APPENDICIES

Detailed Site Investigation
Proposed Utility Waste Disposal Area
Ameren Labadie Power Plant

Prepared by: GREDELL Engineering Resources, Inc.
Appendix 1

Detailed Site Investigation
Work Plan and Approval
Mr. Paul Pike  
Ameren Services  
One Ameren Plaza  
1901 Chouteau Avenue  
St. Louis, MO 63166

Re: Detailed Site Investigation Workplan of the AmerenUE-Labadie Utility Waste Landfill,  
(Sections 17 and 20, Township 44 North, Range 2 East, Labadie 7.5 Minute Quadrangle,  
Franklin County)

Dear Mr. Pike:

The Geological Survey Program (GSP) has reviewed the document "AmerenUE Labadie Power Plant Utility Waste Landfill Detailed Site Investigation Work Plan" dated May 18, 2009. This plan details the workflow and requirements to characterize the 400 acre proposed utility waste landfill located adjacent to the existing AmerenUE Labadie power plant. It details how AmerenUE intends to characterize the alluvial aquifer in order to determine the hydrological conditions that exist below the site. This will include piezometer installation, geotechnical borings, cone penetrometer tests, in situ aquifer tests, bedrock characterization and piezometric surface characterization. The GSP concurs with investigation elements and methodology as proposed and hereby approves this work plan.

Questions regarding this review (Report ID F01709) may be directed to Blake Smotherman at 573-368-2132, P.O. Box 250, Rolla, MO 65402.

Sincerely,

DIVISION OF GEOLOGY AND LAND SURVEY

James W. Duley, RG  
Deputy Division Director  
Geological Survey Program

cc: Mikel Carlson, GREDELL Engineering Resources, Inc.  
Jeffrey L. Fouse, Reitz & Jens, Inc.
The Geological Survey Program (GSP) has reviewed the document "AmerenUE Labadie Power Plant Utility Waste Landfill Detailed Site Investigation Work Plan" dated May 18, 2009. This plan details the workflow and requirements to characterize the 400 acre proposed utility waste landfill located adjacent to the existing AmerenUE Labadie power plant. It details how AmerenUE intends to characterize the alluvial aquifer in order to determine the hydrological conditions that exist below the site. This will include piezometer installation, geotechnical borings, cone penetrometer tests, in situ aquifer tests, bedrock characterization and piezometric surface characterization. The GSP concurs with investigation elements and methodology as proposed and hereby approves this work plan.
May 14, 2009

Mr. Larry Pierce, R.G., Unit Chief
Division of Geology and Land Survey
Missouri Department of Natural Resources
P.O. Box 250
Rolla, MO 65402

Re: Detailed Site Investigation Work Plan, Proposed Ameren Labadie Power Plant Utility Waste Landfill, St. Charles County, MO

Dear Mr. Price:

Attached for your review and approval are two (2) copies of a detailed site investigation work plan for the Labadie Power Plant Utility Waste Landfill. This work plan has been prepared in general accordance with guidance criteria promulgated as Appendix 1 under 10 CSR 80-2.015, effective January 29, 2007. The guidance criteria were further clarified at our meeting with you and your staff on April 21, 2009.

If you have and questions or require clarification on any element of this work plan that might expedite the review process, please contact either myself (314-554-2388) or Mr. Mike Carlson of Gredell Engineering (573-659-9078). We look forward to working with you and your staff on this project.

Sincerely,

Paul R. Pike
Strategic Analyst

Attachment
AmerenUE Labadie Power Plant
Utility Waste Landfill
Detailed Site Investigation Work Plan

Prepared for:

Ameren

Ameren Services
New Generation & Environmental Projects
3700 South Lindbergh Blvd.
St. Louis, Missouri 63127

May 2009
AmerenUE Labadie Power Plant
Utility Waste Landfill
Detailed Site Investigation Work Plan

prepared for:
Ameren Services
New Generation & Environmental Projects
3700 South Lindbergh Road
St. Louis, Missouri 63127

May 2009

Prepared By:

Reitz & Jens, Inc.
St. Louis, Missouri

and

GREDELL Engineering Resources, Inc.
Jefferson City, Missouri
AmerenUE Labadie Power Plant
Utility Waste Landfill
Detailed Site Investigation Work Plan

May 2009

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1.0 INTRODUCTION

On behalf of AmerenUE, Labadie Power Plant, Reitz & Jens, Inc. of St. Louis, Missouri (R&J) and GREDELL Engineering Resources, Inc. of Jefferson City, Missouri (GER) herein presents for review and approval the following detailed site investigation (DSI) work plan for a proposed utility waste landfill (UWL) subject to the permitting requirements promulgated under 10 CSR 80-2 and 10 CSR 80-11. This work plan follows submittal of the preliminary site investigation (PSI) request dated December 3, 2008, which was subsequently approved by the Missouri Department of Natural Resources – Division of Geology and Land Survey (MDNR-DGLS) on February 2, 2009. Both documents are included for reference as Appendix 1.

Ameren Company is an electric power producer and distributor that operates a coal-fired power plant known as the Labadie Power Plant within the Missouri River alluvial plain in northeastern Franklin County, Missouri. Combustion wastes are currently placed in a wastewater treatment device (ash pond) located between the power plant and the proposed UWL site. Regulatory requirements to further reduce air pollutants from the power plant flue gases are anticipated to generate large quantities of flue gas scrubber by-products. A UWL will be required to manage these by-products and the proposed site location for the UWL is the focus of this DSI. Utility wastes generated at this site potentially include various combinations of fly ash, bottom ash, and other coal combustion by-products.

The proposed UWL site is located on properties owned and controlled by Ameren (Figure 1). The planned disposal area footprint measures approximately 400 acres in size, not including ancillary support features to be located outside the accepted limits of solid waste placement. The anticipated disposal area sub-base grade is on average ten (10) feet below existing ground surface based on assumed soil and groundwater conditions. Prospective borrow sources currently are assumed to lie within, or near, proposed investigative boundaries. Those sources will be better defined as both field investigations and the design process proceeds.

Coincident with this proposed DSI investigation will be a geotechnical evaluation of the site. Elements of the geotechnical investigation having application to this DSI work plan are described herein, with the expectation that the results of both field investigations will be utilized in the preparation of the final DSI report. A copy of the proposed geotechnical work plan is included as Appendix 2.
1.1 Purpose

The purpose of this work plan is to define the scope of work necessary to characterize the geologic and hydrologic conditions at the proposed UWL site. It has been prepared in general accordance with the criteria described in 10 CSR 80-2.015, Appendix 1, “Guidance for Conducting and Reporting Detailed Geologic and Hydrologic Investigations at a Proposed Solid-Waste Disposal Area”, dated January 29, 2007. A copy of the Guidance document is included for reference as Appendix 3. Included herein are descriptions of proposed methods for drilling and sampling, data collection, record-keeping procedures, laboratory testing, piezometer construction and development, and reporting. Applicable aspects of the concurrent geotechnical investigation are also described.
2.0 PHYSICAL AND GEOLOGIC SETTING

The proposed UWL site is located approximately two and one-half (2-½) miles northeast of the town of Labadie and immediately southeast of the Missouri River in northeast Franklin County (Figure 1). The National Geodetic Survey indicates the site lies within the northwestern part of Township 44 North, Range 2 East. Portions of the proposed investigation area are part of the "historic" Spanish Land Grant survey system identified as "SUR". The site is located within sections 17 and 20, SUR 384, and SUR 1735. Reference to cares.missouri.edu or similar mapping programs shows that the approximate midpoint of the proposed UWL is located at Latitude 38.5621 and Longitude –90.8168.

2.1 Physical Setting

The proposed UWL site is located within an extensive area of Holocene-age alluvial deposits largely derived from the Missouri River, which bounds the site to the north and forms the most conspicuous natural feature in the area (Figure 1). At its closest approach, the normal flow line of the river lies within approximately 4,000 feet (1,200 m) of the northwest corner of the site. Rock bluffs showing approximately 100 feet (33 m) of topographic relief relative to the alluvial plain bound the area to the south and extend to within approximately 800 feet (240 m) of the southern limits of the proposed UWL site.

Due to the alluvial setting, the most striking aspect of the site is the low topographic relief, which varies less than ten feet (3 m). Based on the topography shown on Figure 1, existing ground surface elevations range from 460 to 471 feet above mean sea level (msl). Drainage through this very flat terrain is facilitated by the creation of small waterways and diversions. Within the northern part of the proposed site, surface runoff generally flows northeastward toward the Missouri River. Within the southern part of the site, surface runoff flows to the southeast through the constructed waterways toward Becker Creek situated near the base of the rock bluffs. From there, the runoff flows northeast and appears to ultimately enter the Missouri River approximately one mile north of the hamlet of St. Albans.

The proposed UWL site and surrounding areas to the north, south, and east are currently used primarily for agricultural (row-crop) production. The AmerenUE Labadie Power Plant facility is located immediately to the west (Figure 1). An agricultural levee along the Missouri River marks the approximate northern boundary of the investigative area, and a Laclede Gas pipeline and another agricultural levee mark the approximate southern boundary of the investigative area. Labadie Bottom Road marks the approximate western boundary of the site and Davis Road marks the
eastern boundary of the site. The agricultural levees serve as partial flood protection for the proposed UWL site. Although the site currently lies within the 100-year floodplain of the Missouri River, proposed waste boundaries have been located so as to remain outside of applicable floodway boundaries. Further protection of the site from the 100-year flood event through construction of a perimeter berm around the UWL is currently anticipated.

2.2 Geologic and Hydrogeologic Setting

The proposed UWL site is situated within Holocene-age alluvium chiefly derived from the Missouri River. These alluvial deposits, which largely consist of sand and small gravel with lesser amounts of silt and clay, are approximately 100 feet thick, based on site-specific borings developed in March 2007. The results of that previous field effort were provided to MDNR-DGLS as part of the preliminary site investigation (PSI) request submitted in December 2008 and are included here for reference in Appendix 4. That site-specific information suggests that the upper 15 feet of alluvial material consists of varying proportions of interbedded clay, silt, and fine-grained sand. Relatively homogeneous, coarser grained sand predominates to a depth of approximately 50 feet. At deeper depths, gravelly sand becomes common, with reported clast size ranging upwards to cobbles and possible boulders. In Boring B-7, limestone bedrock was encountered at a depth of 104.5 feet. In Boring P-1, which was extended to a depth of 91.5 feet and therefore near to the bedrock contact, high-density gravel was encountered that precluded further advancement of the borehole.

Information concerning the local bedrock geology is derived from on-site inspection of the rock bluffs bordering the proposed site to the south, geologic well log data available from the MDNR-Water Resources Center website at www.dnr.mo.gov/env/wrc/logmain, as well as the Missouri Environmental Geology Atlas (MEGA) and other literature sources available from MDNR-DGLS (e.g. Brill, 1991; Thompson and Robertson, 1993; Thompson, 1995). Available well log data are provided in Appendix 5. They consist of three (3) records for wells drilled on the bluffs immediately south of the site (Figure 1). A bedrock map adapted from MEGA is included as Figure 2. A generalized stratigraphic column is also provided for reference as Figure 3.

These data show that the bluffs bordering the site immediately to the south consist of upper Ordovician (Mohawkian Series) St. Peter Sandstone, as well as the overlying Joachim Dolomite. Well log data suggest that the St. Peter Sandstone is between 55 and 110 feet thick, whereas the overlying Joachim Dolomite is less than 50 feet thick (Appendix 5). Below the St. Peter Sandstone are older geologic formations variously assigned to the middle Ordovician (Canadian Series) Powell, Cotter, and Jefferson City formations. These formations are lithologically very similar and for that
reason are not readily differentiated from one another in this part of the state. Regardless, they have an aggregate thickness in excess of 200 feet and likely constitute the uppermost bedrock surface encountered in site-specific borings developed within the proposed UWVL site.

Based on available well data, groundwater resources in the vicinity of the proposed site are primarily derived from the Holocene-age alluvium that overlies bedrock in this area. This alluvial aquifer is marked by a shallow water table surface (10 to 20 feet in depth) and is believed to retain predominantly unconfined hydraulic characteristics. Yields ranging from 1,000 to 2,000 gallons per minute are reported (Appendix 5).

