DRAFT

Coal Combustion Waste Impoundment

Task 3- Dam Assessment Report

John E. Amos Plant (Site 26)

Fly Ash Dam Complex
American Electric Power
St. Albans, West Virginia

Project # 0-381

Assessment of Dam Safety
Coal Combustion Surface Impoundments
for the REAC Program

Prepared for:

Lockheed Martin Services, Inc.
Edison, New Jersey
for
United States Environmental Protection Agency

Prepared by:

Dewberry
Fairfax, Virginia

Under Contract Number: EP-C-04-032: REAC

September 2009

INTRODUCTION, SUMMARY CONCLUSIONS AND RECOMMENDATIONS

The release of over five million cubic yards of coal ash from the Tennessee Valley Authority's Kingston, Tennessee, facility in December 2008, which flooded more than 300 acres of land, damaging homes and property, is a wake-up call for diligence on coal combustion waste disposal units. We must marshal our best efforts to prevent such catastrophic failure and damage. A first step toward this goal is to assess the stability and functionality of the ash impoundments and other units, then quickly take any needed corrective measures.

This assessment of the stability and functionality of the John Amos Fly Ash Dam management unit is based on a review of available documents and on the site assessment conducted by Dewberry personnel on Tuesday, September 8, 2009. We found the supporting technical documentation adequate (Section 1.1.3). As detailed in Section 1.2.6, there are five recommendations that may help to maintain a safe and trouble-free operation, and we recommend an updated dam break analysis.

In summary, the John Amos Fly Ash Dam is SATISFACTORY for continued safe and reliable operation, with no recognized existing or potential management unit safety deficiencies. However, the utility has submitted an application for raising the dam by 27 feet. The State of West Virginia has indicated it will soon issue a Certificate of Approval to raise the dam level.

PURPOSE AND SCOPE

The U.S. Environmental Protection Agency (EPA) is embarking on an initiative to investigate the potential for catastrophic failure of Coal Combustion Surface Impoundments (i.e., management unit) from occurring at electric utilities in an effort to protect lives and property from the consequences of a dam failure or the improper release of impounded slurry. The EPA initiative is intended to identify conditions that may adversely affect the structural stability and functionality of a management unit and its appurtenant structures (if present); to note the extent of deterioration (if present), status of maintenance and/or a need for immediate repair; to evaluate conformity with current design and construction practices; and to determine the hazard potential classification for units not currently classified by the management unit owner or by a state or federal agency. The initiative will address management units that are classified as having a Less-than-Low, Low, Significant or High Hazard Potential ranking. (For Classification, see pp. 3-8 of the 2004 Federal Guidelines for Dam Safety)

In February 2009, the EPA sent letters to coal-fired electric utilities seeking information on the safety of surface impoundments and similar facilities that receive liquid-borne material that store or dispose of coal combustion waste. This letter was issued under the authority of the Comprehensive Environmental Response, Compensation, and Liability Act (CERCLA) Section 104(e), to assist the Agency in assessing the structural stability and functionality of such management units, including which facilities should be visited to perform a safety assessment of the berms, dikes, and dams used in the construction of these impoundments.

EPA asked utility companies to identify all management units: surface impoundments or similar diked or bermed structures; and landfills receiving liquid-borne material that store or dispose of coal-combustion residuals or by-products, including, but not limited to, fly ash, bottom ash, boiler slag, and flue gas emission control residuals. Utility companies responded with information on the size, design, age, and the amount of material placed in the units so that EPA could gauge which management units had or potentially could rank as having High Hazard Potential. The USEPA and its contractors used the following definitions for this study:

> "Surface Impoundment or impoundment means a facility or part of a facility which is a natural topographic depression, man-made excavation, or diked area formed primarily of earthen materials (although it may be lined with man-made materials), which is designed to hold an accumulation of liquid wastes or wastes containing free liquids, and which is not an injection well. Examples of surface impoundments are holding, storage, settling, and aeration pits, ponds, and lagoons."
>
> For this study, the earthen materials could include coal combustion residuals. EPA did not provide an exclusion for small units or based on whether the placement was temporary or permanent. Furthermore, the study covers not only waste units designated as surface impoundments, but also other units designated as landfills which receive free liquids.
>
> EPA is addressing any land-based units that receive fly ash, bottom ash, boiler slag, or flue gas emission control wastes along with free liquids. If the landfill is receiving coal combustion wastes with liquids limited to that for proper compaction, then there should not be free liquids present and EPA did not seek information on such units which are appropriately designated a landfill.
>
> In some cases coal combustion wastes are separated from the water, and the water containing de minimus levels of fly ash, bottom ash, slag, or flue gas emission control wastes, are sent to an impoundment. EPA is including such impoundments in this study, because chemicals of concern may have leached from the solid coal combustion wastes into the waste waters, and suspended solids from the coal combustion wastes remain.

The purpose of this report is to evaluate the condition and potential of waste release from the selected High Hazard Potential management units. This evaluation included a site visit. Prior to conducting the site visit, a two-person team reviewed the information submitted to EPA, reviewed any relevant publicly available information from state or federal agencies regarding the unit hazard potential classification (if any) and accepted information provided via telephone communication with a management unit supervisor.

EPA sent two professional engineers, one licensed in the State of West Virginia, for a one-day site visit. The two-person team met with the owner of the management unit as well as several technical representative and management unit supervisors to discuss the engineering characteristics of the unit as part of the site visit. During the site visit the team collected additional information about the management unit to be used in determining the hazard potential classification of the unit. Subsequent to the site visit the management unit owner provided additional engineering data pertaining to the management unit.

Factors considered in determining the hazard potential classification of the management units(s) included the age and size of the impoundment, the quantity of coal combustion residuals or by-products that were stored or disposed of in these impoundments, its past operating history, and its geographic location relative to down gradient population centers and/or sensitive environmental systems.

This report presents the opinion of the assessment team as to the potential of catastrophic failure and reports on the condition of the management unit(s). The team considered criteria in evaluating dams under the National Inventory of Dams, in making these determinations.

LIMITATIONS

The assessment of dam safety reported herein is based on field observations and review of readily available information provided by the owner/operator of the subject coal combustion waste management unit(s). Qualified Dewberry engineering personnel performed the field observations and review and made the assessment in conformance with the required scope of work and in accordance with reasonable and acceptable engineering practices. No other warranty, either written or implied, is made with regard to our assessment of dam safety.

TABLE OF CONTENTS

DRAFT

APPENDICES

APPENDIX A – REFERENCE DOCUMENTS

Doc 01: Dam Location-Google Aerial
Doc 02: Monitoring & Emergency Action Plan & maintenance Plan
Doc 03: Review Document List
Doc 04: H&H Calculations
Doc 05: H&H Review
Doc 06: Dam Break Analysis
Doc 07: 2009 Dam and Dike Inspection Report - John Amos
Doc 08: 2009 Geotechnical Report - John E. Amos
Doc 09: 2009 DEP Fly Ash Dam Inspection report
Doc 10: Embankment Section
Doc 11: Embankment Plan
Doc 12: Slope Stability-Dam El 830
Doc 13: Slope Stability-Dam El 875
Doc 14: Fly Ash Retention Damn Operation and Maintenance Manual - July 1980

APPENDIX B – PHOTOGRAPHS

Doc 01: Photographs

APPENDIX C – FIELD OBSERVATION CHECKLIST

Doc 01: Dam Inspection Checklist Form

1.0 CONCLUSIONS AND RECOMMENDATIONS

1.1 CONCLUSIONS

Conclusions are based on visual observations from our one-day site visit and review of technical documentation provided by American Electric Power.

1.1.1 Conclusions Regarding the Structural Soundness of the Management Unit(s)

The embankment and spillway appear to be structurally sound based on a review of the engineering data provided by the owner's technical staff and Dewberry engineers' observations during the site visit,

1.1.2 Conclusions Regarding the Hydrologic/Hydraulic Safety of the Management Unit(s)

Adequate freeboard and capacity exist to safely pass the Probable Maximum Flood (PMF) based on the engineering analyses provided for Dewberry's review,

1.1.3 Conclusions Regarding the Adequacy of Supporting Technical Documentation

The supporting technical documentation is adequate. Engineering documentation reviewed is referenced in Appendix A.

1.1.4 Conclusions Regarding the Description of the Management Unit(s)

The description of the management unit provided by American Electric Power (AEP) was an accurate representation of what Dewberry observed in the field.

1.1.5 Conclusions Regarding the Field Observations

Dewberry staff was provided access to all areas in the vicinity of the management units required to conduct a thorough field observation. The visible parts of the embankment dam and outlet structure were observed to have no signs of overstress, significant settlement, shear failure, or other signs of instability, although visual observations were severely hampered by the presence of thick vegetation. Embankments visually appear structurally sound. There are no apparent indications of unsafe conditions or conditions needing remedial action.

1.1.6 Conclusions Regarding the Adequacy of Maintenance and Methods of Operation

The current maintenance and methods of operation appear to be adequate for the fly ash management unit. There was no evidence of repaired embankments or prior releases observed during the field inspection.

