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Safety Factor Assessment Report

Area 15

DTE Monroe Power Plant

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Executive Summary

This Coal Combustion Residuals (CCR) Certification Report for Area 15 at the DTE Monroe Power Plant has been prepared in accordance with the requirements specified in the United States Environmental Protection Agency (USEPA) CCR Rule under 40 Code of Federal Regulations (CFR) §257.100 (e). These regulations require that the specified documentation, assessments and plans for inactive CCR surface impoundments be completed by April 17, 2018.

Required factors of safety were analyzed in compliance with the CCR Rule. The engineering investigations, analyses, and evaluations determined that the Area 15 impoundment meets the regulatory requirements for the safety factor assessment analysis, as summarized in **Table ES-1**.

Table ES-1 – Certification Summary

Report Section	CCR Rule Reference	Requirement Summary	Requirement Met?	Comments
Safety Factor				
4.1	§257.73 (e)(1)(i) per §257.100 (e)	Maximum storage pool safety factor must be at least 1.50	Yes	Safety factors were calculated to be 1.54 and higher.
4.1	§257.73 (e)(1)(ii) per §257.100 (e)	Maximum surcharge pool safety factor must be at least 1.40	Yes	Safety factors were calculated to be 1.45 and higher.
4.2	§257.73 (e)(1)(iii) per §257.100 (e)	Seismic safety factor must be at least 1.00	Yes	Safety factors were calculated to be 1.22 and higher.
4.2	§257.73 (e)(1)(iv) per §257.100 (e)	Liquefaction safety factor must be at least 1.20	Yes	Safety factors were calculated to be 1.30 and higher.

1 Introduction

1.1 Purpose of this Report

Area 15 is an inactive CCR surface impoundment as defined by 40 CFR §257.53. The CCR Rule requires that a safety factor assessment for an inactive CCR surface impoundment be completed by April 17, 2018. 40 CFR §257.100 (e) specifically states:

40 CFR §257.100(e)(3)

- *(v) No later than April 17, 2018, complete the initial hazard potential classification, structural stability, and safety factor assessments as set forth by § 257.73(a)(2), (b), (d), (e), and (f).*

40 CFR §257.73(e)

- *(1) Conduct initial and periodic safety factor assessments for each CCR unit and document whether the calculated factors of safety for each CCR unit achieve minimum safety factors specified in (e)(1)(i) through (iv) of this section for the critical cross section of the embankment. The critical cross section is the cross section anticipated to be the most susceptible of all cross sections to structural failure based on appropriate engineering considerations, including loading conditions. The safety factor assessments must be supported by appropriate engineering calculations.*

The aforementioned regulatory requirements and the corresponding section of this report are summarized in **Table 1-1** below.

Table 1-1 – CCR Rule Cross Reference Table		
Report Section	Title	CCR Rule Reference
4.1	Factor of Safety: Maximum Storage Pool Loading	§257.73 (e)(1)(i) per §257.100 (e)
4.1	Factor of Safety: Maximum Surcharge Pool Loading	§257.73 (e)(1)(ii) per §257.100 (e)
4.2	Factor of Safety: Seismic	§257.73 (e)(1)(iii) per §257.100 (e)
4.2	Factor of Safety: Post-Liquefaction	§257.73 (e)(1)(iv) per §257.100 (e)

Area 15 consists of a bottom ash impoundment bordered by Lake Erie to the east and an existing channel to the west, which discharges cooling water from the Monroe coal power plant to the lake. The impoundment is separated from the discharge channel and Lake Erie by a perimeter dike. The southern boundary of the ash pond is formed by an earthen divider berm constructed of aggregate material, which separates the ash pond from the coal pile runoff pond to the south. Both the perimeter dike and divider berm have been evaluated to determine whether safety factor requirements are met. The following sections summarize the evaluations performed and the results from the analyses.

This report presents the methodology and results of AECOM's geotechnical investigation and stability analyses for Area 15 perimeter dike and divider berm (see **Figure 1** of **Appendix A** for Site Location Map). The purpose of the geotechnical investigation and analyses is to evaluate the design, performance, and condition of the impoundment using available design drawings, construction records, inspection reports, previous engineering investigations, reports and analyses, Station operating records, and other pertinent documents. This information combined with a site-specific subsurface investigation and laboratory testing program were used by AECOM to perform slope stability analyses in accordance with the CCR Rule.

2 Summary of Investigations

A subsurface exploration was performed at Area 15 in November, 2017 which included 12 soil borings, numbered B-01 through B-13, omitting B-06. Borings B-01 through B-05, B-08, and B-09 were drilled on the crest of the impoundment perimeter dike. Borings B-07, B-10, and B-11 were drilled within the northern portion of the impoundment where bottom ash fill had already been placed. Borings B-12 and B-13 were drilled on the crest of the divider berm separating the impoundment from the water bodies to the south. All boring locations are shown on **Figures 2-1** and **2-2** of **Appendix A**. All borings were drilled by AECOM's subcontractor TTL Associates, Inc. of Toledo, Ohio. The borings were drilled from November 6 to 17, 2017 using a CME 75 truck-mounted drilling rig with full-time oversight by an AECOM geotechnical engineer. The borings were drilled to depths ranging from approximately 33 feet to 69 feet below ground surface (ft bgs).

A total of nine stratigraphic units were identified as a result of the subsurface explorations. Seven of the units were relevant to this slope stability analysis. Brief descriptions of each relevant unit are as follows:

- **Fill – Perimeter Dikes**: The perimeter dikes were found to be constructed of fill material, consisting of a variety of interlayered materials including sand (USCS type SP, SP-SM, SW), gravel (GP), silty sand (SM), lean clay (CL), silty clay (CL-ML), organic silt as topsoil (OL), asphalt and cobbles.
- **Fill – Divider Berm**: The divider berm was found to be constructed of fill material consisting of moist to wet, light to dark gray crushed limestone classified as sandy gravel (GP and GW-GM) with trace amounts of silt.
- **Silt and Clay**: A unit of soft cohesive silt and clay was encountered below the fill materials at both the divider berm and outer dikes and consisted of cohesive elastic silt (MH) with organics and cohesive silt (ML) with organics, lean clay (CL) and silty clay (CL-ML) with variable proportions of organics, and minor proportions of non-cohesive silt (ML) with organics. The unit contained moderate organic content, which generally decreased and eventually disappeared with depth.
- **Loose Sand**: A unit of loose sand was encountered below the soft cohesive silt and clay unit and consisted of generally wet, gray poorly graded sand (SP), poorly graded sand with silt (SP-SM), silty sand (SM), clayey sand (SC) and silty clayey sand (SC-SM).
- **Sand and Gravel**: A unit of sand and gravel was encountered below the loose sand and consisted of generally wet, gray and brown interbedded silty sand (SM), clayey sand (SC), well-graded sand with silt and gravel (SW-SM) and gravel layers classified as poorly graded gravel (GP) and well graded sand with silt and gravel GW-GM).
- **Till**: Clay till was encountered at all the borings beneath the sand and gravel unit and consisted of very stiff to hard lean Clay (CL) with interbedded minor proportions of silty Clay (CL-ML) and Silt (ML). The clay material contained variable proportions of gravel, sand, silt and clay.
- **Bedrock:** Bedrock primarily consisted of light gray moderately weathered limestone and/or dolomite In general, the bedrock was found to generally slope downward from west to east (towards Lake Erie), and is shallowest at a possible bedrock ridge noted near Borings B-03 and B-04 which were nearest the existing overflow weir.

Groundwater was encountered in all borings and the static groundwater table was found to be generally consistent with the pool elevations of the Area 15 pond.

3 Safety Factor Assessment

Analyses completed for the safety factor assessment of the ash impoundment are described in this section. Data from AECOM's subsurface exploration and laboratory testing results, design information provided by DTE, and safety factor criteria outlined in 40 CFR §257.100 (e) for inactive CCR impoundments were used to complete the safety factor assessment of the ash impoundment located at the subject site. As part of the assessment, global or general failure analyses were performed by AECOM to evaluate the potential for mass slope instabilities of the perimeter dike and divider berm. The potential for global instability is dependent on factors such as slope geometry, groundwater/phreatic surface conditions, loading conditions, and shear strength of the embankments and foundation materials. This section summarizes the methodology, loading conditions and assumptions of the analyses. Figures and computer program outputs are provided in **Appendix D**.

3.1 Cross-Sections Selected for Analysis

Five (5) cross sections were identified for the stability evaluation of the Area 15 impoundment dikes and berms: four along the dike on the west side of the impoundment, two on the divider berm, and one along the east side of the impoundment. Locations of the sections are shown **on Figure 2** of **Appendix A:**

- **Cross-Section A-A':** This section was analyzed based on stratigraphy from Boring B-02 on the west side of the crest of the western perimeter dike and from Boring B-01 on the east side of the crest of the perimeter dike.
- **Cross-Section B-B':** This section was analyzed based on stratigraphy from Boring B-03 on the crest of the western perimeter dike, south of the fly ash slurry pipeline crossing of the discharge channel and north of the existing overflow weir.
- **Cross-Section C-C':** This section was analyzed based on stratigraphy from Boring B-04 on the crest of the western perimeter dike, south of the existing overflow weir, and just north of the divider berm.
- **Cross-Section D-D':** This section is representative of the southern divider berm and was analyzed based on stratigraphy from Boring B-12 on the crest of the berm. The section was drawn roughly perpendicular to the centerline of the divider berm.
- **Cross-Section E-E':** This section is representative of the eastern perimeter dike and was based on stratigraphy from Borings B-08 and B-09 drilled on the eastern perimeter dike, just north of the divider berm.

Stratigraphy for each cross-section was established from the subsurface information depicted by the borings as indicated above.

The topography for each analysis cross-section was determined based on specific ground surveys performed to support this project. Bathymetry data from plans provided by DTE are available within the impoundment and along a portion of the dredged channel abutting the western dike of the pond. This data was used to establish bathymetric grades for analysis cross-sections A-A', B-B', C-C', and D-D'. Bathymetry was not available within Lake Erie on the eastern side of the pond. AECOM reviewed National Oceanic and Atmospheric Administration (NOAA) nautical charts (specifically Sheet 14 of the document entitled "Nautical Chart 14846: West End of Lake Erie") which depict soundings in Lake Erie adjacent to the project site. The soundings indicate shallow water (water depth of 3 ft or less) within 1,000 ft of the eastern dike of the ash impoundment. Conservatively, the general bathymetry from the dredged discharge channel (which has significantly deeper bathymetric grades) was applied to the model for Section E-E’, along the eastern dike.

3.2 Material Engineering Parameters

Material properties for slope stability analyses were developed using both laboratory testing data (index and strength testing) and strength correlations from SPT and pocket penetrometer data. Material shear strength parameters used in the slope stability analyses for each of the pertinent strata are provided in **Table 3-2** below. Application of the material properties in the table to the specific stability analysis loading conditions is discussed in **Section 3.4**.

To summarize:

- For the fine-grained foundation soils (mostly composed of silt and clay), peak effective and undrained strengths were selected based on conservative interpretation of Consolidated Undrained (CU) Triaxial test data in accordance with the Modified Mohr-Coulomb plot (p-q and p’-q plots) procedures, as described in Appendix D of the United States Corps of Engineers Manual EM-1110-2-1902 “Slope Stability.” In analyzing the test results, the deviator stress corresponding to an axial strain of 12% was used for determining failure criteria. This failure criteria was used to establish the shear strength parameters for both effective and total strengths. Modified Mohr-Coulomb plots are provided in **Appendix B**.
- For the embankment fill soils (perimeter dike and divider berm), and other foundation soils such as loose sand, gravel and till soils, the engineering parameters (unit weight and shear strength) were estimated based on correlations with SPT N-values, pocket penetrometer readings, our experience with these soil types and conservative engineering judgement.

Table 3-2 – Material Properties For Slope Stability Analyses

Material	Unit Weight (pcf)	Peak Effective (drained) Shear Strength Parameters		Peak Total (undrained) Shear Strength Parameters	
		c’ (psf)	Φ’ (°)	c (psf)	Φ (°)
Fill –Dike[1]	120	0	30	0	30
Fill – Divider Berm[1]	130	0	33	0	33
Soft Clay and Silt Strata[2]	100	110	32	220	16.4
Loose Sand[1]	120	0	28	0	28
Sand and Gravel[1]	120	0	33	0	33
Till[1]	130	4,000	0	4,000	0
Bedrock	Assumed to be impenetrable in the slope stability models				

Notes:
1. Engineering parameters estimated based on correlations with SPT N-values, pocket penetrometer readings, our experience with these soil types and engineering judgement.
2. Shear strength parameters estimated based on interpretation of Consolidated Undrained (CU) Triaxial test data in accordance with the Modified Mohr-Coulomb plot (a p-q and p’-q plot) procedures, as described in Appendix D of the United States Corps of Engineers Manual EM-1110-2-1902 “Slope Stability.”

3.3 Loading Conditions

Consistent with the criteria provided in the USEPA CCR Rule §257.100 (e), the stability of the perimeter dike and divider berm was evaluated for four load cases:

3.3.1 Static, Steady-State, Normal Pool Condition

This case models the perimeter dike and divider berm embankments under static, long-term conditions, at normal water level within the impoundment. The USEPA CCR Rule requires a maximum storage pool factor of safety greater than or equal to 1.50.

3.3.2 Static, Maximum Surcharge Pool Condition

This case models the conditions under short-term surcharge pool conditions, which herein was taken as a condition in which the water level in the ash impoundment is at El. 577, which corresponds to between 0 and 1 ft below the top of the dike at any location. This condition requires a minimum factor of safety greater than or equal to 1.40.

3.3.3 Seismic (Pseudostatic) Load Condition

These analyses incorporate a horizontal seismic coefficient, k_h, selected to be representative of expected loading during the design earthquake event (i.e., a “pseudostatic” analysis). The design earthquake event is one with a 2% probability of exceedance in 50 years (approximately 2,500 year recurrence interval), as required by the CCR Rule. The horizontal seismic coefficient was estimated to be 0.11g for the subject site as described in **Section 3.4** and in **Appendix C**. The analyses utilized peak undrained strength parameters for soils that are not considered to be rapidly draining materials (such as the soft clay and till foundation soils), and peak drained strengths for materials that are rapidly draining (the various sand strata at the site). The phreatic surface and pore water pressures corresponding to the steady-state pool from the static analyses were utilized. This loading condition requires a minimum Factor of Safety greater than or equal to 1.00.

3.3.4 Post-Liquefaction Condition

The purpose of the post-liquefaction stability analysis is to assess stability conditions immediately following the design seismic event. No horizontal seismic coefficient is included in these analyses, but selection of strength parameters for the analyses takes into account the potential for the liquefaction of the soils as a result of pore pressures generated in sand-like materials, or cyclic softening in clay-like materials due to the earthquake shaking. Liquefaction potential analysis was performed on the foundation soils as explained in **Section 3.4.2.2**.

3.4 Methodology

Limit equilibrium stability analysis was completed using the two-dimensional Slope/W computer program by Geo-Slope International. Factors of safety were calculated using Spencer’s method and using iterative analyses of both circular and block failure surfaces to determine the critical failure surface for each analysis section and load case. Shallow finite slope failure surfaces were not considered as they correspond to sloughing which can be addressed as part of regular maintenance. Critical surfaces with respect to dam safety were considered to be those which intersected the dike crest at or upstream of its centerline. Such failures are considered to have the potential to create an immediate threat to dike safety. Pore pressures were assigned as hydrostatic pressure under the phreatic surface.

A summary of the analyses is presented in the following sections. Full results of the analyses are presented in **Appendix D**.

3.4.1 Static Analysis Conditions

3.4.1.1 *Pool Elevations*

The static analysis conditions include the steady-state normal pool and maximum surcharge pool loading conditions. Static stability was evaluated for steady-state conditions using a normal pool elevation of 575 ft, and a flood pool surcharge elevation of the lower of 577 ft or the peak elevation of the dike crest at the cross section location (whichever was lower).

3.4.1.2 *Phreatic Surface*

The phreatic surface used in the steady-state normal pool condition was established using the pool elevations of the pond, the discharge channel, and Lake Erie in conjunction with the groundwater levels encountered in each boring. The water elevations were drawn into the stability models with straight line interpolation between the pond pool elevation, boring locations, and discharge channel or Lake Erie pool elevations. The discharge channel and Lake Erie were assumed at El. 570, which corresponds to the average low water level in Lake Erie (as defined by NOAA).

For the maximum surcharge pool condition, the pool level in the pond was raised to the surcharge pool elevation, but the water level in Lake Erie was kept at El. 570 ft. The straight-line interpolation described above was applied for this case as well. Therefore, the phreatic surface used for this loading condition corresponds to steady-state seepage to the raised pool level. This is a conservative representation, as the maximum storage pool water level is likely to be a short-term event and steady state seepage conditions through the dike are unlikely to develop.

3.4.1.3 *Shear Strength Parameters*

For the steady-state normal pool condition, drained (effective stress) shear strength parameters, as shown in **Table 3-2**, were used for all materials.

The change in water level from the normal pool case to the maximum surcharge pool condition is relatively small (less than 3 vertical ft). The small forcing effect created by this change is not expected to generate an undrained stress condition in the dike or its foundation. Therefore, drained (effective stress) shear strength parameters were used for all materials under the maximum surcharge pool loading condition as well.

3.4.2 Earthquake Analysis Conditions

AECOM determined the peak ground acceleration (PGA) at the top of competent bedrock at the subject site associated with a 2 percent probability of exceedance in 50 years (approximately 2,500-year return period) using the seismic unified hazard tool online tool available on the United States Geological Survey website (https://earthquake.usgs.gov/hazards/interactive/). The bedrock PGA was then corrected to account for amplification through the overburden soils at the site, to estimate the design ground accelerations for use in the liquefaction triggering analyses and the horizontal seismic coefficient for use in the seismic (pseudostatic) stability analysis as described in **Appendix C**.

Liquefaction triggering analyses were completed to assess the potential for liquefaction or cyclic softening of the materials and determine the appropriate material properties for use in the post-liquefaction slope stability loading condition.

3.4.2.1 *Horizontal Seismic Coefficient*

Based on USGS data, the peak ground acceleration on bedrock for the design earthquake event at the site location is approximately 0.063g. This acceleration was corrected to account for amplification of the bedrock motions through the soil overburden column, as described in **Appendix C**. After correction, a horizontal seismic coefficient, k_h, of 0.10g was calculated for use in the seismic (pseudostatic) loading condition slope stability analysis. This same acceleration was also used as the peak ground acceleration at the ground surface, used in the liquefaction potential analyses, described in the next section.

3.4.2.2 ***Liquefaction Potential Analysis (SPT-Based Triggering Analyses)***

The subsurface investigation revealed layers of loose granular soils underlying the site. Such materials are candidate for experiencing liquefaction when subject to strong ground motions. Liquefaction potential analyses were performed using the procedure proposed by Idriss and Boulanger (2008, 2014). The procedure considers a stress-based approach to evaluate the potential for liquefaction triggering, and compares calculated earthquake-induced cyclic stress ratios (CSRs) with the estimated cyclic resistance ratios (CRR) of the soil to establish the factor of safety against liquefaction triggering. This methodology is considered to be a conservative, screening-level procedure.

Within the method, the factor of safety against liquefaction triggering is defined as:

$$FS_{liq} = \frac{CRR}{CSR}$$

The CRR is the cyclic resistance ratio at which liquefaction occurs during an earthquake. It is obtained from case history-based semi-empirical correlations with SPT values recorded at sites with level ground conditions, and it also is normalized to $\sigma'_v \approx 1$ atm for an earthquake with M = 7.5. Within the Simplified Procedure, The CRR is a function of a soil's fines content (FC), relative density and effective stress, and penetration resistance (SPT or CPT). The CRR is also dependent on the duration of shaking, and is adjusted to the site-specific design earthquake using a Magnitude Scaling Factor (MSF).

Each sampling interval where a SPT result is available from each boring is analyzed within this method. Intervals that meet all of the following criteria are interpreted as being candidate for liquefaction under the design earthquake: 1) The interval lies below the water table or is saturated; 2) The interval corresponds to a material classified as a sand or gravel, or a fine-grained soil with plasticity index less than 7; 3) The factor of safety against liquefaction triggering is less than or equal to 1.20.

As stated above, the cyclic resistance ratio (CRR) depends on both the magnitude of the design earthquake event, and on the fines content of the soil. To determine the design earthquake magnitude, the Unified Hazard Tool from the U.S. Geological Survey – Earthquake Hazards Program was used to deaggregate the seismic risk of the project site. From this, the earthquakes that compose the greatest risk to the site are earthquakes with magnitudes between 5 and 7. The design magnitude of the earthquake chosen in the liquefaction screening analysis was 6.5. Regarding fines content, laboratory testing of the grain-size analyses performed on the sand was used where available.

The analysis also takes as input the peak ground acceleration at the ground surface (meaning the ground surface at the boring location) induced by the design earthquake. As described previously, the PGA on bedrock is 0.063g based on USGS data. Accounting for amplification through the overburden, this acceleration is 0.10g at the top of the soil column. The latter acceleration was used as the PGA input to the liquefaction potential analyses.

The analyses were performed for each boring where the Loose Sand soil unit was encountered including Borings B-01 to B-04, B-08, B-09, and B-12. The analyses focused on studying the liquefaction potential of this sand soil unit.

Spreadsheets developed by AECOM utilizing the above procedure and in conjunction with SPT data were used for the screening-level analysis performed herein. The spreadsheet calculates a Factor of Safety against liquefaction, which is defined as the quotient of the soil's cyclic resistance ratio and the cyclic stress ratio induced by the earthquake. The spreadsheet limits liquefaction factors of safety to 2.0, even if the computed factor of safety is higher than 2.0.

Complete results of the liquefaction potential analysis are provided in **Appendix E.**

3.4.2.3 ***Pool Elevations and Phreatic Surface***

Pool elevation in the pond and the phreatic surface for both the seismic and post-liquefaction loading conditions were the same as utilized in the steady-state normal pool loading condition.

3.4.2.4 ***Shear Strength Parameters***

– **Pseudostatic Loading Condition:** Peak undrained strength parameters (as summarized in **Table 3-2**) were utilized in the slope stability analyses of the seismic loading condition. As this condition incorporates a horizontal seismic coefficient, liquefied strengths are not pertinent to the analysis and were not utilized.

– **Post-Liquefaction Loading Condition:** The post-liquefaction loading case represents conditions following the design earthquake, and no horizontal seismic coefficient is incorporated. As described in **Section 4.2.1** below and further presented in **Appendix E,** the vast majority of the sand intervals analyzed in the liquefaction analyses had acceptable factors of safety, with only a few exceptions. Therefore, widespread liquefaction is not expected to be triggered within the Loose Sand soil unit during the design seismic event. However, since there were exceptions to the overall trend, the strength characterization of Loose Sand assuming a liquefied sand deposit was determined and modeled in the post-earthquake slope stability analyses, as a conservative measure.

  The liquefied strength (residual strength) of the Loose Sand soil unit was estimated following procedures in Idriss and Boulanger (2008). The method presented in that reference is based on empirical observations and back-analyses made at actual sites that have experienced liquefaction in past earthquakes and is based on correlations with SPT and CPT results. Specifically, the method relates the equivalent fines-corrected clean sand SPT blow count, $(N1)_{60CS\text{-}Sr}$, to the steady-state (post-liquefaction) shear strength, Sr.

  The analyses performed as part of the SPT-based liquefaction screening analysis (**Section 3.4.2.2**) utilizes the fines-corrected blow count, $(N1)_{60CS\text{-}Sr}$, and this parameter is calculated for each sample of sand within the spreadsheets used for the liquefaction potential analyses. These data were used to select the steady-state strength of the sand deposit, as follows:

  - The $(N1)_{60CS\text{-}Sr}$ for each sand sample among all borings were taken from the liquefaction screening analysis spreadsheet, and combined in a single graph. This is shown in **Figure 3**.
  - The median $(N1)_{60CS\text{-}Sr}$ was determined from the graph, and this value was selected for analysis purposes to represent the sand deposit as a whole. From **Figure 3**, the median $(N1)_{60CS\text{-}Sr}$ = 11.
  - **Figure 4** was then used to estimate the residual shear strength that corresponds to $(N1)_{60CS\text{-}Sr}$ = 11. As shown on the figure, the shear strength in the sand was determined to be S_r = 14 kPa or about 292 psf. A value for S_r of 250 psf was used in the analysis.

  The embankment fill soils and the Soft Clay foundation soils were generally soft to stiff. Based on that, the peak undrained shear strength used for these soils were reduced by 20% to account for the possibility of cyclic-softening of these materials.

Table 3-3 below presents a summary of the shear strength parameters used for the different material for checking the seismic (pseudostatic) and post-earthquake stability of the dikes.

Table 3-3 – Material Properties Considered For Seismic Slope Stability Analyses

Material	Unit Weight (pcf)	Pseudostatic Loading Condition		Post-Earthquake Loading Condition	
		c (psf)	Φ (°)	c (psf)	Φ (°)
Fill –Dike[1,3]	120	0	30	0	25
Fill – Divider Berm[1,3]	130	0	33	0	27
Soft Clay[2,3]	100	220	16.4	176	13.2
Loose Sand[1]	120	0	28	S_r = 250 psf	
Sand and Gravel[1]	120	0	33	0	33
Till[1]	130	4,000	0	4,000	0
Bedrock	Assumed to be impenetrable in the slope stability models				

Notes:
1. Engineering parameters estimated based on correlations with SPT N-values, pocket penetrometer readings, our experience with these soil types and engineering judgement.
2. Shear strength parameters estimated based on interpretation of Consolidated Undrained (CU) Triaxial test data in accordance with the Modified Mohr-Coulomb plot (a p-q and p'-q plot) procedures, as described in Appendix D of the United States Corps of Engineers Manual EM-1110-2-1902 "Slope Stability."
3. For post-earthquake loading condition, a 20% reduction in the peak undrained shear strength was considered.

4 Results of Analysis

4.1 Results of Static Stability Analyses

The results of the limit equilibrium slope stability analyses for the static load cases are summarized in **Table 4-1**. The SLOPE/W output figures showing the critical slip surfaces and details of the analyses are included in **Appendix D**.

Table 4-1 – Summary of Minimum Slope Stability Factors of Safety for Static Load Cases

Load Case	Criteria	Slip Surface Type	Analytical Cross-Section				
			A-A’	B-B’	C-C’	D-D’	E-E’
Steady-State (Normal Pool)	FS ≥ 1.50	Circular	2.31	1.75	1.54	2.84	2.42
		Block	2.31	2.04	1.87	2.79	2.41
Maximum Surcharge Pool	FS ≥ 1.40	Circular	2.16	1.69	1.45	2.68	2.28
		Block	2.17	1.98	1.77	2.74	2.30

The calculated factors of safety at all analysis sections are greater than the minimum values required per USEPA CCR Rule §257.100 (e) and §257.73 (e).

4.2 Results of Earthquake Stability Analyses

4.2.1 Liquefaction Triggering Analysis

Figure 5 in **Appendix A** portrays the calculated factors of safety within the Loose Sand soil unit. Data from all borings have been combined into the figure. One outlier lies below a factor of safety of 1.0 while the rest of the data is above the threshold for liquefaction. Based on the results of the screening analysis, it is concluded that widespread liquefaction is not expected to be triggered within the Loose Sand soil unit during the design seismic event. However, the strength characterization of the liquefied sand deposit was determined and modeled in the slope stability analyses to calculate the factor of safety for a post-earthquake condition, as a conservative measure. The results of the liquefaction analyses are presented in detail in **Appendix E**.

4.2.2 Earthquake Slope Stability Analysis

The results of the slope stability analyses for the seismic load cases are summarized in **Table 4-2**. The SLOPE/W output figures showing the critical slip surfaces and details of the analyses are included in **Appendix D**.

Table 4-2 – Summary of Minimum Slope Stability Factors of Safety for Earthquake Load Cases							
Load Case	**Criteria**	**Slip Surface Type**	**Analytical Cross-Section**				
			A-A’	**B-B’**	**C-C’**	**D-D’**	**E-E’**
Seismic (Pseudostatic)	FS ≥ 1.00	Circular	1.26	1.24	1.22	1.97	1.39
		Block	1.28	1.31	1.25	2.01	1.46
Post-Liquefaction	FS ≥ 1.20	Circular	1.38	1.33	1.30	2.60	1.49
		Block	1.36	1.36	1.31	2.52	1.51

The calculated factors of safety at all analysis sections are greater than the minimum values required in USEPA CCR Rule §257.100 (e) and §257.73(e).

5 Conclusions

The calculated factors of safety from the slope stability analysis satisfy the USEPA CCR Rule §257.100(e) and §257.73(e) requirements for all the load cases analyzed at the critical analysis sections for the DTE Monroe Area 15 perimeter dikes and divider berm. Load cases analyzed for this study included static (steady-state) normal pool, maximum surcharge pool, seismic (pseudostatic), and static post-liquefaction.

6 Limitations

Background information, design basis, and other data have been furnished to AECOM by DTE. AECOM has used this data in preparing this report. AECOM has relied on this information as furnished, and is not responsible for the accuracy of this information.

Borings have been spaced as closely as economically feasible, but variations in soil properties between borings, that may become evident at a later date, are possible. The conclusions developed in this report are based on the assumption that the subsurface soil, rock, and groundwater conditions do not deviate appreciably from those encountered in the site-specific exploratory borings. If any variations or undesirable conditions are encountered in any future exploration, we should be notified so that additional analyses can be made, if necessary.

The conclusions presented in this report are intended only for the purpose, site location, and project indicated. The recommendations presented in this report should not be used for other projects or purposes. Conclusions or recommendations made from these data by others are their responsibility. The conclusions and recommendations are based on AECOM's understanding of current plant operations, maintenance, stormwater handling, and ash handling procedures at the station, as provided by DTE. Changes in any of these operations or procedures may invalidate the findings in this report until AECOM has had the opportunity to review the findings, and revise the report if necessary.

