PERIODIC STRUCTURAL STABILITY ASSESSMENT REPORT

ASH BASIN NO. 1
MONTOUR STEAM ELECTRIC STATION
DERRY TOWNSHIP, MONTOUR COUNTY, PENNSYLVANIA

Prepared for:

TALEN GENERATION, LLC
WASHINGTONVILLE, PENNSYLVANIA

Prepared by:

CIVIL & ENVIRONMENTAL CONSULTANTS, INC.

CEC Project 132-065.2006

November 2025

Civil & Environmental Consultants, Inc.

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1.0 PURPOSE

The purpose of this report is to present the results of the 2025 Periodic Structural Stability Assessment of the Montour Steam Electric Station (MSES) Ash Basin 1 (Basin 1). The previous Periodic Structural Stability Assessment was performed by CEC in November 2020. This assessment was performed in accordance with Section 257.73(d)(1) of the USEPA Coal Combustion Residuals (CCR) Rule and evaluated Basin 1 regarding the following.

- Stable foundations and abutments
- Slope protection
- Compaction of dike materials
- Dike Vegetation
- Spillway Adequacy
- Hydraulic structures underlying or passing through the dike
- Stability of downstream slopes after flooding

2.0 SITE DESCRIPTION

Montour, LLC (Montour) is a subsidiary of Talen Energy that operates a coal combustion residuals (CCR) management facility, known as Basin 1, at the Montour Steam Electric Station near Washingtonville, Pennsylvania. Basin 1 is regulated under the Pennsylvania Residual Waste Regulations Title 25 PA Code, Chapters 287 and 289. Basin 1 is permitted as a PADEP Class II Residual Waste Disposal Impoundment and is operated under Permit No. 301315. Basin 1 is also regulated by the PADEP Bureau of Waterways Engineering Division of Dam Safety under Permit No. 47-009 and under the National Pollutant Discharge Elimination System (NPDES) Permit No. PA0008443. All directions referenced herein refer to the plant north as shown on Figure 2.

Basin 1 went into service in 1971 and was developed by excavating site soils to construct an embankment dike around the excavation. The top of the dike is at Elevation 564 (Local Plant Elevation Datum, approximately NGVD 1929 + 0.15'). The perimeter of Basin 1 is approximately 11,000 feet in length and up to approximately 40-feet high. The dike ties into natural grade along the eastern side of the basin. Basin 1 is divided into Subbasins A, B, and C by internal dikes referred to as the Median Dike and the Splitter Dike as shown on Figure 2 in Appendix A. These internal dikes are not subject to the requirements of the structural stability assessment presented in the CCR Rule.

The CCR disposed in Basin 1 have historically included coal fly ash (ceased in 1982), coal bottom ash, Stabil-Fil (lime-amended fly ash), mill rejects, conditioned fly ash and bottom ash fines which were sluiced into the western portion of Subbasin B, which functioned as a settling basin. The MSES ceased burning coal in 2024 and Montour submitted the Notice of Intent to close Basin 1 in November 2024. Montour has initiated the closure of Basin 1 including site grading. Water is discharged from Subbasin C through a spillway consisting of a 36-inch reinforced concrete riser and culvert pipe to the on-site Detention Basin. Refer to Figures 1 and 2 in Appendix A for site location and layout.

3.0 DOCUMENT REVIEW

CEC reviewed documents provided by Montour related to Basin 1 construction and operation. Basin 1 has been inspected in accordance with the PADEP requirements for many years. The Initial Annual Inspection Report of Basin 1 in accordance with the CCR Rule was performed on June 11, 2015 by HDR Engineering, Inc. (HDR). Talen personnel prepared the annual inspection reports for 2017 through 2024. Geosyntec Consultants prepared a History of Construction Report of Basin 1 in October 2016 and Gannett Fleming prepared a revision in June 2021 in accordance with the CCR Rule. CEC prepared, reviewed, and used as references the following documents to assess the requirement in the CCR Rule.

- Initial Inflow Design Flood Control System Plan for Basin 1 – October 2016
- Periodic Inflow Design Flood Control System Plan for Basin 1 – November 2020
- Initial Safety Factor Assessment Report – October 2016
- Periodic Safety Factor Assessment Report – October 2021
- Initial Structural Stability Assessment Report – October 2016
- Periodic Structural Stability Assessment Report – November 2020

4.0 PERIODIC STRUCTURAL STABILITY SITE VISIT

On July 30, 2025, Mr. Jonathan Niemiec, P.E. of CEC performed a site visit to observe the conditions of Basin 1 as it relates to the structural stability assessment required by the CCR Rule. A comprehensive site walk of the entire basin and discussions with Montour personnel were performed during this visit. Select photographs taken during this site visit are included in Appendix B and the approximate photograph locations and viewing directions are shown on Figure 2 in Appendix A.

5.0 STRUCTURAL STABILITY CRITERIA

The following sections address the requirements in Section 257.73(d)(1) of the CCR Rule.

5.1 STABLE FOUNDATIONS AND ABUTMENTS

Based on CEC's site visit, and the documents reviewed, CEC concludes that the dike foundations and abutments appear to be stable. In accordance with Section 257.83(a)(i) of the CCR Rule, Montour monitors the dike slopes, foundations, and abutments for signs of instability on a weekly basis.

According to Section 3.5 of the Basin 1 History of Construction Report, the Basin 1 perimeter dike was primarily founded on bedrock consisting of weathered shale. The perimeter dike ties into natural existing grade at the northeastern and southeastern corners of the basin. According to the History of Construction Report, the abutment material at these locations consists of residual soils overlying weathered shale. A view of these northeastern and southeastern dike corners are shown in Photographs 1 and 2 in Appendix B.

The seep that was observed during CEC's site visit in June 2016 at the toe of the northern dike slope, just upstream of the seepage collection system was not observed during our October 2020 or our July 2025 site visit. According to our conversations with Montour, this seep occurred due to a blockage in the seepage collection system which was cleaned out in October 2016. A view of this area from our July 2025 site visit can be seen in Photograph 3 in Appendix B.

Ponding water was observed between the existing rail line and the toe of the western dike embankment, south of the pipe bridge, at the time of our site visit as shown in Photographs 5 and 6 in Appendix B. Ponding water was also observed at the toe of the western dike embankment, north of the pipe bridge as shown in Photograph 7. No signs of slope instability were observed along the western dike embankment. A rock buttress was constructed in 2007 along the western downstream dike slope in the area of the pipe bridge to increase stability.

5.2 SLOPE PROTECTION

CCR were placed to the top of the dike elevation which covers the upstream side of the dike in Subbasin A (see Photographs 8 and 9) and most of Subbasin B except for the western portion which are exposed (see Photographs 10 and 22). The interior dike slopes in Subbasin C are also exposed as shown in Photographs

13, 14, and 15. The exposed upstream slopes in these two areas are mostly covered with vegetation to protect against erosion. Section 3.6.4 of the History of Construction Report states that a berm was constructed along the perimeter dike to protect against erosion from wave action and was constructed of reclaimed bottom ash and coal mill rejects. Our observations and topography indicates that this berm has remained in place. Refer to Section 5.4 of this report for more information on dike vegetation.

