ALTOONA WATER AUTHORITY

ENGINEERING REPORT

EVALUATION OF MILL RUN DAM

LOGAN TOWNSHIP
BLAIR COUNTY, PA

SEPTEMBER 2010

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EXECUTIVE SUMMARY

1. Mill Run dam was constructed and placed into operation in 1958. The dam is considered in fair physical condition. Major areas of concern are the instability of the rock slope and concrete spillway deterioration.

2. The dam was designed to standards existing at the time. Virtually no structural maintenance has been done at the dam which is a tribute to the original design.

3. A hydrologic evaluation was performed according to updated Probable Maximum Flood (PMF) criteria. Based on computer modeling (HEC-HMS V3.4), the spillway design flood is 22,000 cfs for a net rainfall of 33.4 inches over a 22-hour period. This exceeds a previous PMF study (11,700 cfs) by almost 100%.

4. The existing spillway has a capacity of 12,131 cfs which is 55% of the PMF. The updated PMF would likely result in failure of the dam. PADEP has stated that the spillway is seriously inadequate for this scenario.

5. According to PADEP directives, any new spillway must control discharges up to the 100-year flood. This necessitates a staged spillway arrangement. A primary spillway will convey normal flow (up to the 100-year flood) and the emergency spillway will pass flows up to the PMF.

6. A hydraulic evaluation was performed for four (4) spillway options including: ogee-weir (with floodwall), labyrinth weir, roller compacted concrete (RCC) overtopping protection and the upgrade of the existing spillway (with floodwall).

7. The existing intake tower, though in good structural condition, is showing signs of metal deterioration. Also, access restrictions related to the rock slope failure are another factor. A new reservoir intake tower with access bridge was considered along with renewal of the existing tower.

8. An analysis has revealed that steady state seepage conditions at full reservoir levels range from 200 to 250 gpm. A foundation sand drainage blanket conveys seepage flow to the downstream rock toe/toe drain. No evidence of piping exists. However, a new toe drain is recommended.

9. A slope stability analysis of the dam embankment reveals adequate stability for steady state conditions. However, the rapid drawdown condition reveals a marginal safety factor. Additional geotechnical investigations are needed to confirm operational drawdown procedures.

10. The rock slope adjacent to the intake access road, dam axis and spillway is unstable. This condition, existing since the original construction, has resulted in a progressive deterioration. A major slope failure occurred in 2005 partially blocking the spillway. This situation must ultimately be addressed.

11. Causes of rock slope instability are weathering of underlying siltstone/shale beds, undercutting of a colluvial boulder field, excessive slopes and poor drainage.

EXECUTIVE SUMMARY (Continued)...

12. The minimum necessary stabilization includes reducing the rock slope face (by pre-split/trim cushion blasting techniques) and better drainage facilities. Additional stabilization may take the form of rock anchors/rock bolting, mechanical methods (netting) and structural reinforcement (shotcreting of rock face). A detailed geotechnical investigation and drilling/testing program will provide final scope details.

13. An alternatives evaluation (including a cost effectiveness analysis), has revealed that the two-stage labyrinth spillway is the best solution to attain regulatory compliance. The project will also include rock slope stabilization, seepage collection system (toe drain), demolition/ backfilling the existing spillway, new intake tower, access bridge and widening the downstream Mill Run channel.

14. The estimated total project cost of the recommended improvements in $9.5 million. A typical Pennvest loan (2.2%) would result in an annual debt service payment of $600,000 and a 4% water rate increase.

15. Implementation arrangements would have to consider the loss of storage during construction and possible reactivation of the Allegheny reservoir and pump station. The Allegheny dam and reservoir will also need to be reassessed in light of new PMF criteria.

16. The Mill Run dam modifications project has not been previously considered a mandatory compliance effort by either the Authority or PADEP. Therefore, it is assumed these improvements would be included in an overall upgrade project when funds allow.

PURPOSE - The Altoona Water Authority authorized Gwin, Dobson & Foreman, Inc. to conduct an evaluation of Mill Run dam. This action was prompted by PADEP directives about spillway capacity in relation to new hydrologic criteria. Also, the recurrent failure of the rock slope was an additional concern of PADEP and the Authority.

The purpose of this report is to present the findings of the Mill Run dam evaluation. It principally involved a detailed hydrologic and hydraulic assessment of existing spillway capacity. A cursory study was made of seepage and dam embankment stability. In addition, GD&F examined the causes of rock slope failure at the east abutment. Finally, we evaluated the condition of various dam appurtenances (intake tower, intake line, spillway bridge). An alternatives evaluation and cost-effectiveness study was performed of the various structural improvements.

SCOPE OF PROJECT - The scope of the dam evaluation included the following:

a. Complete reconnaissance of Mill Run dam and reservoir noting all features that affect stability and safety of the dam.

b. Review of existing plans, records, inspection reports, engineering studies and other materials pertinent to the evaluation. Prepare base maps and drawings of the watershed, dam embankment, intake and outlet works, spillway, inflatable dam, access roads, bridges and other appurtenances.

c. Evaluation of physical condition of dam structure, embankment and reservoir. Review previous dam inspection reports to determine progression of deterioration.

d. Review seepage and stability conditions of dam embankment on a cursory basis. Inspect and assess condition of spillway rock slope using desktop geotechnical engineering, geological and soil reconnaissance techniques. Describe causes and severity of slope failure.

e. Evaluate condition and serviceability of intake/outlet works including values and sluice gates, operators, intake pipes, valve chambers and appurtenances. Review condition of access roads, dam crest and spillway bridge.

f. Selection of an appropriate flood relative to size and hazard potential of the dam in conformance with PADEP Division of Dam Safety regulations.

g. Hydrological studies of the Mill Run watershed including delineation and physical characterization of sub-watersheds, development and arrangement of National Weather Service NOAA precipitation data, synthetic unit hydrograph development, flood and channel routing and analysis of peak inflow/peak outflow hydrographs.

h. All hydrologic information was integrated and computed using the US Army Corps of Engineers HEC-HMS software program. Various precipitation and sub-watershed combinations were analyzed to produce the maximum runoff condition. Develop the Spillway Design Flood (SDF) and 100-year flood for Mill Run reservoir.

i. Hydraulic evaluation of the existing overflow spillway including approach channel, ogee weir, spillway chute and discharge channel using the Spillway Design Flood (SDF). Determine hydraulic capacity of the existing spillway.

j. Evaluate various spillway alternatives that will safely pass the SDF including upgrades to the existing spillway (with floodwall), level overflow spillway, multi-stage spillway, labyrinth spillway and roller compacted concrete spillway overtopping protection.

k. Evaluate downstream floodplain and flooding potential due to inflow/outflow characteristics of spillway alternatives.

l. Perform a cost effectiveness and alternatives evaluation of the various flood protection options. Prepare cost estimates along with non-structural considerations. Provide recommendations on the most cost effective and serviceable spillway option.

m. On a cursory basis, review seepage and stability considerations relative to proposed dam modifications. Perform desktop evaluation where appropriate.

n. Perform geotechnical and geological reconnaissance of the rock slope next to the spillway/east dam abutment with possible causes of slope failure. Provide recommendations for remedial measures and future investigation.

o. Assess condition of intake and outlet works including intake tower and appurtenances, control gates and operators, valve chamber, intake line and mechanical piping. Make recommendations for repair and replacement.

p. Prepare a summary report including system description and history, condition of dam and appurtenances, hydrologic/hydraulic evaluation, basic design criteria for spillway improvements, seepage and stability considerations, intake/outlet works assessment, cost effectiveness and alternatives evaluation, recommended improvements and project cost estimates. Attach pertinent maps, drawings, graphics, computations and computer output to technically support the report conclusions.

HISTORY

Mill Run dam was constructed to address regional water shortages and provide enhanced water quality for the Altoona service area. The dam was constructed in 1957 at a cost of $1.5 million (2010 Cost - $15,500,000). It was designed by Albright & Friel (Philadelphia) and Lewis L. Gwin, Consulting Engineers (Altoona). The dam permit was approved by the Pennsylvania Department of Forest and Waters. Sanctis Construction (Pittsburgh) was the original contractor, but was unable to finish the project because of financial difficulties. Construction was completed by New Enterprise Stone and Lime Co.

In 1968, the City elected to increase reservoir capacity by mounting an inflatable "fabridam" on the existing spillway. The fabridam was designed by N.M. Imbertson & Associates; Burbank, CA, for Gwin Engineers. Fabricated and installed by Globe Linings, Long Beach, CA, the fabridam increased the reservoir pool by four feet. The fabridam was replaced in 1990 with a six (6) foot rubber dam. Fabricated by Bridgestone Rubber Co., the rubber dam was installed by Lone Pine Construction Co., Monongahela, PA, at a cost of $494,128. The system is designed to automatically deflate during flood conditions.

A $12 million water treatment plant was built downstream of Mill Run dam in 1998. This plant currently treats raw water from the reservoir. The project did not affect the dam other than relocation of downstream transmission mains.

At the time, Mill Run reservoir materially supplemented the City's water supply. Water quality was and is considered exceptional. Other than the inflatable dams, no improvements or repairs have been required at Mill Run dam since 1955, a considerable return on the original investment. The dam and reservoir continue to form an integral part of the Altoona water supply system.

HAZARD POTENTIAL

Mill Run dam is located on Mill Run stream about two miles upstream from the Altoona city limits in Logan Township, Blair County. The dam is owned and operated by the Altoona Water Authority.

Downstream from the dam, Mill Run flows through a narrow valley. About one mile from the dam, it is confined to a channel about 40-feet wide. This channel is formed by the dike of the off-stream Allegheny Reservoir (PADEP File No. 7-22; USCOE NDI I.D. No. 526) and the natural valley wall. Further downstream, Mill Run traverses a residential area and flows under the Norfolk-Southern railroad embankment.

From this point, Mill Run enters the City of Altoona and is confined to a 25-30 foot wide rectangular channel spanned by numerous bridges. The stream joins the Beaverdam Branch of the Frankstown Branch (of the Juniata River) about one mile south of Altoona.

It is estimated that about 500 business residences (along with a population of about 1,300 to 1,500) are located within the Mill Run floodway. A 2000 dam breach analysis showed the arial extent of flooding in the event of a dam failure. This zone (through the City of Altoona and Logan Township) is shown on the inundation map from this study. Please refer to Figure No. 8 of Mill Run reservoir in relation to the downstream inundation area through the City.

The potential for loss and life and extensive property damage more than justifies the classification of Mill Run as a "high hazard" dam.

DAM DESIGNATION

Mill Run dam has been given the following designations by the regulatory agencies having jurisdiction:

a.	PA Department of Environmental Protection - Division of Dam Safety	DEP File No. D07-082 (Category 1, Class "A", "High Hazard")
b.	U.S. Army Corps of Engineers, Baltimore District	NDI I.D. No. 533 ("Large," "High Hazard")

Due to Mill Run's size and hazard classification, both regulatory agencies specify the Probable Maximum Flood (PMF) as the spillway design flood.

GENERAL INFORMATION

Mill Run dam consists of an earth embankment 1,200 feet long, with a maximum height of 100 feet and a crest width of 20 feet. The combined primary and emergency spillway is located on the left abutment (looking downstream). As designed, flow through the spillway is controlled by an ogee weir located at a level 13 feet below the dam crest. The spillway is spanned by a bridge with a pier at the center of the weir and leaving a 10-foot clear opening between the bottom of the bridge and crest level. Presently, the ogee weir is equipped with inflatable dams, which permit raising the control elevation of the spillway by six (6) feet, leaving a freeboard of seven (7) feet to the crest of the dam. The spillway chute is a rectangular channel with concrete sides and bottom. Flow discharges into a plunge pool at the toe of the dam by a deflector bucket.

The outlet works for the dam consist of an intake tower, a 42-inch diameter combined supply line and outlet pipe, and meter/valve chamber at the downstream toe of the dam. The intake tower is located along the left shore of the lake about 350 feet upstream from the spillway. The 42-inch pipe leads from the intake through the embankment to the valve chamber at the toe of the dam. The pipe is encased in reinforced concrete through the embankment. Flow through this pipe is normally controlled by valves located at the valve chamber downstream of the dam. Flow into this pipe can also be controlled by closing the sluice gates at the intake tower. This pipe constitutes the emergency drawdown facility for the dam. The 1958 aerial photograph (Figure No. 1) shows Mill Run dam and reservoir after completion of construction.

PERTINENT DATA

The following data was extracted from existing records and our evaluation of Mill Run dam.

a. Drainage Area (square miles) - 4.212

b. Discharge at Dam Site (cfs)

Maximum known flood at dam site - 300 (Tropical Storm Ivan, 2004)
Gated spillway capacity at maximum pool elevation - 5,200 (gates inflated)
Total spillway capacity at maximum pool elevation - 11,151 (gates deflated)

c. Elevation (USGS Datum) (feet)

Top of dam - 1,515
Normal pool (normal) - 1,502 (1,508 with spillway gates)
Spillway crest - 1,502 (1,508 with spillway gates inflated)
Upstream intake at tower - 1,424
Downstream intake invert (at blowoff) - 1,414
Streambed at center line of dam - 1,425
Maximum tailwater - Unknown

Figure No. 1: Aerial Photo of Mill Run Dam & Reservoir (1958)

d. Reservoir (feet)

Length of maximum pool - 3,200
Length of normal pool (normal) - 2,800

e. Storage (acre-feet)

Normal pool (El. 1,502) - 1,306.3 (425.6 mg)
Gated Spillway Pool (El. 1,508) - 1,596.0 (520 mg)
Top of dam - (El. 1,515) - 1,966.18 (640.6 mg)

f. Reservoir Surface (acres)

Normal pool - 44.5 (El. 1,502)
Gated Spillway Crest - 50.5 (El. 1,508)
Top of dam - 58.1 (El. 1,502)

g. Dam

Type - Zoned Earth Embankment
Length - 1,200 feet
Height - 100 feet
Top width - 20 feet
Side slopes - 2H:1V, 3H:1V, upstream; 2H:1V, 3H:1V, downstream
Zoning- Yes (Rockfill Downstream, Impervious Upstream Core)
Impervious core - Yes
Grout curtain - Yes (Partial)

h. Diversion and Regulation Intake

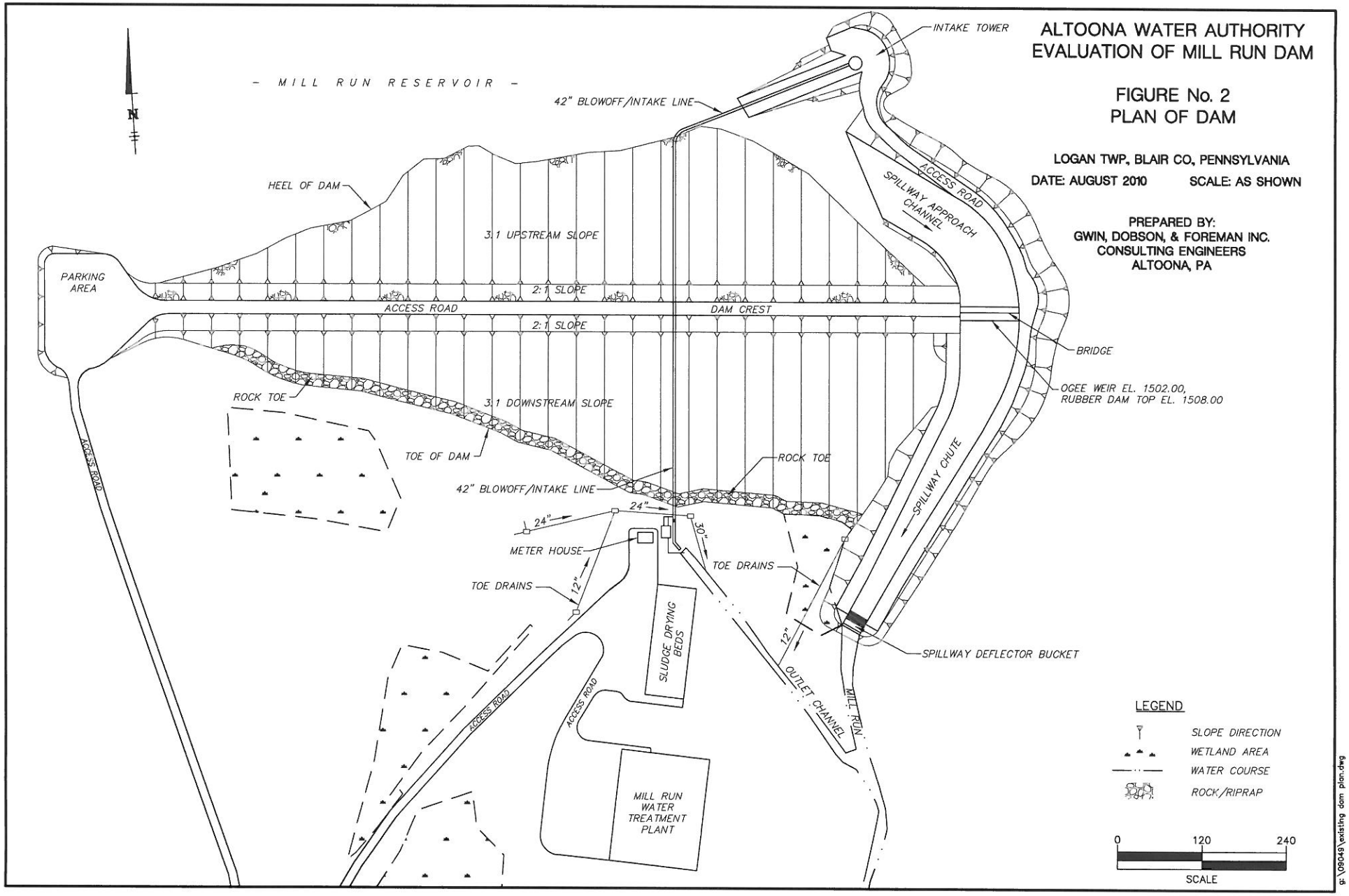
Type - 42-inch cast-iron pipe (encased in concrete through embankment to intake tower)
Length - 858 feet; Inside Diameter = 9 feet
Closure - Gate valve downstream valve pit and sluice gates at upstream intake tower
Access - Valve chamber and intake tower

i. Spillway

Type - Ogee weir (included)
Length - 77.5 feet (clear length of crest)
Crest elevation - 1502
Gates - Inflatable Rubber Dam (6 feet)
Upstream channel - Lake/Reservoir
Downstream channel - Rectangular concrete channel (variable width, 30 to 77.5 feet)
Energy Dissipation - Deflector bucket to plunge pool

j. Plan of Dam

Refer to Figure No. 2 (next page)


ALTOONA WATER AUTHORITY
EVALUATION OF MILL RUN DAM
FIGURE No. 2
PLAN OF DAM
LOGAN TWP., BLAIR CO., PENNSYLVANIA
DATE: AUGUST 2010
SCALE: AS SHOWN
PREPARED BY:
GWIN, DOBSON, & FOREMAN INC.
CONSULTING ENGINEERS
ALTOONA, PA
- MILL RUN RESERVOIR -
N
INTAKE TOWER
42" BLOWOFF/INTAKE LINE
HEEL OF DAM
3:1 UPSTREAM SLOPE
PARKING AREA
2:1 SLOPE
ACCESS ROAD
DAM CREST
2:1 SLOPE
ROCK TOE
3:1 DOWNSTREAM SLOPE
TOE OF DAM
42" BLOWOFF/INTAKE LINE
ROCK TOE
24"
24"
30"
METER HOUSE
TOE DRAINS
TOE DRAINS
12"
12"
SLUDGE DRYING BEDS
MILL RUN WATER TREATMENT PLANT
ACCESS ROAD
ACCESS ROAD
ACCESS ROAD
SPILLWAY APPROACH CHANNEL
ACCESS ROAD
SPILLWAY CHUTE
BRIDGE
OGEE WEIR EL. 1502.00,
RUBBER DAM TOP EL. 1508.00
SPILLWAY DEFLECTOR BUCKET
OUTLET CHANNEL
MILL RUN
LEGEND
SLOPE DIRECTION
WETLAND AREA
WATER COURSE
ROCK/RIPRAP
0
120
240
SCALE
g:\06049\existing dam plan.dwg

DESIGN INFORMATION

General - Albright & Friel and Lewis L. Gwin, P.E. collaborated on the design of Mill Run dam in the mid-1950's. Plans and specifications were filed with the PA Department of Forest and Waters on July 6, 1955 along with a design engineer's report. Additional information included test borings and soil testing for embankment materials. The report included information on hydrology, hydraulics, geotechnical engineering and embankment stability.

Embankment - The dam was designed as a zoned embankment. It consists of a compacted impervious upstream fill with a large rock-fill section forming the downstream slope. See typical cross section, Figure 3. The impervious section extends over a cutoff trench excavated into foundation rock at the center line of the dam. A longitudinal sand drain is located near the downstream limit of the impervious zone. This sand drain is connected to two (2) transverse sand drains which discharge into a rock-fill trench at the toe of the dam. A toe drain system collects seepage for discharge to a blow-off channel.

The embankment was designed to have two-to-one (horizontal-to-vertical) slopes on both the downstream and upstream slopes from the crest to Elevation 1502 and three-to-one slopes below this elevation on both faces. The foundation of the dam was pressure grouted with cement.

The original subsurface investigation conducted for the dam consisted of 18 borings. The typical soil profile consists of silty clay to a depth of up to 50 feet on the right abutment, diminishing to about 8 feet at the valley floor. The soil is underlain by alternating layers of sandstone and shale. The left abutment consists of alternating layers of sandstone, siltstone/mudstone and shale.

Soil testing consisted of classification and direct shear tests. The embankment design was based on the subsurface investigation and the laboratory tests performed by Pittsburgh Testing Laboratory. Review of laboratory test results indicate that the strength parameters for the embankment material obtained from direct shear tests ranged between 0 and 0.49 tons per square foot for cohesion and 14 to 45 degrees for the angles of internal friction. Average values were about 0.10 tons per square foot for cohesion and 24 degrees for the internal friction angle. The factor of safety for slope stability purposes was reported to be 1.5. However, no indication of the embankment condition was noted (rapid drawdown, etc.) in the calculations.

A review of the geotechnical aspects of the design indicates that the design generally followed the accepted practices for subsurface investigation and laboratory testing applicable at the time of the design.

The design incorporated such basic components as zoning of the embankment, cutoff trench, foundation grouting and internal drainage system. However, there is no filter zone between the downstream rock shell and the impervious core. Internal drainage is provided by a longitudinal sand drain at the downstream toe of the impervious zone and by the downstream rock zone.

Seismic Stability - The dam is located in Seismic Zone 1. Static stability of the dam is considered to be adequate. Therefore, based on the recommended criteria for evaluation of seismic stability of dams, the structure is assumed to present no hazard from earthquakes.

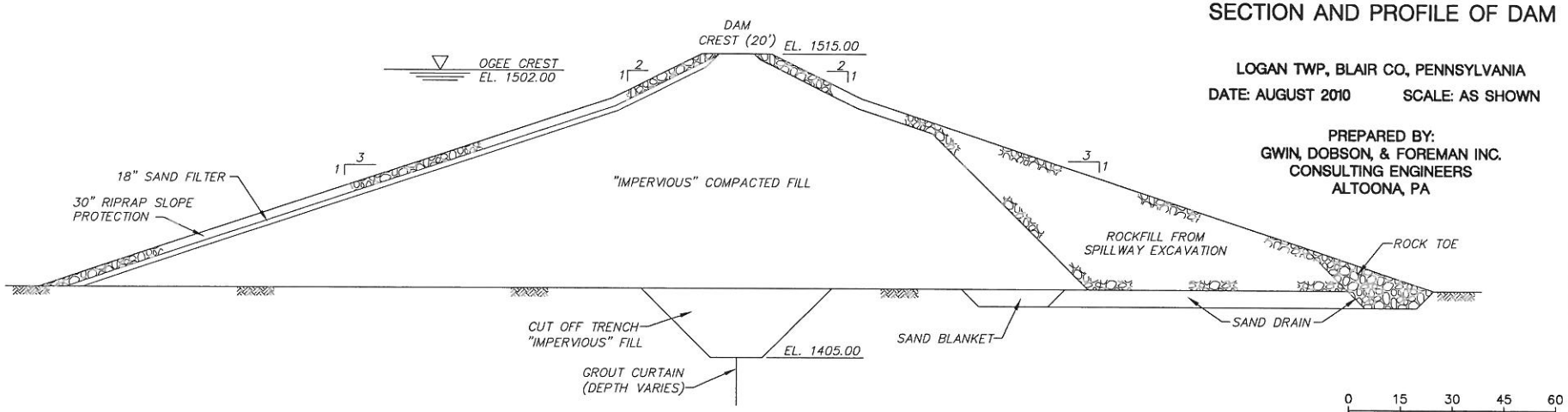
ALTOONA WATER AUTHORITY
EVALUATION OF MILL RUN DAM

FIGURE No. 3
SECTION AND PROFILE OF DAM

LOGAN TWP, BLAIR CO, PENNSYLVANIA

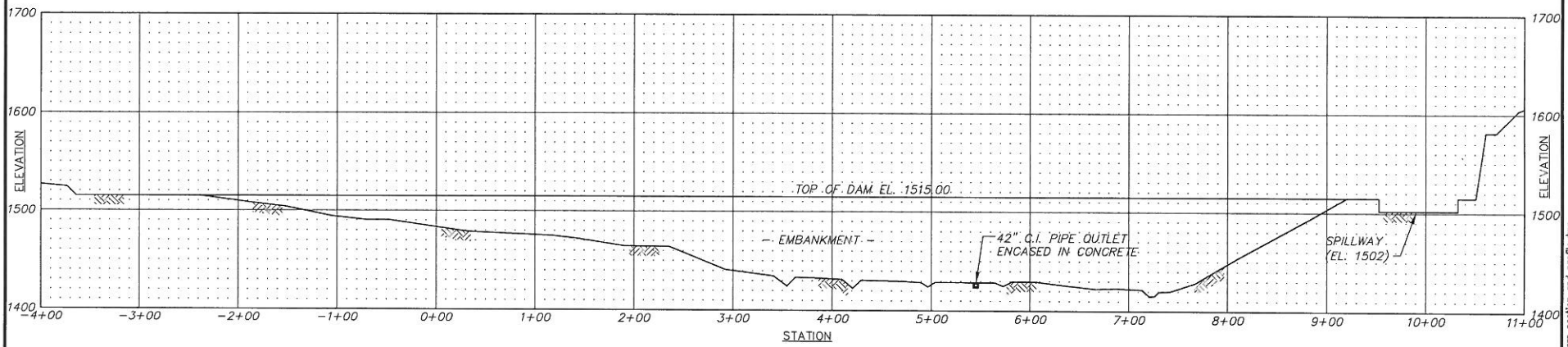
DATE: AUGUST 2010 SCALE: AS SHOWN

PREPARED BY:
GWIN, DOBSON, & FOREMAN INC.
CONSULTING ENGINEERS
ALTOONA, PA



TYPICAL CROSS SECTION OF EXISTING DAM

0 15 30 45 60
SCALE



PROFILE ON AXIS OF EXISTING DAM

0 50 100 150 200
SCALE

g:\09049\existing dam section and profile.dwg

Hydrology and Hydraulics - The hydrology and hydraulic reports indicate that the design followed the criteria set forth by the Pennsylvania Department of Forests and Waters which were applicable at the time of the design.

Dam Appurtenances - The appurtenances of the dam consist of the combined primary and emergency spillway and the outlet works. The spillway crest is equipped with inflatable dams which were installed to increase the storage capacity of the reservoir. The combined primary/emergency spillway for the dam is located on the left abutment.

The inflatable dams were installed across the spillway in 1968. These dams were originally designed by N.M. Imberson and Associates, Inc., of Burbank, California. These were replaced in 1990 with rubber dams manufactured by Bridgestone Rubber Products. Controls for the gates are located in a building next to the right side of the spillway. The inflatable dams are under automatic operation. They start to deflate when the water level over the bags reaches 6 inches and completely deflate when the water depth over the bags reaches 1.5 feet. The bags may also be deflated manually. Refer to the following photographs (Figure No. 4 and 5).

The outlet works for the dam consist of an intake tower, a 42-inch cast-iron combined blow-off and supply line and valve chamber/meter house located at the toe of the dam. The flow through this pipe is normally controlled by the valves located at the valve chamber and the meter house. However, when required, the pipe can be drained by closing the sluice gates at the intake tower. There is no design data available for the appurtenant structures.

A review of the design drawings indicates no significant design deficiencies that should affect the overall performance of the appurtenant structures.

Construction - Construction drawings and specifications prepared by the design engineers were available for review. To the extent observed, the construction of the dam was apparently done in accordance with the plans and specifications. Except for seepage, no reference was found to indicate that any unusual problems were encountered during construction of the dam.

Operation - The dam is operated by personnel of the Altoona Water Authority. It is checked on a daily basis along with operation of the Mill Run water treatment plant. Daily pool levels, seepage flow, daily water withdrawal, precipitation and dam drawdown records are maintained (some on a continuous basis) by the Authority at the water treatment plant.

Historically, the intended operating procedure is to maintain the reservoir level at the crest of the ogee weir or inflatable rubber dam. This would leave 7-13 feet of freeboard. When there is sufficient inflow to fill the reservoir, the inflatable rubber dams are inflated to increase the storage capacity of the dam.

However, supply water take-off often exceeds inflow and the pool is maintained below the spillway crest. Also, the rubber dams have not been operationally reliable, tending to sag at times and allowing storage to be lost downstream. Finally, the Mill Run water treatment plant is shut down periodically for reservoir management purposes. The net effect is that the dam is generally operating at or below the original pool level of El. 1502.

Figure No. 4: Upstream View of Rubber Dam and Bridge

Figure No. 5: Downstream View of Rubber Dam and Bridge

Relative to maximum runoff condition, the City/Authority report that the maximum, storm induced flow over the spillway was about 9 inches or 200-300 cfs during 2004 Tropical Storm Ivan. Records indicate that during Tropical Storm Agnes in 1972, the pool level was below the spillway crest. Although not intended as a flood control reservoir, as a practical matter, Mill Run has controlled downstream flooding in Altoona. Since water drawoff generally exceeds inflow, the pool level is typically below the spillway. Since the top of the pool represents the maximum unit storage (per foot), most storm inflows have been contained in this "wedge" of reservoir storage.

In 1958, drainage pipes were installed along the toe of the dam to collect the seepage flow from the sand filters for discharge to the blow-off pipe discharge channel. A flow-measuring device was installed in the blow-off pipe discharge channel to monitor the seepage through the dam. Available records indicate that normal seepage is 200 gallons per minute at normal pool. Refer to Figure No's. 6 and 7 of the seepage and measuring devices.

Annual Inspections - The Authority has conducted annual dam inspections since 1980. The field inspections and reports have been performed according to PADEP Division of Dam Safety criteria. The report also includes a record of seepage measurements at the dam. Removal of vegetation is an ongoing maintenance function in addition to the periodic removal of rock debris at the spillway and spillway bench.

Post-Construction Changes - In 1968, inflatable fabridams were installed at a cost of $26,000. The dams were placed across the spillway to increase the storage capacity of the reservoir. The fabridams were repaired following a partial failure on June 7, 1968. Mill Creek flowed over its banks at several locations in Altoona which resulted from the fabridam failure. A Gwin Engineer's report entitled, "Report of Fabridam Slippage at Mill Run Reservoir," Altoona, PA, described the causes of the failure and subsequent repairs.

PREVIOUS REPORTS AND DOCUMENTS

A number of reports and studies have been prepared concerning Mill Run watershed. These may be summarized as follows:

- **Water Supply Report, City of Altoona, January 1932** - In response to the serious drought of 1930, the City of Altoona engaged Fuller and McClintock, Consulting Engineers, NY to make a report on additional water supply. The report was the first to recommend the construction of a dam in Mill Run valley. The City subsequently purchased the entire watershed area. However, dam construction was deferred because of the Great Depression.

- **"Report on Project Improvements to Water Supply and Distribution System for City of Altoona,"** The report, dated July 13, 1954, set forth major water system improvements for the Authority including the construction of Mill Run dam. The report included design criteria and technical details for the dam.

  The design flood flow was based on the formula $Q = 6000\ A^{0.5}$, and a drainage area of 4.4 square miles. This resulted in a peak inflow of 12,540 cfs or 2,850 cfs/sq.mi. The selected flood flow was increased 5% to 13,200 cfs which is remarkably similar to later PMF estimates using more sophisticated techniques. A synthetic hydrograph was then developed, showing a time-to-peak of 4 hours from the beginning of the storm.

