APPENDIX F

Workplan – Groundwater Monitoring

Appendix F

Samples will be collected according to procedures detailed in TVA's Quality Assurance Procedure *Groundwater Sample Collection Techniques* (attached to this appendix). An abbreviated summary of these procedures include the following:

1. The elevation of groundwater will be measured prior to sampling.
2. The volume of the water in the well will be calculated, in liters, from measurements of depth to water surface and total depth of the well. If there is insufficient water in a well for pumping, bailers may be used for purging and sampling.
3. The pump will be carefully lowered to approximately 0.5 meters below the water surface before pumping begins. The pump will be lowered with the drop in water surface. This ensures that no stagnant water remains in the well after pumping. Ideally, at least two well volumes of water should be purged before sampling. For wells with slow recharge, the pump rate will need to be reduced to minimize the drawdown of the level in the well, if possible. If insufficient water for sampling exists after purging, the wells can be allowed to recover, but sampling should take place as soon after purging as possible.
4. While pumping, temperature, pH, DO, ORP, and conductivity will be continuously monitored using a calibrated Hydrolab® flow through cell system to avoid air contact. These data will be recorded on form TVA 30066A approximately every five minutes. When the Hydrolab® readings have stabilized and at least two well volumes have been pumped or bailed, unfiltered samples will be collected for the parameters listed in Table 1.
5. Special care will be taken with wells that produce turbid samples. For wells producing turbid water at the time of sampling, water will be allowed to settle in the well before collecting samples, or extra containers filled, kept on ice, and allowed to settle up to two hours. When the particulates have settled, all bottles required will be carefully filled.
6. Samples will be shipped on ice by TVA mail and/or public carrier to TVA's Environmental Chemistry laboratory. Samples not meeting holding times will be rejected and new samples collected.

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TENNESSEE VALLEY AUTHORITY

ENGINEERING SERVICES

QUALITY ASSURANCE PROCEDURE

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No. ES-41.6

Title: GROUNDWATER SAMPLE COLLECTION TECHNIQUES

Revision:	0
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Title: GROUNDWATER SAMPLE COLLECTION TECHNIQUES

No. ES-41.6
Page 1 of 1

REVISION LOG

Rev. No.	Date Approved	Revision Description	Reason for Revision
0	4/29/94	Procedure ES-41.6 replaces DS-41.6. Title and organizational changes made.	To reflect reorganization

TVA 30037 (S&F OPS 5-88)

1.0 OBJECTIVE

To prescribe specific, detailed instructions for Engineering Services (ES) personnel involved in the collection of water samples in accordance with standard practices generally accepted by the U.S. Environmental Protection Agency (EPA), U.S. Geological Survey (USGS), and TVA.

2.0 SCOPE

The techniques described herein are limited to those to be used by ES personnel for routine studies. They do not apply to special studies that may require special apparatus and/or handling or specially trained personnel. For example, the collection of groundwater samples at Comprehensive Environmental Response, Compensation, and Liability Act (CERCLA) sites (i.e., "Superfund" sites), certain Resource Conservation and Recovery Act (RCRA) sites, and those activities which fall under the scope of the Superfund Amendments and Reauthorization Act (SARA) of 1986 are not within the scope of this procedure. This procedure applies to collection of routine groundwater samples in connection with TVA's regional water management program activities and assessment of groundwater quality in the vicinity of TVA power facilities.

3.0 REFERENCES

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3.2 Handbook--Groundwater, Environmental Protection Agency, EPA/625/6-87/016, Cincinnati, OH, 1987.

3.3 A Guide to Groundwater Sampling-Technical Bulletin No. 362, National Council of the Paper Industry for Air and Stream Improvement, Inc., New York, NY, 1982.

3.4 Practical Guide for Groundwater Sampling, Environmental Protection Agency, EPA/600/2-85/104, Ada, Oklahoma, 1985.

3.5 Macrodispersion Experiment Management Policies and Requirements (EPRI RP 2485-05), TVA Engineering Laboratory Report No. WR28-2-520-136, Chapters 4.2.6, "Field Tracer Sampling," and 4.2.7, "Field Monitoring and Sampling," 1987.

3.6 Fletcher G. Driscoll, Groundwater and Wells, Johnson Division, St. Paul, Minnesota, Second Ed., 1982.

3.7 40 CFR 136, "Guidelines Establishing Test Procedures for the Analysis of Pollution," Table II - Required Containers, Preservation Techniques, and Holding Times.

3.8 Methods for Chemical Analysis of Water and Wastes, Environmental Protection Agency, EPA-600/4-79-020, Cincinnati, OH, 1979.

3.9 Standard Methods for the Examination of Water and Wastewater, 18th Ed., American Public Health Association, Washington, D.C., 1992.

3.10 Handbook for Sampling and Sample Preservation of Water and Wastewater, Environmental Protection Agency, EPA-600/4-82-029, Cincinnati, OH, 1982.

3.11 Sampling Guidelines for Groundwater Quality, Electric Power Research Institute, EA-4952, Research Project 2485-1, Palo Alto, CA, 1987.

3.12 Groundwater Manual for the Electric Utility Industry, Electric Power Research Institute, CS-3901, Research Project 2301-1 (volumes 1, 2, and 3), Palo Alto, CA, 1985.

.12.1 Volume 1: Geological Formations and Groundwater Aquifers.

3.12.2 Volume 2: Groundwater Related Problems.

3.12.3 Volume 3: Groundwater Investigations and Mitigation Techniques.

3.13 Resource Conservation and Recovery Act (RCRA) Groundwater Monitoring Technical Enforcement Guidance Document, Environmental Protection Agency, PB87-107751, OSWER-9950.1, Washington, D.C., 1986.

3.14 ES-41.1, "Collection and Handling of Samples."

3.15 ES-41.2, "Water Sample Collection Techniques."

3.16 ES-41.4, "Trace Organics Sample Collection Techniques."

3.17 ES-42.1, 42.3, 42.4, 42.7, 42.8, and 42.11, "Water Quality Field Analyses."

3.18 ES-43.1, 43.2, 43.3, 43.7, and 43.8, "Standardization of Field Instruments."

3.19 ES-5.20, "STORET - Water Quality Data Management."

.20 Lysimeter Evaluation Study, American Petroleum Institute, Publication No. 4433, 1986.

3.21 Handbook of Groundwater Development, Roscoe Moss Company, Los Angeles, California, Published by John Wiley and Sons, 1990.

4.0 ABBREVIATIONS AND DEFINITIONS

4.1 Definitions

4.1.1 Definitions of job titles and general responsibilities of managerial and supervisory personnel in ES are given in section 5.0.

4.2 Abbreviations

4.2.1 BOD--Biochemical Oxygen Demand

4.2.2 DO--Dissolved oxygen

4.2.3 CHATT ENGG--Chattanooga Engineering Services

4.2.4 Dw--Depth of well in meters

4.2.5 Dws--Distance to water surface from top of well R.P. in meters

4.2.6 EDM--Environmental Data Management (CHATT ENGG)

4.2.7 ES--Engineering Services

4.2.8 ENVIR CHEM--Environmental Chemistry, Water Management Services

4.2.9 EPA--United States Environmental Protection Agency

4.2.10 MLS--Multilevel sampling well

4.2.11 NPDES--National Pollutant Discharge Elimination System

4.2.12 ORP-Oxidation-reduction potential (REDOX)

4.2.13 pH--Measure of hydrogen ion concentration

4.2.14 QAC--Quality Assurance Coordinator

4.2.15 R.P.--Reference Point

4.2.16 USGS--United States Geological Survey

4.2.17 Vw--Volume of water in well measured in liters

5.0 RESPONSIBILITIES

5.1 Functional Area Manager--The manager responsible for various functions such as field engineering projects in a geographical area (i.e., eastern, central, or western geographical locations). The manager directly supervises project engineers and team members in his geographical area.

5.2 Project Engineer--The person responsible for a particular area of expertise, subfunction, or specific projects within the geographical area. The project engineer assists and reports directly to the functional area manager, advises and acts as a resource to teams within his area of expertise and provides technical help to other teams as needed.

5.3 Technical Lead Engineer--The person responsible for a particular project(s) or tasks. These responsibilities include coordination with client organizaion(s), workplan preparation, budget estimates, scheduling of field studies to meet project deadlines, technical adequacy of the work performed and report preparation. All of these lead engineer responsibilities are assumed by team members for their own support of the team.

5.4 Quality Assurance Coordinator--The QAC is the functional area manager or his designate and is responsible for Engineering Services procedures that are assigned to that functional area. The QAC assigns a technical writer or reviewer for each procedure. The QAC assures that procedures are correct and up-to-date by requiring technical writers and reviewers to certify in writing on a yearly basis that assigned procedures have received a thorough review. The QAC works closely with the organization Quality Assurance Manager.

5.5 Survey Leader--The survey leader is the individual responsible for a particular piece of work. This individual is responsible for seeing that field work is performed in a technically adequate, timely, and safe manner. The survey leader is responsible for the equipment and supplies; technical supervision of personnel while in the field; collection, handling, and shipping of samples. The survey leader, more than any other person, is responsible for being familiar with the procedures. The survey leader reports directly to the lead engineer for which the work is being done.

5.6 Engineering Services personnel--Personnel assigned to a particular work activity or team. Responsible for conducting tasks in a technically adequate manner and for following QA procedures. Any certification must be current for collection or handling samples (i.e., radiological, hazardous waste, water quality, etc.).

5.7 The Environmental Chemistry Lab, Water Management Services (ECHEM), performs chemical, and physical analyses.

5.8 CHATT ENGG EDM is responsible for coding, keypunching, processing, reviewing, validating, retrieving, and reporting field and laboratory data related to ambient groundwater quality.

6.0 PROCEDURES/REQUIREMENTS

6.1 Workplans

6.1.1 A written workplan is usually prepared in advance of the sampling activities. This written workplan must be coordinated with the client organization and other service organizations. The workplan must receive concurrence by all affected organizations and will address, at a minimum, the purpose of the monitoring activities, the choice of water characteristics to be measured, the method or methods to be employed in collection of the samples, the locations and frequency of sampling, project deadlines, schedules, parameters to be analyzed by the laboratory, budget requirements, and collection of auxiliary data.

6.1.2 If special sample collection requirements, handling techniques, or analyses are required (other than the standard procedures contained in this manual), they will be spelled out in detail in the workplan or in supplemental procedures. All items which will affect the quality of the data to be collected must be addressed in the written workplan and/or referenced to the appropriate ES procedures. The written workplan must be approved by the lead engineer prior to any fieldwork. Also, any workplan revisions must be approved by the lead engineer prior to any field activities associated with a particular workplan revision.

6.2 General Requirements and Instructions for Groundwater Sampling

6.2.1 "Collection and Handling of Samples" (reference 3.14) will be followed as appropriate. In addition, particular attention must be given to the following requirements.

6.2.2 The survey leader will review the workplan in detail and consult with his or her lead engineer prior to the first survey to ensure that no misunderstanding exists about how, when, where, and what samples are to be collected.

6.2.3 Before starting a new work activity at a TVA facility (i.e., nuclear, steam, hydro, etc.), the survey leader will contact the facility manager or his/her designee (usually the Results Section supervisor at a steam plant) and inform them of the work to be performed and on what schedule it will be done. To ensure recognition of any situations which may require special safety awareness, the survey leader will communicate with the plant manager or his/her designee and discuss safety procedures which need to be observed, unusual conditions to be aware of, and names of ES personnel working at the TVA facility.

6.2.4 The survey leader will select and assemble the needed equipment (pumps, meters, Hydrolabs, filtration apparatus, tapes/plunkers, compressor, generators, titration equipment, pH/conductance/ORP standards, buckets, etc), sample containers, workplan, maps, well driller logs, and forms and field worksheets. The survey leader will ensure that all equipment and supplies are appropriately cleaned, in good working order and within their laboratory calibration interval as specified in ES-43.1, attachment 1 (reference 3.18). It is recommended that an equipment checklist be prepared on the initial field survey and that it be referred to and updated on each subsequent survey.

6.2.5 The survey leader may obtain a summary of the last four sets of field data for use to validate and compare information at the time it is being collected. A computer printout can be obtained from CHATT ENGG-EDM to facilitate this data validation process.

6.2.6 Generally, the survey leader should monitor the wells in a particular order as determined by their typical pH values. For instance, all wells below a pH of 7.0 should be sampled, then all wells above a pH of 7.0 should be sampled. The monitoring equipment should be restandardized between the two ranges of wells using the appropriate pH buffers.

6.2.7 Also, water levels of the wells and reference points should be measured prior to any sampling and recorded. These measurements should be made in as short of a time interval (hrs.) as possible. These "snapshot measurements" should be converted to water level elevations (meters above MSL). Both values should be recorded in a table and presented with well/R.P. description, time of measurement, and depth to well bottom (in meters) along with any pertinent remarks.

6.3 <u>Groundwater Sample Collection Techniques</u>

6.3.1 <u>Quality Control of Sampling Operations</u>

3.1.1 Every effort will be made to collect a representative and uncontaminated sample. After each sample is collected, it will be visually examined for any foreign material that is not representative. If any foreign material is observed, or suspected, the sample will be discarded and new sample recollected in a fresh sample container. Do not immerse anything--even a thermometer--in the sample. Always pour the sample directly into the specified containers one at a time. Transferral to another container will greatly increase the opportunity for contamination and cross contamination.

6.3.1.2 Many sample containers contain chemical preservatives. These preservatives may be a source of contamination to other samples, may be ineffective if diluted, or may be harmful if allowed to contact skin or eyes. Use care when handling sample containers with chemical preservatives. Fill sample containers individually, one at a time, to prevent cross contamination of preservatives: uncap the container, fill it directly from the sampler, and recap the sample container immediately. Do not place flexible sample tubing inside the containers unless specifically instructed to do so. Do not lay caps on surfaces that might contaminate them. Do not overfill containers. If any of these potential sources of contamination occur, discard the affected portion of the sample, and collect another portion in a fresh container.

6.3.1.3 Sample collection methods for groundwater may include the use of a submersible centrifugal pump, pneumatic bladder pump, single or 10-channel peristaltic pump, check valve bailer, lysimeter, or perhaps a gas lift pump. The method used to collect a groundwater sample must be compatible with the water quality characteristics of interest. All of these methods, in one or more ways, alter the quality of the sample while it is being collected. In most instances, the submersible centrifugal (low flow, variable speed) pump, the pneumatic bladder pump, or check valve bailer, when used properly, will collect the most representative (least altered) sample for a variety of constituents (particularly volatile organics and reduced/dissolved species). The use of gas lift devices for collection of groundwater quality samples is not recommended. Chapter 6 of reference 3.2 provides additional details.

6.3.1.4 When collecting groundwater samples, the sample should be obtained as close to the discharge of the source or wellhead as possible to reduce the potential for contamination, precipitation of solute, and loss of dissolved gasses. Treated (chlorinated or filtered) or stored groundwater samples, such as from some private or domestic wells are of limited value. Care must be taken to limit sample contact with air and agitation that would interfere with the field determination of pH, ORP, dissolved gasses, acidity, and alkalinity, or the laboratory determination of volatile organics and reduced species.

6.3.1.5 On occasion it may be desirable to determine concentrations of dissolved inorganic constituents (i.e., dissolved minerals or dissolved metals) in groundwater. In such cases, by definition, the sample is filtered through a 0.45 mm average pore diameter cellulose ester membrane filter (Millipore Cat. No. HAWP04700 or equivalent) during (pressure filtration) or immediately after (vacuum filtration) sample collection. Techniques used to filter groundwater samples should be discussed in detail in the project's workplan. In most cases, the preferred method for filtration of groundwater is an "in-line" pressure filtration technique which eliminates sample contact with the atmosphere and utilizes the sampling pump's pressure for filtration. <u>The field worksheets and request for laboratory analysis forms must clearly indicate when samples are filtered in the field</u>. Also, all bottles must be properly marked for which constituent the sample was performed (e.g., DM, dissolved metals and etc.). Samples for field analysis (temperature, DO, pH, conductance, ORP, alkalinity, etc.) and certain laboratory analyses (ferrous and manganous ions, sulfide, organics, turbidity, suspended solids, etc.) are never filtered. Additional details in regard to sample filtration procedures are given in section 6.2.2 of reference 3.15.

Condition the filter prior to sampling with 200 to 300 mL of deionized, distilled water (Super Q). This hydrates the filter to lessen the chance of channelization through the filter during sampling. Collect a filter blank with Super Q water after conditioning at the frquency specified in section 6.3.1.7. If filtration difficulties are anticipated because of high solids concentrations, try to develop the well to reduce the level of solids. If too much mud is still present, measure the Hydrolab parameters and pump up as much sample as possible. Let it stand in a sealed, clean container, and decant enough sample for filtering.

6.3.1.6 Samples collected for extremely low levels (i.e., less than one part per billion) of trace organics and/or trace elements may easily be contaminated by contact with foreign materials. Motor oil, gasoline, soft plastics, etc., may be potential sources of contamination for trace organic/pesticide sampling, while soil and dust, which is ubiquitous at fossil plants, may be potential sources of contamination for many trace elements. Reference 3.16 and section 6.3.3.5 below discuss routine precautions which are taken to minimize potential sources of contamination. The permanent installation of a groundwater sampling device in each monitoring well has many advantages. It will eliminate the possibility of the introduction of foreign material during the lowering of sampling equipment into the well and the potential for cross contamination between wells caused by the possible carryover of contaminants on the sampling equipment from one well to another. In those cases where special attention must be paid to extremely low levels of organics or trace elements, permanent installation of sampling equipment/pumps in each groundwater monitoring well is recommended.

6.3.1.7 Unless otherwise specified in the project's workplan, duplicate groundwater samples will be collected at every 20th well (i.e., five percent site specific of the samples collected). Further details in regard to collection of duplicate samples are given in section 6.15.3 of reference 3.14. Also, filter blanks shall be taken when dissolved samples are collected.

6.3.2 Standardization of Field Equipment and Field Measurements

6.3.2.1 ES procedures for standardization of field instruments (reference 3.18) must be followed, as appropriate, with particular attention given to the following instruments which are commonly used by ES in the collection of groundwater quality samples.

6.3.2.1.1

Field Instruments (reference 3.18)	ES Procedure
Hydrolabs	ES-43.2
YSI Conductance Meters	ES-43.3
Orion pH Instruments	ES-43.7
Thermometers	ES-43.8

6.3.2.1.2 Field instruments will be standardized as specified in the above referenced procedures. At a minimum, instruments will be standardized before and after field measurements are made and whenever the accuracy of the instrument is questioned. Form TVA 30035, "Instrument Standardization, Field Standardization of Instruments," will be completed to document all field standardizations of instruments.

6.3.2.2 ES procedures for water quality field analyses (reference 3.17) must be followed, as appropriate, with particular attention given to the following analyses which are commonly used by ES in the collection of groundwater quality samples.

6.3.2.2.1

Water Quality Field Analyses (reference 3.17)	ES Procedure
Alkalinity and Acidity (Ref. Attachment 6 for summary worksheet)	ES-42.1
Total and fecal coliform bacteria	ES-42.2
Conductance	ES-42.3
Dissolved Oxygen (DO)	ES-42.4
Oxidation-Reduction Potential (ORP)	ES-42.7
pH	ES-42.8
Temperature	ES-42.11

6.3.3 Collection of Well Samples Using a Submersible Pump

6.3.3.1 To obtain a representative sample of groundwater, it must be understood that the composition of the water within the well casing and in close proximity to the well is probably not representative of the overall groundwater quality at the sampling site. This is due to the possible presence of drilling contaminants near the well, introduction of foreign material from the surface, casing corrosion, and/or because environmental conditions such as the oxidation-reduction potential (ORP or REDOX) may differ drastically near the well from the conditions in the surrounding water-bearing materials. Consequently, each well must be flushed (purged) of standing (i.e., stagnant) water until it contains fresh water from the surrounding aquifer. The recommended length of time required to pump a well and the rate at which a well can be pumped before sampling are dependent on many factors including the physical characteristics of the well, the hydrogeological nature of the aquifer (i.e., hydraulic conductivity), the type of sampling equipment being used, and the water quality parameters of interest.

6.3.3.2 Prior to any sampling or pumping of a well, measure and record the distance to the water surface (Dws) with an acoustic or electric plunker. Also measure and record the depth of the well (Dw) on each survey. Do not rely on past well depth data, since the well may be silting in. Depth measurements (measured to the nearest 0.01 meter i.e. nearest cm.) are usually referenced to the top of the inner well casing and not the outer protective casing. All data, measurements, observations, and computations are to be recorded on form TVA 30066A, "Groundwater Quality Data Field Worksheet (Chemical Data)," attachment 1. In addition, if the well to be sampled is a new well or has never been sampled, form TVA 30066B, "Groundwater Quality Data Field Worksheet (Physical Data)," attachment 2, which documents information about type of well, owner of well, location of well, well drillers log/information, etc., must also be completed.

6.3.3.3 Calculate the volume of water in the well as shown below:

Well Casing ID (mm)	Liters Per Meter
51	2.027
76	4.560
102	8.107
127	12.668
153	18.228

$$Vw \text{ (in liters)} = (Dw - Dws) \times \text{liters/meter}$$

where:

Vw = Volume of well, liters;
Dw = Depth of well, meters; and
Dws = Depth to water surface, in meters

6.3.3.4 If a submersible pump is not already permanently installed, such as might be the case at "dedicated" pump wells, private or domestic wells, the preferred method of purging and sampling a well is to use a low flow (variable speed controlled) centrifugal pump, a pneumatic bladder pump, or a peristaltic pump (shallow wells). However, in situations where large volumes of water must be purged from a well, resulting in long pumping times (i.e., greater than one hour), a centrifugal pump with a higher pumping capacity (4 to 16 liters per minute) may be used for purging only instead of the lower capacity bladder pump (1-3 liters per minute). All such cases should be specifically addressed in each project's workplan. Domestic wells with a submersible pump already permanently installed can be sampled from a convenient tap or faucet after letting the water run for several minutes.

6.3.3.5 Prior to lowering the pump into the well, (when advantageous) a large tarpaulin or heavy sheet of plastic should be spread on the ground to cover the necessary portion of the work area. This "good housekeeping" practice will help minimize the potential for contamination caused by contact of the soil with the pump and/or pump tubing. Immediately prior to placing the pump into the well, rinse the outside of the pump and the first meter of pump tubing with deionized water. Successive lengths of pump/sample tubing shall be rinsed/wiped with deionized (DI) water before insertion into the well casing.

6.3.3.6 Carefully lower the pump intake to approximately 0.6 to 1.3 meters below the water surface (dependent upon the length of the pump head). The pump should not be lowered below the top of the well screen or to the bottom of the well unless specific instructions to do so are given in the workplan. Studies have shown that lowering the pump to the bottom of a well (below the well screen) may result in a poor flushing of the column of water above the pump if the transmissivity of the aquifer is high. In such cases the pump would be primarily removing inflowing water from the lower portion of the well casing and not effectively removing the water in the upper water column. Pumping from near the surface (and lowering the pump with the drop in the water surface) ensures that inflowing water moves up through the water column and that no stagnant water will remain in the well after purging. The past performance of a well should be used to indicate the appropriate steps for lowering the pump. If the well's recharge rate is slow, the pumping rate will need to be reduced to minimize the drawdown of the water level in the well, or in extreme cases the well maybe completely evacuated ("pumped dry") and allowed to recharge overnight before sampling. At no time should the water level be drawn below the top of the well screen, unless dictated by a very slow recharge rate, requiring "next day" sampling.

6.3.3.7 While purging the well, continuously monitor the time, pumping rate, and distance to water surface. The pumping rate should be adjusted (when possible or reasonable) to minimize the drawdown of the water surface in the well. Using a Hydrolab flow-through cell system to avoid groundwater-air contact, also monitor the groundwater's temperature, pH, DO, conductance, and ORP. Record all the stabilization test data on form TVA 30066A, "Groundwater Data Field Worksheet," attachment 1, approximately every five minutes or less if purge time is expected to be of a short duration. At each well, while recording and monitoring the field stabilization test data (i.e., pumping rate, water surface, temperature, pH, DO, conductivity, and ORP), the survey leader will compare the data being collected with previously collected field data. A computer printout of the last four sets of field results, obtained from the CHATT ENGG, will facilitate this comparison and ensure, on the spot, that valid and comparable data are being obtained.

6.3.3.8 Unless otherwise stated in the workplan, when at least two well volumes of water have been purged from the well and the Hydrolab readings (temperature, pH, DO, conductivity, and ORP) have stabilized, (i.e., do not change by more than 5 percent or have essentially ceased any obviously upward or downward trend between readings), samples may be collected. If the water quality readings have not stabilized after removal of two well volumes, remove a third well volume (if conditions permit), then begin sampling. When filling the various sample bottles/containers, care must be taken to minimize sample aeration, and to gently fill each bottle. This will often necessitate the lowering of the pumping rate to less than one liter per minute to avoid the turbulence caused by the high velocity of the water as it is discharged from the pump tubing. Be sure to record the pumping rate, temperature, pH, DO, conductivity, ORP, etc., at the time of sample collection and record the distance to the water surface immediately upon completion of sampling.

6.3.3.9 If the well's recharge is slow, the pumping rate will need to be reduced to minimize the drawdown of the water surface level in the well. If a well becomes dry during the purging, it must be allowed to recover before sampling to avoid taking a nonrepresentative sample. It may be necessary to allow 24 hours or longer for recovery. If circumstances are encountered which are not addressed in this procedure or in the project's workplan, notify the lead engineer immediately for instructions.

6.3.3.10 After purging and sampling, sample water should be removed from the pump and tubing before sampling another well. A centrifugal pump should have the check valve removed so that water will drain back into the well when the pump is turned off. Before reuse of any pump/sample tubing at any successive well, place the pump head in a container of deionized water (Super Q) and pump through two line volumes of Super Q water to flush the pump and lines thoroughly. NOTE: The "DI" flush water must be removed with two line volumes of sample water. The outside of lines should be wiped with a clean rag or paper towel soaked with DI water. This process shall be repeated at each well that is sampled.

