \text{“PASS”}$

Check Vertical Bending

Determine Reactions on Impact Baffle

Analyze the impact baffle with the assumption that the vertical baffle section behaves as a fixed cantilever beam spanning from the horizontal baffle section and the jet force is effectively a partially distributed load.

$\phi_{FL} = 1.4$ Fluid Load Factor Per ACI 350-20 Section 9.2.1

$M := \dfrac{w \cdot D_{inside} \cdot \left(L_{baffle} + \left(L_{baffle} - D_{inside}\right)\right)}{2} = 38.292\ kip \cdot ft$ Service Moment

$M_u := \phi_{FL} \cdot M = 53.609\ kip \cdot ft$ Factored Moment

$V := w \cdot D_{inside} = 10.211\ kip$ Service Shear Force

$V_u := \phi_{FL} \cdot V = 14.296\ kip$ Factored Shear Force

Impact Baffle Concrete Reinforcement Design Per ACI 350-20

Rectangular Section in Flexure:

$d_b := 0.75\ in$ No. 6 Bar Diameter (Assumed Bar Size)

$A_b := \dfrac{\pi \cdot {d_b}^2}{4} = 0.442\ in^2$ Area of #6 Bar

$h := t_{baffle\_v} = 1.5\ ft$ Depth of Section

$c_c := 2\ in$ Cover to Reinforcement

$s := 6\ in$ Bar Spacing

$d := h - c_c - \dfrac{d_b}{2} = 15.625\ in$ Depth to Reinforcement

$A_{s\_prov} := \dfrac{b}{s} \cdot A_b = 0.884\ in^2$ Total Area of Reinforcement Provided

$a := \dfrac{A_{s\_prov} \cdot f_y}{0.85 \cdot f'_c \cdot b} = 1.039\ in$ Depth of Equivalent Stress Block

Calculate Net Tensile Strain:

$$\beta_1 := 0.85 - .00005 \cdot \frac{in^2}{lbf} \cdot \left(f'_c - 4000\ psi\right) = 0.8$$

Equivalent Depth Factor

$$\varepsilon_t := \frac{0.003 \cdot \left(\beta_1 \cdot d - a\right)}{a} = 0.033$$

Net Tensile Strain

$$\phi_{RF} := \left. \begin{array}{l} \text{if } \varepsilon_t \geq 0.005 \\ \quad \| \ 0.9 \\ \text{else} \\ \quad \| \ 0.65 \end{array} \right| = 0.9$$

Strength Reduction Factor

$\varepsilon_t \geq 0.005$ Tension-Controlled Section

Calculate Environmental Durability Factor:

$$\beta := \left. \begin{array}{l} \text{if } h \geq 16\ in \\ \quad \| \ 1.2 \\ \text{else} \\ \quad \| \ 1.35 \end{array} \right| = 1.2$$

Strain Gradient Factor

$$\gamma := \frac{M_u}{M} = 1.4$$

Combined Load Factor (Section 9.2.6)

$$f_{smax} := \frac{320\ \frac{kip}{in}}{\beta \cdot \sqrt{s^2 + 4 \cdot \left(2\ in + \frac{d_b}{2}\right)^2}} = 34.846\ ksi$$

Permissible Stress in Reinforcement for Normal Environmental Exposure (Section 10.6.4.5)

$$S_d := \max\left(\frac{\phi_{RF} \cdot f_y}{\gamma \cdot f_{smax}}, 1.0\right) = 1.107$$

Environmental Durability Factor (Section 9.2.6)

$$M_{u1} := S_d \cdot M_u = 59.339\ kip \cdot ft$$

Required Moment Strength

Check Flexural Strength:

$$M_n := A_{s\_prov} \cdot f_y \cdot \left(d - \frac{a}{2}\right) = 66.733\ kip \cdot ft$$

Nominal Flexural Strength

$$\phi_{RF} \cdot M_n = 60.06\ kip \cdot ft$$

Design Flexural Strength

$$\frac{M_{u1}}{\phi_{RF} \cdot M_n} = 0.988$$

Demand Capacity Ratio

$$\begin{array}{l} \text{Check}_{\text{Flex}} := \text{if } M_{u1} \le \phi_{RF} \cdot M_n \\ \qquad \left\| \text{“PASS”} \right. \\ \quad \text{else} \\ \qquad \left\| \text{“FAIL”} \right. \end{array}$$

$\text{Check}_{\text{Flex}} = \text{“PASS”}$

Check Minimum Area of Reinforcement Required:

$A_{s\_prov} = 0.884 \text{ in}^2$

Area of Reinforcement Provided

$$A_{smin} := \max\left(\frac{3 \cdot \sqrt{\dfrac{f'_c}{psi}} \cdot psi}{f_y} \cdot b \cdot d \, , \frac{200 \text{ psi} \cdot b \cdot d}{f_y} \right) = 0.663 \text{ in}^2$$

Minimum Area of Reinforcement Required (Section 10.5.1)

$$\begin{array}{l} \text{Check}_{\text{Reinforcement}} := \text{if } A_{smin} \le A_{s\_prov} \\ \qquad \left\| \text{“PASS”} \right. \\ \quad \text{else} \\ \qquad \left\| \text{“FAIL”} \right. \end{array}$$

$\text{Check}_{\text{Reinforcement}} = \text{“PASS”}$

Rectangular Section in Shear (Per ACI 350-20 Section 11.1):

$\lambda := 1$

Normalweight Concrete

$\phi_s := 0.75$

Shear Reduction Factor

$V_u = 14.296 \text{ kip}$

Factored Shear Force

$$V_c := 2 \text{ psi} \cdot \lambda \cdot \sqrt{\frac{f'_c}{psi}} \cdot b \cdot d = 26.517 \text{ kip}$$

Nominal Shear Strength Provided by Concrete

Demand Capacity Ratio

$$\frac{V_u}{\phi_s \cdot V_c} = 0.719$$

$$\begin{array}{l} \text{Check}_{\text{Shear}} := \text{if } V_u \le \phi_s \cdot V_c \\ \qquad \left\| \text{“PASS”} \right. \\ \quad \text{else} \\ \qquad \left\| \text{“FAIL”} \right. \end{array}$$

$\text{Check}_{\text{Shear}} = \text{“PASS”}$

IMPACT BASIN HEADWALL

Project: Lake Erin Dam Rehabilitation

Task: Headwall Design - Impact Basin

Description:

Design USBR Type VI Impact Basin Headwall structure (vertical and horizontal reinforcement) in accordance with ACI 350-20, Code Requirements for Environmental Structures.

Codes, Standards, & References:

1. ACI 350-20, Code Requirements for Environmental Structures
2. AECOM Basis of Design Report

Assumptions:

- Seepage analysis indicated phreatic surface maximum elevation of EL 943.0 ft. However, elevation conservatively assumed to be midpoint of headwall in calculations below (EL. 943.9 ft).
- Geotechnical parameters provided by geotechnical engineers as part of field investigation.
- Design the Headwall vertical reinforcement on a unit width basis and the assumption that the wall behaves as a cantilever wall.
- Vertical reinforcement is #5 bars spaced 6 inches.
- Design the Headwall horizontal reinforcement with the assumption that the wall behaves as a simply-supported beam spanning between the sidewalls.
- Horizontal reinforcement is #5 bars spaced 6 inches.

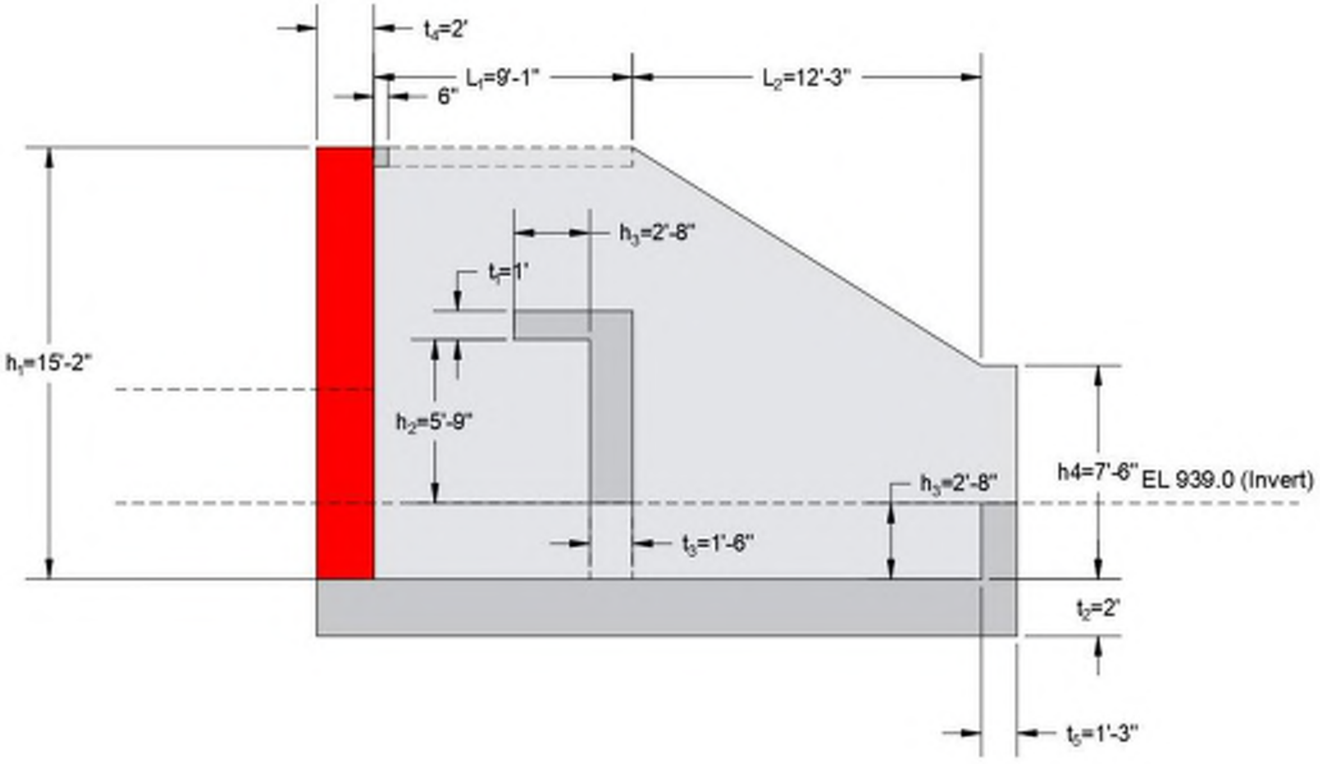


Figure 1 - Impact Basin Cross Section and Headwall (In Red)

Material Properties

$\gamma_c := 150\ \text{pcf}$	Unit Weight of Concrete
$\gamma_w := 62.4\ \text{pcf}$	Unit Weight of Water
$f'_c := 5000\ \text{psi}$	Concrete Compressive Strength
$f_y := 60000\ \text{psi}$	Yield Strength of Reinforcement
$\gamma_{moist\_fill} := 123\ \text{pcf}$	Moist Unit Weight of Backfill
$\gamma_{sat\_fill} := 128\ \text{pcf}$	Saturated Unit Weight of Backfill
$\phi_b := 31\ \text{deg}$	Internal Friction Angle of Backfill
$K_o := 1 - \sin\left(\phi_b\right) = 0.485$	At-Rest Earth Pressure Coefficient

Sidewall Details & Geometry

$EL_{headwall} := 951.5\ \text{ft}$	Top of Headwall Elevation
$EL_{bot} := 936.33\ \text{ft}$	Base of Headwall Elevation
$EL_{s1} := 951.0\ \text{ft}$	Elevation of Upstream Grade
$EL_{HW} := 943.9\ \text{ft}$	Elevation of Upstream Groundwater
$H_{s1} := EL_{s1} - EL_{bot} = 14.67\ \text{ft}$	Height of Upstream Grade
$H_{HW} := EL_{HW} - EL_{bot} = 7.57\ \text{ft}$	Height of Upstream Pool
$t_{headwall} := 2.0\ \text{ft}$	Thickness of Headwall
$b := 1\ \text{ft}$	Unit Width of Headwall
$W_B := 16\ \text{ft}$	Width of Baffle
$t_{sidewall} := 2\ \text{ft}$	Thickness of Sidewalls
$W := W_B + t_{sidewall} \cdot 2 = 20\ \text{ft}$	Width of Impact Basin

Design Horizontal Reinforcement

Calculate Horizontal Loads:

Driving Side Soil Loads (At-Rest Pressure):

$F_{h1} := \frac{1}{2} \cdot K_o \cdot \left(\gamma_{sat\_fill} - \gamma_w\right) \cdot {H_{s1}}^2 \cdot b = 3.423\ \text{kip}$	Laterial Soil Load (Saturated)
$x_{h1} := \frac{H_{s1}}{3} = 4.89\ \text{ft}$	Moment Arm

Driving Side Reservoir Loads:

$F_{h2} := \frac{1}{2} \cdot \gamma_w \cdot {H_{HW}}^2 \cdot b = 1.788\ \text{kip}$	Upstream Hydrostatic Load
$x_{h2} := \frac{H_{HW}}{3} = 2.523\ \text{ft}$	Moment Arm

Calculate Shear/Moment:

$\phi_F := 1.2$	Fluid Load Factor Per ACI 350-20 Section 9.2.1
$\phi_H := 1.6$	Lateral Earth Load Factor Per ACI 350-20 Section 9.2.1
$M := F_{h1} \cdot x_{h1} + F_{h2} \cdot x_{h2} = 21.251\ \text{kip} \cdot \text{ft}$	Service Moment
$V := F_{h1} + F_{h2} = 5.211\ \text{kip}$	Service Shear Force
$M_u := \phi_H \cdot F_{h1} \cdot x_{h1} + \phi_F \cdot F_{h2} \cdot x_{h2} = 32.197\ \text{kip} \cdot \text{ft}$	Factored Moment
$V_u := \phi_H \cdot F_{h1} + \phi_F \cdot F_{h2} = 7.623\ \text{kip}$	Factored Shear Force

Impact Basin Concrete Reinforcement Design Per ACI 350-20

Rectangular Section in Flexure:

$d_b := 0.625\ \text{in}$	No. 5 Bar Diameter (Assumed Bar Size)
$h := t_{headwall} = 2\ \text{ft}$	Depth of Section
$c_c := 2\ \text{in}$	Cover to Reinforcement
$s := 6\ \text{in}$	Bar Spacing
$d := h - c_c - \frac{d_b}{2} = 21.688\ \text{in}$	Depth to Reinforcement

$$A_{s\_prov} := \frac{\pi \cdot d_b^{\ 2} \cdot b}{4 \cdot s} = 0.614\ \text{in}^2$$

Total Area of Reinforcement Provided

$$a := \frac{A_{s\_prov} \cdot f_y}{0.85 \cdot f'_c \cdot b} = 0.722\ \text{in}$$

Depth of Equivalent Stress Block

Calculate Net Tensile Strain:

$$\beta_1 := 0.85 - .00005 \cdot \frac{\text{in}^2}{\text{lbf}} \cdot \left(f'_c - 4000\ \text{psi}\right) = 0.8$$

Equivalent Depth Factor

$$\varepsilon_t := \frac{0.003 \cdot \left(\beta_1 \cdot d - a\right)}{a} = 0.069$$

Net Tensile Strain

$$\phi_{RF} := \begin{array}{l} \text{if } \varepsilon_t \geq 0.005 \\ \quad \| \ 0.9 \\ \text{else} \\ \quad \| \ 0.65 \end{array} = 0.9$$

Strength Reduction Factor

$\varepsilon_t \geq 0.005$ Tension-Controlled Section

Calculate Environmental Durability Factor:

$$\beta := \begin{array}{l} \text{if } h \geq 16\ \text{in} \\ \quad \| \ 1.2 \\ \text{else} \\ \quad \| \ 1.35 \end{array} = 1.2$$

Strain Gradient Factor

$$\gamma := \frac{M_u}{M} = 1.515$$

Combined Load Factor (Section 9.2.6)

$$f_{smax} := \frac{320\ \frac{\text{kip}}{\text{in}}}{\beta \cdot \sqrt{s^2 + 4 \cdot \left(2\ \text{in} + \frac{d_b}{2}\right)^2}} = 35.2\ \text{ksi}$$

Permissible Stress in Reinforcement for Normal Environmental Exposure (Section 10.6.4.5)

$$S_d := \max\left(\frac{\phi_{RF} \cdot f_y}{\gamma \cdot f_{smax}}, 1.0\right) = 1.013$$

Environmental Durability Factor (Section 9.2.6)

$$M_{u1} := S_d \cdot M_u = 32.601\ \text{kip} \cdot \text{ft}$$

Required Moment Strength

Check Flexural Strength:

$$M_n := A_{s\_prov} \cdot f_y \cdot \left(d - \frac{a}{2}\right) = 65.429\ \text{kip} \cdot \text{ft}$$

Nominal Flexural Strength

$$\phi_{RF} \cdot M_n = 58.886\ \text{kip} \cdot \text{ft}$$

Design Flexural Strength

$$\frac{M_{u1}}{\phi_{RF} \cdot M_n} = 0.554$$

Demand Capacity Ratio

$$\text{Check}_{Flex} := \begin{array}{l} \text{if } M_{u1} \le \phi_{RF} \cdot M_n \\ \quad \| \text{“PASS”} \\ \text{else} \\ \quad \| \text{“FAIL”} \end{array}$$

$$\text{Check}_{Flex} = \text{“PASS”}$$

Check Minimum Area of Reinforcement Required (Flexural, Per ACI 350-20 Section 10.5):

$$A_{s\_prov} = 0.614\ \text{in}^2$$

Area of Reinforcement Provided

$$A_{smin} := \max\left(\frac{3 \cdot \sqrt{\frac{f'_c}{\text{psi}}} \cdot \text{psi}}{f_y} \cdot b \cdot d, \frac{200\ \text{psi} \cdot b \cdot d}{f_y}\right) = 0.92\ \text{in}^2$$

Minimum Area of Reinforcement Required (Section 10.5.1)

$$\text{Check}_{Reinforcement} := \begin{array}{l} \text{if } A_{smin} \le A_{s\_prov} \\ \quad \| \text{“PASS”} \\ \text{else} \\ \quad \| \text{“FAIL”} \end{array}$$

$$\text{Check}_{Reinforcement} = \text{“FAIL”}$$

$$\text{Check}_{Reinforcement} := \begin{array}{l} \text{if } M_u \le M_n \cdot 1.33 \\ \quad \| \text{“PASS”} \\ \text{else} \\ \quad \| \text{“FAIL”} \end{array}$$

$$\text{Check}_{Reinforcement} = \text{“PASS”}$$

Section 10.5.3

Per ACI 350-20 Section 10.5.3, the section passes the minimum flexural reinforcement check (10.5.1 not applicable).

Rectangular Section in Shear (Per ACI 350-20 Section 11.1):

$\lambda := 1$ Modification Factor for Normalweight Concrete

$\phi_s := 0.75$ Shear Reduction Factor

$V_u = 7.623\ \text{kip}$ Factored Shear Force

$V_c := 2\ \text{psi} \cdot \lambda \cdot \sqrt{\frac{f'_c}{\text{psi}}} \cdot b \cdot d = 36.805\ \text{kip}$ Nominal Shear Strength Provided by Concrete

Demand Capacity Ratio

$\frac{V_u}{\phi_s \cdot V_c} = 0.276$

$$\text{Check}_{\text{Shear}} := \text{if } V_u \le \phi_s \cdot V_c \\ \quad \| \text{“PASS”} \\ \text{else} \\ \quad \| \text{“FAIL”}$$

$\text{Check}_{\text{Shear}} = \text{“PASS”}$

Design Vertical Reinforcement

Calculate Vertical Loads:

Driving Side Soil Loads (At-Rest Pressure):

$w_{h1} := K_o \cdot \left(\gamma_{sat\_fill} - \gamma_w\right) \cdot H_{s1} \cdot b = 0.467 \ \frac{1}{ft} \cdot kip$ Distributed Soil Load (Saturated)

Driving Side Reservoir Loads:

$w_{h2} := \gamma_w \cdot H_{HW} \cdot b = 0.472 \ \frac{1}{ft} \cdot kip$ Distributed Hydrostatic Load

Calculate Shear/Moment:

$\phi_F := 1.2$ Fluid Load Factor Per ACI 350-20 Section 9.2.1

$\phi_H := 1.6$ Lateral Earth Load Factor Per ACI 350-20 Section 9.2.1

$M := \frac{w_{h1} \cdot \left(W - t_{sidewall}\right)^2}{8} + \frac{w_{h2} \cdot \left(W - t_{sidewall}\right)^2}{8} = 38.032 \ kip \cdot ft$ Service Moment

$V := 0.5 \cdot \left(w_{h1} + w_{h2}\right) \cdot \left(W - t_{sidewall}\right) = 8.452 \ kip$ Service Shear Force

$M_u := \phi_H \cdot \frac{w_{h1} \cdot \left(W - t_{sidewall}\right)^2}{8} + \phi_F \cdot \frac{w_{h2} \cdot \left(W - t_{sidewall}\right)^2}{8} = 53.2 \ kip \cdot ft$ Factored Moment

$V_u := 0.5 \cdot \left(\phi_H \cdot w_{h1} \cdot \left(W - t_{sidewall}\right) + \phi_F \cdot w_{h2} \cdot \left(W - t_{sidewall}\right)\right) = 11.822 \ kip$ Factored Shear Force

Impact Basin Concrete Reinforcement Design Per ACI 350-20

Rectangular Section in Flexure:

$d_b := 0.875 \ in$ No. 7 Bar Diameter (Assumed Bar Size)

$h := t_{headwall} = 2 \ ft$ Depth of Section

$c_c := 2 \ in$ Cover to Reinforcement

$s := 6 \ in$ Bar Spacing

$d := h - c_c - \frac{d_b}{2} = 21.563 \ in$ Depth to Reinforcement

$$A_{s\_prov} := \frac{\pi \cdot {d_b}^2 \cdot b}{4 \cdot s} = 1.203 \text{ in}^2$$

Total Area of Reinforcement Provided

$$a := \frac{A_{s\_prov} \cdot f_y}{0.85 \cdot f'_c \cdot b} = 1.415 \text{ in}$$

Depth of Equivalent Stress Block

Calculate Net Tensile Strain:

$$\beta_1 := 0.85 - .00005 \cdot \frac{\text{in}^2}{\text{lbf}} \cdot \left(f'_c - 4000 \text{ psi}\right) = 0.8$$

Equivalent Depth Factor

$$\varepsilon_t := \frac{0.003 \cdot \left(\beta_1 \cdot d - a\right)}{a} = 0.034$$

Net Tensile Strain

$$\phi_{RF} := \left. \begin{array}{l} \text{if } \varepsilon_t \ge 0.005 \\ \quad \| \; 0.9 \\ \text{else} \\ \quad \| \; 0.65 \end{array} \right| = 0.9$$

Strength Reduction Factor

$\varepsilon_t \ge 0.005$ Tension-Controlled Section

Calculate Environmental Durability Factor:

$$\beta := \left. \begin{array}{l} \text{if } h \ge 16 \text{ in} \\ \quad \| \; 1.2 \\ \text{else} \\ \quad \| \; 1.35 \end{array} \right| = 1.2$$

Strain Gradient Factor

$$\gamma := \frac{M_u}{M} = 1.399$$

Combined Load Factor (Section 9.2.6)

$$f_{smax} := \frac{320 \, \dfrac{\text{kip}}{\text{in}}}{\beta \cdot \sqrt{s^2 + 4 \cdot \left(2 \text{ in} + \dfrac{d_b}{2}\right)^2}} = 34.494 \text{ ksi}$$

Permissible Stress in Reinforcement for Normal Environmental Exposure (Section 10.6.4.5)

$$S_d := \max\left(\frac{\phi_{RF} \cdot f_y}{\gamma \cdot f_{smax}}, 1.0\right) = 1.119$$

Environmental Durability Factor (Section 9.2.6)

$$M_{u1} := S_d \cdot M_u = 59.539 \text{ kip} \cdot \text{ft}$$

Required Moment Strength

Check Flexural Strength:

$M_n := A_{s\_prov} \cdot f_y \cdot \left(d - \frac{a}{2}\right) = 125.406\ kip \cdot ft$

Nominal Flexural Strength

$\phi_{RF} \cdot M_n = 112.865\ kip \cdot ft$

Design Flexural Strength

$\frac{M_{u1}}{\phi_{RF} \cdot M_n} = 0.528$

Demand Capacity Ratio

```
Check_Flex := if M_u1 ≤ ϕ_RF • M_n
                ‖ "PASS"
              else
                ‖ "FAIL"
```

$Check_{Flex} = \text{"PASS"}$

Check Minimum Area of Reinforcement Required:

$A_{s\_prov} = 1.203\ in^2$

Area of Reinforcement Provided

$A_{smin} := \max\left(\frac{3 \cdot \sqrt{\frac{f'_c}{psi}} \cdot psi}{f_y} \cdot b \cdot d, \frac{200\ psi \cdot b \cdot d}{f_y}\right) = 0.915\ in^2$

Minimum Area of Reinforcement Required (Section 10.5.1)

```
Check_Reinforcement := if A_smin ≤ A_s_prov
                         ‖ "PASS"
                       else
                         ‖ "FAIL"
```

$Check_{Reinforcement} = \text{"PASS"}$

Rectangular Section in Shear (Per ACI 350-20 Section 11.1):

$\lambda := 1$

Modification Factor for Normalweight Concrete

$\phi_s := 0.75$

Shear Reduction Factor

$V_u = 11.822\ kip$

Factored Shear Force

$V_c := 2\ psi \cdot \lambda \cdot \sqrt{\frac{f'_c}{psi}} \cdot b \cdot d = 36.593\ kip$

Nominal Shear Strength Provided by Concrete

$$\frac{V_u}{\phi_s \cdot V_c} = 0.431$$

Demand Capacity Ratio

```
Check_Shear := if V_u ≤ ϕ_s · V_c
                  ‖ "PASS"
               else
                  ‖ "FAIL"
```

$Check_{Shear} = \text{“PASS”}$

IMPACT BASIN SIDEWALLS

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Date: 5/20/2024
Project Number: 60727041

Project: Lake Erin Dam Rehabilitation

Task: Sidewall Design - Impact Basin

Description:

Design USBR Type VI Impact Basin sidewall structure in accordance with ACI 350-20, Code Requirements for Environmental Structures.

Codes, Standards, & References:

1. ACI 350-20, Code Requirements for Environmental Structures
2. AECOM Basis of Design Report

Assumptions:

- Geotechnical parameters provided by geotechnical engineers as part of field investigation.
- Conservatively design the sidewall on a unit width basis and the assumption that the sidewall behaves as a cantilever wall.
- Seepage analysis indicated phreatic surface maximum elevation of EL 943.0 ft. However, elevation conservatively assumed to be midpoint of headwall in calculations below (EL. 943.9 ft).

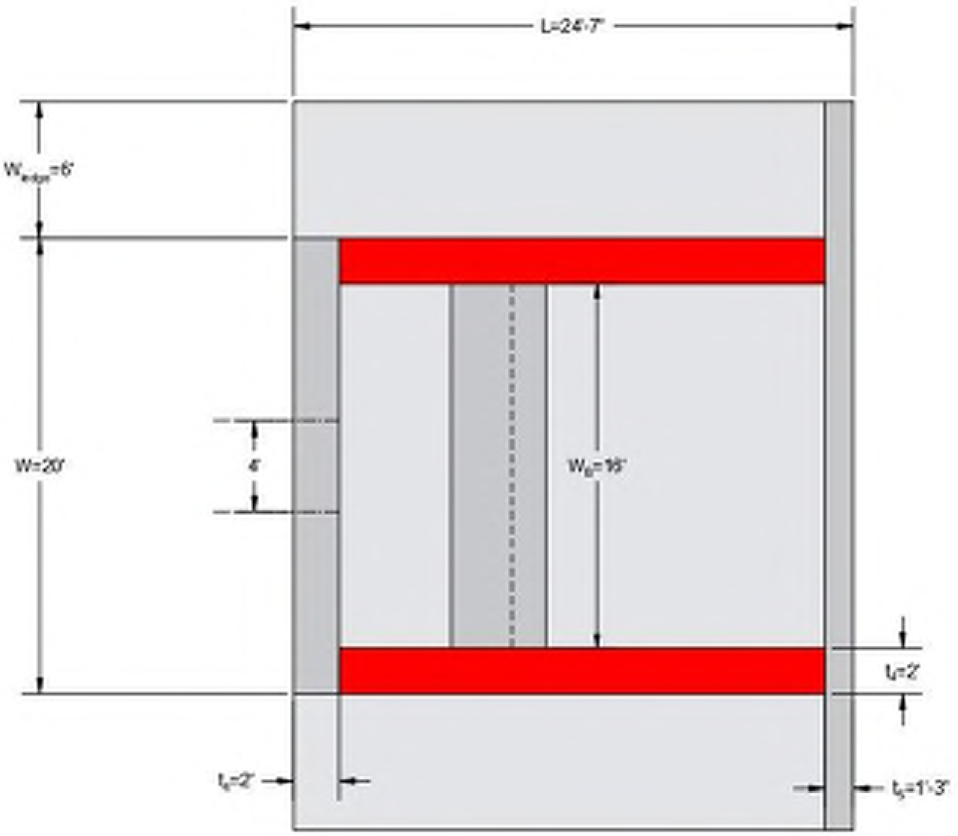

Figure 1 - Impact Basin Plan View and Sidewalls (In Red)

Material Properties

$\gamma_c := 150 \text{ pcf}$	Unit Weight of Concrete
$\gamma_w := 62.4 \text{ pcf}$	Unit Weight of Water
$f'_c := 5000 \text{ psi}$	Concrete Compressive Strength
$f_y := 60000 \text{ psi}$	Yield Strength of Reinforcement
$\gamma_{moist\_fill} := 123 \text{ pcf}$	Moist Unit Weight of Backfill
$\gamma_{sat\_fill} := 128 \text{ pcf}$	Saturated Unit Weight of Backfill
$\phi_b := 31 \text{ deg}$	Internal Friction Angle of Backfill
$K_o := 1 - \sin\left(\phi_b\right) = 0.485$	At-Rest Earth Pressure Coefficient

Sidewall Details & Geometry

$EL_{sidewall1} := 951.5 \text{ ft}$	Top of Sidewall Elevation (Upstream)
$EL_{sidewall2} := 943.83 \text{ ft}$	Top of Sidewall Elevation (Downstream)
$EL_{bot} := 936.33 \text{ ft}$	Base of Sidewall Elevation
$EL_{s1} := 951.0 \text{ ft}$	Elevation of Upstream Grade
$EL_{HW} := 943.9 \text{ ft}$	Elevation of Upstream Groundwater
$H_{s1} := EL_{s1} - EL_{bot} = 14.67 \text{ ft}$	Height of Upstream Grade
$H_{HW} := EL_{HW} - EL_{bot} = 7.57 \text{ ft}$	Height of Upstream Pool
$H_{sidewall1} := EL_{sidewall1} - EL_{bot} = 15.17 \text{ ft}$	Height of Upstream Sidewall
$H_{sidewall2} := EL_{sidewall2} - EL_{bot} = 7.5 \text{ ft}$	Height of Downstream Sidewall
$t_{sidewall} := 24 \text{ in}$	Thickness of Sidewall
$b := 1 \text{ ft}$	Unit Width of Sidewall

Determine Reactions on Sidewall

Calculate Horizontal Loads:

Driving Side Soil Loads (At-Rest Pressure):

$F_{h1} := \frac{1}{2} \cdot K_o \cdot \left(\gamma_{sat\_fill} - \gamma_w\right) \cdot {H_{s1}}^2 \cdot b = 3.423 \text{ kip}$	Laterial Soil Load (Saturated)
$x_{h1} := \frac{H_{s1}}{3} = 4.89 \text{ ft}$	Moment Arm

Driving Side Reservoir Loads:

$F_{h2} := \frac{1}{2} \cdot \gamma_w \cdot {H_{HW}}^2 \cdot b = 1.788 \text{ kip}$	Upstream Hydrostatic Load
$x_{h2} := \frac{H_{HW}}{3} = 2.523 \text{ ft}$	Moment Arm

Calculate Shear/Moment:

$\phi_F := 1.2$	Fluid Load Factor Per ACI 350-20 Section 9.2.1
$\phi_H := 1.6$	Lateral Earth Load Factor Per ACI 350-20 Section 9.2.1
$M := F_{h1} \cdot x_{h1} + F_{h2} \cdot x_{h2} = 21.251 \text{ kip} \cdot \text{ft}$	Service Moment
$V := F_{h1} + F_{h2} = 5.211 \text{ kip}$	Service Shear Force
$M_u := \phi_H \cdot F_{h1} \cdot x_{h1} + \phi_F \cdot F_{h2} \cdot x_{h2} = 32.197 \text{ kip} \cdot \text{ft}$	Factored Moment
$V_u := \phi_H \cdot F_{h1} + \phi_F \cdot F_{h2} = 7.623 \text{ kip}$	Factored Shear Force

Impact Basin Concrete Reinforcement Design Per ACI 350-20

Rectangular Section in Flexure:

$d_b := 0.625 \text{ in}$	No. 5 Bar Diameter (Assumed Bar Size)
$h := t_{sidewall} = 2 \text{ ft}$	Depth of Section
$c_c := 2 \text{ in}$	Cover to Reinforcement
$s := 6 \text{ in}$	Bar Spacing
$d := h - c_c - \frac{d_b}{2} = 21.688 \text{ in}$	Depth to Reinforcement

$$A_{s\_prov} := \frac{\pi \cdot {d_b}^2 \cdot b}{4 \cdot s} = 0.614\ in^2$$

Total Area of Reinforcement Provided

$$a := \frac{A_{s\_prov} \cdot f_y}{0.85 \cdot f'_c \cdot b} = 0.722\ in$$

Depth of Equivalent Stress Block

Calculate Net Tensile Strain:

$$\beta_1 := 0.85 - .00005 \cdot \frac{in^2}{lbf} \cdot \left(f'_c - 4000\ psi\right) = 0.8$$

Equivalent Depth Factor

$$\varepsilon_t := \frac{0.003 \cdot \left(\beta_1 \cdot d - a\right)}{a} = 0.069$$

Net Tensile Strain

$$\phi_{RF} := \left. \begin{array}{l} \text{if } \varepsilon_t \geq 0.005 \\ \quad \| \ 0.9 \\ \text{else} \\ \quad \| \ 0.65 \end{array} \right| = 0.9$$

Strength Reduction Factor

$\varepsilon_t \geq 0.005$ Tension-Controlled Section

Calculate Environmental Durability Factor:

$$\beta := \left. \begin{array}{l} \text{if } h \geq 16\ in \\ \quad \| \ 1.2 \\ \text{else} \\ \quad \| \ 1.35 \end{array} \right| = 1.2$$

Strain Gradient Factor

$$\gamma := \frac{M_u}{M} = 1.515$$

Combined Load Factor (Section 9.2.6)

$$f_{smax} := \frac{320\ \frac{kip}{in}}{\beta \cdot \sqrt{s^2 + 4 \cdot \left(2\ in + \frac{d_b}{2}\right)^2}} = 35.2\ ksi$$

Permissible Stress in Reinforcement for Normal Environmental Exposure (Section 10.6.4.5)

$$S_d := \max\left(\frac{\phi_{RF} \cdot f_y}{\gamma \cdot f_{smax}}, 1.0\right) = 1.013$$

Environmental Durability Factor (Section 9.2.6)

$$M_{u1} := S_d \cdot M_u = 32.601\ kip \cdot ft$$

Required Moment Strength

Check Flexural Strength:

$$M_n := A_{s\_prov} \cdot f_y \cdot \left(d - \frac{a}{2}\right) = 65.429\ \text{kip} \cdot \text{ft}$$

Nominal Flexural Strength

$$\phi_{RF} \cdot M_n = 58.886\ \text{kip} \cdot \text{ft}$$

Design Flexural Strength

$$\frac{M_{u1}}{\phi_{RF} \cdot M_n} = 0.554$$

Demand Capacity Ratio

```
Check_Flex := if  $M_{u1} \le \phi_{RF} \cdot M_n$
                  "PASS"
              else
                  "FAIL"
```

$Check_{Flex} = $ "PASS"

Check Minimum Area of Reinforcement Required (Flexural, Per ACI 350-20 Section 10.5):

$$A_{s\_prov} = 0.614\ \text{in}^2$$

Area of Reinforcement Provided

$$A_{smin} := \max\left(\frac{3 \cdot \sqrt{\frac{f'_c}{\text{psi}}} \cdot \text{psi}}{f_y} \cdot b \cdot d, \frac{200\ \text{psi} \cdot b \cdot d}{f_y}\right) = 0.92\ \text{in}^2$$

Minimum Area of Reinforcement Required (Section 10.5.1)

```
Check_Reinforcement := if  $A_{smin} \le A_{s\_prov}$
                           "PASS"
                       else
                           "FAIL"
```

$Check_{Reinforcement} = $ "FAIL"

```
Check_Reinforcement := if  $M_u \le M_n \cdot 1.33$
                           "PASS"
                       else
                           "FAIL"
```

$Check_{Reinforcement} = $ "PASS"

Section 10.5.3

Per ACI 350-20 Section 10.5.3, the section passes the minimum flexural reinforcement check (10.5.1 not applicable).

Rectangular Section in Shear (Per ACI 350-20 Section 11.1):

$\lambda := 1$ Modification Factor for Normalweight Concrete

$\phi_s := 0.75$ Shear Reduction Factor

$V_u = 7.623\ \text{kip}$ Factored Shear Force

$V_c := 2\ \text{psi} \cdot \lambda \cdot \sqrt{\frac{f'_c}{\text{psi}}} \cdot b \cdot d = 36.805\ \text{kip}$ Nominal Shear Strength Provided by Concrete

Demand Capacity Ratio

$\frac{V_u}{\phi_s \cdot V_c} = 0.276$

$$\text{Check}_{\text{Shear}} := \text{if } V_u \le \phi_s \cdot V_c \; \| \; \text{“PASS”} \; \text{else} \; \| \; \text{“FAIL”}$$

$\text{Check}_{\text{Shear}} = \text{“PASS”}$

IMPACT BASIN SLAB

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Date: 3/15/2024
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Date: 5/20/2024
Project Number: 60727041

Project: Lake Erin Dam Rehabilitation

Task: Base Slab Analysis - Impact Basin

Description:

Design USBR Type VI Impact Basin base slab in accordance with ACI 350-20, Code Requirements for Environmental Structures.

Codes, Standards, & References:

1. ACI 350-20, Code Requirements for Environmental Structures
2. AECOM Basis of Design Report

Assumptions:

- Conservatively design the base slab as a one-way slab spanning between the sidewalls.
- Seepage analysis indicated phreatic surface maximum elevation of EL 943.0 ft. However, elevation conservatively assumed to be midpoint of headwall in calculations below (EL. 943.9 ft). Design for uplift at the foundation/concrete interface based on this constant groundwater elevation.

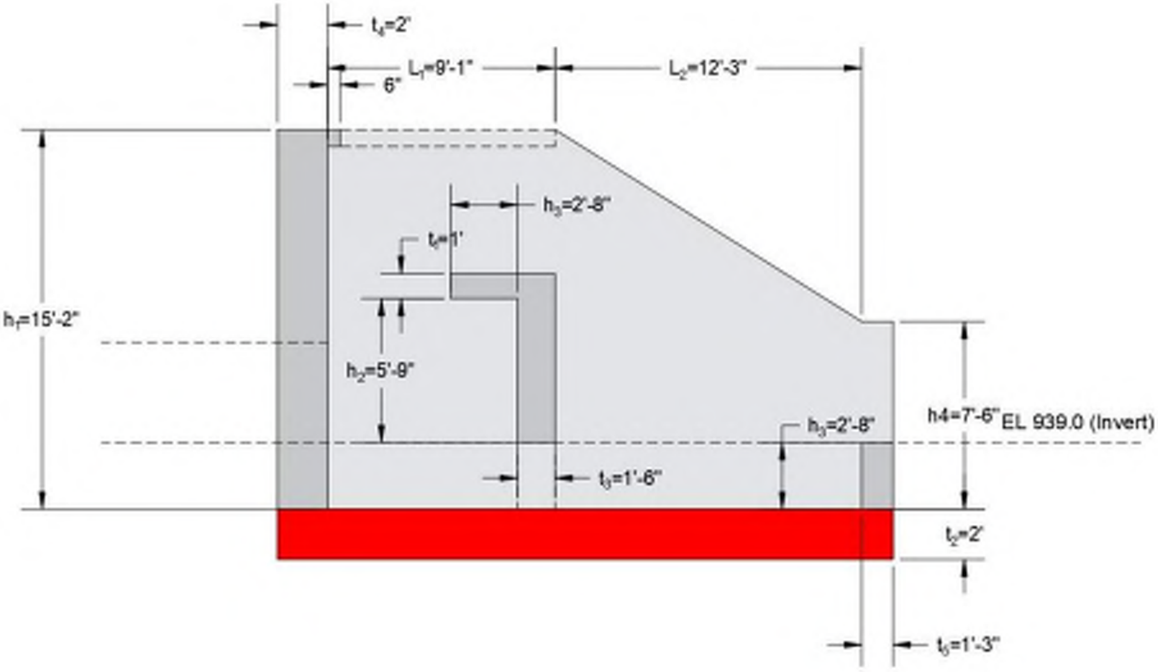

Figure 1 - Impact Basin Cross Section and Base Slab (In Red)

Material Properties

$\gamma_c := 150\ \text{pcf}$	Unit Weight of Concrete
$\gamma_w := 62.4\ \text{pcf}$	Unit Weight of Water
$f'_c := 5000\ \text{psi}$	Concrete Compressive Strength
$f_y := 60000\ \text{psi}$	Yield Strength of Reinforcement

Base Slab Details & Geometry

$EL_{s1} := 951.0\ \text{ft}$	Elevation of Upstream Grade
$EL_{HW} := 943.9\ \text{ft}$	Elevation of Upstream Groundwater
$EL_{bot} := 934.33\ \text{ft}$	Base of Foundation
$H_{HW} := EL_{HW} - EL_{bot} = 9.57\ \text{ft}$	Height of Upstream Pool (Uplift Head)
$h := 2\ \text{ft}$	Height of Base Slab
$b := 12\ \text{in}$	Unit Width of Base Slab
$L := 16\ \text{ft}$	Length of Base Slab

Determine Reactions on Base Slab

Load Combination: U = 0.9D + 1.2F (LRFD)

Calculate Dead Load:

$\phi_{DL} := 0.9$	Dead Load Factor
$w_{u1} := -\gamma_c \cdot b \cdot h = -0.3\ \text{klf}$	Dead Load of Base Slab
$\phi_{DL} \cdot w_{u1} = -0.27\ \text{klf}$	Factored Dead Load

Calculate Fluid Load:

$\phi_{FL} := 1.2$	Fluid Load Factor (ACI 350-20 Section D.9.2.6)
$w_{u2} := \gamma_w \cdot b \cdot H_{HW} = 0.597\ \text{klf}$	Fluid Load Under Base Slab (Uplift)

$\phi_{FL} \cdot w_{u2} = 0.717 \text{ klf}$ Factored Fluid Load

$w_u := \phi_{DL} \cdot w_{u1} + \phi_{FL} \cdot w_{u2} = 0.447 \text{ klf}$ Total Load Combination

Calculate Maximum Moment/Shear:

$M := \frac{w_{u1} \cdot L^2}{8} + \frac{w_{u2} \cdot L^2}{8} = 9.509 \text{ kip} \cdot \text{ft}$ Service Moment

$M_u := \frac{w_u \cdot L^2}{8} = 14.291 \text{ kip} \cdot \text{ft}$ Factored Moment

$V_u := \frac{w_u \cdot L}{2} = 3.573 \text{ kip}$ Factored Shear

Impact Basin Base Slab Concrete Reinforcement Design Per ACI 350-20

Rectangular Section in Flexure:

$h = 2 \text{ ft}$ Depth of Section

$d_b := 0.625 \text{ in}$ No. 5 Bar Diameter (Assumed Bar Size)

$A_b := \frac{\pi \cdot {d_b}^2}{4} = 0.307 \text{ in}^2$ Area of #5 Bar

$c_c := 3 \text{ in}$ Cover to Reinforcement

$s := 6 \text{ in}$ Bar Spacing

$d := h - c_c - \frac{d_b}{2} = 20.688 \text{ in}$ Depth to Reinforcement

$A_{s\_prov} := A_b \cdot \frac{b}{s} = 0.614 \text{ in}^2$ Total Area of Reinforcement Provided

$a := \frac{A_{s\_prov} \cdot f_y}{0.85 \cdot f'_c \cdot b} = 0.722 \text{ in}$ Depth of Equivalent Stress Block

Calculate Net Tensile Strain:

$$\beta_1 := 0.85 - .00005 \cdot \frac{in^2}{lbf} \cdot (f'_c - 4000\ psi) = 0.8$$

Equivalent Depth Factor

$$\varepsilon_t := \frac{0.003 \cdot (\beta_1 \cdot d - a)}{a} = 0.066$$

Net Tensile Strain

$$\phi_{RF} := \begin{array}{l|l} \text{if } \varepsilon_t \geq 0.005 & \\ \quad \| \ 0.9 & \\ \text{else} & \\ \quad \| \ 0.65 & \end{array} = 0.9$$

Strength Reduction Factor

$\varepsilon_t \geq 0.005$ Tension-Controlled Section

Calculate Environmental Durability Factor:

$$\beta := \begin{array}{l|l} \text{if } h \geq 16\ in & \\ \quad \| \ 1.2 & \\ \text{else} & \\ \quad \| \ 1.35 & \end{array} = 1.2$$

Strain Gradient Factor

$$\gamma := \frac{M_u}{M} = 1.503$$

Combined Load Factor (Section 9.2.6)

$$f_{smax} := \frac{320\ \frac{kip}{in}}{\beta \cdot \sqrt{s^2 + 4 \cdot \left(2\ in + \frac{d_b}{2}\right)^2}} = 35.2\ ksi$$

Permissible Stress in Reinforcement for Normal Environmental Exposure (Section 10.6.4.5)

$$S_d := \max\left(\frac{\phi_{RF} \cdot f_y}{\gamma \cdot f_{smax}}, 1.0\right) = 1.021$$

Environmental Durability Factor (Section 9.2.6)

$$M_{u1} := S_d \cdot M_u = 14.588\ kip \cdot ft$$

Required Moment Strength

Check Flexural Strength:

$$M_n := A_{s\_prov} \cdot f_y \cdot \left(d - \frac{a}{2}\right) = 62.361\ kip \cdot ft$$

Nominal Flexural Strength

$$\phi_{RF} \cdot M_n = 56.125\ kip \cdot ft$$

Design Flexural Strength

$$\frac{M_{u1}}{\phi_{RF} \cdot M_n} = 0.26$$

Demand Capacity Ratio

```
$Check_{Flex} := $ if $M_{u1} \le \phi_{RF} \cdot M_n$
    "PASS"
else
    "FAIL"
```

$Check_{Flex} = \text{"PASS"}$

Check Minimum Area of Reinforcement Required (Flexural, Per ACI 350-20 Section 10.5):

$A_{s\_prov} = 0.614\ in^2$

Area of Reinforcement Provided

$$A_{smin} := \max\left(\frac{3 \cdot \sqrt{\frac{f'_c}{psi}} \cdot psi}{f_y} \cdot b \cdot d , \frac{200\ psi \cdot b \cdot d}{f_y}\right) = 0.878\ in^2$$

Minimum Area of Reinforcement Required (Section 10.5.1)

```
$Check_{Reinforcement} := $ if $A_{smin} \le A_{s\_prov}$
    "PASS"
else
    "FAIL"
```

$Check_{Reinforcement} = \text{"FAIL"}$

```
$Check_{Reinforcement} := $ if $M_u \le M_n \cdot 1.33$
    "PASS"
else
    "FAIL"
```

$Check_{Reinforcement} = \text{"PASS"}$

Section 10.5.3

Per ACI 350-20 Section 10.5.3, the section passes the minimum flexural reinforcement check (10.5.1 not applicable).

Rectangular Section in Shear (Per ACI 350-20 Section 11.1):

$\lambda := 1$ Modification Factor for Normal Weight Concrete

$\phi_s := 0.75$ Shear Reduction Factor

$V_u = 3.573\ \text{kip}$ Factored Shear Force

$V_c := 2\ \text{psi} \cdot \lambda \cdot \sqrt{\frac{f'_c}{\text{psi}}} \cdot b \cdot d = 35.108\ \text{kip}$ Nominal Shear Strength Provided by Concrete

Demand Capacity Ratio

$$\frac{V_u}{\phi_s \cdot V_c} = 0.136$$

$$\text{Check}_{\text{Shear}} := \begin{array}{|l} \text{if } V_u \le \phi_s \cdot V_c \\ \quad \Vert \text{“PASS”} \\ \text{else} \\ \quad \Vert \text{“FAIL”} \end{array}$$

$\text{Check}_{\text{Shear}} = \text{“PASS”}$

IMPACT BASIN WINGWALL

Project: Lake Erin Dam Rehabilitation

Task: Wingwall Design - Impact Basin

Description:

Design USBR Type VI Impact Basin Wingwall structure (vertical reinforcement) in accordance with ACI 350-20, Code Requirements for Environmental Structures, and ACI 318-19, Building Code Requirements for Structural Concrete.

Codes, Standards, & References:

1. ACI 350-20, Code Requirements for Environmental Structures
2. ACI 318-19, Building Code Requirements for Structural Concrete
3. AECOM Basis of Design Report

Assumptions:

- Geotechnical parameters provided by geotechnical engineer as part of field investigation.
- Conservatively design the Wingwall vertical reinforcement on a unit width basis and the assumption that the wall behaves as a cantilever wall.
- Assume full hydrostatic pressure in backfill (saturated).

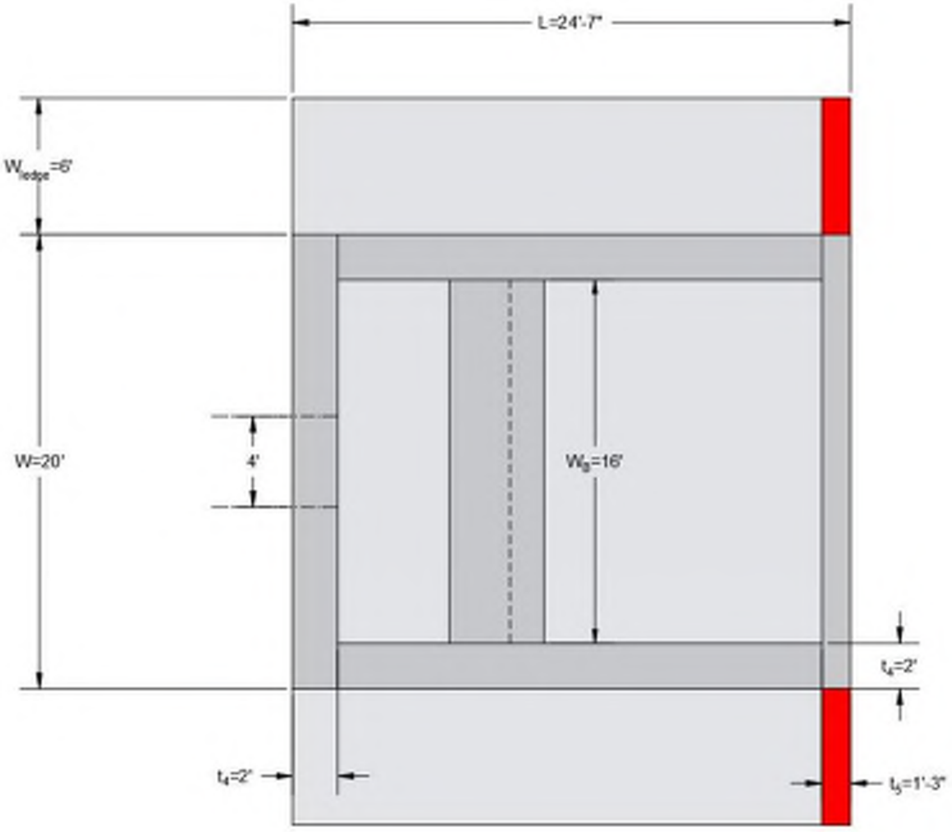

Figure 1 - Impact Basin Plan View and Wingwalls (In Red)

Material Properties

$\gamma_c := 150\ \text{pcf}$	Unit Weight of Concrete
$\gamma_w := 62.4\ \text{pcf}$	Unit Weight of Water
$f'_c := 5000\ \text{psi}$	Concrete Compressive Strength
$f_y := 60000\ \text{psi}$	Yield Strength of Reinforcement
$\gamma_{moist\_fill} := 123\ \text{pcf}$	Moist Unit Weight of Backfill
$\gamma_{sat\_fill} := 128\ \text{pcf}$	Saturated Unit Weight of Backfill
$\phi_b := 31\ \text{deg}$	Internal Friction Angle of Backfill
$K_o := 1 - \sin\left(\phi_b\right) = 0.485$	At-Rest Earth Pressure Coefficient

Sidewall Details & Geometry

$EL_{wingwall} := 943.83\ \text{ft}$	Top of Wingwall Elevation
$EL_{bot} := 936.83\ \text{ft}$	Base of Wingwall Elevation
$EL_{soil} := 942.5\ \text{ft}$	Retained Soil Elevation
$EL_{HW} := EL_{soil} = 942.5\ \text{ft}$	Elevation of Upstream Groundwater (Saturated)
$H_{s1} := EL_{soil} - EL_{bot} = 5.67\ \text{ft}$	Height of Upstream Grade
$H_{HW} := EL_{HW} - EL_{bot} = 5.67\ \text{ft}$	Height of Upstream Pool
$t_{wingwall} := 1.25\ \text{ft}$	Thickness of Wingwall
$b := 1\ \text{ft}$	Unit Width of Wingwall

Design Horizontal Reinforcement

Calculate Horizontal Loads:

Driving Side Soil Loads (At-Rest Pressure):

$F_{h1} := \frac{1}{2} \cdot K_o \cdot \left(\gamma_{sat\_fill} - \gamma_w\right) \cdot {H_{s1}}^2 \cdot b = 0.511\ \text{kip}$ Lateral Soil Load (Saturated)

$x_{h1} := \frac{H_{s1}}{3} = 1.89\ \text{ft}$ Moment Arm

Driving Side Reservoir Loads:

$F_{h2} := \frac{1}{2} \cdot \gamma_w \cdot {H_{HW}}^2 \cdot b = 1.003\ \text{kip}$ Upstream Hydrostatic Load

$x_{h2} := \frac{H_{HW}}{3} = 1.89\ \text{ft}$ Moment Arm

Calculate Shear/Moment:

$\phi_F := 1.2$ Fluid Load Factor Per ACI 350-20 Section 9.2.1

$\phi_H := 1.6$ Lateral Earth Load Factor Per ACI 350-20 Section 9.2.1

$M := F_{h1} \cdot x_{h1} + F_{h2} \cdot x_{h2} = 2.862\ \text{kip} \cdot \text{ft}$ Service Moment

$V := F_{h1} + F_{h2} = 1.514\ \text{kip}$ Service Shear Force

$M_u := \phi_H \cdot F_{h1} \cdot x_{h1} + \phi_F \cdot F_{h2} \cdot x_{h2} = 3.821\ \text{kip} \cdot \text{ft}$ Factored Moment

$V_u := \phi_H \cdot F_{h1} + \phi_F \cdot F_{h2} = 2.022\ \text{kip}$ Factored Shear Force

Impact Basin Concrete Reinforcement Design Per ACI 350-20

Rectangular Section in Flexure:

$d_b := 0.625\ \text{in}$ No. 5 Bar Diameter (Assumed Bar Size)

$h := t_{wingwall} = 1.25\ \text{ft}$ Depth of Section

$c_c := 2\ \text{in}$ Cover to Reinforcement

$s := 6\ \text{in}$ Bar Spacing

$d := h - c_c - \frac{d_b}{2} = 12.688\ \text{in}$ Depth to Reinforcement

$$A_{s\_prov} := \frac{\pi \cdot d_b^{\,2} \cdot b}{4 \cdot s} = 0.614\ \text{in}^2$$

Total Area of Reinforcement Provided

$$a := \frac{A_{s\_prov} \cdot f_y}{0.85 \cdot f'_c \cdot b} = 0.722\ \text{in}$$

Depth of Equivalent Stress Block

Calculate Net Tensile Strain:

$$\beta_1 := 0.85 - .00005 \cdot \frac{\text{in}^2}{\text{lbf}} \cdot \left(f'_c - 4000\ \text{psi}\right) = 0.8$$

Equivalent Depth Factor

$$\varepsilon_t := \frac{0.003 \cdot \left(\beta_1 \cdot d - a\right)}{a} = 0.039$$

Net Tensile Strain

$$\phi_{RF} := \left| \begin{array}{l} \text{if } \varepsilon_t \ge 0.005 \\ \quad \| \ 0.9 \\ \text{else} \\ \quad \| \ 0.65 \end{array} \right. = 0.9$$

Strength Reduction Factor

$\varepsilon_t \ge 0.005$ Tension-Controlled Section

Calculate Environmental Durability Factor:

$$\beta := \left| \begin{array}{l} \text{if } h \ge 16\ \text{in} \\ \quad \| \ 1.2 \\ \text{else} \\ \quad \| \ 1.35 \end{array} \right. = 1.35$$

Strain Gradient Factor

$$\gamma := \frac{M_u}{M} = 1.335$$

Combined Load Factor (Section 9.2.6)

$$f_{smax} := \frac{320\ \frac{\text{kip}}{\text{in}}}{\beta \cdot \sqrt{s^2 + 4 \cdot \left(2\ \text{in} + \frac{d_b}{2}\right)^2}} = 31.289\ \text{ksi}$$

Permissible Stress in Reinforcement for Normal Environmental Exposure (Section 10.6.4.5)

$$S_d := \max\left(\frac{\phi_{RF} \cdot f_y}{\gamma \cdot f_{smax}}, 1.0\right) = 1.293$$

Environmental Durability Factor (Section 9.2.6)

$$M_{u1} := S_d \cdot M_u = 4.94\ \text{kip} \cdot \text{ft}$$

Required Moment Strength

Check Flexural Strength:

$$M_n := A_{s\_prov} \cdot f_y \cdot \left(d - \frac{a}{2}\right) = 37.817\ \text{kip} \cdot \text{ft}$$

Nominal Flexural Strength

$$\phi_{RF} \cdot M_n = 34.036\ \text{kip} \cdot \text{ft}$$

Design Flexural Strength

$$\frac{M_{u1}}{\phi_{RF} \cdot M_n} = 0.145$$

Demand Capacity Ratio

$$\text{Check}_{Flex} := \begin{array}{l} \text{if } M_{u1} \le \phi_{RF} \cdot M_n \\ \quad \Vert \text{“PASS”} \\ \text{else} \\ \quad \Vert \text{“FAIL”} \end{array}$$

$$\text{Check}_{Flex} = \text{“PASS”}$$

Check Minimum Area of Reinforcement Required:

$$A_{s\_prov} = 0.614\ \text{in}^2$$

Area of Reinforcement Provided

$$A_{smin} := \max\left(\frac{3 \cdot \sqrt{\frac{f'_c}{psi}} \cdot psi}{f_y} \cdot b \cdot d , \frac{200\ psi \cdot b \cdot d}{f_y}\right) = 0.538\ \text{in}^2$$

Minimum Area of Reinforcement Required (Section 10.5.1)

$$\text{Check}_{Reinforcement} := \begin{array}{l} \text{if } A_{smin} \le A_{s\_prov} \\ \quad \Vert \text{“PASS”} \\ \text{else} \\ \quad \Vert \text{“FAIL”} \end{array}$$

$$\text{Check}_{Reinforcement} = \text{“PASS”}$$

Rectangular Section in Shear (Per ACI 350-20 Section 11.1):

$$\lambda := 1$$

Modification Factor for Normal Weight Concrete

$$\phi_s := 0.75$$

Shear Reduction Factor

$$V_u = 2.022\ \text{kip}$$

Factored Shear Force

$$V_c := 2\ psi \cdot \lambda \cdot \sqrt{\frac{f'_c}{psi}} \cdot b \cdot d = 21.531\ \text{kip}$$

Nominal Shear Strength Provided by Concrete

$$\frac{V_u}{\phi_s \cdot V_c} = 0.125$$

Demand Capacity Ratio

```
$Check_{Shear} := $ if $V_u \le \phi_s \cdot V_c$
                      “PASS”
                  else
                      “FAIL”
```

$Check_{Shear} = $ “PASS”

CONDUIT ENCASEMENT (48”)

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Calculated By: NRP
Date: 4/4/2024
Checked By: IML
Date: 4/30/2024
Project Number: 60727041

Project: Lake Erin Dam Rehabilitation

Task: Concrete Encasement Design

Description:

Design the concrete encasement thickness and reinforcement size required to resist the embankment loading in accordance with ACI 350-20, Code Requirements for Environmental Structures. Check the encasement for bearing on the soil foundation in accordance with USACE EM 1110-2-2100.

Codes, Standards, & References:

1. ACI 350-20, Code Requirements for Environmental Structures
2. EM No. 14 Beggs Deformeter Stress Analysis of Single-Barrel Conduits, USBR 1986.
3. AECOM Basis of Design Report
4. ASTM C361-22, Reinforced Concrete Low-Head Pressure Pipe
5. USACE EM 1110-2-2100, Stability Analysis of Concrete Structures (2005)

Assumptions:

- The encasement is designed to resist the entire vertical load of the embankment above the conduit.
- Coefficients for moment and shear will be obtained from Beggs Deformeter Stress Analysis Shape D.
- Geotechnical parameters were based on AECOM's subsurface investigation.
- C361 reinforced concrete low-head pressure pipe (RCP) was assumed for the conduit.

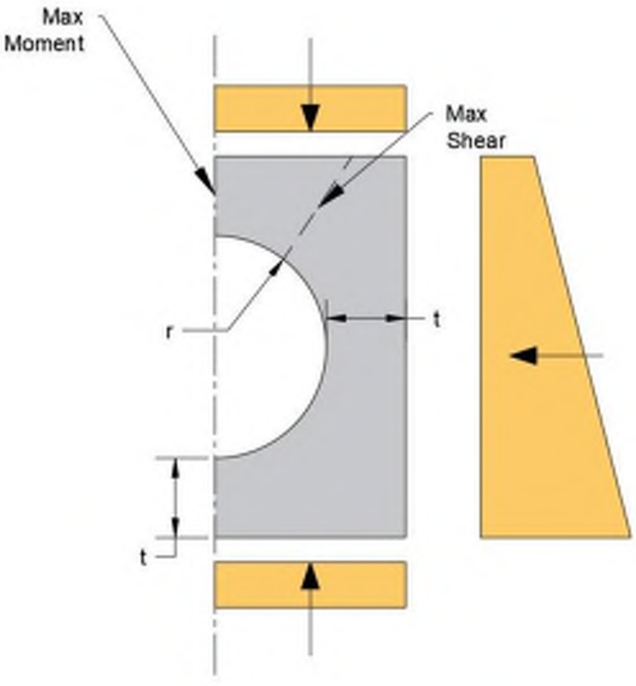


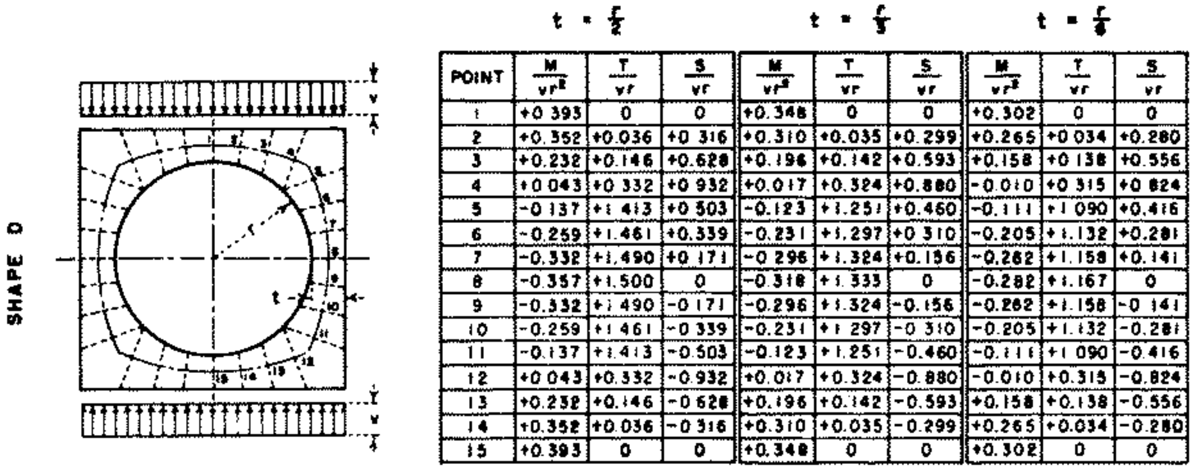
Figure 1 - Shape D and Loading Diagram

Material Properties and Design Parameters

$\gamma_c := 150\ \text{pcf}$	Unit Weight of Concrete
$\gamma_w := 62.4\ \text{pcf}$	Unit Weight of Water
$f'_c := 5000\ \text{psi}$	Concrete Compressive Strength
$f_y := 60000\ \text{psi}$	Yield Strength of Reinforcement
$\gamma_{sat} := 128\ \text{pcf}$	Saturated Unit Weight of Embankment Fill
$\phi := 31\ \text{deg}$	Soil Friction Angle
$K_o := 1 - \sin(\phi) = 0.485$	Embankment At-Rest Pressure
$b := 1\ \text{ft}$	Unit Width
$r := 24\ \text{in}$	Internal Radius of Conduit
$t_p := 5\ \text{in}$	Thickness of RCP Conduit (C361-22, Table 1)
$c_c := 3\ \text{in}$	Clear Cover for Concrete
$\phi_F := 1.2$	Fluid Load Factor Per ACI 350-20 Section 9.2.1
$\phi_H := 1.6$	Lateral Earth Load Factor Per ACI 350-20 Section 9.2.1
$\phi_{LL} := 1.6$	Live Load Load Factor Per ACI 350-20 Section 9.2.1
$\phi_{rf} := 0.9$	Moment Strength Reduction Factor
$\phi_s := 0.75$	Shear Strength Reduction Factor
$h_{emb} := 27\ \text{ft}$	Maximum Height of Embankment Over Conduit
$t := 12\ \text{in}$	Encasement Thickness
$\sigma_{all} := 3500\ \text{psf}$	Allowable Bearing Capacity
$\gamma_{s\_moist} := 123\ \text{pcf}$	Moist Unit Weight of Backfill

Check for Shear Strength

Maximum Shear occurs at point 4



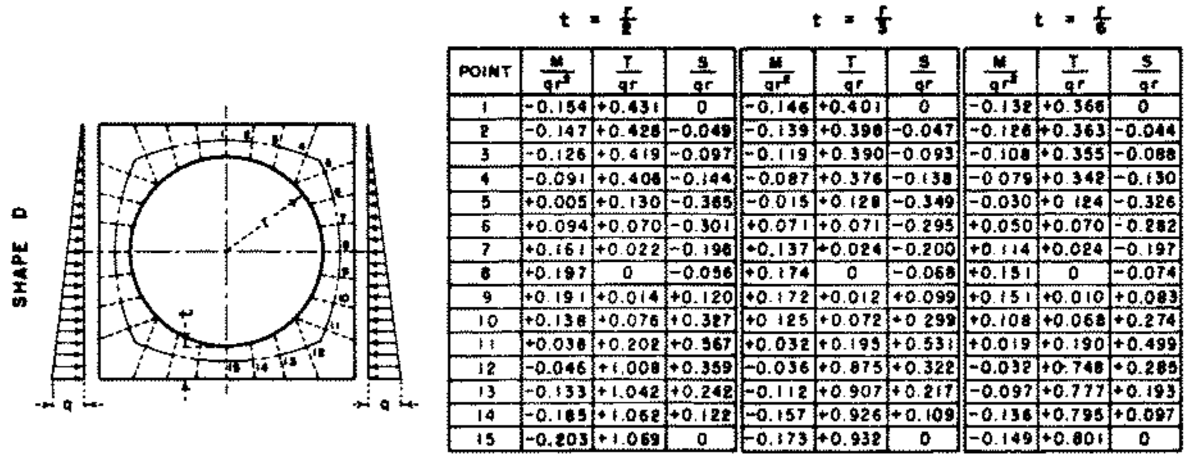
	$t = \frac{r}{2}$			$t = \frac{r}{5}$			$t = \frac{r}{6}$		
POINT	$\frac{M}{vr^2}$	$\frac{T}{vr}$	$\frac{S}{vr}$	$\frac{M}{vr^2}$	$\frac{T}{vr}$	$\frac{S}{vr}$	$\frac{M}{vr^2}$	$\frac{T}{vr}$	$\frac{S}{vr}$
1	+0.393	0	0	+0.348	0	0	+0.302	0	0
2	+0.352	+0.036	+0.316	+0.310	+0.035	+0.299	+0.265	+0.034	+0.280
3	+0.232	+0.146	+0.628	+0.196	+0.142	+0.593	+0.158	+0.138	+0.556
4	+0.043	+0.332	+0.932	+0.017	+0.324	+0.880	-0.010	+0.315	+0.824
5	-0.137	+1.413	+0.503	-0.123	+1.251	+0.460	-0.111	+1.090	+0.416
6	-0.259	+1.461	+0.339	-0.231	+1.297	+0.310	-0.205	+1.132	+0.281
7	-0.332	+1.490	+0.171	-0.296	+1.324	+0.156	-0.262	+1.158	+0.141
8	-0.357	+1.500	0	-0.318	+1.333	0	-0.282	+1.167	0
9	-0.332	+1.490	-0.171	-0.296	+1.324	-0.156	-0.262	+1.158	-0.141
10	-0.259	+1.461	-0.339	-0.231	+1.297	-0.310	-0.205	+1.132	-0.281
11	-0.137	+1.413	-0.503	-0.123	+1.251	-0.460	-0.111	+1.090	-0.416
12	+0.043	+0.332	-0.932	+0.017	+0.324	-0.880	-0.010	+0.315	-0.824
13	+0.232	+0.146	-0.628	+0.196	+0.142	-0.593	+0.158	+0.138	-0.556
14	+0.352	+0.036	-0.316	+0.310	+0.035	-0.299	+0.265	+0.034	-0.280
15	+0.393	0	0	+0.348	0	0	+0.302	0	0

Figure 2 - Beggs Coefficients for Loading (Figure 19)

$v := \gamma_{sat} \cdot h_{emb} \cdot b = 3456\ \frac{lbf}{ft}$ Overburden Pressure

$coeff_{fig19\_D\_S} := 0.914$ Coefficient from Beggs Figure 19 (See Interpolation in Figure 5)

$S_{419} := coeff_{fig19\_D\_S} \cdot v \cdot r = 6317.568\ lbf$ Shear from Loading at Point 4



	$t = \frac{r}{2}$			$t = \frac{r}{5}$			$t = \frac{r}{6}$		
POINT	$\frac{M}{qr^2}$	$\frac{T}{qr}$	$\frac{S}{qr}$	$\frac{M}{qr^2}$	$\frac{T}{qr}$	$\frac{S}{qr}$	$\frac{M}{qr^2}$	$\frac{T}{qr}$	$\frac{S}{qr}$
1	-0.154	+0.431	0	-0.146	+0.401	0	-0.132	+0.366	0
2	-0.147	+0.428	-0.049	-0.139	+0.398	-0.047	-0.126	+0.363	-0.044
3	-0.126	+0.419	-0.097	-0.119	+0.390	-0.093	-0.108	+0.355	-0.088
4	-0.091	+0.408	-0.144	-0.087	+0.376	-0.138	-0.079	+0.342	-0.130
5	+0.005	+0.130	-0.365	-0.015	+0.128	-0.349	-0.030	+0.124	-0.326
6	+0.094	+0.070	-0.301	+0.071	+0.071	-0.295	+0.050	+0.070	-0.282
7	+0.161	+0.022	-0.196	+0.137	+0.024	-0.200	+0.114	+0.024	-0.197
8	+0.197	0	-0.056	+0.174	0	-0.068	+0.151	0	-0.074
9	+0.191	+0.014	+0.120	+0.172	+0.012	+0.099	+0.151	+0.010	+0.083
10	+0.138	+0.076	+0.327	+0.125	+0.072	+0.299	+0.108	+0.068	+0.274
11	+0.038	+0.202	+0.567	+0.032	+0.195	+0.531	+0.019	+0.190	+0.499
12	-0.046	+1.008	+0.359	-0.036	+0.875	+0.322	-0.032	+0.748	+0.285
13	-0.133	+1.042	+0.242	-0.112	+0.907	+0.217	-0.097	+0.777	+0.193
14	-0.165	+1.062	+0.122	-0.157	+0.926	+0.109	-0.136	+0.795	+0.097
15	-0.203	+1.069	0	-0.173	+0.932	0	-0.149	+0.801	0

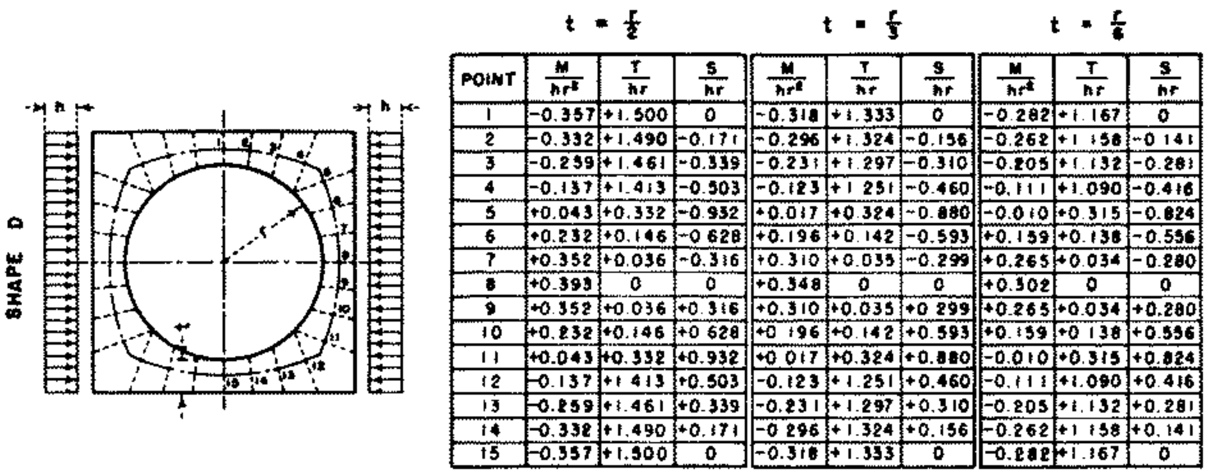
Figure 3 - Beggs Coefficients for Loading (Figure 29)

$h_{enc} := 2 \cdot r + 2 \cdot t + 2 \cdot t_p = 6.833\ ft$ Total Height of Encasement

$q := \left(K_o \cdot \left(\gamma_{sat} - \gamma_w\right) \cdot h_{enc} + \gamma_w \cdot h_{enc}\right) \cdot b = 643.792\ \frac{lbf}{ft}$ Maximum Horizontal Pressure

$coeff_{fig29\_D\_S} := -0.142$ Coefficient from Beggs Figure 29 (See Interpolation in Figure 5)

$S_{429} := coeff_{fig29\_D\_S} \cdot q \cdot r = -182.837\ lbf$ Shear from Loading at Point 4



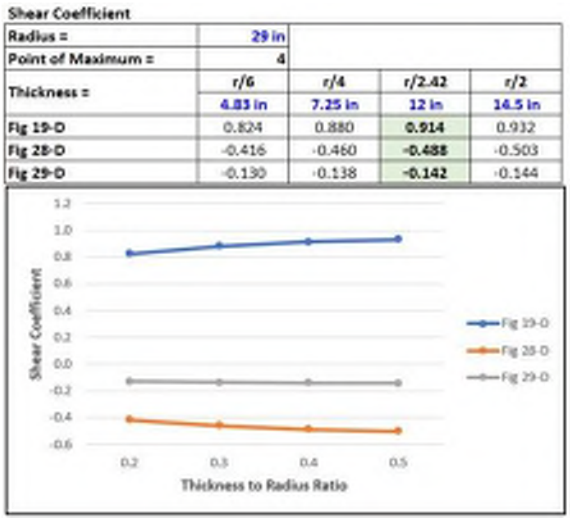
	$t = \frac{r}{2}$			$t = \frac{r}{3}$			$t = \frac{r}{6}$		
POINT	$\frac{M}{hr^2}$	$\frac{T}{hr}$	$\frac{S}{hr}$	$\frac{M}{hr^2}$	$\frac{T}{hr}$	$\frac{S}{hr}$	$\frac{M}{hr^2}$	$\frac{T}{hr}$	$\frac{S}{hr}$
1	−0.357	+1.500	0	−0.318	+1.333	0	−0.282	+1.167	0
2	−0.332	+1.490	−0.171	−0.296	+1.324	−0.156	−0.262	+1.158	−0.141
3	−0.259	+1.461	−0.339	−0.231	+1.297	−0.310	−0.205	+1.132	−0.281
4	−0.137	+1.413	−0.503	−0.123	+1.251	−0.460	−0.111	+1.090	−0.416
5	+0.043	+0.332	−0.932	+0.017	+0.324	−0.880	−0.010	+0.315	−0.824
6	+0.232	+0.146	−0.628	+0.196	+0.142	−0.593	+0.159	+0.138	−0.556
7	+0.352	+0.036	−0.316	+0.310	+0.035	−0.299	+0.265	+0.034	−0.280
8	+0.393	0	0	+0.348	0	0	+0.302	0	0
9	+0.352	+0.036	+0.316	+0.310	+0.035	+0.299	+0.265	+0.034	+0.280
10	+0.232	+0.146	+0.628	+0.196	+0.142	+0.593	+0.159	+0.138	+0.556
11	+0.043	+0.332	+0.932	+0.017	+0.324	+0.880	−0.010	+0.315	+0.824
12	−0.137	+1.413	+0.503	−0.123	+1.251	+0.460	−0.111	+1.090	+0.416
13	−0.259	+1.461	+0.339	−0.231	+1.297	+0.310	−0.205	+1.132	+0.281
14	−0.332	+1.490	+0.171	−0.296	+1.324	+0.156	−0.262	+1.158	+0.141
15	−0.357	+1.500	0	−0.318	+1.333	0	−0.282	+1.167	0