1.1.7 Conclusions Regarding the Adequacy of the Surveillance and Monitoring Program

The surveillance and monitoring program appears to be adequate. The monitoring program includes 37 piezometers and 24 deformation monuments installed on the crest, slope and abutments of the dam. Plant personnel monitor the instrumentation on a monthly basis.

1.1.8 Classification Regarding Suitability for Continued Safe and Reliable Operation

Facility is SATISFACTORY for continued safe and reliable operation. No existing or potential management unit safety deficiencies are recognized. Acceptable performance is expected under all applicable loading conditions (static, hydrologic, seismic) in accordance with the applicable criteria.

1.2 RECOMMENDATIONS

1.2.1 Recommendations Regarding the Structural Stability

None appear warranted at this time.

1.2.2 Recommendations Regarding the Hydrologic/Hydraulic Safety

None appear warranted at this time. However it is recommended that a new analysis be provided with updated survey information, since the last dam break analysis was performed over 15 years ago. The John Amos Monitoring and Emergency Action Plan dated January 24, 2008 (see Appendix A-Doc 02) presents a comprehensive assessment of critical infrastructures downstream of the dam. According to Appendix D of this document, a number of residences and businesses have also been identified to be within the flood inundation area. However, the inundation area is apparently based on the original dam break analysis. Several changes in the assumption and methodology compared to the original analysis would be required to assure the new dam break analysis would be consistent with current engineering practice. These changes are listed in Section 6.1.4 of this report.

1.2.3 Recommendations Regarding the Supporting Technical Documentation

No recommendations appear warranted at this time.

1.2.4 Recommendations Regarding the Description of the Management Unit(s)

No Recommendations appear warranted at this time.

1.2.5 Recommendations Regarding the Field Observations

No recommendations appear warranted at this time.

1.2.6 Recommendations Regarding the Maintenance and Methods of Operation

The maintenance and operation of the dam seem to be adequate. However, the following recommendations may help maintain safe and trouble-free operation:

- The WV – Dam Safety report (see Appendix A- Doc 09) indicated that AEP engineers inspected the tunnel and pipe in October 2008. The AEP inspection report concluded the tunnel and pipe were in good condition. The AEP report included a recommendation that the spillway tunnel be re-lined or abandoned. The WV-Dam Safety report indicated that the tunnel will be abandoned and fully grouted as part of the planned dam expansion project;
- Repair access stairs and walkways to monitoring points if the dam in not expanded;
- Evaluate the cause of movement restrictions of floating docks around the decant riser and repair wooden walkway to overflow structure;
- Evaluate valve condition at the overflow structure to assure it is operational in case flow out of the pond needs to be stopped;
- Monitor, address or otherwise repair minor erosion areas and erosion gullies, wet areas and isolated seepage spots.

1.2.7 Recommendations Regarding the Surveillance and Monitoring Program

Continue monitoring existing groundwater, seepage locations and survey monuments.

1.2.8 Recommendations Regarding Continued Safe and Reliable Operation

No recommendations pertaining to the continued safe and reliable operation of the management unit appear warranted at this time.

1.3 PARTICIPANTS AND ACKNOWLEDGEMENT

1.3.1 List of Participants

Clark Stephen Conover- Environmental Protection Agency (EPA), Region 3
Behrad Zand, American Electric Power (AEP)
Gary Zych, American Electric Power (AEP)
Alan Wood, American Electric Power (AEP)
Son Webster, American Electric Power (AEP)
Roger R. Mc Kinney, American Electric Power (AEP)
R.T. Carroll, American Electric Power (AEP)
Paul Regan, American Electric Power (AEP)
James Filson, P.E. – Dewberry
Joseph P. Klein, III, P.E. – Dewberry

1.3.2 Acknowledgement and Signature

We acknowledge that the management unit referenced herein has been assessed on September 8, 2009.

James Filson, PE (WV # 014013)

Joseph P. Klein, III, P.E. Geotechnical Engineer

2.0 DESCRIPTION OF THE COAL COMBUSTION WASTE MANAGEMENT UNIT(S)

2.1 LOCATION AND GENERAL DESCRIPTION

The John E. Amos Plant is located by the Kanawha River bank in Putnam County, West Virginia just across the river from Raymond City, West Virginia. The plant is operated by Appalachian Power, a subdivision of American Electric Power (AEP). The Fly Ash Dam is approximately 1.5 miles southwest of the plant, north of Interstate 64 in the headwaters of Scary Creek, a tributary of the Kanawha River. The dam is near Teasy Valley, north of Scary in Putnam County. A project location aerial photograph is provided in Appendix A – Doc 01.

The John Amos Fly Ash Dam is a zoned rock fill dam with a clay core constructed in phases in the upper Scary Creek Watershed. The crest of the dam is at elevation 875 feet. The downstream toe of the dam is at elevation 652 feet, making the dam height 223 feet. At the crest elevation of 875 feet, the impoundment area is approximately 175 acres with a total storage capacity of approximately 2,687 acre-feet. The normal pool elevation is 857 to 858 feet, impounding a surface area of approximately 160 acres.

The dam was constructed in three stages. Stage 1 was constructed in the early 1970s to a crest elevation of 810 feet. Stages 2 and 3 were constructed in the mid to late 1970s as a continuous project. Stages 2 and 3 were constructed to elevations of 845 and 875, respectively. Expansion was accomplished by extending the upstream slope upward, widening the crest and enlarging the downstream slope (i.e., making the dam higher and thicker). The construction sequence avoided constructing new foundations in the impounded area.

Material for embankment construction was quarried from the hillside to the north and up gradient from the dam location.

2.2 SIZE AND HAZARD CLASSIFICATION

The classification for size, based on the height of the dam is "Large" and based on the storage capacity is "Intermediate" in accordance with the USACE Recommended Guidelines for Safety Inspection of Dams ER 1110-2-106 criteria summarized in Table 2.2a.

Table 2.2a USACE ER 1110-2-106
Size Classification

Category	Impoundment Storage (Ac-ft)	Height (ft)
Small	50 and < 1,000	25 and < 40
Intermediate	1,000 and < 50,000	40 and < 100
Large	> 50,000	> 100

The John Amos Fly Ash Dam has been classified by the West Virginia Dam Safety as a Class "I", which is a structure which has "High Hazard" potential. The West Virginia Dam Safety rules define High Hazard dams as: "those dams located where failure may cause loss of human life or major damage to dwellings, commercial or industrial buildings, main railroads, important public utilities, or where a high risk highway may be affected or damaged". This classification is similar to "High Hazard" classification per the Federal Guidelines for Dam Safety dated April 2004. As shown in Table 2.2b, dams assigned the "high hazard potential" classification are those dams where failure or mis-operation results in (probable) one or more expected loss of human life, with probable economic loss, environmental damages and disruption of lifeline facilities.

Table 2.2b FEMA Federal Guidelines for Dam Safety Hazard Classification

Hazard Potential Classification	Loss of Human Life	Economic, Environmental, Lifeline Losses
Low	None Expected	Low and generally limited to owner
Significant	None Expected	Yes
High	Probable. One or more expected	Yes (but not necessary for this classification)

2.3 AMOUNT AND TYPE OF RESIDUALS CURRENTLY CONTAINED IN THE UNIT(S) AND MAXIMUM CAPACITY

The data reviewed by Dewberry included reservoir design filling curves and filling rate estimates. Data on the volume of residuals stored in the fly ash pond at the time of inspection were not indicated. The surface area for the pond at the Normal pool Elevation of 857 to 858 ft is approximately 160 acres, and 175 acres maximum, see Table 2.3.

Table 2.3: Amount of Residuals and Maximum Capacity of Unit

	John Amos Fly Ash Pond Dam
Surface Area (acre)	175
Current Storage Capacity (acre-feet)	Data not provided
Total Storage Capacity (acre-feet)	2,687
Crest Elevation (feet)	875
Normal Pond Level (feet)	857 to 858

2.4 PRINCIPAL PROJECT STRUCTURES

2.4.1 Earth Embankment Dam

The dam is a zoned rock fill dam with a clay core constructed in phases. The 30-foot wide crest has a length of 1,977 feet. The alignment of the dam curves upstream in the center. Underdrain pipes allow seepage through and under the embankment. The upstream slope is approximately 2.5 to 3 horizontal to 1 vertical. The downstream slope is approximately 2 to 2.5 horizontal to 1 vertical with a bench at elevation 713 to 716 feet. The upstream slope is covered with grass. The downstream slope of the entire dam is covered with large rock boulders (rip-rap). Stage 3 dam design drawings are available that show the earth embankment as designed with the final dam crest elevation of 875 ft (See Appendix A: Doc 10, "Embankment Section.pdf" and Appendix A: Doc 11, "Embankment Plan.pdf"). The dam crest elevation actually varies from approximately 875 ft on the ends to approximately 877 ft in the middle. More recent and better quality drawings showing the dam plan, profile, service spillway arrangement and tunnel details and dam instrumentations are included in Appendix A: Doc 05 pages 108-111. Table 2.4.1 displays a summary of the dimensions and size specifications of John Amos Fly Ash Dam. Photo Numbers 1-3, 10 and 15-19 in Appendix B show the embankment of the dam.

