REPORT

STRUCTURAL STABILITY AND SAFETY FACTOR ASSESSMENT per CCR Rule 257.73 (d) and (e)

NIPSCO, R.M. Schahfer Generating Station, Waste Disposal Area CCR Unit

Submitted to:

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October 4, 2021

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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 updated 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.

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1.0 INTRODUCTION

1.1 Purpose

The United States Environmental Protection Agency (EPA) published 40 CFR Part 257 – Coal Combustion Residuals (CCR) Final Rule (CCR RCRA Rule) in April 2015 to regulate the solid waste management of CCR generated at electric utilities. The CCR RCRA Rule requires that existing CCR surface impoundments meeting the requirements of Section 257.73(b) conduct initial and periodic structural stability assessments in accordance with Section 257.73(d), and safety factor assessments in accordance with Section 257.73(e). Per rule 257.73(b), this periodic 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 periodic 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 periodic 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 the 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 wall 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 receives primarily bottom ash from the generating station through pipes located at the northern end of the unit. Most of the deposited material is located in the northern half of the WDA. Due to size of the unit and settling/depositional properties of the materials, very little, if any, ash/slag is present in the southern half of the WDA. The east side of the WDA is common with the west side of the adjacent Recycle Settling Basin (RB). Water exits the WDA via an overflow weir (standpipe), to the RB, 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 WDA and the RB are hydraulically connected and the water level within these impoundments will seek equilibrium when the water level is above the invert elevation of the standpipe connecting the impoundments. A survey of the WDA was performed by Marbach, Brady and Weaver, Inc. in December 2011 (Marbach, 2011), see Figure 3. The auxiliary spillway was modified and construction completed in November of 2017. The modifications included the removal of the former closed-conduit spillway and the construction of a concrete open-channel spillway with a concrete down-chute and riprap armoring at the toe of the embankment. It is located near the northwest corner of the WDA.

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.
	February 2013	Golder Associates Inc.

Document	Date	Author
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		
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.
Annual RCRA CCR Unit Inspection Reports – NIPSCO Schahfer Generating Station	January 2016 to January 2021	Golder Associates Inc.
Inflow Design Flood Control System Plan per CCR Rule 257.82 NIPSCO, R.M. Schahfer Generating Station Waste Disposal Area CCR Surface Impoundment	October 2016 and October 2021	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 and June 2021	Golder Associates Inc.

2.0 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 wall, which is located along the embankment centerline. The slurry wall 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 and structures are located above the top of the slurry wall 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 wall extending down to shale bedrock at depth; inlet and outlet pipes that do not penetrate the slurry wall; 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 soil) 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.

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.)	Effective Cohesion (psf)	In-situ Unit Weight (pcf)	Undrained Shear Strength (psf)	Layer Thick-ness (ft)
Embankment Soil	42	0	125	NA	Varies
Topsoil	35	15	120	NA	0.5
Subgrade	39	0	110	NA	Varies
Slurry Wall	NA (0)	300	120	NA	Varies
Riprap	45	0	140	NA	1
Crushed Stone	45	0	140	NA	Varies
Shale	45	0	145	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 used 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. The auxiliary spillway was improved and constructed in 2017.

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:	677.5 ft MSL based on invert elevations of improved spillway structure
Height:	17 feet
Surface Area:	75.5 acres
Reservoir Volume:	1,530 acre-feet

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

The CCR Rule requires an initial and periodic structural stability assessments be conducted by a qualified professional engineer (QPE) to document whether the design, construction, operation and maintenance is consistent with recognized and generally accepted good engineering practices for the maximum volume of CCR and CCR wastewater that can be impounded therein. The following sections provide documentation on the initial structural stability assessment and rely mainly on the recent and historic annual inspections performed at the site. The most recent inspection was completed by Golder on October 27, 2020 while the initial structural stability assessment was performed in September of 2016 (Golder, September 2016). An annual inspection is scheduled for the end of October 2021. If there are any changes in WDA conditions observed during the October 2021 inspection, this report will be updated accordingly.

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 wall, which is located along the embankment centerline. The slurry wall 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 wall 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 October 2020 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 2020 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 wall, which is located along the embankment centerline. The slurry wall 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 wall 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)

. At the time of the October inspection, the WDA's downstream slopes were adequately covered with appropriate vegetation that was well maintained.

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 October 2020 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 was modified and construction was completed in November of 2017. The modifications included the removal of the former closed-conduit spillway and the construction of a concrete open-channel spillway with a concrete down-chute and riprap armoring at the toe of the embankment. At the time of the October 2020 inspection, the water level in the WDA was observed at approximately 2 feet below the invert of the inlet end of the auxiliary spillway.

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 and June 2021), it is required to manage the flow during and following the peak discharge from a PMF event. A HydroCAD 10.0 model (HydroCAD Software Solutions LLC, 2019) 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 with the invert elevation 677.5 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 3, below.

Table 3: Hydrology and Hydraulics Analysis Results

Criteria	Value
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)	677.5
Maximum Inflow from Direct Precipitation (cubic feet per second (cfs))	3,692
Maximum Combined Inflow (cfs) [1]	3,732

Criteria	Value
Depth of Precipitation (in) for a PMF Event	31.9
Maximum WDA Outflow through Spillway (cfs)	335.4
Maximum Water Surface Elevation (ft MSL) [2]	679.1
Height of Wave Action (feet)	1.28
Net Freeboard during Design Storm Event (feet) [3, 7]	0.93

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 3, the current configuration of the WDA's auxiliary spillway is compliant with 40 CFR 257.73(d)(1)(v).

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, no structural stability deficiency were identified. .

4.0 SAFETY FACTOR ASSESSMENT - § 257.73(E)

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

- Maximum Pool Storage - Section 257.73(e)(1)(i) – Defined as the long-term, maximum storage pool (or operating) elevation and equal to the outlet elevation (elevation = 677.5 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 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 0.088g, which is the Maximum Considered Earthquake (MCE) ground motion of 0.2 second spectral response, or the 2 percent exceedance in 50 years. The seismic hazard was determined from the USGS 2014 Hazard Maps with confirmation from the SEAOC/OSHPD seismic mapping tool using ASCE 7-16 (2021) The zip code for the RMSGS was used as the location of the site. The 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 viewpoint of seismic risk.

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 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 4 below.

Table 4: Waste Disposal Area Slope Stability Analysis Results Summary

Case	Pool Elevation	Factor of Safety	Appendix A Reference Figure Number
1 - Steady State, Maximum **Storage** Pool Block - 257.73(e)(1)(i)	677.5 ft MSL	2.6	3
2 - Steady State, Maximum **Storage** Pool Rotational - 257.73(e)(1)(i)	677.5 ft MSL	2.6	4
3 - Steady State, Maximum **Surcharge** Pool Rotational - 257.73(e)(1)(ii)	681 ft MSL	2.7	5
4 - Steady State, Maximum **Surcharge** Pool Block - 257.73(e)(1)(ii)	681 ft MSL	2.6	6
5 - Steady State, Maximum **Storage** Pool Rotational Seismic - 257.73(e)(1)(iii)	677.5 ft MSL	1.8	7
6 - Steady State, Maximum **Storage** Pool Block Seismic - 257.73(e)(1)(iii)	677.5 ft MSL	1.8	8

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

4.2 Liquefaction Potential Assessment

Embankment and foundation soils were evaluated for seismically-induced liquefaction susceptibility using methods the empirical methods outlined by Olson and Stark (2002) for SPT values for non-cohesive materials, the methods outlined by Wright et al (2007) for cohesive materials and using the collected CPT data and running it through the GeoLogismiki software with uses the National Center for Earthquake Research (NCEER) recommended analyses for CPT data that uses Youd et al. (2001) and Robertson and Wride (1998). 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, the full analyses can be found in Appendix A.

5.0 CONCLUSIONS AND RECOMMENDATIONS

Based on our review of the information provided by NIPSCO, onsite observations, and the results of the structural stability assessment, no structural stability deficiencies were identified in the WDA surface impoundment during this assessment.

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).

6.0 CLOSING

This report is intended to summarize the results of the structural stability and factor of safety assessment to fulfill the provisions of Title 40 of the Code of Federal Regulations Section 257.73(d) and (e) (40 CFR Part 257.73(d) and (e)) for the WDA at the R.M. Schahfer Generating Station.

7.0 REFERENCES

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https://golderassociates.sharepoint.com/sites/142116/project files/5 technical work/slope stability analysis/rmsgs wda si and fos report 5yr - final.docx

Figures

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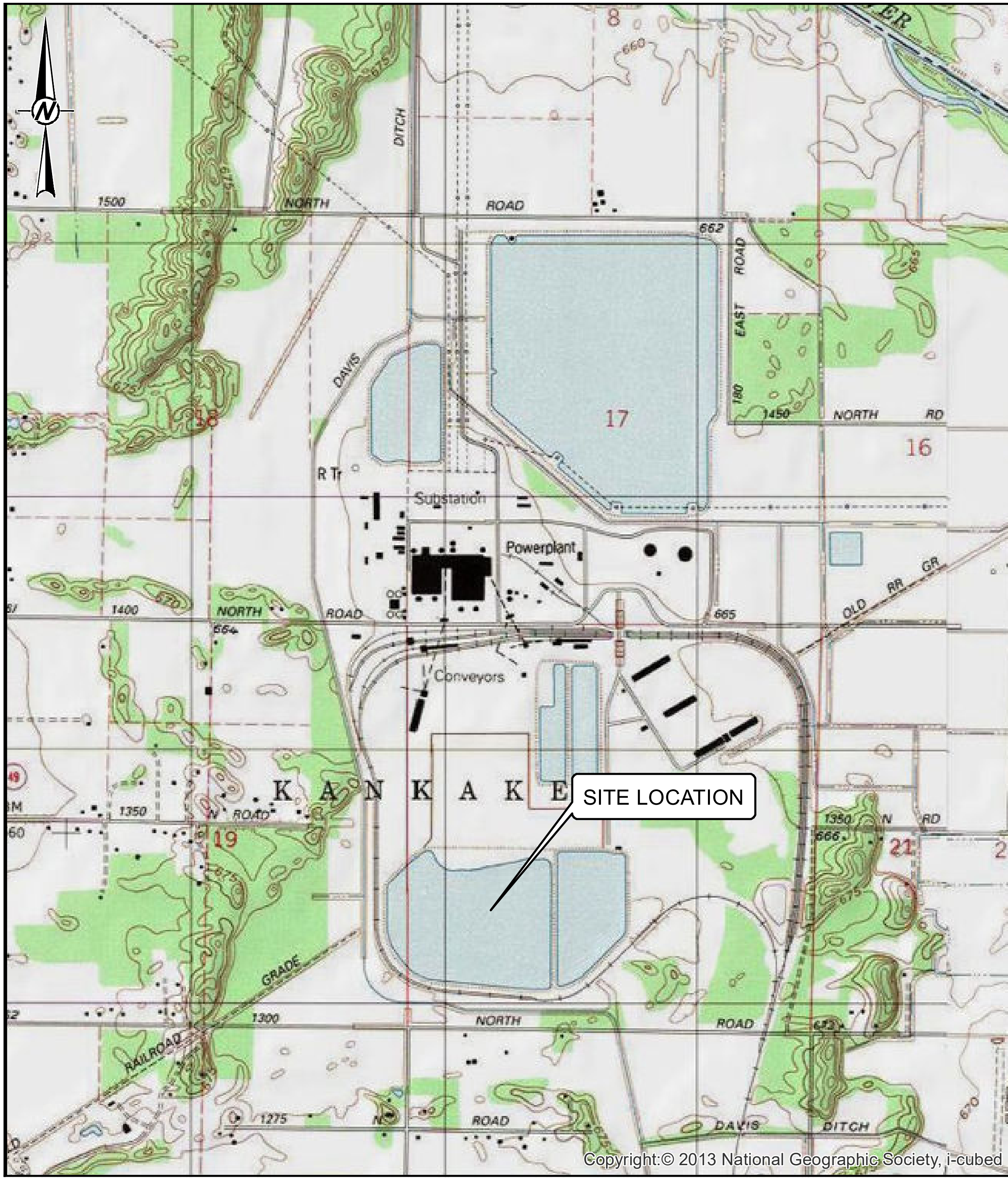
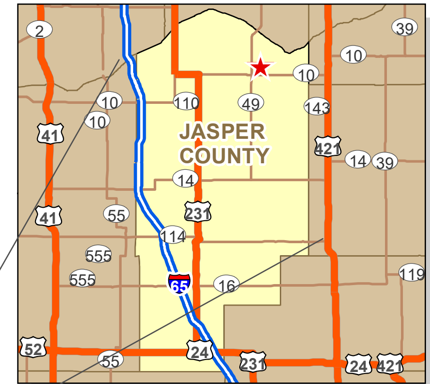




0 2000 4000
Feet

CLIENT
NIPSCO
SCHAHFER GENERATING CENTER
WHEATFIELD, INDIANA

PROJECT
STRUCTURAL STABILITY ASSESSMENT
FOR THE WASTE DISPOSAL AREA

TITLE
SITE LOCATION MAP

CONSULTANT

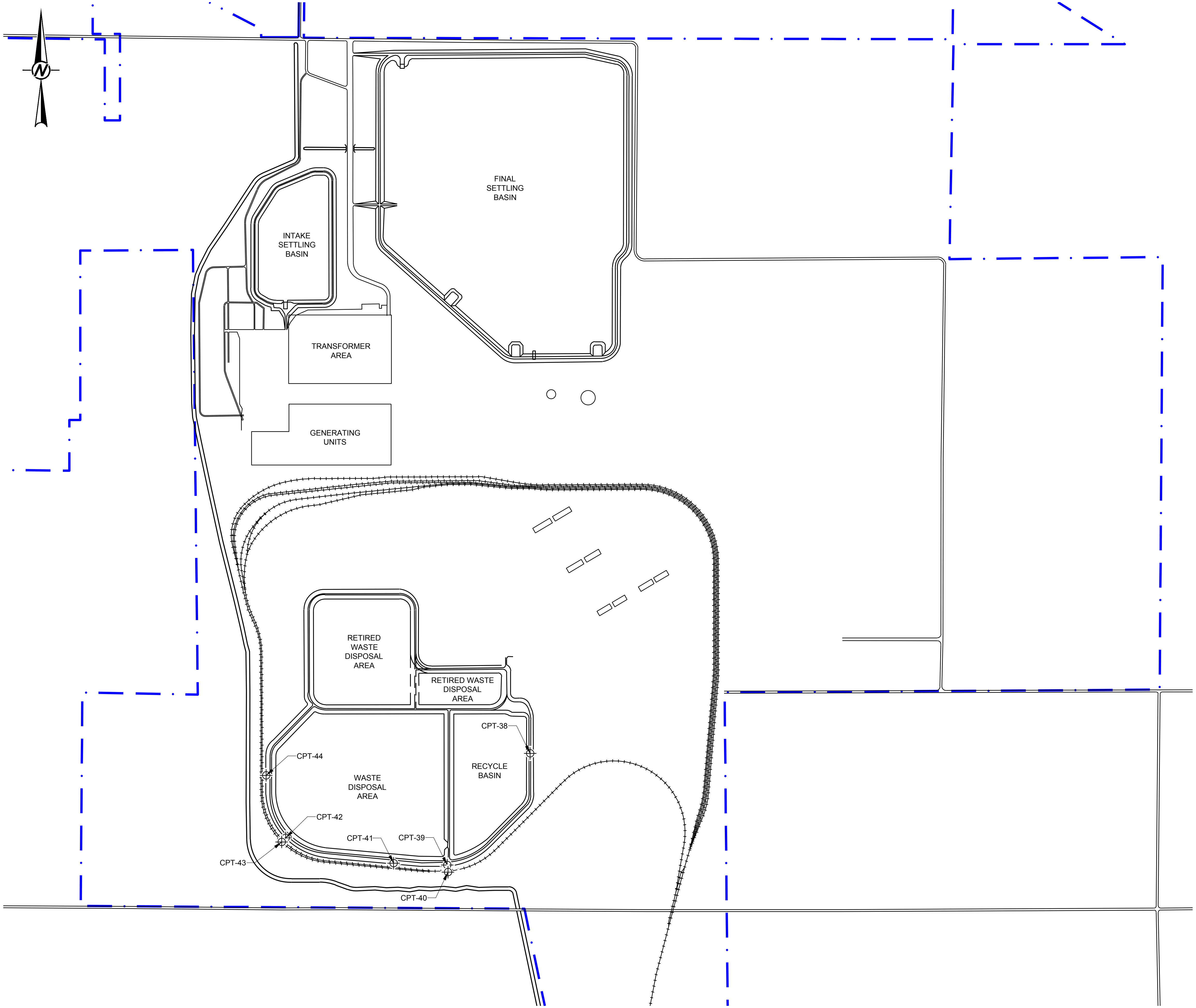
GOLDER
MEMBER OF WSP

YYYY-MM-DD	2021-07-21
PREPARED	DJC
DESIGN	TDJ
REVIEW	PJJ
APPROVED	TDJ

PROJECT No.	CONTROL	Rev.	FIGURE
21455411	21455411C001-GIS.mxd	0	1

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET HAS BEEN MODIFIED FROM: ANSI A

1 in



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
SCHAFHER GENERATING STATION
WHEATFIELD, INDIANA

PROJECT
STRUCTURAL STABILITY ASSESSMENT
FOR THE WASTE DISPOSAL AREA

TITLE
SITE PLAN

CONSULTANT

GOLDER
MEMBER OF WSP

YYYY-MM-DD	2021-07-21
DESIGNED	TDJ
PREPARED	DJC
REVIEWED	PJJ
APPROVED	TDJ

PROJECT NO.	CONTROL	REV.	FIGURE
21455411	21455411A001.dwg	0	2

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

REFERENCE(S)

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
FOR THE WASTE DISPOSAL AREA

TITLE
WASTE DISPOSAL AREA SURVEY

CONSULTANT

GOLDER
MEMBER OF WSP

YYYY-MM-DD	2021-07-21
DESIGNED	TDJ
PREPARED	DJC
REVIEWED	PJJ
APPROVED	TDJ

PROJECT NO.	CONTROL	REV.	FIGURE
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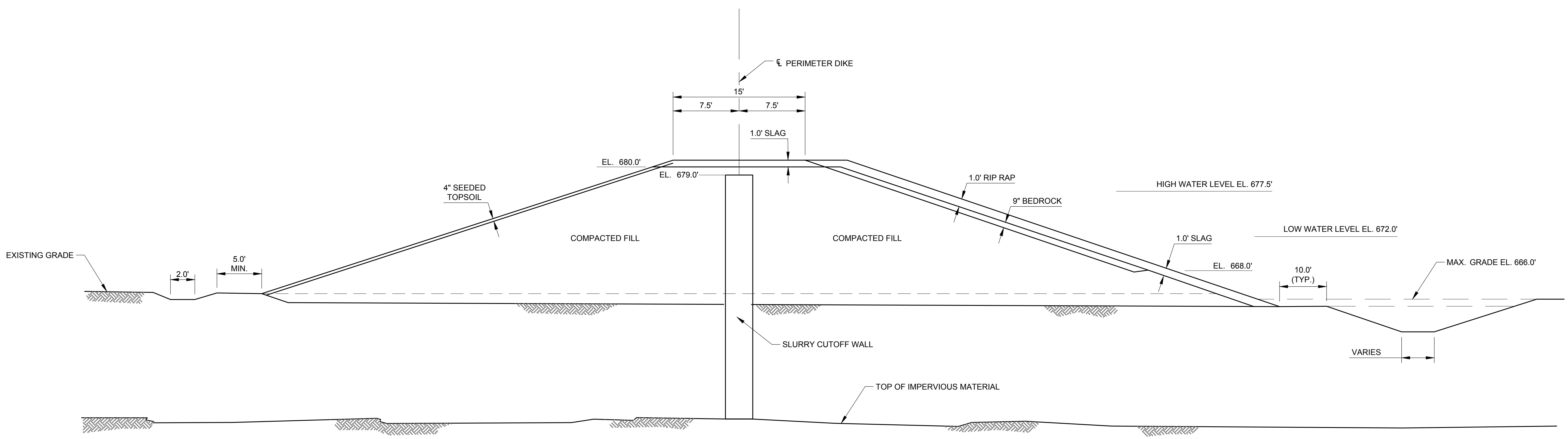


REFERENCE
FROM SARGENT & LUNDY CONSTRUCTION; DRAWING C-19; SETTLING BASINS DIKEWORK SECTIONS AND DETAILS; SHEET - 1; REV. F; 1982-07-13.

