Estling Lake Dam Spillway and Embankment Stability Evaluation

New Jersey Transit

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1 Background

The New Jersey Transit (NJ Transit) Railroad Right-of-Way on the M&E Line in the vicinity of Milepost 34.58 forms an embankment serving as the earthfill dam for the Estling Lake impoundment. It includes a spillway owned by the Estling Lake Corporation and the embankment owned by NJ Transit. See **Figures G-1** and **G-2**.

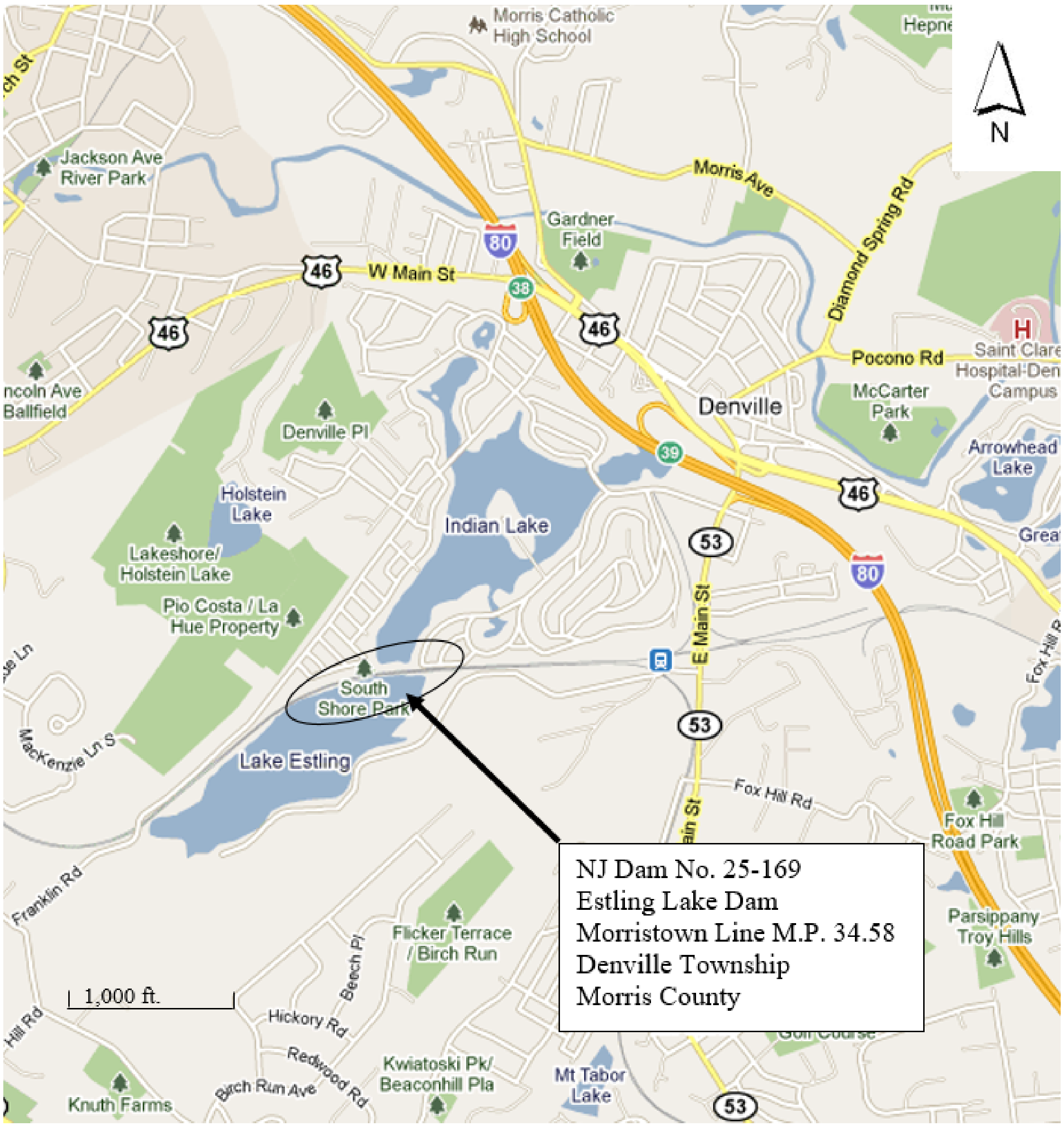


Figure G-1. Site Location

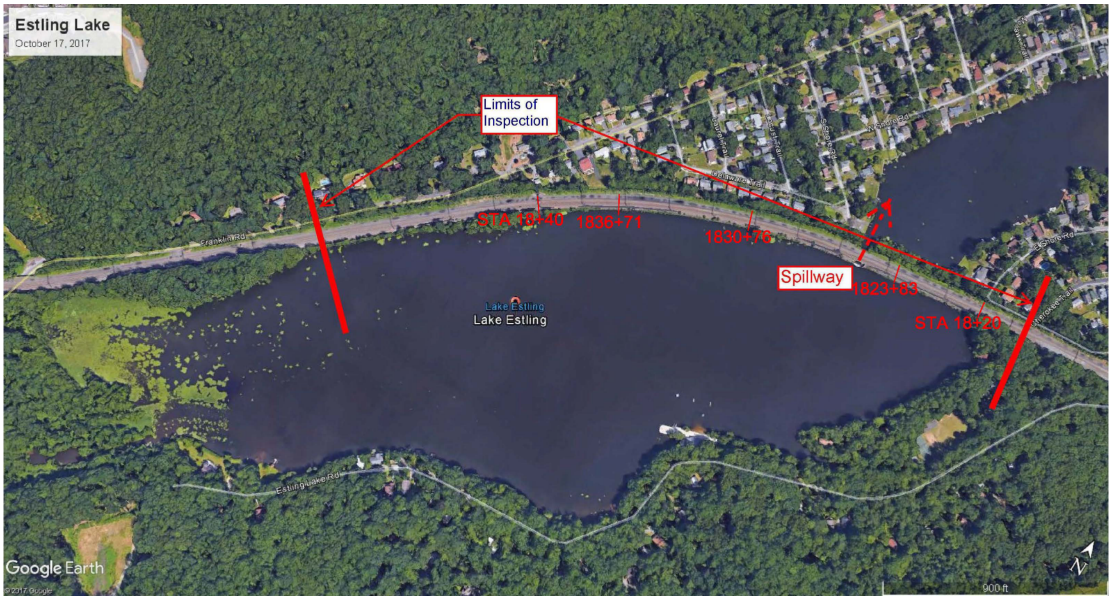


Figure G-2. Estling Lake Limits of Inspection from Formal Inspection Report

A Formal Inspection Report for Estling Lake Dam was submitted to NJDEP on September 5, 2018. The recommendations in this report require that NJ Transit submit the following additional documents to NJDEP:

1. Hydrologic & Hydraulic Evaluation
2. Potential Dam Failure Mode Analysis
3. Spillway and Embankment Stability Evaluation

The Hydrologic & Hydraulic Evaluation was completed in 2019. This report summarizes the objectives, methods, and results of the Spillway and Embankment Stability Evaluation and incorporates the results from the Hydrologic and Hydraulic Evaluation. The Potential Dam Failure Mode Analysis will be completed subsequent to this document.

1.1 Site History

Estling Lake Dam is located in Denville, New Jersey, about 28 miles west of New York, New York. The earthen embankment dam was constructed in 1894, and includes two railroad tracks are both part of the New Jersey Transit's Morris-Essex Line. The location of Estling Lake is shown on the Site Location map in Figure G-1.

The dam consists of an approximately 2000-foot long earthen embankment with a maximum height of 19.0 feet. The crest of the dam is approximately 50-feet wide, with slopes varying from 5:1 (horizontal to vertical) to less than 1:1 (horizontal to vertical) at the upstream slope and 1.5:1 (horizontal to vertical) to 2:1 (horizontal to vertical) at the downstream slope.

The spillway is centrally located on the embankment, and is comprised of an arched masonry design constructed of large granite blocks with a stepped box arrangement. The formal inspection report for the Estling Lake Dam indicated cracking was evident at the masonry spillway. Crack Mapping of the arched masonry spillway at Estling Lake Dam is included as Appendix B-1. A structural stability evaluation of the arched masonry spillway was performed as part of this report and expanded on in Section 5.0.

