



## Initial Structural Stability and Safety Factor Assessment

For Compliance with the EPA Coal  
Combustion Residuals (CCR) Rule  
40 CFR §257.73(d)  
40 CFR §257.73(e)

Erickson Power Station –  
Forebay and Retention Basin

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# 1 Introduction and Purpose

HDR MICHIGAN, Inc. (HDR) has prepared this Structural Stability and Safety Factor Assessment Report for the Forebay and Retention Basin at Erickson Power Station following the requirements of the Federal Coal Combustion Residuals (CCR) Rule to demonstrate compliance of the existing Erickson Power Station in Lansing, Michigan.

On April 17, 2015, the United States Environmental Protection Agency (EPA) issued the final rule (Ref. [2]) for disposal of Coal Combustion Residuals (CCR) under Subtitle D of the Resource Conservation and Recovery Act (RCRA). CCR Rule 40 CFR §257.73(b) requires that owners or operators of an existing CCR surface impoundment that either 1) has a height of five feet or more and a storage volume of 20 acre-feet or more; or 2) has a height of 20 feet or more perform periodic structural stability assessments (40 CFR §257.73(d)) and periodic safety factor assessments (40 CFR §257.73(e)). It was determined that the Forebay and Retention Basin at the Erickson Power Station meet the first criteria with a height of five feet or more and a storage volume greater than 20 acre-feet.

The CCR Final Rule requires that initial and periodic structural stability assessments be conducted in accordance with Section §257.73(d). Section §257.73(e) requires that initial and periodic safety factor assessments be conducted to verify that the stability of the most critical section of the embankment complies with the required minimum factors of safety for the long-term maximum storage pool, maximum surcharge pool, and seismic load cases. This report presents the initial periodic structural stability assessment and initial periodic safety factor assessment for the Forebay and Retention Basin.

The Structural Stability and Safety Factor Assessment Report presented herein addresses the specific requirements of 40 CFR §257.73(d) and 40 CFR §257.73(e). This Structural Stability and Safety Factor Assessment Report was prepared by Mr. Bryce Burkett, P.E., and was reviewed in accordance with HDR's internal review policy by Mr. Adam N. Jones, P.E., both of HDR. Mr. Burkett is a registered Professional Engineer in the State of Michigan.

## 1.1 Site Location

Erickson Power Station is an electrical power generation facility located at 3725 South Canal Road, Lansing, Michigan which is owned and operated by Lansing Board of Water & Light (BWL). The latitude and longitude of the Erickson Power Station are approximately 42.692422 N and 84.657764 W. The site is located southwest of Lansing Michigan, near the intersection of Interstates 69 and 96, as shown in the vicinity map, Figure 1.

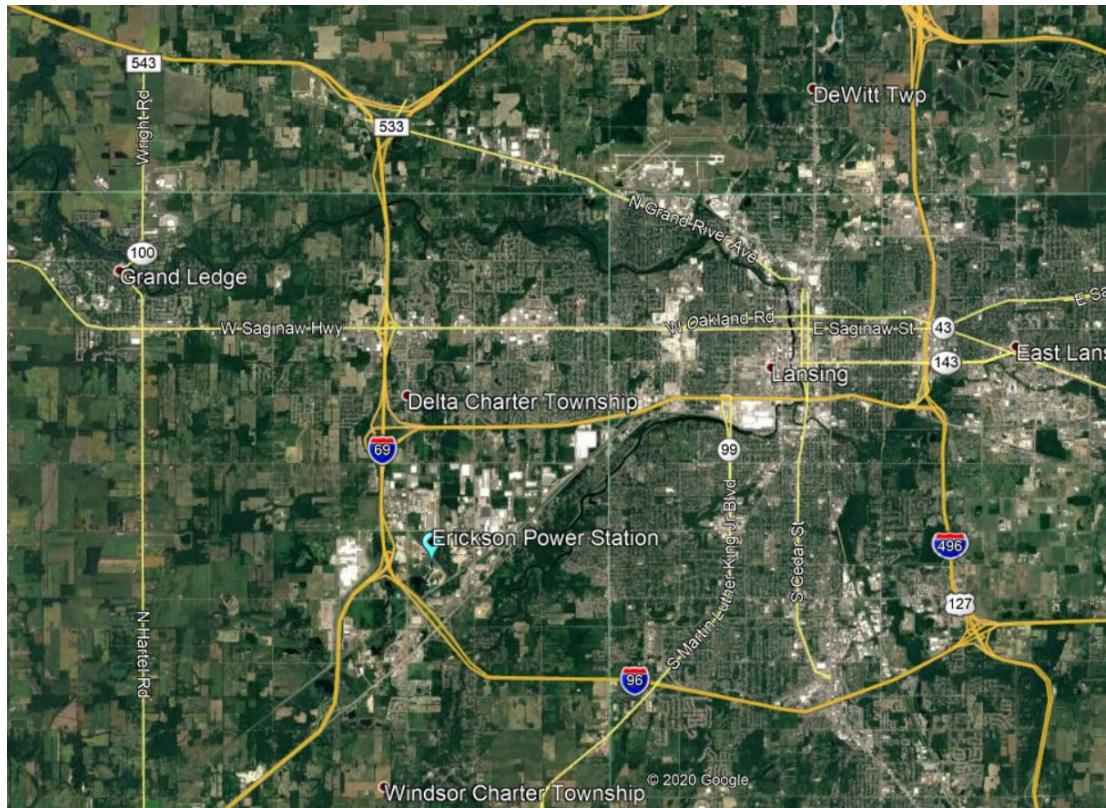


Figure 1. Site Vicinity Map

## 1.2 Site Description

Erickson Power Station was constructed starting in 1970, was completed in 1973, and is scheduled to close in 2025 as part of the BWL's move to cleaner energy sources. Erickson Power Station contains a single coal-fired steam turbine/generator capable of producing 165 megawatts of electricity.

Historically, fly ash and bottom ash resulting from the coal combustion process were mixed with water to form a slurry and pumped from the plant to the 33-acre impoundment system (physically closed in 2014). From the impoundment, the water then flowed hydraulically to the Clear Water Pond. Water from the Clear Water Pond was recycled back to the plant via the Pump House for reuse.

From 2009 through 2014, the ash was removed from the 33-acre impoundment, and a new system (including the construction of the Forebay and Retention Basin) (Ref. [10]) was installed. The Forebay and Retention Basin were installed within the footprint of the excavated 33-acre Former Impoundment and cover approximately 5-acres, leaving the Former Impoundment with a surface area of 28-acres.

Currently, bottom ash from the coal-fired boiler is sluiced from the plant to dewatering tanks (hydro-bins). The dewatered bottom ash is trucked to a sanitary landfill and the decant water is hydraulically fed through the current impoundment system, which consists of a series of three impoundments: the Forebay, Retention Basin, and Clear Water Pond.

The Forebay has an approximate normal pool surface areas of 2.1 acres. The Forebay has a normal operating pool level of approximately El. 882.3 feet NAVD 88<sup>1</sup> (882.9 feet NGVD 29<sup>2</sup>).

The Retention Basin has an approximate normal pool surface areas of 2.6 acres. The Retention Basin has a normal operating pool level of approximately El. 881.8 feet NAVD 88 (882.4 feet NGVD 29).

The Retention Basin is equipped with the Retention Basin Overflow Structure located at the south corner of the Retention Basin. The overflow of the Retention Basin flows through the 72-inch overflow riser and is directed to the Clear Water Pond through a 941 feet long, 36-inch diameter storm sewer which enters the Old Ash Impoundment Transfer Structure located at the southwest corner of the Former Impoundment. From there, the effluent water enters the Clear Water Pond.

Figure 2 displays the Erickson Power Station site configuration.



**Figure 2. Erickson Power Station Site Configuration**

Figure 3 presents a Google Earth view looking NNE, identifying the Forebay and Retention Basin in relation to the impoundment system. Also viewable in Figure 3 is the Clear Water Pond, Lake Delta, Former Impoundment, coal pile, and Erickson Power Station.

<sup>1</sup> North American Vertical Datum of 1988

<sup>2</sup> National Geodetic Vertical Datum of 1929



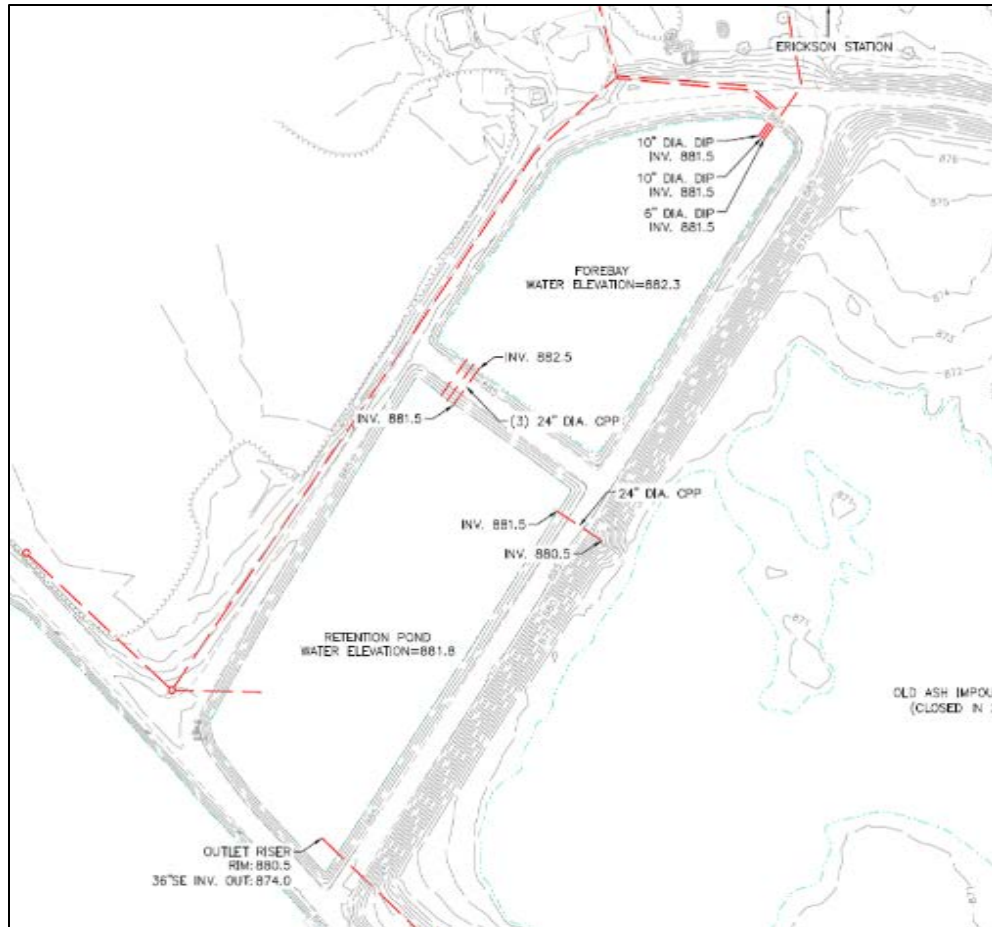
**Figure 3. Google Earth Image of Impoundment System**

The Forebay and Retention Basin have nine hydraulic structures that extend through the embankments:

- Forebay Influent Pipes
  - Three ductile iron pipes (DIP) transferring water from the 1) plant sump, 2) Hydro-Bins, and 3) Coal-Pile Runoff Pond to the Forebay.
- Forebay Overflow
  - Three corrugated plastic pipes (CPP) transferring water from the Forebay to the Retention Basin.
- Former Impoundment Overflow
  - One CPP designed to transfer water from the Former Impoundment to the Retention Basin.
- Retention Basin Overflow Structure
  - The Retention Basin Overflow Structure allows flow from the Retention Basin to the Clear Water Pond.
- By-Pass Pipe
  - One CPP pipe by-pass inlet from transferring water from the plant sump directly to the Retention Basin (bypassing the Forebay).

Figure 4 (Ref. [12]) displays a plan view of the Forebay and Retention Basin with the locations of the associated hydraulic structures and pipes extending through the embankments. Note that the elevations presented in Figure 4 (Ref. [12]) are referenced to NGVD 29 and NAVD 88.





**Figure 4. Location of Forebay and Retention Basin Hydraulic Structures**

The following provides details of each hydraulic structure located at the Forebay and Retention Basin.

A survey was performed by BWL on May 7, 2020, which provided elevations at many of the structures for the impoundment system. At the site, the conversion from NGVD 29 to NAVD 88 is -0.63 feet (i.e. El. 0.00 feet NGVD 29 = El. -0.63 feet NAVD 88). Elevations reference both datums throughout this report for clarity.

**Forebay Influent Pipes**

Water enters the Forebay through the Northeast Embankment through three DIP with inverts set at El. 881.6 feet NAVD 88 (882.2 feet NGVD 29):

- One 12-inch diameter conveyance from plant sump reduced to a 10-inch diameter discharge pipe to the Forebay.
- One 10-inch diameter conveyance from the Hydro-Bins to the Forebay.
- One 6-inch diameter conveyance from the Coal-Pile Runoff Pond to the Forebay.

**Forebay Overflow**

Water travels from northeast to southwest across the Forebay were the water can exit the Forebay through the Central Embankment that separates the Forebay and Retention Basin

via three 24-inch diameter CPP pipes. The invert of the effluent pipes (Forebay side) are set at approximately El. 882.4 to 882.6 feet NAVD (883.1 to 883.3 feet NGVD 29). The invert of the influent pipes (Retention Basin side) are set at approximately El. 881.4 to 881.7 feet NAVD 88 (882.0 to 882.3 feet NGVD 29).

#### **Former Impoundment Overflow**

The Former Impoundment Overflow passes through the Southeast Embankment of Retention Basin and consists of one 24-inch diameter CPP which serves the purpose of allowing water to enter the Retention Basin from the Former Impoundment in the event of flooding in the Former Impoundment. The invert of the effluent pipe (Former Impoundment side) is set at approximately El. 881.5 feet NAVD 88 (882.2 feet NGVD 29) and the invert of the influent pipe (Retention Basin side) is set at approximately El. 880.8 feet NAVD 88 (881.4 feet NGVD 29).

Considering the normal pool level in the Retention Basin (approximately El. 881.8 NAVD 88) and considering that the Former Impoundment is closed and only contains rainfall/runoff, water intermittently flows from the Retention Basin into the Former Impoundment during significant precipitation events.

#### **Retention Basin Overflow Structure**

The Retention Basin Overflow Structure is located at the southeast corner of Retention Basin. Overflow from the Retention Basin flows through the 72-inch overflow riser and is directed to the Clear Water Pond through a 941 feet long, 36-inch diameter CPP which enters into the Old Ash Impoundment Transfer Structure located at the southwest corner of the Former Impoundment. From there, the effluent water then enters the Clear Water Pond. The pipe consists of 36-inch CPP, equipped with square, (8-feet x 8-feet) concrete, anti-seep collars. The overflow structure was installed with a trash rack consisting of high-impact plastic during original construction, however the trash rack was disconnected from the structure during a storm event and is currently resting in the shallow water of the Retention Basin.

The invert of the overflow pipe is set at approximately El. 879.9 feet NAVD 88 (880.5 feet NGVD 29) and the invert of the outlet pipe is at approximately EL. 873.4 feet NAVD 88 (874.0 feet NGVD 29).

#### **By-Pass Pipe**

A 12-inch diameter CPP was installed as a by-pass pipe to transfer water from the Erickson Power Station plant sump under emergency conditions. The by-pass is operated by two valves located at the southwest corner of the Retention Basin. The by-pass pipe travels under the Northwest Embankment of the Retention Basin.

## **1.3 Previous Assessments and Inspections**

A previous assessment was performed by GZA GeoEnvironmental, Inc. (GZA) for the Erickson Power Station Ash Pond in 2011 and a report, referred to as a Round 10 Dam Assessment, was issued detailing the findings from the assessment in 2012 (Ref. [3]). The GZA 2012 report was performed for the Ash Pond which was undergoing closure at the time of the assessment. The Ash Pond has since been closed and is referred to herein as

the Former Impoundment. A site visit was conducted by GZA 2012 on May 19, 2011 which was prior to the construction of the Forebay and Retention Basin.

BWL performs weekly inspections for the entire CCR impoundment system. The weekly inspections are completed by qualified individuals to check for potentially hazardous conditions or structural weakness and the results of the inspections are documented internally on Weekly Inspection Reports.

