2021 Periodic Structural Stability Assessment Report

Brunner Island Ash Basin No. 6 CCR

Prepared for:
Brunner Island, LLC

October 6, 2021

Contents

Appendices

1.0 Executive Summary

This report presents the Five-Year Periodic Structural Stability Assessment for the Brunner Island Ash Basin No. 6 facility. This report was prepared in accordance with the requirements of the U.S. 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) (Coal Combustion Residual [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.

Brunner Island Ash Basin No. 6 is an inactive CCR surface impoundment, which is owned and operated by Brunner Island, LLC, a division of Talen Energy (Talen). Talen permanently discontinued process water and wastewater flow into Ash Basin No. 6 on May 31, 2019, initiated closure activities on June 3, 2019, and is currently removing ash for beneficial reuse as part of their closure plan. While the basin is undergoing closure, it is required to comply with the operating criteria of the CCR Final Rule. 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 a Periodic Structural Stability Assessment performed by a qualified engineer in accordance with the CCR Final Rule. This is the first Five-Year Periodic Structural Stability Assessment performed in accordance with the CCR Final Rule.

Section §257.73 of the CCR Final Rule requires that periodic structural stability assessments be conducted and documented and include the following dam safety-related elements:

- Stable foundations and abutments;
- Adequate slope protection;
- Adequate compaction of dikes;
- Adequate vegetation control;
- Adequate spillway capacity;
- Structural integrity of hydraulic structures underlying or passing through the dikes; and
- Adequate stability of downstream slopes that are affected by sudden drawdown of an adjacent water body.

Based on a review of the information available from the original investigation and construction, the foundation is considered stable. The specified compaction of the dikes complies with the requirements of the CCR Final Rule, although a number of compaction test results were reported which did not meet the requirements of the CCR Final Rule. The specifications, quality control documents, and correspondence from the original construction indicate that sections of the embankment where compaction test results did not comply with the specifications would have been reconditioned and recompacted in accordance with the specifications. The slope protection and control of vegetation, as observed during HDR's site visit on August 11, 2021, is adequate. No evidence of significant deficiencies was observed in the discharge conduits passing through or under the dikes, with the exception of the flap valve on the polishing pond outlet conduit which may be impacted by backwater effects from the Susquehanna River during a major flood. When the Susquehanna River is at flood stage, the flap valve may be closed or may only partially open, restricting discharge from the basin. With the pool elevation in the main basin drawn down and maintained at its current level of 261.7 feet, the inflow design flood (IDF) can be managed without any discharge from the basin, and the spillway capacity is, therefore, adequate. Although not needed to pass the IDF, the discharge conduits are maintained in a clear condition without vegetation or debris obstructions. Talen has implemented a program to control vegetation along the banks of the impoundment to reduce the risk of debris accumulating in the discharge conduits or intakes, which could adversely affect their discharge capacity. Based on the historic drawings, it appears that the conduit bedding is generally in conformance with current standards. There was no evidence of seepage or piping of soils at either of the conduits during previous inspections.

Rapid drawdown analyses of downstream slopes must be conducted where the slopes can be inundated by an adjacent water body that could then be subject to a low pool or sudden drawdown. Ash Basin No. 6 is located immediately adjacent to the Susquehanna River, which is subject to significant swings in flow and stage. Shallow slope failures, attributed to rapid drawdown loading, have been observed in the past immediately after recession of flooding on the Susquehanna River. A transient slope stability analysis was conducted which determined that the factor of safety for critical deep-seated sliding surfaces complies with the recommendations of guidelines recognized by the CCR Final Rule.

The following recommendations are presented:

- Continue slope vegetation cutting and repair measures as necessary to maintain adequate cover and vegetation height within the 6-inch limit and to cut dead vegetation to prevent it from becoming entrained in spillway flows and collecting in or near the discharge conduits.
- Conduct annual inspections and cleaning of the outlet conduits to verify that they are structurally stable and are clear.
- Continue maintenance and inspection of the main basin permanent dewatering pump system and verify that back-up equipment is readily available so that the current drawn down main basin water surface elevation is maintained at approximate elevation 261.7 feet and is quickly restored to that elevation following rainfall events where the runoff exceeds the pump capacity.

2.0 Project Description

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 Five-Year Periodic Dam Failure Analysis and Hazard Potential Classification (HDR 2021a) 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, original construction drawings, and splitter dike conveyance modifications are provided in Appendix A.

The ash basin was designed and constructed between 1975 and 1979. The surface impoundment consists of a main basin with a polishing (water treatment) pond at the southern end of the facility. The ash basin has a total area of approximately 78.7 acres and is surrounded by a perimeter dike with a nominal crest elevation of 290 feet and a length of about 8,300 feet. 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 end of the main basin was previously filled with ash to near the crest of the dike. Prior to the start of phased decommissioning in 2019, bottom ash was removed from the waste stream prior to entering the surface impoundment; therefore, only process water and low volume process wastewater was sluiced into the main basin. The plant permanently discontinued wastewater and process water inflow into Ash Basin No. 6 on May 31, 2019. The southern end of the main basin area (where there were previously 9.4 acres of open water) initiated dewatering in 2019 as part of the decommissioning passive dewatering. The only inflow to the basin is from precipitation. Rainfall or snowmelt is now discharged from the main basin to the polishing pond for final treatment via a pump (installed in the fourth quarter of 2019 and became fully operational January 2, 2020) that discharges to the original flow-through concrete drop structure, prior to discharge to the Susquehanna River. 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 (NGVD29), is approximately 0.76 feet higher than the North American Vertical Datum of 1988 (NAVD88) at Ash Basin No. 6.

The outlet structure located in the splitter dike, separating the main basin from the polishing pond, consists of a concrete riser with a sill elevation of 283.50 feet and a drop structure with an outlet invert elevation of 271.0 feet that discharges into one 48-inch-diameter, reinforced-concrete pipe that discharges to the polishing pond. Two gated perforations in the weir-type riser were made in 2019, once the basin stopped receiving wastewater, to assist with the impoundment drawdown. Floating skimmers were used initially to lower the reservoir below the splitter dike concrete riser sill. The skimmers were then replaced with a floating permanent pump to maintain the main basin water surface at the current elevation of approximately 261.7 feet. Drawings of the ash basin and control structures are provided in Appendix A.

These modifications established the low-flow outlet invert of the bulkhead dewatering fitting at elevation 272.5 feet. As-built drawings of the modifications are included in Appendix A. Daily water surface elevations were recorded beginning in February 2020 through mid-May 2021, the average elevation during this time was 261.7 feet – a 22.5-foot decrease from the 2016 normal water surface elevation. The pumping system maintained the pond elevation between 261.4 to 261.8 feet during several rain events, pumping up to 300 to 340 gallons per minute.

The outlet structure connecting the main basin to the polishing pond includes a 48-inch-diameter buried pipe with a Komax static mixing chamber that is used for environmental testing and control.

The polishing pond was previously used for final treatment of the process water before it was discharged to the Susquehanna River, but now that process water and wastewater are no longer discharged to Ash Basin No. 6, the inflow to the polishing pond is stormwater runoff. The polishing pond geometry remains unchanged, with an area of 2.7 acres controlled by twin, baffled morning glory outlet structures, with top-of-weir elevations of 268.0 feet, which both discharge into a single 48-inch-diameter, reinforced-concrete pipe that discharges to the Susquehanna River. The water elevation in the polishing pond is normally maintained slightly above elevation 268.0 feet. A flap gate, with an invert elevation of approximately 252 feet, is provided at the river end of the 48-inch-diameter discharge pipe to prevent river water from entering the ash basin during Susquehanna River high flow conditions.

3.0 Structural Stability Assessment

Documentation and assessment of the required elements of the Structural Stability Assessment are provided below.

3.1 Stable Foundations and Abutments

Available information regarding the foundation of Ash Basin No. 6 is provided in the Draft History of Construction document (Geosyntec Consultants 2015) and is summarized below:

- A geotechnical investigation in 1975 consisted of 16 borings advanced into rock on a grid-like pattern. Boring logs and a location plan were provided. The site was summarized as being "underlain mostly by sandy soils (i.e., sandy gravels, silty sands, sandy silts) from the surface to depths of 14 to 34 feet below ground surface (ft-bgs). Clay was identified in some borings at depths shallower than 10 ft-bgs. Rock, consisting of soft to very hard sandstone and soft to hard shale were encountered at depths between approximately 10 ft and 29 ft-bgs."

- A geotechnical investigation in 1977 consisted of 12 borings advanced into rock on a grid-like pattern, as well as field permeability and laboratory testing. A boring location plan was provided separately. Subsurface conditions were generally consistent with the 1975 investigation, with the exception of a 6- to 8-foot-thick layer of loose sand encountered in Borings A and D. These borings were located in a part of the basin that was shown as being excavated and it is expected that the loose soils are no longer in place.
- A geotechnical investigation in 2009 consisted of 4 borings drilled through the east embankment into the foundation, installation of piezometers, and index and strength testing of the embankment.
- A geotechnical investigation in 2012 consisted of 5 borings drilled in the embankment and 4 test pits (not included in the history document). The borings likely did not penetrate the foundation.
- During explorations conducted as part of the decommissioning work (not included in the history document), Talen determined that the overburden soils overlying rock within the interior of the original ash basin were not left in place as previously believed but were excavated to rock. At the time of the 2016 Initial Structural Stability Assessment report (HDR 2016a) it was assumed these overburden soils within the interior of the basin were left in place and were considered immobile and would remain in place during the impoundment failure modeling. However, Talen determined these overburden soils, estimated to be 15 to 20 feet thick, were excavated to rock and removed over the full footprint of the basin during construction of the perimeter dike down to bedrock elevation. Based on Talen's investigation findings, the impoundment contained an additional estimated 1,200 acre-feet of ash before decommissioning began than was assumed previously.

The 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 Ash basin 6 is continuous. There are no abutments.

3.2 Adequate Slope Protection

The downstream Ash Basin No. 6 perimeter dike embankment slopes are protected by a thick cover of grass. They are not normally exposed to water or wave action and have withstood flow and wave action from occasional flooding of the Susquehanna River without significant erosion in the past. The Environmental Resources Management (ERM) Flood Impact Memo on Ash Basin 4, 5, 6, and 7 Dikes (ERM 2012) stated that the grassed slopes were adequate to withstand anticipated water velocity and wave action resulting from flooding from the Susquehanna River. Shallow sloughing has occurred during recession of flooding of the Susquehanna River on a few occasions, which has been attributed to a sudden drawdown-type of slope failure. The transient drawdown analysis noted below indicates that the stability of slopes with respect to deep-seated failure surfaces complies with the recommended factors of safety in guidelines recognized by the CCR Final Rule. The upstream slope of the part of the impoundment that contains open water is lined with clay and gravel and is partially protected by vegetation. There is little wave action, and no significant erosion has been observed during recent annual inspections. The crest is formed by a gravel road. Significant erosion of the crest road has not been observed, and Talen periodically re-grades the road to address potholes or low areas. Based on the condition of the slope protection measures observed during HDR's August 2021 site visit and Talen's slope and vegetation maintenance practices, the erosion protection of the upstream and downstream slopes and crest are adequate.

3.3 Dike Compaction

Specifications from the original construction as well as a limited number of field compaction test results are provided in Geosyntec (2015). The specifications call for the density of embankment soils to be within 95 percent of the standard Proctor density established in accordance with ASTM D698, consistent with the requirements of the CCR Final Rule. The compaction test results and the earthwork control summary sheet indicate that a number of field compaction tests did not meet either the specification density requirement of 95 percent of the standard Proctor density or the moisture content requirement. Re-tests are noted in the documentation, but these cannot be definitively correlated to areas that previously had unsatisfactory compaction test results. The specifications and quality control guidance document clearly call for sections of the embankment where compaction tests did not meet the specified moisture content or minimum compaction to be reconditioned, recompacted, and retested. An internal memo PPL (1979) discusses the compaction difficulties and noted that a slight relaxation in water content would be allowed but did not suggest that density requirements could be relaxed. Though it cannot be positively stated that all areas where initial compaction tests did not meet specification requirements were reconditioned as necessary until they satisfied the specification requirements, it is clear that the intent of the Owner was to maintain the specification requirements.

3.4 Vegetation Control

The vegetation on the downstream slope of the Ash Basin No. 6 perimeter dike embankment consists of thick grass as noted above. The vegetation on the upstream slope consists of thick grass and reeds. The erosion protection on the crest consists of gravel and is not vegetated.

Talen's vegetation control program calls for cutting vegetation three times a year during the growing season. Vegetation during HDR's August 2021 site visit was generally within the 6-inch-height limit noted in the CCR Final Rule. Talen indicated that the vegetation control plan would maintain vegetation within the recommended limits.

3.5 Spillway Adequacy

As noted in Section 2, the spillway system at Ash Basin No. 6 consists of:

- a flow-through concrete drop structure in the main basin with a floating permanent pump to maintain the main basin water surface at the current elevation of approximately 261.7 feet, a concrete riser with a sill elevation of 283.50 feet, and a drop structure with an outlet invert elevation of 271.0 feet that discharges into a 48-inch-diameter, reinforced-concrete pipe that discharges to the polishing pond; and
- the terminal outlet structure located in the polishing pond, consisting of two, 60-inch-diameter riser pipes with skimmers draining into a single, 48-inch-diameter, reinforced-concrete discharge pipe that discharges into the Susquehanna River. A flapper gate and an outlet control structure are provided at the river-end of the discharge pipe to prevent river water from entering the ash basin during Susquehanna River high flow conditions.

For a medium-sized, significant-hazard CCR impoundment, the IDF is the 1,000-year flood. With the pool elevation in the main basin drawn down and maintained at its current level of 261.7 feet, the IDF can be managed without any discharge from the basin, and the spillway capacity is, therefore, adequate. Although not needed to pass the IDF, the discharge conduits are maintained in a clear condition without vegetation or debris obstructions. Talen has implemented a program to control vegetation along the banks of the impoundment to reduce the risk of debris accumulating in the discharge conduits or intakes, which could adversely affect their discharge capacity. The methodology, assumptions, results, and conclusions of the spillway adequacy evaluation are described in the Flood Control Plan (HDR 2021b).

3.6 Structural Integrity of Hydraulic Structures

The CCR Final Rule requires that the annual inspection include a visual inspection of any hydraulic structures underlying the base of the CCR unit or passing through the dike of the CCR unit for structural integrity and continued safe and reliable operation. Talen performed the visual inspection of the conduit which passes through the internal splitter dike on August 18, 2021. The inspection was completed by manually entering the conduit and recording a video throughout. Talen reported that the conduit, gates, and mixing chamber were all in good condition.

The outlet structure from the polishing pond to the Susquehanna River consists of an inlet in the polishing pond and a regulating structure just upstream of the discharge point. Both of these structures appeared to be in good condition during the 2021 video inspection. The stem of the control gate operator on the regulating structure was replaced in early 2017. The interior of the conduit through the polishing pond embankment was also inspected by Talen staff. The conduit appeared to be in good condition, and no unusual conditions were observed that would affect the integrity or discharge capacity of the conduit.

The structural integrity of the outlet structures appeared adequate, and no evidence of significant deterioration, deformation, or distortion was observed in the discharge conduits passing through or under the dikes. Based on the historic drawings, it appears that the conduit bedding was designed in general conformance with current standards, except that anti-seep collars were specified. While common at the time of construction in the late 1970s, anti-seep collars have been found to be ineffective in preventing seepage and are no longer a recommended practice. There was no evidence of seepage or piping of soils at either of the conduits during previous inspections.

3.7 Structural Stability of Downstream Slopes After Flooding

Shallow slope failures, attributed to rapid drawdown loading, have been observed in the past immediately after recession of flooding on the Susquehanna River. These slope failures, which have since been repaired, did not threaten the integrity of the embankment, but did indicate that stability of the downstream slope for the rapid drawdown condition should be assessed.

The structural stability of the downstream slope of the embankment for the drawdown condition was assessed through a slope stability analysis as documented in the Brunner Island SES Transient Seepage and Slope Stability Study (Schnabel Engineering 2015 and 2021), and PPL Brunner Island SES Transient Seepage and Slope Stability Study (Schnabel Engineering 2012), both provided in Appendix B.

In 2015, the stability of the downstream slope was analyzed for a condition of rapid drawdown of the Susquehanna River from an elevation of 289.5 feet, which is 0.5 foot below the crest of the ash basin embankment and 0.5 foot above the reported 1,000-year flood level for the Susquehanna River of 289.0 feet. The 2015 analysis, which included evaluation of the sensitivity of the embankment permeability, determined that the minimum factor of safety for the rapid drawdown condition was 1.1, in compliance with the recommendations of the U.S. Army Corps of Engineers (USACE) Manual 1110-2-1902 (USACE 2003), the reference recommended in the CCR Final Rule. This analysis, which was prepared by another consultant, was not reviewed in detail as part of the preparation of this Structural Stability Assessment Report.

In 2021, the stability analysis of the downstream slope was updated for a condition of rapid drawdown of the Susquehanna River assuming an Ash Basin No. 6 normal headwater elevation of 262.0 feet and a surcharged phreatic (headwater) condition of elevation 289.0 feet. The Susquehanna River flood water surface elevation of 289.5 feet, corresponding to 0.5 foot above the 1,000-year recurrence interval river flood level, was assessed through an 8-day transient seepage progression evaluation which incorporated a river rise over a 2-day period from normal river elevation 252 feet to flood river elevation 289.5 feet, followed by a 4-day period where the river remained at 289.5 feet, followed by a 2-day period where the river receded to elevation 252 feet. The minimum factor of safety for the rapid drawdown condition for deep-seated failure surfaces capable of causing a breach of the embankment was estimated at 1.1 to 1.2 under steady-state normal and surcharged ash basin phreatic condition, which meets or exceeds the minimum USACE recommended values of 1.0 and 1.1. Due to the differences in the size of the respective drainage basins and the timing of peak runoff, there is a low likelihood of the Susquehanna River 1,000-year recurrence interval flood occurring simultaneously with an ash basin steady-state surcharge storage pool condition. The resultant factor of safety of 1.1 provides a lower bound for the deep-seated failure surfaces for the specified flood conditions. Similarly, river floods with more frequent recurrence intervals (e.g., 50-yr, 100-yr, etc.) would result in lower tailwater levels and higher factors of safety if all other factors remained the same.

4.0 Site Visit

A site visit was conducted by Adam Jones, P.E. of HDR and Ben Wilburn, P.E. of Talen on August 11, 2021. One intent of this visit, pertinent to this report, was to view site conditions for visual evidence either verifying that the assumptions of the analyses were appropriate, or that changes in site conditions were visually evident that should be considered in the Structural Stability Assessment Report.

No evidence of instability of the embankment was observed, with the exception of evidence of historic shallow sloughs that were observed around 2005 and that were documented previously. These sloughs were repaired in 2009 and their condition has not changed since then. The section adopted for the stability analyses is appropriate, based on visual observations. Conditions observed during the site visit were consistent with those observed in the 2016 visual inspection and the assumptions made in the Initial Safety Factor Assessment Report (HDR 2016b), with the following exceptions:

- Ash slurry and process water are no longer being discharged at the north end of the basin, and the surface elevation and lateral extent of the free water pool at the south end of the basin has been reduced. This has resulted in a reduction in the phreatic surface in the ash fill and embankment, as discussed above, which has a beneficial effect on the stability of the embankment.
- Evidence of significant ash excavation and removal was evident, and ash removal was in progress during the site visit. Ash removal reduces the hydrostatic and lateral earth pressures acting on the embankment section, which has a beneficial effect on embankment stability.

- Conditions at the toe and lower slope of the embankment were drier than observed previously, consistent with the reductions in the phreatic surface discussed above. The section of piezometer casing at Piezometer PZ-6-1B that has historically shown artesian pressures several feet above the ground surface was not flowing, an additional indication that groundwater levels at the toe of the embankment are falling.
- The outlet structures between the main basin and the polishing pond and between the polishing pond and the Susquehanna River appeared to be in good condition. Talen reported that the outlet control valves in both structures are locked in the full open position and that the operating stem for the control valve in the polishing pond outlet structure had recently been replaced to address difficulties in operation that had been reported previously.
- Talen reported that internal inspections of the outlet structure conduits between the main basin and the polishing pond and between the polishing pond and the Susquehanna River were conducted in the summer of 2021, and that the conduits were in good condition, with no obstructions or other conditions that affect the assumptions of clear open pipes that were used in the analyses. Note that the 2021 Flood Control Plan (HDR 2021b) analyses found that the ash basin complies with IDF requirements even if no discharge from the polishing pond is assumed.
- Crews were in the process of removing vegetation along the inside of the basin that could plug the outlet structures during an extreme flood. As noted above, the freeboard of the ash basin is adequate even if there is no discharge from the polishing pond.
- Talen reported that the polishing pond outlet flapper gate valve is exercised and lubricated annually and reportedly operates smoothly through its full range of motion.

5.0 Conclusions and Recommendations

5.1 Conclusions

No significant deficiencies were identified as a result of this Periodic Structural Stability Assessment. Recommendations to maintain continued compliance with the requirements of the CCR Final Rule are presented below.

5.2 Recommendations

The following recommendations are presented:

- Continue slope vegetation cutting and turf repair measures as necessary to maintain adequate cover and vegetation height within the 6-inch limit and to prevent cut or dead vegetation from becoming entrained in spillway flows.

- Conduct annual inspections and cleaning of the outlet conduits as necessary to verify that they are structurally stable and are clear.
- Continue maintenance and inspection of the main basin permanent dewatering pump system, and verify that back-up equipment is readily available, so that the current drawn down main basin water surface elevation is maintained at approximate elevation 261.7 feet and is quickly restored to that elevation following rainfall events where the runoff exceeds the pump capacity.

6.0 Closure

Based on the information currently available, I certify to the best of my knowledge, information, and belief that this Periodic Structural Stability Assessment meets the requirements of CCR Rule §257.73(d) Structural Integrity Criteria for Existing CCR Surface Impoundments, Periodic Structural Stability 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.

Adam N. Jones, P.E.
Senior Geotechnical Engineer

COMMONWEALTH OF PENNSYLVANIA
REGISTERED PROFESSIONAL
ADAM JONES
ENGINEER
No.PE075739

10/6/2021

Jennifer Gagnon, P.E.
Associate Engineer

7.0 References

Environmental Resources Management (ERM). 2012. *Memo on Flood Impact on Ash Basin 4, 5, 6, and 7 Dikes*.

Geosyntec Consultants. 2015. *Draft History of Construction*.

HDR Engineering, Inc. (HDR). 2016a. Initial Structural Stability Assessment.

_____. 2016b. *Initial Safety Factor Assumption Report*.

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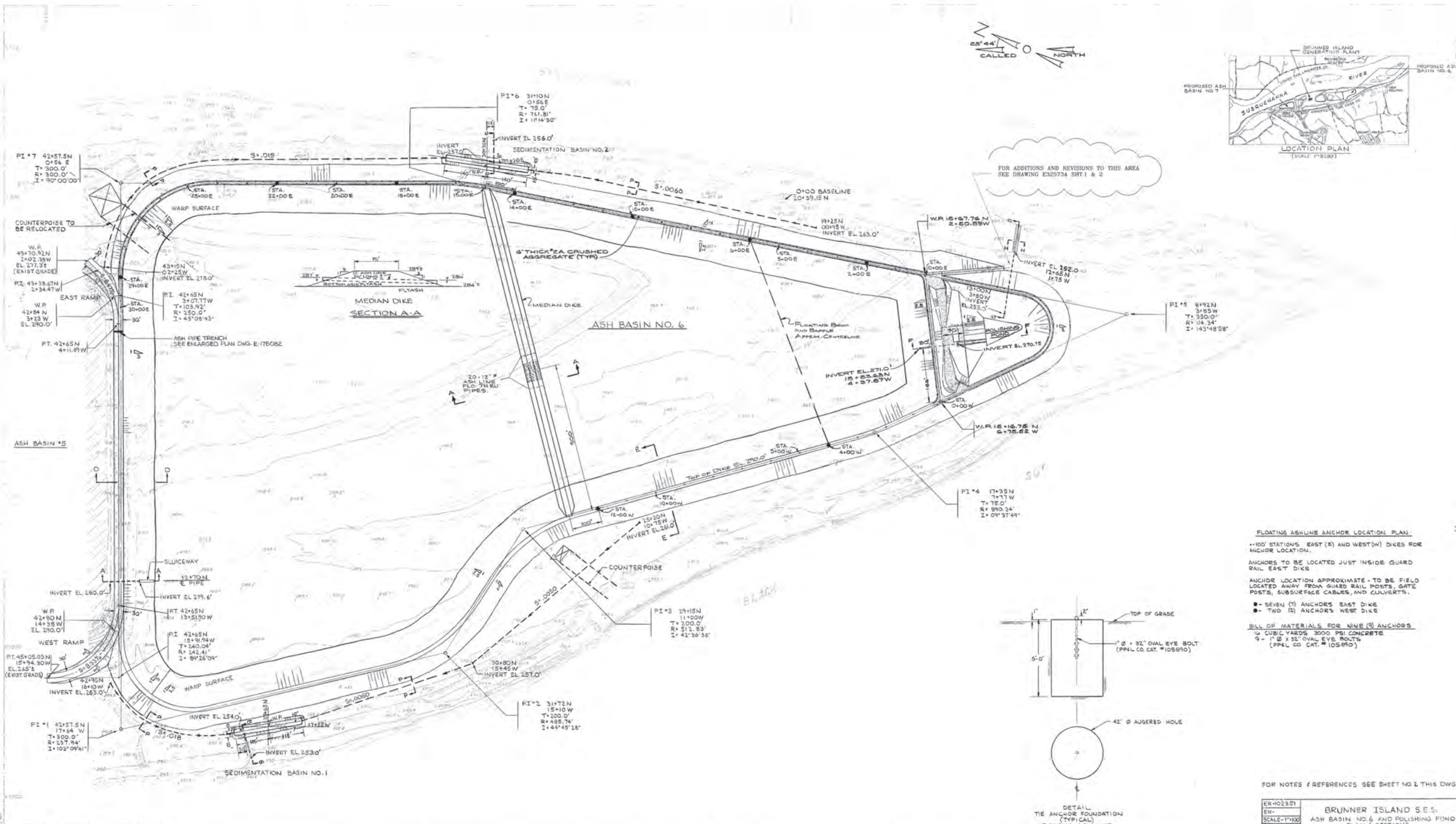
Appendix A. Reference Photos and Drawings


3/2020
Gut Rd
Gut Rd
Imagery Date: 3/8/2020 40°04'47.28" N 76°41'01.26" W



FOR NOTES & REFERENCES SEE SHEET NO. 2 THIS DWG.

NO	DATE	ER	REVISION	BY	CH.	APPROVED
4	11/20/06	103357	ADDED FLOATING ASH LINE ANCHOR FOUNDATIONS	JAC		
3	2/18/83	102351	ADDED ASH PIPE TRENCH REFERENCE	TJL	KEQ	
2	4/1/80	102713	REVISED ... STATIC MIXER PIT & DISCHARGE ... TO POLISHING POND	GHC	FAC	
1	6/8/77	102713	REVISED DIKE BETWEEN POLISHING POND AND ASH POND NO. 5. ADDED ACCESS RAMPS & GUARDRAIL	GHC	DPK	
5	3/8/11	480251	ADDED FLOATING BOOM, MEDIAN DIKE, & SECTION A-A	KK	JEM	

REFERENCE TITLE	NUMBER
SOUTH ACCESS ROAD - PLAN & PROFILE	E-169157
ASH BASIN NO.6 & POLISHING POND - PLAN & DETAILS	E-176088

ER-102351
ER-
SCALE-1"=100'
DATE-1-23-76
DRAWN-EMB
CHECKED-ECB
LEADER-RCS

BRUNNER ISLAND S.E.S.
ASH BASIN NO. 6 AND POLISHING POND
PLAN & SECTIONS

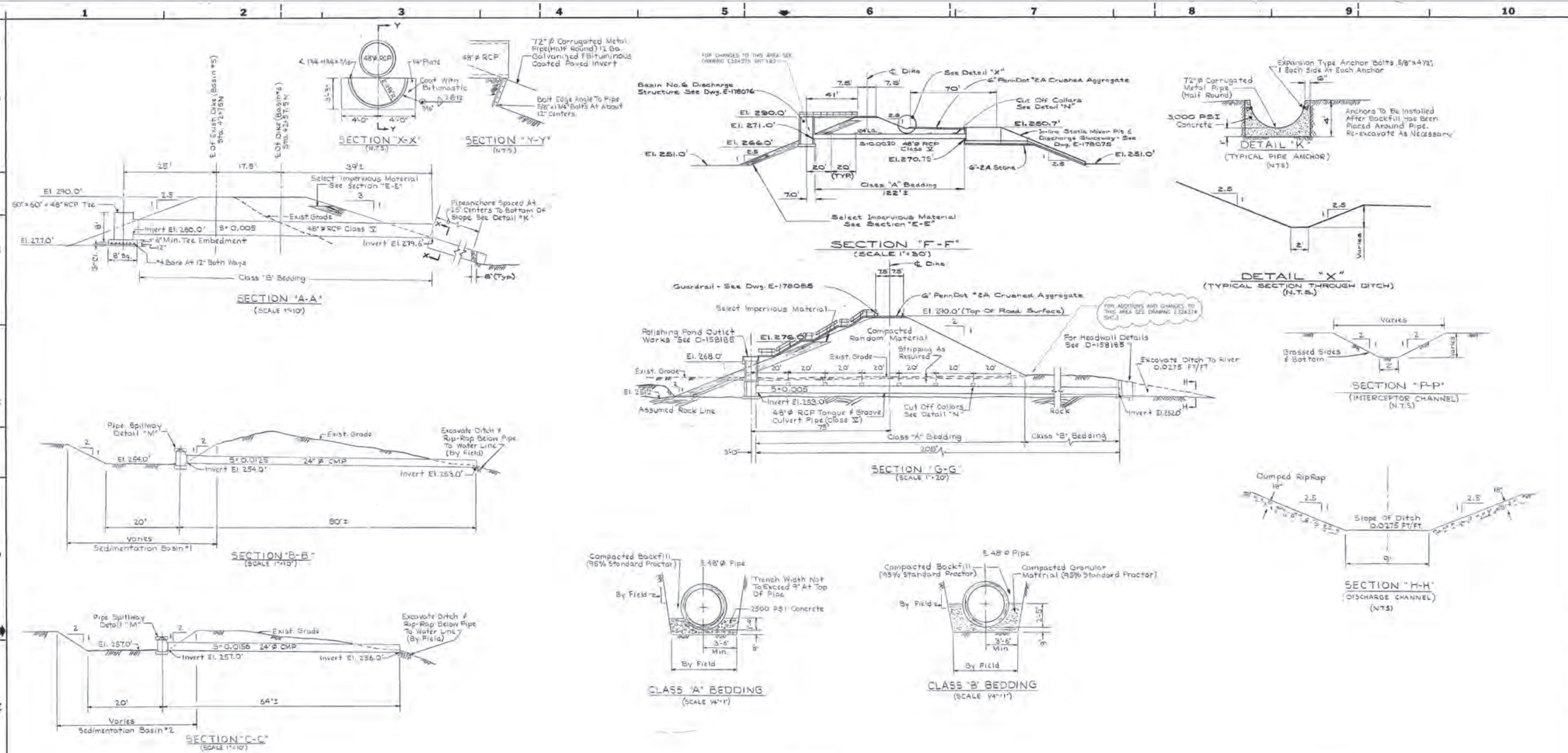
PENNSYLVANIA POWER & LIGHT COMPANY
ALLENTOWN PA.

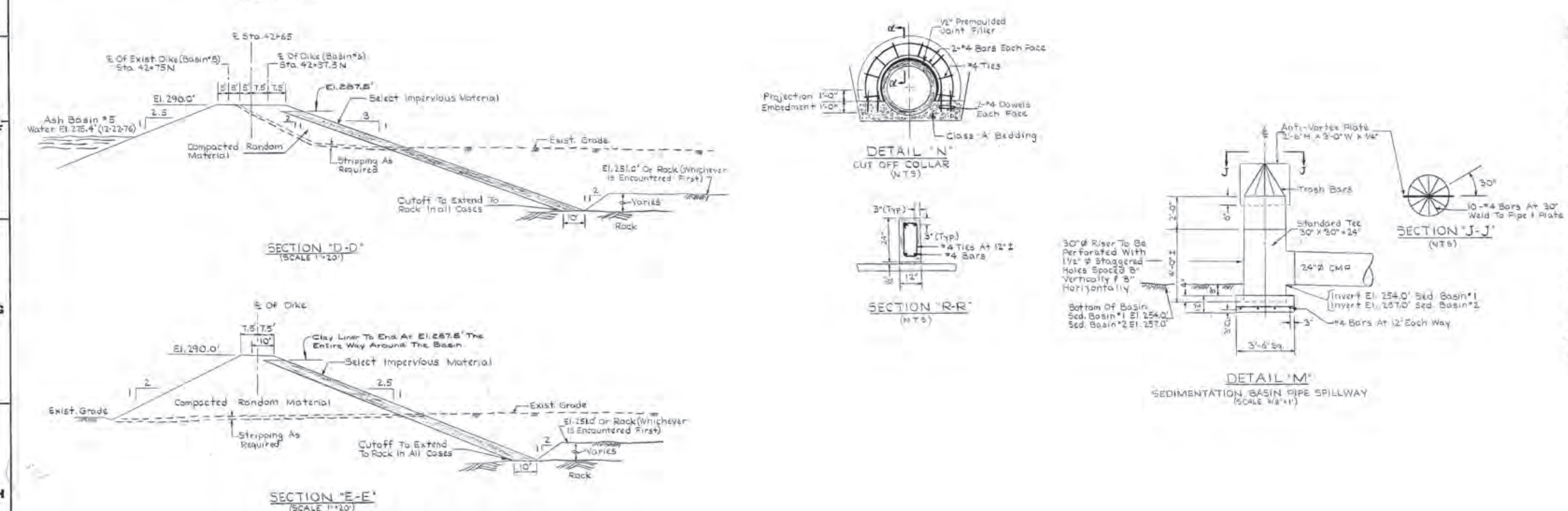
APPROVED RESPONSIBLE ENGINEER

SHEET 1 OF 7

E158595- 6

F H70 BI ASH BASIN NO 6 PLAN 1





NOTES:

For specifications, see specification PPC-1333, "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 – 1,839,000 Cu. Yd's

Random Fill – 566,000 Cu. Yd's

Impervious Fill – 115,000 Cu. Yd's

#2A Crushed Aggregate – 1,880 Cu. Yd's

Guardrail – 8,300 Lineal Feet

REFERENCE TITLE	NUMBER
ASH STORAGE AREAS #4 & #5 - PLAN	LF-75333
ASH BASIN #6 & POLISHING POND - OUTLET WORKS & HEADWALL	C-158185
ASH BASIN #7 - PLAN & SECTIONS	E-157640
ASH BASIN #6 - LOCATION & LOG OF CORE DRILL HOLES	D-129687

REFERENCE TITLE	NUMBER
BILL OF MATERIAL	A-159148

NO.	DATE	ER.	REVISION	BY	CH.	APPROVED
4	11/26/01		ADDED REFERENCE NOTE TO SECTION G-G	DM		GDH
3	11/20/00		ADDED NOTE TO SECTION F-F	DM		GDH

NO.	DATE	ER.	REVISION	BY	CH.	APPROVED
2	4/1/80	102713	ADDED IN-LINE STATIC MIXER PIT, DISCHARGE SLUICEWAY, REVISED INV. & LENGTH OF 36"Ø RCP IN SECT. F-F, DELETED DETAIL "Y"	SHC	FAC	ALN
1	6/6/79	102713	REVISED DETAIL "K" AND ADDED DETAIL "Y"	SHC	GPK	ALN
			ADDED GUARD RAIL & AGGREGATE QUANTITIES & DET. "X"			
			REVISED SECT. "F-F" & CLAY IN SECT. "D-D", "E-E" & "G-G"			

LOC CODE: F H 7 0

DESCRIPTION: BI ASH BASIN NO 6 PLAN 2

ER-102351

ER-

SCALE-AS NOTED

DATE-1-23-78

DRAWN-EMB

CHECKED-RCB

LEADER-RGS

APPR'D-NHJ

APPR'D-JAI

BRUNNER ISLAND S.E.S.

ASH BASIN NO. 6 AND POLISHING POND PLAN & SECTIONS

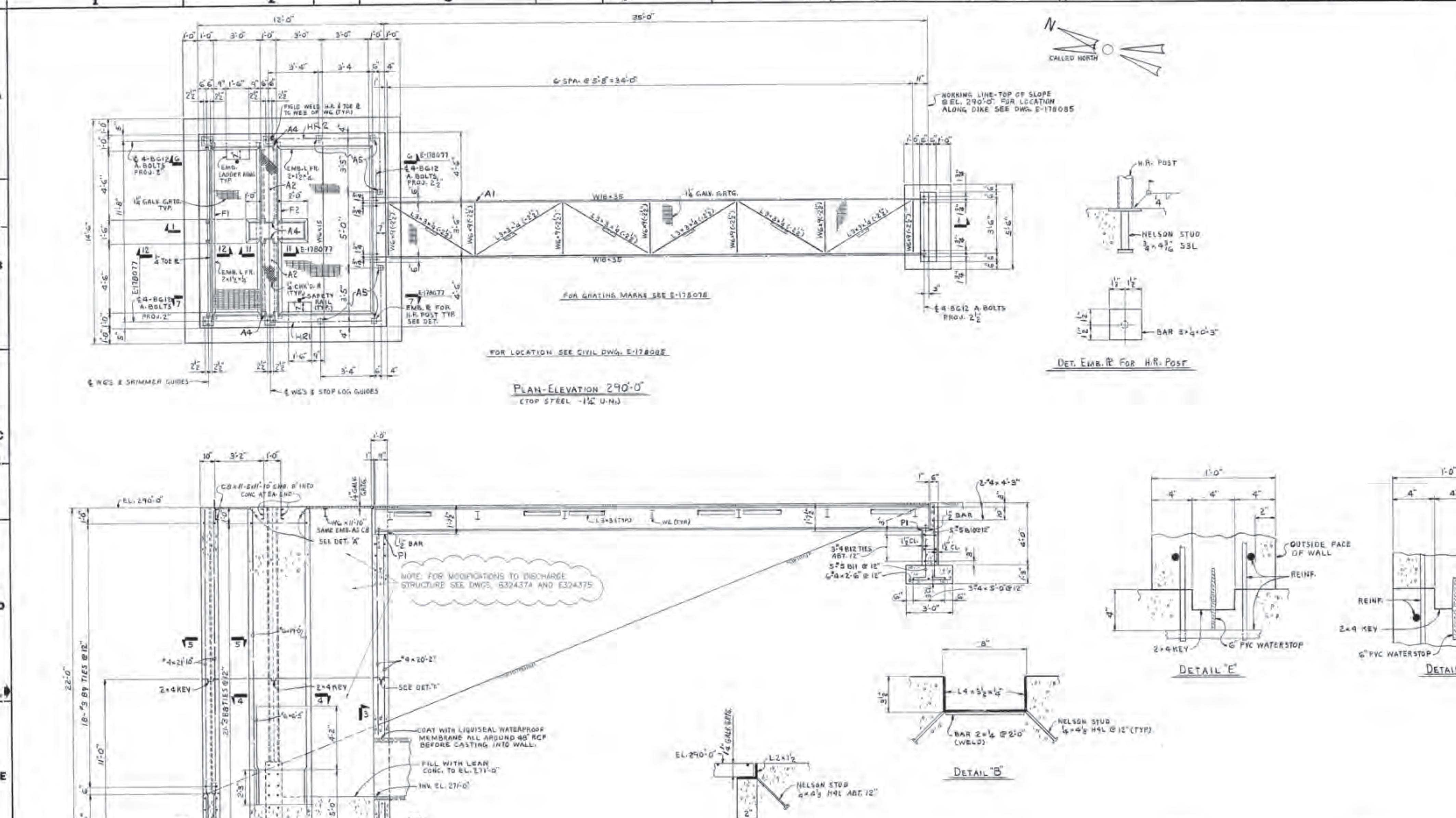
PENNSYLVANIA POWER & LIGHT COMPANY

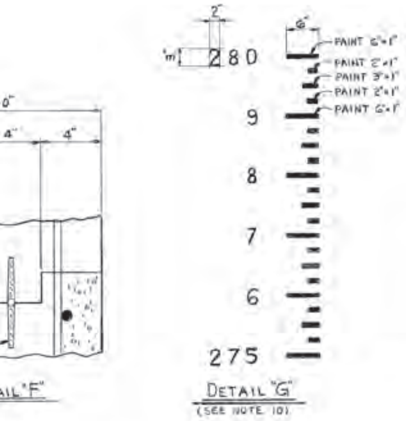
ALLENTOWN, PA.

SHEET 2 OF 2

RESPONSIBLE ENGINEER

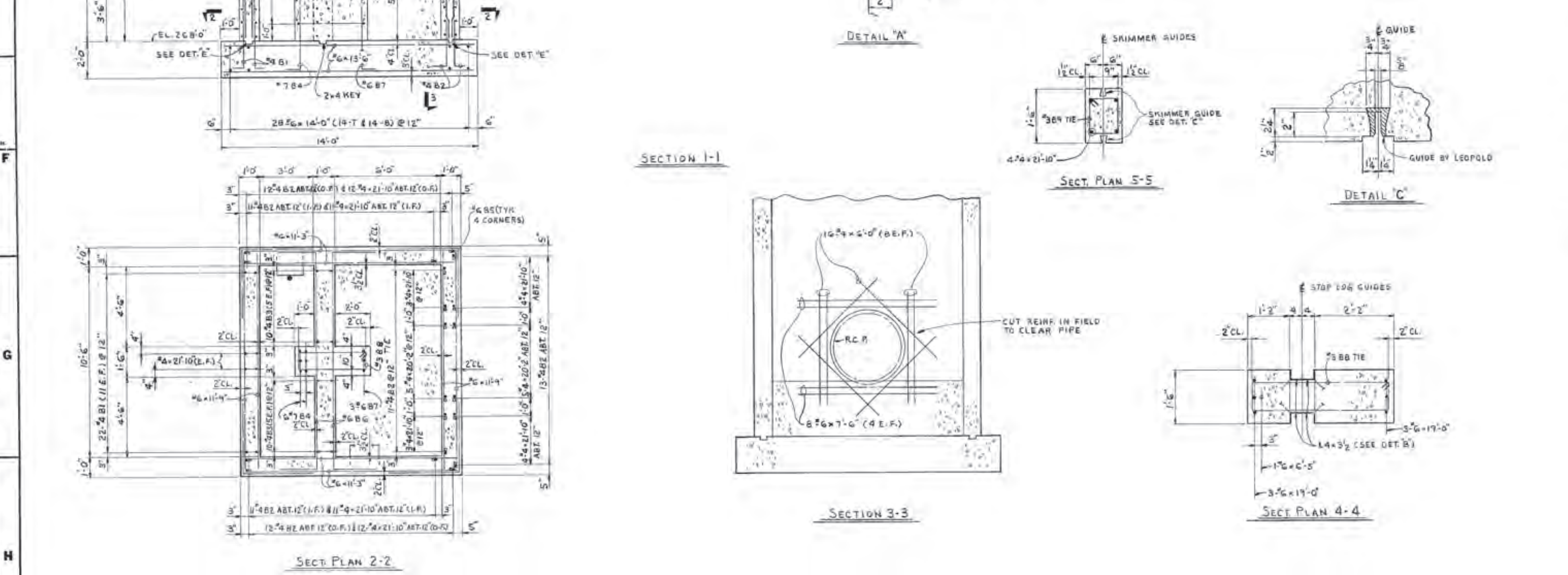
E158595-4





Bill Of Materials

Quantity	Description
54 CU. YDS.	3,000 P.S.I. CONCRETE
86 LIN. FT.	6" PVC WATERSTOP TYPE NO. 6380 W.R. MEADOWS OR EQUAL
1 GAL.	LIQUISEAL WATERPROOF MEMBRANE BY SURE SEAL
16	3/4"# A. BOLTS 8G12 AS PER SHEET NO. 9 OF LA-80165
18	LOGS LONGLEAF YELLOW PINE SELECT STRUCT. GRADE SEE STOP LOG DET. (E-178077) & NOTE 6
2	SKIMMER GATE, FIBERGLASS REINFORCED POLYESTER RESIN, 4'-0" H. x 4'-6" W., COMPLETE WITH 18'-6" H. TYPE EP-103 FRAME, LIFT BOLTS, AND 25 L.F. 5/8" STAINLESS STEEL CABLE. GATE TO HAVE NEOPRENE SEAL STRIPS. AS MANUFACTURED BY F.B. LEOPOLD CO., ZELIENOPLE, PA.
1	FOUR WHEEL I BEAM TROLLEY FOR 8" I BEAM, 4,000 LBS. CAPACITY. McMASTER-CARR CO. NO. 3267T4 OR EQUAL.
2	WORM GEAR WINCH, 2000 LBS. CAPACITY, McMASTERS-CARR NO. 3210T14 OR EQUAL
1	CABLE RATCHET WINCH HOIST, 4,000 LBS. CAPACITY. 25' LIFT WITH SAFETY HOOKS AND STAINLESS STEEL CABLE DETACHABLE HANDLE MARINE SERVICE, LUG ALL MODEL 450-8-M OR EQUAL
7 CU. YDS.	LEAN CONC.
1	LOT OF STEEL & GRTG. AS PER E-178078 & E-178079
1	LOT OF REINF. STEEL AS PER A-178084 SHTS 3 & 4
2	LADDER FALL PREVENTION DEVICE COMPLETE WITH CARRIER RAIL, LADDER RUNG CLAMPS, SLEEVE, AND 4'-6" REMOVABLE TOP EXTENSION, ALL GALVANIZED. ONE ASSEMBLY 26' LONG ONE ASSEMBLY 23' LONG. NORTON COMPANY "SAF-T-CLIMB" OR EQUAL.



NOTES:-

1. ALL CONCRETE WORK SHALL BE IN ACCORDANCE WITH THE AMERICAN CONCRETE INSTITUTE CODE LATEST REVISION, AND SHALL COMPLY WITH PP&L SPECIFICATION FOR CONCRETE LA-50168, EXCEPT CONCRETE SHALL HAVE A MINIMUM COMPRESSIVE STRENGTH OF 3000 PSI AT THE END OF 28 DAYS.
2. REINFORCEMENT BARS TO BE DEFORMED STEEL BARS CONFORMING TO ASTM A615, GRADE 60.
3. ALL STEEL WORK SHALL COMPLY WITH AISC SPECIFICATIONS AND CODE OF STANDARD PRACTICE AND PP&L SPECIFICATION FOR STRUCTURAL STEEL LA-50165 AND SHALL CONFORM TO ASTM A36.
4. WELDING TO COMPLY WITH THE SPECIFICATIONS OF THE AMERICAN WELDING SOCIETY.
5. GRATING TO BE ASTM A36 STEEL, GALVANIZED AS PER ASTM A123-73, WITH 1-1/4" X 1/4" BEARING BARS AT 1-3/16" CENTER-TO-CENTER AND 9/32" CROSS BARS AT 4" CENTER-TO-CENTER.
6. PRESERVATIVE TREATMENT FOR STOP LOGS SHALL BE PRESSURE CREOSOTING AND SHALL BE DONE IN ACCORDANCE WITH "TIMBERS IN MARINE CONSTRUCTION SPECIFICATION C-18" OF THE AMERICAN WOOD PRESERVERS ASSOCIATION, 1958. FINAL RETENTION OF GRADE NO. 1 CREOSOTE PER CUBIC FOOT OF WOOD SHALL BE 16 LBS/CU FT CREOSOTE SHALL CONFORM TO AMERICAN WOOD PRESERVERS ASSOCIATION STANDARD "P1-65".
7. ALL EXPOSED CONCRETE CORNERS SHALL HAVE A 3/4" CHAMFER.
8. TOE PLATE AND HANDRAIL INDICATED THUS——— - ——— - ———
9. INSTALLATION INSTRUCTIONS FOR THE SKIMMER GATE AND GUIDE SHALL BE SUPPLIED BY F.B. LEOPOLD COMPANY. CARE SHALL BE TAKEN TO INSURE THE TIGHTEST SEAL POSSIBLE WITH THE MATERIAL PROVIDED.
10. FIELD TO PAINT (BLACK) WEST FACE OF WALL (NORTHWEST CORNER) WITH A GRADUATED SCALE WITH FIGURES 2" X 3" AND STARTING AT EL. 273'-6" TO EL. 280'-0" AS SHOWN IN DETAIL "G".

Reference Title	Number	Reference Title	Number
ASH BASIN #6 DISCH. STRUCTURE STRL. CONC. & STL. SECT. & DET.	E-178077	LIST OF REINFORCING SHEETS 3 & 4	A-178084
LAYOUT OF PIKE BETWEEN ASH POND #6 & POLISHING POND. CIVIL	E-158595		
STEEL DETAILS	E-178078		
STEEL DETAILS	E-178079		

No.	Date	ER.	Revision	By	Ch.	Approved
(2)	10/23/09		REVISED REFERENCE NUMBER (D3)	DM		GDH
1	11/06		ADDED NOTE FOR DISCHARGE STRUCTURE (D3)	DM		GDH

LOC CODE: F H70 — DESCRIPTION: DISCH. STRUCT. PLAN & SECTS. — SORT: 5

W.O. 844712-041

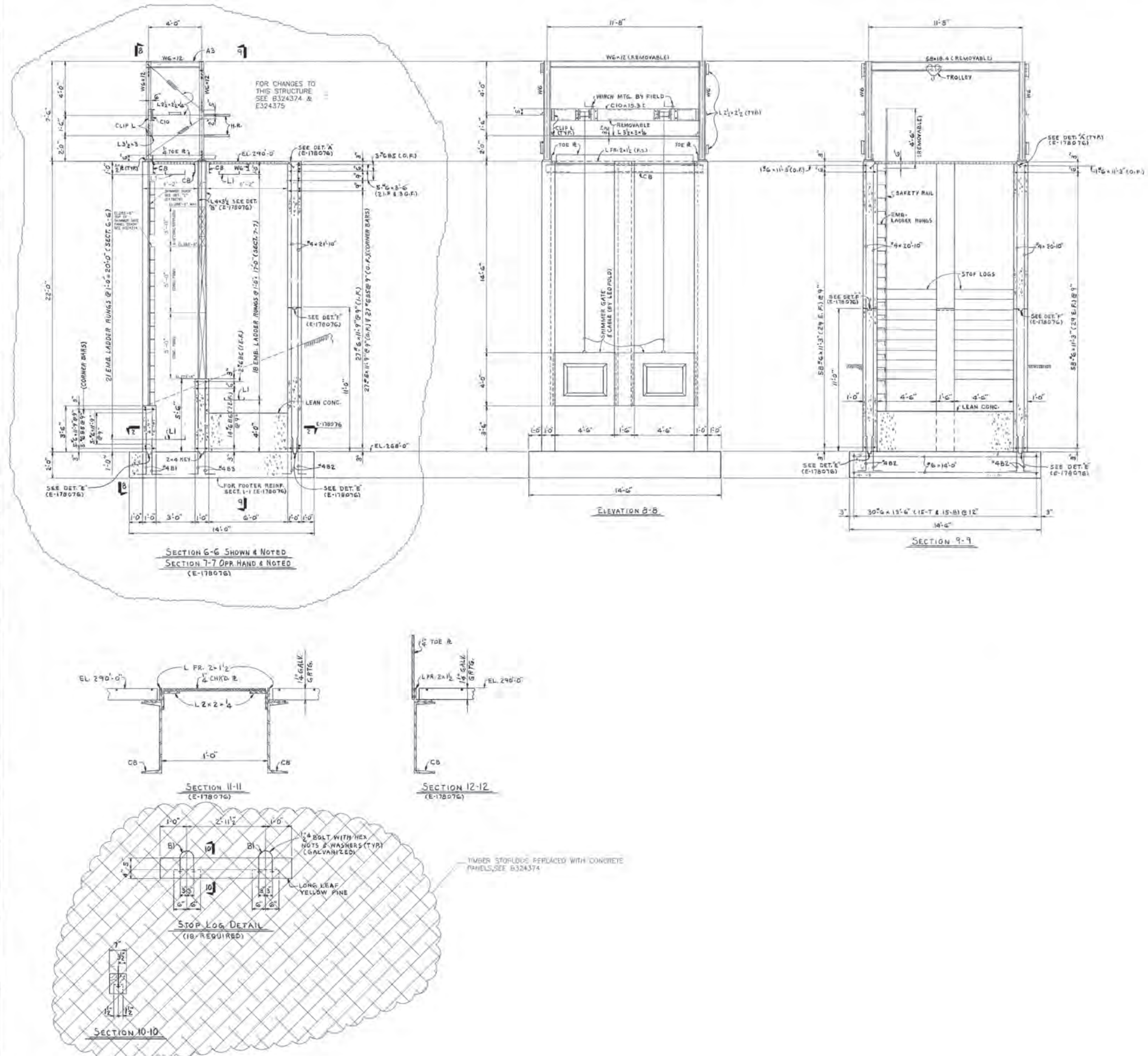
ER- 107351
ER-
SCALE- 3/8"=1'-0"
DATE- 6/8/79
DRAWN- N.K.
CHECKED- L.P.H. 5.16.79
LEADER- J.J.F.
APPR'D-
APPR'D-

BRUNNER ISLAND S. E. STATION
ASH BASIN 6 DISCHARGE STRUCTURE
STRUCTURAL CONCRETE AND STEEL
PLAN AND SECTIONS

PENNSYLVANIA POWER & LIGHT COMPANY
ALLENTOWN, PA.

APPROVED J.A. Stefanik 6/8/79
RESPONSIBLE ENGINEER

SHT. 1
E178076-2



NOTES:-
1. FOR NOTES, SEE DRAWING E-178076.

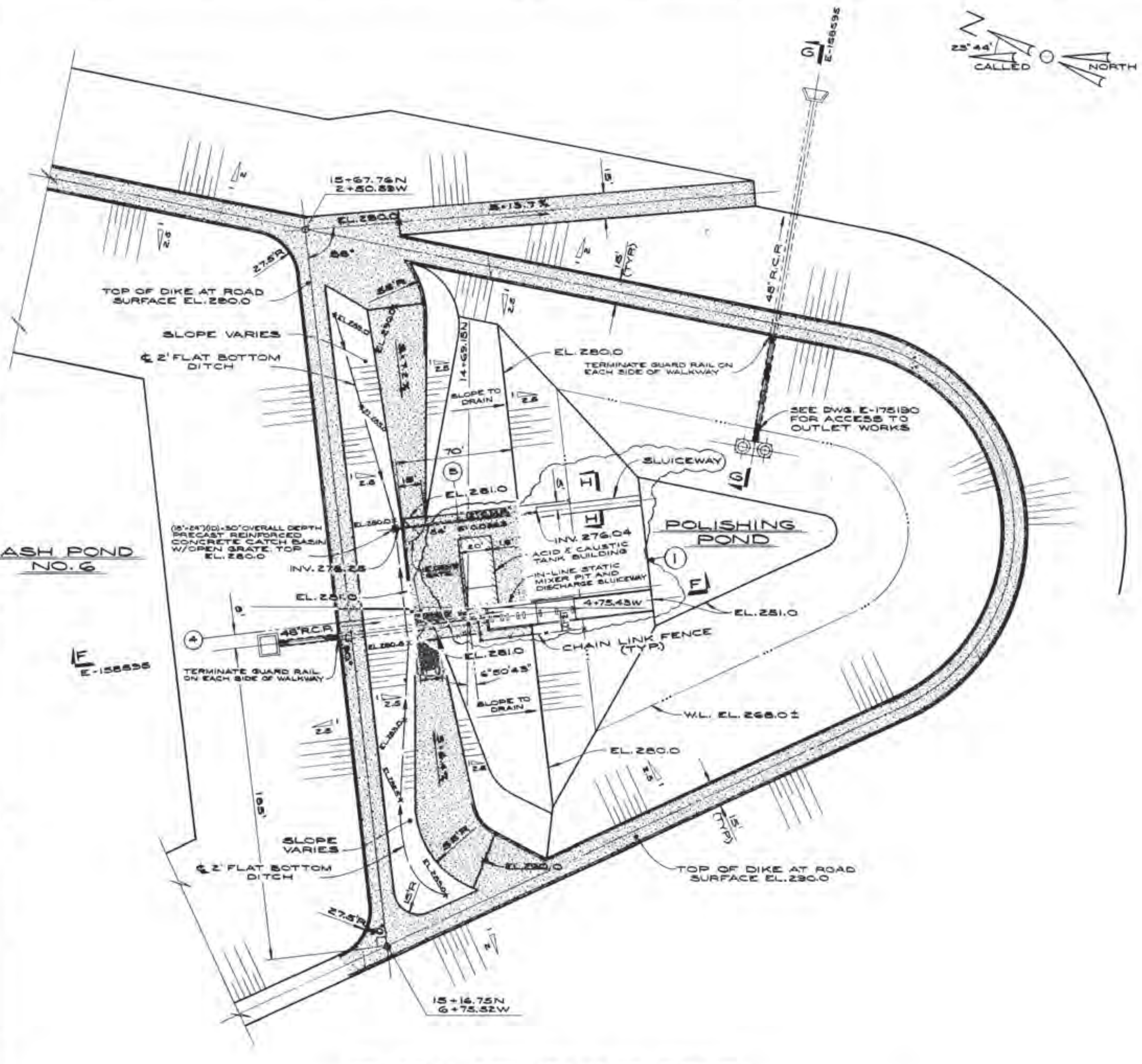
W.O. 044712-041

ER-102351	BRUNNER ISLAND S. E. STATION ASH BASIN #8 DISCHARGE STRUCTURE STRUCTURAL CONCRETE AND STEEL SECTIONS AND DETAILS
ER-	
SCALE-3/8"=1'-0"	
DATE-6/9/77	
DRAWN- N.K.	
CHECKED- LFH 8-18-77	**PENNSYLVANIA POWER & LIGHT COMPANY** ALLENTOWN, PA.
LEADER- J.J.F.	APPROVED J.A. Stefanik 6/9/79 RESPONSIBLE ENGINEER — SHT. 1 — E178077-1
APPR'D-	
APPR'D-	

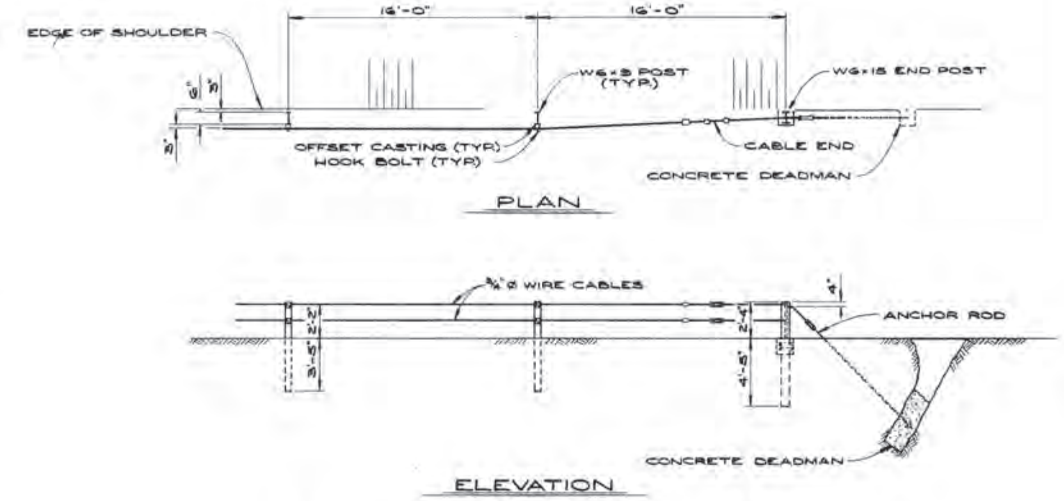
REFERENCE TITLE	NUMBER
ASH BASIN #8 DISCH. STRUCTURE STRL. CONC. & STL. PLAN & SECT.	E-178076

NO.	DATE	ER.	REVISION	BY	CH.	APPROVED
1	11/06		REVISED SECTION 6-6 & 7-7(CD)	DM	DDH	

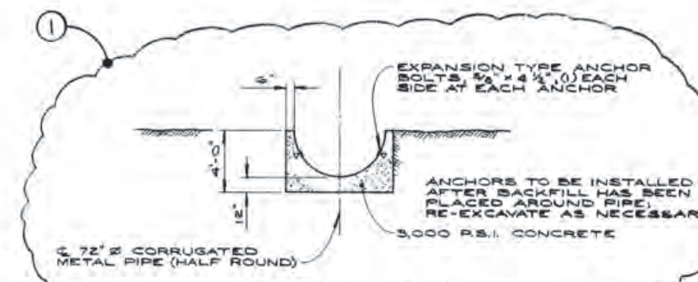
DISCH. STRUCT. SECTS. & DETS.



