SAFETY FACTOR ASSESSMENT REPORT

PONDS 1 & 2, JR WHITING PLANT
ERIE, MICHIGAN

OCTOBER 14, 2016

PREPARED FOR:
CONSUMERS ENERGY COMPANY

Mannik
Smith
GROUP

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CERTIFICATION

Professional Engineer Certification Statement [40 CFR 257.73(e)(1)(i)-(iv)]

I hereby certify that, having reviewed the attached documentation and being familiar with the provisions of Title 40 of the Code of Federal Regulations 40 CFR Part 257.73(e)(1)(i)-(iv), I attest that this Safety Factor Assessment Report is accurate and 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(e)(1)(i)-(iv).

The Mannik Smith Group, Inc.

Ibraheem Shunnar
Signature

STATE OF MICHIGAN
IBRAHEEM
ALSHUNNAR
ENGINEER
No.
39106
LICENSED PROFESSIONAL ENGINEER

10/14/2016
Date of Report Certification

Ibraheem Shunnar, PE
Name

6201039106
Professional Engineer Certification Number

1.0 INTRODUCTION

On April 17, 2015, the United States Environmental Protection Agency (EPA) issued the Coal Combustion Residual (CCR) Resource Conservation and Recovery Act (RCRA) Rule (40 CFR 257 Subpart D) ("CCR RCRA Rule") to regulate the beneficial use and disposal of CCR materials generated at coal-fired electrical power generating complexes. In accordance with the CCR RCRA Rule, any CCR surface impoundment or CCR landfill that was actively receiving CCRs on the effective date of the CCR RCRA Rule (October 19, 2015) was deemed to be an "Existing CCR Unit" on that date and subject to self-implementing compliance standards and schedules. Consumers Energy identified two CCR surface impoundments at the JR Whiting Generating Facility (JR Whiting) located in Erie, Michigan:

- Ponds 1 & 2 (Existing CCR surface impoundment)
- Pond 6 (Inactive CCR surface impoundment)

The CCR RCRA Rule requires that existing CCR surface impoundments meeting the requirements of Section 257.73(b) conduct a safety factor assessment in accordance with Section 257.73(e). This report provides the safety factor assessment for Ponds 1 & 2.

2.0 SITE DESCRIPTION AND BACKGROUND

JR Whiting is a coal-fired power generation facility located in Erie, Michigan as presented on Figure 1 – Site Location Map. JR Whiting formerly operated coal-burning baseload units but ceased electrical generation on April 15, 2016. Ponds 1 & 2 as presented in Figure 2 – General Site Plan, served two primary functions:

- Received outflow of bottom ash for primary detention and settlement
- Received intermittent sluiced fly ash and low-volume wastewater from the generating facility for detention and settlement.

The two ponds comprising the CCR surface impoundment are no longer receiving CCRs from an active power generating plant but are managing stormwater run-on (non-CCR wastewater) per the Site National Pollutant Discharge Elimination System (NPDES) Permit. The pond system is underlain by clay soils and contained by a perimeter dike which has, generally, a 20-foot wide crest and a crest elevation of about 590.1 (NAVD88). The perimeter dikes are designed and constructed of native materials and coal ash utilized as fill. The crest of the dike structure is graded to allow flow of stormwater from the crest into the ponds. The elevation of water in Ponds 1 & 2 is about 584 ft. (NAVD88).

Hydraulically, Ponds 1 & 2are interconnected by a subsurface pipe. Any discharge from Ponds 1 & 2 is combined in Pond 1 and routed through permitted National Pollutant Discharge Elimination System (NPDES) Outfall 001B into the forebay. This discharge pipe was grouted on May 24, 2016.

Based on previous investigations including borings completed along the perimeter dike and within the ponds, the site is underlain by layers of soft to medium clay underlain by layers of stiff to hard clay.

A hazard potential classification was conducted for Ponds 1 & 2 pursuant to Section 257.73(a) (2), which resulted in a significant hazard classification. As a result of the hazard classification potential, the 1000-year flood elevation was used in the models to prepare this report.

3.0 PREVIOUS SAFETY FACTOR ASSESSMENTS

Several investigations, assessments and inspections were completed to assess the structural stability of Ponds 1 & 2. A list of documents related to Ponds 1 & 2 that were reviewed for the structural stability assessment is provided in Table 1. Based on our review, there is no evidence of structural deficiencies at Ponds 1 & 2. A brief summary of the previous assessments is provided below.

In 2009, a dike inspection and a potential mode failure analysis (PMFA) were completed for Ponds 1 & 2. The inspection and the PMFA provided operational and maintenance recommendations and recommended the completion of additional stability analysis. As a follow up to these recommendations, CEC developed a Surveillance Monitoring Program (SMP) and contracted NTH Consultants to complete additional stability evaluations. The results of these evaluations indicated that the existing slopes have adequate factor of safety. In 2012, 2014, and 2015 dike assessments were completed by AECOM, Barr Engineering, and Golder Associates, respectively. These assessments provided additional maintenance and operational recommendations regarding erosion, vegetation, animal burrows and the potential for seepage along the west slope of Pond 2, among others. Following these assessments, CEC updated the SMP. None of these studies identified any structural deficiencies that will require immediate action or repair.

TABLE 1
SUMMARY OF BACKGROUND DOCUMENT REVIEW

No	DOCUMENT	DATE	AUTHOR
1	J. R. Whiting Ponds 1 and 2 - Annual RCRA CCR Surface Impoundment Inspection Report	10/2016	The Mannik & Smith Group, Inc.
2	J. R. Whiting Ponds 1 and 2 - Annual RCRA CCR Surface Impoundment Inspection Report	01/2016	Golder Associates, Inc.
3	Fossil Fuel Generation, Solid Waste Disposal Area - Surveillance Monitoring Programs (SMPs)	12/2010, Revised 2015	Consumers Energy Company
4	J.R. Whiting Ash Disposal Area Triennial Ash Dike Assessment Report – Spring 2014	December 2014	Barr Engineering Company
5	J.R. Whiting Ash Disposal Area, 2012 Ash Dike Risk Assessment Final Inspection Report	July 2012	AECOM Technical Services, Inc.
6	Dam Safety Assessment of CCW Impoundments J.R. Whiting Plant	June 2011	USEPA, O'Brien and Gere Engineers, Inc.
7	Slope Stability Analysis, Ponds 1,2 and 6, J.R. Whiting Ash Disposal Facility	11/2011	NTH Consultants
8	J.R. Whiting Generating Facility Ash Dike Risk Assessment, Inspection Report	December 2009	AECOM Technical Services, Inc.
9	J.R. Whiting Generating Facility Ash Dike Risk Assessment, Potential Failure Mode Analysis (PFMA) Report	December 2009	AECOM Technical Services, Inc.

4.0 SAFETY FACTOR ASSESSMENT

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 loading scenarios:

1. Maximum Pool Storage - Section 257.73(e)(1)(i) – Defined as the long-term, maximum storage pool (or operating) elevation. For this case, we assumed the worst case scenario with the water elevation at the top of the perimeter dike elevation. Static factor of safety for this case must equal or exceed 1.50.
2. 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. Static factor of safety for this case must equal or exceed 1.40.
3. Seismic Loading Conditions - Section 257.73(e)(1)(iii) – Seismic factor of safety must equal or exceed 1.00.
4. Liquefaction Potential - Section 257.73(e)(1)(iv) – Portion of the exterior dikes are built using fly ash and bottom ash, which are susceptible to liquefaction. Factor of safety must equal or exceed 1.20.

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.

4.1 Slope Stability Analysis

Slope stability analyses were performed to evaluate the slope factor of safety for each of the maximum pool storage, maximum pool surcharge, and seismic loading scenarios. In the Preamble to Sections 257 and 261 of the CCR RCRA Rule *General Safety Factor Assessment Considerations* [VI (E)(3)(b)(ii)(a)], limit equilibrium methods are identified as conventional analysis procedures for calculating the factor of safety and specific common methods are identified, including the Modified Bishop and Spencer method of slices, which was used for this stability analysis.

4.2 Cross-Sections Analyzed

A critical cross section developed during the investigation completed by NTH Consultants and reported in "*Slope Stability Analyses Coal Ash Storage Ponds 1, 2, & 6 J.R. Whiting Ash Disposal Facility*", dated November 23, 2011 was used for this evaluation. The cross section was modified to reflect new information about the depth of ash in ponds 1 & 2 collected during the investigation completed by Golder in 2015. The locations of the critical section (A-A) is shown on Figure 2 – Location Map.

In general, slopes are relatively shallow and relatively flat. The downstream slope of the dike is less than 20 feet deep and slopes at about 2.5 horizontal to one vertical.

4.3 Geotechnical Material Properties

Geotechnical material properties developed by NTH during their 2011 investigation were used. Based on review and knowledge of site conditions, there was concurrence that these parameters are appropriate for this project. A summary of the geotechnical material properties is shown in the table below.

TABLE 4.1
SUMMARY OF GEOTECHNICAL MATERIAL PROPERTIES

Layer Description	Unit Weight (pcf)	Long Term		Short Term	
		Cohesion (psf)	Friction Angle (Degree)	Cohesion (psf)	Friction Angle (Degree)
V. Loose to Loose Fly Ash	103	0	35	0	35
Sluiced Fly Ash	80	0	28	0	28
Soft to Medium Clay	121	0	28	400	0
Stiff to Very Stiff Clay	127	0	30	2400	0
Stiff Clay	140	0	33	1500	0
Very Stiff Clay	137	0	31	3500	0

4.4 Pond Elevations

The flood control system evaluation indicated that the ponds will not become filled with water during a 1,000 year storm and through the planned closure in 2018. Therefore, for the purpose of the evaluation, to be conservative, it was assumed that the water level in the ponds will reach its crest (i.e. Elevation 590). This was used for both the maximum pool storage and the maximum pool surcharge. The phreatic surface for the maximum surcharge scenario was then estimated using steady state seepage, assuming the pond elevation remained elevated but the exterior water elevation in Lake Erie receded back to its ordinary elevation.

4.5 Vehicle Loading

The crest of the embankment is periodically used by maintenance vehicles as access roads around Ponds 1 & 2; therefore, a vehicle load was applied to the critical cross-section to model the loading effects of vehicle traffic. The vehicle load was modeled as a line load of 300 pounds per square foot (psf) extending across the top of the dike. This surcharge load corresponds to truck traffic based on the American Association of State Highway and Transportation Officials (AASHTO) recommended loading for truck loads acting parallel to the embankment wall equivalent to approximately two times the unit weight of embankment fill (AASHTO 2012).

4.6 Seismic Loading Conditions

Factors of safety for stability under seismic conditions were calculated using the pseudo-static method. The peak ground acceleration (PGA) based on the 2014 United States Geological Survey (USGS) seismic hazard maps with a two percent probability of exceedance in 50 years (2,475-year return period) is 0.05g.

4.7 Stability Analysis Results

Slope stability analyses were performed for long-term static conditions under maximum storage and maximum surcharge scenarios as well as seismic. The results of the slope stability analyses cases are presented in Table 4.2 and critical failure surface result outputs are contained in Appendix A. The results indicate that the pond exterior slopes meet or exceed the required safety factors under considered loading scenarios.

