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 – Clear Water Pond

June 12, 2020

Prepared for:
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1 Introduction and Purpose

HDR MICHIGAN, Inc. (HDR) has prepared this Structural Stability and Safety Factor Assessment Report for the Clear Water Pond 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 existing Clear Water Pond at the Erickson Power Station meets 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 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. 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.

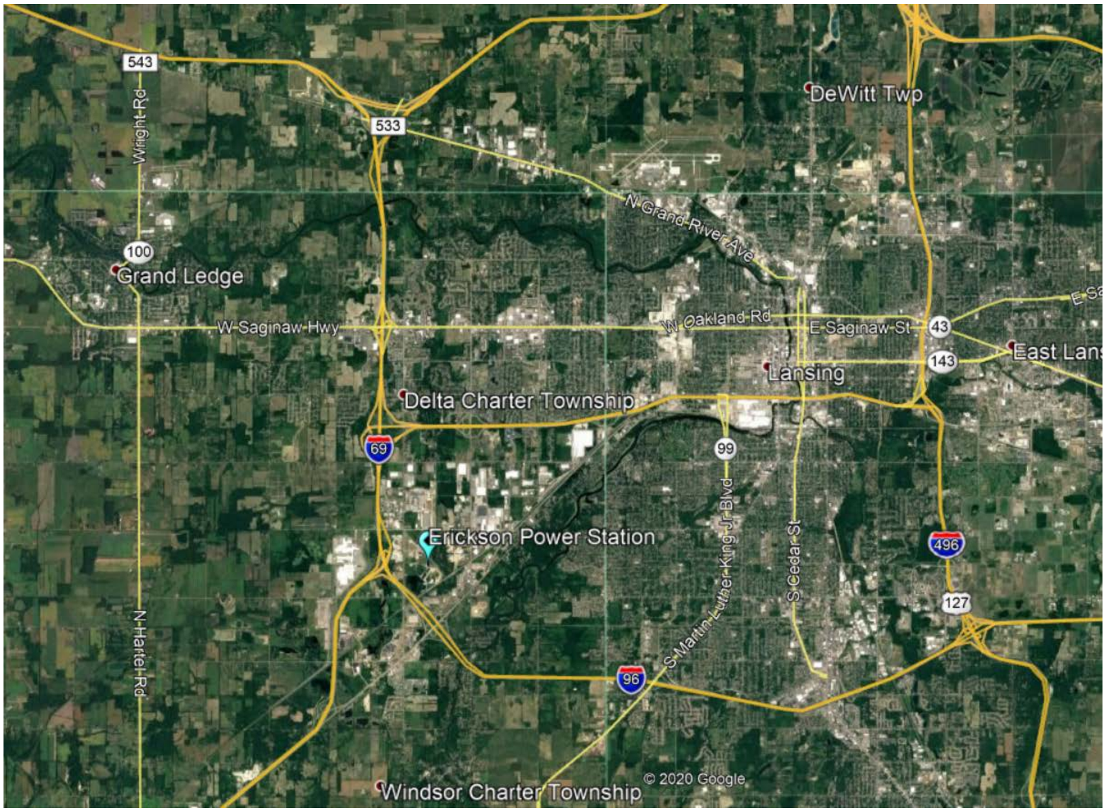


Figure 1. Site Vicinity Map

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. [9]) 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 Clear Water Pond was constructed to provide a storage basin for water prior to recycling it back to Erickson Power Station via the Pump House located on the northwest corner of Clear Water Pond. The Clear Water Pond has a surface area (including top of dike) of approximately 4.7 acres. During normal operating conditions, the water flows between the station, the impoundments, the Clear Water Pond, and back to the station. The Clear Water Pond has a normal operating pool level of approximately El. 881.7 to El. 882.0 feet (NAVD 88[1]).

There is one overflow associated with the impoundment system, which is the Emergency Overflow Structure located in the Clear Water Pond. The overflow structure consists of a 36-inch ductile iron pipe set at El. 883.0 feet NAVD 88. In the event of an emergency overflow, water would enter the overflow structure which discharges to a swale that directs flow north to Carrier Creek, then north to Holly Drain, then to Clements Underhill Drain, and ultimately to the Grand River.

Figure 2 displays the Erickson Power Station site configuration, including the current impoundment system.

Figure 2. Erickson Power Station Site Configuration

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

[1] North American Vertical Datum of 1988

Figure 3. Google Earth Image of Impoundment System

The Clear Water Pond has five hydraulic structures that extend through the embankment:

- Lake Delta Drainage Structure
- Lake Delta Transfer Structure
- Old Ash Impoundment Transfer Structure
- Old Ash Impoundment Drainage Structure
- Emergency Overflow Structure

Figure 4 (Ref. [11]) displays a plan view of the Clear Water Pond with the locations of the associated hydraulic structures and pipes extending through the embankment. Note that the elevations presented in Figure 4 (Ref. [11]) presents survey information referenced to NGVD 29[2] and NAVD 88.

[2] National Geodetic Vertical Datum of 1929

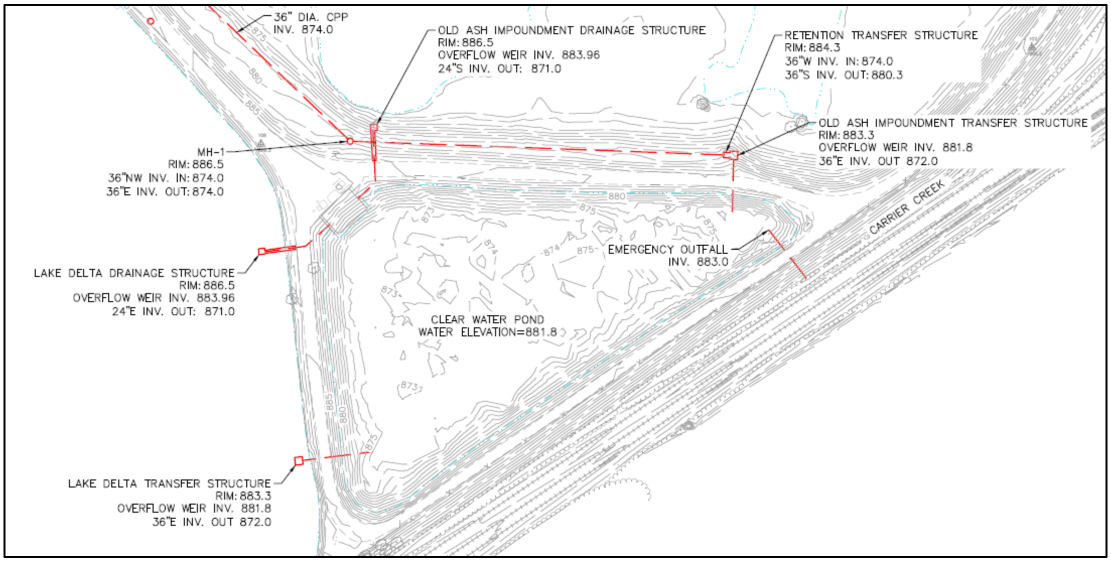


Figure 4. Location of Clear Water Pond Hydraulic Structures

The following provides details of each hydraulic structure located at the Clear Water Pond. It should be noted that elevations presented in this report were provided by a survey performed by BWL on May 7, 2020, along with a review of the existing elevations presented in reports provided by BWL.

Lake Delta Drainage Structure

The Lake Delta Drainage Structure is located between the Clear Water Pond and Lake Delta. Water from Lake Delta flows through the drainage structure (extending through the Clear Water Pond embankment) to the Pump House where it is sent to Erickson Power Station to use. The discharge pipe consists of 24-inch ductile iron pipe, equipped with square, (6-feet x 6-feet) concrete, anti-seep collars. HDR understands that this drainage structure is active and commonly in use.

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 Clear Water Pond and Lake Delta. Water from Lake Delta flows over the overflow weir through the discharge pipe extending through the Clear Water Pond embankment and into the Clear Water Pond. The discharge pipe consists of 36-inch ductile iron pipe, equipped with square, (8-feet x 8-feet) concrete, anti-seep collars. Stop logs are in place to the top of the overflow weir, preventing hydraulic connection between Lake Delta and the Clear Water Pond.

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.

Old Ash Impoundment Transfer Structure

The Old Ash Impoundment Transfer Structure is located between the Clear Water Pond and the Former Impoundment. Water from the Retention Basin flows through piping to the Retention Transfer Structure, which is then transferred to the Old Ash Impoundment Transfer Structure which then flows through the pipe extending through the Clear Water

Pond embankment and in to the Clear Water Pond. The piping extending through the Clear Water Pond embankment consists of 36-inch ductile iron pipe, equipped with square, (8-feet x 8-feet) concrete, anti-seep collars. This structure is the only intake water source to the Clear Water Pond.

The invert of the overflow weir is at approximately El. 880.3 feet NAVD 88 and the invert of the outlet is at approximately El. 871.4 feet NAVD 88.

Old Ash Impoundment Drainage Structure

The Old Ash Impoundment Drainage Structure is located between the Clear Water Pond and the Former Impoundment. The Old Ash Impoundment Drainage Structure was designed to transfer water from the Former Impoundment to the Pump House but is now inactive and not in use. The piping extending through the Clear Water Pond embankment consists of 24-inch ductile iron pipe, equipped with square, (8-feet x 8-feet) concrete, anti-seep collars. According to BWL, the valve of the pipe is currently closed.

Emergency Overflow Structure

The Emergency Overflow Structure is located between the Clear Water Pond and the swale adjacent to the railroad. The Emergency Overflow Structure was designed in an overflow event of the Clear Water Pond to allow water to discharge through the pipe extending through the Clear Water Pond embankment and exit into the swale adjacent to the railroad. The pipe consists of 36-inch ductile iron pipe, equipped with square, (8-feet x 8-feet) concrete, anti-seep collars.

The top of the inlet of the Emergency Overflow Structure was repaired by BWL in approximately May 2017 due to corrosion/deterioration of the pipe. The invert of the overflow pipe is at approximately El. 883.0 feet NAVD 88 and the invert of the outlet pipe is at approximately EL. 873.1 feet NAVD 88. The outlet pipe is equipped with fencing to prevent animals from entering and vegetation was maintained around the outlet.

