REPORT

NIPSCO MICHIGAN CITY GENERATING STATION

PRIMARY SETTLING POND NO. 2 (PRIMARY 2)

STRUCTURAL STABILITY AND SAFETY FACTOR ASSESSMENT

Michigan City, Indiana

Pursuant to 40 CFR 257.73(d) & 257.73(e)

Submitted To: Northern Indiana Public Service Company
2755 Raystone Drive
Valparaiso, IN 46383

Submitted By: Golder Associates Inc.
15851 South US 27, Suite 50
Lansing, MI 48906 USA

March 5, 2018 **Project No. 1787453**

CERTIFICATION

Professional Engineer Certification Statement [40 CFR 257.73(d)(3) & 257.73(e)(2)]

I hereby certify that, having reviewed the attached documentation and being familiar with the provisions of Title 40 of the Code of Federal Regulations Section 257.73 (40 CFR Part 257.73), I attest that this Structural Stability and Safety Factor Assessment Report is accurate has been prepared in accordance with good engineering practices, including the consideration of applicable industry standards, and with the requirements of 40 CFR Part 257.73(d) periodic structural stability assessments and 40 CFR Part 257.73(e) periodic safety factor assessments.

Golder Associates Inc.

Signature

March 5, 2018
Date of Report Certification

Tiffany Johnson, PE
Name

PE 11500730
Indiana Professional Engineer License Number

TIFFANY D. JOHNSON
No. PE11500730
3/5/18

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1.0 INTRODUCTION

1.1 Purpose

The United States Environmental Protection Agency (EPA) published 40 CFR Part 257 – Coal Combustion Residuals (CCR) Final Rule (CCR RCRA Rule) in April 2015 to regulate the solid waste management of CCR generated at electric utilities. The CCR RCRA Rule requires that existing CCR surface impoundments meeting the requirements of Section 257.73(b) conduct initial and periodic structural stability assessments in accordance with Section 257.73(d), and safety factor assessments in accordance with Section 257.73(e). Per rule 257.73(b), this initial stability assessment and factor of safety is required for all CCR units with either (1), a height of five feet or more and a storage volume of 20 acre-feet or more; or (2) a height of 20 feet or more. Golder Associates Inc. (Golder) was retained by the Northern Indiana Public Service Company (NIPSCO) to perform the initial structural stability assessment and the safety factor assessment for the Primary Settling Pond Number 2 (Primary 2) surface impoundment at the Michigan City Generating Station (MCGS or Site). This assessment is being provided at this time in accordance with 81 FR 51802.

This report provides the initial structural stability assessment and the safety factor assessment for the Primary 2 surface impoundment at the NIPSCO MCGS, located in Michigan City, Indiana, see Figures 1 and 2. The Primary 2 is an above grade surface impoundment that is not currently regulated by the Indiana Department of Water Dam Regulations (IDOW). A hazard potential classification was conducted for the Primary 2 pursuant to Section 257.73(a)(2) (Golder, 2017a), which resulted in a significant hazard classification thereby requiring the 1,000-year recurrence interval flood elevation to be used in structural assessment.

1.2 Sources of Information

NIPSCO provided Golder with geotechnical data from historic hydrogeologic and geotechnical investigation reports completed at the site by others. These include boring logs from the initial 1970s facility design/construction efforts in areas that are near the current investigation and the *Pre-RCRA Facility Investigation Study Report* (Geraghty and Miller, 1995). Additionally, Golder performed a geotechnical investigation at the Site in 2012 and prepared the *2012 Geotechnical Investigation and Embankment Stability Analyses* report, dated August 27, 2012.

A list of reviewed documents pertinent to the structural stability assessment is provided in Table 1.

Table 1: Previous Evaluations Related to Structural Stability Assessment

Document	Date	Author
Historical Boring Logs	1970	Sargent & Lundy Engineers
Various Construction Drawings	1972	Sargent & Lundy Engineers
Pre-RCRA Facility Investigation Study Report Northern Indiana Public Service Company Michigan City Generating Station	1995	Geraghty & Miller, Inc.
Final Round 10 Dam Assessment Report Michigan City Generating Station Coal Ash Impoundments	2012	GZA GeoEnvironmental, Inc. for the EPA
2012 Geotechnical Investigation and Embankment Stability Analyses NIPSCO Michigan City Generating Station	2012	Golder Associates Inc.
Embankment, Soil Boring & Monitoring Well Survey; NIPSCO Michigan City Generating Station.	2015	Marbach, Brady, & Weaver, Inc.
Initial Annual RCRA CCR Unit Inspection Report Primary Settling Pond No. 2 – Surface Impoundment	2017	Golder Associates Inc.
Michigan City Generating Station Initial Hazard Potential Classification Assessment – RCRA CCR Units, Primary Settling Pond Number 2 – Surface Impoundment	2018	Golder Associates Inc.
Michigan City Generating Station – RCRA CCR Units, Primary Settling Pond Number 2 - CCR Surface Impoundment Inflow Design Flood Control System Plan.	2018	Golder Associates Inc.

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2.0 SITE DESCRIPTION

The NIPSCO MCGS is an operating coal-fired electric generating plant located at 101 Wabash Street in Michigan City, La Porte County, Indiana. The Site occupies 131 acres in a mixed industrial, commercial, and residential area along the southern shoreline of Lake Michigan. The facility includes power generation and transmission facilities; buildings and associated infrastructure; coal storage and handling operations; pollution control equipment; and various active, inactive, and closed surface impoundments.

The Primary 2 is located northwest of the coal storage pile and southeast of the Lake Michigan shoreline. The Primary 2 is flanked to the southwest and northeast by Secondary Settling Pond No. 1 and Secondary Settling Pond No. 2, respectively, see Figure 2. The Primary 2 was designed by Sargent & Lundy Engineers (S&L) of Chicago, Illinois in 1972, and put into service in 1973. It has been continuously owned and operated by NIPSCO.

The Primary 2 is currently used as storage for air heater wash, reverse osmosis waste water and boiler room sump water. Primary 2 is currently not accepting CCR materials. When operating, water is pumped into the Primary 2, via an aboveground steel pipe, from the generating station. There is one concrete discharge structure in Primary 2 that utilizes stop logs to control water elevation and currently discharges to the Final Settling Pond.

2.1 Subsurface Conditions

The Site is located near the eastern end of the physiographic region of Indiana known as the Calumet Lacustrine Plain and is underlain by more than 200 feet of unconsolidated glacial and lacustrine sediments. The plain is a topographically-low region bordering Lake Michigan, and is a remnant of the Lake Chicago stage of the Wisconsinan glaciations. The geology of the plain is characterized by complex clay, sand, and silt deposits, ranging from ground moraines to aeolian sand and silt, as the shoreline of glacial Lake Chicago moved with its rising and falling stage.

2.2 Physical Properties of Foundation Materials

Twelve Hollow Stem Auger (HSA) boreholes were drilled in and around several of the embankments at the MCGS in 2012 (Golder, 2012). Three of these boreholes (BH-5, BH-6, and BH-7) were drilled from near the center of the crest of the west, north and east embankments of Primary 2. Boreholes BH-10, BH-11 and BH-12 were also advanced downstream of the north embankment. BH-5, BH-6 and BH-7 were advanced to depths of 50 feet below ground surface (ft bgs), 50 ft bgs, and 40 ft bgs, respectively.

These boreholes indicate the subsurface material consists of dense to very dense slag and sand from ground surface to approximately 1 ft bgs. Immediately below this upper layer is a layer of loose to medium dense sand, approximately 35 ft thick. Borings BH-5, BH-6, BH-7 indicate that below the upper sand layer is a layer of dense to very dense black ash 2.5 ft to 5 ft thick. Below the ash layer is a discontinuous layer of dense to very dense sand as indicated in BH-5, BH-10, BH-11, and BH-12. This layer was not present in boreholes BH-6 or BH-7. BH-7 was terminated at a depth of 40 ft bgs (approximately elevation 570 ft above mean sea level; amsl) and neither the black ash nor underlying units were encountered. The borehole log for BH-6 notes that a medium stiff clay layer is present below the black ash to the end of the boring at 50 ft bgs (approximately elevation 560 ft above mean sea level; amsl). The borehole log for BH-10 notes that the clay later extends to approximately elevation 535 ft amsl (75 feet below the embankment crest).

During drilling activities associated with the 2012 Geotechnical Report (Golder, 2012), groundwater was observed and measured at 8.5 ft bgs in BH-5 (north embankment). Groundwater was observed and measured at 18 ft bgs in BH-6 (west embankment) and 11.5 ft bgs in BH-7 (east embankment).

2.3 Engineering Properties of Foundation Materials

Historic construction drawings, data from subsurface investigations, and inspection reports indicate that Primary 2 was constructed with reasonable and sound construction practices. The design drawings (S&L, 1972) indicate reasonable construction configurations, e.g. 2.5 horizontal to 1 vertical (2.5H:1V) upstream and downstream side slopes; embankment constructed of controlled compacted fill; sheet piling along the west embankment; reinforced concrete structures at the inlet and outlet pipes; and engineered surface water control around the structure. Site investigations and inspections indicate the west embankment is reinforced by two rows of steel sheet piling and armored against erosion by riprap.

The 2012 Geotechnical Report (Golder, 2012) summarizes available subsurface information and presents engineering properties of the foundation materials based on the geotechnical investigation performed in 2012, associated laboratory testing, and empirical correlations to published data sources. The engineering properties presented in the 2012 Geotechnical Report were used to perform the geotechnical analyses summarized herein. Additional engineering properties not presented in the 2012 Geotechnical Report were developed based on available in-situ and laboratory testing data and published data sources.

Table 2 summarizes the engineering properties of foundation materials used in the geotechnical analyses presented herein:

Table 2: Material Properties used in Geotechnical Analyses

Material	Internal Friction Angle, ϕ (deg)	Peak Cohesion, c (psf)	Dry Unit Weight, γ_d (pcf)	Saturated Unit Weight, γ_{sat} (pcf)	Undrained Shear Strength, s_u (psf)	Hydraulic Conductivity, k (cm/s)
Loose to Medium Dense Fill (SP)	33	0	100	110	NA	1x10^{-3}
Bottom Ash Fill-Medium Dense	35	0	100	110	NA	1X10^{-3}
Clay-Medium to V. Stiff	30	70	116	136	750 – 2500	1x10^{-6}
Native Sand-Dense	40	0	110	120	NA	1x10^{-3}
Crushed Blast Furnace Slag	40	0	120	130	NA	1
8-inch Riprap	45	0	140	145	NA	100
Limestone Riprap	45	0	140	145	NA	100
Steel Sheeting	NA	NA	120	NA	NA	NA

Notes: deg = degrees, psf = pounds per square foot, pcf = pounds per cubic foot, ft = feet, and cm/s = centimeters per second.

2.4 Primary 2 Design and Construction Details

The Primary 2 was designed by Sargent & Lundy Engineers of Chicago, Illinois in 1972. The structure was constructed for NIPSCO and has been continuously owned and operated by NIPSCO since it was placed into service in 1973. Available applicable construction drawings (S&L, 1972) provided by NIPSCO were reviewed and utilized during the preparation of this report.

The Primary 2 is formed by an above-grade earthfill embankment, the crest of which is at approximate elevation 609 ft amsl. The embankment is approximately 14 feet high on the outside and approximately 19 feet high on the inside. The interior and exterior slopes are approximately 2.5 horizontal to 1 vertical (2.5H:1V). The west portion of this embankment separates Primary 2 from Lake Michigan and incorporates two rows of sheet piling, between which heavy riprap has been placed. The Secondary Settling Pond numbers 1 and 2 are located southwest and northeast of the Primary 2, respectively. The MCGS coal storage area is located east of the Primary 2. Primary 2 is currently used as storage for air heater wash, reverse osmosis waste water, and boiler room sump water. Primary 2 is currently not receiving CCR materials. There is one discharge structure located on the pond's north side. The discharge structure is concrete and utilizes stop logs to control water elevation. Flow through the discharge structure reports to the Final Settling Pond.

A topographic survey of embankments at MCGS was performed in 2015 by Marbach, Brady & Weaver, Inc. (Marbach, 2015). Note that the vertical datum used for the 2015 survey was the North American Vertical Datum of 1988 (NAVD88), whereas the construction drawings (S&L, 1972) reference the National Geodetic Vertical Datum of 1929 (NGVD29). The location of Primary 2 relative to the generating station and surrounding structures is shown on Figure 2.

SIZE AND PHYSICAL DATA

Designed Crest Elevation:	609 ft amsl (NGVD29; S&L, 1972 and GZA, 2012)
Current Lowest Crest Elevation:	609 ft amsl (NAVD88; Marbach, 2015)
Surrounding Ground Elevation:	Approximately 596 ft amsl – 602 ft amsl (NAVD88; Marbach, 2015)
Operating Water Level:	605.79 (Golder, 2017b)
Dike Height:	14 feet above the ground surface (S&L, 1972)
Pond Depth:	19 feet (S&L, 1972)
Surface Area:	3.4 acres (Golder, 2017b)
Reservoir Volume:	55 acre-feet (Golder, 2017b)

3.0 STRUCTURAL STABILITY ASSESSMENT - § 257.73(d)(1)(I)-(VII)

The CCR Rule requires an initial and periodic structural stability assessments be conducted by a qualified professional engineer (QPE) to document whether the design, construction, operation and maintenance is consistent with recognized and generally accepted good engineering practices for the maximum volume of CCR and CCR wastewater that can be impounded therein. The following sections provide documentation on the initial structural stability assessment and rely mainly on the recent and historic annual inspections performed at the site. The most recent inspection was completed by Golder on May 30, 2017 for the initial structural stability assessment (Golder, 2017c).

3.1 Foundations and Abutments - §257.73(d)(1)(i)

Based on the available construction drawings (S&L, 1972) and borehole logs (Golder 2012), the Primary 2 embankment is constructed of compacted sand and ash/slag materials obtained from on-site borrow areas. The embankment footprint was stripped prior to embankment construction (GZA, 2012). The interior of Primary 2 is lined with a layer of compacted blast furnace slag. There has been no indication of foundational or abutment instability or movement in recent or historic site inspections and; therefore, the foundation soils and abutments are considered stable.

3.2 Slope Protection - §257.73(d)(1)(ii)

The downstream slope of the north, south, and east portions of the Primary 2 embankment is protected from erosion and deterioration by a layer of compacted blast furnace slag. The west embankment is protected against erosion and wave action by two rows of sheet piling and riprap. The embankment is inspected by NIPSCO personnel at least every seven days for signs of erosion, seepage, animal burrows, sloughing, and plants that could negatively impact the embankment. The only items repair identified in the May 2017 inspection (Golder, 2017c) relating to slope protection that required investigation or repair were minor vegetation within the rip-rap and minor erosion on the upstream slope. The existing slope protection measures are considered adequate to provide against surface erosion, wave action, and adverse effects of rapid drawdown. Furthermore, the potential for failure of the upstream slope caused by rapid drawdown of water within Primary 2 was evaluated as part of the slope stability analysis discussed in Section 4.1. The results of that analysis indicate that the stability of the slope against such a failure are adequate. Additional discussion of the slope stability analysis are presented in Section 4.1 and Appendix B.

3.3 Dikes (Embankment) - §257.73(d)(1)(iii)

Based on the available construction drawings (S&L, 1972) and borehole logs (Golder 2012), the Primary 2 embankment is constructed of compacted sand and ash/slag materials obtained from on-site borrow areas. The embankment footprint was stripped prior to embankment construction (GZA, 2012). The interior of Primary 2 is lined with a layer of compacted blast furnace slag. Based on the relative density of the material encountered during the investigations, historic inspections, recent observations, and results of the stability analysis; the embankment dikes are considered sufficient to withstand the range of loading conditions in Primary 2.

3.4 Vegetated Slopes - §257.73(d)(1)(iv)

The EPA has removed the requirement from the CCR Rule that vegetative cover on surface impoundment dikes be maintained at no more than six inches. At the time of the May 2017 inspection, the Primary 2's downstream slopes were covered by alternate (non-vegetative) slope protection (blast furnace slag). A new rule establishing requirements relating to the use of vegetation as slope protection for CCR surface impoundments is still pending.

3.5 Spillways - §257.73(d)(1)(v)

The principal spillway of Primary 2 is a concrete discharge structure in the north end of the impoundment. The discharge structure utilizes stop logs to control water elevation and flow through structure reports to the Final Settling Pond. Based on visual observations made on May 30, 2017, no safety issues were observed within the hydraulic structures of the Primary 2. Available information indicates the outlet conduit is a 24-inch diameter corrugated metal pipe. At the time of the May 2017 inspection, the water level in the Primary 2 was observed to be approximately 11 feet deep (elevation 601 ft amsl).

A hydrologic and hydraulic analysis was completed for Primary 2 as part of the requirements for CCR Rule 257.73(d)(1)(v)(B) and 257.82. Per the CCR Rule, the combined capacity of all spillways must adequately manage flow during and following peak discharge from a:

- Probable maximum flood (PMF) for a high hazard potential CCR surface impoundment; or
- 1,000-year flood for a significant hazard potential CCR surface impoundment; or
- 100-year flood for a low hazard potential CCR surface impoundment.

Since Primary 2 has been classified as a significant hazard potential CCR surface impoundment, (Golder, 2017a), it is required to manage the flow during and following the peak discharge from a 1,000-year recurrence interval flood event. A HEC-HMS (USACE, 2015) analysis and wave analysis was performed for Primary 2. The results of this analysis are presented in the Inflow Design Flood Control Plan (Golder, 2017b) and summarized below in Table 3.

Table 3: Hydrology and Hydraulics Analysis Results

Depth of Precipitation (in) for 1,000-year Flood Event	10.4
Primary 2 Catchment Area (acres)	4.1
Primary 2 Lowest Crest Elevation (ft amsl)	608.7
Maximum pool storage elevation (ft amsl)	605.7
Maximum Inflow to Primary 2 (cubic feet per second; cfs)	59.8
Maximum Primary 2 Outflow through Spillway (cfs) [1]	0.0
Maximum Water Surface Elevation (ft amsl)	606.8
Height of Wave Action (feet)	0.44
Net Freeboard during Design Storm Event (feet) [2]	1.4
Maximum Allowable Operational Water Surface Elevation (ft amsl)	607.1

Notes:
[1] The analysis presented in Golder (2017b) assumed there to be no outflow from Primary 2 during the design storm event.
[2] Net freeboard = lowest crest elevation minus the maximum water surface elevation and the height of wave action.

As shown in Table 3, the current configuration and operational status of Primary 2 meet the requirements of 40 CFR 257.73(d)(1)(v).

3.6 Hydraulic Structures - §257.73(d)(1)(vi)

CCR Rule 257.73(d)(1)(vi) requires that "*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.*" No hydraulic structures underlie the base of Primary 2 and the only hydraulic structure passing through the dike is the corrugated metal outlet pipe. No deterioration, sedimentation, debris or other deficiencies were observed in this pipe during the May 2017 inspection.

3.7 Downstream Slopes Adjacent to Water Body - §257.73(d)(1)(vii)

The downstream slope of the west embankment adjacent to Lake Michigan is reinforced with two rows of metal sheet piling and armored with riprap. As such this slope will maintain structural stability during low pool of Lake Michigan. Rapid drawdown of Lake Michigan is not considered a potential mechanism for slope instability, due to the large size of Lake Michigan.

3.8 Structural Stability Deficiencies - §257.73(d)(2)

In accordance with the CCR Rule 257.73(d)(2), the periodic assessment 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 structural stability assessment contained herein, no structural stability deficiencies were identified for Primary 2.

4.0 SAFETY FACTOR ASSESSMENT - § 257.73(e)

According to Section 257.73(e)(1) of the CCR RCRA Rule, periodic safety factor assessments must be conducted for each CCR unit. The safety factor assessment must document the calculated factor of safety for the dike slopes under the following scenarios:

- Maximum Pool Storage - Section 257.73(e)(1)(i) – Defined as the long-term, maximum storage pool (or operating) elevation. Due to the inactive operational status of Primary 2 and the ability to alter the decant elevation of the outlet structure, the water surface elevation associated with the Maximum Pool Storage is assumed to be 605.7 ft amsl; static factor of safety must equal or exceed 1.5
- Maximum Pool Surcharge - Section 257.73(e)(1)(ii) – Defined as the temporary raised pond level above the maximum pool storage elevation due to an inflow design flood (606.8 ft amsl); static factor of safety must equal or exceed 1.4
- Seismic Loading Conditions - Section 257.73(e)(1)(iii) – Seismic factor of safety must equal or exceed 1.0
- Liquefaction Potential - Section 257.73(e)(1)(iv) – Only necessary for dikes constructed of soils that have susceptibility to liquefaction; factor of safety must equal or exceed 1.2

Additionally, the safety factor assessment calculated the factor of safety for the dike slopes during a scenario involving rapid drawdown of water stored within the impoundment. 40 CFR 257 does not specify a minimum acceptable factor of safety for the rapid drawdown scenario. However, factors of safety as low as 1.0 to 1.2 are generally considered acceptable for rapid drawdown (Duncan and Wright, 2005 after USACE, 2003). A minimum acceptable factor of safety of 1.2 was selected for this analysis.

The following sections provide details on the factor of safety assessment and methods used to calculate the slope factor of safety and results of the analysis. Additionally, the factor of safety for the embankment during rapid drawdown conditions was calculated

4.1 Slope Stability Analysis

Analyses were performed for the loading cases on the critical section for Primary 2, see Appendix A. The analyses calculated factors of safety along both circular and planar (block) failure surfaces. The search method of analysis was used, and several thousand trial surfaces for each case and each model were evaluated by the program.

The results of the analyses calculated that the embankment for Primary 2 has adequate factors of safety given the strength parameters used and the conditions analyzed.

A summary of the lowest factors of safety for each scenario analyzed for Primary 2 is included in Appendix A and in Table 4 below.

Table 4: Slope Stability Analysis Results Summary

Scenario	Minimum Calculated Factor of Safety		Minimum Acceptable Factor of Safety
	Circular Failure Surface	**Planar (Block) Failure Surface**	
Static Maximum **Storage** Pool - 257.73(e)(1)(i)	2.0	1.9	1.5
Static Maximum **Surcharge** Pool - 257.73(e)(1)(ii)	1.9	1.6	1.4
Static Maximum **Storage** Pool – Rapid Drawdown 257.73(d)(1)(ii)	1.3	1.3	1.2
Seismic (Pseudo-Static) Maximum **Storage** Pool - 257.73(e)(1)(iii)	1.6	1.5	1.0

Appendix A presents outputs from the slope stability analysis model.

4.2 Liquefaction Potential Assessment

Embankment and foundation soils were assessed for seismically-induced liquefaction susceptibility using the state-of-practice “simplified method,” based on the original work by R.B. Seed and later updated by Idriss and Boulanger et al. (2008). In general, this method employs the effective stress, relative density, and fines content (percentage of soil passing the No. 200 sieve) of the soils to calculate resistance or susceptibility of soil to liquefaction. The calculated factor of safety against seismically-induced liquefaction is shown in Appendix B and was calculated to be equal to or greater than 1.2 throughout the depth of the embankments and underlying foundation in the evaluated SPT boreholes (Golder, 2012) for the considered earthquake loading. These screening-level results indicate that the embankments and foundation soils for Primary 2 are not susceptible to seismically-induced liquefaction for the seismic loading considered.

5.0 CONCLUSIONS AND RECOMMENDATIONS

Based on our review of the information provided by NIPSCO, onsite observations, and the results of the structural stability assessment, no structural stability deficiencies were identified in Primary 2 and no corrective actions are necessary. Based on this same information and on our analyses, the calculated factors of safety against slope failure and liquefaction meet or exceed the minimum values listed in §257.73(e)(1)(i)-(iv).

6.0 CLOSING

This report is intended to summarize the results of the structural stability and factor of safety assessment to fulfill the provisions of Title 40 of the Code of Federal Regulations Section 257.73(d) and (e) (40 CFR Part 257.73(d) and (e)) for Primary Settling Pond No. 2 at the Michigan City Generating Station.

Sincerely,

GOLDER ASSOCIATES INC.

