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)

Former Erickson Power Station –

Former Forebay, Former Retention Basin, and
Former Clear Water Pond

August 12, 2025

Prepared for:
Lansing Board of Water & Light
Former Erickson Power Station
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1 Introduction and Purpose

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

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 previously determined that the Former Forebay, Former Retention Basin, and Former Clear Water Pond at the Former Erickson Power Station met 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 5-year periodic structural stability assessment and 5-year periodic safety factor assessment for the Former Forebay, Former Retention Basin, and Former Clear Water Pond.

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. Andrew Bertapelle, P.E., both of HDR. Mr. Burkett is a registered Professional Engineer in the State of Michigan.

1.1 Site Location

The Former Erickson Power Station was 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 Former 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 site vicinity map, Figure 1.

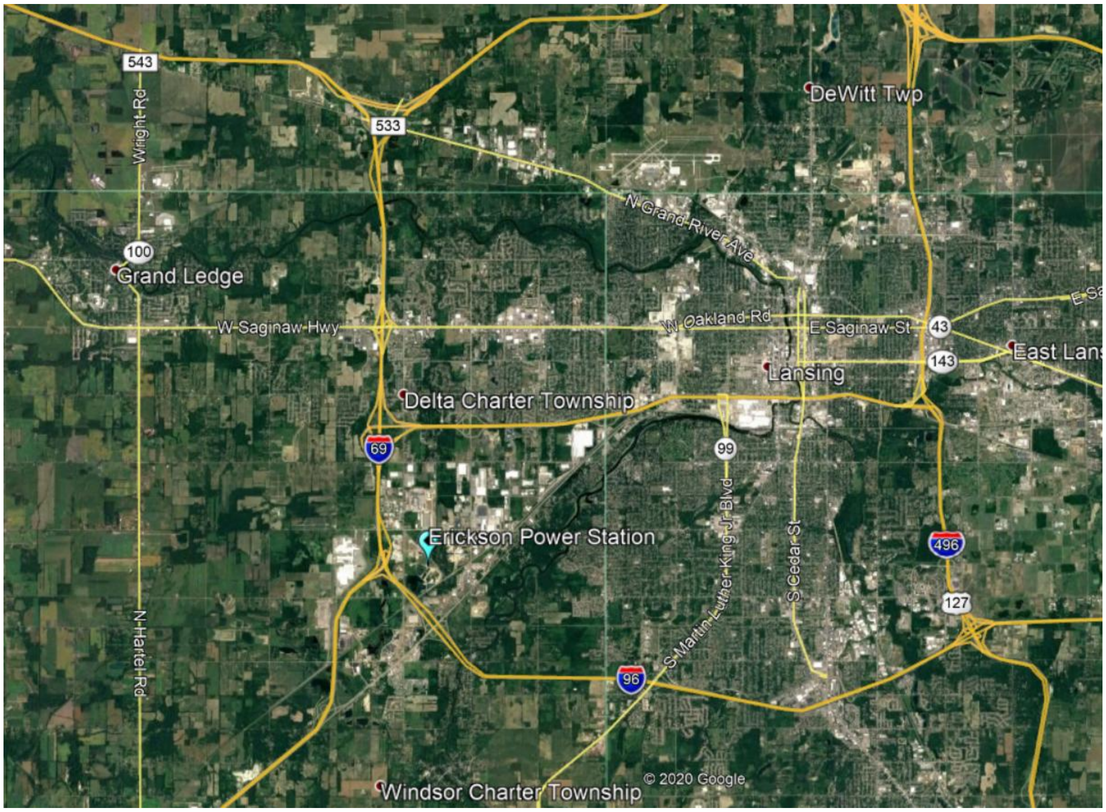


Figure 1. Site Vicinity Map

1.2 Site Description

The Former Erickson Power Station was constructed starting in 1970 and completed in 1973. The Former Station contained a single coal-fired steam turbine/generator capable of producing 165 megawatts of electricity, while it was still in operation. Coal-fired operations and CCR process water discharges at the Former Station ceased in 2022 as part of BWL's move to cleaner energy sources. Closure of the three (3) former regulated CCR surface impoundments began in February 2023. Since then, the former regulated CCR units have been physically closed, and all CCR materials have been removed and appropriately managed. The Former Forebay and Former Retention Basin were then filled with clean fill material and all former impoundments were graded to direct stormwater runoff to the Former Impoundment area. Verification of CCR removal from the regulated former CCR units was completed and documented in the CCR Removal Report, dated November 4, 2024 (Ref. [14]).

Following the decommissioning of the three (3) regulated CCR surface impoundments, the entire impoundment system (including the Former Impoundment) now collectively functions as a limited stormwater collection basin. These former impoundments form a roughly contiguous drainage system. Their only inflow is stormwater from direct rainfall onto the impoundment system.

Figure 2 displays the Former Erickson Power Station site configuration.

Figure 2. Former Erickson Power Station Site Configuration

The Former Forebay and Former Retention Basin no longer have embankments as these former impoundments have been filled. The Former Clear Water Pond has three hydraulic structures that extend through embankments:

- Lake Delta Drainage Structure
- Lake Delta Transfer Structure
- Emergency Overflow Structure

Figure 3 displays a plan view of the Former Clear Water Pond with the locations of the associated hydraulic structures and pipes extending through the embankments.

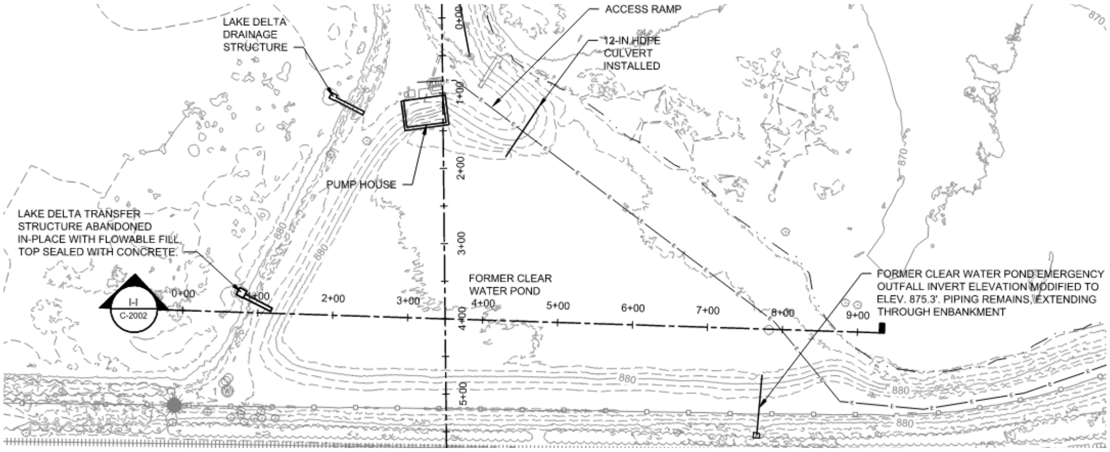


Figure 3. Location of Former Clear Water Pond Hydraulic Structures

The following provides details of each hydraulic structure located at the Former Clear Water Pond.

Lake Delta Drainage Structure

The Lake Delta Drainage Structure is located between the Former Clear Water Pond and Lake Delta. Water from Lake Delta flows through the drainage structure (extending through the Former Clear Water Pond embankment) to the Pump House where it is sent to Delta Energy Park, BWL's natural gas fired combined cycle power plant, to use. The discharge pipe consists of 24-inch ductile iron pipe, equipped with square, (6-feet x 6-feet) concrete, anti-seep collars.

The invert of the overflow weir is at approximately El. 883.6 feet NAVD 88 and the invert of the outlet is at approximately El. 870.4 feet NAVD 88.

Lake Delta Transfer Structure

The Lake Delta Transfer Structure is located between the Former Clear Water Pond and Lake Delta. Historically, water from Lake Delta was controlled by stoplogs to allow flow over the overflow weir through the discharge pipe extending through the Former Clear Water Pond embankment and into the Former Clear Water Pond. After removal of CCR and decommissioning the Former Clear Water Pond, this structure was abandoned in-place with flowable fill, and the top of the intake was sealed with concrete. The abandoned pipe consists of 36-inch ductile iron pipe, equipped with square, (8-feet x 8-feet) concrete, anti-seep collars.

The invert of the overflow weir is at approximately El. 881.9 feet NAVD 88 and the invert of the outlet is at approximately El. 871.4 feet NAVD 88.

Emergency Overflow Structure

The Emergency Overflow Structure is located between the Former Clear Water Pond and the swale adjacent to the property line and Canadian National Railroad right-of-way. The Emergency Overflow Structure was designed for an overflow event of the Former Clear Water Pond to allow water to discharge through the pipe extending through the Former Clear Water Pond embankment and exit into the swale adjacent to the railroad. After removal of CCR and decommissioning the Former Clear Water Pond, the Emergency Overflow Structure vertical riser pipe was removed and only the horizontal pipe remains in place. The pipe consists of 36-inch ductile iron pipe, equipped with square, (8-feet x 8-feet) concrete, anti-seep collars.

1.3 Previous Assessments and Inspections

A previous assessment was performed by GZA GeoEnvironmental, Inc. (GZA) for the Former Erickson Power Station's Former 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 Former Ash Pond which was undergoing closure at the time of the assessment. The Former 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 Former Forebay and Former Retention Basin.

HDR performed the Initial Inspections in accordance with CCR Rule 40 CFR §257.83(b) for the Former Forebay and Former Retention Basin (Ref. [12]), and Former Clear Water Pond (Ref. [11]) in 2020. Additionally, HDR performed the 2021 Annual Inspection (Ref. [5]), the 2022 Annual Inspection (Ref. [6]), the 2023 Annual Inspection (Ref. [7]), the 2024 Annual Inspection (Ref. [8]) and the 2025 Annual Inspection (Ref. [9]).

Prior to the removal of CCR from the former regulated CCR units, BWL performed weekly inspections for the entire former CCR impoundment system, per the requirements of 40 CFR §257.83(a)(1). The weekly inspections were completed by qualified individuals to check for potentially hazardous conditions or structural weakness and the results of the inspections were documented internally on Weekly Inspection Reports. Following verification of CCR removal (Ref. [14]), and subsequent EGLE approval on November 8, 2024, operating criteria and inspection requirements of 40 CFR §257.83 no longer apply to the former impoundments.

There have been no reports of structural instability at the Former Forebay, Former Retention Basin, or Former Clear Water Pond during previous inspections, except for minor sloughing at the Former Clear Water Pond which was documented in previous inspections.

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 Former 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. [17]). 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 Former Forebay and Former Retention Basin for the then new gas-fired combustion turbine power plant (Ref. [19]). From 2019 to 2025, HDR installed 42 wells at 21 distinct locations across the site. Monitoring wells MW-3 and MW-4 were installed in the vicinity of the Former Forebay and Former Retention Basin, respectively, and MW-1 and MW-14 were both installed in the vicinity of the Former Clear Water Pond (Ref. [13]).

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

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

ID	Type	Year	Engineering Firm	Reference
AP-3, AP-4, AP-5, and AP-6	Geotechnical Borings	1969	Dames & Moore	Ref. [17]
AP-2	Test Pit	1969	Dames & Moore	Ref. [17]
ECT-18-B01 thru ECT-18-B03	Geotechnical Borings	2018	SME	Ref. [19]
CW-SB-01 thru CW-SB-03	Geotechnical Borings	2018	MD&E	Ref. [17]
EW-F-1 thru EW-F-6	Test Pits	2018	MD&E	Ref. [17]
MW-1, MW-3, MW-4, MW-14	Monitoring Wells	2019-2025	HDR	Ref. [13]

The approximate locations of the borings, test pits, and monitoring wells are shown on Figure 4. The borings logs, test pit records, and monitoring well logs are provided in Attachment 1.

Figure 4. Approximate Boring/Monitoring Well Locations

The foundation of the Former Forebay and Former Retention Basin was cut to approximately El. 870.9 feet NAVD 88 prior to construction of the embankments. The boring logs, test pit records, and monitoring well logs indicate that the Former Forebay and Former Retention Basin foundation was 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 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, 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 Former Forebay and Former 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). 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.

The historical boring logs prepared by Dames & Moore (1969) prior to the construction of the Former Clear Water Pond indicate that the embankment foundation was comprised primarily by alternating layers of sands and silts (i.e. sand, silty sand, clayey sand, clayey silt) from the surface to depths of approximately 60 feet below existing grade at the time of the investigation. The installation log of MW-1 and MW-14, installed in 2019 and 2023, respectively, indicates the presence of cohesive layers (Lean and Fat Clay) with intermittent granular layers. Gravel, traces of clay, and organic matter were observed in the alternating sand and silt layers. In the deepest boring performed (AP-5), a sandstone layer was encountered at approximately 60 feet below grade, which is the depth that the boring was terminated.

The previous subsurface investigation documentation indicates that the foundation is competent and stable.

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.

There are no longer interior slopes present in the Former Forebay and Former Retention Basin. The exterior slope of the Former Retention Basin, adjacent to Lake Delta, is protected by riprap.

The interior slopes of the Former Clear Water Pond are protected by vegetation and water is no longer impounded in the Former Clear Water Pond. The exterior slopes of the Former Clear Water Pond West Embankment (adjacent to Lake Delta) are protected by vegetation above the water line and riprap below the water line. The crest of the Former Clear Water Pond consists of a gravelly/soil surface. The road on the crest of the embankment is graded and maintained periodically.

The existing slope protection measures of the Former Forebay, Former Retention Basin, and Former Clear Water Pond are considered adequate to provide protection against surface erosion, wave action, and adverse effects of sudden drawdown. Additionally, as the former impoundments are no longer in use and only function as a limited stormwater collection basin for direct rainfall onto the former impoundment system, the only drawdown is from limited stormwater evaporation or infiltration.

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. [18]) for the Former Forebay and Former Retention Basin. 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. [18]).

Construction drawings and specifications, including compaction records for the Former Clear Water Pond, were unavailable for review, however, GZA 2012 (Ref. [3]), referenced the original specifications for the embankment, and noted that "*It was reportedly constructed on clays and silts underlain by silts and sands underlain by bedrock (sandstone). According to the specifications construction for the Ash Pond, the natural ground surface, which also forms the liner, was stripped and scarified to provide a bond with the first layer of the dike fill. The construction specifications indicate that the embankment was constructed primarily with selected on-site clay borrow material from locations shown in Figure 4. The fill was specified to be placed in layers of 8-inch loose thickness and compacted to 95% of the maximum dry density determined by ASTM standard D-1557.*"

A review of the construction records of the Former Forebay, Former Retention Basin, and Former Clear Water Pond indicated that the embankments were mechanically compacted to a density sufficient to withstand the range of loading conditions in the former CCR units.

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 is present on slopes of the Former Forebay, Former Retention Basin, and Former Clear Water Pond embankments, in addition to stone riprap. BWL maintains the vegetation and any overgrown vegetation is 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.***

During decommissioning of the Former Clear Water Pond, the emergency overflow vertical riser pipe (the only potential outflow of the entire impoundment system) was removed and only the horizontal pipe remains in place. However, the results of hydrologic and hydraulic modeling show that this horizontal pipe outfall would not spill over in a 100-year flood event (Ref. [4]) to discharge to the swale for the three low hazard potential former CCR surface impoundments.

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.