According to BWL, an overflow event has not occurred in the Clear Water Pond and the Emergency Overflow Structure has yet to be used for discharge.

1.3 Previous Assessments and Inspections

A previous assessment was performed by 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 for GZA 2012 on May 19, 2011. The GZA 2012 report includes discussion and details of the Clear Water Pond. An additional inspection of the Former Impoundment was performed in 2009 by Inspecsol Engineering, Inc. as noted in GZA 2012, however, that report was not available for review.

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 Clear Water Pond during previous inspections.

There are no records of previous structural stability assessments or safety factor assessments that have been performed for the Clear Water Pond embankment.

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. [10]). In addition to the 1969 soil borings, geoprobe borings and test pits were performed at the site by MD&E in 2018. In 2019 and 2020, HDR installed six monitoring wells across the site, with one monitoring well (MW-1) being installed through the south embankment of the Clear Water Pond. As part of the previous subsurface investigations, three borings (AP-4 through AP-6), three geoprobe borings (CW-SB-1 through CW-SB-3), and one monitoring well (MW-1) were performed/installed in the vicinity of the Clear Water Pond.

The historical boring logs prepared by Dames & Moore (1969) prior to the construction of the Clear Water Pond indicate that the embankment foundation is 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, installed in 2019, indicates the presence of cohesive layers (Lean and Fat Clay) within the 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. The assessment of abutment stability required by the CCR Final Rule is not applicable, as the embankment impounding the Clear Water Pond is 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 Clear Water Pond are protected by vegetation above the water line and riprap below the water line. Some areas along the North Embankment interior slope were observed to have sparse rip rap and some shallow sloughing has occurred. The exterior slopes of the Clear Water Pond Southeast Embankment (adjacent to the railroad) are protected from erosion and deterioration by vegetative cover. The exterior slopes of the Clear Water Pond North Embankment (adjacent to the Former Impoundment) are protected from erosion and deterioration by a combination of vegetative cover and/or riprap. The exterior slopes of the 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 Clear Water Pond consists of a gravelly/soil surface around the perimeter of the entire pond. 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. Erosion and sloughing along the interior slope of the North Embankment was observed, though, it is anticipated that this can be addressed through normal maintenance on an as needed basis without creating a risk to the facility. Except for the sparse riprap and shallow sloughing observed along the interior slope of the North Embankment, the existing slope protection measures for the Clear Water Pond are generally considered adequate to provide protection against surface erosion, wave action, and adverse effects of sudden drawdown. The March 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 for the 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.*"

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.

The interior and exterior slopes of the Clear Water Pond embankment contained vegetation (with the addition of riprap in some areas). The vegetation has been maintained by BWL, and reportedly is cut to maintain a height of 6 inches or less. The embankment vegetation did not exceed a height of six inches at the time of the site inspection in March 2020 (Ref. [6]).

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 Clear Water Pond is impounded by an above ground ring-shaped embankment, and there is no run off from adjacent areas. Inflow to the Clear Water Pond is limited to rainfall and water that is hydraulically flowing under controlled conditions. There are no spillways at the Clear Water Pond. The Clear Water Pond is equipped with an Emergency Overflow Structure located between the Clear Water Pond and the swale adjacent to the Canadian National Railroad right-of way, described in Section 1.2. Overflow from the Clear Water Pond would flow through the pipe and exit into the swale. According to BWL, the Emergency Overflow Structure has never been used for discharge.

The Clear Water Pond is considered to be a low hazard potential embankment, as stated in GZA 2012 (Ref. [3]) in which HDR concurs. Therefore, the combined capacity of all spillways must adequately manage flow during and following the peak discharge from the Inflow Design Flood (IDF), defined as the 100-year flood. No capacity calculations were available for the Emergency Overflow Structure. Discharge from the Clear Water Pond is normally maintained by the Pump House located at the northwest corner of the Clear Water Pond for reuse at Erickson Power Station.

The Emergency Overflow Structure can adequately manage flow resulting from the IDF, including wave action, without overtopping of the embankment, provided that the conduit and overflow is maintained without obstructions or debris. 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.

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

- Lake Delta Drainage Structure
- Lake Delta Transfer Structure
- Old Ash Impoundment Transfer Structure
- Old Ash Impoundment Drainage Structure
- Emergency Overflow Structure

Details of each hydraulic structure are discussed in Section 1.2. Each hydraulic structure observed during the March 2020 inspection (Ref. [6]) appeared to maintain structural integrity. Additionally, the structures appeared 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, with exception to the repair made to the intake of the Emergency Overflow Structure. It should be noted that the interior of the pipes and submerged pipes were not observed and should be inspected internally via remotely operated vehicle (ROV).

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 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 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. 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 Clear Water Pond.

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 GZA 2012 (Ref. [3]) report, weekly inspections performed by BWL, and the inspection performed in March 2020 by HDR (Ref. [6]), no structural stability deficiencies were identified for the embankment of the Clear Water Pond.

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. [13]) 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 (d)(1)(v): 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 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 provided by BWL and acquired from the topographic survey performed in 2018 by Droneview and prepared by NTH Consultants, Ltd. (Ref. [11]), the interpreted subsurface profile from the available boring (MW-1) at the Clear Water Pond, and the interpreted phreatic surface based on observations at the site and from the monitoring well installed on the south embankment of the 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 northeast of the Clear Water Pond, is adjacent to the Former Impoundment. Section 1 was selected due to the geometry of the slopes, the height of the embankment, the differential head acting on the section, and the lack of a downstream berm, which is in place over the majority of the north embankment and as formed by the railroad tracks on the southeast embankment. Due to the geometry that is present for this portion of the Clear Water Pond embankment, it was deemed more critical than the other portions of the embankment alignment. 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.

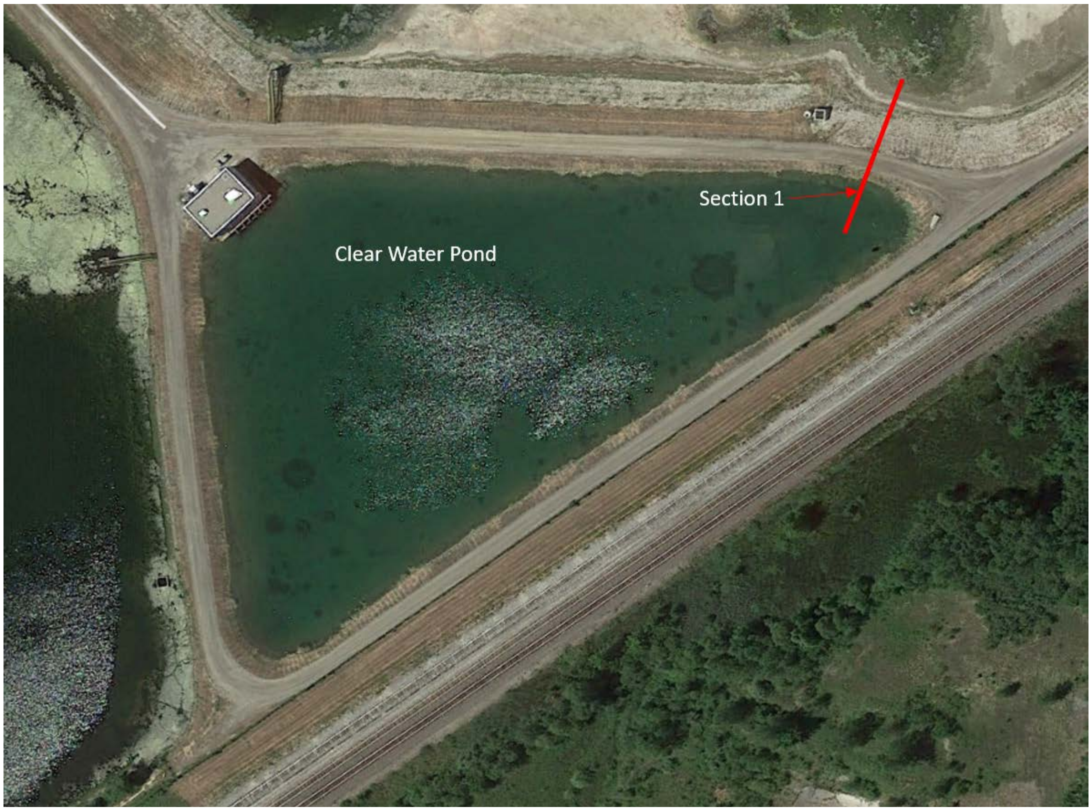


Figure 5. Location of Section 1

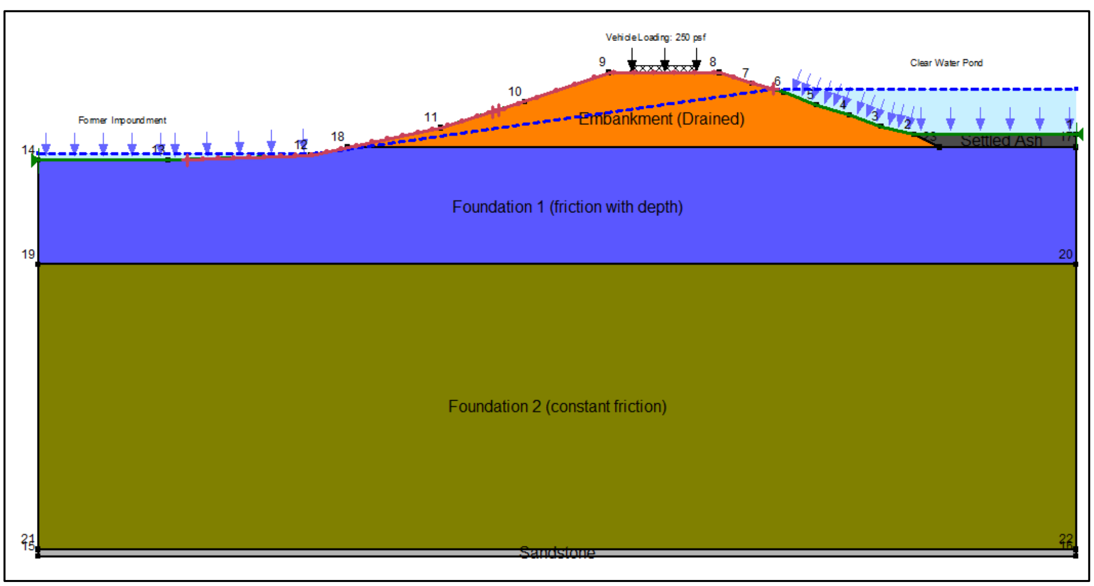


Figure 6. Section 1 Cross Section

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.	Minimum Required Factor of Safety
Maximum Storage Pool (Normal)	882.0[1]	1.5
Maximum Surcharge	884.0[2]	1.4
Seismic[3]	882.0	1.0
Post-earthquake - Liquefaction	882.0	1.2

[1]Assumed to be normal operating pool level of the Clear Water Pond.
[2] Assumed to be approximately at lowest Top of Dike elevation of Clear Water Pond according to 2018 Droneview survey (Ref. [11]).
[3] Using a Peak Ground Acceleration (PGA) = 0.076g with 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 Clear Water Pond and Former Impoundment as well as from historical pond levels provided by BWL. 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 Clear Water Pond is 882.0 feet NAVD 88. The maximum pool surcharge scenario considers the temporary rise of the pond water elevation due to rainfall and collection of site storm water runoff. For the maximum pool surcharge scenario, HDR assumed that the pool level would rise to the lowest surveyed elevation of Top of Dike along the perimeter of the Clear Water Pond: 884.0 feet NAVD 88.