Figure 4 - Beggs Coefficients for Loading (Figure 28)

$h := \left(K_o \cdot \left(\gamma_{sat} - \gamma_w\right) \cdot h_{emb} + \gamma_w \cdot h_{emb}\right) \cdot b = 2543.765\ \frac{lbf}{ft}$ Horizontal Pressure at Top of Encasement

$coeff_{fig28\_D\_S} := -0.488$ Coefficient from Beggs Figure 28 (See Interpolation in Figure 5)

$S_{428} := coeff_{fig28\_D\_S} \cdot h \cdot r = -2482.714\ lbf$ Shear from Loading at Point 4



Shear Coefficient

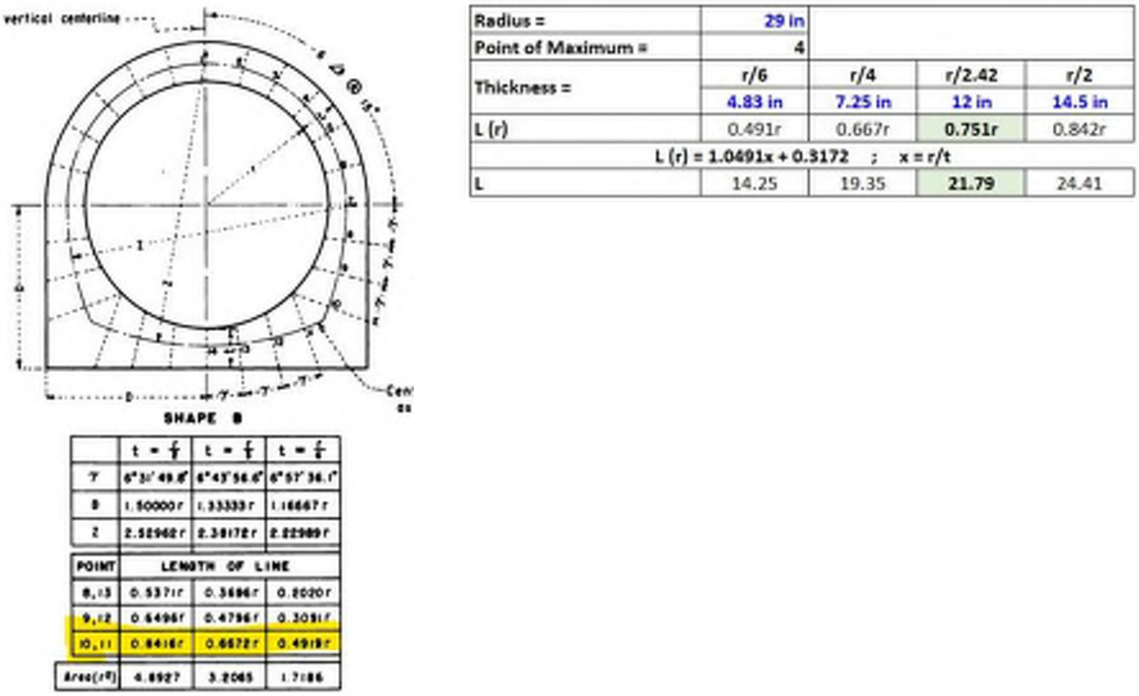
Radius =	29 in			
Point of Maximum =	4			
Thickness =	r/6	r/4	r/2.42	r/2
	4.83 in	7.25 in	12 in	14.5 in
Fig 19-D	0.824	0.880	**0.914**	0.932
Fig 28-D	-0.416	-0.460	**-0.488**	-0.503
Fig 29-D	-0.130	-0.138	**-0.142**	-0.144

Figure 5 - Beggs Shear Coefficients for Loading Interpolations

$S_{max4} := S_{419} + S_{429} + S_{428} = 3652.017\ lbf$ Total Shear from Loading at Point 4

$\sigma_{allow} := 1.1 \cdot \sqrt{f'_c \cdot \frac{lbf}{in^2}} = 77.782\ psi$ Allowable Shear Stress in Concrete (ACI 350-20, A.3.1b)

$\sigma_{shear} := \frac{S_{max4}}{b \cdot t} = 25.361\ psi$ Shear Stress in Concrete at Point 4



	$t = \frac{r}{2}$	$t = \frac{r}{4}$	$t = \frac{r}{6}$
γ	6°31′49.8″	6°43′56.6″	6°57′36.1″
B	1.50000r	1.33333r	1.16667r
Z	2.52962r	2.38172r	2.22989r
POINT	LENGTH OF LINE		
8, 13	0.5371r	0.3696r	0.2020r
9, 12	0.6496r	0.4796r	0.3091r
10, 11	0.8416r	0.6672r	0.4919r
Area(r^2)	4.8927	3.2065	1.7186

Radius =	29 in			
Point of Maximum =	4			
Thickness =	r/6 4.83 in	r/4 7.25 in	r/2.42 12 in	r/2 14.5 in
L (r)	0.491r	0.667r	0.751r	0.842r
L (r) = 1.0491x + 0.3172 ; x = r/t				
L	14.25	19.35	21.79	24.41

Figure 6 - Critical Length Calculation (Figure 1 from Beggs)

$length_{point4} := 21.8\ in$ — Length of Point 4 (Calculated Separately in MS Excel)

$d_{crit} := length_{point4} - 3.5\ in = 18.3\ in$ — Critical Length

$N_u := 0\ kip$ — Factored Axial Force Normal to Cross Section (Conservatively Set to Zero)

$A_g := t \cdot b = 144\ in^2$ — Area of Concrete

$$\phi V_{c\_T} := \phi_s \cdot 2 \cdot \left(1 + \frac{\frac{N_u}{lbf}}{2000 \cdot \frac{A_g}{in^2}}\right) \cdot \sqrt{f'_c \cdot \frac{lbf}{in^2}} \cdot d_{crit} \cdot b = 23.292\ kip$$

Shear Strength Provided by Concrete (ACI 350, Equation 11-4)

$V_u := \phi_{LL} \cdot S_{max4} = 5.843\ kip$ — Factored Shear Demand

$Shear_{check} := \text{if } V_u \le \phi V_{c\_T}$
  "PASS"
else
  "FAIL"

$Shear_{check} = \text{"PASS"}$

Check for Moment Strength

Maximum Moment occurs at point 1

$v = 3456\ \frac{lbf}{ft}$ Overburden Pressure

$coeff_{fig19\_D\_M} := 0.377$ Coefficient from Beggs Figure 19 (See Interpolation in Figure 7)

$M_{19} := coeff_{fig19\_D\_M} \cdot v \cdot r^2 = 5211.648\ ft \cdot lbf$ Moment from Loading at Point 1

$h_{enc} = 6.833\ ft$ Total Height of Encasement

$q = 643.792\ \frac{lbf}{ft}$ Maximum Horizontal Pressure

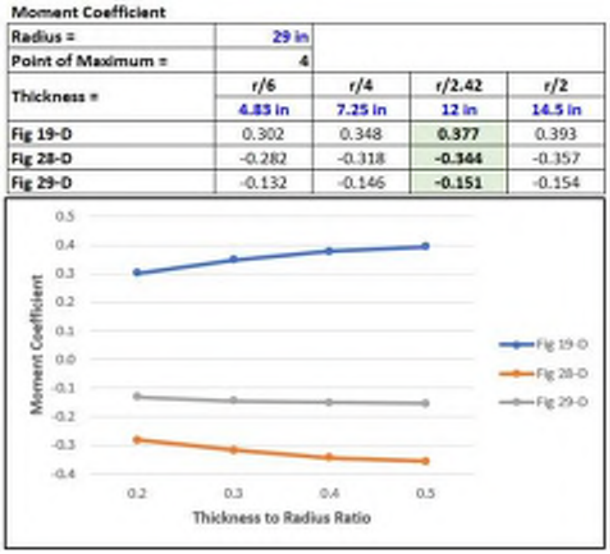
$coeff_{fig29\_D\_M} := -0.151$ Coefficient from Beggs Figure 29 (See Interpolation in Figure 7)

$M_{29} := coeff_{fig29\_D\_M} \cdot q \cdot r^2 = -388.851\ ft \cdot lbf$ Moment from Loading at Point 1

$h = 2543.765\ \frac{lbf}{ft}$ Horizontal Pressure at Top of Encasement

$coeff_{fig28\_D\_M} := -0.344$ Coefficient from Beggs Figure 28 (See Interpolation in Figure 7)

$M_{28} := coeff_{fig28\_D\_M} \cdot h \cdot r^2 = -3500.22\ ft \cdot lbf$ Moment from Loading at Point 1



Moment Coefficient

Radius =	29 in			
Point of Maximum =	4			
Thickness =	r/6	r/4	r/2.42	r/2
	4.83 in	7.25 in	12 in	14.5 in
Fig 19-D	0.302	0.348	0.377	0.393
Fig 28-D	-0.282	-0.318	-0.344	-0.357
Fig 29-D	-0.132	-0.146	-0.151	-0.154

Figure 7 - Beggs Moment Coefficients for Loading Interpolations

$M_{max} := M_{19} + M_{29} + M_{28} = 1322.577\ ft \cdot lbf$ Total Moment from Loading at Point 1

$d_b := 0.75\ \text{in}$ No. 6 Bar Diameter (Assumed Bar Size)

$s := 12\ \text{in}$ Bar Spacing

$A_{s\_prov} := \dfrac{\pi \cdot d_b^{\,2} \cdot b}{4 \cdot s} = 0.442\ \text{in}^2$ Total Area of Reinforcement Provided

$a := \dfrac{A_{s\_prov} \cdot f_y}{0.85 \cdot f'_c \cdot b} = 0.52\ \text{in}$ Depth of Equivalent Stress Block

$d := t - c_c - \dfrac{d_b}{2} = 8.625\ \text{in}$ Depth to Reinforcement

$$\beta := \left.\begin{array}{l} \text{if } t \geq 16\ \text{in} \\ \quad \| \ 1.2 \\ \text{else} \\ \quad \| \ 1.35 \end{array}\right| = 1.35$$

Strain Gradient Factor

$\gamma := \phi_{LL} = 1.6$ Combined Load Factor (Section 9.2.6)

$$f_{smax} := \max\left(\frac{320\ \dfrac{\text{kip}}{\text{in}}}{\beta \cdot \sqrt{s^2 + 4 \cdot \left(2\ \text{in} + \dfrac{d_b}{2}\right)^2}}, 20000\ \text{psi}\right) = 20\ \text{ksi}$$

Permissible Stress in Reinforcement for Normal Environmental Exposure (Section 10.6.4.5)

$S_d := \max\left(\dfrac{\phi_{rf} \cdot f_y}{\gamma \cdot f_{smax}}, 1.0\right) = 1.688$ Environmental Durability Factor (Section 9.2.6)

$M_u := S_d \cdot \phi_{LL} \cdot M_{19} + M_{28} + M_{29} = 10.182\ \text{kip} \cdot \text{ft}$ Required Moment Strength

$M_n := A_{s\_prov} \cdot f_y \cdot \left(d - \dfrac{a}{2}\right) = 18.478\ \text{kip} \cdot \text{ft}$ Nominal Flexural Strength

$\phi_{rf} \cdot M_n = 16.63\ \text{kip} \cdot \text{ft}$ Design Flexural Strength

$\dfrac{M_u}{\phi_{rf} \cdot M_n} = 0.612$ Demand Capacity Ratio

$$Check_{Flex} := \left.\begin{array}{l} \text{if } M_u \leq \phi_{rf} \cdot M_n \\ \quad \| \ \text{“PASS”} \\ \text{else} \\ \quad \| \ \text{“FAIL”} \end{array}\right|$$

$Check_{Flex} = \text{“PASS”}$

Check Minimum Area of Reinforcement Required

$$A_{s\_prov} = 0.442\ \text{in}^2$$

Area of Reinforcement Provided

$$A_{smin} := \max\left(\frac{3 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi}{f_y} \cdot b \cdot d, \frac{200\ psi \cdot b \cdot d}{f_y}\right) = 0.366\ \text{in}^2$$

Minimum Area of Reinforcement Required (Section 10.5.1)

$$\begin{array}{l} Check_{Reinforcement} := \text{if } A_{smin} \le A_{s\_prov} \\ \qquad \qquad \qquad \qquad \Big\| \text{"PASS"} \\ \qquad \qquad \qquad \quad \text{else} \\ \qquad \qquad \qquad \qquad \Big\| \text{"FAIL"} \end{array}$$

$$Check_{Reinforcement} = \text{"PASS"}$$

Check Bearing Stability

$$A_{inside} := \pi \cdot r^2 = 12.566\ \text{ft}^2$$

Area of Inside of RCP

$$A_{shell} := \pi \cdot \left(r + t_p\right)^2 - A_{inside} = 5.781\ \text{ft}^2$$

Area of RCP Concrete Shell

$$A_{encasement} := \left(2 \cdot r + 2 \cdot t + 2 \cdot t_p\right)^2 - A_{inside} - A_{shell} = 28.347\ \text{ft}^2$$

Area of Conduit Encasement

$$F_{v\_water} := A_{inside} \cdot \gamma_w \cdot b = 0.784\ \text{kip}$$

Weight of Water Inside Conduit

$$F_{v\_concrete} := \left(A_{shell} + A_{encasement}\right) \cdot \gamma_c \cdot b = 5.119\ \text{kip}$$

Weight of Concrete

$$F_{v\_soil} := h_{emb} \cdot \gamma_{s\_moist} \cdot b \cdot \left(r + t + t_p\right) = 11.347\ \text{kip}$$

Weight of Embankment Soil (Assume Moist Soil)

$$F_N := F_{v\_water} + F_{v\_concrete} + F_{v\_soil} = 17.25\ \text{kip}$$

Normal Force (Conservatively Ignore Uplift)

$$\sigma_{max} := \frac{F_N}{\left(2 \cdot r + 2 \cdot t + 2 \cdot t_p\right) \cdot b} = 2524.405\ \text{psf}$$

Maximum Bearing Pressure

$$\begin{array}{l} Check_{bearing} := \text{if } \sigma_{max} \le \sigma_{allow} \\ \qquad \qquad \qquad \quad \Big\| \text{"OK"} \\ \qquad \qquad \qquad \text{else} \\ \qquad \qquad \qquad \quad \Big\| \text{"NOT OK"} \end{array} = \text{"OK"}$$

Bearing Check

CONDUIT ENCASEMENT (15”)

Description:

Design the concrete encasement thickness and reinforcement size required to resist the water pressure loading in accordance with ACI 350-20, Code Requirements for Environmental Structures. Check the encasement for bearing on the soil foundation in accordance with USACE EM 1110-2-2100.

Codes, Standards, & References:

1. ACI 350-20, Code Requirements for Environmental Structures
2. EM No. 14 Beggs Deformeter Stress Analysis of Single-Barrel Conduits, USBR 1986.
3. AECOM Basis of Design Report
4. ASTM C361-22, Reinforced Low-Head Concrete Pressure Pipe
5. USACE EM 1110-2-2100, Stability Analysis of Concrete Structures (2005)

Assumptions:

- The encasement is designed to resist the entire vertical load of the flood water pressure above the conduit.
- Coefficients for moment and shear will be obtained from Beggs Deformeter Stress Analysis Shape D.
- Geotechnical parameters were based on AECOM's subsurface investigation.
- C361 reinforced concrete low-head pressure pipe (RCP) was assumed for the conduit.

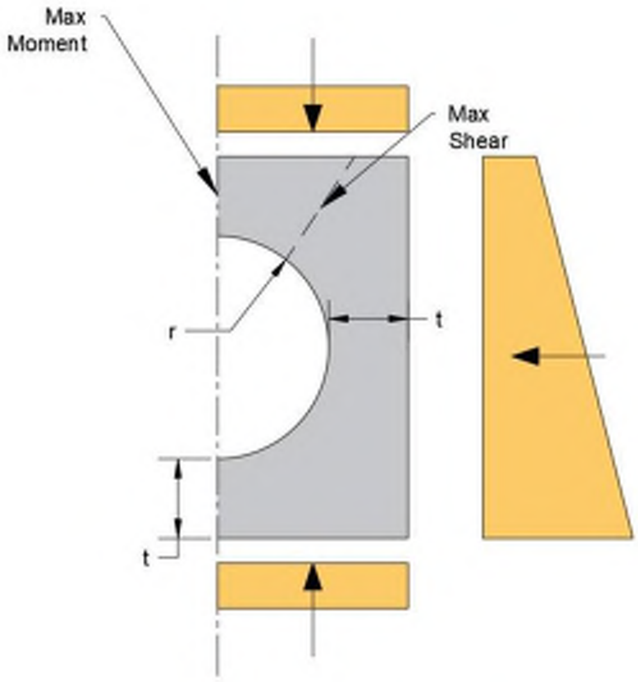


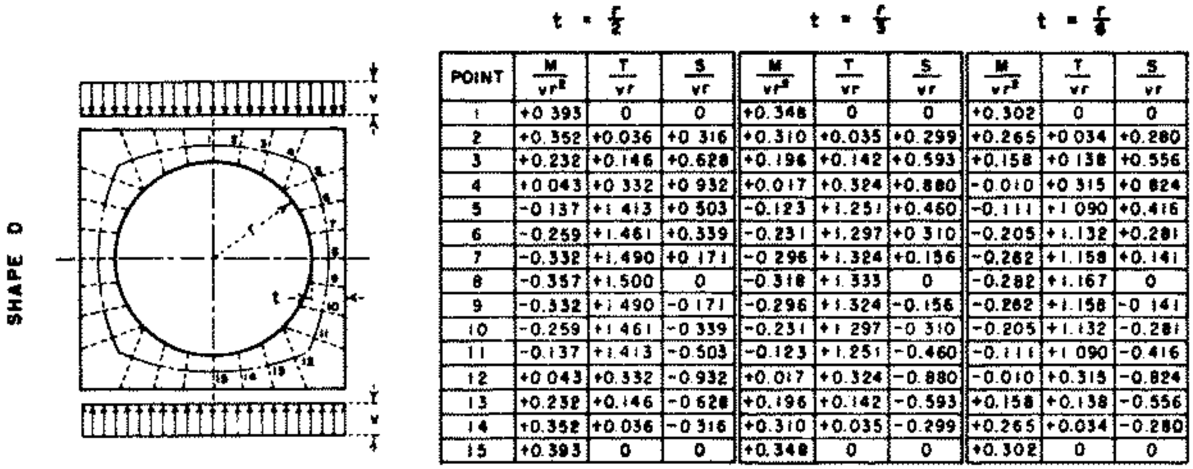
Figure 1 - Shape D and Loading Diagram

Material Properties and Design Parameters

$\gamma_c := 150\ \text{pcf}$	Unit Weight of Concrete
$\gamma_w := 62.4\ \text{pcf}$	Unit Weight of Water
$f'_c := 5000\ \text{psi}$	Concrete Compressive Strength
$f_y := 60000\ \text{psi}$	Yield Strength of Reinforcement
$b := 1\ \text{ft}$	Unit Width
$r := 7.5\ \text{in}$	Internal Radius of Conduit
$t_p := 2\ \text{in}$	Thickness of RCP Conduit (ASTM C361-22, Table 1)
$c_c := 3\ \text{in}$	Clear Cover for Concrete
$\phi_F := 1.2$	Fluid Load Factor Per ACI 350-20 Section 9.2.1
$\phi_H := 1.6$	Lateral Earth Load Factor Per ACI 350-20 Section 9.2.1
$\phi_{LL} := 1.6$	Live Load Factor Per ACI 350-20 Section 9.2.1
$\phi_{rf} := 0.9$	Moment Strength Reduction Factor
$\phi_s := 0.75$	Shear Strength Reduction Factor
$EL_{invert} := 940.25\ \text{ft}$	Invert Elevation of LLO Conduit
$EL_{flood} := 968.9\ \text{ft}$	Water Elevation for 1/3 PMF
$h_{water} := EL_{flood} - EL_{invert} + 2 \cdot r = 29.9\ \text{ft}$	Maximum Height of Water Over Conduit
$t := 8\ \text{in}$	Encasement Thickness
$\sigma_{all} := 3500\ \text{psf}$	Allowable Bearing Capacity

Check for Shear Strength

Maximum Shear occurs at point 4



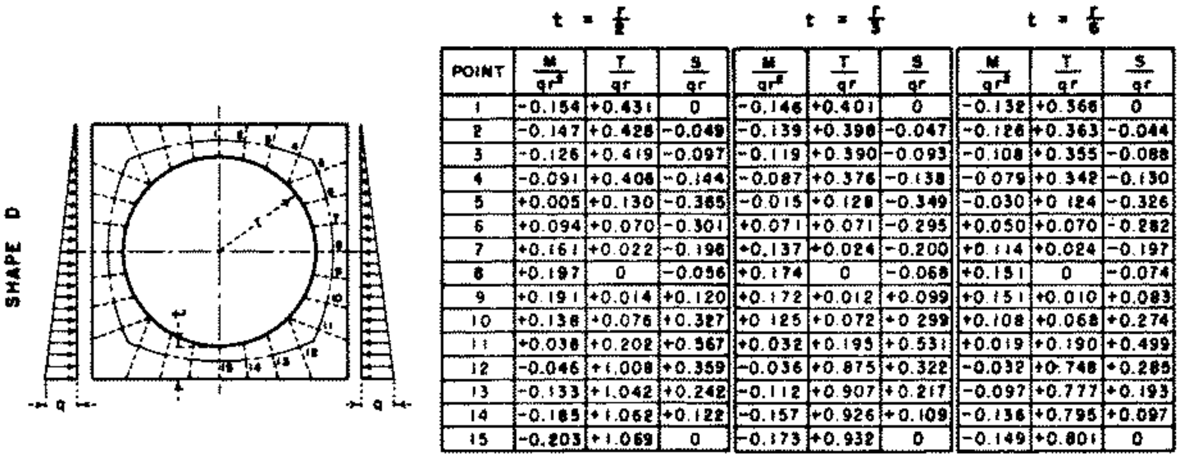
POINT	$t = \frac{r}{2}$			$t = \frac{r}{3}$			$t = \frac{r}{6}$		
	$\frac{M}{vr^2}$	$\frac{T}{vr}$	$\frac{S}{vr}$	$\frac{M}{vr^2}$	$\frac{T}{vr}$	$\frac{S}{vr}$	$\frac{M}{vr^2}$	$\frac{T}{vr}$	$\frac{S}{vr}$
1	+0.393	0	0	+0.348	0	0	+0.302	0	0
2	+0.352	+0.036	+0.316	+0.310	+0.035	+0.299	+0.265	+0.034	+0.280
3	+0.232	+0.146	+0.628	+0.196	+0.142	+0.593	+0.158	+0.138	+0.556
4	+0.043	+0.332	+0.932	+0.017	+0.324	+0.880	-0.010	+0.315	+0.824
5	-0.137	+1.413	+0.503	-0.123	+1.251	+0.460	-0.111	+1.090	+0.416
6	-0.259	+1.461	+0.339	-0.231	+1.297	+0.310	-0.205	+1.132	+0.281
7	-0.332	+1.490	+0.171	-0.296	+1.324	+0.156	-0.262	+1.158	+0.141
8	-0.357	+1.500	0	-0.318	+1.333	0	-0.282	+1.167	0
9	-0.332	+1.490	-0.171	-0.296	+1.324	-0.156	-0.262	+1.158	-0.141
10	-0.259	+1.461	-0.339	-0.231	+1.297	-0.310	-0.205	+1.132	-0.281
11	-0.137	+1.413	-0.503	-0.123	+1.251	-0.460	-0.111	+1.090	-0.416
12	+0.043	+0.332	-0.932	+0.017	+0.324	-0.880	-0.010	+0.315	-0.824
13	+0.232	+0.146	-0.628	+0.196	+0.142	-0.593	+0.158	+0.138	-0.556
14	+0.352	+0.036	-0.316	+0.310	+0.035	-0.299	+0.265	+0.034	-0.280
15	+0.393	0	0	+0.348	0	0	+0.302	0	0

Figure 2 - Beggs Coefficients for Loading (Figure 19)

$v \coloneqq \gamma_w \cdot h_{water} \cdot b = 1865.76\ \frac{lbf}{ft}$ Overburden Pressure

$coeff_{fig19\_D\_S} \coloneqq 1.003$ Coefficient from Beggs Figure 19 (See Interpolation in Figure 5)

$S_{419} \coloneqq coeff_{fig19\_D\_S} \cdot v \cdot r = 1169.598\ lbf$ Shear from Loading at Point 4



POINT	$t = \frac{r}{2}$			$t = \frac{r}{3}$			$t = \frac{r}{6}$		
	$\frac{M}{qr^2}$	$\frac{T}{qr}$	$\frac{S}{qr}$	$\frac{M}{qr^2}$	$\frac{T}{qr}$	$\frac{S}{qr}$	$\frac{M}{qr^2}$	$\frac{T}{qr}$	$\frac{S}{qr}$
1	-0.154	+0.431	0	-0.146	+0.401	0	-0.132	+0.366	0
2	-0.147	+0.428	-0.049	-0.139	+0.398	-0.047	-0.126	+0.363	-0.044
3	-0.126	+0.419	-0.097	-0.119	+0.390	-0.093	-0.108	+0.355	-0.088
4	-0.091	+0.406	-0.144	-0.087	+0.376	-0.138	-0.079	+0.342	-0.130
5	+0.005	+0.130	-0.365	-0.015	+0.128	-0.349	-0.030	+0.124	-0.326
6	+0.094	+0.070	-0.301	+0.071	+0.071	-0.295	+0.050	+0.070	-0.282
7	+0.161	+0.022	-0.196	+0.137	+0.024	-0.200	+0.114	+0.024	-0.197
8	+0.197	0	-0.056	+0.174	0	-0.068	+0.151	0	-0.074
9	+0.191	+0.014	+0.120	+0.172	+0.012	+0.099	+0.151	+0.010	+0.083
10	+0.138	+0.076	+0.327	+0.125	+0.072	+0.299	+0.108	+0.068	+0.274
11	+0.038	+0.202	+0.567	+0.032	+0.195	+0.531	+0.019	+0.190	+0.499
12	-0.046	+1.008	+0.359	-0.036	+0.875	+0.322	-0.032	+0.748	+0.285
13	-0.133	+1.042	+0.242	-0.112	+0.907	+0.217	-0.097	+0.777	+0.193
14	-0.165	+1.062	+0.122	-0.157	+0.926	+0.109	-0.136	+0.795	+0.097
15	-0.203	+1.069	0	-0.173	+0.932	0	-0.149	+0.801	0

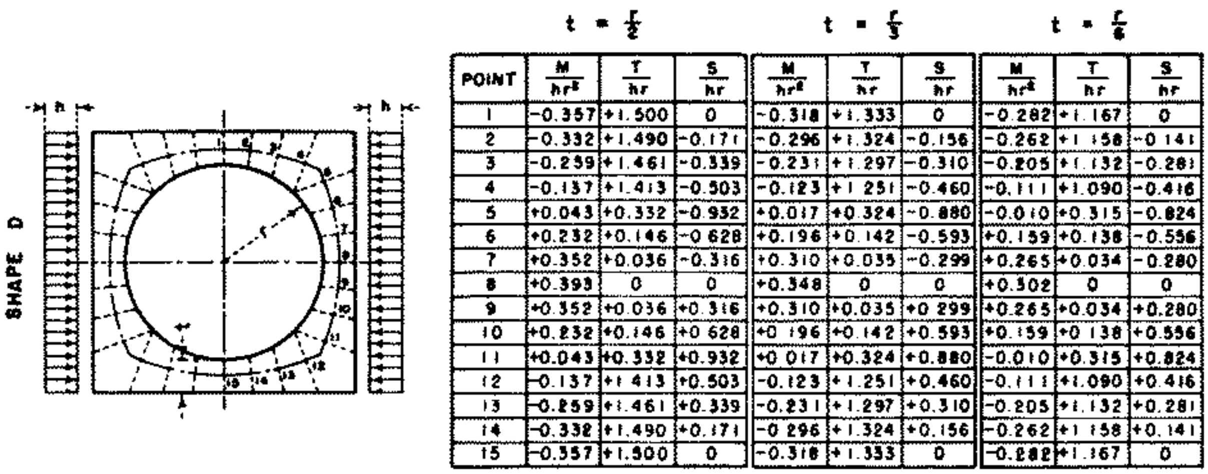
Figure 3 - Beggs Coefficients for Loading (Figure 29)

$h_{enc} \coloneqq 2 \cdot r + 2 \cdot t + 2 \cdot t_p = 2.917\ ft$ Total Height of Encasement

$q \coloneqq \gamma_w \cdot h_{enc} \cdot b = 182\ \frac{lbf}{ft}$ Maximum Horizontal Pressure

$coeff_{fig29\_D\_S} \coloneqq -0.152$ Coefficient from Beggs Figure 29 (See Interpolation in Figure 5)

$S_{429} \coloneqq coeff_{fig29\_D\_S} \cdot q \cdot r = -17.29\ lbf$ Shear from Loading at Point 4



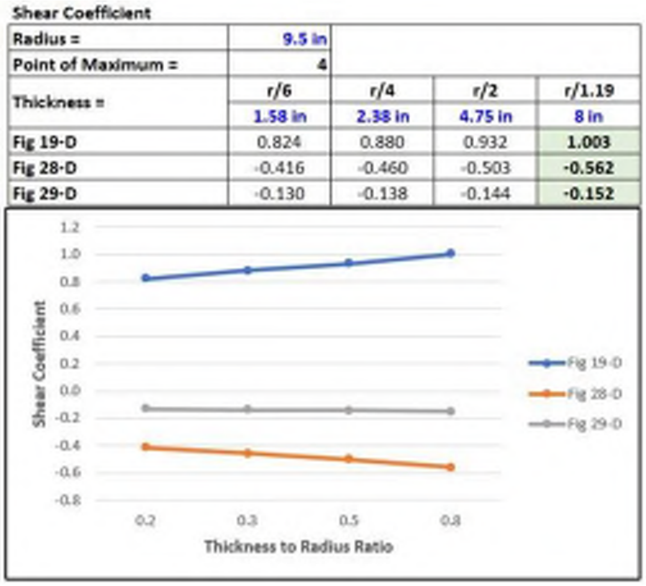
POINT	t = r/2 $\frac{M}{hr^2}$	t = r/2 $\frac{T}{hr}$	t = r/2 $\frac{S}{hr}$	t = r/3 $\frac{M}{hr^2}$	t = r/3 $\frac{T}{hr}$	t = r/3 $\frac{S}{hr}$	t = r/6 $\frac{M}{hr^2}$	t = r/6 $\frac{T}{hr}$	t = r/6 $\frac{S}{hr}$
1	−0.357	+1.500	0	−0.318	+1.333	0	−0.282	+1.167	0
2	−0.332	+1.490	−0.171	−0.296	+1.324	−0.156	−0.262	+1.158	−0.141
3	−0.259	+1.461	−0.339	−0.231	+1.297	−0.310	−0.205	+1.132	−0.281
4	−0.137	+1.413	−0.503	−0.123	+1.251	−0.460	−0.111	+1.090	−0.416
5	+0.043	+0.332	−0.932	+0.017	+0.324	−0.880	−0.010	+0.315	−0.824
6	+0.232	+0.146	−0.628	+0.196	+0.142	−0.593	+0.159	+0.138	−0.556
7	+0.352	+0.036	−0.316	+0.310	+0.035	−0.299	+0.265	+0.034	−0.280
8	+0.393	0	0	+0.348	0	0	+0.302	0	0
9	+0.352	+0.036	+0.316	+0.310	+0.035	+0.299	+0.265	+0.034	+0.280
10	+0.232	+0.146	+0.628	+0.196	+0.142	+0.593	+0.159	+0.138	+0.556
11	+0.043	+0.332	+0.932	+0.017	+0.324	+0.880	−0.010	+0.315	+0.824
12	−0.137	+1.413	+0.503	−0.123	+1.251	+0.460	−0.111	+1.090	+0.416
13	−0.259	+1.461	+0.339	−0.231	+1.297	+0.310	−0.205	+1.132	+0.281
14	−0.332	+1.490	+0.171	−0.296	+1.324	+0.156	−0.262	+1.158	+0.141
15	−0.357	+1.500	0	−0.318	+1.333	0	−0.282	+1.167	0

Figure 4 - Beggs Coefficients for Loading (Figure 28)

$h := \gamma_w \cdot h_{water} \cdot b = 1865.76\ \frac{lbf}{ft}$ Horizontal Pressure at Top of Encasement

$coeff_{fig28\_D\_S} := -0.562$ Coefficient from Beggs Figure 28 (See Interpolation in Figure 5)

$S_{428} := coeff_{fig28\_D\_S} \cdot h \cdot r = -655.348\ lbf$ Shear from Loading at Point 4



Shear Coefficient

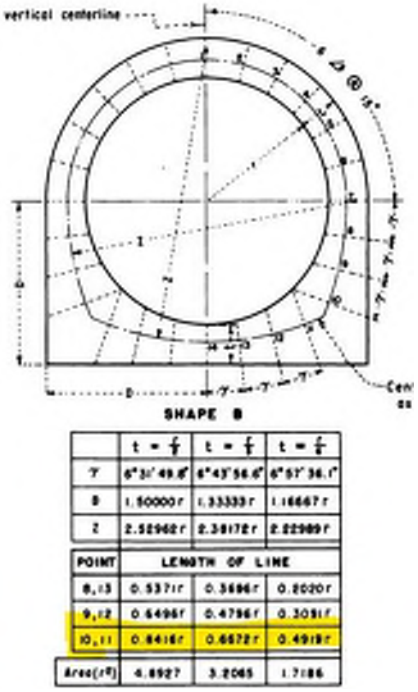
Radius =	9.5 in			
Point of Maximum =	4			
Thickness =	r/6 1.58 in	r/4 2.38 in	r/2 4.75 in	r/1.19 8 in
Fig 19-D	0.824	0.880	0.932	1.003
Fig 28-D	-0.416	-0.460	-0.503	-0.562
Fig 29-D	-0.130	-0.138	-0.144	-0.152

Figure 5 - Beggs Shear Coefficients for Loading Interpolations

$S_{max4} := S_{419} + S_{429} + S_{428} = 496.96\ lbf$ Total Shear from Loading at Point 4

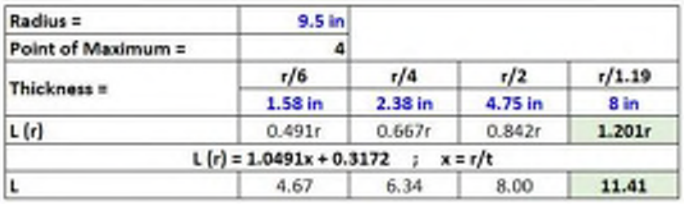
$\sigma_{allow} := 1.1 \cdot \sqrt{f'_c \cdot \frac{lbf}{in^2}} = 77.782\ psi$ Allowable Shear Stress in Concrete (ACI 350-20, A.3.1b)

$\sigma_{shear} := \frac{S_{max4}}{b \cdot t} = 5.177\ psi$ Shear Stress in Concrete at Point 4



	t = r/2	t = r/4	t = r/6
γ	6°31′49.8″	6°43′56.6″	6°57′36.1″
B	1.50000r	1.33333r	1.16667r
Z	2.52962r	2.39172r	2.22989r

POINT	LENGTH OF LINE		
8,13	0.5371r	0.3696r	0.2020r
9,12	0.6496r	0.4796r	0.3091r
10,11	0.8416r	0.6672r	0.4919r
Area(r²)	4.8927	3.2065	1.7186



Radius =	9.5 in			
Point of Maximum =	4			
Thickness =	r/6	r/4	r/2	r/1.19
	1.58 in	2.38 in	4.75 in	8 in
L (r)	0.491r	0.667r	0.842r	1.201r
L (r) = 1.0491x + 0.3172 ; x = r/t				
L	4.67	6.34	8.00	11.41

Figure 6 - Critical Length Calculation (Figure 1 from Beggs)

$length_{point4} := 11.4\ in$ — Length of Point 4 (Calculated Separately in MS Excel)

$d_{crit} := length_{point4} - 3.5\ in = 7.9\ in$ — Critical Length

$N_u := 0\ kip$ — Factored Axial Force Normal to Cross Section (Conservatively Set to Zero)

$A_g := t \cdot b = 96\ in^2$ — Area of Concrete

$$\phi V_{c\_T} := \phi_s \cdot 2 \cdot \left(1 + \frac{\frac{N_u}{lbf}}{2000 \cdot \frac{A_g}{in^2}}\right) \cdot \sqrt{f'_c \cdot \frac{lbf}{in^2}} \cdot d_{crit} \cdot b = 10.055\ kip$$

Shear Strength Provided by Concrete (ACI 350, Equation 11-4)

$V_u := \phi_{LL} \cdot S_{max4} = 0.795\ kip$ — Factored Shear Demand

$$Shear_{check} := \text{if } V_u \le \phi V_{c\_T} \ \| \text{“PASS”} \ \text{else} \ \| \text{“FAIL”}$$

$Shear_{check} = $ “PASS”

Check for Moment Strength

Maximum Moment occurs at point 1

$v = 1865.76\ \frac{lbf}{ft}$ Overburden Pressure

$coeff_{fig19\_D\_M} := 0.455$ Coefficient from Beggs Figure 19 (See Interpolation in Figure 7)

$M_{19} := coeff_{fig19\_D\_M} \cdot v \cdot r^2 = 331.61\ ft \cdot lbf$ Moment from Loading at Point 1

$h_{enc} = 2.917\ ft$ Total Height of Encasement

$q = 182\ \frac{lbf}{ft}$ Maximum Horizontal Pressure

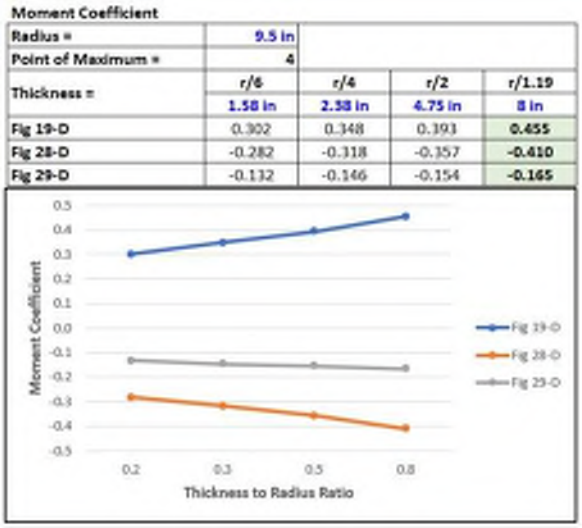
$coeff_{fig29\_D\_M} := -0.165$ Coefficient from Beggs Figure 29 (See Interpolation in Figure 7)

$M_{29} := coeff_{fig29\_D\_M} \cdot q \cdot r^2 = -11.73\ ft \cdot lbf$ Moment from Loading at Point 1

$h = 1865.76\ \frac{lbf}{ft}$ Horizontal Pressure at Top of Encasement

$coeff_{fig28\_D\_M} := -0.410$ Coefficient from Beggs Figure 28 (See Interpolation in Figure 7)

$M_{28} := coeff_{fig28\_D\_M} \cdot h \cdot r^2 = -298.813\ ft \cdot lbf$ Moment from Loading at Point 1



Moment Coefficient

Radius =	9.5 in			
Point of Maximum =	4			
Thickness =	r/6	r/4	r/2	r/1.19
	1.58 in	2.38 in	4.75 in	8 in
Fig 19-D	0.302	0.348	0.393	0.455
Fig 28-D	-0.282	-0.318	-0.357	-0.410
Fig 29-D	-0.132	-0.146	-0.154	-0.165

Figure 7 - Beggs Moment Coefficients for Loading Interpolations

$M_{max} := M_{19} + M_{29} + M_{28} = 21.066\ ft \cdot lbf$ Total Moment from Loading at Point 1

$d_b := 0.625\ \text{in}$	No. 5 Bar Diameter (Assumed Bar Size)
$s := 12\ \text{in}$	Bar Spacing
$A_{s\_prov} := \dfrac{\pi \cdot {d_b}^2 \cdot b}{4 \cdot s} = 0.307\ \text{in}^2$	Total Area of Reinforcement Provided
$a := \dfrac{A_{s\_prov} \cdot f_y}{0.85 \cdot f'_c \cdot b} = 0.361\ \text{in}$	Depth of Equivalent Stress Block
$d := t - c_c - \dfrac{d_b}{2} = 4.688\ \text{in}$	Depth to Reinforcement
$\beta := \begin{array}{l} \text{if } t \geq 16\ \text{in} \\ \quad \Vert\ 1.2 \\ \text{else} \\ \quad \Vert\ 1.35 \end{array} = 1.35$	Strain Gradient Factor
$\gamma := \phi_{LL} = 1.6$	Combined Load Factor (Section 9.2.6)
$f_{smax} := \max\left(\dfrac{320\ \dfrac{\text{kip}}{\text{in}}}{\beta \cdot \sqrt{s^2 + 4 \cdot \left(2\ \text{in} + \dfrac{d_b}{2}\right)^2}}, 20000\ \text{psi}\right) = 20\ \text{ksi}$	Permissible Stress in Reinforcement for Normal Environmental Exposure (Section 10.6.4.5)
$S_d := \max\left(\dfrac{\phi_{rf} \cdot f_y}{\gamma \cdot f_{smax}}, 1.0\right) = 1.688$	Environmental Durability Factor (Section 9.2.6)
$M_u := S_d \cdot \phi_{LL} \cdot M_{19} + M_{28} + M_{29} = 0.585\ \text{kip} \cdot \text{ft}$	Required Moment Strength
$M_n := A_{s\_prov} \cdot f_y \cdot \left(d - \dfrac{a}{2}\right) = 6.914\ \text{kip} \cdot \text{ft}$	Nominal Flexural Strength
$\phi_{rf} \cdot M_n = 6.222\ \text{kip} \cdot \text{ft}$	Design Flexural Strength
$\dfrac{M_u}{\phi_{rf} \cdot M_n} = 0.094$	Demand Capacity Ratio
$Check_{Flex} := \begin{array}{l} \text{if } M_u \leq \phi_{rf} \cdot M_n \\ \quad \Vert\ \text{“PASS”} \\ \text{else} \\ \quad \Vert\ \text{“FAIL”} \end{array}$	$Check_{Flex} = \text{“PASS”}$

Check Minimum Area of Reinforcement Required

$$A_{s\_prov} = 0.307\ \text{in}^2$$

Area of Reinforcement Provided

$$A_{smin} := \max\left(\frac{3 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi}{f_y} \cdot b \cdot d\,,\ \frac{200\ psi \cdot b \cdot d}{f_y}\right) = 0.199\ \text{in}^2$$

Minimum Area of Reinforcement Required (Section 10.5.1)

```
Check_Reinforcement := if  $A_{smin} \le A_{s\_prov}$
                           ‖ "PASS"
                       else
                           ‖ "FAIL"
```

$$Check_{Reinforcement} = \text{“PASS”}$$

Check Bearing Stability

$$A_{inside} := \pi \cdot r^2 = 1.227\ \text{ft}^2$$

Area of Inside of RCP

$$A_{shell} := \pi \cdot \left(r + t_p\right)^2 - A_{inside} = 0.742\ \text{ft}^2$$

Area of RCP Concrete Shell

$$A_{encasement} := \left(2 \cdot r + 2 \cdot t + 2 \cdot t_p\right)^2 - A_{inside} - A_{shell} = 6.538\ \text{ft}^2$$

Area of Conduit Encasement

$$F_{v\_water} := A_{inside} \cdot \gamma_w \cdot b = 0.077\ \text{kip}$$

Weight of Water Inside Conduit

$$F_{v\_concrete} := \left(A_{shell} + A_{encasement}\right) \cdot \gamma_c \cdot b = 1.092\ \text{kip}$$

Weight of Concrete

$$F_{v\_water\_pressure} := h_{water} \cdot \gamma_w \cdot b \cdot \left(r + t + t_p\right) = 2.721\ \text{kip}$$

Weight of Water Above Conduit

$$F_N := F_{v\_water} + F_{v\_concrete} + F_{v\_water\_pressure} = 3.889\ \text{kip}$$

Normal Force (Conservatively Ignore Uplift)

$$\sigma_{max} := \frac{F_N}{\left(2 \cdot r + 2 \cdot t + 2 \cdot t_p\right) \cdot b} = 1333.522\ \text{psf}$$

Maximum Bearing Pressure

```
Check_bearing := if  $\sigma_{max} \le \sigma_{allow}$      = "OK"
                     ‖ "OK"
                 else
                     ‖ "NOT OK"
```

Bearing Check

INTAKE TOWER DEBRIS SCREEN

Project: Lake Erin Dam Rehabilitation

Task: Debris Screen Sizing - Intake Tower

Description:

To size the debris screen structural steel members for the anticipated impact load.

Codes, Standards, & References:

1. AECOM Basis of Design Report
2. ASCE/SEI 7-22, Minimum Design Loads and Associated Criteria for Buildings and Other Structures (2022)
3. AISC 325-23, Steel Construction Manual, 16th Edition (2023)

Assumptions:

- The debris screens will be installed on the opening between the crest wall and the top slab of the intake tower.
- Debris screens will be sized structurally for impact loads. Impact loads are those that result from logs, ice floes, and other objects obstructing the debris screen. Debris object weight of 1,000 lb will be used for structural design based on ASCE 7-22, paragraph C5.4.5.
- Assume structural members are Round HSS 6x0.5.

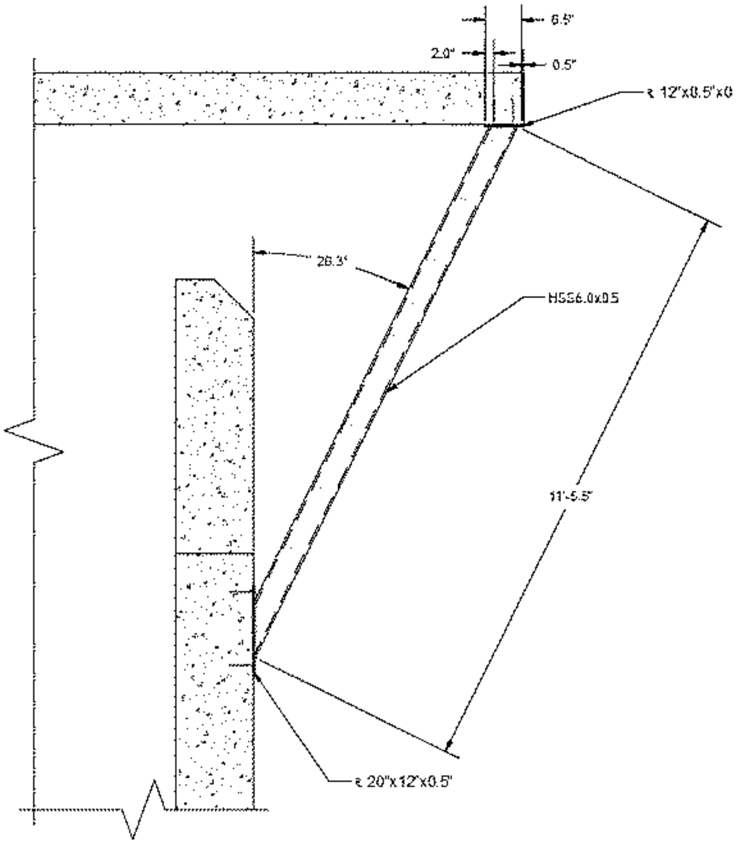


Figure 1 - Debris Screen Section and Geometry

Material Properties and Geometry

$f_y := 46000\ \text{psi}$	Yield Strength (Assume A500 Grade B HSS)
$E := 29000\ \text{ksi}$	Modulus of Elasticity of Steel
$L := 11.5\ \text{ft}$	Length of Members
$D := 6\ \text{in}$	Section Depth (Round HSS 6.0x0.5)
$t := 0.465\ \text{in}$	Section Thickness (Round HSS 6.0x0.5)
$Z := 14.3\ \text{in}^3$	Section Modulus (Round HSS 6.0x0.5)
$A_g := 8.09\ \text{in}^2$	Section Area (Round HSS 6.0x0.5)

Calculate Impact Force and Maximum Moment/Shear (AISC 7-22 Section C5.4.5)

$W := 1000\ \text{lbf}$	Debris Weight
$\theta := 26.3\ \text{deg}$	Angle of Member (From Vertical)
$V_b := 4\ \frac{\text{ft}}{\text{s}}$	Velocity of Object (Assumption)
$C_I := 1$	Importance Coefficient
$C_O := 0.8$	Orientation Coefficient
$C_D := 1.0$	Depth Coefficient
$C_B := 1.0$	Blockage Coefficient
$R_{max} := 1.0$	Maximum Response Ratio for Impulsive Load
$g := 32.174\ \frac{\text{ft}}{\text{s}^2}$	Acceleration due to Gravity
$\Delta t := 0.03\ \text{s}$	Impact Duration
$F := \frac{\pi \cdot W \cdot V_b \cdot C_I \cdot C_O \cdot C_B \cdot C_D \cdot R_{max}}{2 \cdot g \cdot \Delta t} = 5.208\ \text{kip}$	Impact Load (C5.4-3)
$F' := F \cdot \cos(\theta) = 4668.606\ \text{lbf}$	Impact Force Normal to Member
$M_{max} := \frac{F' \cdot L}{4} = 13.422\ \text{kip} \cdot \text{ft}$	Maximum Moment (Impact Applied at Mid-Length)
$V_{max} := F' = 4668.606\ \text{lbf}$	Maximum Shear (Impact Applied at End)

Check for Yielding (AISC Section F8-1)

$$\begin{array}{l} \text{if } \frac{D}{t} \le \frac{0.45 \cdot E}{f_y} \\ \quad \text{“Applicable”} \\ \text{else} \\ \quad \text{“Not Applicable”} \end{array} = \text{“Applicable”}$$

Check if Applies

$M_n := f_y \cdot Z = 54.817 \text{ kip} \cdot \text{ft}$

Nominal Moment Capacity for Yielding (Equation F8-1)

Check for Local Buckling (AISC Section F8-2)

$\lambda_p := \frac{0.07 \cdot E}{f_y} = 44.13$

Check for Compactness (Table B4.1b, Case 20)

$$\begin{array}{l} \text{if } \frac{D}{t} \le \lambda_p \\ \quad \text{“Compact, Does not Apply”} \\ \text{else} \\ \quad \text{“Not Compact”} \end{array} = \text{“Compact, Does not Apply”}$$

Check for Moment Capacity

$$\text{Check}_{flex} := \begin{array}{l} \text{if } M_{max} \le \frac{M_n}{1.67} \\ \quad \text{“Pass”} \\ \text{else} \\ \quad \text{“Fail”} \end{array}$$

$\text{Check}_{flex} = \text{“Pass”}$

Check for Shear Capacity (AISC Section G5)

$F_{cr1} := \frac{1.6 \cdot E}{\sqrt{\frac{L}{D}} \cdot \left(\frac{D}{t}\right)^{1.25}} = 395.623 \text{ ksi}$

Equation G5-2a

$F_{cr2} := \frac{0.78 \cdot E}{\left(\frac{D}{t}\right)^{1.5}} = 488.028 \text{ ksi}$

Equation G5-2b

$F_{cr} := min\left(F_{cr1}, F_{cr2}, 0.6 \cdot f_y\right) = 27.6 \text{ ksi}$

Shear Yielding Stress

$$V_n := F_{cr} \cdot \frac{A_g}{2} = 111.642\ \text{kip}$$

Nominal Shear Strength (Equation G5-1)

$$Check_{shear} := \text{if } V_{max} \le \frac{V_n}{1.67}$$

```
    ‖ "Pass"
else
    ‖ "Fail"
```

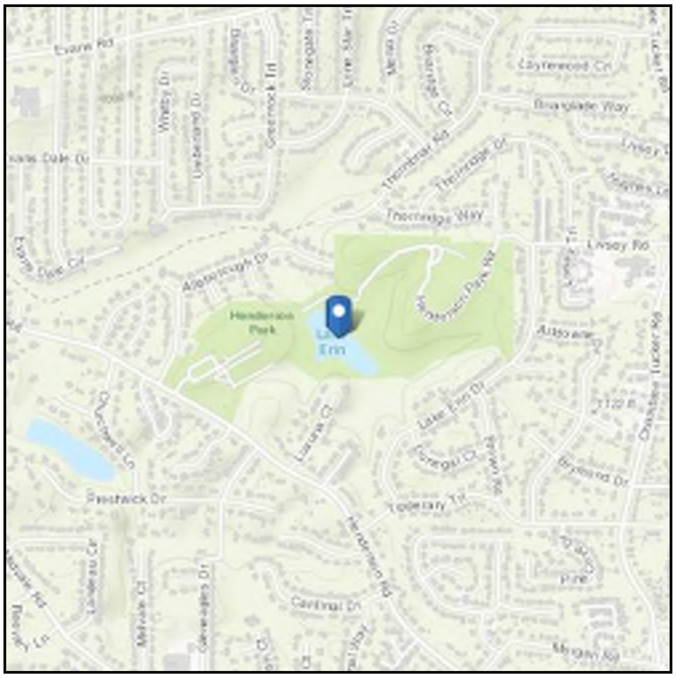
$Check_{shear} = \text{“Pass”}$

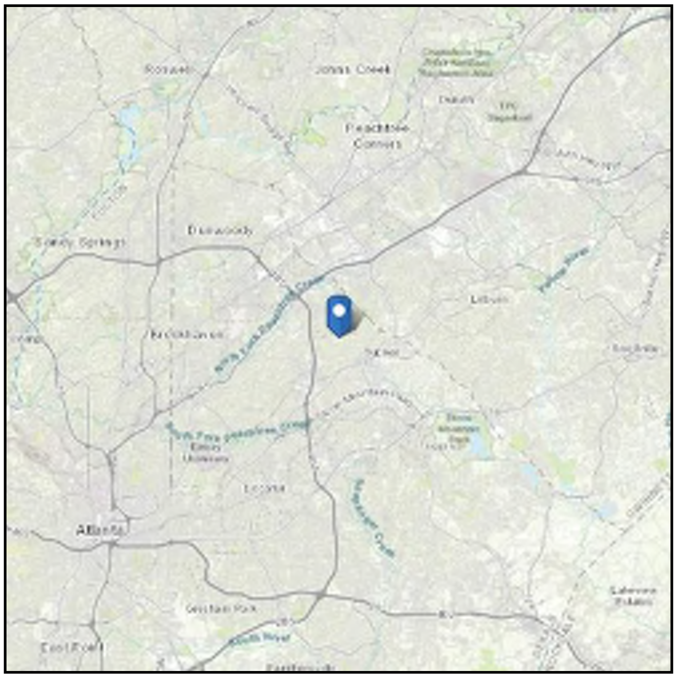
REFERENCES

ASCE Hazards Report

Address:
Lake Erin - Tucker
,

Standard:	ASCE/SEI 7-22	**Latitude:**	33.86634
Risk Category:	IV	**Longitude:**	-84.22934
Soil Class:	D - Stiff Soil	**Elevation:**	956.8242256577443 ft (NAVD 88)





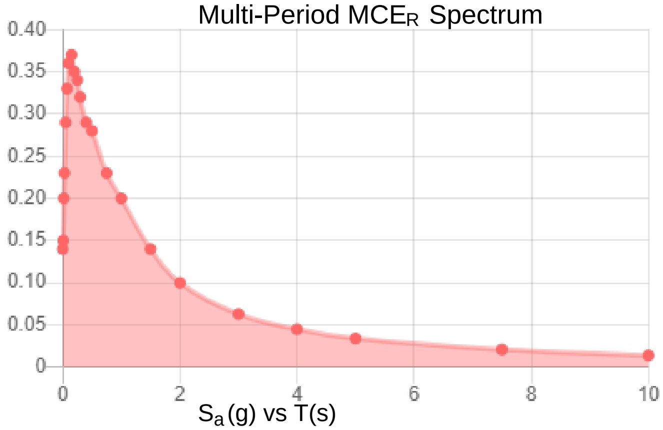
Seismic

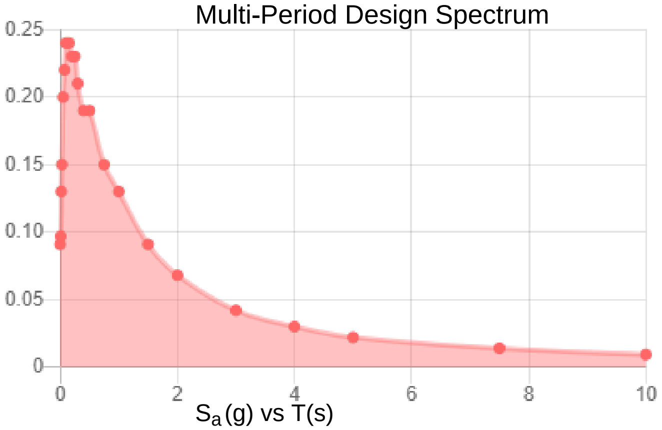
Site Soil Class: D - Stiff Soil

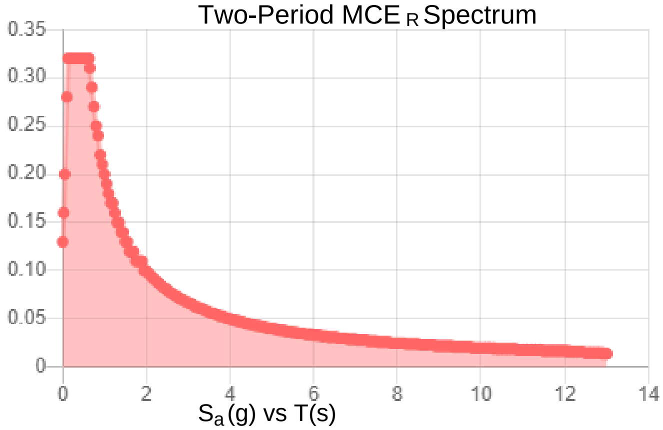
Results:

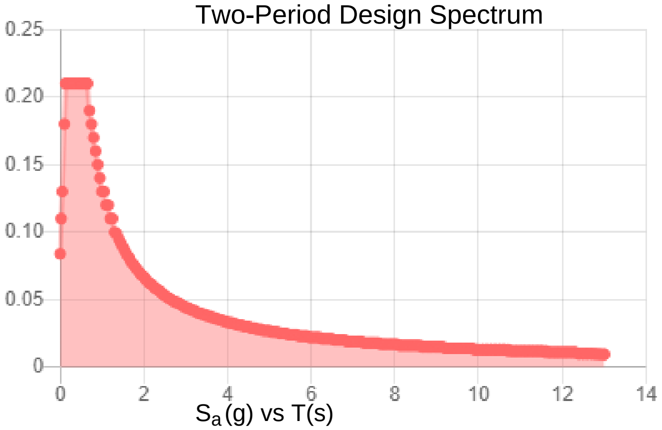
PGA_M :	0.12	T_L :	12
S_{MS} :	0.32	S_S :	0.25
S_{M1} :	0.2	S_1 :	0.094
S_{DS} :	0.21	V_{S30} :	260
S_{D1} :	0.13		

Seismic Design Category: D


Multi-Period MCE_R Spectrum


Multi-Period Design Spectrum


Two-Period MCE_R Spectrum


Two-Period Design Spectrum

MCE_R Vertical Response Spectrum

Vertical ground motion data has not yet been made available by USGS.

Design Vertical Response Spectrum

Vertical ground motion data has not yet been made available by USGS.

Data Accessed: **Mon Jul 01 2024**

Date Source:

USGS Seismic Design Maps based on ASCE/SEI 7-22 and ASCE/SEI 7-22 Table 1.5-2. Additional data for site-specific ground motion procedures in accordance with ASCE/SEI 7-22 Ch. 21 are available from USGS.

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Monthly Mean Avg Temperature for Atlanta Area, GA (ThreadEx)

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
2000	43.1	50.9	57.3	58.6	72.9	77.9	81.4	79.7	70.8	64.1	51.0	37.2	62.1
2001	41.7	50.8	50.4	63.5	69.9	74.4	78.6	78.8	71.3	60.7	59.9	50.1	62.5
2002	46.9	45.4	54.5	64.8	68.3	76.4	80.4	80.2	76.0	65.4	50.9	43.9	62.8
2003	40.0	46.4	55.7	61.7	69.0	74.4	77.9	79.1	72.1	63.5	57.6	43.2	61.7
2004	42.8	43.7	58.3	62.0	72.9	75.9	79.5	76.8	72.2	67.0	56.4	44.6	62.7
2005	46.8	48.8	52.1	60.4	66.7	75.3	79.4	79.6	76.6	64.4	55.2	41.8	62.3
2006	49.4	44.2	54.2	65.9	69.8	77.2	81.0	81.0	71.9	61.7	53.8	50.1	63.4
2007	46.1	45.3	60.2	60.3	71.6	79.6	78.4	85.6	75.8	66.0	53.2	50.7	64.4
2008	42.2	48.3	54.0	61.3	69.5	79.8	79.8	78.2	74.5	62.0	50.6	48.5	62.4
2009	43.9	47.7	55.0	60.7	70.2	79.8	78.1	78.9	73.5	61.0	53.8	42.3	62.1
2010	38.5	39.6	50.7	65.2	73.4	81.4	82.3	82.8	78.0	65.3	54.6	38.3	62.5
2011	40.2	50.0	55.8	65.3	70.9	81.3	82.4	83.0	73.2	62.1	55.7	50.4	64.2
2012	49.1	51.3	64.5	65.8	73.9	77.4	83.8	79.0	74.3	63.4	52.8	51.1	65.5
2013	49.9	46.6	49.1	62.1	68.1	77.5	77.5	77.5	74.4	64.4	50.6	48.1	62.2
2014	37.0	47.3	52.5	62.7	71.2	78.2	77.9	79.0	75.8	66.2	49.0	50.0	62.2
2015	43.1	40.4	57.6	65.7	73.1	79.6	81.3	79.9	73.5	64.1	57.7	57.6	64.5
2016	42.4	48.5	60.5	64.1	71.1	81.0	83.4	82.7	78.8	69.6	58.9	48.9	65.8
2017	52.0	56.1	57.7	67.8	71.1	76.6	81.2	79.2	73.8	65.7	56.2	46.8	65.4
2018	40.4	57.3	53.5	59.8	74.8	79.8	80.8	79.7	81.0	67.4	50.3	48.6	64.5
2019	45.9	53.8	55.4	65.3	76.4	77.7	82.2	82.5	82.4	68.4	52.5	51.2	66.1
2020	49.2	49.9	61.6	61.6	68.9	76.8	82.1	80.9	73.8	67.3	58.9	46.1	64.8
2021	46.2	48.7	59.1	62.9	69.7	77.6	79.9	80.4	73.9	67.2	52.9	56.0	64.5
2022	44.5	51.7	58.1	63.2	73.9	81.1	81.6	79.5	74.7	62.4	56.6	47.5	64.6
2023	51.7	57.1	58.0	63.5	70.3	76.6	83.1	82.6	75.8	65.7	56.0	49.9	65.9
2024	44.3	53.1	58.1	M	M	M	M	M	M	M	M	M	51.8
Mean	44.7	48.9	56.2	63.1	71.1	78.1	80.6	80.3	74.9	64.8	54.4	47.6	63.2
Max	52.0 2017	57.3 2018	64.5 2012	67.8 2017	76.4 2019	81.4 2010	83.8 2012	85.6 2007	82.4 2019	69.6 2016	59.9 2001	57.6 2015	66.1
Min	37.0 2014	39.6 2010	49.1 2013	58.6 2000	66.7 2005	74.4 2003	77.5 2013	76.8 2004	70.8 2000	60.7 2001	49.0 2014	37.2 2000	51.8

Appendix E – Previous Geotechnical Reports

Appendix A

WILLMER ENGINEERING INC. WE

July 19, 2010

VIA EMAIL/U.S. MAIL

J. Mark Kilby, PE
Kimley-Horn and Associates, Inc.
3169 Holcomb Bridge Road
Suite 600
Norcross, Georgia 30071-1367

SUBJECT: **Report of Visual Dam Evaluation**
Erin Lake Dam, Henderson Park
Dekalb County, Georgia
Willmer Project No. 171-3638

Dear Mr. Kilby:

Willmer Engineering Inc. (Willmer) is pleased to present this Report of Visual Dam Evaluation regarding remedial action items resulting from an annual Safe Dams inspection letter dated March 25, 2010 conducted by the Georgia Department of Natural Resources, Environmental Protection Division, Safe Dam's Program. This work was performed in accordance with our proposal P10-208 dated June 11, 2010 and site visits by a Willmer engineer on June 8, 2010 and July 7, 2010.

Introduction

The Erin Lake dam is about 340 feet in length and approximately 30 feet in height, with upstream and downstream slopes that visually appeared to be about 2.5H:1V or flatter. The crest width of the dam is approximately 15 feet. At the time of our visits, the upstream slope was heavily vegetated with shrubs obscuring the slope surface. The downstream slope was vegetated with grass, but in some areas in the lowest part of the slope, the vegetation was missing or very sparse. The emergency spillway is located in the left abutment of the dam and appears to be cut into natural rock.

The Safe Dams March 25, 2010 annual inspection report indicated three geotechnical related issues:

- A potential slough on the downstream slope near the midsection of the slope.
- A hole along the right end of the downstream channel of the emergency spillway.
- A wet area on the downstream slope, near the toe and the emergency spillway channel.

A review of the Safe Dams file on Erin Lake Dam indicated that the first dam inspection was performed by DeKalb County personnel in March 1978. The age or method of construction of the dam is unknown. The lake appears on the 1956 (photo revised 1982) USGS Stone Mountain Quadrangle sheet and is at least 54 years old. It is likely much older. The dam was re-inspected in August 1978 under the U.S. Army Corps of Engineers National Dam Safety Program and was declared to be in very poor condition due to a number of deficiencies. As a result of that inspection, the dam was modified to current standards with flatter slopes, addition of a principal spillway pipe, and addition of a toe drain system.

A review of annual inspection reports copied from the Safe Dams file on the Erin Lake Dam from 2002 to 2008 indicate on-going wet conditions at the downstream toe near the vicinity of the emerging spillway. No downstream slope sloughs were ever noted during these annual site inspections. It should be noted that in 2009, the Atlanta region experienced historic rainfall and subsequent flooding.

Visual Observations

Our site inspection did not indicate any major signs of instability of the dam. Two distinct "sloughs" were observed by the Willmer engineer. The sloughs roughly measured 23 feet in length by 26 feet in width and 13 feet in length by 16 feet in width. The approximate areal extent of the observed sloughs is shown on the attached topographic survey. Photographs of the sloughs are also included. An erosional hole was also observed at the base of the smaller slough. Hand probing of the downstream slope and slough areas did not reveal any "soft" conditions. Probing of the erosional hole indicated 6 to 12 inches of soft materials. The wet area noted in the inspection near the toe and emergency spillway was not evident during our visit. Probing along the downstream toe indicated relatively firm soil conditions during the July 7, 2010 site visit. The erosional hole near the rock wall of the spillway channel was also noted during the visit. The hole is currently filled with branches and other debris.

Conclusions / Recommendations

Based on the observed slough conditions, it is likely the surface soils were saturated by the heavy rains of 2009 and sloughed along a shallow slip surface causing the observed failures. The erosional hole observed at the base of the smaller slough is likely the result of concentrated surface runoff. There is no physical evidence of a deeper failure occurring.

We recommend the removal of all failed slough areas and ersional features and replacing with new compacted fill. On the slopes, the new fill should be keyed into the existing dam. After repair, we recommend that a series of survey stakes be placed along the slope of each slough area and monitored on a regular basis to determine if there is on-going movement particularly after a heavy rainfall event.

Willmer also recommends that a shallow 5 foot auger boring be performed in the observed wet area and a temporary piezometer be installed and monitored to observe water levels in the area. This work will be performed after the repair work is completed on the observed sloughs. The piezometer will consist of a hand slotted 1-inch diameter PVC pipe with the annular space between the pipe and auger hole filled with sand and the top of the auger hole sealed with bentonite chips. We propose monitoring water levels initially on a weekly basis and after heavy rainfall events. Once sufficient data are accumulated, readings will be performed once a month. The data will be provided to Kimely-Horn for transmittal to Safe Dams.

If you have any questions regarding this report, please contact the undersigned.

Sincerely,

WILLMER ENGINEERING INC.

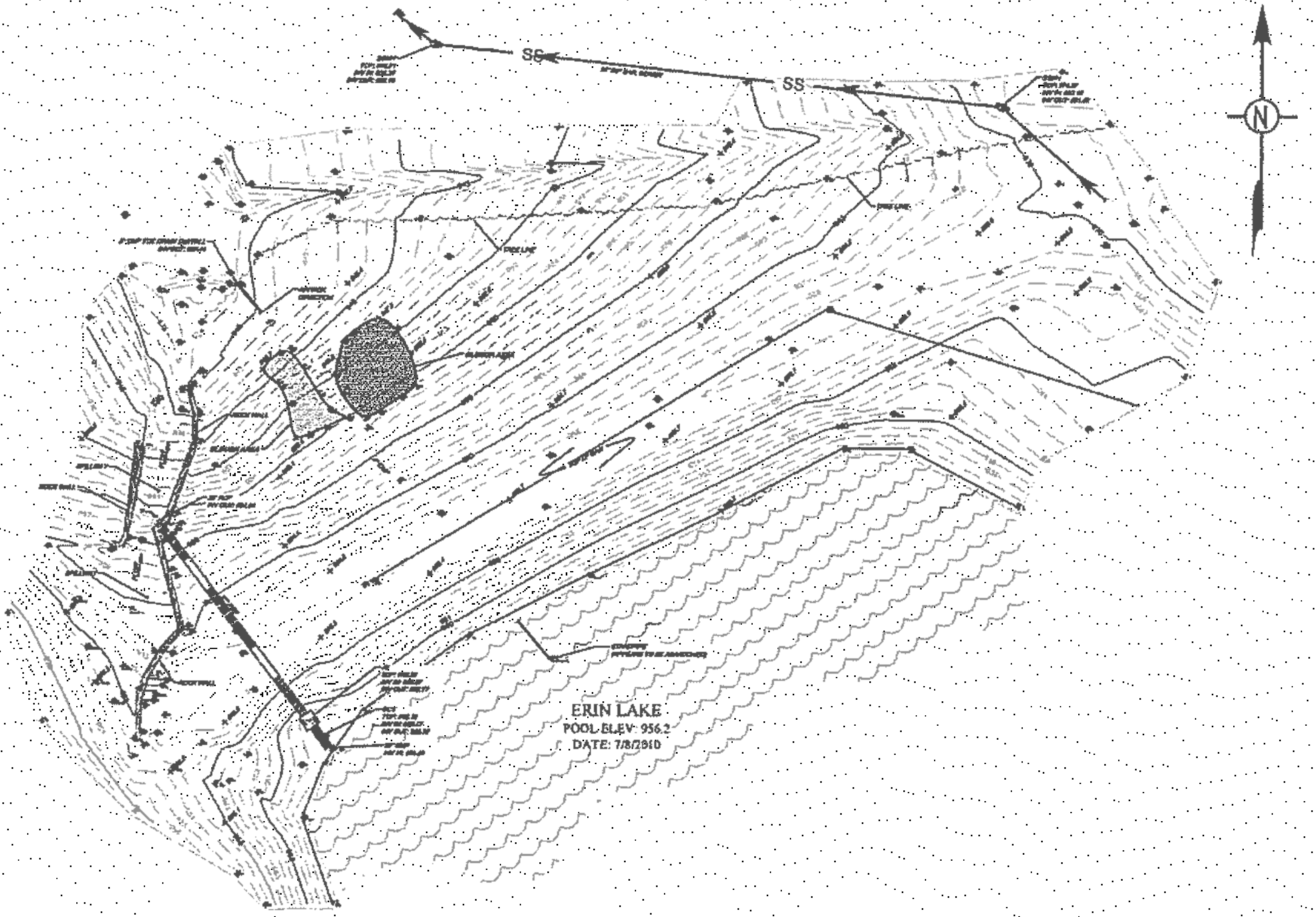
Edmond Leo, PE
Senior Geotechnical Engineer
Safe Dams Engineer of Record for
Geotechnical Engineering

James L. Willmer, PE
Executive Vice President/Principal Consultant

EL/JLW:lmh
[illegible]

Attachments

The original of this document was sealed and signed by
James L. Willmer No. 10780 on July 19, 2010.
THIS REPRODUCTION IS NOT A CERTIFIED DOCUMENT



SCALE: 1" = 50'

DATE: 7/19/2010
DRAWN BY: CDL
REVIEWED BY: EL

WILLMER ENGINEERING INC.

WE

GEOTECHNICAL ENGINEERING • CONSTRUCTION SERVICES
ENVIRONMENTAL SERVICES AND ENGINEERING
3772 PLEASANTDALE ROAD - SUITE 165
ATLANTA, GA 30340-4270

FIGURE 1

ERIN LAKE DAM
TOPOGRAPHIC SURVEY
DEKALB COUNTY, GEORGIA
WILLMER PROJECT No. ATL-171-3838

Larger Slough Area – Downstream Slope, Erin Lake Dam

Larger Slough Area – Downstream Slope, Erin Lake Dam

Smaller Slough Area – Downstream Slope, Erin Lake Dam

Smaller Slough Area – Downstream Slope, Erin Lake Dam

Erosional Hole at Base of Smaller Slough

Kimley-Horn
and Associates, Inc.

Suite 600
3169 Holcomb Bridge Road
Norcross, Georgia
30071-1367

July 27, 2010

Ms. Sree Madichetty
Georgia Environmental Protection Division
Safe Dams Program
4244 International Parkway
Suite 110
Atlanta, Georgia 30354

Re: Engineering Report for Erin Lake Dam
Completed in response to the GSDP letter dated March 25, 2010

Dear Ms. Madichetty,

Kimley-Horn and Associates, Inc. ("KHA") is pleased to submit this letter report to Georgia Safe Dams in order to respond to the March 25, 2010 letter and January 20, 2010 Visual Inspection Report by the Georgia Department of Natural Resources, Environmental Protection Division, Safe Dams Program (GSDP). The following report addresses issues number 3 (slough) and 5 (wetness at toe) as listed in the GSDP letter. Issues 1, 2, 4, and 6 will be addressed directly by DeKalb County. The GSDP letter and Visual Inspection Report are included for reference in Appendix B of this report.

<u>Scope and Strategy</u>

KHA and Wilmer Engineering Inc. (WEI) have performed site visits to observe the six issues and identified potential solutions to address Items 3 and 5. WEI has developed a Report of Visual Dam Evaluation, attached in Appendix A, which recommends removal and replacement of the slough area and the installation of a piezometer at the toe of the dam. After construction, the slough area will be monitored by field survey measurement. It is recommended that survey monuments be installed by the Contractor after repair of the slough area as recommended by WEI's Report dated July 19, 2010. It is recommended that survey monuments be surveyed initially after construction completion and once per quarter following the initial survey. Survey results will be tabulated and provided to the GSDP on an annual basis.

If you have any questions or concerns, please do not hesitate to contact me at 770-825-0744.

Very truly yours,

KIMLEY-HORN AND ASSOCIATES, INC.

J. Mark Kilby

J. Mark Kilby, P.E.
Project Manager

cc: David Pelton
Paige Singer

Appendices Attached:

Appendix A – Willmer Engineering Inc. Report of Visual Dam Evaluation

Appendix B
Exhibit 1: Slough Remediation Area

Appendix C – GSDP Letter and Visual Inspection Report

RECEIVED

JUN - 4 2012

DeKalb County
Roads and Drainage

Georgia Department of Natural Resources
Environmental Protection Division
Safe Dams Program
4244 International Parkway
Suite 110, Atlanta, GA 30354
Mark Williams, Commissioner
Judson H. Turner, Director
404/362-2678

May 25, 2012

Mr. J. Mark Kilby, P.E.
Kimley-Horn and Associates, Inc.
3169 Holcomb Bridge Road
Suite 600
Norcross, Georgia 30071-1367

SUBJECT: Erin Lake Dam
Dekalb County

Dear Mr. Kilby:

The Safe Dams Program has completed its review of the Report prepared by you and Wilmer Engineering, dated July 27, 2010. The following comments relate to our review of the file and your report.