TABLE 4.2
SUMMARY OF SAFETY FACTOR

Case	Condition	Required Safety Factor	Calculated Safety Factor
Maximum Pool with surcharge	Static	1.50	1.76
	Seismic	1.00	1.54
Maximum Pool Surcharge	Static	1.40	1.76
	Seismic	1.00	1.54
Liquefaction		1.20	1.31

As shown, the slip surfaces with the minimum factor of safety do not cross the actual impoundment. This is due to the shallow downstream slopes.

4.8 Liquefaction Potential Assessment

Embankment and foundation soils were screened for seismically-induced liquefaction susceptibility using methods recommended by the National Center for Earthquake Research (NCEER) and presented in a paper by Youd and Idriss (2001). The calculated factor of safety against seismically-triggered liquefaction is greater than 1.2 throughout the depth of CCR for the considered earthquake loading as shown in the calculations included in Appendix B.

5.0 CONCLUSIONS

Based on the results of the safety factor assessment, the calculated factors of safety through the critical cross section in the Ponds 1&2 surface impoundment meet or exceed the minimum values listed in Section 257.73(e)(1)(i)-(iv).

6.0 REFERENCE

Youd, T. L. and Idriss, I. M. (2001), "*Liquefaction Resistance of Soils: Summary From the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils*." Journal of Geotechnical and Geoenvironmental Engineering, April 2001, pp 297-313.

FIGURES

MONROE COUNTY

NOT TO SCALE

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TECHNICAL SKILL.
CREATIVE SPIRIT.

FIGURE 1

SITE LOCATION MAP

PONDS 1 & 2
JR Whiting Generating Facility
Erie, Monroe County, Michigan

DATE	DRAWN BY	DESIGNED BY	PROJECT NO.
9/30/2016	RAR	ISS	C1790015

9/30/2016 2:36:49 PM
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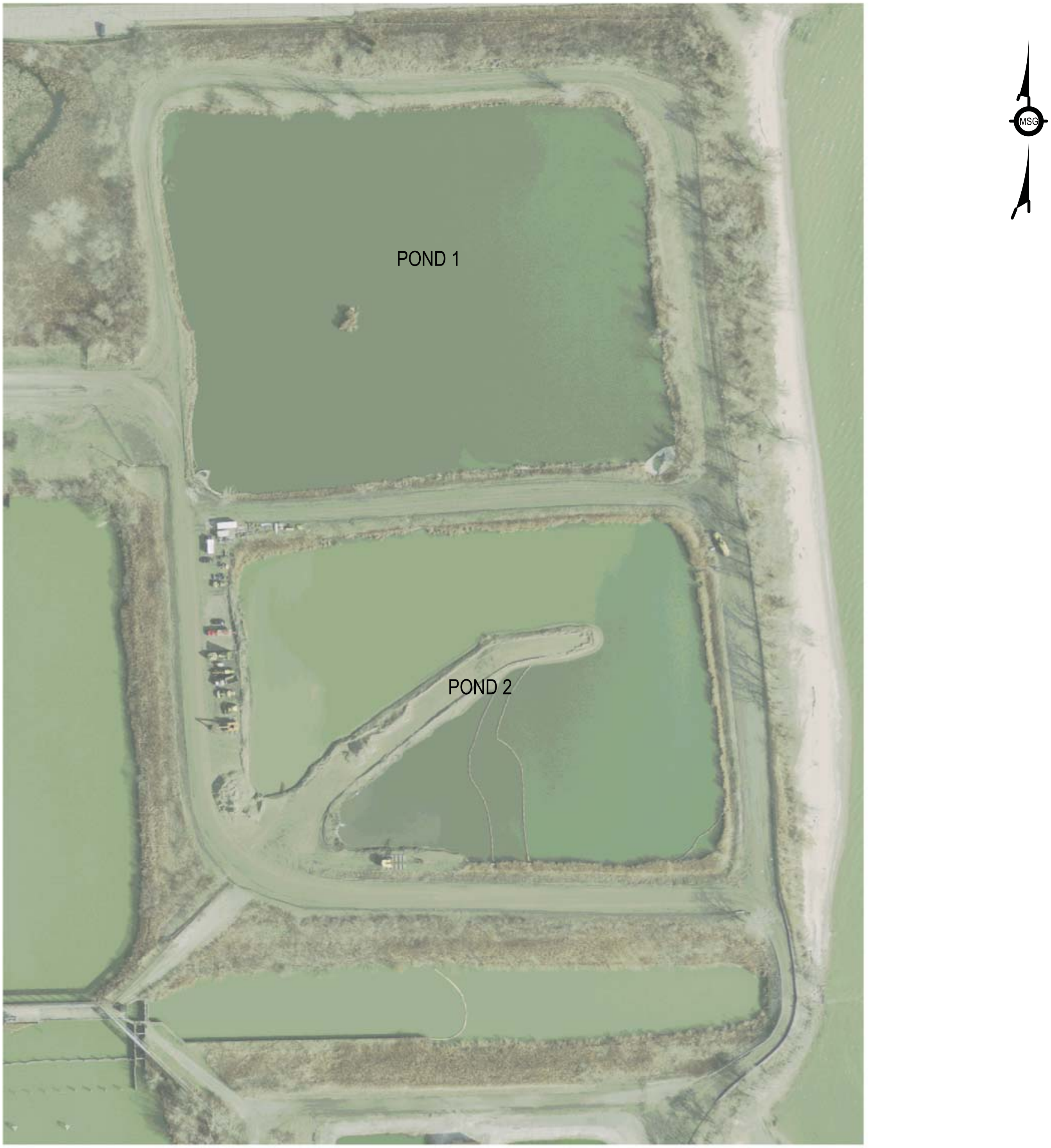

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FIGURE 2

GENERAL SITE PLAN

PONDS 1 & 2
JR Whiting Generating Facility
Erie, Monroe County, Michigan

DATE	DRAWN BY	DESIGNED BY	PROJECT NO.
9/30/2016	RAR	ISS	C1790015

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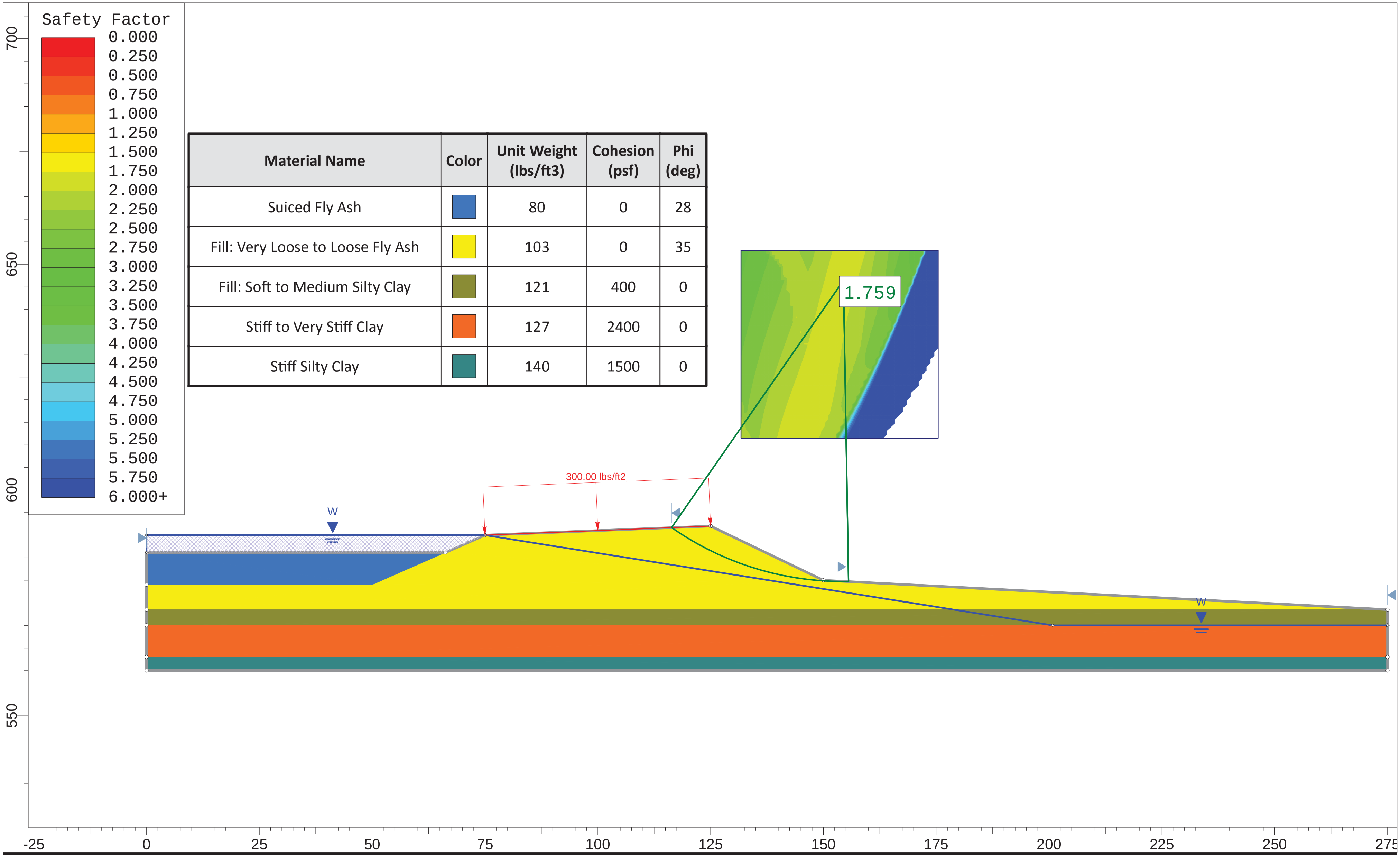
APPENDIX A
RESULTS OF SLOPE STABILITY EVALUATIONS



Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Suiced Fly Ash	🟦	80	0	28
Fill: Very Loose to Loose Fly Ash	🟨	103	0	35
Fill: Soft to Medium Silty Clay	🟫	121	400	0
Stiff to Very Stiff Clay	🟧	127	2400	0
Stiff Silty Clay	🟩	140	1500	0

