ALLIANT ENERGY
Wisconsin Power and Light Company
Ottumwa Generating Station

CCR SURFACE IMPOUNDMENT

SAFETY FACTOR ASSESSMENT

Report Issued: September 29, 2016
Revision 0
EXECUTIVE SUMMARY

This Safety Factor Assessment (Report) is prepared in accordance with the requirements of the United States Environmental Protection Agency (USEPA) published Final Rule for Hazardous and Solid Waste Management System – Disposal of Coal Combustion Residual (CCR) from Electric Utilities (40 CFR Parts 257 and 261, also known as the CCR Rule) published on April 17, 2015 and effective October 19, 2015.

This Report assess the safety factors of each CCR unit at Ottumwa Generating Station in Ottumwa, Iowa in accordance with §257.73(b) and §257.73(e) of the CCR Rule. For purposes of this Report, “CCR unit” refers to an existing or inactive CCR surface impoundment.

Primarily, this Report is focused on assessing if each CCR surface impoundment achieves the minimum safety factors, which include:

- Static factor of safety under long-term, maximum storage pool loading condition,
- Static factor of safety under the maximum surcharge pool loading condition,
- Seismic factor of safety; and,
- Post-Liquefaction factor of safety for embankments constructed of soils that have susceptibility to liquefaction.
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1 Introduction

The owner or operator of the Coal Combustion Residual (CCR) unit must conduct an initial and periodic safety factor assessments to determine if each CCR surface impoundment achieves the minimum safety factors, which include:

- Static factor of safety under long-term, maximum storage pool loading condition,
- Static factor of safety under the maximum surcharge pool loading condition,
- Seismic factor of safety; and,
- Post-Liquefaction factor of safety for embankments constructed of soils that have susceptibility to liquefaction.

This Report has been prepared in accordance with the requirements of §257.73(b) and §257.73(e) of the CCR Rule.

1.1 CCR Rule Applicability

The CCR Rule requires a periodic safety factor assessment by a qualified professional engineer (PE) for existing CCR surface impoundments with a height of 5 feet or more and a storage volume of 20 acre-feet or more; or the existing CCR surface impoundment has a height of 20 feet or more.

On August 5th, 2016, USEPA published revisions to the CCR Rule (the “Extension Rule”) that extend the above requirements to inactive CCR surface impoundments with different deadlines. The effective date of the Extension Rule is October 4th, 2016.

1.2 Safety Factor Assessment Applicability

The Ottumwa Generating Station (OGS) in Ottumwa, Iowa (Figure 1) has one existing and one inactive CCR surface impoundments, identified as follows:

- OGS Ash Pond (existing)
- OGS Zero Liquid Discharge Pond (inactive)
Each of the identified CCR surface impoundments meet the requirements of §257.73(b)(1) and/or §257.73(b)(2), they are subject to the periodic safety factor assessment requirements of §257.73(e) of the CCR Rule.
2 FACILITY DESCRIPTION

OGS is located approximately ten miles northwest of Ottumwa, Iowa on the western shore of the Des Moines River in Wapello County, at 20775 Power Plant Road, Ottumwa, Iowa (Figure 1). The McNeese Wildlife Area is located to the southeast of OGS. Middle Avery Creek, which flows to the northeast into the Des Moines River, is located to the south and east of OGS.

OGS is a fossil-fueled electric generating station consisting of one steam electric generating unit. Sub-bituminous coal is the primary fuel for producing steam. The burning of coal produces a by-product of CCR. The CCR at OGS is categorized into three types; bottom ash, fly ash, and flue gas desulfurization (scrubber) byproducts. The fly ash also can be subdivided into two types, economizer fly ash and precipitator fly ash.

The majority of precipitator fly ash is collected by the electrostatic precipitators and sent to the on-site storage silo located on the west side of the generating plant. Historically, the precipitator fly ash has then either been transported off-site for beneficial reuse or was placed in the fly ash reclamation processing area adjacent to the coal pile storage area for the purposes of producing hydrated fly ash. In the fly ash reclamation processing area, the fly ash was rolled out, compacted, hydrated, and allowed to dry into a very hard, cement-like material that was stored in this area until transported off-site. Although this fly ash hydrating process has occurred in the past, this process ceased prior to October 19, 2015.

The precipitator fly ash that is not collected by the electrostatic precipitators becomes part of the flue gas desulfurization pollution control process at OGS. Activated carbon is injected into the flue gas stream and binds with mercury. This flue gas stream travels to the spray dry desulfurization towers. From there, a water based slurry of hydrated (slaked) lime is injected into the spray dry desulfurization towers. The hydrated lime reacts with the sulfur compounds in the flue gas and the water evaporates. A precipitate is left that consists of activated carbon bound to mercury, calcium sulfate, calcium sulfite,
unreacted slaked lime, and some unreacted fly ash. This flue gas stream is directed to the bag house where the particulate matter is removed. A portion of the solids are recycled back to the process and the rest of the scrubber byproducts are sent to the air quality control system byproduct silo. The material from the byproduct silo is mixed with water in a pin mixer to reduce dust, loaded into trucks, and transported to the off-site Ottumwa-Midland CCR landfill for disposal.