1. The proposed remediation for the two sloughs on the downstream is to excavate the low quality material and replace with competent, well-compacted soil.
    a. Do you know the extent of poor soil in these areas?
    b. Was the material in and around the sloughs evaluated? If so, where is the data?
    c. Where would the borrow soil be obtained for this replacement method?
    d. As noted in the report, portions of metro Atlanta had a significant rainfall event in September 2009. A couple of dams experienced sloughs on their downstream slope. The most notable dam is the Berkeley Lake Dam. A full geotechnical evaluation was performed at this dam and revealed soft soils within the embankment and a compromised internal drain system. An evaluation of the dam (Erin Lake Dam) should be done to verify the internal condition of the dam.
2. The report mentions installing survey monuments on the dam. Information on locations of the monuments and type of monuments should be submitted.
3. Wetness has been observed at the toe of the dam for many years. A review of the Safe Dams Program files notes that Craig Robinson, P.E., with Piedmont Geotechnical Consultants, Inc was retained in 2004 to investigate the wetness. Mr. Robinson's report recommended the installation of observation wells to measure the phreatic surface within the dam. Mr. Robinson's recommendation was never implemented.
4. Review of past inspection reports; showed there were sloughs and holes observed on the upstream slope. This office recommends further investigation of the upstream slope as part of this study. Upstream slope sloughs will affect the overall stability of the dam.
5. The Wilmer Engineering report recommended the installation of a piezometer in the dam. No information was given on the proposed location. Additionally, one well is not sufficient to determine the phreatic surface. Additionally wells should be installed.

DEKALB COUNTY DEPARTMENT OF TRANSPORTATION
ERIN LAKE DAM REHABILITATION
Opinion of Probable Construction Cost, July 27, 2010

BID ITEM NUMBER	ITEM DESCRIPTION	UNIT	QUANTITY	UNIT PRICE	TOTAL PRICE
1	MOBILIZATION	LS	1	$5,000	$5,000
2	EXCAVATION OF SLOUGH AREA (DISPOSE OFF-SITE)	CY	230	$8	$1,840
3	PLACE OFF-SITE STRUCTURAL FILL	CY	275	$16	$4,400
4	SILT FENCE (TYPE 'C')	LF	150	$3	$375
5	PERMANENT VEGETATION	LS	1	$500	$500
6	6" DIA. 36" DEEP CONCRETE MONITORING MONUMENT	EA	8	$50	$400
7	DEMOBILIZATION	LS	1	$5,000	$5,000
8					$0
9					$0
10					$0
11					$0
12					$0
13					$0
14					$0
15					$0
16					$0
17					$0
18					$0
19					$0
20					$0
21					$0
22					$0
23					$0
24					$0
25					$0
26					$0
SUBTOTAL					$17,515
CONVERSION FROM 2010 TO MID 2011 RATES			5%		$18,391
			Total		**$18,400**
		Contingency		**20%**	$22,080
			Grand Total		**$22,100**

* SURVEY MONITORING IS ESTIMATED @ APPROXIMATELY $1,400/VISIT

It is possible the wetness at the toe and occurrence of the sloughs are interrelated. Therefore, this office believes additional investigation is warranted. Because the investigation, analysis and design of any remediation could take over a year this office accepts the proposed replacement of soil as an interim measure. Additionally, at least two sets of piezometers should be installed in cross-section of the dam. At a minimum, there should be piezometers installed at the crest, downstream slope and toe at each cross-section. Additional wells may be warranted. Data from the piezometer and drain outlets should then be collected to aid in determining the phreatic surface and overall stability of this dam.

This office recommends a predesign meeting be conducted to discuss the proposed work and monitoring as well as potential timelines.

If there are any questions, please feel free to contact this office at (404) 362-2678.

Sincerely,

Sree Madichetty
Environmental Engineer
Safe Dams Program

SM:ks
Enclosure
CC: Mr. Carl Glover, Dekalb County Public Works Department
Mr. Mark Dalrymple, Dekalb County Roads and Drainage Department

Appendix C

Georgia Department of Natural Resources
Environmental Protection Division
Safe Dams Program
4244 International Parkway, Suite 110
Atlanta, Georgia 30354
Chris Clark, Commissioner
F. Allen Barnes, Director
(404) 362-2678

7/392-1503

March 25, 2010

Mr. Steve Wesson
Dekalb County Parks & Recreation
4770 North Peachtree Road
Atlanta, Georgia 30338

Subject: Erin's Lake Dam
DeKalb County

Dear Mr. Wesson:

On January 20, 2010, our staff conducted the FY10 annual inspection of this dam. Mr. George McGrade and Mr. Mark Dalrymple accompanied the staff on the inspection. I apologize for the delay in the follow up letter. The following items were noted during the inspection.

1. The vegetation on the dam had been cut. Vegetation on the upstream slope needs to be replaced by low growing grass and maintained. There are animal holes on the upstream slope. These holes need to be filled and compacted.
2. There are some sparse areas of vegetation on the dam. These areas need to be seeded with a low growing grass.
3. There is a potential slough on the downstream slope, at the midsection. You will need to monitor this area for any changes. An Engineer of Record (EOR) should evaluate the slough and recommend repairs.
4. There is a hole along the right end of the downstream channel of the emergency spillway. This should be evaluated by the EOR and repaired.
5. There is a wet area on the downstream slope, near the toe and emergency spillway channel. This area needs to be monitored for any changes.
6. The toe drain channel needs to be cleaned out to allow the channel to drain.

Please inform the Safe Dams Program in writing before **May 27, 2010** of the name of the engineer you have selected to perform the detailed investigation. The engineering report is due no later than **July 30, 2010.**

A copy of the inspection report and photos is enclosed for your records. If there are any questions, please contact our office at (404) 362-2678.

Sincerely,

Sree Madichetty
Environmental Engineer III
Permitting and Compliance Unit
Safe Dams Program

SM:ks
Enclosure
cc: Mr. George McGrade
Mr. Mark Dalrymple

Steve, Our Engineer of Record
is Mark Kilby with
Kimley-Horn & Assoc.
off 678-533-3905
cell 404-291-7009
Mark 3/30/10

S:\Damdocs\2010 annual inspections\follow-up letters\Erins Lake Dam followup.doc

SUBSEQUENT VISUAL INSPECTION

Date of Inspection 1/20/2010

Name of Dam: Erin Lake Dam County: Dekalb

ID#: 044-004-00033 Weather: Sunny and cool

Inspected by: Sree Madichetty and Tom Woosley

Type of Inspection: X Annual ____Construction ____Other

Persons Contacted: No One

Toe Drains Right: flowing could not measure

Toe Drains Left: N/A

Additional Drains: N/A

Type of Principal Pipe: ____

Needs to be Camera Inspected? ____Maybe ____Yes X No

Low-level drain is being tested? ____Not Sure ____Yes ____No

Comments: ____

Number of Pictures Taken: 6

Remarks:

1. The vegetation on the dam had been cut. Vegetation on the upstream slope needs to be replaced by low growing grass and maintained. There are animal holes on the upstream slope. These holes need to be filled and compacted.
2. There are some sparse areas of vegetation on the dam. These areas need to be seeded with a low growing grass.
3. There is a potential slough on the downstream slope, at the midsection. You will need to monitor this area for any changes. An engineer of record should evaluate the slough and slope needs to be repaired.
4. There is a hole along the right end of the downstream channel of the emergency spillway. This should be evaluated by the Engineer of Record and repaired.
5. There is a wet area on the downstream slope, near the toe and emergency spillway channel. This area needs to be monitored for any changes.
6. The toe drain channel needs to be cleaned out to allow the channel to drain.

Erin Lake Dam, Dekalb County
Pictures Taken on January 20, 2010


Upstream slope


Emergency spillway channel


Inlet to the principal spillway pipe


Hole at the right wing-wall of emergency spillway


Left abutment


Slough on the downstream slope (mid-section)

Kimley-Horn
and Associates, Inc.

July 27, 2010

Suite 600
3169 Holcomb Bridge Road
Norcross, Georgia
30071-1367

Ms. Sree Madichetty
Georgia Environmental Protection Division
Safe Dams Program
4244 International Parkway
Suite 110
Atlanta, Georgia 30354

Re: Engineering Report for Erin Lake Dam
Completed in response to the GSDP letter dated March 25, 2010

Dear Ms. Madichetty,

Kimley-Horn and Associates, Inc. ("KHA") is pleased to submit this letter report to Georgia Safe Dams in order to respond to the March 25, 2010 letter and January 20, 2010 Visual Inspection Report by the Georgia Department of Natural Resources, Environmental Protection Division, Safe Dams Program (GSDP). The following report addresses issues number 3 (slough) and 5 (wetness at toe) as listed in the GSDP letter. Issues 1, 2, 4, and 6 will be addressed directly by DeKalb County. The GSDP letter and Visual Inspection Report are included for reference in Appendix B of this report.

Scope and Strategy

KHA and Wilmer Engineering Inc. (WEI) have performed site visits to observe the six issues and identified potential solutions to address Items 3 and 5. WEI has developed a Report of Visual Dam Evaluation, attached in Appendix A, which recommends removal and replacement of the slough area and the installation of a piezometer at the toe of the dam. After construction, the slough area will be monitored by field survey measurement. It is recommended that survey monuments be installed by the Contractor after repair of the slough area as recommended by WEI's Report dated July 19, 2010. It is recommended that survey monuments be surveyed initially after construction completion and once per quarter following the initial survey. Survey results will be tabulated and provided to the GSDP on an annual basis.

If you have any questions or concerns, please do not hesitate to contact me at 770-825-0744.

Very truly yours,

KIMLEY-HORN AND ASSOCIATES, INC.

J. Mark Kilby

J. Mark Kilby, P.E.
Project Manager

cc: David Pelton
Paige Singer

Appendices Attached:
Appendix A – Willmer Engineering Inc. Report of Visual Dam Evaluation

Appendix B
Exhibit 1: Slough Remediation Area

Appendix C – GSDP Letter and Visual Inspection Report

Kimley»Horn

August 26, 2014

Mr. Tom Woosley, P.E.
Georgia Environmental Protection Division
Safe Dams Program
200 Piedmont Avenue, SW
Suite 418
Atlanta, GA 30334

RE: *Erin Lake Dam, DeKalb County*
Status Update and Response to Comments

Dear Mr. Woosley:

Kimley-Horn and Associates, Inc. (Kimley-Horn) is pleased to submit this letter regarding the Erin Lake Dam to the Georgia Department of Natural Resources, Environmental Protection Division, Safe Dams Program (SDP). The purpose of this letter is to give an update of the project status, to transmit resubmittal of the Engineering Report, and to address SDP comments regarding the Engineering Report. A brief project history is shown below:

January 20, 2010
SDP Visual Inspection with six (6) remarks

March 25, 2010
SDP Visual Inspection follow-up letter to DeKalb County Parks and Recreation (DeKalb County) noting the six (6) remarks

July 27, 2010
Kimley-Horn Engineering Report submitted to the SDP with remediation recommendations. Submittal included Willmer Engineering, Inc. (Willmer) Report of Visual Dam Evaluation dated July 19, 2010

May 25, 2012
SDP Review Comments with five (5) comments

August 4, 2014
SDP Letter to Kimley-Horn requesting a progress update

August 26, 2014
Kimley-Horn Engineering Report resubmitted to the SDP (attached to this letter). Submittal includes Willmer's Report of Piezometer Installation and Initial Monitoring dated February 7, 2014

The March 2010 SDP Visual Inspection letter outlined six items that are addressed by the Engineering Report; primarily, slough areas along the downstream slope and wetness at the toe of the dam. In order to respond to the May 2012 SDP Review Comments, the Engineering Report has been updated and is being submitted with this response letter.

Also to address the SDP comments, Willmer has installed piezometers, performed geotechnical exploration at the locations of the piezometers, and made further geotechnical recommendations. The evaluation results are included in Willmer's Report of Piezometer Installation and Initial Monitoring and also summarized in the Engineering Report; the results indicate that there should be further investigation in order to provide final recommendations for repair. This includes additional field sampling and laboratory permeability and triaxial tests to determine phreatic lines and to perform seepage and slope stability analyses. We anticipate incorporating the additional geotechnical recommendations into the Engineering Report and Construction Documents and submitting the documents to the SDP for review.

Although the Engineering Report and Construction Documents are pending additional recommendations, they have been updated and are being submitted with this cover letter. Please accept this letter in response to the SDP comments dated May 25, 2012, relating to the SDP review of the Engineering Report prepared by Kimley-Horn and Willmer dated July 27, 2010. The SDP comments are listed below with our response to each.

1. The proposed remediation for the two sloughs on the downstream is to excavate the low quality material and replace with competent, well-compacted areas.

    a. Do you know the extent of poor soil in these areas?

       The approximate limits of the slough areas are shown on sheet C2-00 of the Construction Documents which are being submitted as an attachment to the Engineering Report. The exact limits of slough remediation area will be determined by the project geotechnical engineer during construction.

    b. Was the material in and around the sloughs evaluated? If so, where is the data?

       Willmer performed 3 hand auger borings to a depth of 3 feet in the sloughed areas of the dam and collected soil samples for classification. See the Willmer Report in Appendix A of the Engineering Report for the results.

    c. Where would the borrow soil be obtained for this replacement method?

       A potential borrow site within the park was evaluated by Willmer but was not found to contain suitable soils therefore an alternate offsite borrow source will be used for the remediation. Once a new borrow source is selected, the borrow soils will be required to be approved by the project geotechnical engineer prior to placement. See the Willmer Report in Appendix A of the Engineering Report for the results of the unsuitable onsite borrow soil analysis and also for recommendations for potential borrow soil properties.

    d. As noted in the report, portions of metro Atlanta had a significant rainfall event in September 2009. A couple of dams experienced sloughs on their downstream slope. The most notable dam is Berkeley Lake Dam. A full geotechnical evaluation was performed at this dam and revealed soft soils within the embankment and a compromised internal drain system. An evaluation of the dam (Erin Lake Dam) should be done to verify the internal condition of the dam.

Willmer performed 6 standard penetration test borings and installed 6 piezometers along the top and downstream face of the dam. Willmer took initial groundwater measurements and will be monitoring the piezometer readings in order to prepare a Report of Geotechnical Investigation after the first year of monitoring. See the Willmer Report included in Appendix A of the Engineering Report for more information.

2. The report mentions installing survey monuments on the dam. Information on locations of the monuments and type of monuments should be submitted.

   Eight 6" diameter concrete monuments at 36" of depth will be constructed within the slough areas during the repairs. After the repair is made, these will be used to facilitate long term monitoring. See sheet C2-00 of the Construction Documents for locations.

3. Wetness has been observed at the toe of the dam for many years. A review of the Safe Dams Program files notes that Craig Robinson, P.E., with Piedmont Geotechnical Consultants, Inc. was retained in 2004 to investigate the wetness. Mr. Robinson's report recommended the installation of observation wells to measure the phreatic surface within the dam. Mr. Robinson's recommendation was never implemented.

   Six observation wells (piezometers) have recently been installed by Willmer in two cross sections to measure the phreatic surface within the dam. Each cross section contains one piezometer located along the top of dam, one in the downstream slope, and one along the toe of the dam embankment. The piezometers will be monitored by Willmer and the readings provided to the SDP. Willmer took initial groundwater readings which are included in the Willmer Report in Appendix A of the Engineering Report. Also, see sheet C2-00 of the Construction Documents for location of wells W1-W6, ranging in depths of 40' to 15'.

4. Review of past inspection reports showed there were sloughs and holes observed on the upstream slope. This office recommends further investigation of the upstream slope as part of this study. Upstream slope sloughs will affect the overall stability of the dam.

   No upstream sloughs or holes were observed during Kimley-Horn site visits. A slope stability analysis is included in the work recommended by Willmer.

5. The Willmer Engineering report recommended the installation of a piezometer in the dam. No information was given on the proposed location. Additionally, one well is not sufficient to determine the phreatic surface. Additionally wells should be installed.

   Six observation wells (piezometers) have recently been installed by Willmer along the top of dam and the downstream embankment to measure the phreatic surface within the dam. Willmer took initial groundwater readings which are included in the Willmer Report in Appendix A of the Engineering Report. Also, see sheet C2-00 of the Construction Documents for location of wells W1-W6.

As indicated in Willmer's Report of Piezometer Installation and Initial Monitoring, further geotechnical investigation is recommended prior to a final recommendation for repair. If you have any questions related to the above responses, please contact me at (678) 533-3932 or kristin.ray@kimley-horn.com.

Very truly yours,

KIMLEY-HORN AND ASSOCIATES, INC.

J. Mark Kilby

By: J. Mark Kilby, P.E.
Engineer of Record

Kristin Ray

Kristin J. Ray, P.E., CFM
Project Manager

Enclosures

cc: David Pelton
Paige Singer

Kimley-Horn
and Associates, Inc.

Suite 450
2 Sun Court
Norcross, Georgia
30092

TEL 770 825 0744

February 10, 2014

Mr. Tom Woosley, P.E.
Georgia Environmental Protection Division
Safe Dams Program
200 Piedmont Avenue, SW
Suite 418
Atlanta, GA 30334

Re: Erin Lake Dam, DeKalb County
Response to Comments Dated May 25, 2012

Dear Mr. Woosley:

Please accept this letter in response to the Safe Dams Program (SDP) comments dated May 25, 2012, relating to the SDP review of the Engineering Report prepared by Kimley-Horn and Associates, Inc. (KHA) and Willmer Engineering, Inc. (Willmer) dated July 27, 2010. The SDP comments are listed below with our response to each. Additionally, the Engineering Report is being resubmitted under separate cover.

1. *The proposed remediation for the two sloughs on the downstream is to excavate the low quality material and replace with competent, well-compacted areas.*
   a. *Do you know the extent of poor soil in these areas?*
   The approximate limits of the slough areas are shown on sheet C2-00 of the Construction Documents which are being submitted as an attachment to the Engineering Report. The exact limits of slough remediation area will be determined by the project geotechnical engineer during construction.
   b. *Was the material in and around the sloughs evaluated? If so, where is the data?*
   Willmer performed 3 hand auger borings to a depth of 3 feet in the sloughed areas of the dam and collected soil samples for classification. See the Willmer Report in Appendix A of the Engineering Report for the results.
   c. *Where would the borrow soil be obtained for this replacement method?*
   A potential borrow site within the park was evaluated by Willmer but was not found to contain suitable soils therefore an alternate offsite borrow source will be used for the remediation. Once a new borrow

Kimley-Horn
and Associates, Inc.

February 10, 2014

Suite 450
2 Sun Court
Norcross, Georgia
30092

TEL 770 825 0744

Mr. Tom Woosley, P.E.
Georgia Environmental Protection Division
Safe Dams Program
200 Piedmont Avenue, SW
Suite 418
Atlanta, GA 30334

Re: Erin Lake Dam, DeKalb County
Response to Comments Dated May 25, 2012

Dear Mr. Woosley:

Please accept this letter in response to the Safe Dams Program (SDP) comments dated May 25, 2012, relating to the SDP review of the Engineering Report prepared by Kimley-Horn and Associates, Inc. (KHA) and Willmer Engineering, Inc. (Willmer) dated July 27, 2010. The SDP comments are listed below with our response to each. Additionally, the Engineering Report is being resubmitted under separate cover.

1. *The proposed remediation for the two sloughs on the downstream is to excavate the low quality material and replace with competent, well-compacted areas.*
    a. *Do you know the extent of poor soil in these areas?*
    The approximate limits of the slough areas are shown on sheet C2-00 of the Construction Documents which are being submitted as an attachment to the Engineering Report. The exact limits of slough remediation area will be determined by the project geotechnical engineer during construction.
    b. *Was the material in and around the sloughs evaluated? If so, where is the data?*
    Willmer performed 3 hand auger borings to a depth of 3 feet in the sloughed areas of the dam and collected soil samples for classification. See the Willmer Report in Appendix A of the Engineering Report for the results.
    c. *Where would the borrow soil be obtained for this replacement method?*
    A potential borrow site within the park was evaluated by Willmer but was not found to contain suitable soils therefore an alternate offsite borrow source will be used for the remediation. Once a new borrow

source is selected, the borrow soils will be required to be approved by the project geotechnical engineer prior to placement. See the Willmer Report in Appendix A of the Engineering Report for the results of the unsuitable onsite borrow soil analysis and also for recommendations for potential borrow soil properties.

d. *As noted in the report, portions of metro Atlanta had a significant rainfall event in September 2009. A couple of dams experienced sloughs on their downstream slope. The most notable dam is Berkeley Lake Dam. A full geotechnical evaluation was performed at this dam and revealed soft soils within the embankment and a compromised internal drain system. An evaluation of the dam (Erin Lake Dam) should be done to verify the internal condition of the dam.*

   Willmer performed 6 standard penetration test borings and installed 6 piezometers along the top and downstream face of the dam. Willmer took initial groundwater measurements and will be monitoring the piezometer readings in order to prepare a Report of Geotechnical Investigation after the first year of monitoring. See the Willmer Report included in Appendix A of the Engineering Report for more information.

2. *The report mentions installing survey monuments on the dam. Information on locations of the monuments and type of monuments should be submitted.*

   Eight 6" diameter concrete monuments at 36" of depth will be constructed within the slough areas during the repairs. After the repair is made, these will be used to facilitate long term monitoring. See sheet C2-00 of the Construction Documents for locations.

3. *Wetness has been observed at the toe of the dam for many years. A review of the Safe Dams Program files notes that Craig Robinson, P.E., with Piedmont Geotechnical Consultants, Inc. was retained in 2004 to investigate the wetness. Mr. Robinson's report recommended the installation of observation wells to measure the phreatic surface within the dam. Mr. Robinson's recommendation was never implemented.*

   Six observation wells (piezometers) have recently been installed by Willmer in two cross sections to measure the phreatic surface within the dam. Each cross section contains one piezometer located along the top of dam, one in the downstream slope, and one along the toe of the dam embankment. The piezometers will be monitored by Willmer and the readings provided to the SDP. Willmer took initial groundwater readings which are included in the

Willmer Report in Appendix A of the Engineering Report. Also, see sheet C2-00 of the Construction Documents for location of wells W1-W6, ranging in depths of 40' to 15'.

4. *Review of past inspection reports; showed there were sloughs and holes observed on the upstream slope. This office recommends further investigation of the upstream slope as part of this study. Upstream slope sloughs will affect the overall stability of the dam.*
   No upstream sloughs or holes were observed during KHA site visits.

5. *The Willmer Engineering report recommended the installation of a piezometer in the dam. No information was given on the proposed location. Additionally, one well is not sufficient to determine the phreatic surface. Additionally wells should be installed.*
   Six observation wells (piezometers) have recently been installed by Willmer along the top of dam and the downstream embankment to measure the phreatic surface within the dam. Willmer took initial groundwater readings which are included in the Willmer Report in Appendix A of the Engineering Report. Also, see sheet C2-00 of the Construction Documents for location of wells W1-W6.

If there are any further outstanding items related to the SDP review of the remedial actions proposed in the Engineering Report or Construction Documents, or if you have any questions related to above responses, please contact me at (678) 533-3932 or kristin.ray@kimley-horn.com.

Very truly yours,

KIMLEY-HORN AND ASSOCIATES, INC.

Kristin J. Ray, PE, CFM
Project Manager

J. Mark Kilby, P.E.
Engineer of Record

cc: David Pelton
Paige Singer

Engineering Report

ERIN LAKE DAM
REHABILITATION

DeKalb County, Georgia

Prepared for:
DeKalb County Transportation Division
1950 West Exchange Place
Tucker, Georgia 30084

Prepared by:
Kimley-Horn and Associates, Inc.
2 Sun Court
Peachtree Corners, Georgia 30092

August 2014
Revision 1

Kimley»Horn

Kimley»Horn

August 26, 2014

Mr. Tom Woosley, P.E.
Georgia Environmental Protection Division
Safe Dams Program
200 Piedmont Avenue, SW
Suite 418
Atlanta, GA 30334

RE: ***Erin Lake Dam, DeKalb County***
Engineering Report, Revision 1

Dear Mr. Woosley:

Kimley-Horn and Associates, Inc. (Kimley-Horn) is pleased to submit this engineering letter report to respond to the January 20, 2010, Visual Inspection and March 25, 2010, follow-up letter by the Georgia Department of Natural Resources, Environmental Protection Division, Safe Dams Program (SDP). Our original Engineering Report dated July 27, 2010, has been updated and is being resubmitted in order to address five (5) review comments received from the SDP dated May 25, 2012.

SDP's 2010 Visual Inspection and follow-up letter outlined six (6) items that are addressed by this report; primarily, slough areas along the downstream slope and wetness at the toe of the dam. Kimley-Horn has teamed with Willmer Engineering, Inc. (Willmer) to address the geotechnical related issues with the dam. Willmer prepared a Report of Visual Dam Evaluation dated July 19, 2010, that was included in with our July 2010 Engineering Report submittal. Also, Willmer has prepared a Report of Piezometer Installation and Initial Monitoring dated February 7, 2014, which is included as Appendix A of this submittal.

Scope and Strategy

Kimley-Horn and Willmer have performed site visits to observe the six issues from the SDP Visual Inspection and have identified potential solutions to address Items 3 (slough area) and 5 (toe wetness). In the July 2010 Kimley-Horn letter report, recommendations were made to remove and replace the slough areas, install survey monuments in the areas, monitor the monuments for movement, and install piezometers. The SDP May 2012 review letter recommended additional investigation and the installation of six piezometers in order to measure the phreatic surface within the dam. Subsequently, six piezometers were installed by Willmer in December 2013. The location of each is shown on the enclosed preliminary Construction Documents and in the Willmer Report. The six piezometers make up two cross-sections with each section having a piezometer on the dam crest, along the downstream slope, and at the dam toe.

Willmer's Report of Piezometer Installation and Initial Monitoring provides an evaluation and recommendations which are summarized below:

1. A possible phreatic surface line corresponding to the lake level at the top of the principal spillway riser pipe would likely exit the downstream slope of the dam at the level of the

observed erosion hole. Existence of various floating debris on the top of the riser pipe indicates that such a condition has occurred in the past.

2. Since the dam embankment consists of loose to very loose silty sand, it is possible that the observed erosion hole was caused by piping of soil due to seepage during heavy rainfall events. It is also possible that the observed sloughing was caused by increased seepage pressure during heavy rainfall events.

3. During the field exploration work, the toe drain outlet (8-inch diameter corrugated metal pipe) was observed to have no significant flow and was partially filled with red-stained sediments. Based on information in the SDP files, the toe drain was possibly constructed in 1979±; therefore, it is about 34 years old. If the drain was constructed with corrugated metal pipe, then it is possible that pipe has corroded and collapsed, blocking any flow through the drain.

4. Willmer recommends that the slough repair work be postponed at this time, and water level monitoring be continued to gather additional data for use in assessment of the cause of the observed sloughing and erosion hole on the downstream slope of the dam.

5. Willmer also recommends that additional field sampling and laboratory permeability and triaxial tests be performed to confirm the possible high phreatic line and to perform slope stability analyses. The results from these analyses, along with the continued water level monitoring data, will be used to confirm their assessment and provide final recommendations for repair.

6. If further analyses indicate that the erosion hole was caused by piping due to seepage through the dam, it will confirm that the existing toe drain is not effective in controlling the phreatic surface.

7. If the existing toe drain cannot be located or made effective by cleaning out, a new toe drain system will be required.

The results shown in the Willmer Report of Piezometer Installation and Initial Monitoring indicate that there should be further investigation in order to provide a final recommendation for repair. Kimley-Horn recommends that further analysis by Willmer be provided before remediation of the slough areas occurs. We anticipate incorporating the additional geotechnical recommendations into the Engineering Report and Construction Documents and submitting the documents to the SDP for review.

Please refer to the Appendices for additional information. Appendix A contains Willmer's Report of Piezometer Installation and Initial Monitoring, Appendix B includes the SDP Visual Inspection and comments letter, and Appendix C contains Kimley-Horn's letter responding to the five (5) SDP comments. Also, a preliminary set of Construction Documents is being submitted.

If you have any questions or concerns, please do not hesitate to contact me at (678)-533-3932 or kristin.ray@kimley-horn.com.

Very truly yours,

KIMLEY-HORN AND ASSOCIATES, INC.

By: J. Mark Kilby, P.E.
Engineer of Record

Kristin J. Ray, P.E., CFM
Project Manager

cc: David Pelton
Paige Singer

Enclosure

Preliminary Construction Documents

Appendices

Appendix A:
Willmer Engineering Inc. Report of Piezometer Installation and Initial Monitoring
Slough Area Laboratory Testing Results provided to KHA via email dated August 23, 2013 and January 3, 2014

Appendix B:
SDP March 25, 2010 Inspection Letter
SDP January 20, 2010 Visual Inspection Report
SDP May 25, 2012 Review Comments Letter

Appendix C:
Kimley-Horn August 26, 2014 Letter Response to SDP Comments

Appendix A

Willmer Engineering, Inc. Report of Piezometer Installation and Initial Monitoring

Slough Area Laboratory Testing Results

February 7, 2014

VIA EMAIL

J. Mark Kilby, PE
Kimley-Horn and Associates, Inc.
2 Sun Court, Suite 450
Norcross, Georgia 30092

SUBJECT: **Report of Piezometer Installation and Initial Monitoring**
Erin Lake Dam Rehabilitation
DeKalb County, Georgia
Willmer Project No. 71.3638

Dear Mr. Kilby:

Willmer Engineering Inc. (Willmer) is pleased to present this report of piezometer installation and initial monitoring for the Erin Lake Dam Rehabilitation project located at Henderson Park in DeKalb County, Georgia. This work was performed in accordance with our subcontract agreement for professional services dated July 15, 2013 with Kimley-Horn and Associates, Inc. (Kimley-Horn). The results of our geotechnical exploration at the piezometer locations, piezometer installation data, initial groundwater level measurements, and our geotechnical assessment/recommendations are presented in this report.

Project Background

The Erin Lake Dam is located at Henderson Park in DeKalb County, Georgia, as shown in Figure 1. The dam is approximately 340 feet long and 30 feet high. The average upstream slope of the dam is about 2.3H:1V and is vegetated with overgrown shrubs. The downstream slope varies from about 2.8H:1V near the southwest end to about 4.6H:1V near the northeast end of the dam. The downstream slope is vegetated with grass. The crest of the dam is approximately 18 feet wide. The dam includes a principal spillway consisting of a 30-inch diameter reinforced concrete pipe (RCP) and an emergency spillway channel that appears to have been cut into natural rock. The principal and emergency spillways are located near the southwest end of the dam (see Figure 2).

The age or method of construction of the dam is unknown. The lake appears on the 1956 (photo-revised 1982) United States Geological Survey Quadrangle Map of Stone Mountain and is, therefore, at least 57 years old. A review of the Georgia Department of Natural Resources - Safe Dams Program (SDP) files on Erin Lake Dam indicates that the first dam inspection was performed by DeKalb County personnel in March 1978. The dam was re-inspected in August 1978 under the U.S. Army Corps of Engineers National Dam Safety Program and was declared to be in very poor condition due to a number of deficiencies. As a result of that inspection, the dam was modified to the current geometry with flatter slopes and the addition of a principal spillway pipe and a toe drain system.

Based on information in the SDP files, the toe drain was designed to consist of an 8-inch diameter perforated pipe encapsulated with No. 57 stone, all wrapped with a filter fabric. The trench for the toe drain was to be 20-inch wide and 3 to 4 feet deep, and the drain was to be covered by a minimum of 12 inches of soil. The toe drain outlet consists of an 8-inch diameter corrugated metal pipe; however, it is not known if corrugated metal pipe was used for the entire toe drain. Although no as-built drawing for the toe drain is available, information in SDP files indicates that the drain was possibly installed along the toe of the original dam prior to flattening the downstream slope.

The Safe Dams annual inspection report dated March 25, 2010 indicated the following three geotechnical-related issues on the downstream slope of the dam: (i) a potential slough on the downstream slope near the midsection of the slope, (ii) a hole along the right end of the downstream channel of the emergency spillway, and (iii) a wet area on the downstream slope, near the toe and the emergency spillway channel. Also the annual inspections reports from 2002 to 2008 (in Safe dams files) indicate on-going wet conditions at the downstream toe near the vicinity of the emergency spillway; no downstream slope sloughs were mentioned in these annual inspection reports. It should be noted that in 2009, the Atlanta region experienced historic rainfall and subsequent flooding.

Willmer performed a visual evaluation of the dam on July 7, 2010. During this inspection, we observed two distinct sloughs at the locations shown in Figure 2. An erosion hole was also observed near the base of the smaller slough (see Figure 2 and 3B). Hand probing of the downstream slope and slough areas did not indicate any unusual soft conditions. Probing of the erosion hole indicated 6 to 12 inches of soft materials. The wet area near the toe and emergency spillway, as noted in the Safe Dams March 25, 2010 inspection report, was not evident during our site visit. Probing along the downstream toe indicated relatively firm soil conditions during our July 7, 2010 site visit. The erosion hole near the rock wall of the spillway channel (as noted in the Safe Dams March 25, 2010 inspection report) was also observed during our visit; the hole was filled with branches and other debris. Based on the visual evaluation, Willmer recommended removing the sloughed material and replacing it with compacted fill, and installing a piezometer in the wet area to monitor the variation of groundwater level with time. These recommendations were presented in a Willmer report dated July 27, 2010.

Upon review of our July 27, 2010 report, Safe Dams recommended that two sets of piezometers be installed to determine the phreatic surface across the dam. Each set was to include one piezometer at the crest, one on the downstream slope, and one at the downstream toe of the dam. The present report pertains to the installation of the piezometers and initial water level monitoring.

Scope of Work

The scope of our work associated with the Erin Lake Dam rehabilitation project consists of the following major tasks:

1. Laboratory testing of soil samples from a potential borrow site for use in repairing the slough areas.
2. Geotechnical inspection and testing during slough repair work.
3. Performance of soil test borings and installation of piezometers at six locations (i.e., three piezometers each along two cross sections).
4. Taking water level readings monthly during the first three months and quarterly thereafter for a total period of three years (i.e., a total of 14 readings).

5. Preparation of a geotechnical report after the first year's piezometer readings to provide an evaluation of the data and our recommendations.

The laboratory testing (i.e., Scope Item 1 listed above) of soil samples from a potential borrow site (designated by DeKalb County) and from the slough areas on the downstream slope of the dam have been completed. The results of these tests were provided to Kimley-Horn and Associates, Inc. in e-mails dated August 23, 2013 and January 3, 2014. Scope Items 2 and 3 will be completed during/after repair of slough areas. The present report pertains to Scope Item 4 (i.e., soil test boring and installation of piezometers) and initial reading of the piezometers.

Standard Penetration Test Borings

For installation of the piezometers, six Standard Penetration Test (SPT) borings (P-1 through P-6) were performed at the crest, mid-slope, and at the toe of the dam. The locations of the SPT borings are shown in Figure 2.

The borings were drilled using a track-mounted rotary drill rig to advance continuous hollow-stem augers. All work was performed under the observation of our geotechnical engineer. The SPT borings were performed in general accordance with ASTM Standard D 1586. The Standard Penetration Test is a widely accepted method for *in situ* testing of soils. A 2-foot long, 2-inch outside-diameter split-barrel sampler attached to the end of a string of drilling rods is driven 18 inches into the ground by successive blows of a 140-pound hammer freely dropping 30 inches. The number of blows needed for each 6 inches of penetration is recorded. The blows required for the first 6 inches of penetration are allowed for seating the sampler into any loose cuttings, and the sum of the blows required for penetration of the second and third 6-inch increments constitutes the penetration resistance or N-value. After the test, the sampler is extracted from the ground and opened to allow visual examination and classification of the retained soil sample. The N-value has been empirically correlated with various soil properties including consistency, relative density, strength, compressibility, and potential for difficult excavation. Correlations between the N-value and the relative density of cohesionless soils (sands) and consistency of cohesive soils (clays/silts) are included in Appendix I. The boreholes were not grouted to allow installation of piezometers.

Visual classification of the soil samples collected was performed in general accordance with ASTM D 2487/D 2488 procedures. Detailed descriptions of the soils encountered in each test boring, along with graphic representations of the standard penetration test blow counts (N-values), are presented on the Soil Boring Logs included in Appendix I.

Subsurface Conditions

Besides the SPT boring results presented in the form of individual boring logs in Appendix I, the boring profiles along with SPT N-values are also shown in Figures 3A and 3B. The stratification lines shown on the boring logs and profiles represent our interpretation of the field logs in accordance with generally accepted geotechnical engineering practice. The stratification lines represent approximate boundaries between soil types; actual transitions between soil types are expected to be gradual. Although individual test borings are representative of the subsurface conditions at the precise boring locations on the dates shown, they are not necessarily indicative of the subsurface conditions at other locations or at other times. Also, in the absence of foreign substances, it is difficult to distinguish between virgin (undisturbed) residual soils and clean soil fill; the soil was classified as fill only at locations and depths where the fill material was visually distinguishable from residual soils.

The dam is comprised of fill soils consisting of silty sand (SM) near the ground surface and generally transitioning to sandy silt (ML) with depth. The thickness of the embankment fill soil found in the borings ranged from about 2 to 29 feet, increasing in thickness from the toe of the dam to the crest, as expected. The relative density of the fill material varied from very loose to loose with SPT N-values ranging from 3 to 9 blows per foot (bpf). The very loose to loose relative density of the fill soils indicates that very little or no compaction efforts were used in construction of the dam.

Underlying the fill, alluvial soils were encountered in all borings and consisted of silty sand (SM), elastic silt (MH) or sandy fat clay (CH). The thickness of the alluvium varied from about 2.5 to 7 feet, and the SPT N-values ranged from 4 to 10 bpf. Residual soils were encountered below the alluvium in borings P-1, P-3, and P-6 and consisted of silty sand (SM). The SPT N-values in the residuum ranged from 7 to 60 bpf. Partially weathered rock (PWR), which is locally defined as material having SPT N-values in excess of 50 for six inches of penetration, was encountered below the alluvium in boring P-5 at a depth of 22 feet below the existing ground surface. The SPT N-value in the PWR was 50 blows for 0 inch of penetration. Borings P-4 and P-6 encountered auger refusal below the alluvium at depths of 33.5 and 12 feet, respectively; auger refusal is generally indicative of the top of parent bedrock.

Groundwater was encountered at each boring location at the time of drilling, and the groundwater elevations are shown on the individual boring logs in Appendix I, on the subsurface profiles in Figures 3A and 3B, and in Table 1. As shown in Table 1, the groundwater elevation recorded during drilling ranged from 933.5 to 942 feet.

Piezometer Installation

Upon completion of SPT boring at each location, a piezometer was installed through the hollow-stem augers. The piezometers were constructed of one-inch diameter, schedule 40 polyvinyl chloride (PVC) pipe with 5 to 10 feet of 0.01-inch machine slotted PVC screen. The screened portion was constructed with a sand pack consisting of washed No. 2 filter pack. The sand pack was placed to 2 feet above the top of the slotted portion of the pipe. Bentonite chips were then placed in the annulus above the sand pack and extended up to about one foot below the ground surface. A protective steel casing was then placed around the PVC pipe and embedded into the bentonite backfill and set in concrete placed above the bentonite. A 2-foot square concrete surface pad was constructed at the ground surface and a locking cap was placed on the casing. The PVC riser and protective casing extend about 3 to 4 feet above the ground surface. Installation records for each piezometer are included in Appendix II.

Initial Monitoring Data

Initial water level measurements for each piezometer were made on December 10, 2013, and a second set of readings were taken on January 10, 2014. Table 1 summarizes the recorded ground surface and groundwater level elevations for each piezometer. In addition to the water level data, we have also recorded the lake level elevation, discharge from the toe drain, and the rainfall data for each monitoring event. The rainfall data was obtained from the National Oceanic and Atmospheric Administration (NOAA) database and corresponds to rainfall observed in the general Atlanta area. As shown in Table 1, the groundwater level reading in the piezometers taken on December 10, 2013 and January 10, 2014 ranged from 938.4 to 949.8 feet, and the lake level elevation ranged between 956.3 and 956.4 feet. No significant discharge was observed at the toe drain outlet. Water was flowing through the principal spillway during both visits to the site.

Preliminary Assessment of Observed Slough and Erosion Hole

The recorded water elevation in the lake and the groundwater elevations in the piezometers (recorded on December 10, 2013) were used to develop approximate phreatic lines for the two cross sections. The phreatic lines are presented in Figures 3A and 3B. As shown in these figures, the water level readings in the lake and the piezometers were generally consistent, resulting in smooth phreatic lines. Figures 3A and 3B also show "possible" phreatic lines representing a rainfall event in which the lake level rises to the top of the principal spillway riser pipe. These "possible" phreatic lines were obtained by translating the December 10, 2013 phreatic lines up along the upstream face of the dam to the elevation of the top of the riser pipe. This scenario would occur if the inlet to the principal spillway pipe were blocked by floating debris or when the principal spillway is inadequate to carry the outflow during a heavy rainfall event. Existence of various flow debris on the top of the riser pipe indicates that such a condition has occurred in the past.

As shown in Figure 3B, the "possible" phreatic line exits the downstream slope of the dam at the level of the observed erosion hole. Since the fill material at this location (P-5) consists of loose to very loose silty sand (SPT N-value of 2 to 6), it is possible that the observed erosion hole was caused by piping of soil due to seepage during heavy rainfall events. It is also possible that the observed sloughing was caused by increased seepage pressure during heavy rainfall events.

Geotechnical Recommendations for Rehabilitation

As indicated earlier in this report, the dam embankment material has a very loose to loose relative density, indicating that very little or no compaction efforts were used in construction of the dam. Also, during our field exploration work, the toe drain outlet was observed to have no significant flow and was partially filled with red-stained sediments. Based on information in the SDP files, the toe drain was possibly constructed in 1979±; therefore, it is about 34 years old. If the drain was constructed with corrugated metal pipe, then it is possible that pipe has corroded and collapsed, blocking any flow through the drain.

In light of the findings described above (i.e., very loose to loose soils in the dam embankment, possibly ineffective toe drain, and possible high phreatic line in the dam during heavy rainfall), we recommend that the slough repair work be postponed at this time, and water level monitoring be continued to gather additional data for use in assessment of the cause of the observed sloughing and erosion hole on the downstream slope of the dam. We also recommend that additional field sampling and laboratory permeability and triaxial tests be performed to confirm the possible high phreatic line and to perform slope stability analyses. The results from these analyses, along with the continued water level monitoring data, will be used to confirm our assessment and provide final recommendations for repair.

If the continued water level monitoring data and seepage and stability analyses indicate that the erosion hole and sloughing were caused by piping due to seepage through the dam, it will confirm that the existing toe drain is not effective in controlling the phreatic surface. At that point, the condition of the toe drain can be evaluated by excavating and manually inspecting the pipe at several locations. If the drain pipe is still is intact, an effort can be made to clean out the drain to make it functional. However, since the exact alignment of the toe drain is not known, it may be difficult to locate the drain for inspection. If the existing toe drain cannot be located or cleaned out, a new toe drain system will be required.

Future Monitoring Schedule

As indicated earlier, piezometer monitoring will be performed monthly during the first three months and quarterly thereafter for a total period of three years (i.e., total of 14 readings). A projected monitoring schedule is provided in Table 2. As we continue monitoring, we will modify the monitoring dates, as needed, in an attempt to obtain water level readings after heavy rainfall events.

Closing Remarks

We appreciate the opportunity to be of service to you on this project. Please contact us if you have any questions regarding the data and recommendations presented in this report or require further assistance.

Sincerely,

WILLMER ENGINEERING INC.

Bradford Drew, EIT
Staff Engineer

Sujit K. Bhowmik, PhD, PE
Chief Engineer/Safe Dams Engineer of
Record for Geotechnical Engineering

James L. Willmer, PE
Executive Vice President/Principal Consultant/Safe
Dams Engineer of Record for Geotechnical
Engineering

Attachments:

TABLES

Table 1

Summary of Piezometer Data
Erin Lake Dam Rehabilitation
DeKalb County, Georgia
Willmer Project No. 71.3638

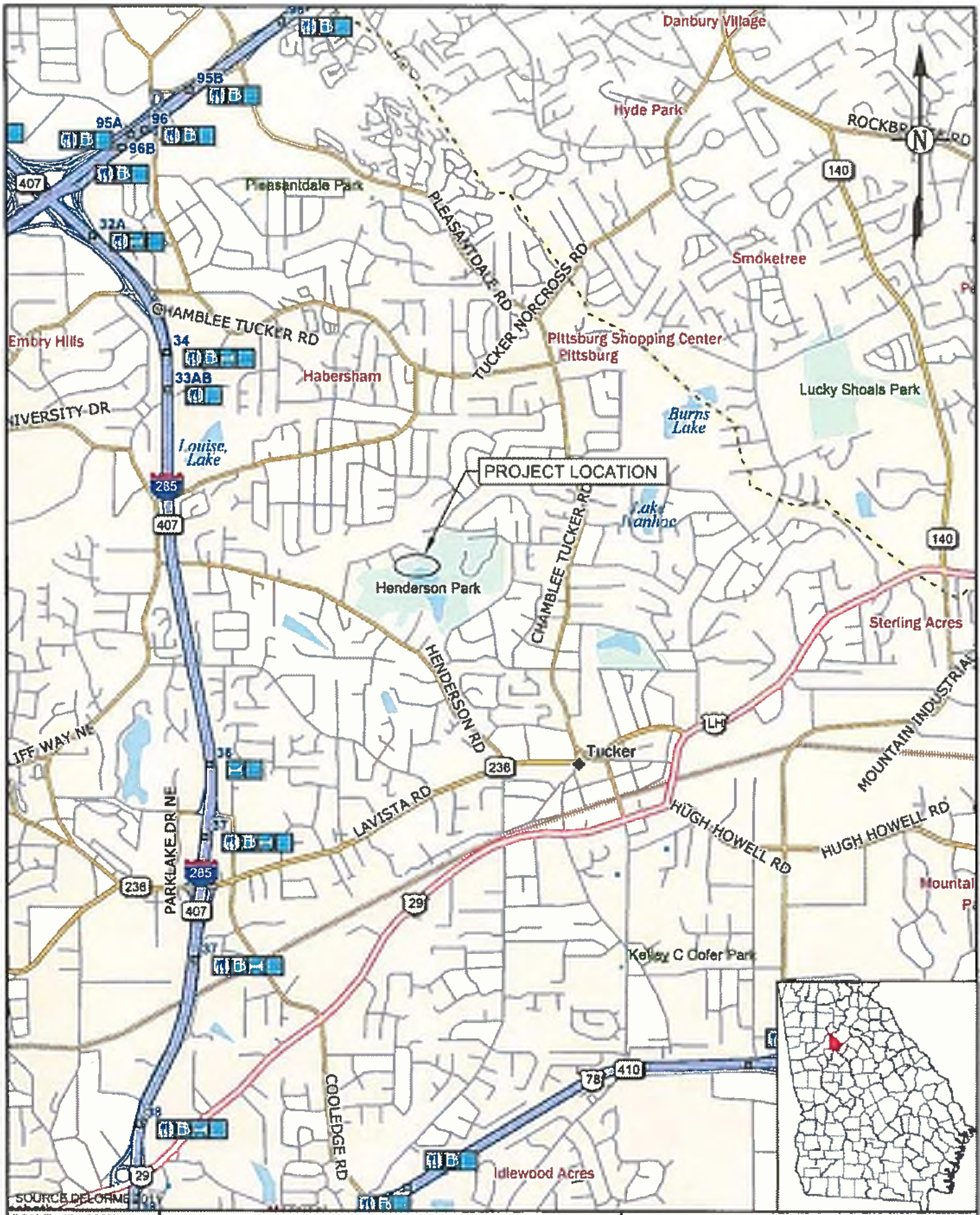
Piezometer No.	Ground Surface Elevation (feet)	Measured Groundwater Elevation (feet)		
		At time of Installation (12/4 and 12/5)	12/10/2013	01/10/2014
P-1	968.5	940.0	949.6	949.8
P-2	959.0	942.0	945.6	945.9
P-3	947.5	937.5	942.4	942.5
P-4	968.5	938.5	946.8	947.0
P-5	956.0	941.0	941.8	941.7
P-6	939.5	933.5	938.4	938.4
Lake Level Elevation (feet)			956.3	956.4
Approximate Toe Drain Discharge (gal/min)			Trickle	Trickle
Observed Rainfall for Previous Month (inches)			1.87	7.80

Table 2

Projected Piezometer Monitoring Schedule
Erin Lake Dam Rehabilitation
DeKalb County, Georgia
Willmer Project No. 71.3638

Record No.	Date
Initial	12/10/2013
1	01/10/2014
2	02/10/2014
3	03/10/2014
4	06/10/2014
5	09/10/2014
6	12/10/2014
7	03/10/2015
8	06/10/2015
9	09/10/2015
10	12/10/2015
11	03/10/2016
12	06/10/2016
13	09/10/2016
14	12/10/2016

FIGURES



SCALE: 1" = 3000'

DATE: 2/6/2014

DRAWN BY: NGD

REVIEWED BY: BD

WILLMER ENGINEERING INC. WE

GEOTECHNICAL ENGINEERING
CONSTRUCTION SERVICES
ENVIRONMENTAL SERVICES AND ENGINEERING
3772 PLEASANTDALE ROAD - SUITE 165
ATLANTA, GA 30340-4270

FIGURE 1

PROJECT LOCATION MAP
ERIN LAKE DAM REHABILITATION
DEKALB COUNTY, GEORGIA
WILLMER PROJECT No. 71.3638

P:\3638-Lake Erin-Henerdon Park Geotech Consult\CADD\PROJECT LOCATION MAP.dwg

APPENDIX I

WILLMER ENGINEERING INC. | WE

BORING RECORD LEGEND

SM, CL, etc: - GROUP SYMBOL based on Unified Soil Classification System.
(Refer to ASTM D-2488 and Table 1 of D-2487)

N-VALUE: BLOWS PER FOOT- Standard Penetration Resistance (SPT) blow count, the sum of the second and third 6-inch increments of the SPT test.
(Refer to ASTM D-1586)

CONSISTENCY / RELATIVE DENSITY Correlated with SPT Blow Count, N:

SILTS AND CLAYS

N (blows per foot)	Consistency
0 - 2	Very Soft
3 - 4	Soft
5 - 8	Firm
9 - 15	Stiff
16 - 30	Very Stiff
31 - 50	Hard
> 50	Very Hard

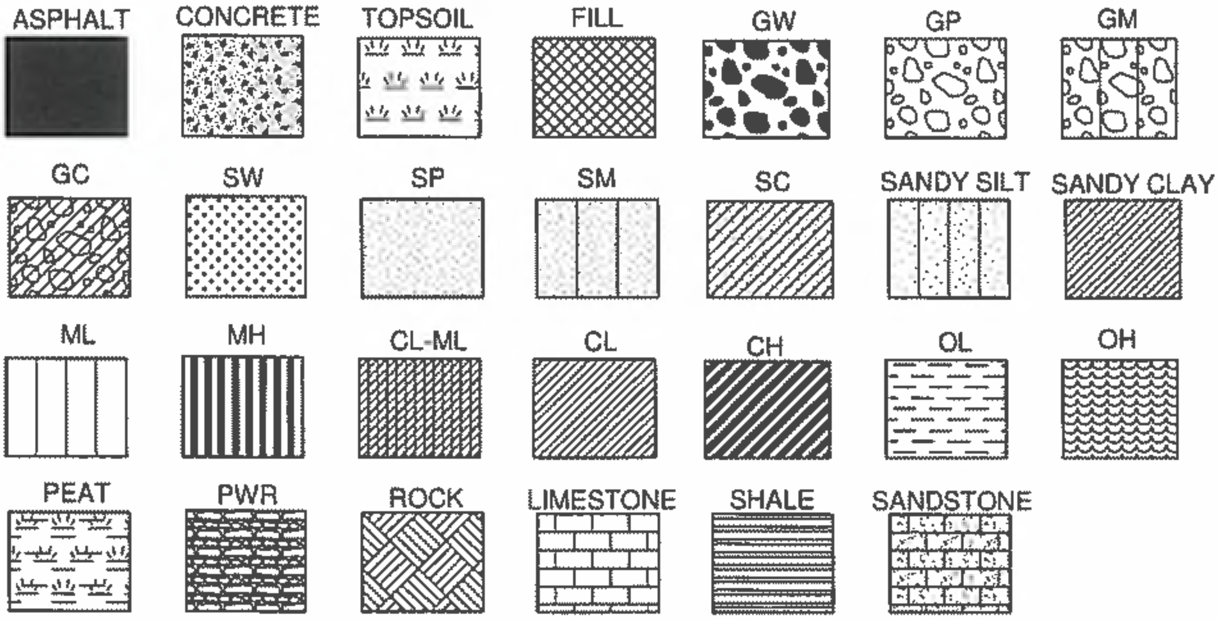
SANDS

N (blows per foot)	Relative Density
0 - 4	Very Loose
5 - 10	Loose
11 - 30	Medium Dense
31 - 50	Dense
> 50	Very Dense

NOTES:

Groundwater Measurements:

▼	Water level at 24 hours
▽	Water level at time of boring
⊠	Caved level at 24 hours



WILLMER ENGINEERING INC. WE

Willmer Engineering Inc.
3772 Pleasantdale Road, Suite 165
Atlanta, Georgia 30340

UNIFIED SOIL CLASSIFICATION SYSTEM REFERENCE SHEET

MAJOR DIVISIONS			LETTER SYMBOL	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN #200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED #4 SIEVE	CLEAN GRAVELS LITTLE OR NO FINES	(GW)	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
			(GP)	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES APPRECIABLE AMOUNT OF FINES	(GM)	SILTY GRAVELS and GRAVEL-SAND-SILT MIXTURES
			(GC)	CLAYEY GRAVELS and GRAVEL-SAND-CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING #4 SIEVE	CLEAN SAND LITTLE OR NO FINES	(SW)	WELL GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
			(SP)	POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
		SANDS WITH FINES APPRECIABLE AMOUNT OF FINES	(SM)	SILTY SANDS and SAND-SILT MIXTURES
			(SC)	CLAYEY SANDS and SAND-CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN #200 SIEVE SIZE	SILTS AND CLAYS LIQUID LIMIT LESS THAN 50		(ML)	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR VERY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
			(CL)	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			(OL)	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50		(MH)	INORGANIC ELASTIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS
			(CH)	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			(OH)	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS			(PT)	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

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WILLMER ENGINEERING INC. WE

Project: **Erin Lake Dam Rehabilitation**
Location: **DeKalb County, Georgia**
Project Number: **71.3638**

HOLE No. P-1

Sheet 1 of 1
Location: **See Figure 2**

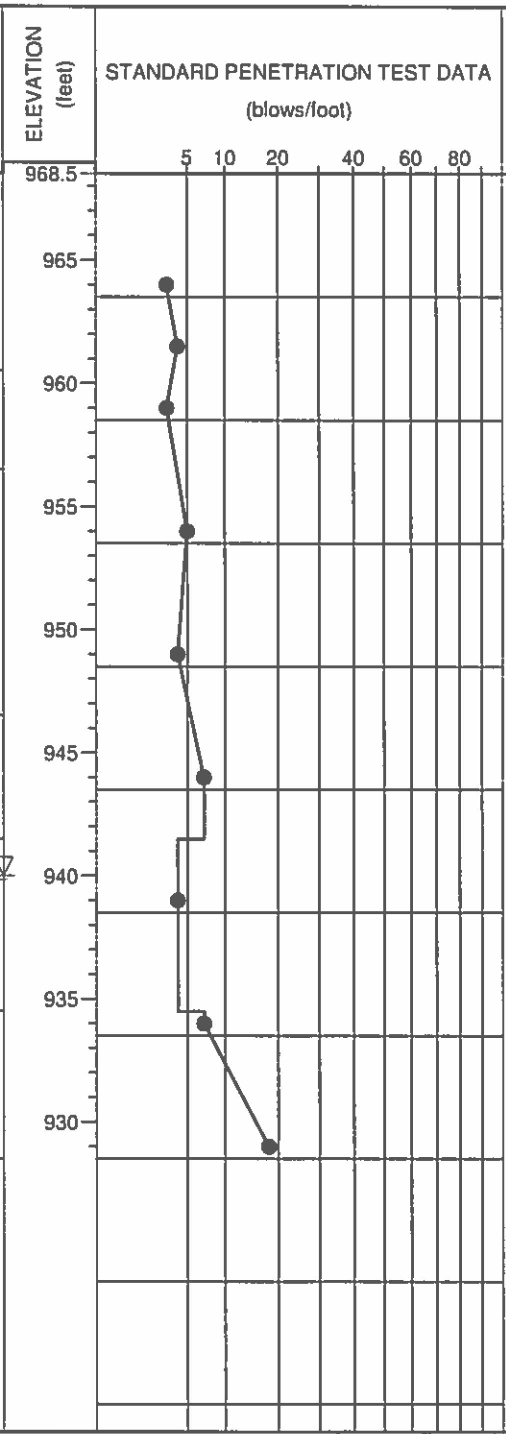
Azimuth: **--** Angle from Horizontal: **90** Surface Elevation (ft): **968.50** Station: **N/A**
Drilling Equipment: **CME 45 Track Mounted Rig** Drilling Method: **HSA - Automatic Hammer**
Core Boxes: **N/A** Samples: **9** Overburden (ft): **N/A** Rock (ft): **N/A** Total Depth (ft): **40.0**
Logged By: **BD** Date Drilled: **12/4/13**

GRAPHIC LOG	SAMPLE TYPE	REC%	RQD %	MATERIAL DESCRIPTION		N-VALUE
▨	SS			FILL: Very loose brown to reddish brown silty medium to fine **SAND** (slightly micaceous)	SM	3
▨	SS					4
▨	SS			Soft reddish brown medium to fine sandy elastic **SILT** (micaceous)	MH	3
▨	SS			Soft to firm grayish brown medium to fine sandy clayey **SILT** (slightly micaceous)	ML	5
▨	SS					4
▨	SS			Loose grayish brown clayey medium to fine **SAND**	SC	7
▧	SS			ALLUVIUM: Very loose gray clayey/silty medium to fine **SAND**	SC SM	4
	SS			RESIDUUM: Loose to medium dense gray and white silty medium to fine **SAND** (slightly micaceous)	SM	7
	SS					18

VERTICAL DEPTH (ft): 5, 10, 15, 20, 25, 30, 35, 40

Boring was terminated at 40 feet below the existing ground surface.

Groundwater was encountered at 28.5 feet below the existing ground surface at the time of boring completion.



SAMPLER TYPE
SS - Split Spoon
ST - Shelby Tube
NQ - Rock Core, 1-7/8"
NX - Rock Core, 2-1/8"
CU - Cuttings
CT - Continuous Tube

DRILLING METHOD
HSA - Hollow Stem Auger
CFA - Continuous Flight Augers
DC - Driving Casing
RW - Rotary Wash
RC - Rock Core

Hole No.
P-1

WILLMER ENGINEERING INC. | WE

Project: **Erin Lake Dam Rehabilitation**
Location: **DeKalb County, Georgia**
Project Number: **71.3638**

HOLE No. P-2

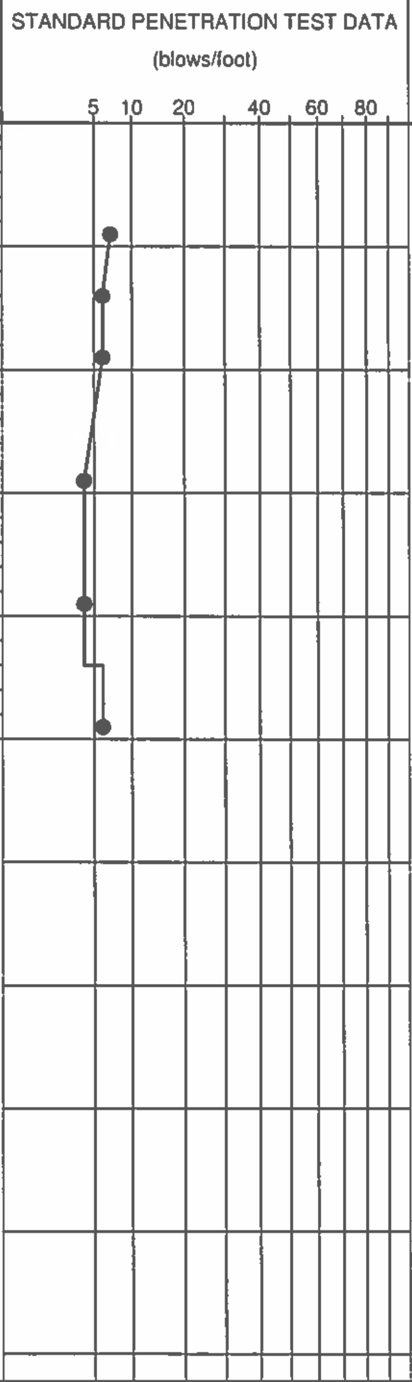
Sheet 1 of 1
Location: **See Figure 2**

Azimuth: -- Angle from Horizontal: 90 Surface Elevation (ft): 959.00 Station: N/A
Drilling Equipment: CME 45 Track Mounted Rig Drilling Method: HSA - Automatic Hammer
Core Boxes: N/A Samples: 6 Overburden (ft): N/A Rock (ft): N/A Total Depth (ft): 25.0
Logged By: BD Date Drilled: 12/5/13

VERTICAL DEPTH (ft)	GRAPHIC LOG	SAMPLE TYPE	REC%	RQD %	MATERIAL DESCRIPTION		ELEVATION (feet)	N-VALUE
					FILL: Loose reddish brown and gray silty medium to fine **SAND** (slightly micaceous)	SM	959.0	
5		SS					955	7
		SS						6
					Soft to firm grayish brown medium to fine sandy **SILT** (slightly micaceous)	ML		
10		SS					950	6
15		SS					945	4
							∇	
20		SS					940	4
					POSSIBLE ALLUVIUM: Loose gray silty medium to fine **SAND** (slightly micaceous) with partially decayed wood debris	SM		
25		SS					935	6

Boring was terminated at 25 feet below the existing ground surface.

Groundwater was encountered at 17 feet below the existing ground surface at the time of boring completion.


STANDARD PENETRATION TEST DATA (blows/foot)

SAMPLER TYPE
SS - Split Spoon
ST - Shelby Tube
NQ - Rock Core, 1-7/8"
NX - Rock Core, 2-1/8"
CU - Cuttings
CT - Continuous Tube

DRILLING METHOD
HSA - Hollow Stem Auger
CFA - Continuous Flight Augers
DC - Driving Casing
RW - Rotary Wash
RC - Rock Core

Hole No.
P-2

WILLMER ENGINEERING INC. | WE

Project: **Erin Lake Dam Rehabilitation**
Location: **DeKalb County, Georgia**
Project Number: **71.3638**

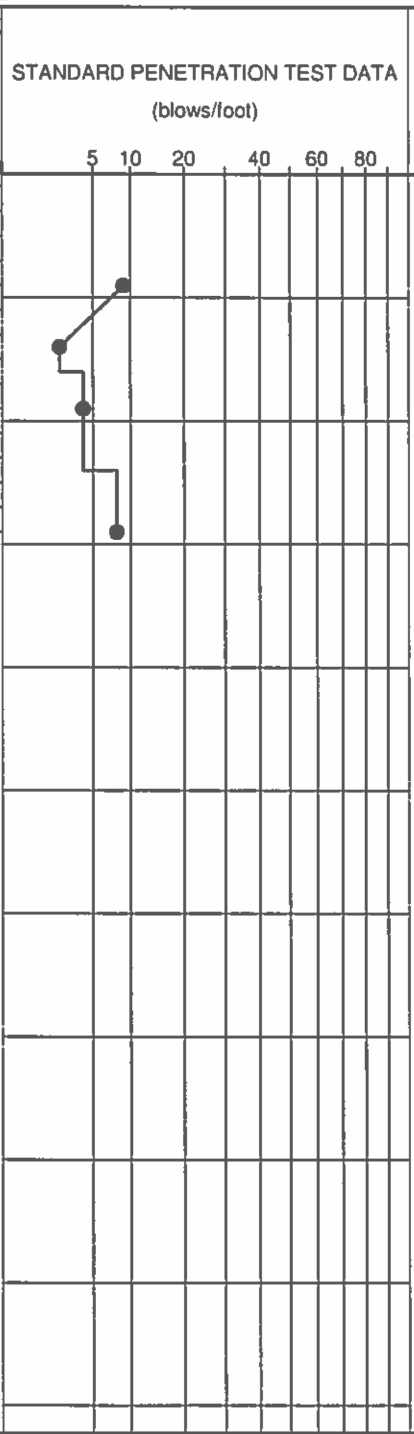
HOLE No. P-3
Sheet 1 of 1
Location: **See Figure 2**

Azimuth: -- Angle from Horizontal: 90 Surface Elevation (ft): 947.50 Station: N/A
Drilling Equipment: CME 45 Track Mounted Rig Drilling Method: HSA - Automatic Hammer
Core Boxes: N/A Samples: 4 Overburden (ft): N/A Rock (ft): N/A Total Depth (ft): 15.0
Logged By: BD Date Drilled: 12/5/13

VERTICAL DEPTH (ft)	GRAPHIC LOG	SAMPLE TYPE	REC%	RQD %	MATERIAL DESCRIPTION		ELEVATION (feet)	N-VALUE
0					FILL: Brown silty medium to fine **SAND** (slightly micaceous)	SM	947.5	
					Very loose to loose grayish brown silty medium to fine **SAND** (micaceous)	SM	945	
5		SS						9
		SS					940	2
10		SS			ALLUVIUM: Soft light blueish gray medium to fine sandy fat **CLAY**	CH	▽	4
					RESIDUUM: Loose gray, white, and brown silty medium to fine **SAND** (very micaceous)	SM	935	
15		SS						8

Boring was terminated at 15 feet below the existing ground surface.

Groundwater was encountered at 10 feet below the existing ground surface at the time of boring completion.



SAMPLER TYPE
SS - Split Spoon
ST - Shelby Tube
NQ - Rock Core, 1-7/8"
NX - Rock Core, 2-1/8"
CU - Cuttings
CT - Continuous Tube

DRILLING METHOD
HSA - Hollow Stem Auger
CFA - Continuous Flight Augers
DC - Driving Casing
RW - Rotary Wash
RC - Rock Core

Hole No. **P-3**

WILLMER ENGINEERING INC. | WE

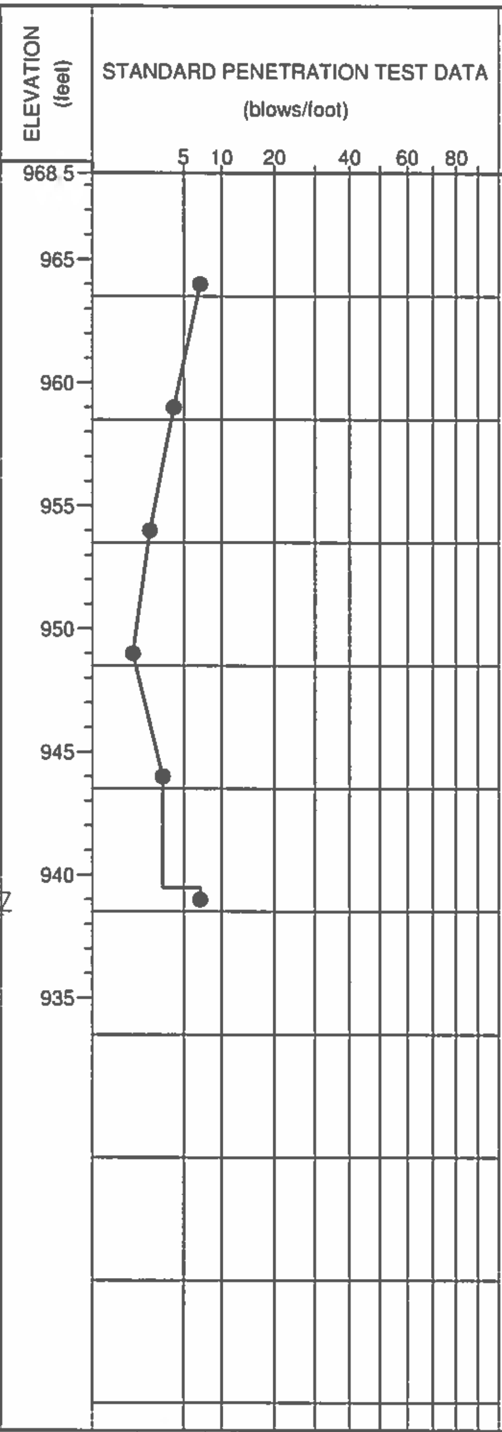
Project: **Erin Lake Dam Rehabilitation**
Location: **DeKalb County, Georgia**
Project Number: **71.3638**

HOLE No. P-4

Sheet 1 of 1
Location: **See Figure 2**

Azimuth: -- Angle from Horizontal: 90 Surface Elevation (ft): 968.50 Station: N/A
Drilling Equipment: **CME 45 Track Mounted Rig** Drilling Method: **HSA - Automatic Hammer**
Core Boxes: N/A Samples: 6 Overburden (ft): N/A Rock (ft): N/A Total Depth (ft): 33.5
Logged By: BD Date Drilled: 12/4/13

VERTICAL DEPTH (ft)	SAMPLE TYPE	REC%	RQD %	MATERIAL DESCRIPTION		ELEVATION (feet)	N-VALUE
0				FILL: Very loose to loose reddish brown to brown silty medium to fine **SAND** (slightly micaceous)	SM	968.5	
5	SS						7
10	SS						4
12				Very soft to soft grayish brown medium to fine sandy **SILT** (slightly micaceous)	ML		
15	SS						2
20	SS						1
25	SS						3
30	SS			ALLUVIUM: Firm gray sandy clayey elastic **SILT** with trace rock fragments	MH ▽		7
33.5				Auger refusal was encountered at 33.5 feet below the existing ground surface. Groundwater was encountered at 30 feet below the existing ground surface at the time of boring completion.			



SAMPLER TYPE		DRILLING METHOD		Hole No.
SS - Split Spoon	NX - Rock Core, 2-1/8"	HSA - Hollow Stem Auger	RW - Rotary Wash	P-4
ST - Shelby Tube	CU - Cuttings	CFA - Continuous Flight Augers	RC - Rock Core	
NQ - Rock Core, 1-7/8"	CT - Continuous Tube	DC - Driving Casing		

SPT BORING LOGS.GPJ 2/4/14

WILLMER ENGINEERING INC. WE

Project: **Erin Lake Dam Rehabilitation**
Location: **DeKalb County, Georgia**
Project Number: **71.3638**

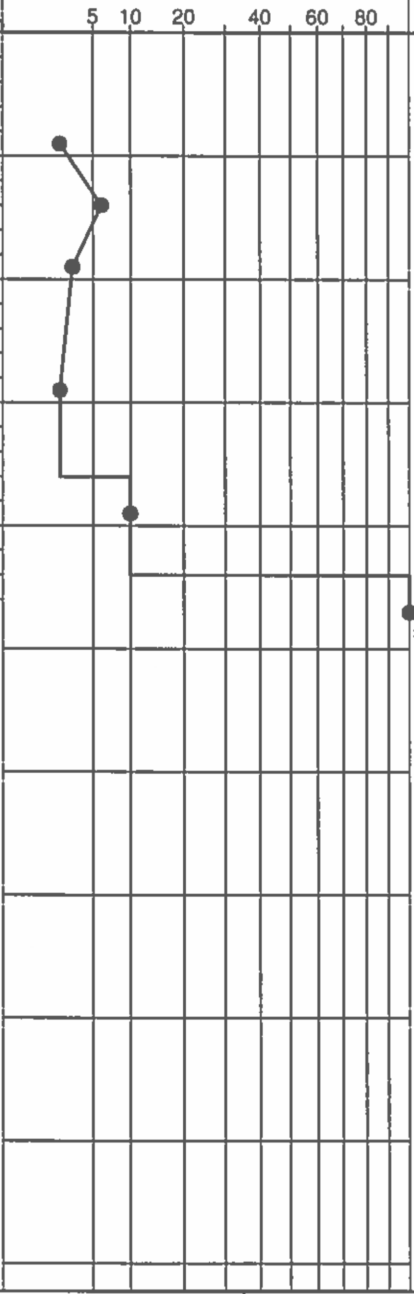
HOLE No. P-5
Sheet 1 of 1
Location: **See Figure 2**

Azimuth: **--** Angle from Horizontal: **90** Surface Elevation (ft): **956.00** Station: **N/A**
Drilling Equipment: **CME 45 Track Mounted Rig** Drilling Method: **HSA - Automatic Hammer**
Core Boxes: **N/A** Samples: **6** Overburden (ft): **N/A** Rock (ft): **N/A** Total Depth (ft): **23.5**
Logged By: **BD** Date Drilled: **12/5/13**

VERTICAL DEPTH (ft)	GRAPHIC LOG	SAMPLE TYPE	REC%	RQD %	MATERIAL DESCRIPTION		ELEVATION (feet)	N-VALUE
					FILL: Very loose reddish brown silty medium to fine **SAND** (slightly micaceous)	SM	956.0 955	
5		☒ SS						2
					Very loose to loose red and grayish brown clayey/silty medium to fine **SAND**	SC/SM	950	
		☒ SS						6
10		☒ SS						3
					Very loose grayish brown silty medium to fine **SAND** (slightly micaceous)	SM	945	
15		☒ SS					▽	2
							940	
					ALLUVIUM: Loose grayish brown silty medium to fine **SAND** with pebbles and trace root fragments	SM		
20		☒ SS						10
							935	
		SS			PARTIALLY WEATHERED ROCK: no sample obtained	PWR		50/0"

Boring was terminated at 23.5 feet below the existing ground surface.

Groundwater was encountered at 15 feet below the existing ground surface at the time of boring completion.



SAMPLER TYPE
SS - Split Spoon
ST - Shelby Tube
NQ - Rock Core, 1-7/8"
NX - Rock Core, 2-1/8"
CU - Cuttings
CT - Continuous Tube

DRILLING METHOD
HSA - Hollow Stem Auger
CFA - Continuous Flight Augers
DC - Driving Casing
RW - Rotary Wash
RC - Rock Core

Hole No.
P-5

SPTM SPT BORING LOGS.GPJ 2/4/14

WILLMER ENGINEERING INC. WE

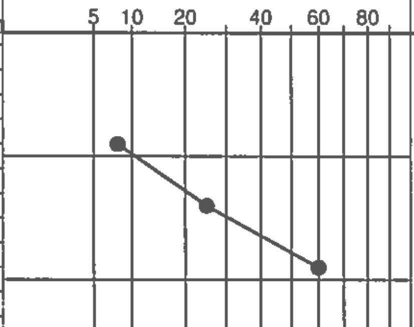
Project: **Erin Lake Dam Rehabilitation**
Location: **DeKalb County, Georgia**
Project Number: **71.3638**

HOLE No. P-6

Sheet 1 of 1
Location: **See Figure 2**

Azimuth: **--** Angle from Horizontal: **90** Surface Elevation (ft): **939.50** Station: **N/A**
Drilling Equipment: **CME 45 Track Mounted Rig** Drilling Method: **HSA - Automatic Hammer**
Core Boxes: **N/A** Samples: **3** Overburden (ft): **N/A** Rock (ft): **N/A** Total Depth (ft): **12.0**
Logged By: **BD** Date Drilled: **12/5/13**

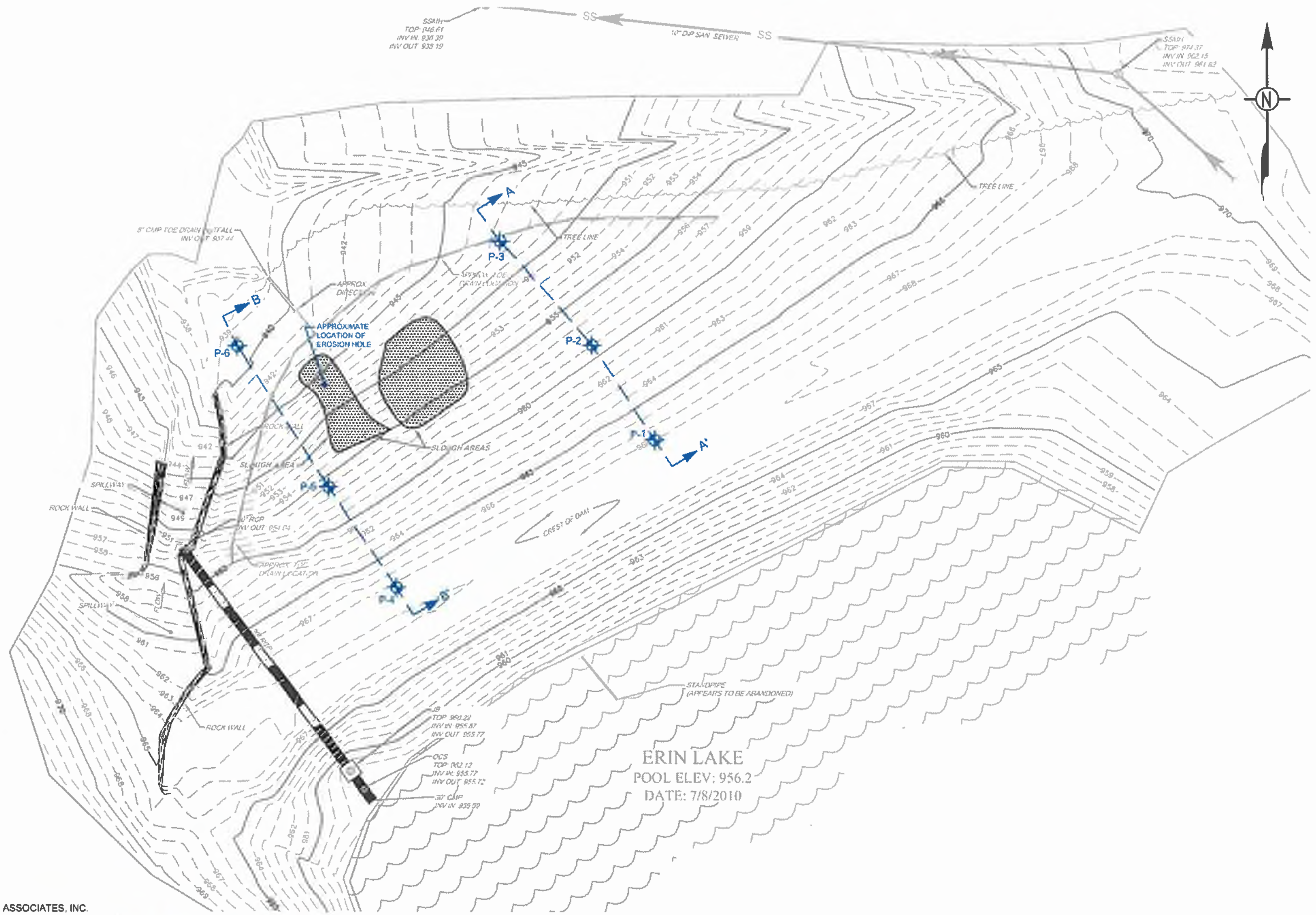
VERTICAL DEPTH (ft)	GRAPHIC LOG	SAMPLE TYPE	REC%	RQD %	MATERIAL DESCRIPTION		ELEVATION (feet)	STANDARD PENETRATION TEST DATA (blows/foot)	N-VALUE
					FILL: Reddish brown silty medium to fine **SAND** (micaceous)	**SM**	939.5		
					ALLUVIUM: Gray silty medium to fine **SAND** with trace pebbles	**SM**			
5		SS			RESIDUUM: Loose gray and white silty medium to fine **SAND**	**SM**	935		8
		SS			Medium dense to very dense brown, red, and tan (mottled black) silty medium to fine **SAND**	**SM**			25
10		SS					930		60



Auger refusal was encountered at 12 feet below the existing ground surface.