Mannik Smith GROUP

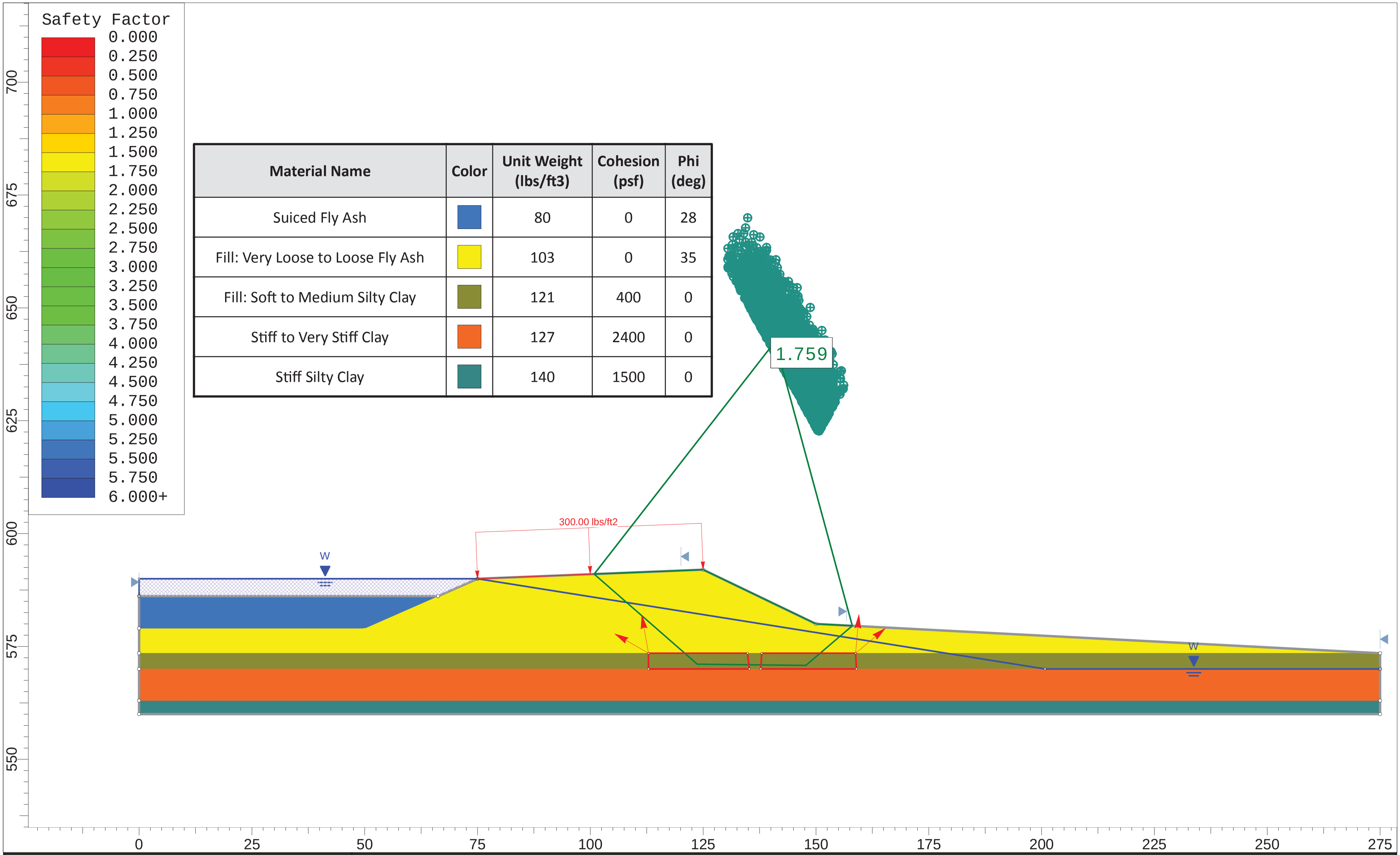
Project Case 1 - Figure A - Short Term - Circle

Analysis Description

Drawn By AD *Scale* 1:353 *Company* The Mannik & Smith Group, Inc.

Date 9/19/2016, 11:29:18 AM *File Name* C1790017- Case 1 - Figure A - Short Term - Circle.slim

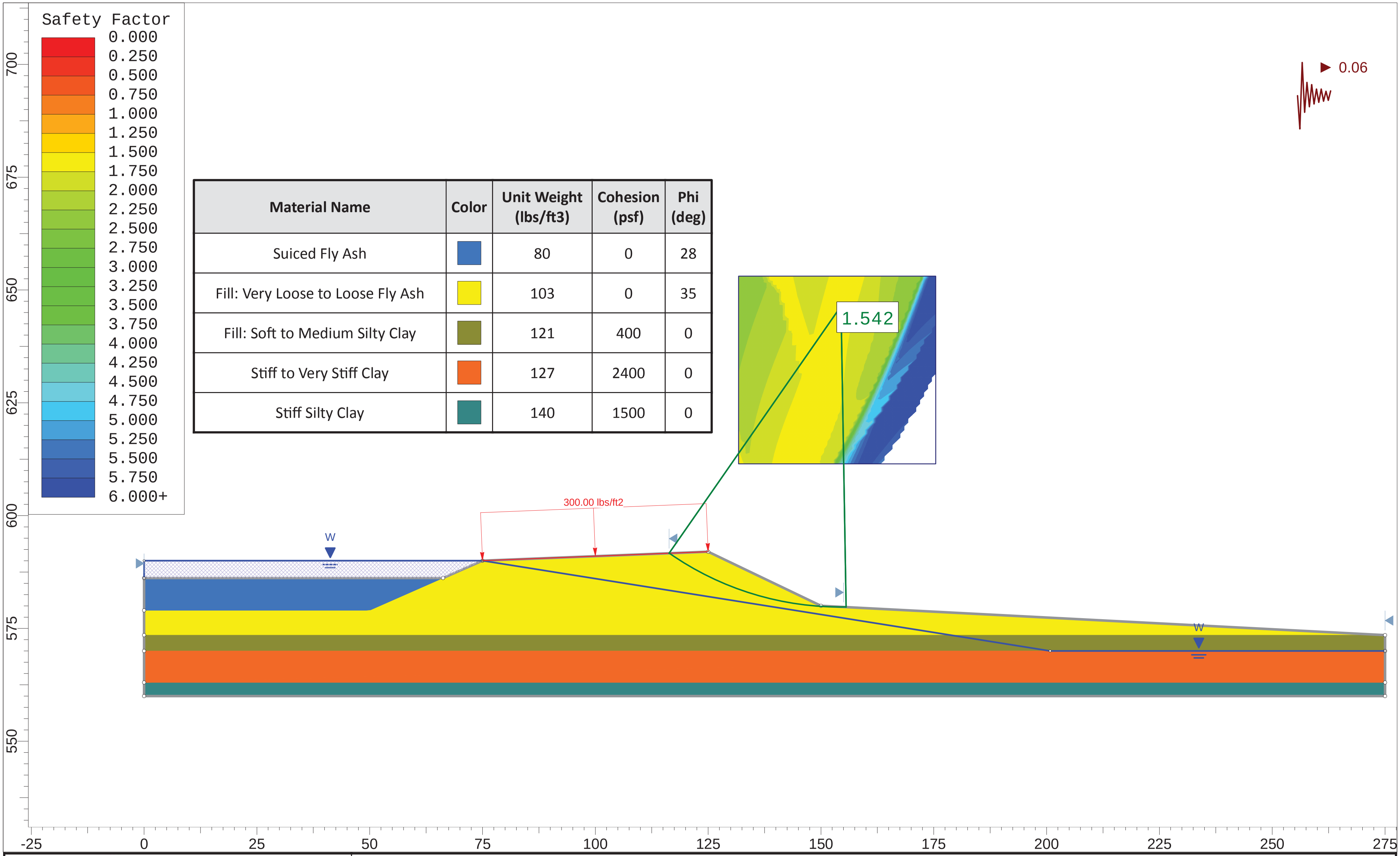
SLIDEINTERPRET 7.010



Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Suiced Fly Ash	🟦	80	0	28
Fill: Very Loose to Loose Fly Ash	🟨	103	0	35
Fill: Soft to Medium Silty Clay	🟫	121	400	0
Stiff to Very Stiff Clay	🟧	127	2400	0
Stiff Silty Clay	🟩	140	1500	0

Project	Case 1 - Figure B - Short Term - Block		
Analysis Description			
Drawn By AD	*Scale* 1:353	*Company*	The Mannik & Smith Group, Inc.
Date	9/19/2016, 11:29:18 AM	*File Name*	C1790017- Case 1 - Figure B - Short Term - Block.slim

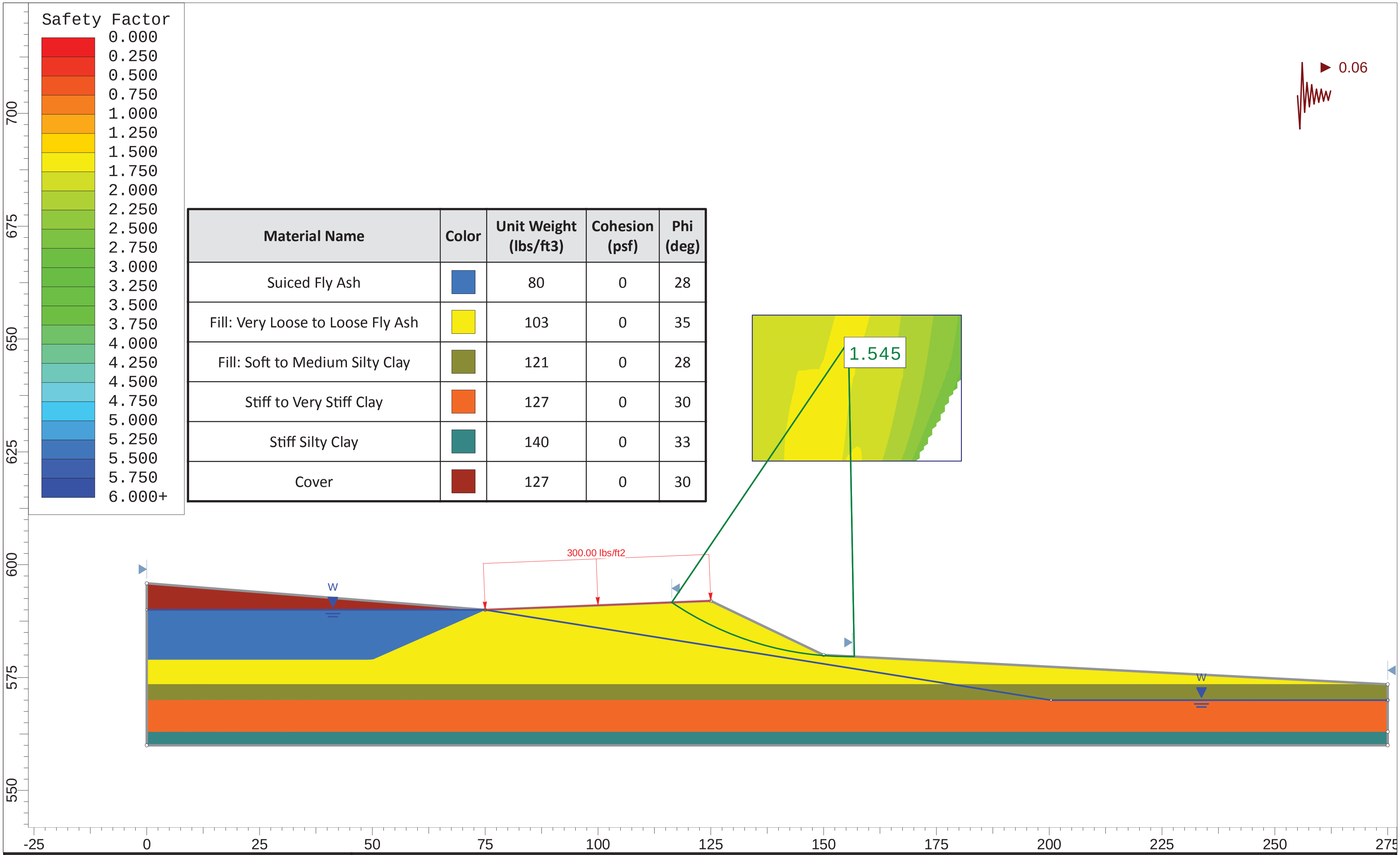
SLIDEINTERPRET 7.010



Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Suiced Fly Ash	🟦	80	0	28
Fill: Very Loose to Loose Fly Ash	🟨	103	0	35
Fill: Soft to Medium Silty Clay	🟫	121	400	0
Stiff to Very Stiff Clay	🟧	127	2400	0
Stiff Silty Clay	🟩	140	1500	0

Project	Case 1 - Figure C - Short term - Seismic		
Analysis Description			
Drawn By	AD	*Scale* 1:353	*Company* The Mannik & Smith Group, Inc.
Date	9/19/2016, 11:29:18 AM		*File Name* C1790017-Case 1 - Figure C - Short Term - Seismic.slim

SLIDEINTERPRET 7.010



Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Suiced Fly Ash	🟦	80	0	28
Fill: Very Loose to Loose Fly Ash	🟨	103	0	35
Fill: Soft to Medium Silty Clay	🟫	121	0	28
Stiff to Very Stiff Clay	🟧	127	0	30
Stiff Silty Clay	🟩	140	0	33
Cover	🟥	127	0	30

SLIDEINTERPRET 7.010

Project	Case 2-Figure C -Long Term-Proposed-Seismic		
Analysis Description			
Drawn By AD	*Scale* 1:353	*Company* The Mannik & Smith Group, Inc.	
Date 9/19/2016, 11:29:18 AM		*File Name* C1790017- Case 2 - Figure C - Long Term - Seismic.slim	

APPENDIX B
LIQUEFACTION POTENTIAL CALCULATIONS

1.0 LIQUEFACTION ANALYSIS

The CCR RCRA Rule requires that existing CCR surface impoundments must have a calculated safety factor for dikes constructed of soils that have a susceptibility to liquefaction equal to or greater than 1.20. JR Whiting Ponds 1&2 was evaluated and retained for further consideration due to the presence of a saturated fly ash layer within the perimeter dike strata. This layer was observed in lithology descriptions presented in "*Slope Stability Analyses Coal Ash Storage Ponds 1, 2, & 6 J.R. Whiting Ash Disposal Facility*", dated November 23, 2011.