Table 2.4.1: Summary of Dam Dimensions and Size

	John Amos Fly Ash Pond Dam
Dam Height	223'
Crest Width	30'
Length	1,977'
Side Slopes (upstream)	2.5-3:1
Side Slopes (downstream)	2-2.5:1
Hazard Classification	High

2.4.2 Outlet Structures

The dam has a concrete decant riser 8 feet in diameter with invert elevation at 799.3 ft and a tunnel/pipe 8 ft in diameter located in original ground off the northwest end of the dam, which discharges through the hillside into an adjacent tributary to Scary Creek. There is a floating pier around the spillway riser with a staff gauge to show the water level. The main spillway enclosure is equipped with a debris grill. According to calculations included in Appendix A- Doc 05 the full flow capacity of the spillway tunnel at 8 feet of head is approximately 420 cfs. Photo Numbers 12-15 in Appendix B show the main spillway and Photos Number 20-22 show the main spillway outlet and outfall conditions.

The dam also has an emergency spillway in the form of an open channel excavated through the bedrock of the hillside northwest of the dam. The emergency spillway is trapezoidal in cross section and acts as a broad crested weir. According to the available documents (see Appendix A- Doc 02) the width of the spillway is 30 ft. However, the onsite assessment report shows the bottom width of the spillway to be 34 ft and the top width to be 58 ft (see Appendix C- Doc 01). Currently the emergency spillway is also the access roadway to the crest of the dam. The invert of the spillway is at 868 ft. According to the hydrologic and hydraulic review (Appendix A- Doc 05) calculation sheets, the emergency spillway discharge at the dam crest elevation of 875 ft is 1,930 cfs. When this value is combined with 333 cfs discharge from the principal spillway, the result is a total discharge value of approximately 2,300 cfs. Photo Numbers 11 and 13 in Appendix B show the emergency spillway.

2.5 CRITICAL INFRASTRUCTURE WITHIN FIVE MILES DOWN GRADIENT

The John Amos Monitoring and Emergency Action Plan (EAP) dated January 24, 2008 evaluates the hazard conditions downstream of the dam and presents an evacuation plan (see Appendix A-Doc 02, "Monitoring & Emergency Action Plan & maintenance Plan.pdf"). The EAP report presents a comprehensive assessment of critical infrastructures downstream of the dam. According to Appendix D of the EAP report, a number of residences and businesses have been identified to be within the flood inundation area. According to the emergency action and evacuation plan the residences of the flood inundation area will be notified to evacuate by plan personnel if failure of the dam is evident or has already occurred. The flood inundation map in Appendix E of the EAP report shows I-64 to be impacted by the flood from Fly Ash Dam failure and Appendix F of that report shows the planned road blocks in case I-64 is flooded.

Other data reviewed for this assessment included two dam break analyses: one dated November 1993 and the second dated October 27, 1994. Both analyses conclude that the dam can withstand the PMP without overtopping and no damage to homes or other structures is expected due to releases from the dam. The 1993 analysis includes an assessment of potential damages in the event of a postulated failure of the dam. The assessment concluded that in the event of a dam failure the I-64 roadway embankment would be overtopped by 22 feet causing severe erosion. Down gradient from I-64, the railroad embankments would be overtopped by 18 feet, also causing severe erosion. Below the railroad embankment, flood water would spread out as it flows over U.S. Route 35 and into the Kanawha River. Flood level in the river will rise to the top of the banks but will not inundate the floodplain. The bank full condition extends 5 miles along the Kanawha River. The Winfield Lock and Dam have the spillway capacity to pass the resulting flood wave without being overtopped.

DRAFT

3.0 SUMMARY OF RELEVANT REPORTS, PERMITS AND INCIDENTS

3.1 SUMMARY OF REPORTS ON THE SAFETY OF THE MANAGEMENT UNIT

In response to a Freedom of Information request, the State of West Virginia provided an extensive package of design information, performance monitoring data and past inspection documents for the John Amos Fly Ash Pond dam. The data were provided in 264 electronic files. A complete list of data files including general data, the Fly Ash Pond and as well as the Bottom Ash Pond data and reports is provided in Appendix A-Doc 03 "Review_Document_List". A few of the most recent reports directly relevant to the safety of the Fly Ash Dam are summarized below.

In the wake of the failure of the Kingston Power Plant in Tennessee, West Virginia Dam Safety issued an order to verify that the "Ash Dams" in West Virginia were visually inspected and evaluated to insure that they were in good visual condition and meet or exceed the requirements of the Dam Safety Act and Regulations and standard dam safety engineering fundamentals. To meet part of the requirements of that order, West Virginia Department of Environmental Protection (DEP) Dam Safety Staff inspected the John Amos Fly Ash Dam on February 10, 2009 and summarized their finding in a March 16, 2009 inspection report (see Appendix A-Doc 09, "2009 DEP Fly Ash Dam Inspection report.pdf"). This report concluded that "In general, the John Amos Fly Ash Dam appears to be working well and in good condition. No major problems or maintenance items were observed during this inspection".

A third party consultant (Stantec Consulting Services Inc.) submitted the 2009 Dam and Dike Inspection report to American Electric Power (AEP) dated May 15, 2009 (see Appendix A-Doc 07, "2009 Dam and Dike Inspection Report - John Amos"). This report was prepared based on field reconnaissance and existing data and provided observations and recommendations for both the Fly Ash and Bottom Ash facilities. According to this report the overall condition of the Fly Ash Complex was good and "No excessive or critical items were observed". Stantec reported several minor issues such as minor erosion spots and made several maintenance recommendations.

The Geotechnical Engineering Department of AEP prepared the March 2009 John E. Amos Plant Fly Ash Dam Stability Analysis Study report (see Appendix A - Doc 08). The report concluded that the Fly Ash dam has stability safety factors at or above minimum recommended values.

3.2 SUMMARY OF LOCAL, STATE AND FEDERAL ENVIRONMENTAL PERMITS

The facility is under regulation by the West Virginia Division of Water and Wastewater Management, Dam Safety regulations. West Virginia inspects the dam on a biannual basis. However, in response to the Kingston Dam failure, the State conducted a supplemental review. The main dam and appurtenances are inspected by plant personnel on a quarterly basis and by qualified engineers annually.

In January 25, 2008 The Environmental Enforcement Division of the West Virginia DEP conditionally approved the Monitoring and Emergency Action Plan prepared by AEP.

The Dam Inspection Report dated February 10, 2009, reported that an application for raising the dam by 27 feet had been submitted by the owner. The report also indicated that WV – Dam Safety is in the final stages of issuing the Certificate of Approval to modify/raise the John Amos Fly Ash Pond dam.

3.3 SUMMARY OF SPILL/RELEASE INCIDENTS

Data reviewed by Dewberry did not indicate any spills, unpermitted release, or other performance related problems with the dam over the last 10 years.

4.0 SUMMARY OF HISTORY OF CONSTRUCTION AND OPERATION

4.1 SUMMARY OF CONSTRUCTION HISTORY

4.1.1 Original Construction

The John Amos Fly Ash Pond dam was constructed in three phases in the early to mid-1970s. The initial phase constructed the dam to a crest elevation of 810 feet. Stages 2 and 3 raised the crest elevation to 845 feet and 875 feet respectively. The Fly Ash Pond was commissioned in 1971. The design data, calculations and construction drawings are available and were reviewed.

4.1.2 Significant Changes/Modifications in Design since Original Construction

The initial phase constructed the dam to a crest elevation of 810 feet. Stages 2 and 3 raised the crest elevation to 845 feet and 875 feet respectively. Expansion was accomplished by extending the upstream slope upward, widening the crest and enlarging the downstream slope (i.e., making the dam higher and thicker). The construction sequence avoided constructing new foundations in the impounded area.

4.1.3 Significant Repairs/Rehabilitation since Original Construction

No information was provided regarding major repairs or rehabilitation. No evidence of prior releases, failures, or patchwork was observed on the earthen embankment during the visual site assessment and no documents or statements were provided to the dam assessor that indicates prior releases, failures, or have occurred. The only exception was repair of some leaks from joints and cracks in the concrete-lined portion of the spillway outlet tunnel that was observed in 1990 and was successfully fixed in 1991 (see Section 9.2.2 under seepage for more explanation).

The owner's 2008 Inspection Report identified an erosion gully on the downstream slope, close to the right groin ditch below the 716 bench. The erosion feature was not cited in the WV-Dam Safety inspection report. The owner's 2008 inspection report discussed a similar erosion feature on the upstream slope that was identified in the 2007 inspection report. The erosion had been repaired and the upstream slope was in good condition. This suggests that the erosion gully observed in 2008 was repaired prior to the WV Dam 2009 inspection.

4.2 SUMMARY OF OPERATIONAL HISTORY

4.2.1 Original Operational Procedures

The dam was designed and operated for fly ash sedimentation and control. The pond receives plant process waste water slurrying coal combustion waste and stormwater runoff from the drainage area on the perimeter of the pond. The inflow water is treated through gravity settling and deposition, and treated process water and stormwater runoff are discharged through an unregulated overflow outlet structure.