Although the geologic and hydrologic settings are relatively straightforward, the proposed site and surrounding region are located along the northern margin of an area of potential seismic activity known as the New Madrid Seismic Zone. That seismic activity is embedded in deeply buried Paleozoic and Precambrian basement rocks beneath the Mississippi Embayment and reflects the vestiges of a failed rift system believed related to the early Pennsylvanian (Morrowan-Atokan Series) Oklahoma Aulocogen (Houseknecht and Kacena, 1983). Spasmodic earthquakes generally of low magnitude are a relatively common occurrence and it is for that reason that the region is considered part of a “seismic impact zone” as defined by regulation under 10 CSR 80-2.010(69) or as otherwise described under the criteria for Holocene age fault displacement found in 10 CSR 80-11.010(4)(B)3.
3.0 SITE INVESTIGATION

The site investigation for the proposed 400-acre UWL facility includes the development of unconsolidated materials borings, sample collection and description, field and laboratory testing, piezometer installation and development, surveying, aquifer testing, and water level monitoring. Bedrock investigation will be limited owing to the significant depth of burial of pre-Holocene geologic formations. Site activities will be performed by or under the direction of a qualified groundwater scientist as defined by 10 CSR 80-2.010(83). All subsequent geologic and hydrologic interpretation shall be under the direction of a geologist registered in the state of Missouri. All geotechnical interpretation shall be under the direction of an engineer registered in the state of Missouri. At a minimum, the qualified groundwater scientist or his firm shall also hold a restricted monitoring well installation contractor's permit. A company properly holding a non-restricted monitoring well installation contractor's permit will perform the drilling and piezometer installation. All drilling rigs shall have permit numbers prominently displayed.

Following completion of field activities, a detailed site investigation report shall be prepared and shall be submitted to the Division of Geology and Land Survey (DGLS) for review and approval as described under 10 CSR 80-2.015. Both field investigation techniques and reporting procedures as described below shall conform to the criteria described in 10 CSR 80-2.015, Appendix 1, "Guidance for Conducting and Reporting Detailed Geologic and Hydrologic Investigations at a Proposed Solid-Waste Disposal Area", dated 1/29/07 (subsequently referred to in this work plan as Guidance) (Appendix 3). The DSI report shall be signed and sealed by a geologist registered in the state of Missouri.

3.1 Field Investigation

The proposed field investigation will consist of the establishment of a minimum of 200 borings and cone penetrometer test (CPT) soundings, and the subsequent conversion of 100 of the borings into piezometers, in accordance with Guidance criteria specifying one boring per two acres and one piezometer per four acres for the proposed 400-acre disposal area. Proposed boring and CPT sounding locations are shown on Figure 4. They conform to a regular grid-like pattern. However, specific drilling locations may vary from a strictly linear pattern due to local field conditions or to better optimize site characterization. Certain piezometers will also be located outside proposed waste disposal boundaries to potentially serve as future groundwater detection monitoring wells for the UWL facility.
All piezometers and most temporary borings will be developed to an approximate depth of at least 35 feet. The proposed minimum depth is based on the Guidance requirement that all borings must extend at least 25 feet below the anticipated disposal area subgrade, which is approximately ten (10) feet. Some temporary borings and CPT soundings may be extended to deeper depths (e.g. 40 to 50 ft) to facilitate geotechnical considerations. All temporary borings not subsequently converted into piezometers shall be immediately backfilled following completion of borehole advancement using approved materials in accordance with 10 CSR 23-6 and the location marked for future survey reference. In addition, one (1) temporary boring is proposed to be extended to the top of bedrock (estimated depth 90 to 105 ft) to supplement the two existing site-specific "deep" boreholes (Borings B-7 and P-1, Appendix 4). The deeper depths of investigation will allow for the basic characterization of older unconsolidated layers, assist in the delineation of potentially confining layers to the overlying alluvial aquifer, and enable a more thorough assessment of geotechnical considerations to be made for future design purposes. Borings selected for deepening will be drilled to total depth in one continuous operation. The deeper borings shall also be promptly backfilled with approved materials in accordance with 10 CSR 23-6 and the location marked for future survey reference. Registration records shall be filed with the MDNR-Wellhead Protection Program within required timeframes.

Initially, borehole locations may be spotted in the field using global positioning system (GPS) techniques. Surveyed northings, eastings, and ground surface elevations performed by a registered land surveyor for the temporary borings will be obtained contemporaneously with piezometer survey data to the extent practicable using field marks as reference points. The previously acquired GPS data may also be necessary in the event field marks are obliterated between the time of backfilling of the temporary borings and mobilization of a survey crew, be it attributable to wind, water, or other climatic conditions, or through continued use of the site for agricultural production.

3.2 Drilling and Sampling

Proposed drilling methods include a combination of hollow-stem auger (HSA), solid-flight auger (SFA), and rotary-wash water techniques. Methods used will be predicated on borehole conditions and specific characterization objectives. Borings will be sampled using a combination of CME continuous samplers, split-spoons, or Shelby tubes at approximate 5-foot intervals (2.5-foot sample intervals will be implemented in the first 10 feet and if complex stratigraphic horizons are encountered). Shelby tubes will also be used if cohesive materials are encountered. Sampling equipment will be advanced within the open borehole, or within the interior of HSA’s as needed to
offset collapsible granular materials. The sampling in the temporary geotechnical borings deeper than 35 feet is described in Appendix 2. Sampling will be supplemented as required by means of “grab” samples taken at approximate 5-ft intervals or at obvious changes in lithology or texture. Borings planned for piezometer construction will have a nominal borehole diameter at least four inches larger than the outer diameter of the well screen and riser pipe used for construction.

Drilling and sampling activities will be logged and described by or under the direction of a qualified groundwater scientist, geologist or geotechnical engineer. Field notes will be recorded in clear, concise fashion in indelible ink for later reference and inclusion in the final report. Notes shall include drilling equipment, drilling personnel, date, start up and end times for drilling, weather conditions at the time of drilling, drill-rate data and observations, and sampling methods and data. Depth to groundwater shall be recorded. Field descriptions for the piezometer borings shall be entered using standard stratigraphic nomenclature and techniques. Note shall be made of color, texture, lithology, porosity, permeability, and significant characteristics of the geologic strata encountered. Sedimentary structures and possible subdivision of discrete lithologic units into depositional facies shall follow conventional terminology, such as described in Reineck and Singh (1980) or similar reference text. The field descriptions for the temporary geotechnical borings will follow ASTM standards.

All geologic and geohydrologic field work shall be completed under the direction of a geologist registered in the state of Missouri per RSMo 256.450 through 256.483 and the rules promulgated pursuant thereto. Applicable aspects of the concurrent geotechnical investigation will also be performed under the supervision of a professional engineer registered in the state of Missouri.

Analytical tests performed as a result of either the DSI or geotechnical investigation may include the following: Atterberg limits, hydraulic conductivity, unconfined compressive strength, unconsolidated-undrained strength (UU triaxial test), and consolidated-undrained strength (R triaxial test with pore pressure measurements). In addition, an estimated minimum of 50 granular samples collected during the investigation will be subjected to grain size analyses, including percent passing a #200 U.S. standard sieve (i.e. percent silt and clay). Selected representative samples will be cut from the continuous tube sample for index testing in the lab. Applicable standards to the geotechnical investigation and subsequent analytical procedures are described in Appendix 2.
3.3 Piezometer Installation

A total of 100 piezometers will be installed in an effort to characterize the behavior of the shallow alluvial aquifer beneath the proposed UWL facility. Proposed locations are presented on Figure 4. Piezometer depths will be dependent on the depth to the water table surface as determined while drilling. However, total depths should be sufficient to ensure submersion of the well screens to the extent practicable. Where the base of any boring extends deeper than what is required for piezometer construction, the lower part of the boring shall be backfilled using approved materials in accordance with 10 CSR 23-4.

Proposed construction methods will be in general accordance with 10 CSR 23-4.060. Proposed piezometer construction diagrams outlining the anticipated methods are presented as Figure 5 and Figure 6. Piezometers will be constructed using new, NSF-rated WC or PW, flush-threaded, nominal 2-inch and 4-inch Schedule 40 PVC well screen and riser pipe. The nominal 4-inch diameter piezometers may be required to facilitate aquifer testing. Proposed well-screen lengths will be ten feet. The well screens will be machine-slotted and will have a maximum aperture size of 0.010 inches (10-slot). Each well screen will be equipped with a sump of approximate four-inch length. The sump will also be flush-threaded. Well-screen depth data will take into account the sump at the base of the piezometers.

Once the well screen and riser pipe are installed, a primary filter pack will be tremied into the annular space, beginning at the base of the piezometer construction interval. The primary filter pack will consist of natural, rounded, well-sorted (poorly graded), quartz-rich sand. For 10-slot well screens, filter pack grain size will conform to the U.S. Standard sieve range of 20-40 (0.85 mm-0.425 mm). Filter pack volumes will be calculated beforehand. Once filter pack volumes are calculated, the sand will be emplaced in the annulus using approximate small diameter, clean PVC tremie pipe. Sand will be poured slowly to ensure that it drops through the water column to the base of the piezometer. Bridging will be corrected by “washing” the interior of the tremie pipe with potable water.

The volume of introduced water will be tracked for later development purposes. As filter pack sand is poured, the tremie will be lifted in stages to a height of between two and five feet above the calculated top of the well screen. Measurements will be determined using a weighted tape. The amount of primary filter pack sand used will be recorded and compared to calculated values. Once the sand is placed, the interior of the well casing may be agitated with a surge block to promote additional settlement. The resulting change in depth will be recorded and sand added, if necessary, to ensure the 2-foot minimum standard is met. Surging will take place before any additional well
construction materials are introduced into the annulus.

Because collapsible sands are anticipated and may be ubiquitous in this geologic setting, natural sand packs may be considered as an alternative to artificially emplaced filter packs, as allowed for under 10 CSR 23-4.060(8)(B). Piezometers where this alternative technique may be required will be discussed with DGLS prior to completion of the piezometer.

A secondary filter pack will be utilized in each piezometer where high-solids bentonite slurry is used as the bentonite seal. The secondary filter pack will also consist of natural, rounded, well-sorted (poorly graded), quartz-rich sand. Sand size will be selected based on the size of the underlying primary filter sand. If 20-40 primary sand is used, the secondary sand will correspond to the U.S. Standard sieve range of 60-120 (0.125-0.25 mm). Tremie methods identical to that described above will also be used to install the secondary filter pack. The volume of secondary sand required will be calculated beforehand. Secondary filter pack thickness will be between one and two feet. Measurements will be recorded using a weighted tape. The amount of secondary sand used will be recorded and compared to calculated volumes.

A bentonite seal will be installed immediately above the secondary filter pack, or the primary filter pack if a secondary filter pack is not used. The bentonite seal will consist either of bentonite chips or high-solids bentonite slurry grout. Bentonite chips may be used in those instances where the upper part of the borehole is unsaturated. The chips shall be placed in approximate one-foot layers and hydrated prior to installing the succeeding layers. The chips will be placed by gravity methods into the annulus to a thickness of between three and five feet. Measurements will be determined using a weighted tape. Volumes will be calculated beforehand and compared to quantities actually used. Prior to use, the chips will be screened to remove fine particulate matter so as to avoid flash swelling during emplacement. If bentonite slurry grout is used, the installation procedure is as follows.

The bentonite slurry grout will be between 20 and 30 percent by weight solids and will be tremied from bottom to top in one continual operation. Side-discharging tremie methods may be used to minimize damage to the underlying filter pack. The grout will be thoroughly mixed to a uniform consistency in an aboveground tank prior to use. Calculations will be made to ensure the 20-30 percent by weight solids standard is met. Annular volume calculations will also be made beforehand and compared to actual quantities of grout used. As grouting proceeds, the tremie pipe will be lifted in stages to ensure minimal displacement of the bentonite slurry once it is injected into the borehole.
An annular seal will be placed immediately above the bentonite seal. It will also consist of 20-30 percent by weight solids, bentonite slurry grout. In those piezometers where both the bentonite seal and annular seal consist of bentonite slurry grout, they will be installed in one continual operation. The methods used to install the annular seal will be as described above. Once the seal is installed, the grout will be allowed to cure and checked for settlement. It will be topped off as necessary with additional bentonite slurry grout to a minimum of within two feet of ground surface.

Each completed piezometer will be flush-cut approximately 2.5 feet above ground surface, equipped with a vented cap, labeled, and marked with a survey reference point by notching the top of the casing. Each piezometer will subsequently be surveyed by a registered land surveyor in the State of Missouri to establish x, y, and z coordinate information. Both ground surface and top-of-casing elevations accurate to within 0.01-ft will be obtained.