The hydraulic structures underlying the base of the Former Clear Water Pond or passing through the Former Clear Water Pond embankment consist of the following:

- Lake Delta Drainage Structure
- Lake Delta Transfer Structure
- Emergency Overflow Structure

Hydraulic structures no longer pass through the Former Forebay or Former Retention Basin embankments. Details of each hydraulic structure are discussed in Section 1.2. Each hydraulic structure observed during the 2025 inspection (Ref. [9]) appeared to maintain structural integrity. 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. 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 slope of the Former Clear Water Pond is Lake Delta. 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 River Pump House located on the Grand River to maintain lake levels for recreation at a design elevation of 882.5 feet. Lake Delta serves as emergency backup water for Delta Energy Park and can be drawn down at times for plant use, if needed. However, the Former Clear Water Pond no longer retains water, and all ash has been removed, therefore during rapid drawdown conditions in Lake Delta, the embankment will maintain structural stability.

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 previous annual inspections performed by HDR, no structural stability deficiencies were identified for the embankments.

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. [20]) 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.***

As discussed in this section, the *Safety Factor Assessment* is focused on the embankment separating the Former Clear Water Pond and Lake Delta as the Former Forebay and Former Retention Basin have been filled.

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 ¼ 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 embankment was determined using the existing topography obtained from the 2025 topographic survey completed as part of the decommissioning efforts of the three former impoundments, the interpreted subsurface profile from the available borings (AP-4, AP-5, AP-6, MW-1 and MW-14) at the Former Clear Water Pond, and the interpreted phreatic surface based on observations at the site and from the monitoring wells (MW-1 and MW-14) installed on the south embankment of the Former Clear Water Pond.

One section of the embankment was considered as potentially being critical based on geometry, described below, and located as shown on Figure 5, and can be seen in Figure 6.

- Section 1, located at the western embankment of the Former Clear Water Pond, is adjacent to Lake Delta. Section 1 was selected due to the geometry of the slope and the differential head acting on the section. Due to the geometry that is present for this portion of the Former Clear Water Pond embankment, it was deemed more critical than the other portions of the embankment alignment.

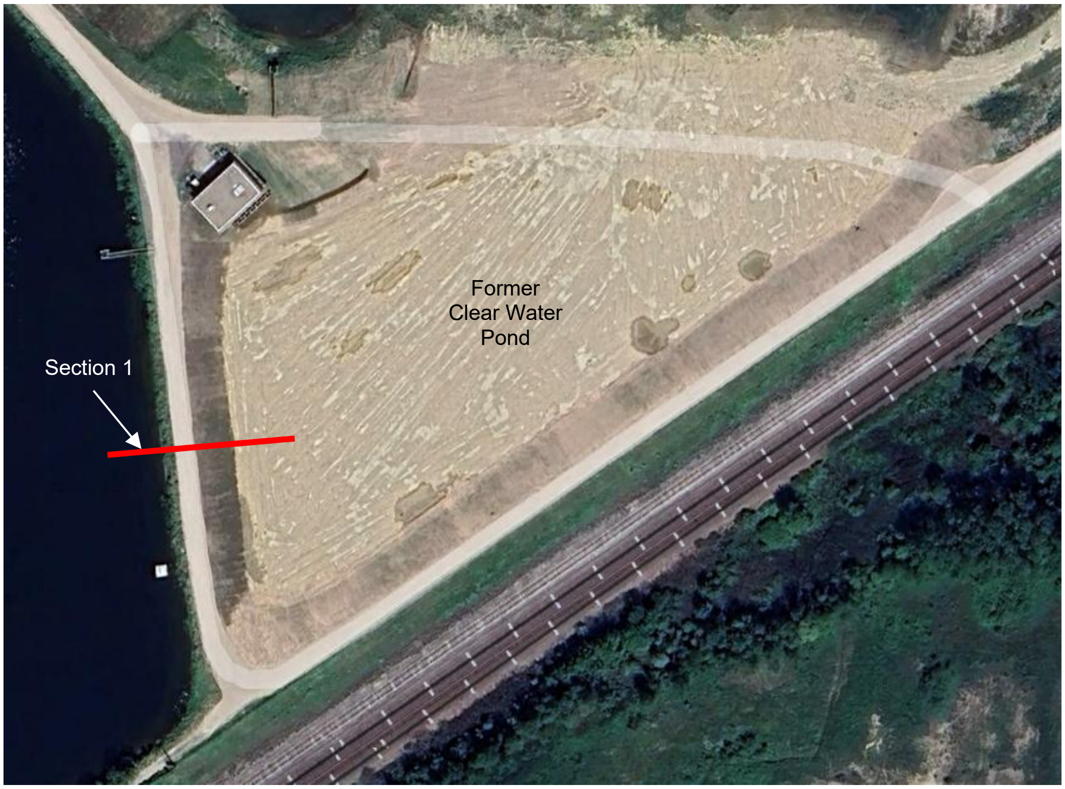


Figure 5. Location of Section 1

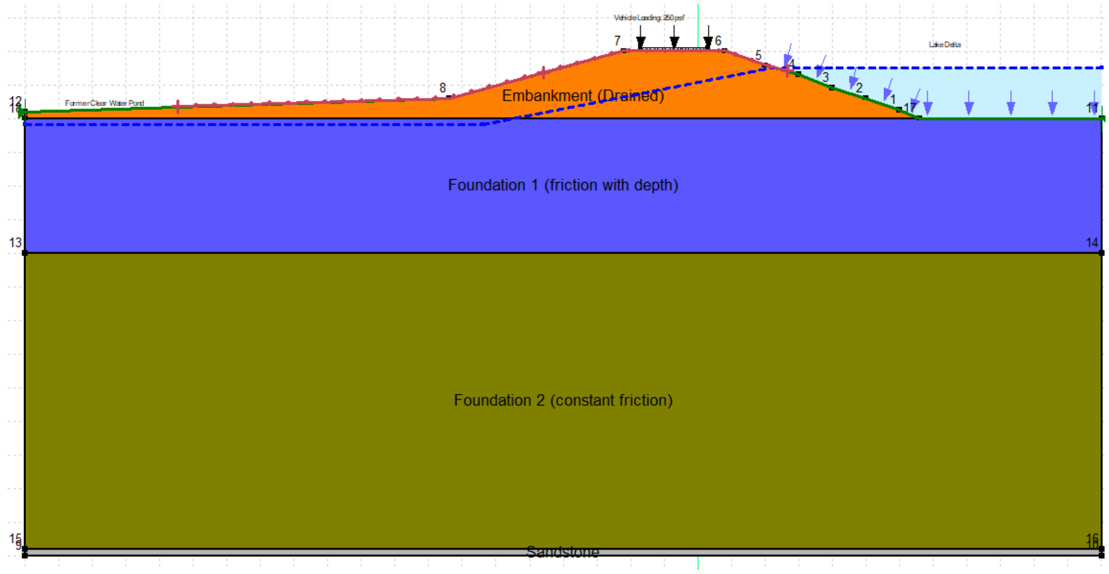


Figure 6. Section 1 Cross Section

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)	882.5[1]	1.5
Maximum Surcharge	885[2]	1.4
Seismic[3]	882.5[1]	1.0
Post-earthquake - Liquefaction	882.5[1]	1.2

[1] Assumed to be normal operating pool level of Lake Delta.
[2] Assumed to be approximately at Top of Dike elevation of the Former Clear Water Pond according to as-built construction drawings (Ref. [10]).
[3] 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 of Lake Delta. 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 Lake Delta is El. 882.5 feet NAVD 88. The phreatic surface was assumed to follow a straight line from headwater on the upstream slope to El. 874.9 in the Former Clear Water Pond, based on historical the maximum readings from nearby MW-1 and MW-14.

The maximum pool surcharge scenario assumes a temporary rise of Lake Delta water elevation due to rainfall to the elevation of the top of embankment, as shown on the as-built construction drawings (Ref. [10]), of El. 885.0 feet NAVD 88. 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. 872 feet NAVD 88. 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 Former Erickson Power Station's former 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. [17]). In addition to the 1969 soil borings, geoprobe borings and test pits were performed at the site by MD&E in 2018. From 2019 to 2025, HDR installed 42 wells at 21 distinct locations across the site. Monitoring wells MW-3 and MW-4 were installed in the vicinity of the Former Forebay and Former Retention Basin, respectively, and MW-1 and MW-14 were both installed in the vicinity of the Former Clear Water Pond (Ref. [13]).

The embankment stratigraphy is shown in Figure 6 and the material properties used for the slope stability analysis are presented in Table 3-2. The estimated material engineering properties were based on correlations with Standard Penetration Testing (SPT) performed in 1969 and HDR's experience with similar conditions. The boring 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	Elevation (feet)	Unit Weight, γ (pcf)	Short-term (Undrained)		Long-term (Drained	
			Cohesion, c (psf)	Friction Angle, φ (degrees)	Cohesion, c' (psf)	Friction Angle, φ' (degrees)
Embankment	886 to 873	130	1,000	0	200	28
Foundation 1*	873 to 855	120	0	26 to 35	0	26 to 35
Foundation 2	855 to 811	120	0	35	0	35
Sandstone	811 to 810	160	2,000	45	2,000	45

* - Friction angle of foundation was modeled to increase linearly (from 26 to 35 degrees) with depth from El. 873 to 855 feet and is constant (35 degrees) thereafter with depth. Friction angle interpretation was taken from a review of the N values provided on boring logs in Attachment 1.

3.7 Vehicle Loading

The crest of the embankment is intermittently used as access roads around the Former Clear Water Pond, 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 clayey and silty sand, and silt that becomes denser with depth. A "triggering analysis" was used to assess the potential for liquefaction of the foundation soils using correlations with the SPT data from Borings AP-4, AP-5, and AP-6. These borings were drilled in 1969 before construction of the embankment in the vicinity of the footprint of the embankment as shown in Figure 7. The borings logs are provided in Attachment 1.

Figure 7. Approximate Boring/Monitoring Well Locations at Former Clear Water Pond

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. [15]). 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%). Two one-dimensional sections were analyzed: 1) a section at the toe of the embankment (i.e. the natural ground) and 2) a section that includes the embankment (i.e. the embankment crest elevation). It was conservatively assumed that the original borings were dry, and, following the start of plant operations, the phreatic surface increased, such that all of the considered layers below the assumed phreatic surface were saturated. Based on these assumptions, the corrected SPT blow counts and soil stresses were calculated for evaluation of cyclic shear strength and stress.

Using the USGS online Unified Hazard Tool (Ref. [22]), 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 and 2 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 degrees 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. [19]), the site is classified as Seismic Site Class C and in accordance with ASCE-7 2016, so a factor of 1.3/0.9 was applied to obtain the site PGA of 0.076g 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 testing was likely not performed. A hammer efficiency of 60% was assumed corresponding to standard rope and cat head SPT method. Due to water level measurements (after ground water stabilization) not being available for review from the historical data, it is assumed that the boreholes were dry in order to be conservative for including the effect of overburden pressure. The methods used to calculate $(N_1)_{60}$ were those that have been proposed by Idriss and Boulanger (Ref. [15]). The raw SPT "N" values (N_{raw}) presented on the boring logs were converted to $(N_1)_{60}$ values using the following equation:

$$\boldsymbol{(N1)_{60}} = N_{RAW}C_NC_EC_BC_RC_S$$

Where:

C_N = Overburden Correction Factor = (P_a/σ'_{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. [15]) and the following equation:

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

Where:

PF = Percent fines passing No. 200 sieve

Using Idriss and Boulanger (Ref. [15]), CRR was then calculated using the following equation:

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

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:

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

Where:

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

$\boldsymbol{\sigma_v}$ = Total Overburden Stress

$\boldsymbol{\sigma'_v}$ = Effective Overburden Stress

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

Where:

a(z) = -1.012-1.126*sin((z/11.73)+5.133)

b(z) = 0.106+0.118*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:

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

Where:

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

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

$\mathbf{K_\sigma}$ = overburden correction factor = 1

Where:

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

$\mathbf{P_a}$ = Pressure at 1 atmosphere

The static shear strength in the liquefaction-susceptible material is small. Therefore, K_α was taken equal to one for the purpose of this analysis. If the FS is greater than 1.2, soil is not susceptible to liquefaction. The calculated factor of safety against seismically-induced liquefaction is presented in on Page 3 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 4 of Attachment 2 shows the calculated value and the assumed bilinear variation of 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 5 of Attachment 2 (Ref. [21]). The average embankment acceleration for a deep failure surface was then obtained from the figure on Page 6 of Attachment 2 (Ref. [16]), using y/h=1, the maximum ratio of 0.47, and an effective seismic coefficient of 0.25*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 earthquake is 1.48 which is greater than 1 and suggests that the deformation of the embankment during and after the 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 7 of Attachment 2 (Ref. [16]), 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	2.9	Attachment 3, Page 1
Maximum Surcharge	1.4	2.6	Attachment 3, Page 2
Seismic	1.0	2.3	Attachment 3, Page 3
Post-earthquake - Liquefaction	1.2	>1.2	Attachment 2, Page 3

4 Professional Engineer Certification

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

Bryce Burkett, P.E.

Senior Geotechnical Project Manager

STATE OF MICHIGAN
TERRY
BRYCE
BURKETT
ENGINEER
No.
6201066757
LICENSED PROFESSIONAL ENGINEER

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. Inflow Design Flood Control System Plan, Erickson Power Station – Former CCR Surface Impoundments, Lansing Board of Water & Light, Lansing, Michigan, June 6, 2025.

Ref. [5] HDR Michigan, Inc. Annual Inspection Report - 2021 – Forebay, Retention Basin, and Clear Water Pond, Lansing Board of Water & Light Erickson Power Station, Lansing, Michigan, April 27, 2022.

Ref. [6] HDR Michigan, Inc. Annual Inspection Report - 2022 – Forebay, Retention Basin, and Clear Water Pond, Lansing Board of Water & Light Erickson Power Station, Lansing, Michigan, May 2, 2022.

Ref. [7] HDR Michigan, Inc. Annual Inspection Report - 2023 – Forebay, Retention Basin, and Clear Water Pond, Lansing Board of Water & Light Erickson Power Station, Lansing, Michigan, February 24, 2023.

Ref. [8] HDR Michigan, Inc. Annual Inspection Report - 2024 – Forebay, Retention Basin, and Clear Water Pond, Lansing Board of Water & Light Erickson Power Station, Lansing, Michigan, February 21, 2024.

Ref. [9] HDR Michigan, Inc. Annual Inspection Report - 2025 – Forebay, Retention Basin, and Clear Water Pond, Lansing Board of Water & Light Erickson Power Station, Lansing, Michigan, February 20, 2025.

Ref. [10] HDR Michigan, Inc. Former Erickson Power Station Ash Impoundments Closure Drawings, As-Built, Lansing Board of Water & Light Erickson Power Station, Lansing, Michigan, July 9, 2025.

Ref. [11] HDR Michigan, Inc. Initial Inspection Report – Clear Water Pond, Lansing Board of Water & Light Erickson Power Station, Lansing, Michigan, June 12, 2020.

Ref. [12] HDR Michigan, Inc. Initial Inspection Report – Forebay and Retention Basin, Lansing Board of Water & Light Erickson Power Station, Lansing, Michigan, August 10, 2020.