Steven A. McManus

Tiffany Johnson

Steven McManus, E.I.T.
Senior Project Geotechnical Engineer

Tiffany D. Johnson, P.E.
Associate

7.0 REFERENCES

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FIGURES

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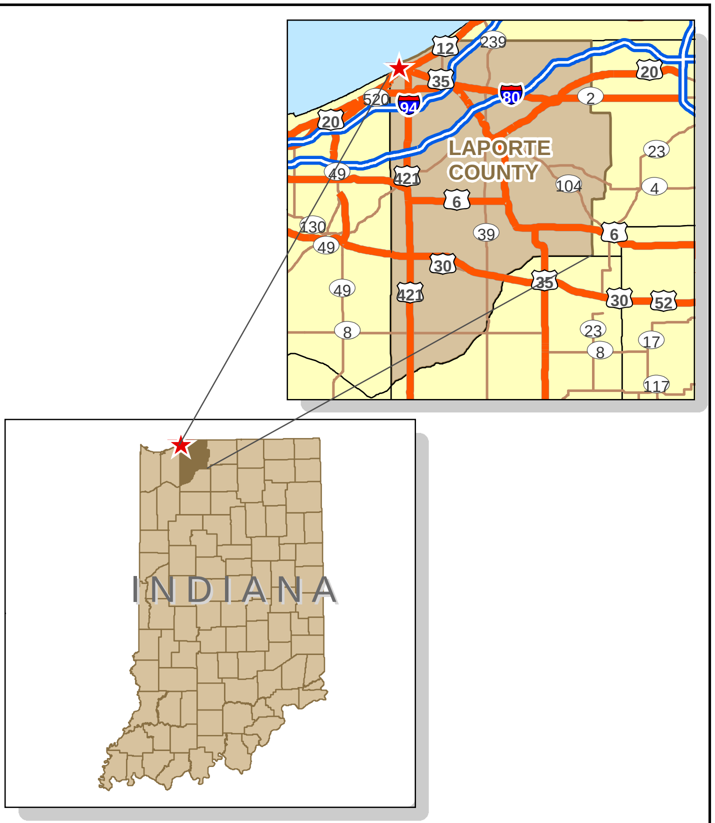

0 1000 2000 Feet

APPROXIMATE SCALE

CLIENT

NIPSCO

PROJECT

MICHIGAN CITY GENERATING STATION
101 WABASH STREET
MICHIGAN CITY, INDIANA

TITLE

SITE LOCATION MAP

CONSULTANT

Golder Associates

YYYY-MM-DD	2017-11-28
PREPARED	DJC
DESIGN	SAM
REVIEW	SAM
APPROVED	TDJ

PROJECT No.	CONTROL	Rev.	FIGURE
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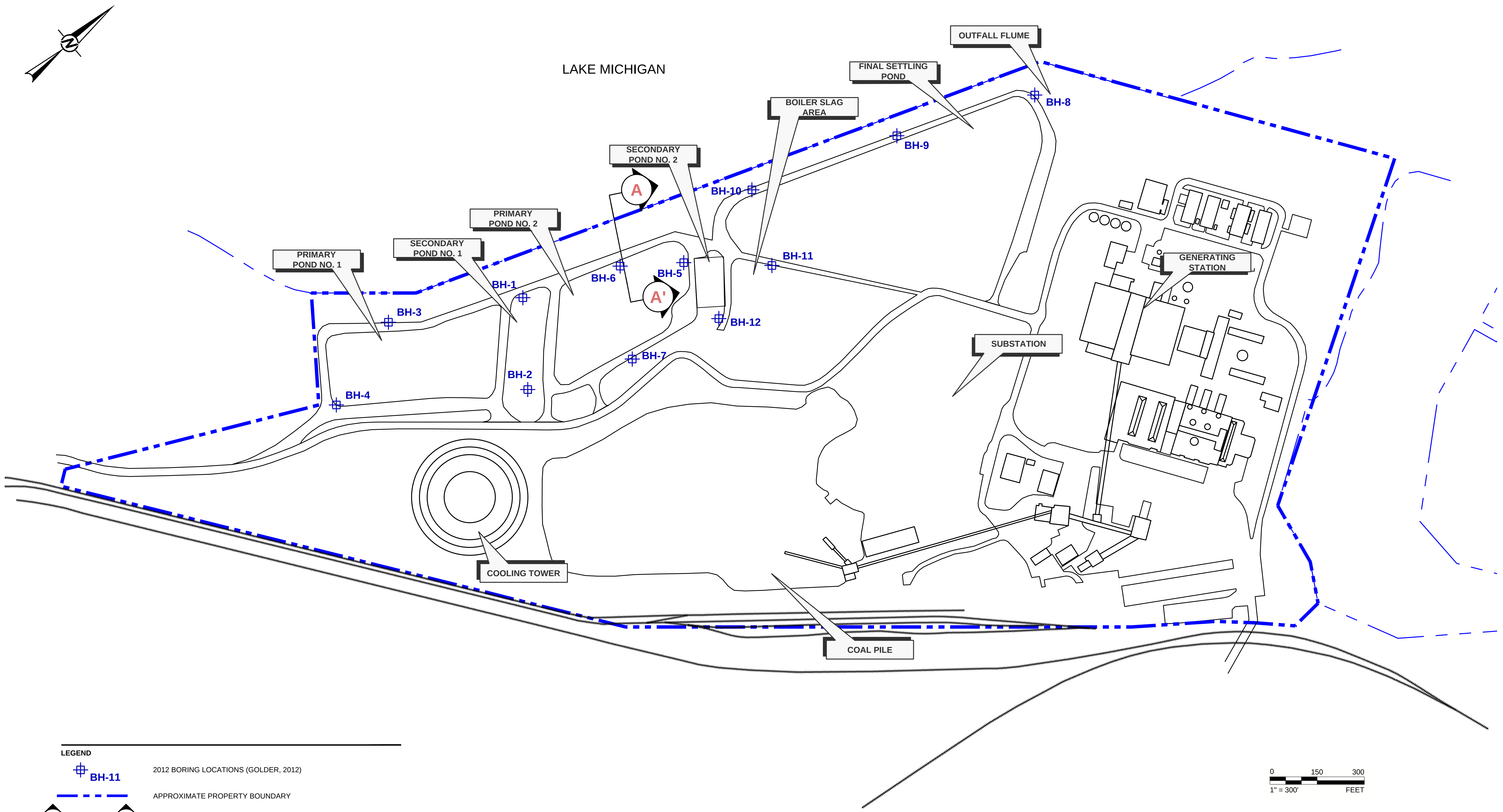

LEGEND

⌖ BH-11	2012 BORING LOCATIONS (GOLDER, 2012)
━ ‑ ‑ ━	APPROXIMATE PROPERTY BOUNDARY
A A'	LOCATION OF SECTION FOR SLOPE STABILITY ANALYSIS

REFERENCE

2012 BOREHOLE LOCATIONS FROM 2012 GEOTECHNICAL INVESTIGATION AND EMBANKMENT STABILITY ANALYSES - NIPSCO MICHIGAN CITY GENERATING STATION (GOLDER, 2012).

CLIENT
NIPSCO

CONSULTANT
Golder Associates

YYYY-MM-DD	2017-11-27
DESIGNED	SAM
PREPARED	DJC
REVIEWED	SAM
APPROVED	TDJ

PROJECT
MICHIGAN CITY GENERATING STATION
101 WABASH STREET
MICHIGAN CITY, INDIANA

TITLE
BOREHOLE AND SLOPE STABILITY SECTION LOCATIONS

PROJECT NO.	CONTROL	REV.	FIGURE
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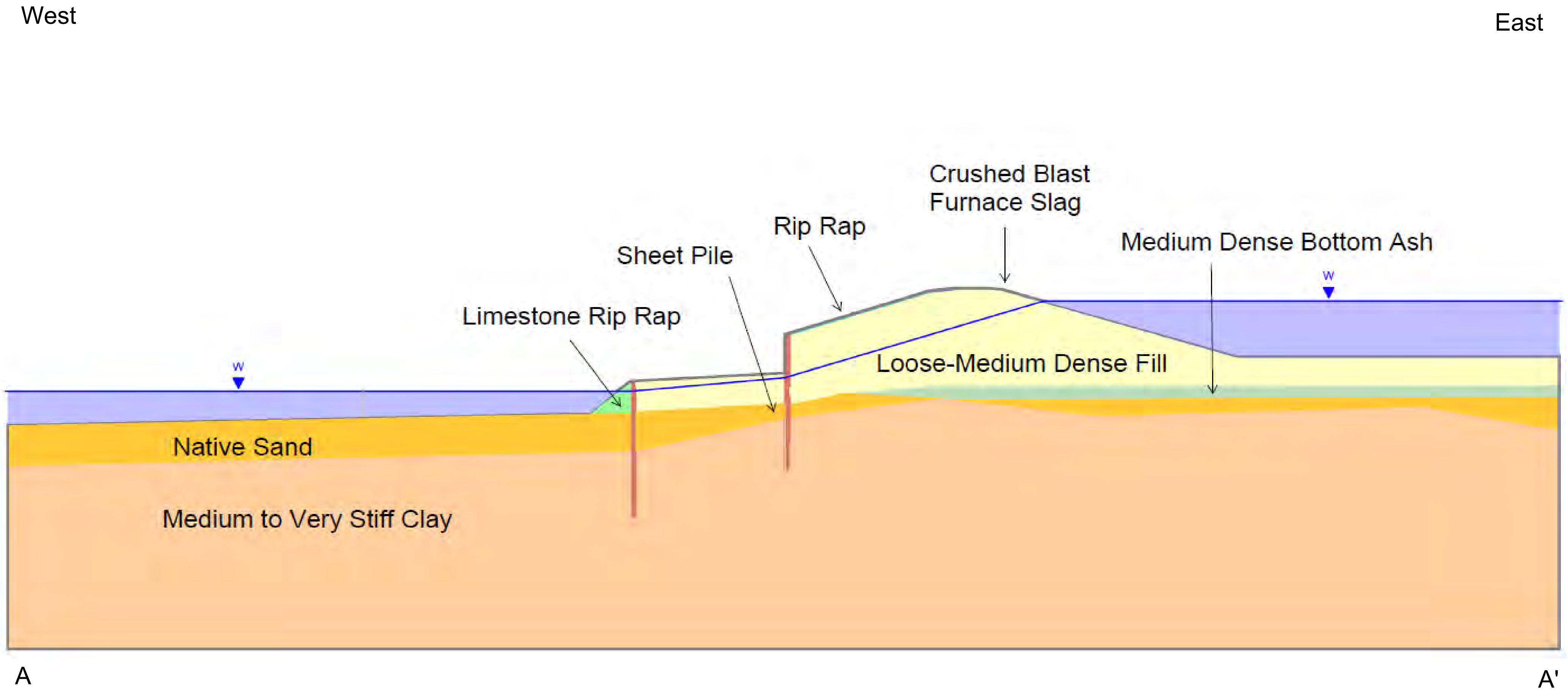

Golder Associates

Golder Associates Inc.

SCALE	AS SHOWN	TITLE		
DATE	NOV-28-2017	Primary Settling Pond No. 2 Slope Stability Section A-A'		
MADE BY	SF			
CAD	-			
FILE	STABILITY	CHECK	AK	Northern Indiana Public Service Company	FIGURE 3
PROJECT No. 1787453.006	REV. 0	REVIEW	TDJ		

Primary Settling Pond No. 2 Slope Stability Section A-A'

Northern Indiana Public Service Company

APPENDIX A
PRIMARY SETTLING POND NO. 2 – SLOPE STABILITY ANALYSIS

CALCULATIONS

Date:	January 12, 2018	**Made by:**	Samantha Fentress
Project No.:	1787453.006	**Checked by:**	Amin Kandi
Site Name:	NIPSCO Michigan City Generating Station	**Reviewed by:**	Tiffany Johnson
Subject	**SLOPE STABILITY ANALYSIS CALCULATION BRIEF FOR PRIMARY SETTLING POND NO. 2 AT MICHIGAN CITY GENERATING STATION**		

1.0 OBJECTIVE

The objective of this document is to provide a slope stability assessment for Primary Settling Pond No. 2 (Primary 2) at the Michigan City Generating Station (MCGS). The Code of Federal Regulations (CFR) specifies structural integrity criteria for existing coal combustion residuals (CCR) surface impoundments under 40 CFR Part 257.73 (e). The rule states: "…*the owner or operator must conduct 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…*"

- *The calculated static factor of safety under the long-term, maximum storage pool loading condition must equal or exceed 1.50,*
- *The calculated static factor of safety under the maximum surcharge pool loading condition must equal or exceed 1.40,*
- *The calculated seismic factor of safety must equal or exceed 1.00, and*
- *For dikes constructed of soils that have susceptibility to liquefaction, the calculated liquefaction factor of safety must equal or exceed 1.20.*

2.0 SUBSURFACE CONDITIONS

Information regarding subsurface conditions in the vicinity of the Primary 2 was summarized in the *2012 Geotechnical Investigation and Embankment Stability Analyses* report (2012 Geotechnical Report; Golder, 2012). Borehole logs and a slope stability section presented therein were used to characterize the subsurface conditions used in this analysis. Figure 1 presents the location of these boreholes presented in the 2012 Geotechnical Report and the slope stability section evaluated in that report and re-evaluated herein.

2.1 Stratigraphy

Boreholes drilled within the embankment of Primary 2 and the surrounding area indicate the subsurface material consists of dense to very dense slag and sand from ground surface to approximately 1 foot below ground surface (ft bgs). Immediately below this upper layer there is a layer of loose to medium dense sand, approximately 35 feet (ft) thick. Borings BH-5, BH-6, BH-7 indicate that below the upper sand layer is a

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layer of dense to very dense black ash 2.5 ft to 5 ft thick. Below this ash layer, is a discontinuous layer of dense to very dense sand as indicated in BH-5, BH-10, BH-11, and BH-12. This layer was not present in boreholes BH-6 or BH-7. BH-7 was terminated at a depth of 40 ft bgs (approximately elevation 570 ft above mean sea level; amsl) and neither the black ash nor any underlying units were encountered. The borehole log for BH-6 indicates a medium stiff clay layer is present below the black ash to the end of the boring at 50 ft bgs (approximately elevation 560 ft above mean sea level; amsl). BH-10 indicates that the clay layer extends to approximately elevation 535 ft amsl (75 feet below the embankment crest).

During drilling activities associated with the 2012 Geotechnical Report, groundwater was observed at 8.5 ft bgs in BH-5 (north embankment). Groundwater was observed at 18 ft bgs in BH-6 (west embankment) and 11.5 ft bgs in BH-7 (east embankment).

2.2 Material Properties

The stability analysis of the Primary 2 was performed using material properties presented in the 2012 Geotechnical Report. These properties were developed based on the geotechnical investigation and associated laboratory testing presented therein, as well as empirical correlations to published data sources. Table 1 summarizes these material properties.

Table 1: Material Properties used in Geotechnical Analyses

Material	Internal Friction Angle, ϕ (deg)	Peak Cohesion, c (psf)	Dry Unit Weight, γ_d (pcf)	Saturated Unit Weight, γ_{sat} (pcf)	Undrained Shear Strength, s_u (psf)	Hydraulic Conductivity, k (cm/s)
Loose to Medium Dense Fill (SP)	33	0	100	110	NA	$1x10^{-3}$
Bottom Ash Fill-Medium Dense	35	0	100	110	NA	$1X10^{-3}$
Clay-Medium to V. Stiff	30	70	116	136	750 – 2500	$1x10^{-6}$
Native Sand-Dense	40	0	110	120	NA	$1x10^{-3}$
Crushed Blast Furnace Slag	40	0	120	130	NA	1
8-inch Riprap	45	0	140	145	NA	100
Limestone Riprap	45	0	140	145	NA	100
Steel Sheeting	NA	NA	120	NA	NA	NA

Notes: deg = degrees, psf = pounds per square foot, pcf = pounds per cubic foot, ft = feet, and cm/s = centimeters per second

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3.0 SLOPE STABILITY ANALYSIS METHODS

A two-dimensional slope stability analysis was performed for the selected critical section of the Primary 2. The geometry and stratigraphy of the slope stability section was based on the slope stability section presented in the 2012 Geotechnical Report and revised to evaluate alternate water pool elevations within the Primary 2. Slope stability analyses employed limit equilibrium methods, with factors of safety against shear failure calculated using the Slide Version 7.0 software, developed by Rocscience. Acceptable factors of safety against shear failure are specified in 40 CFR 257.73(e) and summarized in Section 1.0 of this document. Factors of safety for the various scenarios were calculated for circular and non-circular failure surfaces. Four scenarios were evaluated as part of this analysis:

- Maximum pool storage (long-term),
- Maximum pool surcharge from a storm event,
- Stability of upstream face during rapid drawdown, and.
- Seismic (pseudo-static) stability.

Two slice equilibrium methods were used to calculate the factors of safety for the impoundment: Spencer Method and Janbu (Corrected) Method. The slope stability section was analyzed for circular and planar (block) failure surfaces.

3.1 Maximum Pool Storage

The minimum acceptable factor of safety for the maximum pool storage scenario is 1.5, per 40 CFR 257.73(e). For this scenario, the water elevation within Primary 2 was assumed to be 605.7 ft above mean sea level (amsl), as presented in the Inflow Design Flood Control System Plan for Primary 2 (Golder, 2017). Additionally, a distributed load of 375 pounds-per-square-foot (psf) was applied to the crest of Primary 2 to simulate traffic loading. The upstream and downstream slopes, crest, and toe were analyzed in this scenario. Figures 3, 4, 5 and 6 present the results of the analysis for this scenario.

3.2 Maximum Pool Surcharge

The minimum acceptable factor of safety for the maximum pool surcharge scenario is 1.4, per 40 CFR 257.73(e). For this scenario, the water elevation was assumed to be 606.8 ft (amsl) to reflect the maximum water surface elevation caused by a 24-hour duration, 1,000-year recurrence interval storm event (Golder, 2017). Additionally, a distributed load of 375 psf was applied to the crest of Primary 2 to simulate traffic loading. The upstream and downstream slopes, crest, and toe were analyzed in this scenario. Figures 7, 8, 9, and 10 present the results of the analysis for this scenario.

3.3 Rapid Drawdown

Rapid drawdown occurs when the water level within an impoundment is lowered faster than soils within the embankment can drain. In this scenario, the buttressing effects of the water are no longer present, but the phreatic surface within the embankment remains elevated. Additionally, low-permeability soils below the phreatic surface remain in the undrained (total stress) condition.40 CFR 257 does not specify a minimum acceptable factor of safety for the rapid drawdown scenario. However, factors of safety as low as 1.0 to 1.2 are generally considered acceptable for rapid drawdown (Duncan and Wright, 2005 after USACE, 2003). A minimum acceptable factor of safety of 1.2 was selected for this analysis.

The phreatic surface within the embankment was modeled with the assumption of an initial operating pool water surface level equal to 605.7 ft (amsl), followed by complete drawdown of the impounded water. Additionally, a distributed load of 375 pounds-per-square-foot (psf) was applied to the crest of Primary 2to simulate traffic loading. The upstream slope was analyzed in for this scenario. Figures 11, 12, 13, and 14 present the results of the analysis for this scenario.

3.4 Seismic (Pseudo-Static)

Slope stability under seismic conditions was analyzed using the pseudo-static approach, where a sustained horizontal force is applied to simulate inertial forces due to earthquake motions. Pseudo-static seismic analysis procedures are applicable for slopes made up of materials that do not liquefy or lose significant shear strength due to seismic shaking (Seed, 1979). In the procedure, the inertial force due to shaking is represented as a fraction of the vertical weight force by a horizontal seismic coefficient.

Typically, for pseudo-static stability analyses, the horizontal pseudo-static seismic coefficient is assumed to be 1/2 to 2/3 the value of the peak ground acceleration (PGA). This is because the PGA during a seismic event only occurs for a fraction of a second, with sustained ground accelerations values being significantly lower than the peak. Per guidance provided by the Department of Energy (USDOE, 1989), the seismic coefficient selected for the pseudo-static stability analyses for the site is 2/3 the value of the PGA.

The PGA at the site corresponding to a 2% probability of exceedance in 50 years (average return period of 2,475 years) is 0.09g (see Appendix B). A corresponding pseudo-static seismic coefficient of 0.06g (0.09g * 2/3 = 0.06g) was used in this seismic stability analysis. The maximum pool storage elevation 605.7 ft (amsl) was used. Figures 15, 16, 17, and 18 present the results of the analysis for this scenario.

4.0 RESULTS AND DISCUSSION

Results from analysis of each of the four scenarios are presented in Table 2 below.

Table 2: Slope Stability Analysis Results Summary

Scenario	**Minimum Calculated Factor of Safety**		**Minimum Acceptable Factor of Safety**
	Circular Failure Surface	**Planar (Block) Failure Surface**	
Static Maximum **Storage** Pool - 257.73(e)(1)(i)	2.0	1.9	1.5
Static Maximum **Surcharge** Pool - 257.73(e)(1)(ii)	1.9	1.6	1.4
Static Maximum **Storage** Pool – Rapid Drawdown 257.73(d)(1)(ii)	1.3	1.3	1.2
Seismic (Pseudo-Static) Maximum **Storage** Pool - 257.73(e)(1)(iii)	1.6	1.5	1.0

The calculated factors of safety for each of the scenarios analyzed herein meet the required factors of safety.

5.0 CONCLUSION

The slope stability assessment for Primary 2 was performed using geotechnical modeling software in conjunction with historical construction drawings and site data. Four scenarios were analyzed using two vertical slice limit equilibrium methods. The results calculated in this assessment demonstrate acceptable factors of safety and meet the slope stability requirements of 40 CFR 257.73.

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Date:	January 12,2018	Reviewed by:	Tiffany Johnson

6.0 REFERENCES

Duncan, J.M., Wright, S.G. Soil Strength and Slope Stability. John Wiley & Sons, Inc. Hoboken, New Jersey. 2005. Print

Golder Associates Inc. (Golder), 2012. Geotechnical Investigation and Embankment Stability Analyses. Prepared for Michigan City Generating Station. Station Unit 12; No. Indiana Public Service co., Wheatfield, Indiana.

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Rocscience Inc. Slide 2D Limit Equilibrium Slope Stability for Soil and Rock Slopes. User's Guide. 1989

Sargent and Lundy Engineers of Chicago (Sargent and Lundy), 1972. Construction Drawings for Michigan City Gen. Station Unit 12; No. Indian Public Service Co., Wheatfield Indiana. January

Seed, H.B., 1979. *Considerations in the Earthquake-Resistant Design of Earth and Rockfill Dams*. Geotechnique, Vol. 29, No. 3, pp 215 - 263.

United States Army Corps of Engineers (USACE), 2003. *EM-1110-2-1902: Engineering and Design – Slope Stability Engineer Manual*. October 31, 2003.

U.S. Department of Energy (USDOE), 1989. *Technical Approach Document, Revision II, UMTRA-DOE/AL 050425.0002*, Uranium Mill Tailings Remedial Action Project, Albuquerque, New Mexico.

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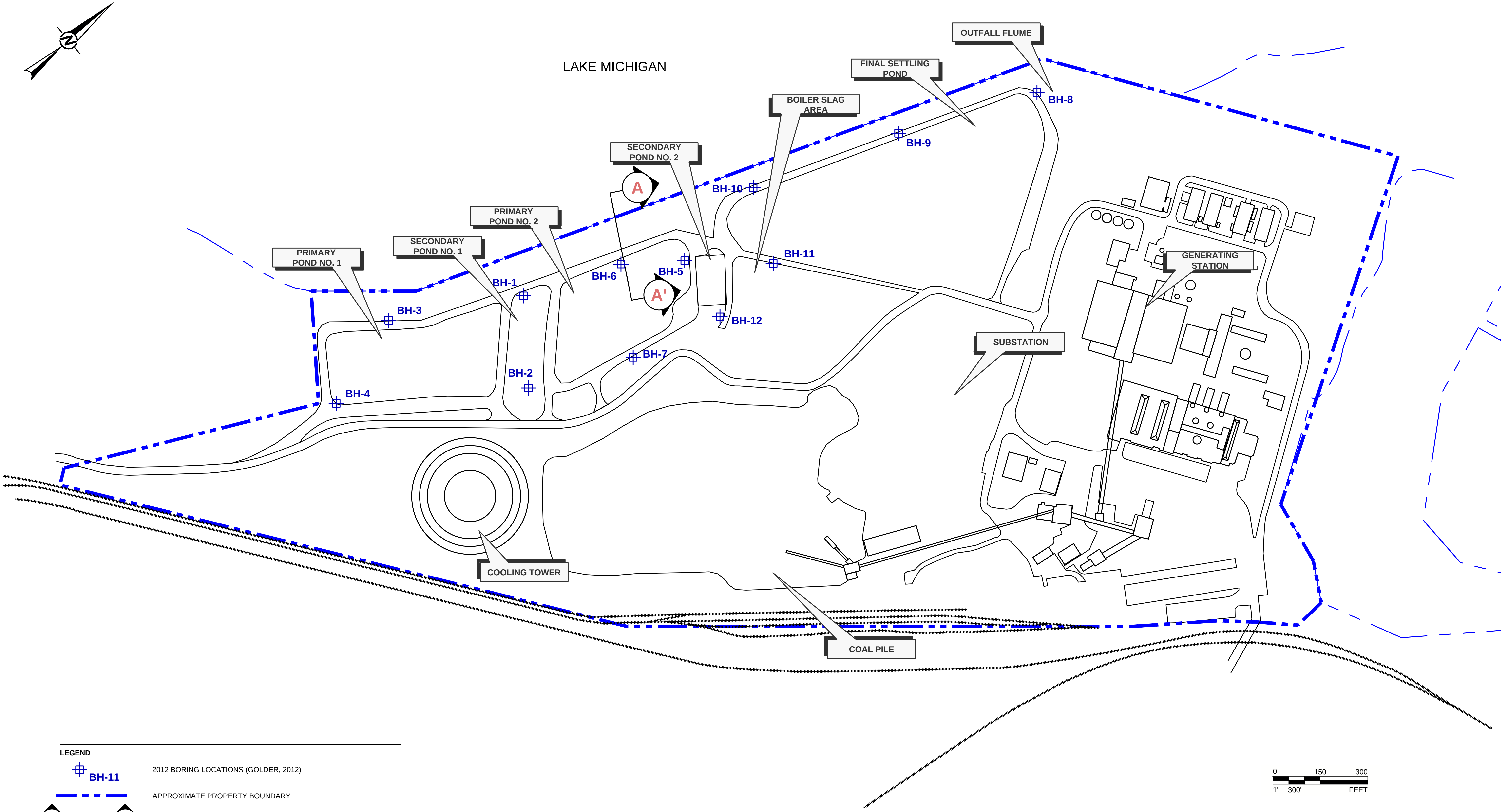
7.0 FIGURES



LEGEND

⌖ BH-11	2012 BORING LOCATIONS (GOLDER, 2012)
— · — ·	APPROXIMATE PROPERTY BOUNDARY
A A'	LOCATION OF SECTION FOR SLOPE STABILITY ANALYSIS

REFERENCE

2012 BOREHOLE LOCATIONS FROM 2012 GEOTECHNICAL INVESTIGATION AND EMBANKMENT STABILITY ANALYSES - NIPSCO MICHIGAN CITY GENERATING STATION (GOLDER, 2012).

