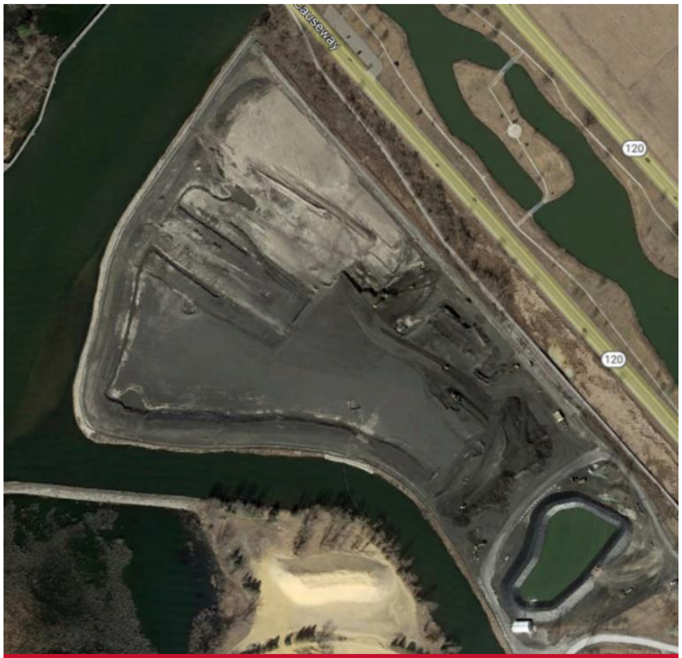

2022 Structural Stability and Safety Factor Assessment

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

August 17, 2022

Prepared for:
Muskegon Environmental Redevelopment Group, LLC (MERG)
Muskegon, Michigan

Prepared by:
HDR MICHIGAN, Inc.
5405 Data Court
Ann Arbor, Michigan 48108

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1 Introduction and Purpose

HDR MICHIGAN, Inc. (HDR) has prepared this 2022 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 periodic structural stability assessment and 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 recently completed excavation by removing CCR material from the ponds as part of pond remediation 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.

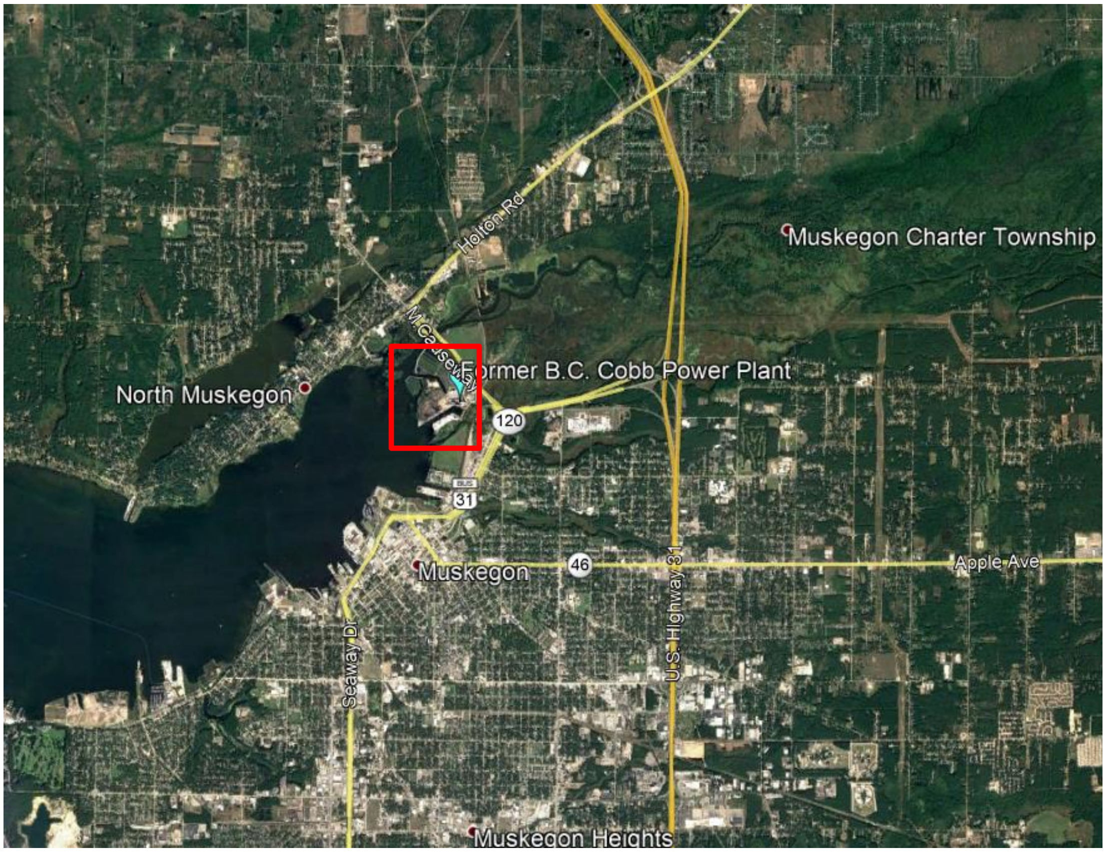

Figure 1. Site Vicinity Map

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. [12]). 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 decommissioned. 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 remediation 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 was completed in May 2022. The interior embankments separating the ponds have been excavated and removed, while the perimeter embankments are still in place adjacent to the Discharge Channel and the Muskegon River.

Figure 2 presents an aerial view of the CCR impoundment as of February 2022, displaying the excavated impoundments, as well as the terminology of the embankment sections used in this report.

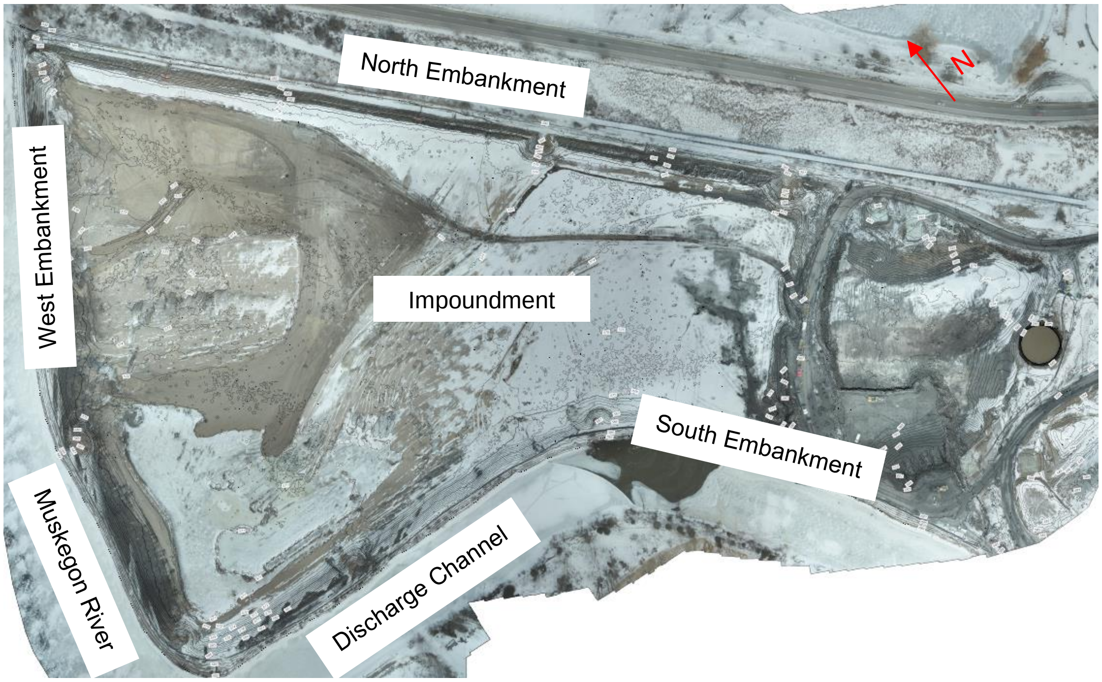

Figure 2. Aerial Image of Impoundment During Excavation

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). HDR performed annual inspections in 2021 and 2022. 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. [1]
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. [12]
B.C. Cobb Ponds 0-8 2019 Annual Surface Impoundment Inspection Report	May 21, 2019	Ref. [13]
2021 Annual Inspection Report, Former B.C. Cobb Power Plant Ponds 0-8 and Bottom Ash Pond	March 24, 2021	Ref. [14]
2022 Annual Inspection Report, Former B.C. Cobb Power Plant Ponds 0-8 and Bottom Ash Pond	April 29, 2022	Ref. [15]

During on-going excavation, MERG performs daily visual inspections of the entire site. 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 documentation requirements 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 subsurface investigation program in 2020 by SME (Ref. [19]) 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. [19]) and also in Figure 3 below. The boring logs are provided in Attachment 1.

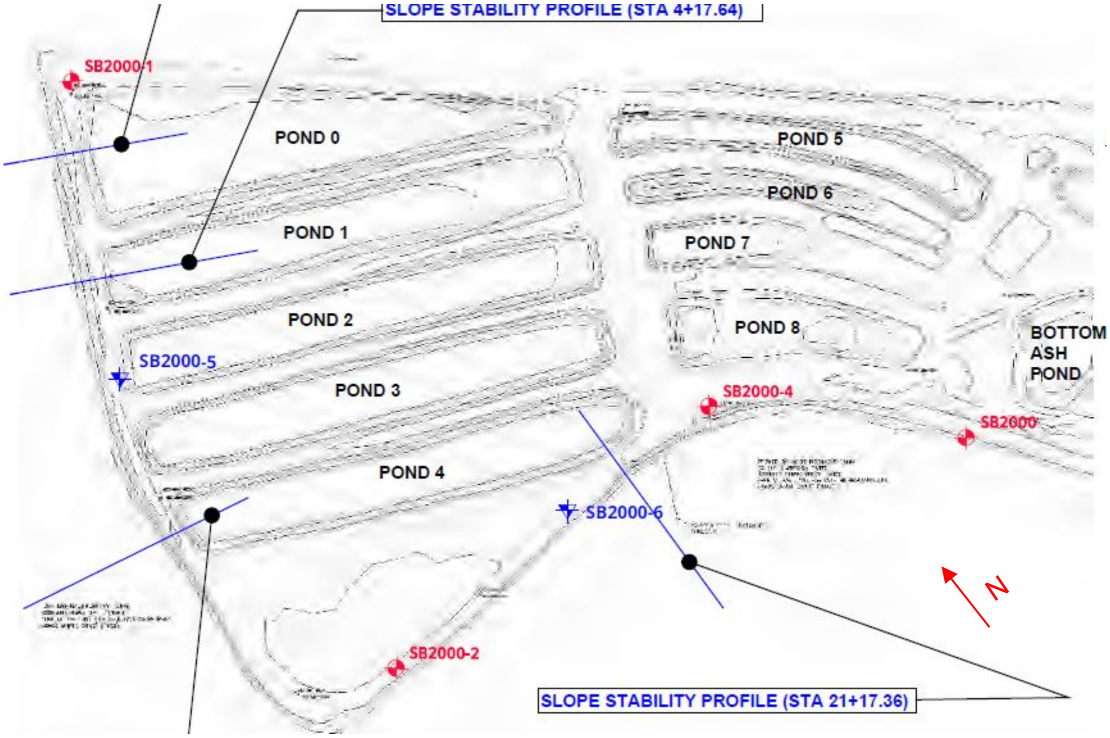

Figure 3. Approximate Boring Locations

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.

Based on subsurface investigation documentation and the history of no observed instability in the foundation materials, the foundation is demonstrating stability and competency. 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 in Muskegon Lake create large waves which have the potential to reach the exterior slopes of the perimeter embankments. The exterior slopes of the perimeter embankments are protected from erosion and deterioration by riprap and vegetative cover. The interior slopes are currently graded with soil/CCR surface. At the time of this report, MERG is currently planning to topsoil and seed the interior slopes to provide additional vegetative protection.

The crest of the perimeter embankment is a gravelly/soil surface. Due to the recent 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 2022 Annual Inspection performed by HDR (Ref. [15]).

The existing slope protection measures for the perimeter embankment have shown and are generally considered adequate to provide protection against surface erosion, wave action, and adverse effects of sudden drawdown. The April 2022 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 investigations conducted at the site (Ref. [6] and Ref. [19]), 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 considered 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 April 2022 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.***

There are no spillways associated with the impoundment.

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 April 2022 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. Portions of these pipes between the soil-bentonite wall and the Discharge Channel have been grouted and 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.

Ponds 0-8 are classified as a significant hazard (Ref. [10]) and 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. 587 feet. The typical water elevation of Muskegon Lake is approximately El. 579.4 feet as estimated by nearby tide gauge data (Ref. [18]) 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. 580 feet). A factor of safety (FS) of 1.91 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 1.1 required for rapid drawdown loading as per USACE Engineering Manual EM 1110-2-1902 guidelines (Ref. [20]).

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 are considered to have 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 April 2022 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. [20]) 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 11.2.0.22838) 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. We used Spencer’s method which uses two-dimensional limit equilibrium analysis to determine the factor of safety for the slope. The computed factor of safety is the ratio of the forces resisting movement to the forces driving movement. 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 considered an embankment safety concern since they are surficial in nature and 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 5.4 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 1) the May 2022 topographic survey provide by MERG which was performed after the completion of excavation, 2) the interpreted subsurface profile from the available borings at the site (discussed in Section 2.1), and 3) the interpreted phreatic surface based on observations at the site and from records of monitoring wells installed at the site.

One embankment section was considered critical based on geometry, described below, and located along the South Embankment as shown on Figure 4 with the section cut on Figure 5.

- The stability cross-section chosen for the Safety Factor Assessment is located along the South Embankment. This section was selected due to the geometry of the slope at this location compared to the other geometries observed from May 2022 topographic survey. 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.