The lake drain structure is located east of the spillway entrance, and consists of a manually controlled 24-inch low-level outlet pipe valve.

Dam characteristics have been summarized in **Table G-1** below.

Table G- 1. Dam Characteristics

General Information	
NJ File Number	25-169
Hazard Classification	Class I
County	Morris
Owner(s)	New Jersey Transit Corporation (Embankment) and Estling Lake Corporation (Spillway)
Structural Information	
Construction	1894
Drainage Area	6.44 square miles
Type of Impoundment	Earthen Embankment, Railroad Embankment
Embankment Length	2000 feet
Embankment Height	19.0 feet
Top Width	50 feet
Upstream Slope	Varies 5H:1V to <1H:1V
Downstream Slope	Varies 1.5H1V to 2H:1V
Lake Drain	24” low level outlet at east wingwall; Manually operated
Control Structure	Uncontrolled Stone Masonry Arched Spillway
Spillway	37-foot wide arched masonry design with stepped box arrangement
Key Elevations	
	Elevation (NGVD29 Datum)
Embankment Crest	Varies along length: 525.0 to 527.3 ft
Principal Spillway	515.5 ft (design elevation)

The Estling Lake Dam is classified as a Class I dam in accordance with Section N.J.A.C. 7:20-1.9 of the New Jersey Dam Safety Standards due to its ability to cause probable loss of life or extensive property damage should failure occur.

2 Evaluation Objectives

The objectives of the Estling Lake embankment and spillway stability evaluation are the following:

1. Evaluate the amount of embankment erosion and geometry modification due to overtopping forces generated during the spillway design storm (SDS), which is the flow event generated by one half of the probable maximum precipitation (0.5 PMP).
2. Evaluate the stability of the existing embankment while loaded by through-seepage, underseepage and overtopping forces generated during the 0.5 PMP event.
3. Evaluate the stability of the existing spillway while loaded with forces generated during the 0.5 PMP event.
4. Evaluate the stability of the existing embankment for other loading conditions consistent with guidance provided by the U.S. Army Corps of Engineers, including normal pool, rapid drawdown, and seismic loading.

3 Embankment Erosion

3.1 Methods

The project team applied the Windows Dam Analysis Modules (WinDAM) software model to evaluate embankment erosion and potential geometry modification caused by erosion during overtopping of the 0.5 PMP flow event. The US Department of Agriculture-Agricultural Research Service, USDA-Natural Resources Conservation Service, and Kansas State University developed the model through a cooperative effort. The numerical modeling tool WinDAM-C (Version 1.1) developed by the U.S. Department of Agriculture, Natural Resources Conservation Service (NRCS) is a legacy model that has undergone a sequential development of modules to simulate flood routing through reservoirs, overtopping, and embankment erosion. The WinDAM module was applied for the Estling lake evaluation. This method utilizes relationships from the headcut erodibility index for the material based primarily on analysis of spillways from USDA research described in the NRCS NEH Part 628 Chapter 51. The WinDAM module was applied for the Estling Lake evaluation.

The three-phase erosion model simulates the following processes:

1. Initial erosion or failure process of the embankment vegetal cover and development of concentrated flow.
2. Downward erosion in the area of concentrated flow, resulting in headcut formation.
3. Downward and upstream movement of the headcut, potentially breaching the spillway or embankment.

Each phase is described by a set of threshold-rate relationships based on the process mechanics. A headcut erodibility index (Kh) describes the resistance of the exposed geologic materials to erosive attack during the third phase of the process. The program utilizes the Hanson and Robinson excess shear stress equation to compute total loss of embankment material during a given overtopping event. That information, combined with known bulk density of embankment material, is used to compute modification of the embankment geometry.

3.2 Data Sources

The WinDAM model requires three general types of input data: embankment geometry, embankment material geologic profiles and parameters, and overtopping flow data. **Table E-1** summarizes the data sources accessed to supply this information for the evaluation.

Table E-1. WinDAM Model Input Data Sources

Data Type	Source
Embankment Geometry	Matrix Topographic and Boundary Survey (2018), AmerCom Topographic Survey (2019)
Embankment Material Parameters	Subsurface Investigation Report (Oweis, 2019)
Flow Distribution	Updated Hydrologic and Hydraulic Analysis, New Jersey Transit – Estling Lake Dam (SWM, 2019)

Table E-2 summarizes the embankment soil material parameters used for both the Hanson/Robinson Stress Headcut Model Embankment parameters and the three soil strata breaks shown on the subsurface profile and sections, as discussed in more detail in Section 4.2.3.

Table E-2. Soil Material Parameters

Parameter	Value	Comment
Total Unit Weight, pcf	113	Hanson/Robinson Stress Headcut Model Embankment Parameter
Erodibility Coefficient (Kd), (ft/h)/psf	10	Hanson/Robinson Stress Headcut Model Embankment Parameter
Undrained Shear Strength, psf	2100	Hanson/Robinson Stress Headcut Model Embankment Parameter
Critical Shear Stress, psf	0	Hanson/Robinson Stress Headcut Model Embankment Parameter
Plasticity Index	0	Average of the three stratums.
Dry Density, pcf	127	Average of the three stratums.
Head Cut Index (Kh)	0.0500	Average of the three stratums.
Percent Clay, %	13	Average of the three stratums.

Parameter	Value	Comment
Representative Material Diameter, inch	0.2861	Average of the three stratums.

Soil parameters from Borings B2, B3, and B4 were utilized based on the location of this overtopping section. Review of the soil parameters for the three soil stratums found little to no significant differences, so an average of the three soil stratums was assigned as one soil layer for the downstream embankment soil material in the WinDAM model. The three soil strata are defined as silty-clayey sand, lean clay with sand, and silty sand with gravel, with low clay content and Plasticity Indexes. Soils with these characteristics are low cohesion and are considered non-erosion resistant soils.

Table E-3 summarizes the discharge distribution from Estling Lake Dam during the SDS as reported in the Updated Hydrologic and Hydraulic Analysis (SWM, 2019). The primary discharge from the dam is to the stone arch culvert under the railroad embankment. Due to the existing topography of the railroad embankment crest, discharge that overtops the railroad embankment is split into two sections; the majority of overtopping flow occurs west (left) of the culvert and discharges over the downstream embankment (Secondary: West Overtopping), and the small remaining portion of the overtopping flow occurs east (right) of the culvert (Tertiary: East Overtopping). The location of these flows are detailed in Appendix E-1.

Table E-3. Flow Distribution

Location	Peak Flow (cfs)
Primary: Spillway Culvert under Railroad Embankment	6,540
Secondary: West Overtopping[1]	4,230
Tertiary: East Overtopping	450

[1]Flows overtop this embankment over a 2.8 hr period reaching a maximum depth of 1.7 ft.

For the purpose of assessing potential embankment erosion due to overtopping flow, only the flow distribution for the West Overtopping section was simulated. The East Overtopping section, with less than 500 cfs of peak flow overtopping the railroad embankment during the SDS, was deemed negligible for potential overtopping embankment erosion. The East Overtopping discharge will flow directly downstream to Indian Lake. The Indian Lake water surface will rise with the SDS flow from the culvert and East Overtopping, creating a tailwater condition on the railroad embankment for the section of East Overtopping embankment, which will reduce the potential for overtopping erosion during the SDS at this location.

3.3 Model Configuration

The Estling Lake Dam is represented in the WinDAM model from railroad embankment Station 1819+80 to 1841+20 to define the discharge over the West Overtopping section. The railroad embankment profile used to define the dam crest is provided in Appendix E-1, ranging from elevation 524.2 to 526.5 feet NAVD88 (525.0 to 527.3 ft NGVD29). The dam upstream embankment was defined with a slope of 1.3:1 (horizontal:vertical) with a riprap armor. The dam crest was defined with a width from upstream to downstream of 57.2 feet based on the 2019 AmerCOM topographic survey, with a riprap/ballast surface cover. The dam downstream embankment was defined with a slope of 3.7:1 (horizontal:vertical), with uniform riprap/ballast protection.

The starting water surface elevation was set to the minimum dam crest elevation, and the hydrograph derived from the Updated Hydrologic and Hydraulic Analysis (SWM, 2019) was input as the Secondary West Overtopping discharge hydrograph to simulate the overtopping discharge of the SDS.