Liquefaction is defined as the sudden loss of soil strength resulting from the increase in pore pressure. The increased pore water pressure is due to volumetric strains caused by cyclic stresses commonly associated with seismic events. The CCR RCRA Rule defines Liquefaction factor of safety as, "the factor of safety (safety factor) determined using analysis under liquefaction conditions (40 CFR 257.53). The preamble of the CCR RCRA Rule notes that liquefaction is a phenomenon which typically occurs in loose, saturated or partially-saturated soils in which the effective stress of the soils reduces to zero, corresponding to a total loss of shear strength of the soil. Additionally, it notes that the most common occurrence of liquefaction is in loose soils, typically sand.

The liquefaction analysis evaluates the in-situ soils susceptibility to liquefaction during these events. This evaluation has been completed in accordance with the referenced liquefaction analysis : "*Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils*" prepared by T.L. Youd and I. M. Idriss from the CCR RCRA Preamble pp. 21317 from FR Vol. 80, No. 74, Friday April 17, 2015.

2.0 LIQUEFACTION SCREENING

Liquefaction screening criteria consists of evaluating the following aspects of the soil material:

- Geologic age and origin,
- Fines content and plasticity,
- Saturation, depth below ground surface, and
- Soil penetration resistance.

If three or more of the above criteria indicate that liquefaction is not likely, then the potential for liquefaction can be dismissed. Otherwise, a more rigorous analysis of the liquefaction potential is required. The following are evaluations of the screening criteria:

1. *Geologic age and origin.* If a soil layer is a fluvial, lacustrine or aeolian deposit of Holocene age, a greater potential for liquefaction exists than for till, residual deposits, or older deposits.

The underlying native soil at JR Whiting Ponds 1&2 is homogenetic in nature and was deposited recently –

before the Holocene age. Thus, this criterion indicates a greater potential for soil liquefaction.

2. *Fines content and plasticity.* Liquefaction potential in a soil layer increases with decreasing fines content and plasticity of the soil. Cohesionless soils having less than 15 percent (by weight) finer than 0.005 mm, a liquid limit less than 35 percent, and in situ water content greater than 0.9 times the liquid limit may be susceptible to liquefaction (Seed and Idriss, 1982).

   The soil testing results for the fly ash strata in the perimeter dike were reviewed. The material exhibited much greater than 15% fines and an in-situ water of approximately 0.9 times the liquid limit. Conservatively, it is estimated that a discrete strata of fly ash would be characterized a cohesionless material without any benefit from potential cohesion from clay sources also noted within the construction of the perimeter dike system. This criterion indicates liquefaction of this material is likely.

3. *Saturation*. Although low water content soils have been reported to liquefy, at least 80 to 85 percent saturation is utilized for this screening level evaluation. Since the observed fly ash strata is at or near saturation condition, this criterion indicates a greater potential for soil liquefaction.

4. *Depth below ground surface*. Liquefaction is generally not likely to occur more than 50 feet below the ground surface for the purposes of this screening level evaluation. Since the observed fly ash strata is observed within the 50-ft threshold of ground surface, this criterion indicates a greater potential for soil liquefaction.

5. *Soil Penetration Distance*. Seed et al, 1985, states that soil layers with a normalized Standard Penetration Test (SPT) blow count less than 22 have been known to liquefy. Marcuson et al, 1990, suggests an SPT value of less than 30 as the threshold to use for suspecting liquefaction potential. Liquefaction has also been shown to occur if the normalized Cone Penetration Test (CPT) cone resistance is less than 157 tsf. (SIbata and Taparaksa, 1988).

   The observed, SPT blow counts (N values) are less than 30 for the fly ash layer in the perimeter dike, averaging 3.7 for the unit. This criterion indicates a greater potential for soil liquefaction. Blow counts from borings TB-2 and TB-3 are as shown below:

Boring	**Feet BGS**	**Blow Counts Field (N)**
TB-2	2.5	5.0
	5.0	10.0
	7.5	1.0
	10.0	1.0
	15.0	0.0
	19.0	3.0
TB-3	2.5	8.0
	5.0	5.0
	7.5	3.0
	10.0	3.0
	15.0	2.0
	Average =	3.7

Through the above screening criteria, the potential of the fly ash material observed in the perimeter dike to liquefy under seismic conditions cannot be dismissed. Therefore, a more rigorous analysis was completed for this layer and the Safety Factor for Liquefaction pursuant to 40 CFR 257.73(e)(4) was completed.

3.0 LIQUEFACTION POTENTIAL ASSESSMENT

The liquefaction potential assessment conducted for the fly ash material used the procedures outlined in the Liquefaction Resistance of Soils: Summary Report From the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils document. This assessment used Standard Penetration Test (SPT) blow count data (N values) provided by NTH dated July 2011 contained in Attachment A.

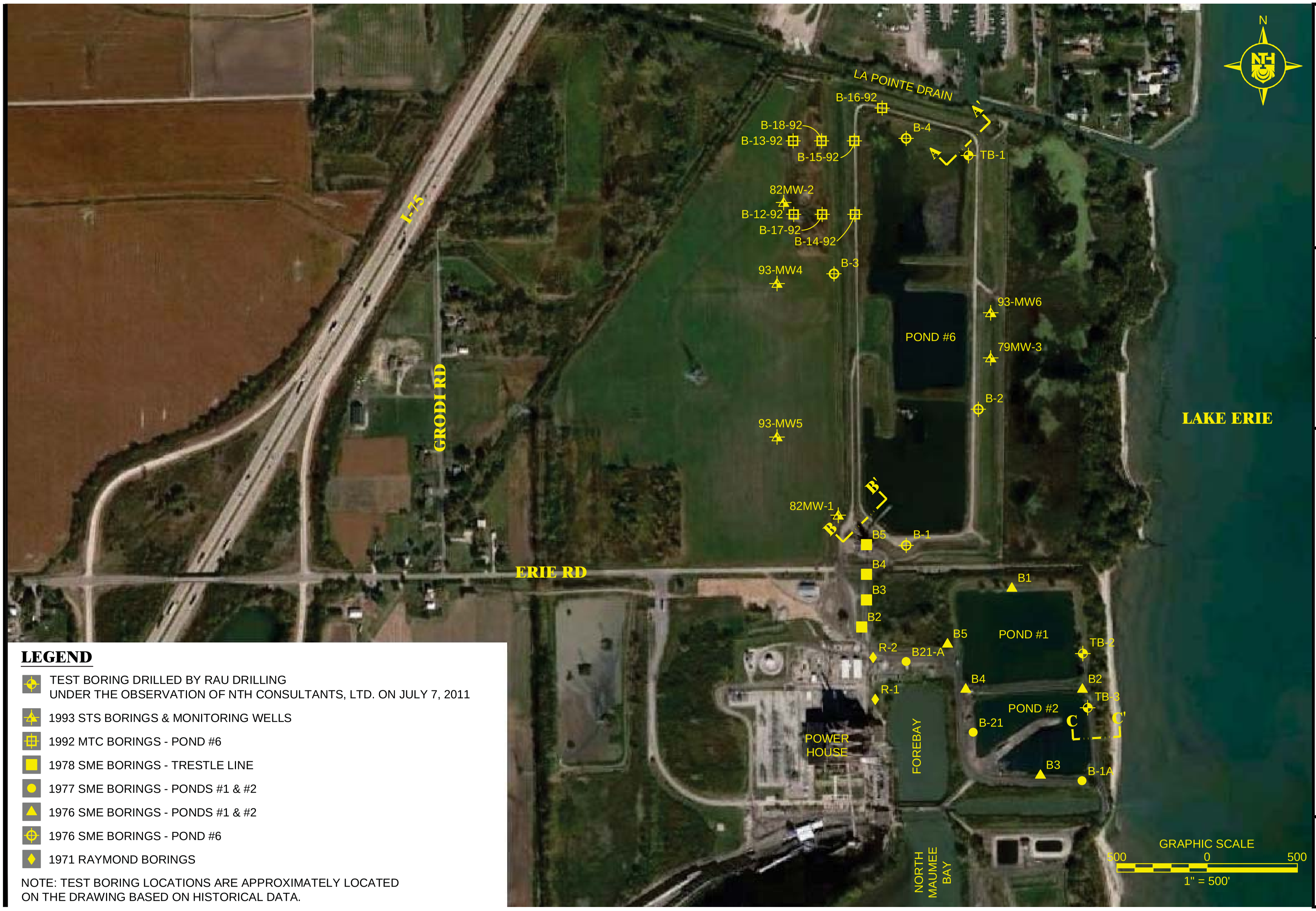
As shown in the calculations, the factor of safety was calculated using the clean sand equivalent corrected N value (N_{60}) for the lowest fly ash material observed, which was 5. The clean sand equivalent corrected N value was then normalized $(N_1)_{60}$. For a conservative analysis, the lowest N value of the fly ash material was used, which was 0.

4.0 CONCLUSION

Calculations of factors of safety against soil liquefaction are contained in Attachment B. As shown on sheet 2 of 2 of the calculations, a clean sand equivalent corrected blow count of 5 yields a minimum factor of safety with regard to liquefaction of 1.31. A sensitivity analysis yields a minimum clean sand equivalent blow count of 4.4 (which equates to less than 0 N value for the fly ash) is necessary to achieve a minimum factor of safety with regard to liquefaction of 1.20.

Therefore, liquefaction of the fly ash material in the perimeter dike in JR Whiting Ponds 1&2 with corrected blow counts of 0 and greater should not occur based on the anticipated seismic activity. It should be noted that a corrected blow count for the fly ash material of 0 yields a clean sand equivalent corrected blow count of 5 and that blow counts (N values) less than 0 are unobtainable.