Ref. [13] HDR Michigan, Inc. Monitoring Wall Installation Report, Lansing Board of Water & Light Former Erickson Power Station, Lansing, Michigan, May 15, 2025.

Ref. [14] HDR Michigan, Inc. CCR Removal Report, Forebay, Retention Basin, & Clear Water Pond, Lansing Board of Water & Light, Lansing, Michigan, November 4, 2024.

Ref. [15] 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. [16] Makdisi, F.I. and Seed, H.B., Simplified Procedure for Estimating Dam and Embankment Earthquake-Induced Deformations, Journal of Geotechnical Engineering, 1978.

Ref. [17] Mayotte Design & Engineering, P.C. Compliance with 40CFR257-Locations Restrictions. Lansing Board of Water & Light Erickson Station. October 10, 2018.

Ref. [18] Mayotte Design & Engineering, P.C. Construction Documentation Report Ash Impoundment System Reconfiguration, Lansing Board of Water & Light Erickson Station, Lansing, Michigan, May 2015.

Ref. [19] 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. [20] USACE. EM 1110-2-1902, Slope Stability, October 31, 2003.

Ref. [21] US Army Corps of Engineers for the Nuclear Regulatory Commission, Technical Bases for Regulatory Guide for Soil Liquefaction, Figure 40, March 2000.

Ref. [22] United States Geologic Survey, Unified Hazard Tool, accessed July 2025, https://earthquake.usgs.gov/hazards/interactive/

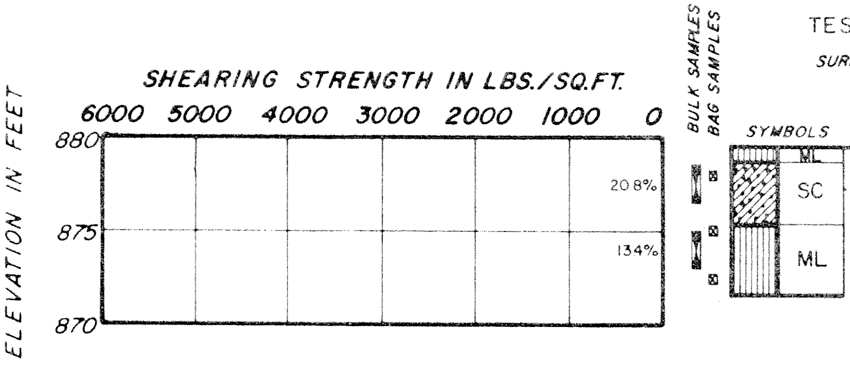
6 Attachments

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

ATTACHMENT 1
BORING LOGS AND MONITORING WELL LOGS

TEST PIT AP-1

SURFACE ELEVATION 879.6



DESCRIPTIONS

DARK BROWN CLAYEY SILT WITH ROOTS - TOPSOIL (9")

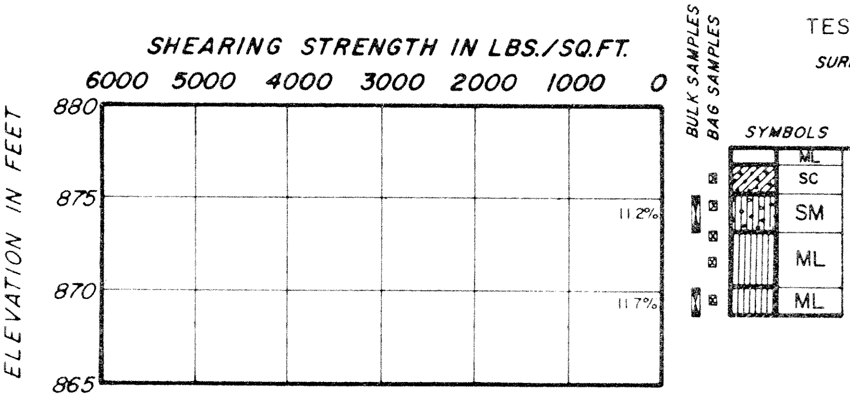
MOTTLED BROWN AND GRAY CLAYEY FINE SAND WITH SOME SMALL GRAVEL
ROOTS TO 2'-6"

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 6.0'

TEST PIT COMPLETED AT 8.0'
ON 6/23/69
MINOR SEEPAGE WATER FROM POCKET OF SAND AT 6.0'

TEST PIT AP-2

SURFACE ELEVATION 877.8



DESCRIPTIONS

DARK BROWN CLAYEY SILT WITH ROOTS - TOPSOIL (10")

MOTTLED BROWN AND GRAY CLAYEY SAND

MOTTLED BROWN AND GRAY FINE SILTY SAND WITH SOME CLAY AND SMALL GRAVEL

BROWN SILT
2" SEAM OF BROWN FINE TO COARSE SAND WITH GRAVEL AT 5.5'

GRAY CLAYEY SILT WITH SOME FINE SAND AND SMALL GRAVEL

TEST PIT COMPLETED AT 9.0'
ON 6/23/69
MINOR SEEPAGE WATER AT 5.5'

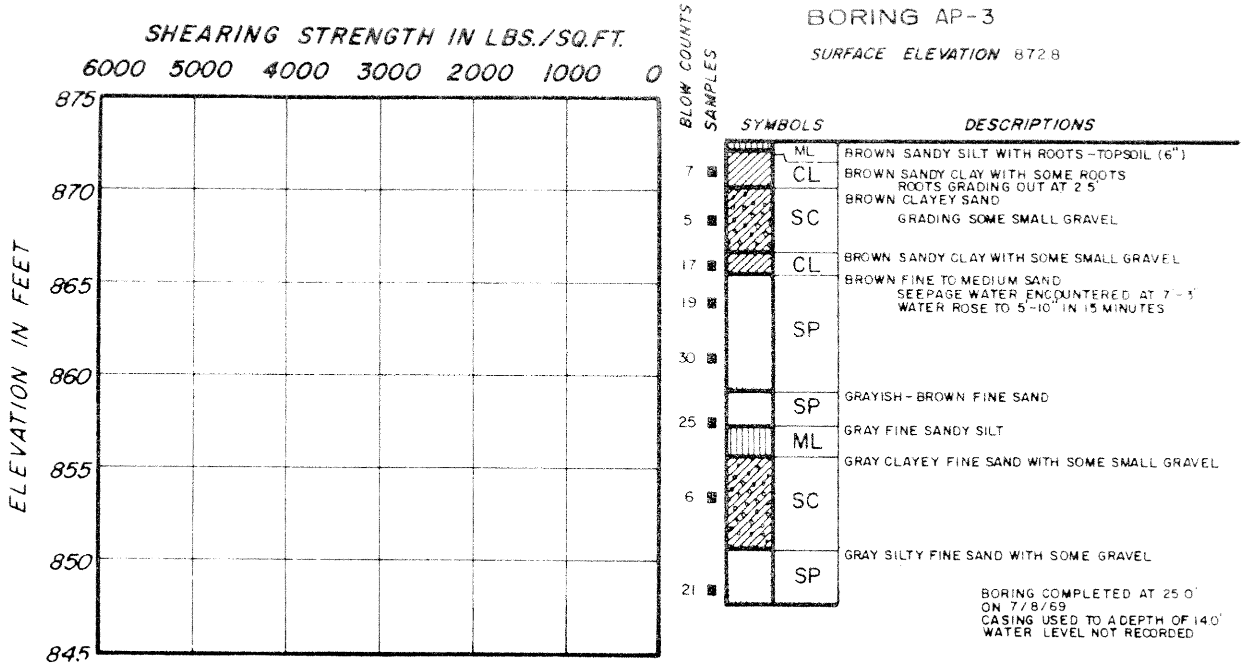
LOG OF TEST PITS

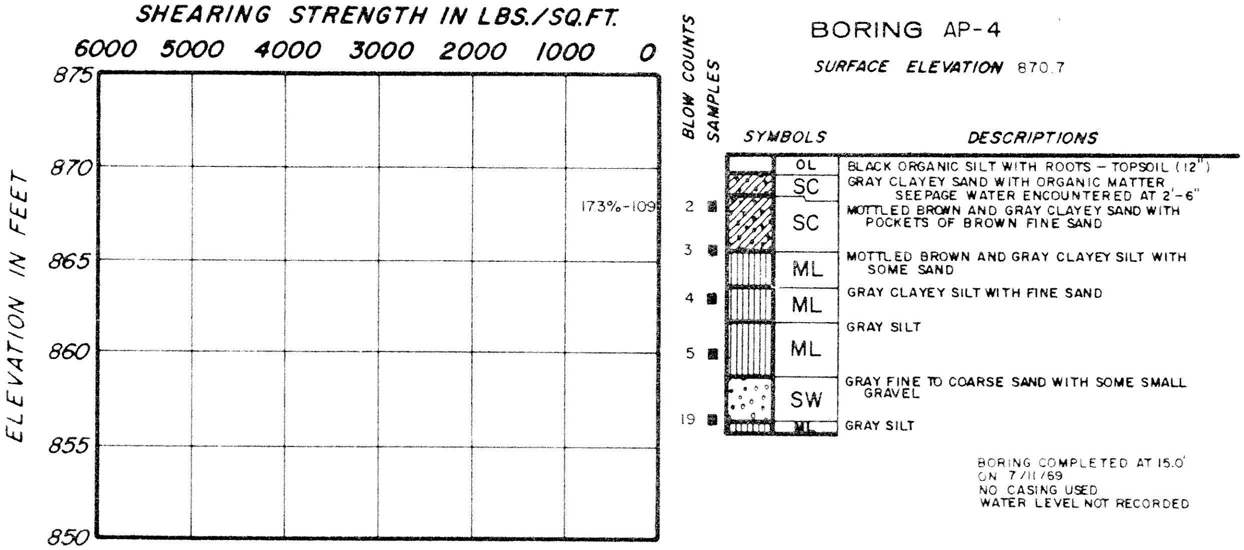
DAMES & MOORE

PLATE A-17

BY_____ DATE_____
BY_____ DATE_____
PLATE_____ OF_____

BY_____ DATE_____
CHECKED BY_____ DATE_____





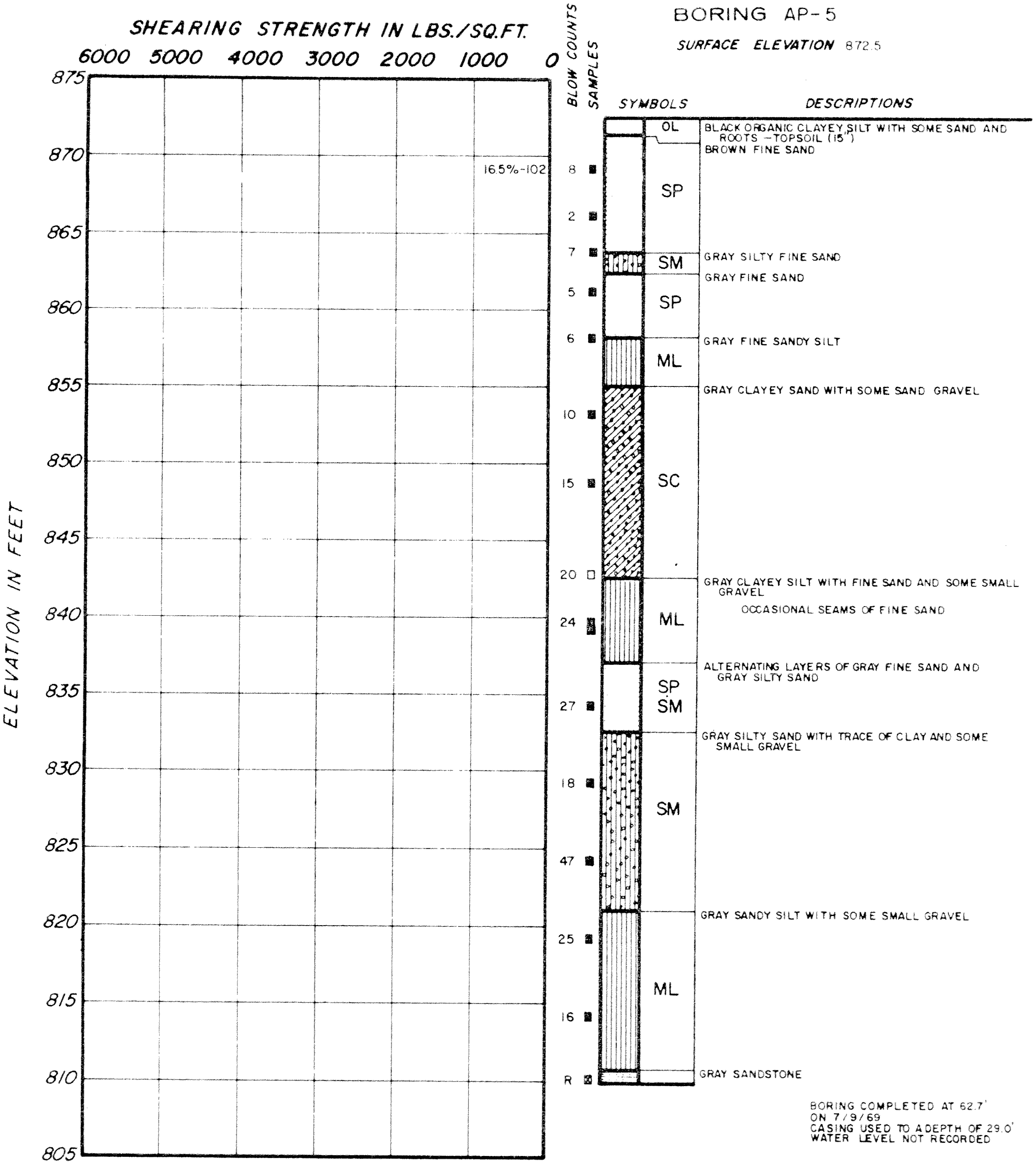
LOG OF BORINGS

DAMES & MOORE

PLATE A-10

BY ____ DATE ____
BY ____ DATE ____
PLATE ____ OF ____

BY ____ DATE ____
CHECKED BY ____ DATE ____



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

LOG OF BORINGS

DAMES & MOORE

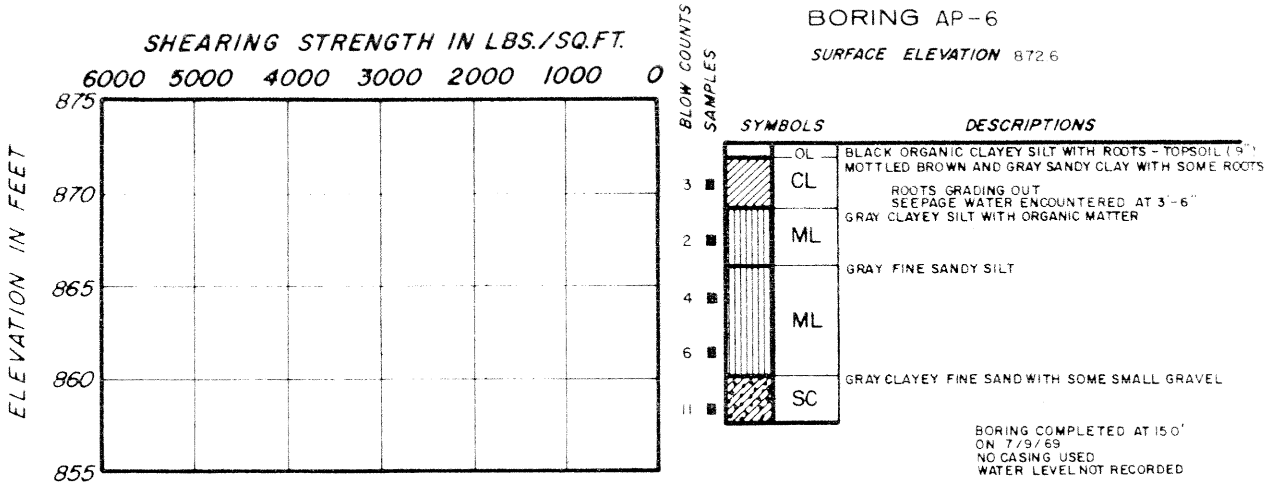
PLATE A-IV

BY ____ DATE ____
BY ____ DATE ____
PLATE ____ OF ____

BY ____ DATE ____
CHECKED BY ____ DATE ____

BORING AP-6

SURFACE ELEVATION 872.6



BLACK ORGANIC CLAYEY SILT WITH ROOTS - TOPSOIL (9")