There have been no reports of structural instability at the Forebay and Retention Basin during previous inspections.

There are no records of previous inspections that have been performed for the Forebay and Retention Basin embankments.

## 2 Structural Stability Assessment - 40 CFR §257.73(d)

The requirements to be documented in the Structural Stability Assessment for existing CCR surface impoundments are detailed in 40 CFR §257.73: *Structural integrity criteria for existing CCR surface impoundments*. CCR Rule 40 CFR §257.73(d) states that the assessment must, at a minimum, document whether the CCR unit has been designed, constructed, operated, and maintained with the items specified in 40 CFR §257.73(d)(1)(i) through (vii). Table 2-1 summarizes the information from paragraphs 40 CFR §257.73(d)(1)(i) through (vii), as well as the location of the information presented in this document.

**Table 2-1. List of Structural Stability Assessment Items**

40 CFR Rule	Rule Information	Document Section
§257.73 (d)(1)(i)	Foundations and Abutments	Section 2.1
§257.73 (d)(1)(ii)	Slope Protection	Section 2.2
§257.73 (d)(1)(iii)	Embankment Compaction	Section 2.3
§257.73 (d)(1)(iv)	Embankment Vegetation	Section 2.4
§257.73 (d)(1)(v)	Spillway	Section 2.5
§257.73 (d)(1)(vi)	Hydraulic Structures	Section 2.6
§257.73 (d)(1)(vii)	Downstream Slope Drawdown	Section 2.7
§257.73 (d)(2)	Structural Stability Deficiencies	Section 2.8

### 2.1 §257.73 (d)(1)(i) - Foundations and Abutments

**§257.73 (d)(1)(i): *Stable foundations and abutments.***

Prior to the construction of the Erickson Power Station impoundment system, a subsurface investigation program was performed in 1969 by Dames & Moore. The soil boring logs performed for that study are presented in the Location Restrictions Report prepared by

Mayotte Design & Engineering (MD&E) (Ref. [11]). In addition to the 1969 soil borings, test pits were performed at the site by MD&E in 2018. In 2018, SME performed three soil borings to the west of the Forebay and Retention Basin for the new gas-fired combustion turbine power plant (Ref. [13]). In 2019 and 2020, HDR installed six monitoring wells across the site, with two monitoring wells (MW-3 and MW-4) being installed in the vicinity of the Forebay and Retention Basin (Ref. [7]).

Table 2-2 details the borings, test pits, and monitoring wells which were reviewed for the foundation material of the Forebay and Retention Basin.

**Table 2-2. List of Available Borings, Test Pits, Monitoring Wells**

ID	Type	Year	Engineering Firm	Reference
AP-3 and AP-5	Geotechnical Borings	1969	Dames & Moore	Ref. [11]
AP-2	Test Pit	1969	Dames & Moore	Ref. [11]
ECT-18-B01 thru ECT-18-B03	Geotechnical Borings	2018	SME	Ref. [13]
EW-F-1 thru EW-F-6	Test Pits	2018	MD&E	Ref. [11]
MW-3 and MW-4	Monitoring Wells	2019/2020	HDR	Ref. [7]

The approximate locations of the borings, test pits, and monitoring wells are shown on Figure 5. The borings logs, test pit records, and monitoring well logs are provided in Attachment 1.



**Figure 5. Approximate Boring/Monitoring Well Locations**

The foundation of the Forebay and Retention Basin embankments was cut to approximately El. 870.9 feet NAVD 88 (871.5 feet NGVD 29) prior to construction of the embankments. The boring logs, test pit records, and monitoring well logs indicate that the Forebay and Retention Basin foundation is comprised primarily of alternating layers of clays, sands and silts (i.e. Lean Clay, Sandy Clay, Clayey Sand, Sand, Silt, and Silty Sand) from the surface to depths of approximately El. 810.4 feet NAVD 88 (811 feet NGVD 29) below existing grade. Refusal was encountered from SPT at approximately El. 837 feet to 847 feet NAVD 88 in the ECT-series borings, however, the borings were not advanced after refusal of the SPT. The borings logs indicated very dense sands and silts with limestone fragments observed at those elevations. Sandstone was encountered in the deepest boring (AP-5) at El. 810.4 feet NAVD 88 (811 feet NGVD 29), which was the limit of the deepest boring in the vicinity of the impoundments. Gravel, shale fragments, and limestone fragments were observed in the alternating cohesive and granular layers.

Laboratory tests were available for samples taken from Borings ECT-18-B01 through ECT-18-B03, which were advanced outside of the footprint of the Forebay and Retention Basin, but in the vicinity of the site (i.e. approximately 300 feet northwest of the impoundments). Laboratory tests were available for the subsurface foundation material (i.e. below El. 870.9 feet NAVD 88 (871.5 feet NGVD 29)). Undrained shear strengths obtained from field estimates with a hand penetrometer or torvane in the cohesive soils ranged from 1,000 psf (stiff) to greater than 4,500 psf (very stiff). Moisture contents in the cohesive soils ranged between 7 and 13 percent. SPT blow counts indicated that the granular soils ranged from

very loose to very dense, with blow counts ranging from 3 to greater than 50 blows per foot.

There is no available boring information that was performed through the embankments of the Forebay or Retention Basin, however, compaction records were available and reviewed (Ref. [10]).

It should also be noted that the location of the footprints of the Forebay and Retention Basin are within the extents of the previously closed Former Impoundment. Therefore, the foundation was previously preloaded prior to the construction of the new impoundments.

The previous subsurface investigation documentation indicates that the foundation is competent and stable. The assessment of abutment stability required by the CCR Final Rule is not applicable, as the embankments impounding the Forebay and Retention Basin are continuous.

## 2.2 §257.73 (d)(1)(ii) - Slope Protection

***§257.73 (d)(1)(ii): Adequate slope protection to protect against surface erosion, wave action, and adverse effects of sudden drawdown.***

The interior slopes of the Forebay and Retention Basin are protected by approximately 6-12 inches of stone riprap (Ref. [10]). Underlying the stone riprap is a layer of geosynthetic clay (GCL) overlain with a 40 millimeter-thick flexible polyvinylchloride membrane (FML) which is protected with geofabric and a 6-12 inch layer of sand. The exterior slopes of the Forebay and Retention Basin are protected from erosion and deterioration by vegetative cover, except for the exterior slope adjacent to the Former Impoundment, which is protected by stone riprap.

The crest of the Forebay and Retention Basin consists of a gravelly/soil surface. According to BWL, the road on the crest of the embankment is graded and maintained periodically.

Weekly inspections performed by BWL monitor the existing slopes for erosion, depressions, cracks, animal burrows, ruts, holes, and seepage. There have been no observations of erosion and/or sloughing along the slopes of the Forebay and Retention Basin during the weekly inspections or the Initial Inspection performed by HDR (Ref. [6]).

The existing slope protection measures for the Forebay and Retention Basin are generally considered adequate to provide protection against surface erosion, wave action, and adverse effects of sudden drawdown. Both impoundments are small, with a maximum fetch of 420 feet and 520 feet for the Forebay and Retention Basin, respectively. This limited fetch prevents the development of sustained wind set-up or wave run-up. The June 2020 inspection performed by HDR (Ref. [6]) did not identify any other concerns relating to slope protection that required investigation or repair.

## 2.3 §257.73 (d)(1)(iii) - Embankment Compaction

***§257.73 (d)(1)(iii): Dikes mechanically compacted to a density sufficient to withstand the range of loading conditions in the CCR unit.***

Construction drawings and specifications, including compaction records, were provided in MD&E 2015 (Ref. [10]). The embankment construction consisted of placement and

compaction of fill material in successive lifts, with a maximum uncompacted thickness of 8-inches. The fill materials were placed in this manner until design elevations were achieved and the slopes of the embankments were graded during the vertical progression of the embankments. Each lift was compacted to within 95% of the maximum dry density (Standard Proctor - ASTM D698) of the source material. Field density testing was performed at many locations to verify the adequacy of embankment lift compaction and the results of the field density testing are presented in Appendix E of MD&E 2015 (Ref. [10]).

A review of the construction records (Ref. [10]) of the Forebay and Retention Basin indicated that the embankments were mechanically compacted to a density sufficient to withstand the range of loading conditions in the CCR unit

## 2.4 §257.73 (d)(1)(iv) - Embankment Vegetation

***§257.73 (d)(1)(iv): Vegetated slopes of dikes and surrounding areas not to exceed a height of six inches above the slope of the dike, except for slopes which have an alternate form or forms of slope protection.***

Vegetation was evident on the interior and exterior slopes of the Forebay and Retention Basin embankments, in addition to stone riprap. The vegetation was overgrown and exceeded a height of 6-inches at the time of the HDR June 2020 inspection (Ref. [6]). BWL stated that the vegetation is typically maintained and the overgrown vegetation will be cut to maintain a height of 6 inches or less.

## 2.5 §257.73 (d)(1)(v) – Spillway

***§257.73 (d)(1)(v): A single spillway or a combination of spillways configured as specified in paragraph (d)(1)(v)(A) of this section. The combined capacity of all spillways must be designed, constructed, operated, and maintained to adequately manage flow during and following the peak discharge from the event specified in paragraph (d)(1)(v)(B) of this section.***

***(A) All spillways must be either:***

***(1) Of non-erodible construction and designed to carry sustained flows;  
or***

***(2) Earth- or grass-lined and designed to carry short-term, infrequent flows at non-erosive velocities where sustained flows are not expected.***

***(B) The combined capacity of all spillways must adequately manage flow during and following the peak discharge from a:***

***(1) Probable maximum flood (PMF) for a high hazard potential CCR surface impoundment; or***

***(2) 1000-year flood for a significant hazard potential CCR surface impoundment; or***

***(3) 100-year flood for a low hazard potential CCR surface impoundment.***

The Forebay and Retention Basin are considered to be low hazard potential embankments (Ref. [4]). Therefore, the combined capacity of the spillways must adequately manage

flow during and following the peak discharge from the Inflow Design Flood (IDF), defined as the 100-year flood. Design capacities and calculations used in the hydraulic analysis of the Forebay and Retention Basin are included in the Construction Documentation Report prepared by MD&E (Ref. [10]). Additionally, HDR prepared an Inflow Design Flood Control System Plan which included hydraulic analyses of the Erickson Power Station impoundment system. These analyses determined that the impoundment system is capable of managing the 100-year, 24-hour storm event without overtopping (Ref. [5]).

The methodology, assumptions, results, and conclusions of the spillway adequacy evaluation are described in the Inflow Design Flood Control System Plan (Ref. [5]).

## 2.6 §257.73 (d)(1)(vi) - Hydraulic Structures

***§257.73 (d)(1)(v): 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.***

Details of each hydraulic structure are discussed in Section 1.2. All of the hydraulic structures are relatively new, having been constructed in 2014. Each hydraulic structure observed during the June 2020 inspection (Ref. [6]) appeared to maintain structural integrity. The hydraulic structures that were observed from the ground surface during the June 2020 inspection include the Forebay Influent Pipes, Forebay Overflow, and Former Impoundment Overflow. These hydraulic structures appeared to be free of significant deterioration, deformation, distortion, bedding deficiencies, sedimentation, and debris and HDR was not aware of deficiencies being observed in the past by BWL. The Retention Basin Overflow Structure was submerged, but viewable at the time of the inspection and appeared to be in good condition aside from the detached trash rack. The portions of the hydraulic structures that were underground or submerged were not inspected, however, there were no indications of settlement or distress of the embankment at the locations of the structures. The By-Pass Pipe was not observable during the inspection as it is buried and the outlet is submerged in the Retention Basin. It should be noted that the interior of the pipes and submerged pipes were not observed, and LBWL reported that no dewatered or remotely operated vehicle (ROV) internal inspections have been conducted.

## 2.7 §257.73 (d)(1)(vii) - Downstream Slope Drawdown

***§257.73 (d)(1)(v): For CCR units with downstream slopes which can be inundated by the pool of an adjacent water body, such as a river, stream or lake, downstream slopes that maintain structural stability during low pool of the adjacent water body or sudden drawdown of the adjacent water body.***

The only water body present on the downstream slopes of the Forebay and Retention Basin is Lake Delta, which is adjacent to and to the south of the Retention Basin. Lake Delta is a shore and dock fishing lake located at Delta Township Park, which is leased to and maintained by Delta Township. Water from the Grand River is fed to the lake by the Erickson's River Pump House, located on the Grand River, to maintain lake levels for recreation at a design elevation of 882.5 feet NAVD 88 (883.1 feet NGVD 29). The water in Lake Delta is not subject to drawdown, thus a rapid drawdown condition was not



considered a potential mechanism for structural instability of the exterior slope of the Retention Basin.

## 2.8 §257.73 (d)(2) - Structural Stability Deficiencies

**§257.73 (d)(1)(v): The periodic assessment described in paragraph (d)(1) of this section 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 the previous weekly inspections performed by BWL and the inspection performed in June 2020 by HDR (Ref. [6]), no structural stability deficiencies were identified for the embankments retaining the Forebay and Retention Basin.

# 3 Safety Factor Assessment - 40 CFR §257.73(e)

## 3.1 Stability Analysis Criteria

The CCR Final Rule does not stipulate the stability analysis methodology directly, although the minimum required factor of safety criteria were adopted from the U.S. Army Corp of Engineers (USACE) guidance manuals, and USACE Engineering Manual EM 1110-2-1902 (Ref. [14]) is referred to by the CCR Rule as a benchmark in the dam engineering community for slope stability analyses. The methodologies in EM 1110-2-1902 were used in this assessment of the static load cases.

Safety Factor Assessment documentation requirements for existing CCR surface impoundments are detailed in 40 CFR §257.73: *Structural integrity criteria for existing CCR surface impoundments*. CCR Rule 40 CFR §257.73(e) states that:

**§257.73 (e)(1): The owner or operator must conduct an initial and periodic safety factor assessments for each CCR unit and document whether the calculated factors of safety for each CCR unit achieve the minimum safety factors specified in paragraphs (e)(1)(i) through (iv) of this section for the critical cross section of the embankment. 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 safety factor assessments must be supported by appropriate engineering calculations.**

**(e)(1)(i) The calculated static factor of safety under the long-term, maximum storage pool loading condition must equal or exceed 1.50.**

**(e)(1)(ii) The calculated static factor of safety under the maximum surcharge pool loading condition must equal or exceed 1.40.**

**(e)(1)(iii) The calculated seismic factor of safety must equal or exceed 1.00.**

***(e)(1)(iv) For dikes constructed of soils that have susceptibility to liquefaction, the calculated liquefaction factor of safety must equal or exceed 1.20.***

## 3.2 Methodology

The slope stability analysis was conducted using the GeoStudio computer program Slope/W, which uses limit equilibrium methodologies to evaluate potential rotational and sliding block failure surfaces. For a given geometry and soil profile, the program evaluates potential failure surfaces and identifies the surface exhibiting the minimum factor of safety. The Spencer Method was used in the evaluation because it satisfies both force and moment equilibrium. The factors of safety against sliding for both shallow and deep failure surfaces were determined. The shallow failure surfaces typically have lower factors of safety but are not typically a dam safety concern since they are surficial in nature and failure of a shallow surface is not likely to result in the release of the impoundment. The “deep” failure surfaces were defined for this study as failure surfaces that penetrate the phreatic surface or penetrate at least  $\frac{1}{4}$  of the crest width (approximately 5 feet) and, therefore, represent the most critical failure surfaces for the embankment stability.

## 3.3 Critical Cross Section Geometry

The critical section of the embankments was determined using the as-built construction drawings of the Forebay and Retention Basin prepared by MD&E (Ref. [10]), the interpreted subsurface profile from the available borings in the vicinity of the Forebay and Retention Basin (discussed in Section 2.1), and the interpreted phreatic surface based on observations at the site and from the monitoring wells installed in the vicinity of the Forebay and Retention Basin.

One embankment section for both the Forebay and Retention Basin was considered as potentially being critical based on geometry, described below, and located as shown on Figure 6 and Figure 7.

