2016 Initial Safety Factor Assessment Report

Brunner Island Ash Basin No. 6

Prepared for:
Brunner Island, LLC

September 27, 2016

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Contents

Tables

Appendices

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1.0 Executive Summary

This report presents the Initial Periodic Safety Factor Assessment for the Brunner Island Ash Basin No. 6 facility. This report was prepared by HDR Engineering, Inc., in accordance with the requirements of the United States Environmental Protection Agency (USEPA) 40 CFR Parts 257 and 261 Hazardous and Solid Waste Management System; Disposal of Coal Combustion Residuals From Electric Utilities, April 17, 2015 (USEPA 2015) (CCR Final Rule).

Brunner Island Ash Basin No. 6 is an operating Coal Combustion Residual (CCR) surface impoundment, referred to as an ash basin, which is owned and operated by Brunner Island LLC, a division of Talen Energy (Talen). The ash basin is formed by an earth embankment with a maximum height of approximately 30 feet. The ash basin is, therefore, required to have an Initial and Periodic Safety Factor Assessments performed by a qualified engineer in accordance with the CCR Final Rule. This is the initial (first) Safety Factor Assessment performed in accordance with the CCR Final Rule.

HDR performed slope stability analyses of the critical section of the embankment in accordance with the CCR Final Rule for the long-term maximum storage (normal) pool, maximum surcharge pool, and seismic load cases. The factors of safety for the critical failure section comply with the requirements of the USEPA rule for the long-term maximum storage pool and seismic load cases for the section analyzed, as well as for the surcharge case using an extrapolated groundwater surface profile.

The embankment satisfies the safety factor and deformation requirements of the CCR Final Rule with respect to liquefaction. The embankment soils are not considered susceptible to liquefaction. Post-earthquake slope stability analyses of the impoundment and embankment were conducted assuming that the ash fill liquefies, and these analyses also satisfied the safety factor requirements of the USEPA final rule and related references for the seismic load case.

2.0 Background

Ash Basin No. 6 is located between Black Gut Creek and the Susquehanna River at the southern end of Brunner Island in East Manchester Township, York County, Pennsylvania. The basin was originally owned by PPL Brunner Island, LLC (PPL). In June of 2015, the company changed their name to Brunner Island, LLC, which is a division of Talen Energy (Talen).

The Dam Failure Analysis and Initial Hazard Potential Classification (HDR 2016) for the Brunner Island Ash Basin No. 6 classified the ash basin as a significant-hazard-potential dam. A plan of the ash basin, aerial photograph, and original construction drawings are provided in Appendix A.

The ash basin was designed and constructed between 1975 and 1979. The basin is formed by an oval-shaped, above-ground embankment constructed with rolled random earth fill. The embankment was constructed of native borrow, generally sandy silt to silty clay, with a specified compaction of at least 95 percent of the maximum density determined in accordance with ASTM standard D698. A 10-foot-thick clay liner was constructed along the upstream slope, from bedrock to elevation 287.5 feet. The maximum height of the embankment is approximately 30 feet, the nominal crest width is 15 feet, though the actual crest width is approximately 20 feet, the upstream slope is 2.5H:1V and the downstream slope is 2H:1V. The nominal crest elevation of the embankment is 290 feet. Overall, the embankment is about 8,300 feet long and the impoundment has a surface area of about 70 acres. The basin is subdivided into three main areas. The northern part of the main basin has been completely filled with ash. The southern part of the main basin has not been completely filled with ash and retains open water. To the south of the main basin is a polishing pond, separated from the main basin by a dike, which also retains open water. The Susquehanna River is located approximately 80 feet east of the ash basin at its closest point, and flooding from the Susquehanna periodically extends up the embankment slopes.

Elevations in this report refer to Plant datum. The Plant vertical datum, the National Geodetic Vertical Datum of 1929 (NGVD 29), is approximately 0.76 feet higher than the North American Vertical Datum of 1988 (NAVD 88) at Ash Basin No. 6.

Ash is no longer being discharged into the basin, although process water which has come into contact with ash is still being discharged at the northwest corner of the basin; therefore, the ash basin is still considered to be active. The plant's equalization pond also discharges into the basin at the northeast corner.

The USEPA released and published in the Federal Register the final rule regarding CCR surface impoundments (USEPA 2015) on April 17, 2015, referred to herein as the CCR Final Rule. The CCR Final Rule establishes nationally applicable minimum criteria for the safe disposal of CCR in landfills and surface impoundments, and requires that the owner or operator of each CCR unit demonstrate and document that the CCR unit complies with these criteria.

Section § 257.73 of the CCR Final Rule requires that initial and periodic safety factor assessments be conducted to verify that the most critical section of the embankment achieves the required minimum factors of safety for embankment slope stability for the long-term maximum storage pool, maximum surcharge pool, and seismic load cases. This report presents the Initial Periodic Safety Factor Assessment for the Brunner Island Ash Basin No. 6 facility.

3.0 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 (USACE 2003) 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.

EM 1110-2-1902 does not address seismic slope stability analyses, noting that the USACE guidance document, Dynamic Analysis of Embankment Dams, is still in draft form. The CCR Final Rule refers to USACE Engineering Circulars that address dynamic analysis that are in draft form or have expired. Seismic analyses were conducted in accordance with Draft 2 of EC 1110-2-6001, Seismic Analysis of Embankment Dams, dated May 27, 2011, through a combination of liquefaction analyses, slope stability analyses using the pseudostatic seismic coefficient method, and deformation analyses. The potential for liquefaction was determined through a triggering analysis using methods proposed by Idriss and Boulanger (2008) and Youd and Idriss (2001), sources which are recognized in the CCR Final Rule. Seismic slope stability analyses were performed using the pseudostatic seismic coefficient method, in accordance with Special Publication 117, Guidelines of Evaluating and Mitigating Seismic Hazards in California, 2008 (California Geological Survey 2008), a reference that is used within the engineering community. Deformation analyses were conducted using the Jansen method (1988), which is consistent with the screening methodology described in EC 1110-2-6001 (USACE 2011) and Engineering Design Manual, Coal Refuse Disposal Facilities(MSHA 2009), which is also recognized by the CCR Final Rule.

3.1 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.

3.2 Critical Cross Section Geometry

The design of the embankment is consistent around its entire perimeter, varying only in height and, potentially, foundation conditions. Two sections of the embankment were considered as potentially being critical based on observations of field performance, described below and located as shown on Figure A-2.

- Section 1-1 at Station 21+80, located within the filled section of the impoundment, is immediately adjacent to a section of the embankment where several shallow failure surfaces had been observed previously. These shallow slope failures suggested that the factor of safety for deep-seated failure surfaces may also be critical at this location. Wet soils were observed up to 1/3 of the way up the embankment face, suggesting that the phreatic surface was relatively high.

- Section 2-2 at Station 7+44 is located within the section of the embankment that retains open water, although this part of the basin has been filled since the boring was drilled in 2009. During planning of the geotechnical investigation conducted in 2009, it appeared that moisture on the downstream face and toe of the embankment was more pronounced than at other sections of the embankment, with wet soils and ponded water extending up to 5 feet up the slope.

A geotechnical investigation was conducted in 2009, which included borings drilled through the crest of the dam and through the downstream slope, and installation of open standpipe piezometers at the two sections discussed above.

The phreatic surface at both sections was lower than that anticipated based on surface indications of moisture, suggesting that the moisture may have been the result of rainfall or localized conditions that were not reflected in the piezometers. The piezometers at Section 2-2 have continued to read dry since 2009. The cause of the variation in phreatic surfaces between the two sections has not been explained, but could be due to the fact that the ground water level against Section 1-1 may be slightly higher than at Section 2-2, which may overtop the upstream clay liner. Variation in the fines content of the embankment fill material, especially the higher fine contents at Section 1-1, may also partially explain the higher embankment phreatic level.

Section 1-1 at Station 21+80 was determined to be the critical section since the piezometric levels were noticeably higher than at Section 2-2, and this section is immediately adjacent to a part of the embankment where shallow slope failures have occurred previously. The section height is approximately 29 feet, only slightly less than the maximum section height of 30 feet.

The critical cross section, as it was modeled in Slope/W, is shown as Figure A-5 in Appendix A. Talen surveyed the piezometers and the basin using ground control surveys and photogrammetry aerial surveys in 2015. The upstream slope, which is not visible, was assumed to have a 2.5H:1V slope, as shown on drawing E158595 Sheet 2, Figure A-4 in Appendix A. The crest width was estimated to be about 20 feet wide, as determined from the 2015 survey CAD files provided by Talen. Piezometer B09-1 was determined to be located about 7 feet from the outer crest guardrail, with the top of steel casing about 7 inches below the top of crest elevation of 290 feet. The ground surface elevation at Piezometer B09-2 was surveyed at EL 270.76, located on the downstream slope. The embankment stratigraphy, including the natural ground surface elevation and bedrock elevation, was determined from the 2009 boring logs for B09-1 (crest) and B09-2 (downstream slope), provided in Appendix C1.

The stability of the splitter dike between the middle sub-basin and the polishing pond was not assessed during this study. The splitter dike is totally contained within the perimeter dike, and a breach of the splitter dike would not result in an uncontrolled release of ash, providing the discharge conduit was closed.

There may be sections with phreatic surfaces that are higher than that encountered at Section 1-1, particularly at the northwest corner of the main basin where artesian conditions have been observed in the riser of Monitoring Well 6-1B. This warrants some conservatism when interpreting the stability analysis results.

3.3 Credible Load Cases

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

Table 1. Loading Conditions and Minimum Required Factors of Safety

Loading Condition	Headwater El.	Minimum Required Factor of Safety
Steady State Seepage – Maximum Storage Pool (Normal)	286.0[1]	1.5
Maximum Surcharge	289.0[2]	1.4
Seismic[3]	286.0	1.0
Post-earthquake – Liquefaction of impoundment	286.0	1.2

[1]Assumed to be approximately 1 foot below ground surface at swale adjacent to embankment
[2] Assumed to be about 1 foot below the top of the embankment
[3] Using a Peak Ground Acceleration (PGA) = 0.08g with 2 percent probability of exceedance in 50 years (2,475 recurrence interval) (USGS 2008).

The new USEPA regulation also requires any CCR unit that has downstream slopes susceptible to inundation by the pool of an adjacent water body such as a river, stream, or lake, to be analyzed for rapid drawdown loading of the downstream slope. The results for this analysis are presented in the Initial Periodic Structural Stability Assessment Report for Brunner Island, in accordance with the requirements of the CCR Final Rule. Rapid drawdown analysis of the upstream face of the embankment is not a likely load condition for a CCR impoundment and is not required by the CCR Final Rule.

3.4 Phreatic Conditions

The normal operating water surface level is measured at the downstream (south) end of the open water part of the basin and, since 2009, the normal operating level has been lowered from EL 288.0 feet to approximately EL 284.2 feet (about 3.8 feet) as a result of operational changes. The elevation of the groundwater surface at the northern end of the basin, where the critical cross section was taken, is likely higher than the pool elevation, as evidenced by the visible gradient in the discharge channels; but it has not been measured. Talen reported that the discharge swale that runs along the upstream face of the west embankment and used to carry free water has been reestablished and is dry during normal operation. The invert of the channel, based on the 2015 survey, is at about EL 287.0 feet; therefore, a normal pool elevation of EL 286.0 was assumed at the upstream face of the embankment at the critical section location. A separate study was conducted by HDR in June, 2015 to evaluate the hydraulic and hydrologic adequacy of the basin. The upstream water level assumed in the stability analysis for the surcharge condition was determined based on the depth of flow in the ditch along the upstream face of the embankment at the critical section location, estimated in the hydrologic and hydraulic study.

The potential for overtopping the embankments was evaluated in the separate hydrologic and hydraulic study report discussed above.

The phreatic surface for the normal, seismic, and liquefaction analyses was conservatively based on the highest piezometer readings observed since 2010. At Piezometer B09-1 on the crest, the piezometric level was modeled at EL 275.5, as measured on May 6, 2010. At Piezometer B09-2 on the downstream slope, the piezometric level was modeled at EL 265.5, as measured on May 24, 2011. The piezometric levels can be seen in Figure A-6 in Appendix A and the time-history plot for Piezometers B09-1 and B09-2 can be seen in Appendix D, Figures D-1 through D-4.

For the surcharge loading conditions, the phreatic surface at each of the piezometers was assumed to rise 1 foot as a result of the short term response to flooding. The basin has not experienced significant flooding since the piezometers were installed, and the actual piezometric response to flooding is not known. Due to the relative impermeability of the embankment soils and expected brief duration of a significant flood, a significant rise in the phreatic surface during flooding is not anticipated.

The phreatic level was assumed to be linear between headwater and B09-1 and between B09-1 and B09-2. Observations during inspections since 2008 have indicated that soils at the toe are wet, so the phreatic surface was assumed to be at ground level at the toe, with saturated soils downstream. The phreatic surface for both normal and surcharge conditions can be seen in Figure A-5 of Appendix A.

3.5 Material Properties

The material properties used for the 2015 slope stability analysis are presented in Table 2. The assumed material properties were based on correlations with Standard Penetration Testing (SPT) performed in 2009, triaxial testing, and other laboratory tests on soil samples from B09-1 and B09-2. The boring logs and laboratory test results are provided in Appendix C. HDR used drained shear strengths related to effective stresses, as recommended by the USACE. Table 2 below provides a summary of soil material properties used in the analysis.

For the post-seismic liquefaction analysis, no significant strength reduction of the embankment fill is anticipated, due to the degree of compaction, the resistance to liquefaction as discussed below, material composition, and relatively low seismicity. The ash, however, was deposited hydraulically, without compaction, and is likely in a very loose state. Stability was assessed using two strength models. For the first model, the impounded ash was assumed to liquefy completely, and the shear strength was neglected, which is a very conservative assumption. For the second model, the shear strength was assumed to vary as a function of effective vertical stress, as recommended by the Mine Safety and Health Association, in accordance with MSHA (2009).

Table 2. Summary of Material Properties Used in Analysis

Material Types	γ_{moist} (pcf)	γ_{sat} (pcf)	c' (psf)	φ' (degrees)
Native Soil	N/A	130	0	30.0
Clay Liner	130	130	0	30.0
Embankment Fill	125	135	0	37.0
Ash Fill (Storage)	N/A	90	0	30.0
Liquefied Ash Fill (Storage) Strength Model 1	N/A	90	0	0
Liquefied Ash Fill (Storage) Strength Model 2	γ =90pcf, Ratio of shear strength to effective vertical stress = 0.04			

4.0 Assessment of Liquefaction Potential

A “triggering analysis” was used to assess the potential for liquefaction of the embankment soils using correlations with the SPT data from Borings B09-1 and B09-2. The calculation is provided in Appendix C2, which also includes the input parameters used in the analysis. The first step in the triggering assessment was to determine the appropriate seismicity. The 2008 USGS Hazard Mapping deaggregation estimates a peak ground acceleration of 0.08g, with a probability of exceedance of 2 percent in 50 years, or a return period of 2,475 years, consistent with the CCR Final Rule. The attenuation model used to estimate the peak ground acceleration, assumed a shear wave velocity of 760 meters per second for a firm rock, Site Class BC, in eastern or central US, which is considered appropriate for the sandstone and mudstone rock identified beneath Brunner Island Basin No. 6 through Borings B09-1 and B09-2. The deaggregation indicated that the source behavior is consistent with a Richter Magnitude (M) M5.8 (Mean) event. For use in the triggering analyses, a M5.8 earthquake with a Peak Ground Acceleration (PGA) of 0.08g was assumed for the design earthquake.

As discussed above, the SPT data from the borings were used as the basis of the liquefaction assessment of the embankment soils. Hammer blow counts were measured in 6-inch increments each time the spoon was driven to recover a sample for classification and laboratory index testing. The blow counts from the middle two 6-inch drives were used to determine the raw SPT “N” values, which are presented on the attached boring logs, expressed in blows per foot.