MOTTLED BROWN AND GRAY SANDY CLAY WITH SOME ROOTS

ROOTS GRADING OUT
SEEPAGE WATER ENCOUNTERED AT 3'-6"

GRAY CLAYEY SILT WITH ORGANIC MATTER

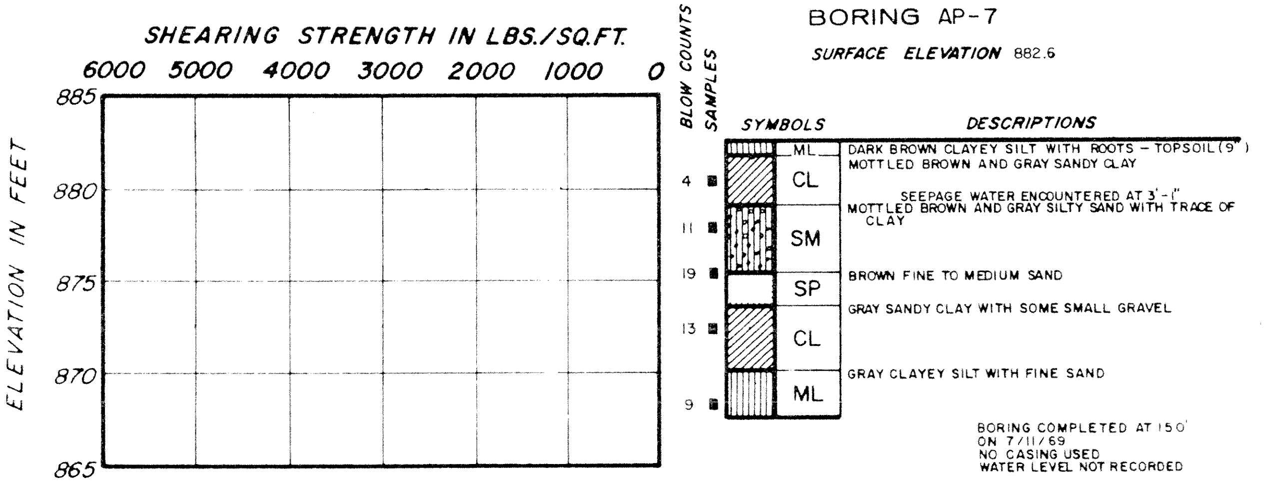
GRAY FINE SANDY SILT

GRAY CLAYEY FINE SAND WITH SOME SMALL GRAVEL

BORING COMPLETED AT 15.0'
ON 7/9/69
NO CASING USED
WATER LEVEL NOT RECORDED

BORING AP-7

SURFACE ELEVATION 882.6



DARK BROWN CLAYEY SILT WITH ROOTS - TOPSOIL (9")

MOTTLED BROWN AND GRAY SANDY CLAY

SEEPAGE WATER ENCOUNTERED AT 3'-1"

MOTTLED BROWN AND GRAY SILTY SAND WITH TRACE OF CLAY

BROWN FINE TO MEDIUM SAND

GRAY SANDY CLAY WITH SOME SMALL GRAVEL

GRAY CLAYEY SILT WITH FINE SAND

BORING COMPLETED AT 15.0'
ON 7/11/69
NO CASING USED
WATER LEVEL NOT RECORDED

LOG OF BORINGS

DAMES & MOORE

PLATE A-1W

BY ____ DATE ____
BY ____ DATE ____
PLATE ____ OF ____

BY ____ DATE ____
CHECKED BY ____ DATE ____

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Civil & Environmental Engineering

PROJECT: LBWL - Erickson
PAGE 1 OF 1
PROJECT NO.:
BORING CW-SB-01
ELEVATION:
DATE 10/2/2018
FIELD GEOLOGIST: Tim Mayotte
RIG Geoprobe

SAMPLE NO., TYPE & DEPTH (ft)	BLOWS/SIX INCHES OR RQD (%)	SAMPLE RECOVERY/SAMPLE LENGTH (ft)	MATERIAL MOISTURE & WATER DEPTH (ft)	MATERIAL DESCRIPTION*			USCS OR ROCK BROKENNESS	REMARKS
				SOIL DENSITY/CONSISTENCY OR ROCK HARDNESS	COLOR	MATERIAL CLASSIFICATION		
1						Void		
2								
3	NA	2.5 ft	Dry	Stiff	Gray-Brown	Sandy Clay	CL	
4								
5								
6								
7	NA	4 ft	Wet	Loose	Gray-Brown	Fine to Medium Sand	SP	Boring consists
8								of layers of
9								saturated soils
10		4 ft	Dry	Loose		Medium Sand	SP	from a depth of
11								5 ft to EOB.
12								
13								
14		4 ft	Moist	Loose		Medium to Coarse Sand	SP	
15								
16			Wet	Stiff		Sandy Clay	SC-CL	
17						End of boring = 16 feet.		
18								
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20								
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40								

REMARKS Boring backfilled with bentonite chips.

MD&E

Civil & Environmental Engineering

PROJECT: LBWL - Erickson

PROJECT NO.:

ELEVATION:

FIELD GEOLOGIST: Tim Mayotte

PAGE 1 OF 1

BORING CW-SB-02

DATE 10/2/2018

RIG Geoprobe

SAMPLE NO., TYPE & DEPTH (ft)	BLOWS/SIX INCHES OR RQD (%)	SAMPLE RECOVERY/SAMPLE LENGTH (ft)	MATERIAL MOISTURE & WATER DEPTH (ft)	MATERIAL DESCRIPTION*			USCS OR ROCK BROKENNESS	REMARKS
				SOIL DENSITY/CONSISTENCY OR ROCK HARDNESS	COLOR	MATERIAL CLASSIFICATION		
1						Void		
2								
3	NA	3 ft	Dry	Stiff	Gray-Brown/	Clay	CL	
4					Black			
5								
6								
7	NA	4 ft	Wet	Loose	Gray-Brown	Medium Sand	SP	Boring consists
8								of layers of
9								saturated soils
10		4 ft	Moist	Loose	Gray-Brown	Medium Sand	SP	from a depth of
11								5 ft to EOB.
12								
13			Wet					
14		4 ft		Loose	Gray-Brown	Medium Sand	SP	
15			Moist					
16								
17						End of boring = 16 feet.		
18								
19								
20								
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40								

REMARKS Boring backfilled with bentonite chips.

MD&E

Civil & Environmental Engineering

PROJECT: LBWL - Erickson

PAGE 1 OF 1

PROJECT NO.:

BORING CW-SB-03

ELEVATION:

DATE 10/2/2018

FIELD GEOLOGIST: Tim Mayotte

RIG Geoprobe

SAMPLE NO., TYPE & DEPTH (ft)	BLOWS/SIX INCHES OR RQD (%)	SAMPLE RECOVERY/SAMPLE LENGTH (ft)	MATERIAL MOISTURE & WATER DEPTH (ft)	MATERIAL DESCRIPTION*			USCS OR ROCK BROKENNESS	REMARKS
				SOIL DENSITY/CONSISTENCY OR ROCK HARDNESS	COLOR	MATERIAL CLASSIFICATION		
1						Void		
2								
3	NA	2.25 ft	Dry	Stiff	Gray-Brown	Clay	CL	
4								
5								
6			Moist -					
7	NA	3 ft	Wet	Loose	Gray-Brown	Medium Sand	SP	Boring consists
8								of layers of
9								saturated soils
10		4 ft	Wet	Loose	Gray-Brown	Medium Sand	SP	from a depth of
11								5 ft to EOB.
12								
13								
14		4 ft	Moist -	Loose	Gray-Brown	Medium Sand	SP	
15			Wet					
16								
17						End of boring = 16 feet.		
18								
19								
20								
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40								

REMARKS Boring backfilled with bentonite chips.

GEOTECHNICAL TESTING SUMMARY

LBWL - Erickson Station - Foundation Samples

MD&E Project No.

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SAMPLE	CLASSIFICATION	%Fines	LL%	PI%	$w\%_{field}$	$w\%_{opt}$	ρ_d (lbs/ft^3)	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
Ranges/Averages:								

NOTES:

BORING ECT-18-B01

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

DRY DENSITY (pcf) -- ■ 90 100 110 120
MOISTURE & ATTERBERG LIMITS (%) PL MC LL 10 20 30 40
▽ HAND PENE.
☒ TORVANE SHEAR
○ UNC.COMP.
⊡ VANE SHEAR (PK)
× VANE SHEAR (REM)
⊕ TRIAXIAL (UU)
SHEAR STRENGTH (KSF) 1 2 3 4

ELEVATION (FEET)	DEPTH (FEET)	SURFACE ELEVATION: 881.5 FT PROFILE DESCRIPTION	SAMPLE TYPE/NO. INTERVAL	RECOVERY LENGTH (INCHES)	BLOWS PER SIX INCHES	N-VALUE -- ○	MOISTURE & ATTERBERG LIMITS (%)	SHEAR STRENGTH (KSF)	REMARKS
880.6	0.9	11 inches of TOPSOIL							
870.5	11.0	Sandy LEAN CLAY- Occasional Roots- Occasional Wet Sand Seams from 5 feet to 8 feet- Brown- Hard to Stiff (CL)	SB1	16	2 4 6	10	9	4.5+	
			SB2	18	6 8 9	17	11	4.5+	
			SB3	18	4 6 7	13	12	▽	
			SB4	18	6 6 11	17	12	4.5+	
868.0	13.5	Sandy LEAN CLAY- Gray- Stiff (CL)	SB5	18	3 4 4	8	13	▽	
866.0	15.5	Fine to Medium SILTY CLAYEY SAND- Brown- Wet- Very Loose (SM-SC)	SB6	18	1 1 2	3			
858.0	23.5	Sandy SILT- Little Clay- Brown to Gray at 18 feet- Wet- Medium Dense (ML)	SB7	18	3 6 8	14			
			SB8	18	4 5 7	12			
856.0	25.5	Sandy LEAN CLAY- Gray- Stiff (CL)	SB9	18	4 7 6	13	12	▽	
	30	Fine to Coarse SAND- Grayish Brown- Wet- Medium Dense to Very Dense (SP)	SB10	12	10 35 16	51			

GROUNDWATER & BACKFILL INFORMATION

	DEPTH (FT)	ELEV (FT)
▽ **DURING BORING:**	13.5	868.0
▼ **AT END OF BORING:**	27.0	854.5

BACKFILL METHOD: 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'

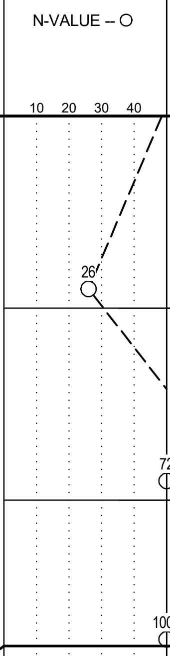
(Continued Next Page)

BORING ECT-18-B01

PROJECT NAME: LBWL New Gas Combined Cycle Plant
PROJECT NUMBER: 079295.00
CLIENT: Lansing Board of Water & Light
PROJECT LOCATION: Delta Township, Michigan

SURFACE ELEVATION: 881.5 FT

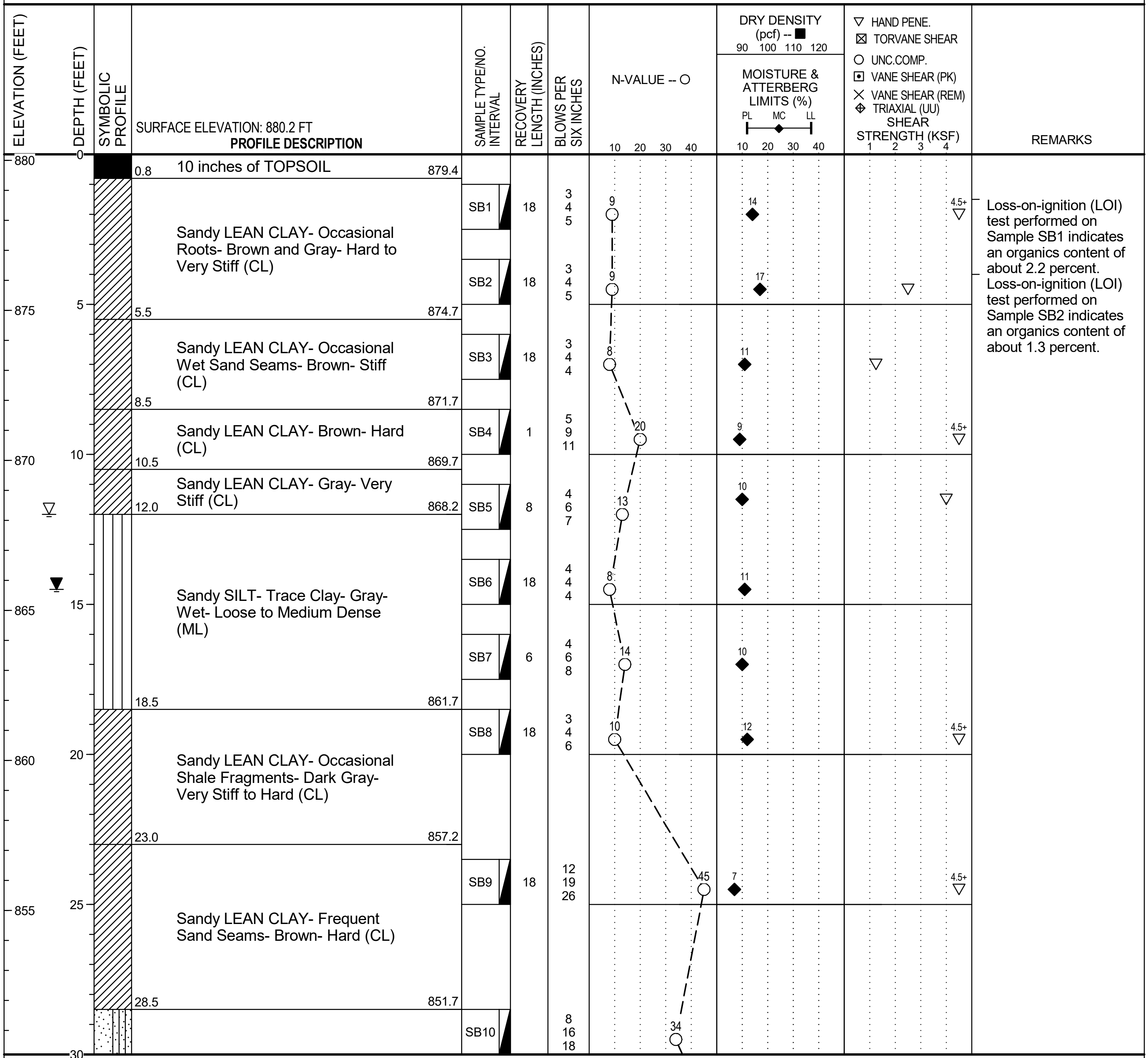
ELEVATION (FEET)	DEPTH (FEET)	PROFILE DESCRIPTION	SAMPLE TYPE/NO. INTERVAL	RECOVERY LENGTH (INCHES)	BLOWS PER SIX INCHES	N-VALUE -- ○	DRY DENSITY (pcf) -- ■ MOISTURE & ATTERBERG LIMITS (%) PL MC LL	HAND PENE. ▽ TORVANE SHEAR ☒ UNC.COMP. ○ VANE SHEAR (PK) ▣ VANE SHEAR (REM) × TRIAXIAL (UU) ◈ SHEAR STRENGTH (KSF)	REMARKS
	30	Fine to Coarse SAND- Grayish Brown- Wet- Medium Dense to Very Dense (SP) *(continued)*							
			SB11	12	8 12 14	26			
			SB12	6	8 17 55	72			
									Driller reported hard drilling from 41.0 feet to 43.5 feet.
837.7	43.8		SB13	0	100/3"	100+			Driller reported no recovery for Sample SB13.
		END OF BORING AT 43.8 FEET.							