Aboveground completions shall be consistent with the requirements of 10 CSR 23-4.060(12). They will consist of properly sized steel protective casing set in concrete. The protective casing will extend at least two inches higher than the interior PVC casing. The base of the casing will extend down into the borehole a minimum of two feet. The borehole will be enlarged to ensure it is at least eight (8) inches in diameter larger than the size of the protective casing. At a minimum, concrete pads will be used for piezometers having utility as future groundwater monitoring wells. They will have a minimum dimension of 2 feet x 2 feet and will extend below finish grade to a depth sufficient to prevent frost heaving, which for eastern Missouri is between 15 and 20 inches. The top of the concrete pad will be shaped to facilitate drainage away from the protective casing. A small diameter (<1/8 inch) weep hole will be placed near the base of the protective casing. The interior of the casing will be partially filled with either coarse sand or gravel. The riser pipe will be equipped with a vented cap and the exterior protective casing provided with a locking cap and lock. For those piezometers located along high-traffic areas, three large-diameter (>4 inch), steel bollards may be installed around the concrete pad. The bollards will be set in concrete at least two feet below finish grade and will extend at least three feet above finish grade. The bollards will be painted to promote easy visibility.

Following completion of the piezometers, including survey data, well certification records will be prepared and submitted to the DGLS-Wellhead Protection Program within required timeframes.
3.4 Piezometer Development

Each piezometer shall be developed in accordance with 10 CSR 23-4.070. Proposed methods include the use of either disposal bailers or a non-dedicated, submersible pump. In no event will the method used introduce any contaminants into the piezometers. A minimum of three well volumes of water will be removed or until the piezometer is effectively "dry". A "well volume" includes both the filter pack and casing, as measured from the base of the well to the initial static water level. In addition, the volume of potable water introduced into the well bore while drilling and/or constructing the piezometer, if any, will be removed.

Field measurements of groundwater temperature, pH, and specific conductivity may be recorded in some piezometers during the development process, particularly those having potential utility for long-term groundwater detection monitoring. If performed, and provided sufficient recharge is realized, field measurements will continue until both temperature and specific conductivity have stabilized to within ten percent between three successive readings. Similarly, pH readings should stabilize within 0.2 pH units.

In addition to the above, development records will include documentation of both pre- and post-development water levels. Final clarity of the water will also be noted.

3.5 Aquifer Testing

Aquifer testing will be performed in one out of every four of the piezometer borings (25 percent of the piezometer borings drilled on-site) as described by Guidance. Consequently, if a total of 100 piezometers are constructed as proposed elsewhere in this work plan, a minimum of 25 percent or 25 piezometers will be selected for aquifer testing. Piezometers selected for analysis will be based on a combination of location and screen interval lithology in order to provide a representative depiction of hydraulic behavior throughout the limits of investigation. Piezometer locations currently under consideration for testing are noted on Figure 4.

Aquifer testing will be conducted after piezometer development is complete. The appropriate test for each piezometer will be dependent on site-specific conditions, including screen interval lithology, water column height, and apparent recharge as determined during development. However, based on prior experience in alluvial settings, piezometers screened across medium- to coarse-grained (or larger) sand will recharge very quickly and for that reason rising head (recovery) test methods are anticipated using 4-inch diameter piezometers. This larger diameter will permit the use of 3-inch or 4-inch submersible pumps (e.g. Grundfos Redi-Flo3 or Redi-Flo4) capable of sustaining 20-30 gpm
pump rates believed necessary to achieve adequate suppression of the water table surface. Continuous recording of water level will be maintained until 90 percent of the drawdown is recovered, relative to initial water level measurements.

Falling head (slug) test methods may be considered if fine-grained (i.e. silt and very fine sand) materials predominate at the level of the screen interval. This method can likely be successfully accomplished in 2-inch diameter piezometers due to the relatively low recharge rate of the formation materials. If used, falling head tests will be continued until the water level has stabilized or until 70 percent of the excess head has dissipated.

Laboratory testing of hydraulic conductivity for aquifer characterization purposes is not proposed.

3.6 Monthly Water Level Monitoring

Monthly water level monitoring will be performed for a period of 12 consecutive months following piezometer installation as required by Guidance in an effort to characterize flow direction and seasonal variation in water table elevation. Measurements will be made to an accuracy of 0.01-ft using the surveyed "top-of-casing" as a reference point. An effort will be made to ensure that all measurements are made within an approximate 48-hr time period. Accumulated data will be evaluated to determine the effects of precipitation-derived recharge on the alluvial aquifer. Possible influences from the Missouri River will also be examined. River flow data will be obtained from the nearest available gauging stations. The U.S. Army Corps of Engineers (USACE) maintains a gauging station upstream at Washington, MO (MOWA, found on the USACE website at http://mvs-wc.mvs.usace.army.mil/trans/gages.html) The United States Geological Survey (USGS) maintains a gauging station upstream at Washington, MO (USGS #06935450, found on the USGS website at http://waterdata.usgs.gov/nwis/uv?06935450).

Precipitation data will be gathered throughout the twelve-month monitoring period. These data will be gathered either from a continuously recording device located at or near the investigative site or from the nearest local weather center in the event of equipment malfunction.
4.0 EVALUATION AND REPORT

Following completion of field activities, a detailed site investigation report will be prepared for review and approval by DGLS. The DSI report will detail the results of the field investigation and interpret the geologic and hydrogeologic conditions of the site based on data collected in accordance with this work plan. The content and format of the report shall be in general conformance with Guidance criteria. All DSI report work shall be prepared under the direction of a qualified groundwater scientist who is a geologist registered in the state of Missouri per RSMo 256.450 through 256.483 and the rules promulgated pursuant thereto. This person shall also sign and seal the report. The concurrent geotechnical investigation results will be described in a separate document appended to the DSI report and signed and sealed by a professional engineer registered in the state of Missouri.

Major narrative elements of the report will include an introductory section, a description of field data collection methodology, a description of field data collection results, and an interpretative summary describing conclusions reached about the hydrogeologic character of the site. The introductory section will provide general information about the proposed UWL site, including the regional geographic and geologic setting and historical land use(s). Siting restrictions for utility waste landfills as described under 10 CSR 80-11.010(4)(B) may also be discussed, although not specifically required by Guidance criteria. Field methods will include discussion of drilling, sampling, and logging procedures, as well as an accurate description of analytical testing. Standard procedures will be appropriately referenced. Field results will be presented in a clear and concise fashion and will include discussion of any anomalous data. The conclusion section of the report will describe how the site-specific hydrogeology may affect the design of the UWL. A conceptual groundwater detection monitoring system will also be described.

Data collected during the field investigation, and as described in the detailed site investigation report, shall be supported by figures, tables, maps, and cross-sections. Drilling logs, piezometer construction diagrams, and development records will be provided as appendices to the report. Appendix information will also include well certification and registration records, raw analytical data, and copies of field notes, including photographic documentation as applicable. An aerial photograph of the proposed site and surrounding area will also be provided. Guidance criteria require the aerial photograph to be taken between November 1st and March 30th within two years of report submittal. However, because the proposed UWL facility is located in a region of minimal tree cover or other limiting obstructions, it is herein proposed that the monthly timeframes be waived in this instance.
Tabular data presented in the report will consist of borehole and piezometer construction summaries, including identification, location, total depth, surface elevation, well-screen interval, and hydrogeologic information. Monthly groundwater elevation summaries will also be provided. Laboratory analytical data shall be summarized in similar fashion. Other tabular data will include a summation of any hydrologic testing, as well as precipitation data recorded during the investigative and monitoring period.

All maps displaying site-specific data acquired during the investigation will be at a scale of one-inch equals 400 feet or less. Initial site topography will be shown using a maximum contour interval of five feet. Each map will contain a scale, north arrow, and concise legend describing all of the symbols used on the map. Proposed facility boundaries will be shown. Maps shall provide information as required by Guidance, or as otherwise applicable.

Geologic cross-sections will be provided both parallel and perpendicular to the facility baseline utilizing borehole information to illustrate the geologic and hydrologic character of the site. At least one cross-section will be constructed parallel to groundwater flow. Cross-sections will clearly depict relevant lithostratigraphic boundaries with appropriate designations defined in the accompanying legend. Applicable boring log and piezometer construction data will be portrayed. Both the anticipated sub-base and final grades for the proposed UWL disposal area will be shown.
5.0 REFERENCES

Brill, K.G., Jr., 1991, Geologic Map St. Louis City and County, Missouri, Missouri Department of Natural Resources, Division of Geology and Land Survey Open File Map OFM-91-259-GI.


Piezometer Construction Quantities:

1. Primary Filter Sand – _________ lb.
2. Bentonite Seal – _________ lb.

Remarks:

PROPOSED 2" PIEZOMETER CONSTRUCTION DIAGRAM

AmerenUE LABADIE POWER PLANT

DETAILED SITE INVESTIGATION WORK PLAN

GREDELL Engineering Resources, Inc.
ENVIROMENTAL ENGINEERING

LAND
AIR
WATER

1505 East High Street
Jefferson City, Missouri 65101
Telephone: (573) 659-8078
Facsimile: (573) 659-8079

DATE: 3/2009
SCALE: N.T.S.

FIGURE 5

FIGURE 5

DRAWN BY: W.J.A. APPROVED BY: M.C.C. PROJECT NO.
Piezometer Construction Quantities:

1. Primary Filter Sand - ____________ lb.
2. Bentonite Seal - ____________ lb.

Remarks:

4" Dia. SCH-40 PVC Screen
No. 6 - No. 10 Machined Slotted

1½" Secondary Filter Pack
(Where Needed)

3" - 5" Bentonite Seal

Minimum of 8 3/8" Borehole

4" Dia. SCH-40 PVC Riser With Flush Threaded Connections

Washed Pen Gravel/Crushed Sand

Well Identification

Vented Cap

Protective Casing (6x6 inch Square)

1½" Whip Hole Above Ground Level

Protective Casing 2' - 3' Below Grade (As Required)

High Solids (20%-30% By Weight)
Bentonite Slurry (Length Varies)

Primary Filter Pack
(2' - 5' Above Well Screen)

Sump

Top Of Riser

+/- 2.5' Above Grade

12 in. Diameter Concrete Pad Minimum

PROPOSED 4" PIEZOMETER CONSTRUCTION DIAGRAM

GREDELL Engineering Resources, Inc.
ENVIROMENTAL ENGINEERING

LAND

WATER

1505 East High Street
Jefferson City, Missouri 65101

Telephone: (573) 659-9078
Facsimile: (573) 659-9079

DETAILED SITE INVESTIGATION WORK PLAN

DATE 3/2009  SCALE N.T.S.  FIGURE  REV

FIGURE 6

DRAWN BY: W.J.A. APPROVED BY: M.C.C. PROJECT NO.
APPENDICES
APPENDIX 1

Preliminary Site Investigation (PSI)

Correspondence
February 2, 2009

CERTIFIED MAIL 7005 3110 0004 3988 9017
RETURN RECEIPT REQUESTED

Mr. Paul Pike
Ameren
One Ameren Plaza
1901 Chouteau Avenue
St. Louis, MO 63166

Re: Preliminary investigation of the proposed expansion of the AmerenUE-Labadie Utility Waste Landfill, (Section 17 and 20, Township 44 North, Range 2 East, Labadie 7.5 Minute Quadrangle, Franklin County)

Dear Mr. Pike:

The Geological Survey Program (GSP) has completed the Preliminary Site Investigation (PSI) for the proposed expansion to the AmerenUE-Labadie Utility Waste Landfill. The proposed landfill is approximately 1042 acres.

The site is approved to proceed to the next phase of the permitting process. Please find enclosed the PSI report (ID# F00409) that summarizes the geologic and hydrologic evaluation of the proposed expansion area.

Also enclosed is a copy of Appendix 1, Guidelines for Planning, Conducting, and Reporting Detailed Geologic and Hydrologic Investigations at a Proposed Solid Waste Disposal Area. This document summarizes the elements and format that should be used to develop a detailed site investigation workplan. We encourage you and your consultant to meet with the GSP staff prior to finalizing a workplan for the detailed site investigation to discuss the elements to be included within the report. Please contact Mr. Larry Pierce, telephone 573-368-2191, or email larry.pierce@dnr.mo.gov, to schedule this workplan meeting.

Current procedures call for an applicant receiving approval at the preliminary site investigation stage to participate in public involvement activities as part of the solid
waste disposal area permit application process. Within 30 days of the approval, the applicant must notify both the governing body of the county or city, and the solid waste management district in which the proposed disposal area is to be located. This notification is to be by certified mail.

Within 90 days of the Preliminary Site Investigation approval, the department will conduct a public awareness session in the county in which the proposed disposal area is to be located. For further information concerning these public involvement requirements, please contact the Solid Waste Management Program at (573) 751-5401.

