



# Structural Stability and Safety Factor Assessment

For Compliance with the EPA Coal  
Combustion Residuals (CCR) Rule  
40 CFR §257.73(d)  
40 CFR §257.73(e)

Former B.C. Cobb Power Plant  
Ponds 0-8 and Bottom Ash Pond

June 29, 2021

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# 1 Introduction and Purpose

HDR MICHIGAN, Inc. (HDR) has prepared this Structural Stability and Safety Factor Assessment Report for Ponds 0-8 and Bottom Ash Pond at the Former B.C. Cobb Power Plant (B.C. Cobb) following the requirements of the Federal Coal Combustion Residuals (CCR) Rule to demonstrate compliance of the Former B.C. Cobb Power Plant in Muskegon, Michigan.

On April 17, 2015, the United States Environmental Protection Agency (EPA) issued the final rule (Ref. [5]) for disposal of Coal Combustion Residuals (CCR) under Subtitle D of the Resource Conservation and Recovery Act (RCRA). CCR Rule 40 CFR §257.73(b) requires that owners or operators of an existing CCR surface impoundment that either 1) has a height of five feet or more and a storage volume of 20 acre-feet or more; or 2) has a height of 20 feet or more perform periodic structural stability assessments (40 CFR §257.73(d)) and periodic safety factor assessments (40 CFR §257.73(e)).

The CCR Final Rule requires that initial and periodic structural stability assessments be conducted in accordance with Section §257.73(d). Section §257.73(e) requires that initial and periodic safety factor assessments be conducted to verify that the stability of the most critical section of the embankment complies with the required minimum factors of safety for the long-term maximum storage pool, maximum surcharge pool, and seismic load cases. This report presents the initial periodic structural stability assessment and initial periodic safety factor assessment for Ponds 0-8 and Bottom Ash Pond.

The Structural Stability and Safety Factor Assessment Report presented herein addresses the specific requirements of 40 CFR §257.73(d) and 40 CFR §257.73(e). This Structural Stability and Safety Factor Assessment Report was prepared by Mr. Bryce Burkett, P.E., and was reviewed in accordance with HDR's internal review policy by Mr. Greg Shafer, P.E., both of HDR. Mr. Burkett is a registered Professional Engineer in the State of Michigan.

## 1.1 Site Location

B.C. Cobb is a former electrical power generation facility located along North Causeway (M-120) in Muskegon, Michigan which was previously owned by Consumers Energy Company (CEC). The Muskegon Environmental Redevelopment Group, LLC (MERG) acquired the B.C. Cobb property in 2020 and has initiated the process of dewatering and removing CCR material from the ponds as part of pond remediation and closure efforts. The latitude and longitude of B.C. Cobb are approximately 43.254355 N and 86.241224 W. The site is located north of Muskegon, Michigan and south of the intersection of North Causeway (M-120) and the Muskegon River, as shown in the vicinity map, Figure 1.

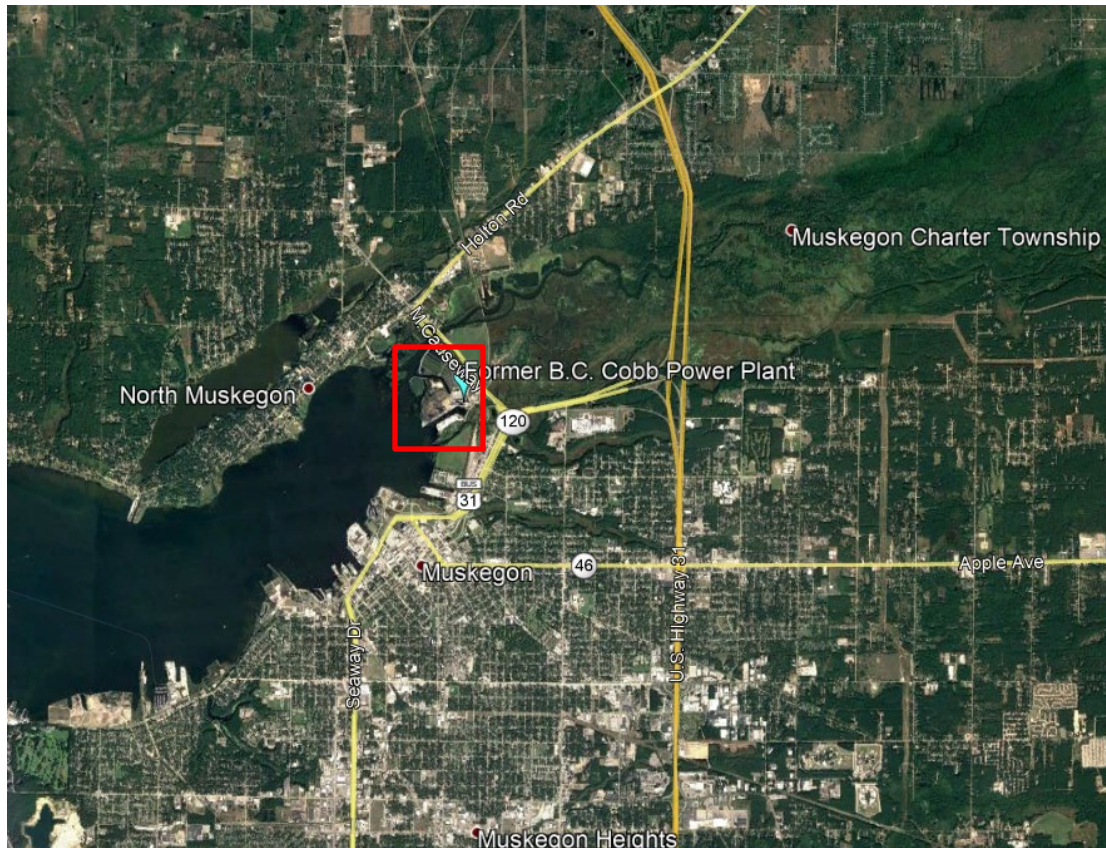


Figure 1. Site Vicinity Map

## 1.2 Site Description

B.C. Cobb began operations in the 1940s with five coal-burning units, later converting three of those units to natural gas until operations were ceased by CEC in 2016. The CCR unit, which includes Ponds 0-8 and the Bottom Ash Pond, are National Pollutant Discharge Elimination System (NPDES) treatment units. Figure 3 displays the pond layout at the site. Historically, CCR was deposited in the ponds by utilizing sluicing methods. Bottom ash slurry was directed into the Bottom Ash Pond, with Bottom Ash Pond overflow directed into either Ponds 5 or 6. Fly ash from the power plant was directed into Ponds 7 and 8. The ponded CCR was routed through the remaining ponds in series. Each pond allowed a portion of CCR particles to settle out before the overflow was transferred to the next pond. The overflow from Pond 4 was discharged to the NPDES outfall located on the Discharge Channel which consisted of a 24-inch diameter high density polyethylene (HDPE) pipe. The NPDES outfall was made inactive prior to the 2017 Annual Inspection (Ref. [11]) and reportedly grouted (Ref. [13]). A portion of the NPDES outfall has since been reactivated to provide outflow for treated water during current excavation activities. Additionally, two 18-inch diameter HDPE outflow pipes connected Pond 4 to the Discharge Channel serving as emergency outflow pipes and have been abandoned. Further details of the outfall structures are as discussed in Section 2.6.

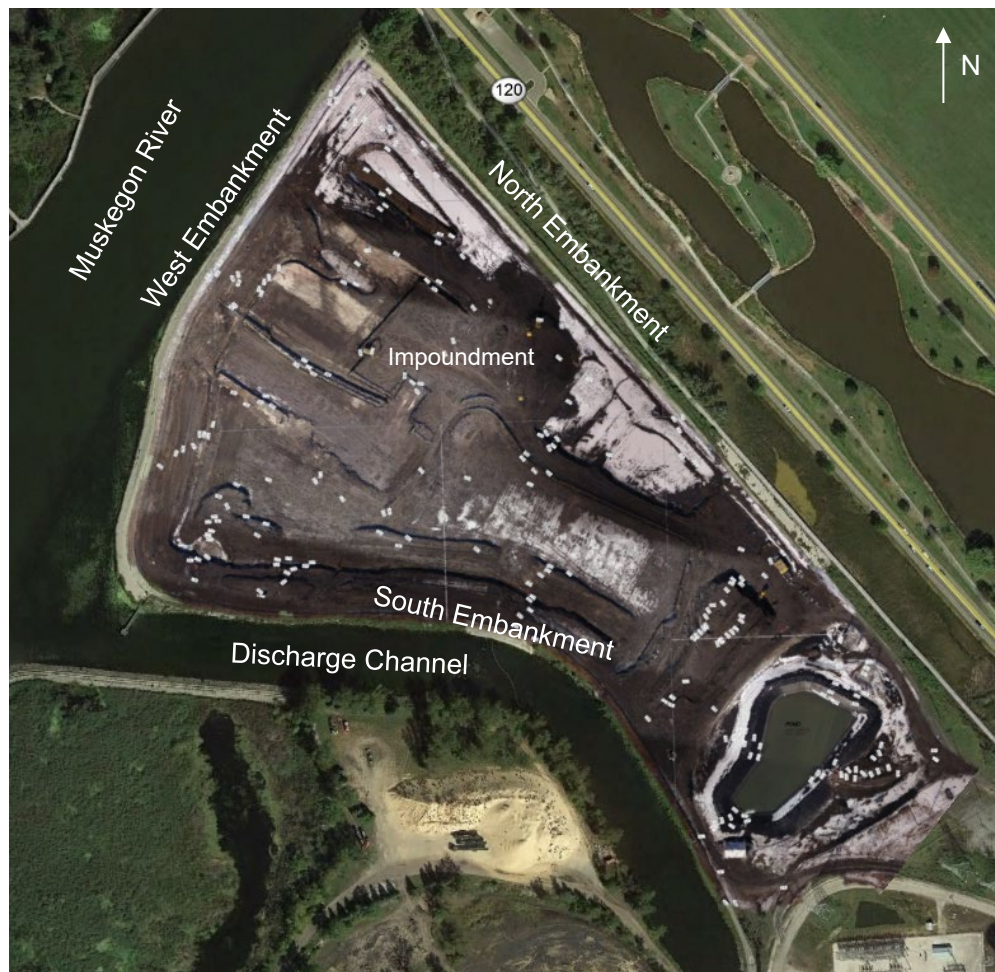
The site is in close proximity to several water bodies. The site is adjacent to the North Branch of the Muskegon River on the West Embankment, and the Veterans Memorial Pond is to the northeast of the North Embankment. The Discharge Channel is adjacent to



the South Embankment and discharges into the North Branch of the Muskegon River. There are no available original construction documents detailing the existing subgrade or embankment information at the site. Based on prior subsurface investigations performed by Golder Associates, Inc. (Golder), the perimeter embankments (collectively referring to the South, West, and North Embankments in this report) are assumed to be constructed with standard earthwork equipment and compacted and/or proof rolled before subsequent lifts based on field geotechnical testing results. The foundation material consists of native sand underlain by silty clay (Ref. [8]).

MERG initiated closure of the ponds in 2020 by installing a soil-bentonite wall in the South and West embankment adjacent to the Discharge Channel and the North Branch of the Muskegon River, respectively, to promote dewatering activities. Dewatering began in July 2020 to prepare for excavation and removal of waste CCR. Ash removal began in August 2020 and is currently ongoing. Currently, the interior embankments separating the ponds are being excavated, while the perimeter embankments are still in place adjacent to the Discharge Channel and the Muskegon River. Dewatering, excavation, and ash removal will continue throughout 2021, with expected completion in 2022.

Figure 2 presents an aerial view of the CCR impoundment as of February 2021, displaying the ongoing excavation of Ponds 0-8 and the Bottom Ash Pond as well as the terminology of the embankment sections used in this report.



**Figure 2. Aerial Image of Impoundment During Excavation**

## 1.3 Previous Assessments and Inspections

In 2009 and 2012, AECOM performed Ash Dike Risk Assessments for the impoundment system. The previous assessments have been reviewed as part of this study. Additionally, Golder previously performed annual inspections for Ponds 0-8. The Bottom Ash Pond was exempt from the inspection due to the size requirements (Ref. [4]) detailed in CCR Rule 40 CFR 257.73(b). The annual inspections were performed in accordance with 40 CFR 257.83(b), including a visual site inspection and associated reporting. The previous annual reports have been reviewed as part of this study.

Table 1-1 lists the previous reports which provide details of the annual inspections along with the date of the visual inspection.

**Table 1-1. List of Previous Assessments and Inspections**

Document Name	Date of Inspection	Reference
Inspection Report, B.C. Cobb Generating Facility, Ash Dike Risk Assessment, Muskegon, MI	August 28, 2009	Ref. [2]
B.C. Cobb Ash Disposal Area, 2012 Ash Dike Risk Assessment, Final Inspection Report	May 24, 2012	Ref. [3]
B.C. Cobb Ponds 0-8 Annual RCRA CCR Surface Impoundment Inspection Report - January 2016	October 14, 2015	Ref. [7]
B.C. Cobb Generating Facility, Pond 0-8 Structural Stability and Safety Factor Assessment Report	May 19, 2016	Ref. [8]
B.C. Cobb Ponds 0-8 2017 Annual Surface Impoundment Inspection Report	May 17, 2017	Ref. [11]
B.C. Cobb Ponds 0-8 2018 Annual Surface Impoundment Inspection Report	May 9, 2018	Ref. [13]
B.C. Cobb Ponds 0-8 2019 Annual Surface Impoundment Inspection Report	May 21, 2019	Ref. [14]

Currently, MERG performs daily visual inspections of the entire site as excavation is currently underway. The daily inspections are completed by qualified individuals to check for potentially hazardous conditions or structural weakness and the results of the inspections are documented internally on daily inspection forms.

There have been no reports of structural instability of the perimeter embankments during previous inspections.

## 2 Structural Stability Assessment - 40 CFR §257.73(d)

The requirements to be documented in the Structural Stability Assessment for existing CCR surface impoundments are detailed in 40 CFR §257.73: *Structural integrity criteria for existing CCR surface impoundments*. CCR Rule 40 CFR §257.73(d) states that the assessment must, at a minimum, document whether the CCR unit has been designed,



constructed, operated, and maintained with the items specified in 40 CFR §257.73(d)(1)(i) through (vii). Table 2-1 summarizes the information from paragraphs 40 CFR §257.73(d)(1)(i) through (vii), as well as the location of the information presented in this document.

**Table 2-1. List of Structural Stability Assessment Items**

40 CFR Rule	Rule Information	Document Section
§257.73 (d)(1)(i)	Foundations and Abutments	Section 2.1
§257.73 (d)(1)(ii)	Slope Protection	Section 2.2
§257.73 (d)(1)(iii)	Embankment/Dike Compaction	Section 2.3
§257.73 (d)(1)(iv)	Embankment/Dike Vegetation	Section 2.4
§257.73 (d)(1)(v)	Spillway	Section 2.5
§257.73 (d)(1)(vi)	Hydraulic Structures	Section 2.6
§257.73 (d)(1)(vii)	Downstream Slope Drawdown	Section 2.7
§257.73 (d)(2)	Structural Stability Deficiencies	Section 2.8

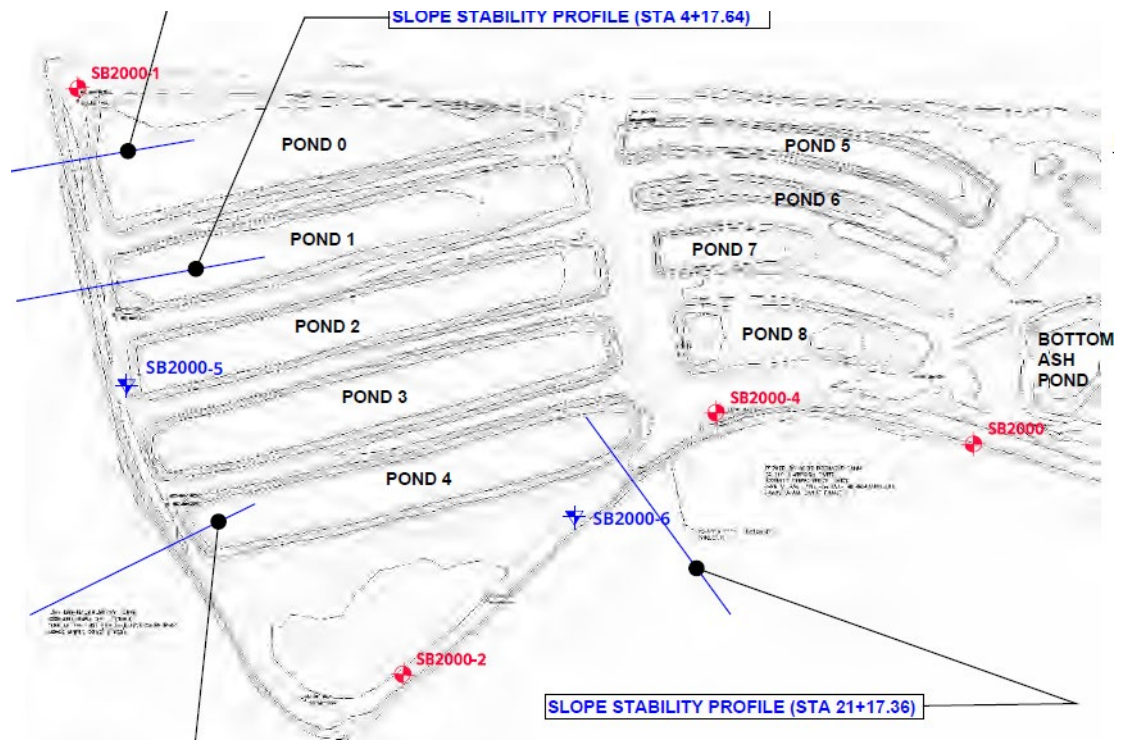
## 2.1 §257.73 (d)(1)(i) - Foundations and Abutments

### **§257.73 (d)(1)(i): *Stable foundations and abutments.***

There are no certified construction documents available that provide information on the foundations of the embankments prior to construction.