BORING ECT-18-B02

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: 6/29/18
COMPLETED: 6/29/18
BORING METHOD: Hollow-stem Augers
DRILLER: BS (Strata)
RIG NO.: CME 55 - ATV
LOGGED BY: JAR
CHECKED BY: JSW



SURFACE ELEVATION: 880.2 FT

Depth (ft)	Elevation (ft)	PROFILE DESCRIPTION
0.8	879.4	10 inches of TOPSOIL
5.5	874.7	Sandy LEAN CLAY- Occasional Roots- Brown and Gray- Hard to Very Stiff (CL)
8.5	871.7	Sandy LEAN CLAY- Occasional Wet Sand Seams- Brown- Stiff (CL)
10.5	869.7	Sandy LEAN CLAY- Brown- Hard (CL)
12.0	868.2	Sandy LEAN CLAY- Gray- Very Stiff (CL)
18.5	861.7	Sandy SILT- Trace Clay- Gray- Wet- Loose to Medium Dense (ML)
23.0	857.2	Sandy LEAN CLAY- Occasional Shale Fragments- Dark Gray- Very Stiff to Hard (CL)
28.5	851.7	Sandy LEAN CLAY- Frequent Sand Seams- Brown- Hard (CL)

SAMPLE TYPE/NO.	RECOVERY LENGTH (INCHES)	BLOWS PER SIX INCHES	N-VALUE	MOISTURE (%)	SHEAR STRENGTH (KSF)
SB1	18	3 4 5	9	14	4.5+
SB2	18	3 4 5	9	17	
SB3	18	3 4 4	8	11	
SB4	1	5 9 11	20	9	4.5+
SB5	8	4 6 7	13	10	
SB6	18	4 4 4	8	11	
SB7	6	4 6 8	14	10	
SB8	18	3 4 6	10	12	4.5+
SB9	18	12 19 26	45	7	4.5+
SB10		8 16 18	34		

REMARKS

Loss-on-ignition (LOI) test performed on Sample SB1 indicates an organics content of about 2.2 percent.

Loss-on-ignition (LOI) test performed on Sample SB2 indicates an organics content of about 1.3 percent.

GROUNDWATER & BACKFILL INFORMATION

	DEPTH (FT)	ELEV (FT)
▽ **DURING BORING:**	12.0	868.2
▼ **AT END OF BORING:**	14.5	865.7

BACKFILL METHOD: Cement- Bentonite Grout

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

(Continued Next Page)

BORING ECT-18-B02

PAGE 2 OF 2

PROJECT NAME: LBWL New Gas Combined Cycle Plant

PROJECT NUMBER: 079295.00

CLIENT: Lansing Board of Water & Light

PROJECT LOCATION: Delta Township, Michigan

ELEVATION (FEET)	DEPTH (FEET)	SYMBOLIC PROFILE	SURFACE ELEVATION: 880.2 FT **PROFILE DESCRIPTION**	SAMPLE TYPE/NO. INTERVAL	RECOVERY LENGTH (INCHES)	BLOWS PER SIX INCHES	N-VALUE -- ○ 10 20 30 40	DRY DENSITY (pcf) -- ■ 90 100 110 120 MOISTURE & ATTERBERG LIMITS (%) PL MC LL 10 20 30 40	▽ HAND PENE. ☒ TORVANE SHEAR ○ UNC.COMP. ⊡ VANE SHEAR (PK) × VANE SHEAR (REM) ⊕ TRIAXIAL (UU) SHEAR STRENGTH (KSF) 1 2 3 4	REMARKS
850	30		Fine to Coarse SAND with Silt and Gravel- Limestone pieces at 33.5 feet- Brown and Gray- Wet- Medium Dense (SP-SM) *(continued)*							
846.6	33.6		END OF BORING AT 33.6 FEET.	SB11	1	50/1"	50+			
845	35									
840	40									
835	45									
830	50									
825	55									
820	60									
815	65									
	70									

BORING ECT-18-B03

PAGE 1 OF 2

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

SURFACE ELEVATION: 879 FT

DEPTH (FEET)	ELEVATION (FEET)	PROFILE DESCRIPTION
0.8	878.2	9 inches of TOPSOIL
8.5	870.5	Sandy LEAN CLAY- Brown and Gray to Brown- Very Stiff to Stiff (CL)
11.5	867.5	Sandy LEAN CLAY- Gray- Stiff (CL)
16.0	863.0	Sandy SILT- Little Clay- Occasional Clay Layers- Gray- Wet- Very Loose (ML)
23.5	855.5	Sandy LEAN CLAY- Dark Gray- Stiff (CL)
28.5	850.5	Fine to Coarse SAND with SIlt- Brown- Wet- Loose (SP-SM)
		Sandy SILT- Limestone pieces at 33.5 feet- Gray- Wet- Medium Dense (ML)

SAMPLE TYPE/NO. INTERVAL	RECOVERY LENGTH (INCHES)	BLOWS PER SIX INCHES	N-VALUE	MOISTURE (%)	SHEAR STRENGTH (KSF)	REMARKS
SB1	18	4 5 5	10	11		
SB2	18	4 4 5	9	12		
SB3	18	4 4 6	10	14		
SB4	18	3 4 4	8	13		
SB5	18	3 2 2	4			
SB6	18	0 1 2	3	12	0.7 ☒	Shear strength test performed on clay layer.
SB7	18	2 2 3	5	12	1.2 ☒	
SB8	18	2 3 3	6	11		
SB9	16	4 3 3	6			
SB10	12	5 12 15	27			Driller reported cobbles from 28.5 feet to 32.0 feet.

DRY DENSITY (pcf) -- ■ 90 100 110 120
MOISTURE & ATTERBERG LIMITS (%) PL MC LL 10 20 30 40
N-VALUE -- ○ 10 20 30 40
▽ HAND PENE.
☒ TORVANE SHEAR
○ UNC.COMP.
⊡ VANE SHEAR (PK)
× VANE SHEAR (REM)
◈ TRIAXIAL (UU)
SHEAR STRENGTH (KSF) 1 2 3 4

GROUNDWATER & BACKFILL INFORMATION

	DEPTH (FT)	ELEV (FT)
▽ **DURING BORING:**	11.5	867.5
▼ **AT END OF BORING:**	11.0	868.0

BACKFILL METHOD: 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'

(Continued Next Page)

BORING ECT-18-B03

PROJECT NAME: LBWL New Gas Combined Cycle Plant **PROJECT NUMBER:** 079295.00

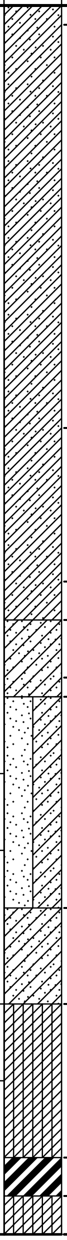
CLIENT: Lansing Board of Water & Light **PROJECT LOCATION:** Delta Township, Michigan

ELEVATION (FEET)	DEPTH (FEET)	SYMBOLIC PROFILE	SURFACE ELEVATION: 879 FT PROFILE DESCRIPTION	SAMPLE TYPE/NO. INTERVAL	RECOVERY LENGTH (INCHES)	BLOWS PER SIX INCHES	N-VALUE -- ○ 10 20 30 40	DRY DENSITY (pcf) -- ■ 90 100 110 120 MOISTURE & ATTERBERG LIMITS (%) PL MC LL 10 20 30 40	▽ HAND PENE. ☒ TORVANE SHEAR ○ UNC.COMP. ▣ VANE SHEAR (PK) ✕ VANE SHEAR (REM) ⊕ TRIAXIAL (UU) SHEAR STRENGTH (KSF) 1 2 3 4	REMARKS
			Sandy SILT- Limestone pieces at 33.5 feet- Gray- Wet- Medium Dense (ML) *(continued)*							Driller reported hard dilling from 32.0 feet to 33.7 feet.
845.3	33.7			SB11	1	50/2"	50+			
			END OF BORING AT 33.7 FEET.							
845										
	35									
840										
	40									
835										
	45									
830										
	50									
825										
	55									
820										
	60									
815										
	65									
810										
	70									

MW-1

PAGE 1 OF 1

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 11:00 **COMPLETED** 10/15/19 12:30
GROUND ELEVATION 885.97 ft MSL **HOLE DIAMETER** 7"
DRILLING CONTRACTOR SME **DRILLER** Rudy Musulin
GROUND WATER LEVELS:
DRILLING METHOD HSA **EQUIPMENT** Track-Mounted CME 55
▽ **AT TIME OF DRILLING** 17.50 ft / Elev 868.47 ft
LOGGED BY Emily Munoz **CHECKED BY**
▼ **75 HRS AFTER DRILLING** 11.85 ft / Elev 874.12 ft
NOTES Sample ID prefix LBWL-MW1-. Driller recorded blow counts on SME logs.

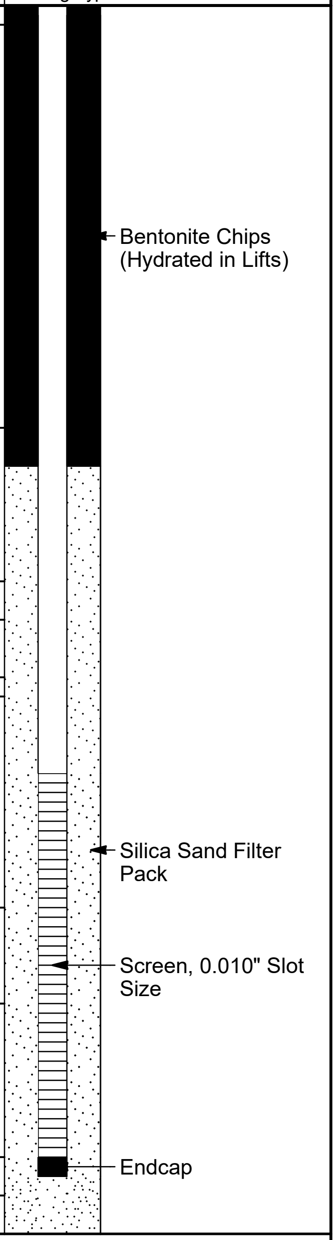


DEPTH (ft)	SAMPLE TYPE NUMBER	MATERIAL DESCRIPTION	
0.5		SANDY LEAN CLAY WITH GRAVEL, (CL) brown (10YR 5/3), dry, stiff, low plasticity	885.5
11.0		SANDY LEAN CLAY WITH GRAVEL, (CL) yellowish brown (10YR 5/4), dry, medium stiff, mottled, low plasticity	875.0
15.0		▼ SANDY LEAN CLAY WITH GRAVEL, (CL) yellowish brown (10YR 5/4), moist, medium stiff, mottled, low plasticity	871.0
16.0		SANDY LEAN CLAY, (CL) very dark gray (2.5Y 3/1), moist, stiff, low plasticity	870.0
17.5		CLAYEY SAND, (SC) dark greenish gray (10GY 4/1), poorly graded, fine grained, moist, medium dense, iron oxide staining	868.5
18.0		▽ CLAYEY SAND, (SC) dark greenish gray (10GY 4/1), poorly graded, fine grained, wet, medium dense, iron oxide staining	868.0
23.5	SS 20-22	POORLY GRADED SAND WITH CLAY, (SP) gray (5Y 5/1), fine to medium grained, wet, medium dense	862.5
26.0		CLAYEY SAND, (SC) gray (5Y 5/1), poorly graded, fine grained, wet, medium dense	860.0
30.0	SS 26-28	LEAN CLAY WITH SAND, SILTY, (CL) gray (5Y 5/1), fine grained, wet, soft, low plasticity	856.0
31.0		FAT CLAY, (CH) gray (5Y 5/1), wet, stiff, medium plasticity	855.0
32.0		LEAN CLAY WITH SAND, SILTY, (CL) gray (5Y 5/1), fine to medium grained, wet, soft, low plasticity	854.0

Bottom of borehole at 32.0 feet.

WELL DIAGRAM

Casing Top Elev: 888.74 (ft)
Casing Type: 2" Sch 40 PVC



MW-3

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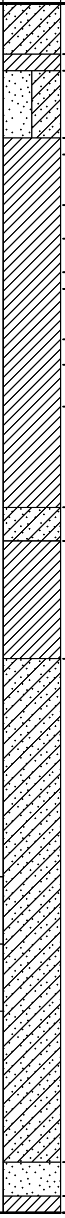
CLIENT Lansing Board of Water and Light
PROJECT NAME LBWL Confidential
PROJECT NUMBER 10173187
PROJECT LOCATION Erickson Power Station, Lansing, MI
DATE STARTED 10/15/19 10:36 **COMPLETED** 10/15/19 12:30
GROUND ELEVATION 885.12 ft MSL **HOLE DIAMETER** 8"
DRILLING CONTRACTOR SME **DRILLER** Derek Blackburn
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.