Figure No. 6: V-Notch Weir in Blowoff Channel

Figure No. 7: Upstream View of Blowoff Channel

Reservoir routing was performed that reduced the peak to 9,625 cfs (or 28%) using the S-curve technique. Current analytical methods have shown this reduction to be excessive.

The spillway, consisting of an 80-feet wide ogee-shaped weir, was designed for a depth of 10 feet, peak discharge of 9,625 cfs and a freeboard of 3-feet. The spillway and downstream chute were designed principally in rock and sloped and curved to minimize excavation.

Concerning inflow reservoir yield, the study used a nearby stream gaging station which was applied to a mass curve to determine reservoir depletion at various consumption drafts. Data from the Frankstown Branch of the Juniata River from 1932 to 1950 were analyzed and adjusted based on average rainfall in Altoona. This produced a yield of 1.2 cfs/sq. mi. at an average annual draft of 4.0 mgd.

The mass curve analysis indicated a reservoir capacity of 876 million gallons (later revised) for a proposed supply of 3.5 MGD. Controlling elevations were a minimum reservoir elevation of 1,450 msl (for gravity system flow); spillway elevation of 1,502 msl; and crest elevation of 1,515 msl. An average pass-by flow of 0.5 MGD was dedicated for Allegheny Reservoir and stream release (0.21 cfs/day or 0.136 mgd). The study concluded that a reliable water supply of 3.0 MGD could be produced. The storage capacity (876 mg) proved to be incorrect.

- **Water Allocation Permit** - On January 12, 1955, the PA Department of Forest and Waters approved the Authority's application to use Mill Run as a public water supply. They approved a withdrawal of 3.5 mgd and a stream release of 0.418 mgd under Permit for Water Allocation WA-171-B.

- **Plans and Specifications for Construction of Mill Run Dam and Reservoir for Altoona City Authority** - Prepared by Gwin and Albright & Friel, the plans and specifications (dated June 20, 1955) provide construction details for the dam. A plan set of 26 drawings along with technical specifications were prepared for two prime contracts. Contract No. 19 - General Construction and Contract No. 20 - Electrical Construction. As-built construction quantities are as follows:

a.	Embankment Material				
	1)	Impervious Material	-	395,000	CY
	2)	Rockfill Material	-	129,000	CY
		Total	-	524,000	CY
b.	Excavation		-	244,000	CY
c.	Reinforced Concrete		-	4,800	CY
d.	Riprap Slope Protection		-	16,600	CY
e.	Drilling/Grouting Foundation		-	3,900	LF/4,250 CY
f.	42-in. Intake/Blow-off Pipe		-	858	LF

- **Report Upon the Application of Altoona City Water Authority and City of Altoona** - This July 6, 1955 report was prepared by G.E. Thomas, Chief-Division of Dams of the PA Department of Forest and Waters on the plans for Mill Run dam. It summarizes the hydraulic and hydrologic data for the project in addition to the design criteria of the spillway (3,655 cfs). The report approved the project plans and specifications. It also specified a stream release of 0.418 mgd.

The report indicated that the maximum discharge capacity of the spillway was calculated to be 10,270 cfs, controlled by a 10-foot, 3-inch clearance between the spillway crest and the overhead bridge leaving two feet, seven inches of freeboard to the top of the dam. It is further stated that the spillway would pass the required design discharge of 3,655 cfs (850 cfs per square mile) with a freeboard of 7.8 feet. This generous freeboard allowance permitted future installation of inflatable dams to increase storage capacity.

- **Report of Fabridam Slippage at Mill Run Reservoir** - Gwin Engineers submitted a report on the partial failure of a fabridam on June 7, 1968. This fabridam was replaced in 1990.

- **USCOE Mill Run Flood Plain Study, January 1974** - Gwin, Dobson & Foreman, Inc. prepared a flood plain study of Mill Run between Allegheny Reservoir to the Meadows Intersection for the US Army Corps of Engineers. Snyder's synthetic unit hydrograph was used in conjunction with a HMR 50 (48 hour rainfall duration) to produce flood discharges. A prorated Standard Project Flood (500-year frequency) of 1,400 cfs for Mill Run dam was computed. This is significantly less than the PMF developed under other studies. The flood plain boundaries through Logan Township and the City of Altoona were useful in showing the extent for storm events less than the PMF.

- **USCOE Phase I National Dam Inspection Report - Mill Run Dam (NDI I.D. No. 533)** - D'Appolonia Consulting Engineers, Pittsburgh, submitted a report to the Baltimore District, U.S. Army Corps of Engineers in September 1978. An inspection report was performed according to the National Dam Inspection Act (PL 92-367). This legislation directed the Corps of Engineers to conduct inspection of all dams throughout the United States.

  The report summarized the results of a comprehensive field inspection and a hydrology/ hydraulics evaluation. Mill Run dam was found to be in good condition and the spillway capacity was classified as "adequate." Recommendations included the monitoring of the spillway rock cut, dam seepage and inflatable dams.

  Regarding the spillway, Mill Run was classified as a "large" dam in the "high hazard" classification. Under the recommended criteria, the dam was required to pass the probable maximum flood (PMF). A PMF inflow peak of 12,900 cfs and spillway capacity of 11,400 cfs were determined. Since the spillway could pass 97% of the PMF (with surcharge storage), the spillway was rated as "adequate." No further structural improvements were recommended.

- **Hydrologic and Hydraulic Evaluation of Allegheny Reservoir, July 1979** - Mill Run dam was included in an evaluation of the downstream Allegheny Reservoir. This report, prepared by Gwin, Dobson & Foreman, included a detailed hydraulic and hydrologic evaluation. GD&F computed a peak PMF inflow of 11,702 cfs or about 10% less than the Phase I report. The spillway capacity was computed to be 12,131 cfs or 6.4% greater than the Phase I report. It was judged that the Mill Run dam spillway was sufficient for the PMF.

- **Federal Emergency Management Agency (FEMA) - National Flood Insurance Program, Flood Boundary and Floodway Map, October 17, 1986** - The FEMA program identified flood plain boundaries for Mill Run from Allegheny Reservoir to the confluence with the Beaverdam Branch. The study noted that the 100-and 500-year floods were contained in the downstream railroad

culverts. Peak discharges were developed for the 10-, 50-, 100- and 500-year floods which we later prorated for Mill Run dam. This yielded 100 and 500-year values of 884 cfs and 1,630 cfs, respectively. The flood discharges are useful in assessing and, therefore, controlling the effects of spillway modifications on the downstream floodplain.

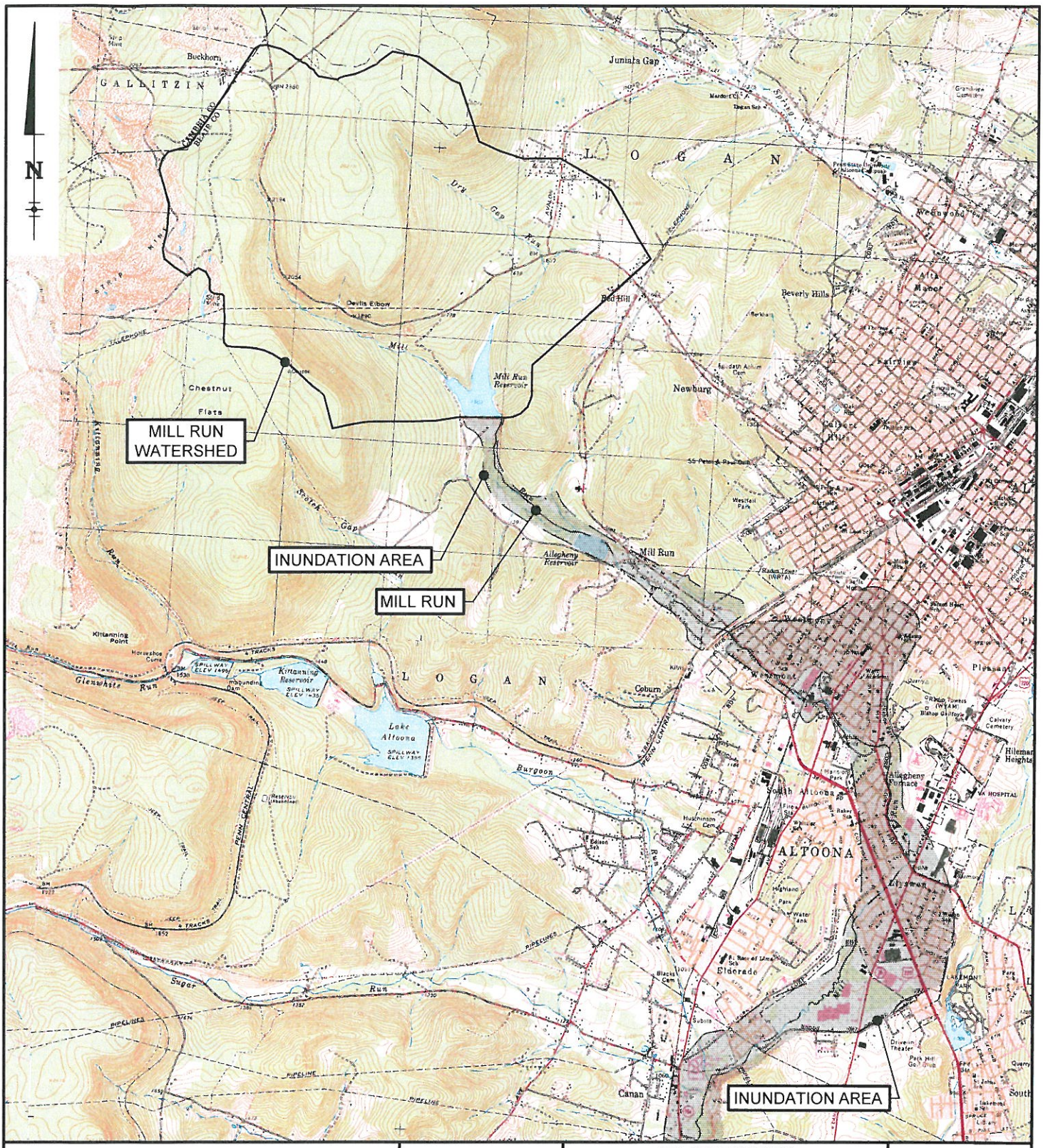
- **Computer Breaching Analysis for Mill Run Dam, February 2000** - As part of the development of an emergency action plan, GD&F did a dam break analysis and downstream inundation map for Mill Run dam. Using BOSS DAMBRK software, a downstream reach of 33 miles was modeled involving 110 cross-sections and the routing of 7 lateral inflows. Simulations included the "Sunny Day with Breach" and "100% PMF with Breach" analyses. Figure 8 shows the downstream inundation area resulting from the breach.

  The PMF hydrograph was developed using the 6-hour probable maximum precipitation from HMR 51 and the 6-hour Snyder's Unit Hydrograph. This yielded a PMF peak discharge of 22,034 cfs with a total precipitation of 26.2 inches.

  This PMF discharge is 80% higher than the PMF calculations from the Phase I reports. This is due to a much smaller time interval (5 minutes) used in the computation of the hydrograph (as opposed to one hour) and the use of HMR 51 (in lieu of HMR No. 40) precipitation data.

- **Hydrologic Study of Mill Run Watershed, 2004** - In conjunction with the Baltimore District, USCOE, Advanced Technology Systems, Inc. developed a hydrologic/hydraulic planning tool for the Mill Run area. Prompted by flooding in 1996, Congress authorized funding for projects to mitigate flooding in the area. Using the USCOE Watershed Modeling System (WMS), peak flow discharges for various storm frequencies and durations were developed. From this data, we prorated a Standard Project Flood (500-yr. frequency) of 2,000 cfs for Mill Run dam. It should be noted that the difference between the SPF (2,000 cfs) and PMF (20,000 cfs) is substantial.

- **Mill Run Stream Improvement Project; FEMA Conditional Letter of Map Revision (CLOMR), Logan Township, March 2010** - Keller Engineers (along with WHM Consulting, Inc.) did HEC-RAS modeling of Mill Run from Allegheny Reservoir to the Mill Run railroad culvert. The intent was to develop new FEMA flood plain boundaries for a stream improvement project. This study utilized the 1986 FEMA peak discharge values. Again, the prorated 100-yr. value for Mill Run dam is 884 cfs. This information was considered in determining the storage/discharge function for floods less than the PMF.



GRAPHIC SCALE

0 4000 8000

1 inch = 4000 ft.

COMMONWEALTH OF PENNSYLVANIA
REGISTERED PROFESSIONAL ENGINEER
MARK V. GLENN
No. 30633-E

SEAL

MILL RUN DAM BREACH
DOWNSTREAM INUNDATION AREA

ALTOONA WATER AUTHORITY

MILL RUN DAM EVALUATION

LOGAN TOWNSHIP, BLAIR CO, PENNSYLVANIA

DATE: 8/11/10	JOB: 09049	SCALE: AS SHOWN
FILE: WATERSHED	DRAWN BY: AJL	CHECKED BY: MVG

GD&F

GWIN DOBSON & FOREMAN INC.
Consulting Engineers
3121 Fairway Drive
Altoona, PA 16602

SHEET NO:

Fig. No. 8

WATER ALLOCATION PERMIT

The PADEP issued a new water allocation permit to the Authority on April 28, 2008. Included in this permit were conditions for Mill Run reservoir which stipulated a maximum withdrawal of 5.0 mgd and conservation release of 0.466 mgd. This conservation release is slightly higher than the 1955 pass-by flow of 0.418 mgd. GD&F computed a total safe yield of 3.0 mgd for Mill Run reservoir. Therefore, the net safe yield is 2.534 mgd which approximates a low flow condition of seven consecutive days at a frequency occurrence of once every ten years ($Q_{7\text{-}10}$).

PHYSICAL CONDITION OF DAM

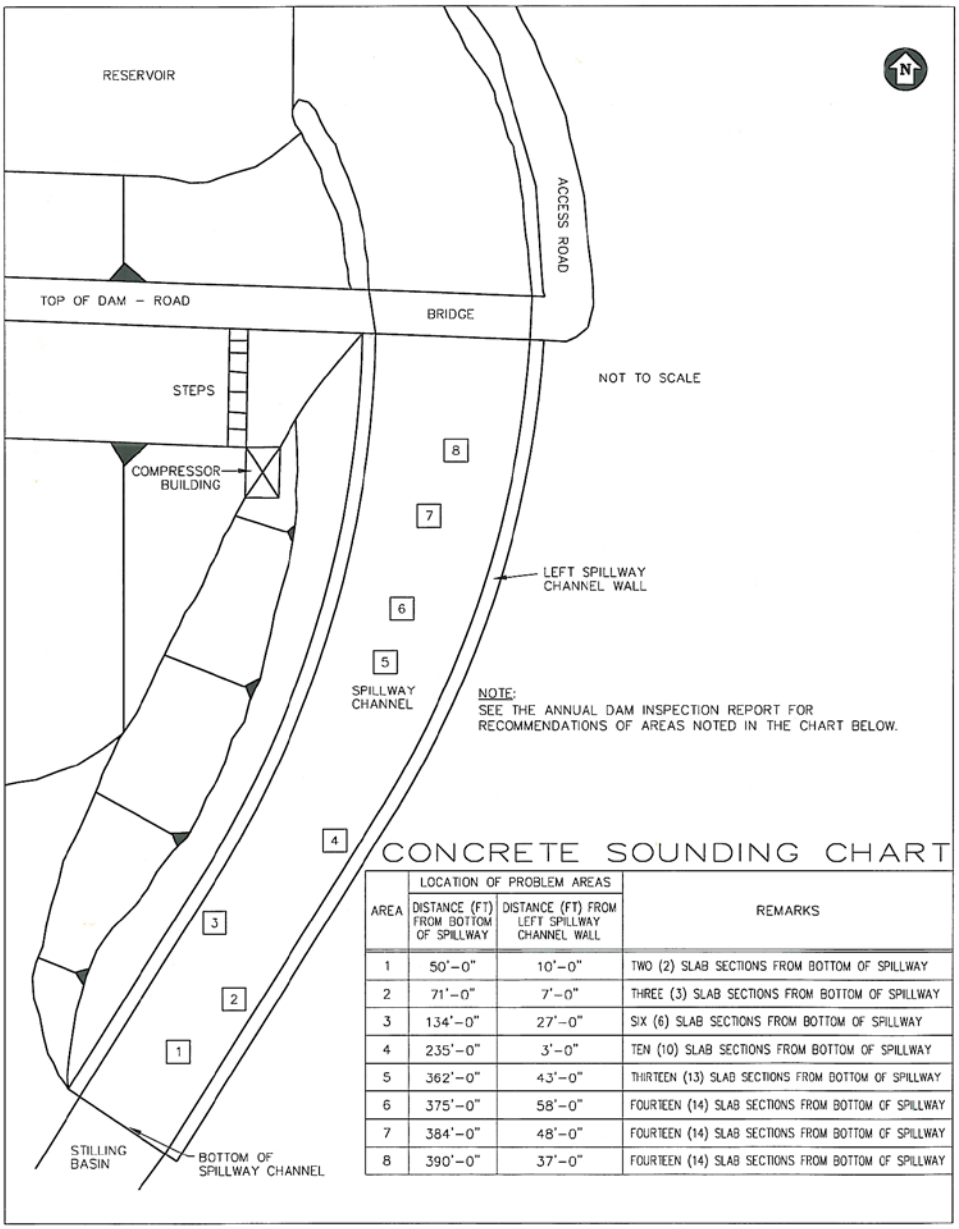
Systematic dam inspections, under the auspices of a professional engineer, are conducted annually by the Altoona Water Authority. Inspection reports for Mill Run dam have been prepared since 1980. Based on our field assessments and review of recent reports, we have summarized the physical condition of the Mill Run dam as follows:

- **Watershed** - The watershed is predominantly woodlands. The shoreline is not considered susceptible to massive landslides which would affect the storage volume of the reservoir or cause overtopping of the dam by displaced water.

- **Embankment** - The dam embankment has not exhibited structural stress, cracking, subsidence, bulging and slope instability since its construction. The dam was designed with 3:1 side slopes along with an engineered zoned embankment (impervious fill upstream and rockfill downstream). These factors, together with a seepage control system, reflect the 1.5 factor of safety used in the dam design. Recent inspections have confirmed that the visible embankment and abutment components are satisfactory.

- **Subsurface Drainage/Seepage** - Seepage from the dam foundation and embankment has been controlled by an internal sand blanket and collected downstream in a toe drain system. The volume of seepage has been a concern at times. In fact, shortly after filling of the dam, seepage quantities of 200-250 gpm caused some anxiety for City and Authority officials. However, the design engineers started that the seepage was not excessive. In 1958, drainage pipes were installed along the downstream toe to collect the seepage flow from the sand filters for discharge to the blowoff pipe channel. Before Authority operation, the City Water Department stated that seepage flow averaged about 200 gpm. At the time of the Phase I inspection (July 10, 1978), the seepage flow was 165 gpm with the reservoir four feet below the spillway. Generally, seepage has varied between 75-250 gpm, depending on pool elevation. The flow is clear with little if any turbidity. However, inspection personnel suspect these pipes are clogged or collapsed due to several wet areas downstream.

- **Spillway** - In 2001 and 2002, the reinforced concrete spillway was examined using a percussion sounding tool (Delam 2000). The purpose of this testing was to find any voids existing under the spillway floor. Six (6) areas were found to have some void potential according to the following plan. These voids were likely caused by erosional underflow. It was recommended that the voids be filled with concrete. Non-intrusive techniques, such as injection grouting (cement or epoxy), could also be effective. This condition has existed for at least 10 years. Please refer to the attached figure.



CONCRETE SOUNDING CHART

AREA	LOCATION OF PROBLEM AREAS		REMARKS
	DISTANCE (FT) FROM BOTTOM OF SPILLWAY	DISTANCE (FT) FROM LEFT SPILLWAY CHANNEL WALL	
1	50'–0"	10'–0"	TWO (2) SLAB SECTIONS FROM BOTTOM OF SPILLWAY
2	71'–0"	7'–0"	THREE (3) SLAB SECTIONS FROM BOTTOM OF SPILLWAY
3	134'–0"	27'–0"	SIX (6) SLAB SECTIONS FROM BOTTOM OF SPILLWAY
4	235'–0"	3'–0"	TEN (10) SLAB SECTIONS FROM BOTTOM OF SPILLWAY
5	362'–0"	43'–0"	THIRTEEN (13) SLAB SECTIONS FROM BOTTOM OF SPILLWAY
6	375'–0"	58'–0"	FOURTEEN (14) SLAB SECTIONS FROM BOTTOM OF SPILLWAY
7	384'–0"	48'–0"	FOURTEEN (14) SLAB SECTIONS FROM BOTTOM OF SPILLWAY
8	390'–0"	37'–0"	FOURTEEN (14) SLAB SECTIONS FROM BOTTOM OF SPILLWAY

Figure No. 9: 2001/2002 Concrete Integrity Testing

The spillway joints have been eroded at the slab sections and most of the joint filler has been eroded. The surface of the concrete floor is poor, with aggregate exposed along the entire channel. Cracking is evident in many of the floor slabs.

The ogee weir, a mass concrete section, has shown signs of deterioration. Surficial cracks are evident and leakage is occurring at the rubber dam-concrete interface. Seepage flow is evident at the west ogee/concrete wall interface which may be indicative of a poor construction joint. The danger of saturated concrete can lead to a porous and structurally weak condition.

The concrete spillway has been affected by rock slides over its history. The condition worsened to the point that a significant portion of the spillway was blocked. This material (6,000 CY) was removed in 2006. However, rock falls and debris continue to accumulate along the spillway. The rock slide condition will be addressed later in the report.

The spillway access bridge was inspected using NBIS inspection techniques. A separate report was prepared detailing the results and is on file at the GD&F office. The bridge consists of a two-span, steel, I-beam bridge with a non-composite concrete deck. The curb-to-curb width is 15-feet and has a total length of 83-feet. The sole purpose of the bridge is to provide maintenance access to the intake tower and left dam abutment. A W27 x 102 beam and two (2) W27 x 94 fascia beams form the bridge superstructure. The bridge is in generally good condition. Painting, cleaning of the bearings and bearing seats and patching the concrete deck should be performed. Negligible section loss of the bridge steel superstructure elements was noted. The bridge has been rated for an AASHTO ML80 load rating of 46.4 tons (operating).

The 6-foot inflatable rubber dam was installed in 1990. The rubber dam has provided, until recently, a dependable and inexpensive means of increasing reservoir storage. However, the rubber dam control system has had difficulty maintaining constant pressure for full inflation. The rubber dam has sagged on occasion allowing a loss of storage downstream. As has been noted, the rubber dam/concrete interface has leaked at the clamping plate. This may be caused by the concrete haunch installed on the ogee face as a mounting platform for the rubber dam. Since 2008, the rubber dam has been deflated for safety reasons. The control building for the inflation system is in fair condition.

The spillway approach channel is submerged at normal pool. When dam levels have permitted, the channel floor has been inspected and found to be in good condition. Just upstream of the bridge, a section of the channel wall has deflected outwards. This represents a growing structural problem. Although not a critical dam component, a wall collapse could lead to sloughing of material into the approach channel, which is not easily removed.

- **Intake Tower** - During annual inspections, only the portion of the intake tower above the water line has been inspected. The access road leading to the tower is blocked with material from the rock slope. The tower is currently not accessible by vehicle. Inspections have revealed that metal elements of the tower have badly deteriorated include access ladder, valve operator standoffs and operating rods. It is not known if the dam drain valve has ever been operated. Although the gate valves and sluice gates are of substantial construction, the operating parts may need to be refurbished as part of a future upgrade project. The intake tower has not been inspected by underwater diving to determine the status of the tower below the waterline. We believe, however, that the structural integrity of the reinforced concrete tower is good.

- **Dam Drain/Intake Line and Valve Vault** - The 42" dam drain line from the intake tower to the downstream valve vault is encased in concrete. No signs of leakage have ever been indicated. The valve vault has the ability to drain the dam or to divert raw water to the water treatment plant. The valves are periodically operated. The meter vault and mechanical piping are considered to be in good condition. The 42" line has not been internally inspected for leakage by means of underwater videography.

Based on the foregoing, we can summarize the condition of Mill Run dam as follows:

- The dam embankment is in good condition with no signs of instability or surface erosion.

- Seepage is effectively controlled by downstream sand drains. The flow has low turbidity. This indicates that no "piping" or internal erosion is presently occurring.

- Steady state seepage at normal pool can range from 200-250 gpm. However, the toe drain system must be checked to verify the pipes are not clogged.

- The concrete spillway, although structurally adequate, is showing signs of significant deterioration. The inflatable rubber dam is not currently operated due to operational problems.

- The intake tower is in good structural condition. Some metal components are in poor condition.

- The intake/drain line appears to be in good condition, but needs to be verified with an internal inspection.

- The rock slope at the east abutment is unstable. Recurrent rock slides have the potential to fill portions of the spillway and affect its capacity.

Based on the above, Mill Run dam is considered to be in fair condition. The dam continues to function according to regulatory standards. Virtually no structural maintenance has ever been done at the dam which is a tribute to the original design. Our condition assessment is merely a reflection of this fact and of the natural progression of deterioration since its construction.

HYDROLOGIC EVALUATION

General - Accurate runoff estimates resulting from extreme rainfall events are necessary to evaluate spillway capacity. The procedures followed in this study to compute standard project flood flows were in accordance with accepted practices of the PADEP Division of Dam Safety and U.S. Army Corps of Engineers, Baltimore District. Input and direction was provided by PADEP personnel throughout the evaluation. PADEP also approved GD&F's spillway design flood.

Hazard Potential Classification - In dam engineering, hazard classification attempts to assess the potential impact of a dam failure on downstream life and property. As was noted, the Mill Run valley downstream of the dam is heavily populated and urbanized. A sudden dam failure would result in extensive loss of life and property. As previously stated, Mill Run dam is classified as a Category 1, Substantial Hazard.

Size Classification - This category is determined by either impoundment storage or height, whichever gives the higher classification. Mill Run reservoir has a storage capacity of 1,306 acre feet (to top of dam) which rates as a Class B structure (less than 50,000 ac-ft., but greater than 1,000 ac-ft.). However, since the dam is 100 feet high, the higher Class A designation will govern.

Spillway Design Flood - According to PADEP Chapter 105 provisions (105.94), *"...Every dam shall be provided with a spillway system which is capable of safely conveying the design flood of the dam without endangering the safety or integrity of the dam..."* In addition, *"...The design flood is intended to represent the largest flood that needs to be considered in the evaluation of a given project..."* and *"...the magnitude that most closely relates to the size and hazard potential shall be selected..."* The PADEP design flood criteria is as follows:

Table 1: PADEP Design Flood Criteria

Size & Hazard Potential Classification	Design Flood
A-1, A-2, B-1	PMF
A-3, B-2, C-1	1/2 PMF to PMF
B-3, C-2	100-year to 1/2 PMF
C-3	50-year to 100-year frequency

Since Mill Run dam is classified as an A-1 structure, the required spillway design flood (SDF) is the probable maximum flood (PMF).

Previous Flood Estimates - For purposes of context and perspective, we compiled the previous flood estimates for Mill Run Dam as follows:

Table 2: Previous Mill Run Flood Estimates

Date	Study	PMP	PMF	Analysis Method/Criteria
July 13, 1954	Mill Run Engineering Report Albright & Friel, Philadelphia/ Lewis L. Gwin Engineers, Altoona	N/A	9,625	$Q = 6000A^{0.5}$ (Routed)
June 20, 1955	Report on Mill Run Dam Appl. PA Department of Forest & Waters	N/A	3,655	Q = 850 cfs/sq.mi. ("Max. Expected Flood")
Jan. 1974	Mill Run Flood Study Baltimore District, Corps of Engrs. By Gwin, Dobson & Foreman	HMR 40 (48 hrs)	1,400 cfs (SPF, est)	USCOE CE Bulletin No. 52-8 (Snyder's Synthetic UH)
Sept. 1978	COE Phase I Inspection Report by D'Applonia, Pittsburgh	HMR 40 (24 hrs)	12,900 cfs	COE Simplified Procedure (cfs/sq mi)
July 1979	Hydrology/Hydraulic Evaluation Gwin, Dobson & Foreman	HMR 40 (72 hrs)	11,700 cfs	USCOE CE Bulletin No. 52-8 (Snyder's Synthetic UH)
Feb. 2000	DEP Dam Breach Evaluation Gwin, Dobson & Foreman	HMR 51 (6 hrs)	22,034 cfs	DAMBRK Computer Model (SCS Synthetic UH)
2004	Mill Run Hydrologic Study Baltimore District, Corps of Engrs.	PDT-IDF (24 hrs)	2,000 cfs (SPF, est)	HEC-HMS (v2.12) (SCS Synthetic UH)

The range of flood flows is significant. For instance, the order of magnitude between the FEMA 100-yr. flood (1,134 cfs) and Dam Breach PMF (22,034 cfs) is significant.

Methodology - Several methodologies were used to determine the spillway design flood.

- **Corps of Engineers - Standard Project Flood (COE-SPF)** - The first involved the use of "Civil Engineering Bulletin No. 52-8 - Standard Project Flood Determinations" and "Engineering Manual, Civil Works, Part 5 - Flood Hydrograph Analysis and Computations." These methods use US Weather Bureau rainfall data which is distributed and arranged to maximize precipitation. This data is applied to a unit hydrograph (Snyder's) based on specific watershed characteristics to produce the SPF. The USCOE method was used by GD&F in the previous flood study of Mill Run dam.

- **Hydrologic Modeling System (COE-HMS)** - Developed by the USCOE, HMS simulates the precipitation-runoff process for watershed systems. This software supersedes HEC-1, which has been a mainstay of hydrologic modeling since the early 1970's. HMS has an array of options for unit hydrographs, hydrologic routing, multiple outlet works and precipitation. Version 3.4, which was released in August 2009, was utilized.

Physical Characteristics of the Watershed - The following parameters were used to characterize the Mill Run watershed for analysis. For the COE-SPF method, the physical geometry of the drainage basin must be determined. These include the drainage area, main channel length (L) and channel length to center of basin (L_c). These values are found on Figure 10 for the Mill Run and Dry Gap Run sub-basins and combined watershed area above the reservoir. Of equal importance are coefficients used to develop the shape and duration of the unit hydrograph (Snyder's). These include the lag coefficient, C_t, (ranging from 1.55 to 2.00) and peaking coefficient, C_p, (between 0.4 to 0.65). We used typical values for Pennsylvania drainage basins.

Relative to COE-HMS, since the SCS unit hydrograph will be utilized, soil maps from the USDA National Resources Conservation Service were obtained. This data, along with the hydrologic soil group information, is shown on Figure 11. Urbanization and impervious areas are shown on a satellite image of the area (Figure 12) along with watershed statistics from the USGS StreamStats website.