A recent subsurface investigation program in 2020 by SME (Ref. [18]) which consisted of six geotechnical borings (SB-2000-1 thru SB-2000-6) performed through the perimeter embankment to assess slope stability during dewatering activities. Other subsurface investigations have been performed, including geoprobes and monitoring well installation, however, there are no geotechnical engineering properties provided from those explorations.

The approximate locations of the borings performed by SME are shown on Figure 3 of SME 2020 (Ref. [18]) and also in Figure 3 below. The boring logs are provided in Attachment 1.



**Figure 3. Approximate Boring Locations**

The recent boring logs prepared by SME 2020 indicate that the foundation of the perimeter embankments (top of the natural sands encountered in the borings) ranges from approximately El. 573 to 582 feet. The foundation material is comprised primarily by alternating layers of granular material (i.e. sand, silty sand, sandy silt) from the foundation surface to depths of approximately 25 to 33 feet (approximately El. 543 to 547 feet ) below the original foundation grade where it is underlain by cohesive material consisting of silty clay and lean clay. Traces of silt and gravel, organic clay, peat, and organic matter were observed in the alternating sand and silt layers.

Field density tests performed on the granular material (Standard Penetration Test (SPT) blow counts) indicated that the granular soils ranged from very loose to dense, with blow counts ranging from 0 to 41 blows per foot. Undrained shear strengths obtained from field estimates with a hand penetrometer or torvane in the cohesive soils ranged from 400 psf (soft) to 2,100 psf (stiff). Moisture contents in the cohesive soils ranged between 22 and 79 percent.

The previous subsurface investigation documentation along with the history of no instability in the foundation material indicates that the foundation is competent and stable. The assessment of abutment stability required by the CCR Final Rule is not applicable, as the embankments are continuous.

## 2.2 §257.73 (d)(1)(ii) - Slope Protection

***§257.73 (d)(1)(ii): Adequate slope protection to protect against surface erosion, wave action, and adverse effects of sudden drawdown.***

High winds developing in Muskegon Lake create large waves which have the potential to reach the exterior slopes of the perimeter embankment. The exterior slopes of the

perimeter embankment are protected from erosion and deterioration by riprap and vegetative cover. The interior slopes are currently graded and consist of soil/CCR surface, however, the interior of the impoundment has been dewatered and water is not being impounded. As such, sudden draw down was not considered a failure mechanism for the interior slopes.

The crest of the perimeter embankment consists of a gravelly/soil surface. Due to the ongoing excavation activities, the road on the crest of the embankment is graded and maintained.

Weekly inspections performed by MERG monitor the existing slopes for erosion, depressions, cracks, animal burrows, ruts, holes, and seepage. There have been no observations of erosion and/or sloughing along the slopes of the perimeter embankment during the weekly inspections or the 2021 Annual Inspection performed by HDR (Ref. [15]).

The existing slope protection measures for the perimeter embankment are generally considered adequate to provide protection against surface erosion, wave action, and adverse effects of sudden drawdown. The March 2021 inspection performed by HDR (Ref. [15]) did not identify any other concerns relating to slope protection that required investigation or repair.

## 2.3 §257.73 (d)(1)(iii) - Embankment Compaction

***§257.73 (d)(1)(iii): Dikes mechanically compacted to a density sufficient to withstand the range of loading conditions in the CCR unit.***

Construction drawings and specifications, including compaction records, for the perimeter embankment were unavailable for review,

Based on the previous subsurface investigation collected at the site (Ref. [6] and Ref. [18]), the perimeter embankments were likely constructed to typical embankment standards using earthwork equipment and compacted. The slope stability analyses discussed in Section 3 provides additional details on the stability of the perimeter embankment.

Based on the information above, along with previous inspections and the recent annual inspection, the perimeter embankment is sufficient to withstand the range of loading conditions in the CCR unit.

## 2.4 §257.73 (d)(1)(iv) - Embankment Vegetation

***§257.73 (d)(1)(iv): Vegetated slopes of dikes and surrounding areas not to exceed a height of six inches above the slope of the dike, except for slopes which have an alternate form or forms of slope protection.***

Vegetation was evident on the exterior slopes of the South and North embankments, in addition to stone riprap on the South Embankment. The vegetation was overgrown and exceeded a height of 6-inches at the time of the HDR March 2021 inspection (Ref. [15]).

## 2.5 §257.73 (d)(1)(v) – Spillway

***§257.73 (d)(1)(v): A single spillway or a combination of spillways configured as specified in paragraph (d)(1)(v)(A) of this section. The combined capacity of all***

***spillways must be designed, constructed, operated, and maintained to adequately manage flow during and following the peak discharge from the event specified in paragraph (d)(1)(v)(B) of this section.***

***(A) All spillways must be either:***

- (1) Of non-erodible construction and designed to carry sustained flows; or***
- (2) Earth- or grass-lined and designed to carry short-term, infrequent flows at non-erosive velocities where sustained flows are not expected.***

***(B) The combined capacity of all spillways must adequately manage flow during and following the peak discharge from a:***

- (1) Probable maximum flood (PMF) for a high hazard potential CCR surface impoundment; or***
- (2) 1000-year flood for a significant hazard potential CCR surface impoundment; or***
- (3) 100-year flood for a low hazard potential CCR surface impoundment.***

Historically, there are no spillways associated with the impoundment. Additionally, the impoundment is not impounding water and is actively being dewatered and decommissioned.

## 2.6 §257.73 (d)(1)(vi) - Hydraulic Structures

***§257.73 (d)(1)(v): Hydraulic structures underlying the base of the CCR unit or passing through the dike of the CCR unit that maintain structural integrity and are free of significant deterioration, deformation, distortion, bedding deficiencies, sedimentation, and debris which may negatively affect the operation of the hydraulic structure.***

Prior to decommissioning the impoundment system, the pond network discharged from Pond 4 through the South Embankment via one 24-inch diameter HDPE outflow pipe to the permitted NPDES outfall which was installed with a concrete headwall and endwall. The NPDES outfall was made inactive prior to the 2017 Annual Inspection (Ref. [11]). During the installation of the soil-bentonite wall, the portion of the outflow towards the interior of the site was removed and the outfall on the exterior of the site (between the Discharge Channel and the soil-bentonite wall) was kept active and connected to the dewatering system to discharge treated water to the Discharge Channel. The remaining portion of the NPDES outfall observed during the March 2021 inspection (Ref. [15]) appeared to maintain structural integrity. The NPDES outfall appeared to be free of significant deterioration, deformation, distortion, bedding deficiencies, sedimentation, and debris and HDR was not aware of deficiencies being observed in the past by MERG. The portion of the outfall that was underground was not inspected, however, there were no indications of settlement or distress of the South Embankment above the structure.

Previously, two HDPE outflow pipes (18-inch diameter) connected Pond 4 to the Discharge Channel to serve as emergency outflow pipes that extend through the South Embankment. The portions of these pipes between the soil-bentonite wall and the Discharge Channel

have been grouted and the portions of the pipes on the interior side of the soil-bentonite wall have been removed.

## 2.7 §257.73 (d)(1)(vii) - Downstream Slope Drawdown

**§257.73 (d)(1)(v): For CCR units with downstream slopes which can be inundated by the pool of an adjacent water body, such as a river, stream or lake, downstream slopes that maintain structural stability during low pool of the adjacent water body or sudden drawdown of the adjacent water body.**

The West Embankment is adjacent to the North Branch of the Muskegon River and the South Embankment is adjacent to the Discharge Channel. The North Embankment is not adjacent to a water body. The Muskegon River and the Discharge Channel both flow into the Muskegon Lake and are hydraulically connected and are assumed to have the same water level elevations.

Considering that Ponds 0-8 are classified as a significant hazard (Ref. [10]), 1000-year flood elevations are considered. The 1000-year flood elevation of Muskegon Lake was estimated by Golder (Ref. [9]) at El. 585.7 feet. The top of embankment elevation ranges from approximately El. 586 to El. 588 feet. The typical water elevation of Muskegon Lake is approximately El. 579.4 feet as estimated by nearby tide gauge data (Ref. [17]) and referenced in the 2016 Structural Stability and Safety Factor Assessment Report (Ref. [8]).

As presented in Section 3, rapid drawdown of the exterior (river side) slope is considered dropping from the 1,000-year flood elevation in Muskegon Lake (El. 585.7 feet) to the typical water elevation (El. 579.4 feet). A factor of safety (FS) of 2.4 was computed during the slope stability analyses for rapid drawdown as presented in Section 3. The computed factor of safety is compared against the minimum factor of safety of 1.0 required for rapid drawdown loading as per USACE Engineering Manual EM 1110-2-1902 guidelines (Ref. [19]).

Furthermore, the river side slopes of the South and West Embankments have adequate protection and did not reveal any sign of erosion or instability. Therefore, the downstream slope adjacent to Discharge Channel and the Muskegon River has adequate structural stability.

## 2.8 §257.73 (d)(2) - Structural Stability Deficiencies

**§257.73 (d)(1)(v): The periodic assessment described in paragraph (d)(1) of this section must identify any structural stability deficiencies associated with the CCR unit in addition to recommending corrective measures. If a deficiency or a release is identified during the periodic assessment, the owner or operator unit must remedy the deficiency or release as soon as feasible and prepare documentation detailing the corrective measures taken.**

Based on the previous weekly inspections performed by MERG and the inspection performed in March 2021 by HDR (Ref. [15]), no structural stability deficiencies were identified for the perimeter embankments.



## 3 Safety Factor Assessment - 40 CFR §257.73(e)

### 3.1 Stability Analysis Criteria

The CCR Final Rule does not stipulate the stability analysis methodology directly, although the minimum required factor of safety criteria were adopted from the U.S. Army Corp of Engineers (USACE) guidance manuals and USACE Engineering Manual EM 1110-2-1902 (Ref. [19]) is referred to by the CCR Rule as a benchmark in the dam engineering community for slope stability analyses. The methodologies in EM 1110-2-1902 were used in this assessment of the static load cases.

Safety Factor Assessment documentation requirements for existing CCR surface impoundments are detailed in 40 CFR §257.73: *Structural integrity criteria for existing CCR surface impoundments*. CCR Rule 40 CFR §257.73(e) states that:

**§257.73 (e)(1): The owner or operator must conduct an initial and periodic safety factor assessments for each CCR unit and document whether the calculated factors of safety for each CCR unit achieve the minimum safety factors specified in paragraphs (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.**

**(e)(1)(i) The calculated static factor of safety under the long-term, maximum storage pool loading condition must equal or exceed 1.50.**

**(e)(1)(ii) The calculated static factor of safety under the maximum surcharge pool loading condition must equal or exceed 1.40.**

**(e)(1)(iii) The calculated seismic factor of safety must equal or exceed 1.00.**

**(e)(1)(iv) For dikes constructed of soils that have susceptibility to liquefaction, the calculated liquefaction factor of safety must equal or exceed 1.20.**

### 3.2 Methodology

The slope stability analysis was conducted using the GeoStudio (Version 11.0.0.21118) computer program Slope/W, which uses limit equilibrium methodologies to evaluate potential rotational and sliding block failure surfaces. For a given geometry and soil profile, the program evaluates potential failure surfaces and identifies the surface exhibiting the minimum factor of safety. The Morgenstern-Price Method was used in the evaluation because it satisfies both force and moment equilibrium. The critical potential failure surface was obtained using the entry-exit search function. The factors of safety against sliding for both shallow and deep failure surfaces were determined. The shallow failure surfaces typically have lower factors of safety but are not typically an embankment safety concern since they are surficial in nature and failure of a shallow surface is not likely to result in the release of the impoundment. The “deep” failure surfaces were defined for this study as failure surfaces that penetrate the phreatic surface or penetrate at least 1/3 of the

crest width (approximately 25 feet) and, therefore, represent the most critical failure surfaces for the embankment stability.

### 3.3 Critical Cross Section Geometry

The critical section of the perimeter embankment was determined using the recent March 2021 topographic survey provide by MERG, the interpreted subsurface profile from the available borings at the site (discussed in Section 2.1), and the interpreted phreatic surface based on observations at the site and from the monitoring well records installed at the site.

One embankment section was considered as potentially being critical based on geometry, described below, and located as shown on Figure 4 and Figure 5.

- The stability cross-section chosen for the Safety Factor Assessment is located along the South Embankment. The stability location was selected due to the geometry of the slopes at this location compared to the other areas where the slopes have been bridged (i.e. material from within the impoundment has been placed adjoined to the interior slope to provide additional passive resistance to the embankment) to support slope stability (which was taken from March 2021 topographic survey provided by MERG). Due to the pond side slopes that are present in this portion of the perimeter embankment, it was deemed more critical than the other portions of the perimeter embankment alignment.



Figure 4. Location of Stability Cross-Section

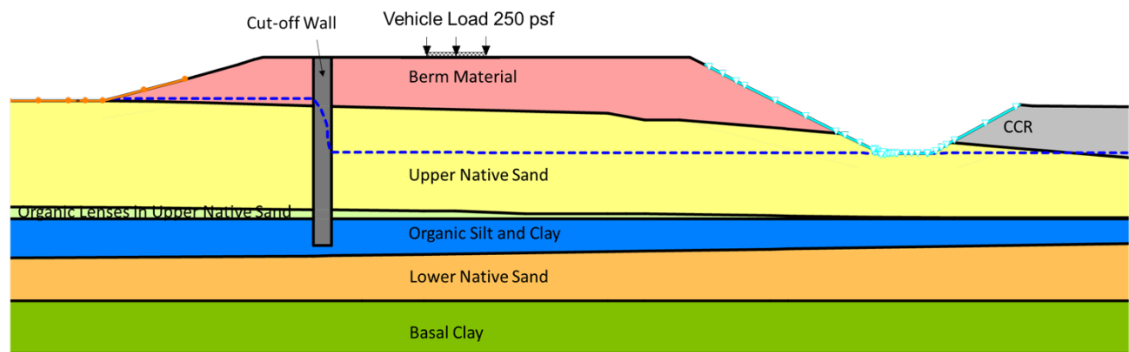


Figure 5. Stability Cross Section Geometry and Stratigraphy (phreatic surface for NWSE case)

### 3.4 Credible Load Cases

The loading conditions that were analyzed and the USEPA required minimum factors of safety are summarized in Table 3-1 below.

**Table 3-1. Loading Conditions and Minimum Required Factors of Safety**

Loading Condition	Minimum Required Factor of Safety
Maximum Storage Pool (Normal)	1.50
Maximum Surcharge Pool Loading (1,000- year flood)	1.40
Seismic <sup>1</sup>	1.00
Post-earthquake - Liquefaction <sup>2</sup>	1.20

**Notes:**

1. A Peak Ground Acceleration (PGA) = 0.034g was adopted, based on a 10 percent probability of exceedance in 50 years (475-year recurrence interval) (USGS 2018).
2. A liquefaction potential assessment was conducted and is discussed in Section 3.8. Results indicate that areas susceptible to liquefaction were not identified at the adopted level of shaking (0.0336 g at 475-year recurrence interval).