PLAN - POLISHING POND
SCALE: 1"=30'



TYPICAL GUARDRAIL DETAIL
NO SCALE

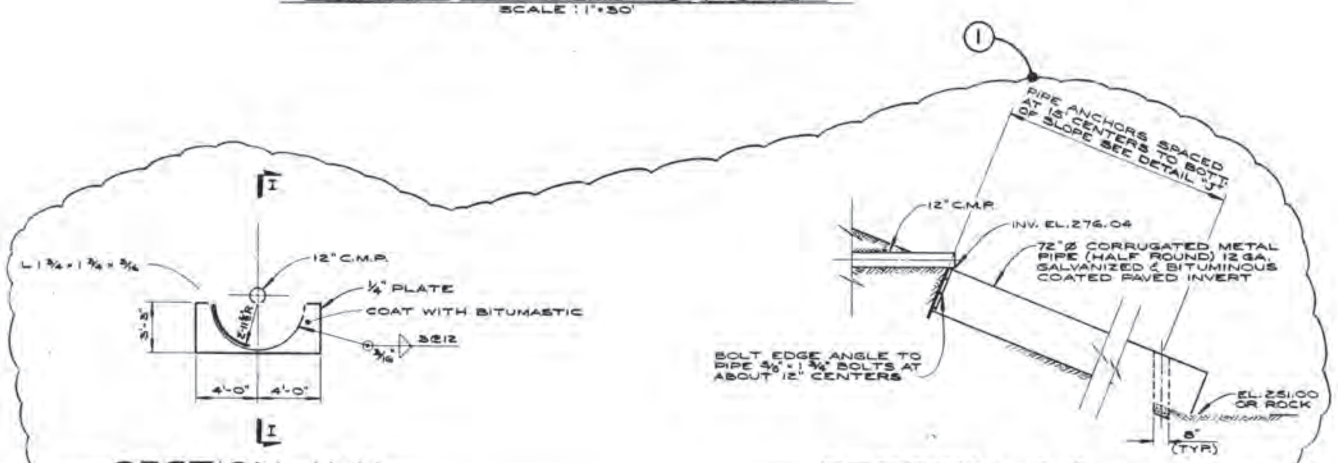


DETAIL "J"
(TYPICAL PIPE ANCHOR)
NO SCALE

6" THICK PENNDOT #2A CRUSHED AGGREGATE

NOTES:
1. ALL FENCE WORK WILL BE IN ACCORDANCE WITH P.P. &L. SPECIFICATION LA 50187.

LIST OF MATERIALS	QUANTITY
15' DRIVE GATE	2
FENCE FABRIC	1715 S.F.
GATE POSTS	4
LINE POSTS	24
PRECAST CATCH BASIN	1
12" C.M.P.	84 L.F.
72" HALF ROUND	(ON SITE)



REFERENCE TITLE	NUMBER	REFERENCE TITLE	NUMBER
SWITCHYARDS & SUBSTATIONS - PROPERTY FENCE	LD-3586	ASH BASIN #6 - ACCESS TO POLISHING POND OUTLET	E-175190
ASH BASIN #6 & POLISHING POND - OUTLET WORKS & HW.	D-168188		
INLINE MIXER FOUNDATION - PLAN & SECTIONS	D-178075		
ASH BASIN #6 & POLISHING POND - PLAN & SECTIONS	E-168585		

NO.	DATE	ER.	REVISION	BY	CH.	APPROVED
1	8/1/80	102713	ADDED SLUICEWAY TO PLAN, DETAIL "J" & SECTIONS H-H & I-I. LOCATED ACID & CAUSTIC TANK BLDG. REVISED FENCE WEST OF BLDG. AND IN-LINE STATIC MIXER PIT & SLUICEWAY.	G.H.C.	FAC	A.L.M.

LOC CODE: F H70 — DESCRIPTION: BI ASH BASIN NO 6 ENL PL — SORT: C

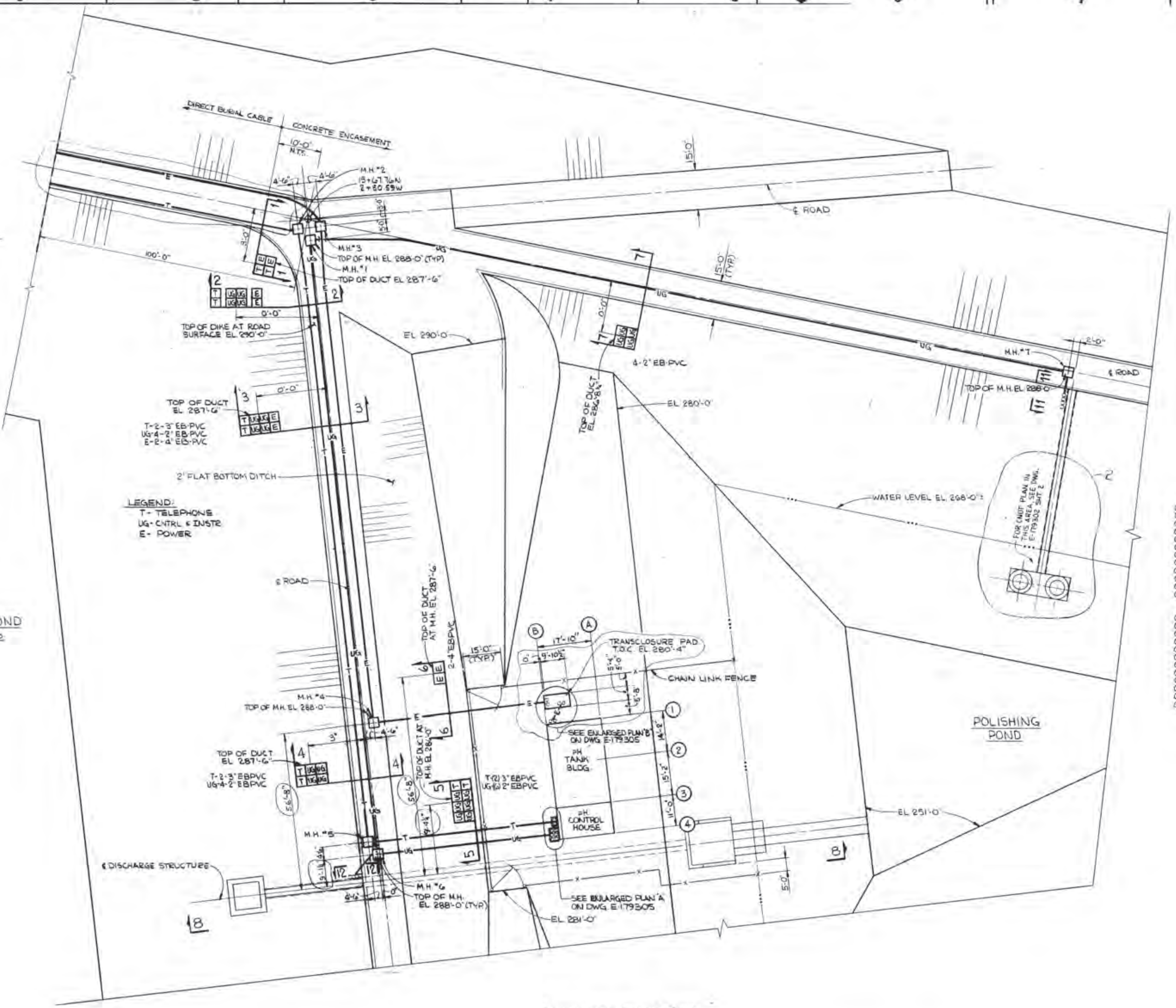
W.O. 044712-042
ER- 102713
ER-
SCALE- 1"=30'
DATE- 6/5/79
DRAWN- G.H.C.
CHECKED- DFK
LEADER- A.L.M.
APPR'D-
APPR'D-

BRUNNER ISLAND S.E.S.
ASH BASIN NO. 6 - POLISHING POND
ENLARGED PLAN

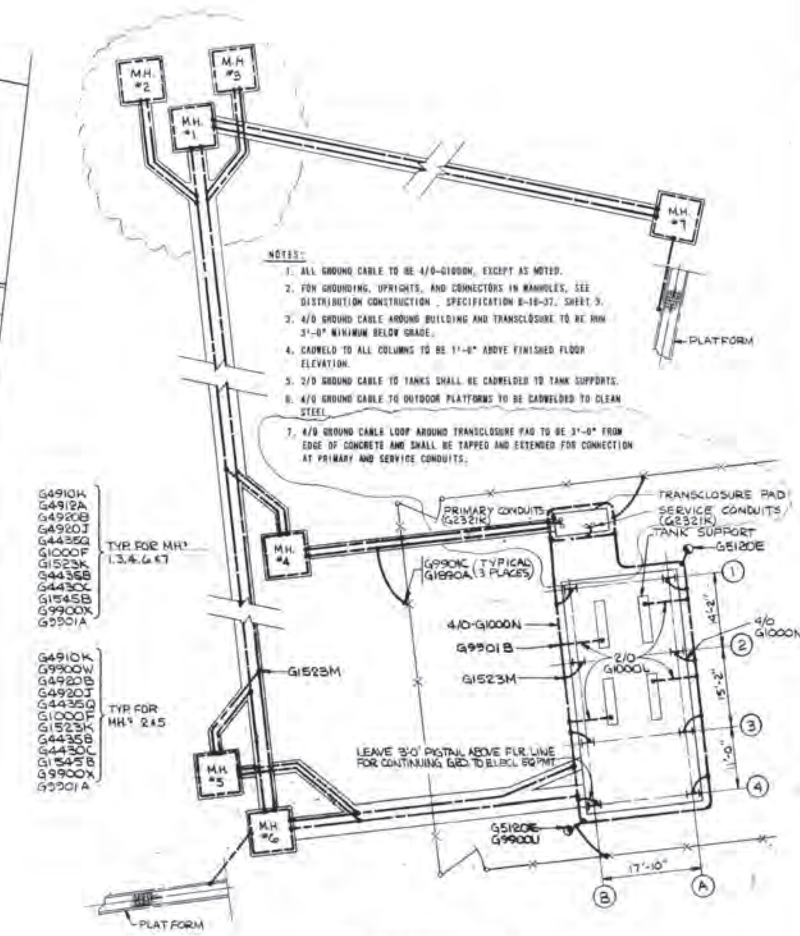
PENNSYLVANIA POWER & LIGHT COMPANY
ALLENTOWN, PA.

APPROVED John A. Stefanik

E-178085-1



PLAN - POLISHING POND
SCALE: 1/16" = 1'-0"



NOTES:

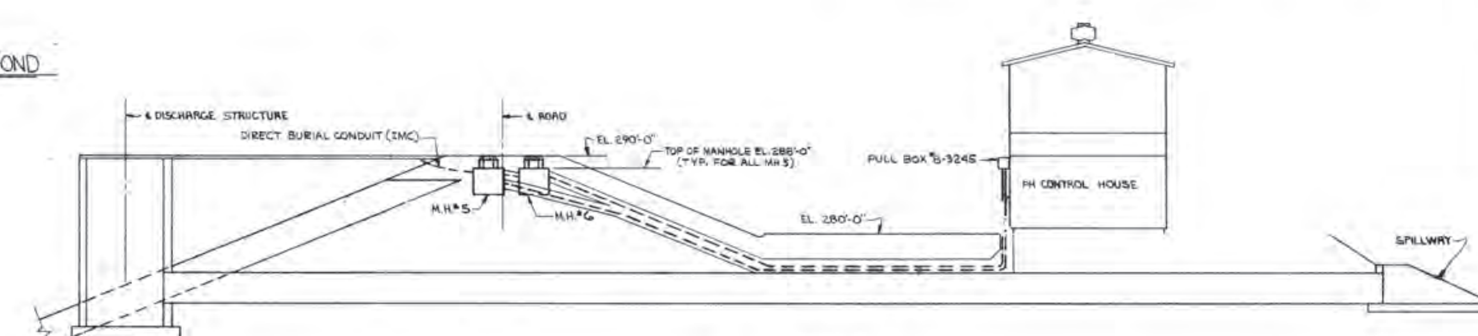
1. ALL GROUND CABLE TO BE 4/0-G1000N, EXCEPT AS NOTED.
2. FOR GROUNDING, UPRIGHTS, AND CONNECTORS IN MANHOLES, SEE DISTRIBUTION CONSTRUCTION, SPECIFICATION 8-18-37, SHEET 3.
3. 4/0 GROUND CABLE AROUND BUILDING AND TRANSCLOSURE TO BE RUN 3'-0" MINIMUM BELOW GRADE.
4. CADWELD TO ALL COLUMNS TO BE 1'-0" ABOVE FINISHED FLOOR ELEVATION.
5. 2/0 GROUND CABLE TO TANKS SHALL BE CADWELDED TO TANK SUPPORTS.
6. 4/0 GROUND CABLE TO OUTDOOR PLATFORMS TO BE CADWELDED TO CLEAN STEEL.
7. 4/0 GROUND CABLE LOOP AROUND TRANSCLOSURE PAD TO BE 3'-0" FROM EDGE OF CONCRETE AND SHALL BE TAPPED AND EXTENDED FOR CONNECTION AT PRIMARY AND SERVICE CONDUITS.

GROUNDING PLAN
SCALE: NONE

NOTES:

INSTALLATION

1. MANHOLE - DISTRIBUTION CONSTRUCTION SPECIFICATION 8-15-67 AND 8-15-66.
2. MANHOLE FRAME AND COVER - DISTRIBUTION CONSTRUCTION SPECIFICATION 8-15-08 AND 8-15-08.
3. UNDERGROUND DUCTS (ENCASED) - DISTRIBUTION CONSTRUCTION SPECIFICATION 8-15-81, SHEET 1 THROUGH 10.
4. ALL DIRECT BURIAL CONDUITS (IMC) AND EXTERIOR OF MANHOLES BELOW GRADE TO BE COATED WITH TWO COATS OF FARBERTITE (C), A COAL PITCH DERIVATIVE AS PER MANUFACTURERS DIRECTIONS. FARBERTITE (C) PROTECTS AGAINST MOISTURE AND CHEMICAL CORROSION AND IS MANUFACTURED BY BRIGGS BITUMINOUS COMPOSITION COMPANY, B/M G8801A.



SECTION 'B-B'
SCALE: 1/8" = 1'-0"

Reference Title	Number	Reference Title	Number
BILL OF MATERIAL SH 1752 TO 1965	LA-50942		
ASH BASIN 6-PH CONT. FACILITY CONDUIT LAYOUT ... & CAUSTIC TANK BLDG	E-179302		
ASH BASIN 6-PH CONT. FACILITY SECTIONS & DETAILS MANHOLES & UGND. DUCTS	E-179305		
ASH BASIN 6 - POLISHING POND - ENLARGED PLAN	E-178085	ASH BASIN 6 DISCH. STRUCTURE - PLAN STEEL	E-178076

No.	Date	ER.	Revision	By	Ch.	Approved	
2	12/8/81	102713	ADDED REFERENCE AS IND.	JPS	JMA	WIP	WJH
1	9-24-80	102713	REV. T-PAD & RELOCATED MH #4 & #6., REV. GRDING @ T-PAD & ADDED NOTE 7	EBE	RAY	WIP	WJH

LOC CODE: F3D70
DESCRIPTION: CNTRL FAC MH UG GRDNG PLN
SORT: E

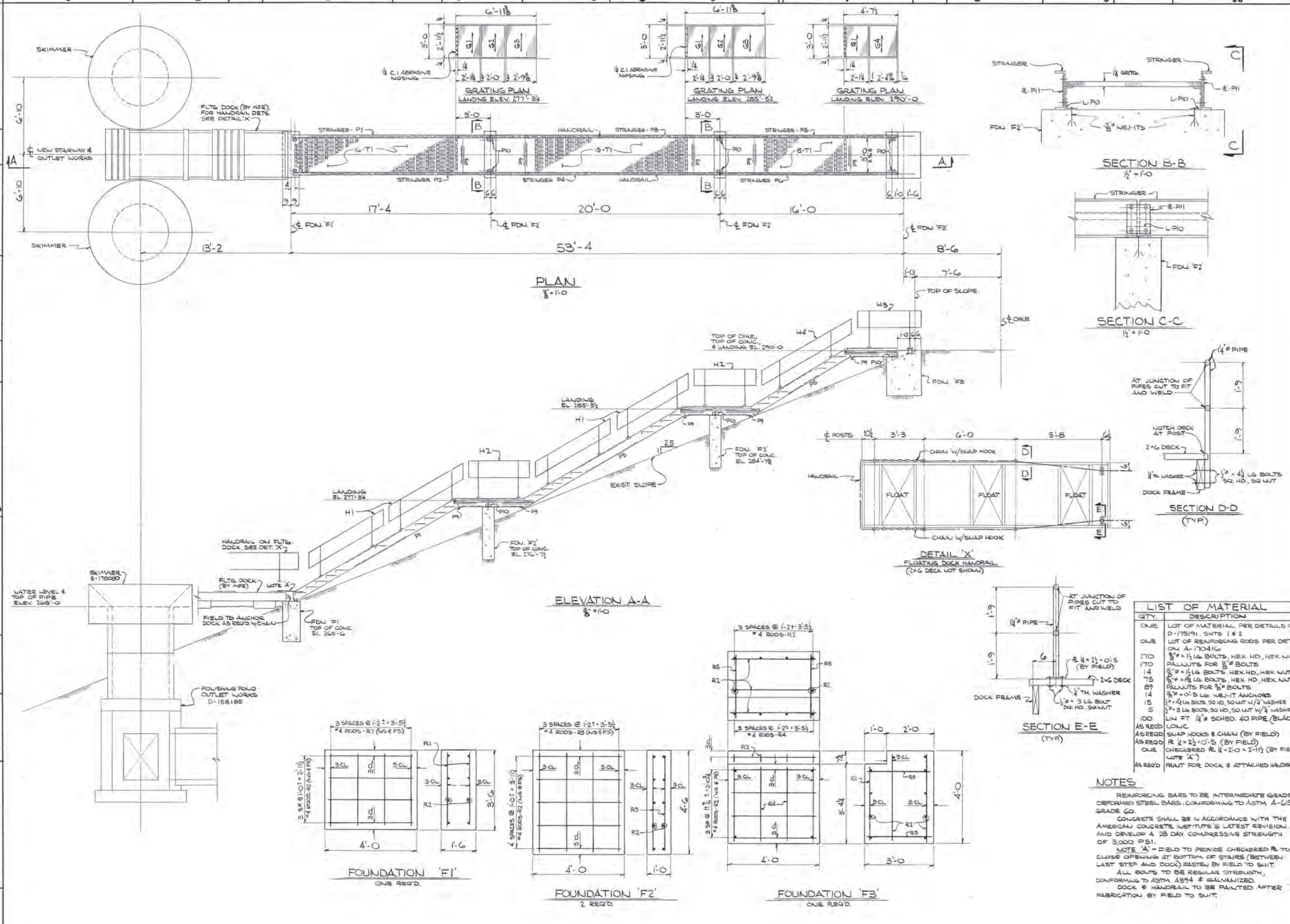
W.O. 044712-042
ER-102713
ER-
SCALE-AS NOTED
DATE- 8-8-80
DRAWN- T.J.W.
CHECKED- ERK
LEADER- RAY
APPR'D-
APPR'D- WJH

BRUNNER ISLAND S. E. STATION
ELECTRICAL
ASH BASIN 6, pH CONTROL FACILITY
PLAN
MANHOLES - UNDERGROUND DUCTS - GROUNDING

PENNSYLVANIA POWER & LIGHT COMPANY
ALLENTOWN, PA.

APPROVED
SENIOR PROJECT ENGINEER

E-179301-2



LIST OF MATERIAL

QTY.	DESCRIPTION
ONE	LOT OF MATERIAL PER DETAILS ON D-175191, SHTS 1 & 2
ONE	LOT OF REINFORCING RODS PER DETS. ON A-170416
170	⅜"ø × 1½ LG. BOLTS, HEX. HD., HEX. NUT
170	PALNUTS FOR ⅜"ø BOLTS
14	⅝"ø × 1½ LG. BOLTS, HEX. HD., HEX. NUT
75	⅝"ø × 1¾ LG. BOLTS, HEX. HD., HEX. NUT
89	PALNUTS FOR ⅝"ø BOLTS
14	⅝"ø × 0'-5 LG. WEJ-IT ANCHORS
15	½"ø × 4¼ LG. BOLTS, SQ. HD., SQ. NUT W/¼" WASHER
5	½"ø × 3 LG. BOLTS, SQ. HD., SQ. NUT W/¼" WASHER
100	LIN. FT. 1¼"ø SCHED. 40 PIPE (BLACK)
AS REQ'D	CONC.
AS REQD	SNAP HOOKS & CHAIN (BY FIELD)
AS REQD	℞ ¼ × 2½ × 0'-5 (BY FIELD)
ONE	CHECKERED ℞ ¼ × 2'-0 × 2'-11½ (BY FIELD - NOTE "A")
AS REQ'D	PAINT FOR DOCK & ATTACHED HANDRAIL

NOTES

REINFORCING BARS TO BE INTERMEDIATE GRADE, DEFORMED STEEL BARS, CONFORMING TO ASTM A-615-GRADE 60.

CONCRETE SHALL BE IN ACCORDANCE WITH THE AMERICAN CONCRETE INSTITUTE'S LATEST REVISION. AND DEVELOP A 28 DAY COMPRESSIVE STRENGTH OF 3,000 PSI.

NOTE "A" – FIELD TO PROVIDE CHECKERED ℞ TO CLOSE OPENING AT BOTTOM OF STAIRS (BETWEEN LAST STEP AND DOCK) FASTEN BY FIELD TO SUIT.

ALL BOLTS TO BE REGULAR STRENGTH, CONFORMING TO ASTM A394 & GALVANIZED.

DOCK & HANDRAIL TO BE PAINTED AFTER FABRICATION, BY FIELD TO SUIT.

REFERENCE TITLE	NUMBER
SHOP DETAILS	D-175191
OUTLET WORKS & WINGWALL	D-158185
PLAN & SECTIONS	E-158855
POLISHING POND - PLAN & DETAILS	E-178085

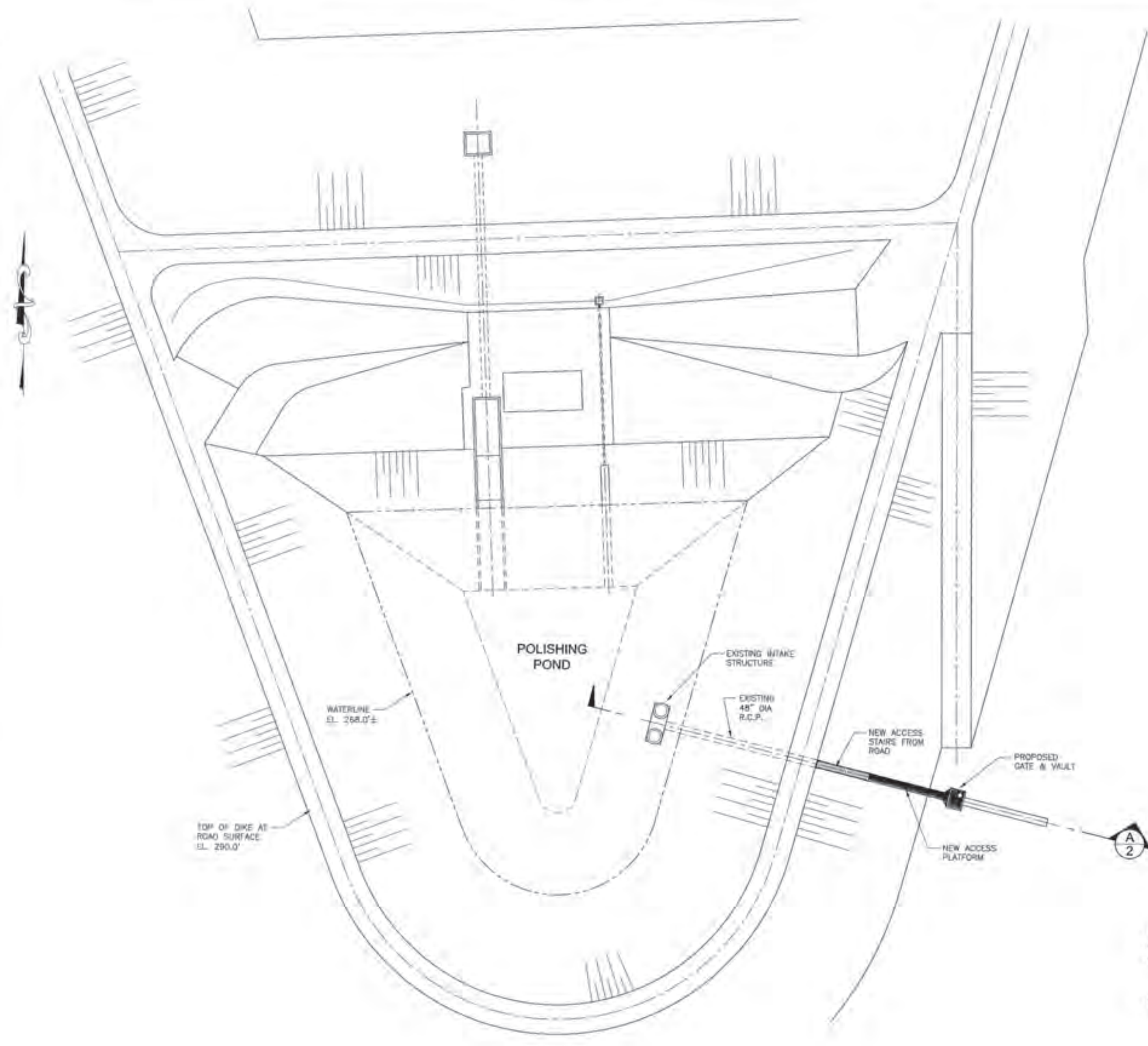
ER-102351
ER-
SCALE-AS NOTED
DATE-7-31-79
DRAWN-WEM
CHECKED-
LEADER-
APPRD-
APPRD-

BRUNNER ISLAND S.E.S.
ASH BASIN #6
ACCESS TO POLISHING POND OUTLET WORKS ASSEMBLY & DETAILS

PENNSYLVANIA POWER & LIGHT COMPANY
ALLENTOWN, PA.

APPROVED [signature] 7/31/79 RESPONSIBLE ENGINEER

E-175190-0



SITE PLAN
1"= 30'

30 0 30 60
SCALE IN FEET

GENERAL CONCRETE NOTES

1.) NEW CONCRETE VAULT CAN EITHER BE CAST-IN-PLACE OR PRE-CAST. PRE-CAST DESIGN SHALL BE SUBMITTED FOR APPROVAL.

2.) ALL LOOSE ROCK AND MUCK SHALL BE REMOVED PRIOR TO CONSTRUCTION. WHEN CONCRETE IS PLACED DIRECTLY AGAINST ROCK SURFACES, THE SURFACE SHALL BE CLEANED WITH HIGH PRESSURE WATER TO REMOVE ALL DIRT OR LOOSE MATERIAL. HIGH PRESSURE WATER IS DEFINED FOR THIS PURPOSE AS 1,500 PSI MINIMUM. IF GREASE, OIL OR OTHER COATINGS ARE ON THE ROCK SURFACE, OTHER METHODS MAY BE REQUIRED FOR REMOVAL.

3.) SURFACE PREPARATION OF EXISTING CONCRETE AND MASONRY SURFACES SHALL REMOVE LOOSE DETERIORATED MATERIAL AND VEGETATION. ACCEPTABLE METHODS INCLUDE: SANDBLASTING, MECHANICAL CHIPPING OR HIGH PRESSURE WATER BLAST (8,000 psi MIN.).

4.) EXCEPT AS OTHERWISE NOTED:

A. CONCRETE: 4000 PSI @ 28 DAYS, AIR ENTRAINED 5% TO 7%

B. REINFORCEMENT: 60,000 PSI, ASTM A615

C. REBAR COVER: 2" UNLESS NOTED OTHERWISE
HOOKS – ACI STANDARD
SPLICES – #7 BAR = 54"
– #5 BAR = 31"
– #4 BAR = 25"

D. CONCRETE ANCHORS: 3/4" MIN. DIA. S.S. SIMPSON EPOXY-TIE FOR DAMP OR SUBMERGED APPLICATIONS OR APPROVED EQUAL.

E. GROUT: 5000psi NON-SHRINK GROUT @ 28 DAYS CRD-C621

5.) PROVIDE ADHESIVE WATERSTOPS OF WATER SWELLING ELASTIC SEALING MATERIAL WHERE INDICATED. END ALL WATERSTOP 3"± FROM EXPOSED FINISHED CONCRETE SURFACE. ACCEPTABLE SUPPLIERS: WATERSTOP RX. INSTALL PER MANUFACTURERS INSTRUCTIONS.

6.) UNLESS NOTED OTHERWISE PROVIDE 3/4" CHAMFER ON ALL UNARMORED CORNERS.

7.) FIELD BEND REINFORCING BARS TO CLEAR INCIDENTAL BOXOUTS WHERE REQUIRED. EXCEPT AS NOTED, NO CUTTING OF REINFORCING BARS WITHOUT PRIOR REVIEW BY OWNER.

8.) VERTICAL CONCRETE SURFACES SHALL HAVE A SMOOTH FORMED FINISH. HORIZONTAL CONCRETE SURFACES SHALL HAVE A SMOOTH RUBBED FINISH (U.N.O.), EXCEPT WALKING SURFACE SHALL HAVE BROOM FINISH.

9.) CONCRETE SHALL BE CURED FOR A MINIMUM OF 7 DAYS IN ACCORDANCE WITH ACI 301.

10.) REPAIR ALL SURFACE VOIDS GREATER THAN 1/4" WITH METHOD APPROVAL BY ENGINEER

SEE SPECIFICATIONS SECTION 03310 FOR ADDITIONAL NOTES.

STEEL NOTES

1.) STEEL – ASTM A-36 U.N.O.
PIPE – A53 GRADE B
BOLTS – ASTM A-325 GALVANIZED

2.) ALL WELDING PER AWS D1-1.

3.) ALL STEEL HOT DIPPED GALVANIZED PER ASTM A-123, HARDWARE PER A-153 U.N.O.

4.) ALL STEEL SHALL BE FABRICATED AND ERECTED PER AISC STEEL CONSTRUCTION MANUAL 13th EDITION.

GENERAL GATE NOTES

SEE SPECIFICATIONS FOR SECTION 11281

KLEINSCHMIDT DRAWING LIST

DRAWING NO.	DRAWING DESCRIPTION
1	PLAN VIEW - POLISHING POND AND PROPOSED ACCESS TO GATE AND VAULT
2	LONGITUDINAL SECTION THRU POND AND ACCESS DETAILS
3	CONCRETE REINFORCEMENT SECTIONS AND DETAILS
4	STEEL FRAMING SECTIONS AND DETAILS

RECORD DRAWING

THIS DRAWING REPRESENTS THE BEST INFORMATION AVAILABLE TO THE ENGINEER UPON COMPLETION OF THE WORK.

KLEINSCHMIDT ASSOCIATES
CONSULTING ENGINEERS

DATE: 1/18/08 BY: TLK

SUBMIT SHOP DRAWINGS
TO E.O.R FOR APPROVAL

No.	Revision	Date
	RECORD DRAWING	1-18-08
1	UPDATED VAULT AND GATE	10-22-07
	RELEASED FOR CONSTRUCTION	9-17-07

Scale:	AS SHOWN
Project No:	112-059
AutoCad Release:	2007
Filename:	112-059 RD Sheet 1.dwg
Design By:	KMG
Drawn By:	HWF
Checked By:	TLK
Date Revised:	1-18-08

CIANBRO CORPORATION
BALTIMORE, MARYLAND

BRUNNER ISLAND
POLISHING POND OUTLET MODIFICATIONS

PLAN VIEW - POLISHING POND AND
PROPOSED ACCESS TO GATE AND VAULT

Kleinschmidt
Energy & Water Resource Consultants

P.O. Box 650 141 Main Street
Pittsfield, Maine 04967
Telephone: (207) 487-3328
Fax: (207) 487-3124
www.KleinschmidtUSA.com

PROFESSIONAL
THOMAS L. KAHL
ENGINEER

1

WALKWAY AND STAIR ASSEMBLY AND DETAILS E325756
SITE PLAN E158565

REFERENCE TITLE NUMBER

ppl

BRUNNER ISLAND S.E.S.
POLISHING POND OUTLET MODIFICATIONS
PLAN VIEW - POLISHING POND AND
PROPOSED ACCESS TO GATE AND VAULT

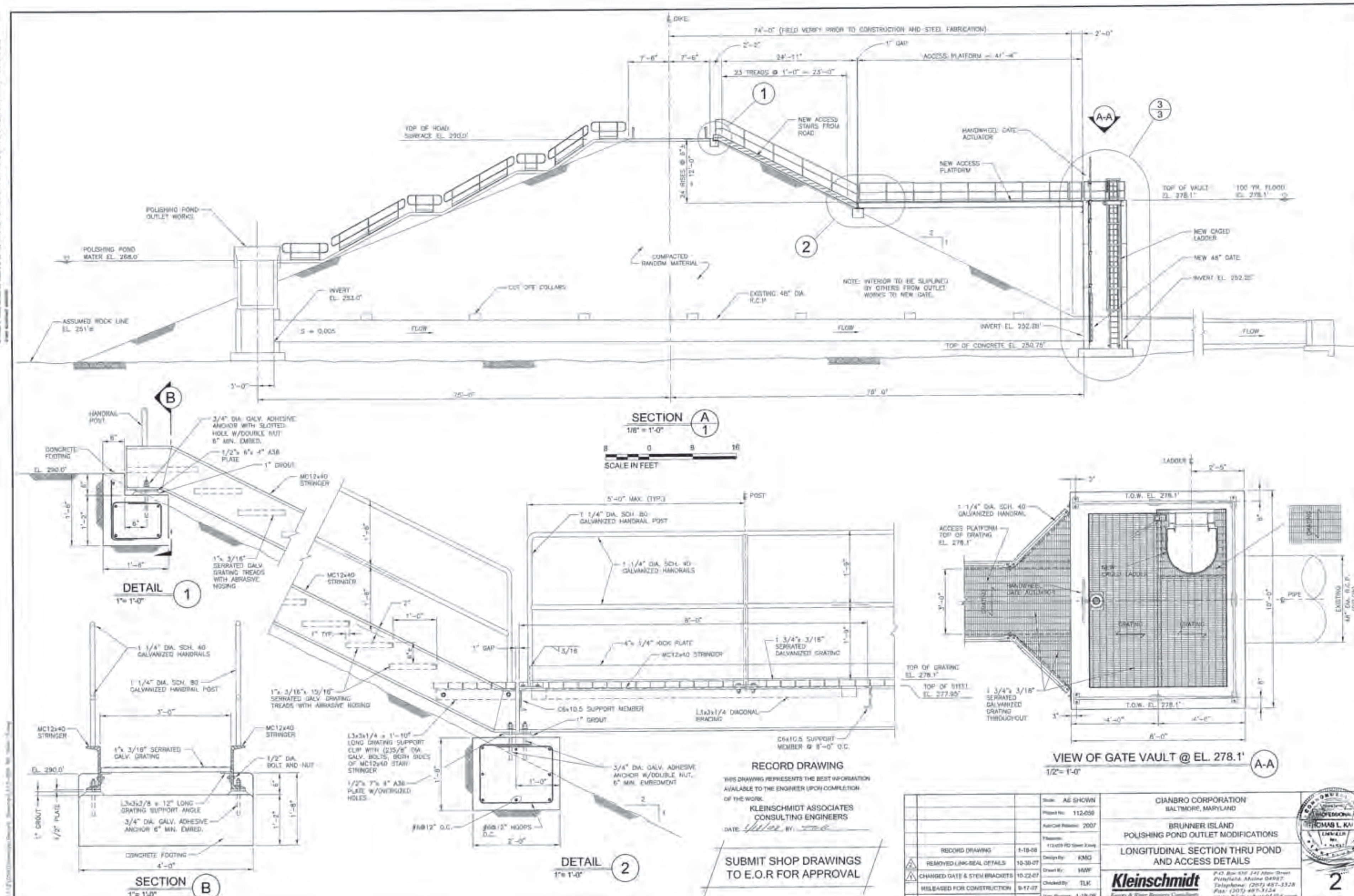
SCALE: AS SHOWN

APPROVED KLEINSCHMIDT DATE

PPL CORP.

PPL DRAWING NO. E325734 SHEET NO. 1 REV. 0

E325734 D.dwg



No.	Revision	Date
	RECORD DRAWING	1-18-08
2	REMOVED LINK-SEAL DETAILS	10-30-07
1	CHANGED GATE & STEM BRACKETS	10-22-07
	RELEASED FOR CONSTRUCTION	9-17-07

Scale:	AS SHOWN
Project No:	112-050
AutoCad Release:	2007
Filename:	
Design By:	KMG
Drawn By:	HWF
Checked By:	TLK
Date Revised:	1-18-08

CIANBRO CORPORATION
BALTIMORE, MARYLAND

BRUNNER ISLAND
POLISHING POND OUTLET MODIFICATIONS

LONGITUDINAL SECTION THRU POND
AND ACCESS DETAILS

Kleinschmidt
Energy & Water Resource Consultants

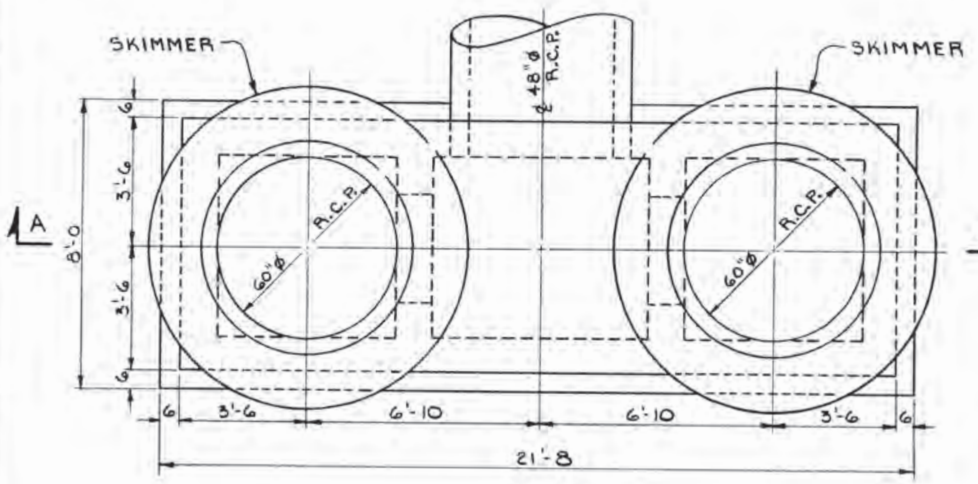
2

BRUNNER ISLAND S.E.S.

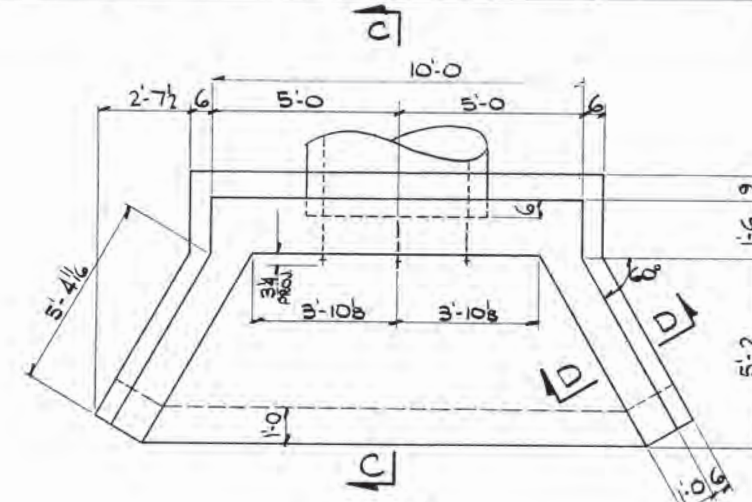
POLISHING POND OUTLET MODIFICATIONS
LONGITUDINAL SECTION THRU POND
AND ACCESS DETAILS

PPL CORP.

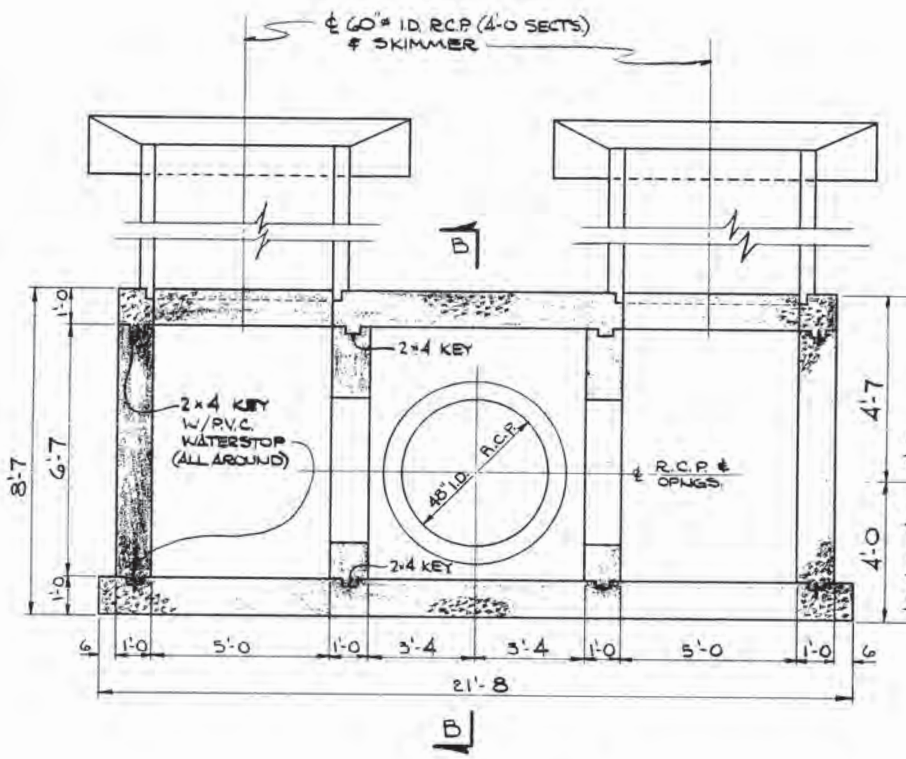
PPL DRAWING NO. E325734 SHEET NO. 2 REV. 0



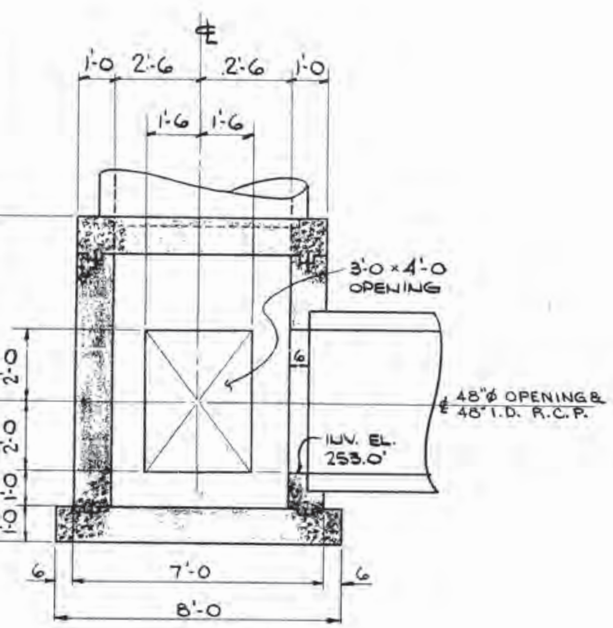
PLAN
(OUTLET WORKS)



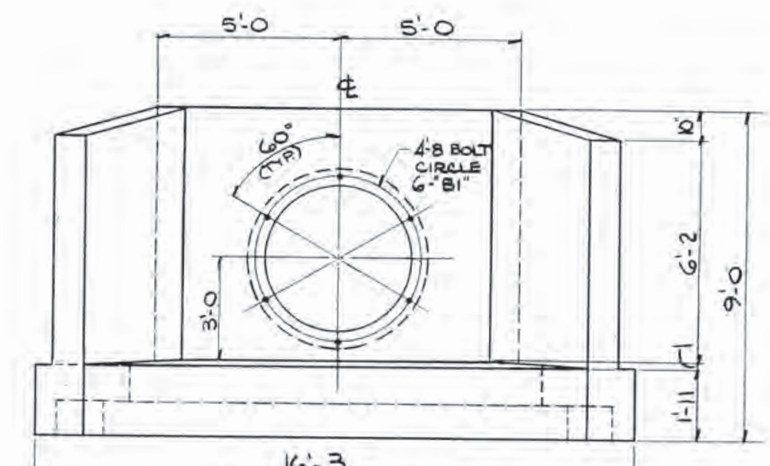
PLAN
WINGWALL
(FLAP VALVE NOT SHOWN)



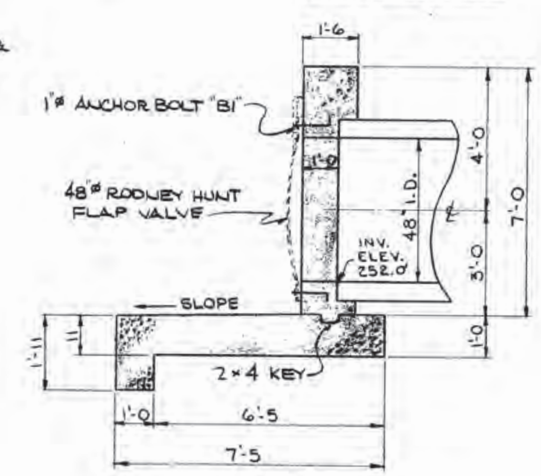
SECTION A-A



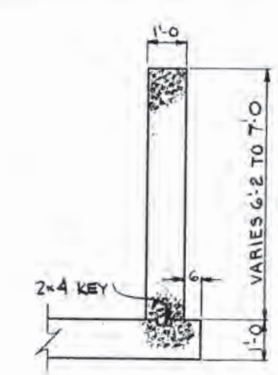
SECTION B-B



ELEVATION



SECTION C-C



SECTION D-D

LIST OF MATERIAL

QUANT.	DESCRIPTION
AS REQ'D	CU. YDS. CONCRETE.
ONE	LOT OF REINF. FOR OUTLET WORKS & WINGWALL PER A-158186
2	SKIMMERS PER DETAILS ON DRAWING E-178080
ONE	48"Ø RODNEY HUNT FLAP VALVE SERIES FV-AC TEMP NO. B-048042 W/BRONZE SEATS AS MFG. BY RODNEY HUNT CO. ORANGE, MASS.
6	ANCHOR BOLTS "B1" PER DETAIL THIS DRAWING, SH. 2.
4	PCS. 60"Ø × 4'-0 LONG EXTRA STRENGTH R.C.P. - T & G.
110	LIN. FT. P.V.C. WATERSTOP #4316 AS MFG. BY W.R. MEADOWS OF PA., INC. YORK, PA. OR EQUAL.

NOTES

REINFORCING BARS TO BE INTERMEDIATE GRADE, DEFORMED STEEL BARS, CONFORMING TO ASTM A-615 - GRADE 60.

CONCRETE SHALL BE IN ACCORDANCE WITH THE AMERICAN CONCRETE INSTITUTE'S LATEST REVISION, AND DEVELOP A 28 DAY COMPRESSIVE STRENGTH OF 3000 PSI.

REFERENCE TITLE	NUMBER
NEW FLY ASH DISPOSAL BASIN #6	E-158595
LIST OF REINFORCING	A-158186
SKIMMER DETAILS	E-178080
POLISHING POND - PLAN & DETAILS	E-178085

FRACTIONAL SCALE — DECIMAL SCALE

NO.	DATE	ER.	REVISION	BY	CH.	APPROVED

LOC CODE: F H70 — DESCRIPTION: BSN 6 OUT & WINGWL PLN & SECT — SORT: C

ER-
ER-102551
SCALE-AS SHOWN
DATE-7-31-79
DRAWN-
CHECKED-
LEADER-
APPR'D-
APPR'D-

BRUNNER ISLAND S.E.S.
ASH BASIN #6
OUTLET WORKS & WINGWALL
PLANS, ELEVATIONS & SECTIONS

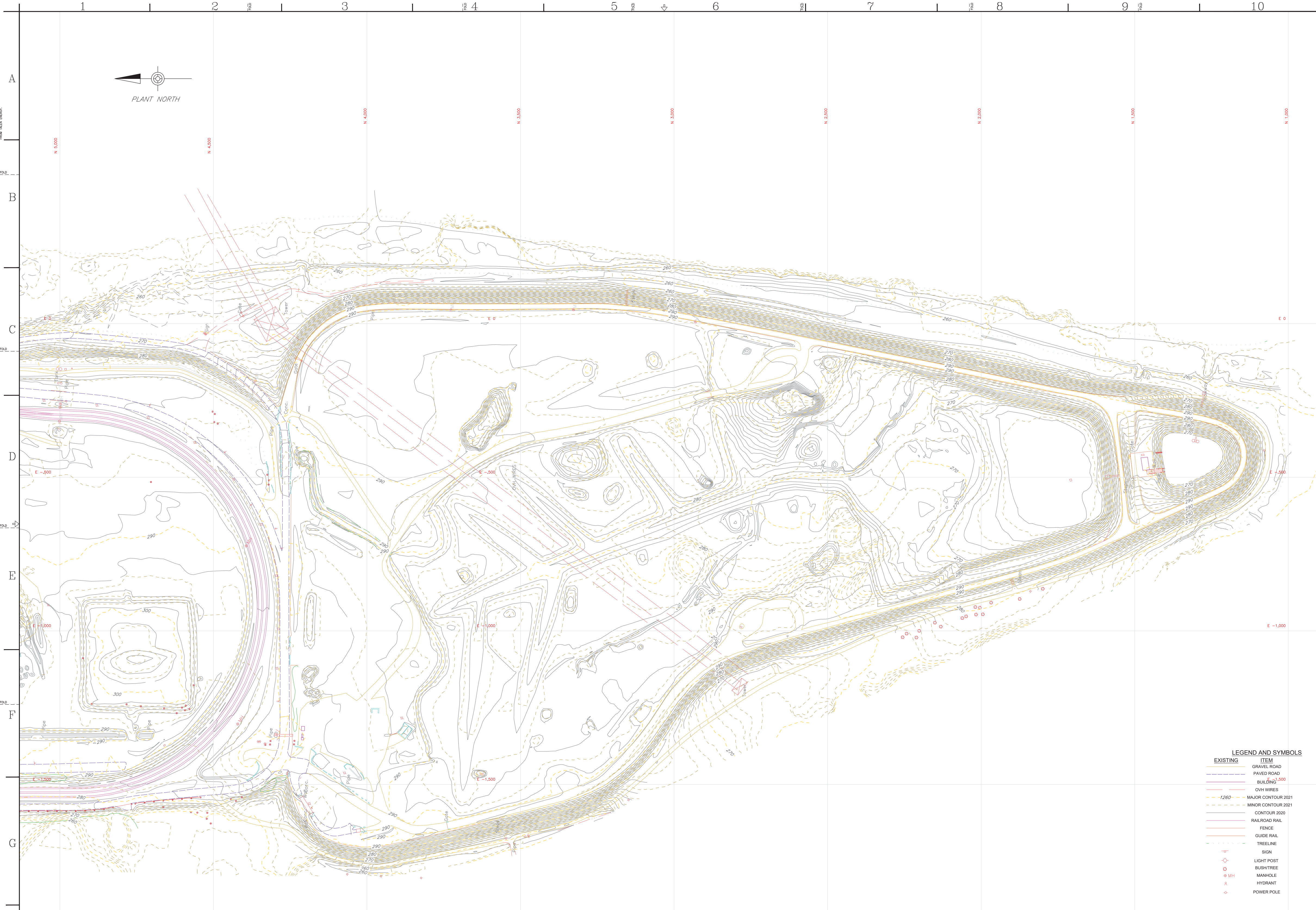
PENNSYLVANIA POWER & LIGHT COMPANY
ALLENTOWN, PA. Nº 1 OF 2

APPROVED John A. Stefanik 7/31/79
RESPONSIBLE ENGINEER

D-158185-0

PP&L Letter - Height Drafting Standard: Dwg. No. - 3/8"; Title - 3/16"; Subtitle - 5/32"; Letter Figures - 1/8" Min.

Appendix A.2 – 2021 Survey of Brunner Ash Basin Plant Datum



NOTES:

1. Aerial photography dated April 28 2021 flown by Borton Lawson.
2. Horizontal control is a plant based system provided by Talen Enregy Services.
3. Vertical control is based on NAVD29 datum provided by Talen Energy Services.
4. Reporting year contours are in tan and dark tan. Previous report year contours are grey.

NO.	DATE	ACCT.	REVISION	BY	REVIEWED	APPROVED
23	6/21		ADDED 2021 ASH CONTOURS & CHANGED TITLE TO REFLECT 2020 RESULTS	SJN	–	–

James A. Kovalik, PLS
PA #SU-075508
FOR BORTON-LAWSON ENGINEERING

ACCT–
SCALE– 1"=100'
BY– SJN

BRUNNER ISLAND S. E. S
ASH BIN NO. 6 PLANT DATUM
2021 TOPOGRAPHICAL MAPPING
FOR 2020 OPERATIONAL REPORT
EAST MANCHESTER TOWNSHIP YORK CO., PA

APPROVED DATE TALEN ENERGY

DRAWING NO. E244272 SHEET NO. 1 REV. 23

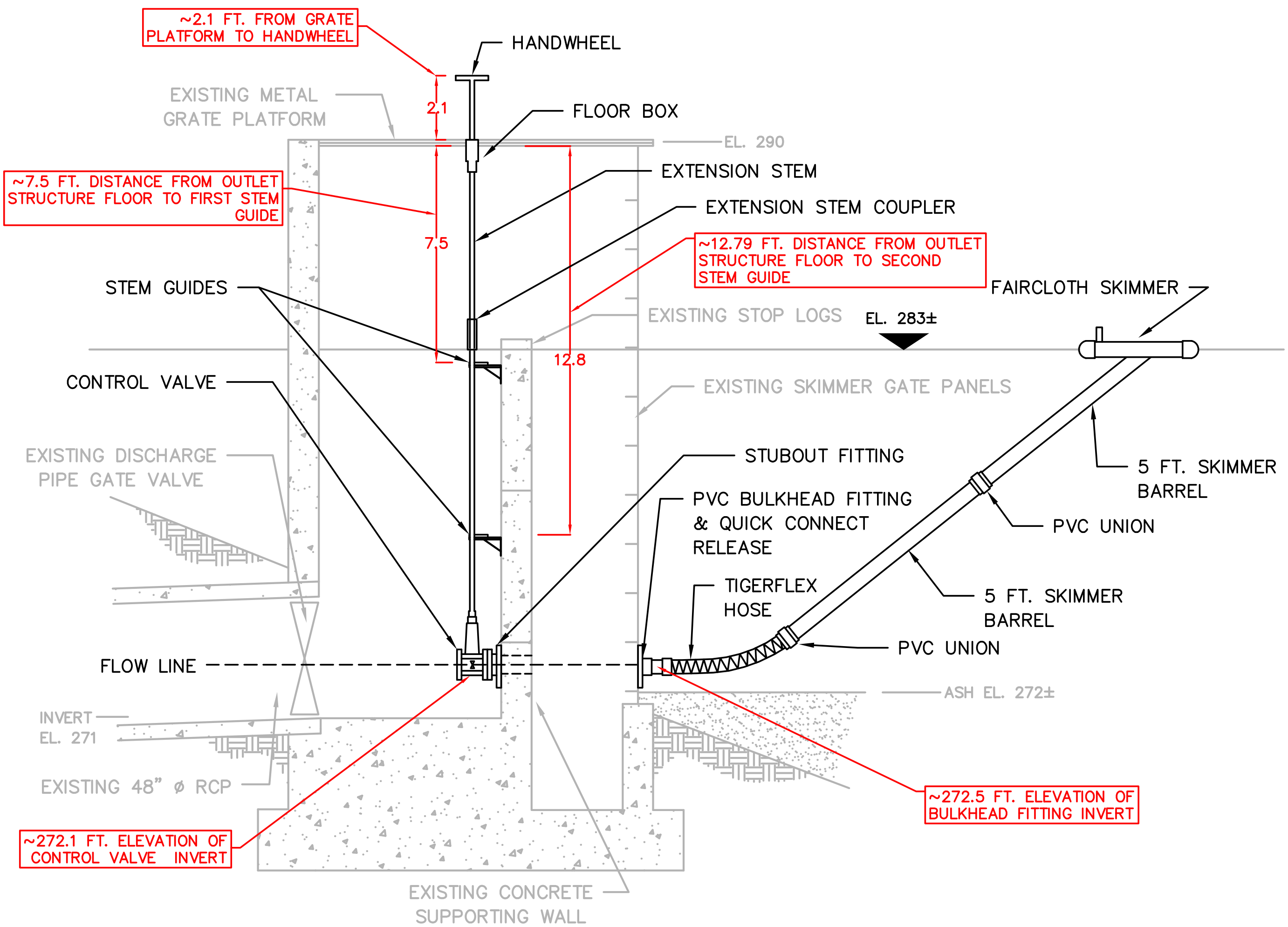
Brunner_Ash_Basin_Plant_Datum_20210429.dwg

June 8, 2021

Appendix A.2-1

Appendix A.3 – Ash Basin No. 6 Outlet Modifications Summary

2019 Outlet Structure Modification to install dewatering system



NOTE: 1.) WATER ELEVATION SHOWN ON JUNE 3RD, 2019 AT TIME OF MODIFICATION.