6.3.4 Collection of Samples Using a Bailer

6.3.4.1 Prior to sampling a well with a bailer, measure and record the distance to the water surface and the depth of the well as given in section 6.3.3.2.

6.3.4.2 Calculate the volume of water in the well as shown in 6.3.3.3.

6.3.4.3 Prior to sampling a well with a bailer, thoroughly flush the sampler with deionized water. (As an alternate method, a pre-cleaned disposable Teflon bailer may be used.) Carefully lower the sampler to the water surface. Do not drop the sampler or let it free fall to the water surface, as this will cause aeration of the sample. Gently lower the sampler into the water. Retrieve the bailer. Repeat this process until two well volumes of water have been removed or as specified in the project's workplan.

6.3.4.4 Collect the samples by carefully lowering the sampler to the well screen or the perforated section of the well casing or to the depth specified in the workplan. Care should be taken to avoid striking the bottom of the well with the sampler.

6.3.4.5 Fill the specified bottles/containers directly from the sampler. Slow and careful transfer is important to minimize sample aeration. When filtered samples are requested, use a bailer fitted with an in-line filter. Measure and record temperature, pH, DO, conductivity, ORP, and the distance to the water surface immediately after collection of the sample.

6.3.5 Collection of Samples From Multilevel Sampling (MLS) Wells

6.3.5.1 A typical MLS well, see attachment 3, will consist of several (often 20 to 30) small diameter, flexible sampling tubes. Each tube will have a filter, usually a nylon mesh, on the intake end of the tube with the intake ends of these tubes spaced at known distances below the ground surface. These flexible sampling tubes are housed and extend to the surface inside a PVC pipe as shown in attachment 3.

6.3.5.2 Groundwater samples will be collected from MLS wells using peristaltic 10-channel pumps (i.e., two 10-channel pumps for 20 flexible sampling tubes, three 10-channel pumps for 30 flexible sampling tubes, etc.). In all sample collections from MLS wells, the 10-channel peristaltic pumps will be used in parallel to purge all tubes and collect all samples simultaneously. Every effort will be made to collect representative and uncontaminated samples. An important consideration in obtaining a valid, representative sample is first the removal of the standing water which has been trapped in the multilevel flexible sample tubing since the last sample collection. However, to avoid stressing the aquifer and perhaps altering its natural movement, this purging of the trapped water in the tubing will be minimized. One of the reasons for using the small diameter flexible tubing is that it minimizes the amount of water which is purged. For example, one meter of 5 mm ID tubing contains approximately 19.6 mL of water. Therefore, the purging of two tubing volumes would result in the purging of approximately one liter of water from each sample tube (assuming 25 meter lengths of 5 mm ID tubing) prior to collection of the samples. Specific purging instructions for individual MLS wells will be detailed in each project's workplan.

6.3.5.3 To collect samples at MLS wells, connect the MLS flexible sampling tubes to the 10-channel peristaltic pump tubes by mating like numbered (colored) tubes number 1 through 30 (assuming there are 30 flexible sample tubes and that three 10-channel pumps are used).

6.3.5.4 Place waste containers beneath each sampling tube, turn on the 10-channel peristaltic pumps, and simultaneously purge all the sample tubes of stagnant water by pumping approximately two volumes of water from each sample tube. (One meter of 5 mm ID tubing contains approximately 19.6 mL of water.) Discard the purge water as appropriate or as outlined in the customer's request documentation. Record on the field worksheets any tubes which do not produce water or produce only small quantities of water.

6.3.5.5 After purging the MLS sample tubes, place sample bottles/containers marked with sample identification numbers and in proper numerical order under each correspondingly numbered sample tube. Fill the bottles/containers to the required volume and repeat this step until all types of sample bottles (i.e., metals, minerals, nutrients, sulfide, etc.) have been collected.

6.3.5.6 During the collection of the MLS groundwater samples, it is important to keep track of the fluid volume in each bottle/container, because each sampling tube will not discharge at the same rate. As a bottle or container reaches the proper volume of sample, the sample collector will clamp off the appropriate peristaltic pump tube while allowing the remaining bottles/containers to continue to fill. Finally, after the last bottle or container has filled and the pump tube has been clamped off, the 10-channel peristaltic pumps can be shut off.

6.3.5.7 Immediately after collection of MLS well samples, make field measurements for those water quality characteristics specified in the project's workplan (e.g., temperature, pH, DO, conductivity, ORP, alkalinity, etc.).

6.3.6 <u>Collection of Samples Using a Peristaltic Pump</u>

6.1 A peristaltic pump can be used to collect a sample from a shallow well (water surface less than 7.6 meters below ground surface), spring or seep.

6.3.6.2 Prior to sampling a shallow well, measure and record the distance to the water surface and the depth of the well as given section 6.3.3.2.

6.3.6.3 Calculate the volume of water in the well as shown in 6.3.3.3.

6.3.6.4 Lower the tygon or teflon tubing connected to the peristaltic pump into the water. Remove at least two volumes of water before collection of samples from a shallow well. No purging of water is necessary if collecting a sample from a spring or seep, since the water is naturally flowing.

6.3.6.4 Fill the specified containers, process the samples, and make the water quality field measurements as specified in the project's workplan. Measure (or estimate) and record the spring or seep discharge rate (or the pumping rate if sampling a shallow well) on form TVA 30066A, "Groundwater Quality Data Field Worksheet," attachment 1.

6.3.7 Collection of Samples Using a Lysimeter (Pressure-Vacuum Soil Water Sampler)

6.3.7.1 General Instructions--Lysimeter (pressure/vacuum soil water samplers) can generally be installed and used at any depth up to approximately 15 meters. The access tubes (i.e., pressure/vacuum tube and sample discharge tube) from the lysimeter can extend above the ground surface directly above the lysimeter, or if conditions require, the access tubes can be laid in a trench, terminating above the ground surface at some distance from the lysimeter. The ends of the access tubes should be installed so that they will be protected from damage by mechanical equipment, livestock, etc. The tube ends should be covered or plugged to prevent debris from entering the tubes and later contaminating the samples. The ground surface directly above the lysimeter should not be covered in any manner that would interfere with the normal percolation of soil moisture down to the depth of the lysimeter. Attachment 4 shows a typical lysimeter installation.

6.3.7.2 Access Tubes--The "pressure/vacuum" access tube and the "sample discharge" access tube are usually small diameter polyethylene tubes (e.g., 5 mm I.D.) that extend from the porous ceramic collection device to the ground surface. Typically the tubes are inserted through a cap or plug at the open end of the porous collection cup as shown in attachment 4. One end of the "sample discharge" tube extends nearly to the bottom of the porous ceramic collection cup with the other (discharge) end extending to the ground surface. The discharge end of this tube must be marked and identified as the tube from which the samples are collected. The "pressure/vacuum" access tube is installed slightly differently. One end of the "pressure/vacuum" tube is inserted only about an 2.5 cm past the cap or plug with the other end also extending to the ground surface. The fit of the tubing through the cap or plug and the fit of the cap or plug at the open end of the porous collection cup must be tight and well seated so as to be able to maintain a pressure-vacuum seal.

6.3.7.3 Installing a Soil Water Sampler--Installation of a lysimeter can be performed in several ways. Methods for installation of a lysimeter must be specified in the project's workplan. Typically a 102 mm hole is cored using a T-handle bucket auger. The augered soil should be sifted through a 6.0 mm mesh screen to remove any larger rocks and pebbles. This sifted soil will provide a reasonably uniform backfill for filling in around the inplaced lysimeter. The following discussion details some of the more common methods for installation of a lysimeter. The primary concern in all the methods is that the porous ceramic cup of the lysimeter be in tight, intimate contact with the soil so that soil moisture can move readily from the soil through the pores of the ceramic cup where it can then be withdrawn through the sample discharge tube.

6.3.7.3.1 Native Soil Backfill Method--After the hole has been cored to the desired depth, insert the lysimeter and backfill the hole with native screened (sifted) soil, tamping continuously with a small-diameter rod to ensure good soil contact with the porous ceramic cup and to prevent surface water from channeling down the cored hole.

6.3.7.3.2 Soil Slurry Method--After the hole has been cored, mix a substantial quantity of the sifted soil from the bottom of the hole with water to make a slurry which has a consistency of cement mortar. This slurry is then poured into the bottom of the cored hole. Immediately after the slurry has been poured, push the lysimeter into the hole so that approximately the bottom third of the lysimeter is completely embedded in the soil slurry. Backfill the remaining voids around the lysimeter with sifted soil, tamping lightly with a small-diameter rod to ensure good soil contact with the lysimeter. Backfill the remainder of the hole, tamping firmly, to prevent surface water from running down the cored hole. The first set(s) of soil water samples collected after installing a lysimeter by this soil slurry method may need to be discarded to avoid differences in water chemistry between the water used to prepare the slurry and the natural soil water.

6.3.7.3.3 Sand and Soil Method--Core hole to the desired depth. Pour into the hole, to a depth of about 51 mm, crushed 200 mesh pure silica sand of almost talcum powder consistency (commercially available under trade names of Super-Sil and Silica Flour). Insert the lysimeter and pour in additional sand until at least the bottom third of the lysimeter is covered. Backfill the remainder of the hole with sifted native soil, tamping to ensure good soil contact with the lysimeter and to prevent surface water from channeling down between the lysimeter and the soil.

6.3.7.3.4 Bentonite-Sand-Soil Method--Core hole to the desired depth. Pour into the hole, to a depth of about 51 mm, a small quantity of wet bentonite clay. This will isolate the lysimeter from soil below. Next, pour in a small quantity of 200 mesh silica-sand and insert the lysimeter. Pour in additional sand until at least the bottom third of the lysimeter is covered. Backfill with sifted native soil to a level about 51 mm above the lysimeter, tamping lightly. Again add about two inches of wet bentonite clay as a plug to further isolate the lysimeter and guard against possible channeling of water down the hole. Finally, backfill the remainder of the hole slowly with sifted native soil, tamping continuously. Allow sufficient time for the wet bentonite clay to harden before using the lysimeter to collect soil water samples.

6.3.7.4 Collecting a Soil Water Sample--After the lysimeter has been installed, a pinch clamp is securely tightened on the sample discharge tube, and a vacuum is applied to the pressure/vacuum tube. A vacuum of approximately 60 centibars (46 cm of mercury) is applied. A pinch clamp is then securely tightened on the pressure/vacuum tube. The lysimeter is then left undisturbed for a predetermined period of time, determined by experience and trial and error or as set forth by work plan instructions.

6.3.7.4.1 The vacuum within the lysimeter causes the soil moisture to move from the soil through and into the porous ceramic cup. The rate at which the soil water will collect in the lysimeter depends on the capillary conductivity of the soil and the amount of vacuum that has been created within the lysimeter. In most soils of good conductivity, substantial soil water samples can be collected within a few hours. Under more difficult conditions it may require several days to collect an adequate volume of sample.

6.3.7.4.2 In general, vacuums of 50-85 centibars (38 cm - 64 cm of mercury) are normally applied to the lysimeter. However, in very sandy soils it has been shown that high vacuums may result in a slow rate of sample collection. In coarse, sandy soils, the high vacuums may deplete the soil moisture in the immediate vicinity of the porous ceramic cup and, hence, reduce the capillary conductivity, which results in lower sample collection rates. In loam and gravelly clay loam, collection rates of 300-500 mL/day at 50 centibars (38 cm of mercury) are common. On waste water disposal sites, collection rates of up to 1500 mL/day have been observed.

6.3.7.4.3 To recover the soil water from the lysimeter, attach the pressure/vacuum access tube to the pressure port on a pump. Place the sample discharge tube into the sample bottle or container being careful to avoid and minimize sample contamination from the surrounding soil excavation. Open both pinch clamps (one on the pressure/vacuum tube and one on the sample discharge tube) and gently apply pressure to develop enough pressure within the lysimeter to force the collected soil water out of the lysimeter and into the sample bottle or container.

6.3.7.4.4 Subsequent samples are collected by again creating a vacuum within the lysimeter and repeating the above steps, sections 6.3.7.4 through 6.3.7.4.3

7.0 HANDLING OF SAMPLES

7.1 Sample Identification--All sample bottles and sample containers shall be labeled with a permanent sample identification number. This sample identification number or tag number must be unique for each sample collected and must be cross referenced on all field sheets (forms TVA 30066A and 30066B), and Analysis Request and Custody Record forms (TVA 30488). Prior to packaging and shipping of samples, all containers and bottles shall be inspected for tag numbers and cross checked against all field sheets, and Analysis Request and Custody Record forms. Additional explanation of sample identification requirements are given in section 6.11, reference 3.14 .

7.2 Packing and Shipping of Samples--Sample containers should be closely protected against contamination while transporting them to the survey site, during sampling, field handling and analysis processes, and while transporting them back to the laboratory. Detailed instructions for packing and shipping the various kinds of samples are given in reference 3.7. These requirements are summarized in attachment 1 of reference 3.15. As soon as possible, samples shall be packed on ice. To avoid breakage, care must be taken when packing bottles and containers in shipping chests. Copies of the Analysis Request and Custody Record forms must be sent to the laboratory with the samples. Check to make sure all paperwork has been accurately completed and sealed in a plastic bag to prevent water damage. All shipping containers shall be clearly addressed and shall be sealed and closed with strapping tape.

7.3 Holding Times--The time which elapses between sample collection and sample analysis is critical for many constituents (e.g., BOD, ortho-phosphorus, turbidity, nitrite, etc.). So that the laboratory can complete the analyses within the appropriate holding times, samples must be shipped or transported so as to arrive within the time limits given in attachment 1, reference 3.15. (ES 41.2) Any time samples are to be collected with holding times less than 48 hours, the laboratory must be notified in advance. All collections of samples should be coordinated with the laboratory.

7.4 Chain-of-Custody--The sample collector is responsible for the care and custody of the samples until they are properly dispatched to the receiving laboratory. The sample collector will ensure that each sample is under his/her control at all times. When samples are dispatched to the laboratory for analyses, the sample collector will retain a copy of the completed Analysis Request and Custody Record form(s), the originals of which accompany the samples. All samples shipped to the laboratory will be listed on the custody record form and cross referenced with their unique sample tag (identification) number. The custody record form should reveal the name and telephone number of the sample collector/shipper and the date of shipment. Shipping record receipts for shipments (UPS, Greyhound bus, etc.) will be retained by the sample collector/shipper as part of the permanent chain-of-custody documentation. Upon receipt, the laboratory will inspect for the shipping container for broken seals and will inspect the samples for breakage, missing samples, tampering, etc. The laboratory will verify all samples by cross referencing tag numbers between the custody record and the sample bottles received to ensure that all samples which were shipped have been received complete and intact. The laboratory will immediately notify the sample collector/ES/shipper of any discrepancies. For non-routine sampling or if shipping after Wednesday of a given week, the shipper should verify the arrival of the samples at the laboratory.

7.5 Field Data Worksheets--Copies of all field data worksheets will be sent to the CHATT ENGG-EDM in Chattanooga. Section 8.3 gives additional details.

8.0 RECORDKEEPING

8.1 Project Notebooks

8.1.1 A project field notebook and/or file shall be maintained by the ES survey leader to record pertinent information and observations. The project field notebook accompanies the survey leader to the field. The survey leader shall record and/or file all physical measurements and field analyses performed in the project notebook/file. In addition, auxiliary data often prove very useful in the interpretation of the results. Thus, water surface elevations of nearby ash ponds, basins, lakes, streams, etc., gas bubbles in the sample line, rapid development of turbidity or color in the sample, equipment problems, clogged sampling ports at MLS wells, weather conditions, deviations from workplans or this procedure, or any number of other observations could prove very helpful and should be recorded. Project field notebooks, should there be a change in personnel, should include all information necessary to properly conduct the field survey. At a minimum this would include: the original project workplan with all approved revisions; sample identification (tag) numbers and descriptions of the well locations; copies of past survey field worksheets and groundwater level observations; computer printouts of prior field data; a survey equipment checklist; and all field instrument calibration records. Also included in the field notebook might be maps, sample collection and handling instructions, bus schedules, names and telephone numbers of project personnel, and any miscellaneous notes to aid in conducting the survey.

8.1.2 A project office notebook and/or file are maintained by the lead engineer. The project office notebooks remain in the office at all times and are available for reference by ES, client, and other project organizations. In addition to containing the original approved project workplan and all approved revisions, it should contain information relating to the project such as memoranda, budget estimates, progress reports, data reports, correspondence with client organizations, etc.

8.2 Survey Reports--Following completion of each groundwater field survey, the ES survey leader will prepare a draft report to the client organization which will be finalized by the lead engineer. The report shall contain:

a. A cover letter addressed to the client from the lead engineer which describes the field activities and notes any unusual conditions (weather, equipment problems, breach of well security, etc.);
b. The Ground Water Quality Data Field Worksheets;
c. Special worksheets (e.g., Acidity and Alkalinity);
d. Instrument Standardization Forms;
e. Groundwater Level Measurements Form; and
f. Analysis Request and Custody Record Form.
g. Other Forms (i.e. bacterial organism worksheet)

Note: The survey leader is responsible for proper routing of the five (color coded) field sheets).

8.3 Disposition of Forms

8.3.1 Forms TVA 30066A and B, Groundwater Quality Data Field Worksheets, attachments 1 and 2, are used any time physical and/or chemical groundwater measurements are made. The original (white copy) is sent to and is filed by CHATT ENGG-EDM. Copies are retained by ES field office per attachment 7 (distribution) and may be sent to the client organization(s) at their request.

8.3.2 Form TVA 11552 (or similar project specific form/table), Groundwater Level Measurements (Field), attachment 5, is used as required, when groundwater elevations are observed or recorded at ash ponds, coal pile runoff ponds, metal cleaning waste ponds, rivers, lakes, groundwater wells, etc. The original (white copy) is sent to and is filed by CHATT ENGG (EDM). Copies are retained by ES field office per attachment 7 (distribution) and may be sent to the client organization(s) at their request.

8.3.3 Form TVA 30488, Tennessee Valley Authority, Water Management Services, Environmental Chemistry Analysis and Custody Record, is used to ship samples to the ECHEM Laboratory and identify the desired analyses. It is to be used anytime samples are shipped or delivered to the ECHEM Laboratory to ensure that the proper number and types of samples as specified in the approved project workplan, are in fact received by the

ECHEM Laboratory. The original (white copy) is sent with the samples to the laboratory. Copies are retained by ES field office per attachment 7 (distribution) and one copy (pink) is sent to CHATT ENGG-EDM. Reference 3.15 contains an example of form TVA 30488.

8.3.4 Form TVA 11064, Sample Custody Record, is only used when samples are shipped or delivered to an external TVA laboratory to aid ES in its internal record keeping functions, or as an aid for shipping/record keeping, for sample custody to an external TVA laboratory. Reference section 3.15 of ES-41.2, contains an example of form TVA 11064.

8.3.5 Form TVA 991, Request for Analysis, is only used for samples requiring external TVA laboratory analyses. It specifies which analyses are to be performed or which workplan is to be followed for sample analyses. The original is sent with the samples to the external TVA laboratory, additional copies will be retained by ES. Reference 3.15 contains an example of form TVA 991.

8.3.6 Form TVA 30533, Acidity and Alkalinity Field Worksheet is to be used by ES, the original is sent to CHATT ENGG-EDM and copies distributed per attachment 7 (distribution).

8.3.7 Retention periods and file locations for these forms are given in attachment 7.

CE 0127V

LIST OF ATTACHMENTS

1. Groundwater Quality Data Field Worksheet (Chemical Data), form TVA 30066A.
2. Groundwater Quality Data Field Worksheet (Physical Data), form TVA 30066B.
3. Schematic Drawing of a Multilevel Sampling (MLS) well.
4. Typical Lysimeter Installation.
5. Groundwater Level Measurements (Field), form TVA 11552.
6. Acidity and Alkalinity Field Worksheet, TVA Form 30533.
7. Records (Use, Distribution, and Retention).

ATTACHMENT 1

GROUNDWATER QUALITY DATA FIELD WORKSHEET (CHEMICAL DATA), FORM TVA 30066A

GROUNDWATER DATA FIELD WORKSHEET

SHEET ____ OF ____ | PRELIMINARY DATA

PROJECT/SITE | PURGE DATE ▶ YEAR / MONTH / DAY

WELL NO. | DEPTH TO WATER (m) 4195 | BOTTOM OF WELL (m) 4194 | SURVEY LEADER | FIELD CREW

WELL DIAM (mm) 4188 | DEPTH OF SCREEN OR OPEN BORE HOLE (CIRCLE) (m) 4191 TO (m) 4190

[BOTTOM OF WELL - DEPTH TO WATER] x VOL. FACTOR = WELL VOLUME | TARGET PURGE VOL | ACTUAL PURGE VOL

[( )m - ( )m] x ( ) l/m = 4187 (l) | (l) | 4186 (l)

SAMPLE TAG NO. | (Circle) UNFLT FLT BOTH | FILTER TYPE/SIZE

PURGE PUMP (Circle) BLADDER CENTRIFUGAL PERISTALTIC BAILER OTHER (list) ____

SAMPLE PUMP (Circle) BLADDER CENTRIFUGAL PERISTALTIC BAILER OTHER (list) ____

NOTES AND WQ OBSERVATIONS	ET CT TIME (mil)	PUMP RATE (l/min)	DEPTH TO WATER (m)	PUMP DEPTH (m)	TEMP (°C)	pH (units)	DO (mg/l)	COND (µs/cm)	(+/-) ORP (mV)	
BEGIN PURGE →										

REMARKS:

Reviewed by:
Survey Ldr: Date
FE Proj. Eng: Date

SAMPLE READINGS →

SAMPLE DATE YR / MO / DAY

PUMP DURATION min

"99" = 2 DAYS

	4193		4192	10	400	300	84	90	
(mil) TIME	(l/min) SAMPLE RATE	(m) WATER SURFACE	(m) SAMPLE DEPTH	(°C) TEMP	pH	(mg/l) DO	(µs/cm) CONDUCTIVITY	(mV) ORP (+/-)	

ADDITIONAL SAMPLE DATA | TURBIDITY 1350 (circle): CLEAR 1 SLIGHTLY TURBID 2 TURBID 3 HIGHLY TURBID 4

COLOR: | ODOR:

PHENOL ALK mg/l 415 | MINERAL ACIDITY mg/l 436

TOTAL ALK mg/l 431 | CO_2 ACIDITY mg/l 437

BOTTLES REQUIRED (circle)

BOD TIC FERROUS MINERAL PHENOL OTHERS (list): ____

COD SOC METALS DIS MINERAL FILT TIC ____ ____

TOC O&G DIS METALS NUTRIENT TSS/TDS ____ ____

WELL DIAM mm		VOL. FACTOR l/m
51	(2 in)	2.027
76	(3 in)	4.560
102	(4 in)	8.107
127	(5 in)	12.668
153	(6 in)	18.228

DISTRIBUTION: (1) Original - Data Mgt (2) Pink - Lab With Samples (3) Blue - Unit Leader (Office Notebook) (4) Green - Survey Leader (Field Notebook) (5) Yellow - F.E. Project Engineer

TVA 30066A (RD BUS 1-93)

ATTACHMENT 2
GROUNDWATER QUALITY DATA FIELD WORKSHEET (PHYSICAL DATA), FORM TVA 30066B

Ground Water Quality Data Field Worksheet
(Physical Data)

Project ______________________________
Well Name/Number ______________ Spring Name/Number ______________
Owner's Name ______________________________
Address ______________________________
Phone Number ______________

Well/Spring Information

Lat ____________ Long ____________ State ____________
Location ______________________________
Well Depth (ft.) ______________________________
Depth of Well Screen (ft.) ______________________________
Approximate Water Surface Depth (ft.) ______________________________
Description of Reference Point Used to Make Depth Measurement ______________________________
____
Elevation of Reference Point (MSL-ft.) ______________________________
Water Use ______________________________
Volume of Water Use (GPD) ______________________________
Type Casing ______________________________
Casing Dimensions ID ________ (in) OD ________ (in) Length ________ (ft)
Does well have permanently installed pump? ________ If so, type of pump ________,
capacity (gpm) ____________, discharge flow rate (gpm) ____________

Well Drillers Log Data

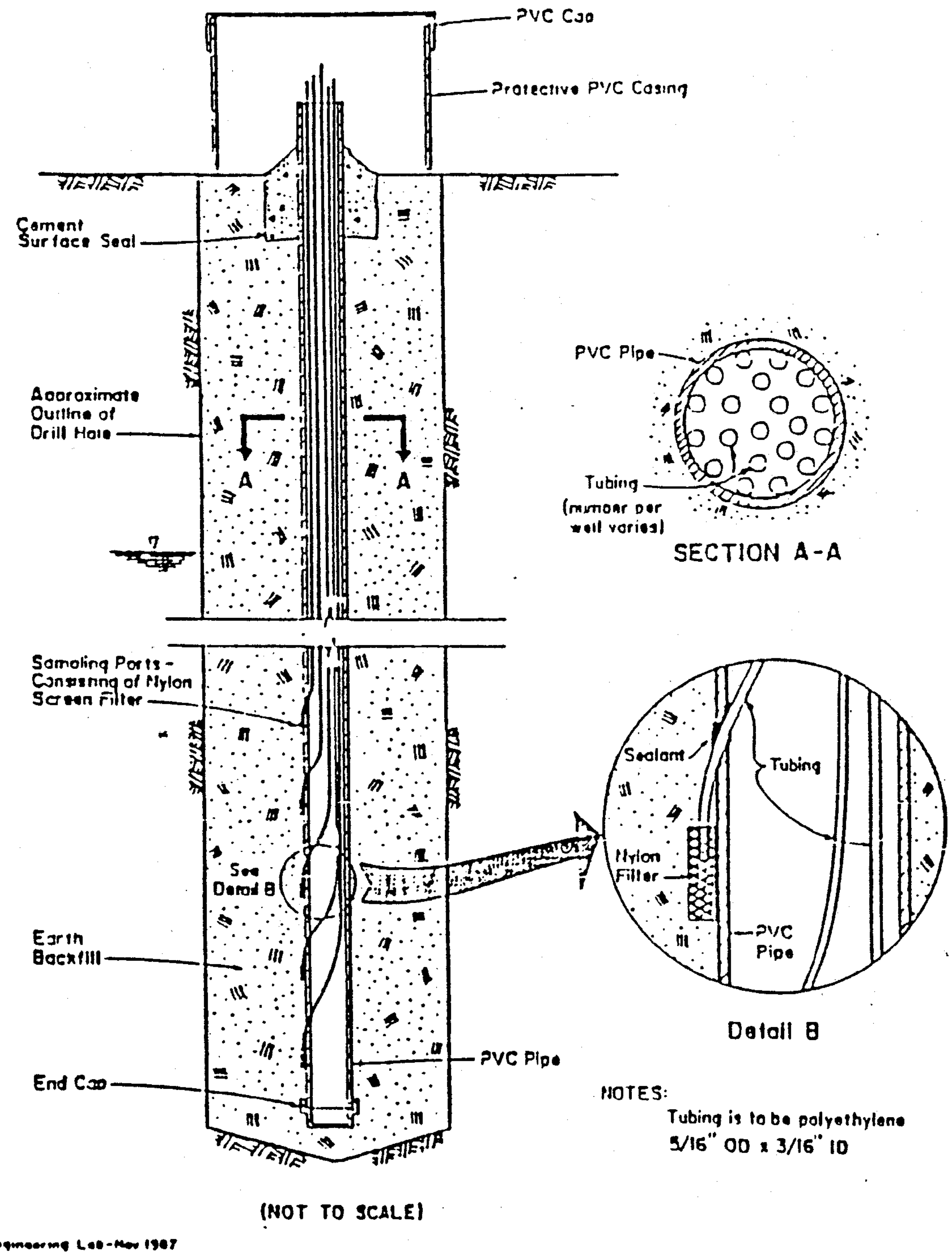
(Attach sketch and/or provide written detailed description)