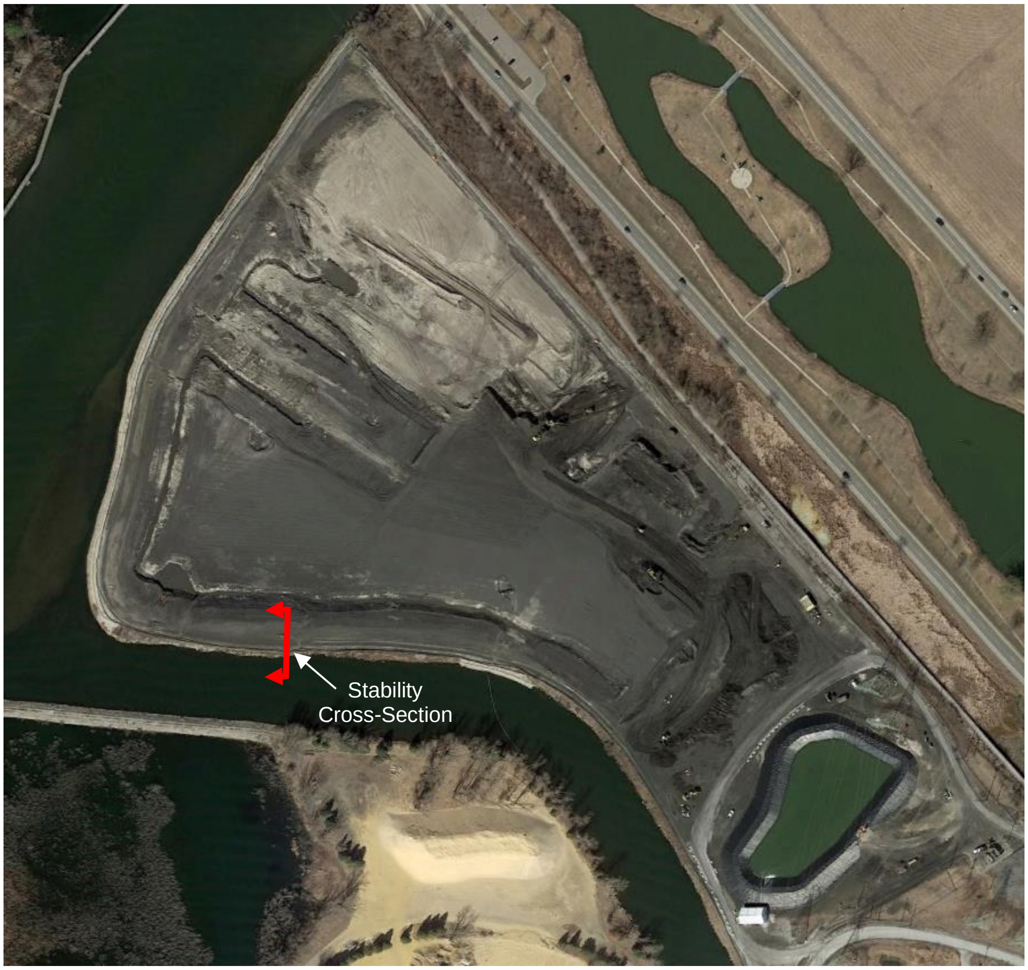

Figure 4. Location of Stability Cross-Section

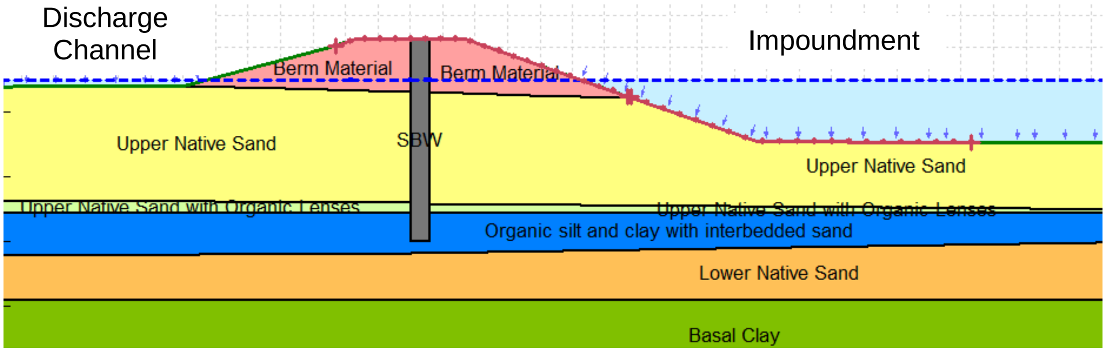

Figure 5. Stability Cross Section Geometry and Stratigraphy (phreatic surface for NWSE case)

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[1]	1.00
Post-earthquake - Liquefaction[2]	1.20

Notes:

1. A Peak Ground Acceleration (PGA) = 0.079g was adopted, based on a 2 percent probability of exceedance in 50 years (2475-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 (0.079 g at 2475-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 excavated impoundment refilling with water. For the maximum storage pool condition, the water in the impoundment and the Discharge Channel was modeled to be at El. 580.0 feet, the approximate Normal Water Surface Elevation (NWSE) of the Discharge Channel (Ref. [8]).

The maximum pool surcharge condition was assumed to be at El. 582.0 feet in accordance with the post-closure recommended pond levels established by HDR (Ref. [16]). An additional 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]).

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, ϕ' (deg)	Drained Cohesion, c' (psf)
Berm Material	90	120	31	0
CCR	70	112	29	0
Upper Native Sand	96	124	27	0
Upper Native Sand with Organic Lenses	98	124	32	0
Organic silt and clay with interbedded sand	74	109	27	0
Lower Native Sand	98	124	31	0
Basal Clay (Lacustrine)	74	109	28	150
Soil Bentonite Wall (SBW)	120	120	31	0

3.7 Vehicle Loading

The crest of the perimeter embankment is used as an access road 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 maximum pool storage, 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 lifetime) 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 same earthquake return period of 2475 years to address the potential risks after the construction period. The probability of occurrence of the 2475-year earthquake is 2% during the typical lifetime of 50 years.

In the current study, Borings SB2000-1 to SB2000-6 (Ref. [19]) 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. [21]), the Peak Ground Acceleration (PGA), assuming a Site Class B/C boundary was obtained as 0.0331g. The earthquake magnitude is assumed 6.2, similar to the previous liquefaction potential assessment (Ref. [8]). Pages 1 through 5 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. 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 0.0794g and magnitude of 6.2.

The triggering analysis is based on the procedure proposed by Idriss and Boulanger (Ref. [17]). 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:

$$\boldsymbol{(N1)_{60}} = N_{RAW} C_N C_E C_B C_R C_S$$

Where:

$\mathbf{C_N}$ = Overburden Correction Factor = (P_a/σ'_{vo})^(0.784-0.0768[$(N_1)_{60}$^$^{0.5}$] ≤ 1.7

C_E = Hammer Energy Correction factor = 60% efficient safety hammer = 1.0

C_B = Borehole Diameter Correction Factor = 1.0

C_R = Rod Length Correction Factor

= 0.75 (0-9.75 ft.)
= 0.8 (9.75 to 13 ft.)
= 0.85 (13 to 19.5 ft.)
= 0.95 (19.5 to 32 ft.)
= 1 (>32 ft.)

C_S = Spoon Liner Correction

= 1.0 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. [17]) and the following equation:

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

Where:

PF = Percent fines passing No. 200 sieve

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

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

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 0.0794g 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:

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

Where:

a_{max}/g = 0.0794

σ_v = Total Overburden Stress

σ'_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 = 6.2

z = depth in meters

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

$$\textbf{FS} = CRR/CSR \text{ x } MSF \text{ x } K_{\sigma} \text{ x } K_{\alpha}$$

Where:

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

K_{α} = correction factor for the effects of an initial static shear stress ratio = 1

K_{σ} = overburden correction factor = $1\text{-}C_{\sigma} \text{ x } Ln(\sigma'_{V}/P_{a}) \leq 1.1$

Where:

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

P_a = Pressure at 1 atmosphere

The static shear strength in the liquefaction-susceptible material is small. Therefore, K_{α} 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 on Page 6 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 lose clean sand material with N≤1 (identified in the box on Page 7 of Attachment 2). As shown on Page 6 of Attachment 2, the results provided a FS greater than 1.2 for the assumed amplified peak ground acceleration factor of 2.4.

Because neither the embankment, nor foundation soil, were considered 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 0.0794g.

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 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)	Pond 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
Maximum Pool Storage	580.0	580.0	1.50	1.53	Page 1	1.91	Page 2
1000-year Flood Elevation	585.7	582.0	1.40	1.52	Page 3	2.38	Page 4
Maximum Pool Surcharge	580.0	582.0	1.40	1.52	Page 5	1.91	Page 6
Pseudo-Static Seismic Stability	580.0	580.0	1.00	1.04	Page 7	1.41	Page 8
Rapid Drawdown	585.7 to 580	582 to 580	1.10	--	--	1.91	Page 9

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

Bryce Burkett, P.E.
Senior Geotechnical Project Manager

Greg Shafer, P.E.
Geotechnical Engineer

STATE OF MICHIGAN
TERRY
BRYCE
BURKETT
ENGINEER
No.
6201066757
LICENSED PROFESSIONAL ENGINEER

17 August 2022

5 References

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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, Ponds 0-8, 2018 Annual Surface Impoundment Inspection Report, October 15, 2018.

Ref. [13] Golder Associates, Inc. B.C. Cobb Generating Facility, Ponds 0-8 2019 Annual Surface Impoundment Inspection Report, October 10, 2019.

Ref. [14] HDR Michigan, Inc. 2021 Annual Inspection Report, Former B.C. Cobb Power Plant, Ponds 0-8 and Bottom Ash Pond, May, 26, 2021.

Ref. [15] HDR Michigan, Inc. 2022 Annual Inspection Report, Former B.C. Cobb Power Plant, Ponds 0-8 and Bottom Ash Pond, May, 26, 2022.

Ref. [16] HDR Michigan, Inc. Memorandum: Post-Closure Pond Elevation Recommendations, BC Cobb CCR Removal, May, 27, 2022.

Ref. [17] 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. [18] National Oceanic and Atmospheric Administration (NOAA). Holland, MI - Station ID: 9087031, accessed April 2021, https://tidesandcurrents.noaa.gov/stationhome.html?id=9087031.

Ref. [19] SME. Geotechnical Evaluation Report, B.C. Cobb Ash Basin Dewatering and Dredging, Muskegon, Michigan. SME Project No. 083742.01, August 27, 2020.

Ref. [20] USACE. EM 1110-2-1902, Slope Stability, October 31, 2003.

Ref. [21] United States Geologic Survey, Unified Hazard Tool, accessed July 2022, 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

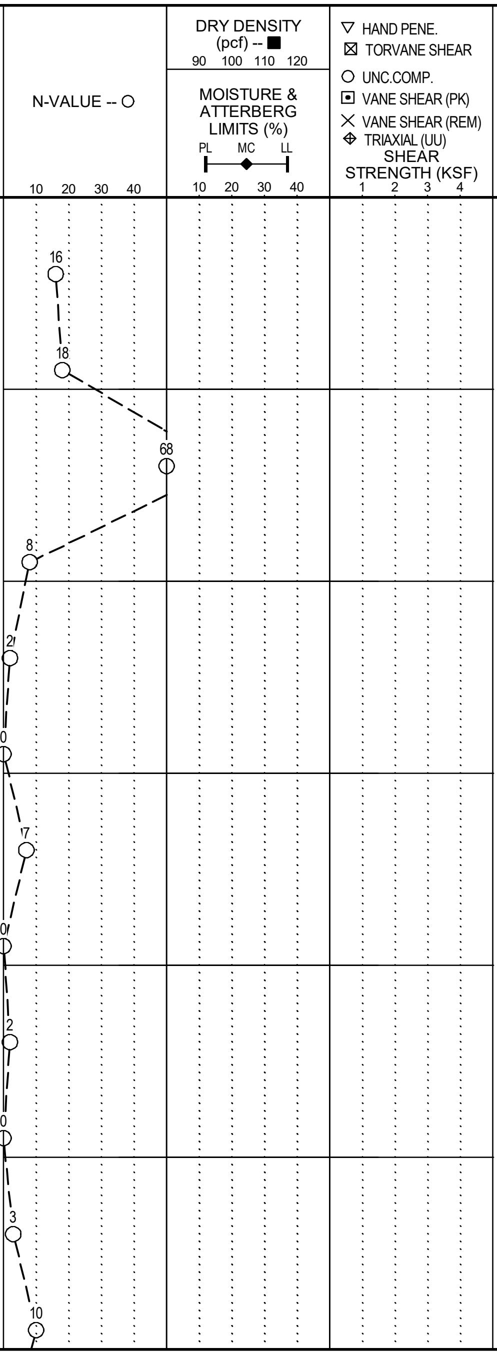
PAGE 1 OF 2

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

LATITUDE: 43.26127 N
LONGITUDE: -86.246722 W
SURFACE ELEVATION: 589.8 FT (NAVD88)

DEPTH (FEET)	ELEVATION (FEET)	PROFILE DESCRIPTION
0–1.5	588.3	FILL- GRAVEL
1.5–7.0	582.8	FILL- COAL ASH- Frequent Sand Layers- Black
7.0–11.0	578.8	FILL- Fine to Medium SAND- Occasional Wood Pieces- Brown- Moist to Wet- Very Dense to Loose (SP)
11.0–13.5	576.3	FILL- Fine to Medium SAND with Silt- Gray- Wet- Very Loose (SP-SM)
13.5–16.0	573.8	FILL- SILT- Occasional Peat Layers- Gray- Wet- Very Loose (ML)
16.0–24.5	565.3	Fine to Medium SAND- Gray- Wet- Loose to Very Loose (SP)
24.5–29.5	560.3	Fine to Medium SILTY SAND- Trace Organics- Dark Gray- Wet- Very Loose (SM)