The downstream water boundary condition was set at the current pond elevation of the Former Impoundment: 872.0 feet NAVD 88. The Former Impoundment is no longer in service therefore the water boundary condition should be relatively stable.

The phreatic surface was estimated inside the embankment from the assumed water levels discussed above. Consideration was given to the monitoring well installed at the Clear Water Pond (MW-1), however, a more conservative phreatic surface condition was chosen using a straight line connecting the water level conditions of the Clear Water Pond to the Former Impoundment.

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. [10]). In addition to the 1969 soil borings, geoprobe borings and test pits were performed at the site by MD&E in 2018. In 2019 and

2020, HDR installed six monitoring wells across the site, with one monitoring well (MW-1) being installed through the south embankment of the Clear Water Pond (Ref. [4]).

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	884.5 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
Settled Ash	875 to 873	90	0	30	0	30

* - 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.

The embankment stratigraphy and elevations were interpreted from the 1969 boring logs and MW-1 (Attachment 1), the 2018 Droneview topography (Ref. [11]), 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 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* (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 foot print of the embankment as shown in Figure 7. The borings logs are provided in Attachment 1.

Figure 7. Approximate Boring/Monitoring Well Locations at 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. [5]). 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. [15]), 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 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. [12]), 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 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. [5]). The raw SPT "N" values (N_{raw}) presented on the boring logs were converted to $(N_1)_{60}$ values using the following equation:

$$\boldsymbol{(N1)_{60}} = N_{RAW}C_N C_E C_B C_R C_S$$

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. [5]) and the following equation:

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

Where:

PF = Percent fines passing No. 200 sieve

Using Idriss and Boulanger (Ref. [5]), 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:

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

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 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 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 6 of

Attachment 2 (Ref. [14]). The average embankment acceleration for a deep failure surface was then obtained from the figure on Page 7 of Attachment 2 (Ref. [8]), 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 8 of Attachment 2 (Ref. [8]), 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	1.6	Attachment 3, Page 1
Maximum Surcharge	1.4	1.4	Attachment 3, Page 2
Seismic[2]	1.0	1.5	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.

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

Bryce Burkett, P.E.
Senior Geotechnical Project Manager

Adam Jones, P.E.
Engineering Manager

12 June 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 Engineering, Inc. Groundwater Monitoring 2019 Annual Report, Lansing Board of Water & Light Erickson Station, Lansing, Michigan, January 30, 2020.

Ref. [5] HDR Engineering, 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 Engineering, Inc. Initial Inspection Report, Erickson Station – Clear Water Pond, Lansing Board of Water & Light Erickson Station, Lansing, Michigan, June 12, 2020.

Ref. [7] 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. [8] Makdisi, F.I. and Seed, H.B., Simplified Procedure for Estimating Dam and Embankment Earthquake-Induced Deformations, Journal of Geotechnical Engineering, 1978.

Ref. [9] Mayotte Design & Engineering, P.C. Construction Documentation Report Ash Impoundment System Reconfiguration, Lansing Board of Water & Light Erickson Station, Lansing, Michigan, May 2015.

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

Ref. [11] NTH Consultants, Ltd. Closure Plan, CCR Surface Impoundment System, Erickson Power Station. August 16, 2019.

Ref. [12] 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. [13] USACE. EM 1110-2-1902, Slope Stability, October 31, 2003.

Ref. [14] US Army Corps of Engineers for the Nuclear Regulatory Commission, Technical Bases for Regulatory Guide for Soil Liquefaction, Figure 40, March 2000.

Ref. [15] United States Geologic Survey, Unified Hazard Tool, accessed April 2020, https://earthquake.usgs.gov/hazards/interactive/

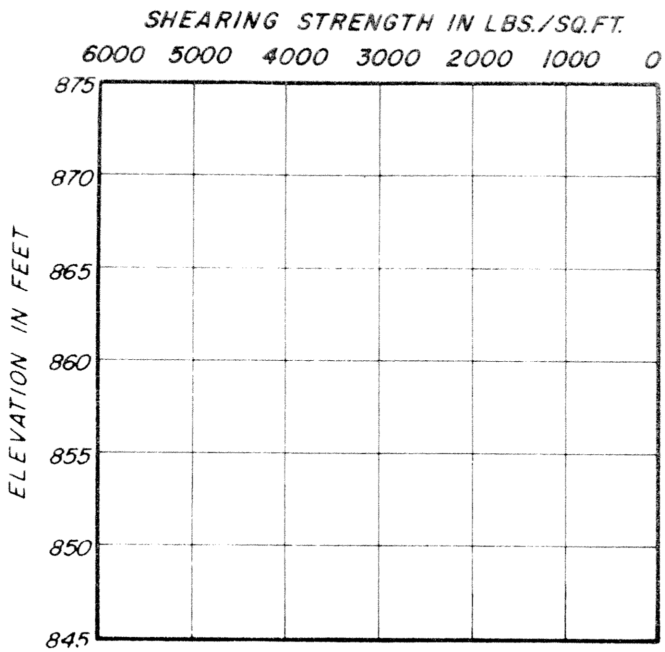
6 Attachments

Attachment 1 Boring Logs and Monitoring Well Logs at Clear Water Pond

Attachment 2 Liquefaction Analysis Figures and Results

Attachment 3 Stability Analyses Results

ATTACHMENT 1
BORING LOGS AND MONITORING WELL LOGS
AT CLEAR WATER POND

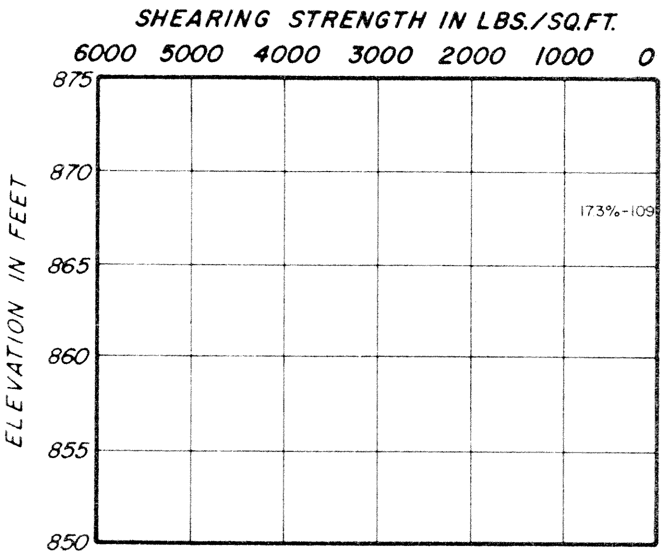


BORING AP-3

SURFACE ELEVATION 872.8

BLOW COUNTS	SYMBOLS	DESCRIPTIONS
	ML	BROWN SANDY SILT WITH ROOTS – TOPSOIL (6")
7	CL	BROWN SANDY CLAY WITH SOME ROOTS ROOTS GRADING OUT AT 2.5'
5	SC	BROWN CLAYEY SAND GRADING SOME SMALL GRAVEL
17	CL	BROWN SANDY CLAY WITH SOME SMALL GRAVEL
19, 30	SP	BROWN FINE TO MEDIUM SAND SEEPAGE WATER ENCOUNTERED AT 7'-3" WATER ROSE TO 5'-10" IN 15 MINUTES
	SP	GRAYISH-BROWN FINE SAND
25	ML	GRAY FINE SANDY SILT
6	SC	GRAY CLAYEY FINE SAND WITH SOME SMALL GRAVEL
21	SP	GRAY SILTY FINE SAND WITH SOME GRAVEL