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 percent efficiency (referred to as the $(N_1)_{60}$ value). This allows correlation of the site SPT data with empirical liquefaction correlations. The methods used to calculate $(N_1)_{60}$ were those that have been proposed by Youd and Idriss (2001) and Idriss and Boulanger (2008). The raw SPT “N” values (N_{raw}) presented on the boring logs were converted to $(N_1)_{60}$ values using the following equation:

$(N_1)_{60} = N_{RAW}C_NC_EC_BC_RC_S$

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 - Circular reference

= 1.1	**[$(N_1)_{60}$ = 0 to 10]**
= 1+ [$N_1)_{60}$/100]	**[$(N_1)_{60}$ = 10 to 30]**
=1.3	**[$(N_1)_{60}$ = >30]**

Additional corrections were then made to correct the $(N_1)_{60}$ value to an equivalent “clean sand” value for use in determining cyclic resistance ratio (CRR), which was used for assessing triggering of liquefaction. The clean sand value, $(N_1)_{60cs}$, was determined based on the grain size analysis results from the laboratory testing and using the method proposed by Idriss and Boulanger (2008) and the following equation:

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

where,

PF = Percent fines passing No. 200 sieve

Using Idriss and Boulanger (2008), CRR was then calculated using the following equation:

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

The Cyclic Stress Ratio (CSR) was then calculated using the loading applied by 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 0.08g, determined from the U.S. Geological Survey (USGS) geologic hazards website, was assumed at the base of the dam and the distribution of CSR through the dam cross-section was determined. For critical projects in high seismic areas, seismic amplification is typically assessed using Finite Element Analysis. Since the seismicity at Brunner Island is relatively low, a simplified approach was used by evaluation of a conservative crest acceleration using empirical methods and then linearly interpolating the acceleration through the dam cross-section by depth below the crest. The crest acceleration was estimated using Figure 12, from Jansen and Leps, provided in the attachments, which is a plot of peak ground acceleration versus amplification. With the

PGA for the dam site of 0.08g, the corresponding amplification factor is approximately 5 times the base acceleration, resulting in a peak acceleration of 5 (0.08g) = 0.4g. The analysis then estimated the shear stress reduction with depth by using the reduction factor r_d. The CSR was then calculated using the following equation:

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

where,

a_{max}/g = 0.08 at base and 0.4 at crest (interpolate in between)

σ_v = Total Overburden Stress

σ'_v = Effective Overburden Stress

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

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 = 5.8

z = depth in meters

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

FS = CRR/CSR (MSF) (Kσ) (Kα) [Youd and Idriss 2001]

where,

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

Kσ= Confining stress correction = $(\sigma'_v/P_a)^{(f-1)}$

where,

P_a = Pressure at 1 atmosphere

f = relative density factor = 0.7 to 0.8 (relative density 40 to 60%)

0.6 to 0.7 (relative density 60 to 80%)

The static shear stress coefficient, Kα was conservatively taken equal to one for the purpose of this analysis, because N_{60} values are generally greater than 10, the downstream slope is relatively shallow, and the embankment is relatively short.

If the calculated triggering factor of safety (FS) was 1.2 or less, then liquefaction was assumed to occur, consistent with USEPA criteria. If it was greater than 1.2, liquefaction was not indicated.

5.0 Potential Seismic Deformation

Deformation under the maximum credible earthquake loading was estimated using the Jansen (1988) Method, which is an industry recognized empirical relationship for estimating deformation of an embankment dam under seismic loading. The first step in this analysis was to determine the yield acceleration of the embankment. This was done by analyzing the seismic slope stability and incrementally increasing the pseudostatic seismic coefficient until a minimum factor of safety of 1.0 was computed. The seismic coefficient that achieved this factor of safety is considered the yield acceleration. As shown on Figure 8, the minimum yield acceleration was found to be 0.15g for the downstream slope. The yield acceleration was conservatively determined for a shallow failure surface. As noted in EC 1110-2-6001, yield accelerations for deep-seated failure surfaces that disrupt the crest are typically considered, which would result in higher yield accelerations.

Similar to the triggering analysis discussed above, an M5.8 earthquake with a PGA of 0.08g was assumed for the deformation analysis, and the embankment response was estimated using the Leps and Jansen Plot (shown on Figure B1-6 of Appendix B). With the PGA for the dam site of 0.08g, the corresponding amplification factor was conservatively assumed to be 5 times the base acceleration, or 5 (0.08g) = 0.4g. Using the above data, the deformation under the Maximum Credible Earthquake (MCE) for the site was estimated using the Jansen Equation (Jansen 1988):

$$U = (19(M/10)^{8} * (k_m - k_y))/(k_y^{0.5})$$

where,

U = estimated seismic deformation, feet

M = earthquake magnitude = M5.8

k_y = yield acceleration = 0.15g

k_m = maximum crest acceleration (based on estimated amplification, Figure 12, for a PGA of 0.08g) = 5 * 0.08g = 0.4g

6.0 Results and Conclusions

6.1 Stability Analysis Results and Conclusions

Analysis summary diagrams for each loading case are provided as Figures B1-1 through B1-6 in Appendix B. Table 3 below also summarizes the results of the analyses conducted for all loading cases.

As shown in Table 3, the factors of safety against slope instability, for deep failure surfaces that are capable of breaching the basin, satisfy the requirements of the CCR Final Rule under all loading conditions. The embankment is not expected to liquefy and, if the impounded ash were to liquefy assuming a total loss of strength, the stability of the embankment would still be adequate. The critical seismic yield acceleration (to obtain a Factor of Safety of 1.0) is also considerably greater than the PGA with 2 percent probability of exceedance in 50 years.

Table 3. Summary of Stability Analyses Results

Loading Condition	Failure Surface	Required Minimum Factor of Safety	Computed Factor of Safety
Normal	Deep	1.5	1.5
Surcharge	Shallow	1.4	1.4[1]
Seismic[2]	Shallow	1.0	1.2[1]
Liquefaction Potential	NA	1.2	1.4
Post-earthquake – liquefied Ash Fill 1	Shallow	1.2	1.4[1]
Post-earthquake – liquefied Ash Fill 2	Shallow	1.2	1.4[1]

[1] The factor of safety for a deep failure surface would be greater than the computed minimum factor of safety, which corresponded to a shallow failure surface. Shallow failure surfaces 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. Deep failure surfaces represent the most critical failure surfaces for the embankment stability.
[2] Using PGA = 0.08g with 2 percent probability of exceedance in 50 years (2,475 recurrence interval) (USGS 2008).

Using the Jansen Method to calculate deformation, a yield acceleration of 0.15g was calculated and it was estimated that a fraction of a foot (approximately 2 inches) of seismic deformation could occur under the MCE. This is considerably less than the deformation limits of 3.0 feet or less recommended in MSHA (2009), a source referenced by the CCR Final Rule.

7.0 Summary

HDR analyzed the stability of the embankment at Brunner Island Ash Basin No 6. Section 1-1, located in the east embankment, which was determined to be representative of the critical embankment section. The analysis was conducted in accordance with the criteria of the CCR Final Rule and the referenced methodologies published by the USACE (2003), MSHA (2009), Idriss and Boulanger (2008), Youd and Idriss (2001) and Jansen (1988).

The calculated factors of safety satisfy the minimum factor of safety criteria for the long-term maximum storage (normal) pool, maximum surcharge pool, and seismic load cases. The calculated factors of safety against liquefaction satisfy the minimum factors of safety criteria; therefore, the embankment is not considered vulnerable to liquefaction. Seismic deformation of approximately 2 inches is possible under the design earthquake, which satisfies deformation criteria.

8.0 Closure

Based on the information currently available, I certify to the best of my knowledge, information and belief that this initial Periodic Safety Factor Assessment meets the requirements of CCR Rule §257.73(e) Structural Integrity Criteria for Existing CCR Surface Impoundments, Periodic Safety Factor Assessments, in accordance with professional standards of care for similar work. HDR appreciates the opportunity to assist Talen with this project. Please contact us if you have any questions or comments.

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9.0 References

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Appendix A. Plans and Sections

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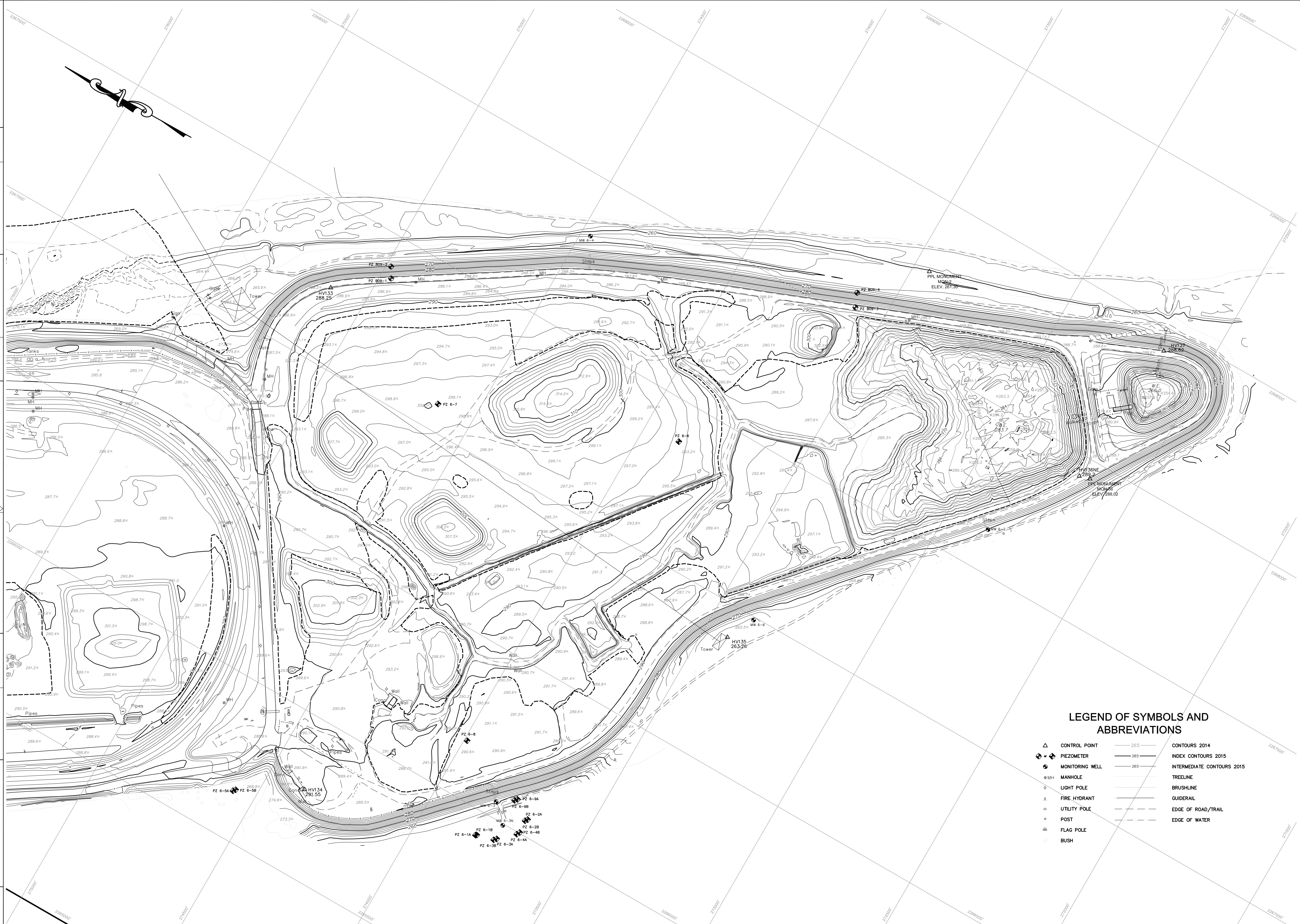


L.R. Kimball

NOTES:

1. From ground control surveys dated May 20 – 21, 2015 by L.R. Kimball, aerial photography dated April 2, 2015 by Air Photographics, Inc. and from the following plans:

   – Brunner Island SES, Ash Basin No. 6 2014 Mapping for 2013 Operational Report by PPL Services

2. Horizontal control is based on NAD83, PA South Coordinates provided by Talen Energy Services.

3. Vertical control is based on NAVD88 datum provided by Talen Energy Services.

4. 2014 Contours are depicted in the main pond and polishing pond as light grey. The 2015 contours are darker.

NO.	DATE	ACCT.	REVISION	BY	REVIEWED	APPROVED
18	6/15	-	ADDED 2014 ASH CONTOURS & CHANGED TITLE TO REFLECT 2014 RESULTS	JAW	AL	-
17	6/14	750189	ADDED 2014 ASH CONTOURS & CHANGED TITLE TO REFLECT 2013 RESULTS	JTE	AL	BRW
16	6/13	750189	ADDED 2013 ASH CONTOURS & CHANGED TITLE TO REFLECT 2012 RESULTS	JTE	AL	BRW
15	6/12	750639	ADDED 2012 ASH CONTOURS & CHANGED TITLE TO REFLECT 2011 RESULTS	JTE	AL, BRW	GDH
14	6/11	701046	ADDED 2011 ASH CONTOURS & CHANGED TITLE TO REFLECT 2010 RESULTS	JTE	JLS	ADS

ACCT- -
SCALE- 1"=100'
BY- JAW

BRUNNER ISLAND S.E.S.
ASH BASIN NO. 6
2015 TOPOGRAPHICAL MAPPING
FOR 2014 OPERATIONAL REPORT
EAST MANCHESTER TOWNSHIP YORK CO., PA.

