AECOM

Root Cause Analysis of TVA Kingston Dredge Pond Failure from December 22, 2008

Volume II – Geological and Field Explorations

Kingston Fossil Plant, Harriman, Tennessee

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2.1 Geological and Field Explorations

2.1.1 General

AECOM field operations included drilling, sampling, relic identification, observation of site conditions, and logistical support of the drilling crews and sampling staff. Within the figures section of this volume, is an aerial photograph of the site identifying geographic features prior to the failure of Dredge Cells (Figure 2.1.1_1). These features are referenced throughout this RCA.

Logistical support was also provided by TVA Heavy Equipment Division (HED) in the form of:

- Site safety information and briefings,
- Material acquisition and repair (e.g., crane mats, supply handling, protective concrete barriers around instrumentation, etc.)
- Earthwork (e.g., building, clearing, and maintenance of roads and drilling platforms),
- Moving crane mats necessary to access at least half of the drilling locations on liquefied ash,
- Equipment for excavation of Test Trenches No. 1 and 2 for the former spillway outfall pipe and Test Trench No. 3,
- Dust control,
- Surveying, and
- Vehicle washing (daily) and washing/inspection prior to demobilization from the site.

The primary activity of the field exploration program was drilling and sampling of ash and native soils. The drilling locations were selected using a variety of criteria including, but not limited to:

- The position of the former dikes,
- Mapping and characterization of the remnant unfailed ash at the south side of Dredge Cell 1,
- Collection of samples from and beneath the failed ash immediately north of the ash stack,
- Exploration of Dike D and the south facing dikes on Dredge Cell 1 that did not fail, and
- Gathering of specific information if a preceding boring suggested that additional investigation was warranted in a particular area.

A general protocol for advancement of the borings was as follows:

- Piezocone Penetrometer Soundings (CPT_u) was generally advanced first at each location.
- Following CPT_u soundings, an SPT boring was advanced at each location for collection of continuous samples using a split-barrel sampler. SPT blow counts were recorded during advancement of these borings. Jar samples were classified according to type of material, relative grain size, color, texture, relative moisture content and structures if present.
- Results of the SPT borings were reviewed by AECOM field engineers who then made a determination as to whether additional testing warranted at each location. Additional testing included various combinations of vane shear testing, Osterberg samples and instrumentation.

- If additional testing at a location was deemed necessary, the drill rig was offset at least five feet (or as necessary based on site access) from the original location and additional exploration holes were advanced to facilitate the additional sampling, testing or instrumentation requirements.
- Multiple exploration holes were performed for special sampling or testing at most locations.

The procedures used for the various drilling and sampling methods, and for relic identification are discussed in detail in relevant sections of this volume. The location of all the borings and CPT_u soundings are present on Figure 2.1.1_2 Exploration Location Diagram also in the figure section of this volume.

2.1.2 Field Observations to Locate Relics

2.1.2.1 Description and Purpose

As part of the RCA, relics transported and deposited by the liquefied ash were located and identified within the debris flow. Relics were defined as identifiable pieces of man-made and natural materials found after the flow failure that had the potential to provide information regarding point of origin. Using point of origin and location where an object was found, an estimate of the flow path of the debris could be inferred.

2.1.2.2 Methods

On January 18, 2009 a relic search crew of two individuals were tasked with making a reconnaissance of the ash debris to locate, identify, photograph and map relics throughout the debris field. The recon was conducted primarily by walking. Portions of the Watts Bar Reservoir were surveyed by riding an amphibious swamp buggy. Equipment used to complete the reconnaissance included a Magellan® Triton™ 2000 hand-held global positioning satellite (GPS) unit, compass, digital camera, marking paint, flagging tape, cellular telephones and survey prism poles. The prism poles were used as a safety measure to probe the stability of the ash flow ahead of the relic hunters. Because of the nature of the wet loose ash, safety concerns dictated that no relic hunting was permitted without a partner. While searching for relics, personnel maintained visual contact at all times and generally stayed within 150 feet of each other. At least one team member also carried a two-way radio that could be used to contact the project manager and other site personnel in the event of an emergency.

Initially, the ash was frozen and could be traversed easily. In general, personnel walked roughly parallel paths across the debris flow. When items of interest were identified, a white board was used to record an identification number for the relic and a pin-flag would be placed at the location for later survey. This worked well, but as the number of relics increased and site activity required larger time commitments from the TVA surveyors, the relic search team used the GPS unit to record the location of each relic. Following the recon, data from the TVA surveyor and GPS were plotted on an air photo. The GPS proved relatively accurate, although minor adjustments were made as needed based on a review of the relic location when plotted on the air photo.

The relic search team also used the amphibious swamp buggy to access, observe and map relics identified in areas too soft or wet to traverse on foot. The swamp buggy was used primarily in the former Emory River channel area and on isolated occasions when a relic was unearthed in an inaccessible area within the debris flow.

As site restoration activities removed surficial features and disturbed relic locations, the relic search was suspended. The full time relic search was completed on or about February 8, 2009. All equipment operators were informed that if they unearthed any relics or pieces of debris, they were to contact a TVA foreman who would then contact one of the relic search team who would travel to the location to observe and document the relic. Only a small number of observations were performed on an as-needed basis after February 8, 2009.

2.1.2.3 Observations

The first relics mapped were selected blocks of ash that had floated intact to the north and northwest of the former containment area and also into the channel between Dike C and a small island to the east of the ash collection pond. The stratigraphy of these relic blocks was described and sketched with thought to possibly reconstructing their point of origin. Determination of point of origin for the blocks proved futile, although reconstruction of the very large blocks having remnants of dike bench gravel drainage layers installed in early 2008 was successful using air photographs.

After approximately one week, the focus of the recon turned to smaller man-made relics and large identifiable remnants of dike materials. A representative sample of relics identified and located included:

- Polyethylene drainage piping and associated stone and/or geotextile
- Dike remnants with and without unique features (e.g., specific vegetation, traceable patterns, cattails cover with ash, etc.)
- Boats, dock remnants, a mailbox, buoys, tires, outdoor furniture, etc

Interviews with TVA employees familiar with the site provided insight into the original position of many relics. Figure 1.3.2_1 in Volume I is map of the location of relics along with estimated vectors and distances of travel. A photographic log of relics with identification numbers is included in Appendix 2A.

Of particular value are relics with precise points of origin. Relics fitting that description included:

- A cellular telephone repeater tower originally located on a portion of the setback northwest of the perimeter Dike C.
- TVA Survey Monument CHT-2.
- Cables and 24-inch ID polyethylene piping.
- Remnants of set back area behind original Dike C covered with cattails in Slough 2.

Assistance with determination of original position of relics was provided by TVA personnel. Interviews were conducted with TVA staff familiar with the site or responsible for daily observations of site conditions. These interviews along with a site walkover on February 4, 2009 provided AECOM with invaluable information about pre-failure conditions.

2.1.3 Geology

2.1.3.1 Physiographic Setting

Regional geologic information was obtained through TVA, library searches, field observations and from the State of Tennessee Department of Environment and Conservation Division of Geology.

Initial documentation was provided from the TVA archives. The following documents were received from TVA and reviewed for this report:

- Benziger, Charles P. and Kellberg, John M., 1951, Preliminary Geological Investigation for Eastern Area Steam Plant, Tennessee Valley Authority, Division of Water Control Planning, Geologic Branch, Knoxville, Tennessee.
- Boggs, Mark J., P.G., et al., 1995, Hydrogeologic Evaluation of Ash Pond Area, Kingston Fossil Plant, Report No. WR28-2-36-124, Tennessee Valley Authority Resource Group, Engineering Services, Norris Engineering Laboratory.
- The Kingston Steam Plant: A Report on the Planning, Design, Construction, Costs, and First Power Operations, Technical Report No. 34 (a.k.a. TVA Brown Book, TVA 00020036), Tennessee Valley Authority, Knoxville, Tennessee, 1965.