In their 1980 "Fly Ash Retention Dam Operation and Maintenance Manual", the original dam designers, Acres American Incorporated provided the dam owners with operation and maintenance procedures and details (see Appendix A- Doc 14). Items covered included service spillway, instrumentation and monitoring, discharge rate measurements and water quality monitoring.

4.2.2 Significant Changes in Operational Procedures since Original Startup

No documents are provided to indicate any operational procedures have changed.

4.2.3 Current Operational Procedures

Original operational procedures are in effect. Appendix A-Doc 02 includes a Maintenance Plan but it is not clear from what date this plan has been in effect.

4.2.4 Other Notable Events since Original Startup

No additional information was provided.

5.0 FIELD OBSERVATIONS

5.1 PROJECT OVERVIEW AND SIGNIFICANT FINDINGS

Dewberry personnel James Filson, P.E. and Joseph P. Klein, III, P.E. performed a site visit on Tuesday, September 8th, 2009, in company with the participants.

The site visit began at 09:00 AM. The weather was warm and cloudy. Photographs were taken of conditions observed. Please refer to photographs in Appendix B and the Dam Inspection Checklist in Appendix C: Doc 01. Selected photographs are included here for ease of visual reference. All pictures were taken by Dewberry personnel during the site visit.

The overall assessment of the dam was that it was in satisfactory condition and no significant findings were noted.

5.2 EARTH EMBANKMENT DAM

5.2.1 Crest

The dam crest had no signs of any depressions, tension cracks or other indications of settlement or shear failure, and appeared to be in satisfactory condition. Previous inspection reports reviewed by Dewberry indicated minor surface cracking in the clays on the upstream face. The cracking was described as "the result of desiccation". Photographs included with the reviewed report supported the desiccation description. The data did not indicate cracking along the crest or downstream face of the dam. Figure 5.2.1-1 shows the conditions of the dam crest.

Figure 5.2.1-1. Photo Showing the Dam Crest.

5.2.2 Upstream Slope

The upstream slope is protected with grass. There were no observed scarps, sloughs, bulging, cracks or scraps or depressions or other indications of slope instability or signs of erosion. Figures 15 and 16 in Appendix B show the upstream slope. Figure 5.2.2-1 depicts part of the upstream slope of the dam embankment.

Figure 5.2.2-1. Photo Showing the Upstream Slope.

5.2.3 Downstream Slope and Toe

The downstream slope of the entire dam is covered with large rock boulders (rip-rap). Also, the downstream slope has a bench that has a sloped surface with the elevation carrying from 713 ft to 716 ft (referred to as "716 Bench" in some dam documents). There were no observed scarps, sloughs, depressions or other indications of slope instability or signs of erosion or uncontrolled seepage. Isolated locations of weathered rip-rap on downstream face were observed. Seepage was observed from underdrains as well as isolated points on the embankment slope. The seepage on the embankment slope was believed to be due to percolation of local runoff from the dam bench and not from the water stored in the reservoir. This was evident from the local drainage rill disappearing up gradient from the seepage spot. No seepage was observed over a widespread area or the downstream foundation area and there was no water against the downstream toe. Figures 1-3, 5-7, 10 and 17-19 in Appendix B depict various views of the downstream slope. Figure 5.2.3-1 shows the downstream slope from North Abutment of the dam. Figure 5.2.3-2 shows the Seepage Collection Basin in the downstream embankment toe receiving seepage water from a drain pipe.

The photographs included in the WV- Dam report indicated deterioration was limited to moderate to severe weathering of approximately 10 percent of the sandstone and shale rip-rap on the downstream face of the dam. Weathering is reported to be most apparent below the bench at approximately Elevation 716 ft in the downstream slope. During the 9/8/09 dam inspection, Dewberry personnel observed weathered rip-rap on isolated spots on the downstream slope (see Figure 5.2.3-3).

Figure 5.2.3-1. Downstream Slope from North Abutment,

Figure 5.2.3-2. Seepage Collection Basin, Downstream Embankment.

Figure 5.2.3-3. Weathered Rip-Rap at Downstream Slope.

The 2008 Inspection Report prepared for the owner identified an erosion gully on the downstream slope, close to the right groin ditch below the 716 bench. The erosion feature was not cited in the WV- - Dam Safety inspection report.

No other significant deterioration was indicated in the data reviewed.

5.2.4 Abutments and Groin Areas

Erosion or uncontrolled seepage was not observed along either groin. The abutments and groin areas appeared to be in excellent condition. The groin slopes are protected with rip-rap. Figure 5.2.4-1 shows the Abutment/Groin at the "716 Bench".

Figure 5.2.4-1. Downstream North Abutment/Groin at "716 Bench".

5.3 OUTLET STRUCTURES

5.3.1 Overflow Structure

The dam has a concrete decant riser 8 feet in diameter with invert elevation at 799.3 ft and a tunnel/pipe 8 ft in diameter located in original ground off the northwest end of the dam, which discharges through the hillside into an adjacent tributary to Scary Creek. There is a floating pier around the spillway riser with a staff gauge to show the water level. The main spillway enclosure is equipped with a debris grill. Photo

DRAFT

Numbers 12-15 in Appendix B show the main spillway and Photos Number 20-22 show the main spillway outlet and outfall conditions. According to calculations included in Appendix A- Doc 05 the full flow capacity of the spillway tunnel at 8 feet of head is approximately 420 cfs.

The primary overflow structure was observed to be working properly, discharging flow from the pond, and visually appeared to be in satisfactory condition. There was no sign of clogging of the spillway and the water exiting the outlet was flowing clear. Figure 5.3.1-1 shows the main outlet structure and the floating pier around it within the reservoir.

Figure 5.3.1-1. Outlet Structure (Principal Spillway) and the Floating Pier Around it on the Northwest End of Dam.

5.3.2 Outlet Conduit

The outlet conduit appeared to be in good shape and operating normally with no sign of clogging and the water exiting the outlet was flowing clear. Figure 5.3.2-1 shows the water discharging from the main spillway tunnel outfall.


Figure 5.3.2-1. Main Spillway Tunnel Outfall.

5.3.3 Emergency Spillway

The dam also has an emergency spillway in the form of an open channel excavated through the bedrock of the hillside northwest of the dam. The emergency spillway is trapezoidal in cross section and acts as a broad crested weir. According to the available documents (see Appendix A- Doc 02) the width of the spillway is 30 ft. However, Dewberry staff measured the bottom width of the spillway to be 34 ft and the top width to be 58 ft (see Appendix C Doc 01). Currently the emergency spillway is also the access roadway to the crest of the dam. The invert of the spillway is at 868 ft. According to the hydrologic and hydraulic review (Appendix A- Doc 05) calculation sheets the emergency spillway discharge at the dam crest elevation of 875 ft is 1,930 cfs which when combined with 333 cfs discharge from the principal spillway results in a total discharge value of approximately 2,300 cfs. Photo Numbers 11 and 13 in Appendix B show the emergency spillway. Figure 5.3.3-1 shows the trapezoidal emergency spillway excavated through the hillside northwest of the dam.

The emergency spillway appeared to be in good condition with no sign of clogging.

Figure 5.3.3-1. Emergency Spillway at the North end of Dam.

5.3.4 Low Level Outlet

No low level outlet is present.

6.0 HYDROLOGIC/HYDRAULIC SAFETY

6.1 SUPPORTING TECHNICAL DOCUMENTATION

6.1.1 Floods of Record

No documentation has been provided about the floods of record.

6.1.2 Inflow Design Flood

The John Amos Fly Ash Dam has been classified by the West Virginia Dam Safety as a Class "1" which is a structure which has "High Hazard" potential. According to rule 7.1.a.2.A in Dam Safety rules: "The Class 1 dams shall be designed for the probable maximum precipitation of six (6) hours in duration. The design precipitation for a Class 1 dam may be reduced based on Risk Assessment (subdivision 3.5.d.), but in no case to less than seventy percent (70%) of the probable maximum precipitation." The Probable Maximum Precipitation (PMP) is defined by American Meteorological Society as the theoretically greatest depth of precipitation for a given duration that is physically possible over a particular drainage area at a certain time of year. The National Weather Service (NWS) further states that in consideration of our limited knowledge of the complicated processes and interrelationships in storms, PMP values are identified as estimates. The NWS has published application procedures that can be used with PMP estimates to develop spatial and temporal characteristics of a Probable Maximum Storm (PMS). A PMS thus developed can be used with a precipitation-runoff simulation model to calculate a probable maximum flood (PMF) hydrograph.

The Fly Ash Dam Stage 2 design document for Hydrologic and Hydraulic calculations by Acres American Incorporated Engineering and Woodward-Clyde Consultants is included as Appendix A-Doc 4. This is a 1975 report with hand-written calculation sheets for various inflows (including the 6-hr PMP and 24-hr PMP generated runoff) and pipe sizes analyzed. It is not clear from this report which design was adopted for the construction of the Fly Ash Dam but the WV – Dam Safety report (see Appendix A-Doc 05, "H&H Review") indicates the dam was designed to safely handle the full probable maximum precipitation design storm without overtopping, assuming the reservoir pool is at the maximum normal pool elevation prior to the storm. The report indicates that the operational pool elevation at the time of the inspection provided 15 to 17 feet of freeboard. The report estimates that a probable maximum precipitation event would raise the reservoir approximately 4.5 feet with no outflow, leaving a freeboard of at least 10.5 feet.