BRUNNER ISLAND ASH BASIN 6
OUTLET STRUCTURE MODIFICATIONS

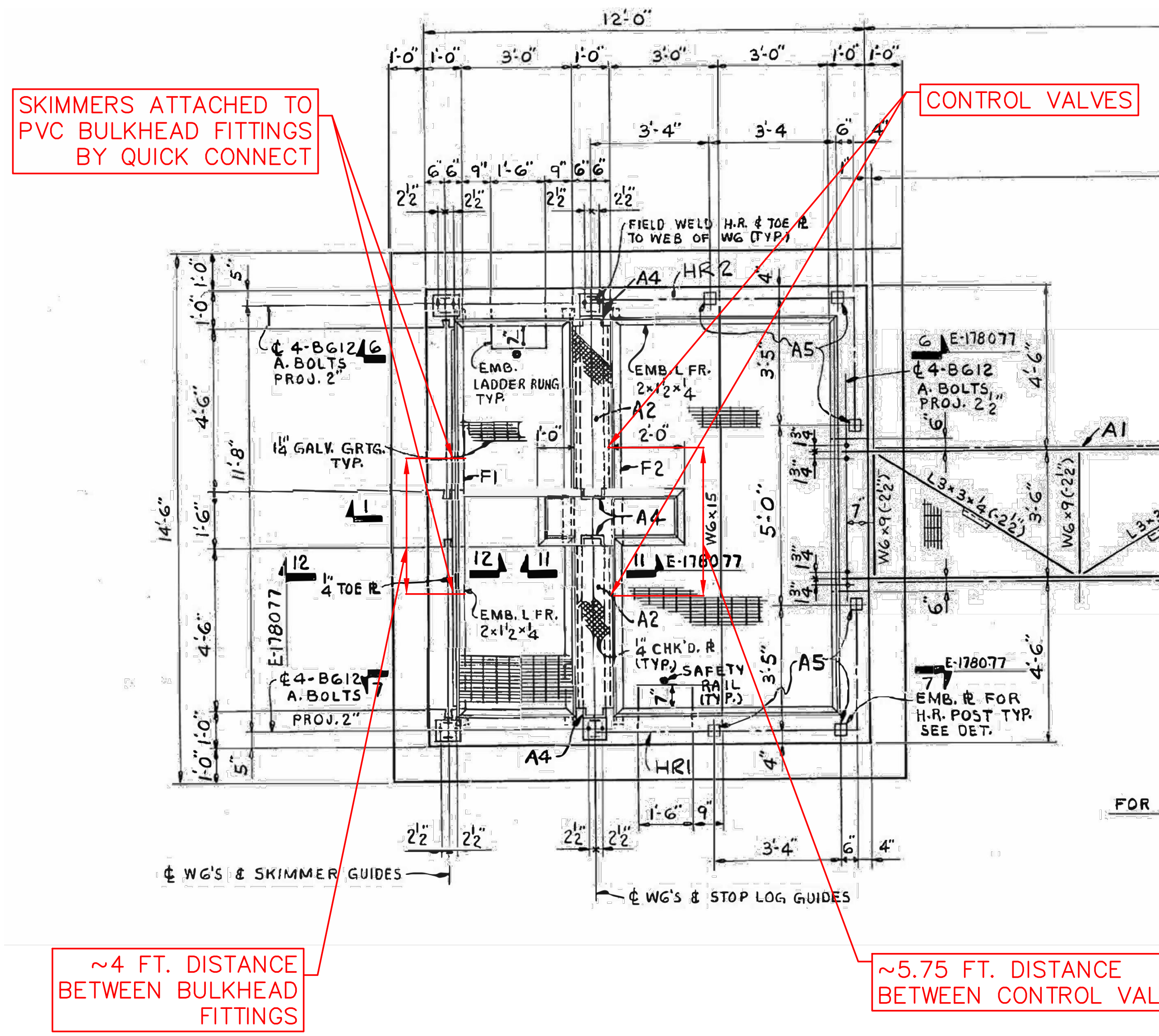
BRUNNER ISLAND ASH BASIN 6
EAST MANCHESTER TWP, YORK COUNTY, PENNSYLVANIA

FIGURE 2

ADVANCED GeoServices
WEST CHESTER, PENNSYLVANIA 19380

DATE: JUNE 2019 | PROJ NO.: 2011-2740 | SCALE: NTS

2019 Outlet Structure Modification to install dewatering system



BRUNNER ISLAND ASH BASIN 6
OUTLET STRUCTURE MODIFICATIONS

BRUNNER ISLAND ASH BASIN 6

EAST MANCHESTER TWP, YORK COUNTY, PENNSYLVANIA

ADVANCED GeoServices

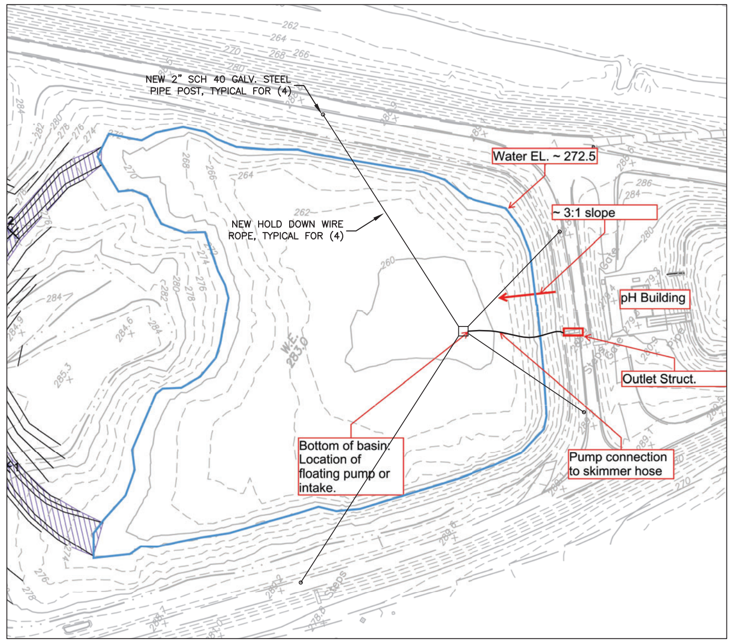
WEST CHESTER, PENNSYLVANIA 19380

DATE: JUNE 2019 | PROJ NO.: 2011-2740 | SCALE: NTS

FIGURE 3

F:\Projects\2011\20112740-Brunner Ash Basin 6 Closure\Cad\dwg\Outlet Structure Modifications FIG3.dwg

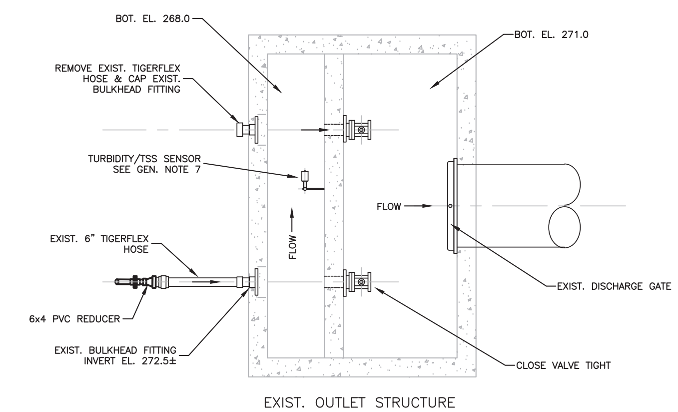
2020 Outlet Structure Modification to install permanent dewatering pump



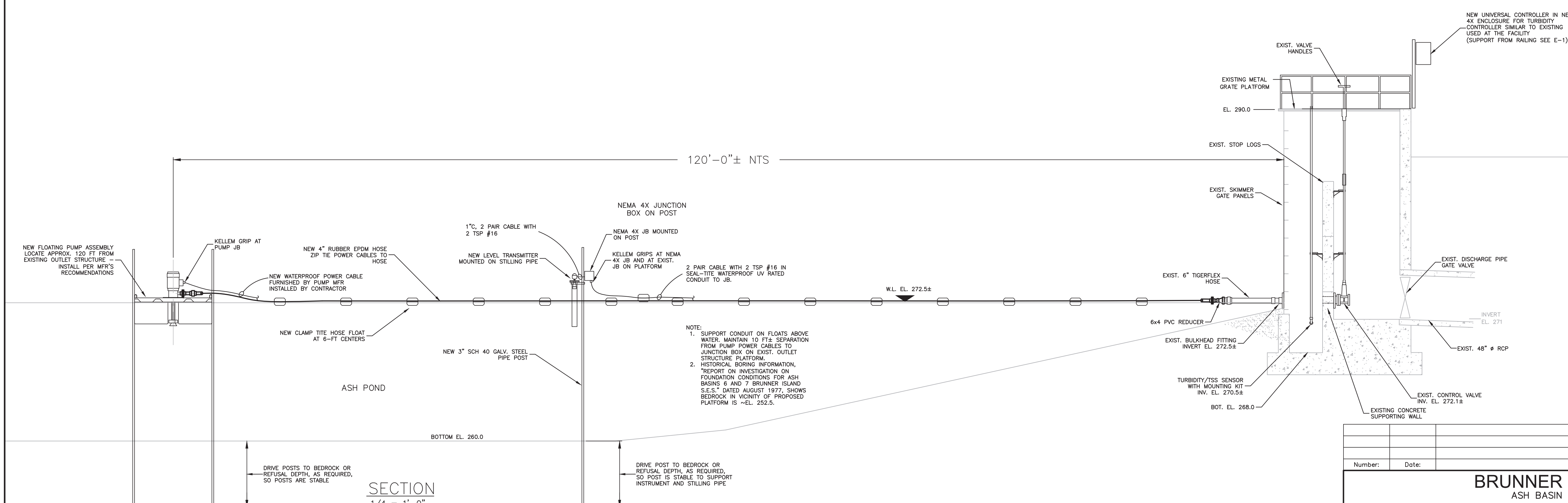
SITE PLAN
NTS

GENERAL NOTES:

1. FLOATING PLATFORM SHALL BE 7'x7' 4" DESIGNED FOR MOUNTING AN S-SERIES VERTICAL PUMP RATED AT 400 GPM AT 125' TDH IN THE FUTURE . PLATFORM SHALL BE PROVIDED WITH A VERTICAL PUMP RATED AT 100 GPM AT 20' TDH, 5 HP TEFC MOTOR 460/3/60 PLATFORM SHALL BE CONSTRUCTED OF CORROSION RESISTANT MATERIALS AND PROVIDED WITH U-SHAPED HOLDING LUGS AT CORNERS. FLOATING PLATFORM WITH PUMP SHALL AS MADE BY CRISAFULLI OR APPROVED EQUAL. CONTACT TRAVIS STROH, TEL. 888-817-7011
2. TURBIDITY SENSOR SHALL BE SOLITAXsc TURBIDITY & SUSPENDED SOLIDS SENSOR WITH sc200 UNIVERSAL CONTROLLER AS MANUFACTURED BY HACH OR APPROVED EQUAL.
3. LEVEL TRANSMITTER SHALL BE ROSEMOUNT 5408 TWO-WIRE NON-CONTACTING RADAR OR APPROVED EQUAL.
4. RUBBER HOSE SHALL BE QUALITY EPDM RUBBER SUCTION/DISCHARGE HOSE RATED AT 150 PSI AS MODEL RN1 AS MANUFACTURED BY HOSECRAFT USA OR APPROVED EQUAL. PROVIDE CORROSION RESISTANT FITTINGS AS REQUIRED. PROVIDE
5. SCHEME 1 – FLOATING PUMP PLATFORM WILL BE HELD USING 3/8" GALVANIZED STEEL WIRE ROPE TIED TO THE EYES ON THE PLATFORM AND TO 3" SCH 40 GALV. STEEL POST DRIVEN SECURELY TO THE BANKS. LOCATE IN THE FIELD. PROVIDE GALV. STEEL FITTINGS AS REQUIRED. PROVIDE WIRE ROPE WITH SUFFICIENT SLACK TO ALLOW FOR VERTICAL MOVEMENT OF THE PLATFORM DUE TO VARYING WATER LEVEL IN THE POND.
6. SCHEME 2 – FLOATING PUMP PLATFORM WILL BE HELD USING O-RINGS IN EACH OF THE FOUR (4) – 3" SCH 40 GALV. STEEL POST DRIVEN SECURELY CLOSE TO CORNERS OF THE PLATFORM. TIE O-RING TO THE EYES ON THE PLATFORM WITH GALV. STEEL CHAIN. SET UP SHALL ALLOW THE PLATFORM TO MOVE VERTICALLY AS A FLOATING DOCK.
7. TURBIDITY/TOTAL SUSPENDED SOLIDS MONITOR SHALL BE HACH SOLITAXsc, STAINLESS STEEL WITH STAINLESS STEEL POLE MOUNT KIT FOR IMMERSION MOUNTING. PROVIDE EXTENSION POLE, BRACKETS AND SUPPORTS AS REQUIRED. LOCATION AS DETERMINED IN THE FIELD.



PLAN @ BOTTOM
1/4 = 1'-0"



SECTION
1/4 = 1'-0"

FOR CONSTRUCTION

Number:	Date:	Revision:

BRUNNER ISLAND
ASH BASIN NO. 6
EAST MANCHESTER TOWNSHIP, PENNSYLVANIA

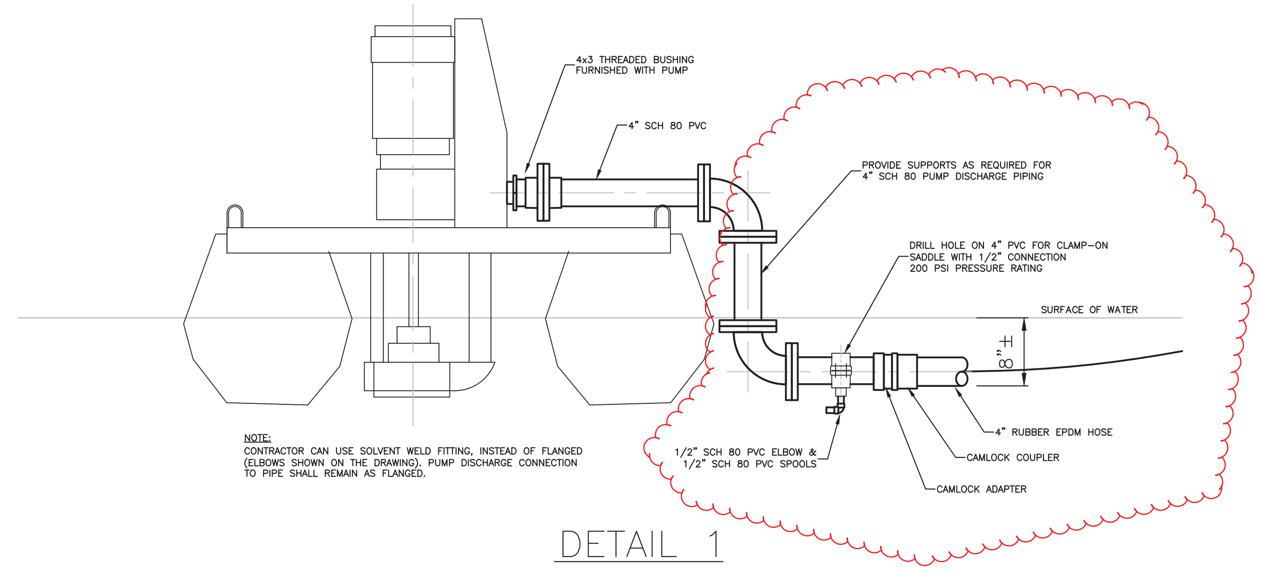
MECHANICAL/ELECTRICAL
SITE PLAN AND SECTION

ADVANCED GeoServices
Engineering for the Environment. Planning for People.
1055 ANDREW DRIVE, SUITE A
WEST CHESTER, PENNSYLVANIA 19380
Tel: 610.840.9100 Fax: 610.840.9199 Web: www.advancedgeoservices.com

Scale:	NTS
Drawn By:	WCG
Checked By:	JSD
Project Mgr.:	CTR
Originated By:	KO
Project No.:	2011-2740
Drawing Date:	11/7/19
Sheet No.:	1 OF 2
Revision Number:	0

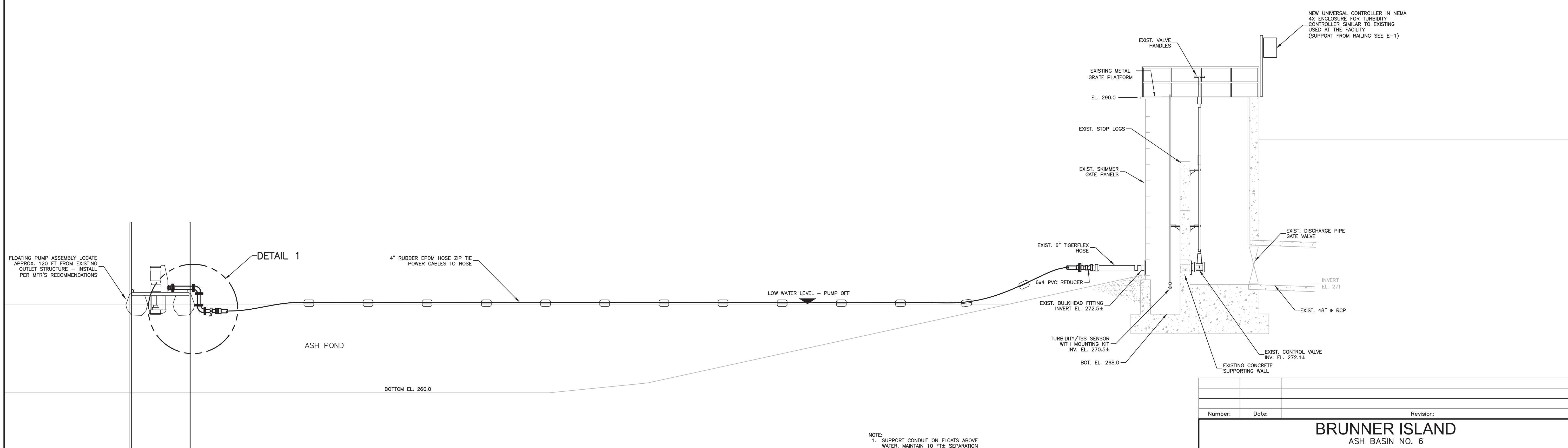
M-1

2020 Outlet Structure Modification to install permanent dewatering pump



NOTE:
CONTRACTOR CAN USE SOLVENT WELD FITTING, INSTEAD OF FLANGED (ELBOWS SHOWN ON THE DRAWING). PUMP DISCHARGE CONNECTION TO PIPE SHALL REMAIN AS FLANGED.

DETAIL 1
NTS



SECTION – POND AT LOW WATER LEVEL CONDITION
1/4 = 1'-0"

NOTE:
1. SUPPORT CONDUIT ON FLOATS ABOVE WATER. MAINTAIN 10 FT± SEPARATION FROM PUMP POWER CABLES TO JUNCTION BOX ON EXIST. OUTLET STRUCTURE PLATFORM.
2. HISTORICAL BORING INFORMATION, "REPORT ON INVESTIGATION ON FOUNDATION CONDITIONS FOR ASH BASINS 6 AND 7 BRUNNER ISLAND S.E.S." DATED AUGUST 1977, SHOWS BEDROCK IN VICINITY OF PROPOSED PLATFORM IS ~EL. 252.5.

Number:	Date:	Revision:

BRUNNER ISLAND
ASH BASIN NO. 6
EAST MANCHESTER TOWNSHIP, PENNSYLVANIA

MECHANICA
SECTION AND DETAILS

ADVANCED GeoServices
Engineering for the Environment. Planning for People.
1055 ANDREW DRIVE, SUITE A
WEST CHESTER, PENNSYLVANIA 19380
Tel: 610.840.9100 Fax: 610.840.9199 Web: www.advancedgeoservices.com

Scale:	NTS
Drawn By:	WCG
Checked By:	JSD
Project Mgr.:	CTR
Originated By:	KO
Project No.:	2011-2740
Drawing Date:	12/9/19
Sheet No.:	3 OF 3
Revision Number:	0

M-2

Appendix B. Drawdown Analysis

GEOTECHNICAL ENGINEERING REPORT

Brunner Island SES Ash Basin No. 6 Transient Seepage and Slope Stability Evaluation
East Manchester Township, York County
Pennsylvania

Schnabel Reference 21C25010.00
August 25, 2021

Keith Toombs, PE
Project Engineer

Scott A. Raschke, PhD, PE
Sr. Vice President

Schnabel ENGINEERING

Schnabel ENGINEERING
T 518.348.8575
28 Corporate Drive, Suite 204 / Clifton Park, NY 12065 schnabel-eng.com

August 25, 2021

Mr. Adam Jones, PE
HDR
970 Baxter Boulevard Suite 301
Portland, ME 04103

Subject: **Brunner Island SES Ash Basin No. 6 Transient Seepage and Slope Stability Evaluation; East Manchester Township, York County, Pennsylvania (Schnabel Project 21C25010.00)**

Dear Mr. Jones:

SCHNABEL ENGINEERING, LLC (Schnabel) is pleased to submit our Geotechnical Engineering Report for this project. This report includes tables, figures, and attachments with relevant data pertinent to this study. This study was performed in accordance with our proposal dated March 30, 2021, as authorized by HDR, Subconsultant Agreement Number 10001000-Conformed, signed by Scott A. Raschke of Schnabel on May 27, 2021, and James Carnahan of HDR on June 4, 2021.

We appreciate the opportunity to be of service for this project. Please call us if you have any questions regarding this report.

PREVIOUS EVALUATIONS

Schnabel Engineering Consultants (SEC) previously performed seepage and stability analyses in 2012 and 2015 to evaluate a rapid drawdown (RDD) event of the Susquehanna River along the eastern-most impoundment dike at the Brunner Island Ash Basin No. 6 facility. The dike is roughly 30 feet (ft) tall with an approximate 2.5H:1V upstream (i.e., ash storage basin side) slope and an approximate 2H:1V downstream (i.e., river side) slope, and is located adjacent to the west bank of the Susquehanna River. The original studies and their findings are presented in SEC's February 17, 2012, report to Pennsylvania Power and Light (PPL) Generation, LLC (SEC, 2012) and SEC's December 17, 2015, report to Talen Generation, LLC (Talen) (SEC, 2015).

For the 2012 study, a transient seepage analysis was performed to consider slope stability under an RDD event from a 500-yr recurrence interval flood corresponding to a river elevation at EL 288.8. The 2015 study re-evaluated the RDD event from a level slightly greater than a 1,000-yr event corresponding to a river elevation at EL 289.5. The duration of the various stages was based on our interpretation and evaluation of readily available historical data prepared by others. The 2012 and 2015 studies calculated a minimum factor of safety (FOS) for RDD to be greater than 1.1 for both the initial and the revised scenarios and conditions. The most critical representative cross-section (Section 1-1 at Station 21+80) was chosen and used for both evaluations based on observed piezometric levels at the time of the evaluation.

SCOPE

Current Coal Combustion Residuals (CCR) regulations by the United States Environmental Protection Agency (USEPA) require periodic report updates including re-evaluation of the stability of the embankment slopes of the dike. Talen requested that HDR retain Schnabel as a subconsultant to update the previous RDD analyses to the current conditions.

Our agreement signed by HDR on June 4, 2021, defines the scope of the current study which includes re-evaluation of the RDD event modeled for the 2015 analysis using updated data including current geometric conditions of the representative cross-section and phreatic levels within the impoundment, storage basin, and dike. Similar to the 2015 study, the maximum surcharge river elevation was evaluated at EL 289.5 ft corresponding to 0.5 ft above the 1,000-yr recurrence interval flood stage elevation. HDR had initially provided Schnabel with updated cross-sections that included the current external and internal geometries of the embankment and ash storage, as well as the assumed phreatic levels within the cross-section shown in Attachments 1a and 1b. The "2021 assumed Normal Pool El 262.0" was used in our current evaluation as the initial steady state condition. We were later provided updated Attachments 2a and 2b, which reflect a higher phreatic level ("PZ6-7 average El 271.5") from the centerline of the embankment through the embankment/clay liner and ash fill.

Services not described in our agreement are not included in this study. We would be happy to provide any additional services to the project team that are required.

PROJECT APPROACH

Our current evaluation and model are similar to SEC's previous analyses in 2012 and 2015; however, the model was revised with updated external and internal geometries reflected in the representative cross-section and the phreatic levels provided to us. The basis of our analyses and the development of the transient loading condition and parameters adopted for the current model are described in detail in the 2012 and 2015 reports.

The previous seepage and stability studies performed in 2012 and 2015 used SEEP/W and SLOPE/W within the GeoStudio 2007 (ver. 7.14) suite of software. GeoStudio 2007 is no longer supported by the developer. The current seepage and stability models were updated to SEEP/W and SLOPE/W within the GeoStudio 2019 R2 (ver. 10.1.1.18972) suite of software.

TRANSIENT SEEPAGE ANALYSIS AND MODELING

Seepage was modeled using SEEP/W which is a two-dimensional finite element computer program commonly used to model unconfined and confined seepage problems, including groundwater movement and pore water pressure distribution within porous materials, such as soil and rock. SEEP/W can be used to model seepage conditions and evaluate various parameters, including hydraulic head/pore water pressure distribution, hydraulic gradient, volume of flow, and many others. SEEP/W can be used to model both steady-state and transient seepage conditions. Steady-state conditions include situations in which model parameters (soil properties, boundary conditions, etc.) do not change over time. Transient conditions involve scenarios in which model parameters do change over time.

The previous analyses were based on a normal headwater elevation (i.e., the elevation of groundwater within the storage basin) at EL 288.0. According to the data provided to Schnabel, dewatering of the storage basin was begun in mid-2019, and the normal headwater level was observed to stabilize around EL 262 in mid-2020. The 2021 normal headwater elevation was assumed to be at EL 262.0 through the ash fill, rising to approximately EL 268 below the crest of the embankment, and daylighting near the downstream toe at about EL 263.5. Based on data provided by HDR, the surcharge pool within the storage basin was assumed to be at EL 289.0. The assumed phreatic surfaces are shown on updated cross-sections provided to us in Attachments 1 and 2, as well as the 2021 readings within the piezometers installed in the embankment. Both normal and surcharge phreatic conditions were considered in the current RDD analysis using the updated model.

As evaluated in the 2015 study, the current model also used a maximum surcharge (flood) elevation of the Susquehanna River at EL 289.5, which corresponds to 0.5 ft above the 1,000-yr recurrence interval loading event. The progression of the 8-day transient seepage evaluation incorporates a river rise over a 2-day period from EL 252 (normal river elevation) to EL 289.5 (flood elevation), followed by a 4-day period where the river remains at EL 289.5, followed by a 2-day period where the river recedes to EL 252. The hydrologic and hydraulic (H&H) analysis used to evaluate the flooding scenario used for the Brunner Island transient RDD event are discussed in detail in the 2012 and 2015 SEC reports. The seepage model was evaluated using the river loading varying in ½-day increments for the RDD event to observe the progression and regression of the wetting front within the embankment model.

It should be noted that the surcharge pool and associated assumed phreatic surface developed by HDR reflects a steady-state (stabilized) condition. It would take a significant period of time to develop steady-state pore water pressures under the surcharge loading within the basin particularly with the significant reduction of pore water pressures compared to the pre-dewatering conditions previously evaluated in earlier studies. Therefore, the evaluation also considered the transient nature of the storage surcharge pool phreatic conditions from the current steady-state conditions at normal pool prior to a transient flood loading condition. The transient model considered phreatic levels in the basin beginning at current normal pool (EL 262) and increasing to a surcharge loading at EL 289. The pore water pressures developing from the surcharge pool loading were evaluated for a period of 1,000 days at EL 289 allowing for consideration of antecedent transient pore water pressures from the surcharge basin loading prior to a potential transient riverine flood loading event.

The isotropic and anisotropic parameters for the hydraulic conductivity of the embankment fill material were not modified for the current 2021 evaluation. The previous studies used the following cases based on the saturated hydraulic conductivity used for the embankment fill material:

<u>Isotropic Hydraulic Conductivity (Embankment Fill)</u>
Case 1: $K_v = K_h = 6.8*10^{-6}$ ft/sec (maximum saturated hydraulic conductivity, isotropic)
Case 2: $K_v = K_h = 2.8*10^{-6}$ ft/sec (average saturated hydraulic conductivity, isotropic)
Case 3: $K_v = K_h = 6.8*10^{-9}$ ft/sec (minimum saturated hydraulic conductivity, isotropic)

<u>Anisotropic Hydraulic Conductivity (Embankment Fill)</u>
Case 4: $K_v = 0.50 * K_h = 2.8*10^{-6}$ ft/sec (average saturated hydraulic conductivity, anisotropy ratio = 2)
Case 5: $K_v = 0.25 * K_h = 2.8*10^{-6}$ ft/sec (average saturated hydraulic conductivity, anisotropy ratio = 4)
Case 6: $K_v = 0.13 * K_h = 2.8*10^{-6}$ ft/sec (average saturated hydraulic conductivity, anisotropy ratio = 8)

While the “Ash Fill (Storage)” and “Clay Liner” materials were updated to a saturated/unsaturated model type due to the change in normal pool in the storage basin, the saturated hydraulic conductivities of the materials in the model were not changed from those in the 2012 and 2015 studies. A summary of the hydraulic conductivities, excluding the Embankment Fill, are included in Table 1.

Table 1: Material Hydraulic Conductivity Values (Excluding Embankment Fill)

Material	Material Model	Saturated Hydraulic Conductivity (ft/sec)	Anisotropy Ratio
Ash Fill (Storage)	Saturated / Unsaturated	1e-08	1
Clay Liner	Saturated / Unsaturated	1e-09	1
Native Soil	Saturated	1e-08	1
Bedrock	Saturated	1e-10	1

DEEP-SEATED GLOBAL SLOPE STABILITY ANALYSIS

The downstream side of the impoundment dike was evaluated for global stability using Spencer’s Method, as implemented in SLOPE/W. Soil parameters (unit weight, shear strength, etc.) used in the 2012 and 2015 studies were adopted for the slope stability evaluation and were not modified. The previously described transient seepage analysis was used to model the change in pore water pressure over time, and effective shear strengths were used in the stability model.

Spencer’s Method was used to evaluate global slope stability of the downstream slope using the pore water pressure distribution from SEEP/W. The minimum FOS resulting from the RDD from a surcharge level 0.5 ft higher than the 1,000-yr flood stage to normal water levels in the river was calculated at discrete time increments starting at flood stage (EL 289.5), and ending when river levels return to the normal water level elevation (EL 252). Consistent with the 2012 and 2015 studies, the model only considered deep-seated potential failure planes to avoid evaluating minor surficial sloughing. The model limited failure planes to extend from the crest of the embankment to the downstream embankment toe or beyond. The slope stability of the embankment was evaluated at each ½-day time step within the transient seepage analysis to assess the critical FOS during the RDD event. The minimum FOS for each slope stability model was recorded at the critical time-step within the transient evaluation of the river drawdown and may occur prior to the river reaching its original normal pool condition (i.e., the minimum FOS values among the slope stability models may occur at different time steps within the transient riverine drawdown).

The 2012 study showed that Case 1 and Case 6 were the most critical isotropic and anisotropic scenarios, respectively (i.e., these cases provided the lowest FOS results). Therefore, the current evaluation was limited to these two cases for RDD under transient riverine loading and variable upstream (ash storage) pool conditions including steady-state normal pool, transient surcharge pool, and steady-state surcharge pool. The FOS results of the slope stability evaluation during RDD are reported in Table 2.

Table 2: Minimum Factors of Safety for RDD from EL 289.5 to Normal River Water Levels

Condition	Embankment Hydraulic Conductivity (k, ft/sec)	Min. FOS (Plate #)
Isotropic Hydraulic Conductivity (Case 1)		
Steady-state normal storage pool	$K_v = K_h = 6.8*10^{-6}$ ft/sec (max sat hydr cond, isotropic)	1.1 (Attachment 3)
1,000-day transient surcharge storage pool		1.1 (Attachment 4)
Anisotropic Hydraulic Conductivity (Case 6)		
Steady-state normal storage pool	$K_v = 0.13 * K_h = 2.8*10^{-6}$ ft/sec (avg sat hydr cond, anisotropy ratio = 8)	1.2 (Attachment 5)
1,000-day transient surcharge storage pool		1.2 (Attachment 6)

There are a couple of observations worth noting based on the 1,000-day transient surcharge evaluation shown in Attachments 4 and 6. First, the ash fill is completely saturated with a phreatic surface at the level of the ash within the basin, well above an initial “normal pool” represented by the “2021 assumed Normal Pool El 262.0” in Attachment 1 and “PZ6-7 average El 271.5” in Attachment 2, resulting in comparable FOS values to the initial “normal pool” using the “2021 assumed Normal Pool El 262.0” in Attachments 3 and 5. This suggests that the RDD Minimum FOS is not particularly sensitive to the initial phreatic levels within the ash, which is consistent considering also the earlier SEC evaluations. Second, the phreatic surface from the 1,000-day transient surcharge RDD evaluation results in approximately the same or higher piezometric levels from the approximate centerline of the embankment outward towards the riverine side of the embankment compared to the “assumed phreatic surface for surcharge condition” shown by HDR in Attachments 1 and 2. This suggests the 1,000-day transient surcharge RDD evaluation includes reasonably conservative piezometric levels, which are naturally higher within the portion of the embankment subject to the riverine flood loading and control the RDD Minimum FOS.

CONCLUSIONS

The USEPA Final Rules 257.73 and 257.74 require an analysis of the downstream slope of a CCR surface impoundment that is subject to rapid drawdown of an adjacent water body in addition to an evaluation of structural stability during low pool of the adjacent water body. The USEPA rules do not provide minimum factors of safety for the analyses. Conventional guidelines for minimum factors of safety include recommendations in United States Army Corps of Engineers (USACE) engineering manuals. Recommended minimum values of 1.1 (drawdown from maximum riverine surcharge pool) to 1.3 (drawdown from maximum riverine storage [normal] pool) are provided for new earth and rock-fill dams in Table 3-1 in USACE EM 1110-2-1902 (USACE, 2003). Recommended minimum values of 1.0 to 1.2 for new and existing levees, and other embankments and dikes, are provided in USACE EM 1110-2-1913 (USACE, 2000).

The minimum FOS results for stability of the downstream embankment slope under the rapid drawdown scenarios presented herein correspond to values of 1.1 to 1.2 under steady-state normal storage pool and 1,000-day transient surcharge storage pool conditions. These results meet and/or exceed the minimum values of 1.0 and 1.1 provided in the USACE engineering manuals. Additionally, the likelihood of the 1,000-yr recurrence interval event corresponding with a prolonged steady-state surcharge storage pool condition is unlikely, and therefore the resultant FOS of 1.1 provides a lower bound for this event. Similarly, floods with more frequent recurrence intervals (e.g., 50-yr, 100-yr, etc.) would result in even higher factors of safety if all other factors remain the same.

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SEEP/W included in GeoStudio 2019R2 (seepage analysis).

SLOPE/W included in GeoStudio 2019R2 (slope stability analysis using Spencer's Method).

United States Army Corps of Engineers (USACE). (2000). "Engineering Manual (EM) 1110-2-1913: Design and Construction of Levees." Washington, DC.

United States Army Corps of Engineers (USACE). (2003). "Engineering Manual (EM) 1110-2-1902: Slope Stability." Washington, DC.

Various Piezometer Readings, Groundwater Levels, and Dewatering Data, as provided by HDR Engineering, Inc., on June 3, 2021.

LIMITATIONS

We based the analyses and recommendations submitted in this report on the information revealed by the exploration performed by others and interpretation of data prepared by others. We attempted to provide for normal contingencies, but the possibility remains that unexpected conditions may exist.

We prepared this report to aid in the evaluation of this site and to assist in the geotechnical evaluation described herein. We intend it for use concerning this specific project. We based our recommendations on information on the site and understanding of information as described in this report.

We have endeavored to complete the services identified herein in a manner consistent with that level of care and skill ordinarily exercised by members of the profession currently practicing in the same locality and under similar conditions as this project. No other representation, express or implied, is included or intended, and no warranty or guarantee is included or intended in this report, or any other instrument of service.

We appreciate the opportunity to be of service for this project. Please call us if you have any questions regarding this report.

Sincerely,

SCHNABEL ENGINEERING, LLC

Scott A. Raschke, PhD, PE
Sr. Vice President

KST:SAR:scc

Attachments:

(1) Initial Cross-Section (provided by HDR)
  a. Updated Section 1-1
  b. Updated Section 1-1: Historic Piezometric Levels
(2) Updated Cross-Section (provided by HDR)
  a. Section 1-1
  b. Section 1-1: Historic Piezometric Levels
(3) RDD from EL 289.5 to River at Normal Water Level Elevation with Steady-State Normal Storage Pool (Case 1: $K_v=K_h=6.8*10^{-6}$ ft/sec)
(4) RDD from EL 289.5 to River at Normal Water Level Elevation with 1,000-day Transient Surcharge Storage Pool (Case 1: $K_v=K_h=6.8*10^{-6}$ ft/sec)
(5) RDD from EL 289.5 to River at Normal Water Level Elevation with Steady-State Normal Storage Pool (Case 6: $K_v=0.13*K_h=2.8*10^{-6}$ ft/sec)
(6) RDD from EL 289.5 to River at Normal Water Level Elevation with 1,000-day Transient Surcharge Storage Pool (Case 6: $K_v=0.13*K_h=2.8*10^{-6}$ ft/sec)

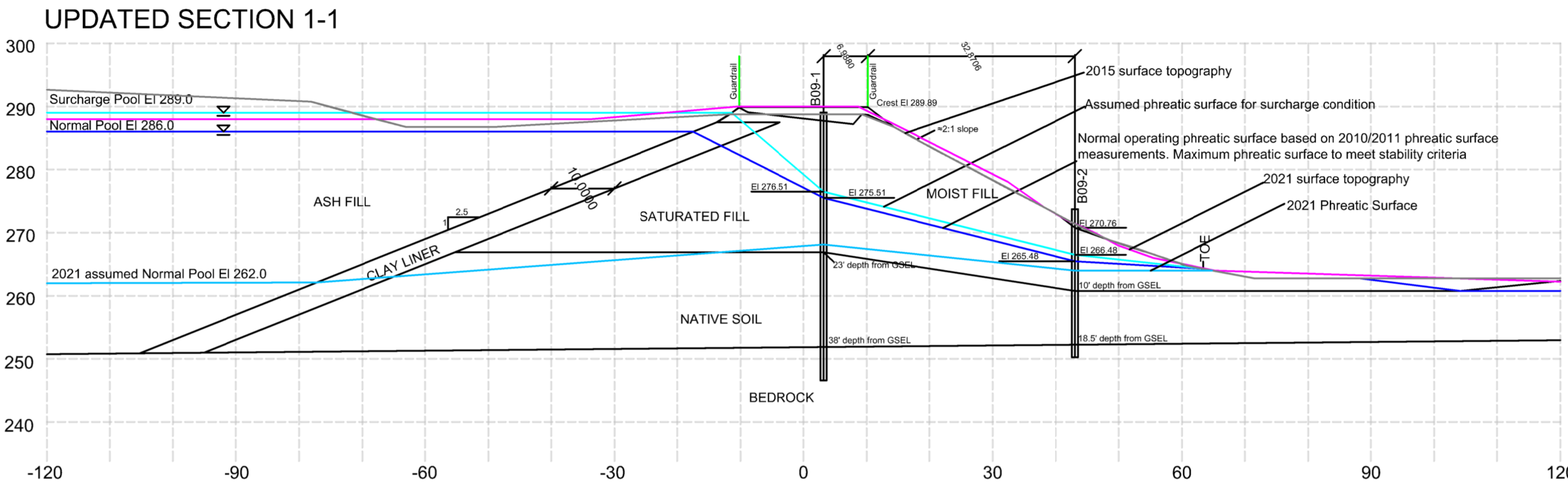
Distribution (PDF by email only):

HDR

Attn: Mr. Adam Jones, PE

Attn: Ms. Jennifer Gagnon, PE

Attachment 1a
Initial Cross-Section



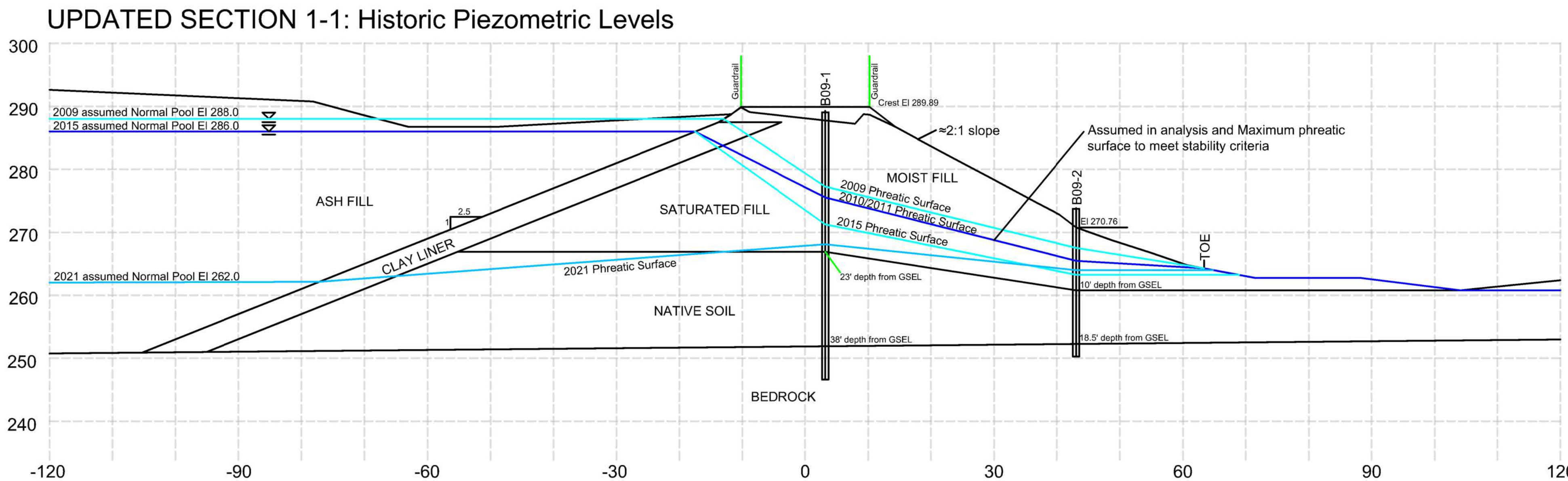
NOTES:
Surface topography from 2015 aerial survey (NAVD88 and state plane)
-Adjusted up 0.76 feet to convert to Plant datum (NGVD29)
-Crest assumed to be level at El 289.89 feet based on 2015 piezometer survey
-Surface topography from 2021 aerial survey (NGVD29) overlaid on 2015 surface topography. Topography shows only minor changes. Crest at El. 290.0 feet. Aerial survey provided to HDR by Talen as an AutoCAD file via email.

Pool level and piezometric levels:
-Normal WSEL previously assumed to be El 286.0 feet in previous analysis, 2021 Normal WSEL 262.0 feet, 24 feet lower.
-Surcharge WSEL assumed to be El 289.0 feet
-Stratigraphy and piezometric levels from boring logs, historical piezometer data, and Dwg E158595-4
-See updated Section 1-1: Historic Piezometric Levels for history of pool and piezometric levels.

No new boring data or other changes are known to have occurred at Ash Basin 6 that would result in the need to adjust stratigraphy and material properties. Therefore material properties and stratigraphy from 2009 analysis remain valid:

Summary of Material Properties			
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

Attachment 1b
Initial Cross-Section

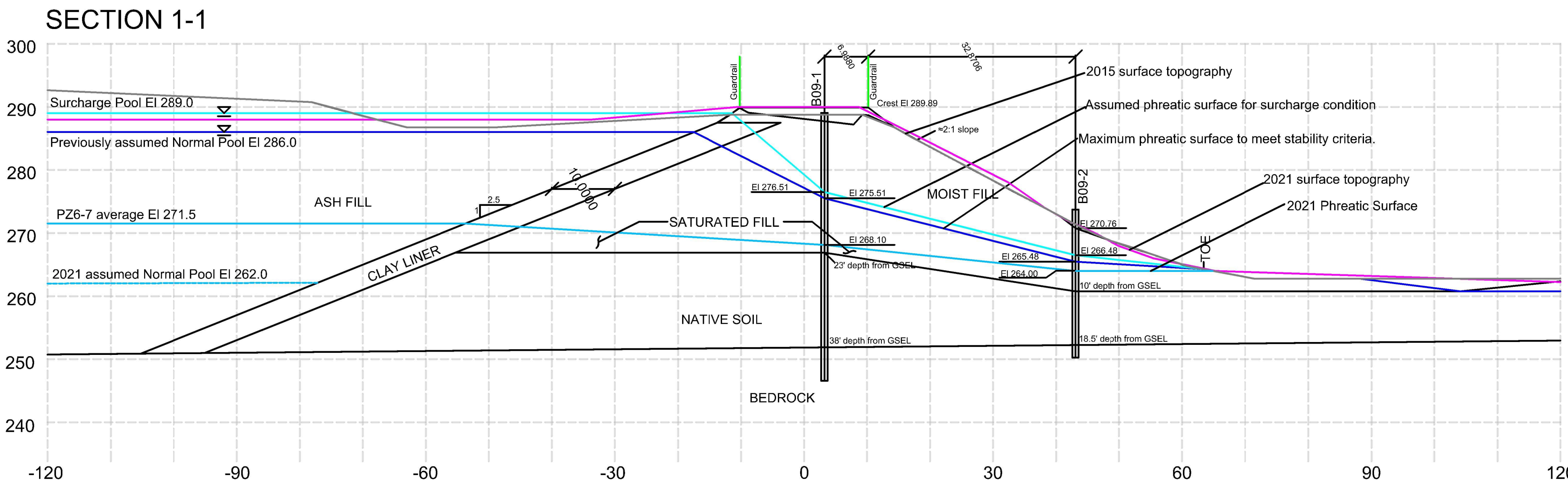


NOTES:
The 2010/2011 phreatic surface was conservatively chosen for the 2015 analysis. This is also the maximum phreatic surface through which stability of the embankment will satisfy EPA stability criteria.

Piezometer	2009 Level	2010/2011 Level	2015 Level	2021 Level
B09-1*	277.3	275.5	271.3	268.1
B09-2	267.5	265.5	263.2	263.6

* B09-1 abandoned on 12/13/2019 due to blockage. Replaced with B09-1N on 12/13/2019.

Attachment 2a
Updated Cross-Section



NOTES:

- 2015 surface topography from aerial survey (NAVD88). Adjusted up 0.76 feet to convert to Plant datum (NGVD29).
- 2021 surface topography from aerial survey (NGVD 29) flown by Borton Lawson April 28, 2021. Aerial survey provided to HDR by Talen.
- Surface topography from 2021 aerial survey (NGVD29) overlaid on 2015 surface topography and Section 1-1 used in 2015 stability analsysis. Topography shows only minor changes. Crest at El. 290.0 feet.
- Stratigraphy unchanged from 2009 and 2015 analyses. Developed from 2009 boring logs and Dwg E158595-4.

Pool level and piezometric levels:

- Normal WSEL previously assumed to be El. 286.0 feet in prior analyses.
- 2021 Normal WSEL assumed to be El. 262.0 feet, 24 feet lower than prior analyses, based on readings from March 11, 2020 through April 15, 2021 which range from El. 260.9 to 262.7. Average piezometric El. 261.8.
- Piezometer PZ6-7 is the nearest piezometer within Ash Basin 6 to Section 1-1, located about 400 feet southwest. PZ6-7 average piezometric level 271.5 based on October 22, 2020 through May 11, 2021 readings.
- Surcharge WSEL assumed to be El 289.0 feet consistent with prior analyses.
- See Section 1-1: Historic Piezometric Levels for history of pool and piezometric levels.

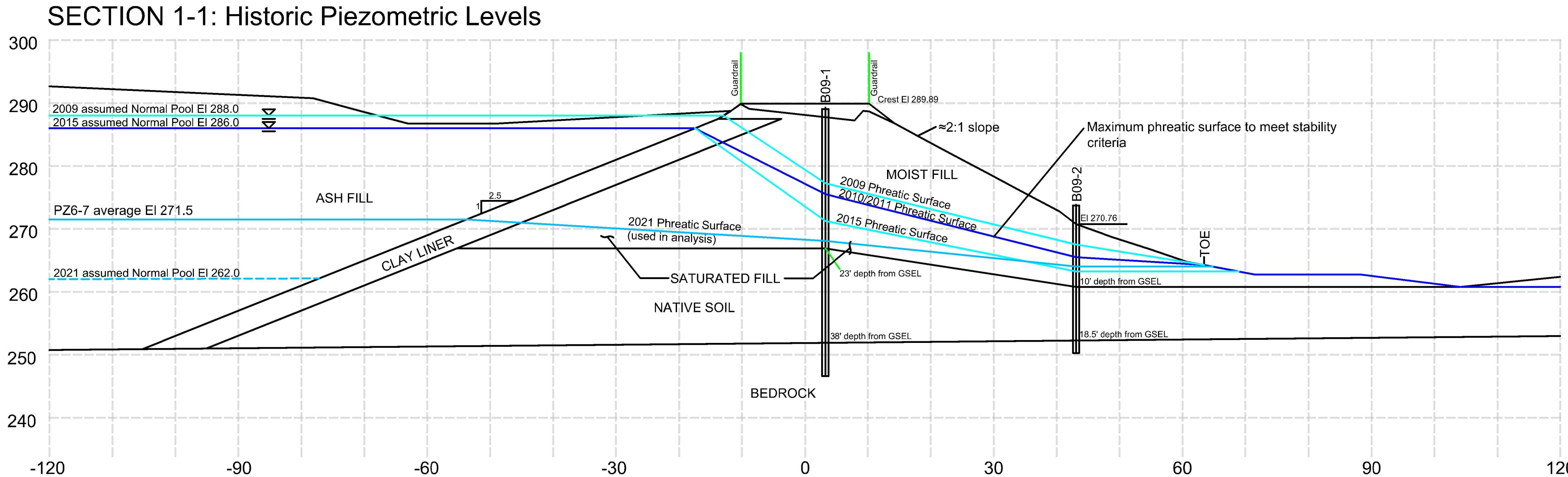
No new boring data or other changes are known to have occurred at Ash Basin 6 that would result in the need to adjust stratigraphy and material properties. Therefore material properties and stratigraphy used in 2009 and 2015 analyses remain valid:

Summary of Material Properties			
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

LEGEND:

- 2021 surface topography
- Assumed Surcharge Pool Phreatic Surface
- Max. Phreatic Surface to Meet Stability Criteria
- 2015 surface topography
- Analysis Section Stratigraphy

Attachment 2b
Updated Cross-Section



NOTES:
- The 2010/2011 phreatic surface is the maximum phreatic surface through which stability of the embankment will satisfy EPA stability criteria.
- Phreatic surface assumed for 2021 analysis shown above.
- B09-1 has a decreasing trend therefore piezometric level based on April 15, 2021 reading.
- B09-2 has a relatively static piezometric level. 2021 level based on average of readings from January 14, 2020 through April 15, 2021 which ranged from El. 262.4 to El. 264.7.
- See notes on Section 1-1 for discussion on Piezometer PZ6-7.

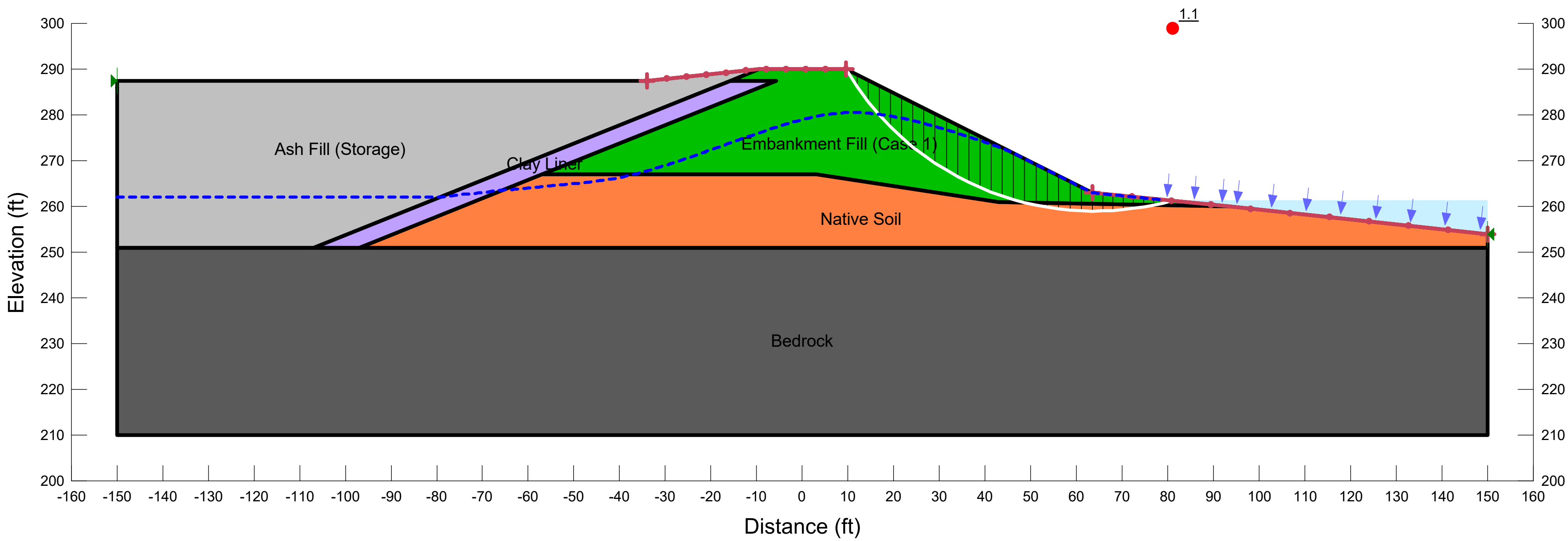
Piezometer	2009 Level	2010/2011 Level	2015 Level	2021 Level
B09-1*	277.3	275.5	271.3	268.1
B09-2	267.5	265.5	263.2	263.6

* B09-1 abandoned on 12/13/2019 due to blockage. Replaced with B09-1N on 12/13/2019.