BORING COMPLETED AT 25.0'
ON 7/8/69
CASING USED TO A DEPTH OF 14.0'
WATER LEVEL NOT RECORDED



BORING AP-4

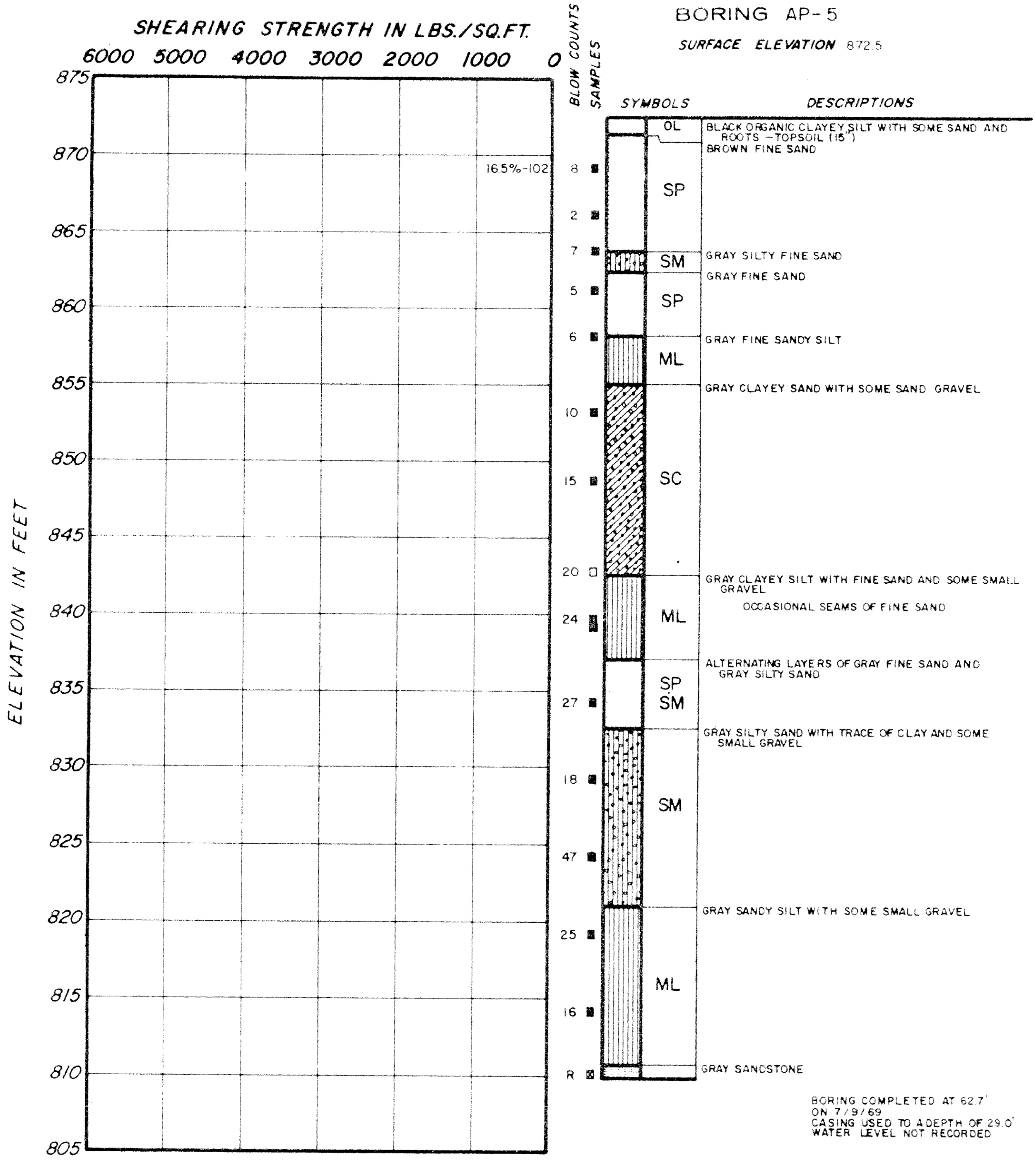
SURFACE ELEVATION 870.7

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

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

LOG OF BORINGS

DAMES & MOORE

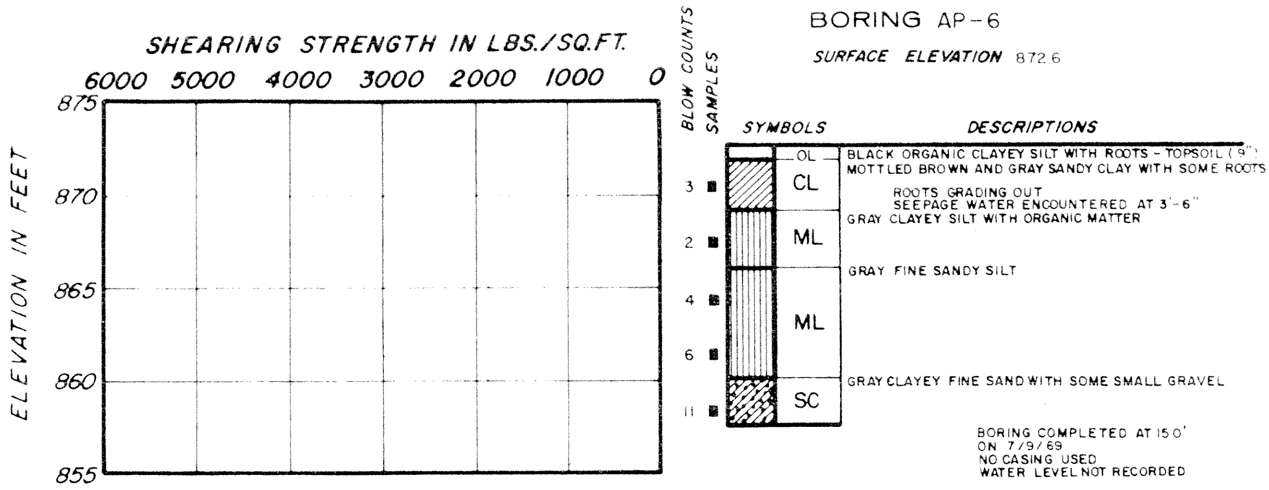


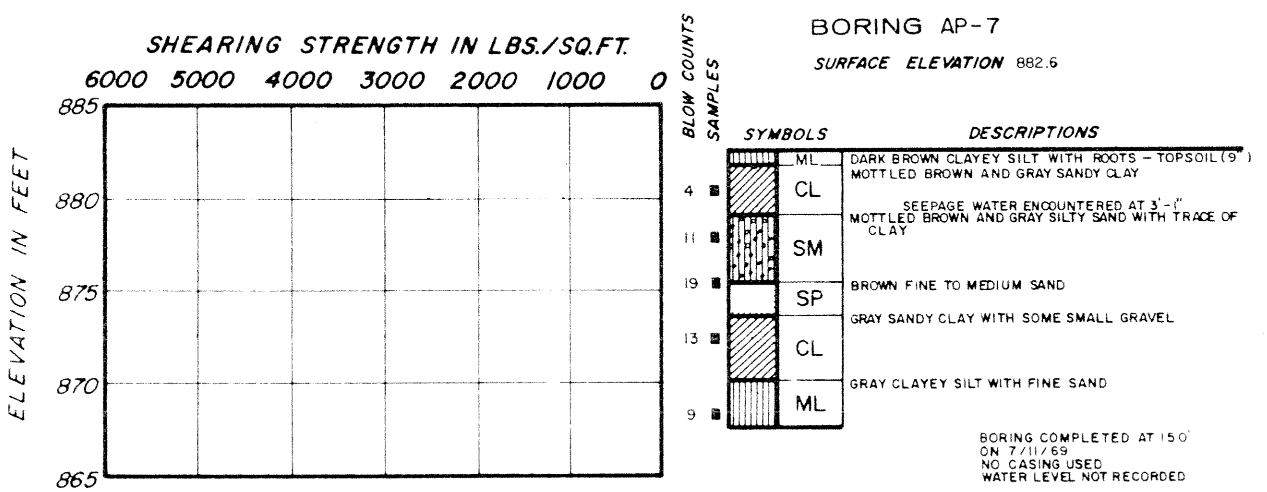
LOG OF BORINGS

Attachment 1, Page 2

DAMES & MOORE

PLATE A-IV





LOG OF BORINGS

DAMES & MOORE

MW-1

PAGE 1 OF 1

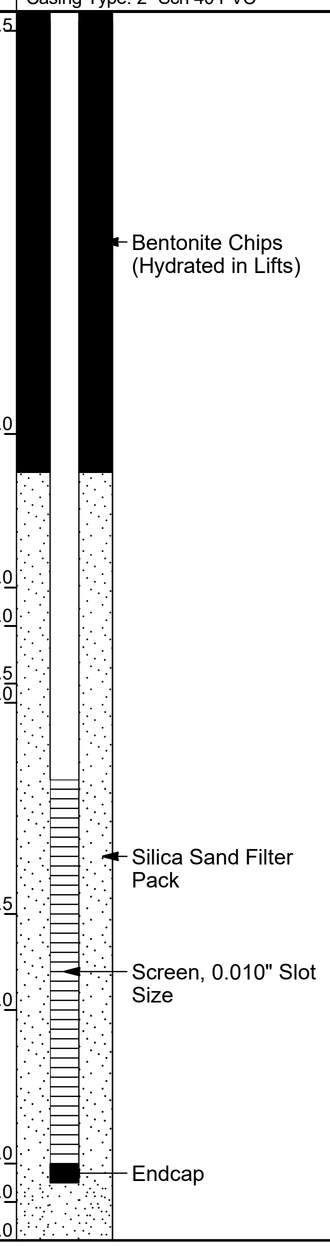
CLIENT Lansing Board of Water and Light
PROJECT NAME LBWL Confidential
PROJECT NUMBER 10173187
PROJECT LOCATION Erickson Power Station, Lansing, MI
DATE STARTED 10/15/19 11:00 **COMPLETED** 10/15/19 12:30
GROUND ELEVATION 885.97 ft MSL **HOLE DIAMETER** 7"
DRILLING CONTRACTOR SME **DRILLER** Rudy Musulin
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	Elev.
0.5		SANDY LEAN CLAY WITH GRAVEL, (CL) brown (10YR 5/3), dry, stiff, low plasticity	885.5
		SANDY LEAN CLAY WITH GRAVEL, (CL) yellowish brown (10YR 5/4), dry, medium stiff, mottled, low plasticity	
11.0		▼ SANDY LEAN CLAY WITH GRAVEL, (CL) yellowish brown (10YR 5/4), moist, medium stiff, mottled, low plasticity	875.0
15.0		SANDY LEAN CLAY, (CL) very dark gray (2.5Y 3/1), moist, stiff, low plasticity	871.0
16.0		CLAYEY SAND, (SC) dark greenish gray (10GY 4/1), poorly graded, fine grained, moist, medium dense, iron oxide staining	870.0
17.5 ▽			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
	SS 20-22	POORLY GRADED SAND WITH CLAY, (SP) gray (5Y 5/1), fine to medium grained, wet, medium dense	
23.5		CLAYEY SAND, (SC) gray (5Y 5/1), poorly graded, fine grained, wet, medium dense	862.5
26.0	SS 26-28	LEAN CLAY WITH SAND, SILTY, (CL) gray (5Y 5/1), fine grained, wet, soft, low plasticity	860.0
30.0		FAT CLAY, (CH) gray (5Y 5/1), wet, stiff, medium plasticity	856.0
31.0		LEAN CLAY WITH SAND, SILTY, (CL) gray (5Y 5/1), fine to medium grained, wet, soft, low plasticity	855.0
32.0			854.0

Bottom of borehole at 32.0 feet.