If you have any questions, please feel free to contact Larry "Boot" Pierce at P.O. Box 250, Rolla, Missouri 65402, telephone (573) 368-2191, or email at larry.pierce@dnr.mo.gov. Thank you for your interest.

Sincerely,

DIVISION OF GEOLOGY AND LAND SURVEY

James W. Duley, RG
Deputy Division Director

cc: Charlene Fitch, Waste Management Program, w/enclosure
Paul Reitz, P.E., Reitz & Jens, Inc., w/enclosure
Mike Carlson, R.G., Gredell Engineering Resources, Inc., w/enclosure
Region 1 – East Central SWMD
Missouri Department Of Natural Resources
Division of Geology and Land Survey
P.O. Box 250
Rolla, Missouri 65402-0250
Phone - 573.368.2161 Fax - 573.368.2111
E-mail - gpgeol@dnr.mo.gov

Solid Waste Disposal Site - Geologic Evaluation

Project Name  AmerenUE Labadie, Utility Waste Landfill  Quadrangle  LABADIE
Location  Section 17  Township 44N  Range 2E
Additional Location Information  Also Section 20, includes SUR 354 and 735
Latitude  38 Deg 34 Min 0 Sec North  Longitude  90 Deg 49 Min 31 Sec West

Owner  Ameren
One Ameren Plaza, 1901 Chouteau Ave., St. Louis, MO 63166-1419  (314) 342-1000

Requestor  Ameren Services
Paul Pike, One Ameren Plaza, 1901 Chouteau Ave., St. Louis, MO 63166-1419  (314) 554-2388

Date of Field Visit

<table>
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<td>Perched</td>
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<tr>
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<td>Ridgetop</td>
<td>Local</td>
<td>Bedrock</td>
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<tr>
<td>8% to 15%</td>
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</tr>
<tr>
<td>&gt;15</td>
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Bedrock

The uppermost bedrock is the Ordovician-age Everton Formation or the Jefferson City-Cotter Dolomite, which both exhibit low permeability in this area. These formations are typically composed of an undulating dolomite overlying a thicker, massive sandstone, and a light-gray to light brown, medium- to finely crystalline, cherty dolomite, respectively.

Overburden

The surficial materials are best described as ranging from brown to tan, silty-sand alluvium with some clay (SM/SC) to brown to gray, highly plastic, inorganic clay (CH) in the upper 20–30 feet. Two soil borings (P-1 and B-7) indicate cobble and boulder-size limestone and dolomite debris below 50 feet. Bedrock is contacted at approximately 100 feet. These materials typically exhibit moderate to high permeability in this environment.

Site Hydrology

The Missouri River alluvial aquifer is the uppermost continuous water-bearing unit. Groundwater in the alluvial aquifer largely flows in the same direction but is variable depending on the river stage. Surface water and groundwater also flow due north from the Franklin Low Hills of the Ozark Uplands south of the site and contribute to the overall water balance at the site. Overland flow on the site tends to travel north-northeast.

Underlying the Quaternary alluvium of the proposed site, the uppermost continuous bedrock water-bearing unit is within the Ordovician dolomites. Though no confining unit separating the alluvial aquifer from the underlying Ozark Aquifer has been identified, the thickness of saturated alluvium and the groundwater direction and gradient makes it highly unlikely that this lower aquifer could become contaminated by the proposed site.
The proposed AmerenUE utility-waste landfill was visited to conduct preliminary site investigations and determine the general suitability for use as a utility waste disposal area. The site is located in the east halves of Sections 17 and 26, Township 44 North, Range 2 East, in the Lower Missouri River Alluvial Plain. The elevation of the proposed site is approximately 465 masl. The area of the proposed site is an alluvial bottomland bounded by the Missouri River on the north, east and west; and the Ozark Uplands to the south. The proposed utility landfill is tentatively sited in the alluvial bottoms, approximately one-third of a mile to the east of the existing Ameren Labadie power plant.

Examination of the well logs of on-site boreholes indicates the presence of alluvial materials ranging from silts and clays to fine to coarse grained sand, to gravel, cobble and boulder-size clasts of limestone, dolomite and insoluble clasts at depth. Some organic materials, such as decaying trees were observed at depth in the logs. There is no evidence of a lower confining unit within the alluvium. However, the thickness of the alluvium (over 100 feet thick) and the shallow depth to groundwater (ranging from eight to 20 feet) and the existing groundwater gradients indicate a low probability of groundwater contamination from this facility into the lower Missouri River alluvial aquifer or the Ozark Aquifer.

During the site visit for the preliminary site investigation, fault displacement was observed in the bed cut of a railroad bordering the southern edge of the proposed utility landfill. This fault appeared to transect the Ordovician-age Everton Formation and the overlying Ordovician-age St. Peter Sandstone. Inactive bedrock faults are not uncommon, however, further exploration may be warranted during the detailed site investigation.

<table>
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<th>Results of Preliminary Investigation</th>
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<th>☐ Disapproval</th>
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<td>☐ Hydrological ☐ Collapse Potential ☐ Bedrock ☐ Soil</td>
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</table>

Report By | Blake Smotherman | Report Date | 02/03/2009 |

CC Charlene Fitch, Paul Reitz (Reitz & Jens, Inc.), Mike Carlson (Gredell Engineering), Region I - East Central SWMD
December 3, 2008

Mr. Larry Pierce, R.G.
Unit Chief - Geological Survey Program
Division of Geology and Land Survey
Department of Natural Resources
P.O. Box 250
Rolla, MO 65402-0250

RE: Preliminary Site Investigation Request - Proposed Utility Waste Landfill
AmerenUE Labadie Power Plant, Franklin County, Missouri

Dear Mr. Pierce,

As discussed in our November 12 meeting in your office, enclosed is a Preliminary Site Investigation (PSI) request for a proposed Utility Waste Landfill at AmerenUE's Labadie Power Plant in Franklin County, Missouri. This PSI request is being made in accordance with 10 CSR 80-2.015(1)(A). The PSI request encompasses approximately 1,042 acres, however only a portion of the area will be permitted as a solid waste disposal area. Following the Department's PSI findings, AmerenUE will identify and delineate a smaller footprint for the Detailed Site Investigation (DSI), design and permitting of the actual solid waste disposal area.

Ameren either currently owns, or has a verbal agreement to purchase all of the land within the PSI limits by February 27, 2009. The purchase agreement includes the rights to access the site for the purpose of completing the DSI.

A USGS map at 1” = 2000’ scale is attached to the PSI request form. The limits of the PSI area are shown on this map. Generally the PSI area extends from Labadie Bottom Road on the west to a property line approximately 1500 feet east of Davis Road on the east, and from the existing agricultural levee on the south to the existing levee on the north. Additional site information, including site maps, boring logs, piezometer locations, and piezometric water level data were provided to you on November 12th. This additional information is referenced but not submitted with this PSI request. Additional copies of this information can be provided at your request.

We understand that 10 CSR 80-2.015(1)(A) requires review and approval/disapproval of the PSI within sixty (60) days of receipt. It is also our understanding that Department staff will make a site visit during this 60 day time...
period to observe site conditions. AmerenUE requests notification of this site visit so that the necessary and appropriate representatives can be present during that visit. Please coordinate the date of your site visit with either myself or Paul H. Reitz, P.E. with Reitz & Jens, Inc. I can be reached at prpike@ameren.com or 314-554-2388. Mr. Reitz can be reached at preitz@reitzjens.com or 314-993-4132, ext. 224. Once contacted, we will subsequently notify other appropriate AmerenUE representatives of the planned date and time of your staff's site visit.

If you have any questions or would like additional information regarding this PSI request, please contact me at 314-554-2388 or prpike@ameren.com.

Sincerely,

[Signature]

Paul R. Pike
Strategic Analyst
Environmental Services

Enclosures

cc: Bill Duley, R.G., Geological Survey Program w/enclosure
Charlene Fitch, Waste Management Program, w/enclosure
Paul Reitz, P.E., Reitz & Jens, Inc., w/enclosure
Mikel Carlson, R.G., GREDELL Engineering Resources, Inc., w/enclosure
### AMERENUE Labadie Plant Utility Waste Landfill

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<td><strong>NAME AND COMPANY OF REQUESTOR</strong></td>
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<td></td>
<td><strong>TELEPHONE</strong></td>
</tr>
<tr>
<td></td>
<td>One Ameren Plaza, 1901 Chouteau Ave</td>
</tr>
<tr>
<td></td>
<td>MO</td>
</tr>
<tr>
<td></td>
<td>(314) 342-1000</td>
</tr>
</tbody>
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**FACILITY INFORMATION**

- **TYPE OF DISPOSAL AREA PROPOSED**
  - [x] Utility Waste Landfill
  - **SPECIAL WASTE LANDFILL**
  - [ ] Sanitary Landfill
  - [ ] Demolition Landfill

- **ESTIMATED SIZE OF DISPOSAL AREA IN ACRES**
  - 4/- 1042

- **ESTIMATED ELEVATION OF THE SUB-BASE GRADE IN FEET ABOVE MEAN SEA LEVEL**
  - 455 ft.

**A special waste is defined as "solid-waste requiring handling other than normally used for municipal waste."**

**SKETCH OR MAP MUST BE SUBMITTED WITH REQUEST!**

A topographic map must be provided with this request that contains the following information: all known wells, springs, sinkholes, caves, mines, roads, and dwellings within 1/4 mile of the facility. Show the estimated boundaries of the disposal facility and any existing borings, test pits, or excavations which expose soil or bedrock. Include a scale and north arrow on the map.

**COMMENTS**

**A USGS topographic map is attached, which outlines the approximate boundaries of the requested PSI investigation area. The area delineated includes water supply features of the future solid waste disposal area. Following receipt of DGLS's PSI report, AMERENUE intends to delineate a final, smaller disposal area footprint for evaluation during the DSI process.**

Additional site information, including site maps, soil borings, and groundwater data, was provided to DGLS at our November 12, 2008 meeting in Rolla.

Please contact Paul Pike at 314-554-2388 or Paul Reitz (Reitz & Jens, Inc.) at 314-993-4132 to coordinate the PSI site visit.

**REQUESTOR'S SIGNATURE**

Date: 12/3/2008

**OWNER'S SIGNATURE (INDICATING PERMISSION TO ACCESS PROPERTY)**

Date: 12/3/2008
Bcc: J. Thee, w/enclosure
    K.D. Stumpe w/enclosure
    M. J. Tomasovic w/enclosure
    D. V. Fox, w/o enclosure
    E. J. Kammerer w/o enclosure
    S. B. Knowles, w/o enclosure
    T. J. Fox w/o enclosure
    B. S. Skitt, w/o enclosure
    C. R. Henderson w/o enclosure
    W.E. Kahl w/o enclosure
    M. L. Menne, w/o enclosure
    S. C. Whitworth, w/o enclosure
    J. C. Pozzo, w/o enclosure
    File WM 3.5.8 w/enclosure
APPENDIX 2

Geotechnical Work Plan for
AmerenUE Labadie Plant UWL
Appendix 2

AMEREN LABADIE PLANT UWL
DETAILED SITE INVESTIGATION – GEOTECHNICAL WORKPLAN

1.0 SCOPE OF GEOTECHNICAL INVESTIGATION

1.1 This geotechnical investigation is a component of a Detailed Site Investigation (DSI) for the proposed Utility Waste Landfill (UWL) for the Ameren U.E. Labadie Power Plant in Franklin County, Missouri. The purpose of this geotechnical investigation is to provide data of subsurface conditions for: 1) the geologic and hydrogeologic characterization of the site for the DSI to be submitted to MDNR-DGLS, and 2) the geotechnical analyses and design of the UWL.

1.2 The geotechnical component of the DSI will consist of 100 temporary borings or cone penetrometer test (CPT) soundings. Three borings from the 2007 investigation by Reitz & Jens will be included. We anticipate a minimum of 21 new temporary borings, to supplement the geological borings, and 76 CPT soundings. The geotechnical component will be done after the geological borings and piezometers are completed. These borings and CPT soundings will be distributed evenly across the proposed site, unless the geological borings indicate portions of the site that need further investigation.

1.3 If CPT soundings indicate an unexpected soil stratigraphy in an area that was not found in the geological or geotechnical borings, then additional geotechnical borings will be made to verify the soil stratigraphy and to obtain soil samples.

2.0 DEPTH CRITERION FOR GEOTECHNICAL BORINGS

2.1 The geotechnical borings will be made to a minimum depth of 35 feet, which is a minimum of 25 feet below the proposed depth of the UWL. The borings will extend beyond the minimum depth of 30 feet to a depth where the following two criteria are met: 1) the uncorrected N-value from the Standard Penetration Test (SPT) with an automatic hammer is a minimum of 12 blows/foot, AND 2) the last 15 feet of soil is classified as sand or gravel (Unified Soil Classifications of SW, SP, SM, GW, GP, GP-SP).