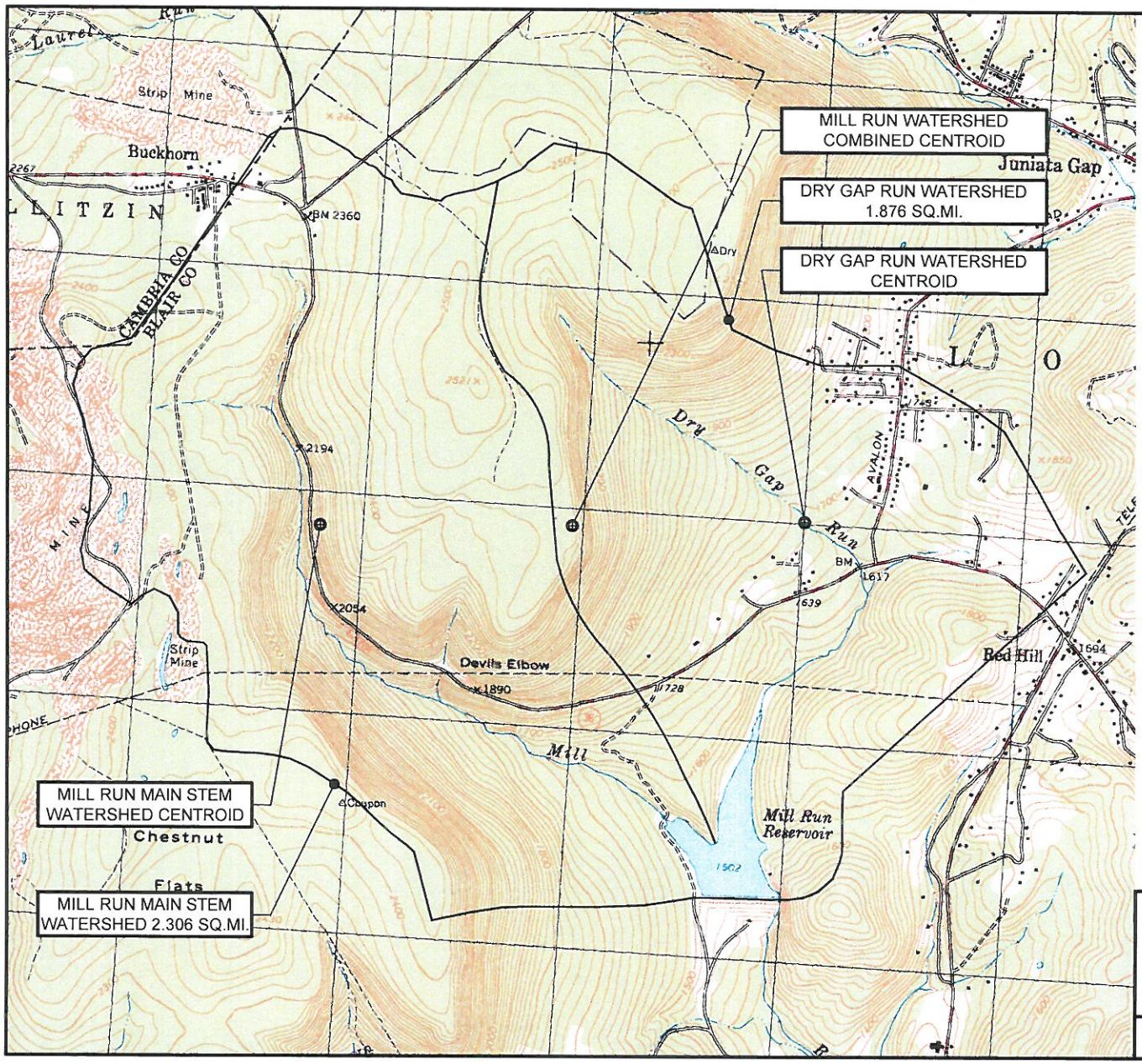
An important component of the SCS unit hydrograph is the time of concentration (t_c). The t_c is defined as the time for water to flow from the most hydrologically remote point in the watershed to the watershed outlet. Two methods were computed and compared including the NRCS and Kirpich/Kerry-Hathoway techniques. Please refer to the Appendix for computations.

Reservoir Characteristics - The storage capacity of Mill Run reservoir was determined from a 1955 survey of the basin. Storage-Area curves were developed from the bottom of the reservoir to five (5) feet above the top of the dam. Please refer to the Appendix for details.

Relevant data concerning Mill Run reservoir is as follows:

Table 3: Mill Run Reservoir Characteristics

a.	Normal Pool Elevation, Crest - Ogee Weir:	1,502.0 MSL
b.	Reservoir Area, Crest-Ogee Weir:	44.5 acres
c.	Storage, Crest-Ogee Weir:	425.6 MG (1,306.3 ac.ft.)
d.	Normal Pool Elevation, Crest-Rubber Dam:	1,508.0 MSL
e.	Reservoir Area, Crest-Rubber Dam:	50.5 acres
f.	Storage, Crest-Rubber Dam:	520 MG (1,596 - ac.ft.)
g.	Pool Elevation, Top of Dam:	1,515.0 MSL
h.	Reservoir Area, Top of Dam:	58.1 acres
i.	Storage, Top of Dam:	640.6 MG (1,966.18 ac.ft.)
j.	Pool Elevation, 5-ft. Above Dam:	1,520.0 MSL
k.	Reservoir Area, 5-ft. Above Dam:	64.5 acres
l.	Storage, 5-ft. Above Dam:	735 MG (2,256 ac.ft.)
m.	Surcharge Storage to Top of Dam:	215 MG (660 ac.ft.) from El. 1,502
n.	Surcharge Storage to 5-ft. Above Dam:	309.4 MG (950 ac.ft.) from El. 1,502



PHYSICAL WATERSHED PROPERTIES

A. DRY GAP RUN SUB-WATERSHED
1. DRAINAGE AREA = 1.875 mi²
2. SNYDER'S MAIN CHANNEL LENGTH (L) = 2.77 mi²
3. SNYDER'S CHANNEL LENGTH TO CENTROID (Lc) =1.35 mi
4. SNYDER'S LAG COEFFICIENT (Ct) = 1.55 - 1.61
5. SNYDER'S PEAKING COEFFICIENT (Cp) = 0.40 - 0.65
6. FETCH = 0.526 mi

B. MILL RUN (MAIN STEM) SUB-WATERSHED
1. DRAINAGE AREA = 2.337 mi²
2. SNYDER'S MAIN CHANNEL LENGTH (L) = 3.04 mi
3. SNYDER'S CHANNEL LENGTH TO CENTROID (Lc) = 1.48 mi
4. SNYDER'S LAG COEFFICIENT (Ct) = 1.55 - 1.61
5. SNYDER'S PEAKING COEFFICIENT (Cp) = 0.40 - 0.65
6. FETCH = 0.313 mi

C. MILL RUN WATERSHED (COMBINED)
1. DRAINAGE AREA = 4.212 mi²
2. SNYDER'S MAIN CHANNEL LENGTH (L) = 3.04 mi
3. SNYDER'S CHANNEL LENGTH TO CENTROID (Lc) = 1.02 mi
4. SNYDER'S LAG COEFFICIENT (Ct) = 1.55 - 1.61
5. SNYDER'S PEAKING COEFFICIENT (Cp) = 0.40 - 0.65
6. FETCH = 0.526 mi

GRAPHIC SCALE
0 1500 3000
(IN FEET)
1 inch = 1500 ft.

SEAL

PHYSICAL WATERSHED PROPERTIES

ALTOONA WATER AUTHORITY

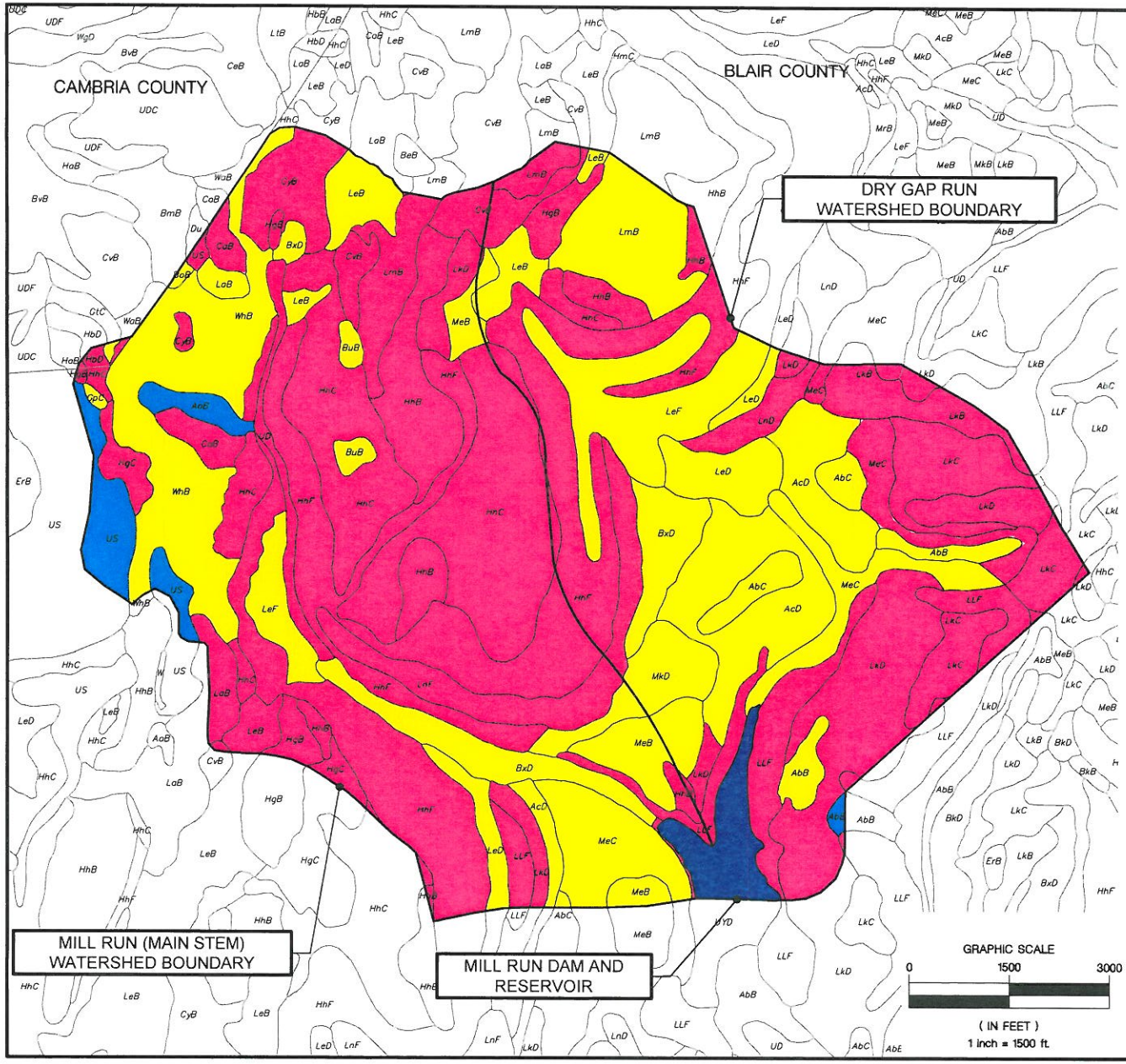
MILL RUN DAM EVALUATION

LOGAN TOWNSHIP, BLAIR CO, PENNSYLVANIA

DATE: 7/28/10	JOB: 09049	SCALE: AS SHOWN
FILE: USGS MAP	DRAWN BY: AJL	CHECKED BY: MVG

GD&F
GWIN DOBSON & FOREMAN INC.
Consulting Engineers

SHEET NO:
Fig. No.10



MILL RUN COMBINED WATERSHED SOIL GROUP CHART

SOIL GROUP	LEGEND	SQ.MI. TOTAL	PERCENT
HYDROLOGIC SOIL GROUP "B"		2.510	59.90%
HYDROLOGIC SOIL GROUP "C"		1.525	36.39%
HYDROLOGIC SOIL GROUP "D"		0.090	2.14%
WATER		0.066	1.57%
TOTAL AREA		4.190	100%

MILL RUN SUB-WATERSHED SOIL GROUP CHART

SOIL GROUP	LEGEND	SQ.MI. TOTAL	PERCENT
HYDROLOGIC SOIL GROUP "B"		1.529	66.05%
HYDROLOGIC SOIL GROUP "C"		0.675	29.16%
HYDROLOGIC SOIL GROUP "D"		0.086	3.71%
WATER		0.025	1.08%
TOTAL AREA		2.315	100%

DRY GAP RUN SUB-WATERSHED SOIL GROUP CHART

SOIL GROUP	LEGEND	SQ.MI. TOTAL	PERCENT
HYDROLOGIC SOIL GROUP "B"		0.981	52.29%
HYDROLOGIC SOIL GROUP "C"		0.850	45.31%
HYDROLOGIC SOIL GROUP "D"		0.004	0.21%
WATER		0.041	2.19%
TOTAL AREA		1.876	100%

REGISTERED PROFESSIONAL MARK V. GLENN ENGINEER No. 36538-E COMMONWEALTH OF PENNSYLVANIA

SEAL

NRCS SOILS MAP – WATERSHED AREA

ALTOONA WATER AUTHORITY

MILL RUN DAM EVALUATION

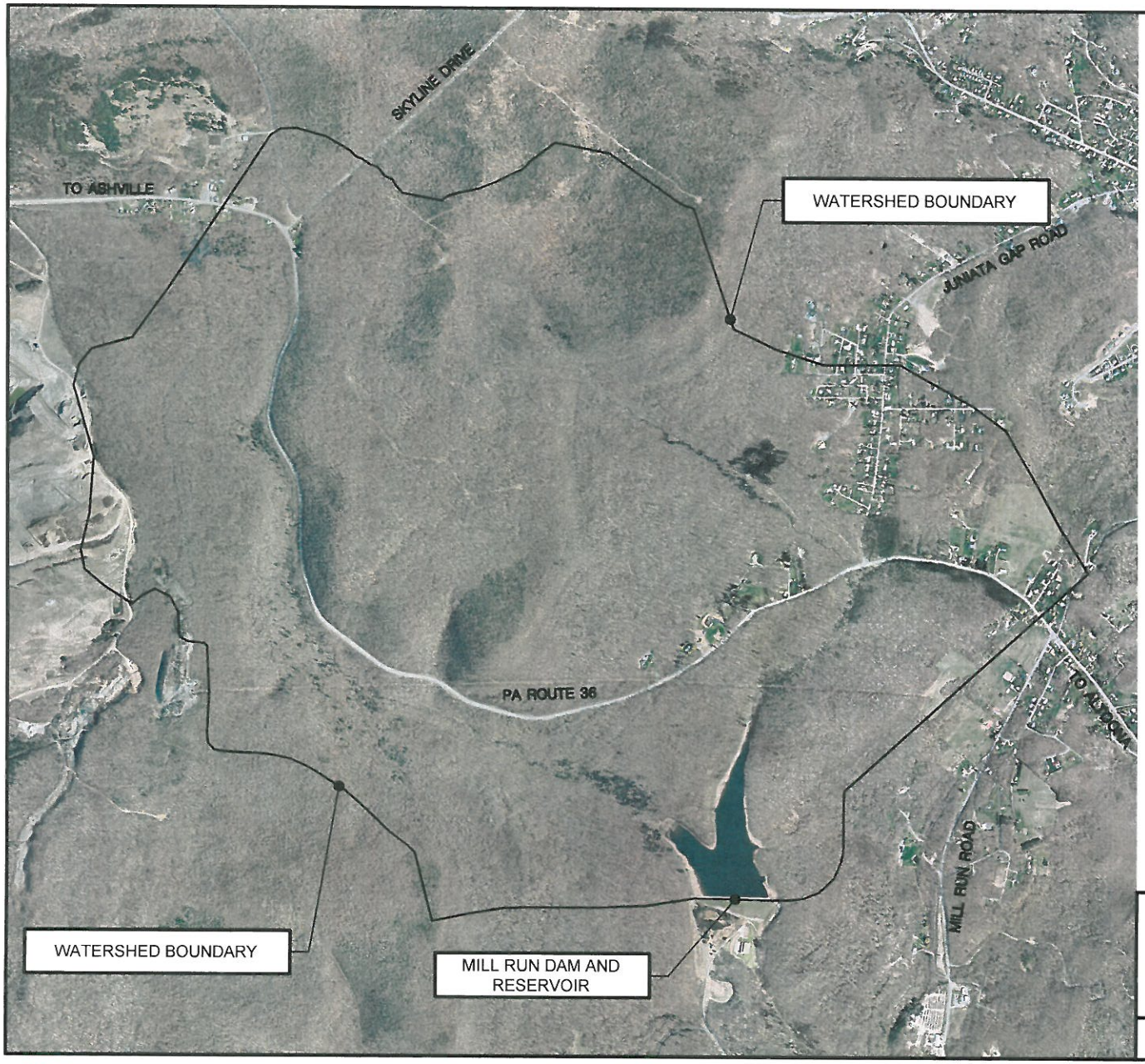
LOGAN TOWNSHIP, BLAIR CO., PENNSYLVANIA

DATE: 7/26/10	JOB: 09049	SCALE: AS SHOWN
FILE: SOILS MAP	DRAWN BY: AJL	CHECKED BY: MVG

GD&F

GWIN DOBSON & FOREMAN INC.
Consulting Engineers
3121 Fairway Drive
Altoona, PA 16602

SHEET NO:
Fig. No.11



N

GRAPHIC SCALE

0 1500 3000

(IN FEET)
1 inch = 1500 ft.

SEAL

SATELLITE IMAGE – WATERSHED AREA

ALTOONA WATER AUTHORITY

MILL RUN DAM EVALUATION

LOGAN TOWNSHIP, BLAIR CO, PENNSYLVANIA

DATE: 7/28/10	JOB: 09040	SCALE: AS SHOWN	
FILE: AERIAL MAP	DRAWN BY: AJL	CHECKED BY: MVG	

GD&F

GWIN DOBSON & FOREMAN INC.
Consulting Engineers
1011 Railway Drive
Altoona, PA 16602

SHEET NO:
Fig. No.12

Existing Spillway and Reservoir Routing - The theory of routing is related to the inflow (entering reservoir), outflow (spillway) and storage capacity of the reservoir. As water begins to flow over the spillway, the rise in the reservoir will, for a time, increase faster than it goes over the spillway. Known as "surcharge storage," this capacity has a regulating or retarding effect on the peak flow over the spillway. A study of a storm flow passing through a reservoir is called "flood routing." Spillway outflow in relation to reservoir storage is needed for proper routing of the inflow hydrograph. Spillway outflow at a specific level correlates to a corresponding volume of reservoir storage.

The spillway has an inclined upstream face with an ogee-shaped crest. An intermediate rounded pier and spillway bridge are also factors in the computation. The procedure for evaluation are set forth on p. 378-382 of the "Design of Small Dams," 2nd Edition, U.S. Department of the Interior, Bureau of Reclamation. Please refer to the Appendix for additional information.

A stage-discharge relationship was developed for the existing spillway. The capacity of the spillway to the underside of the bridge is 12,131 cfs. Above this point, the capacity of the spillway is reduced by bridge-created orifice flow. Therefore, the capacity at the top of dam is 11,151 cfs (El. 1,515).

For flow over the dam, the crest is simulated as a broad crested weir. The weir coefficient for a crest width of 20-feet is assumed to be 2.63 (Reference, p. 373, "Engineering for Dams"). The stage-discharge relationship is shown on Figure 13. The storage area curve is shown on Figure 14.

Channel Routing - Reservoir inflow can also be affected by the storage capacity of upstream drainage channels. However, channel storage attenuation is not as great as reservoir storage. Regardless, "channel routing" will be considered in the HEC-HMS model.

Several channel routing methodologies were considered. The Muskingum-Cunge has found common usage and has been adopted for this study. Channel geometry consists of trapezoid channels (with 1:1 side slopes). Manning's roughness coefficients (n = 0.05) were assumed for natural mountain streams (V.T. Chow). Please refer to the Appendix for details of channel length and slope for each sub-basin (Mill Run and Dry Gap Run sub-basins).

Precipitation - Estimates of Probable Maximum Precipitation (PMP) are provided in Hydrometerological Report No. 51 (June 1978) of the National Weather Service. Regional maps for "All Season PMP for 10 mi^2 Watersheds" (Figures 18-21) provide maximum rainfall amounts for Mill Run. A required adjustment factor of 105% (per HMR) is applied to the data as follows:

Table 4: HMR 51 PMP Rainfall

Duration (hrs.)	Cumulative Rainfall (in.)	Cumulative Rainfall with Regional Adjustment (in.)
0	0	0
6	26.3	27.6
12	30.2	31.7
24	32.9	34.5
48	36.1	37.9
72	37.8	39.7

The total rainfall (37.8 in.) exceeds the PMP estimates computed by previous Authority studies. These reports relied on the older HMR No. 40 (PMP Precipitation for Susquehanna River Drainage above Harrisburg, PA) which produced a total PMP of 34 inches. The updated precipitation total is 17% higher than previous estimates.

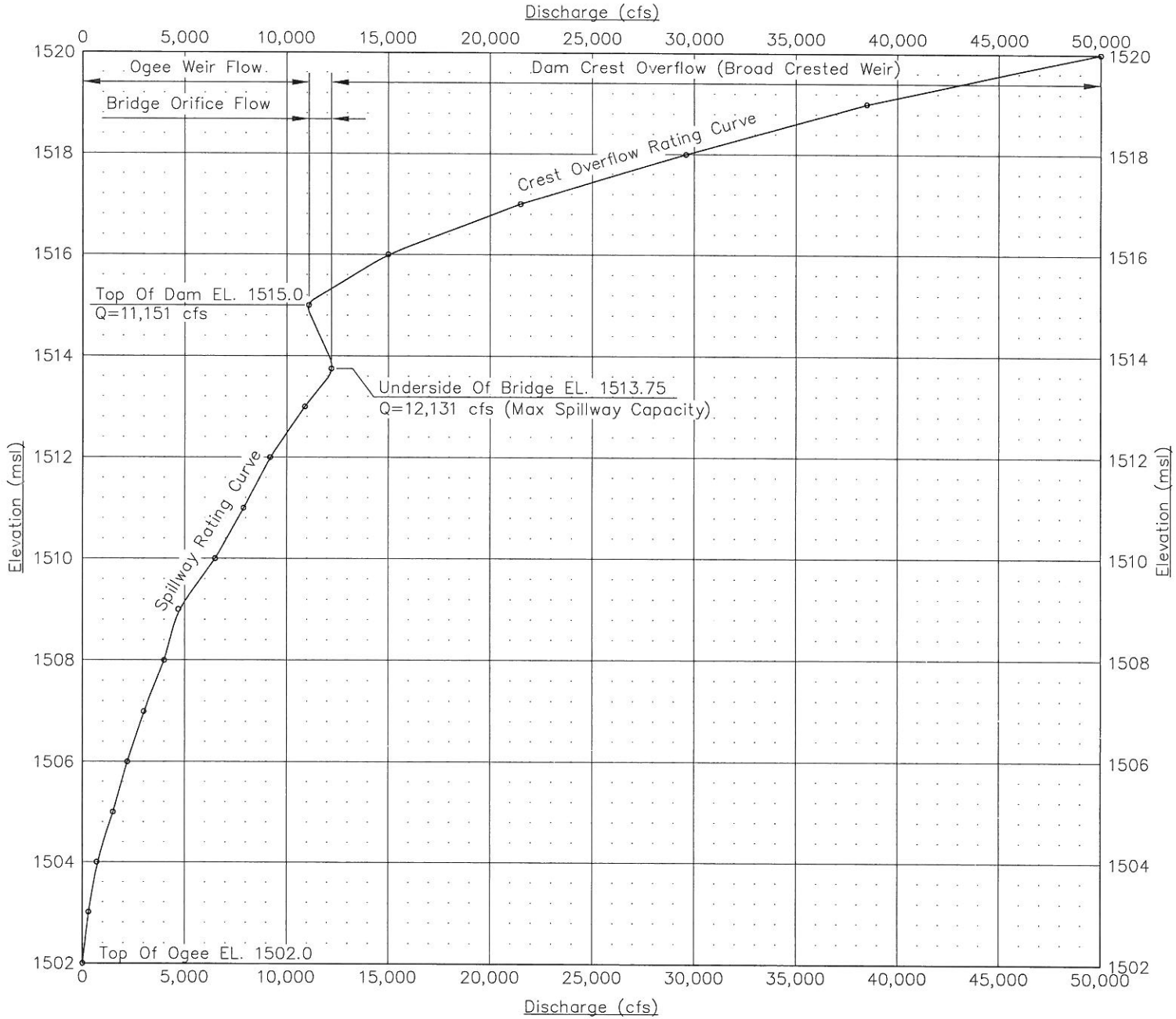


FIGURE No. 13
DISCHARGE RATING CURVE FOR RESERVOIR ROUTING PURPOSES

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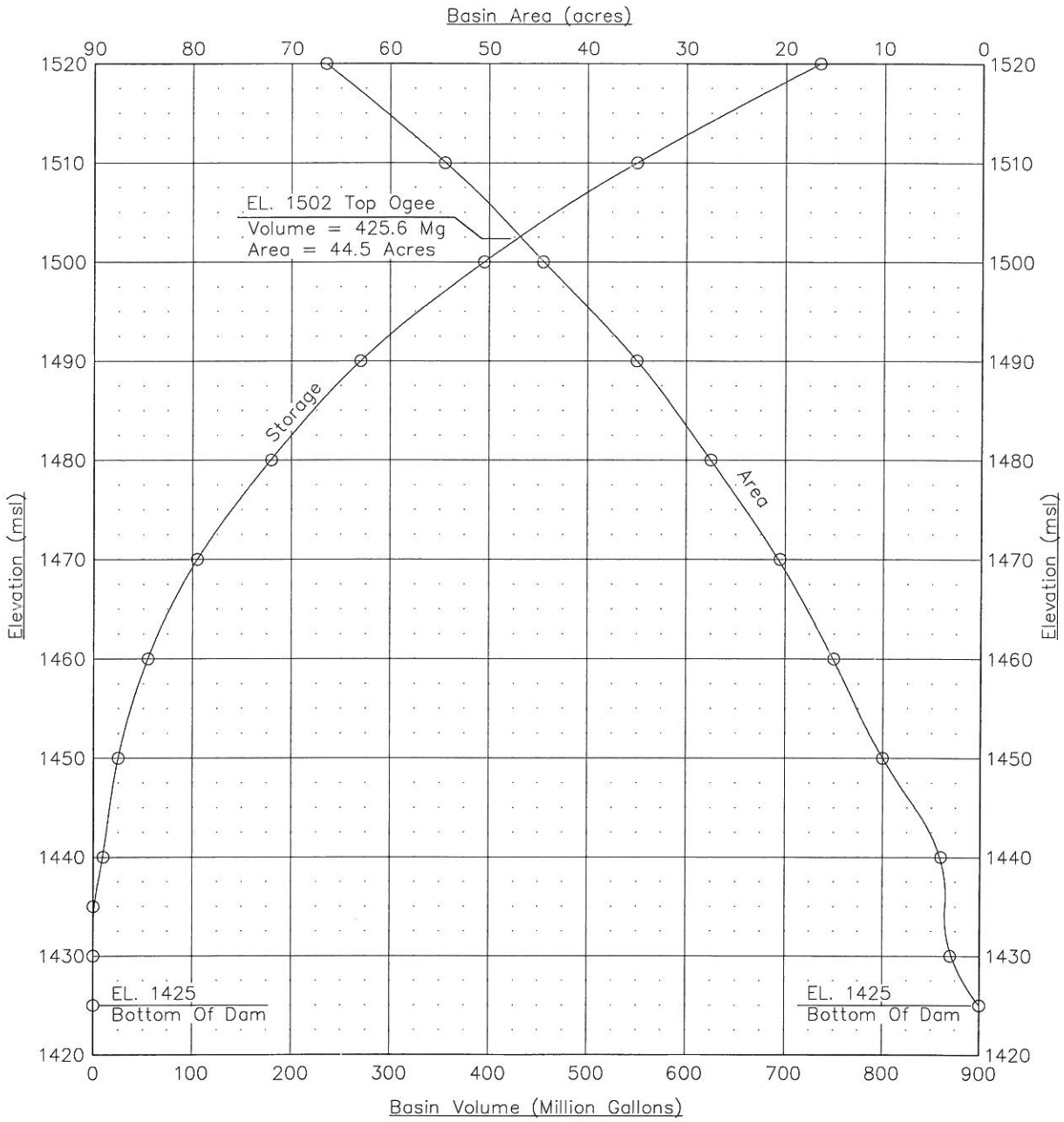


FIGURE No. 14
STORAGE-AREA CURVE-MILL RUN RESERVOIR

g:\09049\storage area curves mill run dam.dwg

This data is then plotted and intermediate rainfall depths in six (6) hour increments are determined. The increments are then grouped in 24-hour segments and arranged so that the rainfall increments produce the maximum rainfall. These groups are shown in the following table:

Table 5: PMP Increments and Sequencing for Maximum Rainfall

Time (hrs.) from Start of Storm	PMP (in.) for Time Interval
0	0.0
6	0.7
12	0.8
18	1.1
24	0.8
30	1.5
36	4.1
42	27.6
48	1.3
54	0.3
60	0.5
66	0.5
72	0.5

The time distribution of maximum hourly rainfall is further defined by COE Bulletin 52-8 which subdivides this period by the following percentages:

Table 6: Hourly Subdivision of Maximum Rainfall

Rainfall Period (Subdivision of 6-Hr.)	Time Distribution of Max. 6-Hr. PMP, % of Total 6-Hr. Rainfall
1st	10%
2nd	12%
3rd	15%
4th	38%
5th	14%
6th	11%
	100% (total)

The above percentages were applied to each 6-hour rainfall period to provide an hourly rainfall increment. Under the COE-SPF method, these hourly amounts are applied to the unit hydrograph values to produce surface runoff from rainfall excess. In HEC-HMS, the time subdivision can be selected for any interval, typically ranging from 5-15 minutes for various precipitation models. These additional time sub-divisions produce much higher rainfall peaks within the one hour time increment. Please refer to the Appendix for computations.

Losses due to soil infiltration in the watershed were computed along the guidelines set forth in COE Design Manual, "Flood Hydrograph and Analysis and Computations." An initial loss of 0.5 inches was assumed and is very small compared with the rainfall volume. The infiltration index (in./hr.) is the average rate of loss such that the volume of rainfall in excess of that rate will equal the volume of direct runoff. A loss of 0.1 in./hr. was selected. These values fall within the range of infiltration indices for the Susquehanna River basin. The total infiltration loss for the PMP event is 6.31 in. resulting in a net rainfall excess of 33.39 in.

PMF Hydrograph Development: COE-SPF Method - In this procedure, the synthetic unit hydrograph corresponds to empirical relationships developed by Snyder. These are set forth in USA COE Design Manual "Flood Hydrograph Analysis and Development." This method has been found particularly useful for ungaged watersheds similar to Mill Run. Snyder's procedure attempts to develop a relationship between the physical geometry of the basin and the resulting hydrograph. The direct runoff hydrograph (with the same storm duration) can then be computed by applying the unit hydrograph to the rainfall excess values.

The use of Snyder's coefficients has a significant effect on ultimate peak flow rates. A 1983 study entitled "Calibration of Snyder's Coefficients for PA" attempted to quantify these coefficients based on a statistical analysis of Pennsylvania drainage basins. The report provided the following coefficients for the Tyrone, PA gaging station (USGS Sta. No. 5575) on the Little Juniata River: C_p = 0.40 and C_t = 1.61. Since Mill Run is near the gaging station (if not in the watershed), the values seem to have application. They tend to approximate the coefficients used in the 1980 Phase I Study for Allegheny Reservoir. However, the coefficients have major standard deviations of 50% for C_p and 72% for C_t. We would anticipate that the C_p value of 0.40 is the minimum condition. The 1979 GD&F study for Allegheny Reservoir had a C_p value of 0.65 and would be considered the maximum condition. Therefore, the following Snyder unit hydrograph data is summarized below:

Table 7: Snyder's Unit Hydrograph Data

Minimum Condition			**Maximum Condition**		
t_p	=	2.26 hrs.	t_p	=	2.18 hrs.
t_r	=	0.41 hrs.	t_r	=	0.40 hrs.
q_p	=	113.3 cfs/sq.mi.	q_p	=	190.8 cfs/sq.mi.
Q_p	=	477 cfs	Q_p	=	803.6 cfs
t_{pr}	=	2.41 hrs.	t_{pr}	=	2.33 hrs.
q_{pr}	=	106.2 cfs/sq.mi.	q_{pr}	=	178.54 cfs/sq.mi.
Q_{pr}	=	447 cfs	Q_{pr}	=	752 cfs
t_B	=	13 hrs.	t_B	=	8 hrs.
W_{75}	=	335.3 cfs @ 3 hrs.	W_{75}	=	564 cfs @ 2 hrs.
W_{50}	=	223.5 cfs @ 5 hrs.	W_{50}	=	376 cfs @ 3 hrs.

The following assumptions were used in the preparation of the unit hydrographs:

- No separate sub-basin analysis was performed. The watershed was analyzed as a whole.
- Separate analyses were performed for the maximum and minimum Snyder's coefficients.
- Use of HMR-51 PMP, data is previously described with an interval of one (1) hour.

This analysis provides a good comparison with the original Mill Run PMF using similar techniques. The only changes are updated rainfall data and the analysis of the Mill Run watershed as a whole (no sub-basins). The results of the evaluation are as follows:

Table 8: COE-SPF Evaluation

Study	Snyder's C_t/C_p	PMF-Peak Flow (cfs)
Original 1979 GD&F Study	2.00/0.65	11,702
1980 Phase I Dam Inspection Report	N/A	12,900
Minimum Condition, Current Study	1.61/0.40	10,099
Maximum Condition, Current Study	1.55/0.65	14,190

The 1979 PMF value was 11,702 cfs. The maximum PMF value (14,190 cfs) exceeds this figure by 21%, which is attributable to the greater rainfall increments and higher peaking factor (C_p).