Groundwater was encountered at 6 feet below the existing ground surface at the time of boring completion.

SAMPLER TYPE		DRILLING METHOD		Hole No.
SS - Split Spoon	NX - Rock Core, 2-1/8"	HSA - Hollow Stem Auger	RW - Rotary Wash	
ST - Shelby Tube	CU - Cuttings	CFA - Continuous Flight Augers	RC - Rock Core	
NQ - Rock Core, 1-7/8"	CT - Continuous Tube	DC - Driving Casing		**P-6**



LEGEND:

⌖ P-1 = APPROXIMATE SPT BORING/ PIEZOMETER LOCATION

NOTE: BASE DRAWING PROVIDED BY KIMLEY-HORN & ASSOCIATES, INC.

SCALE: 1" = 30'

DATE: 2/6/2014

DRAWN BY: ZMH

REVIEWED BY: BD

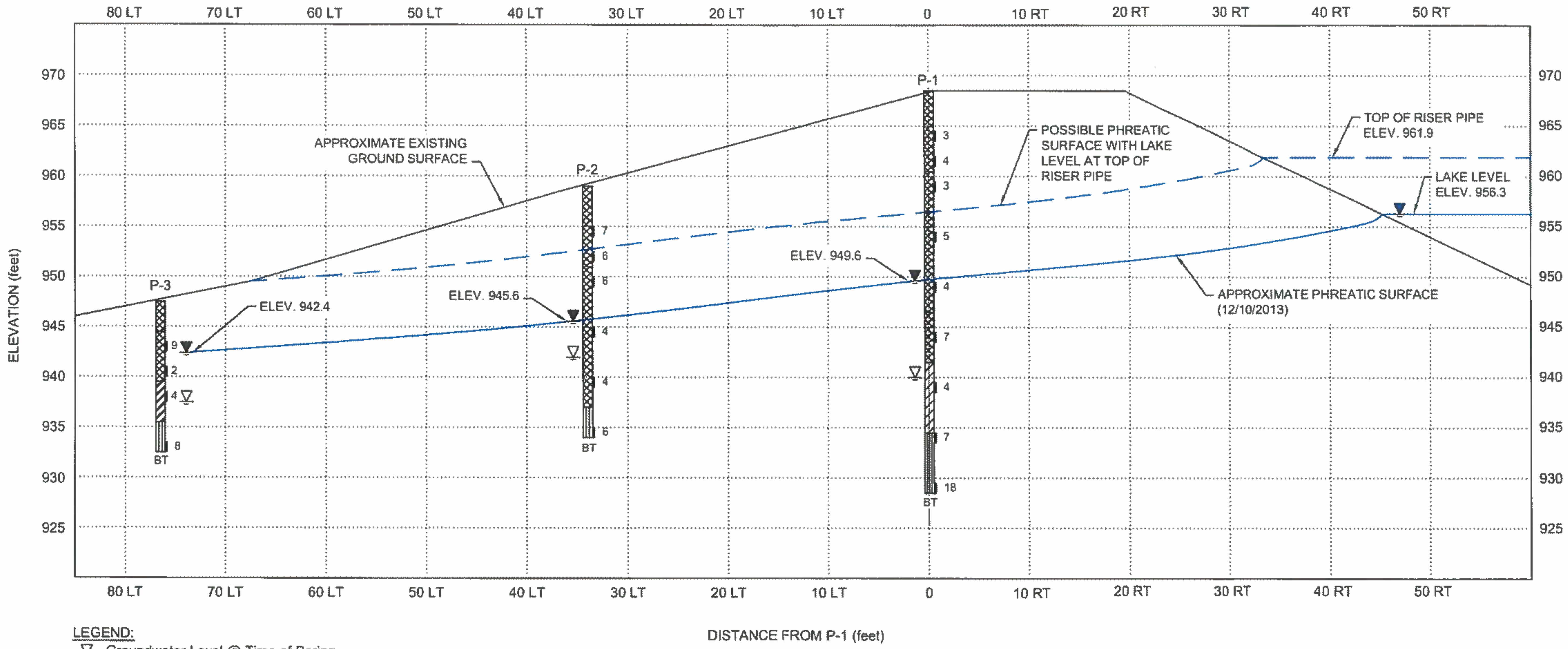
WILLMER ENGINEERING INC.

GEOTECHNICAL ENGINEERING ¤ CONSTRUCTION SERVICES
ENVIRONMENTAL SERVICES AND ENGINEERING
3772 PLEASANTDALE ROAD - SUITE 165
ATLANTA, GA 30340-4270

FIGURE 2

BORING/PIEZOMETER LOCATION PLAN
ERIN LAKE DAM REHABILITATION
DEKALB COUNTY, GEORGIA
WILLMER PROJECT No. 71.3638

P:\3638-Lake Erin-Henerdon Park Geotech Consult\CADD\Piezometer Location Plan.dwg



LEGEND:

∇ - Groundwater Level @ Time of Boring

▼ - Groundwater Level on 12/10/2013

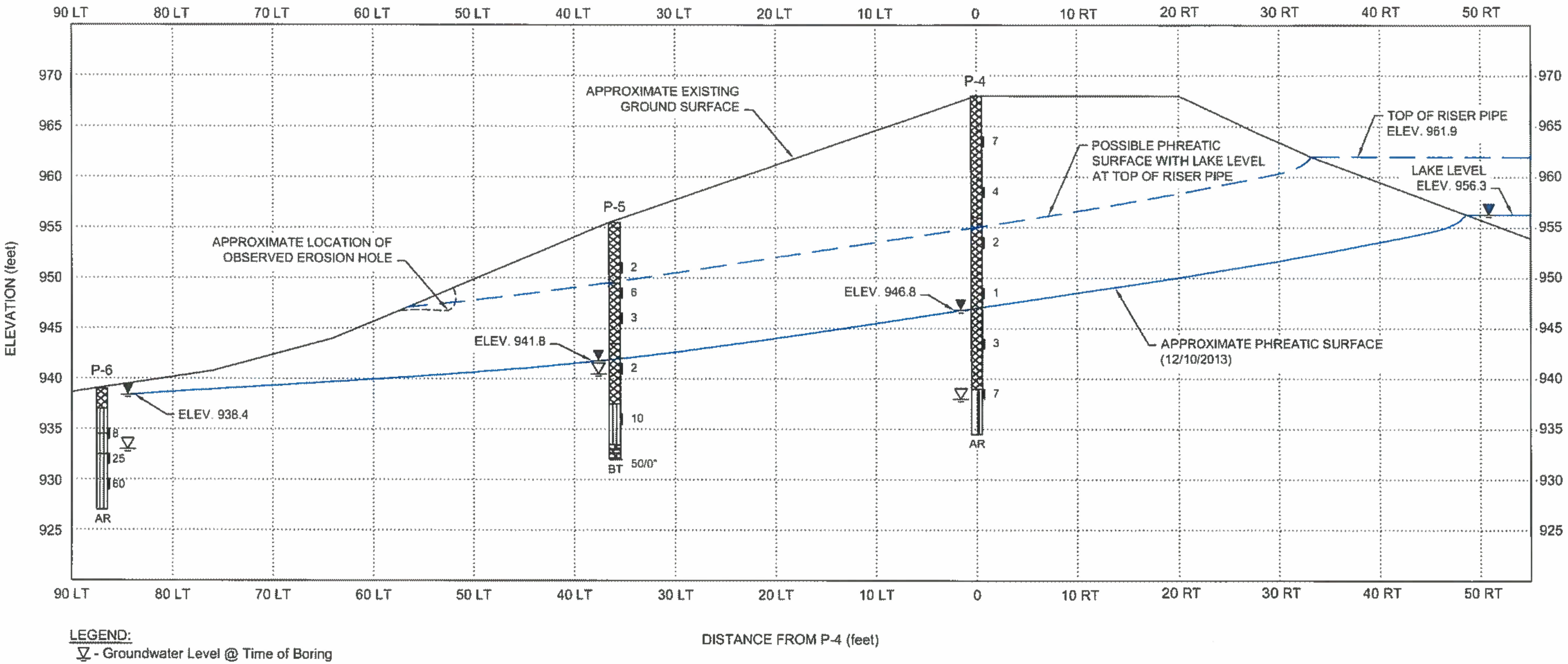
BT - Boring Terminated

SCALE: 1" = 10' DATE: 2/3/2014 DRAWN BY: ZMH REVIEWED BY: BD	WILLMER ENGINEERING INC. WE	GEOTECHNICAL ENGINEERING ¤ CONSTRUCTION SERVICES ENVIRONMENTAL SERVICES AND ENGINEERING 3772 PLEASANTDALE ROAD - SUITE 165 ATLANTA, GA 30340-4270

FIGURE 3A

GENERALIZED SUBSURFACE PROFILE A-A'
ERIN LAKE DAM REHABILITATION
DEKALB COUNTY, GEORGIA
WILLMER PROJECT No. 71.3638

P:\3638-Lake Erin-Henerdon Park Geotech Consult\CADD\Subsurface Profiles.dwg



LEGEND:
▽ - Groundwater Level @ Time of Boring
▼ - Groundwater Level on 12/10/2013
BT - Boring Terminated
AR - Auger Refusal

SCALE: 1" = 10'
DATE: 2/6/2014
DRAWN BY: ZMH
REVIEWED BY: BD

WILLMER ENGINEERING INC.
WE

GEOTECHNICAL ENGINEERING ▫ CONSTRUCTION SERVICES
ENVIRONMENTAL SERVICES AND ENGINEERING
3772 PLEASANTDALE ROAD - SUITE 165
ATLANTA, GA 30340-4270

FIGURE 3B

GENERALIZED SUBSURFACE PROFILE B-B'
ERIN LAKE DAM REHABILITATION
DEKALB COUNTY, GEORGIA
WILLMER PROJECT No. 71.3638

P:\3638-Lake Erin-Henderdon Park Geotech Consult\CADD\Subsurface Profiles.dwg

APPENDIX II

WILLMER ENGINEERING INC. WE

PIEZOMETER INSTALLATION RECORD P-1

Project Name: Erin Lake Dam Rehabilitation

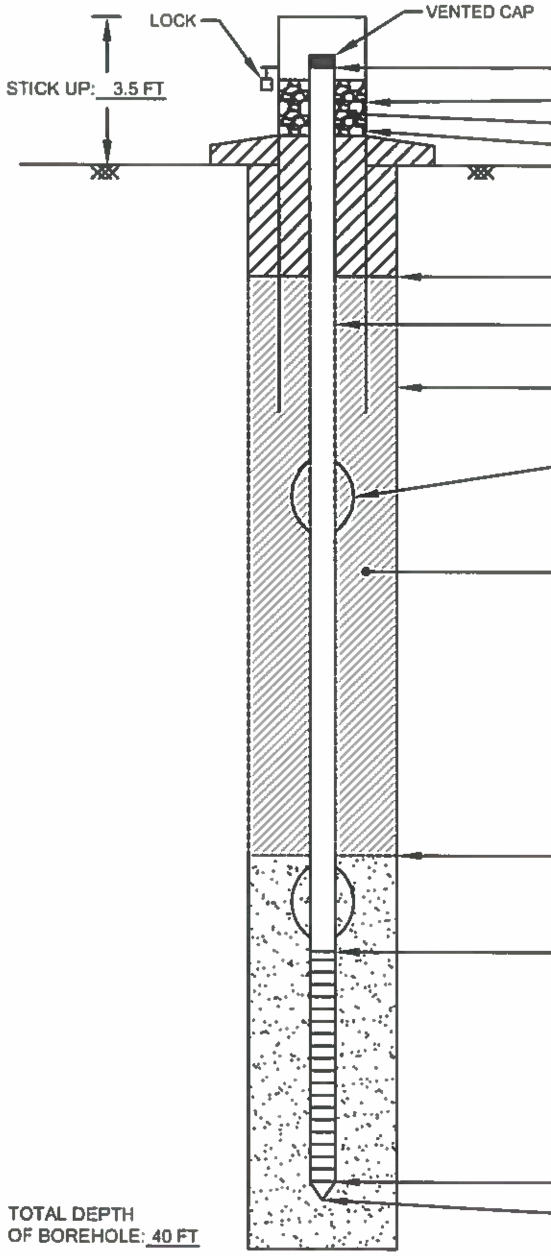
Project Number: 71.3638

Client: Kimley-Horn & Associates, Inc.

Location: DeKalb County, GA

Completion Date: 12/04/2013

Drilling Method: 6-1/4" Hollow Stem Auger



TOP OF RISER PIPE ELEVATION: 971.5 FT

PROTECTIVE CASING ([✓ yes]/no): STEEL - 4" x 4" x 5'

PEA GRAVEL

¼" WEEP HOLE

GROUND SURFACE ELEVATION: 968.5 FT

CONCRETE SEAL DEPTH: 1 FT BGS

DIAMETER OF RISER PIPE: 1 IN

DIAMETER OF BOREHOLE: 6 IN

CENTRALIZER (yes / [✓ no]) - TYPE: NO - SEE NOTES

TYPE OF BENTONITE SEAL: MEDIUM CHIPS

TOP OF SAND DEPTH: 27.5 FT BGS

TOP OF SCREEN DEPTH: 29.5 FT BGS

TYPE OF SCREEN: 1" x 10' SCHEDULE 40 PVC

SCREEN SLOT SIZE: 0.01 IN

SIZE OF SAND PACK: WASHED #2 FILTER PACK

BOTTOM OF SCREEN DEPTH: 39.5 FT BGS

BOTTOM OF PIEZOMETER DEPTH: 39.6 FT BGS

TOTAL DEPTH OF BOREHOLE: 40 FT

BGS = BELOW GROUND SURFACE

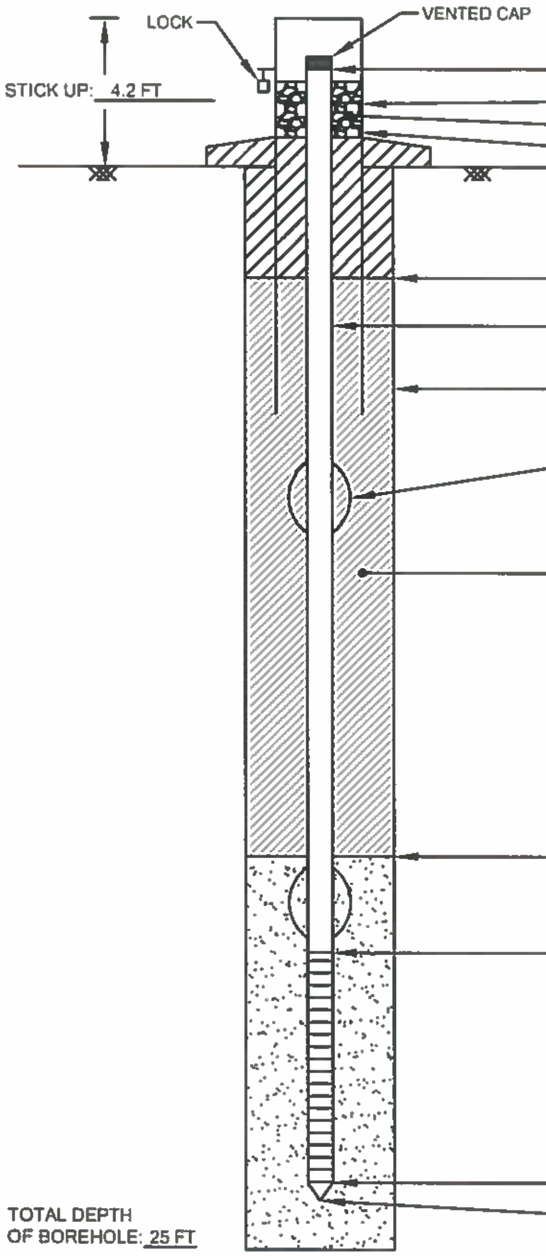
ADDITIONAL NOTES: PIEZOMETER INSTALLED THROUGH HOLLOW STEM AUGERS.

P:\3638-Lake Erin-Henerdon Park Geotech Consult\CADD\Piezometer Installation Records.dwg

WILLMER ENGINEERING INC. WE

PIEZOMETER INSTALLATION RECORD P-2

Project Name: Erin Lake Dam Rehabilitation	**Project Number:** 71.3638
Client: Kimley-Horn & Associates, Inc.	**Location:** DeKalb County, GA
Completion Date: 12/05/2013	**Drilling Method:** 6-1/4" Hollow Stem Auger



STICK UP: 4.2 FT

TOP OF RISER PIPE ELEVATION: 962.9 FT

PROTECTIVE CASING ([✓ yes]/no): STEEL - 4" x 4" x 5'

PEA GRAVEL

¼" WEEP HOLE

GROUND SURFACE ELEVATION: 959.0 FT

CONCRETE SEAL DEPTH: 1 FT BGS

DIAMETER OF RISER PIPE: 1 IN

DIAMETER OF BOREHOLE: 6 IN

CENTRALIZER (yes / [✓ no]) - TYPE: NO - SEE NOTES

TYPE OF BENTONITE SEAL: MEDIUM CHIPS

TOP OF SAND DEPTH: 17.5 FT BGS

TOP OF SCREEN DEPTH: 19.5 FT BGS

TYPE OF SCREEN: 1" x 5' SCHEDULE 40 PVC

SCREEN SLOT SIZE: 0.01 IN

SIZE OF SAND PACK: WASHED #2 FILTER PACK

BOTTOM OF SCREEN DEPTH: 24.5 FT BGS

BOTTOM OF PIEZOMETER DEPTH: 24.6 FT BGS

TOTAL DEPTH OF BOREHOLE: 25 FT

BGS = BELOW GROUND SURFACE

ADDITIONAL NOTES: PIEZOMETER INSTALLED THROUGH HOLLOW STEM AUGERS.

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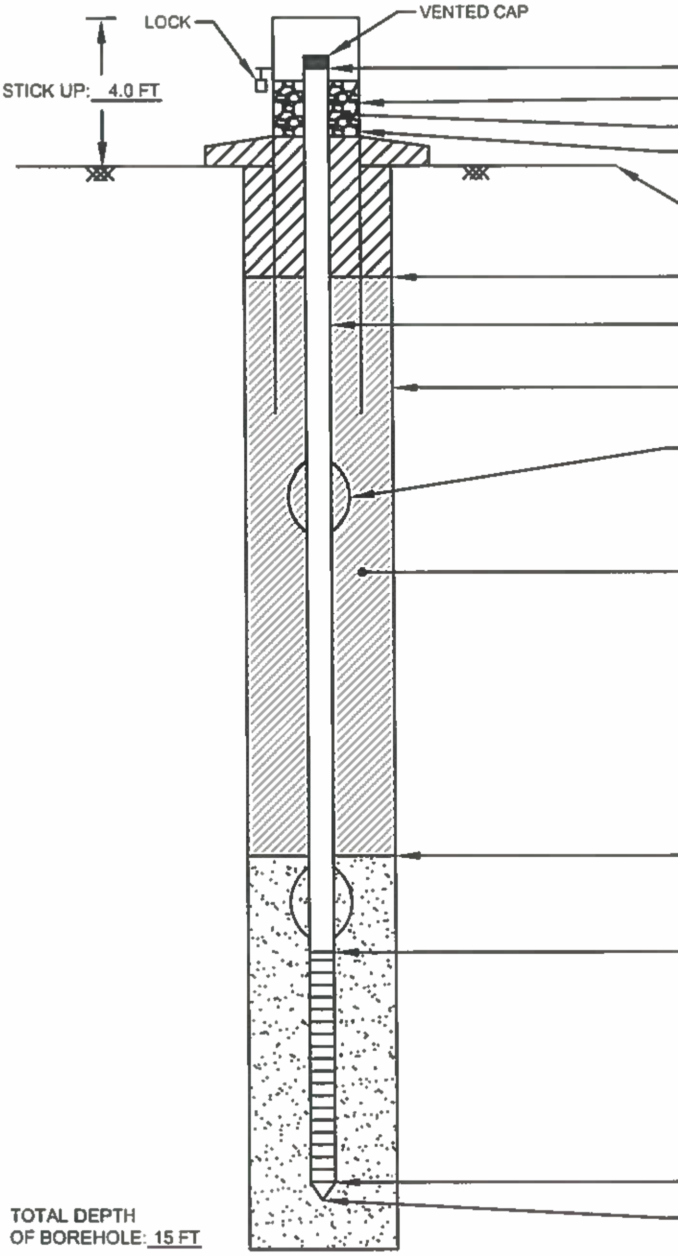
WILLMER ENGINEERING INC. WE

PIEZOMETER INSTALLATION RECORD P-3

Project Name: Erin Lake Dam Rehabilitation	**Project Number:** 71.3638
Client: Kimley-Horn & Associates, Inc.	**Location:** DeKalb County, GA
Completion Date: 12/05/2013	**Drilling Method:** 6-1/4" Hollow Stem Auger



STICK UP: 4.0 FT

TOP OF RISER PIPE ELEVATION: 951.3 FT

PROTECTIVE CASING ([✓ yes]/no): STEEL - 4" x 4" x 5'

PEA GRAVEL

¼" WEEP HOLE

GROUND SURFACE ELEVATION: 947.5 FT

CONCRETE SEAL DEPTH: 1 FT BGS

DIAMETER OF RISER PIPE: 1 IN

DIAMETER OF BOREHOLE: 6 IN

CENTRALIZER (yes / [✓ no]) - TYPE: NO - SEE NOTES

TYPE OF BENTONITE SEAL: MEDIUM CHIPS

TOP OF SAND DEPTH: 7.5 FT BGS

TOP OF SCREEN DEPTH: 9.5 FT BGS

TYPE OF SCREEN: 1" x 5' SCHEDULE 40 PVC

SCREEN SLOT SIZE: 0.01 IN

SIZE OF SAND PACK: WASHED #2 FILTER PACK

BOTTOM OF SCREEN DEPTH: 14.5 FT BGS

BOTTOM OF PIEZOMETER DEPTH: 14.6 FT BGS

TOTAL DEPTH OF BOREHOLE: 15 FT

BGS = BELOW GROUND SURFACE

ADDITIONAL NOTES: PIEZOMETER INSTALLED THROUGH HOLLOW STEM AUGERS.

P:\3638-Lake Erin-Henerdon Park Geotech Consult\CADD\Piezometer Installation Records.dwg

WILLMER ENGINEERING INC. WE

PIEZOMETER INSTALLATION RECORD P-5

Project Name: Erin Lake Dam Rehabilitation

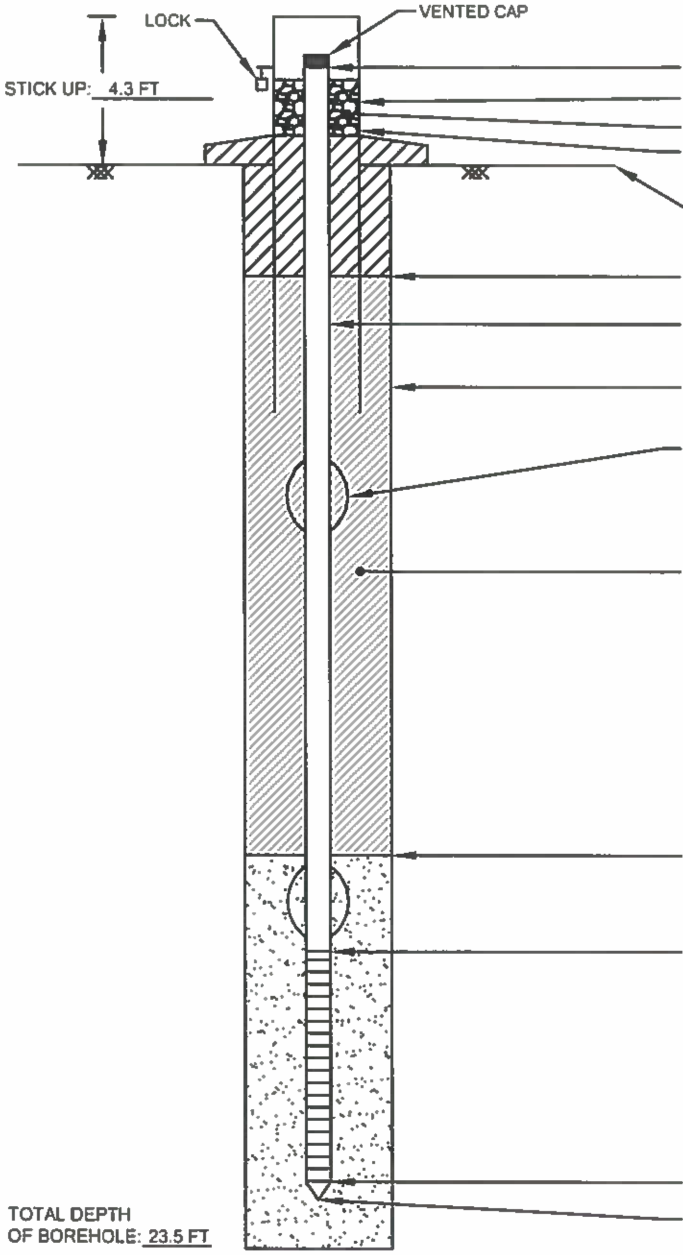
Project Number: 71.3638

Client: Kimley-Horn & Associates, Inc.

Location: DeKalb County, GA

Completion Date: 12/05/2013

Drilling Method: 6-1/4" Hollow Stem Auger



STICK UP: 4.3 FT

TOP OF RISER PIPE ELEVATION: 960.1 FT

PROTECTIVE CASING ([✓ yes]/no): STEEL - 4" x 4" x 5'

GROUND SURFACE ELEVATION: 956.0 FT

CONCRETE SEAL DEPTH: 1 FT BGS

DIAMETER OF RISER PIPE: 1 IN

DIAMETER OF BOREHOLE: 6 IN

CENTRALIZER (yes / [✓ no]) - TYPE: NO - SEE NOTES

TYPE OF BENTONITE SEAL: MEDIUM CHIPS

TOP OF SAND DEPTH: 16.0 FT BGS

TOP OF SCREEN DEPTH: 18.0 FT BGS

TYPE OF SCREEN: 1" x 5' SCHEDULE 40 PVC

SCREEN SLOT SIZE: 0.01 IN

SIZE OF SAND PACK: WASHED #2 FILTER PACK

BOTTOM OF SCREEN DEPTH: 23.0 FT BGS

BOTTOM OF PIEZOMETER DEPTH: 23.1 FT BGS

TOTAL DEPTH OF BOREHOLE: 23.5 FT

BGS = BELOW GROUND SURFACE

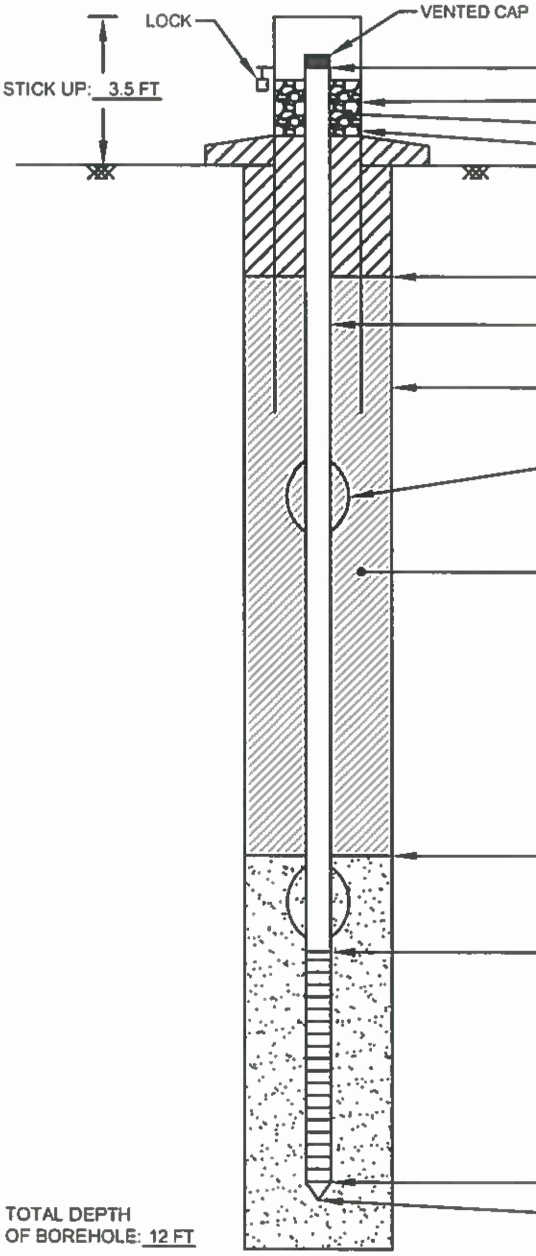
ADDITIONAL NOTES: PIEZOMETER INSTALLED THROUGH HOLLOW STEM AUGERS.

P:\13638-Lake Erin-Henderson Park Geotech Consult\CADD\Piezometer Installation Records.dwg

WILLMER ENGINEERING INC. WE

PIEZOMETER INSTALLATION RECORD P-6

Project Name: Erin Lake Dam Rehabilitation	**Project Number:** 71.3638
Client: Kimley-Horn & Associates, Inc.	**Location:** DeKalb County, GA
Completion Date: 12/05/2013	**Drilling Method:** 6-1/4" Hollow Stem Auger



STICK UP: 3.5 FT

TOP OF RISER PIPE ELEVATION: 942.6 FT

PROTECTIVE CASING ([✓ yes]/no): STEEL - 4" x 4" x 5'

PEA GRAVEL

¼" WEEP HOLE

GROUND SURFACE ELEVATION: 939.5 FT

CONCRETE SEAL DEPTH: 1 FT BGS

DIAMETER OF RISER PIPE: 1 IN

DIAMETER OF BOREHOLE: 6 IN

CENTRALIZER (yes /[✓ no]) - TYPE: NO - SEE NOTES

TYPE OF BENTONITE SEAL: MEDIUM CHIPS

TOP OF SAND DEPTH: 5.0 FT BGS

TOP OF SCREEN DEPTH: 6.5 FT BGS

TYPE OF SCREEN: 1" x 5' SCHEDULE 40 PVC

SCREEN SLOT SIZE: 0.01 IN

SIZE OF SAND PACK: WASHED #2 FILTER PACK

BOTTOM OF SCREEN DEPTH: 11.5 FT BGS

BOTTOM OF PIEZOMETER DEPTH: 11.6 FT BGS

TOTAL DEPTH OF BOREHOLE: 12 FT

BGS = BELOW GROUND SURFACE

ADDITIONAL NOTES: PIEZOMETER INSTALLED THROUGH HOLLOW STEM AUGERS.

P:\3638-Lake Erin-Henerdon Park Geotech Consult\CADD\Piezometer Installation Records.dwg

APPENDIX III

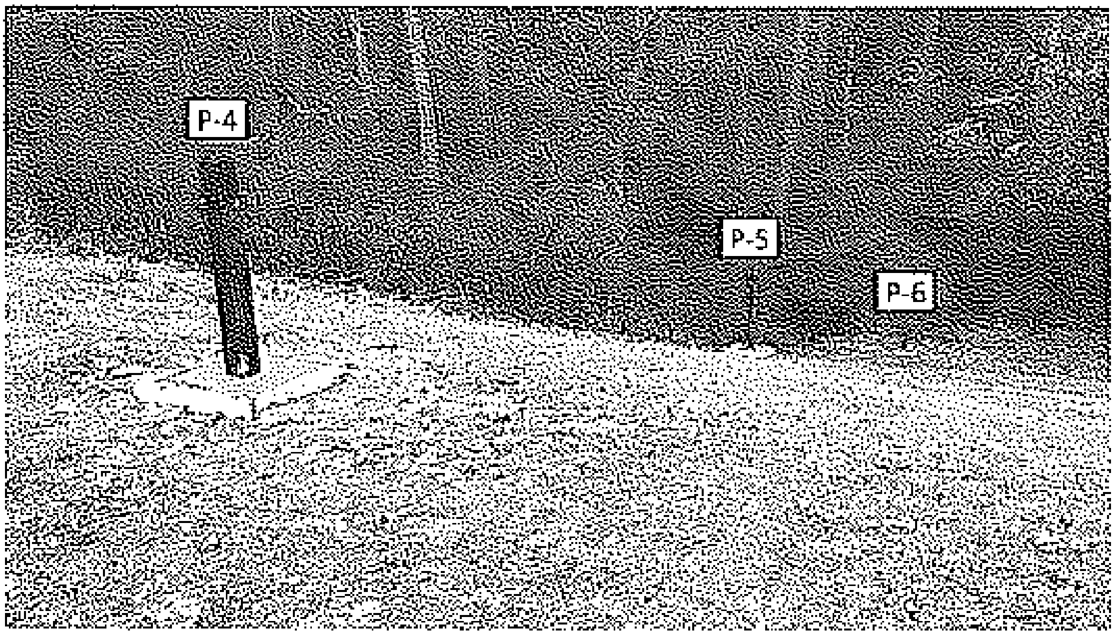
Western slough area and erosion hole; photographed on 3/14/2009

Eastern slough area; photographed on 3/14/2009

Erosion hole; photographed on 10/28/2013

Principal spillway inlet and riser pipe; photographed on 10/28/2013

Piezometers P-1 through P-3; photographed on 12/6/2013



Piezometers P-4 through P-6; photographed on 12/6/2013

Toe drain outlet (left) and possible abandoned drain (right); photographed on 1/10/2014

Ray, Kristin

From: Bradford Drew <bdrew@willmerengineering.com>
Sent: Friday, January 03, 2014 4:14 PM
To: Kilby, Mark
Cc: Ray, Kristin; Sujit Bhowmik
Subject: RE: Erin Lake Dam - Results of Borrow Study

Mark,

We have completed three hand auger borings (HA-1 through HA-3) to a depth of 3 feet in the sloughed areas of the dam and collected soil samples for classification. The soil type encountered in the borings are similar; therefore, only one sample was used for laboratory testing. A summary of the laboratory test results and soil descriptions are presented in the table below.

Sample No.	Sample Location	Sample Depth (feet)	Soil Description	Natural Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	Fines Content (%)	Standard Proctor Compaction Test	
								Maximum Dry Density (pcf)	Optimum Moisture Content (%)
TP-4	Proposed borrow pit	2' – 5'	Reddish brown medium to fine sandy Elastic SILT	25.7	52	11	68.6	94.5	25.1
HA-1	Slough area of existing dam	0 – 3'	Brown silty medium to fine SAND (slightly micaceous)	25.2	47	12	32.4	--	--

As we discussed earlier in August, the soil obtained from the proposed borrow pit classifies as a sandy elastic silt with a liquid limit of 52% and a fines content of 68.6%. The liquid limit is higher than what would be preferred for the slope repair work. Also, if this borrow material gets wet, it will be very difficult to dry it because of its low permeability (high fines content and liquid limit).

The soil sample obtained from the sloughed areas on the face of the dam classifies as a silty sand with a fines content of 32.4%. The fines content for this soil is significantly lower than that of the borrow pit soil (fines content = 68.6%); hence, the permeability is likely much higher than that of the borrow pit soil.

It would be undesirable to use the borrow pit soil as the outer layer of the dam because of its lower permeability relative to the existing dam material. In the event that the phreatic surface through the dam rises, groundwater seepage may get blocked by the outer, less permeable material and eventually cause a 'blow out' characterized by surficial slope failures. Therefore, we recommend that an alternative borrow source be evaluated for suitability as fill soil for the repair work. Once a new borrow source is selected, we can collect representative samples and perform laboratory tests to determine the suitability of the soil.

We have also completed the installation of six piezometers along the downstream face of the dam and have taken initial groundwater measurements. We are in the process of preparing a letter report for our work performed thus far and can have a PDF emailed to you next week.

Please contact us with any questions.

Thanks,

Bradford Drew, EIT
Staff Engineer

Willmer Engineering Inc.
3772 Pleasantdale Road, Suite 165
Atlanta, Georgia 30340
Phone: 770.939.0089 Ext.22
Cell: 770.296.7253
www.willmerengineering.com

From: Sujit Bhowmik
Sent: Friday, August 23, 2013 4:41 PM
To: 'Mark.Kilby@kimley-horn.com'
Cc: Bradford Drew; Jim Willmer
Subject: Erin Lake Dam - Results of Borrow Study

Mark,

We have completed our laboratory tests on the borrow soil. The tested soil sample was obtained from Test Pit No. 4 (TP-4), and the location of the test pit is shown in the attached Figure 1. The laboratory tests performed on the soil sample consisted of grain size distribution analysis, Atterberg limits, and standard proctor compaction tests. Results of these tests are presented in the attached individual test sheets, and a summary of the results is presented below:

Test Pit Number	Sample Depth (ft)	Soil Description	Percent Passing No. 200 Sieve (%)	Natural Moisture Content (%)	Atterberg Limits		Standard Proctor Compaction Test Results	
					Liquid Limit (%)	Plasticity Index (%)	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
TP-4	2 - 5	Reddish brown medium to fine sandy **ELASTIC SILT**	68.6	25.7	52	11	94.5	25.1

As shown above, the soil classifies as a sandy elastic silt with a liquid limit of 52%. This liquid limit is higher than what would be preferred for the slope repair work. The natural moisture content of the soil is 25.7 percent, which is close to the optimum moisture content (25.1%); thus, significant wetting/drying will not be required to achieve the optimum moisture content during construction. However, if this borrow material gets wet, it will be very difficult to dry it because of its low permeability (high fines content and liquid limit).

I will call you to discuss these borrow soil properties further.

Thanks,

Sujit

Sujit K. Bhowmik, PhD, PE
Chief Engineer

Willmer Engineering Inc.
3772 Pleasantdale Road, Suite 165
Atlanta, Georgia 30340
Phone 770-939-0089 ext. 30
Fax 770-939-4299
www.willmerengineering.com

Table 1

Summary of Piezometer Data
Erin Lake Dam Rehabilitation
DeKalb County, Georgia
Willmer Project No. 71.3638

Piezometer No.	Ground Surface Elevation (feet)	Measured Groundwater Elevation (feet)				
		At time of Installation (12/4 and 12/5)	12/10/2013	01/10/2014	02/10/2014	03/07/2014
P-1	968.5	940.0	949.6	949.8	949.7	949.5
P-2	959.0	942.0	945.6	945.9	945.9	945.9
P-3	947.5	937.5	942.4	942.5	942.5	942.5
P-4	968.5	938.5	946.8	947.0	946.8	946.7
P-5	956.0	941.0	941.8	941.7	941.7	941.6
P-6	939.5	933.5	938.4	938.4	938.4	938.4
Lake Level Elevation (feet)			956.3	956.4	956.5	956.5
Approximate Toe Drain Discharge (gal/min)			Trickle	Trickle	Trickle	Trickle
Recorded Rainfall for Previous Month (inches)			1.87 (Nov)	7.80 (Dec)	3.35 (Jan)	3.81 (Feb)

Appendix B

SDP March 25, 2010 Inspection Letter

SDP January 20, 2010 Visual Inspection

SDP May 25, 2012 Review Comments Letter

Georgia Department of Natural Resources
Environmental Protection Division
Safe Dams Program
4244 International Parkway, Suite 110
Atlanta, Georgia 30354
Chris Clark, Commissioner
F. Allen Barnes, Director
(404) 362-2678

7/392-1503

March 25, 2010

Mr. Steve Wesson
Dekalb County Parks & Recreation
4770 North Peachtree Road
Atlanta, Georgia 30338

Subject: Erin's Lake Dam
DeKalb County

Dear Mr. Wesson:

On January 20, 2010, our staff conducted the FY10 annual inspection of this dam. Mr. George McGrade and Mr. Mark Dalrymple accompanied the staff on the inspection. I apologize for the delay in the follow up letter. The following items were noted during the inspection.

1. The vegetation on the dam had been cut. Vegetation on the upstream slope needs to be replaced by low growing grass and maintained. There are animal holes on the upstream slope. These holes need to be filled and compacted.
2. There are some sparse areas of vegetation on the dam. These areas need to be seeded with a low growing grass.
3. There is a potential slough on the downstream slope, at the midsection. You will need to monitor this area for any changes. An Engineer of Record (EOR) should evaluate the slough and recommend repairs.
4. There is a hole along the right end of the downstream channel of the emergency spillway. This should be evaluated by the EOR and repaired.
5. There is a wet area on the downstream slope, near the toe and emergency spillway channel. This area needs to be monitored for any changes.
6. The toe drain channel needs to be cleaned out to allow the channel to drain.

Please inform the Safe Dams Program in writing before **May 27, 2010** of the name of the engineer you have selected to perform the detailed investigation. The engineering report is due no later than **July 30, 2010.**

A copy of the inspection report and photos is enclosed for your records. If there are any questions, please contact our office at (404) 362-2678.

Sincerely,

Sree Madichetty
Environmental Engineer III
Permitting and Compliance Unit
Safe Dams Program

SM:ks
Enclosure
cc: Mr. George McGrade
Mr. Mark Dalrymple

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Steve, Our Engineer of Record
is Mark Kilby with
Kimley-Horn & Assoc.
off 678-533-3905
cell 404-291-7009
Mark 3/30/10

SUBSEQUENT VISUAL INSPECTION

Date of Inspection 1/20/2010

Name of Dam: Erin Lake Dam County: Dekalb

ID#: 044-004-00033 Weather: Sunny and cool

Inspected by: Sree Madichetty and Tom Woosley

Type of Inspection: X Annual ____ Construction ____ Other

Persons Contacted: No One

Toe Drains Right: flowing could not measure

Toe Drains Left: N/A

Additional Drains: N/A

Type of Principal Pipe:

Needs to be Camera Inspected? ____ Maybe ____ Yes X No

Low-level drain is being tested? ____ Not Sure ____ Yes ____ No

Comments:

Number of Pictures Taken: 6

Remarks:

1. The vegetation on the dam had been cut. Vegetation on the upstream slope needs to be replaced by low growing grass and maintained. There are animal holes on the upstream slope. These holes need to be filled and compacted.
2. There are some sparse areas of vegetation on the dam. These areas need to be seeded with a low growing grass.
3. There is a potential slough on the downstream slope, at the midsection. You will need to monitor this area for any changes. An engineer of record should evaluate the slough and slope needs to be repaired.
4. There is a hole along the right end of the downstream channel of the emergency spillway. This should be evaluated by the Engineer of Record and repaired.
5. There is a wet area on the downstream slope, near the toe and emergency spillway channel. This area needs to be monitored for any changes.
6. The toe drain channel needs to be cleaned out to allow the channel to drain.

Erin Lake Dam, Dekalb County
Pictures Taken on January 20, 2010


Upstream slope


Emergency spillway channel


Inlet to the principal spillway pipe


Hole at the right wing-wall of emergency spillway


Left abutment


Slough on the downstream slope (mid-section)

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Georgia Department of Natural Resources
Environmental Protection Division
Safe Dams Program
4244 International Parkway
Suite 110, Atlanta, GA 30354
Mark Williams, Commissioner
Judson H. Turner, Director
404/362-2678

May 25, 2012

Mr. J. Mark Kilby, P.E.
Kimley-Horn and Associates, Inc.
3169 Holcomb Bridge Road
Suite 600
Norcross, Georgia 30071-1367

SUBJECT: Erin Lake Dam
Dekalb County

Dear Mr. Kilby:

The Safe Dams Program has completed its review of the Report prepared by you and Wilmer Engineering, dated July 27, 2010. The following comments relate to our review of the file and your report.

1. The proposed remediation for the two sloughs on the downstream is to excavate the low quality material and replace with competent, well-compacted soil.
    a. Do you know the extent of poor soil in these areas?
    b. Was the material in and around the sloughs evaluated? If so, where is the data?
    c. Where would the borrow soil be obtained for this replacement method?
    d. As noted in the report, portions of metro Atlanta had a significant rainfall event in September 2009. A couple of dams experienced sloughs on their downstream slope. The most notable dam is the Berkeley Lake Dam. A full geotechnical evaluation was performed at this dam and revealed soft soils within the embankment and a compromised internal drain system. An evaluation of the dam (Erin Lake Dam) should be done to verify the internal condition of the dam.
2. The report mentions installing survey monuments on the dam. Information on locations of the monuments and type of monuments should be submitted.
3. Wetness has been observed at the toe of the dam for many years. A review of the Safe Dams Program files notes that Craig Robinson, P.E., with Piedmont Geotechnical Consultants, Inc was retained in 2004 to investigate the wetness. Mr. Robinson's report recommended the installation of observation wells to measure the phreatic surface within the dam. Mr. Robinson's recommendation was never implemented.
4. Review of past inspection reports; showed there were sloughs and holes observed on the upstream slope. This office recommends further investigation of the upstream slope as part of this study. Upstream slope sloughs will affect the overall stability of the dam.
5. The Wilmer Engineering report recommended the installation of a piezometer in the dam. No information was given on the proposed location. Additionally, one well is not sufficient to determine the phreatic surface. Additionally wells should be installed.

It is possible the wetness at the toe and occurrence of the sloughs are interrelated. Therefore, this office believes additional investigation is warranted. Because the investigation, analysis and design of any remediation could take over a year this office accepts the proposed replacement of soil as an interim measure. Additionally, at least two sets of piezometers should be installed in cross-section of the dam. At a minimum, there should be piezometers installed at the crest, downstream slope and toe at each cross-section. Additional wells may be warranted. Data from the piezometer and drain outlets should then be collected to aid in determining the phreatic surface and overall stability of this dam.

This office recommends a predesign meeting be conducted to discuss the proposed work and monitoring as well as potential timelines.

If there are any questions, please feel free to contact this office at (404) 362-2678.

Sincerely,

Sree Madichetty
Environmental Engineer
Safe Dams Program

SM:ks
Enclosure
CC: Mr. Carl Glover, Dekalb County Public Works Department
Mr. Mark Dalrymple, Dekalb County Roads and Drainage Department

Appendix C

Kimley-Horn August 26, 2014 Letter Response to SDP Comments

Kimley»Horn

August 26, 2014

Mr. Tom Woosley, P.E.
Georgia Environmental Protection Division
Safe Dams Program
200 Piedmont Avenue, SW
Suite 418
Atlanta, GA 30334

RE: ***Erin Lake Dam, DeKalb County***
Status Update and Response to Comments

Dear Mr. Woosley:

Kimley-Horn and Associates, Inc. (Kimley-Horn) is pleased to submit this letter regarding the Erin Lake Dam to the Georgia Department of Natural Resources, Environmental Protection Division, Safe Dams Program (SDP). The purpose of this letter is to give an update of the project status, to transmit resubmittal of the Engineering Report, and to address SDP comments regarding the Engineering Report. A brief project history is shown below:

January 20, 2010
SDP Visual Inspection with six (6) remarks

March 25, 2010
SDP Visual Inspection follow-up letter to DeKalb County Parks and Recreation (DeKalb County) noting the six (6) remarks

July 27, 2010
Kimley-Horn Engineering Report submitted to the SDP with remediation recommendations. Submittal included Willmer Engineering, Inc. (Willmer) Report of Visual Dam Evaluation dated July 19, 2010

May 25, 2012
SDP Review Comments with five (5) comments

August 4, 2014
SDP Letter to Kimley-Horn requesting a progress update

August 26, 2014
Kimley-Horn Engineering Report resubmitted to the SDP (attached to this letter). Submittal includes Willmer's Report of Piezometer Installation and Initial Monitoring dated February 7, 2014

The March 2010 SDP Visual Inspection letter outlined six items that are addressed by the Engineering Report; primarily, slough areas along the downstream slope and wetness at the toe of the dam. In order to respond to the May 2012 SDP Review Comments, the Engineering Report has been updated and is being submitted with this response letter.

Also to address the SDP comments, Willmer has installed piezometers, performed geotechnical exploration at the locations of the piezometers, and made further geotechnical recommendations. The evaluation results are included in Willmer's Report of Piezometer Installation and Initial Monitoring and also summarized in the Engineering Report; the results indicate that there should be further investigation in order to provide final recommendations for repair. This includes additional field sampling and laboratory permeability and triaxial tests to determine phreatic lines and to perform seepage and slope stability analyses. We anticipate incorporating the additional geotechnical recommendations into the Engineering Report and Construction Documents and submitting the documents to the SDP for review.

Although the Engineering Report and Construction Documents are pending additional recommendations, they have been updated and are being submitted with this cover letter. Please accept this letter in response to the SDP comments dated May 25, 2012, relating to the SDP review of the Engineering Report prepared by Kimley-Horn and Willmer dated July 27, 2010. The SDP comments are listed below with our response to each.

1. The proposed remediation for the two sloughs on the downstream is to excavate the low quality material and replace with competent, well-compacted areas.

    a. Do you know the extent of poor soil in these areas?

       The approximate limits of the slough areas are shown on sheet C2-00 of the Construction Documents which are being submitted as an attachment to the Engineering Report. The exact limits of slough remediation area will be determined by the project geotechnical engineer during construction.

    b. Was the material in and around the sloughs evaluated? If so, where is the data?

       Willmer performed 3 hand auger borings to a depth of 3 feet in the sloughed areas of the dam and collected soil samples for classification. See the Willmer Report in Appendix A of the Engineering Report for the results.

    c. Where would the borrow soil be obtained for this replacement method?

       A potential borrow site within the park was evaluated by Willmer but was not found to contain suitable soils therefore an alternate offsite borrow source will be used for the remediation. Once a new borrow source is selected, the borrow soils will be required to be approved by the project geotechnical engineer prior to placement. See the Willmer Report in Appendix A of the Engineering Report for the results of the unsuitable onsite borrow soil analysis and also for recommendations for potential borrow soil properties.

    d. As noted in the report, portions of metro Atlanta had a significant rainfall event in September 2009. A couple of dams experienced sloughs on their downstream slope. The most notable dam is Berkeley Lake Dam. A full geotechnical evaluation was performed at this dam and revealed soft soils within the embankment and a compromised internal drain system. An evaluation of the dam (Erin Lake Dam) should be done to verify the internal condition of the dam.

Willmer performed 6 standard penetration test borings and installed 6 piezometers along the top and downstream face of the dam. Willmer took initial groundwater measurements and will be monitoring the piezometer readings in order to prepare a Report of Geotechnical Investigation after the first year of monitoring. See the Willmer Report included in Appendix A of the Engineering Report for more information.

2. The report mentions installing survey monuments on the dam. Information on locations of the monuments and type of monuments should be submitted.

   Eight 6" diameter concrete monuments at 36" of depth will be constructed within the slough areas during the repairs. After the repair is made, these will be used to facilitate long term monitoring. See sheet C2-00 of the Construction Documents for locations.

3. Wetness has been observed at the toe of the dam for many years. A review of the Safe Dams Program files notes that Craig Robinson, P.E., with Piedmont Geotechnical Consultants, Inc. was retained in 2004 to investigate the wetness. Mr. Robinson's report recommended the installation of observation wells to measure the phreatic surface within the dam. Mr. Robinson's recommendation was never implemented.

   Six observation wells (piezometers) have recently been installed by Willmer in two cross sections to measure the phreatic surface within the dam. Each cross section contains one piezometer located along the top of dam, one in the downstream slope, and one along the toe of the dam embankment. The piezometers will be monitored by Willmer and the readings provided to the SDP. Willmer took initial groundwater readings which are included in the Willmer Report in Appendix A of the Engineering Report. Also, see sheet C2-00 of the Construction Documents for location of wells W1-W6, ranging in depths of 40' to 15'.

4. Review of past inspection reports showed there were sloughs and holes observed on the upstream slope. This office recommends further investigation of the upstream slope as part of this study. Upstream slope sloughs will affect the overall stability of the dam.

   No upstream sloughs or holes were observed during Kimley-Horn site visits. A slope stability analysis is included in the work recommended by Willmer.

5. The Willmer Engineering report recommended the installation of a piezometer in the dam. No information was given on the proposed location. Additionally, one well is not sufficient to determine the phreatic surface. Additionally wells should be installed.

   Six observation wells (piezometers) have recently been installed by Willmer along the top of dam and the downstream embankment to measure the phreatic surface within the dam. Willmer took initial groundwater readings which are included in the Willmer Report in Appendix A of the Engineering Report. Also, see sheet C2-00 of the Construction Documents for location of wells W1-W6.

As indicated in Willmer's Report of Piezometer Installation and Initial Monitoring, further geotechnical investigation is recommended prior to a final recommendation for repair. If you have any questions related to the above responses, please contact me at (678) 533-3932 or kristin.ray@kimley-horn.com.

Very truly yours,

KIMLEY-HORN AND ASSOCIATES, INC.

J. Mark Kilby

By: J. Mark Kilby, P.E.
Engineer of Record

Kristin Ray

Kristin J. Ray, P.E., CFM
Project Manager

Enclosures

cc: David Pelton
Paige Singer

Kimley»Horn

August 7, 2015

Ms. Kate Betsill
Georgia Environmental Protection Division
Safe Dams Program
200 Piedmont Avenue, SW
Suite 418
Atlanta, GA 30334

RE: ***Erin Lake Dam, DeKalb County***
EOR and Status Update

Dear Ms. Betsill:

I am writing this letter in response to your July 9, 2015, letter addressed to Paige Singer with DeKalb County. The purpose of this letter is to update of the project's Engineer of Record, to transmit recent piezometer readings, and to request a statement from the Safe Dams Program regarding our recommended path forward.

Kimley-Horn has been involved with this repair project since 2010 with Mark Kilby acting as EOR at the onset and then myself, Kristin Ray, taking over as EOR upon Mark's retirement in 2014. Sujit Bhowmik with Willmer Engineering has been involved as the project's geotechnical engineer since 2010. Both Kristin and Sujit intend to remain involved through slough repair completion.

A letter from you dated August 4, 2014, required "a full geotechnical evaluation" be performed. Willmer has previously performed some geotechnical analysis, installed six (6) piezometers, monitored the piezometers, and prepared a report of findings and recommendations dated February 7, 2014. In that report, Willmer recommended that additional geotechnical analysis be performed prior to repairing the sloughs. Thus, Willmer provided DeKalb County with a proposal to provide additional services. I have attached the associated scope of services. DeKalb County is requesting that the Safe Dams Program review the scope and concur that it will suffice to meet the requirement for the full geotechnical evaluation. Please respond accordingly so that the county may authorize the contract to perform the required evaluation.

Lastly, as noted in your July 9 inspection notes, some of the piezometers that were installed in December 2013 were vandalized in late summer 2014. They were subsequently repaired and Willmer has been monitoring the water levels. I have attached a current report from Willmer outlining the repairs and the readings to date. Willmer intends to continue monitoring the water levels, perform the additional services outlined in the attached scope, and then make final recommendations for repair.

If you have any questions, please contact me at (678) 533-3932 or kristin.ray@kimley-horn.com.

Very truly yours,
KIMLEY-HORN AND ASSOCIATES, INC.

By: Kristin J. Ray, P.E., CFM

Enclosures

cc: David Pelton
Paige Singer

COST ESTIMATE SUMMARY WORKSHEET

Additional Sampling, Testing, and Stability Analyses
Erin Lake Dam Slough Evaluation and Repair
Henderson Park, DeKalb County, Georgia
Willmer Project No. 71.3638
(Prepared for Kimley-Horn and Associates, Inc.)

CATEGORY	COST
Personnel	$11,250
Field & Lab	$7,380
TOTAL ESTIMATED COST	**$18,630**

Assumptions/Project Information:

1. This cost estimate is for providing additional soil sampling, laboratory testing, and geotechnical engineering analyses as part of the slough evaluation and repair project at Erin Lake Dam in Henderson Park, DeKalb County, Georgia.

2. In our report dated February 7, 2014, we recommended that additional field sampling and laboratory permeability and triaxial tests be performed for use in determination of phreatic lines and to perform seepage and slope stability analyses. The results from these analyses, along with continued water level monitoring data, will be used to confirm our assessment and provide final recommendations for slough repair.

3. The additional field sampling will consist of obtaining six undisturbed Shelby tube samples at the crest, mid-slope, and toe of the dam. The soil samples will be used for laboratory permeability and triaxial tests. Upon completion of drilling, the bore holes will be backfilled with grout.

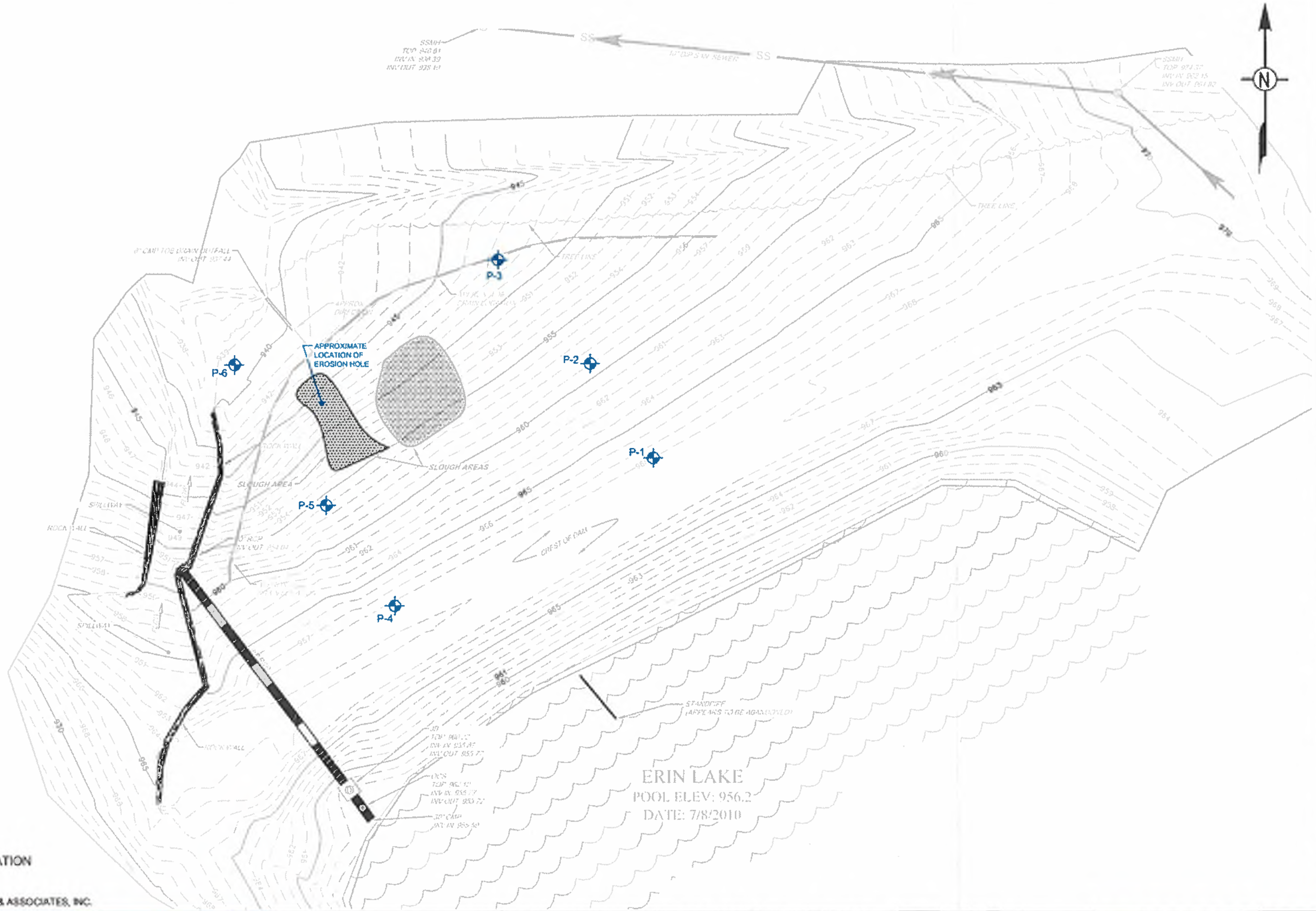
4. Willmer will perform three permeability tests and three triaxial tests on the undisturbed soil samples. As mentioned above, the soil permeability and strength parameters obtained from these tests will be used to evaluate possible high phreatic lines and to perform seepage and slope stability analyses. Classification tests (grain size analysis and Atterberg limits) will also be performed on the soil samples.

5. Upon completion of field sampling, laboratory testing, and engineering analyses, Willmer will prepare a report that includes the additional data and our recommendations for the slough repair.

6. We estimate that it will take about two weeks to coordinate drilling activities, two weeks to perform laboratory tests, and four weeks to perform the analyses and prepare a report with our recommendations.

7. This cost estimate was prepared by Bradford Drew, EIT on March 10, 2014 and reviewed by Sujit K. Bhowmik, PhD, PE and Jim Willmer, PE.

Willmer Engineering Inc.
3772 Pleasantdale Road, Suite 165, Atlanta, GA 30340-4270



LEGEND:

◆ P-1 = APPROXIMATE PIEZOMETER LOCATION

NOTE: BASE DRAWING PROVIDED BY KIMLEY-HORN & ASSOCIATES, INC.

SCALE: 1" = 30'

DATE: 7/29/2015
DRAWN BY: ZMH
REVIEWED BY: BD

WILLMER ENGINEERING INC.

WE

GEOTECHNICAL ENGINEERING ▫ CONSTRUCTION SERVICES
ENVIRONMENTAL SERVICES AND ENGINEERING
3772 PLEASANTDALE ROAD - SUITE 165
ATLANTA, GA 30340-4270

FIGURE 1

PIEZOMETER LOCATION PLAN
ERIN LAKE DAM REHABILITATION
DEKALB COUNTY, GEORGIA
WILLMER PROJECT No. 71.3638

P:\3638-Lake Erin-Henderson Park Geotech Consult\CADD\Piezometer Location Plan - No Cross Sections.dwg

WILLMER ENGINEERING INC. WE

July 31, 2015

VIA EMAIL

Kristin J. Ray, PE, CFM
Water Resources
Kimley-Horn and Associates, Inc.
817 W Peachtree St NW, Suite 601
Atlanta, Georgia 30308

SUBJECT: **Report of Piezometer Monitoring Data (December 2013 – June 2015)**
Erin Lake Dam Rehabilitation
DeKalb County, Georgia
Willmer Project No. 71.3638

Dear Ms. Ray:

Willmer Engineering Inc. (Willmer) is pleased to present this letter containing piezometer data obtained from December 2013 through June 2015 for Erin Lake Dam located at Henderson Park in DeKalb County, Georgia. This work was performed in accordance with our subcontract agreement for professional services dated July 15, 2013 with Kimley-Horn and Associates, Inc. (Kimley-Horn).

Description of Installed Piezometers

Six piezometers (P-1 through P-6) were installed by Willmer in December 2013. The locations of the piezometers are shown in the attached Figure 1. As shown, the piezometers were installed in two sets with three piezometers in each set to determine the phreatic surface across the dam. Each set included one piezometer at the crest, one on the downstream slope, and one at the downstream toe of the dam. The piezometer installation records were provided in our report dated February 7, 2014. Our scope of work for this project included taking water level readings monthly during the first three months and quarterly thereafter for a total period of three years (i.e., a total of 14 readings).

During a site visit made by Willmer on September 8, 2014 to take scheduled recordings of the water levels in the piezometers, we observed that three of the six piezometers (P-3, P-4, and P-5) had been vandalized. We assessed that the piezometers were vandalized sometime between mid-June and early September of 2014, as our previous site visit was made on June 10, 2014, and no damages to the piezometers were observed at that time.

Repairs to the damaged piezometers were made under a separate Purchase Order issued by DeKalb County on January 22, 2015, and the elevations of the repaired piezometers were surveyed on February 10, 2015 for future scheduled water level recordings. A letter summarizing the piezometer repairs is provided in Attachment A.

Piezometer Monitoring Data

The piezometer data obtained from December 2013 through June 2015 is provided in Table 1 (attached). The table provides the groundwater elevations obtained at each piezometer at the scheduled recording date, the mean and standard deviation of the groundwater elevations for each piezometer, the approximate lake level and observed toe drain discharge at the time of the piezometer reading, and the total rainfall recorded in the Atlanta area for the month preceding the piezometer reading date (obtained from NOWData – NOAA Online Weather Data). As shown in Table 1, the maximum fluctuation in the water level readings during the 18-month monitoring period was 1.2 feet, and the maximum fluctuation in lake level readings was 0.9 feet. As expected, the maximum water elevations in the piezometers corresponded to the maximum lake elevation (except for P-4 and P-5), which occurred in April 2014. No unusual fluctuation in the piezometer readings was observed during the monitoring period

Closing Remarks

We appreciate the opportunity to be of service to you on this project. Please contact us if you have any questions regarding this report.

Sincerely,

WILLMER ENGINEERING INC.

Bradford Drew
G. Bradford Drew, PE
Project Engineer

Sujit K Bhowmik
Sujit K. Bhowmik, PhD, PE
Chief Engineer/Safe Dams Engineer of Record
for Geotechnical Engineering

J. L. Willm, P.E.
James L. Willmer, PE
Executive Vice President/Principal Consultant/Safe
Dams Engineer of Record for Geotechnical
Engineering

Attachments:

Figure 1 – Piezometer Location Plan
Table 1 – Summary of Piezometer Monitoring Data (December 2013 through June 2015)
Attachment A – Report of Piezometer Repairs

Table 1

Summary of Piezometer Monitoring Data (December 2013 through June 2015)

Erin Lake Dam Rehabilitation

DeKalb County, Georgia

Willmer Project No. 71.3638

Date	Groundwater Elevation (feet)						Lake Level Elevation (feet)	Toe Drain Discharge Observation	Atlanta Area Monthly Total Rainfall	
	P-1	P-2	P-3	P-4	P-5	P-6			Inches	Month/Year
12/10/2013	949.6	945.6	942.4	946.8	941.8	938.4	956.3	Trickle	1.87	Nov 2013
1/10/2014	949.8	945.9	942.5	947.0	941.7	938.4	956.4	Trickle	7.80	Dec 2013
2/10/2014	949.7	945.9	942.5	946.8	941.7	938.4	956.5	Trickle	3.35	Jan 2014
3/7/2014	949.5	945.9	942.5	946.7	941.6	938.4	956.5	Trickle	3.81	Feb 2014
4/8/2014	950.1	946.4	943.1	947.1	941.9	938.6	956.5	Trickle	3.12	Mar 2014
6/10/2014	949.3	945.5	942.1	946.5	941.4	938.2	956.0	Trickle	2.29	May 2014
9/8/2014	949.2	945.2	*	*	*	938.2	956.0	Trickle	5.80	Aug 2014
12/10/2014	949.2	945.4	*	*	*	938.3	955.6	Trickle	3.85	Nov 2014
3/10/2015	949.6	946.1	**	947.4	942.0	938.4	956.0	Trickle	4.15	Feb 2015
6/11/2015	949.0	945.4	**	946.8	941.7	938.2	955.7	***	4.44	May 2015
Mean	949.5	945.7	942.5	946.9	941.7	938.4	956.1			
St. Dev.	0.3	0.4	0.3	0.3	0.2	0.1	0.3			

Notes:

* Groundwater levels could not be obtained due to piezometer damage by vandalism.

** The water level indicator could not be lowered into Piezometer P-3 due to an apparent blockage in the piezometer pipe. The issue is currently being investigated.

*** The toe drain discharge could not be observed due to overgrown vegetation blocking access.

ATTACHMENT A

REPORT OF PIEZOMETER REPAIRS

WILLMER ENGINEERING INC. WE

July 31, 2015

VIA EMAIL

Ms. Paige Singer
Project Manager
DeKalb County Recreation, Parks, and Cultural Affairs
1300 Commerce Drive
Decatur, Georgia 30030

SUBJECT: **Report of Piezometer Repairs**
Erin Lake Dam Rehabilitation
DeKalb County, Georgia
Willmer Project No. 71.4017

Dear Ms. Singer:

Willmer Engineering Inc. (Willmer) is pleased to present this report of piezometer repairs for the Erin Lake Dam Rehabilitation project located at Henderson Park in DeKalb County, Georgia. This work was performed in accordance with the DeKalb County Purchase Order No. 951040, dated December 12, 2014. Descriptions of the installed piezometers, piezometer damages, and repair methods are presented in this report.

Installed Piezometers

Six piezometers (P-1 through P-6) were installed by Willmer in December 2013. The piezometer installation records were provided in our report dated February 7, 2014. Each piezometer was provided with a 2-foot square concrete surface pad, a protective steel casing extending 3 to 4 feet above the ground surface, and a locking cap on the casing. Photographs showing the piezometers after installation are presented in Appendix I.

Description of Piezometer Damages

During a site visit by Willmer on September 8, 2014 to take scheduled readings of the water levels in the piezometers, we observed that three of the six piezometers (P-3, P-4, and P-5) had been vandalized. Photographs of the vandalized piezometers are presented in Appendix II. It appeared that the protective steel casings of the piezometers were rocked back and forth until the ground surrounding the concrete pads loosened enough to allow the piezometers to be bent over. The steel covers that were encased in the concrete pads were uprooted, and the PVC piezometer pipes were either bent or broken off below the protective casing. We assess that the piezometers were vandalized sometime between mid-June and early September of 2014, as our previous site visit was made on June 10, 2014, and no damages to the piezometers were observed at that time.

Description of Piezometer Repairs

A subsequent site visit was made by Willmer on September 18, 2014 to assess the conditions of the PVC pipes and to evaluate possible repair methods. A cost estimate for repairs of the damaged piezometers was prepared by Willmer and submitted to DeKalb County on October 7, 2014. A Purchase Order authorizing the work was issued on December 12, 2014 by DeKalb County, and Willmer completed the piezometer repairs on January 22, 2015. The elevations of the repaired piezometers were surveyed on February 10, 2015 for future scheduled water level recordings.

The piezometers were repaired by cutting the PVC pipe just below the point of damage (i.e., bent section or broken end). The concrete pad, protective steel casing, and damaged PVC piezometer pipe were then removed. A new section of PVC pipe was connected to the existing PVC pipe using a coupling, and the new section of pipe was extended to just below the ground surface. A vent port was then installed near the top of the PVC pipe extension. After repairing the PVC pipe, a 12 to 18-inch deep hole was excavated around the pipe. The hole was filled with concrete and an 8-inch diameter by 12-inch deep drop-in manhole was placed in the concrete to protect the PVC pipe. Prior to pouring the concrete, three 4-foot long #4 rebars were driven into the ground to provide strong ground anchorage for the concrete encasement. A locking cap plug was installed at the top of the PVC pipe and the drop-in manhole cover was bolted shut. A schematic of the piezometer repair is presented in Figure 1, and photographs of the repaired piezometers are attached in Appendix III. It should be noted that the repaired piezometers were fitted with top of casings flush with the ground surface to make them less visible and minimize the potential for future vandalism.

Closing Remarks

We appreciate the opportunity to be of service to you on this project. Please contact us if you have any questions regarding this report.

Sincerely,

WILLMER ENGINEERING INC.

Bradford Drew

G. Bradford Drew, PE
Project Engineer

Sujit K Bhowmik

Sujit K. Bhowmik, PhD, PE
Chief Engineer/Safe Dams Engineer of Record for Geotechnical Engineering

James L. Willmer, P.E.