CLIENT
NIPSCO

CONSULTANT
Golder Associates

YYYY-MM-DD	2017-11-27
DESIGNED	SAM
PREPARED	DJC
REVIEWED	SAM
APPROVED	TDJ

PROJECT
MICHIGAN CITY GENERATING STATION
101 WABASH STREET
MICHIGAN CITY, INDIANA

TITLE
BOREHOLE AND SLOPE STABILITY SECTION LOCATIONS

PROJECT NO.	CONTROL	REV.	FIGURE
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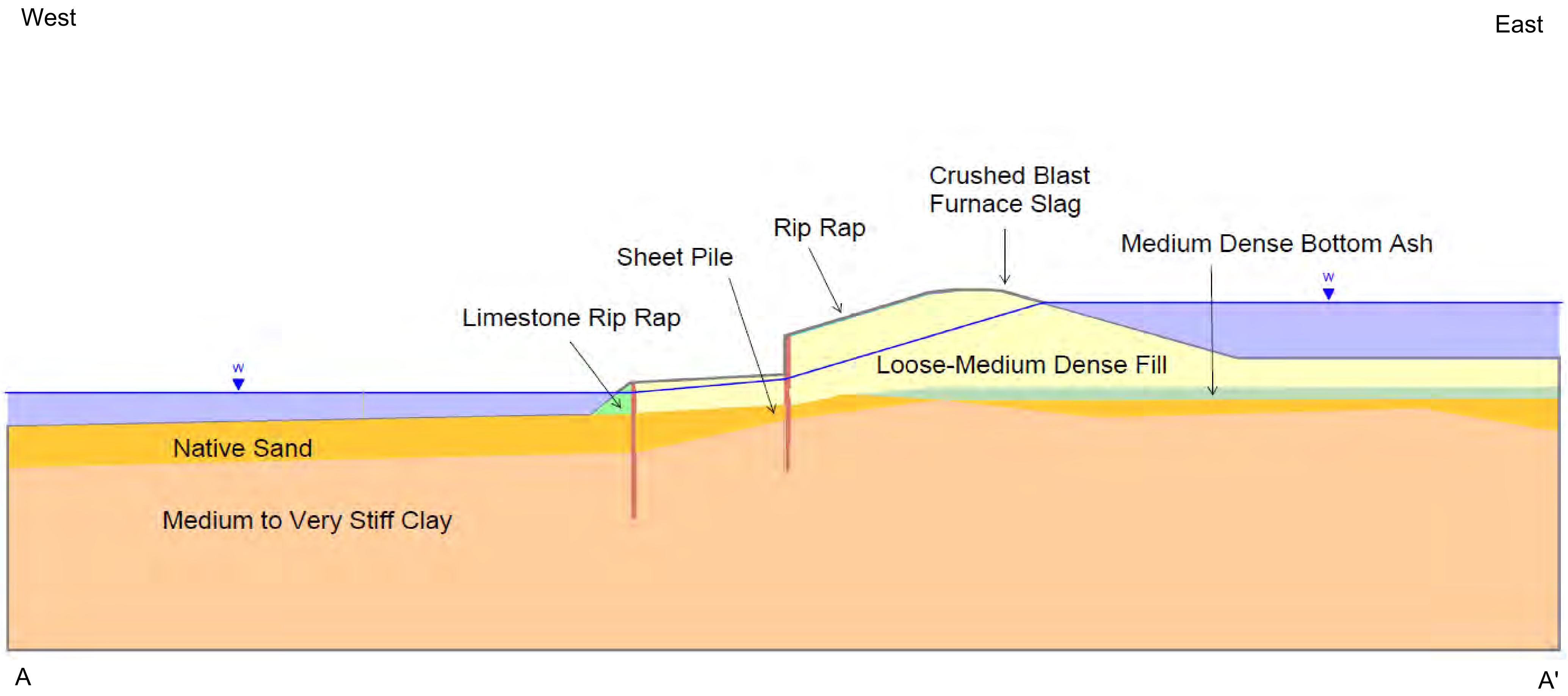

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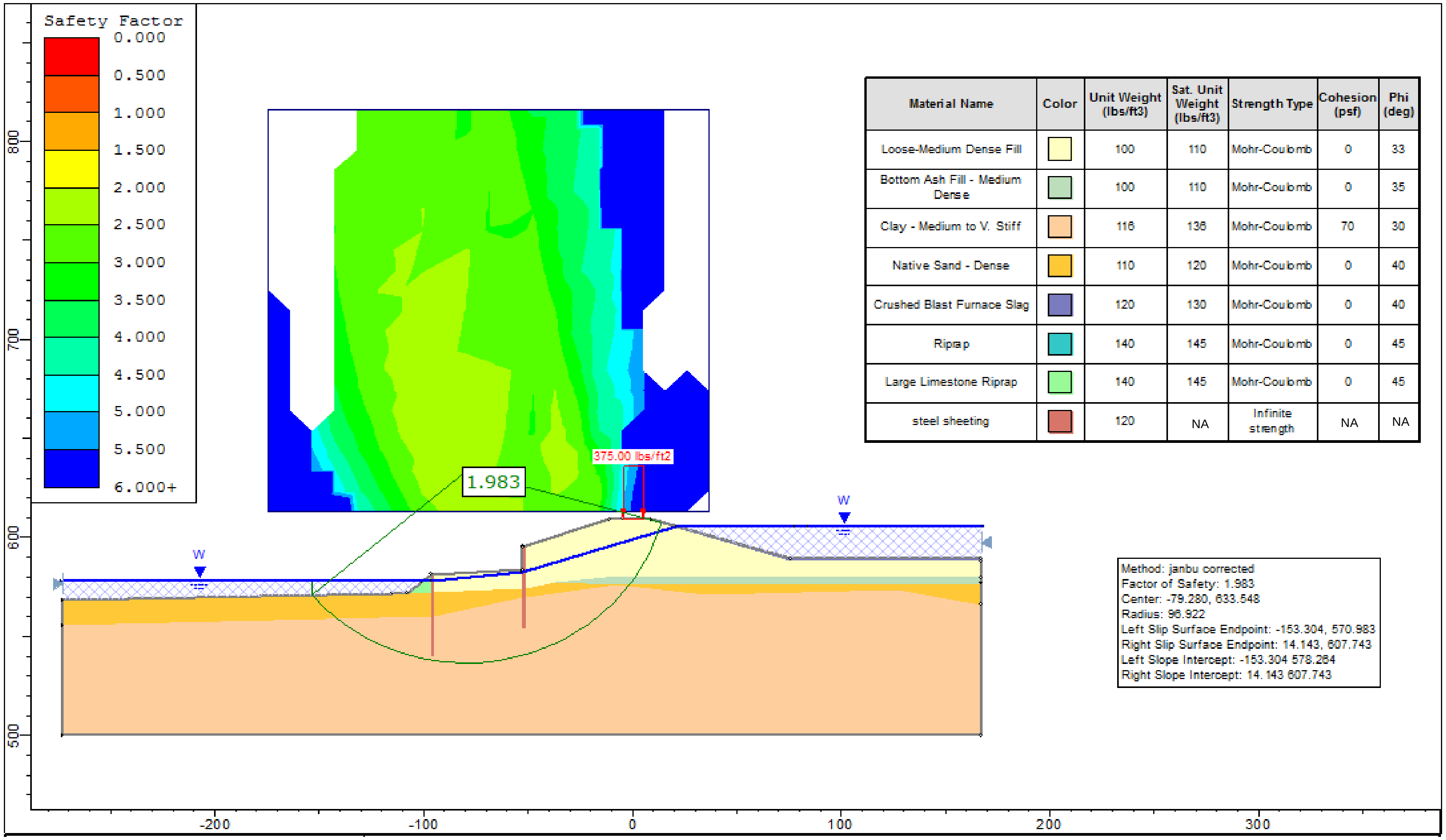
FILE STABILITY PROJECT No. 1787453.006 REV. 0	CHECK AK REVIEW TDJ	**Northern Indiana Public Service Company**	FIGURE **2**



Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Loose-Medium Dense Fill	🟨	100	110	Mohr-Coulomb	0	33
Bottom Ash Fill - Medium Dense	🟩	100	110	Mohr-Coulomb	0	35
Clay - Medium to V. Stiff	🟧	116	136	Mohr-Coulomb	70	30
Native Sand - Dense	🟨	110	120	Mohr-Coulomb	0	40
Crushed Blast Furnace Slag	🟪	120	130	Mohr-Coulomb	0	40
Riprap	🟦	140	145	Mohr-Coulomb	0	45
Large Limestone Riprap	🟩	140	145	Mohr-Coulomb	0	45
steel sheeting	🟥	120	NA	Infinite strength	NA	NA

Method: janbu corrected
Factor of Safety: 1.983
Center: -79.280, 633.548
Radius: 96.922
Left Slip Surface Endpoint: -153.304, 570.983
Right Slip Surface Endpoint: 14.143, 607.743
Left Slope Intercept: -153.304 578.264
Right Slope Intercept: 14. 143 607.743

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Primary Settling Pond No. 2 Maximum Pool Storage Circular Slip Surface

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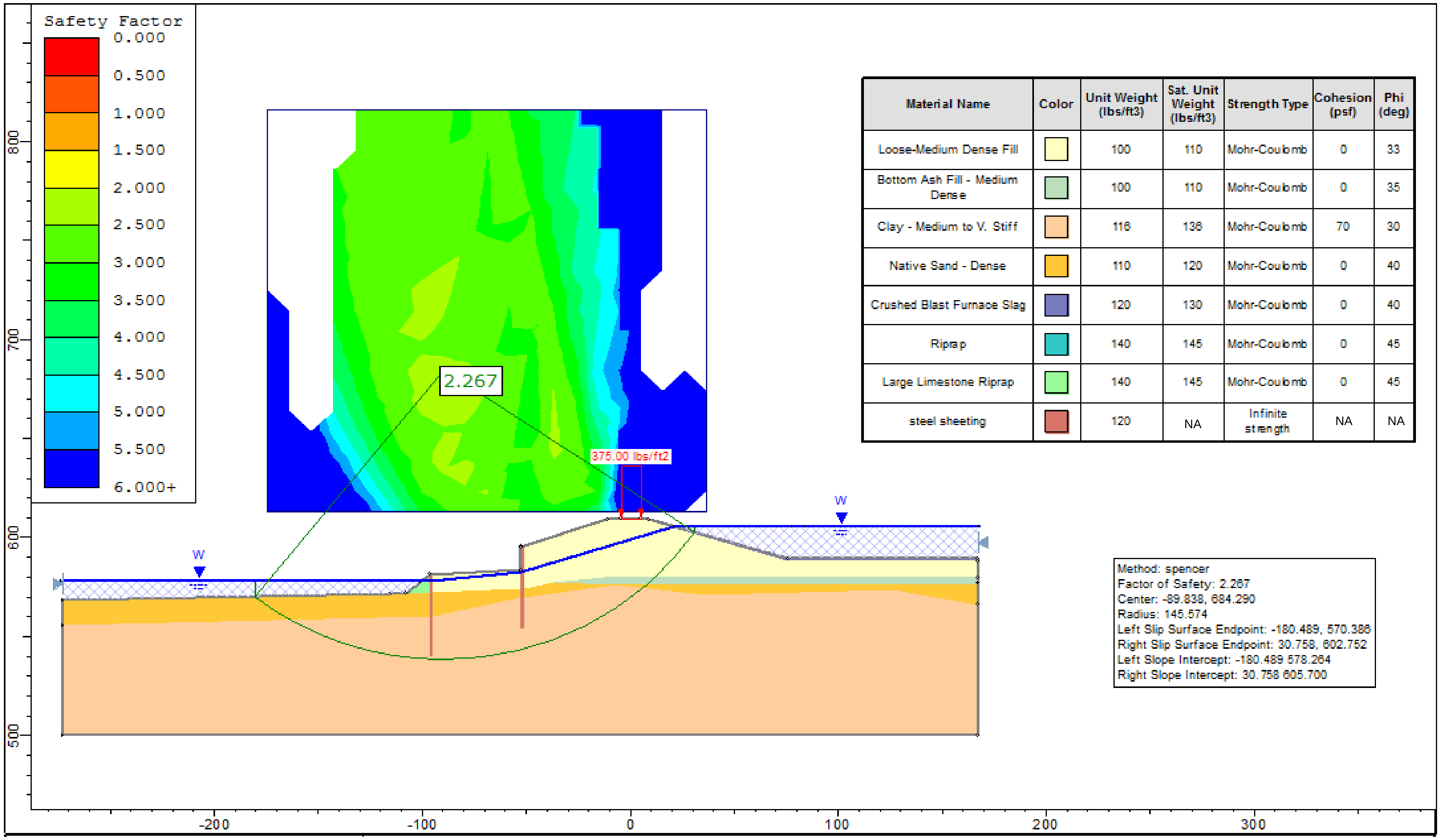
FIGURE 3



Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Loose-Medium Dense Fill	🟨	100	110	Mohr-Coulomb	0	33
Bottom Ash Fill - Medium Dense	🟩	100	110	Mohr-Coulomb	0	35
Clay - Medium to V. Stiff	🟧	116	136	Mohr-Coulomb	70	30
Native Sand - Dense	🟧	110	120	Mohr-Coulomb	0	40
Crushed Blast Furnace Slag	🟪	120	130	Mohr-Coulomb	0	40
Riprap	🟦	140	145	Mohr-Coulomb	0	45
Large Limestone Riprap	🟩	140	145	Mohr-Coulomb	0	45
steel sheeting	🟥	120	NA	Infinite strength	NA	NA

Method: spencer
Factor of Safety: 2.267
Center: -89.838, 684.290
Radius: 145.574
Left Slip Surface Endpoint: -180.489, 570.386
Right Slip Surface Endpoint: 30.758, 602.752
Left Slope Intercept: -180.489 578.264
Right Slope Intercept: 30.758 605.700

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Primary Settling Pond No. 2 Maximum Pool Storage
Circular Slip Surface

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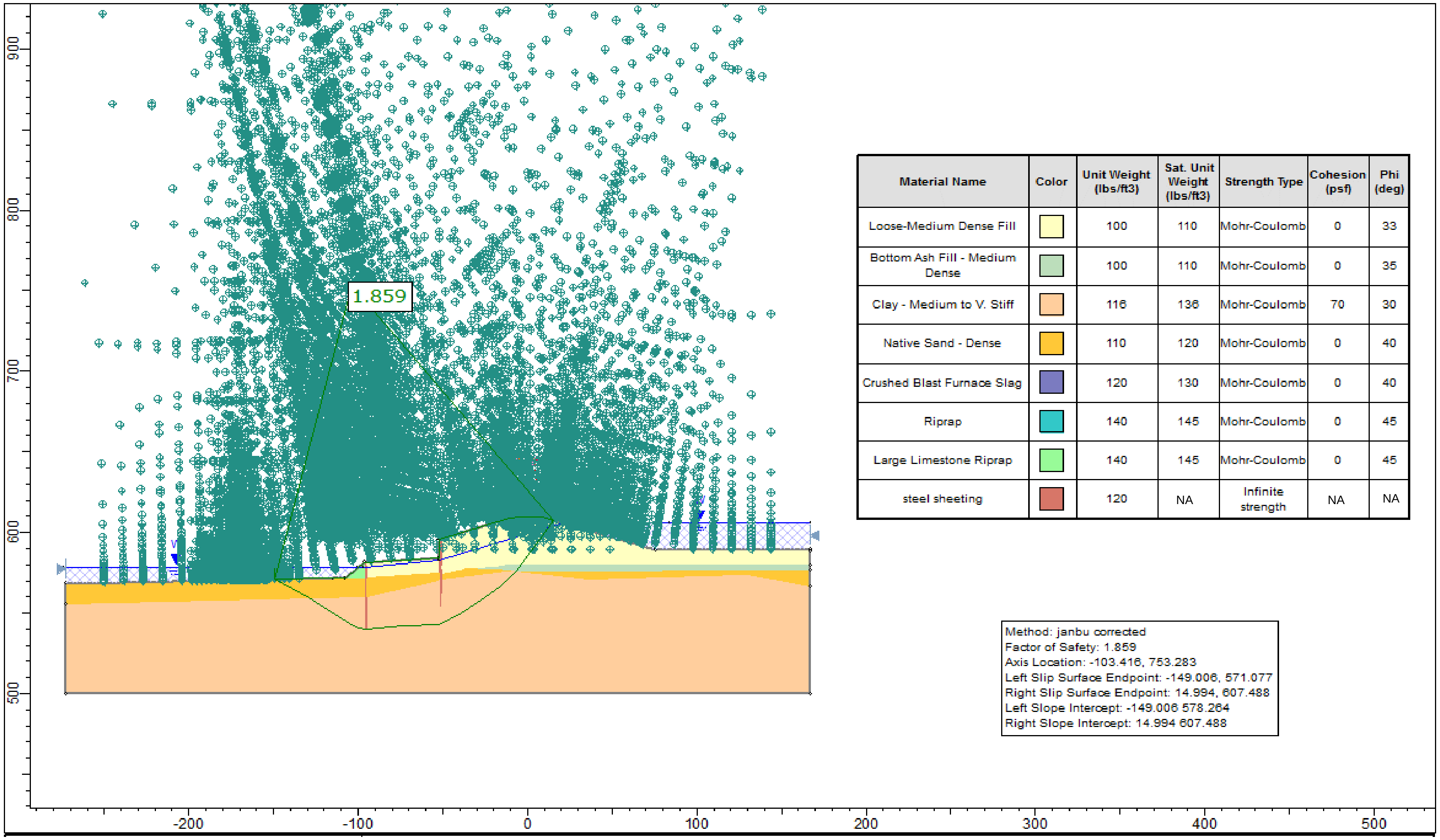
FIGURE 4



Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Loose-Medium Dense Fill	🟨	100	110	Mohr-Coulomb	0	33
Bottom Ash Fill - Medium Dense	🟩	100	110	Mohr-Coulomb	0	35
Clay - Medium to V. Stiff	🟧	116	136	Mohr-Coulomb	70	30
Native Sand - Dense	🟨	110	120	Mohr-Coulomb	0	40
Crushed Blast Furnace Slag	🟪	120	130	Mohr-Coulomb	0	40
Riprap	🟦	140	145	Mohr-Coulomb	0	45
Large Limestone Riprap	🟩	140	145	Mohr-Coulomb	0	45
steel sheeting	🟥	120	NA	Infinite strength	NA	NA

Method: janbu corrected
Factor of Safety: 1.859
Axis Location: -103.416, 753.283
Left Slip Surface Endpoint: -149.006, 571.077
Right Slip Surface Endpoint: 14.994, 607.488
Left Slope Intercept: -149.006 578.264
Right Slope Intercept: 14.994 607.488

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TITLE

Primary Settling Pond No. 2 Maximum Pool Storage
Non-Circular Slip Surface

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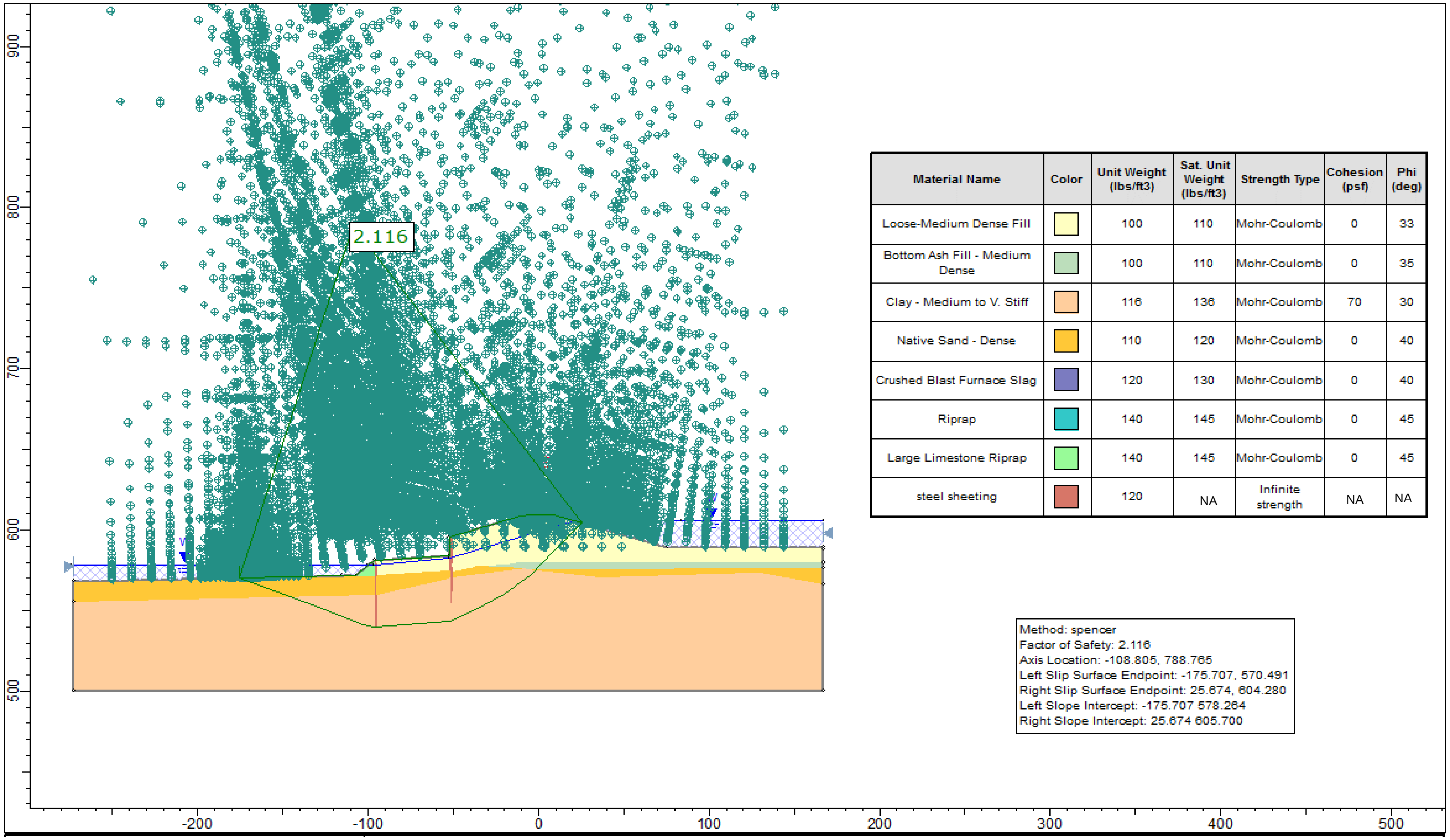
FIGURE 5



Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Loose-Medium Dense Fill	🟨	100	110	Mohr-Coulomb	0	33
Bottom Ash Fill - Medium Dense	🟩	100	110	Mohr-Coulomb	0	35
Clay - Medium to V. Stiff	🟧	116	136	Mohr-Coulomb	70	30
Native Sand - Dense	🟨	110	120	Mohr-Coulomb	0	40
Crushed Blast Furnace Slag	🟪	120	130	Mohr-Coulomb	0	40
Riprap	🟦	140	145	Mohr-Coulomb	0	45
Large Limestone Riprap	🟩	140	145	Mohr-Coulomb	0	45
steel sheeting	🟥	120	NA	Infinite strength	NA	NA

Method: spencer
Factor of Safety: 2.116
Axis Location: -108.805, 788.765
Left Slip Surface Endpoint: -175.707, 570.491
Right Slip Surface Endpoint: 25.674, 604.280
Left Slope Intercept: -175.707 578.264
Right Slope Intercept: 25.674 605.700

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Primary Settling Pond No. 2 Maximum Pool Storage
Non-Circular Slip Surface

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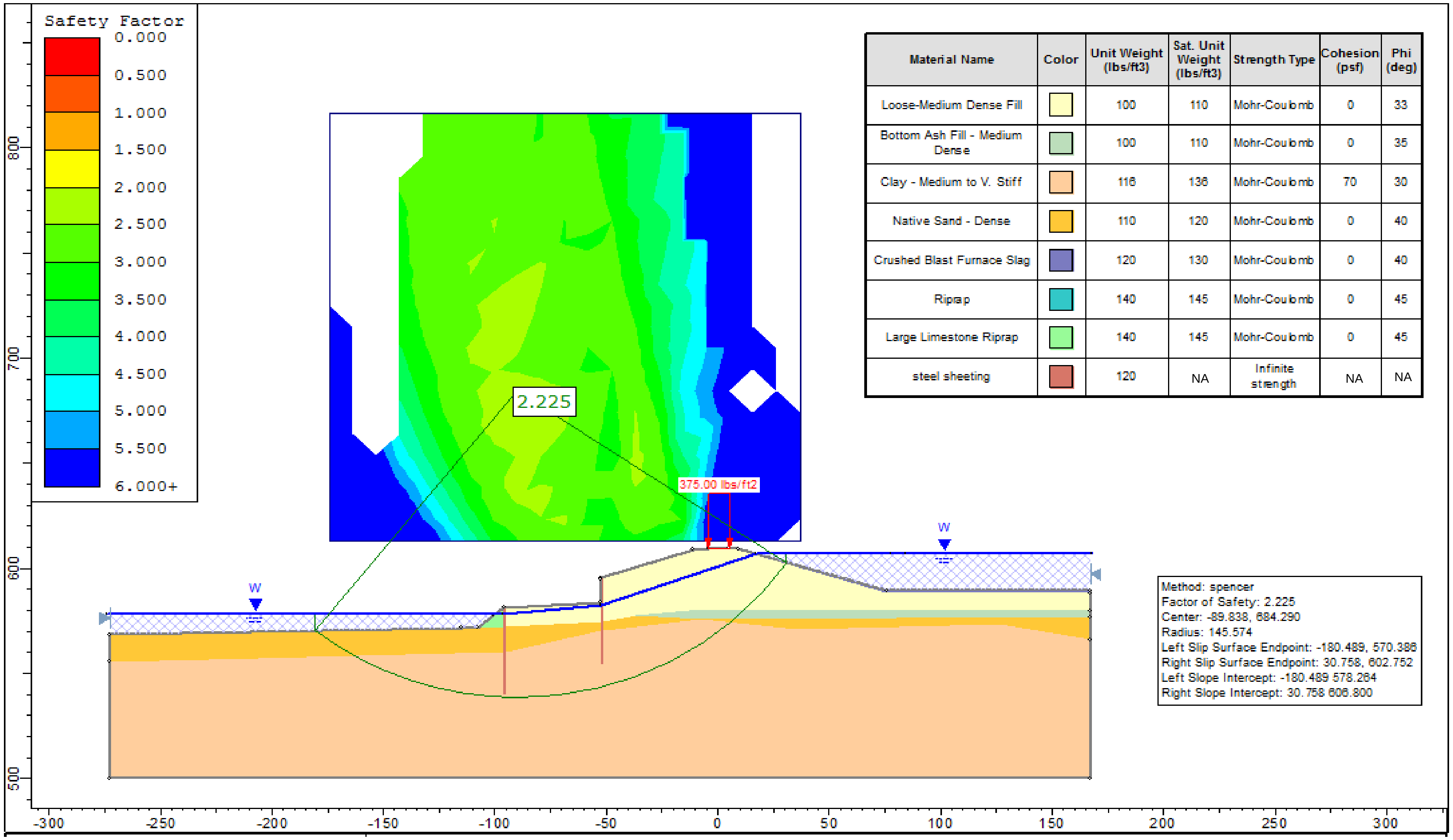
FIGURE 6



Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Loose-Medium Dense Fill	🟨	100	110	Mohr-Coulomb	0	33
Bottom Ash Fill - Medium Dense	🟩	100	110	Mohr-Coulomb	0	35
Clay - Medium to V. Stiff	🟧	116	136	Mohr-Coulomb	70	30
Native Sand - Dense	🟨	110	120	Mohr-Coulomb	0	40
Crushed Blast Furnace Slag	🟪	120	130	Mohr-Coulomb	0	40
Riprap	🟦	140	145	Mohr-Coulomb	0	45
Large Limestone Riprap	🟩	140	145	Mohr-Coulomb	0	45
steel sheeting	🟥	120	NA	Infinite strength	NA	NA

Method: spencer
Factor of Safety: 2.225
Center: -89.838, 684.290
Radius: 145.574
Left Slip Surface Endpoint: -180.489, 570.386
Right Slip Surface Endpoint: 30.758, 602.752
Left Slope Intercept: -180.489 578.264
Right Slope Intercept: 30.758 606.800

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Primary Settling Pond No. 2 Maximum Pool Surcharge
Circular Slip Surface

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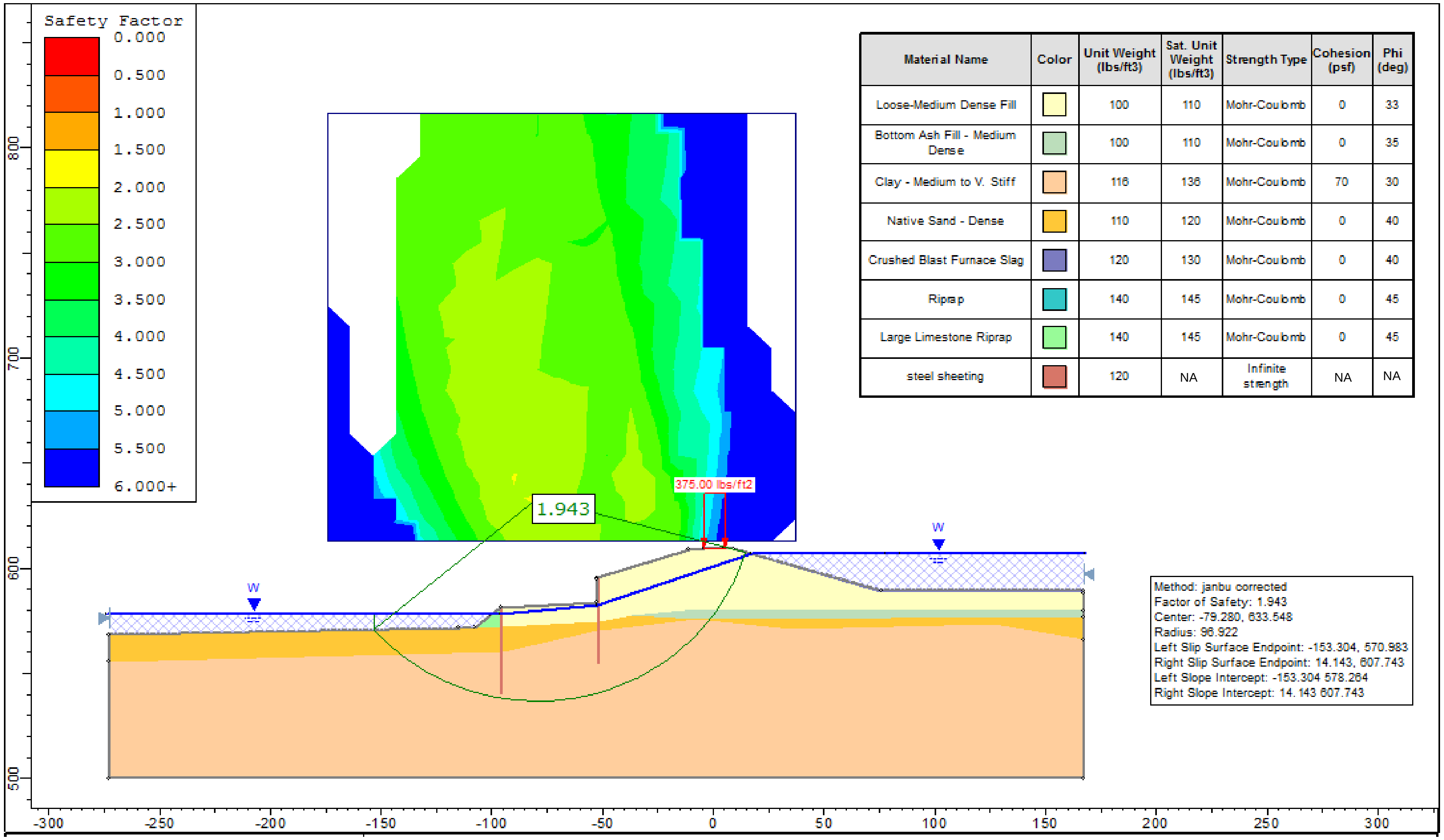
FIGURE 7



Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
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Native Sand - Dense	🟨	110	120	Mohr-Coulomb	0	40
Crushed Blast Furnace Slag	🟪	120	130	Mohr-Coulomb	0	40
Riprap	🟦	140	145	Mohr-Coulomb	0	45
Large Limestone Riprap	🟩	140	145	Mohr-Coulomb	0	45
steel sheeting	🟥	120	NA	Infinite strength	NA	NA

Method: janbu corrected
Factor of Safety: 1.943
Center: -79.280, 633.548
Radius: 96.922
Left Slip Surface Endpoint: -153.304, 570.983
Right Slip Surface Endpoint: 14.143, 607.743
Left Slope Intercept: -153.304 578.264
Right Slope Intercept: 14. 143 607.743

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Circular Slip Surface

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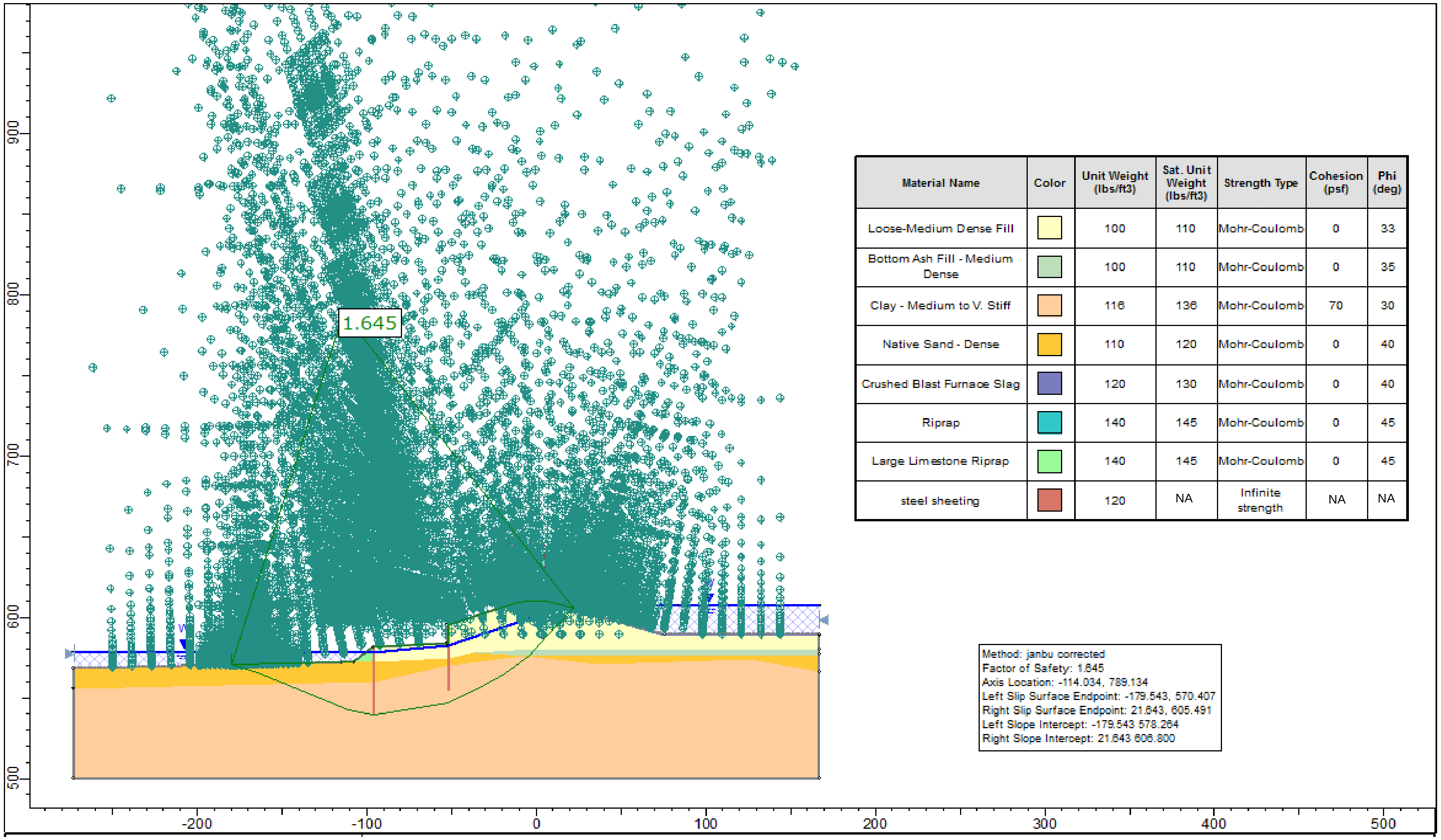
FIGURE 8



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Loose-Medium Dense Fill	🟨	100	110	Mohr-Coulomb	0	33
Bottom Ash Fill - Medium Dense	🟩	100	110	Mohr-Coulomb	0	35
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Native Sand - Dense	🟨	110	120	Mohr-Coulomb	0	40
Crushed Blast Furnace Slag	🟪	120	130	Mohr-Coulomb	0	40
Riprap	🟦	140	145	Mohr-Coulomb	0	45
Large Limestone Riprap	🟩	140	145	Mohr-Coulomb	0	45
steel sheeting	🟥	120	NA	Infinite strength	NA	NA

Method: janbu corrected
Factor of Safety: 1.645
Axis Location: -114.034, 789.134
Left Slip Surface Endpoint: -179.543, 570.407
Right Slip Surface Endpoint: 21.643, 605.491
Left Slope Intercept: -179.543 578.264
Right Slope Intercept: 21.643 606.800

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Non-Circular Slip Surface

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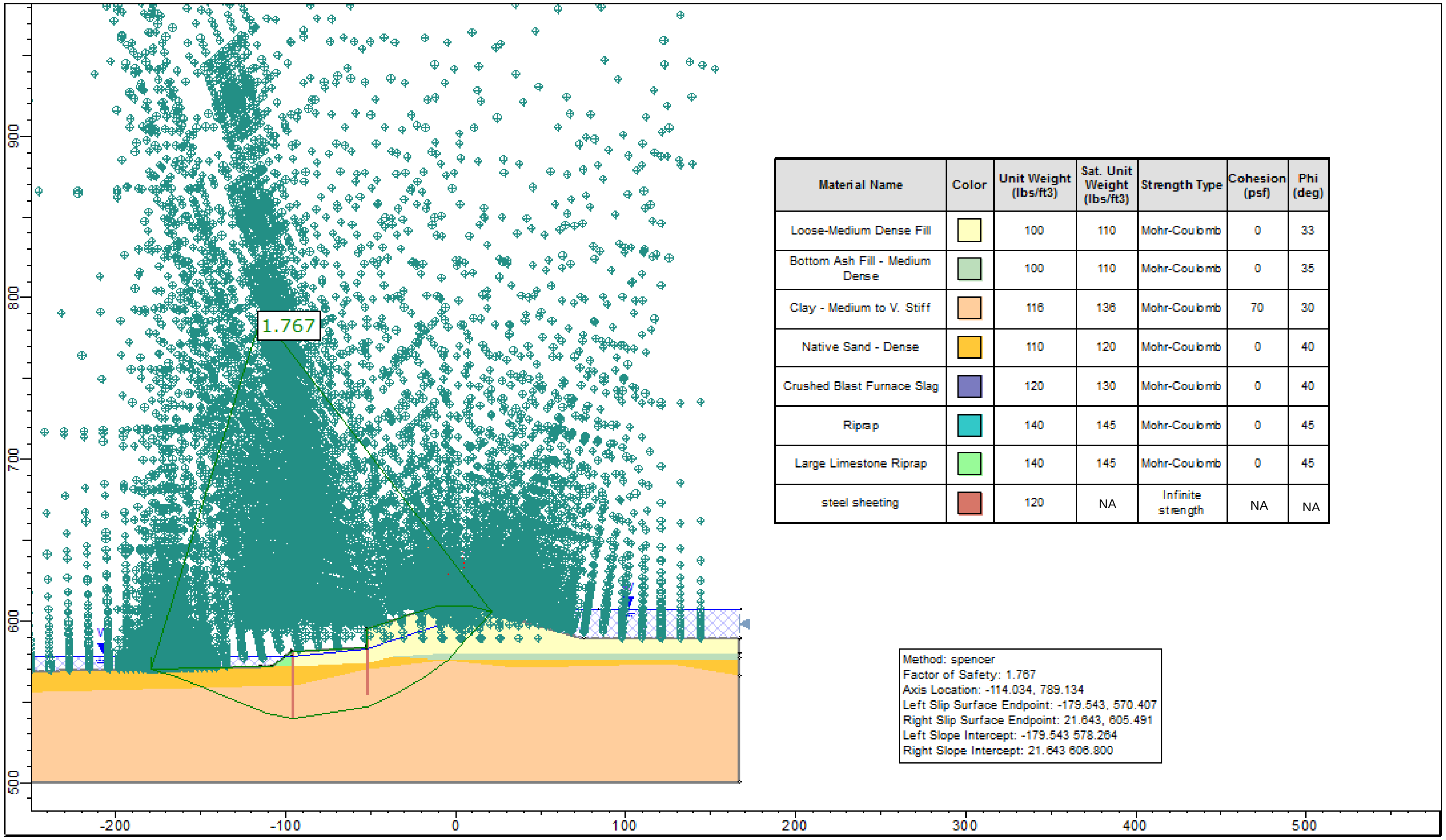
FIGURE 9



Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Loose-Medium Dense Fill	🟨	100	110	Mohr-Coulomb	0	33
Bottom Ash Fill - Medium Dense	🟩	100	110	Mohr-Coulomb	0	35
Clay - Medium to V. Stiff	🟧	116	136	Mohr-Coulomb	70	30
Native Sand - Dense	🟨	110	120	Mohr-Coulomb	0	40
Crushed Blast Furnace Slag	🟪	120	130	Mohr-Coulomb	0	40
Riprap	🟦	140	145	Mohr-Coulomb	0	45
Large Limestone Riprap	🟩	140	145	Mohr-Coulomb	0	45
steel sheeting	🟥	120	NA	Infinite strength	NA	NA

Method: spencer
Factor of Safety: 1.767
Axis Location: -114.034, 789.134
Left Slip Surface Endpoint: -179.543, 570.407
Right Slip Surface Endpoint: 21.643, 605.491
Left Slope Intercept: -179.543 578.264
Right Slope Intercept: 21.643 606.800

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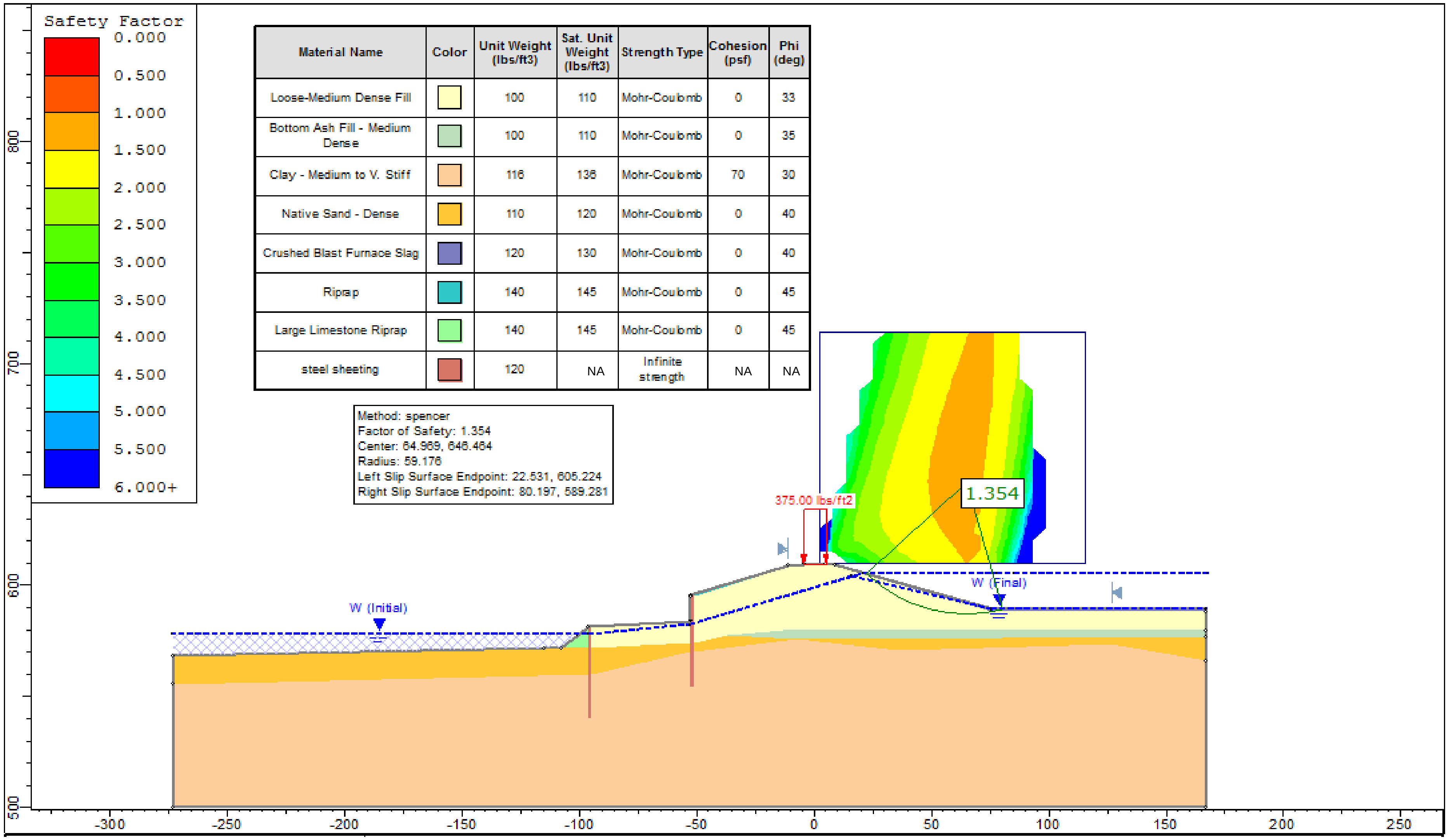
TITLE

Primary Settling Pond No. 2 Maximum Pool Surcharge Non-Circular Slip Surface

Northern Indiana Public Service Company

FIGURE 10



Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Loose-Medium Dense Fill	🟨	100	110	Mohr-Coulomb	0	33
Bottom Ash Fill - Medium Dense	🟩	100	110	Mohr-Coulomb	0	35
Clay - Medium to V. Stiff	🟧	116	136	Mohr-Coulomb	70	30
Native Sand - Dense	🟧	110	120	Mohr-Coulomb	0	40
Crushed Blast Furnace Slag	🟪	120	130	Mohr-Coulomb	0	40
Riprap	🟦	140	145	Mohr-Coulomb	0	45
Large Limestone Riprap	🟩	140	145	Mohr-Coulomb	0	45
steel sheeting	🟥	120	NA	Infinite strength	NA	NA