This geotechnical investigation was performed in accordance with the standard of care commonly used as state-of-practice in our profession. Specifically, our services have been performed in accordance with accepted principles and practices of the geological and geotechnical engineering profession. The conclusions presented in this report are professional opinions based on the indicated project criteria and data available at the time this report was prepared. Our services were provided in a manner consistent with the level of care and skill ordinarily exercised by other professional consultants under similar circumstances. No other representation is intended.

7 References

U.S. Environmental Protection Agency [USEPA]. (2015). Standards for the Disposal of Coal Combustion Residuals in Landfills and Surface Impoundments. 40 CFR §257. Federal Register 80, Subpart D, April 17, 2015.

Idriss, I.M., and Boulanger, R. W. (2008). “SPT-Based Liquefaction Triggering Procedures”, Report No. UCD/CGM-10-02, Center for Geotechnical Modeling, Department of Civil and Environmental Engineering, University of California, Davis, CA.

Idriss, I.M. and Boulanger, R.W. (2014). “CPT and SPT Based Liquefaction Triggering Procedures”, Center of Geotechnical Modeling, Department of Civil and Environmental Engineering, University of California, Davis, California.

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AECOM (NYSE: ACM) is a global provider of professional technical and management support services to a broad range of markets, including transportation, facilities, environmental, energy, water and government. With nearly 100,000 employees around the world, AECOM is a leader in all of the key markets that it serves. AECOM provides a blend of global reach, local knowledge, innovation, and collaborative technical excellence in delivering solutions that enhance and sustain the world's built, natural, and social environments. A Fortune 500 company, AECOM serves clients in more than 100 countries and has annual revenue in excess of $19 billion.

More information on AECOM and its services can be found at www.aecom.com.

Appendix A
Figures

Map Data ©2018 Google

0 0.5 1.0

SCALE: 1"=1 mi.

N

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DTE MONROE POWER PLANT AREA 15

MONROE, MICHIGAN

GENERAL LOCATION MAP

DRAWN BY:	CHECKED BY:	PROJECT No:	DATE:	FIGURE No:
EJB	VGK	60516675	2/21/2018	1

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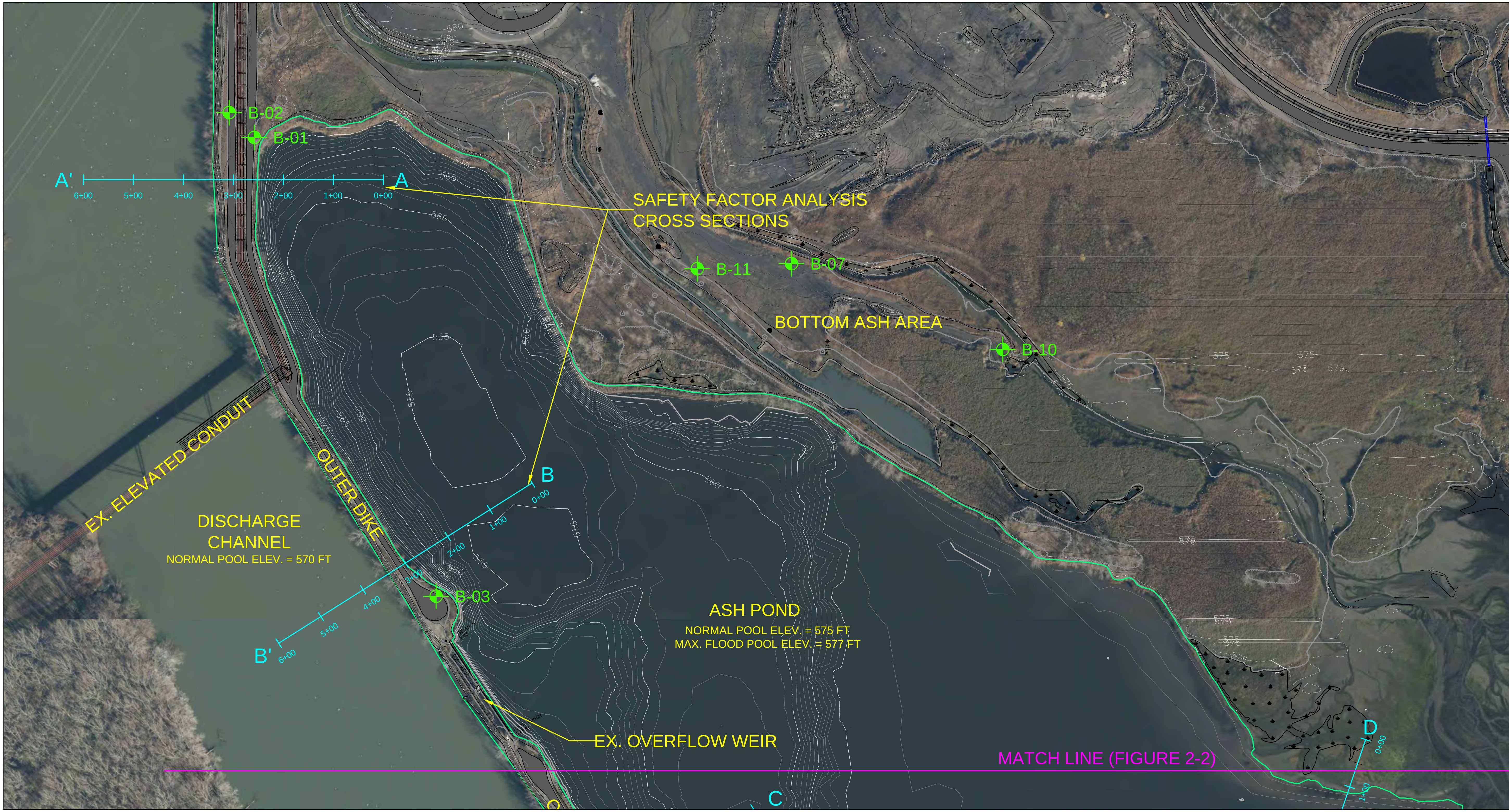

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SCALE: 1"=200'

SOIL BORING LOCATION

AECOM

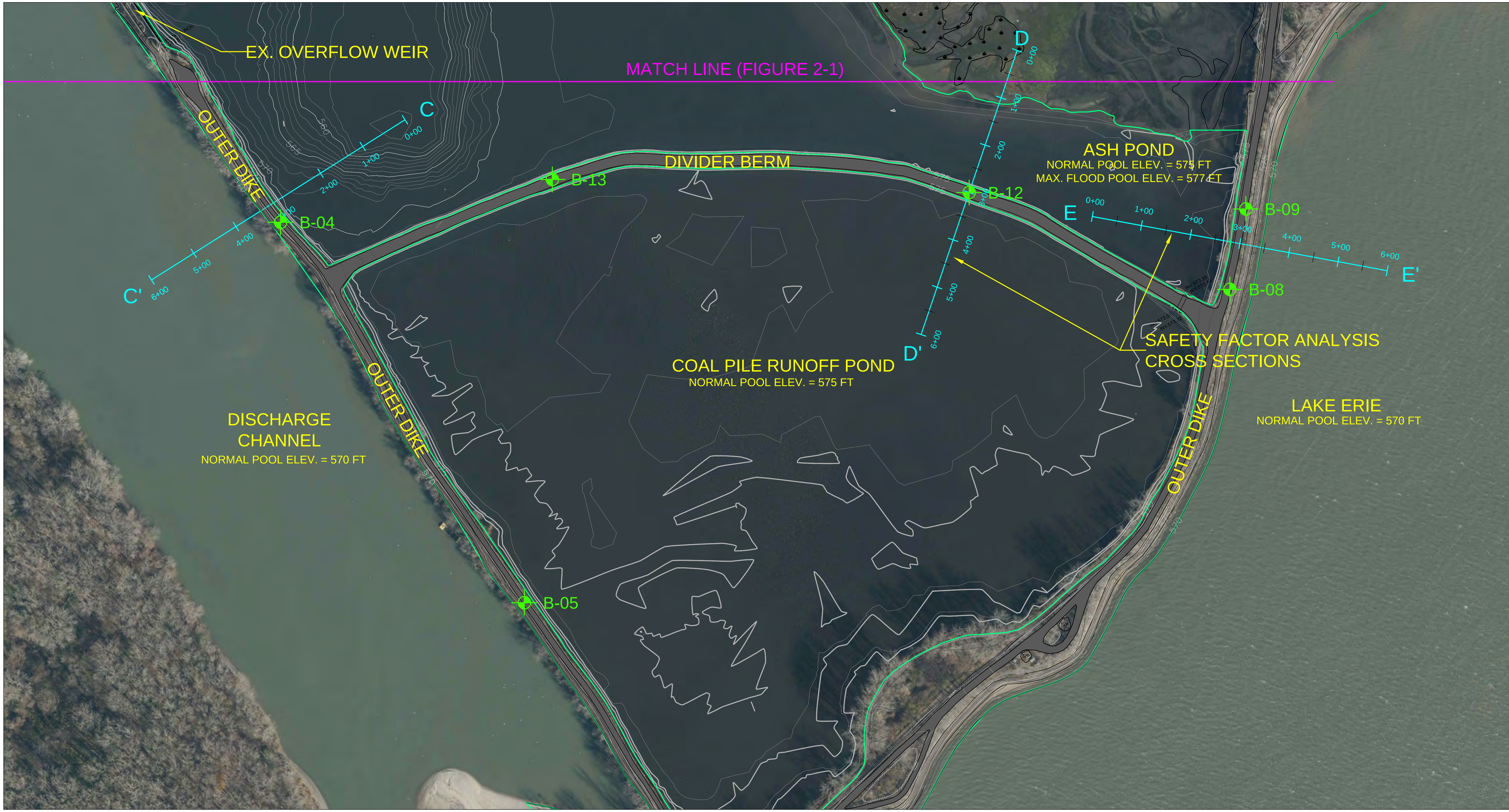
DTE MONROE POWER PLANT AREA 15

MONROE, MICHIGAN

GEOTECHNICAL SITE PLAN

SHEET 1 of 2

DRAWN BY:	CHECKED BY:	PROJECT No:	DATE:	FIGURE No:
EJB	VKG	60516675	3/8/2018	2-1



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SCALE: 1"=200'

SOIL BORING LOCATION

AECOM

DTE MONROE POWER PLANT AREA 15
MONROE, MICHIGAN

GEOTECHNICAL SITE PLAN
SHEET 2 of 2

DRAWN BY:	CHECKED BY:	PROJECT No:	DATE:	FIGURE No:
EJB	VKG	60516675	3/8/2018	2-2

Appendix B
Shear Strength Parameters

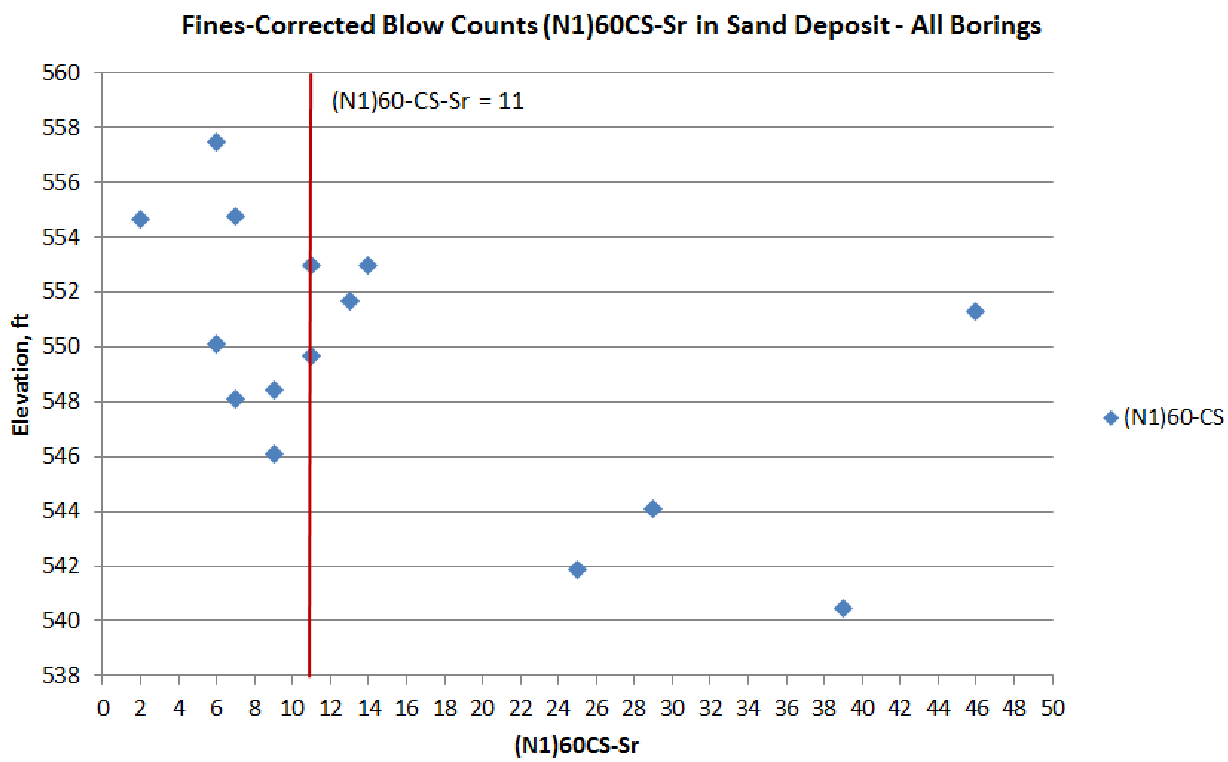


Figure 3 - Compilation of Fines Corrected Blow Counts in Loose Sand Soil Unit

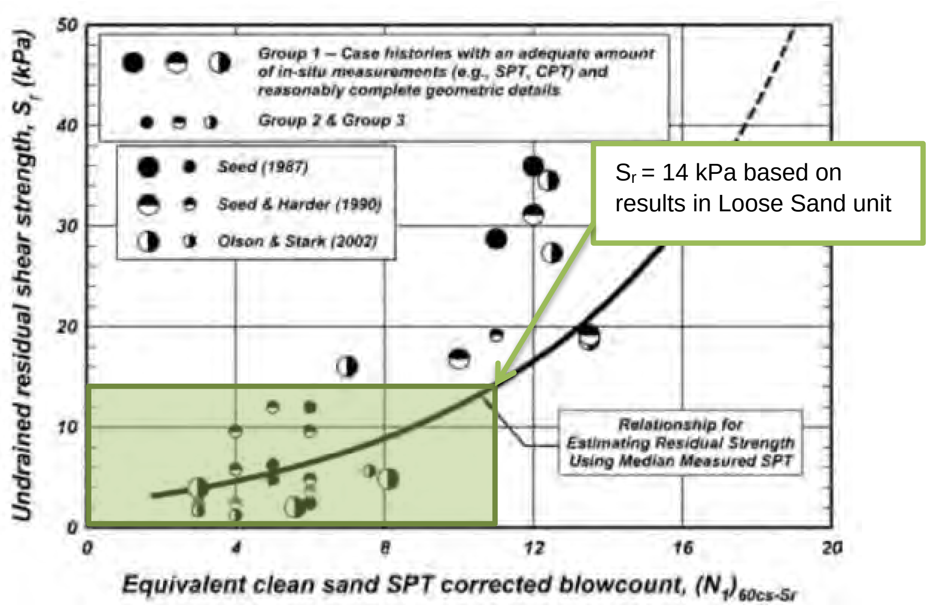


Figure 4 - Undrained Residual Shear Strength vs. Equivalent Clean Sand Blow Count

(Idriss and Boulanger, 2008)

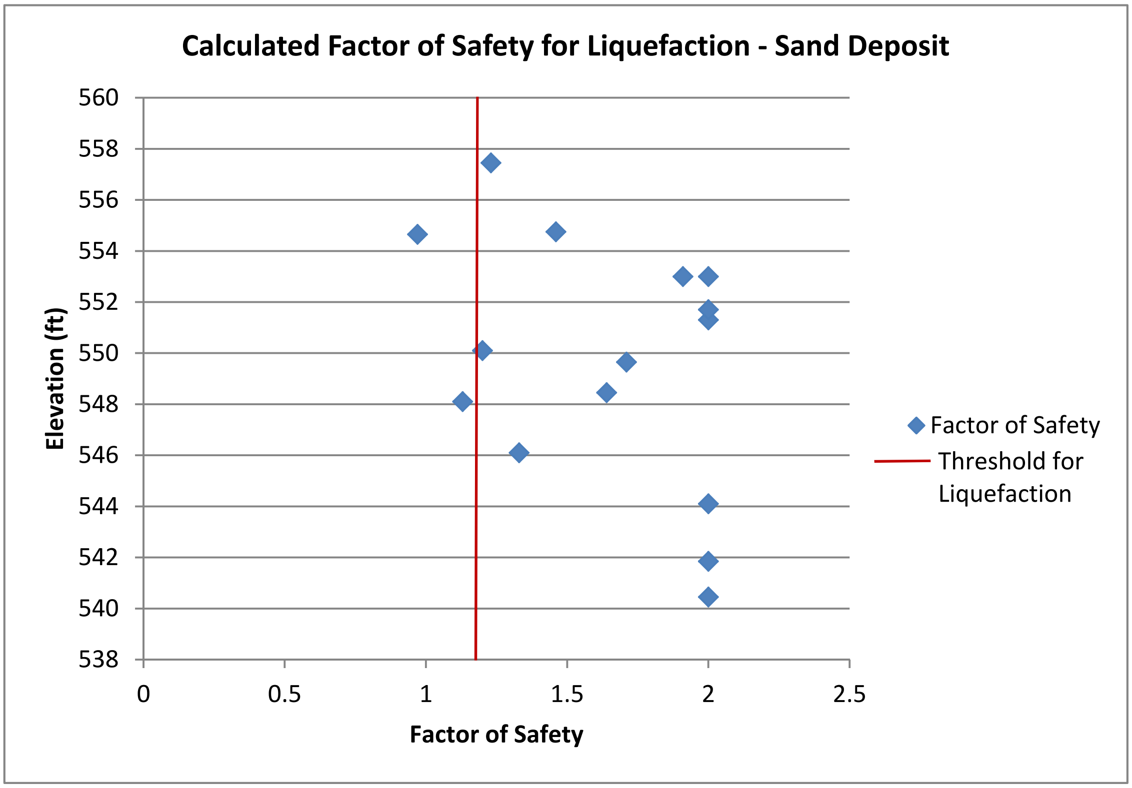
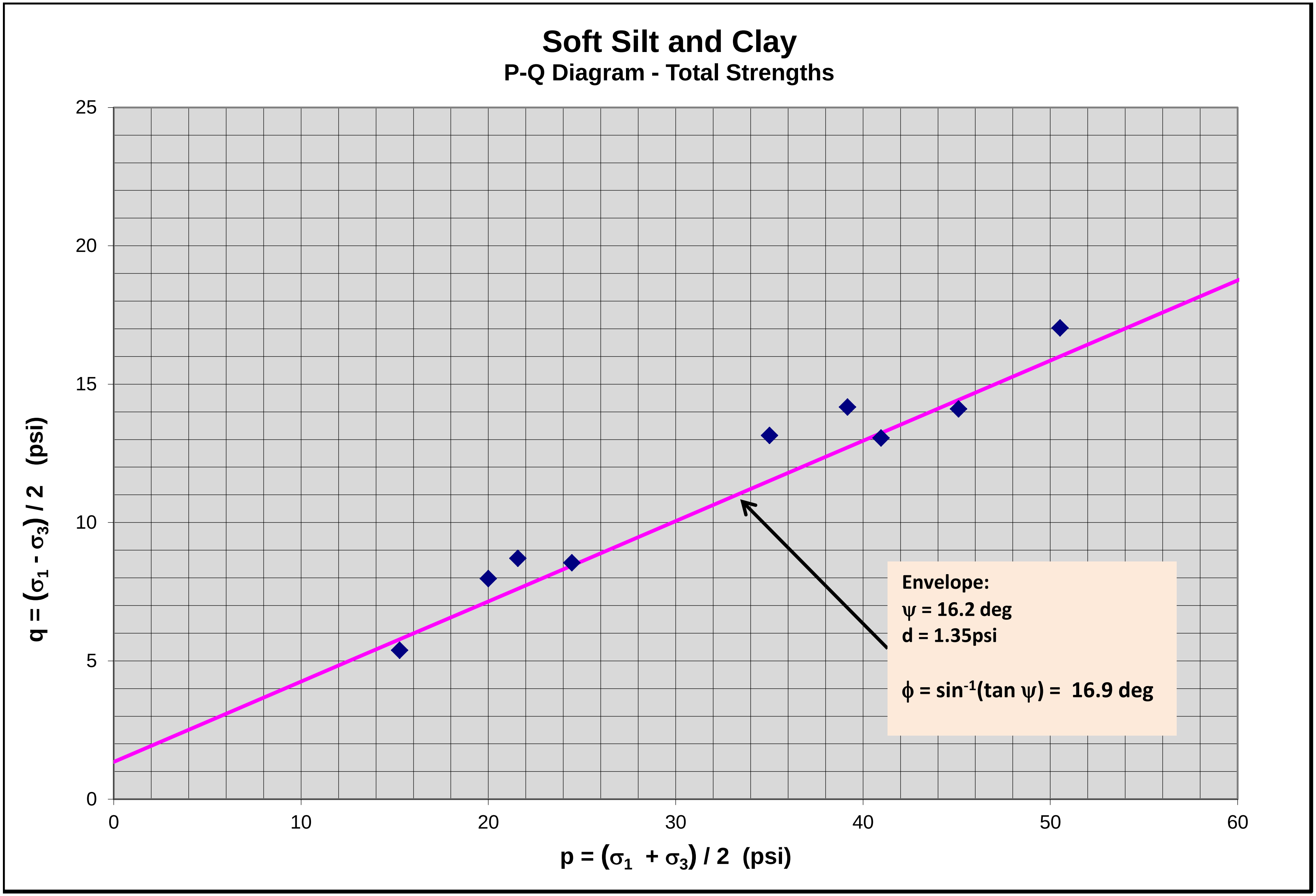
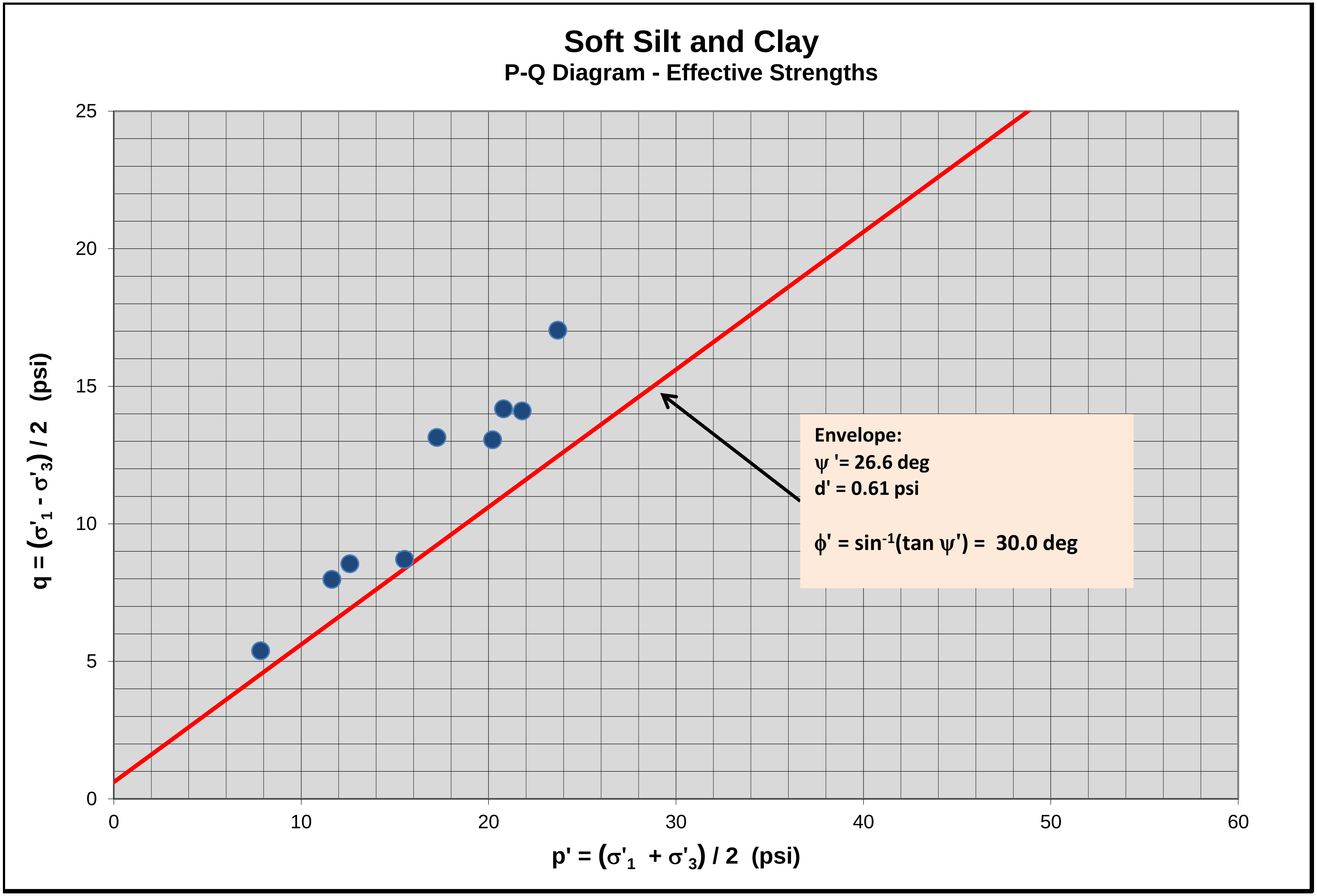


Figure 5 - Compilation of Liquefaction Factors of Safety in Loose Sand Unit


Soft Silt and Clay
P-Q Diagram - Total Strengths
25
20
15
10
5
0
0
10
20
30
40
50
60
q = (σ1 - σ3) / 2 (psi)
p = (σ1 + σ3) / 2 (psi)
Envelope:
ψ = 16.2 deg
d = 1.35psi
ϕ = sin-1(tan ψ) = 16.9 deg


Soft Silt and Clay
P-Q Diagram - Effective Strengths
Envelope:
ψ '= 26.6 deg
d' = 0.61 psi
ϕ' = sin-1(tan ψ') = 30.0 deg
q = (σ'1 - σ'3) / 2 (psi)
p' = (σ'1 + σ'3) / 2 (psi)
0
5
10
15
20
25
0
10
20
30
40
50
60

Appendix C
Estimation of Horizontal Seismic Coefficient, k_h

Estimation of Horizontal Seismic Coefficient, k_h

The horizontal seismic coefficient, k_h, is calculated based on the seismic hazard identified at the site. The seismic coefficient is typically the only variable necessary to be determined to perform the pseudo-static analysis and is used directly in the limit equilibrium analyses in a manner similar to a static analysis. The horizontal seismic coefficient was calculated as follows:

1. Using the seismic unified hazard tool online tool available on the United States Geological Survey website (https://earthquake.usgs.gov/hazards/interactive/), AECOM determined that the peak ground acceleration (PGA) associated with a 2 percent probability of exceedance in 50 years (2,475-year return period) at the subject site (latitude-longitude coordinates of 41.879611°, -83.348419°) would be 0.063g (see **Figure 1**) at the top of competent deep rock.
2. By using a 1.6 amplification multiplier (see **Figure 3)** associated with an assumed **Seismic Site Class D** materials (see **Figures 2** per ASCE/SEI Standard 7-10), the PGA at the ground surface (or base of the dike) would be near 0.106g (0.063g x 1.6 = 0.101g).

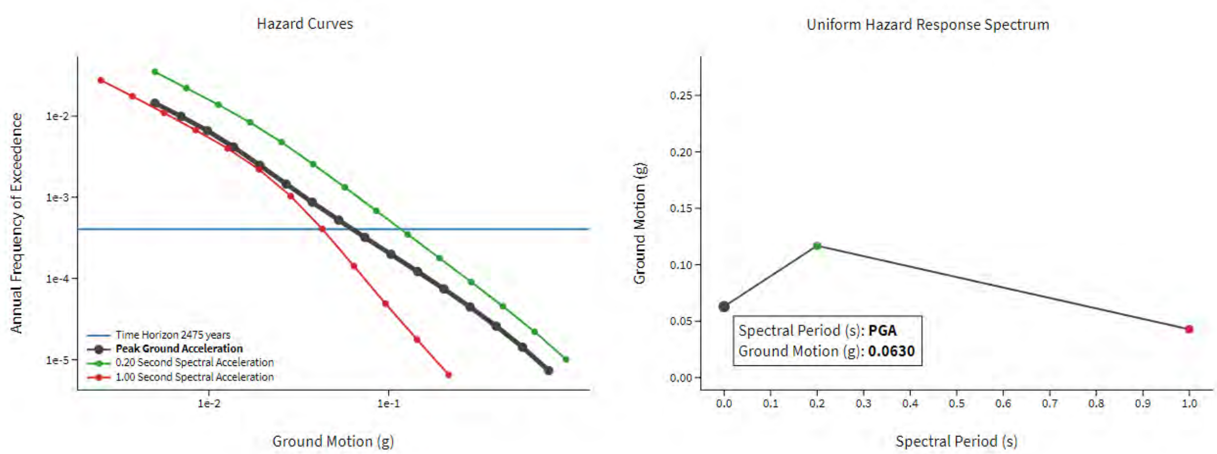


Figure 1. Site-Specific Uniform Hazard Spectrum From USGS.