The maximum PMF exceeds the capacity of the existing spillway (11,151 cfs). The excess flow (3,039 cfs) overtops the dam crest by 0.98 feet. Whether this is sufficient to breach the dam is uncertain. However, an unsafe condition would exist, given the erosion potential on the downstream face.

Our COE-SPF calculation used an intensity duration of one (1) hour which may limit the time when the PMF peak can occur. Considering the time of concentration (54 minutes), the likelihood of the PMF peak occurring within the hour interval is probable. Typically, intervals of 5-15 minutes are used for evaluation purposes. PADEP has recommended a 5-minute evaluation interval. The HEC-HMS computational modeling will evaluate this time interval and several hydrograph and meteorological techniques.

PMF Hydrograph Development: HEC-HMS Method - We evaluated the following meteorological models using HEC-HMS as directed by PADEP.

- **Frequency Storm Method** - 48-hour rainfall (HMR-51) and COE rainfall distribution (CE Bulletin No. 52-8) with one hour intensity duration; evaluated over 5-minute intervals with a 0.2% probability (partial duration series) for generating the Probable Maximum Flood; SCS unit hydrograph.

- **Precipitation Gage Method** - 72-hour rainfall (HMR-51) and COE rainfall distribution (CE Bulletin No. 52-8) with one hour intensity duration; evaluated over 5-mintue intervals and entered as a "precipitation gage" time-series for generating the Probable Maximum Flood; SCS unit hydrograph.

- **SCS Storm Method** - 24-hour NOAA Atlas 14 (36-0140) precipitation event with a Type II SCS rainfall distribution; evaluated inflow-outflow characteristics of various spillway configurations up to 100-year flood; SCS unit hydrograph.

The physical parameters including watershed, stream reaches, routing criteria, reservoir and spillway data were entered into the model as previously described. Please refer to the Appendix for relevant HEC-HMS data screens for this information.

The model computed reservoir outflow for the single basin (entire Mill Run watershed) and sub-basin (Mill Run and Dry Gap Run sub-watersheds) scenarios through the existing spillway as follows:

	Meteorological Model	Single Basin PMF	Multi-Basin PMF
a.	48-Hour COE Frequency Storm	20,341 cfs	20,898 cfs
b.	72-Hour COE Precipitation Gage Storm	21,447 cfs	21,987 cfs
c.	24-Hour NOAA 100-Year SCS Storm	1,442 cfs	1,472 cfs

The above data indicates a unit discharge of 5,220 cfs/sq.mi. Several model runs using the SCS unit hydrograph/rainfall distribution and 5-minute duration interval produced PMF peaks of 26,000 - 29,000 cfs or unit values of 6,200 - 6,900 cfs/sq.mi. respectively. When compared to recent PMF studies on other comparable, regional watersheds, the SCS unit values appear to be excessive. Therefore, the SCS methodology could excessively concentrate runoff and produce peak values that, in our judgment, are excessive for this application.

In addition, the lag time for the SCS unit hydrograph (UH) uses a reduction factor of 0.6 times the t_c value. This will also provide a high SCS UH peak. Given the inherent uncertainty of computing completely accurate times of concentration values for mountainous/upland watersheds and the fact that there is no suitable nearby gaging station to calibrate the watershed, the value of 54 minutes (Mill Run, Main Stem) and 40 minutes (Dry Gap Run) were directly entered in the watershed transform. PADEP agreed with this assessment as they computed a lag time of 47 minutes which is close to the GD&F value. They indicated t_c should not be construed as an acceptable substitute for lag time in all SCS models.

The 72-Hour COE precipitation gage storm applied to the multi-basin model produces the maximum flow of 21,987 CFS. This value compares to the February 2000 dam breach flow of 22,034 cfs using a 6-hour HMR-51 rainfall and SCS storm distribution.

Based on our computations, the existing spillway has a maximum spillway capacity of 12,131 cfs or 55% of the probable maximum flood. A larger spillway is needed for the proposed Spillway Design Flood of 22,000 cfs. In addition, the maximum stage at this flow is El. 1517 ft. msl or two feet above the dam crest. In all likelihood, this stage would be sufficient to breach the dam.

Downstream Floodplain Considerations - We then examined the downstream effects of a larger spillway during certain flood events. Specifically, the 100-year storm yields a flow of 1,442 cfs through the existing spillway. The pool elevation and storage associated with this flow is El. 1505.1 feet (msl) and 471.50 million gallons, respectively. This equates to a total head on the existing ogee weir (El. 1502 feet (msl) of 3.1 feet.

Although a wider spillway (130-ft. at same overflow elevation) will be sufficient for the SDF, routing of the new inflow-outflow characteristics produces a 100-year flow of 1,768 cfs. In other words, a wider spillway at the same overflow elevation will measurably increase discharges (+23%) for storm events up to the 100-year flood. The Department has stated that the 100-year flood plain must be maintained downstream for any spillway reconfiguration.

The 1986 FEMA flood study assigned a 100-year peak discharge of 1,455 cfs for the Mill Run reach between Allegheny Reservoir and the twin-railroad culverts (6.93 sq.mi.). We "prorated" the FEMA flow to the upstream Mill Run reservoir watershed (4.212 sq.mi.) by using the Bureau of Reclamation "gage transfer" equation ($Q_1 = Q_2$ (sq. root A_1/sq. root A_2). This yields a 100-year flood of 1,134 cfs equivalent to a rainfall depth of 5.4-inches in the SCS Type II storm model.

The model generated a peak discharge of 1,455 cfs for the NOAA 24-hour, 100-year rainfall (5.96-inches) and SCS Type II storm. A staged spillway will control pool levels and discharges up to 1,442 cfs which will be sufficient for the prorated FEMA flow (1,134 cfs) and 1,455 cfs in the downstream reach.

HYDRAULIC ALTERNATIVES EVALUATION

General - The alternatives evaluation of preventing PMF overtopping of Mill Run dam is based on several factors including serviceability and cost-effectiveness. We evaluated the following options:

- Staged Primary and Emergency Ogee Weir Spillways with Floodwall
- Staged Primary and Emergency Labyrinth Spillways
- Roller Compacted Concrete Overtopping Protection Spillway
- Upgrade of Existing Spillway with Floodwall
- Unstaged Ogee Weir Level Spillway

Each alternative will be designed according to the previous hydrologic criteria. A separate cost-evaluation will also be performed.

Location - The location of the spillway is governed by the downstream channel. The existing spillway discharges to Mill Run to the east of the water treatment plant. Any new spillway configuration must discharge at this location. The proposed location of several spillway options is about 100-feet west of the existing spillway. This provides, hydraulically, the simplest and, economically, the most cost effective solution. The discharge of the spillways (except for a roller compacted concrete overtopping protection option) will be directly to the rear of the water treatment. The stream channel will be enlarged to prevent tailwater formation. It will also, as far as possible, prevent flooding of the Mill Run water treatment for all but the most extreme events.

This spillway location will place its foundation on the earth/rockfill embankment rather than bedrock. This arrangement has been typical for many of the Authority spillway projects. (Kittanning Point, Impounding, Kettle and Plane 9 dams). The justification is that this embankment material has been preconsolidated by virtue of material settlement since construction of the dam. This assumption has proven valid for the above spillways built over the last 25 years. No evidence of settlement or displacement is in evidence. Therefore, the proposed spillways will be placed in that portion of the dam forming the west shoulder of the existing spillway in rock/soil embankment material.

Pool Elevation - Concerning "normal pool" elevation, consideration must be given to the operational characteristics of Mill Run reservoir. The Altoona Water Authority has operated an inflatable dam for added water storage. A 6-foot inflatable dam has functioned since 1990 replacing an earlier 4-foot inflatable dam installed in 1968. The state had permitted installation of these devices because of the freeboard available at Mill Run dam (13-feet).

For maintenance purposes, the Authority wishes to decommission the inflatable dam but raise the normal pool to compensate for the loss of storage. Therefore, we have modeled the staged spillway for various pool elevations including El. 1502 (existing weir overflow), El. 1506 (original 4-ft. inflatable dam) and El. 1508 (current 6-ft. inflatable dam). Obviously, the higher pool elevations will necessitate a floodwall across the dam crest to maintain spillway head. The storage-discharge and elevation-discharge functions for these alternatives are included in the Appendix.

Staged-Primary and Emergency Ogee-Weir Spillways Option - It is clear that a wider spillway set at the same overflow elevation will not meet PADEP criteria. The new spillway configuration must not exceed the 100-year discharge (1,455 cfs) at the maximum pool elevation (El. 1,505 ft.) under existing conditions. This condition typically necessitates a staged spillway. For analysis purposes, we have tested a staged spillway model to examine its effect. The first stage would control the pool elevation and spillway discharge up to the 100-year flood. The second stage spillway would be set at the maximum pool elevation (El. 1505 ft, msl) under existing conditions. Both stages would be designed to pass the 22,000 cfs PMF.

In order to increase the effective head on the spillway and minimize its length, a 4-foot (nominal) floodwall is proposed. The maximum water surface during the SPF is El. 1,519. Our preliminary calculations find the weir length of the first stage spillway is 86-feet with an available head of 13 head (13,747 cfs total capacity). The depth of the first stage spillway is 3.1 feet. The second stage spillway also has a 68-foot weir, but with an available head of 10 feet (9,038 cfs total capacity). The top of the second stage ogee weir was set at the pool elevation (El. 1505.1 msl) corresponding to the 100-year NOAA flood through the existing spillway. The total capacity of the spillway including the first and second stages is 22,000 cfs.

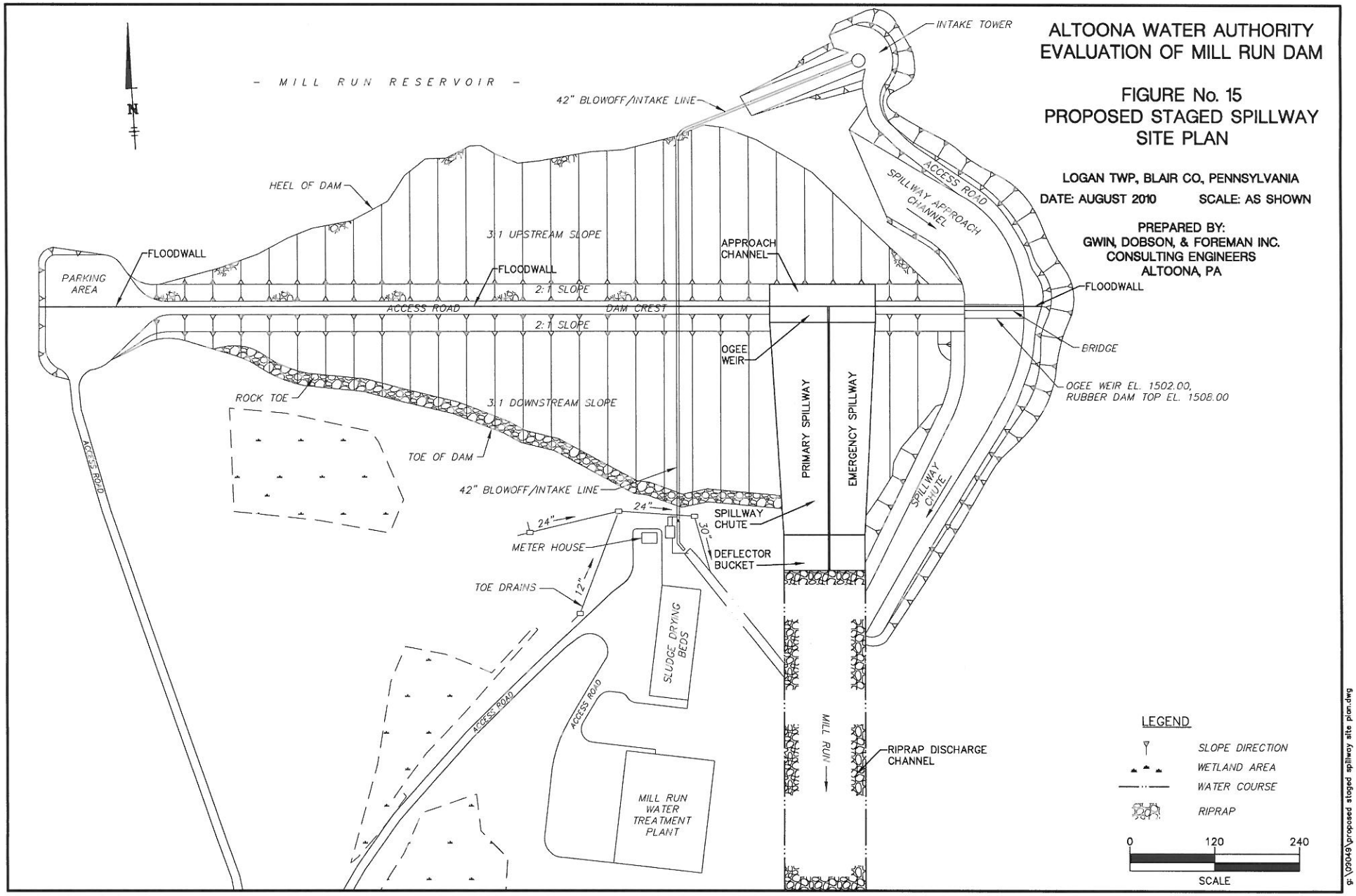
This alternative involves basically two (2) reinforced concrete spillways. A primary or "service spillway," 86-feet long, will convey all flow up to the 100-year storm event (1,400 cfs). This places the top of weir at elevation 1,506 with a depth of flow of 3-feet. An emergency spillway, with a width of 68-feet and a ogee weir elevation of 1,509, would assist in discharging extreme storms up to the Probable Maximum Flood. The primary spillway would have a capacity of 14,177 cfs and emergency spillway with a capacity of 7,823 cfs. The spillways will be divided by a rounded pier at the approach channel and common wall in the spillway chute. The layout of the weir is controlled by a common approach channel elevation (El. 1,499.5). This necessarily affects the respective hydraulic capacity of the respective spillways.

The ogee crest design relative to shape and submergence potential was based on criteria in "Design of Small Dams," p. 377. Design of the spillway chute involved a conventional, gradually-varied flow analysis. The "standard-step" computational method established flow depths and velocities in the chute.

The terminal structure, spillway chute freeboard and convergence analysis are also based on "Design of Small Dams" criteria. The energy dissipater utilized is a "deflector bucket" which is similar to other Authority spillways. This device is effective in directing the high velocity discharge beyond the toe of the dam. Although the downstream channel may be heavily eroded by the discharged jet, energy dissipation will occur and prevent formation of eddy currents near the spillway wing walls.

The complete design of the staged spillway structure is shown in the Appendix. The layout of the spillway and its location on the embankment is shown in the following figures (No.'s 15 - 17).

HEC-HMS was run based on the storage-discharge relationship of this staged ogee configuration. Routing of the PMF through the reservoir and spillway produces a maximum outflow of 21,190 cfs based on a peak inflow of 22,258 cfs, a reduction of 1,068 cfs (4.8%). Please refer to the Appendix for model output results.


ALTOONA WATER AUTHORITY
EVALUATION OF MILL RUN DAM
FIGURE No. 15
PROPOSED STAGED SPILLWAY
SITE PLAN
LOGAN TWP, BLAIR CO., PENNSYLVANIA
DATE: AUGUST 2010
SCALE: AS SHOWN
PREPARED BY:
GWIN, DOBSON, & FOREMAN INC.
CONSULTING ENGINEERS
ALTOONA, PA
- MILL RUN RESERVOIR -
N
INTAKE TOWER
42" BLOWOFF/INTAKE LINE
HEEL OF DAM
3:1 UPSTREAM SLOPE
FLOODWALL
PARKING AREA
FLOODWALL
2:1 SLOPE
ACCESS ROAD
DAM CREST
2:1 SLOPE
APPROACH CHANNEL
SPILLWAY APPROACH CHANNEL
ACCESS ROAD
FLOODWALL
BRIDGE
OGEE WEIR
OGEE WEIR EL. 1502.00,
RUBBER DAM TOP EL. 1508.00
ROCK TOE
3:1 DOWNSTREAM SLOPE
PRIMARY SPILLWAY
EMERGENCY SPILLWAY
TOE OF DAM
ACCESS ROAD
42" BLOWOFF/INTAKE LINE
24"
24"
SPILLWAY CHUTE
SPILLWAY CHUTE
METER HOUSE
30"
DEFLECTOR BUCKET
TOE DRAINS
12"
SLUDGE DRYING BEDS
ACCESS ROAD
ACCESS ROAD
MILL RUN WATER TREATMENT PLANT
MILL RUN
RIPRAP DISCHARGE CHANNEL
LEGEND
SLOPE DIRECTION
WETLAND AREA
WATER COURSE
RIPRAP
0
120
240
SCALE
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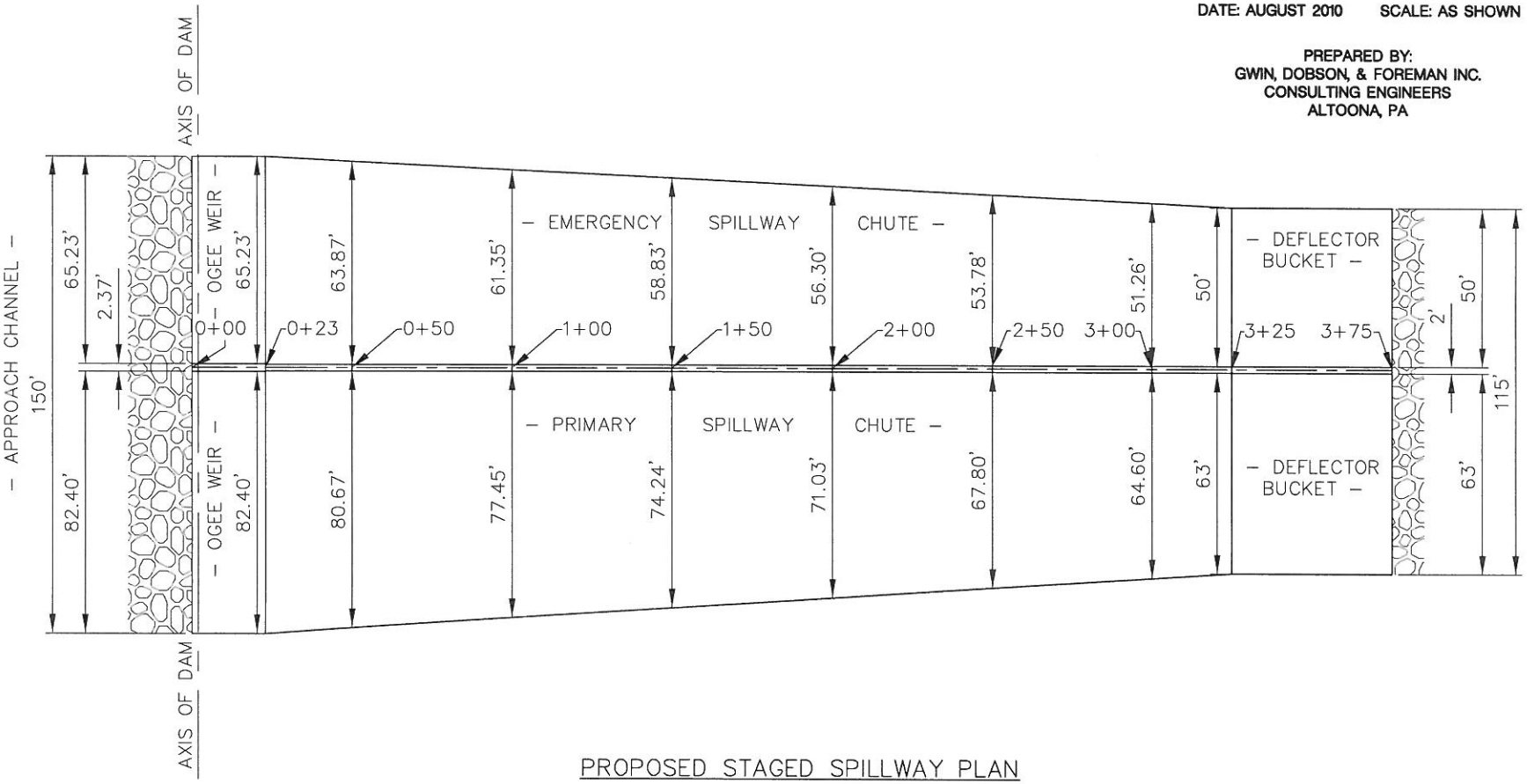
ALTOONA WATER AUTHORITY
EVALUATION OF MILL RUN DAM

FIGURE No. 16
PROPOSED STAGED SPILLWAY PLAN

LOGAN TWP, BLAIR CO, PENNSYLVANIA

DATE: AUGUST 2010 SCALE: AS SHOWN

PREPARED BY:
GWIN, DOBSON, & FOREMAN INC.
CONSULTING ENGINEERS
ALTOONA, PA



PROPOSED STAGED SPILLWAY PLAN

0 25 50
SCALE

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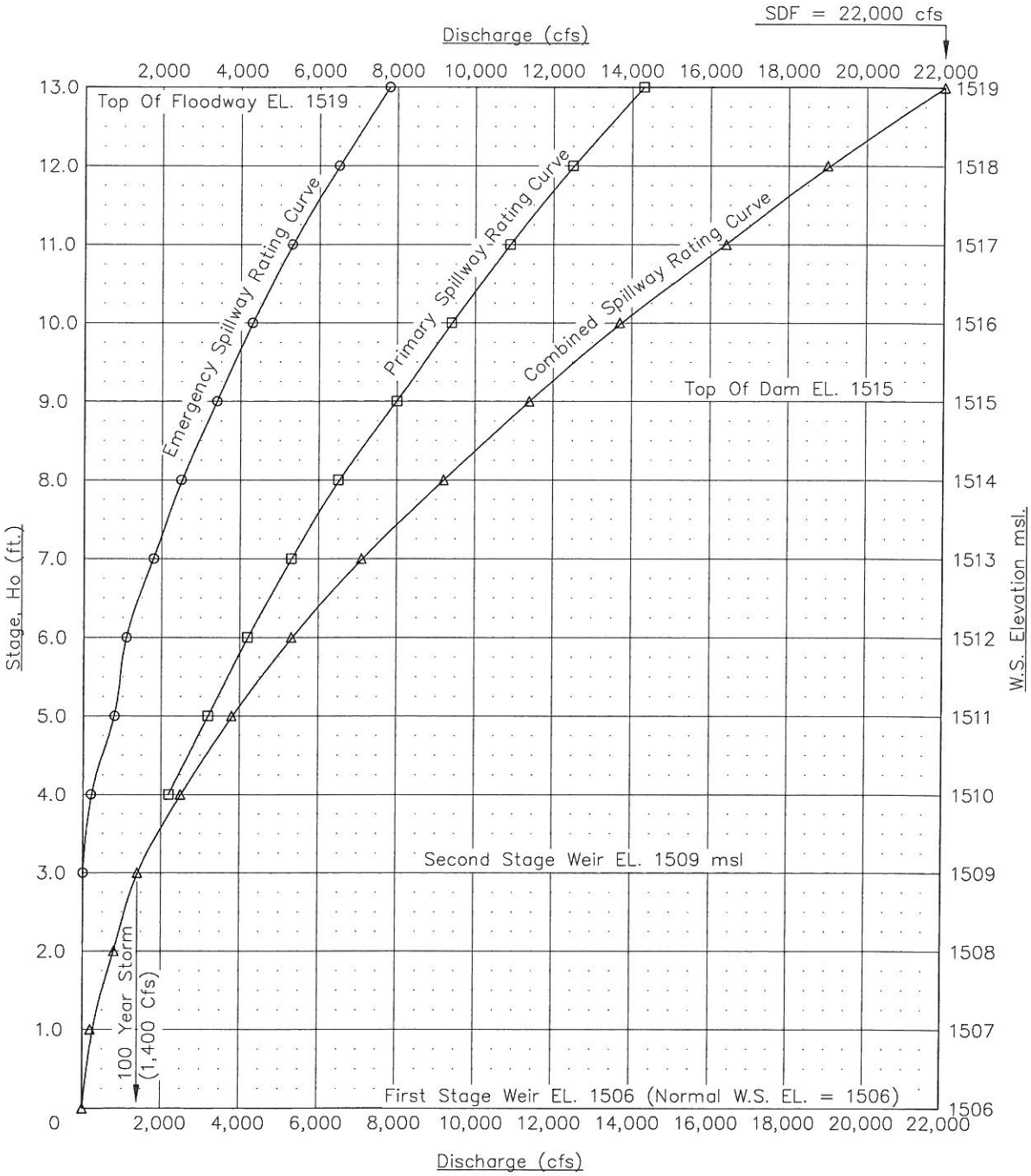

FIGURE No. 17
PROPOSED STAGE–DISCHARGE CURVE
TWO–STAGE SPILLWAY (W.S. EL. 1506)

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Staged Primary and Emergency Labyrinth Spillways - The most significant parameters in computing weir capacity are its height relative to the upstream depth, the crest shape and the crest length. Of these, the crest length has the greatest influence on spillway capacity. Most weirs are placed perpendicular to the flow direction.

The crest length can be increased by folding the weirs in "accordion" fashion. These weirs are known as labyrinth spillways. The following photo is a typical example. The principal advantage of a labyrinth weir is that spillway capacity can be increased for a given channel width. The total length of the labyrinth weir is typically 3-5 times the spillway width and its capacity is twice that of a standard weir of the same width. They have seen increased application in rehabilitation of existing dams with low spillway capacities.

Key factors in labyrinth weir design include length and width, crest height, labyrinth angle, number of cycles and other minor variables (wall thickness, crest shape and apex configuration). GD&F used "Design of Labyrinth Spillways," by Tullis, Amanian & Waldron, 1995, ASCE Journal of Hydraulic Engineering, for design purposes.

Key design parameters include the following:

a. Labyrinth should be mounted on a horizontal apron with the upstream and downstream elements at the same elevation. The flow downstream from the labyrinth should be supercritical to avoid submergence of the crest.

b. The ratio of total head (H_t) to the weir height (P) should be less than 0.9.

c. The crest shape should be "quarter round" which produces a higher weir coefficient and higher heads.

d. The inside width of the opex should be twice the wall thickness. Walls may be tapered.

e. Relationship of weir coefficients and H_t/P based on labyrinth angle regression equations (Tullis, Waldron, Amanian, Baasirj, et.al).

f. The height aspect ratio (W/P) or unit labyrinth width to weir height should be greater than 2.0 and less than 3.0.

g. The magnification ratio (length of the labyrinth crest divided by cycle width) should be less than 10.

h. Optimum angle of side legs ranges from 7° to 16°.

i. The major design variables are the side wall angle and number of cycles. These are varied to provide maximum capacity for a given spillway channel width and to minimize the total volume of concrete.

The design for the labyrinth spillway is provided in the Appendix. A staged spillway concept has been considered. The cycles and side angles have been varied to produce the most functional and cost effective layout. Key design elements are as follows:

Figure No. 18: Labyrinth Spillway Discharge

Figure No. 19: Labyrinth Spillway

Table 9: Labyrinth Spillway Design Elements & Capacity

a.	Primary Spillway			
	1. Weir Crest Elevation	-	1506	msl.
	2. Weir Height (P)	-	12.0	ft.
	3. Head on Weir (H_t)	-	8.7	ft.
	4. No. of Cycles	-	1	
	5. Depth Aspect Ratio, H_t/P	-	0.725	
	6. Angle of Side Wall	-	16°	
	7. Crest Coefficient (C_o)	-	0.425	
	8. Total Weir Length (L)	-	106.67	ft.
	9. Labyrinth Width (W)	-	35.24	ft.
	10. Height Aspect Ratio (W/P)	-	2.94	
	11. Apron Length	-	49.33	ft.
	12. Magnification Ratio (L/W)	-	3.03	
	13. Total Volume of Concrete	-	223	CY
	14. Primary Spillway Capacity at W.S. El. 1509.0	-	1,400	cfs (100-yr. flood)
	15. Total Primary Spillway Capacity	-	5,573	cfs
b.	Secondary Spillway			
	1. Weir Crest Elevation	-	1509	msl.
	2. Weir Height	-	15	ft.
	3. Head on Weir (H_t)	-	5.85	ft.
	4. No. of Cycles	-	3	
	5. Depth Aspect Ratio, H_t/P	-	0.39	
	6. Angle of Side Wall	-	14°	
	7. Crest Coefficient (C_o)	-	0.525	
	8. Total Weir Length (L)	-	449.81	ft.
	9. Labyrinth Width (W)	-	127.31	ft.
	10. Height Aspect Ratio (W/P)	-	2.82	
	11. Apron Length	-	70.82	ft.
	12. Magnification Ratio (L/W)	-	3.53	
	13. Total Volume of Concrete	-	1,168	CY
	14. Secondary Spillway Capacity	-	16,430	cfs
c.	Combined Spillway			
	1. Total Capacity	-	22,011	cfs (SPF)
	2. Total Length of Weir	-	162.55	ft.

The rating curves for the primary, secondary and combined labyrinth spillway are shown in Figure 20.

Several considerations should be noted about labyrinth spillway performance and serviceability. These are summarized as follows:

- Nappe interference can occur when jets from the two sidewalls (or the apex and sidewall) intersect and cause decreased capacity of the weir. As long as the aspect ratios (W/P <3.0 and Ht/P <0.9) are maintained, this effect should not be significant. The minor "length of disturbance" is accounted for in the calculation.

- At low heads, nappe vibration can occur causing noise and pressure fluctuations on the labyrinth sidewalls. Lack of aeration and general flow instability can be alleviated by placing air vents in the sidewalls and flow splitters on the crest.

- Projecting a labyrinth weir into the reservoir enhances its capacity. The Mill Run labyrinth weirs project 52 feet from the dam axis into the reservoir.

- The magnification ratio (or the crest length divided by the width), should be more than 2.0 and less than 10.0. For the Mill Run primary and secondary labyrinth spillways, the ratios are 3.03 and 5.53, respectively.

- Concerning deposition of sediment in the apron between the weir bays, model studies have shown the labyrinth weir to be self-cleaning. Any sediment deposited during low flows will be scoured out by the violent turbulent action at the downstream apex.

- Ice formation can exert a pressure on the labyrinth walls of up to 10,000 lb./ft^2. One method of relieving pressure is to provide a mechanism to break the ice sheet upstream of the labyrinth. An upstream slope wall can cause the ice sheet to break, thus alleviating the problem.

- Hydraulic modeling studies of the labyrinth weir are an aid to design of non-typical spillway applications. These studies can confirm design principles or provide data on specific weir coefficients and overflow stability.

Most of the above considerations would be validated in final design. It is sufficient to observe that many of the basic design parameters have been considered in this preliminary study. This is considered sufficiently accurate for purposes of project feasibility.

Labyrinth weirs have found general applications in the dam safety industry. Numerous projects have been designed and constructed over the last 30 years with satisfactory performance. Certain design principles have evolved over this time and have proven generally reliable in practice.

The downstream spillway chute has been designed in a similar fashion as the staged ogee spillway. A gradually-varied flow, standard step computation has been performed along with terminal structures computations. These calculations are shown in the Appendix. Please refer to the following figures of the labyrinth spillway proposed for Mill Run dam.

HEC-HMS was run based on the storage-discharge relationship of this labyrinth spillway configuration. Routing of the PMF through the reservoir and spillway produces a maximum outflow of 21,585 cfs based on a peak inflow of 22,258, a reduction of 673 cfs (3%). Please refer to the Appendix for model output results.