The bottom ash and economizer fly ash at OGS are sluiced to a surface impoundment identified as the OGS Ash Pond (Figure 2). The OGS Ash Pond is located east of the generating plant and is presently the only existing CCR surface impoundment at OGS.

In addition to the OGS Ash Pond, OGS has one inactive CCR surface impoundment identified as the OGS Zero Liquid Discharge (ZLD) Pond. The OGS ZLD Pond is located northeast of the generating plant and north of the OGS Ash Pond. The OGS ZLD Pond, presently, only receives surface water runoff from the surrounding area.

General Facility Information:

- Date of Initial Facility Operations: 1981
- NPDES Permit Number: IA90-001-01
- Latitude / Longitude: 41°5’53”N 92°33’17”W
- Nameplate Ratings: Unit 1 (1981) 725 MW

2.1 OGS Ash Pond
The OGS Ash Pond is located east of the generating plant on the eastern portion of the site. The OGS Ash Pond receives influent flows from the generating plant floor drains, oil/water separator, boiler blow down water, solid contact unit sludge, sluiced CCR (bottom ash and economizer fly ash), recirculating media sanitary treatment plant, and surface water runoff from the generating site proper.
The sluiced CCR is discharged into the west end of the OGS Ash Pond. The sluiced CCR is discharged into a collection pad area where the majority of CCR is recovered. A dozer is used to scrape the collection pad and push the CCR into a stockpile for dewatering. Once dewatered, the CCR is then loaded into over-the-road haul trucks for transporting off-site. The sluiced water from the CCR drains into a narrow channel that flows into the southwest portion of the OGS Ash Pond. Routine maintenance dredging of the narrow channel occurs as the CCR settles out in the channel. Approximately 4 million gallons per day (MGD) of process water is recirculated back into OGS for reuse.

The water in the OGS Ash Pond from other sources flows to the east and discharges through the facility’s National Pollution Discharge Elimination System (NPDES) Outfall 001, located in the northeast corner of the OGS Ash Pond. NPDES Outfall 001 consists of a concrete discharge structure with a six foot wide overflow weir and includes a Parshall flume and instrumentation to measure the flow of the discharged water. The water flows through the NPDES Outfall 001 and discharges into an unnamed creek at an average rate of 1.54 MGD. The water flows through the NPDES Outfall 001 and discharges into an unnamed creek. The unnamed creek flows into the Des Moines River downstream of the water intake structure and before the confluence of Middle Avery Creek.

The surface area of the OGS Ash Pond is approximately 18 acres and has an embankment height of approximately 25 feet from the crest to the toe of the downstream slope. The interior storage depth of the OGS Ash Pond is approximately 20 feet. Currently, the total volume of impounded CCR and water within the OGS Ash Pond is approximately 556,000 cubic yards.

2.2 OGS Zero Liquid Discharge Pond

The OGS Zero Liquid Discharge (ZLD) Pond is located northeast of the generating plant on the eastern portion of the site and north of the OGS Ash Pond. The OGS ZLD Pond historically received influent flows from the generating plant that consisted of boiler wash water, air heater wash, turbine chemical cleaning water, and boiler chemical
cleaning water. Presently, the OGS ZLD Pond only receives storm water runoff from the surrounding area, which includes the inactive hydrated fly ash area located west of the surface impoundment, as well as occasional excess storm water runoff from the coal pile storage area. One 24-inch diameter high-density polyethylene culvert connects the coal pile runoff pond to the OGS ZLD Pond. The culvert is used as an emergency overflow to route storm water from the coal pile runoff pond into the OGS ZLD Pond.

The OGS ZLD Pond does not currently discharge. Two 48-inch diameter concrete culverts, located along the south embankment, previously connected the OGS ZLD Pond to the OGS Ash Pond prior to being permanently sealed off with concrete.

The OGS ZLD Pond covers a surface area of approximately 19 acres and has an embankment height of approximately 29 feet from crest to toe of the downstream slope. The interior storage depth of the OGS ZLD Pond is approximately 25 feet. Based on readily available information, the OGS ZLD Pond has a total storage capacity of approximately 515,000 cubic yards.
3 SAFETY FACTOR ASSESSMENT- §257.73(e)

This Report documents if each CCR surface impoundment achieves the minimum safety factors, which are identified on the table below.