### 3.5 Pond Elevation and Phreatic Conditions

The phreatic surface for the stability model was developed based on current water level conditions of the Discharge Channel along with the current dewatered state within the impoundment. The water from the Discharge Channel was modeled as the headwater in this case as the interior of the site is no longer impounding water. The headwater was selected to be at El. 579.4 feet, the approximate Normal Water Surface Elevation (NWSE) of the Discharge Channel (Ref. [8]). The maximum pool surcharge condition was assumed to be the Muskegon Lake 1,000-year flood at El. 585.7 feet as per Golder's B.C. Cobb Generating Facility Ponds 0-8, Inflow Design Flood Control System Plan (Ref. [9]). Consideration was given to the monitoring well MW-17001R installed in the vicinity of the selected cross-section. The monitoring well measured a phreatic surface elevation of 570.4 feet when dewatering activities were active. A steady state seepage analysis was set up with the soil-bentonite wall and seepage face boundary condition on the pond side slope. The NWSE steady state seepage analysis resulted in a phreatic surface that is at Elevation 570.4 feet on the impoundment side of the cut off wall. This agrees with the observed phreatic surface in well MW-17001R at 570.4 feet.

The maximum pool surcharge scenario assumes a temporary rise of the Discharge Channel to the 1,000-year flood elevation of 585.7 feet. Similar to the NWSE case, a steady state seepage analysis was set up to determine the resulting phreatic surface to be used for proceeding stability analysis for the maximum pool surcharge scenario. Since the flood surcharge loading condition is likely to exist for a short period of time, the assumption of steady state seepage is conservative. The stability of the exterior slope during rapid drawdown of the Discharge Channel was analyzed by considering an instantaneous drop in phreatic surface from the maximum pool surcharge elevation to the NWSE.

### 3.6 Material Properties

The embankment stratigraphy is shown in Figure 5 and the material properties used for the slope stability analysis are presented in Table 3-2. The estimated material engineering

properties were based on the classifications on the encountered subsurface soils, correlations with Standard Penetration Testing (SPT), and shear strength data obtained from the soil borings.

**Table 3-2. Summary of Material Properties Used in Analysis**

Material	Dry Unit Weight (pcf)	Saturated Unit Weight (pcf)	Effective Stress Parameters	
			Drained Friction Angle, $\phi'$ (deg)	Drained Cohesion, $c'$ (psf)
<b>Berm Material</b>	90	120	31	0
<b>CCR</b>	70	112	29	0
<b>Upper Native Sand</b>	96	124	27	0
<b>Organic silt and clay with interbedded sand</b>	74	109	27	0
<b>Interbedded sands and silty sands</b>	98	124	31	0
<b>Basal Clay (Lacustrine)</b>	74	109	28	150
<b>Bentonite Soil Cutoff</b>	120	120	31	0

## 3.7 Vehicle Loading

The crest of the perimeter embankment is used as access roads around the impoundment, therefore, a vehicle load of 250 psf was used on the crest of the embankment in the stability analyses. The vehicle loading was applied to the loading conditions for the NWSE, maximum pool surcharge (flood loading), and rapid drawdown cases. The vehicle load used in the analysis is based on American Association of State Highway and Transportation Officials (AASHTO) recommended loading for *Equivalent Height of Soil for Vehicular Loading on Abutments* for maintenance trucks (Ref. [1]).

## 3.8 Assessment of Liquefaction Potential

A previous liquefaction potential assessment (Ref. [8]) utilized the Cone Penetration Test (CPT) results and the 2008 United States Geological Survey (USGS) 2475-year return period (2% probability of occurrence in 50 years life time) earthquake and concluded that the embankment and foundation soils are not susceptible to seismically induced liquefaction.

The current study evaluates the liquefaction potential for the period of dewatering and short construction time of about 2 years when the stabilized effect of accumulated water in pond cannot be realized. Considering the short duration of the dewatering period, a shorter



earthquake return period of 475 years, typically used for building structures, is considered reasonable based on understanding of risk during construction period. The probability of occurrence of the 475-year earthquake is less than 0.5% during the assumed two-year period, which is considerably lower than 2% probability for typical lifetime of 50 years.

In the current study, Borings SB2000-1 to SB2000-6 (Ref. [18]) and the most recent 2014 and 2018 USGS published data were used for assessment of liquefaction potential. These borings were drilled along the Perimeter Embankment as shown previously in Figure 3. The borings logs are provided in Attachment 1.

A “triggering analysis” was used to assess the potential for liquefaction of the embankment and foundation soils using correlations with the SPT blow counts (N) data. Based on the observed stratigraphy and blow count data, the fill above is generally medium-dense, fine to coarse silty sand or sand with silt content. Below the water elevation, both fill and foundation material density reduce such that loose to very loose material can be identified in foundation material.

The foundation soils were screened for seismically-induced liquefaction susceptibility using methods recommended by the National Center for Earthquake Research (NCEER), which uses SPT data (Ref. [16]). For liquefaction triggering analysis, the corrected SPT blow counts and soil stresses were calculated for evaluation of cyclic shear strength and stress and minimum factor of safety for each boring from the three analyzed cases were obtained. The fine contents of SM and SC material is conservatively taken based on the lower bound of USCS fine contents (12%). Similarly, fine material is taken as silt (ML) with maximum 50% fine contents for this evaluation. The analysis is conducted for each six borings located at the crest of embankment as well as at the foundational very loose, saturated sand located at the toe of embankment which has no embankment overburden weight.

The site is located at very low seismic zone according to USGS data. Using the 2014 USGS online Unified Hazard Tool (Ref. [20]), the Peak Ground Acceleration (PGA), assuming a Site Class B/C boundary was obtained as 0.014g. The earthquake magnitude is conservatively assumed 7.5. Pages 1 through 3 of Attachment 2 present a summary of the 2014 Unified Hazard Tool data. The USGS Unified Hazard Tool has not been developed for 2018, however grid data is available in the form of tables and map. Based on the site location and the interpolated 2018 data that are available for 0.05-degree grids, the PGA was found same as the year 2014 as shown on Page 4 of Attachment 2. It should be noted that the USGS PGA is defined at the rock outcrop surface and should be adjusted for overburden soil material. The presence of loose foundation material suggests that in the absence of data for upper 100 feet of foundation material, the largest amplification corresponding to Site Class E (ASCE 7-16) should be selected. As such a factor of 2.4 (ASCE 7-16) is applied to the rock PGA yielding to the liquefaction triggering analysis for earthquake level with PGA of 0.0336g and magnitude of 7.5.

The triggering analysis is based on the procedure proposed by Idriss and Boulanger (Ref. [16]). The triggering analysis requires that the raw SPT “N” values be corrected to a confining pressure of 1 ton per square foot and a drive energy of 60% efficiency (referred to as a  $(N_1)_{60}$  value). The raw SPT “N” values ( $N_{raw}$ ) presented on the boring logs were converted to  $(N_1)_{60}$  values using the following equation:

$$(N_1)_{60} = N_{RAW} C_N C_E C_B C_{RC} S$$

Where:

$$C_N = \text{Overburden Correction Factor} = (P_a/\sigma'_{vo})^{(0.784-0.0768[(N_1)_{60}]^{0.5})}$$

$$C_E = \text{Hammer Energy Correction factor} = 60\% \text{ efficient safety hammer} = 1.0$$

$$C_B = \text{Borehole Diameter Correction Factor} = 1.0$$

$$C_R = \text{Rod Length Correction Factor}$$

$$= 0.75 \quad (0-9.75 \text{ ft.})$$

$$= 0.8 \quad (9.75 \text{ to } 13 \text{ ft.})$$

$$= 0.85 \quad (13 \text{ to } 19.5 \text{ ft.})$$

$$= 0.95 \quad (19.5 \text{ to } 32 \text{ ft.})$$

$$= 1 \quad (>32 \text{ ft.})$$

$$C_S = \text{Spoon Liner Correction}$$

$$= 1.0 \quad \text{No liner was used}$$

Additional corrections were then made to correct the  $(N_1)_{60}$  value to an equivalent “clean sand” value for use in determining cyclic stress resistance (CRR), which was used for assessing triggering of liquefaction. The clean sand value,  $(N_1)_{60cs}$ , was determined based on the lowest possible fine contents from soil classification noted on the boring logs and using the method proposed by Idriss and Boulanger (Ref. [16]) and the following equation:

$$\Delta(N_1)_{60cs} = e^{(1.63+9.7/(PF+0.01)-(15.7/(PF+0.01))^2)}$$

Where:

$$PF = \text{Percent fines passing No. 200 sieve}$$

Using Idriss and Boulanger (Ref. [16]), CRR was then calculated using the following equation:

$$CRR = e^{[(N_1)_{60cs}/14.1 + ((N_1)_{60cs}/126)^2 - ((N_1)_{60cs}/23.6)^3 + ((N_1)_{60cs}/25.4)^4 - 2.8]}$$

The Cyclic Stress Ratio (CSR) was then calculated using the design earthquake. The CSR is defined as the ratio of the cyclic shear stress acting on a horizontal plane to the initial (pre-earthquake) effective or overburden stress. The PGA of 0.096g was assumed in the analysis and the distribution of CSR through the foundation cross-section was determined. The CSR was then calculated using the following equation:

$$CSR = 0.65*(a_{max}/g)*(\sigma_v/\sigma'_v)*r_d$$

Where:

$$a_{max}/g = 0.096$$

$\sigma_v$  = Total Overburden Stress

$\sigma'_v$  = Effective Overburden Stress

$$r_d = e^{(a(z) + B(z)M)}$$

Where:

$$a(z) = -1.012 - 1.126 \sin((z/11.73) + 5.133)$$

$$b(z) = 0.106 + 0.118 \sin((z/11.28) + 5.142)$$

$$M = 7.5$$

$z$  = depth in meters

Once the CSR and CRR values were calculated, the factor of safety against triggering liquefaction was calculated as:

$$FS = CRR/CSR \times MSF \times K_\sigma \times K_\alpha$$

Where:

$$MSF = \text{magnitude scaling factor} = 6.9 \cdot e^{(-M/4)} - 0.058, \leq 1.8$$

$K_\alpha$  = correction factor for the effects of an initial static shear stress ratio = 1

$K_\sigma$  = overburden correction factor = 1

Where:

$$C_\sigma = 1 / \{18.9 - 2.55 \cdot \text{SQRT}((N_1)_{60cs})\} \leq 0.3$$

$P_a$  = Pressure at 1 atmosphere

The static shear strength in the liquefaction-susceptible material is small. Therefore,  $K_\alpha$  was taken equal to one for the purpose of this analysis. If the FS is greater than 1.2, the soil is considered not susceptible to liquefaction. The calculated factor of safety against seismically-induced liquefaction is presented in on Page 5 of Attachment 2 and was calculated to be greater than 1.2 for all borings. The analysis showed that with the embankment overburden weight of 10 to 20 feet, the potential liquefaction can be excluded.

Considering that the database for the existing procedures for low level of earthquake excitation is not as vast as the cases of large earthquakes and that the sand foundation near the toe of embankment is subjected to smaller vertical effective stress, a saturated cross section is analyzed further for the loose to very loose clean sand material with  $N \leq 1$  (identified in the box on Page 6 of Attachment 2). As shown on Page 7 of Attachment 2, the results provided a FS of approximately 1.5 ( $>1.2$ ) for the assumed amplified peak ground acceleration factor of 2.4.

Because neither the embankment, nor foundation soil, were considered to be liquefiable, a pseudo static seismic stability analysis was conducted assuming no strength loss for the embankment materials. The amplification factor that accounts for the quasi-elastic response of the embankment assumed failure surface is conservatively taken equal to the amplified peak acceleration of 0.0336g.

### 3.9 Stability Analysis Results and Conclusions

Analysis summary diagrams for each loading case are provided in Attachment 3. Table 3-3 below also summarizes the results of the analyses conducted for each loading case.

As presented in Table 3-3, the factors of safety against slope instability for deep failure surfaces that are capable of breaching the embankment satisfy the requirements of the CCR Final Rule under all loading conditions.

**Table 3-3. Summary of Stability Analyses Results and Factors of Safety**

Case	River Water Surface Elevation (feet)	Required Minimum Factor of Safety	Interior Slope/ Pond Side Factor of Safety	Attachment 3 Figure Location	Exterior Slope/ River Side Factor of Safety	Attachment 3 Figure Location
Normal Water Surface Elevation	579.4	1.50	2.0	Page 1	2.0	Page 2
1000-year Flood Elevation	585.7	1.40	2.0	Page 3	2.4	Page 4
Pseudo-Static Seismic Stability	579.4	1.00	1.8	Page 5	1.7	Page 6
Rapid Drawdown	Flood to NWSE	1.00	--	--	2.4	Page 7

## 4 Certification

Based on the information provided to HDR by MERG, information available on MERG's CCR website, and HDR's visual observations and analyses, this Structural Stability Assessment and Safety Factor Assessment was conducted in accordance with the requirements of the USEPA 40 CFR Parts §257 and §261 Hazardous and Solid Waste Management System; Disposal of Coal Combustion Residuals from Electric Utilities; Final Rule, April 17, 2015 (CCR Final Rule). Based on the information currently available, I certify to the best of my knowledge, information and belief that this Structural Stability Assessment and Safety Factor Assessment meets the requirements of CCR Rule §257.73(d) and (e) in accordance with professional standards of care for similar work.



Bryce Burkett, P.E.  
Senior Geotechnical Project Manager



Greg Shafer, P.E.  
Geotechnical Engineer

29 Jun 2021



## 5 References

- Ref. [1]* American Association of State Highway and Transportation Officials (AASHTO), Load Resistant Factor Design (LFRD) Bridge Design Specifications, 2012.
- Ref. [2]* AECOM. Inspection Report, B.C. Cobb Generating Facility, Ash Dike Risk Assessment, Muskegon, MI. December 8, 2009.
- Ref. [3]* AECOM. B.C. Cobb Ash Disposal Area, 2012 Ash Dike Risk Assessment, Final Inspection Report. July 2012.
- Ref. [4]* Consumers Energy Company. B.C. Cobb History of Construction Ponds 0-8. October 17, 2016.
- Ref. [5]* Environmental Protection Agency, 40 CFR Parts §257 and §261; Hazardous and Solid Waste Management System; Disposal of Coal Combustion Residuals from Electric Utilities; Final Rule, Washington D.C., April 2015.
- Ref. [6]* Golder Associates, Inc. B.C. Cobb Plant Ash Pond Characterization, November 30, 2015.
- Ref. [7]* Golder Associates, Inc. B.C. Cobb Generating Facility, Ponds 0-8, Annual RCRA CCR Surface Impoundment Inspection Report – January 2016, January 15, 2016.
- Ref. [8]* Golder Associates, Inc. B.C. Cobb Generating Facility, Ponds 0-8, Structural Stability and Safety Factor Assessment Report, October 14, 2016.
- Ref. [9]* Golder Associates, Inc. B.C. Cobb Generating Facility, Ponds 0-8, Inflow Design Flood Control System Plan, October 14, 2016.
- Ref. [10]* Golder Associates, Inc. B.C. Cobb Generating Facility, Ponds 0-8, Hazard Potential Classification Assessment Report, October 14, 2016.
- Ref. [11]* Golder Associates, Inc. B.C. Cobb Generating Facility, Ponds 0-8, 2017 Annual Surface Impoundment Inspection Report, October 12, 2017.
- Ref. [12]* Golder Associates, Inc. B.C. Cobb Generating Facility, Bottom Ash Pond and Ponds 0-8 Closure Work Plan, Muskegon, MI, May 30, 2018.
- Ref. [13]* Golder Associates, Inc. B.C. Cobb Generating Facility, Ponds 0-8, 2018 Annual Surface Impoundment Inspection Report, October 15, 2018.
- Ref. [14]* Golder Associates, Inc. B.C. Cobb Generating Facility, Ponds 0-8 2019 Annual Surface Impoundment Inspection Report, October 10, 2019.
- Ref. [15]* HDR Michigan, Inc. 2021 Annual Inspection Report, Former B.C. Cobb Power Plant, Ponds 0-8 and Bottom Ash Pond, May, 26, 2021.
- Ref. [16]* Idriss, I.M. and Boulanger, R.W., SPT-Based Liquefaction Triggering Procedures, Report No. UCD/CGM-10/02, Department of Civil and Environmental Engineering, University of California at Davis, December 2010
- Ref. [17]* National Oceanic and Atmospheric Administration (NOAA). Holland, MI - Station ID: 9087031, accessed April 2021,  
<https://tidesandcurrents.noaa.gov/stationhome.html?id=9087031>.
- Ref. [18]* SME. Geotechnical Evaluation Report, B.C. Cobb Ash Basin Dewatering and Dredging, Muskegon, Michigan. SME Project No. 083742.01, August 27, 2020.
- Ref. [19]* USACE. EM 1110-2-1902, Slope Stability, October 31, 2003.
- Ref. [20]* United States Geologic Survey, Unified Hazard Tool, accessed May 2021,  
<https://earthquake.usgs.gov/hazards/interactive/>