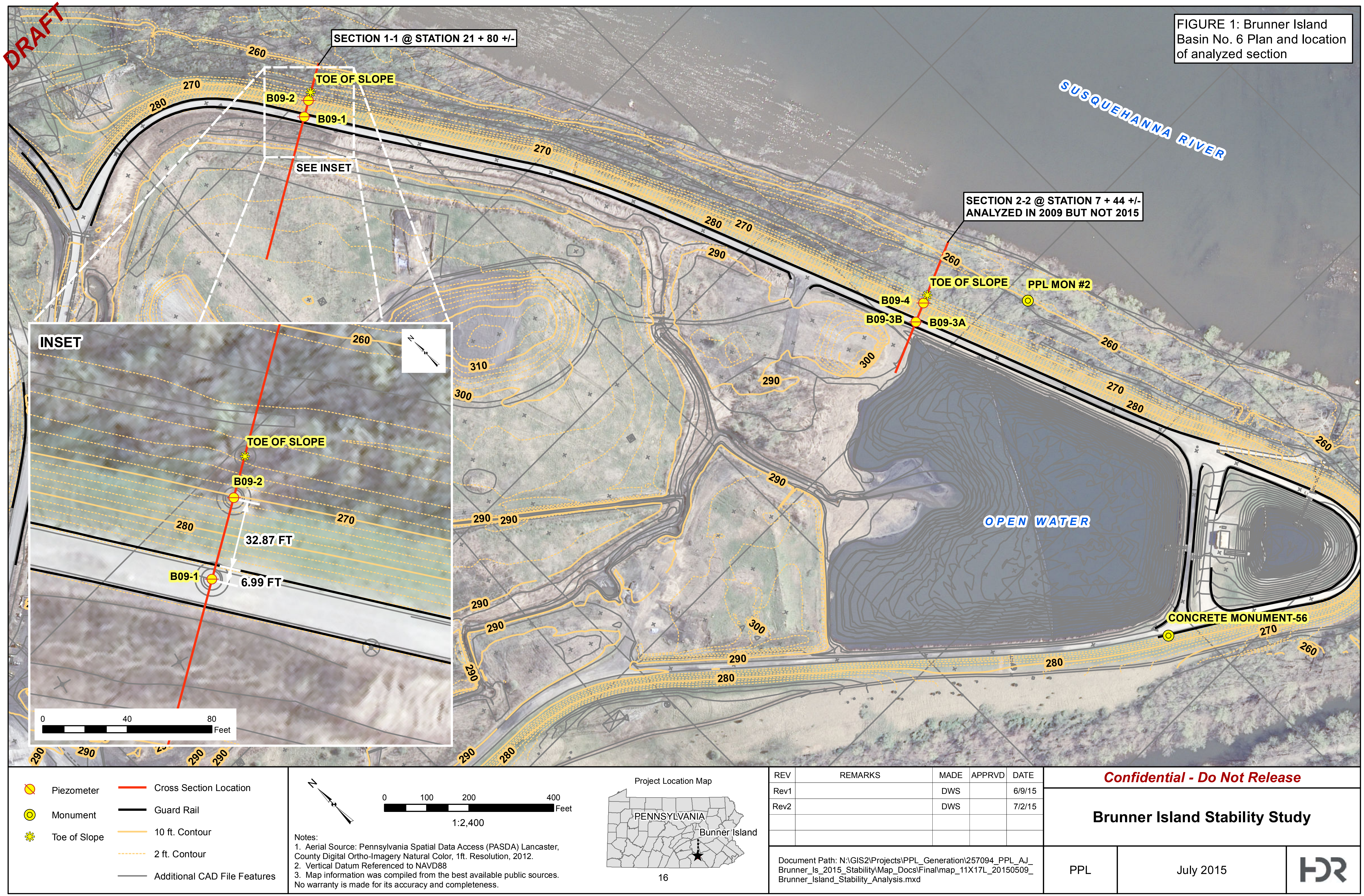
APPROVED A. D. Spear DATE 5/28/04

Talen Energy Corp

PPL DRAWING NO. 750189 SHEET NO. 1 REV. 18

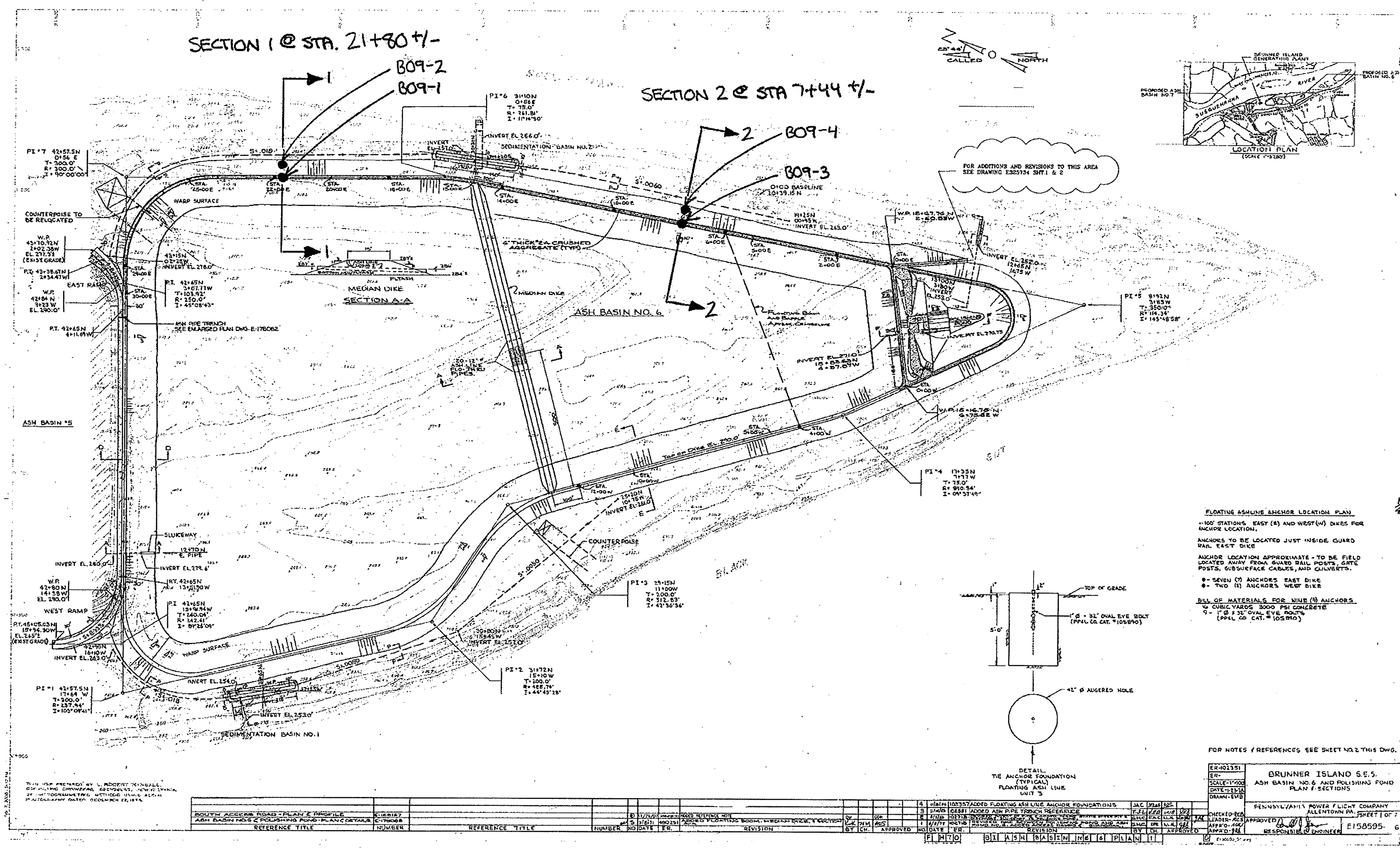
15-0068_Brunner_Ash_Basin_5-26-15.dwg

Figure A-1



DRAFT
FIGURE 1: Brunner Island Basin No. 6 Plan and location of analyzed section
SECTION 1-1 @ STATION 21 + 80 +/-
TOE OF SLOPE
B09-2
B09-1
SEE INSET
SUSQUEHANNA RIVER
SECTION 2-2 @ STATION 7 + 44 +/-
ANALYZED IN 2009 BUT NOT 2015
TOE OF SLOPE
PPL MON #2
B09-4
B09-3B
B09-3A
OPEN WATER
CONCRETE MONUMENT-56
INSET
TOE OF SLOPE
B09-2
32.87 FT
B09-1
6.99 FT
0 40 80 Feet
Piezometer
Monument
Toe of Slope
Cross Section Location
Guard Rail
10 ft. Contour
2 ft. Contour
Additional CAD File Features
0 100 200 400 Feet
1:2,400
Notes:
1. Aerial Source: Pennsylvania Spatial Data Access (PASDA) Lancaster, County Digital Ortho-Imagery Natural Color, 1ft. Resolution, 2012.
2. Vertical Datum Referenced to NAVD88
3. Map information was compiled from the best available public sources. No warranty is made for its accuracy and completeness.
Project Location Map
PENNSYLVANIA
Brunner Island
16
REV | REMARKS | MADE | APPRVD | DATE
Rev1 | DWS | 6/9/15
Rev2 | DWS | 7/2/15
Document Path: N:\GIS2\Projects\PPL_Generation\257094_PPL_AJ_Brunner_Is_2015_Stability\Map_Docs\Final\map_11X17L_20150509_Brunner_Island_Stability_Analysis.mxd
Confidential - Do Not Release
Brunner Island Stability Study
PPL
July 2015
HDR

Figure A-2



PPL/BRUNNER ISLAND
GEOTECHNICAL EXPLORATION
BORING PLAN (REVISED)
5/28/09

Figure A-3

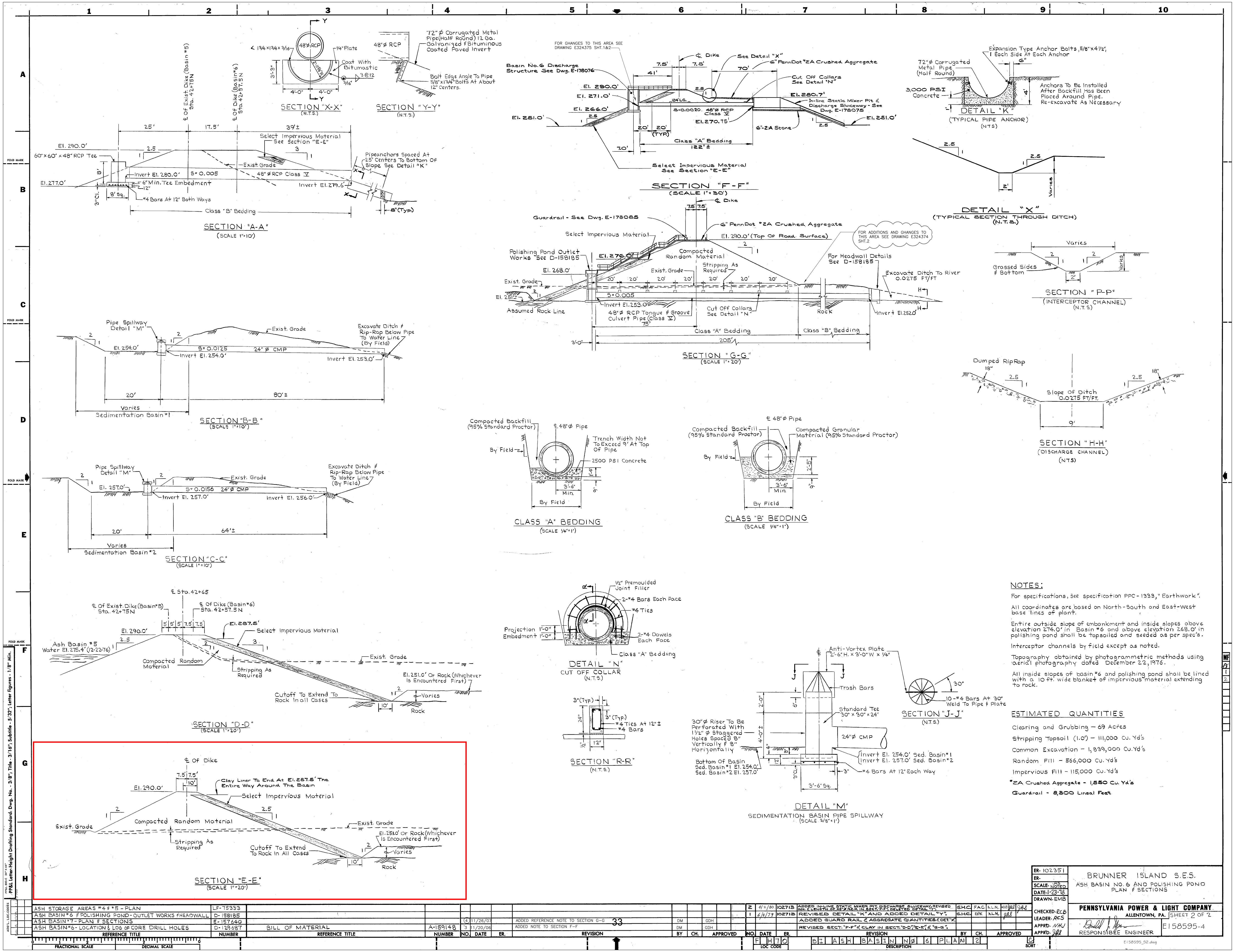
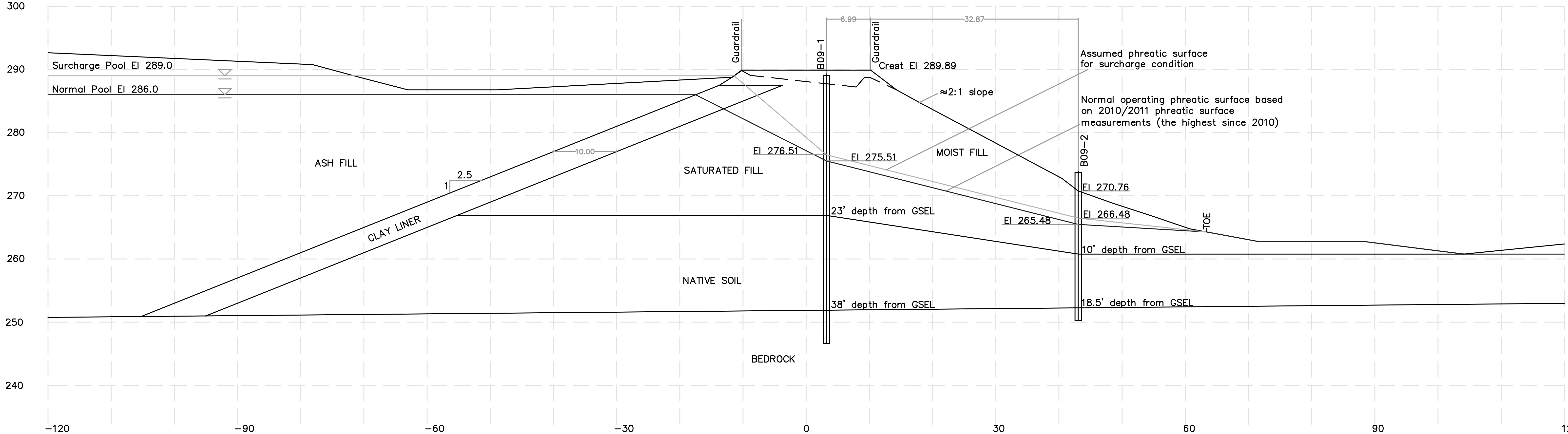

SECTION "X-X" (N.T.S.)
SECTION "Y-Y" (N.T.S.)
SECTION "A-A" (SCALE 1"=10')
SECTION "B-B" (SCALE 1"=10')
SECTION "C-C" (SCALE 1"=10')
SECTION "D-D" (SCALE 1"=20')
SECTION "E-E" (SCALE 1"=20')
SECTION "F-F" (SCALE 1"=30')
SECTION "G-G" (SCALE 1"=20')
DETAIL "K" (TYPICAL PIPE ANCHOR) (NTS)
DETAIL "X" (TYPICAL SECTION THROUGH DITCH) (N.T.S.)
SECTION "P-P" (INTERCEPTOR CHANNEL) (N.T.S.)
SECTION "H-H" (DISCHARGE CHANNEL) (NTS)
CLASS "A" BEDDING (SCALE 1/4"=1')
CLASS "B" BEDDING (SCALE 1/4"=1')
DETAIL "N" CUT OFF COLLAR (N.T.S.)
SECTION "R-R" (N.T.S.)
SECTION "J-J" (N.T.S.)
DETAIL "M" SEDIMENTATION BASIN PIPE SPILLWAY (SCALE 3/8"=1')
NOTES:
For specifications, see specification PPC-1339, "Earthwork".
All coordinates are based on North-South and East-West base lines of plant.
Entire outside slope of embankment and inside slopes above elevation 274.0' in Basin #6 and above elevation 268.0' in polishing pond shall be topsoiled and seeded as per spec's.
Interceptor channels by field except as noted.
Topography obtained by photogrammetric methods using aerial photography dated December 22, 1976.
All inside slopes of basin #6 and polishing pond shall be lined with a 10 ft. wide blanket of impervious material extending to rock.
ESTIMATED QUANTITIES
Clearing and Grubbing – 69 Acres
Stripping Topsoil (1.0') – 111,000 Cu. Yd's
Common Excavation – 1,339,000 Cu. Yd's
Random Fill – 566,000 Cu. Yd's
Impervious Fill – 115,000 Cu. Yd's
#2A Crushed Aggregate – 1,550 Cu. Yd's
Guardrail – 8,300 Lineal Feet
BRUNNER ISLAND S.E.S.
ASH BASIN NO. 6 AND POLISHING POND PLAN & SECTIONS
PENNSYLVANIA POWER & LIGHT COMPANY
ALLENTOWN, PA.
SHEET 2 OF 2
E158595-4
RESPONSIBLE ENGINEER
ASH STORAGE AREAS #4 & #5 – PLAN LF-75333
ASH BASIN #6 & POLISHING POND-OUTLET WORKS & HEADWALL D-158185
ASH BASIN #7 – PLAN & SECTIONS E-157640
ASH BASIN #6 – LOCATION & LOG OF CORE DRILL HOLES D-128687
BILL OF MATERIAL A-159148
REFERENCE TITLE
NUMBER
REVISION
33

Figure A-4

UPDATED SECTION 1–1



<u>NOTES:</u>

Surface topography from 2015 aerial survey (NAVD88 and state plane)
- Adjusted up 0.76 feet to convert to Plant datum (NGVD29)
- Crest assumed to be level at El 289.89 feet based on 2015 piezometer survey

Normal WSEL assumed to be El 286.0 ft

Surcharge WSEL assumed to be El 289.0 feet

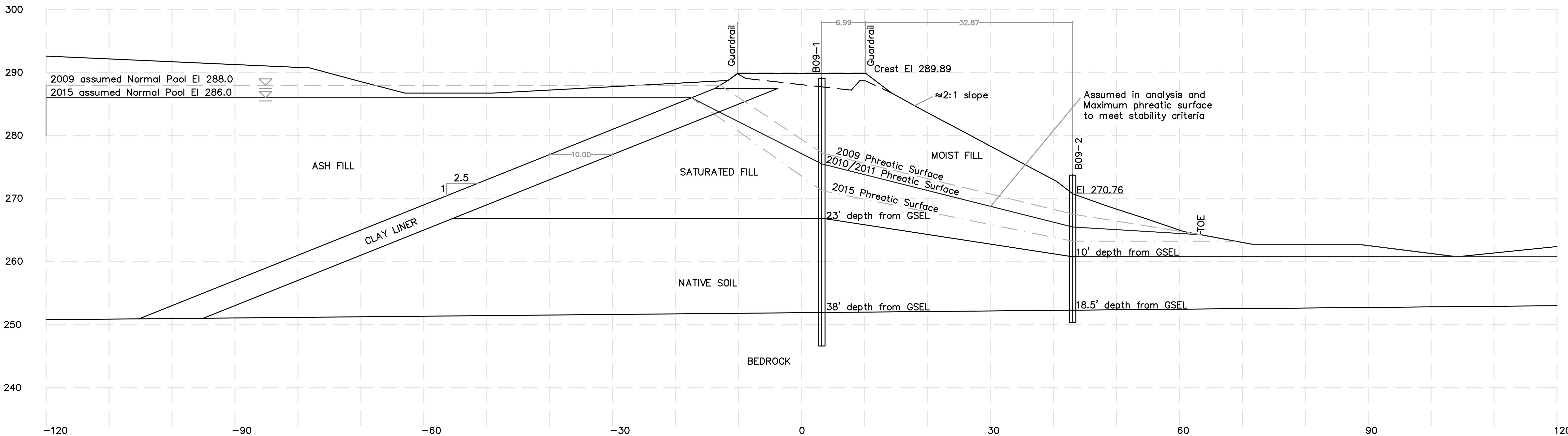
Stratigraphy and piezometric levels from boring logs, historical piezometer data, and Dwg E158595–4

Assume same material properties as 2009 analysis:

Material	Unit Weight (pcf)	Cohesion (psf)	Friction Angle
Ash Fill	90	0	30
Clay Liner	130	0	30
Moist Fill	125	0	37
Saturated Fill	135	0	37
Native Soil	130	0	30
Bedrock	160	2000	45

Figure A-5

UPDATED SECTION 1-1: Historic Piezometric Levels



NOTES:
The 2010/2011 phreatic surface was conservatively chosen for the 2015 analysis. This is also the maximum phreatic surface through which stability of the embankment will satisfy EPA stability criteria.