2.1.3.2 Native Soils

Information regarding native soils at the site was obtained through the Natural Resource Conservation Service (NRCS) and the University of Tennessee (UT). NRCS data was obtained from their Web Soil Survey located at:

- http://websoilsurvey.nrcs.usda.gov/app/WebSoilSurvey.aspx

Staff at UT also provided assistance with research of soil survey data compiled prior to construction of the reservoir and power plant. An earlier soil survey of the area was completed by the United States Department of Agriculture, Soil Conservation Service in 1942; however, a copy of this soil survey could not be located at the time this report was completed.

2.1.3.3 Bedrock Geology

Initial research included a visit to the site and location of bedrock outcrop in the vicinity of the site. An AECOM Senior Project Geologist conducted a reference search at UT in Knoxville, Tennessee. The following materials were identified at UT and reviewed:

- Mebane, R. Allen, The Geology of the South Harriman Area, Roane County, Tennessee [master's thesis], University of Tennessee, December 1957.
- Moore, James L., Finlayson, C. P., and Milici, R. C., 1993, Geologic Map and Mineral Resources Summary of the Harriman Quadrangle, Tennessee. State of Tennessee Department of Environment and Conservation.
- Poole, Joe L., The Geology of the Harriman Quadrangle, Roane and Morgan Counties, Tennessee [master's thesis], University of Tennessee, August 1949.
- Swingle, George D., et al., 1966, Geologic Map of Tennessee, East-Central Sheet, State of Tennessee Department of Environment and Conservation.
- Poole, Joe L., The Geology of the Harriman Quadrangle, Roane and Morgan Counties, Tennessee [master's thesis], University of Tennessee, August 1949.
- Isham, Tim, 2009, Personal communication.

Field observations were also used to supplement documents. Outcrops of bedrock were visited at the northeast corner of the intersection of Tennessee State Highway 70 (Hwy 70) and Pine Ridge Road, along the intake channel for the power plant, and along the south side of Hwy 70 southeast of the intersection of Swan Pond Road and Hwy 70. Strike and dip measurements were collected and general observations of the rock were made at each location. The site visit locations are presented on Figure 2.1.3_1.

Historic topographic maps and aerial photographs were reviewed to observe changes with time and to obtain historical information regarding topography and land use prior to construction of the power plant and ash disposal facility. Topographic maps and aerial photographs were obtained from the TVA archives. Topographic maps reviewed for this report included:

- Land Acquisition Map, Map No. 10N39, TVA Maps and Surveys Division, 1940.
- United States Department of the Interior Geologic Survey and TVA Maps and Surveys Division, Harriman Quadrangle, Tennessee, 1941.
- United States Department of the Interior Geologic Survey and TVA Maps and Surveys Division, Harriman Quadrangle, Tennessee, 1952.
- United States Department of the Interior Geologic Survey and TVA Maps and Surveys Division, Harriman Quadrangle, Tennessee, 1968 (and photorevised).
- United States Department of the Interior Geologic Survey, Harriman Quadrangle, Tennessee, 1998.

These maps are included in Appendix 2 B.

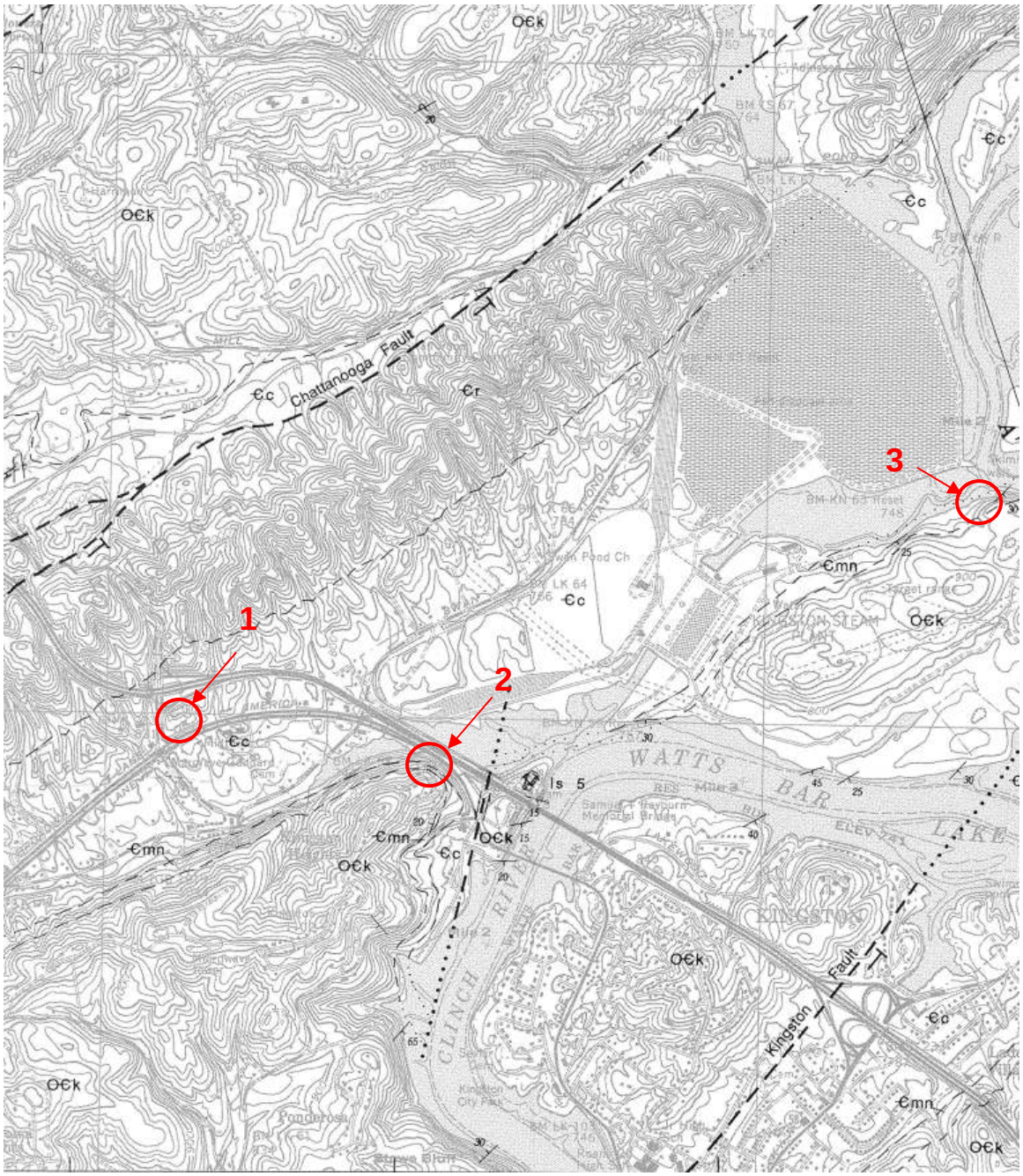

Figure 2.1.3_1: Bedrock outcrops visited during field investigation at the Kingston Fossil Plant.

1. Northeast corner of Tennessee State Highway 70 (TN Hwy 70) and Pine Ridge Road.
2. Highway Right of Way along the south side of TN Hwy 70 east of Keylon Drive.
3. Maynardville Limestone southwest of the eastern bridge approach.

2.1.3.4 Regional Groundwater Information

Regional groundwater information was acquired from TVA Kingston documentation listed in Section 2.1.3.1 and reviewed by AECOM. Additional information regarding groundwater flow was gathered as part of the field investigation through the use of pneumatic piezometers installed at locations around the site.

Groundwater information was also inferred around the perimeter of the site by observation of Watts Bar Reservoir pool elevation.

2.1.3.5 Seismic Activity

A review of online databases was conducted to assess seismic activity in the vicinity of the site. Databases reviewed included:

- The University of Memphis Center for Earthquake Research and Information (CERI) at url: http://www.ceri.memphis.edu/index.shtml
- The United States Geologic Survey Earthquakes Hazards Program at: http://earthquake.usgs.gov/eqcenter/ (Individual files locate at urls: http://earthquake.usgs.gov/eqcenter/dyfi/events/se/hnw1110a/us/ and http://earthquake.usgs.gov/eqcenter/dyfi/events/se/hnw1218a/us/)

The earthquake data table (1.3.3_T1 in Volume I) was compiled from events shown at these web sites.