The WinDAM integrity analysis modeled a section of the downstream embankment as an auxiliary spillway, to assess the potential for the gross shear stress or the time-integral of the erodibility effective shear stress to generate erosion. Railroad embankment Station 1836+71.40 cross section stations and elevations from the 2019 AmerCOM survey defined the spillway slope and channel profile, with a spillway crest elevation of 525.43 feet NAVD88. The boring logs in the 2019 Oweis Subsurface Investigation Results were assessed to define the three soil strata defining the spillway crest (Borings B2 and B3) and downstream slope (Boring B4) that underlay the riprap/ballast protection. An image of the WinDAM model spillway material is provided in Appendix E-1 as well as Figure E-1. The spillway profile downstream station defines the limit of potential erosion, as the “valley floor” within the model, defined as the downstream bottom data point from the Station 1836+71.40 cross section. The spillway bottom width was assumed as the distance between the two highest crest profile survey elevations between Station 1837+25 and 1835+25, as a width of 200 feet.

The USDA auxiliary spillway erosion headcut model was used to simulate the overtopping erosion process. This method utilizes relationships from the headcut erodibility index for the material based primarily on analysis of spillways from USDA research described in the NRCS NEH Part 628 Chapter 51.

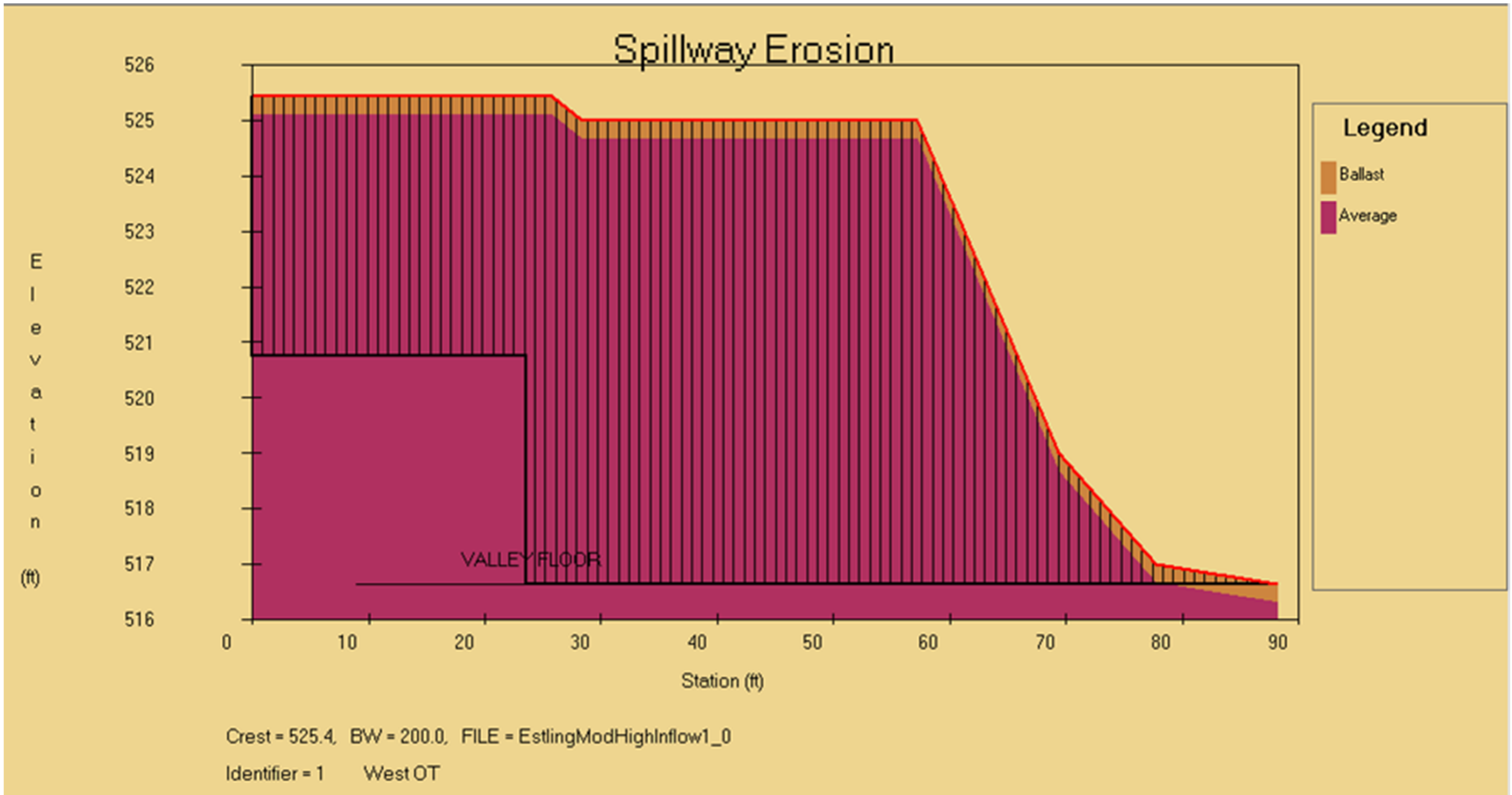
3.4 Model Results

The WinDAM analysis indicated that the railroad embankment downstream slope would experience significant erosion and head cutting, estimated to advance through the railroad embankment crest to the upstream embankment top of bank. This can be seen in **Figure E-1**, where the hatched areas represent the material eroded through the SDS overtopping event, and zero is the upstream embankment top of bank station. Due to the variability of the downstream slope, ranging from 3.5 to 49.7 percent, erosion progression would initiate with loss of riprap/ballast protection and partial head cutting (“nicking”) at several locations, then progress to development of the headcut through the entire embankment. The model results show the overflow began at 1.5 hours, peaked at 41.3 hours, and flow duration was

53 hours. Head cutting began at multiple locations including the downstream embankment toe at 40.0 hours and advanced upstream to breach the crest at 43.3 hours.

The model computations terminate at the end of the discharge hydrograph duration, estimating a breach of the 57-foot-wide railroad crest through to the upstream top of embankment. The model estimates that the deepest head cutting will advance through more than half of the crest toward the upstream embankment, resulting in loss of half of the railroad crest and all of the downstream embankment removed down to the valley floor elevation of 516.63 feet NAVD88, with a head cut height of 8.8 feet. Headcut depth reached the valley floor elevation for the entire overflow length of the downstream embankment and a portion of the crest.

A limitation of the WinDAM model is that it simulates an inflow hydrograph through the reservoir storage. The 2019 HydroCAD outflow hydrograph (SWM, 2019) for the overtopping of the railroad embankment was provided and utilized as the inflow hydrograph to WinDAM. The starting water surface elevation was set to the minimum railroad embankment crest elevation, used as the spillway crest elevation, so there would be minimal routing storage resulting in this embankment section within the WinDAM model for areas of the dam higher than the spillway crest elevation. However, even with this conservative minimal storage causing WinDAM peak outflow to be slightly smaller than the peak outflow from HydroCAD, the WinDAM model showed that the railroad embankment will have significant erosion during the overtopping of the SDS. An additional scenario was simulated where the WinDAM inflow hydrograph was modified to result in a close match of peak outflow from the WinDAM model to the HydroCAD model results, to estimate the full erosion potential of the peak outflow during the SDS, which are the results reported and shown in **Figure E-1**.



Note: WinDAM model Station 0 represents the upstream embankment top of bank station.

Figure E- 1. STA 1836+71.40 Embankment Material Zones and Potential Erosion Results from WinDAM

3.5 Limitations

The hydrographs provided by SWM for this dam were reviewed for reasonableness but not recalculated. The railroad embankment profile is a vertical curve but the overtopping crest is modeled as a level section to accommodate WinDAM input requirements.

4 Embankment Stability

HDR performed a geotechnical assessment to evaluate the existing earthen embankment. The geotechnical assessment is based on a review of the 1983 report provided by Langan Engineering Associates (Phase IIA – Investigation, Analysis and Design, Estling Lake Dam). HDR utilized results of Langan's report to analyze seepage and slope stability factors of safety for the embankment.

The following sections report the methods, data sources, results, and limitations of the stability evaluation completed for the Estling Lake earthen embankment.