5.3 COMPACTION OF DIKE MATERIALS

The History of Construction Report states that based on Drawing G-199944-11 by Ebasco Services, Inc. dated March 28, 1968, the materials used to construct the dike were to be compacted to at least 95% of the maximum dry density based on the standard Proctor (ASTM D698). The Safety Factor Assessment Reports indicate that the dike materials are adequate to withstand the range of loading conditions expected to be experienced by the dike. The conditions of the dike materials used in the Safety Factor Assessment Reports were based on field and laboratory testing data obtained during CEC's 2015 subsurface investigation and from previous subsurface investigations. CEC did not make any observations at the site in July 2025 that would suggest the dike materials have changed since our most recent Safety Factor Assessment.

5.4 DIKE VEGETATION

The CCR Rule states that the vegetation on the dikes and surrounding areas shall not exceed 6 inches above the slope of the dike, except for slopes which have an alternate form or forms of slope protection. Montour's vegetation control program calls for cutting vegetation on the dike slopes at least three times a year during the growing season. In accordance with Section 257.83(a)(i) of the CCR Rule, Montour performs weekly inspections of the dike slopes. During these inspections the condition of the vegetation is documented and any issues reported are promptly addressed. Mowing of a portion of the dike slopes was being performed during CEC's site visit in July as shown in Photograph 11 in Appendix B.

At the time of CEC's site visit, downstream dike slopes were mostly covered with grassy vegetation excluding the areas where riprap has been placed and where the overtopping protection was installed. CEC observed 12 animal burrow holes, each approximately 6 to 12 inches in diameter along the downstream dike slopes. These holes were present along the southern dike and southern portion of the western dike. One of these holes is shown in Photograph 4 in Appendix B. Based on our observations and conversations with Montour, animal burrow holes are filled on a regular basis as part of their normal maintenance and the holes observed by CEC will be filled in. On November 7, 2025, Talen informed CEC that 42 animal burrows along the dike slopes at Basin 1were recently filled with grout. The downstream dike slopes vegetation was generally in accordance with the Rule requirements. Larger vegetation such as shrubs or trees were not present on the downstream dike slopes. Photographs 1, 2, 3, 4, 7, 11, 12, 16, and 23 show the condition of the vegetation on the downstream dike slopes at the time of our site visit.

5.5 SPILLWAY ADEQUACY

The spillway is located in Subbasin C and consists of a 36-inch reinforced concrete riser and culvert pipe. This spillway discharges into the on-site Detention Basin which is then piped to discharge to the Middle Branch Susquehanna River.

Montour installed overtopping protection on the northwestern portion of the Subbasin C dike that consists of lowering and armoring with articulating concrete block a section of the dike approximately 150-feet wide to Elevation 562 which was approved by the Pennsylvania Department of Environmental Protection (PADEP) Division of Dam Safety and as a Minor Modification to the Solid Waste Permit No. 301315. The installation of this overflow section increased the discharge capacity of the basin to adequately manage the CCR Rule design storm, which is the Probable Maximum Flood (PMF) for Ash Basin No. 1. This design is presented in the Periodic Inflow Design Flood Control System Plan by Gannett Fleming, Inc dated November 2020. Photographs 24 and 25 show the overtopping protection. The construction was completed in 2020.

Subsequent to CEC's site visit in July 2025, a remotely operated vehicle (ROV) was used to inspect the spillway pipe in Subbasin C. CEC reviewed the videos provided by Montour. No obstructions were observed within the pipe.

5.6 HYDRAULIC STRUCTURES UNDERLYING OR PASSING THROUGH THE DIKE

The integrity of the spillway discharge pipe located in Subbasin C is inspected as part of the annual inspections. Based on the photographs provided by Montour taken at the time the pipe was dewatered and the blockage was removed in 2016, the pipe integrity appeared to be in very good condition. An inspection performed in submerged conditions of this pipe was performed on September 16, 2025 using an ROV. CEC reviewed this video and observed no indication of structural deformation.

Two abandoned reinforced concrete pipe culverts are present beneath the dike on the north side of Basin 1 as shown on Figure 2. The outlet of the western plugged culvert is exposed and is shown in Photograph 17 in Appendix B. According to the Initial Annual Inspection Report, this pipe was inspected from the downstream end with a ROV by Montour in 2014. A concrete plug was encountered during the inspection approximately 59 feet from the outlet end. Drawing G-199945-13 by Ebasco Services, Inc. dated March 15, 1968 indicates that the eastern plugged culvert was temporarily installed to allow flow of an existing creek through the dike embankment, most likely during construction. The exact location of the eastern plugged culvert is unknown.

Six 15- to 24-inch HDPE stormwater pipes pass through the dike in Subbasin A. These pipes were previously capped prior to placing conditioned fly ash (CFA) in Basin 1 and to direct all surface water run-off to Subbasin C. These pipes were permanently plugged by filling with grout in accordance with the PADEP Subbasin A Stormwater Conduits Maintenance approval letter dated April 14, 2025 and the field work was completed on September 12, 2025.

In 1973, a seepage collection system was installed on the northwestern side of the basin for collecting seepage water and conveying it back to Basin 1. In 1979, the system was extended farther to the northeast and an additional pump station was added to convey the seepage water back to the basin. The collection system consists of a buried interceptor trench at the downstream toe of the northern dike. The trench contains a pipe that is sloped to convey water to four manholes positioned along its length. The manholes are equipped with submersible pumps that operate via level controls to pump the accumulated water back into the basin. Pipes pass through the northern dike to convey pumped water from the seepage collection system to Subbasin B. The pumps were not running during our site visit.

5.7 STABILITY OF DOWNSTREAM SLOPES AFTER FLOODING

Based on the Montour SES Lake Chillisquaque Dambreak Analysis dated November 1999, inundation of a portion of the northern dike slope adjacent to the Chillisquaque Creek is possible if a dam breach should occur to the reservoir north of Ash Basin No. 1. Therefore, as part of the Initial Structural Stability Assessment CEC evaluated the stability of the exterior embankment at Cross Section 1-1 considering a rapid drawdown scenario of the maximum flood elevation. Figure 2 in Appendix A shows the location of Cross Section 1-1. As part of the Initial Structural Stability Assessment, CEC evaluated the stability of the exterior embankment at Cross Section 1-1 considering a rapid drawdown scenario of the maximum flood elevation. Figure 2 in Appendix A shows the location of Cross Section 1-1. The stability of this cross section under this scenario was not re-evaluated as part of this Periodic Structural Stability Assessment because of the variables that affect the outcome of this analysis have not changed since it was evaluated as part of the Initial Structural Stability Assessment.

