NIPSCO R.M. SCHAFFER 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
2723 East 1500 North
Wheatfield, IN 46392

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

October 5, 2016
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

Tiffany Johnson, PE
Name

PE 11500730
Indiana Professional Engineer License Number
Table of Contents

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

1. INTRODUCTION.............................................................................................................................. 1
   1.1 Purpose ........................................................................................................................................ 1
   1.2 WDA Background ......................................................................................................................... 1
   1.3 Previous Evaluations .................................................................................................................... 2

2. SITE DESCRIPTION ........................................................................................................................ 4
   2.1 SUBSURFACE CONDITIONS..................................................................................................... 4
   2.2 Physical Properties of Foundation Materials ............................................................................ 4
   2.3 Engineering Properties of Foundation Materials ..................................................................... 4
   2.4 Waste Disposal Area Design and Construction Details ........................................................... 6

3. STRUCTURAL STABILITY ASSESSMENT - § 257.73(D)(1)(I)-(VII) .............................................. 8
   3.1 Foundations and Abutments - §257.73(d)(1)(i) ............................................................................ 8
   3.2 Slope Protection - §257.73(d)(1)(ii) ............................................................................................. 8
   3.3 Dikes (Embankment) - §257.73(d)(1)(iii) .................................................................................... 9
   3.4 Vegetated Slopes - §257.73(d)(1)(iv) ........................................................................................... 9
   3.5 Spillways - §257.73(d)(1)(v) ......................................................................................................... 9
   3.6 Hydraulic Structures - §257.73(d)(1)(vi) ..................................................................................... 11
   3.7 Downstream Slopes Adjacent to Water Body - §257.73(d)(1)(vii) ............................................. 11
   3.8 Structural Stability Deficiencies - §257.73(d)(2) ......................................................................... 11

4. SAFETY FACTOR ASSESSMENT - § 257.73(E) ......................................................................... 12
   4.1 Slope Stability Analysis .............................................................................................................. 12
      4.1.1 Cross-Section Analyzed ......................................................................................................... 13
      4.1.2 Geotechnical Material Properties ........................................................................................ 13
      4.1.3 Seismic analysis .................................................................................................................... 13
      4.1.4 Factor of Safety Results ....................................................................................................... 13
   4.2 Liquefaction Potential Assessment ............................................................................................ 14

5. CONCLUSIONS AND RECOMMENDATIONS ........................................................................... 15

6. CLOSING ....................................................................................................................................... 16

7. REFERENCES ................................................................................................................................... 17

List of Tables

Table 1  Previous Evaluations Related to Structural Stability Assessment
Table 2  Geotechnical Model Material Properties
Table 3  Slope Stability Analysis Results Summary
Table 4  Hydrology and Hydraulics Analysis Results
Table 5  Slope Stability Analysis Results Summary
List of Figures

Figure 1  Site Location Map  
Figure 2  Site Plan (with CPT Borehole Locations Noted)  
Figure 3  Waste Disposal Area  
Figure 4  Typical Cross Section  
Figure 5  Typical Geotechnical Model at the Waste Disposal Area Embankment  

List of Appendices

Appendix A  Slope Stability Analysis Results  
Appendix B  Liquefaction Potential Analysis Results
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

<table>
<thead>
<tr>
<th>Document</th>
<th>Date</th>
<th>Author</th>
</tr>
</thead>
<tbody>
<tr>
<td>Various construction drawings</td>
<td>1982</td>
<td>Sargent &amp; Lundy Engineers</td>
</tr>
<tr>
<td>Assessment of Dam Safety of Coal Combustion Surface Impoundments, NIPSCO, RM Schahfer Generating Station</td>
<td>July 2010</td>
<td>CDM for the EPA</td>
</tr>
<tr>
<td>Embankment Elevation Survey, Waste Disposal Area and Recycle Pond, NIPSCO Schahfer Generating Station</td>
<td>December 2011</td>
<td>Marbach, Brady and Weaver, Inc.</td>
</tr>
<tr>
<td>Schahfer Spillway Hydrologic and Hydraulic Evaluation</td>
<td>December 2011</td>
<td>Golder Associates Inc.</td>
</tr>
<tr>
<td>Final Geotechnical Investigation and Embankment Stability Analyses</td>
<td>June 2012</td>
<td>Golder Associates Inc.</td>
</tr>
<tr>
<td>Construction in a Floodway Permit Application, NIPSCO R.M. Schahfer Generating Station</td>
<td>November 2012</td>
<td>Golder Associates Inc.</td>
</tr>
<tr>
<td>Waste Disposal and Recycle Ponds Hydrographic Survey, NIPSCO R.M. Schahfer Generating Station</td>
<td>December 2012</td>
<td>DLZ Industrial, LLC (DLZ)</td>
</tr>
<tr>
<td>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</td>
<td>February 2013</td>
<td>Golder Associates Inc.</td>
</tr>
<tr>
<td>State of Indiana Department of Natural Resources (DNR), Certificate of Approval, After-the-Fact, Construction in a Floodway</td>
<td>April 23, 2013</td>
<td>State of Indiana DNR</td>
</tr>
<tr>
<td>---------------------------------------------------------------</td>
<td>----------------</td>
<td>---------------------</td>
</tr>
</tbody>
</table>
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 probes 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