Remarks: ______________________________
____
____
____
____

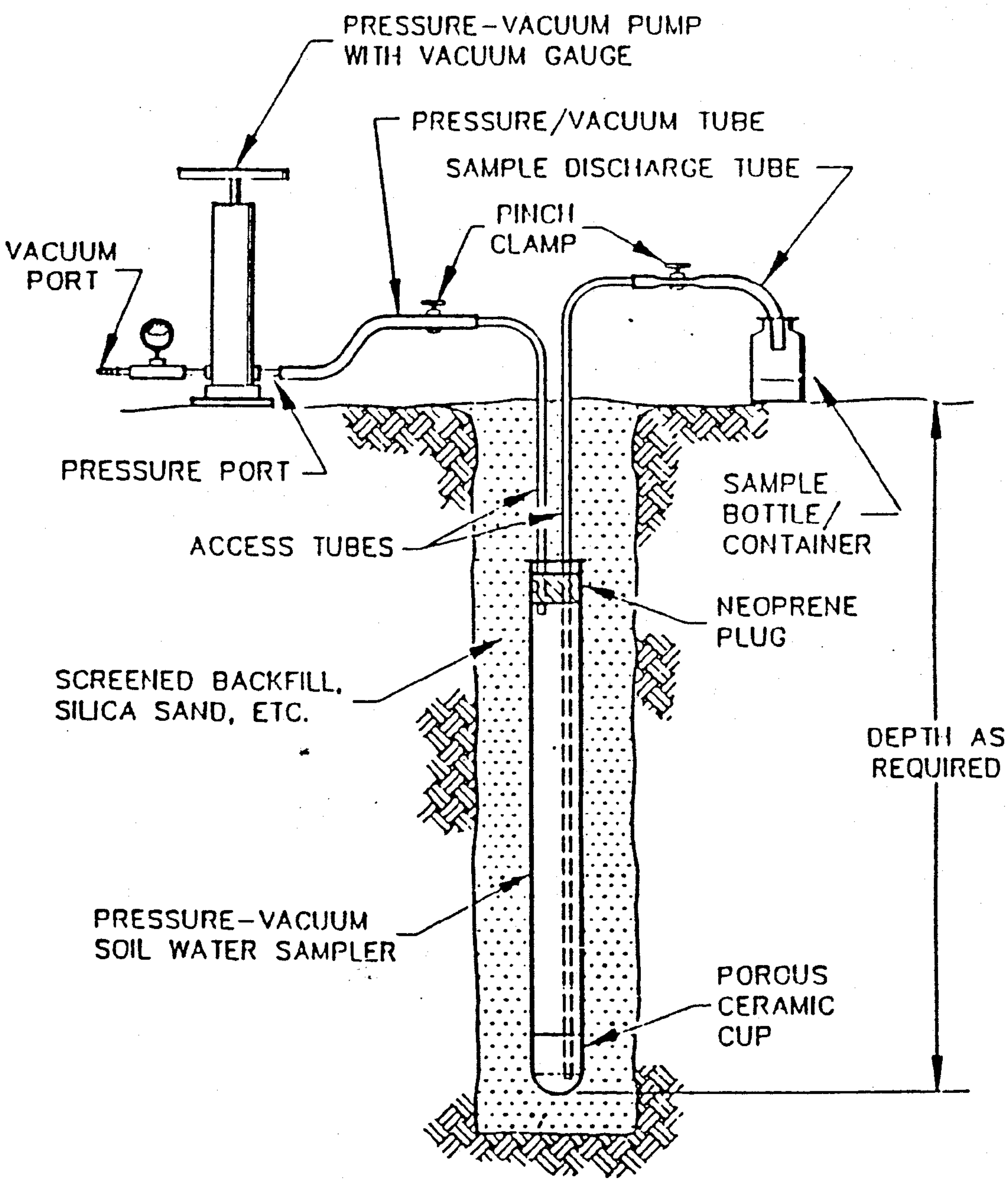
TVA 30066B (NROPS-1-88)

Reviewed by ______________ Date ________

ATTACHMENT 3
SCHEMATIC DRAWING OF A MULTILEVEL SAMPLING (MLS) WELL



ATTACHMENT 4
TYPICAL LYSIMETER INSTALLATION (PRESSURE-VACUUM SOIL WATER SAMPLER)



TYPICAL LYSIMETER INSTALLATION
(PRESSURE-VACUUM SOIL WATER SAMPLER)

ATTACHMENT 5

TENNESSEE VALLEY AUTHORITY
ENGINEERING SERVICES
GROUND-WATER LEVEL MEASUREMENTS (FIELD)

Date____________
Location_________________
Measured by_______________

Well/ RP No. (P84068)*	Description	Reference Point (RP)	Time (ET)	Well RP Elevation(m)	Depth to Water Surface(m) (4195)*	Water Level Elevation (m) (4189)*	Depth to Well Bottom (m) (4194)*	Remarks

* STORET codes

TVA 11552 (5/94)

ATTACHMENT 6
ACIDITY AND ALKALINITY FIELD WORKSHEET, FORM TVA 30533

ACIDITY AND ALKALINITY FIELD WORKSHEET

PRELIMINARY DATA

PROJECT/SITE	COLLECTION DATE
SURVEY LEADER	
FIELD CREW	

NORMALITY
ACID
BASE

				ALKALINITY TITRATION(S)				ACIDITY TITRATION(S)			
SAMPLE TAG NO.	INITIAL pH	SAMPLE VOL TI-TRATED (ml)	FAC-TOR	ml ACID TO pH 8.3	PHENOL ALK (mg/L) ($CaCO_3$)	TOTAL ml ACID TO pH 4.5	TOTAL ALK. (mg/L) ($CaCO_3$)	ml BASE TO pH 3.7	MINERAL ACIDITY (mg/L) ($CaCO_3$)	TOTAL ml BASE TO pH 8.3	CO_2 ACIDITY (mg/L) ($CaCO_3$)

ALKALINITY OR ACIDITY (mg/L $CaCO_3$) = $\frac{A \times N \times 50{,}000}{V}$

A = ml TITRANT
N = TITRANT NORMALITY
V = SAMPLE VOLUME

NORMALITY	SAMPLE VOL TITRATED	FACTOR
0.02N	100 ml	10
0.02N	50 ml	20

REVIEWED BY:

_______________ Survey Leader _______________ Date

_______________ FE Proj. Eng. _______________ Date

DISTRIBUTION: (1) Original - Data Management (2) Pink - Lab with samples (3) Blue - Unit Leader (Office Notebook) (4) Green - survey Leader (Field Notebook) (5) Yellow - F.E. Project Engineer (AJDMDS)

TVA 30533 (RD-BUS 4-92)

ATTACHMENT 7
RECORDS (USE, DISTRIBUTION, AND RETENTION)

Record	Use	Distribution[d]	Retention	Time[a,b,c]
TVA 30066A	GW Data Chemical, Field Worksheet	1-Original to CHATT ENGG-EDM	Files	20 yrs
		2-Copy (pink) ECHEM	as needed	as needed
		3-Copy (blue) ES Field Office	Office notebook	2-3 yrs
		4-Copy (green) ES Field Office	Field notebook	2-3 yrs
		5-Copy (yellow) ES Field Office	Lead Engineer	2-3 yrs
TVA 11552	Groundwater Elevations (wells, water bodies, etc.)	1-Original to CHATT ENGG-EDM	Files	20 yrs
		2-Copy (pink) extra	Lead Engineer	
		3-Copy (blue) ES Field Office	Office notebook	2-3 yrs
		4-Copy (green) ES Field Office	Field notebook	2-3 yrs
		5-Copy (yellow) ES Field Office	Lead Engineer	2-3 yrs
TVA 30488	Request for Analysis and Custody Record	1-Original to ECHEM	ES sample analysis (marked up copy immediately to ES if discrepancies occur)	as needed
		2-Copy (pink) to CHATT ENGG -EDM	Files	20 yrs
		3-Copy (blue) ES Field Office	Office notebook	2-3 yrs
		4-Copy (green) ES Field Office	Field notebook	2-3 yrs
		5-Copy (yellow) ES Field Office	Lead Engineer	2-3 yrs
TVA 11064	Sample Custody Record	1-Original to Lab (outside TVA)	Return to ES w/ sample analysis (Field notebook)	2-3 yrs
		2-Copy (pink) extra	Lead Engineer	
		3-Copy (blue) ES Field Office	Office notebook	2-3 yrs
		4-Copy (green) ES Field Office	Field notebook	2-3 yrs
		5-Copy (yellow) ES Field Office	Lead Engineer	2-3 yrs
TVA 991	Request for Analysis	1-Original to Lab (outside TVA)	Return to ES w/ sample analysis (Field notebook)	2-3 yrs
		2-Copy (pink) extra	Lead Engineer	
		3-Copy (blue) ES Field Office	Office notebook	2-3 yrs
		4-Copy (green) ES Field Office	Field notebook	2-3 yrs
		5-Copy (yellow) ES Field Office	Lead Engineer	2-3 yrs
TVA 30533	Acidity and Alkalinity Field Worksheet	1-Original to CHATT ENGG-EDM	Files	20 years
		2-Copy (pink) extra (client)	Lead Engineer	-
		3-Copy (blue) ES Field Office	Office notebook	2-3 yrs
		4-Copy (green) ES Field Office	Field notebook	2-3 yrs
		5-Copy (yellow) ES Field Office	Lead Engineer	2-3 yrs
Various	Laboratory Results	1-Original to CHATT ENGG by Lab	Files STORET	2 yrs
		2-Copy to ES Field Office	Office notebook	2-3 yrs
		3-Copy to client as required by ES after review	as needed	as needed

a. Retention time for STORET-related data and field sheets is 20 years
b. Retention time for STORET-related laboratory results report forms is 2 years beyond project completion.
c. ES retention time is 2 years MINIMUM after total completion of project and 3 years MINIMUM for on-going projects.
d. Color coded copies may not be available for all forms.

APPENDIX G

Stability and Seismic Impact Analysis

PARSONS

CALCULATION COVER SHEET

CLIENT	TVA
PROJECT	Kingston Fossil Plant – Dredge Cell Expansion
SUBJECT	Slope Stability Evaluation and Recommendations
JOB NUMBER	55090501 WBS NUMBER -
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DESCRIPTION/PURPOSE

Review available subsurface data including that obtained recently, develop subsurface profiles for critical locations, determine design soil parameters, and evaluate factor of safety against failure of slopes of both the ash pile (existing cell area) and the gypsum-ash stack (existing ash-pond area).

METHOD OF ANALYSIS

Pseudostatic method (cylindrical surface of failure and sliding-block analysis) using computer program PC STABL5M

CODES AND STANDARDS

1. Tennessee Division of Solid Waste Management, Technical Guidance Document – Earthquake Evaluation Guidance Policy (Guidance Document)

INFORMATION SOURCES

See REFERENCES list on Page 29.

ASSUMPTIONS

Read Pages 3 through 23.

CONCLUSIONS OR RESULTS

See Pages 27 and 28.

REV	DATE	DESCRIPTION	PAGES REVISED	PAGES ADDED	PAGES DELETED	BY/DATE	REV/DATE	LDE/DATE
3								
2								
1								
0		ORIGINAL ISSUE	NA	NA	NA	Y.S.Shah 05-26-04	W.Anundsn 05-26- 04	D.R.Smith 05-26-04

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1. INTRODUCTION

Reference 1 drawings show the existing or present topography of the ash site and the proposed Phase 1, 2 and 3 construction plans. The site is divided into three primary areas:

A. Cell Area, consisting of cells 1, 2 and 3, where ash has been deposited to-date to Elev. ~**810'**.

B. Ash Pond Area, where ash has been deposited to-date to Elev. **760'** or lower, and

C. Stilling Basin, wherein water from the above two areas is drained and where the surface of pond water now is at Elev. **756'±**.

Currently, a new cell area is being created between Cell Area and Ash Pond Area, located inside Ash Pond Area, where a Stage 1 dike to Elev. 780' is being constructed. This area is called Phase 1, where ash will be temporarily deposited and later raised to be even with Cell Area elevation (810').

(NOTE: For convenience herein, Cell Area is referred to as the area located on the north side of the ash site, and Stilling Pond on the south side. Thus, the ash site is bounded by Dike B on the north, Dike C on the east, and North Dike and Road Dike on the west. Dike B and North Dike form the north and west boundaries, respectively, of Cell Area; Road Dike forms the west boundary of both Ash Pond Area and Stilling Basin; and Divider Dike separates Ash Pond Area and Stilling Basin.)

The original topography of the ash site may be assumed as shown in the Reference 2 drawing. This drawing shows that the original ground surface (GS) in the eastern half of Cell Area was approximately at Elev. 730', and dipped gently to Elev. 724' at its west edge. In Ash Pond Area, the GS dipped gently westward from Elev. 735' at its east edge to 724' or lower at its west edge. The GS varied from Elev. 745' to 730' in Stilling Basin Area.

Thus, the original GS at the ash site was roughly at Elev. 730' ± a few feet and that ash has been stacked up by at least **80 feet** (= 810' – 730') in Cell Area and approximately **30 feet** (= 760' – 730') in Ash Pond Area over the original GS. If the Stilling Basin bottom consisted of the original GS (i.e., the bottom was left uneven), the water and sediment depth there is maximum **26 feet** (= 756' – 730'). For this analysis, however, it is assumed that the bottom was excavated to Elev. **729'** and that the basin is silted up so far to Elev. **746'**; i.e., a loose silt/flyash deposit of 17 feet exists at the bottom of the pond.

The proposed plan is to stack ash to Elevation as high as **868 feet** in Cell Area (i.e., raise the area further by 58**feet**) and stack gypsum and fly ash both to Elevation approximately **970 feet** in Ash Pond Area (i.e., raise the area there further by **210 feet**) as shown on the ~~drawings (Ref.1). Both ash and gypsum will be placed wet primarily (sluiced in from the~~

plant) until the year 2019 and gypsum will be placed dry thereafter. For this analysis, it is assumed conservatively that the Ash Pond stack consists of wet placed gypsum and fly ash up to Elevation approximately 930 feet (maximum expected) with dry-placed gypsum above it.

Prior to performing the latest subsurface exploration, it was believed that (as no subsurface data was then available for the interior Ash Pond area of Phase 1) the foundation condition beneath the intermediate cell dikes might be incapable of supporting the proposed intermediate cell construction at the south edge of Cell Area. However, based both on the exploratory data and satisfactory performance of the dike built so far to Elevation 780' it is evident that the foundation condition at the dikes is capable of supporting the proposed construction. Therefore, a separate evaluation of stability for the intermediate cell dikes is not performed as it is considered that the stability evaluation for Section 1-1 as done herein is adequate to demonstrate stability of these dikes also.

The static stability evaluation is performed also for the existing ash stack where a groundwater blowout occurred in the Fall of 2003 at Elevation 770' at the Swan Pond side slope of Dike B, outside Cell 3. This is done to support the conclusion that the failure was due not to the slope stability but to the piping or the excessive seepage gradient. The excessive seepage gradient may have resulted from the raised phreatic surface inside the ash stack as a result of inadequate drainage of both the storm water and water drained from the wet stacking operations.

Except the stability evaluation for the blowout location (for which only the static condition is considered), stability evaluation for three critical sections across the proposed two stacks includes evaluation for the design seismic condition in accordance with the Guidance Document; i.e., assuming a peak or maximum ground acceleration of 0.22g.

2. SITE HISTORY & PERTINENT DATA

Based on the data from References 2 through 6, the developmental history for the ash site and other information pertinent to this analysis are summarized as follows.

1. *Referring to Drawing 10N400 (Ref. 2), it is evident that*

    - *The initial North Dike (top at Elev. 746'), along with East Dike (top at Elev. 750'), was planned in August 1951. Both dikes were to be built of earth materials.*
    - *The initial Dike C (top at Elev. 748'±) was planned in January 1958. It was to be built of borrowed earth materials.*

- *The initial Dike B (top at Elev. 748') and future raising of all these dikes were planned in August 1967. Use of bottom ash (BA) was planned for raising the dikes and construction of the initial Dike B.*

Thus, the ash site (then called, "'New Ash Disposal Area") was created after January 1958 when Dike C was built. Further, it should be noted that the dike slopes below Elev. 735' were to be constructed under the submerged condition (i.e., the Watts Bar Lake water level then was at Elev. 735' and the site was perhaps a part of Swan Pond Embayment or was mostly a swamp). That also implies that the lake water level might have been drawn down to Elevation 735' ± during construction of these dikes and that the ash site might have remained water-logged prior to ash disposal there.

2. *The TVA document dated June 26, 1974 (Ref. 4) indicates that*

   - *The dikes were not yet raised, and Dike B was not yet built. Although Dike B was planned to be built "in the wet on previously deposited ash .... by end-dumping to minimum depth and compacting with tracked equipment", a hand-written note dated November 10, 1975 on the document states that "as ash is of poor quality, Dike B will be built all earth". Thus, it is indicated that Dike B foundation consisted of loose, wet ash and that the ash would not be suitable for the initial Dike B construction (although, borings B-1 and B-2 drilled through Dike B show that it was built of ash)!!*

   - *Southern portion of the initial Dike C was built using ash.*

3. *TVA's soil investigation report dated November 3, 1975 (borings SS-1 through SS-11 on the initial Dike C and initial Road Dike; borings SS-12 through SS-24 into ash adjacent to these two dikes and initial North Dike; Ref. 5) shows that*

   - *Dike B was not yet built. However, ash was deposited to Elev. varying between 749' and 755'± within the area enclosed by Dike C, Road Dike and North Dike. See sketch titled, Plan of Foundation Investigation (Ref.5).*

   - *The top of initial Dike C and initial Road Dike was at Elev. 752'± 1' and the top two feet consisted of crushed shale and limestone. Also, it was not clear from the borings that the southern portion of Dike C at SS-6, SS-7 and SS-8 locations was built all of ash. It appeared that ash, if used, was mixed with clayey earth fill.*

   - *Approximately 11 feet below the top 2 feet of crushed stone fill (i.e., to Elev. ~739') of initial Dike C and initial Road Dike consisted of compacted ash and soil (SPT N greater than 10). Below that depth, both the fill soil and alluvial soil were soft or loose (SPT N of 4 or less). The fill soil was fine-grained, consisting of CL, CH and SM containing chert fragments. The GWL into the dikes varied between Elev. 735' and 750', dipping southward.*

   - *The top 5 to 8 feet of ash in the ash pile adjacent to the dikes or nearby areas was medium compact to compact, but was loose (SPT N less than 4) below that depth. The GWL into the ash was approximately 6 feet higher than into the dikes.*

4. *Drawings 10N420 and 421 (Ref. 3), both dated May 1976, indicate that*

- *All the <u>initial</u> dikes noted above were built and raised to Elev. 765' (with the top of the initial dikes left as bench) before May 1976, although Dike C and Road Dike were raised using "Rolled Earth Fill", and not BA as originally planned. That was perhaps because enough BA, called "heavy ash", wasn't available! (See Sect's A-A and B-B on 10N421)*

- *The Divider Dike, to be built of BA, was planned in May 1976. The BA would be placed by end-dumping below water level (then perhaps at Elev. 746'± as indicated in Sect. AA – AA on 10N421), and to be placed in compacted lifts above up to Elev. 765'.*

- *The lake water level then varied between Elev.'s 735' and 741' (Sect. A-A on 10N421).*

5. *Logs of hand-auger borings AH-1 through AH-17 and exploratory borings SS-35 through SS-38 (Ref. 6), drilled by TVA during May and December 1984 along the initial Dike C and initial Road Dike, show that:*

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   - *The southern part of the initial Dike C along Stilling Basin between AH-1 and AH-3 was apparently constructed of coarse BA. The BA was found in all four SS borings (located between AH-1 and AH-3) from Elev. ~748' down to approximately Elev. 739' ± 1'. Soft clayey soil (SPT N = 2 to 8) was encountered below BA for several feet. Silty clay fill was encountered above BA to the top of the dike (i.e., to Elev. 753' ± 1'), except for some coarse material at the surface. The top 5 feet of BA in the dike was found to be compact, and loose to medium compact below.*

   - *The remaining northern portion of the initial Dike C and the entire initial Road Dike were apparently built of highly plastic clayey (CH) earth fill, perhaps mixed occasionally with ash.*

6. *Borings SB-1 through SB-10, drilled by Singleton during July-August 1994 on the perimeter dikes along Cell Area (Ref. 7), show that*

   - *Dike C at Cell Area (Cell No. 2) was raised to Elev. 773', and North Dike (Cell 3) had been raised to Elev. 797.5'. (Note that Cell Area then, as now, was divided into Cell 1 on the west side, Cell 2 on the east side, and Cell 3 in between those two.) Other dikes forming Cells 1 and 3, including Dike B, then had been raised to Elev. ~795'. (See Location Plan in Ref. 7.) Apparently, the dikes were raised above Elev. ~765' using compacted ash.*

   - *The surface of ash in Cell 1 was at Elev. ~785', at Elev. ~ 770' in Cell 3, and at Elev. ~769' or lower in Cell 2. Apparently, Cells 1 and 2 were active then. (See Location Plan in Ref. 7.)*

7. *A blowout at the exterior (Swan Pond Road side) slope of Dike B outside Cell 3 at Elev. 770' occurred in the Fall of 2003. (A stability evaluation for this slope is included herein, as stated earlier, in support of a conclusion that the blowout occurred as a result of excessive seepage pressure of water from the ash pile and not the sliding failure of the slope.)*

8. *The existing or recent ground-surface condition at the ash site is shown on the drawings in Reference 1. The dikes surrounding Cell 1 now are at Elev. ~810' and the remaining dikes*

surrounding Cells 2 and 3 are at Elev. ~805'. The ash level in all three cells are at Elev. 805' or lower. None of these cells is active at present.

NOTE: Reviewing all the subsurface data, especially the 2004 Normalized CPT Plots (Ref. 9), it is apparent that several feet of ash overlying the natural clay layer (top approximately at Elev. 730') has remained apparently loose despite years of being under the existing ash overburden. Also, the loose ash generally is described as silt; i.e., it is primarily fly ash. The CPT logs also show that the dynamic pore-water pressure generated in this ash during sounding was high and the nature of dissipation of the pore-water pressure resembled that for clay. This means that the ash has not been consolidating or that it may undergo significant strength loss when disturbed or shaken.. Interestingly, John Boschuk of JLT laboratories also observed that the fly ash "liquefies under even slight vibrations"; i.e., if pore-water pressure induced by shaking is not allowed to dissipate, the ash looses its strength. This may also explain why the SPT blowcount in this ash is very low – almost zero. Thus, the need for a provision for a quicker relief of this pressure and for a speedier gain in strength of this ash at critical locations for an effective improvement of the stability of the proposed stacks during a seismic event is perhaps indicated. Furthermore, it is also interesting to note that the subterranean water from the adjacent Pine Ridge area, located northwest of the ash site, drains into the lake as shown in Fig. 2-5 of the hydrogeology report (Ref.8). Thus, any downward seepage of water from the wet-sluiced ash deposited in the cell area recharges the GWL and raises it just upstream of Cell Area. This is important to note when planning an interceptor drain enveloping the cell and pond areas, especially to control the exit gradient of water seepage from the ash stack at safer levels and thereby to help mitigate future recurrence of the blowout that occurred in the Fall of 2003.)

3. SUBSURFACE EXPLORATIONS

Locations of all exploratory borings drilled at the ash site prior to 2004 are shown on the Reference 10 drawing.

No deep borings were drilled in the interior cell and ash pond areas during the past investigations. Therefore, an additional subsurface exploration was undertaken in March 2004 that consisted of the following:

- Twelve borings (B-1 through B-12),
- Eleven cone-penetrometer (CPT) soundings (CPT-1, 1A, 4, 6, 8, 9, 10, 11, 12A, DN and DS) with pore-water pressure measurement located adjacent to selected boring locations,
- Field permeability testing (at the blowout location), and
- Laboratory testing of disturbed and undisturbed ash and soil samples collected from the borings.

The exploration was performed to obtain the subsurface conditions of ash and natural subsoil in the interior areas and also to verify those obtained from the past explorations. The data obtained from the 2004 exploration is given in Reference 9 and is used primarily to determine the design conditions for this analysis, although the data from the past explorations is also considered both as complimentary and supplementary data.

Also, undisturbed Shelby-tube samples of both sedimented Gypsum-fly ash mixture and cast Gypsum were obtained from the active Cumberland Fossil Plant disposal facility by Mactec and tested in their laboratory for its shear strength (Ref. 11). The values of the strength obtained from this testing were compared with the extensive data available from the existing TVA and EPRI sources (References 12, 13 and 14) and the design values were chosen based on a review of the entire data base.