<table>
<thead>
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<th>Safety Factor Assessment</th>
<th>Minimum Safety Factor</th>
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<tr>
<td>Static Safety Factor Under Maximum Surcharge Pool Loading</td>
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<tr>
<td>Seismic Safety Factor</td>
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<tr>
<td>Post-Liquefaction Safety Factor</td>
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3.1 Safety Factor Assessment Methods

The safety factor assessment is completed with the two dimensional limit-equilibrium slope stability analyses program STABL5M (1996)\(^1\). The program analyzes many potential failure circles or block slides by random generation of failure surfaces using the toe and crest search boundaries set for each analysis. The solution occurs by balancing the resisting forces along the failure plane due to the Mohr-Columb failure strength parameters of friction angle and cohesion. The gravity driving forces are divided by the resisting forces to produce a safety factor for the slope. The minimum of hundreds of searches is presented as the applicable safety factor.

There are both total stress and effective stress friction angle and cohesion values for clay. For the total stress case clay has only cohesion. For effective stress clay has both cohesion and friction angle. When clay receives a load that is applied only briefly (i.e., earthquake or high water), it responds as a total stress soil. For long term loadings such as normal water elevation, the clay resistance to failure is based on effective stress parameters. The total stress parameters for compacted and stiff clay yield a conservative answer for safety

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\(^1\) STABL User Manual by Ronald A. Siegal, Purdue University, June 4, 1975 and STABL5 – The Spencer Method of Slices: Final Report by J. R. Carpenter, Purdue University, August 28, 1985
factor, and the static analysis with normal operating water elevation is performed with the total stress parameters for the clay components in the embankments.

3.1.1 Soil Conditions in and under the impoundments
The soil conditions at the embankments is documented by SCS Engineers\(^2\) boring logs MW-304 and MW-305, Figure 2. The results indicate that the embankments of both impoundments are constructed of stiff compacted clay from the site overlying the medium stiff native clay which overlies very dense sand of the Des Moines River. The boring logs are shown in Appendix A.

During the construction of the OGS in 1974, the native clay was sampled and tested for Atterberg limits, unconfined compressive strength and both consolidated undrained (CU) and unconsolidated undrain (UU) triaxial strength. The test results are shown in Appendix B and indicated that the native clay under the embankments is a low plasticity clay (CL) with unconfined compression values from 1,500 to 2,500 psf. Triaxial UU tests indicated a range of 750 to 2,000 psf for cohesion and the CU tests indicated 29° to 34° for friction angle and 0 to 600 psf cohesion. The CU test results imply the clay is normally consolidated.

Information on the compacted clay and river valley sand is available from the SCS soil boring standard split spoon (SPT) blowcount information, Appendix A. The Terzaghi and Peck relationship of SPT blowcount to clay cohesion for the average blowcounts in each clay layer yields a value of cohesion of 1,000 psf for the native clay and 1,600 psf for the embankment clay, Appendix C. The very dense sand is assigned a friction angle of 38°, based on the correlation of cohesionless soil strength to density provided in NAVFACs DM-7\(^3\), Appendix C.

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\(^3\) Naval Facilities Engineering Command, Soil Mechanics, Foundations, and Earth Structures, Figure 3-7, NAVFAC DM-7, January 1971
The analysis was completed with a cohesion value of 1,600 psf for the embankment clay, 1000 psf for the native clay and a friction angle of 38° for the very dense sand.

3.1.2 Design water surface in impoundments maximum normal pool and maximum pool under design inflow storm

The OGS Ash Pond receives both circulating sluicing water and other process water sources from the facility. The sluicing water is recirculated back into facility. The other sources of water discharge at an average rate of 1.54 MGD. The impoundment discharge is controlled by a six foot wide weir with its top elevation at approximately 675.5 feet making the normal impoundment water elevation approximately 676 feet. During the design inflow storm the water elevation increases to elevation 677.25 feet.

The OGS ZLD Pond only receives water from storm flows and its normal water elevation is determined by the balance of rainfall and evaporation. The impoundment has a clay bottom and embankment so exfiltration seepage is not significant. The normal water elevation based on topographic surveys is approximately elevation 673 feet. During the design inflow storm the water elevation rises to 675.25 feet.

The water elevation in the embankment is assumed to conservatively exit at the toe of the embankment and saturated the native clay and river sand at the toe. This provides a conservative strength projection for the soils at the toe of the embankment.

3.1.3 Selection of Seismic Design Parameters and Description of Method

The design earthquake ground acceleration is selected from the United States Geologic Survey (USGS) detailed seismic design maps based on the latitude and longitude of the OGS. The peak ground acceleration (PGA) value is selected for a 2% probability of exceedance in 50 years (2,500 year return period) as required by §257.53. Since the site soils are clay with cohesion greater than 1,000 psf, or very dense sand and extend to bedrock at elevation 625 feet, the site class as defined in the 2009 International Building

\[\text{Cross Section KK, Appendix B} \]

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\[\text{Safety Factor Assessment} \]

\[\text{September 29, 2016} \]

\[\text{9} \]
Code 1613.5.5 is Site Class D. For Site Class D the ground surface Peak Ground Acceleration (PGA) for slope stability and liquefaction assessment is 0.058g, Appendix D.

