REPORT

NIPSCO R.M. SCHAHFER GENERATING FACILITY

WASTE DISPOSAL AREA (WDA)

STRUCTURAL STABILITY AND SAFETY FACTOR ASSESSMENT

Wheatfield, Indiana

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

Submitted To: Northern Indiana Public Service Company
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October 5, 2016

Project No. 1651599

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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

October 5, 2016

Date of Report Certification

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1. 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. At the Northern Indiana Public Service Company (NIPSCO), R.M. Schahfer Generating Station (RMSGS), the only CCR unit which meets this criteria is the Waste Disposal Area (WDA).

This report provides the initial structural stability assessment and the safety factor assessment for the WDA surface impoundment at the NIPSCO RMSGS, located in Wheatfield, Indiana, see Figures 1 and 2. A hazard potential classification was conducted for the WDA pursuant to Section 257.73(a)(2), which resulted in a high hazard classification thereby requiring the probable maximum flood (PMF) elevation to be used in structural assessment.

1.2 WDA Background

The WDA was designed by Sargent & Lundy Engineers of Chicago, Illinois in 1982. The WDA is formed by a ring earth-fill dike with slurry trench core that is approximately 17 feet high and 7,540 feet long (including the common embankment) with a crest elevation of 681 feet above mean sea level (Marbach, 2011). The WDA was constructed for NIPSCO, put in service in 1982, and has been continuously owned and operated by NIPSCO.

The WDA accepts sluiced bottom ash and boiler slag CCR and various sump discharges from the generating station. The sluiced CCR enters the WDA via elevated pipes at the north side and also via buried pipes located at the northwest corner, the pipes do not penetrate the slurry wall core. Water exits the WDA via an overflow weir, to the Recycle Basin, or through the auxiliary spillway located at the northwest side. The overflow weir is located at the southern end of the east side of the WDA. The auxiliary spillway consisting of two, 24 inch diameter corrugated steel pipes with a concrete down-slope channel transitioning to a rip-rap lined downstream channel, is located near the northwest corner of the WDA. The east side of the WDA is common with the west side of the adjacent Recycle Basin. A survey of the WDA was performed by Marbach, Brady and Weaver, Inc. in December 2011 (Marbach, 2011), see Figure 3.

1.3 Previous Evaluations

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
Various construction drawings	1982	Sargent & Lundy Engineers
Assessment of Dam Safety of Coal Combustion Surface Impoundments, NIPSCO, RM Schahfer Generating Station	July 2010	CDM for the EPA
Report on Inspection of The Waste Disposal Area	January 2011	Golder Associates Inc.
Final Hazard Classification Review Report – NIPSCO Schahfer Generating Station	January 2011	Golder Associates Inc.
Embankment Elevation Survey, Waste Disposal Area and Recycle Pond, NIPSCO Schahfer Generating Station	December 2011	Marbach, Brady and Weaver, Inc.
Schahfer Spillway Hydrologic and Hydraulic Evaluation	December 2011	Golder Associates Inc.
Final Geotechnical Investigation and Embankment Stability Analyses	June 2012	Golder Associates Inc.
Report on Inspection of The Waste Disposal Area	September 2012	Golder Associates Inc.
Construction in a Floodway Permit Application, NIPSCO R.M. Schahfer Generating Station	November 2012	Golder Associates Inc.
Waste Disposal and Recycle Ponds Hydrographic Survey. NIPSCO R.M. Schahfer Generating Station	December 2012	DLZ Industrial, LLC (DLZ)
Basin Operation, Maintenance and Inspection Plan, NIPSCO R. M. Schahfer Generating Station	February 2013	Golder Associates Inc.
Emergency Action Plan**,** Final Settling Basin (FSB), Intake Settling Basin (ISB), Waste Disposal Area (WDA), Recycle Basin (RB), Northern Indiana Public Service Company (NIPSCO), R.M. Schahfer Generating Station	February 2013	Golder Associates Inc.

State of Indiana Department of Natural Resources (DNR), Certificate of Approval, After-the-Fact, Construction in a Floodway	April 23, 2013	State of Indiana DNR
Report on Inspection of The Waste Disposal Area	April 2014	Golder Associates Inc.
Construction Observation Documentation Report, Surface Water Basin Erosion Repairs, NIPSCO R.M. Schahfer Generating Station	October 2014	Golder Associates Inc.
First Annual RCRA CCR Unit Inspection Report – NIPSCO Schahfer Generating Station	January 2016	Golder Associates Inc.
Hazard Potential Classification Assessment and Visual Inspection Report – RCRA CCR Units, Waste Disposal Area, Drying Area, Material Storage Runoff Basin, & Metal Cleaning Waste Basin – Surface Impoundments, NIPSCO, R.M. Schahfer Generating Station	September 2016	Golder Associates Inc.

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2. SITE DESCRIPTION

2.1 SUBSURFACE CONDITIONS

Soil borings and laboratory testing programs were completed in 2010, 2011 and 2012 around the WDA to develop site specific stratigraphy and engineering material properties. Golder performed a geotechnical investigation of the WDA in 2011 and prepared the *2012 Geotechnical Investigation and Embankment Stability Analyses* report, dated August 27, 2012. Topographically, the area is generally flat to gently rolling with isolated hills. In the northern and northeastern portions of Jasper County where the WDA is located, the soil is sandy, and is interspersed with sandy knolls and ridges. The northern part of the county is covered by Pleistocene aged, alluvial sand overlying shale of Carboniferous age.

The WDA is located in a rural area and is surrounded by farmland, forested areas, and isolated farm buildings to the south, and by the generating station and other infrastructure to the north. The Recycle Basin is contiguous to the east. The Drying Area is contiguous to the north.

2.2 Physical Properties of Foundation Materials

Based on the site specific available boring logs (Golder, 2012), the site is underlain by a relatively uniform deposit of coarse to fine sand with traces of gravel and silt overlying shale bedrock. Locally, there is a clayey or fine-grained deposit just above the shale bedrock, but this stratum is not evident at all boring locations.

Based on the available construction drawings (Sargent and Lundy, 1982), the WDA embankment is constructed of the native sand materials obtained from on-site borrow areas. The embankment footprint was stripped to a depth of approximately 1 foot below natural grade prior to embankment construction. The embankment fill placement and compaction was completed prior to construction of the slurry trench, which is located along the embankment centerline. The slurry trench is approximately 1.5 feet wide, and extends from 2 feet below the embankment crest down to the shale bedrock. The interior of the WDA is at approximately original ground surface elevation less the approximate 1 foot strip depth. The WDA's inlet and outlet pipes are located above the top of the slurry trench and do not penetrate it.

2.3 Engineering Properties of Foundation Materials

Historic construction drawings and technical specifications suggest that the WDA was constructed with reasonable and sound construction practices. Select drawings (Sargent and Lundy, 1982) can be attributed to the WDA, and these drawings indicate reasonable construction configurations, e.g. 3 horizontal to 1 vertical (3H:1V) upstream and downstream side slopes; embankment constructed of controlled compacted fill; central slurry trench extending down to shale bedrock at depth; inlet and outlet pipes that do not penetrate the slurry trench; rip-rap with bedding on the upstream slope; reinforced concrete structures at

the primary and auxiliary spillway, and inlet and outlet pipes; and detailed surface water control around the structure.

The available historic construction drawings also contain some geotechnical data indicating relatively uniform embankment foundation conditions at the WDA consisting of coarse to fine sand with traces of gravel and silt down to shale bedrock at a depth of approximately 40 feet.

The *Final 2012 Geotechnical Investigation and Embankment Stability Analyses*, prepared by Golder, was referenced during the file review for the WDA. Based on the *2012 Geotechnical Investigation and Embankment Stability Analyses (Golder, 2012),* cone penetration soundings were conducted in June 2011 at the WDA. Six cone penetration test (CPT) probes (noted at CPT-39 though CPT-44 on Figure 2) were advanced in and around the WDA. One CPT probe (CPT-38) was advanced in the adjacent Recycle Basin, which was built at the same time and has the same construction. CPT-38 was deeper than the 6 CPT probed advanced in the WDA, so CPT-38 was included in this analysis for the WDA. The subsurface conditions encountered during the June 2011 investigation are reasonably consistent with those encountered during the previous CPT probing performed at the site, and also with information available from previous historic geotechnical information at the site. The exploration indicated subsurface conditions are dense to very dense sand to silty sand from ground surface to the full depth of the exploration.

Laboratory testing was also performed on samples collected during the geotechnical investigation. The test results indicate a relatively uniform deposit of poorly graded, fine sand with typically less than 10 percent medium sand and less than 10 percent fines. The material is variously classified as a poorly graded sand with little or no fines (SP); a silty sand or sand silt mixture (SM); or a “SP-SM” which is a borderline classification used for materials with between 5 percent and 12 percent fines. The measured water contents ranged from approximately 10 percent to 20 percent. The distribution of water content with depth indicates with reasonable certainty where the water table is in the field. Laboratory samples consistently showed lower water contents in the upper portions of holes, and higher water contents in the lower portions.

The geotechnical model for the WDA is dense silty sand (embankment fill) overlying dense silty sand (subgrade). Figure 4, attached, shows the typical designed cross section of the WDA. Figure 5, attached, shows the geotechnical model for the WDA to be used for the factor of safety analysis. It should be noted that for the purposes of the factor of safety analysis prepared for the WDA and described in Section 4 of this report, the designed crest elevation (681 feet above mean sea level (ft MSL)) was used as the highest elevation found on the WDA, which is a worst case scenario. The surveyed lowest crest elevation (680 ft MSL, Marbach, 2011) was used in the spillway capacity calculations, because that is a worst case scenario for the spillway capacity calculations.

Material properties of each of the modeled layers are included in Table 2 below. These properties are based on the geotechnical investigation and associated laboratory testing that was performed by Golder (Golder, 2012).

Table 2: Geotechnical Model Material Properties

Material	Internal Friction Angle (deg.)	Peak Cohesion (psf)	Dry Unit Weight (pcf)	Saturated Unit Weight (pcf)	Undrained Shear Strength (psf)	Layer Thick-ness (ft)
Embank-ment Fill	42	0	125	135	NA	Varies
Topsoil	35	0	120	120	NA	0.5
Existing Subgrade	39	0	110	125	NA	Varies
Slurry Wall	NA	300	120	NA	NA	Varies
Riprap	45	0	140	145	NA	1
Crushed Stone	45	0	140	145	NA	Varies
Loose Silty Sand Subgrade	37	0	125	132	NA	Varies
Shale	45	0	145	150	0	Varies

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

2.4 Waste Disposal Area Design and Construction Details

Available applicable Sargent & Lundy (1982) construction drawings provided by NIPSCO were reviewed and utilized during the preparation of this report.

A crest survey was performed the week of December 19, 2011 by Marbach, Brady & Weaver, Inc. (Marbach 2011). Survey data was obtained at 50 foot intervals along the crest centerline and embankment cross-section data was obtained on 500 foot intervals. Note that the 2011 survey reference vertical datum is North American Vertical Datum (NAVD) 88, while the original Sargent & Lundy construction drawing reference is U.S. Geological Survey (USGS) 1929 vertical datum adjustment.

The WDA was constructed for NIPSCO, put in service in 1982, and has been continuously owned and operated by NIPSCO. The WDA was designed by Sargent & Lundy Engineers of Chicago, Illinois. The WDA is formed by a ring dike approximately 7,540 feet long (including the common embankment). The constructor of the WDA is not known. Salisbury Engineering of Griffiths, Indiana performed at least some of the historical geotechnical soil borings and geotechnical laboratory testing associated with the WDA

geotechnical investigation and subsurface characterization. An additional geotechnical investigation was performed by Golder in 2011/2012.

A general description of the WDA is presented in Section 1.2. The location of the WDA relative to the generating station and surrounding structures is shown on Figures 1 and 2, attached.

SIZE AND PHYSICAL DATA

Designed Crest Elevation:	681 ft MSL (USGS 29) based on construction drawings
Current Lowest Crest Elevation:	680 ft MSL based on the December 2011 (Marbach, 2011) crest survey (NAVD 88)
Surrounding Ground Elevation:	Approximately 664 ft MSL
High Water Level:	678.9 ft MSL based on invert elevations of spillway pipes
Height:	17 feet
Surface Area:	75.5 acres
Reservoir Volume:	1,530 acre-feet

3. 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 June 2, 2016 for the initial structural stability assessment (Golder, September 2016).