2.2 One of the geotechnical borings will extend to drilling or sampler refusal on bedrock or boulders. Drilling refusal is defined as a penetration rate with a fishtail or similar drill bit of less than 0.2-inches per minute for 5 minutes and with a downward pressure of at least 500 psi. Sampler refusal is defined as less than 6 inches of penetration after 50 blows with an automatic SPT hammer. The deep boring will be located to supplement the two deep borings completed by Reitz & Jens in 2007.

2.3 Borings may be advanced from the ground surface to the depth of the ground water table using 4.25-inch I.D. hollow-stem augers. The depth of the water inside the hollow-stem augers shall be maintained at the same depth as the surrounding ground water. The water levels shall equalize before a SPT is performed. After the underlying sand strata has been reached below the prevailing ground water table, the borings shall be advanced using rotary drilling techniques with Bentonite or drilling revert to stabilize the hole. The drilling fluid and cuttings shall be re-circulated using a metal drilling-mud pit.
3.0 CONE PENETROMETER (CPT) SOUNDINGS

3.1 CPT soundings will be made with a dedicated cone penetrometer truck or drill rig, and in general accordance with ASTM D 5778-07, "Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils." A copy of the standard is included in Appendix 6. A piezocone penetrometer will be used, capable of measuring end bearing of the cone, side friction on the sleeve behind the cone, and pore water pressure. A hydraulic ram will be used to push the rod string and cone into the ground at a constant rate of 4 feet per minute. The penetration of the cone may be paused at selected intervals in fine-grain soils to measure pore water pressure dissipation, which will be used to estimate hydraulic conductivity and consolidation properties.

3.2 The depths of the CPT soundings will be a minimum of 30 feet, or to the depth of the dense coarse sand and gravel stratum found in the previous borings by R&I at about 30 to 40 feet deep.

3.3 As the penetrometer is advanced, Bentonite grout will be pumped into the annular space formed between the smaller diameter sounding rods and the larger diameter cone penetrometer. After retrieval of the rod string, grout levels will be checked and more grout will be added if necessary.

3.4 In addition to the planned CPT soundings, a minimum of two piezometer borings will be selected and a CPT sounding will be made adjacent to each of the piezometer borings, to correlate the readings from the CPT with a continuously-sampled boring. The borings will be selected to provide a range of different subsurface conditions, if possible.

3.5 A separate report of the CPT soundings will be generated. The report will contain the data from the CPT in 6-inch increments, both in tables and plotted. The data will be analyzed to estimate: soil classification, undrained shear strength ($s_u$) or drained friction angle ($\phi'$), relative density, and normalized Standard Penetration Test N-value.

4.0 SAMPLING CRITERION FOR GEOTECHNICAL BORINGS

4.1 From the ground surface (GS) to a depth of 10 feet: The thickness of the ploughed zone with organics and root balls, etc., shall be noted on the boring log based upon initial auger cuttings or a shallow test pit adjacent to the boring. Samples of the subsurface soils will be taken at depths of: 1.0 feet, 4.0 feet, 7.0 feet, and at 10.0 feet. Samples will be taken using either: 1) a hydraulically pushed, 3-in. O.D., thin-wall "Shelby tube" sampler (ASTM D-1587); or 2) a 2-in. O.D., split-spoon sampler driven by an automatic hammer in conjunction with a Standard Penetration Test (ASTM D-1586). One Shelby tube sample of fine-grain soils will be taken in the first 10 feet of each boring. The depth of the Shelby tube sample will be rotated between the above depths except that if the uncorrected N-value from a SPT is less than 5 blows/foot AND is a fine-grain soil, then the following sample will be a Shelby tube regardless of whether a Shelby tube sample has already been taken at a shallower depth.

4.2 From a depth of 10 feet to a depth of 50 feet: Samples will be taken at intervals of 5.0 feet. Samples will be obtained using a 2-in. O.D., split-spoon sampler driven by an automatic hammer in conjunction with a Standard Penetration Test (ASTM D-1586). However, if the uncorrected N-value from a SPT is less than 5
blows/foot AND is a fine-grain soil, then the boring will be cleaned out to the depth of the SPT sample and a 3-in. O.D., thin-walled "Shelby tube" sample (ASTM D-1587) will be taken.

4.3 From a depth of 50 feet to refusal: Samples will be taken at intervals of 10.0 feet. Samples will be obtained using a 2-in. O.D., split-spoon sampler driven by an automatic hammer in conjunction with a Standard Penetration Test (ASTM D-1586).

5.0 FIELD PROCEDURES

5.1 A qualified geologist or geotechnical engineer will supervise and maintain quality control of the drilling and sampling program, and will log the borings, determine the completion depths, collect and prepare samples for transport, and obtain water level readings. Borings will be backfilled after the 24-hour groundwater level readings. Backfilling of borings will be in accordance with 10 CSR 23-6.

5.2 A continuous field log of the boring shall be recorded in general accordance with ASTM D-5434 "Field Logging of Subsurface Explorations of Soil and Rock," and shall include the following information at a minimum:

1. All of the information at the top of the field boring log form.
2. An accurate description of any deviation from the planned boring location.
3. Drilling method(s) used, including diameter of augers.
4. Depths of generalized soil and rock boundaries encountered, based upon drilling characteristics, samples, cuttings, color of drilling fluid, etc.
5. Depths of samples, including: type, length sampled, length recovered, hammer blows for each 6-in. interval for Standard Penetration Tests (SPT=§).
6. Loss of drilling fluid, if applicable.
7. Water level readings (to 0.1 foot) when free water is first encountered, at the conclusion of drilling and 24 hours after drilling.
8. Type of drilling fluid used to stabilize the hole, if applicable.
9. Identification of the soil as specified below.
10. Pocket penetrometer readings on firm to stiff cohesive soil samples, or torvane reading on soft soil samples (Shelby-tube samples only).
11. Note the length and cause of significant delays in field operations.
12. If the boring does not stand open, record the depth of material sloughed into the hole or the depth to collapse of the hole.
13. Note how the boring is backfilled, including installation of piezometer if applicable.

5.3 Soils will be classified in general accordance with ASTM D-2487 AStandard Classification of Soils for Engineering Purposes (Unified Soil Classification System)@ and ASTM D-2488 AStandard Practice for Description and Identification of Soils (Visual-Manual Procedure),@ except that the descriptions will be in the following sequence:

a) Modifying Major Component (ex. AClayey@ or ASilty@)
b) Major Component (ex. ACLAY,@ ASAND,@ ASILT,@ etc.)

REITZ & JENS, INC. Consulting Engineers
c) USCS abbreviation in parentheses [ex. A(CL)@ or A(SM)@]
d) Color
e) Secondary Components or Inclusions (ex. A with sand@ or A trace gravel@)
f) Structure (ex. A homogeneous, @ A laminated, @ A slickensided, @ etc.)
g) Consistency (ex. A soft @ or A medium stiff @ for cohesive soils; or A loose @ or A medium dense @ for cohesionless soils)
h) Moisture (ex. A moist @ or A wet @)

5.4 If a clay or silt is high plastic, put A High Plastic@ as the Modifying Major Component (ex. A High Plastic CLAY@). Similarly, if silt is non-plastic, put A Non-Plastic SILT@. If a material is primarily man-made, put A Miscellaneous FILL@ for the Modifying Major Component and Major Component. If the material is primarily a specific soil type but was placed as a fill, then list the Origin as A (fill) . @

5.5 All personnel in the vicinity of the drill rig shall wear hard hat, steel-toed safety boots, and long pants or coveralls. All personnel operating the drill rig or handling drilling equipment and tools shall wear eye protection and gloves, or as otherwise specified in the Project's approved health & safety plan.

5.6 If unknown or unexpected contamination of the soil and ground water is encountered, then field work shall stop immediately and Reitz & Jens shall be contacted. Secure the area to prevent the spread of the contamination and to prevent anyone from approaching the boring.

5.7 Soil cuttings and material sloughed from the side of the boring shall be removed prior to sampling. The field technician shall note the length of drilling rod in the hole before it is extracted and after the sampler is set on the bottom of the boring. If the change in depth is more than 3 inches, then the sampler shall be removed and the hole cleaned out.

5.8 Hollow-stem augers shall be advanced with the center plug attached to the drill rods, to prevent soil from entering the augers.

5.9 The procedures for drilling a test hole and obtaining split-spoon samples are described in ASTM D-1586. The following major points shall be followed and noted on the field boring log:

1. Drag, chopping, fishtail and roller-cone bits between 2.2 in. and 6.5 in. diameter are permitted. Bottom discharge bits are not permitted in rotary wash borings. The inside diameter of hollow-stem augers, and the outside diameter of continuous-flight augers, must also be between 2.2 in. and 6.5 in.

2. Flush-joint steel AA@ drill rods (1-5/8" O.D. and 1-1/8" I.D.) shall be used. The rods must be tight so that there is no movement at the joints.

3. Count and record the number of blows for each 6 inches of penetration.

4. THE SPLIT-SPOON SAMPLER SHALL NOT BE DRIVEN MORE THAN 18 INCHES.
5. A representative soil sample shall be taken from each split-spoon. If there is a change in soil type, two samples shall be taken. Each sample shall be placed in a glass jar and immediately sealed to prevent loss of moisture. The jar should be filled as much as possible. Label each jar (not the lid) with the boring number, sample number, sample depth, and the blow count for each 6-in. increment. Protect the samples against extreme temperature changes.

5.10 The procedures for obtaining thin-walled tube samples are described in ASTM D-1587. The following major points shall be followed and noted on the boring log:

1. Jetting or bottom discharge bits are not permitted.

2. Loose material from the bottom of the hole shall be removed without disturbing the interval to be sampled.

3. The bottom of the hollow-stem auger or casing shall not be below the interval to be sampled.

4. Only clean, new, galvanized Shelby tubes, 30 or 36 inches long, shall be used. The tubes shall be round, and the cutting edge shall be sharp and free of burrs or nicks.

5. The tube shall be hydraulically pushed in one continuous, rapid motion without rotation.

6. THE SAMPLER SHALL NOT BE PUSHED MORE THAN 24 INCHES.

7. Trim the bottom end of the sampler and obtain a pocket-penetrometer or torvane reading, and then seal the bottom end of the tube with a tight-fitting plastic cap and duct tape.

8. Remove the excess fluid and loose material from the upper end of the tube and measure the length of the sample recovered.

9. Seal the top end of the tube with a tight-fitting plastic cap and duct tape.

10. Clean and dry the outside of the tube and write with a permanent marker on the tube (not the cap) the boring number, sample number, sample depth, and recovered length.

11. Tube samples shall be maintained in a vertical position with the bottom end down at all times. Protect the sample from extreme changes in temperature or disturbance.

5.11 ASTM D 4220-95 AStandard Practice for Preserving and Transporting Soil Samples@ will be used as a guide. The following shall be the minimum requirements:

1. Relatively undisturbed, thin-walled samples shall be transported vertically in a rack to prevent disturbance.

2. All samples shall be protected from freezing or extreme temperatures at all times.
3. All samples shall be transported to Reitz & Jens' laboratory daily, or more frequently if necessary to prevent damage due to temperature extremes.

4. Bulk samples shall be transported in clean plastic 5-gallon buckets with a sealed lid. (A sealed lid is not required if the moisture content of the sample will be determined from other samples.)

5. Jar samples shall be transported in the cardboard box with dividers that come with the jars.

6. Reitz & Jens shall be responsible for the transporting of all geotechnical samples, and for maintaining records of chain of custody.

6.0 LABORATORY PROCEDURES

6.1 All samples will be transported to Reitz & Jens' AMRL-approved laboratory. Shelby tube samples will be stored in the vertical position.

6.2 All testing will be done in general accordance with the latest applicable ASTM procedures. The following minimum guidelines will be followed:

   1. All samples shall be classified in the lab by a different technician, geotechnical engineer or geologist than the one who performed the field classifications.

   2. Moisture contents will be determined on all fine-grained soil samples.

6.3 All split-spoon jar samples shall be retained until the final boring logs are completed, or for a period of 90 days, whichever is longer. Relatively-undisturbed tube soil samples, either un-tested or unused portions of tested samples, shall be discarded after the final boring logs are completed or after 45 days, whichever is longer. Selected representative specimens from tube samples may be sealed in paraffin in plastic tubes for long-term storage.