6.1.3 Spillway Rating

The original Hydrologic and Hydraulic calculations report (Appendix A- Doc 4) includes spillway rating curve values listed in the inflow files for various hydrological simulations. These values were hand calculated as is evident from the report. Also, the hydrologic and hydraulic review (Appendix A- Doc 05) and the Dam Break Analysis report (see Appendix A- Doc 06) include calculation sheets with the spillway rating tables for the Principal Spillway and Emergency Spillway and the Total Spillway rating curve. As presented above, the hydrologic and hydraulic review (Appendix A- Doc 05) calculation sheets result in a total discharge value of approximately 2,300 cfs.

6.1.4 Downstream Flood Analysis

The WV – Dam Safety inspection report does not address critical facilities down gradient of the unit.

Data reviewed for this assessment included two dam break analyses: one dated November 1993 (see Appendix A-Doc 06) and the second dated October 27, 1994 (see Appendix A-Doc 05). Both analyses conclude that the dam can withstand the PMP without overtopping and no damage to homes or other structures is expected due to releases from the dam.

The 1993 analysis includes an assessment of potential damages in the event of a postulated failure of the dam. The assessment concluded that in the event of a dam failure the I-64 roadway embankment would be overtopped by 22 feet causing severe erosion. Down gradient from I-64, the railroad embankments would be overtopped by 18 feet, also causing severe erosion. Below the railroad embankment, flood water would spread out as it flows over U.S. Route 35 and into the Kanawha River. Flood level in the river will rise to the top of the banks but will not inundate the floodplain. The bank full condition extends 5 miles along the Kanawha River. The Winfield Lock and Dam have the spillway capacity to pass the resulting flood wave without being overtopped.

As it has been over 15 years since the analysis of down gradient facilities, an updated survey would be beneficial. The John Amos Monitoring and Emergency Action Plan dated January 24, 2008 (see Appendix A-Doc 02) presents a comprehensive assessment of critical infrastructures downstream of the dam. According to Appendix D of this document, a number of residences and businesses have also been identified to be within the flood inundation area. However, the inundation area is apparently based on the 1993 analysis. There was no information provided that presented or referenced a new analysis, including an updated survey of critical facilities. Also, several changes in the assumption and methodology would be required to assure any new dam break analysis would be consistent with current engineering practice:

1. The 1993 Dam Break Analysis was performed using the National Weather Service program "DAMBRK". Since 1998, DAMBRK has been replaced by a newer National Weather Service Model "FLDWAV". The new dam break analysis needs to use FLDWAY or other current appropriate software such as the U.S. Army Corps of Engineers HEC-HMS or HEC-RAS.

2. The new analysis needs to consider updated survey data reflecting the current level of settled fly ash and the available storage capacity.

3. The new analysis needs to consider any changes or additions in critical facilities and residential structures downstream of the dam that may be impacted by a dam failure flood.

4. The new analysis needs to consider the runoff from area downstream of the dam in assessing the difference between dam break and no dam break scenarios and evaluate the hazards from a dam failure. The original 1993 analysis had ignored this factor.

6.2 ADEQUACY OF SUPPORTING TECHNICAL DOCUMENTATION

Supporting technical documentation reviewed by Dewberry is adequate.

6.3 ASSESSMENT OF HYDROLOGIC/HYDRAULIC SAFETY

Based on the calculations provided in the 1994 hydrologic and hydraulic review (Appendix A- Doc 05) the Fly Ash Dam can safely pass the PMF with a freeboard safety of 4.5 feet. Hence, the dam failure by overtopping seems to be improbable.

7.0 STRUCTURAL STABILITY

7.1 SUPPORTING TECHNICAL DOCUMENTATION

7.1.1 Stability Analyses and Load Cases Analyzed

The 2009 Geotechnical Report for John Amos Fly Ash Dam summarizes a recent stability analysis following the general guidelines of the US Army Corps of Engineers in slope stability engineering manual (see Appendix A- Doc 08, "2009 Geotechnical Report - John E. Amos"). This investigation used information from 12 boreholes drilled in the dam area in late 2007 and early 2008. Based on the results from this analyses it was concluded that the Fly Ash Dam has stability safety factors at or above minimum recommended values.

An older stability analysis relied on the results of extensive computer generated slope stability analyses. The analyses included seismic loading on the dam and were conducted for both total and effective soil strength conditions. At each of the three stages of embankment construction a stability analysis was performed and documented.

The WV –Dam Safety inspection report does not include a review of design loads relative to potential credible loading conditions. The available design information includes the original structural calculations for the spillway structure and foundation and spillway tunnel and terminal structures as well as the reservoir loading schedule. As the loading rate can impact soil strength parameters, actual loading rates should be compared to the design values.

A comparison of the geologic data and the design slope stability analyses indicates that mylonite and/or clay seams in the rock beneath the dam represent potential sliding failure surfaces.

7.1.2 Design Properties and Parameters of Materials

The only documentation identified in the review data that may refer to the design parameters used for the original dam design is a plan showing the downstream slope stability flow net analysis (see Appendix A-Doc 12). This plan is for the condition of Top of Dam Elevation of 830 ft. Stage 1 was constructed in the early 1970s to a crest elevation of 810 ft. Stages 2 and 3 were constructed in the mid to late 1970s as a continuous project. Stages 2 and 3 were constructed to elevations of 845 and 875 respectively. Assuming the plan for the Top of Dam at 830 ft more or less reflects the Stage 1 conditions the design parameters may be taken from this plan. The density values listed in the parameter tables for the downstream slope range from 125 to 150 PCF. Angle of shearing resistance under total stress analysis range is 0° to 32° for various zones in downstream slope. The underdrain shear stress under total stress analysis ranges from 0 to 2500 PSF, except for the sandstone zone which is identified with a value of 9,999 PSF.

For Stage 3 dam expansion which took the Top of Dam Elevation to 875 ft (the current conditions), parameters may be taken from the downstream stability analysis plan (see Appendix A- Doc 13). The density values listed in the parameter tables range from 125 to 150 PCF. The range for the Angle of shearing resistance under Total Stress Analysis is from 0° to 32°.

In the new stability model (Appendix A – Doc 08), a peak ground acceleration of 0.08g was used for seismic loading.

7.1.3 Uplift and/or Phreatic Surface Assumptions

A flow net analysis for Stage 3 dam expansion (current conditions) is included in Appendix A- Doc 13. However, no uplift calculations were provided. Based on the stability model (Appendix A – Doc 08), the phreatic surface and pore water pressure distribution in the dam and foundation were obtained from a series of transient seepage analyses, consistent with the piezometer readings. Negative pore pressures were set to zero in the calculations.

7.1.4 Factors of Safety and Base Stresses

In the new stability model (Appendix A – Doc 08) Safety Factors were computed as listed in Table 7.1.4

Table 7.1.4: Factors of Safety John Amos Fly Ash Dam

Location/Loading Condition	Required Safety Factor (Army Corps)	Computed Safety Factor
Downstream Static (Drained)	1.5	1.5
Downstream Seismic (Drained)	1.2	1.2
Upstream Static (Drained)	1.5	1.8
Upstream Static (Undrained)	1.5	3.5
Upstream Seismic	2.2	2.4

Based on the results summarized in Table 7.1.4, the Fly Ash Dam was found to have stability safety factors at or above the minimum recommended values.

Embankment internal stress data for the final crest elevation of 875 feet were provided in a series of summary drawings showing calculated principal stresses on longitudinal and cross sections of the dam.

The magnitude and orientation of the principal stresses are reasonable for the dam configuration analyzed.

The critical downstream failure surface for each cross-section reported passed through a mylonite seam or clay layer between two rock strata (see Section 7.1.6).

7.1.5 Liquefaction Potential

The documentation reviewed by Dewberry did not include an evaluation of liquefaction potential. Foundation soil conditions do not appear susceptible to liquefaction.

7.1.6 Critical Geological Conditions and Seismicity

In the new stability model (Appendix A – Doc 08), a peak ground acceleration of 0.08g was used for seismic loading conditions derived from the 2008 USGS National Seismic Hazard Map for 2% probability of exceedance in 50 years.

The original design data reviewed indicates that seismicity was considered in the design. Based on the 1969 Seismic Risk Map of the United States, the site was located in Zone 1, a zone of minor damage. The site was described as approximately 40 miles from a Zone 2 boundary and 175 miles from a Zone 3 boundary.

Three alternative design earthquakes were reviewed as part of the design. The alternatives are shown in Table 7.1.6.

Table 7.1.6: Alternative Design Earthquakes

Earthquake	Distance from Site	Site Acceleration
Distant strong earthquake; e.g., New Madrid fault location	400 miles	<0.010 g
Moderate to strong Appalachian earthquake	50 miles	0.020 g
Local moderate earthquake	At site	0.075 g

The data reviewed indicated that the local moderate earthquake was selected as the design alternative.

The current Seismic Risk Map of the United States was also reviewed using the U.S. Geologic Survey web site. The 2% probability of exceedance in 50 years ground acceleration for rock at the site is 0.05g to 0.1g. Based on the above, the seismic design criteria are appropriate for this dam.