3.1.4 Liquefaction Assessment Method and Parameters
Certain soils may have zero effective stress (liquefaction) during an earthquake or from static shear of a saturated embankment slope. Soils that will liquefy include loose or very loose uniform fine sand or silt, and low plasticity clay (plastic index (PI) of less than 12). The native clay and embankment both have PI higher than 12 and are stiff and medium stiff in consistency. The river valley sand is very dense.

None of the soil types at OGS is susceptible to liquefaction and no analysis of liquefaction potential is required for the embankments.

3.2 OGS Ash Pond
The critical cross-section for the OGS Ash Pond is the location where the embankment toe is closest to Middle Avery Creek, just upstream of the railroad embankment, Figure 2. At this location, top of the creek bank is approximately 25 feet from the toe of the embankment. For determination of safety factors, the bottom of Middle Avery Creek was taken to be in the very dense sand and the water elevation in the creek was set at the same elevation.

3.2.1 Static Safety Factor Assessment Under Maximum Storage Pool Loading - §257.73(e)(1)(i)
The OGS Ash Pond receives 2.4 cubic feet per second of process water flow that discharges over the outlet weir. The process flow maintains a maximum average storage pool of 676 feet in the impoundment. Analysis of both circular and block sliding surfaces, Appendix E, show a minimum factor of safety of 2.1 for the circular failure surface passing through the foundation soil and exiting in Middle Avery Creek.

3.2.2 Static Safety Factor Assessment Under Maximum Surcharge Pool Loading - §257.73(e)(1)(ii)
The OGS Ash Pond will contain the 100 year return period design storm through a combination of storage in the impoundment and discharge to the Middle Avery Creek.
The maximum surcharge pool elevation is 677.25 at the peak of the storm. Analysis for both circular and block sliding surface, Appendix E, show a minimum factor of safety of 2.1 for the circular surface passing through the foundation soil and exiting in Middle Avery Creek.

3.2.3 **Seismic Safety Factor Assessment - §257.73(e)(1)(iii)**
The OGS Ash Pond was assigned a pseudo-static earthquake coefficient equal to 0.058 g acceleration and a vertical downward component equal to \( \frac{2}{3} \) of the horizontal component (0.039 g) as recommended by Newmark\(^5\). Analysis for both a circular and block sliding surface, Appendix E, show a minimum factor of safety of 1.7 for the circular sliding surface through the foundation soil and into Middle Avery Creek.

3.2.4 **Liquefaction Safety Factor Assessment - §257.73(e)(1)(iv)**
The OGS Ash Pond foundation and embankment soils are not susceptible to liquefaction, Section 3.1.4.

3.3 **OGS Zero Liquid Discharge Pond**
The critical cross-section for the OGS ZLD Pond is the location where the embankment is highest in the southern part of the embankment, Figure 2. At this location, the Des Moines River bank is approximately 500 feet to the northeast from the toe of the embankment. For determination of safety factors, the water elevation in the embankment was set at the toe with the native clay in the river valley assumed to be saturated.

3.3.1 **Static Safety Factor Assessment Under Maximum Storage Pool Loading - §257.73(e)(1)(i)**
The OGS ZLD Pond receives only storm water inflow. Its normal water elevation is control by the balance between storm water inflow and evaporation. A normal water elevation of 673 feet was selected as representative of measurements taken on the impoundment water elevation. Analysis of both circular and block sliding surfaces, 

\(^5\) Newmark, N. M. and W. J. Hall, “Earthquake Spectra and Design”, EERI Monograph, Earthquake Engineering Research Institute, Berkeley, California, 1982

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Appendix E, show a minimum factor of safety of 3.0 for the circular failure surface passing through the foundation soil.

3.3.2 Static Safety Factor Assessment Under Maximum Surcharge Pool Loading - §257.73(e)(1)(ii)
The OGS ZLD Pond will contain the 100 year return period design storm through storage in the impoundment without discharge. The maximum surcharge pool elevation is 677.25 feet at the conclusion of the storm. Analysis for both circular and block sliding surface, Appendix E, show a minimum factor of safety of 2.9 for the block slide surface passing through the foundation clay.

3.3.3 Seismic Safety Factor Assessment - §257.73(e)(1)(iii)
The OGS ZLD Pond was assigned a pseudo-static earthquake coefficient equal to 0.058 g acceleration and a vertical downward component equal to \( \frac{2}{3} \) of the horizontal component (0.039 g) as recommended by Newmark\(^6\). Analysis for both a circular and block sliding surface, Appendix E, show a minimum factor of safety of 2.5 for the circular sliding surface through the foundation soil.

3.3.4 Liquefaction Safety Factor Assessment - §257.73(e)(1)(iv)
The OGS ZLD Pond foundation and embankment soils are not susceptible to liquefaction, Section 3.1.4.