Piezometer	2009 Level	2010/2011 level	2015 level
B09-1	277.26	275.51	271.26
B09-2	267.53	265.48	263.21

Figure A-6

Appendix B. Analysis Results

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Figure B1-1: Normal Pool, Deep Failure Surface

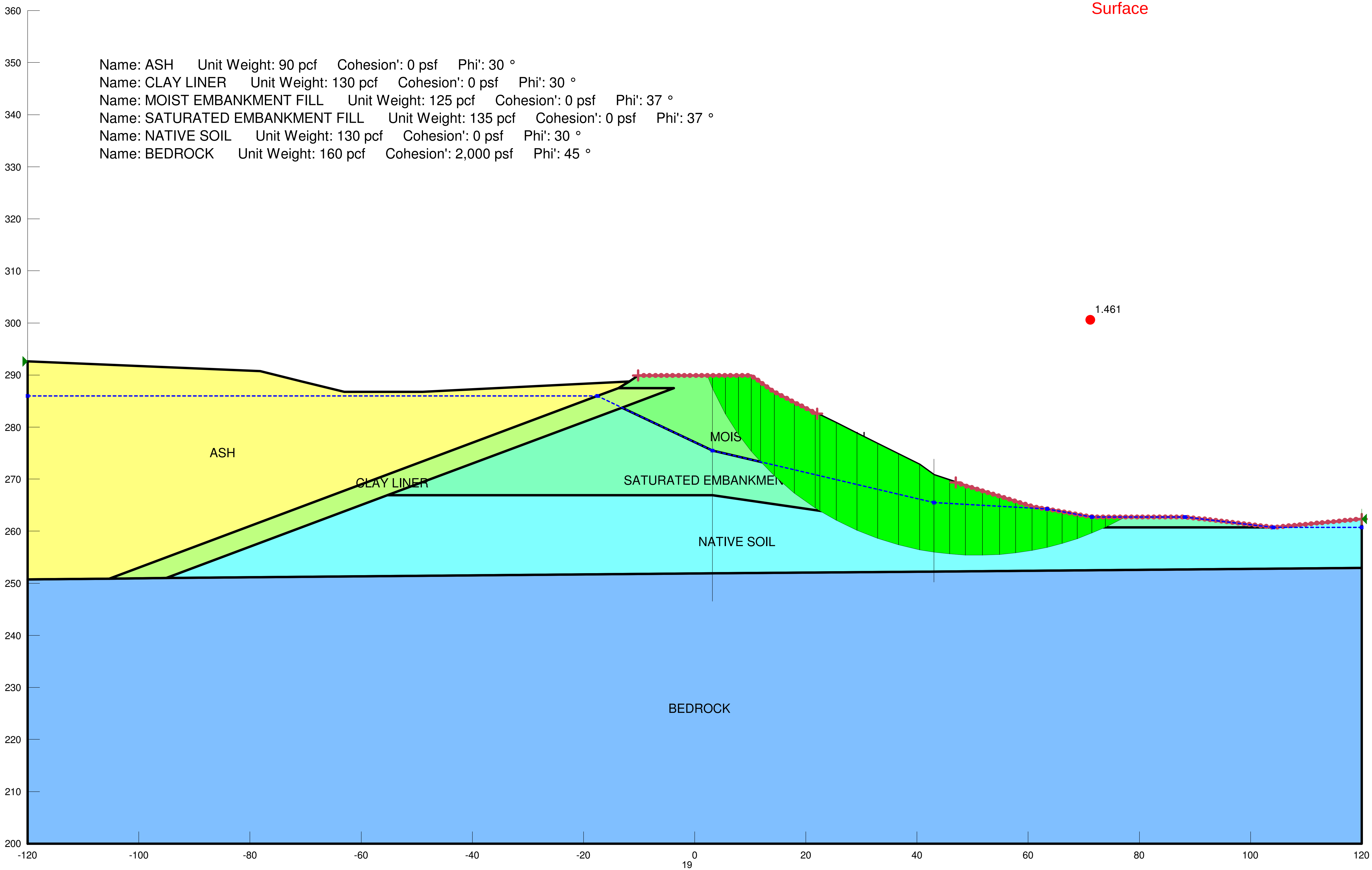


FIGURE B1-2: Surcharge

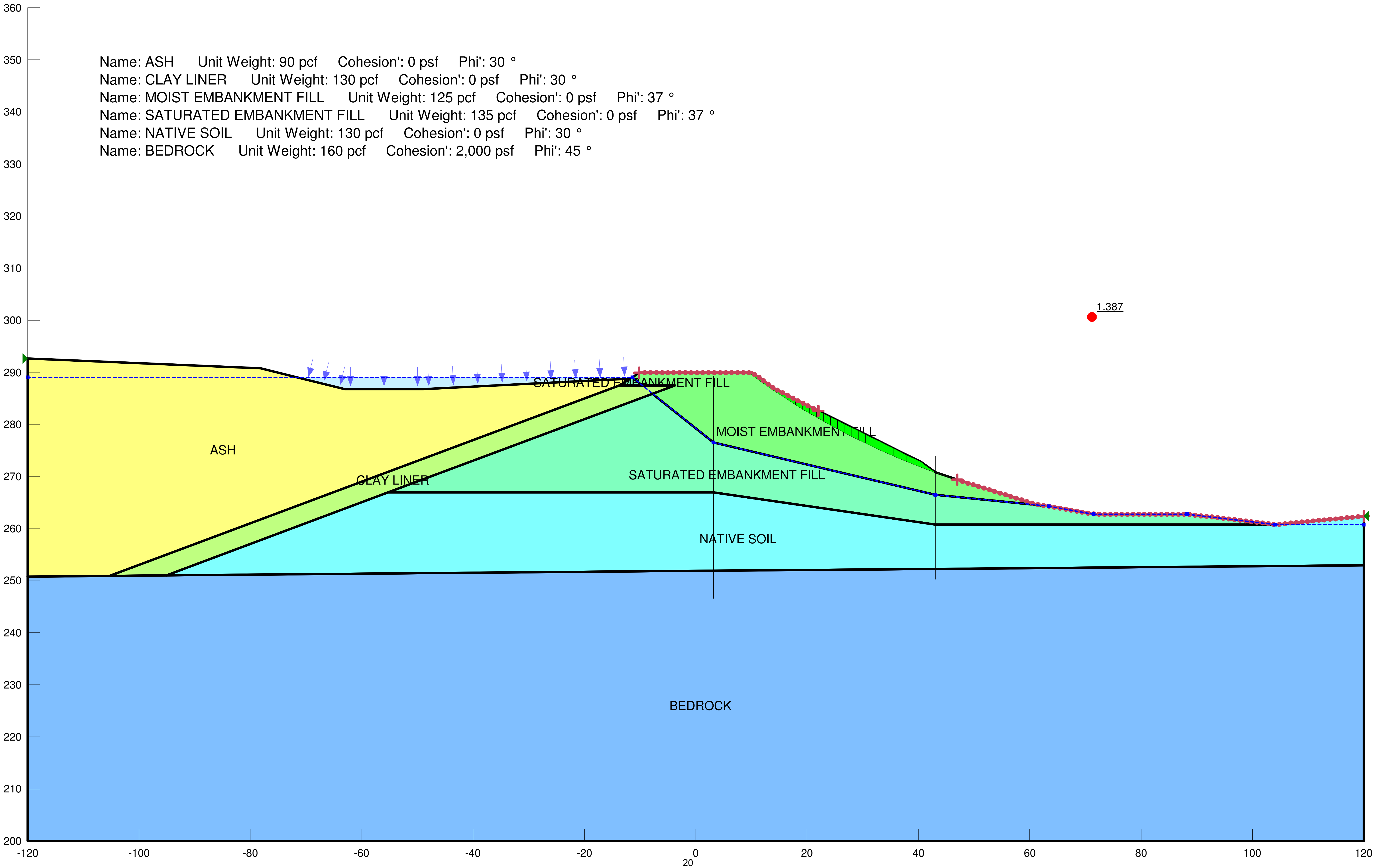


FIGURE B1-3: Liquefaction Load Case 1

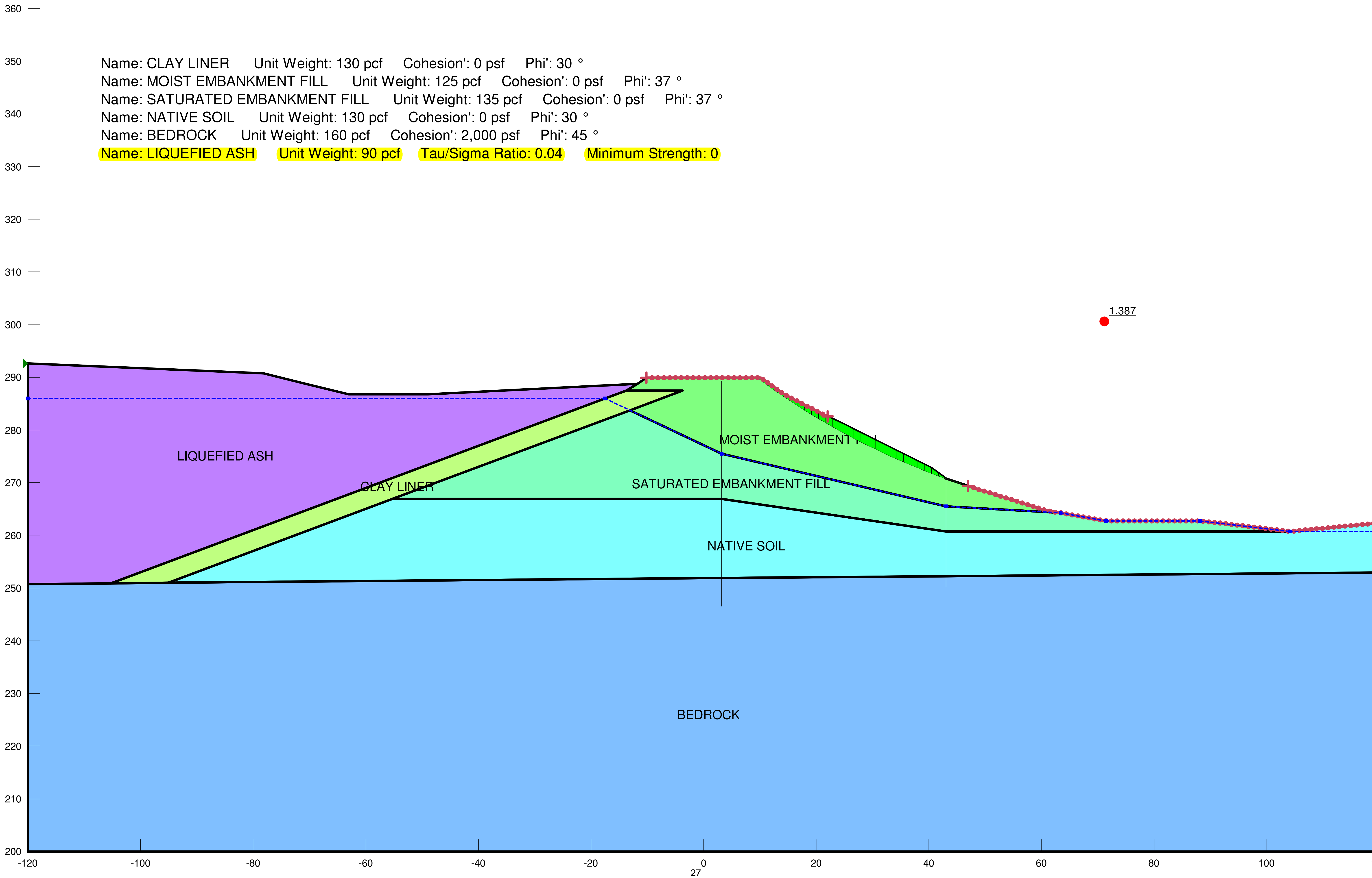


Figure B1-4: Liquefaction Load Case 2

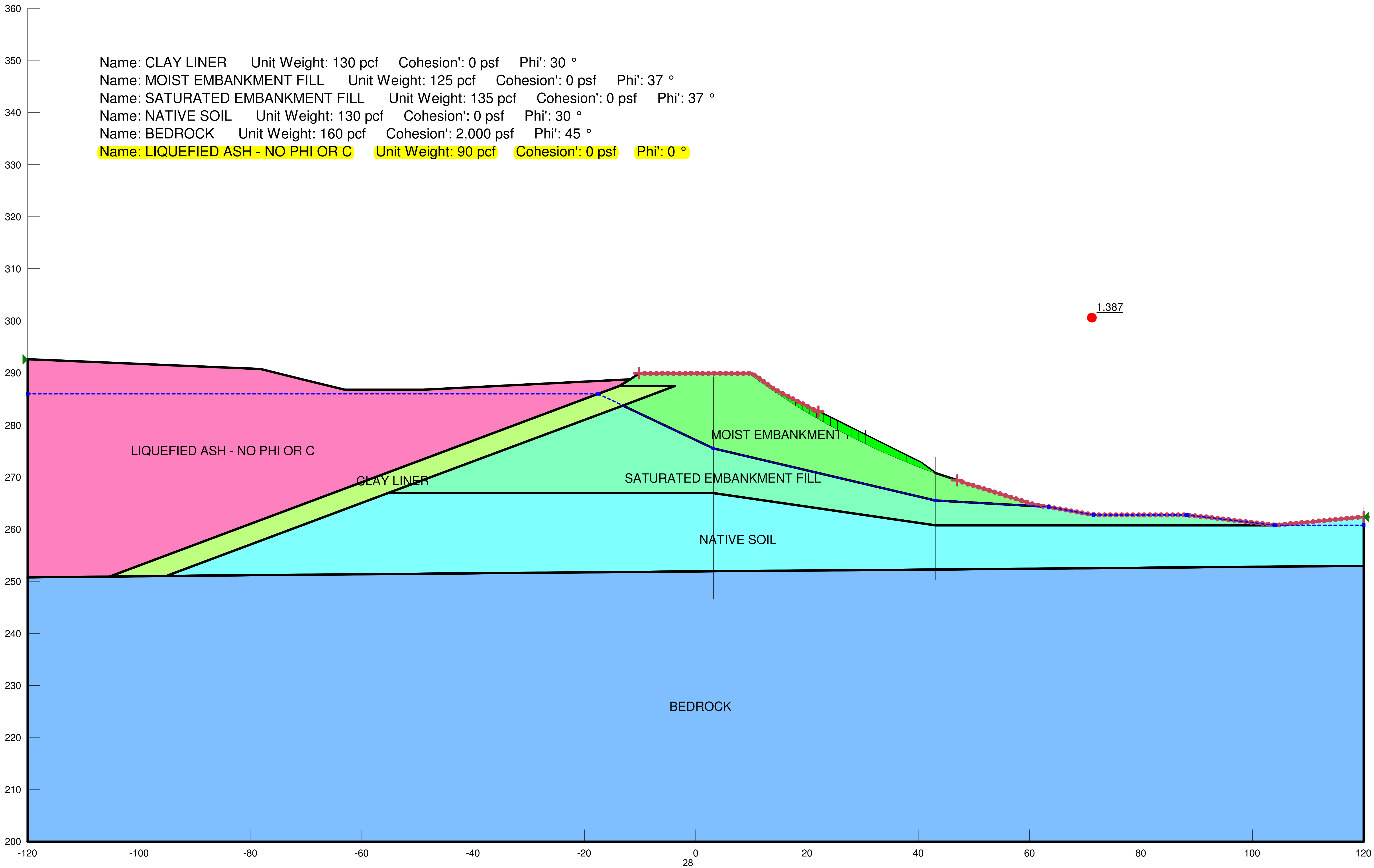


FIGURE B1-5: Seismic (PGA = 0.08g)