SAMPLE TYPE/NO. INTERVAL	RECOVERY LENGTH (INCHES)	BLOWS PER SIX INCHES	N-VALUE
SB1	18	11 8 8	16
SB2	18	5 8 10	18
SB3	18	27 35 33	68
SB4	18	4 5 3	8
SB5	15	1 1 1	2
SB6	18	1 0 0	0
SB7	18	1 3 4	7
SB8	18	1 0 0	0
SB9	18	1 1 1	2
SB10	18	2 0 0	0
SB11	18	1 1 2	3
SB12	18	2 5 5	10



REMARKS

GROUNDWATER & BACKFILL INFORMATION

	DEPTH (FT)	ELEV (FT)
▽ DURING BORING:	8.5	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)

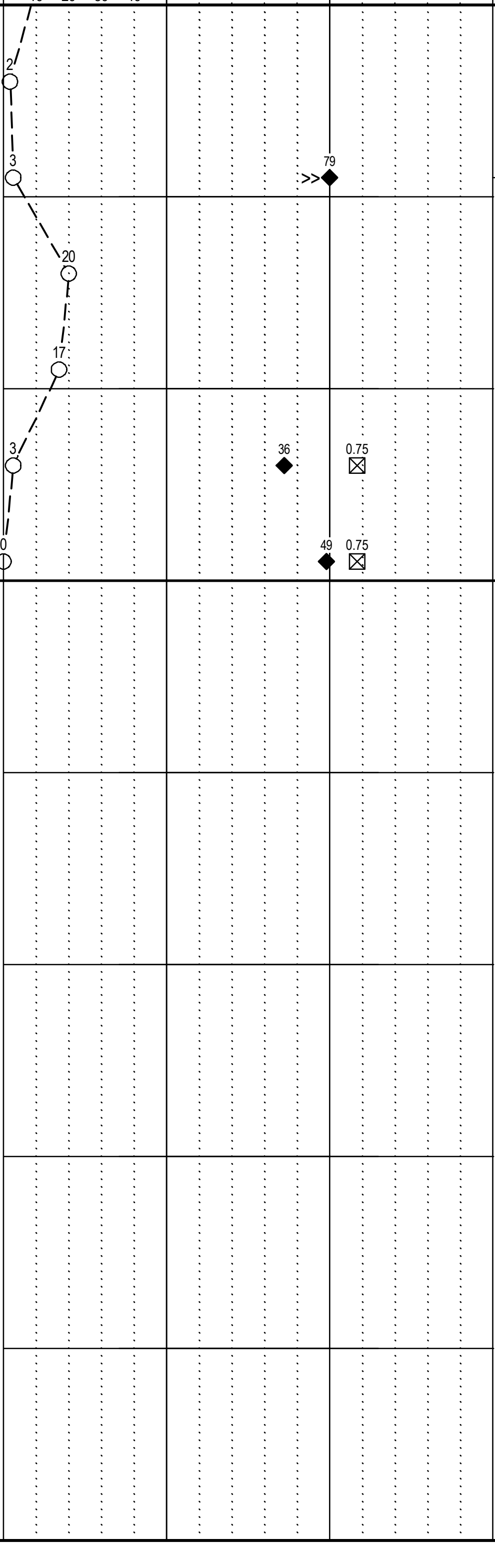
BORING SB2000-1

PAGE 2 OF 2

PROJECT NAME: BC Cobb Slurry Wall Design
PROJECT NUMBER: 083742.01
CLIENT: HDR Michigan Inc.
PROJECT LOCATION: North Muskegon, Michigan

LATITUDE: 43.26127 N
LONGITUDE: -86.246722 W
SURFACE ELEVATION: 589.8 FT (NAVD88)

DRY DENSITY (pcf) -- ■ 90 100 110 120
MOISTURE & ATTERBERG LIMITS (%) PL MC LL 10 20 30 40
SHEAR STRENGTH (KSF) 1 2 3 4
▽ HAND PENE.
⊠ TORVANE SHEAR
○ UNC.COMP.
⊡ VANE SHEAR (PK)
× VANE SHEAR (REM)
⊕ TRIAXIAL (UU)



ELEVATION (FEET)	DEPTH (FEET)	PROFILE DESCRIPTION	SAMPLE TYPE/NO. INTERVAL	RECOVERY LENGTH (INCHES)	BLOWS PER SIX INCHES	N-VALUE	MOISTURE (%)	SHEAR STRENGTH (KSF)	REMARKS
557.8	32.0	Fine to Medium SAND- Gray- Wet- Loose (SP) *(continued)*	SB13	18	1 1 1	2			
555.8	34.0	Fine to Medium SILTY SAND- Dark Gray- Wet- Very Loose (SM)	SB14	18	1 1 2	3	>>79		Sample SB14 was too disturbed to perform a shear strength test.
553.8	36.0	SILTY CLAY- Dark Brown- Very Soft (CL/ML)							
		Fine to Medium SAND- Brown and Gray- Wet- Medium Dense (SP)	SB15	18	2 7 13	20			
548.3	41.5		SB16	18	4 6 11	17			
		SILTY CLAY- Gray- Medium (CL/ML)	SB17	18	2 2 1	3	36	0.75	
544.8	45.0		SB18	18	1 0 0	0	49	0.75	

END OF BORING AT 45.0 FEET.

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

LATITUDE: 43.25792 N
LONGITUDE: -86.247977 W
SURFACE ELEVATION: 586.2 FT (NAVD88)

DRY DENSITY (pcf) -- ■ 90 100 110 120
MOISTURE & ATTERBERG LIMITS (%) PL MC LL 10 20 30 40
N-VALUE -- ○ 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

DEPTH (FEET)	PROFILE DESCRIPTION	ELEVATION (FEET)
1.0	FILL- GRAVEL	585.2
3.5	FILL- Fine to Coarse SAND with Silt and Gravel- Brown- Moist- Medium Dense (SP-SM)	582.7
6.0	FILL- Fine to Coarse SILTY SAND- Few Gravel- Brown and Gray- Wet- Loose (SM)	580.2
8.5	FILL- Fine to Medium SAND with Silt- Occasional Coal Fragments- Gray and Brown- Wet- Loose (SP-SM)	577.7
13.5	FILL- Fine to Medium SAND with Silt- Frequent Wood Pieces- Gray- Wet- Very Loose to Loose (SP-SM)	572.7
22.0	Fine to Medium SAND- Trace Organics- Occasional Wood Pieces- Gray- Wet- Very Loose (SP)	564.2
26.0	Fine SILTY SAND- Gray- Wet- Very Loose (SM)	560.2
28.0	Fine SILTY SAND- Trace Organics- Gray and Brown- Wet- Very Loose (SM)	558.2
	SILTY to CLAYEY Fine SAND- Dark Brown- Moist- Very Loose (SC/SM)	

SAMPLE TYPE/NO. INTERVAL	RECOVERY LENGTH (INCHES)	BLOWS PER SIX INCHES	N-VALUE	MOISTURE & ATTERBERG LIMITS (%)	REMARKS
SB1	18	6 12 10	22		
SB2	18	4 3 2	5		
SB3	15	6 3 2	5		
SB4	7	2 0 0	0		
SB5	15	1 2 3	5		
SB6	12	1 1 2	3		
SB7	18	1 0 0	0		
SB8	18	2 0 0	0		
SB9	18	1 1 1	2		
SB10	6	1 1 2	3		
SB11	18	1 2 2	4		
SB12	14	1 2 1	3	>> 62	

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)

BORING SB2000-2

PROJECT NAME: BC Cobb Slurry Wall Design
PROJECT NUMBER: 083742.01
CLIENT: HDR Michigan Inc.
PROJECT LOCATION: North Muskegon, Michigan

LATITUDE: 43.25792 N
LONGITUDE: -86.247977 W
SURFACE ELEVATION: 586.2 FT (NAVD88)

DRY DENSITY (pcf) -- ■ 90 100 110 120

∇ HAND PENE.
☒ TORVANE SHEAR
○ UNC.COMP.
⊡ VANE SHEAR (PK)
× VANE SHEAR (REM)
⊕ TRIAXIAL (UU)

ELEVATION (FEET)	DEPTH (FEET)	PROFILE DESCRIPTION	SAMPLE TYPE/NO. INTERVAL	RECOVERY LENGTH (INCHES)	BLOWS PER SIX INCHES	N-VALUE -- ○	MOISTURE & ATTERBERG LIMITS (%) PL MC LL	SHEAR STRENGTH (KSF)	REMARKS
555.2	31.0								
		SILTY CLAY- Dark Brown and Gray- Very Soft (CL/ML)	SB13	18	1 1 2	3	>>60	0.25	
552.7	33.5								
		Fine to Medium SAND- Light Brown- Wet- Loose (SP)	SB14	7	0 1 5	6			
			SB15	6	1 4 5	9			
547.7	38.5								
		Fine to Coarse SAND- Light Brown- Wet- Loose (SP)	SB16	6	4 6 4	10			
546.2	40.0								
		SILTY CLAY- Light Brown and Gray- Very Soft (CL/ML)					>>52	0.5	
			SB17	18	1 1 1	2			
			SB18	18	1 0 0	0	>>58	0.5	
541.2	45.0								

END OF BORING AT 45.0 FEET.

BORING SB2000-3

PAGE 1 OF 2

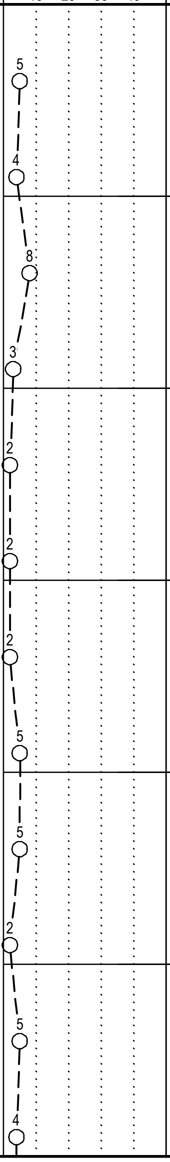
PROJECT NAME: BC Cobb Slurry Wall Design
PROJECT NUMBER: 083742.01
CLIENT: HDR Michigan Inc.
PROJECT LOCATION: North Muskegon, Michigan

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

LATITUDE: 43.256708 N
LONGITUDE: -86.243934 W
SURFACE ELEVATION: 589.2 FT (NAVD88)

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

DEPTH (FEET)	ELEVATION (FEET)	PROFILE DESCRIPTION	SAMPLE TYPE/NO. INTERVAL	RECOVERY LENGTH (INCHES)	BLOWS PER SIX INCHES	N-VALUE -- ○
1.0	588.2	FILL- GRAVEL				
4.5	584.7	FILL- Fine SILTY SAND- Frequent Roots- Dark Gray- Moist- Loose (SM)	SB1	18	3 2 3	5
			SB2	18	1 1 3	4
6.0	583.2	FILL- Fine to Coarse SAND with Silt- Dark Brown and Black- Moist- Loose (SP-SM)				
8.5 ⊻	580.7	FILL- Fine to Coarse SAND with Silt and Gravel- Frequent Wood Pieces- Reddish Brown- Wet- Loose (SP-SM)	SB3	18	4 4 4	8
11.0	578.2	FILL- Fine to Coarse SILTY SAND with Gravel- Gray- Wet- Very Loose (SM)	SB4	18	6 2 1	3
13.5	575.7	FILL- Fine SILTY SAND- Trace Organics- Gray- Wet- Very Loose (SM)	SB5	18	1 1 1	2
18.5	570.7	Fine SAND- Trace Silt- Gray- Wet- Very Loose (SP)	SB6	18	1 1 1	2
			SB7	18	1 1 1	2
21.0	568.2	Fine to Coarse SAND- Trace Silt- Trace Gravel- Gray- Wet- Loose (SP)	SB8	18	1 2 3	5
26.0	563.2	Fine to Medium SAND- Trace Organics- Brownish Gray- Wet- Loose to Very Loose (SP)	SB9	12	1 2 3	5
			SB10	12	0 1 1	2
27.0	562.2	Fine SILTY SAND with Wood- Gray and Dark Brown- Wet- Loose (SM)	SB11	18	1 2 3	5
29.0	560.2	Fine to Medium SAND- Gray- Wet- Loose (SP)				
		Fine SILTY SAND- Trace	SB12	18	1 1 3	4



GROUNDWATER & BACKFILL INFORMATION

	DEPTH (FT)	ELEV (FT)
⊻ DURING BORING:	8.5	580.7
▼ 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)

BORING SB2000-3

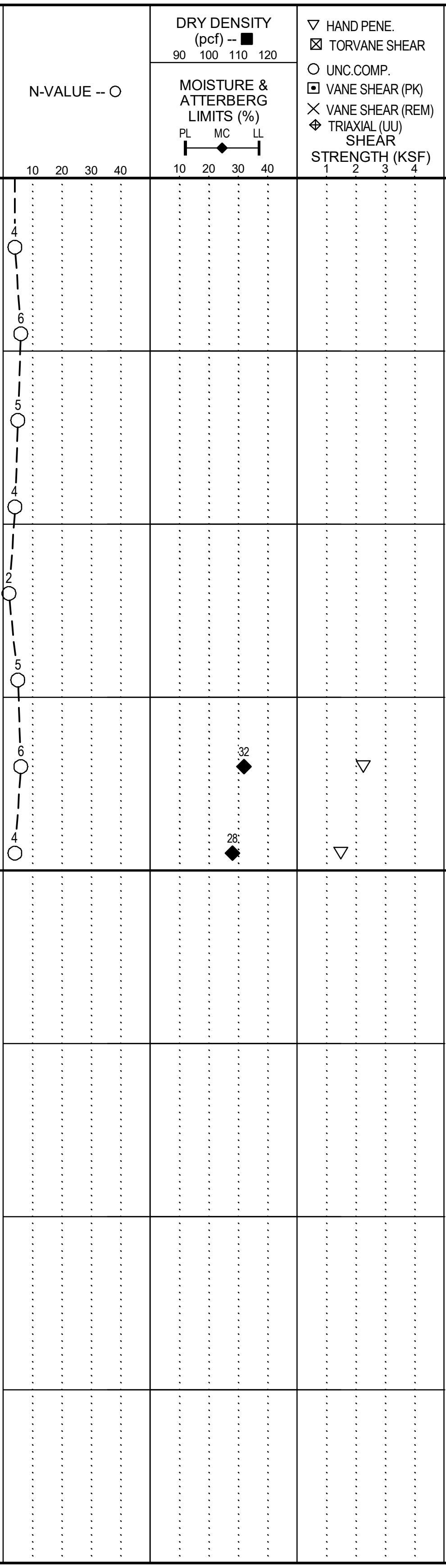
PROJECT NAME: BC Cobb Slurry Wall Design **PROJECT NUMBER:** 083742.01