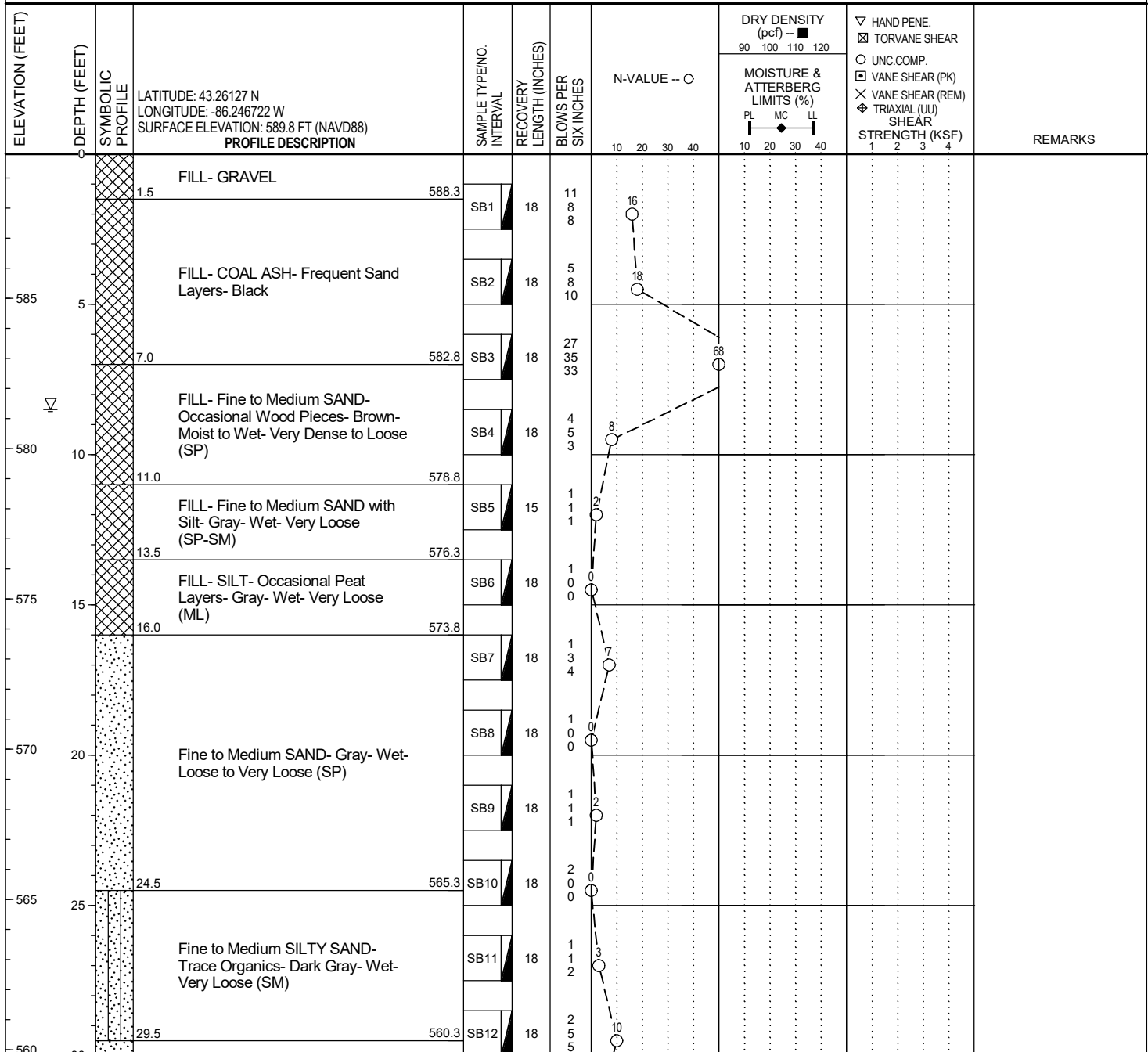
## 6 Attachments

- Attachment 1 Boring Logs
- Attachment 2 Liquefaction Analysis Figures and Results
- Attachment 3 Stability Analyses Results

**ATTACHMENT 1**  
**BORING LOGS**

**BORING SB2000-1**

PAGE 1 OF 2

**PROJECT NAME:** BC Cobb Slurry Wall Design**PROJECT NUMBER:** 083742.01**CLIENT:** HDR Michigan Inc.**PROJECT LOCATION:** North Muskegon, Michigan**DATE STARTED:** 2/24/20**COMPLETED:** 2/24/20**BORING METHOD:** Hollow-stem Augers**DRILLER:** MH (Stearns Drilling)**RIG NO.:** ATV (CME 55)**LOGGED BY:** RLS**CHECKED BY:** ATB**GROUNDWATER & BACKFILL INFORMATION**

▽ DURING BORING: DEPTH (FT) 8.5 ELEV (FT) 581.3  
▽ AT END OF BORING: Note 3

**BACKFILL METHOD:** Note 2

NOTES: 1. The indicated stratification lines are approximate. In situ, the transition between materials may be gradual.  
2. The borehole was backfilled by tremie method with bentonite and cement grout to the ground surface.  
3. An accurate groundwater level measurement was not obtained after the completion of drilling activities due to the use of grout.

(Continued Next Page)



PROJECT NAME: BC Cobb Slurry Wall Design

PROJECT NUMBER: 083742.01

CLIENT: HDR Michigan Inc.

PROJECT LOCATION: North Muskegon, Michigan

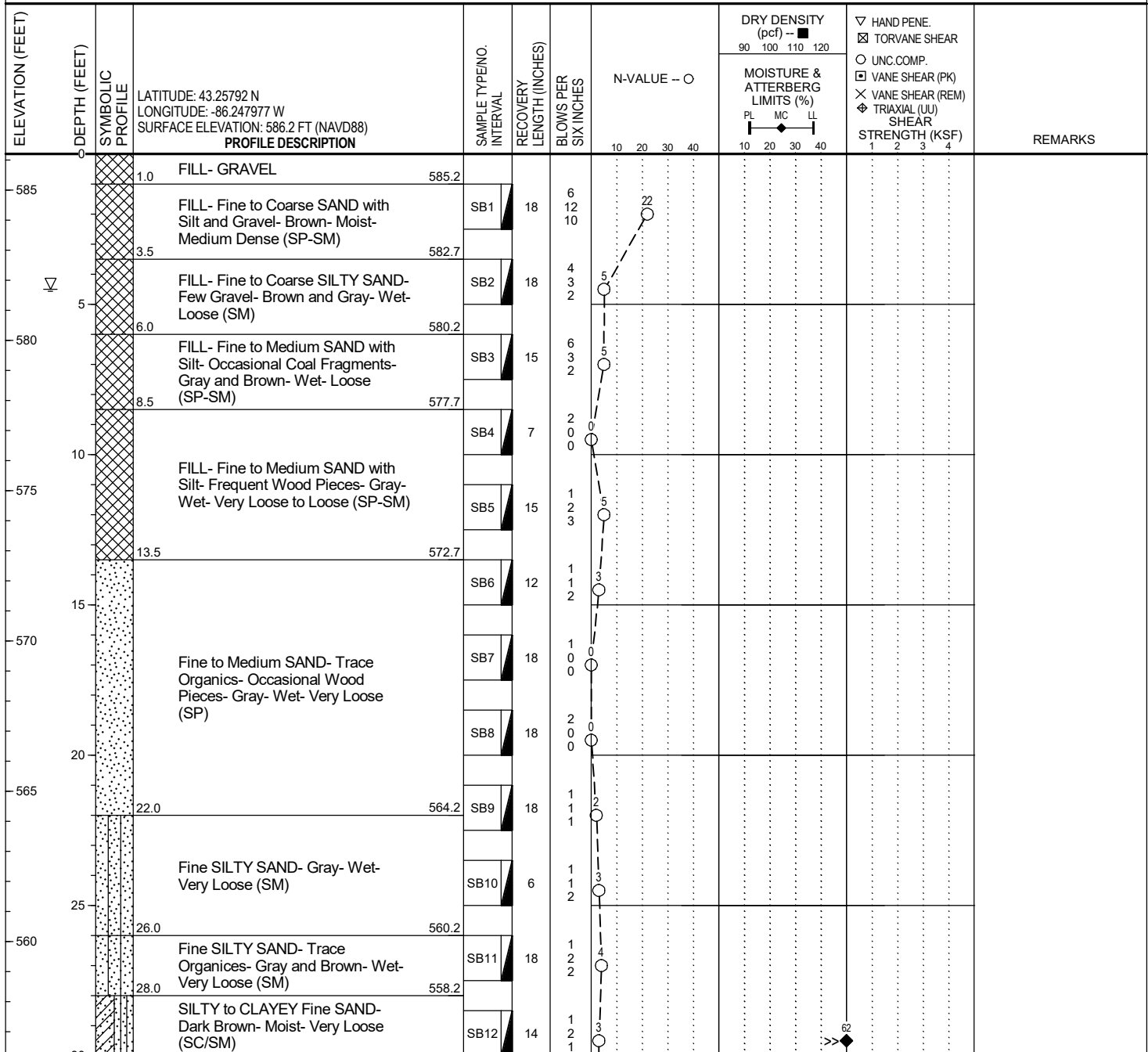
ELEVATION (FEET)	DEPTH (FEET)	SYMBOLIC PROFILE	LATITUDE: 43.26127 N LONGITUDE: -86.246722 W SURFACE ELEVATION: 589.8 FT (NAVD88) PROFILE DESCRIPTION	SAMPLE TYPE/NO. INTERVAL	RECOVERY LENGTH (INCHES)	BLOWS PER SIX INCHES	N-VALUE -- O	DRY DENSITY (pcf) -- ■ 90 100 110 120	MOISTURE & ATTERBERG LIMITS (%) PL MC LL 10 20 30 40	▽ HAND PENE. ☒ TORVANE SHEAR ○ UNC.COMP. ☐ VANE SHEAR (PK) ✕ VANE SHEAR (REM) ◆ TRIAXIAL (UU) SHEAR STRENGTH (KSF) 1 2 3 4	REMARKS
30			Fine to Medium SAND- Gray- Wet- Loose (SP) (continued)	557.8	SB13	18	1 1 1				
	32.0										
			Fine to Medium SILTY SAND- Dark Gray- Wet- Very Loose (SM)	555.8	SB14	18	1 1 2			79	
555	34.0										
			SILTY CLAY- Dark Brown- Very Soft (CL/ML)	553.8	SB15	18	2 7 13				
	36.0										
			Fine to Medium SAND- Brown and Gray- Wet- Medium Dense (SP)		SB16	18	4 6 11				
550	40										
				548.3	SB17	18	2 2 1			36 0.75	
	41.5										
			SILTY CLAY- Gray- Medium (CL/ML)		SB18	18	1 0 0			49 0.75	
545	45.0			544.8							
			END OF BORING AT 45.0 FEET.								
540	50										
535	55										
530	60										
525	65										
520	70										

Sample SB14 was too disturbed to perform a shear strength test.



**BORING SB2000-2**

PAGE 1 OF 2

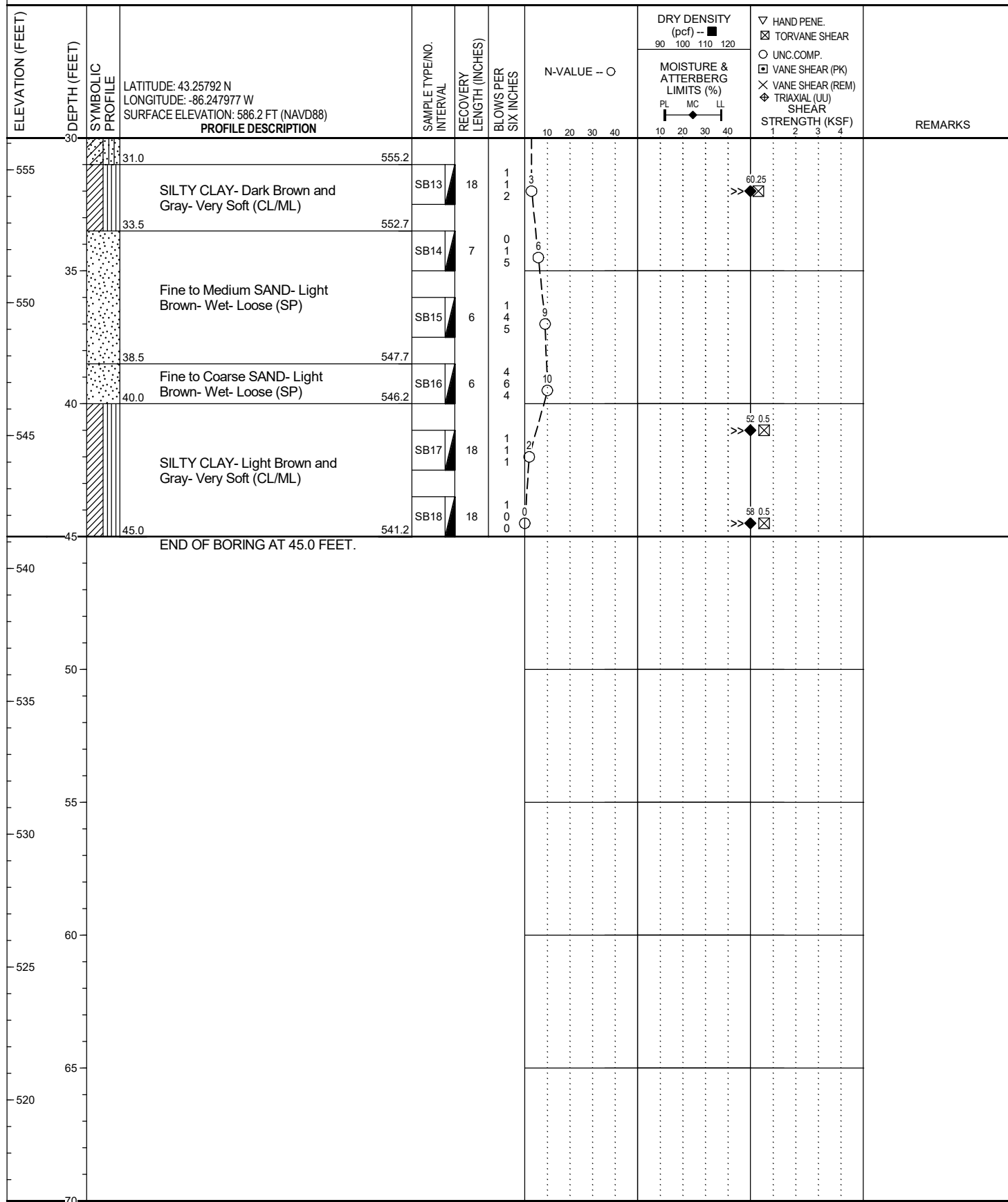
**PROJECT NAME:** BC Cobb Slurry Wall Design**PROJECT NUMBER:** 083742.01**CLIENT:** HDR Michigan Inc.**PROJECT LOCATION:** North Muskegon, Michigan**DATE STARTED:** 2/25/20**COMPLETED:** 2/25/20**BORING METHOD:** Hollow-stem Augers**DRILLER:** MH (Stearns Drilling)**RIG NO.:** ATV (CME 55)**LOGGED BY:** RLS**CHECKED BY:** ATB**GROUNDWATER & BACKFILL INFORMATION**

DEPTH (FT) ELEV (FT)  
▽ DURING BORING: 4.5 581.7  
▽ AT END OF BORING: Note 3

**BACKFILL METHOD:** Note 2

NOTES: 1. The indicated stratification lines are approximate. In situ, the transition between materials may be gradual.  
2. The borehole was backfilled by tremie method with bentonite and cement grout to the ground surface.  
3. An accurate groundwater level measurement was not obtained after the completion of drilling activities due to the use of grout.