ATTACHMENT A
PREVIOUS STUDY, NTH DATA



LEGEND

- ⌖ TEST BORING DRILLED BY RAU DRILLING UNDER THE OBSERVATION OF NTH CONSULTANTS, LTD. ON JULY 7, 2011
- ▲ 1993 STS BORINGS & MONITORING WELLS
- ⊞ 1992 MTC BORINGS - POND #6
- ■ 1978 SME BORINGS - TRESTLE LINE
- ● 1977 SME BORINGS - PONDS #1 & #2
- ▲ 1976 SME BORINGS - PONDS #1 & #2
- ⊕ 1976 SME BORINGS - POND #6
- ◆ 1971 RAYMOND BORINGS

NOTE: TEST BORING LOCATIONS ARE APPROXIMATELY LOCATED ON THE DRAWING BASED ON HISTORICAL DATA.

NTH Consultants, Ltd.
Infrastructure Engineering
and Environmental Services

NTH PROJECT No.: 62-110458-01	CAD FILE NAME: D11045801
DESIGNED BY: EB	INCEP DATE: 29 AUG 2011
DRAWN BY: SHB	DRAWING SCALE: 1" = 500'
CHECKED BY: IS	PLOT DATE: 09 NOV 2011

TEST BORING LOCATION PLAN - POND #1, 2, AND 6
NTH 2011 INVESTIGATION / HISTORICAL INVESTIGATIONS

J.R. WHITING POWER PLANT
MONROE COUNTY, MICHIGAN

FIGURE No.
1

NTH Consultants, Ltd.
Infrastructure Engineering
and Environmental Services

GENERAL NOTES

TERMINOLOGY

Unless otherwise noted, all terms utilized herein refer to the Standard Definitions presented in ASTM D 653.

PARTICLE SIZES

Boulders	Greater than 12 inches (305mm)
Cobbles	3 inches (76.2mm) to 12 inches (305mm)
Gravel - Coarse	3/4 inches (19.05 mm) to 3 inches (76.2mm)
Fine	No. 4 - 3/16 inches (4.75mm) to 3/4 inches (19.05 mm)
Sand - Coarse	No. 10 (2.00mm) to No. 4 (4.75mm)
Medium	No. 40 (0.425mm) to No. 10 (2.00mm)
Fine	No. 200 (0.074mm) to No. 40 (0.425mm)
Silt	0.005mm to 0.074mm
Clay	Less than 0.005mm

CLASSIFICATION

The major soil constituent is the principal noun, i.e., clay, silt, sand, gravel. The second major soil constituent and other minor constituents are reported as follows:

Second Major Constituent (percent by weight)	**Minor Constituents (percent by weight)**
Trace - 1 to 12%	Trace - 1 to 12%
Adjective - 12 to 35% (clayey, silty, etc.)	Little - 12 to 23%
And - Over 35%	Some - 23 to 33%

COHESIVE SOILS

If clay content is sufficient so that clay dominates soil properties, clay becomes the principal noun with the other major soil constituent as modified; i.e., silty clay. Other minor soil constituents may be included in accordance with the classification breakdown for cohesionless soils; i.e., silty clay, trace of sand, little gravel.

Consistency	**Unconfined Compressive Strength (psf)**	**Approximate Range of (N)**
Very Soft	Below 500	0 - 2
Soft	500 - 1000	3 - 4
Medium	1000 - 2000	5 - 8
Stiff	2000 - 4000	9 - 15
Very Stiff	4000 - 8000	16 - 30
Hard	8000 - 16000	31 - 50
Very Hard	Over 16000	Over 50

Consistency of cohesive soils is based upon an evaluation of the observed resistance to deformation under load and not upon the Standard Penetration Resistance (N).

COHESIONLESS SOILS

Density Classification	**Relative Density %**	**Approximate Range of (N)**
Very Loose	0 - 15	0 - 4
Loose	16 - 35	5 - 10
Medium Compact	36 - 65	11 - 30
Compact	66 - 85	31 - 50
Very Compact	86 - 100	Over 50

Relative density of cohesionless soils is based upon the evaluation of the Standard Penetration Resistance (N), modified as required for depth effects, sampling effects, etc.

SAMPLE DESIGNATIONS

AS - Auger Sample - directly from auger flight
BS - Miscellaneous Sample - bottle or bag
S - Split Spoon Sample - ASTM D 1586
LS - Split Spoon Sample S with Liner Insert 3 inches in length
ST - Shelby Tube Sample - 3 inch diameter unless otherwise noted
PS - Piston Sample - 3 inch diameter unless otherwise noted
RC - Rock Core - NX core unless otherwise noted
CS - Continuous Sample - from rock core barrel or continuous sampling device
VS - Vane Shear

STANDARD PENETRATION TEST (ASTM D 1586) - A 2.0" outside-diameter, 1-3/8" inside-diameter, split barrel sampler is driven into undisturbed soil by means of a 140-pound weight falling freely through a vertical distance of 30 inches. The sampler is normally driven three successive 6-inch increments. The total number of blows required for the final 12 inches of penetration is the Standard Penetration Resistance (N).

FIGURE NO. 2

LOG OF TEST BORING NO: TB-1

Project Name: *Whiting Slope Stability Analyses*
Project Location: *Erie, Michigan*

NTH Consultants, Ltd.
NTH Proj. No.: 62-110458-01
Checked By: ACE

SUBSURFACE PROFILE					SOIL SAMPLE DATA								
ELEV. (FT)	PRO-FILE	ELEV	GROUND SURFACE ELEVATION: 600.0	DEPTH	DEPTH (FT)	SAMPLE TYPE/NO.	BLOWS/ 6-INCHES	STD. PEN RESIST. (N)	REC (in)	FIELD TEST (ppm)	MOIST. CONTENT (%)	DRY DENSITY (PCF)	UNCONF. COMP ST (PSF)
600													
		597.5	FILL: Loose Gray and Black BOTTOM ASH (Root Fibers)	2.5		LS-1	1 2 3	5	8				-
595		595.0	FILL: Very Stiff Brown SILTY CLAY with Trace Sand and Gravel (Root Fibers)	5.0	5	LS-2	2 4 3	7	6				7000*
						LS-3	3 7 7	14	6				>9000*
590					10	LS-4	4 7 9	16	7				8000*
585					15	LS-5	4 7 8	15	8				9000*
		583.0	FILL: Very Stiff to Hard Gray SILTY CLAY with Trace Sand and Gravel (Root Fibers)	17.0									
580					20	LS-6	2 5 6	11	8		14.8	117.8	6600
575					25	LS-7	3 6 9	15	7				7000*
570		570.0	FILL: Stiff to Very Stiff Brown and Gray SILTY CLAY with Trace Sand and Gravel	30.0	30	LS-8	3 5 6	11	10				5000*
565					35	LS-9	7 10 8	18	18		15.5	117.9	6920
		562.0	Very Stiff Brown SILTY CLAY with Trace Sand and Gravel (Occasional Sand Lenses)	38.0									
560					40	LS-10	1 4 5	9	8		13.8	123.3	3240

Total Depth: *59.4 FT*
Drilling Date: *7/7/11*
Inspector: *S. Sulzman*
Contractor: *Rau Drilling*
Driller: *A. Rau*
Drilling Method:
CME-750 truck mounted drill rig using 3-1/4" ID HSA to EOB.

Plugging Procedure:
Borehole backfilled with bentonite cement grout.

Water Level Observation:
No groundwater encountered during or upon completion of drilling.

Notes:
** = pocket penetrometer value*
SPT testing completed using a standard 140 lbs auto hammer.
TB-1 located at the median of Pond #6 perimeter dike.

GPS Coordinates:

Figure No. 3

LOG OF TEST BORING NO: TB-1

Project Name: *Whiting Slope Stability Analyses*
Project Location: *Erie, Michigan*

NTH Consultants, Ltd.
NTH Proj. No.: 62-110458-01
Checked By: ACE

SUBSURFACE PROFILE					SOIL SAMPLE DATA								
ELEV. (FT)	PRO-FILE	ELEV	GROUND SURFACE ELEVATION: 600.0	DEPTH	DEPTH (FT)	SAMPLE TYPE/NO.	BLOWS/ 6-INCHES	STD. PEN RESIST. (N)	REC (in)	FIELD TEST (ppm)	MOIST. CONTENT (%)	DRY DENSITY (PCF)	UNCONF COMP ST (PSF)
560													
			Stiff Brown and Gray SILTY CLAY with Trace Sand and Gravel										
555					45	LS-11	2 4 7	11	10		14.2	122.3	3440
		552.0		48.0									
550					50	LS-12	4 6 10	16	6				6000*
			Very Stiff Gray SILTY CLAY with Trace Sand and Gravel										
		547.0		53.0									
545					55	LS-13	3 9 12	21	10				9000*
			Hard to Very Hard Gray SILTY CLAY with Some Sand and Trace Gravel										
		540.6		59.4		LS-14	39 50/5"		10		7.2		>9000*
540			END OF BORING AT 59.4 FEET.										
535													
530													
525													
520													
515													
510													

LOG OF TEST BORING NO: TB-2

Project Name: *Whiting Slope Stability Analyses*
Project Location: *Erie, Michigan*

NTH Consultants, Ltd.
NTH Proj. No.: 62-110458-01
Checked By: ACE

SUBSURFACE PROFILE					SOIL SAMPLE DATA								
ELEV. (FT)	PRO-FILE	ELEV	GROUND SURFACE ELEVATION: 590.0	DEPTH	DEPTH (FT)	SAMPLE TYPE/NO.	BLOWS/ 6-INCHES	STD. PEN RESIST. (N)	REC (in)	FIELD TEST (ppm)	MOIST. CONTENT (%)	DRY DENSITY (PCF)	UNCONF. COMP ST (PSF)
590													
585					5	LS-1	3 5 5	10	6				-
						LS-2	1 WOH 1	1	6		48.6	65.2	-
580			FILL: Very Loose to Loose Black and Gray FLY ASH with Trace Sand (Root Fibers)		10	LS-3	1 WOH 1	1	18		47.3	70.2	-
575					15	LS-4	WOH WOH WOH	0	18		53.5	64.8	-
		571.0		19.0									
570			Medium Brown and Gray SILTY CLAY with Trace Sand and Gravel (Occasional Sand and Silt Lenses)		20	LS-5	1 2 2	4	12		31.8	89.4	1280
		568.0		22.0									
565			Very Stiff Brown and Gray SILTY CLAY with Trace Sand and Gravel (Occasional Sand and Silt Lenses)		25	LS-6	4 6 6	14	11		21.7	104	4340
		561.0		29.0									
560		560.0	Stiff Gray SILTY CLAY with Trace Sand and Gravel	30.0	30	LS-7	1 5 7	12	2		19.2		2380
			END OF BORING AT 30.0 FEET.										
555													
550													

Total Depth: *30 FT*
Drilling Date: *7/7/11*
Inspector: *S. Sulzman*
Contractor: *Rau Drilling*
Driller: *A. Rau*
Drilling Method:
CME-750 truck mounted drill rig using 3-1/4" ID HSA to EOB.

Plugging Procedure:
Borehole backfilled with bentonite cement grout.

Water Level Observation:
Groundwater encountered 5.0' during drilling and upon completion of drilling.

Notes:
** = pocket penetrometer value WOH = weight of hammer*
SPT testing completed using a standard 140 lbs auto hammer.
TB-2 located at the inboard side of Pond #1 and #2 perimeter dike.