2.1.4 Field Explorations

The exploration locations generally included several boreholes soundings used for various sampling and testing purposes and are shown on Figure 2.1.2_2 Field Exploration Location Diagram. In general, soil borings without a letter suffix (e.g. Boring 09-100) were used for continuous standard penetration tests and sampling to the surface of the bedrock. Soil borings labeled with an “A” suffix (e.g., 09-100A), generally were reserved for in-situ vane shear testing (VST). The “B” suffix (e.g., 09-100B), represents soil boring locations where fixed piston Osterberg sampling using 3-inch diameter steel tubes were performed. Locations labeled with a “C” suffix (09-303C1) are locations where cone penetrometer testing (CPT_u) was performed. Each of these methods and procedures will be discussed in more detail elsewhere in this report.

The exploration locations were grouped into nine series (e.g., 100 through 900):

- 100-series boring/sounding locations were performed 10 feet downstream of the toe of the upstream Dike A. The 10-test locations for this series were spaced at approximately 200-foot intervals along the north side of Dredge Cell 2. A summary of the field results for the 100-series are included on Figure 2.1.4_1.
- 200-series boring/sounding locations were performed along the original Dike C crest level (El. 748 feet). There were 13 test locations spaced at approximate 200-foot intervals. A summary of the field results for the 200-series are included on Figure 2.1.4_2.

- The 300-series borings/soundings consisted of four locations and were located on the remaining unfailed portion of Dredge Cell 1.
- The 400-series boring/sounding locations were generally performed along the toe of the western Dredge Cells 1 and 2 along Swan Pond Road, with the exception of locations where instrumentation was installed. Instrumented locations were offset eastward to avoid inference with the construction of Swan Pond Road at the time of exploration. These locations were spaced at 200-foot intervals. At several locations, 09-401, 09-403, 09-405, 09-407 and 09-411, only CPTu soundings were conducted. A summary of the field results for the 400-series are included on Figure 2.1.4_3 (2 sheets).
- The 500-series boring locations were performed within the failed portions of Dredge Cell 2.
- The 600-series boring locations were performed along the unfailed Dike D, east of the Dredge Cells. A summary of the field results for the 600-series are included on Figure 2.1.4.4 (2 sheets).
- The 700-series boring locations were performed along the Phase 2 Lateral Expansion for Stantec and TVA.
- The 800-series were performed to obtain sampling and testing on the Divider Dikes between the Ash Collection Pond and Settling Pond away from the failed portion of the dredge cell.
- The 900-series boring locations were performed as part of the seismic design and categorization effort drilled near the location of 09-103 by Stantec and TVA.

2.1.4.1 CPTu Testing

General Description

The Piezocone Penetration Test (CPT_u) is performed by hydraulically pushing a cylindrical rod into the soil from the back of a drill rig. The rod has an instrumented, 60 degree conical tip with a maximum diameter of 1.4 inches. The bearing stress (q_c) is measured on this tip as the probe is advanced into the soil at a constant rate of 2 centimeters per second (cm/s) unless otherwise noted. Immediately behind the tip, a porous polyethylene filter with a thickness of 1/8-inch and an outside diameter equal to the diameter of the cone measures the induced dynamic pore pressure (U) generated during the test. Immediately behind the pore pressure filter is an approximately 5.2-inch-tall cylindrical sleeve which reacts against and measures the local friction (f_s) of the soil. Inside the cone, an inclinometer measures the deviation of the cone tip from vertical. A stand-alone rotary proximity switch measures the depth of advancement as the cone is pushed into the ground.

Values of tip resistance, sleeve friction, pore pressure, inclination and depth were recorded automatically by a field computer for every 0.065 ft (2 cm) of soil penetrated. Thus, the CPT_u test provides a nearly continuous soil profile which indicates changes in relative density and stratigraphy (layers of different materials). The data was collected using CPTSound Software version 2007.2.1 by Applied Research Associates, Vertek Division.

Pore Pressure Measurement Procedures

Three different piezocones (1, 2, 5, and 10-ton maximum range) were used to perform the tests for this project. Calibration sheets for each of the cones are provided in Appendix 2C. All three cones had a base area of 10 square centimeters (cm^2) and a sleeve area of 150 cm^2. The pore pressure filter for these cones is mounted directly behind the base of the cone tip. The friction sleeve is designed with equal end areas so that the correction of the sleeve readings for water pressure effects is generally unnecessary.

The polyethylene filters were pre-saturated in glycerol under high vacuum in the laboratory, and then carried to the site submerged in de-aired glycerol in air tight containers. Glycerol is used instead of water because it helps to prevent the loss of saturation in the ash and soils above the water table.

Immediately before each test, the internal cavity which links the pore pressure transducer to the filter was flushed and saturated with de-aired glycerol by direct injection with a hypodermic needle. The saturated filter was then placed on the cone and the tip was tightened in place. The cone tip was removed at the end of each test and a new filter was installed for each test location.

The measured pore pressure is generally not the hydrostatic pore pressure, but rather a generated pore pressure due to shearing of the soil as the cone advances. This pore pressure tends to be higher than hydrostatic in normally consolidated clays, silty clays and silts. In loose sands where drainage is more rapid, the measured pore pressure is often closer to the hydrostatic value. Dense sands and silts or overconsolidated clays tend to produce negative pore pressures since these materials tend to dilate during shear.

Pore Pressure Dissipation Tests

Pore pressure dissipation (PPD) tests were conducted at most of the piezecone test locations. PPD tests are conducted in saturated soils below the water table to back-calculate an estimate of the in-situ soil permeability, as well as to provide a measure of the piezometric pore pressure. The probe is stopped at the depth of interest and the drop or increase in pore pressure is recorded as a function of time. The dissipation test results are included in Appendix 2D.

Data Collection

A total of 69 piezocone locations noted on Figure 2.1.2_2, were tested during the course of this exploration program. CPT_u tests were conducted within a few feet of the corresponding SPT borings for the purposes of correlating with physical samples. Some of the locations required multiple advances of the cone due to shallow refusal in hard zones. Refusal was rarely achieved by approaching the allowable limits of the cone sensors, but rather by the drill rig lifting off its outriggers, usually with 325 to 400 tons per square foot of pressure measured on the cone tip. Where zones could not be penetrated, they were drilled either with hollow-stem augers or mud rotary methods for a few feet and then backfilled with clean sand and the cone test continued as a subsequently

numbered cone advance. The naming convention is described in the following example: 09-103-C1 indicates the year (2009), the boring location number (103) and the first advance of the cone at that location (C1). Subsequent advances were recorded as C2, C3, etc. A summary of the CPT_u field notes is included in Appendix 2C that helps document why the multiple advances were necessary at some test locations.

Pore pressure dissipation tests were collected at nearly all of the CPT_u locations, with two to seven tests conducted in each location. Generally, the test depths were determined during data collection where a sharp rise in pore pressure was noted in relatively soft materials. Two tests were also performed at a high rate of advance (18 to 36 cm/s) to assess whether the subsurface materials were truly reacting in a drained or undrained condition.

A database was created to store and catalog each of the tests which were performed. The data has been summarized and presented as a series of graphs. The CPT_u plots and pore pressure dissipation graphs are presentations of the raw data that was collected in the field. This information includes tip resistance (q_t), skin friction (f_s), friction ratio (R_f), and pore water pressure (u_2). The soil behavior type (SBT) and static pore water pressure (u_0) that are shown on the CPTu plots are calculated based on the other raw data and parameters. The SBT was determined on the chart of Tip Resistance vs. Friction Ratio for Normalized Soil Behavior Types by Robertson (1990).