- Section 1, located along the Southeast Embankment of the Forebay, is adjacent to the Former Impoundment. Section 1 was selected due to the geometry of the slopes, the height of the embankment, and the differential head acting on the section. Due to the geometry that is present for this portion of the Forebay embankment, it was deemed more critical than the other portions of the embankment alignment for the Forebay and Retention Basin. Only one section was analyzed for the Forebay and Retention Basin due to the similar layout and being adjacent to one another. The location of the section was chosen at the Forebay due to the slightly higher normal pool level. Although this section is anticipated to have the most critical factor of safety, discharge from a breach, were it to occur, would be contained within the Former Impoundment.



Figure 6. Location of Section 1

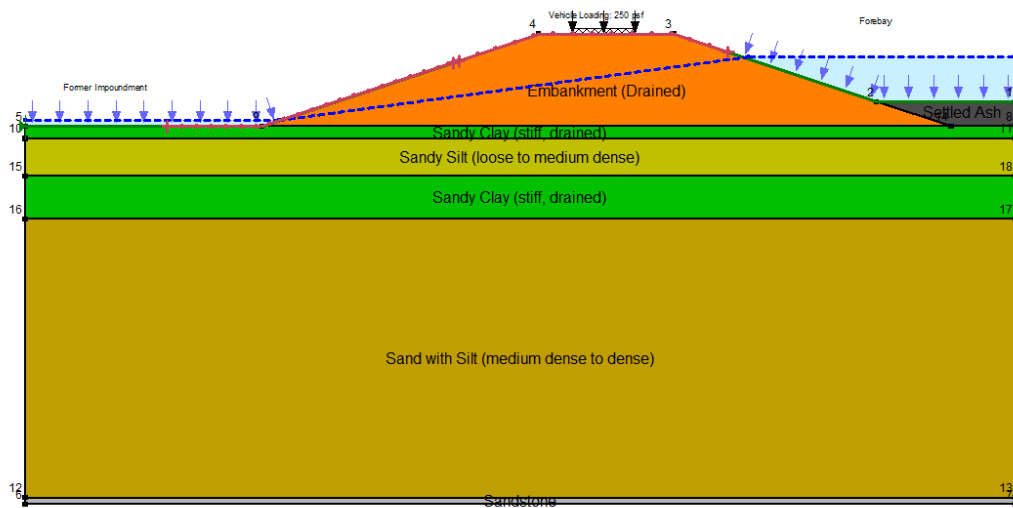


Figure 7. Section 1 Cross Section

### 3.4 Credible Load Cases

The loading conditions that were analyzed and the USEPA required minimum factors of safety are summarized in Table 3-1 below.

**Table 3-1. Loading Conditions and Minimum Required Factors of Safety**

Loading Condition	Headwater El. (feet NAVD 88)	Minimum Required Factor of Safety
Maximum Storage Pool (Normal)	882.3 <sup>1</sup>	1.5
Maximum Surge	885.9 <sup>2</sup>	1.4
Seismic <sup>3</sup>	882.3 <sup>1</sup>	1.0
Post-earthquake - Liquefaction	882.3 <sup>1</sup>	1.2

<sup>1</sup> Assumed to be normal operating pool level of the Forebay.

<sup>2</sup> Assumed to be approximately at Top of Dike elevation of the Forebay according to as-built construction drawings from MD&E (Ref. [10]).

<sup>3</sup> A Peak Ground Acceleration (PGA) = 0.076g was adopted, based on a 2 percent probability of exceedance in 50 years (2,475 recurrence interval) (USGS 2018).

### 3.5 Pond Elevation and Phreatic Conditions

The phreatic surface for the stability models was developed based on current water level conditions within the Forebay and Former Impoundment. Two upstream water boundary conditions were considered in the analyses; the maximum pool storage and the maximum pool surcharge conditions.

The maximum pool storage (i.e. normal operating condition) of the Forebay is El. 882.3 feet NAVD 88 (882.9 feet NGVD 29). The phreatic surface was assumed to follow a straight line from headwater on the upstream slope to tailwater on the downstream slope. Consideration was given to the monitoring wells installed in the vicinity of the Forebay (MW-3 and MW-4), however, the straight line assumption was found to be more conservative, and was adopted due to the limited instrumentation available.

The maximum pool surcharge scenario assumes a temporary rise of the pond water elevation due to rainfall and collection of site storm water runoff to the elevation of the top of dike, as shown on the as-built construction drawings (Ref. [10]), of El. 885.9 feet NAVD 88 (886.5 feet NGVD 29). The phreatic surface was assumed to follow a straight line from headwater to tailwater, as before, assuming that steady state seepage conditions develop. Since the surcharge loading condition is likely to exist for a short period of time, the assumption of steady state seepage is conservative.

The downstream water boundary condition was set at the current pond elevation of the Former Impoundment: El. 871.4 feet NAVD 88 (872.0 feet NGVD 29). The Former Impoundment is no longer in service therefore the water boundary condition should be relatively stable, and surcharge during flood conditions is not anticipated.

### 3.6 Material Properties

Prior to the construction of the Erickson Power Station impoundment system, a subsurface investigation program was performed in 1969 by Dames & Moore. The soil boring logs

performed for that study are presented in the Location Restrictions Report prepared by Mayotte Design & Engineering (MD&E) (Ref. [11]). In addition to the 1969 soil borings, test pits were performed at the site by MD&E in 2018. In 2018, SME performed three soil borings to the west of the Forebay and Retention Basin for the new gas-fired combustion turbine power plant for BWL (Ref. [13]). In 2019 and 2020, HDR installed six monitoring wells across the site, with two monitoring wells (MW-3 and MW-4) being installed in the vicinity of the Forebay and Retention Basin (Ref. [7]).

The embankment stratigraphy is shown in Figure 7 and the material properties used for the slope stability analysis are presented in Table 3-2. The estimated material engineering properties were based on the classifications on the encountered subsurface soils, correlations with Standard Penetration Testing (SPT), shear strength data obtained from the soil borings, and HDR’s experience with similar conditions. The borings logs, test pit records, and monitoring well logs are provided in Attachment 1. HDR used undrained and drained shear strengths related to effective stresses, as recommended by the USACE.

**Table 3-2. Summary of Material Properties Used in Analysis**

Material Types	Approximate Elevation (feet NAVD 88)	Unit Weight, $\gamma$ (pcf)	Short-term (Undrained)		Long-term (Drained)	
			Cohesion, $c$ (psf)	Friction Angle, $\phi$ (degrees)	Cohesion, $c'$ (psf)	Friction Angle, $\phi'$ (degrees)
Embankment	886 to 871	120	1,000	0	200	28
Sandy Clay	871 to 869	125	1,500	0	150	18
Sandy Silt	869 to 863	125	0	28	0	28
Sandy Clay	863 to 856	125	1,500	0	150	18
Sand with Silt	856 to 811	125	0	40	0	40
Sandstone	811 to 810	160	2,000	45	2,000	45
Settled Ash*	875 to 871	90	0	30	0	30

\* - The thickness of settled ash in the Forebay was assumed.

The embankment stratigraphy and elevations were interpreted from the borings logs, test pit records, and monitoring well logs, the as-built construction drawings of the Forebay and Retention Basin prepared by MD&E (Ref. [10]), and measurements taken during the HDR 2020 site inspection (Ref. [6]).

### 3.7 Vehicle Loading

The crest of the embankment is intermittently used as access roads around the Forebay, therefore, a vehicle load was used on the crest of the embankment in the stability analyses. The vehicle loading was applied to the loading conditions for the maximum pool storage and maximum pool surcharge cases. The vehicle load used in the analysis is based on American Association of State Highway and Transportation Officials (AASHTO)

recommended loading for *Equivalent Height of Soil for Vehicular Loading on Abutments* for maintenance trucks (Ref. [1]).

### 3.8 Assessment of Liquefaction Potential

The embankment is an engineered compacted fill that is classified as sandy lean clay (CL) and founded on foundation soils generally consisting of stiff sandy clay and loose to dense clayey sand. A “triggering analysis” was used to assess the potential for liquefaction of the foundation soils using correlations with the SPT data from and ECT-18-B01 through ECT-18-B03 (Ref. [13]). The lowest SPT blow counts (N) in the ECT-series borings were measured to be N=3 in silt and clayey sand materials. The stratigraphy observed in the closest historical boring, AP-3, was found to be consistent with the 2018 ECT-series borings, indicating high fine content, however the minimum N=6 was greater.

These borings were drilled in the vicinity of the Forebay as shown in Figure 8. The borings logs are provided in Attachment 1.



**Figure 8. Approximate Boring/Monitoring Well Locations**

The foundation soils were screened for seismically-induced liquefaction susceptibility using methods recommended by the National Center for Earthquake Research (NCEER), which uses SPT data (Ref. [8]). For liquefaction triggering analysis, the fine contents of SM and SC material is conservatively taken based on the lower bound of USCS fine contents (12%). Three one-dimensional sections were analyzed: 1) a section at the toe of the embankment (i.e. the natural ground), 2) a section that includes the embankment (i.e.

the embankment crest elevation) and 3) a section corresponding to the existing ground elevation using the 2018 ECT-series borings and measured ground water level. For Cases 1 and 2, it was conservatively assumed that the groundwater level is located at the ground surface (i.e. the toe and crest for Cases 1 and 2, respectively). Based on these assumptions, the corrected SPT blow counts and soil stresses were calculated for evaluation of cyclic shear strength and stress and minimum factor of safety for each boring from the three analyzed cases were obtained.

Using the USGS online Unified Hazard Tool (Ref. [16]), the Peak Ground Acceleration (PGA) and earthquake magnitude, assuming a Site Class B/C boundary were selected as 0.0466g and 5.5, respectively. Pages 1 through 3 of Attachment 2 present a summary of the Unified Hazard Tool data. The USGS Unified Hazard Tool has not been developed for 2020, however grid data is available in the form of tables and map. Based on the site location and the interpolated 2018 data that are available for 0.05 degree grids, the PGA was estimated at 0.0544g, slightly higher than 0.0466g and, as such, the higher value was used for analysis. According to most recent geotechnical report performed in the vicinity of the site (Ref. [13]), the site is classified as Seismic Site Class C. In accordance with ASCE-7 2016, a factor of 1.3/0.9 was applied to obtain the site PGA of 0.076g, which was used for the analysis.

As discussed above, the triggering analysis requires that the raw SPT “N” values be corrected to a confining pressure of 1 ton per square foot and a drive energy of 60% efficiency (referred to as a  $(N_1)_{60}$  value). Hammer efficiency of 78% was used based on the average reported measured efficiency for the 2018 ECT-series borings. The methods used to calculate  $(N_1)_{60}$  were those that have been proposed by Idriss and Boulanger (Ref. [8]). The raw SPT “N” values ( $N_{raw}$ ) presented on the boring logs were converted to  $(N_1)_{60}$  values using the following equation:

$$(N_1)_{60} = N_{RAW} C_N C_E C_B C_R C_S$$

Where:

$C_N$  = Overburden Correction Factor =  $(P_a/\sigma'_{vo})^{(0.784-0.0768[(N_1)_{60}^{0.5}]}$

$C_E$  = Hammer Energy Correction factor = 60% efficient safety hammer = 1.0

$C_B$  = Borehole Diameter Correction Factor = 1.0

$C_R$  = Rod Length Correction Factor

= 0.75 (0-9.75 ft.)

= 0.8 (9.75 to 13 ft.)

= 0.85 (13 to 19.5 ft.)

= 0.95 (19.5 to 32 ft.)

= 1 (>32 ft.)

$C_S$  = Spoon Liner Correction

= 1.0 No liner was used

Additional corrections were then made to correct the  $(N_1)_{60}$  value to an equivalent “clean sand” value for use in determining cyclic stress resistance (CRR), which was used for assessing triggering of liquefaction. The clean sand value,  $(N_1)_{60cs}$ , was determined based

on the lowest possible fine contents from soil classification noted on the boring logs and using the method proposed by Idriss and Boulanger (Ref. [8]) and the following equation:

$$\Delta(N_1)_{60cs} = e^{(1.63+9.7/(PF+0.01)-(15.7/(PF+0.01))^2)}$$

Where:

**PF** = Percent fines passing No. 200 sieve

Using Idriss and Boulanger (Ref. [8]), CRR was then calculated using the following equation:

$$\mathbf{CRR} = e^{[(N_1)_{60cs}/14.1 + ((N_1)_{60cs}/126)^2 - ((N_1)_{60cs}/23.6)^3 + ((N_1)_{60cs}/25.4)^4 - 2.8]}$$

The Cyclic Stress Ratio (CSR) was then calculated using the design earthquake. The CSR is defined as the ratio of the cyclic shear stress acting on a horizontal plane to the initial (pre-earthquake) effective or overburden stress. The PGA of 0.076g was assumed in the analysis and the distribution of CSR through the foundation cross-section was determined. The CSR was then calculated using the following equation:

$$\mathbf{CSR} = 0.65 \cdot (a_{max}/g) \cdot (\sigma_v/\sigma'_v) \cdot r_d$$

Where:

$$a_{max}/g = 0.076$$

$\sigma_v$  = Total Overburden Stress

$\sigma'_v$  = Effective Overburden Stress

$$r_d = e^{(a(z) + B(z)M)}$$

Where:

$$a(z) = -1.012 - 1.126 \cdot \sin((z/11.73) + 5.133)$$

$$b(z) = 0.106 + 0.118 \cdot \sin((z/11.28) + 5.142)$$

$$M = 5.5$$

**z** = depth in meters

Once the CSR and CRR values were calculated, the factor of safety against triggering liquefaction was calculated as:

$$\mathbf{FS} = \mathbf{CRR/CSR} \times \mathbf{MSF} \times \mathbf{K}_\sigma \times \mathbf{K}_\alpha$$

Where:

$$\mathbf{MSF} = \text{magnitude scaling factor} = 6.9 \cdot e^{(-M/4)} - 0.058, \leq 1.8$$

$\mathbf{K}_\alpha$  = correction factor for the effects of an initial static shear stress ratio = 1

$\mathbf{K}_\sigma$  = overburden correction factor = 1

Where:

$$\mathbf{K}_\sigma = 1 / \{18.9 - 2.55 \cdot \text{SQRT}((N_1)_{60cs})\} \leq 0.3$$

$\mathbf{P}_a$  = Pressure at 1 atmosphere

The static shear strength in the liquefaction-susceptible material is small. Therefore,  $K_\alpha$  was taken equal to one for the purpose of this analysis. If the FS is greater than 1.2, the soil is considered not susceptible to liquefaction. The calculated factor of safety against seismically-induced liquefaction is presented in on Page 4 of Attachment 2 and was



calculated to be greater than 1.20 throughout the foundation depth. Considering that the embankment is classified as CL (USCS standard) and compacted material, the screening-level results indicate that the embankment and foundation soils are not susceptible to seismically-induced liquefaction for the seismic loading considered. In summary, the foundation was determined to be stable with respect to liquefaction for earthquakes up to the considered 2475-year return interval, which would have a PGA of 0.076g.

The corrected blow counts were also used for evaluation of foundation shear strength for stability analysis. Page 5 of Attachment 2 shows the calculated value and the assumed friction angle in foundation soil for slope stability analysis.

Because neither the embankment nor foundation soil were considered to be liquefiable, a pseudo static seismic stability analysis was conducted assuming no strength loss for the embankment materials, and the embankment yield acceleration was evaluated. In order to include the amplification factor that accounts for the quasi-elastic response of the embankment, the peak transverse crest acceleration was evaluated to be 0.25g, using a peak transverse base acceleration of 0.076g from the figure presented on Page 6 of Attachment 2 (Ref. [15]). The average embankment acceleration for a deep failure surface was then obtained from the figure on Page 7 of Attachment 2 (Ref. [9]), using  $y/h=1$ , the maximum ratio of 0.47, and an effective seismic coefficient of  $0.25 \times 0.47 = 0.1175$  was used for the calculation of the factor of safety during an earthquake based on a conservative undrained shear strength of 1,000 psf. The results indicate that the factor of safety during an earthquake is 1.48, which is greater than 1 and suggests that the deformation of the embankment during and after an earthquake would be very small. The yield acceleration of the embankment was calculated as 0.23g. The ratio of the effective acceleration to the yield acceleration, as shown on the figure on Page 8 of Attachment 2 (Ref. [9]), indicates that the deformation during an earthquake is anticipated to be negligible.

### 3.9 Stability Analysis Results and Conclusions

Analysis summary diagrams for each loading case are provided in Attachment 3. Table 3-3 below also summarizes the results of the analyses conducted for each loading case.