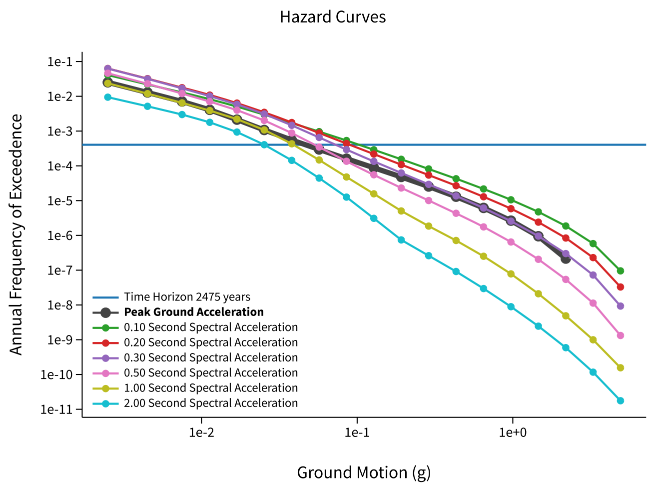
WELL DIAGRAM

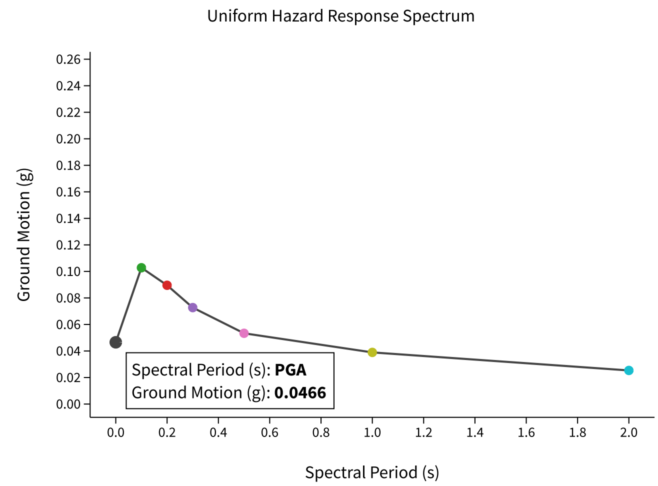
Casing Top Elev: 888.74 (ft)
Casing Type: 2" Sch 40 PVC