4.1 Methods

HDR utilized Geostudios SEEP/W program to analyze groundwater seepage, estimate exit gradients, estimate seepage flow through the embankment, and generate a phreatic surface for subsequent slope stability analysis. HDR evaluated the earthen embankment utilizing the Geostudios SLOPE/W program for the steady state seepage condition with normal pool and transient phreatic surface with maximum storage pool for the earthen embankment. The stability analyses utilized the phreatic surface from the results of the seepage analysis. **Tables S-1** and **S-2** summarize the seepage and stability scenarios evaluated.

Table S-1. Seepage Scenarios

Cross Section	Pool Level	Analysis Type	Water Surface Elevation (ft. NAVD88)
STA 1823+83.40	Normal Pool	Steady State	513.9
	Maximum Pool	Transient	525.0[1]
STA 1830+76.69	Normal Pool	Steady State	513.9
	Maximum Pool	Transient	525.0[1]
STA 1836+71.40	Normal Pool	Steady State	513.9
	Maximum Pool	Transient	525.0[1]

[1] Peak elevation was set equal to the approximate top of embankment elevation for critical loading conditions – because the embankment elevation varies.

Table S-2. Stability Scenarios

Cross Section	Loading Condition	Slope	Material Properties
STA 1823+83.40	Normal Pool	Downstream/Upstream	Effective Stress
	Maximum Pool	Downstream/Upstream	Effective Stress
	Rapid Drawdown	Upstream	Effective/Total Stress
	Pseudostatic (kh=0.16g)[1]	Downstream/Upstream	Total Stress
STA 1830+76.69	Normal Pool	Downstream/Upstream	Effective Stress
	Maximum Pool	Downstream/Upstream	Effective Stress
	Rapid Drawdown	Upstream	Effective/Total Stress
	Pseudostatic (kh=0.16g)[1]	Downstream/Upstream	Total Stress
STA 1836+71.40	Normal Pool	Downstream/Upstream	Effective Stress
	Maximum Pool	Downstream/Upstream	Effective Stress
	Rapid Drawdown	Upstream	Effective/Total Stress
	Pseudostatic (kh=0.16g)[1]	Downstream/Upstream	Total Stress

[1] Pseudostatic loading developed from the USGS horizontal ground accelerations with a return period of 2475 years as the Maximum Credible Earthquake (MCE).

4.2 Site Data

4.2.1 Geology

The project site is located in the central portion of Morris County, New Jersey within the Highlands Province physiographic region of the New England Province of the Appalachian Highlands of the United States. The Highlands Province is characterized by rugged topography consisting of a series of intermittent rounded ridges separated by deep, narrow valleys. Elevations range from less than 400 feet above mean sea level (MSL) along the Delaware River to 1,496 feet above MSL at Wawayanda Mountain. Site specifically, the elevation ranges from 525 to 535 feet above MSL, based on United States Geological Survey 7.5-minute series map of the Dover and Boonton, New Jersey Quadrangles.

A custom soil resource report obtained from the United States Department of Agriculture (USDA), Natural Resources Conservation Service (NRCS) indicates that the surficial soil deposits within approximately 60 inches of the ground surface belong to the Parker-Gladstone, Parker-Rock, and Rockaway soils consisting of very gravelly sandy loam, very gravelly loam, gravelly sandy loam, gravelly sandy clay loam, sandy loam, and gravelly sandy loam. The USDA report also indicates that the USCS classification of the surficial soils consist of SM and SC.

Published geological mapping by the New Jersey Geological and Water Survey indicates the proposed site lies primarily on bedrock consisting of Mesoproterozoic-aged metamorphic and igneous rocks (1.6 to 1.0 billion years ago) known as the Byram Intrusive Suite and an undifferentiated metasedimentary and metavolcanic rock unit. The Byram Intrusive Suite consists of mainly of microperthite alaskite, a pale pink-white, medium- to coarse-grained, massive, moderately foliated granite consisting of microcline, microperthite, quartz, oligoclase, with minor amounts of horneblende and/or biotite, zircon, apatite, and magnetite. The undifferentiated metasedimentary and metavolcanic rock unit is primarily composed of biotite-quartz-feldspar gneiss. This gneiss is described as pale pink-white or pink-gray, locally weathers to a rust color, medium- to coarse-grained, moderately layered and foliated, containing microcline, microperthite, oligoclase, quartz, biotite, garnet, and sillimanite. Some local variations are associated with thin, moderately-layered quartzite containing biotite, feldspar, and graphite.

Based on mapping by the New Jersey Geological and Water Survey, the Rockaway Valley Fault lies along the eastern edge of the project site and traverses. The Rockaway Valley Fault is described as a major regional structural feature, strikes northeast and dips southeast, and is one of many faults located within the Dover and Boonton quadrangles.

4.2.2 Subsurface Explorations

From December 7, 2018 to January 9, 2019, Craig Test Boring Co., Inc. (CTB) executed a subsurface exploration program consisting of ten (10) borings and the installation of seven (7) temporary groundwater observation wells. CTB advanced the borings using 3 7/8-inch outside diameter hollow-stem augers and a track mounted CME-55 drilling rig. CTB obtained Standard Penetration Test (SPT) samples continuously to a depth of 12.0 feet then 5.0-foot intervals thereafter until termination. CTB obtained Undisturbed Shelby tube samples in the silty clay layer encountered for further testing. CTB installed temporary groundwater observation wells in Boring B-2, B-3, B-4, B-6, B-7, B-8, and B-9. See **Table S-3. Appendix S-1** depicts the locations of the borings. The test wells extended to varying depths in the borings to correspond with differing materials. CTB performed field slug testing in the materials encountered to develop hydraulic permeability's for the encountered soils.

Table S-3. Summary of CTB Subsurface Exploration

Boring Number	General Location	Ground Surface Elevation (NAVD 88)	Termination Elevation (ft)
B-1	STA 1839+40 – Downstream Toe	524.0	462.0
B-2	STA 1836+60 – Crest	524.0	475.0
B-3	STA 1836+50 – Crest	524.0	457.0
B-4	STA 1836+70 – Downstream Toe	520.0	418.0
B-5	STA 1833+70 – Crest	524.0	457.0
B-6	STA 1827+00 – Crest	525.0	468.0

Boring Number	General Location	Ground Surface Elevation (NAVD 88)	Termination Elevation (ft)
B-7	STA 1827+20 – Downstream Toe	519.0	467.0
B-8	STA 1830+90 – Downstream Toe	522.0	465.0
B-9	STA 1830+90 – Crest	524.0	467.0
B-10	STA 1823+30 – Crest	525.0	460.0

CTB transported the collected samples to TerraSense, LLC's (TerraSense) laboratory for further examination and laboratory testing at the completion of the drilling program. Oweis Engineering Inc. assigned laboratory tests to classify the soil, determine grain size distribution, shear strength, and soil permeability for representative samples. The laboratory performed the following tests on selected soil samples:

- Natural water content (ASTM D 2216)
- Grain-size distribution (ASTM D 422)
- Atterberg limits (ASTM D 4318)
- Drained Direct Shear Test (ASTM D 3080)
- Unconsolidated-Undrained Compressive Strength Test (ASTM D 2850)
- Consolidated Undrained Triaxial Test (ASTM D 4767)

Detailed laboratory tests completed by TerraSense are presented in **Appendix S-2**.

4.2.3 General Subsurface Conditions

The following sections present the generalized subsurface conditions encountered during the subsurface exploration. HDR reviewed the subsurface data collected to correlate and clarify the geologic significance of the subsurface conditions encountered. A summary of the geologic units is presented in **Table S-4**. For more detailed information, please refer to the boring logs presented in **Appendix S-3**. Please note that the strata breaks shown on the subsurface profile and sections are approximate based on HDR's geologic interpretation at the borings drilled, and that the actual unit separation will be gradational and vary between the borings.