Attachment 3: RDD from EL 289.5 to River at Normal Water Level Elevation
with Steady-State Normal Storage Pool
(Case 1: $K_v = K_h = 6.8*10^{-6}$ ft/sec)

Brunner Island Ash Basin No. 6
Case 1: Normal Pool, Slope Stability
Station 21+80 (Updated Section 1-1)
Manchester Township, Pennsylvania
August 2021

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Constant Unit Wt. Above Water Table (pcf)
	Ash Fill (Storage)	Mohr-Coulomb	90	0	30	
	Bedrock	Mohr-Coulomb	160	2,000	45	
	Clay Liner	Mohr-Coulomb	130	0	30	
	Embankment Fill (Case 1)	Mohr-Coulomb	135	0	37	125
	Native Soil	Mohr-Coulomb	130	0	30	



Attachment 4: RDD from EL 289.5 to River at Normal Water Level Elevation
with 1,000-day Transient Surcharge Storage Pool
(Case 1: $K_v = K_h = 6.8*10^{-6}$ ft/sec)

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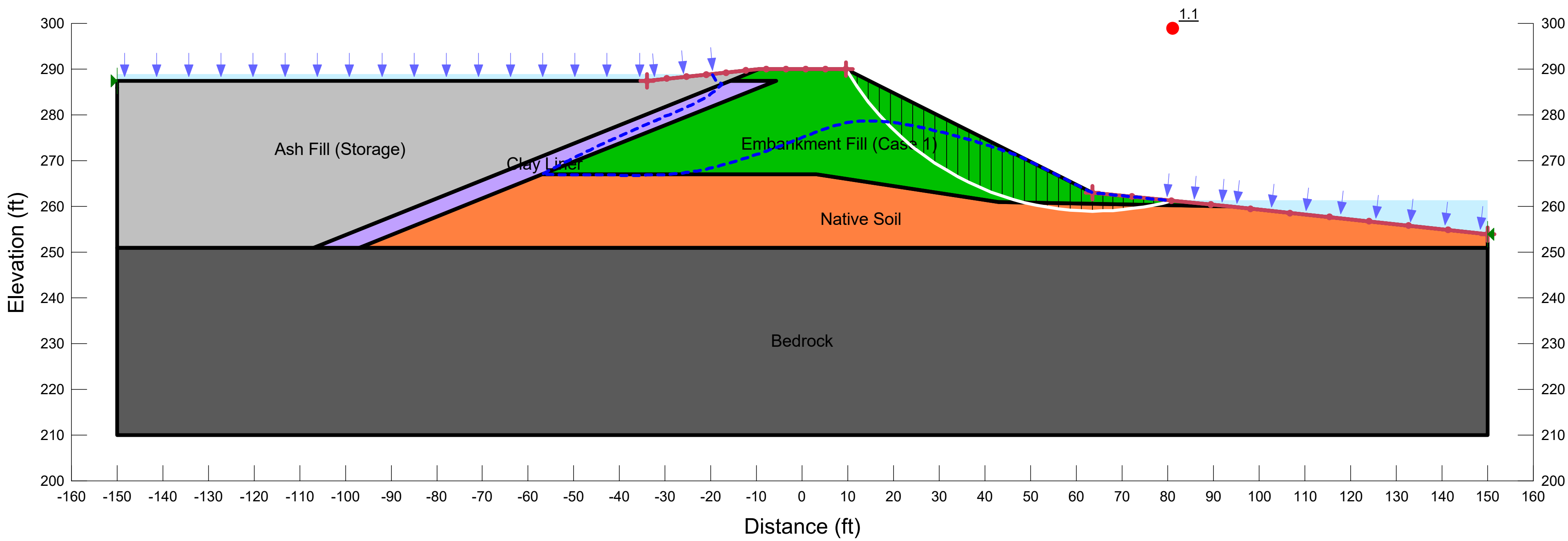
Brunner Island Ash Basin No. 6
Case 1: Storage, Slope Stability 1000day RDD
Station 21+80 (Updated Section 1-1)
Manchester Township, Pennsylvania
August 2021

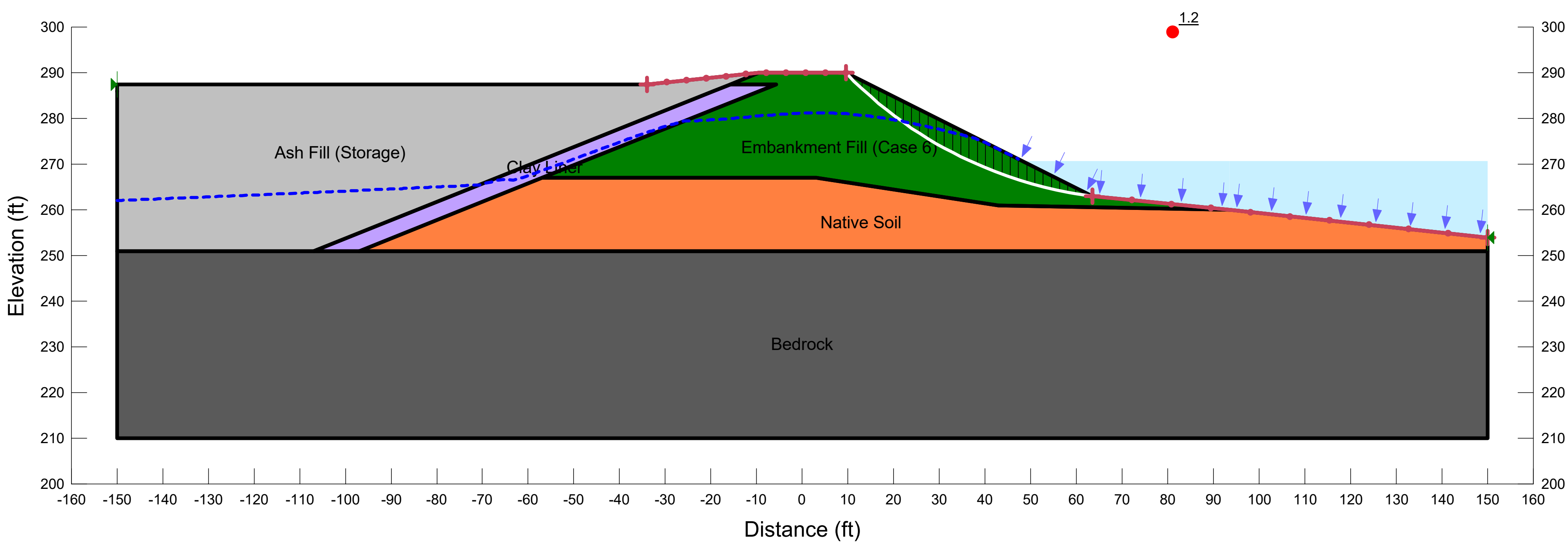
Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Constant Unit Wt. Above Water Table (pcf)
	Ash Fill (Storage)	Mohr-Coulomb	90	0	30	
	Bedrock	Mohr-Coulomb	160	2,000	45	
	Clay Liner	Mohr-Coulomb	130	0	30	
	Embankment Fill (Case 1)	Mohr-Coulomb	135	0	37	125
	Native Soil	Mohr-Coulomb	130	0	30	



Attachment 5: RDD from EL 289.5 to River at Normal Water Level Elevation
with Steady-State Normal Storage Pool
(Case 6: $K_v = 0.13*K_h = 2.8*10^{-6}$ ft/sec)

Brunner Island Ash Basin No. 6
Case 6: Normal Pool, Slope Stability
Station 21+80 (Updated Section 1-1)
Manchester Township, Pennsylvania
August 2021

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Constant Unit Wt. Above Water Table (pcf)
	Ash Fill (Storage)	Mohr-Coulomb	90	0	30	
	Bedrock	Mohr-Coulomb	160	2,000	45	
	Clay Liner	Mohr-Coulomb	130	0	30	
	Embankment Fill (Case 6)	Mohr-Coulomb	135	0	37	125
	Native Soil	Mohr-Coulomb	130	0	30	

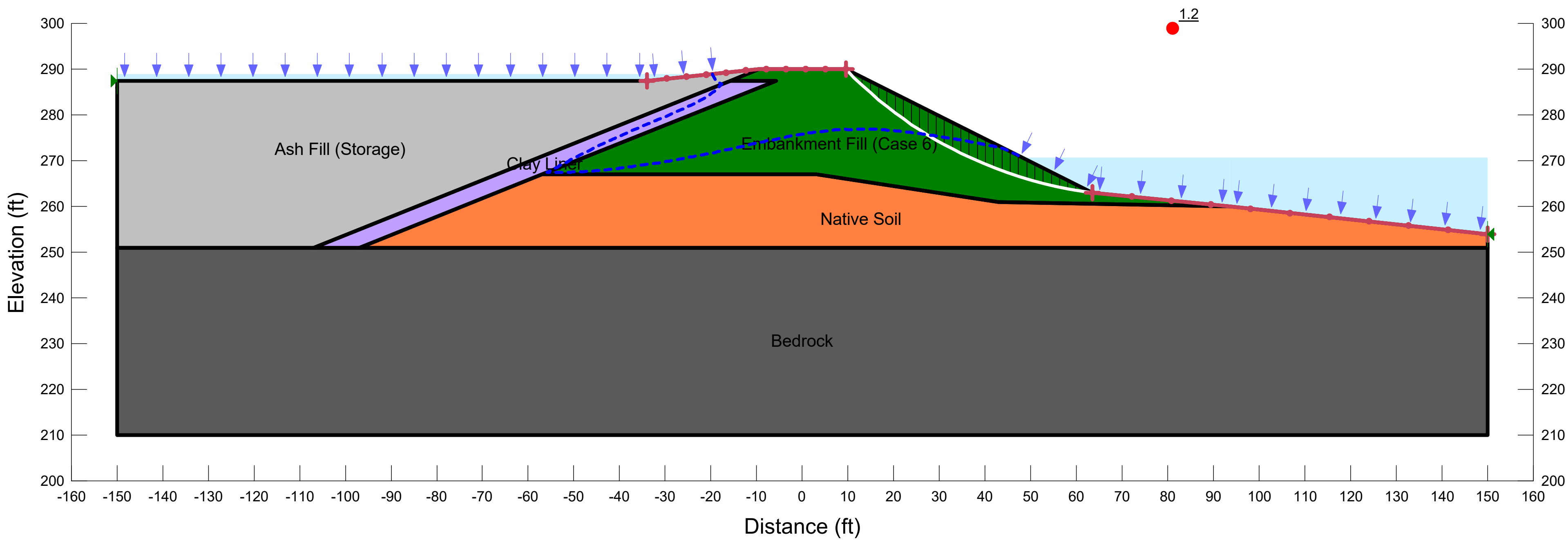


Attachment 6: RDD from EL 289.5 to River at Normal Water Level Elevation
with 1,000-day Transient Storage Surcharge Pool
(Case 6: $K_v = 0.13*K_h = 2.8*10^{-6}$ ft/sec)

Schnabel ENGINEERING

Brunner Island Ash Basin No. 6
Case 6: Storage, Slope Stability 1000day RDD
Station 21+80 (Updated Section 1-1)
Manchester Township, Pennsylvania
August 2021

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Constant Unit Wt. Above Water Table (pcf)
	Ash Fill (Storage)	Mohr-Coulomb	90	0	30	
	Bedrock	Mohr-Coulomb	160	2,000	45	
	Clay Liner	Mohr-Coulomb	130	0	30	
	Embankment Fill (Case 6)	Mohr-Coulomb	135	0	37	125
	Native Soil	Mohr-Coulomb	130	0	30	



GEOTECHNICAL ENGINEERING REPORT

Brunner Island SES Transient Seepage and Slope Stability Study
East Manchester Township, York County Pennsylvania

Schnabel Reference 15615015
December 17, 2015

Schnabel
ENGINEERING

December 17, 2015

Mr. Ben Wilburn, PE
Talen Generation LLC
835 Hamilton Street
Allentown, PA 18101

Subject: **Project 15615015, Brunner Island SES Transient Seepage and Slope Stability Study, Wago Road, East Manchester Township, York County, Pennsylvania**

Dear Mr. Wilburn:

SCHNABEL ENGINEERING CONSULTANTS, INC. (Schnabel) is pleased to submit our geotechnical engineering report for this project. This report includes tables, figures, and attachments with relevant data pertinent to this study. This study was performed in accordance with our revised proposal dated May 22, 2015, as authorized by Talen Generation LLC (Talen), Contract No. 628213–C, dated June 2, 2015.

We appreciate the opportunity to be of service for this project. Please call us if you have any questions regarding this report.

EXECUTIVE SUMMARY

We are providing this executive summary solely for purposes of overview. Any party that relies on this report must read the full report. This executive summary omits several details, any one of which could be very important to the proper application of the report.

This study re-evaluated the stability of the eastern-most impoundment dike at the Brunner Island Ash Basin No. 6 facility, which is adjacent to the Susquehanna River. The original study was performed by Schnabel, as summarized in our February 17, 2012, report to Pennsylvania Power and Light (PPL) Generation, LLC (Schnabel, 2012). In Schnabel's 2012 study, a transient seepage analysis was performed to consider slope stability under a rapid drawdown (RDD) event from a 500-yr recurrence interval (RI) flood corresponding to a river elevation at EL 288.8. The current study includes re-evaluation of the RDD event from a level slightly greater than a 1000-yr event corresponding to a river elevation at EL 289.5. The present study suggests a minimum factor of safety (FOS) under RDD to still be greater than 1.1 for the revised scenarios and conditions that were considered. As in the previous study, the most critical representative section (Section 1-1 at Station 21+80) was chosen based on observed piezometric levels.

SCOPE

Our agreement dated June 2, 2015, defines the scope of this study. We previously completed a transient seepage and slope stability analysis of one of the Brunner Island Ash Basin (AB) No. 6 impoundment dikes (Schnabel, 2012). The results of our previous analysis in 2012 focused on the stability of the eastern-most downstream (e.g., river side) slope of the embankment under rapid drawdown of the Susquehanna River from the 500-yr recurrence interval (RI) flood stage elevation. The duration of the various stages was based on our interpretation and evaluation of readily available historical data prepared by others.

Recent changes in Coal Combustion Residuals (CCR) regulations released by the United States Environmental Protection Agency (USEPA) require re-evaluation of the stability of the embankment slopes of Brunner Island Ash Basin No. 6 (HDR, 2015a). Based on these new regulations, Talen requested that Schnabel update the previous analyses to consider rapid drawdown of the Susquehanna River from the 1000-yr recurrence interval flood stage elevation. This maximum surcharge corresponds to a river elevation of 289.0 (HDR, 2015b). Schnabel re-evaluated RDD from this maximum surcharge, as well as ½ ft above the corresponding 1000-yr event (i.e., at EL 289.5).

Services not described in our agreement are not included in this study. We would be happy to provide any additional services to the project team that are required.

PROJECT APPROACH

Our analyses were identical to Schnabel's earlier (2012) study, with the exception of a higher surcharge corresponding to a 1000-yr (and slightly greater) loading event from flooding on the Susquehanna River. The basis of our analyses and development of the transient loading condition and parameters adopted are described in detail in the 2012 report.

The previous analyses assumed a normal headwater elevation (i.e., the elevation of groundwater within the basin) at EL 288.0. We understand that operational changes have resulted in a reduction to approximately EL 284.3. However, as a worst-case scenario, and to account for potential (but unlikely) changes in operations, the present analyses maintained the headwater elevation within the basin at EL 288.0. The lower operational level has relatively minor impact to the stability of the downstream embankment under RDD transient conditions, which are controlled primarily by the change in seepage caused by flooding in the river.

TRANSIENT SEEPAGE ANALYSIS AND MODELING

Seepage was modeled using GeoStudio's SEEP/W (ver 7.14) computer program. SEEP/W is a two-dimensional finite element computer program commonly used to model unconfined and confined seepage problems, including groundwater movement and pore water pressure distribution within porous materials, such as soil and rock. SEEP/W can be used to model seepage conditions and evaluate various parameters, including hydraulic head/pore water pressure distribution, hydraulic gradient, volume of flow, and many others. SEEP/W can be used to model both steady state and transient seepage conditions. Steady state conditions include situations in which model parameters (soil properties, boundary conditions, etc.) do not change over time. Transient conditions involve scenarios in which model parameters do change over time.

The initial water table adopted was identical to that defined in the Schnabel (2012) Report for the analysis of Section 1 at Sta. 21+80. The water table extended from a normal water level (NWL) at EL 288.0 on the upstream side of the impoundment dike, through the embankment at levels as measured by the two piezometers, daylighting near the downstream toe of the impoundment dike at EL 263. The transient seepage scenario described in the 2012 Schnabel Report was used in modeling the RDD condition under transient loading, with the exception that the flood event was modeled using a river elevation as high as EL 289.5.

The previous study used the following cases based on the saturated hydraulic conductivity used for the impoundment dike embankment:

Isotropic Hydraulic Conductivity
Case 1: $K_v = K_h = 6.8*10^{-6}$ ft/sec (maximum saturated hydraulic conductivity, isotropic)
Case 2: $K_v = K_h = 2.8*10^{-6}$ ft/sec (average saturated hydraulic conductivity, isotropic)
Case 3: $K_v = K_h = 6.8*10^{-9}$ ft/sec (minimum saturated hydraulic conductivity, isotropic)

Anisotropic Hydraulic Conductivity
Case 4: $K_v = 0.50 * K_h = 2.8*10^{-6}$ ft/sec (average saturated hydraulic conductivity, anisotropy ratio = 2)
Case 5: $K_v = 0.25 * K_h = 2.8*10^{-6}$ ft/sec (average saturated hydraulic conductivity, anisotropy ratio = 4)
Case 6: $K_v = 0.13 * K_h = 2.8*10^{-6}$ ft/sec (average saturated hydraulic conductivity, anisotropy ratio = 8)

DEEP-SEATED GLOBAL SLOPE STABILITY ANALYSIS

The downstream side of the impoundment dike was evaluated for global stability using Spencer's Method, as implemented in GeoStudio's SLOPE/W (ver 7.14) computer program. Soil parameters (unit weight, shear strength, etc.) used in the previous Schnabel Report (2012) were adopted for the slope stability analyses. The transient seepage analysis was used to model the change in pore water pressure over time (as described previously), and effective shear strengths were used in the stability model.

Spencer's Method was used to evaluate global slope stability of the downstream slope using the pore water pressure distribution from SEEP/W. The minimum FOS resulting from the RDD from 1000-yr (EL 289.0) and slightly higher (EL 289.5) flood stage to normal water levels in the river was calculated at discrete time increments starting at flood stage, and ending when river levels return to the normal water level elevation. Only deep-seated potential failure planes were considered, which are failure planes that extend from the crest of the embankment beyond the downstream embankment toe.

The results of the previous study showed that Case 1 and Case 6 were the most critical, in terms of providing the lowest Factors of Safety. As such, only these two cases were evaluated for RDD under transient loading from the two-flood stage elevations considered. The Factors of Safety corresponding to the highest flood stage evaluated (EL 289.5), which is greater than the 1000-yr RI flood, are reported in the following table.

Minimum Factor of Safety for RDD from EL 289.5 to Normal River Water Levels: Cases 1 and 6

CONDITION	Min. FOS (Plate #)
Isotropic Hydraulic Conductivity	
Case 1: $K_v = K_h = 6.8*10^{-6}$ ft/sec (max sat hydr cond, isotropic)	1.13 (Attachment 1)
Anisotropic Hydraulic Conductivity	
Case 6: $K_v = 0.13 * K_h = 2.8*10^{-6}$ ft/sec (avg sat hydr cond, anisotropy ratio = 8)	1.12 (Attachment 2)

CONCLUSIONS

Conventional guidelines for minimum factors of safety include recommendations in United States Army Corps of Engineers (USACE) engineering manuals. Recommended minimum values of 1.1 (drawdown from maximum surcharge pool) to 1.3 (drawdown from maximum storage pool) are provided for new earth and rock-fill dams in Table 3-1 in USACE EM 1110-2-1902 (USACE, 2003). Recommended minimum values of 1.0 to 1.2 for new and existing levees, and other embankments and dikes, are provided in USACE EM 1110-2-1913 (USACE, 2000).

The minimum FOS for stability of the downstream embankment slope under the rapid drawdown scenarios presented herein corresponds to a value of 1.12, which is greater than the value of 1.1 for earth dams drawn down from maximum surcharge pool (which most closely represents the scenario used in this study). The study used a flood event corresponding to a river flooding elevation of EL 289.5, approximately 0.5-ft higher than that corresponding to a 1000-yr RI event. Floods with more frequent RIs (e.g., 50-yr, 100-yr, etc.) would result in even higher factors of safety if all other factors remain the same.

REFERENCES

GEO-SLOPE International Ltd. (2008). “Seepage Modeling with SEEP/W 2007: An Engineering Methodology.” 3rd Ed. Calgary, Alberta, Canada.

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SEEP/W Version 2007 by GeoStudio (seepage analysis).

SLOPE/W Version 2007 by GeoStudio (slope stability analysis using Spencer’s Method).

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United States Army Corps of Engineers (USACE). (2003). “Engineering Manual (EM) 1110-2-1902: Slope Stability.” Washington, DC.

LIMITATIONS

We based the analyses and recommendations submitted in this report on the information revealed by the exploration performed by others, and interpretation of data prepared by others. We attempted to provide for normal contingencies, but the possibility remains that unexpected conditions may exist.

We prepared this report to aid in the evaluation of this site and to assist in the geotechnical evaluation described herein. We intend it for use concerning this specific project. We based our recommendations on information on the site and understanding of information as described in this report.

We have endeavored to complete the services identified herein in a manner consistent with that level of care and skill ordinarily exercised by members of the profession currently practicing in the same locality and under similar conditions as this project. No other representation, express or implied, is included or intended, and no warranty or guarantee is included or intended in this report, or any other instrument of service.

We appreciate the opportunity to be of service for this project. Please call us if you have any questions regarding this report.

Sincerely,

SCHNABEL ENGINEERING CONSULTANTS, INC.

Scott A. Raschke, PhD, PE
Senior Associate

SAR:clp

Attachments:

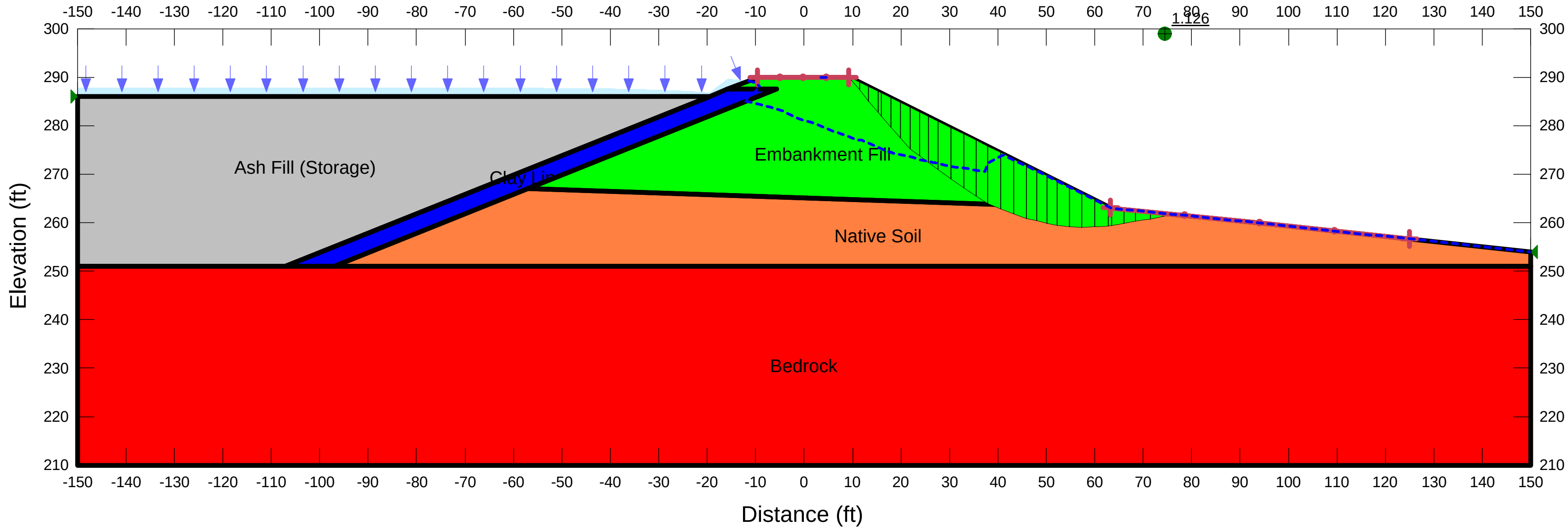
(1) RDD from EL 289.5 to River at Normal Water Level Elevation (Case 1: Kv=Kh=6.8*10^-6 ft/sec)

(2) RDD from EL 289.5 to River at Normal Water Level Elevation (Case 6: Kv=0.13*Kh=2.8*10^-6 ft/sec)

Distribution:

Talen Generation LLC (2)

Attn: Mr. Ben Wilburn, PE

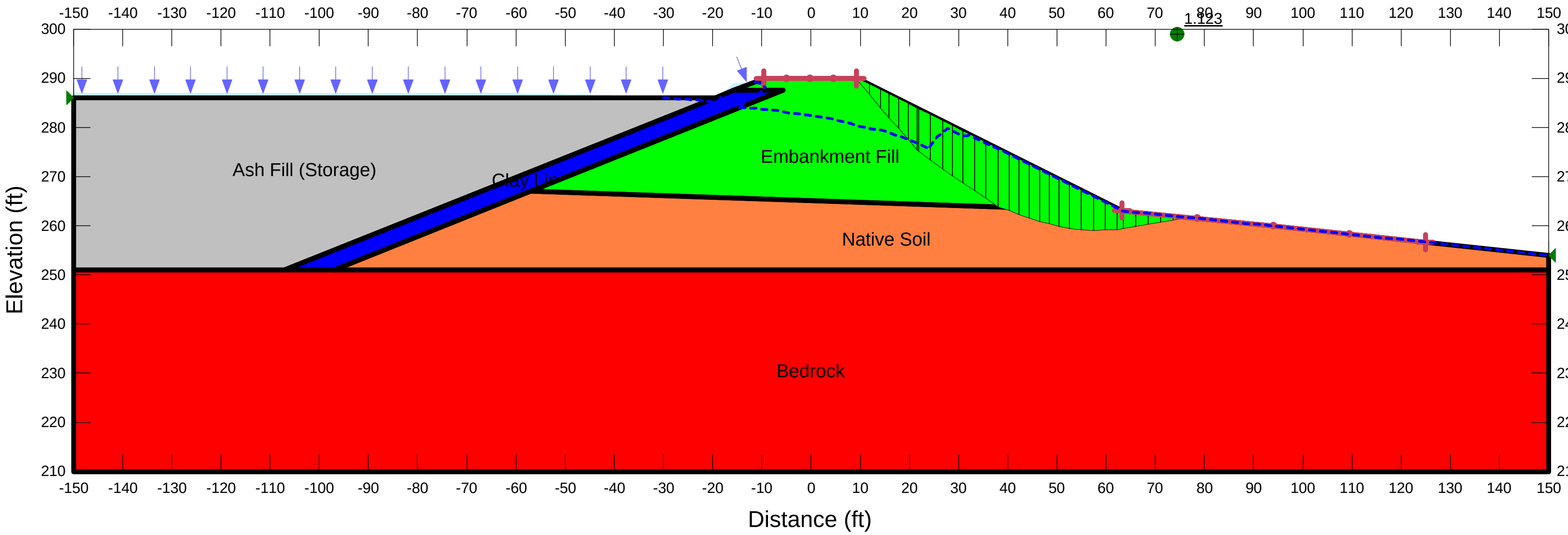


Material Input Properties
Name: Bedrock Model: Mohr-Coulomb Unit Weight: 160 pcf Cohesion: 2000 psf Phi: 45 °
Name: Native Soil Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion: 0 psf Phi: 30 °
Name: Clay Liner Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion: 0 psf Phi: 30 °
Name: Ash Fill (Storage) Model: Mohr-Coulomb Unit Weight: 90 pcf Cohesion: 0 psf Phi: 30 °
Name: Embankment Fill Model: Mohr-Coulomb Unit Weight: 135 pcf Unit Wt. Above Water Table: 125 pcf Cohesion: 0 psf Phi: 37 °

Attachment 1 - RDD from EL 289.5 to River at Normal Water Level Elevation
(Case 1: Kv=Kh=6.8*10^-6 ft/sec)

Slope Stability (6)

Brunner Island Ash Basin No. 6
Station 21+80 (Section 1-1)
Manchester Township, Pennsylvania



Material Input Properties
Name: Bedrock   Model: Mohr-Coulomb   Unit Weight: 160 pcf   Cohesion: 2000 psf   Phi: 45 °
Name: Native Soil   Model: Mohr-Coulomb   Unit Weight: 130 pcf   Cohesion: 0 psf   Phi: 30 °
Name: Clay Liner   Model: Mohr-Coulomb   Unit Weight: 130 pcf   Cohesion: 0 psf   Phi: 30 °
Name: Ash Fill (Storage)   Model: Mohr-Coulomb   Unit Weight: 90 pcf   Cohesion: 0 psf   Phi: 30 °
Name: Embankment Fill   Model: Mohr-Coulomb   Unit Weight: 135 pcf   Unit Wt. Above Water Table: 125 pcf   Cohesion: 0 psf   Phi: 37 °

Attachment 2 - RDD from EL 289.5 to River at Normal Water Level Elevation
(Case 6: Kv=0.13*Kh=2.8*10^-6 ft/sec)

Slope Stability (6)

Brunner Island Ash Basin No. 6
Station 21+80 (Section 1-1)
Manchester Township, Pennsylvania

GEOTECHNICAL ENGINEERING REPORT

PPL Brunner Island SES Transient Seepage and Slope Stability Study
PPL Contract No. 523528-C
East Manchester Township, York County
Pennsylvania

Schnabel Reference 11615019
February 17, 2012

Schnabel
ENGINEERING

February 17, 2012

Mr. James P. Lynch, CCM
PPL Civil TIP Coordinator
GENPL6
2 North 9th Street
Allentown, PA 18101

Subject: **Project 11615019, PPL Brunner Island SES Transient Seepage and Slope Stability Study, Wago Road, East Manchester Township, York County, Pennsylvania**

Dear Mr. Lynch:

SCHNABEL ENGINEERING CONSULTANTS, INC. (Schnabel) is pleased to submit our geotechnical engineering report for this project. This report includes tables, figures, and appendices with relevant data collected for this study. This study was performed in accordance with our proposal dated May 16, 2011, with addendum dated August 29, 2011, as authorized by Mr. Larry Ehrenreich originally on June 2, 2011, and as amended on September 12, 2011.

We appreciate the opportunity to be of service for this project. Please call us if you have any questions regarding this report.

EXECUTIVE SUMMARY

We are providing this executive summary solely for purposes of overview. Any party that relies on this report must read the full report. This executive summary omits several details, any one of which could be very important to the proper application of the report.

This study evaluated the stability of the eastern-most impoundment dike at the Brunner Island Ash Basin No. 6 facility, which is adjacent to the Susquehanna River. A transient seepage analysis was performed to consider slope stability under a rapid drawdown event from a 500-yr recurrence interval (RI) flood corresponding to a river elevation at EL 288.8. The models developed for this evaluation included data from explorations and analyses prepared by others as described herein.

The study suggests a minimum factor of safety (FOS) under rapid drawdown greater than 1.1 for the scenarios and conditions that were considered.

schnabel-eng.com

T/ 610-696-6066 F/ 610-696-7771
1380 Wilmington Pike, Suite 100 / West Chester, PA / 19382

SCOPE

Our agreement dated May 16, 2011, as amended by our addendum dated August 29, 2011, defines the scope of this study. Our services included retention of a subconsultant (Advantage Engineers [Advantage]) to perform subsurface exploration, field testing and evaluation, and soil laboratory testing which are included in a Geotechnical Data Summary Report (DSR).

Based on the Geotechnical DSR prepared by Advantage and data provided to us which were developed by others, we completed a transient seepage and slope stability analysis of one of the Brunner Island Ash Basin (AB) No. 6 impoundment dikes. Our analysis focused on the stability of the eastern-most downstream (e.g., river side) slope of the embankment under rapid drawdown of the Susquehanna River from the 500-yr recurrence interval (RI) flood stage elevation. The duration of the various stages described herein is based on our interpretation and evaluation of readily available historical data prepared by others. Our evaluation of the eastern impoundment dike along the Susquehanna River was requested since results of steady state seepage and slope stability analysis performed by another consultant (HDR Engineering, Inc., 2009) indicated that the minimum Factor of Safety (FOS) for slope stability of the downstream slope under a rapid drawdown condition (under steady state seepage) may be unsatisfactory for the eastern impoundment dike.

Pennsylvania Power and Light (PPL) provided a copy of the HDR Engineering Report (2009) to Schnabel, as well as a copy of a report prepared by Borings, Soils & Testing Company (BST, 1977) which was prepared to evaluate foundation conditions for Ash Storage Basins 6 and 7 at the Brunner Island facility. For the project described herein, Schnabel prepared a transient seepage and slope stability analysis for the downstream slope of the eastern Brunner Island impoundment dike at AB No. 6 under a rapid drawdown condition. This report presents our approach and the results of our evaluation.

Services not described in our agreement are not included in this study. We would be happy to provide any additional services to the project team that are required.

PROJECT APPROACH

The HDR Report (2009) included subsurface exploration, piezometer installation, and testing and evaluation at two cross section locations on the eastern impoundment dike (Section 1 at Sta. 21+80 and Section 2 at Sta. 7+44). The geometry and subsurface soil conditions were nearly identical at the two cross section locations; however, water levels observed in the Section 1 piezometers were found to be higher than at Section 2. The higher phreatic surface at Section 1 would make that section more critical for slope stability, so the geometric configuration and piezometric levels based on Section 1 were adopted for this study.

We performed preliminary transient seepage and slope stability analyses based on the parameters adopted in the HDR Report (2009), including embankment geometry, subsurface conditions and stratification, phreatic surface, shear strength (friction angle and cohesion), and unit weights. The HDR Report (2009) did not include testing and evaluation of the embankment soil hydraulic conductivity since analyses were made based upon steady state seepage conditions.

Preliminary transient analyses used a range of reasonable parameters to perform a sensitivity analysis of the transient seepage condition, including the saturated hydraulic conductivity of the embankment soils. The range of values adopted for parameters used in the sensitivity analysis was based upon embankment soil gradation from laboratory testing and visual descriptions in test borings, all performed by others, including values reported in the BST Report (1977).

Our preliminary sensitivity evaluation showed that the penetration of the wetting front during transient seepage caused by rising flood levels in the Susquehanna River, and the subsequent dissipation of pore pressures as the flood levels recede, was mostly dependent on the saturated hydraulic conductivity of the embankment soils. The factor of safety for deep-seated slope failures of the embankment under transient seepage conditions could range from acceptable to unacceptable based on the pore water pressure distribution resulting from various transient models which incorporated reasonable values of hydraulic conductivity. The factors of safety were typically lower as the saturated hydraulic conductivity increased, due to the deeper penetration of the wetting front moving through the embankment during transient seepage. Therefore, it was decided that further characterization of the embankment soils was necessary to complete the dike stability evaluation under rapid drawdown using transient seepage analysis.

SUPPLEMENTAL FIELD EXPLORATION AND LABORATORY TESTING – GEOTECHNICAL DATA SUMMARY REPORT (DSR)

Schnabel retained Advantage Engineers to perform a supplemental field exploration and laboratory testing program, and to summarize the results into a Geotechnical DSR. The subsurface exploration program included the following:

- Five Standard Penetration Test (SPT) Borings located along the crest of the existing embankment extending to a depth of approximately 20 ft (designated TB-C1 through TB-C5).

- Four hand-excavated test pits located mid-way between the riverside embankment toe and crest (designated HA-E1 through HA-E4).

Exploration locations are shown on Figure 1 of the Advantage Report (2012) that is included as Appendix A. Within each of the hand-excavated test pits, in-place soil density and moisture content were measured according to ASTM D1556 (sand cone). The infiltration rate was measured within the test pits using a double ring infiltrometer. Infiltration rates were also measured at depths selected by Schnabel in cased holes advanced as auger probes adjacent to the SPT boring locations. These infiltration tests were performed by Advantage personnel in general accordance with Appendix C of the Pennsylvania Department of Environmental Protection (PADEP) Pennsylvania Stormwater Best Management Practices Manual (PADEP, 2006). Results of the field testing are summarized in Tables I and II of the Advantage Report (2012). In addition to the SPT samples, bulk samples were also collected from auger cuttings over each 5-ft depth interval (e.g., 0-5 ft, 5-10 ft, 10-15 ft, and 15-20 ft) and from the hand-excavated test pits.

Draft test boring logs provided to Schnabel by Advantage were used to select samples to perform initial laboratory testing to further characterize the soils. Samples were selected to evaluate the various types of embankment soils encountered in the field exploration. Embankment soils (based on visual classifications) were generally either: (1) lean clay or silt with varying amounts of sand and gravel; (2)

sand with varying amounts of silt/clay and gravel; or (3) gravel with varying amounts of silt/clay and sand. The initial laboratory testing included the following:

- 50 natural moisture content determinations (ASTM D2216)
- 14 sieve and hydrometer tests (ASTM D422)
- 7 Atterberg (plastic and liquid) Limit determinations (ASTM D4318)

Standard Proctor Tests (ASTM D698) were performed to evaluate the maximum dry unit weight and optimum moisture content of representative samples of the three fundamental embankment soil types. Based on in situ density tests, the average relative compaction (RC) of the embankment soils was approximately 85 percent. While in situ density tests were only performed in the shallow hand-excavated test pits, SPT blowcounts suggest a lower bound average relative compaction of 85 percent for the deeper embankment soils is reasonable as well.

Seven bulk samples were selected for hydraulic conductivity testing (ASTM D5084) to represent the various embankment soil types. Specimens were prepared from the bulk samples, which included samples from the hand auger locations and test borings. Specimens from the hand auger locations were remolded at the approximate in situ moisture content and dry density (as determined from the field testing). Soil samples from test borings were remolded at optimum moisture content and a dry unit weight corresponding to an RC of 85 percent (based on Proctor tests most suitable for each particular soil sample). The complete results of the laboratory testing are included with the Advantage Report (2012) that is provided in Appendix A.

Saturated hydraulic conductivity values for representative embankment soils were evaluated from the seven flexible wall permeameter (ASTM D5084) tests. Saturated hydraulic conductivities were also estimated from the measured infiltration rates using the empirical relationship described by Fritton et al. (1986) which were developed based on tests in Pennsylvania soils. The saturated hydraulic data are summarized in tabular format in Appendix B.

Tables 1 and 2 included in Appendix B summarize the saturated hydraulic conductivity data from the in-situ infiltration testing and laboratory, respectively. Figure 1 in Appendix B is a box plot showing the statistical distribution in the saturated hydraulic conductivity data. Maximum, minimum, average, and lower and upper quartile values of the saturated hydraulic conductivity are shown.

TRANSIENT SEEPAGE ANALYSIS AND MODELING

Seepage was modeled using GeoStudio's SEEP/W (ver 7.14) computer program. SEEP/W is a two-dimensional finite element computer program commonly used to model unconfined and confined seepage problems, including groundwater movement and pore water pressure distribution within porous materials such as soil and rock. SEEP/W can be used to model seepage conditions and evaluate various parameters, including hydraulic head/pore water pressure distribution, hydraulic gradient, volume of flow, and many others. SEEP/W can be used to model both steady state and transient seepage conditions. Steady state conditions include situations in which model parameters (soil properties, boundary conditions, etc.) do not change over time. Transient conditions involve scenarios in which model parameters do change over time.

To model both steady state and transient seepage in SEEP/W, the saturated hydraulic conductivity is required for the underlying soils. Both natural soil deposits and man-made soil structures (e.g., dikes, levees, earthen embankments, etc.) may exhibit anisotropy, which means that the resistance to flow is different in different directions. This means that different values of hydraulic conductivity are required to model flow in different directions (e.g., different values of K_h and K_v for saturated hydraulic conductivity in the horizontal and vertical directions, respectively). Anisotropic hydraulic conductivity can be (and was) modeled in SEEP/W. The saturated hydraulic conductivity values evaluated from the field and laboratory testing are mostly controlled by the vertical hydraulic conductivity of the embankment soils. Finally, boundary conditions associated with the phreatic surface (i.e., the water table) defined in the seepage model must be established.

Transient (non-steady state) seepage modeled in SEEP/W requires definition of additional soil parameters to model unsaturated flow and appropriate boundary conditions applied to the ground surface profile. The boundary conditions can be changed over time to produce realistic stages of varying infiltration and water elevations to various surfaces.

The unsaturated hydraulic conductivity of a soil is both nonlinear and hysteretic. SEEP/W can model the nonlinear relationship between hydraulic conductivity and matric potential/volumetric water content, but cannot model hysteresis. Hysteresis is the phenomena associated with unsaturated flow, whereby the unsaturated hydraulic conductivity is not only a function of the matric potential/volumetric water content, but whether the soil is going through a drying or wetting phase.

Modeling a soils' unsaturated hydraulic conductivity in SEEP/W requires definition of two relationships:

1. The volumetric water content / matric potential curve (VWC-MPC), which defines the non-linear relationship between the volumetric water content and matric potential.

2. The unsaturated hydraulic conductivity / pore water pressure (matric potential) UP-PWP curve, which defines the non-linear relationship between unsaturated hydraulic conductivity and matric potential.

In addition, the coefficient of volume compressibility (m_v) must also be defined.

SEEP/W has several semi-empirical models that can be used to develop the VWC-MPC curves for soils, which depend on the soil type (fine versus coarse grained) and material properties (e.g., plasticity of fine grained soils, grain-size distribution of coarse grained soils, etc.). The pertinent soil properties for the strata (including m_v) were taken from an evaluation of the laboratory test data. The UP-PWP was modeled using the relationship developed by Fredlend and Xing, which depends on the saturated hydraulic conductivity, residual water content, range of matric potential, and VWC-MPC relationship. Details of this model can be found in the SEEP/W User's Manual (GEO-SLOPE International Ltd, 2008) and references included therein. It should be noted that a sensitivity analysis was performed prior to finalizing the transient analysis. The sensitivity analysis revealed that the results from the models were relatively insensitive to residual water content and coefficient of volume compressibility, and showed that the saturated hydraulic conductivity of the dike embankment was the primary factor affecting the stability of the embankment using the pore pressure distribution from a transient seepage analysis under the rapid drawdown condition. This was primarily due to deeper penetration of the wetting front at higher hydraulic conductivities, which did not dissipate over the rapid drawdown time period.

The initial water table must also be defined to perform a transient seepage analysis. The initial water table adopted was identical to that defined in the HDR Report (2009) for the analysis of Section 1 at Sta. 21+80. The water table extended from a normal water level (NWL) at EL 288.0 on the upstream side of the impoundment dike, through the embankment at levels as measured by the two piezometers, daylighting near the downstream toe of the impoundment dike at EL 263.

Once the initial water table and material properties for transient flow were defined for the unsaturated embankment soil in the analysis section, appropriate boundary conditions were assigned. The boundary conditions were established assuming the following staged "rainy day" scenario, which is based on available historical climatic, meteorological, and hydraulic data (including the rise, high stage, and recede time intervals for the storm of record, which is Hurricane Agnes that occurred in June 1972). A summary of the available climatic, meteorological, and hydraulic data that was reviewed for this project is included in Appendix C.

1. <u>DAY 0 to 353:</u> A surface boundary flux was applied representing annual infiltration at a rate twice as great as the average daily precipitation for the project area for a period of 353 days. Based on data from the United States National Oceanic and Atmospheric Administration (NOAA), the average daily precipitation near the project area is about 0.11 inches per day.

2. <u>DAY 353 to 357:</u> The rate of infiltration for the surface boundary flux was increased to correspond to a total of 9 inches of precipitation over a 24 hour period. Based on NOAA data, this corresponds to the 24-hour rainfall from a storm with an RI of 200 years, with a 90% confidence interval (i.e., 95% assurance that the value is less than 9 inches). This flux was applied for a total period of four days, corresponding to a total of 36 inches of rainfall.

3. <u>DAY 357 to 359:</u> The surface boundary flux was reduced back to a value equal to twice the average infiltration rate; however, the river level was raised from a normal water level elevation (considered as the top of bank elevation at EL 252) to the flood elevation corresponding to the 500-year RI flood event (EL 288.8). The river level was increased linearly to the peak elevation over a period of two days.

4. <u>DAY 359 to 363:</u> The river elevation was held at the 500-yr flood elevation for a period of four days.

5. <u>DAY 363 to 365:</u> The river elevation was allowed to recede (fall) to the initial normal water level elevation over a period of two days. This is the time period for rapid drawdown, and the pore water pressure distribution at the end of two days was used for the slope stability analysis under rapid drawdown.

The transient seepage scenario described above was modeled using the following cases based on the saturated hydraulic conductivity used for the impoundment dike embankment:

<u>Isotropic Hydraulic Conductivity</u>
Case 1: $K_v = K_h = 6.8*10^{-6}$ ft/sec (maximum saturated hydraulic conductivity, isotropic)
Case 2: $K_v = K_h = 2.8*10^{-6}$ ft/sec (average saturated hydraulic conductivity, isotropic)
Case 3: $K_v = K_h = 6.8*10^{-9}$ ft/sec (minimum saturated hydraulic conductivity, isotropic)

<u>Anisotropic Hydraulic Conductivity</u>
Case 4: $K_v = 0.50 * K_h = 2.8*10^{-6}$ ft/sec (average saturated hydraulic conductivity, anisotropy ratio = 2)
Case 5: $K_v = 0.25 * K_h = 2.8*10^{-6}$ ft/sec (average saturated hydraulic conductivity, anisotropy ratio = 4)
Case 6: $K_v = 0.13 * K_h = 2.8*10^{-6}$ ft/sec (average saturated hydraulic conductivity, anisotropy ratio = 8)

Representative plates displaying graphical output from the transient seepage analyses are provided in Appendix D for Cases 1 and 3. As suggested earlier, Plates D2a and D3a in Appendix D illustrate how the lower saturated hydraulic conductivity limits the penetration of the wetting front through the embankment.

DEEP-SEATED GLOBAL SLOPE STABILITY ANALYSIS

The downstream side of the impoundment dike was evaluated for global stability using Spencer's Method as implemented in GeoStudio's SLOPE/W (ver 7.14) computer program. Soil parameters (unit weight, shear strength, etc.) used in the HDR Report (2009) were adopted for the slope stability analyses. The transient seepage analysis was used to model the change in pore water pressure over time (as described previously), and effective shear strengths were used in the stability model.

Spencer's Method was used to evaluate global slope stability of the downstream slope using the pore water pressure distribution from SEEP/W. The minimum FOS resulting from the rapid drawdown (flood recede over two days) from a 500-yr flood stage to normal water levels in the river was calculated at discrete time increments starting at flood stage and ending when river levels return to the normal water level elevation. Only deep-seated potential failure planes were considered, which are failure planes that extend from the crest of the embankment beyond the downstream embankment toe.

Plates displaying graphical output from the global slope stability analyses are provided in Appendix E for Case 2 at selected stages during rapid drawdown (Plates E2a through E2d), and at the completion of drawdown for all cases (Cases 1 through 6 in Plates E3a through E3f, respectively). The following table summarizes the minimum FOS for rapid drawdown that was calculated for Cases 1 through 6.

Minimum Factor of Safety for Rapid Drawdown: Cases 1 through 6

CONDITION	Min. FOS (Plate #)
Isotropic Hydraulic Conductivity	
Case 1: $K_v = K_h = 6.8*10^{-6}$ ft/sec (max sat hydr cond, isotropic)	1.13 (E3a)
Case 2: $K_v = K_h = 2.8*10^{-6}$ ft/sec (avg sat hydr cond, isotropic)	1.22 (E3b)
Case 3: $K_v = K_h = 6.8*10^{-9}$ ft/sec (min sat hydr cond, isotropic)	1.32 (E3c)
Anisotropic Hydraulic Conductivity	
Case 4: $K_v = 0.50 * K_h = 2.8*10^{-6}$ ft/sec (avg sat hydr cond, anisotropy ratio = 2)	1.20 (E3d)
Case 5: $K_v = 0.25 * K_h = 2.8*10^{-6}$ ft/sec (avg sat hydr cond, anisotropy ratio = 4)	1.17 (E3e)
Case 6: $K_v = 0.13 * K_h = 2.8*10^{-6}$ ft/sec (avg sat hydr cond, anisotropy ratio = 8)	1.13 (E3f)

CONCLUSIONS

Conventional guidelines for minimum factors of safety include recommendations in United States Army Corps of Engineers (USACE) engineering manuals. Recommended minimum values of 1.1 (drawdown from maximum surcharge pool) to 1.3 (drawdown from maximum storage pool) are provided for new earth and rock-fill dams in Table 3-1 in USACE EM 1110-2-1902 (USACE 2003). Recommended minimum values of 1.0 to 1.2 for new and existing levees, and other embankments and dikes, are provided in USACE EM 1110-2-1913 (USACE 2000).

The minimum FOS for stability of the downstream embankment slope under the rapid drawdown scenarios presented herein corresponds to a value of 1.13, which is greater than the value of 1.1 for earth dams drawn down from maximum surcharge pool (which most closely represents the scenario used in this study). The study used a flood event with a 500-yr RI, so floods with more frequent RI's (e.g., 50-yr, 100-yr, etc.) would result in even higher factors of safety if all other factors remain the same.

REFERENCES

Advantage Engineers. (2012). "Geotechnical Data Summary Report: PPL Ash Basin Brunner Island Transient Seepage and Embankment Stability Study," Mechanicsburg, Pennsylvania.

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HDR Engineering, Inc. (2009). "Slope Stability Assessment Brunner Island Ash Basin No. 6," Portland, Maine.

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SLOPE/W Version 2007 by GeoStudio (slope stability analysis using Spencer's Method).

United States Army Corps of Engineers (USACE). (2000). "Engineering Manual (EM) 1110-2-1913: Design and Construction of Levees." Washington, DC.

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LIMITATIONS

We based the analyses and recommendations submitted in this report on the information revealed by the exploration performed by others, and interpretation of data prepared by others. We attempted to provide for normal contingencies, but the possibility remains that unexpected conditions may exist.

We prepared this report to aid in the evaluation of this site and to assist in the geotechnical evaluation described herein. We intend it for use concerning this specific project. We based our recommendations on information on the site and understanding of information as described in this report.

We have endeavored to complete the services identified herein in a manner consistent with that level of care and skill ordinarily exercised by members of the profession currently practicing in the same locality and under similar conditions as this project. No other representation, express or implied, is included or intended, and no warranty or guarantee is included or intended in this report, or any other instrument of service.

We appreciate the opportunity to be of service for this project. Please call us if you have any questions regarding this report.

Sincerely,

SCHNABEL ENGINEERING CONSULTANTS, INC.

Scott A. Raschke, PhD, PE
Senior Associate

JMB:SAR:PIW:jlc

Appendix A: Advantage Geotechnical Data Summary Report
Appendix B: Summary of Saturated Hydraulic Conductivity Data
Appendix C: Summary of Climatic, Meteorological, and Hydraulic Conductivity Data
Appendix D: Seepage Analysis Plates
Appendix E: Slope Stability Analysis Plates

Distribution:
PPL Generation, LLC (2)
Attn: Mr. James P. Lynch

APPENDIX A

ADVANTAGE GEOTECHNICAL DATA SUMMARY REPORT (2012)

GEOTECHNICAL DATA SUMMARY REPORT

PPL ASH BASIN
BRUNNER ISLAND TRANSIENT SEEPAGE AND EMBANKMENT STABILITY STUDY

YORK HAVEN, YORK COUNTY, PENNSYLVANIA

PREPARED FOR:

DR. SCOTT A. RASCHKE, P.E.
SCHNABEL ENGINEERING CONSULTANTS, INC.
1380 WILMINGTON PIKE, SUITE 100
WEST CHESTER, PA 19382

PREPARED BY:

DANIEL R. SCHAUBLE, JR
DIRECTOR OF GEOTECHNICAL SERVICES

EDWARD L. BALASAVAGE, P.E.
MANAGING PARTNER

ADVANTAGE PROJECT NO. - 1100517

JANUARY 2012

telecommunications | environmental | geotechnical

910 Century Drive, Mechanicsburg, Pennsylvania 17055
(717) 458-0800 (717) 458-0801(fax)

TABLE OF CONTENTS

APPENDIX

1.0 INTRODUCTION

This report was prepared by Advantage Engineers, LLC (Advantage), on behalf of Schnabel Engineering Consultants, Inc. (Schnabel), of West Chester, Pennsylvania, and contains the results of a subsurface geotechnical engineering study and laboratory testing program conducted at the site of the existing ash basin at the PPL Brunner Island power generation facility in York Haven, Pennsylvania. The purpose of this investigation has been to gather supplemental subsurface data to establish the parameters required for Schnabel to complete the final seepage and stability analysis.

The scope of work for this project included the completion of a subsurface field investigation, laboratory testing program, and preparation of this geotechnical data summary report. This report summarizes the results of the work performed and provides factual geotechnical engineering data for use in Schnabel's engineering analysis.

2.0 SITE AND PROJECT DESCRIPTION

The project site currently consists of the eastern earthen embankment of Ash Basin No. 6 at the existing PPL Brunner Island power generation facility in York Haven, York County, Pennsylvania. The site is bordered to the east by the Susquehanna River, to the south by undeveloped property and the Susquehanna River, to the west by the existing Ash Basin No. 6, and to the north by Ash Basin No. 5. The approximate location of the site in relation to the surrounding area is presented on the attached *Topographic Map*.

3.0 SUBSURFACE INVESTIGATION PROGRAM

In an effort to evaluate subsurface conditions within the existing earthen embankment, a series of standard SPT earth borings and hand-dug test pits were conducted in September and October 2011, in accordance with the following schedule:

- Five (5) test borings along the crest of the existing embankment, each extending to a termination depth of approximately 20 feet below existing site grades.
- Four (4) hand-dug test pits within the embankment, each extending to a depth of approximately 2 to 3 feet below existing site grades.

Supervision and monitoring of the field operation were provided by a representative of Advantage. The test borings and test pits were field surveyed and staked by Schnabel in advance of our field investigation. The approximate locations of the test borings and test pits, designated as TB-C1 through TB-C5 and HA-E1 through HA-E4, respectively, are shown on *Figure 1 – Brunner Island SES Impoundment Dike Supplemental Exploration Locations*, prepared by Schnabel, presented in the Appendix.

The test borings were advanced using a truck-mounted CME-55 drilling rig equipped with hollow-stem augers and an automatic hammer. Split-spoon samples, conducted in accordance with ASTM standard D1586, were taken throughout the entire depth of the borings and the Standard Penetration Test (SPT) values were recorded for each sample obtained. The SPT values, which are a measure of relative density or consistency, are the number of blows required to drive a 2-inch (outer-diameter), split-barrel sampler 2 feet using a 140-pound weight dropped 30 inches. The number of blows required to advance the sampler over the 12-inch interval from 6 to 18 inches is considered the "N" value.

Data pertaining to the subsurface investigation was documented in the field and is presented in detail on the *Test Boring Logs*, presented within the Appendix. The *Test Boring Logs* contain general information about the subsurface program and specific data regarding each test boring, including: sample depths, blow counts per six (6) inches of penetration, and detailed characterizations of the subsurface materials encountered.

Within each of the hand-excavated test pits, the in-place density and moisture content were determined via Sand Cone Method (ASTM D1556). In addition, infiltration testing was conducted at varying depths within auger probes adjacent to the test boring locations using the “cased pipe method” and within the test pit locations via a “double ring infiltrometer”.

4.0 SUMMARY OF IN-SITU FIELD TESTING

A summary of the results of the field moisture-density testing and infiltration analyses are presented below in Tables I and II. Additional details of the testing completed are presented in the Appendix.

TABLE I

SAND CONE TEST RESULTS – ASTM D1556				
Test Location	HA-E1	HA-E2	HA-E3	HA-E4
Moisture Content (%)	12.4	9.6	8.5	5.4
Wet Density (pcf)	123.2	117.6	126.9	132.5
Dry Density (pcf)	109.5	107.3	117.0	125.7

TABLE II

INFILTATION TEST RESULTS – CASED PIPE & DOUBLE-RING METHODS			
Test Location	**Test Depth (ft)**	**Test Method**	**Infiltration Rate (in/hr)**
TB-C1	8.0	CASED PIPE	1.08
TB-C2	5.0	CASED PIPE	0.60
TB-C3	8.0	CASED PIPE	4.68
TB-C4	4.0	CASED PIPE	0.36
TB-C5	4.5	CASED PIPE	NO MEASURABLE RATE
HA-E1	2.0	DOUBLE-RING	0.20
HA-E2	2.0	DOUBLE-RING	0.84
HA-E3	2.3	DOUBLE-RING	0.31
HA-E4	2.5	DOUBLE-RING	0.25

5.0 LABORATORY TESTING

All soils encountered at the site were visually reviewed and classified by Advantage personnel. The client selected samples collected from the field investigation for laboratory analysis. Advantage delivered the samples to GTS Laboratories where they were subjected to the following analyses:

- 50 natural moisture content determinations per ASTM D2216
- 14 sieve & hydrometer analyses per ASTM D422
- 7 Atterberg Limits (Liquid and Plastic Limits) per ASTM D4318
- 3 Standard Proctor analyses per ASTM D698
- 7 hydraulic conductivity/permeability tests per ASTM D5084 flexible wall permeameter

A detailed account of the laboratory testing completed is presented in the Appendix of this report.

6.0 DESCRIPTION OF CONDITIONS ENOUNTERED

6.1 SOIL

The surfaces of the test borings were found to be covered by approximately 4 to 6 inches of crushed stone (gravel road base). Beneath the topsoil, subsurface conditions were found to be generally homogenous throughout the embankment ranging from silty sand and gravel to sandy clay with gravel. In general, the soils encountered consisted of rounded sand and gravel with varying amounts of silt and clay. Based on the laboratory testing completed, the fines content ranges from approximately 11.5% to 66.8% and the soils are of low to moderate plasticity.

6.2 GROUNDWATER

Groundwater was encountered and measured only within test boring TB-C4 at a depth of approximately 11.3 feet below existing site grades at completion of the test boring. Water was not encountered within the remaining test borings or hand-excavated test pits completed at the project site. These observations were made at the time of the field investigation and groundwater elevations will change with daily, seasonal, and climatological variations.

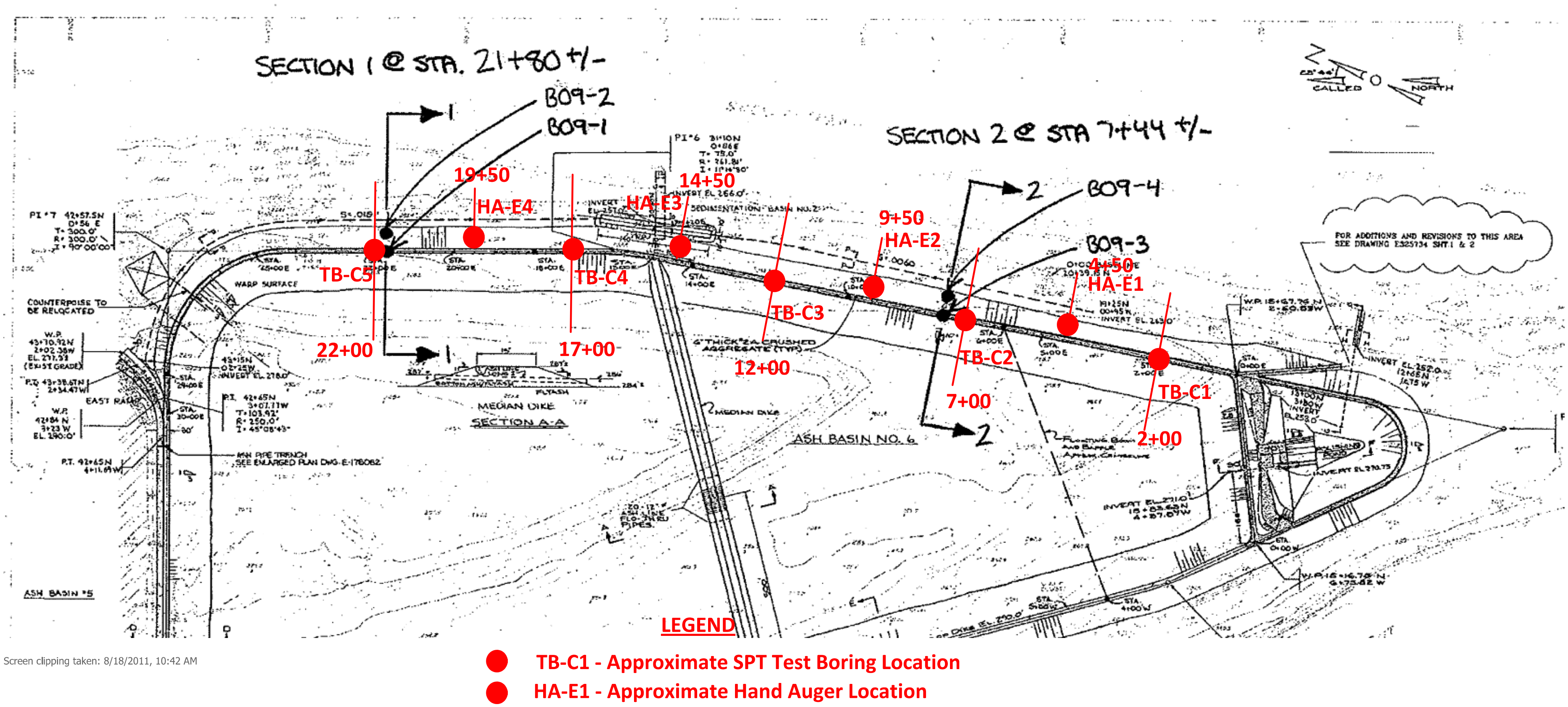
7.0 LIMITATIONS

This report has been prepared in accordance with generally accepted geotechnical design practices for specific application to this project. This report has been based on assumed conditions and characteristics of the proposed development where specific information was not available.