4. CRITICAL SECTIONS FOR STABILITY EVALUATION

An examination of the proposed stacking plan (Phases 1, 2 and 3 or the final phase) and the subsurface data shows that the critical locations for the slope stability evaluation are in Ash Pond Area adjacent to the proposed Drainage Pond and existing Stilling Basin. Noting that the proposed stack toe will be located 100 feet and 200 feet from the two ponds, respectively, the following three critical sections, one for each of the three phases of construction are chosen for the stability evaluation. Also, a section at the blowout location is analyzed as noted before. The critical sections chosen for the stability evaluation are:

a. Section 1-1: North-South section, through Cell Area and Drainage Pond (End of Phase 1)

b. Section 2-2: East-West section, through Gypsum-Flyash Stack and Drainage Pond (End of Phase 2)

c. Section 3-3: North-South section, through final Gypsum-Flyash and Stilling Basin (End of Phase 3 or Final Phase)

d. Section 4-4: Section through existing Cell Area at the "blowout" location

The first three sections are illustrated on the drawings (Ref.1).

The computer program PCSTABL5M is used for the stability evaluation, assuming a cylindrical surface of failure. Further, a sliding block analysis for the most critical Section 3-3 (Final Phase condition) is also performed using the same computer program.

The simplified versions of all sections are illustrated in the computer printouts of the respective stability evaluation.

5. FOUNDATION STRATIFICATION FOR ANALYTICAL MODELS

An extensive review of data from all past and recent borings and CPT soundings was performed for determination of the generalized existing subsurface stratification to be used for the stability evaluation. Generally, data from the past borings matched the subsurface conditions revealed from the investigation performed in 2004. However, unlike the past investigations, the 2004 investigation included CPT soundings. The continuous record of data obtained from these soundings was found to be more definitive of changes in the stratification and, hence, was the determining factor in choosing the design profile.

The most critical area for the stability is clearly the existing ash pond area due to location of Stilling Basin and the proposed drainage pond ("Drainage Pond") adjacent to the proposed stack and also due to the anticipated maximum loading condition (i.e., maximum proposed stack height) in that area. Therefore, the stratification used for the stability evaluation at Sections 2-2 and 3-3 corresponds to Ash Pond Area. The same stratification also is used for apparently less critical Section 1-1: Further, it is proposed that

1. The existing Ash Pond Area be graded (where the existing GS is at or lower than Elev. 760'),
2. The graded surface (Elev. ~758') be stabilized and compacted using a heavy roller or compaction equipment and then
3. A well-compacted fly ash pad, gently sloping towards Stilling Basin, be constructed.

This construction will raise the bottom of the proposed stack from the graded existing surface (Elev. ~758') to Elev. 760' at Stilling Basin and to Elev. 770' at the south edge of the existing Cell Area. Bottom ash and/or Tensar geogrid may be required to stabilize the area to be occupied by the fly ash base during construction to support construction equipment. A 3-foot thick filter blanket of coarse bottom ash (two feet) and bottom ash-fly ash mixture (1 foot) will be placed over the compacted fly-ash pad in the stack area. (See Ref.1 drawings)

Thus, the subsurface profile below the stack is generalized as follows for the stability evaluation:

Stratum No.	Elevation Range	General description
1	763'+ to 760'+	Bottom Ash, lightly compacted
2	760'+ to 758'	Compacted Fly Ash
3	758' to 739'	Loose Fly Ash - Bottom Ash Mixture (FA+BA)
4	739' to 729'	Loose Fly Ash (FA)
5	729' to 714'	Natural Clay, soft to stiff (CL)
6	714' to 703'	Clayey Silty Sand, Residuum (SC-SM)
7	Below703'	Bedrock (Soft Shale)

6. ANALYTICAL MODELS FOR PROPOSED STACKS

A. GYPSUM-ASH STACK:

The foundation stratification for this model is given in the preceding section.

After constructing the filter blanket over the fly-ash pad, the perimeter dike to Elev. 780' will be constructed of compacted BA/fly ash mixture. Gypsum slurry will be deposited into the area enclosed by the perimeter dike. The gypsum sedimented from this initial gypsum deposit will be scooped from areas adjacent to the perimeter dike to build the initial cast gypsum dikes above Elev. 780', using the rim-ditch operation as shown on the Reference-1 drawings. The subsequent construction of the stack also is shown on these drawings.

As sedimented gypsum is to be deposited first to Elev. 780', the bottom of the stack up to Elev. 780' consists of sedimented gypsum for the analytical model for this stack. It is assumed as stated earlier that the stack will be raised to Elev. 930' with wet-stacking operation; and, with dry-stacking operation above it.

The outer slope of the stack will consist of cast-gypsum dikes, raised in 10-feet vertical heights, with a 15-foot wide bench at every 30-foot height interval. Also, drains will be installed as shown in the Ref. 1 drawings at the bottom of each perimeter dike. Simplifying this condition, a cast-gypsum zone of 150 feet horizontal width is assumed conservatively for the stability evaluation as shown in the computer printout of the model. The phreatic surface inside the stack for the stability evaluation is assumed conservatively to be as high as the top of the wet-stacking operation (Elev. 930') and bounded by the inner boundary of the assumed cast-gypsum zone along the stack slope. Thus, all simplifying assumptions for this analytical model are conservative.

B. ASH-ONLY STACK

TVA also wanted PE&C to perform the stability evaluation of this stack assuming that only ash would be deposited in the stack, instead of ash and gypsum. For that, the stack is assumed to be raised over the BA filter blanket by the wet operation, using compacted BA perimeter dikes. The outer slopes, drains, dike height, etc. and the foundation condition are assumed similar to the gypsum-ash stack. However, for this stack, the width of the outer compacted BA-zone is conservatively assumed to be only 120-feet horizontally, instead of the 150-foot width of cast-gypsum zone used for the gypsum-ash stack. Note that this width is greater for the gypsum-ash stack due to the rim-ditch operation.

For the ash-only stack, the evaluation is performed to examine the maximum height attainable using only the wet operation. An evaluation is also performed additionally for this stack where the wet operation is used first, followed by the dry operation.

The phreatic surface for all-wet operation stack is conservatively assumed at a depth of 10 feet below the final top based on recent observations of GWL in Cell Area condition, although the proposed new ash stack will have more efficient drainage than the existing cells in Cell Area. The phreatic surface for the wet-and-dry stack is assumed conservatively at the top of the termination of the wet operation, although it is likely to be lower than that with the planned provision for the drainage.

C. EXISTING CELLS: (For Blowout Location Stability Evaluation)

The analytical model for the interior of the existing cells and the foundation stratification at the blowout location are based primarily on the borings and soundings within Cell Area; specifically, B-1 through B-5 and CPT-1, 4 and 6. The simplified model of existing cells and foundation for this location is as follows:

Stratum No.	Elevation Range	General description
1	810'+ to 794'+	Medium dense to dense FA + BA
2	794' to 773'	Loose FA
3	773' to 763'	Medium dense to dense FA + BA
4	763' to 745'	Loose FA
5	745' to 737'	Loose FA + BA

6	737' to 730'	Loose FA
7	730' to 718'	Natural Clay, soft to stiff (CL)
8	718' to 703'	Clayey Silty Sand, Residuum (SC-SM)
9	Below703'	Bedrock (Soft Shale)

The top of the phreatic surface for this model is assumed at Elev. ~ 785'; i.e., approximately 2 feet above that observed in the monitoring well MW-3 temporarily installed near boring B-3 during the April 2004 investigation. The profile and the phreatic surface along the slope are based on the data from borings B-1, 2, 3 and monitoring wells MW-1, 2, 3 and are shown on the computer printout sketch for the blowout-location stability evaluation. *(Note that the stack height used for this evaluation corresponds to the recent condition under which the blowout occurred and not the future raised-stack condition. The latter is apparently not more critical for stability than the other conditions analyzed herein, especially those for the Ash-Only options.)*

Other details of the interior of all stacks used for the evaluation are illustrated in the computer printout sketches for each stack.

7. DESIGN MATERIAL/SOIL PROPERTIES

The design properties of various materials constituting the proposed stacks and existing ash deposits (namely, FA, FA+BA, Gypsum, and Gypsum+FA) and foundation subsoils have been determined based on the data referenced herein and as interpreted below.

Note that the test data referenced was obtained over the years from 1974 till the current year for the existing ash and foundation soils and that for the gypsum was obtained under variable conditions and locations; specifically, undisturbed and remolded conditions, and had variable aging effect. This is important to note in the case of a material like fly ash, bottom ash or gypsum that is known to harden or attain increased strength with aging in place and, when remolded, exhibits a significantly reduced strength. It should be noted further that these materials do not behave exactly like naturally occurring soils.

As far as the existing subsurface soils/materials are concerned, it should be noted that these will undergo further consolidation under a gradually raised stack over a period of more than 20 years; i.e. the loading would not be imposed suddenly and in a manner like that by a structural mat foundation but by a relatively much more flexible stack of materials of relatively huge-size and that will exhibit internal arching. In view of these factors, the strength properties selected based on past or recent data and laboratory conditions are conservative. Therefore, the reduction of shear-strength of these materials during a seismic event, although done for the natural clay (CL) soil, is unwarranted.

A. FA:

The FA is encountered as wet-placed ash in the past at the site and also will be deposited primarily by the wet operation (i.e., similarly as was done in the past) for the Ash Only Option stack. In the dry operation (a probability after termination of wet operation in the Ash Only Option), both FA and BA are likely to be mixed in a variable proportion. However, it is assumed conservatively that the dry-placed ash will consist primarily of FA and, accordingly, the design properties are determined herein.

1. Dry-Placed FA:

Per Ref.15, (noting that compacted ash gains strength as it ages in place),

For the three U.S. ashes tested @ 100% modified Proctor max. dry density (Tables 3 and 4),

Ave. Max. Dry Density, Y_{dmax} = 92.0 pcf.....(0.85 x 92.0 pcf = 78.2 pcf)

Ave. Opt. Moisture Content, w_{opt} = 24.8%, say 25%

Ave. 28-day strength (saturated):

Cohesion, c = 12 psi

Friction, $\Phi = 40.3^0$

For the four British ashes tested @ 100% std. Proctor max. dry density (Table 5),

Ave. 28-day strength (undrained):

Cohesion, c = 24 psi

Friction, $\Phi = 40.4^0$

Assume: (a) Cohesion, c = 12 psi, and $\Phi = 40^0$, at 100% density; (b) The ash actually will have an average density of 85% the maximum density (i.e., Y_d = 78.2 pcf and Y_t = say 1.25 x 78.2 pcf ~ 98 pcf).

Then, the strength @ 85% density is as follows:

Strength @ 85% density = 0.60 (Strength @ 100% density)... (P. 369, Ref. 15)

$$c = 0.6 \times 12 \text{ psi} = 7.2 \text{ psi} = 1{,}037 \text{ psf}$$

$$\Phi = \tan^{-1}(0.6 \tan 40^0) \quad = 27^0$$

For the Bull Run Facility ash-pile stability analysis, TVA used $Y_t = Y_{sat} = 106$ pcf, c = 200 psf and $\Phi = 30^0$.

Based on these data, the following properties are assumed conservatively for the stability evaluation,

c = 200 psf ; $\Phi = 30^0$; Y_t, Y_{sat} = 100, 108.4 pcf, resp. .... Dry-placed Ash Only stack

c = 100 psf ; $\Phi = 38^0$; $Y_t = Y_{sat}$ = 113.4 pcf......Well-Compacted Ash below BA Filter*

* Low cohesion value is assumed due to probable addition of bentonite to reduce its permeability, although friction angle of 38^0 (smaller than 40^0 test value) is reasonable for the well-compacted placement of this ash based on the test data presented below for the wet-placed ash.

2. Wet-Placed FA:

Per Ref. 9, for loose FA,

Ave. G_s = (2.58 + 2.42 + 2.35 + 2.52) / 4 = 2.47

Ave. w = (39 + 40 + 34 + 37.2 + 37.6 + 32 + 39 + 41 + 48) % / 9 = 38.6 %

Ave. Y_d = (76.3 + 80.3) pcf / 2 = 78.3 pcf (Y_t = 78.3 pcf x 1.386 = 108.5 pcf)

c = 0; $\Phi = 32^0$..... Effective; sample remolded @ Y_d = 78.4 pcf and saturated

Based on CPT data, for this ash, shear strength, s_u = 0.17 tsf = 340 psf. If a Mohr's envelope is drawn for the corresponding unconfined compression strength and Φ is assumed to be **28^0**, the corresponding cohesion intercept, **c = 200 psf**.

Per Ref. 7, Table 1,

Ave. G_s = (2.32 + 2.30 + 2.25 + 2.28 + 2.31 + 2.22 + 2.29 + 2.27) / 8 = 2.28

Ave. w = (34.5 + 25.8 + 42 + 34.5 + 33.2 + 35.2 + 29.7 + 31.2) % / 8 = 33.3 %

Ave.Y_d = (77.9 + 84.9 + 74.4 + 74.9 + 79.1 + 81.6 + 79.9 + 85.2 + 75.7)/8=79.9 pcf

Y_t = 79.9 pcf x 1.333 = 106.5 pcf (Y_{sat} = 108.2 pcf... w_{sat} = 35.04%)

c = 0; $\Phi = 37.5^0$ ..... Effective, Y_d = 85.2 pcf

c = 2600 psf; $\Phi = 22.3^0$ ....Undisturbed; @ field moisture content; Y_d = 85.2 pcf

TVA used the following values for their analysis,

c = 540 psf ; $\Phi = 28.3^0$ ..... Effective; saturated sample; Y_t = 99.9 pcf

c = 2,080 psf; $\Phi = 23.7^0$ .... Total; unsaturated sample; Y_t = 99.9 pcf

Based on all of the above data, the following values are selected conservatively for FA for various locations/depths (*note that the ash in-place in the stack well above general GWL should attain greater strength with age as discussed in Ref.15*):

	Cohesion, c, psf	Friction, Φ
Loose FA, existing, just above CL layer	0	28^0
Loose FA, existing, near existing GS	200	28^0
Wet-Placed FA, lowest level (Ash Only stack)...	500	28^0
Wet-Placed FA, middle level (Ash Only stack)...	200	28^0

For all wet-placed FA, it's assumed conservatively that **$Y_t = Y_{sat}$ = 108.4 pcf.**

B. FA + BA:

The grain-size analysis of the samples per data in Ref. 9 for FA+BA mixture is ML to SM-ML.

Per Ref. 9,

Ave. G_s = (2.40 + 2.35 + 2.49 + 2.29 + 2.28) / 5 = 2.36

For LOOSE condition,

Ave. w = (39 + 43 + 32.2 + 30 + 45 + 32 + 48 + 38.1 + 36.5) % / 9 = 38.2 % .. Below GWL

Ave.Y_d = (81.8 + 74.0 + 78.4) pcf / 3 = 78.0 pcf.... (Y_t = 78.0 pcf x 1.382 = 107.8 pcf)

c = 0; $\Phi = 32^0$ ..... Effective; sample was remolded @ Y_d = 78.4 pcf and was saturated

For MEDIUM DENSE condition,

Ave. w = (31 + 29 + 34 + 28) % / 4 = 30.5 % .... Below GWL

Ave.Y_d = (87.4 + 89.4) pcf / 2 = 88.4 pcf.... (Y_t = 88.4 pcf x 1.305 = 115.4 pcf)

c = 0; $\Phi = 37^0$ ..... Effective; sample remolded @ Y_d = 89.4 pcf and saturated

Per Ref. 7, Table 1,

Ave. G_s = (2.21 + 2.22 + 2.29 + 2.37) / 4 = 2.27

Ave. w = (22.3 + 26.3 + 30.3) % / 3 = 26.3 %

Ave. Y_d = (87.1 + 82.8 + 90.7) / 3=86.9 pcf ... (Y_t = 86.9 pcf x 1.263 = 109.8 pcf...S_r < 1.0)

c = 980 psf; $\Phi = 29.1^0$ ..... Effective; Y_d = 90.7 pcf

c = 0; $\Phi = 37.4^0$ .... Undisturbed; sample @ field moisture content; Y_d = 90.7 pcf

For the Ash Only stack, the perimeter dikes will consist of dry-placed and compacted mixture of BA and FA. As this zone will be exposed to air and above the phreatic surface due to the planned drainage system under each dike, a cohesion value of 100 psf and a friction angle of 38^0 along with saturated unit weight of 120.4 pcf are conservatively assumed.

Thus, and if Y_d = 78 pcf, 88 pcf and w = 39%, 30% for the LOOSE and MEDIUM DENSE conditions are assumed, respectively, the following design properties are selected for the stability evaluation:

	Y_t=Y_{sat}, pcf	Cohesion, c, psf	Friction, Φ
Loose FA+BA	**108.4**	**0**	**31⁰**
Medium Dense FA+BA..	**114.4**	**0**	**37⁰**
Compacted FA+BA in dike..	**120.4**	**100**	**38⁰**

C. Natural CL:

Per Ref. 7, Table 1,

Ave. G_s = (2.53 + 2.63 + 2.72 + 2.66) / 4 = 2.64

Ave. w = (28.8 + 22.8) % / 2 = 25.8 %

Ave. Y_d = (94.2 + 97.8) / 2 = 96.0 pcf ..... (Y_t = 96.0 pcf x 1.258 = 120.8 pcf...S_r < 1.0)

c = 800 psf; $\Phi = 22.6^0$ ..... Effective; saturated sample; Y_d = 94.2 pcf

Per Ref. 5,

Ave. w = (25.4 + 25.1) % / 2 = 25.3 % ...... Foundation CL; US-7; saturated moisture content

Ave. Y_d = (99.9 + 99.6) / 2 = 99.8 pcf .... Foundation CL; US-7

Y_{sat} = 99.8 pcf x 1.253 = 125.0 pcf

NOTE: Triaxial-shear test results in this data appear unreliable and are not considered.

Per Ref.9,

G_s = 2.68

W = 21.9%

Y_d = (102.2 + 102.4) / 2 = 102.3 pcf ...... w_{sat} = 23.6%

Y_{sat} = 102.3 pcf x !.236 = 126.4 pcf

The CPT data for this stratum gives the following interpreted strength values:

CPT Ave. s_u in tsf

1A	0.68	
4	0.82	
6	0.53	
DN	0.375	
DS	0.84	
8	0.87	
9	0.47	
12	1.00	Average s_u = 0.70 tsf = 1,400 psf

If the Mohr's envelope is drawn for this strength and if $\Phi = 23^0$ is assumed for this soil, a cohesion intercept of 1,000 psf is obtained. This is close to Ref. 7 data of 800 psf. However, for the static-condition stability evaluation herein, the following values are conservatively used for this soil:

$$Y_t = Y_{sat} = 126.4 \text{ pcf}$$

$$c = 400 \text{ psf}$$

$$\Phi = 23^0$$

For the seismic condition, the strength is reduced to 80% of the maximum strength per the Guidance Document as follows:

$$Y_t = Y_{sat} = 126.4 \text{ pcf}$$

$$c = 0.8(1{,}000 \text{ psf}) = 800 \text{ psf}$$

$$\Phi = \tan^{-1}(0.8 \times \tan 23^0) = 19^0$$

It should be noted that the design seismic event is a low probability occurrence. Also, both the clayey CL subsoil and the overlying existing loose ash are likely to gain strength due to further consolidation under a significantly greater surcharge load in the future compared to the present condition under which the strength was measured insitu anddue to a planned provision of enhancement of drainage of the loose ash. Thus, the strength values assumed above for these two materials are conservative.

D. Clayey Silty Sand (SC-SM)

This naturally existing soil is primarily the residuum soil originating from the parent bedrock. Therefore, although it apparently includes pockets of soft or loose soil, the shear strength of the overall stratum is likely to be significantly high enough not to be a concern for the stability. As it was difficult to obtain really undisturbed and representative samples of this soil for the strength testing during the investigations due to variability and sand content under below GWL condition, the design properties as explained below are based on the available unit weight- moisture content, triaxial shear testing of remolded samples and conservative average value of the CPT tip-resistance data.

Per Ref. 7, Table 1,

Ave. $G_s = (2.66 + 2.66 + 2.64 + 2.67) / 4 = \underline{2.66}$

Ave. w $= (19.9 + 20.6 + 18.6 + 17.2 + 22.0)\ \% / 5 = 19.9\ \%$, say, $20.0\ \% = w_{sat}$

Ave.$\Upsilon_d = (106.8 + 105.6 + 112.9 + 112.6 + 103.5) /5 = 108.3$ pcf

$\Upsilon_t = \Upsilon_{sat} = 108.3$ pcf x $1.20 = \underline{130.0}$ pcf

c = $\underline{1,200}$ psf; $\Phi = \underline{29.6}^0$ ..... Effective; saturated sample; $\Upsilon_d = 112.9$ pcf

Per Ref. 5,

Ave. w $= (22.7 + 17.1)\ \% / 2 = 19.9\ \%$ ...... ($w_{sat} = 21.4\%$) ....US-1 and US-7 samples

Ave. $\Upsilon_d = (102.8 + 110.7) / 2 = 107.3$ pcf .... US-1 and US-7 samples

$\Upsilon_{sat} = 107.3$ pcf x $1.214 = \underline{130.3}$ pcf

c = $\underline{620\text{ psf}}$; $\Phi = \underline{31}^0$ ..... Effective; saturated sample; $\Upsilon_d = 102.8$ pcf

Per Ref.9,

$G_s = 2.67$

$w = (21.9 + 20.0) / 2 = 21.0\ \%$

$\Upsilon_d = 108.3$ pcf ...... $w_{sat} = 20.2\%$

$\Upsilon_{sat} = 108.3$ pcf x $1.202 = \underline{130.2}$ pcf

The CPT data for this stratum gives an interpreted unconfined compressive strength of approx. 2.0 tsf. Assuming $\Phi = \underline{30^0}$, a cohesion intercept of approximately <u>1,200</u> psf is obtained for this strength if a Mohr's envelope is drawn. Thus, for this soil, the following conservative values are used conservatively for the analysis:

$$\mathbf{Y_t = Y_{sat} = 130.4\ pcf}$$

c = use 0.5 (1,200 psf) = 600 psf

$$\mathbf{\Phi = 30^0.}$$

As the strength of this soil is already reduced, considering it to be the residuum soil, there is no need to reduce its strength further for the seismic condition.

E. <u>Soft Shale</u>:

No strength testing was required for the bedrock as the slip circles are not likely to penetrate it. However, the values are required for the computer-program input. The following values for the rock are used for both static and seismic conditions:

$$\mathbf{Y_t = Y_{sat} = 150\ pcf}$$

c = 3,000 psf

$$\mathbf{\Phi = 42^0}$$

F. <u>Gypsum Sludge</u>:

It is assumed that the sludge will be piped in the form of water-based slurry (wet-placement) and discharged at the stack using the rim-ditch concept until the stack reaches Elev. 930' above which it will be placed using the dry-placement method. The sludge will consist primarily of calcium sulphate or gypsum. Thus, design properties for cast gypsum, sedimented (wet-placed) gypsum and dry-placed gypsum are determined herein.

It is important to note that the gypsum stacks reportedly are observed to sustain steep slopes, indicative of its relatively high shear strength, especially the cohesive bondage when exposed to air. Ref. 13 states (P. 10-137), "Gypsum stacks over 100 feet high with average side slopes as steep as 1.5 horizontal to 1.0 vertical are not uncommon". Mactec also observed (Ref. 11) that after the sedimented gypsum is allowed to dry "Near vertical cuts of 20 feet or more show little if any signs of slope failure or even raveling after being exposed for several months." On the other hand, there is an indication (Ref. 13, page. 10-137) that there is "no measurable change in the shear strength or permeability of gypsum within the stack" where gypsum is not exposed to air and "indicated no cementation" (although it perhaps does crystallize in the interior portions of stacks also as long as free water is available).

The properties of the sludge at disposal sites have been studied extensively elsewhere besides TVA sites (Ref.13). Reviewing this information, it can be seen that various factors govern the properties of gypsum in a stack. That is also likely to be the case at the Kingston site just like other existing disposal sites. Therefore, the design properties selected for this analysis are primarily derived from the EPRI data (Ref. 13), using TVA-site data (Ref. 11) as supplementary data, and also are based on the observations described in the preceding paragraph.

Per data presented in Ref. 11 for sedimented gypsum,

Y_{sat} = (104.3 + 103.8 + 100.6 + 102.0) pcf / 4 = 102.7 pcf ...(Samples 2 & 4, S_r = 1.0)

c = 0

$\Phi = (40.4^0 + 39^0) / 2 = 39.7^0$, say 40^0

Per data presented in Figures 10-62 and 10-63 of Ref. 13,

c = 0

$\Phi = 40^0$ to 42^0 @ Y_d = 78 to 82 pcf or Y_{sat} = 107.1 to 109.4 pcf....... *Sedimented Gypsum*

$\Phi = 41^0$ to 47^0 @ Y_d = 87 to 103 pcf or Y_{sat} = 112.2 to 121.4 pcf .... *Cast Gypsum*

(Assumed G_s = 2.34)

Per Ref. 14 (gypsum + FA mixture),

c = 0

$\Phi = 41^0 \pm 2^0$ @ Y_d = 91.5 pcf or Y_{sat} = 117.4 pcf ...... *Sedimented*

$\Phi = 43^0$ @ Y_d = 96.2 pcf or Y_{sat} = 120.2 pcf .... *Cast*

For dry-placed gypsum, using Table 3-11 of Ref. 13,

c ~ (0 + 5) psi / 2 = 2.5 psi =360 psf.

$\Phi = (31^0 + 39^0)/2 = 35^0$

Arbitrarily, assume Y_t = 102 pcf and Y_{sat} = 107 pcf due to some compaction effort.