<table>
<thead>
<tr>
<th>Material</th>
<th>Internal Friction Angle (deg.)</th>
<th>Peak Cohesion (psf)</th>
<th>Dry Unit Weight (pcf)</th>
<th>Saturated Unit Weight (pcf)</th>
<th>Undrained Shear Strength (psf)</th>
<th>Layer Thickness (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Embankment Fill</td>
<td>42</td>
<td>0</td>
<td>125</td>
<td>135</td>
<td>NA</td>
<td>Varies</td>
</tr>
<tr>
<td>Topsoil</td>
<td>35</td>
<td>0</td>
<td>120</td>
<td>120</td>
<td>NA</td>
<td>0.5</td>
</tr>
<tr>
<td>Existing Subgrade</td>
<td>39</td>
<td>0</td>
<td>110</td>
<td>125</td>
<td>NA</td>
<td>Varies</td>
</tr>
<tr>
<td>Slurry Wall</td>
<td>NA</td>
<td>300</td>
<td>120</td>
<td>NA</td>
<td>NA</td>
<td>Varies</td>
</tr>
<tr>
<td>Riprap</td>
<td>45</td>
<td>0</td>
<td>140</td>
<td>145</td>
<td>NA</td>
<td>1</td>
</tr>
<tr>
<td>Crushed Stone</td>
<td>45</td>
<td>0</td>
<td>140</td>
<td>145</td>
<td>NA</td>
<td>Varies</td>
</tr>
<tr>
<td>Loose Silty Sand Subgrade</td>
<td>37</td>
<td>0</td>
<td>125</td>
<td>132</td>
<td>NA</td>
<td>Varies</td>
</tr>
<tr>
<td>Shale</td>
<td>45</td>
<td>0</td>
<td>145</td>
<td>150</td>
<td>0</td>
<td>Varies</td>
</tr>
</tbody>
</table>

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
- 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**

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth of Precipitation (in) for a PMF Event</td>
<td>31.9</td>
</tr>
<tr>
<td>WDA Catchment Area (acres)</td>
<td>83.5</td>
</tr>
<tr>
<td>WDA Lowest Crest Elevation (ft MSL, Marbach, 2011)</td>
<td>680</td>
</tr>
<tr>
<td>Invert Elevation of Auxiliary Spillway (ft MSL)</td>
<td>678.9</td>
</tr>
<tr>
<td>Maximum Inflow from Direct Precipitation (cubic feet per second (cfs))</td>
<td>3,668</td>
</tr>
<tr>
<td>Maximum Combined Inflow (cfs)</td>
<td>3,708</td>
</tr>
<tr>
<td>Maximum WDA Outflow through Spillway (cfs)</td>
<td>37.9</td>
</tr>
<tr>
<td>Maximum Water Surface Elevation (ft MSL)</td>
<td>682.2</td>
</tr>
<tr>
<td>Height of Wave Action (feet)</td>
<td>1.28</td>
</tr>
<tr>
<td>Net Freeboard during Design Storm Event (feet)</td>
<td>-3.4</td>
</tr>
</tbody>
</table>

**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

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

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.

GOLDER ASSOCIATES INC.