As presented in Table 3-3, the factors of safety against slope instability for deep failure surfaces that are capable of breaching the embankment satisfy the requirements of the CCR Final Rule under all loading conditions.

**Table 3-3. Summary of Stability Analyses Results**

Loading Condition	Required Minimum Factor of Safety	Computed Factor of Safety	Figure Location
Maximum Storage Pool (Normal)	1.5	1.7	Attachment 3, Page 1
Maximum Surcharge	1.4	1.6	Attachment 3, Page 2
Seismic	1.0	2.1	Attachment 3, Page 3
Post-earthquake - Liquefaction	1.2	>1.2	Attachment 2, Page 4

## 4 Closure

Based on the information provided to HDR by BWL, information available on BWL's CCR website, and HDR's visual observations and analyses, this Initial Structural Stability Assessment and Safety Factor Assessment was conducted in accordance with the requirements of the USEPA 40 CFR Parts §257 and §261 Hazardous and Solid Waste Management System; Disposal of Coal Combustion Residuals from Electric Utilities; Final Rule, April 17, 2015 (CCR Final Rule). Based on the information currently available, I certify to the best of my knowledge, information and belief that this Initial Structural Stability Assessment and Safety Factor Assessment meets the requirements of CCR Rule §257.73(d,e) in accordance with professional standards of care for similar work. HDR appreciates the opportunity to assist BWL with this project. Please contact us if you have any questions or comments.



Bryce Burkett, P.E.  
Senior Geotechnical Project Manager



Adam Jones, P.E.  
Engineering Manager



13 Aug 2020

## 5 References

- Ref. [1]* American Association of State Highway and Transportation Officials (AASHTO), Load Resistant Factor Design (LFRD) Bridge Design Specifications, 2012.
- Ref. [2]* Environmental Protection Agency, 40 CFR Parts §257 and §261; Hazardous and Solid Waste Management System; Disposal of Coal Combustion Residuals from Electric Utilities; Final Rule, Washington D.C., April 2015.
- Ref. [3]* GZA GeoEnvironmental, Inc. Draft Round 10 Dam Assessment Report, Lansing Board of Water & Light, Erickson Station, Ash Pond. April 30, 2012.
- Ref. [4]* HDR Michigan, Inc. Hazard Potential Classification Assessment Report, Erickson Power Station – Forebay, Retention Basin, & Clear Water Pond, Lansing Board of Water & Light, Lansing, Michigan, June 19, 2020.
- Ref. [5]* HDR Michigan, Inc. Inflow Design Flood Control System Plan, Erickson Power Station – CCR Surface Impoundments, Lansing Board of Water & Light, Lansing, Michigan, June 9, 2020.
- Ref. [6]* HDR Michigan, Inc. Initial Inspection Report, Erickson Power Station – Forebay and Retention Basin, Lansing Board of Water & Light Erickson Station, Lansing, Michigan, August 10, 2020.
- Ref. [7]* HDR Engineering, Inc. Monitoring Wall Installation Report, Lansing Board of Water & Light Erickson Power Station, Lansing, Michigan, March 25, 2020.
- Ref. [8]* Idriss, I.M. and Boulanger, R.W., SPT-Based Liquefaction Triggering Procedures, Report No. UCD/CGM-10/02, Department of Civil and Environmental Engineering, University of California at Davis, December 2010
- Ref. [9]* Makdisi, F.I. and Seed, H.B., Simplified Procedure for Estimating Dam and Embankment Earthquake-Induced Deformations, Journal of Geotechnical Engineering, 1978.
- Ref. [10]* Mayotte Design & Engineering, P.C. Construction Documentation Report Ash Impoundment System Reconfiguration, Lansing Board of Water & Light Erickson Station, Lansing, Michigan, May 2015.
- Ref. [11]* Mayotte Design & Engineering, P.C. Compliance with 40CFR257-Locations Restrictions. Lansing Board of Water & Light Erickson Station. October 10, 2018.
- Ref. [12]* NTH Consultants, Ltd. Closure Plan, CCR Surface Impoundment System, Erickson Power Station. August 16, 2019.
- Ref. [13]* SME. Geotechnical Data Report, Lansing Board of Water and Light, New Gas Combined Cycle Plant, Delta Township, Michigan. SME Project No. 079295.00, August 16, 2018.
- Ref. [14]* USACE. EM 1110-2-1902, Slope Stability, October 31, 2003.
- Ref. [15]* US Army Corps of Engineers for the Nuclear Regulatory Commission, Technical Bases for Regulatory Guide for Soil Liquefaction, Figure 40, March 2000.
- Ref. [16]* United States Geologic Survey, Unified Hazard Tool, accessed July 2020, <https://earthquake.usgs.gov/hazards/interactive/>

## 6 Attachments

- Attachment 1 Boring Logs and Monitoring Well Logs
- Attachment 2 Liquefaction Analysis Figures and Results
- Attachment 3 Stability Analyses Results

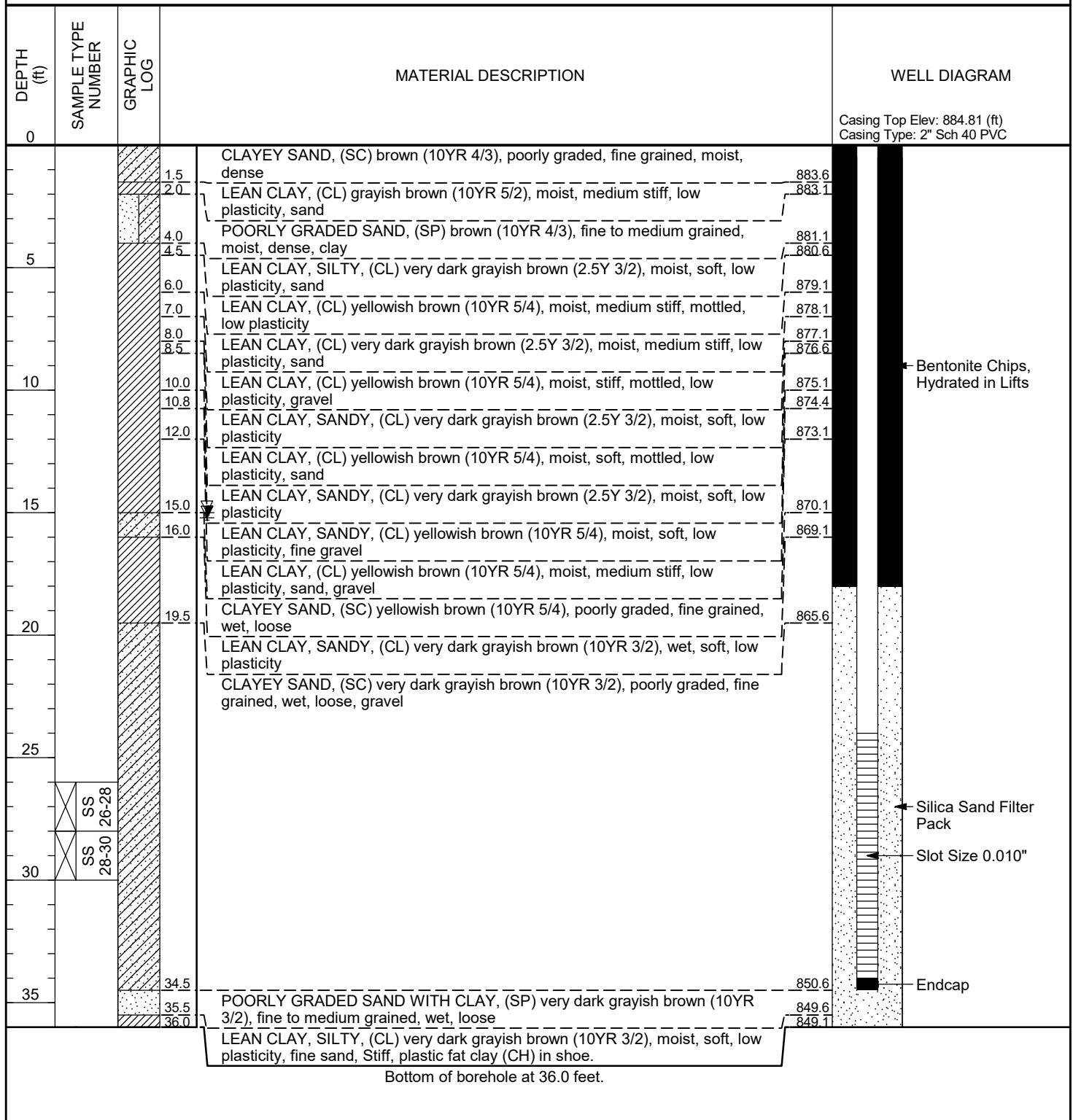
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# **ATTACHMENT 1**

## **BORING LOGS AND MONITORING WELL LOGS**



CLIENT Lansing Board of Water and Light PROJECT NAME LBWL Confidential  
 PROJECT NUMBER 10173187 PROJECT LOCATION Erickson Power Station, Lansing, MI  
 DATE STARTED 10/15/19 10:36 COMPLETED 10/15/19 12:30 GROUND ELEVATION 885.12 ft MSL HOLE DIAMETER 8"  
 DRILLING CONTRACTOR SME DRILLER Derek Blackburn GROUND WATER LEVELS:  
 DRILLING METHOD HSA EQUIPMENT Truck-Mounted CME 55 ∇ AT TIME OF DRILLING 15.00 ft / Elev 870.12 ft  
 LOGGED BY Emily Munoz CHECKED BY \_\_\_\_\_ ∇ 72 HRS AFTER DRILLING 15.21 ft / Elev 869.91 ft  
 NOTES Sample ID prefix LBWL-MW3-. Driller recorded blow counts on SME logs.





**CLIENT** Lansing Board of Water and Light      **PROJECT NAME** LBWL Confidential  
**PROJECT NUMBER** 10173187      **PROJECT LOCATION** Erickson Power Station, Lansing, MI  
**DATE STARTED** 01/06/20 10:09 **COMPLETED** 01/06/20 11:05      **GROUND ELEVATION** 885.23 ft MSL **HOLE DIAMETER** 8"  
**DRILLING CONTRACTOR** SME **DRILLER** Derek Blackburn      **GROUND WATER LEVELS:**  
**DRILLING METHOD** HSA      **EQUIPMENT** Truck-Mounted CME 55      ▽ **AT TIME OF DRILLING** 13.00 ft / Elev 872.23 ft  
**LOGGED BY** Emily Munoz      **CHECKED BY** \_\_\_\_\_      ▽ **94.3 HRS AFTER DRILLING** 11.51 ft / Elev 873.72 ft  
**NOTES** \_\_\_\_\_

DEPTH (ft)	SAMPLE TYPE NUMBER	GRAPHIC LOG	MATERIAL DESCRIPTION	WELL DIAGRAM
0				Casing Top Elev: 889.15 (ft) Casing Type: 2" Sch 40 PVC
1.0			LEAN CLAY, SILTY, (CL) very dark brown (7.5YR 2.5/2), moist, soft, low plasticity, fine sand	
			LEAN CLAY, SILTY, (CL) brown (10YR 4/3), moist, soft, low plasticity	
5.0			LEAN CLAY, SILTY, (CL) dark brown (7.5YR 3/2), moist, soft, low plasticity, fine sand	
6.0			LEAN CLAY, SILTY, (CL) brown with dark brown (10YR 5/3), moist, medium stiff, mottled, low plasticity, fine sand, fine gravel	
7.0			LEAN CLAY, SILTY, (CL) dark yellowish brown with dark grayish brown (10YR 4/6), moist, soft, mottled, low plasticity, fine sand, fine gravel	
9.0			LEAN CLAY, SILTY, (CL) yellowish brown (10YR 5/4), moist, soft, medium plasticity, fine sand, fine gravel	
13.0			▽	
14.0			LEAN CLAY, SILTY, (CL) yellowish brown (10YR 5/4), wet, soft, medium plasticity, fine sand, fine gravel	
14.5			WELL GRADED SAND WITH GRAVEL, (SW) brown (10YR 4/3), fine to coarse grained, wet, loose	
15.5			LEAN CLAY, SILTY, (CL) yellowish brown (10YR 5/4), wet, stiff, medium plasticity, fine sand, fine gravel	
16.5			CLAYEY SAND, (SP) yellowish brown (10YR 5/4), fine grained, wet, loose, fine gravel	
19.8			LEAN CLAY, (CL) brown (7.5YR 4/2), wet, medium stiff, low plasticity, fine sand, fine gravel	
20.0			CLAYEY SAND, (SP) brown (7.5YR 5/2), fine to coarse grained, wet, loose, fine gravel	
20.3			CLAYEY SAND, (SP) brown (7.5YR 5/2), fine grained, wet, loose	
21.0			LEAN CLAY, (CL) brown (7.5YR 5/2), wet, soft, low plasticity, fine sand	
22.0			POORLY GRADED SAND, (SP) dark gray (7.5YR 4/1), coarse grained, wet, loose, fine gravel	
23.0			LEAN CLAY, (CL) gray (7.5YR 5/1), moist, stiff, low plasticity, fine sand, fine gravel	
26.3			LEAN CLAY, (CL) brown (7.5YR 5/2), wet, stiff, low plasticity, fine sand	
28.0			LEAN CLAY, SANDY, (CL) dark gray to black (7.5YR 4/1), wet, medium stiff, low plasticity	

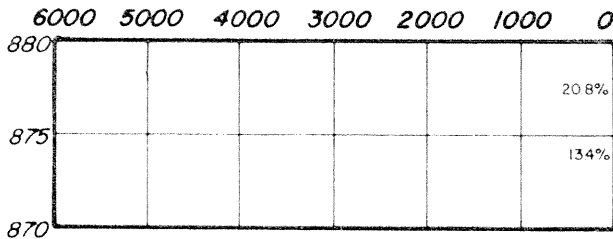
Hydrated bentonite chips

0.010" Slotted PVC Screen

Bottom of borehole at 28.0 feet.

ELEVATION IN FEET

SHEARING STRENGTH IN LBS./SQ.FT.



BULK SAMPLES  
BAG SAMPLES

TEST PIT AP-1

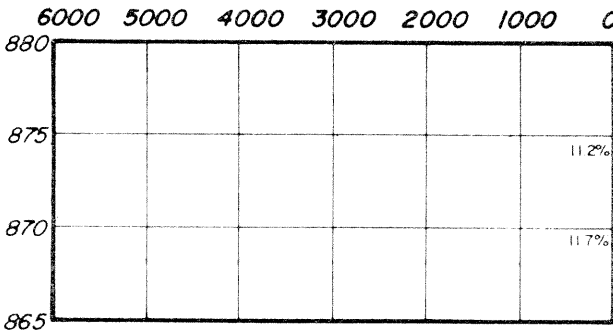
SURFACE ELEVATION 879.6

SYMBOLS		DESCRIPTIONS
	ML	DARK BROWN CLAYEY SILT WITH ROOTS - TOPSOIL (9')
	SC	MOTTLED BROWN AND GRAY CLAYEY FINE SAND WITH SOME SMALL GRAVEL. ROOTS TO 2'-6"
	ML	BROWN FINE SANDY SILT WITH SOME SMALL GRAVEL AND TRACE OF CLAY. 3' POCKET OF WATER BEARING FINE SAND ON WEST WALL OF PIT AT 60'

TEST PIT COMPLETED AT 80'  
ON 6/23/69  
MINOR SEEPAGE WATER FROM POCKET OF SAND AT 60'

ELEVATION IN FEET

SHEARING STRENGTH IN LBS./SQ.FT.