The following design properties for the sludge are used for the analysis:

	Y_t, pcf	Y_{sat}, pcf	Cohesion, c, psf	Friction, Φ
Sedimented Gypsum ...	**116.4**	**116.4**	**0**	**40^0**
Cast Gypsum ...	**120.4**	**120.4**	**100**	**43^0**
Dry-Placed Gypsum ...	**102**	**105**	**350**	**35^0**

8. SLOPE STABILITY EVALUATION

The slope stability evaluation is performed using the computer program PC STABL5M. The program is based on the pseudo-static method of analysis where a mass or a part of the slope of varying size is assumed to fail along a cylindrical or predetermined surface (for sliding block analysis). The resistance to sliding is provided by friction and adhesion along the surface of sliding. The program automatically searches for the most critical cylindrical surface of sliding that gives the least factor of safety against such a failure and uses the same method for both static and seismic conditions.

For the seismic condition, a horizontal destabilizing force is added to the total sliding force, that is equal to the weight of the sliding mass times a seismic coefficient, k_s. Based on extensive studies performed in the past, as discussed in Reference 16, the coefficient is found to be significantly smaller than the peak or maximum ground acceleration a_{max}/g, where a_{max} is the peak or maximum horizontal ground acceleration and g is the acceleration due to the gravity. This is simply due to the fact that the sliding mass is subjected to the peak acceleration at any one point in the mass at a time only for a fraction of a second and does not occur simultaneously at all points during an earthquake. Thus, for a simplified analysis such as the pseudo-static analysis, the coefficient corresponds to an average effective acceleration across the mass. Due to the complexity factors and difficulty involved in the determination of this effective acceleration in a mass during an earthquake, accurate value of the coefficient for such an analysis perhaps will never be determined and, therefore, an empirical value has been recommended based on observations and studies for failures in the past. A coefficient value of equal to 0.5 is found to be adequate and is recommended in Reference 16 and other publications referenced in it for the pseudo-static analyses of slopes.

According to Guidance Document, the peak ground acceleration is the "maximum horizontal acceleration in lithified earth material", corresponding to a "90 percent or greater probability that the acceleration will not be exceeded in 250 years." The document also states, "lithified earth materials means all rock, including all naturally occurring and naturally formed aggregates of masses of minerals or small particles of older rock that formed by crystallization of magma or by induration of loose sediments.... This term does not include man-made materials, such as fill, concrete, and asphalt or UNCONSOLIDATED earth materials, soil, or regolith lying at or near the earth's surface".

The peak acceleration in the bedrock at the site in accordance with Guidance Document is approximately 0.22g. Since the natural soil overburden over the bedrock at the site is very shallow (hardly 730 feet – 703 feet = 27 feet) and generally stiff, with the ash or ash-gypsum stack being medium stiff to stiff, the maximum ground acceleration for this evaluation is assumed to be the same as that in the rock (i.e., 0.22*g*) in accordance with Figure 4.2 of Reference 16. This means that the average effective acceleration in a sliding mass of the stack during the design earthquake is likely to be 0.11*g*; i.e. equal to one-half of the peak ground acceleration (a_{max}).

In accordance with the recommended procedure (Ref. 16, Page 84), the computer program is utilized herein to obtain the acceleration, called the *yield acceleration* k_y, at which the factor of safety equals approximately 1.00 against the failure. The procedure further recommends that if the yield acceleration so obtained is equal to or greater than 0.5 a_{max} *(i.e.,* 0.11*g* in this case*)*, the slope is likely to be stable during the design earthquake and no further verification by computing the deformation based on the Seed-Makdisi procedure is required. The deformation in such a case is found to be almost always less than one foot which is generally acceptable and, hence, not necessary to be computed. Thus, as the yield acceleration is equal to or greater than 0.11*g* in all cases (See Table 1) and as conservative soil/material parameters were used for the evaluation, the deformation analysis is not required and is not included herein.

9. RESULTS OF SLOPE STABILITY EVALUATION

The results of the stability evaluation for the three critical sections listed before, representing the three phases of the proposed construction, and for the section at the blowout location are summarized in Table 1. The table gives the factor of safety against the slope failure corresponding to the static condition and the yield acceleration values corresponding to the design seismic event. The table also includes the results of the sliding block analysis performed for the most critical condition; i.e., Phase 3 or Final condition at Section 3-3. The details of both input and output for each computer run are given in the printouts in Attachment A.

TABLE 1

SLOPE STABILITY FACTOR OF SAFETY & YIELD ACCELERATION

Run No.	Section (Phase)	Stack Type	Condition	F.S.	Yield Accel
1	1-1 (Phase 1 End)	Ash Cells	Static	2.00	-
2	1-1 (Phase 1 End)	Ash Cells	Seismic	-	0.18*g*
3	2-2 (Phase 2 End)	Gypsum+Ash	Static	1.90	-
4	2-2 (Phase 2 End)	Gypsum+Ash	Seismic	-	0.20*g*
5	3-3 (Final)	Gypsum+Ash	Static	1.73	-
6	3-3 (Final)	Gypsum+Ash	Seismic	-	0.16*g*
7	3-3 (Final)	Ash Only (Wet)	Static	1.51*	-
8	3-3 (Final)	As Only (Wet)	Seismic	-	0.11*g*
9	3-3 (Final)	Ash Only (Wet&Dry)	Static	1.52**	-
10	3-3 (Final)	As Only (Wet&Dry)	Seismic	-	0.11*g*
11	3-3 (Final)	Gypsum+Ash	Static(Slid.Block)	1.77	-
12	3-3 (Final)	Gypsum+Ash	Seismic(Slid.Block)	-	0.16*g*
13	1-1 (Current)	Ash Cell 3 (Blowout)	Static	1.48***	

* Maximum Stack Elev. 930'

** Maximum Stack Elev. 965'

*** Top of Cell assumed at Elev. 810'; GWL assumed at least 5 feet above the level on May 14, 2004.

It should be noted that the actual factor of safety will be significantly greater than the tabulated values due to the three-dimensional effect since the values calculated above were based not only on conservative soil or material parameters but also an assumption that a stack consists of a two-dimensional or an infinitely long embankment whereas the actual stack would have finite length and would be closer to a square-shaped body than that resembling a long embankment. Also, the stacks are bounded by the perimeter dikes composed of much stronger materials than that in the interior areas. Therefore, the resistance to a slide was derived primarily from the weaker interior slope, yielding lower values of the factor of safety than in the actual case.

10. VENEER STABILITY EVALUATION

The veneer stability is evaluated along the sloped surface of the final stack using the *landfilldesign.com* calculators (Attachment B). In accordance with the recommendation in the Guidance Document, a seismic coefficient of 0.11*g* is used. As the final cover is required to consist of a cohesive clayey soil, perhaps mixed with gypsum, two soil-cover cohesion values are used: 250 psf (12.0 kN/m^2) and 100 psf (4.8 kN/m^2) along with the friction angle of 26^0. The latter value of cohesion may be considered to correspond conservatively to a softened condition after rain. As the slope will have 15 feet wide benches at 30-feet vertical height intervals, a slope length of 90 feet (= 3 x 30' or 27.43 m) is used for the evaluation. Also, it is assumed for this analysis that the surficial slope material underlying the cover consists of either the cast gypsum or BA for which the surface friction will be approximately equal to two-thirds of 38^0 (lower of the friction angle values corresponding to the two materials); i.e., equal to 25^0. The results are summarized in Table 2 below.

TABLE 2

VENEER STABILITY FACTOR OF SAFETY

Condition	Seismic Coefficient	Cover Soil Cohesion, psf	F.S.
Static	-	250	1.761
Static	-	100	1.588

Seismic	0.11*g*	250	1.283
Seismic	0.11*g*	100	1.154

Thus, the soil cover is likely to be stable during both the static and design seismic conditions.

11. SITE LIQUEFACTION POTENTIAL

The ash site where the proposed facility will be located has been a permitted facility that has been used so far to deposit wet-sluiced ash. Therefore, several tens of feet of sedimented or settled ash (approximately 30 feet in the Ash Pond area and 75 feet in the Cells area) cover the natural soil strata at the site. Furthermore, the top natural soil stratum consists of generally stiff cohesive clay soil that is underlain by a stratum of cohesive residuum soil stronger than the clay soil. Bedrock exists below these two soil strata. Thus, the two natural soil strata are not likely to liquefy except at isolated loose cohesionless sand pockets that may exist in these two strata. Liquefaction of such pockets, if any, is likely to be inconsequential for this facility.

However, a 7 feet to 10 feet thick stratum of loose ash appears to exist immediately above the natural clay stratum; i.e., at a depth of approximately 20 feet in Ash Pond Area and more than 60 feet in Cell Area below the present GS in those areas. This stratum of loose ash may undergo an an initial liquefaction in Ash Pond Area due to insufficient existing overburden load on it if a design seismic event occurs at the site before it is buried under a sufficient overburden of ash; i.e., roughly 10 feet to 30 feet of additional ash or ash and gypsum, depending on the depth to GWL at the time of such an occurrence. The probability of such an occurrence is extremely low.

Theoretically, once this stratum is buried under a sufficient overburden load, it is not likely to liquefy but it is likely to undergo significant settlement subsequent to the occurrence of a design seismic event. A rough estimate shows that the total settlement resulting from such an occurrence is not likely to be greater than one foot and, hence, of no serious consequence. Based on Figure 8 in the Guidance Document, which is based on a 1985 study by Ishihara, it is not likely that the surface manifestation of liquefaction will occur as long as this 3-meter thick stratum is at least 3 meters (10 feet) or more below GS.

The subsurface exploration data also shows, as is expected due to the nature of wet-ash disposal and due to the very large area of the disposal site, that pockets (and not continuous strata or layers) of liquefiable ash may exist occasionally at depths shallower than the liquefiable ash stratum discussed above. However, liquefaction of such pockets for a facility such as this should not be of any serious consequence.

There are no theories that can accurately predict degrees of liquefaction as it depends on the distance of the epicenter of the earthquake, energy released, and the nature of dissipation or dispersal of this energy that depends on the nature and extent or continuity of soil overburden above bedrock strata in its path. The methods that predict liquefaction and its effects are empirical and have often proved to be insufficient. Therefore, it is recommended that measures be taken to improve drainage and consequently rate of consolidation of this loose ash stratum at least at a critical location. For this, it is suggested that columns of coarse ash, similarly to gravel columns, be inserted into this stratum and connected to the proposed BA filter system located at the bottom of the proposed gypsum-ash stack. The appropriate location of these columns would be at or near the inner toe of the starter perimeter dike along the Stilling-Basin side of the stack. This provision will facilitate dissipation of generated higher pore-water pressure in this stratum, if any, and allow it to consolidate faster. This will also improve stability of the critical toe area.

12. CONCLUSION AND RECOMMENDATIONS

1. The proposed raised Cell Area ash stack and gypsum-ash stack (wet-placed to Elevation 930' and dry-placed above it) are likely to be stable during any stage of construction and after completion of construction including during the occurrence of the design seismic event. Although a stability evaluation is not performed for a Phase 2 condition that may require the stack raised at the end of Phase 2 to Elevation 870' instead of 840', it can be deduced based on the factor of safety values obtained for the Phase 3 that the factor of safety for that condition will be satisfactory for both static and seismic conditions.

2. If instead of gypsum and ash only ash is used, it is estimated that the stack can be raised maximum to Elevation 930' if the ash is deposited using only wet operation, and to Elevation 965' if wet operation is terminated at Elevation 870' and dry stacking used above that.

3. If a clayey soil cover or veneer is used to cap the final stack, it is likely to remain stable even during the design seismic event if the cohesion and friction values of the cover soil are greater than 100 psf and 26^0, respectively.

4. The existing Ash Pond Area should be graded flat and the graded surface should be compacted using heavy compaction equipment prior to placement of the

compacted fly-ash base. The GWL should be lowered by several feet to help stabilize the graded surface. Note that bottom ash and/or Tensar geogrid or similar geonet reinforcement may be required to support construction equipment in soft areas.

5. Adequate drainage must be provided to control the phreatic water surface inside the stack.

6. It is also recommended that measures be taken to enhance drainage and consolidation of the existing approximately 7 to 10 feet thick loose ash stratum immediately overlying the natural clayey soil stratum, especially below the starter perimeter dike for the gypsum-ash stack and adjacent to the Stilling Basin. Use of coarse bottom ash columns, like gravel columns, installed by drilling to the bottom of this stratum and tying the columns to the bottom filter blanket should be adequate for improving the strength against probable instability during the design seismic event.

END

.REFERENCES

1. Drawing Nos. 10W425-22, 23, 34A, 34B, 34C (showing proposed Phase 1, 2 and 3 plans and selected critical sections 1-1, 2-2, and 3-3).

2. TVA Drawing No. 10N400 – R6, dated 7-5-56 (showing original surface topography).

3. TVA Drawings No. 10N420 and 10N421, dated 5-6-77 and 10-13-77, resp., (showing sections of Dike C, Road Dike and Divider Dike).

4. TVA document titled, "Ash Disposal Area Dike Raising – Soils Exploration and Testing" by Gene Farmer and W.W. Engle, dated June 26, 1974.

5. TVA's "Soil Investigation" report by G.L. Buchanan and Gene Farmer, dated November 3, 1975.

6. U.S. Government reports titled, *KINGSTON STEAM PLANT - DIKE C, - SOILS INVESTIGATION – EN DES SOIL SCHEDULE 82.3*, dated June 22, 1984 and January 10, 1985.

7. Singleton Laboratories' report titled, *KINGSTON FOSSIL PLANT – DREDGE CELLS CLOSURE SOILS INVESTIGATION*, dated September 29, 1994.

8. TVA report titled, *Hydro geologic Evaluation of Ash Pond Area*, dated June 1995.

9. Mactec report titled, *REPORT OF GEOTECHNICAL EXPLORATION, ASH DISPOSAL AREA,* dated May 4, 2004.

10. Drawing No. SK PR0637 C80 (showing locations of borings drilled prior to 2004).

11. Mactec report titled, *Laboratory Testing Results – Samples from Gypsum Pond at Cumberland Fossil Plant,* dated May 13, 2004.

12. Law Engineering's FINAL REPORT - *Fly Ash, Bottom Ash and Scrubber Gypsum Study* - to TVA dated November 7, 1995, along with transmittal letter dated November 10, 1995.

13. EPRI Manual TR104731 titled, *FGD by-Product Disposal Manual, Fourth Edition*, August 1995.

14. Report by Ardaman & Associates, Inc., titled, "Interim Report on Evaluation of Gypsum-Flyash Wet-Stacking Disposal facility, Widows Creek Steam Plant", dated April 22, 1991.

15. D. H. Gray and Y.-K. Lin, *Engineering Properties of Compacted Ash,* ASCE Journal of The Soil Mechanics & Foundation Division, April 1972.

16. Seminar proceeding titled, Seismic Analysis and Design Considerations for Municipal Solid Waste Landfills, March 2-3, 1994, sponsored by New York Association for Solid Waste Management, NY Dept. of Environmental Conservation and U.S. EPA.

17. The University of British Columbia, Canada, soil Mechanics Series Nos. 157 & 158, *Interpretation of Piezocone Test Data for Geotechnical Design, by R.G. Campanella et al*, September 1995.

ATTACHMENT 1

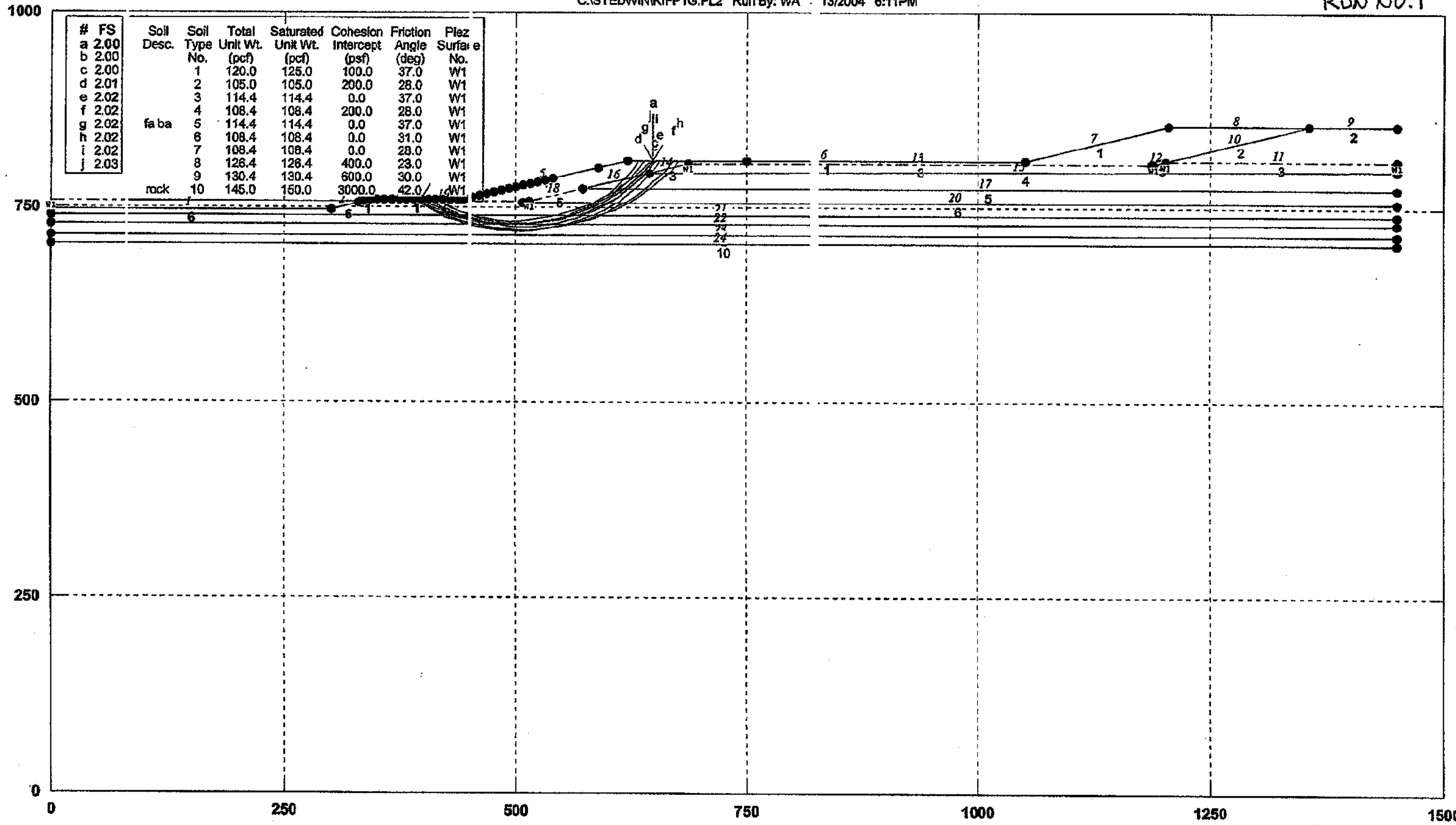
SLOPE STABILITY COMPUTER PRINTOUTS

(13 Pages)

KIF Phase 1 (Sectio ι 1-1)

C:\STEDWIN\KIFP1G.PL2 Run By: WA · 13/2004 6:11PM

Run No. 1



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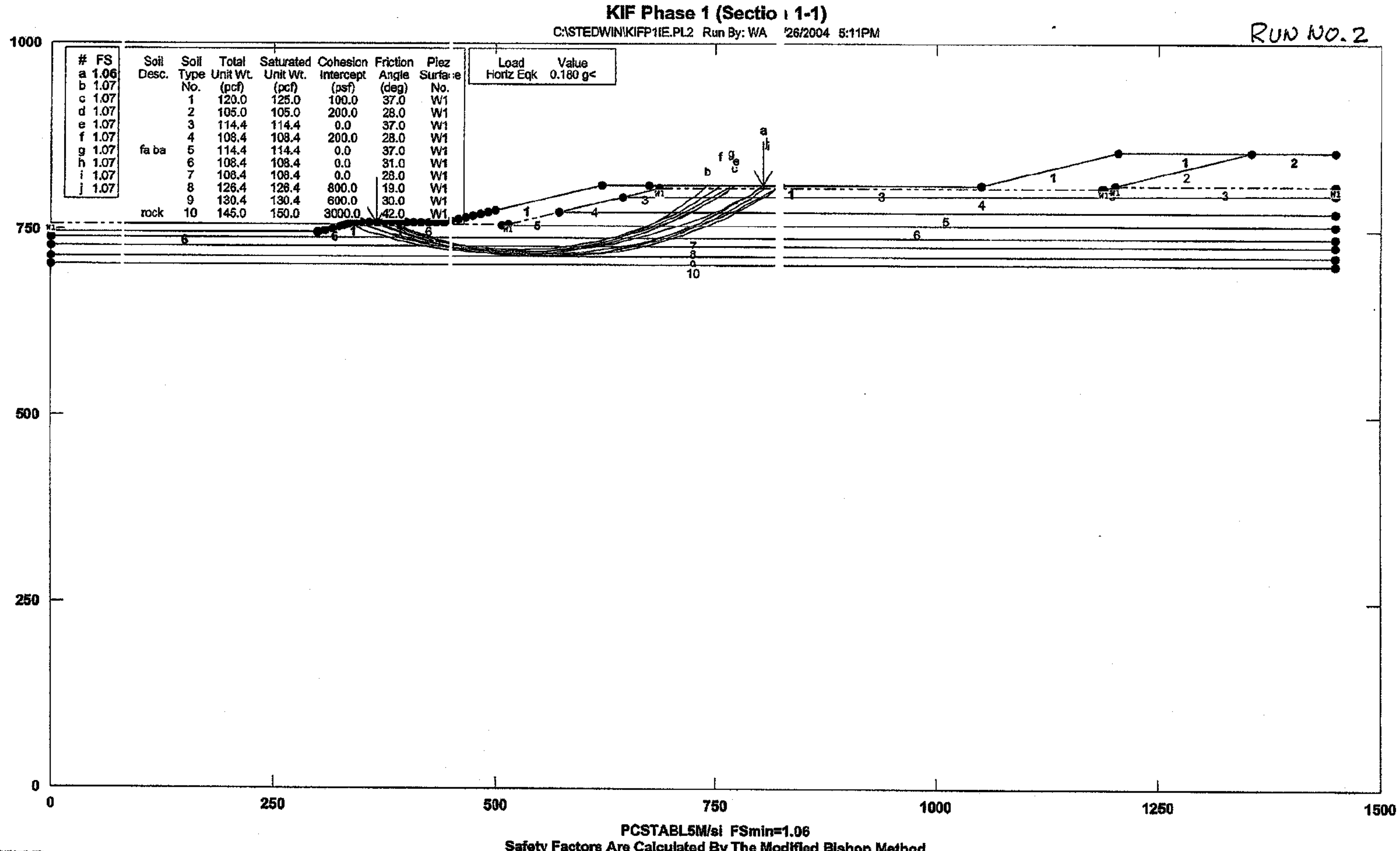
Safety Factors Are Calculated By The Modified Bishop Method

STED



#	FS
a	**1.06**
b	1.07
c	1.07
d	1.07
e	1.07
f	1.07
g	1.07
h	1.07
i	1.07
j	1.07

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez Surface No.
	1	120.0	125.0	100.0	37.0	W1
	2	105.0	105.0	200.0	28.0	W1
	3	114.4	114.4	0.0	37.0	W1
	4	108.4	108.4	200.0	28.0	W1
fa ba	5	114.4	114.4	0.0	37.0	W1
	6	108.4	108.4	0.0	31.0	W1
	7	108.4	108.4	0.0	28.0	W1
	8	126.4	126.4	800.0	19.0	W1
	9	130.4	130.4	600.0	30.0	W1
rock	10	145.0	150.0	3000.0	42.0	W1

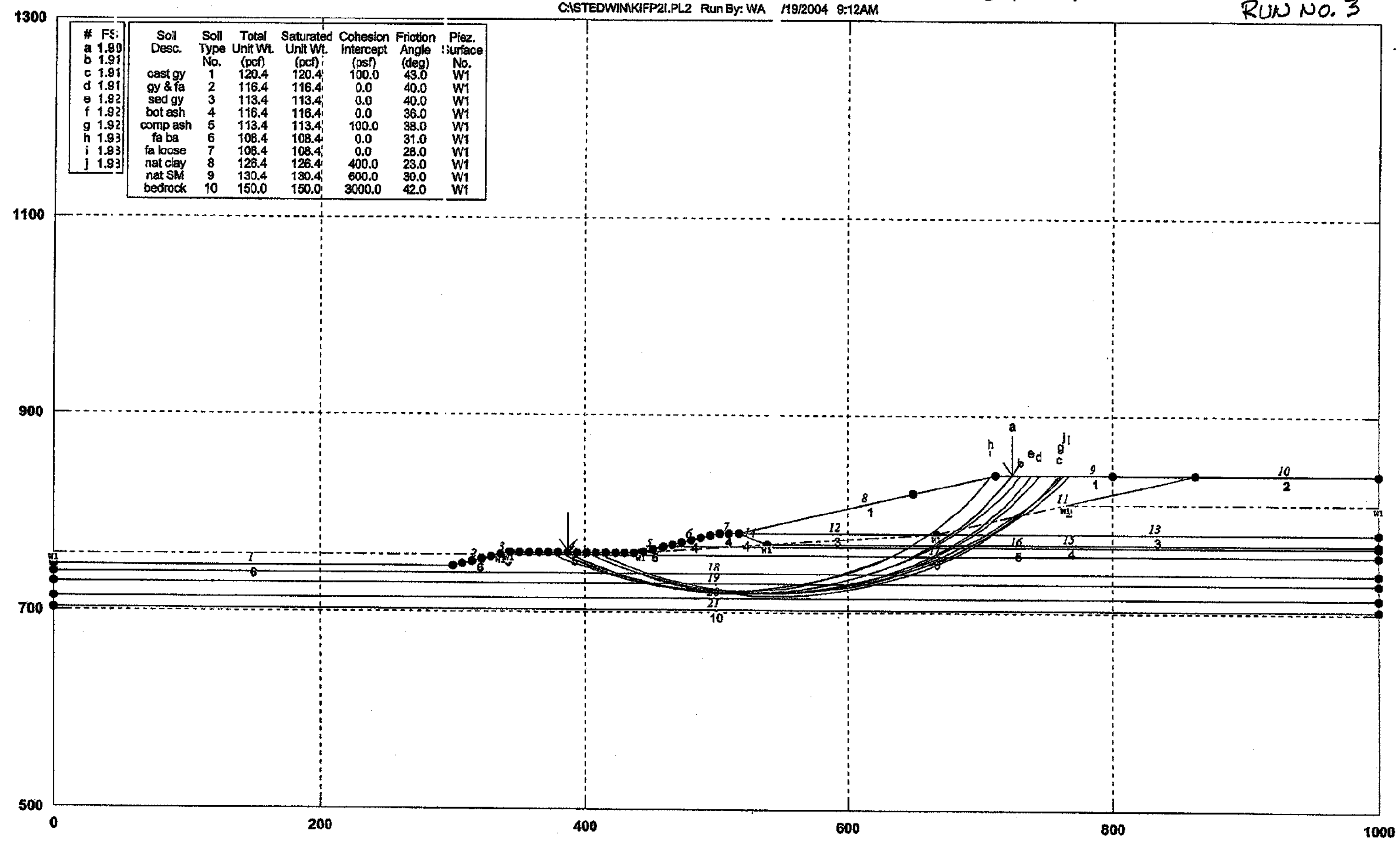
Load	Value
Horiz Eqk	0.180 g<

STED

KIF Phase 2 (Section 2-2) Gypsum Ash WetPlacement Intermediate Stage (el 840')