1615.1.1 Site class definitions. The site shall be classified as one of the site classes defined in Table 1615.1.1. Where the soil shear wave velocity, $\bar{v}_s$, is not known, site class shall be determined, as permitted in Table 1615.1.1, from standard penetration resistance, $\bar{N}$, or from soil undrained shear strength, $\bar{s}_u$, calculated in accordance with Section 1615.1.5. Where site-specific data are not available to a depth of 100 feet (30 480 mm), appropriate soil properties are permitted to be estimated by the registered design professional preparing the soils report based on known geologic conditions.

When the soil properties are not known in sufficient detail to determine the site class, Site Class D shall be used unless the building official determines that Site Class E or F soil is likely to be present at the site.

1615.1.2 Site coefficients and adjusted maximum considered earthquake spectral response acceleration parameters. The maximum considered earthquake spectral response acceleration for short periods, S_{MS}, and at 1-second period, S_{M1}, adjusted for site class effects, shall be determined by Equations 16-38 and 16-39, respectively:

$$S_{MS} = F_a S_s \qquad \textbf{(Equation 16-38)}$$

$$S_{M1} = F_v S_1 \qquad \textbf{(Equation 16-39)}$$

where:

F_a = Site coefficient defined in Table 1615.1.2(1).

F_v = Site coefficient defined in Table 1615.1.2(2).

S_S = The mapped spectral accelerations for short periods as determined in Section 1615.1.

S_1 = The mapped spectral accelerations for a 1-second period as determined in Section 1615.1.

1615.1.3 Design spectral response acceleration parameters. Five-percent damped design spectral response acceleration at short periods, S_{DS}, and at 1-second period, S_{D1}, shall be determined from Equations 16-40 and 16-41, respectively:

$$S_{DS} = \frac{2}{3} S_{MS} \qquad \textbf{(Equation 16-40)}$$

$$S_{D1} = \frac{2}{3} S_{M1} \qquad \textbf{(Equation 16-41)}$$

where:

S_{MS} = The maximum considered earthquake spectral response accelerations for short period as determined in Section 1615.1.2.

S_{M1} = The maximum considered earthquake spectral response accelerations for 1-second period as determined in Section 1615.1.2.

1615.1.4 General procedure response spectrum. The general design response spectrum curve shall be developed as indicated in Figure 1615.1.4 and as follows:

1. For periods less than or equal to T_O, the design spectral response acceleration, S_a, shall be determined by Equation 16-42.
2. For periods greater than or equal to T_O and less than or equal to T_S, the design spectral response acceleration, S_a, shall be taken equal to S_{DS}.

TABLE 1615.1.1
SITE CLASS DEFINITIONS

SITE CLASS	SOIL PROFILE NAME	AVERAGE PROPERTIES IN TOP 100 feet, AS PER SECTION 1615.1.5		
		Soil shear wave velocity, $\bar{v}_s$, (ft/s)	Standard penetration resistance, $\bar{N}$	Soil undrained shear strength, $\bar{s}_u$, (psf)
A	Hard rock	$\bar{v}_s > 5{,}000$	N/A	N/A
B	Rock	$2{,}500 < \bar{v}_s \le 5{,}000$	N/A	N/A
C	Very dense soil and soft rock	$1{,}200 < \bar{v}_s \le 2{,}500$	$\bar{N} > 50$	$\bar{s}_u \ge 2{,}000$
D	Stiff soil profile	$600 \le \bar{v}_s \le 1{,}200$	$15 \le \bar{N} \le 50$	$1{,}000 \le \bar{s}_u \le 2{,}000$
E	Soft soil profile	$\bar{v}_s < 600$	$\bar{N} < 15$	$\bar{s}_u < 1{,}000$
E	—	Any profile with more than 10 feet of soil having the following characteristics: 1. Plasticity index $PI > 20$, 2. Moisture content $w \ge 40\%$, and 3. Undrained shear strength $\bar{s}_u < 500$ psf		
F	—	Any profile containing soils having one or more of the following characteristics: 1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils. 2. Peats and/or highly organic clays ($H >$ 10 feet of peat and/or highly organic clay where H = thickness of soil) 3. Very high plasticity clays ($H > 25$ feet with plasticity index $PI > 75$) 4. Very thick soft/medium stiff clays ($H > 120$ feet)		

For SI: 1 foot = 304.8 mm, 1 square foot = 0.0929 m², 1 pound per square foot = 0.0479 kPa. N/A = Not applicable

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Figure 2. Site Classes from IBC 2003.

Table 11.8-1 Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.1	PGA = 0.2	PGA = 0.3	PGA = 0.4	PGA ≥ 0.5
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7				

Note: Use straight-line interpolation for intermediate values of PGA.

Figure 3. Site Class Amplification Factors for Calculating $(PGA)_{design}$ from $(PGA)_{rock}$, NEHRP2009

DTE Monroe - Area 15 Safety Factor Analysis

Cross Section A-A'

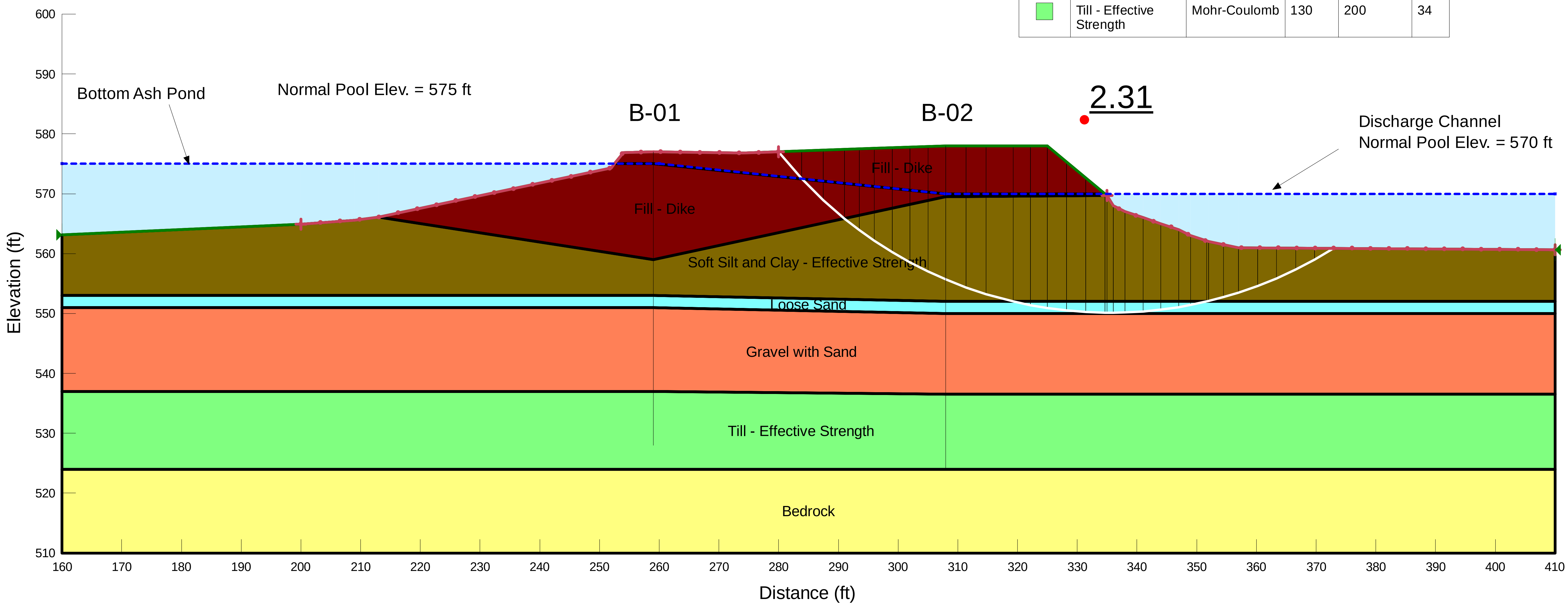
Alignment Reference Point (Local Plant Datum):
N 5861.23 ft
E 7227.31 ft

Alignment Azimuth Angle (Local Plant Datum):
270.0 degrees

Static Analysis
Circular Slip Surfaces

Material Properties

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Bedrock	Bedrock (Impenetrable)			
	Fill - Dike	Mohr-Coulomb	120	0	30
	Gravel with Sand	Mohr-Coulomb	125	0	33
	Loose Sand	Mohr-Coulomb	120	0	28
	Soft Silt and Clay - Effective Strength	Mohr-Coulomb	100	100	30
	Till - Effective Strength	Mohr-Coulomb	130	200	34



DTE Monroe - Area 15 Safety Factor Analysis

Cross Section A-A'

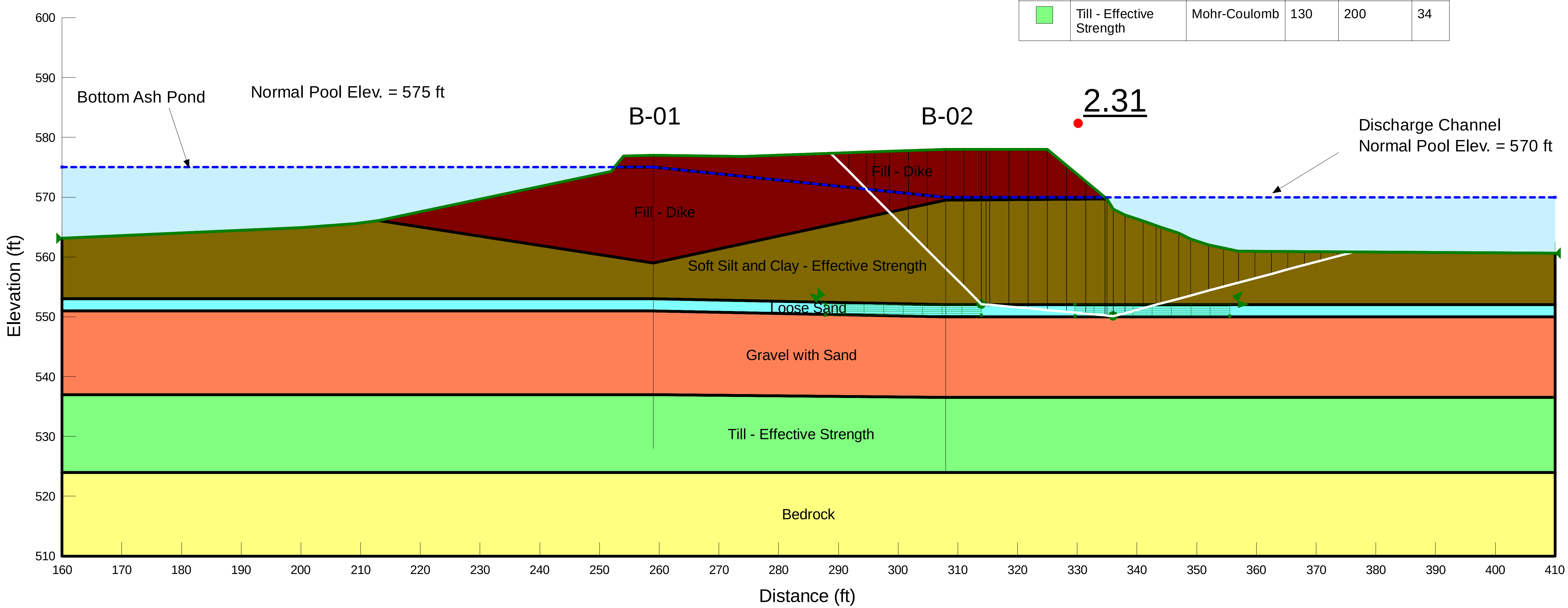
Alignment Reference Point (Local Plant Datum):
N 5861.23 ft
E 7227.31 ft

Alignment Azimuth Angle (Local Plant Datum):
270.0 degrees

Static Analysis
Block Failure

Material Properties

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Bedrock	Bedrock (Impenetrable)			
	Fill - Dike	Mohr-Coulomb	120	0	30
	Gravel with Sand	Mohr-Coulomb	125	0	33
	Loose Sand	Mohr-Coulomb	120	0	28
	Soft Silt and Clay - Effective Strength	Mohr-Coulomb	100	100	30
	Till - Effective Strength	Mohr-Coulomb	130	200	34



DTE Monroe - Area 15 Safety Factor Analysis

Cross Section A-A'

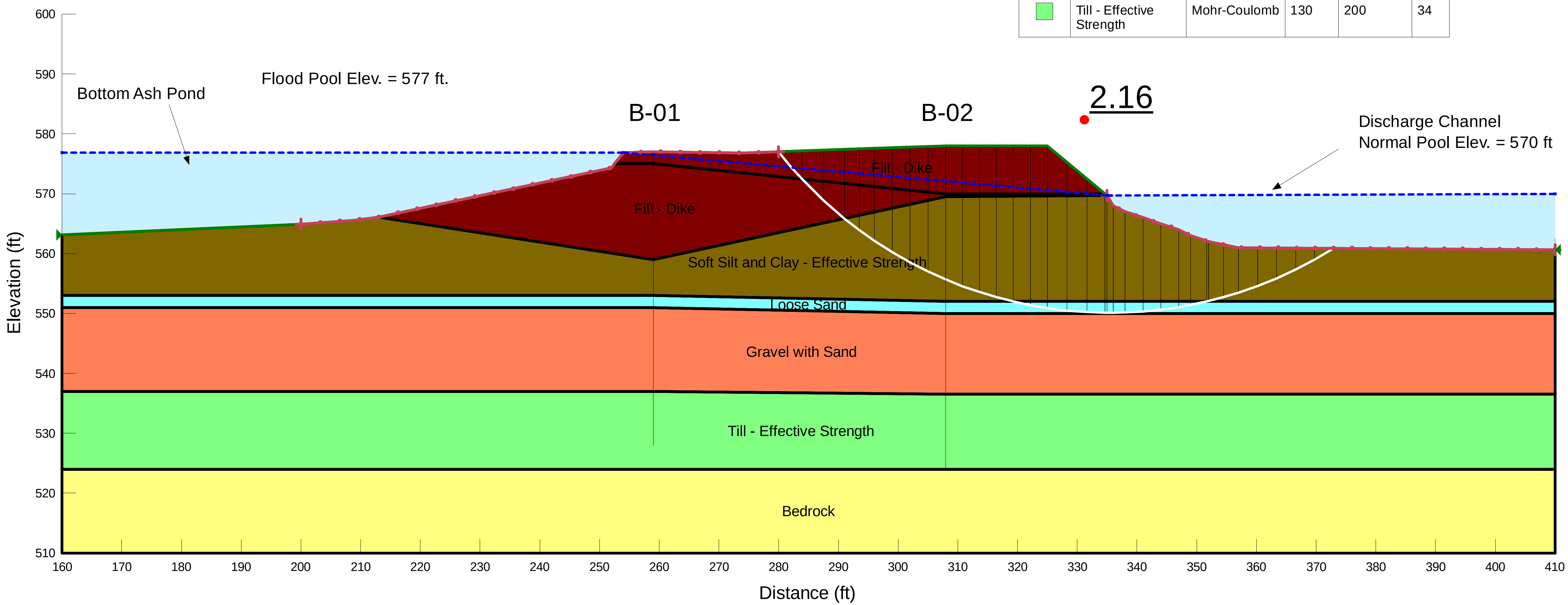
Alignment Reference Point (Local Plant Datum):
N 5861.23 ft
E 7227.31 ft

Alignment Azimuth Angle (Local Plant Datum):
270.0 degrees

Static Analysis with Flood Pool Surcharge
Circular Slip Surfaces

Material Properties

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Bedrock	Bedrock (Impenetrable)			
	Fill - Dike	Mohr-Coulomb	120	0	30
	Gravel with Sand	Mohr-Coulomb	125	0	33
	Loose Sand	Mohr-Coulomb	120	0	28
	Soft Silt and Clay - Effective Strength	Mohr-Coulomb	100	100	30
	Till - Effective Strength	Mohr-Coulomb	130	200	34



DTE Monroe - Area 15 Safety Factor Analysis

Cross Section A-A'

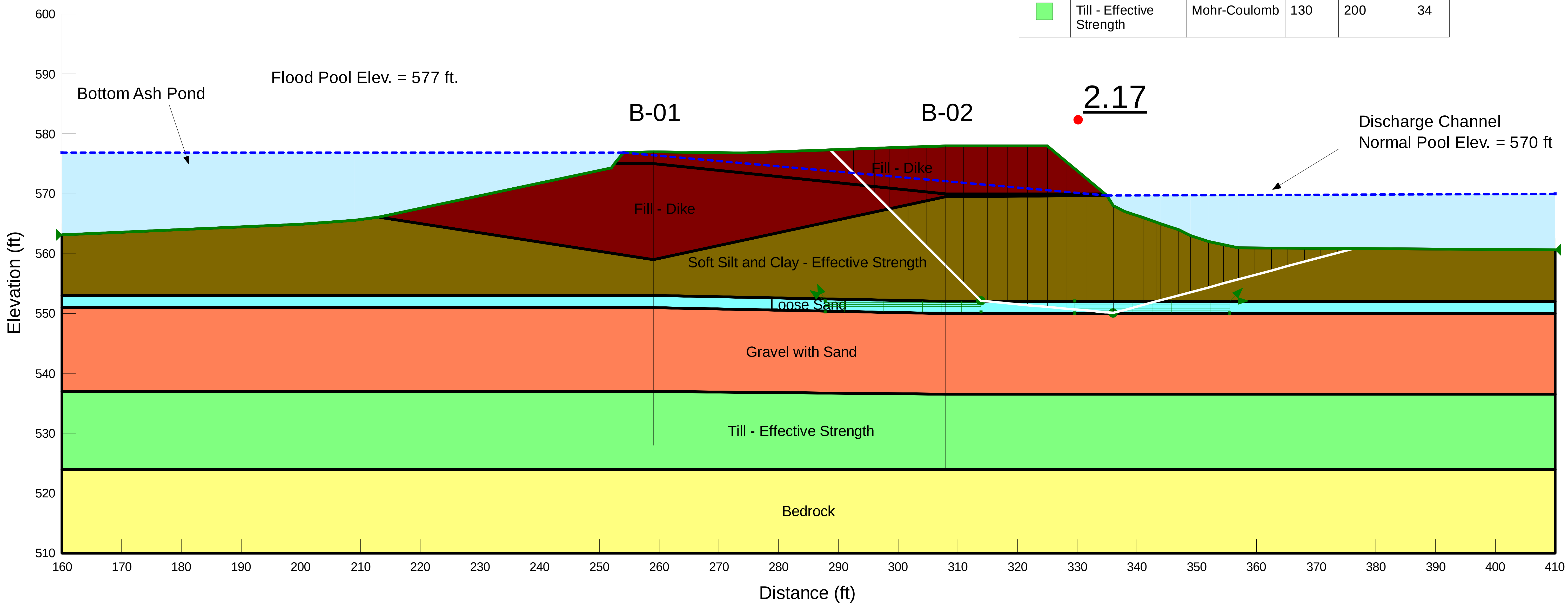
Alignment Reference Point (Local Plant Datum):
N 5861.23 ft
E 7227.31 ft

Alignment Azimuth Angle (Local Plant Datum):
270.0 degrees

Static Analysis with Flood Pool Surcharge
Block Failure

Material Properties

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Bedrock	Bedrock (Impenetrable)			
	Fill - Dike	Mohr-Coulomb	120	0	30
	Gravel with Sand	Mohr-Coulomb	125	0	33
	Loose Sand	Mohr-Coulomb	120	0	28
	Soft Silt and Clay - Effective Strength	Mohr-Coulomb	100	100	30
	Till - Effective Strength	Mohr-Coulomb	130	200	34



DTE Monroe - Area 15 Safety Factor Analysis

Cross Section A-A'

Alignment Reference Point (Local Plant Datum):
N 5861.23 ft
E 7227.31 ft

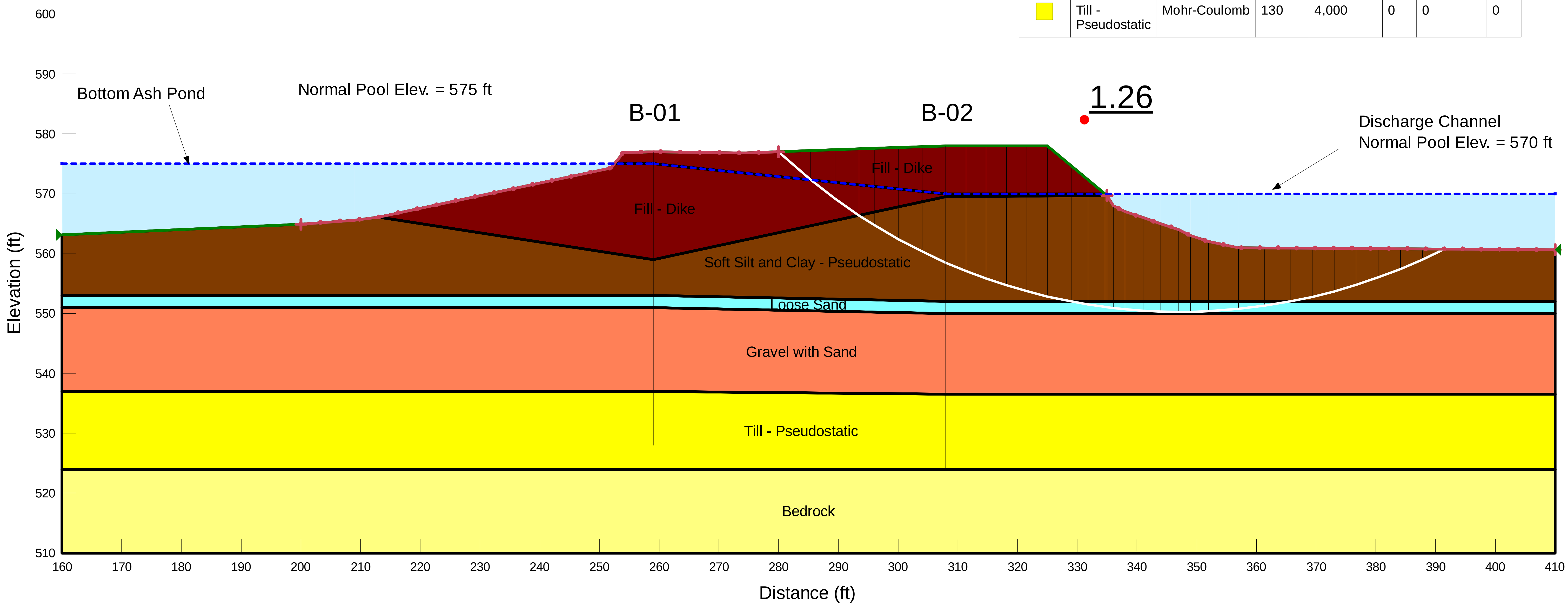
Alignment Azimuth Angle (Local Plant Datum):
270.0 degrees

Pseudostatic (Seismic) Analysis
Circular Slip Surfaces

Pseudostatic Analysis
Seismic Coefficient = 0.11g

Material Properties

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Cohesion R (psf)	Phi R (°)
	Bedrock	Bedrock (Impenetrable)					
	Fill - Dike	Mohr-Coulomb	120	0	30	0	0
	Gravel with Sand	Mohr-Coulomb	125	0	33	0	0
	Loose Sand	Mohr-Coulomb	120	0	28	0	0
	Soft Silt and Clay - Pseudostatic	Mohr-Coulomb	100	220	16	0	0
	Till - Pseudostatic	Mohr-Coulomb	130	4,000	0	0	0



DTE Monroe - Area 15 Safety Factor Analysis

Cross Section A-A'

Alignment Reference Point (Local Plant Datum):
N 5861.23 ft
E 7227.31 ft

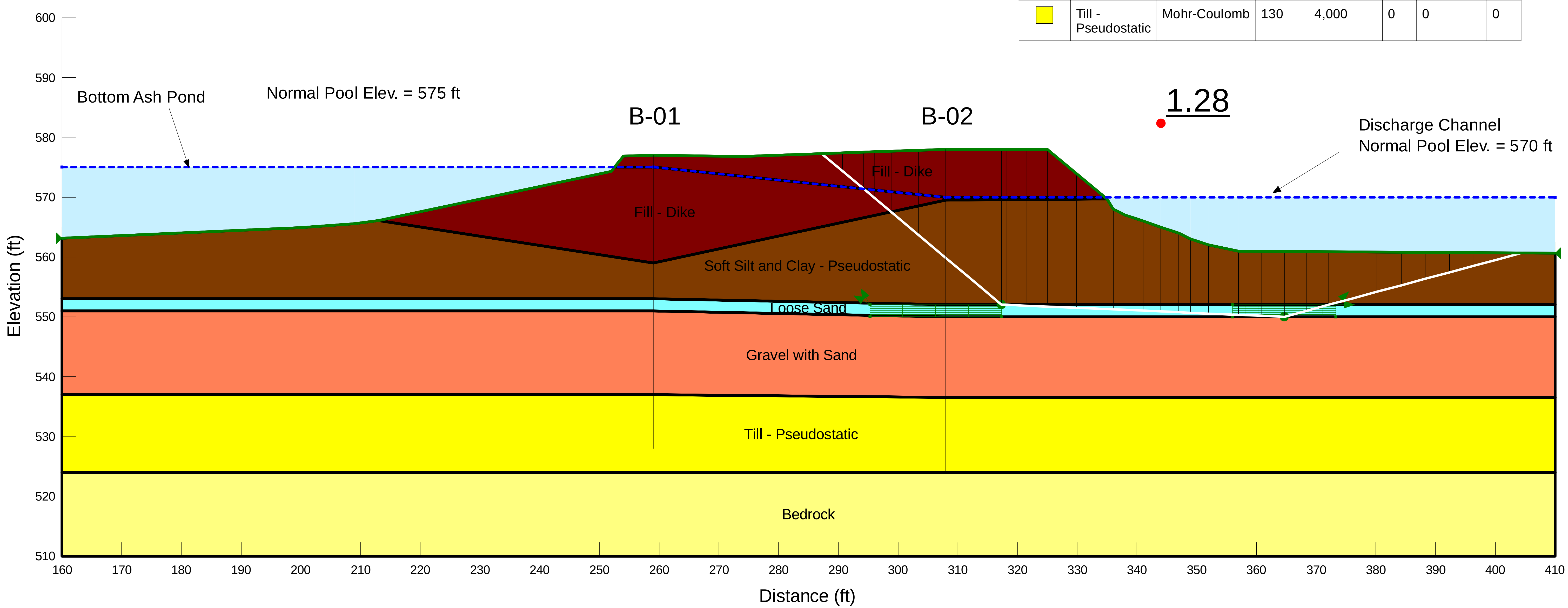
Alignment Azimuth Angle (Local Plant Datum):
270.0 degrees

Pseudostatic (Seismic) Analysis
Block Failure

Pseudostatic Analysis
Seismic Coefficient = 0.11g

Material Properties

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Cohesion R (psf)	Phi R (°)
	Bedrock	Bedrock (Impenetrable)					
	Fill - Dike	Mohr-Coulomb	120	0	30	0	0
	Gravel with Sand	Mohr-Coulomb	125	0	33	0	0
	Loose Sand	Mohr-Coulomb	120	0	28	0	0
	Soft Silt and Clay - Pseudostatic	Mohr-Coulomb	100	220	16	0	0
	Till - Pseudostatic	Mohr-Coulomb	130	4,000	0	0	0



DTE Monroe - Area 15 Safety Factor Analysis

Cross Section A-A'

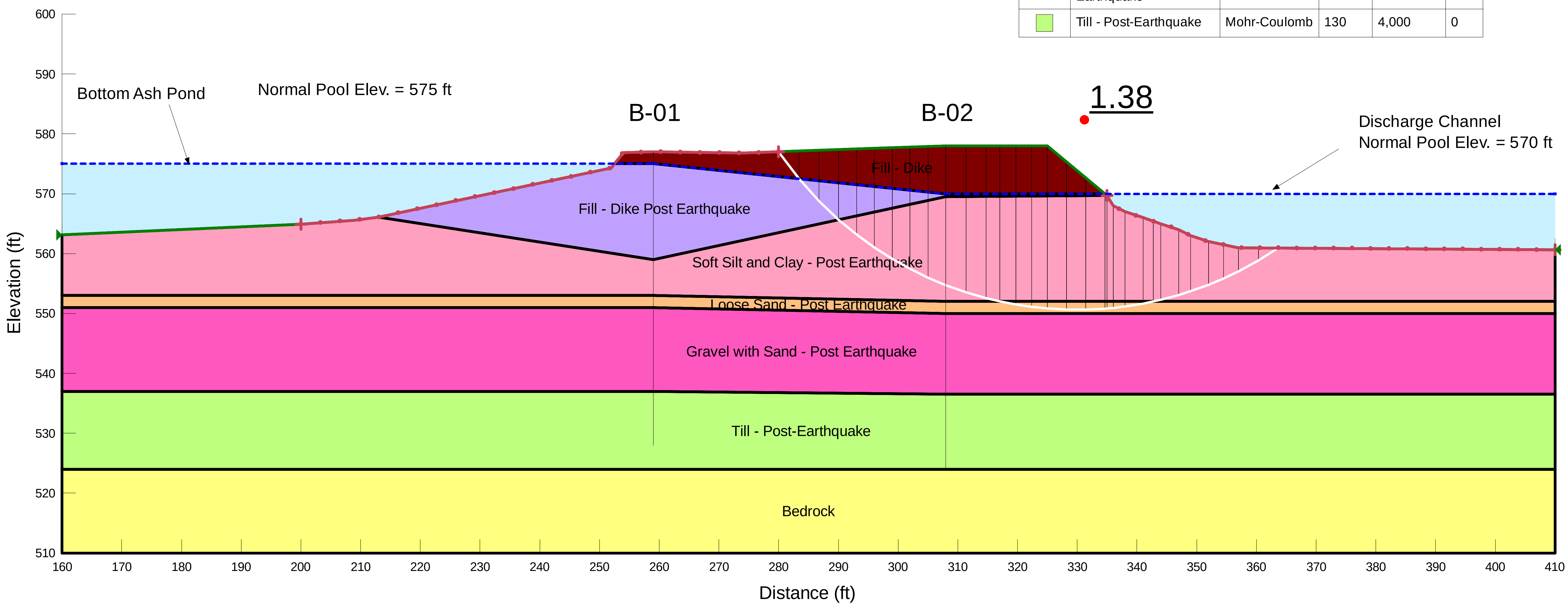
Alignment Reference Point (Local Plant Datum):
N 5861.23 ft
E 7227.31 ft