The static pore water pressure is based on the unit weight water and the depth of the reading. The static pore pressures determined from this program increase linearly with depth and do not necessarily represent actual piezometric conditions. The calculated static pore water pressures are shown on the plots as dashed lines.

2.1.4.2 Soil Borings

The soil borings were performed using truck, all-terrain vehicle (ATV) and trailer mounted drilling rigs. The drill rigs used in this exploration program include two (2) truck-mounted Mobile B-57's, one (1) truck-mounted Diedrich D-120, one (1) ATV-mounted Diedrich D-50, one (1) ATV-mounted CME-850 and one (1) trailer-mounted Diedrich D-25.

Site conditions and access generally dictated the equipment necessary to perform the required tasks. Due to extremely loose ash and unstable ground conditions, ATV's were needed to perform many of the soil borings in the failed ash areas. However, most of these locations were not accessible even to ATV equipment without some remedial stabilization of the ground surface and placement of crane mats. Pontoon mounted excavation equipment was needed to access these areas to provide material that could stabilize the area, redirect water flow and place crane mats which would allow drill rig and drill crew access to these soft ground condition areas. At some locations the drill rig was several hundreds of feet from the support truck and relatively stable working surfaces.

Each of the borings performed for continuous split-spoon soil sampling, vane shear testing and Osterberg sampling was initiated utilizing solid flight augering methods. Augering was generally limited to the near surface portions of the borings, as the saturated conditions, did not permit much advancement with this method. Casing was extended through the previously sampled zone and drilling fluid, consisting of water and bentonite was introduced. The borehole was then advanced by means of rotary wash methods with circulating drilling fluid and rotating tricone cutting bits. Occasionally, steel casing needed to be extended beyond the originally installed depth due to the loss of drilling fluid or borehole instability in the relatively near surface zones.

The soil borings used for continuous soil sampling (i.e., borings without a suffix) were advanced to the surface of apparent bedrock or into the bedrock. Selected borings were extended into the bedrock and rock core samples obtained by means of N-sized coring tools. Representative soil samples were obtained in these borings using split-spoon sampling procedures performed in general conformance with ASTM Standard D-1586.

The hammers used for Standard Penetration Tests (SPT) on each of the above noted drill rigs were evaluated at the site for the energy delivered to the drill rods for determining the hammer's average Energy Transfer Ratio (ETR) for calculating N60 corrected blow counts. The results of these tests are included in the GRL Engineers, Inc. report "Standard Penetration Test Energy Measurements", dated March 3, 2009, included in Appendix 2F.

A field log, recording observed soil conditions, groundwater conditions, and drilling procedures was maintained by the drill crew. Where split-spoon samples were collected, the drill crew noted the number of hammer blows to advance the sampler each 6 inches in depth. All soil samples obtained from the drilling operations were sealed immediately in the field. At selected boring locations, representative soil samples from the split-spoons were placed into pre-weighed 2 oz. jars which were shipped to the Vernon Hills, IL laboratory for water content determination. Within another container a larger amount of sample was saved for classification in our laboratory.

Observations of water levels encountered in the boring during and upon completion of the drilling and sampling operations are noted on the representative soil boring logs. These observations are noted on the individual soil boring logs which are included in the Appendix 2E – SPT Soil Borings.

2.1.4.3 Vane Shear Testing

Borings labeled with an “A” suffix (e.g., Boring 09-100A) were primarily used for the performance of in-situ vane shear testing. Vane shear test depths were selected following completion of the SPT split-spoon, continuous sampling borehole location. The test depths were based on visual classification of soft to stiff, cohesive soil from the adjacent soil boring location. Vane shear testing was performed in general accordance with ASTM D-2573.

Three vane sizes were used to perform the various tests and chosen based on the samples obtained in the SPT borehole. Each vane is four-bladed and has a tapered end. The vanes were attached to a slip coupling attached to steel rods with centralizers. The vane was then pushed to the prescribed interval using the drill rig.

The torque applied was measured by apparatus that made a continuous record. Following failure of the soil, the measurement apparatus was removed and the vane was rotated 5 to 10 times. The measurement apparatus was placed back over the torque rods and the remolded strength test was performed. A table that summarizes the peak and remolded undrained shear strength and sensitivity at the test location is presented in the Appendix 2G – Vane Shear Tests of this volume.

For each vane shear test that reached peak strength, a qualitative assessment of the observed type of failure was recorded. The failure types were described as well-defined, moderately well-defined and not well-defined. These descriptions were based on visual inspection and are meant to provide a relative basis for comparison between tests. The well-defined peak failure describes a peak failure in which there was a rapid decrease in the shear strength after failure. The not well-defined peak failure describes a peak failure in which there was little discernable decrease in the shear strength following failure. The moderately well-defined peak failure describes a failure in which the peak shear strength was easily discernable; however, the decrease in shear strength immediately following peak strength was not as great or rapid as in the well-defined failures. Several tests did not reach peak strength. These particular tests reached the maximum torque capacity of the equipment being used.

Following the completion of vane shear testing, samples were recovered through the tested zones. Where it was possible, 2-inch Shelby tubes were pushed to obtain recovery of the soil disturbed during vane shear testing to sample the soil for classification and other index tests described below. Where no recovery was made with the Shelby tubes, oversized (i.e., 2.5 or 3 inch I.D.) split spoons were often pushed for sample recovery. Since these samples were considered disturbed, no attempt to record the Standard Penetration Test blow counts were made. The purpose of this sampling was to provide for visual and index laboratory testing of the soils in which vane shear tests were performed. In general, the tests performed on this material included moisture content, liquid and plastic limit determinations and combined hydrometer and sieve analyses. Details of these laboratory testing methods are included in Section 3.1.1 Geotechnical Index Tests, Gradations and Classifications.

These soil borings were drilled, backfilled or instrumented according to the methods described in the previous Section 2.1.4.2.

2.1.4.4 Undisturbed Sampling

Borings labeled with a “B” suffix (e.g., Boring 09-100B) were primarily used to obtain undisturbed sampling using 3 inch thin-walled tubes and an Osterberg Sampler. Sampling depths were selected following the completion of the split-spoon, continuous sampling borehole.

This type of sampler uses hydraulic water pressure to activate a piston that advances a 3 inch diameter thin-walled tube into the bottom of the borehole. The Osterberg Tube samples were obtained using the general procedure outlined in USACE No. 30, “Soil Sampling” (2000) and Walton, et al., (2002).

Moisture contents of the tube samples were generally obtained from the top and/or bottom of each tube sample for testing. Each upright tube sample was sealed with wax at the top and bottom of the tube in the field at the boring location. Each tube was carefully packed to minimize sample disturbance and shipped upright from the project site to our Vernon Hills, IL laboratory or to the laboratory at the University of Massachusetts–Amherst, MA via a passenger vehicle operated by an AECOM engineer.

These boreholes were drilled, backfilled and instrumented according to the methods described in the previous Section 2.1.4.2.

2.1.4.5 Site Instrumentation

In several of the boreholes, AECOM installed instrumentation, including pneumatic piezometers and inclinometer casings. These instruments were installed to monitor the existing conditions as well as to measure any possible ongoing changes or set baselines for future changes in the various states of stresses and deformations that may occur due to ongoing operations at the project site.

Inclinometer casings were installed for two (2) primary purposes. The first purpose is for the monitoring of deformation over time to attempt to locate residual movements along the slide plane and the second was as a casing for the performance of geophysical surveys.

For the inclinometer casings installed to monitor short and long-term deformations, the inclinometer casings were set with grout into bedrock a minimum of four (4) feet. For the geophysical surveys, the inclinometers were installed to an approximate depth of 100 feet below the existing ground surface. All boreholes where inclinometer casing was installed were grouted with a relatively low strength cement-bentonite grout mix. The installation depth for each inclinometer casing is shown on the individual boring logs. Results of the inclinometer plots are included in Appendix 2H – Site Instrumentation.