GPS Coordinates:

Figure No. 4

LOG OF TEST BORING 62-110458-01.GPJ NTH CORPORATE NEW.GDT 11/23/11

LOG OF TEST BORING NO: TB-3

Project Name: *Whiting Slope Stability Analyses*
Project Location: *Erie, Michigan*

NTH Consultants, Ltd.
NTH Proj. No.: 62-110458-01
Checked By: ACE

SUBSURFACE PROFILE						SOIL SAMPLE DATA							
ELEV. (FT)	PRO-FILE	ELEV	GROUND SURFACE ELEVATION: 590.0	DEPTH	DEPTH (FT)	SAMPLE TYPE/NO.	BLOWS/ 6-INCHES	STD. PEN RESIST. (N)	REC (in)	FIELD TEST (ppm)	MOIST. CONTENT (%)	DRY DENSITY (PCF)	UNCONF COMP ST (PSF)
590													
			FILL: Very Loose to Loose Gray and Black FLY ASH with Trace Sand and Gravel			LS-1	4 4 4	8	8				-
585					5	LS-2	3 3 2	5	16		44.8	70.5	-
						LS-3	1 1 2	3	12		44.8	71.6	-
580					10	LS-4	2 2 1	3	14		43.5	72.2	-
575					15	LS-5	1 1 1	2	18		44.2	73.5	-
		573.5		16.5									
			FILL: Soft Black and Gray SILTY CLAY										
570		570.0		20.0	20	LS-6	1 1 2	3	10		23.6	97.3	780
			Very Stiff Brown SILTY CLAY with Trace Sand and Gravel										
565					25	LS-7	2 4 5	9	18				5300
		563.0		27.0									
			Stiff Mottled Brown and Gray SILTY CLAY with Trace Sand and Gravel (Frequent Sandy Gravel Seams)										
560		560.0		30.0	30	LS-8	2 5 5	10	18		13.9	124.2	2680
			END OF BORING AT 30.0 FEET.										
555													
550													

Total Depth: *30 FT*
Drilling Date: *7/7/11*
Inspector: *S. Sulzman*
Contractor: *Rau Drilling*
Driller: *A. Rau*
Drilling Method:
CME-750 truck mounted drilling using 3-1/4" ID HSA to EOB.

Plugging Procedure:
Borehole backfilled with bentonite cement grout.

Water Level Observation:
Groundwater encountered 5.0' during drilling and upon completion of drilling.

Notes:
** = pocket penetrometer value*
SPT testing completed using a standard 140 lbs auto hammer.
TB-3 located at the inboard side of Pond #1 and #2 perimeter dike.

GPS Coordinates:

Figure No. 5

Project No.	62-110458-01	NTH Consultants, Ltd.	Whiting Slope Stability Analyses

TABULATION OF LABORATORY TEST DATA

Boring / Test Pit / Probe Designation	Sample Number	Depth of Sample Tip (ft)	Elevation of Sample Tip (ft)	Unconfined Compressive Strength (psf)	Failure Strain (%)	Natural Water Content (% of dry weight)	In-Place Dry Density (lbs/cu.ft)	PERMEABILITY (CM/SEC)	PARTICLE SIZE DISTRIBUTION (%)							ATTERBERG LIMITS (%)			Apparent Specific Gravity	Loss On Ignition (%)	Unified Soil Classification
									Colloids	Clay	Silt	Fine Sand	Medium Sand	Coarse Sand	Gravel	Liquid Limit	Plastic Limit	Plasticity Index			
TB-1	LS-6	20.0	580.0	6,660	14.2	14.8	117.8									31	16	15			
	LS-9	**35.0**	**565.0**	**6,920**	**15.0**	**15.5**	**117.9**									**26**	**15**	**11**			
	LS-10	40.0	560.0	3,240	15.0	13.8	123.3									25	14	11			
	LS-11	45.0	555.0	3,440	15.0	14.2	122.3									24	14	10			
	LS-14	59.4	540.6			7.2															
TB-2	LS-2	7.5	582.5			48.6	65.2														
	LS-3	10.0	580.0			47.3	70.2														
	LS-4	15.0	575.0			53.5	64.8			6.7	84.3	9.0	0.0	0.0	0.0						
	LS-5	20.0	570.0	1,280	15.0	31.8	89.4									52	23	29			
	LS-6	25.0	565.0	4,340	12.5	21.7	104.0									43	20	23			
	LS-7	30.0	560.0	2,380	15.0	19.2										30	16	14			
TB-3	LS-2	5.0	585.0			44.8	70.5														
	LS-3	7.5	582.5			44.8	71.6														
	LS-4	10.0	580.0			43.5	72.2														
	LS-5	15.0	575.0			44.2	73.5														
	LS-6	20.0	570.0	780	15.0	23.6	97.3			47.4	49.6	3.0	0.0	0.0	0.0	46	19	27			
	LS-7	25.0	565.0	5,300	15.0																
	LS-8	30.0	560.0	2,620	15.0	13.9	124.2									22	13	9			

FIGURE No. 6

NTH Consultants, Ltd.
Infrastructure Engineering
and Environmental Services

NTH Consultants, Ltd.
Southeast Michigan Laboratory

Telephone: 248.553.6300
Fax: 734.524.0927

Aggregate/Soil Test Report

Report No: MAT:62-110458-01-S008
Issue No: 1

Client: Consumers Energy

Project: Whiting Slope Stability Analyses
Slope Stability Analysis

Job No: 62-110458-01

This laboratory is accredited by the American Association of State Highway and Transportation Officials (AASHTO). The tests reported have been completed in accordance with the terms of the

Zeerak Paydawy

Date of Issue: 8/10/2011
Approved Signatory: Zeerak Paydawy

Sample Details

Boring No: TB-2
Field Sample No: LS-4
Sample Depth: 15
Date Sampled:
Sampled By: Stephen Sulzman
LWO No: 000719
Sample Location:

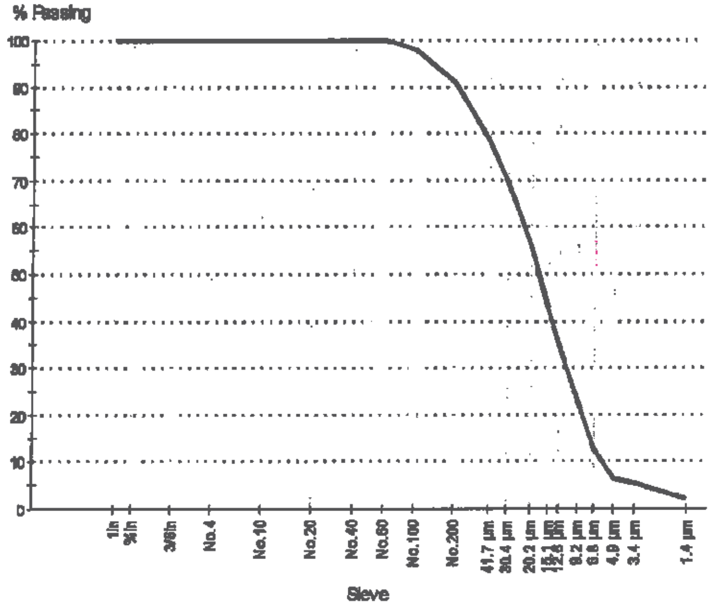
Sample Description:

Grading: Particle Size Analysis of Soils (ASTM D 422 - 07)

Drying by:
Date Tested: 8/10/2011

Sieve Size	% Passing	Limits
1in	100	
¾in	100	
3/8in	100	
No.4	100	
No.10	100	
No.20	100	
No.40	100	
No.60	100	
No.100	98	
No.200	91	
41.7 µm	78.5	
30.4 µm	70.0	
20.2 µm	56.2	
15.1 µm	42.4	
12.6 µm	35.0	
9.2 µm	23.3	
6.8 µm	12.7	
4.9 µm	6.4	
3.4 µm	5.3	
1.4 µm	2.1	

Particle Size Distribution



COBBLES	GRAVEL		SAND			FINES	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
(0.0%)	(0.0%)	(0.0%)	(0.0%)	(0.0%)	(9.0%)	(84.3%)	(6.7%)

NTH Consultants, Ltd.
Infrastructure Engineering
and Environmental Services

NTH Consultants, Ltd.
Southeast Michigan Laboratory

Telephone: 248.553.6300
Fax: 734.524.0927

Aggregate/Soil Test Report

Report No: MAT:62-110458-01-S015
Issue No: 1

Client:	Consumers Energy
Project:	Whiting Slope Stability Analyses Slope Stability Analysis
Job No:	62-110458-01

This laboratory is accredited by the American Association of State Highway and Transportation Officials (AASHTO). The tests reported have been completed in accordance with the terms of the

Date of Issue: 8/10/2011
Approved Signatory: Zeerak Paydawy

Sample Details

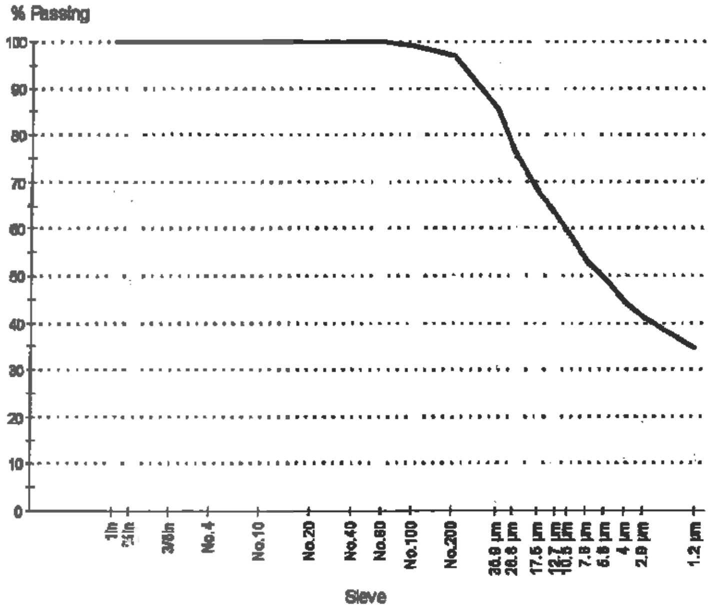
Boring No:	TB-3
Field Sample No:	LS-6
Sample Depth:	20
Date Sampled:	
Sampled By:	Stephen Sulzman
LWO No:	000719
Sample Location:	

Atterberg Limit:

Liquid Limit:	46
Plastic Limit:	19
Plasticity Index:	27
Linear Shrinkage (%):	N/A

Sample Description:

Particle Size Distribution



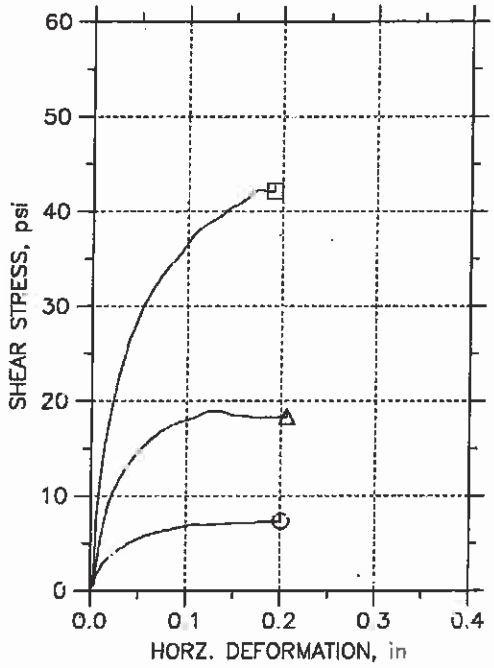
COBBLES	GRAVEL		SAND			FINES	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
(0.0%)	(0.0%)	(0.0%)	(0.0%)	(0.0%)	(3.0%)	(49.6%)	(47.4%)