The John Amos dam site is located in the Allegheny plateau physiographic province. Bedrock belongs to the Pennsylvanian System, mainly to the Monongahela and Conemaugh Series. The sedimentary rock includes randomly stratified sandstone, siltstone, clay shale and cemented shale. Strata are variable in both thickness and lateral continuity.

The data reviewed indicated that rock at the site is generally sub-horizontally bedded with a maximum dip of 10 degrees. Strata in both abutments indicated a geologic history of sliding along bedding planes. The sliding has resulted on the formation of mylonite seams, varying in thickness from less than ¼ inch to approximately 4 inches. Overburden thickness generally varied from 1 to 2 feet on the abutment slopes to 15 to 20 feet in the center of the valley. Overburden consisted of colluvial and weathered local bedrock.

7.2 ADEQUACY OF SUPPORTING TECHNICAL DOCUMENTATION

Structural stability documentation is adequate.

7.3 ASSESSMENT OF STRUCTURAL STABILITY

The WV –Dam Safety inspection report does not include a review of design loads relative to potential credible loading conditions. The available design information includes a reservoir loading schedule. As the loading rate can impact soil strength parameters, actual loading rates should be compared to the design values.

Overall, the structural stability of the embankment appears to be satisfactory based on the following observations during the September 8th field visit and dam inspection by Dewberry, available recent dam inspection reports and the 2009 Geotechnical Report and the 2009 Dam and Dike Inspection Report:

- The internal drains are flowing clear and at a consistent rate which is a good indication internal soil piping is not occurring;
- There were no indications of scarps, sloughs, depressions or bulging anywhere along the dam;
- Boils, sinks or uncontrolled seepage was not observed along the slopes, groins or toe;
- The crest appeared free of depressions and no significant vertical or horizontal alignment variations were observed; and
- The computed factors of safety comply with accepted criteria.

The review conducted for this assessment indicates that settlements and horizontal movement of the crest and downstream slope appear to be consistent with design parameters. Sliding along a mylonite or clay seam does not appear to be occurring. A review of the original design earthquake design parameters indicates they are compliant with current values.

8.0 ADEQUACY OF MAINTENANCE AND METHODS OF OPERATION

8.1 OPERATIONAL PROCEDURES

The facility is operated for storage of fly ash deposits. Coal combustion process waste water and limited stormwater runoff from within the reservoir exterior perimeters discharge into the reservoir, inflow water is treated through gravity settling and deposition, and treated process water and stormwater runoff is discharged through an unregulated overflow outlet structure.

8.2 MAINTENANCE OF THE DAM AND PROJECT FACILITIES

The 1980 Operation and Maintenance Plan were organized in a manual format (see Appendix A- Doc 14). Maintenance procedures for the Fly Ash Dam are in effect as a comprehensive Maintenance Plan and Schedule (See Appendix A- Doc 02). The maintenance plan includes:

- Monthly dam and dike inspection;
- Annual engineering inspection;
- Annual embankment mowing;
- Annual repair of erosion gullies;
- Removal of unwanted trees/brush and obstructive vegetation from rocks;
- Monthly maintenance of the overflow structure;
- Annual clearing of outlet channel trees/brush.

8.3 ASSESSMENT OF MAINTENANCE AND METHODS OF OPERATION

8.3.1 Adequacy of Operational Procedures

Based on the assessments of this report operation procedures seem to be adequate.

8.3.2 Adequacy of Maintenance

Various dam inspections reports including the WV- Dam Safety report of March 16, 2009, Dam Inspection Checklist of September 8th, 2009 by Dewberry (see Appendix C- Doc 01, "Dam Inspection Checklist") as well as the 2009 Dam and Dike Inspection Report by Stantec Consulting Services Inc. reported no major maintenance issues. The latter report includes several maintenance recommendations but none of them are considered critical or imminent. This indicates that the maintenance plan is probably followed in practice and adequate maintenance is provided for the dam and the project facilities.

Although the maintenance program is adequate, several recommendations have been made to improve the maintenance and insure trouble-free operation:

- The WV – Dam Safety report (see Appendix A-Doc 09) indicated that AEP engineers inspected the tunnel and pipe in October 2008. The AEP inspection report concluded the tunnel and pipe were in good condition, but included a recommendation that the spillway tunnel be re-lined or abandoned. The WV-Dam Safety report indicated that the tunnel will be abandoned and fully grouted as part of the planned dam expansion project;

- Continue monitoring existing groundwater and survey monuments;

- Repair access stairs and walkways to monitoring points if the dam in not expanded;

- Evaluate the cause of movement restrictions of floating docks around the decant riser and repair wooden walkway to overflow structure;

- Install a staff gage at the overflow structure for monitoring purposes;

- Evaluate the valve at the overflow structure to assure it is operational in case flow out of pond needs to be stopped;

- Monitor, address or otherwise repair minor erosion areas and erosion gullies, partially saturated areas and isolated seepage spots.

9.0 ADEQUACY OF SURVEILLANCE AND MONITORING PROGRAM

9.1 SURVEILLANCE PROCEDURES

Quarterly Inspections:
Quarterly dam and dike inspections are conducted by the AEP. Data from the dam monitoring system is collected monthly. inspection reports are submitted to the WV DEP.

Annual Inspections:
Annual dam engineering inspections are conducted by the AEP engineers and inspection reports are submitted to the WV DEP. A third party consultant (Stantec Consulting Services Inc.) submitted the 2009 Dam and Dike Inspection report to American Electric Power (AEP) dated May 15, 2009 (see Appendix A- Doc 07, "2009 Dam and Dike Inspection Report - John Amos").

Special Inspections:
In the wake of the failure of the Kingston Power Plant in Tennessee, West Virginia Dam Safety issued an order to verify that the "Ash Dams" in West Virginia were visually inspected and evaluated to insure that they were in good visual condition and meet or exceed the requirements of the Dam Safety Act and Regulations and standard dam safety engineering fundamentals. To meet part of the requirements of that order, West Virginia Department of Environmental Protection (DEP) Dam Safety Staff inspected the Fly Ash Dam on February 10, 2009 and summarized their findings in a March 16, 2009 inspection report (see Appendix A-Doc 09,"2009 DEP Fly Ash Dam Inspection report.pdf"). This report concluded that "In general, the John Amos Fly Ash Dam appears to be working well and in good condition. No major problems or maintenance items were observed during this inspection". It was noted in the report that an application for raising the dam by 27 feet has been submitted by the owner. The report also indicated that WV – Dam Safety is in the final stages of issuing the Certificate of Approval to modify/raise the John Amos Fly Ash Pond dam.

9.2 INSTRUMENTATION MONITORING

The John Amos Fly Ash Dam is well equipped with monitoring instrumentation.

9.2.1 Instrumentation Plan

Deformation monitoring data are available for 23 points on the dam: 15 points along the crest and eight points on the downstream face of the dam. The dam configuration and location of monitoring points are shown on Figure 9.2.1-1.