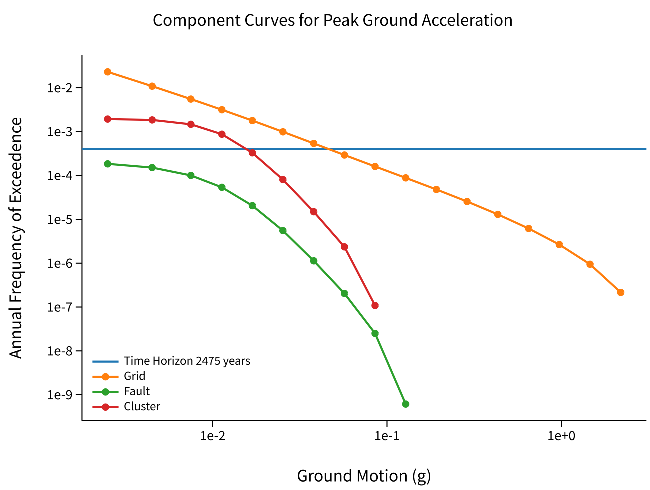


ATTACHMENT 2
LIQUEFACTION ANALYSIS FIGURES AND RESULTS

Hazard Curve





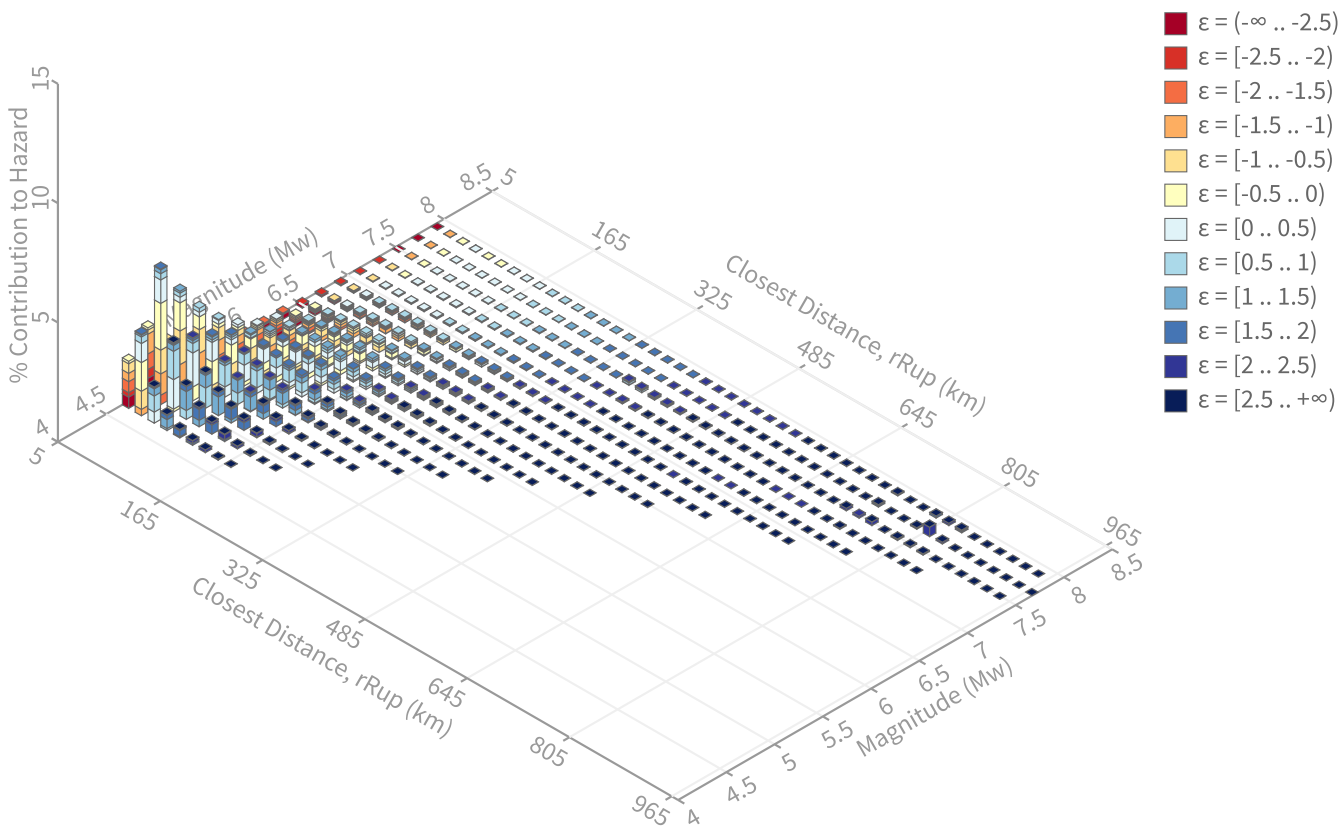


View Raw Data

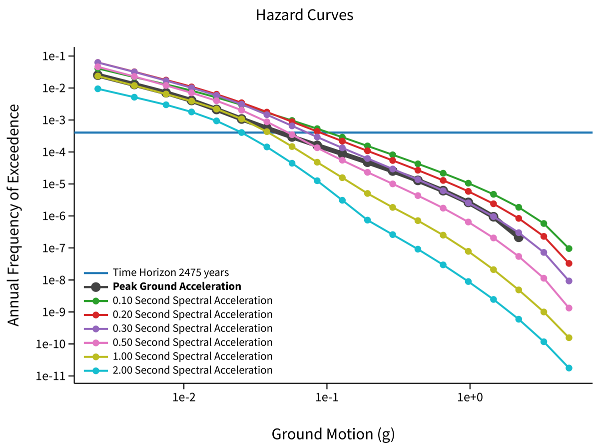
Deaggregation

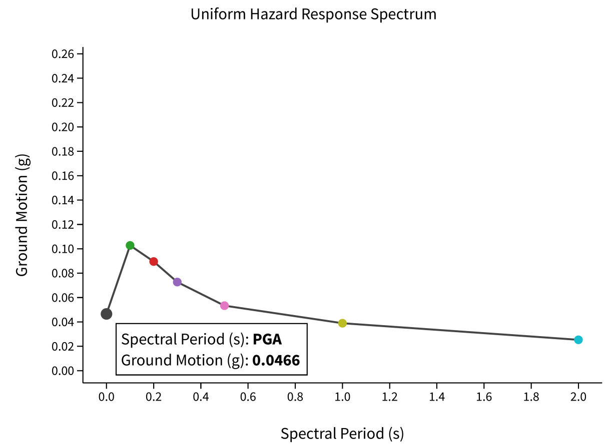
Component

Total



Hazard Curve

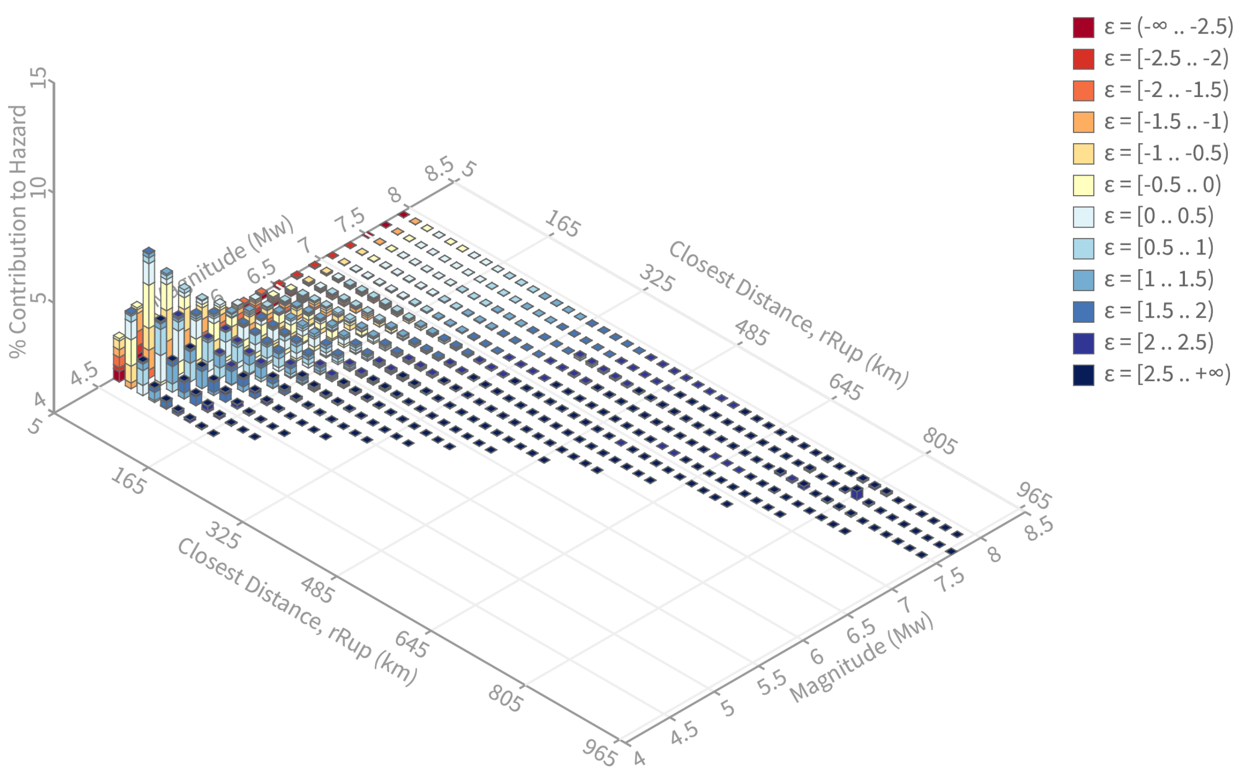


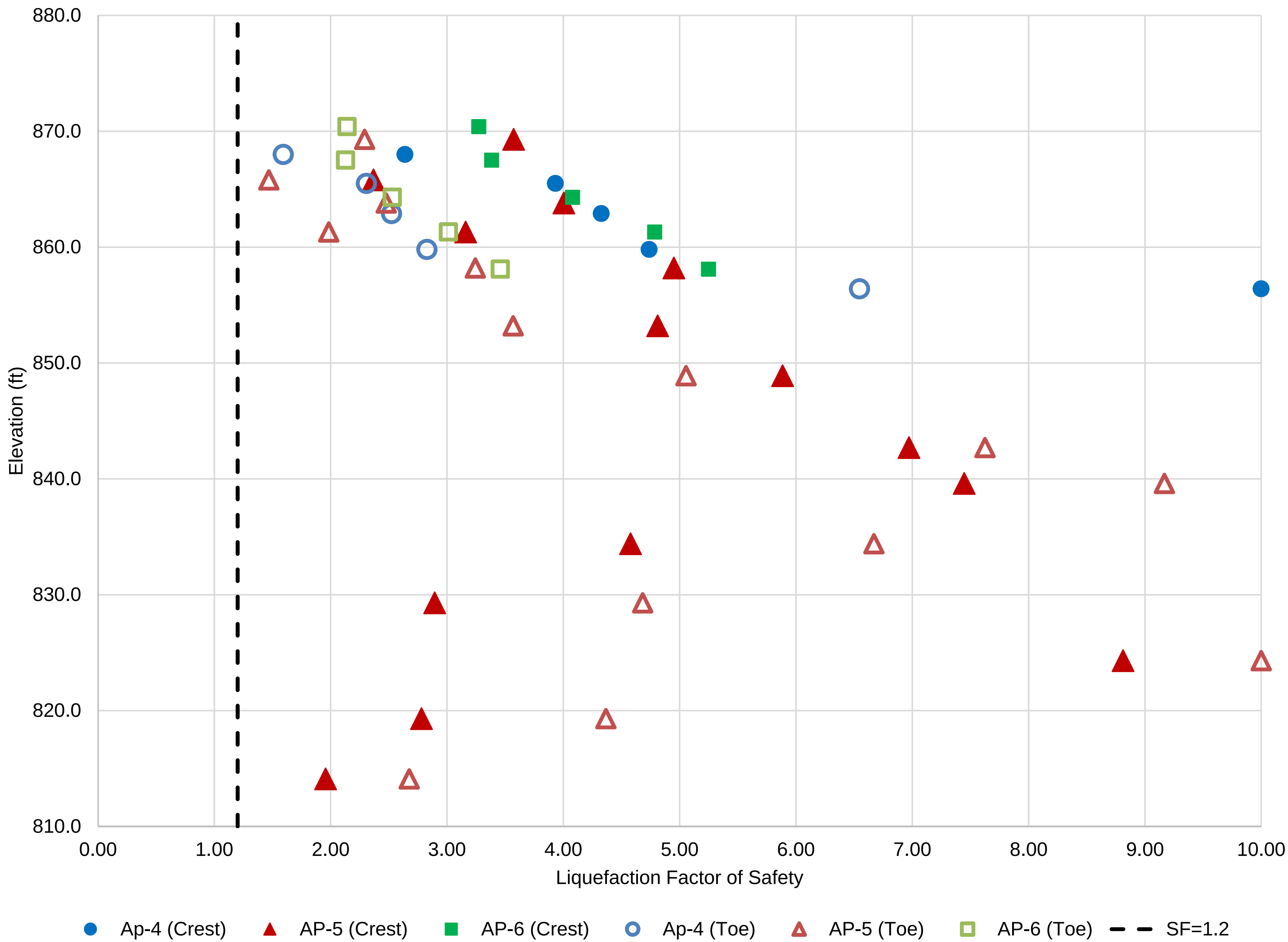


Deaggregation