\(^6\) Newmark, N. M. and W. J. Hall, “Earthquake Spectra and Design”, EERI Monograph, Earthquake Engineering Research Institute, Berkeley, California, 1982
4 Results Summary

The results of the safety factor assessment indicate that the OGS embankments meet the requirements of §257.73(e). The results are:

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5 QUALIFIED PROFESSIONAL ENGINEER CERTIFICATION

To meet the requirements of 40 CFR 257.73(e)(2), I Mark W. Loerop hereby certify that I am a licensed professional engineer in the State of Iowa; and that, to the best of my knowledge, all information contained in this document is correct and the document was prepared in compliance with all applicable requirements in 40 CFR 257.73(b) and 40 CFR 257.73(e).

By: 

Name: Mark Loerop 

Date: 9/30/2016
FIGURES

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Ottumwa Generating Station
Ottumwa, Iowa

Safety Factor Assessment
APPENDIX A – 2016 Soil Borings

Alliant Energy
Wisconsin Power and Light Company
Ottumwa Generating Station
Ottumwa, Iowa

Safety Factor Assessment
null
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<tr>
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<td>Length (ft) &amp; Recovered (in)</td>
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<tr>
<td>S3</td>
<td>12</td>
</tr>
<tr>
<td>S4</td>
<td>22</td>
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<td>S13</td>
<td>18</td>
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</table>

**Soil/Rock Description**

- **FAT CLAY, yellowish brown (10YR 3/4). (continued)**
- **FAT CLAY, DARK OLIVE BROWN (2.5Y 3/3).**

**Soil Properties**

- Standard Penetration
- Moisture Content
- Liquid Limit
- Plasticity Index
- P 200
- RQD/Comments

**U.S.C.S.**

**Graphic Log**

**Well Diagram**

**FIP/FID**

- M
- M
- M
- M
- M
- M
- M
- M
- M
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<td>S14 24</td>
<td>FAT CLAY, DARK OLIVE BROWN (2.5Y 3/3), (continued) SANDY SILT, very dark gray, POORLY GRADED SAND, medium grained, gray (5Y 6/1), (weathered bedrock).</td>
<td>CH</td>
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<td>W</td>
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<td>S17 5</td>
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<td>W</td>
</tr>
<tr>
<td>S18</td>
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End of Boring at 52 feet bgs.
### Soil Boring Log Information

- **Facility/Project Name:** IPL-Ottumwa Generating Station
- **SCS #:** 25215135.40
- **License/Permit/Monitoring Number:** MW-305
- **Boring Number:** MW-305
- **Date Drilling Started:** 12/7/2015
- **Date Drilling Completed:** 12/8/2015
- **Drilling Method:** 4-1/4 hollow stem auger
- **Borehole Diameter:** 8.5 in

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<th>Local Grid Location</th>
<th>Lat</th>
<th>Long</th>
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<td>401,473 N, 1,903,023 E</td>
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<td>S1</td>
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<tr>
<td>S2</td>
<td>same as above except, brown (10YR 4/3).</td>
</tr>
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</table>

I hereby certify that the information on this form is true and correct to the best of my knowledge.

**Signature:** [Signature]
**Firm:** SCS Engineers
**Tel:** (608) 224-2830
**2830 Dairy Drive Madison, WI 53718**
**Fax:**
<table>
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<td>Recovered (ft)</td>
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<td>Depth In Feet</td>
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<tr>
<td>Soil/ Rock Description And Geologic Origin For Each Major Unit</td>
<td></td>
</tr>
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<td>USC S</td>
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<td>Graphic Log</td>
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<td>Well Diagram</td>
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<td>PTD/PID</td>
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<tr>
<td>Standard Penetration</td>
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</tr>
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<td>Moisture Content</td>
<td></td>
</tr>
<tr>
<td>Liquid Limit</td>
<td></td>
</tr>
<tr>
<td>Plasticity Index</td>
<td></td>
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<td>P-200</td>
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</tr>
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<td>RQD/Comments</td>
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**MW-305**

<table>
<thead>
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<th>22</th>
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<th>5</th>
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<td>20</td>
<td>5</td>
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<td>15</td>
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<td>S5</td>
<td>24</td>
<td>5</td>
<td>7</td>
<td>11</td>
<td>M</td>
</tr>
<tr>
<td>S6</td>
<td>20</td>
<td>7</td>
<td>11</td>
<td>20</td>
<td>M</td>
</tr>
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<td>S7</td>
<td>24</td>
<td>8</td>
<td>12</td>
<td>21</td>
<td>M</td>
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<td>24</td>
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<td>11</td>
<td>12</td>
<td>M</td>
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<td>S9</td>
<td>13</td>
<td>4</td>
<td>4</td>
<td>12</td>
<td>M</td>
</tr>
<tr>
<td>S10</td>
<td>24</td>
<td>5</td>
<td>6</td>
<td>9</td>
<td>W</td>
</tr>
<tr>
<td>S11</td>
<td>24</td>
<td>4</td>
<td>4</td>
<td>5</td>
<td>W</td>
</tr>
<tr>
<td>S12</td>
<td>22</td>
<td>2</td>
<td>2</td>
<td>3</td>
<td>W</td>
</tr>
<tr>
<td>S13</td>
<td>6</td>
<td>3</td>
<td>9</td>
<td>11</td>
<td>W</td>
</tr>
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</table>