CLIENT
NIPSCO
SCHAHFER GENERATING STATION
WHEATFIELD, INDIANA

PROJECT
STRUCTURAL STABILITY ASSESSMENT
FOR THE WASTE DISPOSAL AREA

TITLE
TYPICAL CROSS SECTION

CONSULTANT

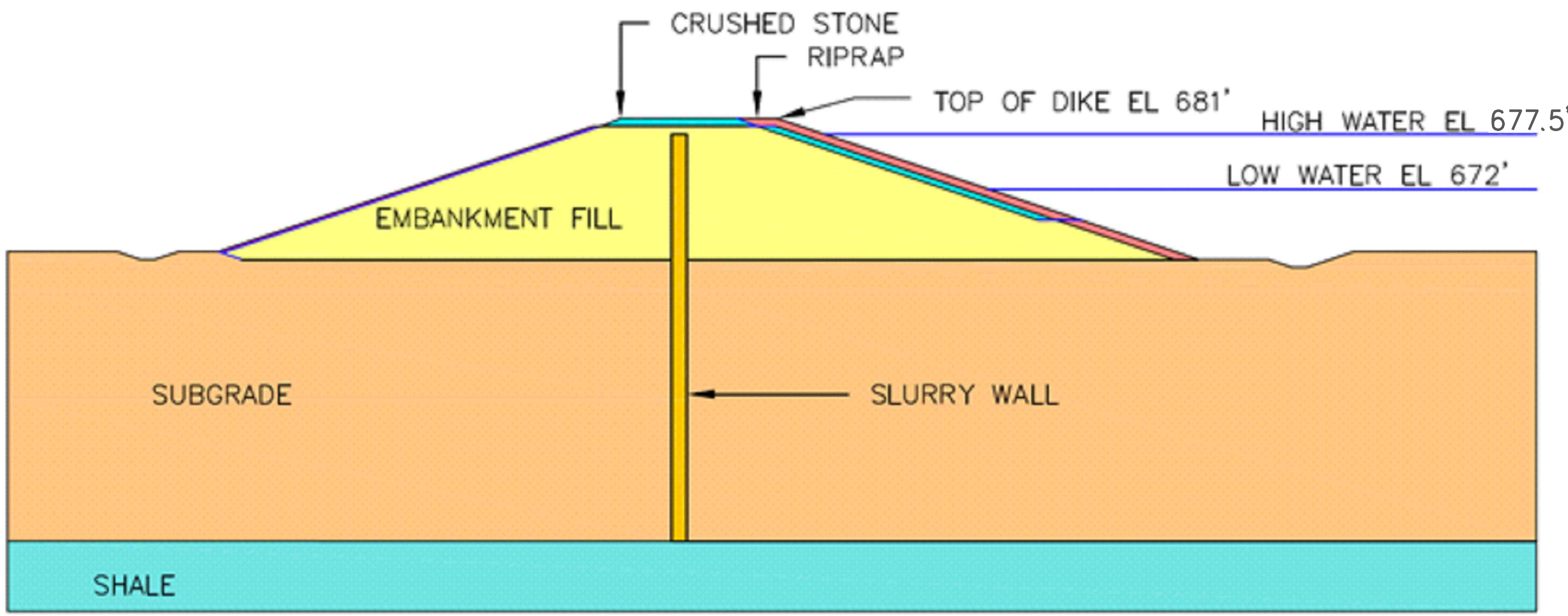
GOLDER
MEMBER OF WSP

YYYY-MM-DD	2021-07-21
DESIGNED	TDJ
PREPARED	DJC
REVIEWED	PJJ
APPROVED	TDJ

PROJECT NO.	CONTROL	REV.	FIGURE
21455411	21455411A003.dwg	0	4

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

WASTE DISPOSAL AREA
TYPICAL PROFILE



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 FOR THE WASTE DISPOSAL AREA

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

CONSULTANT
GOLDER
MEMBER OF WSP

YYYY-MM-DD	2021-07-21
DESIGNED	TDJ
PREPARED	DJC
REVIEWED	PJJ
APPROVED	TDJ

PROJECT NO.	CONTROL	REV.	FIGURE
21455411	21455411A003.dwg	0	5

APPENDIX A

Slope Stability and Liquefaction Analysis Results

	SUBJECT:	NIPSCO - CCR Pond Assessment - R.M. Schahfer			
GOLDER MEMBER OF WSP	Job No.:	21455411	Prepared:	MSG	6/8/2021
	Location:	Jasper County, IN	Checked:	PJJ	6/14/2021
	Date:	Jun-14-2021	Reviewed:	TDJ	7/14/2021

CCR Dam Hazard Assessment - WDA Pond

Objective:

Evaluate the static and pseudo-static stability of the earth impoundments and critical cross-section within the Waste Disposal Area (WDA) for the NIPSCO R.M. Schahfer Generating Station for EPA CCR Final Rule compliance.

References:

Reference No.	Source of Information
1	Golder Associates Inc (Golder) - 2011 Geotechnical Investigation and Embankment Stability Analyses - NIPSCO R.M. Schahfer Generating Station, June 2012.
2	Golder Associates Inc (Golder) - NIPSCO R.M. Schahfer Generating Facility - Waste Disposal Area (WDA) - Structural Stability and Safety Factor Assessment, October 5, 2016
3	Golder Associates Inc. (Golder) - Previous experience with CCR
4	Golder Associates Inc. (Golder) - Subsurface Geotechnical Investigation of the Intake Settling Basin - NIPSCO - RMSGS - April 15, 2016
5	Stratigraphics - Piezometric Cone Penetration Testing with Soil Electrical Conductivity Measurements - RMSGS CCR MUA 10387265 - September 2010.
6	Abramson, L.W. et al; "Slope Stability and Stabilization Methods"; 2nd Edition; Wiley; 2002
7	Hynes-Griffin, M.E. and Franklin, A.G. (1984), "Rationalizing the Seismic Coefficient Method, " Miscellaneous Paper GL-84-13, U.S. Army Engineer Waterways Experiment Station, Vicksbug, Mississippi, 34p.
8	Rocscience (2021), SLIDE2 Version 9.017 64-bit
9	United States Geological Society (USGS) online hazard mapping website (http://earthquake.usgs.gov/research/hazmaps/design/).
10	SEAOC/OSHPD Seismic Design Map Tool (2021) (https://seismicmaps.org/)
11	Boulanger & Idriss (2004); Evaluating the Potential for Liquefaction or Cyclic Failure of Silts and Clays
12	Boulanger & Idriss (2005); Evaluating Cyclic Failure in Silts and Clays
13	Boulanger & Idriss (2007); Evaluation of Cyclic Softening in Silts and Clays
14	Olson, SM; Stark, TD (2002); Liquefied Strength Ratio from Liquefaction Flow Failure Case Histories
15	Wright, SG; Zornberg, JG; Aguettant, JE (2007); The Fully Softened Shear Strength of High Plasticity Clays

Analysis Sections:

Analyses performed included the loading conditions based on the 40 CFR Parts 257 & 261 (EPA Final CCR Rule) and included: (1) Maximum Pool Long-Term Steady State Static, (2) Maximum Pool Surcharge, (3) Seismic (Pseudo-static), and (4) Post-Liquefaction. Undrained (End-of-Construction) conditions were not considered for the berm slopes due to no new construction for closure.

Notes for the loading conditions:
(1) Maximum Pool Long-Term Steady State Static conditions considered the maximum pool height experienced and expected in the ash pond prior to closure.
(2) Maximum Pool Surcharge looks at conditions where the temporary pool level reaches EL 681 ft MSL, but the steady state condition at that water level was not achieved.
(3) For Pseudo-static Limit Equilibrium method, see next section of report.
(4) Post-liquefaction conditions assessed the containment system with the expectations that any ash would liquefy and the shear strength of the materials within the berm would reduce. In this situation, cohesive materials would reduce to residual strength conditions (a conservative approach; conceptualized by Boulanger & Idriss 2004, 2005, 2007) and non-cohesive contractive materials would reduce to cyclic softened conditions, if applicable (per Olson and Stark (2002). Estimation of the residual strength for cohesive materials would use an empirical method developed by Wright et al (2007) based on liquid limit, confining stresses, and effective friction angles from the field investigations and laboratory testing results.

Analysis Methods:

Slope stability was evaluated using the computer program SLIDE Version 7.030 64-bit (Rocscience, 2017) using the generalized limit equilibrium method of stability analysis developed by Spencer (Abramson et al., 2002). Circular and block search patterns were used to find the failure surface, which resulted in a minimum calculated factor of safety for the preliminary desktop review. Static and pseudo-static methods were performed during the analyses.

Pseudo-static limit equilibrium analyses were conducted to evaluate the stability of the existing berm slope under seismic loads for earthquake hazards. Pseudo-static stability analyses apply a constant horizontal force to the system to represent the forces generated during an earthquake event, with the magnitude of the applied force typically related to the peak ground acceleration modified for soil amplification (PGAM) of a specific earthquake hazard risk. A pseudo-static limit equilibrium analysis was conducted to evaluate the stability of the containment berms under a seismic load for the earthquake hazard representing a 2% probability of exceedance in 50 years (equaling 0.088g; i.e. a return period of 2475 years) based on the United States Geological Survey (USGS) Hazard Maps. The seismic hazard as determined from the USGS 2014 Hazard Maps with confirmation from the SEAOC/OSHPD seismic mapping tool using ASCE7-16 (2021) and data for 2% probability of exceedance in 50 years is presented in Figure 1A. Figure 1B shows the USGS 2008 Deggregation plot and information from the SEAOC/OSHPD site for reference. A modification of the Hynes-Griffin and Franklin (1984) method was used where a horizontal force of 1/2 of the PGAM (EPA 1995) was used in this analysis (0.044g). In addition, the shear strength of the site material properties were reduced by 20% per the method's requirements.

Material Properties:

The material properties outlined below were used in the this stability analysis. Values were determined from data obtained from the following resources: (a) Golder's investigation "2011 Geotechnical Investigation and Embankment Stability Analyses", June 2012, (b) values obtained from Golder's investigation "Waste Disposal Area (WDA) - Structural Stability and Safety Factor Assessment", October 5, 2016, (c) Golder's investigation "Subsurface Geotechnical Investigation of the Intake Settling Basin - NIPSCO - RMSGS", April 15, 2016, d) published typical values for the observed soil and rock types, e) published engineering correlations between in-situ data and characteristic soil and rock properties, and f) Golder's professional experience. Table 1 summarizes the soil/material parameters used for the stability analyses.

Table 1. Golder's Material Parameters

Material	Shear Strength		In-situ Unit Weight (lb/ft^3)	Source (Resources)
	Effective Friction Angle (degrees)	Effective Cohesion (lb/ft^2)		
Embankment Soil	42	0	125	1, 2, 3, 4
Shale	45	0	145	1, 2, 3, 4
Subgrade	39	0	110	1, 2, 3, 4
Slurry Wall	0	300	120	1, 2, 3, 4
Rip Rap	45	0	140	1, 2, 3, 4
Crushed Stone	45	0	140	1, 2, 3, 4
Topsoil	35	15	120	1, 2, 3, 4

Note: Sands contain minor amounts of cohesive materials

Liquefied Shear Strength of Non-Cohesive Materials:

Post-liquefaction analysis and liquefied shear strength for non-cohesive materials were evaluated by reviewing SPT blow counts and using the empirical method outlined by Olson & Stark (2002) for SPT N values. The equation below was used to evaluate the liquefaction potential of the saturated sands and contained ash within the WDA pond boundaries/footprint. Ash within the containment berms is potentially liquefiable. From experience and published literaure, a vertical stress ratio of 0.08 has been established for ash based on Golder's experience. However, the WDA facility has limited ash within the containment system and sands within the berms and underlying the WDA footprint have an average blow count of 16. In addition, CPT testing conducted at the site also indicated, stiff sand conditions with no liquefaction potential at the site, except for one data point in CPT 27, however only CPTs 39 through 44 were near the WDA, with CPT-38 in the recycle basin and likely applicable. Olson & Stack method confirms that a SPT blow count range of 15 to 20 is the transition between contractive (liquefiable) sands and dilatative (non-liquefiable) sands. The equation provided by Olson & Stark (provided below) is only used for blows less than or equal to 12. Blow counts greater than 12 (note that equation only applies to blows less than or equal to 12) are considered too dense for liquefaction to occur and would be considered non-liquefiable. Therefore, post-liquefaction strength analyses would not apply to this site. CPT data was also be evaluated to confirm Golder's assertion on post-liquefaction analyses for the site.

Olson & Stark (2002) Equation

$$[19b] \quad \frac{s_u(\text{LIQ})}{\sigma'_{vo}} = 0.03 + 0.0075[(N_1)_{60}] \pm 0.03$$

$$\text{for } (N_1) \lesssim 12$$

where:

s_u (LIQ) = liquefied undrained stear strength

$(N_1)_{60}$ = corrected SPT blow counts

σ'_{vo} = effective overburden pressure

$s_u / \sigma'vo$ vertical stress ratio

(N1)60 = 16

Liquefied Shear Strength of Cohesive Materials:
Post-liquefaction analysis and liquefied shear strength for cohesive materials was evaluated using the empirical method outlined by Wright et al (2007) for liquid limit (LL) values (equation provided below). For the WDA site, the only material considered cohesive is the Slurry Wall. However, the slurry wall is cemented with portland cement. Therefore, there was no LL values available for using the Wright et al (2007) method. Based on previous studies, the worst case scenario for long-term steady state conditions of the slurry wall (made of cement bentonite) was a friction angle of 0 and cohesion of 300 psf. Golder decreased the strength by 20% (similar to Hynes-Griffin and Franklin) given limited information on the Mohr Coulomb parameters for the material and the conservative nature of the Hynes-Grffin and Franklin method. It was further determined that due to the high strength of the site's sands within the embankment and subsurface, post-liquefactions analyses were not needed for the site. Additional analysis documents for the CPT analyses are attached to this calculation package.

Wright et al (2007) Equation

$$\varphi_{secant,r} = 52.5° - 21.3° \log_{10}(\omega_{LL}) - 3° \log_{10}\left(\frac{\sigma'_f}{p_a}\right) \quad (8.1)$$

where:

ϕ'_{secant} = secant friction angle
ω_{LL} = liquid limit (Atterberg limits)
σ'_f = effective overburden pressure
p_a = atmospheric pressure

Summary of Stability Analyses Results:

Table 3 summarizes the minimum required safety factors for the critical cross-section per Section 257.73 analyzed using the defined material properties for the project site. The table lists the minimum factor of safety for the slope and the factor of safety for a failure. For additional information, see the attached stability figures presented for each analysis case.