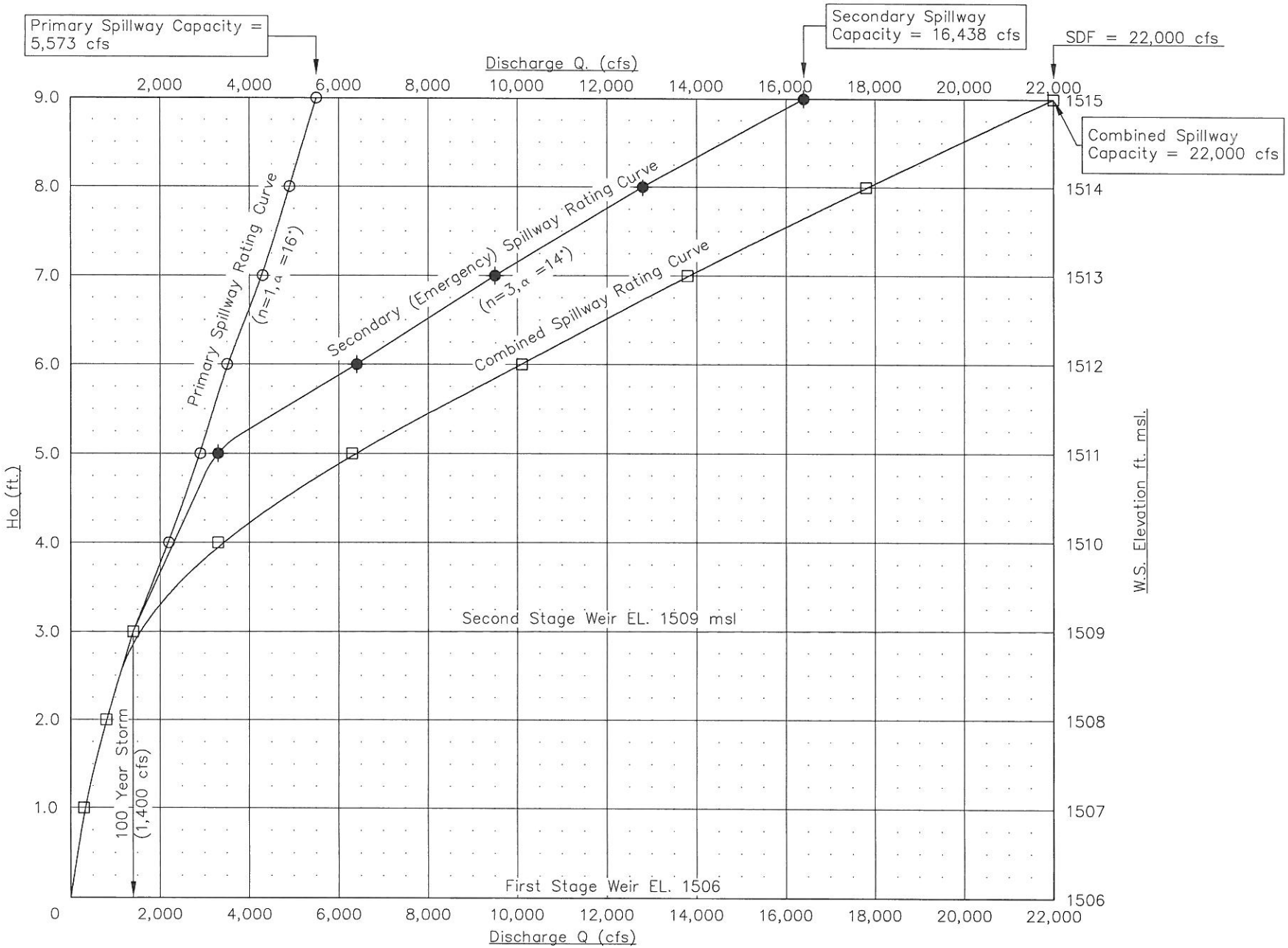

FIGURE No. 20 – LABYRINTH SPILLWAY RATING CURVE

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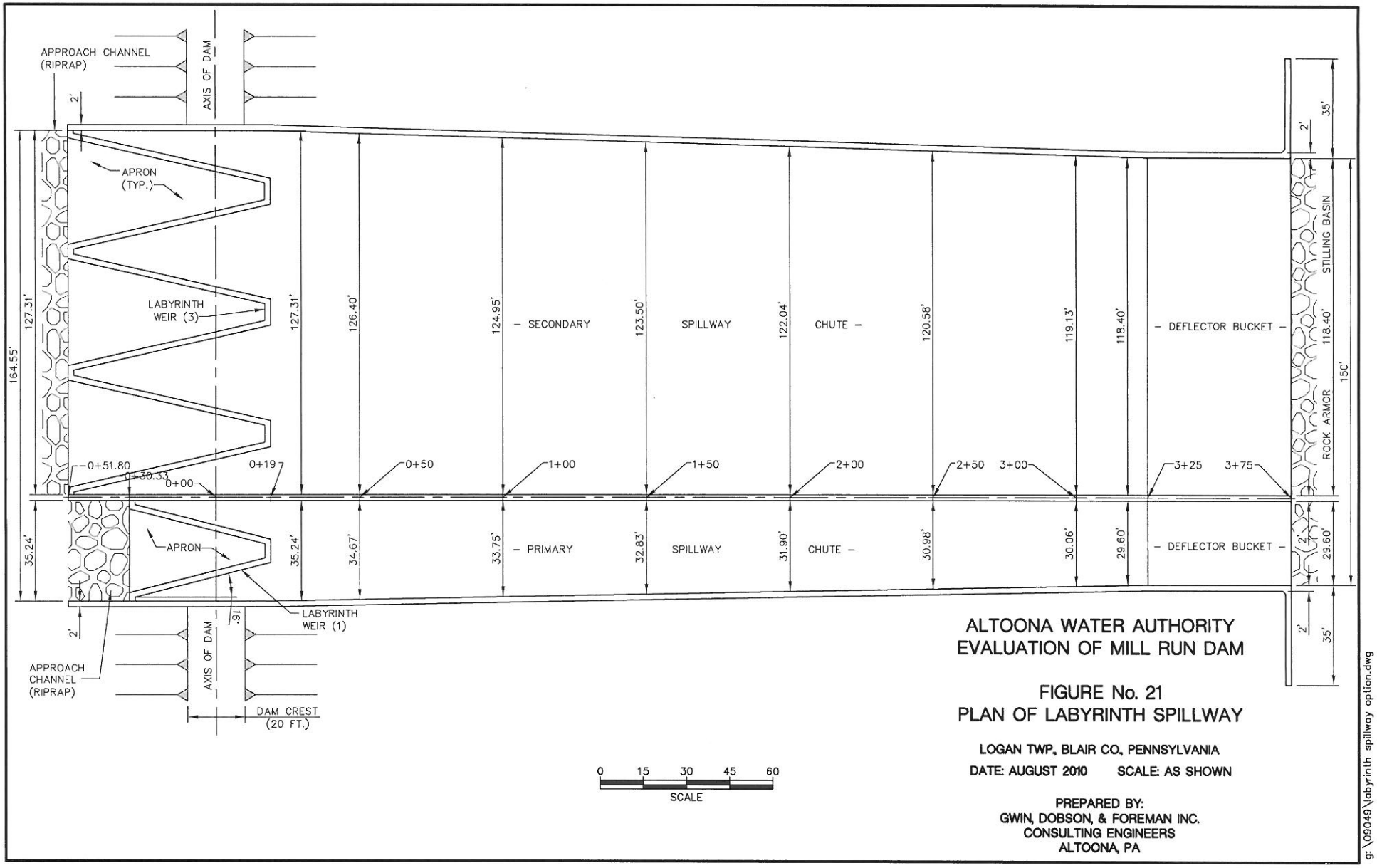

ALTOONA WATER AUTHORITY
EVALUATION OF MILL RUN DAM

FIGURE No. 21
PLAN OF LABYRINTH SPILLWAY

LOGAN TWP., BLAIR CO., PENNSYLVANIA

DATE: AUGUST 2010 SCALE: AS SHOWN

PREPARED BY:
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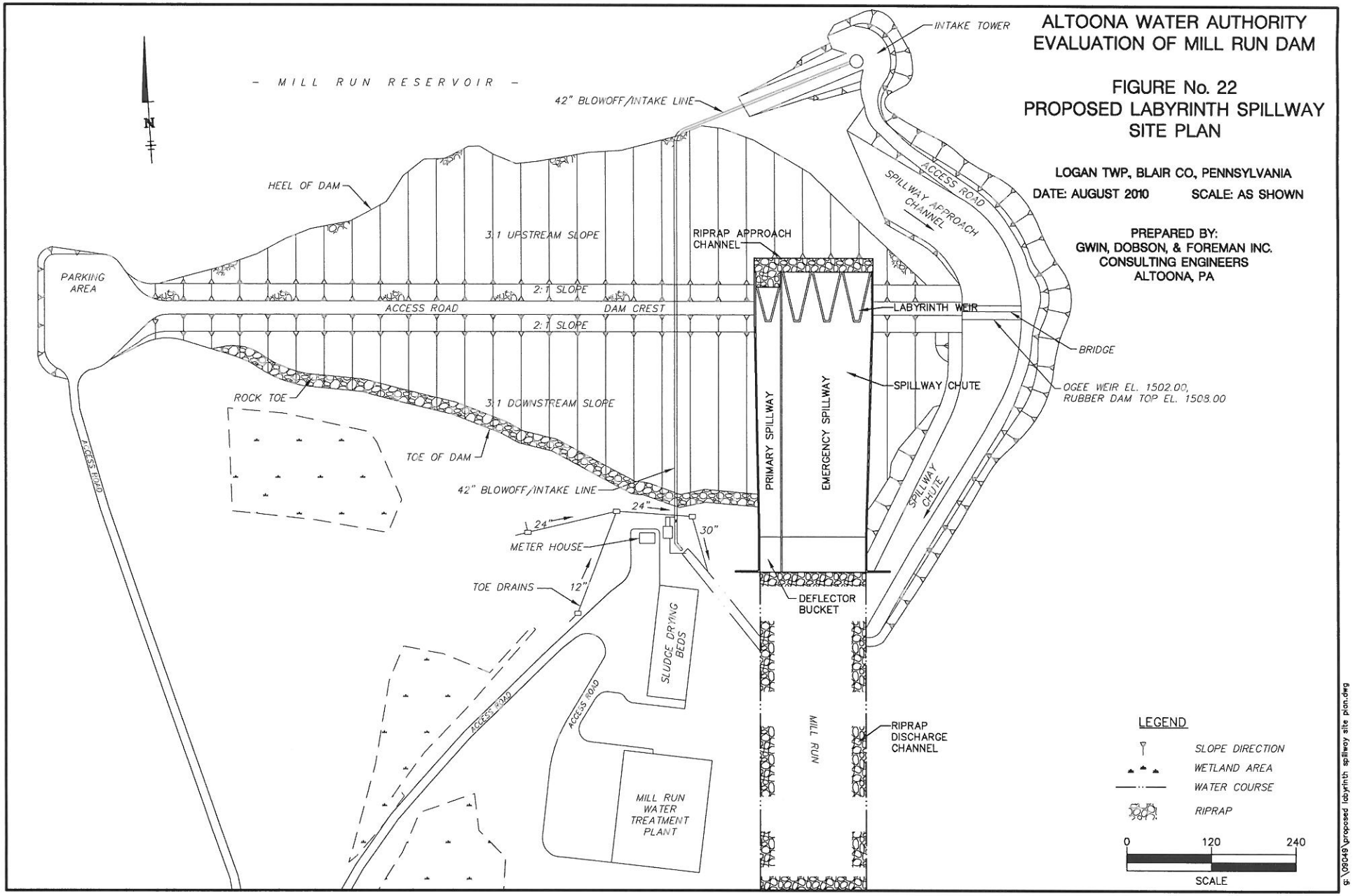
CONSULTING ENGINEERS
ALTOONA, PA

g:\09049\labyrinth spillway option.dwg


ALTOONA WATER AUTHORITY
EVALUATION OF MILL RUN DAM
FIGURE No. 22
PROPOSED LABYRINTH SPILLWAY
SITE PLAN
LOGAN TWP, BLAIR CO, PENNSYLVANIA
DATE: AUGUST 2010
SCALE: AS SHOWN
PREPARED BY:
GWIN, DOBSON, & FOREMAN INC.
CONSULTING ENGINEERS
ALTOONA, PA
- MILL RUN RESERVOIR -
N
INTAKE TOWER
42" BLOWOFF/INTAKE LINE
HEEL OF DAM
3:1 UPSTREAM SLOPE
RIPRAP APPROACH CHANNEL
SPILLWAY APPROACH CHANNEL
ACCESS ROAD
PARKING AREA
2:1 SLOPE
ACCESS ROAD
DAM CREST
2:1 SLOPE
LABYRINTH WEIR
BRIDGE
OGEE WEIR EL. 1502.00,
RUBBER DAM TOP EL. 1503.00
SPILLWAY CHUTE
ROCK TOE
3:1 DOWNSTREAM SLOPE
PRIMARY SPILLWAY
EMERGENCY SPILLWAY
TOE OF DAM
ACCESS ROAD
42" BLOWOFF/INTAKE LINE
24"
24"
30"
METER HOUSE
TOE DRAINS
12"
DEFLECTOR BUCKET
SLUDGE DRYING BEDS
ACCESS ROAD
ACCESS ROAD
MILL RUN
RIPRAP DISCHARGE CHANNEL
MILL RUN WATER TREATMENT PLANT
LEGEND
SLOPE DIRECTION
WETLAND AREA
WATER COURSE
RIPRAP
0
120
240
SCALE
g:\09049\proposed labyrinth spillway site plan.dwg

Roller Compacted Concrete Overtopping Protection Emergency Spillway and Primary Spillway Option - Roller Compacted Concrete (RCC) was initially developed to produce a material exhibiting the structural properties of concrete with the placement characteristics of embankment materials. Theoretically, the result is a material that, when properly designed and constructed as a gravity structure, should be more economical than comparable earth-rockfill or conventional concrete structures.

The American Concrete Institute (ACI) defines RCC as a *"concrete compacted by roller compaction concrete that, in its unhardened state, will support a roller while being compacted."* Properties of hardened RCC can be similar to those of conventionally placed concrete. The term "roller compaction" is also defined by ACI as a *"process for compacting concrete using a roller, often a vibratory roller."* Please refer to the attached photos of completed RCC projects and construction placement techniques.

RCC has become a cost effective method for new dam construction and the rehabilitation of existing structures. It has become a popular means of providing overtopping protection, especially for dams with inadequate spillways. Hundreds of RCC dam structures have been built throughout the United States and the world. As previously noted, construction techniques have made RCC an economically competitive alternative to conventional concrete and embankment dams due to the following factors:

1. **Cost** - The unit cost of RCC is about 25 to 50 percent less than conventional concrete. The savings result from reduced forming, placement and compaction costs along with reduced construction times.

2. **Rapid Construction** - RCC construction encourages near continuous placement of material which results in high production rates.

3. **Integral Spillways and Appurtenances** - As with conventional dams, spillways can be directly incorporated into the structure. A typical layout allows the discharging of flow over the dam crest and downstream face.

4. **Material** - Savings in materials can often be a cost advantage of RCC, especially when compared to conventional earth/rockfill embankment construction.

The adaptation of RCC for Mill Run dam would involve overtopping protection for the PMF and provision of a service spillway for normal flow conveyance. Conventional RCC construction can be easily adopted for the overtopping component. However, conventional concrete will have to be formed and poured for some elements of the service spillway. Please refer to the following photos.

Figure No. 23: Roller Compacted Concrete Dam

Figure No. 24: Placement of RCC

The Mill Run RCC overtopping protection and spillway option will be a staged-spillway arrangement. Please note that no floodwall is proposed to raise the effective head on the spillway.

- **Overtopping Protection** - The entire downstream embankment could be protected with roller compacted concrete along with a service spillway for normal flow. For a large event, this condition would not only flood the water treatment plant but cause structural damage by the force of flood water. It is possible that a sloped, lateral collection channel at the toe of slope could convey flood waters into Mill Run downstream of the plant. However, a large portion of the toe drain, access road, valve vault, sludge drying beds, etc., would have to be relocated. This is considered unfeasible from a plant operating perspective and also impractical given downstream dam topography.

  To confine the overtopping protection to the east side of the dam, a broad crested weir will be designed with a nominal height of 6 feet. The capacity of the broad crested weir will be the difference between the peak PMF flow and service spillway capacity. The weir formula ($Q = CLH^{1.5}$) will be used to compute the required length. Downstream of the weir, a stepped RCC facing will be applied on the downstream slope. A weir, stilling basin will provide energy dissipation at the toe of slope. This will allow the discharge to be directed toward the Mill Run stream channel to the east of the plant.

- **Hydraulic Design** - The shape and size of the primary spillway will conform to the layout of an ogee-weir spillway. A large portion of this spillway will be formed and poured using conventional reinforced concrete. The need to insure watertight construction at the ogee section is evident. RCC tends to provide a seepage path if the RCC lifts are not properly bonded.

  The spillway rating curve used to size the primary spillway is based on the secondary ogee-weir spillway of the previous option. The ratio (1.05) of height of weir (9.5) to total head (9.0) is the same. Based on the 9.0 feet available head, the unit discharge is 101.30 cfs/ft. The width of the primary ogee-weir spillway is 82.4 feet which results in a total discharge capacity of 8,350 cfs for the primary spillway. Therefore, the total discharge for the broad crested RCC spillway would be 13,650 cfs. The total spillway capacity is 22,000 cfs. The primary service spillway chute and terminal structure would be identical to the staged-ogee weir primary spillway. As previously stated, the primary service spillway will control discharges up to the 100-year flood.

  The hydraulic design procedures for the RCC overtopping protection/broad crested weir are based on "Design Manual for RCC Spillways and Overtopping Protection," 2002, by URS Greiner Woodwoard Clyde for the Portland Cement Association. Based on the conventional weir formula ($Q = CLH^{1.5}$) and a broad crested weir coefficient (2.60), the required length of overtopping spillway is 360-feet for a PMF flood of 13,650 cfs. The width of the broad crested weir is 45-feet. A concrete cut-off wall is placed at the upstream end of the weir for seepage control.

  The downstream embankment will be faced with stepped RCC. Stepped chutes significantly increase the rate of energy dissipation on the downstream face. This helps reduce the size of the energy dissipation structure at the base of the dam and the potential for undermining and scour. The flow regime of the spillway determines the required energy dissipation. Either nappe flow (when water bounces from one step to the next in a free-falling manner) or skimming flow will occur. Skimming flow results when water flows as a coherent stream over the step edges.

According to tables based on stepped spillway modeling studies (3:1 embankment slope and 38.4 inch step height), skimming flow will occur if the step-height is 3.66 feet. The height of the spillway chute walls is 6-feet which accounts for the required freeboard.

The energy dissipation structure can also be designed using equations developed from hydraulic modeling studies. A Type II USBR stilling basin with a length of 60-feet and a depth of 13.5 feet will contain the hydraulic jump produced from the skimming flow regime. The top of the stilling basin is the required tailwater elevation to fully develop the hydraulic jump. Please refer to the Appendix for detailed computations.

Relative to the RCC spillway details, the stepped-spillway will have a placement width of 10 feet and step-height of 3.2 feet. The approach apron (or RCC broad crested weir) has a total thickness of 4-feet (2-2 foot lifts) with a cutoff wall depth of 8 feet. Finally, the stilling basin is constructed in four (4) 2-foot lifts for a total thickness of 8-feet. The total quantity of RCC (only) is about 23,000 cubic yards exclusive of the poured reinforced concrete service spillway. The site plan and profile of the RCC overtopping protection spillway option is shown on the attached Figure No.'s 25 and 26.

The following considerations must be taken into account in the design and construction of RCC dam projects:

- **Watertightness and Seepage Control** - Joints between RCC concrete lifts and interface with adjacent structural elements are potential seepage pathways through RCC dams. This condition is caused primarily by segregation at the lift boundaries, discontinuity between successive lifts and excessive time intervals between lift placements. Seepage can be controlled by contraction joints with waterstops, sealing the RCC interface layers and draining and collecting seepage.

  Seepage at Mill Run dam should not be a significant problem. The phreatic (seepage) line in the dam is controlled by a foundation sand blanket and sand drains that convey seepage to the downstream rock toe. The seepage line is well within the interior of the dam and should not penetrate the downstream RCC slope covering.

- **Horizontal Joint Treatment** - Bond strength and permeability are major concerns at the horizontal lift joints. These are accomplished by improving RCC mixture compactability, cleaning the joint surfaces and placing a bedding mortar between lifts.

- **Seepage Collection** - The seepage collection and drainage system can prevent excessive hydrostatic pressures against conventional concrete spillway or downstream facing. It also serves to reduce uplift pressures and increase stability. Collection methods include vertical drains with waterstop at the upstream face and horizontal drains at intervals beneath the downstream dam facing. As noted, seepage is contained within the interior of the dam.

- **Aesthetics/Freeze Thaw Considerations** - Aesthetically, RCC dam facing may be one of the least attractive alternatives. When compared to formed, finished concrete, there is no comparison. Freeze-thaw protection is also a concern in severe climates. Typically, thicker lifts require a flatter slope. The exposed edge of an uncompacted slope will have a rough, stair-stepped, natural gravel appearance. The concrete step has limited strength within 12-inches of the face.

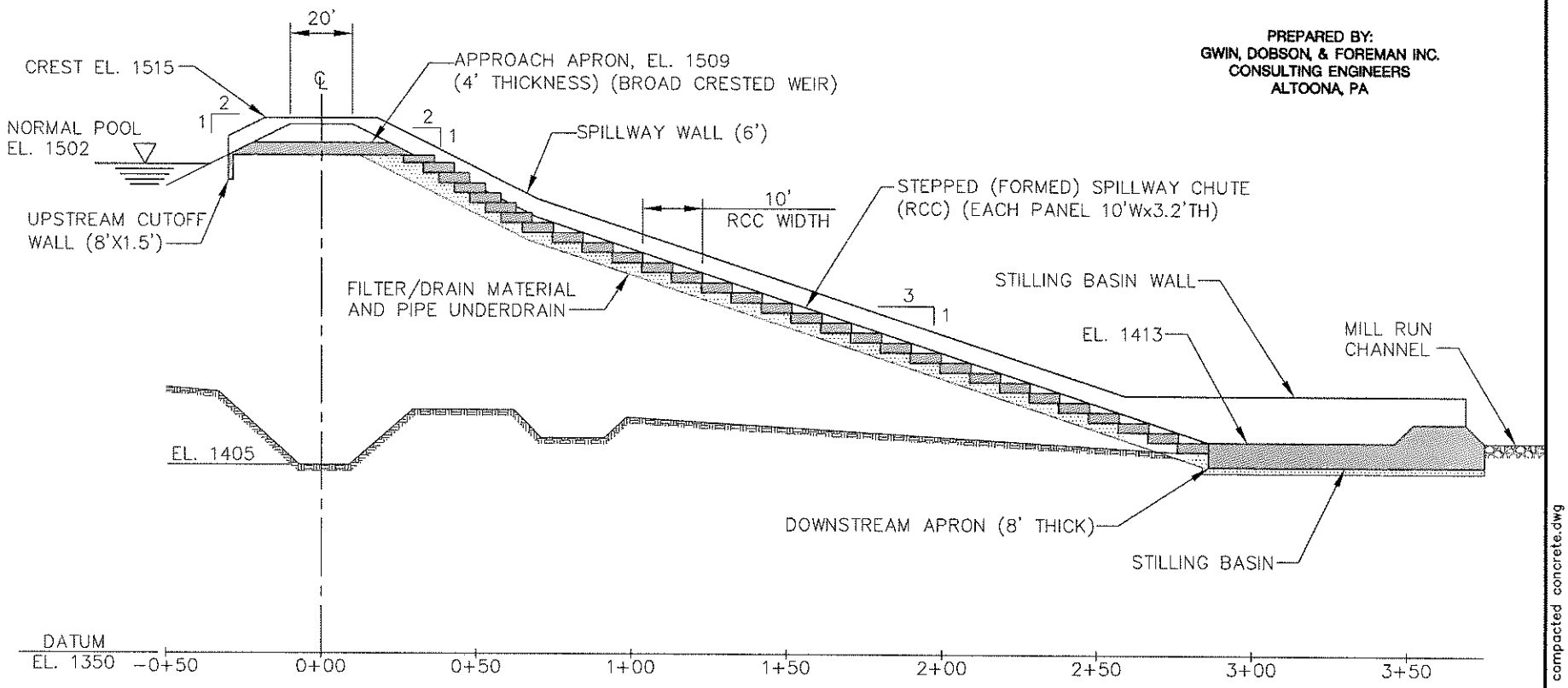
ALTOONA WATER AUTHORITY
EVALUATION OF MILL RUN DAM

FIGURE No. 25
ROLLER COMPACTED CONCRETE
OVERTOPPING PROTECTION SPILLWAY

LOGAN TWP, BLAIR CO, PENNSYLVANIA

DATE: AUGUST 2010 SCALE: AS SHOWN

PREPARED BY:
GWIN, DOBSON, & FOREMAN INC.
CONSULTING ENGINEERS
ALTOONA, PA



ROLLER COMPACTED CONCRETE
OVERTOPPING PROTECTION SPILLWAY
PROFILE

0 25 50
SCALE

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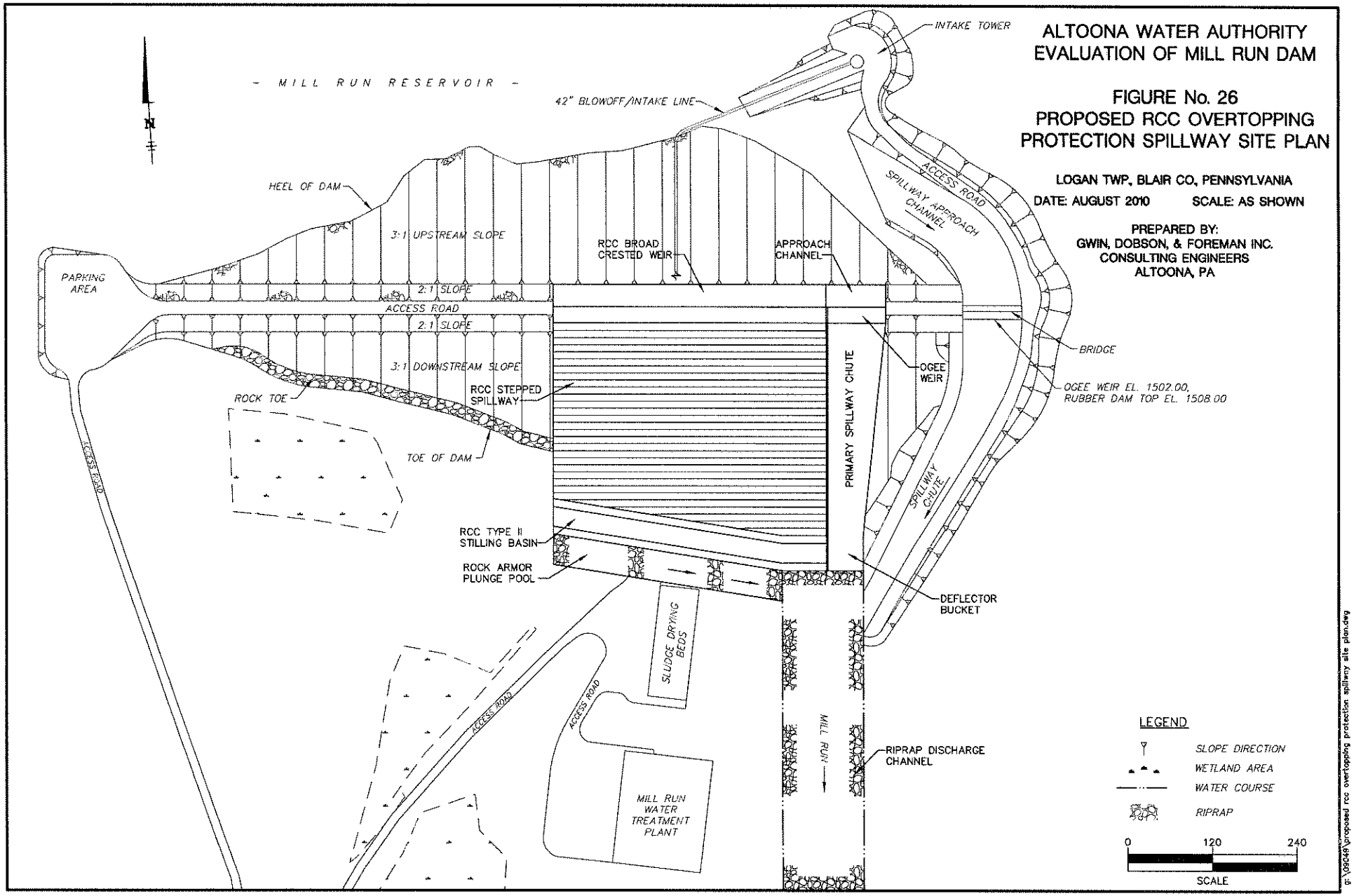

ALTOONA WATER AUTHORITY
EVALUATION OF MILL RUN DAM
FIGURE No. 26
PROPOSED RCC OVERTOPPING
PROTECTION SPILLWAY SITE PLAN
LOGAN TWP, BLAIR CO, PENNSYLVANIA
DATE: AUGUST 2010
SCALE: AS SHOWN
PREPARED BY:
GWIN, DOBSON, & FOREMAN INC.
CONSULTING ENGINEERS
ALTOONA, PA
- MILL RUN RESERVOIR -
N
INTAKE TOWER
42" BLOWOFF/INTAKE LINE
HEEL OF DAM
3:1 UPSTREAM SLOPE
PARKING AREA
2:1 SLOPE
ACCESS ROAD
2:1 SLOPE
3:1 DOWNSTREAM SLOPE
ROCK TOE
RCC STEPPED SPILLWAY
TOE OF DAM
RCC BROAD CRESTED WEIR
APPROACH CHANNEL
SPILLWAY APPROACH CHANNEL
ACCESS ROAD
PRIMARY SPILLWAY CHUTE
OGEE WEIR
BRIDGE
OGEE WEIR EL. 1502.00,
RUBBER DAM TOP EL. 1508.00
SPILLWAY CHUTE
RCC TYPE II STILLING BASIN
ROCK ARMOR PLUNGE POOL
DEFLECTOR BUCKET
SLUDGE DRYING BEDS
ACCESS ROAD
ACCESS ROAD
ACCESS ROAD
MILL RUN
RIPRAP DISCHARGE CHANNEL
MILL RUN WATER TREATMENT PLANT
LEGEND
SLOPE DIRECTION
WETLAND AREA
WATER COURSE
RIPRAP
0
120
240
SCALE

These conditions can be mitigated by horizontal slip-forming; compaction-finishing on the sloped facing (by vibrating plates); placement against vertical forms; or placement by a concrete paving machine. Of course, these techniques add cost to the project.

- **Transverse Contraction Joints** - The potential for cracking are minimized in RCC dams because of reduction in mixing water, reduced temperature rise in rapid placement and lower lift heights. Also, increased point-to-point aggregate contact decreases volume shrinkage. Nevertheless, contraction joints are typically designed for RCC dams to prevent any shrinkage or thermal cracking that may serve as a leakage path. Contraction joint placement by use of a vibrating blade (for insertion of galvanized steel sheeting) is a typical method.

All of the above factors must be considered during the design process. The requirement for detailed construction plans and details is imperative. Construction specifications using precise mix designs and construction quality assurance/quality control are mandatory.

Standard RCC practices and procedures have evolved over the years. The US Army Corps of Engineers and Portland Cement Association have been at the forefront of standardizing specifications and construction details. They have also served as a clearinghouse for new technical practices and research. Their documents have been widely disseminated throughout the dam engineering profession.

There is no reason that a RCC overtopping spillway protection system cannot be effectively implemented at Mill Run dam.

Upgrade of Existing Spillway Option - The hydraulic evaluation has revealed that the existing spillway has inadequate capacity for the PMF. Two options exist for upgrading it. The first is to install a staged spillway at the current location. A second option is to increase the head on the existing spillway. These options will be assessed in the following narrative.

- **Install Staged-Spillway/Use Existing Spillway Channel** - Since the length of staged-spillway will be excessive in relation to spillway channel geometry, this option is considered unfeasible and will not be evaluated.

- **Increase Spillway Head/Use Existing Spillway Channel** - By increasing the head on the existing spillway, additional flow capacity can be generated. Raising the dam or installing a floodwall on the dam crest will serve this function. Because of topographic limitations, raising the existing dam is not possible. The installation of a floodwall could increase the available spillway head. This would require the removal of the bridge pier and rubber dam haunch.

  Because of the higher head-to-weir height ratio (H/P), the spillway coefficient (C) is less efficient in the weir equation. This results in a slightly lower spillway rating (see spillway rating curve in Appendix). Computations show that a four (4) foot addition will increase the available spillway head to seventeen (17) feet. Obviously, no staging is necessary since the width of the existing spillway and crest elevation (1,502 msl) remains unchanged.