6.4 The procedures contained in Reitz & Jens' AAP-approved Quality Manual shall be followed.

6.5 Once all laboratory testing for given boring is complete, it will be combined with the information in the field boring log using GEOSYS. The draft log will be reviewed both for accuracy (i.e. correct blow counts, sample types, sample depths, recovery percentages and laboratory test results) and consistency of the information provided. The final boring log=s consistency will be reviewed by an experienced registered geotechnical engineer. In this review the engineer will look at the sample description on the field log, the sample description from the laboratory, and the laboratory test results to make verify they are consistent in representing the sample. In certain instances, a judgment call will need to be made to reconcile these three aspects of each sample. Unless there is strong evidence to the contrary, the stratification lines identified in the field boring log will be shown on the final boring log.

6.6 The final test results will include the following:

REITZ & JENS, INC. Consulting Engineers
1. Index Properties B unless other testing is requested, moisture contents and liquid and plastic limits will be transmitted only on the final boring logs.

2. Grain Size Distribution and Hydrometer Tests B graph of grain size distribution and USCS designation (CL, ML, SM, SP, etc.).

3. Unconfined Compressive Strength Tests B the sample description from the laboratory classification, USCS designation (CL, CH, ML, etc.), moisture content, dry density, liquid limit and plasticity index (if requested), unconfined compressive strength, mode of failure, strain at failure, and a graph of the stress-strain curve.

4. Unconsolidated-Undrained Triaxial Shear Strength Tests B the sample description from the laboratory classification, USCS designation (CL, CH, ML, etc.), moisture content, dry density, liquid limit and plasticity index (if requested), unconfined compressive strength, mode of failure, strain at failure, the stress and strain readings, and graphs of the stress-strain curve and the total stress Mohr=’s circle.

5. Consolidated-Undrained Triaxial Shear Strength Tests B the sample description from the laboratory classification, USCS designation (CL, CH, ML, etc.), moisture content, dry density, liquid limit and plasticity index (if requested), unconfined compressive strength, mode of failure, strain at failure, the stress and strain readings, and graphs of the stress-strain curve, the total and effective stress paths, and the total and effective stress Mohr=’s circles.

6. Consolidation and Swell Tests B the sample description from the laboratory classification, USCS designation (CL, CH, ML, etc.), moisture content, dry density, liquid limit and plasticity index (if requested), the graph of consolidation curve (strain vs. pressure), tables of $c_v$ and $c_s$ for each load increment, and the graphs of the time-deflection curve for each load increment.

7. Moisture-Density Relationship (standard or modified Proctor compaction tests) B graph of the dry unit weight versus moisture content, the sample description from the laboratory classification, USCS designation (CL, CH, ML, etc.), natural moisture content, maximum dry unit weight, optimum moisture content, and the liquid limit and plasticity index.

7.0 LABORATORY TESTING PROGRAM

7.1 The laboratory testing program will be determined based upon the types of subsurface soils encountered. The following general guidelines will be followed in developing the laboratory testing program.

7.2 Grain-size analyses will be performed on cohesionless samples (Unified Soil Classifications of SW, SP, SM, GW, GP, GP-SP). If the percentage of fines (passing #200 U.S. sieve) is greater than 25%, then a hydrometer analysis will be performed on the fine-grain portion of the sample. Samples that appear to be from the same stratum may be combined.
7.3 Moisture and dry unit weight shall be measured on all undisturbed Shelby tube samples if possible.

7.4 Two series of unconsolidated-undrained triaxial shear strength tests will be performed on each major cohesive soil stratum. Each series will have a minimum of three points.

7.5 Two series of consolidated-undrained triaxial shear strength tests will be performed on each major cohesive soil stratum that will be below the berm. Each series will have a minimum of three points.

7.6 The following tests will be performed on samples of prospective construction materials:

a) natural moisture content,
b) Atterberg liquid and plastic limits,
c) grain-size analyses,
d) moist-density relationship (standard or modified Proctor),
e) unconsolidated-undrained triaxial shear strength tests on compacted samples, and
f) flexible-wall permeability tests on compacted samples.

Samples will be compacted to the appropriate compaction criterion (95% of standard Proctor or 90% of modified Proctor) and at moisture contents approximately 3% higher than the optimum moisture content determined by the corresponding test.

7.7 One-dimensional consolidation test on each major cohesive soil stratum beneath the UWL.
APPENDIX 3

Guidance for Conducting and Reporting Detailed Geologic and Hydrologic Investigations at a Proposed Solid - Waste Disposal Area

(January 2007)

(copy of 10 CSR 80-2.015 Appendix 1)
APPENDIX 1

GUIDANCE FOR CONDUCTING AND REPORTING DETAILED GEOLOGIC AND HYDROLOGIC INVESTIGATIONS AT A PROPOSED SOLID-WASTE DISPOSAL AREA

Missouri Department of Natural Resources
Division of Environmental Quality
Division of Geology and Land Survey

This appendix contains the following:

- Elements and format of a workplan for conducting the Detailed Site Investigation.
- Guidance for conducting an acceptable detailed geologic and hydrologic investigation of a proposed solid-waste disposal area.
- Guidance for the acceptable presentation of site characterization data.
- Form for requesting a preliminary investigation for a proposed solid-waste disposal area.

ELEMENTS AND FORMAT OF A DETAILED SITE INVESTIGATION WORKPLAN

The detailed site investigation workplan must contain the following elements plus any additional site-specific elements which may be requested by the Geological Survey Program (GSP).

1. Topographic map at a scale of 1:24,000 showing the pertinent property boundaries, as well as the location of the proposed solid-waste disposal area, and potential borrow areas.

2. Site map at a suitable scale to display proposed locations for pits, borings, and piezometers.

3. A general description of the proposed facility to include:
   a. Maximum depth of excavation
   b. Total acreage to be developed as a solid-waste disposal area.

4. Description of proposed methods for site exploration to include:
   a. Drilling methods
   b. Sampling methods
   c. Piezometer and monitoring well construction methods (must comply with 10CSR23-4):
      1) Approximate depth intervals to be screened
      2) Specific grout mixtures and emplacement methods to be used
   d. Aquifer test methods
   e. Alternative exploration methods (such as geophysical methods)

5. Record keeping procedures for:
   a. Well logs, boring logs, drilling logs, pit logs
   b. On-site precipitation data
   c. Periodic water-level measurement data from piezometers
   d. Aquifer test data

DETAILED SITE INVESTIGATION
General Procedures for Detailed Site Investigations

The potential disposal area construction permit applicant is responsible for retaining a qualified groundwater scientist to provide the GSP with a complete and accurate evaluation of the geologic and hydrologic conditions of the proposed solid-waste disposal area. All geologic and geohydrologic work must be completed under the direction of a geologist registered in the State of Missouri per RSMo 256.450 through 256.483 and the rules promulgated pursuant thereto. A consultant who subcontracts the drilling of piezometers or monitoring wells must hold a restricted or a nonrestricted monitoring well installation contractor's permit. Drilling must be done by a driller holding a nonrestricted monitoring well installation contractor's permit and appropriate permit numbers must be prominently displayed on all drill rigs used for site characterization, as required by 10 CSR 23 Chapters 1, 2 and 4. The detailed site investigation is intended to provide the GSP with sufficient geohydrologic data to determine if the site is suitable for the development of a solid waste disposal area.

The minimum elements of a detailed site investigation are partially dependent on site-specific geologic conditions. As a result of data gathered during the preliminary or detailed site investigation, the GSP may require additional investigations to adequately define the geology and hydrology of the site. The GSP may require less detailed investigation based upon site geohydrologic conditions.

Geophysical methods may be used to help characterize the site; however borings or pits must be located and drilled to verify the results of the geophysical survey(s). Where geologic structures or solution features are present or suspected, additional borings or pits will be required to adequately define the extent and distribution of these features across the site, and to determine the relationships between these features and hydrostratigraphic units.

Sinkholes, solution-enlarged fractures and caves may have very small, near-surface expressions that a boring program would not be expected to detect. Sites will be rejected during preliminary or detailed site investigations where the site is characterized by karst terrane features which may affect the structural integrity or effective monitoring of the site.

Field Direction

A qualified groundwater scientist must direct the excavation of all pits, the drilling of all borings, the performance of any geophysical surveys, and the installation, development and abandonment of all exploratory wells or piezometers. Interpretations of geological data must be conducted under the direction of a geologist registered in the State of Missouri per RSMo 256.450 through 256.483.

A qualified groundwater scientist must supervise all field testing to determine the geologic and hydrologic characteristics of the material encountered or intended for use at the proposed site. A qualified groundwater scientist must maintain accurate and complete field notes of the investigation activities.

A land surveyor registered in the State of Missouri must determine the location and elevation of all wells and piezometers. Borings, excavation pits and all transects performed as part of a geophysical exploration will be located to the nearest one-tenth (0.1) foot by a land surveyor registered in the State of Missouri. All elevation measurements, grid patterns, and coordinates must be established and used consistently throughout the investigation and referenced to North American Datum (NAD) 1983 and National Geodetic Vertical Datum (NGVD) 1929 or North American Vertical Datum (NAVD) 1988. Monitoring well and piezometer measuring-point elevations must be accurate to the nearest one-hundredth (0.01) foot.

Field Investigations

The minimum requirements for conducting a detailed subsurface investigation are listed below. Alternative investigation techniques and procedures may be approved at the discretion of the GSP. Additional borings or pits may be required, subject to site-specific conditions, to fully characterize the geology of the area. The number of borings, pits, and piezometers required is dependent upon the anticipated size of the proposed disposal area and the site geohydrology. Borings that are not used as monitoring wells or piezometers must be permanently abandoned and reported as per 10 CSR 23-4. Exploration pits must be backfilled using native material, compacted to natural density condition, and their locations clearly marked on site maps.
1. Surficial Materials

A qualified groundwater scientist must determine the thickness, and geotechnical characteristics of significant hydrostratigraphic units, where they exist at the site, above competent bedrock. At least one boring must be drilled per two acres of the proposed disposal area. All borings must be extended to at least 25 feet below the anticipated disposal area sub-base grade or to competent bedrock, whichever is less. All borings must be continuously sampled. Exploration pits may be substituted for borings in areas where the surficial materials can be fully penetrated by the pits. For sites that meet the conditions pursuant to 10 CSR 80-2.015(1)(A)3 the GSP shall require only one boring per four acres of the site.

If geologic structures or solution features are suspected, at least one boring must be completed per acre of the proposed disposal area. All of these borings will be drilled to competent bedrock. Exploration pits may be substituted if approved by GSP.

The borings or pits must be distributed in a grid pattern across the site or located in a manner that will optimize characterization of the site. Deviations from a regular grid pattern must be approved by the GSP. The locations and elevations of borings or pits must be surveyed by a land surveyor.

2. Aquifers

A qualified groundwater scientist must determine the depth, thickness and lateral extent of the uppermost aquifer(s) beneath the proposed site and additional aquifers which are potentially at risk (as determined by the GSP).

Piezometers are required to adequately characterize the groundwater at the proposed site. There must be at least five piezometers, or one piezometer per four acres of the site, whichever is greater, installed in each aquifer to be characterized. For sites that meet the conditions pursuant to 10 CSR 80-2.015(1)(A)3 there must be at least five piezometers, or one piezometer per eight acres of the site, whichever is greater. Piezometer construction and development standards must be in accordance with 10 CSR 23-4.

All piezometers must be distributed in a grid pattern across the proposed site or located in a manner that will optimize characterization of the site. Deviations from a regular grid pattern must be approved by the GSP. An adequate number of piezometers must be located outside the anticipated fill area to sufficiently characterize each aquifer investigated. The measuring-point elevation of the piezometers must be determined by a land surveyor. Additional piezometers may be required to demonstrate the effectiveness of confining units and extent of aquifers. If geophysical methods are used, piezometers must be installed to verify the results of the geophysical survey(s).

A continuously recording precipitation gauge, capable of measuring precipitation events greater than one-tenth (0.1) inch, must be installed at the site concurrent with, or prior to, installation of piezometers. Data from the gauge will be used to interpret any fluctuations in potentiometric level(s) throughout the site characterization period and may be used for other purposes later, at the discretion of the department.

The hydraulic conductivity of the uppermost aquifer(s) beneath the proposed disposal area must be determined. The hydraulic conductivity must be determined in one out of every four piezometers installed for each aquifer tested. The hydraulic conductivity must be determined in the field. Accepted field tests are in situ slug and/or pump tests, as determined through the workplan process, which isolate the geologic unit of interest.