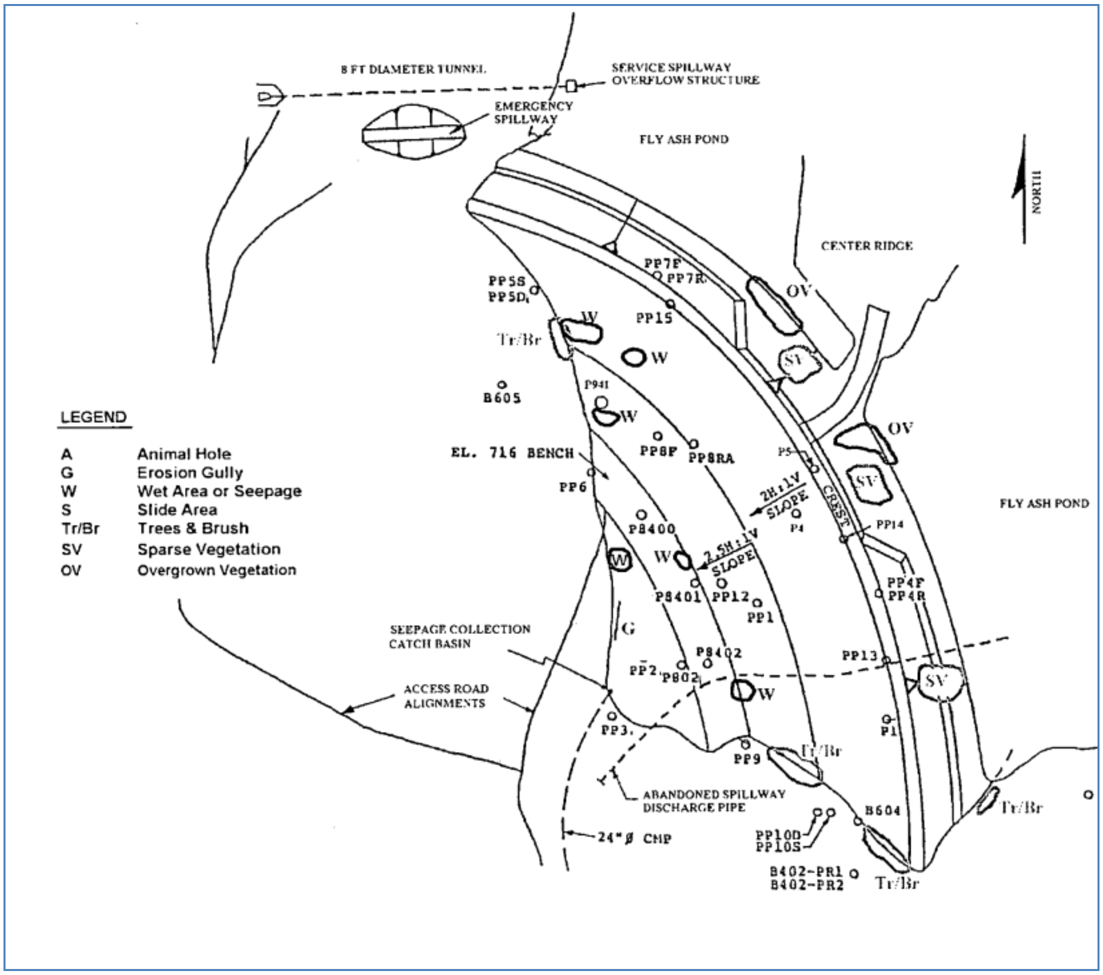

Figure 9.2.1-1- John Amos Fly Ash Pond Dam Configuration and Monitoring points.

Deformation measurements are performed using dual frequency Global Positioning System (GPS) Methods. Vertical deformation is measured using GPS Methods. Deformation Monuments are shown in Figure 9.2.1-2.

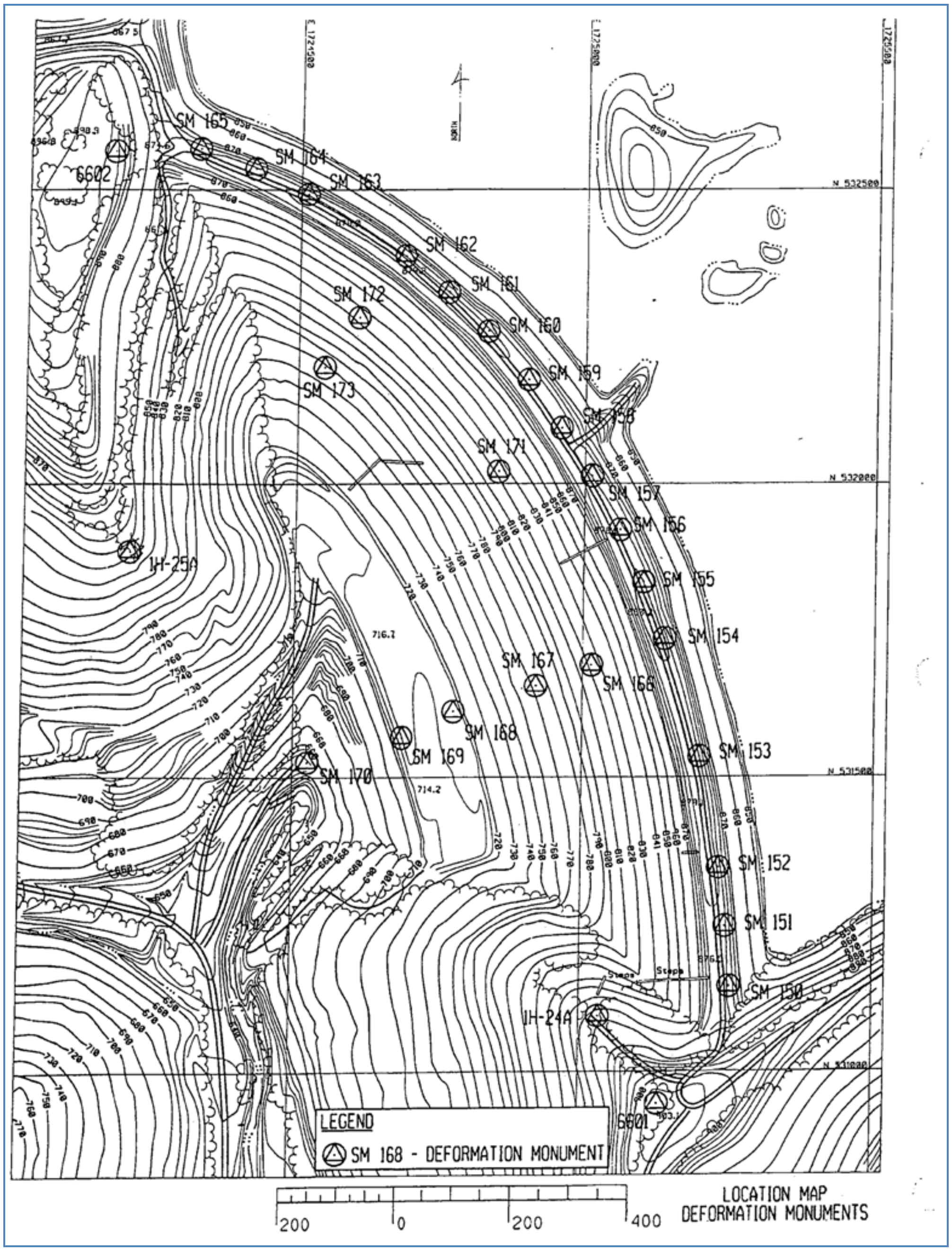

Figure 9.2.1-2. Location Map of Deformation Monuments.

Piezometer readings are obtained monthly for eleven piezometers and quarterly for the remaining 26 by John Amos personnel using a battery powered Sinco water level indicator. Figure 9.2.1-3 is the location map of the piezometers.

The reviewed data indicate the presence of a network of foundation drains designed to provide a drainage curtain along the longitudinal axis of the dam. Individual pressure relief drains consist of holes drilled 40 to 120 feet into the rock foundation materials. After drilling, each drain was filled to within 4 feet of the rock surface with granular backfill. The upper 4-feet of the drain were cased with a 3-inch diameter perforated plastic well screen extending 1-foot above the rock surface. Water from the drains discharged into a granular drainage blanket for discharge outside the dam.

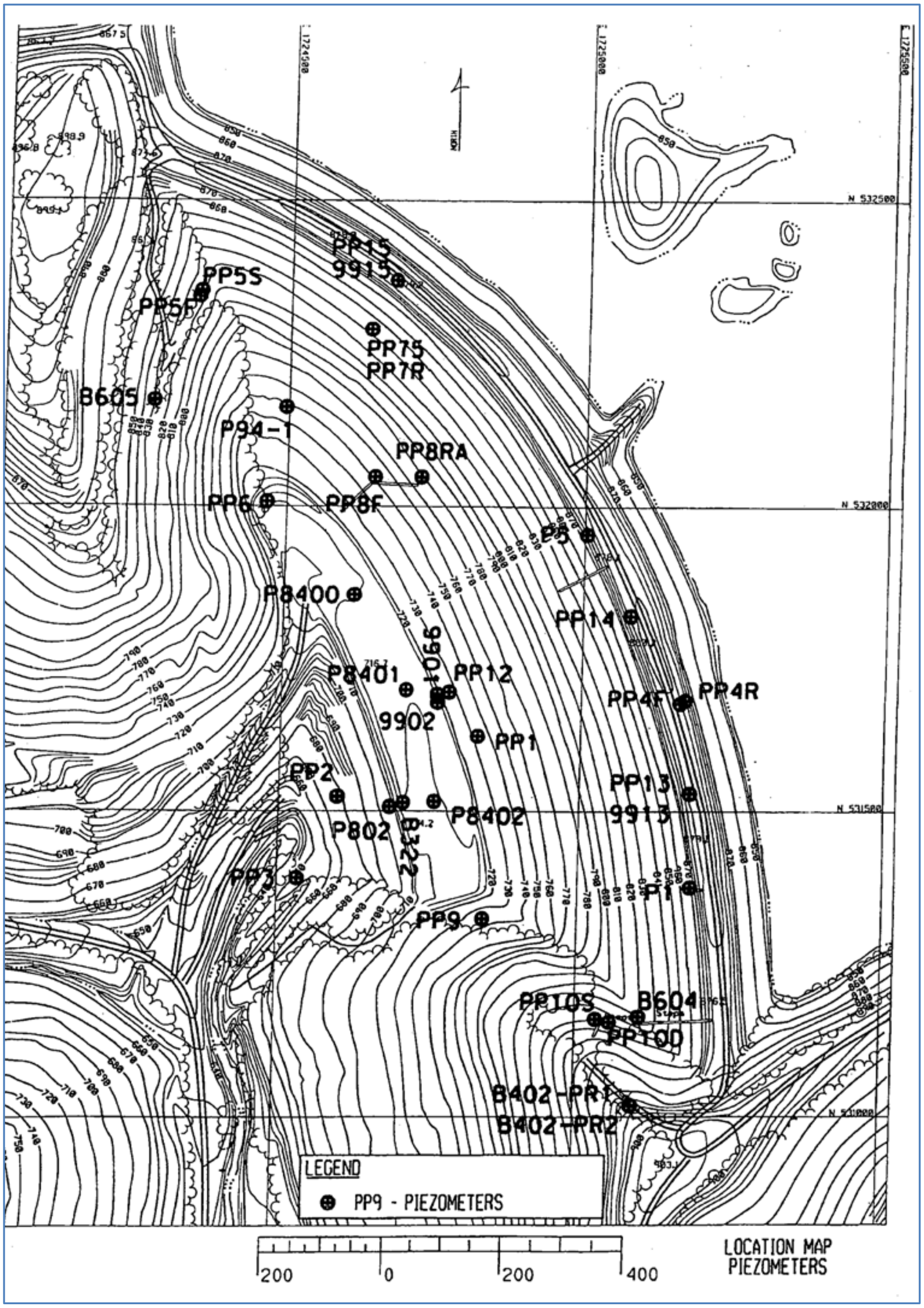

Figure 9.2.1-3. Location Map of Piezometers.