  The downstream spillway chute was evaluated for a discharge of 20,000 cfs. These computations are shown in the Appendix and reveal that the spillway walls will need to be raised 50% (2-6 feet) to contain flows at or near the Probable Maximum Flood. An evaluation of the terminal structure shows that the existing deflector bucket is inadequate. The existing bucket radius of 35-feet is well under the required radius of 135-feet. This results in excessive soil pressure beneath the existing bucket (about 3,000 psf > 500 psf allowable). Therefore, the deflector bucket will need to be entirely replaced.

  Finally, a cursory hydraulic evaluation of the curvature effect of the existing spillway was performed. Cross waves are usually found in channels of non-linear alignment where supercritical flow exists. These waves form a disturbance pattern that can persist a considerable distance downstream. They are capable of producing oblique hydraulic jumps and wave fronts that could overtop the channel near the embankment/abutment contact.

  These cross waves can be cancelled in a curved channel if a specific radius of curvature is maintained. The analysis reveals that the central angle of the spillway curvature needs to be doubled (from 29° to 58° at the same radius) to prevent this condition. From this analysis, the question of cross wave formation may not be of a sufficient magnitude to cause concern. However, the default assumption for all other spillway options is to safely pass all floods up to the PMF. Consideration must therefore be given for redesigning the downstream chute to conform with the required geometry.

The following considerations are drawn from the hydraulic evaluation and a condition assessment of the existing spillway.

- The hydraulic upgrade of the existing spillway must also take into account its physical condition. Testing has revealed structural deficiencies of the spillway slabs and seepage through the ogee-weir. At a minimum, the ogee-weir needs to be rehabilitated with epoxy-injected grout and possible replacement of certain sections to abate the leakage condition. The approach channel wall would need to be replaced and raised to match the flood wall height.

- The required curvilinear alignment and need for a new terminal structure (deflector bucket) suggest that the need for a new spillway channel, 80-feet downstream of the ogee weir. Since the condition of the spillway is poor to begin with, a new spillway channel is entirely justified.

- The floodwall would take the form of an inverted-zee wall and installed along the entire crest of the dam. The exposed portion (6.5 feet) would be placed on the upstream side. The slab portion would be extended under the road and terminate at a six-foot deep stem footer. This design was previously used at Kettle dam to increase available head on the spillway.

- A different means of access to the intake tower is required if this alternative is constructed. Any bridge spanning the new spillway would have to be raised 6.5 feet so as not to interfere with the waterway opening. The required approach work is not practical on either side of the spillway bridge. Consideration must be given to either a new intake tower/bridge to the west of the spillway or fording the downstream channel and climbing the bench area next to the spillway chute.

- This option necessitates optimum rock slope stability along the spillway channel. Because of the proximity to the rock slope, a massive "block" failure (which could fill/damage the spillway) cannot be dismissed. More sophisticated slope stability techniques may be necessary than a simple slope "lay back." Please refer to Figure 27 for the 1958 photo of the spillway.

Our analysis of the existing spillway shows that the current spillway alignment is not the most hydraulically efficient or economic solution. Although this structure is largely founded in rock, the curvilinear alignment has the potential to form standing waves. The required radius to provide cross-wave cancellation in a supercritical flow regimen may be impractical from a cost standpoint.

Modification to existing structures often involve unforeseen additional costs. Differing site conditions may be uncovered during construction. These may be the source of added expense during construction. Provided a thorough design is done, many of these cost escalation factors can be mitigated or controlled. However, the retrofitting of the existing spillway, although practical and feasible, may not be cost-effective.

Figure No. 27: Spillway Chute (1958)

New Unstaged Ogee Weir Spillway Option - The analysis considered the design of a new ogee-weir spillway with no staging. In other words, this option would provide a "level" spillway that does not control the outflow up to the 100-year flood elevation.

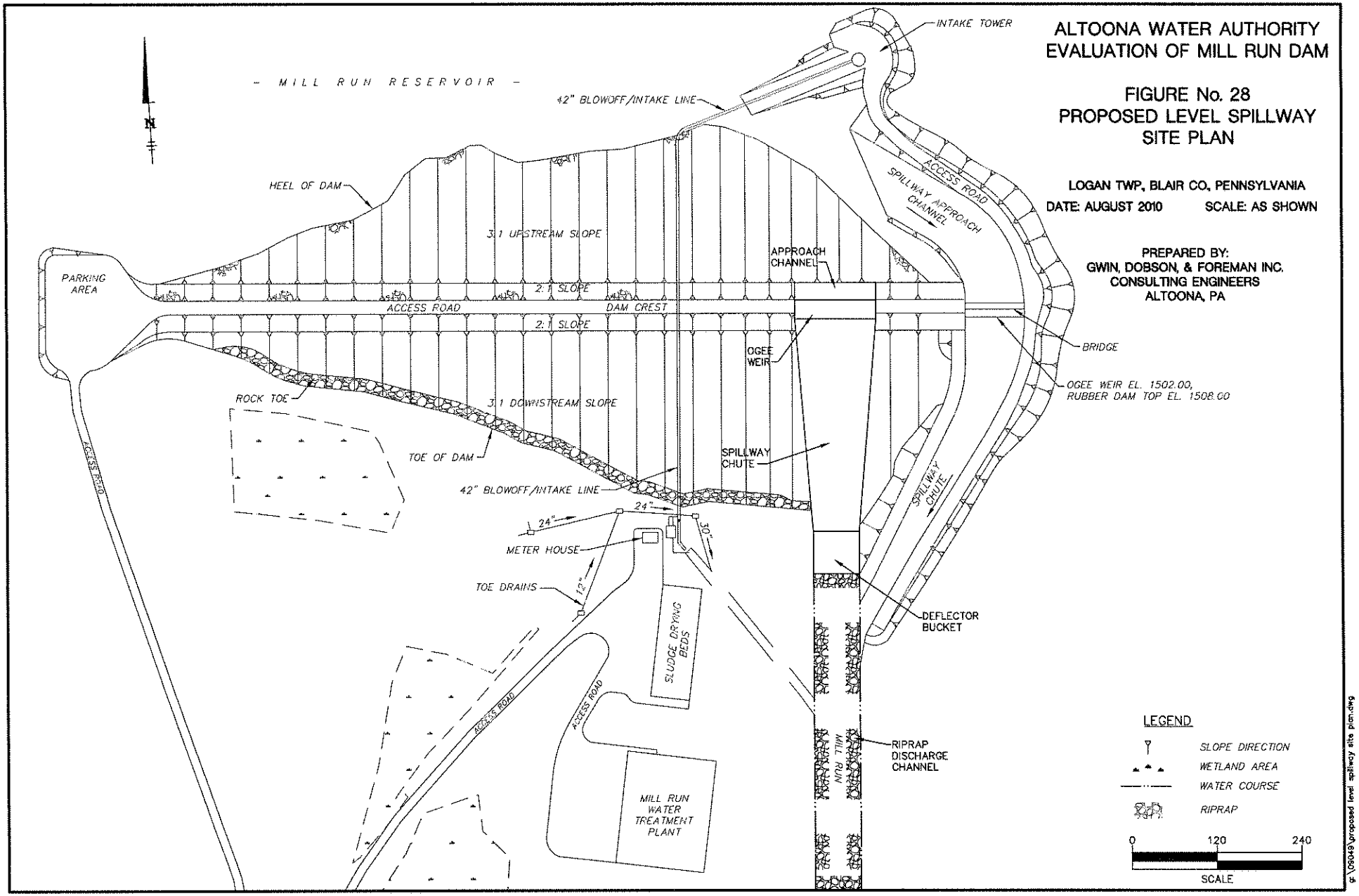
We evaluated the "level" spillway option with the weir crest at its original elevation of 1,502 msl. This provided a spillway head of 13 feet to the top of dam. No intervening piers or obstructions are present. The discharge is about 20,000 cfs. The detailed computations are found in the Appendix. Figures 28 and 29 show the site plan and spillway details for this alternative.

The analysis reveals that the required spillway width is 115 feet. The spillway chute is about 325 feet long, 115 feet wide at the top and 70 feet wide at the bottom. The slope varies between 26-33%. An 85-foot deflector bucket radius serves as the terminal structure.

Modeling of the stage/discharge/storage characteristics of this alternative was run using HEC-HMS. The discharge from the wider spillway produces a flow of about 1,950 cfs for the same 100-year rainfall through for the existing spillway. This produces a 33% increase in the 100-year discharge (1,400 cfs) that would otherwise be controlled by the existing spillway.

The PADEP has mandated that reservoir inflow-outflow be controlled at pre-existing conditions, in other words, the elevation/storage/discharge characteristics of the existing spillway. Mill Run would be flooded on a more frequent basis than the current 100-year flood.

It is acknowledged that this alternative will not be approved by PADEP. However, the Authority should be aware of the cost implications of the DEP staging requirement. A separate cost estimate will be prepared for this option and compared with the other alternatives for information purposes only.


ALTOONA WATER AUTHORITY
EVALUATION OF MILL RUN DAM
FIGURE No. 28
PROPOSED LEVEL SPILLWAY
SITE PLAN
LOGAN TWP., BLAIR CO., PENNSYLVANIA
DATE: AUGUST 2010
SCALE: AS SHOWN
PREPARED BY:
GWIN, DOBSON, & FOREMAN INC.
CONSULTING ENGINEERS
ALTOONA, PA
- MILL RUN RESERVOIR -
N
INTAKE TOWER
42" BLOWOFF/INTAKE LINE
HEEL OF DAM
3:1 UPSTREAM SLOPE
PARKING AREA
2:1 SLOPE
ACCESS ROAD
DAM CREST
2:1 SLOPE
APPROACH CHANNEL
SPILLWAY APPROACH CHANNEL
ACCESS ROAD
BRIDGE
OGEE WEIR
OGEE WEIR EL. 1502.00,
RUBBER DAM TOP EL. 1508.00
ROCK TOE
3:1 DOWNSTREAM SLOPE
TOE OF DAM
SPILLWAY CHUTE
42" BLOWOFF/INTAKE LINE
24"
24"
30"
METER HOUSE
TOE DRAINS
72"
SLUDGE DRYING BEDS
ACCESS ROAD
ACCESS ROAD
ACCESS ROAD
DEFLECTOR BUCKET
RIPRAP DISCHARGE CHANNEL
MILL RUN
MILL RUN WATER TREATMENT PLANT
LEGEND
SLOPE DIRECTION
WETLAND AREA
WATER COURSE
RIPRAP
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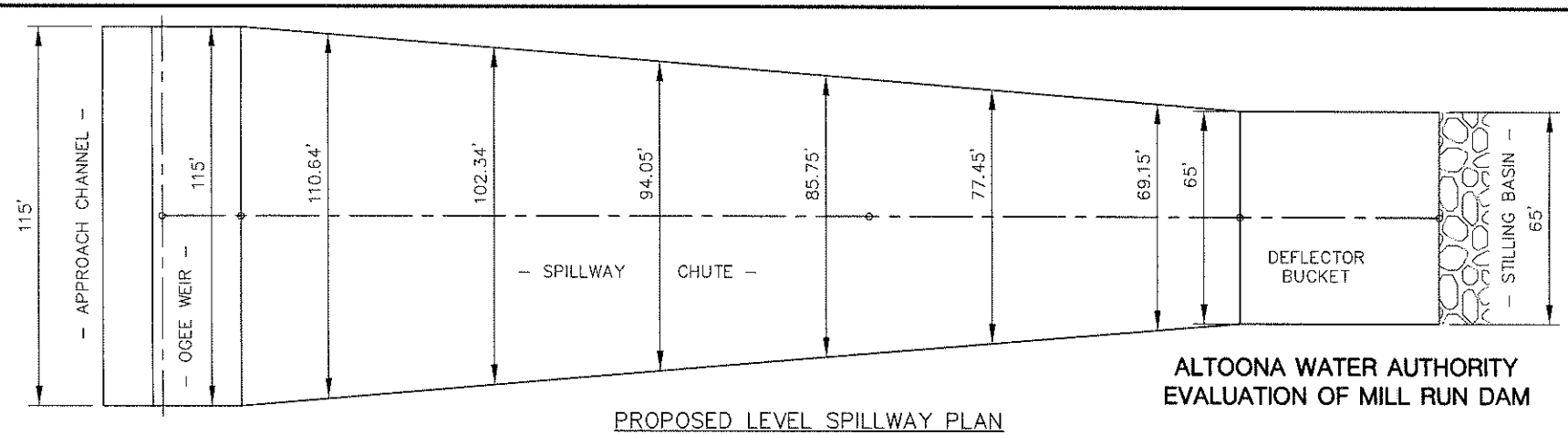


PROPOSED LEVEL SPILLWAY PLAN

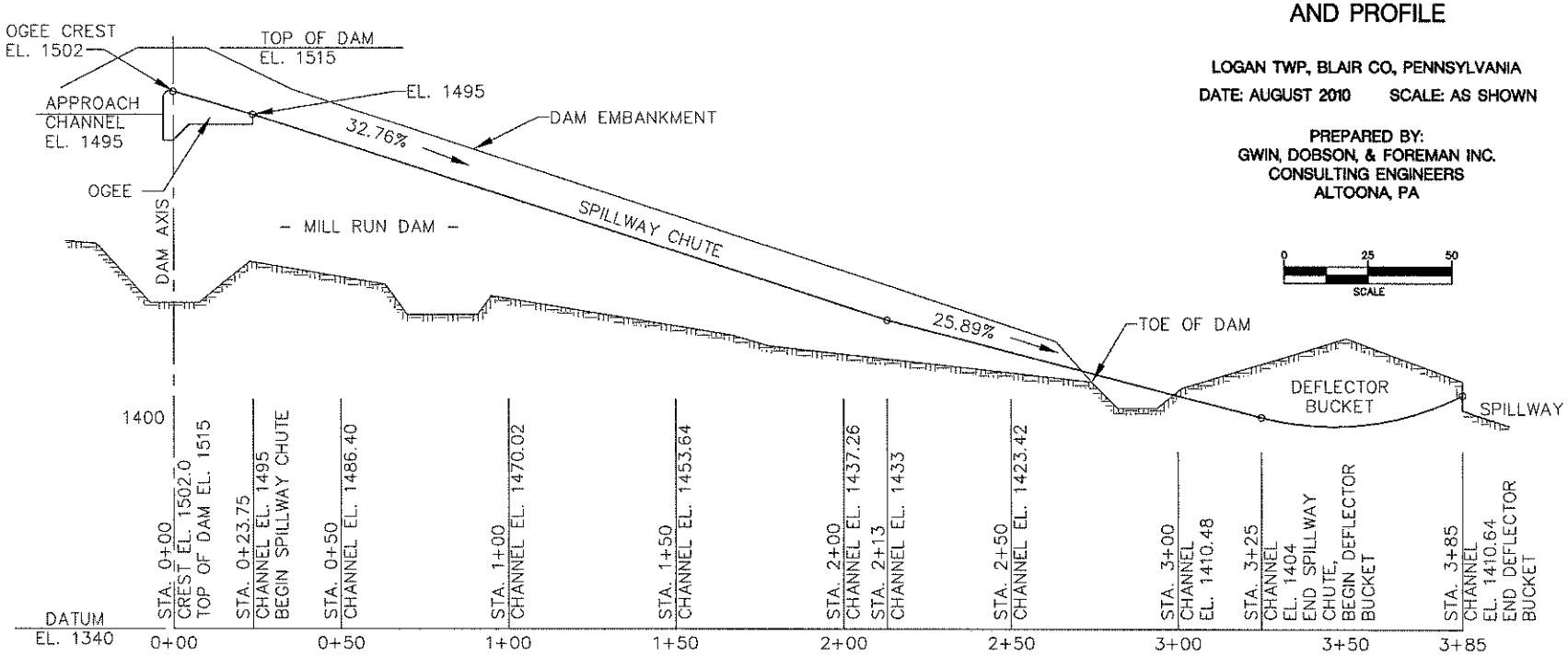
ALTOONA WATER AUTHORITY
EVALUATION OF MILL RUN DAM

FIGURE No. 29
PROPOSED LEVEL SPILLWAY PLAN AND PROFILE

LOGAN TWP, BLAIR CO, PENNSYLVANIA

DATE: AUGUST 2010 SCALE: AS SHOWN

PREPARED BY:
GWIN, DOBSON, & FOREMAN INC.
CONSULTING ENGINEERS
ALTOONA, PA



PROPOSED LEVEL SPILLWAY PROFILE

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Spillway Cost Evaluation - The following cost analysis is provided for each spillway option. Estimated costs are confined to the spillway portion only. Additional cost factors (rock slope stabilization, access roads/bridges, etc.) will be considered in the alternatives evaluation section.

Table 10: Engineer's Opinion of Probable Construction Cost-Staged Ogee Spillway Option

Item	Description	Quantity	Unit	Unit Price	Total Price
1.	Unclassified Excavation - Spillway Floodwall and Riprap Channel	24,600	CY	$ 15	$ 369,000
2.	Earth Embankment - Backfill	3,650	CY	$ 30	$ 109,500
3.	AASHTO No. 57 Coarse Aggregate - Underdrains	700	CY	$ 40	$ 28,000
4.	6" PVC Pipe and Fittings - Underdrains and Weep Holes	2,075	LF	$ 57	$ 118,275
5.	10" PVC Pipe and Fittings - Underdrain	240	LF	$ 100	$ 24,000
6.	20" PVC Pipe and Fittings - Deflector Bucket Drain	400	LF	$ 125	$ 50,000
7.	Reinforced Concrete Floodwall	1,250	CY	$ 575	$ 718,750
8.	Reinforced Concrete Approach Channel	285	CY	$ 400	$ 114,000
9.	Reinforced Concrete Ogee Weirs	1,850	CY	$ 550	$1,017,500
10.	Reinforced Concrete Spillway Chute and Deflector Bucket	3,115	CY	$ 585	$1,822,275
11.	Expansion and Construction Joints	3,000	LF	$ 11	$ 33,000
12.	Concrete Protective Coating	1,460	SY	$ 35	$ 51,100
13.	Rock Armor, 36" Thickness	850	SY	$ 200	$ 170,000
14.	Riprap, 18" Thickness - Riprap Channel	165	SY	$ 42	$ 6,930
15.	Riprap, 12" Thickness - Floodwall	1,200	SY	$ 70	$ 84,000
16.	Grading, Seeding and Mulching	1,500	SY	$ 4	$ 6,000
17.	Spillway Access Walkway, Stairway and Handrailing	JOB	JOB	L.S.	$ 20,000
	Total, Staged Ogee Spillway Option				$4,742,330
				USE	$5,000,000

Table 11: Engineer's Opinion of Probable Construction Cost - Labyrinth Spillway Option

Item	Description	Quantity	Unit	Unit Price	Total Price
1.	Unclassified Excavation - Spillway Floodwall and Riprap Channel	16,800	CY	$ 15	$ 252,000
2.	Earth Embankment - Backfill	2,500	CY	$ 30	$ 75,000
3.	AASHTO No. 57 Coarse Aggregate - Underdrains	500	CY	$ 40	$ 20,000
4.	6" PVC Pipe and Fittings - Underdrains and Weep Holes	1,420	LF	$ 57	$ 80,940
5.	10" PVC Pipe and Fittings - Underdrain	170	LF	$ 100	$ 17,000
6.	20" PVC Pipe and Fittings - Deflector Bucket Drain	280	LF	$ 125	$ 35,000
7.	Labyrinth Spillway Weir Walls	1,168	CY	$ 585	$ 683,280
8.	Labyrinth Spillway Apron	223	CY	$ 400	$ 89,200
9.	Reinforced Concrete Spillway Chute and Deflector Bucket	3,050	CY	$ 585	$ 1,784,250
10.	Expansion and Construction Joints	2,025	LF	$ 11	$ 22,275
11.	Concrete Protective Coating	1,000	SY	$ 35	$ 35,000
12.	Rock Armor, 36" Thickness, Existing Intake Tower Masonry	580	SY	$ 200	$ 116,400
13.	Riprap, 18" Thickness - Riprap Channel	1,120	SY	$ 42	$ 47,040
14.	Grading, Seeding and Mulching	1,025	SY	$ 4	$ 4,100
	Total, Staged - Ogee Spillway				$ 3,261,485
				USE	$ 3,300,000

Table 12: Engineer's Opinion of Probable Construction Cost
Roller Compacted Concrete (RCC) Overtopping
Protection Spillway and Primary Spillway

Item	Description		Quantity	Unit	Unit Price	Total Price
A.	**PRIMARY REINFORCED CONCRETE SPILLWAY**					
	1.	Unclassified Excavation - Spillway Floodwall and Riprap Channel	12,800	CY	$ 15	$ 192,000
	2.	Earth Embankment - Backfill	1,900	CY	$ 30	$ 57,000
	3.	PennDOT No. 2B Coarse Aggregate - Underdrains	370	CY	$ 40	$ 14,800
	4.	6" PVC Pipe and Fittings - Underdrains and Weep Holes	1,080	LF	$ 57	$ 61,560
	5.	10" PVC Pipe and Fittings - Underdrain	125	LF	$ 100	$ 12,500
	6.	20" PVC Pipe and Fittings - Drain	200	LF	$ 125	$ 25,000
	7.	Reinforced Concrete - Approach Channel	160	CY	$ 400	$ 64,000
	8.	Reinforced Concrete - Ogee Weir	935	CY	$ 575	$ 537,625
	9.	Reinforced Concrete Primary Spillway Chute and Deflector Bucket	2,355	CY	$ 585	$ 1,377,675
	10.	Expansion and Construction Joints	1,540	LF	$ 35	$ 53,900
	11.	Concrete Protective Coating	760	SY	$ 35	$ 26,600
	12.	Rock Armor, 36" Thickness	440	SY	$ 200	$ 88,000
	13.	Riprap, 18" Thickness - Riprap Channel	950	SY	$ 42	$ 39,900
	14.	Grading, Seeding and Mulching	800	SY	$ 4	$ 3,200
		Sub-Total, Primary Reinforced Concrete Spillway				$ 2,553,760
B.	**RCC OVERTOPPING PROTECTION SPILLWAY**					
	1.	Structural Concrete (Spillway Walls)	900	CY	$ 585	$ 526,500
	2.	Bedding Mortar	1,250	CY	$ 170	$ 212,500
	3.	Foundation Preparation	15,000	SY	$ 11.50	$ 172,500
	4.	Roller Compacted Concrete	22,200	CY	$ 100	$ 2,220,000

Item	Description	Quantity	Unit	Unit Price	Total Price
B.	**RCC OVERTOPPING PROTECTION SPILLWAY (Continued)...**				
5.	Cement for RCC	1,350	TONS	$ 108	$ 145,800
6.	Pozzolan for RCC	2,220	TONS	$ 28	$ 62,160
7.	Pre-Cast Facing Elements	JOB	JOB	L.S.	$ 275,000
8.	Aggregate/Underdrains	1,700	CY	$ 75	$ 127,500
9.	Geotextile	14,800	SY	$ 4	$ 59,200
10.	Rock Armor/Riprap	5,000	CY	$ 42	$ 210,000
11.	Topsoil, Seeding & Mulching	2,000	SY	$ 4	$ 8,000
	Sub-Total, RCC Overtopping Protection Spillway				$ 4,019,160
	Sub-Total, Primary Reinforced Concrete Spillway				$ 2,553,760
	TOTAL, RCC OVERTOPPING PROTECTION SPILLWAY AND PRIMARY SPILLWAY				$ 6,572,920
				USE	$ 6,600,000

Table 13: Engineer's Opinion of Probable Construction Cost - Existing Spillway Modifications

Item	Description	Quantity	Unit	Unit Price	Total Price
1.	Unclassified Excavation - Spillway	6,400	CY	$ 15	$ 96,000
2.	Earth Embankment - Backfill	1,000	CY	$ 30	$ 30,000
3.	PennDOT No. 2B Coarse Aggregate - Underdrains	250	CY	$ 50	$ 12,500
4.	Demolition - Existing Spillway - Chute, Walls, Slabs and Deflector Bucket	2,500	SY	$ 50	$ 125,000
5.	6" PVC Pipe and Fittings - Underdrains and Weep Holes	750	LF	$ 57	$ 42,750
6.	10" PVC Pipe and Fittings - Underdrain	200	LF	$ 100	$ 20,000
7.	20" PVC Pipe and Fittings - Deflector Bucket Drain	250	LF	$ 130	$ 32,500
8.	Reinforced Concrete - Approach Channel Walls	500	CY	$ 600	$ 300,000
9.	Reinforced Concrete Spillway Chute Slab and Walls	2,250	CY	$ 585	$ 1,316,250
10.	Reinforced Concrete Floodwall	1,250	CY	$ 575	$ 718,750
11.	Reinforced Concrete Reflector Bucket	370	CY	$ 585	$ 216,450
12.	Expansion and Construction Joint	1,770	LF	$ 11	$ 19,470
13.	Existing Ogee Weir Concrete Repairs and Epoxy Injection	JOB	JOB	L.S.	$ 200,000
14.	Rock Armor, 36" Thickness, Existing Intake Tower Masonry	500	SY	$ 200	$ 100,000
15.	Riprap, 18" Thickness - Riprap Channel	1,000	SY	$ 42	$ 42,000
16.	Riprap, 12" Thickness - Floodwall	720	SY	$ 70	$ 50,400
17.	Grading, Seeding and Mulching	900	SY	$ 4	$ 3,600
	Total, Existing Spillway Modifications				$ 3,325,670
				USE	$ 3,400,000

Table 14: Engineer's Opinion of Probable Construction Cost
Unstaged Ogee Weir Spillway Option

Item	Description	Quantity	Unit	Unit Price	Total Price
1.	Unclassified Excavation - Spillway Floodwall and Riprap Channel	18,900	CY	$ 15	$ 283,500
2.	Earth Embankment - Backfill	2,800	CY	$ 30	$ 84,000
3.	PennDOT No. 2B Coarse Aggregate - Underdrains	540	CY	$ 40	$ 21,600
4.	6" PVC Pipe and Fittings - Underdrains and Weep Holes	1,600	LF	$ 57	$ 91,200
5.	10" PVC Pipe and Fittings - Underdrain	185	LF	$ 100	$ 18,500
6.	20" PVC Pipe and Fittings - Deflector Bucket Drain	200	LF	$ 125	$ 25,000
7.	Reinforced Concrete Approach Channel	220	CY	$ 400	$ 88,000
8.	Reinforced Concrete Ogee Weir	1,420	CY	$ 575	$ 816,500
9.	Reinforced Concrete	3,360	CY	$ 585	$ 1,965,600
10.	Expansion and Construction Joint	2,300	LF	$ 11	$ 25,300
11.	Concrete Protective Coating	1,120	SY	$ 35	$ 39,200
12.	Rock Armor, 36" Thickness	650	SY	$ 42	$ 27,300
13.	Riprap, 18" Thickness - Riprap Channel	920	SY	$ 70	$ 64,400
14.	Grading, Seeding and Mulching	1,150	SY	$ 4	$ 4,600
	Total, Unstaged - Ogee Weir Spillway				$ 3,554,700
				USE	$ 3,600,000

INTAKE TOWER

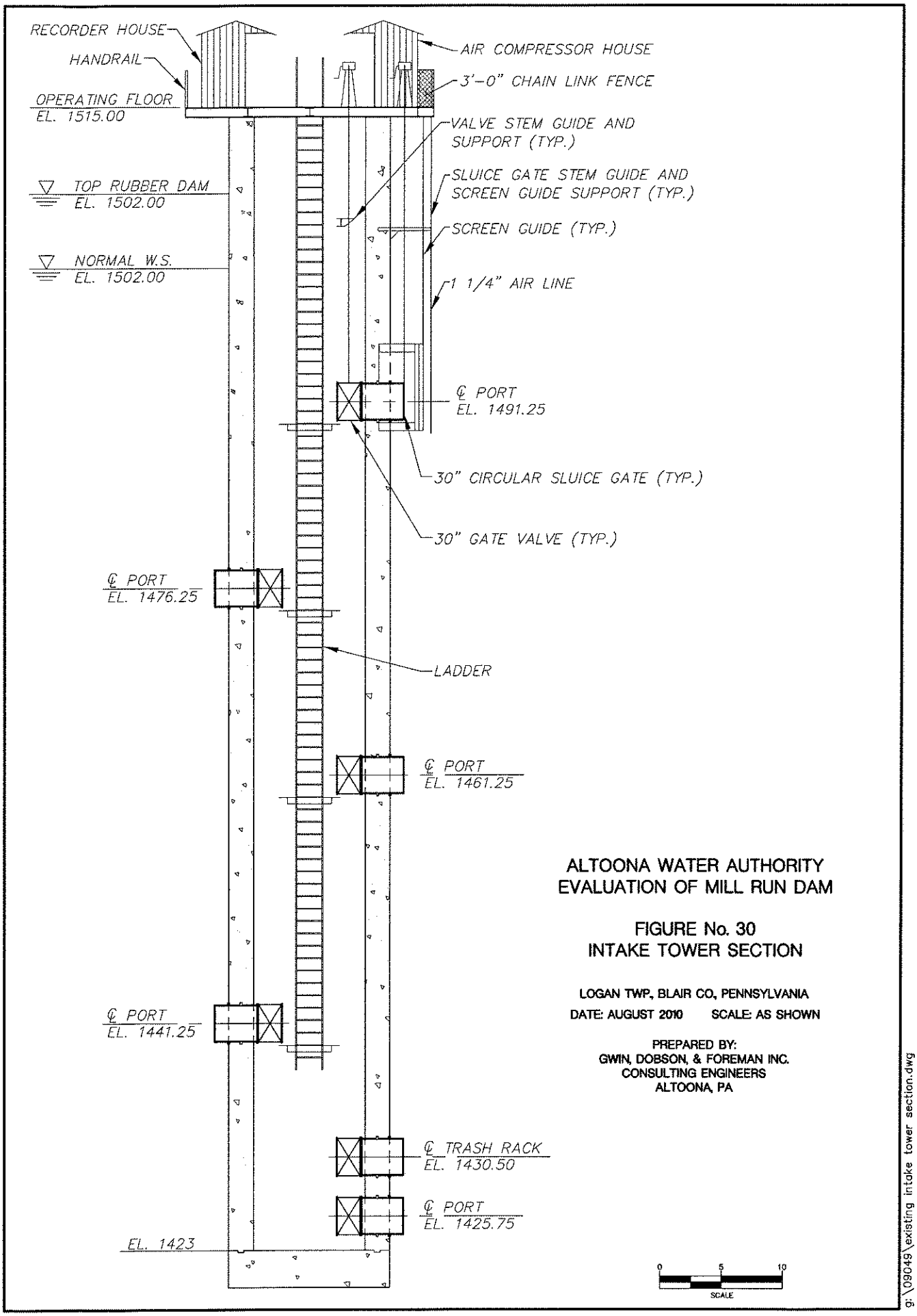
General - The assessment of the intake tower includes a physical evaluation of the structure and appurtenances, accessibility, stability of the adjacent rock slope and hydraulics of tower operations. Please refer to Figure 30 which is a cross-section of the existing intake tower.

Intake Tower Hydraulics - Computations were performed to determine the minimum time to drain the dam. The hydraulic analysis of the 42" conduit operating under a reservoir head is similar to a highway culvert problem. This involves two (2) conditions of flow. First, when the upper pool is at low stage, open channel flow may occur within the pipe. As the pool level is raised, pressure conditions prevail when the pipe flows full. Analysis techniques used US Army Corps of Engineer and United States Bureau of Reclamation (Design of Small Dams criteria) criteria. Please refer to the Appendix for detailed computations.

For the partial flow condition, it appears that maximum pipe capacity is 94.2 cfs with a velocity of 9.8 fps. Under this condition, "critical" depth and "normal" depth are essentially the same at the end of the 42" line. This location establishes the "control section" of the pipe under the open channel flow condition. All flow under 95 cfs would be controlled by the sluice gate at the intake tower. Known as "inlet control," a reservoir pool level of El. 1,430 would control all flow up to 95 cfs. This is only 7 feet above the upstream pipe invert.

For all pool levels, above El. 1,430, pressure pipe conditions exist. This flow regime is governed by all head loss potential at the intake tower and in the 42" pipe. For the "maximum loss" condition at normal pool elevation (El. 1,502 msl), the discharge through the 42" line is 187 cfs. This produces a velocity of 19.4 cfs. The major head loss contributors are (in order of magnitude): pipe entrance loss (at tower); friction loss through pipe; exit loss at discharge and the intake tower 3' x 4' sluice gate.