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

Based on the available construction drawings (Sargent and Lundy, 1982), the WDA embankment is constructed of the native sand materials obtained from on-site borrow areas. The embankment footprint was stripped to a depth of approximately 1 foot below natural grade prior to embankment construction. The embankment fill placement and compaction was completed prior to construction of the slurry trench, which is located along the embankment centerline. The slurry trench is approximately 1.5 feet wide, and extends from 2 feet below the embankment crest down to the shale bedrock. The interior of the WDA is at approximately original ground surface elevation less the approximate 1 foot strip depth. The WDA's inlet and outlet pipes are located above the top of the slurry trench and do not penetrate it.

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 WDA embankment is protected from erosion and deterioration by the establishment of a vegetative cover. The vegetative cover is inspected by NIPSCO personnel weekly for signs of erosion, seepage, animal burrows, sloughing, and plants that could negatively impact the embankment. The June 2016 inspection did not identify items relating to slope protection that required investigation or repair and the downstream slopes of the WDA are not subjected to wave or sudden drawdown effects. To reduce the possible impact of rising water surface elevations, waves, or ice sheets, upstream shoreline rip-rap protection has been installed along the upstream slope of the dike. Additionally, the downstream and upstream slopes are inspected weekly for erosion, signs of seepage, animal burrows, sloughing, and vegetation that could negatively impact the embankment. The 2016 annual inspection report did not identify any items relating to slope protection that required investigation or repair. The existing slope protection measures are considered adequate to provide against surface erosion, wave action, and adverse effects of sudden drawdown.

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

Based on the available construction drawings (Sargent and Lundy, 1982), the WDA embankment is constructed of the native sand materials obtained from on-site borrow areas. The embankment footprint was stripped to a depth of approximately 1 foot below natural grade prior to embankment construction. The embankment fill placement and compaction was completed prior to construction of the slurry trench, which is located along the embankment centerline. The slurry trench is approximately 1.5 feet wide, and extends from 2 feet below the embankment crest down to the shale bedrock. The interior of the WDA is at approximately original ground surface elevation less the approximate 1 foot strip depth. The WDA's inlet and outlet pipes are located above the top of the slurry trench and do not penetrate it. 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 the WDA.

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

The EPA has vacated the requirement that vegetative cover on surface impoundment dikes be maintained at no more than six inches. At the time of the June inspection, the WDA's downstream slopes were adequately covered with appropriate vegetation that was well maintained. 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 the WDA is considered the overflow weir which is hydraulically linked to the adjacent Recycle Basin. The overflow weir was visually inspected during the June 2016 inspection, and is generally in good condition where visible. The overflow weir is located at the southeast side of the WDA where it connects to the Recycle Basin and is constructed of reinforced concrete (based on historical construction drawing review). Available drawings indicate the outlet conduit is a 36 inch diameter steel pipe with an energy dissipating reinforced concrete structure at the outlet end. Much of this structure is buried or was submerged and could not be inspected.

The auxiliary spillway is considered the two 24 inch diameter corrugated metals pipes (CMPs) located at the northwest side of the WDA, and were observed to be in acceptable condition. The 24 inch diameter CMPs are located side by side and at the outlet end there is a concrete downslope channel. Below the concrete downslope channel is a rip-rap lined channel leading to a perimeter ditch. At the time of the June 2016 inspection, the water level in the WDA was observed at approximately 2 feet below the invert of the inlet ends of the pipes.

A hydrologic and hydraulic analysis was completed for the WDA 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
- 1000-year flood for a significant hazard potential CCR surface impoundment; or
- 100-year flood for a low hazard potential CCR surface impoundment.

Since the WDA has been classified as having a high hazard potential (Golder, September 2016), it is required to manage the flow during and following the peak discharge from a PMF event. A HEC-HMS (USACE, 2015) analysis and wave analysis was performed for the WDA. Since the principal spillway is an interconnecting pipe to the Recycle Basin, from which water is pumped as a discharge, the only applicable spillway for the WDA is the auxiliary spillway. Therefore, the analysis was performed using the auxiliary spillway, which includes the two 24 inch diameter CMP's with the invert elevation 678.9 ft MSL, as the only spillway available to manage the PMF event.

Results of the hydrology and hydraulics analysis of the WDA are summarized below in Table 4. These include the results of HEC-HMS (USACE, 2015) modelling analysis and the results of the wave action analysis.

Table 4: Hydrology and Hydraulics Analysis Results

Depth of Precipitation (in) for a PMF Event	31.9
WDA Catchment Area (acres)	83.5
WDA Lowest Crest Elevation (ft MSL, Marbach, 2011)	680
Invert Elevation of Auxiliary Spillway (ft MSL)	678.9
Maximum Inflow from Direct Precipitation (cubic feet per second (cfs))	3,668
Maximum Combined Inflow (cfs) [1]	3,708
Maximum WDA Outflow through Spillway (cfs)	37.9
Maximum Water Surface Elevation (ft MSL) [2]	682.2
Height of Wave Action (feet)	1.28
Net Freeboard during Design Storm Event (feet) [3, 7]	-3.4

Notes:
[1] Includes direct precipitation and 40 cfs from overflow weir.
[2] Assumes extra storage capacity is available above embankment crest (e.g. there is no outflow from the impoundment due to overtopping)

[3] Negative freeboard indicates that the embankment will overtop.
[5] All spillway configurations assume 2% longitudinal slope at embankment crest.
[6] All spillway cross-sections are trapezoidal.
[7] Net freeboard = minimum freeboard required for storm event plus the height of wave action.

As shown in Table 4, the current configuration of the WDA's auxiliary spillway is not compliant with 40 CFR 257.73(d)(1)(v). It is Golder's recommendation that NIPSCO explore options to improve the WDA emergency spillway to satisfy those requirements.

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

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.

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

The downstream slopes of the WDA are not adjacent to water bodies and therefore rapid-drawdown was not considered a potential mechanism for structural instability in the exterior slope.

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, one structural stability deficiency was identified as follows:

- The auxiliary spillway is not sized to manage the flow produced by a PMF event.

As a result, it is recommended that NIPSCO remedy this deficiency by improving the size of the auxiliary spillway, operationally controlling the water level in the WDA, or implementing an equivalent engineering or operational control.

4. 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 and equal to the outlet elevation (elevation = 678.9 ft MSL) for this facility; 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 (681 ft MSL); 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

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 Spencer and Janbu method of slices (Abramson et al. 2002), which was used for this stability analysis.

The specific analysis types are:

- Steady state seepage, Maximum Pool Storage (257.73 (e)(1)(i)), downstream slope
- Steady state seepage, Maximum Pool Surcharge (257.73 (e)(1)(ii)), downstream slope
- Seismic (pseudo-static) with Maximum Pool Storage, steady state seepage, (257.73(e)(1)(iii)), downstream slope

The steady state analyses were performed with the fully developed phreatic surface as indicated by the site geotechnical investigation and as extrapolated based on inferred subsurface conditions. This phreatic surface begins at the upstream water level, extends horizontally to the upstream side of the slurry wall, then extends downward at a steep angle through the slurry wall to near the elevation where the groundwater level was encountered in exploratory holes in the downstream side of the embankment. The inferred

piezometric levels in each model are illustrated in Appendix A. Drained shear strength parameters were used in all of the slope stability analyses for all of the material types except the slurry wall.

4.1.1 Cross-Section Analyzed

The critical section of the exterior dike was determined by using the existing topography (2011), and considering the interpreted soil profile from the subsurface investigations, and phreatic surface. The critical cross section is the cross section anticipated to be the most susceptible of all cross sections to structural failure based on appropriate engineering considerations, including loading conditions.

The critical section used for the slope stability analysis is shown on Figure 4.

4.1.2 Geotechnical Material Properties

Based on the subsurface investigations and laboratory testing, representative material properties were selected for use in the stability analysis. These properties are included in Table 2 - Geotechnical Model Material Properties.

4.1.3 Seismic analysis

A pseudo static seismic analysis was performed on the downstream slope of the WDA. The analyses were performed with the same steady state, fully developed phreatic surface in the embankments as was used in the initial two cases analyzed for the WDA. The ground acceleration used in the seismic analysis was 0.1472g, which is the Maximum Considered Earthquake (MCE) ground motion of 0.2 second spectral response, or the 2 percent exceedance in 50 years. The value of the acceleration was obtained from the United States Geologic Survey (USGS) online seismic hazard tool, which provides such information for any location in the United States. The zip code for the RMSGS was used as the location of the site. Contour intervals of this same seismic acceleration are included in Appendix D of the US Army Corps of Engineers (USCOE) publication number: ER 1110-2-1806 titled *Engineering and Design – Earthquake Design and Evaluation for Civil Works Projects*. This contour map, which illustrates the seismic acceleration contours for the 0.2 sec spectral response and 2 percent probability of exceedance in 50 years is also included in Appendix A of this report. This map shows how the area of northwest Indiana is a relatively low hazard area from the view point of seismic risk. The RMSGS is in Risk Zone 1 in the ASCE seismic risk categorization which is also illustrated in the USACOE publication referenced above. This is the second lowest category in a five category system. This ASCE seismic risk map is also included in Appendix A.

4.1.4 Factor of Safety Results

As previously indicated, analyses were performed for the loading cases on the representative cross section for the WDA. Analyses were performed with both circular and planar (block) analyses. 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 indicate the embankment for the WDA has adequate factors of safety given the strength parameters used and the conditions analyzed.

A summary of the lowest factors of safety for each case analyzed for the WDA is included in Table 5 below.

Table 5: Slope Stability Analysis Results Summary

Waste Disposal Area		
Case	**Pool Elevation**	**Factor of Safety**
1 - Steady Sate, Maximum **Storage** Pool Block - 257.73(e)(1)(i)	679 ft MSL	2.6
2 - Steady Sate, Maximum **Storage** Pool Block Seismic - 257.73(e)(1)(iii)	679 ft MSL	1.8
3 - Steady Sate, Maximum **Storage** Pool Rotational - 257.73(e)(1)(i)	679 ft MSL	2.7
4 - Steady Sate, Maximum **Storage** Pool Rotational Seismic - 257.73(e)(1)(iii)	679 ft MSL	1.8
5 - Steady State, Maximum **Surcharge** Pool Block - 257.73(e)(1)(ii)	681 ft MSL	2.6
6 - Steady State, Maximum **Surcharge** Pool Block Seismic - 257.73(e)(1)(iii)	681 ft MSL	1.7
7 - Steady State, Maximum **Surcharge** Pool Rotational - 257.73(e)(1)(ii)	681 ft MSL	2.8
8 - Steady State, Maximum **Surcharge** Pool Rotational Seismic - 257.73(e)(1)(iii)	681 ft MSL	1.8

Models from the slope stability analyses are included in Appendix A.

4.2 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), which uses CPT data (Youd et al. 2001; Robertson and Wride 1998). The calculated factor of safety against seismically-induced liquefaction is shown in Appendix B and was calculated to be greater than 1.2 throughout the depth of the embankments and underlying foundation in the evaluated CPT soundings (Golder, 2012) for the considered earthquake loading, see Figure 2. These screening-level results indicate that the embankments and foundation soils for the WDA are not susceptible to seismically-induced liquefaction for the seismic loading considered.

5. CONCLUSIONS AND RECOMMENDATIONS

Based on our review of the information provided by NIPSCO, onsite observations, and the results of the structural stability assessment, one structural stability deficiency was identified in the WDA surface impoundment during this assessment. As a result, it is recommended that NIPSCO remedy the deficiency by improving the size of the auxiliary spillway, operationally controlling the water level in the WDA, or implementing an equivalent engineering or operational control.

Based on this same information and on our analyses, the calculated factor of safety through the critical cross section in the WDA surface impoundment meets or exceeds the minimum values listed in §257.73(e)(1)(i)-(iv).

6. 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 the WDA at the R.M. Schahfer Generating Station.