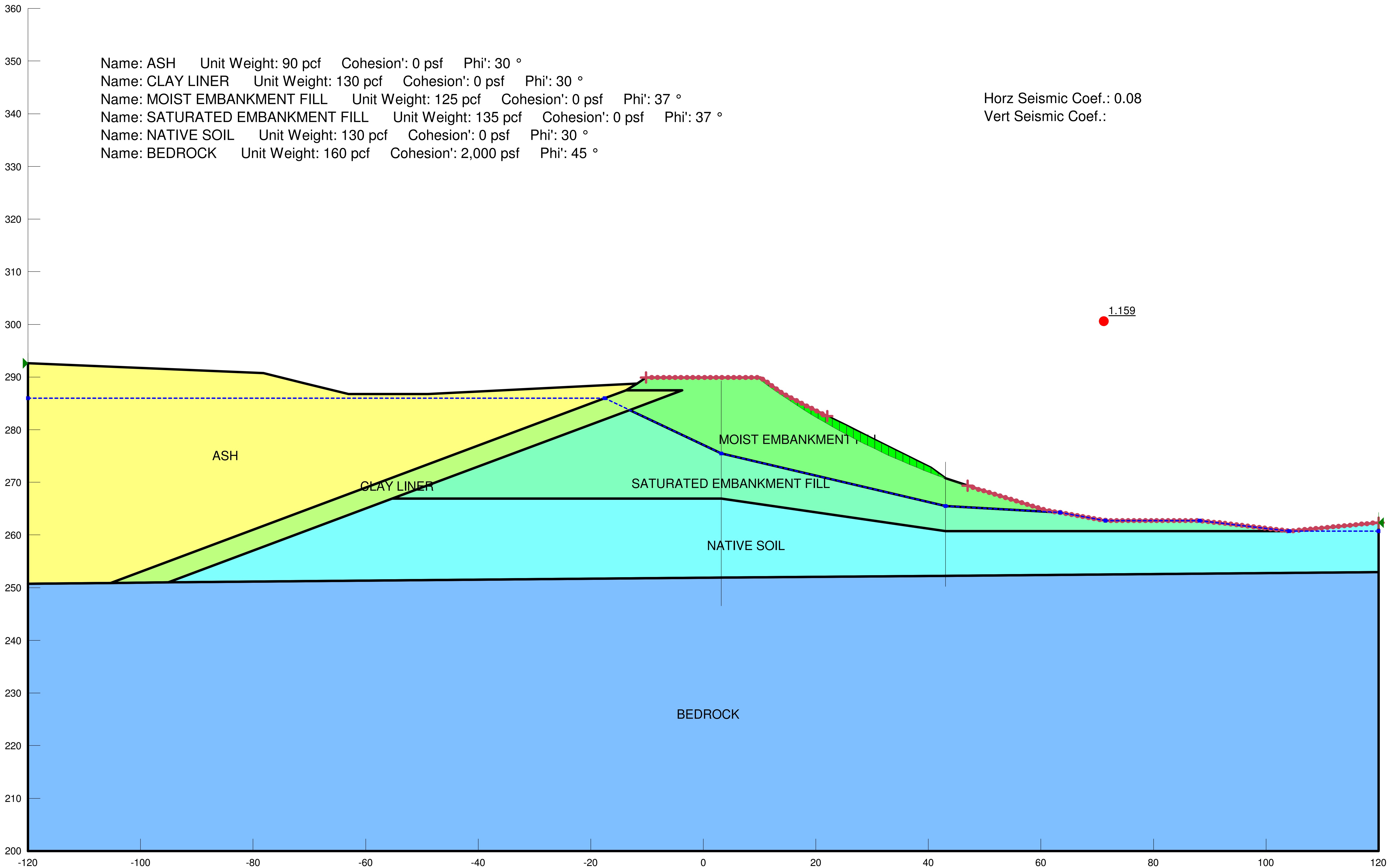
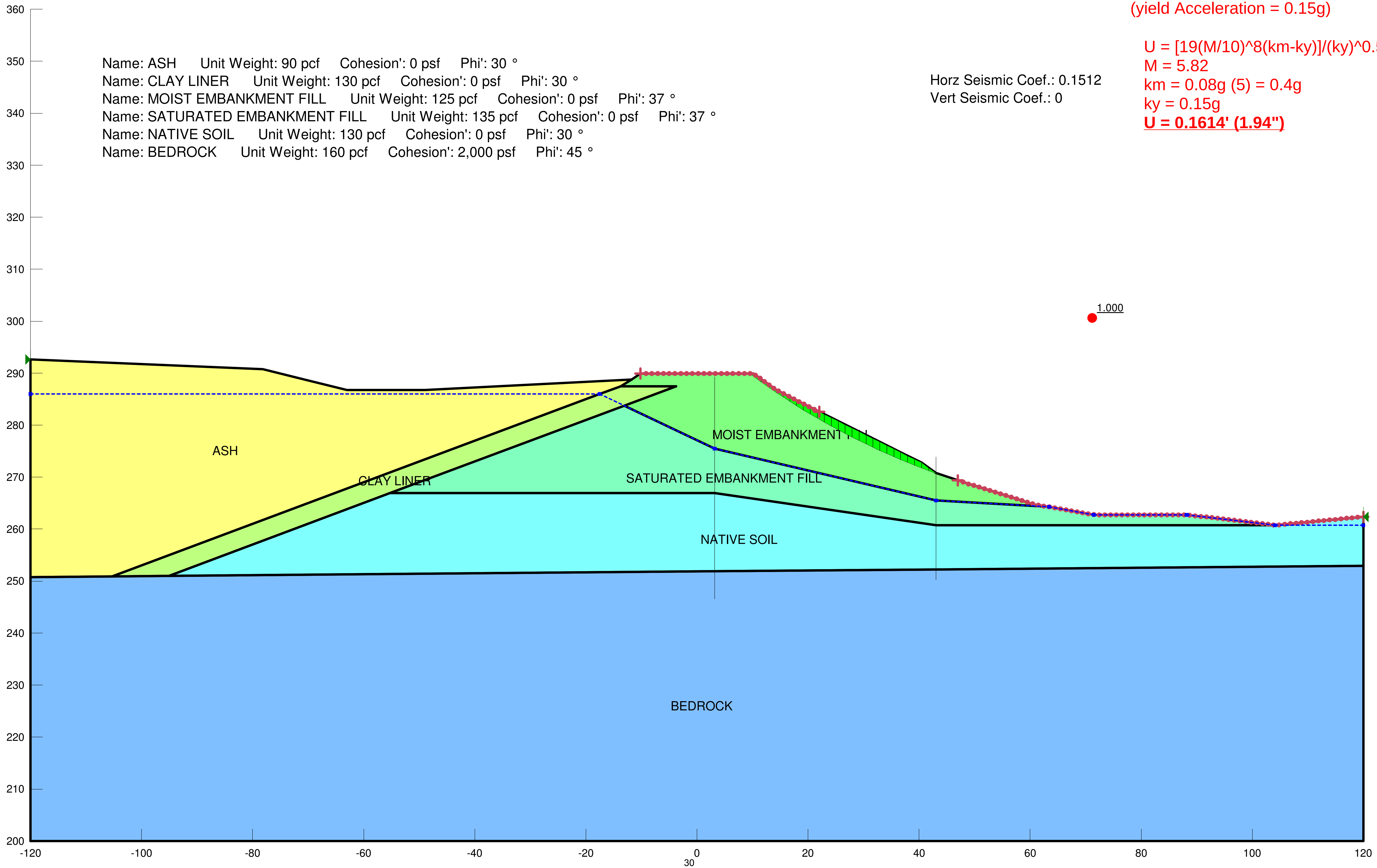


FIGURE B1-6: Seismic Deformation
(yield Acceleration = 0.15g)



Appendix C. Geotechnical Data

Appendix C1. 2009 Boring Logs

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HDR	Project : Brunner Island Ash Basin #6 Geotechnical Exploration Project Location: York Haven, PA Project Number: 106864	Boring: **B09-1** Sheet: **1 of 2** Location: Sta. 21+80 - Crest Coordinates: , Ground Surface Elevation: ± 290 ft. MSL Datum:
Client: PPL Allentown, PA		

Boring Contractor:	CGC Geoservices
Boring Foreman:	Dan Bowles
Drilling Method:	4-1/4" HSA/SPT
Core Barrel:	N/A
Drilling Equipment:	Acker Soilmax
Boring Logged By:	BRR
Dates Started: 6/10/09	Finished: 6/10/09

Water Level Observations

	Date	Time	Casing (ft)	Water (ft)	Caved (ft)
Completion (Borehole)	6/10/09	4:50 pm	43.3	39.4	
Completion (Piezometer)	6/11/09	1:00 pm		12.6	
24 to 72 Hours	6/12/09	9:00 am		11.8	
Extended Reading	6/15/09	7:30 am		12.3	
Extended Reading	6/18/09	7:45 am		12.5	

Depth & Elevation (Feet)	Sampling Type	Sample Name	Data	Rec. (in.)	Material Description & Classification	N-Value, blows/ft	Laboratory Tests & Comments
	SS	1 - SS	41+19+17+24 N=36	20"	AGGREGATE BASE COURSE (6 inch) : Silty Sand, fine to coarse, contains gravel **FILL** : Gravelly Silt with Sand, hard, brown, contains zones of silty clay (Qp = 4.5+ tsf)		Soil classifications based on Visual-Manual Procedure in general accordance with ASTM D 2488
	SS	2 - SS	20+15+16+28 N=31	19"	**FILL** : Silty Clay, hard, brown, contains trace sand and gravel		
5 285.0	SS	3 - SS	12+21+17+12 N=38	18"	**FILL** : Silty Sand with Gravel, dense, brown		Obtained bulk sample #1 from auger cuttings (Approx. depth: 4' - 10')
	SS	4 - SS	32+42+38+36 N=80	20"	**FILL** : Gravelly Silt with fine to coarse Sand, hard, brown, contains trace clay and zones of silty clay (Qp = 4.0-4.5+ tsf)		
	SS	5 - SS	45+50/3"	8"	Continued:	50/3" >>	
10 280.0	SS	6 - SS	9+23+27+15 N=50	19"	**FILL** : Gravelly Silt with fine to coarse Sand, hard, brown, contains trace clay		
	SS	7 - SS	27+16+31+23 N=47	18"	**FILL** : Silty Clay with Gravel and Sand, hard, brown, contains zones of gravelly silt (Qp = 4.5+ tsf)		
15 275.0	SS	8 - SS	14+11+10+11 N=21	17"	Continued: very stiff, contains zones of gravelly silt and moist silty sand (Qp = 4.0-4.5+ tsf)		Obtained bulk sample #2 from auger cuttings (Approx. depth: 15' - 19') Wet zone at approx. 17'
	SS	9 - SS	28+23+24+24 N=47	17"	**FILL** : Fine Sandy Silt with Gravel, hard, brown, contains trace of medium to coarse sand and clay		Drilled through cobles and boulders at approx. 19'
20 270.0	SS	10 - SS	6+12+19+20 N=31	18"	**FILL** : Fine Sandy Silt, medium stiff, brown, contains trace medium to coarse sand **FILL** : Silty Sand with Gravel, dense, brown, moist		
	SS	11 - SS	23+19+10+10 N=29	18"	**FILL** : Silty Sand/Sandy Silt , dense/hard, brown, contains gravel, portions moist to wet **NATURAL SOIL**: Clayey Silt (CL-ML), very stiff, brown to gray brown (Qp = 4.5+ tsf)		Fill/Natural Soil contact approx. at 23'
25 265.0	SS	12 - SS	12+11+5+4 N=16	18"	**NATURAL SOIL**: Silty Sand/Sandy Silt (SM/ML), medium dense/very stiff, brown, contains coarse sand and gravel with zones of silty clay, portions moist to wet (Qp = 4.0 - 4.5+ tsf)		
	SS	13 - SS	4+5+6+9 N=11	20"	**NATURAL SOIL**: Clayey Silt (CL-ML), stiff, brown to brown gray, contains portions of silty clay and traces of small roots (Qp = 2.0 - 3.0+		
30 260.0	SH	14 - SH		8.5"			Push 3" Shelby Tube 13" (Refusal)

(Continued)

Notes: 1. SPT performed with automatic safety hammer. 2. Installed 1" diameter piezometer with screening interval approx. from 19' to 24'.
3. Obtained bulk samples from auger cuttings (Approx. depth: 4' - 10' and 15' - 19').

LOG OF BORING WITHOUT CASING BLOWS BRUNNERISLAND_200906.GPJ DTA.GDT 8/7/09

HDR

Project :	Brunner Island Ash Basin #6 Geotechnical Exploration
Project Location:	York Haven, PA
Project Number:	106864
Client:	PPL Allentown, PA

Boring: B09-1

Location: Sta. 21+80 - Crest
Coordinates: ,
Ground Surface Elevation: ± 290 ft. MSL
Datum:

Boring Contractor: CGC Geoservices
Boring Foreman: Dan Bowles
Drilling Method: 4-1/4" HSA/SPT
Core Barrel: N/A
Drilling Equipment: Acker Soilmax
Boring Logged By: BRR
Dates Started: 6/10/09 Finished: 6/10/09

Water Level Observations

	Date	Time	Casing (ft)	Water (ft)	Caved (ft)
Completion (Borehole)	6/10/09	4:50 pm	43.3	39.4	
Completion (Piezometer)	6/11/09	1:00 pm		12.6	
24 to 72 Hours	6/12/09	9:00 am		11.8	
Extended Reading	6/15/09	7:30 am		12.3	
Extended Reading	6/18/09	7:45 am		12.5	

Depth & Elevation (Feet)	Sampling: Type	Sample Name	Data	Rec. (in.)	Material Description & Classification	Lithology	PL / MC / LL; N-Value, blows/ft (0 – 50 – 100)	Laboratory Tests & Comments	Well
					tsf, Tor = 0.75 kg/cm2)				
					NATURAL SOIL: Silty Clay (CL), brown, contains trace sand (Qp = 4.0 - 4.5+ tsf, Tor = 0.7-0.9 kg/cm2)*(Continued)*				
35 / 255.0	SS	15 - SS	4+7+9+11 N=16	22"	**NATURAL SOIL**: Silty Clay (CL), very stiff, brown, contains some small roots with softer material near root zones (Qp = 4.0 - 4.5 tsf, Near root zone: Qp = 2.0 tsf)				
	SS	16 - SS	50/5"	5"	**PARTIALLY WEATHERED ROCK (PWR)** Sampled as brown silty fine sand with portions of sandy silt and pieces of brown sandstone, dry		>>		
40 / 250.0									
	SS	17 - SS	50/4"	4"	Continued: Sampled as dark brown mudstone with saturated brown silty fine sand above sample		>>		

BORING TERMINATED AT 43.3 FEET (SPT Refusal).
Safety Hammer used for SPT.
Bottom of Boring at 43.3 feet.

Notes: 1. SPT performed with automatic safety hammer. 2. Installed 1" diameter piezometer with screening interval approx. from 19' to 24'. 3. Obtained bulk samples from auger cuttings (Approx. depth: 4' - 10' and 15' - 19').

LOG OF BORING WITHOUT CASING BLOWS BRUNNERISLAND_200906.GPJ DTA.GDT 8/7/09

HDR

Project : Brunner Island Ash Basin #6
Geotechnical Exploration

Project Location: York Haven, PA

Project Number: 106864

Client: PPL
Allentown, PA

Boring: **B09-2**

Sheet: **1 of 1**

Location: Sta. 21+80 - Downstream Slope
Coordinates: ,
Ground Surface Elevation: ± 271.1 ft. MSL
Datum:

Boring Contractor:	CGC Geoservices
Boring Foreman:	Dan Bowles
Drilling Method:	ROTARY/SPT
Core Barrel:	N/A
Drilling Equipment:	Skid
Boring Logged By:	BRR
Dates Started: 6/12/09	Finished: 6/16/09

Water Level Observations

	Date	Time	Casing (ft)	Water (ft)	Caved (ft)
Completion (Piezometer)	6/16/09	1:45 pm		3.8	
24 to 72 Hours	6/17/09	9:15 am		5.5	
24 to 72 Hours	6/18/09	7:45 am		5.1	
Extended Reading	6/25/09	9:00 am		5.6	

Depth & Elevation (Feet)	Type	Sample Name	Data	Rec. (in.)	Material Description & Classification	Lithology	PL / MC / LL; N-Value, blows/ft (0 / 50 / 100)	Laboratory Tests & Comments	Well
	SS	1 - SS	2+2+5+6 N=7	10"	**FILL** : Gravelly Silt with fine to coarse Sand, medium stiff, brown, moist to wet			Soil classifications based on Visual-Manual Procedure in general accordance with ASTM D 2488	
	SS	2 - SS	6+11+12+13 N=23	21"	Continued: **FILL** : Fine Sandy Silt, very stiff, brown, dry to moist				
5 266.1	SS	3 - SS	30+44+50/4"	13"	**FILL** : Gravelly Silt with fine to coarse Sand, hard, brown, moist to wet		50/4" >>	Cobbles or gravel at 4.5 ft	
	SS	4 - SS	24+42+27+8 N=69	14"	**FILL** : Silty Sand with Gravel, very dense, brown, wet, contains portions of moist to wet gravelly silt with sand				
10 261.1	SS	5 - SS	7+8+11+17 N=19	14"	**NATURAL SOIL**: Silty Clay (CL), very stiff, brown to gray brown, contains portions of clayey silt (Qp = 3.5 tsf)				
	SS	6 - SS	13+16+20+26 N=36	4"	Continued: hard (Qp = 3.0 tsf)				
15 256.1	SH	7 - SH		13"	Continued: contains trace sand (Qp = 3.0-3.5 tsf, Tor = 0.8 kg/cm2)			Set 4" Casing to 15 ft Push 3" Shelby Tube 19" (Refusal)	
	SS	8 - SS	10+23+57+50/3" N=80	18"	Continued: hard (Qp = 3.0 tsf)		50/3"		
					NATURAL SOIL: Shale, gray			Difficult drilling starting approx. at 18.0'	
20 251.1					**PARTIALLY WEATHERED ROCK (PWR)** Sampled as red brown silt with trace sand (Weathered mudstone)				
					TRICONE REFUSAL AT 20.5 FEET. Safety Hammer used for SPT. Bottom of Boring at 20.5 feet.				