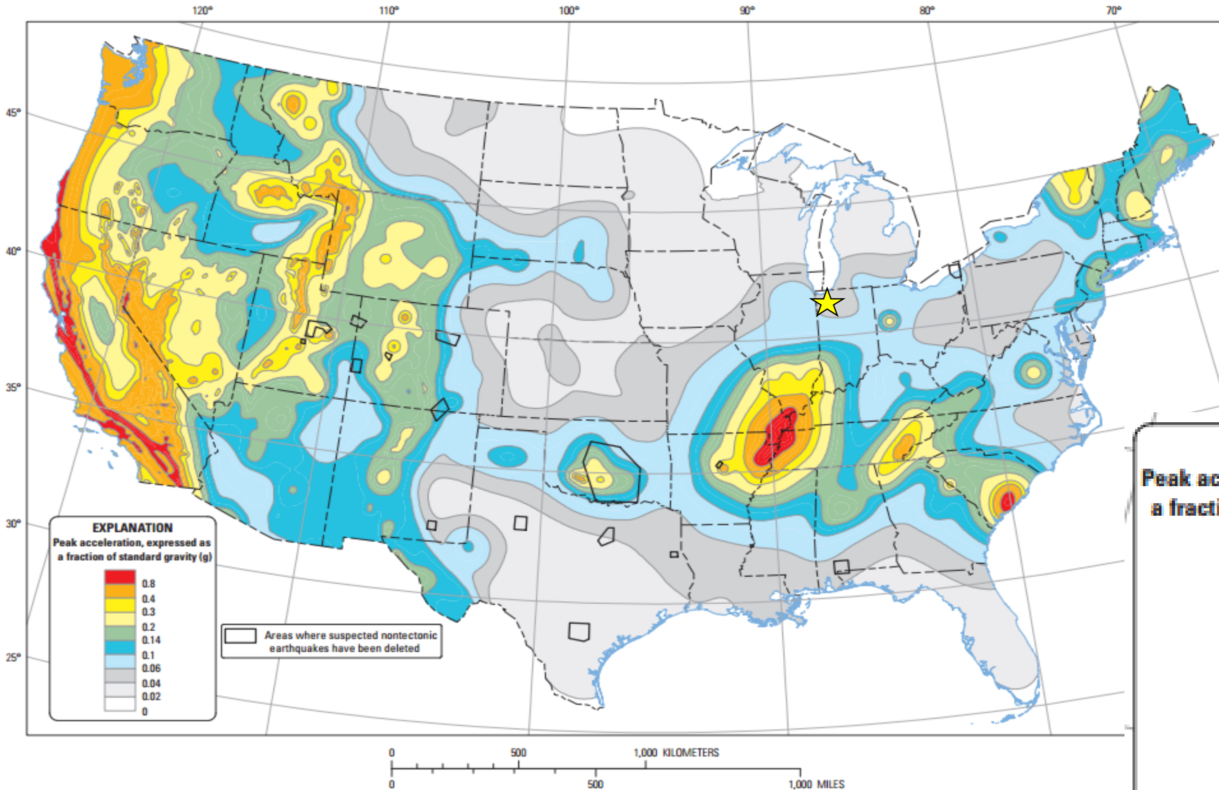
Table 3. Calculated factors of safety - Slope Cross-Section A-A'

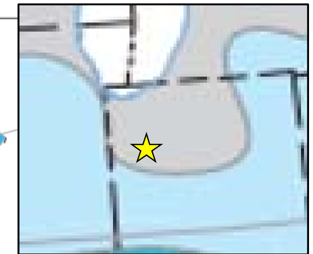
Slope Stability Case	Minimum Acceptable Factor of Safety	Calculated Factor of Safety		Evaluation	Figure(s)
		Circular	Non-Circular		
Maximum Pool Depth Long-Term Steady State	1.50	2.60	2.60	Satisfactory	3,4
Maximum Pool Surcharge	1.40	2.70	2.60	Satisfactory	5,6
Maximum Pool Depth Pseudo-Static Seismic	1.00	1.80	1.80	Satisfactory	7,8

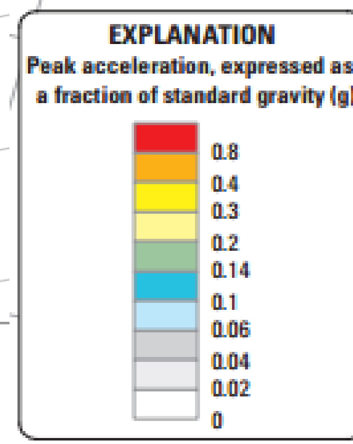
Discussion:
The results indicate that the slopes along WDA and within the existing pond do meet the minimum requirements set forth by the EPA CCR Final Rule for Maxim Pool Long-Term Steady State, Maximum Pool Surcharge, and Pseudo-Static Seismic. This conclusion includes cross-sections A-A'. Post-liquefaction was determined to not apply to the site due to SPT and CPT field investigations determining dense nature of Embankment and Subgrade sand layers. As stated above, end-of-construction, rapid drawdown, and post-liquefaction analyses using 2D limit equilibrium analysis methods were determined to not be needed for the WDA assessment.

Based on Map, Horizontal Ground Acceleration for Analysis = (1/2)*0.06g = 0.03g (per Hynes-Griffin & Franklin (1984)

Golder has elected to use the PGA_M = 0.088g, representing value within soil region of strata. Horizontal Ground Acceleration for Analysis = (1/2)*0.088g = 0.044g (per Hynes-Griffin & Franklin (1984).







Two-percent probability of exceedance in 50 years map of peak ground acceleration

GOLDER
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Golder Associates Inc.

SCALE	AS SHOWN	PROJECT	NIPSCO - CCR Pond Assessment - R.M. Schahfer
DATE	Jun 2021	TITLE	Slope USGS Peak Horizontal Ground Acceleration Hazard Assessment (2% in 50 Year Event) (2014)
MADE BY	MSG		
CAD	-		
CHECK	PJJ	CLIENT	NIPSCO - Indiana
REVIEW	TDJ	FIGURE	1A

FILE: STABILITY

PROJECT No. 2145411 REV. 0

Based on Map, Horizontal Ground Acceleration for Analysis = (1/2)*0.06g = 0.03g (per Hynes-Griffin & Franklin (1984)

Golder has elected to use the PGA_M = 0.088g, representing a value within the soil region of the substrata. Horizontal Ground Acceleration for Analysis = (1/2)*0.088g = 0.044g (per Hynes-Griffin & Franklin (1984).

OSHPD

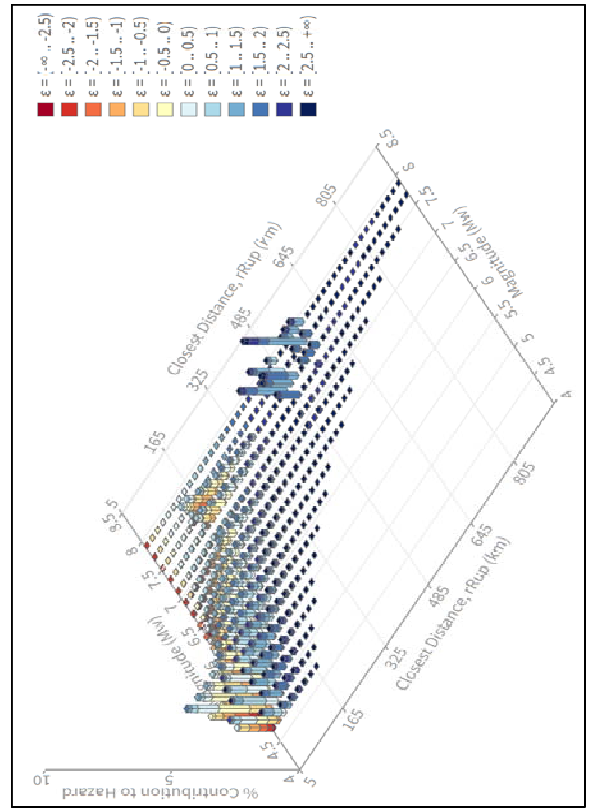
NIPSCO

Latitude, Longitude: 41.20899261, -87.02159155

Date	6/7/2021, 12:36:21 PM
Design Code Reference Document	ASCE7-16
Risk Category	IV
Site Class	D - Stiff Soil

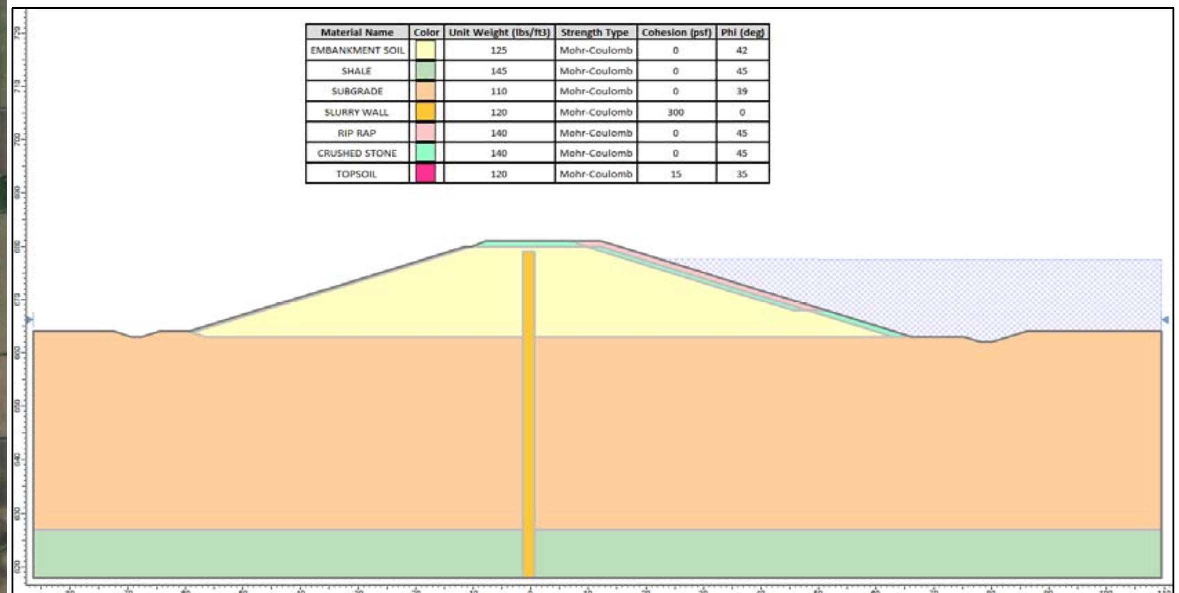
Type	Value	Description
S_S	0.114	MCE_R ground motion. (for 0.2 second period)
S_1	0.068	MCE_R ground motion. (for 1.0s period)
S_{MS}	0.183	Site-modified spectral acceleration value
S_{M1}	0.164	Site-modified spectral acceleration value
S_{DS}	0.122	Numeric seismic design value at 0.2 second SA
S_{D1}	0.109	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	C	Seismic design category
F_a	1.6	Site amplification factor at 0.2 second
F_v	2.4	Site amplification factor at 1.0 second
PGA	0.055	MCE_G peak ground acceleration
F_{PGA}	1.6	Site amplification factor at PGA
PGA_M	0.088	Site modified peak ground acceleration



GOLDER MEMBER OF WSP Golder Associates Inc.	SCALE: AS SHOWN	PROJECT	NIPSCO - CCR Pond Assessment - R.M. Schahfer
	DATE: Jun 2021	TITLE	Data provided by OSHPD Seismic Hazard Map Tool (2021) based on ASCE 7 (2016) and 2014 Deaggregation Chart
	MADE BY: MSG		
	CAD: -		
FILE: STABILITY	CHECK: PJJ	CLIENT	NIPSCO - Indiana
PROJECT No.: 2145411 / REV.: 0	REVIEW: TDJ	FIGURE	1B

Cross-Section A-A' Profile Typical Embankment Cross-Section
for West and South Sides of the WDA
(per Sargent & Lundy Construction Drawing C-19 (1982)



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

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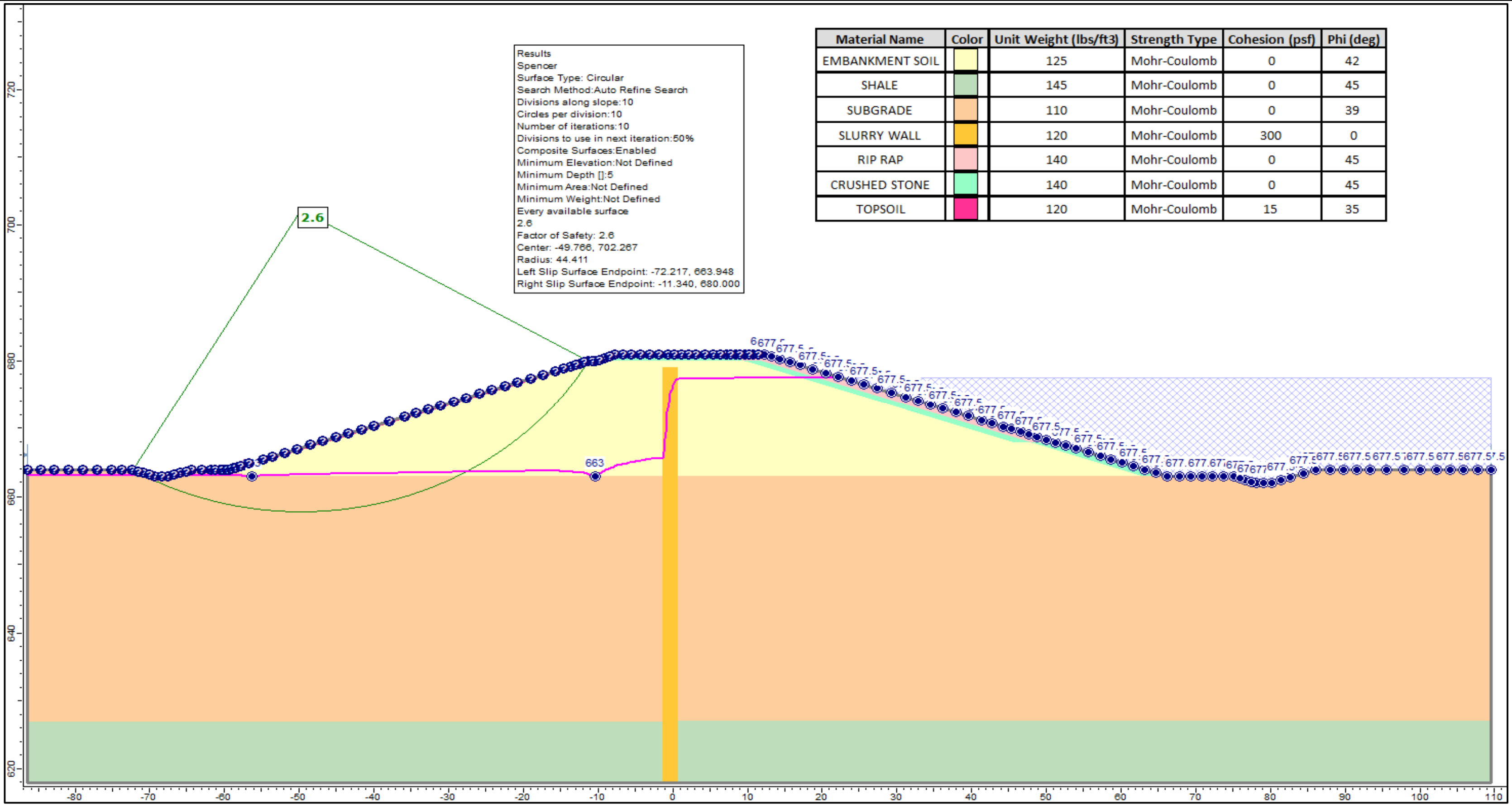
MEMBER OF WSP
Golder Associates Inc.

SCALE	AS SHOWN	PROJECT	**NIPSCO - CCR Pond Assessment - R.M. Schahfer**
DATE	Jun 2021	TITLE	**WDA Cross-Section A-A'**
MADE BY	MSG		
CAD	-		
CHECK	PJJ	CLIENT	**NIPSCO - Indiana**
REVIEW	TDJ	FIGURE	**2A**

FILE: STABILITY
PROJECT No. 2145411 REV. 0

Minimum
FS = 2.6



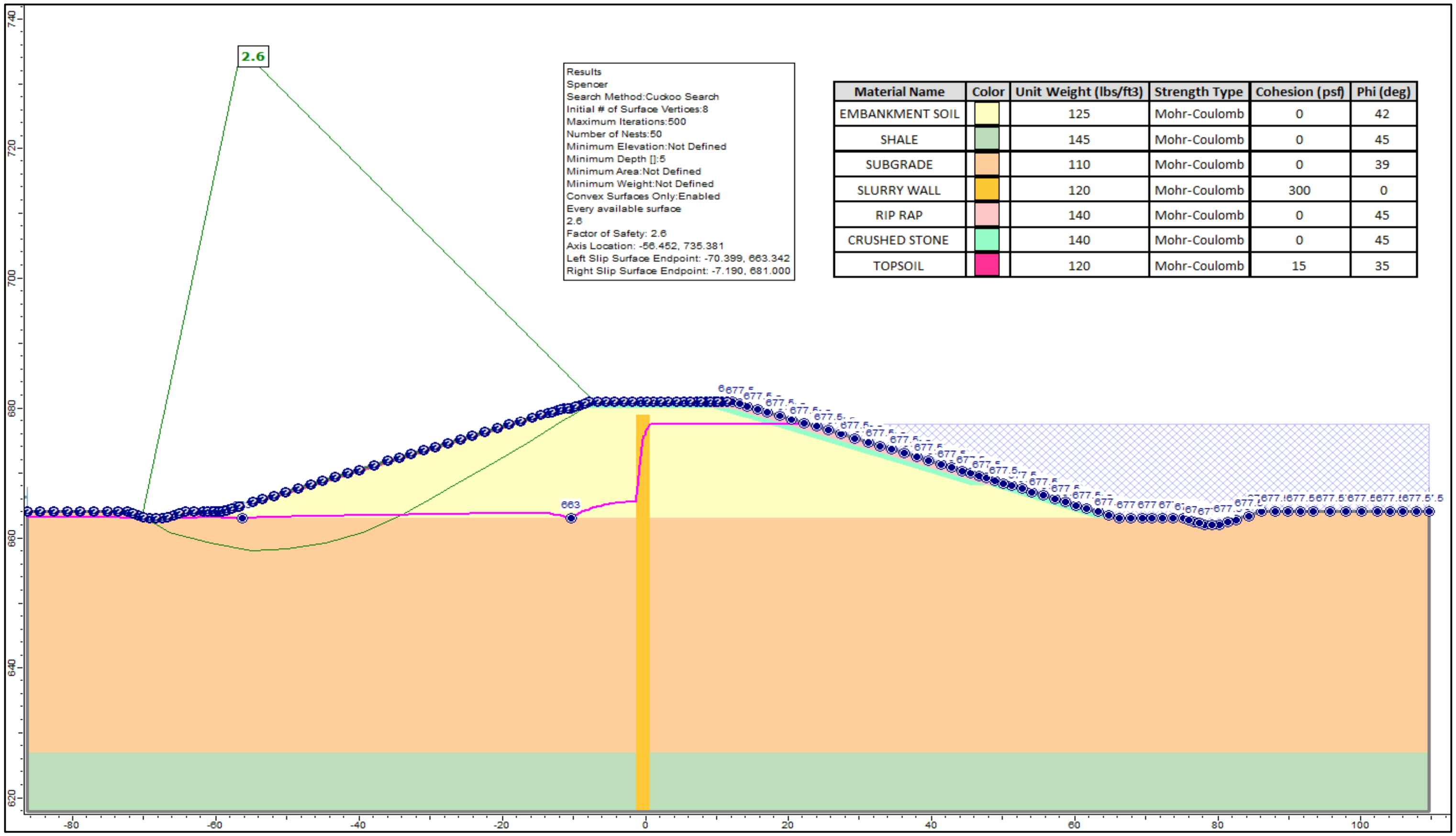
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
EMBANKMENT SOIL		125	Mohr-Coulomb	0	42
SHALE		145	Mohr-Coulomb	0	45
SUBGRADE		110	Mohr-Coulomb	0	39
SLURRY WALL		120	Mohr-Coulomb	300	0
RIP RAP		140	Mohr-Coulomb	0	45
CRUSHED STONE		140	Mohr-Coulomb	0	45
TOPSOIL		120	Mohr-Coulomb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

GOLDER
MEMBER OF WSP
Golder Associates Inc.