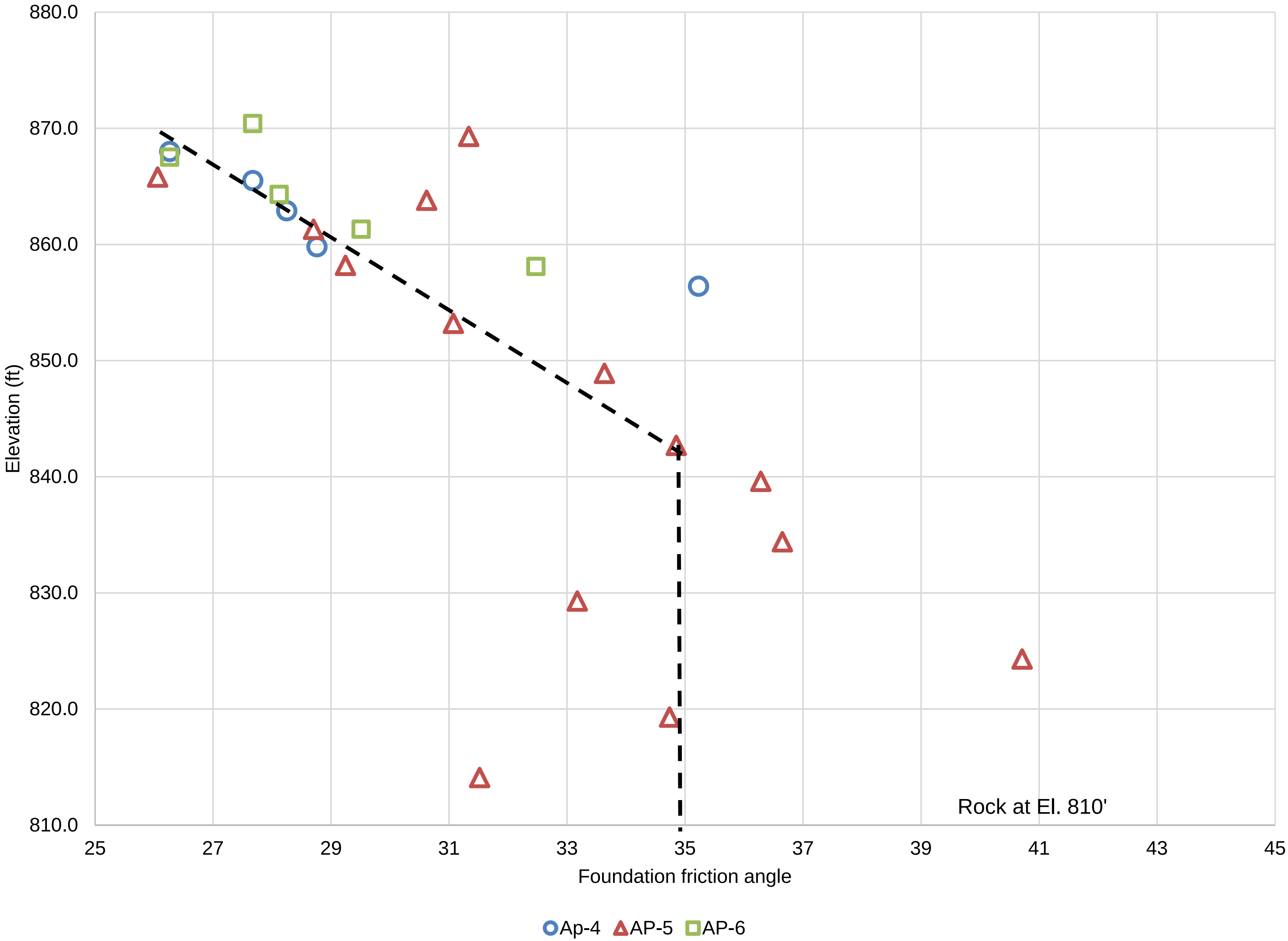
Component

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
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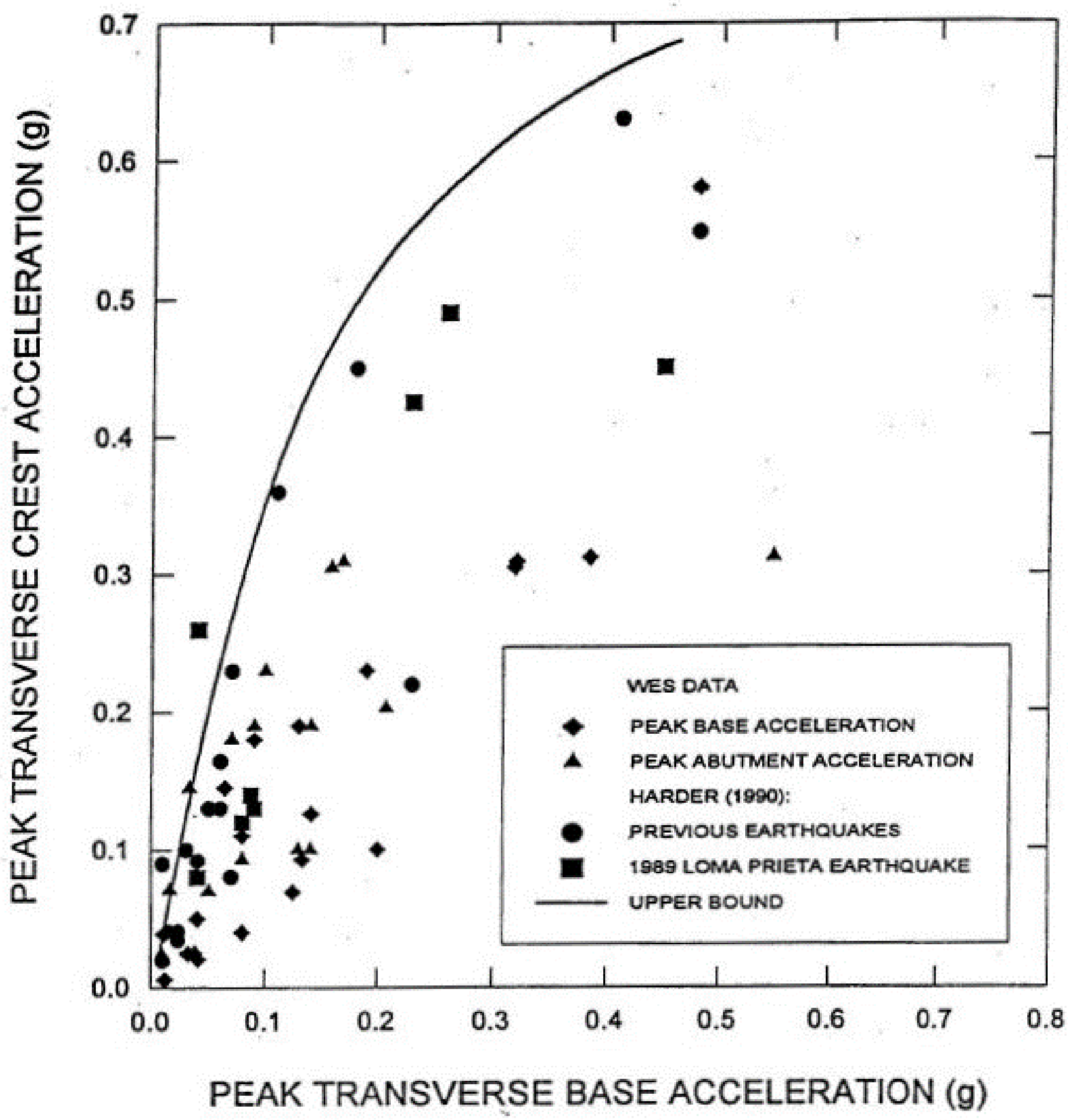
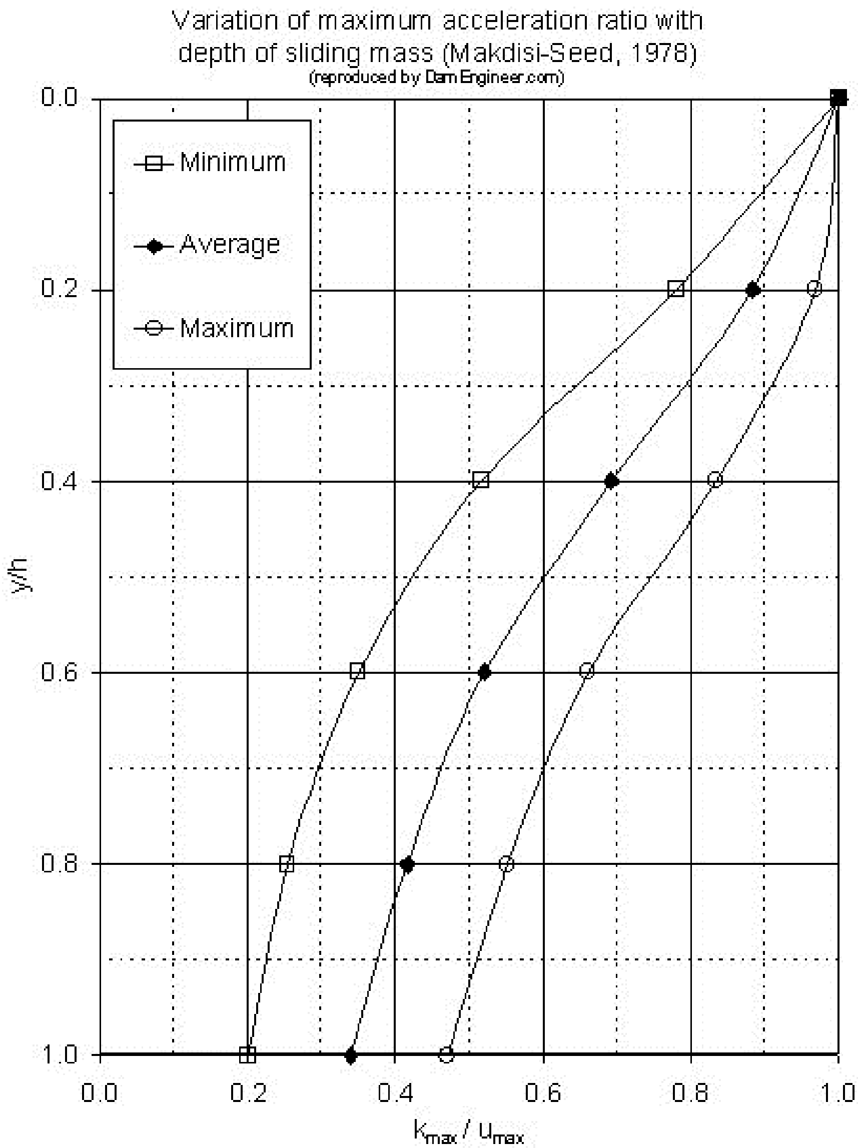
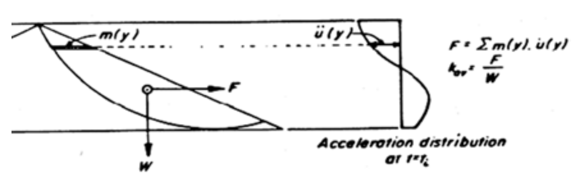
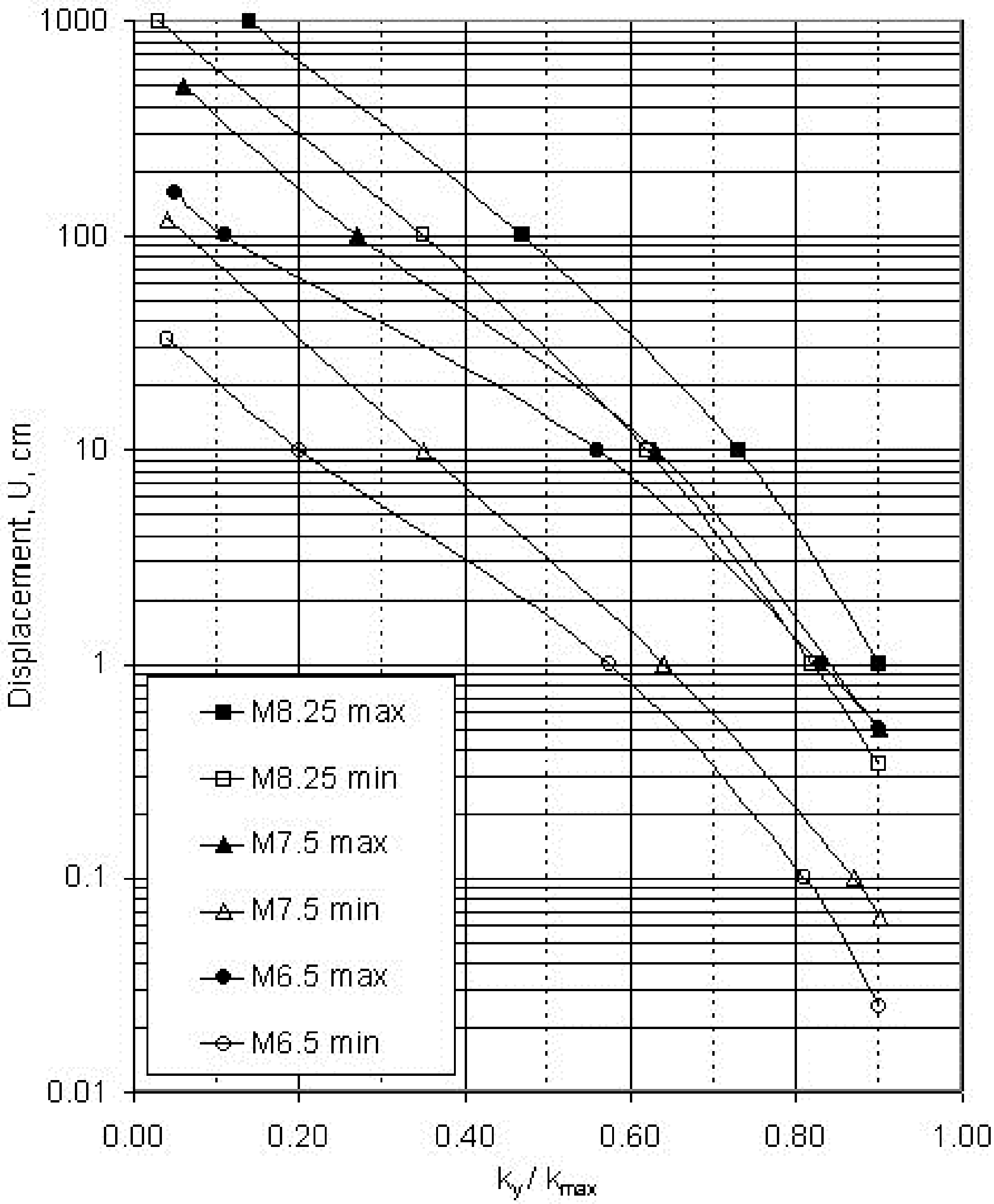
35
37
39
41
43
45
Foundation friction angle
Rock at El. 810'
Ap-4
AP-5
AP-6