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7. REFERENCES

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FIGURES

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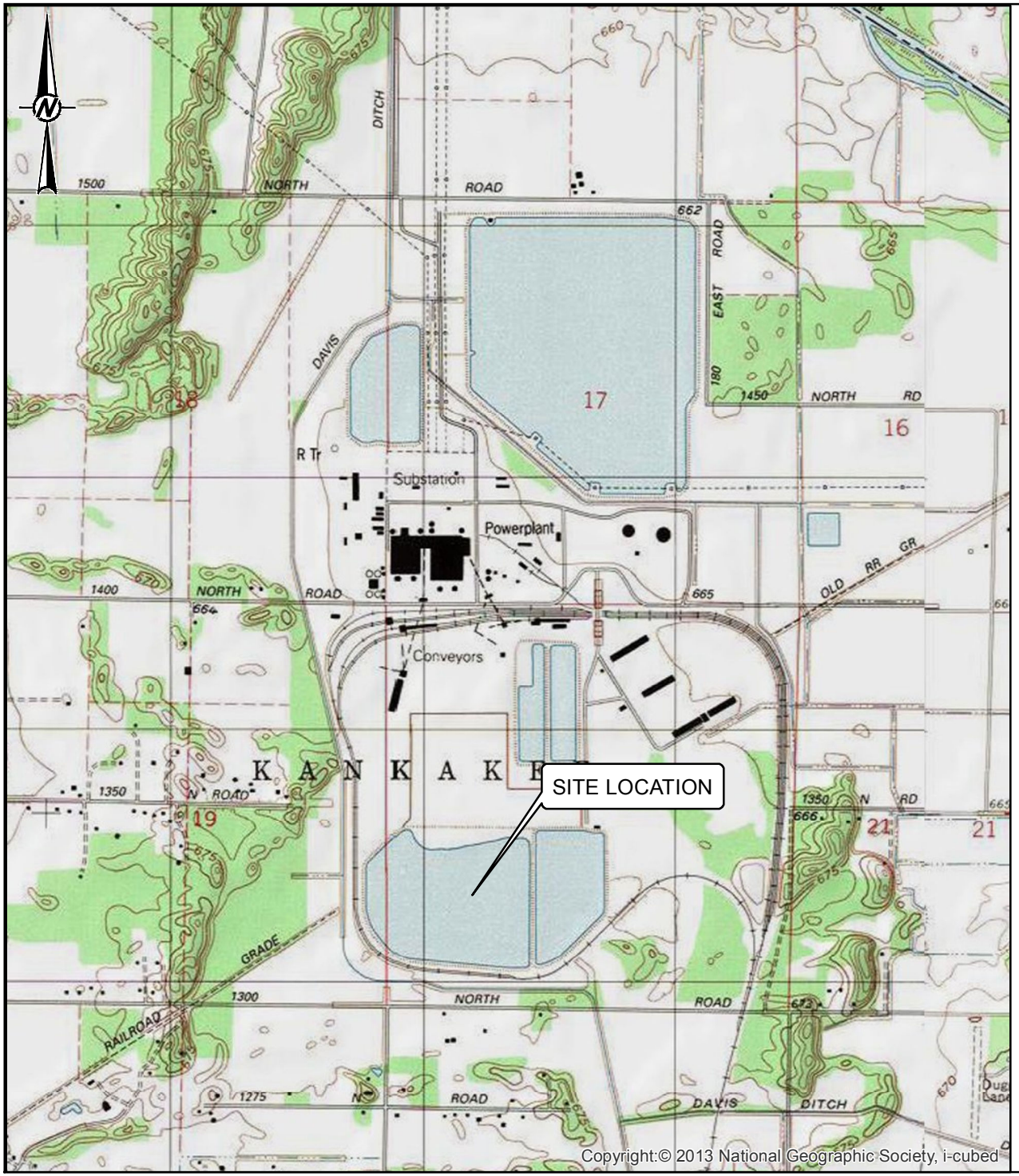

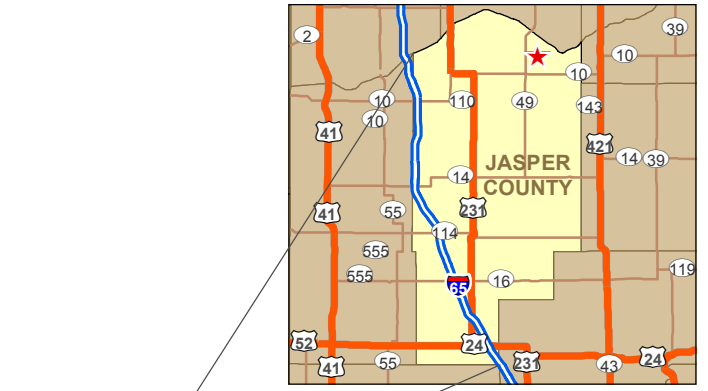

CLIENT
NIPSCO
SCHAHFER GENERATING STATION
WHEATFIELD, INDIANA

PROJECT
STRUCTURAL STABILITY ASSESSMENT

TITLE
SITE LOCATION MAP

CONSULTANT
Golder Associates

YYYY-MM-DD	2016-09-07
PREPARED	DJC
DESIGN	TDJ
REVIEW	TDJ
APPROVED	DML

PROJECT No.	CONTROL	Rev.	FIGURE
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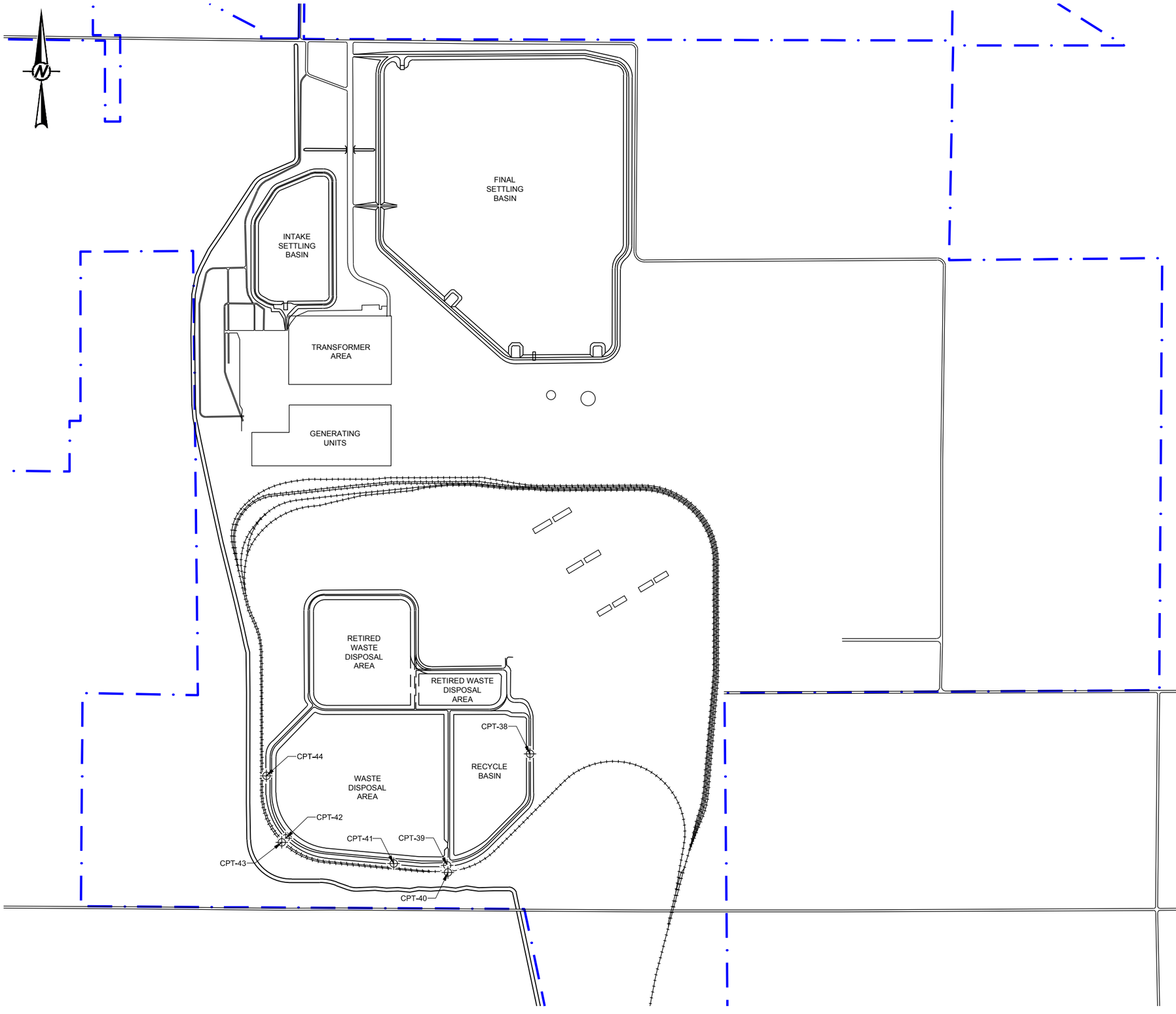

LEGEND

- APPROXIMATE PROPERTY BOUNDARY
- RAILROAD TRACKS
- CPT-XX 2011 / 2012 GEOTECHNICAL INVESTIGATION CPT BOREHOLE LOCATION

REFERENCE

BASE MAP TAKEN FROM MARBACH, BRADY & WEAVER ENGINEERING & SURVEYING; EMBANKMENT ELEVATION SURVEY; DRAWING NO. A-31565; FILE NO. 0221-2011.DWG; DATED 2011-12-30; DELIVERED IN .DWG FORMAT

0 100 200
1" = 200' FEET

CLIENT
NIPSCO
SCHAHFER GENERATING STATION
WHEATFIELD, INDIANA

PROJECT
STRUCTURAL STABILITY ASSESSMENT

TITLE
SITE PLAN

CONSULTANT
Golder Associates

YYYY-MM-DD	2016-09-07
DESIGNED	TDJ
PREPARED	DJC
REVIEWED	TDJ
APPROVED	DML

PROJECT NO.	CONTROL	REV.	FIGURE
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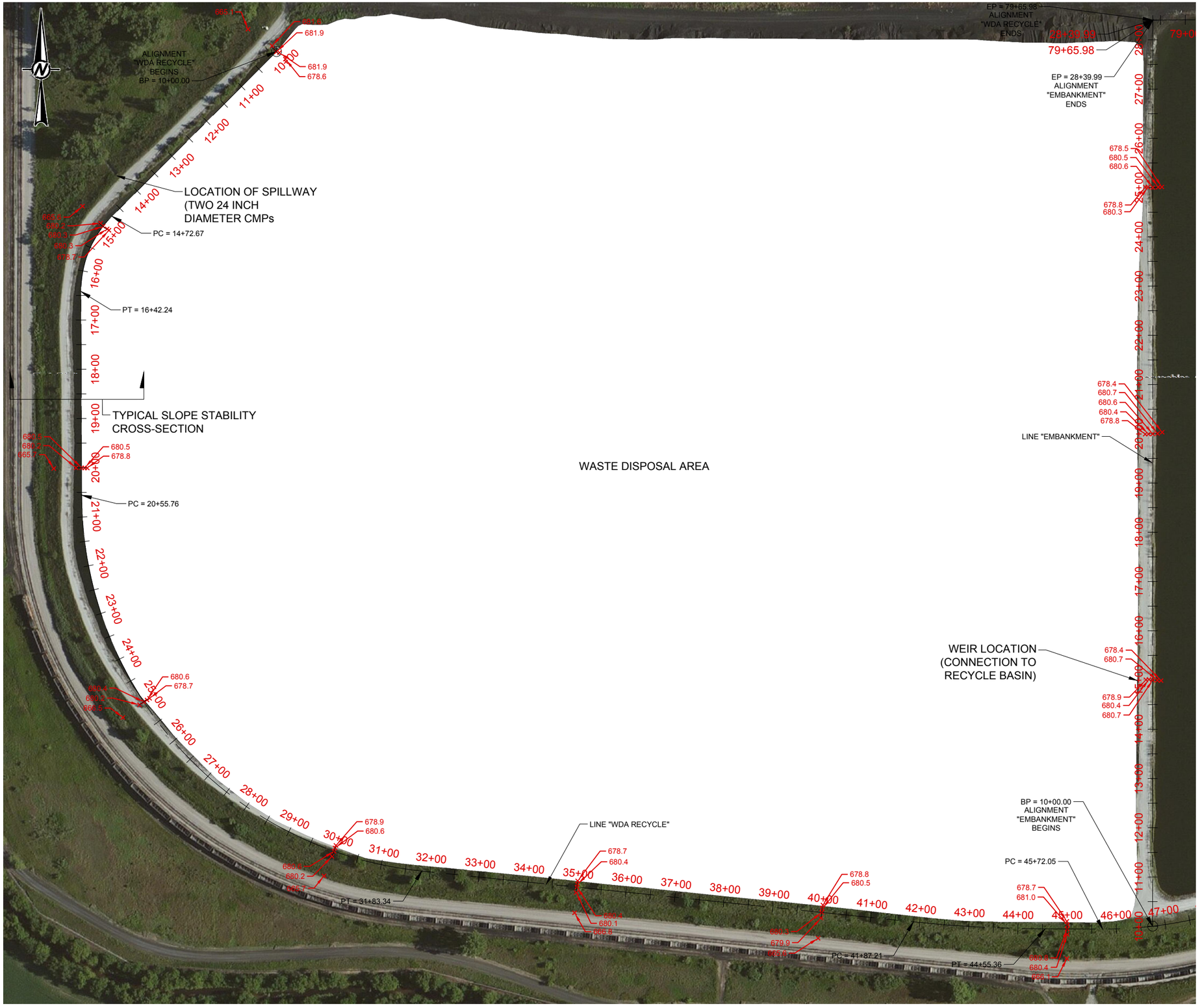