Pneumatic SINCO piezometers were also installed into selected boreholes, following completion of the sampling and testing protocols. The pneumatic piezometers were placed in boreholes, most often as multiple stage installations. The boreholes were flushed with water and the pneumatic piezometer with a saturated filter tip encapsulated in a canvas sack filled with filter sand was lowered to the selected depth. An additional intake zone of filter sand was generally placed one foot above and one foot below the pneumatic piezometer tip depth. To create a seal, bentonite chips were placed a minimum depth of five feet or up to another piezometer installation level. The borehole was often then grouted using cement-bentonite mix to the ground surface or backfilled with additional bentonite chips.

As the instruments were installed, baseline readings and subsequent readings, if time allowed, were obtained on each instrument. Tabulated or graphical results of the inclinometer and pneumatic piezometer readings are contained in Appendix 2H – Site Instrumentation.

2.1.5 Test Excavations

2.1.5.1 Test Trenches No. 1 and No. 2 – Search for Outfall Pipe

Based on available historic drawings as well as TVA staff recollections, two test trenches were completed to locate former outfall piping associated with the Starter and Dike C. The outfall piping was illustrated at two locations on drawings from the TVA archives, leading to uncertainty regarding the actual location. Recollection of long-time TVA staff also provided somewhat conflicting locations. The locations physically explored by excavating test trenches at locations depicted on Figure 1.3.5_1 in Volume I.

Coordinates for the potential locations were extrapolated from the historic drawings. Using the amphibious swamp buggy (ASB), TVA surveyors initially located and staked the two most probable locations for the outfall pipe. On January 25, 2009, AECOM personnel aboard the ASB along with an amphibious trackhoe (ATH) mobilized to one of the locations deemed most likely (Trench No. 1) matching the 1951TVA Drawing No. 10N400, Rev. 6. Trench No. 1 site was located near the center of a post-failure surface water drainage swale cut through the ash after the failure. The drainage cut minimized the required depth of excavation although the ash that was saturated and excavation proved problematic as the sides of the cut continually flowed into the excavation. It was estimated that the drain pipe(s) would be approximately four to seven feet below the bottom of the drainage. After digging through the ash to a depth of seven to eight feet, the ABH “probed” back and forth across several feet on either side of the staked location in an attempt to locate dike remnants or the concrete pipe. After approximately

30 minutes effort when no alluvium clay, dike material or buried concrete pipes were encountered, it was determined that a long-stick ATH would be needed to dig deeper, and the first attempt was abandoned.
On the morning of January 30, 2009 a long-stick ATH mobilized to the initial location and assisted the conventional ATH. Approximately one hour was spent "probing" for the pipe at depths at least 5 feet below the expected pipe invert of El. 735 feet. When no clay, dike material, or concrete pipe was encountered, this location was abandoned in favor of the second identified location south of a former headwall thought to be the actual original spillway (Trench No. 2).

Excavation of Trench No. 2 commenced on the afternoon of January 30, 2009 using a conventional ATH and a long-stick ATH. Dike C dike and foundation clay was encountered almost immediately and the outfall pipe was exposed on February 1, 2009. Only one of two concrete outfall pipes was identified and fully excavated. Despite probing in the vicinity of the first pipe, the second was not located.

Following discovery of one of the outfall pipes, the excavation was expanded laterally to expose as much of the pipe as possible. The pipe is 30-inch inside diameter, bell and socket concrete with okum gasket material in the joints. The pipe was exposed over a length of approximately 55 feet. The outfall end (northeast) of the pipe also extended beneath one of the major west-to-east drainages. Exposing the northeast end of the outfall pipe next to the filled in slough would have risked flooding the entire excavation, so excavation was halted and a berm created adjacent to the drainage. The outfall intake structure was located to the southwest of the initial Trench 2 excavation. However, seepage from the ash required that a berm be left in place between the intake structure and the piping run to prevent collapse of the side walls and flooding. Only the upper rim of the intake structure was exposed by excavation. The intake was plugged with concrete and covered with ash. The transverse sections of pipe had only minor amounts (less than two inch thick) of ash in them. Staining on the pipe walls indicate that when in operation, the pipes flowed at least half-full.

Although the dike material had been disturbed by excavation a cross-section through the dike with the outfall pipe shown on the section was prepared.

Following exposure of the concrete pipe, a four-foot long section of the pipe was removed approximately half-way along the exposed section for observation of the interior condition. The pipe walls did not show distress. Photographs of this outfall pipe are shown in Appendix 2A – Relics Location Data.

2.1.5.2 Test Trench No. 3 – South Dike

During the early stages of the root cause failure analysis, it was recommended that a portion of the remaining south dike be excavated and the construction materials examined with respect to the design for raising the impoundment. Procedures, logistics and safety considerations were discussed at a meeting on March 4, 2009. The meeting was attended by representatives of TVA Heavy Equipment Division (HED), TVA safety personnel, AECOM, equipment operators and representatives of Stantec. Prior to commencement of excavation, TVA, along with AECOM personnel completed a Work Package (WP) and Job Safety Analysis (JSA) for the dig. As part of the WP and JSA, AECOM engineers provided TVA safety personnel with stamped drawings depicting the proposed slopes and depths of the excavation. Results of piezocone penetrometer testing suggested that stability of the ash would be the largest safety concern, followed by equipment hazards and weather. Procedures specific for working in the vicinity of and down slope from unstable, saturated ash conditions were defined in writing in the event that unstable ash was encountered. Weather conditions would also dictate work since ash stability decreased under rainy/wet conditions. Based on the meeting discussion and drawings, TVA approved the WP and JSA for Test Trench No. 3. Job safety briefings were held prior to the start of work each day of the excavation with all staff scheduled to be working in or around the excavation.

The design perimeter of the excavation was staked out by TVA surveyors. A temporary benchmark was also installed to facilitate manual surveys if necessary.

Excavation of the south dikes of Cell 1 allowed direct observation, sampling, and testing of the dikes themselves (placed and compacted ash) and also the sluiced ash (pumped Dredge Cell 1 material) that was hydraulically placed behind the dikes. The excavation also facilitated observation and sampling of the lateral drainage system placed along the heel of each dike as it was raised.

Material excavated from the south side of the Cell 1 dikes (D1 down to C1) was transported to an instrumented area located to the northwest of the excavation and east of Boring 09-404 identified as the Trench Ash Disposal Area (TADA) that was approved by TDEC. Prior to ash placement, four settlement plates were constructed and installed at random locations within the TADA. The four settlement plates are illustrated on Figure 1.3.5_5 included with this Appendix. The settlement plate elevations (surface plate and top of pipe) were surveyed using both GPS Station and conventional transit methods prior to ash placement. The area was not amenable to proof rolling prior to ash placement due to the loose wet surface and underlying ash material.

Also installed as part of the excavation activities were four surveyed monuments located on the slope created by the grading of the former high wall. These TDEC requested monuments were given the designation UT-1 through UT-4. TVA performs surveys of these stakes to monitor cut slope stability and Cell 1 settlement.

Prior to commencement of the excavation, AECOM subcontracted Roto Rooter of Knoxville, Tennessee, to televise accessible drainage pipes penetrating the south dike face of Cell 1. Roto Rooter used a Spartan PROvision 2.0 camera system and small diameter camera to investigate several drain pipes that daylight on the face of the south face of Cell 1 dikes. The video equipment was used to determine if the drains contained sediment or standing/trapped water prior to excavation. Only non-perforated polyethylene (PE) pipe extended back into the dike was video taped. The perforated PE pipe was present as laterals, and the camera was not able to negotiate the 90 degree corners to view perforated PE pipe in place. The video illustrates that most of the drains were clear of debris and carried water as evidenced by staining on the sidewalls of the pipe. Two of the drains had standing water where the drain pipe was installed with a downward sag in the pipe. None of the video-taped pipes were obstructed. One pipe was observed to be obstructed at the surface. Roto Rooter provided videos of the drain pipe surveys.