CEC reviewed the Federal Emergency Management Agency (FEMA) flood insurance rate map and the Montour SES Lake Chillisquaque Dambreak Analysis dated November 1999. The Dambreak Analysis reported a maximum flood level at the MSES (located approximately 2.4 miles downstream of the dam) of approximately Elevation 528. The FEMA map reports a flood elevation of approximately Elevation 524 at the location of Basin 1. Elevation 528 was used in our analysis, which is higher and therefore conservative. The dike does not extend down to Elevation 528 at the other critical cross sections that were evaluated as part of the Initial Safety Factor Assessment, so they were not evaluated for rapid drawdown. The FEMA flood map and an excerpt from the dambreak analysis are included in Appendix C.

Section 257.73(e) of the CCR Rule does not specify a minimum FS for rapid drawdown. However, the regulations suggest that this evaluation be completed, if applicable. The US Army Corps of Engineers (USACE) Engineering Manual EM 1110-2-1902 “Slope Stability” (October 2003) recommends a minimum FS of 1.1 (drawdown from maximum surcharge pool) and 1.3 (drawdown from maximum storage pool). The maximum water level used in the analysis is an extreme event (dam breach under the probable maximum precipitation event) so the lower FS is recommended. Based on our analysis, a FS of 1.4 was calculated for this drawdown scenario. Refer to the Basin 1 Safety Factor Assessment Reports for more information regarding the subsurface conditions and analysis methodology.

6.0 CONCLUSIONS

Based on our site visit and document review, CEC concludes that Basin 1 meets the criteria outlined in Section 257.73(d)(1) of the CCR Rule. No structural stability deficiencies were observed and no corrective measures are recommended.

7.0 CERTIFICATION

The following is provided in accordance with Section 257.73(d)(3) of the CCR Rule.

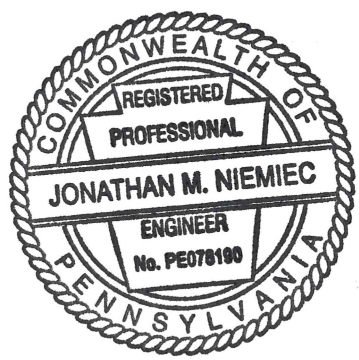
By affixing my seal to this, I do hereby certify to the best of my knowledge, information, and belief that the information contained in this report is true and correct. I further certify I am licensed to practice in the Commonwealth of Pennsylvania and that it is within my professional expertise to verify the correctness of the information. I am aware that there are significant penalties for submitting false information, including the possibility of fines and imprisonment.

Jonathan M. Niemiec, P.E.