BULK SAMPLES  
BAG SAMPLES

TEST PIT AP-2

SURFACE ELEVATION 877.8

SYMBOLS		DESCRIPTIONS
	ML	DARK BROWN CLAYEY SILT WITH ROOTS - TOPSOIL (10')
	SC	MOTTLED BROWN AND GRAY CLAYEY SAND
	SM	MOTTLED BROWN AND GRAY FINE SILTY SAND WITH SOME CLAY AND SMALL GRAVEL
	ML	BROWN SILT. 2" SEAM OF BROWN FINE TO COARSE SAND WITH GRAVEL AT 5.5'
	ML	GRAY CLAYEY SILT WITH SOME FINE SAND AND SMALL GRAVEL

TEST PIT COMPLETED AT 90'  
ON 6/23/69  
MINOR SEEPAGE WATER AT 5.5'

LOG OF TEST PITS

BY \_\_\_\_\_ DATE \_\_\_\_\_  
BY \_\_\_\_\_ DATE \_\_\_\_\_  
CHECKED BY \_\_\_\_\_ DATE \_\_\_\_\_

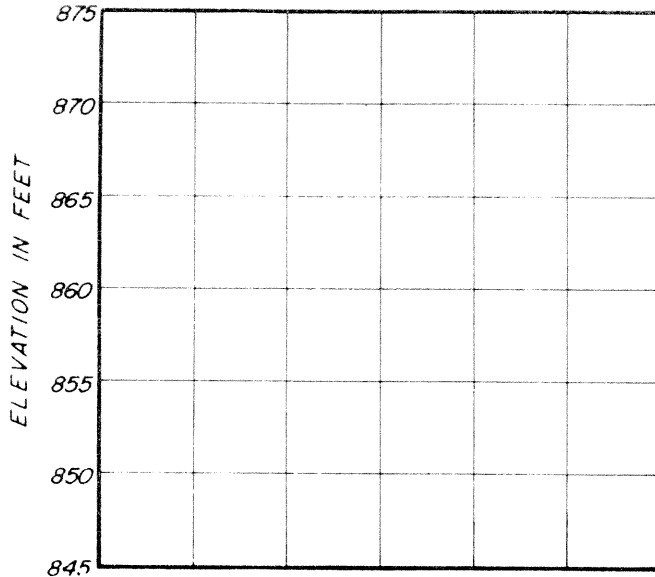
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BY \_\_\_\_\_ DATE \_\_\_\_\_  
 BY \_\_\_\_\_ DATE \_\_\_\_\_  
 CHECKED BY \_\_\_\_\_ DATE \_\_\_\_\_

SHEARING STRENGTH IN LBS./SQ.FT.  
 6000 5000 4000 3000 2000 1000 0



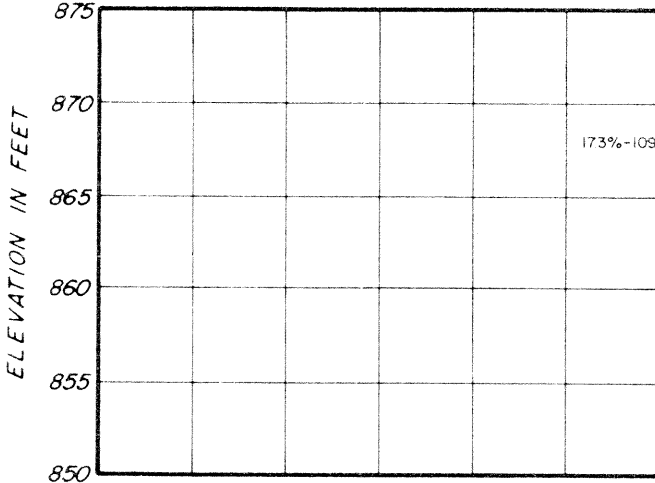
BLOW COUNTS  
 SAMPLES

BORING AP-3  
 SURFACE ELEVATION 872.8

SYMBOLS		DESCRIPTIONS
ML		BROWN SANDY SILT WITH ROOTS - TOPSOIL (6")
CL		BROWN SANDY CLAY WITH SOME ROOTS ROOTS GRADING OUT AT 2.5'
SC		BROWN CLAYEY SAND GRADING SOME SMALL GRAVEL
CL		BROWN SANDY CLAY WITH SOME SMALL GRAVEL
SP		BROWN FINE TO MEDIUM SAND SEEPAGE WATER ENCOUNTERED AT 7-8' WATER ROSE TO 5-10" IN 15 MINUTES
SP		GRAYISH - BROWN FINE SAND
ML		GRAY FINE SANDY SILT
SC		GRAY CLAYEY FINE SAND WITH SOME SMALL GRAVEL
SP		GRAY SILTY FINE SAND WITH SOME GRAVEL

BORING COMPLETED AT 25.0'  
 ON 7/8/59  
 CASING USED TO A DEPTH OF 14.0'  
 WATER LEVEL NOT RECORDED

SHEARING STRENGTH IN LBS./SQ.FT.  
 6000 5000 4000 3000 2000 1000 0



BLOW COUNTS  
 SAMPLES

BORING AP-4  
 SURFACE ELEVATION 870.7

SYMBOLS		DESCRIPTIONS
OL		BLACK ORGANIC SILT WITH ROOTS - TOPSOIL (12")
SC		GRAY CLAYEY SAND WITH ORGANIC MATTER SEEPAGE WATER ENCOUNTERED AT 2-6"
SC		MOTTLED BROWN AND GRAY CLAYEY SAND WITH POCKETS OF BROWN FINE SAND
ML		MOTTLED BROWN AND GRAY CLAYEY SILT WITH SOME SAND
ML		GRAY CLAYEY SILT WITH FINE SAND
ML		GRAY SILT
ML		GRAY SILT
SW		GRAY FINE TO COARSE SAND WITH SOME SMALL GRAVEL
ML		GRAY SILT

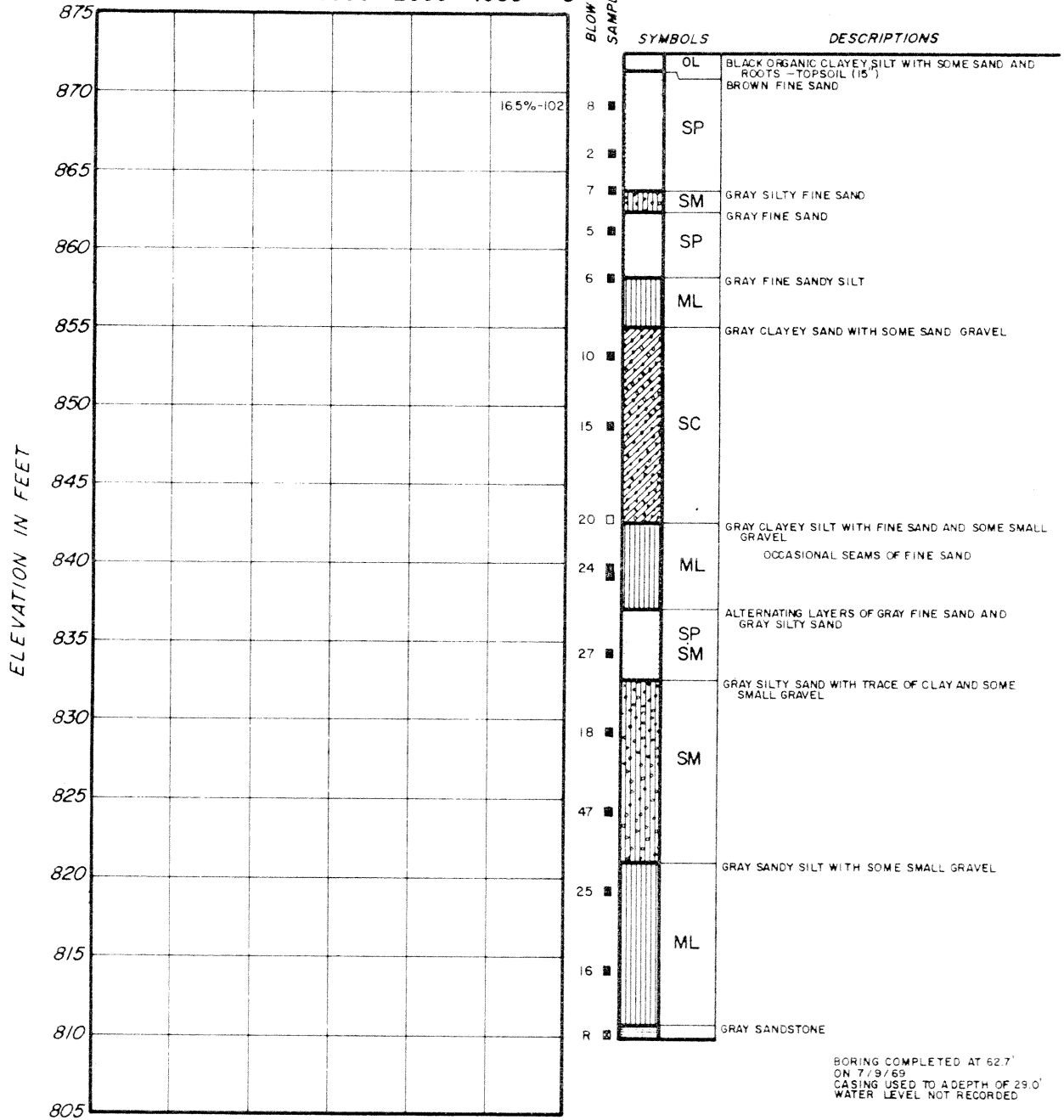
BORING COMPLETED AT 15.0'  
 ON 7/11/59  
 NO CASING USED  
 WATER LEVEL NOT RECORDED

# LOG OF BORINGS

BY \_\_\_\_\_ DATE \_\_\_\_\_  
 BY \_\_\_\_\_ DATE \_\_\_\_\_  
 CHECKED BY \_\_\_\_\_ DATE \_\_\_\_\_  
 PLATE \_\_\_\_\_ OF \_\_\_\_\_

SHEARING STRENGTH IN LBS./SQ.FT.  
 6000 5000 4000 3000 2000 1000 0

BORING AP-5  
 SURFACE ELEVATION 872.5



BORING COMPLETED AT 62.7'  
 ON 7/9/69  
 CASING USED TO A DEPTH OF 29.0'  
 WATER LEVEL NOT RECORDED

LOG OF BORINGS



**PROJECT NAME:** LBWL New Gas Combined Cycle Plant

**PROJECT NUMBER:** 079295.00

**CLIENT:** Lansing Board of Water & Light

**PROJECT LOCATION:** Delta Township, Michigan

**DATE STARTED:** 6/28/18

**COMPLETED:** 6/28/18

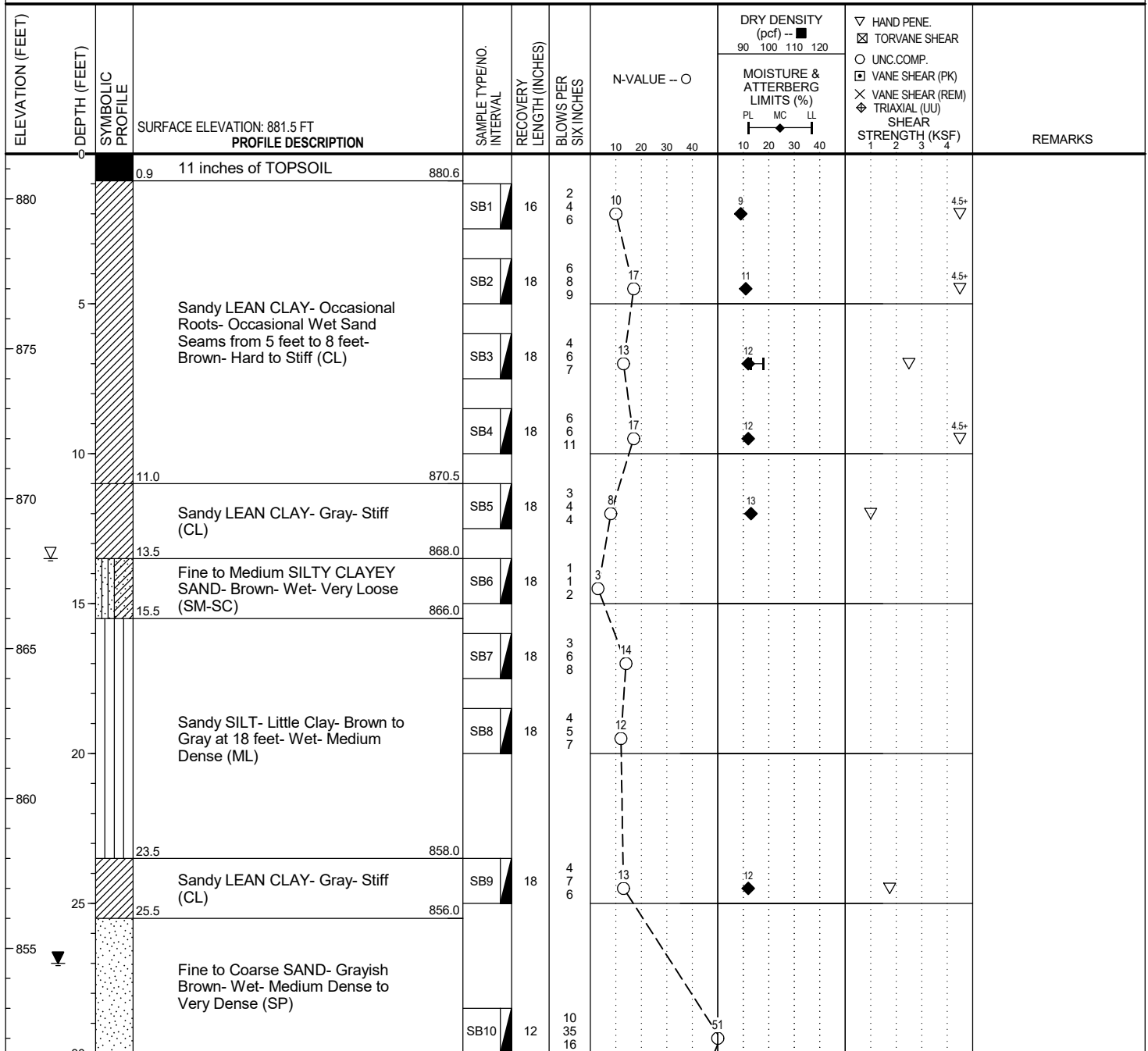
**BORING METHOD:** Hollow-stem Augers

**DRILLER:** BS (Strata)

**RIG NO.:** CME 55 - ATV

**LOGGED BY:** JAR

**CHECKED BY:** JSW



GROUNDWATER & BACKFILL INFORMATION		
	DEPTH (FT)	ELEV (FT)
▽ DURING BORING:	13.5	868.0
▽ AT END OF BORING:	27.0	854.5
<b>BACKFILL METHOD:</b> Cement- Bentonite Grout		

NOTES: 1. The indicated stratification lines are approximate. In situ, the transition between materials may be gradual.  
2. Bulk sample obtained from auger cuttings while drilling from 0' to 10'