Excavation of the south facing Dredge Cell commenced on March 7, 2009 at El. 821 feet, in material placed as part of another lift of the south dike. Equipment dedicated to the excavation included one dozer, one rubber-tired backhoe, one long-stick conventional trackhoe, one regular-stick conventional trackhoe, water trucks (for dust control) and varying numbers of off-road dump trucks to transport ash. Trackhoes used a flat bucket to prevent disturbing lower test lifts as the excavation proceeded. A flagman was stationed at the loading site to guide trucks and keep pedestrians clear of the loading platform. During excavation activity, TVA personnel periodically applied water and/or Flexterra® for dust control as needed.

Ash was transported to the TADA by articulated off-road dump trucks. The trucks used a makeshift ramp constructed of shot-rock and breaker run stone. The trucks backed onto the ramp and dumped onto the ash. The ash was then spread using a wide-track dozer. Since most of the ash transported from the excavation was dry, the TADA attained sufficient stability to walk over the fill without rutting. A three-day rain event with associated runoff from the adjacent slopes caused the surface to become saturated and the fill surface was too wet for vehicle or foot travel. Surface stability was restored by aeration of the material using a trackhoe and re-compaction with the wide-track dozer. However, areas immediately adjacent to the settlement plates remained soft because of the effort not to disturb the plates with machinery. The inclinometer installed at 09-404 was read frequently during the test trench fill area to monitor rapid filling over previously failed ash.

Initial excavation activity consisted of removing the upper three feet of ash in the test area from El. 821 to 818 feet. This upper layer represented early construction of an initial lift for the Stage D3 Dike and was considered disturbed. The first testing was conducted on an ash lift at El. ~818 feet. As testing of each lift was completed, test pits were advanced to the next lift elevation (e.g., El. ~816 feet). The excavation proceeded downward at two-foot depth increments. Samples were collected from five TVA design dike levels. Figure 1.3.5_4 in Volume I illustrates the excavation cut on a cross section with TVA upstream dikes labeled. After the initial 8 feet of excavation was completed, the JSA was revisited and deemed adequate for the remainder of the excavation unless conditions changed.

Periodically, drain piping installed during construction was encountered. Drain pipe between the lateral and daylight, perpendicular to the dike axis, was installed using 8-inch inside diameter (ID) non-perforated polyethylene (PE) piping. The non-perforated piping was placed directly into the ash. Lateral drains running parallel to the dike were completed using 8-inch ID, perforated PE pipe. The laterals were placed into a trench lined with geotextile (both woven and non-woven were observed). The lined trench was backfilled with Number 57 crushed stone gravel and a layer of geotextile was placed over the top and stapled in place. All of the drains observed had transmitted water as evidenced by gravel dust and rust on and inside the pipes. In general, the gravel packs were clean with only small amounts of ash present inside the observed geotextile. Only one of the pipes had significant sediment inside, the remainder appeared to be virtually sediment free. Samples of the drain pipe, geotextile, and filter pack were collected for potential testing.

As the excavation proceeded, three randomly selected sand cone density (SC) tests were conducted on each of 15 dike excavation lifts. A surface was prepared by back scraping the ash at the test location nearly level using the trackhoe bucket. The surface was then finished smooth using the edge of the sand cone plate as a bevel. The material recovered from the SC sample location was returned to a mobile laboratory van located near the test area. Samples from each SC sample were placed in pre-tared glass jars and placed in a thermostatically controlled oven overnight to determine water content in general conformance with ASTM D2216. SC Sample Nos. 1 through 48 were collected during the initial excavation, working from El. 818 feet downward to El. 790 feet. Material in these samples was collected from both compacted bottom ash in the Stage C1 through Stage D2 dikes and non-compacted sluiced ash placed behind each dike. SC Sample Nos. 49 through 68 were collected exclusively from sluiced ash placed behind the dikes from El. 816 feet downward to El. 792 feet. Sand cone locations are identified as “SC-” in this report and on figures and data tables. Results for those tests are included in Volume I, Section 1.3.5 and Table 1.3.5_T2.

Upon completion of the sand cone tests, two of three test locations were chosen for Shelby Tube (ST) sampling. Shelby tube (ST) samples are identified as “ST-” in the report and on figures and data tables. ST samples were collected using one-foot long 3-inch diameter tubes rather than standard 30-inch long tubes. The shorter tubes limited the potential for damage by buckling during sampling. Four ST samples were collected from each lift of the excavation, two horizontally and two vertically. Tubes were pushed using one of the hoes (rubber tired or track). Tubes were extracted from the sample location by digging adjacent to the tube with the trackhoe and removing the tube by hand. The locations for the ST samples were chosen to collect samples from differing types of materials if present (e.g., bottom ash versus fly ash, wet versus dry, etc.). The ST samples were returned to the mobile laboratory van for processing (moisture content sampling, weight, wax seal and sealed caps). The ST samples were handled and stored in the general configuration in which they were gathered (horizontal or vertical) and kept from freezing. The location and elevation of the ST samples were surveyed by TVA or AECOM personnel and were recorded on field data logs. ST Sample Nos. 1 through 62 were collected during the initial excavation working from El. 818 feet downward to El. 790 feet. ST Sample Nos. 63 through 90 were collected exclusively from sluiced ash placed behind the dikes from elevation El. 816 feet downward to El. 792 feet. Table 1.3.5_T3 in Volume I is a summary of the results of SC and ST sampling. Volume I, Part 2 Attachments contains representative photos of activities conducted during the excavation.

Density of the in-situ ash was calculated in general conformance with the latest version of ASTM D 1556, “Test Method for Density and Unit Weight of Soil in Place by the Sand-Cone Method.” Table 1.3.5_T3 is a summary of the results of SC and ST density calculations. Figure 1.3.5_20 is a plot of dry density versus elevation. Sluiced ash and compacted ash (dikes) are plotted separately. Determination of sluiced versus compacted ash was made based primarily on location. However dry density differences noted between ash placement types was also considered. Squares represent ash believed to have been sluiced (non-compacted) and triangles represent compacted ash placed during dike construction. The range of dry density of sluiced ash (58.2 to 82.2 pounds per cubic foot [pcf]) is slightly lower than that for compacted ash (63.5 to 106.5 pcf).

2.1.6 Topographic Surveys

According to correspondence from TVA, original survey data for the Kingston Fossil Plant was generated using a horizontal datum corresponding to the North American Datum 1927 (NAD27 - Tennessee Lambert Plant Adjustment). The vertical datum used was the National Geodetic Vertical Datum of 1929 (NGVD 29).

Surveys performed using Light Detecting and Ranging (LIDAR) technology were generated following the ash incident, using the North American Datum 1983 (NAD83). Data from the LIDAR surveys were then re-projected into the NAD27 Plant datum. The LIDAR surveys proved to be problematic, as a persistent, approximately 24-foot horizontal difference (offset north) was noticed when comparing to historical data. After the LIDAR survey data were corrected, those data continued to demonstrate a slight variation from the historical NAD27 data. For the most part, these variations were corrected by TVA, but some irregularities are still apparent when the data are plotted, with deviations of one to two feet depending upon location.

In addition to the broad-scale LIDAR surveys, location data for the borings and a portion of the relics were provided “on the ground” by TVA survey personnel. TVA surveyors initially set up a Global Positioning System (GPS) base station near the intersection of the unpaved northern plant access and perimeter dike roads (east of the plant discharge channel). Traffic and site activities at this location became problematic, so the GPS base station was moved to near the center of the divider dike between the ash and settling ponds (east of Boring 09-800). The GPS base station was used in conjunction with one or more mobile GPS receivers to provide location and elevation at the various locations to sub-centimeter accuracy. In addition to TVA surveying of individual points, local benchmarks were set to provide a datum for ongoing activities such as the excavation of the trench through the south dike, University of Tennessee (UT) benchmarks on the flattened slope of the former high-wall, and the settlement plates in the Trench Ash Disposal Area near Boring 09-404. In Appendix 2H are plots of the results of the TVA Surveys of the UT benchmarks.