3. Other Hydrostratigraphic Units

At least one boring per four acres of the proposed disposal area or five borings, whichever is greater, must be drilled to characterize hydrostratigraphic units, including the uppermost confining unit, below the anticipated sub-base grade of the site. The depth of these borings will be determined based on geohydrologic conditions at the site. At least five of these borings must be continuously sampled, unless otherwise approved by the GSP. For sites that meet the conditions pursuant to 10 CSR 80-2.015(1)(A)3 there must be at least five of these borings or one boring per eight acres of the site, whichever is greater.
A qualified groundwater scientist must determine the occurrence, thickness, depth and lateral extent of the uppermost confining unit beneath the proposed solid-waste disposal area. If the uppermost confining unit is more than 150 feet below the lowest anticipated sub-base grade, the GSP will determine the need for characterization of the unit. If the thickness of the confining unit is greater than 50 feet, the depth of drilling required will be determined by GSP. The hydraulic conductivity of the uppermost confining bed must be determined by in situ tests in at least one out of every two, but a minimum of five, borings that penetrate the confining unit.

For investigation of horizontal expansions and investigations near previously existing disposal areas, piezometers and borings must be located within 500 feet of the limits of the existing filled area such that there is a minimum of one piezometer per 400 lineal feet extending along the periphery of the existing filled area. As determined by the GSP, if geologic structures or features are present or suspected, one piezometer/boring must be installed per 200 lineal feet along the periphery of the existing filled area. Piezometers will not be installed within the boundary of the pre-existing waste.

Records (Field Notes)

The geologic materials in each boring, exploration pit, piezometer or well must be logged in detail during drilling or excavation by a qualified groundwater scientist. The qualified groundwater scientist must describe and record the physical and lithologic characteristics of each geologic material encountered as well as other information pertaining to drilling or excavation. Field logs and notes pertaining to the field investigation shall be retained by the applicant or owner/operator of a permitted solid waste disposal area until closure.

At a minimum, a qualified groundwater scientist must, in the field, note on a descriptive log the following:

1. Texture of geologic material
2. Color (qualitative descriptions - include mottling) of geologic material
3. Relative degree of saturation (description)
4. Voids
5. Geologic origin
6. Secondary permeability features
7. Zones of incomplete sample recovery
8. Depth at which water is encountered
9. Depth and rate of drilling fluid gain or loss
10. Type and size of drilling/excavation equipment
11. Drilling rate and penetration rate (blow counts), as appropriate
12. Packer tests (intervals tested and results), as appropriate
13. Start and stop times for drilling/excavation
14. Names of field personnel
15. Date, time, weather conditions
16. Depth to water upon completion

All borings or pits must be observed until the water level has stabilized for at least 24 hours following completion. This observation must determine if groundwater has entered the hole, the depth to water, and, if possible, the water bearing hydrostratigraphic units. During observation all borings and pits must be protected from rainfall and runoff.

Laboratory Analysis

All samples collected for laboratory analyses must be clearly labeled (sampling location - boring/pit number, depth, date of sample) and preserved. Soil samples not destroyed by testing and rock core must be stored, protected from the weather, and available for the GSP's inspection in Missouri until closure.
Laboratory Testing

A laboratory must be retained to conduct geotechnical analyses for each unconsolidated material encountered to verify field observations. The following must be recorded for each sample tested.

1. Texture
2. Color (based on a Munsell color chart - include mottling)
3. Grain size distribution (reported in percent)
4. Soil classification (reported in Unified Soil Classification System)
5. Moisture content (reported in percent)
6. Liquid Limit
7. Plasticity Index
8. Standard Proctor density
9. Names of lab personnel
10. Date

Monitoring Wells

While monitoring wells are not normally required as part of the detailed site investigation, background water quality data will be required prior to operation of a solid-waste disposal facility. The number of monitoring wells required will be dependent upon the presence and number of aquifers monitored and the presence and number of confining beds. Well construction standards and development must be in accordance with 10 CSR 23-4.

A minimum of one monitoring well must be located hydraulically upgradient and three monitoring wells located hydraulically downgradient for each aquifer monitored. These wells must be located outside of but not greater than 500 feet from the anticipated limit of the area. The screen and/or filter-pack must not extend through confining units.

Water Level Data Collection

Measurements of water level, to the nearest hundredth (0.01) of a foot, must be made every month for one year for all wells and piezometers. For sites that meet the conditions pursuant to 10 CSR 80-2.015(1)(A)3 the GSP may allow termination of water-level measurements after six (6) months. Water-level measurements in all wells and piezometers should be made within a 48-hour period, if possible. Additional measurements may be necessary as determined by the GSP.

PRESENTATION OF DATA AND INTERPRETATIONS

The following information must be provided in the order specified below. The report must be prepared under the direction of a qualified groundwater scientist who is a geologist registered in the State of Missouri per RSMo 256.450 through 256.483 and the rules promulgated pursuant thereto. This person must sign and seal the report.

1. Table of Contents
2. Introduction (general information about the site vicinity and the investigation)
   A. Location:
      A written narrative of the geographic setting with legal description (section, township, and range)
   B. Regional Geology:
      A written narrative describing the regional lithologic, stratigraphic, structural and hydrologic settings of the area
   C. Historic Land Uses:
      A written narrative describing previous land use such as mining or mineral exploration
3. Method of Study

A written narrative must be provided which describes field and laboratory procedures used to characterize geologic and hydrologic conditions of the site. Standardized laboratory and field procedures may be referenced. All other procedures must be described in detail. Deviations from and amendments to the approved workplan during the detailed site investigation should be described.

4. Results of Investigation

A written detailed narrative must be provided that describes the site-specific geology and hydrology based on data collected. The narrative must include explanations of any anomalous data. Interpretations of results must be presented in a clear and concise manner.

5. Conclusions

A written narrative must be provided that details how the site-specific geology and hydrology will impact the design of the disposal area and groundwater monitoring system. The narrative must assess the inadequacies of the investigation and propose future investigations if needed. The narrative must describe the proposed monitoring system design.

6. References

All published information sources used in the compilation or research of the hydrogeologic investigation must be listed.

7. Appendices

The appendices of the site characterization report must include:

- Compiled logs of all borings, excavations, wells and piezometers.
- The raw data for any and all tests (e.g., pumping tests)
- All additional information that may facilitate the GSP’s assessment of the acceptability of the proposed site.

A. Logs

Lithologic logs of all borings and excavations, including well construction diagrams, must be provided. Each log must include borehole identification, borehole grid location, soil and rock description, sample depths, methods of sampling, sampling date, land surface elevation, borehole total depth, moisture content, and test results such as: blow counts, vane shear, or pocket penetrometer measurements.

B. Tables

Presentations of tabular data that must be supplied include the following:

1. All borehole, well and piezometer construction data. Such data should include the borehole, well or piezometer identification, grid location, total depth, surface elevation and, if applicable, screened interval and hydrogeologic unit monitored.
2. Monthly groundwater elevation measurements for each piezometer or well. The table(s) should indicate the well or the piezometer identification, depth to water from measuring-point, groundwater elevation and date of measurement.
3. The results of all unconsolidated-material testing. The table(s) must include the sample location, depth, sampling date, and test results.
4. The results of all hydrologic testing. The table(s) must include the well or piezometer identification, method and date of test, depths of interval tested, hydrologic unit tested and results.
5. The daily precipitation data collected at the site.
C. Maps
   All detailed site maps for the report must be drawn on a scale where one inch equals 400 feet or less. As appropriate, maps should be drawn on a consistent scale. All maps must include a scale, north arrow, and a clear and concise legend describing all of the symbols used on the map. More than one map will be required to include the following information:
   (1) A base map showing initial topography (on 5 foot contour intervals unless otherwise specified by the GSP), borrow area(s), and proposed disposal area boundary.
   (2) Map(s) showing land use, ownership, residences, septic systems, lateral lines, buildings, wells, cisterns, mined or quarried areas, mine shafts, spoil piles, and all other man-made features within 1/4 mile of the proposed disposal area boundary.
   (3) Map(s) showing springs, water courses, streams, lakes, caves, sinkholes, rock outcrops, and other significant geologic features within 1/4 mile of the proposed disposal area boundary.
   (4) Map(s) showing all borings, excavations, piezometers, and wells constructed for the study.
   (5) Monthly piezometric maps per aquifer to be monitored. The maps must include labels showing water elevations next to each well or piezometer and must indicate the date when the water elevation was measured.
   (6) Map(s) showing inferred results of geophysical explorations with survey tracks (if applicable).
   (7) Map(s) locating cross-sections showing borings used in cross-section representation.
   (8) Map(s) locating floodplains, wetlands and fault(s).
   (9) Map delineating seismic impact zones.
   (10) Bedrock contour map (where applicable).

D. Cross-sections
   Geologic cross-sections must be constructed through all appropriate borings both perpendicular and parallel to the facility baseline as well as along and across all transects which include major geologic features such as faults, sinkholes, and buried valleys. At least one cross-section must be constructed parallel to groundwater flow. The subsurface conditions of the site must be illustrated in these cross-sections. Where more than one interpretation may be reasonably made, conservative assumptions must be used.

The following information must be included on the cross-sections:
   (1) A dashed line or question mark for inferred lithostratigraphic boundaries, a number or symbol to label major soil units (instead of extensive shading) and legend containing a description of the soil units.
   (2) The anticipated sub-base, and final grades for the proposed disposal area.
   (3) All boring logs, the Unified Soil Classification System soil classifications and the geologic origin for each soil unit. The results of all lab and field tests, and all well construction details including screen and seal length along with the stabilized water elevations should be shown on the logs beside the descriptions of the materials encountered.

E. Aerial Photographs
   One or more vertical aerial photographs, representing the entire area of the proposed site plus the area within 1/4 mile of the site must be included in the report. The photos must be taken between November 1 and March 30, within two years of the submittal of the report unless significant excavation has occurred at the site. If significant excavation has occurred at the site during the previous two years, the photos must be taken between November 1 and March 30, within one year of the submittal of the report. The extent of the proposed disposal area, the anticipated limits of the proposed fill area and a north arrow must be added to the photos. Photocopies of the photographs will not be accepted.
APPENDIX 4

Preliminary Report of Feasibility Study,
Reitz & Jens, Inc., May 1, 2007
MEMORANDUM

TO:      Mr. Carl Rezsonya, PE, PMP
          New Generation & Environmental Projects
          Ameren Services

FROM:    Jeffrey Fouse, PE

SUBJECT: Preliminary Report of Feasibility Study
          Labadie Power Plant Utility Waste Landfill
          ESA No. E223, Task No. GEN-56

DATE:    May 1, 2007

This preliminary report presents the results of the field exploration that has been completed for the feasibility investigation for the development of a Utility Waste Landfill (UWL) on property adjacent to AmerenUE’s Labadie Power Plant. This UWL would accept gypsum from the future Wet Flue Gas Desulphurization (WFGD) scrubbers to be constructed at the Plant, and potentially other ash by-products. This investigation was done in general accordance with Task No. GEN-56, dated February 5, 2007, of Reitz & Jens’ Engineering Services Agreement E223 with Ameren Services.

Site Description

The proposed site of the UWL is located east of the Labadie Power Plant as shown in Figure 1. The site is composed of three parcels belonging to Heisel, Drewel and Newman. The site is bounded on the west by Labadie Bottom Road, on the south by the Laclede Gas pipeline, and on the north by a Missouri River levee. The ground surface is relatively flat, ranging between el. 465 and 471. The site is protected from flooding by levees.

An AmerenUE transmission line runs diagonally across the Heisel and Drewel parcels. The east-west portion of Labadie Bottom Road also divides the Heisel and Drewel parcels. Our field investigation located an Explorer pipeline which crosses diagonally the southern half of the Heisel and Drewel parcels, and then runs north along a field road between the Drewel and Newman parcels (see Figure 2).

Field Investigation

Our field investigation consisted of 8 borings, located as shown in Figure 2. Temporary standpipe piezometers were installed in three borings, designated P-1, P-2 and P-3. Borings P-1 and B-7 were drilled to refusal in cobbles or limestone bedrock. The completed depths of these borings were 91.5 feet and 104.5 feet, respectively. The other borings were 20 to 30 feet deep, and were terminated in the underlying medium-dense sand.
Below the surface topsoil and disturbed zone (due to farming), the 8 borings encountered 0 to 8.5 feet of high plastic clay which should have a permeability of $1 \times 10^{-7}$ cm/sec or less when compacted, and thus would be suitable for a composite liner for the UWL. Boring P-1 on the north end and Boring B-8 on the south end had no clay stratum. The thickness of the high plastic clay in the remaining 6 borings ranged from 2.5 feet to 8.5 feet, and averaged about 6 feet. The remainder of the soils in the upper 13.5 feet consisted of sandy silts, silty clay, silt, and silty sand.