It is emphasized that this geotechnical investigation was completed for the areas indicated on the plan enclosed with this report and described herein. The validity of the projections and data contained in this report may be affected by the number of borings completed. The recommendations presented herein are based upon the number of borings purchased by the owner and while, depending upon the actual nature of subsurface conditions, those projections and conclusions may accurately set forth the nature of the subsurface conditions where the borings were made, the data presented herein are not to be applied to the remainder of the site.

APPENDIX

Figure 1 - Brunner Island SES Impoundment Dike Supplemental Exploration Locations



Screen clipping taken: 8/18/2011, 10:42 AM

TEST BORING LOG

SHEET 1 OF 1

PROJECT NAME: PPL Ash Basin 6 - Brunner Island Seepage & Embankment Stability Study
PROJECT NUMBER: 1100517
CLIENT: Schnabel Engineering Consultants, Inc.
LOCATION: Station 2+00

BORING NO.: TB-C1

☐ FIELD SURVEYED ☐ TOPO ESTIMATE

ELEV

TOP OF GROUND:
GROUNDWATER DATA: Dry
Depth: Not Encountered
Time: Completion

DEPTH (feet)	SAMPLE NUMBER	SAMPLE DEPTH (ft)	BLOWS PER 6"		SOIL DESCRIPTION	REMARKS
				0.0'-0.5'	Gray sand and gravel	**Road Base**
	S1	0'-2'	23-17-19-21	0.5' - 7.25'	Very stiff brown sandy clay with gravel; 100% Recovery	Uc= >4.5TSF
	S-2	2'-4'	8-7-13-14		Very stiff brown sandy clay; some gravel; [moist]	Uc= 1.5TSF
5					Stiff brown sandy clay with gravel;	Uc= 2.5TSF
	S3	4'-6'	8-6-4-9		[no gravel; moist to wet from 5'-5.5'] 100% Recovery	Uc= 1.25TSF
	S4	6'-8'	10-15-18-20	7.25' - 8.0'	Very dense brown clayey sand with gravel	100% Recovery
				8.0' - 12.0'		
10	S5	8'-9.3'	16-49-50/4"		Very dense brown sand and rounded gravel; some silt	100% Recovery
	S6	10'-12'	23-30-35-38		Very dense brown sand and rounded gravel; some silt	100% Recovery
				12.0' - 14.0'	Very dense brown silty fine to medium sand with gravel;	
	S7	12'-14'	32-34-29-35		[moist from 13.0' - 14.0']	100% Recovery
15				14.0' - 15.5'	Very dense brown sand and rounded gravel; some silt	
	S8	14'-16'	20-21-16-12	15.5' - 16.25'	Very stiff brown fine sandy clay; trace gravel	100% Recovery
				16.25' - 17.5'	Very dense brown sand and rounded gravel; some silt	
	S9	16'-18'	15-21-20-21	17.5' - 18.0'	Very dense brown silty fine sand	100% Recovery
				18.0' - 20.0'		
20	S10	18'-20'	17-22-31-44		Very dense sand and rounded gravel; some silt	100% Recovery
					-End of Boring at 20.0 feet-	
25						

ADVANTAGE ENGINEERS

910 Century Drive, Mechanicsburg, PA 17055
(717) 458-0800 FAX: (717) 458-0801
www.advantageengineers.com

RIG TYPE: Truck-Mounted CME-55

DRILLING METHOD: Hollow Stem Auger

ADVANTAGE REP.: Brian K. Hilsabeck

DRAWN/COMPILED BY: Brian K. Hilsabeck

DATE DRILLED: September 12, 2011

TEST BORING LOG

SHEET 1 OF 1

PROJECT NAME: PPL Ash Basin 6 - Brunner Island Seepage & Embankment Stability Study
PROJECT NUMBER: 1100517
CLIENT: Schnabel Engineering Consultants, Inc.
LOCATION: Station 7+00

FIELD SURVEYED | TOPO ESTIMATE

BORING NO.: TB-C2

ELEV

TOP OF GROUND:
GROUNDWATER DATA: Dry
Depth: Not Encountered
Time: Completion

DEPTH (feet)	SAMPLE NUMBER	SAMPLE DEPTH (ft)	BLOWS PER 6"		SOIL DESCRIPTION	REMARKS
				0.0'-0.3'	Gray sand and gravel	**Road Base**
	S1	0'-2'	20-12-12-18	0.3' - 2.0'	Very stiff brown clay; some sand, some gravel; 100% rec.	Uc= >4.5TSF
				2.0' - 3.0'	Very dense brown clayey sand with rounded gravel	
	S-2	2'-4'	11-17-14-15	3.0' - 4.0'	Very stiff brown clay; some sand, some gravel; 100% rec.	Uc= >4.5TSF
5				4.0' - 4.75'	Very dense brown clayey sand with rounded gravel	
	S3	4'-6'	6-10-17-20	4.75' - 5.25'	Very stiff brown clay	100% Recovery
				5.25' - 6.0'	Very dense brown clayey sand with rounded gravel	Cave at 6.5'
	S4	6'-8'	18-15-13-21	6.0' - 8.0'	Very stiff brown sandy clay with gravel; 30% Recovery	
				8.0' - 16.0'		
10	S5	8'-10'	15-25-22-23		Very dense brown sand and rounded gravel; some silt	100% Recovery
	S6	10'-12'	20-32-24-13		Very dense brown sand and rounded gravel; some silt	100% Recovery
	S7	12'-14'	11-23-30-31		Very dense brown sand and rounded gravel; some silt	100% Recovery
15						
	S8	14'-16'	19-30-21-26		Very dense brown clayey sand with rounded gravel; -tan to yellow sand seam from 15.0' - 15.25'	100% Recovery
				16.0' - 17.0'	Very dense brown clayey sand with rounded gravel	
	S9	16'-18'	23-18-18-22	17.0' - 17.5'	Very stiff brown clay; Uc= 3.5TSF	100% Recovery
				17.5' - 18.0'	Very dense brown silty fine sand	
				18.0' - 20.0'		
20	S10	18'-20'	6-11-10-15		Very stiff brown to gray clay; some fine sand; Uc= >4.5TSF	100% Recovery
					-End of Boring at 20.0 feet-	
25						

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RIG TYPE: Truck-Mounted CME-55

DRILLING METHOD: Hollow Stem Auger

ADVANTAGE REP.: Brian K. Hilsabeck

DRAWN/COMPILED BY: Brian K. Hilsabeck

DATE DRILLED: September 12, 2011

TEST BORING LOG

SHEET 1 OF 1

PROJECT NAME: PPL Ash Basin 6 - Brunner Island Seepage & Embankment Stability Study
PROJECT NUMBER: 1100517
CLIENT: Schnabel Engineering Consultants, Inc.
LOCATION: Station 12+00

☐ FIELD SURVEYED ☐ TOPO ESTIMATE

ELEV

BORING NO.: TB-C3
TOP OF GROUND:
GROUNDWATER DATA: Dry
Depth: Not Encountered
Time: Completion

DEPTH (feet)	SAMPLE NUMBER	SAMPLE DEPTH (ft)	BLOWS PER 6"		SOIL DESCRIPTION	REMARKS
				0.0'-0.4'	Gray sand and gravel	**Road Base**
	S1	0'-2'	35-20-18-12	0.4' - 2.0'	Very stiff brown sandy clay with rounded gravel; 100% rec.	Uc= >4.5TSF
				2.0' - 5.5'		
	S-2	2'-4'	8-8-13-21		Very dense brown clayey sand with rounded gravel	70% Recovery
5						
	S3	4'-6'	13-17-12-18	5.5' - 6.0'	Very stiff brown sandy clay with rounded gravel; 100% rec.	Uc= 3.5TSF
				6.0' - 8.0'		
	S4	6'-8'	13-17-15-13		Very dense brown clayey sand with rounded gravel	100% Recovery
				8.0' - 10.0'	Very dense brown silty sand and rounded gravel; 4" clay seam from 9.0' - 9.3'; Uc=>4.5TSF	
10	S5	8'-10'	14-21-23-12			100% Recovery
				10.0' - 10.75'	Very stiff brown sandy clay; trace gravel	Uc= >4.5TSF
	S6	10'-12'	19-24-22-42	10.75' - 12.0'	Very dense brown sand with rounded gravel; some silt	100% Recovery
				12.0' - 16.0'		
	S7	12'-14'	31-25-20-48		Very dense brown clayey sand with rounded gravel	100% Recovery
15						
	S8	14'-16'	10-10-20-25		Very dense brown clayey sand with rounded gravel; [MOIST]	100% Recovery
				16.0' - 18.0'		
	S9	16'-18'	38-31-27-31		Very dense brown silty sand and rounded gravel; Light brown silty fine sand from 17.75' to 18.0'	100% Recovery
				18.0' - 19.0'	Very dense brown clayey sand with rounded gravel	
20	S10	18'-20'	19-25-26-23	19.0' - 19.2'	Very dense light brown silty fine sand	100% Recovery
				19.2' - 20.0'	Very stiff brown clay; trace sand, trace gravel Uc=>4.5TSF	
					-End of Boring at 20.0 feet-	
25						

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www.advantageengineers.com

RIG TYPE: Truck-Mounted CME-55
DRILLING METHOD: Hollow Stem Auger
ADVANTAGE REP.: Brian K. Hilsabeck
DRAWN/COMPILED BY: Brian K. Hilsabeck
DATE DRILLED: September 2, 2011

TEST BORING LOG

SHEET 1 OF 1

PROJECT NAME: PPL Ash Basin 6 - Brunner Island Seepage & Embankment Stability Study
PROJECT NUMBER: 1100517
CLIENT: Schnabel Engineering Consultants, Inc.
LOCATION: Station 17+00

☐ FIELD SURVEYED ☐ TOPO ESTIMATE

ELEV

BORING NO.: TB-C4

TOP OF GROUND:

GROUNDWATER DATA: Wet
Depth: 11.3 ft
Time: Completion

DEPTH (feet)	SAMPLE NUMBER	SAMPLE DEPTH (ft)	BLOWS PER 6"	SOIL DESCRIPTION		REMARKS
				0.0'-0.3'	Gray sand and gravel	**Road Base**
	S1	0'-2'	35-20-18-12	0.3' - 2.0'	Very dense brown clayey sand with rounded gravel	70% Recovery
				2.0' - 3.0'	Very stiff brown clay; some sand, some rounded gravel	Uc= >4.5TSF
	S-2	2'-4'	8-8-13-21	3.0' - 4.0'	Very dense brown sand and rounded gravel; some clay	100% Recovery
5				4.0' - 5.25'	Very dense brown clayey sand with rounded gravel	
	S3	4'-6'	13-17-12-18	5.25' - 6.0'	Very stiff brown clay; some sand, some rounded gravel	Uc= >4.5TSF
				6.0' - 8.0'		
	S4	6'-8'	13-17-15-13		Very dense brown clayey sand with rounded gravel	60% Recovery
				8.0' - 9.5'	Very dense brown silty sand and rounded gravel	
10	S5	8'-10'	14-21-23-12	9.5' - 10.0'	Very dense brown clayey sand with rounded gravel	83% Recovery
				10.0' - 12.0'	Very stiff brown clay; some sand; gravel and sand from 10.2' to 10.4' and 11.8' to 12.0'; 100% Recovery	Uc= >4.5TSF
	S6	10'-12'	19-24-22-42			**H_2O at 11.3'**
				12.0' - 17.25'		
	S7	12'-14'	31-25-20-48		Very dense brown clayey sand with rounded gravel; [WET from 12.1' to 12.2' and 13.25' to 13.5']	100% Recovery
15						
	S8	14'-16'	10-10-20-25		Very dense brown clayey sand with rounded gravel; [WET]	45% Recovery
	S9	16'-18'	38-31-27-31	17.25' - 18.0'	Very stiff brown clay; some sand, some gravel [DRY]	Uc= 4.0TSF
				18.0' - 18.25'	Very dense brown clayey sand with rounded gravel [WET]	
20	S10	18'-20'	19-25-26-23	18.25' - 20.0'	Very dense light brown silty fine sand [DRY]	100% Recovery
					-End of Boring at 20.0 feet-	
25						

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(717) 458-0800 FAX: (717) 458-0801
www.advantageengineers.com

RIG TYPE: Truck-Mounted CME-55

DRILLING METHOD: Hollow Stem Auger

ADVANTAGE REP.: Brian K. Hilsabeck

DRAWN/COMPILED BY: Brian K. Hilsabeck

DATE DRILLED: September 2, 2011

TEST BORING LOG

SHEET 1 OF 1

PROJECT NAME: PPL Ash Basin 6 - Brunner Island Seepage & Embankment Stability Study
PROJECT NUMBER: 1100517
CLIENT: Schnabel Engineering Consultants, Inc.
LOCATION: Station 22+00

FIELD SURVEYED   TOPO ESTIMATE

BORING NO.: TB-C5

ELEV

TOP OF GROUND:
GROUNDWATER DATA: Dry
Depth: Not Encountered
Time: Completion

DEPTH (feet)	SAMPLE NUMBER	SAMPLE DEPTH (ft)	BLOWS PER 6"		SOIL DESCRIPTION	REMARKS
				0.0'-0.4'	Gray sand and gravel	**Road Base**
	S1	0'-2'	18-12-21-29	0.4' - 2.0'	Very dense brown silty sand with rounded gravel	100% Recovery
				2.0' - 6.5'		
	S-2	2'-4'	8-13-10-10		Very stiff brown sandy clay; some rounded gravel; 75% Recovery, Uc= >4.5TSF	
5						
	S3	4'-6'	12-10-10-13		Very stiff brown sandy clay with rounded gravel; 100% Recovery, Uc= >4.5TSF	
				6.5' - 10.0'		Cave at 7.0 ft
	S4	6'-7.4'	10-32-50/5"		Very dense brown silty sand with rounded gravel; 42% Recovery; auger chatter from 7.0' to 7.7'	
10	S5	8'-10'	8-16-17-16		Very dense brown clayey sand with rounded gravel; 83% Recovery; Uc= 2.0TSF	
				10.0' - 11.0'	Very stiff brown sandy clay with rounded gravel	Uc= >4.5TSF
	S6	10'-12'	6-15-16-23	11.0' - 12.0'	Very dense brown sand with rounded gravel; some silt	100% Recovery
				12.0' - 13.25'	Very stiff brown sandy clay, some rounded gravel	Uc= >4.5TSF
	S7	12'-14'	9-16-19-50	13.25' - 14.0'	Very dense brown silty sandy rounded gravel	
15				14.0' - 15.5'	Very dense brown silty sand with rounded gravel	100% Recovery
	S8	14'-16'	16-25-29-24	15.5' - 16.0'		Uc= >4.5TSF
				16.0'-16.5'	Very dense brown silty sand with rounded gravel	
				16.5 - 17.5'	Very stiff brown sandy clay with rounded gravel	Uc= >4.5TSF
	S9	16'-18'	18-24-26-22	17.5' - 18.0	Very dense brown clayey sand with rounded gravel	100% Recovery
				18.0' - 18.5'	Very dense brown silty fine sand	
20	S10	18'-20'	10-7-9-13	18.5' - 19.75'	Very stiff brown sandy clay; trace gravel;	Uc= >4.5TSF
				19.75' - 20.0'	Brown silty sand with gravel from 100% Recovery	
					-End of Boring at 20.0 feet-	
25						

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RIG TYPE: Truck-Mounted CME-55

DRILLING METHOD: Hollow Stem Auger

ADVANTAGE REP.: Brian K. Hilsabeck

DRAWN/COMPILED BY: Brian K. Hilsabeck

DATE DRILLED: September 1, 2011

Density and Unit Weight of Soil in Place by Sand-Cone Method

(per ASTM Designation D 1556)

Date:	September 2, 2011	**Project:**	PPL Ash Basin - Brunner Island Transient Seepage & Embankment Stability Study
Client:	Schnabel Engineering Consultants, Inc.	**Project No.:**	1100517

Test Number	1	2	3	4	5
Material					
Test Location	HA-E1	HA-E2	HA-E3	HA-E4	
Test Elevation/Lift					
Wt. of sand before (lbs.)	14.43	15.22	15.55	15.58	
Wt. of sand after (lbs.)	4.32	5.71	6.98	3.06	
Wt. of sand in cone (lbs.)	3.82	3.82	3.82	3.82	
Wt. of sand in hole (lbs.)	6.29	5.69	4.75	8.70	
Volume of hole (ft^3)	0.0645	0.0583	0.0487	0.0892	
Wt. of wet soil (lbs.)	7.94	6.86	6.18	11.82	
Moisture sample wet wt. (g)	3601.5	3111.6	2801.9	5361.5	
Moisture sample dry wt. (g)	3202.8	2838.1	2581.7	5085.7	
Wt. of water in sample	398.7	273.5	220.2	275.8	
Percent field moisture (%)	**12.4%**	**9.6%**	**8.5%**	**5.4%**	
Wt. of dry soil (lbs.)	7.06	6.26	5.69	11.21	
Wet density (lbs./ft^3)	123.2	117.6	126.9	132.5	
Dry density (lbs./ft^3)	109.5	107.3	117.0	125.7	
Field compaction (%)					
Maximin unit weight (lbs./ft^3)					
Optimum moisture content (%)					
Specified compaction					

The results stated on this report relate only to the material specifically identified.
These relative humidity results reflect the condition of the concrete floor at the time of this test.

Reviewed by: ______________________

6520 Stonegate Drive, Suite 110, Allentown, Pennsylvania 18106
(610) 366-7120 (610) 366-7121 (fax)

RESULTS OF INFILTRATION ANALYSIS			
TEST PIT LOCATION	**INVERT ELEVATION** (feet below existing grade)	**TEST METHOD**	**INFILTRATION RATE (in/hr)**
TB-C1	8.0	CASE-PIPE	1.08
TB-C2	5.0	CASE-PIPE	0.6
TB-C3	8.0	CASE-PIPE	4.68
TB-C4	4.0	CASE-PIPE	0.36
TB-C5	4.5	CASE-PIPE	No Measurable Rate
HA-E1	2.0	DOUBLE RING	0.2
HA-E2	2.0	DOUBLE RING	0.84
HA-E3	2.25	DOUBLE RING	0.31
HA-E4	2.5	DOUBLE RING	0.25

910 Century Drive, Mechanicsburg, Pennsylvania 17055
(717) 458-0800 (717) 458-0801(fax)

Schnabel Engineering Consultants, Inc.
PPL Ash Basin Brunner Island
Transient Seepage and Slope Stability Study
Addendum No. 1 - Additional Project Data Acquisition
Laboratory Testing Assignments

Date: 9/19/2011
By: SAR

Test: NMC (D2216)

No.	9	2	4	2	10	1	4	2	10	2
Depth (ft)										
0	TB-C1		TB-C2		TB-C3		TB-C4		TB-C5	
1	S-1		S-1		S-1		S-1		S-1	
2										
3	S-2	B-1	S-2	B-1	S-2	B-1	S-2	B-1	S-2	B-1
4										
5	S-3		S-3		S-3		S-3		S-3	
6										
7	S-4		S-4		S-4		S-4		S-4	
8		B-2		B-2		B-2		B-2		B-2
9	S-5		S-5		S-5		S-5		S-5	
10										
11	S-6		S-6		S-6		S-6		S-6	
12										
13	S-7	B-3	S-7	B-3	S-7	B-3	S-7	B-3	S-7	B-3
14										
15	S-8		S-8		S-8		S-8		S-8	
16										
17	S-9		S-9		S-9		S-9		S-9	
18		B-4		B-4		B-4		B-4		B-4
19	S-10		S-10		S-10		S-10		S-10	
20										

No.	1	1	1	1	TOT 50
Depth (ft)					
0	HA-E1	HA-E2	HA-E3	HA-E4	
1					
2	B-1	B-1	B-1	B-1	
3					
4					
5					

S-1, etc (split spoon samples)
B-1, etc (bulk samples from auger cuttings)

MOISTURE CONTENT OF SOIL
AASHTO T-265 or ASTM D-2216

Project #: 11001-37
Project: Ash Basin #6, Brunner Island
Date: 9/21/2011

BORING NO.	SAMPLE NO.	weight of tare	weight wet soil + tare	weight dry soil + tare	MOISTURE CONTENT (%)
TB-C1	S-1	9.08	237.11	213.58	11.51
TB-C1	S-2	8.50	253.36	220.59	15.45
TB-C1	S-3	9.31	282.13	247.16	14.70
TB-C1	S-4	9.15	251.67	234.34	7.70
TB-C1	S-5	8.74	290.61	276.88	5.12
TB-C1	S-6	8.42	312.79	298.70	4.85
TB-C1	S-7	9.28	354.27	331.85	6.95
TB-C1	S-8	6.87	288.79	254.89	13.67
TB-C1	S-9	9.10	169.75	154.48	10.50
TB-C1	S-10	9.10	261.55	251.41	4.18
TB-C2	S-3	8.44	266.79	250.60	6.69
TB-C2	S-4	8.60	245.19	230.57	6.59
TB-C2	S-6	8.39	343.67	326.88	5.27
TB-C2	S-10	8.47	209.17	184.78	13.83
TB-C3	S-1	8.39	276.92	253.45	9.58
TB-C3	S-2	9.04	232.54	220.62	5.63
TB-C3	S-3	9.12	238.25	207.22	15.66
TB-C3	S-4	8.47	282.53	266.98	6.02
TB-C3	S-5	8.40	309.30	293.70	5.47
TB-C3	S-6	8.31	290.48	276.73	5.12
TB-C3	S-7	8.56	221.14	209.12	5.99
TB-C3	S-8	8.49	275.95	259.08	6.73
TB-C3	S-9	9.08	208.12	192.72	8.39
TB-C3	S-10	9.22	257.26	228.48	13.13
TB-C4	S-3	8.44	271.87	247.22	10.32
TB-C4	S-4	8.35	90.44	84.24	8.17
TB-C4	S-6	8.28	238.10	210.35	13.73
TB-C4	S-8	9.02	353.44	328.99	7.64
TB-C4	S-9	8.45	199.45	185.05	8.15
TB-C5	S-1	9.11	244.77	224.65	9.33
TB-C5	S-2	8.65	221.71	195.09	14.28
TB-C5	S-3	9.79	284.40	251.00	13.85
TB-C5	S-4	9.70	233.75	214.37	9.47
TB-C5	S-5	9.60	238.35	223.62	6.88
TB-C5	S-6	9.86	315.69	297.32	6.39
TB-C5	S-7	9.72	268.09	255.50	5.12
TB-C5	S-8	9.63	221.38	205.62	8.04
TB-C5	S-9	9.72	247.96	236.13	5.23
TB-C5	S-10	9.75	258.41	223.83	16.15

By: DFS Ck'd: MCM

GTS TECHNOLOGIES

MOISTURE CONTENT OF SOIL
AASHTO T-265 or ASTM D-2216

Project #: 11001-37
Project: Ash Basin #6, Brunner Island
Date: 9/26/2011

BORING NO.	SAMPLE NO.	weight of tare	weight wet soil + tare	weight dry soil + tare	MOISTURE CONTENT (%)
TB-C1	B-1	42.62	2674.21	2422.81	10.56
TB-C1	B-2	14.38	1314.13	1226.32	7.25
TB-C2	B-2	14.29	1367.91	1289.54	6.15
TB-C2	B-3	43.09	3035.81	2865.74	6.03
TB-C3	B-2	42.59	2772.42	2596.25	6.90
TB-C4	B-2	14.29	1393.91	1308.74	6.58
TB-C5	B-1	11.07	1223.61	1141.27	7.29
TB-C5	B-2	11.68	1311.00	1236.11	6.12
HA-E1	B-1	43.68	3091.70	2877.73	7.55
HA-E2	B-1	43.26	3097.79	2892.80	7.19
HA-E3	B-1	43.87	3099.52	2835.42	9.46
HA-E4	B-1	44.29	3096.24	2888.89	7.29

By: DFS Ck'd: MCM

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Schnabel ENGINEERING

Schnabel Engineering Consultants, Inc.
PPL Ash Basin Brunner Island
Transient Seepage and Slope Stability Study
Addendum No. 1 - Additional Project Data Acquisition
Laboratory Testing Assignments

Date: 9/19/2011
By: SAR

Test: Sieve/Hydr (D422)

No.	2	1	2	1	1	1	1		1	
Depth (ft)										
0	TB-C1		TB-C2		TB-C3		TB-C4		TB-C5	
1	S-1		S-1		S-1		S-1		S-1	
2										
3	S-2	**B-1**	S-2	B-1	S-2	B-1	S-2	B-1	**S-2**	B-1
4										
5	**S-3**		**S-3**		S-3		**S-3**		S-3	
6										
7	S-4		S-4		**S-4**		S-4		S-4	
8		B-2		B-2		**B-2**		B-2		B-2
9	**S-5**		S-5		S-5		S-5		S-5	
10										
11	S-6		**S-6**		S-6		S-6		S-6	
12										
13	S-7	B-3	S-7	**B-3**	S-7	B-3	S-7	B-3	S-7	B-3
14										
15	S-8		S-8		S-8		S-8		S-8	
16										
17	S-9		S-9		S-9		S-9		S-9	
18		B-4		B-4		B-4		B-4		B-4
19	S-10		S-10		S-10		S-10		S-10	
20										

No.	1	1	1	1	TOT 14
Depth (ft)					
0	HA-E1	HA-E2	HA-E3	HA-E4	
1					
2	**B-1**	**B-1**	**B-1**	**B-1**	
3					
4					
5					

S-1, etc (split spoon samples)
B-1, etc (bulk samples from auger cuttings)

Schnabel Engineering Consultants, Inc.
PPL Ash Basin Brunner Island
Transient Seepage and Slope Stability Study
Addendum No. 1 - Additional Project Data Acquisition
Laboratory Testing Assignments

Date: 9/19/2011
By: SAR

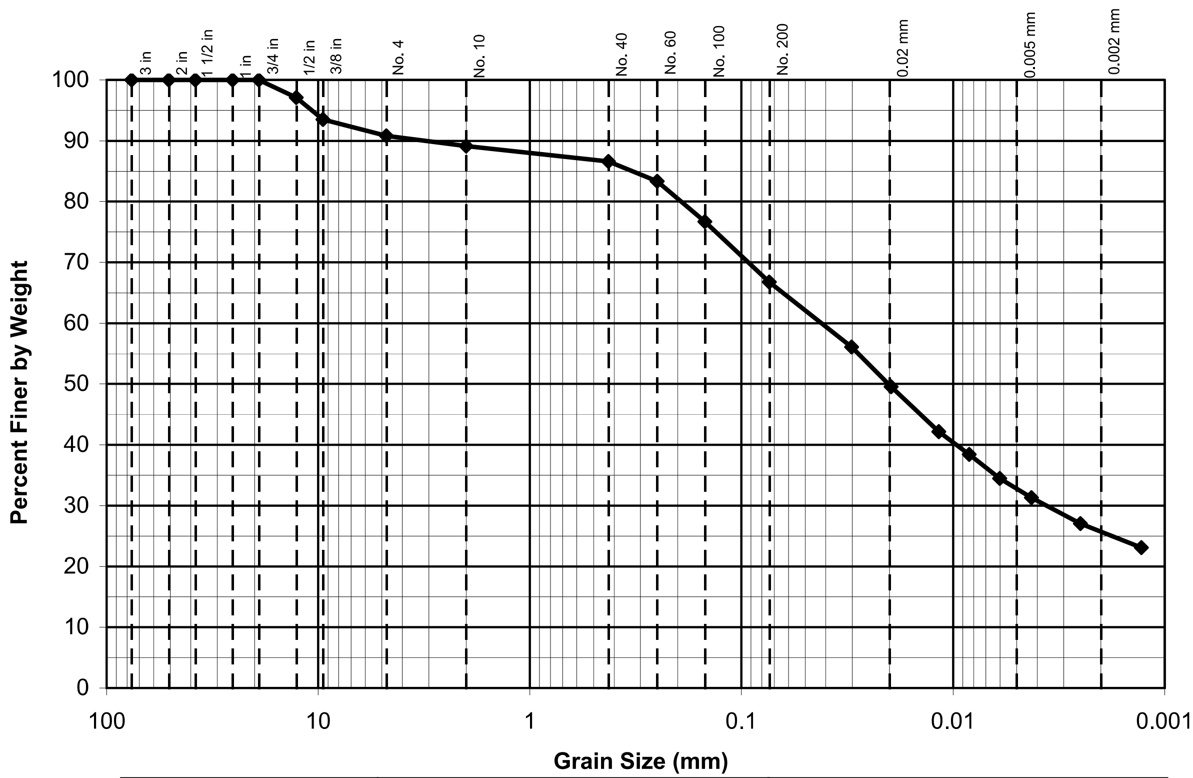
Test: A. Limits (D4318)

Depth (ft)	TB-C1		TB-C2		TB-C3		TB-C4		TB-C5	
No.	1		1				2		1	
0	TB-C1		TB-C2		TB-C3		TB-C4		TB-C5	
1	S-1		S-1		S-1		S-1		S-1	
2										
3	S-2	B-1	S-2	B-1	S-2	B-1	S-2	B-1	**S-2**	B-1
4										
5	**S-3**		S-3		S-3		**S-3**		S-3	
6										
7	S-4		**S-4**		S-4		S-4		S-4	
8		B-2		B-2		B-2		B-2		B-2
9	S-5		S-5		S-5		S-5		S-5	
10										
11	S-6		S-6		S-6		**S-6**		S-6	
12										
13	S-7	B-3	S-7	B-3	S-7	B-3	S-7	B-3	S-7	B-3
14										
15	S-8		S-8		S-8		S-8		S-8	
16										
17	S-9		S-9		S-9		S-9		S-9	
18		B-4		B-4		B-4		B-4		B-4
19	S-10		S-10		S-10		S-10		S-10	
20										

Depth (ft)	HA-E1	HA-E2	HA-E3	HA-E4	TOT
No.	1	1			7
0	HA-E1	HA-E2	HA-E3	HA-E4	
1					
2	**B-1**	**B-1**	B-1	B-1	
3					
4					
5					

S-1, etc (split spoon samples)
B-1, etc (bulk samples from auger cuttings)

Grain Size Distribution Curve



GRAVEL		SAND			FINES	
COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
9.2 %		24.0 %			66.8 %	
0.0 %	9.2 %	1.7 %	2.5 %	19.8 %	34.2 %	32.6 %

USCS

Project:	Ash Basin #6 - Brunner Island	**Soil Type:**	sandy lean CLAY
Boring No.:	TB-C5		
Station:		**Classification:**	CL, A-4 (4)
Offset:		LL = 27 %	PL = 18 %
Sample No.:	S-2	PI = 9 %	w = 14.3 %
Depth:	2.0 - 4.0 ft	**Spec. Grav.:**	2.65 (assumed)

ote: Minimum mass requirement was not met. Mass used for the test = 186.44 grams

GTS TECHNOLOGIES

CLASSIFICATION TEST RESULTS

AASHTO T-88, T-89, T-90, M-145

or ASTM D 422, D 4318, D 2487

10/6/2011

AASHTO R18

GTS No. 11001-37 By: DFS Ckd: MCM

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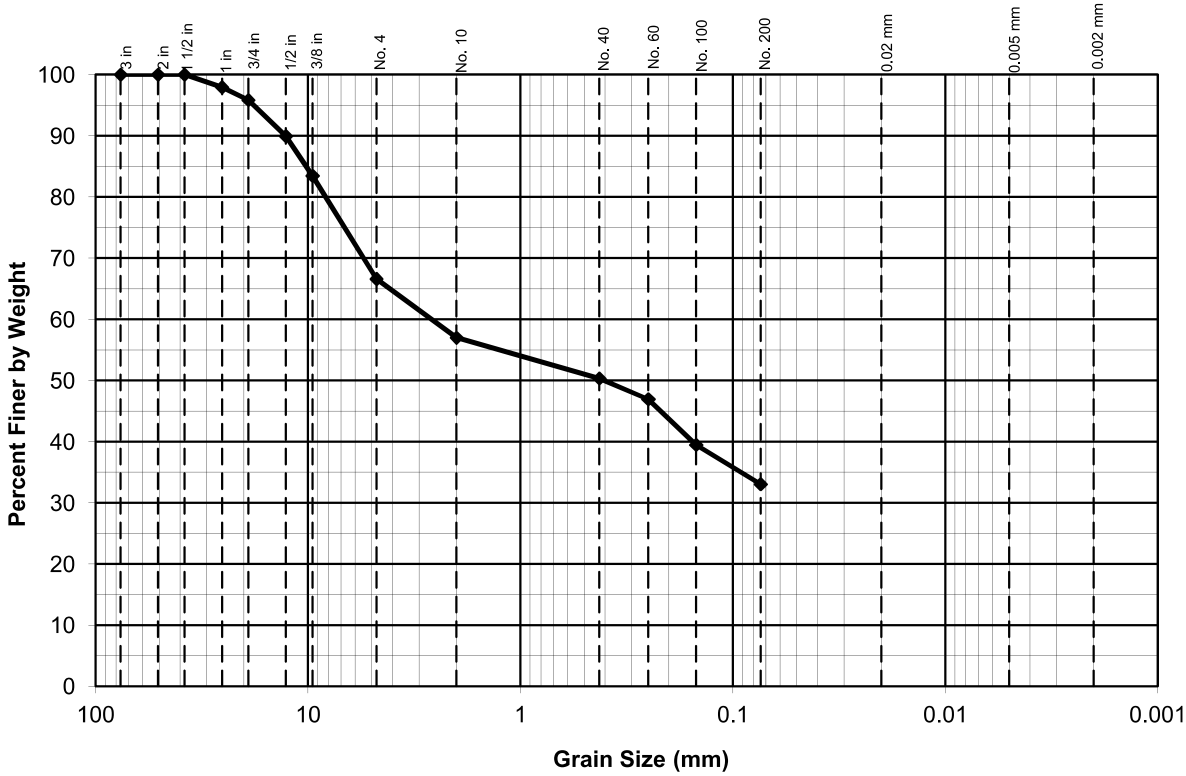
Grain Size Distribution Curve



GRAVEL		SAND			FINES	
COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
33.4 %		33.5 %				
4.2 %	29.2 %	9.6 %	6.7 %	17.3 %	--	--

USCS

Project: Brunner Island - Ash Basin #6
Boring No.: TB-C5

Sample No.: B-1
Depth: 0.0 - 5.0 ft

GTS TECHNOLOGIES

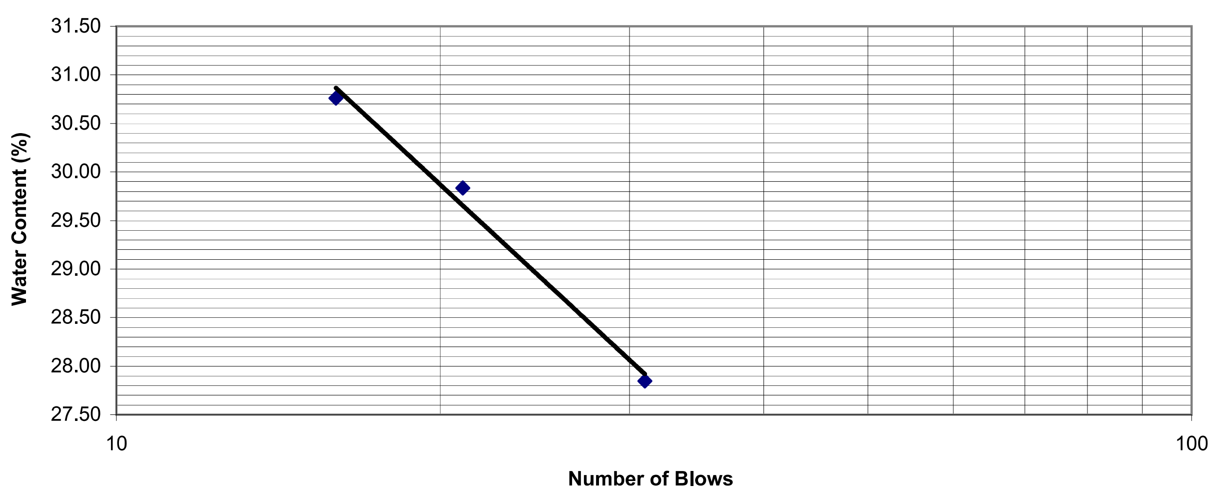
GRADATION TEST RESULTS
AASHTO T-88
or ASTM 422
10/24/2011

AASHTO R18

GTS No. 11001-37 By: DFS Ckd: dsc

LIQUID LIMIT					
Dish No.					
Blows	31	21	16	0	0
Wt. of Dish	2.48	2.50	2.49	0.00	0.00
Wt. Dish + Wet Soil	14.05	14.25	14.18	0.00	0.00
Wt. Dish + Dry Soil	11.53	11.55	11.43	0.00	0.00
Wt. Of Dry Soil	9.05	9.05	8.94	0.00	0.00
Wt. Of Water	2.52	2.70	2.75	0.00	0.00
% Moist	27.85	29.83	30.76		

PLASTIC LIMIT					
Dish No.					
Wt. of Dish	2.53	2.45	0.00	0.00	0.00
Wt. Dish + Wet Soil	8.95	8.19	0.00	0.00	0.00
Wt. Dish + Dry Soil	7.89	7.24	0.00	0.00	0.00
Wt. Of Dry Soil	5.36	4.79	0.00	0.00	0.00
Wt. Of Water	1.06	0.95	0.00	0.00	0.00
% Moist	19.78	19.83			



Project: Ash Basin #6 - Brunner Island
Boring No.: TB-C4
Sample No.: S-6
Depth: 10.0 - 12.0 ft

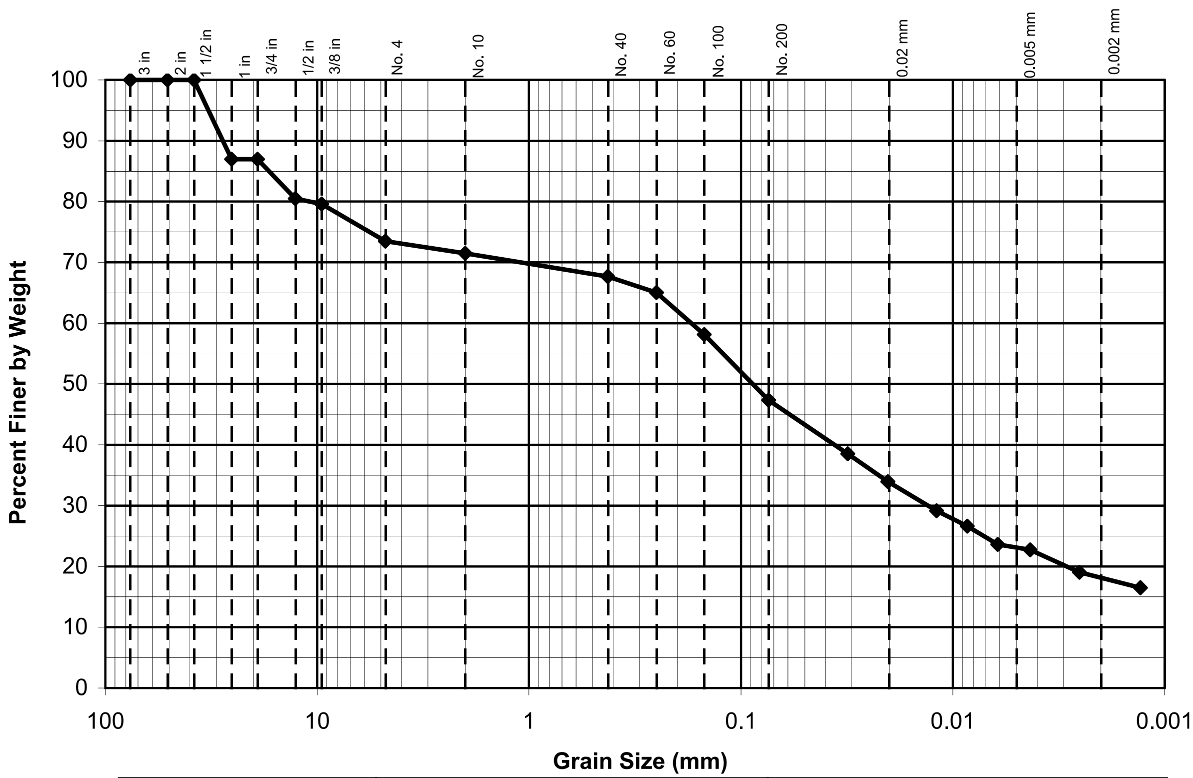
LL	29 %
PL	20 %
PI	9 %
w	13.7%

ATTERBERG LIMITS TEST DATA
AASHTO T-88, T-99 or ASTM 4318

GTS TECHNOLOGIES, INC.
10/7/2011

GTS No. 11001-37 By: DFS Ckd: MCM

Grain Size Distribution Curve



GRAVEL		SAND			FINES	
COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
26.5 %		26.2 %			47.3 %	
13.0 %	13.5 %	2.0 %	3.8 %	20.4 %	24.2 %	23.1 %

USCS

Project:	Ash Basin #6 - Brunner Island	**Soil Type:**	silty, clayey GRAVEL with sand
Boring No.:	TB-C4		
Station:		**Classification:**	GC-GM, A-4 (1)
Offset:		LL = 24 %	PL = 17 %
Sample No.:	S-3	PI = 7 %	w = 10.3 %
Depth:	4.0 - 6.0 ft	**Spec. Grav.:**	2.65 (assumed)

ote: Minimum mass requirement was not met. Mass used for the test = 238.78 grams

GTS TECHNOLOGIES

CLASSIFICATION TEST RESULTS

AASHTO T-88, T-89, T-90, M-145

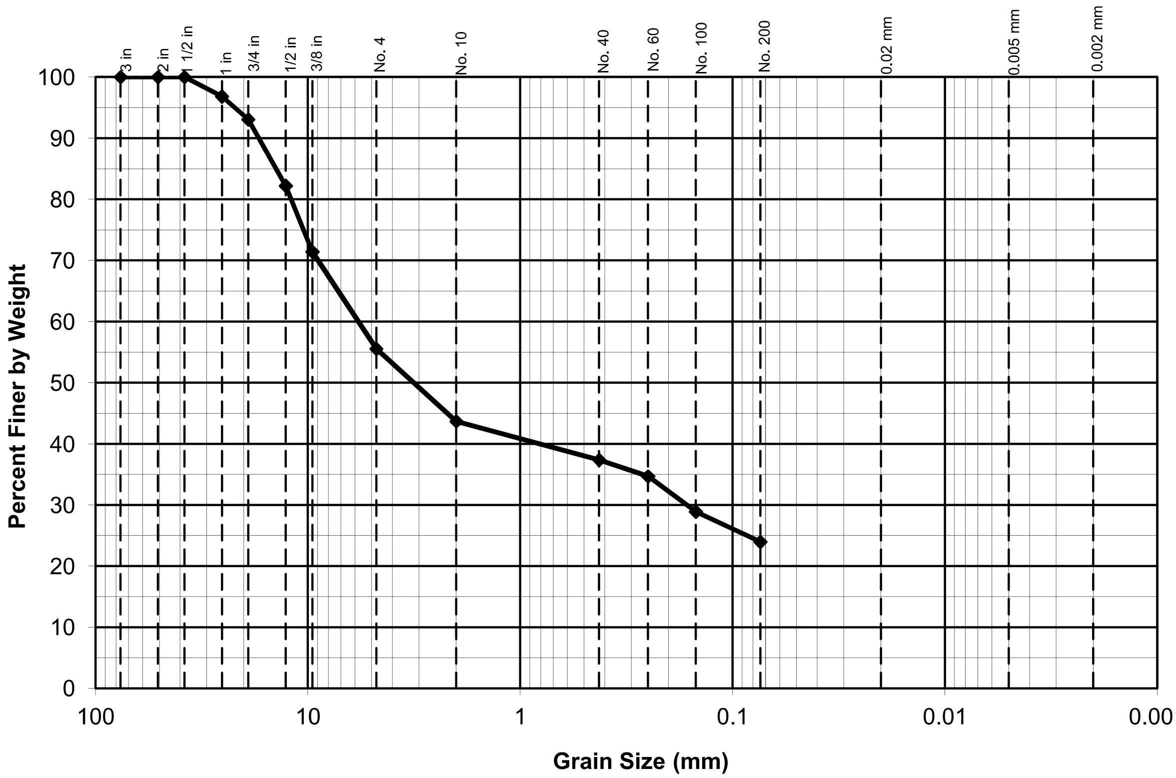
or ASTM D 422, D 4318, D 2487

10/5/2011

AASHTO R18

GTS No. 11001-37 By: DFS Ckd: MCM

Grain Size Distribution Curve



GRAVEL		SAND			FINES	
COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
44.5 %		31.6 %				
7.0 %	37.5 %	11.9 %	6.3 %	13.4 %	--	--

USCS

Project: Brunner Island - Ash Basin #6
Boring No.: TB-C4

Sample No.: B-2
Depth: 5.0 - 10.0 ft

GTS TECHNOLOGIES

GRADATION TEST RESULTS
AASHTO T-88
or ASTM 422
10/24/2011

AASHTO R18

GTS No. 11001-37 By: KJE Ckd: dsc

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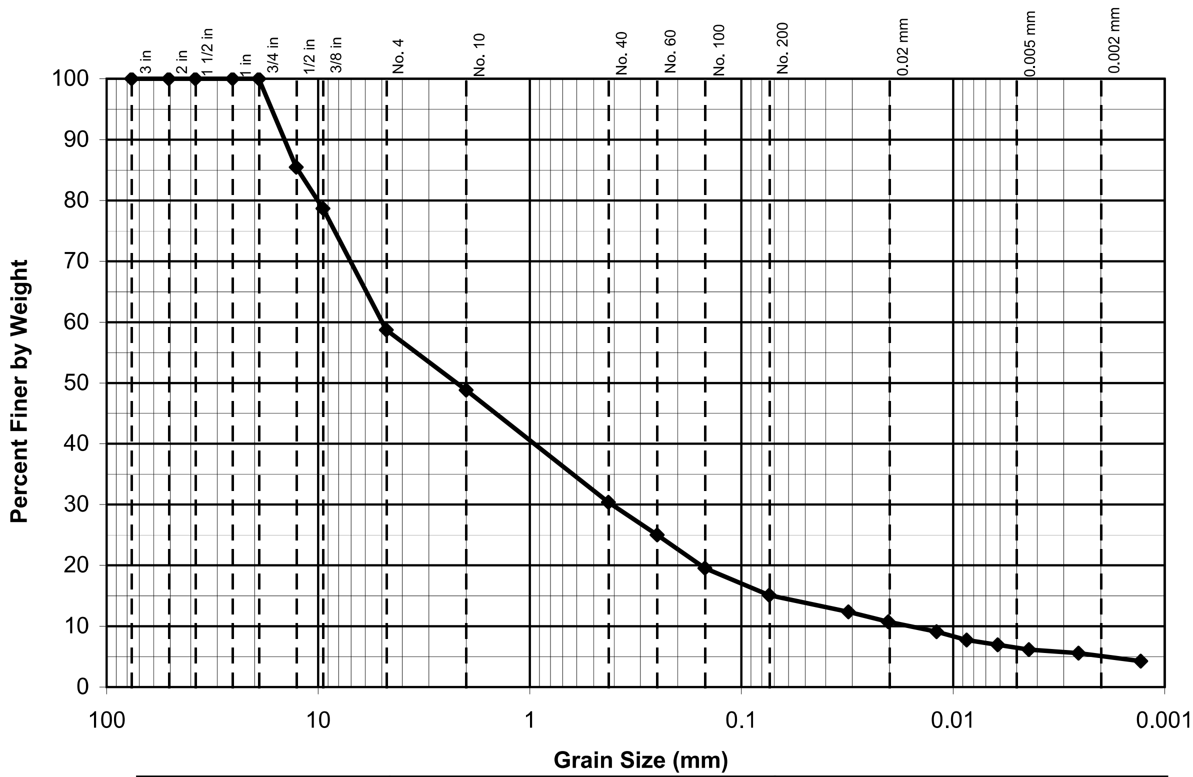
Grain Size Distribution Curve



GRAVEL		SAND			FINES	
COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
41.3 %		43.6 %			15.1 %	
0.0 %	41.3 %	9.9 %	18.4 %	15.3 %	8.6 %	6.5 %

USCS

Project: Ash Basin #6 - Brunner Island
Boring No.: TB-C3

Sample No.: S-4
Depth: 6.0 - 8.0 ft

Moisture Content: w = 6.0 %

GTS TECHNOLOGIES

GRADATION TEST RESULTS
AASHTO T-88
or ASTM 422
10/6/2011

GTS No. 11001-37 By: DFS Ckd: MCM

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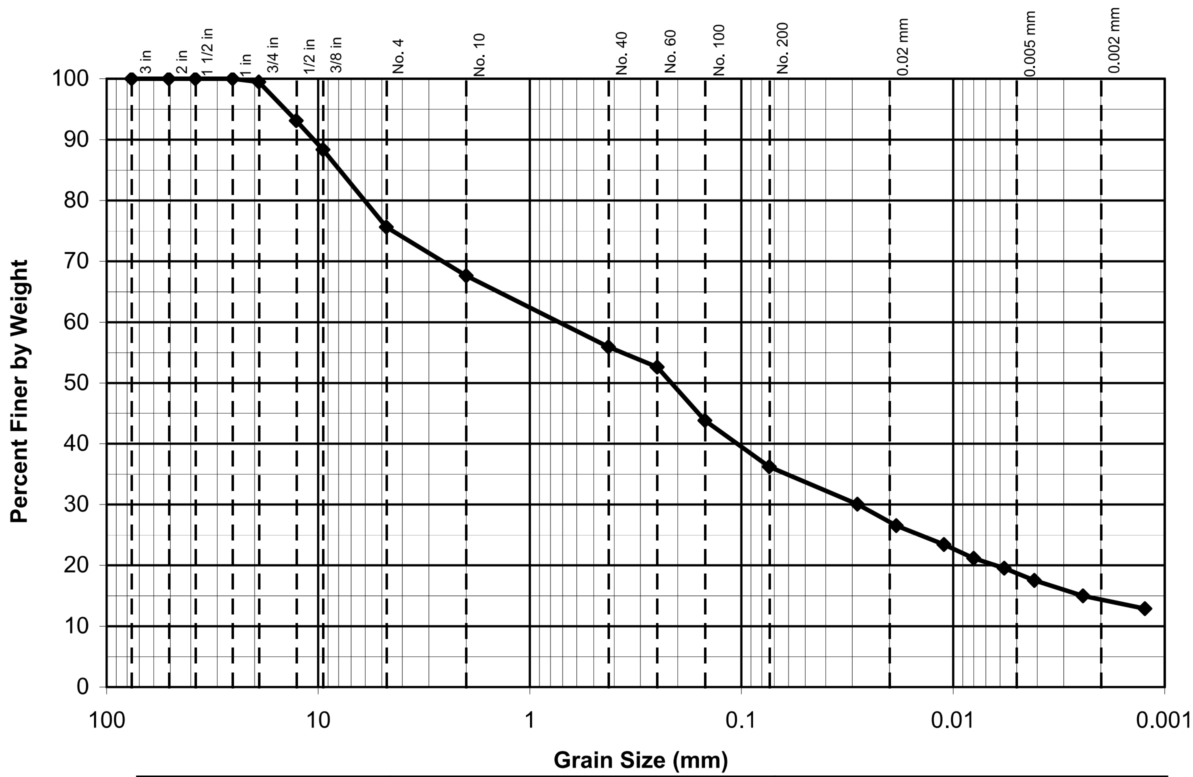
Grain Size Distribution Curve



GRAVEL		SAND			FINES	
COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
24.4 %		39.4 %			36.2 %	
0.5 %	23.9 %	8.0 %	11.7 %	19.7 %	17.6 %	18.6 %

USCS

Project: Ash Basin #6 - Brunner Island
Boring No.: TB-C3

Sample No.: B-2
Depth: 5.0 - 10.0 ft

Moisture Content: w = 6.9 %

GTS TECHNOLOGIES

GRADATION TEST RESULTS
AASHTO T-88
or ASTM 422
10/6/2011

AASHTO R18

GTS No. 11001-37 By: DFS Ckd: MCM

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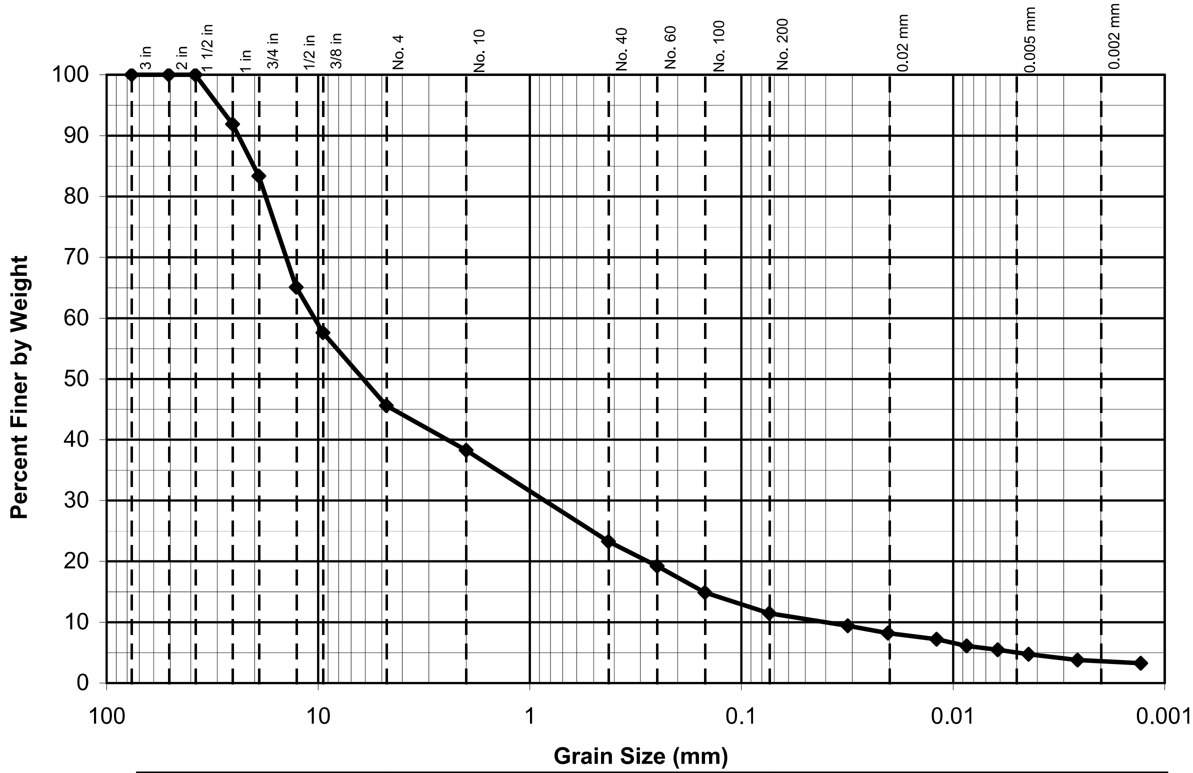
Grain Size Distribution Curve



GRAVEL		SAND			FINES	
COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
54.4 %		34.1 %			11.5 %	
16.7 %	37.7 %	7.3 %	15.0 %	11.8 %	6.5 %	5.0 %

USCS

Project: Ash Basin #6 - Brunner Island
Boring No.: TB-C2

Sample No.: S-6
Depth: 10.0 - 12.0 ft
Moisture Content: w = 5.3 %

GTS TECHNOLOGIES

GRADATION TEST RESULTS

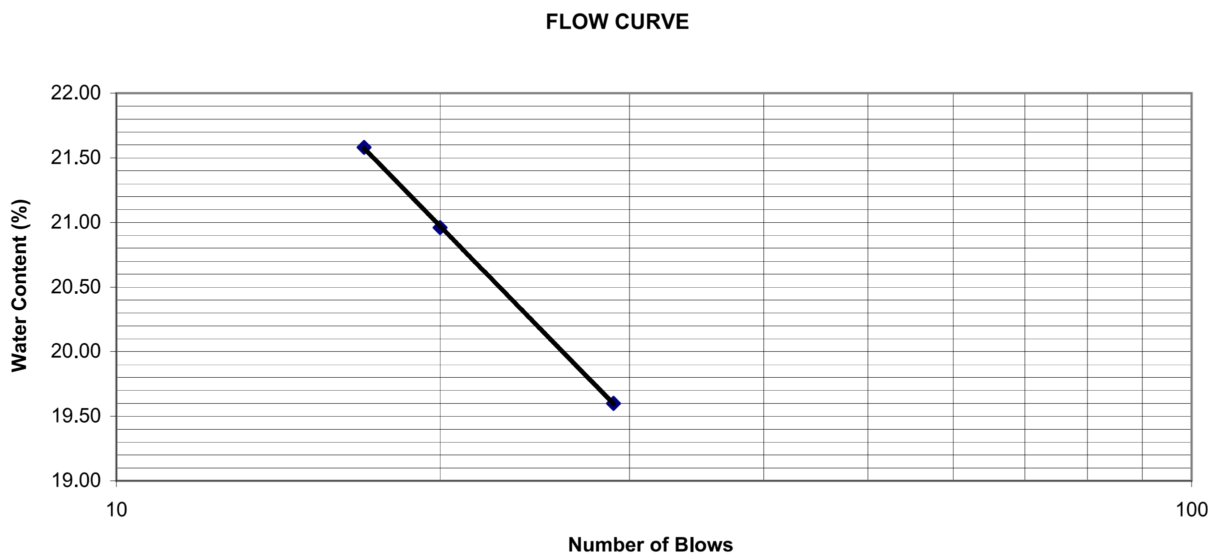
AASHTO T-88
or ASTM 422
10/6/2011

AASHTO R18

GTS No. 11001-37 By: DFS Ckd: MCM

LIQUID LIMIT					
Dish No.					
Blows	29	20	17	0	0
Wt. of Dish	2.55	2.50	2.53	0.00	0.00
Wt. Dish + Wet Soil	12.68	13.58	16.22	0.00	0.00
Wt. Dish + Dry Soil	11.02	11.66	13.79	0.00	0.00
Wt. Of Dry Soil	8.47	9.16	11.26	0.00	0.00
Wt. Of Water	1.66	1.92	2.43	0.00	0.00
% Moist	19.60	20.96	21.58		

PLASTIC LIMIT					
Dish No.					
Wt. of Dish	2.51	2.51	0.00	0.00	0.00
Wt. Dish + Wet Soil	8.21	6.92	0.00	0.00	0.00
Wt. Dish + Dry Soil	7.41	6.29	0.00	0.00	0.00
Wt. Of Dry Soil	4.90	3.78	0.00	0.00	0.00
Wt. Of Water	0.80	0.63	0.00	0.00	0.00
% Moist	16.33	16.67			



Project: Ash Basin #6 - Brunner Island
Boring No.: TB-C2
Sample No.: S-4
Depth: 6.0 - 8.0. ft

LL 20 %
PL 16 %
PI 4 %
w 6.6%

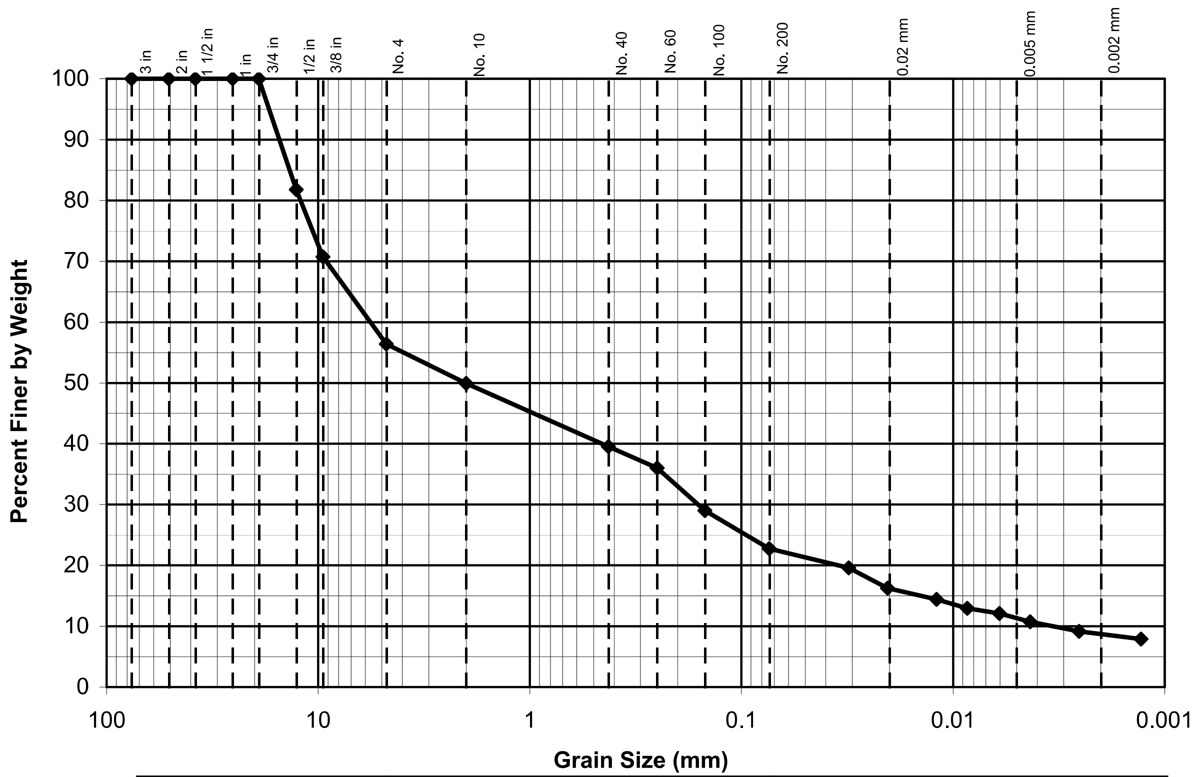
ATTERBERG LIMITS TEST DATA

AASHTO T-88, T-99 or ASTM 4318

GTS TECHNOLOGIES, INC.