C:\STEDWIN\KIFP2I.PL2 Run By: WA /19/2004 9:12AM

RUN NO. 3



#	FS
a	1.90
b	1.91
c	1.91
d	1.91
e	1.92
f	1.92
g	1.92
h	1.93
i	1.93
j	1.93

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
cast gy	1	120.4	120.4	100.0	43.0	W1
gy & fa	2	116.4	116.4	0.0	40.0	W1
sed gy	3	113.4	113.4	0.0	40.0	W1
bot ash	4	116.4	116.4	0.0	36.0	W1
comp ash	5	113.4	113.4	100.0	38.0	W1
fa ba	6	108.4	108.4	0.0	31.0	W1
fa loose	7	108.4	108.4	0.0	28.0	W1
nat clay	8	126.4	126.4	400.0	23.0	W1
nat SM	9	130.4	130.4	600.0	30.0	W1
bedrock	10	150.0	150.0	3000.0	42.0	W1

PCSTABL5M/sl FSmin=1.90

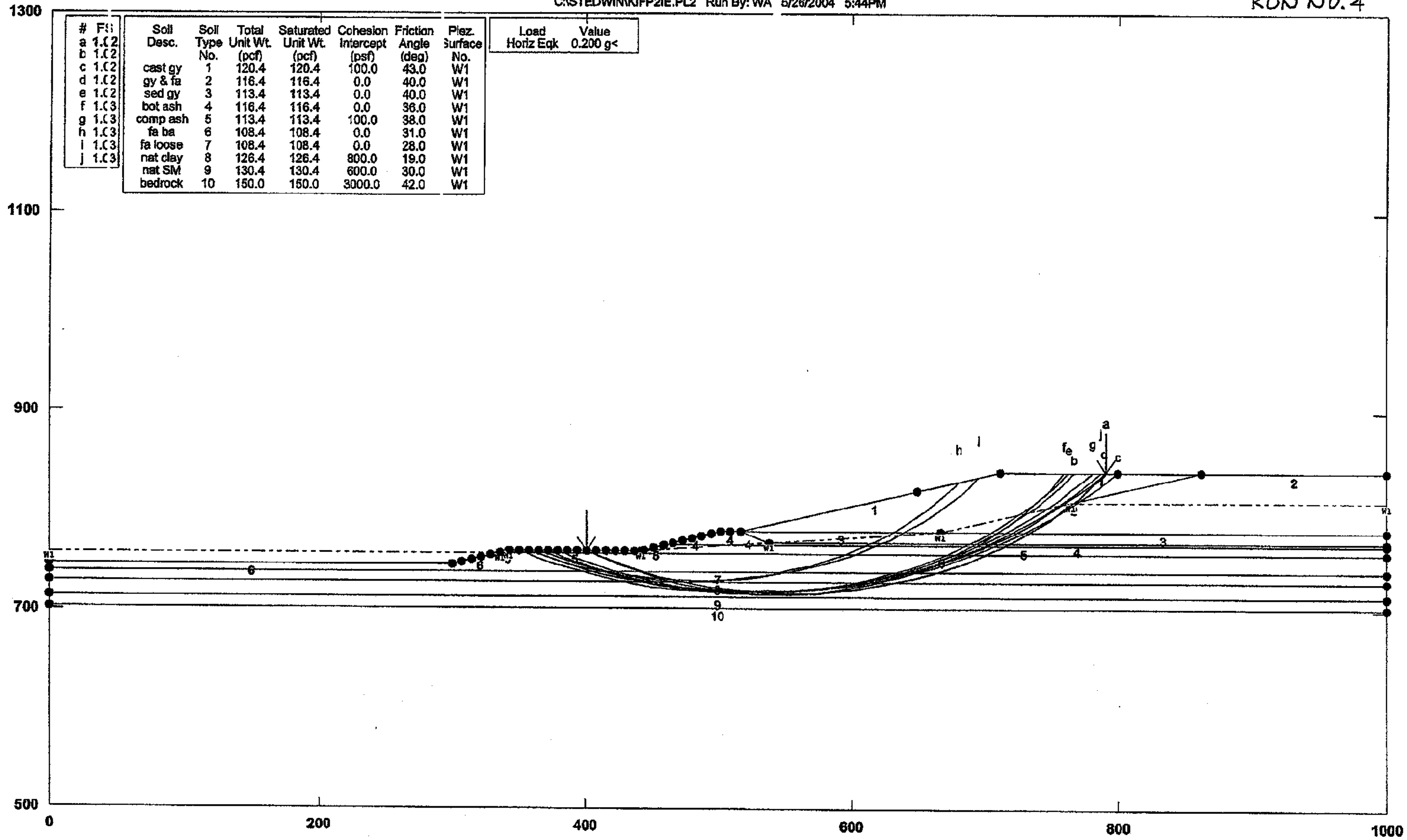
Safety Factors Are Calculated By The Modified Bishop Method

STED

KIF Phase 2 (Section 2-2) Gypsum Ash WetPlacement Intermediate Stage (el 840')

C:\STEDWIN\KIFP2IE.PL2 Run By: WA 5/26/2004 5:44PM

RUN NO. 4



#	FS
a	1.02
b	1.02
c	1.02
d	1.02
e	1.02
f	1.03
g	1.03
h	1.03
i	1.03
j	1.03

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
cast gy	1	120.4	120.4	100.0	43.0	W1
gy & fa	2	116.4	116.4	0.0	40.0	W1
sed gy	3	113.4	113.4	0.0	40.0	W1
bot ash	4	116.4	116.4	0.0	36.0	W1
comp ash	5	113.4	113.4	100.0	38.0	W1
fa ba	6	108.4	108.4	0.0	31.0	W1
fa loose	7	108.4	108.4	0.0	28.0	W1
nat clay	8	126.4	126.4	800.0	19.0	W1
nat SM	9	130.4	130.4	600.0	30.0	W1
bedrock	10	150.0	150.0	3000.0	42.0	W1

Load	Value
Horiz Eqk	0.200 g<

PCSTABL5M/si FSmin=1.02

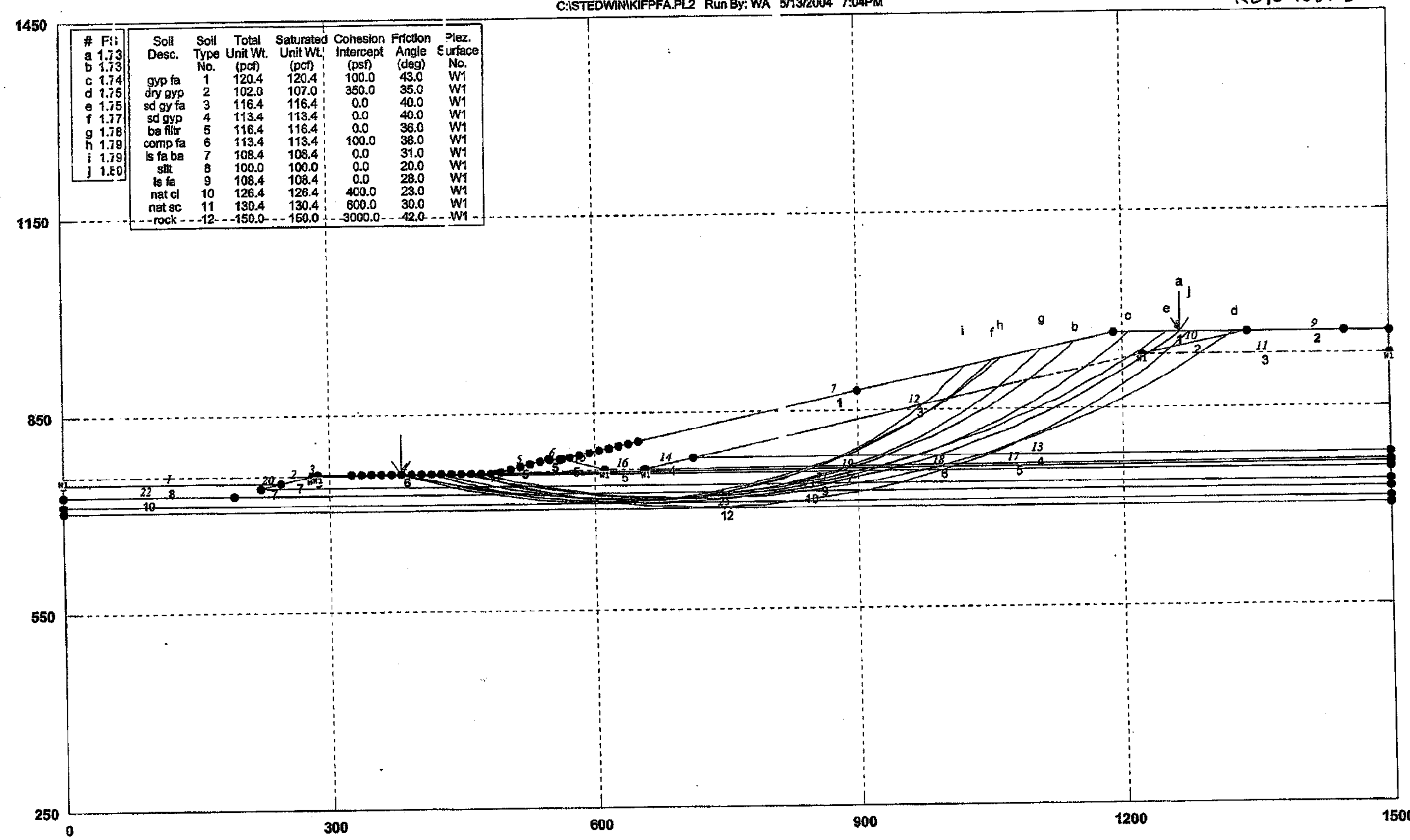
Safety Factors Are Calculated By The Modified Bishop Method

STED

KIF Phase 3 (Section 3-3) Final Stack Gypsum and Wet Ash Placement to 930

C:\STEDWIN\KIFPFA.PL2 Run By: WA 5/13/2004 7:04PM

RUN NO. 5



#	FS
a	1.73
b	1.73
c	1.74
d	1.75
e	1.75
f	1.77
g	1.78
h	1.78
i	1.79
j	1.80

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
gyp fa	1	120.4	120.4	100.0	43.0	W1
dry gyp	2	102.0	107.0	350.0	35.0	W1
sd gy fa	3	116.4	116.4	0.0	40.0	W1
sd gyp	4	113.4	113.4	0.0	40.0	W1
ba filtr	5	116.4	116.4	0.0	36.0	W1
comp fa	6	113.4	113.4	100.0	38.0	W1
ls fa ba	7	108.4	108.4	0.0	31.0	W1
silt	8	100.0	100.0	0.0	20.0	W1
ls fa	9	108.4	108.4	0.0	28.0	W1
nat cl	10	126.4	126.4	400.0	23.0	W1
nat sc	11	130.4	130.4	600.0	30.0	W1
rock	12	150.0	150.0	3000.0	42.0	W1

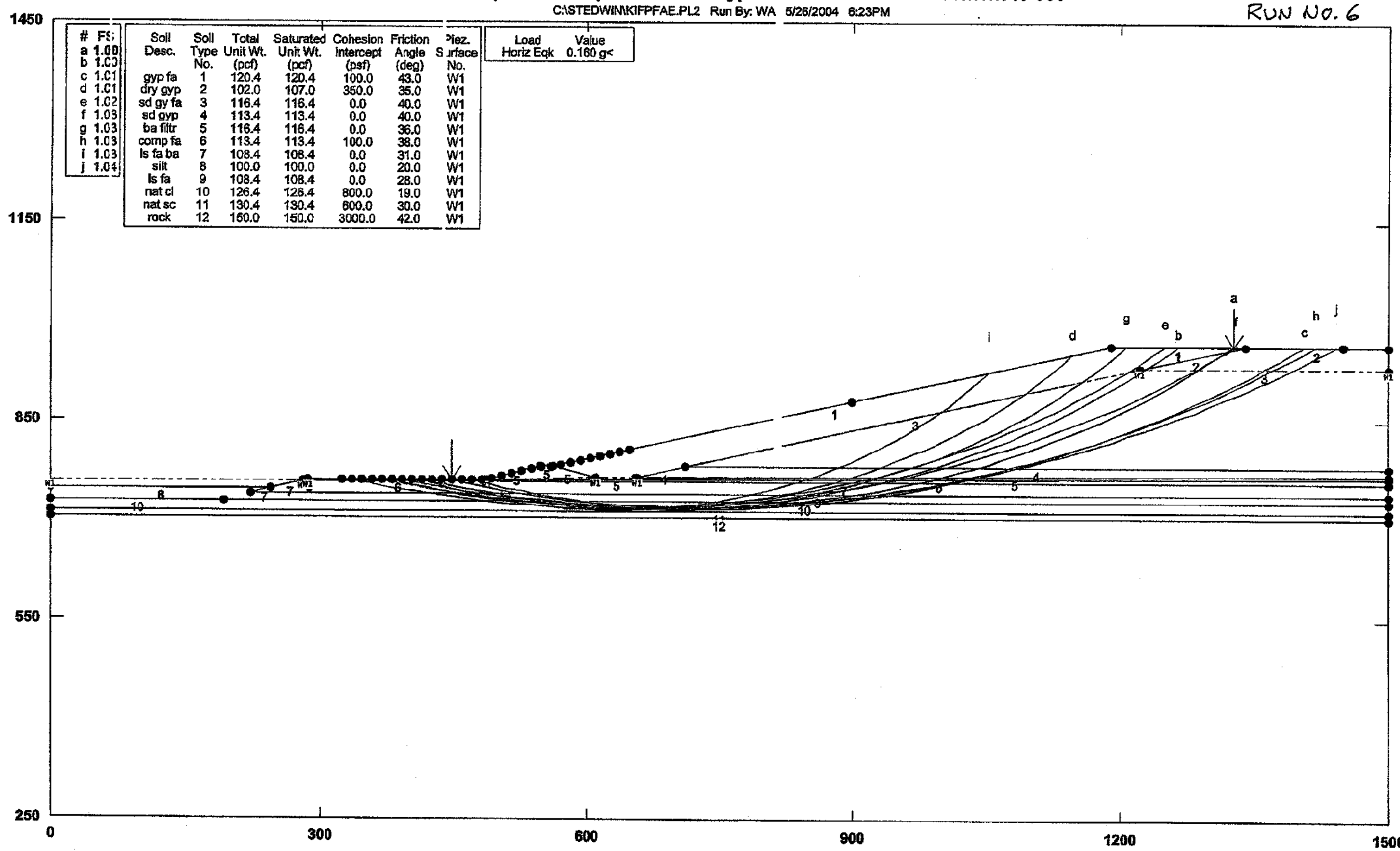
PCSTABL5M/si FSmin=1.73
Safety Factors Are Calculated By The Modified Bishop Method

STED

KIF Phase 3 (Section 3-3) Final Stack Gypsum and Wet Ash Placement to 930

C:\STEDWIN\KIFPFAE.PL2 Run By: WA 5/28/2004 6:23PM

RUN NO. 6



PCSTABL5M/si FSmin=1.00

Safety Factors Are Calculated By The Modified Bishop Method

STED

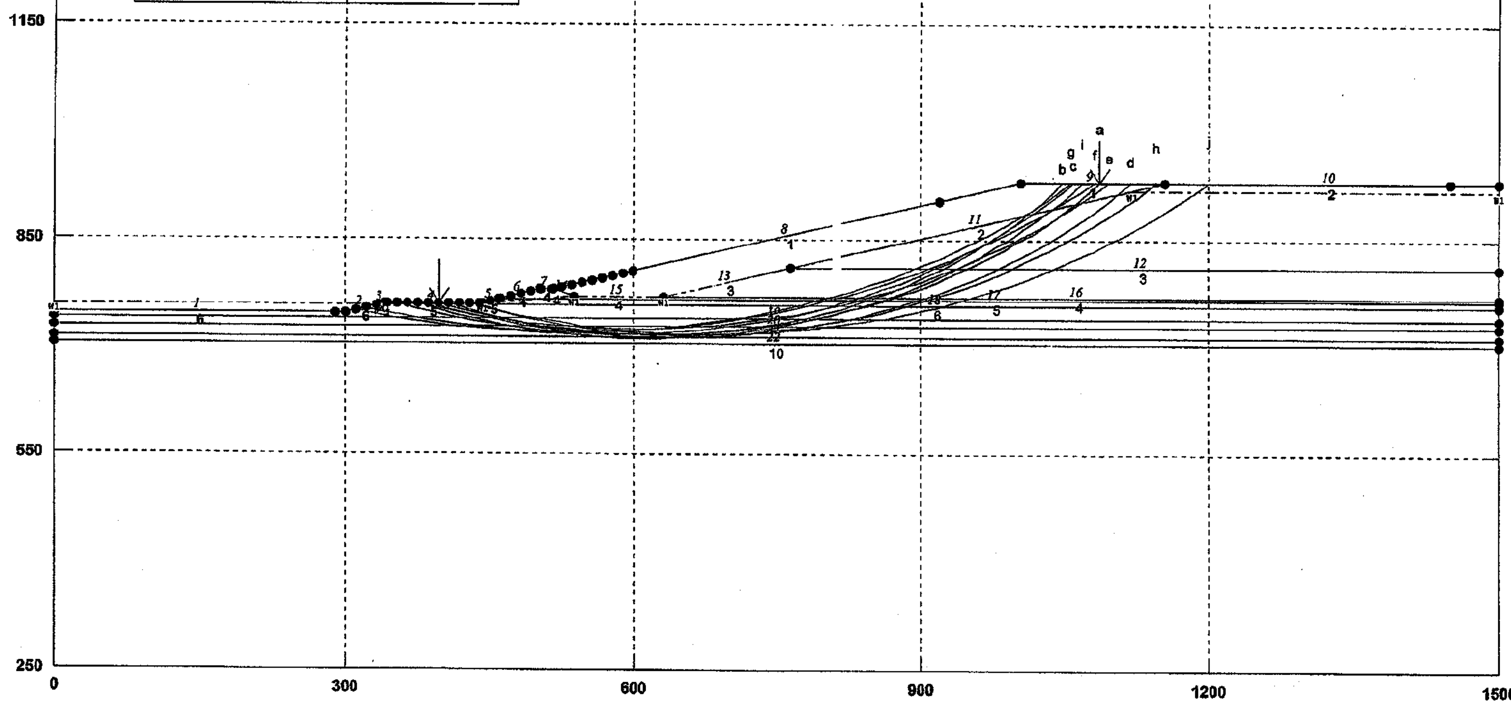
KIF All Wet Option T p at 930

C:\STEDWIN\KIFPWA.PL2 Run By: WA 5/14/2004 1:00PM

RUN NO. 7

#	FS
a	1.51
b	1.52
c	1.52
d	1.54
e	1.56
f	1.58
g	1.59
h	1.59
i	1.60
j	1.61

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
ba fa	1	120.4	120.4	100.0	38.0	W1
sed fa	2	108.0	108.0	200.0	28.0	W1
sda fa	3	108.4	108.4	500.0	28.0	W1
ba filtr	4	116.4	116.4	0.0	36.0	W1
comp fa	5	113.4	113.4	100.0	38.0	W1
ls fa ba	6	108.4	108.4	0.0	31.0	W1
ls fa	7	108.4	108.4	0.0	28.0	W1
nat cl	8	126.4	126.4	400.0	23.0	W1
sc sm	9	130.4	130.4	600.0	30.0	W1
rock	10	150.0	150.0	3000.0	42.0	W1



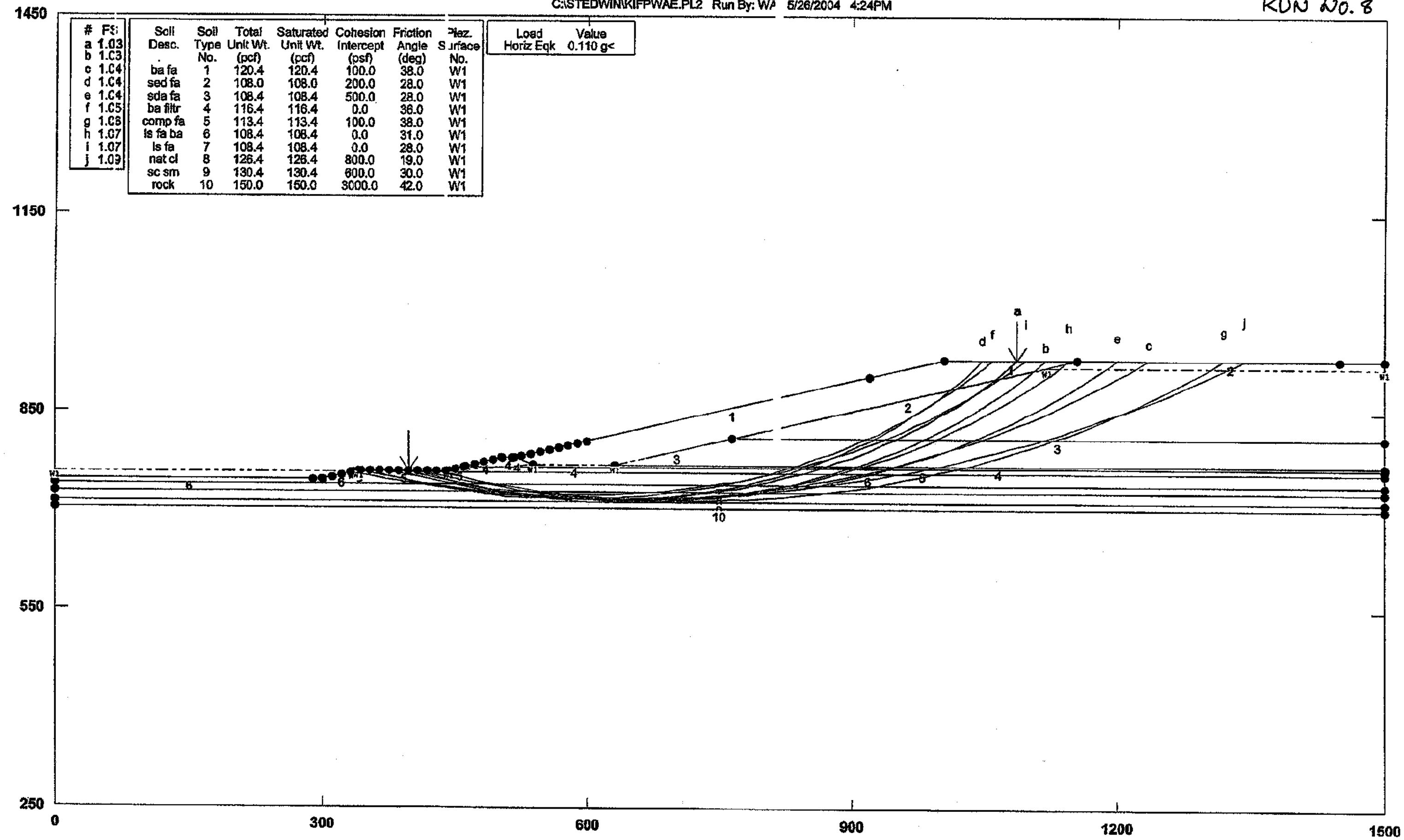
PCSTABL5M/si FSmin=1.51
Safety Factors Are Calculated By The Modified Bishop Method