Notes: 1. SPT performed with automatic safety hammer. 2. Installed 1" diameter piezometer with screening interval approx. from 5' to 10'.

LOG OF BORING WITHOUT CASING BLOWS BRUNNERISLAND_200906.GPJ DTA.GDT 8/7/09

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Appendix C2. Corrected Blow Counts

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STANDARD PENETRATION TEST BLOW COUNT CORRECTION

Project:	Brunner 6				
Client:	PPL				
		El Dam Base	252	feet	
Boring:	B09-1	Crest El	290	feet	
Moist Overburden Unit Weight (total):	0.125	kcf	assumed	Top of boring	290 feet
Sat. Overburden Unit Weight Total):	0.135	kcf	assumed		
Depth to Water Table:	14.38	ft			
Hammer Type:	Safety Hammer				
Hammer efficiency, $E_{M:}$	0.6				

References: McCarthy, D.F. 2007. Essentials of Soil Mechanics and Foundati
Idriss and Bouldanger 2008
Youd & Idriss 2001

Sample	classification	Depth (feet)	% fines	Uncorrected Blow Counts 0-6"	6-12"	12-18"	18-24"	Raw SPT N (bpf)	Static Pore Pressure (ksf)	Borehole Dia. Correction C_B
S1		1		41	19	17	24	**36**	0.00	1.00
S2		3		20	15	16	28	**31**	0.00	1.00
S3	Silty clayey gravel with sand (GC-GM)	5	18.2	12	21	17	12	**38**	0.00	1.00
S4	Silty clayey gravel with sand (GC-GM)	7	18.2	32	42	38	36	**80**	0.00	1.00
S6		11		9	23	27	15	**50**	0.00	1.00
S7		13		27	16	31	23	**47**	0.00	1.00
S8	Silty clayey sand with gravel (SC-SM)	16	30.8	14	11	10	11	**21**	0.10	1.00
S9	Silty clayey sand with gravel (SC-SM)	18	30.8	28	23	24	24	**47**	0.23	1.00
S10		21		6	12	19	20	**31**	0.41	1.00
S11		23		23	19	10	10	**29**	0.54	1.00
S12		26		12	11	5	4	**16**	0.73	1.00
S13		28		4	5	6	9	**11**	0.85	1.00
S15		34		4	7	9	11	**16**	1.22	1.00

STANDARD PENETRATION TEST BLOW COUNT CORRECTION CONT'D

Boring: B09-1

Average

ons

Spoon Liner Correction C_S	Rod Length Correction C_R	**Corrected Blowcount $(N)_{60}$ (bpf)**	Eff. Overburden σ'v (ksf)	Overburden Correction C_N	Hammer Type Correction C_E	**Corrected Blowcount $(N_1)_{60}$ (bpf)**
1.30	0.75	**35**	0.13	1.70	1.00	**60**
1.30	0.75	**30**	0.38	1.56	1.00	**47**
1.30	0.75	**37**	0.58	1.36	1.00	**51**
1.30	0.75	**78**	0.88	1.07	1.00	**84**
1.30	0.80	**52**	1.38	1.09	1.00	**57**
1.30	0.80	**49**	1.63	1.06	1.00	**52**
1.23	0.85	**22**	1.92	1.04	1.00	**23**
1.30	0.85	**52**	2.06	1.01	1.00	**52**
1.30	0.95	**38**	2.28	0.98	1.00	**37**
1.30	0.95	**36**	2.42	0.96	1.00	**34**
1.16	0.95	**18**	2.64	0.90	1.00	**16**
1.10	0.95	**11**	2.79	0.86	1.00	**10**
1.15	1.00	**18**	3.22	0.82	1.00	**15**

STANDARD PENETRATION TEST BLOW COUNT CORRECTION CONT'D

Boring: B09-1

Clean Sand Correction $\Delta(N_1)_{60cs}$	**Corrected Blowcount $(N_1)_{60cs}$ (bpf)**	CRR M7.5	Tot. Overburden σv (ksf)	Depth z (meters)	rd	Interpolated amax	CSR	MSF	Assumed Relative Density Factor, f	Kσ	Kα	**Triggering FS**
0	**60**	8452386	0.13	0.3048	1.004097	0.392215958	0.255985	1.552463	0.7	2.336657	1	**Above H20**
0	**47**	96.83596	0.38	0.9144	0.994726	0.375447874	0.242754	1.552463	0.7	1.680578	1	**Above H20**
4	**55**	29361.06	0.58	1.524	0.984535	0.358679791	0.229536	1.552463	0.7	1.478317	1	**Above H20**
4	**88**	3.92E+41	0.88	2.1336	0.973575	0.341911707	0.21637	1.552463	0.7	1.303364	1	**Above H20**
0	**57**	270220.4	1.38	3.3528	0.94956	0.30837554	0.190334	1.552463	0.7	1.138092	1	**Above H20**
0	**52**	2656.868	1.63	3.9624	0.936616	0.291607456	0.177531	1.552463	0.7	1.08246	1	**Above H20**
5	**28**	0.39434	2.02	4.8768	0.916195	0.26645533	0.167057	1.552463	0.7	1.030409	1	**3.7760351**
5	**58**	697985.1	2.29	5.4864	0.901997	0.249687247	0.162441	1.552463	0.7	1.008064	1	**6724493.3**
0	**37**	1.94127	2.69	6.4008	0.879971	0.224535121	0.151718	1.552463	0.7	0.978128	1	**19.429685**
0	**34**	0.949236	2.96	7.0104	0.864888	0.207767037	0.142728	1.552463	0.7	0.960164	1	**9.9136201**
0	**16**	0.16331	3.37	7.9248	0.84181	0.182614912	0.127355	1.552463	0.7	0.93569	1	**1.8627311**
0	**10**	0.117381	3.64	8.5344	0.826205	0.165846828	0.116232	1.552463	0.7	0.920787	1	**1.443617**
0	**15**	0.156085	4.45	10.3632	0.778928	0.115542577	0.080729	1.552463	0.7	0.881524	1	**2.645981**

NO LIQUEFACTION (FOS>1.2)

STANDARD PENETRATION TEST BLOW COUNT CORRECTION

Boring:	B09-2		
Moist Overburden Unit Weight:	0.125	kcf	assumed
Sat. Overburden Unit Weight:	0.135	kcf	assumed
Depth to Water Table:	5.28	ft	
Hammer Type:	Safety Hammer		
Hammer efficiency, $E_{M:}$	0.6		

El Dam Base	252	feet
Crest El	290	feet
Top of boring	271	feet

References: McCarthy, D.F. 2007. Essentials of Soil Mechanics and Foundati
Idriss and Bouldanger 2008
Youd & Idriss 2001

Sample	classification	Depth (feet)	% fines	Uncorrected Blow Counts				Raw SPT N (bpf)	Static Pore Pressure (ksf)	Borehole Dia. Correction C_B
				0-6"	6-12"	12-18"	18-24"			
S1		1		2	2	5	6	**7**	0.00	1.00
S2		3		6	11	12	13	**23**	0.00	1.00
S4		8		24	42	27	8	**69**	0.17	1.00
S5		11		7	8	11	17	**19**	0.36	1.00
S6		13		13	16	20	26	**36**	0.48	1.00
S8		18		10	23	57	50	**80**	0.79	1.00

STANDARD PENETRATION TEST BLOW COUNT CORRECTION CONT'D

Average

Boring: B09-2

ons

Spoon Liner Correction C_S	Rod Length Correction C_R	**Corrected Blowcount $(N)_{60}$ (bpf)**	Eff. Overburden σ'v (ksf)	Overburden Correction C_N	Hammer Type Correction C_E	**Corrected Blowcount $(N_1)_{60}$ (bpf)**
1.10	0.75	**6**	0.13	1.70	1.00	**10**
1.30	0.75	**22**	0.38	1.70	1.00	**38**
1.30	0.75	**67**	0.86	1.11	1.00	**75**
1.25	0.80	**19**	1.08	1.31	1.00	**25**
1.30	0.80	**37**	1.22	1.16	1.00	**44**
1.30	0.85	**88**	1.58	1.02	1.00	**90**

STANDARD PENETRATION TEST BLOW COUNT CORRECTION CONT'D

Boring: B09-2

Clean Sand Correction $\Delta(N_1)_{60cs}$	**Corrected Blowcount $(N_1)_{60cs}$ (bpf)**	CRR M7.5	Tot. Overburden σv (ksf)	Depth z (meters)	rd	Interpolated amax	CSR	MSF	Assumed Relative Density Factor, f	Kσ	Kα	**Triggering FS**
0	**10**	0.116796	0.13	0.3048	1.004097	0.231829237	0.151306	1.552463	0.7	2.336657	1	**Above H20**
0	**38**	2.350511	0.38	0.9144	0.994726	0.215061154	0.139052	1.552463	0.7	1.680578	1	**Above H20**
0	**75**	1.7E+20	1.03	2.4384	0.967823	0.173140944	0.13048	1.552463	0.7	1.3113	1	**2.653E+21**
0	**25**	0.287736	1.43	3.3528	0.94956	0.147988819	0.121661	1.552463	0.7	1.225216	1	**4.4986042**
0	**44**	16.27346	1.70	3.9624	0.936616	0.131220735	0.111419	1.552463	0.7	1.179532	1	**267.45484**
0	**90**	6.36E+45	2.38	5.4864	0.901997	0.089300526	0.078601	1.552463	0.7	1.0909	1	**1.371E+47**

NO LIQUEFACTION (FOS>1.2)

Appendix C3. Laboratory testing data

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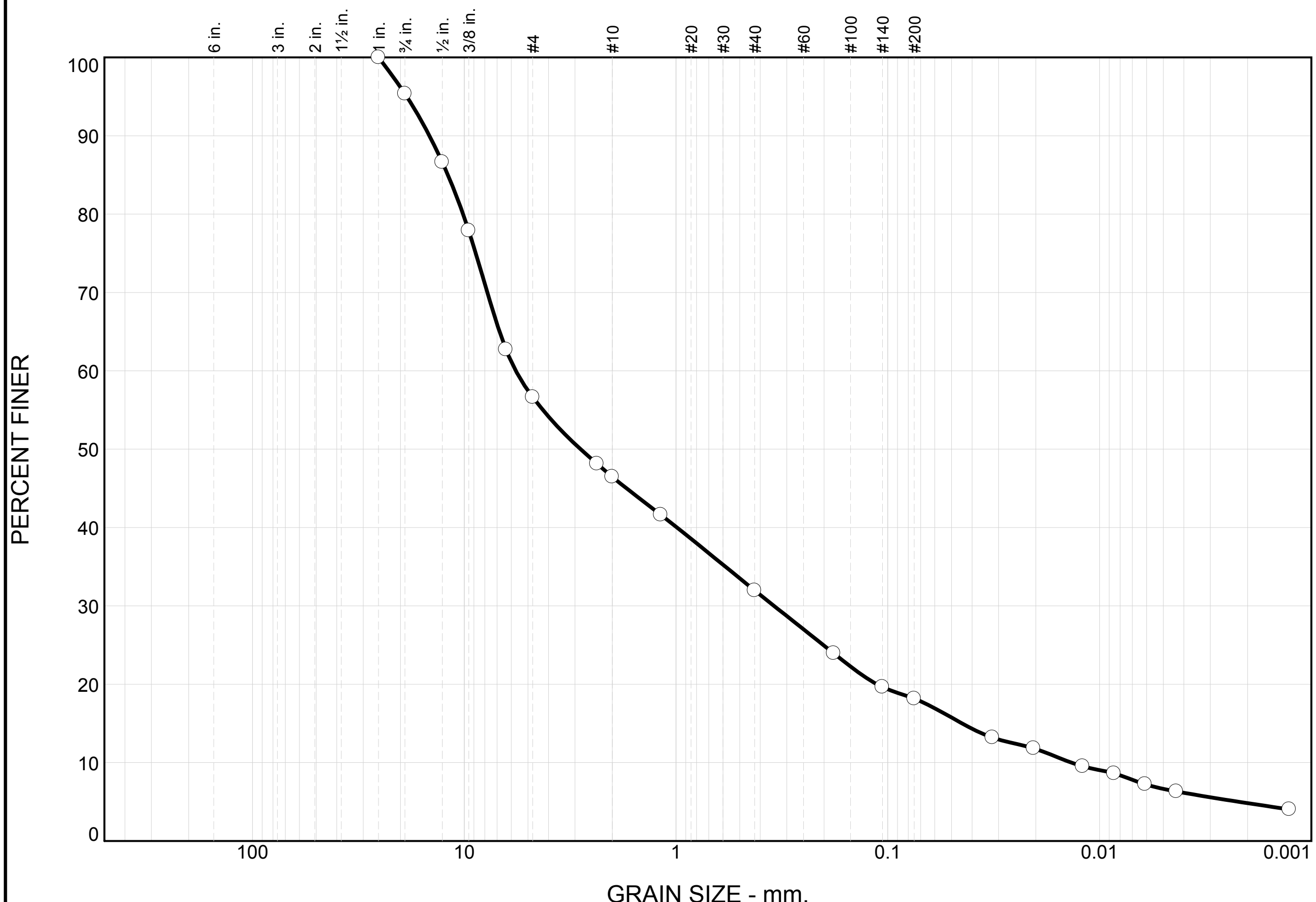
Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	4.6	38.8	10.1	14.5	13.8	11.6	6.6

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	20	13	11.9430	5.7115	2.8216	0.3446	0.0446	0.0136	1.53	420.49

Material Description	USCS	AASHTO
○ USCS Classification: Silty clayey gravel with sand	GC-GM	

Project No. 08146.ZA **Client:** PPL Generation, LLC.