A rating curve for the 42" line under pressure and open channel flow conditions is shown in the Appendix. Using this curve, 4.26 days are required to drain the dam at normal pool (El. 1,502) and 5 days at the crest of the rubber dam (El. 1,508). The average drawdown rate is 17.8 ft./day and 16.4 ft./day, respectively.



ALTOONA WATER AUTHORITY
EVALUATION OF MILL RUN DAM

FIGURE No. 30
INTAKE TOWER SECTION

LOGAN TWP, BLAIR CO, PENNSYLVANIA

DATE: AUGUST 2010 SCALE: AS SHOWN

PREPARED BY:
GWIN, DOBSON, & FOREMAN INC.
CONSULTING ENGINEERS
ALTOONA, PA

g:\09049\existing intake tower section.dwg

This information is useful in determining how quickly the dam could be drained in an emergency. However, draining a dam at this rate has potentially negative consequences. Known as the "rapid draw down" condition, slope stability problems can often occur. It is generally considered the most critical scenario in a slope stability analysis.

Some regulatory agencies limit drawdown for these reasons. For instance, New Jersey limits drawdown to one foot per day for dams which impound water on a permanent basis. This would translate to a 75-90 day drawdown period for Mill Run reservoir.

Normal drawdown for Mill Run reservoir should be limited to 75 days for non-critical inspection of the dam, basin silt removal, etc. In an emergency, an analysis of slope stability (along with visual inspection) would be required, prior to operation of the maximum drawdown rate. An attempt to do a preliminary desktop assessment is included in the slope stability portion of this report.

Intake Tower Access - The existing intake tower is not presently accessible by vehicle. Debris accumulation and danger of rock falls are hazards for operating personnel. The required work to stabilize the rock slope will be addressed later in this report.

The type of slope stability technique is an economic question. The highest rock slope exists at the dam axis and along a portion of the intake access road. In all likelihood, the new spillway will be located farther to the west than the existing spillway. This would result in backfilling the existing spillway. The degree of stability would not be as critical because falling debris, if any, would have a greater fall zone without endangering the new spillway.

The same logic would apply to the intake tower. An alternative would be to construct a new intake tower into the reservoir. A logical location is at 42-inch intake line where it becomes perpendicular with the dam axis. This would require a two-span access bridge 175-feet long (with two (2) 87.5 ft. spans). The tower would be identical, in many respects, to the existing tower in terms of height, intake ports, etc. Undoubtedly, this alternative would be more costly when compared to refurbishing the existing tower. However, this cost must be weighed with the degree and scope of required slope stability. For purposes of this analysis, we will evaluate intake tower alternatives only, leaving rock slope stability options for a separate evaluation.

Another consideration is that a bridge over the new spillway is needed if the existing intake tower remains. The length of this bridge would range from 80 feet for the existing spillway modifications; 150 feet for the staged spillway; 162-feet for labyrinth spillway; and 442-feet for the RCC overtopping protection spillway option. Obviously, a bridge over the RCC spillway is impractical. The bridges over other spillways would be not inexpensive, especially those with an associated floodwall. These factors are a cost consideration for retaining the existing intake tower.

If the tower was relocated, access to the east abutment would still be required. However, the existing bench between the rock slope and dam could serve as an access road. Fording of Mill Run below the spillway would be necessary. Though not as convenient, this roadway is practical since access to this part of the dam is infrequent.

Intake Tower Rehabilitation - This alternative involves retaining the intake at its current location and replacing certain appurtenances. We base this assessment largely on annual dam inspection reports. Obviously, additional work may be needed during final design to confirm the need for rehabilitation.

The estimated scope and cost of these improvements are tabulated as follows:

- Refurbishment of six (6) gate valves, sluice gates and operators
- Replacement of sluice gate guides, wall guides and brackets, valve stems and operators
- Replacement of trash rack
- Replacement of access ladder and internal platforms
- Replacement of operating platform and hoists
- Concrete patching, crack repair and sealing of intake and 42" intake line
- Replacement of air piping system (ice prevention), compressors and building
- Site improvements including new chain link fence, parking area, access road upgrade and guiderails

Table 15: Engineer's Opinion of
Probable Construction Cost-Existing Intake Tower Rehabilitation

Item	Description	Quantity	Unit	Unit Price	Total Price
1.	Renovation of 30" Gate Valves	6	EA.	$ 20,000	$ 120,000
2.	Renovation of 36" Sluice Gates	6	EA.	$ 15,000	$ 90,000
3.	Replacement of Gate Operators	12	EA.	$ 1,500	$ 18,000
4.	Valve Guides, Brackets, Standoffs and Stems Replacement	JOB	JOB	L.S.	$ 22,000
5.	Trash Rack Replacement	1	EA.	$ 15,000	$ 15,000
6.	Access Ladders & Platform Replacement	JOB	JOB	L.S.	$ 37,500
7.	Top Platform/Hoist Replacement	JOB	JOB	L.S.	$ 18,000
8.	Air Piping & Compressor Building Replacement	JOB	JOB	L.S.	$ 35,000
9.	Concrete Patching, Repairs & Sealing - Tower and 42" Intake/Line	1,670	SY	$ 75	$ 125,250
10.	Chain Link Fence Replacement	50	LF	$ 60	$ 3,000
11.	Access Road	850	SY	$ 30	$ 25,500
12.	New Guiderail	500	LF	$ 50	$ 25,000
	Total, Intake Tower Renovation				$ 534,250
				USE	$ 535,000

New Intake Tower - Constructing a new intake tower involves many of the same components as the existing structure. These include reinforced concrete structure, foundation, mechanical piping, intake ports, operating platforms, access ladder, ice prevention system, access bridge and appurtenances. We anticipate that an 8-foot circular intake structure would be built over the existing 42-inch discharge line and into the upstream embankment. For preliminary evaluation purposes, we have estimated that the tower would be 175 feet from the dam axis. This would involve a two-span (87.5 feet each span), structural steel bridge (10-feet wide) with open steel grid deck and intervening bridge pier.

The estimated cost of the intake tower is based on comparable structures built at the Kettle, Plane 9 and Cochran Impounding dams. A detailed tabulation of system components is set forth below. It should be mentioned that Authority personnel prefer a new intake tower.

Table 16: Engineer's Opinion of Probable Construction Cost-New Intake Tower

Item	Description	Quantity	Unit	Unit Price	Total Price
1.	Unclassified Excavation - Intake Tower, Bridge Pier and Abutment	500	CY	$ 20	$ 10,000
2.	Earth Embankment - Backfill	300	CY	$ 50	$ 15,000
3.	Steel Sheet Piling - Intake Tower Bridge Pier and Tower Structure	3,150	LF	$ 35	$ 110,250
4.	Core Drilling, NX Diameter and Rotary Drilling for Rock Anchors	650	LF	$ 35	$ 22,750
5.	Rock Anchors with Grouting Complete-in-Place	600	LF	$ 90	$ 54,000
6.	Reinforced Concrete				
	a. Intake Tower and Foundation	270	CY	$ 1,350	$ 364,500
	b. Intake Tower Bridge Pier	110	CY	$ 1,100	$ 121,000
	c. Intake Tower Bridge Abutment and Approach Slab	35	CY	$ 1,000	$ 35,000
7.	Valves and Sluice Gates with Wall Thimbles				
	a. 30" Diameter Valves	6	EA	$ 30,000	$ 180,000
	b. 36"x42" Rectangular Sluice Gates	6	EA	$ 40,000	$ 240,000
8.	Floor Stands, Operators, Stems and Guides	JOB	JOB	L.S.	$ 20,000
9.	Miscellaneous Metal	JOB	JOB	L.S.	$ 250,000
10.	Air Line Pipe and Fittings	JOB	JOB	L.S.	$ 10,000

Item	Description	Quantity	Unit	Unit Price	Total Price
11.	Intake Pipe Interconnections	JOB	JOB	L.S.	$ 25,000
12.	Intake Tower Bridge - Structural Steel	175	LF	$ 1,750	$ 306,250
13.	Intake Tower Bridge - Guiderail	350	LF	$ 50	$ 17,500
14.	Intake Tower Bridge - Open Grid Steel Deck	1,750	SF	$ 40	$ 70,000
15.	Intake Tower Bridge - Surface Preparation and Painting	JOB	JOB	L.S.	$ 75,000
16.	Concrete Protective Coating	650	SY	$ 35	$ 22,750
17.	Intake Tower Control Building	JOB	JOB	L.S.	$ 40,000
18.	Intake Tower Staff Gauge	92	LF	$ 130	$ 11,960
19.	42" Diameter Intake Line - Concrete Rehabilitation	500	LF	$ 40	$ 20,000
	Total, New Intake Tower				$2,020,960
				USE	$2,000,000

SEEPAGE ANALYSIS

General - All earth dams are permeable to some degree and will generate a certain amount of seepage. The challenge is to minimize and control seepage so that it will have no harmful affects. The character of the materials comprising the foundation and the embankment has an important influence on seepage and its effects. For earth dams, it is important that seepage be contained well within the downstream face.

Earth dams are founded on rock or pervious material that, after sufficient treatment, are relatively impervious to seepage. The dam foundation may be considered as the lower seepage boundary. The top most seepage boundary (or phreatic line) corresponds to the "water table" in the embankment. By definition, the seepage zone represents saturated material in the dam and is important for slope stability purposes. The location of the phreatic line is generally controlled by drainage facilities.

Figure No. 31: Downstream Embankment and Rock Toe (1958)

A statistical analysis of historical seepage data is provided in the Appendix. Using the "least squares" method, the average seepage flow (based on reservoir level) is as follows:

Table 17: Statistical Results of Seepage Flow

Water Surface Elevation	Seepage Flow (gpm)
1508*	197
1506	185
1504	173
1502**	161
1500	149
1495	119
1490	89
1485	60

* Crest of Rubber Dam
** Crest of Ogee Weir

Existing Seepage Control - For any dam of homogeneous and isotropic material formed on an impervious base, the seepage will pass through the dam and appear on the downstream face regardless of the material or how it was placed. If allowed to intersect the outside downstream face much above the toe, serious sloughing may result. Therefore, drainage is provided beneath the dam to convey the seepage safely downstream. For Mill Run dam, the following seepage control features are provided:

- The upstream portion of the dam is constructed of compacted earth. According to the specifications, this material was classified as "impervious fill." The material was furnished in a borrow area downstream of the dam. The laboratory tests on this soil included moisture content, grain size analysis, liquid and plastic limits and direct shear. This borrow material was placed and compacted with moisture control testing according to the specifications. No permeability tests were performed for use in a seepage analysis.

- The downstream portion is constructed of what is described as "random fill." This material is from the spillway and approach channel excavation. This fill zone (consisting generally of rock) is not designed within the saturation zone of the dam and, therefore, not subject to seepage. The plans or specifications do not call for a filter zone between the rockfill and impervious fill zone.

- Together, these components form a "zoned" embankment. Seepage is confined to the earth embankment zone. As stated, the design did not provide for a transition zone between the rockfill zone. This filter zone prevents penetration of finer material into coarser material due to water pressure or seepage. The filter also helps to prevent the earth from being carried away through the large interstices of the rockfill, thus resulting in piping. Fortunately, the drainage system has prevented seepage from entering the rockfill zone.

- A cutoff trench is located directly beneath the axis of the dam and extends to bedrock. Generally, the depth ranges from 5 to 10 feet and 15 to 30 feet in width. Impervious fill was placed and compacted in this area.

- To provide an "impervious foundation," a grout curtain was installed in the cutoff trench and extended into bedrock. The grouting was provided along the axis of the dam and concentrated at the left abutment/spillway. Overburden grouting was provided at the right abutment. The center portion of the dam is grouted to an average depth of 25 feet at a spacing of 25-feet.

- The cutoff trench and grout curtain should theoretically provide a "tight" foundation. Because of limited foundation treatment, the bedrock must have been judged (through core drill analysis and pressure testing) to be tightly jointed and relatively impermeable by the design engineers. The cutoff trench, made of denser soil, would reduce seepage in the upper "pervious" zone by virtue of the soil characteristics. Whether a completely impervious condition exists is unlikely since some contribution to seepage flow must occur. The amount of this seepage contribution is unknown, but is considered to be low when compared to the embankment.

- A sand drainage blanket is located in the impervious zone at the bottom of the dam. The drainage blanket extends from Sta. 2+00 to Sta. 8+30 (630 feet) and is installed parallel to the dam. Two sand drains convey seepage flow from the blanket 200 feet downstream to the rock toe. The sand blanket has a width of 20-feet and depth of 5 feet. The material for the blanket was specified as "concrete" sand.

- A rock toe helps to safely channelize and convey seepage to the downstream end of the embankment. The rock toe extends along the entire downstream toe of the dam.

- Finally, a toe drain system (consisting of a network of open joint and perforated pipes), collects the subsurface flow at the rock toe and conveys it to the blow-off channel below the valve vault. The flow is monitored by a V-notch weir.

Please refer to Figure 32 which shows a cross-section of the embankment and foundation drilling and grouting work.

The major purpose of these elaborate seepage control measures is three fold: a) prevent piping or conveyance of soil through the dam, b) confine seepage within the dam structure and c) control the quantity of seepage. Based on our historical observations, the first two measures have been entirely effective. No evidence of a "piping" condition has ever been observed. The sand drain system has also been effective in controlling seepage. It is confined within the dam and no evidence of saturation exists on the downstream face of the dam. Although wet areas are noted below the toe of the dam, they are not extensive and may be more indicative of an inadequate toe drain. Consideration of a new toe drain system should be a feature of any dam upgrade project (estimated cost $125,000).

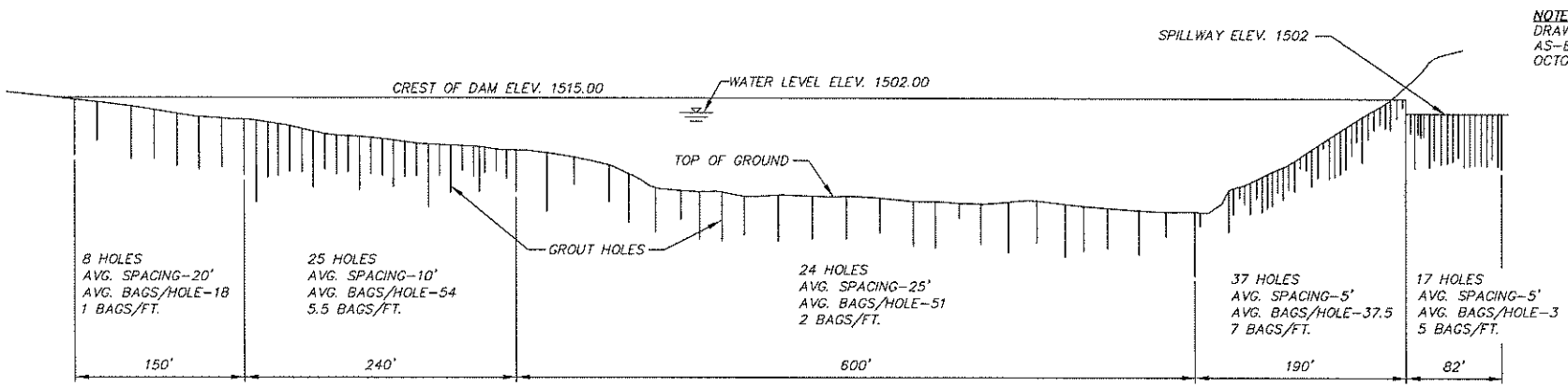
The final factor, seepage quantity, deserves a fuller examination. We plotted seepage flow against reservoir levels since 1980. It appears that at full capacity (El. 1502 to El. 1508, depending on the inflatable dam), seepage seems to vary between 200 - 250 gpm.

Our inspection of reports and available design documents does not reveal seepage calculations or expected seepage flow. However, the original design engineer, at a March 17, 1959 board meeting, indicated that the seepage flow "…was not unusual…"

Seepage Calculation - A preliminary assessment of seepage volume was computed on the basis of a flow net and parabolic curve (after Casagrande) calculations. Assumptions were made for soil permeability. A range of 250 to 350 gpm has been computed for a soil permeability coefficient ("K") of 0.0015 cm/sec. Although many variables are involved and certain simplifications were made, our calculation of seepage flow seems to agree with historical observations. Therefore, given the height and length of the dam, embankment material, methods of construction and drainage control system, the seepage flow does not appear to be excessive. In fact, this seepage comprises most of the required conservation release for Mill Run dam (323 gpm or 0.466 mgd). Please refer to the Appendix for seepage computations.

No further recommendations are offered for seepage control at this time other than continuous monitoring and observation by operating personnel and installation of a new toe drain at an estimated cost of $125,000.

NOTE:
DRAWING BASED ON AS-BUILT PLAN DATED OCTOBER 13, 1958



FOUNDATION DRILLING & GROUTING

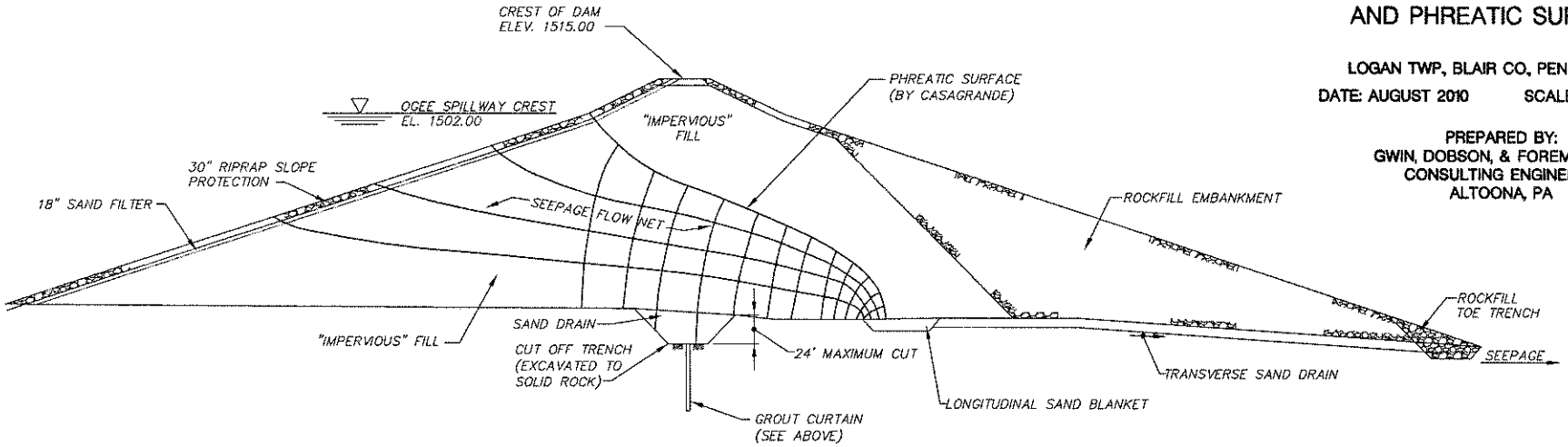
ALTOONA WATER AUTHORITY
EVALUATION OF MILL RUN DAM

FIGURE No. 32
FOUNDATION GROUTING AND PHREATIC SURFACES

LOGAN TWP, BLAIR CO, PENNSYLVANIA

DATE: AUGUST 2010 SCALE: AS SHOWN

PREPARED BY:
GWIN, DOBSON, & FOREMAN INC.
CONSULTING ENGINEERS
ALTOONA, PA



DAM EMBANKMENT PROFILE AND PHREATIC SURFACES & SEEPAGE FLOW NET
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SLOPE STABILITY

Embankment Slope Stability - Design engineers performed a stability analysis using soil test results from the planned borrow area. It appears that direct shear testing provided the necessary soil strength parameters (cohesion, internal friction angle and shear strength). The reported factors were: angle of internal friction (14° to 45° (24° degrees used)) and cohesion (0 to 0.49 tons/SF (0.10 tons/SF used)). The slope stability analysis yielded a safety factor of 1.5. A safety factor greater than unity is generally acceptable.

It is not known what stability analysis method was used. Since hand calculations at the time were laborious, this limited the analysis to a few critical failure circles. Finally, it is not known what embankment condition was used in the analysis.

Direct shear test results can be problematic. Samples are typically remolded for the test and may not represent in-situ strength parameters. Ideally, triaxial shear tests using "undisturbed" Shelby tube samples are the best method. However, undisturbed Shelby tube samples are difficult to obtain and are more suitable for clay soils than coarse, less cohesive soils (i.e., Mill Run dam). However, with proper judgement, direct shear test results can be used for a stability analysis, particularly those that have a coarser material component. It has been argued that borrow materials of this nature are essentially remolded during the excavation, placement and compaction phase of embankment construction.

Our interest in slope stability is to confirm the previous analysis. Stability analysis software can model numerous failure modes and embankment conditions. We entered the previous soil strength data and embankment geometry to determine safety factors. This "desk top" analysis is done for information and screening purposes only. No new soil sampling and testing was performed. The following assumptions were used:

1. The impervious fill which comprises the upstream slope, top of dam and part of the downstream slope (which is defined by "Soil Type No. 3" in the stability analysis) is assumed to have a total unit weight of 125 pcf and a saturated unit weight of 130 pcf. According to the September 1978 “Phase I Inspection Report National Dam Inspection Report," the embankment material has an average cohesion value of 0.10 tsf and an average internal friction angle of 24 degrees. To be conservative, a cohesion value of zero was used in the stability analysis.

2. It was assumed that the rockfill which makes up the majority of the downstream slope has a total and saturated unit weight of 145 pcf, an internal friction angle of 35 degrees and a cohesion value of zero.

3. The phreatic surface presumably takes the shape of a half parabola which terminates at the sand drain. Please refer to the seepage analysis for additional information. This is indicated by the blue dashed line in the stability sketch. This was derived since no phreatic water surface data is available from observation wells.

4. The Modified-Bishop method in the GSTABL7 (Version 2) computer software program was used. This program has been widely used in the engineering field including the PADEP Division of Dam Safety.

Preliminary Evaluation of Slope Stability - Both the upstream and downstream slopes have a factor of safety greater than 1.5 when evaluating the presumed, existing conditions (normal pool). The upstream slope value is 1.616 and the downstream safety factor is 1.794. When the phreatic surface is raised six (6) feet (inflation of rubber dam) the factor of safety values actual increases for both the upstream and downstream sections due to pore water pressure. However, seepage flow should increase due to the increased hydraulic head.

The only direct concern is during a "rapid drawdown" condition. Evaluating the upstream slope during rapid drawdown indicates that if the impervious fill material is undrained and cohesionless, the safety factor is less than unity (0.924) and may be indicative of a potential sloughing condition. However, if the impervious soil is assumed to have a cohesion value of 200 psf (as indicated in the report), the embankment would be considered stable (FS = 1.173). Please refer to the Appendix for stability analysis calculations and computer output for more information.

Our conclusion is that the original stability analysis was confirmed (F.S. = 1.5) if assuming, steady state conditions (i.e., reservoir at normal pool). Obviously, this is further confirmed by the fact that no evidence of slope failure has ever been observed. However, the rapid drawdown condition is considered the most critical failure mode for earth embankments. In this failure modeling condition, the safety factor is below 1.0. Therefore, consideration should be given to obtaining in-situ soil samples, performing soil testing and modeling an updated stability analysis during the design stage.

Table 18: Slope Stability Results

Slope Condition	Reservoir Level	Factor of Safety	Remarks
Downstream Slope	Full, Ogee Weir Crest	1.794	FS Greater than 1.0
Upstream Slope	Full, Ogee Weir Crest	1.616	FS Greater than 1.0
Downstream Slope	Full, Rubber Dam Crest	1.829	FS Greater than 1.0
Upstream Slope	Full, Rubber Dam Crest	1.687	FS Greater than 1.0
Upstream Slope	Rapid Drawdown (Ogee Weir Crest)	0.924	FS Less than 1.0 (No Cohesion)
Upstream Slope	Rapid Drawdown (Ogee Weir Crest)	1.173	FS Greater than 1.0 (No Cohesion)

Rock Slope Stability - The purpose of this evaluation is to assess the rock slope at the left (east) abutment of Mill Run dam. Historically, slope stability has been a concern of the Authority for many years. Rock falls, material sloughing and dangerous slide conditions have occurred, partially blocking the spillway channel and preventing access to the intake tower. In 2005, a rock removal project was completed at a cost of $67,000.

Original Design - The contract drawings show a steep rock cut from the intake tower to the stilling basin, a distance of 850 feet. The rock slope was installed at a slope of 1 horizontal to 6 vertical (80.5°). A twenty foot bench (access road/rock fall zone) is placed at the toe of the slope. At the top of the rock cliff, the slope is 1:1 in a boulder colluvial mass. The maximum height of the cliff is 85 feet. The average height along the left abutment is about 60.75 feet.

It is not known if a rock slope stability analysis was performed based on the test borings or other observations. The jagged, overbroken appearance of the rock face after construction (See Figure No. 33) shows that no pre-splitting blasting was performed. A smoother rock face would have resulted in less exposure to weathering of erodible beds (siltstones/mudstones).

Figure No. 33: Left Abutment Rock Slope from Spillway Approach Channel (1958)
(Note Jagged/Overbroken Rock Slope Face)

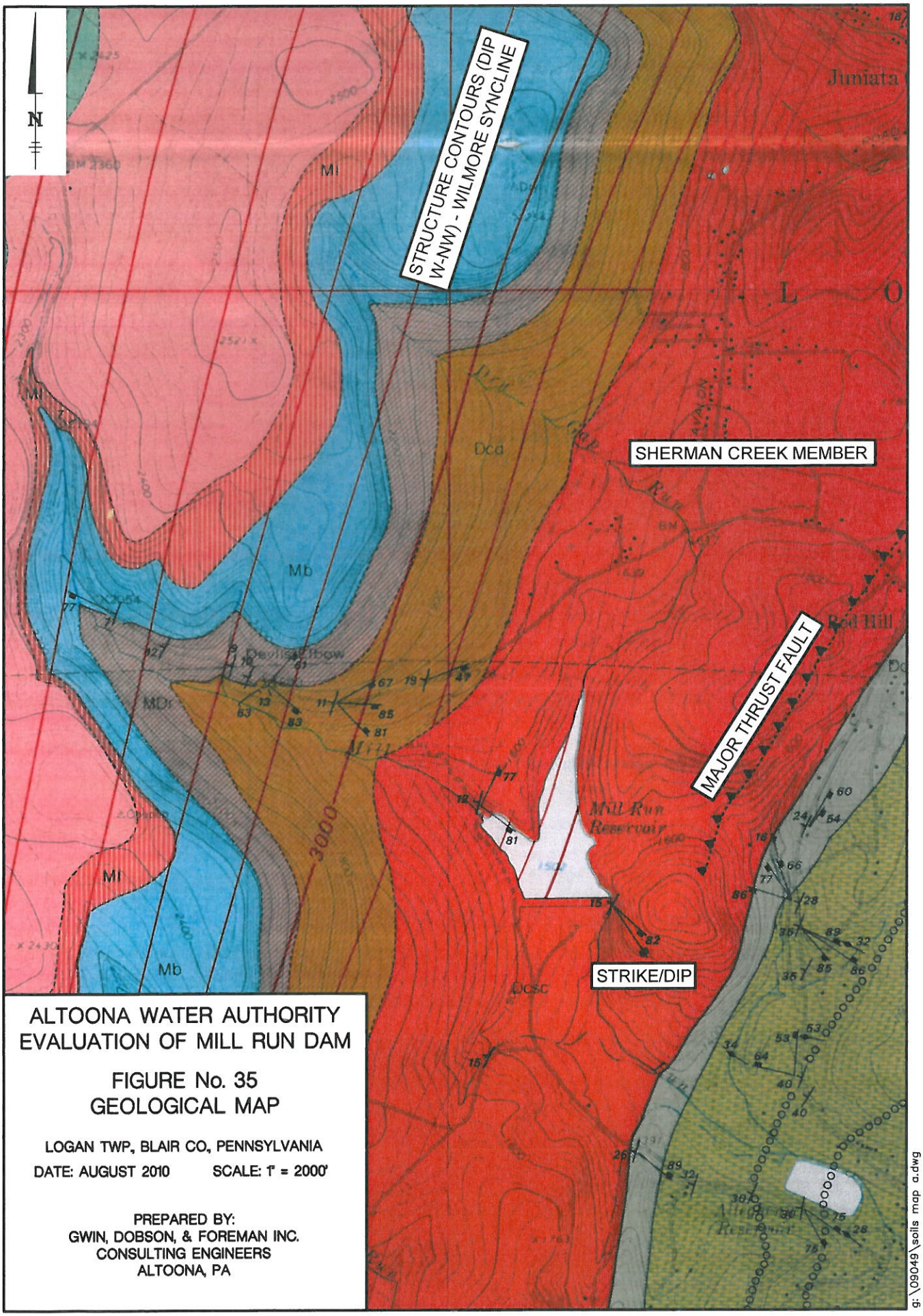
Physical Geology Description - In 1989, the PA Topographic and Geologic Survey (PAT&GS) published the "Geology and Mineral Resources of the Blandburg, Tipton, Altoona and Bellwood Quadrangles, Atlas 86." This report encompassed the Mill Run dam area. In summary, the dam is located in the Sherman Creek Member, Catskill Unit of the Devonian System (Upper Devonian). According to the report, *"…the Sherman Creek Member contains interbedded red and gray mudstone and siltstone, and gray sandstone. …Forms rolling hills of moderate to high relief at the foot of the Allegheny Front; moderate to steep natural slopes…Moderate to moderately high resistance to weathering. …Thickness: 1,400 feet to 2,000 feet… Fractures are moderately well developed, planar, curved and irregular and have moderate-to-wide spacing… Cut-slope stability is fair to good…"* The report references the cut-slope at Mill Run dam and includes a photo (Figure 25, p. 44).

Concerning structural geology, Mill Run dam is located on the southeastern limb of the Wilmore Syncline. A syncline is a fold of layered rock in which the layers dip toward the hinge; it is concave upward (like a bowl). This condition is opposite to the anticline which is a subsurface "dome" structure. A strike-and-dip measurement was taken at the east abutment. The beds dip to the northwest 15° (toward Buckhorn mountain) at a strike of N 40°E. A major thrust fault was inferred (or estimated) about 1,200 feet east of the rock slope. A thrust fault is defined as a "hanging" wall (upper block) which has moved or "thrusted" upward over the "footwall." Concerning fractures, measurements were taken at the rock face showing an 82° dip to the southwest on a NW to SE strike (See Figure No. 34).

Regarding surface geology features, Plate 3 of the report shows Mill Run dam surrounded by "boulder colluvium." This is described as an unsorted mixture of clay, cobbles, and boulders, and lesser amounts of silt, sand and pebbles. Thickness typically ranges from 15 to 20 feet. Cut-slope stability is judged to be fair-to-poor.

It can be inferred from these reports that the dam is founded on a heterogeneous bed of mudstones and sillstones with interbedded sandstone features. The beds dip under the dam toward the reservoir and have generally low permeability. The report does not provide detailed slope stability conclusions. Please refer to the following Figure 35 for the geologic map for the site area.

Figure No. 34: Rock Slope from Right Abutment Along Dam Axis (Note Dip of Beds Upstream and Flat Beds Downstream)



ALTOONA WATER AUTHORITY
EVALUATION OF MILL RUN DAM

FIGURE No. 35
GEOLOGICAL MAP

LOGAN TWP, BLAIR CO, PENNSYLVANIA

DATE: AUGUST 2010 SCALE: 1" = 2000'

PREPARED BY:
GWIN, DOBSON, & FOREMAN INC.
CONSULTING ENGINEERS
ALTOONA, PA

g:\09049\soils map a.dwg

Geotechnical Inspection - After a large rock slide in 2004-2005, GD&F investigated the conditions in greater detail. GD&F contacted its geotechnical consultant, Dr. James V. Hamel, P.E., to perform a site inspection and make recommendations. Dr. Hamel is a renowned expert in geotechnical engineering and rock mechanics and has worked on several Authority dams in a consulting capacity including the Kettle and Plane 9 dams.

On April 26, 2005, Dr. Hamel performed a detailed site investigation. His May 2, 2005 report is provided in the Appendix. The following technical discussion summarizes the inspection:

- Rock fall debris extended 90 feet along the spillway channel about 50 feet below the dam axis. A large block (bounded by fractures) slumped from the cut slope onto the bench. Portions of the rock broke off and fell into the spillway. The upper part consists of massive sandstone and the lower part of broken shale. The block was estimated to be 30 feet high by 30 feet deep by 30 feet wide.