10/7/2011

Grain Size Distribution Curve



GRAVEL		SAND			FINES	
COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
43.6 %		33.6 %			22.8 %	
0.0 %	43.6 %	6.5 %	10.3 %	16.8 %	11.5 %	11.3 %

USCS

Project: Ash Basin #6 - Brunner Island
Boring No.: TB-C2

Sample No.: S-3
Depth: 4.0 - 6.0 ft

Moisture Content: w = 6.7 %

GTS TECHNOLOGIES

GRADATION TEST RESULTS

AASHTO T-88
or ASTM 422
10/5/2011

AASHTO R18

GTS No. 11001-37 By: DFS Ckd: MCM

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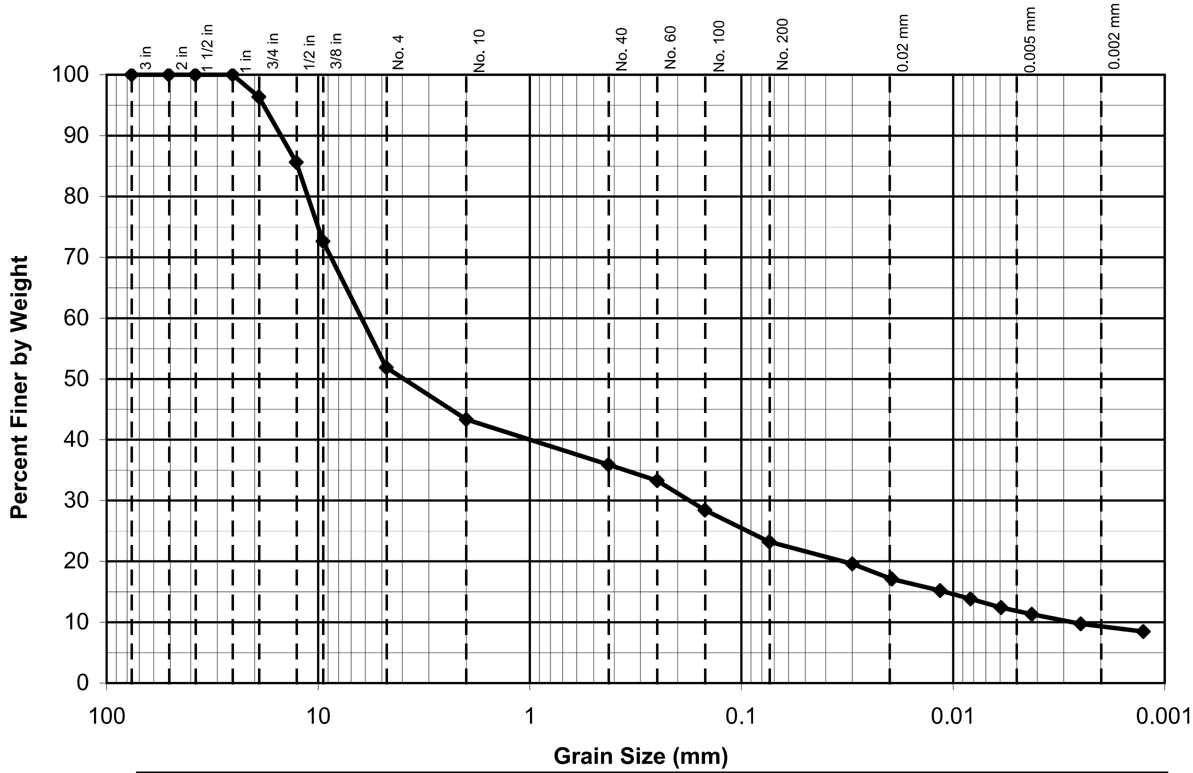
Grain Size Distribution Curve



GRAVEL		SAND			FINES	
COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
48.2 %		28.6 %			23.2 %	
3.6 %	44.5 %	8.5 %	7.4 %	12.7 %	11.4 %	11.8 %

USCS

Project: Ash Basin #6 - Brunner Island
Boring No.: TB-C2

Sample No.: B-3
Depth: 10.0 - 15.0 ft

Moisture Content: w = 6.0 %

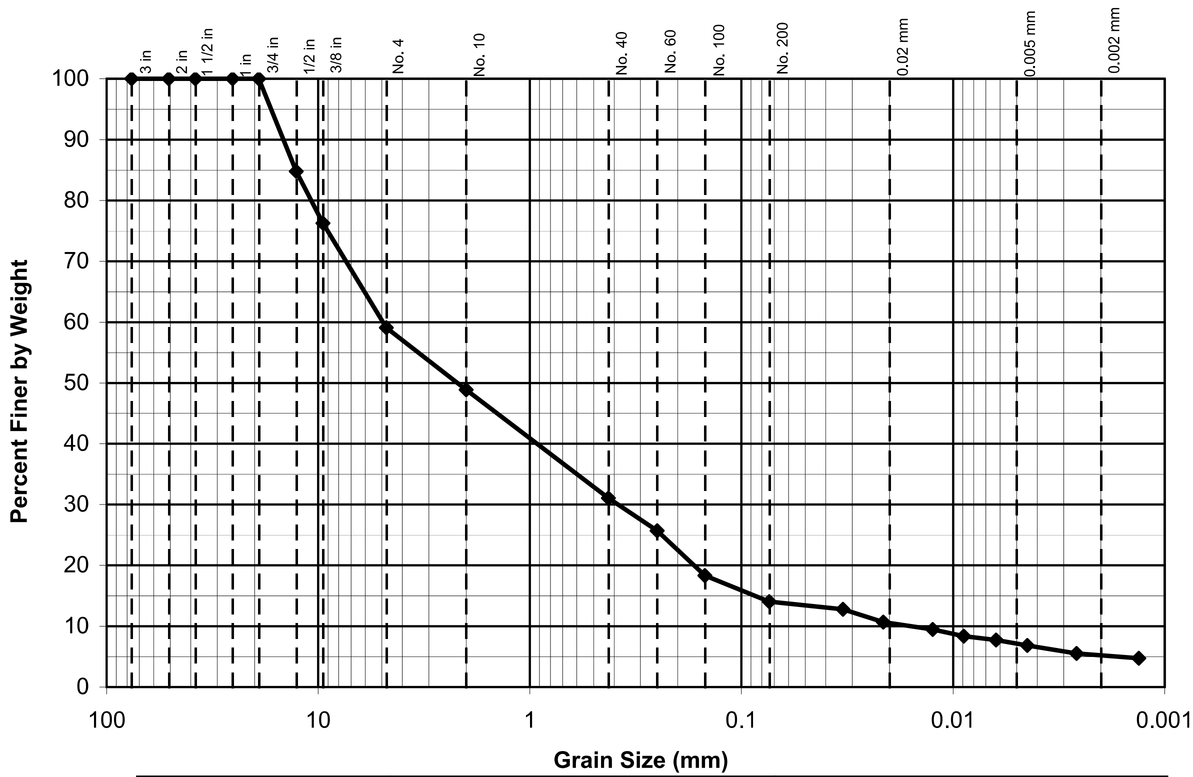
GTS TECHNOLOGIES

GRADATION TEST RESULTS
AASHTO T-88
or ASTM 422
10/6/2011

AASHTO R18

GTS No. 11001-37 By: DFS Ckd: MCM

Grain Size Distribution Curve



GRAVEL		SAND			FINES	
COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
40.9 %		45.0 %			14.0 %	
0.0 %	40.9 %	10.2 %	17.8 %	17.0 %	6.9 %	7.1 %

USCS

Project: Ash Basin #6 - Brunner Island
Boring No.: TB-C1

Sample No.: S-5
Depth: 8.0 - 10. ft
Moisture Content: w = 5.1 %

GTS TECHNOLOGIES

GRADATION TEST RESULTS

AASHTO T-88
or ASTM 422
10/5/2011

AASHTO R18

GTS No. 11001-37
By: DFS Ckd: MCM

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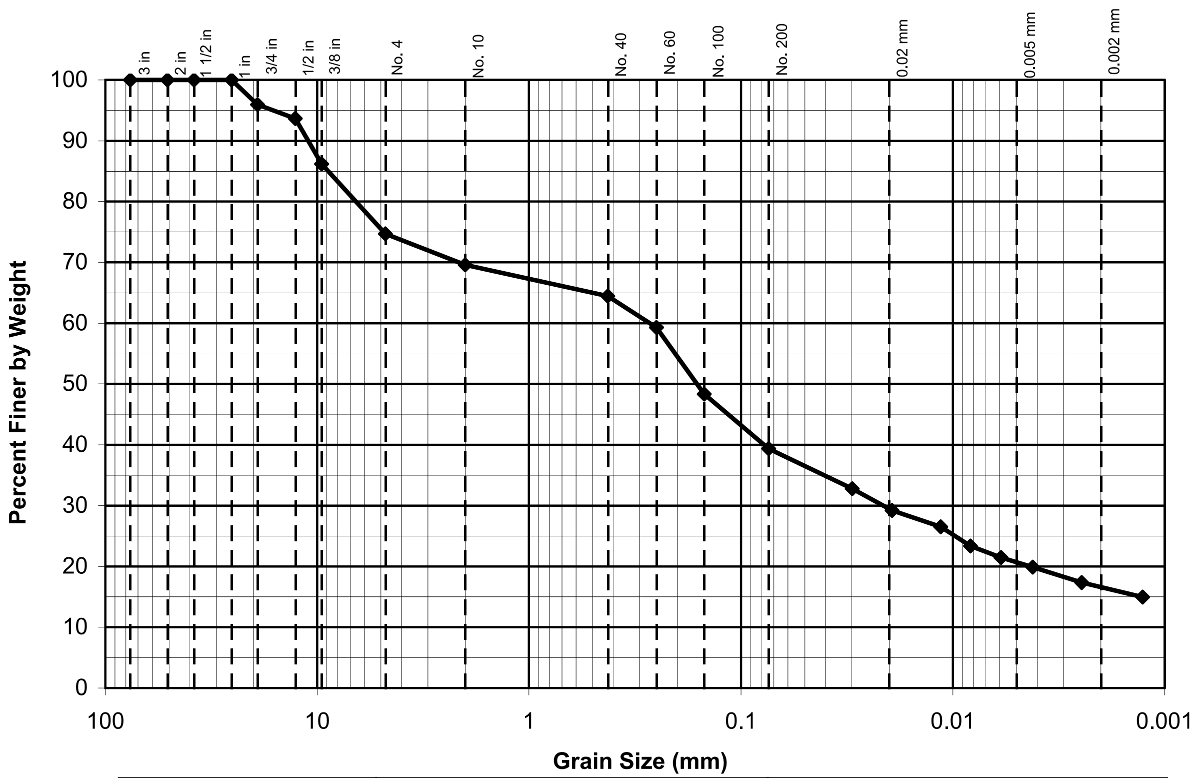
Grain Size Distribution Curve



GRAVEL		SAND			FINES	
COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
25.3 %		35.3 %			39.4 %	
4.1 %	21.3 %	5.1 %	5.1 %	25.1 %	18.8 %	20.6 %

USCS

Project:	Ash Basin #6 - Brunner Island	**Soil Type:**	clayey SAND with gravel
Boring No.:	TB-C1		
Station:		**Classification:**	SC, A-4 (0)
Offset:		LL = 23 %	PL = 15 %
Sample No.:	S-3	PI = 8 %	w = 14.7 %
Depth:	4.0 - 6.0 ft	**Spec. Grav.:**	2.65 (assumed)

ote: Minimum mass requirement was not met. Mass used for the test = 237.85 grams

GTS TECHNOLOGIES

CLASSIFICATION TEST RESULTS

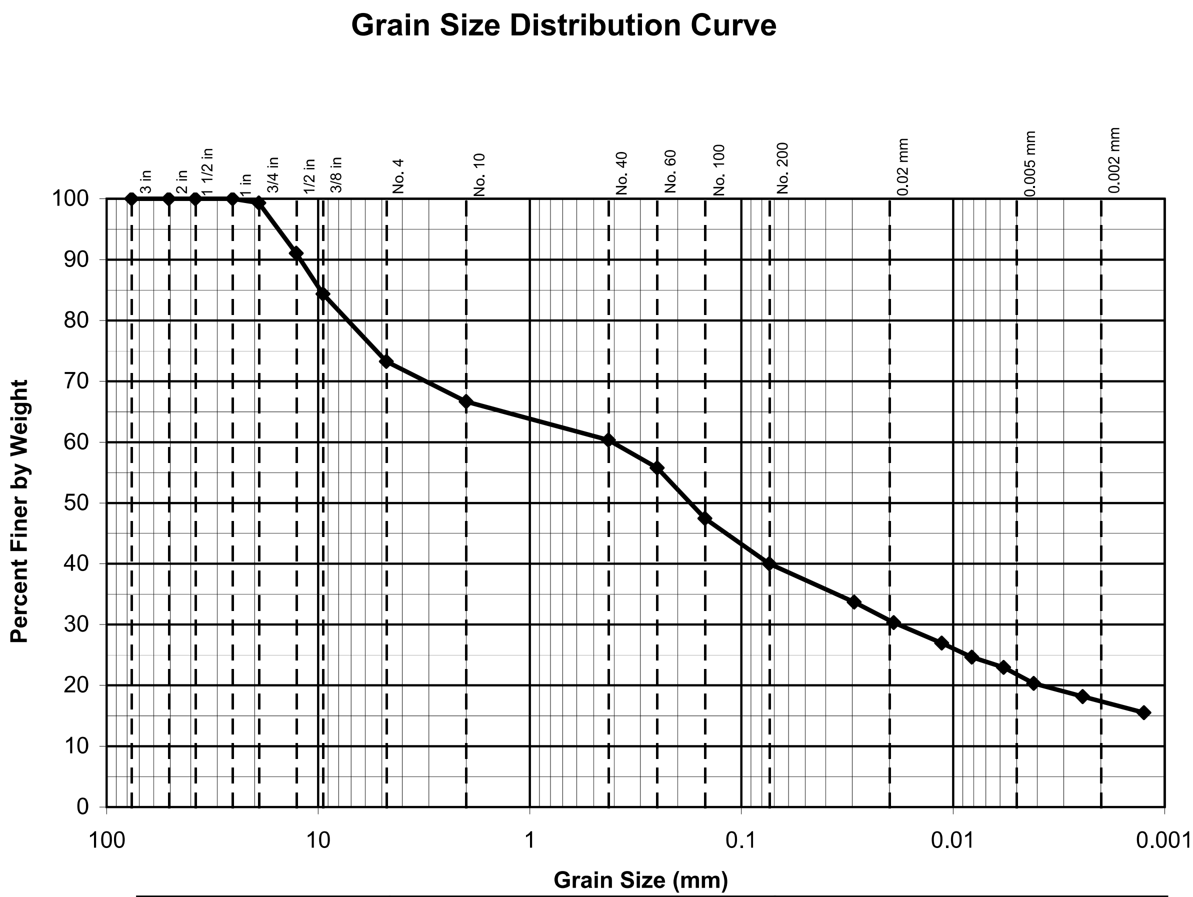
AASHTO T-88, T-89, T-90, M-145

or ASTM D 422, D 4318, D 2487

10/5/2011

AASHTO R18

GTS No. 11001-37 By: DFS Ckd: MCM



GRAVEL		SAND			FINES	
COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
26.8 %		33.2 %			40.0 %	
0.7 %	26.1 %	6.6 %	6.3 %	20.3 %	18.4 %	21.7 %

USCS

Project: Ash Basin #6 - Brunner Island
Boring No.: TB-C1

Sample No.: B-1
Depth: 0.0 - 5.0 ft **Moisture Content:** w = 10.6 %

GTS TECHNOLOGIES

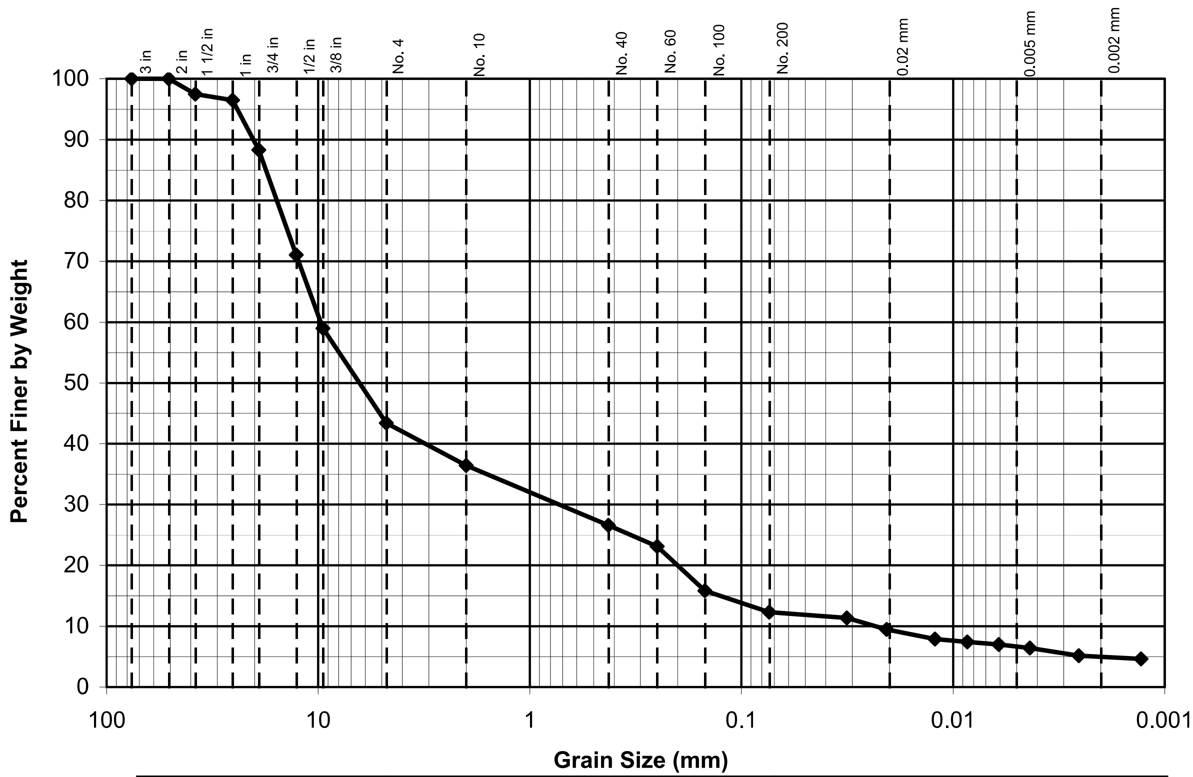
GRADATION TEST RESULTS

AASHTO T-88
or ASTM 422
10/5/2011

AASHTO R18

GTS No. 11001-37 By: DFS Ckd: MCM

Grain Size Distribution Curve



GRAVEL		SAND			FINES	
COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
56.6 %		31.1 %			12.3 %	
11.7 %	44.9 %	7.0 %	9.8 %	14.3 %	5.7 %	6.6 %

USCS

Project: Ash Basin #6 - Brunner Island
Boring No.: HA-E4

Sample No.: B-1
Depth: 1.5 - 2.5 ft
Moisture Content: w = 7.3 %

GTS TECHNOLOGIES

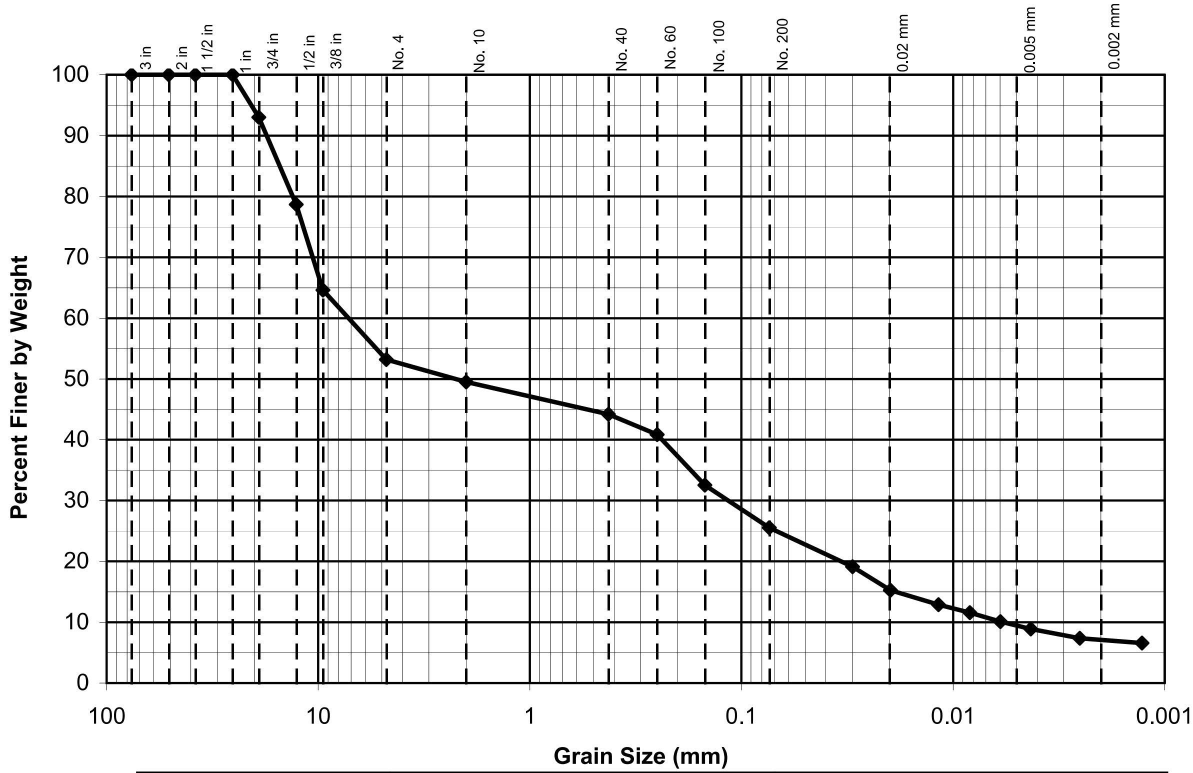
GRADATION TEST RESULTS
AASHTO T-88
or ASTM 422
10/7/2011

AASHTO R18

GTS No. 11001-37 By: DFS Ckd: MCM

441 Friendship Road . Harrisburg, PA 17111 . Ph: 717/236-3006 . Fax: 717/233-0994 . www.gtstech.com

Grain Size Distribution Curve



GRAVEL		SAND			FINES	
COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
46.8 %		27.6 %			25.6 %	
7.0 %	39.8 %	3.7 %	5.3 %	18.6 %	16.1 %	9.4 %

USCS

Project: Ash Basin #6 - Brunner Island
Boring No.: HA-E3

Sample No.: B-1
Depth: 1.5 - 2.5 ft

Moisture Content: w = 9.5 %

GTS TECHNOLOGIES

GRADATION TEST RESULTS

AASHTO T-88
or ASTM 422
10/7/2011

AASHTO R18

GTS No. 11001-37

By: DFS Ckd: MCM

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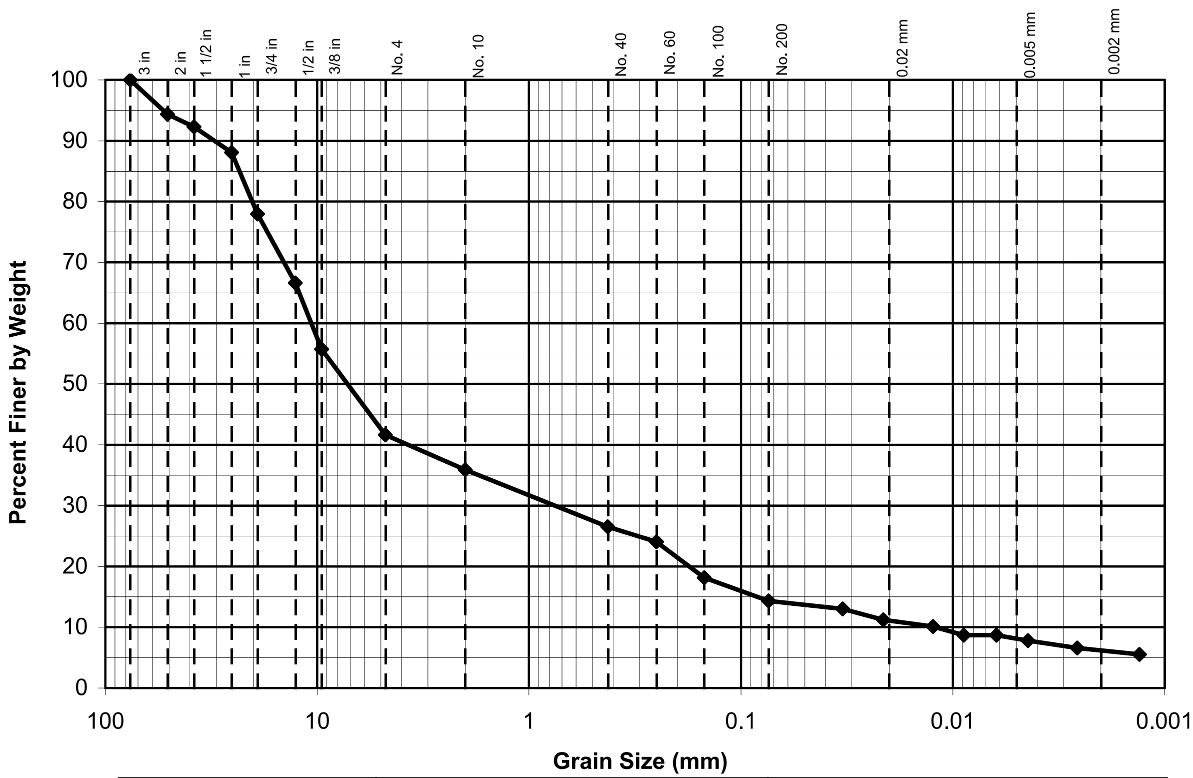
Grain Size Distribution Curve



GRAVEL		SAND			FINES	
COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
58.4 %		27.3 %			14.4 %	
22.0 %	36.3 %	5.8 %	9.4 %	12.1 %	6.2 %	8.1 %

USCS

Project:	Ash Basin #6 - Brunner Island	**Soil Type:**	silty GRAVEL with sand
Boring No.:	HA-E2		
Station:		**Classification:**	GM, A-1-a (0)
Offset:		LL = 19 %	PL = 17 %
Sample No.:	B-1	PI = 2 %	w = 7.2 %
Depth:	1.0 - 2.0 ft	**Spec. Grav.:**	2.65 (assumed)

ote: Minimum mass requirement was not met. Mass used for the test = 2849.54 grams

GTS TECHNOLOGIES

CLASSIFICATION TEST RESULTS

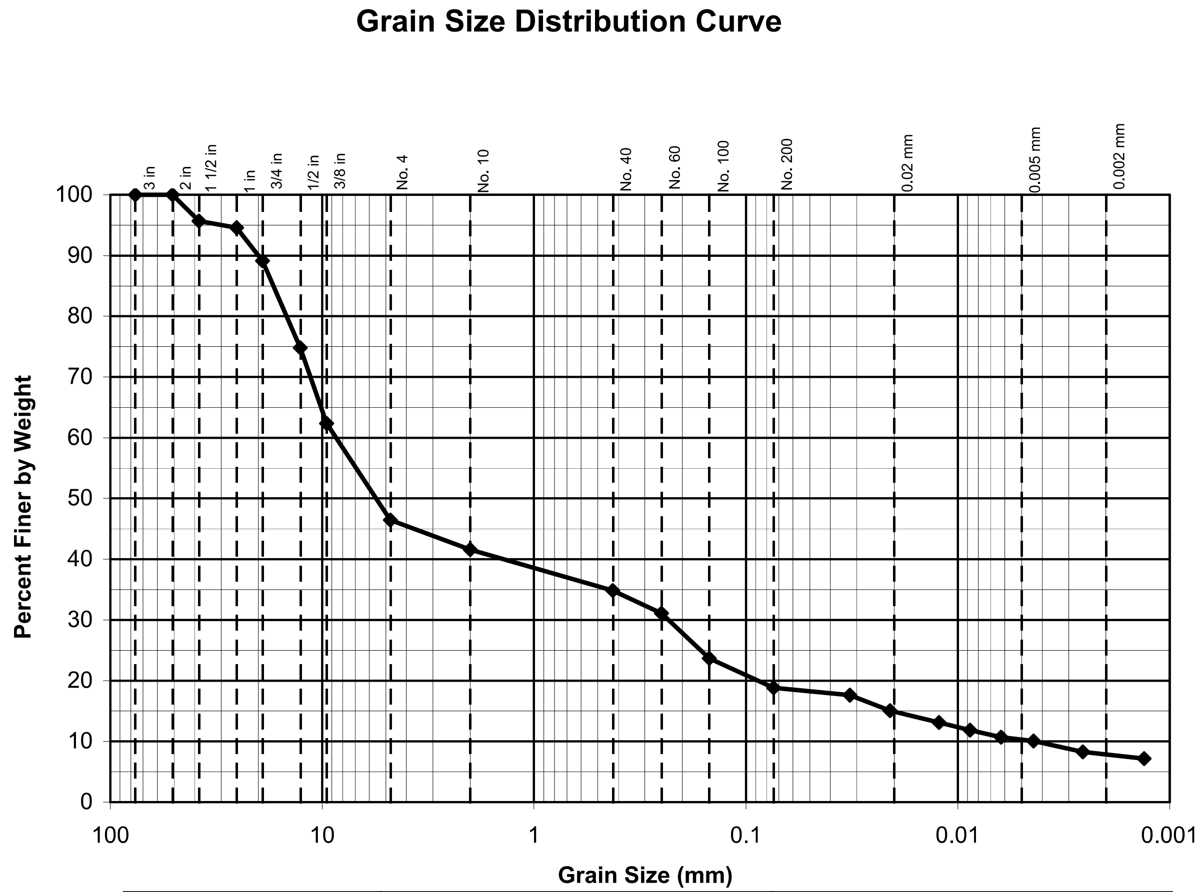
AASHTO T-88, T-89, T-90, M-145

or ASTM D 422, D 4318, D 2487

10/6/2011

AASHTO R18

GTS No. 11001-37 By: DFS Ckd: MCM



GRAVEL		SAND			FINES	
COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
53.5 %		27.6 %			18.8 %	
10.8 %	42.7 %	4.9 %	6.8 %	16.0 %	8.5 %	10.3 %

USCS

Project:	Ash Basin #6 - Brunner Island	**Soil Type:**	silty GRAVEL with sand
Boring No.:	HA-E1		
Station:		**Classification:**	GM, A-1-b (0)
Offset:		LL = 19 %	PL = 17 %
Sample No.:	B-1	PI = 2 %	w = 7.5 %
Depth:	1.0 - 2.0 ft	**Spec. Grav.:**	2.65 (assumed)

ote: Minimum mass requirement was not met. Mass used for the test = 2834.05 grams

GTS TECHNOLOGIES

CLASSIFICATION TEST RESULTS

AASHTO T-88, T-89, T-90, M-145
or ASTM D 422, D 4318, D 2487
10/6/2011

AASHTO R18

GTS No. 11001-37 By: DFS Ckd: MCM

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Schnabel ENGINEERING

Schnabel Engineering Consultants, Inc.
PPL Ash Basin Brunner Island
Transient Seepage and Slope Stability Study
Addendum No. 1 - Additional Project Data Acquisition
Laboratory Testing Assignments

Date: 10/17/2011
By: SAR

Test: Std Proctor (D698)

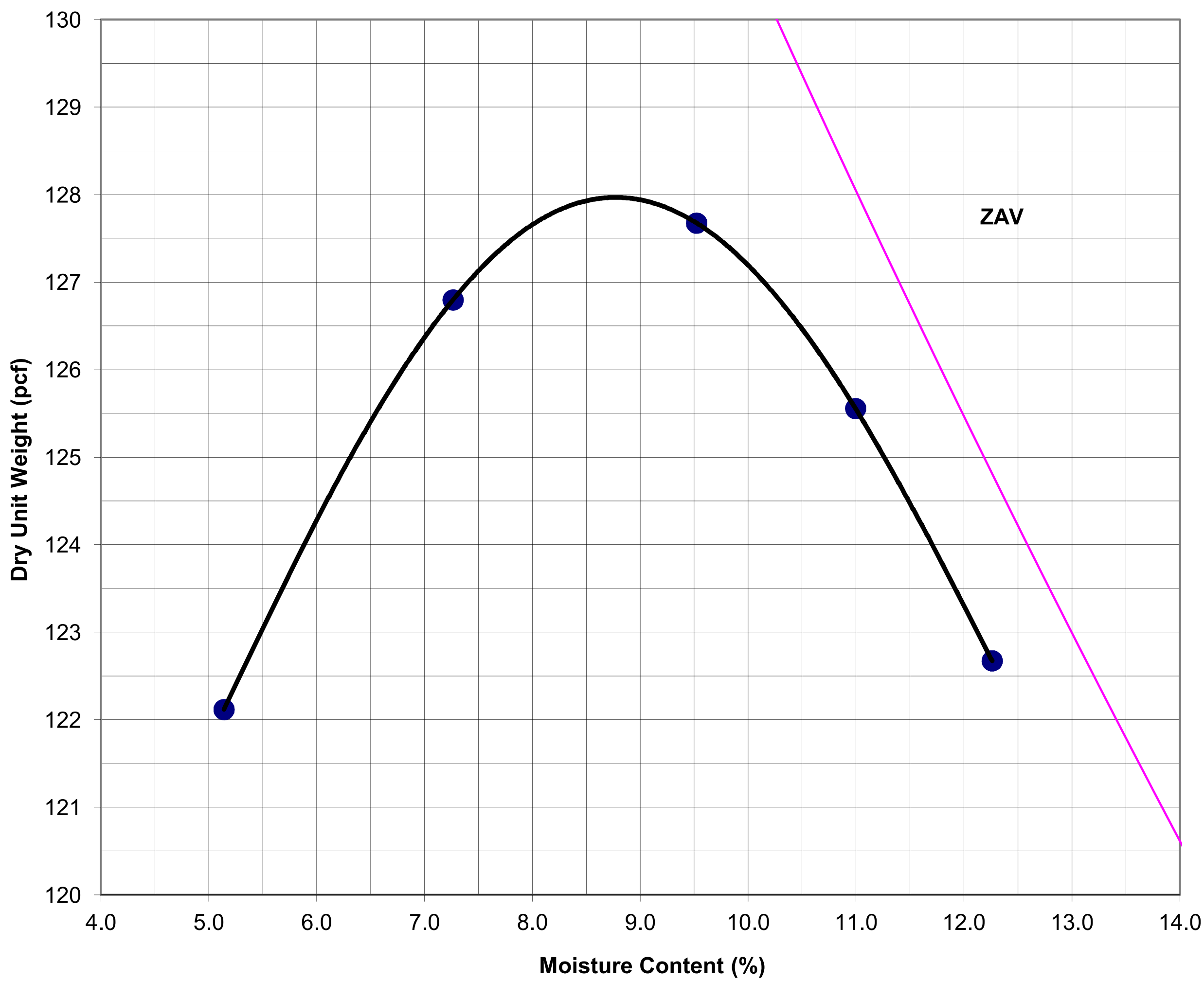
Depth (ft)	TB-C1		TB-C2		TB-C3		TB-C4		TB-C5	
0										
1	S-1		S-1		S-1		S-1		S-1	
2										
3	S-2	B-1	S-2	B-1	S-2	B-1	S-2	B-1	S-2	B-1
4		Proctor "B"								
5	S-3		S-3		S-3		S-3		S-3	
6										
7	S-4		S-4		S-4		S-4		S-4	
8		B-2		B-2		B-2		B-2		B-2
9	S-5		S-5		S-5		S-5		S-5	
10										
11	S-6		S-6		S-6		S-6		S-6	
12										
13	S-7	B-3	S-7	B-3	S-7	B-3	S-7	B-3	S-7	B-3
14				Proctor "C"						
15	S-8		S-8		S-8		S-8		S-8	
16										
17	S-9		S-9		S-9		S-9		S-9	
18		B-4		B-4		B-4		B-4		B-4
19	S-10		S-10		S-10		S-10		S-10	
20										

Depth (ft)	HA-E1	HA-E2	HA-E3	HA-E4
0				
1				
2	B-1	B-1	B-1	B-1
3		Proctor "A"		
4				
5				

S-1, etc (split spoon samples)
B-1, etc (bulk samples from auger cuttings)

T/ 610-696-6066 F/ 610-696-7771
1380 Wilmington Pike, Suite 100 / West Chester, PA / 19382

Compaction Curve



Project:	Brunner Island - Ash Basin #6		
Boring No.:	TB-C1		
Station:			
Offset:			
Sample No.:	B-1	**Max. Dry Density:**	128.0 pcf
Depth:	0.0 - 5.0 ft	**Opt. Moisture:**	8.8 %

GTS TECHNOLOGIES

STANDARD PROCTOR COMPACTION TEST RESULTS

AASHTO T-99 or ASTM D-698

10/25/2011

AASHTO R18

GTS No. 11001-37 By: DFS Ckd: dsc

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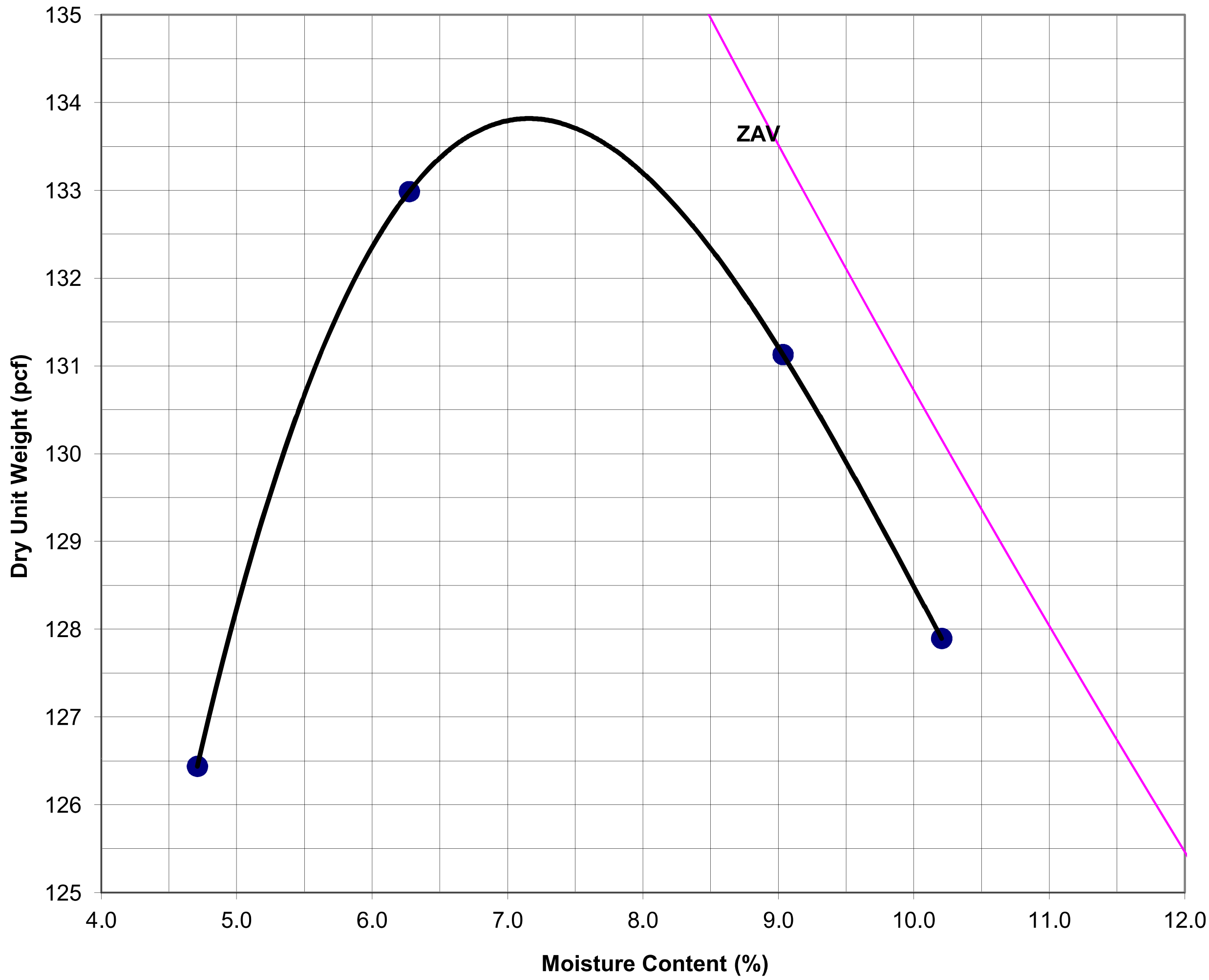
Compaction Curve



Project:	Brunner Island - Ash Basin #6		
Boring No.:	TB-C2		
Station:			
Offset:			
Sample No.:	B-3	**Max. Dry Density:**	133.8 pcf
Depth:	10.0 - 15.0 ft	**Opt. Moisture:**	7.2 %

GTS TECHNOLOGIES

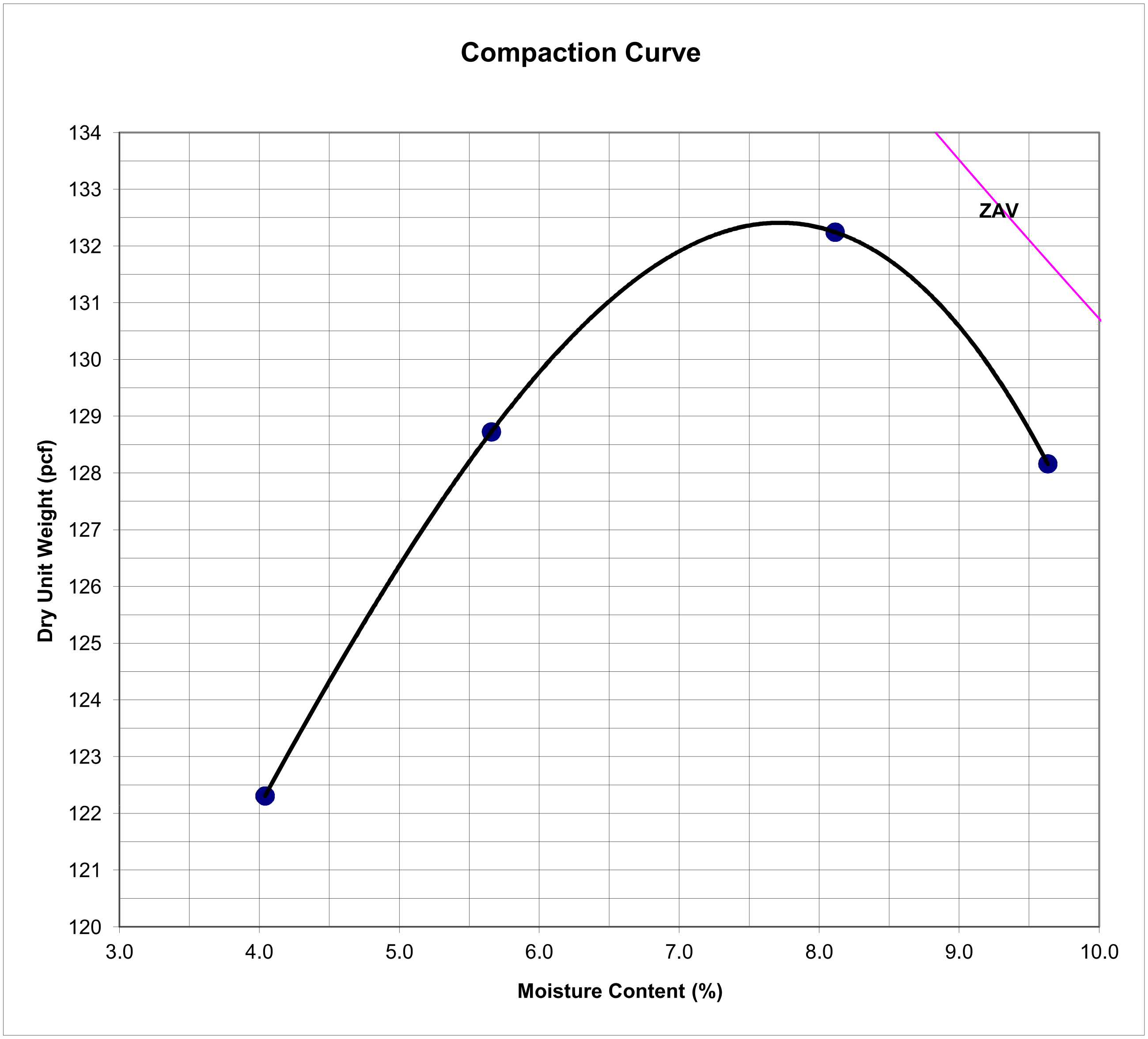
STANDARD PROCTOR COMPACTION TEST RESULTS

AASHTO T-99 or ASTM D-698

10/27/2011

AASHTO R18

GTS No. 11001-37 By: KJE Ckd: dsc



Project:	Brunner Island - Ash Basin #6		
Boring No.:	HA-E2		
Sample No.:	B-1	**Max. Dry Density:**	132.4 pcf
Depth:	1.0 - 3.0 ft	**Opt. Moisture:**	7.7 %

GTS TECHNOLOGIES

STANDARD PROCTOR COMPACTION TEST RESULTS

AASHTO T-99 or ASTM D-698

10/27/2011

AAP® AASHTO R18

GTS No. 11001-37

By: KJE Ckd: dsc

Schnabel ENGINEERING

Schnabel Engineering Consultants, Inc.
PPL Ash Basin Brunner Island
Transient Seepage and Slope Stability Study
Addendum No. 1 - Additional Project Data Acquisition
Laboratory Testing Assignments

Date: 10/17/2011
By: SAR

Test: Hydraulic Conductivity/Permeability (D2434/5084/5856)

Depth (ft)	TB-C1		TB-C2		TB-C3		TB-C4		TB-C5	
0										
1	S-1		S-1		S-1		S-1		S-1	
2										
3	S-2	B-1	S-2	B-1	S-2	B-1	S-2	B-1	S-2	B-1
4		Perm A								Perm E
5	S-3		S-3		S-3		S-3		S-3	
6										
7	S-4		S-4		S-4		S-4		S-4	
8		B-2		B-2		B-2		B-2		B-2
9	S-5		S-5		S-5	Perm C	S-5	Perm D	S-5	
10										
11	S-6		S-6		S-6		S-6		S-6	
12										
13	S-7	B-3	S-7	B-3	S-7	B-3	S-7	B-3	S-7	B-3
14				Perm B						
15	S-8		S-8		S-8		S-8		S-8	
16										
17	S-9		S-9		S-9		S-9		S-9	
18		B-4		B-4		B-4		B-4		B-4
19	S-10		S-10		S-10		S-10		S-10	
20										

S-1, etc (split spoon samples)
B-1, etc (bulk samples from auger cuttings)

Depth (ft)	HA-E1	HA-E2	HA-E3	HA-E4
0				
1				
2	B-1	B-1	B-1	B-1
3		Perm F	Perm G	
4				
5				

Table 1 - Permeability Testing

Test ID	Boring/HA Loc	Sample	Description
A	TB-C1	B-1	@ opt w/c; x % Relative Compaction (RC) based on Proctor B
B	TB-C2	B-3	@ opt w/c; x % Relative Compaction (RC) based on Proctor C
C	TB-C3	B-2	@ opt w/c; x % Relative Compaction (RC) based on Proctor B
D	TB-C4	B-2	@ opt w/c; x % Relative Compaction (RC) based on Proctor Y - **perform supplemental sieve (only - no hydromete**
E	TB-C5	B-1	@ opt w/c; x % Relative Compaction (RC) based on Proctor Y - **perform supplemental sieve (only - no hydromete**
F	HA-E2	B-1	w/c = 9.6%; dry unit weight = 107.5 pcf
G	HA-E3	B-1	w/c = 8.5%; dry unit weight = 117.5 pcf

Note D2434 not appropriate for these soil samples (all greater than 10% fines)

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Consultants in the Geosciences

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Fax (717) 245-9656
www.duffnet.com

STANDARD TEST METHOD FOR
MEASUREMENT OF HYDRAULIC CONDUCTIVITY OF SATURATED POROUS
MATERIALS USING A FLEXIBLE WALL PERMEAMETER

ASTM DESIGNATION: D 5084

Test Specimen Data

Sample Type:	Remold	**Unified Classification:**	
Water Content:	8.9 %	**Saturation:**	45.0 %
Dry Density:	108.9 pcf	**Diameter:**	4.00 in
Void Ratio:	.5186	**Height:**	4.584 in

Test Results

Consolidation Pressure:	10.00 psi	**Height:**	4.543 in
Cell Pressure:	65 psi	**Water Content:**	19.1 %
Back Pressure:		**Dry Density:**	109.9 pcf
At bottom of specimen:	59 psi	**Void Ratio:**	.5050
At top of specimen:	55 psi	**Saturation:**	100.0 %
Hydraulic Gradient:	23.4		

PERMEABILITY: 6.69 x 10^{-6} cm/sec

Sample No.: TB-C1 B-1

Sample Description: Brown Sandy Silt and Gravel

Source: TB-C1 B-1

Remarks: Sample compacted to 85.0% Standard Proctor Density at a moisture content of 8.9%

Project No.: 9339.ZA
Brunner Island - Ash Basin No.6
York County, PA
November 17, 2011

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STANDARD TEST METHOD FOR MEASUREMENT OF HYDRAULIC CONDUCTIVITY OF SATURATED POROUS MATERIALS USING A FLEXIBLE WALL PERMEAMETER

ASTM DESIGNATION: D 5084

Test Specimen Data

Sample Type:	Remold	**Unified Classification:**	
Water Content:	7.3 %	**Saturation:**	42.6 %
Dry Density:	113.7 pcf	**Diameter:**	4.00 in
Void Ratio:	.4545	**Height:**	4.584 in

Test Results

Consolidation Pressure:	10.00 psi	**Height:**	4.535 in
Cell Pressure:	65 psi	**Water Content:**	16.6 %
Back Pressure:		**Dry Density:**	114.9 pcf
At bottom of specimen:	59 psi	**Void Ratio:**	.4389
At top of specimen:	55 psi	**Saturation:**	100.0 %
Hydraulic Gradient:	23.4		

PERMEABILITY: 6.98 x 10^{-5} cm/sec

Sample No.: TB-C2 B-3

Sample Description: Brown Silty Sand and Gravel

Source: TB-C2 B-3

Remarks: Sample compacted to 85.0% Standard Proctor Density at a moisture content of 7.3%

Project No.: 9339.ZA
Brunner Island - Ash Basin No.6
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STANDARD TEST METHOD FOR MEASUREMENT OF HYDRAULIC CONDUCTIVITY OF SATURATED POROUS MATERIALS USING A FLEXIBLE WALL PERMEAMETER

ASTM DESIGNATION: D 5084

Test Specimen Data

Sample Type:	Remold	Unified Classification:	
Water Content:	8.8 %	Saturation:	45.0 %
Dry Density:	108.9 pcf	Diameter:	4.00 in
Void Ratio:	.5186	Height:	4.584 in

Test Results

Consolidation Pressure:	10.00 psi	Height:	4.548 in
Cell Pressure:	65 psi	Water Content:	18.9 %
Back Pressure:		Dry Density:	110.1 pcf
At bottom of specimen:	59 psi	Void Ratio:	.5015
At top of specimen:	55 psi	Saturation:	100.0 %
Hydraulic Gradient:	22.0		

PERMEABILITY: 1.87 x 10^{-4} cm/sec

Sample No.: TB-C3 B-2

Sample Description: Brown Silty Sand and Gravel

Source: TB-C3 B-2

Remarks: Sample compacted to 85.0% Standard Proctor Density at a moisture content of 8.8%

Project No.: 9339.ZA
Brunner Island - Ash Basin No.6
York County, PA
November 18, 2011

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STANDARD TEST METHOD FOR MEASUREMENT OF HYDRAULIC CONDUCTIVITY OF SATURATED POROUS MATERIALS USING A FLEXIBLE WALL PERMEAMETER

ASTM DESIGNATION: D 5084

Test Specimen Data

Sample Type:	Remold	**Unified Classification:**	
Water Content:	7.1 %	**Saturation:**	41.4 %
Dry Density:	113.7 pcf	**Diameter:**	4.00 in
Void Ratio:	.4545	**Height:**	4.584 in

Test Results

Consolidation Pressure:	10.00 psi	**Height:**	4.551 in
Cell Pressure:	65 psi	**Water Content:**	16.8 %
Back Pressure:		**Dry Density:**	114.5 pcf
At bottom of specimen:	59 psi	**Void Ratio:**	.4440
At top of specimen:	55 psi	**Saturation:**	100.0 %
Hydraulic Gradient:	23.4		

PERMEABILITY: 5.68 x 10^{-6} cm/sec

Sample No.: TB-C4 B-2

Sample Description: Brown Sandy Silt and Gravel

Source: TB-C4 B-2

Remarks: Sample compacted to 85.0% Standard Proctor Density at a moisture content of 7.1%

Project No.: 9339.ZA
Brunner Island - Ash Basin No.6
York County, PA
November 21, 2011

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STANDARD TEST METHOD FOR MEASUREMENT OF HYDRAULIC CONDUCTIVITY OF SATURATED POROUS MATERIALS USING A FLEXIBLE WALL PERMEAMETER

ASTM DESIGNATION: D 5084

Test Specimen Data

Sample Type:	Remold	Unified Classification:	
Water Content:	9.1 %	Saturation:	46.5 %
Dry Density:	108.9 pcf	Diameter:	4.00 in
Void Ratio:	.5186	Height:	4.584 in

Test Results

Consolidation Pressure:	10.00 psi	Height:	4.548 in
Cell Pressure:	65 psi	Water Content:	19.1 %
Back Pressure:		Dry Density:	109.8 pcf
At bottom of specimen:	59 psi	Void Ratio:	.5066
At top of specimen:	55 psi	Saturation:	100.0 %
Hydraulic Gradient:	22.5		

PERMEABILITY: 8.11 x 10^{-5} cm/sec

Sample No.: TB-C5 B-1

Sample Description: Brown Silty Sand and Gravel

Source: TB-C5 B-1

Remarks: Sample compacted to 85.0% Standard Proctor Density at a moisture content of 9.1%

Project No.: 9339.ZA
Brunner Island - Ash Basin No.6
York County, PA
November 17, 2011

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STANDARD TEST METHOD FOR
MEASUREMENT OF HYDRAULIC CONDUCTIVITY OF SATURATED POROUS
MATERIALS USING A FLEXIBLE WALL PERMEAMETER

ASTM DESIGNATION: D 5084

Test Specimen Data

Sample Type:	Remold	Unified Classification:	
Water Content:	8.5 %	Saturation:	97.5 %
Dry Density:	117.5 pcf	Diameter:	4.00 in
Void Ratio:	.4062	Height:	4.584 in

Test Results

Consolidation Pressure:	1.44 ksf	Height:	4.551 in
Cell Pressure:	65 psi	Water Content:	14.9 %
Back Pressure:		Dry Density:	118.4 pcf
At bottom of specimen:	59 psi	Void Ratio:	.3961
At top of specimen:	55 psi	Saturation:	100.0 %
Hydraulic Gradient:	24.4		

PERMEABILITY: 7.06 x 10^{-7} cm/sec

Sample No.: HA E3 B-1

Sample Description: Brown Silty Sand

Source: HA E3 B-1

Remarks: Sample compacted to 117.5 pcf Dry Density at a moisture content of 8.5%.