DEPTH (ft) | SAMPLE TYPE NUMBER | GRAPHIC LOG | MATERIAL DESCRIPTION | WELL DIAGRAM

Casing Top Elev: 884.81 (ft)
Casing Type: 2" Sch 40 PVC



CLAYEY SAND, (SC) brown (10YR 4/3), poorly graded, fine grained, moist, dense

LEAN CLAY, (CL) grayish brown (10YR 5/2), moist, medium stiff, low plasticity, sand

POORLY GRADED SAND, (SP) brown (10YR 4/3), fine to medium grained, moist, dense, clay

LEAN CLAY, SILTY, (CL) very dark grayish brown (2.5Y 3/2), moist, soft, low plasticity, sand

LEAN CLAY, (CL) yellowish brown (10YR 5/4), moist, medium stiff, mottled, low plasticity

LEAN CLAY, (CL) very dark grayish brown (2.5Y 3/2), moist, medium stiff, low plasticity, sand

LEAN CLAY, (CL) yellowish brown (10YR 5/4), moist, stiff, mottled, low plasticity, gravel

LEAN CLAY, SANDY, (CL) very dark grayish brown (2.5Y 3/2), moist, soft, low plasticity

LEAN CLAY, (CL) yellowish brown (10YR 5/4), moist, soft, mottled, low plasticity, sand

LEAN CLAY, SANDY, (CL) very dark grayish brown (2.5Y 3/2), moist, soft, low plasticity

LEAN CLAY, SANDY, (CL) yellowish brown (10YR 5/4), moist, soft, low plasticity, fine gravel

LEAN CLAY, (CL) yellowish brown (10YR 5/4), moist, medium stiff, low plasticity, sand, gravel

CLAYEY SAND, (SC) yellowish brown (10YR 5/4), poorly graded, fine grained, wet, loose

LEAN CLAY, SANDY, (CL) very dark grayish brown (10YR 3/2), wet, soft, low plasticity

CLAYEY SAND, (SC) very dark grayish brown (10YR 3/2), poorly graded, fine grained, wet, loose, gravel

POORLY GRADED SAND WITH CLAY, (SP) very dark grayish brown (10YR 3/2), fine to medium grained, wet, loose

LEAN CLAY, SILTY, (CL) very dark grayish brown (10YR 3/2), moist, soft, low plasticity, fine sand, Stiff, plastic fat clay (CH) in shoe.

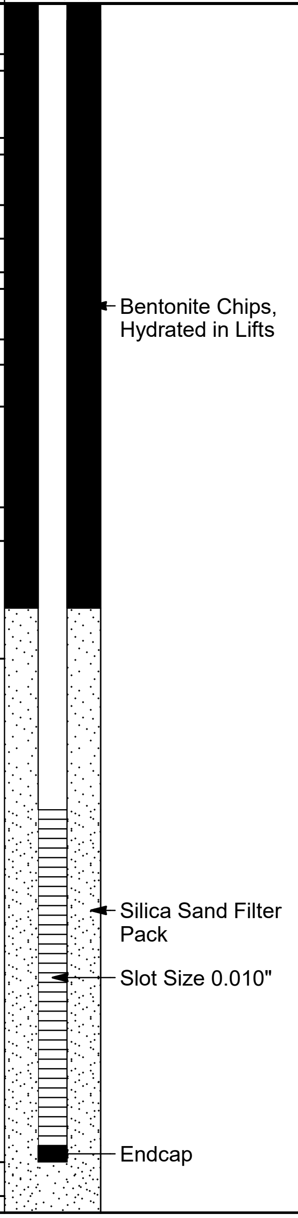
Bottom of borehole at 36.0 feet.

1.5	883.6
2.0	883.1
4.0	881.1
4.5	880.6
6.0	879.1
7.0	878.1
8.0	877.1
8.5	876.6
10.0	875.1
10.8	874.4
12.0	873.1
15.0	870.1
16.0	869.1
19.5	865.6
34.5	850.6
35.5	849.6
36.0	849.1

Depth scale: 0, 5, 10, 15, 20, 25, 30, 35

SS 26-28

SS 28-30



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PAGE 1 OF 1

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

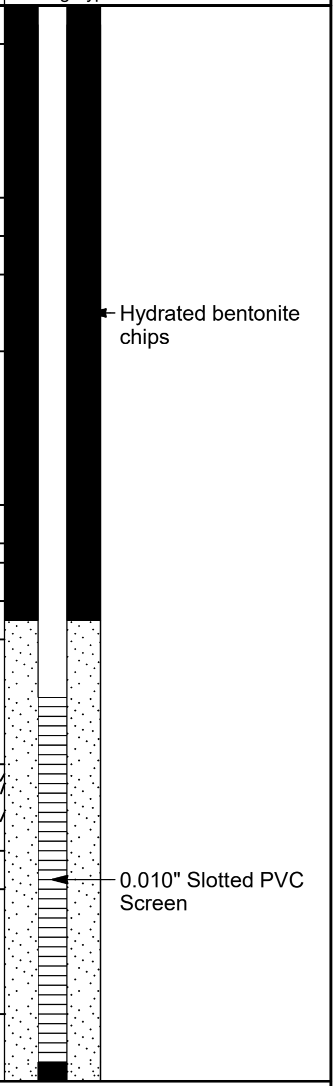
DEPTH (ft)	MATERIAL DESCRIPTION	
1.0	LEAN CLAY, SILTY, (CL) very dark brown (7.5YR 2.5/2), moist, soft, low plasticity, fine sand	884.2
5.0	LEAN CLAY, SILTY, (CL) brown (10YR 4/3), moist, soft, low plasticity	880.2
6.0	LEAN CLAY, SILTY, (CL) dark brown (7.5YR 3/2), moist, soft, low plasticity, fine sand	879.2
7.0	LEAN CLAY, SILTY, (CL) brown with dark brown (10YR 5/3), moist, medium stiff, mottled, low plasticity, fine sand, fine gravel	878.2
9.0	LEAN CLAY, SILTY, (CL) dark yellowish brown with dark grayish brown (10YR 4/6), moist, soft, mottled, low plasticity, fine sand, fine gravel	876.2
13.0	LEAN CLAY, SILTY, (CL) yellowish brown (10YR 5/4), moist, soft, medium plasticity, fine sand, fine gravel	872.2
14.0	LEAN CLAY, SILTY, (CL) yellowish brown (10YR 5/4), wet, soft, medium plasticity, fine sand, fine gravel	871.2
14.5	WELL GRADED SAND WITH GRAVEL, (SW) brown (10YR 4/3), fine to coarse grained, wet, loose	870.7
15.5	LEAN CLAY, SILTY, (CL) yellowish brown (10YR 5/4), wet, stiff, medium plasticity, fine sand, fine gravel	869.7
16.5	CLAYEY SAND, (SP) yellowish brown (10YR 5/4), fine grained, wet, loose, fine gravel	868.7
19.8	LEAN CLAY, (CL) brown (7.5YR 4/2), wet, medium stiff, low plasticity, fine sand, fine gravel	865.5
20.0	CLAYEY SAND, (SP) brown (7.5YR 5/2), fine to coarse grained, wet, loose, fine gravel	865.2
20.3	CLAYEY SAND, (SP) brown (7.5YR 5/2), fine grained, wet, loose	865.0
21.0	LEAN CLAY, (CL) brown (7.5YR 5/2), wet, soft, low plasticity, fine sand	864.2
22.0	POORLY GRADED SAND, (SP) dark gray (7.5YR 4/1), coarse grained, wet, loose, fine gravel	863.2
23.0	LEAN CLAY, (CL) gray (7.5YR 5/1), moist, stiff, low plasticity, fine sand, fine gravel	862.2
26.3	LEAN CLAY, (CL) brown (7.5YR 5/2), wet, stiff, low plasticity, fine sand	859.0
28.0	LEAN CLAY, SANDY, (CL) dark gray to black (7.5YR 4/1), wet, medium stiff, low plasticity	857.2

SAMPLE TYPE NUMBER: GB

Bottom of borehole at 28.0 feet.

WELL DIAGRAM

Casing Top Elev: 889.15 (ft)
Casing Type: 2" Sch 40 PVC



HDR

MW-14

PAGE 1 OF 1

CLIENT Lansing Board of Water & Light

PROJECT NAME Erickson Power Station

PROJECT NUMBER 10173187

PROJECT LOCATION Eaton County, MI

DATE STARTED 01/09/23 08:00 **COMPLETED** 01/09/23 13:00

GROUND ELEVATION 885.028 ft MSL **HOLE DIAMETER** 6"

DRILLING CONTRACTOR Cascade **DRILLER**

GROUND WATER LEVELS:

DRILLING METHOD Sonic **EQUIPMENT**

▽ **AT TIME OF DRILLING** 19.00 ft / Elev 866.03 ft

LOGGED BY Tanten Buszka **CHECKED BY**

▼ **AFTER DRILLING** 15.48 ft / Elev 869.55 ft

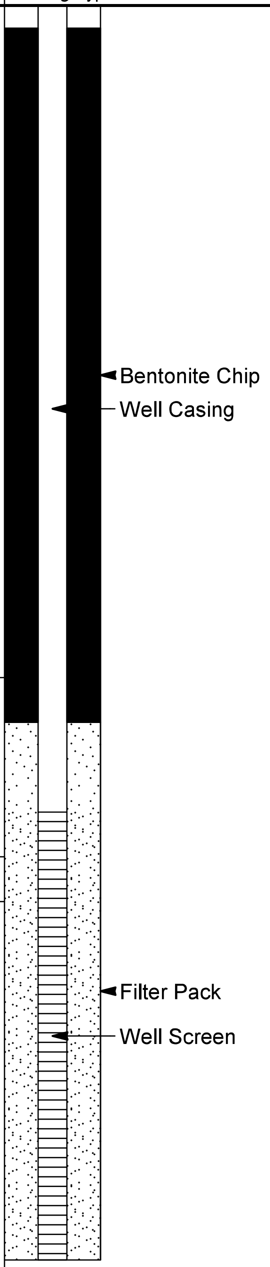
NOTES

DEPTH (ft)	SAMPLE TYPE NUMBER	GRAPHIC LOG		MATERIAL DESCRIPTION	
0		▨		SANDY LEAN CLAY, (CL) brown (10YR 5/4), poorly graded, fine to medium grained, dry, stiff, low plasticity, Gravel noted at 10 - 15'	
15		▨	15.0	▼ SANDY LEAN CLAY, (CL) brown (10YR 5/4), poorly graded, fine to medium grained, dry, loose, high plasticity	870.0
		░	19.0	▽ POORLY GRADED SAND, (SP) gray (GLEY 1 5/N), poorly graded, fine to medium grained, wet	866.0
20		▨	20.0	SANDY LEAN CLAY, (CL) gray (GLEY 1 5/2), poorly graded, fine grained, wet, loose, low plasticity, Trace silt	865.0
30			30.0		855.0

Bottom of borehole at 30.0 feet.