PEAK TRANSVERSE CREST ACCELERATION (g)
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)
Minimum
Average
Maximum
y/h
0.0
0.2
0.4
0.6
0.8
1.0
k_max / u_max


m(y)
ü(y)
F
W
F = Σ m(y). ü(y)
Acceleration distribution

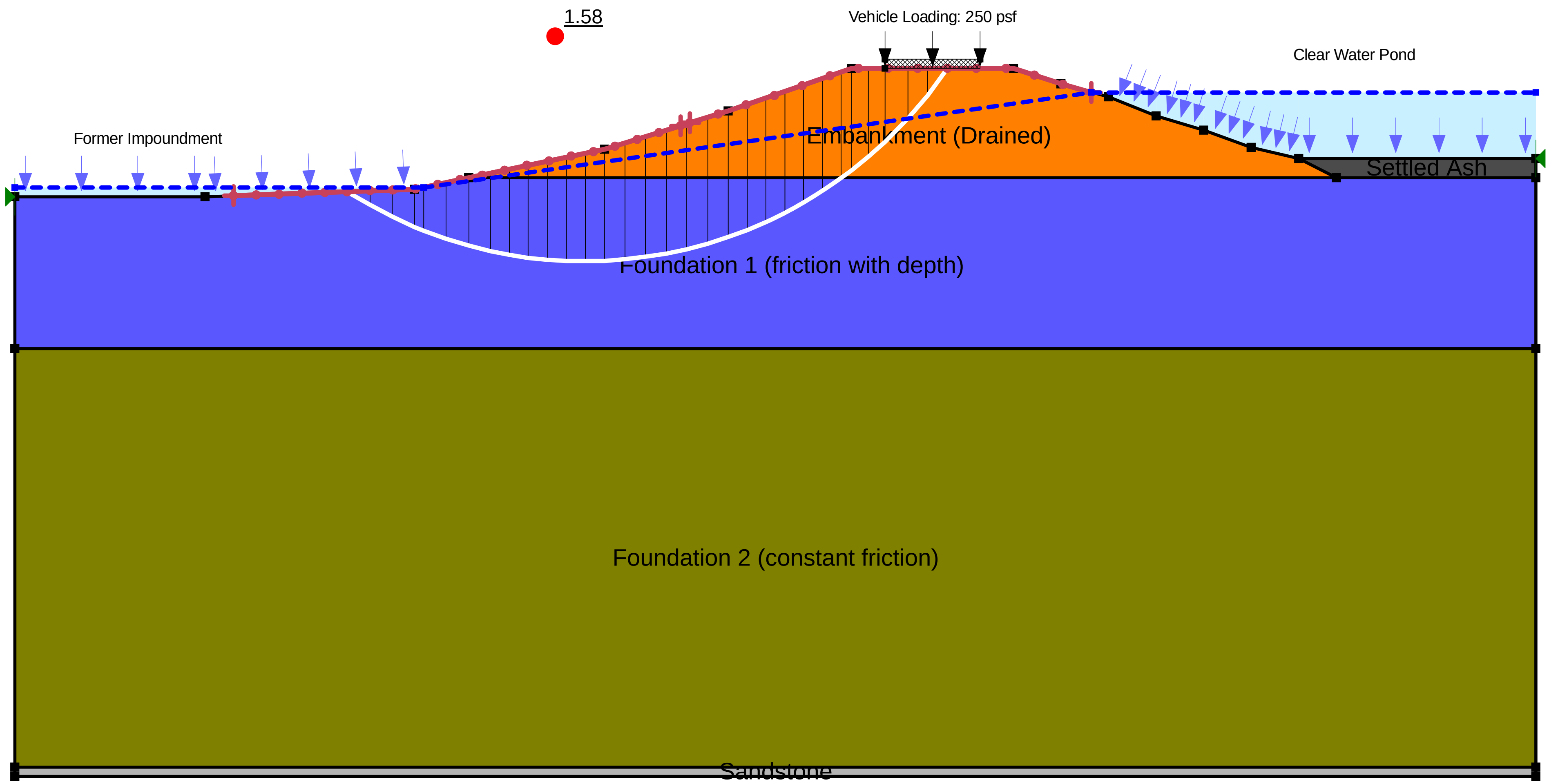
Variation of permanent displacement with
yield acceleration (Makdisi-Seed, 1978)
(reproduced by DamEngineer.com)



ATTACHMENT 3
STABILITY ANALYSES RESULTS

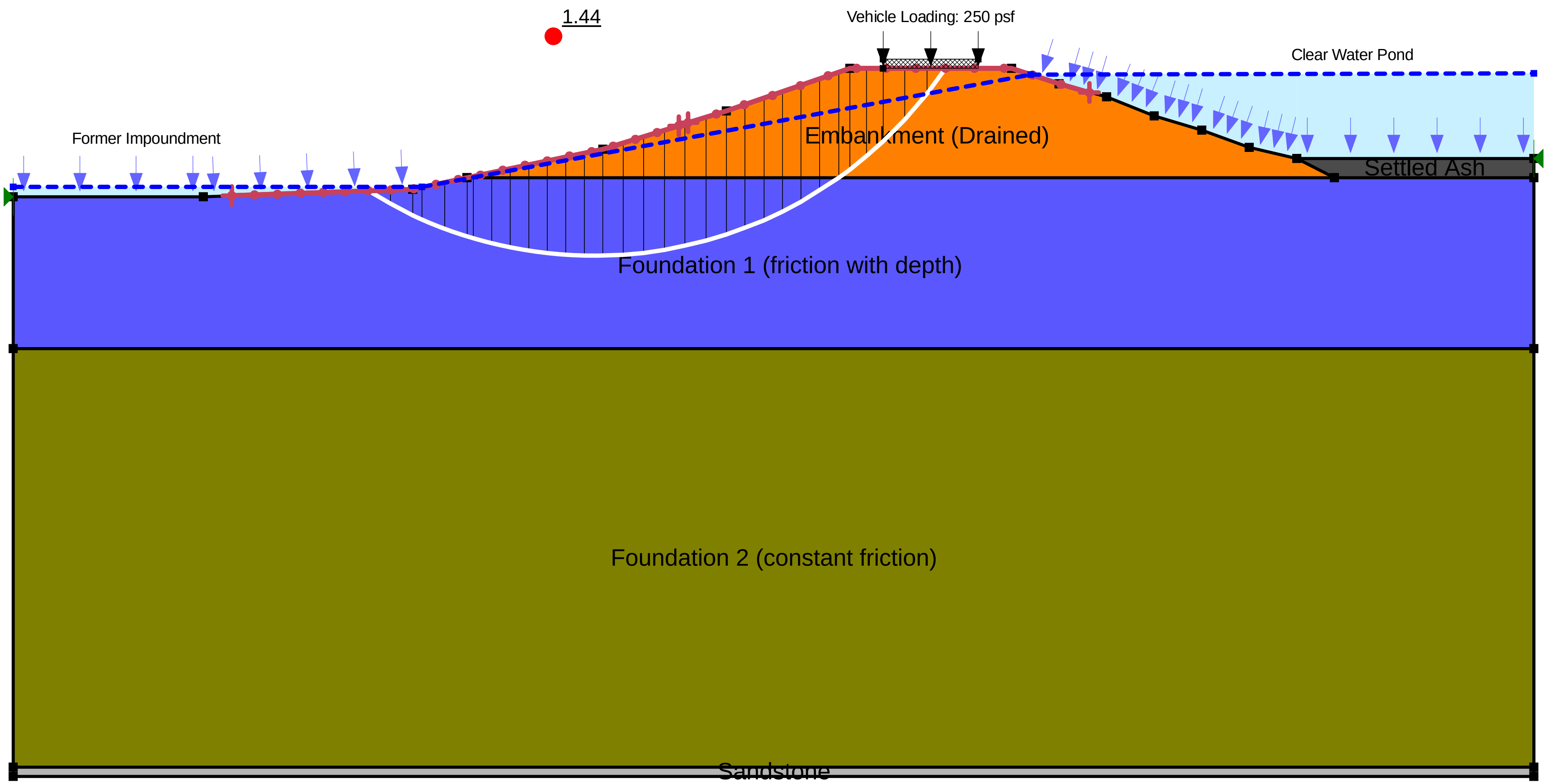
Name: Clear Water Pond - El. 882 feet
Description: Normal Pool, Drained Conditions
Method: Spencer
FS: 1.58

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi Fn	Phi' (°)
	Embankment (Drained)	120	200		28
	Foundation 1 (friction with depth)	115	0	loose silt/sand foundation	
	Foundation 2 (constant friction)	120	0		35
	Sandstone	160	2,000		45
	Settled Ash	90	0		30



Name: Clear Water Pond - El. 884 feet
Description: Surcharge Pool, Drained Conditions
Method: Spencer
FS: 1.44

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi Fn	Phi' (°)
	Embankment (Drained)	120	200		28
	Foundation 1 (friction with depth)	115	0	loose silt/sand foundation	
	Foundation 2 (constant friction)	120	0		35
	Sandstone	160	2,000		45
	Settled Ash	90	0		30



Name: Earthquake
Description: Normal Pool, Undrained Conditions
Method: Spencer
FS: 1.48

Color	Name	Unit Weight (pcf)	Cohesion (psf)	Phi Fn	Phi (°)
	Embankment (Undrained)	120	1,000		0
	Foundation 1 (friction with depth)	115	0	loose silt/sand foundation	
	Foundation 2 (constant friction)	120	0		35
	Sandstone	160	2,000		45
	Settled Ash	90	0		30