P.E. License Number: PE078190

Signature: ______________________

Date: 11/11/25

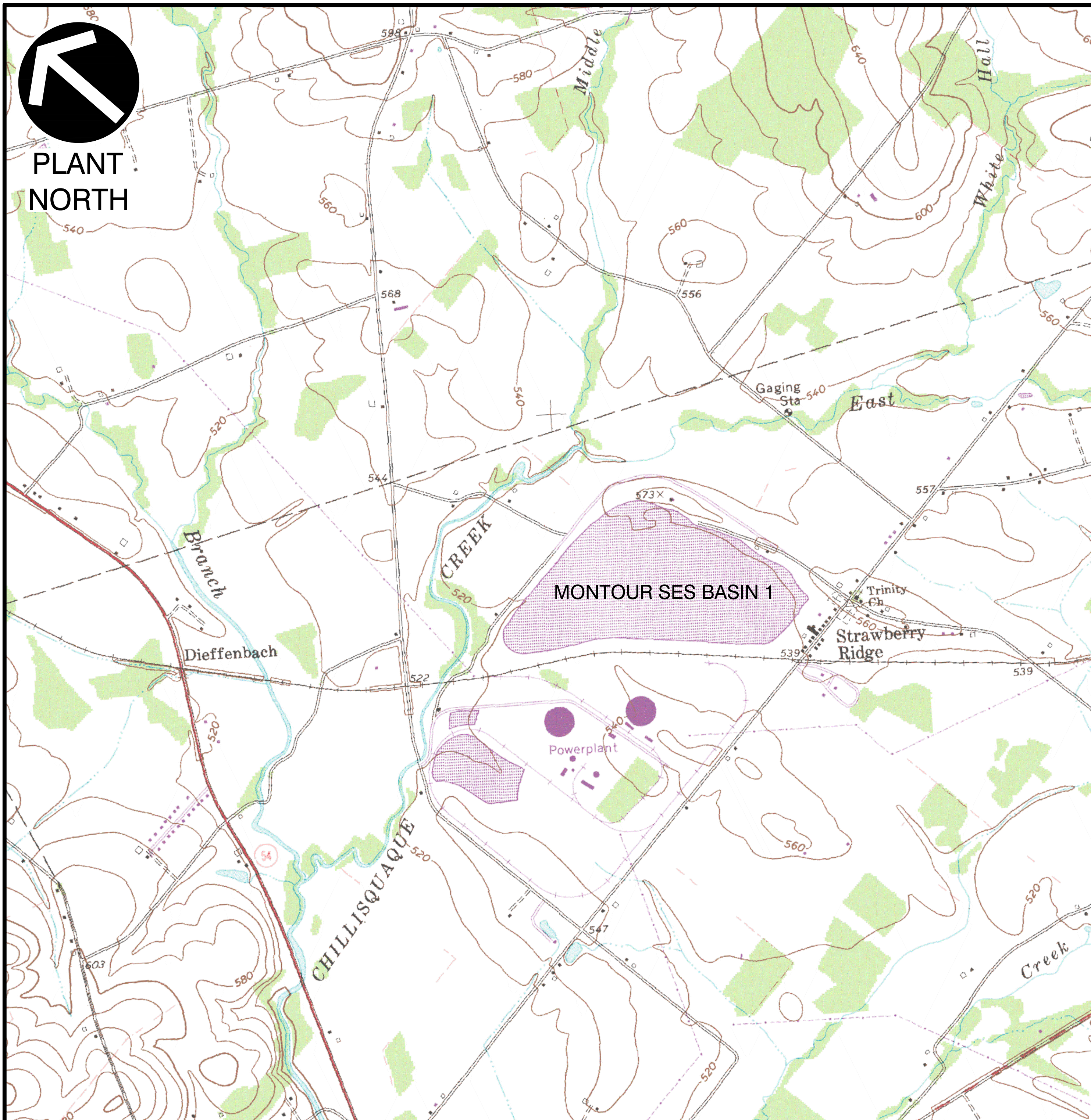


8.0 REFERENCES

1. HDR Engineering, Inc. January 2016. Initial Annual Inspection Report of Basin 1.
2. Geosyntec Consultants. October 2016. History of Construction Report.
3. November 1999. Montour SES Lake Chillisquaque Dambreak Analysis.
4. Civil & Environmental Consultants, Inc. October 2016. Initial Inflow Design Flood Control System Plan Montour Ash Basin No. 1.
5. Civil & Environmental Consultants, Inc. October 2016. Initial Safety Factor Assessment Report.
6. Civil & Environmental Consultants, Inc. October 2021. Periodic Safety Factor Assessment Report.
7. Civil & Environmental Consultants, Inc. October 2016. Initial Structural Stability Assessment Report.
8. Civil & Environmental Consultants, Inc. November 2020. Periodic Structural Stability Assessment Report.
9. Talen Energy. January 2022. 2021 USEPA CCR Surface Impoundment Annual Inspection Report Montour Steam Electric Station Ash Basin No. 1.
10. Talen Energy. January 2023. 2022 USEPA CCR Surface Impoundment Annual Inspection Report Montour Steam Electric Station Ash Basin No. 1.
11. Talen Energy. January 2024. 2023 USEPA CCR Surface Impoundment Annual Inspection Report Montour Steam Electric Station Ash Basin No. 1.
12. Talen Energy. November 2024. 2024 USEPA CCR Surface Impoundment Annual Inspection Report Montour Steam Electric Station Ash Basin No. 1.
13. Gannett Fleming. November 2020. Periodic Inflow Design Flood Control System Plan.

APPENDIX A

FIGURES



REFERENCE

1. U.S.G.S. 7.5' TOPOGRAPHIC MAP, WASHINGTONVILLE QUADRANGLE, PA DATED: 1969, PHOTOREVISED: 1977. PHOTOINSPECTED: 1983.

SCALE IN FEET

0 2000 4000

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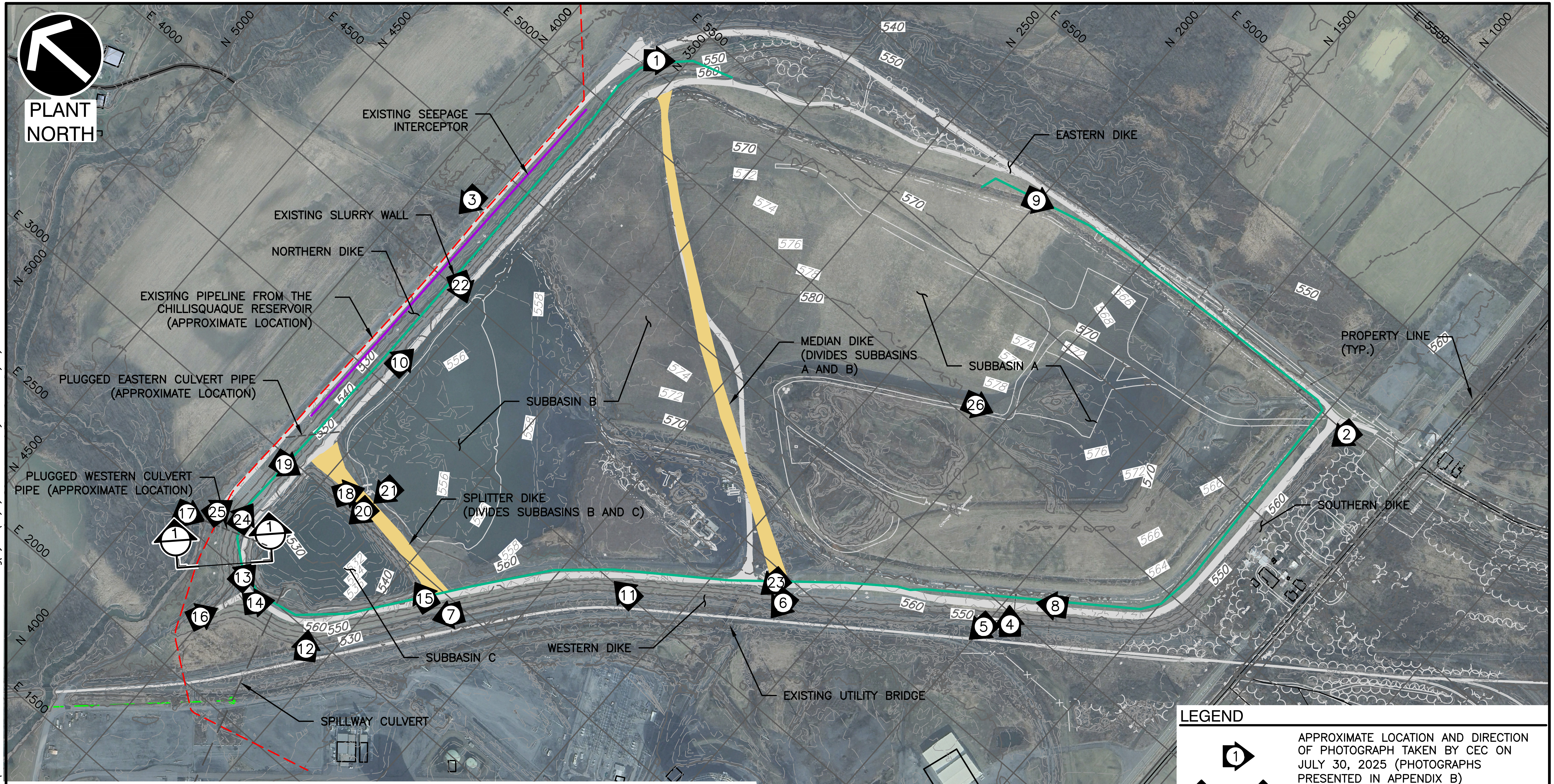
MONTOUR, LLC.
STRUCTURAL STABILITY ASSESSMENT
MONTOUR SES
WASHINGTONVILLE, PA

SITE LOCATION MAP

DRAWN BY: DWD	CHECKED BY: JMN	APPROVED BY: RJB	FIGURE NO.: 1
DATE: NOVEMBER 2025	DWG SCALE: 1"=2000'	PROJECT NO: 132-065.2006	



REFERENCES

1. THE EXISTING CONDITIONS TOPOGRAPHY IN THE PROPOSED BORROW AREA PROVIDED BY BORTON–LAWSON USING CONTOURS GENERATED FROM AN UNMANNED AERIAL SYSTEM (UAS) SURVEY ON APRIL 4, 2024. THE EXISTING CONDITIONS TOPOGRAPHY IN BASIN NO. 1 PROVIDED BY BORTON–LAWSON USING CONTOURS GENERATED FROM AN UAS ON MARCH 29, 2022. THE TOPOGRAPHY IS PROVIDED IN PLANT HORIZONTAL COORDINATES AND VERTICAL DATUM. THE EXISTING CONDITIONS OUTSIDE OF THESE AREAS IS FROM THE PAMAP PROGRAM LIDAR DATA AT 2–FEET INTERVALS DATED 2006 AND 2008.
2. PROPERTY BOUNDARIES PROVIDED BY TALEN ENERGY AND MODIFIED BY CIVIL & ENVIRONMENTAL CONSULTANTS, INC. TO SHOW CURRENT PROPERTY OWNERSHIP.
3. THE LIMITS OF THE BASIN NO. 1 PERMIT AREA IS FROM PPL DRAWING E244381, DATED 9/2007. THE APPROXIMATE TOP OF DIKE/LIMIT OF CCR SHOWN IS AN ESTIMATED LOCATION.
4. IMAGE TAKE FROM GOOGLE EARTH PRO, DATE OF AERIAL MARCH 7, 2024.