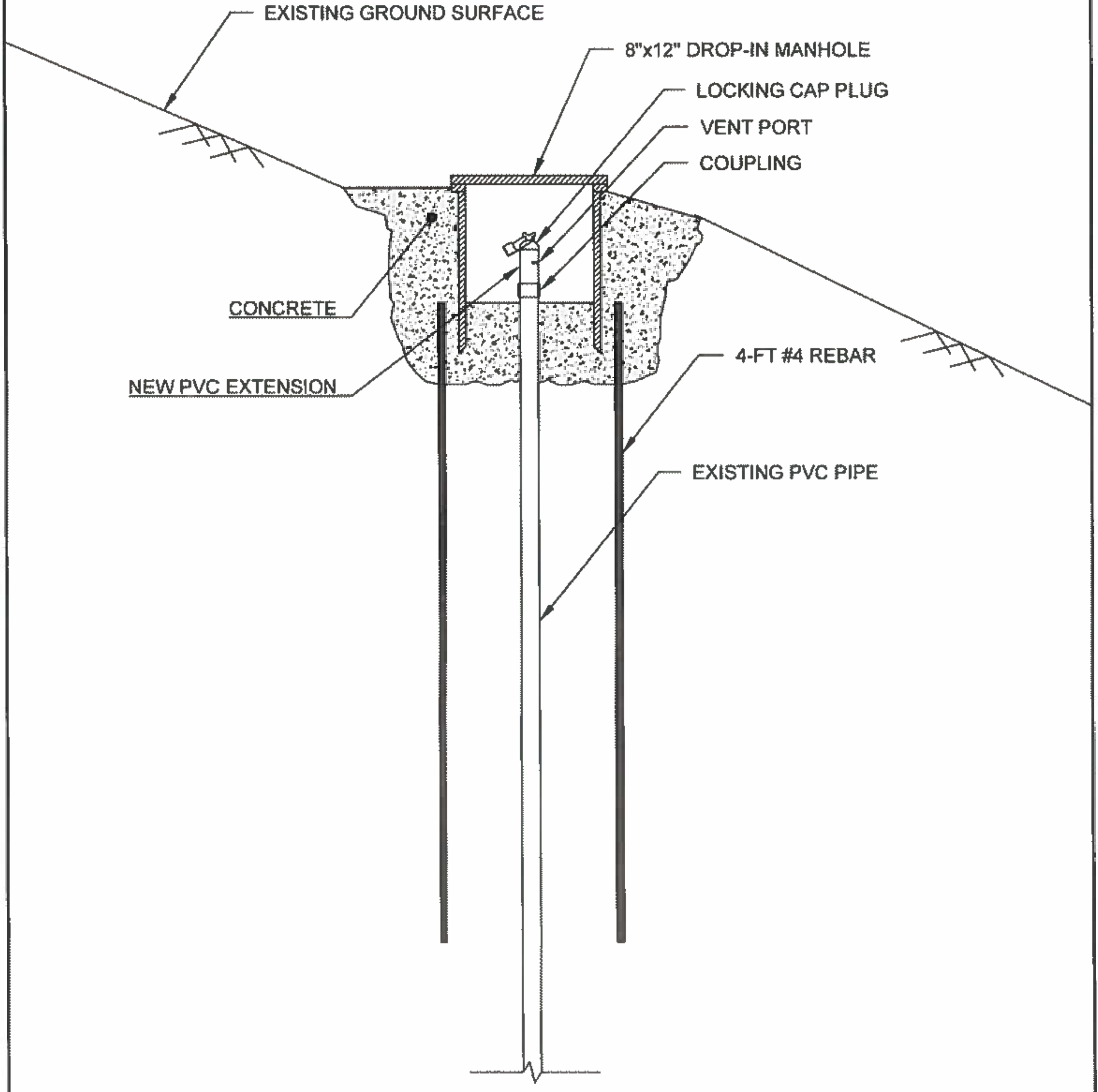
James L. Willmer, PE
Executive Vice President/Principal Consultant/Safe Dams Engineer of Record for Geotechnical Engineering

Attachments:

Figure 1 – Piezometer Repair Schematic
Appendix I – Photographs of Piezometers after Installation
Appendix II – Photographs of Damaged Piezometers
Appendix III – Photographs of Repaired Piezometers

FIGURE 1

PIEZOMETER REPAIR SCHEMATIC



SCALE: NTS

DATE: 2/13/2015
DRAWN BY: JB
REVIEWED BY: BD

WILLMER ENGINEERING INC. WE

GEOTECHNICAL ENGINEERING
CONSTRUCTION SERVICES
ENVIRONMENTAL SERVICES AND ENGINEERING
3772 PLEASANTDALE ROAD - SUITE 165
ATLANTA, GA 30340-4270

FIGURE 1

PIEZOMETER REPAIR SCHEMATIC
ERIN LAKE DAM PIEZOMETER REPAIRS
DEKALB COUNTY, GEORGIA
WILLMER PROJECT No. 71.4017

P:\4017 Erin Lake Dam Piezometer Repairs\CADD\Piezometer Repair Schematic.dwg

APPENDIX I

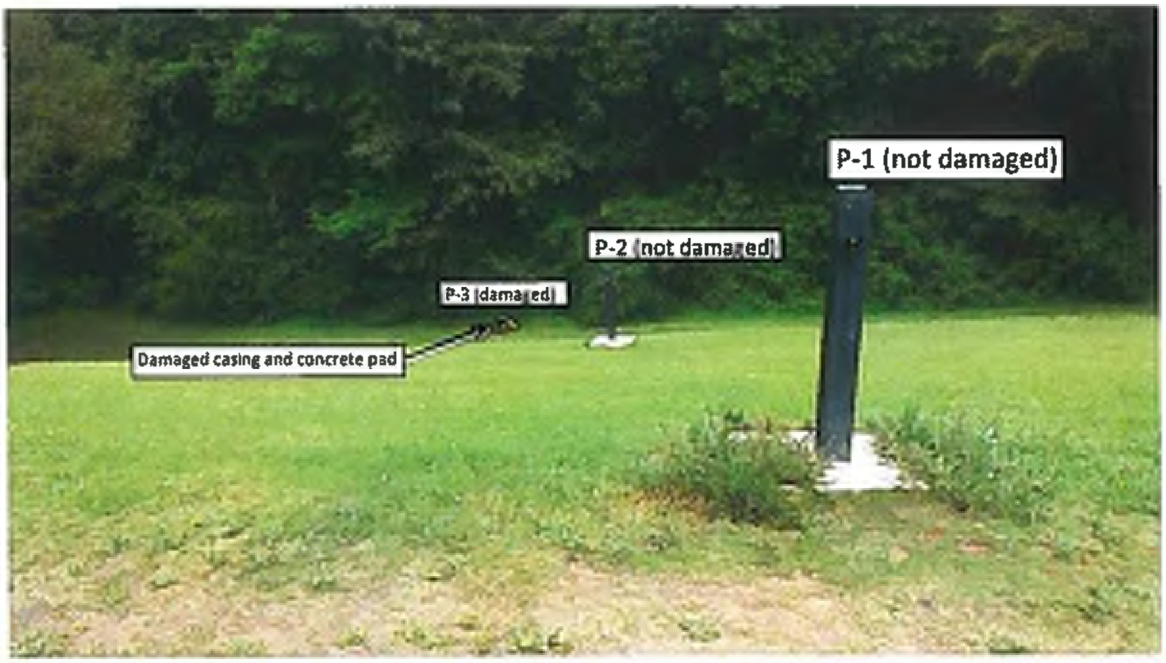
PHOTOGRAPHS OF PIEZOMETERS AFTER INSTALLATION

Piezometers P-1 through P-3; photographed on 12/6/2013

Piezometers P-4 through P-6; photographed on 12/6/2013

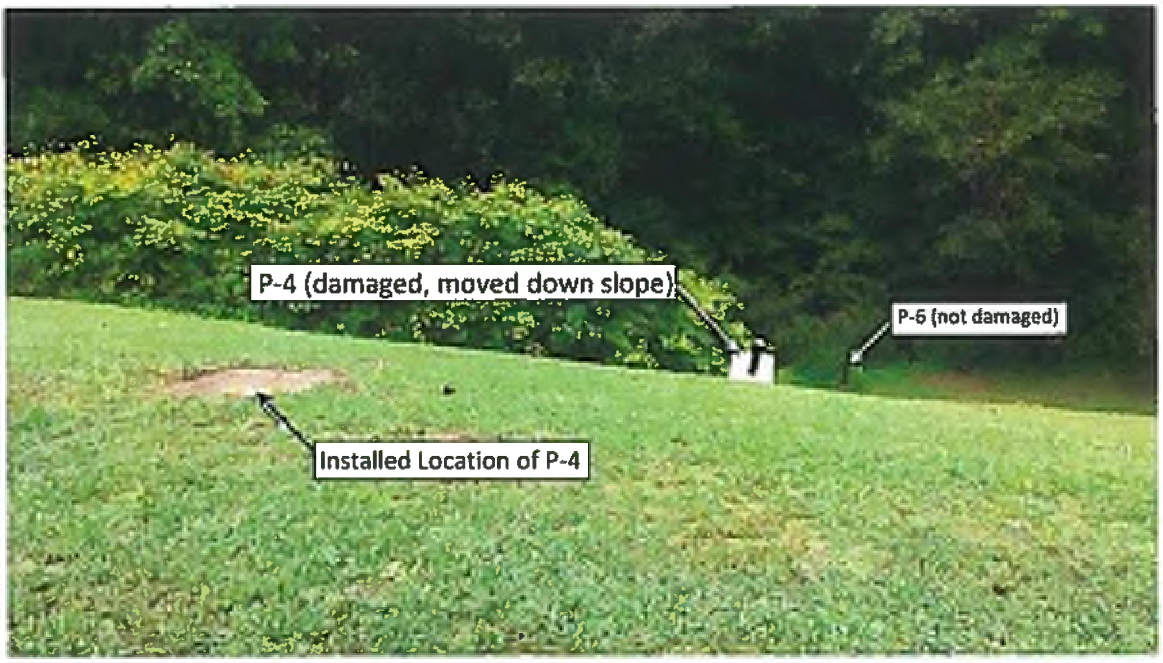
APPENDIX II

PHOTOGRAPHS OF DAMAGED PIEZOMETERS

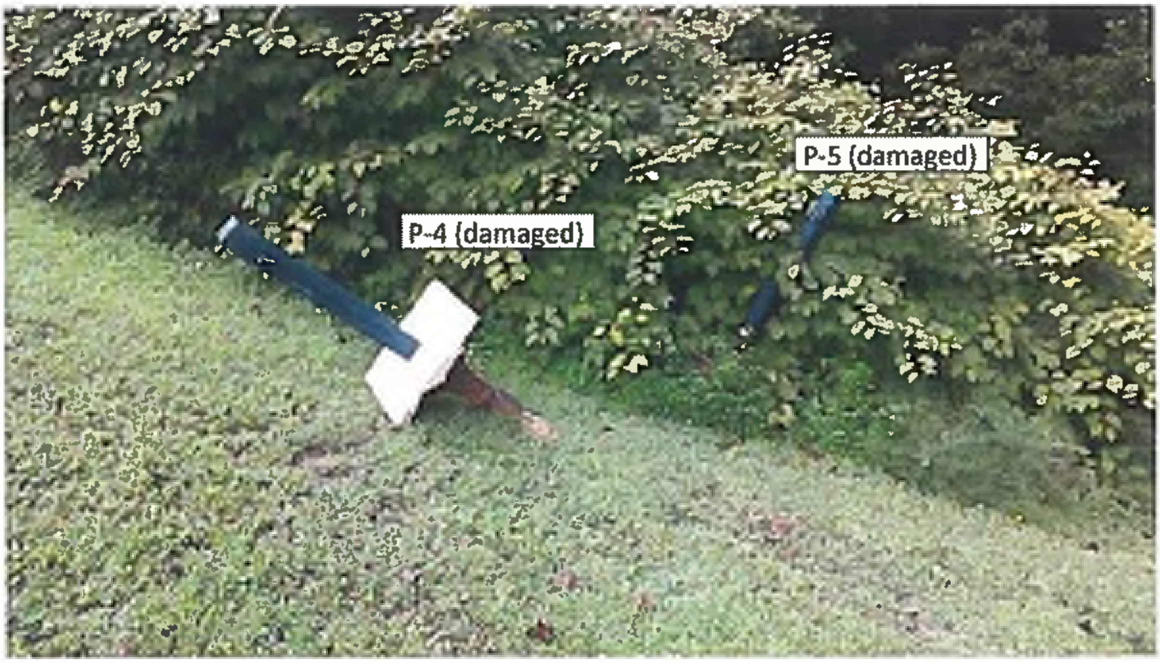


Piezometers P-1 through P-3; Facing Northwest; 09/08/2014

Damaged Piezometer P-3; Facing East; 09/08/2014



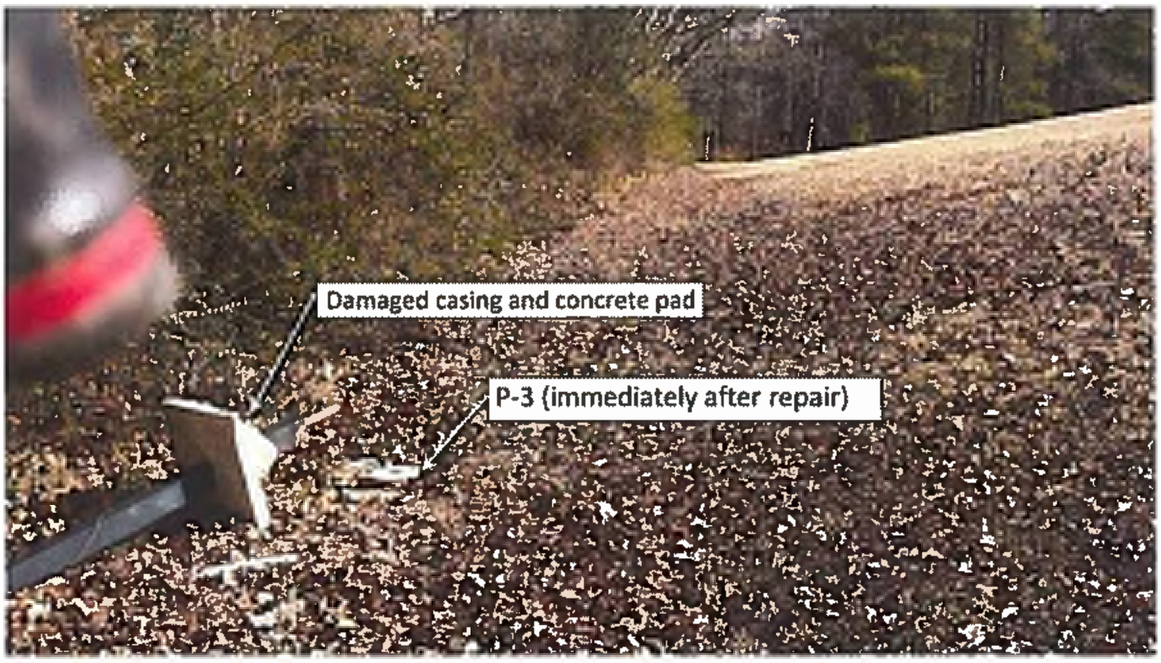
Piezometers P-4 through P-6; Facing Northwest; 09/08/2014



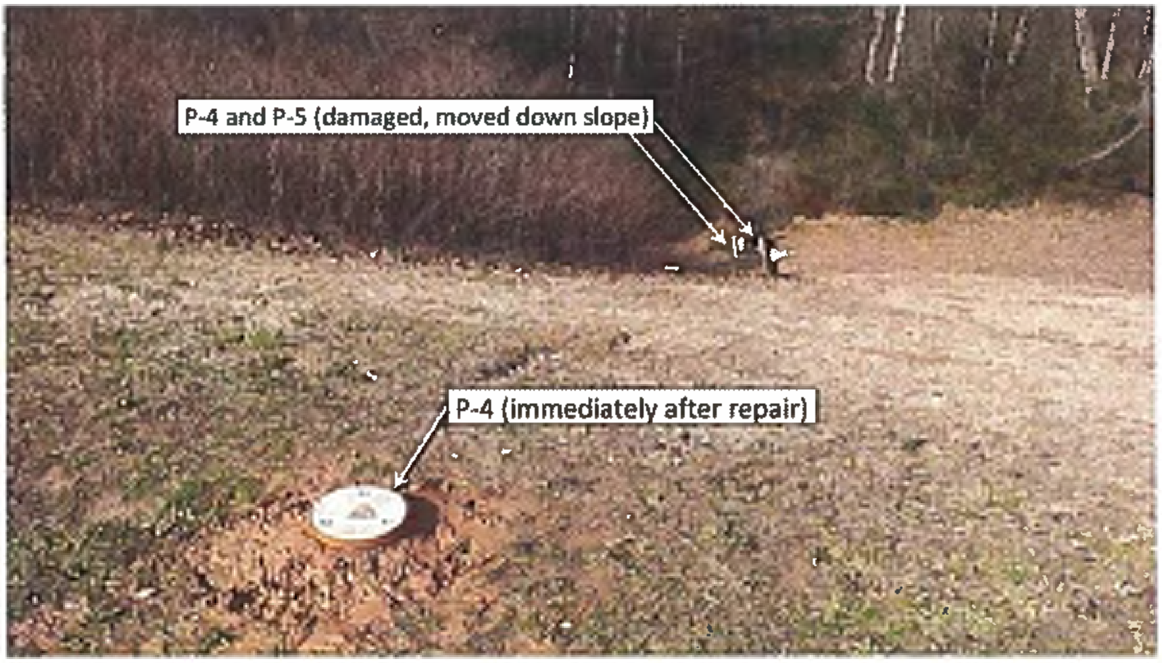
Damaged Piezometers P-4 and P-5; Facing West; 09/08/2014

APPENDIX III

PHOTOGRAPHS OF REPAIRED PIEZOMETERS



Repaired Piezometer P-3; Facing East; 01/27/2015



Repaired Piezometer P-4; Facing Northwest; 01/27/2015

Repaired Piezometer P-4 with Protective Cover Removed; 01/27/2015

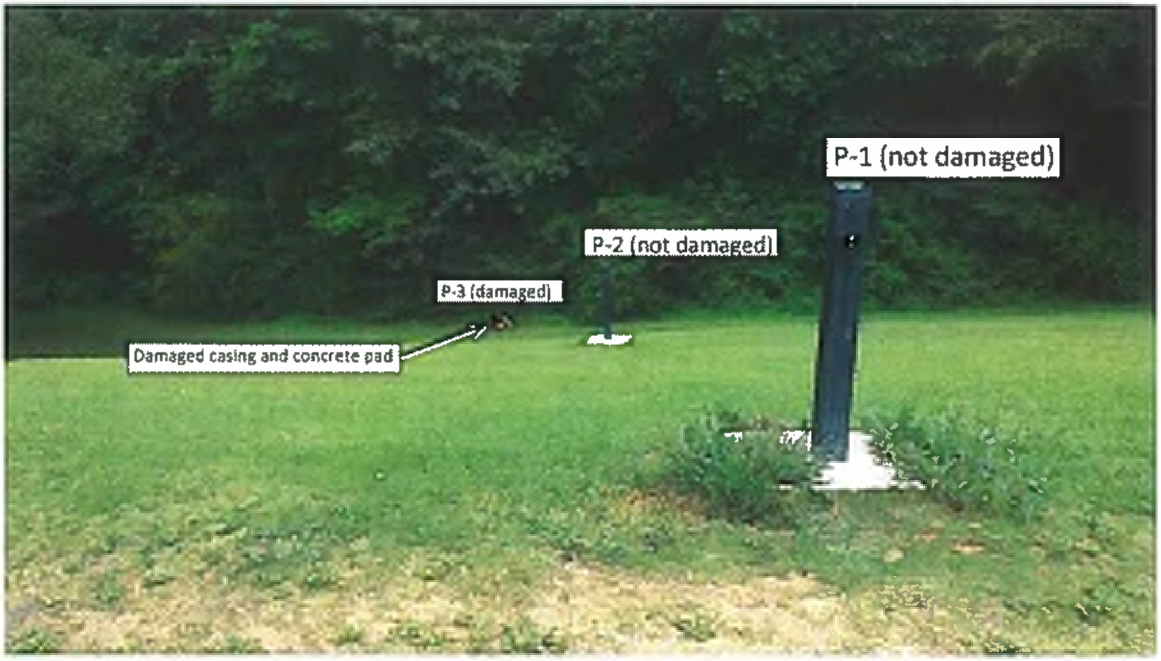
Repaired Piezometer P-5; Facing West; 01/27/2015

Piezometers P-1 through P-3; photographed on 12/6/2013

Piezometers P-4 through P-6; photographed on 12/6/2013

APPENDIX II

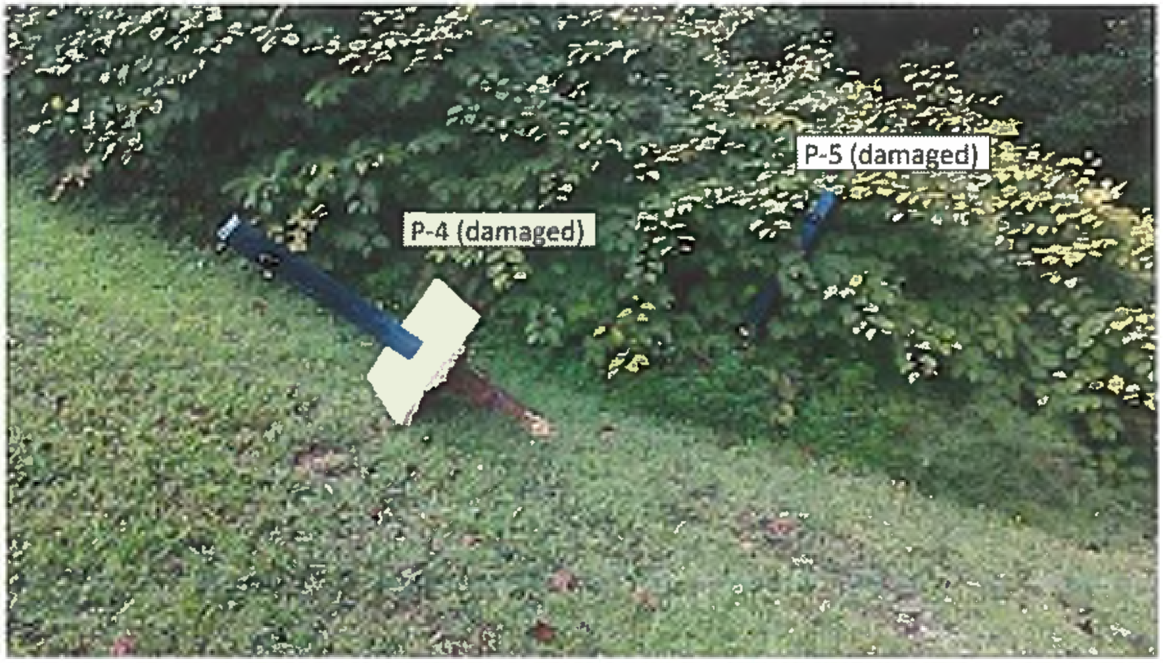
PHOTOGRAPHS OF DAMAGED PIEZOMETERS



Piezometers P-1 through P-3; Facing Northwest; 09/08/2014

Damaged Piezometer P-3; Facing East; 09/08/2014

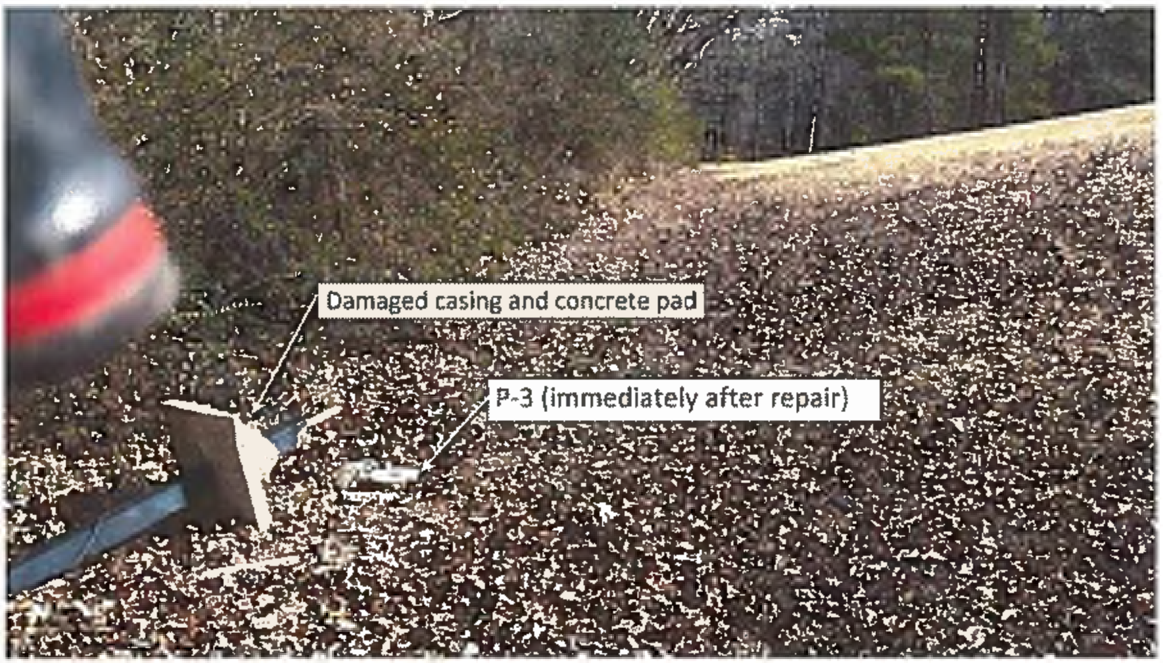
Piezometers P-4 through P-6; Facing Northwest; 09/08/2014



Damaged Piezometers P-4 and P-5; Facing West; 09/08/2014

APPENDIX III

PHOTOGRAPHS OF REPAIRED PIEZOMETERS



Repaired Piezometer P-3; Facing East; 01/27/2015

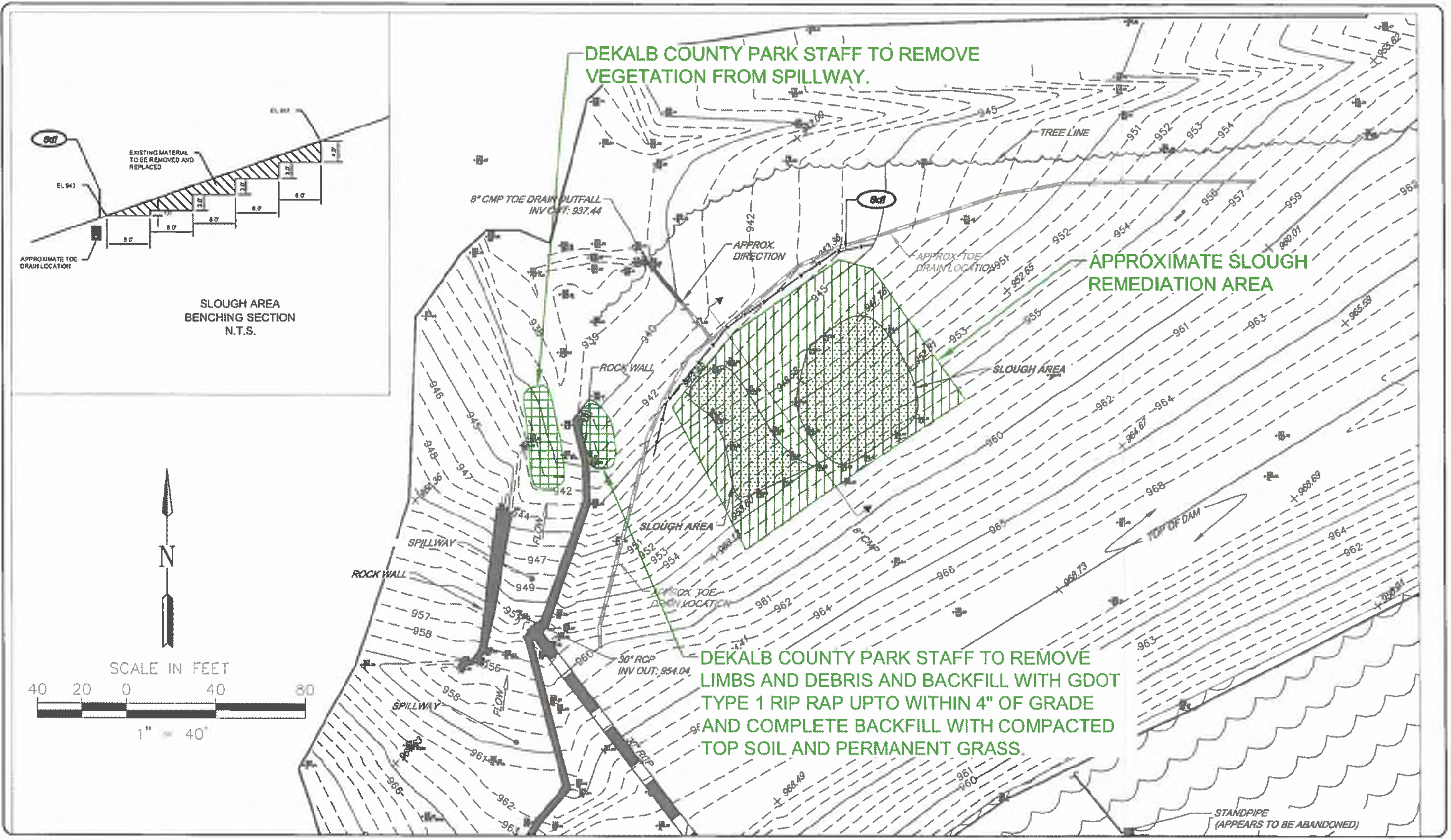
Repaired Piezometer P-4; Facing Northwest; 01/27/2015

Repaired Piezometer P-4 with Protective Cover Removed; 01/27/2015

Repaired Piezometer P-5; Facing West; 01/27/2015

Appendix B

K:\ATL_Civil\019303043_DeKalb Cty - Erin Lake Dam Rehabilitation Svcs\CADD\CONSTR\C4-00_Grading Plan.dwg



DEKALB COUNTY PARK STAFF TO REMOVE VEGETATION FROM SPILLWAY.

APPROXIMATE SLOUGH REMEDIATION AREA

DEKALB COUNTY PARK STAFF TO REMOVE LIMBS AND DEBRIS AND BACKFILL WITH GDOT TYPE 1 RIP RAP UPTO WITHIN 4" OF GRADE AND COMPLETE BACKFILL WITH COMPACTED TOP SOIL AND PERMANENT GRASS.

SLOUGH AREA BENCHING SECTION N.T.S.

Kimley-Horn and Associates, Inc.
3169 HOLCOMB BRIDGE ROAD, SUITE 600, NORCROSS, GEORGIA 30071
PHONE: (770) 825-0744 FAX: (770) 825-0074
www.kimley-horn.com CA 000379

TITLE
SLOUGH REMEDIATION AREA

PROJECT
ERIN LAKE DAM

CLIENT
DEKALB COUNTY, GEORGIA

EXHIBIT 1

Georgia Department of Natural Resources
Environmental Protection Division
Safe Dams Program
200 Piedmont Avenue, S.W., Suite 418
Atlanta, Georgia 30334
(404) 463-1511

July 9, 2015

Ms. Paige Singer
DeKalb County Natural Resources Management Office
1300 Commerce Drive, 3rd Floor
Decatur, Georgia 30030

SUBJECT: Erin Lake Dam
DeKalb County

Dear Ms. Singer:

On February 3, 2015, our staff performed the FY15 quality assurance inspection on the above mentioned dam. During the inspection, we noted the following items:

1. Much of the upstream slope and some of the downstream slope need to be mowed. These areas could not be properly observed.
2. Much of the crest and several other areas are bare and need to be grassed with low growing grasses.
3. Three piezometers have fallen over. One of the piezometers which is still standing has several inches of clearance between the ground and concrete pad which is supposed to be at ground level.
4. There is a slough about 20 feet by 10 feet with an erosion rill on the right (looking downstream) side of it located on the downstream slope. There are at least two slightly smaller sloughs nearby. One of these has a sunken hole about 18 inches in diameter and about 7 inches deep downhill of it.
5. Part of the left spillway wall has collapsed. Other parts of the left wall are deteriorating. There appears to be seepage down the right side of the spillway above the pipe outlet.
6. The spillway needs to be cleared of debris.
7. There is seepage beyond the toe of the dam at the left and center of the dam.
8. The toe drains need to be cleaned out and to have trees and brush cleared so they can be accessed.

Plans have been submitted to the Safe Dams Program for improvements to Erin Lake Dam, and have been reviewed by the Safe Dams Program. However, neither DeKalb County nor the engineer who prepared the plans has responded adequately to comments on those plans and requests for additional information sent to you by letters of May 25, 2012 and August 4, 2014 (copies attached). Inspections indicate that the downstream slope is not stable and that sloughs have been forming and increasing in extent. You must retain an Engineer of Record (EOR) with the Safe Dams Program and notify us of such by **August 10, 2015**. The full geotechnical evaluation requested in both letters must contain an evaluation of the extent, nature, and stability of the soils in the dam and address in detail remediation of the sloughs and seepage through the dam. A response to all issues and requests for information in both letters should be submitted with the full geotechnical report by **January 5, 2016**.

As owner/operator of the dam, you are responsible for inspecting your dam on a routine basis. This office recommends at least quarterly inspections be performed. An inspection form is enclosed for your use. A more detailed inspection form can be used in lieu of the one provided. Photographs should also be taken and provided as part of the report.

Please retain the completed checklists, photographs, instrumentation readings, and any other notes taken during your inspections. The inspection reports should be sent to this office at least annually. If you notice any significant changes with your dam(s), such as new cracks, sloughs (shallow slope failures), excessive or new seepage, or muddy flow from the dam or internal drains, please contact our office immediately.

A copy of our inspection report with photos is enclosed for your records. If you have any questions, please contact our office at (404) 463-2461.

Sincerely,

Kate Betsill
Environmental Engineer
Safe Dams Program

KB:ks
Enclosures

Sujit Bhowmik

From: Sujit Bhowmik
Sent: Friday, August 23, 2013 4:41 PM
To: 'Mark.Kilby@kimley-horn.com'
Cc: Bradford Drew; Jim Willmer
Subject: Erin Lake Dam - Results of Borrow Study
Attachments: 71.3638 Erin Lake Dam Rehabilitation - Borrow Pit Study.pdf

Mark,

We have completed our laboratory tests on the borrow soil. The tested soil sample was obtained from Test Pit No. 4 (TP-4), and the location of the test pit is shown in the attached Figure 1. The laboratory tests performed on the soil sample consisted of grain size distribution analysis, Atterberg limits, and standard proctor compaction tests. Results of these tests are presented in the attached individual test sheets, and a summary of the results is presented below:

Test Pit Number	Sample Depth (ft)	Soil Description	Percent Passing No. 200 Sieve (%)	Natural Moisture Content (%)	Atterberg Limits		Standard Proctor Compaction Test Results	
					Liquid Limit (%)	Plasticity Index (%)	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
TP-4	2 - 5	Reddish brown medium to fine sandy **ELASTIC SILT**	68.6	25.7	52	11	94.5	25.1

↳ Less than 50% for a dam

As shown above, the soil classifies as a sandy elastic silt with a liquid limit of 52%. This liquid limit is higher than what would be preferred for the slope repair work. The natural moisture content of the soil is 25.7 percent, which is close to the optimum moisture content (25.1%); thus, significant wetting/drying will not be required to achieve the optimum moisture content during construction. However, if this borrow material gets wet, it will be very difficult to dry it because of its low permeability (high fines content and liquid limit).

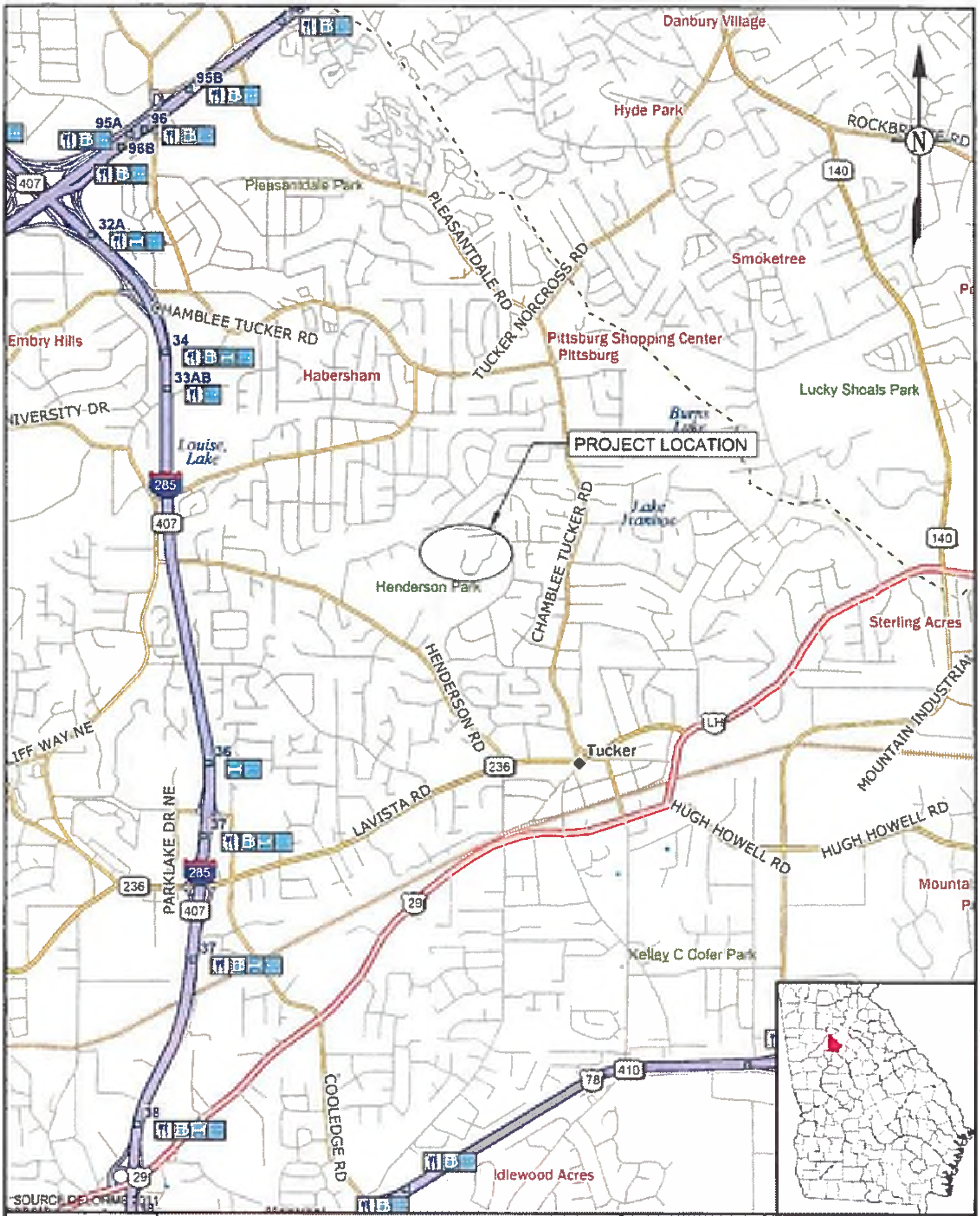
I will call you to discuss these borrow soil properties further.

Thanks,

Sujit

Sujit K. Bhowmik, PhD, PE
Chief Engineer

Willmer Engineering Inc.
3772 Pleasantdale Road, Suite 165
Atlanta, Georgia 30340
Phone 770-939-0089 ext. 30
Fax 770-939-4299
www.willmerengineering.com



SCALE: 1" = 3000'

DATE: 8/23/2013

DRAWN BY: NGD

REVIEWED BY: BD

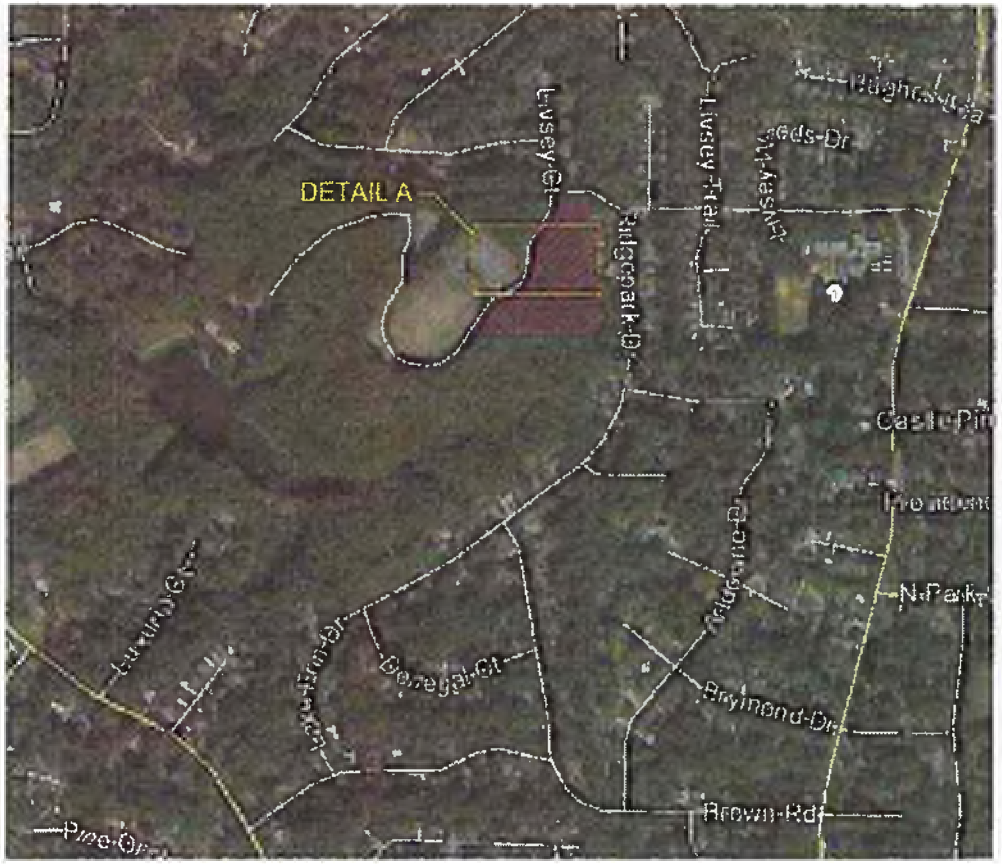
WILLMER ENGINEERING INC. | WE

GEOTECHNICAL ENGINEERING
CONSTRUCTION SERVICES
ENVIRONMENTAL SERVICES AND ENGINEERING
3772 PLEASANTDALE ROAD - SUITE 165
ATLANTA, GA 30340-4270

FIGURE 1

PROJECT LOCATION MAP
ERIN LAKE DAM REHABILITATION
DEKALB COUNTY, GEORGIA
WILLMER PROJECT No. 71.3638

P:\3638-Lake Erin-Henerdon Park Geotech C[illegible]\CADD\PROJECT LOCATION MAP.dwg



DETAIL A
(SCALE: 1" = 100')

LEGEND:

TP-1 ⌖ APPROXIMATE TEST PIT LOCATION

▨ PROPOSED BORROW PIT AREA

SOURCE: GOOGLE EARTH

SCALE: 1" = 600'

DATE: 8/23/2013

DRAWN BY: HGO

REVIEWED BY: BD

WILLMER ENGINEERING INC.

GEOTECHNICAL ENGINEERING ▪ CONSTRUCTION SERVICES
ENVIRONMENTAL SERVICES AND ENGINEERING
3772 PLEASANTDALE ROAD - SUITE 165
ATLANTA, GA 30340-4270

FIGURE 2

TEST PIT LOCATION PLAN
ERIN LAKE DAM REHABILITATION
DEKALB COUNTY, GEORGIA
WILLMER PROJECT No. 71.3636

TEST PIT SUMMARY

Date: August 8, 2013

Test Pit Number: TP-1

Location: See Figure 2

Total Depth: 3 feet

Excavation Kubota KX057-4

Depth to Ground Water N/A

Contamination Encountered: N/A

SOIL DESCRIPTION/CLASSIFICATION	
Depth	**Soil Description/Classification**
0 – 0.5'	Topsoil/roots
0.5' – 3'	RESIDUUM: Reddish brown medium to fine sandy CLAY
	Rock was encountered at 3 feet below the existing ground surface. No groundwater was encountered at the time of test pit excavation.

Additional Comments:

WILLMER ENGINEERING INC. WE

Willmer Project No.: 71.3638

Project Name: Erin Lake Dam Rehabilitation
DeKalb County, Georgia

TEST PIT SUMMARY

Date: August 8, 2013

Test Pit Number: TP-2

Location: See Figure 2

Total Depth: 8 feet

Excavation Kubota KX057-4

Depth to Ground Water N/A

Contamination Encountered: N/A

SOIL DESCRIPTION/CLASSIFICATION	
Depth	**Soil Description/Classification**
0 – 0.5'	Topsoil/roots
0.5' – 4'	RESIDUUM: Reddish brown medium to fine sandy CLAY
4' – 8'	RESIDUUM: Reddish brown and tan medium to fine sandy elastic SILT (slightly micaceous)
	No groundwater was encountered at the time of test pit excavation.

Additional Comments:

WILLMER ENGINEERING INC. | WE

Willmer Project No.: 71.3638

Project Name: Erin Lake Dam Rehabilitation
DeKalb County, Georgia

TEST PIT SUMMARY

Date: August 8, 2013

Test Pit Number: TP-3

Location: See Figure 2

Total Depth: 8 feet

Excavation Kubota KX057-4

Depth to Ground Water N/A

Contamination Encountered: N/A

SOIL DESCRIPTION/CLASSIFICATION

Depth	Soil Description/Classification
0 – 0.5'	Topsoil/roots
0.5' – 5'	RESIDUUM: Reddish brown medium to fine sandy elastic SILT
5' – 8'	RESIDUUM: Reddish brown and tan medium to fine sandy SILT (slightly micaceous)
	No groundwater was encountered at the time of test pit excavation.

Additional Comments:

WILLMER ENGINEERING INC. | WE

Willmer Project No.: 71.3638

Project Name: Erin Lake Dam Rehabilitation
DeKalb County, Georgia

TEST PIT SUMMARY

Date: August 8, 2013

Test Pit Number: TP-4

Location: See Figure 2

Total Depth: 8 feet

Excavation Kubota KX057-4

Depth to Ground Water N/A

Contamination Encountered: N/A

SOIL DESCRIPTION/CLASSIFICATION	
Depth	**Soil Description/Classification**
0 – 0.5’	Topsoil/roots
0.5’ – 5’	RESIDUUM: Reddish brown medium to fine sandy elastic SILT
5’ – 8’	RESIDUUM: Reddish brown and tan medium to fine sandy SILT (slightly micaceous)
	No groundwater was encountered at the time of test pit excavation.

Additional Comments:

WILLMER ENGINEERING INC. WE

Willmer Project No.: 71.3638

Project Name: Erin Lake Dam Rehabilitation
DeKalb County, Georgia

TEST PIT SUMMARY

Date: August 8, 2013

Test Pit Number: TP-5

Location: See Figure 2

Total Depth: 3 feet

Excavation Kubota KX057-4

Depth to Ground Water N/A

Contamination Encountered: N/A

SOIL DESCRIPTION/CLASSIFICATION	
Depth	**Soil Description/Classification**
0 – 0.5’	Topsoil/roots
0.5’ – 3’	RESIDUUM: Reddish brown medium to fine sandy elastic SILT
	Rock was encountered at 3 feet below the existing ground surface. No groundwater was encountered at the time of test pit excavation

Additional Comments:

WILLMER ENGINEERING INC. WE

Willmer Project No.: 71.3638
Project Name: Erin Lake Dam Rehabilitation
DeKalb County, Georgia

TEST PIT SUMMARY

Date: August 8, 2013
Test Pit Number: TP-6
Location: See Figure 2
Total Depth: 6 feet
Excavation Kubota KX057-4
Depth to Ground Water N/A
Contamination Encountered: N/A

SOIL DESCRIPTION/CLASSIFICATION	
Depth	**Soil Description/Classification**
0 – 0.5’	Topsoil/roots
0.5’ – 6’	RESIDUUM: Reddish brown medium to fine sandy elastic SILT
	Rock was encountered at 6 feet below the existing ground surface. No groundwater was encountered at the time of test pit excavation.

Additional Comments:

Job No. 71.3638 Date 8/15/13

Project Erin Lake Dam Rehabilitation

DeKalb County, Georgia

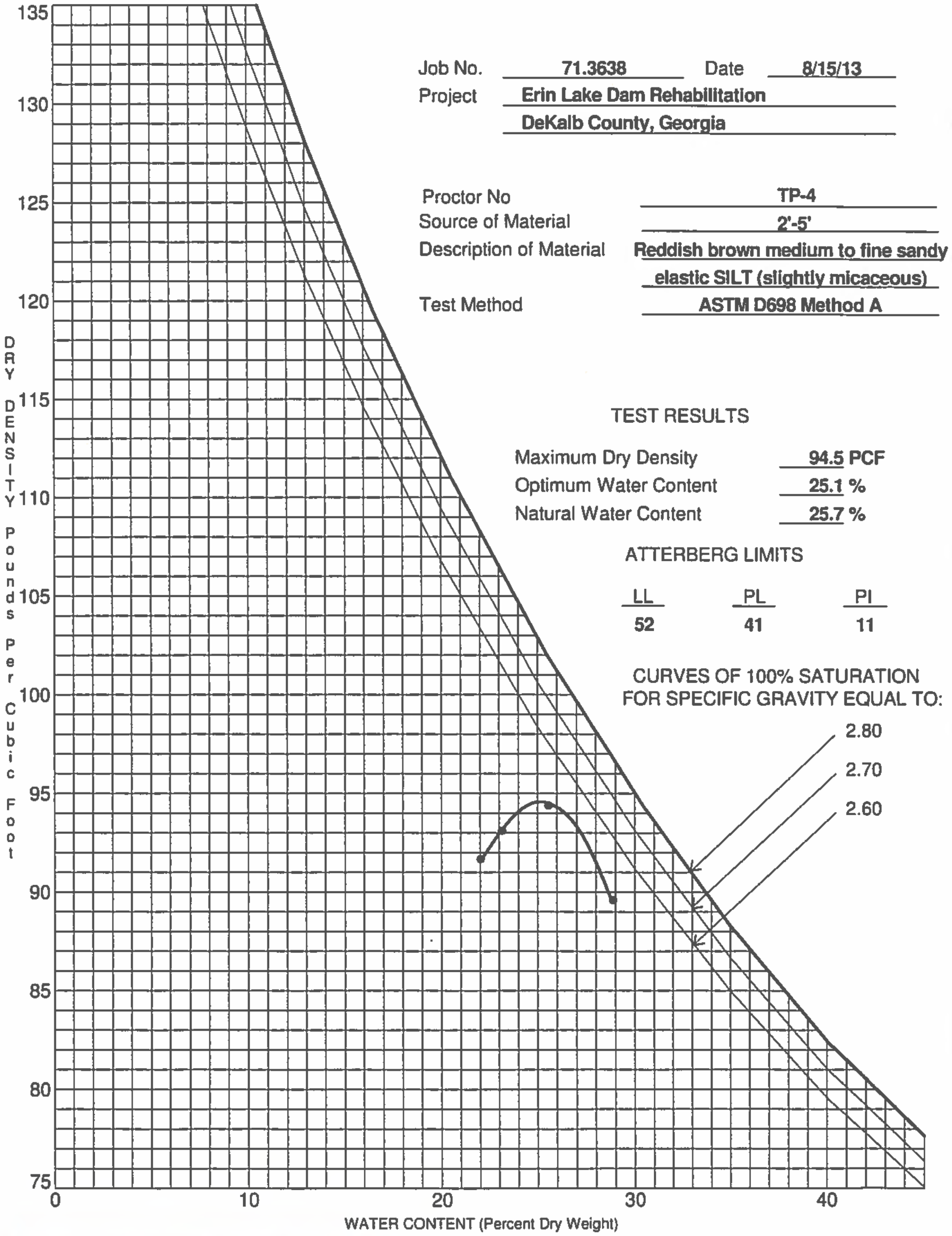
Proctor No	TP-4
Source of Material	2'-5'
Description of Material	Reddish brown medium to fine sandy elastic SILT (slightly micaceous)
Test Method	ASTM D698 Method A

TEST RESULTS

Maximum Dry Density	94.5 PCF
Optimum Water Content	25.1 %
Natural Water Content	25.7 %

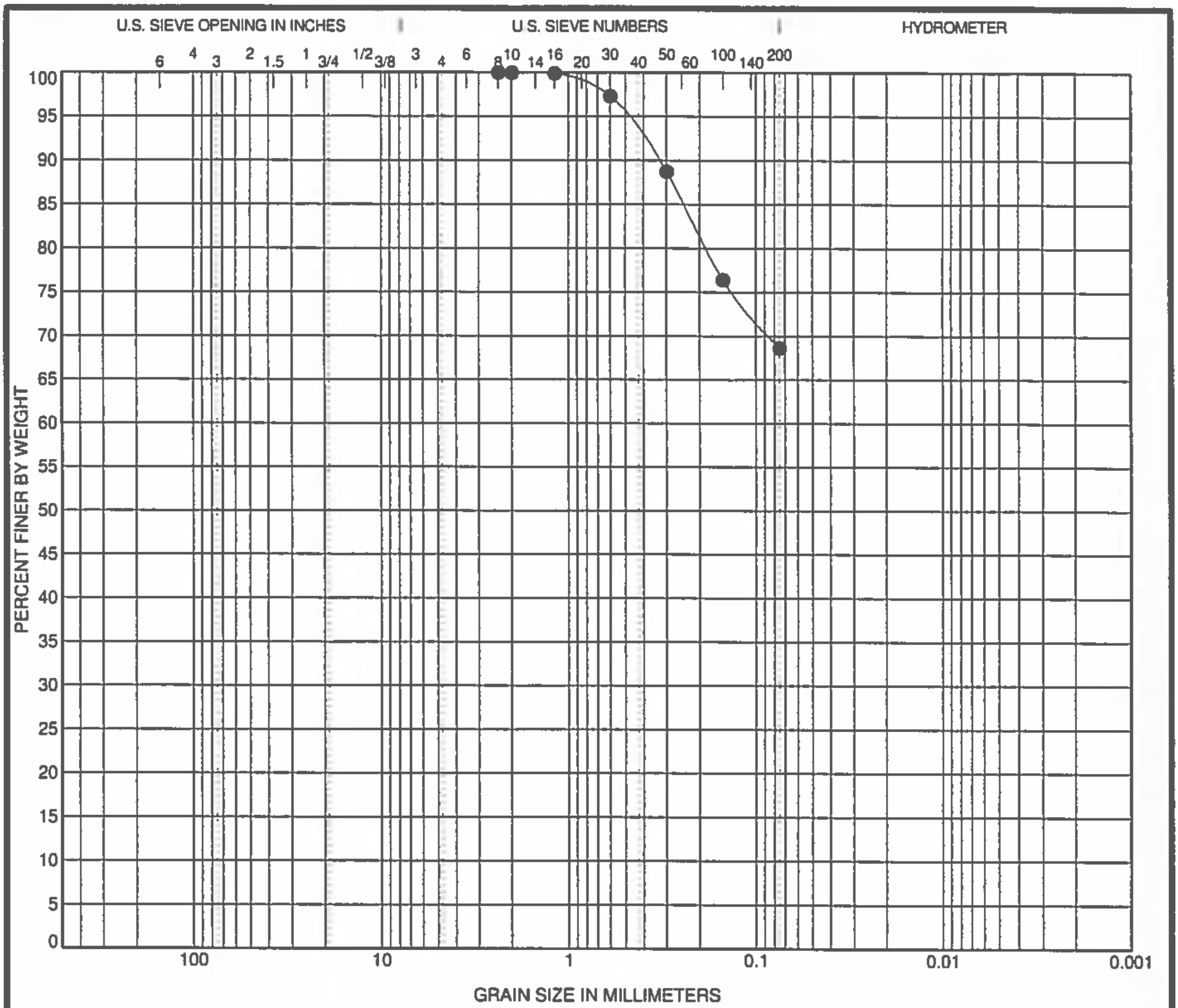
ATTERBERG LIMITS

LL	PL	PI
52	41	11



WILLMER ENGINEERING INC. WE

MOISTURE-DENSITY RELATIONSHIP



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

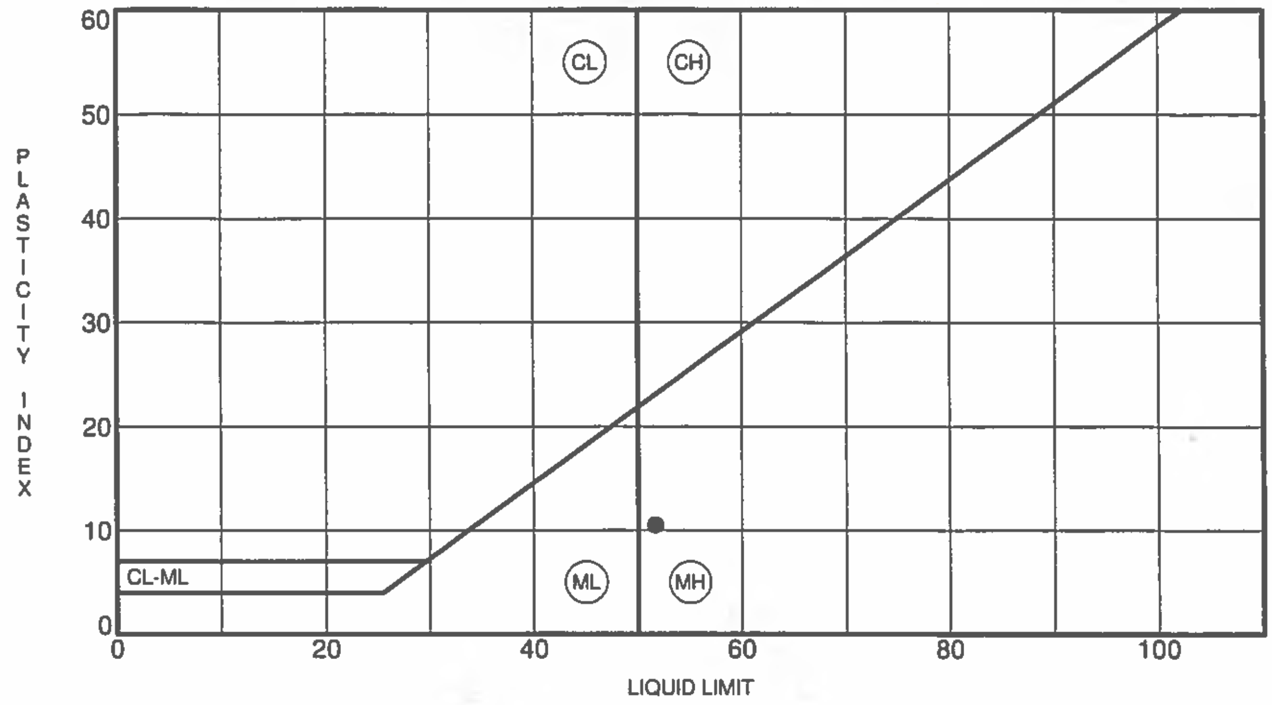
	Specimen Identification	Classification	LL	PL	PI	Cc	Cu
●	TP-4 (2'-5')	Reddish brown medium to fine sandy elastic SILT (slightly micaceous)	52	41	11		

	Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	TP-4 (2'-5')	2.36				0.0	31.4	68.6	

WILLMER ENGINEERING INC. WE

GRAIN SIZE DISTRIBUTION

Project: Erin Lake Dam Rehabilitation
Location: DeKalb County, Georgia
Number: 71.3638



	Specimen Identification	LL	PL	PI	Fines	Classification
●	TP-4 (2'-5')	52	41	11	68.6	Reddish brown medium to fine sandy elastic SILT (slightly micaceous)

WILLMER ENGINEERING INC. WE

ATTERBERG LIMITS RESULTS

Project: Erin Lake Dam Rehabilitation
Location: DeKalb County, Georgia
Number: 71.3638

ACCURA
GEOTECHNICAL DATA REPORT

Erin Lake Dam
Tucker, DeKalb County, Georgia

Submitted to:
AECOM
Attn: Malavika Tripathi, PE, LEED Green Associate, ENV SP
Project Manager
One Midtown Plaza
1360 Peachtree Street, NE, Suite 500
Atlanta, Georgia 30309

Submitted by:

ACCURA
Accura Engineering and Consulting Services, Inc.
3200 Presidential Drive
Atlanta, GA 30340

August 2021

ACCURA

August 27, 2021

AECOM
Ms. Malavika Tripathi, PE, LEED Green Associate, ENV SP
Project Manager
One Midtown Plaza
1360 Peachtree Street, NE, Suite 500
Atlanta, Georgia 30309, USA
Email: malavika.tripathi@aecom.com

Subject: **Geotechnical Data Report**
Erin Lake Dam
Tucker, DeKalb County, Georgia

Dear Ms. Tripathi:

The enclosed report presents the results of the geotechnical subsurface exploration program undertaken by Accura Engineering and Consulting Services, Inc. (Accura) in connection with the above referenced project. Our services were performed in general conformance with the scope of work presented in Accura Proposal dated March 30, 2021. This report presents our understanding of the project, reviews our exploration procedures, describes the general subsurface conditions at the boring locations, laboratory test results, and our findings.

We have enjoyed working with you on this project, and we are prepared to assist you with the recommended quality assurance monitoring and testing services during construction. Please contact us if you have any questions regarding this report or if we may be of further service.

Sincerely,

ACCURA ENGINEERING AND CONSULTING SERVICES, INC.

Ken Khanidokht, P.E.
Geotechnical Group Leader-Senior Engineer

GEORGIA
REGISTERED
NO. PE032545
PROFESSIONAL
ENGINEER
KHEIBAR KHANIDOKHT

Justin Lawrence, EIT
Civil Engineer

Accura Engineering and Consulting Services, Inc.
3200 Presidential Drive Atlanta, Georgia 30340 Phone: 404.241.8722 Fax: 404.241.4577

TABLE OF CONTENTS

1.0 SITE LOCATION AND PROJECT INFORMATION

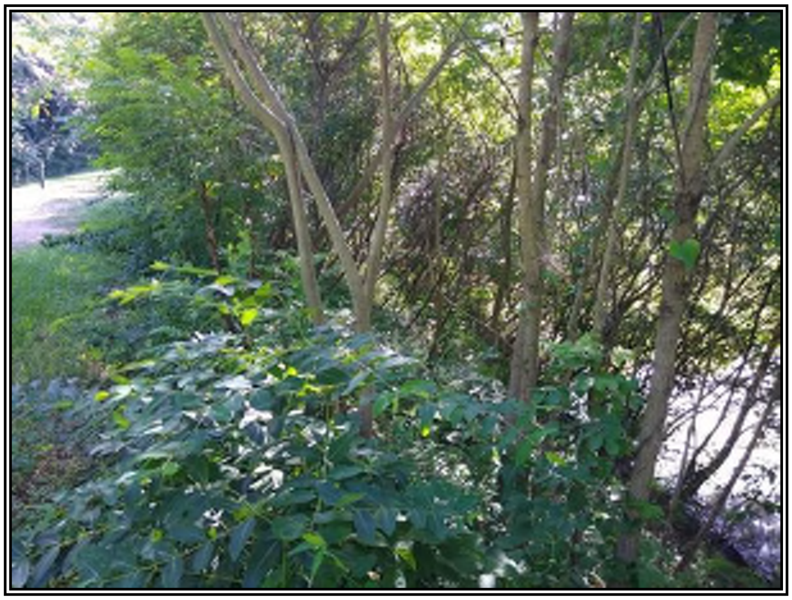
Erin Lake Dam is located at Henderson Park in Tucker, Dekalb County, Georgia. The dam and lake are surrounded by wooded areas and dense residential properties. We understand that the project consists of the rehabilitation and improvement of the dam to bring it into compliance with the requirements of GSDP (Georgia Safe Dams Program).

Crest of the dam

The existing dam is an earthen structure that is approximately 350 feet long and has an east-west orientation. The maximum height of the dam is about 30 feet. The crest of the dam is 15 feet wide and covered with soil and grass. No longitudinal or transverse cracks were observed within the top of crest. The general condition of the crest of the dam is shown below:

Upstream Face

The upstream face of the dam had an average slope of roughly 2(H):1(V) and was covered in thick vegetation with trees along the water line. There was no evidence of slope distress or erosion on the upstream face at the time field work was conducted.



Downstream Face

The downstream face of the dam on average had a slope that ranged from 2(H):1(V) to 3(H):1(V). The downstream face is covered predominantly with grass terminating at wooded areas to the north and west.

There were instances of slope distress as evidenced by erosive soils near the change in grade, but no evidence of seepage was observed on the downstream face at the time field work was conducted.

Spillway Locations

Based on information provided by the client and our review of a survey plan performed by Accura, we understand that the principal spillway is an approximately 12-foot-wide natural rock spillway located at the northwestern end of the dam. The outlet control system is a 30” corrugated metal pipe at the end of the left abutment that connects to a junction box. The drain outlet for the lake consists of a 30-inch diameter reinforced concrete pipe running through the west side of the dam near the left abutment and outflows onto the old stone spillway.

An emergency spillway system was observed for this dam. It consisted of the old spillway system on the northwest corner of the dam. In the event of high water, a low point on the corner of the left abutment would allow water to flow out onto a rock spillway abutted by mortared rock walls.

Probe Rod

The accessible areas along the downstream toe of the dam were visually observed and probed with a probe rod. The probed areas along the downstream toe were generally firm, and there were no obvious visual indications of seepage. The probed area along the creek bank near the piezometer, P-6, was found to be loose in consistency. We note that some large trees and dense vegetation were present along the lower portion of the downstream face and toe of the dam.

The bottom 2 inches of the downstream face and toe of the dam were noted to be firm when probed with the probe rod. No trees along the downstream face were noted to be leaning, which is often an indication of long-term slope movement.

Project locations and general surroundings are shown on the attached Site Vicinity Map and Test Location Plan in Appendix I, Drawing No. 1 and Drawing No. 2.

2.0 PURPOSE & SCOPE OF SERVICE

The purpose of our involvement on this project was to: 1. provide general descriptions of the subsurface soil, rock, and groundwater conditions within the crest area, abutments, and downstream toe of dam, 2. excavating the test pits to determine the expose the pipe of toe drain of dam, 3. obtain laboratory tests, and 4. provide a geotechnical data report.

In order to accomplish these objectives, we undertook the following scope of services:

- Visited the site to observe existing surface conditions and marked the proposed boring locations.
- Coordinated underground utility clearance with GA 811.
- Reviewed and summarized readily available geologic information relative to the project site.
- Executed the subsurface exploration program consisting of 5 Standard Penetration Test (SPT) soil boring drilled up to a 46-foot at the crest of the dam, the toe of the dam, and the left and right abutments. Borings were performed near perpendicular to each other to accurately profile the dam and subsurface soils at one cross section.
- Measured groundwater after the completion of drilling.
- Two (2) Test Pits were excavated up to a depth of 12 feet on the dam embankment near the downstream toe at the location of the toe drain and of previously observed slope discontinuities.
- Performed four (4) In-Place Density and Water Content of Soil by Nuclear Gauge Methods after the excavated material is placed into the test pit hole.
- Four (4) Shelby tube were pushed at a depth from 10 feet to 12 feet, 20 feet to 22 feet, and 22 feet to 24 feet within the offset boring AB-2A top of the crest of dam, and at a depth from 7 feet to 9 feet within boring AB-3 at downstream toe of dam. The Shelby tube at boring AB-2A from 20 feet to 22 feet had no recovery, therefore only three (3) samples were obtained.
- The existing 1-inch diameter piezometer, P-3, was drilled out (approximately 15 feet deep) and tremie grouted completely closed from the bottom.
- We Installed a two (2)-inch diameter open well piezometer at borehole P-3 to a depth of 25 feet and is screened for the bottom 15 feet. The piezometer was capped with a “stick-up” style cap mounted approximately 24 to 36 inches above existing grade.
- The existing 1-inch diameter piezometer, P-4, was drilled out (approximately 35 feet deep) and extended 4 feet deeper into native soil with two spoon samples. A new two (2)-inch diameter

open well piezometer was installed in the same location with a new “flush-mount” style cap. It screened in the lowest 4 feet.

- Representative samples, collected from the subsurface investigation, were visually classified by a qualified member of our geotechnical staff. In addition, some of those samples were subjected to soil index testing to assist in and verify the classifications and for use in engineering analysis and design. We proposed fifteen (15) Moisture Content (ASTM D2216) tests, seven (7) Grain Size Analysis (ASTM D422) with hydrometer tests, seven (7) Grain Size Analysis (ASTM D6913), eight (8) Atterberg Limits (ASTM D4318) tests, two (2) USCS Classification (ASTM D2487), two Standard Proctor (ASTM D698) tests, and two (2) Consolidated Undrained (CU) Triaxial Test with Pore Pressure Measurement Tests on soil (ASTM D4767), due to the soft cohesive soils was encountered in offset boring AB-2A within the crest of the dam., one (1) One-Dimensional Consolidation Test (ASTM D2435), one (1) Falling Head Permeability Test (ASTM D5084), and one (1) Specific Gravity (-10 Materials) (ASTM D854).

- Preparation of this written geotechnical data report for the project summarizing our work on the project, providing descriptions of the subsurface conditions encountered, laboratory testing program, visual classifications in accordance with USCS and adjusted based on the results of the laboratory testing, logs of all borings delineated the limits of stratum encountered (Fill and Embankment materials, Alluvium, Residuum, etc.), and the results of all field and laboratory testing for the design team and the planned construction.

Accura’s geotechnical services did not include topographic or field surveying (we note that we performed GPS surveying of boring locations and approximate ground elevation), development of quantity estimates, preparation of plans and specifications, or an environmental site assessment for determining the presence or absence of wetlands, hazardous or toxic materials in the soil, surface water, groundwater, or air on, below, or around the site.

3.0 EXPLORATION PROCEDURES

Five (5) Standard Penetration Test (SPT) borings were drilled at the approximate locations indicated on the attached Test Location Plan (Drawing No. 2) provided in Appendix I. Borings AB-2 and AB-4 were drilled within the crest and right abutment of dam to a depth of 46 feet, and boring AB-1 was drilled within the left abutments to an auger refusal depth of 8.5 feet. Borings AB-3 and AB-5 were drilled at downstream toe of dam to respective boring termination depths of about 26 feet and 36 feet below the existing grade, respectively.

Continuous split spoon sampling was performed during HSA drilling in general compliance with ASTM standards. Split-spoon and three (3) Shelby tube samples were returned to the laboratory for classification and boring log preparation.

Boring locations were determined and field-staked by utilizing a handheld GPS unit to obtain northing and easting coordinates and ground surface elevations at each location.

TABLE 3.1 - BORING REFERENCE

Boring	Ground Surface Elevation (ft)	Northing	Easting	Latitude	Longitude	Boring Depth (ft))
AB-1	969	1406712.375	2277147.032	33.8671	-84.2307	18.5
AB-2	968	1406791.693	2277279.083	33.8673	-84.2303	46
AB-3	942	1406860.962	2277234.674	33.8675	-84.2304	26
AB-4	969	1406867.725	2277426.385	33.8675	-84.2298	46
AB-5	960	1406876.421	2277335.013	33.8676	-84.2301	36

Soil samples obtained using the split spoon sampler were visually evaluated by the Project Engineer and classified according to the visual manual procedure described in ASTM D 2488.

Rock coring was performed below the auger refusal level in boring AB-1 at depths from 8.5 feet to 18.5 feet.

Core drilling procedures are utilized to determine the characteristics and continuity of materials below the soil drilling refusal level. The core drilling procedure is performed in general accordance with ASTM designation D 2113-70. Initially, casing is set through the overburden soils or hollow stem augers are utilized to keep the hole from collapsing. Refusal materials are then cored with a diamond-studded bit fastened to the end of a hollow core barrel. This device is rotated at high speeds and is capable of cutting through the hardest rock. The cuttings are brought to the surface by circulating water. Rock core samples of the materials penetrated are protected and retained in the inner core barrel. Upon completion of the drill run, the core barrel is brought to the surface and the samples are removed and placed in partitioned boxes. The samples are then returned to laboratory where the rock is identified, and "recovery" and rock quality designation (RQD) are determined.

The ratio of the length of core obtained to the distance drilled is known as the "core recovery" and expressed as a percentage. The "rock quality designation" (RQD) is the ratio of recovered rock sample in sections four or more inches long to the distance drilled. This designation is generally applied only to samples of NX size or larger and to samples described as moderately hard or harder. The NX size designates a bit which obtains core samples 2-1/8 inches in diameter. The percent recovery and RQD are related to rock soundness and continuity.

The boring locations and approximate ground elevation were determined in the field by our engineering representative by using a TRIMBLE GeoXH GPS. The approximate boring locations are indicated on the attached Test Location Plan (Drawing No. 2). The boring locations and elevations should, therefore, be considered approximate. The borings were backfilled with cement-bentonite upon completion of drilling.

4.0 REGIONAL GEOLOGY

The project site is located in the Piedmont physiographic province of Georgia, which is characterized by medium-to-high grade metamorphic rocks and scattered igneous intrusions. The metamorphic rocks comprising the Piedmont region were formed when older "parent" rocks were subjected to high temperatures and/or pressures during regional metamorphism that occurred during the creation of the Appalachian Mountains. The same high temperatures and pressures also caused some "parent" rocks to fully melt and subsequently recrystallize as intrusive igneous rocks. Topography in the province is variable and ranges from gently rolling hills in the south to moderate-to-steep hills in the north. Based on the Geologic Map of Georgia, Geological Survey of Georgia (1976), the project site is underlain by the Granite Gneiss/Amphibolite formation in the Piedmont region.

The boundary between soil and rock is typically not sharply defined. A transitional zone termed partially weathered rock (PWR) is normally found overlying bedrock. PWR is defined for engineering purposes as residual material that can be penetrated with soil drilling equipment, but which has a standard penetration resistance exceeding 100 blows per foot (bpf). Differential weathering of the parent rock has resulted in highly variable subsurface conditions over short horizontal distances. Lenses and boulders of hard rock and zones of PWR may be present within the soil above the general bedrock level. The upper surface of rock is irregular.

5.0 USDA SOIL SURVEY

According to the Natural Resources Conservation Services (NRCS) Soil Survey of DeKalb County, Georgia, primary soils in the area of the project site are mapped as Cartecay silt loam (Ca), Gwinnett sandy loam (GeE), and Pacolet-Urban land complex (PuE). The description, property, quality, and typical profile of these soils type are provided in the following table 5.1.

TABLE 5.1 – SUMMARY OF THE SERIES SOILS WITHIN THE PROJECT SITE

Map Unit Symbol	Map Unit Description	Properties			Soil Type	Hydrologic Soil Group	Seasonal High Groundwater Table (in)
		Slope %	Drainage Class	Hydraulic Permeability (in/hr)			
Ca	Cartecay silt loam	0 to 2	Somewhat poorly drained	1.98 to 5.95	Sandy Silt loam	A/D	6 to 18
GeE	Gwinnett sandy loam	15 to 30	Well drained	0.57 to 1.98	Sandy Clay loam	B	More than 80
PuE	Pacolet-Urban land complex	10 to 25	Well drained	0.57 to 1.98	Sandy Clay loam	B	More than 80

6.0 SUBSURFACE CONDITIONS

The subsurface conditions discussed in the following paragraphs and those shown on the attached boring logs represent an estimate of the subsurface conditions based on interpretation of the field using normally accepted geotechnical engineering judgments. Given the spacing between boring locations, it is anticipated that subsurface conditions may vary between each boring location. Strata breaks designated on the boring logs represent approximate boundaries between soil types. The transitions between different soil strata are usually less distinct than those shown on the boring logs. Although individual soil test borings are representative of the subsurface conditions at the boring locations on the dates shown, they are not necessarily indicative of subsurface conditions at other locations or at other times. Data from the specific soil test borings are shown on the individual boring logs included in Appendix II.

Crest of the Dam

Existing Fill: Below a thin two-inch layer of topsoil, boring AB-2 encountered fill soils to a depth of approximately 16 feet below the existing grade. The fill soils encountered in the boring generally consisted of loose to very loose silty sand (SM) with trace of clay and rock fragments, with Standard Penetration Test (SPT) resistances (N-values) ranging from 2 to 5 blows per foot (bpf).

Alluvial Soils: Alluvial soils are deposited by flowing water such as creeks, rivers, etc. Alluvium was encountered below the fill soils in boring AB-2 and extended to about 30 feet below the existing grade. The alluvium generally consisted of very loose silty sand (SM) with trace of clay or very loose to loose clayey sand (SC), with Standard Penetration Test (SPT) resistances (N-values) ranging from zero to 5 blows per foot (bpf), and very soft sandy clay (CL), with SPT N-values of about 1 bpf.

Residual Soils: Residual soils typical of the Piedmont Physiographic Region were encountered below the alluvium in boring AB-2. The residual soils generally consisted of very loose to medium dense silty sand (SM) with traces of clay and had SPT N-values that ranged from 1 to 24 bpf.

Groundwater: Groundwater was encountered in boring AB-2 at a depth of 26 feet after completion of drilling.

Right Abutment

Existing Fill: Below a thin 2-inch layer of topsoil, boring AB-4 encountered fill soils to a depth of approximately 12 feet below the existing grade. The fill soils encountered in the boring generally consisted of loose to medium dense silty sand with traces of clay and rock fragments, with Standard Penetration Test (SPT) resistances (N-values) ranging from 4 to 11 bpf.

Residual Soils: Residual soils was encountered below the fill soils in boring AB-4. The residual soils generally consisted of very loose to dense silty sand with traces of clay and rock fragments, with SPT N-values ranging from 2 to 31 bpf.

Partially Weathered Rock (PWR): PWR was encountered in boring AB-4 at a depth of 44 feet to terminated depth of 46 feet. PWR is a term for the residuum that can be penetrated by soil drilling techniques and has standard penetration resistance values (N-values) in excess of 100 bpf.

Groundwater: Groundwater was encountered in boring AB-4 at a depth of 9 feet after completion of drilling.

Left Abutment

Existing Fill: Below a thin 2-inch layer of topsoil, boring AB-1 encountered fill soils to a depth of approximately 6 feet below the existing grade. The fill soils encountered in boring generally consisted of loose to medium dense silty sand (SM), and Standard Penetration Test (SPT) resistances (N-values) ranging from 5 to 14 bpf.

Residual Soils: Residual soils were encountered below the fill soils in boring AB-1. The residual soils generally consisted of medium dense silty sand (SM) with trace of clay, with SPT N-values of 15 bpf.

Partially Weathered Rock (PWR): PWR was encountered in boring AB-1 at depths of about 8 feet to auger refusal depths of 8.5 feet. PWR is a term for the residuum that can be penetrated by soil drilling techniques and has standard penetration resistance values (N-values) in excess of 100 bpf.

Auger Refusal: Auger refusal materials was encountered in boring AB-1 at a depth of 8.5 feet below the existing grade, respectively. Auger refusal indicates the depth at which the boring cannot be drilled further using soil drilling tools and techniques. Auger refusal levels may represent the top of massive bedrock, a boulder or other hard obstruction.

Groundwater: No groundwater was encountered in boring AB-1 after completion of drilling.

Downstream Toe Area

Existing Fill: Below a thin 2-inch layer of topsoil, boring AB-3 and AB-5 encountered fill soils to a depth of approximately 12 feet and 10 feet below the existing grade, respectively. The fill soils encountered in the boring generally consisted of loose silty/clayey sand (SM/SC) and Standard Penetration Test (SPT) resistances (N-values) ranging from 4 to 10 blows per foot (bpf) or firm sandy silt (SM) with some clay and N-values of about 6 bpf .

Residual Soils: Residual soils were encountered below the fill soils in borings AB-3 and AB-5. The residual soils generally consisted of loose to dense silty sand (SM) with traces of clay and SPT N-values ranging from 4 to 37 bpf.

Groundwater: Groundwater was encountered in borings AB-3 and AB-5 at a depth of 11 feet and 18 feet after completion of drilling, respectively.

A summary of subsurface conditions encountered at the test locations is provided in the following Table 6.1.

TABLE 6.1 – SUMMARY OF SUBSURFACE CONDITIONS

Boring Location	Existing Ground Elevation (ft)	Depth to PWR (ft)	PWR Elevation (ft)	Auger Refusal Depth (ft)	Auger Refusal Elevation (ft)	GW Elevation After Completion of Drilling (ft)	Existing Fill/Alluvium (ft)
AB-1	969	8	961	8.5	960.5	N/E	6/0
AB-2	968	N/E	N/E	N/E	N/E	942	16/14
AB-3	942	22	920	N/E	N/E	931	12/0
AB-4	969	44	925	N/E	N/E	960	12/0
AB-5	960	N/E	N/E	N/E	N/E	942	10/0

N/E – Not Encountered
Note: The elevations indicated on this table are provided by Accura's Engineer by using GPS and should be considered approximate. The provided elevations are for general informational purposes only. Subsurface conditions can vary considerably within short horizontal distances in this geology.

It should be noted that the groundwater levels fluctuate depending upon seasonal factors such as precipitation and temperature. As such, soil moisture and groundwater conditions at other times may vary from those described in this report. Accura notes that due to the presence relatively impervious silty soils noted on the project site, trapped or perched water conditions may be encountered during periods of inclement weather and during seasonally wet periods.

Rock coring was performed below the auger refusal levels in boring AB-1 at depths ranging from 8.5 to 18.5 feet. The following table provide a summary of the core recoveries and the rock quality designations (RQD) in the core hole.

TABLE 6.2 – SUMMARY OF ROCK CORE DATA

Core location AB-1			
Core Run Number	**Depth (feet)**	**Recovery (%)**	**RQD (%)**
Run 1	8.5-13.5	100	100
Run 2	13.5-18.5	100	100

The results of the rock coring indicate generally excellent rock quality. The definition of the terms used related to rock quality is provided in the following.