CLIENT: HDR Michigan Inc. **PROJECT LOCATION:** North Muskegon, Michigan

LATITUDE: 43.256708 N
LONGITUDE: -86.243934 W
SURFACE ELEVATION: 589.2 FT (NAVD88)

DEPTH (FEET)	ELEVATION (FEET)	PROFILE DESCRIPTION	SAMPLE TYPE/NO. INTERVAL	RECOVERY LENGTH (INCHES)	BLOWS PER SIX INCHES	N-VALUE	MOISTURE (%)
31.0	558.2	Organics- Gray- Wet- Loose (SM)					
36.0	553.2	Fine SILTY SAND- Gray- Wet- Loose (SM)	SB13	18	1 2 2	4	
			SB14	18	1 2 4	6	
42.0	547.2	Fine SILTY SAND- Occasional Roots- Gray- Moist- Loose (SM)	SB15	18	1 3 2	5	
			SB16	18	1 2 2	4	
			SB17	18	0 1 1	2	
46.0	543.2	Fine to Medium SAND- Gray- Wet- Loose (SP)	SB18	18	0 2 3	5	
48.5	540.7	SILTY CLAY- Grayish Brown- Very Stiff (CL/ML)	SB19	18	2 2 4	6	32
50.0	539.2	SILTY CLAY- Brownish Gray- Stiff (CL/ML)	SB20	18	1 2 2	4	28

END OF BORING AT 50.0 FEET.



REMARKS

SME

BORING SB2000-4

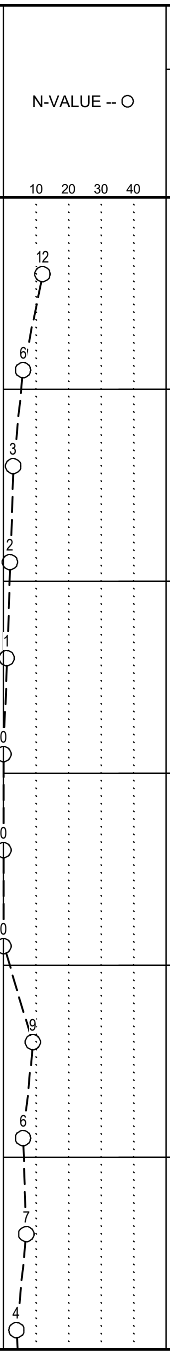
PAGE 1 OF 2

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

LATITUDE: 43.257754 N
LONGITUDE: -86.245089 W
SURFACE ELEVATION: 593.6 FT (NAVD88)

DEPTH (FEET)	PROFILE DESCRIPTION	ELEVATION (FEET)
0 – 1.0	FILL- GRAVEL	592.6
1.0 – 21.0	FILL- COAL ASH- Occasional Sand Layers- Black	572.6
21.0 – 26.0	Fine to Medium SAND- Gray- Wet- Loose (SP)	567.6
26.0 –	Fine to Medium SAND- Trace Silt and Organics- Gray- Wet- Loose (SP)	

SAMPLE TYPE/NO. INTERVAL	RECOVERY LENGTH (INCHES)	BLOWS PER SIX INCHES	N-VALUE
SB1	18	8 7 5	12
SB2	18	3 3 3	6
SB3	18	1 2 1	3
SB4	18	1 1 1	2
SB5	18	1 1 0	1
SB6	18	1 0 0	0
SB7	18	1 0 0	0
SB8	18	1 0 0	0
SB9	18	2 6 3	9
SB10	18	1 3 3	6
SB11	18	1 3 4	7
SB12	18	2 3 1	4



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

GROUNDWATER & BACKFILL INFORMATION

	DEPTH (FT)	ELEV (FT)
▽ **DURING BORING:**	9.5	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)

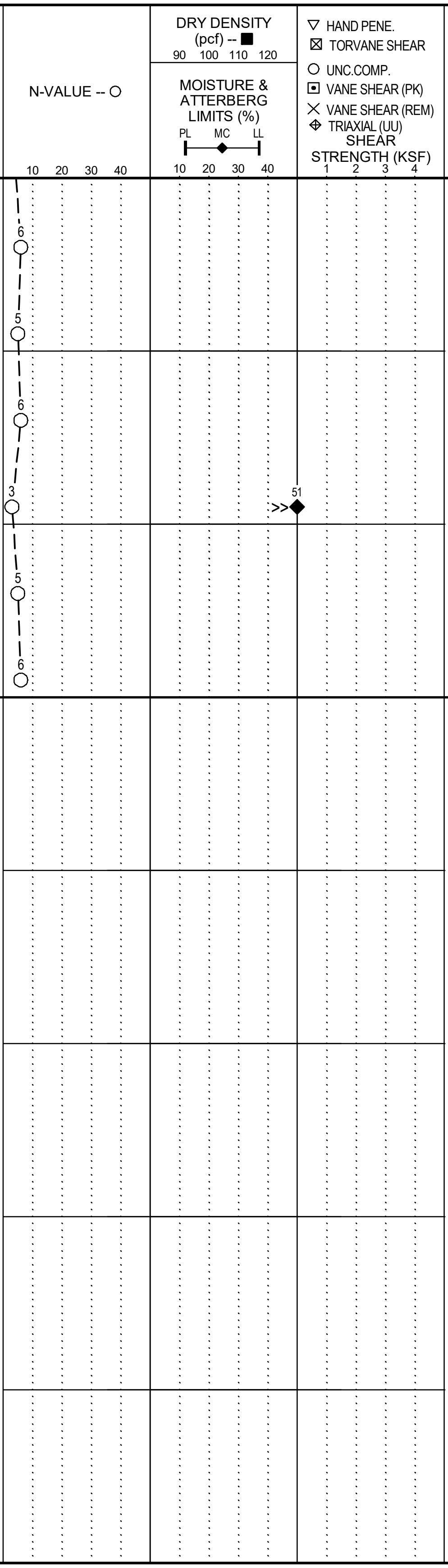
BORING SB2000-4

PROJECT NAME: BC Cobb Slurry Wall Design
PROJECT NUMBER: 083742.01
CLIENT: HDR Michigan Inc.
PROJECT LOCATION: North Muskegon, Michigan

LATITUDE: 43.257754 N
LONGITUDE: -86.245089 W
SURFACE ELEVATION: 593.6 FT (NAVD88)

DEPTH (FEET)	PROFILE DESCRIPTION	ELEVATION (FEET)	SAMPLE TYPE/NO. INTERVAL	RECOVERY LENGTH (INCHES)	BLOWS PER SIX INCHES	N-VALUE -- ○	MOISTURE & ATTERBERG LIMITS (%)	REMARKS
33.0	Fine to Medium SAND- Trace Silt and Organics- Gray- Wet- Loose (SP) *(continued)*	560.6	SB13	18	4 3 3	6		
34.0	Fine SILTY SAND- Dark Gray- Wet (SM)	559.6	SB14	18	1 2 3	5		
38.5	Fine to Medium SAND- Trace Silt- Brown- Wet- Loose (SP)	555.1	SB15	18	2 3 3	6		
41.0	Sandy SILT- Dark Brown and Gray- Moist- Very Loose (ML)	552.6	SB16	18	1 1 2	3	>> 51	
45.0	Fine to Medium SAND- Grayish Brown- Wet- Loose (SP)	548.6	SB17	18	1 2 3	5		
			SB18	18	0 0 6	6		

END OF BORING AT 45.0 FEET.



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

LATITUDE: 43.260001 N
LONGITUDE: -86.24798 W
SURFACE ELEVATION: 587 FT (NAVD88)

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)

DEPTH (FEET)	PROFILE DESCRIPTION	ELEVATION (FEET)	SAMPLE TYPE/NO. INTERVAL	RECOVERY LENGTH (INCHES)	BLOWS PER SIX INCHES	N-VALUE -- ○	MOISTURE & ATTERBERG LIMITS (%)	REMARKS
0	FILL- Fly Ash- Trace Sand- Black- Moist	587	SB1	18	2 2 3	5	>>82	
			3ST2	11				
∇ 5.1		581.9						
	Fine SAND- Occasional Organics- Light Brown and Brown- Moist to Wet- Very Loose to Loose (SP)		SB3	18	1 1 1	2	20	
			SB4	12	2 2 2	4	25	Organic odor
			SB5	10	1 2 2	4	30	
			SB6	8	1 1 2	3	28	
15.5		571.5						
	Fine SAND with Organics and Wood Fragments- Light Brown- Wet- Loose (SP)		SB7	8	4 2 3	5	>>79	
			SB8	8	2 3 2	5	36	
20.5		566.5						
	Fine SAND- Trace Organics- Light Brown- Wet- Very Loose (SP)		SB9	14	3 2 1	3	19	
22.4		564.6						
	PEAT- Black (PT)							
23.0		564.0						
	Fine SAND- Trace Organic and Wood Fragments- Brown- Wet- Very Loose to Loose (SP)		SB10	9	1 2 1	3	25	
			SB11	8	3 2 2	4	23	
			3ST12	24				
30								

GROUNDWATER & BACKFILL INFORMATION

	DEPTH (FT)	ELEV (FT)
∇ **DURING BORING:**	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)

BORING SB2000-5

PAGE 2 OF 2

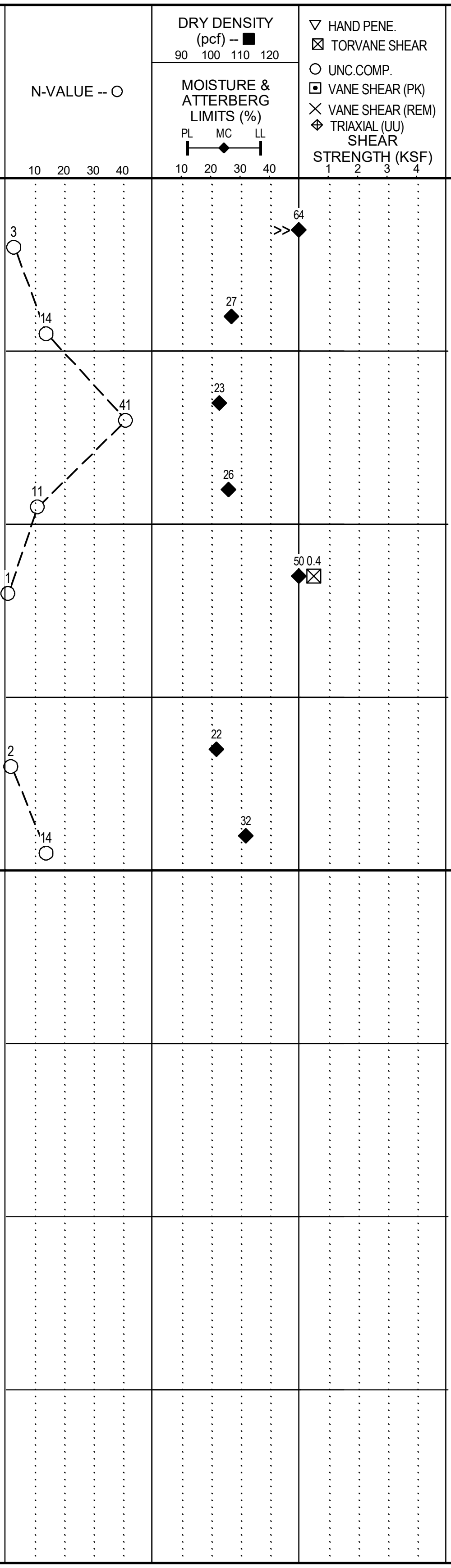
PROJECT NAME: BC Cobb Slurry Wall Design **PROJECT NUMBER:** 083742.01

CLIENT: HDR Michigan, Inc. **PROJECT LOCATION:** North Muskegon, MI

LATITUDE: 43.260001 N
LONGITUDE: -86.24798 W
SURFACE ELEVATION: 587 FT (NAVD88)

ELEVATION (FEET)	DEPTH (FEET)	PROFILE DESCRIPTION	SAMPLE TYPE/NO. INTERVAL	RECOVERY LENGTH (INCHES)	BLOWS PER SIX INCHES	N-VALUE -- ○	MOISTURE & ATTERBERG LIMITS (%)	SHEAR STRENGTH (KSF)	REMARKS
556.5	30.5								
552.5	34.5	ORGANIC CLAY- Trace Shells- Dark Brown- Soft (OL)	SB13	18	1 1 2	3	>> 64		
			SB14	18	2 4 10	14	27		
546.5	40.5	Fine SAND- Light Brown- Wet- Medium Dense to Dense (SP)	SB15	10	10 18 23	41	23		
			SB16	18	4 6 5	11	26		
540.0	47.0	LEAN CLAY with Organics- Trace Shells- Gray- Very Soft (CL)	SB17	18	1 0 1	1	50	0.4 ☒	
			3ST18	24					
			SB19	18	1 1 1	2	22		
538.0	49.0	LEAN CLAY with Organics and Peat- Trace Sand- Gray- Very Soft (CL)							
537.0	50.0	CLAYEY SAND- Brown- Wet- Medium Dense (SC)	SB20	18	3 6 8	14	32		

END OF BORING AT 50.0 FEET.