Project No.: 9339.ZA
Brunner Island - Ash Basin No. 6
York County, Pennsylvania
December 2, 2011

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STANDARD TEST METHOD FOR MEASUREMENT OF HYDRAULIC CONDUCTIVITY OF SATURATED POROUS MATERIALS USING A FLEXIBLE WALL PERMEAMETER

ASTM DESIGNATION: D 5084

Test Specimen Data

Sample Type:	Remold	Unified Classification:	
Water Content:	9.6 %	Saturation:	98.5 %
Dry Density:	107.5 pcf	Diameter:	4.00 in
Void Ratio:	.5398	Height:	4.584 in

Test Results

Consolidation Pressure:	1.44 ksf	Height:	4.560 in
Cell Pressure:	65 psi	Water Content:	20.1 %
Back Pressure:		Dry Density:	107.9 pcf
At bottom of specimen:	59 psi	Void Ratio:	.5317
At top of specimen:	55 psi	Saturation:	100.0 %
Hydraulic Gradient:	23.4		

PERMEABILITY: 2.06×10^{-7} cm/sec

Sample No.: HA E2 B-1

Sample Description: Brown Sandy Silt

Source: HA E2 B-1

Remarks: Sample compacted to 107.5 pcf Dry Density at a moisture content of 9.6%.

Project No.: 9339.ZA
Brunner Island - Ash Basin No. 6
York County, Pennsylvania
December 2, 2011

APPENDIX B

SUMMARY OF SATURATED HYDRAULIC CONDUCTIVITY DATA

Schnabel Project No. 11615019
Brunner Island AB No. 6 Transient Seepage Analysis
Saturated Hydraulic Conductivity Evaluation

Table 1 - In-Situ (Field) Hydraulic Conductivity Values from Measured Infiltration Rates

Location	Sample No.	Depth (ft bgs)	Test Method	Infiltration Rate (in/hr)	Infiltration Rate (cm/sec)	Infiltration Rate (sec/m)	Sat. Hyd Cond, K (m/sec)	Sat. Hyd Cond, K (ft/sec)	Sat. Hyd Cond, K (cm/sec)
TB-C1	n/a	8	Case-Pipe	1.08	7.62E-04	1.31E+05	1.33E-06	4.35E-06	1.33E-04
TB-C2	n/a	5	Case-Pipe	0.6	4.23E-04	2.36E+05	1.11E-06	3.63E-06	1.11E-04
TB-C3	n/a	8	Case-Pipe	4.68	3.30E-03	3.03E+04	2.09E-06	6.84E-06	2.09E-04
TB-C4	n/a	4	Case-Pipe	0.36	2.54E-04	3.94E+05	9.44E-07	3.10E-06	9.44E-05
TB-C5	n/a	4.5	Case-Pipe	(1)					
HA-E1	n/a	2	Double Ring	0.2	1.41E-04	7.09E+05	7.87E-07	2.58E-06	7.87E-05
HA-E2	n/a	2	Double Ring	0.84	5.93E-04	1.69E+05	1.23E-06	4.03E-06	1.23E-04
HA-E3	n/a	2.25	Double Ring	0.31	2.19E-04	4.57E+05	9.02E-07	2.96E-06	9.02E-05
HA-E4	n/a	2.5	Double Ring	0.25	1.76E-04	5.67E+05	8.44E-07	2.77E-06	8.44E-05
MAX								***6.84E-06***	***2.09E-04***
AVG								***3.78E-06***	***1.15E-04***
MIN								***2.58E-06***	***7.87E-05***

(1) Not measureable

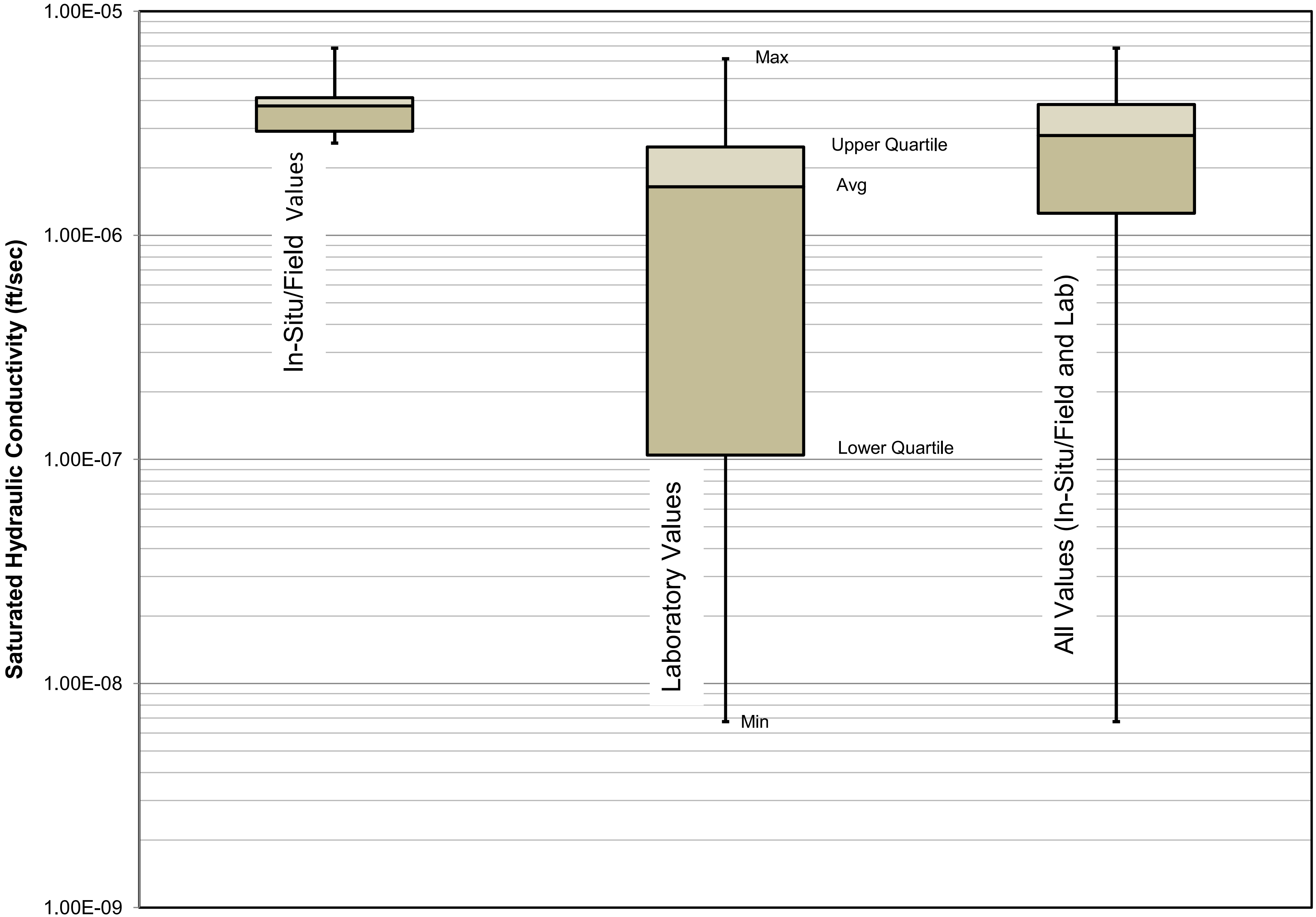
Table 2 - Laboratory Hydraulic Conductivity Measured Values

					Initial Specimen					Final Specimen					Sat.	Sat.
	Sample No.	Depth (ft bgs)	USCS	% Passing # 200	Saturation, S (%)	Moist. Content, m (%)	Dry Unit Wt, γ_d (pcf)	Moist Unit Wt, γ_w (pcf)	Void Ratio, e	Saturation, S (%)	Moist. Content, m (%)	Dry Unit Wt, γ_d (pcf)	Moist (Sat) Unit Wt, γ_w (pcf)	Void Ratio, e	Hyd Cond, K (ft/sec)	Hyd Cond, K (cm/sec)
HA-E2	B-1	0-5	GM	14.4	98.5	9.6	107.5	117.8	0.540	100	20.1	107.9	129.6	0.532	6.76E-09	2.06E-07
HA-E3	B-1	0-5	GM/C	25.6	97.5	8.5	117.5	127.5	0.406	100	14.9	118.4	136.0	0.396	2.32E-08	7.06E-07
TB-C1	B-1	0-5	SM/C	40.0	45.0	8.9	108.9	118.6	0.519	100	19.1	109.9	130.9	0.505	2.19E-07	6.69E-06
TB-C2	B-3	10-15	GM/C	23.2	42.6	7.3	113.7	122.0	0.455	100	16.6	114.9	134.0	0.439	2.29E-06	6.98E-05
TB-C3	B-2	5-10	SM/C	36.2	45.0	8.8	108.9	118.5	0.519	100	18.9	110.1	130.9	0.502	6.14E-06	1.87E-04
TB-C4	B-2	5-10	GM/C	23.9	41.4	7.1	113.7	121.8	0.455	100	16.8	114.5	133.7	0.444	1.86E-07	5.68E-06
TB-C5	B-1	0-5	SM/C	33.1	46.5	9.1	108.9	118.8	0.519	100	19.1	109.8	130.8	0.507	2.66E-06	8.11E-05
MAX				***40.0***	***98.5***	***9.6***	***117.5***	***127.5***	***0.540***	***100.0***	***20.1***	***118.4***	***136.0***	***0.532***	***6.14E-06***	***1.87E-04***
AVG				***28.1***	***59.5***	***8.5***	***111.3***	***120.7***	***0.487***	***100.0***	***17.9***	***112.2***	***132.3***	***0.475***	***1.65E-06***	***5.02E-05***
MIN				***14.4***	***41.4***	***7.1***	***107.5***	***117.8***	***0.406***	***100.0***	***14.9***	***107.9***	***129.6***	***0.396***	***6.76E-09***	***2.06E-07***

ALL DATA:

MAX		***6.84E-06***	***2.09E-04***
AVG		***2.79E-06***	***8.49E-05***
MIN		***6.76E-09***	***2.06E-07***

Figure 1 - Box Plot Showing Saturated Hydraulic Conductivity Values



APPENDIX C

SUMMARY OF CLIMATIC, METEORLOGICAL, AND HYDRAULIC CONDUCTIVITY DATA

Hydraulic Analysis of Flood Flows at Brunner Island

Purpose:
Define peak flow frequency curve
Define typical times for rise, high stage, and fall of hydrographs during major floods
Develop precipitation-frequency-duration data

Brunner Island is on the Susquehanna River between two USGS stream gages:

USGS 01570500 Susquehanna River at Harrisburg, PA
LOCATION.--Lat 40`15'17", long 76`53'11", Dauphin County, Hydrologic Unit 02050305, on east bank of City Island, 60 ft downstream from Market Street bridge in Harrisburg, 3,670 ft upstream from sanitary dam, and 1.7 mi upstream from Paxton Creek.
DRAINAGE AREA.--24,100 mi2.
PERIOD OF RECORD.--October 1890 to current year.
GAGE.--Water-stage recorder. Concrete control since Aug. 29, 1916. Datum of gage is 290.01 ft above National Geodetic Vertical Datum of 1929. Prior to Oct. 1, 1928, nonrecording gage at Walnut Street Bridge 600 ft upstream, and Oct. 1, 1928, to Aug. 31, 1975, water-stage recorder at site 3,170 ft downstream, all gages at same datum.
EXTREMES OUTSIDE PERIOD OF RECORD.--Maximum stage known during period 1786 to 1890, 26.8 ft at Walnut Street bridge, June 2, 1889, discharge, 654,000 ft3/s.

USGS 01576000 Susquehanna River at Marietta, PA
LOCATION.--Lat 40`03'16", long 76`31'52", Lancaster County, Hydrologic Unit 02050306, on left bank 420 ft upstream from Chickies Creek, and 1.0 mi downstream from Marietta. Records include flow of Chickies Creek.
DRAINAGE AREA.--25,990 mi2, approximately, includes that of Chickies Creek.
PERIOD OF RECORD.--October 1931 to current year.
GAGE.--Water-stage recorder. Datum of gage is 200.56 ft above sea level.
EXTREMES OUTSIDE PERIOD OF RECORD.--Flood of June 2, 1889, reached a stage of 58.2 ft, from floodmark, discharge, about 630,000 ft3/s.

The site is closer to the Marietta gage and there are no dams between the site and this gage; therefore, the Marietta gage will be used for the analysis.

Peak Flow Frequency Curve

Using the USGS Program PeakFQ, Annual Flood-Frequency Analysis Using Bulletin 17B Guidelines, the following peak flow frequency curve was developed. It was assumed that the 1889 peak flow was an historic peak (but not necessarily the historic peak, as it was recorded by a means prior to establishment of the stream gage in 1932) for the historic period of 111 years, from 1889 to 2010.

The results are summarized below and compared with the peak flows contained in the Lancaster Flood Insurance Study. Because the results compare relatively well, the peak elevations shown in the IFS will be used.

ANNUAL EXCEEDANCE PROBABILITY	Percent Chance	Return period	BULL.17B ESTIMATE	Lancaster County FIS	
				Peak Flow	Peak Elevation
			(cfs)	(cfs)	(ft, NAVD 88)
0.995			123,800		
0.99			130,900		
0.95			155,400		
0.9			172,300		
0.8			197,600		
0.6667			227,300		
0.5	50	2-yr	266,800		
0.4292			286,200		
0.2	20	5-yr	379,700		
0.1	10	10-yr	466,700	420,000	270.7
0.04	4	25-yr	591,800		
0.02	2	50-yr	696,500	615,000	276.8
0.01	1	100-yr	811,900	725,000	279.2
0.005	0.5	200-yr	939,300		
0.002	0.2	500-yr	1,129,000	1,100,000	288.8

As shown in the attached peak flow frequency analysis results, the peak of record occurred during Hurricane Agnus in June 1972. The only other storm t exceed the 2 percent chance event occurred in 1936.While causing significant damage elsewhere, Hurricane Diane in October 1955 was less than a 50-percent-chance event on the Susquehanna in this area.

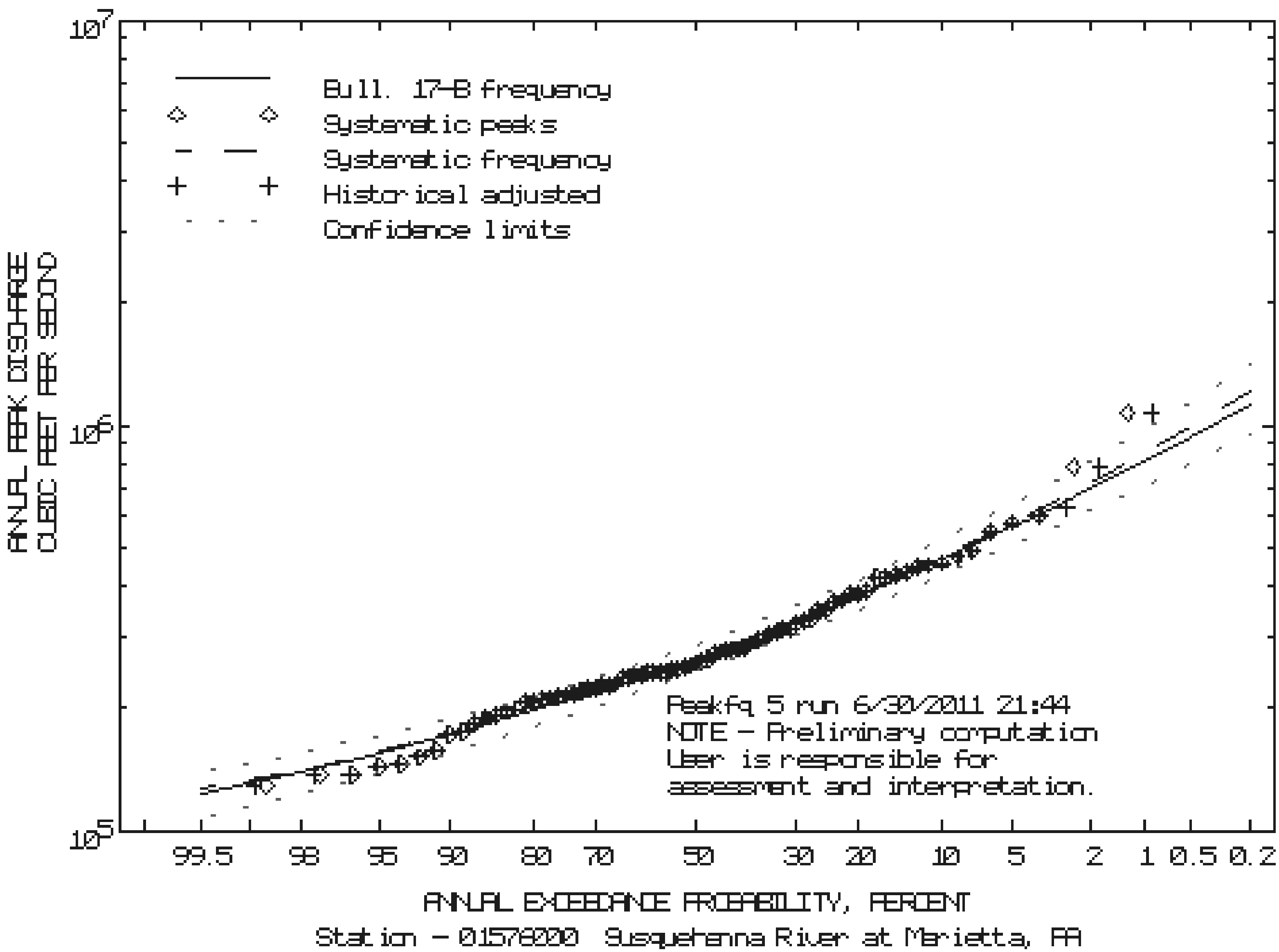
Typical Times for Rise, High Stage, And Fall of Hydrographs During Major Floods

Daily flows for the Marietta gage were observed for the major flood events.

It was found that, for the record storm, Hurricane Agnes in June 1972, the period of rise to the 2 percent chance (50-year) event was about 2 days. The period of high stage above the 2 percent chance event was about 3 days. Using the 50 percent chance (2-year) peak flow to identify the end of the flood event, the period of fall was about 2 days.

Precipitation-Frequency-Duration Data

The attached table shows the results of the precipitation frequency data, developed using NOAA’s Atlas 14.



Bull. 17-B frequency
Systematic peaks
Systematic frequency
Historical adjusted
Confidence limits
ANNUAL PEAK DISCHARGE
CUBIC FEET PER SECOND
10^7
10^6
10^5
99.5
98
95
90
80
70
50
30
20
10
5
2
1
0.5
0.2
Peakfq 5 run 6/30/2011 21:44
NOTE - Preliminary computation
User is responsible for
assessment and interpretation.
ANNUAL EXCEEDANCE PROBABILITY, PERCENT
Station - 01576000 Susquehanna River at Marietta, PA

```
1
  Program PeakFq           U. S. GEOLOGICAL SURVEY              Seq.000.000
  Ver. 5.2            Annual peak flow frequency analysis      Run Date / Time
  11/01/2007          following Bulletin 17-B Guidelines      06/30/2011 21:13

                         --- PROCESSING OPTIONS ---

                      Plot option          = None
                      Basin char output    = None
                      Print option         = Yes
                      Debug print          = No
                      Input peaks listing = Long
                      Input peaks format  = WATSTORE peak file

                      Input files used:
                         peaks (ascii)  -
G:\2011-SEC-JOBS\11615019_00-ASH_BASIN_6_SLOPE_STABILITY\DATA\1576000.TXT
                         specifications - PKFQWPSF.TMP

                      Output file(s):
                         main -
G:\2011-SEC-JOBS\11615019_00-ASH_BASIN_6_SLOPE_STABILITY\DATA\1576000.PRT

1


  Program PeakFq           U. S. GEOLOGICAL SURVEY              Seq.001.001
  Ver. 5.2            Annual peak flow frequency analysis      Run Date / Time
  11/01/2007          following Bulletin 17-B Guidelines      06/30/2011 21:13

             Station - 01576000  Susquehanna River at Marietta, PA


                       I N P U T   D A T A   S U M M A R Y

                  Number of peaks in record           =       80
                  Peaks not used in analysis          =        0
                  Systematic peaks in analysis        =       79
                  Historic peaks in analysis          =        1
                  Years of historic record            =      111
                  Generalized skew                    =    0.560
                       Standard error                 =    0.550
                       Mean Square error              =    0.303
                  Skew option                         =   WEIGHTED
                  Gage base discharge                 =      0.0
                  User supplied high outlier threshold =   --
                  User supplied low outlier criterion  =   --
                  Plotting position parameter         =     0.00


  *********  NOTICE  --  Preliminary machine computations.        *********
  *********  User responsible for assessment and interpretation.  *********

    WCF134I-NO SYSTEMATIC PEAKS WERE BELOW GAGE BASE.                    0.0
    WCF156I-17B HI-OUTLIER TEST SUPERSEDED BY MIN HIST PK  898061.8
    WCF165I-HIGH OUTLIERS AND HISTORIC PEAKS ABOVE HHBASE.  2  1   630000.3
    WCF195I-NO LOW OUTLIERS WERE DETECTED BELOW CRITERION.         83347.2
1


  Program PeakFq           U. S. GEOLOGICAL SURVEY              Seq.001.002
  Ver. 5.2            Annual peak flow frequency analysis      Run Date / Time
```

11/01/2007 following Bulletin 17-B Guidelines 06/30/2011 21:13

Station - 01576000 Susquehanna River at Marietta, PA

ANNUAL FREQUENCY CURVE PARAMETERS -- LOG-PEARSON TYPE III

	FLOOD BASE		LOGARITHMIC		
	DISCHARGE	EXCEEDANCE PROBABILITY	MEAN	STANDARD DEVIATION	SKEW
SYSTEMATIC RECORD	0.0	1.0000	5.4428	0.1739	0.672
BULL.17B ESTIMATE	0.0	1.0000	5.4423	0.1709	0.569

ANNUAL FREQUENCY CURVE -- DISCHARGES AT SELECTED EXCEEDANCE PROBABILITIES

ANNUAL EXCEEDANCE PROBABILITY	BULL.17B ESTIMATE	SYSTEMATIC RECORD	'EXPECTED PROBABILITY' ESTIMATE	95-PCT CONFIDENCE LIMITS FOR BULL. 17B ESTIMATES LOWER	UPPER
0.9950	123800.0	126900.0	121700.0	107100.0	139100.0
0.9900	130900.0	133400.0	129000.0	114100.0	146300.0
0.9500	155400.0	156200.0	154100.0	138200.0	171100.0
0.9000	172300.0	172300.0	171300.0	155000.0	188300.0
0.8000	197600.0	196700.0	197000.0	180200.0	214100.0
0.6667	227300.0	225800.0	227000.0	209500.0	244900.0
0.5000	266800.0	265100.0	266800.0	247700.0	287000.0
0.4292	286200.0	284700.0	286400.0	266000.0	308200.0
0.2000	379700.0	380800.0	381300.0	350700.0	415600.0
0.1000	466700.0	472600.0	471100.0	425600.0	521400.0
0.0400	591800.0	607500.0	602700.0	529000.0	680000.0
0.0200	696500.0	723100.0	715500.0	613100.0	817500.0
0.0100	811900.0	852500.0	842500.0	703800.0	972800.0
0.0050	939300.0	997800.0	986300.0	802100.0	1148000.0
0.0020	1129000.0	1218000.0	1207000.0	945300.0	1416000.0

1

Program PeakFq U. S. GEOLOGICAL SURVEY Seq.001.003
Ver. 5.2 Annual peak flow frequency analysis Run Date / Time
11/01/2007 following Bulletin 17-B Guidelines 06/30/2011 21:13

Station - 01576000 Susquehanna River at Marietta, PA

I N P U T D A T A L I S T I N G

WATER YEAR	DISCHARGE	CODES	WATER YEAR	DISCHARGE	CODES
-1889	630000.0	H	1971	238000.0	
1932	256000.0		1972	1080000.0	
1933	296000.0		1973	224000.0	
1934	152000.0		1974	218000.0	
1935	263000.0		1975	545000.0	
1936	787000.0		1976	260000.0	
1937	241000.0		1977	283000.0	
1938	176000.0		1978	277000.0	
1939	213000.0		1979	452000.0	
1940	432000.0		1980	220000.0	

1941	249000.0	1981	316000.0
1942	307000.0	1982	207000.0
1943	428000.0	1983	276000.0
1944	211000.0	1984	458000.0
1945	254000.0	1985	137000.0
1946	492000.0	1986	384000.0
1947	214000.0	1987	238000.0
1948	310000.0	1988	200000.0
1949	227000.0	1989	230000.0
1950	298000.0	1990	138000.0
1951	420000.0	1991	216000.0
1952	329000.0	1992	172000.0
1953	227000.0	1993	448000.0
1954	246000.0	1994	365000.0
1955	183000.0	1995	192000.0
1956	325000.0	1996	601000.0
1957	249000.0	1997	277000.0
1958	274000.0	1998	336000.0
1959	241000.0	1999	247000.0
1960	370000.0	2000	224000.0
1961	386000.0	2001	158000.0
1962	265000.0	2002	197000.0
1963	245000.0	2003	289000.0
1964	473000.0	2004	577000.0
1965	129000.0	2005	391000.0
1966	280000.0	2006	421000.0
1967	191000.0	2007	247000.0
1968	208000.0	2008	352000.0
1969	143000.0	2009	146000.0
1970	350000.0	2010	316000.0

Explanation of peak discharge qualification codes

PeakFQ CODE	NWIS CODE	DEFINITION
D	3	Dam failure, non-recurrent flow anomaly
G	8	Discharge greater than stated value
X	3+8	Both of the above
L	4	Discharge less than stated value
K	6 OR C	Known effect of regulation or urbanization
H	7	Historic peak

- Minus-flagged discharge -- Not used in computation
  -8888.0 -- No discharge value given
- Minus-flagged water year -- Historic peak used in computation

1

```
Program PeakFq           U. S. GEOLOGICAL SURVEY              Seq.001.004
Ver. 5.2              Annual peak flow frequency analysis     Run Date / Time
11/01/2007            following Bulletin 17-B Guidelines      06/30/2011 21:13
```

Station - 01576000 Susquehanna River at Marietta, PA

EMPIRICAL FREQUENCY CURVES -- WEIBULL PLOTTING POSITIONS

WATER   RANKED   SYSTEMATIC   BULL.17B

YEAR	DISCHARGE	RECORD	ESTIMATE
1972	1080000.0	0.0125	0.0089
1936	787000.0	0.0250	0.0179
-1889	630000.0	--	0.0268
1996	601000.0	0.0375	0.0375
2004	577000.0	0.0500	0.0500
1975	545000.0	0.0625	0.0626
1946	492000.0	0.0750	0.0751
1964	473000.0	0.0875	0.0876
1984	458000.0	0.1000	0.1001
1979	452000.0	0.1125	0.1127
1993	448000.0	0.1250	0.1252
1940	432000.0	0.1375	0.1377
1943	428000.0	0.1500	0.1502
2006	421000.0	0.1625	0.1627
1951	420000.0	0.1750	0.1753
2005	391000.0	0.1875	0.1878
1961	386000.0	0.2000	0.2003
1986	384000.0	0.2125	0.2128
1960	370000.0	0.2250	0.2254
1994	365000.0	0.2375	0.2379
2008	352000.0	0.2500	0.2504
1970	350000.0	0.2625	0.2629
1998	336000.0	0.2750	0.2755
1952	329000.0	0.2875	0.2880
1956	325000.0	0.3000	0.3005
1981	316000.0	0.3125	0.3130
2010	316000.0	0.3250	0.3255
1948	310000.0	0.3375	0.3381
1942	307000.0	0.3500	0.3506
1950	298000.0	0.3625	0.3631
1933	296000.0	0.3750	0.3756
2003	289000.0	0.3875	0.3882
1977	283000.0	0.4000	0.4007
1966	280000.0	0.4125	0.4132
1978	277000.0	0.4250	0.4257
1997	277000.0	0.4375	0.4383
1983	276000.0	0.4500	0.4508
1958	274000.0	0.4625	0.4633
1962	265000.0	0.4750	0.4758
1935	263000.0	0.4875	0.4883
1976	260000.0	0.5000	0.5009
1932	256000.0	0.5125	0.5134
1945	254000.0	0.5250	0.5259
1941	249000.0	0.5375	0.5384
1957	249000.0	0.5500	0.5510
1999	247000.0	0.5625	0.5635
2007	247000.0	0.5750	0.5760
1954	246000.0	0.5875	0.5885
1963	245000.0	0.6000	0.6011
1937	241000.0	0.6125	0.6136
1959	241000.0	0.6250	0.6261
1971	238000.0	0.6375	0.6386
1987	238000.0	0.6500	0.6511
1989	230000.0	0.6625	0.6637
1949	227000.0	0.6750	0.6762
1953	227000.0	0.6875	0.6887
1973	224000.0	0.7000	0.7012
2000	224000.0	0.7125	0.7138
1980	220000.0	0.7250	0.7263
1974	218000.0	0.7375	0.7388
1991	216000.0	0.7500	0.7513

1947	214000.0	0.7625	0.7639
1939	213000.0	0.7750	0.7764
1944	211000.0	0.7875	0.7889
1968	208000.0	0.8000	0.8014
1982	207000.0	0.8125	0.8139
1988	200000.0	0.8250	0.8265
2002	197000.0	0.8375	0.8390
1995	192000.0	0.8500	0.8515
1967	191000.0	0.8625	0.8640
1955	183000.0	0.8750	0.8766
1938	176000.0	0.8875	0.8891
1992	172000.0	0.9000	0.9016
2001	158000.0	0.9125	0.9141
1934	152000.0	0.9250	0.9267
2009	146000.0	0.9375	0.9392
1969	143000.0	0.9500	0.9517
1990	138000.0	0.9625	0.9642
1985	137000.0	0.9750	0.9768
1965	129000.0	0.9875	0.9893

1

End PeakFQ analysis.
Stations processed : 1
Number of errors : 0
Stations skipped : 0
Station years : 80

Data records may have been ignored for the stations listed below.
(Card type must be Y, Z, N, H, I, 2, 3, 4, or *.)
(2, 4, and * records are ignored.)

For the station below, the following records were ignored:

FINISHED PROCESSING STATION: 01576000 USGS Susquehanna River at Marietta

For the station below, the following records were ignored:

FINISHED PROCESSING STATION:

NOAA Atlas 14, Volume 2, Version 3
Location name: Mt Wolf, Pennsylvania, US*
Coordinates: 40.0999, -76.6967
Elevation: 266ft*
* source: Google Maps

POINT PRECIPITATION FREQUENCY ESTIMATES

G.M. Bonnin, D. Martin, B. Lin, T. Parzybok, M.Yekta, and D. Riley

NOAA, National Weather Service, Silver Spring, Maryland

PF_tabular | PF_graphical | Maps_&_aerials

PF tabular

PDS-based point precipitation frequency estimates with 90% confidence intervals (in inches)[1]

Duration	Average recurrence interval(years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.318 (0.287-0.354)	0.379 (0.340-0.421)	0.447 (0.401-0.497)	0.496 (0.444-0.550)	0.556 (0.496-0.616)	0.599 (0.533-0.663)	0.641 (0.568-0.710)	0.680 (0.599-0.753)	0.727 (0.636-0.805)	0.764 (0.664-0.845)
10-min	0.508 (0.458-0.565)	0.605 (0.544-0.674)	0.715 (0.642-0.796)	0.793 (0.711-0.880)	0.887 (0.790-0.982)	0.954 (0.848-1.06)	1.02 (0.903-1.13)	1.08 (0.950-1.19)	1.15 (1.01-1.27)	1.20 (1.05-1.33)
15-min	0.635 (0.572-0.706)	0.761 (0.684-0.847)	0.905 (0.813-1.01)	1.00 (0.899-1.11)	1.12 (1.00-1.25)	1.21 (1.07-1.34)	1.29 (1.14-1.43)	1.36 (1.20-1.51)	1.45 (1.27-1.60)	1.51 (1.31-1.67)
30-min	0.870 (0.784-0.968)	1.05 (0.944-1.17)	1.29 (1.15-1.43)	1.45 (1.30-1.61)	1.67 (1.48-1.84)	1.82 (1.62-2.02)	1.97 (1.75-2.18)	2.12 (1.87-2.35)	2.30 (2.02-2.55)	2.44 (2.13-2.71)
60-min	1.09 (0.978-1.21)	1.32 (1.19-1.47)	1.65 (1.48-1.83)	1.89 (1.70-2.10)	2.22 (1.98-2.46)	2.47 (2.19-2.73)	2.72 (2.41-3.01)	2.97 (2.62-3.29)	3.31 (2.89-3.66)	3.57 (3.10-3.95)
2-hr	1.27 (1.15-1.42)	1.54 (1.39-1.72)	1.96 (1.76-2.17)	2.28 (2.05-2.53)	2.75 (2.45-3.03)	3.13 (2.78-3.45)	3.54 (3.12-3.90)	3.97 (3.47-4.37)	4.59 (3.97-5.06)	5.10 (4.38-5.62)
3-hr	1.39 (1.25-1.55)	1.69 (1.52-1.88)	2.14 (1.93-2.38)	2.50 (2.25-2.78)	3.00 (2.68-3.32)	3.42 (3.03-3.78)	3.87 (3.41-4.27)	4.34 (3.80-4.79)	5.02 (4.34-5.55)	5.57 (4.78-6.17)
6-hr	1.71 (1.54-1.92)	2.07 (1.87-2.32)	2.61 (2.35-2.93)	3.07 (2.74-3.42)	3.73 (3.31-4.14)	4.29 (3.79-4.76)	4.90 (4.29-5.43)	5.57 (4.83-6.16)	6.56 (5.61-7.25)	7.39 (6.24-8.17)
12-hr	2.08 (1.86-2.37)	2.51 (2.24-2.86)	3.18 (2.83-3.62)	3.76 (3.33-4.26)	4.63 (4.07-5.23)	5.39 (4.70-6.07)	6.24 (5.38-7.01)	7.18 (6.13-8.05)	8.61 (7.22-9.64)	9.85 (8.15-11.0)
24-hr	2.39 (2.20-2.63)	2.89 (2.66-3.18)	3.70 (3.39-4.07)	4.40 (4.03-4.83)	5.49 (4.97-5.99)	6.45 (5.80-7.00)	7.53 (6.70-8.14)	8.76 (7.69-9.45)	10.6 (9.19-11.4)	12.3 (10.5-13.2)
2-day	2.77 (2.56-3.06)	3.35 (3.09-3.70)	4.29 (3.94-4.72)	5.09 (4.65-5.59)	6.28 (5.70-6.88)	7.32 (6.59-8.00)	8.48 (7.58-9.24)	9.77 (8.63-10.6)	11.7 (10.2-12.8)	13.4 (11.5-14.6)
3-day	2.95 (2.72-3.24)	3.56 (3.29-3.91)	4.54 (4.19-4.98)	5.38 (4.94-5.90)	6.65 (6.06-7.26)	7.75 (7.01-8.44)	8.98 (8.05-9.76)	10.3 (9.18-11.2)	12.4 (10.8-13.5)	14.2 (12.2-15.4)
4-day	3.12 (2.89-3.41)	3.77 (3.49-4.12)	4.80 (4.44-5.25)	5.68 (5.23-6.21)	7.02 (6.41-7.64)	8.18 (7.42-8.89)	9.47 (8.52-10.3)	10.9 (9.73-11.9)	13.1 (11.5-14.2)	15.0 (13.0-16.3)
7-day	3.66 (3.40-3.98)	4.40 (4.09-4.80)	5.55 (5.14-6.04)	6.53 (6.02-7.09)	7.99 (7.33-8.68)	9.27 (8.44-10.0)	10.7 (9.65-11.6)	12.2 (11.0-13.2)	14.6 (12.9-15.8)	16.6 (14.5-18.0)
10-day	4.20 (3.92-4.54)	5.04 (4.71-5.46)	6.28 (5.85-6.78)	7.31 (6.80-7.89)	8.82 (8.15-9.50)	10.1 (9.28-10.9)	11.5 (10.5-12.3)	13.0 (11.7-13.9)	15.2 (13.5-16.3)	17.0 (15.0-18.3)
20-day	5.72 (5.39-6.10)	6.80 (6.40-7.26)	8.19 (7.71-8.74)	9.33 (8.75-9.95)	10.9 (10.2-11.6)	12.2 (11.4-13.0)	13.6 (12.6-14.4)	15.0 (13.8-15.9)	16.9 (15.5-18.0)	18.5 (16.8-19.8)
30-day	7.07 (6.69-7.51)	8.36 (7.90-8.87)	9.91 (9.36-10.5)	11.2 (10.5-11.8)	12.9 (12.1-13.7)	14.3 (13.4-15.2)	15.7 (14.7-16.7)	17.2 (16.0-18.3)	19.2 (17.7-20.4)	20.8 (19.0-22.2)
45-day	8.90 (8.47-9.37)	10.5 (9.98-11.0)	12.2 (11.6-12.8)	13.6 (12.9-14.3)	15.4 (14.5-16.1)	16.7 (15.8-17.5)	18.1 (17.0-19.0)	19.4 (18.2-20.4)	21.1 (19.8-22.3)	22.5 (20.9-23.7)
60-day	10.6 (10.2-11.2)	12.5 (11.9-13.1)	14.4 (13.7-15.1)	15.9 (15.1-16.6)	17.8 (16.9-18.6)	19.2 (18.2-20.1)	20.5 (19.4-21.6)	21.9 (20.6-23.0)	23.6 (22.2-24.8)	24.8 (23.3-26.1)

[1] Precipitation frequency (PF) estimates in this table are based on frequency analysis of partial duration series (PDS).
Numbers in parenthesis are PF estimates at lower and upper bounds of the 90% confidence interval. The probability that precipitation frequency estimates (for a given duration and average recurrence interval) will be greater than the upper bound (or less than the lower bound) is 5%. Estimates at upper bounds are not checked against probable maximum precipitation (PMP) estimates and may be higher than currently valid PMP values.
Please refer to NOAA Atlas 14 document for more information.

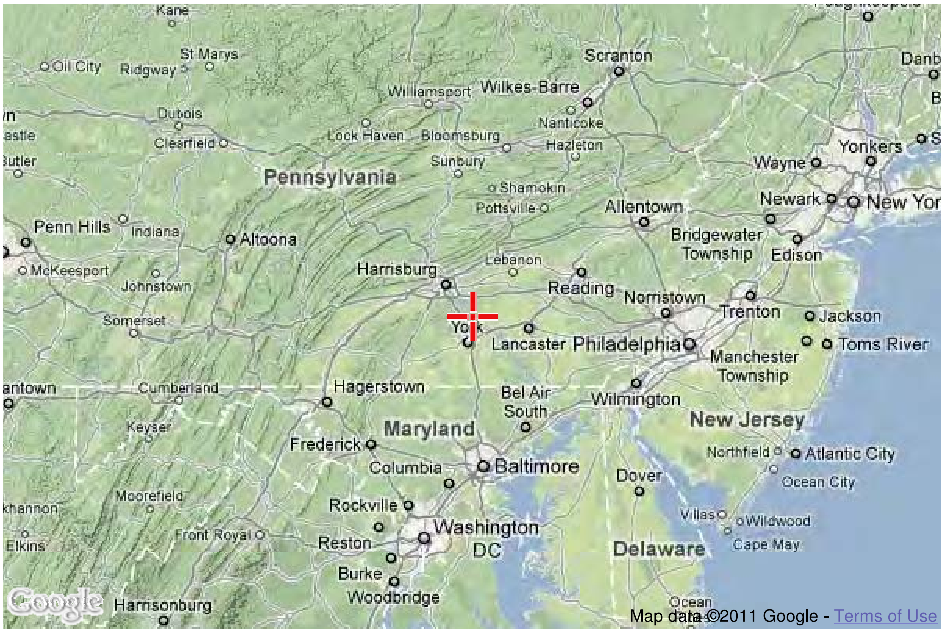
Back to Top

PF graphical

Back to Top

Maps & aerials

Small scale terrain



Large scale terrain

Large scale map

Large scale aerial

Back to Top

US Department of Commerce
National Oceanic and Atmospheric Administration
National Weather Service
Office of Hydrologic Development
1325 East West Highway
Silver Spring, MD 20910
Questions?: HDSC.Questions@noaa.gov

Select Other Date

These data are preliminary and have not undergone final quality control by the National Climatic Data Center (NCDC). Therefore, these data are subject to revision. Final and certified climate data can be accessed at the NCDC - http://www.ncdc.noaa.gov.

Climatological Report (Annual)

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CXUS51 KCTP 021406
CLAMDT

CLIMATE REPORT
NATIONAL WEATHER SERVICE STATE COLLEGE PA
905 AM EST SUN JAN 2 2011

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...THE HARRISBURG PA CLIMATE SUMMARY FOR THE YEAR OF 2010...

CLIMATE NORMAL PERIOD 1971 TO 2000
CLIMATE RECORD PERIOD 1888 TO 2011

WEATHER         OBSERVED          NORMAL  DEPART
                 VALUE   DATE(S)  VALUE   FROM
                                          NORMAL
...................................................
TEMPERATURE (F)
RECORD
 HIGH              107   07/03/1966
 LOW               -22   01/21/1994

2010...
HIGHEST            100   07/06
LOWEST              13   01/31
AVG. MAXIMUM      64.1              62.4     1.7
AVG. MINIMUM      46.1              44.1     2.0
MEAN              55.1              53.3     1.8
DAYS MAX >= 90      34              22.4    11.6
DAYS MAX <= 32      20              19.7     0.3
DAYS MIN <= 32     100             101.7    -1.7
DAYS MIN <= 0        0               0.9    -0.9

PRECIPITATION (INCHES)
RECORD
 MAXIMUM         59.27   1972
 MINIMUM         25.52   1941

2010...
```

```
TOTALS           39.43                41.45   -2.02
DAILY AVG.        0.11                 0.11    0.00
DAYS >= .01       100                 119.2   -19.2
DAYS >= .10        68                  75.0    -7.0
DAYS >= .50        25                  25.0     0.0
DAYS >= 1.00        9                   9.8    -0.8
GREATEST
 24 HR. TOTAL     3.42


SNOWFALL (INCHES)
RECORDS
 TOTAL            81.3   1960
 24 HR TOTAL      25.0   02/11-02/12/1983

2010...
TOTALS            44.0                 36.9     7.1
 LIQUID EQUIV     4.40                 3.70    0.70
SINCE 7/1          0.8                  7.7    -6.9
 LIQUID 7/1       0.08
DAYS >= TRACE       33                 18.4    14.6
DAYS >= 1.0          7                 12.8    -5.8
GREATEST
 SNOW DEPTH         22   02/11
 24 HR TOTAL      12.3   02/10


DEGREE_DAYS
HEATING TOTAL     4894                 5347    -453
 SINCE 7/1        1951                 1949       2
COOLING TOTAL     1421                  962     459
 SINCE 1/1        1421                  955     466

FREEZE DATES
RECORD
 EARLIEST      09/24/1963
 LATEST        05/11/1966

2010...
EARLIEST       11/02
LATEST         03/26
.................................................

WIND (MPH)
AVERAGE WIND SPEED                7.0
HIGHEST WIND SPEED/DIRECTION      41/270     DATE   04/16
HIGHEST GUST SPEED/DIRECTION      63/250     DATE   04/16