(Continued Next Page)

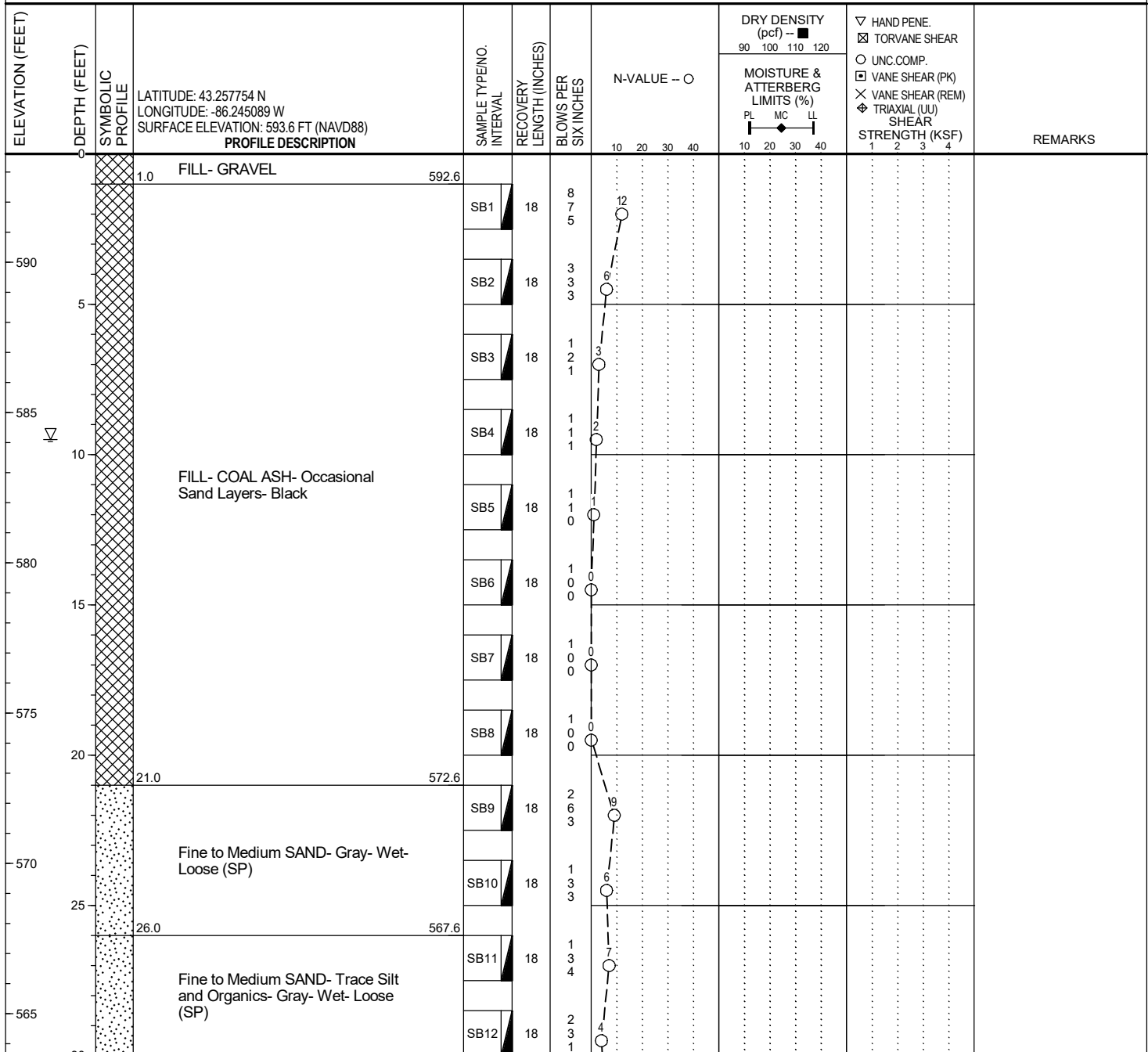


(Continued Next Page)

[illegible]

**BORING SB2000-4**

PAGE 1 OF 2

**PROJECT NAME:** BC Cobb Slurry Wall Design**PROJECT NUMBER:** 083742.01**CLIENT:** HDR Michigan Inc.**PROJECT LOCATION:** North Muskegon, Michigan**DATE STARTED:** 2/26/20**COMPLETED:** 2/26/20**BORING METHOD:** Hollow-stem Augers**DRILLER:** MH (Stearns Drilling)**RIG NO.:** ATV (CME 55)**LOGGED BY:** RLS**CHECKED BY:** ATB**GROUNDWATER & BACKFILL INFORMATION**

▽ DURING BORING: DEPTH (FT) 9.5 ELEV (FT) 584.1  
▽ AT END OF BORING: Note 3

**BACKFILL METHOD:** Note 2

NOTES: 1. The indicated stratification lines are approximate. In situ, the transition between materials may be gradual.  
2. The borehole was backfilled by tremie method with bentonite and cement grout to the ground surface.  
3. An accurate groundwater level measurement was not obtained after the completion of drilling activities due to the use of grout.

(Continued Next Page)





# BORING SB2000-5

PAGE 1 OF 2

**PROJECT NAME:** BC Cobb Slurry Wall Design

**PROJECT NUMBER:** 083742.01

**CLIENT:** HDR Michigan, Inc.

**PROJECT LOCATION:** North Muskegon, MI

**DATE STARTED:** 5/14/20

**COMPLETED:** 5/14/20

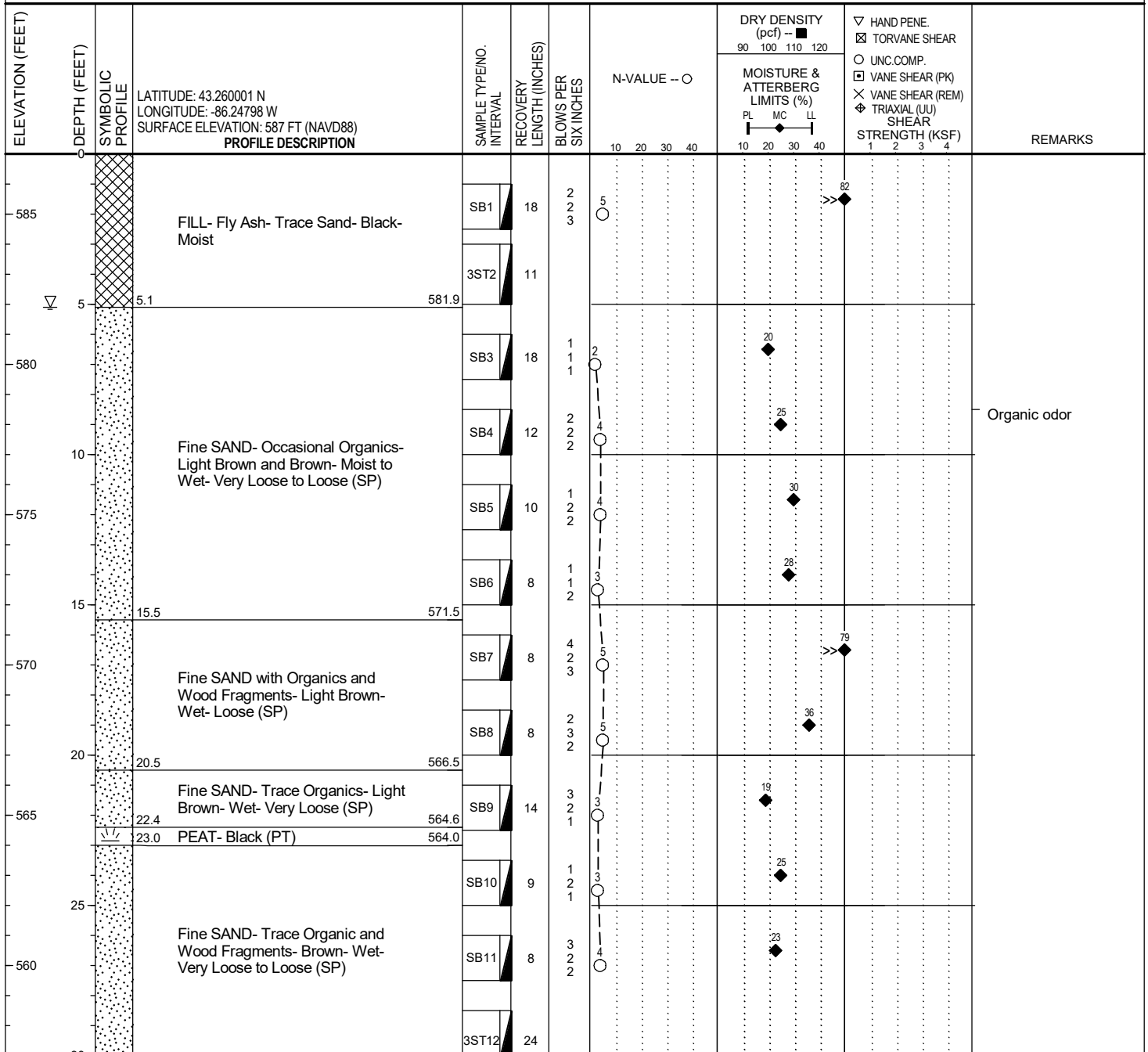
**BORING METHOD:** 4-1/4" Hollow-stem Auger

**DRILLER:** DK (Stearns Drilling)

**RIG NO.:** CME 55 LCX

**LOGGED BY:** JF

**CHECKED BY:** AJE



## GROUNDWATER & BACKFILL INFORMATION

▽ DURING BORING:      DEPTH (FT)    ELEV (FT)  
    5.1      581.9

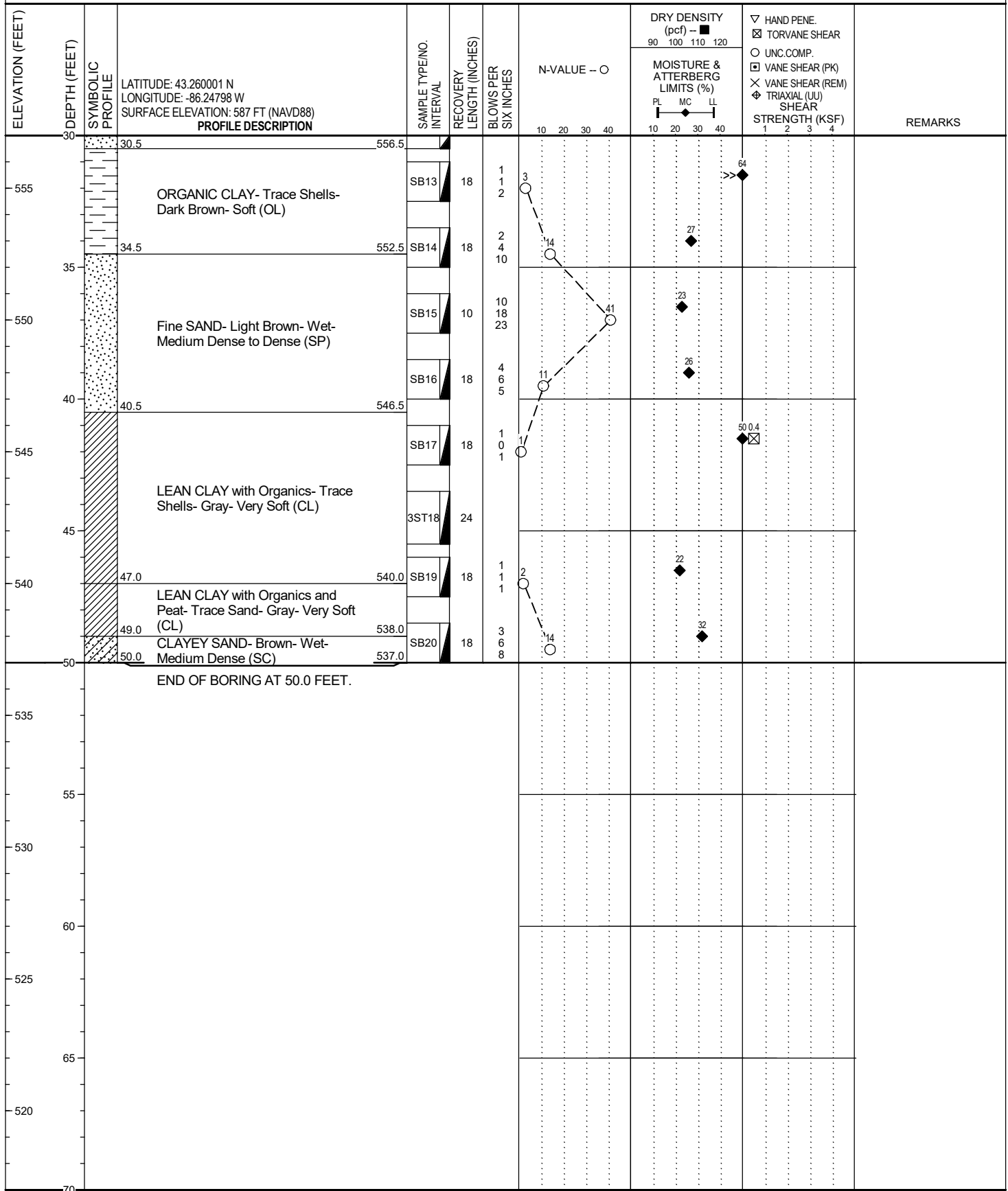
CAVE-IN OF BOREHOLE AT:      9.0      578.0

BACKFILL METHOD:    Auger Cuttings

NOTES: 1. The indicated stratification lines are approximate. In situ, the transition between materials may be gradual.

(Continued Next Page)

**PROJECT LOCATION:** North Muskegon, MI



# BORING SB2000-6

PAGE 1 OF 2

**PROJECT NAME:** BC Cobb Slurry Wall Design

**PROJECT NUMBER:** 083742.01

**CLIENT:** HDR Michigan, Inc.

**PROJECT LOCATION:** North Muskegon, MI

**DATE STARTED:** 5/15/20

**COMPLETED:** 5/15/20

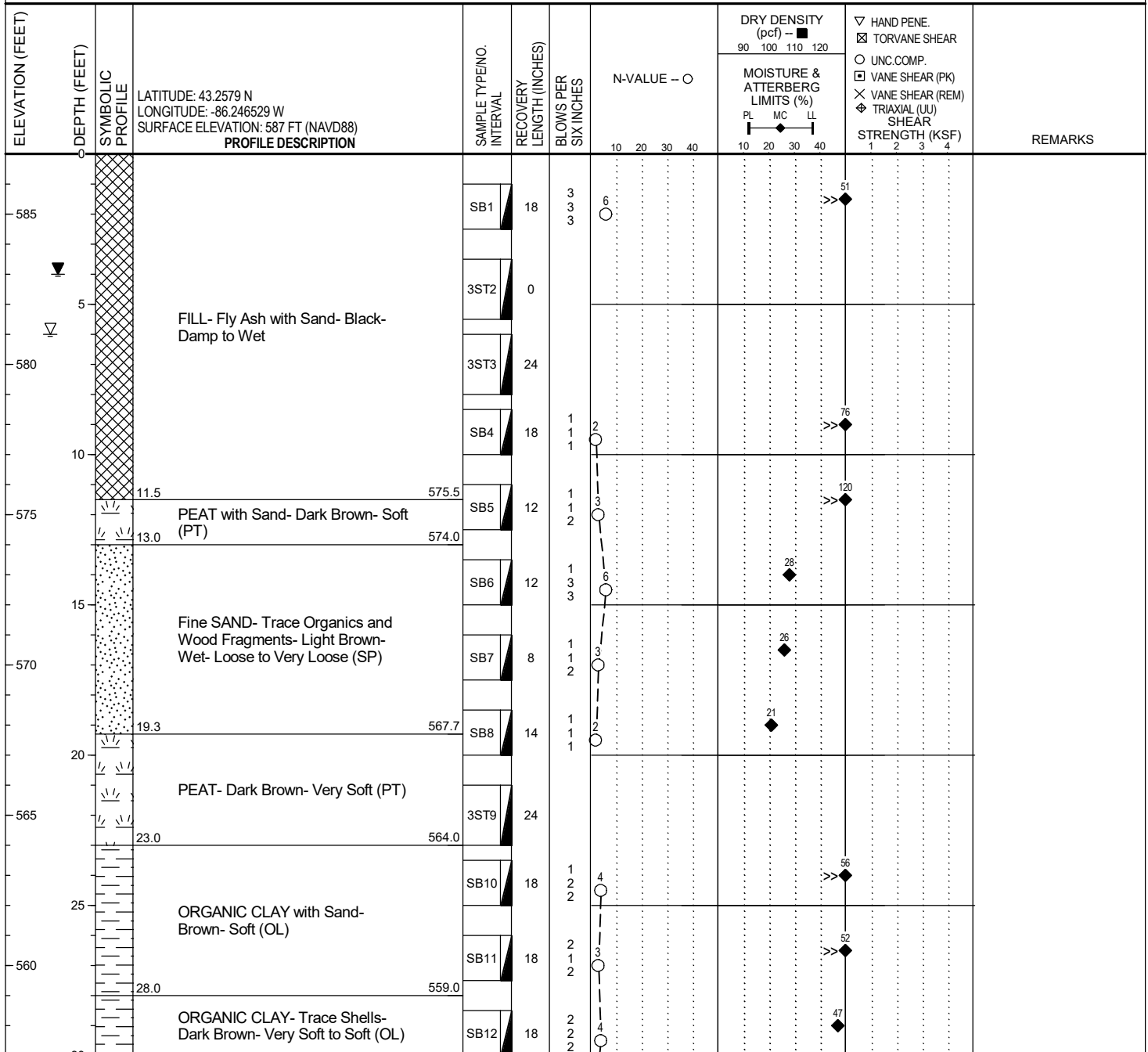
**BORING METHOD:** 4-1/4" Hollow-stem Auger