Alignment Azimuth Angle (Local Plant Datum):
270.0 degrees

Post-Earthquake (Post-Liquefaction) Analysis
Circular Slip Surfaces

Material Properties

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Bedrock	Bedrock (Impenetrable)			
	Fill - Dike	Mohr-Coulomb	120	0	30
	Fill - Dike Post Earthquake	Mohr-Coulomb	120	0	25
	Gravel with Sand - Post Earthquake	Mohr-Coulomb	125	0	33
	Loose Sand - Post Earthquake	Mohr-Coulomb	120	250	0
	Soft Silt and Clay - Post Earthquake	Mohr-Coulomb	100	176	13.2
	Till - Post-Earthquake	Mohr-Coulomb	130	4,000	0



DTE Monroe - Area 15 Safety Factor Analysis

Cross Section A-A'

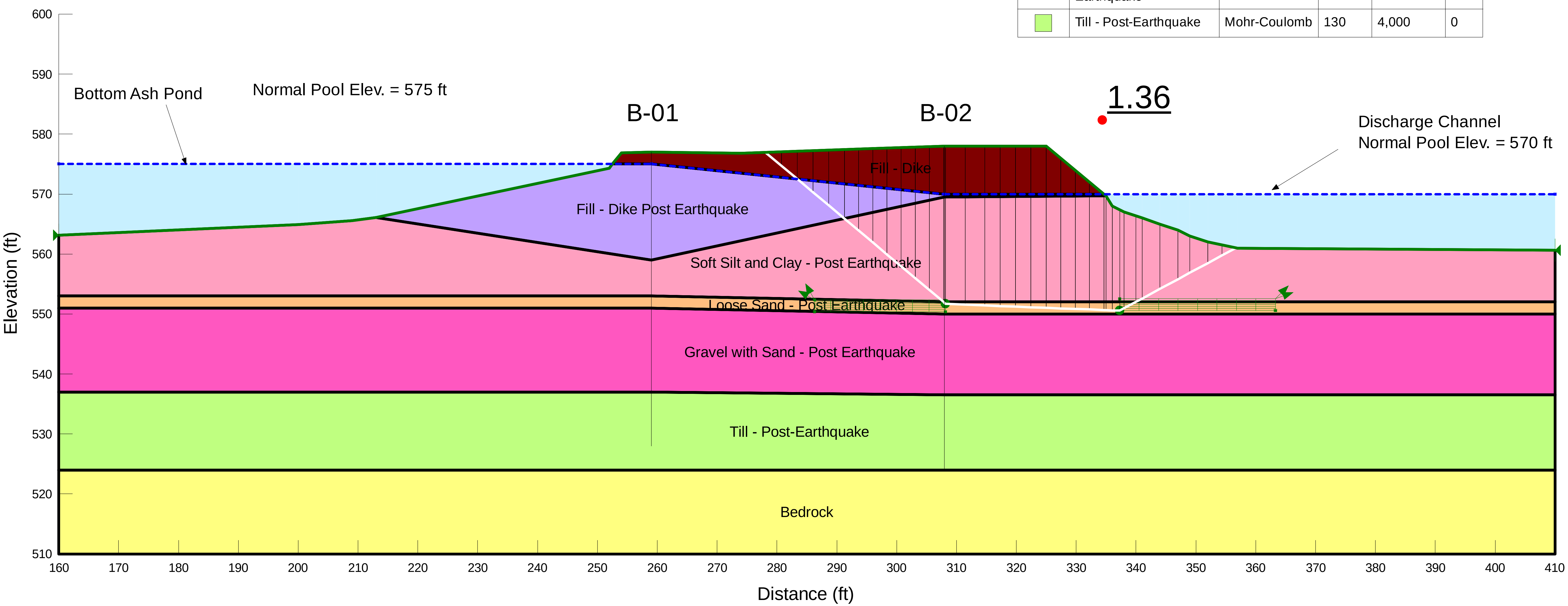
Alignment Reference Point (Local Plant Datum):
N 5861.23 ft
E 7227.31 ft

Alignment Azimuth Angle (Local Plant Datum):
270.0 degrees

Post-Earthquake (Post-Liquefaction) Analysis
Block Failure

Material Properties

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Bedrock	Bedrock (Impenetrable)			
	Fill - Dike	Mohr-Coulomb	120	0	30
	Fill - Dike Post Earthquake	Mohr-Coulomb	120	0	25
	Gravel with Sand - Post Earthquake	Mohr-Coulomb	125	0	33
	Loose Sand - Post Earthquake	Mohr-Coulomb	120	250	0
	Soft Silt and Clay - Post Earthquake	Mohr-Coulomb	100	176	13.2
	Till - Post-Earthquake	Mohr-Coulomb	130	4,000	0



DTE Monroe - Area 15 Safety Factor Analysis

Cross Section B-B'

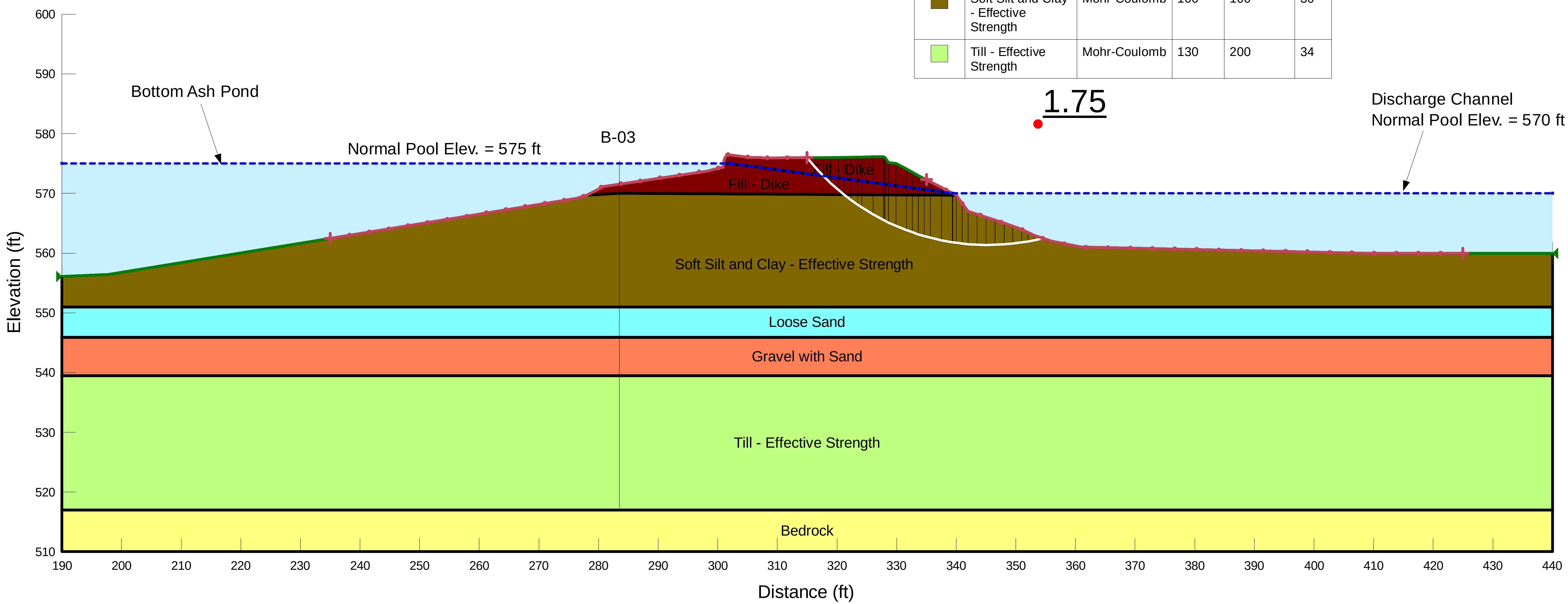
Alignment Reference Point (Local Plant Datum):
N 5254.53 ft
E 7525.40 ft

Azimuth Angle (Local Plant Datum):
237.7 degrees

Static Analysis
Circular Slip Surfaces

Material Properties

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Bedrock	Bedrock (Impenetrable)			
	Fill - Dike	Mohr-Coulomb	120	0	30
	Gravel with Sand	Mohr-Coulomb	125	0	33
	Loose Sand	Mohr-Coulomb	120	0	28
	Soft Silt and Clay - Effective Strength	Mohr-Coulomb	100	100	30
	Till - Effective Strength	Mohr-Coulomb	130	200	34



DTE Monroe - Area 15 Safety Factor Analysis

Cross Section B-B'

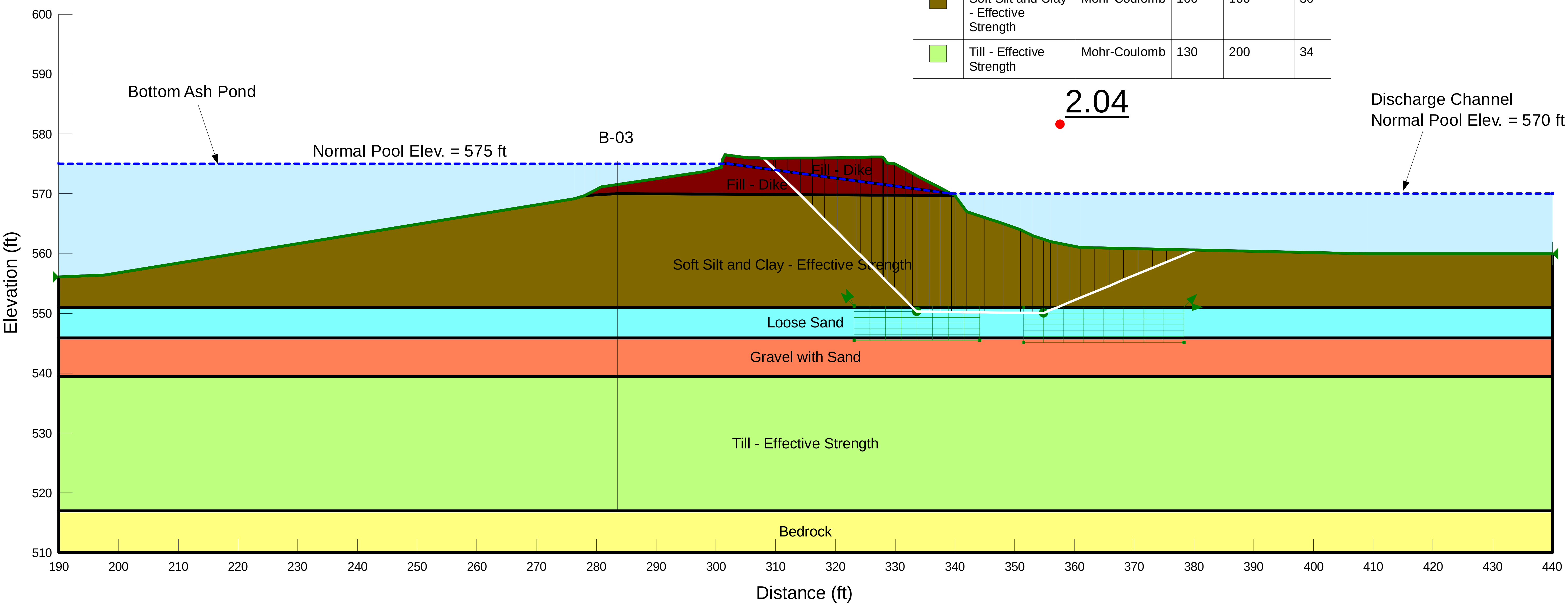
Alignment Reference Point (Local Plant Datum):
N 5254.53 ft
E 7525.40 ft

Azimuth Angle (Local Plant Datum):
237.7 degrees

Static Analysis
Block Failure

Material Properties

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Bedrock	Bedrock (Impenetrable)			
	Fill - Dike	Mohr-Coulomb	120	0	30
	Gravel with Sand	Mohr-Coulomb	125	0	33
	Loose Sand	Mohr-Coulomb	120	0	28
	Soft Silt and Clay - Effective Strength	Mohr-Coulomb	100	100	30
	Till - Effective Strength	Mohr-Coulomb	130	200	34



DTE Monroe - Area 15 Safety Factor Analysis

Cross Section B-B'

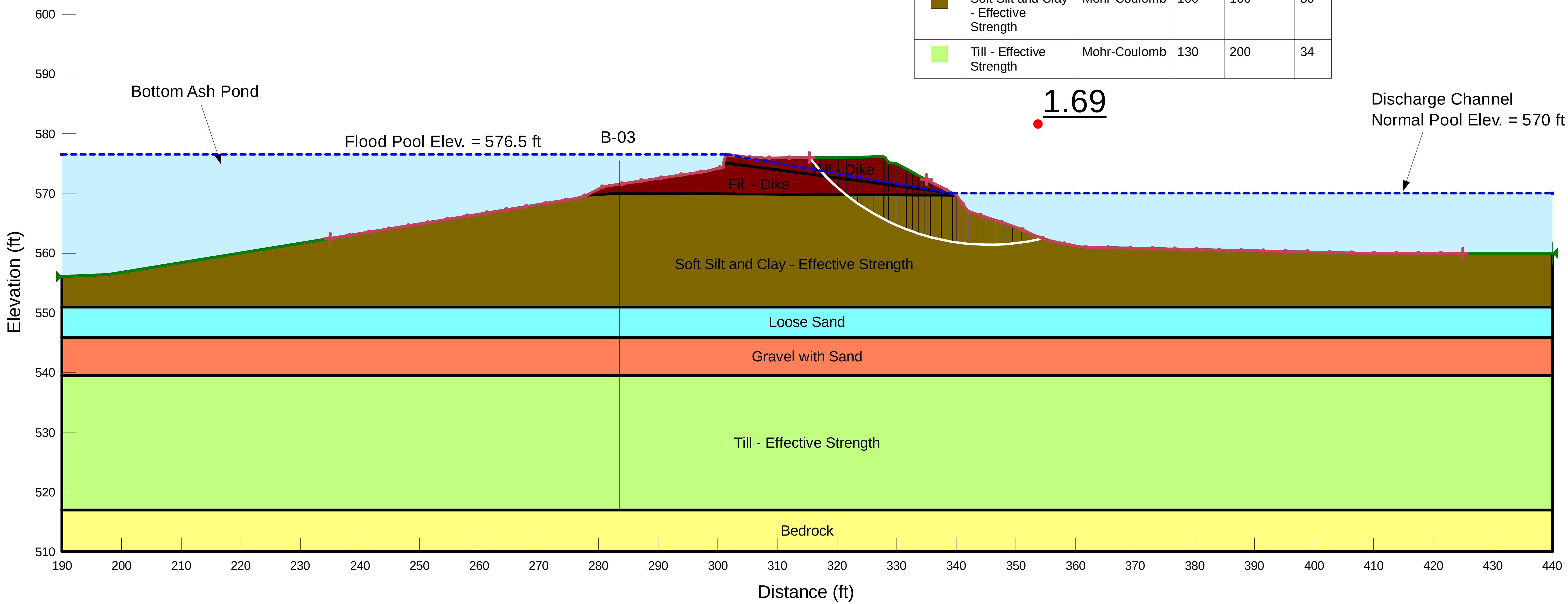
Alignment Reference Point (Local Plant Datum):
N 5254.53 ft
E 7525.40 ft

Azimuth Angle (Local Plant Datum):
237.7 degrees

Static Analysis with Flood Pool Surcharge
Circular Slip Surfaces

Material Properties

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Bedrock	Bedrock (Impenetrable)			
	Fill - Dike	Mohr-Coulomb	120	0	30
	Gravel with Sand	Mohr-Coulomb	125	0	33
	Loose Sand	Mohr-Coulomb	120	0	28
	Soft Silt and Clay - Effective Strength	Mohr-Coulomb	100	100	30
	Till - Effective Strength	Mohr-Coulomb	130	200	34



DTE Monroe - Area 15 Safety Factor Analysis

Cross Section B-B'

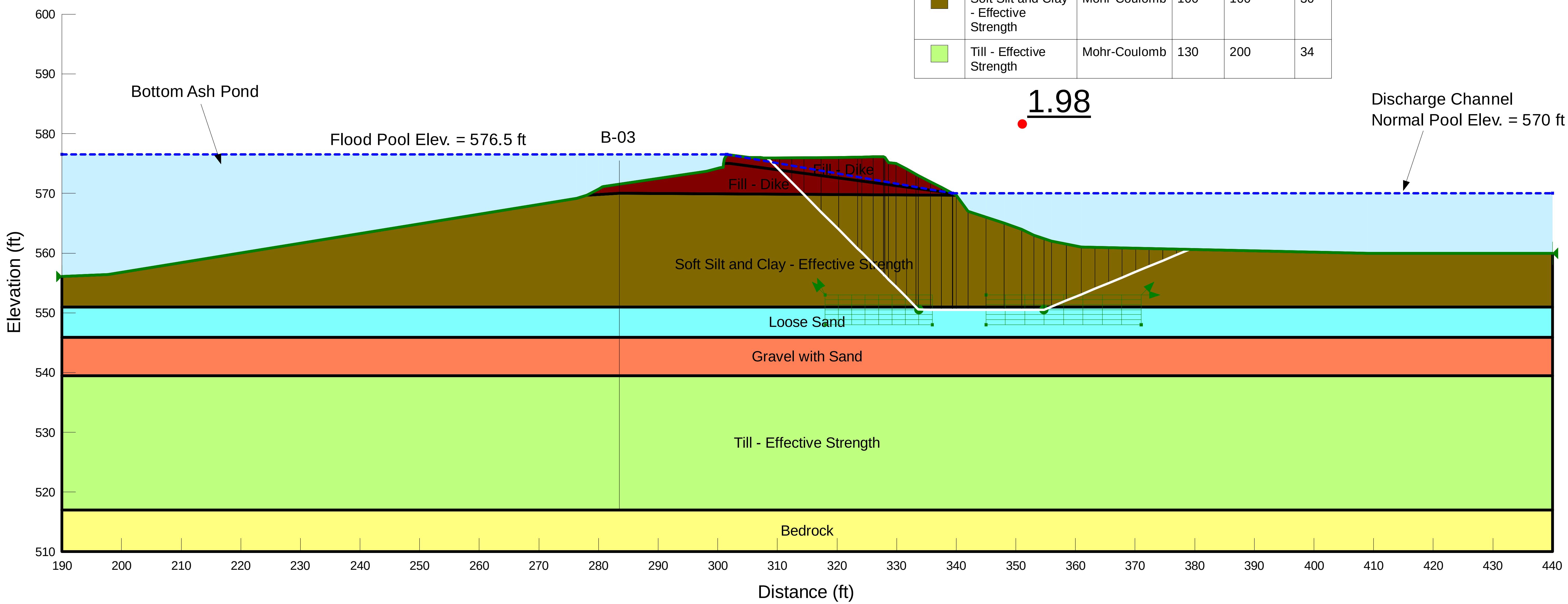
Alignment Reference Point (Local Plant Datum):
N 5254.53 ft
E 7525.40 ft

Azimuth Angle (Local Plant Datum):
237.7 degrees

Static Analysis with Flood Pool Surcharge
Block Failure

Material Properties

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Bedrock	Bedrock (Impenetrable)			
	Fill - Dike	Mohr-Coulomb	120	0	30
	Gravel with Sand	Mohr-Coulomb	125	0	33
	Loose Sand	Mohr-Coulomb	120	0	28
	Soft Silt and Clay - Effective Strength	Mohr-Coulomb	100	100	30
	Till - Effective Strength	Mohr-Coulomb	130	200	34



DTE Monroe - Area 15 Safety Factor Analysis

Cross Section B-B'

Alignment Reference Point (Local Plant Datum):
N 5254.53 ft
E 7525.40 ft

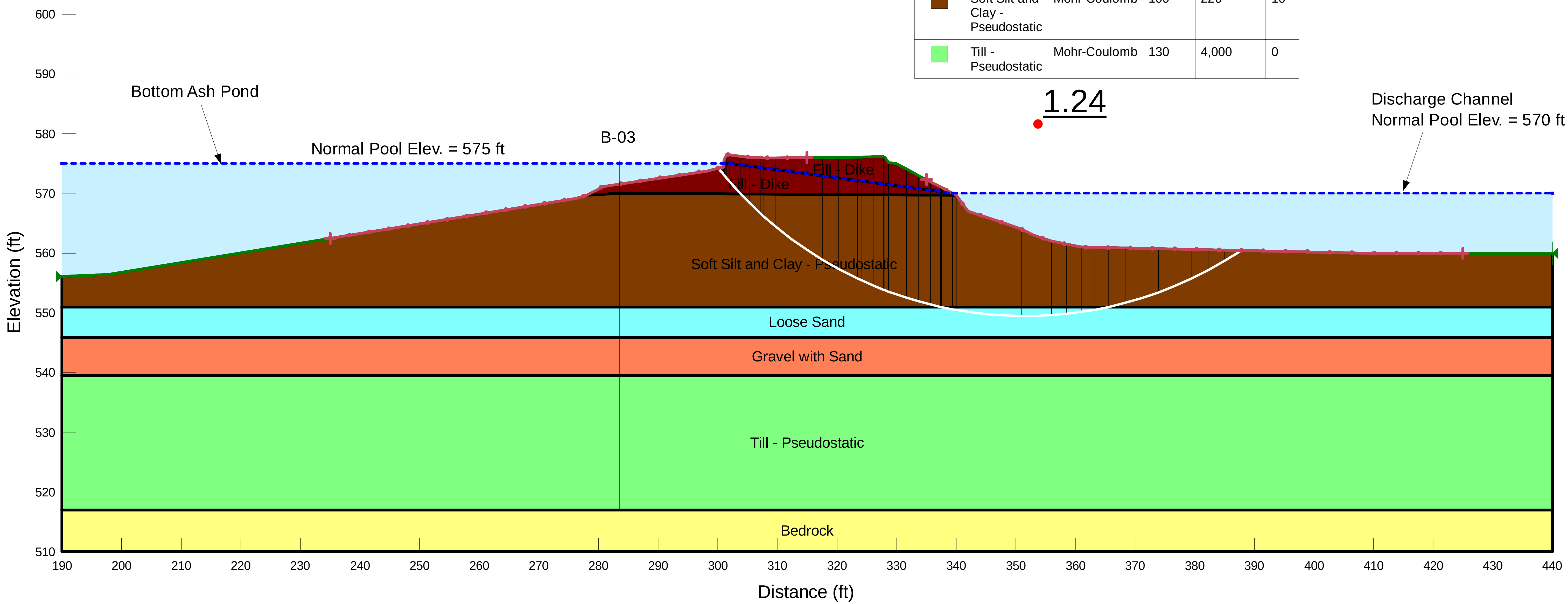
Azimuth Angle (Local Plant Datum):
237.7 degrees

Pseudostatic (Seismic) Analysis
Circular Slip Surfaces

Pseudostatic Analysis
Seismic Coefficient = 0.11g

Material Properties

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Bedrock	Bedrock (Impenetrable)			
	Fill - Dike	Mohr-Coulomb	120	0	30
	Gravel with Sand	Mohr-Coulomb	125	0	33
	Loose Sand	Mohr-Coulomb	120	0	28
	Soft Silt and Clay - Pseudostatic	Mohr-Coulomb	100	220	16
	Till - Pseudostatic	Mohr-Coulomb	130	4,000	0



DTE Monroe - Area 15 Safety Factor Analysis

Cross Section B-B'

Alignment Reference Point (Local Plant Datum):
N 5254.53 ft
E 7525.40 ft

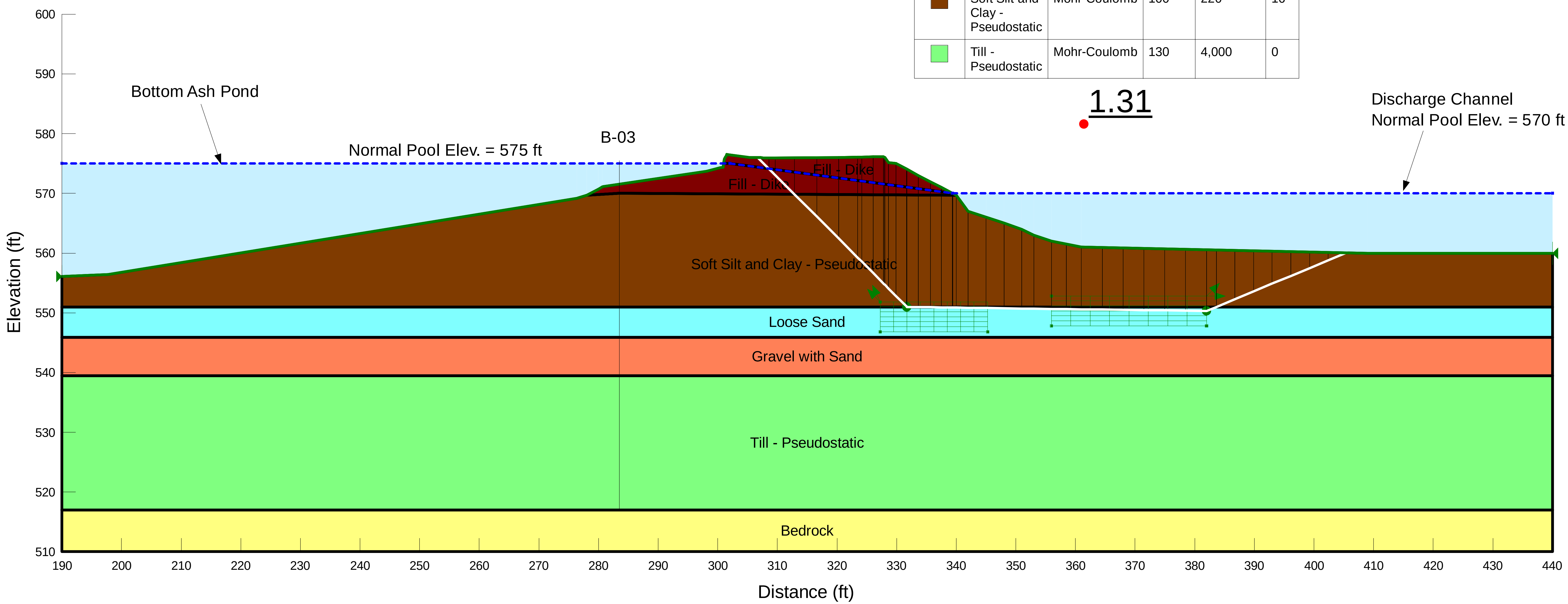
Azimuth Angle (Local Plant Datum):
237.7 degrees

Pseudostatic (Seismic) Analysis
Block Failure

Pseudostatic Analysis
Seismic Coefficient = 0.11g

Material Properties

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Bedrock	Bedrock (Impenetrable)			
	Fill - Dike	Mohr-Coulomb	120	0	30
	Gravel with Sand	Mohr-Coulomb	125	0	33
	Loose Sand	Mohr-Coulomb	120	0	28
	Soft Silt and Clay - Pseudostatic	Mohr-Coulomb	100	220	16
	Till - Pseudostatic	Mohr-Coulomb	130	4,000	0



DTE Monroe - Area 15 Safety Factor Analysis

Cross Section B-B'

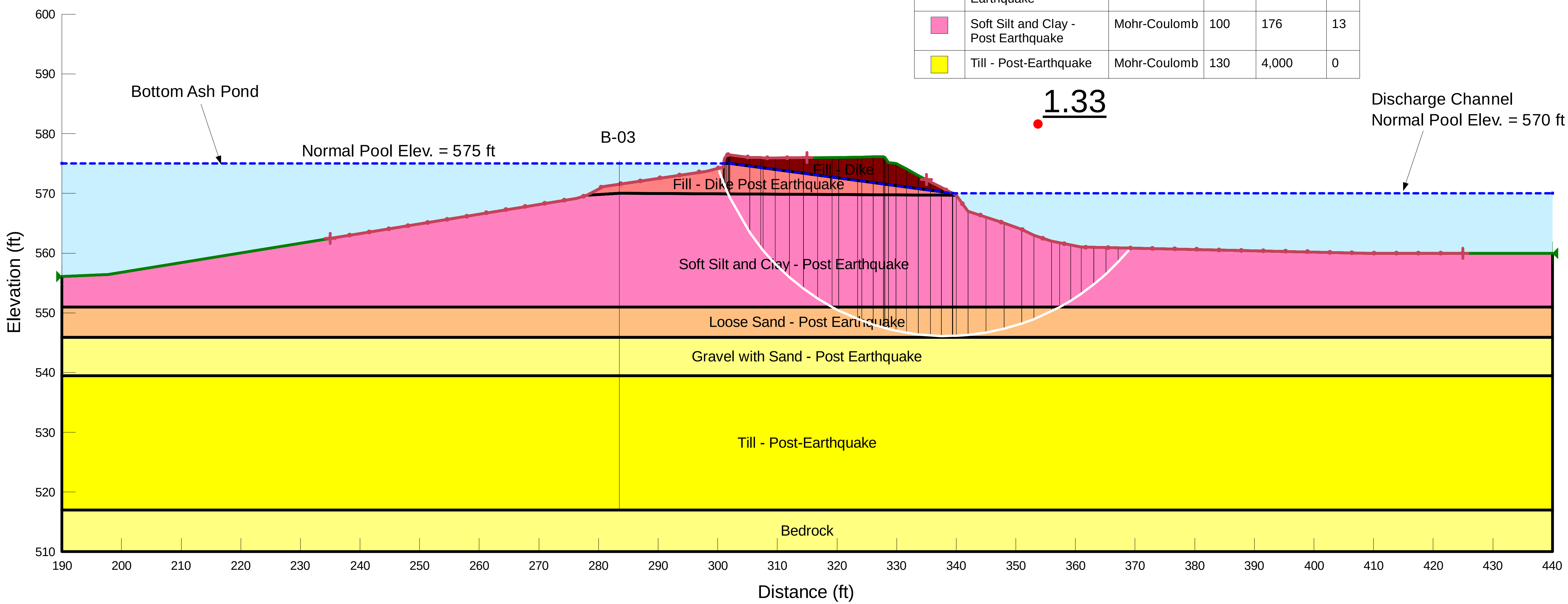
Alignment Reference Point (Local Plant Datum):
N 5254.53 ft
E 7525.40 ft