SCALE	AS SHOWN	PROJECT	NIPSCO - CCR Pond Assessment - R.M. Schahfer
DATE	Jun 2021	TITLE	Cross-Section A-A' Maximum Pool Depth Steady State Loading - Circular Failure Plane
MADE BY	MSG		
CAD	-		
CHECK	PJJ	CLIENT	NIPSCO - Indiana
REVIEW	TDJ	FIGURE	3

FILE: STABILITY
PROJECT No. 2145411
REV. 0

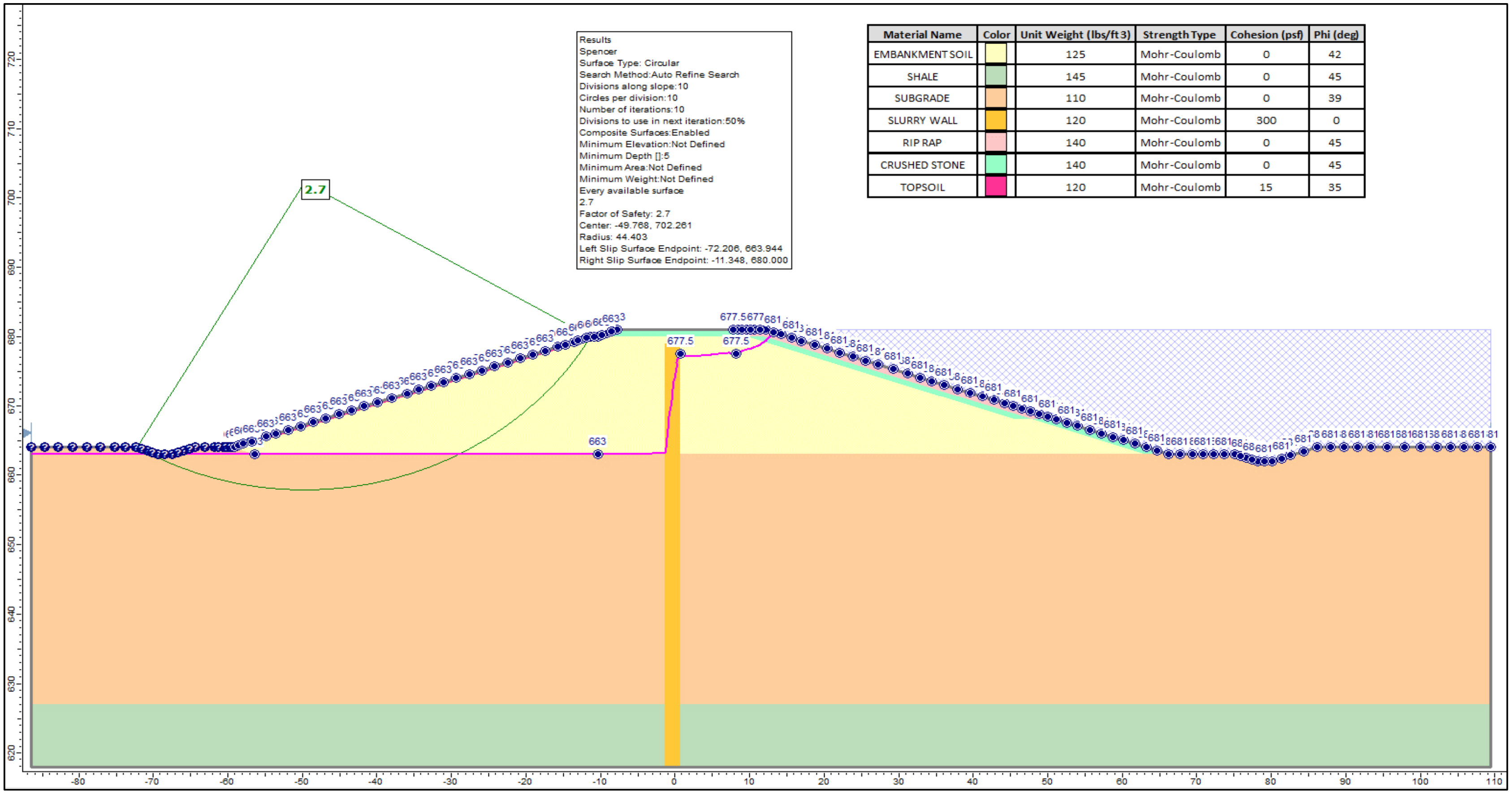
Minimum
FS = 2.6



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

GOLDER MEMBER OF WSP Golder Associates Inc.	SCALE: AS SHOWN DATE: Jun 2021 MADE BY: MSG CAD: -	PROJECT	**NIPSCO - CCR Pond Assessment - R.M. Schahfer**
		TITLE	**Cross-Section A-A' Maximum Pool Depth Steady State Loading - Non-Circular Failure Plane**
FILE: STABILITY PROJECT No. 2145411 REV. 0	CHECK: PJJ REVIEW: TDJ	CLIENT	**NIPSCO - Indiana** — FIGURE 4

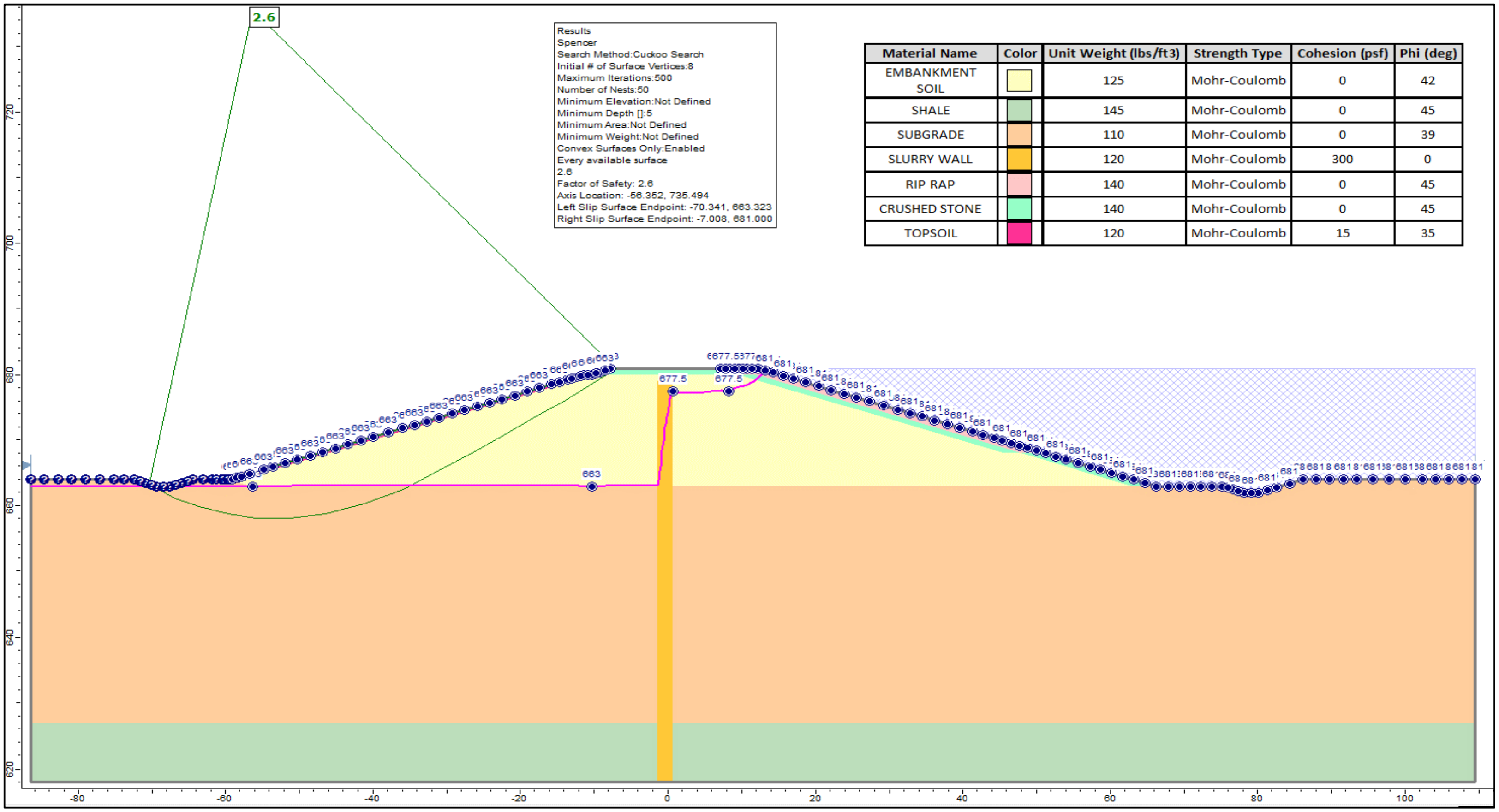
Minimum FS = 2.7



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

GOLDER MEMBER OF WSP Golder Associates Inc.	SCALE AS SHOWN	PROJECT	NIPSCO - CCR Pond Assessment - R.M. Schahfer
	DATE Jun 2021	TITLE	Cross-Section A-A' Maximum Pool Surcharge - Circular Failure Plane
	MADE BY MSG		
	CAD -		
FILE STABILITY	CHECK PJJ	CLIENT	NIPSCO - Indiana
PROJECT No. 2145411 / REV. 0	REVIEW TDJ	FIGURE	5

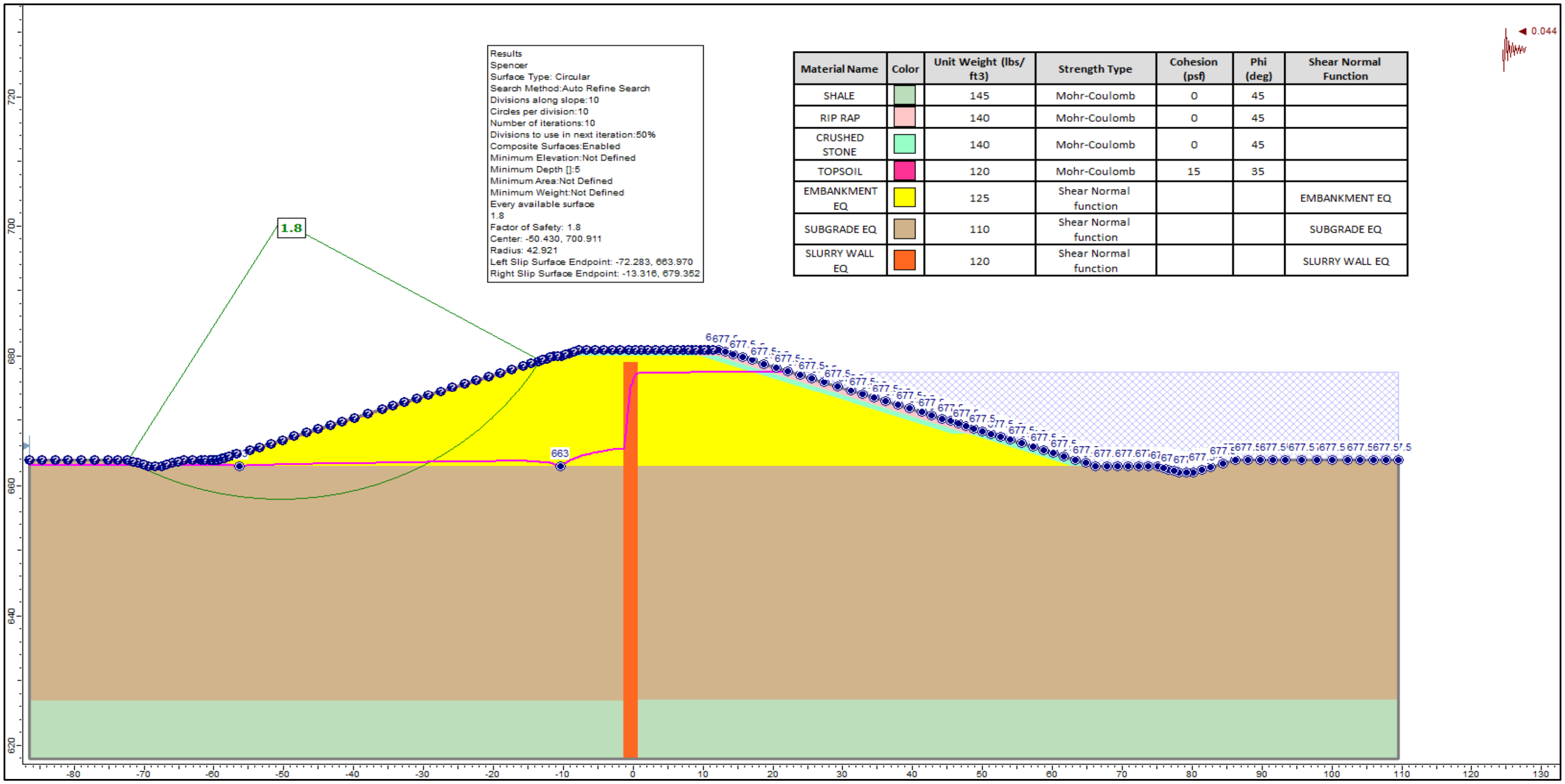
Minimum
FS = 2.6



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

GOLDER MEMBER OF WSP Golder Associates Inc.	SCALE AS SHOWN	PROJECT	NIPSCO - CCR Pond Assessment - R.M. Schahfer
	DATE Jun 2021	TITLE	Cross-Section A-A'Maximum Pool Surcharge - Non-Circular Failure Plane
	MADE BY MSG		
	CAD -		
FILE STABILITY	CHECK PJJ	CLIENT	NIPSCO - Indiana
PROJECT No. 2145411 REV. 0	REVIEW TDJ	FIGURE	6

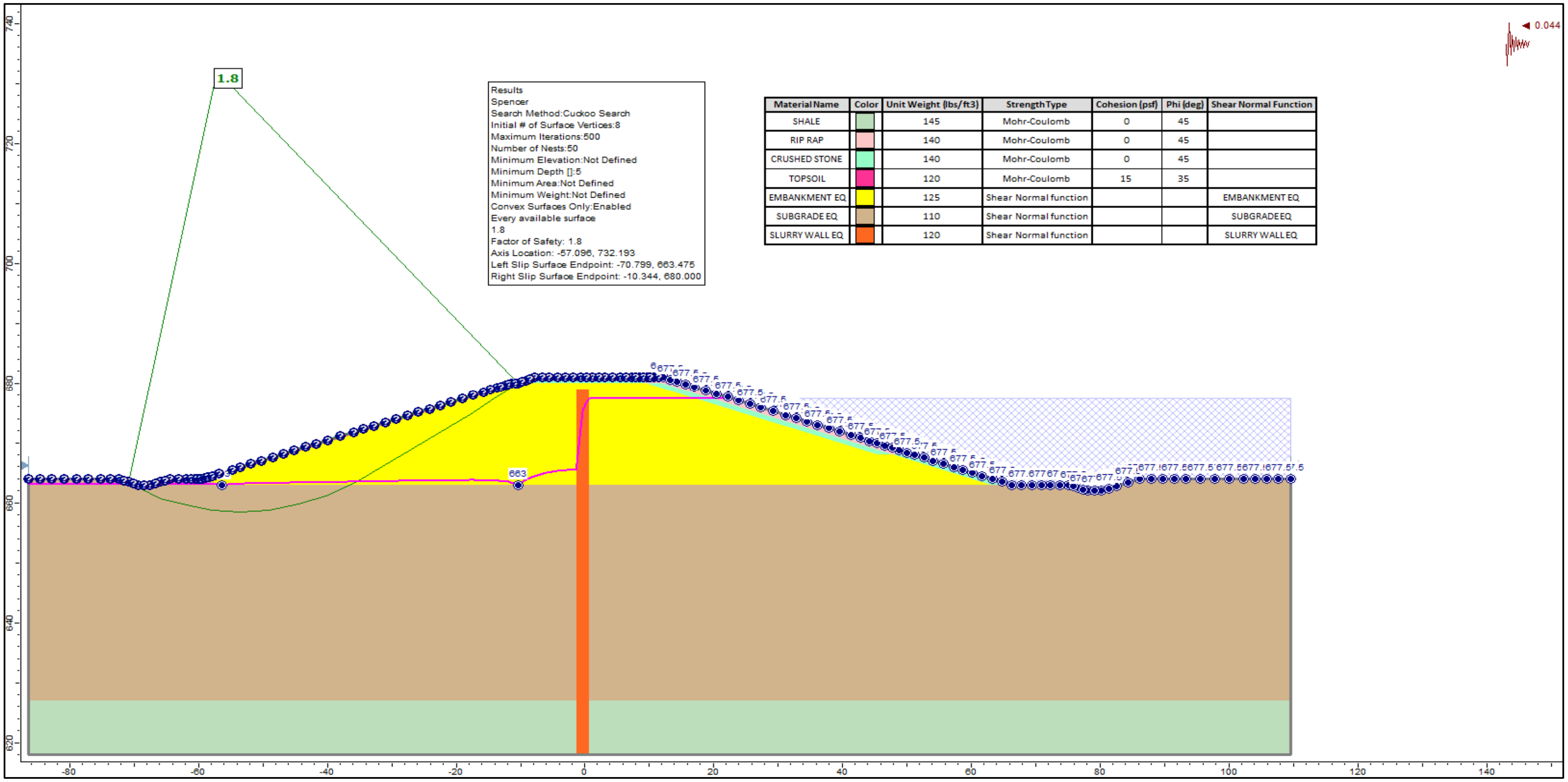
Minimum
FS = 1.8



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

GOLDER MEMBER OF WSP Golder Associates Inc.	SCALE: AS SHOWN	PROJECT	NIPSCO - CCR Pond Assessment - R.M. Schahfer
	DATE: Jun 2021	TITLE	Cross-Section A-A' Maximum Pool Depth Pseudo-Static Seismic Loading - Circular Failure Plane
	MADE BY: MSG		
	CAD: -		
FILE: STABILITY	CHECK: PJJ	CLIENT	NIPSCO - Indiana
PROJECT No. 2145411 REV. 0	REVIEW: TDJ	FIGURE	7