BORING SB2000-6

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

LATITUDE: 43.2579 N
LONGITUDE: -86.246529 W
SURFACE ELEVATION: 587 FT (NAVD88)

DRY DENSITY (pcf) -- ■ 90 100 110 120

MOISTURE & ATTERBERG LIMITS (%) PL MC LL 10 20 30 40

N-VALUE -- ○ 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

DEPTH (FEET)	PROFILE DESCRIPTION	ELEVATION (FEET)
0–11.5	FILL- Fly Ash with Sand- Black- Damp to Wet	575.5
11.5–13.0	PEAT with Sand- Dark Brown- Soft (PT)	574.0
13.0–19.3	Fine SAND- Trace Organics and Wood Fragments- Light Brown- Wet- Loose to Very Loose (SP)	567.7
19.3–23.0	PEAT- Dark Brown- Very Soft (PT)	564.0
23.0–28.0	ORGANIC CLAY with Sand- Brown- Soft (OL)	559.0
28.0–	ORGANIC CLAY- Trace Shells- Dark Brown- Very Soft to Soft (OL)	

SAMPLE TYPE/NO. INTERVAL	RECOVERY LENGTH (INCHES)	BLOWS PER SIX INCHES	N-VALUE	MOISTURE (MC)	REMARKS
SB1	18	3 3 3	6	>> 51	
3ST2	0				
3ST3	24				
SB4	18	1 1 1	2	>> 76	
SB5	12	1 1 2	3	>> 120	
SB6	12	1 3 3	6	28	
SB7	8	1 1 2	3	26	
SB8	14	1 1 1	2	21	
3ST9	24				
SB10	18	1 2 2	4	>> 56	
SB11	18	2 1 2	3	>> 52	
SB12	18	2 2 2	4	47	

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)

BORING SB2000-6

PAGE 2 OF 2

PROJECT NAME: BC Cobb Slurry Wall Design

PROJECT NUMBER: 083742.01

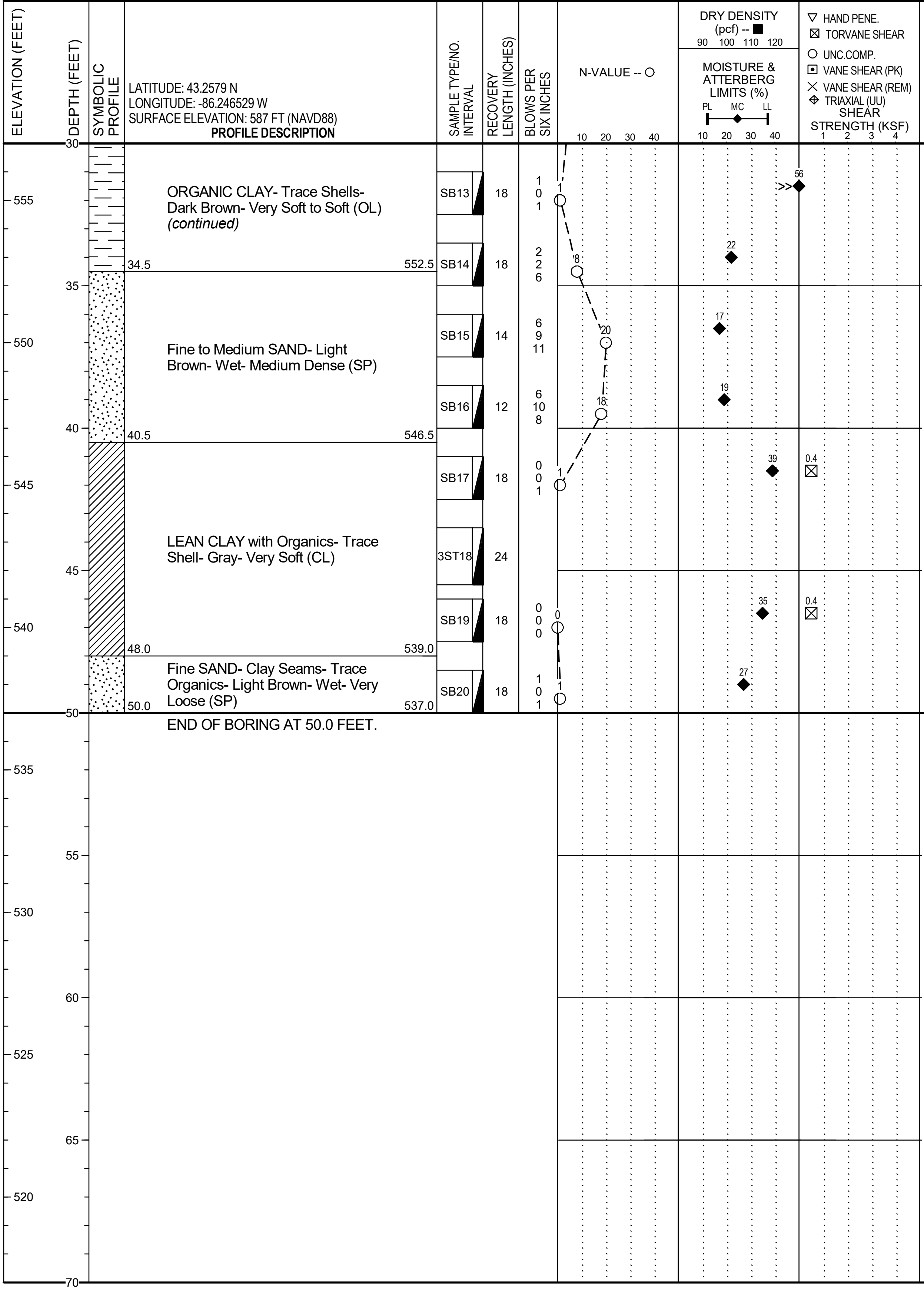
CLIENT: HDR Michigan, Inc.

PROJECT LOCATION: North Muskegon, MI

LATITUDE: 43.2579 N
LONGITUDE: -86.246529 W
SURFACE ELEVATION: 587 FT (NAVD88)

DEPTH (FEET)	PROFILE DESCRIPTION	ELEVATION (FEET)	SAMPLE TYPE/NO. INTERVAL	RECOVERY LENGTH (INCHES)	BLOWS PER SIX INCHES	N-VALUE -- O	MOISTURE & ATTERBERG LIMITS (%)	SHEAR STRENGTH (KSF)	REMARKS
30–34.5	ORGANIC CLAY- Trace Shells- Dark Brown- Very Soft to Soft (OL) *(continued)*	552.5	SB13	18	1 0 1	1	>>56		
			SB14	18	2 2 6	8	22		
34.5–40.5	Fine to Medium SAND- Light Brown- Wet- Medium Dense (SP)	546.5	SB15	14	6 9 11	20	17		
			SB16	12	6 10 8	18	19		
40.5–48.0	LEAN CLAY with Organics- Trace Shell- Gray- Very Soft (CL)	539.0	SB17	18	0 0 1	1	39	0.4 ☒	
			3ST18	24					
			SB19	18	0 0 0	0	35	0.4 ☒	
48.0–50.0	Fine SAND- Clay Seams- Trace Organics- Light Brown- Wet- Very Loose (SP)	537.0	SB20	18	1 0 1	1	27		

END OF BORING AT 50.0 FEET.



ATTACHMENT 2
LIQUEFACTION ANALYSIS FIGURES AND RESULTS

U.S. Geological Survey - Earthquake Hazards Program

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

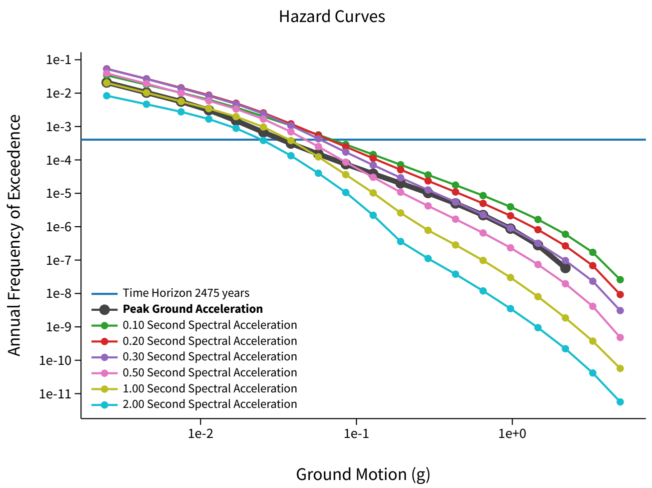
Latitude
Decimal degrees
43.259

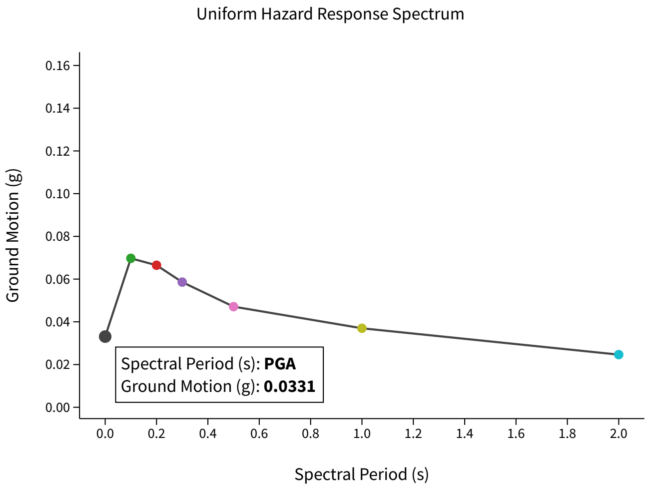
Time Horizon
Return period in years
2475

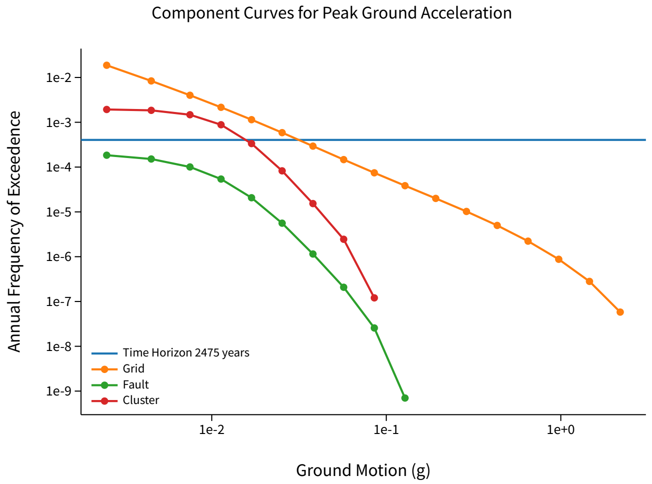
Longitude
Decimal degrees, negative values for western longitudes
-86.246

Site Class
760 m/s (B/C boundary)

Hazard Curve


Hazard Curves


Uniform Hazard Response Spectrum

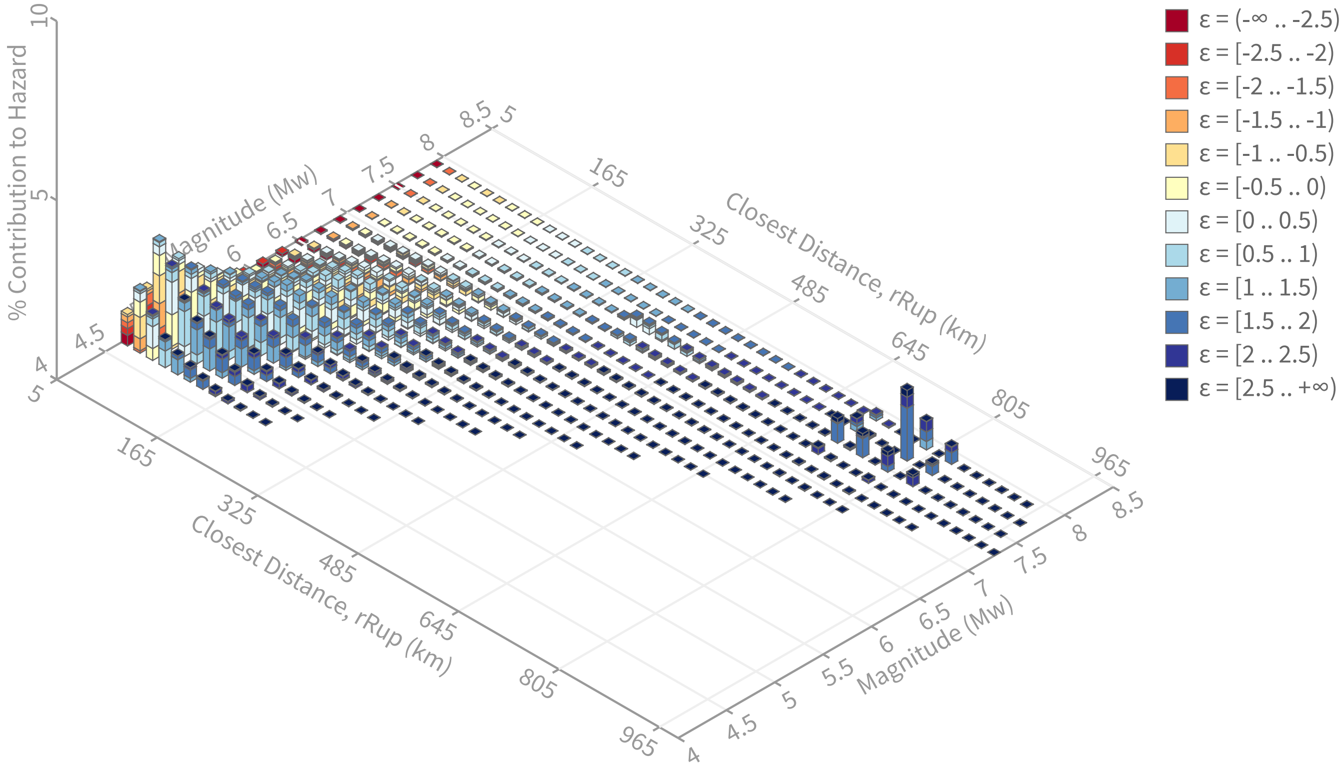

Component Curves for Peak Ground Acceleration

View Raw Data

^ Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
PGA ground motion: 0.033091268 g