Azimuth Angle (Local Plant Datum):
237.7 degrees

Post-Earthquake (Post-Liquefaction) Analysis
Circular Slip Surfaces

Material Properties

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Bedrock	Bedrock (Impenetrable)			
	Fill - Dike	Mohr-Coulomb	120	0	30
	Fill - Dike Post Earthquake	Mohr-Coulomb	120	0	25
	Gravel with Sand - Post Earthquake	Mohr-Coulomb	125	0	33
	Loose Sand - Post Earthquake	Mohr-Coulomb	120	250	0
	Soft Silt and Clay - Post Earthquake	Mohr-Coulomb	100	176	13
	Till - Post-Earthquake	Mohr-Coulomb	130	4,000	0



DTE Monroe - Area 15 Safety Factor Analysis

Cross Section B-B'

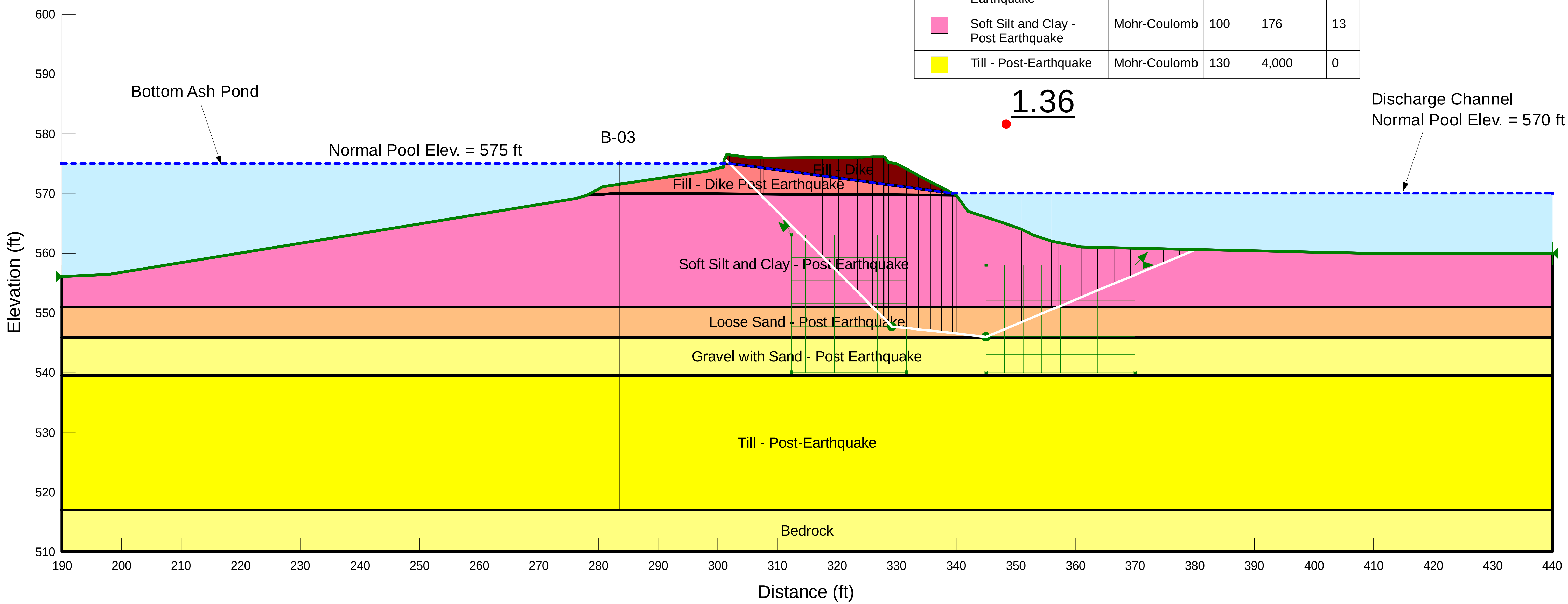
Alignment Reference Point (Local Plant Datum):
N 5254.53 ft
E 7525.40 ft

Azimuth Angle (Local Plant Datum):
237.7 degrees

Post-Earthquake (Post-Liquefaction) Analysis
Block Failure

Material Properties

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Bedrock	Bedrock (Impenetrable)			
	Fill - Dike	Mohr-Coulomb	120	0	30
	Fill - Dike Post Earthquake	Mohr-Coulomb	120	0	25
	Gravel with Sand - Post Earthquake	Mohr-Coulomb	125	0	33
	Loose Sand - Post Earthquake	Mohr-Coulomb	120	250	0
	Soft Silt and Clay - Post Earthquake	Mohr-Coulomb	100	176	13
	Till - Post-Earthquake	Mohr-Coulomb	130	4,000	0



DTE Monroe - Area 15 Safety Factor Analysis

Cross Section C-C'

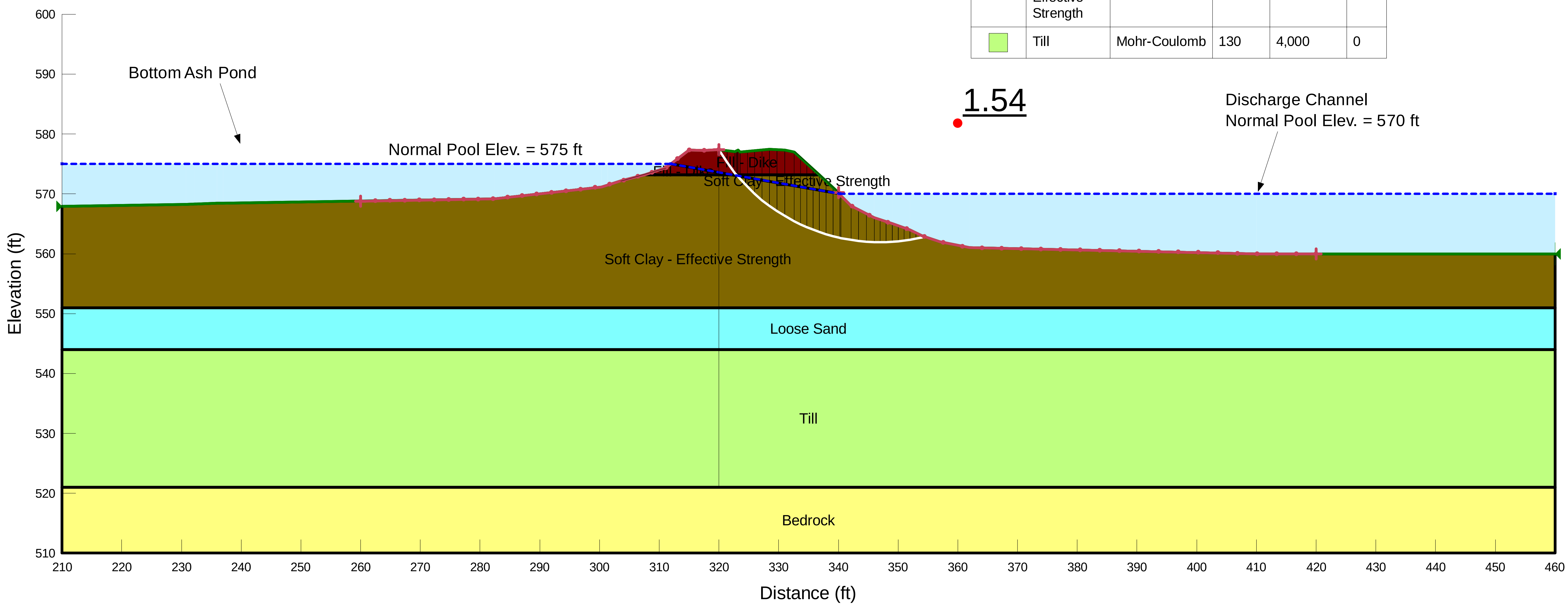
Alignment Reference Point:
N 4602.51 ft
E 7968.22 ft

Alignment Azimuth Angle
237.7 degrees

Static Analysis
Circular Slip Surfaces

Material Properties

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Bedrock	Bedrock (Impenetrable)			
	Fill - Dike	Mohr-Coulomb	120	0	30
	Loose Sand	Mohr-Coulomb	120	0	28
	Soft Clay - Effective Strength	Mohr-Coulomb	100	100	30
	Till	Mohr-Coulomb	130	4,000	0



DTE Monroe - Area 15 Safety Factor Analysis

Cross Section C-C'

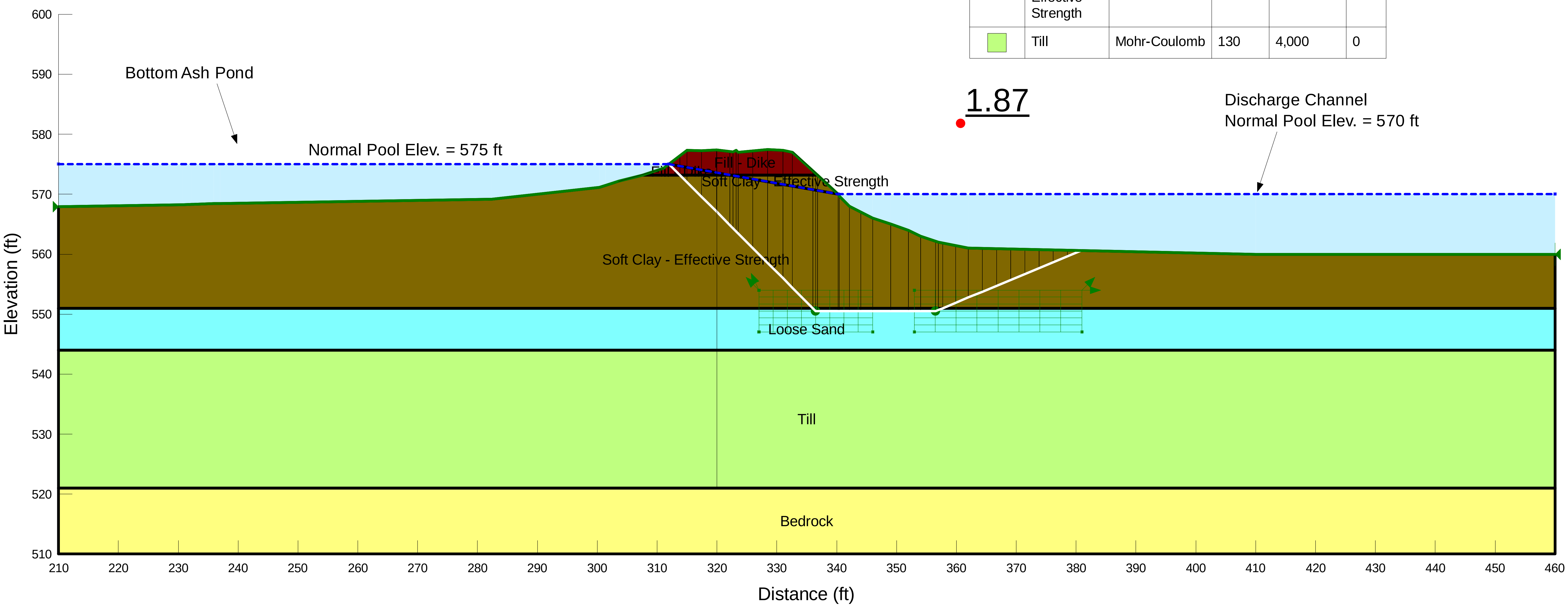
Alignment Reference Point:
N 4602.51 ft
E 7968.22 ft

Alignment Azimuth Angle
237.7 degrees

Static Analysis
Block Failure

Material Properties

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Bedrock	Bedrock (Impenetrable)			
	Fill - Dike	Mohr-Coulomb	120	0	30
	Loose Sand	Mohr-Coulomb	120	0	28
	Soft Clay - Effective Strength	Mohr-Coulomb	100	100	30
	Till	Mohr-Coulomb	130	4,000	0



DTE Monroe - Area 15 Safety Factor Analysis

Cross Section C-C'

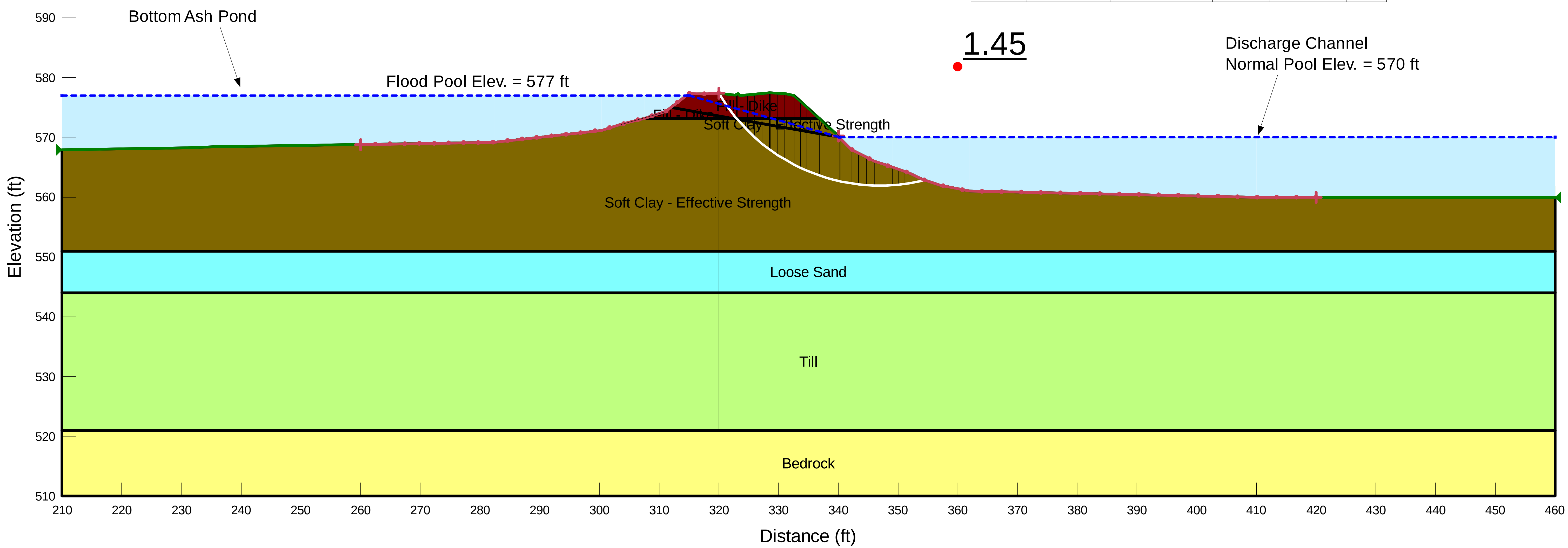
Alignment Reference Point:
N 4602.51 ft
E 7968.22 ft

Alignment Azimuth Angle
237.7 degrees

Static Analysis with Flood Pool Surcharge
Circular Slip Surfaces

Material Properties

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Bedrock	Bedrock (Impenetrable)			
	Fill - Dike	Mohr-Coulomb	120	0	30
	Loose Sand	Mohr-Coulomb	120	0	28
	Soft Clay - Effective Strength	Mohr-Coulomb	100	100	30
	Till	Mohr-Coulomb	130	4,000	0



DTE Monroe - Area 15 Safety Factor Analysis

Cross Section C-C'

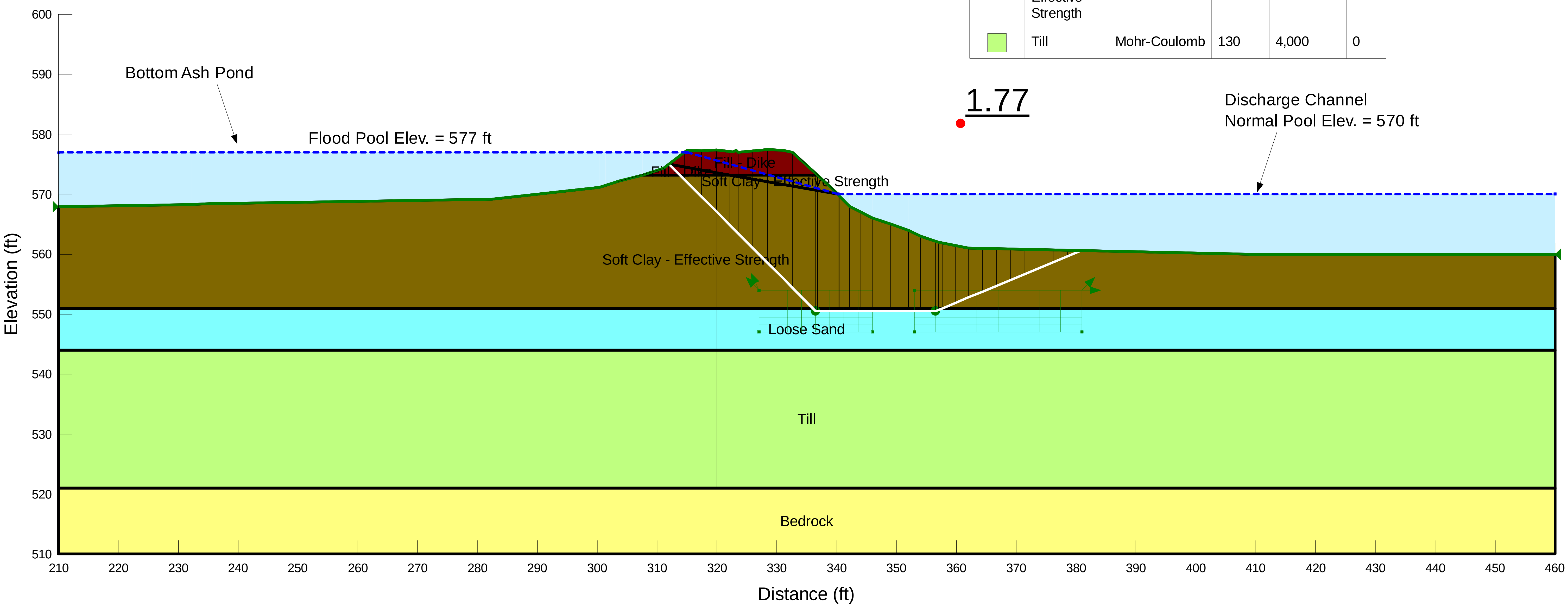
Alignment Reference Point:
N 4602.51 ft
E 7968.22 ft

Alignment Azimuth Angle
237.7 degrees

Static Analysis with Flood Pool Surcharge
Block Failure

Material Properties

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Bedrock	Bedrock (Impenetrable)			
	Fill - Dike	Mohr-Coulomb	120	0	30
	Loose Sand	Mohr-Coulomb	120	0	28
	Soft Clay - Effective Strength	Mohr-Coulomb	100	100	30
	Till	Mohr-Coulomb	130	4,000	0



DTE Monroe - Area 15 Safety Factor Analysis

Cross Section C-C'

Alignment Reference Point:
N 4602.51 ft
E 7968.22 ft

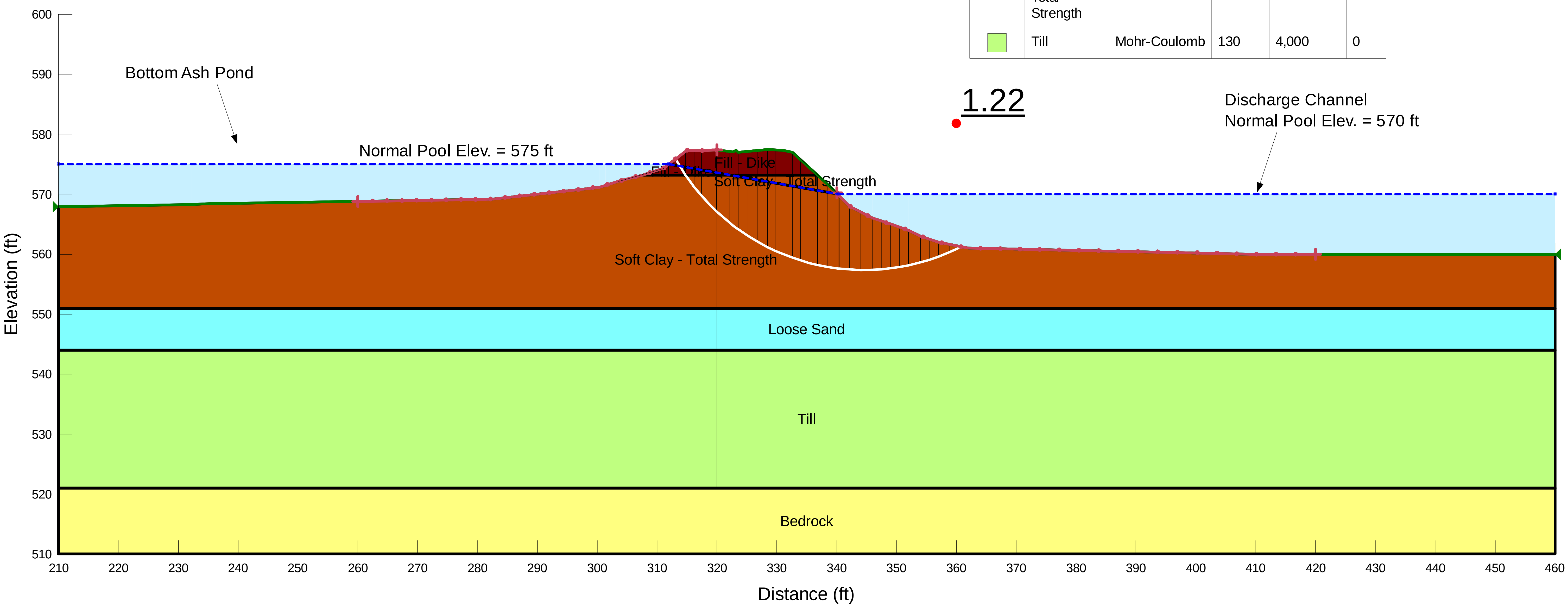
Alignment Azimuth Angle
237.7 degrees

Pseudostatic (Seismic) Analysis
Circular Slip Surfaces

Pseudostatic Analysis
Seismic Coefficient = 0.11g

Material Properties

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Bedrock	Bedrock (Impenetrable)			
	Fill - Dike	Mohr-Coulomb	120	0	30
	Loose Sand	Mohr-Coulomb	120	0	28
	Soft Clay - Total Strength	Mohr-Coulomb	100	220	16
	Till	Mohr-Coulomb	130	4,000	0



DTE Monroe - Area 15 Safety Factor Analysis

Cross Section C-C'

Alignment Reference Point:
N 4602.51 ft
E 7968.22 ft

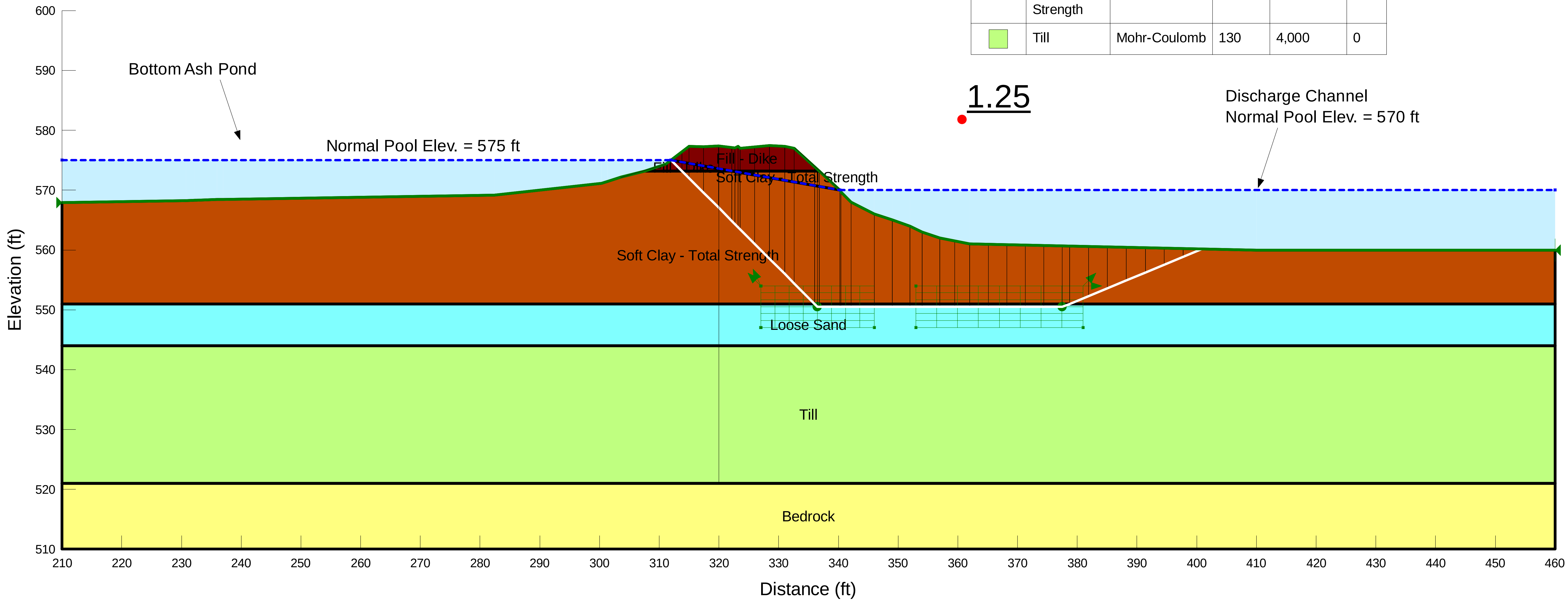
Alignment Azimuth Angle
237.7 degrees

Pseudostatic (Seismic) Analysis
Block Failure

Pseudostatic Analysis
Seismic Coefficient = 0.11g

Material Properties

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Bedrock	Bedrock (Impenetrable)			
	Fill - Dike	Mohr-Coulomb	120	0	30
	Loose Sand	Mohr-Coulomb	120	0	28
	Soft Clay - Total Strength	Mohr-Coulomb	100	220	16
	Till	Mohr-Coulomb	130	4,000	0



DTE Monroe - Area 15 Safety Factor Analysis

Cross Section C-C'

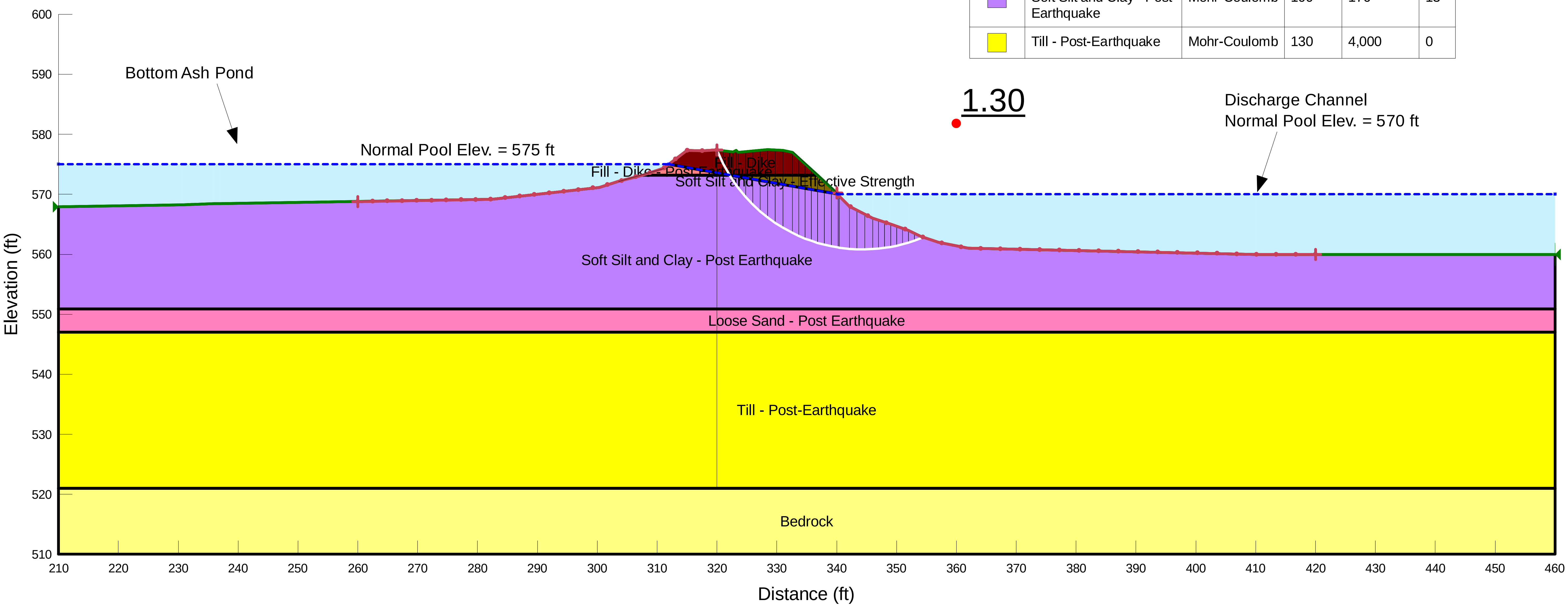
Alignment Reference Point:
N 4602.51 ft
E 7968.22 ft