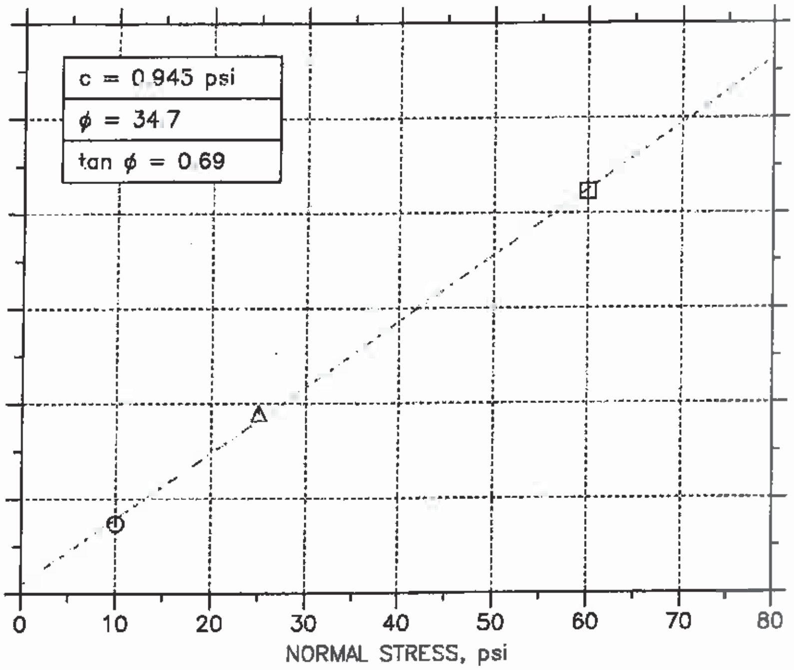
Grading: Particle Size Analysis of Soils (ASTM D 422 - 07)

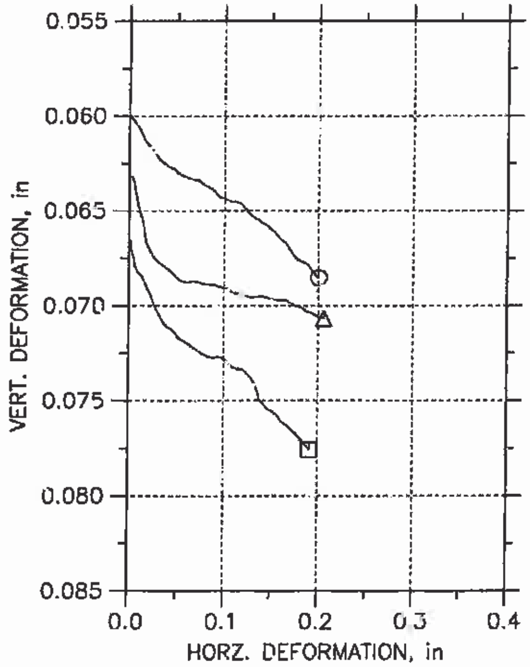
Drying by:
Date Tested: 8/10/2011

Sieve Size	% Passing	Limits
1in	100	
¾in	100	
3/8in	100	
No.4	100	
No.10	100	
No.20	100	
No.40	100	
No.60	100	
No.100	99	
No.200	97	
35.9 µm	85.6	
26.8 µm	76.7	
17.5 µm	67.9	
12.7 µm	62.9	
10.5 µm	59.0	
7.6 µm	53.1	
5.6 µm	49.2	
4.0 µm	44.3	
2.9 µm	41.3	
1.2 µm	34.4	

DIRECT SHEAR TEST REPORT







	Symbol	○	△	□	
	Test No.	1	2	3	
	Sample No.	1	2	3	
	Shape	Circular	Circular	Circular	
Initial	Dimension, in	2.5	2.5	2.5	
	Area, in^2	4.9087	4.9087	4.9087	
	Height, in	1	1	1	
	Water Content, %	54.43	50.13	48.87	
	Dry Density, pcf	63.321	65.137	65.688	
	Saturation, %	94.21	91.09	90.12	
	Void Ratio	1.4155	1.3481	1.3284	
	Consol. Height, in	0.94339	0.93993	0.93483	
	Consol. Void Ratio	1.2787	1.2071	1.1767	
Final	Water Content, %	51.02	48.25	46.85	
	Dry Density, pcf	67.979	70.091	71.212	
	Saturation, %	100.00	100.00	100.00	
	Void Ratio	1.2499	1.1821	1.1478	
	Normal Stress, psi	9.9912	24.997	59.998	
	Max. Shear Stress, psi	7.3484	18.983	42.259	
	Ult. Shear Stress, psi	7.3484	18.358	42.119	
	Time to Failure, min	238.21	175	240	
	Disp. Rate, in/min	0.0009	0.0009	0.0009	
	Estimated Specific Gravity	2.45	2.45	2.45	
	Liquid Limit	NP	NP	NP	
	Plastic Limit	NP	NP	NP	
	Plasticity Index	NP	NP	NP	

Project: Whiting
Location: MI
Project No.: 62-11045801
Boring No.: 1
Sample Type: Bulk
Description: Fly ash
Remarks:

Tue, 06-SEP-2011 09:56:38

FIGURE No. 7

ATTACHMENT B
LIQUEFACTION CALCULATIONS

The Mannik & Smith Group, Inc. 1800 Indian Wood Circle Maumee, Ohio 43537-4086 (419) 891-2222 FAX (419) 891-1595	Whiting Ash Pond Material Liquefaction Assessment	JOB C1790017 SHEET NO 1 OF 3 CALCULATED BY GAB DATE 10/14/16 CHECKED BY ISS DATE SCALE NA

Objective:

Determine liquefaction potential at the Whiting Ash Pond. The factor of safety against liquefaction shall not be less than 1.20.

Method:

Use step by step method outlined in the National Center for Earthquake Engineering Research (NCEER) paper: **Liquefaction Resistance of Soils: Summary Report From The 1996 NCEER And 1998 NCEER/NSF Workshops On Evaluation Of Liquefaction Resistance Of Soils**.

Procedure:

1. Determine total vertical overburden stress (σ), effective vertical overburden stress (σ'), peak ground acceleration (a_{max}), and normalized SPT resistance. For internal slopes, assume no benefit from increase in confining stress due to waste loads.

2. Evaluate stress reduction factor (r_d) - See Figure 1 - Use average value.

$$r_d = \frac{(1.000 - 0.4113z^{0.5} + 0.04052z + 0.001753z^{1.5})}{1.000 - 0.4177z^{0.5} + 0.05729z - 0.006205z^{1.5} + 0.001210z^{2}}$$

Where: z = depth beneath ground surface in meters

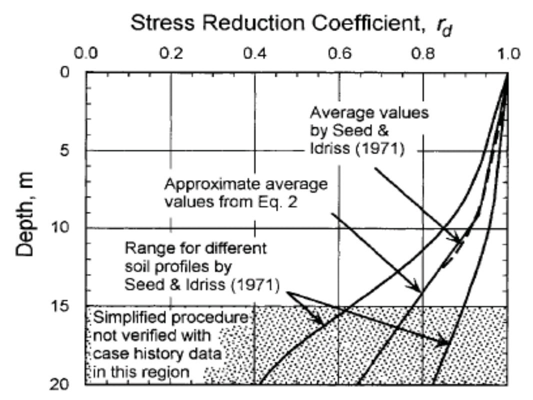


FIG. 1. r_d versus Depth Curves Developed by Seed and Idriss (1971) with Added Mean-Value Lines Plotted from Eq. (2)

3. Calculate the seismic demand on the soil layer expressed as the cyclic stress ratio (CSR).

$$CSR = 0.65 \frac{a_{MAX}}{g} r_d \frac{\sigma}{\sigma'}$$

Where: g = the acceleration of gravity
a_{max} = peak ground surface acceleration
= 0.05 (USGS 2% probability in 50 years - 2008 map see attached)

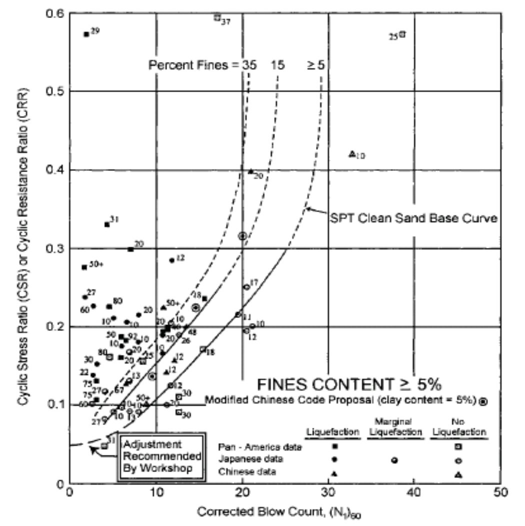
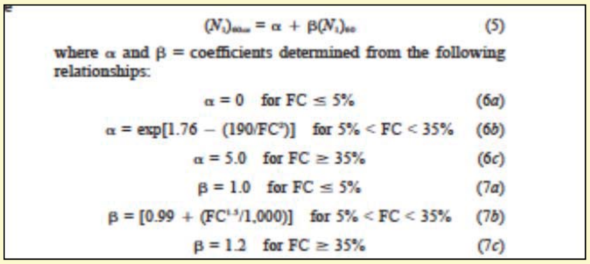


FIG. 2. SPT Clean-Sand Base Curve for Magnitude 7.5 Earthquakes with Data from Liquefaction Case Histories (Modified from Seed et al. 1985)

4. Calculate the clean sand equivalent N_{60} value - See Equations below:

==> >35% fines



$(N_1)_{60cs} = \alpha + \beta(N_1)_{60}$ (5)

where α and β = coefficients determined from the following relationships:

$\alpha = 0$ for FC ≤ 5% (6a)

$\alpha = \exp[1.76 - (190/FC^2)]$ for 5% < FC < 35% (6b)

$\alpha = 5.0$ for FC ≥ 35% (6c)

$\beta = 1.0$ for FC ≤ 5% (7a)

$\beta = [0.99 + (FC^{1.5}/1{,}000)]$ for 5% < FC < 35% (7b)

$\beta = 1.2$ for FC ≥ 35% (7c)

Minimum Corrected Fly Ash Zone (N_{60}) value = 0 bpf
Use Equation 6c for FC greater than 35% α = 5.0
Use Equation 7c for FC greater than 35% β = 1.2

$(N_{60})_{cs}$ = 5.0 + 1.2*0
$(N_{60})_{cs}$ = 5.0

5. Calculate the capacity of the soil to resist liquefaction expressed as the cyclic resistance ratio ($CRR_{7.5}$) - See Figure 2

==> >35% fines

$$CRR_{7.5} = \frac{1}{34 - (N_1)_{60}} + \frac{(N_1)_{60}}{135} + \frac{50}{[10 \cdot (N_1)_{60} + 45]^2} - \frac{1}{200}$$