- Minor seepage was noted at the bottom of the detached block near the shale-sandstone interface. This groundwater flow at the exposed fase would correspond to the regional dip of the bedding planes (toward the reservoir). AWA reported icicles on the rock face which may be indicative of frost wedging action. The rockfall debris consisted of fissile (split) red and gray-green shale and sandstone blocks. Weathering of the shale/siltstone beneath the sandstone seems to have caused the failure.

- Strike and dip measurements were take at several locations. A 12° dip (N30°W) along a strike of N60°E was measured on the sandstone bed about 150 feet upstream of the dam. Considerable water was seen at this rock face. The rock cliff in this area is quite ragged, overbroken and undercut on the weathered beds. At the dam axis, a measurement showed a flattening of the beds (dip 6°/S40° W; strike S50°E). The shale talus inclined at 36° to horizontal in the slide area. A concave recess at the slide area indicates a long-term slope retreat.

- Concerning structural geology, observation of the rock face shows a local anticline or "roll" near the fall zone. Downstream from the dam axis, the beds are relatively flat. A strong sandstone bed is located at the fold axis (hinge zone) with a fractured, axial plane front. Vertical flexural cracks were noted at both sides of the sandstone arch. They appear to be of a stress relief or tectonic nature. An examination of aerial PAT&GS photos may reveal a possible fault (extension of inferred thrust fault from PAT&GS report?). The rock slide is located directly down dip and eroded along the beds. Above the rock cliff, minor cracks (1-2 feet) were noted which may admit surface water into the rock mass.

- Rock in the vicinity of the recent slide appears to be more highly weathered than rock elsewhere in the cut slope. This localized weathering pattern along with boulder colluvium deposits, probably developed along rock fractures (joints, small faults) extending from the major thrust fault located northeast of the dam.

- Farther down the spillway cut, bent trees are evidence of a possible colluvium boulder slide developed by a fault. This rock mass may have been undercut by the original spillway excavation, thus causing a gradual creep of the slope. Above this area, rock shows deeper weathering with red/brown clay color and rock fragments. The whole slope is creeping downstream as evidenced by sandstone boulders on the slope and random sandstone slabs on-end. A creeping, talus/colluvium slide mass is pointed downstream from just below the slide area towards the stilling basin.

Figure No. 36: Rock Slide (2005)

Figure No. 37: Rock Slide (2005)

Figure No. 38: Rock Slide (2005)

Figure No. 39: Rock Slide (2005)

Figure No. 40: Rock Slope Above Slide Zone (2005) Toward Axis of Dam

Figure No. 41: Rock Slope Upstream of Dam Axis (2005)
(Note Dip of Beds (12° - 15°))

Figure No. 42: Rock Debris on Access Road at Axis of Dam

Figure No. 43: Rock Fall & Debris Above Axis of Dam
(Note Weathering)

Figure No. 44: Rock Fall & Debris Along Access Road to Intake Tower (Note Weathering Under Sandstone)

- Upslope of the rock cut, old logging roads and swales are diverting surface water into the slope area. No diversion ditches are in evidence. This condition can promote erosion of the shale/ siltstone bedding planes and frost heave of sandstone blocks along master joints and fracture zones.

Based on this field inspection, rock slope instability is caused by an excessive rock slope, lack of surface drainage, dip-slope erosion of shales/siltstones/mudstones beds under massive sandstone units and undercutting of the boulder-colluvial surface mass.

Remedial Measures - The following remediation measures are offered based on our very preliminary evaluation. Of course, these would be confirmed by a subsurface exploration and geotechnical/ geological study during final design.

- Create a new rock slope at a lower slope angle. This would extend from the intake tower to the spillway stilling basin area using pre-split drilling and blasting techniques. The pre-split blasting method uses lightly loaded, closely spaced drill holes fired before the production blast to form a fracture plane across which the radial cracking from the production blast cannot travel. This is accomplished by delayed blasting techniques used to separate the detonation times of explosive charges. The fracture plane is straight, not jagged or overbroken. It is also aesthetically appealing. Consideration of supplement benches parallel to the dip direction or surface slope will be evaluated during final design.

- Removal of surface rocks and boulders (colluvium) above the slope to prevent creep movement, particularly at the lower spillway area where this rock mass has been undercut by the original spillway excavation.

- Installation of diversion ditches and swales above the excavation to direct surface water away from the slope. Installation of a drainage ditch at the toe of rock slope to direct surface water downstream from the rock slope.

- As added protection, (particularly if the existing spillway and/or intake tower is to be retained), rock bolts/rock anchors may be required to stabilize and anchor the sandstone units that are subject to movement. Rock anchors are deformed steel bars inserted into predrilled holes in rock and secured with non-shrink grout or tensioned plates. The anchors may also be post-tensioned to induce a resisting force to ground movement. The steel members may be in the form of fabricated rock bolts, cable bolts, resin grouted thread bars, or rock dowels.

- Mechanical stabilization methods such as protective blankets and wire netting can prevent rock falls and reduce raveling on cut slopes. Structural stabilization in the form of shotcreting or gunite is another option. It is spray-applied on the slope face to a depth of 2-3 inches and binds together small rock fragments on the face. This approach is used primarily to prevent weathering and spalling of the rock surface.

Future Geotechnical Investigation - In order to confirm the scope of recommended improvements and to provide information for final design, a geotechnical evaluation and subsurface investigation and testing program is required. The basic elements of this plan include the following:

- Core borings drilled above and parallel to the rock slope advanced to a depth of 30 to 100 feet. Rock quality index and core samples will be obtained to indicate the character of the rock, structural geology data, acquicludes (impermeable barrier), possible disconformities/fault zones in the project area.

- Review all available geologic literature, aerial photographs and construction information. Field map all slope features.

- Photogrammetric mapping (1" = 20') of the rock cut to obtain evidence of fracture traces, linaments, fault lines and colluvial movement.

- Installation of observation wells and piezometers to determine groundwater levels and hydrostatic pressure distribution within the rock mass.

- Collection or rock samples for laboratory determination of rock unit weight and compressive strength.

- Collection of samples along potential dip-slope bedding planes (shales) for laboratory determination of cohesion and friction angle for use in plane shear analysis.

- Detailed analysis of potential failure surfaces and stability calculations of block stability along existing and proposed slope angles.

- Evaluation of additional stabilization techniques including rock anchors, rock bolts and rock dowels.

- Assessment of protective blankets and wire netting to control rock falls.

- Evaluation of structural stabilization including guniting and shotcreting for binding of slope face and prevention of weathering.

- Calculation of quantities, establishment of unit costs and preparation of cost estimates for various alternatives.

- Prepare geotechnical report summarizing subsurface exploration program, geological findings, rock/soil test results, alternatives evaluation, project cost estimates, conclusions and recommendations with exhibits, mapping, photos and plans.

Rock Slope Stability Evaluation - A very preliminary slope stability evaluation was made to examine worst-case conditions under this option. We assumed a "planar" failure along the maximum dip slope of the sandstone bed (15°) about 100 feet upstream of the dam axis. Values for cohesion (C=200 psf) and friction angle (14°) at the underlying shale/siltstone contact used published literature. A fracture crack was assumed near the top of the slope and extended vertically to the failure plane. A nominal hydrostatic force (tension crack) and uplift force (failure plane) was applied to the sliding mass.

The analysis yielded a factor-of-safety of just over unity (1.0), thus indicating a marginally stable slope. However, the slope in this area has not shown the degree of instability as the spillway slide area. Therefore, the analysis may not best truly represent the slope failure mode. Undercutting of the sandstone unit (by weathering of the weaker rocks) appears to be a more dominant factor.

Cost Evaluation - For purposes of this preliminary evaluation, we estimated construction costs for stabilization of the rock slope. The degree of stability depends on whether the existing spillway or intake tower are to be retained. If so, we believe a much greater degree of slope protection is warranted for safety reasons. Therefore, costs have been developed for two options as follows:

- **Minimal Rock Slope Stabilization** - This option assumes that the existing spillway will be demolished and the intake tower will be relocated into the reservoir. Therefore, the need to access the east abutment area will be greatly diminished. This option will also create a greater rockfall area downstream of the dam where the potential is greater. It should be mentioned that the east abutment is accessible by fording Mill Run downstream of the spillway and traversing the former spillway channel to the dam crest.

  The following work elements are considered the minimum for rock slope stabilization. Quantity calculations are provided in the Appendix.

  1. Reduction of rock slope from 6:1 to 2:1 (vertical-to-horizontal). This will involve pre-split and trim cushion blasting techniques along with rock excavation.

  2. Installation of toe of slope drainage ditches for conveyance of water away from the slope area.

3. Removal of boulder-colluvium above the rock slope that contributes to slope instability. Existing cross-sections show that this surface material is 10 feet thick above consolidated bedrock.

4. Installation of diversion ditches (upslope of the rock cliff) for conveyance of surface water away from the rock mass.

Table 19: Engineer's Opinion of Probable Construction Cost
Minimum Rock Slope Stabilization

Item	Description	Quantity	Unit	Unit Price	Total Price
1.	Mobilization/Demobilization	JOB	JOB	LS	$ 100,000
2.	Clearing and Grubbing	JOB	JOB	LS	$ 30,000
3.	Rock Slope Scaling Debris Removal	1,500	CY	$ 25.00	$ 37,500
4.	Rock Slope Scaling	200	CRHR	$ 350.00	$ 70,000
5.	Pre-Split Drilling, 3" Diameter @ 3 ft c-c	16,000	LF	$ 12.50	$ 200,000
6.	Production Drilling, 6" Diameter @ 8 ft c-c	6,500	LF	$ 20.00	$ 130,000
7.	Pre-Split/Trim (Cushion) Blasting	4,600	SY	$ 15.00	$ 69,000
8.	Rock Slope Excavation	15,800	CY	$ 30.00	$ 474,000
9.	Boulder Colluvium Excavation	22,250	CY	$ 20.00	$ 445,000
10.	Diversion Ditch and R-4 Riprap	850	LF	$ 40.00	$ 34,000
11.	Topsoil, Seeding & Mulching/ Erosion Control Blanket	6,700	CY	$ 10.00	$ 67,000
	Total, Minimum Rock Slope Stabilization			USE	$1,656,500
					$1,650,000

- **Optimum Rock Slope Stabilization** - This option involves additional structural and mechanical slope protection measures. As was discussed, if the existing spillway and the intake tower are to be retained, additional slope stability is assumed for safe accessibility. These measures are set forth below:

  1. All of the work elements associated with the minimum rock slope stability including drainage measures, reduction of rock slope, removal of boulder colluvium, rock excavation and pre-split drilling and blasting are included for this option.

  2. Installation of rock bolts to stabilize the sandstone rock mass(es) on the rock slope will provide greater resistance to sliding.

3. Application of shotcrete to the entire exposed face of the rock slope to seal and bind the rock fragments on the face will prevent weathering and spalling of the rock surface as well as knit together the slope surface. Provisions for wire mesh should be considered to provide additional tensile strength to the shotcrete.

4. Wire netting would be considered over the slope face to prevent rockfalls from bouncing outward from the toe region. Pinning the wire netting to the face holds the rock in place and reduces rock removal at the toe. Draping the wire netting involves anchoring the net to the crest of the slope and hanging it down over the face of the slope. Weights are added at the toe of slope to contain any rockfall.

Our preliminary calculations show that rock bolts will increase the slope factor of safety to 1.5, principally by supporting the sliding sandstone rock mass. Grade 60, 1¾ inch diameter by 30 feet long rock bolts have been sized. Spacing for the rock bolts is roughly half the length or every 15 feet. The rock bolts have been analyzed in a "passive" force mode. They will be "activated" only when the mass moves and subsequently loads the assembly. That is, the rock bolts are installed untensioned. Obviously, "active" rock bolts/rock anchors can be installed which are immediately tensioned upon installation.

Table 20: Engineer's Opinion of Probable Construction Cost
Optimum Rock Slope Stabilization

Item	Description	Quantity	Unit	Unit Price	Total Price
1.	Mobilization/Demobilization (Drilling/ Rock Bolts/Shotcrete/Wire Netting)	JOB	JOB	LS	$ 100,000
2.	Minimum Rock Slope Stability Measures	JOB	JOB	LS	$ 1,650,000
3.	Rock Bolt Drilling, 3½" Diameter	1,620	LF	$ 25.00	$ 40,500
4.	Rock Bolt Installation, 60K, 1¾" Diameter, Complete-in-Place	1,620	LF	$ 125.00	$ 202,500
5.	Wire Net Slope Protection	41,500	SF	$ 3.00	$ 124,500
6.	Shotcreting, Rock Slope Face	4,600	SY	$ 40.00	$ 184,000
	Total, Optimum Rock Slope Stabilization				$ 2,301,500
				USE	$ 2,300,000

The additional construction cost is $650,000 for optimum slope protection stabilization.

Stipulations - The above slope stability options are based on the best available information. A detailed geotechnical analysis, subsurface exploration and rock/soil testing program are needed to verify the above parameters during final design. Depending on the results, it is possible that full stabilization measures will be required regardless of the spillway option.

ALTERNATIVES EVALUATION

General - GD&F did a cost-effectiveness evaluation of various spillway options along with the necessary appurtenances to implement the option. For purposes of this evaluation, we have assumed the following:

1. Operation and costs are not considered for the alternatives evaluation as they were considered minimal.

2. Project cost components (engineering, legal, contingency, etc.) were considered the same for all options.

3. Accordingly, cost-effectiveness will be governed by construction costs for each of the components required for that alternative.

4. Construction costs are based on previous Authority projects updated with published historical cost indices (Engineering News Record) and supplemented with as-bid cost data from recent projects.

5. The RCC spillway option will require a new intake tower/bridge. The bridge length needed to span the spillway and reach the existing intake tower is excessive and not cost effective.

6. Bridge costs for spanning proposed spillways (except for RCC) are based on structural steel (open grid decks) superstructures. The amount of approach work depends on the location of the dam crest and presence of a floodwall. Costs are based on comparable Authority projects.

7. Toe drains are required for each alternative as the existing system is suspected of being clogged. The estimated replacement cost is $125,000.

8. Rock slope stabilization is required for all alternatives. However, we have assumed a higher degree of slope protection if either the intake tower and spillway are retained at their current location. This is reflected in a base cost for minimum stabilization and additional costs for optimum stabilization (rock anchors, netting, shotcrete, etc.).

9. The Mill Run stream channel below the spillways will need to be widened and protected with riprap. We have estimated a minimum channel length of 500-feet. Also required is a fording area and stabilized road. Based on preliminary quantities, the estimated cost is $450,000.

10. A cost of $250,000 has been assigned for demolition of the existing spillway along with backfilling. Material for filling the spillway is available from the proposed spillway excavation.

Cost Evaluation - The cost effectiveness evaluation for the various spillway alternatives follows:

Table 21: Cost Effectiveness Evaluation (New Intake Tower)

Construction Cost Component	Existing Spillway Improvements	Staged Ogee Spillway	Staged Labyrinth Spillway	RCC Overtopping Spillway Protection
PMF Capacity Spillway Improvements	$3,400,000	$5,000,000	$3,300,000	$6,600,000
Existing Spillway Demolition/Backfill	0	250,000	250,000	250,000
New Intake Tower and Access Bridge	2,000,000	2,000,000	2,000,000	2,000,000
New Toe Drain	125,000	125,000	125,000	125,000
Mill Run Channel Excavation & Protection	400,000	400,000	400,000	400,000
Minimum Rock Slope Stabilization	1,650,000	1,650,000	1,650,000	1,650,000
Additional Rock Slope Stabilization	650,000	0	0	0
Total Cost	**$8,225,000**	**$9,425,000**	**$7,725,000**	**$11,025,000**

Table 22: Cost Effectiveness Evaluation (Existing Intake Tower)

Construction Cost Component	Existing Spillway Improvements	Staged Ogee Spillway	Staged Labyrinth Spillway	RCC Overtopping Spillway Protection
PMF Capacity Spillway Improvements	$3,400,000	$5,000,000	$3,300,000	$6,600,000
Existing Spillway Demolition/Backfill	0	250,000	250,000	250,000
New Intake Tower Rehabilitation	535,000	535,000	535,000	0
New Intake Tower and Bridge	0	0	0	2,000,000
New Toe Drain	125,000	125,000	125,000	125,000
Mill Run Channel Widening & Protection	400,000	400,000	400,000	400,000
Optimum Rock Slope Stabilization	2,300,000	2,300,000	2,300,000	2,300,000
Spillway Bridge and Approach Work	250,000	650,000	500,000	0
Total Cost	**$7,010,000**	**$9,260,000**	**$7,410,000**	**$11,675,000**

Alternatives Evaluation - It is apparent that several options are relatively cost effective or within the "standard deviation" of comparable construction costs. Generally, a 5-10% (±) deviation is an acceptable range where the alternatives would be considered equivalent from a cost standpoint. These options are as follows:

a.	Existing Spillway Improvements (with Existing Intake Tower)	-	$7,010,000
b.	Staged Labyrinth Spillway (with Existing Intake Tower)	-	$7,410,000
c.	Staged Labyrinth Spillway (with New Intake Tower)	-	$7,725,000

Existing Spillway Improvements (with Existing Intake Tower) - The existing spillway option may appear to be the lowest alternative. However, this option heavily relies on the integrity of existing structural elements (ogee weir, spillway walls, etc.). Relying on a 50-year old structure often leads to differing site conditions (bad concrete, etc.) uncovered during construction. Another variable is the extent of rock slope stabilization. Our analysis is necessarily limited to a brief geotechnical inspection and a desktop stability analysis based on many assumptions. The final design may be significantly more extensive than our preliminary analysis. Finally, when weighing "new" versus "old" (or partially "old") construction, there really is no comparison. In terms of serviceability and long term structural integrity, this option is not recommended.

Staged Labyrinth Spillway (with Existing Intake Tower) - The analysis then is reduced to a choice of labyrinth spillway options, using either the existing of proposed intake tower. Both have their pros and cons. Obviously, using the existing tower will only involve renovation, not new construction. An access road already exists on the landward side, thus avoiding the need of a bridge projecting over the reservoir. However, a substantial bridge will be required over the labyrinth spillway, regardless. The uncertainty of rock stabilization is a cost variable, as already noted. This option will also rely on an existing structural element (concrete intake tower), though one apparently in good condition.

Staged Labyrinth Spillway (with New Intake Tower) - In the final analysis, a labyrinth spillway and new intake tower is only marginally more expensive ($300,000). It is also the option with the fewest unknowns and safety variables. Rock slope stabilization becomes less of a concern and may simply involve "trimming back" the slope (as we have assumed). Any debris and rock fall (if any) will be confined to an area that will not endanger the safety of structures or, more importantly, operating personnel. Although a long access bridge will be required, no bridge will be needed over the spillway. Access to the east abutment can still be attained by fording Mill Run and traversing the dam along the old spillway channel. Constructing a new intake into the reservoir may also improve water quality due to enhanced mixing and reservoir stratification. Finally, Authority personnel simply want a new intake tower for ease of accessibility and safety. Therefore, a new labyrinth spillway and intake tower is the recommended option for Mill Run dam.

PROJECT COST

General - The following project cost estimate includes those costs necessary to fully implement the recommended alternative. These include construction, engineering, legal, administrative and contingency costs and are set forth below.

Table 23: Estimated Project Cost
New Labyrinth Spillway and Intake Tower

No.	Component		Cost
1.	Construction Cost	$	7,725,000
2.	Engineering, Design and Construction Administration, Geotechnical Investigation and Subsurface Exploration and Testing	$	775,000
3.	Legal Cost	$	40,000
4.	Administrative Cost	$	35,000
5.	Interest During Construction	$	400,000
6.	Property Acquisition	$	25,000
7.	Contingency (5%)	$	500,000
	Total Estimated Project Cost New Labyrinth Spillway and Intake Tower	**$**	**9,500,000**

Summary of Recommended Improvements - The following summary sets forth our recommended improvements for modifications to Mill Run dam. The project will result in regulatory compliance with the PADEP Division of Dam Safety. The proposed improvements are shown on Figure No. 45.

- A multi-stage labyrinth spillway will provide sufficient capacity to pass the probable maximum flood (PMF) of 22,000 cfs and control downstream discharges up to the 100-year flood (approximately 1,400 cfs). The labyrinth spillway consists of the following components:

    1. Single Cycle, 16° Primary Spillway (35 feet wide) will control normal discharges up to 1,400 cfs. The crest elevation is set at El. 1,506 msl. The total capacity of the primary spillway is 5,573 cfs for the maximum head condition of El. 1,515 (top of dam).

    2. Three Cycle, 14° Secondary (Emergency) Spillway (127 feet wide) will have a capacity of 16,438 cfs for the maximum head condition of El. 1,515 (top of dam). The total combined spillway capacity is 22,011. The crest elevation is 3 feet above the primary spillway crest at El. 1,509.

    3. Spillway chute, separated by a common wall, will convey normal and emergency flow a distance of 309 feet to the deflector buckets, each with a radius of 51 feet and length of 40 feet.

- Downstream channel improvements include widening Mill Run a distance of 500 feet below the spillway. Rock armor and riprap will provide erosion protection for energy dissipation of the deflector bucket jet. Increased channel capacity will protect the Mill Run water treatment plant from increased flood events.

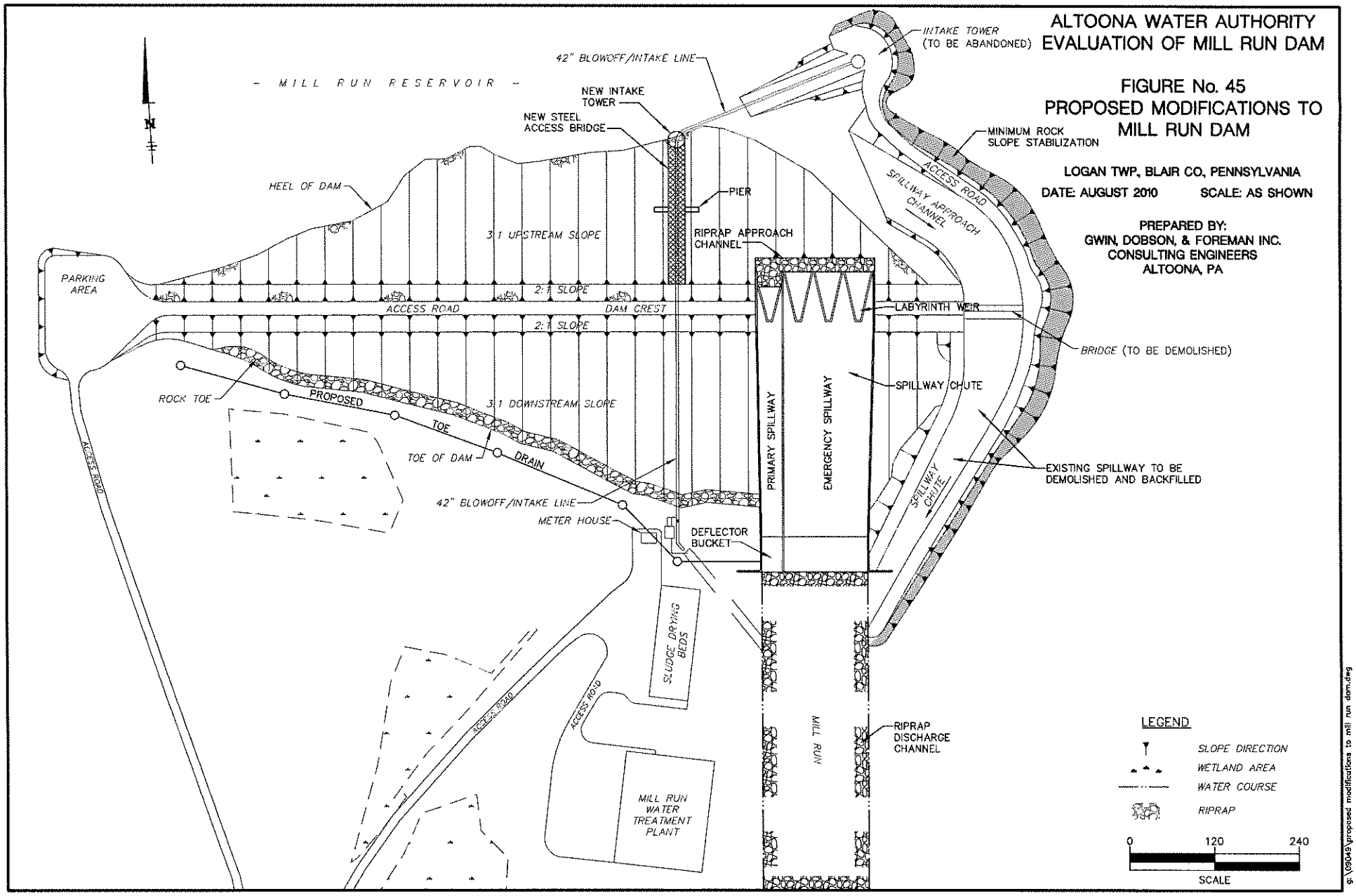
- A new, multi-port intake tower will be constructed in the reservoir about 250 feet west of the current location. The tower will have a 9-foot inside diameter and be 92 feet high. A new 175-foot long, double span steel bridge will provide access from the dam crest. The tower at this location will provide better water quality and offer a safer, more accessible access.

- Stabilization of the rock slope will consist, initially, of laying back the existing slope by selective drilling and blasting techniques. Removal of loose boulders and rock above the slope area will be done. Better drainage facilities are also planned. Final design work may warrant the need for additional measures.

- A new toe drain system will be installed to more efficiently collect and convey seepage. A seepage flow of 200-250 gpm at steady state normal pool conditions is not considered excessive.

- The existing spillway will be demolished and backfilled with excavation from the new labyrinth spillway. This area will serve as a rock-fall zone (if required) and as access to the left abutment area.

- The total estimated project cost to implement the Mill Run dam improvements is $9,500,000.



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Implementation Arrangements - We anticipate the following tasks will be required to implement the project.

1. Submission of the approved report to the PADEP Division of Dam Safety for review and approval of the design concept. In our opinion, this project has not previously been considered a mandatory compliance effort by either the PADEP or the Authority. It has been assumed that these improvements would be included in an overall upgrade project when funds permit.

2. Authorization of engineering design work and preparation of plans and specifications as precedent to submission of a PADEP Division of Dam Safety permit application. This work includes a subsurface exploration and soil/rock testing program for a geotechnical engineering report.

3. Submission of applications to the various funding agencies to finance the project. If PennVEST funding is approved, the project debt service for a traditional PennVEST loan (2.23%) is $600,000/year.

4. The next several years will see the retirement of several PennVEST loans (Kettle Dam, Plane 9/Tipton WTP's) totaling $870,000/year. Although these loan retirements would appear to offset the Mill Run project, the amortization schedule for the 2007 Refunding Series has accounted for these apparent savings. Therefore, the projected rate increase to fund the Mill Run project would be about 4%.

5. The estimated construction time is 2 years. The overall time to implement the project is 3 to 4 years.

6. The project will require draining Mill Run dam for the duration of construction. This represents the loss of a large storage component of the system (20%) for an extended duration. In addition, the Mill Run water treatment plant will not have a gravity source of supply. Certain operational adjustments will be needed to ensure interrupted water supply to the service area, especially during drought conditions.

7. This project may necessitate diversion of Mill Run into Allegheny Reservoir and actuation of the Allegheny pump station to convey raw water back to the Mill Run treatment plant. In addition, the Allegheny dam and reservoir will need to be reassessed in light of new PMF criteria.

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BIBLIOGRAPHY

1. Advanced Technology Systems, *Hydrologic Study of Mill Run Watershed*, 2004, U.S. Army Corps of Engineers

2. Albright and Friel, Inc., *Report to City of Altoona on Water Supply*, 1953, Philadelphia, PA

3. Altoona Water Authority, *Annual Dam Inspections*, 1980-2009, Blair Co., PA

4. Brater, E.F., King, H., *Handbook of Hydraulics*, 1976, McGraw-Hill, N.Y., N.Y., 6th Ed.

5. Casanave, E.A., Orner, M.R., Gwin, Dobson & Foreman, Inc., *Computer Breaching Analysis for Mill Run Dam*, PA DEP No. 007-082, February 2000, Altoona Water Authority, Blair Co.

6. Chow, V.T., *Open Channel Hydraulics*, 1959, McGraw-Hill Book Co., N.Y., N.Y.

7. Chow, V.T., Maidment, D.R., Mays, L.W., *Applied Hydrology*, 1988, McGraw-Hill, N.Y., N.Y.

8. Creager, W.P., Justin, J.D., and Hinds, J., 1964, *Engineering for Dams*, Vol. I&II, John Wiley and Sons, Inc., N.Y., N.Y., 8th Printing

9. D'appolonia Consulting Engineers, *U.S. Army Corps of Engineers, Phase I Dam Inspection Report, ID No. 533 Mill Run Dam (1978) and ID No. 526 (1980)*, Allegheny Storage Dam, Blair Co., PA

10. Falvey, H.T., *Hydraulic Design of Labyrinth Weirs*, 2003, American Society of Civil Engineers, N.Y., N.Y.

11. Glenn, M.V., Young, K.A., Gwin, Dobson & Foreman, Inc., *Hydrologic and Hydraulic Evaluation of Water Supply Reservoirs*, Horseshoe Curve Area, June 1979, Altoona City Authority, Altoona, PA

12. Gwin, Dobson & Foreman, Inc., *Mill Run Flood Plain Information Report*, January 1974, U.S. Army Corps of Engineers, Baltimore District

13. Kliche, C., *Rock Slope Stability*, 1999, Society of Mining Engineers, Littleton, CO

14. Miller, A.C., Kerr, S.N., Spaeder, D.J., *Calibration of Snyder Coefficients for Pennsylvania*, August 1983, Water Resources Bulletin, No. 83020, American Water Resources Association

15. Morris, H.M. and Wiggert, J.M., *Applied Hydraulics in Engineering*, 1972, Ronald Press, N.Y., N.Y.

16. Tullis, J.P., Amanian, N., Waldron, D., *Design of Labyrinth Spillways*, 1995, Journal of Hydraulic Engineering, American Society of Civil Engineers, N.Y., N.Y.

17. U.S. Department of Commerce, National Weather Service, *Hydrometerological Report No. 51, Determination of Probable Maximum Precipitation*, June 1978, U.S. Government Printing Office.

18. U.S. Department of Commerce, National Weather Service, *Hydrometerological Report No. 40, Determination of Probable Maximum Precipitation - Susquehanna River Drainage Above Harrisburg, PA,* 1965, U.S. Government Printing Office.

19. U.S. Department of Defense, Army Corps of Engineers, Civil Engineering Bulletin No. 52-8, *Standard Project Flood Determinations*, 1965, Washington, D.C.

20. U.S. Department of Defense, Army Corps of Engineers, Civil Engineering Bulletin No. 1110-2-1602, *Hydraulic Design of Reservoir Outlet Works*, 1980, Washington, D.C.

21. U.S. Department of Defense, Army Corps of Engineers, EM 1110-2-2006 Design Manual - *Roller Compacted Concrete*, Jan. 2000, Washington, D.C.

22. U.S. Department of Defense, Army Corps of Engineers, *Flood Hydrograph and Analysis*, 1959, Washington, D.C.

23. U.S. Department of Defense, Army Corps of Engineers Hydrologic Engineering Center, *Hydrologic Modeling System (HEC-HMS Version 3.4)*, August 2009, Washington, D.C.

24. U.S. Department of Homeland Security, Federal Emergency Management Agency, *National Flood Insurance Program, Flood Boundary and Floodway Map*, October 17, 1986, Washington, D.C.

25. United States Department of the Interior, Bureau of Reclamation, *Design of Small Dams*, 1974, U.S. Government Printing Office, Washington, D.C. Revised Reprint, 2nd Edition

26. URS Greiner Woodward Clyde Consultants, *Design Manual for RCC Spillways and Overtopping Protection*, 2002, Portland Cement Association, Skokie, Illinois

27. Wyllie, C.D., Mah, C.W., *Rock Slope Engineering*, 2004, Spon Press, 4 Ed.

28. Young, K.A., Glenn, M.V., Gwin, Dobson & Foreman, Inc., *Hydrologic and Hydraulic Evaluation of Allegheny Reservoir*, July 1979, Altoona City Authority, Altoona, PA