An RQD ratio of 90 percent or more denotes excellent rock, 75 to 90 percent denotes good rock, 50 to 75 percent denotes fair rock, and 25 to 50 percent denotes poor rock. Hardness terms are based on the following descriptions:

Soft:	May be broken with fingers

Moderately Soft:	May be scratched with a nail, corners and edges may be broken with fingers
Moderately Hard:	Light blow of hammer required to break sample
Hard:	Hard blow of hammer required to break sample
Very Hard:	Rock core rings when struck with hammer

The borings were backfilled with the cement-bentonite grout upon completion of drilling for safety considerations. For a more precise description of the conditions encountered within the borings, please refer to the Boring Logs provided in The Appendix II.

Test Pit Area

As requested, Accura Engineering visited the above referenced site on August 3, 2021, to observe the excavation of test pits in two selected locations on the downstream slope of dam.

Accura subcontracted a Takeuchi TB370 Compact Excavator and operator for the work. The first test pit, TP-1 was excavated to locate the T-intersection of the lateral dam toe drains with the outflow pipe as shown in Drawing No. 2 and to locate a second open lateral drainpipe. In order to locate the lateral drainpipes, the test pit had to be excavated into the dam, approximately 30 feet in length, while following the outflow pipe back to the intersection. The second test pit, TP-2, was excavated to identify any potential seepage issues occurring under eroded soils; none were discovered. Soil samples were obtained by the AECOM representative on site during test pit excavation and the soils in the test pit areas were observed by the Geotechnical Engineer and classified according to the visual-manual procedure described in ASTM D 2488-00.

Below the existing grade, the test pits TP-1 and TP-2 encountered fill soils to termination depths ranging from 3 feet to 8 feet over the full length of TP-1 and at 7 feet in TP-2. The fill encountered in the test pits within the subject areas generally consisted of silty sand with traces of clay, roots, and rock fragments.

Alluvial soils, deposited by flowing water such as creek, river, etc., were encountered below the fill in test pits TP-1 and TP-2 to termination depths ranging from 4 feet to 12 feet over the full length of TP-1 and at 7 feet in TP-2. The alluvial soil was described as sandy silt with some clay, roots, rock fragments, and wood chips. No groundwater was encountered in the test pits. For a more precise description of the conditions encountered in our test pits, please refer to the summary of test pit data on the next page.

TABLE 6.3 – SUMMARY OF TEST PIT DATA

<table>
<tr><th colspan="5">Test Pit TP-1, at Start of Trench</th></tr>
<tr><th>Depth in Feet</th><th>Soil Description</th><th>Depth to Refusal (feet)</th><th>Depth to Groundwater (feet)</th><th>Comment</th></tr>
<tr><td>0'-3'</td><td>Sand-silty, trace clay and roots, brown</td><td>NE</td><td>NE</td><td>Fill</td></tr>
<tr><td>3'-4'</td><td>Silt-sandy, some clay, roots, and wood chips, gray</td><td>NE</td><td>NE</td><td>Alluvial</td></tr>
<tr><td colspan="5">Test Pit Terminated @ 4 feet to exposure of outflow pipe</td></tr>
<tr><th colspan="5">Test Pit TP-1, at End of Trench</th></tr>
<tr><th>Depth in Feet</th><th>Soil Description</th><th>Depth to Refusal (feet)</th><th>Depth to Groundwater (feet)</th><th>Comment</th></tr>
<tr><td>0-2</td><td>Sand-silty, trace clay, brown</td><td>NE</td><td>NE</td><td>Fill</td></tr>
<tr><td>2-4</td><td>Sand-silty, trace clay, brown</td><td>NE</td><td>NE</td><td>Fill</td></tr>
<tr><td>4-6</td><td>Sand-silty, traces of clay, roots, and rock fragments, brown</td><td>NE</td><td>NE</td><td>Fill</td></tr>
<tr><td>6-8</td><td>Sand-silty, traces of clay, roots, and rock fragments, brown</td><td>NE</td><td>NE</td><td>Fill</td></tr>
<tr><td>8-10</td><td>Silt-sandy, some clay, roots, and wood chips, gray</td><td>NE</td><td>NE</td><td>Alluvial</td></tr>
<tr><td>10-12</td><td>Silt-sandy, some clay, roots, and wood chips, gray</td><td>NE</td><td>NE</td><td>Alluvial</td></tr>
<tr><td colspan="5">Test Pit Terminated @ 12 feet to exposure of outflow pipe</td></tr>
<tr><th colspan="5">Test Pit TP-2</th></tr>
<tr><th>Depth in Feet</th><th>Soil Description</th><th>Depth to Refusal (feet)</th><th>Depth to Groundwater (feet)</th><th>Comment</th></tr>
<tr><td>0-2</td><td>Sand-silty, trace clay and roots, brown</td><td>NE</td><td>NE</td><td>Fill</td></tr>
<tr><td>2-4</td><td>Sand-silty, trace clay and roots, brown</td><td>NE</td><td>NE</td><td>Fill</td></tr>
<tr><td>4-6</td><td>Sand-silty, trace clay and roots, some rocks, brown</td><td>NE</td><td>NE</td><td>Fill</td></tr>
<tr><td>6-7</td><td>Silt-sandy, some clay and roots, organics and wood chips, gray</td><td>NE</td><td>NE</td><td>Alluvial</td></tr>
<tr><td colspan="5">Test Pit Terminated @ 7 feet</td></tr>
</table>

TABLE 6.4 – NUCLEAR GAUGE READINGS FOR COMPACTION

Location	Wet Density (pcf)	Dry Density (pcf)	Percent of Dry Density (%)
Target Point	114.0	88.0	-
TP-1	113.0	86.8	99%
TP-1	114.3	88.1	100%
TP-2	107.4	80.8	92%
TP-2	119.3	91.1	104%

The test pits were backfilled pits immediately after completion of the excavation for safety with the excavated soils stockpiled beside test pits. The backfill of native silty soil which were segregated from fill soils during excavation were replaced first to the original depth and tamped (compacted) with the excavator bucket to the in-situ density of nearby similar soils as per the direction of the representative on site. The remaining fill soils were replaced in 12-inch loose lifts and tamped (compacted) with the backhoe-curling bucket, and the density was confirmed to be greater than 95% of the in-situ soil density, which was measured at 114 pcf, using a nuclear gauge to test the fill soil every 12-inches of compaction.

7.0 LABORATORY TEST PROGRAM

Laboratory tests were performed on selected samples obtained from the area explored. The laboratory testing program included fifteen (15) Natural Moisture Content tests (ASTM D2216), seven (7) Grain Size with Hydrometer tests (ASTM D422), seven (7) Grain Size Analysis (ASTM D6913), eight (8) Atterberg Limits tests (ASTM D4318) on selected samples, two (2) Consolidated Undrained (CU) Triaxial Test with Pore Pressure Measurement Tests (ASTM D4767) on a relatively undisturbed Shelby tube sample, two (2) Standard Proctor Tests (ASTM D698), one (1) One-Dimensional Consolidation Test (ASTM D2435), one (1) Falling Head Permeability Test (ASTM D5084), and one (1) Specific Gravity (-10 Materials) (ASTM D854). The results of the Moisture Content and Atterberg Limits tests are indicated on the boring logs at the respective sample locations. Narrative descriptions of the laboratory tests are included in the Appendix. The results of the laboratory-testing program are summarized below:

TABLE 7.1 – LABORATORY CLASSIFICATION TEST RESULTS

Sample Type	Boring No	Depth (feet)	Natural Moisture Content (%)	Atterberg Limits (%)			Particle-Size Analysis (%)				USCS
				LL	PL	PI	Gravel	Sand	Silt	Clay	
JAR	AB-1	2 – 4	14.9	NP	NP	NP	4.4	60.5	35.1		SM
JAR	AB-2	2 – 4	20.7	NP	NP	NP	0.0	57.9	27.7	14.4	SM
JAR	AB-2	6 – 8	25.0	-	-	-	0.1	61.1	38.8		SM
JAR	AB-2	16 – 18	21.0	NP	NP	NP	9.8	47.6	21.7	20.9	SM
JAR	AB-2	20 – 22	25.4	37	21	16	0.0	46.9	27.3	25.8	CL
JAR	AB-2	26 – 28	19.5	29	21	8	2.8	57.5	22.4	17.3	SC

Sample Type	**Boring No**	**Depth (feet)**	**Natural Moisture Content (%)**	**Atterberg Limits (%)**			**Particle-Size Analysis (%)**				**USCS**
				LL	**PL**	**PI**	**Gravel**	**Sand**	**Silt**	**Clay**	
JAR	AB-2	30 – 32	21.8	-	-	-	0.0	77.4	22.6		SM
JAR	AB-3	0 - 2	21.1	NP	NP	NP	3.2	51.7	45.1		SM
JAR	AB-3	6 - 8	28.5	32	22	10	0.0	52.4	26.8	20.8	SC
JAR	AB-3	12 - 14	49.5	NP	NP	NP	0.0	68.3	31.7		SM
JAR	AB-4	4 – 6	13.8	-	-	-	0.0	59.7	27.7	12.6	SM
JAR	AB-4	18 – 20	27.6	NP	NP	NP	0.0	64.4	25.6		SM
JAR	AB-5	0 - 2	20.3	-	-	-	3.4	57.6	21.1	17.9	SM
JAR	AB-5	10 - 12	33.2	NP	NP	NP	0.0	68.7	31.3		SM

LL: Liquid limit **USCS:** Unified Soil Classification System **PI**: Plasticity index
PL: Plastic limit **NP:** Non-Plastic

TABLE 7.2 – SUMMARY OF TRI-AXIAL SHEAR TEST RESULTS

Sample Type	**Boring No.**	**Sample Depth (feet)**	**Tri-axial Shear Test (Consolidated Undrained with Pore Water Pressure)**			
			Total		**Effective**	
			c psi	**φ Deg**	**c' psi**	**φ ' Deg**
UD	AB-2A	22-24	2.09	14.9	0.232	28.2
UD	AB-3	7-9	1.75	14.6	0.0784	26.6

φ = Angle of Internal Friction (Total)
c = Cohesion (Total)
φ′ = Angle of Internal Friction (Effective)
c′ = Cohesion (Effective)

TABLE 7.3 – STANDARD PROCTOR TEST RESULTS

Sample Type	**Boring No**	**Depth (feet)**	**Maximum Dry Density (pcf)**	**Optimum Moisture Content (%)**
BULK#1	Combination AB-1, AB-2, and AB-4	0-10	105.9	18.5
BULK#2	Combination AB-3 and AB-5	0-10	103.6	14.7

Accura conducted a total of one (1) Permeability test (ASTM D5084) on a relatively undisturbed Shelby tube sample obtained from boring AB-2A at a depth from 10 feet to 12 feet within the crest of dam in order to provide the soil permeability value for use. The result of the permeability test is summarized below:

TABLE 7.4 – PERMEABILITY TEST RESULT

Sample Type	Boring No	Depth (ft)	Hydraulic Conductivity (Permeability) (cm/sec)
UD	AB-2A	10 – 12	2.7×10^{-7}

Laboratory testing program also included a total of one (1) unconfined compressive strength test of selected core rock specimens from boring AB-1. A summary of unconfined compressive strength test result is provided in the following Table 7.5.

Table 7.5 – Unconfined Compressive Strength Test Results

Boring No.	Sample Depth (Feet)	Unconfined Compressive Strength of Rock (psi)
AB-1	8.5-18.5	12,570

Unconfined compressive strengths of about 12,570 psi, which is generally indicative of high strength rock. Narrative descriptions of the laboratory tests are included in the Appendix.

Laboratory test results are provided in Appendix III.

8.0 LIMITATIONS

This report has been prepared for **AECOM**, in accordance with generally accepted geotechnical engineering practice. No other warranty, express or implied, is made. Our conclusions and findings are based on information furnished to us at the time the work was performed. The findings do not reflect variations in subsurface conditions, which could exist in unexplored areas of the site. In areas where variations from the available subsurface data become apparent during construction, it will be necessary to re-evaluate our conclusions based upon on-site observations of the conditions.

9.0 CLOSURE

We appreciate the opportunity to assist you with this project. Please contact us if you have any questions or if we can be of further assistance.

Sincerely,

ACCURA ENGINEERING AND CONSULTING SERVICES, INC.

Ken Khanidokht, P.E.
Geotechnical Group Leader-Senior Engineer

GEORGIA
REGISTERED
NO. PE032545
PROFESSIONAL
ENGINEER
KHEIBAR KHANIDOKHT

Justin Lawrence, EIT
Civil Engineer

APPENDIX I

Site Vicinity Map (Drawing No. 01)

Test Location Plan (Drawing No. 02)

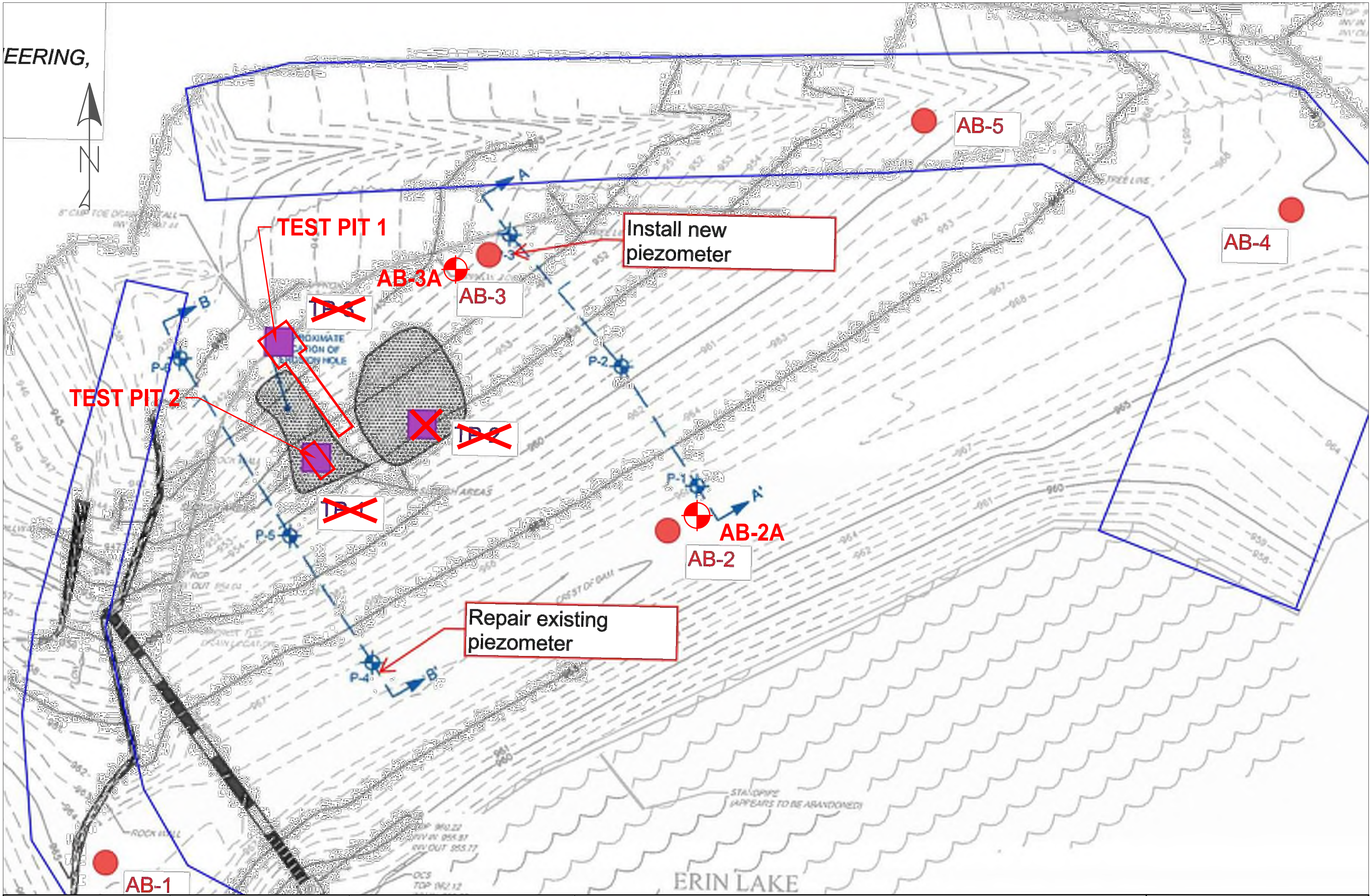
Subsurface Profile (Drawing No. 03-01)

Subsurface Profile (Drawing No. 03-02)

Subsurface Profile (Drawing No. 03-03)

SCALE: AS SHOWN	DATE: 08/20/2021	PROJECT NO: 10205	**TITLE:** **SITE VICINITY MAP** ERIN LAKE DAM DEKALB COUNTY, GEORGIA
PREPARED: JL	CHECKED: KK	REVISIONS:	
CLIENT: **AECOM**			**ACCURA ENGINEERING AND CONSULTING SERVICES, INC.** 3200 PRESIDENTIAL DR ATLANTA, GA 30340 PHONE: 404.241.8722 FAX: 404.241.4577

DRAWING NO. 01



SCALE: AS SHOWN
DATE: 08/20/2021
PROJECT NO: 10205
PREPARED: JL
CHECKED: KK
REVISIONS:
CLIENT: **AECOM**

TITLE:

TEST LOCATION PLAN
ERIN LAKE DAM
DEKALB COUNTY, GEORGIA

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3200 PRESIDENTIAL DR ATLANTA, GA 30340
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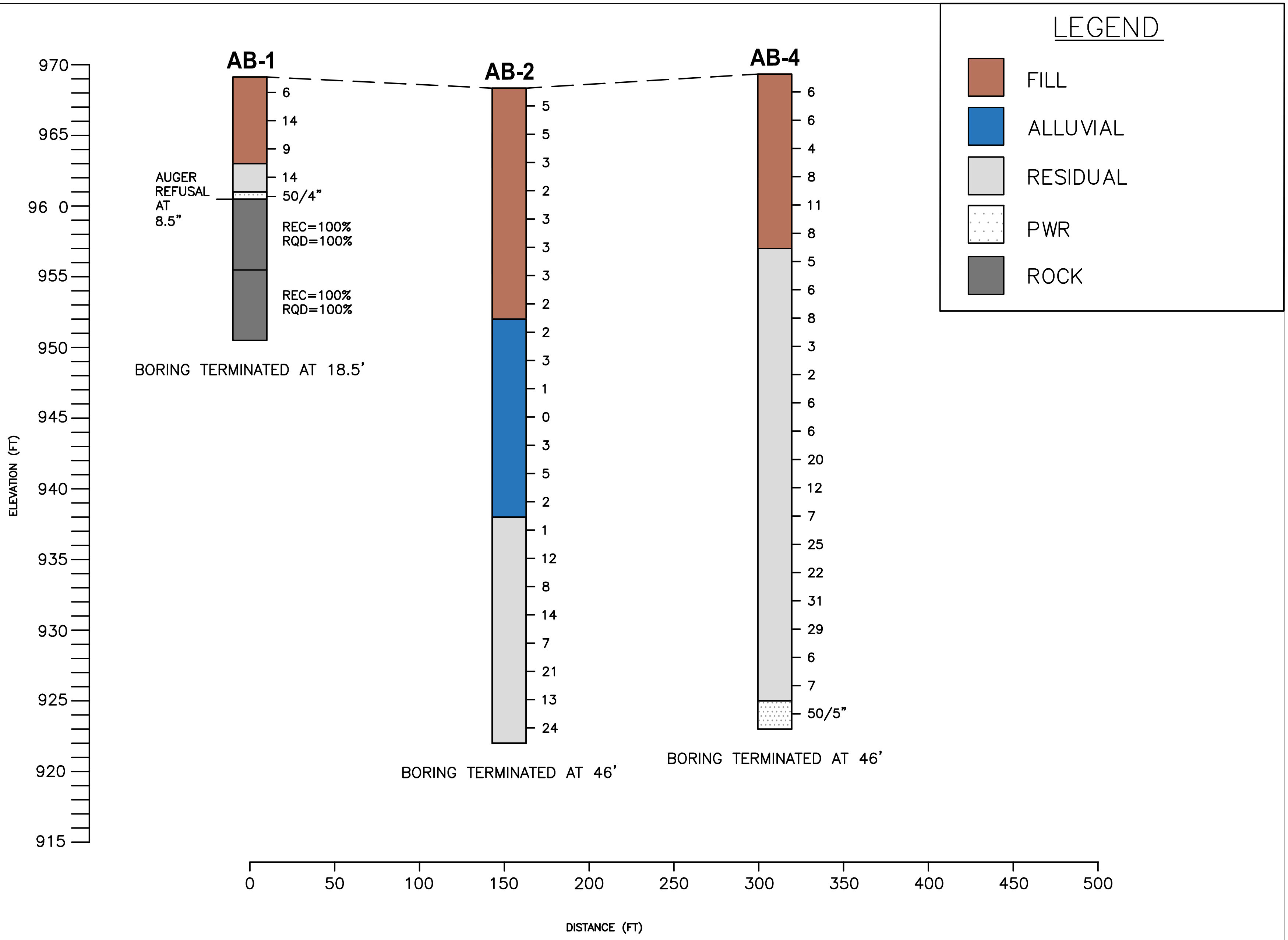
DRAWING NO. 02



SCALE: AS SHOWN
DATE: 08/27/2021
PROJECT NO: 10205
PREPARED: JL
CHECKED: KK
REVISIONS:
CLIENT: **AECOM**

TITLE:

SUBSURFACE PROFILE
ERIN LAKE DAM
DEKALB COUNTY, GEORGIA

ACCURA ENGINEERING AND CONSULTING SERVICES, INC.
3200 PRESIDENTIAL DR ATLANTA, GA 30340
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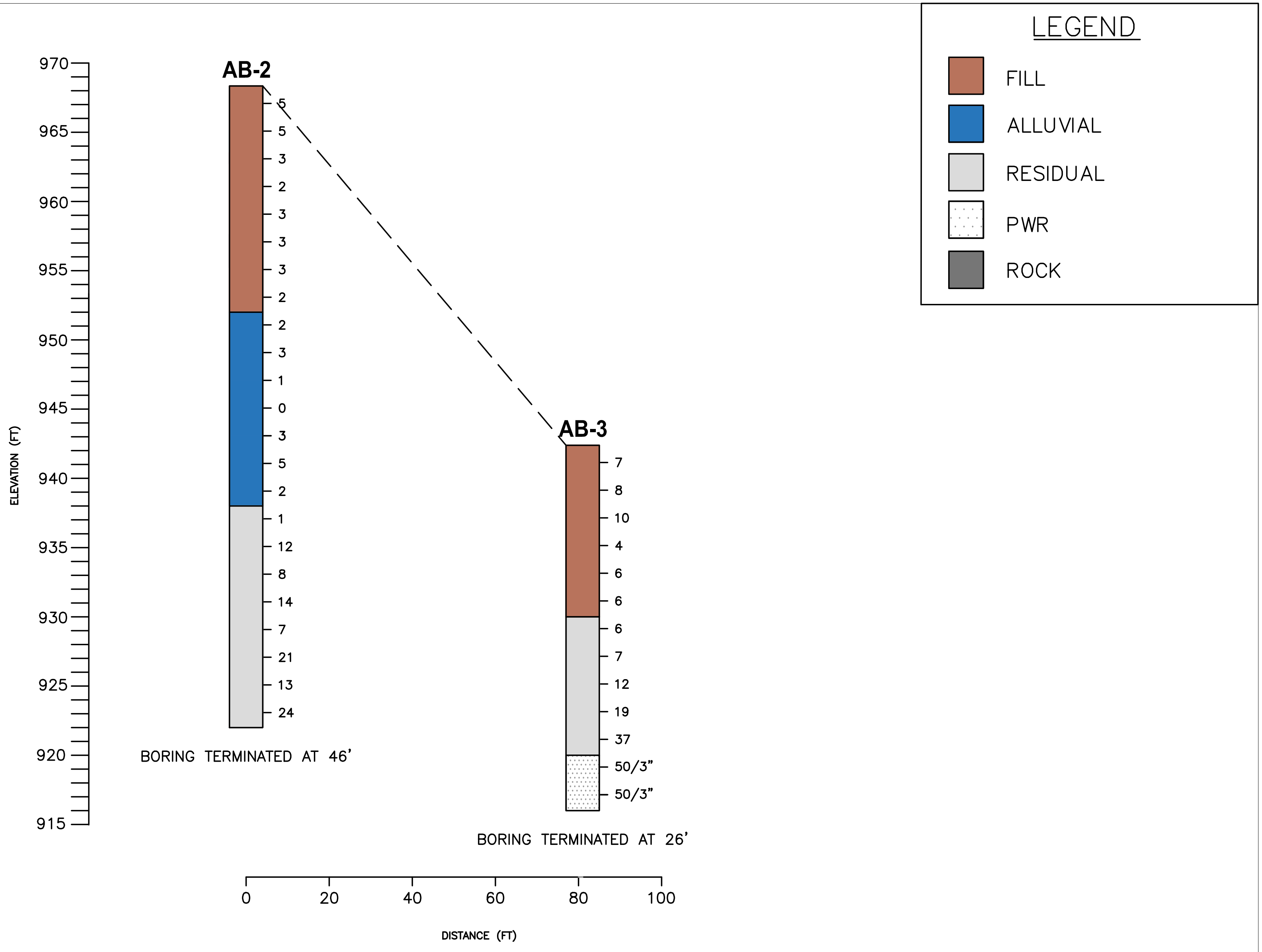
DRAWING NO. 03-01



SCALE: AS SHOWN
DATE: 08/27/2021
PROJECT NO: 10205
PREPARED: JL
CHECKED: KK
REVISIONS:
CLIENT: **AECOM**

TITLE: **SUBSURFACE PROFILE**
ERIN LAKE DAM
DEKALB COUNTY, GEORGIA

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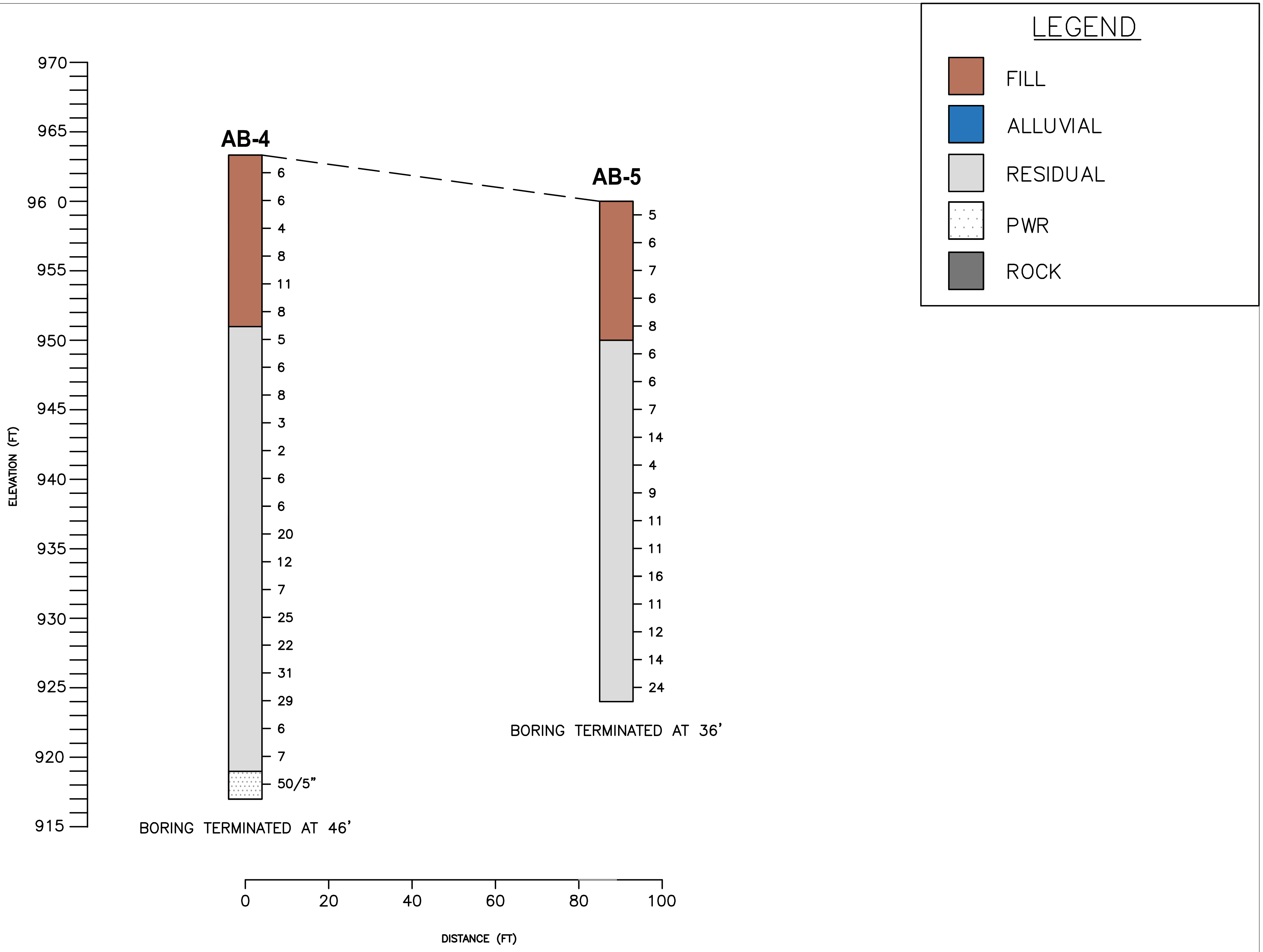
DRAWING NO. 03-02



SCALE: AS SHOWN	DATE: 08/27/2021	PROJECT NO: 10205	**TITLE:** **SUBSURFACE PROFILE** ERIN LAKE DAM DEKALB COUNTY, GEORGIA
PREPARED: JL	CHECKED: KK	REVISIONS:	
CLIENT: **AECOM**			**ACCURA ENGINEERING AND CONSULTING SERVICES, INC.** 3200 PRESIDENTIAL DR ATLANTA, GA 30340 PHONE: 404.241.8722 FAX: 404.241.4577

DRAWING NO. 03-03

APPENDIX II

Boring Logs (AB-1 through AB-5, AB-2A and AB-3A)

Key to Soil Classification

Soil Classification Chart

Rock Coring Photographs (1 page)

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Sheet 1 of 1

BORING LOG

CONTRACTED WITH: AECOM
BORING NO.: AB-1
PROJECT NAME: Erin Lake Dam
DATE: 7/15/2021
JOB NO.: 10205
DRILLER: GEOLAB
RIG: GEOPROBE
LOGGED BY: SY

ELEV.	DESCRIPTION	DEPTH in FEET	SAMPLES					NOTES
			NO.	TYPE	BLOWS/6"	RECOV.	W	
970	2" TOPSOIL	0						
	FILL: Sand-silty, trace clay and roots, brown, loose (SM)		1	◤	1-2-3-4	24"		Lat: 33.8671211043 Long: -84.2307046105
	-medium dense		2	◤	11-8-6-7	22"	15	LL=NP, PL=NP, PI=NP
965	-loose	5	3	◤	8-5-4-9	23"		
	RESIDUAL: Sand-silty, trace clay, white and gray, medium dense (SM)		4	◤	8-4-11-50/3"	19"		Auger refusal at 8.5 feet
	Partially Weathered Rock Sampled as Sand-silty, trace clay, rock fragments, white gray, very dense		5	◤	50/4"	2"		Start 1st core run at 8.5 feet
960	Hard to hard black and white, solid, continuous granite	10	6					REC=100% RQD=100%
955	Hard to hard black and white, solid, continuous granite	15	7					End 1st core run at 13.5 feet Start 2nd core run at 13.5 feet REC=100% RQD=100%
950	ROCK CORING TERMINATED AT 18.5 FEET	20						End 2nd core run at 18.5 feet
945		25						Bulk Sample from 0'-10' combination of Boring AB-1, AB-2 and AB-4 Max. Dry Density= 105.9 pcf Optimum Moisture=18.5%
940		30						
935		35						
930		40						LL=Liquid Limit PL=Plastic Limit PI=Plasticity Index NP=None Plastic

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BORING LOG

CONTRACTED WITH: AECOM
BORING NO.: AB-2
PROJECT NAME: Erin Lake Dam
DATE: 07/15/2021
JOB NO.: 10205
DRILLER: GEOLAB
RIG: GEOPROBE
LOGGED BY: SY

ELEV.	DESCRIPTION	DEPTH in FEET	SAMPLES					NOTES
			NO.	TYPE	BLOWS/6"	RECOV.	W	
	2" TOPSOIL	0						
	FILL: Sand-silty, trace clay, trace rock fragments, tan, loose (SM)		1	◩	2-2-3-3	20"		Lat: 33.8673365235 Long:-84.2302673286
965	-		2	◩	2-2-3-3	22"	21	LL=NP, PL=NP, PI=NP
	-very loose	5	3	◩	2-2-1-1	23"		
	-		4	◩	3-1-1-2	24"	25	Bulk Sample from 0'-10' combination of Boring AB-1, AB-2 and AB-4 Max. Dry Density= 105. 9 pcf Optimum Moisture=18.5
960	-orange and brown	10	5	◩	1-1-2-3	24"		
	-		6	◩	1-1-2-2	27"		
955	-some clay, gray		7	◩	1-1-2-1	17"		
	-wet	15	8	◩	0-1-1-1	18"		
	ALLUVIUM: Sand-silty, trace clay, gray, very loose, moist (SM)		9	◩	1-1-1-3	24"	21	LL=NP, PL=NP, PI=NP
950	-	20	10	◩	0-1-2-5	24"		
	Clay-sandy, gray, very soft, moist (CL)		11	◩	0-1-0-1	24"	25	LL=37, PL=21, PI=16
945	Sand-clayey, some silt, gray and brown, very loose, trace roots (SC)		12	◩	1-0-0-4	24"		
	-	25	13	◩	1-1-2-3	24"		Groundwater was encountered at 26 feet after completion of drilling
	-loose	▽	14	◩	0-3-2-2	24"	20	LL=29, PL=21, PI=8
940	-very loose, gray	30	15	◩	0-1-1-1	24"		
	RESIDUAL: Sand-silty, trace clay, brown and gray, very loose, wet (SM)		16	◩	0-0-1-3	20"	22	
935	-medium dense		17	◩	0-7-5-6	15"		
	-white, loose, wet	35	18	◩	0-2-6-8	24"		
	-medium dense		19	◩	4-6-8-10	24"		
930	-loose	40	20	◩	3-2-5-13	24"		LL=Liquid Limit PL=Plastic Limit
	-medium dense		21	◩	5-8-13-16	24"		PI=Plasticity Index NP=None Plastic

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Sheet 2 of 2

BORING LOG

CONTRACTED WITH: AECOM
BORING NO.: AB-2
PROJECT NAME: Erin Lake Dam
DATE: 07/15/2021
JOB NO.: 10205
DRILLER: GEOLAB
RIG: GEOPROBE
LOGGED BY: SY

ELEV.	DESCRIPTION	DEPTH in FEET	SAMPLES					NOTES
			NO.	TYPE	BLOWS/6"	RECOV.	W	
925	-		22	◤	4-6-7-8	24"		
	-	45	23	◤	4-10-14-21	24"		
	BORING TERMINATED AT 46 FEET							
920								
		50						
915								
		55						
910								
		60						
905								
		65						
900								
		70						
895								
		75						
890								
		80						
885								

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BORING LOG

CONTRACTED WITH: AECOM
BORING NO.: AB-2A
PROJECT NAME: Erin Lake Dam
DATE: 8/9/2021
JOB NO.: 10205
DRILLER: GEOLAB
RIG: GEOPROBE
LOGGED BY: SY

ELEV.	DESCRIPTION	DEPTH in FEET	SAMPLES					NOTES
			NO.	TYPE	BLOWS/6"	RECOV.	W	
		0						
965 960	Straight Auger to 10 feet	5		■				Offset 2' east of AB-2
	UD from 10'-12'	10	1			24"		
955 950	Straight Auger to 20 feet	15		■				
	Attempted UD from 20'- 22' No Recovery	20	2			0		
945	UD from 22'-24'		3			24"		
940 935 930	AUGER TERMINATED AT 24 FEET	25 30 35 40						

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BORING LOG

CONTRACTED WITH: AECOM
BORING NO.: AB-3
PROJECT NAME: Erin Lake Dam
DATE: 8/9/2021
JOB NO.: 10205 DRILLER: John Kilman RIG: CME 55
LOGGED BY: JL

ELEV.	DESCRIPTION	DEPTH in FEET	SAMPLES					NOTES
			NO.	TYPE	BLOWS/6"	RECOV.	W	
	2" TOPSOIL	0						
	FILL: Sand-silty, trace clay, reddish, loose (SM)		1	◤	2-3-4-4	20"		Lat:33.8675 Long:-84.2304
940	-gray		2	◤	5-4-4-5	14"	21	LL=NP, PL=NP, PI=NP
	-	5	3	◤	6-5-5-5	18"		
935	-clayey, light brown (SC)		4	◤	3-2-2-3	12"	29	LL=32, PL=22, PI=10
	Silt-sandy, some clay, reddish brown, firm (SM)	10	5	◤	3-3-3-3	18"		
		▽	6	◤	3-3-3-5	12"		Groundwater was encountered at 11 feet after completion of drilling
930	RESIDUAL: Sand-silty, trace clay, dark blue and white, loose (SM)		7	◤	3-3-3-4	16"	50	wet
	-reddish	15	8	◤	3-3-4-4	18"		Bulk Sample from 0'-10' combination of Boring AB-3 and AB-5
925	-medium dense		9	◤	6-6-6-9	18"		Max. Dry Density= 103.6 pcf Optimum Moisture=14.7%
	-	20	10	◤	6-9-10-12	18"		
	-dense		11	◤	12-17-20-31	10"		
920	Partially Weathered Rock Sampled as sand-silty, trace rock fragments, gray and white, very dense		12	◤	23-31-50/3	10"		
	-	25	13	◤	20-25-50/3	10"		
915	BORING TERMINATED AT 26 FEET							
		30						
910								
		35						
905								
	-	40						LL=Liquid Limit PL=Plastic Limit PI=Plasticity Index NP=None Plastic

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BORING LOG

CONTRACTED WITH: AECOM
BORING NO.: AB-3A
PROJECT NAME: Erin Lake Dam
DATE: 8/9/2021
JOB NO.: 10205
DRILLER: John Kilman
RIG: CME 55
LOGGED BY: JL

ELEV.	DESCRIPTION	DEPTH in FEET	SAMPLES					NOTES
			NO.	TYPE	BLOWS/6"	RECOV.	W	
	Straight Auger to 7 feet	0		■				Offset 2' west of AB- 3
940				■				
		5		■				
935	UD from 7'-9'					24"		
	AUGER TERMINATED AT 9 FEET							
		10						
930								
		15						
925								
		20						
920								
		25						
915								
		30						
910								
		35						
905								
		40						

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BORING LOG

CONTRACTED WITH: AECOM
BORING NO.: AB-4
PROJECT NAME: Erin Lake Dam
DATE: 7/12/2021
JOB NO.: 10205
DRILLER: GEOLAB
RIG: GEOPROBE
LOGGED BY: JL

ELEV.	DESCRIPTION	DEPTH in FEET	SAMPLES					NOTES
			NO.	TYPE	BLOWS/6"	RECOV.	W	
970	2" TOPSOIL	0						
	FILL: Sand-silty, some clay, brown, loose (SM)		1	◤	3-2-3-3	24"		Lat: 33.8675448452 Long: -84.2297820487
	-trace clay, tan		2	◤	4-4-2-4	20"	14	
965	-	5	3	◤	3-2-2-3	20"		
	-		4	◤	3-3-5-8	22"		
960	-trace rock fragments, medium dense	▽ 10	5	◤	5-7-4-4	19"		Groundwater was encountered at 9 feet after completion of drilling
	-loose, moist		6	◤	6-4-4-6	18"		
	RESIDUAL: Sand-silty, trace clay and mica, gray and brown, loose, moist (SM)		7	◤	2-2-3-4	24"		Bulk Sample from 0'-10' combination of Boring AB-1, AB-2 and AB-4 Max. Dry Density= 105. 9 pcf Optimum Moisture=18.5
955	-	15	8	◤	3-3-3-3	24"		
	-		9	◤	2-3-5-6	19"		
950	-very loose	20	10	◤	1-1-2-5	22"	28	
	-wet		11	◤	0-0-2-2	19"		
	-trace rock fragments, loose, wet		12	◤	0-3-3-3	24"		
945	-gray and dark brown	25	13	◤	4-3-3-5	24"		
	-medium dense		14	◤	3-8-12-16	24"		
940	-	30	15	◤	0-4-8-11	24"		
	-white tan and brown, loose, moist		16	◤	0-1-6-13	24"		
	-medium dense		17	◤	6-10-15-20	24"		
935	-	35	18	◤	2-6-16-25	24"		
	-white and brown, dense		19	◤	8-15-16-24	24"		
930	-medium dense	40	20	◤	14-13-16-19	24"		
	-loose		21	◤	0-0-6-14	24"		

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BORING LOG

CONTRACTED WITH: AECOM
BORING NO.: AB-4
PROJECT NAME: Erin Lake Dam
DATE: 7/12/2021
JOB NO.: 10205
DRILLER: GEOLAB
RIG: GEOPROBE
LOGGED BY: JL

ELEV.	DESCRIPTION	DEPTH in FEET	SAMPLES					NOTES
			NO.	TYPE	BLOWS/6"	RECOV.	W	
	-loose		22	◣	2-4-3-14	24"		
925	Partially Weathered Rock Sampled as sand-silty, trace clay, white and tan, very dense	45	23	◣	3-50/5"	12"		
	BORING TERMINATED AT 46 FEET							
920		50						
915		55						
910		60						
905		65						
900		70						
895		75						
890		80						
885								

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Sheet 1 of 1

BORING LOG

CONTRACTED WITH: AECOM
BORING NO.: AB-5
PROJECT NAME: Erin Lake Dam
DATE: 8/9/2021
JOB NO.: 10205
DRILLER: John Kilman
RIG: CME 55
LOGGED BY: JL

ELEV.	DESCRIPTION	DEPTH in FEET	SAMPLES					NOTES
			NO.	TYPE	BLOWS/6"	RECOV.	W	
960	2" TOPSOIL	0						
	FILL: Sand-silty, trace clay, reddish, loose (SM)		1	◩	1-2-3-3	18"		Lat:33.8676 Long:-84.2301
	-gray		2	◩	3-2-4-4	18"	20	
955	-	5	3	◩	5-3-4-5	24"		
	-		4	◩	3-3-3-4	24"		
950		10	5	◩	5-5-3-5	22"		
	RESIDUAL: Sand-silty, trace clay, reddish and gray, loose (SM)		6	◩	4-3-3-3	24"	33	LL=NP. PL=NP, PI=NP
			7	◩	3-3-3-3	24"		
945	-	15	8	◩	3-3-4-5	22"		
	-dark gray, medium dense		9	◩	4-6-8-6	18"		Groundwater was encountered at 18 feet after completion of drilling
	-loose	▽	10	◩	2-2-2-4	18"		
940	-reddish	20	11	◩	2-3-6-6	18"		Bulk Sample from 0'-10' combination of Boring AB-3 and AB-5
	-medium dense		12	◩	6-6-5-5	20"		Max. Dry Density= 103. 6 pcf Optimum Moisture=14.7%
935	-	25	13	◩	3-4-7-7	18"		
	-		14	◩	8-8-8-8	20		
	-	30	15	◩	4-5-6-8	20		
930	-gray		16	◩	4-6-6-8	16		
	-		17	◩	4-6-8-12	16		
925	-	35	18	◩	12-12-12-14	18		
	BORING TERMINATED AT 36 FEET							
920		40						LL=Liquid Limit PL=Plastic Limit PI=Plasticity Index NP=None Plastic

KEY TO BORING LOG SOIL CLASSIFICATION

Particle Size and Proportion

Verbal descriptions are assigned to each soil sample or stratum based on estimates of the particle size of each component of the soil and the percentage of each component of the soil.

Particle Size Descriptive Terms		Proportion Descriptive Terms		
Soil Component	Particle Size	Component	Term	Percentage
Boulder	> 12 inch	Major	Uppercase Letters	>50%
Cobble	3 – 12 inch		(e.g., SAND, CLAY)	
Gravel-Coarse	¾ - 3 inch			
-Fine	#4 – ¾ inch	Secondary	Adjective	20%-50%
Sand-Coarse	#10 - #4		(e.g. sandy, clayey)	
-Medium	#40 - #10			
-Fine	#200 - #40	Minor	Some	15%-25%
Silt (non-cohesive)	< #200		Little	5%-15%
Clay (cohesive)	< #200		Trace	0%-5%

Notes:

1. Particle size is designated by U.S. Standard Sieve Sizes
2. Because of the small size of the split spoon sampler relative to the size of gravel, the true percentage of gravel may not be accurately estimated.

Density or Consistency

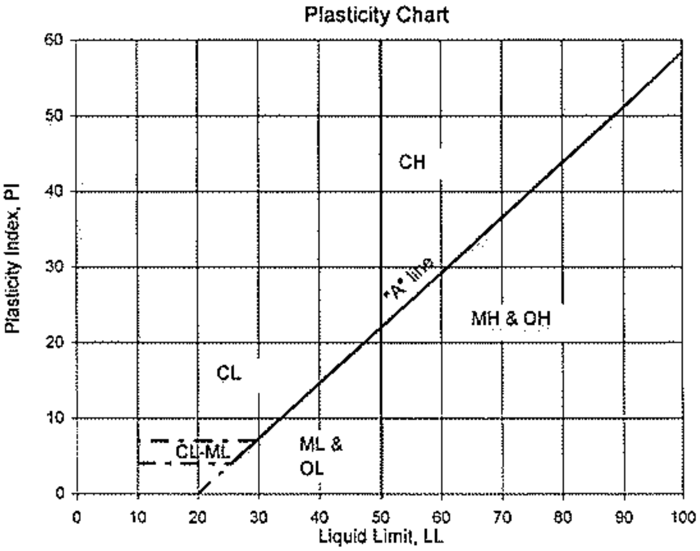
The standard penetration resistance values (N-values are used to describe the density of coarse-grained soils (GRAVEL, SAND) or the consistency of fine-grained soils (SILT, CLAY). Sandy silts of very low plasticity may be assigned a density instead of a consistency.

DENSITY		CONSISTENCY	
Term	N-Value	Term	N-Value
Very Loose	0 – 4	Very Soft	0 – 1
Loose	5 – 10	Soft	2 – 4
Medium-Dense	11 – 30	Medium Stiff	5 – 8
Dense	31 – 50	Stiff	9 – 15
Very Dense	> 50	Very Stiff	16 – 30
		Hard	>30

Notes:

1. The N-value is the number of blows of a 140 lb. hammer freely falling 30 inches required to drive a standard split-spoon sampler (2.0 in. O.D., 1-3/8 in. I.D.) 12 inches into the soil after properly seating the sampler 6 inches.
2. When encountered, gravel may increase the N-value of the standard penetration test and may not accurately represent the in-situ density or consistency of the soil sampled.

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D-2487)

Major Divisions			Group Symbols	Typical Names	Laboratory Classification Criteria		
Coarse-grained soils (More than half of material is larger than No. 200 sieve size)	Gravels (More than half of coarse fraction is larger than No. 4 sieve size)	Clean gravels (little or no fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines	Determine percentages of sand and gravel from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 Sieve), coarse-grained soils are classified as follows: Less than 5 per cent GW, GP, SW, SP More than 12 per cent GM, GC, SM, SC 5 to 12 per cent Borderline cases requiring dual symbols	C_u=D_{60}/D_{10} greater than 4; C_c=$(D_{30})^2/(D_{10}$x $D_{60})$ between 1 and 3	
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines		Not meeting all gradation requirements for GW	
		Gravels with fines (Appreciable amount of fines)	GM	Silty gravels, gravel-sand-silt mixtures		Atterberg limits below "A" line or PI less than 4	Above "A" line with PI between 4 and 7 are borderline cases requiring use of dual symbols
			GC	Clayey gravels, gravel-sand-clay mixtures		Atterberg limits below "A" line or PI greater than 7	
	Sands (More than half of coarse fraction is smaller than No.4 sieve size)	Clean sands (little or no fines)	SW	Well-graded sands, gravelly sands, little or no fines		C_u=D_{60}/D_{10} greater than 6; C_c=$(D_{30})^2/(D_{10}$x $D_{60})$ between 1 and 3	
			SP	Poorly graded sands, gravelly sands, little or no fines		Not meeting all gradation requirements for SW	
		Sands with fines (Appreciable amount of fines)	SM	Silty sands, sand-silt mixtures		Atterberg limits above "A" line or PI less than 4	Above "A" line with PI between 4 and 7 are borderline cases requiring use of dual symbols
			SC	Clayey sands, sand-clay mixtures		Atterberg limits above "A" line or PI greater than 7	
Fine-grained soils (More than half material is smaller than No. 200 sieve	Silts and clays (Liquid limit less than 50)		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity			
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays			
			OL	Organic silts and organic silty clays of low plasticity			
	Silts and clays (Liquid limit greater than 50)		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts			
			CH	Inorganic clays of high plasticity, fat clays			
			OH	Organic clays of medium to high plasticity			
	Highly organic soils		Pt	Peat and other highly organic soils			

Rock Core from Boring AB-1 (1st Run @ 8.5'-13.5', 2nd Run @ 13.5'-18.5')

APPENDIX III

Test Pit Photos (10 pages)

Photographs of Site and Surroundings (7 pages)

Laboratory Test Results (39 pages)

Laboratory Test Procedures

Test Pit Photographs

Project: Erin Lake Dam **Accura Project No:** 10205

[redacted]

[redacted]

Test Pit Photographs

Project: Erin Lake Dam **Accura Project No:** 10205

[redacted]

[redacted]

Test Pit Photographs

Project: Erin Lake Dam **Accura Project No:** 10205

[redacted]

[redacted]

Test Pit Photographs

Project: Erin Lake Dam **Accura Project No:** 10205

[redacted]

[redacted]

Test Pit Photographs

Project: Erin Lake Dam **Accura Project No:** 10205

[redacted]

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Test Pit Photographs

Project: Erin Lake Dam **Accura Project No:** 10205

[redacted]

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Test Pit Photographs

Project: Erin Lake Dam **Accura Project No:** 10205

[redacted]

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Test Pit Photographs

Project: Erin Lake Dam **Accura Project No:** 10205

[redacted]

[redacted]

Test Pit Photographs

Project: Erin Lake Dam **Accura Project No:** 10205

[redacted]

[redacted]

Test Pit Photographs

Project: Erin Lake Dam **Accura Project No:** 10205

[redacted]

[redacted]

Site Photographs

Project: Erin Lake Dam **Accura Project No:** 10205

From east to west showing the crest of the Dam.

Site conditions showing the wooded area of the upstream slope.

Site conditions showing the wooded area of the downstream of the dam.

Showing the test pits at the downstream slope area of the dam.

Site conditions from west of the dam looking east.

Concrete outlet located at the northern end of the left abutment.

Downstream toe of the dam.

Spillway system located at the southwest end of the dam.

Outlet of the principal spillway constituted of riprap rocks.

Old spillway located by the downstream face of the Dam.

Showing boring AB-1 at the downstream area of the dam.

Upstream face of the dam located on the southwest end of the dam.

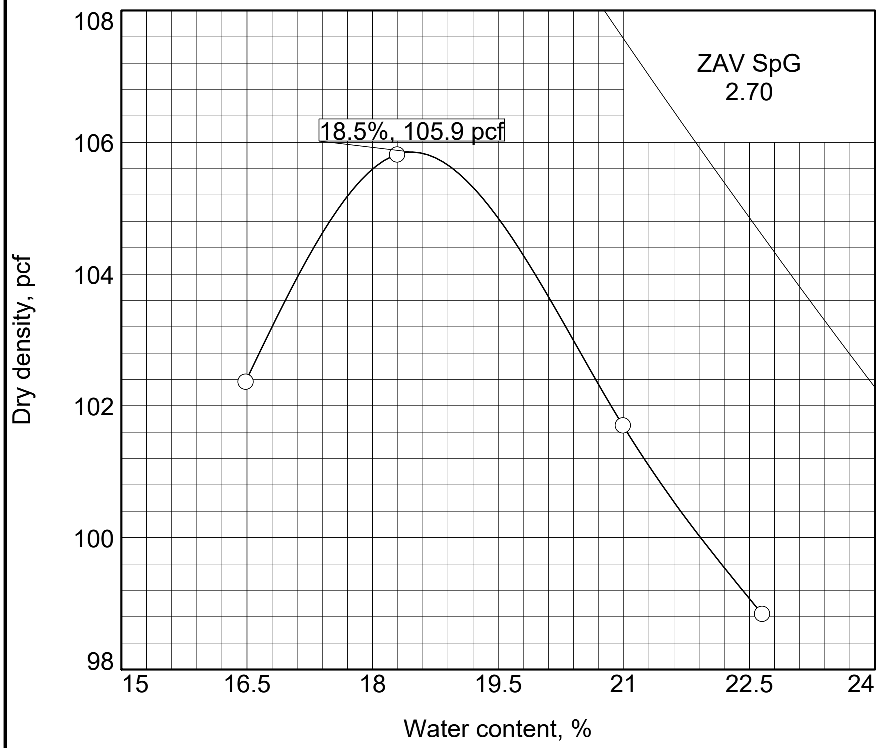
Site Photographs

Project: Erin Lake Dam **Accura Project No:** 10205

Probe test done on the north area of the dam.

Surroundings of the project area with the lake Erin.

COMPACTION TEST REPORT



Curve No.
1

Test Specification:
ASTM D 698-12 Method A Standard

Hammer Wt.: 5.5 lb.
Hammer Drop: 12 in.
Number of Layers: three
Blows per Layer: 25
Mold Size: 0.03333 cu. ft.
Test Performed on Material Passing #4 **Sieve**

Soil Data

NM NT **Sp.G.** 2.7
LL NT **PI** NT
%>#4 NT **%<#200** NT
USCS ML **AASHTO** N/A

TESTING DATA

	1	2	3	4	5	6
WM + WS	3866.0	3955.7	3923.6	3896.1		
WM	2063.3	2063.3	2063.3	2063.3		
WW + T #1	490.5	474.5	440.6	496.5		
WD + T #1	433.7	414.7	381.1	421.3		
TARE #1	89.1	87.8	97.6	89.3		
WW + T #2						
WD + T #2						
TARE #2						
MOISTURE	16.5	18.3	21.0	22.7		
DRY DENSITY	102.4	105.8	101.7	98.8		

TEST RESULTS

Maximum dry density = 105.9 pcf

Optimum moisture = 18.5 %

Material Description

Light brown Silty SAND

Project No. 6162191251 **Client:** Accura Engineering
Project: Erin Lake Dam

○ **Source:** Combination AB 1/2/4 **Depth:** 0-10' **Sample No.:** N/A

wood.

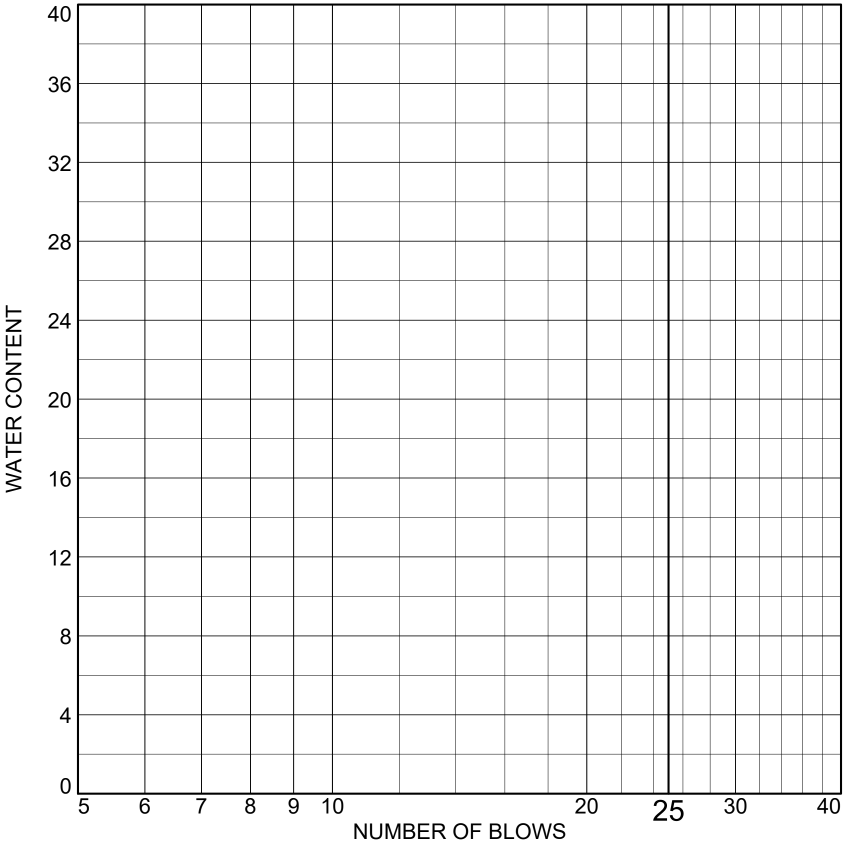
Remarks:
Lab No.20055

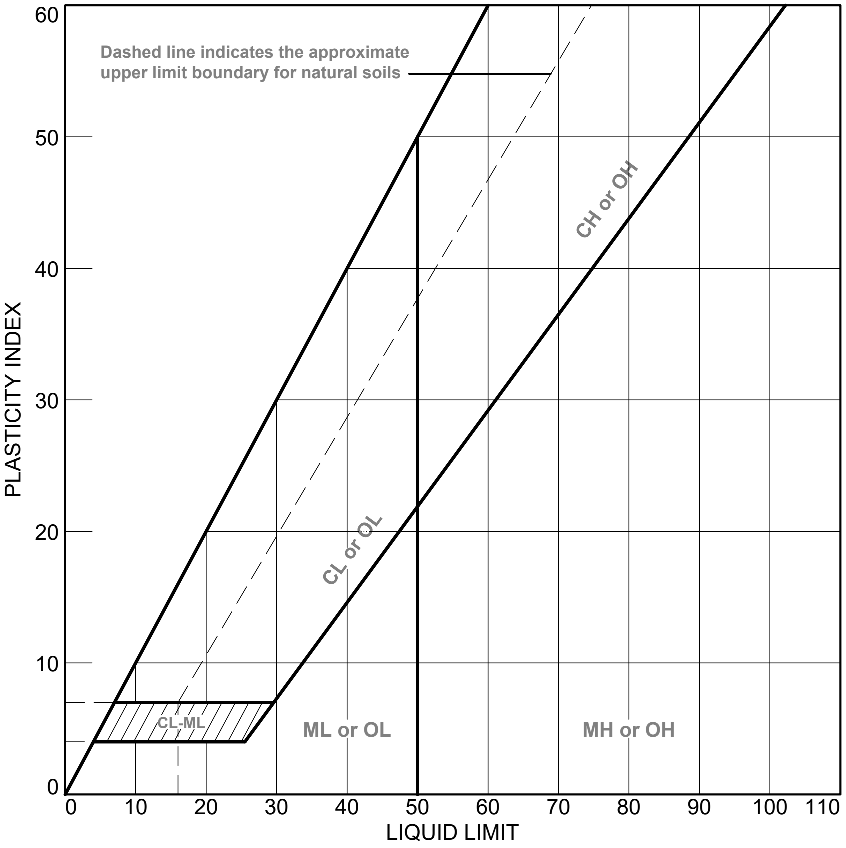
Figure

These results relate only to the items tested. This report shall not be reproduced, except in full, without written approval from Wood.

Tested By: AW **Checked By:** ML

LIQUID AND PLASTIC LIMITS TEST REPORT





	SOURCE	SAMPLE #	DEPTH/ELEV.	DATE SAMPLED	USCS	MATERIAL DESCRIPTION	NM %	LL	PI
●	AB-1	N/A	2-4'	7/19/21	SM	Brown Silty SAND	14.9	NV	NP

Client Accura Engineering

Project Erin Lake Dam

Project No. 6162191251

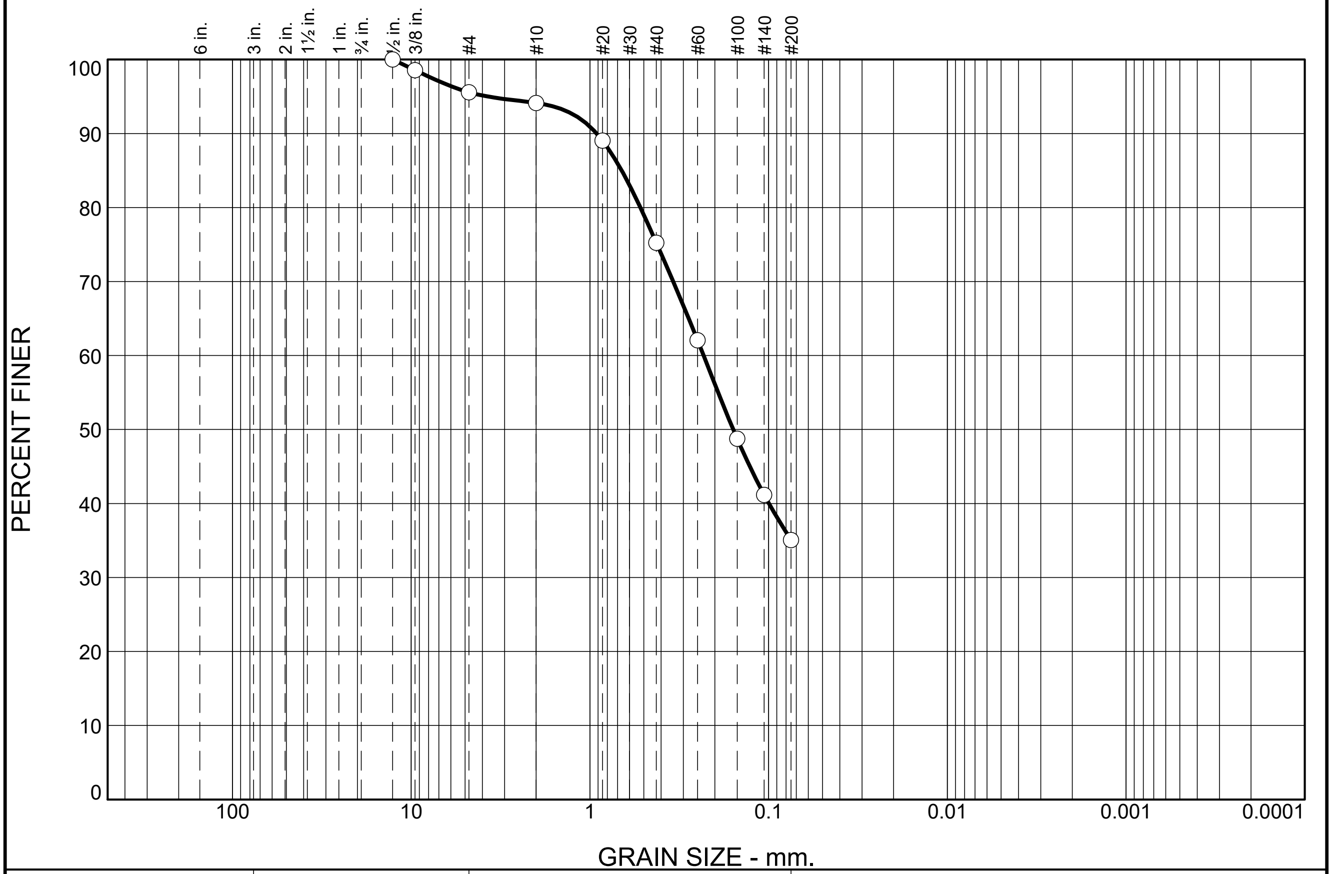
Figure

● Lab No.20056

These results relate only to the items tested. This report shall not be reproduced, except in full, without written approval from Wood.

Tested By: DW **Checked By:** ML

Grain Size Analysis (ASTM D6913-17)



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	4.4	1.5	18.9	40.1	35.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.500"	100.0		
0.375"	98.6		
#4	95.6		
#10	94.1		
#20	89.0		
#40	75.2		
#60	62.1		
#100	48.8		
#140	41.2		
#200	35.1		

* (no specification provided)

Material Description

Brown Silty SAND

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{90}= 0.9170 D_{85}= 0.6645 D_{60}= 0.2313
D_{50}= 0.1577 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO= A-2-4(0)

Remarks

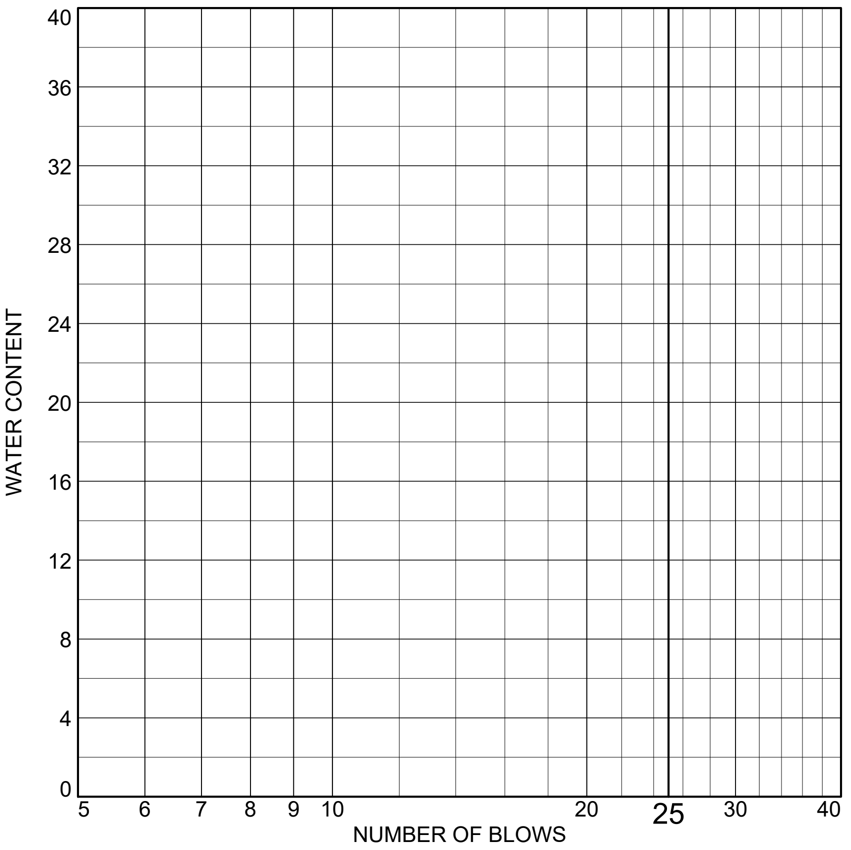
Lab No.20056

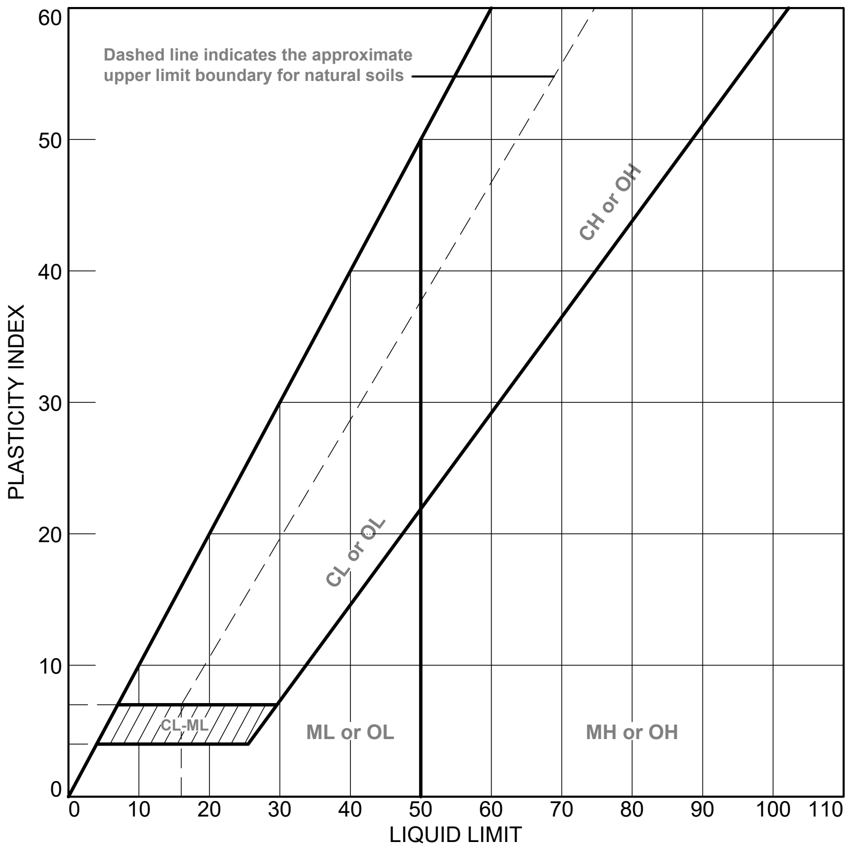
Source of Sample: AB-1 **Depth:** 2-4'
Sample Number: N/A **Date:** 7/19/21

Client: Accura Engineering
Project: Erin Lake Dam
Project No: 6162191251 **Figure**

Tested By: DW **Checked By:** ML

LIQUID AND PLASTIC LIMITS TEST REPORT





	SOURCE	SAMPLE #	DEPTH/ELEV.	DATE SAMPLED	USCS	MATERIAL DESCRIPTION	NM %	LL	PI
●	AB-2	N/A	2-4'	7/26/21	SM	Brown Silty SAND	20.7	NV	NP

Client Accura Engineering

Project Erin Lake Dam

Project No. 6162191251 Figure

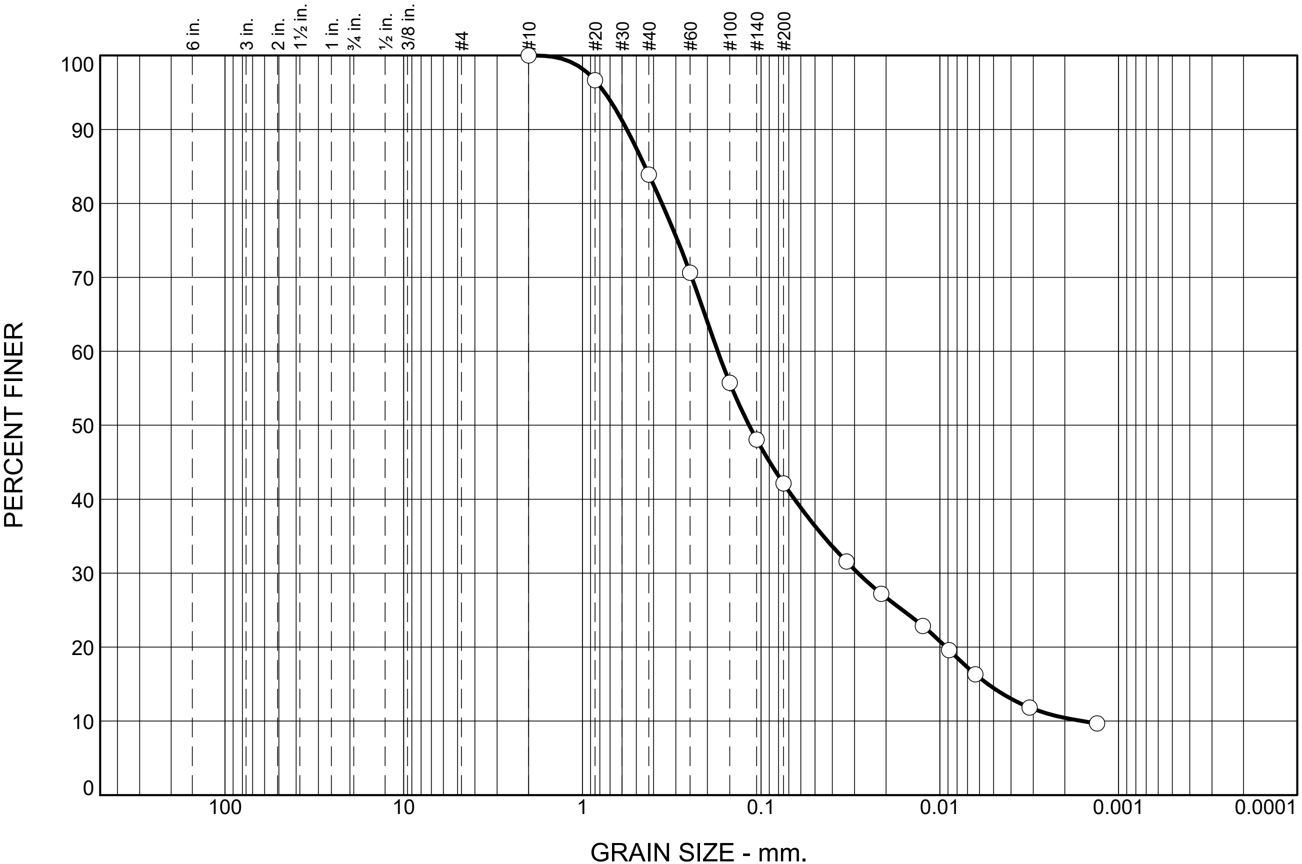
wood.

● Lab No.20057

These results relate only to the items tested. This report shall not be reproduced, except in full, without written approval from Wood.

Tested By: DW **Checked By:** ML

Grain Size Analysis (ASTM D422-63)



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	16.1	41.8	27.7	14.4

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#20	96.7		
#40	83.9		
#60	70.6		
#100	55.7		
#140	48.1		
#200	42.1		

* (no specification provided)

Material Description

Brown Silty SAND

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{90}= 0.5644 D_{85}= 0.4463 D_{60}= 0.1748

D_{50}= 0.1169 D_{30}= 0.0287 D_{15}= 0.0054

D_{10}= 0.0016 C_u= 109.75 C_c= 2.95

Classification

USCS= SM AASHTO= A-4(0)

Remarks

Lab No.20057

Source of Sample: AB-2 **Depth:** 2-4'
Sample Number: N/A

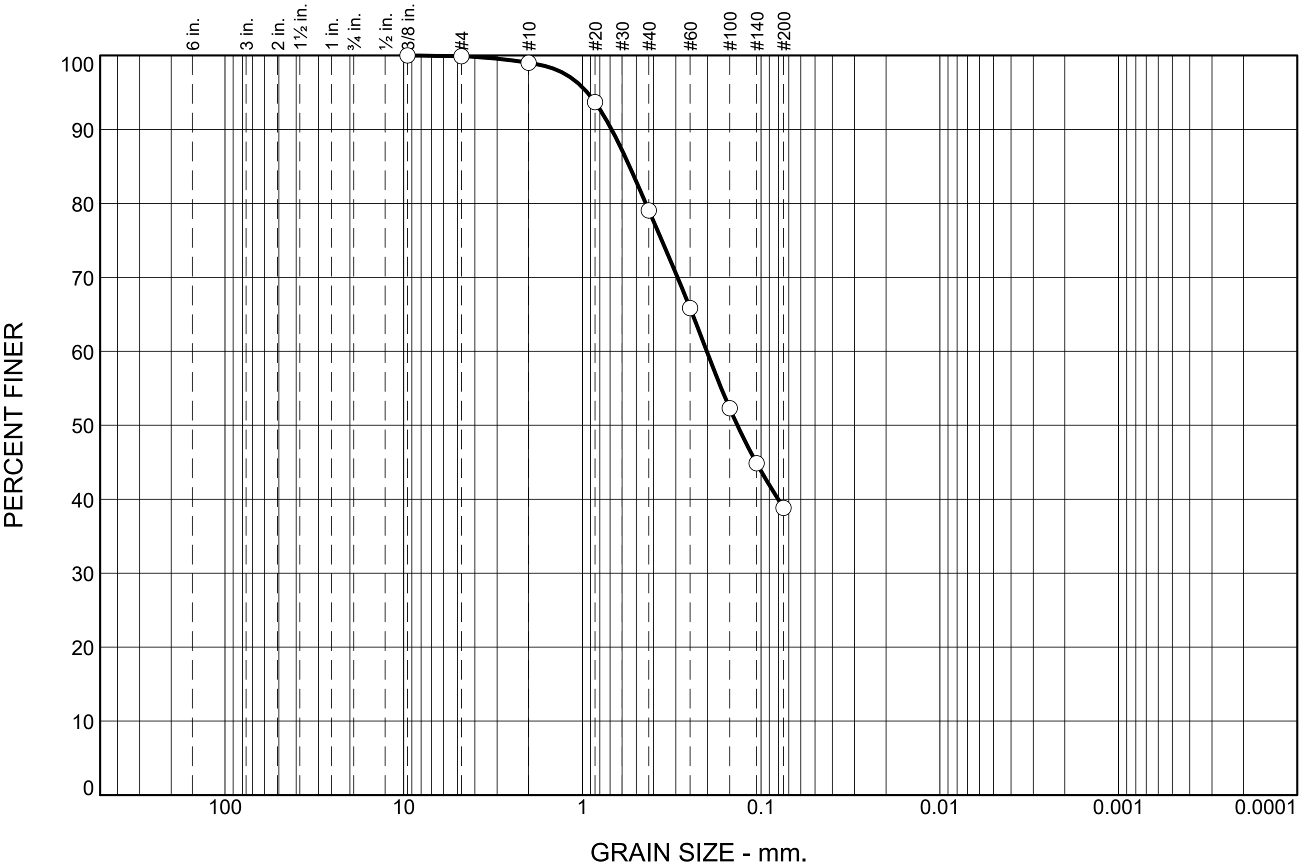
Date: 7/26/21

wood.