6. Calculate the factor of safety against liquefaction (resisting force divided by driving force).

Where: FS = $CRR_{7.5}$/CSR

The Mannik & Smith Group, Inc.
1800 Indian Wood Circle
Maumee, Ohio 43537-4086
(419) 891-2222
FAX (419) 891-1595

Whiting Ash Pond Material

Liquefaction Assessment

JOB **C1790017**
SHEET NO **2** OF **3**
CALCULATED BY **GAB** DATE **10/14/16**
CHECKED BY **ISS** DATE
SCALE **NA**

Calculations:

Assumptions and Input Parameters

Assumed Surface Elevation	590	ft	Assume fly ash is located at 0 to 18 feet beneath surface grades.		
Assumed Phreatic Surf. Elev.	584	ft	Assumed Atmospheric Pressure (P_a)	2100	psf
Avg Unit Wt of Insitu Soil (γ_s)	96	pcf*	Clean Sand Equivalent: Blow Count for Fly Ash Zone (N_{60})	5	bpf
Magnitude	7.5		(See Sheet 1 procedure step 4 for clean sand equivalent)		
Peak Ground Acceleration (a_{max})	0.057	g (given as %g sheet 3)	This value is indicative of a loose sand		

Factor of Safety calculation for internal slopes

Saturated Ash Elevation (ft)	Total Vert. Overburden Stress (σ_{vo}) (psf)	Effective Vertical Overburden Stress (σ'_{vo}) (psf)	Depth to Sand (z) (ft)	Depth to Sand (z) (m)	Stress Ratio (σ/σ')	Rep. Norm. Blow Count for Ash Zone $(N_1)_{60}$ (bpf)	Stress Red. Factor (r_d)	CSR	$CRR_{7.5}$	MSF	FS
590	0	0	0	0.00	#DIV/0!	9	1.00	#DIV/0!	0.11	1.00	**#DIV/0!**
589	96	96	1	0.30	1.00	9	1.00	0.037	0.10	1.00	**2.78**
588	192	192	2	0.61	1.00	9	1.00	0.037	0.10	1.00	**2.71**
587	288	288	3	0.91	1.00	8	1.00	0.037	0.10	1.00	**2.64**
586	384	384	4	1.22	1.00	8	1.00	0.037	0.10	1.00	**2.58**
585	480	480	5	1.52	1.00	8	1.00	0.037	0.09	1.00	**2.52**
584	576	576	6	1.83	1.00	7	1.00	0.037	0.09	1.00	**2.47**
583	672	610	7	2.13	1.10	7	1.00	0.041	0.09	1.00	**2.23**
582	768	643	8	2.44	1.19	7	1.00	0.044	0.09	1.00	**2.04**
581	864	677	9	2.74	1.28	7	1.00	0.047	0.09	1.00	**1.90**
580	960	710	10	3.05	1.35	7	0.99	0.050	0.09	1.00	**1.78**
579	1056	744	11	3.35	1.42	7	0.99	0.052	0.09	1.00	**1.69**
578	1152	778	12	3.66	1.48	7	0.99	0.055	0.09	1.00	**1.61**
577	1248	811	13	3.96	1.54	7	0.99	0.057	0.09	1.00	**1.54**
576	1344	845	14	4.27	1.59	7	0.99	0.058	0.09	1.00	**1.48**
575	1440	878	15	4.57	1.64	7	0.99	0.060	0.09	1.00	**1.43**
574	1536	912	16	4.88	1.68	7	0.99	0.062	0.09	1.00	**1.38**
573	1632	946	17	5.18	1.73	7	0.99	0.063	0.08	1.00	**1.34**
572	1728	979	18	5.49	1.76	7	0.99	0.065	0.08	1.00	**1.31**

*Assume dry density of 65 pcf at an average moisture content of 47%

Determine SPT resistiance required to decrease FS<1.2

Assumptions and Input Parameters

Assumed Surface Elevation	590	ft	Assume fly ash is located at 0 to 18 feet beneath surface grades.		
Assumed Phreatic Surf. Elev.	584	ft	Assumed Atmospheric Pressure (P_a)	2100	psf
Avg Unit Wt of Insitu Soil (γ_s)	96	pcf	Clean Sand Equivalent: Blow Count for Fly Ash Zone (N_{60})	4.4	bpf
Peak Ground Acceleration (a_{max})	0.057	g			

Factor of Safety calculation for internal slopes

Saturated Sand Elevation (ft)	Total Vert. Overburden Stress (σ_{vo}) (psf)	Effective Vertical Overburden Stress (σ'_{vo}) (psf)	Depth to Sand (z) (ft)	Depth to Sand (z) (m)	Stress Ratio (σ/σ')	Rep. Norm. Blow Count for Sand Zone $(N_1)_{60}$ (bpf)	Stress Red. Factor (r_d)	CSR	$CRR_{7.5}$	MSF	FS
590	0	0	0	0.00	#DIV/0!	8	1.00	#DIV/0!	0.10	1.00	**#DIV/0!**
589	96	96	1	0.30	1.00	8	1.00	0.037	0.09	1.00	**2.53**
588	192	192	2	0.61	1.00	7	1.00	0.037	0.09	1.00	**2.47**
587	288	288	3	0.91	1.00	7	1.00	0.037	0.09	1.00	**2.42**
586	384	384	4	1.22	1.00	7	1.00	0.037	0.09	1.00	**2.37**
585	480	480	5	1.52	1.00	7	1.00	0.037	0.09	1.00	**2.32**
584	576	576	6	1.83	1.00	7	1.00	0.037	0.08	1.00	**2.28**
583	672	610	7	2.13	1.10	6	1.00	0.041	0.08	1.00	**2.05**
582	768	643	8	2.44	1.19	6	1.00	0.044	0.08	1.00	**1.88**
581	864	677	9	2.74	1.28	6	1.00	0.047	0.08	1.00	**1.75**
580	960	710	10	3.05	1.35	6	0.99	0.050	0.08	1.00	**1.65**
579	1056	744	11	3.35	1.42	6	0.99	0.052	0.08	1.00	**1.56**
578	1152	778	12	3.66	1.48	6	0.99	0.055	0.08	1.00	**1.48**
577	1248	811	13	3.96	1.54	6	0.99	0.057	0.08	1.00	**1.42**
576	1344	845	14	4.27	1.59	6	0.99	0.058	0.08	1.00	**1.37**
575	1440	878	15	4.57	1.64	6	0.99	0.060	0.08	1.00	**1.32**
574	1536	912	16	4.88	1.68	6	0.99	0.062	0.08	1.00	**1.28**
573	1632	946	17	5.18	1.73	6	0.99	0.063	0.08	1.00	**1.24**
572	1728	979	18	5.49	1.76	6	0.99	0.065	0.08	1.00	**1.21**

Results of the liquefaction analysis show that a representative blow count (average) results in a minimum factor of safety of approximately 1.3 to 2.78. As shown in the table, the factor of safety decreases with depth.

In addtion, the analysis shows that though there is limited blow count data for the fly ash zone, a clean sand equavalent SPT blow count as low as 4.4 results in a factor of safety greater than 1.2 for the fly ash material. However when adjusting to the clean sand curve, a blow count of 0 in the fines rich material becomes clean sand equivalent of 5, indicating liquefaction is unlikely.

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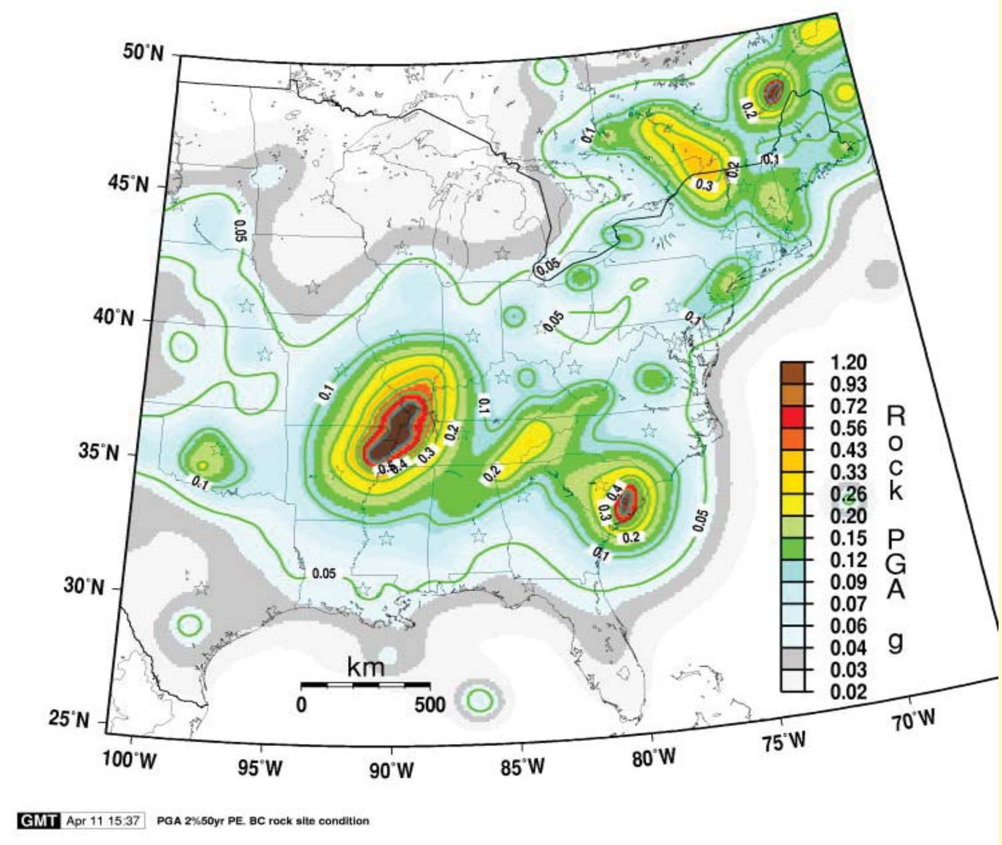
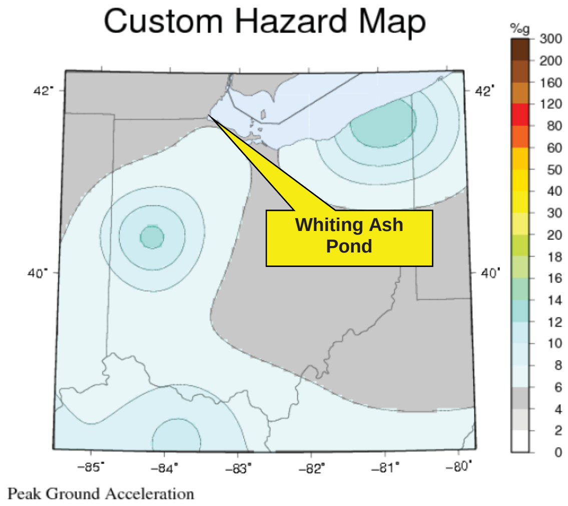
Whiting Ash Pond Material

Liquefaction Assessment

JOB	C1790017		
SHEET NO	3	OF	3
CALCULATED BY	GAB	DATE	10/14/16
CHECKED BY	ISS	DATE	
SCALE	NA		





USGS
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Whiting Ash Pond

Location Lat. Long.

The interpolated Probabilistic ground motion values, in %g, at the requested point are:

	10% PE in 50 yr	2% PE in 50 yr
PGA		

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SEISMIC HAZARD: Hazard by Lat/Lon, 2008

Attachment 1: 2008 USGS Seismic Hazard Map