Method: spencer
Factor of Safety: 1.354
Center: 64.969, 646.464
Radius: 59.176
Left Slip Surface Endpoint: 22.531, 605.224
Right Slip Surface Endpoint: 80.197, 589.281

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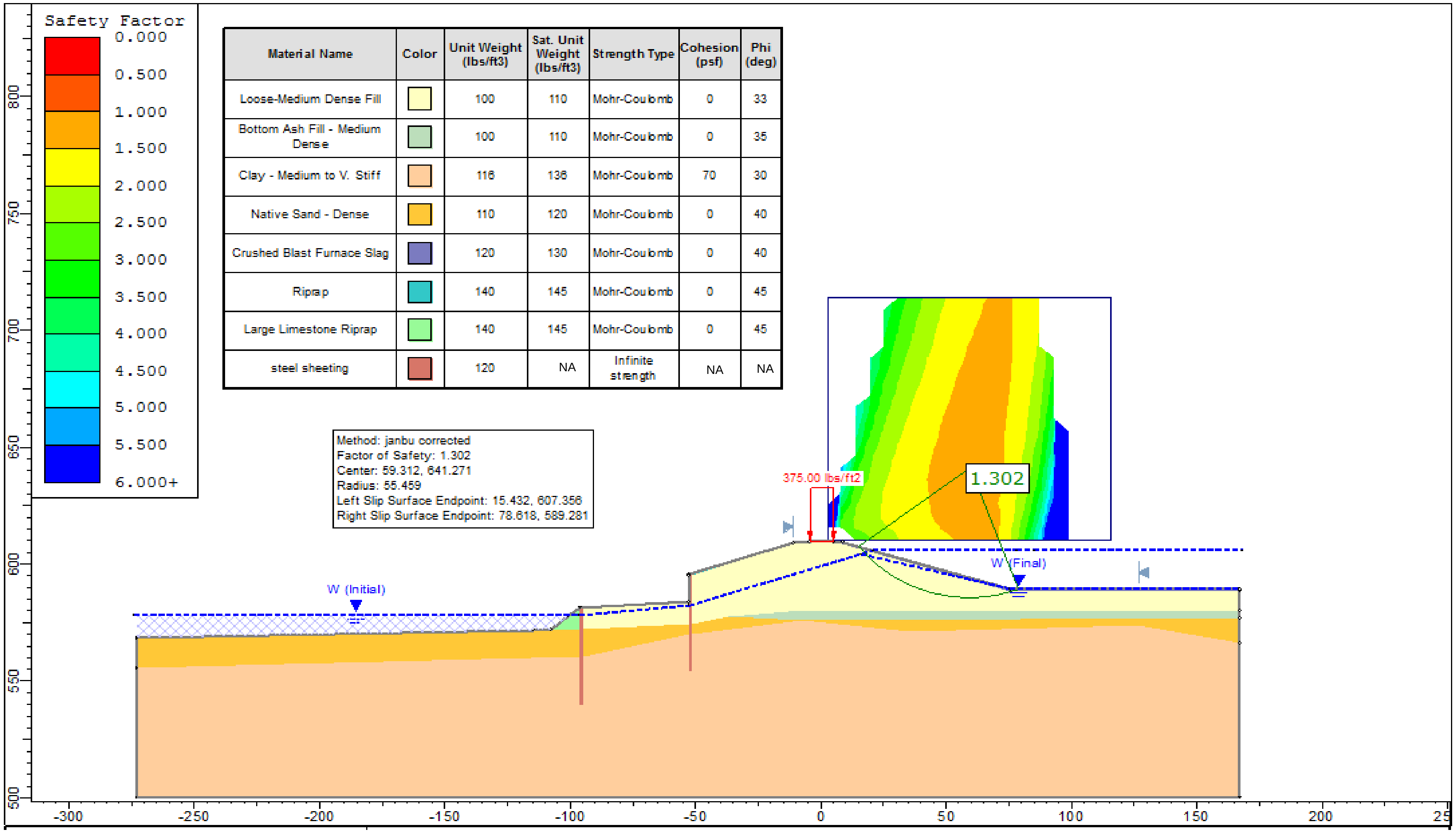
TITLE

Primary Settling Pond No. 2 Rapid Drawdown
Circular Slip Surface

Northern Indiana Public Service Company

FIGURE **11**



Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Loose-Medium Dense Fill	🟨	100	110	Mohr-Coulomb	0	33
Bottom Ash Fill - Medium Dense	🟩	100	110	Mohr-Coulomb	0	35
Clay - Medium to V. Stiff	🟧	116	136	Mohr-Coulomb	70	30
Native Sand - Dense	🟧	110	120	Mohr-Coulomb	0	40
Crushed Blast Furnace Slag	🟪	120	130	Mohr-Coulomb	0	40
Riprap	🟦	140	145	Mohr-Coulomb	0	45
Large Limestone Riprap	🟩	140	145	Mohr-Coulomb	0	45
steel sheeting	🟥	120	NA	Infinite strength	NA	NA

Method: janbu corrected
Factor of Safety: 1.302
Center: 59.312, 641.271
Radius: 55.459
Left Slip Surface Endpoint: 15.432, 607.356
Right Slip Surface Endpoint: 78.618, 589.281

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Primary Settling Pond No. 2 Rapid Drawdown
Circular Slip Surface

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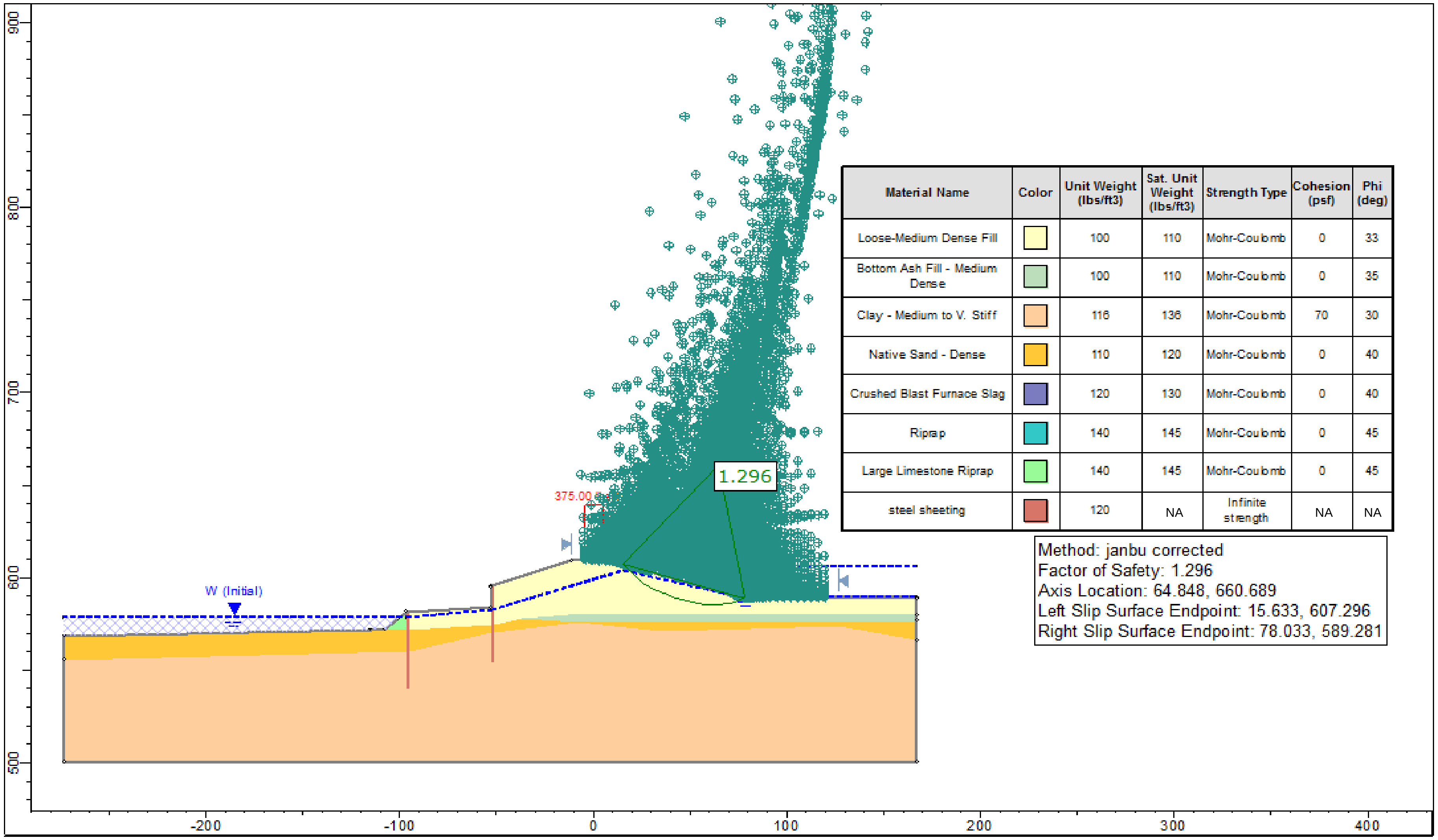
Northern Indiana Public Service Company

FIGURE 12



Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Loose-Medium Dense Fill	🟨	100	110	Mohr-Coulomb	0	33
Bottom Ash Fill - Medium Dense	🟩	100	110	Mohr-Coulomb	0	35
Clay - Medium to V. Stiff	🟧	116	136	Mohr-Coulomb	70	30
Native Sand - Dense	🟨	110	120	Mohr-Coulomb	0	40
Crushed Blast Furnace Slag	🟪	120	130	Mohr-Coulomb	0	40
Riprap	🟦	140	145	Mohr-Coulomb	0	45
Large Limestone Riprap	🟩	140	145	Mohr-Coulomb	0	45
steel sheeting	🟥	120	NA	Infinite strength	NA	NA

Method: janbu corrected
Factor of Safety: 1.296
Axis Location: 64.848, 660.689
Left Slip Surface Endpoint: 15.633, 607.296
Right Slip Surface Endpoint: 78.033, 589.281

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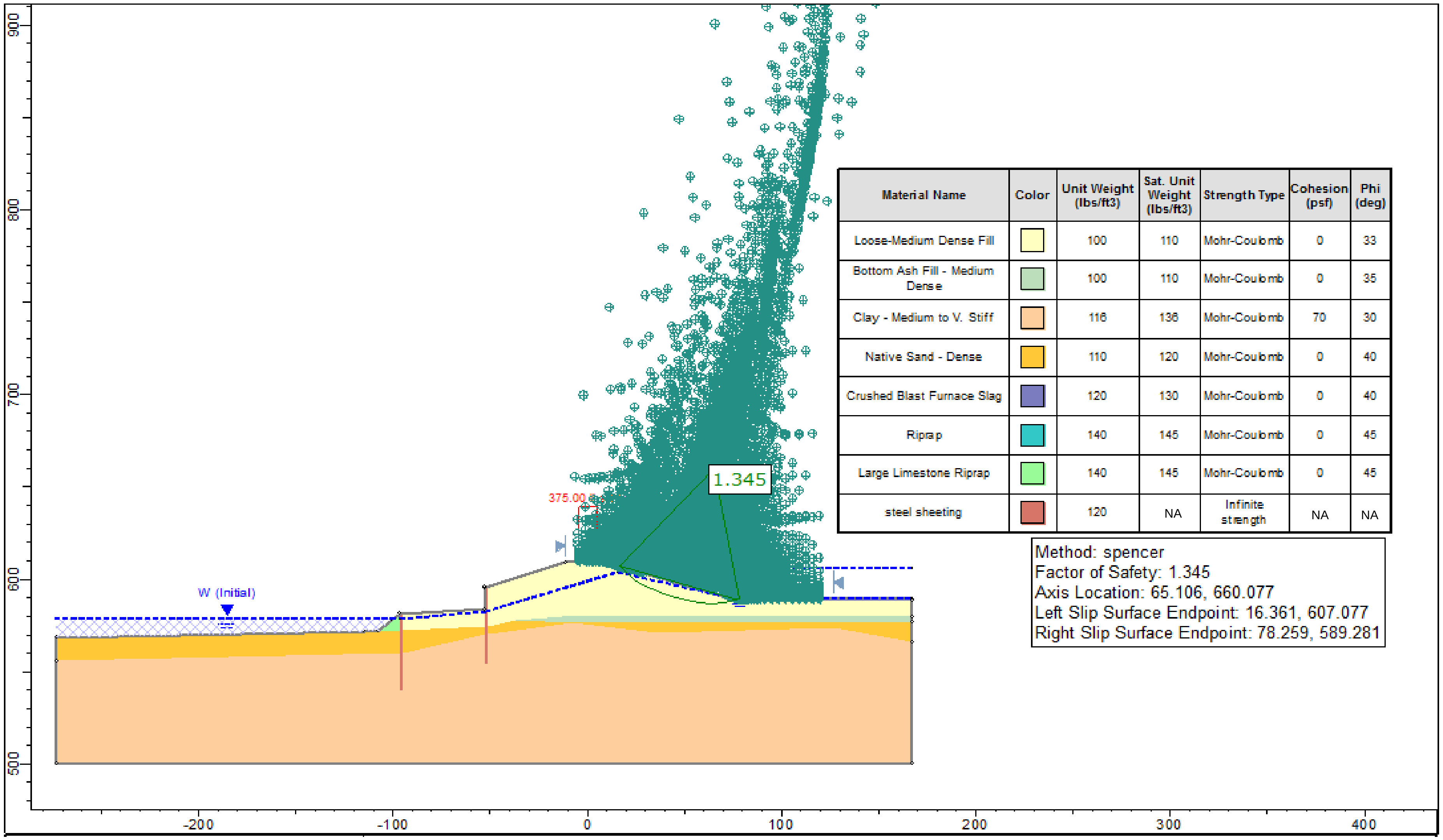
TITLE

Primary Settling Pond No. 2 Rapid Drawdown Non-Circular Slip Surface

Northern Indiana Public Service Company

FIGURE 13



Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Loose-Medium Dense Fill	🟨	100	110	Mohr-Coulomb	0	33
Bottom Ash Fill - Medium Dense	🟩	100	110	Mohr-Coulomb	0	35
Clay - Medium to V. Stiff	🟧	116	136	Mohr-Coulomb	70	30
Native Sand - Dense	🟨	110	120	Mohr-Coulomb	0	40
Crushed Blast Furnace Slag	🟪	120	130	Mohr-Coulomb	0	40
Riprap	🟦	140	145	Mohr-Coulomb	0	45
Large Limestone Riprap	🟩	140	145	Mohr-Coulomb	0	45
steel sheeting	🟥	120	NA	Infinite strength	NA	NA

Method: spencer
Factor of Safety: 1.345
Axis Location: 65.106, 660.077
Left Slip Surface Endpoint: 16.361, 607.077
Right Slip Surface Endpoint: 78.259, 589.281

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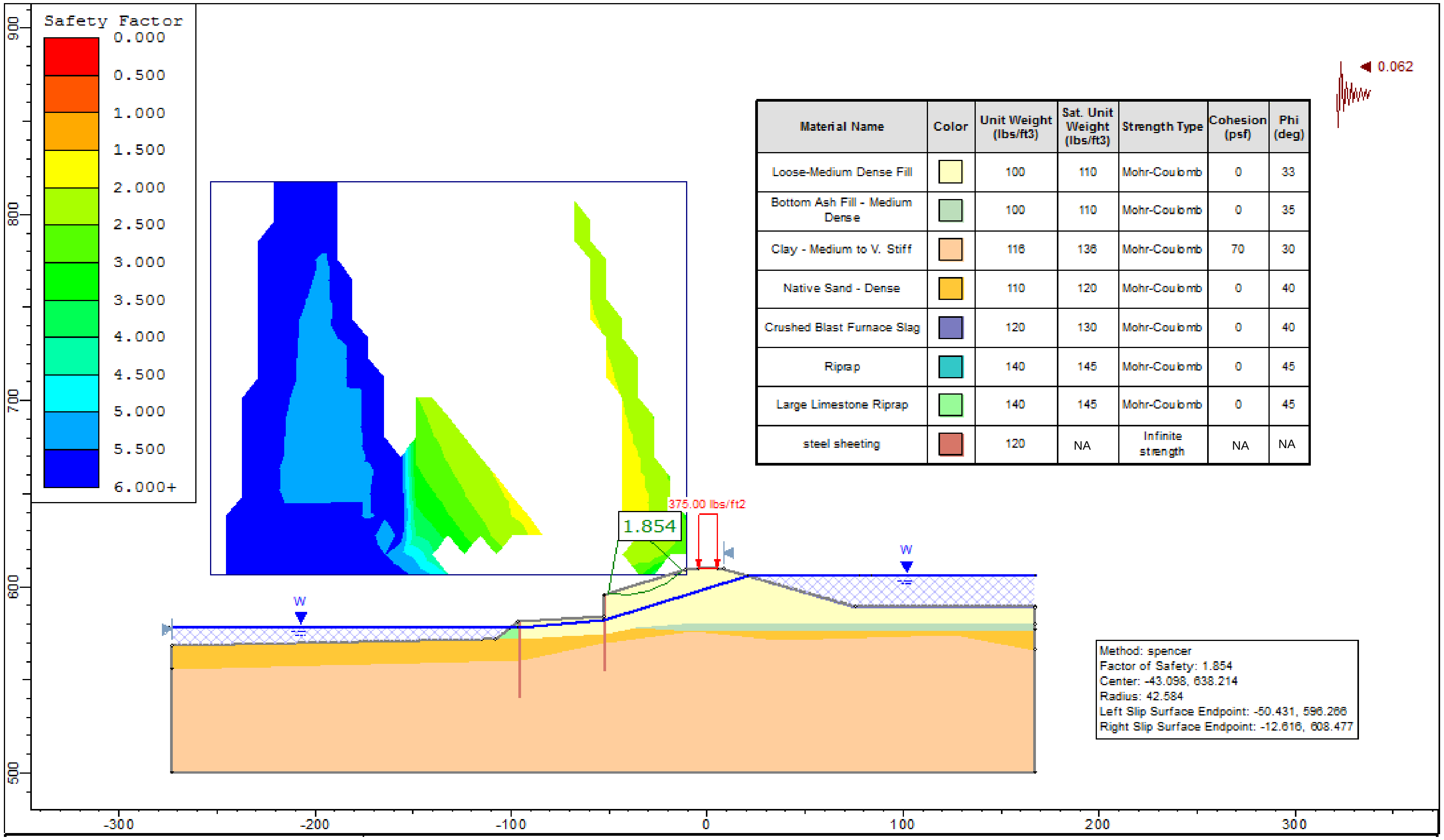
Primary Settling Pond No. 2 Rapid Drawdown
Non-Circular Slip Surface

Northern Indiana Public Service Company

FIGURE **14**



Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Loose-Medium Dense Fill	🟨	100	110	Mohr-Coulomb	0	33
Bottom Ash Fill - Medium Dense	🟩	100	110	Mohr-Coulomb	0	35
Clay - Medium to V. Stiff	🟧	116	136	Mohr-Coulomb	70	30
Native Sand - Dense	🟨	110	120	Mohr-Coulomb	0	40
Crushed Blast Furnace Slag	🟪	120	130	Mohr-Coulomb	0	40
Riprap	🟦	140	145	Mohr-Coulomb	0	45
Large Limestone Riprap	🟩	140	145	Mohr-Coulomb	0	45
steel sheeting	🟥	120	NA	Infinite strength	NA	NA

Method: spencer
Factor of Safety: 1.854
Center: -43.098, 638.214
Radius: 42.584
Left Slip Surface Endpoint: -50.431, 596.266
Right Slip Surface Endpoint: -12.616, 608.477

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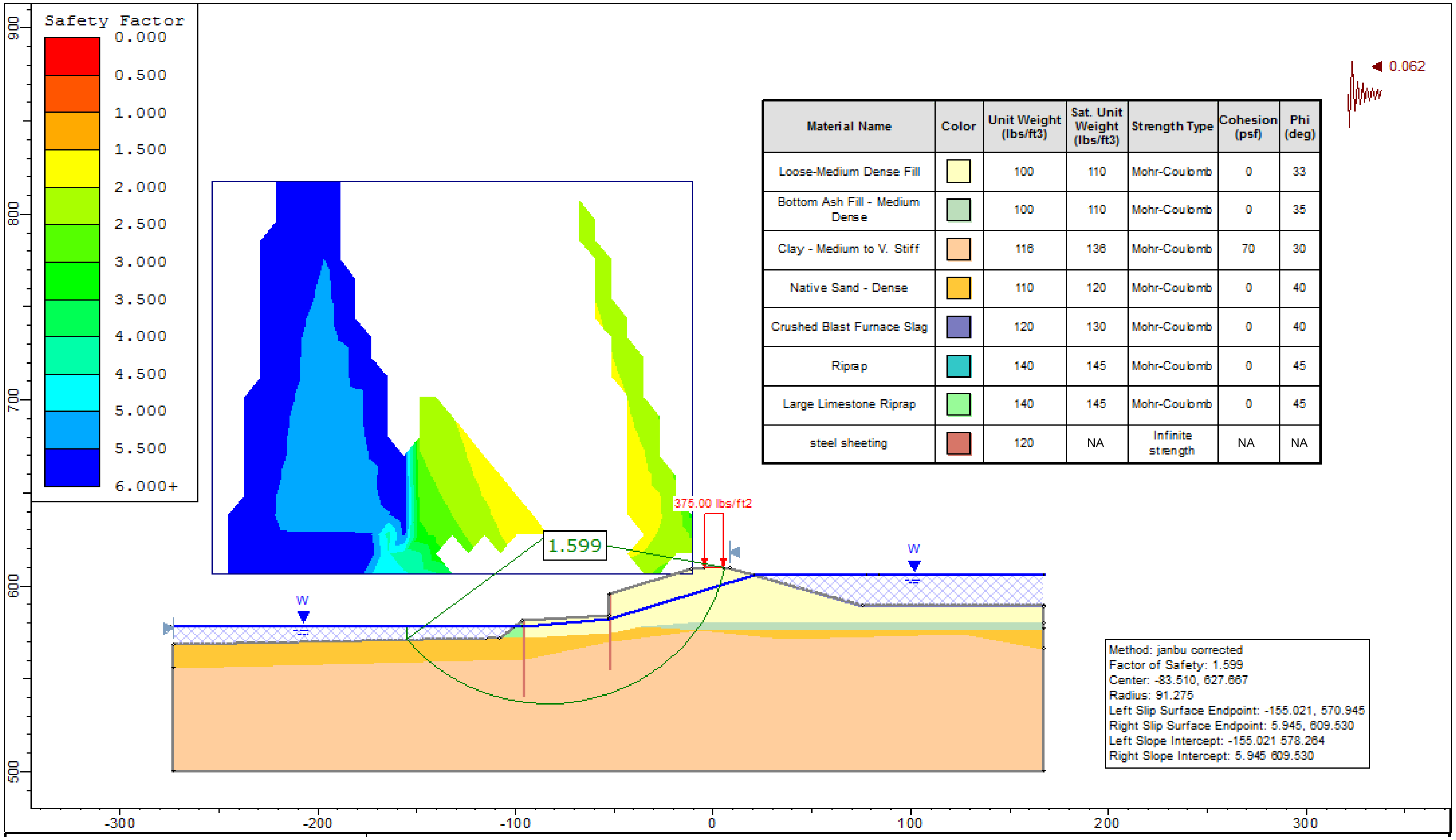
TITLE

Primary Settling Pond No. 2 Pseudo-Static
Circular Slip Surface

Northern Indiana Public Service Company

FIGURE **15**

Note: Slide software disregards potential failure surfaces that intersect materials of infinite strength. As such there is an area of "null" results in the Safety Factor map above.



Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Loose-Medium Dense Fill	🟨	100	110	Mohr-Coulomb	0	33
Bottom Ash Fill - Medium Dense	🟩	100	110	Mohr-Coulomb	0	35
Clay - Medium to V. Stiff	🟧	116	136	Mohr-Coulomb	70	30
Native Sand - Dense	🟨	110	120	Mohr-Coulomb	0	40
Crushed Blast Furnace Slag	🟪	120	130	Mohr-Coulomb	0	40
Riprap	🟦	140	145	Mohr-Coulomb	0	45
Large Limestone Riprap	🟩	140	145	Mohr-Coulomb	0	45
steel sheeting	🟥	120	NA	Infinite strength	NA	NA

Method: janbu corrected
Factor of Safety: 1.599
Center: -83.510, 627.667
Radius: 91.275
Left Slip Surface Endpoint: -155.021, 570.945
Right Slip Surface Endpoint: 5.945, 609.530
Left Slope Intercept: -155.021 578.264
Right Slope Intercept: 5.945 609.530

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PROJECT No. 1787453.006
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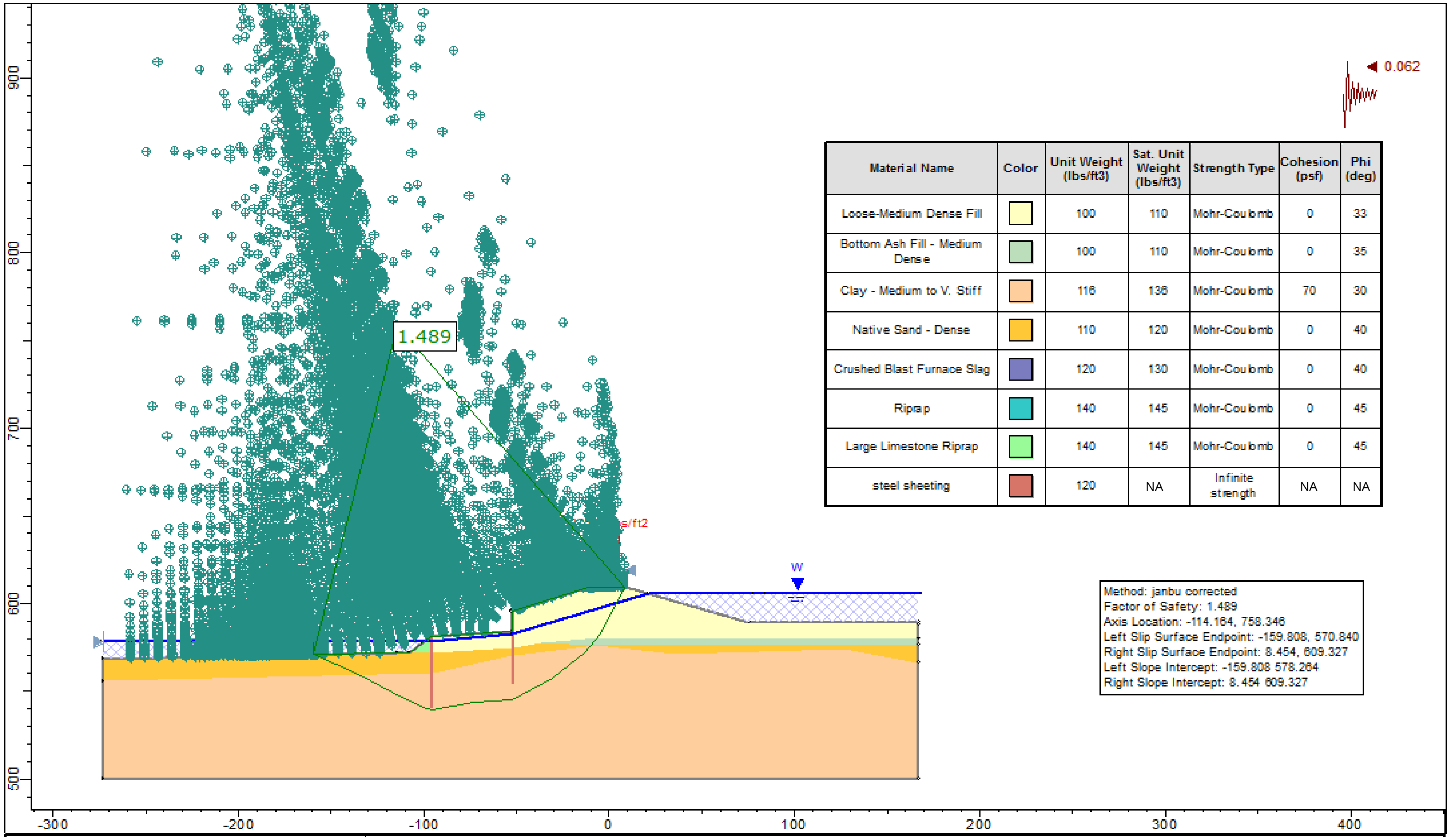
TITLE

Primary Settling Pond No. 2 Psuedo-Static
Circular Slip Surface

Northern Indiana Public Service Company

FIGURE 16

Note: Slide software disregards potential failure surfaces that intersect materials of infinite strength. As such there is an area of "null" results in the Safety Factor map above.



Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Loose-Medium Dense Fill	🟨	100	110	Mohr-Coulomb	0	33
Bottom Ash Fill - Medium Dense	🟩	100	110	Mohr-Coulomb	0	35
Clay - Medium to V. Stiff	🟧	116	136	Mohr-Coulomb	70	30
Native Sand - Dense	🟨	110	120	Mohr-Coulomb	0	40
Crushed Blast Furnace Slag	🟪	120	130	Mohr-Coulomb	0	40
Riprap	🟦	140	145	Mohr-Coulomb	0	45
Large Limestone Riprap	🟩	140	145	Mohr-Coulomb	0	45
steel sheeting	🟥	120	NA	Infinite strength	NA	NA

Method: janbu corrected
Factor of Safety: 1.489
Axis Location: -114.164, 758.346
Left Slip Surface Endpoint: -159.808, 570.840
Right Slip Surface Endpoint: 8.454, 609.327
Left Slope Intercept: -159.808 578.264
Right Slope Intercept: 8.454 609.327

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PROJECT No. 1787453.006
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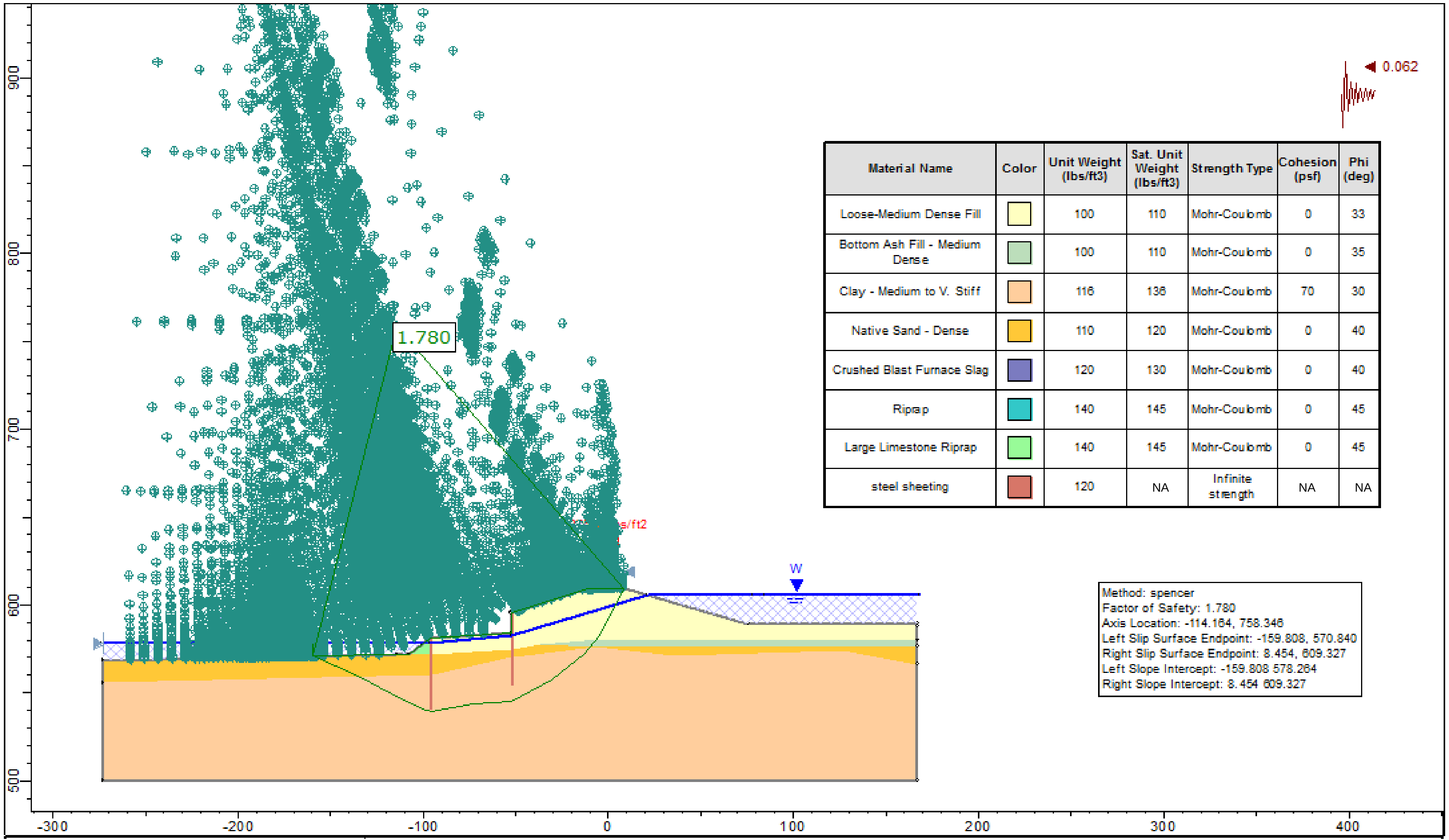
Primary Settling Pond No. 2 Psuedo-Static
Non-Circular Slip Surface

Northern Indiana Public Service Company

FIGURE **17**



Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Loose-Medium Dense Fill	🟨	100	110	Mohr-Coulomb	0	33
Bottom Ash Fill - Medium Dense	🟩	100	110	Mohr-Coulomb	0	35
Clay - Medium to V. Stiff	🟧	116	136	Mohr-Coulomb	70	30
Native Sand - Dense	🟨	110	120	Mohr-Coulomb	0	40
Crushed Blast Furnace Slag	🟪	120	130	Mohr-Coulomb	0	40
Riprap	🟦	140	145	Mohr-Coulomb	0	45
Large Limestone Riprap	🟩	140	145	Mohr-Coulomb	0	45
steel sheeting	🟥	120	NA	Infinite strength	NA	NA

Method: spencer
Factor of Safety: 1.780
Axis Location: -114.164, 758.346
Left Slip Surface Endpoint: -159.808, 570.840
Right Slip Surface Endpoint: 8.454, 609.327
Left Slope Intercept: -159.808 578.264
Right Slope Intercept: 8. 454 609.327

Golder Associates Inc.

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Primary Settling Pond No. 2 Pseudo-Static Non-Circular Slip Surface

Northern Indiana Public Service Company

FILE STABILITY

PROJECT No. 1787453.006 REV. 0

FIGURE 18

APPENDIX B
PRIMARY SETTLING POND NO. 2 – LIQUEFACTION POTENTIAL ANALYSIS

CALCULATIONS

Date:	01/12/18	**Made by:**	Amin Kandi
Project No.:	1787453	**Checked by:**	Steve McManus
Subject:	Liquefaction Assessment for Primary Settling Pond No. 2	**Reviewed by:**	Tiffany Johnson
Project:	**MCGS – PRIMARY 2 – CCR RULE COMPLIANCE PLANS**		

1.0 OBJECTIVE

The objective of this calculation is to assess the liquefaction potential of the foundation soils and surrounding dikes at Primary Settling Pond No. 2 (Primary 2) at the Michigan City Generating Station (MCGS). This is a screening-level assessment, performed using Standard Penetration Test (SPT) results to calculate the factor of safety (FS) against liquefaction within boreholes. The borehole locations assessed are shown on the attached Figure 1 (Attachment 1).

2.0 SUBSURFACE CONDITIONS

Twelve Hollow Stem Auger (HSA) boreholes were drilled in and around several of the embankments at the MCGS in 2012 (Golder, 2012). Three of these boreholes (BH-5, BH-6, and BH-7) were drilled from near the center of the crest of the west, north and east embankments of Primary 2. Boreholes BH-10, BH-11 and BH-12 were also advanced downstream of the north embankment. BH-5, BH-6 and BH-7 were advanced to depths of 50 feet below ground surface (ft bgs), 50 ft bgs, and 40 ft bgs, respectively.

These boreholes indicate the subsurface material consists of dense to very dense slag and sand from ground surface to approximately 1 ft bgs. Immediately below this upper layer there is a layer of loose to medium dense sand, approximately 35 ft thick. Borings BH-5, BH-6, BH-7 indicate that below the upper sand layer is a layer of dense to very dense black ash 2.5 ft to 5 ft thick. Below this ash layer, is a discontinuous layer of dense to very dense sand as indicated in BH-5, BH-10, BH-11, and BH-12. This layer was not present in boreholes BH-6 or BH-7. BH-7 was terminated at a depth of 40 ft bgs (approximately elevation 570 ft above mean sea level; amsl) and neither the black ash nor any underlying units were encountered. The borehole log for BH-6 indicates a medium stiff clay layer is present below the black ash to the end of the boring at 50 ft bgs (approximately elevation 560 ft above mean sea level; amsl). BH-10 indicates that the clay later extends to approximately elevation 535 ft amsl (75 feet below the embankment crest).

During drilling activities associated with the 2012 Geotechnical Report (Golder, 2012), groundwater was observed at 8.5 ft bgs in BH-5 (north embankment). Groundwater was observed at 18 ft bgs in BH-6 (west embankment) and 11.5 ft bgs in BH-7 (east embankment).

3.0 LIQUEFACTION ASSESSMENT METHODOLOGY

A preliminary liquefaction analysis was conducted to assess the liquefaction hazard potential based on the results of SPT. The state-of-practice “simplified method,” based on the original work by R.B. Seed and later updated by Idriss and Boulanger et al. (2008), was utilized. In general, this method employs the effective stress, relative density, and fines content (percentage of soil passing the No. 200 sieve) of the soils to calculate resistance or susceptibility of soil to liquefaction. These inputs are used to calculate the Cyclic Resistance Ratio (CRR) that indicates the capacity of the soil to resist liquefaction. The CRR is compared to the seismic “demand" on the soil, which is expressed in terms of the Cyclic Stress Ratio (CSR). The CSR was estimated using seismic hazard assessment resources provided by the United States Geologic Survey (USGS). The calculated FS against liquefaction is the ratio of these two values (CRR/CSR).

3.1 CSR Determination

The CSR is defined as:

$$CSR = \frac{\tau_{ave}}{\sigma'_v} = 0.65\left(\frac{a_{max}}{g}\right)\left(\frac{\sigma_v}{\sigma'_v}\right)r_d$$

Where:

a_{max}: is the peak horizontal acceleration at the ground surface (g),

g: is the acceleration due to gravity at Earth’s surface (feet per second squared; ft/s^2),

σ_v: is the total vertical overburden stress (pounds per square foot; psf),

τ_{ave}: is the average shear stress (psf)

σ'_v: is the effective vertical overburden stress (psf), and

r_d: is a depth-dependent stress reduction factor defined as:

$$r_d = \exp(\alpha(z) + \beta(z)M)$$

$$\alpha(z) = -1.012 - 1.126\sin\left(\frac{z}{11.73} + 5.133\right)$$

$$\beta(z) = 0.106 + 0.118\sin\left(\frac{z}{11.28} + 5.142\right)$$

Where:

z: is the depth in meters (meters; m)

M: is the design earthquake moment magnitude.

3.1.1 Determination of Reference Site Peak Ground Acceleration

The peak horizontal acceleration at the ground surface is termed a_{max}. This value is determined by first estimating the peak ground acceleration (PGA) for a reference site (with an assumed average shear wave velocity for the upper 30 m (V_{s30}) of the soil profile), and then by adjusting that reference PGA to a site-specific PGA by using knowledge of the estimated average shear wave velocity for the upper 30 m of the soil profile.

The reference PGA, termed PGA,BC, corresponds to a reference site on the border between National Earthquake Reductions Hazard Program (NEHRP) site classes B and C, with a V_{s30} of 760 meters per second (m/s) (Ref. 6). The PGA,BC was estimated from a probabilistic seismic hazard analysis. An online tool that provides de-aggregated data from the 2014 USGS seismic hazard maps was used to calculate the PGA,BC (Ref. 7). The de-aggregation plot for the MCGS (41.719˚N, 86.915˚W) is shown below and indicates that the PGA,BC for a 2,475-year return period is 0.0508g (Figure 2).

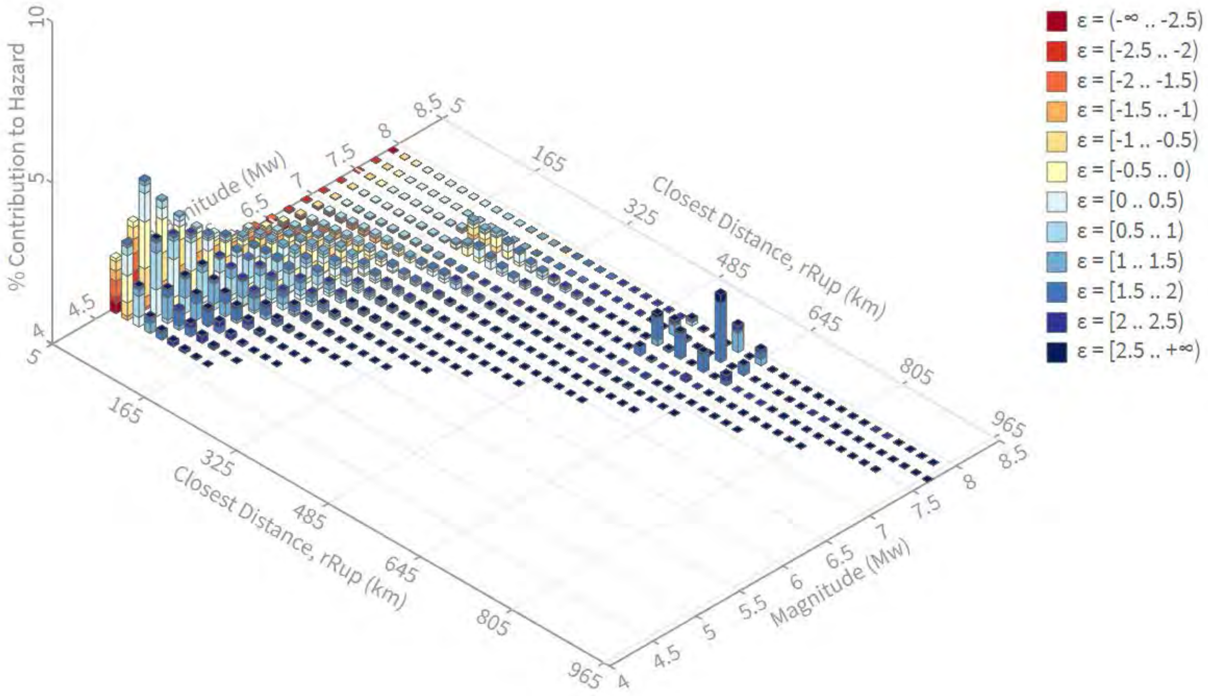


Figure 2: De-aggregation Data from Probabilistic Seismic Hazard Analysis

3.1.2 Determination of Site-Specific Peak Ground Acceleration

Based on the site-specific Vs30 value, an amplification (or de-amplification) factor needs to be applied to the PGA,BC value. Historical borehole BH-8 (Golder, 2012) was used to estimate the V_{s30} for the site since it extends deeper than the boreholes at Primary 2 (75 ft bgs). Blow counts observed in the field (N_m) for BH-5, BH-6, and BH-7 were documented in field notes and are presented in Attachment 2 to this document. Observed blow counts were corrected using the method presented in Idriss and Boulanger (2008) (Attachment 5):

$$N_{60} = C_E C_B C_R C_S N_m$$

Where:
N_{60}: blow count for an energy ratio of 60%
C_E: energy ratio correction factor
C_B: correction factor for borehole diameter (equal to 1.15 for boreholes with D ≥ 8 inches; in)
C_R: correction factor for rod length
C_S: correction factor for samplers without liners (equal to 1.0 for a split spoon sampler with liners)
N_m: measured blow count
ER_m: delivered energy ratio

The N_{60} was normalized to 1 atmosphere (approximately 100 kilopascals; kPa or 1 ton per square foot; tsf) to obtain (N1)60

(Liao and Whitman, 1986) as:

$$(N_1)_{60} = C_N \cdot N_{60}$$

$$C_N = \left(\frac{P_a}{\sigma'_{vc}}\right)^{0.5} \leq 1.7$$

Where:
$(N_1)60$: blow count for an energy ratio of 60%, normalized to 1 atmosphere (atm)
C_N: SPT overburden correction factor
P_a: atmospheric pressure (approximately 2000 psf for the site)
σ'_{vc}: vertical effective stress (psf)

Corrected blow counts from SPTs within BH-8 were used to estimate the shear modulus (G):

$$G = 15.09\ [(N_1)_{60}]^{0.74} \qquad \text{(Anbazhagan, 2012)}$$

The shear wave velocity then was calculated as:

$$v_s = \sqrt{\frac{G}{\rho}}$$

Where :

G : Shear modulus (pounds per square foot; psf)

ρ : Density (slug per cubic foot)

As shown in Table 1, the average site-specific V_{s30} is 766 feet per second (ft/s; 234 m/s). Since this V_{s30} value is lower than what was assumed (760 m/s) when the PGA,BC value was calculated, the PGA,BC value was amplified to obtain a_{max}.

Table 1: Average Shear Wave Velocity in the Upper 30 m (V_{s30}) at BH-8

Depth to top of SPT, D (ft)	Thickness, d_i (ft)	Observed Blow Count, N	Dry Unit Weight, γ_d (pcf)	Overburden Correction Factor, C_N	Corrected Blow Count, $(N_1)_{60}$	Shear Modulus, G (megapascals, Mpa)	Shear Velocity, V_s (ft/s)	Ratio of Thickness to Shear Velocity, d_i/V_{si} (seconds; sec)
1	2.5	18	100	1.7	23.0	153.3	1015.5	0.0025
3.5	2.5	8	100	1.7	10.2	84.1	752.3	0.0033
6	2.5	16	100	1.7	20.4	140.5	972.2	0.0026
8.5	2.5	3	100	1.7	3.8	40.7	523.3	0.0048
11	2.5	1	100	1.6	1.2	17.3	341.4	0.0073
13.5	2.5	1	100	1.5	1.3	18.0	348.3	0.0072
18.5	5	1	100	1.3	1.1	16.5	333.4	0.0150
23.5	5	39	100	1.2	44.8	251.5	1300.6	0.0038
28.5	5	39	100	1.1	41.3	237.0	1262.6	0.0040
33.5	5	35	100	1.0	36.4	215.9	1205.1	0.0042
38.5	5	12	100	1.0	11.8	93.5	793.0	0.0063
43.5	5	12	116	0.9	10.8	88.0	714.4	0.0070
48.5	5	16	116	0.8	13.5	103.4	774.3	0.0065
53.5	5	16	116	0.8	12.7	98.8	757.0	0.0066
58.5	5	36	116	0.8	27.0	173.0	1001.4	0.0050
63.5	5	19	116	0.7	13.6	103.9	776.2	0.0064
68.5	5	66	116	0.7	45.1	252.6	1210.3	0.0041
73.5	5	30	116	0.7	19.7	136.7	890.4	0.0056
75	23.4	30	116	0.6	19.4	135.6	886.5	0.0264
							Average V_s (ft/s)=	**765.6**

The amplification factor was calculated to be 1.86 using the site response term from the Atkinson and Boore (2006) publication on earthquake ground-motion prediction equations for Eastern North America.

a_{max} was thus calculated to be 0.0508g x 1.86 = 0.094g for a 2,475 year return period.

This a_{max} value was used to calculate CSR.

3.2 CRR Determination

To assess the liquefaction susceptibility of soils using the simplified method, the CRR must be estimated. Idriss and Boulanger (2008) developed a procedure for calculating CRR from the SPT as a function of the “clean sand” standard penetration resistance normalized to 1 atmosphere (atm; approximately 100 kPa) and given as $(N1)_{60cs}$. The CRR is based on an earthquake magnitude of 7.5 and a magnitude scaling factor (MSF) adjusts the CRR for magnitudes other than 7.5.