Client: Accura Engineering
Project: Erin Lake Dam
Project No: 6162191251 **Figure**

Tested By: DW **Checked By:** ML

Grain Size Analysis (ASTM D6913-17)



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.1	0.9	19.9	40.3	38.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.375"	100.0		
#4	99.9		
#10	99.0		
#20	93.7		
#40	79.1		
#60	65.8		
#100	52.3		
#140	44.9		
#200	38.8		

* (no specification provided)

Material Description

Brown Silty SAND

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.6869 D_{85}= 0.5451 D_{60}= 0.2014
D_{50}= 0.1360 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO= N/A

Remarks

Lab No.20058
Natural Moisture Content:25.0 %

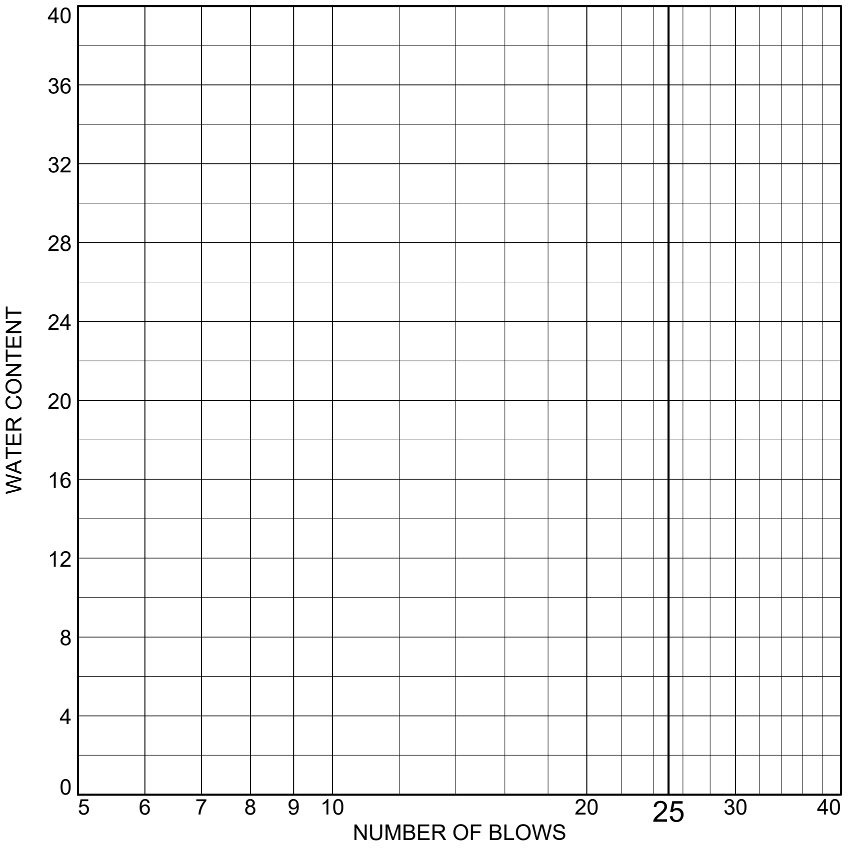
Source of Sample: AB-2 **Depth:** 6-8'
Sample Number: N/A **Date:** 7/19/21

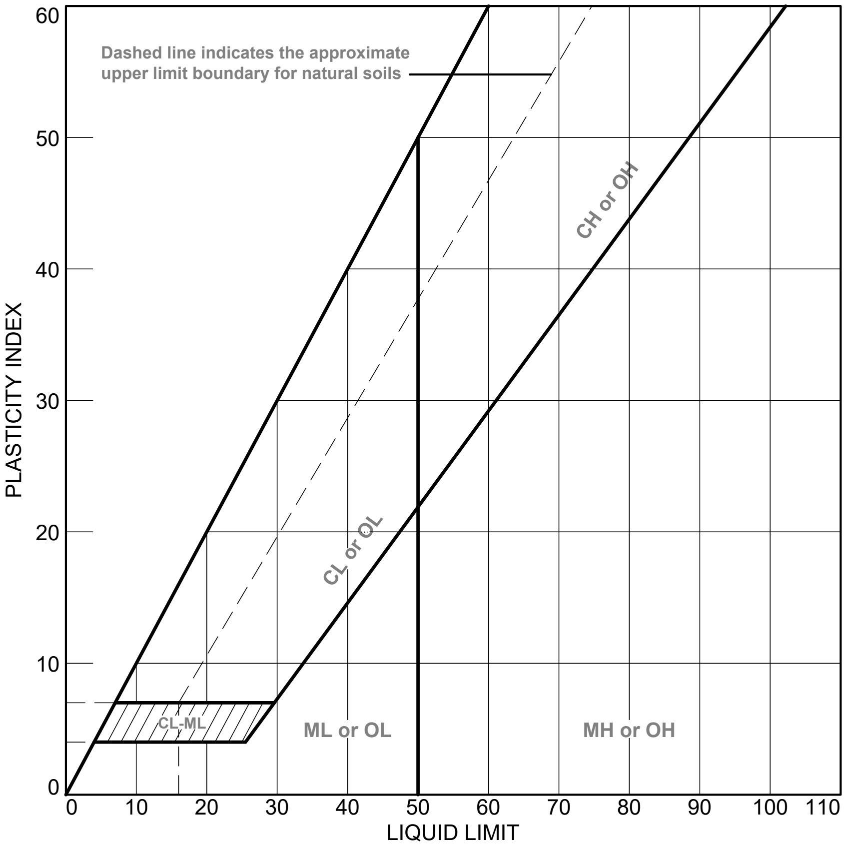
Client: Accura Engineering
Project: Erin Lake Dam
Project No: 6162191251 **Figure**

Tested By: DW **Checked By:** ML

These results relate only to the items tested. This report shall not be reproduced, except in full, without written approval from Wood.

LIQUID AND PLASTIC LIMITS TEST REPORT





	SOURCE	SAMPLE #	DEPTH/ELEV.	DATE SAMPLED	USCS	MATERIAL DESCRIPTION	NM %	LL	PI
●	AB-2	N/A	16-18'	7/19/21	SM	Brown Silty SAND	21.0	NV	NP

Client Accura Engineering

Project Erin Lake Dam

Project No. 6162191251

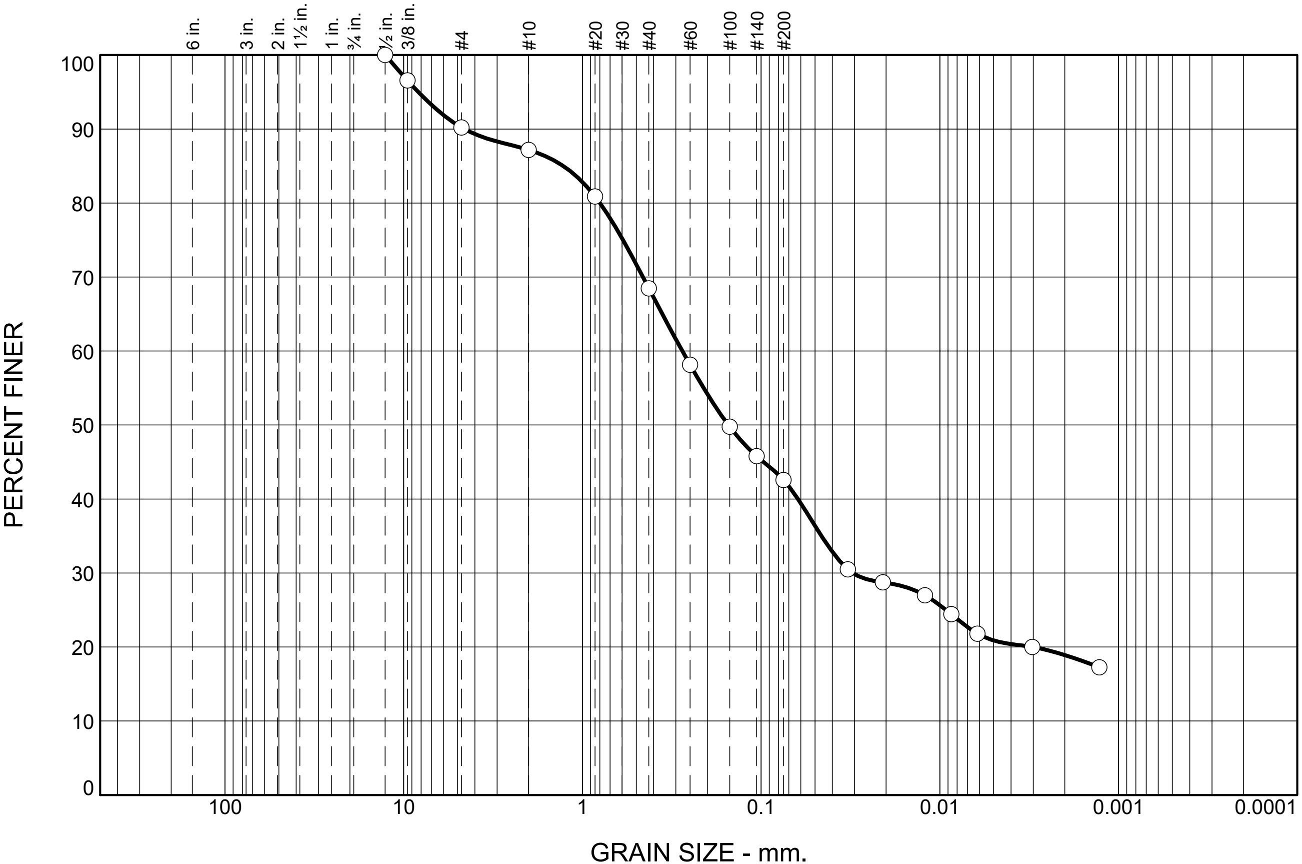
Figure

● Lab No.20059

These results relate only to the items tested. This report shall not be reproduced, except in full, without written approval from Wood.

Tested By: DW **Checked By:** ML

Grain Size Analysis (ASTM D422-63)



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	9.8	3.0	18.7	25.9	21.7	20.9

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.500"	100.0		
0.375"	96.6		
#4	90.2		
#10	87.2		
#20	80.9		
#40	68.5		
#60	58.1		
#100	49.8		
#140	45.8		
#200	42.6		

* (no specification provided)

Material Description

Brown Silty SAND

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{90}= 4.5817 D_{85}= 1.2804 D_{60}= 0.2762
D_{50}= 0.1525 D_{30}= 0.0307 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO= A-4(0)

Remarks

Lab No.20059

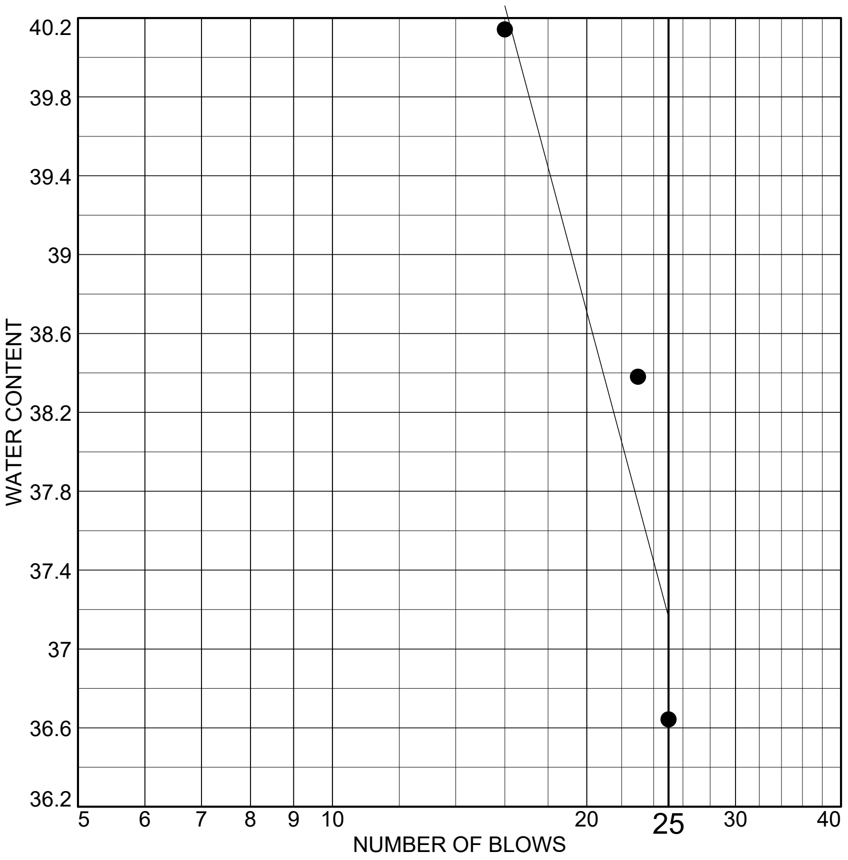
Source of Sample: AB-2 **Depth:** 16-18'
Sample Number: N/A **Date:** 7/19/21

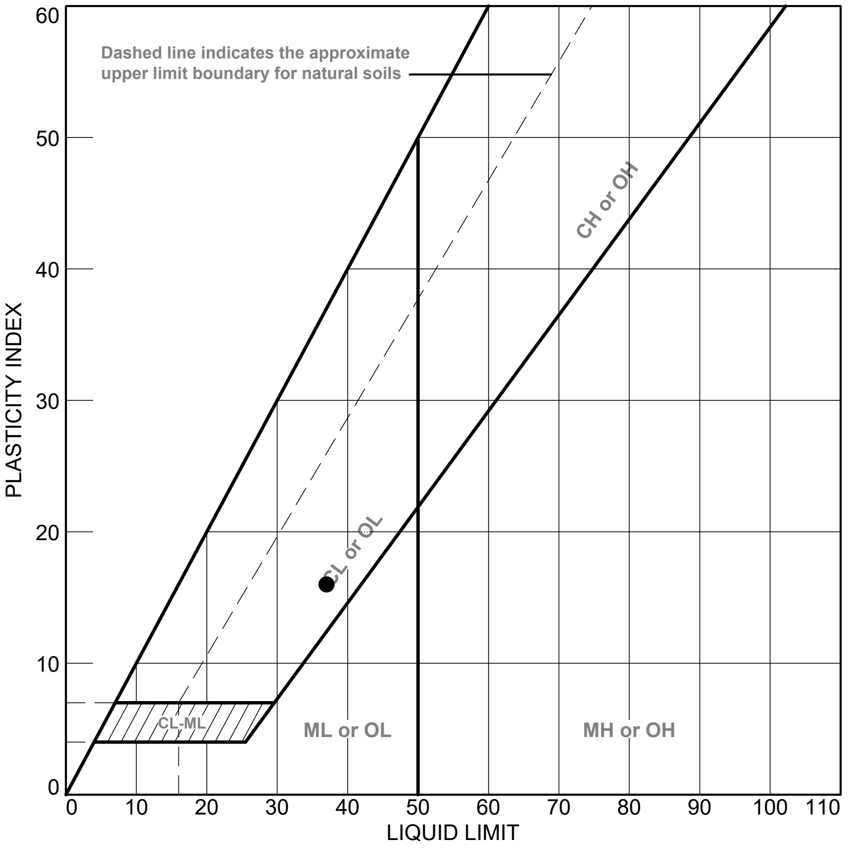
Client: Accura Engineering
Project: Erin Lake Dam

Project No: 6162191251 **Figure**

Tested By: DW **Checked By:** ML

LIQUID AND PLASTIC LIMITS TEST REPORT





	SOURCE	SAMPLE #	DEPTH/ELEV.	DATE SAMPLED	USCS	MATERIAL DESCRIPTION	NM %	LL	PI
●	AB-2	N/A	20-22'	7/19/21	CL	Brown Sandy Lean CLAY	25.4	37	16

Client Accura Engineering

Project Erin Lake Dam

Project No. 6162191251

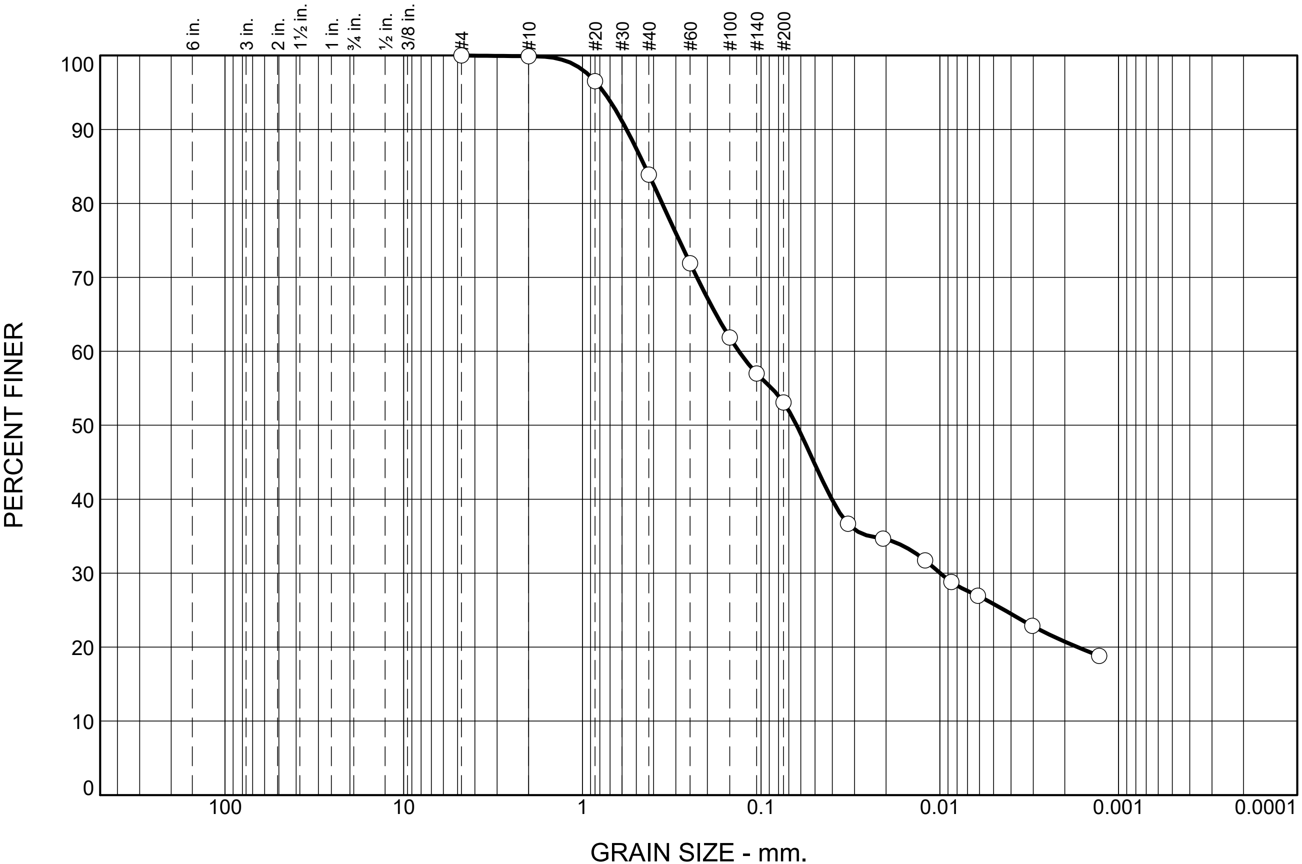
Figure

● Lab No.20060

These results relate only to the items tested. This report shall not be reproduced, except in full, without written approval from Wood.

Tested By: DW **Checked By:** ML

Grain Size Analysis (ASTM D422-63)



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.1	16.0	30.8	27.3	25.8

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	99.9		
#20	96.5		
#40	83.9		
#60	71.9		
#100	61.9		
#140	57.0		
#200	53.1		

* (no specification provided)

Material Description

Brown Sandy Lean CLAY

Atterberg Limits

PL= 21 LL= 37 PI= 16

Coefficients

D_{90}= 0.5665 D_{85}= 0.4468 D_{60}= 0.1333

D_{50}= 0.0633 D_{30}= 0.0100 D_{15}=

D_{10}= C_u= C_c=

Classification

USCS= CL AASHTO= A-6(6)

Remarks

Lab No.20060

Source of Sample: AB-2 **Depth:** 20-22'

Sample Number: N/A

Date: 7/19/21

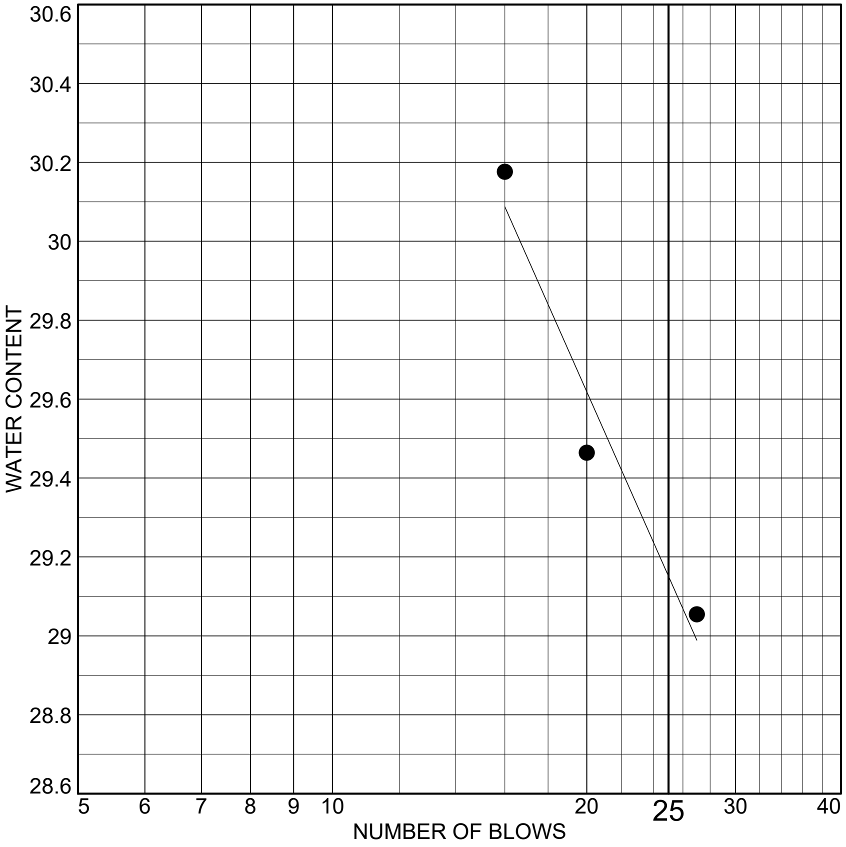
Client: Accura Engineering

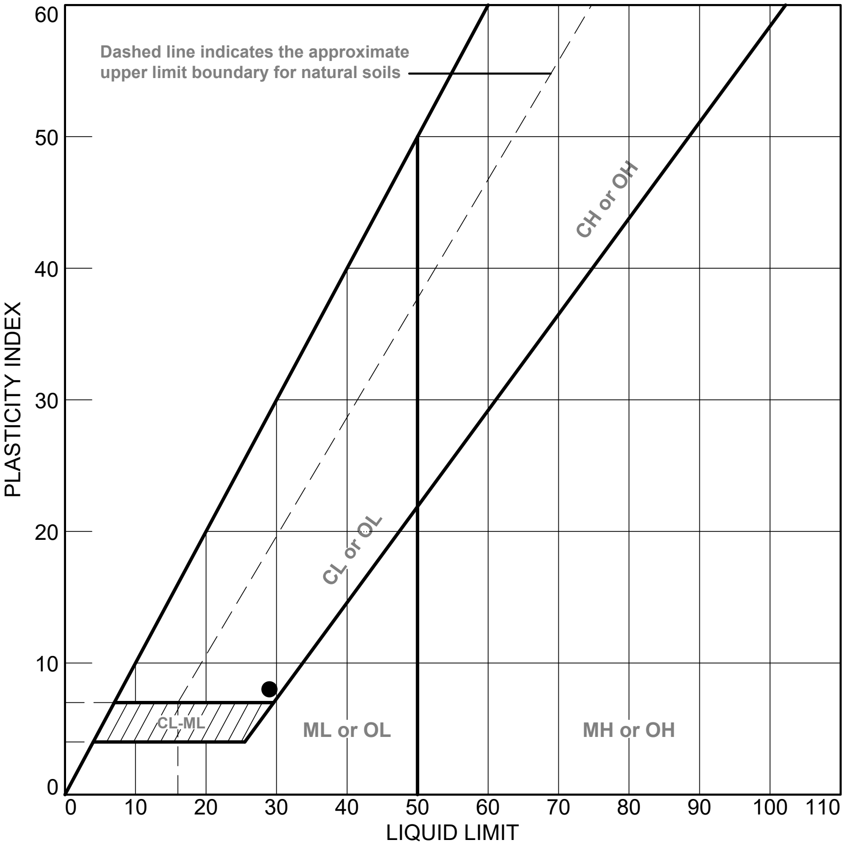
Project: Erin Lake Dam

Project No: 6162191251 **Figure**

Tested By: DW **Checked By:** ML

LIQUID AND PLASTIC LIMITS TEST REPORT





	SOURCE	SAMPLE #	DEPTH/ELEV.	DATE SAMPLED	USCS	MATERIAL DESCRIPTION	NM %	LL	PI
●	AB-2	N/A	26-28'	7/19/21	SC	Brown Clayey SAND	19.5	29	8

Client Accura Engineering

Project Erin Lake Dam

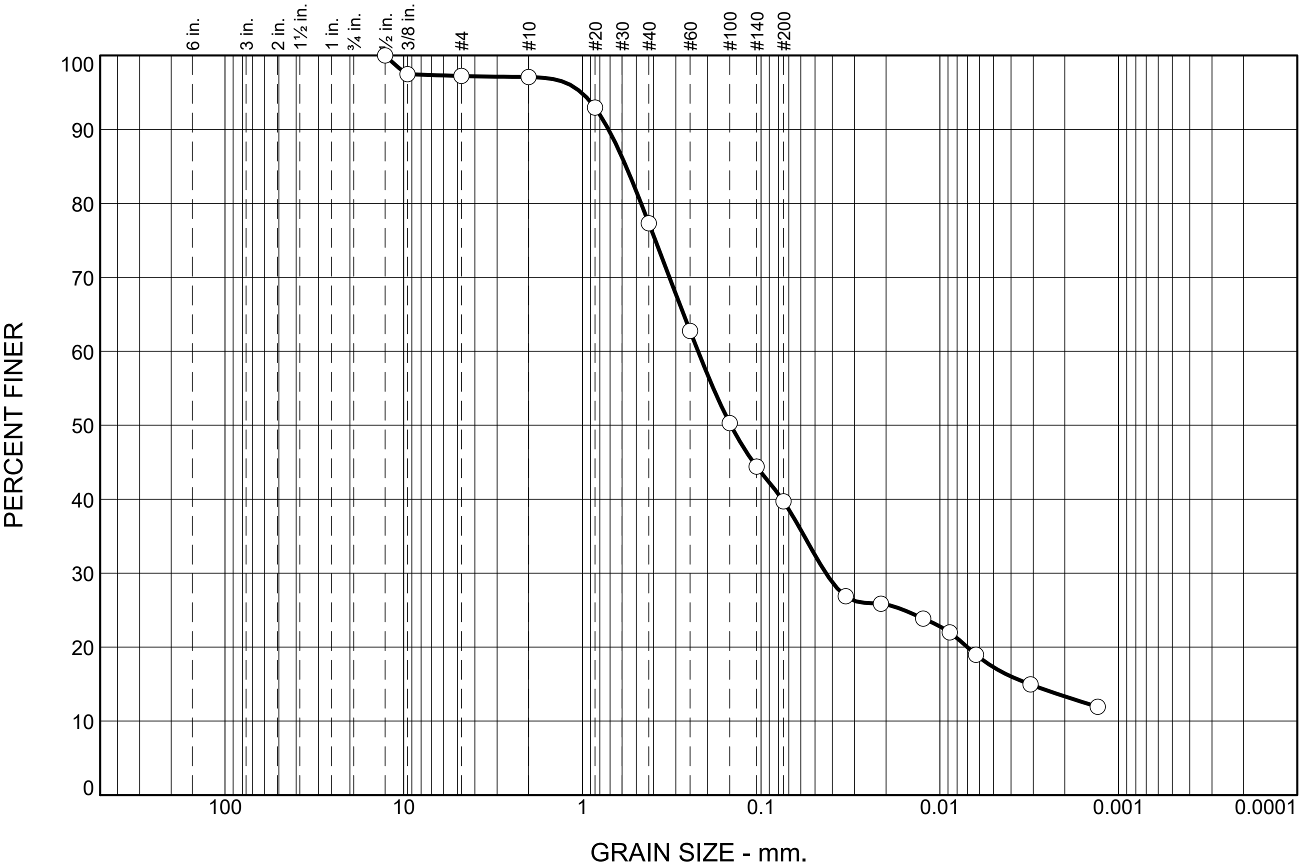
Project No. 6162191251

Figure

● Lab No.20061

Tested By: DW **Checked By:** ML

Grain Size Analysis (ASTM D422-63)



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	2.8	0.1	19.8	37.6	22.4	17.3

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.500"	100.0		
0.375"	97.5		
#4	97.2		
#10	97.1		
#20	93.0		
#40	77.3		
#60	62.8		
#100	50.3		
#140	44.4		
#200	39.7		

* (no specification provided)

Material Description

Brown Clayey SAND

Atterberg Limits

PL= 21 LL= 29 PI= 8

Coefficients

D_{90}= 0.7146 D_{85}= 0.5704 D_{60}= 0.2254
D_{50}= 0.1478 D_{30}= 0.0434 D_{15}= 0.0031
D_{10}= C_u= C_c=

Classification

USCS= SC AASHTO= A-4(0)

Remarks

Lab No.20061

Source of Sample: AB-2 **Depth:** 26-28'
Sample Number: N/A **Date:** 7/19/21

wood.

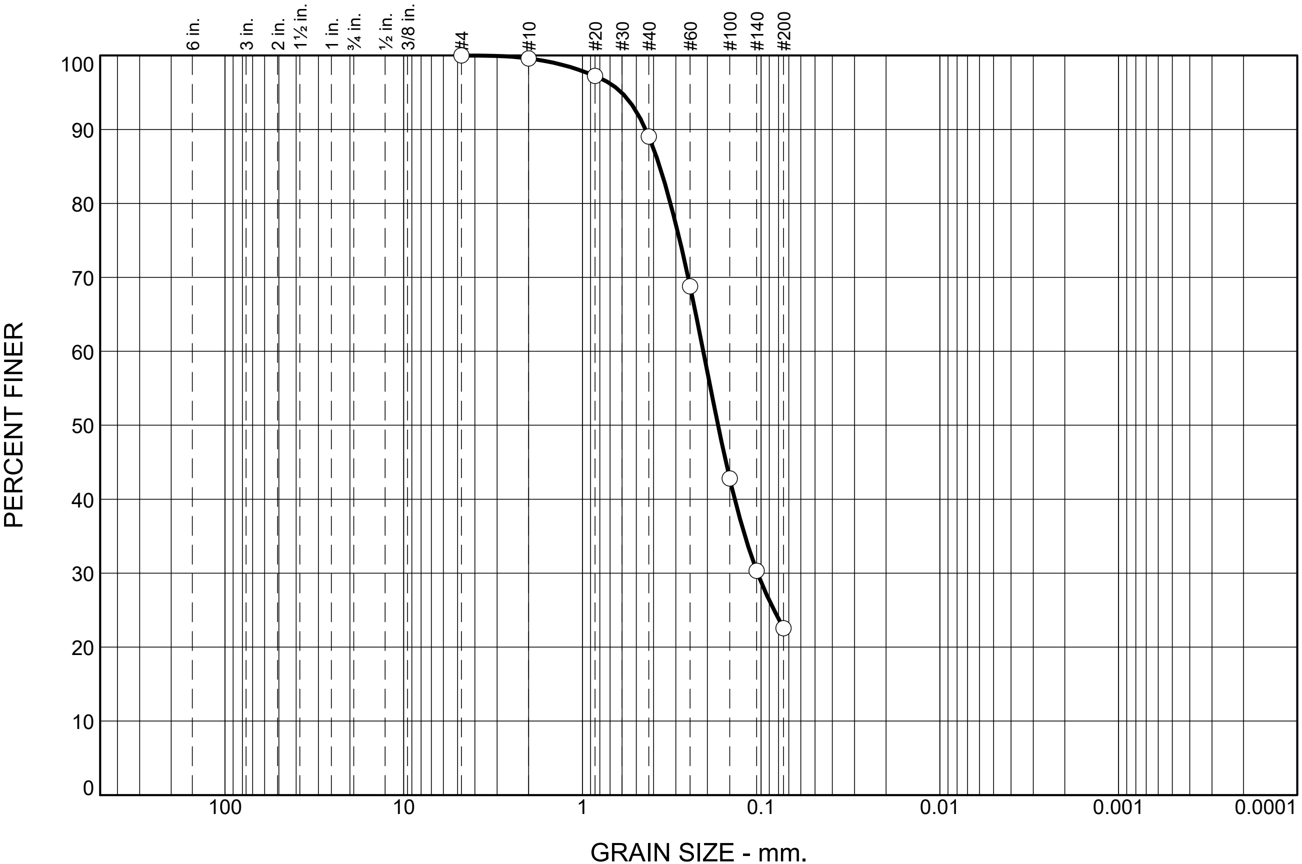
Client: Accura Engineering
Project: Erin Lake Dam

Project No: 6162191251 **Figure**

Tested By: DW **Checked By:** ML

These results relate only to the items tested. This report shall not be reproduced, except in full, without written approval from Wood.

Grain Size Analysis (ASTM D6913-17)



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.4	10.6	66.4	22.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	99.6		
#20	97.2		
#40	89.0		
#60	68.8		
#100	42.8		
#140	30.3		
#200	22.6		

* (no specification provided)

Material Description

Brown Silty SAND

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.4422 D_{85}= 0.3695 D_{60}= 0.2108
D_{50}= 0.1742 D_{30}= 0.1047 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO= N/A

Remarks

Lab No.20062
Natural Moisture Content:21.8%

Source of Sample: AB-2 **Depth:** 30-32'
Sample Number: N/A **Date:** 7/19/21

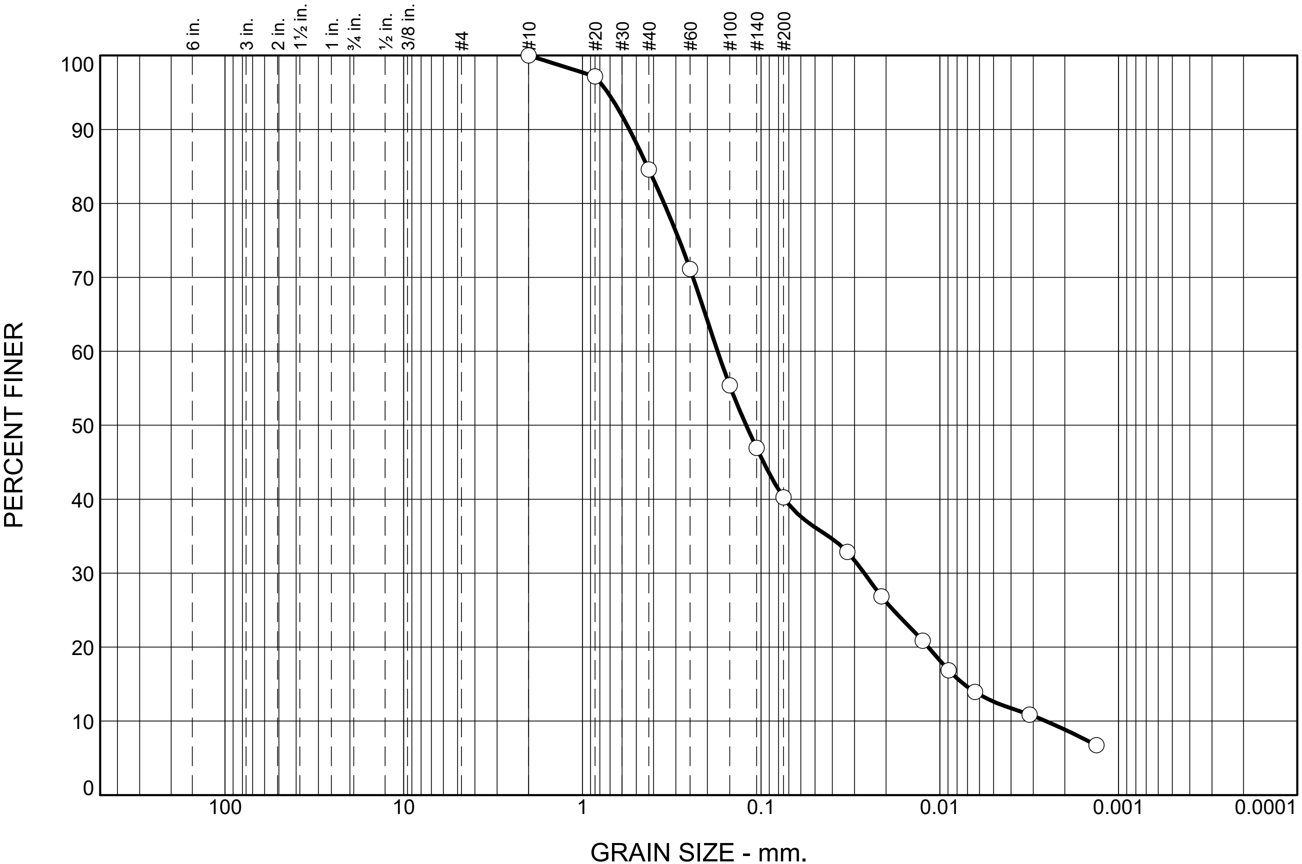
These results relate only to the items tested. This report shall not be reproduced, except in full, without written approval from Wood.

wood.

Client: Accura Engineering
Project: Erin Lake Dam
Project No: 6162191251 **Figure**

Tested By: DW **Checked By:** ML

Grain Size Analysis (ASTM D422-63)



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	15.4	44.3	27.7	12.6

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#20	97.2		
#40	84.6		
#60	71.1		
#100	55.4		
#140	46.9		
#200	40.3		

* (no specification provided)

Material Description

Brown Silty SAND with Clay

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.5463 D_{85}= 0.4329 D_{60}= 0.1752
D_{50}= 0.1214 D_{30}= 0.0265 D_{15}= 0.0073
D_{10}= 0.0026 C_u= 68.70 C_c= 1.57

Classification

USCS= SM AASHTO= N/A

Remarks

Lab No.20063
Natural Moisture Content:13.8%

Source of Sample: AB-4 **Depth:** 4-6'
Sample Number: N/A **Date:** 7/19/21

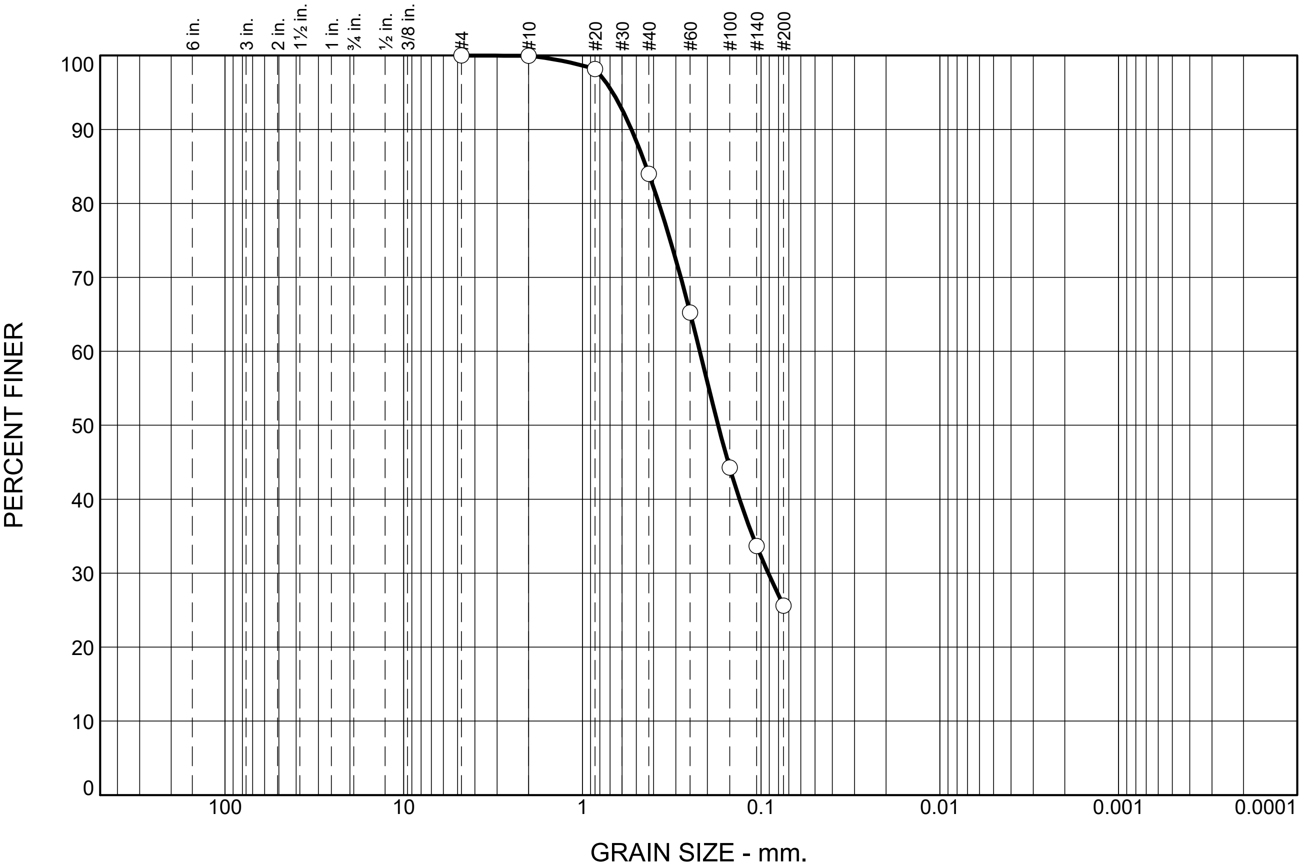
wood.

Client: Accura Engineering
Project: Erin Lake Dam

Project No: 6162191251 **Figure**

Tested By: DW **Checked By:** ML

Grain Size Analysis (ASTM D6913-17)



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	16.0	58.4	25.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	100.0		
#20	98.2		
#40	84.0		
#60	65.3		
#100	44.3		
#140	33.7		
#200	25.6		

* (no specification provided)

Material Description

Brown Silty SAND

Atterberg Limits

PL= NT LL= NT PI= NT

Coefficients

D_{90}= 0.5318 D_{85}= 0.4398 D_{60}= 0.2205

D_{50}= 0.1739 D_{30}= 0.0912 D_{15}=

D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO= N/A

Remarks

Lab No.20064

Natural Moisture Content:27.6%

Source of Sample: AB-4 **Depth:** 18-20'

Sample Number: N/A

Date: 7/19/21

Client: Accura Engineering

Project: Erin Lake Dam

Project No: 6162191251 **Figure**

Tested By: DW **Checked By:** ML

SPECIFIC GRAVITY OF SOILS

ASTM D854-14

Project No. 6162191251

Project Name Erin Lake Dam

Boring No. AB-2A

Sample No. N/A

Sample Depth 10-12'

Sample Description Brown Silty Sand with Clay

Tested By JPK

Test Date 7/29/2021

Reviewed By ML

Review Date 8/3/2021

Lab No. 20066

Pan No. N/A

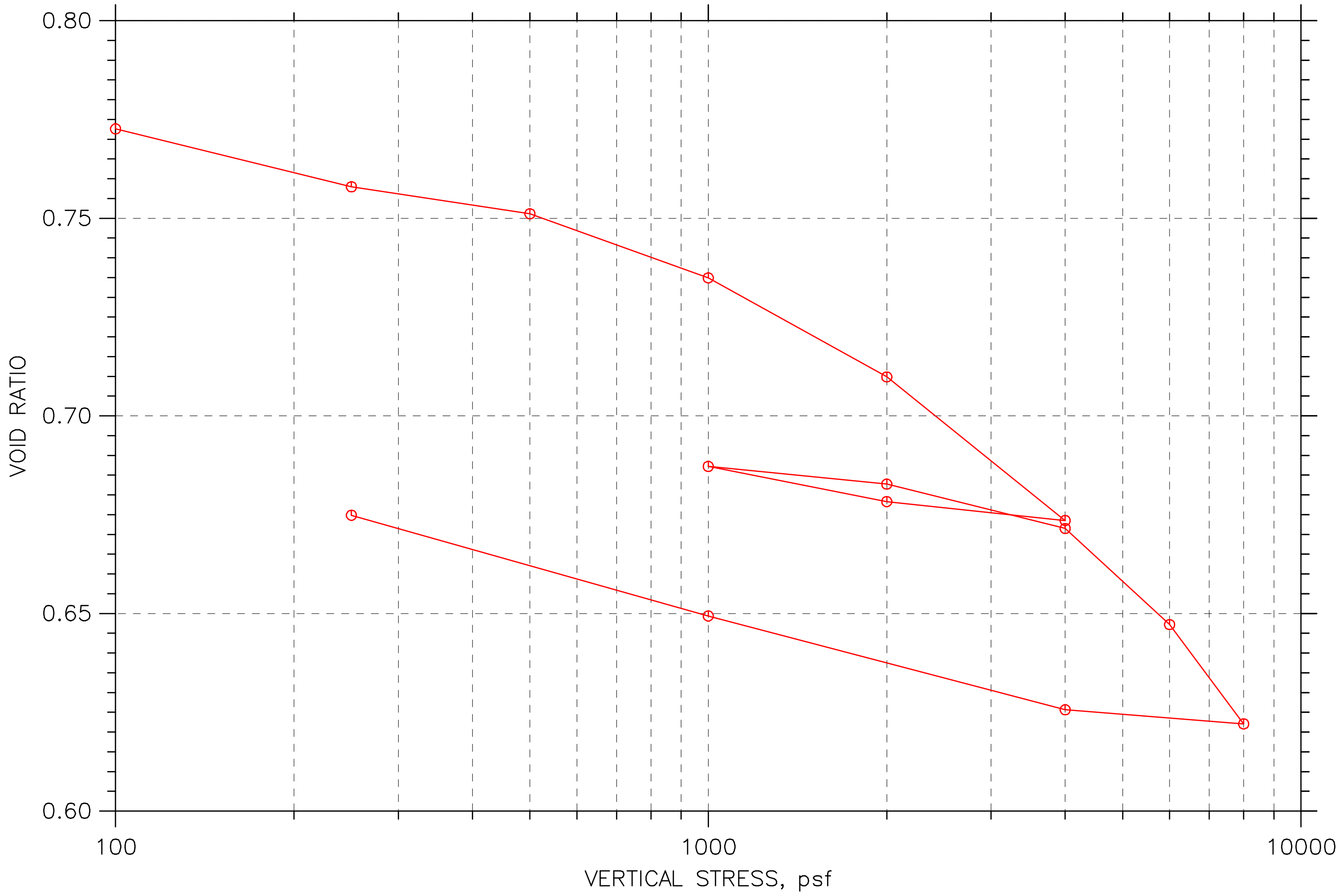
Tare No.	LS-61
Tare Mass, gram	88.92
Dry Soil + Tare Mass, grams	141.68
Mass of oven-dried soil, grams, M_s	52.76
Mass of pycnometer with water at test temperature (T), grams, $M_{pw,t}$	682.00
Mass of pycnometer, water and soil, grams, $M_{pws,t}$	715.39
Test Temperature, ºC , T_t	22.5
Specific Gravity at test temperature, M_s / [$M_{pw,t}$ - ($M_{pws,t}$ - M_s)], G_t	2.724
Temperature Coefficient, K	0.99945
SPECIFIC GRAVITY @ 20ºC: G_{20} o_C = K*G_t	**2.72**

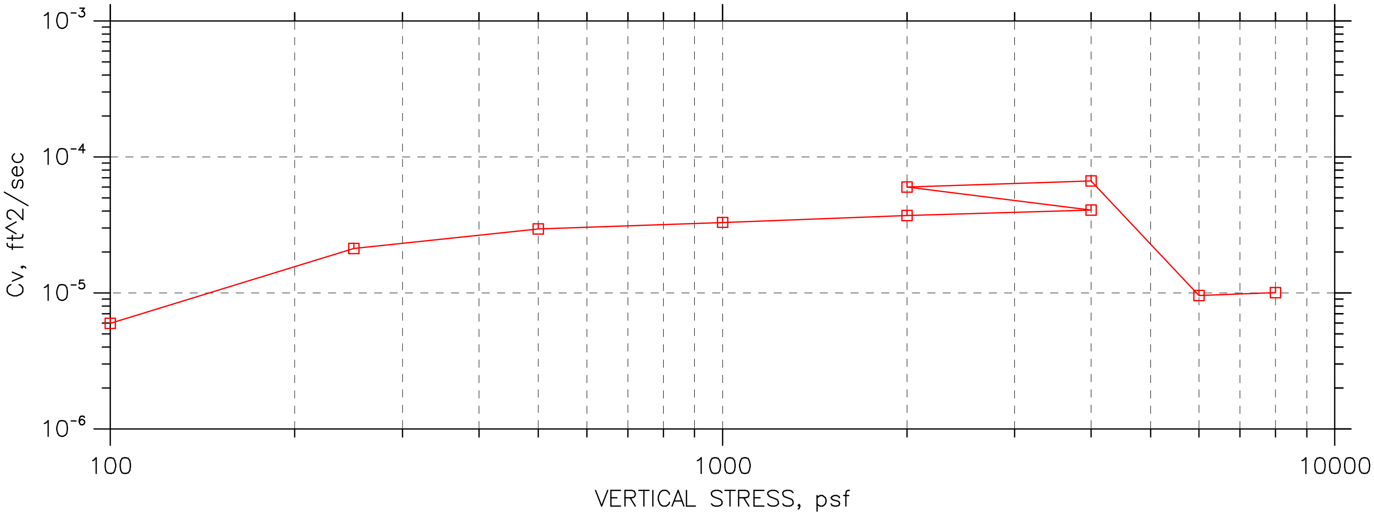
PREPARATION METHOD: ☒ Method A, Wet ☐ Method B, Dry

REMARKS:

CONSOLIDATION TEST DATA

SUMMARY REPORT



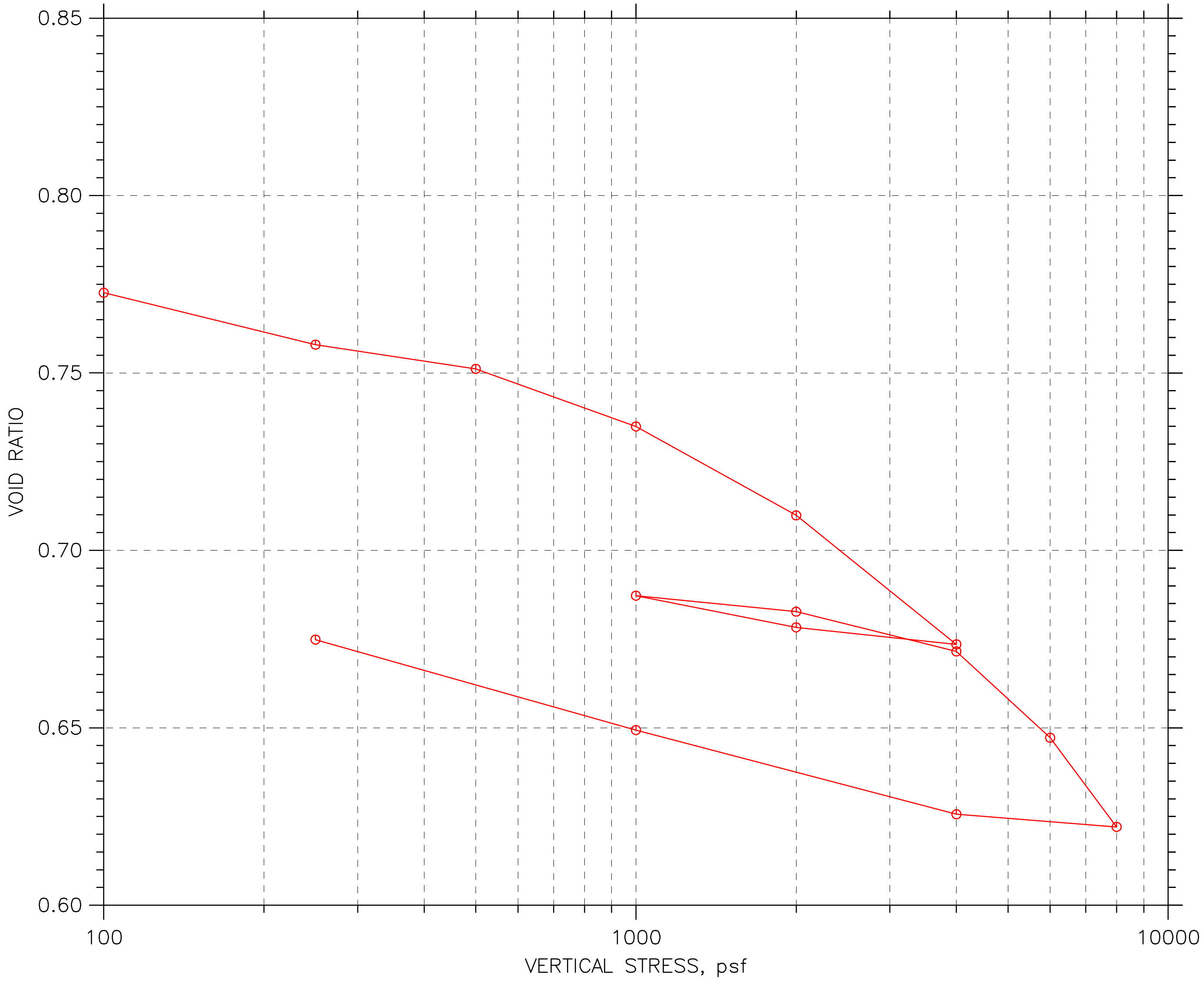


Project: Erin Lake Dam	Location: N/A	Project No.: 6162191251
Boring No.: AB-2A	Tested By: ML	Checked By: MRF
Sample No.: N/A	Test Date: 7/28/21	Depth: 10-12'
Test No.: 1	Sample Type: Intact	Elevation: N/A
Description: Brown Silty SAND with Clay. Rebound Index:0.005		
Remarks: Lab No.20060		

Wed, 25-AUG-2021 10:02:28

CONSOLIDATION TEST DATA

SUMMARY REPORT



					Before Test	After Test
Overburden Pressure: 951.4 psf				Water Content, %	23.95	26.42
Preconsolidation Pressure: 3700 psf				Dry Unit Weight, pcf	95.14	101.6
Compression Index: 0.0125				Saturation, %	82.80	106.72
Diameter: 2.49 in		Height: 0.99 in		Void Ratio	0.79	0.67
LL: ---	PL: ---	PI: ---	GS: 2.73			

Project: Erin Lake Dam	Location: N/A	Project No.: 6162191251
Boring No.: AB-2A	Tested By: ML	Checked By: MRF
Sample No.: N/A	Test Date: 7/28/21	Depth: 10-12'
Test No.: 1	Sample Type: Intact	Elevation: N/A
Description: Brown Silty SAND with Clay. Rebound Index:0.005		
Remarks: Lab No.20060		

Wed, 25-AUG-2021 10:02:28

CONSOLIDATION TEST DATA

Project: Erin Lake Dam
Boring No.: AB-2A
Sample No.: N/A
Test No.: 1

Location: N/A
Tested By: ML
Test Date: 7/28/21
Sample Type: Intact

Project No.: 6162191251
Checked By: MRF
Depth: 10-12'
Elevation: N/A

Soil Description: Brown Silty SAND with Clay. Rebound Index:0.005
Remarks: Lab No.20060

Measured Specific Gravity: 2.73
Initial Void Ratio: 0.79
Final Void Ratio: 0.67

Liquid Limit: ---
Plastic Limit: ---
Plasticity Index: ---

Initial Height: 0.99 in
Specimen Diameter: 2.49 in

	Before Consolidation		After Consolidation	
	Trimmings	Specimen+Ring	Specimen+Ring	Trimmings
Container ID	2578	RING	P-28	P-28
Wt. Container + Wet Soil, gm	293.76	164.14	167.11	167.11
Wt. Container + Dry Soil, gm	264.96	135.3	135.3	135.3
Wt. Container, gm	144.56	14.9	14.9	14.9
Wt. Dry Soil, gm	120.4	120.4	120.4	120.4
Water Content, %	23.92	23.95	26.42	26.42
Void Ratio	---	0.79	0.67	---
Degree of Saturation, %	---	82.80	106.72	---
Dry Unit Weight, pcf	---	95.144	101.61	---

CONSOLIDATION TEST DATA

Project: Erin Lake Dam
Boring No.: AB-2A
Sample No.: N/A
Test No.: 1

Location: N/A
Tested By: ML
Test Date: 7/28/21
Sample Type: Intact

Project No.: 6162191251
Checked By: MRF
Depth: 10-12'
Elevation: N/A

Soil Description: Brown Silty SAND with Clay. Rebound Index:0.005
Remarks: Lab No.20060

	Applied	Final	Void	Strain	T50 Fitting		Coefficient of Consolidation		
	Stress	Displacement	Ratio	at End	Sq.Rt.	Log	Sq.Rt.	Log	Ave.
	psf	in		%	min	min	ft^2/sec	ft^2/sec	ft^2/sec
1	100	0.008877	0.773	0.90	1.5	0.4	3.74e-006	1.46e-005	5.96e-006
2	250	0.01698	0.758	1.72	0.3	0.2	1.98e-005	2.28e-005	2.12e-005
3	500	0.02076	0.751	2.10	0.2	0.2	2.91e-005	2.98e-005	2.94e-005
4	1e+003	0.02974	0.735	3.00	0.2	0.1	3.05e-005	3.58e-005	3.30e-005
5	2e+003	0.04361	0.710	4.41	0.1	0.1	3.48e-005	3.94e-005	3.70e-005
6	4e+003	0.06373	0.674	6.44	0.1	0.1	4.15e-005	3.97e-005	4.06e-005
7	2e+003	0.06108	0.678	6.17	0.1	0.0	8.76e-005	0.00e+000	8.76e-005
8	1e+003	0.05614	0.687	5.67	0.2	0.1	2.80e-005	5.07e-005	3.61e-005
9	2e+003	0.05863	0.683	5.92	0.1	0.0	5.99e-005	0.00e+000	5.99e-005
10	4e+003	0.06484	0.672	6.55	0.1	0.1	5.74e-005	7.87e-005	6.64e-005
11	6e+003	0.0783	0.647	7.91	0.9	0.1	5.41e-006	4.00e-005	9.54e-006
12	8e+003	0.09221	0.622	9.31	0.8	0.1	5.97e-006	3.18e-005	1.00e-005
13	4e+003	0.09022	0.626	9.11	0.1	0.0	7.84e-005	1.45e-004	1.02e-004
14	1e+003	0.07709	0.649	7.79	0.3	0.1	1.34e-005	5.49e-005	2.16e-005
15	250	0.06298	0.675	6.36	2.4	0.0	2.03e-006	0.00e+000	2.03e-006

PERMEABILITY TEST
(ASTM D5084 - 10) (Method F, Constant Volume Falling Head)

wood.

Project Number 6162191251.14
Project Name Erin Lake Dam
Boring No. AB-2A
Sample No. N/A
Sample Depth 10-12'
Sample Description Brown Silty SAND with Clay

Tested By ML
Test Date 07/26/21
Reviewed By JPK
Review Date 08/03/21
Lab No. 20066

Initial Sample Data				Final Sample Data	
Length, in		Diameter, in		Pan No.	MF-24
Location 1	6.069	Location 1	2.855	Wet Soil+Pan, grams	1451.60
Location 2	6.056	Location 2	2.855	Dry Soil + Pan, grams	1198.00
Location3	6.062	Location 3	2.858	Pan Weight, grams	189.47
Average	6.062	Average	2.856	Moisture Content, %	25.1
Volume, in^3	38.83	Wet Soil + Tare, grams	1248.73	Dry Unit Weight, pcf	99.8
SG Assumed	2.73	Tare Weight, grams	0.00	Saturation, %	97.4
Soil Sample Wt., g	1248.73	Dry Soil +Tare, grams	1008.53	Diameter, in.	N/A
Dry UW, pcf	98.9	Moisture Content, %	23.8	Length, in.	N/A
Saturation, %	90.3			Volume, in^3	N/A

Consolidation	
Chamber Pressure, psi	70.0
Back Pressure, psi	60.0
Confining Pressure, psi	10.0
Initial Burett Reading	9.8
Final Burett Reading	4.1
Volume Change, cc	5.7

Permeant used water

Elapsed Time (sec)	z_o (cm)	za (cm)	zb (cm)	Δz_p (cm)	Temp (°C)	Intial Hydraulic Gradient	Final Hydraulic Gradient	k cm/sec	k cm/sec at 20 °C
180	2.50	18.30	17.40	0.90	22.7	12.9	12.1	3.04E-07	2.85E-07
360	2.50	18.30	16.50	1.80	22.7	12.9	11.4	3.14E-07	2.94E-07
540	2.50	18.30	15.80	2.50	22.7	12.9	10.8	2.98E-07	2.80E-07
720	2.50	18.30	15.20	3.10	22.7	12.9	10.3	2.84E-07	2.66E-07
900	2.50	18.30	14.80	3.50	22.7	12.9	9.9	2.61E-07	2.44E-07
1080	2.50	18.30	14.40	3.90	22.7	12.9	9.6	2.46E-07	2.31E-07

No. of Trials	Sample Type	Max. Density (pcf)	Compaction %	Sample Orientation
6	UD	N/A	N/A	Vertical

Avg. k at 20 °C 2.7E-07 cm/sec

a_a = 0.76712 cm²
A = 41.33 cm²
L = 15.40 cm
S=L/A= 0.37258 1/cm

a_p= 0.031416 cm²
M_1= 0.03018
M_2= 1.04095
$C = M_1S/(G_{Hg}-1)$= 0.0008945 for 15° to 25°

Remarks:

HYDRAULIC CONDUCTIVITY

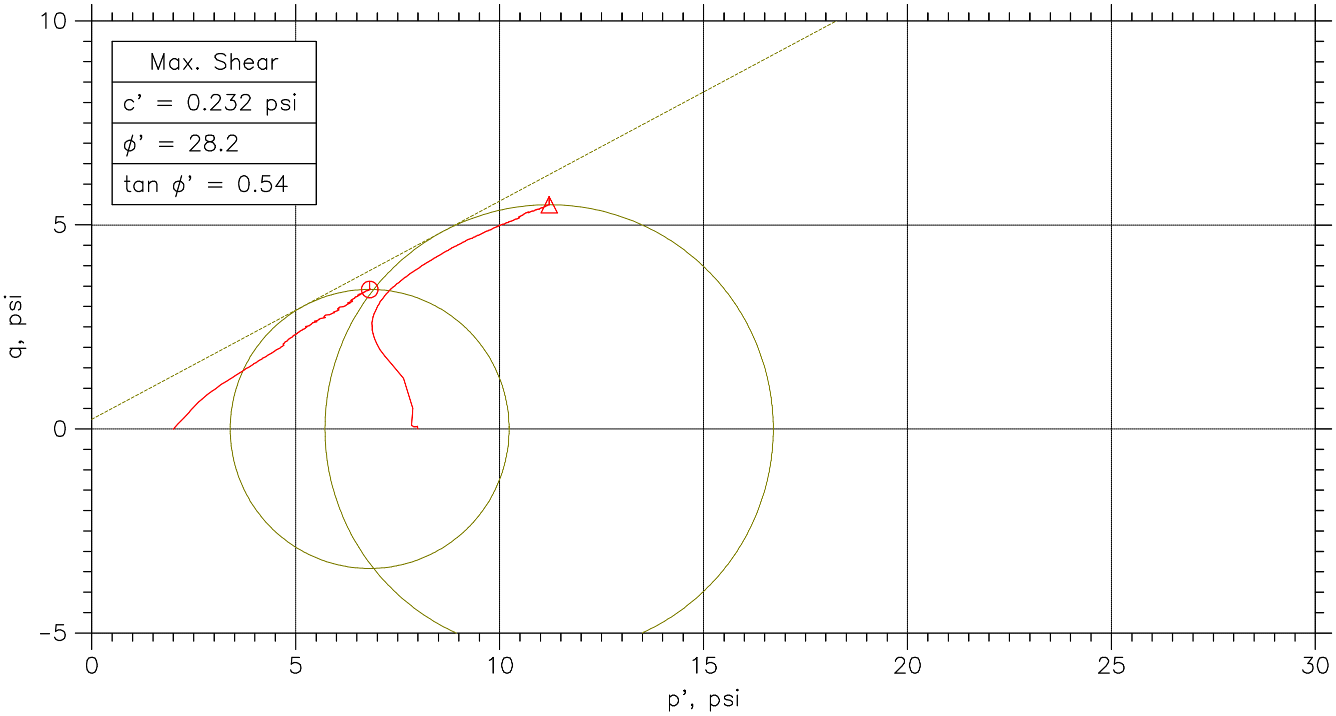
Project No.	*6162191251.14*	Tested By	*ML*
Project Name	*Erin Lake Dam*	Test Date	*7/26/2021*
Boring No.	*AB-2A*	Reviewed By	*JPK*
Sample No.	*N/A*	Review Date	*8/3/2021*
Sample Depth	*10-12'*	Lab No.	*20066*
Sample Description	*Brown Silty SAND with Clay*		

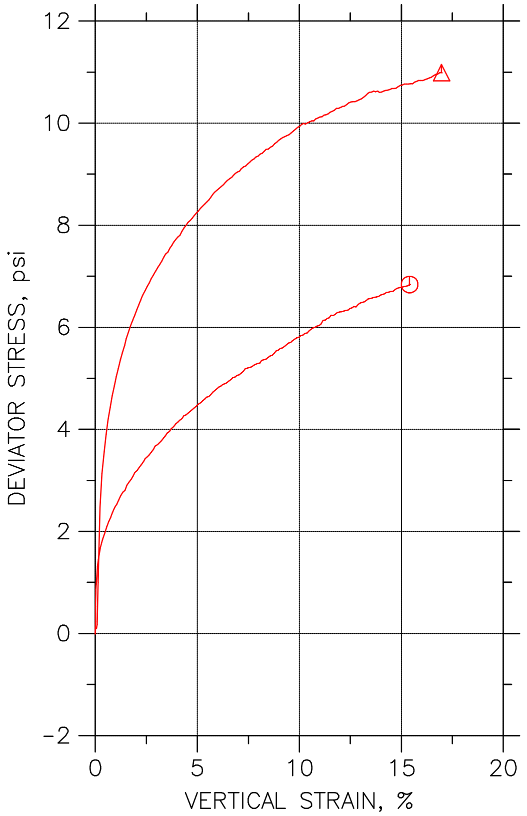
ASTM D5084 - Method F (CVFH)

Sample Type:	*UD*
Sample Orientation:	*Vertical*
Initial Water Content, %:	*23.8*
Wet Unit Weight, pcf:	*122.5*
Dry Unit Weight, pcf:	*98.9*
Compaction, %:	*N/A*
Hydraulic Conductivity, cm/sec. @20 °C	***2.7E-07***

Remarks: ______________________________

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767





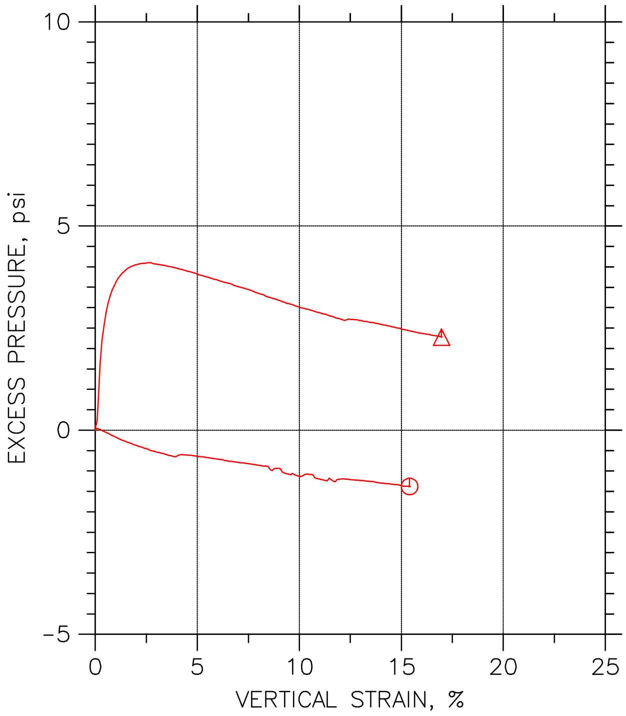
Symbol		○	△		
Sample No.		N/A	N/A		
Test No.		1	1		
Depth		22-24'	22-24'		
Initial	Diameter, in	2.85	2.84		
	Height, in	4.07	5.99		
	Water Content, %	28.0	26.7		
	Dry Density, pcf	93.96	97.44		
	Saturation, %	95.4	98.7		
	Void Ratio	0.794	0.73		
Before Shear	Water Content, %	29.4	26.4		
	Dry Density, pcf	94.	98.47		
	Saturation*, %	100.0	100.0		
	Void Ratio	0.793	0.712		
	Back Press., psi	89.99	90.		
Ver. Eff. Cons. Stress, psi		1.997	7.997		
Shear Strength, psi		3.419	5.495		
Strain at Failure, %		15.4	17		
Strain Rate, %/min		0.05	0.05		
B-Value		0.96	0.98		
Estimated Specific Gravity		2.7	2.7		
Liquid Limit		---	---		
Plastic Limit		---	---		

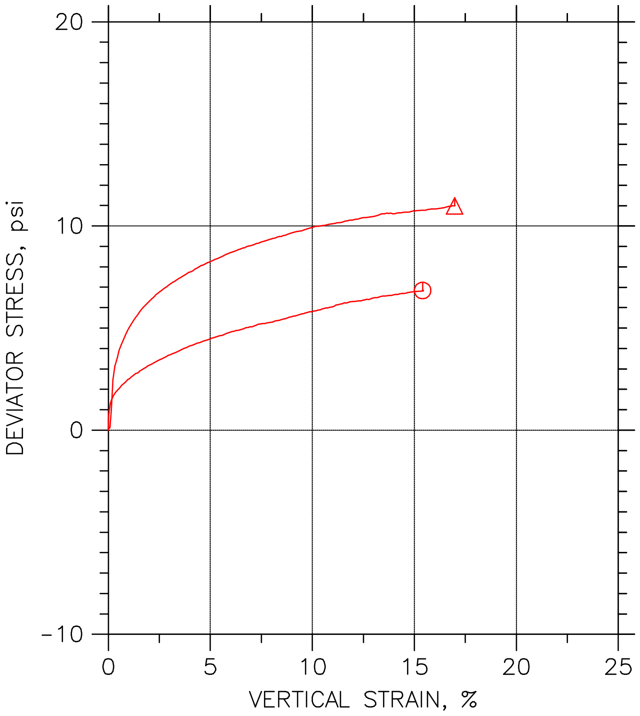
Project: Erin Lake Dam
Location: N/A
Project No.: 6162191251
Boring No.: AB-2A
Sample Type: Intact
Description: Brown Clayey SAND
Remarks: Lab No.20067

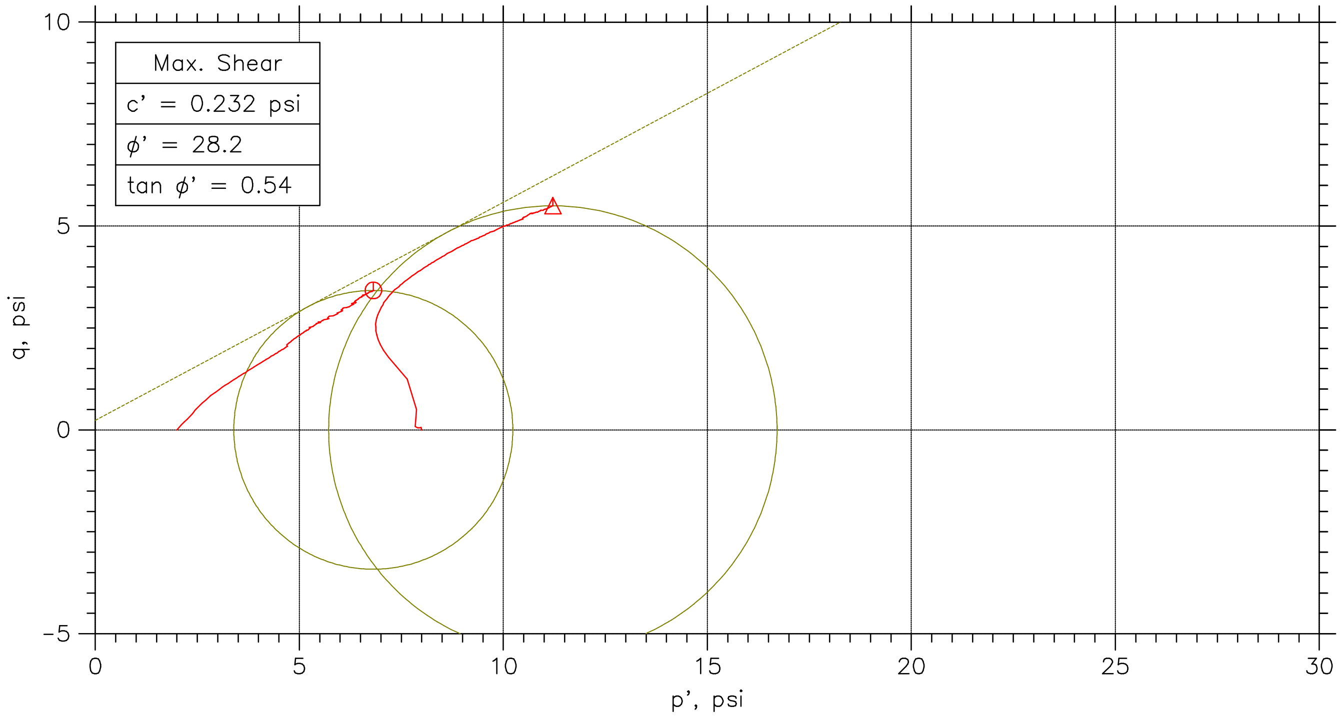
Wed, 25-AUG-2021 14:26:57

Phase calculations based on start of test.
* Saturation is set to 100% for phase calculations.