Minimum
FS = 1.8



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

GOLDER MEMBER OF WSP Golder Associates Inc.	SCALE: AS SHOWN	PROJECT	NIPSCO - CCR Pond Assessment - R.M. Schahfer
	DATE: Jun 2021	TITLE	Cross-Section A-A' Maximum Pool Depth Pseudo-Static Seismic Loading - Non-Circular Failure Plane
	MADE BY: MSG		
	CAD: -		
FILE: STABILITY	CHECK: PJJ	CLIENT	NIPSCO - Indiana
PROJECT No.: 2145411 REV.: 0	REVIEW: TDJ	FIGURE	8

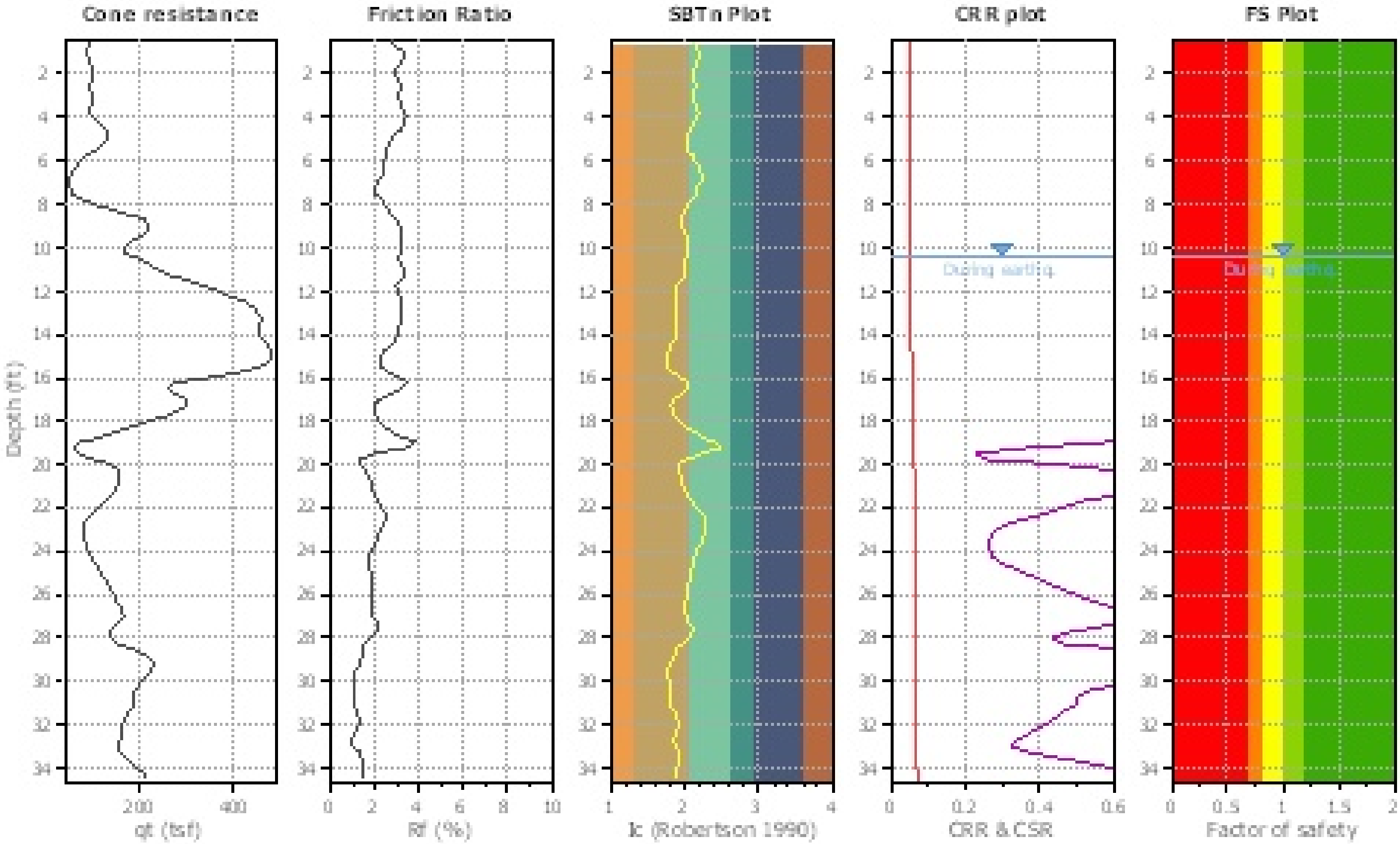
LIQUEFACTION ANALYSIS REPORT

Project title : NIPSCO Schafer **Location : Wheatfield, IN**

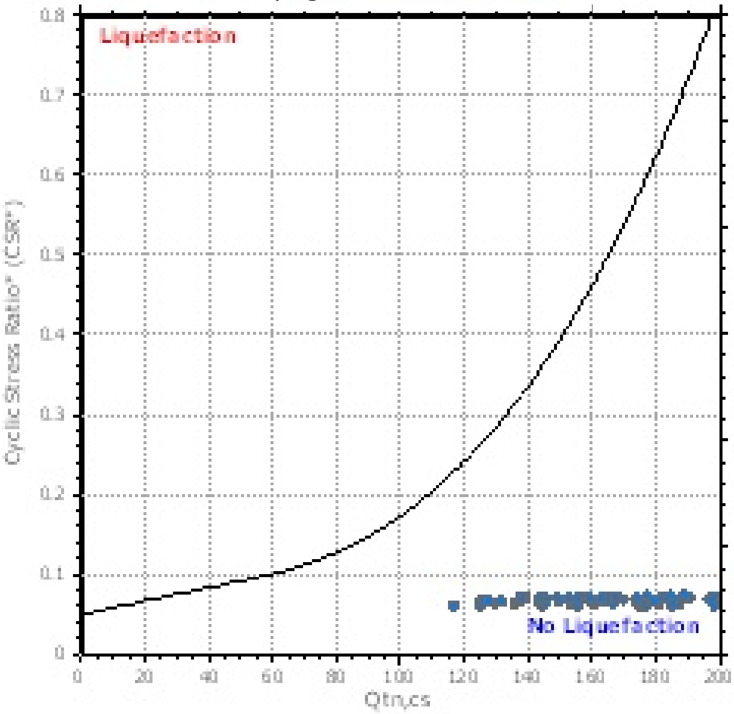
CPT file : CP038

Input parameters and analysis data

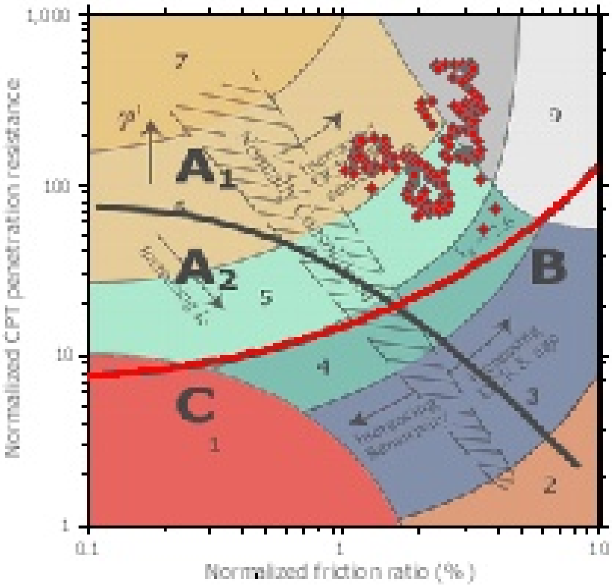
Analysis method:	NCEER (1998)	G.W.T. (in-situ):	10.50 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	10.50 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.00	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.09	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		



$M_w=7^{1/2}$, sigma'=1 atm base curve



Summary of liquefaction potential



Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

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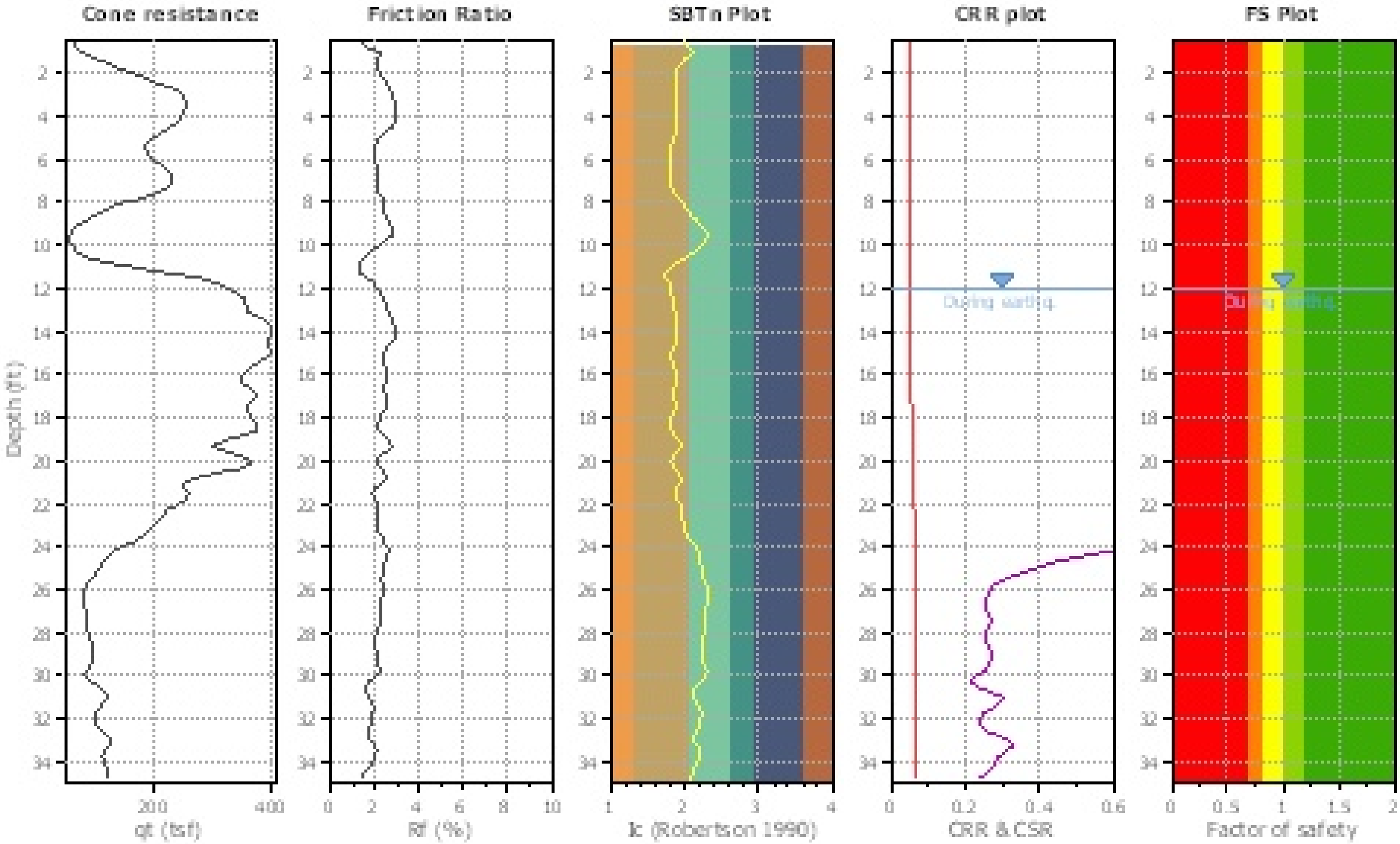
LIQUEFACTION ANALYSIS REPORT

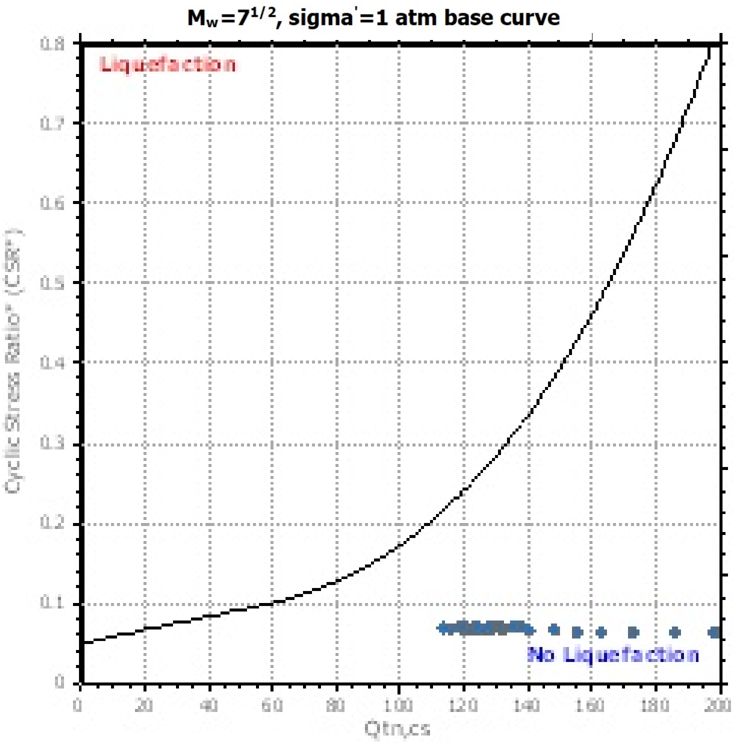
Project title : NIPSCO Schafer **Location : Wheatfield, IN**

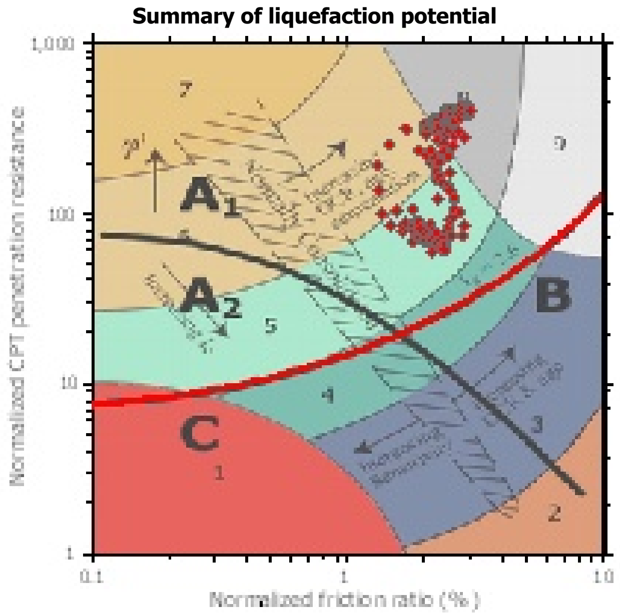
CPT file : CP039

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	12.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	12.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.00	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.09	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