**DRILLER:** DK (Stearns Drilling)

**RIG NO.:** CME 55 LCX

**LOGGED BY:** JF

**CHECKED BY:** AJE



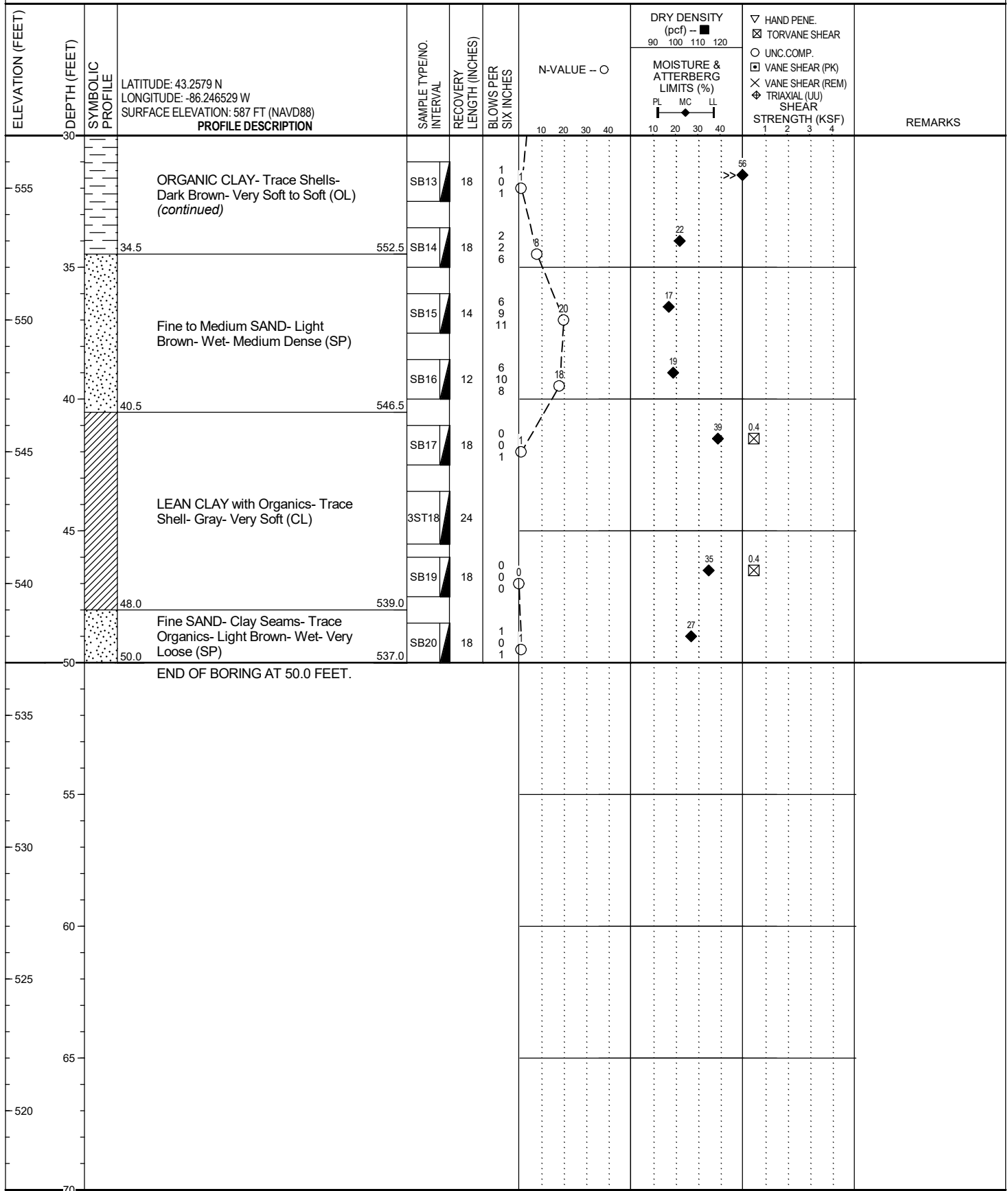
## GROUNDWATER & BACKFILL INFORMATION

	DEPTH (FT)	ELEV (FT)
▽ DURING BORING:	6.0	581.0
▽ AT END OF BORING:	4.0	583.0
CAVE-IN OF BOREHOLE AT:	13.0	574.0
BACKFILL METHOD:	Auger Cuttings	

NOTES: 1. The indicated stratification lines are approximate. In situ, the transition between materials may be gradual.

(Continued Next Page)

**PROJECT LOCATION:** North Muskegon, MI



**ATTACHMENT 2**  
**LIQUEFACTION ANALYSIS FIGURES AND RESULTS**

# Unified Hazard Tool

Please do not use this tool to obtain ground motion parameter values for the design code reference documents covered by the [U.S. Seismic Design Maps web tools](#) (e.g., the International Building Code and the ASCE 7 or 41 Standard). The values returned by the two applications are not identical.

## ^ Input

Edition

Dynamic: Conterminous U.S. 2014 (u... ▼

Spectral Period

Peak Ground Acceleration ▼

Latitude

Decimal degrees

43.259023

Time Horizon

Return period in years

475

Longitude

Decimal degrees, negative values for western longitudes

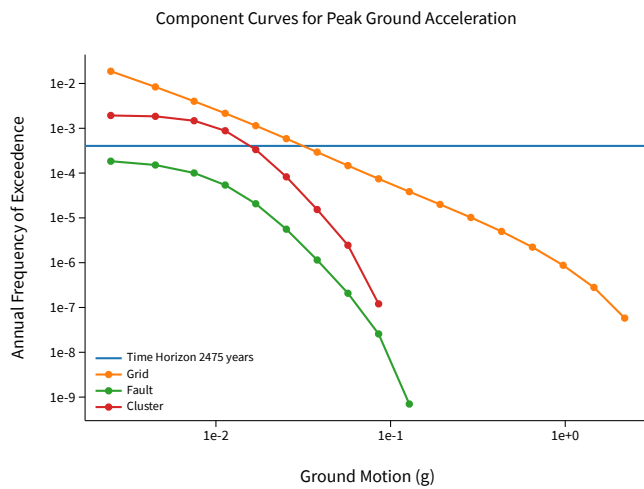
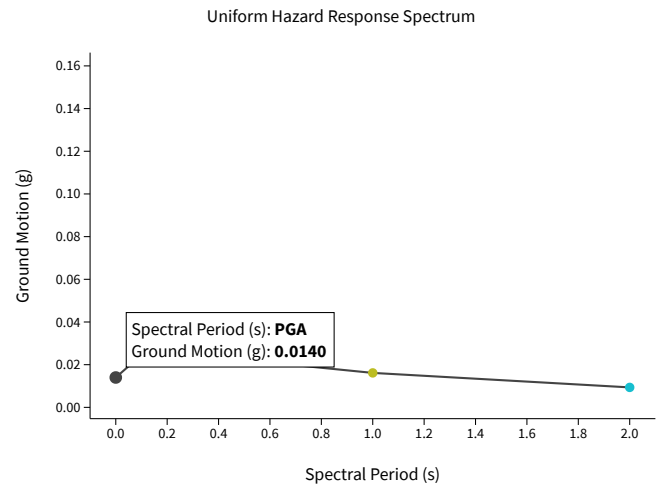
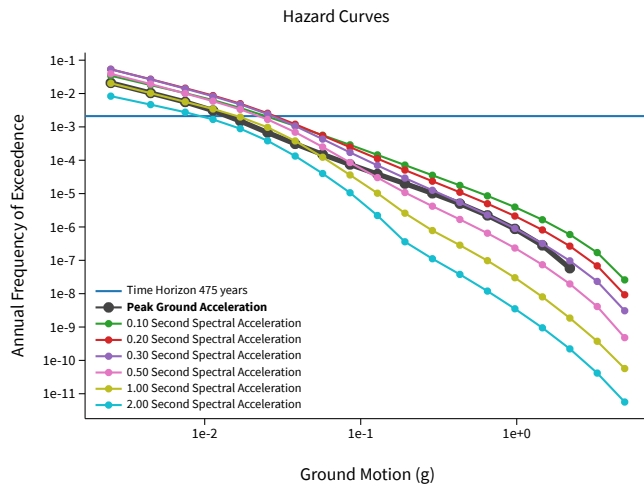
-86.246016

Site Class

760 m/s (B/C boundary) ▼



## ^ Hazard Curve

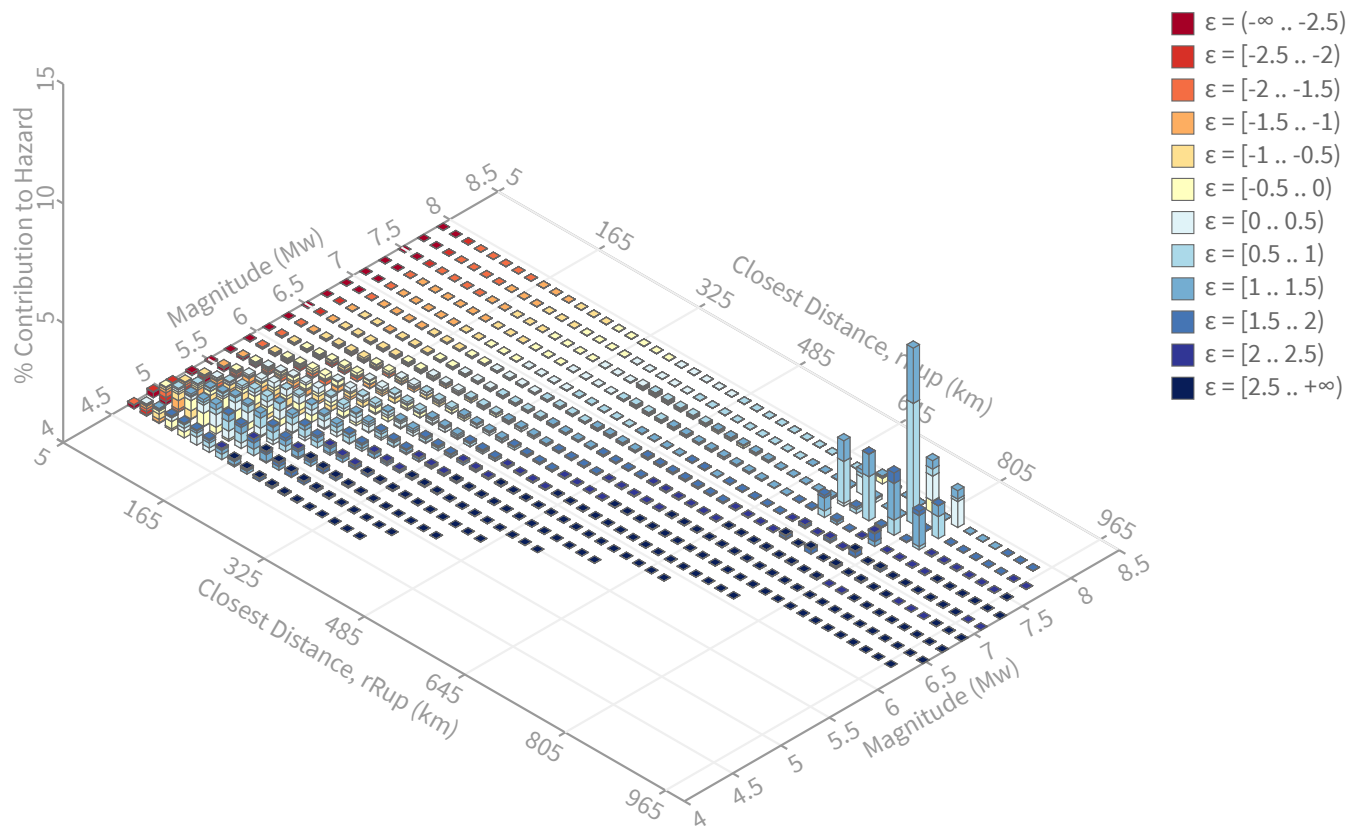


[View Raw Data](#)