The data reviewed identified two long-term seepage areas on the downstream face of the dam. One of the seeps is near the left groin at about elevation 750. This seepage area is referenced as near piezometer P94-1. The second seep is near the right groin at about elevation 740. Both seeps are monitored monthly for flow rates and clarity.

9.2.2 Instrumentation Monitoring Results

Settlement: The documents reviewed included a graphical summary of estimated time-settlement calculation results. The curve was sealed by a WV licensed engineer. The plot indicates an estimated maximum post construction settlement of approximately 1.6 percent of the dam height. The estimated rate of settlement becomes asymptotic approximately 25 years after completion of construction. The estimated load settlement data are summarized in Table 9.2.2-1.

Table 9.2.2-1. Summary Load-settlement Calculations.

Estimated Settlement (% of Dam Height)	Time (Years)
0.2	1.0
0.4	1.5
0.6	2.5
0.8	4.0
1.0	5.5
1.2	7.5
1.4	11.0
1.6	25.0

Two sets of settlement data were reviewed. The first set of data includes settlement measurements of the Stage 1 dam for a period of 36 months after completion. The data indicate settlements near the center and north end of the dam very closely match the estimated values. Settlements at the south end of the dam are slightly higher and are occurring at a slower rate than estimated. The document presenting this data is dated February 1975.

The second set of data includes measurement of the completed Stage 3 dam for the period of January 1978 to January 2009. The data indicate that the average settlement on the center of the dam is slightly less than originally estimated, and that the rate of settlement has been nearly zero for the last five to ten years. Settlements of the northern portion of the dam very closely match the estimated values, and the rate of settlement has been near zero since 1988 to 1993. Settlements of the southern portion of the dam are slightly higher than the estimated values, and settlements are continuing, although at a relatively slow rate of approximately ¼ inch per year. Both sets of settlement data are summarized in Table 9.2.2-2.

Table 9.2.2-2: Summary of Settlement Data

	Dam Crest Settlement Data by Section (Inches)		
	North	Central	South
Final Stage 1 Post Construction Data	8.0	10.3	7.1
Post Stage 3 Construction: January 2009 Readings	6.6	3.8	7.5
Total Settlement[1]	14.6	14.1	14.6
Settlement as % of Embankment Height[2]	1.38	0.84	1.83
Rate of Settlement over Last Ten Years	Flat	Flat	¼ inch/year

[1] Total settlement does not include settlements during Stage 2 and Stage 3 construction.
[2] Estimated maximum settlements was 1.6 percent of embankment height.

Movement: The documents reviewed included horizontal location monitoring data for 23 points on the dam: 15 points along the crest and eight points on the downstream face of the dam. Coordinates were provided for each point for the period of July 1978 to May 2008. Horizontal movements of points in the central and southern portion of the dam were relatively consistent. One of the points reviewed on the northern portion of the dam was somewhat inconsistent with the other data. The direction of movement of the northern portion of the dam is toward the upstream side. The direction of movement of the central and southern portion of the dam is to the downstream side. The results of our review of the horizontal movement at the dam crest data are presented in Table 9.2.2-3.

Table 9.2.2-3: Summary of Dam Crest Horizontal Movement Data

	Dam Settlement Data by Section (feet)		
	North	**Central**	**South**
Net Movement July 1978 to May 2008	0.019 NNE 0.337 ENE	0.240 SSW 0.205 SSW	0.346 WNW 0.368 SW
Estimated Maximum Movement	0.134 NNE (Apr 1985) 0.361 ENE (Nov 2004)	0.258 SSW (Nov 2006) 0.227 SW (Jun 1999)	0.410 WNW (Nov 2004) 0.379 SW (Nov 2006)

Movement data for monuments on the downstream slope face were also reviewed. The data were analyzed for movements for the 30 year period of 1978 to 2008, and for the most recent 10 year period of 19980 to 2008. For completeness, corresponding vertical movement of the slope face monuments is also shown in Table 9.2.2-4.

Table 9.2.2-4: Summary of Dam Downstream Slope Movement

Location	Initial Elevation	1978 to 2008 Movements (ft)		1998 to 2008 Movements (ft)	
		Horizontal	Vertical[1]	Horizontal	Vertical[1]
Dam Crest	878.929	0.368 (SW)[2]	0.663	0.061 (SW)	0.114
Near Mid-Point	773.468	0.393 (SW)	0.874	0.158 (W)	0.010
Bench	714.185	0.107 (SW)	0.070	0.011 (W)	0.058 (up)[3]
Toe	657.849	0.051(SE)	0.004	0.023 (N)	0.101(up)[3]

[1] Vertical movement is downward settlements unless otherwise noted.
[2] Directions are approximate.
[3] Detailed 10 year data for the bench and toe locations indicate vertical movements vary in direction from reading to reading.

Seepage: The data reviewed identified two long-term seepage areas on the downstream face of the dam. The seepage areas have been active since at least the mid-1990s. One of the seeps is near the left groin at about elevation 750. This seepage area is referenced as near piezometer P94-1. The second seep is near the right groin at about elevation 740. Both seeps are monitored monthly for flow rates and clarity. Reported 2006 through 2008 monthly flow rates for the two seeps are presented in Table 9.2.2-5.

Table 9.2.2-5: Reported Seepage Flow Rates

Side	Year	Monthly Seepage (gallons per minute - gpm)											
		Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Left	2006	6.0	5.0	2.5	6.64	3.17	1.7	1.6	2.2	8.9	12.7	N/A	3.64
	2007	11.8	11.4	5.81	6.91	4.2	2.1	7.2	3.8	2.3	12.7	3.9	5.7
	2008	4.6	16.8	7.6	5.06	5.1	4.1	4.1	3.2	3.2	2.3	5.5	15.4
Right	2006	25.3	19.3	16.5	17.44	14.27	15.4	14.8	12.7	17.0	27.3	N/A	18.52
	2007	34.7	67.8	24.93	29.37	10.8	10.8	10.7	10.1	9..0	27.3	15.1	20.2
	2008	18.6	55.4	18.7	16.1	16.1	9.6	9.6	7.9	8.7	9.5	14.2	30.8

Data for the first six months of 2009 is generally consistent with 2008, except for the unusual increase in both the February 2007 and 2008 right side monthly seepage rates. The corresponding 2009 recorded seepage rate was 8.9 gpm.

The data reviewed identified occasional evidence of temporary seepage, generally near the downstream slope bench. Based on the occasional nature of the observations and occurrence of local precipitation, the suspect seepage appears to be local runoff. Other than seepage discussed above, the data reviewed did not indicate any major leakage issue with the facility. The data did not indicate issues with maintaining desired water elevations in the impoundment.

The only case of leakage reported occurred in 1990 when a visual inspection revealed leaks through cracks and joints in the concrete-lined portion of the tunnel. These leaks were repaired in 1991 using a two-phased approach. First 330 gallons of a two-component polyurethane grout mixture was injected into the rock surrounding the tunnel to consolidate the rock and reduce the quantity of water flowing into the liner. Then, chemical grout was injected through small packers placed in holes drilled to intersect the liner/rock interface and at locations near existing cracks or joints in the liner. Based on visual inspection of the tunnel in April 1991 the repair appeared to be successful. (See the April 25, 1991 letter by Woodward-Clyde Consultants, Inc. in Appendix A-Doc 04).

9.2.3 Evaluation

The historical data indicate that the embankment dam is performing adequately.

9.3 ASSESSMENT OF SURVEILLANCE AND MONITORING PROGRAM

9.3.1 Adequacy of Inspection Program

Based on the data reviewed by Dewberry, including observations during the site visit, the inspection program is adequate.

9.3.2 Adequacy Instrumentation Monitoring Program

Instrumentation monitoring program is adequate. Although the WV –Dam Safety inspection report (Appendix A- Doc 09) concludes that the dam appears to be in good condition, it does not directly address structural adequacy and stability. The report references numerous monitoring wells and survey monuments used to monitor the dam's behavior; however no analysis of data from the wells or survey monuments is presented. The report indicates that previous inspection reports by the owner did not mention abnormal readings of concern in those reports. The owner's 2008 inspection report is limited to a presentation of data and lacks documentation of data analysis. It is unclear that data indicating potential adverse performance would be noticed in a timely manner.