**PROJECT NAME:** LBWL New Gas Combined Cycle Plant

**PROJECT NUMBER:** 079295.00

**CLIENT:** Lansing Board of Water & Light

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**COMPLETED:** 6/29/18

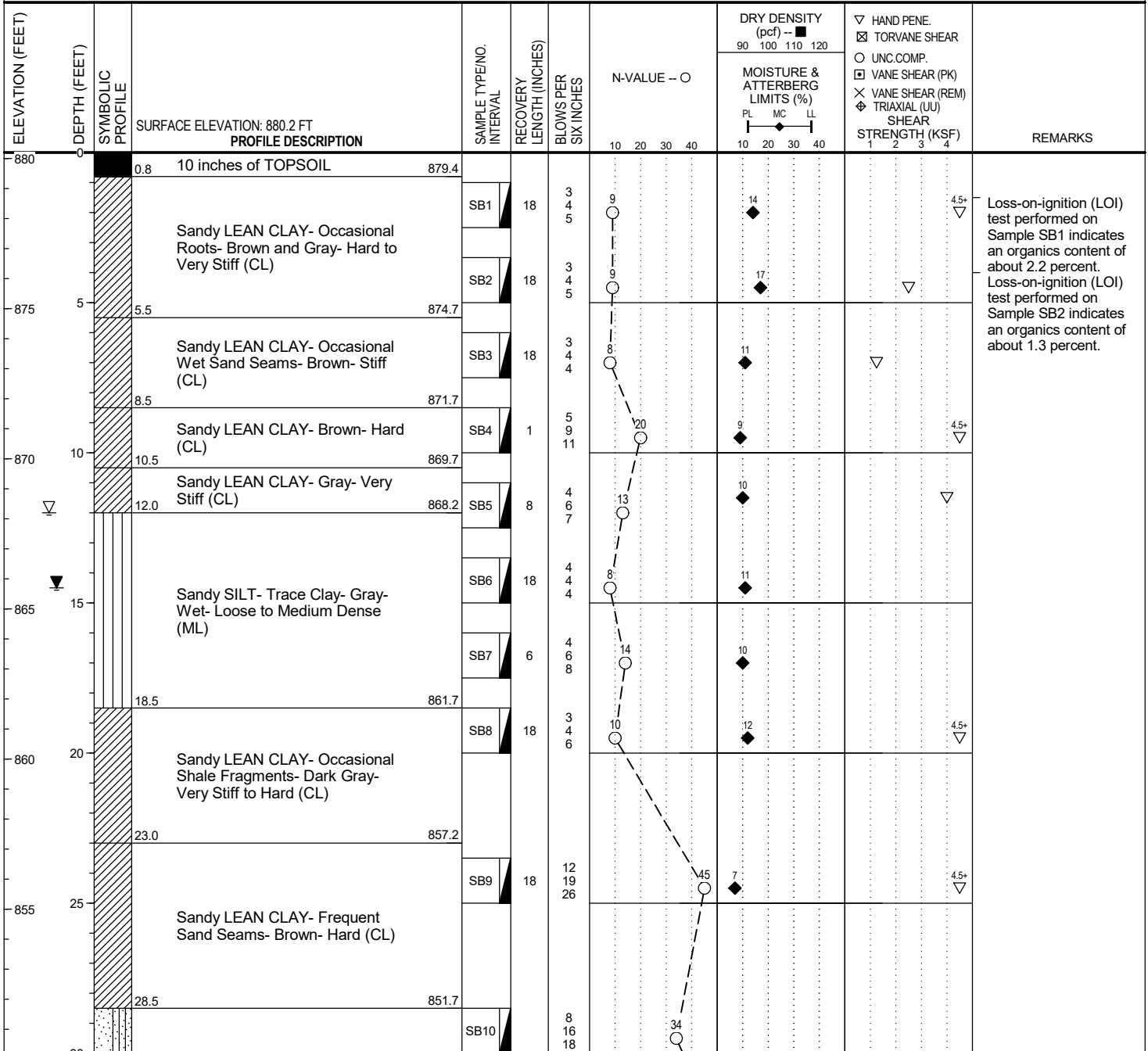
**BORING METHOD:** Hollow-stem Augers

**DRILLER:** BS (Strata)

**RIG NO.:** CME 55 - ATV

**LOGGED BY:** JAR

**CHECKED BY:** JSW



GROUNDWATER & BACKFILL INFORMATION		
	DEPTH (FT)	ELEV (FT)
▽ DURING BORING:	12.0	868.2
▽ AT END OF BORING:	14.5	865.7
<b>BACKFILL METHOD:</b> Cement- Bentonite Grout		

NOTES: 1. The indicated stratification lines are approximate. In situ, the transition between materials may be gradual.





**PROJECT NAME:** LBWL New Gas Combined Cycle Plant

**PROJECT NUMBER:** 079295.00

**CLIENT:** Lansing Board of Water & Light

**PROJECT LOCATION:** Delta Township, Michigan

**DATE STARTED:** 6/28/18

**COMPLETED:** 6/28/18

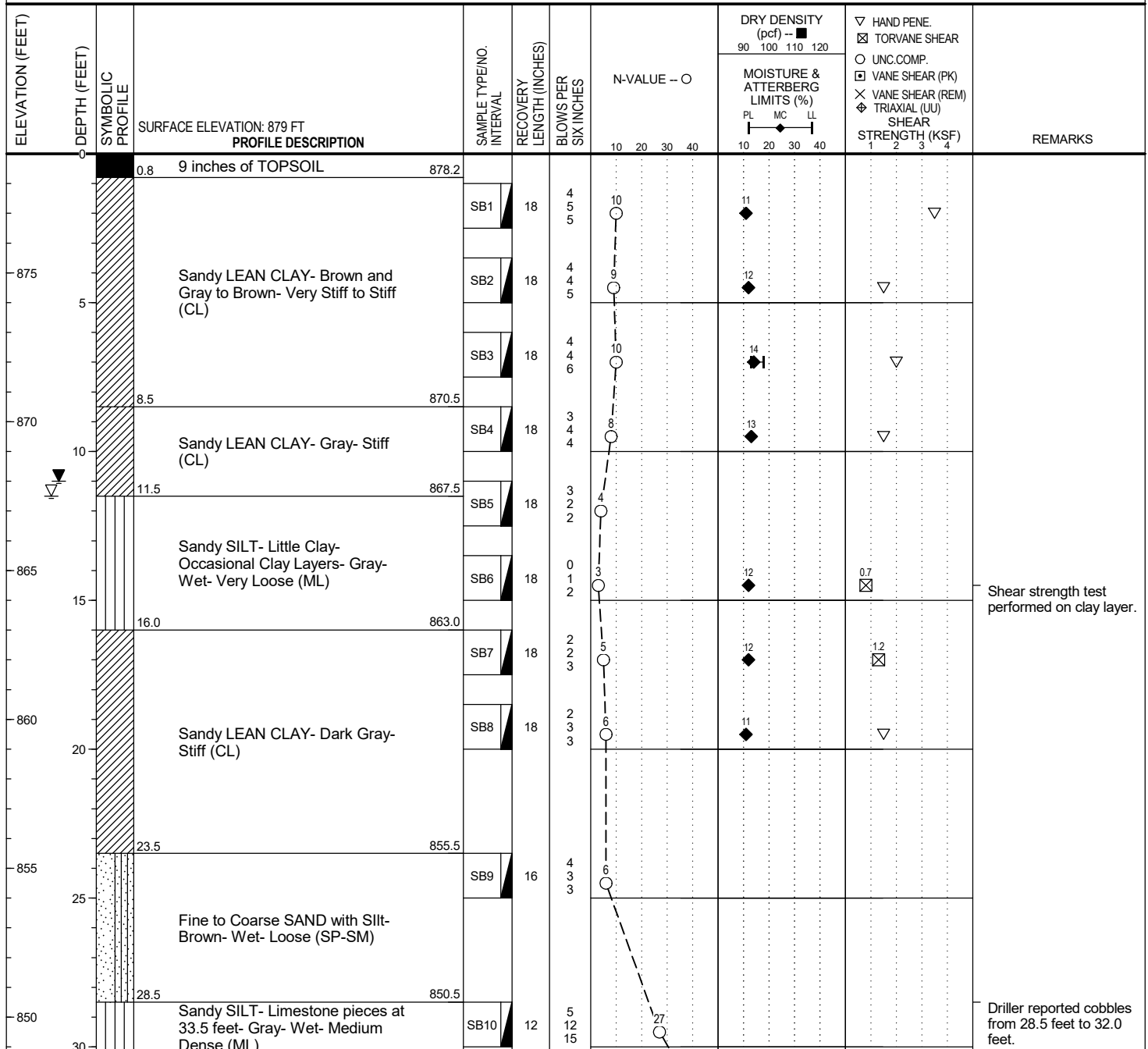
**BORING METHOD:** Hollow-stem Augers

**DRILLER:** BS (Strata)

**RIG NO.:** CME 55 - ATV

**LOGGED BY:** JAR

**CHECKED BY:** JSW



GROUNDWATER & BACKFILL INFORMATION		
	DEPTH (FT)	ELEV (FT)
▽ DURING BORING:	11.5	867.5
▽ AT END OF BORING:	11.0	868.0
<b>BACKFILL METHOD:</b> Cement- Bentonite Grout		

NOTES: 1. The indicated stratification lines are approximate. In situ, the transition between materials may be gradual.  
2. Bulk sample obtained from auger cuttings while drilling from 0' to 10'





# GEOTECHNICAL TESTING SUMMARY

LBWL - Erickson Station - Foundation Samples  
MD&E Project No.



SAMPLE	CLASSIFICATION	%Fines	LL%	PI%	w <sup>o</sup> / <sub>o</sub> field	w <sup>o</sup> / <sub>o</sub> opt	ρ <sub>d</sub> (lbs/ft <sup>3</sup> )	K (cm/s)
EW-F-01	Clayey Sand	29.50	NA	NA	NA	9.20	124.24	NA
EW-F-02	Clayey Sand	14.10	NA	NA	NA	8.25	129.23	NA
EW-F-03	Clayey Sand	9.70	NA	NA	NA	12.00	121.11	NA
EW-F-04	Clayey Sand	9.80	NA	NA	NA	8.50	125.92	NA
EW-F-05	Clayey Sand	16.30	NA	NA	NA	8.30	126.86	NA
EW-F-06	Clayey Sand	12.20	NA	NA	NA	7.85	131.10	NA
EW-T-01	Clayey Sand	NA	NA	NA	NA	10.00	133.60	NA
EW-T-02	Clayey Sand	NA	NA	NA	NA	9.80	127.67	NA
EW-T-03	Clayey Sand	NA	NA	NA	NA	9.30	127.98	NA
West Floor	Clayey Sand	13.10	NA	NA	NA	9.00	128.61	NA
South Floor	Clayey Sand	17.60	NA	NA	NA	7.95	129.98	NA
<b>Ranges/Averages:</b>								

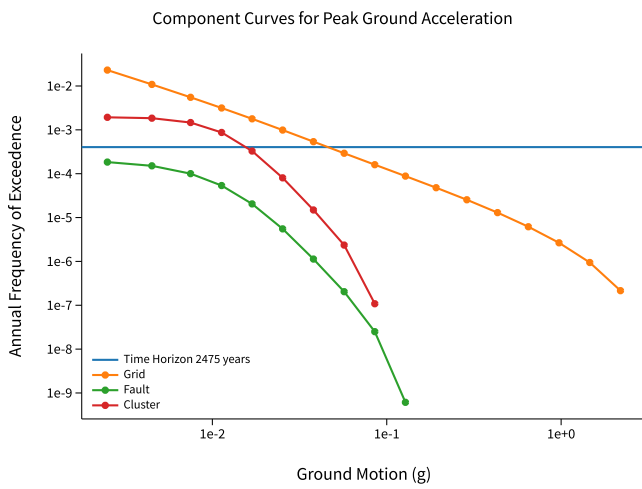
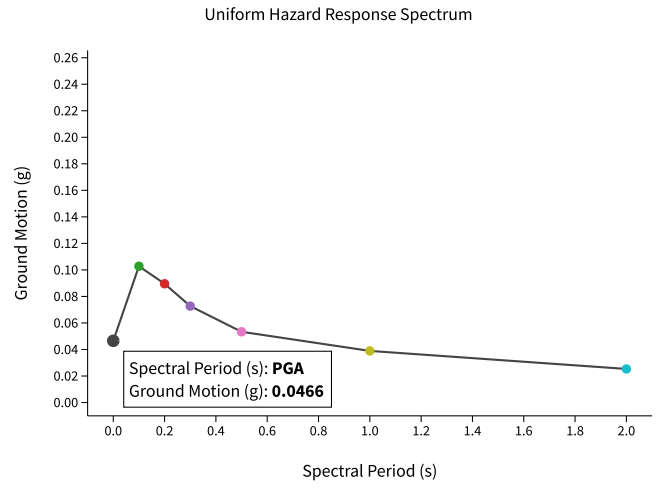
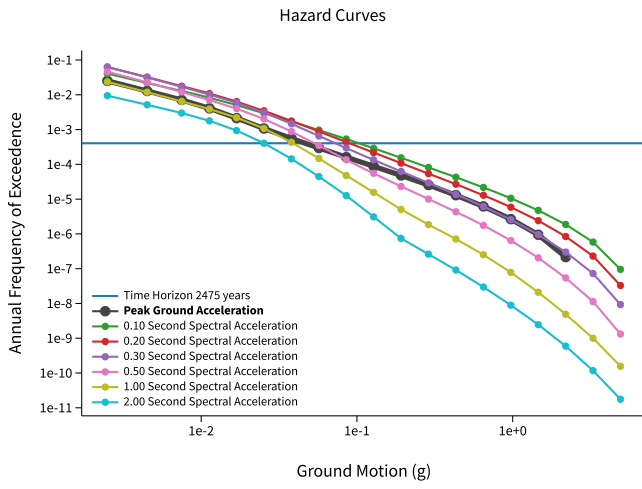
NOTES:



## **ATTACHMENT 2**

# **LIQUEFACTION ANALYSIS FIGURES AND RESULTS**

# ^ Hazard Curve

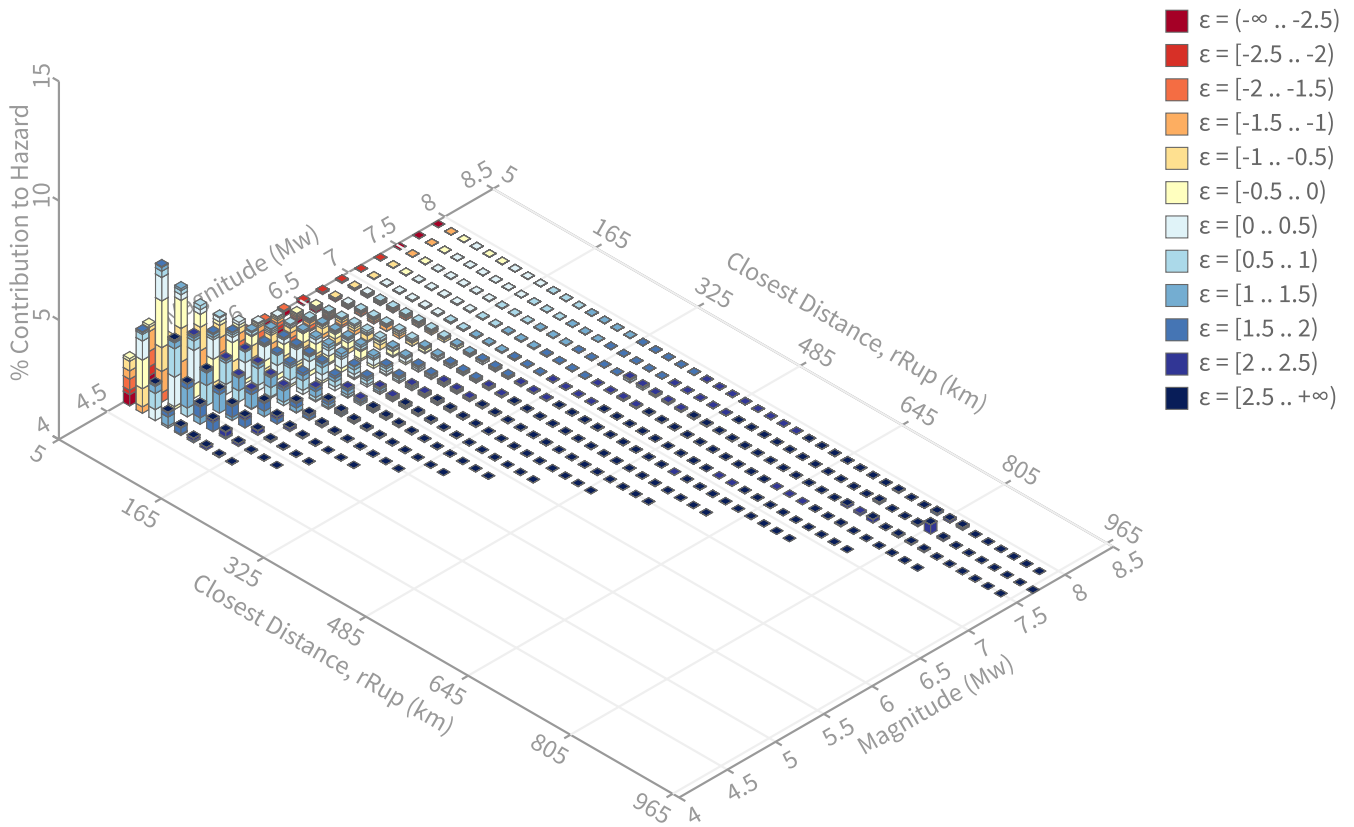


[View Raw Data](#)