The CRR for an earthquake magnitude (M) of 7.5 when σ'_{vc} =1 atm is given as:

$$CRR_{7.5,\sigma'_{vc}=1} = \exp\left(\frac{(N_1)_{60cs}}{14.1} + \left(\frac{(N_1)_{60cs}}{126}\right)^2 - \left(\frac{(N_1)_{60cs}}{23.6}\right)^3 + \left(\frac{(N_1)_{60cs}}{25.4}\right)^4 - 2.8\right)$$

Where:
$(N_1)_{60cs}$: equivalent clean sand corrected $(N_1)_{60}$

$$(N_1)_{60cs} = (N_1)_{60} + \Delta(N_1)_{60}$$

$$\Delta(N_1)_{60} = \exp\left(1.63 + \frac{9.7}{FC + 0.01} - \left(\frac{15.7}{FC + 0.01}\right)^2\right)$$

Where:
FC: fines content (expressed in percent)
$\Delta(N_1)_{60}$: fines content correction factor for $(N_1)_{60}$
$(N_1)_{60}$: blow count for an energy ratio of 60%, normalized to 1 atm

The CRR of sand depends on the effective overburden stress, therefore the overburden correction factor (K_σ) was used to adjust the CRR for the effective overburden stress:

$$CRR_{7.5} = K_\sigma * CRR_{7.5,\sigma'_{vc}=1}$$

$$K_\sigma = 1 - C_\sigma \ln\left(\frac{\sigma'_{vc}}{P_a}\right) \leq 1.1$$

$$C_\sigma = \frac{1}{18.9 - 2.55((N1)_{60})^{0.5}} \leq 0.3$$

3.2.1 Magnitude Scaling Factor (MSF)

The magnitude scaling factor (MSF) adjusts the CRR for magnitudes other than 7.5 where the FS against liquefaction is calculated as

$$FS = \frac{CRR_{7.5}}{CSR} \times MSF$$

The MSF values used in this assessment are calculated as (Idriss, 1999):

$$MSF = 6.9 \exp\left(\frac{-M}{4}\right) - 0.058$$

$$where\ MSF \leq 1.8$$

A probabilistic seismic hazard analysis was used to estimate the PGA (see Section 3.1.2), but such an analysis includes the aggregate contributions of all possible combinations of magnitude and distance from all sources, all weighted by their relative likelihoods of occurrence (Kramer, 2008). However, the simplified method requires the selection of a single earthquake magnitude. Since liquefaction is sensitive to ground motion duration, which is correlated to earthquake magnitude, this selection is an important component of liquefaction assessments.

The selection of either the mean or modal magnitude produces inconsistent risks of liquefaction because the relationship between duration (represented by magnitude) and liquefaction potential is non-linear. Kramer (2008) suggests that the best way to handle this issue is to perform liquefaction calculations for all magnitudes, and to weight the results according to the relative contribution of each magnitude.

Golder has implemented this approach by recognizing that the MSF is the only term in the simplified approach that is affected by the magnitude selection. Golder calculated a weighted-average MSF (weighted by the relative contribution of each magnitude) and then calculated the design earthquake magnitude corresponding to that MSF.

Golder calculated the design earthquake magnitude to be 5.84. This value is slightly less than the mean magnitude (5.92), and is greater than the modal magnitude (4.90).

3.3 Factor of Safety Against Liquefaction

The factor of safety against liquefaction was calculated as:

$$FS = \frac{CRR_{7.5}}{CSR} \times MSF$$

The FS was calculated for each point where SPT results were recorded. The results are presented in the attached Tables 2 through 4 (Attachment 4). The attached Figure 3 (Attachment 3) presents the calculated FSs in graphical form.

4.0 RESULTS AND CONCLUSIONS

Golder used the "simplified method," in-situ measurements, and probabilistic seismic hazard data to assess the likelihood of liquefaction within or beneath the embankment of Primary 2. This approach addresses the likelihood that seismic shaking will trigger liquefaction. The calculations indicate that the three boreholes assessed exhibit acceptable factors of safety against liquefaction (greater than or equal to 1.2).

References

1. Anbazhagan, P., Parihar, A., & Rashmi, H. N. (2012). Review of correlations between SPT N and shear modulus: A new correlation applicable to any region. Soil Dynamics and Earthquake Engineering, 36, 52-69. doi:10.1016/j.soildyn.2012.01.005

2. Atkinson, G.M. and D.M. Boore (2006) "Earthquake Ground-Motion Prediction Equations for Eastern North America," *Bulletin of the Seismological Society of America,* Vol. 96, No. 6, pp. 2181-2205.

3. Idriss, I.M. and R.W Boulanger (2008) "Soil Liquefaction During Earthquakes", Earthquake Engineering Research Institute Monograph No. MNO-12.

4. Kramer, S.L. (2008). "Evaluation of Liquefaction Hazards in Washington State" Final Research report WA-RD 668.1, December 2008.

5. Liao, S.S.C. and R.V. Whitman (1986) "Overburden Correction Factors for SPT in Sand", Journal of Geotechnical Engineering, A.S.C.E., v. 112:3, p. 373-377.

6. National Earthquake Reductions Hazard Program (2015), "NEHRP Recommended Seismic Provisions for New Buildings and Other Structures".

7. The United States Geological Survey. "Unified Hazard Tool". Retrieved November 23, 2017, from https://earthquake.usgs.gov/hazards/interactive/

Attachments

Attachment 1 – Figure 1: Borehole Location Map

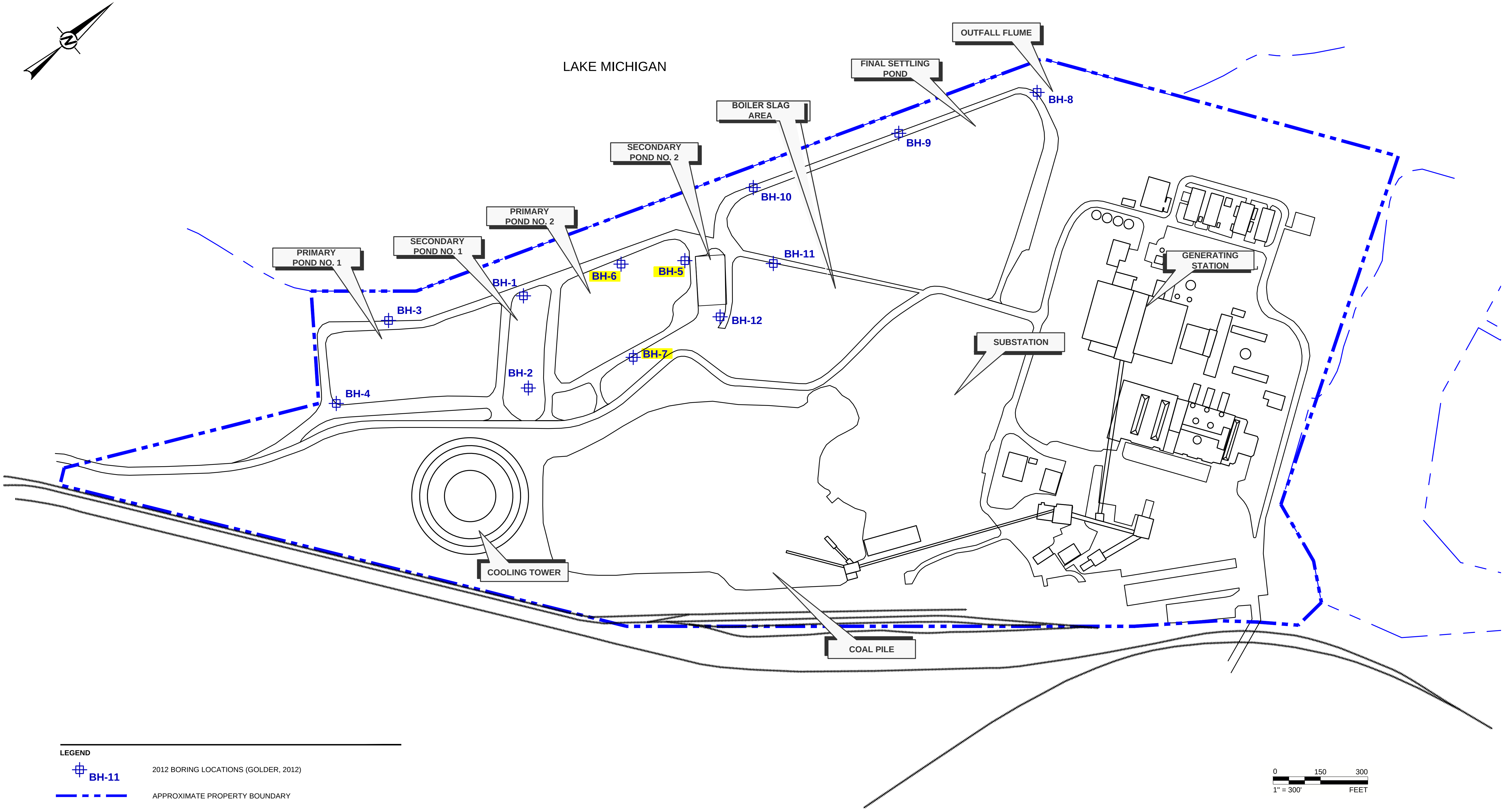
Attachment 2 – Borehole Logs for BH-5, BH-6, and BH-7 with Blow Counts Amended

Attachment 3 - Figure 3: Primary 2 Factor of Safety against Liquefaction

Attachment 4 – Tables 2 through 4: SPT Blow Count Corrections and Liquefaction Potential

Attachment 5 – References

ATTACHMENT 1



LEGEND

⌖ BH-11 — 2012 BORING LOCATIONS (GOLDER, 2012)

— · — APPROXIMATE PROPERTY BOUNDARY

⌖ BH-5 — 2012 BOREHOLE LOCATION (GOLDER, 2012) ANALYZED FOR SUSCEPTIBILITY TO LIQUEFACTION

REFERENCE

2012 BOREHOLE LOCATIONS FROM 2012 GEOTECHNICAL INVESTIGATION AND EMBANKMENT STABILITY ANALYSES - NIPSCO MICHIGAN CITY GENERATING STATION (GOLDER, 2012).

CLIENT
NIPSCO

CONSULTANT
Golder Associates

YYYY-MM-DD	2017-11-27
DESIGNED	AK
PREPARED	DJC
REVIEWED	SAM
APPROVED	TDJ

PROJECT
MICHIGAN CITY GENERATING STATION
101 WABASH STREET
MICHIGAN CITY, INDIANA

TITLE
BOREHOLE LOCATIONS

PROJECT NO.	CONTROL	REV.	FIGURE
1787453	1787453C001.dwg	0	1

Path: \\detroit\cad\Projects\17x-Projects\1787453-NIPSCO\PRODUCTION\C-MCGS PB2 LIQUEFACTION ANALYSIS\ | File Name: 1787453C001.dwg | Last Edited By: dcoss Date: 2017-11-27 Time:6:30:17 PM | Printed By: dcoss Date: 2017-11-28 Time:10:29:16 AM

0 1 in IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ANSI B

ATTACHMENT 2

PROJECT No.: 123-88898

RECORD OF AUGERHOLE: BH-5

SHEET 1 OF 1

LOCATION:
N: 6000.1833 E: 5296.855

DRILLING DATE: JUNE 27, 2012
DRILLING CONTRACTOR: EARTH EXPLORATION

DATUM: Geodetic

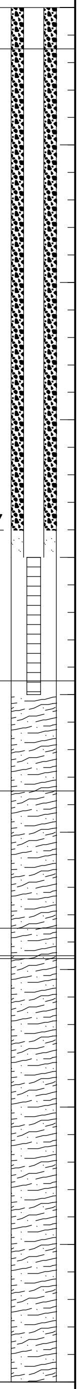
INCLINATION: -90°

BORING METHOD: Track Mounted Auger Drill, Hollow Stem Auger

DEPTH SCALE FEET	SOIL PROFILE DESCRIPTION	ELEV. / DEPTH (ft)	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/ft
	Ground Surface	609.40			
0	Gravel, small, dry, some sand/slag [FILL]	0.00			
	Very loose to loose, brown, fine SAND, dry [FILL]	607.90 1.50		51 SS	5
5				51 SS	4
				51 SS	7
	Very loose to loose, brown, fine SAND, wet [FILL]	600.90 8.50		51 SS	4
10				51 SS	2
15				51 SS	2
20				51 SS	1
	Dense, brown, fine to medium SAND, wet, trace ash [FILL]	585.90		51 SS	42
25	Very dense, gray, GRAVEL/SLAG, wet, trace sand [FILL]	585.00 24.50			
	Very dense, brown, fine to medium sand, wet, trace ash [FILL]				
30	Medium dense, black ASH, wet [FILL]	580.90 28.50		51 SS	23
	Very loose, brown fine SAND, wet, trace ash [FILL]	575.90		51 SS	6
35	Loose, black ASH, wet [FILL]	574.90 34.60			
	Very loose, brown fine to medium SAND, wet, some ash [FILL]				
40	Hard, fine to coarse SAND, brown/gray, wet [NATIVE]	570.90 38.50		51 SS	48
45	Medium dense, gray, fine to medium sand, wet [NATIVE]	565.90 43.50		51 SS	29
				51 SS	
50	End of Boring End of Augerhole.	559.40 50.00			
55					

DYNAMIC PENETRATION RESISTANCE, BLOWS/ft: 20 40 60 80
SHEAR STRENGTH Cu, psf: nat V. + Q - ● rem V. ⊕ U - ○ 400 800 1200 1600
HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}
WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl NP - Non-Plastic 10 20 30 40
ADDITIONAL LAB. TESTING

PIEZOMETER, STANDPIPE OR THERMISTOR INSTALLATION



1" Solid PVC with bentonite chip seal

7/2/2012 ▼

1" Solid PVC with filter sand

1" Slotted PVC with filter sand

DEPTH SCALE
1 in to 7 ft

Golder Associates

LOGGED: JRR
CHECKED:

PROJECT No.: 123-88898

RECORD OF AUGERHOLE: BH-6

SHEET 1 OF 1

LOCATION:
N: 5833.1477 E: 5130.5199

DATUM: Geodetic

DRILLING DATE: JUNE 28, 2012
DRILLING CONTRACTOR: EARTH EXPLORATION

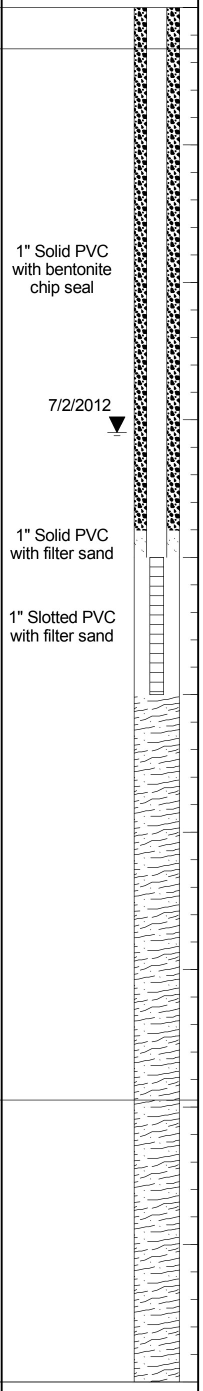
INCLINATION: -90°

BORING METHOD: Track Mounted Auger Drill — Hollow Stem Auger

DYNAMIC PENETRATION RESISTANCE, BLOWS/ft: 20 40 60 80
SHEAR STRENGTH Cu, psf: nat V. + Q - ● rem V. ⊕ U - ○ ; 400 800 1200 1600
HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}
WATER CONTENT PERCENT: Wp ⊢—⊖W—⊣ Wl ; NP - Non-Plastic ; 10 20 30 40
ADDITIONAL LAB. TESTING

DEPTH SCALE FEET	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (ft)	SAMPLES TYPE	BLOWS/ft
	Ground Surface	609.61		
0	Sand/Gravel, gray, coarse, dry [FILL]	608.61		
	Dense, gray, coarse SAND and GRAVEL, dry [FILL]			
	Very loose to loose, light brown, fine to medium SAND, dry to moist [FILL]	1.50	51 SS 51 SS 51 SS 51 SS	9 9 2 5
10	Medium dense to dense, light brown, fine to medium SAND, moist [FILL]	598.61 11.00	51 SS 51 SS	34 16
	Very loose, brown, fine to medium SAND, wet [FILL]	591.11 18.50	51 SS	3
	Very loose to medium dense, gray, fine to medium SAND, wet [FILL]	586.11 23.50	51 SS 51 SS	3 27
	Very loose, black/dark gray, bottom ASH and fine to medium SAND, wet [FILL]	576.11 33.50	51 SS 51 SS	3 6
40	Medium stiff to stiff, gray, SILTY CLAY, moist, trace sand/gravel, medium plasticity, 2.5 TSF pocket pen [NATIVE]	569.86 39.75	76 ST 51 SS 51 SS	 11 10
50	End of Boring End of Augerhole.	559.61 50.00		

PIEZOMETER, STANDPIPE OR THERMISTOR INSTALLATION



DEPTH SCALE
1 in to 7 ft

LOGGED: JRR
CHECKED:

File:P:\12X-PROJECTS\12388898 NIPSCO MCGS GEOTECH INVESTIGATION\300 FIELD INFORMATION\320 BORINGS-WELL LOGS\MCGS.GPJ Output Form:DYNAMIC ADDITIONAL LAB TESTING (WIP) Template:BC REGION TEMPLATE BETA 1.GDT Library:BC REGION LIBRARY.GLB JReese 7/17/12

PROJECT No.: 123-88898

RECORD OF AUGERHOLE: BH-7

SHEET 1 OF 1

LOCATION:
N: 5699.0208 E: 5395.0681

DATUM: Geodetic

DRILLING DATE: JUNE 27, 2012
DRILLING CONTRACTOR: EARTH EXPLORATION

INCLINATION: -90°

DEPTH SCALE FEET	BORING METHOD	SOIL PROFILE: DESCRIPTION	SOIL PROFILE: STRATA PLOT	SOIL PROFILE: ELEV. DEPTH (ft)	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: BLOWS/ft	DYNAMIC PENETRATION RESISTANCE, BLOWS/ft 20 40 60 80 / SHEAR STRENGTH Cu, psf nat V. + Q - ● rem V. ⊕ U - ○ 400 800 1200 1600	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3} / WATER CONTENT PERCENT Wp ⊢ ○W ⊣ Wl NP - Non-Plastic 10 20 30 40	ADDITIONAL LAB. TESTING	PIEZOMETER, STANDPIPE OR THERMISTOR INSTALLATION
		Ground Surface		609.39							
0	Track Mounted Auger Drill Hollow Stem Auger	Gravel, gray coarse sand [FILL]	▨	0.00							
		Medium dense, light brown, fine to medium SAND, dry, trave gravel, possible textile at 5' [FILL]		608.39 1.00		51 SS	11				
5						51 SS	10				
						51 SS	10				
10						51 SS	11				
		Very dense, gray/black, SLAG/ASH, wet [FILL]		597.89 11.50		51 SS	>50				
15		Medium dense to dense, brown, fine to medium SAND, trace black ash/slag, wet, wood at 19.25' [FILL]		595.39 14.00		51 SS	31				
20						51 SS	18				
25						51 SS	14				
30		Medium dense to dense, gray, fine to medium SAND, trace ash and gravel, wet [FILL]		580.89 28.50		51 SS	44				
35						51 SS	19				
						51 SS	17				
40		End of Boring End of Augerhole.		569.39 40.00							
45											

File:P:\12X-PROJECTS\12388898 NIPSCO MCGS GEOTECH INVESTIGATION\300 FIELD INFORMATION\320 BORINGS-WELL LOGS\MCGS.GPJ Output Form:DYNAMIC ADDITIONAL LAB TESTING (WIP) Template:BC REGION TEMPLATE BETA 1.GDT Library:BC REGION LIBRARY.GLB JReese 7/17/12

DEPTH SCALE
1 in to 6 ft

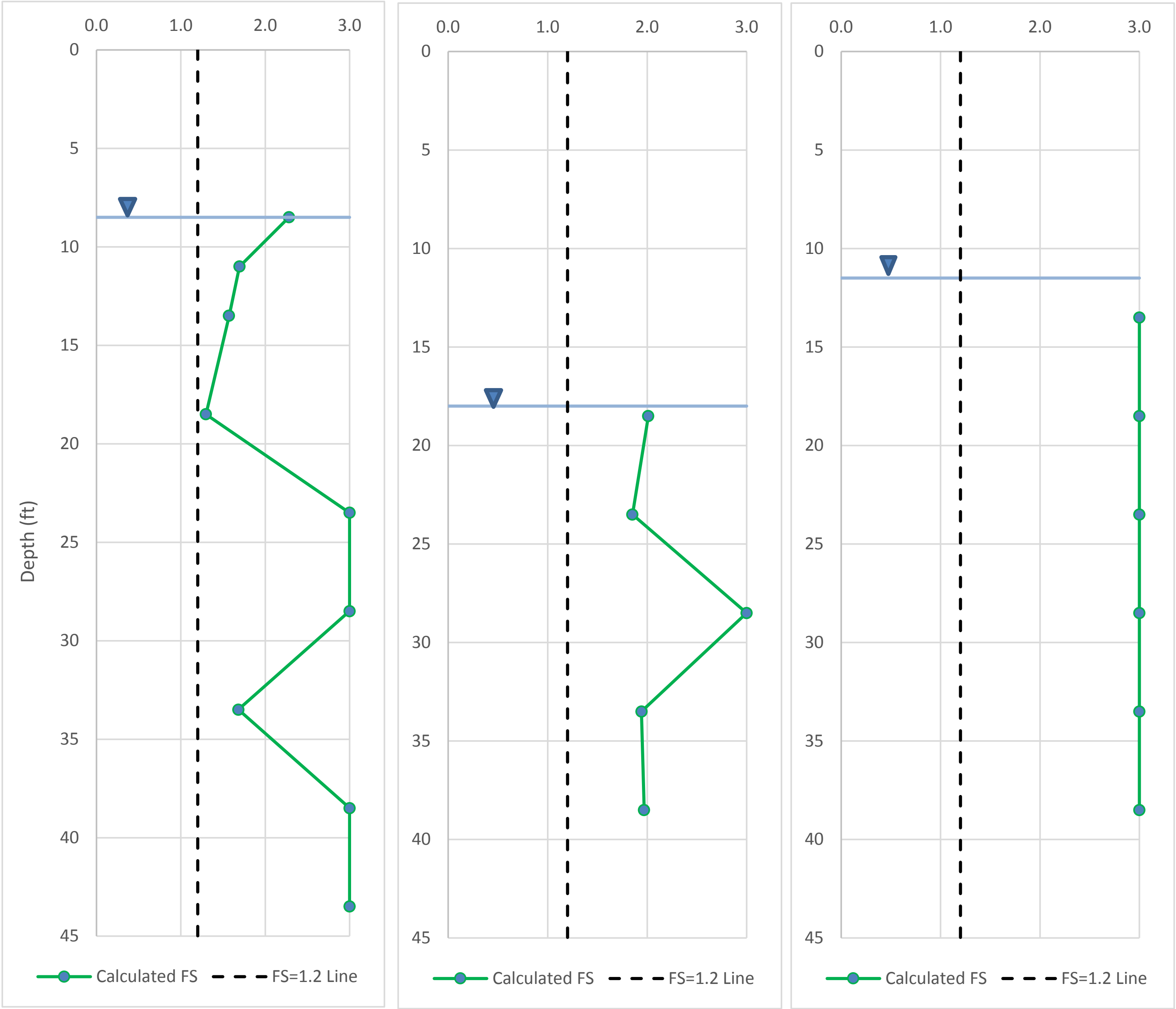
Golder Associates

LOGGED: JRR
CHECKED:

ATTACHMENT 3

BH ID:	BH-5	**BH ID:**	BH-6	**BH ID:**	BH-7
Northing [1]:	6000.1833	**Northing:**	5833.1477	**Northing:**	5699.02
Easting:	5296.8550	**Easting:**	5130.5199	**Easting:**	5395.07
Elevation [2]:	609.4 ft	**Elevation:**	609.6 ft	**Elevation:**	609.4 ft
Water Table:	8.5 ft BGS	**Water Table:**	18 ft BGS	**Water Table:**	11.5 ft BGS

FACTOR OF SAFETY AGAINST LIQUEFACTION



Notes:1. The coordinate system is US State Plane 1983, NAD 1983 (CONUS), Indiana West Zone.
2. Elevations are based upon the 1988 North American Vertical Datum (NAVD88).
3. Factors of safety (FS) greater than 3 are shown equal to 3.
4. It has been assumed that soils above the water table are not susceptible to liquefaction.