LEGEND

1 — APPROXIMATE LOCATION AND DIRECTION OF PHOTOGRAPH TAKEN BY CEC ON JULY 30, 2025 (PHOTOGRAPHS PRESENTED IN APPENDIX B)

1 — 1 — CROSS SECTION LOCATION

SCALE IN FEET

0 400 800

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PHOTOGRAPH LOCATION PLAN

DRAWN BY:	DWD	CHECKED BY:	JMN	APPROVED BY:	RJB	FIGURE NO.:
DATE:	NOVEMBER 2025	DWG SCALE:	1"=400'	PROJECT NO:	132-065.2006	2

P:\2013\132-065\-CADD\Dwg\GT05-Task 2006\132065-GT05-PHOTOGRAPH LOCATION PLAN.dwg[2] LS:(10/1/2025 - ddiable) - LP: 11/11/2025 10:18 AM

APPENDIX B

PHOTOGRAPHS

Photograph 1 – Northeast corner downstream dike slope

Photograph 2 – Southern dike slope and crest

Photograph 3 – Area of previously observed seep along northern dike slope

Photograph 4 – View of animal burrow hole on western dike slope

Photograph 5 – Evidence of ponding water at toe of western dike slope

Photograph 6 – Iron oxidizing bacteria was observed in the sanding water at the toe of the western dike slope

Photograph 7 – Ponding water at western dike slope toe, north of the pipe bridge

Photograph 8 – Western dike embankment crest and perimeter channel

Photograph 9 – Eastern dike crest and perimeter channel

Photograph 10 – Northern dike crest and upstream embankment

Photograph 11 – Mowing of western dike slope

Photograph 12 – Sparse vegetation on southern dike slope

Photograph 13 – Subbasin C western upstream dike slope

Photograph 14 – Subbasin C western and southern upstream dike slopes

Photograph 15 – Subbasin B southeastern slope

Photograph 16 – Southwestern dike slope and access road

Photograph 17 –Abandoned, plugged overflow culvert pipe outlet

Photograph 18 – Subbasin B staff gage

Photograph 19 – Subbasin C staff gage

Photograph 20 – Subbasin B intake pipe with trash rack

Photograph 21 – Subbasin B to C culvert outlet

Photograph 22 – View of Subbasin B

Photograph 23 – Utility bridge

Photograph 24 – Overtopping protection installed at Subbasin C

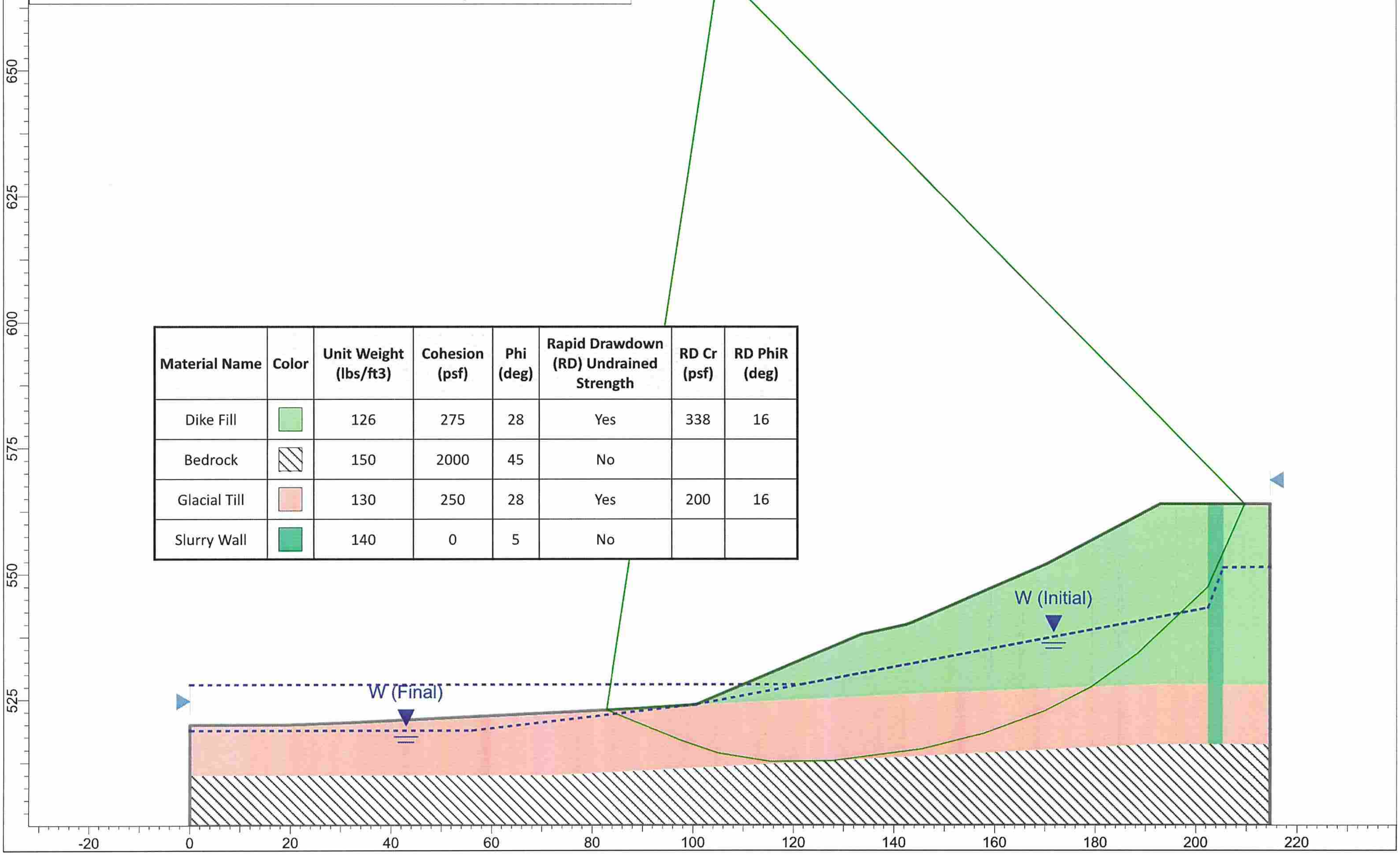
Photograph 25 – Overtopping protection installed at Subbasin C