Project: PPL Brunner Island, Ash Basin No. 6

○ **Loc.:** Ash Basin No. 6 **Depth:** 4.0'-10.0' **Sample No.:** B09-1 Bulk No. 1

Remarks:
○ B09-01 Bulk Sample No. 1
4.0'-10.0'

CUMBERLAND GEOSCIENCE CONSULTANTS

Carlisle, Pennsylvania

GRAIN SIZE DISTRIBUTION TEST DATA

7/22/2009

Client: PPL Generation, LLC.
Project: PPL Brunner Island, Ash Basin No. 6
Project Number: 08146.ZA
Location: Ash Basin No. 6
Depth: 4.0'-10.0' **Sample Number:** B09-1 Bulk No. 1
Material Description: USCS Classification: Silty clayey gravel with sand
Liquid Limit: 20 **Plastic Limit:** 13
USCS Classification: GC-GM
Testing Remarks: B09-01 Bulk Sample No. 1
4.0'-10.0'

Sieve Test Data

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
1326.20	0.00	0.00	1"	0.00	100.0
			3/4"	61.63	95.4
			1/2"	177.30	86.6
			3/8"	293.22	77.9
			1/4"	494.40	62.7
			#4	575.00	56.6
			#8	687.80	48.1
			#10	709.80	46.5
50.00	0.00	0.00	#16	5.22	41.6
			#40	15.61	32.0
			#80	24.23	24.0
			#140	28.86	19.7
			#200	30.46	18.2

Hydrometer Test Data

Hydrometer test uses material passing #10
Percent passing #10 based upon complete sample = 46.5
Weight of hydrometer sample =50
Hygroscopic moisture correction:
Moist weight and tare = 56.05
Dry weight and tare = 55.86
Tare weight = 31.05
Hygroscopic moisture = 0.8%
Table of composite correction values:

Temp., deg. C:	23.0	27.5	26.0	25.0	22.0	19.5
Comp. corr.:	-7.6	-8.6	-8.3	-8.0	-7.4	-6.9

Meniscus correction only = 0.5
Specific gravity of solids = 2.75
Hydrometer type = 152H
Hydrometer effective depth equation: L = 16.294964 - 0.164 **x Rm**

CUMBERLAND GEOSCIENCE CONSULTANTS

Hydrometer Test Data (continued)

Elapsed Time (min.)	Temp. (deg. C.)	Actual Reading	Corrected Reading	K	Rm	Eff. Depth	Diameter (mm.)	Percent Finer
2.00	23.0	22.0	14.4	0.0128	22.5	12.6	0.0321	13.2
5.00	23.0	20.5	12.9	0.0128	21.0	12.9	0.0205	11.8
15.00	23.0	18.0	10.4	0.0128	18.5	13.3	0.0120	9.5
30.00	23.0	17.0	9.4	0.0128	17.5	13.4	0.0085	8.6
60.00	23.0	15.5	7.9	0.0128	16.0	13.7	0.0061	7.2
120.00	23.0	14.5	6.9	0.0128	15.0	13.8	0.0043	6.3
1440.00	23.0	12.0	4.4	0.0128	12.5	14.2	0.0013	4.0

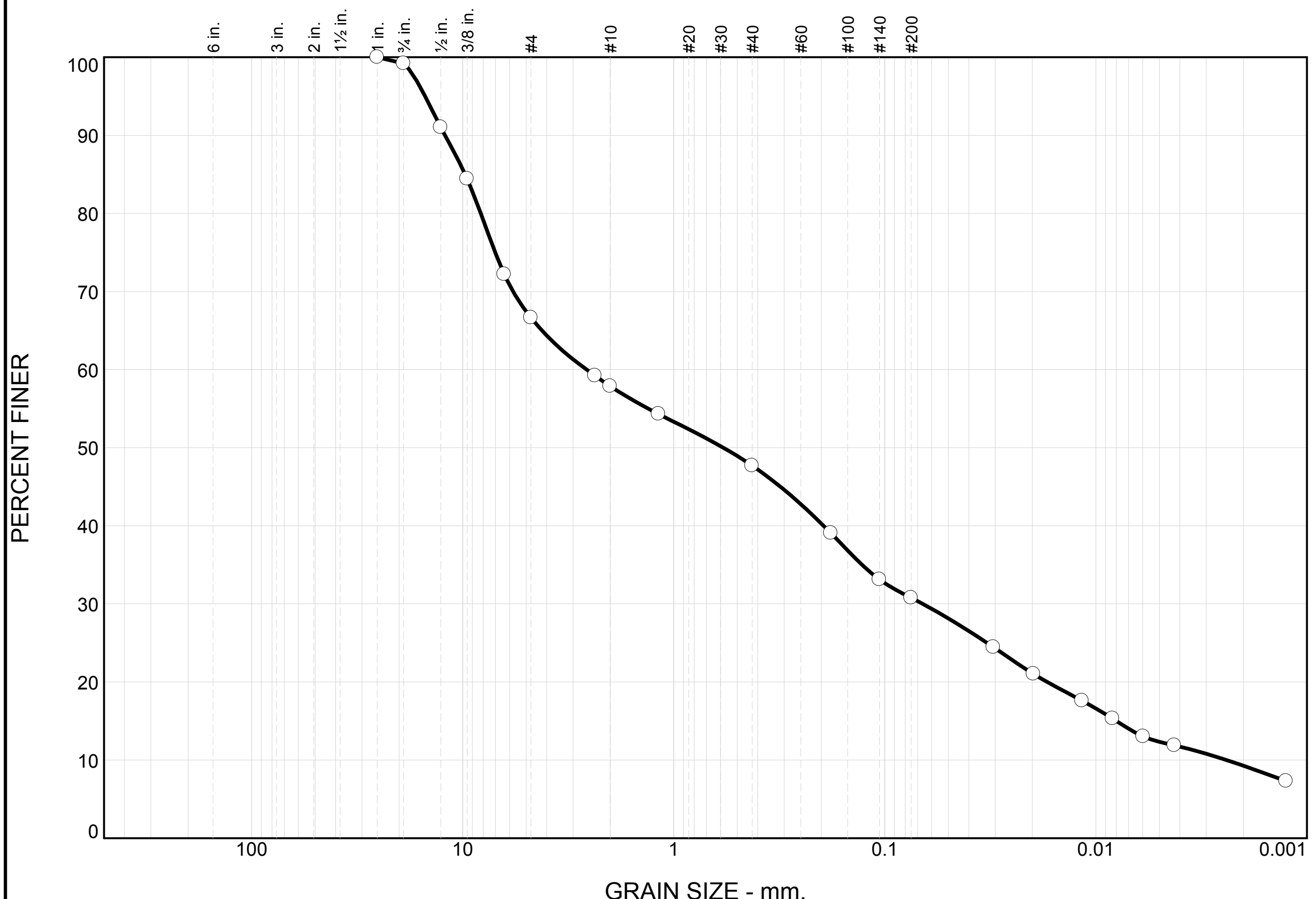
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	4.6	38.8	43.4	10.1	14.5	13.8	38.4	11.6	6.6	18.2

D_{10}	D_{15}	D_{20}	D_{30}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
0.0136	0.0446	0.1124	0.3446	2.8216	5.7115	10.1200	11.9430	14.5971	18.6778

Fineness Modulus	C_u	C_c
3.94	420.49	1.53

Particle Size Distribution Report



% +3"	% Gravel Coarse	% Gravel Fine	% Sand Coarse	% Sand Medium	% Sand Fine	% Fines Silt	% Fines Clay
0.0	0.8	32.6	8.7	10.2	16.9	18.5	12.3

LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
23	17	9.7328	2.5867	0.5823	0.0661	0.0080	0.0024	0.70	1075.34

Material Description	USCS	AASHTO
USCS Classification: Silty, clayey sand with gravel	SC-SM	

Project No. 08146.ZA **Client:** PPL Generation, LLC.

Project: PPL Brunner Island, Ash Basin No. 6

Loc.: Ash Basin No. 6 **Depth:** 15.0'-19.0' **Sample No.:** B09-1 Bulk No. 2

CUMBERLAND GEOSCIENCE CONSULTANTS

Carlisle, Pennsylvania

Remarks:
B09-1 Bulk Sample No. 2
15.0'-19.0'

GRAIN SIZE DISTRIBUTION TEST DATA

7/22/2009

Client: PPL Generation, LLC.
Project: PPL Brunner Island, Ash Basin No. 6
Project Number: 08146.ZA
Location: Ash Basin No. 6
Depth: 15.0'-19.0' **Sample Number:** B09-1 Bulk No. 2
Material Description: USCS Classification: Silty, clayey sand with gravel
Liquid Limit: 23 **Plastic Limit:** 17
USCS Classification: SC-SM
Testing Remarks: B09-1 Bulk Sample No. 2
15.0'-19.0'

Sieve Test Data

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
1471.50	0.00	0.00	1"	0.00	100.0
			3/4"	11.62	99.2
			1/2"	132.04	91.0
			3/8"	229.11	84.4
			1/4"	409.10	72.2
			#4	490.80	66.6
			#8	600.00	59.2
			#10	619.80	57.9
50.00	0.00	0.00	#16	3.08	54.3
			#40	8.79	47.7
			#80	16.26	39.1
			#140	21.39	33.1
			#200	23.41	30.8

Hydrometer Test Data

Hydrometer test uses material passing #10
Percent passing #10 based upon complete sample = 57.9
Weight of hydrometer sample =50
Hygroscopic moisture correction:
Moist weight and tare = 50.07
Dry weight and tare = 49.85
Tare weight = 25.07
Hygroscopic moisture = 0.9%
Table of composite correction values:

Temp., deg. C:	23.0	27.5	26.0	25.0	22.0	19.5
Comp. corr.:	-7.6	-8.6	-8.3	-8.0	-7.4	-6.9

Meniscus correction only = 0.5
Specific gravity of solids = 2.75
Hydrometer type = 152H
Hydrometer effective depth equation: L = 16.294964 - 0.164 **x Rm**

CUMBERLAND GEOSCIENCE CONSULTANTS

Hydrometer Test Data (continued)

Elapsed Time (min.)	Temp. (deg. C.)	Actual Reading	Corrected Reading	K	Rm	Eff. Depth	Diameter (mm.)	Percent Finer
2.00	23.0	29.0	21.4	0.0128	29.5	11.5	0.0306	24.5
5.00	23.0	26.0	18.4	0.0128	26.5	11.9	0.0197	21.0
15.00	23.0	23.0	15.4	0.0128	23.5	12.4	0.0116	17.6
30.00	23.0	21.0	13.4	0.0128	21.5	12.8	0.0083	15.3
60.00	23.0	19.0	11.4	0.0128	19.5	13.1	0.0060	13.0
120.00	23.0	18.0	10.4	0.0128	18.5	13.3	0.0042	11.9
1440.00	23.0	14.0	6.4	0.0128	14.5	13.9	0.0013	7.3

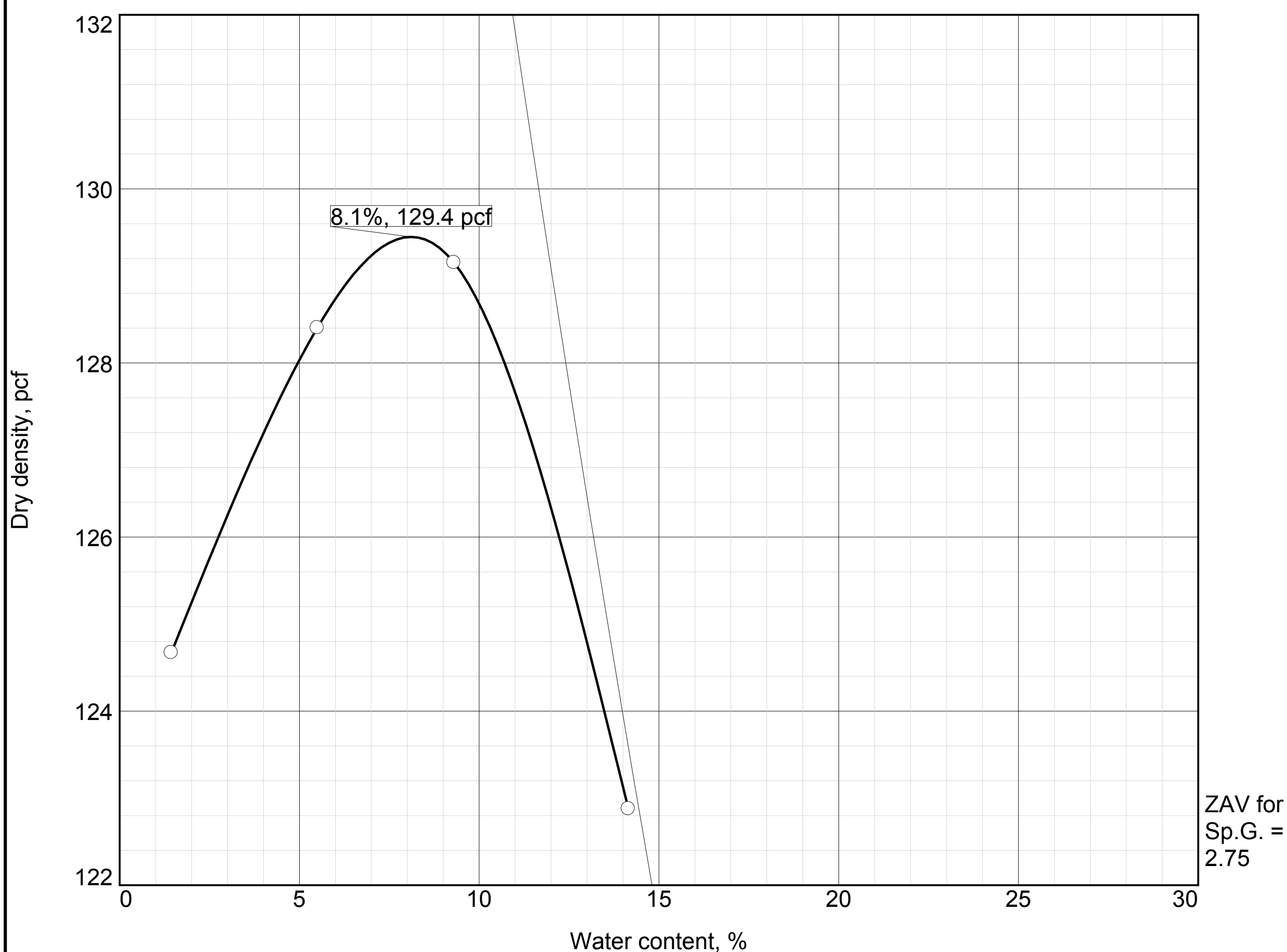
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.8	32.6	33.4	8.7	10.2	16.9	35.8	18.5	12.3	30.8

D_{10}	D_{15}	D_{20}	D_{30}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
0.0024	0.0080	0.0170	0.0661	0.5823	2.5867	8.2136	9.7328	12.1247	15.0591

Fineness Modulus	C_u	C_c
3.05	1075.34	0.70

COMPACTION TEST REPORT



Test specification: ASTM D 698-00a Method B Standard

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > 3/8 in.	% < No.200
	USCS	AASHTO						
15.0'-19.0'	SC-SM			2.75	23	7	15.6	30.8

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 129.4 pcf Optimum moisture = 8.1 %	USCS Classification: Silty, clayey sand with gravel

Project No. 08146.ZA **Client:** PPL Generation, LLC.

Project: PPL Brunner Island, Ash Basin No. 6

○ **Loc.:** Ash Basin No. 6 **Depth:** 15.0'-19.0'

CUMBERLAND GEOSCIENCE CONSULTANTS

Carlisle, Pennsylvania

Remarks:

B09-1 Bulk Sample No. 2
15.0'-19.0'

MOISTURE DENSITY TEST DATA

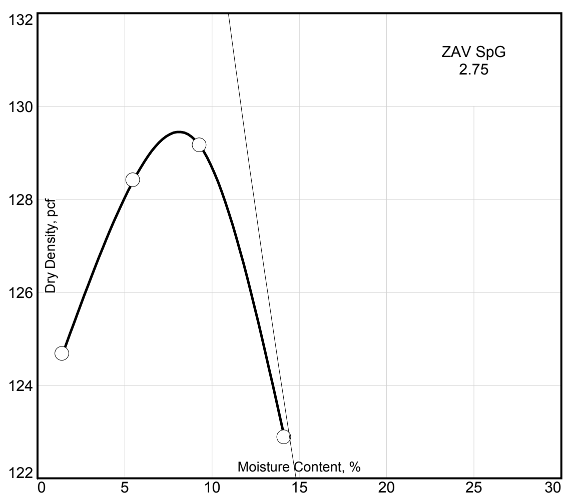
7/22/2009

Client: PPL Generation, LLC.
Project: PPL Brunner Island, Ash Basin No. 6
Project Number: 08146.ZA
Location: Ash Basin No. 6
Depth: 15.0'-19.0' **Sample Number:** B09-1 Bulk No. 2
Description: USCS Classification: Silty, clayey sand with gravel
USCS Classification: SC-SM
Liquid Limit: 23 **Plasticity Index:** 7
Testing Remarks: B09-1 Bulk Sample No. 2
15.0'-19.0'
Percent passing 3/8 in. sieve: 84.4

Test Data and Results

Test Specification:

Type of Test: ASTM D 698-00a Method B Standard
Mold Dia: 4.00 **Hammer Wt.:** 5.5 lb. **Drop:** 12 in. **Layers:** three **Blows per Layer:** 25



Point No.	1	2	3	4
Wt. M+S	13.76	14.06	14.25	14.22
Wt. M	9.54	9.54	9.54	9.54
Wt. W+T	92.2	118.6	139.4	113.7
Wt. D+T	91.2	114.1	130.2	103.4
Tare	30.8	31.7	31.1	31.2
Moist.	1.6	5.5	9.3	14.2
Wt. W+T	89.9	113.2	145.9	125.0
Wt. D+T	89.1	108.9	135.6	112.7
Tare	25.1	31.4	25.2	25.2
Moist.	1.2	5.5	9.3	14.1
Moist.*	1.4	5.5	9.3	14.1
Dry Den.*	124.7	128.4	129.2	122.9