All historical and current survey information and data were provided by TVA with the exception of location data for most of the relics. Relic location data was obtained using a hand-held GPS for the majority of the relics located during the initial stages of the investigation. The Magellan® Triton™ 2000 GPS provided reasonable accuracy and the locations were checked against plots of the relics on an air photo. Minor adjustments to the locations were made as needed based on discussions with and reviews of air photos by field staff.

2.1.7 Geophysical Surveys

Shear wave velocity testing was performed at three locations to measure the shear wave velocity of the ash, alluvium, and the upper shale unit at the site.

<u>Crosshole Seismic Testing</u>

Crosshole Seismic (CHS) surveys were conducted in general accordance with ASTM D-4428/D-4428M. These procedures provide a means of measuring in-situ shear-wave velocity (V_S). AECOM used an SE&G mechanical, reversible downhole hammer as the *S*-wave acoustic source. Operation of the source consisted of securing the hammer to the source (S) borehole with an inflatable bladder. When the hammer is activated in an up or down direction, the energy source produces directionally distorted energy into the surrounding overburden soil and sediments to produce repeatable and identifiable *P*- and *S*-waves. When the source is activated in the opposite

direction, the energy produces S-waves with reversed polarity. By stacking multiple hammer strikes on the recording instrument (seismograph), *S*-waves are typically enhanced, whereas other seismic waves and random noise are theoretically out-of-phase and are diminished.

Multichannel Analysis of Surface Waves/Microtremor Array Measurements (MASW/MAM) Testing

The combined multichannel analysis of surface waves (MASW) and microtremor array measurement (MAM) from the in situ shear wave velocity testing was performed by an AECOM geologist/geophysicist. MASW is based on changes observed in surface wave frequency with increasing distance from the energy source. For MASW testing, the waves are initiated by a source at the surface (hammer and plate, or acoustic wave generator), and the response is measured by geophones placed at discrete intervals along a specified survey line. AECOM utilized a Geometrics Geode Ultra-light Exploration Seismograph to digitally record the shear wave records. A 12-pound sledge hammer was used to initiate shear waves at both ends of each profile.

Microtremor array measurements as described by Louie (2001) are merged with the MASW data to provide a broader frequency range from which to create shear wave velocity sections. The MAM technique is based on two fundamental principles. The first is that common seismic-refraction recording equipment, set out in a way almost identical to shallow P-wave refraction surveys, can effectively record surface waves at frequencies as low as 2 hertz. The second principle is that a simple, two-dimensional slowness-frequency (p-f) transform of a microtremor record can separate Rayleigh waves from other seismic arrivals, and allow recognition of true phase velocity against apparent velocities. Two essential factors that allow exploration equipment to record surface-wave velocity dispersion, with a minimum of field effort, are the use of a single geophone sensor at each channel rather than a geophone “group array”, and the use of a linear spread of 12 or more geophone sensor channels. Single geophones are the most commonly available type, and are typically used for refraction rather than reflection surveying. The advantages of MAM are several, including: It requires only standard refraction equipment already owned by most consultants and universities; it requires no triggered source of wave energy; and it will work best in a seismically noisy urban setting. Traffic and other construction vehicles, and possibly the wind responses of trees, buildings and utility standards provide the surface waves this method analyzes.

Inclinometer Cased Holes

The three (3) CHS cased holes (09-103, 09-211 and 09-301) were installed in a collinear alignment. The acoustic source borehole (S) was selected as one of the end test holes and the far receiver borehole (R2) as the furthest test hole. At the surface, the S borehole was positioned approximately 10 feet from the near receiver borehole (R1) and the R2 borehole was approximately 10 feet beyond R1. Inclinometer measurements were collected in each cased hole at 2-foot depth intervals from 1-to-100 feet below ground surface (bgs), and completed in each of the test holes. Inclinometer-corrected distance measurements were then used to calculate final shear wave velocities.

Crosshole Seismic Testing

For data collection and recording purposes, GeoStuff™ triaxial downhole geophones with mechanical sidearm were operated in the R1 and R2 receiver boreholes. The geophones were coupled to the inside wall of the receiver boreholes using an electromechanically-operated steel sidearm band. The reversible downhole hammer was lowered down the S borehole and triggered in the same horizontal plane as the receiver (geophone) array. The first test was conducted at 0.5 to 1 feet bgs. The remainder of the digital seismic records were recorded at 5-foot vertical depth increments to 100 feet bgs. Companion tests were also performed for quality control (QC) purposes at 20-foot depth intervals as the source and geophones were extracted from the boreholes. Multiple hammer impacts were stacked for each field record. A Geometrics Geode 24-channel exploration seismograph was used to digitally capture the field records.

Multichannel Analysis of Surface Waves/Microtremor Array Measurements (MASW/MAM) Testing

Two seismic profiles (MASW and MAM) were collected near boreholes 09-103 and 09-211 (where CHS was performed) as well as near 09-208 where surface conditions were favorable for laying out the necessary arrays and where bedrock was thought to be relatively shallow. The first profile (MASW) incorporated a 5-foot geophone spacing and maximum 30-foot source offset to maximize resolution of the 10 to 30 Hz frequency range. The second portion of the data collection (for MAM) at each location involved passively recording background noise using a 20-foot geophone spacing and creating L-shaped profiles that were 100 feet long on each leg to record data in the 2 to 15 Hz range. This L-shaped array is used to reduce directional bias in the data that might be caused by the location from which the low frequency waves originated.

After collection of the field data, the electronic files were transferred to a personal computer for detailed analysis in the office. Wave EQ shear-wave analysis software was used to process the seismic records. The processing for the shear-wave velocities included a Fourier transform of the seismic records, which generates a plot of phase-velocity versus frequency. The dispersion curve can then be identified and an initial velocity model generated. An inversion is then performed to create a velocity model that approximates the observed data.

Data Analysis - Crosshole Seismic Testing

In our opinion, crosshole seismic testing is the most reliable way to get discrete shear-wave interval velocities versus depth through a soil and rock section. The inclinometer-cased holes were installed at Boring Locations 09-103, 09-211, and 09-301. The first two locations encountered bedrock at 52.5 feet and 53.5 feet respectively. Boring location 09-301 was installed entirely in unconsolidated fill and native soil material (above bedrock). A schematic of the test layout and a summary of results are included in Appendix 2I.

At Boring 09-103, shear wave velocity testing was performed at 5-foot depth intervals starting at 1 foot bgs. Table 1 in Appendix 2I shows the calculated shear wave velocities for the crosshole tests. The unsaturated near-surface material had shear-wave velocities ranging from 844 to 908 feet per second (ft/s). The saturated ash (11, 16 and 21-foot depth intervals) shows very slow shear-wave velocities of 425 to 488 ft/s. In the 26, 31 and 36-foot depth intervals the velocity appears to increase to 604 to 704 ft/s where the soil description indicates bottom ash. The natural silty clay below the ash at 41 feet bgs showed a shear wave velocity of 757 ft/s. The 46-foot depth interval (in native sandy silt) showed a velocity of 498 ft/s. Overall the shear wave velocity increased from 425 to 757 ft/s through overburden materials which had Standard Penetration Test (SPT) blowcounts (N-values) ranging from 1 to weight-of-hammer (WOH), and weight-of-rods (WOR) for 1 foot or more penetration. No shear waves were observed in bedrock for seismic records below 46 feet. See the paragraph at the end of this section for a discussion of this observed condition. The piezocone penetrometer testing (CPT_u) performed at this location agrees with the descriptions and SPT N-values for the materials observed.