Recovered targets

Return period: 2503.8898 yrs
Exceedance rate: 0.00039937859 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 2.73 %

Mean (over all sources)

m: 5.83
r: 171.69 km
ε_0: 0.26 σ

Mode (largest m-r bin)

m: 4.9
r: 30.09 km
ε_0: -0.75 σ
Contribution: 2.8 %

Mode (largest m-r-ε_0 bin)

m: 7.77
r: 795.49 km
ε_0: 1.73 σ
Contribution: 1.47 %

Discretization

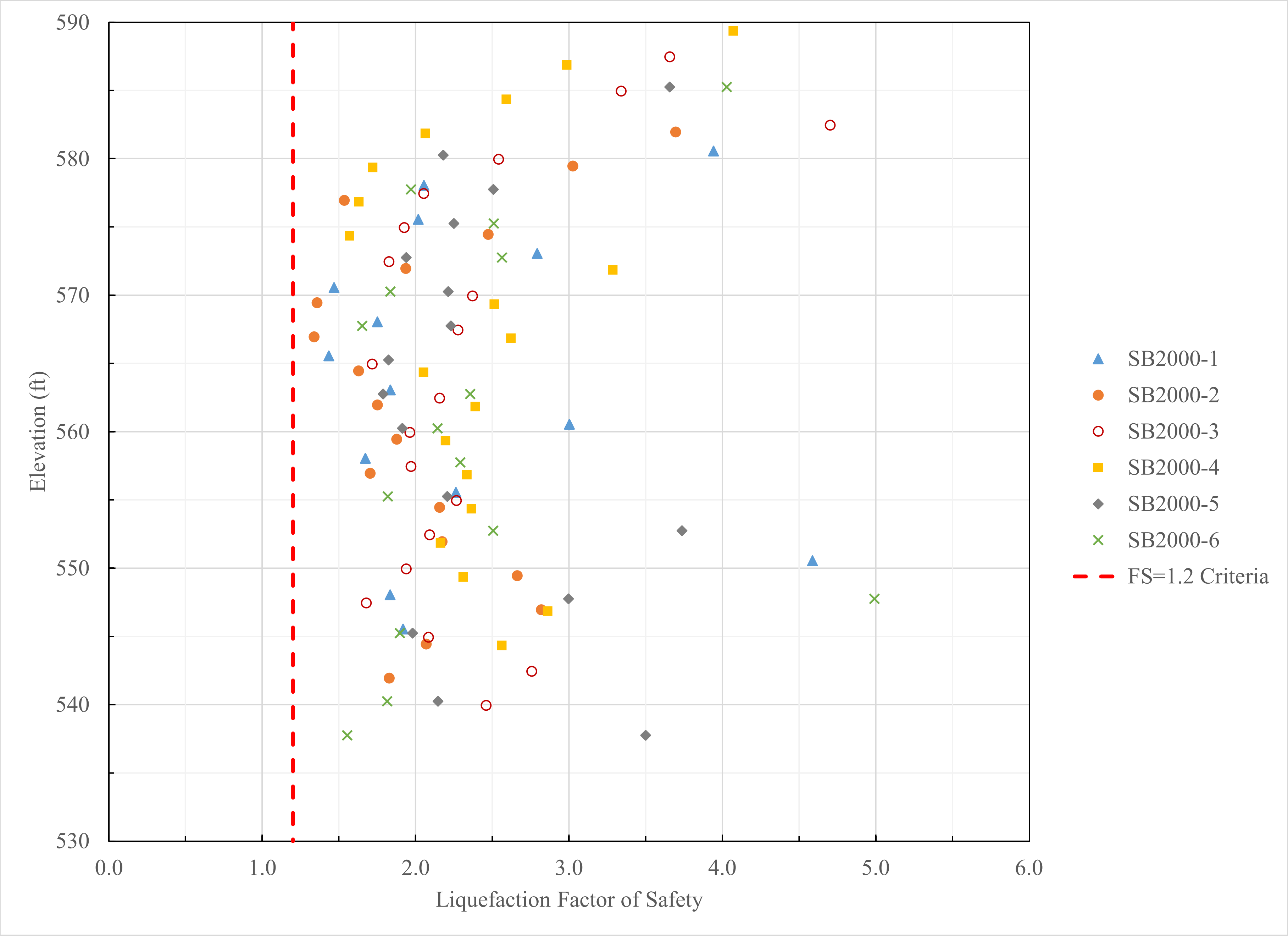
r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ

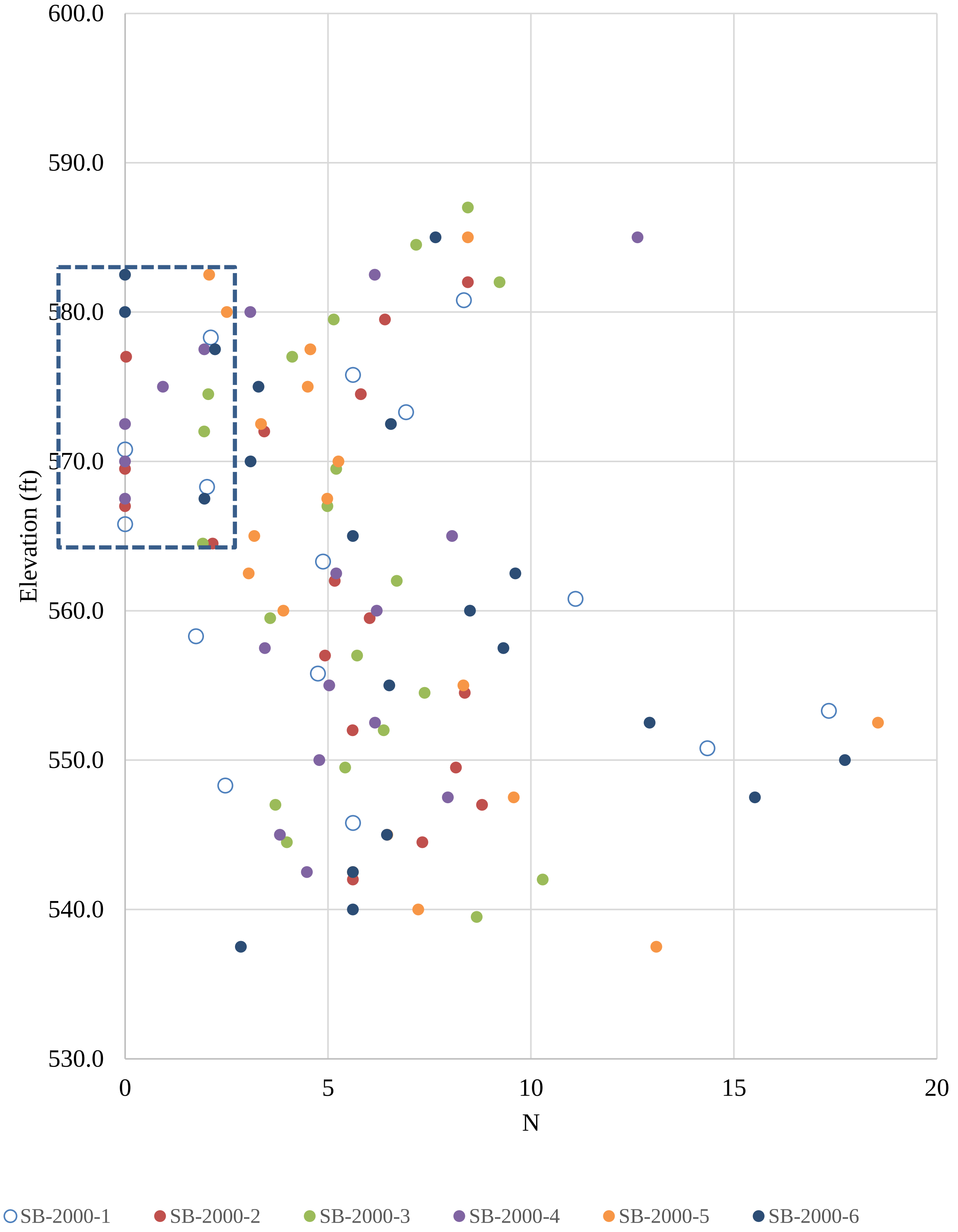
Epsilon keys

ε0: [-∞ .. -2.5)
ε1: [-2.5 .. -2.0)
ε2: [-2.0 .. -1.5)
ε3: [-1.5 .. -1.0)
ε4: [-1.0 .. -0.5)
ε5: [-0.5 .. 0.0)
ε6: [0.0 .. 0.5)
ε7: [0.5 .. 1.0)
ε8: [1.0 .. 1.5)
ε9: [1.5 .. 2.0)
ε10: [2.0 .. 2.5)
ε11: [2.5 .. +∞]

Deaggregation Contributors

Source Set ↳ Source	Type	r	m	ε_0	lon	lat	az	%
USGS Fixed Smoothing Zone 1 (opt)	Grid							27.81
PointSourceFinite: -86.246, 43.641		42.43	5.26	-0.58	86.246°W	43.641°N	0.00	1.04
PointSourceFinite: -86.246, 43.506		27.72	5.17	-1.27	86.246°W	43.506°N	0.00	1.02
SSCn Fixed Smoothing Zone 1 (opt)	Grid							27.48
PointSourceFinite: -86.246, 43.641		42.43	5.26	-0.58	86.246°W	43.641°N	0.00	1.04
PointSourceFinite: -86.246, 43.506		27.72	5.17	-1.27	86.246°W	43.506°N	0.00	1.02
USGS Adaptive Smoothing Zone 1 (opt)	Grid							17.41
SSCn Adaptive Smoothing Zone 1 (opt)	Grid							17.26
SSCn New Madrid	Cluster							3.98
NMFS RLME 1		783.94	7.70	1.97	89.288°W	36.995°N	201.37	1.17
Wabash Valley	Grid							2.25
USGS New Madrid 500-year	Cluster							1.20

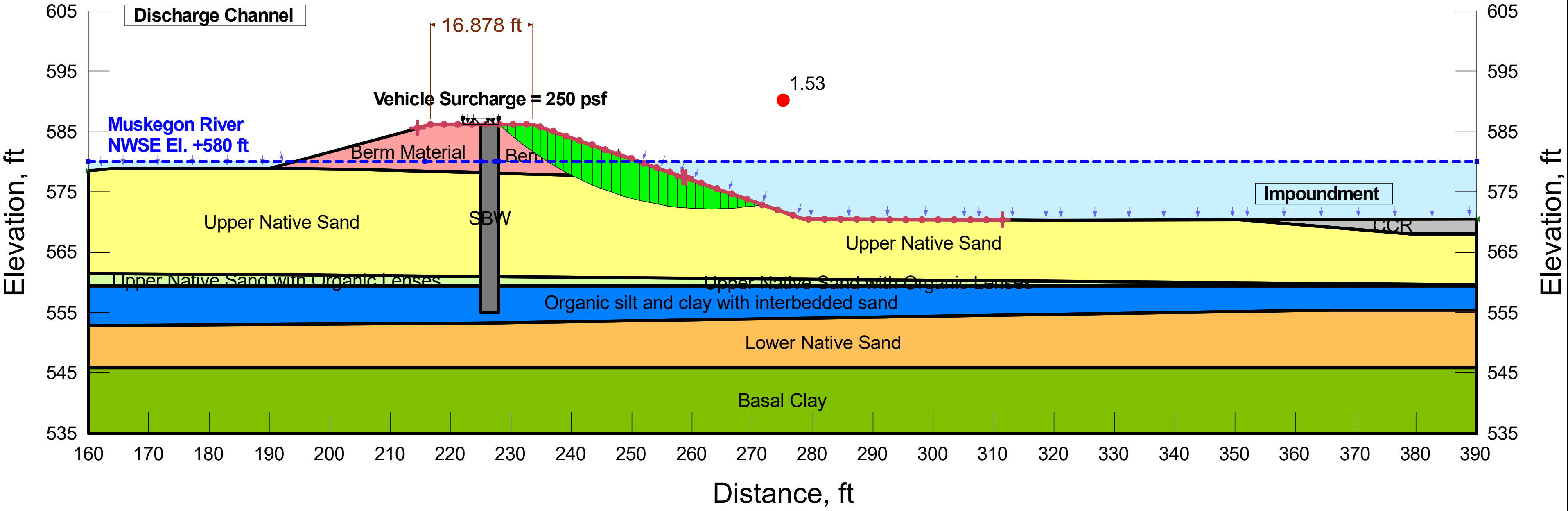

590
580
570
560
550
540
530
Elevation (ft)
0.0
1.0
2.0
3.0
4.0
5.0
6.0
Liquefaction Factor of Safety
SB2000-1
SB2000-2
SB2000-3
SB2000-4
SB2000-5
SB2000-6
FS=1.2 Criteria


600.0
590.0
580.0
570.0
560.0
550.0
540.0
530.0
Elevation (ft)
0
5
10
15
20
N
○ SB-2000-1
● SB-2000-2
● SB-2000-3
● SB-2000-4
● SB-2000-5
● SB-2000-6