Photograph 26 – Fill placement and grading in Subbasin A

APPENDIX C

RAPID DRAWDOWN ANALYSIS OUTPUT AND REFERENCES

Montour SES Basin 1
Initial Safety Factor Assessment
Cross Section 1-1 -- Downstream Rapid Drawdown



Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)	Rapid Drawdown (RD) Undrained Strength	RD Cr (psf)	RD PhiR (deg)
Dike Fill		126	275	28	Yes	338	16
Bedrock		150	2000	45	No		
Glacial Till		130	250	28	Yes	200	16
Slurry Wall		140	0	5	No		

EM 1110-2-1902
31 Oct 2003

US Army Corps
of Engineers®

ENGINEERING AND DESIGN

Slope Stability

ENGINEER MANUAL

Table 3-1
Minimum Required Factors of Safety: New Earth and Rock-Fill Dams

Analysis Condition[1]	Required Minimum Factor of Safety	Slope
End-of-Construction (including staged construction)[2]	1.3	Upstream and Downstream
Long-term (Steady seepage, maximum storage pool, spillway crest or top of gates)	1.5	Downstream
Maximum surcharge pool[3]	1.4	Downstream
Rapid drawdown	1.1-1.3[4,5]	Upstream

[1] For earthquake loading, see ER 1110-2-1806 for guidance. An Engineer Circular, "Dynamic Analysis of Embankment Dams," is still in preparation.

[2] For embankments over 50 feet high on soft foundations and for embankments that will be subjected to pool loading during construction, a higher minimum end-of-construction factor of safety may be appropriate.

[3] Pool thrust from maximum surcharge level. Pore pressures are usually taken as those developed under steady-state seepage at maximum storage pool. However, for pervious foundations with no positive cutoff steady-state seepage may develop under maximum surcharge pool.

[4] Factor of safety (FS) to be used with improved method of analysis described in Appendix G.

[5] FS = 1.1 applies to drawdown from maximum surcharge pool; FS = 1.3 applies to drawdown from maximum storage pool.

For dams used in pump storage schemes or similar applications where rapid drawdown is a routine operating condition, higher factors of safety, e.g., 1.4-1.5, are appropriate. If consequences of an upstream failure are great, such as blockage of the outlet works resulting in a potential catastrophic failure, higher factors of safety should be considered.

(1) During construction of embankments, materials should be examined to ensure that they are consistent with the materials on which the design was based. Records of compaction, moisture, and density for fill materials should be compared with the compaction conditions on which the undrained shear strengths used in stability analyses were based.

(2) Particular attention should be given to determining if field compaction moisture contents of cohesive materials are significantly higher or dry unit weights are significantly lower than values on which design strengths were based. If so, undrained (UU, Q) shear strengths may be lower than the values used for design, and end-of-construction stability should be reevaluated. Undisturbed samples of cohesive materials should be taken during construction and unconsolidated-undrained (UU, Q) tests should be performed to verify end-of-construction stability.

d. Pore water pressure. Seepage analyses (flow nets or numerical analyses) should be performed to estimate pore water pressures for use in long-term stability computations. During operation of the reservoir, especially during initial filling and as each new record pool is experienced, an appropriate monitoring and evaluation program must be carried out. This is imperative to identify unexpected seepage conditions, abnormally high piezometric levels, and unexpected deformations or rates of deformations. As the reservoir is brought up and as higher pools are experienced, trends of piezometric levels versus reservoir stage can be used to project piezometric levels for maximum storage and maximum surcharge pool levels. This allows comparison of anticipated actual performance to the piezometric levels assumed during original design studies and analysis. These projections provide a firm basis to assess the stability of the downstream slope of the dam for future maximum loading conditions. If this process indicates that pore water pressures will be higher than those used in design stability analyses, additional analyses should be performed to verify long-term stability.

e. Loads on slopes. Loads imposed on slopes, such as those resulting from structures, vehicles, stored materials, etc. should be accounted for in stability analyses.

EXCERPT FROM: MONTOUR SES LAKE CHILLASQUAQUE DAMBREAK ANALYSIS

MONTOUR SES
LAKE CHILLASQUAQUE
DAMBREAK ANALYSIS

PP&L, INC.
2 N. 9TH STREET
ALLENTOWN, PA 18101

NOVEMBER 1999

PREPARED BY: G.D. Hopper
SENIOR ENGINEER

138 Sheets

7

MONTOUR SES
LAKE CHILLASQUAQUE
DAMBREAK STUDY

1. Description of Dam

The dam is a 54' high by 2,000 feet long earthfill structure built in 1971 to impound a make-up water supply for PP&L's Montour Steam Electric Station. The crest elevation is 605.0 feet. The maximum reservoir volume is 4,400 acre-feet. The outlet facilities for normal operation include an 8-inch bypass valve, an 18-inch Howell-Bunger valve and a 36-inch sluice gate. There is a 750-foot long emergency spillway channel with outlet crest at elevation 600 feet.

Normal operating level of the lake is elevation 595 feet. Lake level operating limit is elevation 596.5 to 596.7 feet, per agreement with the Susquehanna River Basin Commission. The lake is used as a plant water supply only when there is low flow in the Susquehanna River or when the river intake pumps are out of service.

The dam can be located on the Washingtonville quadrangle of the USGS topographic map 7.5-minute series for the Commonwealth of Pennsylvania.

PP&L owns the dam and regularly inspects it under its inspection program.

2. Description of Watershed

The study area is comprised of the Chillasquaque Creek watershed located primarily in Montour County, Pennsylvania. The watershed is a 112 square mile rural area draining to the west branch of the Susquehanna River. The upper reaches of Chillasquaque Creek include a west, middle and east branch. Lake Chillasquaque is a 185-acre man-made impoundment located on the middle branch.

On its journey to the Susquehanna, the Chillasquaque Creek passes PP&L's Montour power station and the towns of Washingtonville, Pottsgrove, and Chillasquaque. The portion of creek that serves as the receiving stream for Lake Chillasquaque travels 19.6 miles and passes under 20 bridges including the Interstate 80 bridge. In addition to the east and west branches in the upper watershed, two other major tributaries join the creek further downstream: Mud Creek, at the town of Washingtonville, and Beaver Run, just south of Pottsgrove.

The watershed is comprised of a mixture of gently rolling rural terrain and a few mountain ridges. Most of its soils are classified as hydrologic soil type "C".

3. Stream Hydraulics and Controls

The receiving channel has a channel slope ranging from 20 feet per mile (fpm) in the upper reaches to 2 ½ fpm further downstream, with an average of 6 fpm. The entire stream slope is subcritical. The downstream control is the Susquehanna River level.