**POORLY GRADED SANDY GRAVEL, fine, brown (10 YR 4/5).**

**water @ 41.0 ft bgs.**
<table>
<thead>
<tr>
<th>Boring Number</th>
<th>MW-305</th>
<th>Soil/Rock Description And Geologic Origin For Each Major Unit</th>
<th>Soil Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td>S14</td>
<td>22</td>
<td>23 50</td>
<td>POORLY GRADED SAND, medium grained, yellowish brown (10YR 5/4), (weathered bedrock). (continued)</td>
</tr>
<tr>
<td>S15</td>
<td>6</td>
<td>5 10 50</td>
<td></td>
</tr>
<tr>
<td>S16</td>
<td>6</td>
<td>50</td>
<td></td>
</tr>
</tbody>
</table>

End of Boring at 50 ft bgs.
APPENDIX B – 1974 Soil Laboratory Results

Alliant Energy
Wisconsin Power and Light Company
Ottumwa Generating Station
Ottumwa, Iowa

Safety Factor Assessment
<table>
<thead>
<tr>
<th>TABLE</th>
<th>ACTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.1</td>
<td>1.2</td>
</tr>
</tbody>
</table>

**APPENDIX I - RECIPE FORMATION**

1. Prepare the base ingredients.

2. Mix the wet and dry ingredients.

3. Add the flavorings.

**APPENDIX II - SUGAR LEVELS**

- Low: 10-15% sugar by weight
- Medium: 15-20% sugar by weight
- High: 20-25% sugar by weight

**APPENDIX III - BAKERY EQUIPMENT**

- Mixer
- Oven
- Sheeter
- Dough Roller

**APPENDIX IV - SUGAR FORMATION**

1. Combine sugar and water.
2. Heat mixture until sugar dissolves.
3. Cool mixture to room temperature.
4. Store mixture in an airtight container.

**APPENDIX V - BAKERY SAFETY**

- Wear protective clothing.
- Use proper tools.
- Keep work area clean.

**APPENDIX VI - BAKERY SERVICES**

- Delivery
- Catering
- Bulk purchases

**APPENDIX VII - BAKERY LICENSING**

- City licensing
- State licensing
- Federal licensing

**APPENDIX VIII - BAKERY BANKING**

- Opening a business bank account.
- Managing cash flow.
- Understanding financial statements.

**APPENDIX IX - BAKERY MARKETING**

- Social media marketing.
- Email marketing.
- Print advertising.

**APPENDIX X - BAKERY TIPS**

- Use fresh ingredients.
- Pay attention to texture.
- Offer diverse flavors.

**APPENDIX XI - BAKERY RESOURCES**

- Industry associations.
- Trade shows.
- Educational resources.

**APPENDIX XII - BAKERY LEGISLATION**

- Food safety regulations.
- Health codes.
- Environmental laws.

**APPENDIX XIII - BAKERY INNOVATION**

- Innovative baking techniques.
- Sustainable baking practices.
- New product development.

**APPENDIX XIV - BAKERY TECHNOLOGY**

- Automation.
- Robotics.
- Artificial intelligence.

**APPENDIX XV - BAKERY RISK MANAGEMENT**

- Insurance.
- Liability.
- Risk assessment.

**APPENDIX XVI - BAKERY INSURANCE**

- General liability.
- Product liability.
- Business interruption.

**APPENDIX XVII - BAKERY BUSINESS PLANNING**

- Market research.
- Feasibility study.
- Startup costs.

**APPENDIX XVIII - BAKERY TRAINING**

- Employee training.
- Customer service.
- Safety training.

**APPENDIX XIX - BAKERY INTELLECTUAL PROPERTY**

- Trademarks.
- Copyrights.
- Patents.

**APPENDIX XX - BAKERY QUALITY CONTROL**

- Sampling.
- Bench testing.
- Consumer feedback.

**APPENDIX XXI - BAKERY DISTRIBUTION**

- Wholesale.
- Retail.
- Online sales.

**APPENDIX XXII - BAKERY CO-OP**

- Cooperative business model.
- Member benefits.
- Shared resources.

**APPENDIX XXIII - BAKERY CONSULTANTS**

- Business consultants.
- Financial advisors.
- Legal services.

**APPENDIX XXIV - BAKERY CONSULTANTS**

- Business consultants.
- Financial advisors.
- Legal services.