Alignment Azimuth Angle
237.7 degrees

Post-Earthquake (Post-Liquefaction) Analysis
Circular Slip Surfaces

Material Properties

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Bedrock	Bedrock (Impenetrable)			
	Fill - Dike	Mohr-Coulomb	120	0	30
	Fill - Dike - Post Earthquake	Mohr-Coulomb	120	0	25
	Loose Sand - Post Earthquake	Mohr-Coulomb	120	250	0
	Soft Silt and Clay - Effective Strength	Mohr-Coulomb	100	100	30
	Soft Silt and Clay - Post Earthquake	Mohr-Coulomb	100	176	13
	Till - Post-Earthquake	Mohr-Coulomb	130	4,000	0



DTE Monroe - Area 15 Safety Factor Analysis

Cross Section C-C'

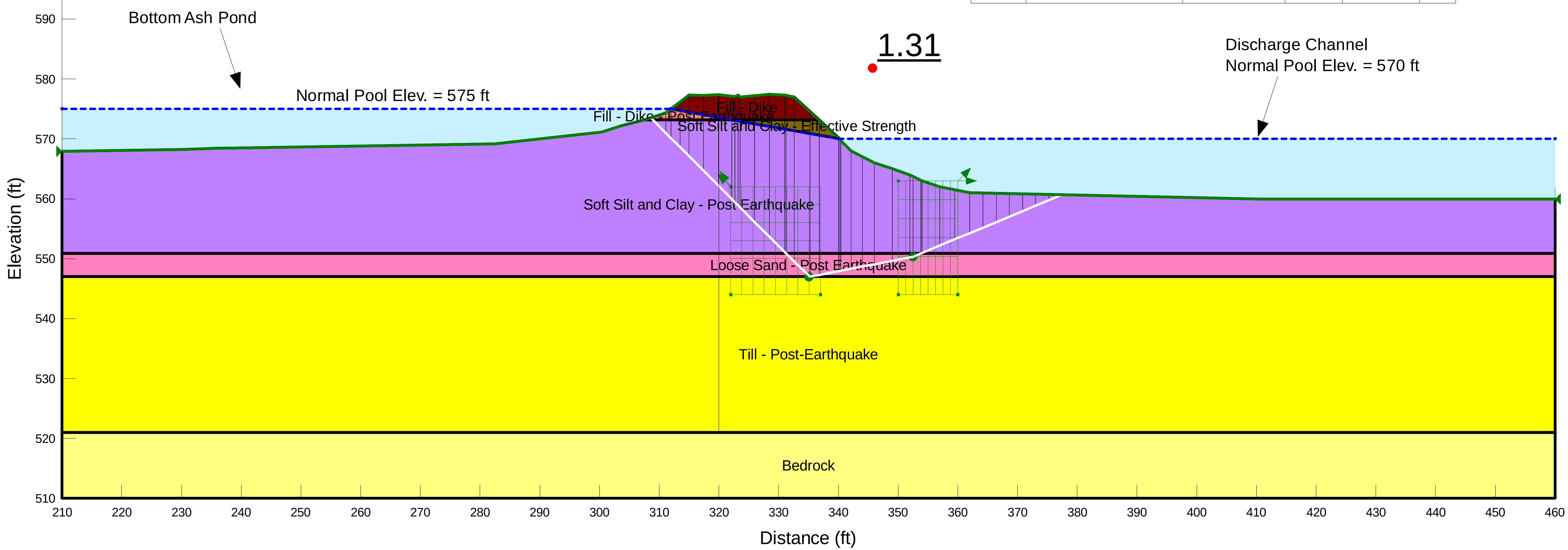
Alignment Reference Point:
N 4602.51 ft
E 7968.22 ft

Alignment Azimuth Angle
237.7 degrees

Post-Earthquake (Post-Liquefaction) Analysis
Block Failure

Material Properties

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Bedrock	Bedrock (Impenetrable)			
	Fill - Dike	Mohr-Coulomb	120	0	30
	Fill - Dike - Post Earthquake	Mohr-Coulomb	120	0	25
	Loose Sand - Post Earthquake	Mohr-Coulomb	120	250	0
	Soft Silt and Clay - Effective Strength	Mohr-Coulomb	100	100	30
	Soft Silt and Clay - Post Earthquake	Mohr-Coulomb	100	176	13
	Till - Post-Earthquake	Mohr-Coulomb	130	4,000	0



DTE Monroe - Area 15 Safety Factor Analysis

Cross Section D-D'

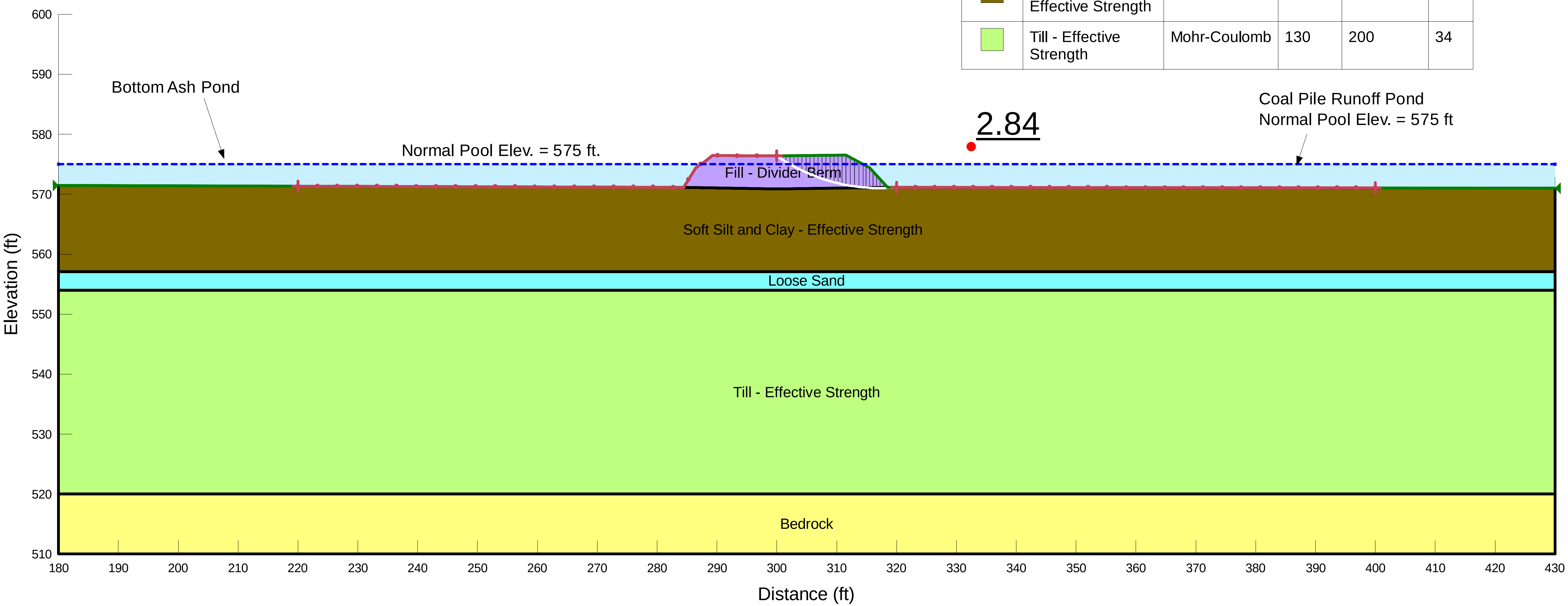
Alignment Reference Point:
N 4741.27 ft
E 9193.09 ft

Alignment Azimuth Angle
198.7 degrees

Static Analysis
Circular Slip Surfaces

Material Properties

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Bedrock	Bedrock (Impenetrable)			
	Fill - Divider Berm	Mohr-Coulomb	130	0	33
	Loose Sand	Mohr-Coulomb	120	0	28
	Soft Silt and Clay - Effective Strength	Mohr-Coulomb	100	100	30
	Till - Effective Strength	Mohr-Coulomb	130	200	34



DTE Monroe - Area 15 Safety Factor Analysis

Cross Section D-D'

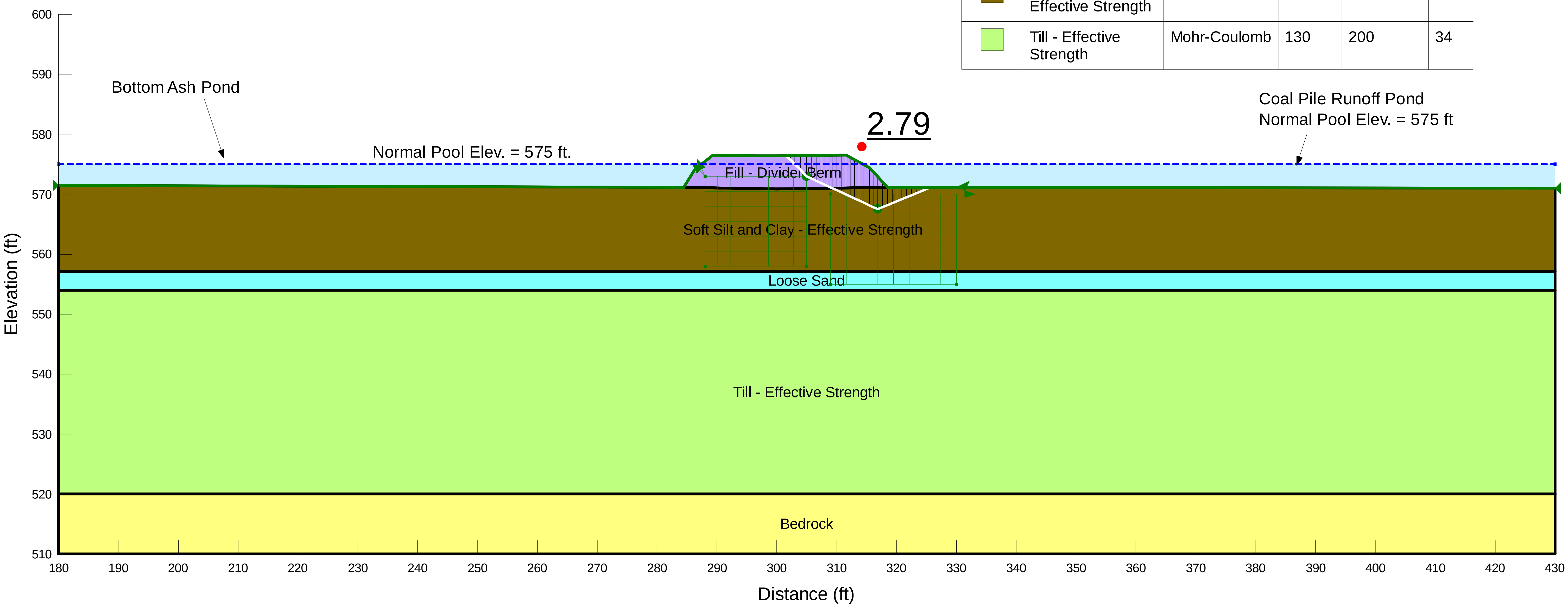
Alignment Reference Point:
N 4741.27 ft
E 9193.09 ft

Alignment Azimuth Angle
198.7 degrees

Static Analysis
Block Failure

Material Properties

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Bedrock	Bedrock (Impenetrable)			
	Fill - Divider Berm	Mohr-Coulomb	130	0	33
	Loose Sand	Mohr-Coulomb	120	0	28
	Soft Silt and Clay - Effective Strength	Mohr-Coulomb	100	100	30
	Till - Effective Strength	Mohr-Coulomb	130	200	34



DTE Monroe - Area 15 Safety Factor Analysis

Cross Section D-D'

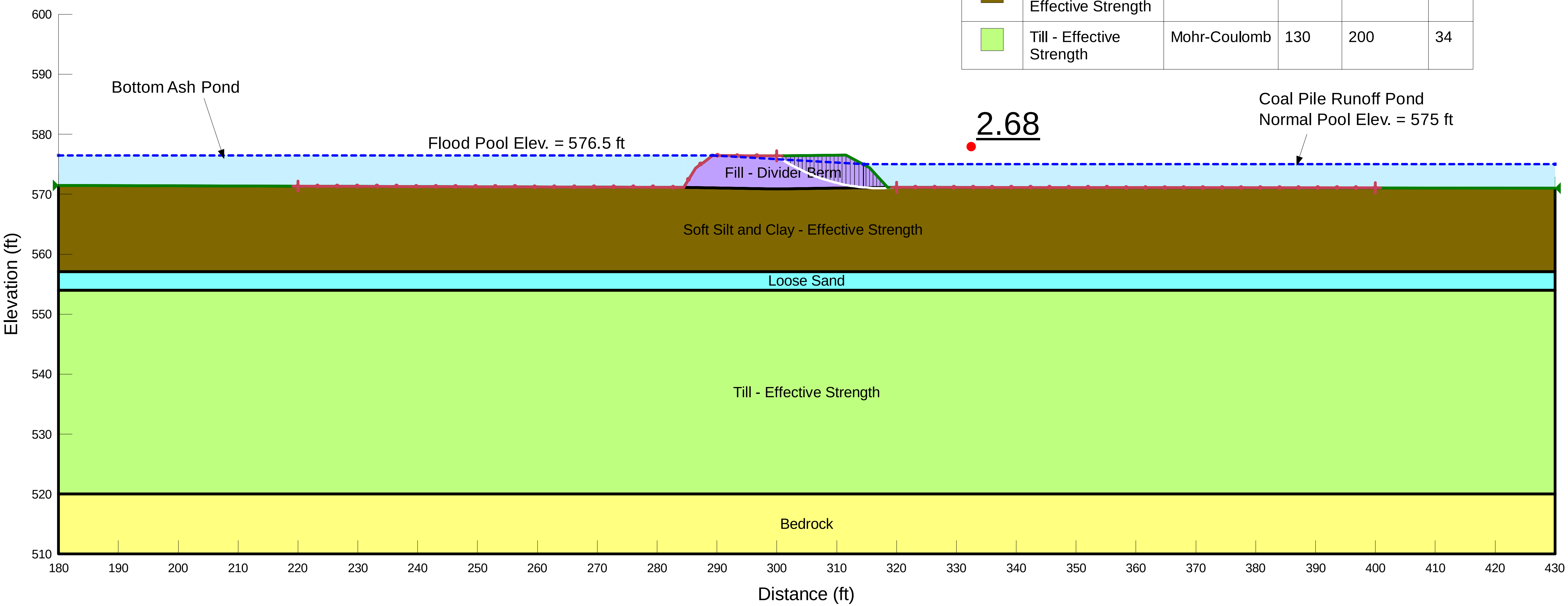
Alignment Reference Point:
N 4741.27 ft
E 9193.09 ft

Alignment Azimuth Angle
198.7 degrees

Static Analysis with Flood Pool Surcharge
Circular Slip Surfaces

Material Properties

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Bedrock	Bedrock (Impenetrable)			
	Fill - Divider Berm	Mohr-Coulomb	130	0	33
	Loose Sand	Mohr-Coulomb	120	0	28
	Soft Silt and Clay - Effective Strength	Mohr-Coulomb	100	100	30
	Till - Effective Strength	Mohr-Coulomb	130	200	34



DTE Monroe - Area 15 Safety Factor Analysis

Cross Section D-D'

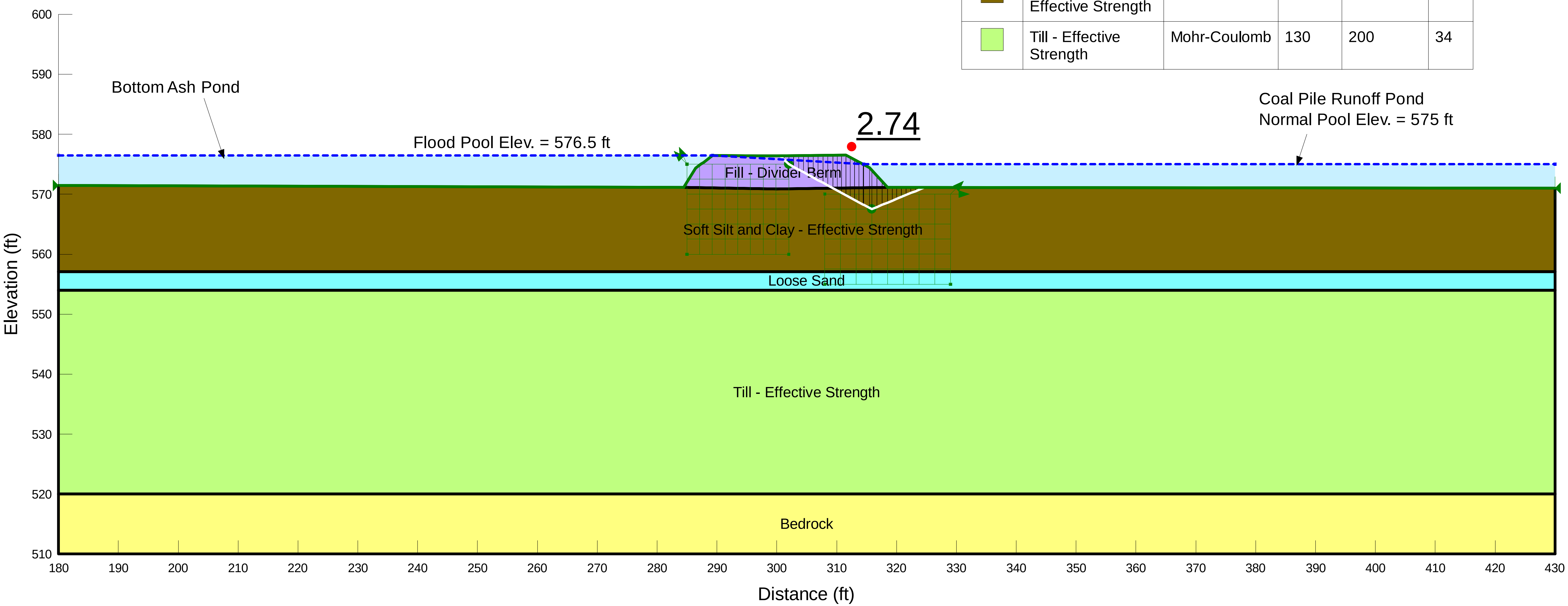
Alignment Reference Point:
N 4741.27 ft
E 9193.09 ft

Alignment Azimuth Angle
198.7 degrees

Static Analysis with Flood Pool Surcharge
Block Failure

Material Properties

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Bedrock	Bedrock (Impenetrable)			
	Fill - Divider Berm	Mohr-Coulomb	130	0	33
	Loose Sand	Mohr-Coulomb	120	0	28
	Soft Silt and Clay - Effective Strength	Mohr-Coulomb	100	100	30
	Till - Effective Strength	Mohr-Coulomb	130	200	34



DTE Monroe - Area 15 Safety Factor Analysis

Cross Section D-D'

Alignment Reference Point:
N 4741.27 ft
E 9193.09 ft

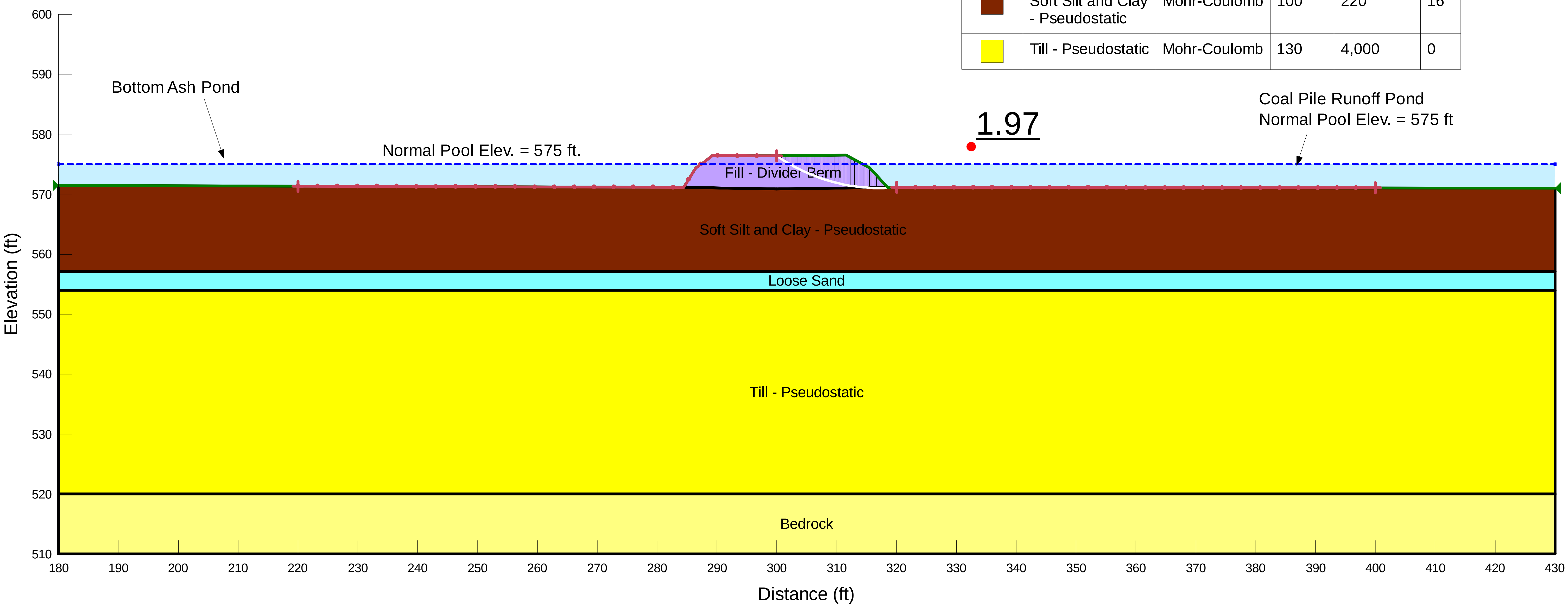
Alignment Azimuth Angle
198.7 degrees

Pseudostatic (Seismic) Analysis
Circular Slip Surfaces

Pseudostatic Analysis
Seismic Coefficient = 0.11g

Material Properties

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Bedrock	Bedrock (Impenetrable)			
	Fill - Divider Berm	Mohr-Coulomb	130	0	33
	Loose Sand	Mohr-Coulomb	120	0	28
	Soft Silt and Clay - Pseudostatic	Mohr-Coulomb	100	220	16
	Till - Pseudostatic	Mohr-Coulomb	130	4,000	0



DTE Monroe - Area 15 Safety Factor Analysis

Cross Section D-D'

Alignment Reference Point:
N 4741.27 ft
E 9193.09 ft

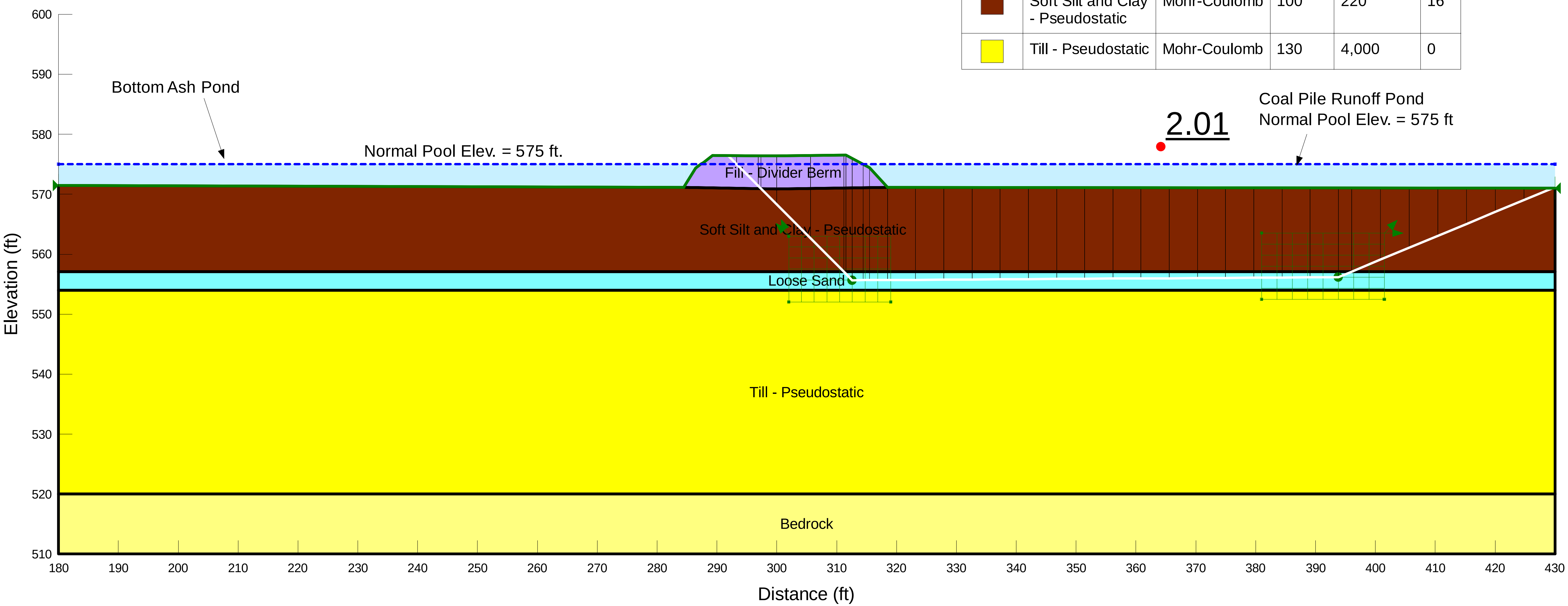
Alignment Azimuth Angle
198.7 degrees

Pseudostatic (Seismic) Analysis
Block Failure

Pseudostatic Analysis
Seismic Coefficient = 0.11g

Material Properties

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Bedrock	Bedrock (Impenetrable)			
	Fill - Divider Berm	Mohr-Coulomb	130	0	33
	Loose Sand	Mohr-Coulomb	120	0	28
	Soft Silt and Clay - Pseudostatic	Mohr-Coulomb	100	220	16
	Till - Pseudostatic	Mohr-Coulomb	130	4,000	0



DTE Monroe - Area 15 Safety Factor Analysis

Cross Section D-D'

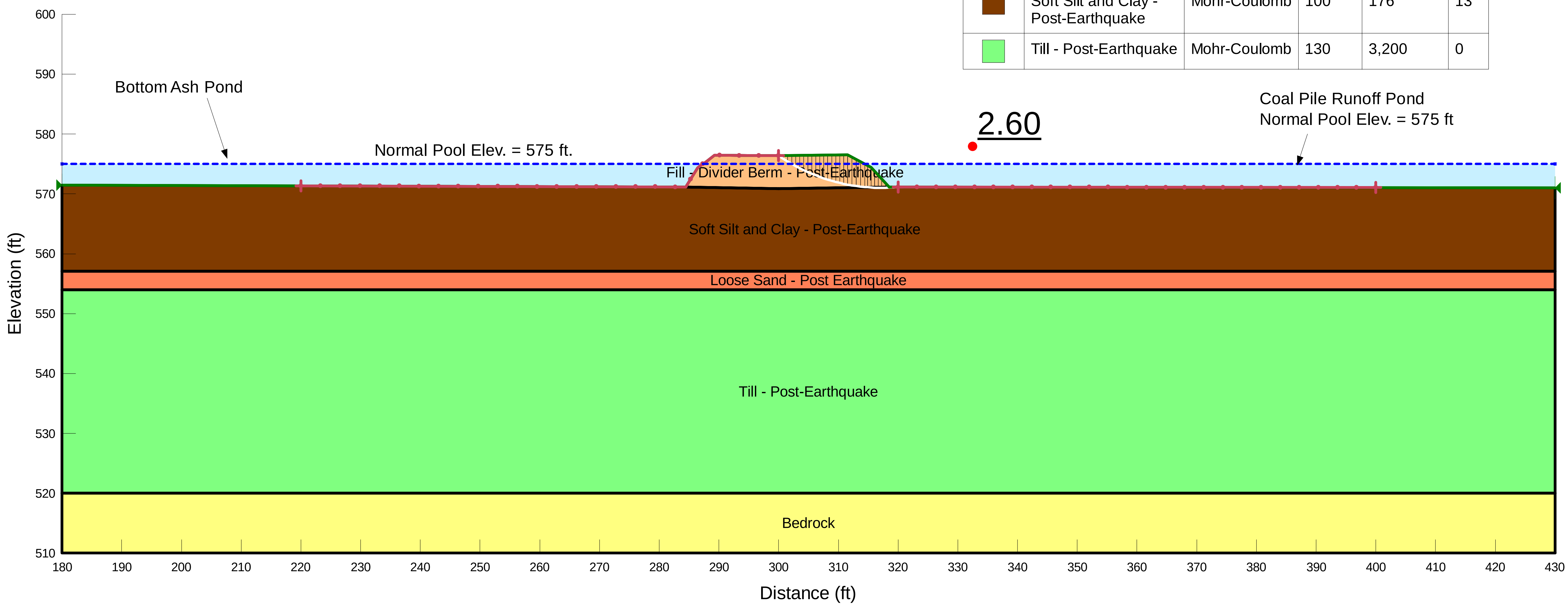
Alignment Reference Point:
N 4741.27 ft
E 9193.09 ft

Alignment Azimuth Angle
198.7 degrees

Post-Earthquake (Post-Liquefaction) Analysis
Circular Slip Surfaces

Material Properties

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Bedrock	Bedrock (Impenetrable)			
	Fill - Divider Berm - Post-Earthquake	Mohr-Coulomb	130	0	28
	Loose Sand - Post Earthquake	Mohr-Coulomb	120	250	0
	Soft Silt and Clay - Post-Earthquake	Mohr-Coulomb	100	176	13
	Till - Post-Earthquake	Mohr-Coulomb	130	3,200	0



DTE Monroe - Area 15 Safety Factor Analysis

Cross Section D-D'