WELL DIAGRAM

Casing Top Elev: 884.594 (ft)
Casing Type: 2" Sch 40 PVC

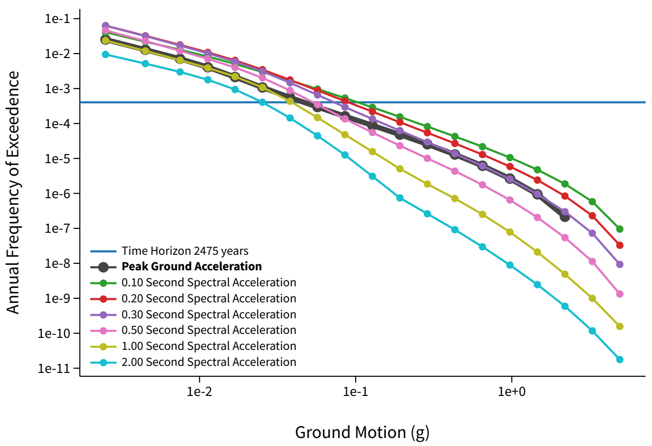


ATTACHMENT 2

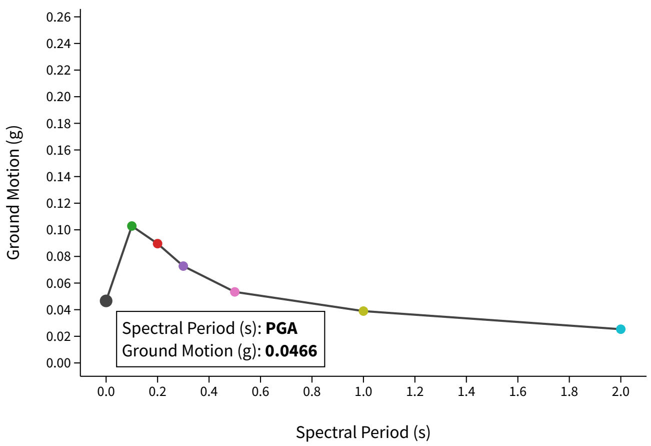
LIQUEFACTION ANALYSIS FIGURES AND RESULTS

Hazard Curve

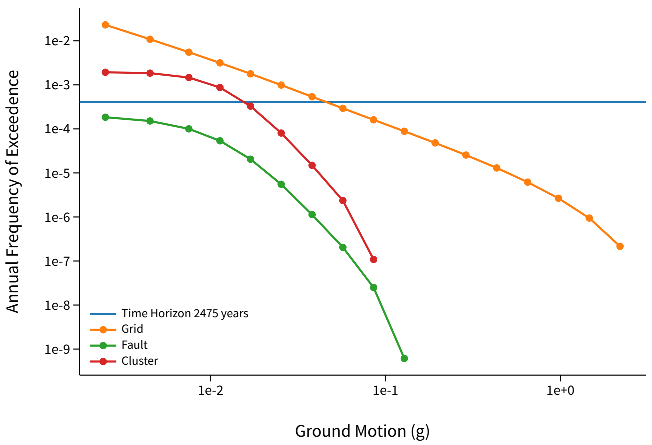
Hazard Curves



Uniform Hazard Response Spectrum



Component Curves for Peak Ground Acceleration

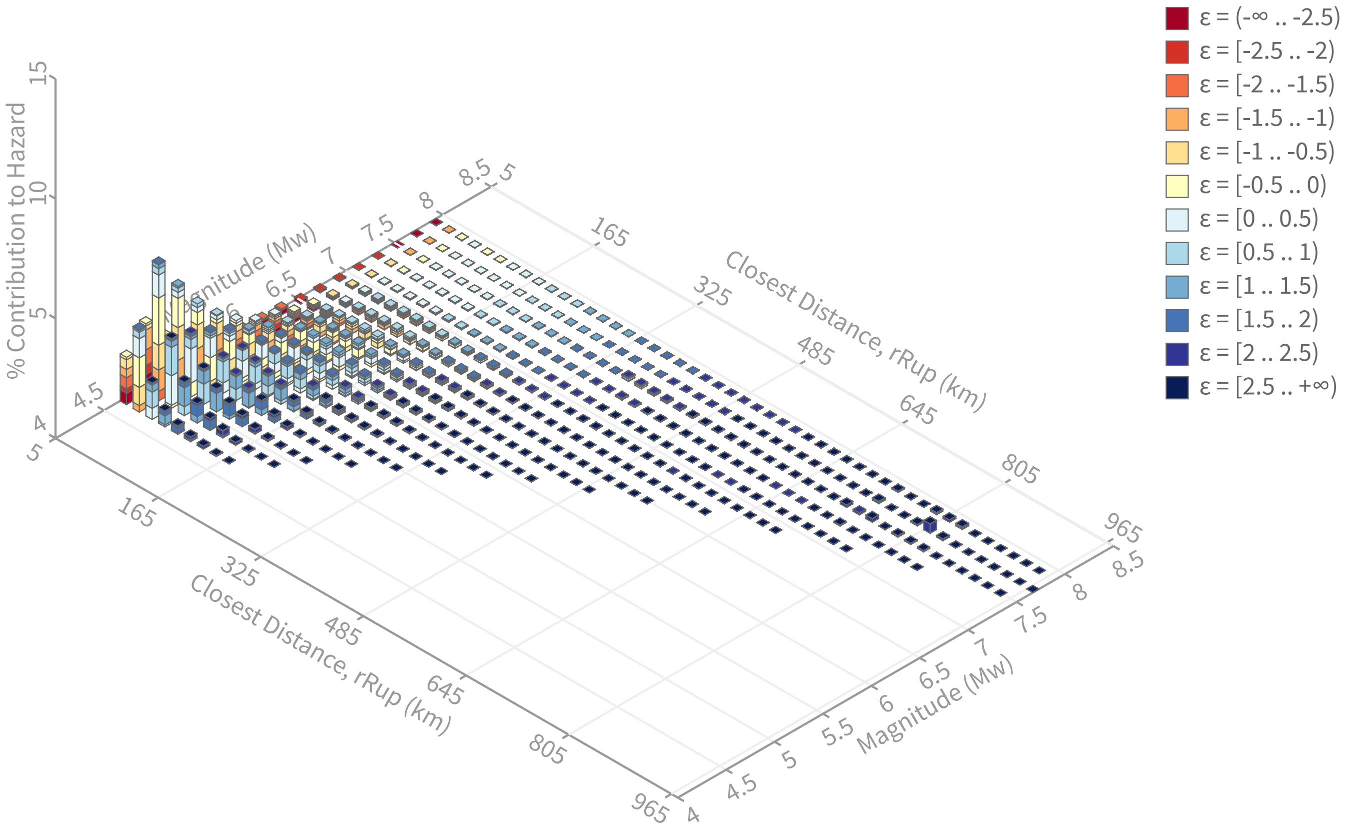


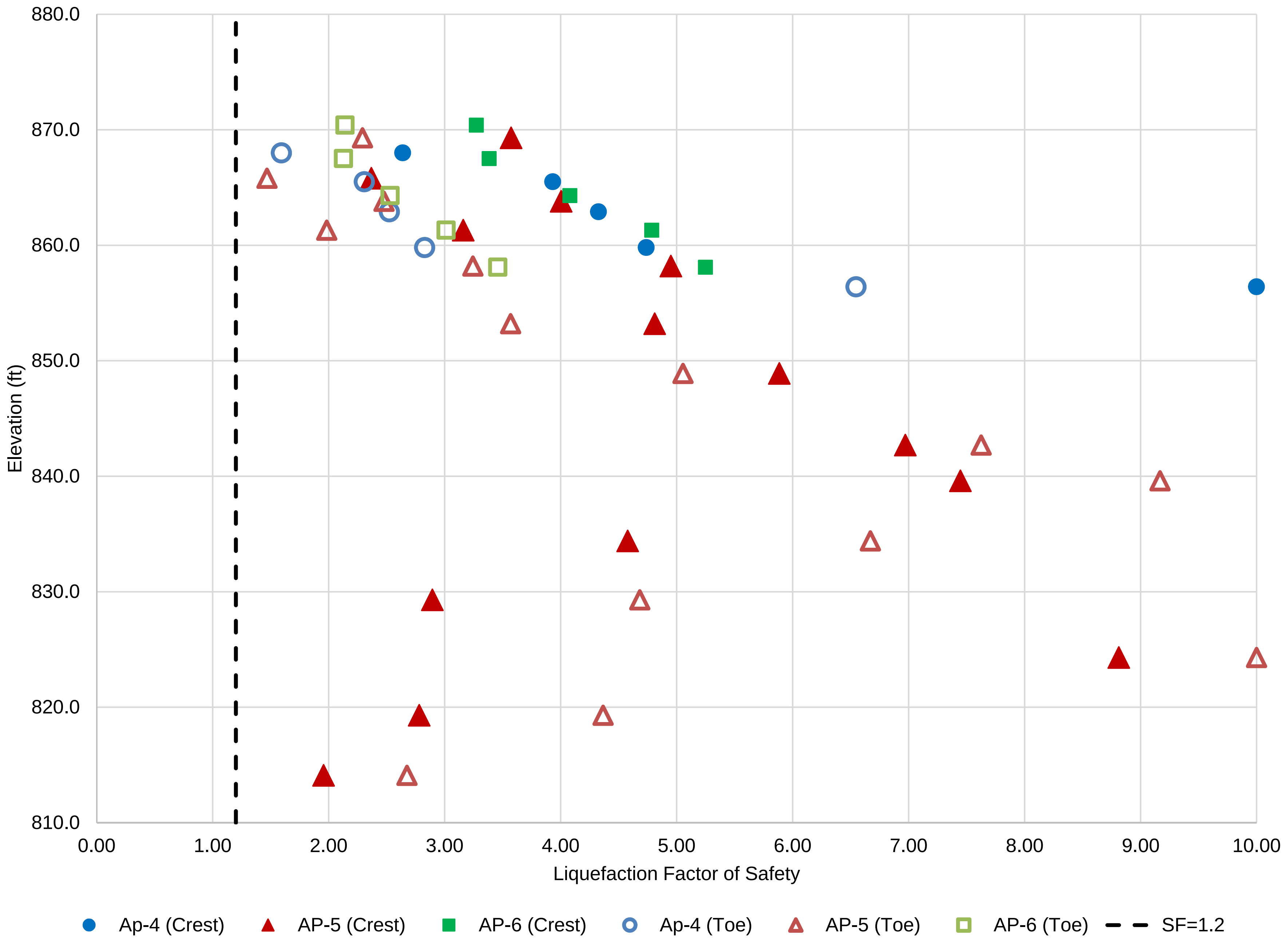
View Raw Data

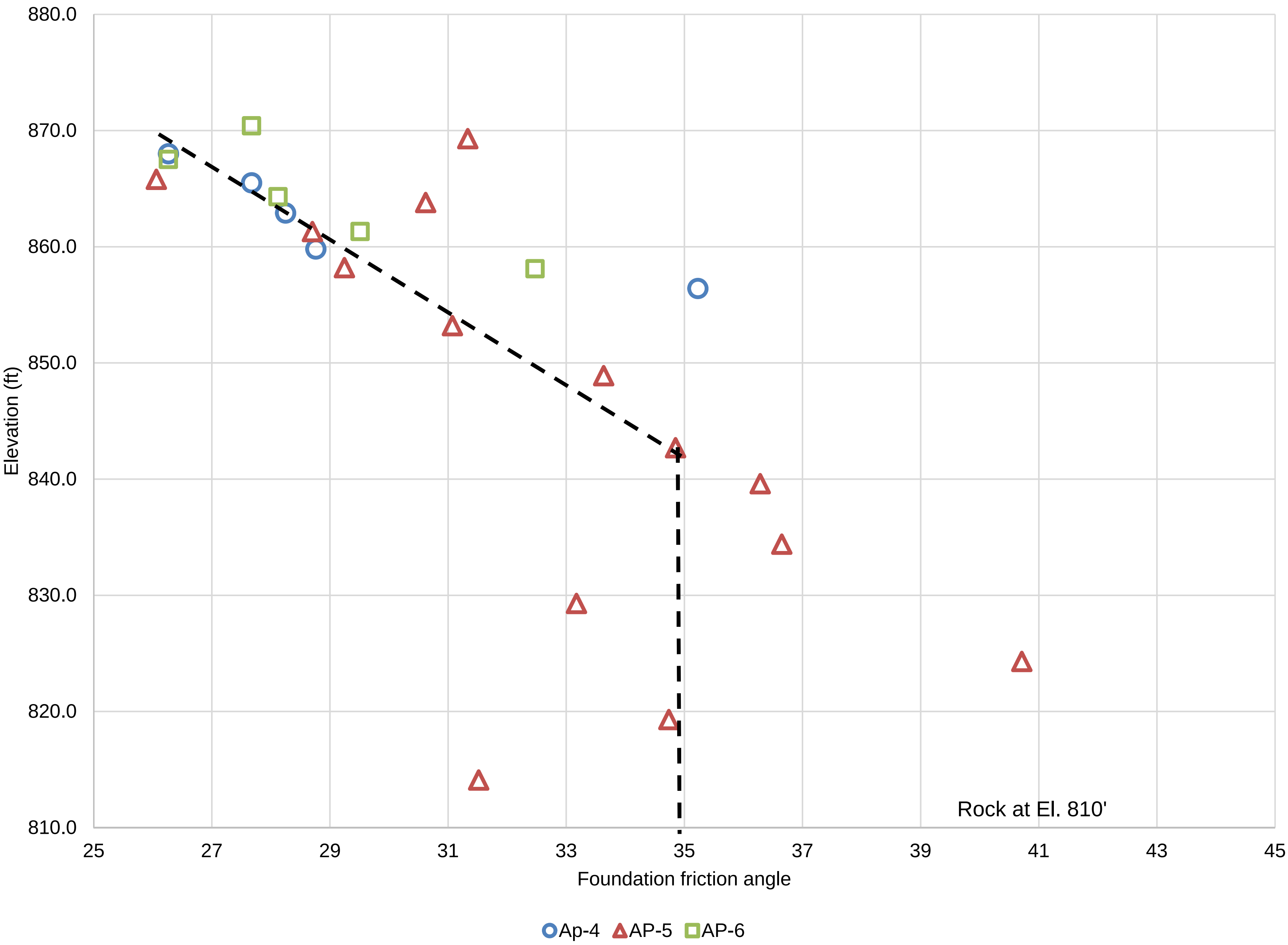
^ Deaggregation

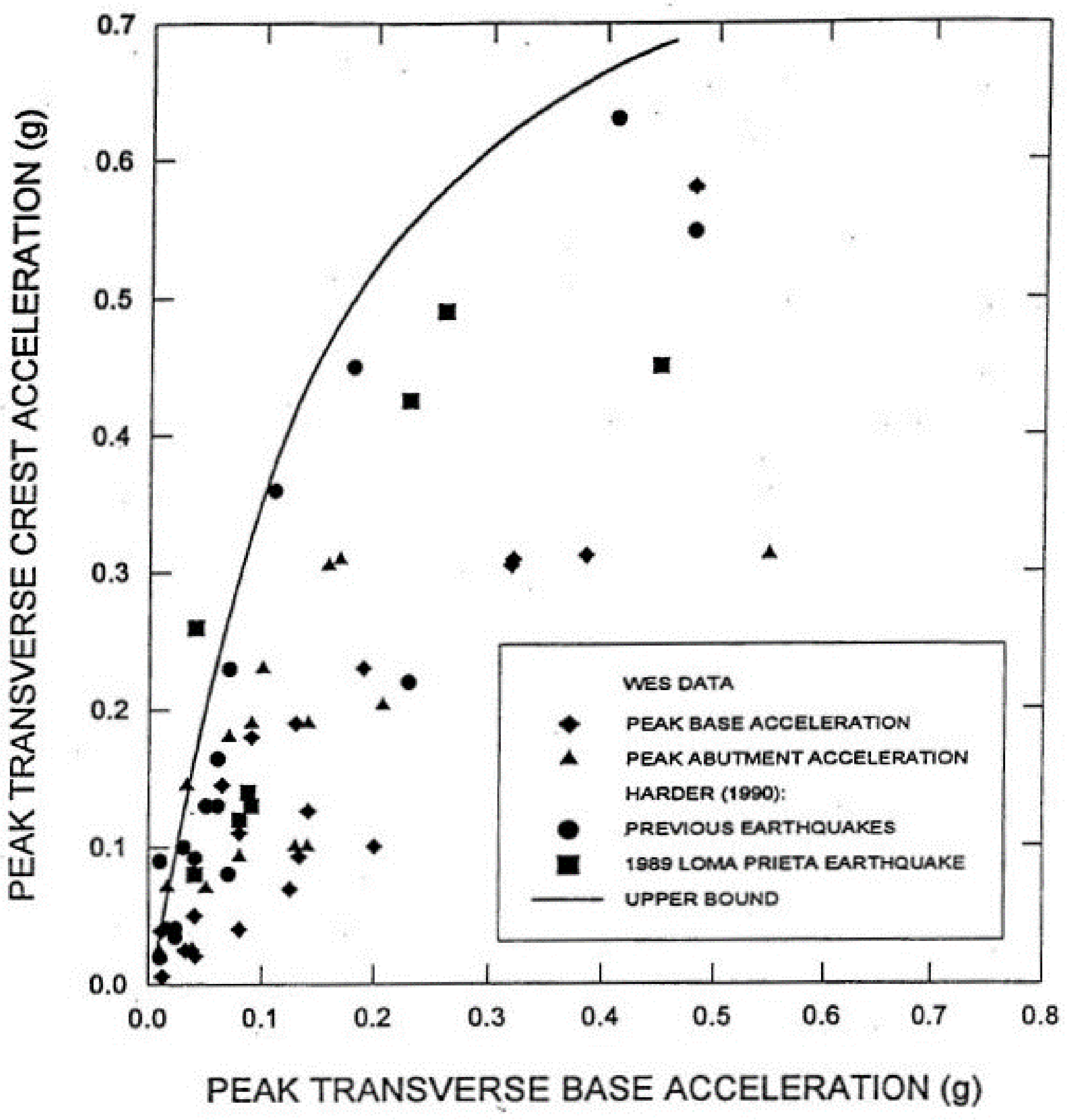
Component

Total



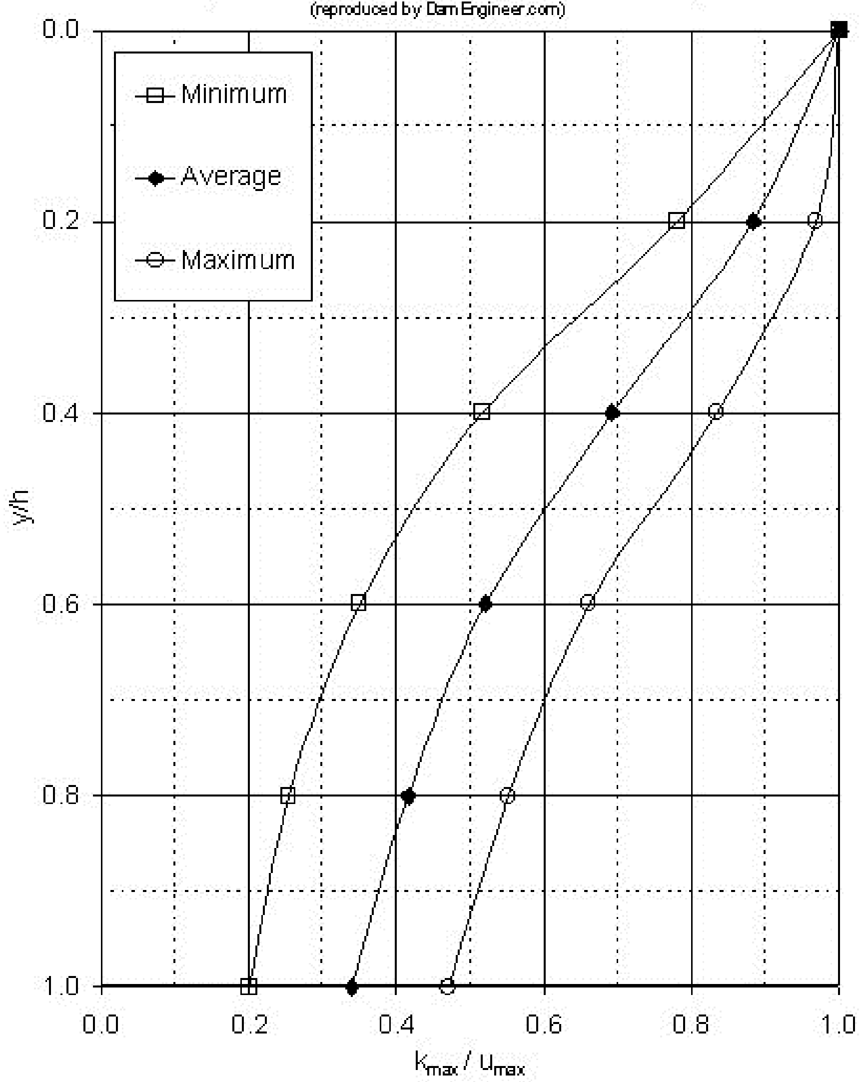

880.0
870.0
860.0
850.0
840.0
830.0
820.0
810.0
Elevation (ft)
0.00
1.00
2.00
3.00
4.00
5.00
6.00
7.00
8.00
9.00
10.00
Liquefaction Factor of Safety
Ap-4 (Crest)
AP-5 (Crest)
AP-6 (Crest)
Ap-4 (Toe)
AP-5 (Toe)
AP-6 (Toe)
SF=1.2