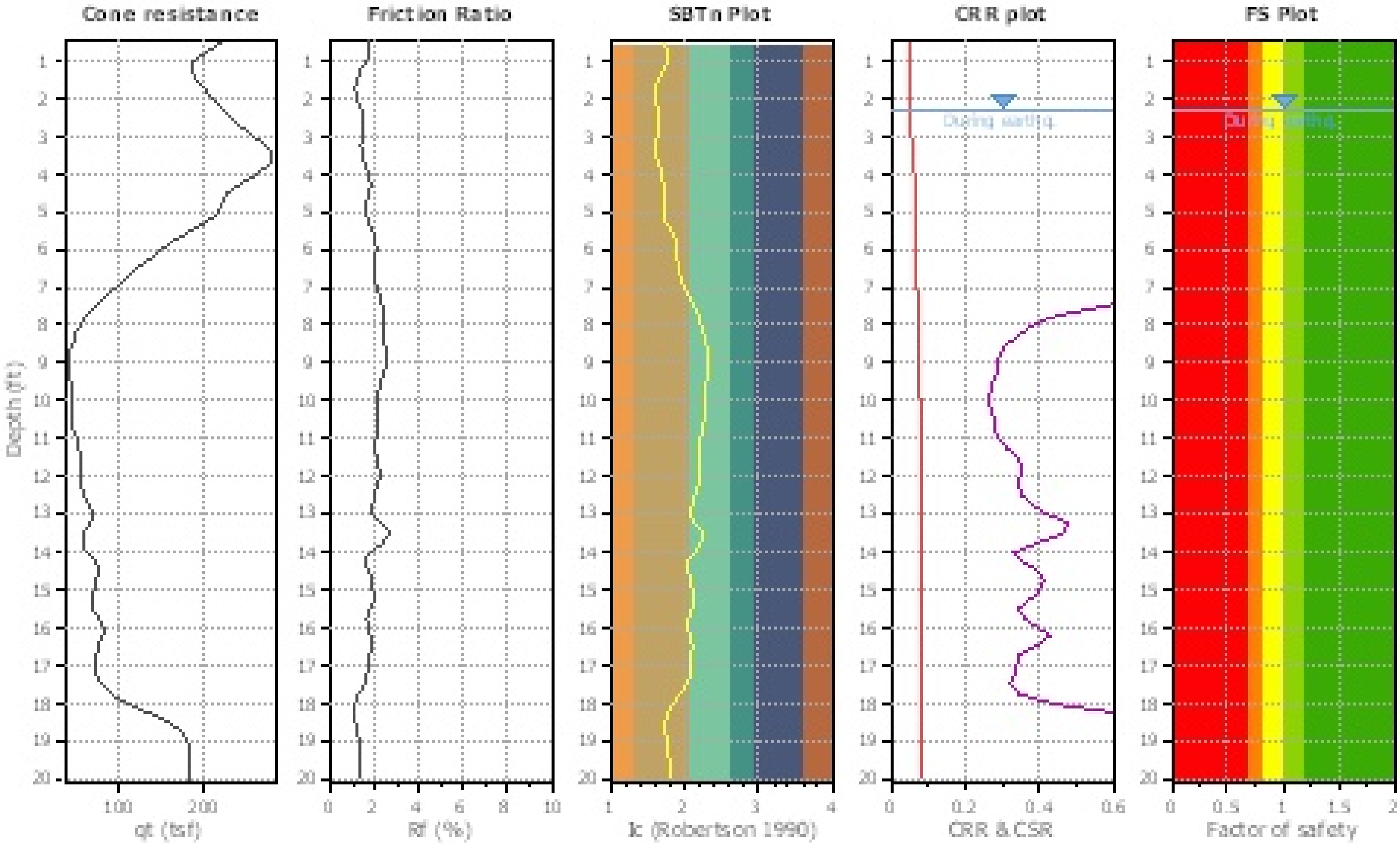
LIQUEFACTION ANALYSIS REPORT

Project title : NIPSCO Schafer **Location : Wheatfield, IN**

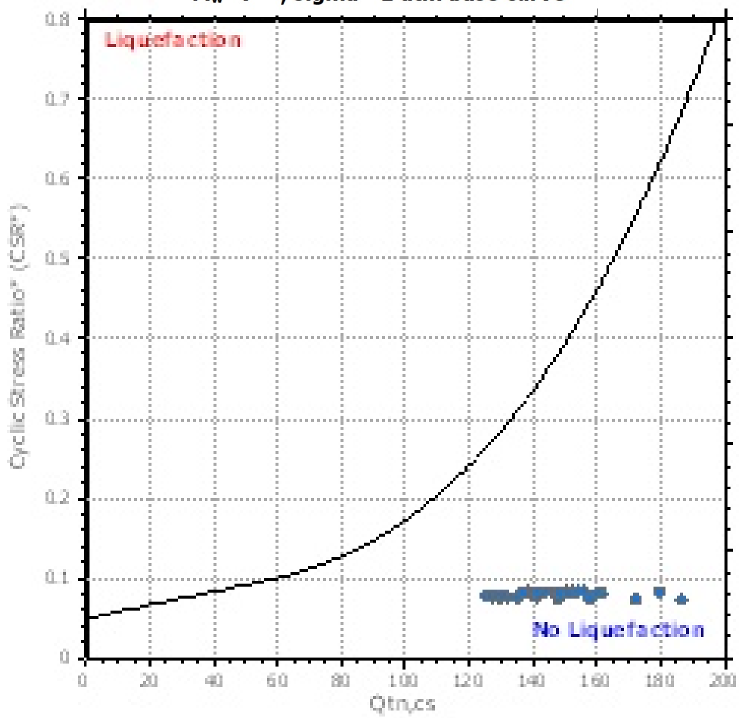
CPT file : CP040

Input parameters and analysis data

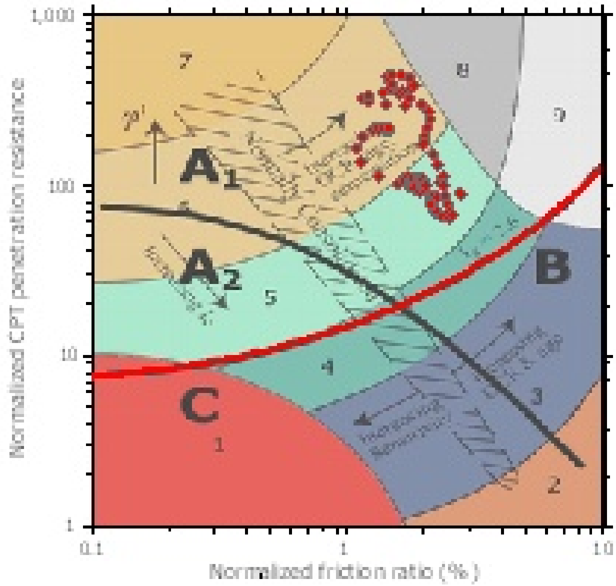
Analysis method:	NCEER (1998)	G.W.T. (in-situ):	2.30 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	2.30 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.00	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.09	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		



M_w=7$^{1/2}$, sigma'=1 atm base curve



Summary of liquefaction potential



Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

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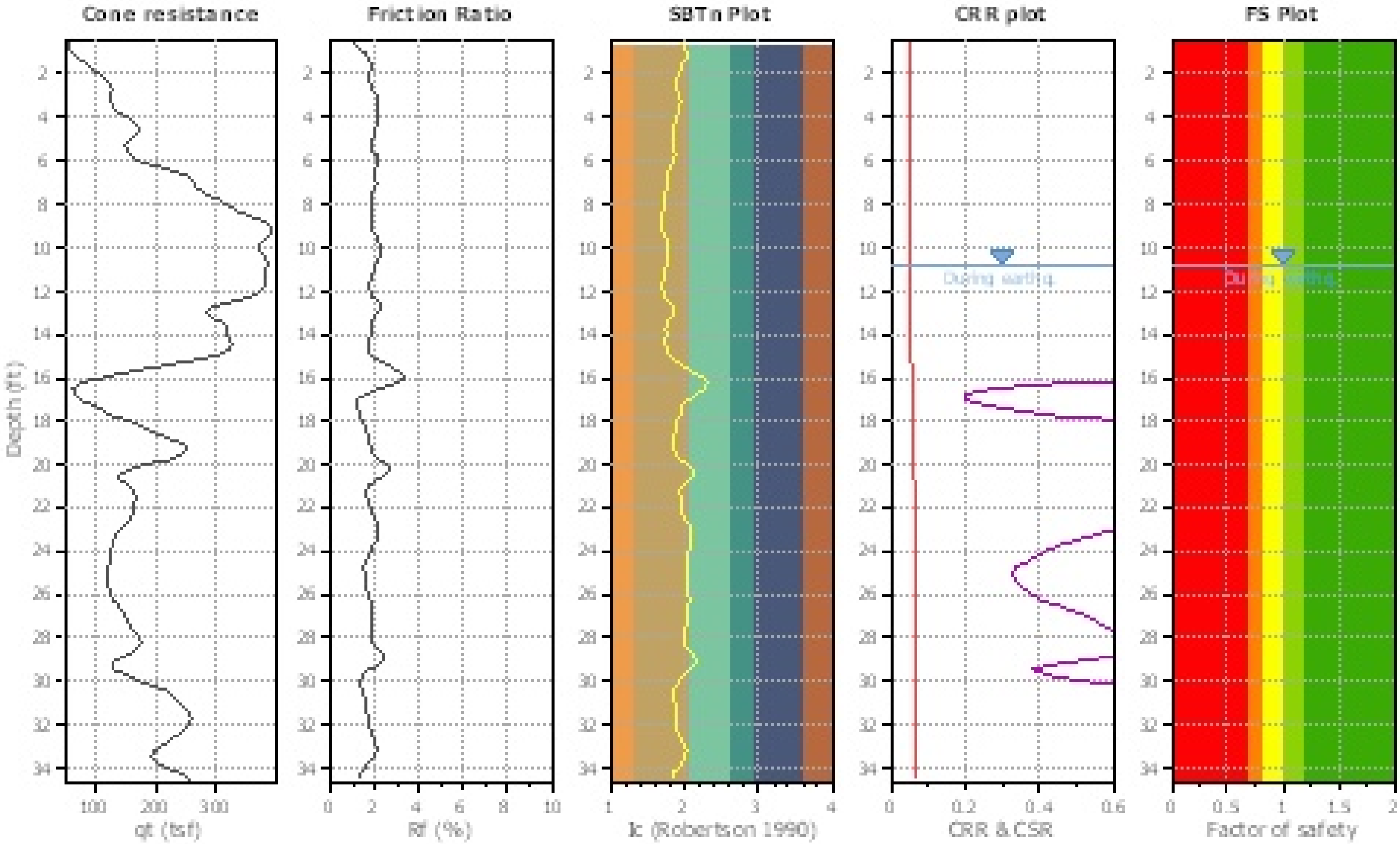
LIQUEFACTION ANALYSIS REPORT

Project title : NIPSCO Schafer **Location : Wheatfield, IN**

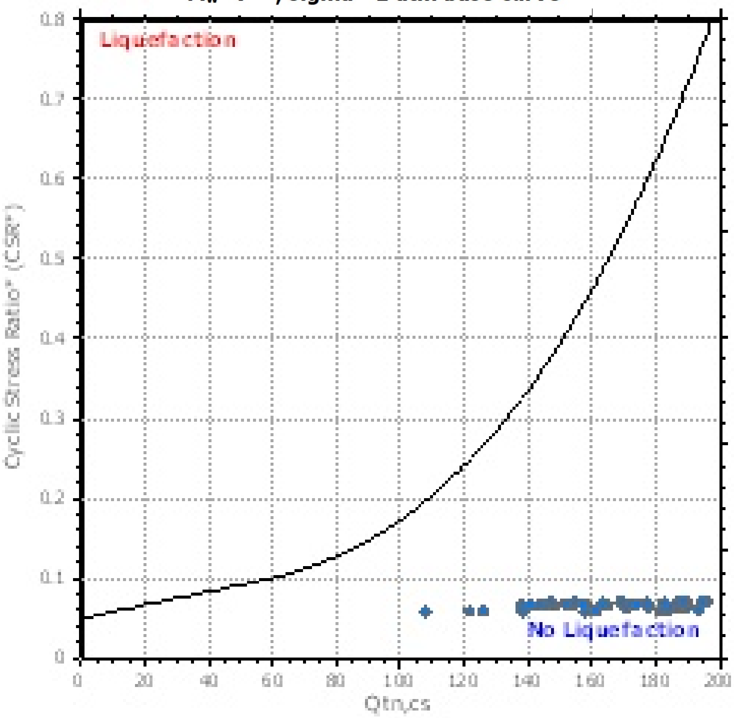
CPT file : CP041

Input parameters and analysis data

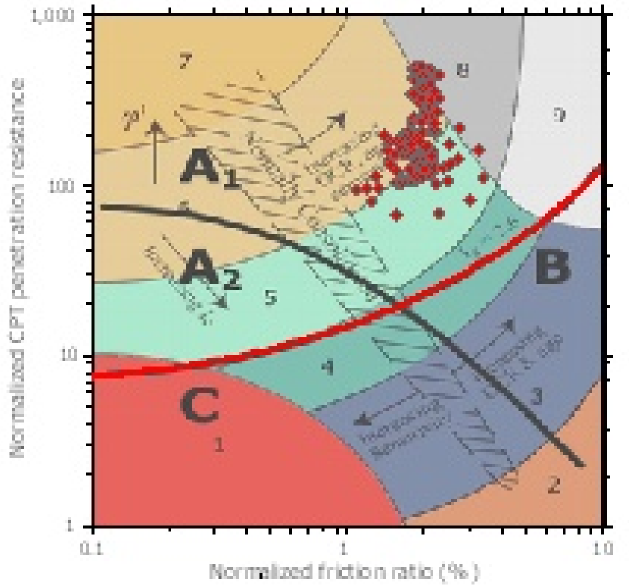
Analysis method:	NCEER (1998)	G.W.T. (in-situ):	10.80 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	10.80 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.00	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.09	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		



$M_w=7^{1/2}$, sigma'=1 atm base curve



Summary of liquefaction potential



Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

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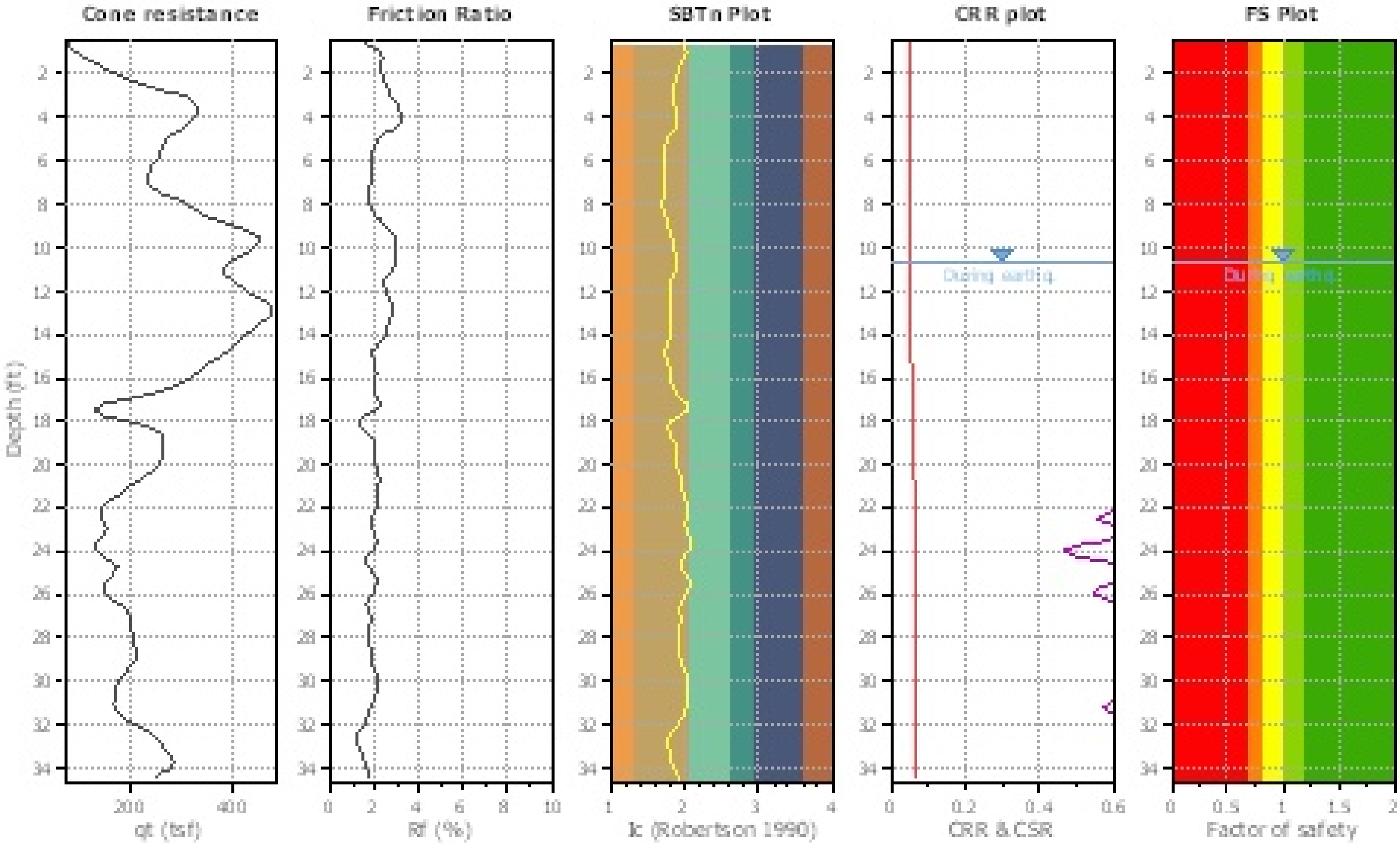
LIQUEFACTION ANALYSIS REPORT

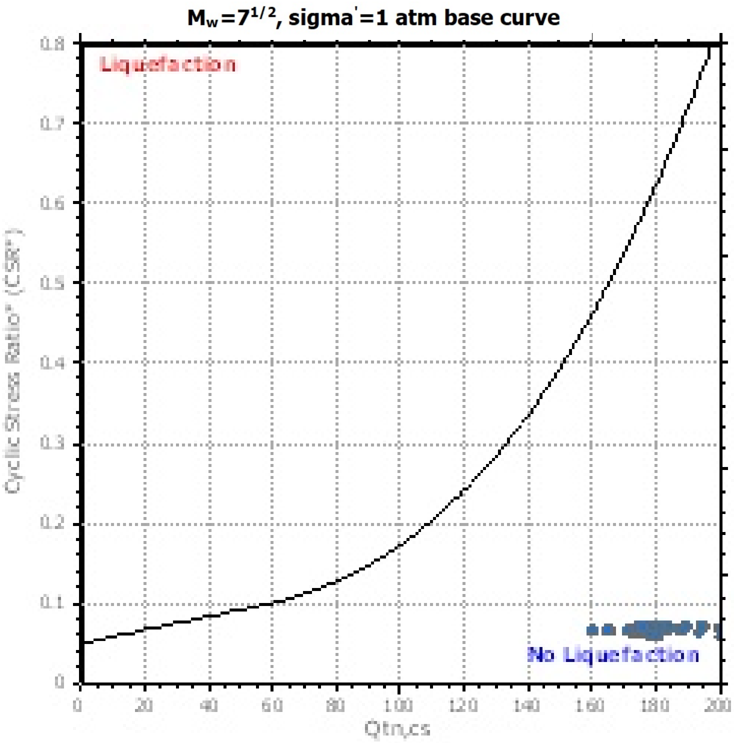
Project title : NIPSCO Schafer **Location : Wheatfield, IN**

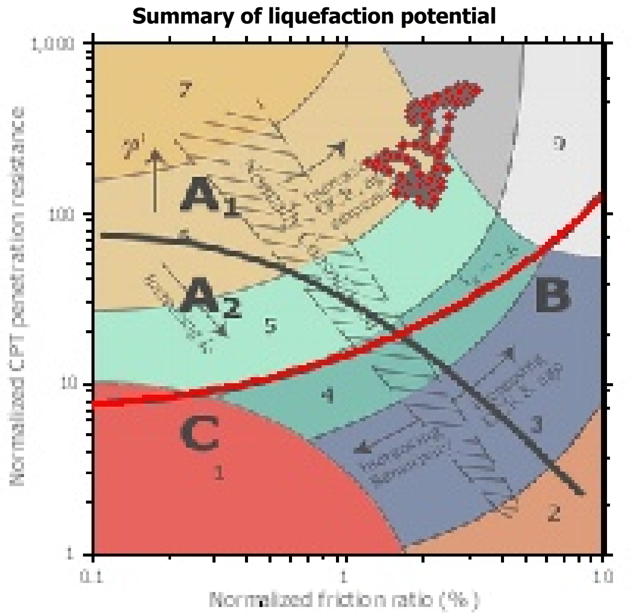
CPT file : CP042

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	10.70 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	10.70 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.00	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.09	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