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767







	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
○	N/A	1	22-24'	ML	7/29/21	JPK		20067.1.dat
△	N/A	1	22-24'	ML	7/28/21	JPK		20067.3.dat

Project: Erin Lake Dam	Location: N/A	Project No.: 6162191251
Boring No.: AB-2A	Sample Type: Intact	
Description: Brown Clayey SAND		
Remarks: Lab No.20067		

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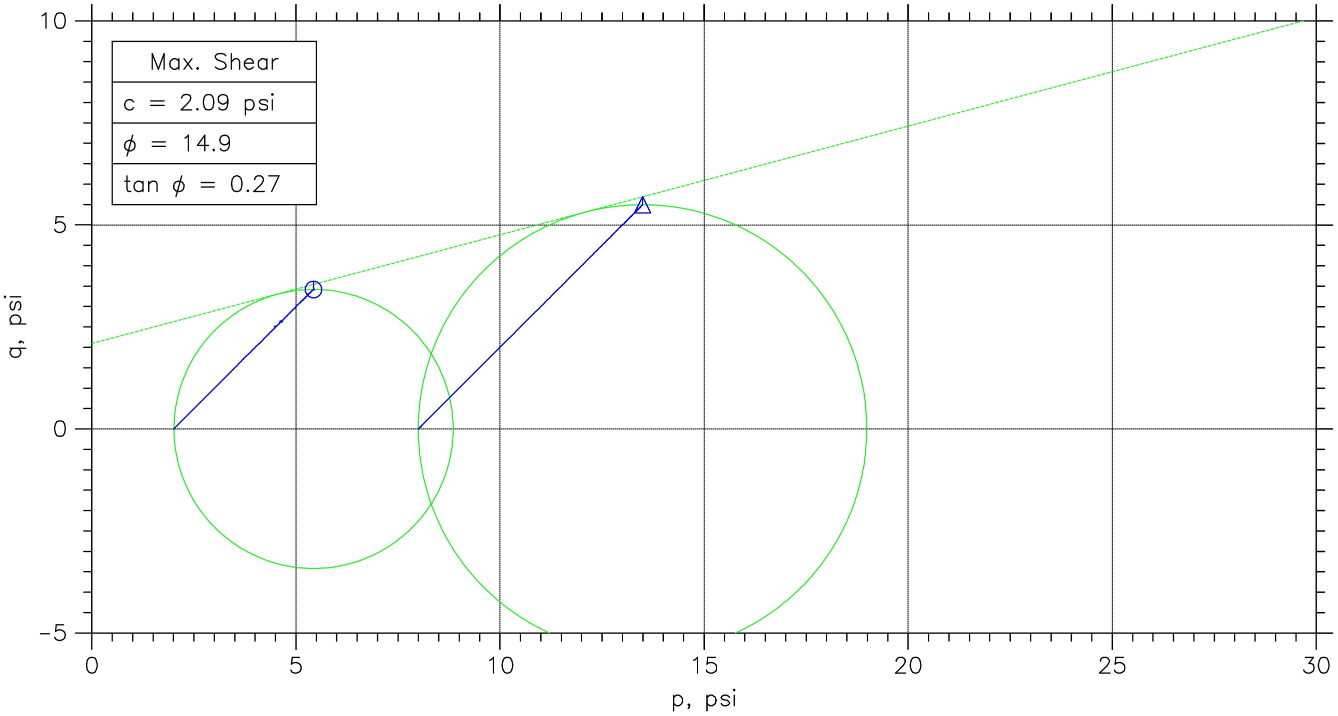
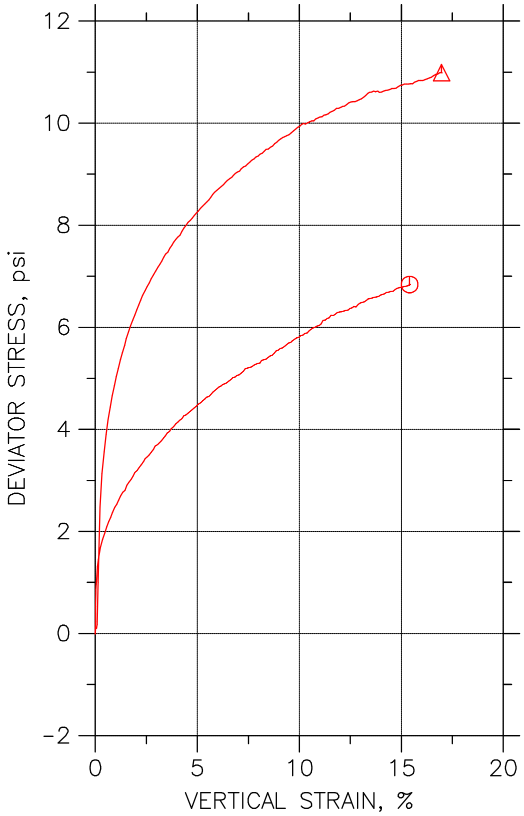
CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767





Symbol		○	△		
Sample No.		N/A	N/A		
Test No.		1	1		
Depth		22-24'	22-24'		
Initial	Diameter, in	2.85	2.84		
	Height, in	4.07	5.99		
	Water Content, %	28.0	26.7		
	Dry Density, pcf	93.96	97.44		
	Saturation, %	95.4	98.7		
	Void Ratio	0.794	0.73		
Before Shear	Water Content, %	29.4	26.4		
	Dry Density, pcf	94.	98.47		
	Saturation*, %	100.0	100.0		
	Void Ratio	0.793	0.712		
	Back Press., psi	89.99	90.		
Ver. Eff. Cons. Stress, psi		1.997	7.997		
Shear Strength, psi		3.419	5.495		
Strain at Failure, %		15.4	17		
Strain Rate, %/min		0.05	0.05		
B-Value		0.96	0.98		
Estimated Specific Gravity		2.7	2.7		
Liquid Limit		---	---		
Plastic Limit		---	---		

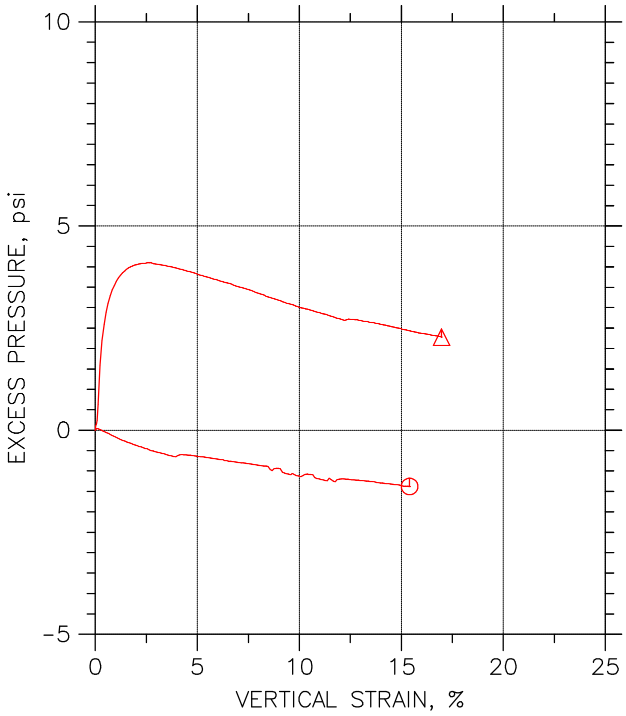
Project: Erin Lake Dam
Location: N/A
Project No.: 6162191251
Boring No.: AB-2A
Sample Type: Intact
Description: Brown Clayey SAND
Remarks: Lab No.20067

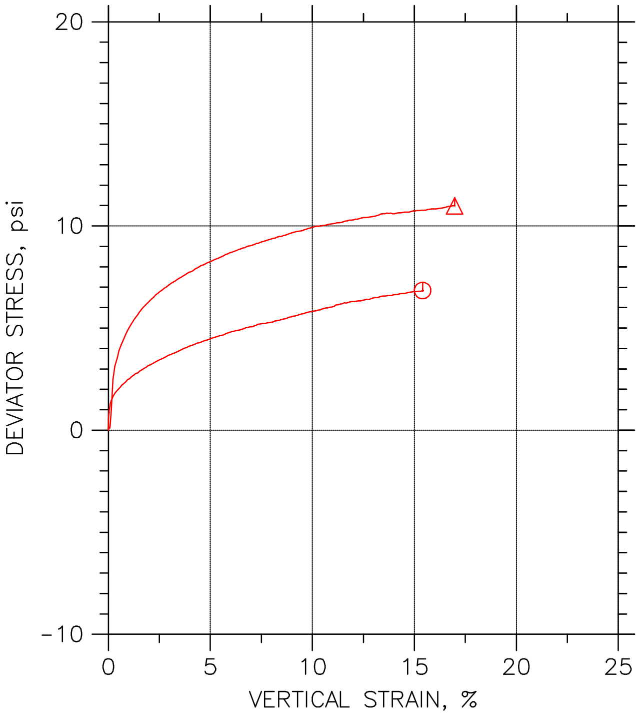
Wed, 25-AUG-2021 14:26:34

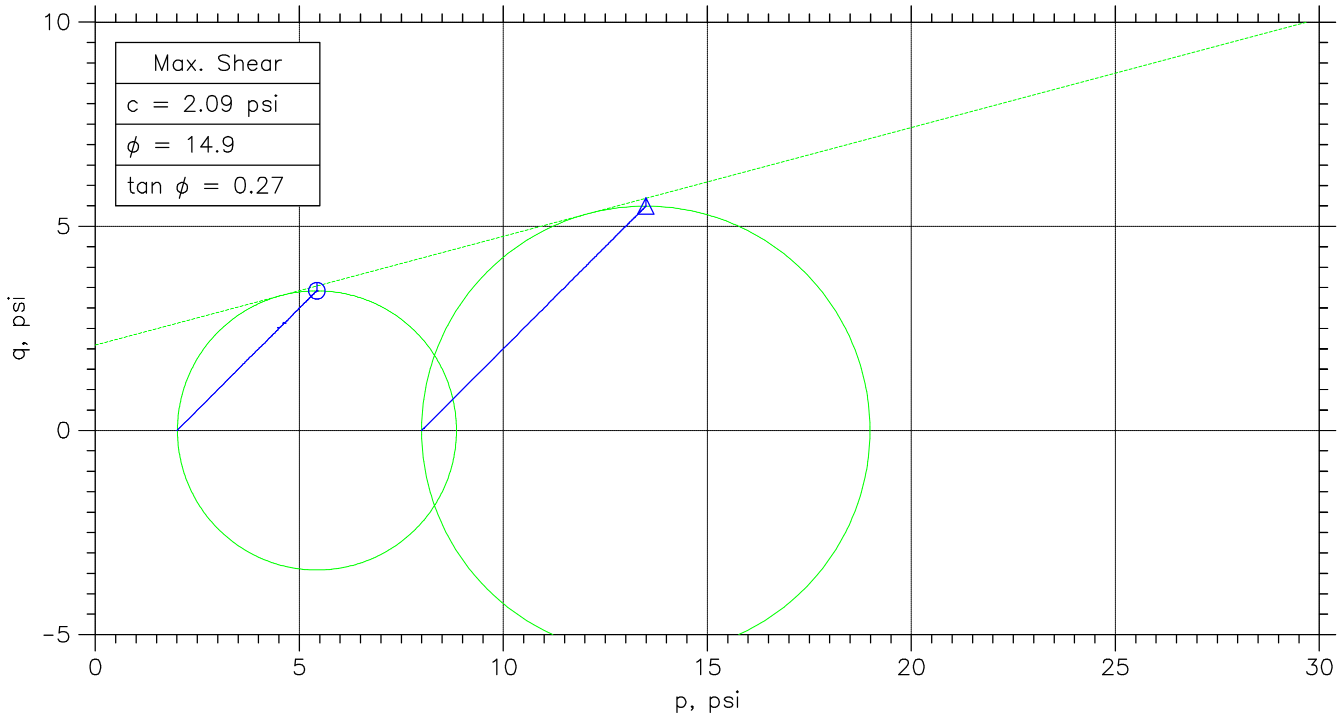
Phase calculations based on start of test.

* Saturation is set to 100% for phase calculations.

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767





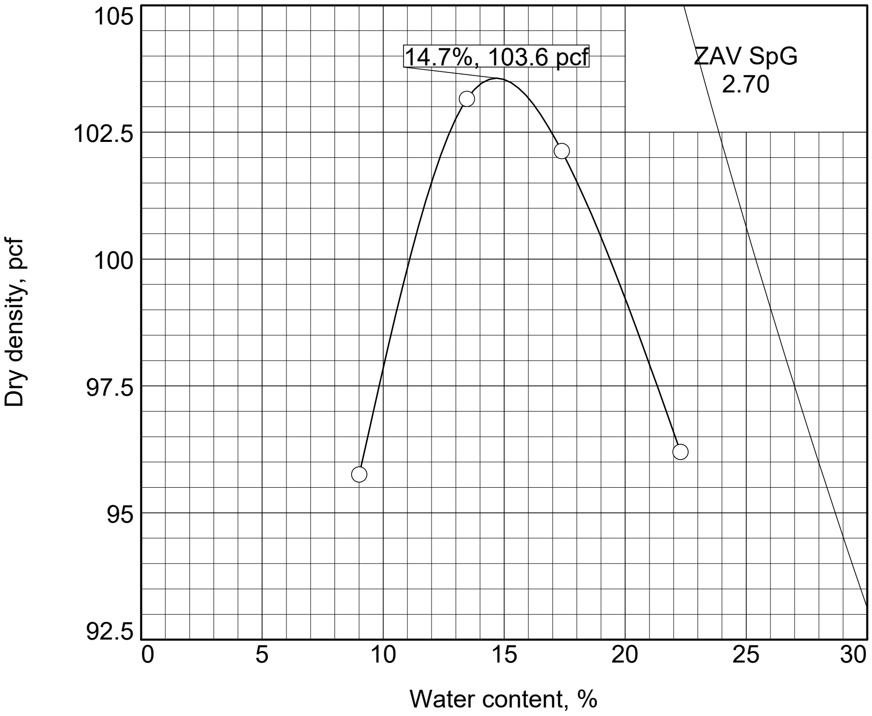


	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
○	N/A	1	22-24'	ML	7/29/21	JPK		20067.1.dat
△	N/A	1	22-24'	ML	7/28/21	JPK		20067.3.dat

Project: Erin Lake Dam	Location: N/A	Project No.: 6162191251
Boring No.: AB-2A	Sample Type: Intact	
Description: Brown Clayey SAND		
Remarks: Lab No.20067		

Wed, 25-AUG-2021 14:26:35

COMPACTION TEST REPORT



Curve No.
2

Test Specification:
ASTM D 698-12 Method A Standard

Hammer Wt.: 5.5 lb.
Hammer Drop: 12 in.
Number of Layers: three
Blows per Layer: 25
Mold Size: 0.03333 cu. ft.
Test Performed on Material Passing #4 **Sieve**

Soil Data

NM	NT	**Sp.G.**	2.7
LL	NT	**PI**	NT
%>#4		**%<#200**	
USCS	SM	**AASHTO**	N/A

TESTING DATA

	1	2	3	4	5	6
WM + WS	3641.1	3832.4	3875.3	3841.5		
WM	2063.0	2063.0	2063.0	2063.0		
WW + T #1	599.3	645.2	661.2	620.9		
WD + T #1	557.8	585.5	584.5	526.0		
TARE #1	97.3	141.9	143.1	100.2		
WW + T #2						
WD + T #2						
TARE #2						
MOISTURE	9.0	13.5	17.4	22.3		
DRY DENSITY	95.8	103.2	102.1	96.2		

TEST RESULTS

Maximum dry density = 103.6 pcf

Optimum moisture = 14.7 %

Material Description

Light Brown Silty SAND

Project No. 6162191251 **Client:** Accura Engineering
Project: Erin Lake Dam
Depth:0-10'
○ **Source:** Combination AB-3 and AB-5 **Sample No.:** N/A

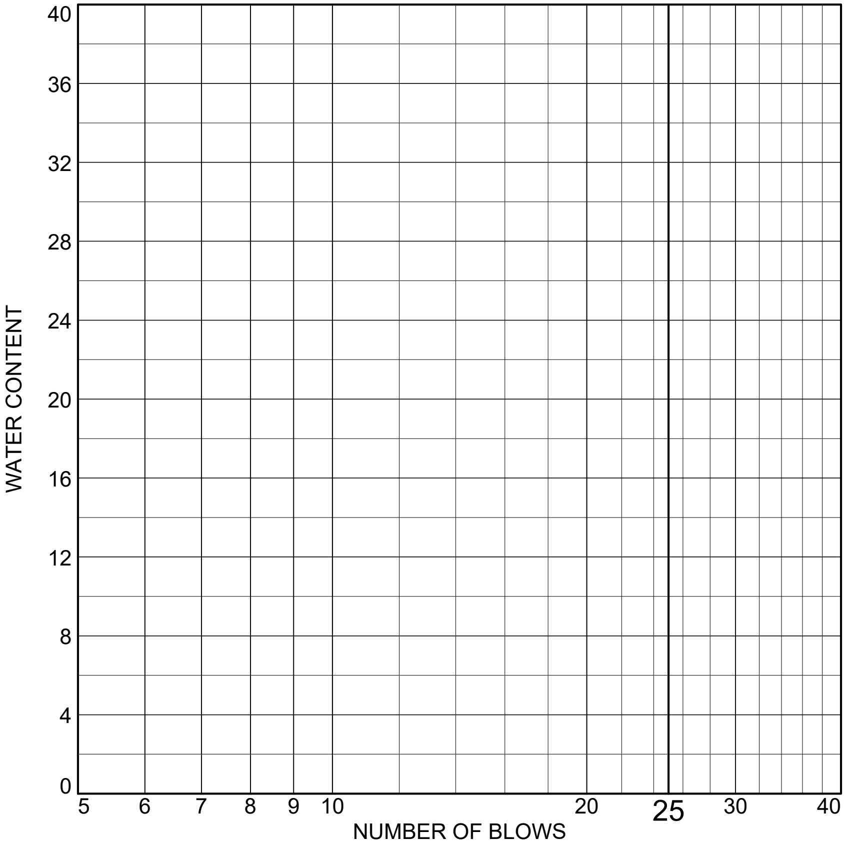
Remarks:
Lab No.20122

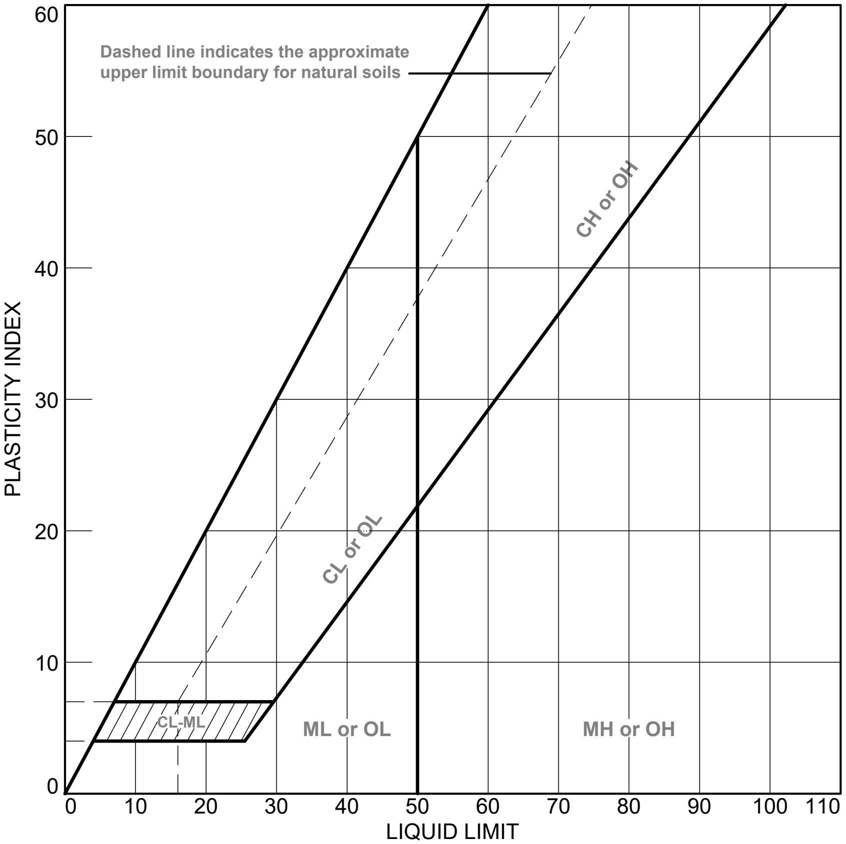
Figure

These results relate only to the items tested. This report shall not be reproduced, except in full, without written approval from Wood.

Tested By: DC **Checked By:** ML

LIQUID AND PLASTIC LIMITS TEST REPORT





	SOURCE	SAMPLE #	DEPTH/ELEV.	DATE SAMPLED	USCS	MATERIAL DESCRIPTION	NM %	LL	PI
●	AB-3	N/A	0-2'	8/20/21	SM	Reddish Brown Silty SAND	21.1	NV	NP

Client Accura Engineering

Project Erin Lake Dam

Project No. 6162191251

Figure

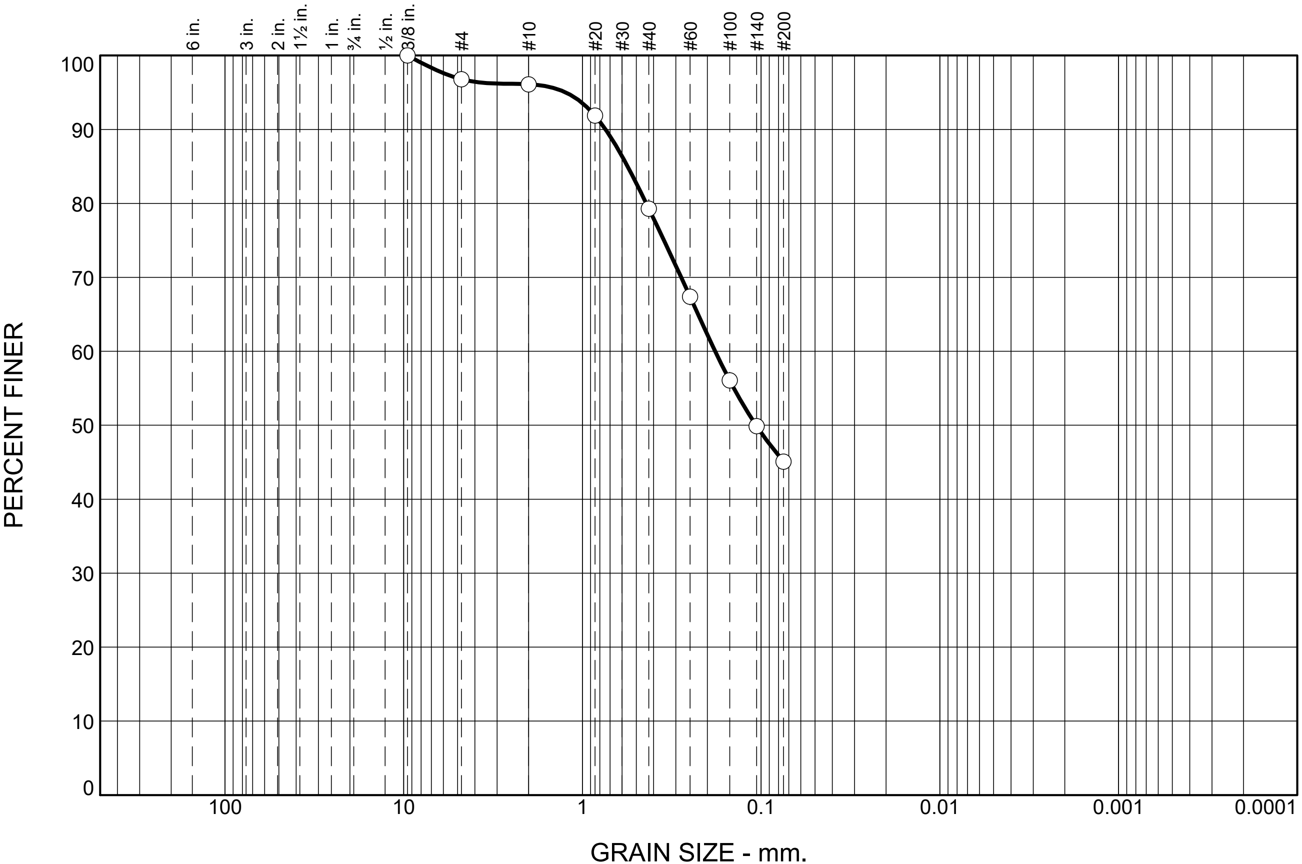
wood.

● Lab No.20123

These results relate only to the items tested. This report shall not be reproduced, except in full, without written approval from Wood.

Tested By: DS **Checked By:** ML

Grain Size Analysis (ASTM D6913-17)



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	3.2	0.7	16.8	34.2	45.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.375"	100.0		
#4	96.8		
#10	96.1		
#20	91.9		
#40	79.3		
#60	67.4		
#100	56.1		
#140	49.9		
#200	45.1		

* (no specification provided)

Material Description

Reddish Brown Silty SAND

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{90}= 0.7421 D_{85}= 0.5588 D_{60}= 0.1807

D_{50}= 0.1068 D_{30}= D_{15}=

D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO= A-4(0)

Remarks

Lab No.20123

Source of Sample: AB-3 **Depth:** 0-2'

Sample Number: N/A

Date: 8/20/21

wood.

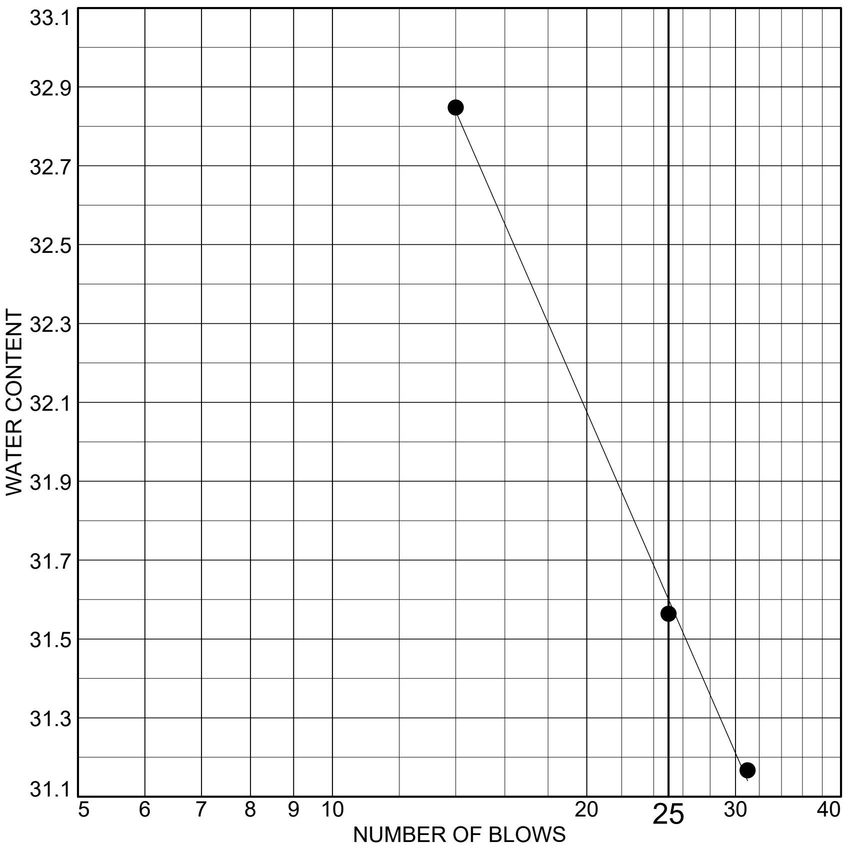
Client: Accura Engineering

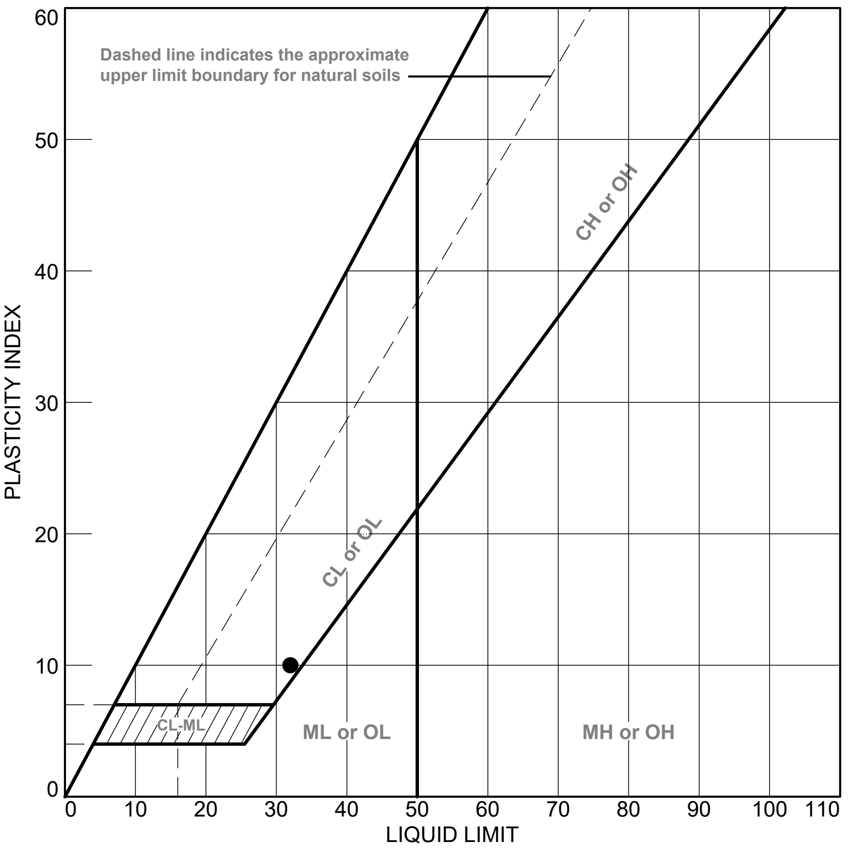
Project: Erin Lake Dam

Project No: 6162191251 **Figure**

Tested By: JF **Checked By:** ML

LIQUID AND PLASTIC LIMITS TEST REPORT





	SOURCE	SAMPLE #	DEPTH/ELEV.	DATE SAMPLED	USCS	MATERIAL DESCRIPTION	NM %	LL	PI
●	AB-3	N/A	6-8'	8/16/21	SC	Light Brown Clayey SAND	28.5	32	10

Client Accura Engineering

Project Erin Lake Dam

Project No. 6162191251

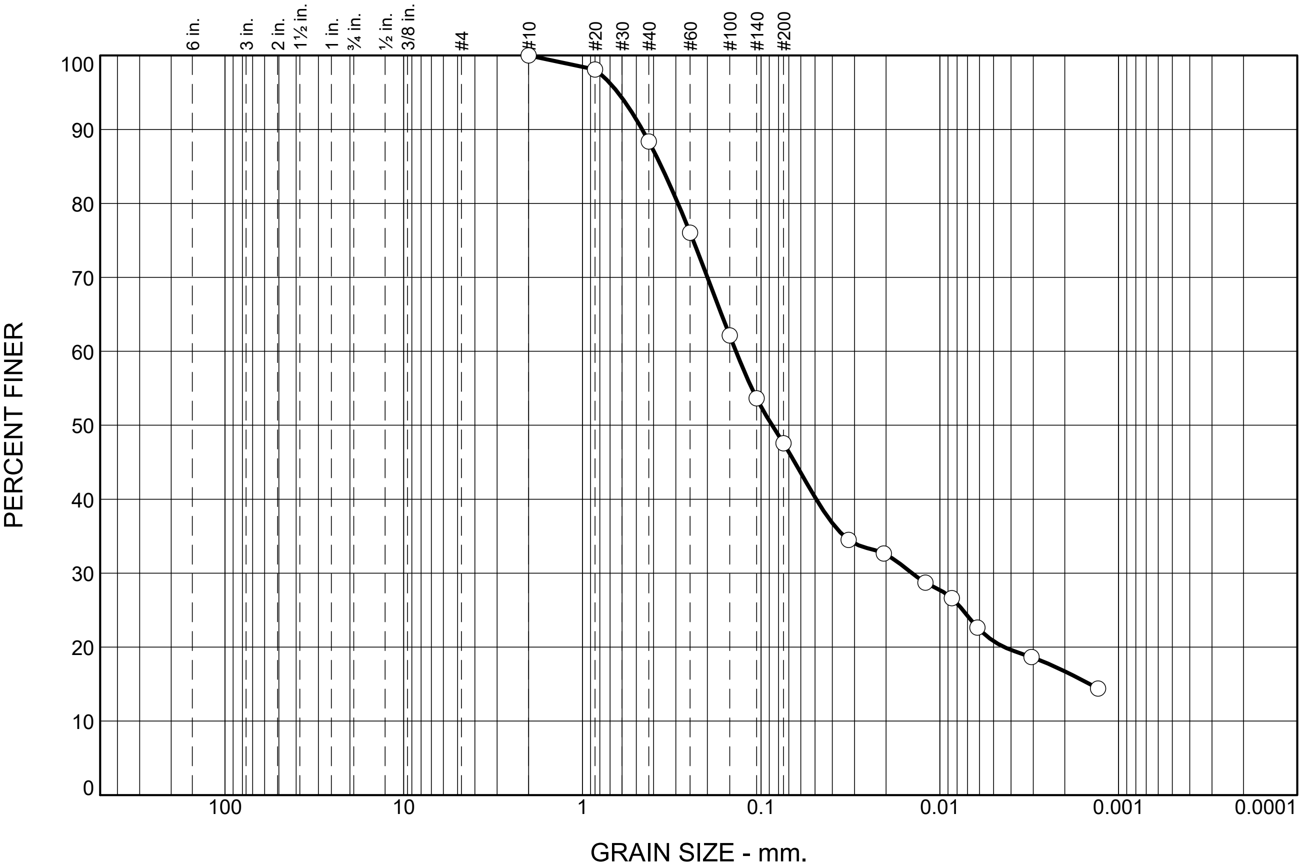
Figure

● Lab No.20124

These results relate only to the items tested. This report shall not be reproduced, except in full, without written approval from Wood.

Tested By: DS **Checked By:** ML

Grain Size Analysis (ASTM D422-63)



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	11.6	40.8	26.8	20.8

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#20	98.1		
#40	88.4		
#60	76.0		
#100	62.1		
#140	53.6		
#200	47.6		

* (no specification provided)

Material Description

Light Brown Clayey SAND

Atterberg Limits

PL= 22 LL= 32 PI= 10

Coefficients

D_{90}= 0.4628 D_{85}= 0.3618 D_{60}= 0.1384
D_{50}= 0.0868 D_{30}= 0.0143 D_{15}= 0.0014
D_{10}= C_u= C_c=

Classification

USCS= SC AASHTO= A-4(2)

Remarks

Lab No.20124

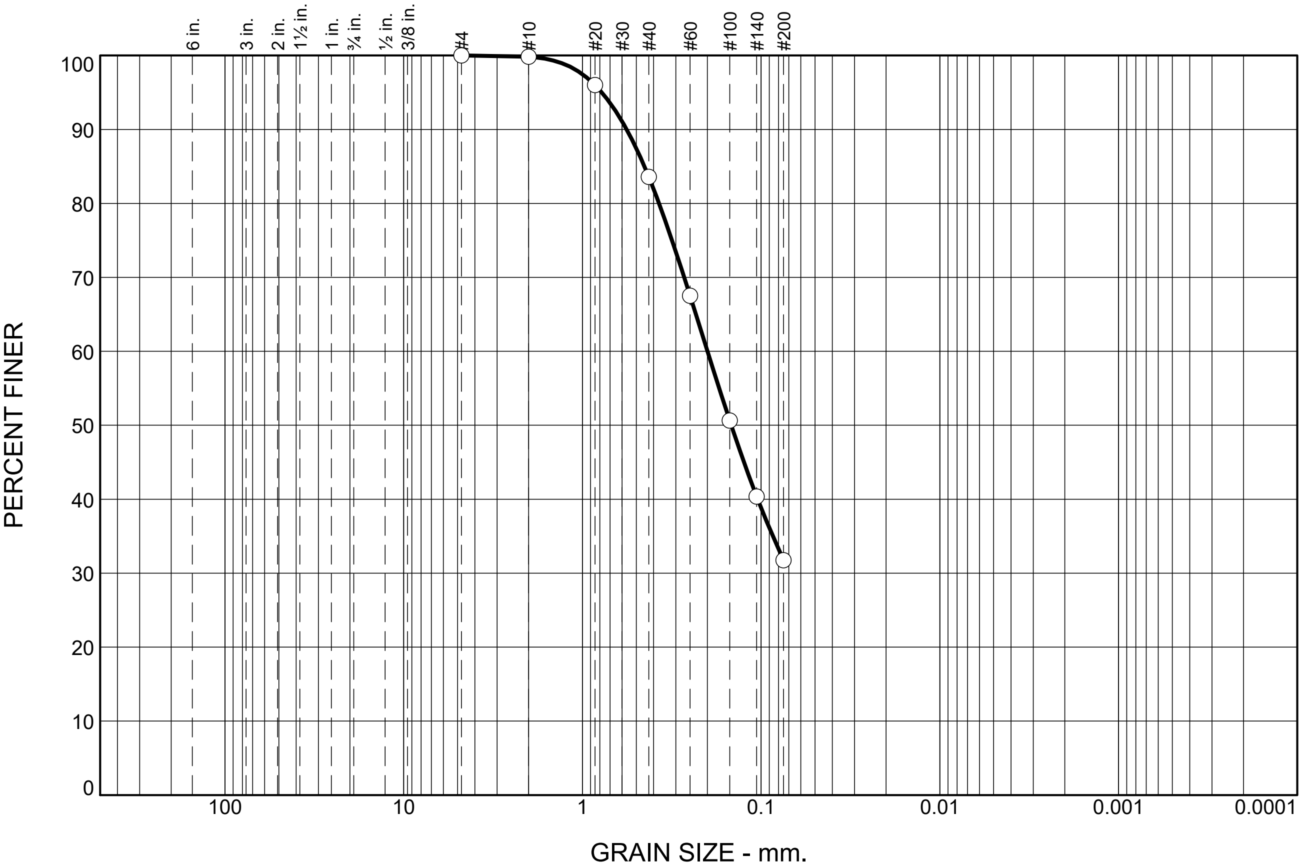
Source of Sample: AB-3 **Depth:** 6-8'
Sample Number: N/A **Date:** 8/16/21

wood.

Client: Accura Engineering
Project: Erin Lake Dam
Project No: 6162191251 **Figure**

Tested By: DW **Checked By:** ML

Grain Size Analysis (ASTM D6913-17)



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.2	16.2	51.9	31.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	99.8		
#20	96.0		
#40	83.6		
#60	67.5		
#100	50.6		
#140	40.4		
#200	31.7		

* (no specification provided)

Material Description

Light Brown Silty SAND

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.5659 D_{85}= 0.4497 D_{60}= 0.1996
D_{50}= 0.1470 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO= N/A

Remarks

Lab No.20125
Natural Moisture Content:49.8%

Source of Sample: AB-3 **Depth:** 12-14'
Sample Number: N/A **Date:** 8/13/21

wood.

Client: Accura Engineering
Project: Erin Lake Dam
Project No: 6162191251 **Figure**

Tested By: JF **Checked By:** ML

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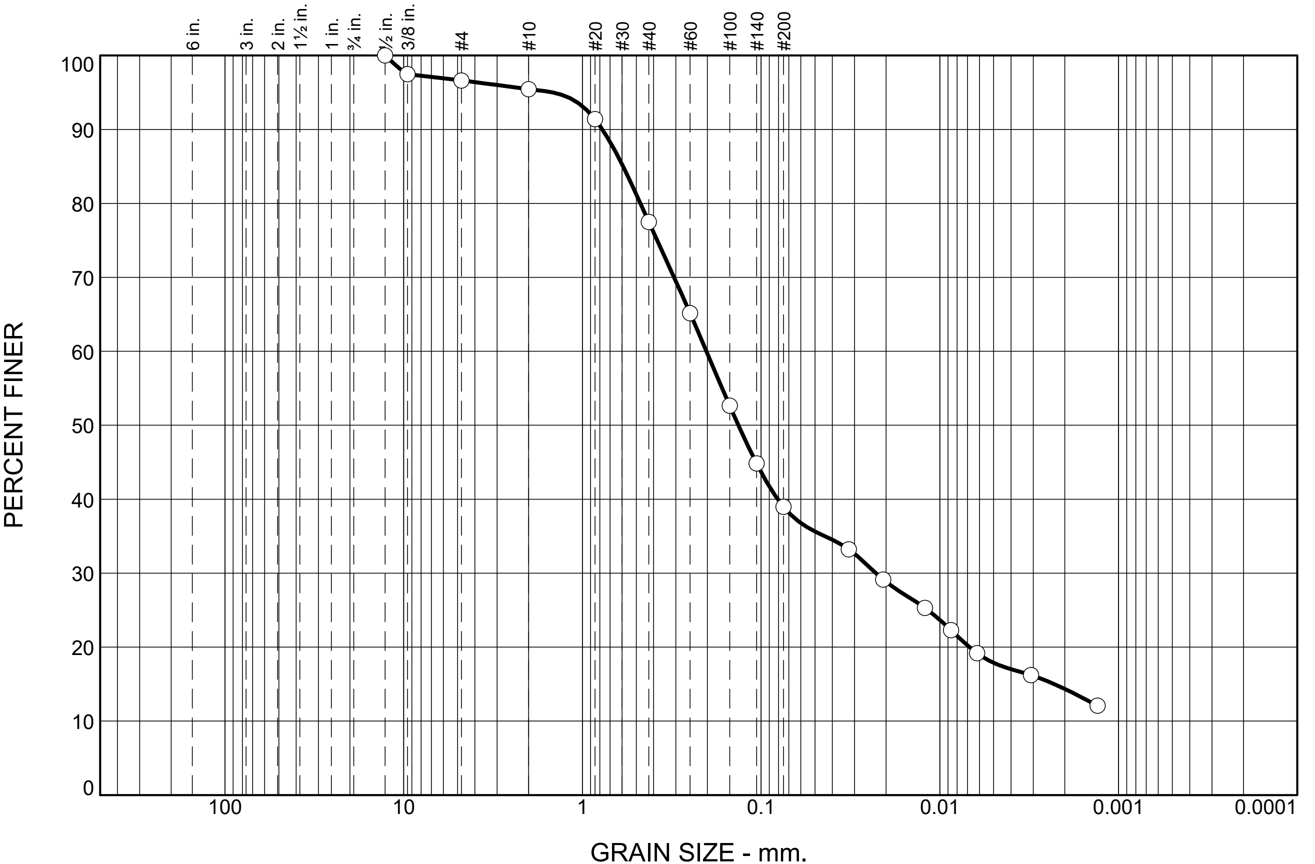
Grain Size Analysis (ASTM D422-63)



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	3.4	1.2	17.9	38.5	21.1	17.9

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.500"	100.0		
0.375"	97.5		
#4	96.6		
#10	95.4		
#20	91.4		
#40	77.5		
#60	65.2		
#100	52.6		
#140	44.8		
#200	39.0		

* (no specification provided)

Material Description

Reddish Brown Silty SAND

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.7727 D_{85}= 0.5926 D_{60}= 0.2024
D_{50}= 0.1341 D_{30}= 0.0228 D_{15}= 0.0023
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO= N/A

Remarks

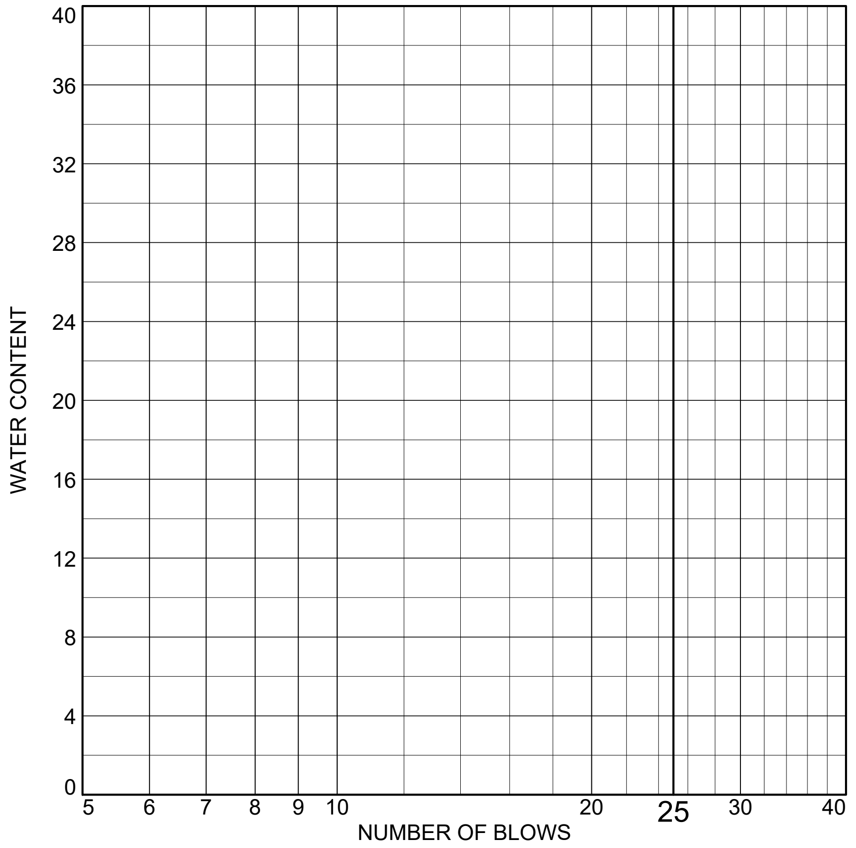
Lab No.20126
Natural Moisture Content:20.3

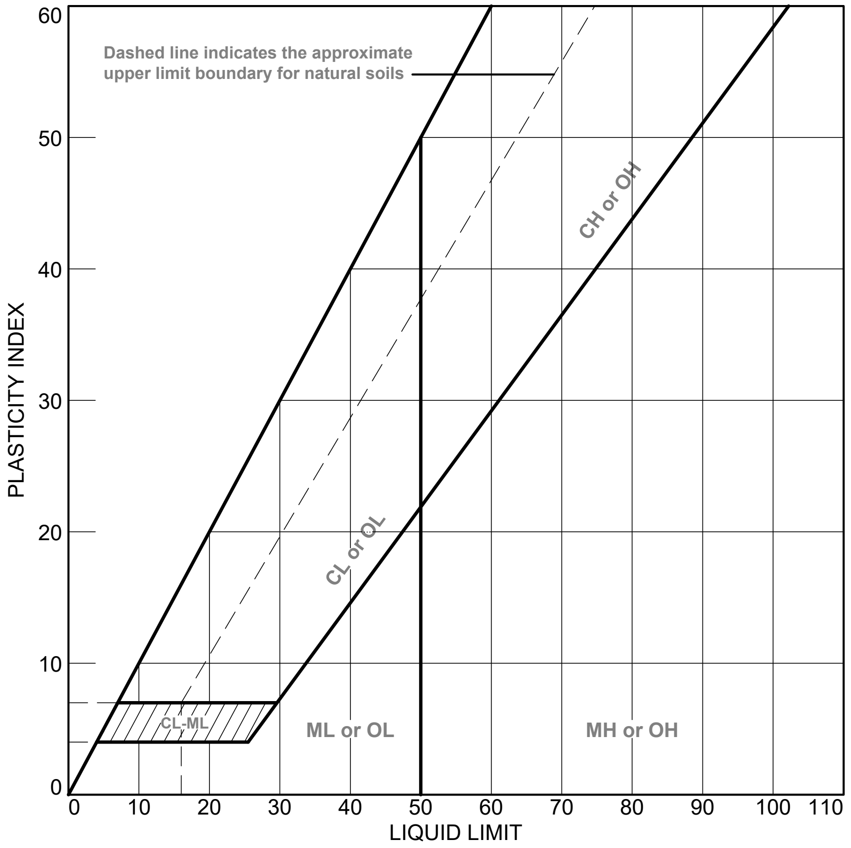
Source of Sample: AB-5 **Depth:** 0-2'
Sample Number: N/A **Date:** 8/16/21

Client: Accura Engineering
Project: Erin Lake Dam
Project No: 6162191251 **Figure**

Tested By: DW **Checked By:** ML

LIQUID AND PLASTIC LIMITS TEST REPORT





	SOURCE	SAMPLE #	DEPTH/ELEV.	DATE SAMPLED	USCS	MATERIAL DESCRIPTION	NM %	LL	PI
●	AB-5	N/A	10-12'	8/13/21	SM	Brown Silty SAND	33.2	NV	NP

Client Accura Engineering

Project Erin Lake Dam

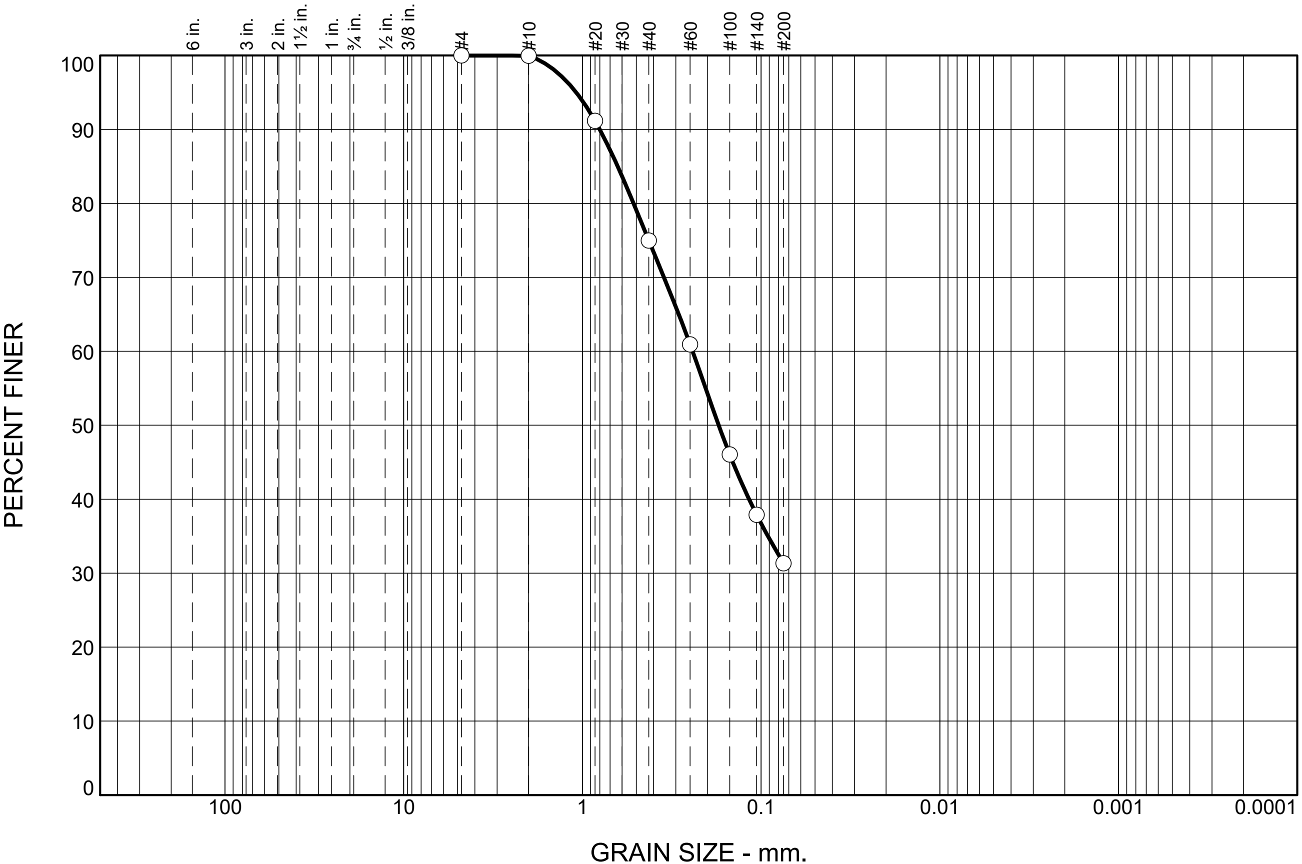
Project No. 6162191251 Figure

wood.

● Lab No.20127

Tested By: DW **Checked By:** ML

Grain Size Analysis (ASTM D6913-17)



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	25.0	43.7	31.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	100.0		
#20	91.2		
#40	75.0		
#60	60.9		
#100	46.0		
#140	37.9		
#200	31.3		

* (no specification provided)

Material Description

Brown Silty SAND

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{90}= 0.7997 D_{85}= 0.6351 D_{60}= 0.2419
D_{50}= 0.1728 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO= A-2-4(0)

Remarks

Lab No.20127

Source of Sample: AB-5 **Depth:** 10-12'
Sample Number: N/A **Date:** 8/13/21

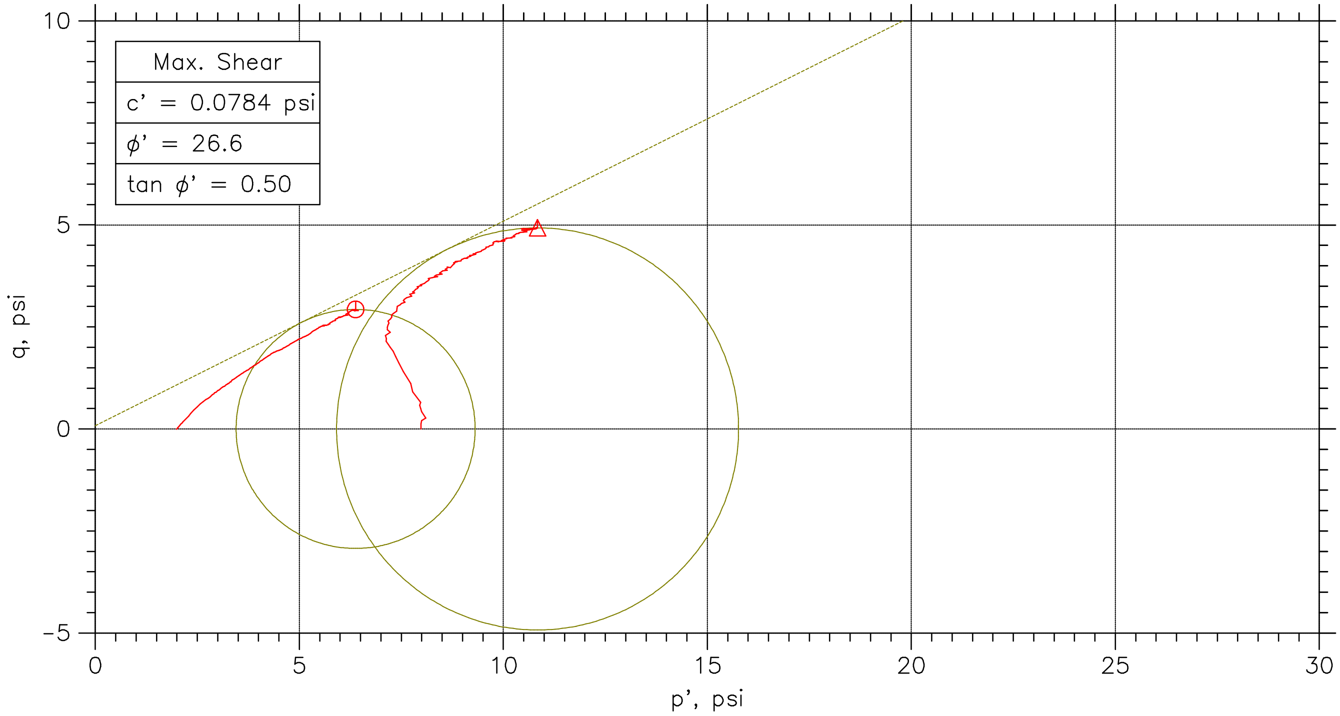
wood.

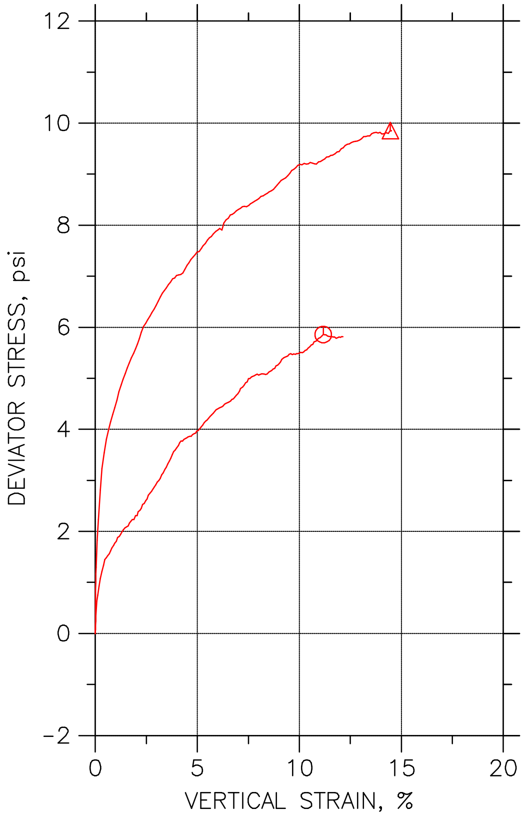
Client: Accura Engineering
Project: Erin Lake Dam
Project No: 6162191251 **Figure**

These results relate only to the items tested. This report shall not be reproduced, except in full, without written approval from Wood.

Tested By: JF **Checked By:** ML

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767





Symbol		○	△		
Sample No.		N/A	N/A		
Test No.		1	1		
Depth		7-9'	7-9'		
Initial	Diameter, in	2.8	2.86		
	Height, in	5.69	5.79		
	Water Content, %	25.6	21.4		
	Dry Density, pcf	106.4	99.15		
	Saturation, %	118.2	82.4		
	Void Ratio	0.585	0.7		
Before Shear	Water Content, %	25.3	19.6		
	Dry Density, pcf	100.1	110.2		
	Saturation*, %	100.0	100.0		
	Void Ratio	0.683	0.53		
	Back Press., psi	90.	89.99		
Ver. Eff. Cons. Stress, psi		1.997	7.981		
Shear Strength, psi		2.929	4.926		
Strain at Failure, %		11.2	14.5		
Strain Rate, %/min		0.05	0.05		
B-Value		0.96	0.96		
Estimated Specific Gravity		2.7	2.7		
Liquid Limit		---	---		
Plastic Limit		---	---		

wood.

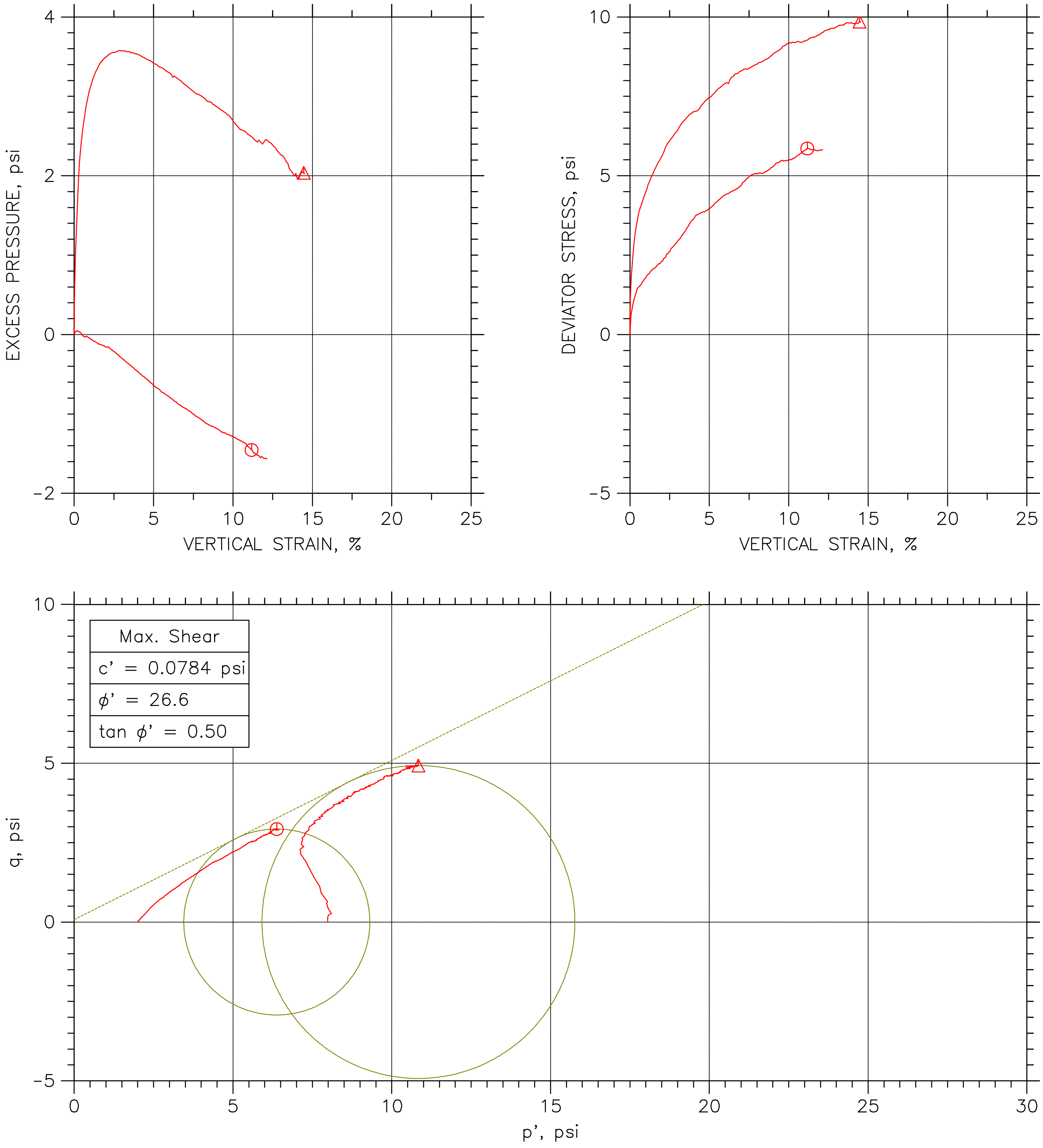
Project: Erin Lake Dam
Location: N/A
Project No.: 6162191251
Boring No.: AB-3A
Sample Type: Intact
Description: Reddish Brown Sandy SILT
Remarks: Lab No.20128

Wed, 25-AUG-2021 14:24:38

Phase calculations based on start of test.

* Saturation is set to 100% for phase calculations

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767



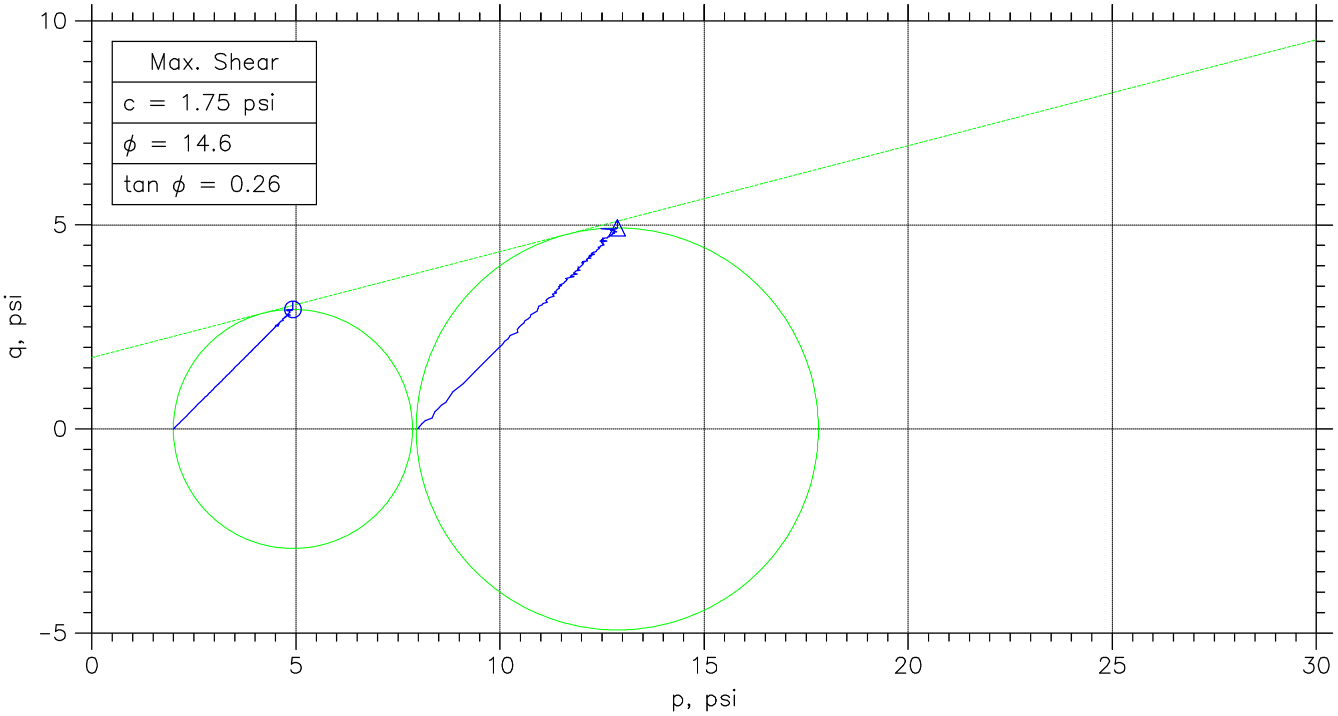
	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
○	N/A	1	7-9'	ML	8/16/21	MRF		20128.1.dat
△	N/A	1	7-9'	ML	8/18/21	MRF		20128.3.dat

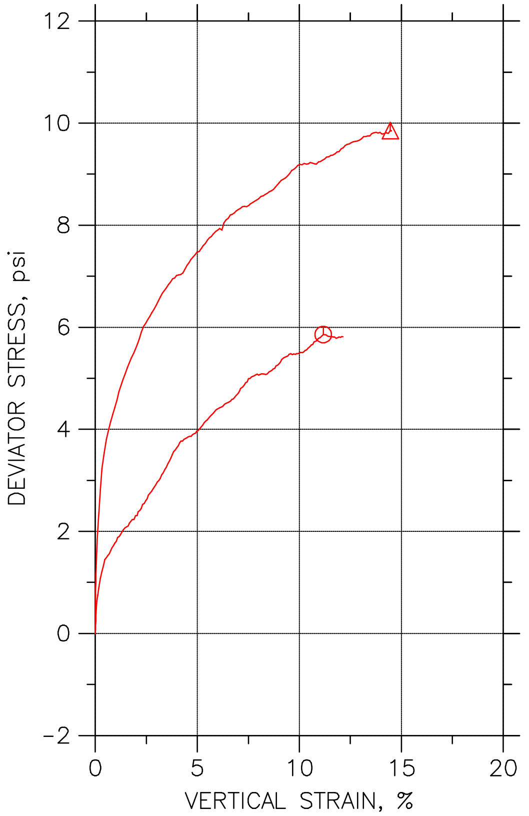
wood.

Project: Erin Lake Dam
Location: N/A
Project No.: 6162191251
Boring No.: AB-3A
Sample Type: Intact
Description: Reddish Brown Sandy SILT
Remarks: Lab No.20128

Wed, 25-AUG-2021 14:24:39

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767





Symbol		○	△		
Sample No.		N/A	N/A		
Test No.		1	1		
Depth		7-9’	7-9’		
Initial	Diameter, in	2.8	2.86		
	Height, in	5.69	5.79		
	Water Content, %	25.6	21.4		
	Dry Density, pcf	106.4	99.15		
	Saturation, %	118.2	82.4		
	Void Ratio	0.585	0.7		
Before Shear	Water Content, %	25.3	19.6		
	Dry Density, pcf	100.1	110.2		
	Saturation*, %	100.0	100.0		
	Void Ratio	0.683	0.53		
	Back Press., psi	90.	89.99		
Ver. Eff. Cons. Stress, psi		1.997	7.981		
Shear Strength, psi		2.929	4.926		
Strain at Failure, %		11.2	14.5		
Strain Rate, %/min		0.05	0.05		
B-Value		0.96	0.96		
Estimated Specific Gravity		2.7	2.7		
Liquid Limit		---	---		
Plastic Limit		---	---		

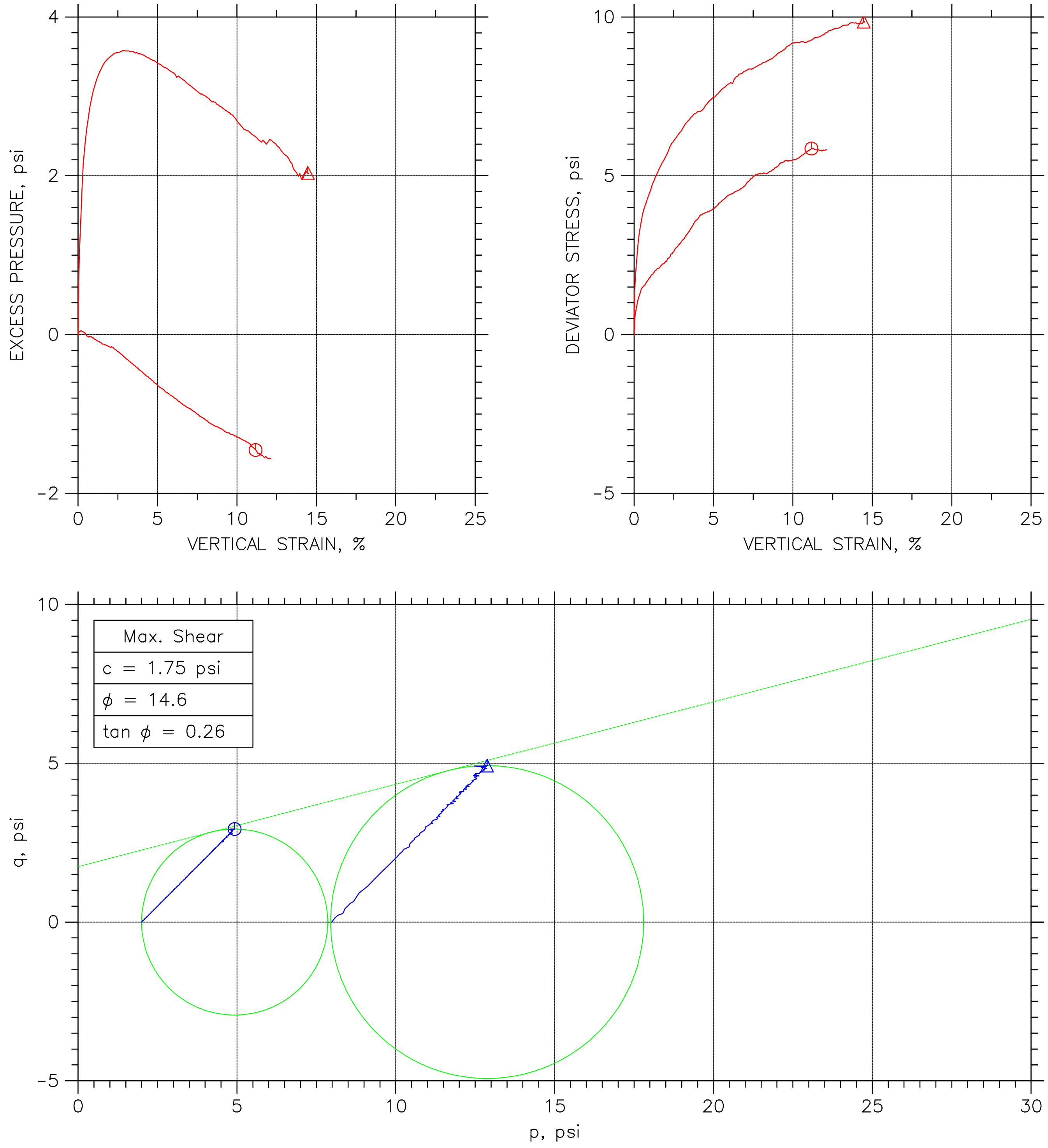
wood.

Project: Erin Lake Dam
Location: N/A
Project No.: 6162191251
Boring No.: AB-3A
Sample Type: Intact
Description: Reddish Brown Sandy SILT
Remarks: Lab No.20128

Wed, 25-AUG-2021 14:25:02

Phase calculations based on start of test.
* Saturation is set to 100% for phase calculations.

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767



	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
○	N/A	1	7-9'	ML	8/16/21	MRF		20128.1.dat
△	N/A	1	7-9'	ML	8/18/21	MRF		20128.3.dat

wood.

Project: Erin Lake Dam

Location: N/A

Project No.: 6162191251

Boring No.: AB-3A

Sample Type: Intact

Description: Reddish Brown Sandy SILT

Remarks: Lab No.20128

Wed, 25-AUG-2021 14:25:03

LABORATORY TESTING PROCEDURES

Moisture Content

The moisture content was determined for selected soil samples obtained in the split-barrel sampler. A representative portion of each sample was weighed and then placed in an oven and dried at 110 degrees Centigrade for at least 15 to 16 hours. After removal from the oven, the soil was again weighed. The weight of the moisture lost during drying thus was determined. From this data, the moisture content of the sample was then calculated as the weight of moisture divided by dry weight of soil, expressed as a percentage. This test was conducted according to ASTM D 2216.

Moisture content is a useful index of a soil's compressibility. If the soil is to be used as fill, the moisture content may be compared to the range of water contents for which proper compaction may be achieved. These moisture contents may be found at the appropriate depths on the respective Boring Logs and are denoted by "w".

Grain Size (Sieve) Analysis with or without Hydrometer

Grain Size Analysis tests were performed to determine the particle size distribution of selected samples tested. The grain size distribution of soils coarser than a number 200 sieve was determined by passing the samples through a standard set of nested sieves. Materials finer than the number 200 sieve were suspended in water and the grain size distribution computed from the time rate of settlement of the different size particles. Air-dried soil passed through #200 sieve. 50 grams of that must soak in s/c agent for a minimum of 8 hours. Soil is then put in graduated cylinder with a hydrometer. Readings are taken at specified times. A graph is drawn from data. These tests were similar to those described by ASTM D 421 and D 422. The results are included in the Appendix.

Liquid and Plastic Limits (Atterberg Limits)

Liquid Limit and Plastic Limit tests aid in the classification of the soils and provide an indication of the soil behavior with moisture change. The Plasticity Index is calculated by subtracting the Plastic Limit (PL) from the Liquid Limit (LL). The Liquid Limit is the moisture content at which the soil will flow as a heavy viscous fluid and is the upper limit of the plastic range, as determined in accordance with ASTM D 4318. The Plastic Limit is the moisture content at which the soil begins to lose its plasticity, as determined in accordance with ASTM D 4318. The Liquidity Index is the ratio of the difference between the in-place moisture and the plastic limit to the Plasticity Limit. Soil is air-dried and pulverized to pass through #40 sieve prior to running the test. The results are shown on the attached Liquid and Plastic Limits reports in the Appendix.

Undisturbed Sampling

Split-barrel samples and/or auger cuttings are suitable for visual examination and classification tests but are not sufficiently intact for quantitative laboratory testing. Alternate sample methods are required.

For quantitative laboratory testing, relatively undisturbed samples were obtained by pushing sections of three-inch O.D., 16-gauge, steel or brass tubing (Shelby tube) into the soil at the desired sampling levels, as described in ASTM D 1587. The tube, together with the encased soil, was carefully removed from the ground, made airtight, and transported to the laboratory. Locations and depths of undisturbed samples were recorded on "Log of Boring".

Tri-axial Consolidated Undrained Test (CU)

Three specimens (with minimum of 6-inch long) are prepared from a relatively undisturbed sample. For insufficient recovery, a multistage tri-axial shear test was performed on one specimen. After preparation, the specimen is encased in a rubber membrane and is placed in the triaxial cell. The specimen is initially saturated using increasing confining pressures and increasing backpressures. Once the saturation is obtained, the desired all-around confining pressures are applied, and the axial load is increased until the specimen fails in shear or in excess of 15% strain is achieved. Readings are taken and then plotted in the form of Mohr's circles using the computer program.

Consolidation Test

A section of a selected undisturbed sample was extruded from its sampling tube for consolidation testing. The section was trimmed into a disc 2.5 inches in diameter and 0.75 inch thick. The disc was confined in a stainless-steel ring and sandwiched between porous stone plates. After being submerged in water (only for samples obtained below groundwater table), the sample was then subjected to incrementally increasing vertical loads and the resulting deformations measured with a micrometer dial gauge. The test procedure is described in ASTM D 2435. The test results are presented in the form of a pressure versus void ratio or pressure versus strain curve on the accompanying Consolidation Test Data Sheet using a computer program.

One-Dimensional Consolidation Test

This method covers the determination of the magnitude of one-dimensional swell or settlement potential of cohesive soils in accordance with ASTM D4546. Method A measures the free swell, percent heave for vertical confining pressures up to the swell pressure and the swell pressure.

Samples for this testing are prepared in accordance with test method for One-Dimensional Consolidation ASTM D2435. Specifically, a section of a selected undisturbed or remolded sample is trimmed into a disc 2.5 inches in diameter and 0.75 inch in height. The disc is confined in a stainless-steel ring and sandwiched between porous stone plates. Once the trimmed specimen is assembled in the odometer apparatus, a seating pressure of 50 lbf/ft^2 is applied.

After the initial deformation reading at the seating load, the sample is inundated, and the deformation is recorded at various elapsed times using a digital dial gauge. The readings are taken for at least 72 hours and until primary swell is completed. This is determined by plotting deformation versus log of time. After completion of swell, successive load increments are applied after 100% primary consolidation is reached for each load until the specimen is recompressed to its initial height. The swell test results are provided in the appendix to this report.

Specific Gravity

The specific gravity of soils is determined in accordance with the ASTM method D854. This method covers the determination of the specific gravity of soil solids by means of a water pycnometer. The specific gravity of soil solids is used in calculating the phase relationships of soils, such as void ratio and degree of saturation. In this method the volume of a soil sample with a known mass is determined by measuring the water displacement using a water pycnometer. The specific gravity is then calculated by the ratio of mass to volume.

Soil Compaction (Standard Proctor Test)

This test determines the maximum dry density that could be achieved by using a uniform compactive effort at varying moisture contents. Two primary methods of compaction are used. For standard Proctor, 5.5-lb (2.49-kg) rammer is dropped 12 inches (305-mm) and for modified Proctor, 10-lb (4.54-kg) rammer is dropped 18 inches (457-mm) for compaction on the bulk sample in the cylindrical mold. Compaction is done in 3 and 5 equal layers, respectively. The methods are explained in ASTM D 698 and ASTM D 1557, respectively.

Falling Head Permeability Testing

Hydraulic conductivity (permeability) tests were performed on remolded samples obtained of bulk sample from boring B-1. This method covers the determination of the hydraulic conductivity also known as coefficient of permeability of saturated fine-grained soils using a flexible wall permeameter in accordance with ASTM D 5084 “C”. This test method may be utilized on an undisturbed or remolded sample. A sample is either trimmed from a Shelby tube or remolded. The specimen is then encased in a rubber membrane and is placed in a triaxial cell. The specimen is then saturated by applying increasing back pressure. The saturation of the sample is verified by measuring the B coefficient. Once the saturation is obtained, the desired confining pressure (effective stress) is applied, and the sample is allowed to consolidate. Once the consolidation is completed, the permeation stage is initiated. The permeation of the specimen is initiated by increasing the influent (headwater) water level and decreasing the effluent (tailwater) level thus creating a hydraulic gradient (differential head of water). Head loss is recorded at measured time intervals. From these data, the permeability of the soil is calculated.

Unconfined Compressive Strength Test

This Standard Operating Procedure (SOP) establishes a standard method for testing and evaluating the unconfined compressive strength (UCS) of intact rock core specimens. This procedure provides all departments with a method for determining the UCS of a sample as defined by the American Society of Testing and Materials (ASTM) in accordance with ASTM D7012-Method C. This SOP addresses protocols for preparation of the sample, performance of the UCS test, and acquiring and reporting data. The test specimen should be cylinder with length to diameter ration of 2 to 2.5 and have a diameter of no less than 1.87 inch. The ends of the specimen should be cut parallel to each other and at right angles. The end surfaces should be flat Place the sample on the base plate of the loading frame and raise the sample by turning the loading frame switch to up until the sample is securely held between the top and bottom plate. Apply the load continuously and without shock. The strain rate should be approximately 0.05 in/min. Record the maximum load sustained by the specimen. The compressive strength in the test specimen calculated from the maximum compressive load on the specimen and the initial computed cross-sectional area.

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