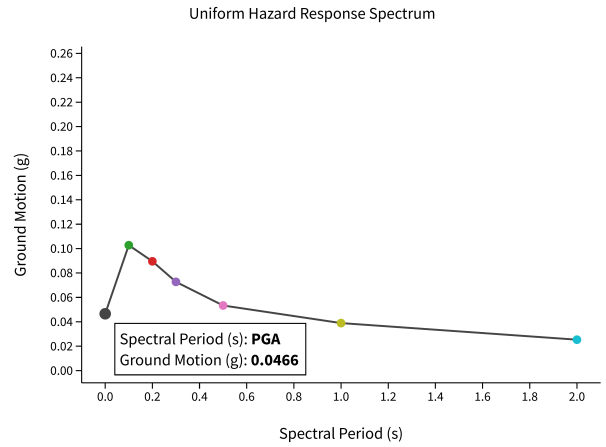
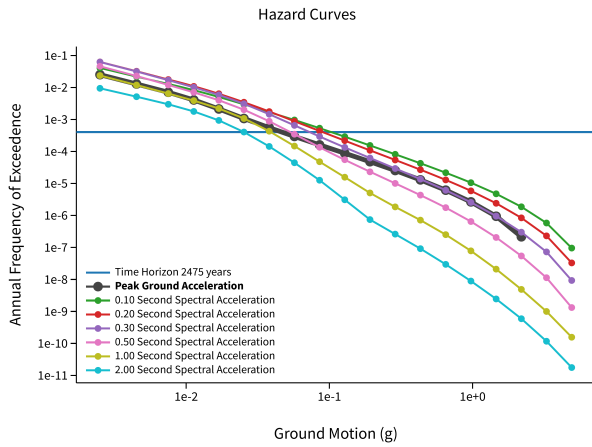
^ Deaggregation

Component

Total



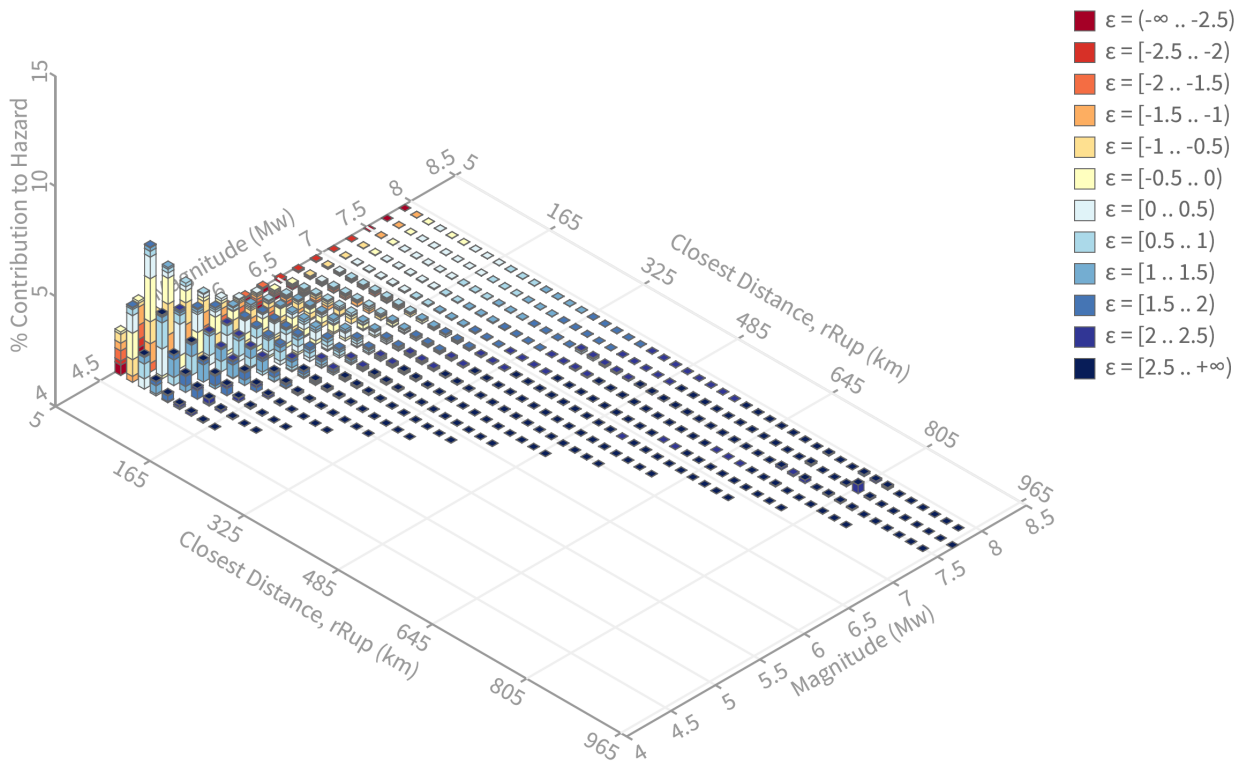
## ^ Hazard Curve

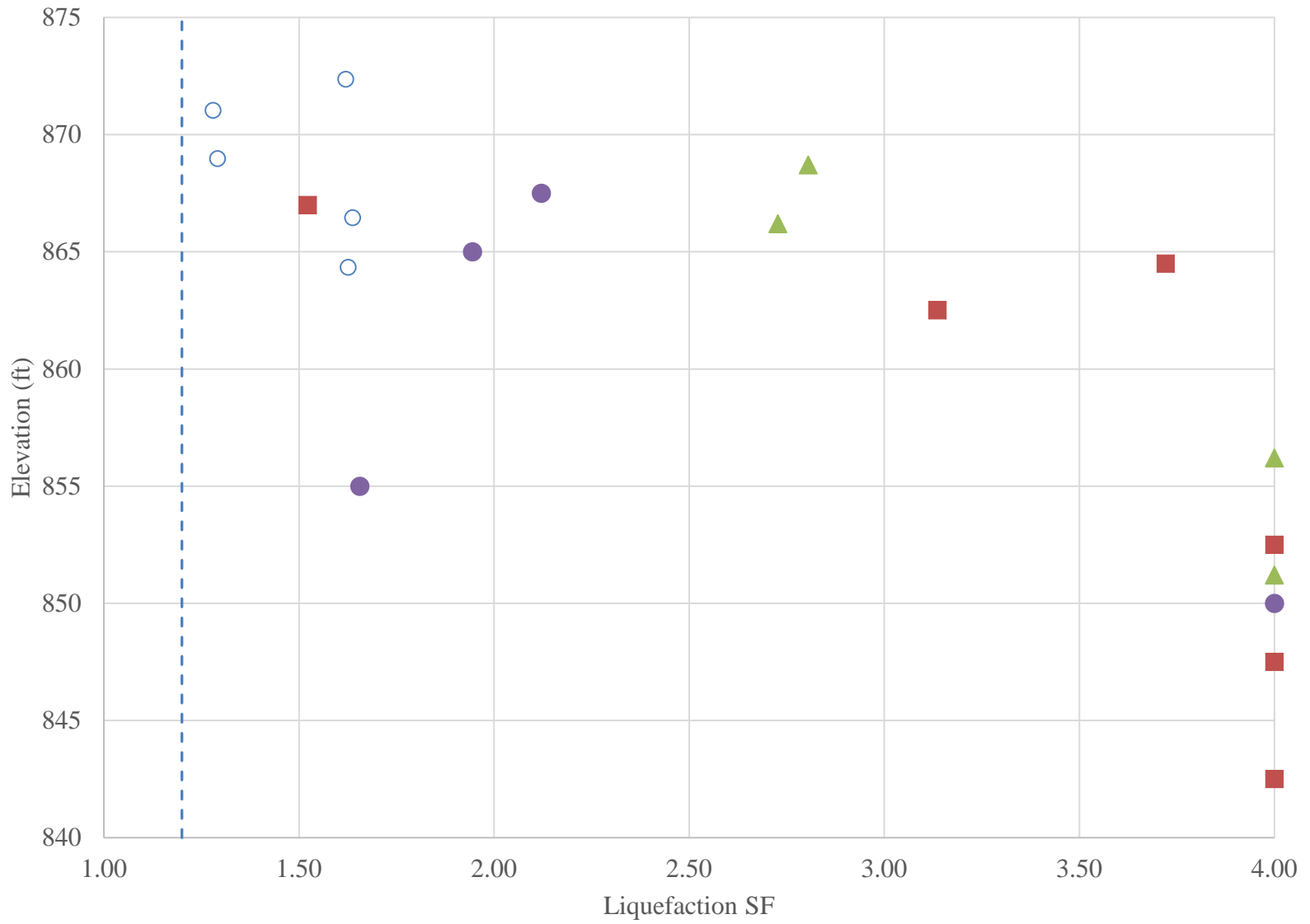


## ^ Deaggregation

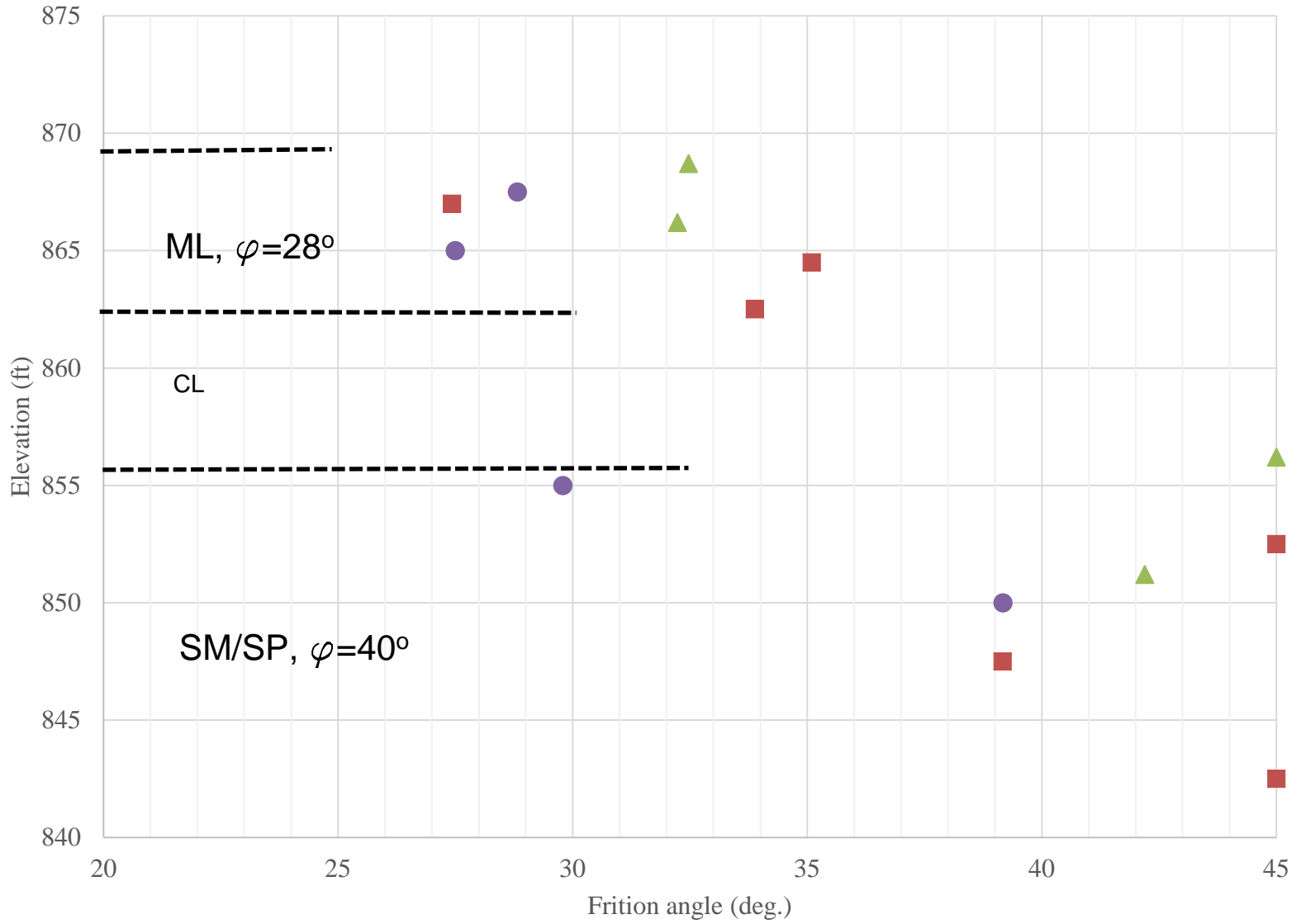
### Component

Total v





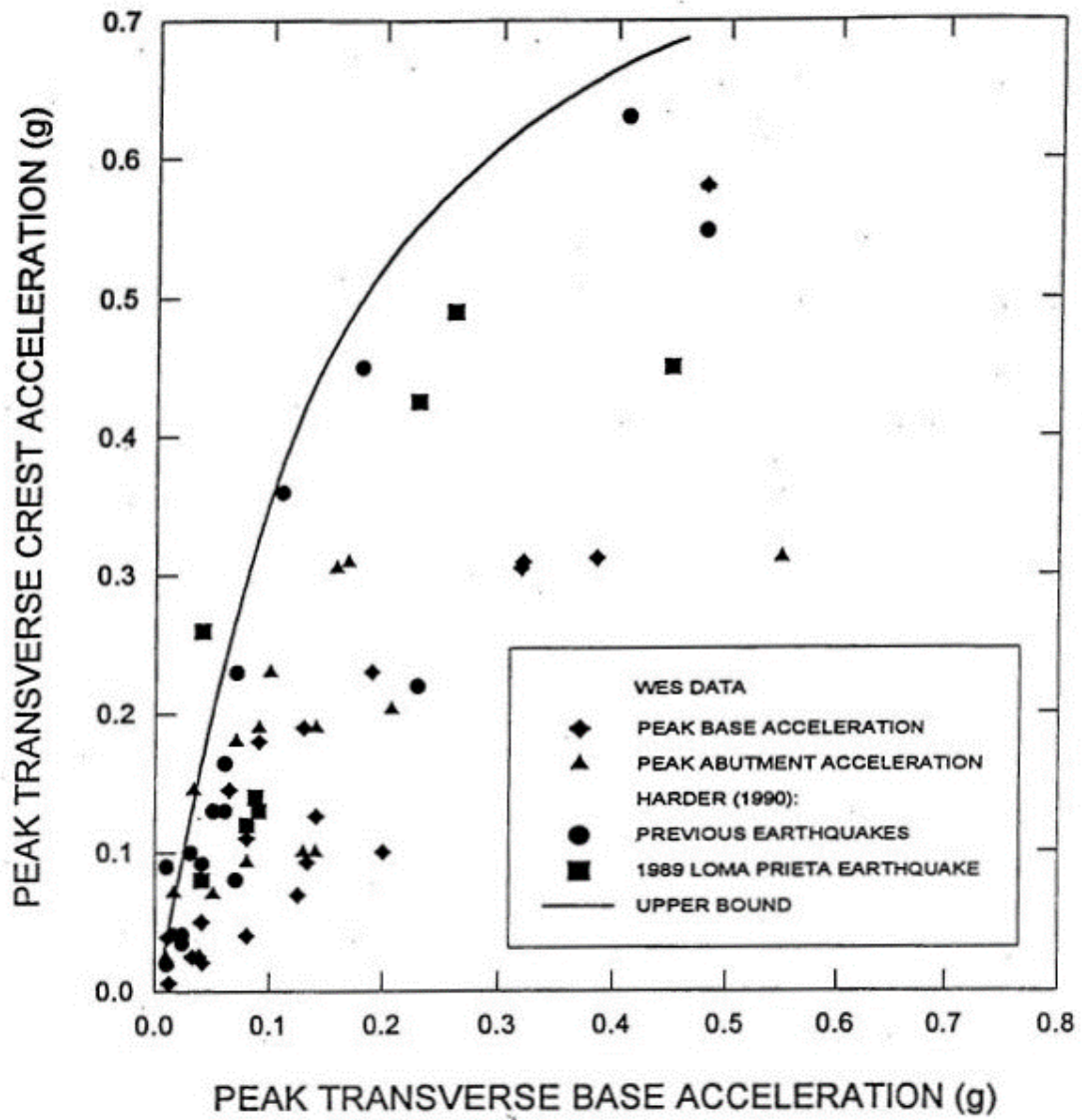
○ Ap-3 (Conservatively assumed N=0)    ■ ECT-18-B01    ▲ ECT-18-B02    ● ECT-18-B03    - - - SF=1.2 criteria ((SF=4 indicates SF>=4))