Table S-4. Summary of Geologic Units

Geologic Unit Number	Geologic Description	Predominant Soil Type(s)
1	Poorly Graded Gravel, Clayey Sand, Silty Sand, Silty Gravel [FILL] (GP, SC, SM, GM, SC-SM, SP-SM, GP-GM, SW)	Granular
2	Lean Clay with Sand, Silt with Sand, Silty Clay (CL, ML, CL-ML)	Cohesive
3	Silty Sand with Gravel (SM) [B-1, B-2, B-3, B-4 only]	Granular
4	Silty Sand with Gravel, Clayey Sand (SM, SC)	Granular
5	Silty Sand with Gravel (SM)	Granular
6	Poorly Graded Gravel, Poorly Graded Gravel with Silt, Poorly Graded Sand with Gravel (SP-SM, SM, SC-SM)	Granular
7	Silty Sand with Gravel, Silty Sand, Silty Clayey Sand (SP-SM, SM, SC-SM)	Granular
8	Poorly Graded Gravel with Silt (GP-GM) [B-1, B-5 only]	Granular
9	Clayey Sand, Lean Clay with Sand (SC, CL) [B-4 only]	Cohesive

All borings were drilled through the existing Earthen Dam embankment at either the crest or the downstream toe. The borings advanced generally encountered a granular fill layer overlying cohesive clay/silt layer, overlying granular silty sand and gravel until boring termination. Elevations and depths of soil stratum breaks varied throughout each boring. Standard Penetration Test (SPT) field blow counts ranged from 2 to 89 blows per foot in the upper granular fill layer. The upper cohesive layer encountered blow counts ranging from 3 to 28. Silty Sand underlying the cohesive clay layer encountered blow counts ranging from 7 to >100 blows per foot.

4.2.4 Groundwater Conditions

CTB recorded the water levels within the borings observed at various times throughout the subsurface exploration program. CTB installed piezometers to depths shown on the Configuration of Monitoring Wells sheets in **Appendix S-3**. **Table S-5** provides a summary of the groundwater elevations measured during the subsurface investigation.

Table S-5. Summary of Groundwater Elevations

Boring	General Location	Ground Surface Elevation (ft)	Average Groundwater Elevation (ft)
B-2	STA 1836+60 – Crest	524.0	517.10
B-3	STA 1836+50 – Crest	524.0	516.28
B-4	STA 1836+70 – Downstream Toe	520.0	516.87
B-6	STA 1827+00 – Crest	525.0	511.70
B-7	STA 1827+20 – Downstream Toe	519.0	511.52
B-8	STA 1830+90 – Downstream Toe	522.0	514.89
B-9	STA 1830+90 – Crest	524.0	514.94

4.2.5 Soil Design Parameters

Material parameters selected for the analysis of the existing earthen embankment of Estling Lake Dam are based on the results of the subsurface exploration, laboratory testing program, published soil information/correlations, general experience with similar material in the geologic area, and engineering judgment. **Table S-6** and **Table S-7** below summarize the material parameters utilized during the seepage and stability analysis, respectively. Detailed laboratory test results and a general discussion outlining factors considered to estimate material parameters is presented in **Appendix S-2**.

Table S-6. Summary of Seepage Analysis Parameters

Geologic Unit No.	Seepage Analysis Location	Description	Material Model	Permeability (hydraulic conductivity)		
				K_v/k_h (---)	k_h	
					(ft/day)	(cm/sec)
1	Dam Embankment	Layer 1: FIll	Saturated Only	1.00	66.69	2.4E-2
2	Dam Embankment	Layer 2: CL, ML, CL-ML	Saturated Only	0.25	1.51	5.3E-4
3	Dam Embankment	Layer 3: SM	Saturated Only	1.00	4.11	1.5E-3
4	Dam Foundation	Layer 4: SM, SC	Saturated Only	1.00	0.69	2.4E-4
5	Dam Embankment	Layer 5: SM	Saturated Only	1.00	4.11	1.5E-3

Geologic Unit No.	Seepage Analysis Location	Description	Material Model	Permeability (hydraulic conductivity)		
				K_v/k_h (---)	k_h	
					(ft/day)	(cm/sec)
6	Dam Embankment	Layer 6: GP, GP-GM, SP	Saturated Only	1.00	4.11	1.5E-3
7	Dam Embankment	Layer 7: SP-SM, SM, SC-SM	Saturated Only	1.00	4.11	1.5E-3
8	Dam Embankment	Layer 8: GP-GM	Saturated Only	1.00	198.43	7.0E-2
9	Dam Embankment	Layer 9: SC, CL	Saturated Only	0.25	0.02	7.1E-6

Table S-7. Summary of Stability Analysis Parameters: Developed by Langan Engineering Associates

Geologic Unit No.	Stability Analysis Location	Description	Total Unit Weight	Shear Strength			
				Effective Stress		Total Stress	
			γ	ϕ'	c'	ϕ	c
			(pcf)	(°)	(psf)	(°)	(psf)
1	Dam Embankment	Layer 1: FIll	134.0	33.6	0.0	33.6	0.0
2	Dam Embankment	Layer 2: CL, ML, CL-ML	128.0	33.0	350	0.0	2100
3	Dam Embankment	Layer 3: SM	119.0	35.0	0.0	35.0	0.0
4	Dam Foundation	Layer 4: SM, SC	116.0	34.0	0.0	34.0	0.0
5	Dam Embankment	Layer 5: SM	122.0	35.0	0.0	35.0	0.0
6	Dam Embankment	Layer 6: GP, GP-GM, SP	135.0	35.0	0.0	35.0	0.0
7	Dam Embankment	Layer 7: SP-SM, SM, SC-SM	117.0	35.0	0.0	35.0	0.0
8	Dam Embankment	Layer 8: GP-GM	135.0	35.0	0.0	35.0	0.0
9	Dam Embankment	Layer 9: SC, CL	120.0	28.0	50	0.0	4000

4.3 Seepage and Stability Analyses

HDR performed seepage and stability analyses for the Estling Lake earthen embankment to evaluate the stability of the existing embankment and evaluate the seepage rates through the existing embankment and the exit gradients near the toe of the embankment. The following subsections present the model configurations and discuss the results of the analyses.

4.3.1 Existing Earthen Embankment Analysis

The USACE guidance (EM 1110-2-1901 and EM 1110-2-1902) provided the minimum required factors of safety for the required loading cases of the existing earthen embankment along with the maximum exit gradients allowed. The embankment seepage analysis analyzed the foundation seepage and uplift pressures within the earthen embankment. HDR utilized topographical survey data of the embankment to create the cross section geometry utilized for embankment analyses. HDR developed the internal geometry of the embankment based on the results from the subsurface exploration program.

The following subsections discuss the results of the seepage and slope stability analyses performed for the existing embankment configuration. See **Appendix S-5** for additional details regarding the seepage and stability analysis of the embankment. Cross-section geometry for the analyzed cross sections are shown below as **Figure S-1, Figure S-2** and **Figure S-3.**

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)
	Layer 1: FILL	Mohr-Coulomb	134	0	33.6	0
	Layer 2: CL, ML, CL-ML (long term)	Mohr-Coulomb	128	350	33	0
	Layer 6: GP, GP-GM, SP	Mohr-Coulomb	135	0	35	0
	Layer 7: SP-SM, SM, SC-SM	Mohr-Coulomb	117	0	35	0