**APPENDIX XXV - BAKERY CONSULTANTS**

- Business consultants.
- Financial advisors.
- Legal services.
Section A

Section B
<table>
<thead>
<tr>
<th>Section</th>
<th>Subsection</th>
<th>Table</th>
<th>Column 1</th>
<th>Column 2</th>
<th>Column 3</th>
<th>Column 4</th>
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</table>

Note: The results of all tests are included in the remainder of Appendix C and
the information provided is based on the assistance provided by the
authors. Information, numerical and other data were obtained in the course of
the investigation prepared on certain of the tasks carried out.
<table>
<thead>
<tr>
<th>Date</th>
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<th>Temperature</th>
<th>Humidity</th>
<th>Precipitation</th>
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<td>1/1/2023</td>
<td>City A</td>
<td>32°F</td>
<td>65%</td>
<td>0.00 in</td>
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<tr>
<td>1/2/2023</td>
<td>City B</td>
<td>45°F</td>
<td>40%</td>
<td>0.25 in</td>
</tr>
<tr>
<td>1/3/2023</td>
<td>City C</td>
<td>52°F</td>
<td>70%</td>
<td>0.10 in</td>
</tr>
<tr>
<td>1/4/2023</td>
<td>City D</td>
<td>38°F</td>
<td>55%</td>
<td>0.05 in</td>
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<tr>
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<td>City E</td>
<td>48°F</td>
<td>60%</td>
<td>0.15 in</td>
</tr>
<tr>
<td>1/6/2023</td>
<td>City F</td>
<td>40°F</td>
<td>45%</td>
<td>0.00 in</td>
</tr>
<tr>
<td>1/7/2023</td>
<td>City G</td>
<td>55°F</td>
<td>75%</td>
<td>0.20 in</td>
</tr>
<tr>
<td>1/8/2023</td>
<td>City H</td>
<td>42°F</td>
<td>65%</td>
<td>0.12 in</td>
</tr>
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</table>

*Note: All data recorded in Fahrenheit and inches.*
<table>
<thead>
<tr>
<th>Boring No.</th>
<th>Depth, ft.</th>
<th>Total Incl.</th>
<th>Determined of Compaction, Void Ratio</th>
<th>Estimated Coefficient of Permeability, ( 10^{-4} ) ft/sec</th>
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<tbody>
<tr>
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<td>6.4</td>
<td>25.0 0.25</td>
<td>0.840</td>
<td>2.2 \times 10^{-4}</td>
</tr>
<tr>
<td>1A</td>
<td>6.5</td>
<td>30.0 0.25</td>
<td>0.840</td>
<td>2.2 \times 10^{-4}</td>
</tr>
<tr>
<td>1A</td>
<td>6.6</td>
<td>35.0 0.25</td>
<td>0.840</td>
<td>2.2 \times 10^{-4}</td>
</tr>
<tr>
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<td>6.7</td>
<td>40.0 0.25</td>
<td>0.840</td>
<td>2.2 \times 10^{-4}</td>
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<td>0.840</td>
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<td>7.0</td>
<td>55.0 0.25</td>
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<td>0.840</td>
<td>2.2 \times 10^{-4}</td>
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<td>65.0 0.25</td>
<td>0.840</td>
<td>2.2 \times 10^{-4}</td>
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<td>7.3</td>
<td>70.0 0.25</td>
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</tr>
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<td>1A</td>
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<td>80.0 0.25</td>
<td>0.840</td>
<td>2.2 \times 10^{-4}</td>
</tr>
</tbody>
</table>

Table 3-2

- **Note:** This table contains data from the Ottawa Sandstone - Unit 1, as referenced in the text "E-7560."
APPENDIX C – Conversion of Blowcount to Soil Strength

Alliant Energy
Wisconsin Power and Light Company
Ottumwa Generating Station
Ottumwa, Iowa

Safety Factor Assessment
dicated shear tests are satisfactorily approximated by an equation (Table 1-3 and Equation 1-6) and a correction can be obtained with a few laboratory tests. As for other geologic conditions, triaxial shear tests are not well suited for a practical application. In the mass earthwork, when the cohesion is substantial and the friction angle is not high, cohesionless soils are employed in determining relative density and in the design of borrow materials.