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0 100 200
1" = 200' FEET

CLIENT
NIPSCO
SCHAHFER GENERATING STATION
WHEATFIELD, INDIANA

PROJECT
STRUCTURAL STABILITY ASSESSMENT

TITLE
WASTE DISPOSAL AREA

CONSULTANT
Golder Associates

YYYY-MM-DD	2016-09-07
DESIGNED	TDJ
PREPARED	DJC
REVIEWED	TDJ
APPROVED	DML

PROJECT NO.	CONTROL	REV.	FIGURE
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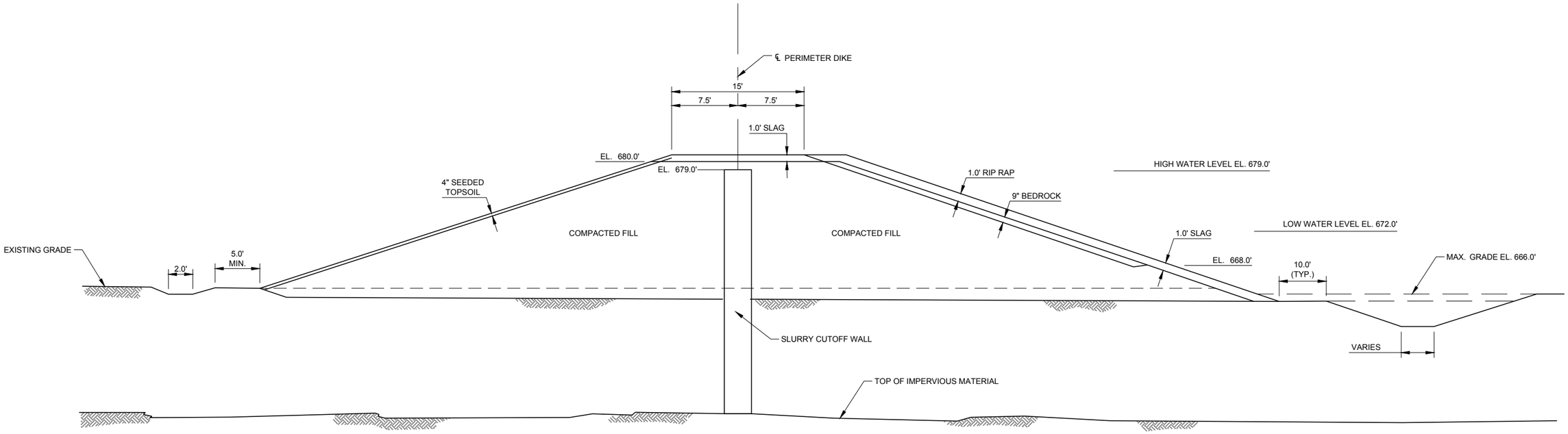

REFERENCE

FROM SARGENT & LUNDY CONSTRUCTION; DRAWING C-19; SETTLING BASINS DIKEWORK SECTIONS AND DETAILS; SHEET - 1; REV. F; 1982-07-13.

CLIENT
NIPSCO
SCHAHFER GENERATING STATION
WHEATFIELD, INDIANA

PROJECT
STRUCTURAL STABILITY ASSESSMENT

TITLE
TYPICAL CROSS SECTION

CONSULTANT
Golder Associates

YYYY-MM-DD	2016-09-07
DESIGNED	TDJ
PREPARED	DJC
REVIEWED	TDJ
APPROVED	DML

PROJECT NO.	CONTROL	REV.	FIGURE
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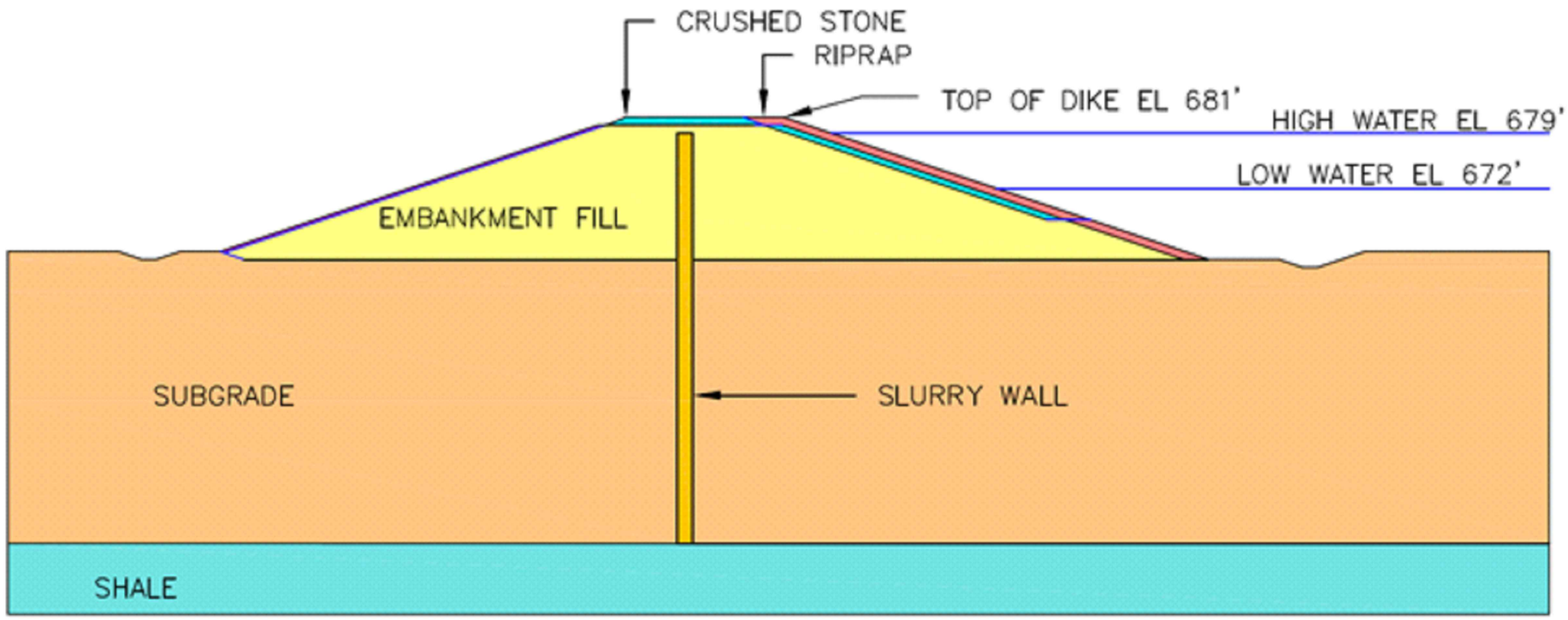
WASTE DISPOSAL AREA
TYPICAL PROFILE



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FROM SARGENT & LUNDY CONSTRUCTION; DRAWING C-19; SETTLING BASINS DIKEWORK SECTIONS AND DETAILS; SHEET - 1; REV. F; 1982-07-13.

CLIENT
NIPSCO
SCHAHFER GENERATING STATION
WHEATFIELD, INDIANA

PROJECT
STRUCTURAL STABILITY ASSESSMENT

TITLE
TYPICAL GEOTECHNICAL MODEL AT THE WASTE DISPOSAL AREA EMBANKMENT (NOT TO SCALE)

CONSULTANT
Golder Associates

YYYY-MM-DD	2016-09-07
DESIGNED	TDJ
PREPARED	TDJ
REVIEWED	TDJ
APPROVED	DML

PROJECT NO.	CONTROL	REV.	FIGURE
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APPENDIX A
SLOPE STABILITY ANALYSIS RESULTS


2008 United States National Seismic Hazard Maps - Windows Internet Explorer
http://earthquake.usgs.gov/hazards/products/conterminous/2008/
Favorites
Popular Sites - Choose yo...
Suggested Sites
Web Slice Gallery
2008 United States National Seismic Hazard Maps
Page
Safety
Tools
USGS
science for a changing world
USGS Home
Contact USGS
Search USGS
Earthquake Hazards Program
Home
About Us
Contact Us
Search
EARTHQUAKES
HAZARDS
LEARN
PREPARE
MONITORING
RESEARCH
NSHM Home
Maps and Data
2008 Update
Maps
Data
Software
Fault Parameters
Catalogs
Documentation
2002 Update
1996 Update
2008 United States National Seismic Hazard Maps
Open File Report 2008-1128
Ground Motion Attenuation Relations for the CEUS
Fact Sheet
Revision I, May 2008
Revision II, December 2008 (The Basis for Design Maps)
Revision III, January 2010
The 2008 U.S. Geological Survey (USGS) National Seismic Hazard Maps display earthquake ground motions for various probability levels across the United States and are applied in seismic provisions of building codes, insurance rate structures, risk assessments, and other public policy. This update of the maps incorporates new findings on earthquake ground shaking, faults, seismicity, and geodesy. The resulting maps are derived from seismic hazard curves calculated on a grid of sites across the United States that describe the frequency of exceeding a set of ground motions.
The USGS National Seismic Hazard Mapping Project developed these maps by incorporating information on potential earthquakes and associated ground shaking obtained from interaction in science and engineering workshops involving hundreds of participants, review by several science organizations and State surveys, and advice from two expert panels.
The new probabilistic hazard maps represent an update of the 2002 seismic hazard maps developed by Frankel and others (2002), which used the methodology developed for the 1996 version of the maps (Frankel and others, 1996). Algermissen and Perkins (1978) published the first probabilistic seismic hazard map of the United States which was updated in Algermissen and others (1990). The national seismic maps represent our assessment of the "best available science" in earthquake hazards estimation for the United States.
DOI and USGS link policies apply.
Share this page: Facebook Twitter Google Email
EARTHQUAKES
Past - Present - Future
Info by State
HAZARDS
Faults
Hazard Maps & Data
LEARN
Earthquake Topics
FAQ
PREPARE
How do I prepare?
A quake just happened...
MONITORING
ANSS - United States
GSN - World
RESEARCH
Projects
Data
Internet | Protected Mode: On
75%