Golder Associates

SCALE --
DATE Nov-17
MADE BY AK
CAD -
CHECK SEF
REVIEW SAM

PROJECT **MCGS - PB2 - CCR Rule Compliance Plans**
TITLE **PB2 Factor of Safety Against Liquefaction**
CLIENT **NiSource-Nipsco**
FIGURE 3

FILE --
PROJECT No. 1787453
REV. 0

ATTACHMENT 4

SPT BLOW COUNT CORRECTIONS AND LIQUEFACTION POTENTIAL

Input Parameters:

Hammer / Split Spoon

Spoon OD (in.) = 2.0
Spoon ID (in.) = 1.375
Hammer weight (lb) = 140
Hammer Drop (in.) = 30
Hammer Type = Automatic
Drilling Method = 4.25" HSA
Standard SPT Spoon OD (in.) = 2.0
Standard SPT Spoon ID (in.) = 1.375
Standard SPT Hammer weight (lb) = 140
Standard SPT Hammer Drop (in.) = 30
Energy Ratio (ER) = 60 %
Hole Dia (I.D. for HSA) = 83 (65-115, 150 or 200) mm
Sampler Lined (Y/N) = N/A

Earthquake Motion

Return Period = 2,475 years
Mw = 5.84
PGA = 0.094 g
PGA_{ROCK} = 0.051 g
F_{PGA} = 1.9

Groundwater

Depth GW = 8.5 ft

Surcharge: 0 psf
Height of Surcharge: 0 ft

Ground Surface Elevation

609.4 ft

Borehole = BH-5

Depth (ft)	Elevation (ft)	Soil Type	γ_{bulk} (pcf)	N_{Field} (blows/ft)	Sample Comments (Heave, etc.)	Fines Content (%)	σ_v (psf)	σ'_v (psf)	overburden factor C_N	non-standard sampler size & hammer C_{HS}	sample liner factor C_s	energy ratio factor C_E	rod length factor C_R	borehole dia. factor C_B	N_{60} (blows/ft)	$(N_1)_{60}$ (blows/ft)	Dr %	ΔN	ΔN (based on surcharge, k= 0.4	$(N_1)_{60\,CS}$ (blows/ft)	r_d	CSR	$CRR_{7.5}$	MSF	K_σ	CRR	FS_{LIQ}	Potentially Liquefiable?
1	608.4	Sand	110	5	Unsaturated	3	110	110	1.70	1.00	1.00	1.00	0.75	1.00	4	6	34	0.000	0	6	1.004	0.062	0.09	1.54	1.100	0.16	-	No
3.5	605.9	Sand	110	4	Unsaturated	3	385	385	1.70	1.00	1.00	1.00	0.75	1.00	3	5	30	0.000	0	5	0.992	0.061	0.09	1.54	1.100	0.15	-	No
6	603.4	Sand	110	7	Unsaturated	3	660	660	1.70	1.00	1.00	1.00	0.75	1.00	5	9	40	0.000	0	9	0.979	0.060	0.11	1.54	1.100	0.19	-	No
8.5	600.9	Sand	110	4	-	2	935	934	1.46	1.00	1.00	1.00	0.75	1.00	3	4	30	0.000	0	4	0.965	0.059	0.08	1.54	1.060	0.14	2.3	No
11	598.4	Sand	110	2	-	2	1,210	1,053	1.38	1.00	1.00	1.00	0.75	1.00	2	2	30	0.000	0	2	0.950	0.067	0.07	1.54	1.046	0.11	1.7	No
13.5	595.9	Sand	110	2	-	2	1,485	1,172	1.31	1.00	1.00	1.00	0.85	1.00	2	2	30	0.000	0	2	0.934	0.073	0.07	1.54	1.039	0.11	1.6	No
18.5	590.9	Sand	110	1	-	2	2,035	1,410	1.19	1.00	1.00	1.00	0.85	1.00	1	1	30	0.000	0	1	0.899	0.080	0.07	1.54	1.025	0.10	1.3	No
23.5	585.9	Gravel	110	42	Gravel/Slag	6	2,585	1,648	1.10	1.00	1.00	1.00	0.95	1.00	40	44	89	0.009	0	44	0.862	0.083	2.00	1.54	1.073	2.00	3.0	No
28.5	580.9	Sand	110	23	Sand/Ash	3	3,135	1,886	1.03	1.00	1.00	1.00	0.95	1.00	22	22	64	0.000	0	22	0.823	0.084	0.24	1.54	1.016	0.38	3.0	No
33.5	575.9	Sand	110	6	-	1	3,685	2,124	0.97	1.00	1.00	1.00	1.00	1.00	6	6	32	0.000	0	6	0.784	0.084	0.09	1.54	0.999	0.14	1.7	No
38.5	570.9	Sand	110	48	-	1	4,235	2,362	0.92	1.00	1.00	1.00	1.00	1.00	48	44	89	0.000	0	44	0.745	0.082	2.00	1.54	0.967	2.00	3.0	No
43.5	565.9	Sand	110	29	-	7	4,785	2,600	0.88	1.00	1.00	1.00	1.00	1.00	29	25	68	0.212	0	26	0.707	0.080	0.31	1.54	0.965	0.46	3.0	No

Legend

Depth	Depth of sample (ft)
Soil Type	Dominate soil type: Gravel, Sand, Silt, or Clay
γ_{bulk}	Bulk unit weight of soil (pcf)
N_{Field}	Uncorrected field blowcounts
Fines Content	Approximate percent of fines (minus No. 200)
σ_v	Total vertical stress
σ'_v	Effective vertical stress
ER	Energy ratio (efficiency) for the hammer and sampling system (%)
Depth GW	Groundwater level below ground surface (ft)
C_N	Overburden correction for blowcounts
C_E	Energy Ratio correction (ER/60); relative to 60% theoretical energy of standard SPT sampling
C_B	Borehole diameter correction factor; relative to standard SPT sampling methods
C_R	Rod length correction factor; relative to standard SPT sampling methods
C_S	Sampler liner correction factor; relative to standard SPT sampling methods
C_{HS}	Sampler size and hammer energy correction factor; relative to standard SPT sampling methods

$$C_{HS} = \frac{W\ H\ (ODspt^3 - IDspt^3)}{Wspt\ Hspt\ (OD^3 - ID^3)}$$ (for cohesionless soil only)

$$C_{HS} = Rs(spt) / Rs$$

Rs and Rs(spt) Burmistor's input energy and sampler size correction factor

$$Rs = \frac{OD^3 - ID^3}{144\ W\ H}$$

Dia	Borehole diameter (mm)
Sampler Lined	Sampler lined or unlined
GSE	Ground surface elevation (feet)
$(N_1)_{60}$	SPT blowcount corrected for all factors (SSC Liao and RV Whitman 1986)
$(N_1)_{60\,CS}$	SPT blowcount corrected for fines content
Dr	Relative Density, % (max of 90% utilized in this analysis)
r_d	Stress reduction factor
CSR	Cyclic Stress Ratio (seismic demand)
$CRR_{7.5}$	Cyclic Resistance Ratio (soil capacity) normalized for a Mw = 7.5 earthquake
MSF	Magnitude scaling factor
FS_{LIQ}	Factor of Safety against liquefaction being triggered
Mw:	Earthquake moment magnitude
PGA & PGA_{ROCK}	Peak ground acceleration, horizontal, at ground surface and at the rock surface, respectively (g) $PGA = F_{PGA} * PGA_{ROCK}$
F_{PGA}	Site coefficient to account for amplification or attenuation of PGA_{ROCK} seismic motions in rock to the ground surface

Abbreviations

in. = inches
ft. = feet
mm = millimeters
dia = diameter
pcf= pounds per cubic foot
OD=outside diameter
ID=inside diameter
lb=pound
g= gravitational acceleration (32.174 ft/s^2)
HSA=Hollow Stem Auger
psf=pounds per square foot

Reference:

Soil Liquefaction During Earthquakes, I.M. Idriss, and R.W. Boulanger, Earthquake Engineering Research Institute Monograph No. MNO-12, 2008.

Golder Associates	DONE BY:	AK	PROJECT NAME: MCGS - PB2 - CCR Rule Compliance Plans
	DATE:	11/29/2017	TITLE: Corrected SPT data and liquefaction potential at BH-5
	CHECK:	SAM	
PROJECT NUMBER: 1787453.006	DATE:	11/29/2017	NiSource-Nipsco / MCGS / Michigan City, Indiana — Table 2

SPT BLOW COUNT CORRECTIONS AND LIQUEFACTION POTENTIAL

Input Parameters:

Hammer / Split Spoon

Spoon OD (in.) = 2.0
Spoon ID (in.) = 1.375
Hammer weight (lb) = 140
Hammer Drop (in.) = 30
Hammer Type = Automatic
Drilling Method = 4.25" HSA
Standard SPT Spoon OD (in.) = 2.0
Standard SPT Spoon ID (in.) = 1.375
Standard SPT Hammer weight (lb) = 140
Standard SPT Hammer Drop (in.) = 30

Energy Ratio (ER) = 60 %
Hole Dia (I.D. for HSA) = 83 (65-115, 150 or 200) mm
Sampler Lined (Y/N) = N/A

Earthquake Motion

Return Period = 2,475 years
Mw = 5.84
PGA = 0.094 g
PGA_{ROCK} = 0.051 g
F_{PGA} = 1.9

Groundwater

Depth GW = 18.0 ft

Surcharge: 0 psf
Height of Surcharge: 0 ft

Ground Surface Elevation

609.6 ft

Borehole = BH-6

Depth (ft)	Elevation (ft)	Soil Type	γ_{bulk} (pcf)	N_{Field} (blows/ft)	Sample Comments (Heave, etc.)	Fines Content (%)	σ_v (psf)	σ'_v (psf)	overburden factor C_N	non-standard sampler size & hammer C_{HS}	sample liner factor C_s	energy ratio factor C_E	rod length factor C_R	borehole dia. factor C_B	N_{60} (blows/ft)	$(N_1)_{60}$ (blows/ft)	Dr %	ΔN	ΔN (based on surcharge, k= 0.4	$(N_1)_{60\,CS}$ (blows/ft)	r_d	CSR	$CRR_{7.5}$	MSF	K_σ	CRR	FS_{LIQ}	Potentially Liquefiable?
1	608.6	Sand	110	9	Unsaturated	0.4	110	110	1.70	1.00	1.00	1.00	0.75	1.00	7	11	46	0.000	0	11	1.004	0.062	0.13	1.54	1.100	0.22	-	No
3.5	606.1	Sand	110	9	Unsaturated	0.4	385	385	1.70	1.00	1.00	1.00	0.75	1.00	7	11	46	0.000	0	11	0.992	0.061	0.13	1.54	1.100	0.22	-	No
6	603.6	Sand	110	2	Unsaturated	0.4	660	660	1.70	1.00	1.00	1.00	0.75	1.00	2	3	30	0.000	0	3	0.979	0.060	0.07	1.54	1.078	0.12	-	No
8.5	601.1	Sand	110	5	Unsaturated	0.4	935	935	1.46	1.00	1.00	1.00	0.75	1.00	4	5	32	0.000	0	5	0.965	0.059	0.09	1.54	1.063	0.15	-	No
11	598.6	Sand	110	34	Unsaturated	0.4	1,210	1,210	1.29	1.00	1.00	1.00	0.75	1.00	26	33	77	0.000	0	33	0.950	0.058	0.73	1.54	1.100	1.24	-	No
13.5	596.1	Sand	110	16	Unsaturated	0.4	1,485	1,485	1.16	1.00	1.00	1.00	0.85	1.00	14	16	53	0.000	0	16	0.934	0.057	0.16	1.54	1.040	0.26	-	No
18.5	591.1	Sand	110	3	Saturated	1.9	2,035	2,004	1.00	1.00	1.00	1.00	0.85	1.00	3	3	30	0.000	0	3	0.899	0.056	0.07	1.54	1.003	0.11	2.0	No
23.5	586.1	Sand	110	3	Saturated	1.9	2,585	2,242	0.94	1.00	1.00	1.00	0.95	1.00	3	3	30	0.000	0	3	0.862	0.061	0.07	1.54	0.996	0.11	1.9	No
28.5	581.1	Sand	110	27	Saturated	1.4	3,135	2,480	0.90	1.00	1.00	1.00	0.95	1.00	26	23	65	0.000	0	23	0.823	0.064	0.25	1.54	0.976	0.38	3.0	No
33.5	576.1	Sand	110	3	Saturated	11.8	3,685	2,718	0.86	1.00	1.00	1.00	1.00	1.00	3	3	30	2.000	0	5	0.784	0.065	0.08	1.54	0.981	0.13	1.9	No
38.5	571.1	Sand	110	6	Saturated	1.4	4,235	2,956	0.82	1.00	1.00	1.00	1.00	1.00	6	5	30	0.000	0	5	0.745	0.066	0.09	1.54	0.975	0.13	2.0	No

Legend

Depth	Depth of sample (ft)
Soil Type	Dominate soil type: Gravel, Sand, Silt, or Clay
γ_{bulk}	Bulk unit weight of soil (pcf)
N_{Field}	Uncorrected field blowcounts
Fines Content	Approximate percent of fines (minus No. 200)
σ_v	Total vertical stress
σ'_v	Effective vertical stress
ER	Energy ratio (efficiency) for the hammer and sampling system (%)
Depth GW	Groundwater level below ground surface (ft)
C_N	Overburden correction for blowcounts
C_E	Energy Ratio correction (ER/60); relative to 60% theoretical energy of standard SPT sampling
C_B	Borehole diameter correction factor; relative to standard SPT sampling methods
C_R	Rod length correction factor; relative to standard SPT sampling methods
C_S	Sampler liner correction factor; relative to standard SPT sampling methods
C_{HS}	Sampler size and hammer energy correction factor; relative to standard SPT sampling methods

$C_{HS} = \frac{W\ H\ (ODspt\text{^}3 - IDspt\text{^}3)}{Wspt\ Hspt\ (OD\text{^}3 - ID\text{^}3)}$ (for cohesionless soil only)

C_{HS} = Rs(spt) / Rs

Rs and Rs(spt) Burmistor's input energy and sampler size correction factor

$Rs = \frac{OD\text{^}3 - ID\text{^}3}{144\ W\ H}$

Dia	Borehole diameter (mm)
Sampler Lined	Sampler lined or unlined
GSE	Ground surface elevation (feet)
$(N_1)_{60}$	SPT blowcount corrected for all factors (SSC Liao and RV Whitman 1986)
$(N_1)_{60\,CS}$	SPT blowcount corrected for fines content
Dr	Relative Density, % (max of 90% utilized in this analysis)
r_d	Stress reduction factor
CSR	Cyclic Stress Ratio (seismic demand)
$CRR_{7.5}$	Cyclic Resistance Ratio (soil capacity) normalized for a Mw = 7.5 earthquake
MSF	Magnitude scaling factor
FS_{LIQ}	Factor of Safety against liquefaction being triggered
Mw:	Earthquake moment magnitude
PGA & PGA_{ROCK}	Peak ground acceleration, horizontal, at ground surface and at the rock surface, respectively (g) $PGA = F_{PGA} * PGA_{ROCK}$
F_{PGA}	Site coefficient to account for amplification or attenuation of PGA_{ROCK} seismic motions in rock to the ground surface

Abbreviations

in. = inches
ft. = feet
mm = millimeters
dia = diameter
pcf= pounds per cubic foot
OD=outside diameter
ID=inside diameter
lb=pound
g= gravitational acceleration (32.174 ft/s^2)
HSA=Hollow Stem Auger
psf=pounds per square foot

Reference:
Soil Liquefaction During Earthquakes, I.M. Idriss, and R.W. Boulanger, Earthquake Engineering Research Institute Monograph No. MNO-12, 2008.

Golder Associates	DONE BY:	AK	PROJECT NAME: MCGS - PB2 - CCR Rule Compliance Plans
	DATE:	11/29/2017	TITLE: Corrected SPT data and liquefaction potential at BH-6
	CHECK:	SAM	
PROJECT NUMBER: 1787453.006	DATE:	11/29/2017	NiSource-Nipsco / MCGS / Michigan City, Indiana — Table 3

SPT BLOW COUNT CORRECTIONS AND LIQUEFACTION POTENTIAL

Input Parameters:

Hammer / Split Spoon

Spoon OD (in.) = 2.0
Spoon ID (in.) = 1.375
Hammer weight (lb) = 140
Hammer Drop (in.) = 30
Hammer Type = Automatic
Drilling Method = 4.25" HSA
Standard SPT Spoon OD (in.) = 2.0
Standard SPT Spoon ID (in.) = 1.375
Standard SPT Hammer weight (lb) = 140
Standard SPT Hammer Drop (in.) = 30

Energy Ratio (ER) = 60 %
Hole Dia (I.D. for HSA) = 83 (65-115, 150 or 200) mm
Sampler Lined (Y/N) = N/A

Earthquake Motion

Return Period = 2,475 years
Mw = 5.84
PGA = 0.094 g
PGA_{ROCK} = 0.051 g
F_{PGA} = 1.9

Groundwater

Depth GW = 11.5 ft

Ground Surface Elevation

609.4 ft

Surcharge: 0 psf
Height of Surcharge: 0 ft

Borehole = BH-7

Depth (ft)	Elevation (ft)	Soil Type	γ_{bulk} (pcf)	N_{Field} (blows/ft)	Sample Comments (Heave, etc.)	Fines Content (%)	σ_v (psf)	σ'_v (psf)	overburden factor C_N	non-standard sampler size & hammer C_{HS}	sample liner factor C_s	energy ratio factor C_E	rod length factor C_R	borehole dia. factor C_B	N_{60} (blows/ft)	$(N_1)_{60}$ (blows/ft)	Dr %	ΔN	ΔN (based on surcharge, k= 0.4	$(N_1)_{60\ CS}$ (blows/ft)	r_d	CSR	$CRR_{7.5}$	MSF	K_σ	CRR	FS_{LIQ}	Potentially Liquefiable?
1	608.4	Sand	110	11	Unsaturated	10.2	110	110	1.70	1.00	1.00	1.00	0.75	1.00	8	14	53	1.218	0	15	1.004	0.062	0.16	1.54	1.100	0.27	-	No
3.5	605.9	Sand	110	10	Unsaturated	10.2	385	385	1.70	1.00	1.00	1.00	0.75	1.00	8	13	50	1.218	0	14	0.992	0.061	0.15	1.54	1.100	0.25	-	No
6	603.4	Sand	110	10	Unsaturated	10.2	660	660	1.70	1.00	1.00	1.00	0.75	1.00	8	13	50	1.218	0	14	0.979	0.060	0.15	1.54	1.100	0.25	-	No
8.5	600.9	Sand	110	11	Unsaturated	13.8	935	935	1.46	1.00	1.00	1.00	0.75	1.00	8	12	52	2.829	0	15	0.965	0.059	0.16	1.54	1.090	0.26	-	No
11	598.4	Sand	110	50	Unsaturated	13.8	1,210	1,210	1.29	1.00	1.00	1.00	0.75	1.00	38	48	90	2.829	0	51	0.950	0.058	2.00	1.54	1.100	2.00	-	No
13.5	595.9	Sand	110	31	Saturated	13.8	1,485	1,360	1.21	1.00	1.00	1.00	0.85	1.00	26	32	79	2.829	0	35	0.934	0.063	1.06	1.54	1.100	1.80	3.0	No
18.5	590.9	Sand	110	18	Saturated	13.8	2,035	1,598	1.12	1.00	1.00	1.00	0.85	1.00	15	17	60	2.829	0	20	0.899	0.070	0.21	1.54	1.037	0.33	3.0	No
23.5	585.9	Sand	110	14	Saturated	20.6	2,585	1,836	1.04	1.00	1.00	1.00	0.95	1.00	13	14	58	4.574	0	18	0.862	0.075	0.19	1.54	1.017	0.30	3.0	No
28.5	580.9	Sand	110	44	Saturated	13.8	3,135	2,074	0.98	1.00	1.00	1.00	0.95	1.00	42	41	89	2.829	0	44	0.823	0.076	2.00	1.54	1.005	2.00	3.0	No
33.5	575.9	Sand	110	19	Saturated	6.8	3,685	2,312	0.93	1.00	1.00	1.00	1.00	1.00	19	18	57	0.101	0	18	0.784	0.077	0.18	1.54	0.989	0.28	3.0	No
38.5	570.9	Sand	110	17	Saturated	6.8	4,235	2,550	0.89	1.00	1.00	1.00	1.00	1.00	17	15	52	0.101	0	15	0.745	0.076	0.16	1.54	0.979	0.24	3.0	No

Legend

Depth	Depth of sample (ft)
Soil Type	Dominate soil type: Gravel, Sand, Silt, or Clay
γ_{bulk}	Bulk unit weight of soil (pcf)
N_{Field}	Uncorrected field blowcounts
Fines Content	Approximate percent of fines (minus No. 200)
σ_v	Total vertical stress
σ'_v	Effective vertical stress
ER	Energy ratio (efficiency) for the hammer and sampling system (%)
Depth GW	Groundwater level below ground surface (ft)
C_N	Overburden correction for blowcounts
C_E	Energy Ratio correction (ER/60); relative to 60% theoretical energy of standard SPT sampling
C_B	Borehole diameter correction factor; relative to standard SPT sampling methods
C_R	Rod length correction factor; relative to standard SPT sampling methods
C_S	Sampler liner correction factor; relative to standard SPT sampling methods
C_{HS}	Sampler size and hammer energy correction factor; relative to standard SPT sampling methods

$$C_{HS} = \frac{W\ H\ (ODspt^3 - IDspt^3)}{Wspt\ Hspt\ (OD^3 - ID^3)}$$ (for cohesionless soil only)

C_{HS} = Rs(spt) / Rs

Rs and Rs(spt) Burmistor's input energy and sampler size correction factor

$$Rs = \frac{OD^3 - ID^3}{144\ W\ H}$$

Dia	Borehole diameter (mm)
Sampler Lined	Sampler lined or unlined
GSE	Ground surface elevation (feet)
$(N_1)_{60}$	SPT blowcount corrected for all factors (SSC Liao and RV Whitman 1986)
$(N_1)_{60\ CS}$	SPT blowcount corrected for fines content
Dr	Relative Density, % (max of 90% utilized in this analysis)
r_d	Stress reduction factor
CSR	Cyclic Stress Ratio (seismic demand)
$CRR_{7.5}$	Cyclic Resistance Ratio (soil capacity) normalized for a Mw = 7.5 earthquake
MSF	Magnitude scaling factor
FS_{LIQ}	Factor of Safety against liquefaction being triggered
Mw:	Earthquake moment magnitude
PGA & PGA_{ROCK}	Peak ground acceleration, horizontal, at ground surface and at the rock surface, respectively (g) PGA = F_{PGA} * PGA_{ROCK}
F_{PGA}	Site coefficient to account for amplification or attenuation of PGA_{ROCK} seismic motions in rock to the ground surface

Abbreviations

in. = inches
ft. = feet
mm = millimeters
dia = diameter
pcf= pounds per cubic foot
OD=outside diameter
ID=inside diameter
lb=pound
g= gravitational acceleration (32.174 ft/s^2)
HSA=Hollow Stem Auger
psf=pounds per square foot

Reference:

Soil Liquefaction During Earthquakes, I.M. Idriss, and R.W. Boulanger, Earthquake Engineering Research Institute Monograph No. MNO-12, 2008.

Golder Associates	DONE BY:	AK	PROJECT NAME: MCGS - PB2 - CCR Rule Compliance Plans
	DATE:	11/29/2017	TITLE: Corrected SPT data and liquefaction potential at BH-7
	CHECK:	SAM	
PROJECT NUMBER: 1787453.006	DATE:	11/29/2017	NiSource-Nipsco / MCGS / Michigan City, Indiana — Table 4

ATTACHMENT 5

Table 3 Correction factors for SPT N values.

Factor	**Description**
Energy ratio	Energy measurements are required to determine the delivered energy ratios or to calibrate the specific equipment being used. The correction factor is then computed as $$C_E = \frac{ER_m}{60}$$ where ER_m is the measured energy ratio as a percentage of the theoretical maximum. Empirical estimates of C_E (for rod lengths of 10 m or more) involve considerable uncertainty, as reflected by the following ranges: Doughnut hammer $C_E = 0.5\text{–}1.0$ Safety hammer $C_E = 0.7\text{–}1.2$ Automatic triphammer $C_E = 0.8\text{–}1.3$ (Seed et al. 1984, Skempton 1986, NCEER 1997)
Borehole diameter	Borehole diameter of 65–115 mm $C_B = 1.0$ Borehole diameter of 150 mm $C_B = 1.05$ Borehole diameter of 200 mm $C_B = 1.15$ (Skempton 1986)
Rod length	Where the ER_m is based on rod lengths of 10 m or more, the ER delivered with shorter rod lengths may be smaller. Recommended values from Youd et al. (2001) are as follows: Rod length < 3 m $C_R = 0.75$ Rod length 3–4 m $C_R = 0.80$ Rod length 4–6 m $C_R = 0.85$ Rod length 6–10 m $C_R = 0.95$ Rod length 10–30 m $C_R = 1.00$
Sampler	Standard split spoon without room for liners (the inside diameter is a constant $1^3/_8$ in.), $C_S = 1.0$. Split-spoon sampler with room for liners but with the liners absent (this increases the inside diameter to $1^1/_2$ in. behind the driving shoe): $C_S = 1.1 \quad \text{for} \quad (N_1)_{60} \leq 10$ $C_S = 1 + \frac{(N_1)_{60}}{100} \quad \text{for} \quad 10 \leq (N_1)_{60} \leq 30$ $C_S = 1.3 \quad \text{for} \quad (N_1)_{60} \geq 30$ (from Seed et al. 1984, equation by Seed et al. 2001)

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