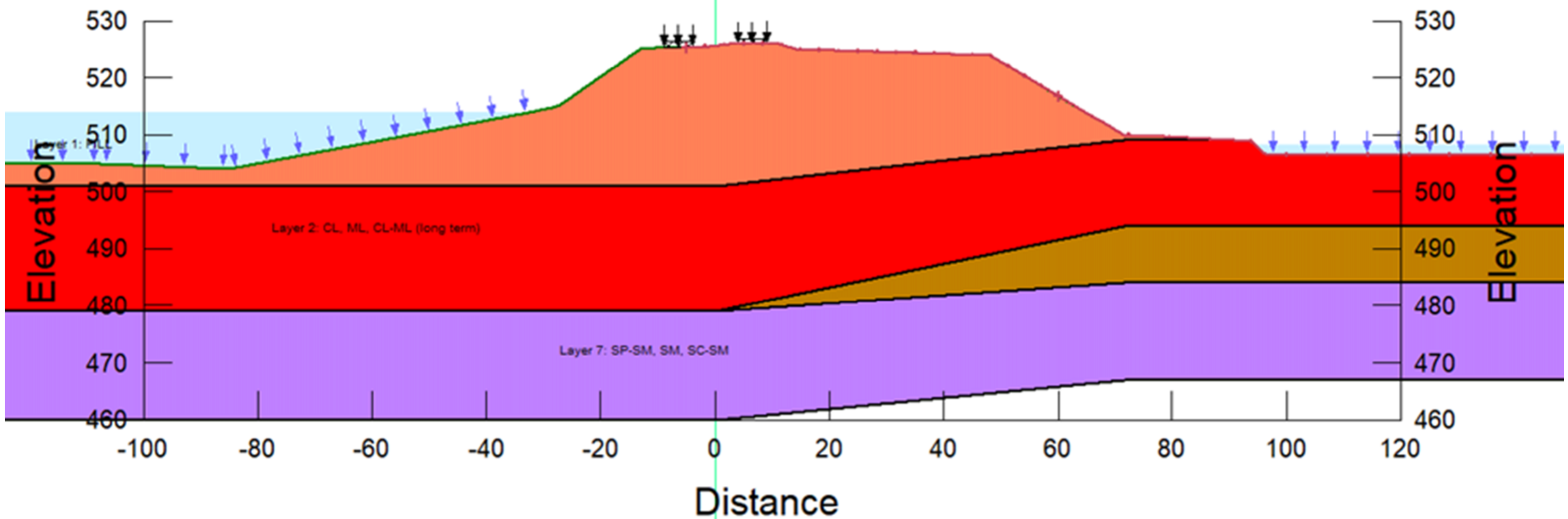


Figure S-1. Embankment Geometry and Material Zones - STA 1823+83.40

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)
	Layer 1: FILL	Mohr-Coulomb	134	0	33.6	0
	Layer 2: CL, ML, CL-ML (long term)	Mohr-Coulomb	126	350	33	0
	Layer 4: SM, SC	Mohr-Coulomb	116	0	34	0
	Layer 5: SM	Mohr-Coulomb	122	0	35	0
	Layer 6: GP, GP-GM, SP	Mohr-Coulomb	135	0	35	0
	Layer 7: SP-SM, SM, SC-SM	Mohr-Coulomb	117	0	35	0

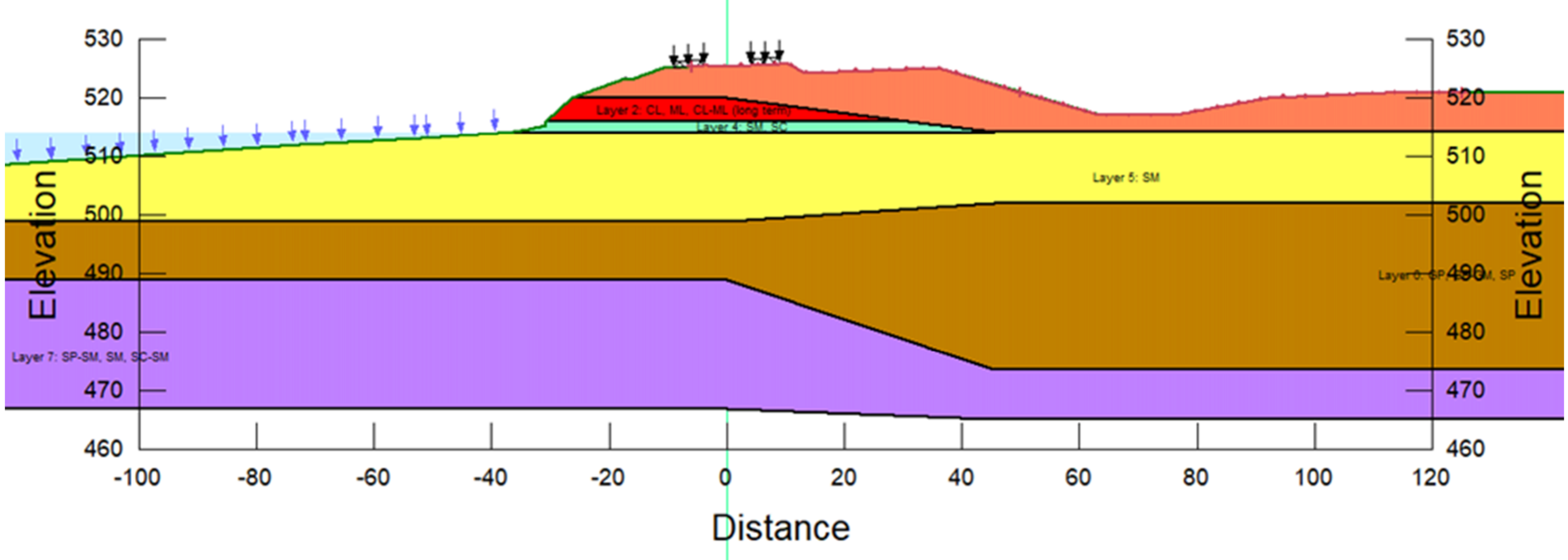


Figure S-2. Embankment Geometry and Material Zones - STA 1830+76.69

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)
	Layer 1: FILL	Mohr-Coulomb	134	0	33.6	0
	Layer 2: CL, ML, CL-ML (long term)	Mohr-Coulomb	128	350	33	0
	Layer 3: SM	Mohr-Coulomb	119	0	35	0
	Layer 4: SM, SC	Mohr-Coulomb	116	0	34	0
	Layer 5: SM	Mohr-Coulomb	122	0	35	0
	Layer 6: GP, GP-GM, SP	Mohr-Coulomb	135	0	35	0
	Layer 7: SP-SM, SM, SC-SM	Mohr-Coulomb	117	0	35	0
	Layer 9: SC, CL (long term)	Mohr-Coulomb	120	50	28	0

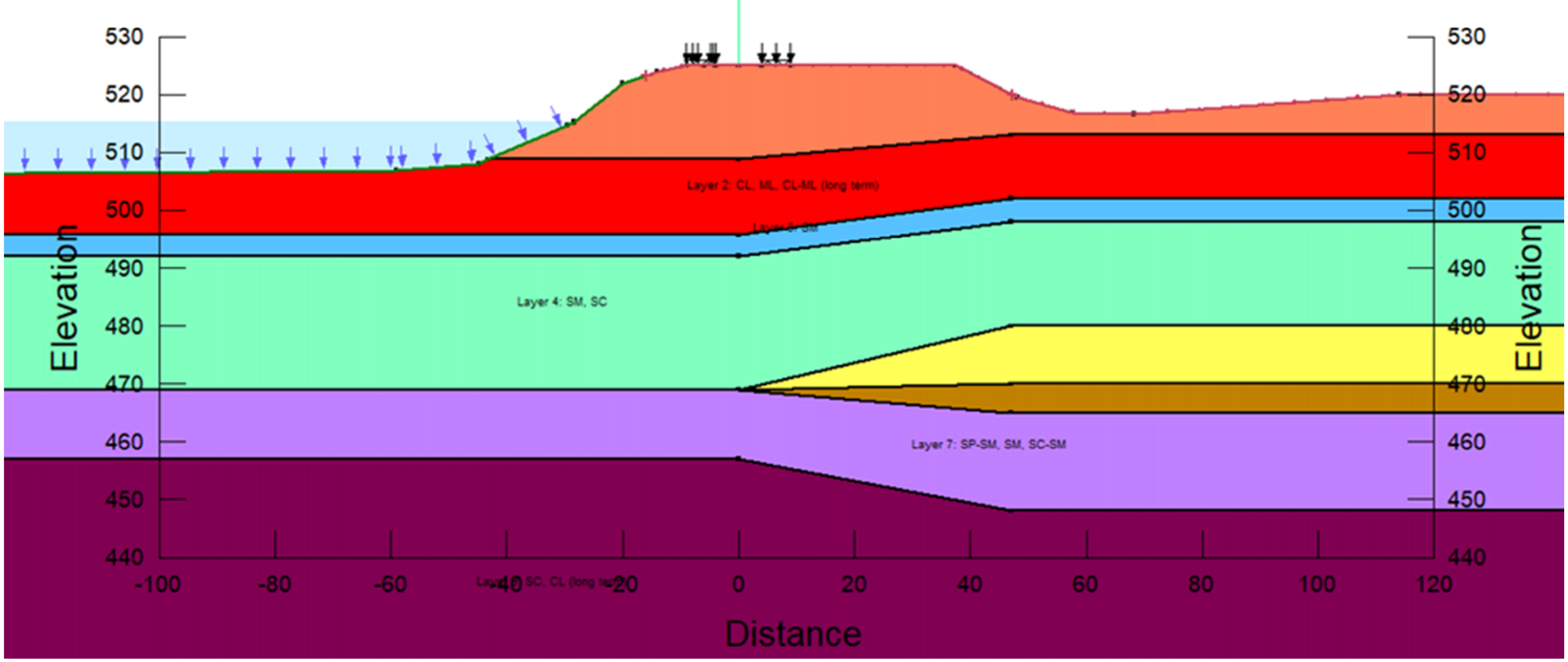


Figure S-3. Embankment Geomtery and Material Zones – STA 1836+71.40

4.4 Seepage and Stability Modeling Results

Tables S-8 and **S-9** provide results of the seepage and stability modeling, respectively. The seepage criteria described in the USACE EM 1110-2-1901 Seepage Analysis and Control for Dam are utilized in this analysis. The stability criteria described in the USACE EM 1110-2-1902 Slope Stability are utilized in this analysis. In general, embankments with seepage results that do not meet criteria are at greater risk of developing conditions that lead to failure than those that do meet the criteria. The same is true for stability results with factors of safety below criteria but are greater than 1.0. Stability factors below 1.0 predict failure for the noted loading condition.