ATTACHMENT 3

STABILITY ANALYSES RESULTS

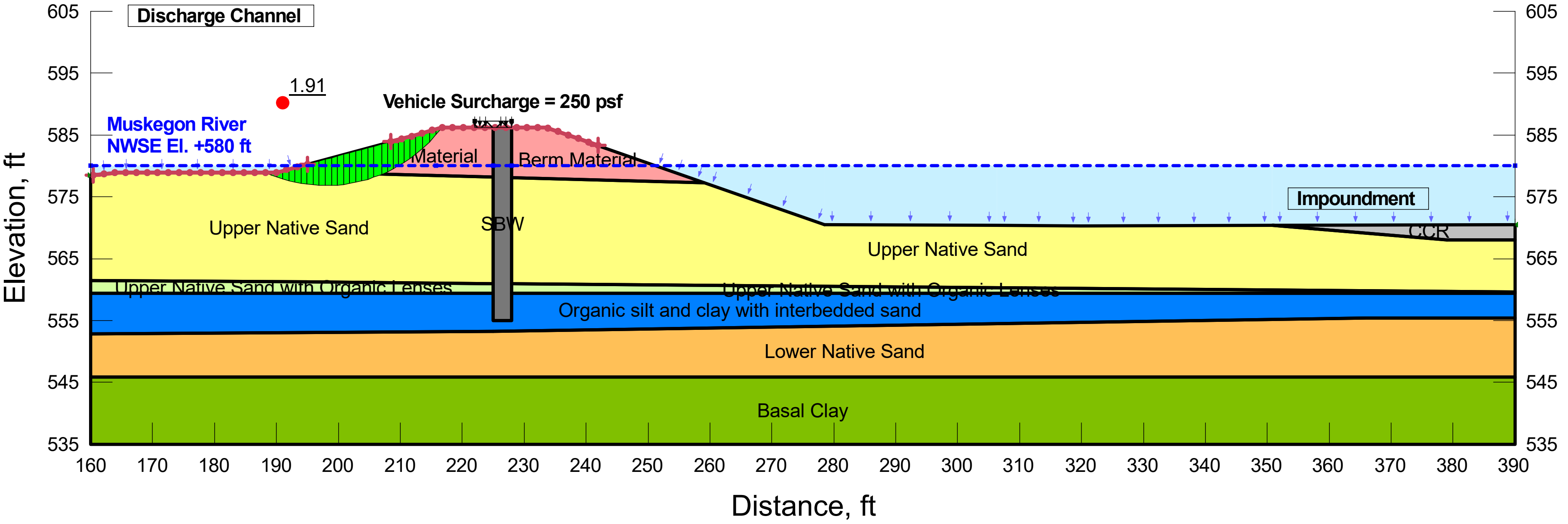
Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Constant Unit Wt. Above Water Table (pcf)
🟩	Basal Clay	109	150	28	107
🟥	Berm Material	120	0	31	120
⬜	CCR	112	0	29	110
🟧	Lower Native Sand	124	0	31	123
🟦	Organic silt and clay with interbedded sand	109	0	27	107
⬛	SBW	120	0	31	
🟨	Upper Native Sand	124	0	27	123
🟩	Upper Native Sand with Organic Lenses	124	0	32	123



Project Name: BC Cobb Ash Pond Closure
Analysis: Ponds 0-8 and Bottom Ash Pond
Project Location: Muskegon, Michigan

File Name: STA 16+50 - Stability.gsz
Method of Analysis: Spencer
Case Analyzed: 1 - Pond Side, NWSE, Long-Term
Minimum FS: 1.53

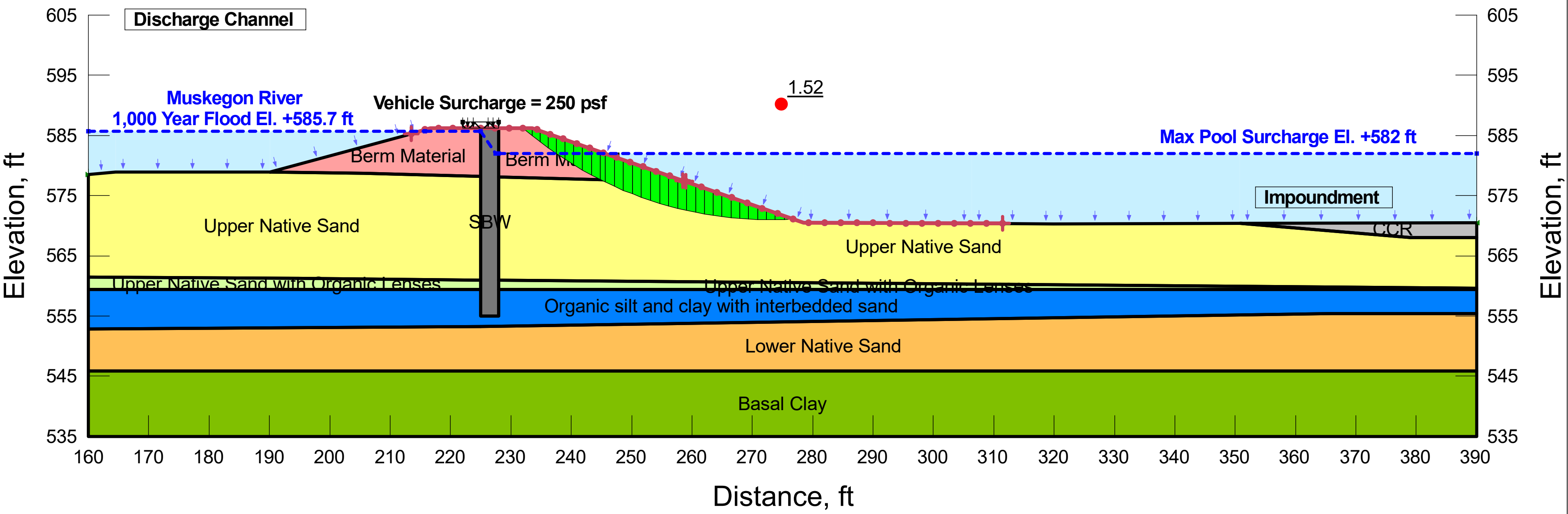
Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Constant Unit Wt. Above Water Table (pcf)
🟩	Basal Clay	109	150	28	107
🟥	Berm Material	120	0	31	120
⬜	CCR	112	0	29	110
🟧	Lower Native Sand	124	0	31	123
🟦	Organic silt and clay with interbedded sand	109	0	27	107
⬛	SBW	120	0	31	
🟨	Upper Native Sand	124	0	27	123
🟩	Upper Native Sand with Organic Lenses	124	0	32	123



Project Name: BC Cobb Ash Pond Closure
Analysis: Ponds 0-8 and Bottom Ash Pond
Project Location: Muskegon, Michigan

File Name: STA 16+50 - Stability.gsz
Method of Analysis: Spencer
Case Analyzed: 2 - River Side, NWSE, Long-Term
Minimum FS: 1.91

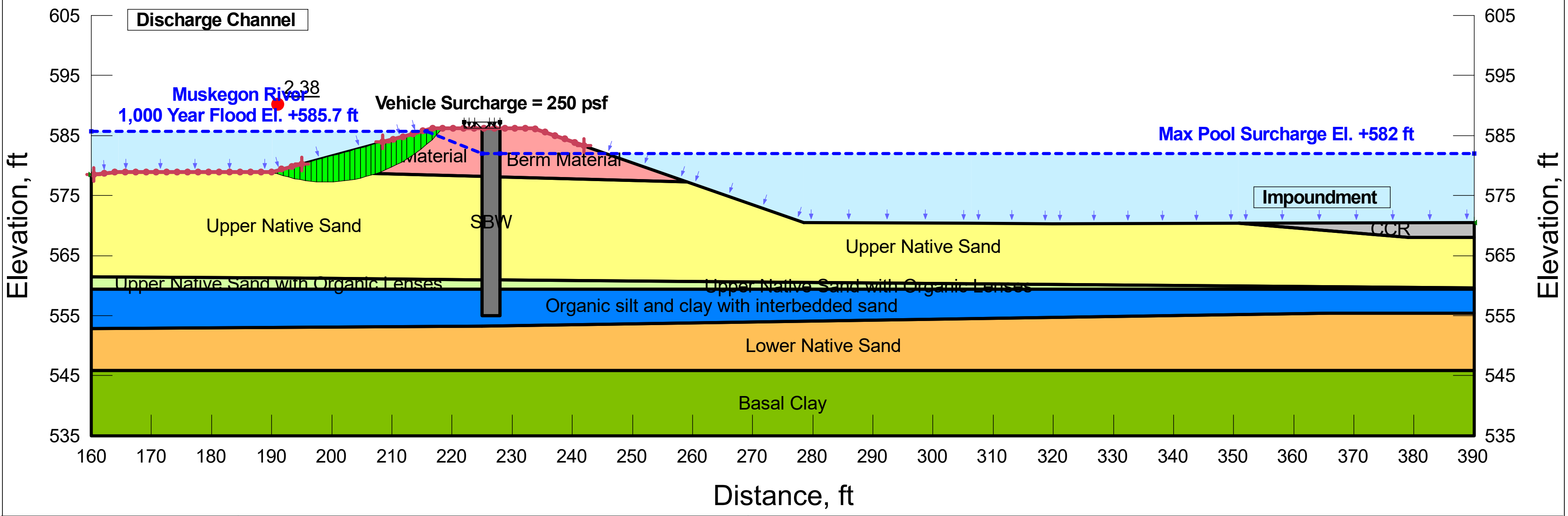
Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Constant Unit Wt. Above Water Table (pcf)
🟩	Basal Clay	109	150	28	107
🟥	Berm Material	120	0	31	120
⬜	CCR	112	0	29	110
🟧	Lower Native Sand	124	0	31	123
🟦	Organic silt and clay with interbedded sand	109	0	27	107
⬛	SBW	120	0	31	
🟨	Upper Native Sand	124	0	27	123
🟩	Upper Native Sand with Organic Lenses	124	0	32	123



Project Name: BC Cobb Ash Pond Closure
Analysis: Ponds 0-8 and Bottom Ash Pond
Project Location: Muskegon, Michigan

File Name: STA 16+50 - Stability.gsz
Method of Analysis: Spencer
Case Analyzed: 3 - Pond Side, 1000Yr Flood, Long-Term
Minimum FS: 1.52

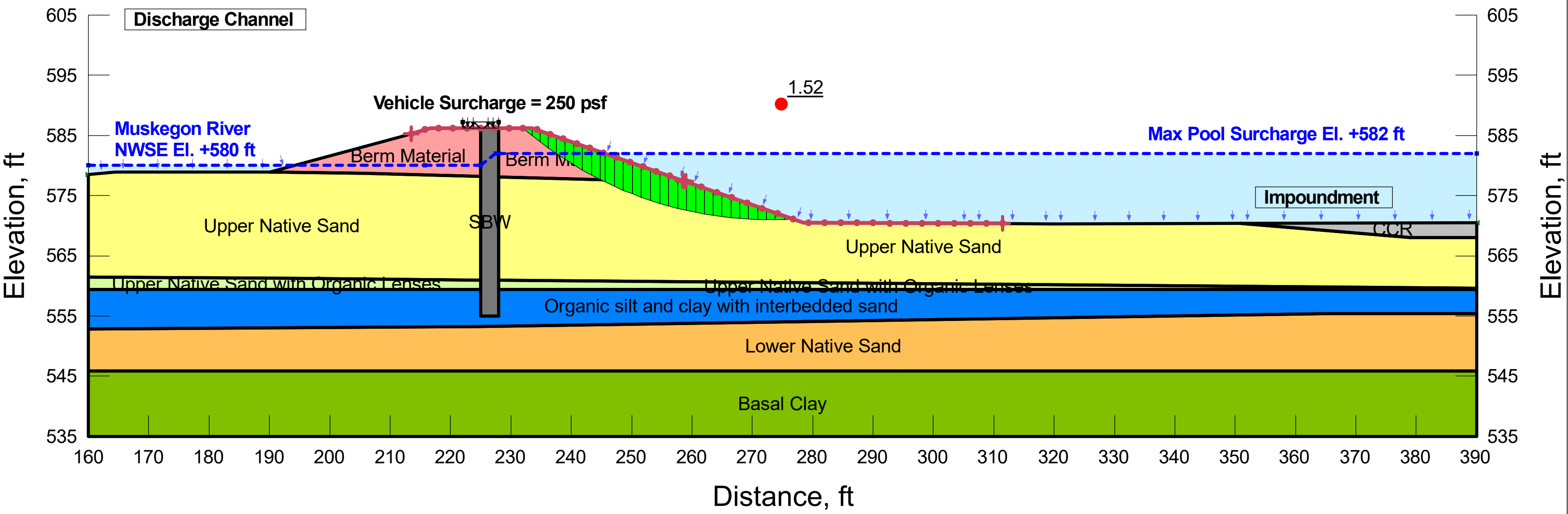
Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Constant Unit Wt. Above Water Table (pcf)
🟩	Basal Clay	109	150	28	107
🟥	Berm Material	120	0	31	120
⬜	CCR	112	0	29	110
🟧	Lower Native Sand	124	0	31	123
🟦	Organic silt and clay with interbedded sand	109	0	27	107
⬛	SBW	120	0	31	
🟨	Upper Native Sand	124	0	27	123
🟩	Upper Native Sand with Organic Lenses	124	0	32	123



Project Name: BC Cobb Ash Pond Closure
Analysis: Ponds 0-8 and Bottom Ash Pond
Project Location: Muskegon, Michigan

File Name: STA 16+50 - Stability.gsz
Method of Analysis: Spencer
Case Analyzed: 4 - River Side, 1000Yr Flood, Long-Term
Minimum FS: 2.38