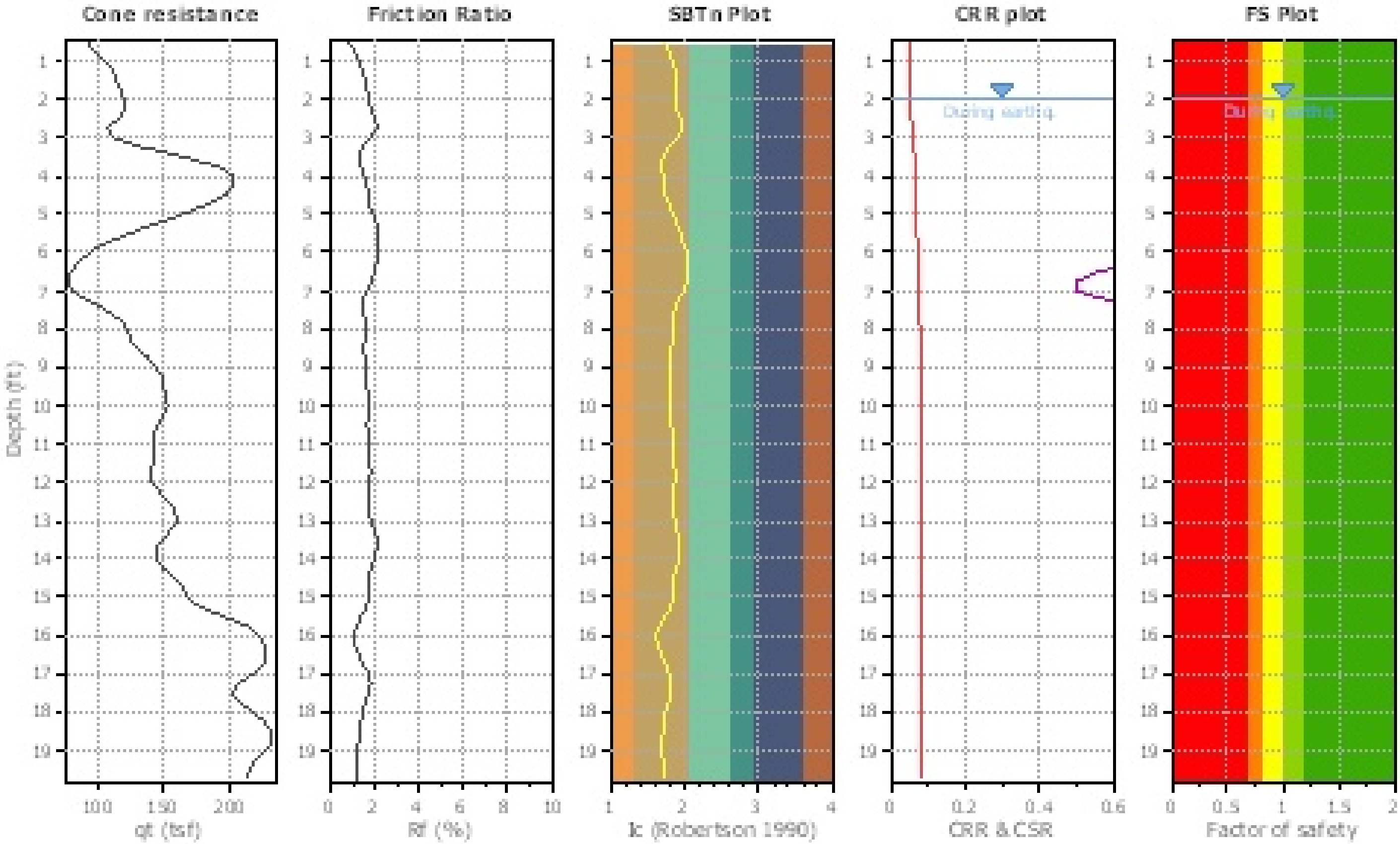
LIQUEFACTION ANALYSIS REPORT

Project title : NIPSCO Schafer **Location : Wheatfield, IN**

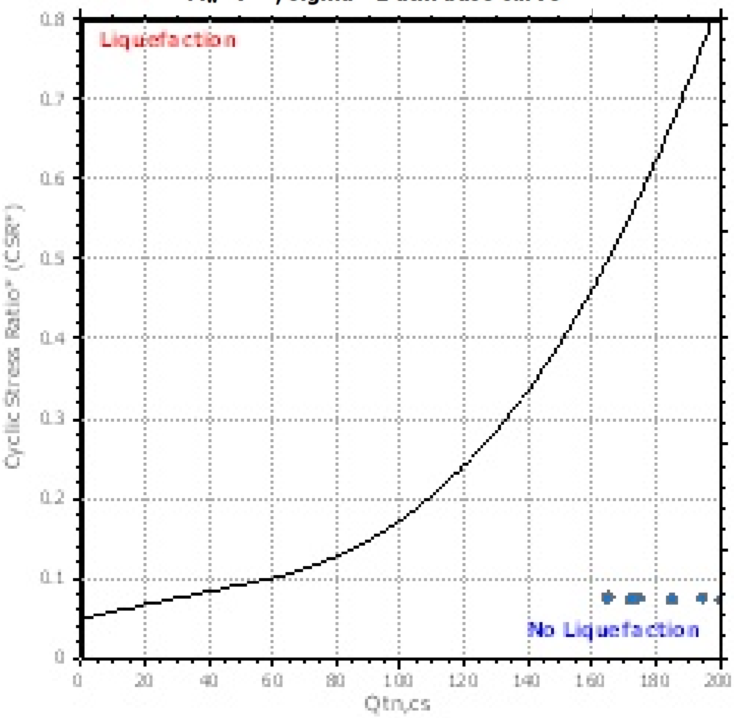
CPT file : CP043

Input parameters and analysis data

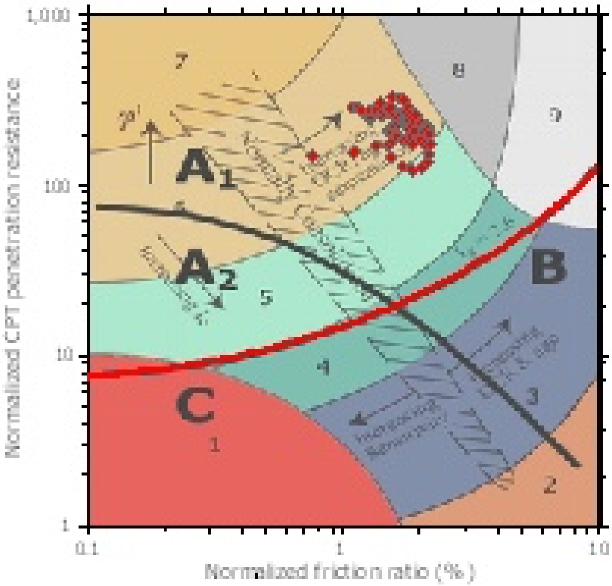
Analysis method:	NCEER (1998)	G.W.T. (in-situ):	2.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	2.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.00	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.09	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		



$M_w=7^{1/2}$, sigma'=1 atm base curve



Summary of liquefaction potential



Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

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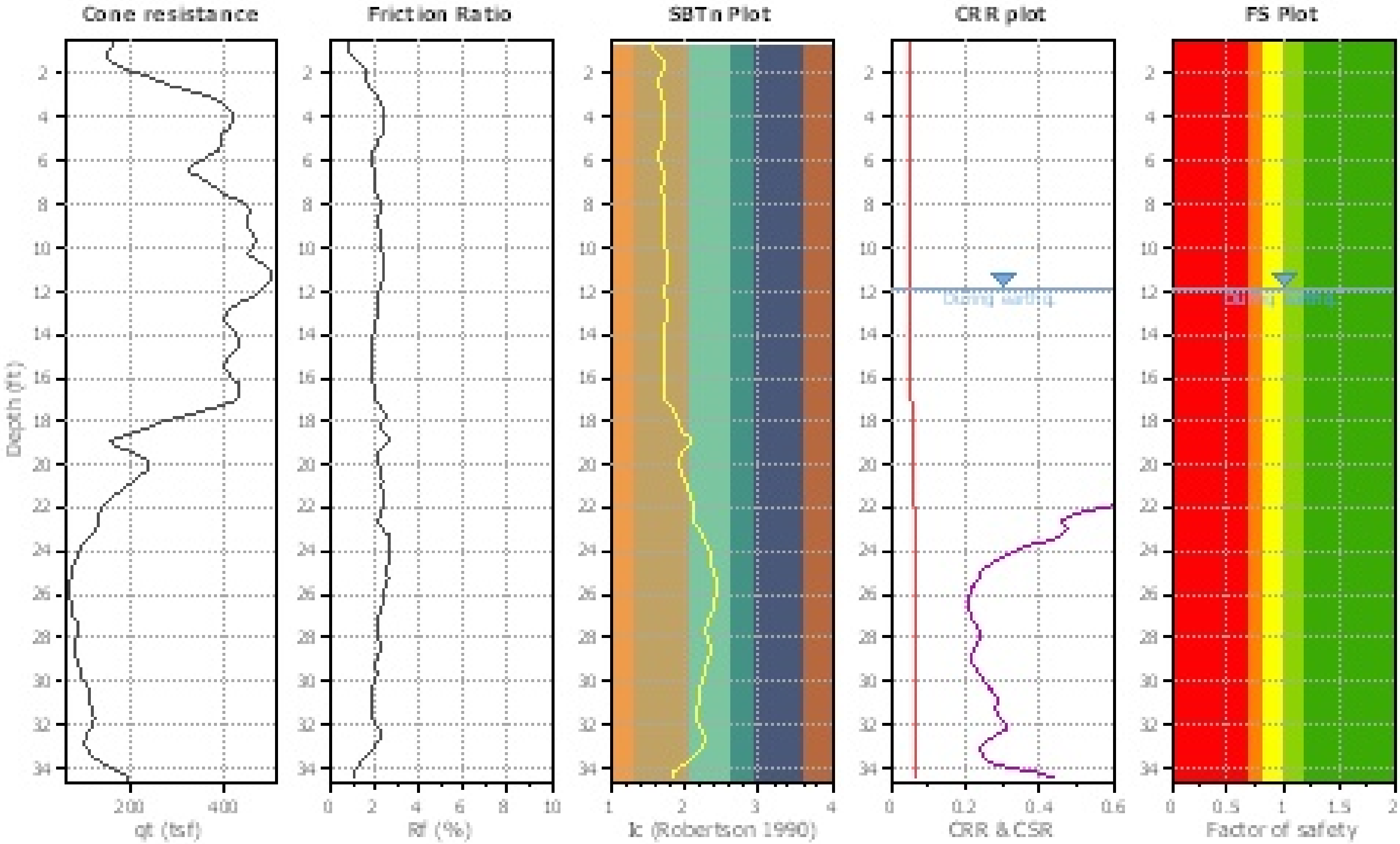
LIQUEFACTION ANALYSIS REPORT

Project title : NIPSCO Schafer **Location : Wheatfield, IN**

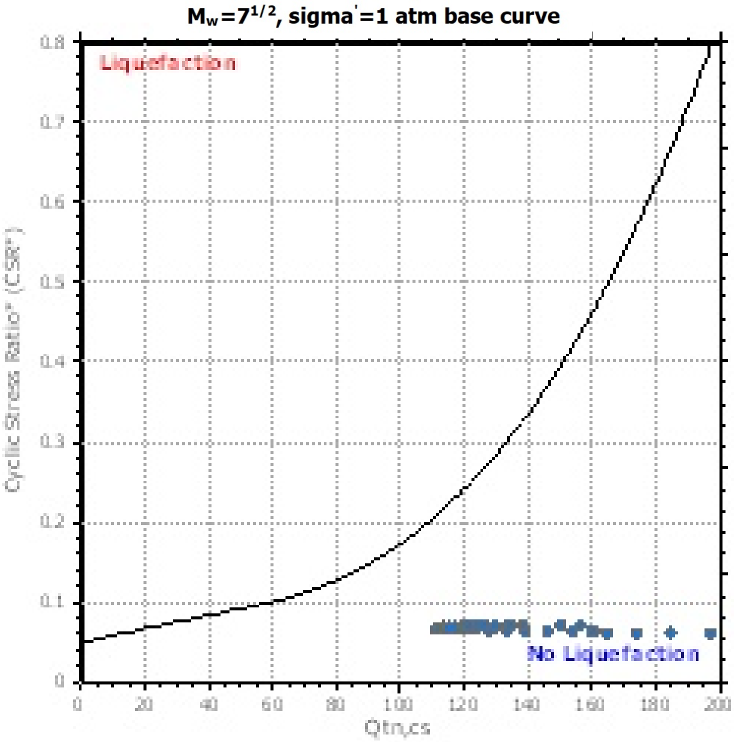
CPT file : CP044

Input parameters and analysis data

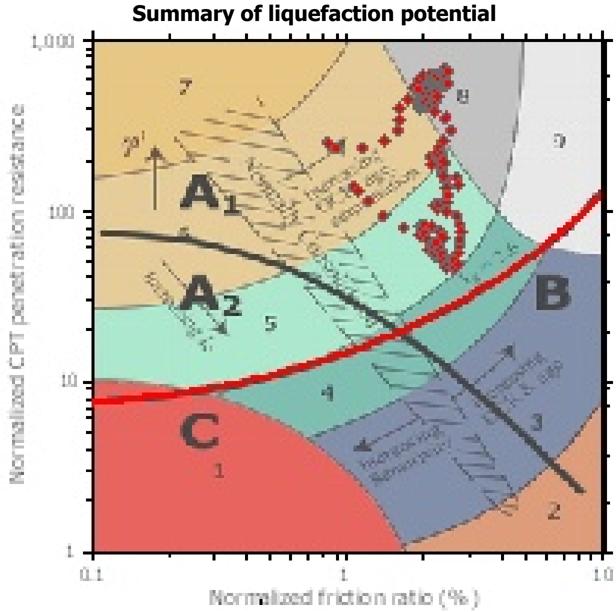
Analysis method:	NCEER (1998)	G.W.T. (in-situ):	11.80 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	11.80 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.00	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.09	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		



$M_w=7^{1/2}$, sigma'=1 atm base curve



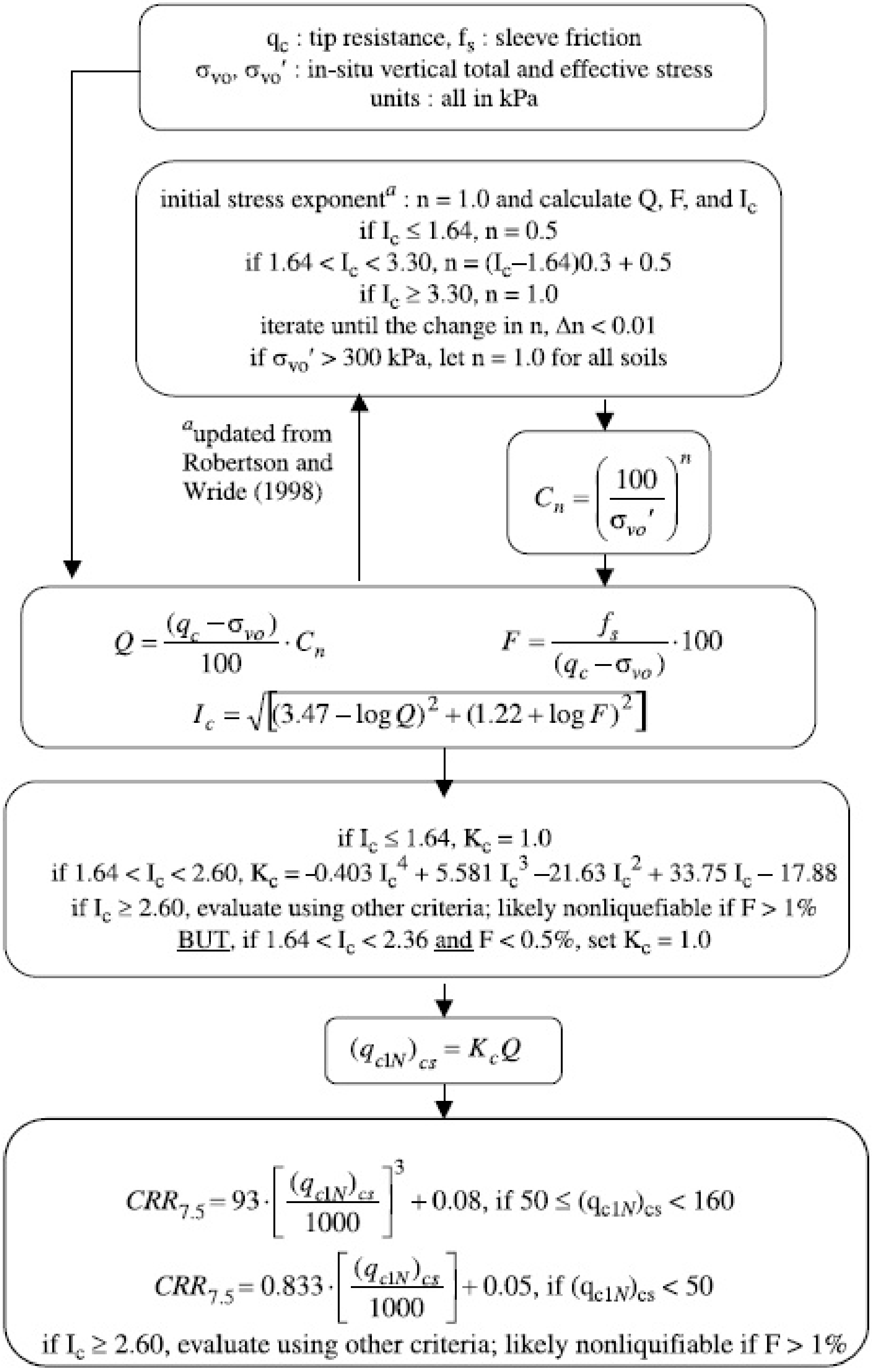
Summary of liquefaction potential



Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Procedure for the evaluation of soil liquefaction resistance, NCEER (1998)