The seepage results satisfy the criteria for all loading conditions at each station evaluated. The stability results varied by section.

Results for STA 1823+83.4 do not meet criteria for the loading conditions specified, including the Normal Pool, Pseudostatic (i.e. seismic loading), which has a factor of safety less than 1.0 – a predicted failure condition where loading forces exceed the resistance the downstream embankment materials generate. HDR utilized a design MCE of 0.16g as defined by USGS for a 2475 year return event in the analyses. A Probabilistic Seismic Hazard Analysis (PHSA) of the site may be utilized to further define the MCE. Results for STA 1830+76.69 all meet criteria. Results for STA 1836+76.40 meet all criteria for loading conditions focused on the downstream slope; however, some of the results for the upstream slope are below the criteria.

Slope stability factors of safety were compared to USACE guidance provided for new embankment construction. Existing embankments must take into account information regarding the past slope performance over the life of the embankment. USACE EM 1110-2-1902 addresses existing slopes as follows:

> *A history free of signs of slope movements provide firm evidence that a slope has been stable under the conditions it has experienced. Conversely, signs of significant movement indicate marginally stable or unstable conditions. In either case, the degree of uncertainty regarding shear strength and piezometric levels can be reduced through back analyses. Therefore, values of factors of safety that are lower than those required for the new slopes can often be justified for existing slopes.*

Embankment conditions vary as observed during the 2018 formal inspection. The side slopes of the Estling Lake earthen embankment are generally steeper than that defined in the New Jersey Administrative Code (NJAC) 7:20, which define a 3(H):1(V) limit for the upstream embankment and a 2(H):1(V) limit for the downstream embankment. The up- and downstream slopes range from 1.5(H):1.0(V) to 2.0(H):1.0(V), with isolated downstream areas that are steeper. The downstream slopes are hummocky in regions, and generally undulating. Numerous areas of seepage and ponded water were noted on the downstream slopes and areas near the downstream toe of the dam. These observations are consistent with a slope that is too steep for generally clay materials, but temporarily stabilized by vegetative tree growth which has not been maintained. Field observations are consistent with the Normal Pool, Steady-State stability factor of safety greater than 1.0 reported in Table S-9.

NJ Transit noted that these slopes have not reported recent major damages. As an example, NJ Transit did not report any major damages after Hurricane Irene in August 2011 which was a major flooding event.

Table S-8. Summary of Seepage Analysis Results for Existing Embankment

Condition		Appendix S-5 Figure	Vertical Exit Gradient		Flux Seepage Rate (ft^3/day/lf)
			Maximum Allowable	Calculated	Downstream Face[1]
STA 1823+83.40	Normal Pool Long Term Steady State	B	0.50	0.25	29.750
	Maximum Storage Pool Transient[2]	D	0.50	0.30	172.683
STA 1830+76.69	Normal Pool Long Term Steady State	F	0.50	0.00[3]	6.13e-^{13} (3)
	Maximum Storage Pool Transient[2]	H	0.50	0.09	56.561
STA 1836+71.40	Normal Pool Long Term Steady State	J	0.50	0.00[3]	3.72e^{-13} (3)
	Maximum Storage Pool Transient[2]	L	0.50	0.09	84.138

[1] Seepage out of the downstream embankment face.

[2] Transient surfaces were developed based on data provided in the updated H&H report.

[3] Downstream ground surfaces are above water surface elevation during normal pool conditions.

Table S-9. Summary of Stability Analysis Results for Existing Embankment

Case		Appendix S-5 Figure	Slope	Factor of Safety (FS) For New Embankments[1]	Calculated Factor of Safety (FS)
STA 1823+83.40	Normal Pool, Steady Seepage	N	Downstream	1.5	1.3
		O	Upstream	1.5	1.1
	Normal Pool, Pseudostatic (kh=0.16g)	P	Downstream	1.0	0.9
		Q	Upstream	1.0	0.9
	Normal Pool, Rapid Drawdown	R	Upstream	1.3	1.1
	Maximum Storage Pool, Transient	S	Downstream	1.4	1.0
		T	Upstream	1.4	1.1
STA 1830+76.69	Normal Pool, Steady Seepage	V	Downstream	1.5	2.7
		W	Upstream	1.5	1.6
	Normal Pool, Pseudostatic (kh=0.16g)	X	Downstream	1.0	1.7
		Y	Upstream	1.0	1.4
	Normal Pool, Rapid Drawdown	Z	Upstream	1.3	1.7
	Maximum Storage Pool, Transient	AA	Downstream	1.4	2.0
		AB	Upstream	1.4	1.7
STA 1836+71.40	Normal Pool, Steady Seepage	AD	Downstream	1.5	2.1
		AE	Upstream	1.5	1.2
	Normal Pool, Pseudostatic (kh=0.16g)	AF	Downstream	1.0	1.4
		AG	Upstream	1.0	0.8
	Normal Pool, Rapid Drawdown	AH	Upstream	1.3	1.2
	Maximum Storage Pool, Transient	AI	Downstream	1.4	1.5
		AJ	Upstream	1.4	1.2

4.5 Limitations

HDR's scope of work included the evaluation of seepage and slope stability factors of safety for Estling Lake Dam. HDR did not evaluate or identify any alternatives to address low factors of safety or high vertical exit gradients calculated in the modeling.

A Potential Failure Mode Analysis (PFMA) workshop will be held in the near future to identify potential modes and progression of failure for the embankment during such loading events. The PFMA workshop will develop risk reduction measures to reduce the potential for failure.

This section presents the findings and conclusions for the geotechnical aspects of the Estling Lake Dam Embankment for NJ Transit near Milepost 34.58. It has been prepared in accordance with generally accepted engineering practice and in a manner consistent with the level of care and skill for this type of project within this geographic area. No warranty, expressed or implied, is made.

5 Spillway Stability

A stability analysis was performed on the masonry spillway dam at Estling Lake. The dam is a masonry structure which was designed and constructed in the 1890s. It is unclear whether the dam was designed to be a gravity dam or an arch dam. The design criteria stated in the applicable U.S. Army Corps of Engineers (USACE) Engineering Manuals that pertain to Gravity Dams and Arch Dams were consulted for this exercise. This section will discuss the analyses utilized to determine the global stability of the structure.

5.1 Methods

5.1.1 Design Criteria and Basis of Analysis

Safety Evaluation of Existing Dams – United States Department of the Interior – Bureau of Reclamation, 2000

USACE - EM 1110-2-2100 Stability Analysis of Concrete Structures, Dec 2005

USACE - EM 1110-2-2201 Arch Dam Design, May 1994

USACE - EM 1110-2-2200 Gravity Dam Design, June 1995

Material Information

- Water – 63 pcf
- Stone – 162 pcf
- Soil shearing resistance – 45 degrees
- Soil cohesion – 10 ksf

The design criteria and load cases were adopted from EM 1110-2-2100 Stability Analysis of Concrete Structures. It states an overturning analysis and a sliding stability analysis is to be performed on the structures. The load cases were compared to the load cases in the Gravity and Arch Dam Design EMs to see if there was any noticeable differences. Each EM had similar load cases that were geared towards its specific dam type.

5.1.2 Applied Loads

The following loads are the loads that were calculated and applied to the masonry dam.

- Vertical Loads
  - Weight of Stone
    - The actual stone used in the construction of the masonry dam is unknown. It is assumed that the stone used is granite at 162lb/cf.
  - Weight of Water
  - Hydrostatic Uplift Pressure
    - The hydrostatic uplift pressure was calculated being fully pervious and a linear difference between the upstream and downstream pressures.
- Lateral Loads
  - Hydrostatic Water Pressure (Crest, Sidewall Overtopping, PMF)
  - Ice
    - Per EM 1110-2-2201, in the absence of ice design data, a 5k/lf should be assumed.