STED

KIF All Wet Option T(p at 930

C:\STEDWIN\KIFPWAE.PL2 Run By: W/ 5/26/2004 4:24PM

RUN NO. 8



PCSTABL5M/si FSmin=1.03

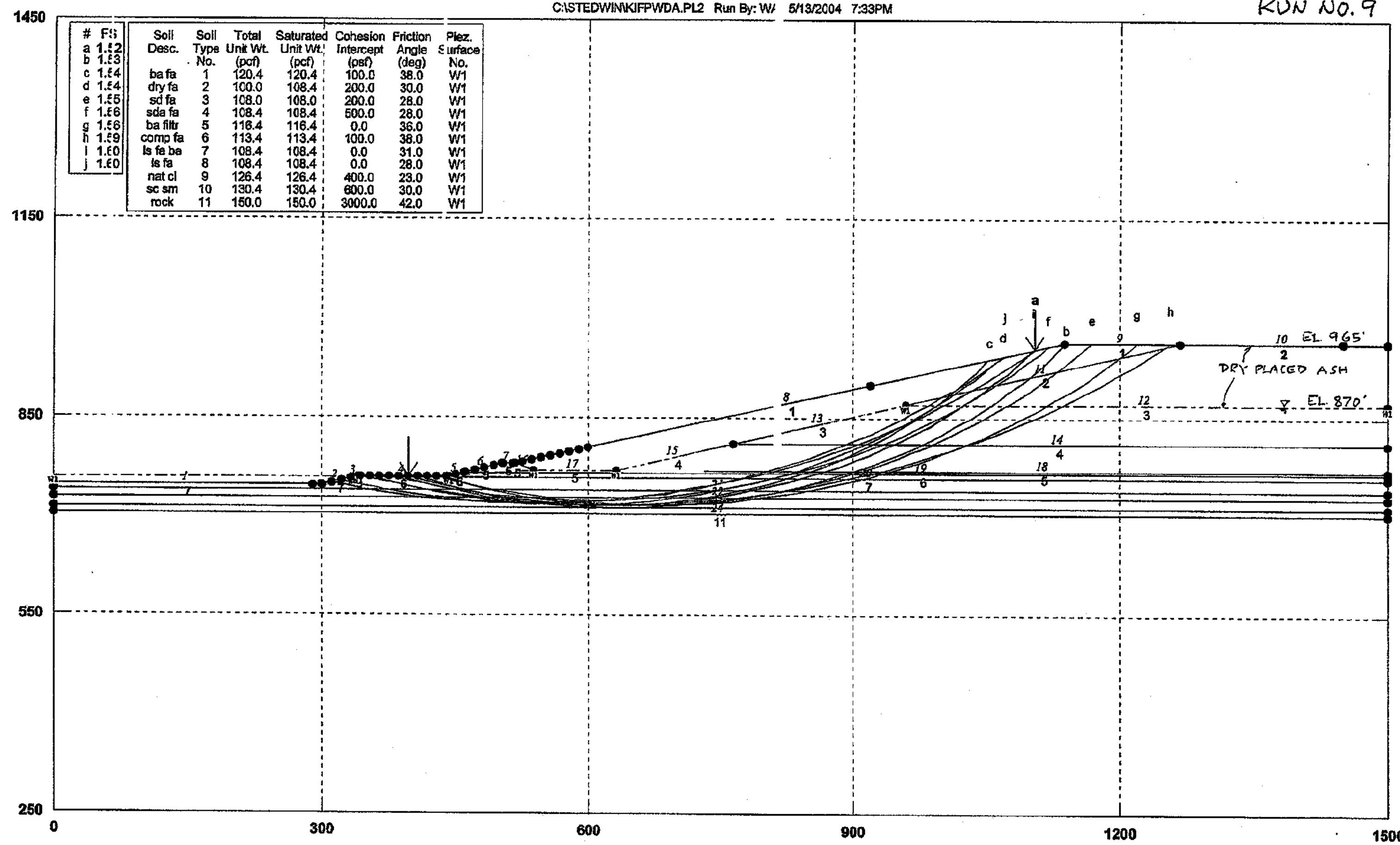
Safety Factors Are Calculated By The Modified Bishop Method

STED

KIF Wet + Dry O| tion

C:\STEDWIN\KIFPWDA.PL2 Run By: W/ 5/13/2004 7:33PM

RUN NO. 9



#	FS
a	1.52
b	1.53
c	1.54
d	1.54
e	1.55
f	1.56
g	1.56
h	1.59
i	1.60
j	1.60

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
ba fa	1	120.4	120.4	100.0	38.0	W1
dry fa	2	100.0	108.4	200.0	30.0	W1
sd fa	3	108.0	108.0	200.0	28.0	W1
sda fa	4	108.4	108.4	500.0	28.0	W1
ba filtr	5	116.4	116.4	0.0	36.0	W1
comp fa	6	113.4	113.4	100.0	38.0	W1
ls fa be	7	108.4	108.4	0.0	31.0	W1
ls fa	8	108.4	108.4	0.0	28.0	W1
nat cl	9	126.4	126.4	400.0	23.0	W1
sc sm	10	130.4	130.4	600.0	30.0	W1
rock	11	150.0	150.0	3000.0	42.0	W1

PCSTABL5M/si FSmin=1.52

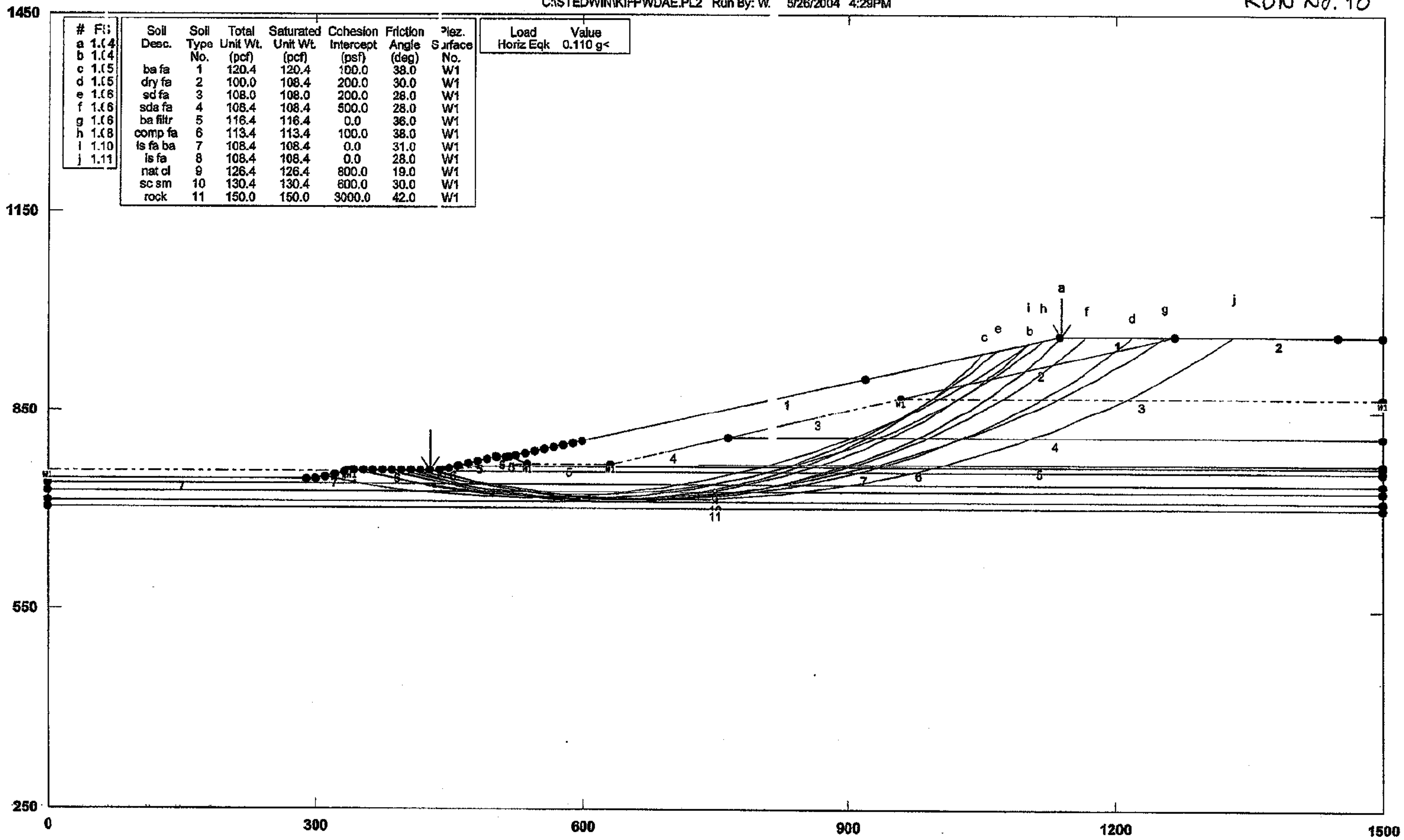
Safety Factors Are Calculated By The Modified Bishop Method

STED

KIF Wet + Dry Option

C:\STEDWIN\KIFPWDAE.PL2 Run By: W. 5/26/2004 4:29PM

RUN NO. 10



PCSTABL5M/si FSmin=1.04

Safety Factors Are Calculated By The Modified Bishop Method

STED

KIF Phase 3 (Section 3-3) Final Stack Gypsu[illegible] and Wet Ash Placement to 930

C:\STEDWIN\KIFPFC.PLT Run By: WA 5/26/2004 6:55PM

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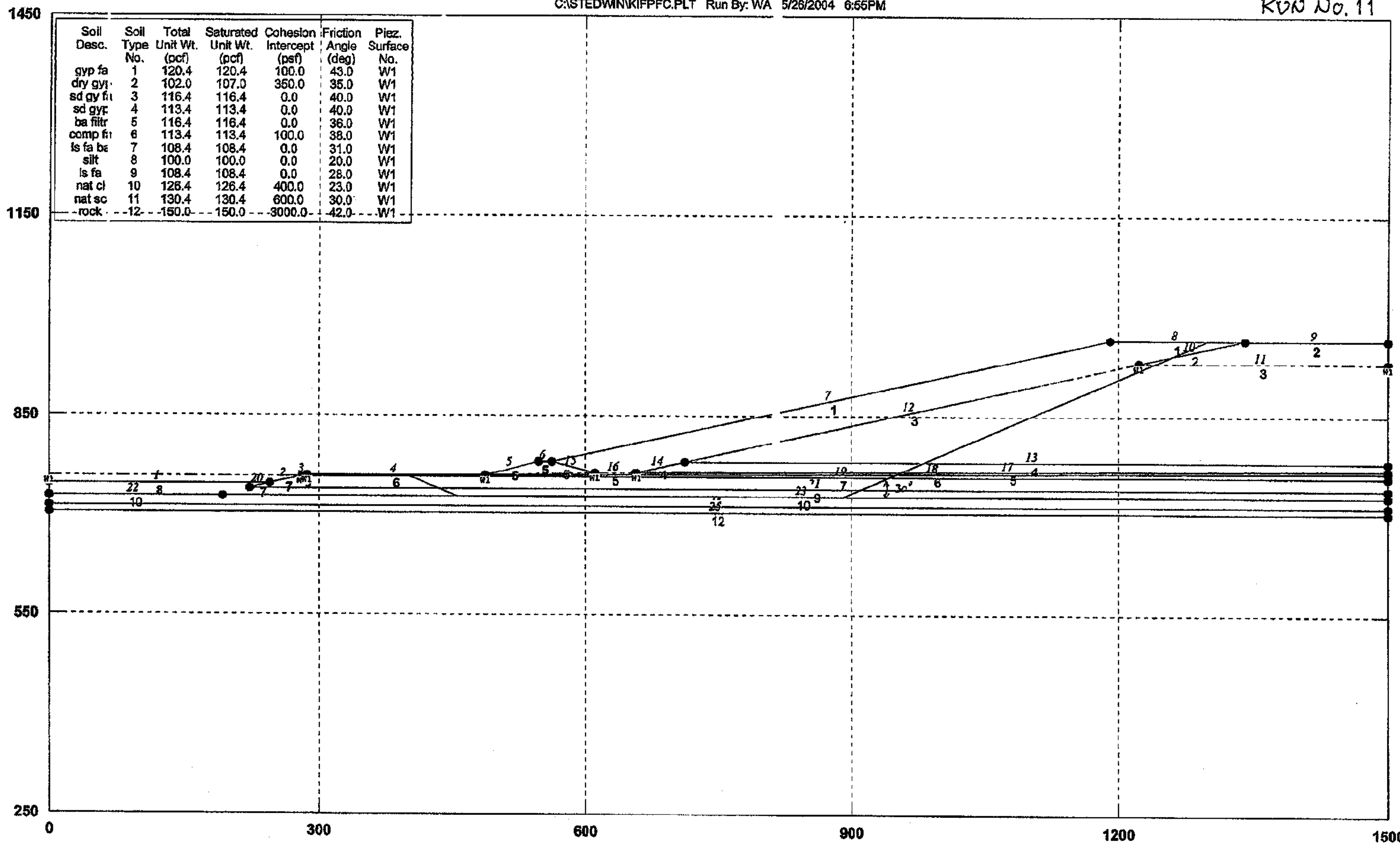


PCSTABL5M/si FSmin=1.77

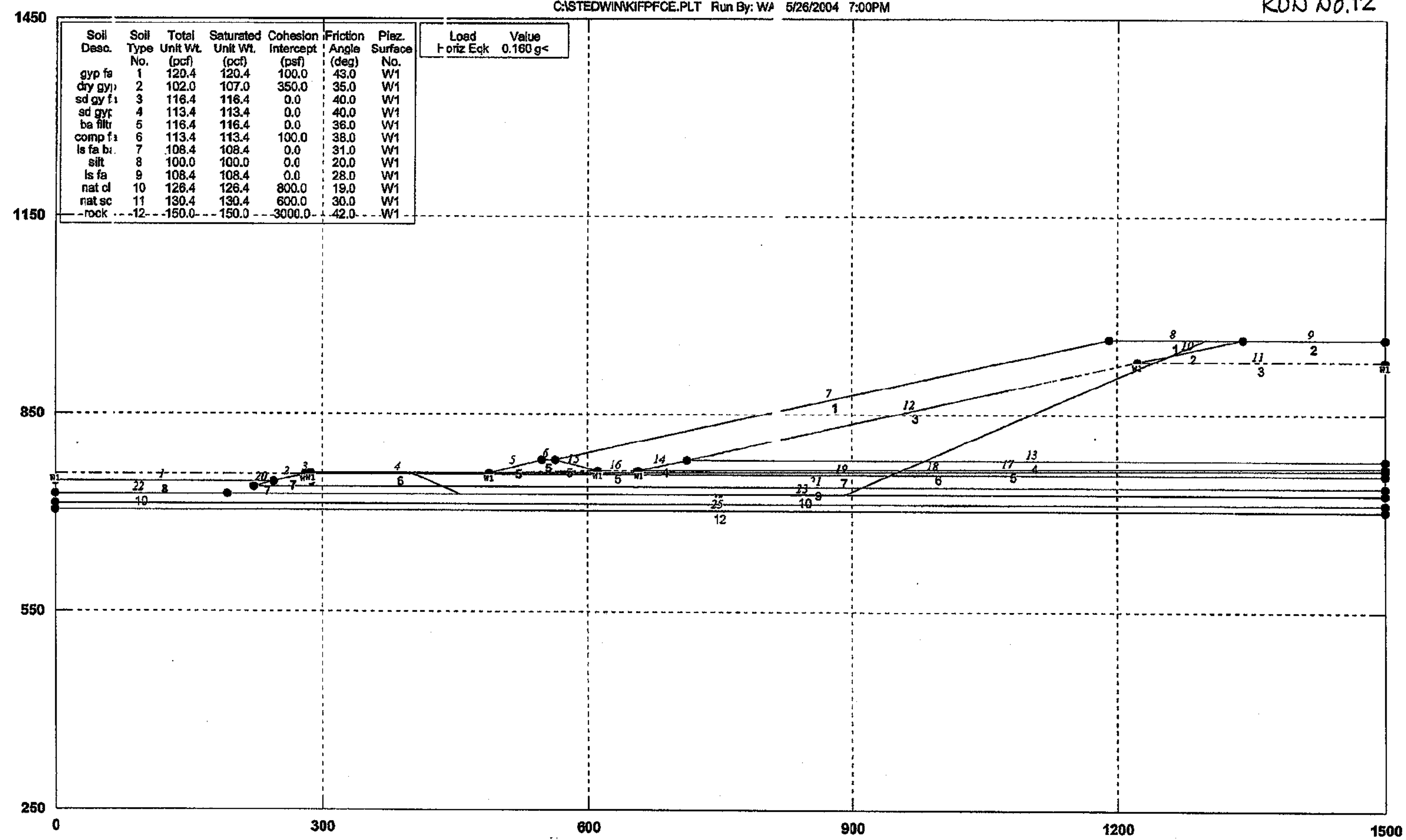
Factor Of Safety Is Calculated By The Modified Bishop Method

STED

KIF Phase 3 (Section 3-3) Final Stack Gypsum and Wet Ash Placement to 930

C:\STEDWIN\KIFPFCE.PLT Run By: WA 5/26/2004 7:00PM

RUN NO.12



Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
gyp fa	1	120.4	120.4	100.0	43.0	W1
dry gyp	2	102.0	107.0	350.0	35.0	W1
sd gy f	3	116.4	116.4	0.0	40.0	W1
sd gyp	4	113.4	113.4	0.0	40.0	W1
ba filtr	5	116.4	116.4	0.0	36.0	W1
comp f	6	113.4	113.4	100.0	38.0	W1
ls fa b	7	108.4	108.4	0.0	31.0	W1
silt	8	100.0	100.0	0.0	20.0	W1
ls fa	9	108.4	108.4	0.0	28.0	W1
nat cl	10	126.4	126.4	800.0	19.0	W1
nat sc	11	130.4	130.4	600.0	30.0	W1
rock	12	150.0	150.0	3000.0	42.0	W1

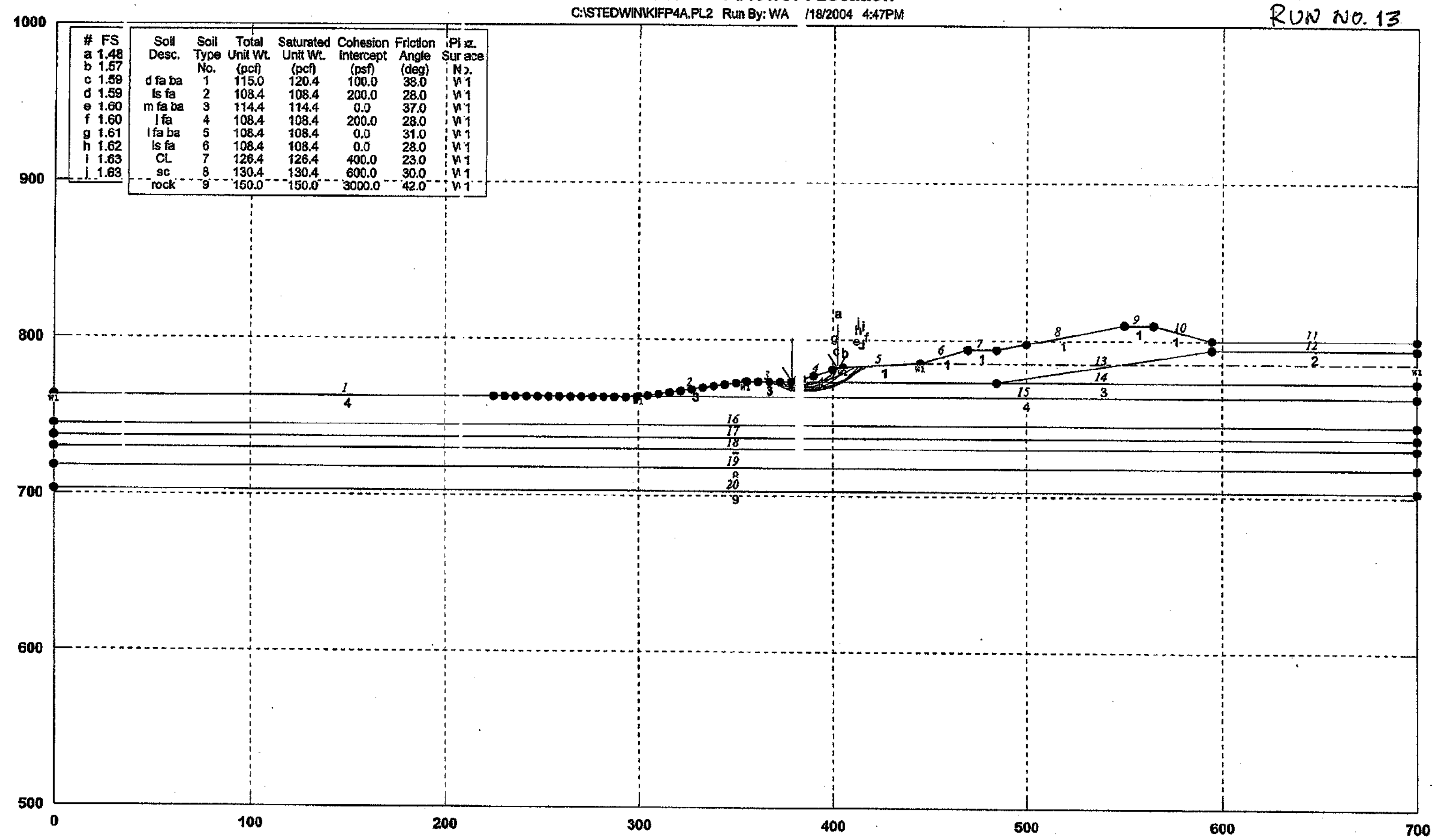
PCSTABL5M/si FSmin=1.02
Factor Of Safety Is Calculated By The Modified Bishop Method

STED

KIF Section 4 - 4 Blowo[illegible] Location

C:\STEDWIN\KIFP4A.PL2 Run By: WA /18/2004 4:47PM

RUN NO. 13



#	FS
a	1.48
b	1.57
c	1.59
d	1.59
e	1.60
f	1.60
g	1.61
h	1.62
i	1.63
j	1.63

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
d fa ba	1	115.0	120.4	100.0	38.0	W1
ls fa	2	108.4	108.4	200.0	28.0	W1
m fa ba	3	114.4	114.4	0.0	37.0	W1
l fa	4	108.4	108.4	200.0	28.0	W1
l fa ba	5	108.4	108.4	0.0	31.0	W1
ls fa	6	108.4	108.4	0.0	28.0	W1
CL	7	126.4	126.4	400.0	23.0	W1
sc	8	130.4	130.4	600.0	30.0	W1
rock	9	150.0	150.0	3000.0	42.0	W1

PCSTABL5M/si FSmin=1.48

Safety Factors Are Calculated By The Modified Bishop Method

STED

ATTACHMENT 2

VENEER STABILITY PRINTOUTS

(Six Pages)

Connection: Close

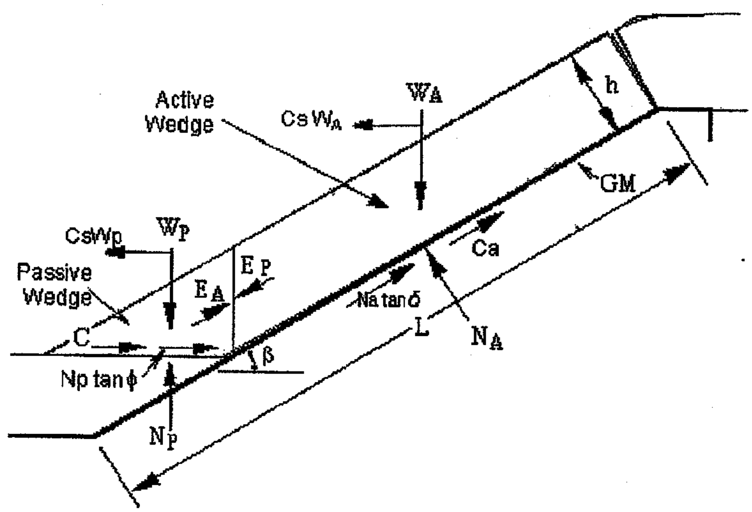
landfilldesign.com

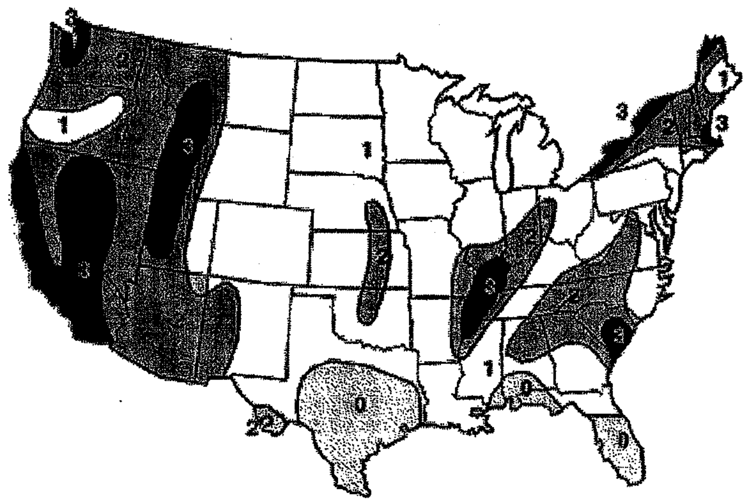
Slope Stability:Seismic Force - Design Calculator

Problem Statement

This slope stability calculator utilizes a pseudo-static analysis to determine the factor of safety (FS) of a geosynthetic lined slope. This calculator assumes that no seepage forces are present. The unit gradient calculator can be used to calculate the required transmissivity of the drainage geocomposite to assure adequate drainage.

Subtitle "D" of the U.S. EPA regulations requires a seismic analysis if the site has experienced a 0.1 g horizontal acceleration, or more, in the past 250 years. For the continental USA, this does not only include the western states, but major sections of the midwest and northeast as well. The map below shows the seismic coefficients for various zones in the USA.





Legend

Zone 0: No damage

Zone 1: Minor damage; corresponds to intensities V and VI on the modified Mercalli intensity scale

Zone 2: Moderate damage; corresponds to intensity VII on the modified Mercalli intensity scale

Zone 3: Major damage; corresponds to intensity VIII and higher on the modified Mercalli intensity scale

Seismic coefficients corresponding to each zone

Zone	Remark	Modified Mercalli Scale	Average Seismic Coefficient (Cs)
0	No damage	-	0
1	Minor damage	V and VI	0.03 to 0.07
2	Moderate damage	VII	0.13
3	Major damage	VIII and higher	0.27

Input Values

Design Inputs

Slope characteristics

Thickness of cover soil (h)	0.6	m
Slope angle (ß)	18.43	degrees
Lenght of slope measured along geomembrane (L)	27.43	m

Soil characteristics

Unit weight of the cover soil (g)	18.85	kN/m^3
Friction angle of the cover soil (F)	26	degrees
Cohesion of the cover soil (c)	12.0	kN/m^2
Interface friction(d)		degrees

	25	
Interface adhesion (Ca)	0	kN/m^2
Seismic characteristic		
Seismic coefficient (Cs)	0.11	g

Seismic Stability Calculation

Solution

Factor of Safety with seismic activity (FS) 1.283

Factor of Safety no seismic activity (FS) 1.761

Material Selection

Follow the GFR link to view our extensive database of geosynthetic materials reprinted with permission of IFAI

GFR

Additional Assistance

If you would like to have Advanced Geotech Systems provide material specifications that meet your performance criteria, please fill in the following fields and click the submit button. All information is kept strictly confidential.