Test Results: **Max. Dry Den.=** 129.4 pcf **Opt. Moist.=** 8.1%

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Appendix C4. Seismic Data

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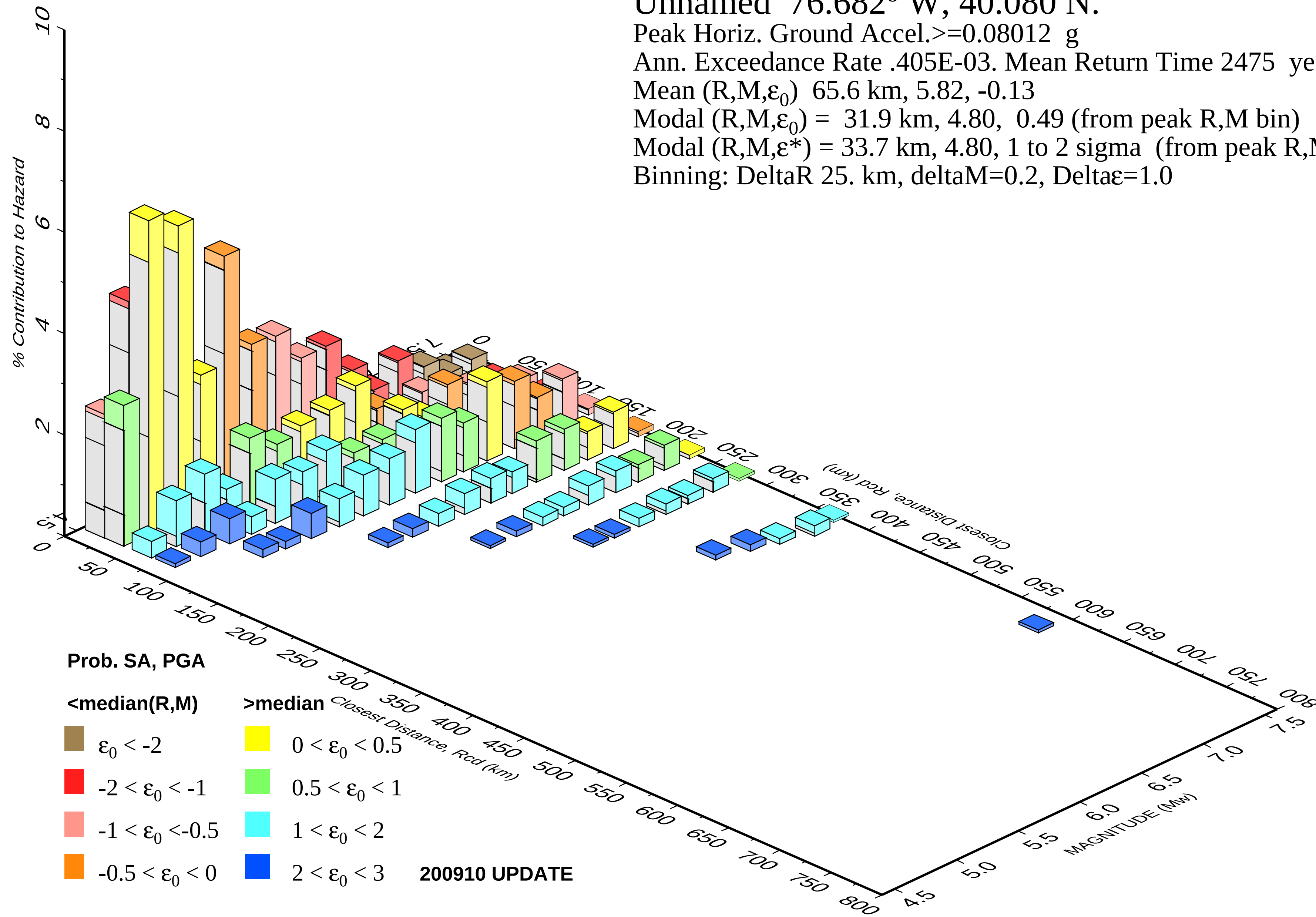
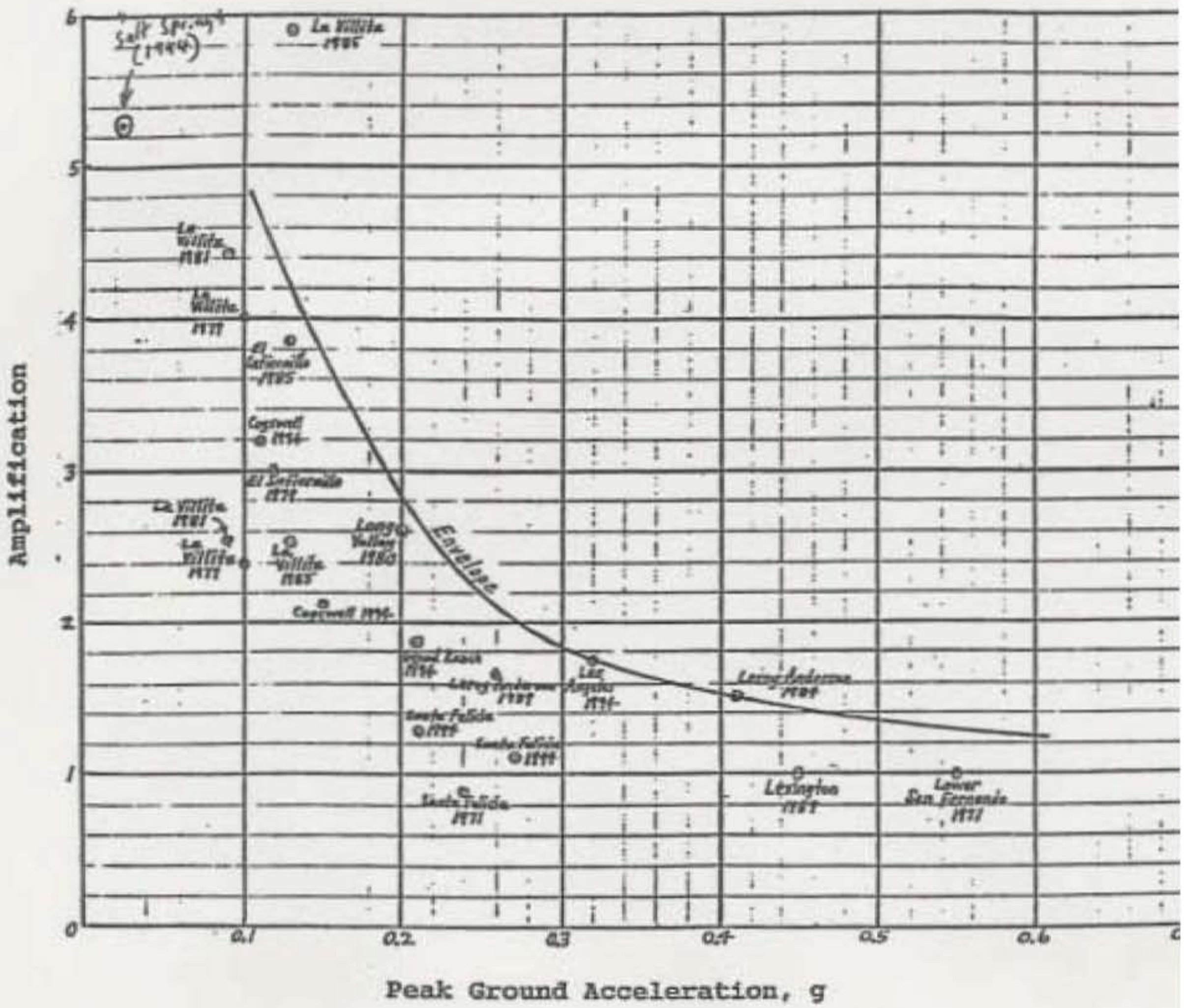
PSH Deaggregation on NEHRP BC rock
Unnamed 76.682° W, 40.080 N.

Peak Horiz. Ground Accel.>=0.08012 g
Ann. Exceedance Rate .405E-03. Mean Return Time 2475 years
Mean (R,M,ε_0) 65.6 km, 5.82, -0.13
Modal (R,M,ε_0) = 31.9 km, 4.80, 0.49 (from peak R,M bin)
Modal (R,M,ε*) = 33.7 km, 4.80, 1 to 2 sigma (from peak R,M,ε bin)
Binning: DeltaR 25. km, deltaM=0.2, Deltaε=1.0



GMT 2015 May 14 20:42:40 Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on rock with average vs= 760. m/s top 30 m. USGS CGHT PSHA2008 UPDATE Bins with lt 0.05% contrib. omitted

MEASURED RATIOS (AMPLIFICATION) OF CREST AND BASE ACCELERATIONS AT EMBANKMENT DAMS IN RESPONSE TO EARTHQUAKES



Notes: This graph represents measured accelerations at embankment dams ranging widely in size, geometry, materials, and foundation conditions.

The two plotted values for La Villita Dam for each indicated year are based on the positive and negative amplitudes from asymmetric accelerograms of crest motion.

The envelope is drawn as an upper limit of amplifications, reflecting the average of La Villita peak crest accelerations in the 1985 earthquake.

FIGURE 12 – EMBANKMENT RESPONSE BASED ON MEASURED SEISMICITY
(after Jansen and Leps)

Appendix D. Phreatic Surface Data

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PPL BRUNNER ISLAND
PIEZOMETERS
Survey Date: May 20, 2015
Date: May 26, 2015

NUMBER	NORTHING PAS(NAD83) US FEET	EASTING PAS(NAD83) US FEET	TOP CAP ELEVATION NAVD88 US FEET	TOP PVC ELEVATION NAVD88 US FEET	GROUND ELEVATION NAVD88 US FEET	TOP CAP ELEVATION NGVD29 US FEET	TOP PVC ELEVATION NGVD29 US FEET	GROUND ELEVATION NGVD29 US FEET
						Note: Plant Elevations are NGVD29		
B09-3A	273597.918'	2268045.634'	289.58'	289.22'	289.58'	290.34'	289.98'	290.34'
B09-3B	273597.918'	2268045.634'	289.58'	289.09'	289.58'	290.34'	289.85'	290.34'
B09-4	273616.784'	2268089.935'	270.80'	270.57'	268.52'	271.56'	271.33'	269.28'
502	273623.201'	2268109.541'	261.76'	TOE SLOPE		262.52'		
B09-2	274984.435'	2267399.727'	273.10'	272.97'	270.00'	273.86'	273.73'	270.76'
B09-1	274964.569'	2267365.172'	288.54'	288.30'	289.13'	289.30'	289.06'	289.89'
506	274994.464'	2267417.433'	263.51'	TOE SLOPE		264.27'		

CONTROL HELD

100	272650.588'	2267943.449'	288.02'	CONCRETE MONUMENT-56
101	273446.599'	2268268.934'	261.16'	PPL MON #2

NOTE: Horizontal and vertical control is based on PA South NAD83, State Plane Coordinates and NAVD88 Elevations provided by PPL.

Figure D-1

Client: ***PPL***
Project: ***Brunner Island Ash Basin No. 6 Geotechnical Exploration***
Subject: ***Water Level Readings***

Piezometer ID	**B09-1**		Note:
Screen Depth	19.0 -24.0	ft	TP = top of pipe
Screen Elev.	266.0 - 271.0	ft msl	GS = ground surface
Ground Surface Elev.	289.9	ft NGVD29	
Top of Pipe Elev.	289.1	ft NGVD29	
Stickup	0.0	ft	Elevations are estimated

Reading Date & Time	**Water Depth (ft) TP**	**Water Elev. (ft NGVD29)**	**Remarks**
6/10/09 4:50 PM	39.4		After boring completion
6/11/09 1:00 PM	12.6	276.5	After piezometer installed
6/12/09 9:00 AM	11.8	277.3	24 - 72 hr reading
6/15/09 7:30 AM	12.3	276.8	Extended reading
6/16/09 7:00 AM	12.3	276.8	Extended reading
6/17/09 9:15 AM	12.6	276.5	Extended reading
6/18/09 7:45 AM	12.5	276.6	Extended reading
6/25/09 12:00 AM	13.5	275.6	Extended reading
5/6/10 1:00 PM	13.6	275.5	Extended reading
5/24/11 12:00 AM	15.3	273.8	Measured by PPL emailed to HDR 05/25/11
7/28/11 12:00 AM	14.6	274.5	2011 HDR Inspection
6/7/12 1:00 PM	15.9	273.2	2012 HDR Inspection
6/18/13 11:00 AM	16.3	272.8	2013 HDR Inspection
6/27/14 2:00 PM	16.1	273.0	2014 HDR Inspection
5/20/15	17.8	271.3	Measured by PPL emailed to HDR 05/20/15

Client: ***PPL***
Project: ***Brunner Island Ash Basin No. 6 Geotechnical Exploration***
Subject: ***Water Level Readings***

Piezometer ID	**B09-2**		Note:
Screen Depth	5.0 -10.0	ft	TP = top of pipe
Screen Elev.	261.1 - 266.1	ft msl	GS = ground surface
Ground Surface Elev.	270.8	ft NGVD29	
Top of Pipe Elev.	273.7	ft NGVD29	
Stickup	3.0	ft	Elevations are estimated

Reading Date & Time	Water Depth (ft) TP	Water Elev. (ft NGVD29)	Remarks
6/16/09 1:45 PM	6.2	267.5	After piezometer installed
6/17/09 9:15 AM	7.9	265.8	24 - 72 hr reading
6/18/09 7:45 AM	7.5	266.2	24 - 72 hr reading
6/25/09 12:00 AM	8.0	265.7	Extended reading
5/6/10 1:00 PM	9.4	264.3	Extended reading
5/24/11 12:00 AM	8.3	265.5	Measured by PPL emailed to HDR 05/25/11
7/28/11 12:00 AM	10.4	263.3	2011 HDR Inspection
6/7/12 1:00 PM	9.6	263.7	2012 HDR Inspection (stickup = 2.5')
6/18/13 11:00 AM	9.9	263.4	2013 HDR Inspection (stickup = 2.5')
6/27/14 2:00 PM	10.2	263.3	2014 HDR Inspection (stickup = 2.7')
5/20/15	10.5	263.2	Measured by PPL emailed to HDR 05/20/15

Figure D-3

Figure 5. Piezometer Time-History Plot

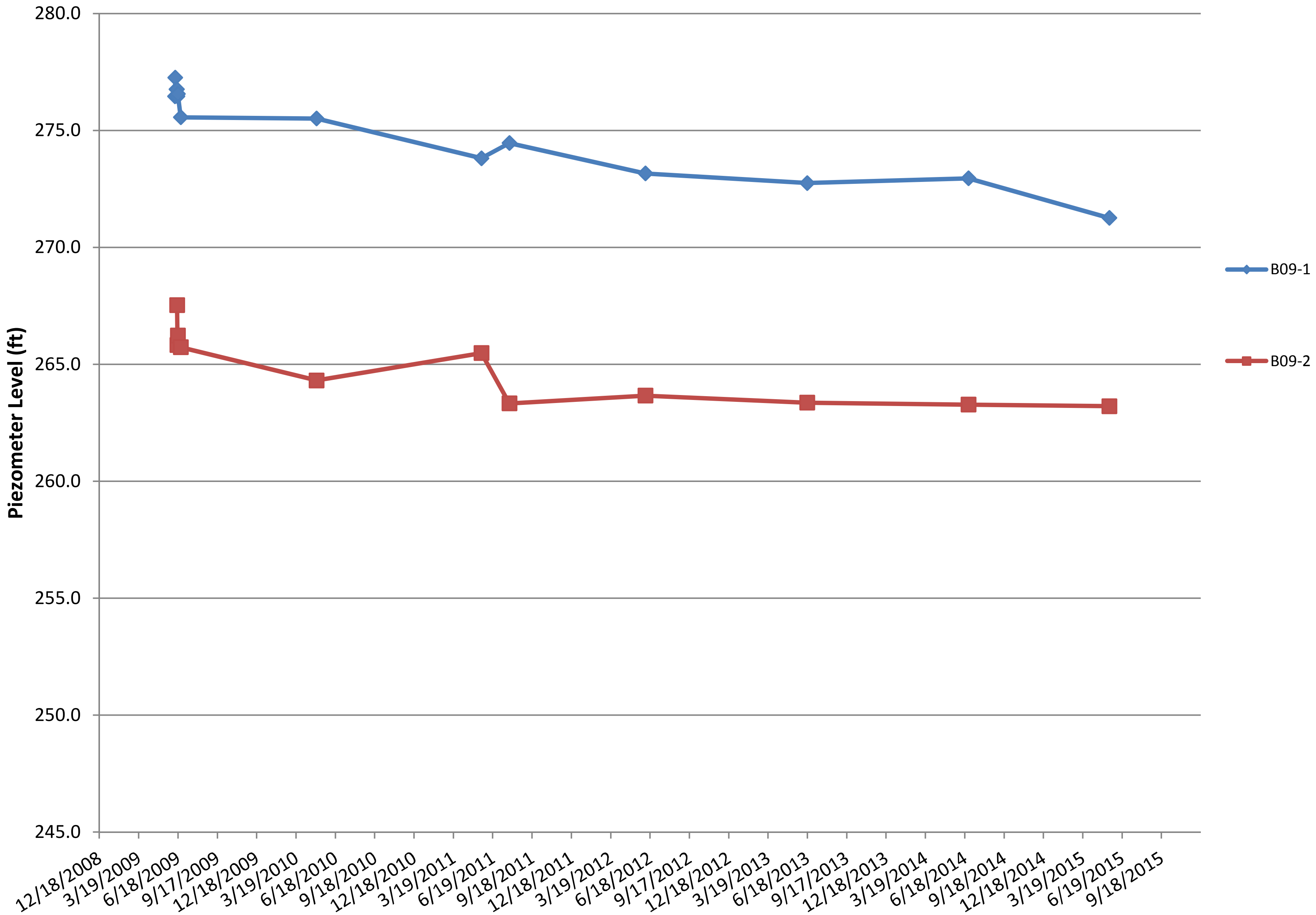

Figure D-4