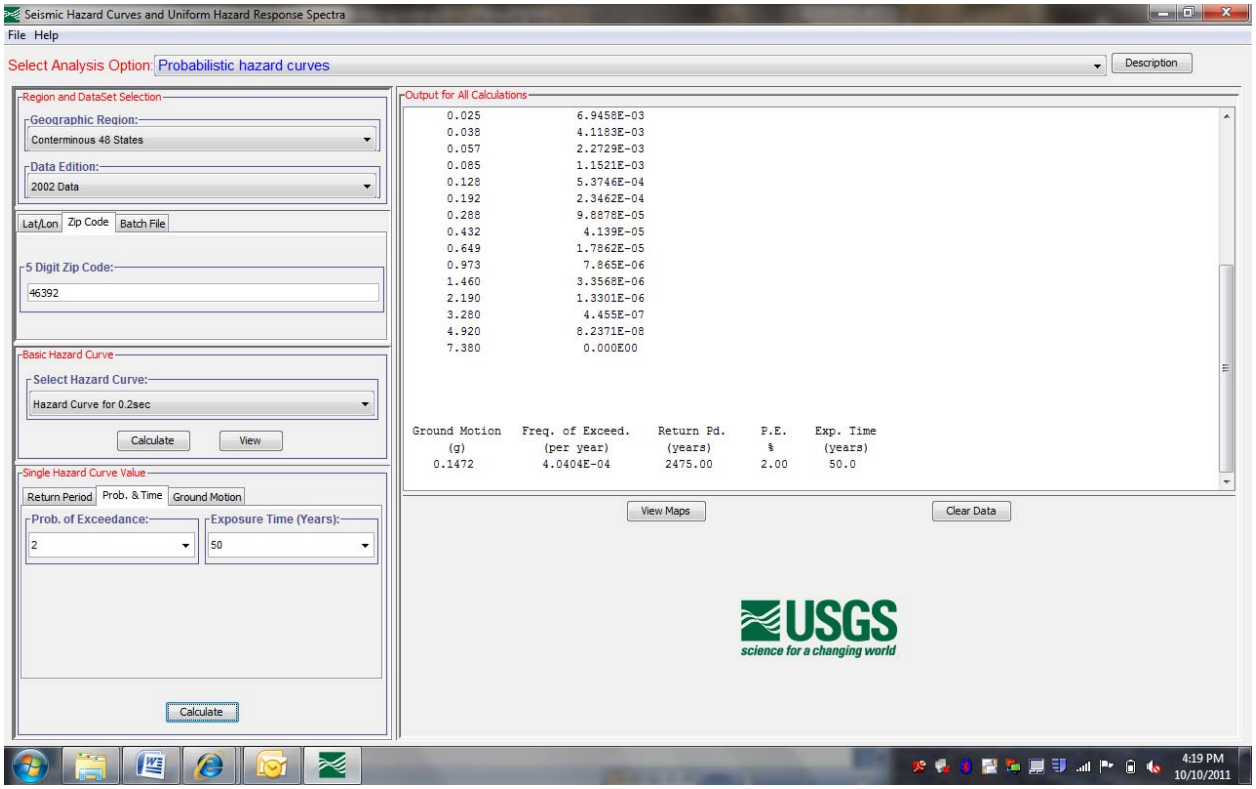
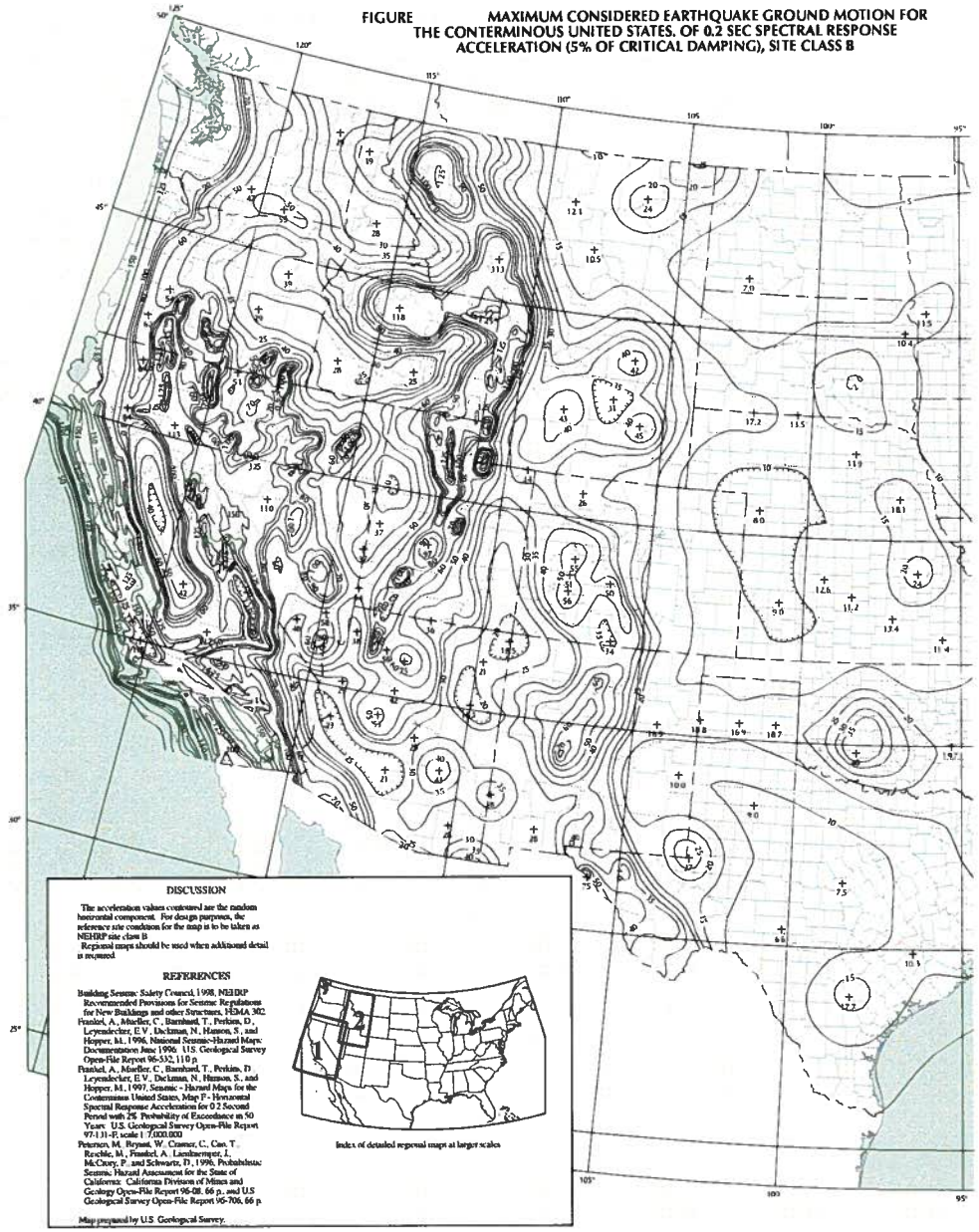

Seismic Hazard Curves and Uniform Hazard Response Spectra
File Help
Select Analysis Option: Probabilistic hazard curves
Description
Region and DataSet Selection
Geographic Region:
Conterminous 48 States
Data Edition:
2002 Data
Lat/Lon
Zip Code
Batch File
5 Digit Zip Code:
46392
Basic Hazard Curve
Select Hazard Curve:
Hazard Curve for 0.2sec
Calculate
View
Single Hazard Curve Value
Return Period
Prob. & Time
Ground Motion
Prob. of Exceedance:
2
Exposure Time (Years):
50
Calculate
Output for All Calculations
0.025 6.9458E-03
0.038 4.1183E-03
0.057 2.2729E-03
0.085 1.1521E-03
0.128 5.3746E-04
0.192 2.3462E-04
0.288 9.8878E-05
0.432 4.139E-05
0.649 1.7862E-05
0.973 7.865E-06
1.460 3.3568E-06
2.190 1.3301E-06
3.280 4.455E-07
4.920 8.2371E-08
7.380 0.000E00
Ground Motion (g) 0.1472
Freq. of Exceed. (per year) 4.0404E-04
Return Pd. (years) 2475.00
P.E. % 2.00
Exp. Time (years) 50.0
View Maps
Clear Data
USGS
science for a changing world
4:19 PM
10/10/2011

FIGURE MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR THE CONTERMINOUS UNITED STATES. OF 0.2 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B



DISCUSSION

The acceleration values contoured are the random horizontal component. For design purposes, the reference site condition for the map is to be taken as NEHRP site class B.

Regional maps should be used when additional detail is required.

REFERENCES

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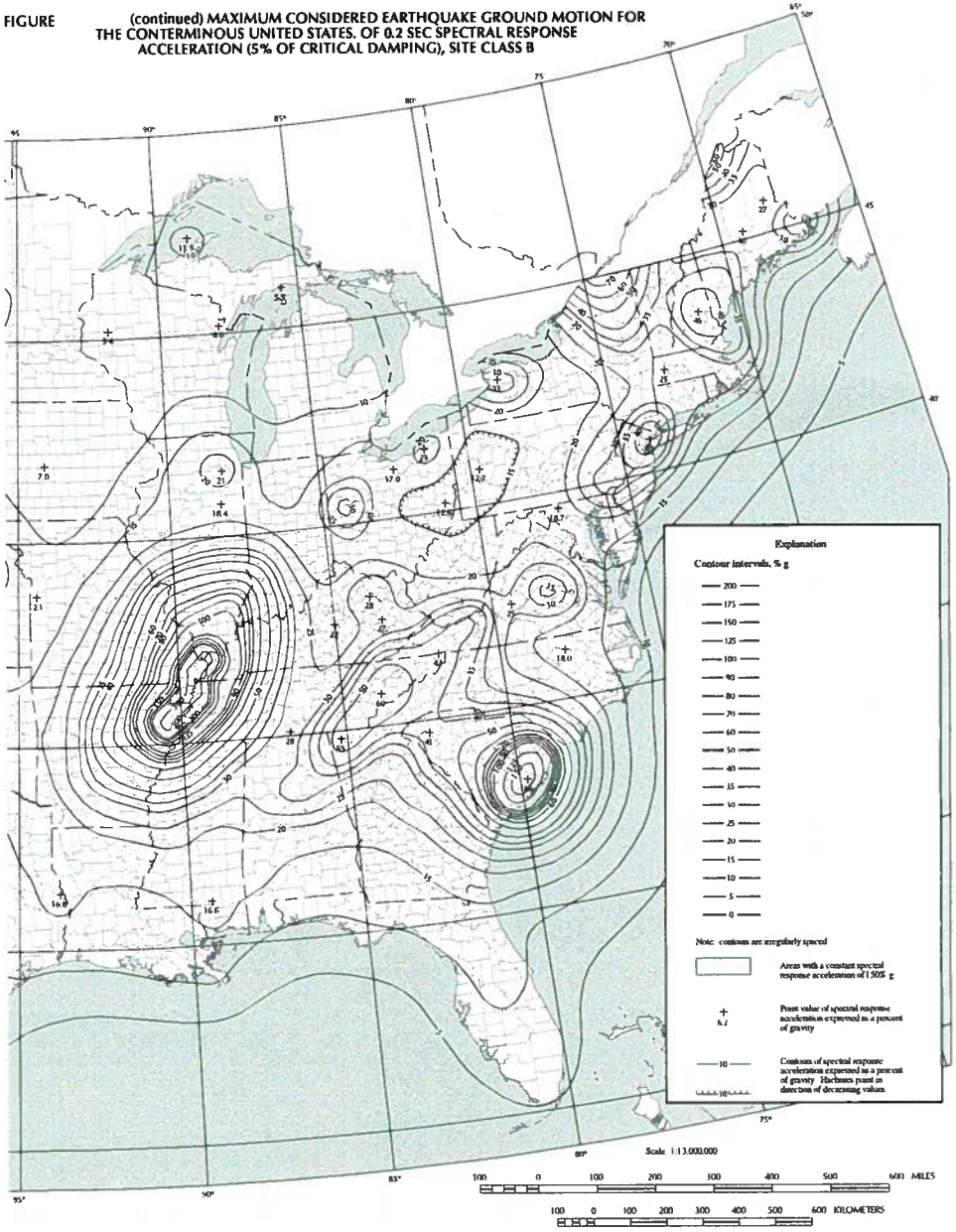
Frankel, A., Mueller, C., Barnhard, T., Perkins, D., Leyendecker, E.V., Dickman, N., Hanson, S., and Hopper, M., 1997, Seismic - Hazard Maps for the Conterminous United States, Map F - Horizontal Spectral Response Acceleration for 0.2 Second Period with 2% Probability of Exceedance in 50 Years: U.S. Geological Survey Open-File Report 97-131-F, scale 1:7,000,000.

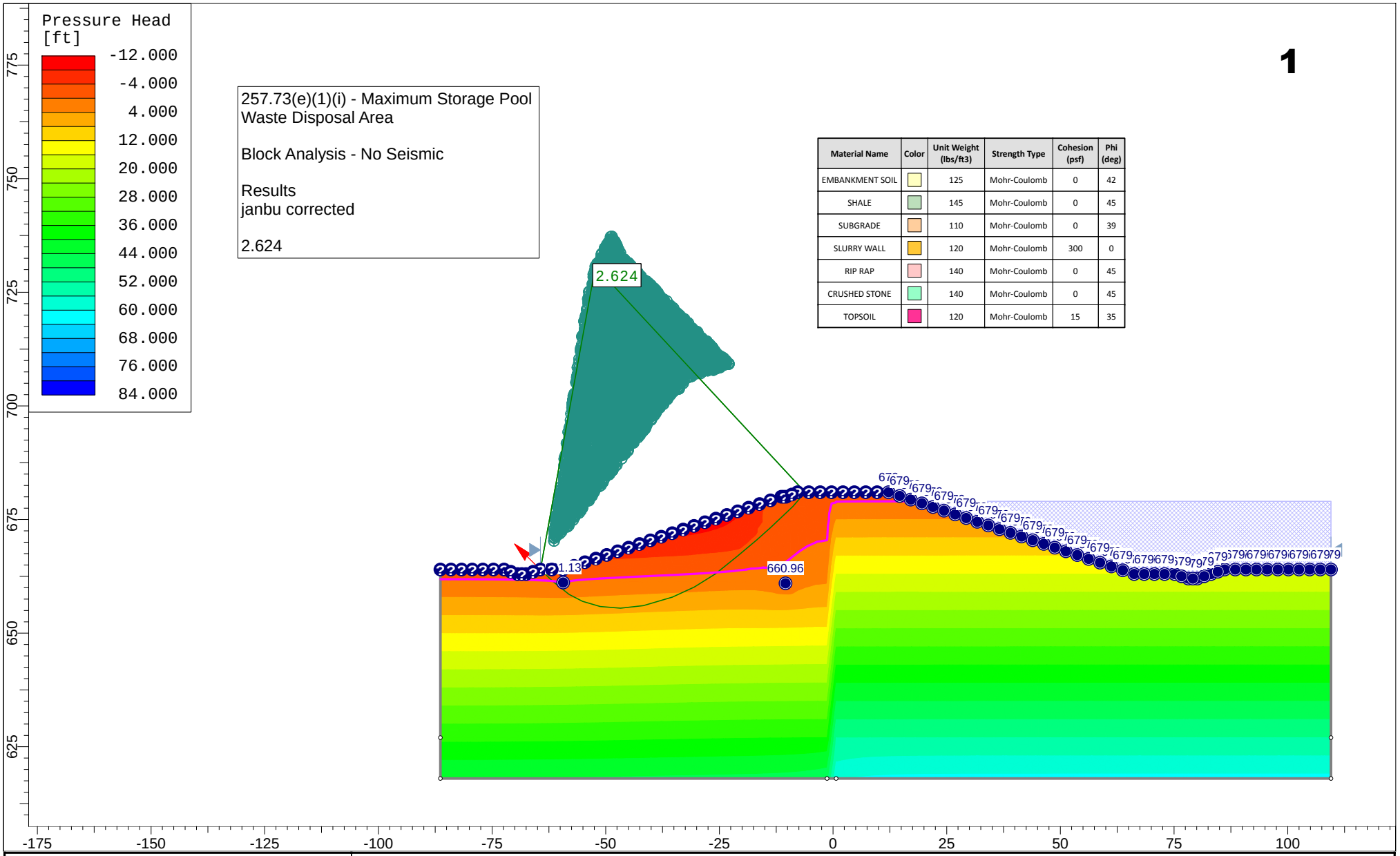
Petersen, M., Bryant, W., Cramer, C., Cao, T., Reichle, M., Frankel, A., Lienkaemper, J., McCrory, P., and Schwartz, D., 1996, Probabilistic Seismic Hazard Assessment for the State of California: California Division of Mines and Geology Open-File Report 96-08, 66 p., and U.S. Geological Survey Open-File Report 96-706, 66 p.