5.1.3 Load Cases

The load cases were chosen based on the referenced EM's noted above. Load cases including Construction and Maintenance were not analyzed.

- SU - Static Usual Load Case (Normal Operating Reservoir Condition, Crest)
- SUN - Static Unusual Load Case (Reservoir Spilling over Sidewalls)
- SE1 - Static Extreme Load Case 1 (Reservoir at PMF)
- SE2 - Static Extreme Load Case 2 (Reservoir at Crest, With Ice load)

5.1.4 Basis of Analysis

Stability Analysis of Concrete Structures was first used to assemble design criteria as it is the newest updated EM as compared to the Designs on Arch and Gravity Dams. As the Estling Lake spillway dam is a masonry structure and the fact that there is essentially no available design criteria for masonry structures, multiple design references were used to complete this analysis. Below is the list of pertinent design elevations of the masonry dam:

Elevations (NAVD88)

- Crest Elevation – 514.7
- Base Elevation – 505.7
- Side wall Elevation – 519.2
- Upstream (UP) PMF Elevation – 527.3

- Downstream (DS) Water - Normal – 508.0
- DS Water – Side Wall – 515.0
- DS Water - PMF – 527.3

All water surface elevation data was taken from the Updated Hydrologic and Hydraulic Analysis, New Jersey Transit – Estling Lake Dam (SWM, 2019). The conditions for each loading case are summarized on **Table SS-1.**

Table SS-1. Loading Cases

Load Case		US WSEL (ft)	DS WSEL (ft)
SU	Usual	514.7	508.0
SUN	Unusual	519.2	515.0
SE1	Extreme	527.3	527.3
SE2	Extreme	514.7[1]	508.0

[1] Includes ice load.

5.2 Model Configuration/Results

Following the guidance in the Engineering Manuals, a global stability analysis was performed including overturning and sliding. The arch shape was ignored and the dam was analyzed as a gravity dam using the cross section of the dam (**Figure SS-1**). This is a conservative assumption as the geometric properties of an arch shape allowed the loads to be transferred to adjacent abutments embedded within the embankment rather than having the loads being resisted by only the soil interaction at the base. EM 1110-2-2201 Arch Dam Design states that all arch dam design shall be completed using a finite element analysis. Considering the scope of this task and using engineering judgment, this conservative approach was taken by treating the section as a gravity dam. The dam's cross section closely follows the design principals of gravity dams which most likely was the intent of the original designers. It is understandable the design intent was to incorporate an arch shape into the gravity dam layout.

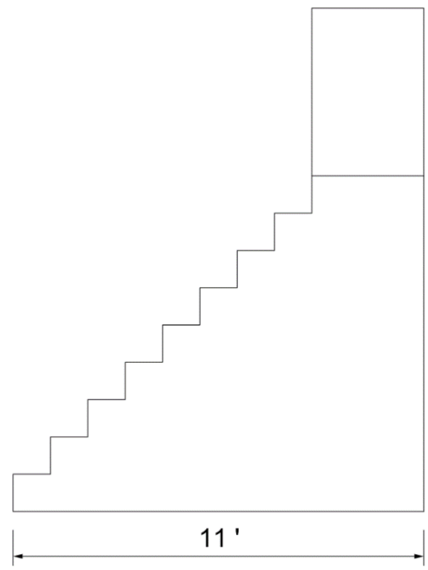


Figure SS-1. Masonry Dam Cross Section

5.2.1 Overturning Stability Analysis

A free body diagram was developed for each load case for the overturning stability analysis. A resultant vertical force and a location was then calculated. The location of this resultant determines if the structure will pass or fail. The resultant location criteria are as follows:

- SU (Usual) – Within the middle third of the structure
- SUN (Unusual) – Within the middle half of the structure
- SE (Extreme) – Within the structure

Table SS-2. Overturning Stability

Load Case		Resultant Location Criteria			Res. Loc.	PASS/FAIL	
SU	Usual	Middle 3rd	3.67	7.33	4.9	PASS	PASS
SUN	Unusual	Middle Half	2.75	8.25	5.0	PASS	PASS
SE1	Extreme	Within Base	0	11	4.9	PASS	PASS
SE2	Extreme	Within Base	0	11	12.1	PASS	FAIL

Table SS-2 shows that the structure passes the overturning stability analysis for 3 of the 4 load cases. In SE2, which is the ice loading load case, the resultant moves outside of the resultant location criteria. The 5 kip load placed at the crest creates enough of a moment to shift the resultant outside of the criteria. Under this load case it was assumed the ice loading was only on the upstream side of the dam. If the same ice load is applied on the downstream side at the downstream water level, the resultant moves back into the location criteria. Also per EM 1110-2-2100, "*Loads due to ice are usually not critical factors in the stability analysis for hydraulic structures. They are more important in the design of gates and other appurtenances. Ice damage to gates is quite common, but there is no known case of a dam failure due to ice. Where ice loads must be considered, refer to **EM 1110-2-1612**.*" Therefore, it is understood that the ice load case failure can be neglected for global stability. Ice would most likely have more of an impact in damaging the mortar joints of the dam rather than the overall stability of the dam.

5.2.2 Sliding Stability Analysis

In the sliding stability analysis the same free body diagrams were used to compare the overall normal load of the dam to the friction force between the dam's base and the soil. The soil parameters used within this analysis include a soil shear resistance of 35 degrees and a soil cohesion of 0 psf. This data comes from soil layer 5 within Table S-7. We are assuming the structure is sitting directly on this soil layer even though the drawings do call out that there is cribbing under the structure. This is because assigning material parameters for the cribbing would require physical testing of that structure, which itself would require removing the spillway – an infeasible option. The required factor of safety for each load case are as follows:

- SU (Usual) – 2
- SUN (Unusual) – 1.7
- SE (Extreme) – 1.33

Table SS-3. Sliding Stability

Load Case		Req. FOS	FOS	PASS/FAIL
SU	Usual	2	1.8	FAIL
SUN	Unusual	1.7	2.6	PASS
SE1	Extreme	1.33	1.4	PASS
SE2	Extreme	1.33	0.6	FAIL

Table SS-3 shows that the structure passes in sliding for two load cases and fails for two. Examining the two load cases that fail, it shows that for SU – Usual load case, a required FOS of 2 is required and the structure has a FOS of 1.8. This means the existing structure does not meet the standard design guidelines states within the Engineering Manuals, but the evaluation does not predict structural failure. For the SE1 – Extreme load case, the evaluation predicts that the structure fails with an FOS of 0.6.

5.2.3 Sensitivity Analysis

Inflow conditions to the reservoir normally vary and affect different overflow water surface elevations at the masonry spillway. To account for this variation a sensitivity evaluation was completed for water surface elevations up to 0.5 feet greater than the crest elevation of 514.7 ft. The analyses were re-run for the modified hydraulic loading associated with a water surface elevation of 515.2 ft. Although the factors of safety changed in some cases, the PASS/FAIL results remained the same for overturning and sliding as shown above.

5.3 Limitations

An overturning stability analysis and a sliding stability analysis was performed on the masonry dam at Estling Lake. These analyses are both in a global failure mode analysis. They do not take into account other possible failure modes. The main concern of failure should be the mortar joints between the stones that comprise the dam. These mortar joints can crack and leach water through the dam. This is of concern during the winter months as once a crack starts, the freezing water that has leached into the joints can propagate the crack to the point of failure. But there is a benefit to the arch design. During the previous inspection it was observed that these stones looked to be cut and interlocked in a fashion that the joints and stones are being compressed under normal load.

Notable Assumptions

- Analyzed as a gravity dam
- Stone is granite at a density of 0.162 kip/cf
- Soil Shear Resistance is 35 degrees
- Soil Cohesion is 0 ksf
- Linear drop in water surface elevation from upstream to downstream that follows the same angle as the dam.
- Ice Load is 5 kips/ft only on upstream side

6 Conclusion and Recommendations

The series of evaluations completed for this report document conditions that could lead to embankment and spillway failure. HDR recommends that these evaluations be considered within the context of the risk these conditions present, and that a plan be developed to reduce those risks. The PFMA to be completed for Estling Lake Dam will provide the information needed to begin development of that plan.