At Boring Location 09-211, CHS data was collected through an undisturbed section of Dike C fill material, emplaced ash and underlying natural sediments. The shear wave velocity of the upper clay fill (dike material) varied from 1,988 ft/s in the highly compacted surface material, to 922 to 1,076 ft/s down to 15 feet bgs. Shear wave velocities varied from 815 to 1,334 ft/s in the bottom ash noted from 15.3 to 24 feet bgs. A sandy clay layer was observed in SPT Boring 09-211 from 25 to 28 feet and CHS testing in this interval calculated a shear wave velocity of 1,334 ft/s. Clayey silt and silty sand was noted in the 09-211 boring log from 28 to 37.5 feet bgs and the calculated shear wave velocities were 834 to 856 ft/s respectively. Well-graded sands were observed from 37.5 to 53.5 feet bgs and the shear wave velocity varied from 654 to 729 ft/s. This decrease in shear wave velocity corresponds with the zone from 32 to 48 feet bgs where SPT N-values ranged from WOH for 1.0 foot to one hammer blow for 2.5 feet (N-value < 1). The CPT_u performed at this location shows tip resistances that correspond with the SPT N-values and material classifications on the boring log for this boring. Similar to the CHS test performed at 09-103, no shear waves were able to be measured below the bedrock interface at 53.5 feet (see below).

At Boring 09-301, a CHS test was performed from the top of the remaining intact Cell No. 1 ash pile. The cased holes are entirely in unconsolidated materials (unfailed ash). The shear wave velocities vary from 450 to 651 ft/s from 0 to 50 feet in the sluiced fly ash. Low SPT N-values through the unfailed ash correspond with the low shear wave velocities. A zone of harder materials described as bottom ash correlates with an increase in shear wave velocity (850 to 877 ft/s) from 50 to 60 feet. Deeper portions of fly ash show velocities of 672 to 911 ft/s. Softer natural clays below 90 feet bgs showed shear wave velocities of 619 to 718 ft/s.

Data Analysis - CHS Testing in Bedrock

It is possible that conditions exist in the bedrock that caused borehole sidewall softening or a layer of high water content material or smeared drill cuttings to be formed around the grout placed within the annular space adjacent to the inclinometer casing. If the material around the well casing is fluid-like, there can be little or no transmission of the shear wave. In an attempt to gather data at intervals which may still contain competent grout below the bedrock surface, the CHS array was collected with the source at either end of the well array as well as shot from the middle inclinometer casing. Further, testing was performed at 2.5 ft intervals below the elevation where shear wave loss was observed in case some zones of grout remained intact. No shear waves were measured or detected during the additional testing. The compression wave (p-wave) is still visible so equipment malfunction was ruled out. Additional troubleshooting included shifting the recording time to start before the trigger in case a timing issue existed. Analysis of these records determined that there was no “missing” signal before time zero. A summary of the shear wave velocities from cross hole seismic testing is presented in Table 2.1.7_1 in Appendix 2I.

Data Analysis - MASW/MAM Testing

The MASW/MAM methods are not a direct measurement of the s-wave (i.e., shear wave) at discrete depth intervals, but rather an average across the length of the seismic array. Therefore, modeled sections are expected to vary from direct interval measurements as would be performed in CHS surveys.

The results of the MASW/MAM surveys are tabulated as layer velocities and are shown on Table 2 in Appendix 2I. The shear wave velocities range from 360 fps in loose near-surface materials to 1,790 ft/s at a depth of 100 feet below grade. The average shear wave velocities over the 100-foot-deep profiles at 09-301, 09-211, and 09-208 are 850, 1,210, and 1,100 ft/s respectively. The calculated 100-foot average for Boring 09-211 may be slightly higher due to space considerations near the borehole. The arrays were collected in a straight-line configuration only because there was not enough space to lay out an L-shaped array for the MAM. Wave fronts that encounter the array at a high angle appear to pass through the array faster than waves that originate off the ends of the array.

The modeled section created for Boring 09-103 shows shear wave velocities that correlate with the materials described in the boring log. A low velocity is shown from 8 to 11 feet bgs where very loose saturated ash material was noted on boring log. The modeled velocity is 150 ft/s and is probably lower than the actual shear wave velocity. Shear wave velocities modeled for the fly ash vary from 360 to 630 ft/s with the majority of the velocities falling in the 500 to 550 ft/s range. The average velocity in the natural silts and clays increases slightly and varies from 670 to 850 ft/s. Bedrock shale velocities vary from 1,020 to 1,330 ft/s

The tested MASW/MAM section for Boring 09-211 also shows good correlation between model velocities and the described material and standard penetration resistance numbers. The clayey dike material velocities vary from 650 to 833 ft/s in the upper 12 feet. A hard zone of bottom ash was noted from 15 to 20 feet bgs and is seen on the velocity model as a 900 to 950 ft/s layer. Extremely soft and/or loose materials having SPT N-values of WOH or N-values less than 8 blows per foot consisting of a thin layer of fly ash over natural clays and sands. The modeled velocities for these layers vary from 510 to 670 ft/s. The bottom 10 feet of the natural sands show shear wave velocities of 970 ft/s, although the material still has very low SPT N-values. Computed shale bedrock velocities increase quickly with depth from 1,200 to 1,790 ft/s

An additional MASW/MAM survey was conducted near the Boring 09-208 to obtain more shale bedrock velocities. This location had enough relatively flat ground to layout the MASW and the MAM L-shaped array. The ground surface elevation for the array was about 5 feet higher than the borehole location and the elevations shown on Table 2.1.7_T2 in Appendix 2I have been adjusted accordingly. The velocity of the fly ash in the upper 6 feet varied from 260 to 350 ft/s in the model. The natural clayey sand and sand velocities vary from 420 to 780 ft/s and show an increase in velocity with depth. The velocity increases gradually into the bedrock and varies from 880 ft/s near the bedrock interface to 1,410 ft/s at 92 feet bgs. A summary of the shear wave velocities from the MASW/MAM are presented in Table 2.1.7_2 in Appendix 2I.

Summary

The following points summarize the findings of the surveys performed to determine shear wave velocities.

- CHS testing was performed in inclinometer-cased holes at 3 locations within the dredge cell area.
- No shear waves from the CHS tests were observable below the top of the bedrock surface due to problems in the casing/grout interface with the drilled shale.
- The testing program was supplemented by MASW/MAM surface wave methods to obtain shale bedrock shear wave velocity information.
- Velocity ranges observed for the various materials at the site are as follows:
    - Clayey dike material - 650 to 900 ft/s;
    - Loose surface materials and fly ash - 250 to 680 ft/s; bottom ash 780 to 1,040 ft/s;
    - Natural clays below ash - 420 to 800 ft/s;
    - Natural sands above shale bedrock - 700 to 950 ft/s; and
    - Fractured shale bedrock - 883 to 1,787ft/s.
- In general, there is a strong correlation between the CPT_u resistances across the site. The shear wave velocities also correlate well with variations shown on the boring log (i.e., material descriptions). The SPT N-values show a general conformance to shear wave variations in the section but still vary considerably when in materials has SPT readings of weight of rod or hammer (WOH or WOR).

- The MASW/MAM testing performed at 09-211 shows shear wave velocities that are 25 to 35 percent higher than the tests performed at Borings 09-301 and 09-208. This may be a directional bias in the deeper values caused by using a MAM array that had to be laid out in a straight line and not an "L" shaped array due to space considerations next to wet ash.
- The MASW/MAM modeled average velocities through a large zone of material and as a result, is less precise than CHS testing at discrete depth intervals. The CHS should be relied upon for shear wave velocities above bedrock and MASW/MAM data used to shear wave velocities in shale bedrock.

See the attached tables and figures that summarize the collected data versus depth and test method. There is some data scatter due to variations between failed and unfailed ash and other geologic variability.

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Appendices

Appendix 2A – Relics Location Data

Appendix 2B – Geology

Appendix 2C – CPTu Soundings

Appendix 2D – CPTu Dissipation Tests

Appendix 2E – SPT Soil Borings

Appendix 2F – GRL Report

Appendix 2G – Vane Shear Tests

Appendix 2H – Site Instrumentation

Appendix 2I – Geophysical Surveys

Appendix 2A - Relics Location Data

Atterberg Limits

Dike C Laboratory Results

Grain Size Analysis

Dike C Laboratory Results

Appendix 2B - Geology

Appendix 2C - CPTu Soundings

Appendix 2D - CPTu Dissipation

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