SKY COVER
POSSIBLE SUNSHINE (PERCENT)    MM
```

NUMBER OF DAYS FAIR	88
NUMBER OF DAYS PC	139
NUMBER OF DAYS CLOUDY	120

AVERAGE RH (PERCENT) 64

WEATHER CONDITIONS. NUMBER OF DAYS WITH

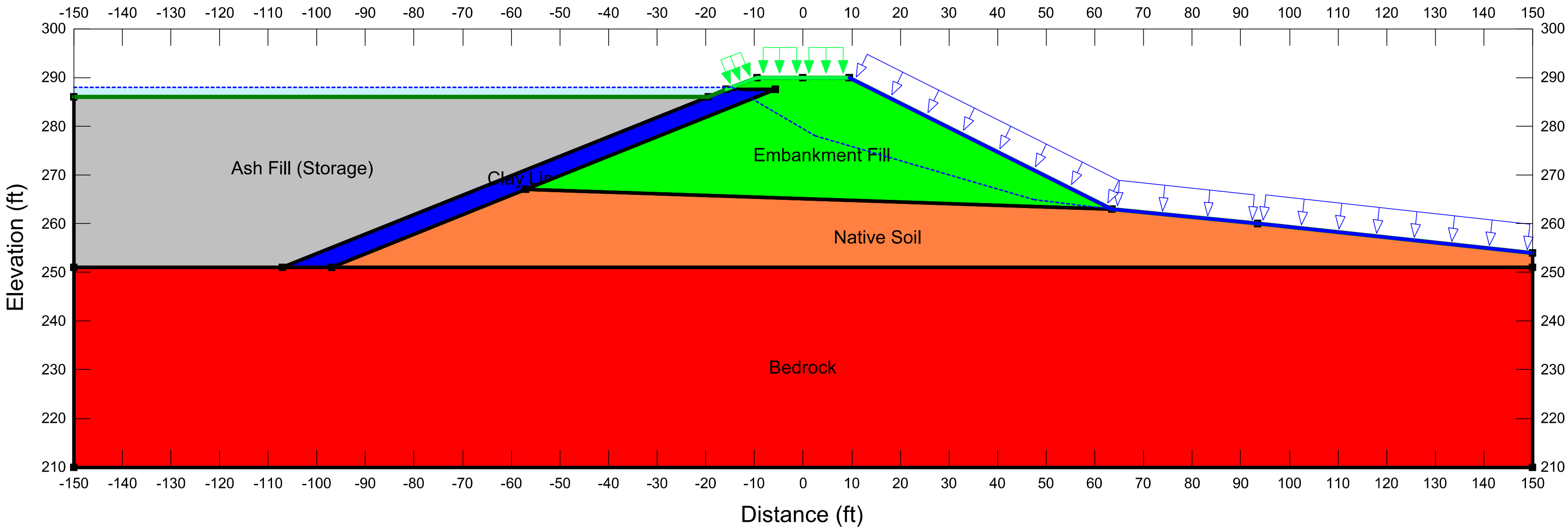
THUNDERSTORM	30	MIXED PRECIP	0
HEAVY RAIN	40	RAIN	61
LIGHT RAIN	118	FREEZING RAIN	0
LT FREEZING RAIN	2	HAIL	0
HEAVY SNOW	4	SNOW	6
LIGHT SNOW	31	SLEET	2
FOG	152	FOG W/VIS <= 1/4 MILE	12
HAZE	135		

- INDICATES NEGATIVE NUMBERS.
R INDICATES RECORD WAS SET OR TIED.
MM INDICATES DATA IS MISSING.
T INDICATES TRACE AMOUNT.

LA CORTE

APPENDIX D

SEEPAGE ANALYSIS PLATES



Material Input Properties

Name: Bedrock Model: Saturated Only K-Sat: 1e-010 ft/sec Volumetric Water Content: 0 ft³/ft³ Mv: 0 /psf K-Ratio: 1 K-Direction: 0 °

Name: Native Soil Model: Saturated Only K-Sat: 1e-008 ft/sec Volumetric Water Content: 0 ft³/ft³ Mv: 0 /psf K-Ratio: 1 K-Direction: 0 °

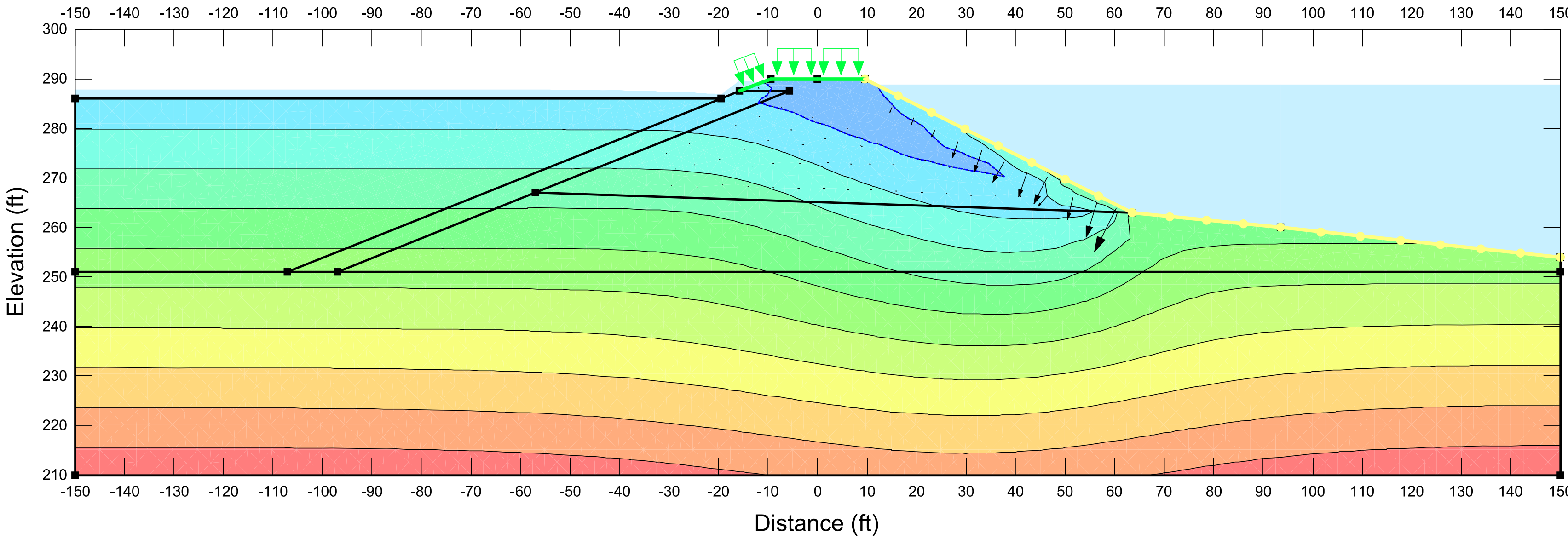
Name: Clay Liner Model: Saturated Only K-Sat: 1e-009 ft/sec Volumetric Water Content: 0 ft³/ft³ Mv: 0 /psf K-Ratio: 1 K-Direction: 0 °

Name: Ash Fill (Storage) Model: Saturated Only K-Sat: 1e-008 ft/sec Volumetric Water Content: 0 ft³/ft³ Mv: 0 /psf K-Ratio: 1 K-Direction: 0 °

Name: Embankment Fill Model: Saturated / Unsaturated K-Function: Embankment Fill Unsat K Vol. WC. Function: Embankment Fill - Vol. WC K-Ratio: 1 K-Direction: 0 °

Transient Seepage
Brunner Island Ash Basin No. 6
Station 21+80 (Section 1-1)
Manchester Township, Pennsylvania

Plate D1 - Seepage Model



Material Input Properties

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Name: Native Soil Model: Saturated Only K-Sat: 1e-008 ft/sec Volumetric Water Content: 0 ft³/ft³ Mv: 0 /psf K-Ratio: 1 K-Direction: 0 °

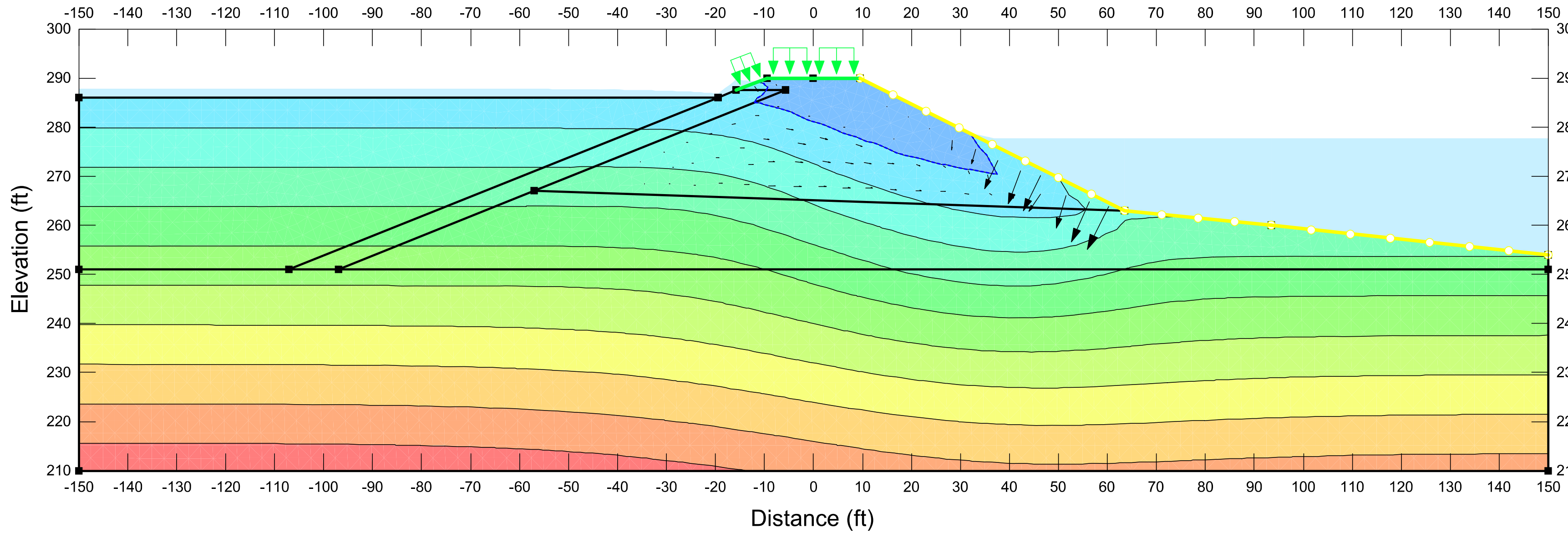
Name: Clay Liner Model: Saturated Only K-Sat: 1e-009 ft/sec Volumetric Water Content: 0 ft³/ft³ Mv: 0 /psf K-Ratio: 1 K-Direction: 0 °

Name: Ash Fill (Storage) Model: Saturated Only K-Sat: 1e-008 ft/sec Volumetric Water Content: 0 ft³/ft³ Mv: 0 /psf K-Ratio: 1 K-Direction: 0 °

Name: Embankment Fill Model: Saturated / Unsaturated K-Function: Embankment Fill Unsat K Vol. WC. Function: Embankment Fill - Vol. WC K-Ratio: 1 K-Direction: 0 °

Transient Seepage (4)
Brunner Island Ash Basin No. 6
Station 21+80 (Section 1-1)
Manchester Township, Pennsylvania

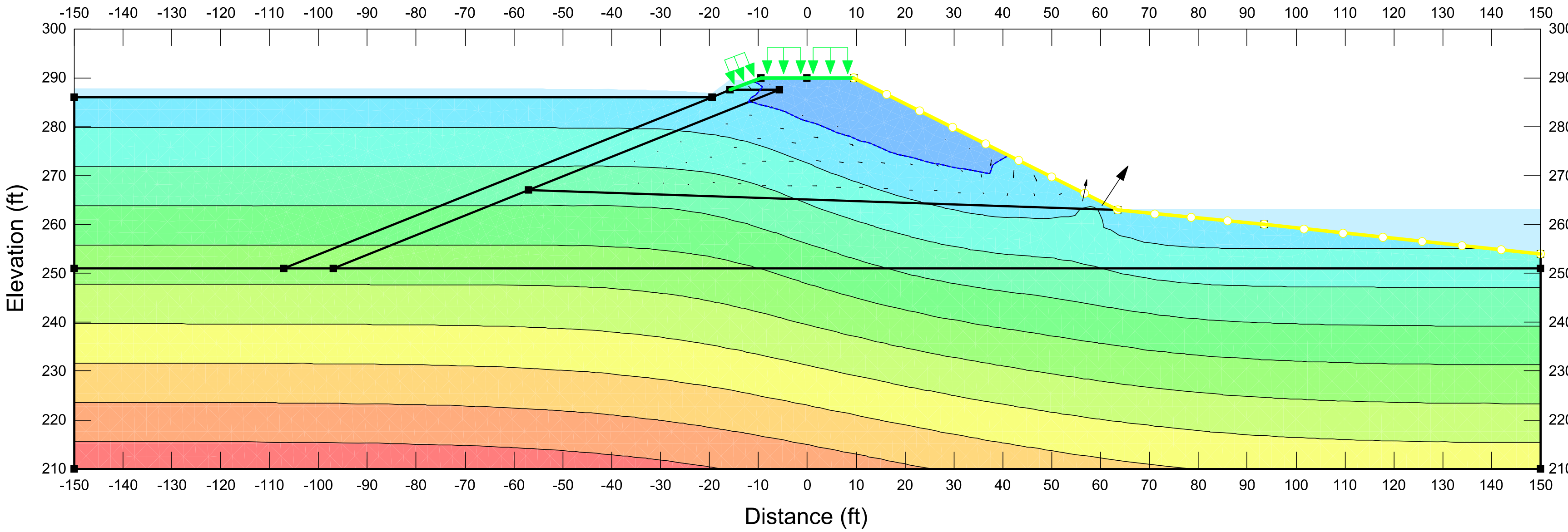
Plate D2a - River at 500-yr Flood Elevation
(Case 1: Kv=Kh=6.8*10^-6 ft/sec)



Material Input Properties
Name: Bedrock Model: Saturated Only K-Sat: 1e-010 ft/sec Volumetric Water Content: 0 ft³/ft³ Mv: 0 /psf K-Ratio: 1 K-Direction: 0 °
Name: Native Soil Model: Saturated Only K-Sat: 1e-008 ft/sec Volumetric Water Content: 0 ft³/ft³ Mv: 0 /psf K-Ratio: 1 K-Direction: 0 °
Name: Clay Liner Model: Saturated Only K-Sat: 1e-009 ft/sec Volumetric Water Content: 0 ft³/ft³ Mv: 0 /psf K-Ratio: 1 K-Direction: 0 °
Name: Ash Fill (Storage) Model: Saturated Only K-Sat: 1e-008 ft/sec Volumetric Water Content: 0 ft³/ft³ Mv: 0 /psf K-Ratio: 1 K-Direction: 0 °
Name: Embankment Fill Model: Saturated / Unsaturated K-Function: Embankment Fill Unsat K Vol. WC. Function: Embankment Fill - Vol. WC K-Ratio: 1 K-Direction: 0 °

Transient Seepage (5)
Brunner Island Ash Basin No. 6
Station 21+80 (Section 1-1)
Manchester Township, Pennsylvania

Plate D2b - River at Mid-Slope Elevation
(Case 1: Kv=Kh=6.8*10^-6 ft/sec)



Material Input Properties

Name: Bedrock Model: Saturated Only K-Sat: 1e-010 ft/sec Volumetric Water Content: 0 ft³/ft³ Mv: 0 /psf K-Ratio: 1 K-Direction: 0 °

Name: Native Soil Model: Saturated Only K-Sat: 1e-008 ft/sec Volumetric Water Content: 0 ft³/ft³ Mv: 0 /psf K-Ratio: 1 K-Direction: 0 °

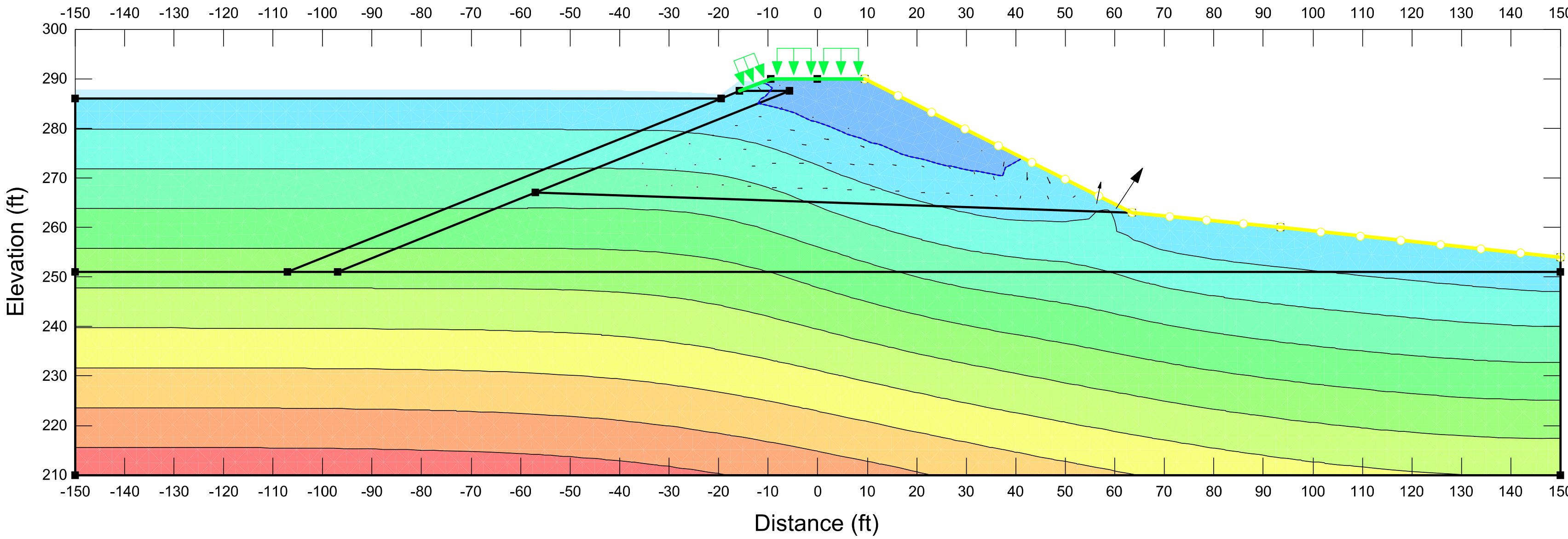
Name: Clay Liner Model: Saturated Only K-Sat: 1e-009 ft/sec Volumetric Water Content: 0 ft³/ft³ Mv: 0 /psf K-Ratio: 1 K-Direction: 0 °

Name: Ash Fill (Storage) Model: Saturated Only K-Sat: 1e-008 ft/sec Volumetric Water Content: 0 ft³/ft³ Mv: 0 /psf K-Ratio: 1 K-Direction: 0 °

Name: Embankment Fill Model: Saturated / Unsaturated K-Function: Embankment Fill Unsat K Vol. WC. Function: Embankment Fill - Vol. WC K-Ratio: 1 K-Direction: 0 °

Transient Seepage (5)
Brunner Island Ash Basin No. 6
Station 21+80 (Section 1-1)
Manchester Township, Pennsylvania

Plate D2c - River at Toe of Slope Elevation
(Case 1: Kv=Kh=6.8*10^-6 ft/sec)



Material Input Properties

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Name: Native Soil Model: Saturated Only K-Sat: 1e-008 ft/sec Volumetric Water Content: 0 ft³/ft³ Mv: 0 /psf K-Ratio: 1 K-Direction: 0 °

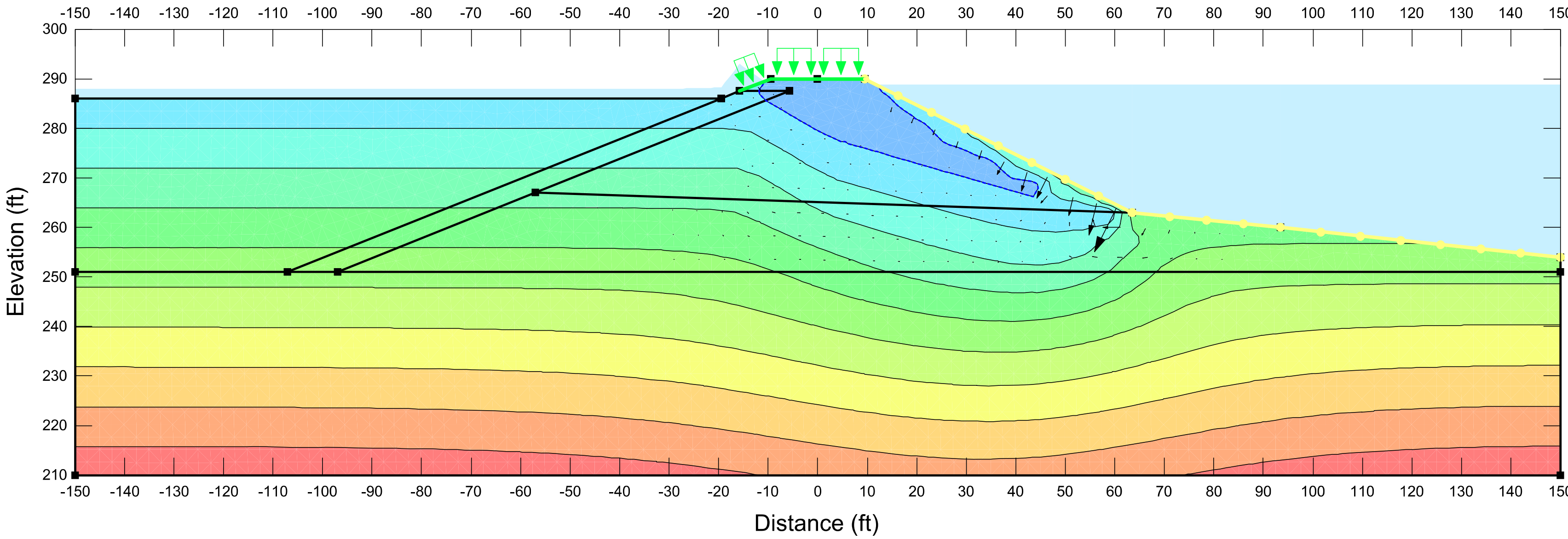
Name: Clay Liner Model: Saturated Only K-Sat: 1e-009 ft/sec Volumetric Water Content: 0 ft³/ft³ Mv: 0 /psf K-Ratio: 1 K-Direction: 0 °

Name: Ash Fill (Storage) Model: Saturated Only K-Sat: 1e-008 ft/sec Volumetric Water Content: 0 ft³/ft³ Mv: 0 /psf K-Ratio: 1 K-Direction: 0 °

Name: Embankment Fill Model: Saturated / Unsaturated K-Function: Embankment Fill Unsat K Vol. WC. Function: Embankment Fill - Vol. WC K-Ratio: 1 K-Direction: 0 °

Transient Seepage (5)
Brunner Island Ash Basin No. 6
Station 21+80 (Section 1-1)
Manchester Township, Pennsylvania

Plate D2d - River at Normal Water Level Elevation
(Case 1: Kv=Kh=6.8*10^-6 ft/sec)



Material Input Properties

Name: Bedrock Model: Saturated Only K-Sat: 1e-010 ft/sec Volumetric Water Content: 0 ft³/ft³ Mv: 0 /psf K-Ratio: 1 K-Direction: 0 °

Name: Native Soil Model: Saturated Only K-Sat: 1e-008 ft/sec Volumetric Water Content: 0 ft³/ft³ Mv: 0 /psf K-Ratio: 1 K-Direction: 0 °

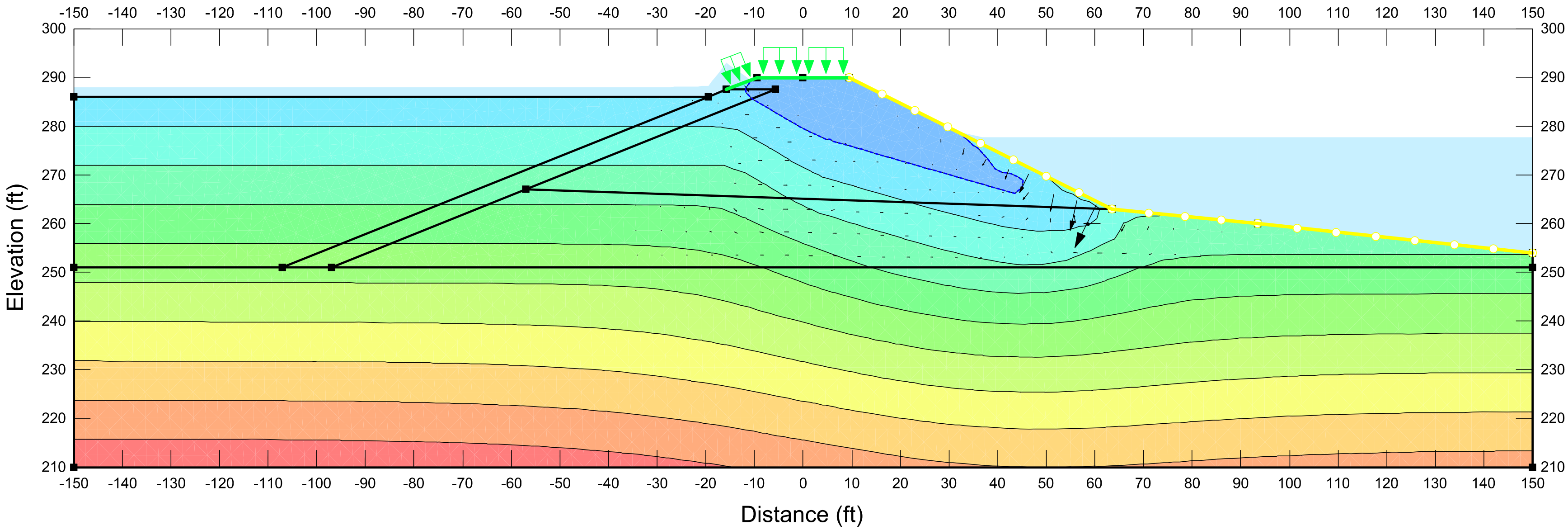
Name: Clay Liner Model: Saturated Only K-Sat: 1e-009 ft/sec Volumetric Water Content: 0 ft³/ft³ Mv: 0 /psf K-Ratio: 1 K-Direction: 0 °

Name: Ash Fill (Storage) Model: Saturated Only K-Sat: 1e-008 ft/sec Volumetric Water Content: 0 ft³/ft³ Mv: 0 /psf K-Ratio: 1 K-Direction: 0 °

Name: Embankment Fill Model: Saturated / Unsaturated K-Function: Embankment Fill Unsat K Vol. WC. Function: Embankment Fill - Vol. WC K-Ratio: 1 K-Direction: 0 °

Transient Seepage (4)
Brunner Island Ash Basin No. 6
Station 21+80 (Section 1-1)
Manchester Township, Pennsylvania

Plate D3a - River at 500-yr Flood Elevation
(Case 3: Kv=Kh=6.8*10^-9 ft/sec)



Material Input Properties

Name: Bedrock Model: Saturated Only K-Sat: 1e-010 ft/sec Volumetric Water Content: 0 ft³/ft³ Mv: 0 /psf K-Ratio: 1 K-Direction: 0 °

Name: Native Soil Model: Saturated Only K-Sat: 1e-008 ft/sec Volumetric Water Content: 0 ft³/ft³ Mv: 0 /psf K-Ratio: 1 K-Direction: 0 °

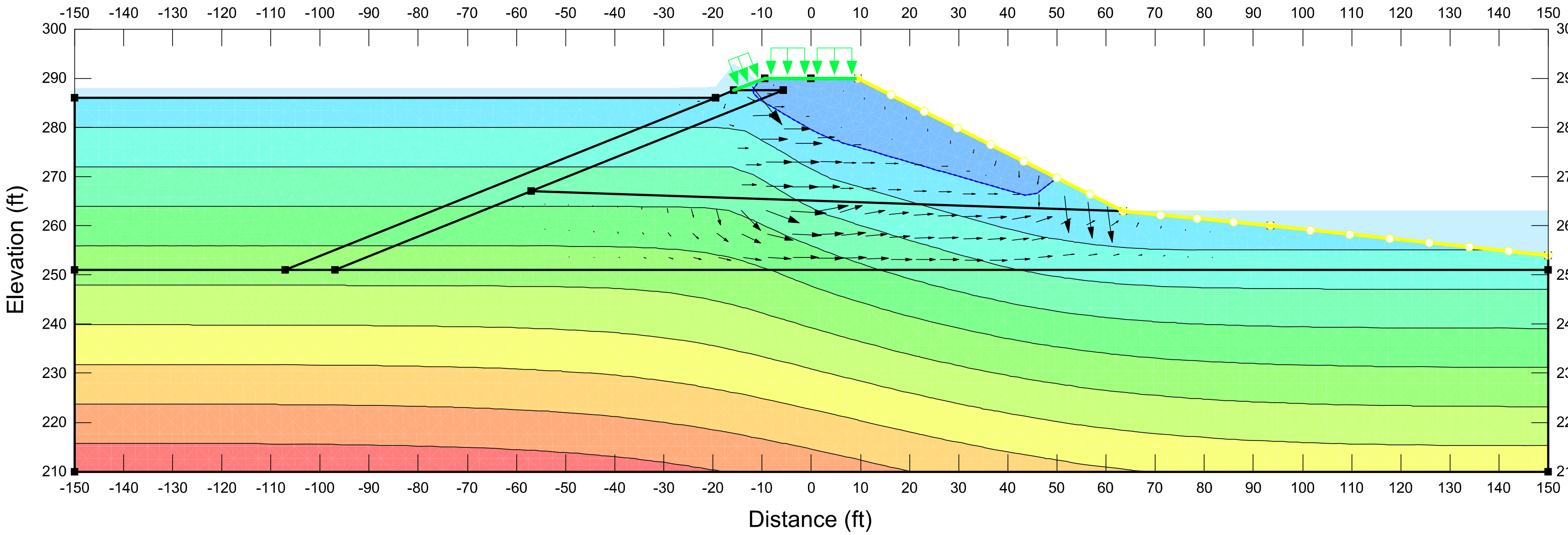
Name: Clay Liner Model: Saturated Only K-Sat: 1e-009 ft/sec Volumetric Water Content: 0 ft³/ft³ Mv: 0 /psf K-Ratio: 1 K-Direction: 0 °

Name: Ash Fill (Storage) Model: Saturated Only K-Sat: 1e-008 ft/sec Volumetric Water Content: 0 ft³/ft³ Mv: 0 /psf K-Ratio: 1 K-Direction: 0 °

Name: Embankment Fill Model: Saturated / Unsaturated K-Function: Embankment Fill Unsat K Vol. WC. Function: Embankment Fill - Vol. WC K-Ratio: 1 K-Direction: 0 °

Transient Seepage (5)
Brunner Island Ash Basin No. 6
Station 21+80 (Section 1-1)
Manchester Township, Pennsylvania

Plate D3b - River at Mid-Slope Elevation
(Case 3: Kv=Kh=6.8*10^-9 ft/sec)



Material Input Properties

Name: Bedrock   Model: Saturated Only   K-Sat: 1e-010 ft/sec   Volumetric Water Content: 0 ft³/ft³   Mv: 0 /psf   K-Ratio: 1   K-Direction: 0 °

Name: Native Soil   Model: Saturated Only   K-Sat: 1e-008 ft/sec   Volumetric Water Content: 0 ft³/ft³   Mv: 0 /psf   K-Ratio: 1   K-Direction: 0 °

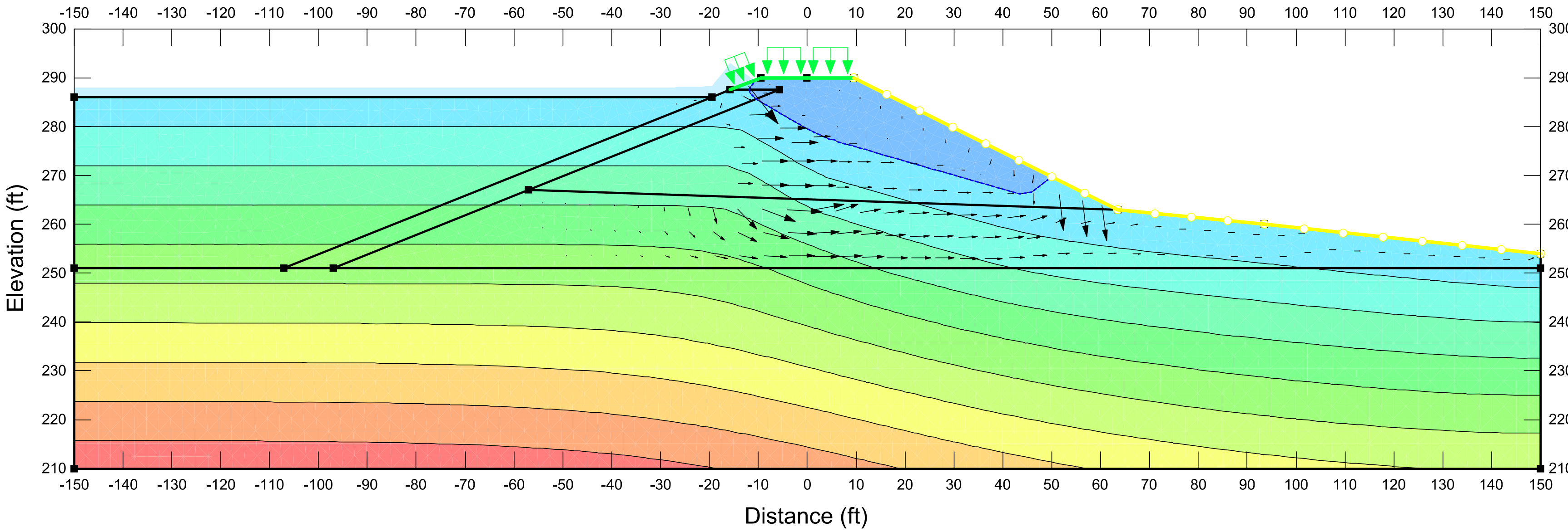
Name: Clay Liner   Model: Saturated Only   K-Sat: 1e-009 ft/sec   Volumetric Water Content: 0 ft³/ft³   Mv: 0 /psf   K-Ratio: 1   K-Direction: 0 °

Name: Ash Fill (Storage)   Model: Saturated Only   K-Sat: 1e-008 ft/sec   Volumetric Water Content: 0 ft³/ft³   Mv: 0 /psf   K-Ratio: 1   K-Direction: 0 °

Name: Embankment Fill   Model: Saturated / Unsaturated   K-Function: Embankment Fill Unsat K   Vol. WC. Function: Embankment Fill - Vol. WC   K-Ratio: 1   K-Direction: 0 °

Transient Seepage (5)
Brunner Island Ash Basin No. 6
Station 21+80 (Section 1-1)
Manchester Township, Pennsylvania

Plate D3c - River at Toe of Slope Elevation
(Case 3: Kv=Kh=6.8*10^-9 ft/sec)



Material Input Properties

Name: Bedrock Model: Saturated Only K-Sat: 1e-010 ft/sec Volumetric Water Content: 0 ft³/ft³ Mv: 0 /psf K-Ratio: 1 K-Direction: 0 °

Name: Native Soil Model: Saturated Only K-Sat: 1e-008 ft/sec Volumetric Water Content: 0 ft³/ft³ Mv: 0 /psf K-Ratio: 1 K-Direction: 0 °

Name: Clay Liner Model: Saturated Only K-Sat: 1e-009 ft/sec Volumetric Water Content: 0 ft³/ft³ Mv: 0 /psf K-Ratio: 1 K-Direction: 0 °

Name: Ash Fill (Storage) Model: Saturated Only K-Sat: 1e-008 ft/sec Volumetric Water Content: 0 ft³/ft³ Mv: 0 /psf K-Ratio: 1 K-Direction: 0 °

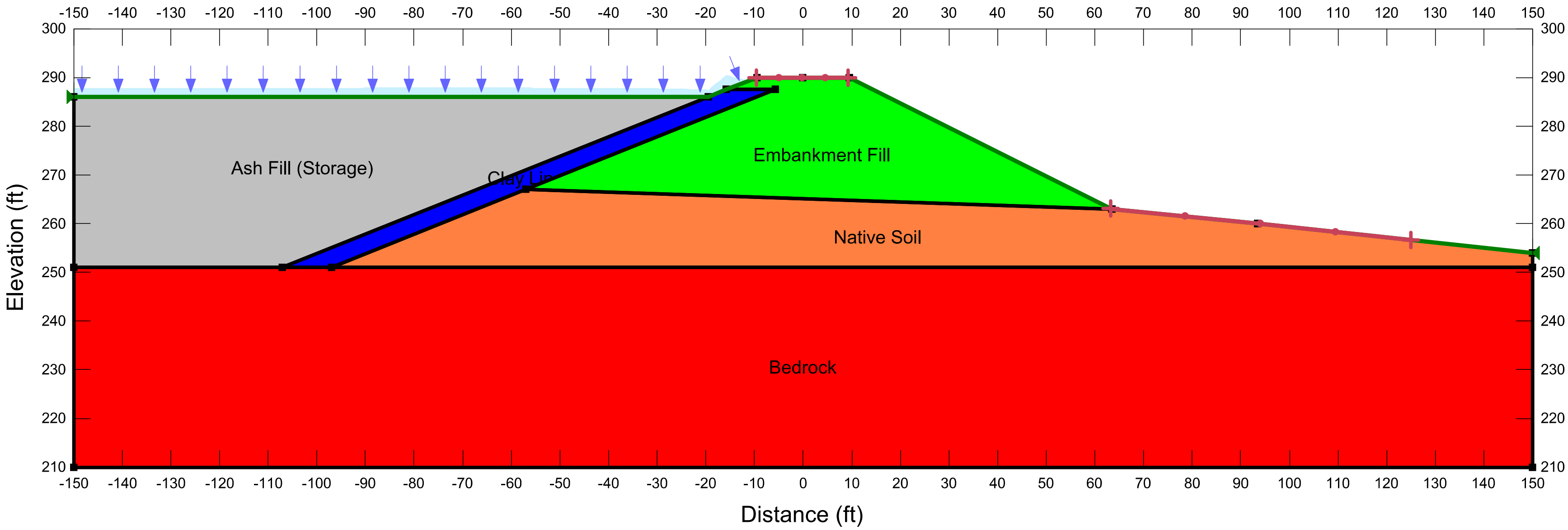
Name: Embankment Fill Model: Saturated / Unsaturated K-Function: Embankment Fill Unsat K Vol. WC. Function: Embankment Fill - Vol. WC K-Ratio: 1 K-Direction: 0 °

Plate D3d - River at Normal Water Level Elevation
(Case 3: Kv=Kh=6.8*10^-9 ft/sec)

Transient Seepage (5)
Brunner Island Ash Basin No. 6
Station 21+80 (Section 1-1)
Manchester Township, Pennsylvania

APPENDIX E

SLOPE STABILITY ANALYSIS PLATES



Material Input Properties

Name: Bedrock Model: Mohr-Coulomb Unit Weight: 160 pcf Cohesion: 2000 psf Phi: 45 °

Name: Native Soil Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion: 0 psf Phi: 30 °

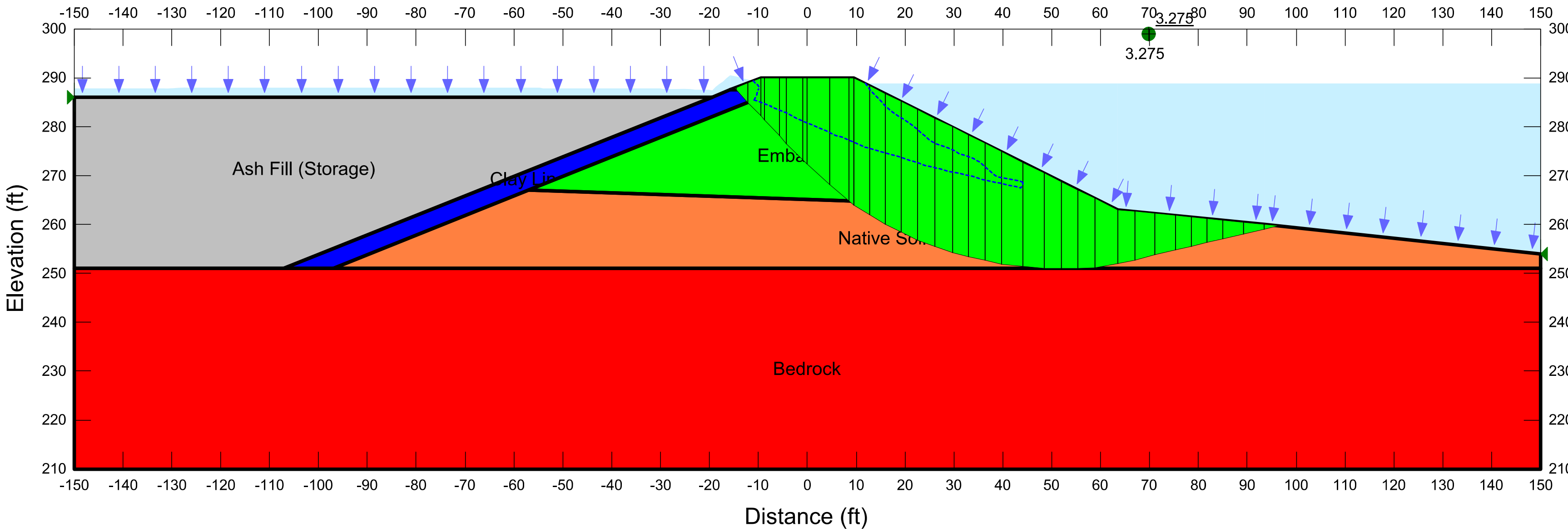
Name: Clay Liner Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion: 0 psf Phi: 30 °

Name: Ash Fill (Storage) Model: Mohr-Coulomb Unit Weight: 90 pcf Cohesion: 0 psf Phi: 30 °

Name: Embankment Fill Model: Mohr-Coulomb Unit Weight: 135 pcf Unit Wt. Above Water Table: 125 pcf Cohesion: 0 psf Phi: 37 °

Plate E1 - Slope Model

Slope Stability (6)
Brunner Island Ash Basin No. 6
Station 21+80 (Section 1-1)
Manchester Township, Pennsylvania



Material Input Properties

Name: Bedrock Model: Mohr-Coulomb Unit Weight: 160 pcf Cohesion: 2000 psf Phi: 45 °

Name: Native Soil Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion: 0 psf Phi: 30 °

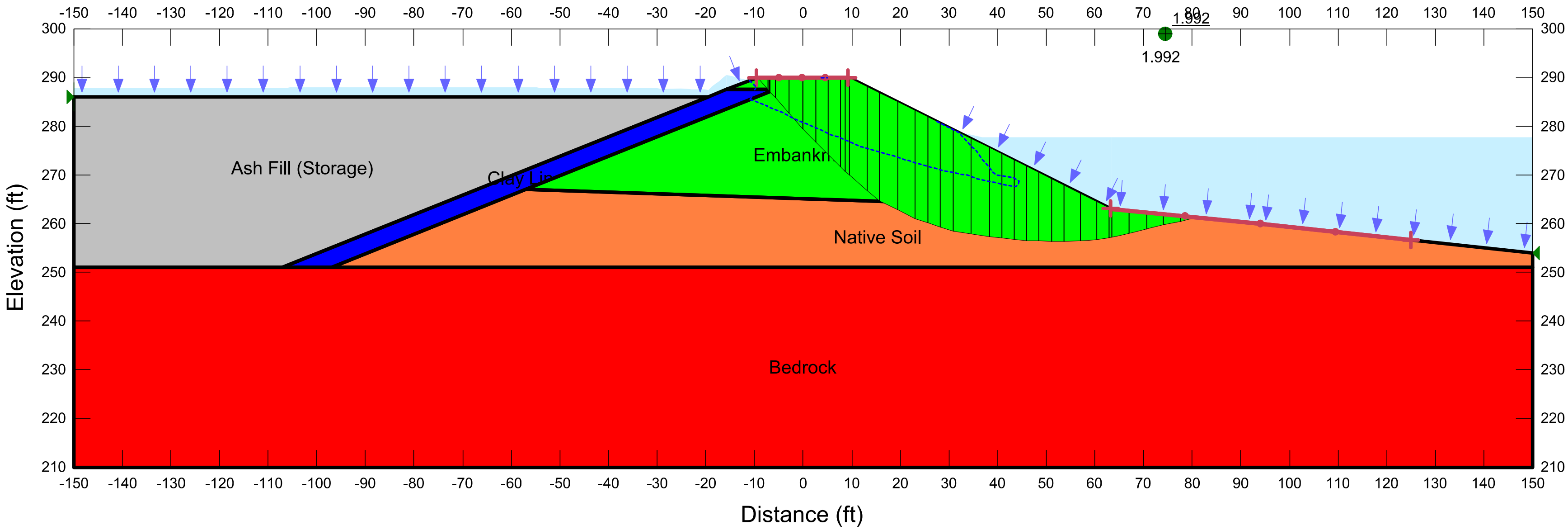
Name: Clay Liner Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion: 0 psf Phi: 30 °

Name: Ash Fill (Storage) Model: Mohr-Coulomb Unit Weight: 90 pcf Cohesion: 0 psf Phi: 30 °

Name: Embankment Fill Model: Mohr-Coulomb Unit Weight: 135 pcf Unit Wt. Above Water Table: 125 pcf Cohesion: 0 psf Phi: 37 °

Slope Stability (2)
Brunner Island Ash Basin No. 6
Station 21+80 (Section 1-1)
Manchester Township, Pennsylvania

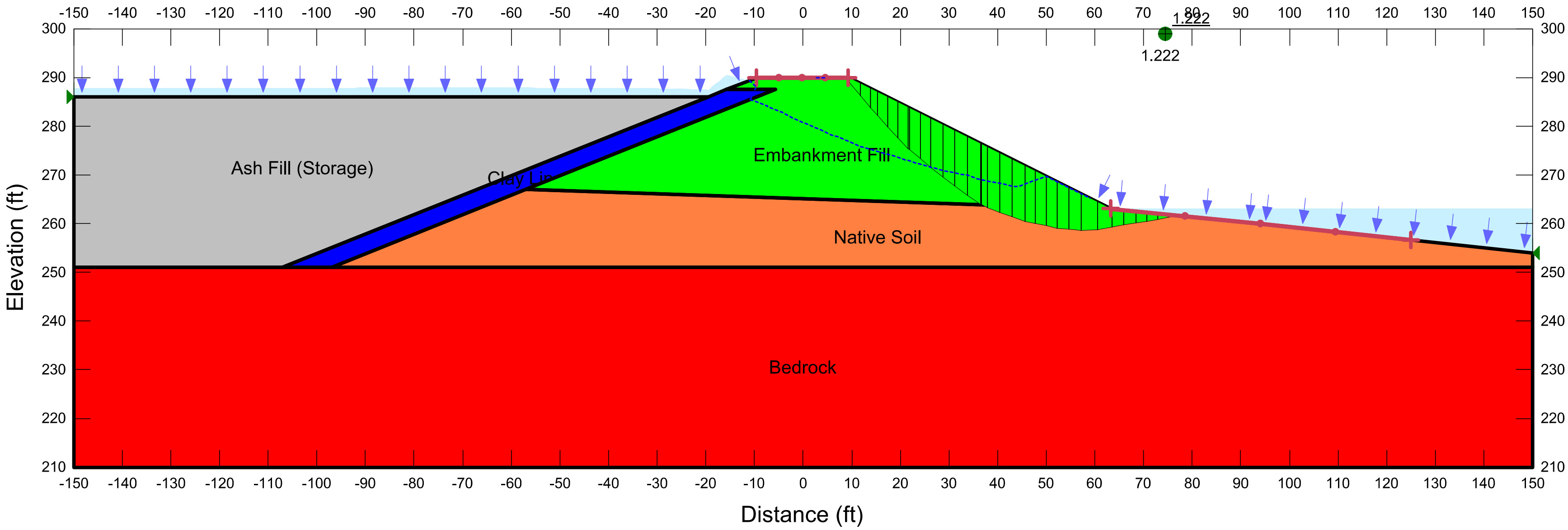
Plate E2a - River at 500-yr Flood Elevation
(Case 2: Kv=Kh=2.8*10^-6 ft/sec)



Material Input Properties
Name: Bedrock Model: Mohr-Coulomb Unit Weight: 160 pcf Cohesion: 2000 psf Phi: 45 °
Name: Native Soil Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion: 0 psf Phi: 30 °
Name: Clay Liner Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion: 0 psf Phi: 30 °
Name: Ash Fill (Storage) Model: Mohr-Coulomb Unit Weight: 90 pcf Cohesion: 0 psf Phi: 30 °
Name: Embankment Fill Model: Mohr-Coulomb Unit Weight: 135 pcf Unit Wt. Above Water Table: 125 pcf Cohesion: 0 psf Phi: 37 °

Slope Stability (6)
Brunner Island Ash Basin No. 6
Station 21+80 (Section 1-1)
Manchester Township, Pennsylvania

Plate E2b - River at Mid-Slope Elevation
(Case 2: Kv=Kh=2.8*10^-6 ft/sec)



Material Input Properties

Name: Bedrock Model: Mohr-Coulomb Unit Weight: 160 pcf Cohesion: 2000 psf Phi: 45 °

Name: Native Soil Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion: 0 psf Phi: 30 °

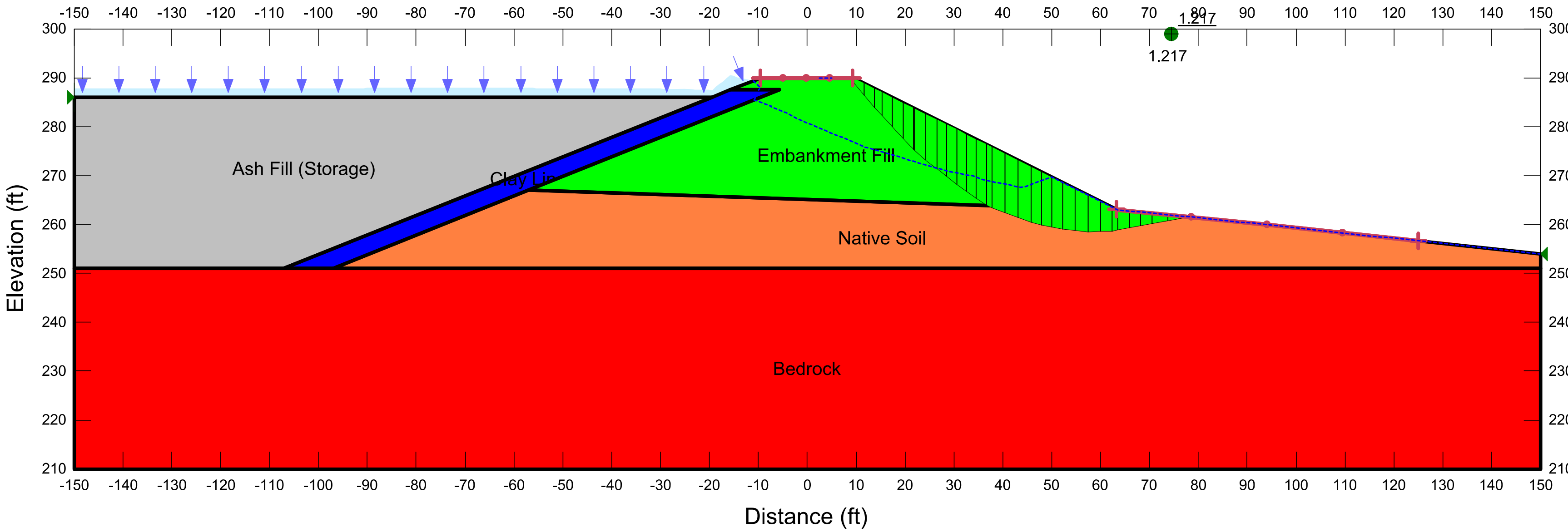
Name: Clay Liner Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion: 0 psf Phi: 30 °

Name: Ash Fill (Storage) Model: Mohr-Coulomb Unit Weight: 90 pcf Cohesion: 0 psf Phi: 30 °

Name: Embankment Fill Model: Mohr-Coulomb Unit Weight: 135 pcf Unit Wt. Above Water Table: 125 pcf Cohesion: 0 psf Phi: 37 °

Slope Stability (6)
Brunner Island Ash Basin No. 6
Station 21+80 (Section 1-1)
Manchester Township, Pennsylvania

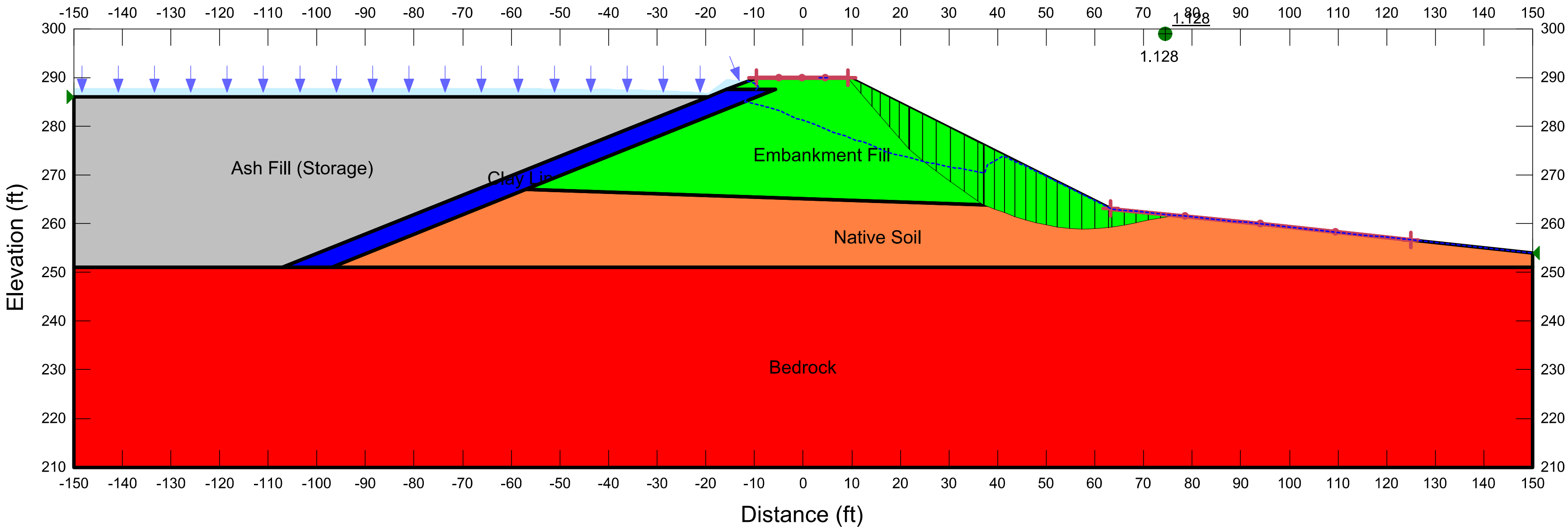
Plate E2c - River at Toe of Slope Elevation
(Case 2: Kv=Kh=2.8*10^-6 ft/sec)



Material Input Properties
Name: Bedrock Model: Mohr-Coulomb Unit Weight: 160 pcf Cohesion: 2000 psf Phi: 45 °
Name: Native Soil Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion: 0 psf Phi: 30 °
Name: Clay Liner Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion: 0 psf Phi: 30 °
Name: Ash Fill (Storage) Model: Mohr-Coulomb Unit Weight: 90 pcf Cohesion: 0 psf Phi: 30 °
Name: Embankment Fill Model: Mohr-Coulomb Unit Weight: 135 pcf Unit Wt. Above Water Table: 125 pcf Cohesion: 0 psf Phi: 37 °

Plate E2d - River at Normal Water Level Elevation
(Case 2: Kv=Kh=2.8*10^-6 ft/sec)

Slope Stability (6)
Brunner Island Ash Basin No. 6
Station 21+80 (Section 1-1)
Manchester Township, Pennsylvania



Material Input Properties

Name: Bedrock Model: Mohr-Coulomb Unit Weight: 160 pcf Cohesion: 2000 psf Phi: 45 °

Name: Native Soil Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion: 0 psf Phi: 30 °

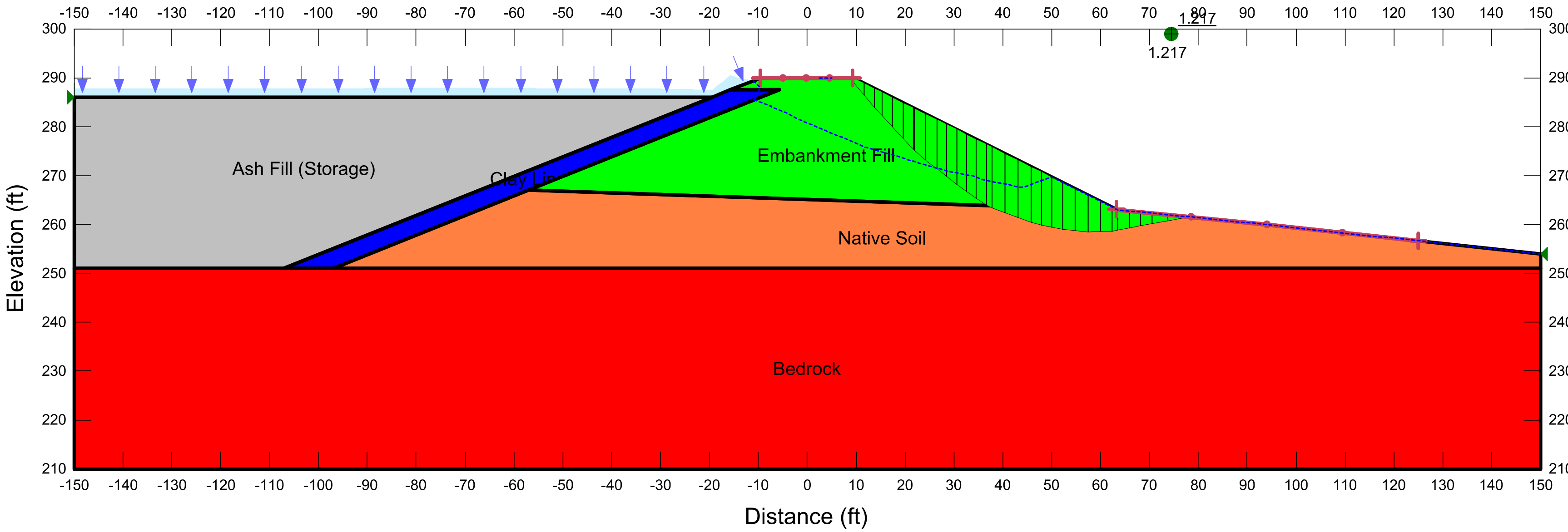
Name: Clay Liner Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion: 0 psf Phi: 30 °

Name: Ash Fill (Storage) Model: Mohr-Coulomb Unit Weight: 90 pcf Cohesion: 0 psf Phi: 30 °

Name: Embankment Fill Model: Mohr-Coulomb Unit Weight: 135 pcf Unit Wt. Above Water Table: 125 pcf Cohesion: 0 psf Phi: 37 °

Plate E3a - River at Normal Water Level Elevation
(Case 1: Kv=Kh=6.8*10^-6 ft/sec)

Slope Stability (6)
Brunner Island Ash Basin No. 6
Station 21+80 (Section 1-1)
Manchester Township, Pennsylvania



Material Input Properties

Name: Bedrock Model: Mohr-Coulomb Unit Weight: 160 pcf Cohesion: 2000 psf Phi: 45 °

Name: Native Soil Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion: 0 psf Phi: 30 °

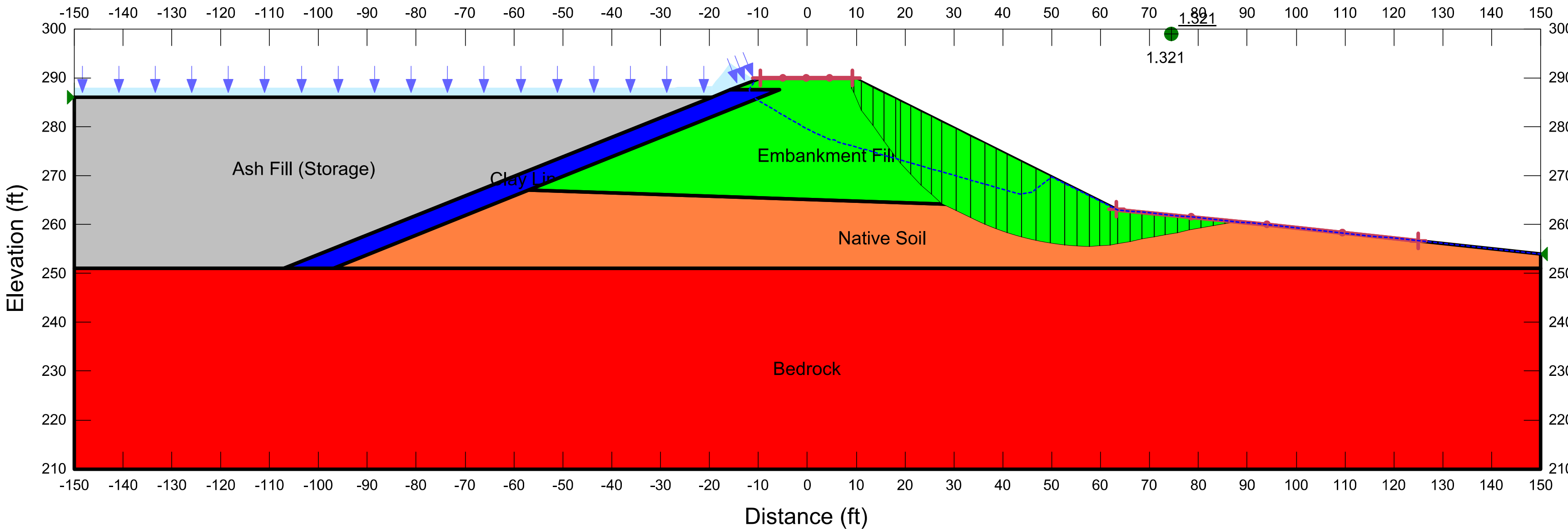
Name: Clay Liner Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion: 0 psf Phi: 30 °

Name: Ash Fill (Storage) Model: Mohr-Coulomb Unit Weight: 90 pcf Cohesion: 0 psf Phi: 30 °

Name: Embankment Fill Model: Mohr-Coulomb Unit Weight: 135 pcf Unit Wt. Above Water Table: 125 pcf Cohesion: 0 psf Phi: 37 °

Plate E3b - River at Normal Water Level Elevation
(Case 2: Kv=Kh=2.8*10^-6 ft/sec)

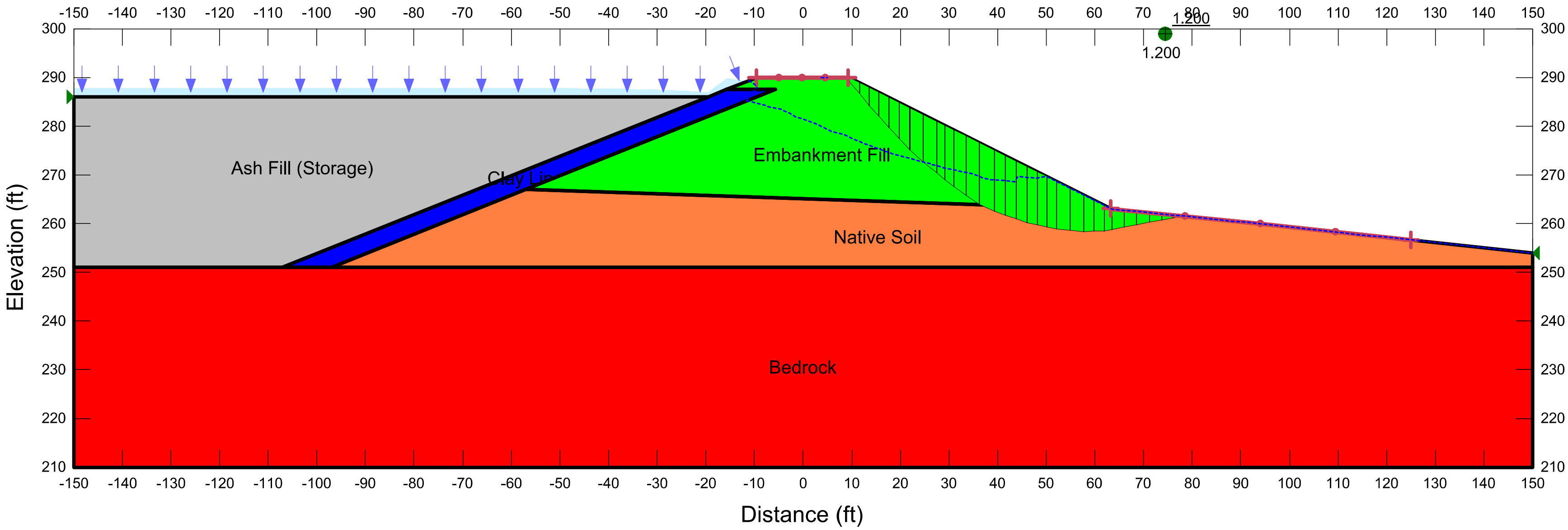
Slope Stability (6)
Brunner Island Ash Basin No. 6
Station 21+80 (Section 1-1)
Manchester Township, Pennsylvania



Material Input Properties
Name: Bedrock Model: Mohr-Coulomb Unit Weight: 160 pcf Cohesion: 2000 psf Phi: 45 °
Name: Native Soil Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion: 0 psf Phi: 30 °
Name: Clay Liner Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion: 0 psf Phi: 30 °
Name: Ash Fill (Storage) Model: Mohr-Coulomb Unit Weight: 90 pcf Cohesion: 0 psf Phi: 30 °
Name: Embankment Fill Model: Mohr-Coulomb Unit Weight: 135 pcf Unit Wt. Above Water Table: 125 pcf Cohesion: 0 psf Phi: 37 °

Plate E3c - River at Normal Water Level Elevation
(Case 3: Kv=Kh=6.8*10^-9 ft/sec)

Slope Stability (6)
Brunner Island Ash Basin No. 6
Station 21+80 (Section 1-1)
Manchester Township, Pennsylvania



Material Input Properties

Name: Bedrock Model: Mohr-Coulomb Unit Weight: 160 pcf Cohesion: 2000 psf Phi: 45 °

Name: Native Soil Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion: 0 psf Phi: 30 °

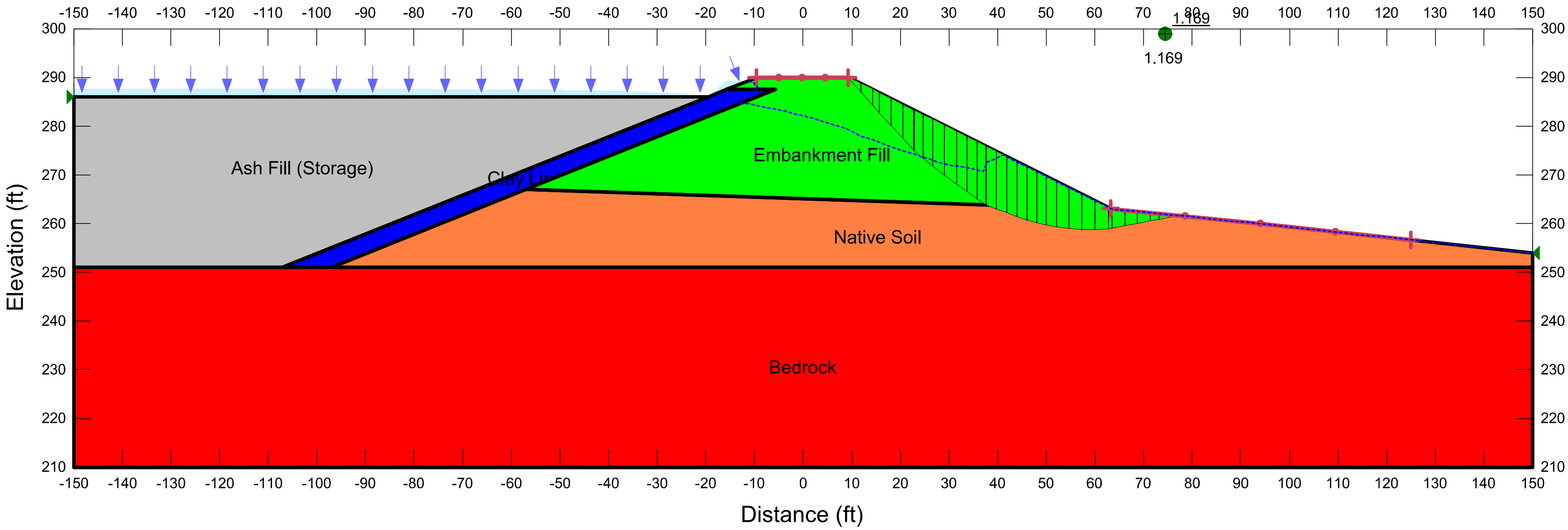
Name: Clay Liner Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion: 0 psf Phi: 30 °

Name: Ash Fill (Storage) Model: Mohr-Coulomb Unit Weight: 90 pcf Cohesion: 0 psf Phi: 30 °

Name: Embankment Fill Model: Mohr-Coulomb Unit Weight: 135 pcf Unit Wt. Above Water Table: 125 pcf Cohesion: 0 psf Phi: 37 °

Plate E3d - River at Normal Water Level Elevation
(Case 4: Kv=0.5*Kh=2.8*10^-6 ft/sec)

Slope Stability (6)
Brunner Island Ash Basin No. 6
Station 21+80 (Section 1-1)
Manchester Township, Pennsylvania



Material Input Properties

Name: Bedrock Model: Mohr-Coulomb Unit Weight: 160 pcf Cohesion: 2000 psf Phi: 45 °

Name: Native Soil Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion: 0 psf Phi: 30 °

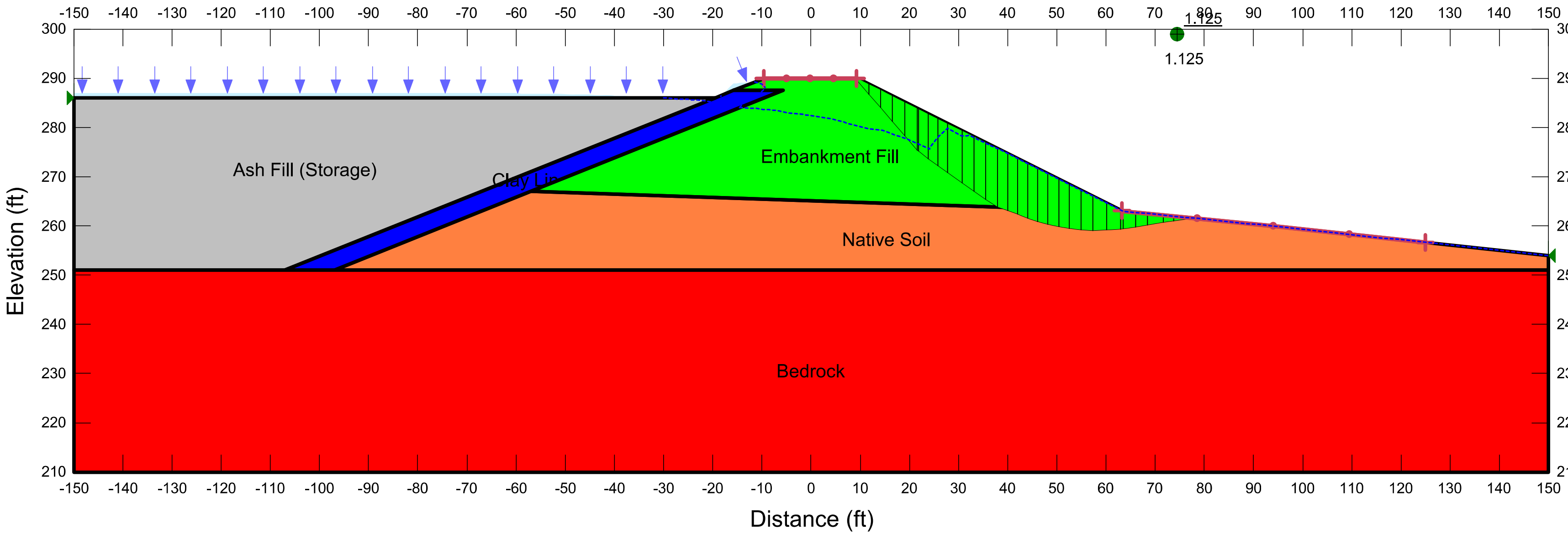
Name: Clay Liner Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion: 0 psf Phi: 30 °

Name: Ash Fill (Storage) Model: Mohr-Coulomb Unit Weight: 90 pcf Cohesion: 0 psf Phi: 30 °

Name: Embankment Fill Model: Mohr-Coulomb Unit Weight: 135 pcf Unit Wt. Above Water Table: 125 pcf Cohesion: 0 psf Phi: 37 °

Plate E3e - River at Normal Water Level Elevation
(Case 5: Kv=0.25*Kh=2.8*10^-6 ft/sec)

Slope Stability (6)
Brunner Island Ash Basin No. 6
Station 21+80 (Section 1-1)
Manchester Township, Pennsylvania



Material Input Properties

Name: Bedrock Model: Mohr-Coulomb Unit Weight: 160 pcf Cohesion: 2000 psf Phi: 45 °

Name: Native Soil Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion: 0 psf Phi: 30 °

Name: Clay Liner Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion: 0 psf Phi: 30 °

Name: Ash Fill (Storage) Model: Mohr-Coulomb Unit Weight: 90 pcf Cohesion: 0 psf Phi: 30 °

Name: Embankment Fill Model: Mohr-Coulomb Unit Weight: 135 pcf Unit Wt. Above Water Table: 125 pcf Cohesion: 0 psf Phi: 37 °

Plate E3f - River at Normal Water Level Elevation
(Case 6: Kv=0.13*Kh=2.8*10^-6 ft/sec)

Slope Stability (6)
Brunner Island Ash Basin No. 6
Station 21+80 (Section 1-1)
Manchester Township, Pennsylvania