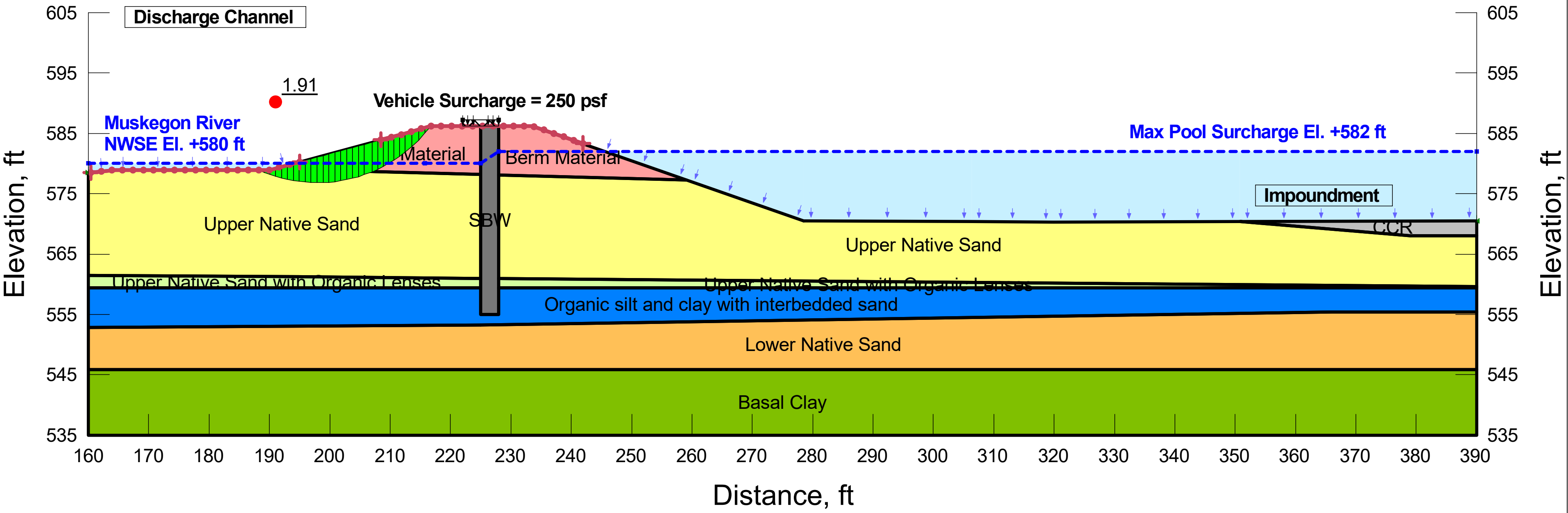
Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Constant Unit Wt. Above Water Table (pcf)
🟩	Basal Clay	109	150	28	107
🟥	Berm Material	120	0	31	120
⬜	CCR	112	0	29	110
🟧	Lower Native Sand	124	0	31	123
🟦	Organic silt and clay with interbedded sand	109	0	27	107
⬛	SBW	120	0	31	
🟨	Upper Native Sand	124	0	27	123
🟩	Upper Native Sand with Organic Lenses	124	0	32	123



Project Name: BC Cobb Ash Pond Closure
Analysis: Ponds 0-8 and Bottom Ash Pond
Project Location: Muskegon, Michigan

File Name: STA 16+50 - Stability.gsz
Method of Analysis: Spencer
Case Analyzed: 5 - Pond Side, Max Pool Surcharge, Long-Term
Minimum FS: 1.52

Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Constant Unit Wt. Above Water Table (pcf)
🟩	Basal Clay	109	150	28	107
🟥	Berm Material	120	0	31	120
⬜	CCR	112	0	29	110
🟧	Lower Native Sand	124	0	31	123
🟦	Organic silt and clay with interbedded sand	109	0	27	107
⬛	SBW	120	0	31	
🟨	Upper Native Sand	124	0	27	123
🟩	Upper Native Sand with Organic Lenses	124	0	32	123

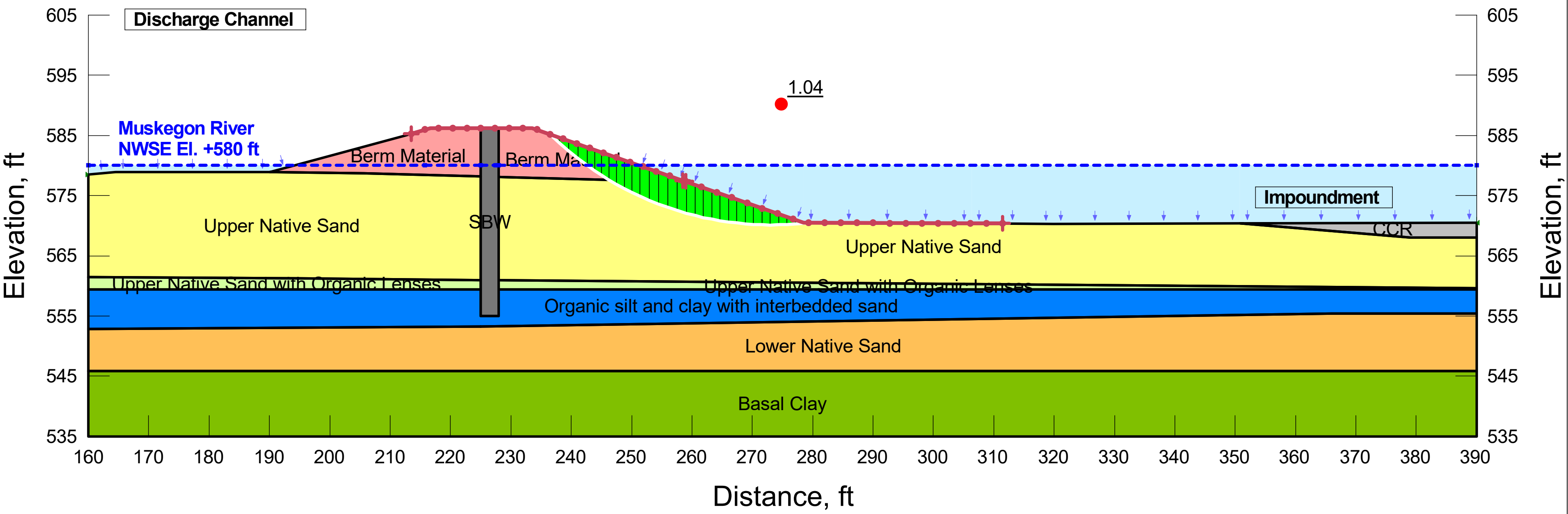


Project Name: BC Cobb Ash Pond Closure
Analysis: Ponds 0-8 and Bottom Ash Pond
Project Location: Muskegon, Michigan

File Name: STA 16+50 - Stability.gsz
Method of Analysis: Spencer
Case Analyzed: 6 - River Side, Max Pool Srucharge, Long-Term
Minimum FS: 1.91

Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Constant Unit Wt. Above Water Table (pcf)
🟩	Basal Clay	109	150	28	107
🟥	Berm Material	120	0	31	120
⬜	CCR	112	0	29	110
🟧	Lower Native Sand	124	0	31	123
🟦	Organic silt and clay with interbedded sand	109	0	27	107
⬛	SBW	120	0	31	
🟨	Upper Native Sand	124	0	27	123
🟩	Upper Native Sand with Organic Lenses	124	0	32	123

Seismic, 2475-year Return Period
Horizontal Seismic Coefficient = 0.079

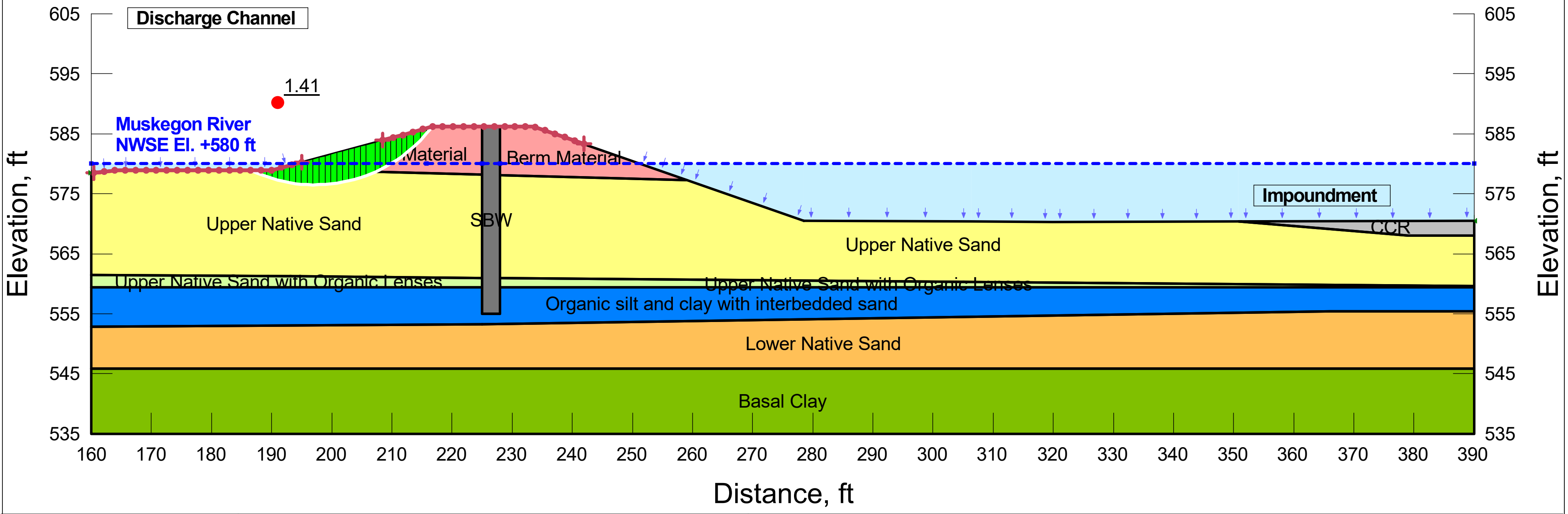


Project Name: BC Cobb Ash Pond Closure
Analysis: Ponds 0-8 and Bottom Ash Pond
Project Location: Muskegon, Michigan

File Name: STA 16+50 - Stability.gsz
Method of Analysis: Spencer
Case Analyzed: 7 - Pond Side, NWSE, Seismic-2475 year
Minimum FS: 1.04

Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Constant Unit Wt. Above Water Table (pcf)
🟩	Basal Clay	109	150	28	107
🟥	Berm Material	120	0	31	120
⬜	CCR	112	0	29	110
🟧	Lower Native Sand	124	0	31	123
🟦	Organic silt and clay with interbedded sand	109	0	27	107
⬛	SBW	120	0	31	
🟨	Upper Native Sand	124	0	27	123
🟩	Upper Native Sand with Organic Lenses	124	0	32	123

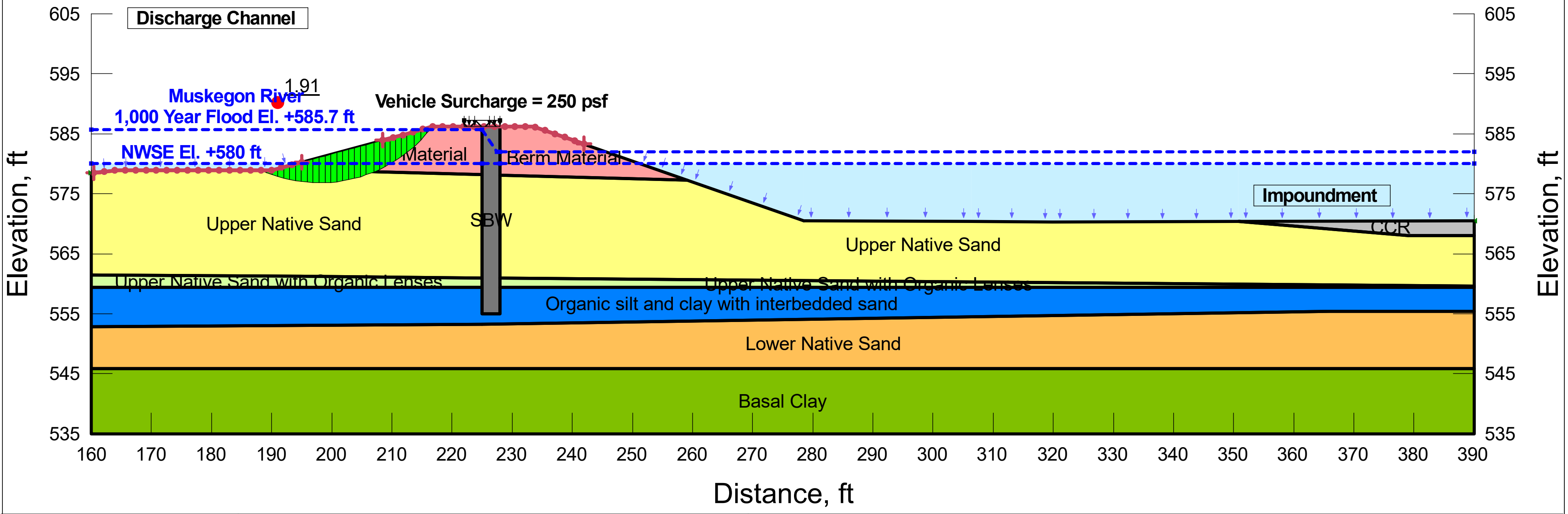
Seismic, 2475-year Return Period
Horizontal Seismic Coefficient = 0.079



Project Name: BC Cobb Ash Pond Closure
Analysis: Ponds 0-8 and Bottom Ash Pond
Project Location: Muskegon, Michigan

File Name: STA 16+50 - Stability.gsz
Method of Analysis: Spencer
Case Analyzed: 8 - River Side, NWSE, Seismic-2475 year
Minimum FS: 1.41

Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Cohesion R (psf)	Phi R (°)	Constant Unit Wt. Above Water Table (pcf)
🟩	Basal Clay	109	150	28	500	0	107
🟥	Berm Material	120	0	31	0	31	120
⬜	CCR	112	0	29	0	29	110
🟧	Lower Native Sand	124	0	31	0	31	123
🟦	Organic silt and clay with interbedded sand	109	0	27	0	27	107
⬛	SBW	120	0	31	0	31	
🟨	Upper Native Sand	124	0	27	0	27	123
🟩	Upper Native Sand with Organic Lenses	124	0	32	0	32	123



Project Name: BC Cobb Ash Pond Closure
Analysis: Ponds 0-8 and Bottom Ash Pond
Project Location: Muskegon, Michigan

File Name: STA 16+50 - Stability.gsz
Method of Analysis: Spencer
Case Analyzed: 9 - River Side, Rapid Drawdown
Minimum FS: 1.91