Index of detailed regional maps at larger scales

Map prepared by U.S. Geological Survey.

FIGURE (continued) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR THE CONTERMINOUS UNITED STATES. OF 0.2 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

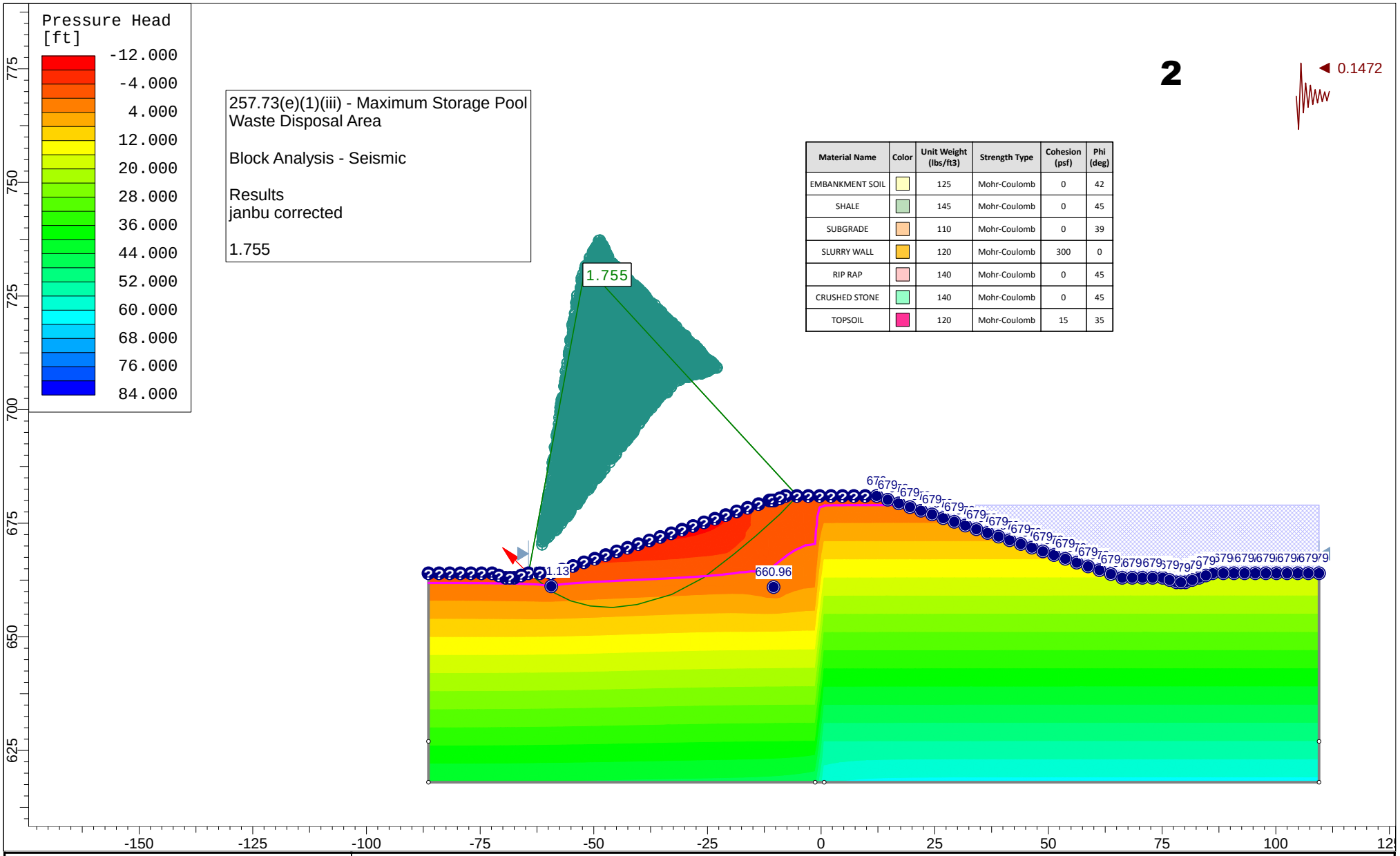




Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
EMBANKMENT SOIL		125	Mohr-Coulomb	0	42
SHALE		145	Mohr-Coulomb	0	45
SUBGRADE		110	Mohr-Coulomb	0	39
SLURRY WALL		120	Mohr-Coulomb	300	0
RIP RAP		140	Mohr-Coulomb	0	45
CRUSHED STONE		140	Mohr-Coulomb	0	45
TOPSOIL		120	Mohr-Coulomb	15	35

Project	NIPSCO - WASTE DISPOSAL AREA		
Analysis Description	MAXIMUM STORAGE POOL FOS		
Drawn By	T. JOHNSON	Scale 1:350	Company GOLDER
Date	9/1/16		File Name WDA 679.sli

SLIDEINTERPRET 7.018



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SHALE		145	Mohr-Coulomb	0	45
SUBGRADE		110	Mohr-Coulomb	0	39
SLURRY WALL		120	Mohr-Coulomb	300	0
RIP RAP		140	Mohr-Coulomb	0	45
CRUSHED STONE		140	Mohr-Coulomb	0	45
TOPSOIL		120	Mohr-Coulomb	15	35

SLIDEINTERPRET 7.018

Project	NIPSCO - WASTE DISPOSAL AREA		
Analysis Description	MAXIMUM STORAGE POOL FOS		
Drawn By	T. JOHNSON	Scale	1:350
Company	GOLDER		
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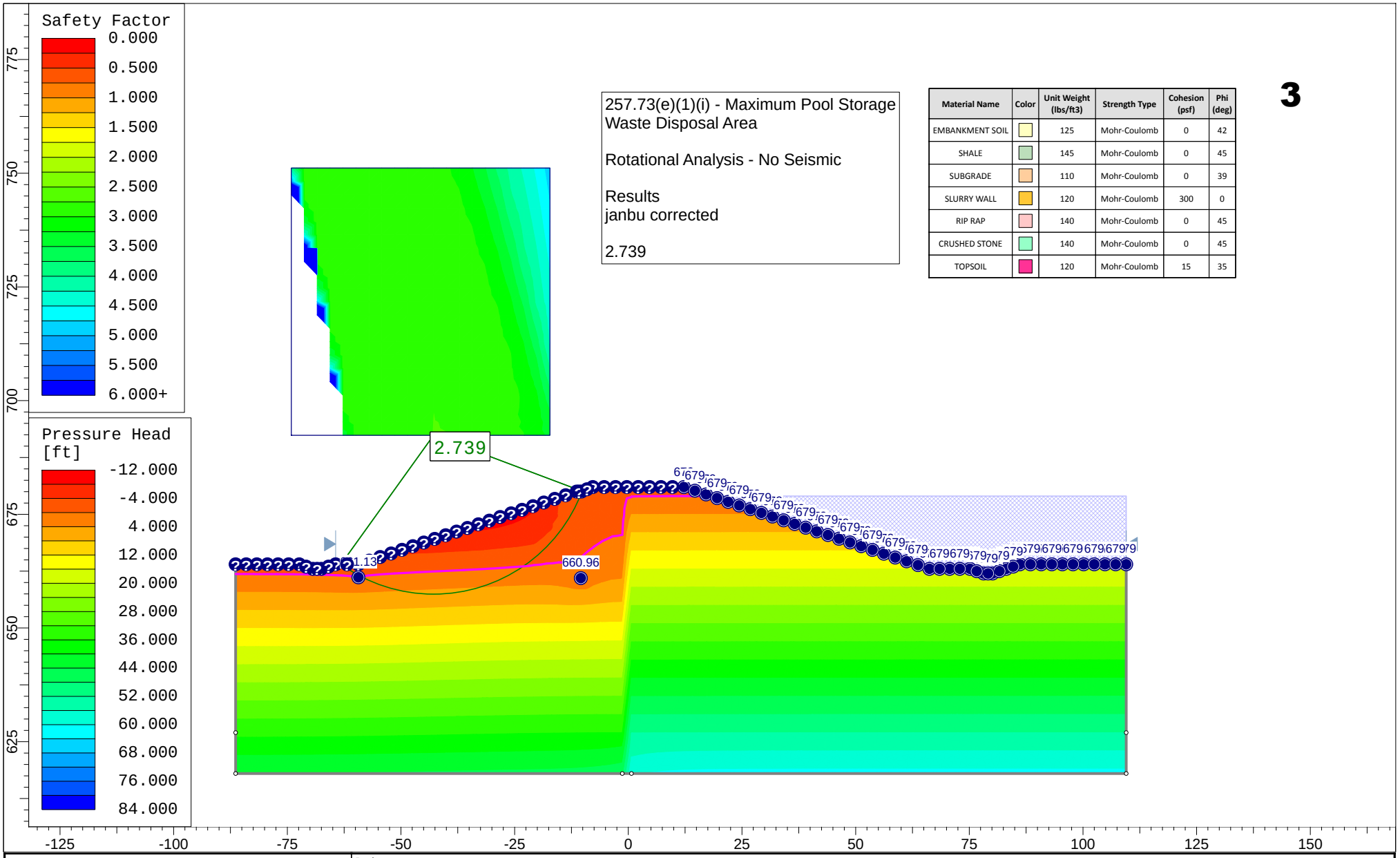