## ^ Deaggregation

### Component

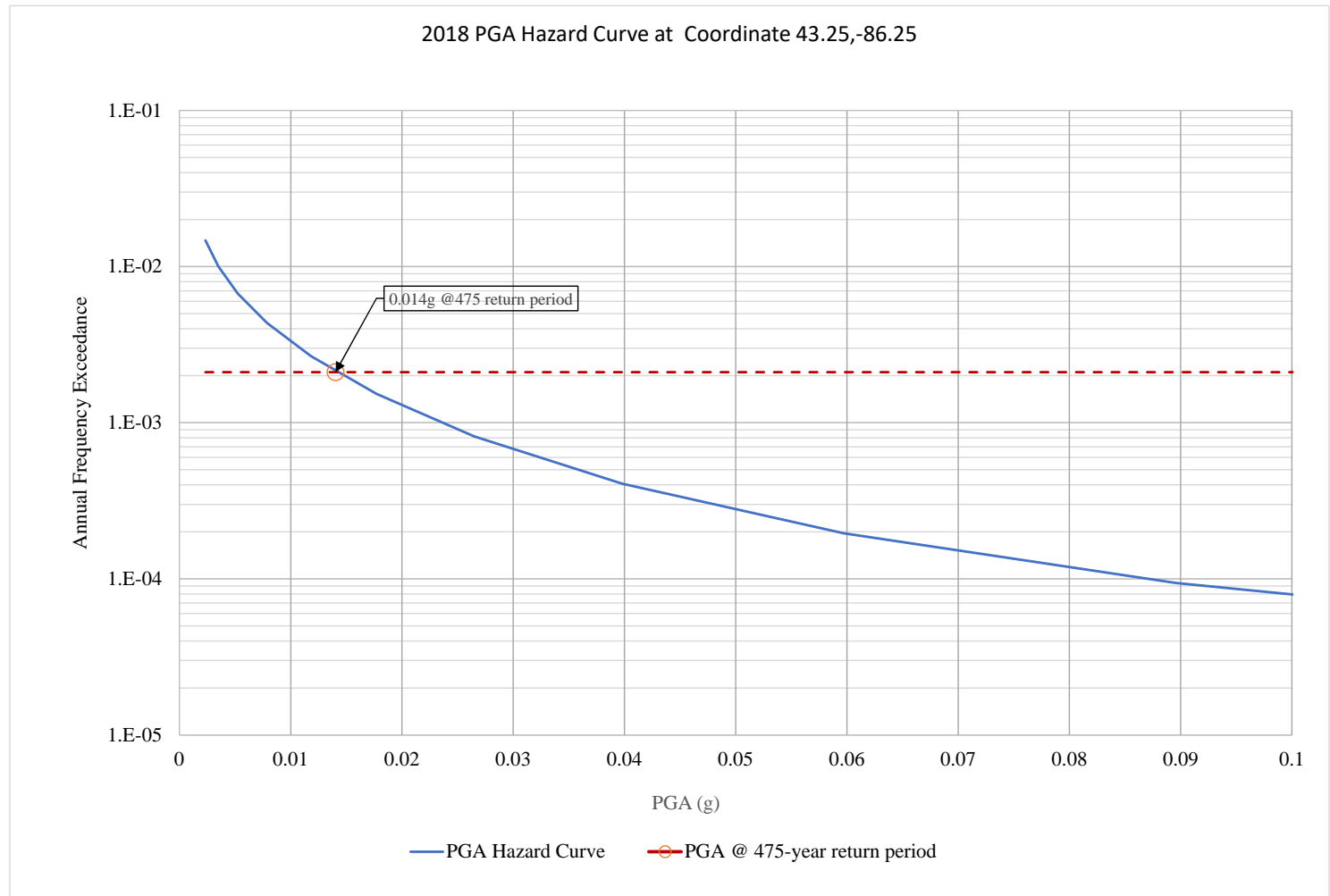
Total

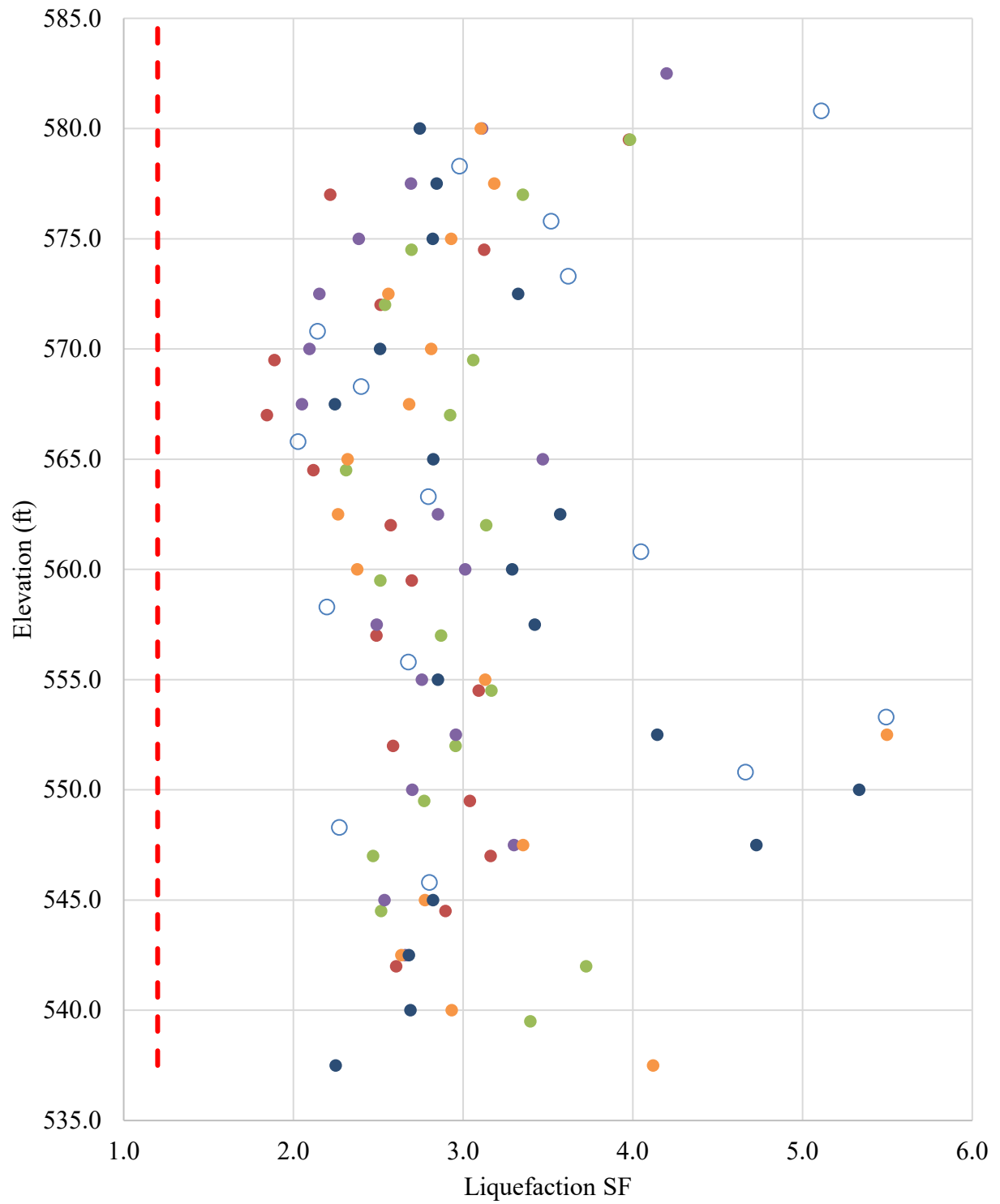


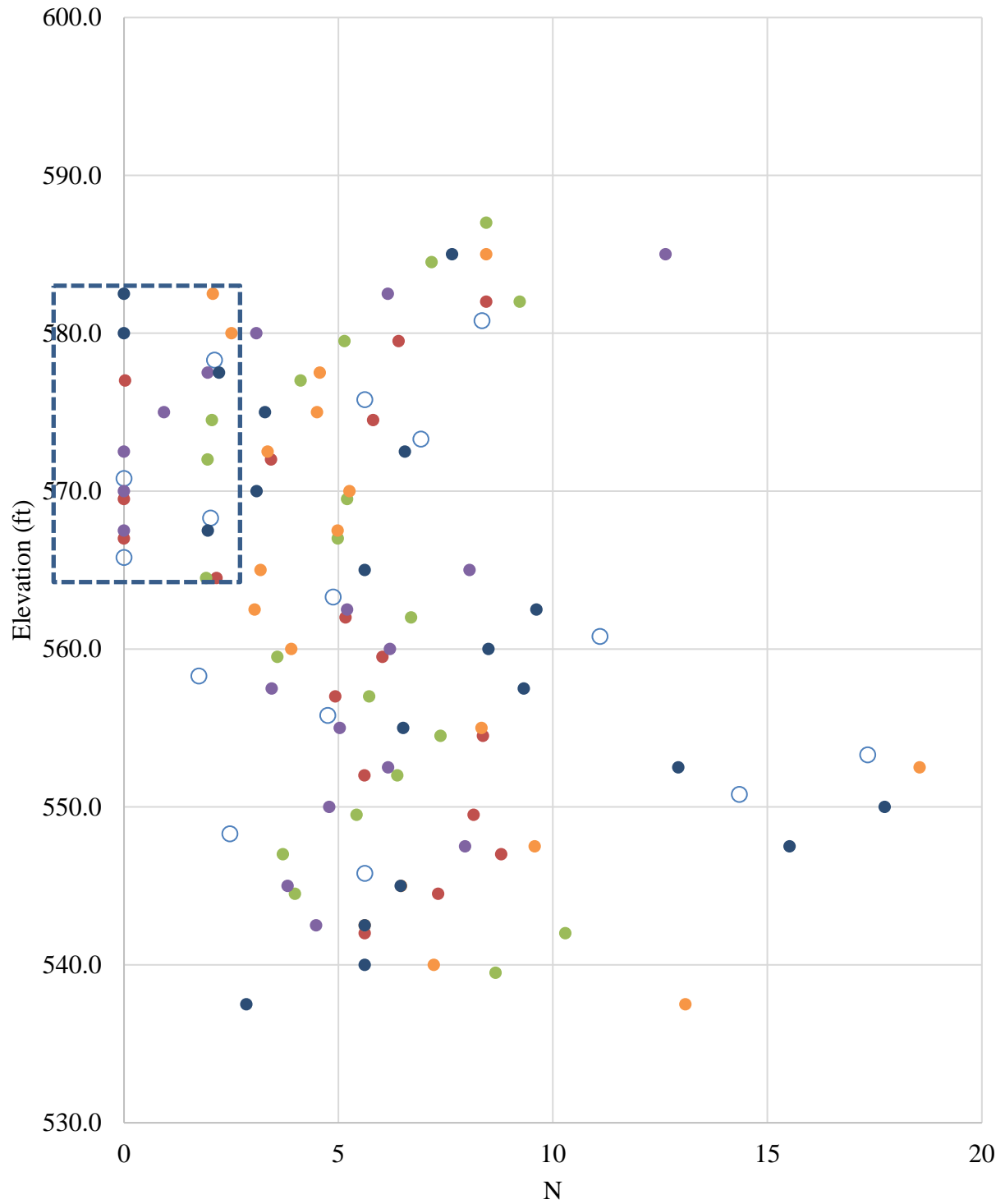
Site coordinate at 43.259023, -86.246016 take (43.25,-86.25)

PGA (g)	0.00233	0.0035	0.00524	0.00786	0.0118	0.0177	0.0265	0.0398	0.0597	0.0896	0.134	0.202	0.302	0.454	0.68
Probability of Exceed.	1.47E-02	1.00E-02	6.72E-03	4.36E-03	2.67E-03	1.53E-03	8.17E-04	4.07E-04	1.96E-04	9.40E-05	4.59E-05	2.25E-05	1.12E-05	5.44E-06	2.56E-06
Log (PGA)	-2.63E+00	-2.46E+00	-2.28E+00	-2.10E+00	-1.93E+00	-1.75E+00	-1.58E+00	-1.40E+00	-1.22E+00	-1.05E+00	-8.73E-01	-6.95E-01	-5.20E-01	-3.43E-01	-1.67E-01
Log (Prob)	-1.8326827	-2	-2.1726307	-2.3605135	-2.5734887	-2.8153086	-3.0877779	-3.3904056	-3.7077439	-4.0268721	-4.338187	-4.647817	-4.950782	-5.264401	-5.59176

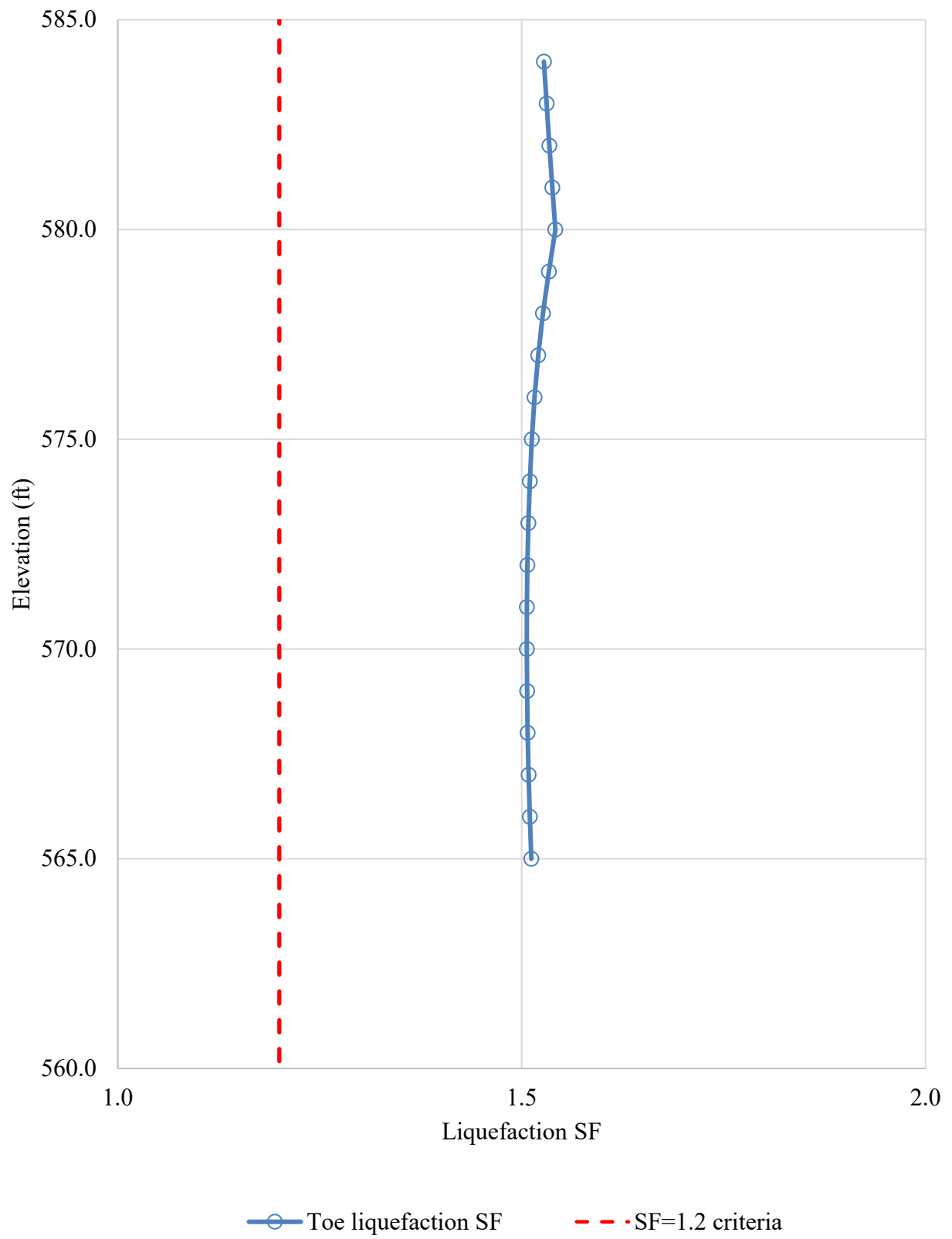
Return period 475  
 Prob. Of exceedance 0.00210526  
 Log(Prob. Of Exceed.) -2.6766936  
 Log (PGA Interpolate) -1.852965  
 PGA Interpolated 0.01402927  
 Site Class E Amplification 2.4  
 Ground Acceleration 0.03367024  
 Magnitude <5.5







○ SB-2000-1    ● SB-2000-2    ● SB-2000-3    ● SB-2000-4    ● SB-2000-5    ● SB-2000-6





**ATTACHMENT 3**  
**STABILITY ANALYSES RESULTS**



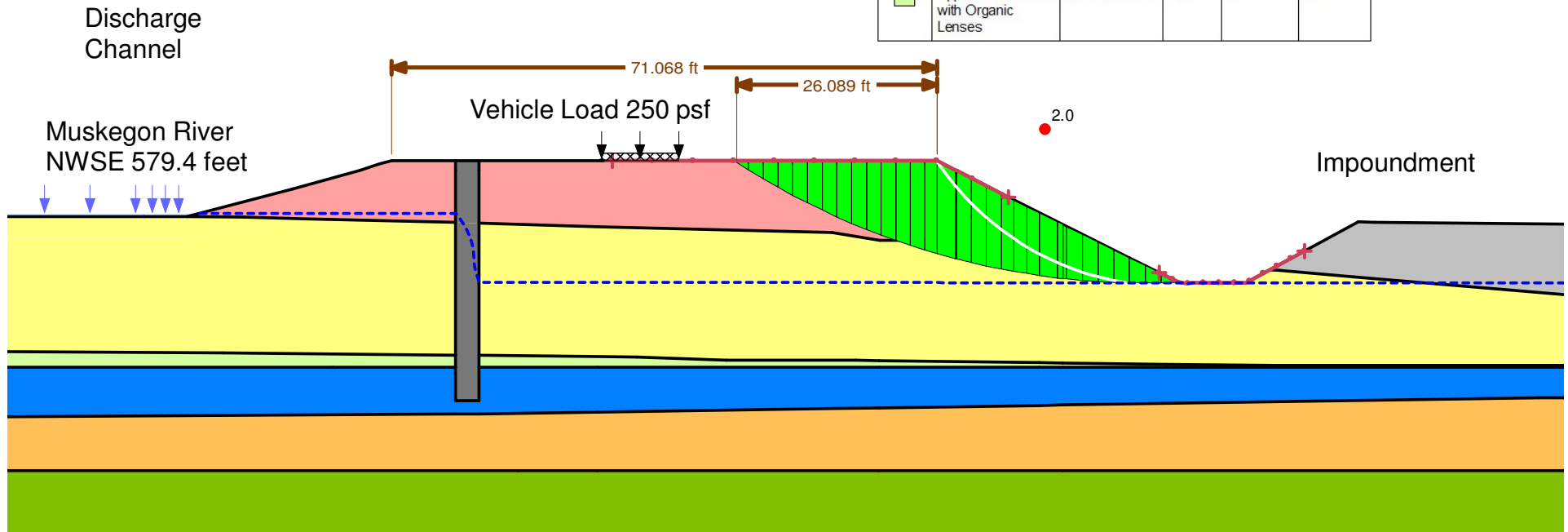
Name: B.C.Cobb Ash Pond - Critical Section

Description: Normal Water Surface Elevation- Static- Pond Side

Method: Morgenstern-Price

FS: 2.0

Color	Name	Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
■	Basal Clay	Mohr-Coulomb	109	150	28
■	Berm Material	Mohr-Coulomb	120	0	31
■	CCR	Mohr-Coulomb	112	0	29
■	Lower Native Sand	Mohr-Coulomb	124	0	31
■	Organic silt and clay with interbedded sand	Mohr-Coulomb	109	0	27
■	SBW	Mohr-Coulomb	120	0	31
■	Upper Native Sand	Mohr-Coulomb	124	0	27
■	Upper Native Sand with Organic Lenses	Mohr-Coulomb	124	0	32













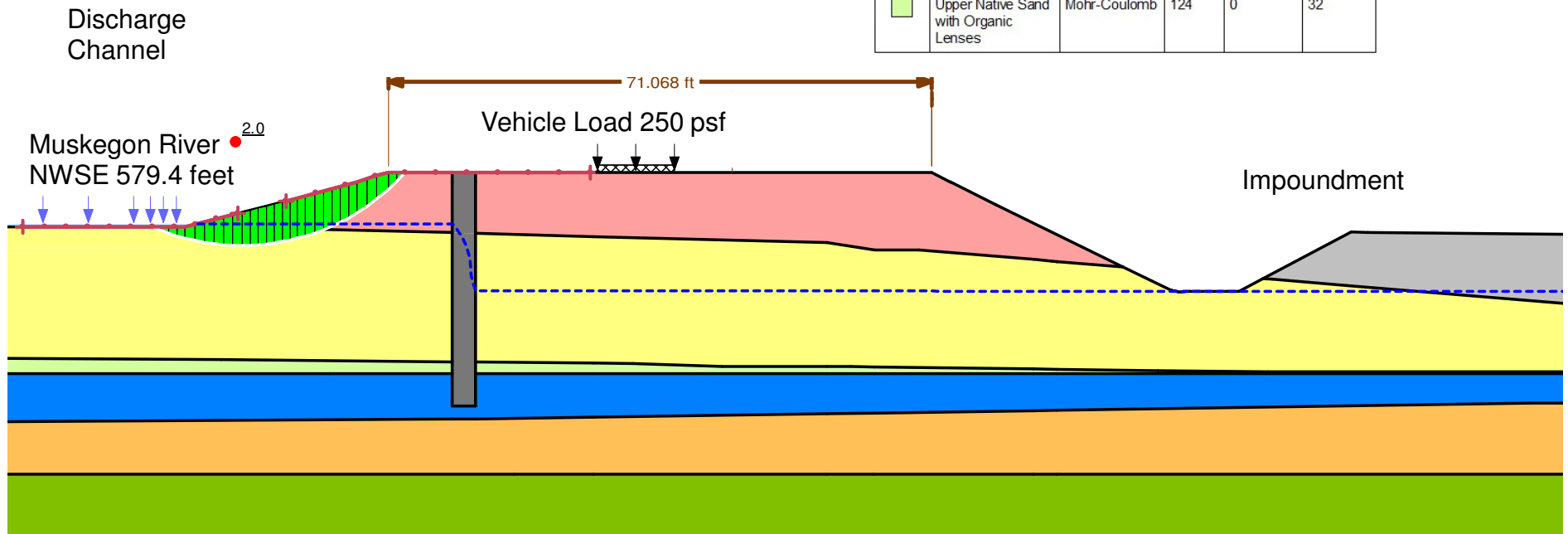
Name: B.C.Cobb Ash Pond - Critical Section