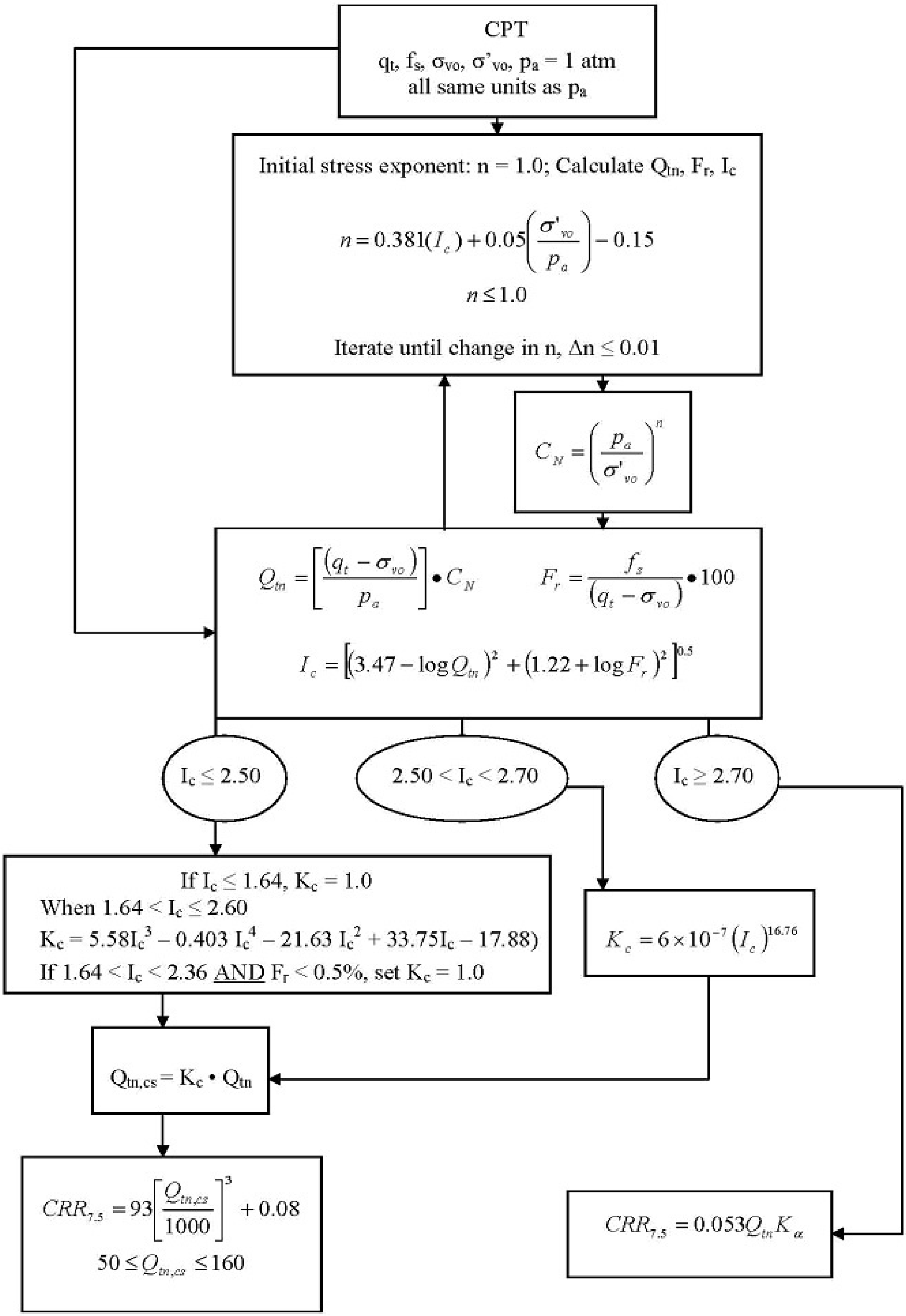
Calculation of soil resistance against liquefaction is performed according to the Robertson & Wride (1998) procedure. The procedure used in the software, slightly differs from the one originally published in NCEER-97-0022 (Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils). The revised procedure is presented below in the form of a flowchart[1]:



[1] "Estimating liquefaction-induced ground settlements from CPT for level ground", G. Zhang, P.K. Robertson, and R.W.I. Brachman

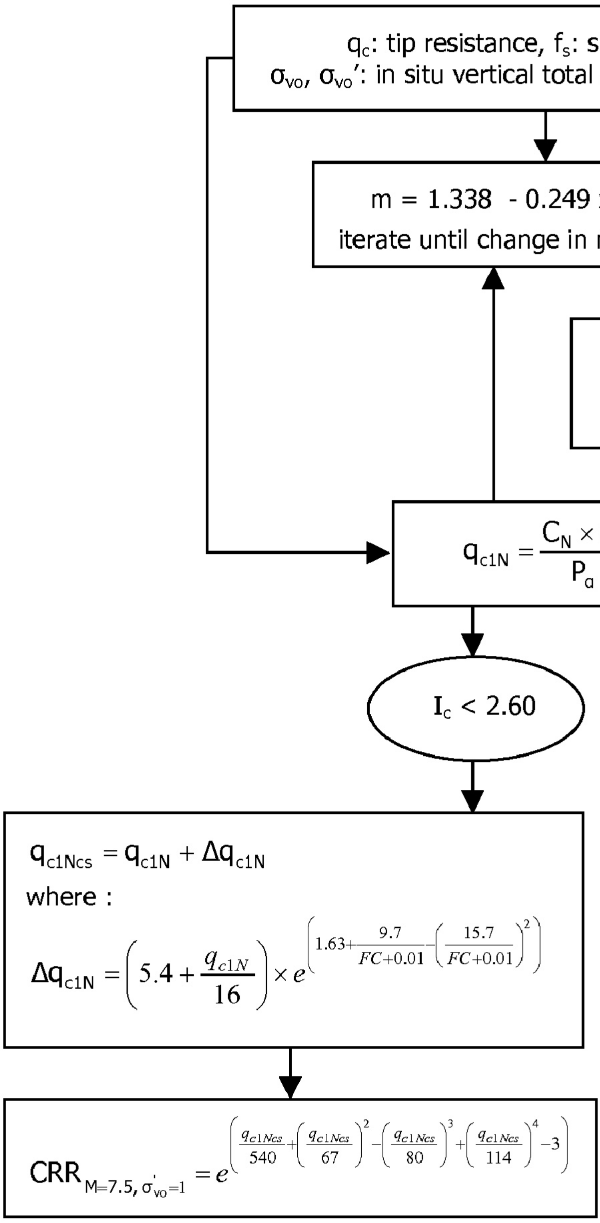
Procedure for the evaluation of soil liquefaction resistance (all soils), Robertson (2010)

Calculation of soil resistance against liquefaction is performed according to the Robertson & Wride (1998) procedure. This procedure used in the software, slightly differs from the one originally published in NCEER-97-0022 (Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils). The revised procedure is presented below in the form of a flowchart[1]:



[1] P.K. Robertson, 2009. "Performance based earthquake design using the CPT", Keynote Lecture, International Conference on Performance-based Design in Earthquake Geotechnical Engineering – from case history to practice, IS-Tokyo, June 2009

Procedure for the evaluation of soil liquefaction resistance, Idriss & Boulanger (2008)



Procedure for the evaluation of soil liquefaction resistance (sandy soils), Moss et al. (2006)

CPT

q_t, f_s, I_c

$I_c < I_c$ cut-off

Initial estimate using raw tip measurements, friction ratio. Calculate $q_{t,1}$. Repeat until an acceptable convergence tolerance is achieved.

$$c = f_1 \cdot \left(\frac{R_f}{f_3}\right)^{f_2}$$

$$C_q = \left(\frac{P_a}{\sigma'_v}\right)^c$$

$$q_{t,1} = C_q \cdot q_t$$

$$CRR = \exp\left[\frac{q_{t,1}^{1.045} + q_{t,1} \cdot (0.110 \cdot R_f) + (0.001 \cdot R_f) + c \cdot (1 + 0.850 \cdot R_f) - 0.848 \cdot \ln(M_w) - 0.002 \cdot \ln(\sigma'_v) - 20.923 + 1.632 \cdot \Phi^{-1}(P_L)}{7.177}\right]$$

Procedure for the evaluation of soil liquefaction resistance, Boulanger & Idriss(2014)

$$FS_{liq} = \frac{CRR_{M=7.5,\sigma'_v=1atm}}{CSR_{M=7.5,\sigma'_v=1atm}}$$

$$CSR_{M=7.5,\sigma'_v=1atm} = 0.65\frac{\sigma_v}{\sigma'_v}\frac{a_{\max}}{g}r_d\frac{1}{MSF}\frac{1}{K_\sigma}$$

$$r_d = \exp\left[\alpha(z)+\beta(z)\cdot M\right]$$

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

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

$$K_\sigma = 1-C_\sigma\ln\left(\frac{\sigma'_v}{P_a}\right)\le 1.1$$

$$C_\sigma = \frac{1}{37.3-8.27\left(q_{c1Ncs}\right)^{0.264}}\le 0.3$$

$$MSF = 1+\left(MSF_{max}-1\right)\left(8.64\exp\left(\frac{-M}{4}\right)-1.325\right)$$

$$MSF_{max} = 1.09+\left(\frac{q_{c1Ncs}}{180}\right)^3\le 2.2$$

σ'_v *and* σ_v *at start of earthquake shaking*

$$CRR_{M=7.5,\sigma'_v=1atm} = \exp\left(\frac{q_{c1Ncs}}{113}+\left(\frac{q_{c1Ncs}}{1000}\right)^2-\left(\frac{q_{c1Ncs}}{140}\right)^3+\left(\frac{q_{c1Ncs}}{137}\right)^4-2.80\right)$$

$$q_{c1Ncs} = q_{c1N}+\Delta q_{c1N}$$

$$\Delta q_{c1N} = \left(11.9+\frac{q_{c1N}}{14.6}\right)\exp\left(1.63-\frac{9.7}{FC+2}-\left(\frac{15.7}{FC+2}\right)^2\right)$$

$$q_{c1N} = C_N\frac{q_c}{P_a}$$

$$C_N = \left(\frac{P_a}{\sigma'_v}\right)^m\le 1.7$$

$$m = 1.338-0.249\left(q_{c1Ncs}\right)^{0.264}\quad with\quad 0.264\le m\le 0.782$$

$$FC = 80\left(I_C+C_{FC}\right)-137\quad with\quad 0\%\le FC\le 100\%$$

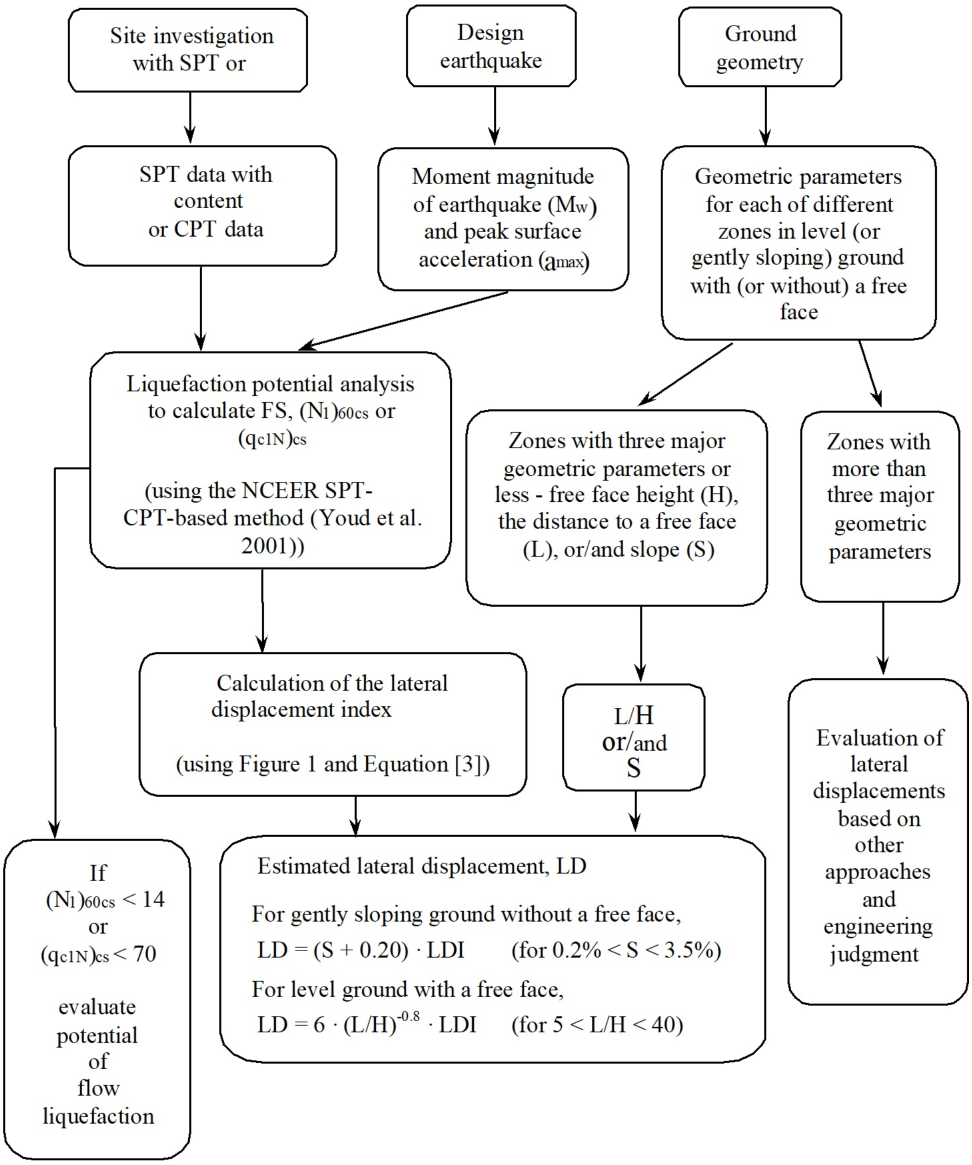
$$I_c = \left[\left(3.47-\log(Q)\right)^2+\left(1.22+\log(F)\right)^2\right]^{0.5}$$

$$Q = \left(\frac{q_c-\sigma_{vc}}{P_a}\right)\left(\frac{P_a}{\sigma'_{vc}}\right)^n\ with\ 0.5\le n\le 1.0\ per\ Robertson\ \&\ Wride\ (1998)$$

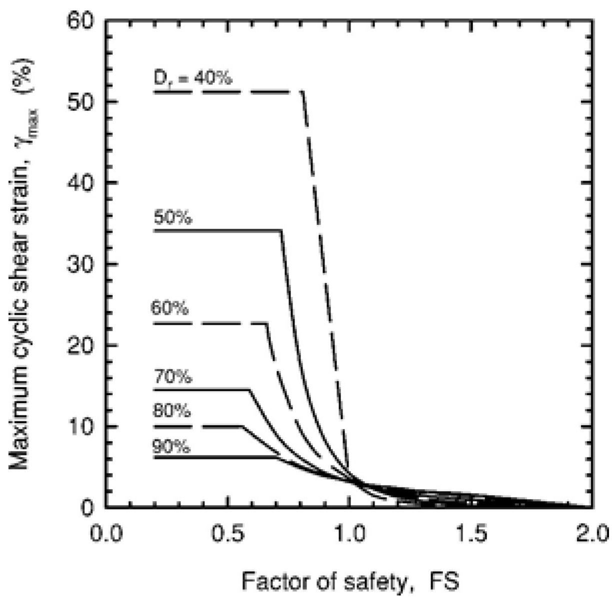
$$F = \left(\frac{f_s}{q_c-\sigma_{vc}}\right)\cdot 100\%$$

σ'_v *at time of CPT sounding*

Procedure for the evaluation of liquefaction-induced lateral spreading displacements



[1] Flow chart illustrating major steps in estimating liquefaction-induced lateral spreading displacements using the proposed approach



[1] **Figure 1**

$$LDI = \int_0^{Z_{max}} \gamma_{max} dz$$

[1] **Equation [3]**

[1] "Estimating liquefaction-induced ground settlements from CPT for level ground", G. Zhang, P.K. Robertson, and R.W.I. Brachman

Procedure for the estimation of seismic induced settlements in dry sands

Average shear stress, τ_{av}

$$\tau_{av} = CSR^{*} \cdot \sigma_{v0}' = 0.65 \cdot \frac{a_{max}}{g} \cdot \sigma_{v0} \cdot r_d$$

Estimate small shear strain modulus, G_0

$$G_0 = 0.0188 \cdot \left[10^{(0.55 I_c - 1.68)}\right] \cdot (q_t - \sigma_v)$$

Estimate shear strain amplitude, γ

(based on Pradel (1998))

$$\gamma = \left[\frac{1 + a \cdot e^{bR}}{1 + a}\right] \cdot R \cdot 100\ (\%)$$

$$R = \frac{\tau_{av}}{G_0} \text{ (Note } \tau_{av} \text{ and } G_0 \text{ same units)}$$

$$a = 0.0389 \cdot \left(\frac{\sigma_v}{p_a}\right) + 0.124$$

$$b = 6400 \cdot \left(\frac{\sigma_v}{p_a}\right)$$

Estimate volumetric strain in 15 cycles

$$\varepsilon_{vol(15)} = \gamma \cdot \left[\frac{(N_1)_{60,cs}}{20}\right]^{-1.20}$$

$$(N_1)_{60,cs} = \frac{Q_{tn,cs}}{8.5 \cdot \left[1 - \frac{I_c}{4.6}\right]}$$

Volumetric strain in design earthquake

$$\Sigma_{vol} = \varepsilon_{vol(15)} \cdot \left[\frac{N_c}{15}\right]^{0.45}$$

$$N_c = (M - 4)^{2.17}$$

Seismic settlement, s

$$s = 2 \cdot \int_0^{GWT} \Sigma_{vol} \cdot dz$$

Robertson, P.K. and Lisheng, S., 2010, "Estimation of seismic compression in dry soils using the CPT" FIFTH INTERNATIONAL CONFERENCE ON RECENT ADVANCES IN GEOTECHNICAL EARTHQUAKE ENGINEERING AND SOIL DYNAMICS, Symposium in honor of professor I. M. Idriss, San Diego, CA

Liquefaction Potential Index (LPI) calculation procedure

Calculation of the Liquefaction Potential Index (LPI) is used to interpret the liquefaction assessment calculations in terms of severity over depth. The calculation procedure is based on the methology developed by Iwasaki (1982) and is adopted by AFPS.

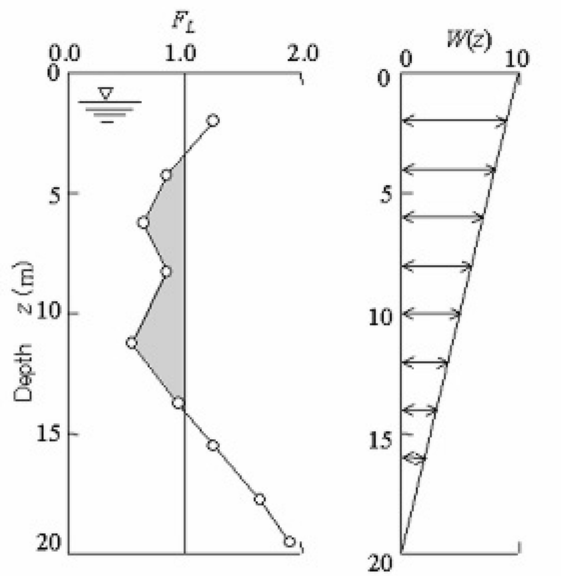
To estimate the severity of liquefaction extent at a given site, LPI is calculated based on the following equation:

$$\textbf{LPI} = \int_0^{20} (10 - 0{,}5_z) \times F_L \times d_z$$

where:
F_L = 1 - F.S. when F.S. less than 1
F_L = 0 when F.S. greater than 1
z depth of measurment in meters

Values of LPI range between zero (0) when no test point is characterized as liquefiable and 100 when all points are characterized as susceptible to liquefaction. Iwasaki proposed four (4) discrete categories based on the numeric value of LPI:

- LPI = 0 : Liquefaction risk is very low
- 0 < LPI <= 5 : Liquefaction risk is low
- 5 < LPI <= 15 : Liquefaction risk is high
- LPI > 15 : Liquefaction risk is very high



Graphical presentation of the LPI calculation procedure

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