Name *

Company

Email Address *

Phone

Project Reference

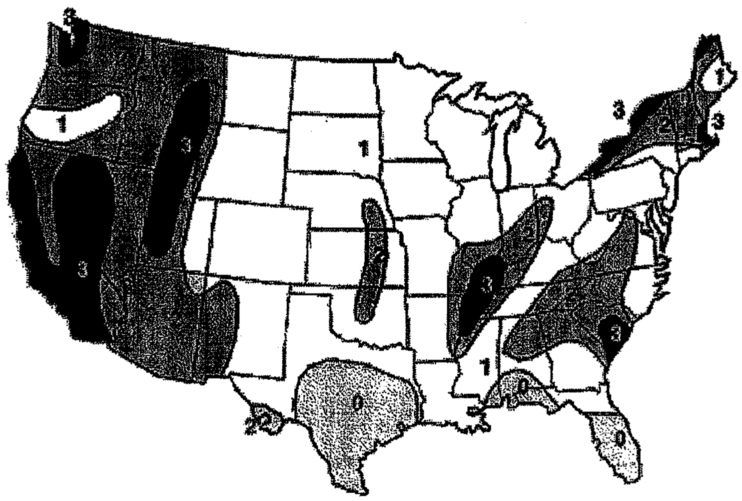
Comments

*required fields

Submit Design Results

Sponsored by

The following companies can service any of your geomembrane protection material selection needs.



Legend

Zone 0: No damage

Zone 1: Minor damage; corresponds to intensities V and VI on the modified Mercalli intensity scale

Zone 2: Moderate damage; corresponds to intensity VII on the modified Mercalli intensity scale

Zone 3: Major damage; corresponds to intensity VIII and higher on the modified Mercalli intensity scale

Seismic coefficients corresponding to each zone

Zone	Remark	Modified Mercalli Scale	Average Seismic Coefficient (Cs)
0	No damage	-	0
1	Minor damage	V and VI	0.03 to 0.07
2	Moderate damage	VII	0.13
3	Major damage	VIII and higher	0.27

Input Values

Design Inputs

Slope characteristics

Thickness of cover soil (h)	0.6	m
Slope angle (ß)	18.43	degrees
Lenght of slope measured along geomembrane (L)	27.43	m

Soil characteristics

Unit weight of the cover soil (g)	18.85	kN/m^3
Friction angle of the cover soil (F)	26	degrees
Cohesion of the cover soil (c)	4.8	kN/m^2
Interface friction(d)		degrees

TVA-00002196

	25	
Interface adhesion (Ca)	0	kN/m^2
Seismic characteristic		
Seismic coefficient (Cs)	0.11	g

Seismic Stability Calculation

Solution

Factor of Safety with seismic activity (FS) 1.154

Factor of Safety no seismic activity (FS) 1.588

Material Selection

Follow the GFR link to view our extensive database of geosynthetic materials reprinted with permission of IFAI

GFR

Additional Assistance

If you would like to have Advanced Geotech Systems provide material specifications that meet your performance criteria, please fill in the following fields and click the submit button. All information is kept strictly confidential.

Name *

Company

Email Address *

Phone

Project Reference

Comments

*required fields

Submit Design Results

Sponsored by

The following companies can service any of your geomembrane protection material selection needs.

TC Mirafi

References

R. M. Koerner, and T-Y. Soong, 1998. "Analysis and Design of Veneer Cover Soils". Proceedings of 6th International Conference on Geosynthetics, Vol. 1, pp. 1-23, Atlanta, Georgia, USA.

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APPENDIX H

Closure/Post Closure Plan

CLOSURE/POST CLOSURE PLAN
DREDGE CELL LATERAL EXPANSION
TENNESSEE VALLEY AUTHORITY
KINGSTON FOSSIL PLANT

Prepared By:
Tennessee Valley Authority
1101 Market Street
Chattanooga, TN 37401-2801

Revision 0
June 7, 2004

TABLE OF CONTENTS

1 INTRODUCTION

1.1 Site Location

The TVA KIF is located near the confluence of the Clinch and Emory Rivers (Watts Bar Lake) at Clinch River mile 2 (Emory River mile 2) in Roane Co. Tennessee, approximately 1 mi northwest of the City of Kingston. Access to the site is by state Highway 70 and Swan Pond Road. Refer to drawing 10W425-21, which depicts the plant layout and location of the existing dredge cells, and proposed dredge cell expansion.

1.2 Site Description

The site selected for the disposal facility is the existing fly ash pond, and is an expansion of the existing dredge cells, as shown on drawing 10W425-21. The ash pond is entirely within the KIF Reservation. Existing benchmarks are located as shown on the drawings.

The area surrounding the KIF is primarily agricultural, industrial, and rural in nature (refer to Drawing 10W425-21). The fossil plant powerhouse is just south of the proposed location for this disposal facility.

The methods of placement of gypsum and coal ash in this facility are discussed in the operations manual. Ash conveyance to the pond is by sluicing from the plant, and ash is dredged from the pond to the dredge cells. Dikes are progressively raised as cells are filled with waste material.

1.3 Expected Year of Closure

<u>1.3.1 Existing Ash Dredge Cells</u>

On a yearly basis, approximately 398,000 cubic yards of ash are produced at the KIF. Based upon the existing topographic contours, it is estimated that approximately 10 years of additional disposal capacity. When factored with the three-year expected capacity of the Phase 1 Lateral Expansion, the expected year of closure is 2017. The Operation Plan contains additional details.

<u>1.3.2 Lateral Expansion of Dredge Cells</u>

The Phase 1 expansion is expected to have a three-year life. However, closure of this portion of the facility will not occur until the remaining Phase 2 and Phase 3 portions of the facility reach the end of their useful life. The Operations Plan addresses the overall facility life.

1.4 Facility Contact

The name, address, and telephone number of the TVA personnel that may be contacted during the Closure/Post-Closure care period are listed as follows:

Owner: Tennessee Valley Authority (TVA)
Contact: Plant Manager

Tennessee Valley Authority
Kingston Fossil Plant
P.O. Box 2000
Kingston, Tennessee 37763
(865) 717-2501

As of the date of this revision, the plant manager is Mr. Earl Deskins.

2 FACILITY CLOSURE

2.1 Complete Closure Steps for Existing Dredge Cells

The TDEC/DSWM will be notified in writing of the intent to close this facility at least 60 days prior to the date final closure is expected to begin. Upon achieving the appropriate final grades for the ash fill (see drawing 10W425-76), the final cover, which includes compacted soil and vegetative layers, will be placed as shown on drawing 10W425-74. The final cover may also consist of the following components (see drawing 10W425-75) placed on top of the final ash grade: 1) a low density polyethylene liner, 40 mil thick; 2) a geocomposite drainage layer (consisting of an extruded polyethylene net heat bonded on both sides to a non-woven, needlepunched geotextile); 3) a one ft thick layer of soil placed above the geocomposite drainage layer; and 4) a one-half ft thick vegetative soil layer. The final cover may consist of a combination of these two methods, depending on material availability or other factors.

This will be accomplished in the shortest time practical, but not exceeding 90 days after completion of final grading of the ash fill. Closure activities (including grading, drainage, and establishment of vegetative cover) will be complete in the shortest time practical, but not exceeding 180 days after completion of final grading of the ash fill.

Closure will be in accordance with this plan and as shown on the permit drawings as approved by the TDEC/DSWM. Drainage structures such as run-on and runoff ditches, culverts, sediment basin, etc., will remain functional beyond final closure in order to minimize erosion and sediment migration into surface waters. After closure is complete, agreement will be obtained from the TDEC/DSWM for elimination of the sediment basin.

2.2 Complete Closure Steps for Dredge Cell Expansion

Complete closure Steps for the Dredge Cell Expansion will be as described for the existing dredge cells.

2.3 Partial Closure of Existing Dredge Cells

A basis premise for partial closure of the existing dredge cells is that this facility, if closed before the projected closure date (see Section 1.3), will result in final grades that are less the proposed final grades shown on the drawings submitted as part of this permit application. If such a partial closure is submitted, TVA will be required to submit revisions to the Closure/Post Closure Plan and closure drawings.

2.4 Partial Closure of Dredge Cell Expansion

TVA does not intend to undergo partial closure. However, in the event that partial closure may become likely, TVA will contact TDEC, DSWM in advance, and coordinate a timetable for partial closure acceptable to TDEC, DSWM.

2.5 Notice in Deed to Property

TVA is required to ensure that within 90 days of completion of final closure of the facility and prior to sale or lease of the property on which the facility is located, there is recorded, in accordance with state law, a notation on the deed to the property or some other instrument, which is normally examined during a title search that will in perpetuity notify any person conducting a title search that the land has been used as a disposal facility.

2.6 Closure Certification

Closure of this facility shall be in accordance with this Closure/Post Closure Plan. A closure certification report prepared by an independent registered professional engineer, licensed in the State of Tennessee, shall be submitted to the Division of Solid Waste Management for review and approval.

3 POST-CLOSURE CARE

The post-closure period will be 30 years. During the post-closure care period the owner must, at a minimum, perform the following activities on the closed portions of the facility:

A. Maintain the approved final contours and drainage systems of the site such that precipitation run-on is minimized, erosion of the cover/cap is minimized, precipitation on the fill is controlled and directed off the stack, and ponding is eliminated.

B. Ensure that a healthy vegetative cover is established and maintained on the site.

C. Maintain the drainage facilities, Stilling Pond, and other erosion/sediment controls (if present) in a functional state until the vegetative cover is established sufficiently to render such maintenance unnecessary. Removal or cessation of maintenance must be approved by the TDEC/DSWM.

D. Maintain and monitor the ground water monitoring system. The approved monitoring system and sampling and analysis program shall be continued during the post-closure period, unless the Closure/Post-Closure Plan is modified to establish a different system or program. Groundwater monitoring will be conducted in accordance with the requirements contained in the operations manual for this facility. Monitoring data must be reported in writing to the DSWM within 30 days after completion of analysis.

E. Post Closure verification. Post-closure of this facility shall be in accordance with this Closure/Post Closure Plan. A post-closure certification report prepared by an independent registered professional engineer, licensed in the State of Tennessee, shall be submitted to the Division of Solid Waste Management for review and approval. There are currently no plans for future use of this site.

4 COST ESTIMATE/FINANCIAL ASSURANCE

TVA is an agency and instrumentality of the United States created by the TVA Act of 1933, 16 U.S.C. 831-831dd (1988). TVA is not required to provide financial assurance in accordance with DSWM solid waste regulations rule 1200-1-7-.03 (1)(b)(3).

APPENDIX I

Quality Assurance/Quality Control (QA/QC) Plan

CONSTRUCTION QUALITY ASSURANCE/ QUALITY CONTROL PLAN DREDGE CELL LATERAL EXPANSION TENNESSEE VALLEY AUTHORITY KINGSTON FOSSIL PLANT

Prepared By:
Tennessee Valley Authority
1101 Market Street
Chattanooga, TN 37401-2801

Revision 0
June 7, 2004

CONSTRUCTION QUALITY ASSURANCE/ QUALITY CONTROL PLAN DREDGE CELL LATERAL EXPANSION TENNESSEE VALLEY AUTHORITY KINGSTON FOSSIL PLANT

Prepared By:
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1101 Market Street
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DRAFT
Revision 0
June 1, 2004

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ATTACHMENTS

1 INTRODUCTION

This Plan describes construction quality assurance/quality control (QA/QC) procedures for the successful construction and performance of the TVA Kingston Fossil Plant (KIF) Dredge Cell Expansion. This QA/QC Plan has been prepared in accordance with the criteria established by the State of Tennessee Department of Environment and Conservation (TDEC), Division of Solid Waste Management (DSWM) Regulations. The elements of construction the expansion and final cover requiring field monitoring and documentation under this plan include; continuation of existing dredge cell construction, Phase 1 Dredge Cell construction, starter dike for Phase 2 and 3 construction, construction of wet gypsum stack outer dikes, final cover, and vegetative layer. In addition, field monitoring and documentation and inspection of associated construction activities will also be required.

The purpose of this Plan is to outline procedures for verifying that proper materials, construction techniques, and installation procedures are used by the Constructor, and that the design intent is met. In addition, this Plan is intended to define problems that may occur during construction and to provide a mechanism to resolving these problems.

The program described by this Plan is independent of the quality control (QC) program conducted by the Constructor. This QA/QC Plan is intended to provide verification that the Constructor has met its obligation in the supply and installation of the specified materials. This Plan does not replace the contract documents (design drawings and documents) regarding the selection and installation of materials.

The construction and operation of this facility involves initial facility construction, as well as on-going operations. TVA conducts dike inspections at all fossil plants yearly, and this will continue for this facility. Because this facility will be raised during the operational phase, certification activities should be an on-going process during operation, but limited to those periods where dikes are being raised. This can be viewed as an extension of the yearly dike inspections. It is anticipated that during dike raising activities, surveillance by technicians to sample and test material and observe construction techniques would ensure that dikes are properly constructed. Less frequent site visits by the Certification Engineer would also provide assurance that construction activities are in conformance with the drawings. As stack construction proceeds, the Certification Engineer can adjust the frequency and type of testing and inspection/surveillance as needed.

2 DEFINITIONS

This section provides definitions for terms used in this QA/QC Plan.

Design Engineer — the individual(s) or firm(s) responsible for preparation of design documents and significant design changes during construction as determined by the Certification Engineer. The design engineer shall be a registered Professional Engineer in the state of Tennessee. TVA Fossil Engineering Services (FES) is the responsible engineering organization for design and certification of this facility.

Conformance Testing — refers to those activities that can take place prior to material installation.

Constructor — the individual or firm responsible for disposal facility-related construction and operational activities. This definition applied to any party performing work defined in the construction documents. TVA may use their own construction organization, Heavy Equipment Division (HED), for initial construction activities, and plant operations personnel (TVA Yard Operations) may perform dike raising activities described herein. TVA may also subcontract construction at it's discretion.

Construction Manager — the official representative of the Owner responsible for overseeing construction of the project. If TVA uses HED for initial construction, and TVA Yard Operations for operation, the Construction Manager and Constructor are one in the same.

Construction Testing — includes those activities than occur during and following material installation, including dike raising activities during facility operation.

Earthwork — an activity involving the use of soil or rock materials. It also includes activities involving the use of coal combustion byproducts in the construction of waste disposal facilities.

Certification Engineer — individual appointed by the Owner who is responsible for performing tasks outlined in this QA/QC Plan. The Certification Engineer will be selected by TVA FES and shall be a registered Professional Engineer in the state of Tennessee.

Project Design Drawings and Documents — all project-related drawings and documents, including design modifications and record drawings.

Project Documents — includes Constructor submittals, construction drawings, record drawings, specifications, shop drawings, field inspection reports, and project schedule. The drawings issued with the solid waste permit will principally used; however, TVA FES may develop additional drawings in more detail if needed to convey the original design intent. These drawings will not be submitted to TDEC DWSM. However, any changes that modify the facility operation or otherwise alter the permitted airspace will be discussed with TDEC in accordance with the Tennessee Solid Waste Regulations.

Quality Assurance/QA — provides verification that QC functions have been performed in substantial compliance with the project design drawings and documents.

CQA Consultant — individual appointed by the Constructor who is responsible for accomplishing work in accordance with the project design drawings and documents.

Quality Control/QC — functions done by the Constructor and material supplier to verify that work performed conforms to project design drawings and documents.

Record Drawings — drawings recording the locations, elevations, and details of the facility after construction in completed.

Surveyor — the individual responsible for preparation of as-constructed surveys of the completed subgrade, clay liner, starter dike, final surface of ash fill, final cover, and completed vegetation layer. The surveyor shall be a registered Surveyor in the state of Tennessee.

Testing Laboratory — a laboratory capable of conducting the tests required by this QA/QC Plan.

3 CERTIFICATION ENGINEER

The Certification Engineer (or personnel under his direct supervision) will closely monitor construction of the various soil components of the compacted base, the dike construction, and cap for the ash/gypsum fill. The Certification Engineer will be a Professional Engineer licensed to practice in the state of Tennessee, who is knowledgeable in the field of soil mechanics, and will have a good working knowledge of the equipment and procedures generally used in the construction of landfills.

The Certification Engineer has the following duties:

- provide written, certified documentation attesting to conformance to the design requirements and the QA/QC Plan with respect to conditions of subgrade, construction related to dike raising, construction of starter dikes, construction of outer wet cast gypsum dikes, final cover, and vegetative cover;
- be present at appropriate intervals during construction of the soil components, perform appropriate tests, and obtain samples for laboratory analyses;
- observe material delivery and unloading;
- use the results of tests and laboratory analyses to document conformance to performance requirements;
- furnish to the Owner and the Constructor the results of all observations and test as the work progresses. Coordinate with Constructor when modifications to the plans are necessary to ensure compliance with the design;
- educate other QA/QC personnel on the QA/QC requirements and procedures;
- schedule and coordinate the QA/QC inspection and testing activities;
- Reject defective work and verify that corrective measures have been implemented.

The Certification Engineer may utilize qualified field technicians to perform testing described and to provide additional observational oversight during construction.

4 MEETINGS

4.1 Design Review Meeting (Optional)

Following completion of the design and after review and approval by the TDEC-DSWM, a design review meeting will be held. The purpose of this meeting, which the Owner, Construction Manager, and the Certification Engineer shall attend, is to accomplish the following activities:

- identify key personnel;
- provide all parties with relevant documents;
- review the project design drawings and documents, and QA/QC Plan;
- confirm responsibilities of each party;
- review reporting and documenting procedures;
- define lines of communication;
- establish work area procedures;
- review sampling and testing procedures.

The meeting will be documented by the Certification Engineer or person designated by the Construction Manager. Copies of the minutes and relevant documents will be provided to all parties.

4.2 Preconstruction Meeting

A pre-construction meeting will be held at the site prior to the start of construction. The Owner, Construction Manager, Certification Engineer, Constructor, and others designated by the Owner will attend this meeting. In certain cases, many, if not most of these functions may be performed directly by TVA. The purpose of the meeting is to accomplish the following activities:

- review the construction drawings and documents, QA/QC Plan, work area procedures, construction procedures, and other related issues;
- define the responsibilities of each party;
- define lines of communication and authority;
- review the project schedule;
- review best management practices for erosion and sediment control during construction;
- review testing procedures and procedures for correcting and documenting; construction deficiencies, repairs, and retesting;
- review testing and record drawing documentation procedures;
- conduct a site inspection to discuss work areas, work plans, stockpiling, equipment and material laydown areas, access roads, and related items.

This meeting will be documented by the Construction Manager or authorized representative, and copies of the documentation will be distributed to all parties.

4.3 Progress Meetings

A progress meeting will be held daily just prior to commencement or just following the completion of work. This meeting will be attended by the Certification Engineer, and the Constructor's on-site superintendent and CQA Consultant. The following activities will be discussed during this meeting:

- review the previous days activities and accomplishments;
- review work locations and scheduled work;
- discuss problems;
- review test data.

This meeting will be documented by the Certification Engineer, and copies of the documentation will be distributed to the Owner, Construction Manager, and Constructor.

4.4 Deficiency Meetings

As required, meetings will be held to discuss problems or deficiencies. At a minimum, these meetings will be attended by the Certification Engineer and the Constructor's on-site superintendent and CQA Consultant. If the problem requires a design modification, the Design Engineer, Constructor, and Construction Manager should also be present. The meeting will be documented by the Certification Engineer on a daily meeting form.

5 INSTALLATION OF UNDERDRAINAGE SYSTEM FOR EXISTING DREDGE CELL SLOPES

The under drainage system for the existing dredge cells shall be installed as required in the Operations Plan. Materials specifications are shown on drawing 10W425-73, and other requirements are shown on the stage 3 drawings 10W425-42 through 45. A technician shall verify that materials meet the requirements and that installation is in accordance with the drawings. The Certification Engineer shall conduct site visits at least weekly during construction, and review daily reports.

6 CONSTRUCTION AND INSPECTION TESTING FOR FLY ASH & BOTTOM ASH DIKE RAISING

6.1 Materials Specification

Materials used to construct dikes for raising dredge cells for the existing Dredge Cell and Phase 1 expansion, and Phase 2/3 expansion (if Phases 2 and 3 are utilized for fly ash disposal instead of combined gypsum/fly ash disposal) shall be fly and bottom ash obtained from KIF. At TVA's option, bottom ash may be imported from Bull Run Fossil Plant if needed for construction. The Constructor shall make a reasonable effort to blend bottom ash to create as uniform a mixture as is practicable. Other materials used for dike raising are as specified on the drawings.

6.1 Pre-construction Testing

No testing is required prior to construction. Conduct testing as specified in the following section.

6.2 Placement

6.2.1 Subgrade Preparation and Dike Construction

Prior to dike raising, scarify all surfaces to prior to receiving fill. Ensure that grade stakes are set prior to proceeding with fill placement.

Place fill in alternating six-inch thick fly ash and bottom as layers. After placement of an initial one-foot layer, use a roto-tiller to blend the two layers together. The Certification Engineer shall inspect the initial placement of material to ensure that the tilling depth is correct. Exercise care with the tiller to ensure that woven geotextile is not damaged where placed. Mix an appropriate amount of water with ash during this process as directed by the Certification Engineer. Attachment 4 contains a suggested compaction procedure. In general, the material will likely have a narrow moisture window of compaction. This window range is also directly related to the compactive effort; i.e., the heavier the roller and the more compactive effort applied the narrower this window becomes. Compact the one-foot thick lift in place after tilling is completed using smooth drum rollers. If scrapers (pans) can yield the desired compaction, the smooth drum roller will not be required. Conduct proctor density testing to ascertain that compaction meets the compaction criteria discussed below. Continue this process as the dikes are raised. The Constructor shall utilize care in subsequent lift construction so that the roto-tiller depth is sufficient, yet not too deep so as to disturb previously placed material.

6.2.2 Testing

Testing should be performed more frequently at the beginning of dike construction, and can be decreased as directed by the Certification Engineer when consistent test results are obtained as the Constructor becomes accustomed to placing and mixing the fill material. Attachment 3 contains a suggested procedure for determining a compaction window for bottom and fly ash.

Initially, the Technician should be at the site continuously for at least the first week. Testing should include grain size analysis, standard proctor testing, and insitu density testing. Grain size testing should be initially be performed on the first lift every 800 feet. The Certification Engineer will review the data for uniformity. Compaction testing shall also be performed at the same frequency for the initial lift. Proctor tests shall be performed on at least two samples and these compared with previous testing results for uniformity by the Certification Engineer. If satisfactory tests are obtained with the first lift, subsequent lifts can be placed and testing can be decreased to four grain size tests and four standard proctor tests per lift. If satisfactory test results are obtained, grain size testing can be further reduced as directed by the Certification Engineer. Compaction shall be meet 95 percent standard proctor density of the material; however this requirement can be adjusted by the Certification Engineer depending on the results obtained.

7 COMPACTED FLY ASH BASE AND DRAINAGE/FILTER LAYER BENEATH PHASES 2 AND 3

7.1 Materials Specification for Fly Ash Base

Materials used to construct dikes for constructing the Phase 2 and 3 Dredge Cell Expansion shall be fly ash obtained from KIF. It is desired that the use of fly ash be maximized to construct this base to conserve available bottom ash for dike raising. However, bottom ash may be used as initial fill as needed to provide a suitable working surface for equipment. At TVA's option, bottom ash may be imported from Bull Run Fossil Plant if needed for construction. Other fill such as crushed stone may also be used if approved by the Certification Engineer. Tensar grid can also be used to stabilize the base if needed to allow equipment to initially place material, if approved by the Certification Engineer.

7.2 Material Specification for Construction of Hydraulic Isolation of Phase 2/3 from Phase 1/Existing Dredge Cells

For material specification for LLDPE geomembrane, see Section 10.4.

7.3 Pre-construction Testing

No testing is required prior to construction. Conduct testing as specified in the following section.

7.4 Placement of Compacted Fly Ash Base/Hydraulic Isolation

7.4.1 Preparation

Prior to construction, ensure that the fly ash base for Phase 2 is properly staked to locate the toe of the fill so that adequate distance is maintained from the outer ash area dikes, and the adjacent area constructed to allow continued dredging operations for wet ash stacking. New weir installation and abandonment of existing weirs should be accomplished prior to construction of the fly ash base in accordance with the drawings.

7.4.2 Hydraulic Isolation

Place the drainage layers beneath and above the LLDPE geomembrane. Place bottom ash as described in Section 7.6, except that the bottom ash drainage layer placed above the LLDPE geomembrane shall be placed in horizontal lifts to avoid placing stress on the geomembrane during installation. It is anticipated that the isolation between the existing dredge cell/Phase 1 and Phase 2/3 will be done in segments (between benches) rather than all at once. It is important that the geomembrane be properly secured to avoid damage by wind, and covered to avoid UV exposure. The geomembrane shall be tied into the fly ash base as shown on the drawings.

7.4.3 Fly Ash Base Construction

Prior to placing ash, temporary dikes can be constructed to isolate this area from the area. Standing water can be pumped out of the diked area continuously as fill is placed to provide a firm surface for equipment access. Construct access roads into the area area using fly ash and/or bottom ash and Tensar grid as necessary to provide a working surface for equipment. Material may be end-dumped from trucks and progressively pushed out into the areas using dozers. Initially, dozers can be D5 dozers with low ground pressure tracks to allow initial placement of material out into the area. Once a firm base is established, fill can be placed in six-inch thick lifts and compacted as described in Attachment 3. Continue to place fill until the grades are achieved.

7.5 Testing/Inspection

7.5.1 Fly Ash Base

After a firm based is established, insitu density testing should be performed for subsequent material placement to verify that the material is being compacted to at least 95 percent standard proctor density. This density requirement can be adjusted at the discrection of the Certification Engineer, depending upon results obtained with respect to the workability of the material as determined in the compaction window procedure in Attachment 4. Density testing should be performed random locations at an interval of five tests per acre (or approximately 50,000 ft^2 area). Attachment 3 contains a suggested procedure for determining a compaction window.

7.5.2 Hydraulic Isolation

Installation of bottom ash drainage layers above and below shall be in accordance with the same procedures outline in Section 7.7 below. Inspection and testing of the LLDPE geomembrane shall follow the requirements referenced in Section 10.4.