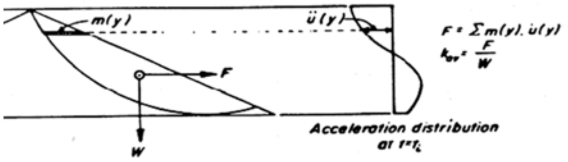

880.0
870.0
860.0
850.0
840.0
830.0
820.0
810.0
Elevation (ft)
25
27
29
31
33
35
37
39
41
43
45
Foundation friction angle
Rock at El. 810'
Ap-4
AP-5
AP-6


PEAK TRANSVERSE CREST ACCELERATION (g)
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
PEAK TRANSVERSE BASE ACCELERATION (g)
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
WES DATA
PEAK BASE ACCELERATION
PEAK ABUTMENT ACCELERATION
HARDER (1990):
PREVIOUS EARTHQUAKES
1989 LOMA PRIETA EARTHQUAKE
UPPER BOUND

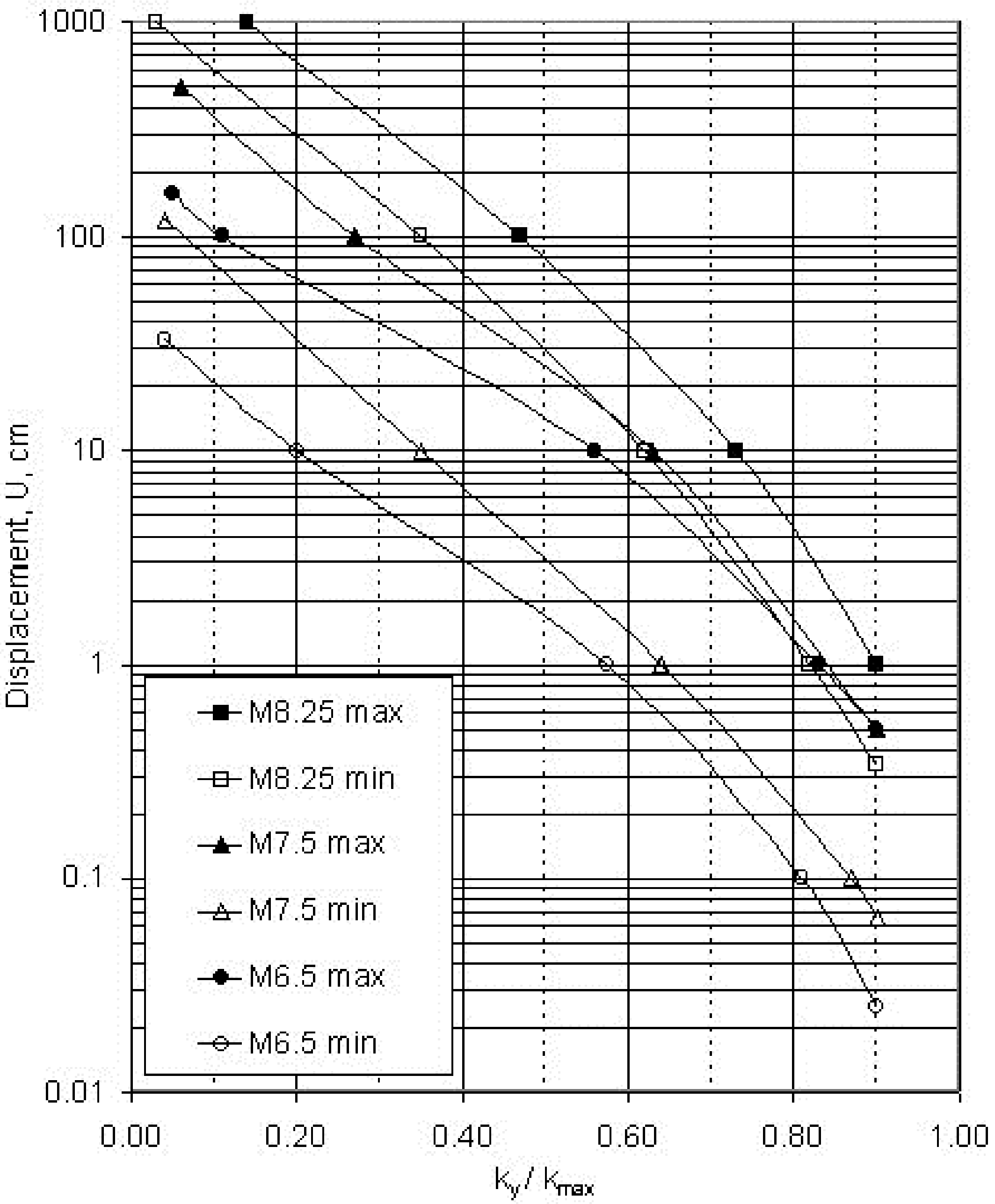
Variation of maximum acceleration ratio with depth of sliding mass (Makdisi-Seed, 1978)

(reproduced by DamEngineer.com)





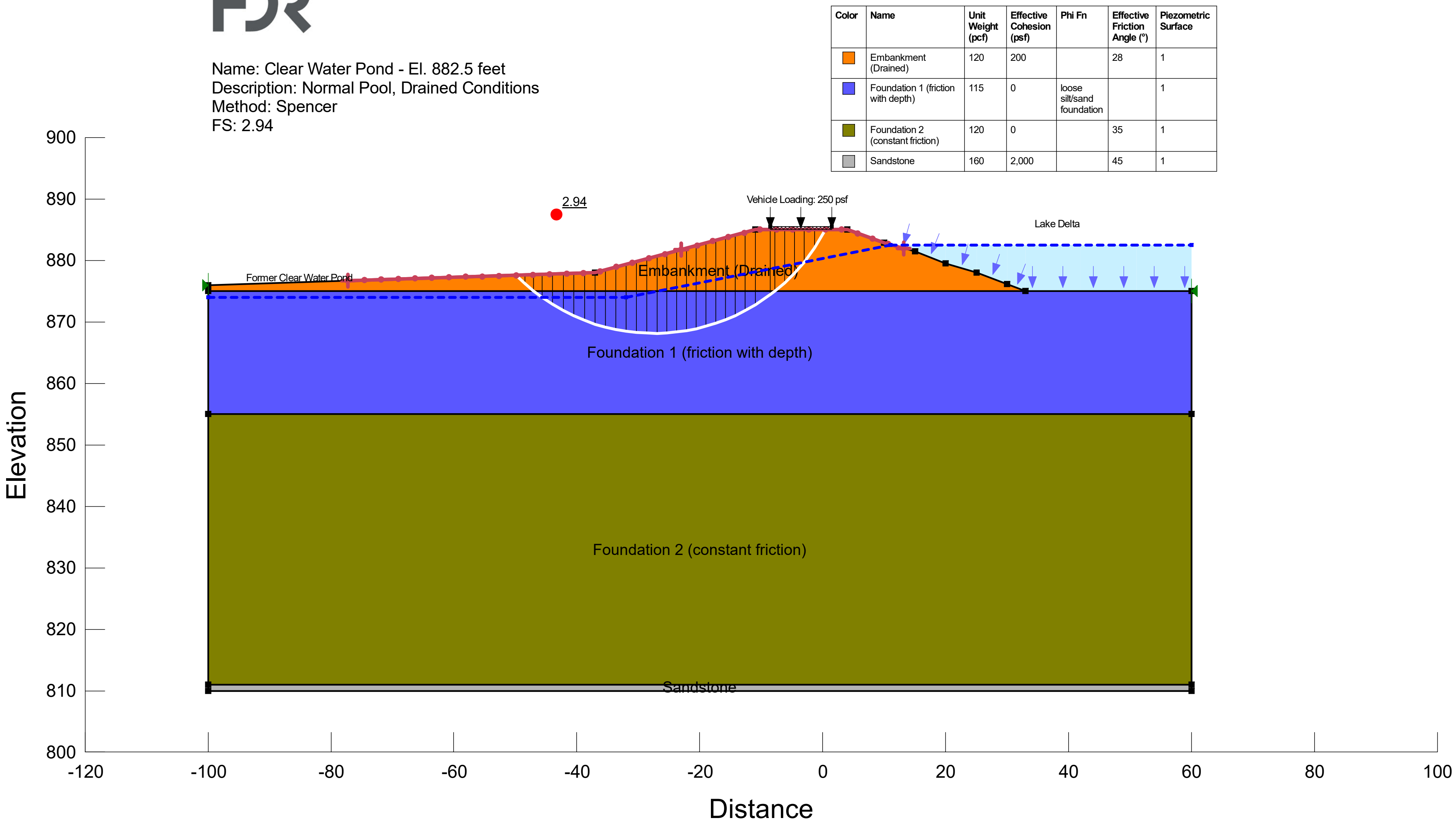
Variation of permanent displacement with
yield acceleration (Makdisi-Seed, 1978)
(reproduced by DamEngineer.com)



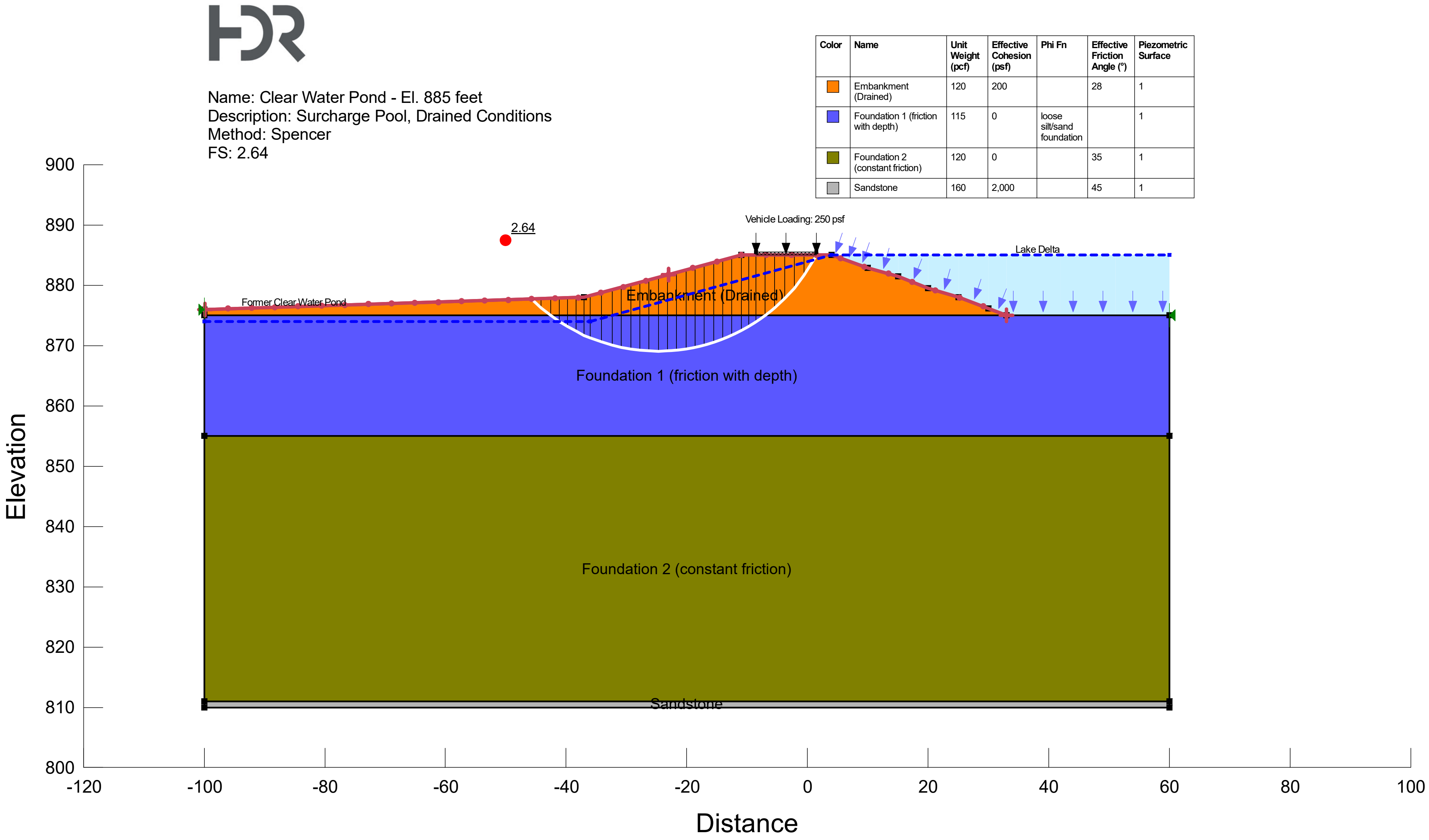
ATTACHMENT 3

STABILITY ANALYSES RESULTS

Name: Clear Water Pond - El. 882.5 feet
Description: Normal Pool, Drained Conditions
Method: Spencer
FS: 2.94



Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Phi Fn	Effective Friction Angle (°)	Piezometric Surface
🟧	Embankment (Drained)	120	200		28	1
🟦	Foundation 1 (friction with depth)	115	0	loose silt/sand foundation		1
🟫	Foundation 2 (constant friction)	120	0		35	1
⬜	Sandstone	160	2,000		45	1



Name: Clear Water Pond - El. 885 feet
Description: Surcharge Pool, Drained Conditions
Method: Spencer
FS: 2.64

Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Phi Fn	Effective Friction Angle (°)	Piezometric Surface
🟧	Embankment (Drained)	120	200		28	1
🟦	Foundation 1 (friction with depth)	115	0	loose silt/sand foundation		1
🟫	Foundation 2 (constant friction)	120	0		35	1
⬜	Sandstone	160	2,000		45	1

Name: Earthquake
Description: Normal Pool, Undrained Conditions
Method: Spencer
FS: 2.25

Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Phi Fn	Effective Friction Angle (°)	Piezometric Surface
🟥	Embankment (Undrained)	120	1,000		0	1
🟦	Foundation 1 (friction with depth)	115	0	loose silt/sand foundation		1
🟫	Foundation 2 (constant friction)	120	0		35	1
⬜	Sandstone	160	2,000		45	1