Aside from the overall channel size, the appearance of the stream is fairly consistent over its entire length: a defined rocky earthen channel with moderately vegetated overbanks. Manning's n-values used in the program range from .030 for the main channel to .040- .050 for the overbanks.

4. Approach to Modeling

Streamflow and water surface profiles through the above-described stream network are computed for "with-break" and "without break" conditions for the PMF, 100-year and sunny day background conditions. The modeling program used is the National Weather Service DAMBRK computer program. Modeling "Option 12" is the selected option: simultaneous dynamic computation for multiple dams and/or bridges.

A runoff hydrograph for the lake's inflow is computed using the "Pondpack" program. The SCS dimensionless unit hydrograph method, utilizing watershed CN runoff coefficients, times of concentration, and appropriate rainfall amounts is the selected hydrologic option. The highest expected normal lake level (596.7) is used as the starting lake level.

Runoff hydrographs for five additional subwatersheds are computed using Pondpack and introduced to the main creek channel as lateral inflows. For the sunny day condition, only nominal average flows are considered throughout the stream network.

The 20 bridges were examined during field inspections. Seven of the more significant bridge contractions were selected for modeling in the DAMBRK program and are included in all of the runs.

The downstream control of Chillasquaque Creek is taken as the Susquehanna River level at 10-year flood stage.

5. Breach Characteristics

In all cases a 100' wide breach with 1H:2V side slopes is the assumed failure shape in the 2,000-ft long earthen embankment. Average breach width is 126 ft., or 2.33 times the height of the dam. The trigger elevation for the breach is the highest pond elevation computed during the "without breach" modeling runs. Time of complete breach formation is taken as 45 minutes in all cases. The breach is assumed to progress down to El. 560, which is the bottom of the dam.

6. Discussion of Results

Water surface elevations and stream flows computed for the respective "break" and "no break" conditions are summarized on the attached charts.

As expected, the "with break" flows for the PMF background condition produce the highest overall water surface elevations. Immediately downstream of the Lake Chillasquaque dam, the additional flood surge resulting from the PMF dambreak amounts to 70,000 cfs and a 7 ft. rise in stream level. The surge quickly attenuates to a 1 ½' rise in stream level at Washingtonville and a little more than a 1' rise at Pottsgrove. At the mouth of the Chillasquaque, the flow surge from this dambreak decreases to approximately 10,000 cfs.

The time to peak flow at downstream locations, as measured from the beginning of the dam break formation for the PMF background condition, is as follows:

Location	Time (hh:mm)
Montour power plant	1:19
Washingtonville, Rt. 54	2:13
Route I80	3:56
Chillasquaque village, Rt. 147	6:34
Susquehanna River	7:17

For the 100-year and sunny day background conditions, the overall dambreak flood levels are less than those for the PMF background condition, but the amount of water level rise due to the dambreak is greater than for the PMF background condition. For all three background conditions, the amount of water level rise due to the dambreak dissipates to less than one foot between the towns of Pottsgrove and Chillasquaque.

7. Effect on Population

The attached map delineates the inundated area for the PMF background condition superimposed with the dambreak flood surge.

PP&L has several buildings immediately downstream of the dam. These would be severely flooded. The Montour power station is outside of the flood zone.

Low-lying portions of the town of Washingtonville would be covered with slightly greater than an additional foot of water as a result of the dambreak.

Approximately 1/3 of the town of Pottsgrove would be covered with an additional foot of water.

Low-lying portions of the town of Chillasquaque would see an additional few inches of water as a result of the dambreak.

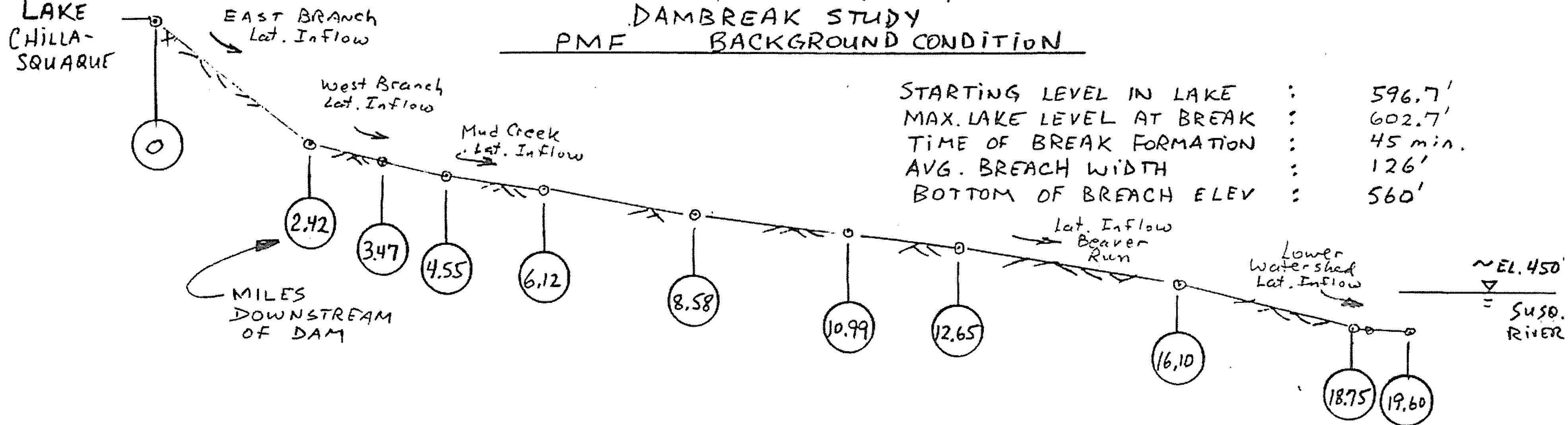
Most of the bridge decks crossing Chillasquaque Creek would be flooded under PMF conditions even without a dambreak. With a PMF dambreak, only two additional bridge decks (immediately downstream of the lake) would be flooded. For the 100-year flood background condition, seven of the bridge decks (in the upper watershed) would be flooded due to the dambreak whereas none is flooded with no dambreak. For sunny day background condition, none of the bridge decks is flooded with or without a dambreak. A table of flood levels for each of the 20 bridge decks is included in the report.

8. Conclusion

It is proposed to use the PMF plus dambreak flooding levels as the basis in preparing the inundation map for the Lake Chillasquaque dambreak emergency action plan.