Tiffany Johnson, P.E.
Senior Consultant

David M. List, P.E.
Principal
7. REFERENCES


FIGURES
REFERENCE

BASE MAP TAKEN FROM MARBACH, BRADY & WEAVER ENGINEERING & SURVEYING; EMBANKMENT ELEVATION SURVEY; DRAWN BY A.J. GUTIERREZ. FILE NO. 1651599. DATED 2011-12-30 DELIVERED IN DWG FORMAT.
WASTE DISPOSAL AREA
TYPICAL PROFILE

CRUSHED STONE RIPRAPP

TOP OF DIKE EL 681'
HIGH WATER EL 679'
LOW WATER EL 672'

EMBANKMENT FILL

SUBGRADE

SLURRY WALL

SHALE

REFERENCE:
FROM SARGENT & LUNDY CONSTRUCTION. DRAWING C-19: SETTLING BASINS DEDICATION.
APPENDIX A
SLOPE STABILITY ANALYSIS RESULTS
2008 United States National Seismic Hazard Maps

The 2008 U.S. Geologic 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 risk analysis, risk assessments, and other public policy. The values of the maps incorporate the latest findings on earthquake ground shaking, faults, and regional geology. The resulting maps are derived from the latest seismic source scenarios developed at the USGS and in collaboration with other national and regional seismic hazard experts.

The USGS National Seismic Hazard Mapping Project developed these maps by using an innovative methodology, which included a comprehensive review of existing seismic hazard maps, a comprehensive review of existing seismic hazard maps, and a comprehensive review of existing seismic hazard maps. The project also used a variety of data sources, including historical earthquake data, geologic data, and seismological data, to develop a comprehensive understanding of the seismic hazard in the United States.
257.73(e)(1)(i) - Maximum Storage Pool
Waste Disposal Area

Block Analysis - No Seismic

Results
janbu corrected

2.624
### Material Properties

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

### Project Information

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

Rotational Analysis - No Seismic

Results

janbu corrected

2.739
257.73(e)(1)(iii) - Maximum Pool Storage
Waste Disposal Area

Rotational Analysis - Seismic

Results
janbu corrected

1.811
257.73(e)(1)(ii) - Maximum Surcharge Pool Loading

Waste Disposal Area
Block - No Seismic

Results
janbu corrected
Every available surface

<table>
<thead>
<tr>
<th>Material Name</th>
<th>Color</th>
<th>Unit Weight (lbs/ft³)</th>
<th>Strength Type</th>
<th>Cohesion (psf)</th>
<th>Phi (deg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>EMBANKMENT SOIL</td>
<td></td>
<td>125</td>
<td>Mohr-Coulomb</td>
<td>0</td>
<td>42</td>
</tr>
<tr>
<td>SHALE</td>
<td></td>
<td>145</td>
<td>Mohr-Coulomb</td>
<td>0</td>
<td>45</td>
</tr>
<tr>
<td>SUBGRADE</td>
<td></td>
<td>110</td>
<td>Mohr-Coulomb</td>
<td>0</td>
<td>39</td>
</tr>
<tr>
<td>SLURRY WALL</td>
<td></td>
<td>120</td>
<td>Mohr-Coulomb</td>
<td>300</td>
<td>0</td>
</tr>
<tr>
<td>RIP RAP</td>
<td></td>
<td>140</td>
<td>Mohr-Coulomb</td>
<td>0</td>
<td>45</td>
</tr>
<tr>
<td>CRUSHED STONE</td>
<td></td>
<td>140</td>
<td>Mohr-Coulomb</td>
<td>0</td>
<td>45</td>
</tr>
<tr>
<td>TOPSOIL</td>
<td></td>
<td>120</td>
<td>Mohr-Coulomb</td>
<td>15</td>
<td>35</td>
</tr>
</tbody>
</table>
257.73(e)(1)(iii) - Maximum Surcharge Pool Loading

Waste Disposal Area

Block - Seismic

Results janbu corrected

Every available surface
257.73(e)(1)(iii) - Maximum Surcharge

Waste Disposal Area

Rotational - Seismic

Results
janbu corrected

1.777
APPENDIX B
LIQUEFACTION POTENTIAL ANALYSIS RESULTS
CALCULATED LIQUEFACTION FACTOR OF SAFETY

Note: Factor of safeties (FS) greater than 2 are shown equal to 2.
At Golder Associates we strive to be the most respected global group of companies specializing in ground engineering and environmental services. Employee owned since our formation in 1960, we have created a unique culture with pride in ownership, resulting in long-term organizational stability. Golder professionals take the time to build an understanding of client needs and of the specific environments in which they operate. We continue to expand our technical capabilities and have experienced steady growth with employees now operating from offices located throughout Africa, Asia, Australasia, Europe, North America and South America.