Description: Normal Water Surface Elevation- Static- River Side

Method: Morgenstern-Price

FS: 2.0

Color	Name	Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Basal Clay	Mohr-Coulomb	109	150	28
	Berm Material	Mohr-Coulomb	120	0	31
	CCR	Mohr-Coulomb	112	0	29
	Lower Native Sand	Mohr-Coulomb	124	0	31
	Organic silt and clay with interbedded sand	Mohr-Coulomb	109	0	27
	SBW	Mohr-Coulomb	120	0	31
	Upper Native Sand	Mohr-Coulomb	124	0	27
	Upper Native Sand with Organic Lenses	Mohr-Coulomb	124	0	32













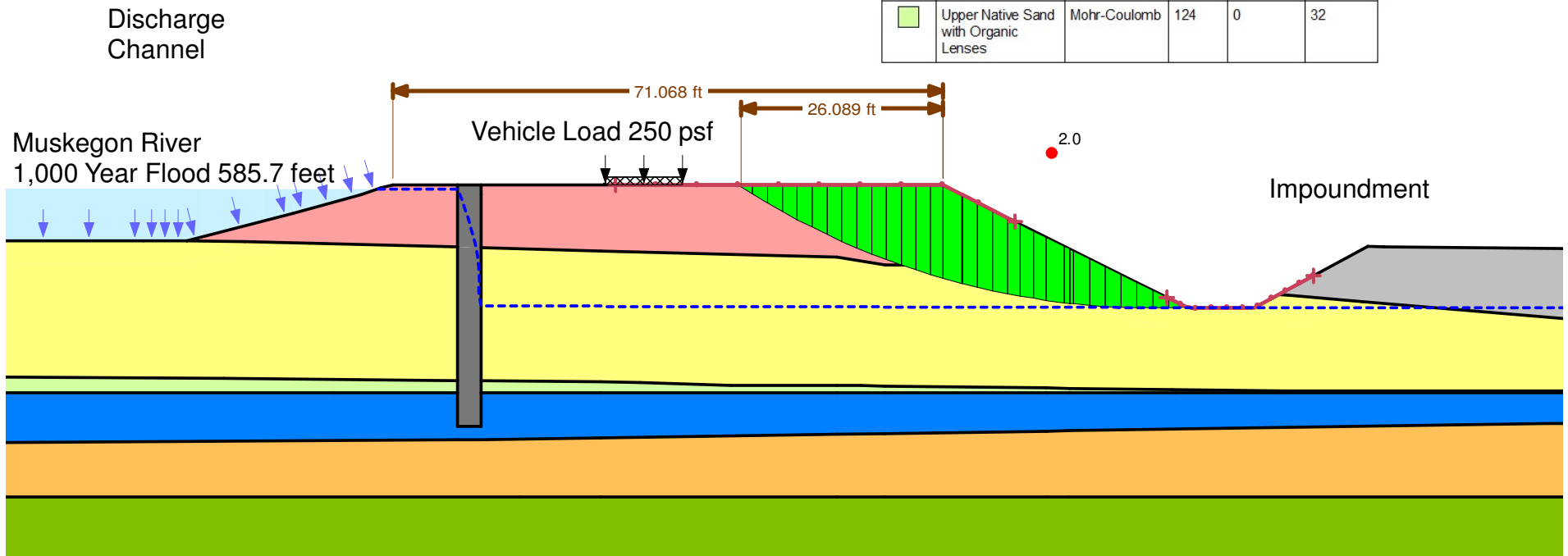
Name: B.C.Cobb Ash Pond - Critical Section

Description: Flood Water Surface Elevation- Static- Pond Side

Method: Morgenstern-Price

FS: 2.0

Color	Name	Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Basal Clay	Mohr-Coulomb	109	150	28
	Berm Material	Mohr-Coulomb	120	0	31
	CCR	Mohr-Coulomb	112	0	29
	Lower Native Sand	Mohr-Coulomb	124	0	31
	Organic silt and clay with interbedded sand	Mohr-Coulomb	109	0	27
	SBW	Mohr-Coulomb	120	0	31
	Upper Native Sand	Mohr-Coulomb	124	0	27
	Upper Native Sand with Organic Lenses	Mohr-Coulomb	124	0	32





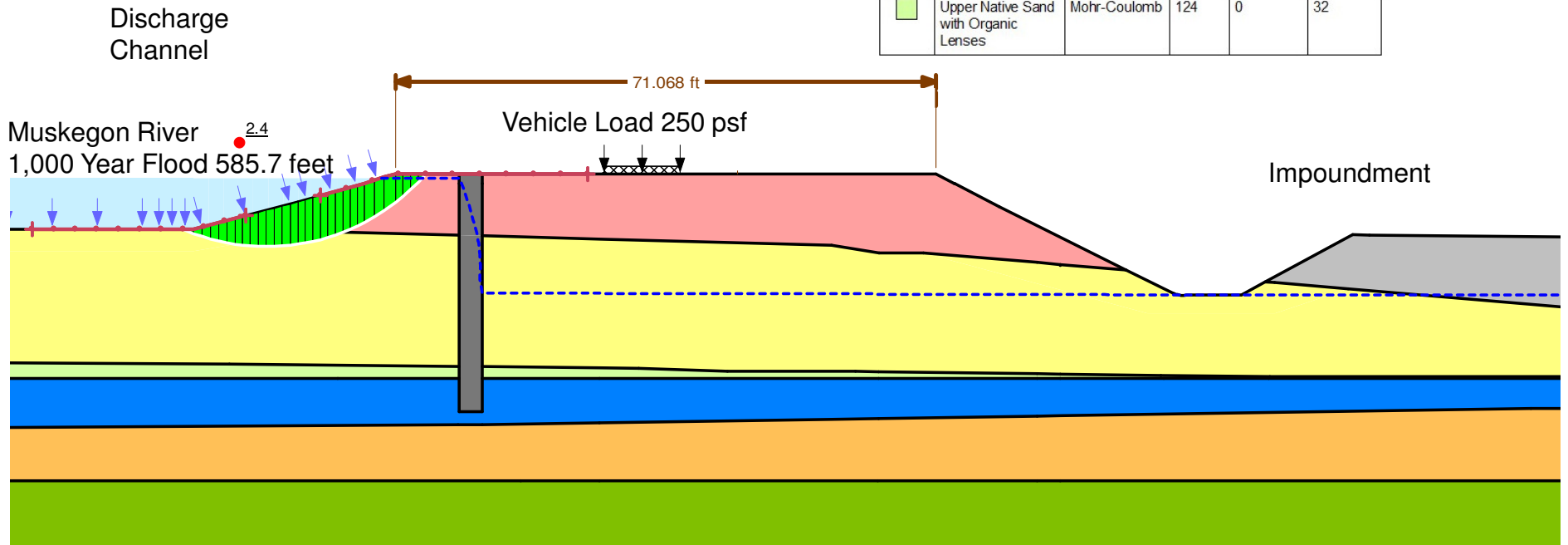
Name: B.C.Cobb Ash Pond - Critical Section

Description: Flood Water Surface Elevation- Static- River Side

Method: Morgenstern-Price

FS: 2.4

Color	Name	Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
Green	Basal Clay	Mohr-Coulomb	109	150	28
Pink	Berm Material	Mohr-Coulomb	120	0	31
Grey	CCR	Mohr-Coulomb	112	0	29
Orange	Lower Native Sand	Mohr-Coulomb	124	0	31
Blue	Organic silt and clay with interbedded sand	Mohr-Coulomb	109	0	27
Dark Grey	SBW	Mohr-Coulomb	120	0	31
Yellow	Upper Native Sand	Mohr-Coulomb	124	0	27
Light Green	Upper Native Sand with Organic Lenses	Mohr-Coulomb	124	0	32













Name: B.C.Cobb Ash Pond - Critical Section

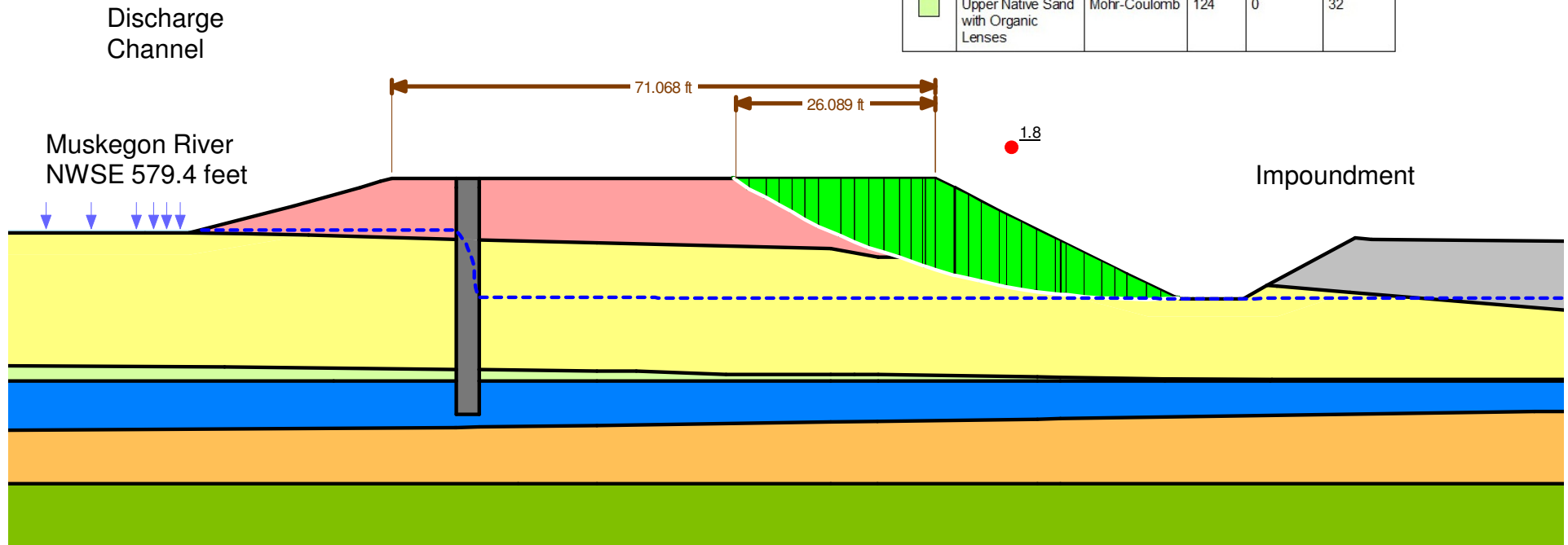
Description: Normal Water Surface Elevation- Pseudostatic- Pond Side

Acceleration:  $K_h = 0.034 \text{ g}$  (475- year return period ground motion)

Method: Morgenstern-Price

FS: 1.8

Color	Name	Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Basal Clay	Mohr-Coulomb	109	150	28
	Berm Material	Mohr-Coulomb	120	0	31
	CCR	Mohr-Coulomb	112	0	29
	Lower Native Sand	Mohr-Coulomb	124	0	31
	Organic silt and clay with interbedded sand	Mohr-Coulomb	109	0	27
	SBW	Mohr-Coulomb	120	0	31
	Upper Native Sand	Mohr-Coulomb	124	0	27
	Upper Native Sand with Organic Lenses	Mohr-Coulomb	124	0	32















Name: B.C.Cobb Ash Pond - Critical Section

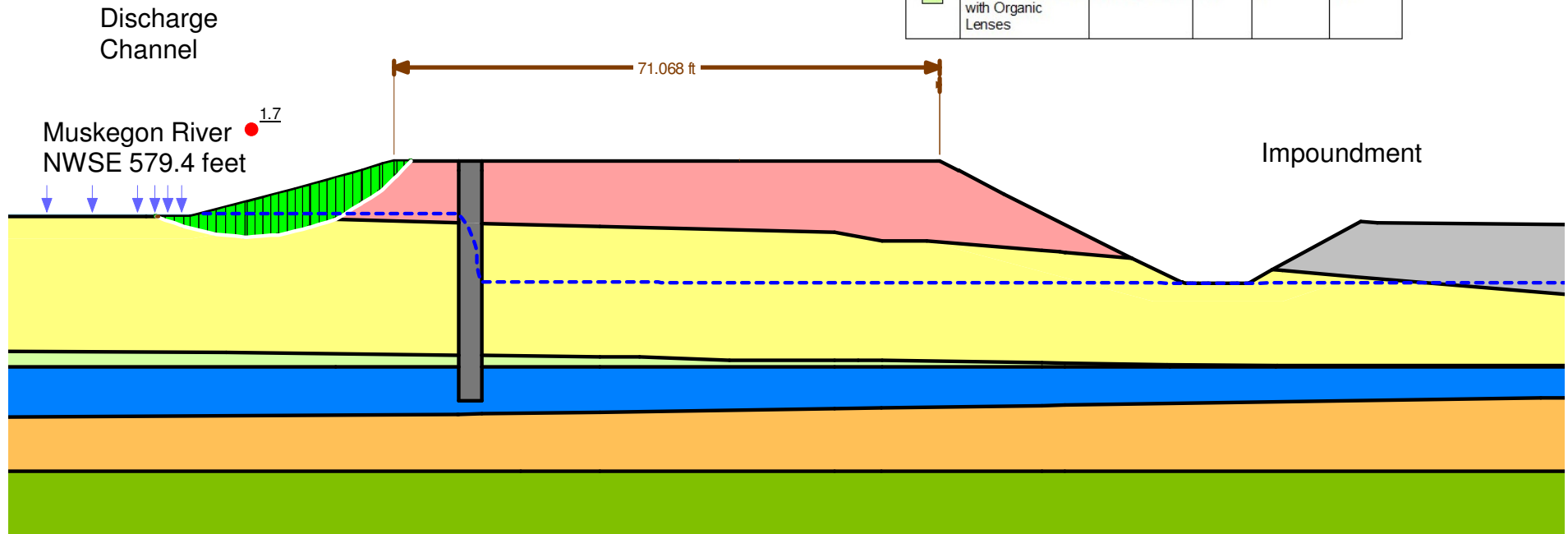
Description: Normal Water Surface Elevation- Pseudostatic- River Side

Acceleration:  $K_h = 0.034 \text{ g}$  (475- year return period ground motion)

Method: Morgenstern-Price

FS: 1.7

Color	Name	Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Basal Clay	Mohr-Coulomb	109	150	28
	Berm Material	Mohr-Coulomb	120	0	31
	CCR	Mohr-Coulomb	112	0	29
	Lower Native Sand	Mohr-Coulomb	124	0	31
	Organic silt and clay with interbedded sand	Mohr-Coulomb	109	0	27
	SBW	Mohr-Coulomb	120	0	31
	Upper Native Sand	Mohr-Coulomb	124	0	27
	Upper Native Sand with Organic Lenses	Mohr-Coulomb	124	0	32





Name: B.C.Cobb Ash Pond - Critical Section  
 Description: Rapid Draw Down Analysis- Flood to Normal Water Surface  
 Method: Morgenstern-Price  
 FS: 2.4

Color	Name	Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
<span style="color: green;">■</span>	Basal Clay	Mohr-Coulomb	109	150	28
<span style="color: pink;">■</span>	Berm Material	Mohr-Coulomb	120	0	31
<span style="color: gray;">■</span>	CCR	Mohr-Coulomb	112	0	29
<span style="color: orange;">■</span>	Lower Native Sand	Mohr-Coulomb	124	0	31
<span style="color: blue;">■</span>	Organic silt and clay with interbedded sand	Mohr-Coulomb	109	0	27
<span style="color: darkgray;">■</span>	SBW	Mohr-Coulomb	120	0	31
<span style="color: yellow;">■</span>	Upper Native Sand	Mohr-Coulomb	124	0	27
<span style="color: lightgreen;">■</span>	Upper Native Sand with Organic Lenses	Mohr-Coulomb	124	0	32