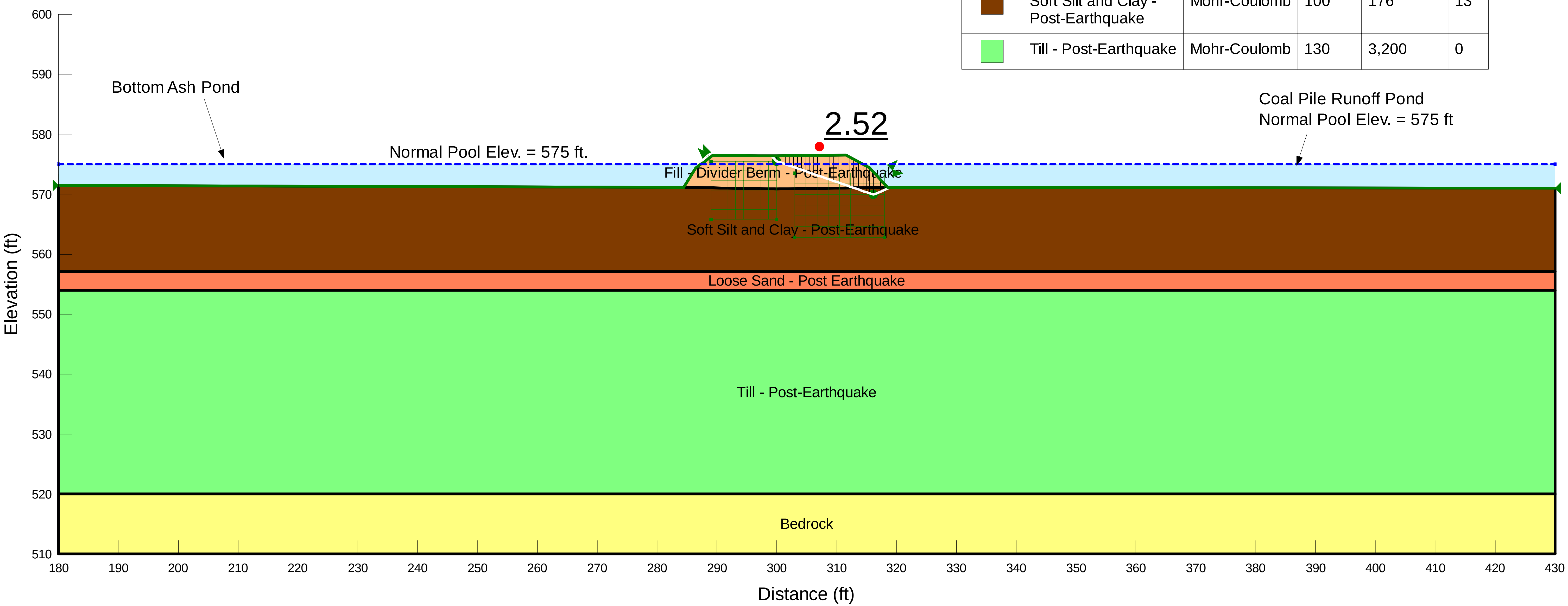
Alignment Reference Point:
N 4741.27 ft
E 9193.09 ft

Alignment Azimuth Angle
198.7 degrees

Post-Earthquake (Post-Liquefaction) Analysis
Block Failure

Material Properties

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Bedrock	Bedrock (Impenetrable)			
	Fill - Divider Berm - Post-Earthquake	Mohr-Coulomb	130	0	28
	Loose Sand - Post Earthquake	Mohr-Coulomb	120	250	0
	Soft Silt and Clay - Post-Earthquake	Mohr-Coulomb	100	176	13
	Till - Post-Earthquake	Mohr-Coulomb	130	3,200	0



DTE Monroe - Area 15 Safety Factor Analysis

Cross Section E-E'

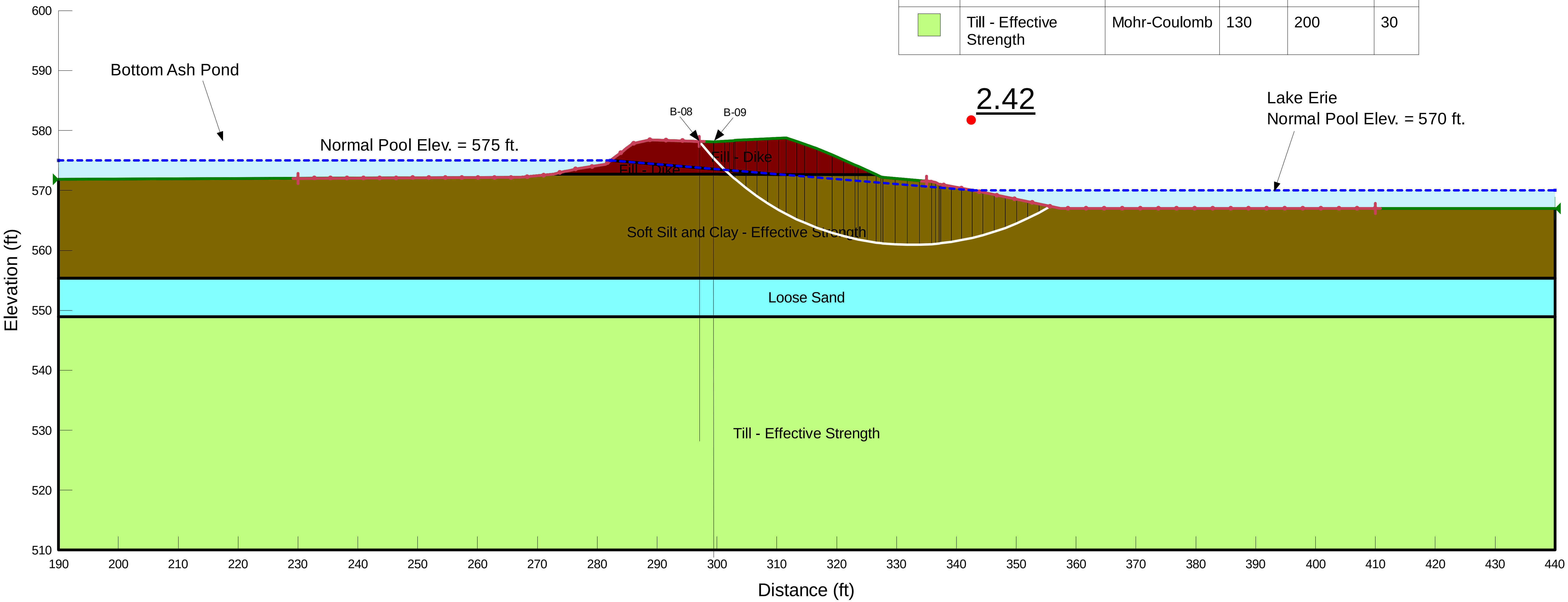
Alignment Reference Point:
N 4409.03 ft
E 9343.45 ft

Alignment Azimuth Angle
100.4 degrees

Static Analysis
Circular Slip Surfaces

Material Properties

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Fill - Dike	Mohr-Coulomb	120	0	30
	Loose Sand	Mohr-Coulomb	120	0	28
	Soft Silt and Clay - Effective Strength	Mohr-Coulomb	100	100	30
	Till - Effective Strength	Mohr-Coulomb	130	200	30



DTE Monroe - Area 15 Safety Factor Analysis

Cross Section E-E'

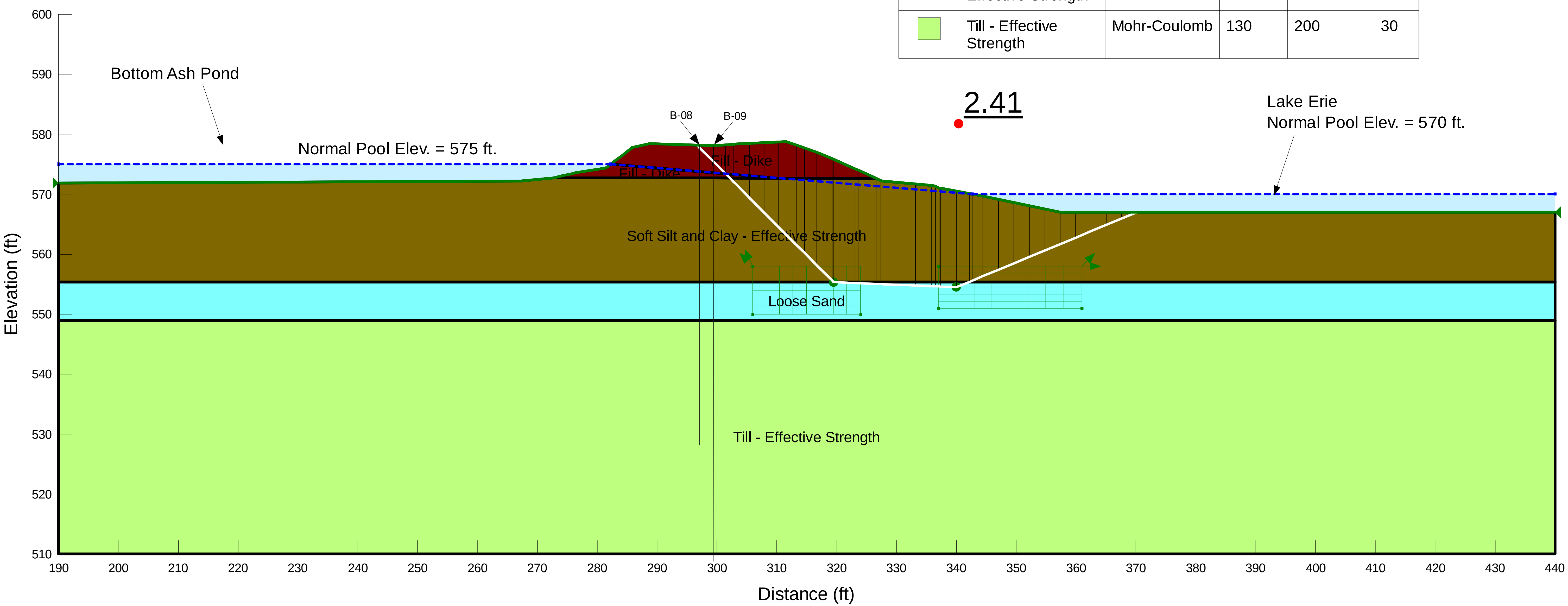
Alignment Reference Point:
N 4409.03 ft
E 9343.45 ft

Alignment Azimuth Angle
100.4 degrees

Static Analysis
Block Failure

Material Properties

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Fill - Dike	Mohr-Coulomb	120	0	30
	Loose Sand	Mohr-Coulomb	120	0	28
	Soft Silt and Clay - Effective Strength	Mohr-Coulomb	100	100	30
	Till - Effective Strength	Mohr-Coulomb	130	200	30



DTE Monroe - Area 15 Safety Factor Analysis

Cross Section E-E'

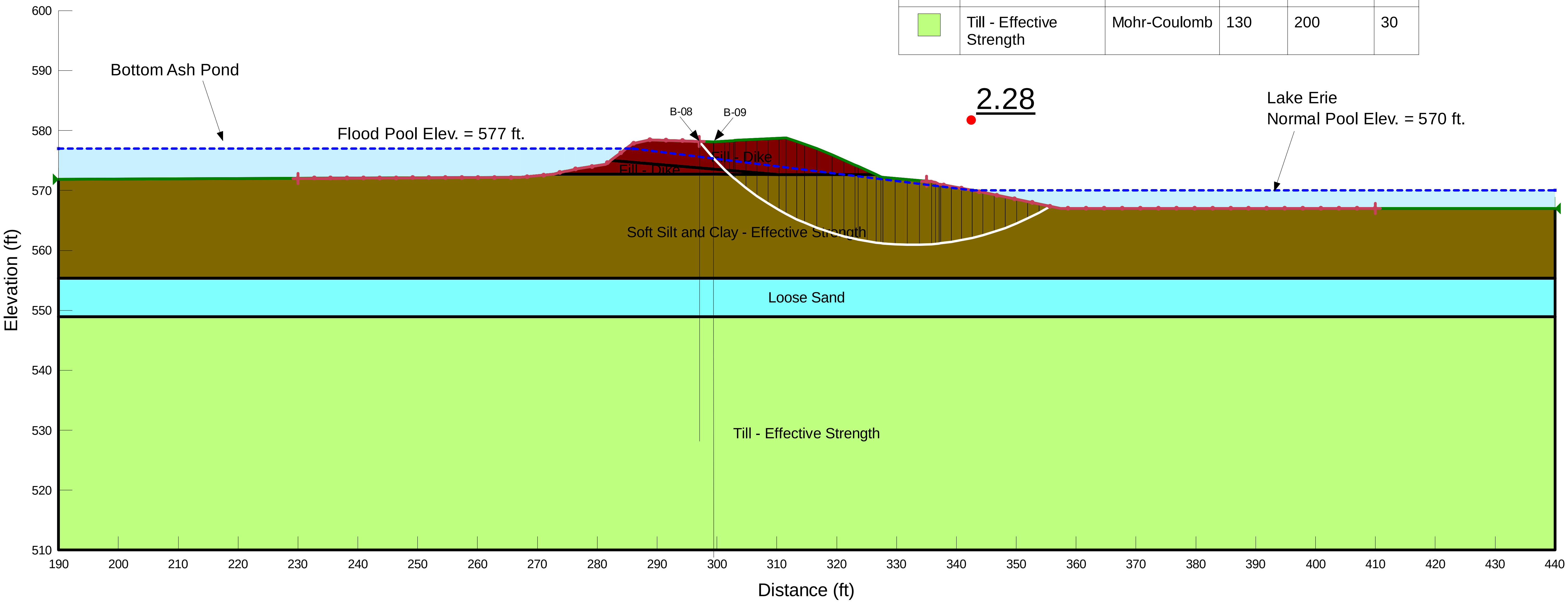
Alignment Reference Point:
N 4409.03 ft
E 9343.45 ft

Alignment Azimuth Angle
100.4 degrees

Static Analysis with Flood Pool Surcharge
Circular Slip Surfaces

Material Properties

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Fill - Dike	Mohr-Coulomb	120	0	30
	Loose Sand	Mohr-Coulomb	120	0	28
	Soft Silt and Clay - Effective Strength	Mohr-Coulomb	100	100	30
	Till - Effective Strength	Mohr-Coulomb	130	200	30



DTE Monroe - Area 15 Safety Factor Analysis

Cross Section E-E'

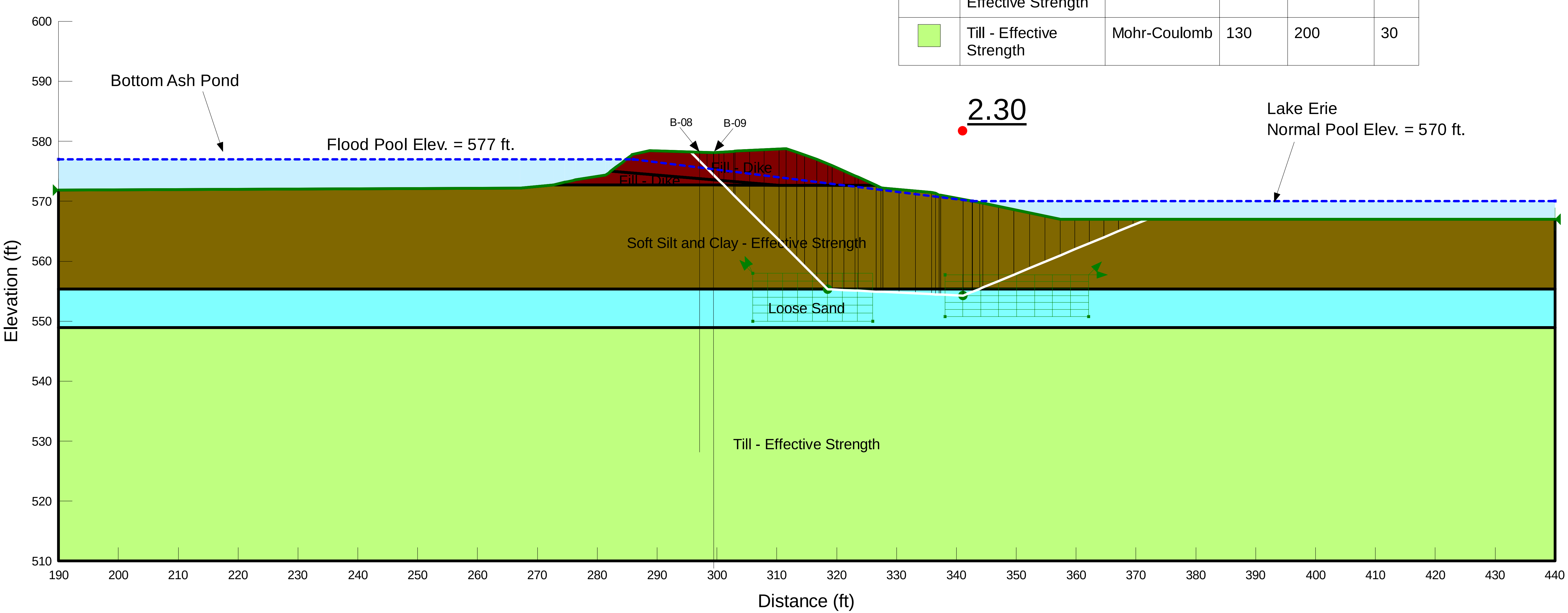
Alignment Reference Point:
N 4409.03 ft
E 9343.45 ft

Alignment Azimuth Angle
100.4 degrees

Static Analysis with Flood Pool Surcharge
Block Failure

Material Properties

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Fill - Dike	Mohr-Coulomb	120	0	30
	Loose Sand	Mohr-Coulomb	120	0	28
	Soft Silt and Clay - Effective Strength	Mohr-Coulomb	100	100	30
	Till - Effective Strength	Mohr-Coulomb	130	200	30



DTE Monroe - Area 15 Safety Factor Analysis

Cross Section E-E'

Alignment Reference Point:
N 4409.03 ft
E 9343.45 ft

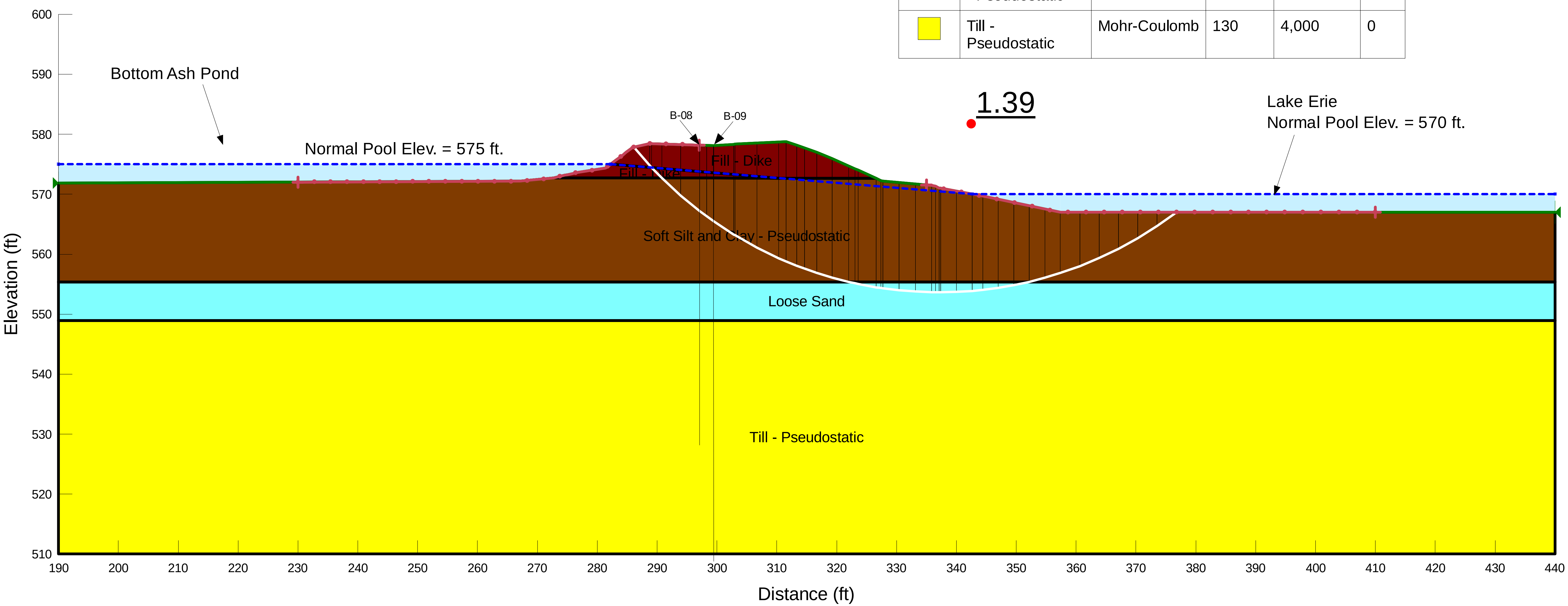
Alignment Azimuth Angle
100.4 degrees

Pseudostatic (Seismic) Analysis
Circular Slip Surfaces

Pseudostatic Analysis
Seismic Coefficient = 0.11g

Material Properties

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Fill - Dike	Mohr-Coulomb	120	0	30
	Loose Sand	Mohr-Coulomb	120	0	28
	Soft Silt and Clay - Pseudostatic	Mohr-Coulomb	100	220	16
	Till - Pseudostatic	Mohr-Coulomb	130	4,000	0



DTE Monroe - Area 15 Safety Factor Analysis

Cross Section E-E'

Alignment Reference Point:
N 4409.03 ft
E 9343.45 ft

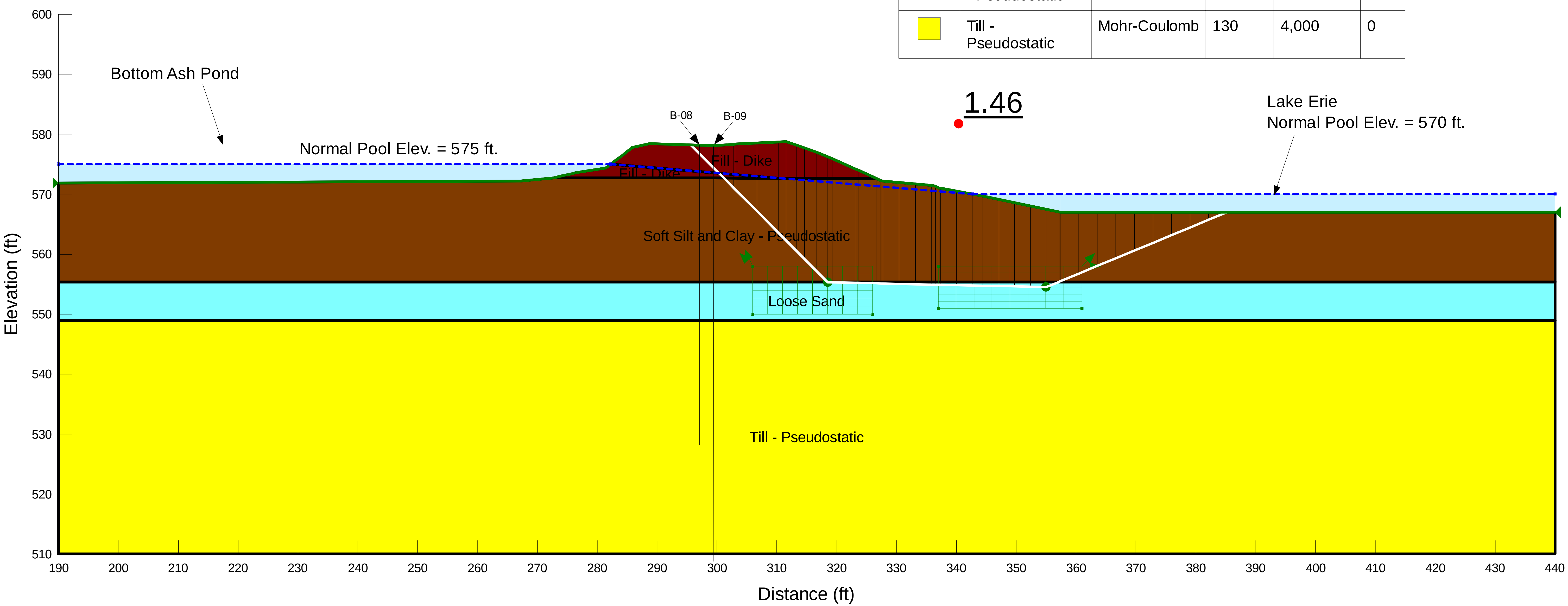
Alignment Azimuth Angle
100.4 degrees

Pseudostatic (Seismic) Analysis
Block Failure

Pseudostatic Analysis
Seismic Coefficient = 0.11g

Material Properties

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Fill - Dike	Mohr-Coulomb	120	0	30
	Loose Sand	Mohr-Coulomb	120	0	28
	Soft Silt and Clay - Pseudostatic	Mohr-Coulomb	100	220	16
	Till - Pseudostatic	Mohr-Coulomb	130	4,000	0



DTE Monroe - Area 15 Safety Factor Analysis

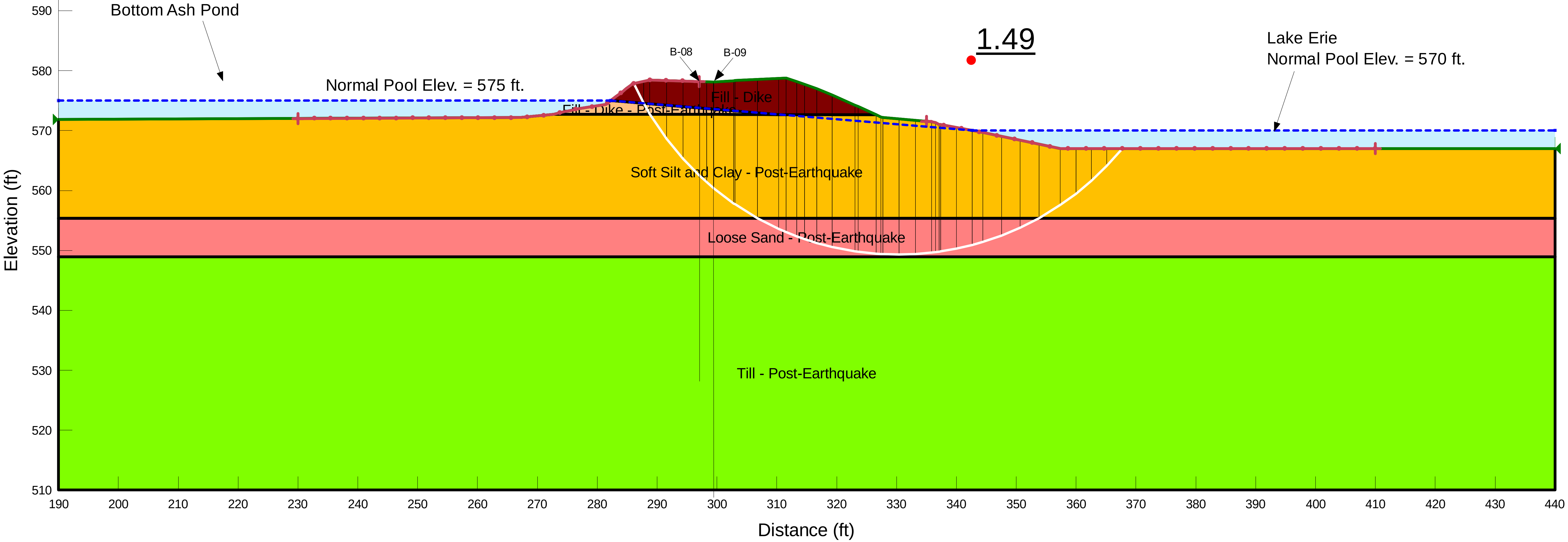
Cross Section E-E'

Alignment Reference Point:
N 4409.03 ft
E 9343.45 ft

Alignment Azimuth Angle
100.4 degrees

Post-Earthquake (Post-Liquefaction) Analysis
Circular Slip Surfaces

Color	Material Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Fill - Dike	Mohr-Coulomb	120	0	30
	Fill - Dike - Post-Earthqake	Mohr-Coulomb	120	0	25
	Loose Sand - Post-Earthquake	Mohr-Coulomb	120	250	0
	Soft Silt and Clay - Post-Earthquake	Mohr-Coulomb	100	176	13
	Till - Post-Earthquake	Mohr-Coulomb	130	3,200	0



DTE Monroe - Area 15 Safety Factor Analysis

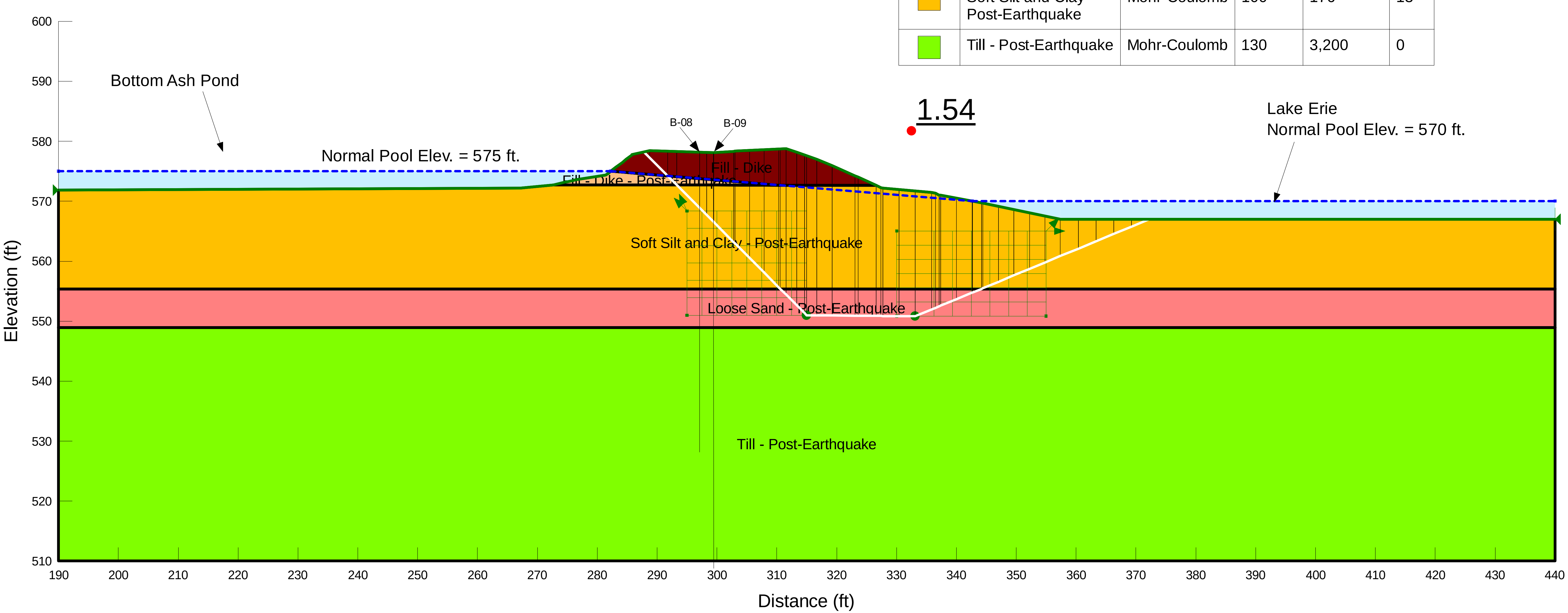
Cross Section E-E'

Alignment Reference Point:
N 4409.03 ft
E 9343.45 ft

Alignment Azimuth Angle
100.4 degrees

Post-Earthquake (Post-Liquefaction) Analysis
Block Failure

Color	Material Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Fill - Dike	Mohr-Coulomb	120	0	30
	Fill - Dike - Post-Earthqake	Mohr-Coulomb	120	0	25
	Loose Sand - Post-Earthquake	Mohr-Coulomb	120	250	0
	Soft Silt and Clay - Post-Earthquake	Mohr-Coulomb	100	176	13
	Till - Post-Earthquake	Mohr-Coulomb	130	3,200	0



Appendix E
Results of Liquefaction Analysis

Method: Idriss and Boulanger (2008), *Soil Liquefaction during Earthquakes*, EERI MNO-12

Title:	DTE Monroe		**Input Parameters:**			
			Peak ground acceleration, pga (g):	0.1008		
Project:	Area 15		Earthquake Magnitude (M):	6.5		
Project No.:	60516675		Water Table Depth at the time of drilling	3	ft	0.91 m
			Water Table Depth at the time of earthquake	3	ft	0.91 m
Date:	2/16/2018		Avg Unit Weight above GWT	117	pcf	18.3792333 kN/m^3
Boring No.	B-01		Avg Unit Weight below GWT	120	pcf	18.8504957 kN/m^3
Units	American	feet, pounds, pcf	Borehole Diameter	0.667	ft	203 mm
			Correction for Sampler Liner (N/Y)	N	ft	
			Rod stickup above ground at start of drive	5	ft	1.524 m
			Boring Total Depth	48.5	ft	14.7828 m
			Ground Surface Elevation	577.3	ft	175.96104 m

Calculated Volumetric Settlement:	0.00 ft
Calculated LDI:	0.0 ft
MSF for Sand	1.30

Bold values for N and Fines were directly mesured.