257.73(e)(1)(i) - Maximum Pool Storage
Waste Disposal Area

Rotational Analysis - No Seismic

Results
janbu corrected

2.739

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
EMBANKMENT SOIL		125	Mohr-Coulomb	0	42
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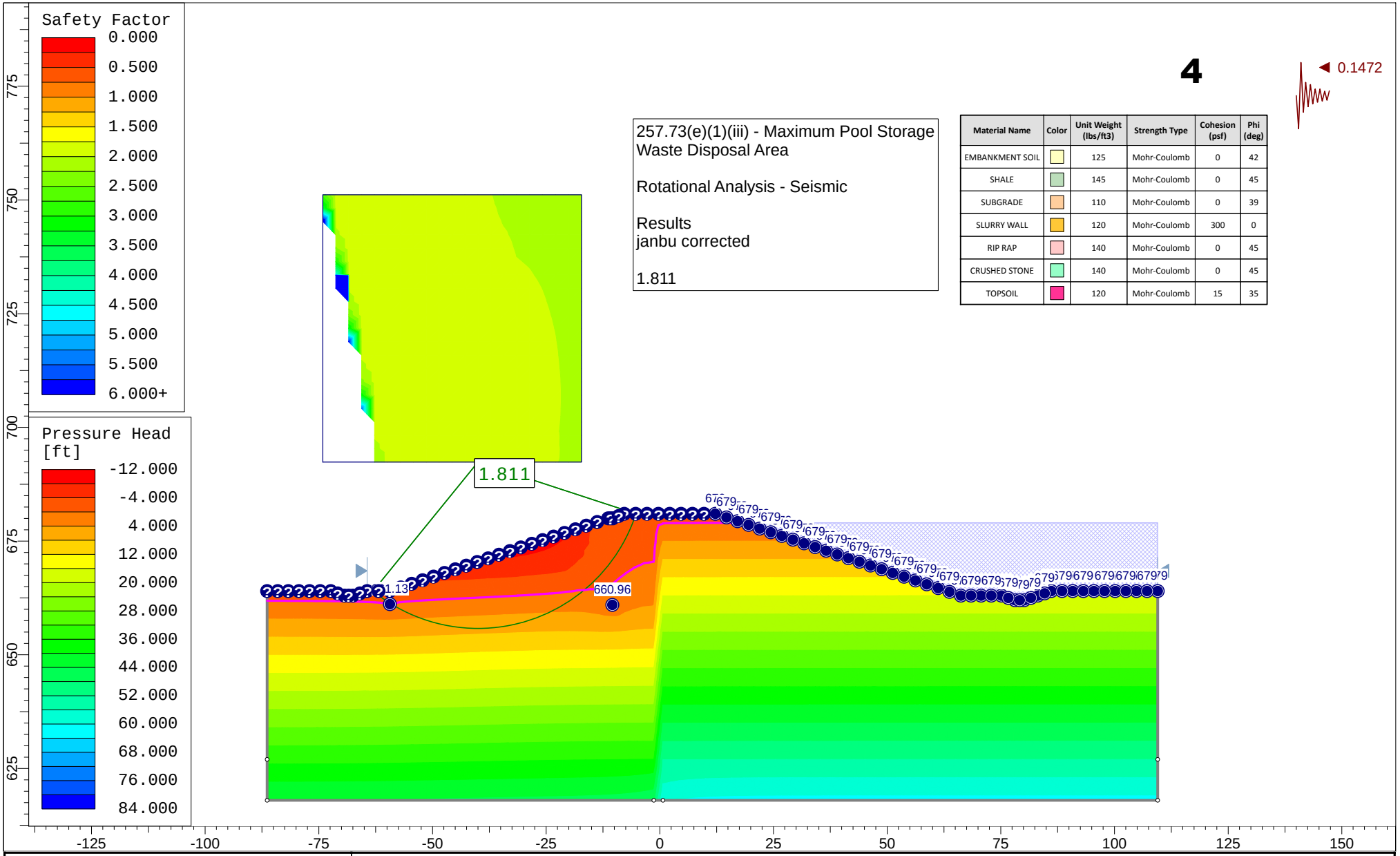
RIP RAP		140	Mohr-Coulomb	0	45
CRUSHED STONE		140	Mohr-Coulomb	0	45
TOPSOIL		120	Mohr-Coulomb	15	35

3

SLIDEINTERPRET 7.018

Project	NIPSCO - WASTE DISPOSAL AREA		
Analysis Description	MAXIMUM STORAGE POOL FOS		
Drawn By	T. JOHNSON	Scale 1:350	Company GOLDER
Date	9/1/16		File Name WDA 679 - ROT.slim



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SUBGRADE		110	Mohr-Coulomb	0	39
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TOPSOIL		120	Mohr-Coulomb	15	35

Project	NIPSCO - WASTE DISPOSAL AREA		
Analysis Description	MAXIMUM STORAGE POOL FOS		
Drawn By	T. JOHNSON	Scale	1:350
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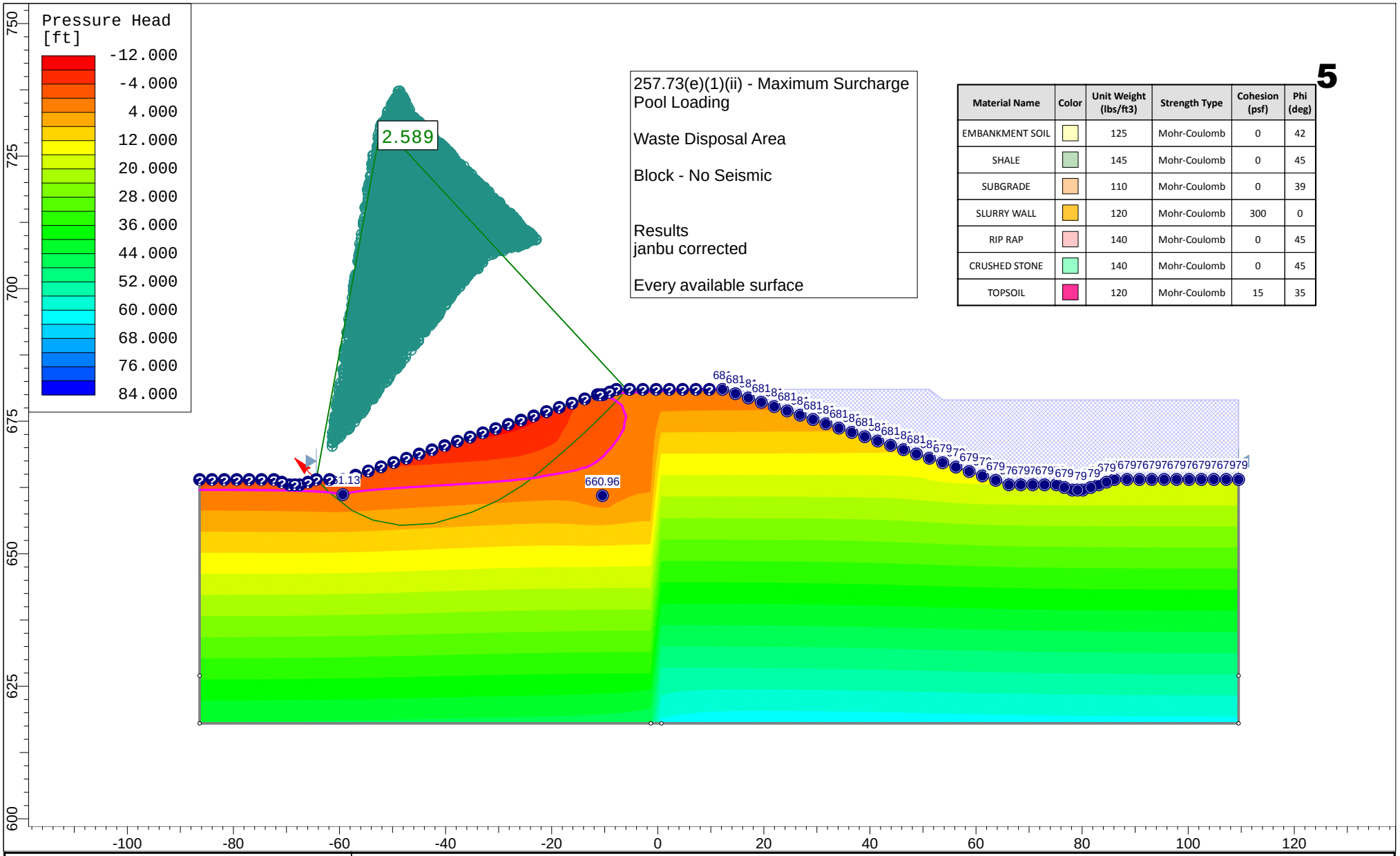
SLIDEINTERPRET 7.018



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Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
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RIP RAP		140	Mohr-Coulomb	0	45
CRUSHED STONE		140	Mohr-Coulomb	0	45
TOPSOIL		120	Mohr-Coulomb	15	35

SLIDEINTERPRET 7.018

Project	NIPSCO - WASTE DISPOSAL AREA		
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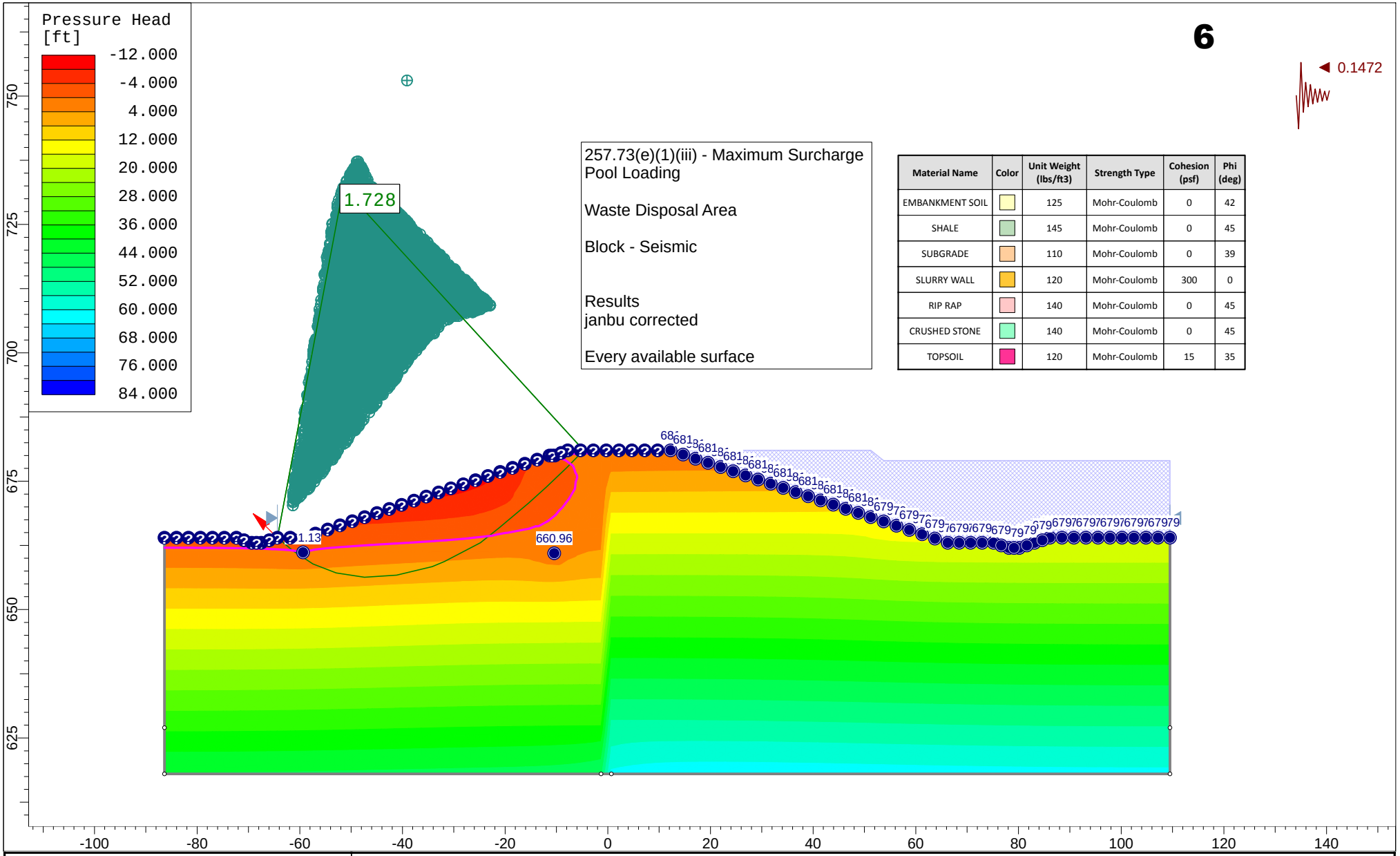