For a free draining cohesionless material, the relative density determined in the mass earthwork, when the cohesion is very small, is not satisfactory. As a guide, the relative density is generally accepted as being two-thirds of the relative density of a cohesionless soil. This value is classified in the compact range of the heave analysis graph.

\[
\phi' = \text{True Angle of Internal Friction}^* \\
\theta = \text{Average}
\]

FIGURE 3-7
Correlations of Strength Characteristics
APPENDIX D – USGS Earthquake Design PGA

Alliant Energy
Wisconsin Power and Light Company
Ottumwa Generating Station
Ottumwa, Iowa

Safety Factor Assessment
Ottumuwa Generating Station
Latitude = 41.000°N, Longitude = 92.543°W

Reference Document
2015 NEHRP Provisions

Site Class
D (determined): Stiff Soil

Risk Category
I or II or III

\[ S_s = 0.078 \text{ g} \quad S_{ms} = 0.124 \text{ g} \quad S_{ds} = 0.083 \text{ g} \]
\[ S_1 = 0.064 \text{ g} \quad S_{ms} = 0.154 \text{ g} \quad S_{d1} = 0.103 \text{ g} \]

MCE Spectrum

Design Response Spectrum

Since \( S_{ms} < S_{n13} \) for this response spectrum \( S_{ms} \) has been set equal to \( S_{n11} \) (and hence \( S_{ds} \) has...
Mapped Acceleration Parameters, Long-Period Transition Periods, and Risk Coefficients

Note: The $S_s$ and $S_1$ ground motion maps provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain $S_s$) 1.3 (to obtain $S_1$).

- FIGURE 22-1 $S_s$ Risk-Targeted Maximum Considered Earthquake (MCE) Ground Motion Parameter for the Conterminous United States for 0.2 s Spectral Response Acceleration (5% of Critical Damping), Site Class B

- FIGURE 22-2 $S_1$ Risk-Targeted Maximum Considered Earthquake (MCE) Ground Motion Parameter for the Conterminous United States for 1.0 s Spectral Response Acceleration (5% of Critical Damping), Site Class B

- FIGURE 22-9 Maximum Considered Earthquake Geometric Mean (MCE) PGA, $\%g$, Site Class B for the Conterminous United States

- FIGURE 22-14 Mapped Long-Period Transition Period, $T_l$ (s), for the Conterminous United States

- FIGURE 22-18 Mapped Risk Coefficient at 0.2 s Spectral Response Period, $C_{ps}$

- FIGURE 22-19 Mapped Risk Coefficient at 1.0 s Spectral Response Period, $C_{p1}$
Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site class as Site Class, based on the site soil properties in accordance with Chapter 20.

Table 20.3-1 Site Classification

<table>
<thead>
<tr>
<th>Site Class</th>
<th>$\overline{v_s}$</th>
<th>$\overline{N}$ or $\overline{N_{ch}}$</th>
<th>$\overline{s_u}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A. Hard Rock</td>
<td>&gt;5,000 ft/s</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>B. Rock</td>
<td>2,500 to 5,000 ft/s</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>C. Very dense soil and soft rock</td>
<td>1,200 to 2,500 ft/s</td>
<td>&gt;50</td>
<td>&gt;2,000 psf</td>
</tr>
<tr>
<td>D. Stiff Soil</td>
<td>600 to 1,200 ft/s</td>
<td>15 to 50</td>
<td>1,000 to 2,000 psf</td>
</tr>
<tr>
<td>E. Soft clay soil</td>
<td>&lt;600 ft/s</td>
<td>&lt;15</td>
<td>&lt;1,000 psf</td>
</tr>
</tbody>
</table>

Any profile with more than 10 ft of soil having the characteristics:
- Plasticity index Pl > 20
- Moisture content w ≥ 40%, and
- Undrained shear strength $\overline{s_u} < 500$ psf

F. Soils requiring site response analysis in accordance with Section 21.1

See Section 20.3.1

For Sl: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²
Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

Table 11.8-1: Site Coefficient for $F_{PGA}$

<table>
<thead>
<tr>
<th>Site Class</th>
<th>Mapped MCE Geometric Mean ($MCE_6$) Peak Ground Acceleration</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>PGA $\leq 0.10$</td>
</tr>
<tr>
<td>A</td>
<td>0.8</td>
</tr>
<tr>
<td>B (measured)</td>
<td>0.9</td>
</tr>
<tr>
<td>B (unmeasured)</td>
<td>1.0</td>
</tr>
<tr>
<td>C</td>
<td>1.3</td>
</tr>
<tr>
<td>D (determined)</td>
<td>1.6</td>
</tr>
<tr>
<td>D (default)</td>
<td>1.6</td>
</tr>
<tr>
<td>E</td>
<td>2.4</td>
</tr>
<tr>
<td>F</td>
<td>See Section 11.4.7</td>
</tr>
</tbody>
</table>

Note: Use straight-line interpolation for intermediate values of PGA.

Note: Where Site Class D is selected as the default site class per Section 11.4.2, the value of $F_{PGA}$ shall not be less than 1.2.

For Site Class = D (determined) and PGA = 0.037 g, $F_{PGA} = 1.600$

Mapped $MCE_6$

$PGA = 0.037$ g

Site-adjusted $MCE_6$

$PGA_{adj} = F_{PGA}PGA = 1.600 \times 0.037 = 0.058$ g
APPENDIX E – Slope Stability Analysis

Alliant Energy
Wisconsin Power and Light Company
Ottumwa Generating Station
Ottumwa, Iowa

Safety Factor Assessment