■ ECT-18-B01

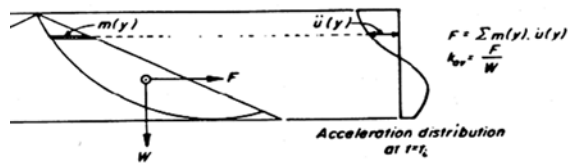
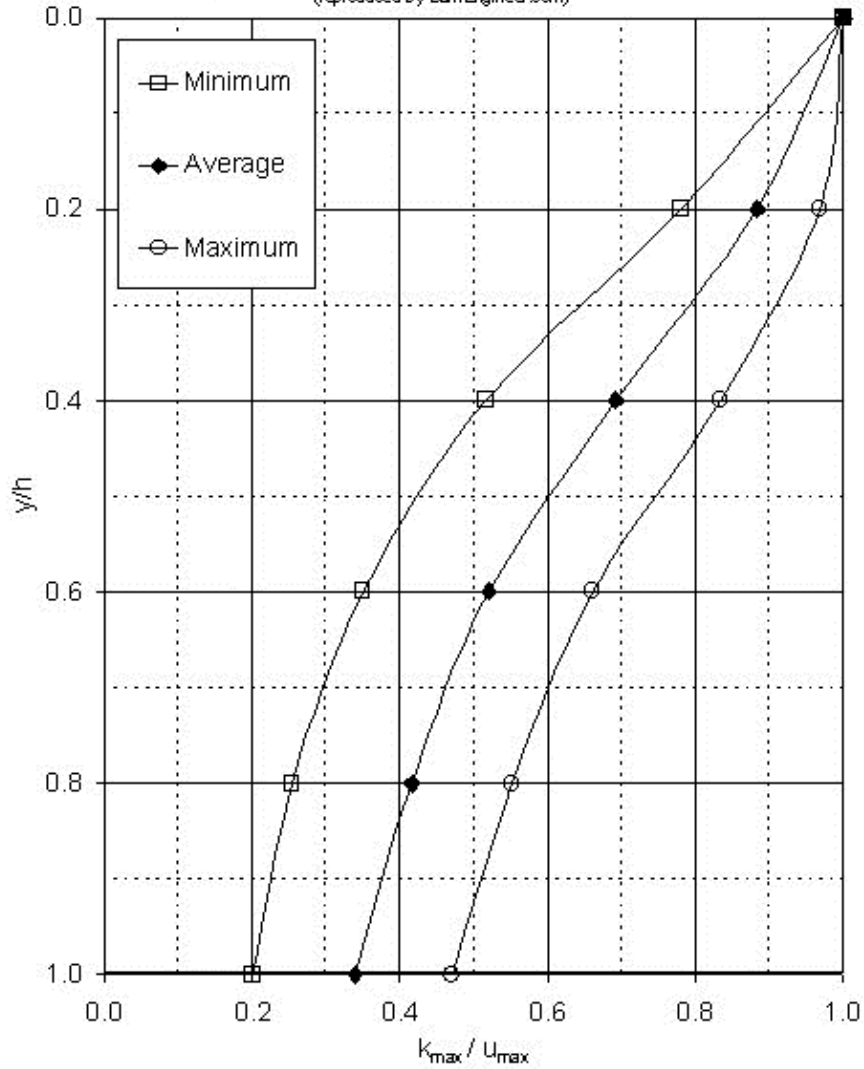
▲ ECT-18-B02

● ECT-18-B03

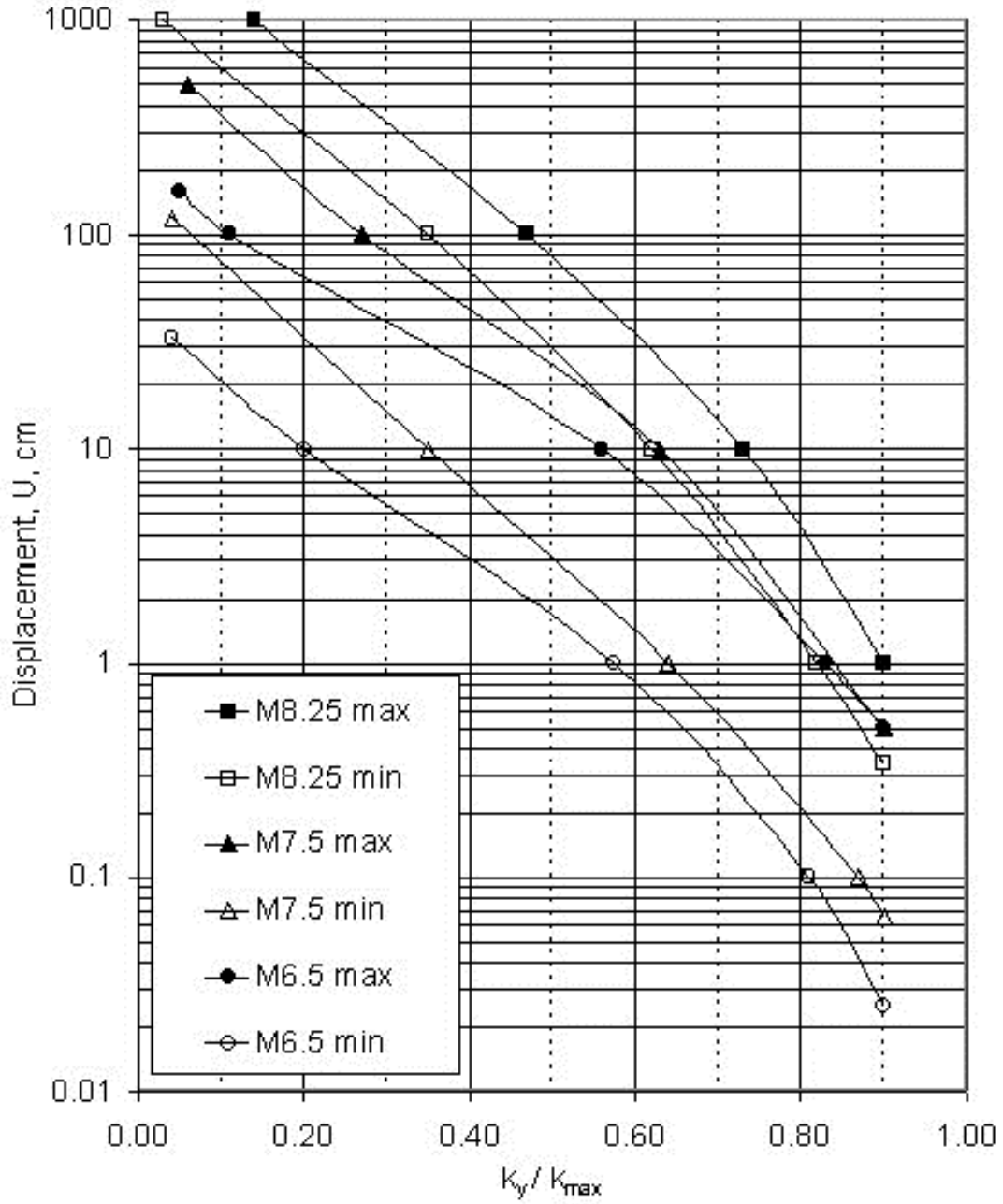




Variation of maximum acceleration ratio with depth of sliding mass (Makdisi-Seed, 1978)  
 (reproduced by Dam Engineer.com)



Variation of permanent displacement with yield acceleration (Makdisi-Seed, 1978)  
 (reproduced by DamEngineer.com)









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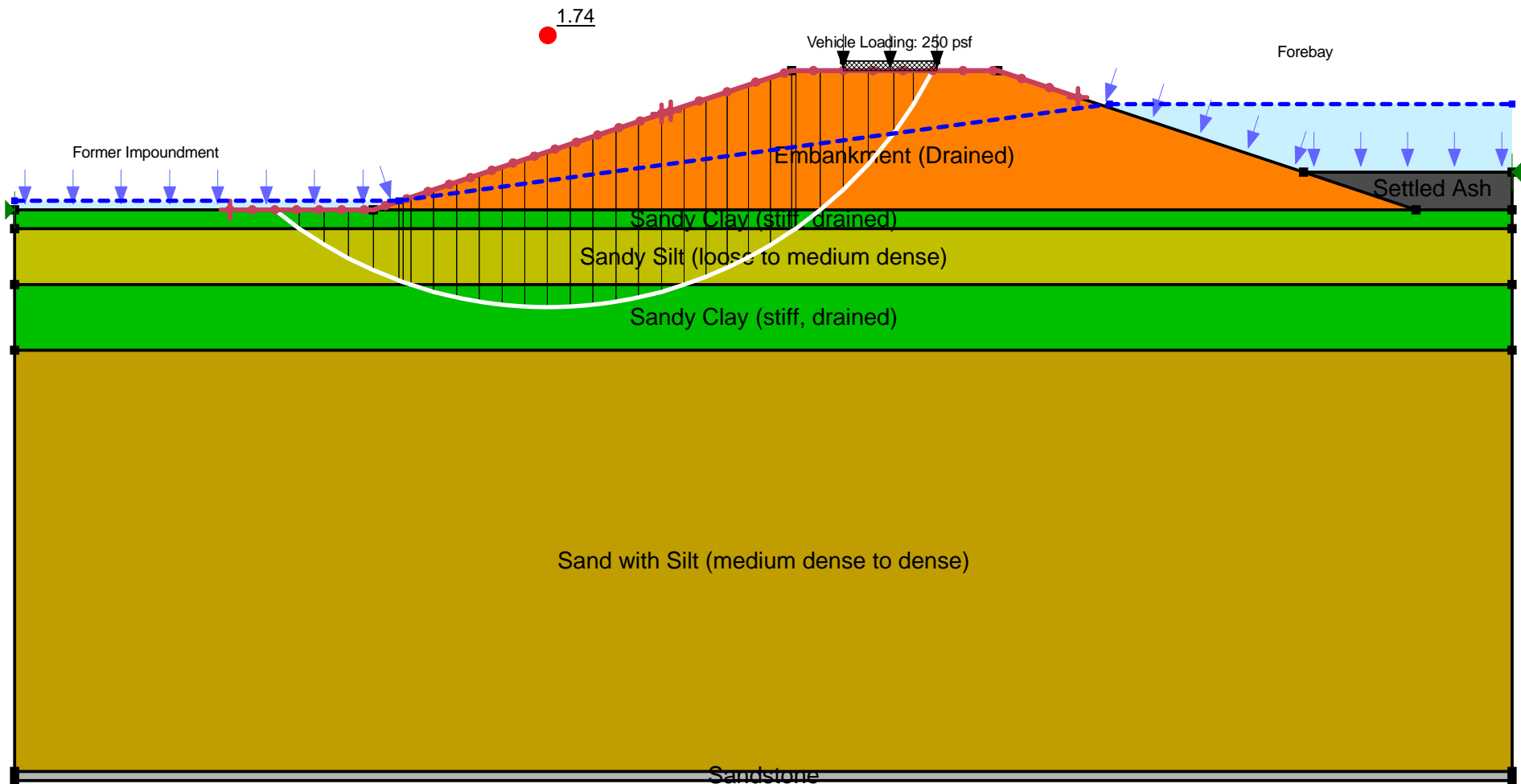
# **ATTACHMENT 3**

## **STABILITY ANALYSES RESULTS**



Name: Forebay - El. 882.3 feet  
 Description: Maximum Storage Pool, Drained Conditions  
 Method: Spencer  
 FS: 1.74

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Embankment (Drained)	120	200	28
	Sand with Silt (medium dense to dense)	125	0	40
	Sandstone	160	2,000	45
	Sandy Clay (stiff, drained)	125	150	18
	Sandy Silt (loose to medium dense)	125	0	28
	Settled Ash	90	0	30





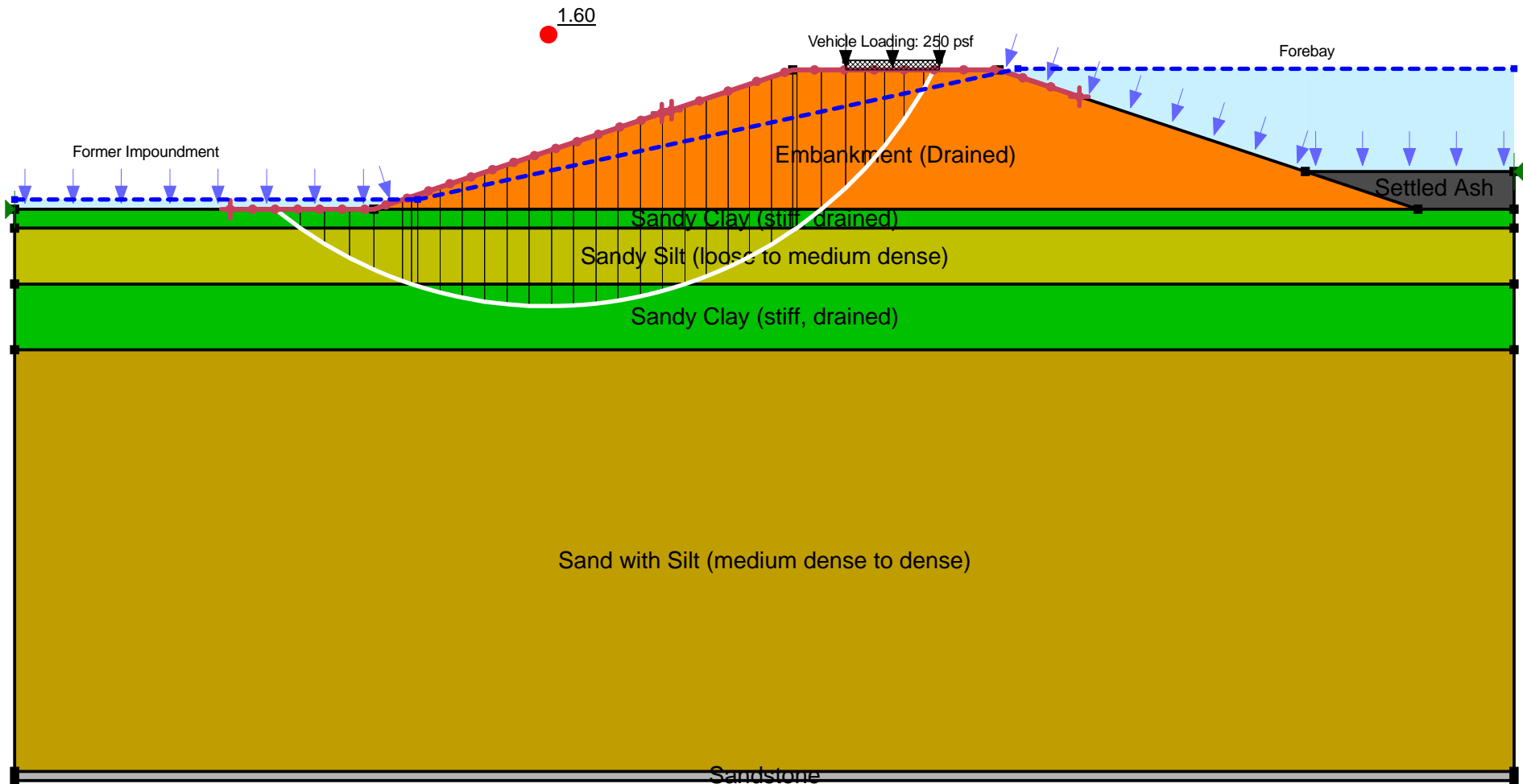
Name: Forebay - El. 885.9 feet

Description: Maximum Surcharge Pool, Drained Conditions

Method: Spencer

FS: 1.60

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
Orange	Embankment (Drained)	120	200	28
Yellow-Green	Sand with Silt (medium dense to dense)	125	0	40
Grey	Sandstone	160	2,000	45
Green	Sandy Clay (stiff, drained)	125	150	18
Light Green	Sandy Silt (loose to medium dense)	125	0	28
Dark Grey	Settled Ash	90	0	30





Name: Forebay - Earthquake Loading

Description: Seismic, Maximum Storage Pool, Undrained Conditions

Method: Spencer

FS: 2.13

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
Pink	Embankment (Undrained)	120	1,000	0
Yellow-Green	Sand with Silt (medium dense to dense)	125	0	40
Grey	Sandstone	160	2,000	45
Green	Sandy Clay (stiff, undrained)	125	1,500	0
Light Green	Sandy Silt (loose to medium dense)	125	0	28
Dark Grey	Settled Ash	90	0	30