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
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SHALE		145	Mohr-Coulomb	0	45
SUBGRADE		110	Mohr-Coulomb	0	39
SLURRY WALL		120	Mohr-Coulomb	300	0
RIP RAP		140	Mohr-Coulomb	0	45
CRUSHED STONE		140	Mohr-Coulomb	0	45
TOPSOIL		120	Mohr-Coulomb	15	35

rocscience

SLIDEINTERPRET 7.018

Project	NIPSCO - WASTE DISPOSAL AREA		
Analysis Description	MAXIMUM STORAGE POOL FOS		
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Company	GOLDER		
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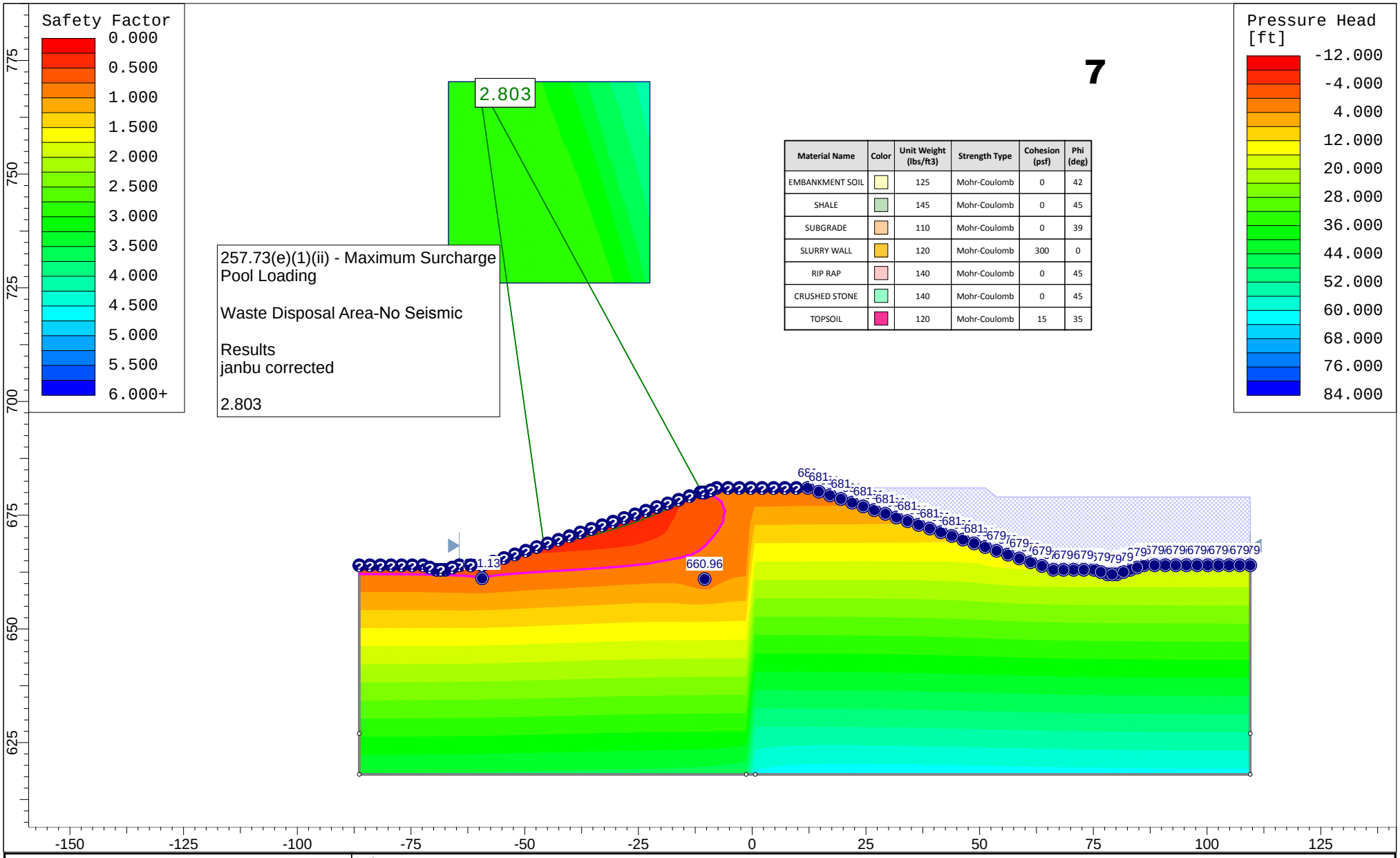
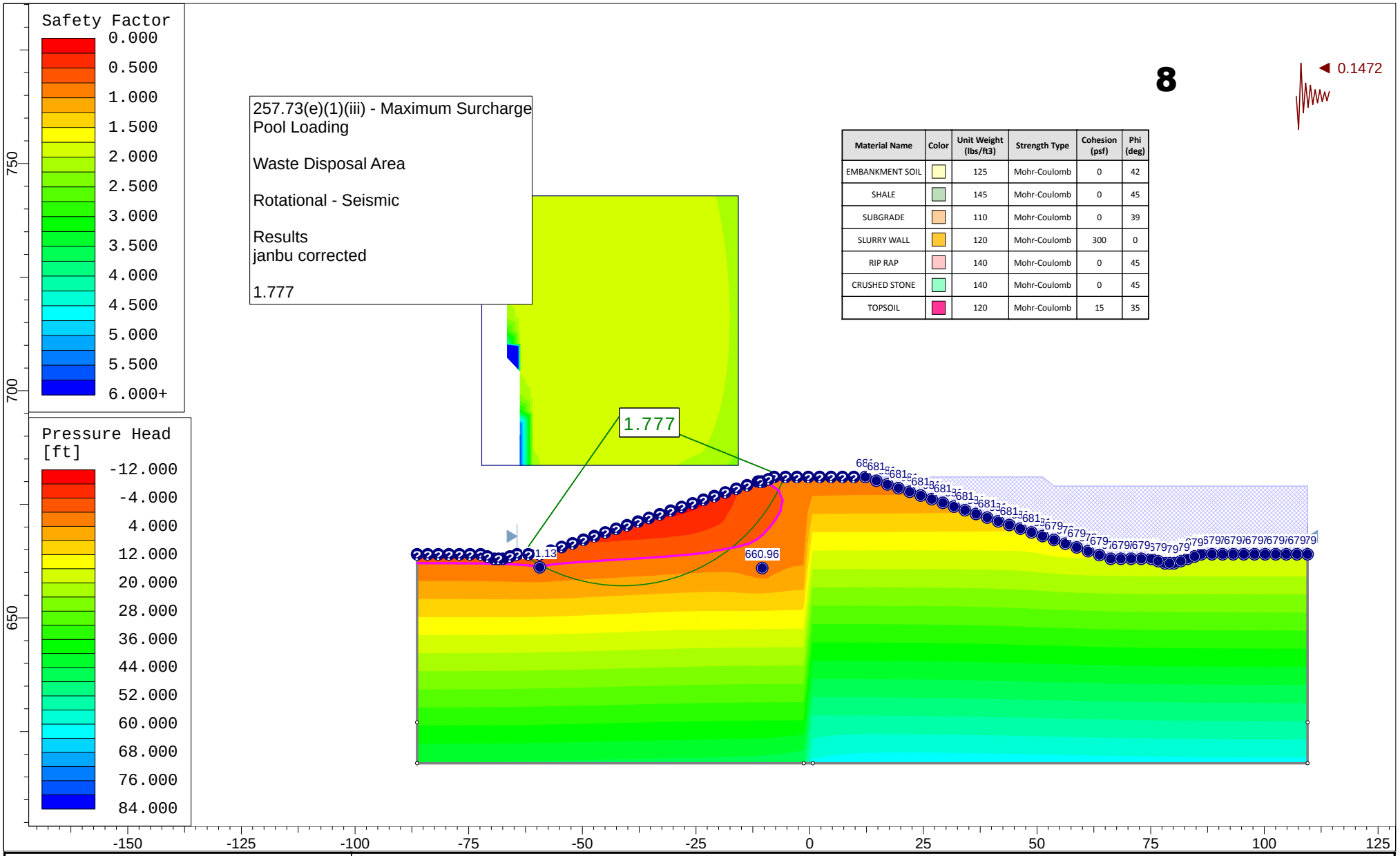


Project	NIPSCO - WASTE DISPOSAL AREA		
Analysis Description	MAXIMUM STORAGE POOL FOS		
Drawn By	T. JOHNSON	Scale 1:350	Company GOLDER
Date	9/1/16		File Name WDA 681 - ROT.slim

SLIDEINTERPRET 7.018



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
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CRUSHED STONE		140	Mohr-Coulomb	0	45
TOPSOIL		120	Mohr-Coulomb	15	35

Project	NIPSCO - WASTE DISPOSAL AREA		
Analysis Description	MAXIMUM STORAGE POOL FOS		
Drawn By	T. JOHNSON	Scale	1:350
Company	GOLDER		
Date	9/1/16	File Name	WDA 681 - ROT.slim

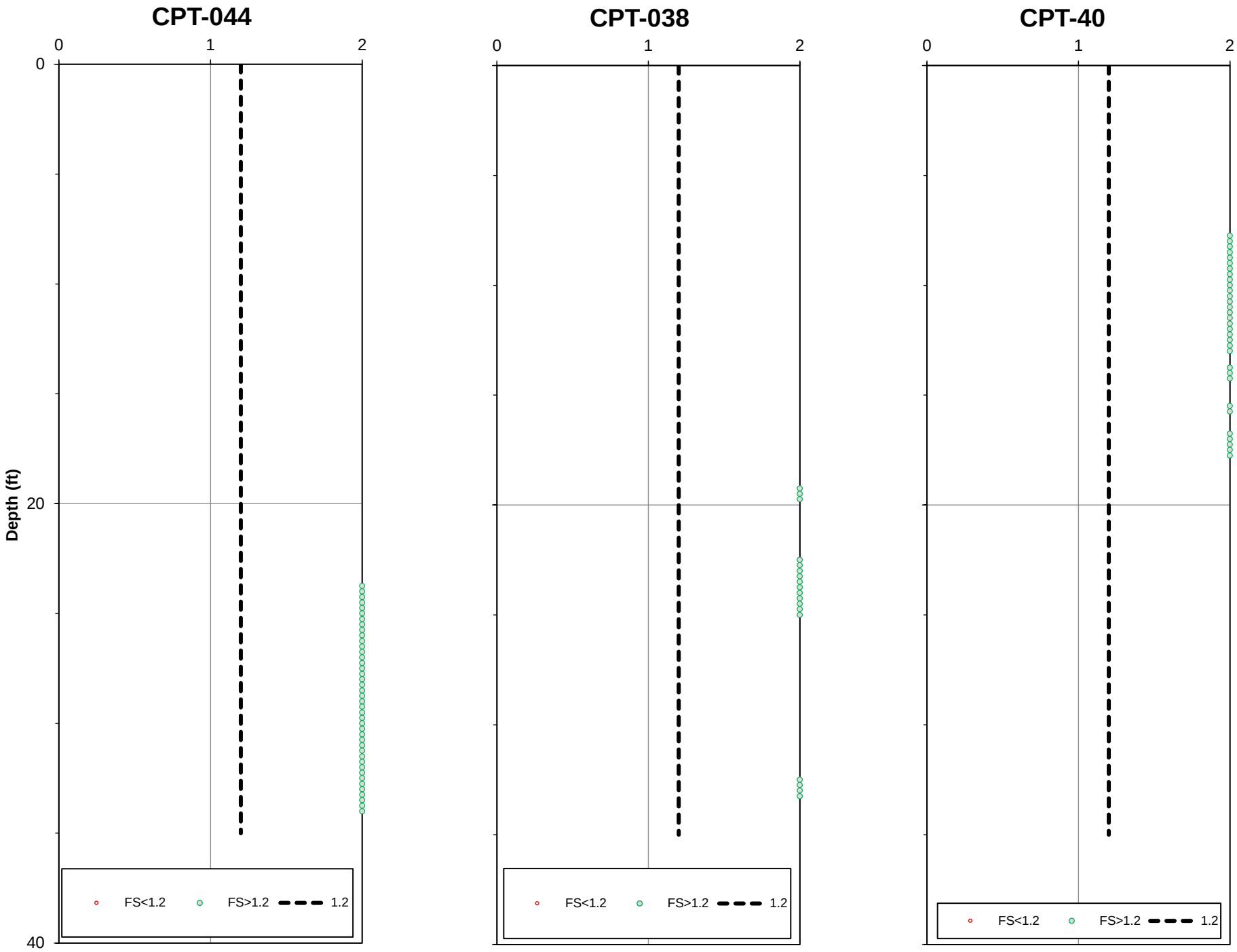
SLIDEINTERPRET 7.018

APPENDIX B
LIQUEFACTION POTENTIAL ANALYSIS RESULTS

Project:	WDA RCRA	**Test Type:**	CPTU	**Golder Eng:**	AK	**Design EQ 1**	
Location:	Wheatfield IN	**Device:**	15 cm^2, Type 2 filter	**Check**	AF	**Magnitude:**	4.8 g
Client:	NIPSCO	**Standard:**	ASTM D5778	**Review:**	TDJ		
Proj No.:	1651599	**Push Co.:**	STRATIGRAPHICS	**Max Depth:**	34 ft	**Date:**	10/5/16
Termination:	Target Depth	**Operator:**	--				

CPT ID:	CPT-044	**CPT ID:**	CPT-038	**CPT ID:**	CPT-040
Test Date:	6/6/2011	**Test Date:**	6/6/2011	**Test Date:**	6/6/2011
Northing:	2169979	**Northing:**	2170181	**Northing:**	2168808
Easting:	2968359	**Easting:**	2971542	**Easting:**	2970555
Elevation:	680.4 ft	**Elevation:**	680.3 ft	**Elevation:**	664.9 ft
a_{max}:	0.09 g	**a_{max}:**	0.09 g	**a_{max}:**	0.1 g
Water Table:	11.8 ft	**Water Table:**	10.5 ft	**Water Table:**	2.3 ft

CALCULATED LIQUEFACTION FACTOR OF SAFETY



Note: Factor of safeties (FS) greater than 2 are shown equal to 2.

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