Data No.	Depth	Elevation	Measured N Previously corrected for gravel content (*)	Soil Type (USCS)	Flag: "Unsaturated", "Clay", "85% Sat"	Fines Content (%)	Energy Ratio (%)	N_{60}	C_N	$(N_1)_{60}$	$(N_1)_{60\text{-cs}}$ for liquefaction triggering	$(N_1)_{60\text{-cs}}$ for residual strength	CRR	CSR	Factor of Safety	Layer Thickness ΔH_i	ΔLDI_i	Vertical Reconsol. Strain, ε_v	Layer Settlement ΔS_i
	ft	ft														ft	ft		ft
1	1.75	575.55	**4**	OL	Unsaturated	10	80	4.6	1.70	7.8	9.0	8	#N/A	0.066	2.00	0.88	0.00	0.000	0.000
2	4.25	573.05	**17**	SP	85% Sat	7.1	80	19.6	1.70	33.2	33.4	33	1.162	0.065	2.00	2.13	0.00	0.000	0.000
3	6.75	570.55	**12**	GP	85% Sat	15	80	14.7	1.70	25.0	28.3	26	0.566	0.085	2.00	2.50	0.00	0.000	0.000
4	9.25	568.05	**5**	GP	85% Sat	15	80	6.1	1.70	10.4	13.7	11	0.208	0.095	2.00	2.50	0.00	0.000	0.000
5	11.75	565.55	**29**	GP	85% Sat	15	80	37.8	1.30	49.1	52.3	50	2.000	0.101	2.00	2.50	0.00	0.000	0.000
6	14.25	563.05	**37**	GP	85% Sat	15	80	48.2	1.24	59.9	63.1	61	2.000	0.104	2.00	2.50	0.00	0.000	0.000
7	19.25	558.05	**4**	CL	Clay	72	80	5.8	1.18	na	na	na	#N/A	0.107	2.00	3.75	0.00	0.000	0.000
8	24.25	553.05	**5**	SC-SM	85% Sat	48.3	80	7.3	1.21	8.8	14.4	11	0.205	0.107	1.91	5.00	0.00	0.000	0.001
9	26	551.3	**29**	SP/GP	85% Sat	13.9	80	42.2	1.07	45.3	48.1	46	2.000	0.107	2.00	3.38	0.00	0.000	0.000
10	29.25	548.05	**33**	GP	85% Sat	13.9	80	48.1	1.05	50.4	53.2	51	2.000	0.106	2.00	2.50	0.00	0.000	0.000
11	34.25	543.05	**19**	GP	85% Sat	13.9	80	29.1	1.02	29.6	32.5	31	0.920	0.104	2.00	4.13	0.00	0.000	0.000
12	39.25	538.05	**29**	GP	85% Sat	13.9	80	44.5	0.98	43.5	46.4	44	2.000	0.101	2.00	5.00	0.00	0.000	0.000
13	44.25	533.05	**36**	CL	Clay	90	80	55.2	0.95	na	na	na	#N/A	0.098	2.00	5.00	0.00	0.000	0.000
																#VALUE!	######		#VALUE!

Method: Idriss and Boulanger (2008), *Soil Liquefaction during Earthquakes*, EERI MNO-12

Title: DTE Monroe
Project: Area 15
Project No.: 60516675
Date: 2/16/2018
Boring No. B-02
Units American feet, pounds, pcf

Input Parameters:

Parameter	Value	Unit	SI
Peak ground acceleration, pga (g):	0.1008		
Earthquake Magnitude (M):	6.5		
Water Table Depth at the time of drilling	8.5	ft	2.59 m
Water Table Depth at the time of earthquake	8.5	ft	2.59 m
Avg Unit Weight above GWT	117	pcf	18.3792333 kN/m^3
Avg Unit Weight below GWT	120	pcf	18.8504957 kN/m^3
Borehole Diameter	0.667	ft	203 mm
Correction for Sampler Liner (N/Y)	N	ft	
Rod stickup above ground at start of drive	5	ft	1.524 m
Boring Total Depth	54	ft	16.4592 m
Ground Surface Elevation	579.7	ft	176.69256 m

0

Calculated Volumetric Settlement: 0.00 ft
Calculated LDI: 0.0 ft
MSF for Sand 1.30

Bold values for N and Fines were directly mesured.

Data No.	Depth	Elevation	Measured N Previously corrected for gravel content (*)	Soil Type (USCS)	Flag: "Unsaturated", "Clay", "85% Sat"	Fines Content (%)	Energy Ratio (%)	N_{60}	C_N	$(N_1)_{60}$	$(N_1)_{60\text{-}cs}$ for liquefaction triggering	$(N_1)_{60\text{-}cs}$ for residual strength	CRR	CSR	Factor of Safety	Layer Thickness ΔH_i	ΔLDI_i	Vertical Reconsol. Strain, ε_v	Layer Settlement ΔS_i
	ft	ft														ft	ft		ft
1	1.75	577.95	**15**	CL	Clay	90	80	17.3	1.70	na	na	na	#N/A	0.066	2.00	0.88	0.00	0.000	0.000
2	4.25	575.45	**24**	SM	Unsaturated	10	80	27.6	1.61	44.4	45.6	44	#N/A	0.065	2.00	2.13	0.00	0.000	0.000
3	6.75	572.95	**15**	SP/CL	Unsaturated	10	80	18.4	1.55	28.4	29.6	28	#N/A	0.065	2.00	2.50	0.00	0.000	0.000
4	9.25	570.45	**5**	CL	Clay	61.3	80	6.1	1.24	na	na	na	#N/A	0.064	2.00	2.50	0.00	0.000	0.000
5	14.25	565.45	**9**	CL	Clay	61.3	80	11.7	1.16	na	na	na	#N/A	0.073	2.00	3.75	0.00	0.000	0.000
6	19.25	560.45	**3**	CL	Clay	61.3	80	4.4	1.10	na	na	na	#N/A	0.082	2.00	5.00	0.00	0.000	0.000
7	24.25	555.45	**3**	CL	Clay	61.3	80	4.4	1.05	na	na	na	#N/A	0.087	2.00	5.00	0.00	0.000	0.000
8	28	551.7	**8**	SM/GP	85% Sat	15.2	80	11.7	1.02	11.9	15.3	13	0.207	0.089	2.00	4.38	0.00	0.000	0.000
9	30	549.7	**42**	GP	85% Sat	18.5	80	64.4	0.99	63.9	68.1	65	2.000	0.089	2.00	2.88	0.00	0.000	0.000
10	34.25	545.45	**23**	GP	85% Sat	18.5	80	35.3	0.97	34.1	38.3	35	2.000	0.089	2.00	3.13	0.00	0.000	0.000
11	39.25	540.45	**27**	SP	85% Sat	9.9	80	41.4	0.94	38.8	39.9	39	2.000	0.088	2.00	4.63	0.00	0.000	0.000
12	44.25	535.45	**21**	CL	Clay	90	80	32.2	0.92	na	na	na	#N/A	0.087	2.00	5.00	0.00	0.000	0.000
13	49.25	530.45	**55**	ML	85% Sat	50	80	84.3	0.90	75.6	81.2	78	2.000	0.085	2.00	5.00	0.00	0.000	0.000
																#VALUE!	######		#VALUE!

Method: Idriss and Boulanger (2008), *Soil Liquefaction during Earthquakes* , EERI MNO-12

Input Parameters:

Title:	DTE Monroe	Peak ground acceleration, pga (g):	0.1008			Calculated Volumetric Settlement:	0.02 ft
Project:	Area 15	Earthquake Magnitude (M):	6.5			Calculated LDI:	0.0 ft
Project No.:	60516675	Water Table Depth at the time of drilling	1	ft	0.30 m	MSF for Sand	1.30
		Water Table Depth at the time of earthquake	1	ft	0.30 m		
Date:	2/20/2018	Avg Unit Weight above GWT	117	pcf	18.3792333 kN/m^3		
Boring No.	B-03	Avg Unit Weight below GWT	120	pcf	18.8504957 kN/m^3		
Units	American feet, pounds, pcf	Borehole Diameter	0.667	ft	203 mm		
		Correction for Sampler Liner (N/Y)	N	ft			
		Rod stickup above ground at start of drive	5	ft	1.524 m		
		Boring Total Depth	58.6	ft	17.86128 m		
		Ground Surface Elevation	576.1	ft	175.59528 m		

Bold values for N and Fines were directly mesured.

Data No.	Depth	Elevation	Measured N Previously corrected for gravel content (*)	Soil Type (USCS)	Flag: "Unsaturated", "Clay", "85% Sat"	Fines Content (%)	Energy Ratio (%)	N_{60}	C_N	$(N_1)_{60}$	$(N_1)_{60-cs}$ for liquefaction triggering	$(N_1)_{60-cs}$ for residual strength	CRR	CSR	Factor of Safety	Layer Thickness ΔH_i	ΔLDI_i	Vertical Reconsol. Strain, ε_v	Layer Settlement ΔS_i
	ft	ft														ft	ft		ft
1	1.75	574.35	**17**	SP/CL	Unsaturated	4.8	80	19.6	1.70	33.2	33.2	33	#N/A	0.066	2.00	0.88	0.00	0.000	0.000
2	4.25	571.85	**3**	CL/SP	Unsaturated	4.8	80	3.5	1.70	5.9	5.9	6	#N/A	0.100	2.00	2.13	0.00	0.000	0.000
3	6.75	569.35	**2**	CL	Clay	64.9	80	2.5	1.57	na	na	na	#N/A	0.113	2.00	2.50	0.00	0.000	0.000
4	9.25	566.85	**1**	CL	Clay	64.9	80	1.2	1.45	na	na	na	#N/A	0.118	2.00	2.50	0.00	0.000	0.000
5	14.25	561.85	**2**	OL	Clay	90	80	2.6	1.32	na	na	na	#N/A	0.120	2.00	3.75	0.00	0.000	0.000
6	19.25	556.85	**0**	CL	Clay	90	80	0.0	1.21	na	na	na	#N/A	0.120	2.00	5.00	0.00	0.000	0.000
7	24.25	551.85	**2**	CL	Clay	90	80	2.9	1.13	na	na	na	#N/A	0.117	2.00	5.00	0.00	0.000	0.000
8	26	550.1	**3**	SM	85% Sat	13.2	80	4.4	1.21	5.3	7.9	6	0.139	0.115	**1.20**	3.38	0.02	0.003	0.011
9	28	548.1	**4**	SP	85% Sat	5.7	80	5.8	1.17	6.8	6.8	7	0.129	0.114	**1.13**	1.88	0.02	0.005	0.009
10	30	546.1	**5**	SP	85% Sat	9.2	80	7.7	1.12	8.6	9.4	9	0.150	0.113	1.33	2.00	0.01	0.002	0.004
11	32	544.1	**18**	SP	85% Sat	9.2	80	27.6	1.05	29.0	29.8	29	0.631	0.111	2.00	2.00	0.00	0.000	0.000
12	34.25	541.85	**16**	SP	85% Sat	9.2	80	24.5	1.03	25.2	26.0	25	0.416	0.110	2.00	2.13	0.00	0.000	0.000
13	39.25	536.85	**34**	CL	Clay	90	80	52.1	0.99	na	na	na	#N/A	0.107	2.00	3.63	0.00	0.000	0.000
14	44.25	531.85	**27**	CL	Clay	90	80	41.4	0.96	na	na	na	#N/A	0.103	2.00	5.00	0.00	0.000	0.000

Method: Idriss and Boulanger (2008), *Soil Liquefaction during Earthquakes* , EERI MNO-12

Title:	DTE Monroe	**Input Parameters:**				Calculated Volumetric Settlement: 0.00 ft
Project:	Area 15	Peak ground acceleration, pga (g):	0.1008			Calculated LDI: 0.0 ft
Project No.:	60516675	Earthquake Magnitude (M):	6.5			MSF for Sand 1.30
		Water Table Depth at the time of drilling	5.5	ft	1.68 m	
		Water Table Depth at the time of earthquake	5.5	ft	1.68 m	
Date:	2/20/2018	Avg Unit Weight above GWT	117	pcf	18.3792333 kN/m^3	
Boring No.	B-04	Avg Unit Weight below GWT	120	pcf	18.8504957 kN/m^3	
Units	American feet, pounds, pcf	Borehole Diameter	0.667	ft	203 mm	
		Correction for Sampler Liner (N/Y)	N	ft		
		Rod stickup above ground at start of drive	5	ft	1.524 m	
		Boring Total Depth	56.2	ft	17.12976 m	
		Ground Surface Elevation	577.7	ft	176.08296 m	

Bold values for N and Fines were directly mesured.

Data No.	Depth	Elevation	Measured N Previously corrected for gravel content (*)	Soil Type (USCS)	Flag: "Unsaturated", "Clay", "85% Sat"	Fines Content (%)	Energy Ratio (%)	N_{60}	C_N	$(N_1)_{60}$	$(N_1)_{60\text{-cs}}$ for liquefaction triggering	$(N_1)_{60\text{-cs}}$ for residual strength	CRR	CSR	Factor of Safety	Layer Thickness ΔH_i	ΔLDI_i	Vertical Reconsol. Strain, ε_v	Layer Settlement ΔS_i
	ft	ft														ft	ft		ft
1	1.75	575.95	**15**	CL	Clay	25.2	80	17.3	1.70	na	na	na	#N/A	0.066	2.00	0.88	0.00	0.000	0.000
2	4.25	573.45	**6**	CL	Clay	59.4	80	6.9	1.60	na	na	na	#N/A	0.065	2.00	2.13	0.00	0.000	0.000
3	6.75	570.95	**2**	CL	Clay	59.4	80	2.5	1.37	na	na	na	#N/A	0.065	2.00	2.50	0.00	0.000	0.000
4	9.25	568.45	**2**	CL	Clay	59.4	80	2.5	1.30	na	na	na	#N/A	0.077	2.00	2.50	0.00	0.000	0.000
5	14.25	563.45	**0**	OL	Clay	59.4	80	0.0	1.22	na	na	na	#N/A	0.087	2.00	3.75	0.00	0.000	0.000
6	19.25	558.45	**0**	CL	Clay	59.4	80	0.0	1.14	na	na	na	#N/A	0.094	2.00	5.00	0.00	0.000	0.000
7	24.25	553.45	**0**	CL	Clay	59.4	80	0.0	1.08	na	na	na	#N/A	0.097	2.00	5.00	0.00	0.000	0.000
8	29.25	548.45	**6**	SP	85% Sat	10	80	8.7	1.07	9.3	10.5	9	0.160	0.097	1.64	5.00	0.01	0.001	0.004
9	30.75	546.95	**1**	CL	Clay	90	80	1.5	1.01	na	na	na	#N/A	0.097	2.00	3.25	0.00	0.000	0.000
10	34.25	543.45	**41**	CL	Clay	90	80	62.9	0.99	na	na	na	#N/A	0.096	2.00	2.50	0.00	0.000	0.000
11	39.25	538.45	**30**	CL	Clay	90	80	46.0	0.96	na	na	na	#N/A	0.095	2.00	4.25	0.00	0.000	0.000
12	44.25	533.45	**53**	CL	Clay	90	80	81.3	0.93	na	na	na	#N/A	0.092	2.00	5.00	0.00	0.000	0.000

Method: Idriss and Boulanger (2008), *Soil Liquefaction during Earthquakes*, EERI MNO-12

Input Parameters:

Title:	DTE Monroe	
Project:	Area 15	
Project No.:	60516675	
Date:	2/20/2018	
Boring No.	B-08	
Units	American	feet, pounds, pcf

Parameter	Value	Unit	SI
Peak ground acceleration, pga (g):	0.1008		
Earthquake Magnitude (M):	6.5		
Water Table Depth at the time of drilling	6.5	ft	1.98 m
Water Table Depth at the time of earthquake	6.5	ft	1.98 m
Avg Unit Weight above GWT	117	pcf	18.3792333 kN/m^3
Avg Unit Weight below GWT	120	pcf	18.8504957 kN/m^3
Borehole Diameter	0.667	ft	203 mm
Correction for Sampler Liner (N/Y)	N	ft	
Rod stickup above ground at start of drive	5	ft	1.524 m
Boring Total Depth	50	ft	15.24 m
Ground Surface Elevation	578.9	ft	176.44872 m

Calculated Volumetric Settlement:	0.34 ft
Calculated LDI:	0.4 ft
MSF for Sand	1.30

Bold values for N and Fines were directly mesured.

Data No.	Depth	Elevation	Measured N Previously corrected for gravel content (*)	Soil Type (USCS)	Flag: "Unsaturated", "Clay", "85% Sat"	Fines Content (%)	Energy Ratio (%)	N_{60}	C_N	$(N_1)_{60}$	$(N_1)_{60-cs}$ for liquefaction triggering	$(N_1)_{60-cs}$ for residual strength	CRR	CSR	Factor of Safety	Layer Thickness ΔH_i	ΔLDI_i	Vertical Reconsol. Strain, ε_v	Layer Settlement ΔS_i
	ft	ft														ft	ft		ft
1	1.75	577.15	**24**	GM/CL-ML	Clay	90	80	27.6	1.70	na	na	na	#N/A	0.066	2.00	0.88	0.00	0.000	0.000
2	4.25	574.65	**14**	CL-ML	Clay	90	80	16.1	1.60	na	na	na	#N/A	0.065	2.00	2.13	0.00	0.000	0.000
3	6.75	572.15	**3**	SP/CL-ML	85% Sat	10	80	3.7	1.70	6.3	7.4	6	0.144	0.065	2.00	2.50	0.00	0.000	0.000
4	9.25	569.65	**3**	CL-ML	Clay	78.7	80	3.7	1.27	na	na	na	#N/A	0.071	2.00	2.50	0.00	0.000	0.000
5	14.25	564.65	**2**	CL-ML	Clay	78.7	80	2.6	1.20	na	na	na	#N/A	0.082	2.00	3.75	0.00	0.000	0.000
6	19.25	559.65	**0**	CL-ML	Clay	60.5	80	0.0	1.12	na	na	na	#N/A	0.090	2.00	5.00	0.00	0.000	0.000
7	24.25	554.65	**1**	SP	85% Sat	1.3	80	1.5	1.19	1.7	1.7	2	0.091	0.093	**0.97**	5.00	0.36	0.067	0.336
8	29.25	549.65	**7**	SP	85% Sat	1.3	80	10.2	1.05	10.7	10.7	11	0.161	0.094	1.71	5.00	0.01	0.001	0.003
9	34.25	544.65	**38**	CL	Clay	90	80	58.3	0.99	na	na	na	#N/A	0.094	2.00	5.00	0.00	0.000	0.000
10	39.25	539.65	**28**	ML	85% Sat	40	80	42.9	0.96	41.1	46.6	43	2.000	0.092	2.00	5.00	0.00	0.000	0.000
11	44.25	534.65	**40**	CL	Clay	90	80	61.3	0.93	na	na	na	#N/A	0.090	2.00	5.00	0.00	0.000	0.000
12	49.25	529.65	**30**	CL	Clay	90	80	46.0	0.91	na	na	na	#N/A	0.088	2.00	5.00	0.00	0.000	0.000
																#VALUE!	######		#VALUE!
																#VALUE!	######		#VALUE!

Method: Idriss and Boulanger (2008), *Soil Liquefaction during Earthquakes*, EERI MNO-12

Title: DTE Monroe
Project: Area 15
Project No.: 60516675
Date: 2/20/2018
Boring No. B-09
Units American feet, pounds, pcf

Input Parameters:

Parameter	Value	Unit	Converted
Peak ground acceleration, pga (g):	0.1008		
Earthquake Magnitude (M):	6.5		
Water Table Depth at the time of drilling	6	ft	1.83 m
Water Table Depth at the time of earthquake	6	ft	1.83 m
Avg Unit Weight above GWT	117	pcf	18.3792333 kN/m^3
Avg Unit Weight below GWT	120	pcf	18.8504957 kN/m^3
Borehole Diameter	0.667	ft	203 mm
Correction for Sampler Liner (N/Y)	N	ft	
Rod stickup above ground at start of drive	5	ft	1.524 m
Boring Total Depth	69.3	ft	21.12264 m
Ground Surface Elevation	579	ft	176.4792 m

Calculated Volumetric Settlement: 0.01 ft
Calculated LDI: 0.0 ft
MSF for Sand 1.30

Bold values for N and Fines were directly mesured.

Data No.	Depth	Elevation	Measured N Previously corrected for gravel content (*)	Soil Type (USCS)	Flag: "Unsaturated", "Clay", "85% Sat"	Fines Content (%)	Energy Ratio (%)	N_{60}	C_N	$(N_1)_{60}$	$(N_1)_{60\text{-}cs}$ for liquefaction triggering	$(N_1)_{60\text{-}cs}$ for residual strength	CRR	CSR	Factor of Safety	Layer Thickness ΔH_i	ΔLDI_i	Vertical Reconsol. Strain, ε_v	Layer Settlement ΔS_i
	ft	ft														ft	ft		ft
1	1.75	577.25	**50**	SM	Unsaturated	18.2	80	57.5	1.70	97.8	101.9	99	#N/A	0.066	2.00	0.88	0.00	0.000	0.000
2	4.25	574.75	**30**	CL-ML	Clay	90	80	34.5	1.60	na	na	na	#N/A	0.065	2.00	2.13	0.00	0.000	0.000
3	6.75	572.25	**4**	CL	Clay	90	80	4.9	1.37	na	na	na	#N/A	0.065	2.00	2.50	0.00	0.000	0.000
4	9.25	569.75	**3**	CL	Clay	90	80	3.7	1.28	na	na	na	#N/A	0.074	2.00	2.50	0.00	0.000	0.000
5	14.25	564.75	**4**	CL	Clay	90	80	5.2	1.21	na	na	na	#N/A	0.085	2.00	3.75	0.00	0.000	0.000
6	19.25	559.75	**1**	CL-ML	Clay	73.3	80	1.5	1.13	na	na	na	#N/A	0.092	2.00	5.00	0.00	0.000	0.000
7	24.25	554.75	**4**	SP	85% Sat	10	80	5.8	1.17	6.8	7.9	7	0.139	0.095	1.46	5.00	0.01	0.001	0.005
8	26	553	**9**	SP	85% Sat	10	80	13.1	1.08	14.1	15.3	14	0.210	0.096	2.00	3.38	0.00	0.000	0.000
9	29.25	549.75	**23**	CL	Clay	90	80	33.5	1.02	na	na	na	#N/A	0.096	2.00	2.50	0.00	0.000	0.000
10	39.25	539.75	**22**	ML	85% Sat	90	80	33.7	0.97	32.7	38.2	38	2.000	0.095	2.00	6.63	0.00	0.000	0.000
11	44.25	534.75	**62**	CL	Clay	90	80	95.1	0.93	na	na	na	#N/A	0.091	2.00	7.50	0.00	0.000	0.000
12	49.25	529.75	**38**	CL	Clay	90	80	58.3	0.91	na	na	na	#N/A	0.089	2.00	5.00	0.00	0.000	0.000
13	54.25	524.75	**44**	CL	Clay	90	80	67.5	0.89	na	na	na	#N/A	0.086	2.00	5.00	0.00	0.000	0.000
																#VALUE!	######		#VALUE!

Method: Idriss and Boulanger (2008), *Soil Liquefaction during Earthquakes*, EERI MNO-12

Title: DTE Monroe	**Input Parameters:**				Calculated Volumetric Settlement: 0.01 ft
Project: Area 15	Peak ground acceleration, pga (g):	0.1008			Calculated LDI: 0.0 ft
Project No.: 60516675	Earthquake Magnitude (M):	6.5			MSF for Sand 1.30
	Water Table Depth at the time of drilling	1.8	ft	0.55 m	
	Water Table Depth at the time of earthquake	1.8	ft	0.55 m	
Date: 2/20/2018	Avg Unit Weight above GWT	117	pcf	18.3792333 kN/m^3	
Boring No. B-12	Avg Unit Weight below GWT	120	pcf	18.8504957 kN/m^3	
Units American feet, pounds, pcf	Borehole Diameter	0.667	ft	203 mm	
	Correction for Sampler Liner (N/Y)	N	ft		
	Rod stickup above ground at start of drive	5	ft	1.524 m	
	Boring Total Depth	48.8	ft	14.87424 m	
	Ground Surface Elevation	576.7	ft	175.77816 m	

Bold values for N and Fines were directly mesured.

Data No.	Depth	Elevation	Measured N Previously corrected for gravel content (*)	Soil Type (USCS)	Flag: "Unsaturated", "Clay", "85% Sat"	Fines Content (%)	Energy Ratio (%)	N_{60}	C_N	$(N_1)_{60}$	$(N_1)_{60-cs}$ for liquefaction triggering	$(N_1)_{60-cs}$ for residual strength	CRR	CSR	Factor of Safety	Layer Thickness ΔH_i	ΔLDI_i	Vertical Reconsol. Strain, ε_v	Layer Settlement ΔS_i
	ft	ft														ft	ft		ft
1	1.75	574.95	**38**	GP	Unsaturated	6.8	80	43.7	1.70	74.3	74.4	74	#N/A	0.066	2.00	0.88	0.00	0.000	0.000
2	4.25	572.45	**16**	GP	Unsaturated	6.8	80	18.4	1.70	31.3	31.4	31	#N/A	0.083	2.00	2.13	0.00	0.000	0.000
3	6.75	569.95	**2**	CL-ML	Clay	90	80	2.5	1.53	na	na	na	#N/A	0.100	2.00	2.50	0.00	0.000	0.000
4	9.25	567.45	**2**	CL-ML	Clay	90	80	2.5	1.41	na	na	na	#N/A	0.108	2.00	2.50	0.00	0.000	0.000
5	12.75	563.95	**0**	CL-ML	Clay	90	80	0.0	1.32	na	na	na	#N/A	0.112	2.00	3.00	0.00	0.000	0.000
6	14.25	562.45	**0**	CL-ML	Clay	90	80	0.0	1.26	na	na	na	#N/A	0.113	2.00	2.50	0.00	0.000	0.000
7	19.25	557.45	**3**	CL-ML/SP	85% Sat	10	80	4.4	1.47	6.4	7.6	6	0.140	0.114	1.23	3.25	0.02	0.003	0.009
8	24.25	552.45	**30**	CL	Clay	90	80	43.7	1.12	na	na	na	#N/A	0.113	2.00	5.00	0.00	0.000	0.000
9	29.25	547.45	**24**	CL	Clay	90	80	35.0	1.07	na	na	na	#N/A	0.111	2.00	5.00	0.00	0.000	0.000
10	34.25	542.45	**34**	CL	Clay	90	80	52.1	1.02	na	na	na	#N/A	0.108	2.00	5.00	0.00	0.000	0.000
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About AECOM

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