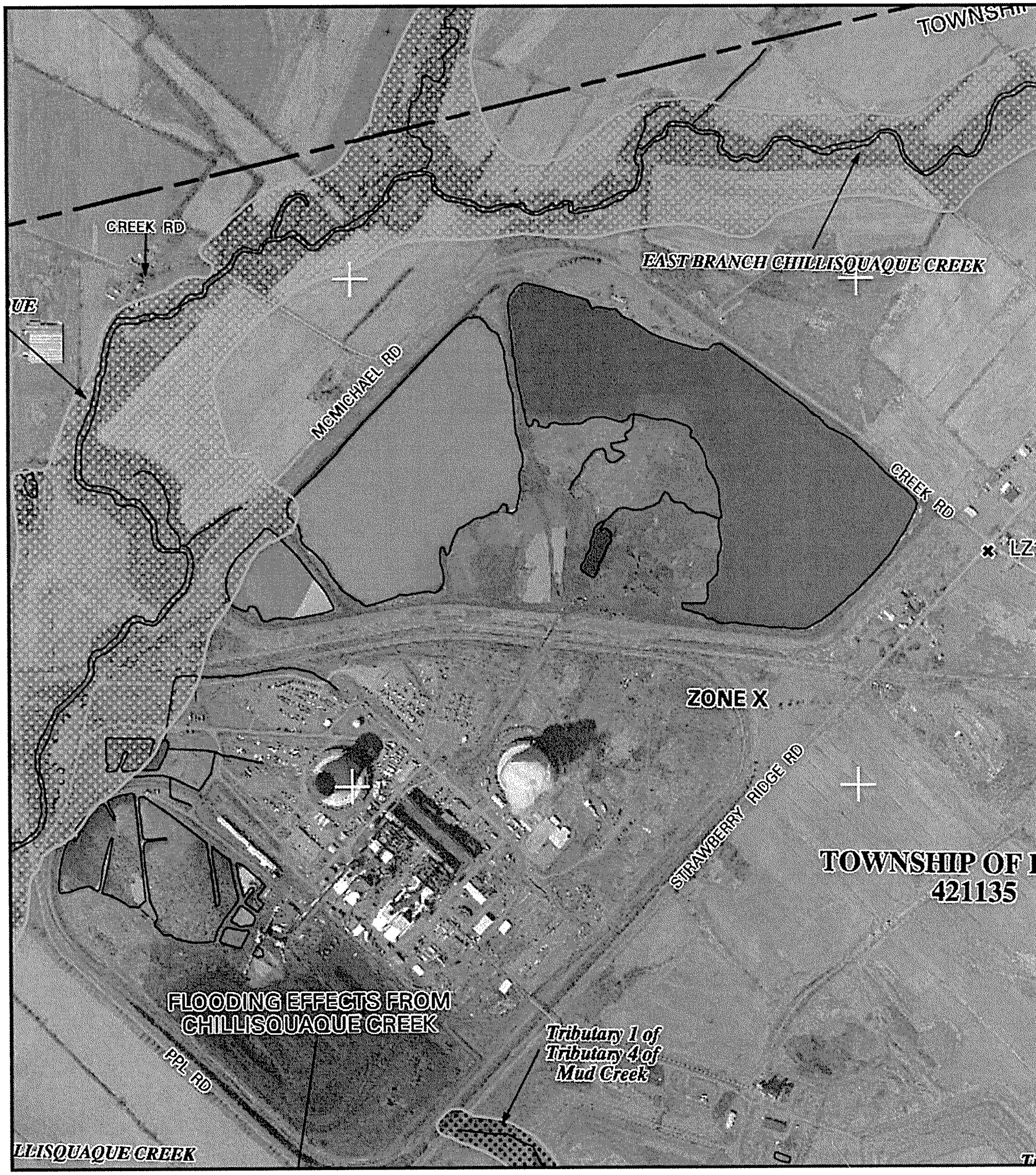
LAKE CHILLASQUAQUE
DAMBREAK STUDY
PMF BACKGROUND CONDITION

STARTING LEVEL IN LAKE : 596.7'
MAX. LAKE LEVEL AT BREAK : 602.7'
TIME OF BREAK FORMATION : 45 min.
AVG. BREACH WIDTH : 126'
BOTTOM OF BREACH ELEV : 560'



LOCATION	JUST D/S OF DAM	MONTOUR POWER PLANT	WASH'VILLE RT 254	WASH'VILLE RT 54	COVERED BRIDGE	METAL BRIDGE	RT. I80	POTTSGR'VE RT. 642	RISHEL COV'D BRIDGE	CHILL'SQ'UE RT. 147	CHILL'SQ'UE RT. 405	SUSQ. RIVER
STREAM CHANNEL MILE	0	2.424	3.466	4.545	6.117	8.580	10.985	12.652	16.098	18.750	18.939	19.602
W.S.E.L. WITH DAMBREAK	569.29	527.95	518.76	515.00	512.06	508.24	502.30	496.20	485.14	461.10	452.12	452.22
W.S.E.L. WITH NO DAMBREAK	562.38	525.77	517.09	513.68	510.58	507.00	501.70	495.05	483.87	460.27	451.86	451.96
Δ W.S.E.L. (Ft.)	6.91	2.18	1.67	1.32	1.48	1.24	0.6	1.15	1.27	0.83	0.26	0.26
Q PEAK WITH BREAK (CFS)	80,676	63,006	84,647	79,004	99,554	89,258	87,514	85,176	101,254	108,803	108,773	108,281
Q PEAK WITH NO BRK (CFS)	10,786	21,861	52,719	52,909	77,104	75,291	74,405	73,798	91,220	98,302	98,317	98,372
Δ Q (CFS)	69,890	41,145	31,928	26,095	22,450	13,967	13,109	11,378	10,034	10,501	10,456	9,909
TIME TO PEAK FOLLOWING START OF BREAK (MIN.)	41	1h+19	1h+39	2h+13	3h+5	3h+25	3h+56	4h+35	6h+41	6h+34	7h+17	7h+17
CHANNEL INVERT EL. (Ft.)	555	507	500	495	490	480	472	468	452	435	435	435

FEMA FLOOD MAP



...ne if flood insurance is available in this community, co... the National Flood Insurance Program at (800) 638-6620.

N

MAP SCALE 1" = 1000'

500 0 1000 2000 FE

NFIP

NATIONAL FLOOD INSURANCE PROGRAM

PANEL 0060C

FIRM

FLOOD INSURANCE RATE MAP

MONTOUR COUNTY,
PENNSYLVANIA
(ALL JURISDICTIONS)

PANEL 60 OF 170

(SEE MAP INDEX FOR FIRM PANEL LAYOUT)

CONTAINS:

COMMUNITY	NUMBER	PANEL	SUFFIX
ANTHONY, TOWNSHIP OF	421232	0060	C
DERRY, TOWNSHIP OF	421135	0060	C

Notice to User: The **Map Number** shown below should be used when placing map orders; the **Community Number** shown above should be used on insurance applications for the subject community.

MAP NUMBER
42093C0060C

EFFECTIVE DATE:
MAY 16, 2008

Federal Emergency Management Agency

This is an official copy of a portion of the above referenced flood map. It was extracted using F-MIT On-Line. This map does not reflect changes or amendments which may have been made subsequent to the date on the title block. For the latest product information about National Flood Insurance Program flood maps check the FEMA Flood Map Store at www.msc.fema.gov