Below about 13.5 feet, the borings encountered strata of medium-dense to very dense sand and gravelly sand. Cobbles and boulders were encountered below about 50 feet in the two deep borings.

Geology

The site is located on the floodplain adjacent to the Missouri River between River Mile 57 and 58. The nearest river gage is located in Washington, Missouri at river mile 67.0. The site is contained within the floodplain, approximately 0.5 miles south of the Missouri River. Alluvium, or sediment deposited by flowing water, covers the entire site. To the south, the site is bordered by loess covered uplands or the River Hills landform.

Geologic structural features closest to the proposed site are the Eureka-House Springs anticline, Moselle normal fault and the Jeffreisburg fault. These features were formed as a result of periods of uplift in the Ozarks and seismic activity from the New Madrid fault system. The Eureka-House Springs anticline is located approximately 7 miles to the northeast. The Moselle normal fault is approximately 10 miles to the southwest. The Jeffreisburg fault is approximately 14 miles to the southwest. There is no literature indicating that these faults are currently active or have been active in the recent geologic past.

There do not appear to be any geologic issues that would preclude the construction of a UWL, such as recent fault, unstable ground or karst topography.

Preliminary Findings

The borings reveal alluvial deposits that are typical of the Missouri River floodplain. The general soil stratigraphy will support a gypsum stack that is 100 feet high, or higher. The upper (0 to 13.5 feet) strata contain suitable materials for construction of the perimeter berms, but may not have sufficient clays to construct the 2-foot composite liner and final cover. Other borrow areas probably will need to be identified.

The existing Explorer pipeline is a controlling factor in the plan of the UWL. We contacted Pat Nwakoby with Explorer Pipeline (918-493-5172) to discuss the possibility of relocating the pipeline. He said that Explorer would be willing to relocate a portion of the pipeline, if AmerenUE provides a new easement and pays the construction costs. He estimated that the cost to relocate the pipeline would be on the order of $250 per linear foot.

The UWL would store WFGD gypsum utilizing the wet gypsum stack method similar to the UWL proposed for the Sioux Power Plant. The UWL would consist of a perimeter berm constructed using onsite soils or ash from the Labadie Power Plant. For this study, the berm was assumed to be 25 feet high and have a width at the crest of 12 feet. The exterior slope would be 3 horizontal to 1 vertical (3:1) and the interior slope would be about 2.4:1. The boundary of the wet gypsum stack would be coincident.
with the inside edge of the crest of the berm. The wet gypsum stack would extend to a total height of 100 feet above the existing ground surface. The side slopes of the wet gypsum stack would be 3:1. A recycle pond would be required to store all of the potential water discharge: decanted water from the top of the wet gypsum stack, seepage water from the interior consolidation of the stack, and all storm water. The recycle pond was assumed to be 25 feet deep, formed by the construction of a perimeter berm with the same geometry of the berm for the wet gypsum stack.

State regulations require that the boundary of the solid waste (i.e. the wet gypsum stack) shall be a minimum of 100 feet from an exterior property boundary or right-of-way. Also, the minimum distance from the outside edge of the perimeter berm to any utility (AmerenUE transmission lines or buried gas pipeline) was assumed to be 60 feet.

We computed the volume of the wet gypsum stack on the basis of three possible scenarios:

1. Assuming that the Explorer pipeline was not relocated, and that the Newman property was not acquired, then a single wet gypsum stack could be constructed as shown in Figure 3 (Stack 1). The recycle pond could be located west of the transmission line and north of the Explorer pipeline. The pond could have a footprint of 35 acres, and a volume of 472 acre-feet (to the top of the berm). The wet gypsum stack could have a footprint of 178 acres and a volume of 21.4 million cubic yards (CY). The land west of the transmission line and south of the Explorer pipeline is too small to contain a wet gypsum stack operation; however, a dry stack could be constructed with a volume of 1.7 million CY. The quantities of fill required for construction of the perimeter berms are approximately: 1) 900,000 CY for the Wet Gypsum Stack 1, 2) 500,000 CY for the recycle pond, and 3) 300,000 CY for the dry stack.

2. If the Newman property was acquired, then a second wet gypsum stack could be constructed, as shown in Figure 3 (Stack 2). The second stack could have a footprint of 138 acres, and a storage volume of 15.3 million CY. The total volume of gypsum storage of Stacks 1 & 2 would be 36.7 million CY. The quantity of fill required for the perimeter berm of Stack 2 is 800,000 CY.

3. If the Explorer pipeline was relocated to AmerenUE's right-of-way on the west side of Labadie Bottom Road as shown in Figure 4 and the Newman property was acquired, then a wet gypsum stack with a footprint of 328 acres could be constructed. The calculated volume of gypsum storage is 41.9 million CY, or 5.2 million CY greater than the volume of the two stacks shown in Figure 3. The quantity of fill required for the construction of the perimeter berm is 1,300,000 CY, or about 400,000 CY less than the fill required for the two stacks. About 8800 feet of pipeline would have to be relocated for the scheme shown in Figure 4, for an estimated cost of $2.2 million.

We are completing our feasibility study, which will include zoning requirements, floodplain, wetlands, and ground water issues. All of our data and findings, including the information in this preliminary report, will be summarized in our forthcoming report. Please contact us if you have any comments or questions which should be addressed in that report.
# Boring Log P-1

**AmerenUE Labadie Power Plant UWL**  
Franklin County, Missouri  
**CLIENT:** Ameren Services  
**LOCATION:** N  
**ELEVATION:** 471  
**DATE DRILLED:** 3-14-2007  
**DATUM:** U.S.G.S.

<table>
<thead>
<tr>
<th>DEPTH (FEET)</th>
<th>ELEVATION</th>
<th>WATER TABLE</th>
<th>GRAPHIC LOG</th>
<th>SAMPLE TYPE</th>
<th>PERCENT RECOVERY</th>
<th>MATERIAL DESCRIPTION</th>
<th>DRY UNIT WEIGHT (PCF)</th>
<th>MOISTURE CONTENT</th>
<th>DENSITY</th>
<th>STANDARD PENETRATION TEST</th>
<th>N-VALUE (BLOWS PER LAST FOOT)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>470</td>
<td></td>
<td></td>
<td>TOPSOIL (4&quot;)</td>
<td>83</td>
<td>Sandy SILT (ML), brown, loose, dry</td>
<td>3-4-3</td>
<td>18.4</td>
<td></td>
<td>△ QU2 □ PP □ SV □ TV</td>
<td>1</td>
</tr>
<tr>
<td>5</td>
<td>465</td>
<td></td>
<td></td>
<td>Becoming grayish brown, less sandy, with clay laminations, moist</td>
<td>78</td>
<td>2-2-3</td>
<td>28.5</td>
<td></td>
<td></td>
<td>△ N-VALUE (%)</td>
<td>(BLOWS PER LAST FOOT)</td>
</tr>
<tr>
<td>10</td>
<td>460</td>
<td></td>
<td></td>
<td>Becoming more sandy, medium-dense, dry</td>
<td>67</td>
<td>2-3-5</td>
<td>15.7</td>
<td></td>
<td></td>
<td>△ N-VALUE (%)</td>
<td>(BLOWS PER LAST FOOT)</td>
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<tr>
<td>15</td>
<td>455</td>
<td></td>
<td></td>
<td>Becoming tan</td>
<td>72</td>
<td>3-5-6</td>
<td>15.0</td>
<td></td>
<td></td>
<td>△ N-VALUE (%)</td>
<td>(BLOWS PER LAST FOOT)</td>
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<tr>
<td>20</td>
<td>450</td>
<td></td>
<td></td>
<td>Silty SAND (SP-SM), tan, fine, loose</td>
<td>89</td>
<td>3-3-4</td>
<td>12.4</td>
<td></td>
<td></td>
<td>△ N-VALUE (%)</td>
<td>(BLOWS PER LAST FOOT)</td>
</tr>
<tr>
<td>25</td>
<td>445</td>
<td></td>
<td></td>
<td>Becoming gray, slightly silty, very loose, with clay balls, free water</td>
<td>100</td>
<td>1-1-1</td>
<td></td>
<td></td>
<td></td>
<td>△ N-VALUE (%)</td>
<td>(BLOWS PER LAST FOOT)</td>
</tr>
<tr>
<td>30</td>
<td>440</td>
<td></td>
<td></td>
<td>SAND (SP), tan, fine, medium-dense</td>
<td>94</td>
<td>3-5-7</td>
<td></td>
<td></td>
<td></td>
<td>△ N-VALUE (%)</td>
<td>(BLOWS PER LAST FOOT)</td>
</tr>
<tr>
<td>35</td>
<td>435</td>
<td></td>
<td></td>
<td>Becoming gray, fine- to medium-grain, with decaying wood, loose</td>
<td>89</td>
<td>3-2-4</td>
<td></td>
<td></td>
<td></td>
<td>△ N-VALUE (%)</td>
<td>(BLOWS PER LAST FOOT)</td>
</tr>
</tbody>
</table>

**DRILLER:** Midwest  
**METHOD:** CFA/Mod Rotary  
**TYPE OF SPT HAMMER:** Automatic  
**HAMMER EFFICIENCY (%):**  
**LOGGED BY:** J. Pruett  

**WATER LEVELS:**  
DURING DRILLING 19 FEET  
N BORING DRY AT COMPLETION OF DRILLING  
AT FEET AFTER HOURS  
AT FEET AFTER HOURS  
**PIEZOMETER:** INSTALLED AT 30 FEET

Figure A-1   Sheet 1 of 3
<table>
<thead>
<tr>
<th>Depth (Feet)</th>
<th>Elevation</th>
<th>Material Description</th>
<th>Shear Strength, tsf</th>
</tr>
</thead>
<tbody>
<tr>
<td>35</td>
<td>435</td>
<td>Becoming tan, fine- to coarse, medium- dense</td>
<td>6-8-10</td>
</tr>
<tr>
<td>40</td>
<td>430</td>
<td>Becoming gray, fine- to medium-grain, dense, with decaying wood</td>
<td>10-15-10</td>
</tr>
<tr>
<td>45</td>
<td>425</td>
<td>Without decaying wood</td>
<td>13-21-15</td>
</tr>
<tr>
<td>50</td>
<td>420</td>
<td>SAND and GRAVEL (SP-GP), gray, dense, with clay balls and fragments of dolomite (up to 1.5&quot;)</td>
<td>9-16-7</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Very stiff drilling at 51' (heavy gravels and cobbles)</td>
<td></td>
</tr>
<tr>
<td>60</td>
<td>410</td>
<td>Becoming medium-dense, without clay and fragments of dolomite</td>
<td>12-10-8</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Very stiff drilling from 60.5' to 61.5' possible limestone cobbles or boulders</td>
<td></td>
</tr>
</tbody>
</table>

Figure A-1    Sheet 2 of 3
### BORING LOG P-1

#### AmerenUE Labadie Power Plant UWL

<table>
<thead>
<tr>
<th>Depth (Feet)</th>
<th>Elevation</th>
<th>Water Table</th>
<th>Graphic Log</th>
<th>Sample Recovery</th>
<th>Material Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>75</td>
<td>395</td>
<td></td>
<td></td>
<td></td>
<td>Very stiff drilling at 71', 73', and 77' to 78', possible cobbles or boulders</td>
</tr>
<tr>
<td>80</td>
<td>390</td>
<td>44</td>
<td></td>
<td></td>
<td>Very stiff drilling from 80' to 91.5' possible cobbles</td>
</tr>
<tr>
<td>90</td>
<td>385</td>
<td>33</td>
<td></td>
<td></td>
<td>Becoming dense</td>
</tr>
<tr>
<td>95</td>
<td>380</td>
<td></td>
<td></td>
<td></td>
<td>Boring terminated at 91'-6&quot; in cobbles.</td>
</tr>
<tr>
<td>100</td>
<td>375</td>
<td></td>
<td></td>
<td></td>
<td>Note: terminated boring due to very difficult drilling; rods were binding during advancement, near breaking point.</td>
</tr>
</tbody>
</table>

#### Shear Strength, lsf

<table>
<thead>
<tr>
<th>N-Value (Blows per Last Foot)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
</tr>
<tr>
<td>2</td>
</tr>
<tr>
<td>3</td>
</tr>
<tr>
<td>4</td>
</tr>
</tbody>
</table>

#### Moisture Content, %

<table>
<thead>
<tr>
<th>% Fines (Passing #200 Sieve)</th>
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</thead>
<tbody>
<tr>
<td>1</td>
</tr>
<tr>
<td>2</td>
</tr>
<tr>
<td>3</td>
</tr>
</tbody>
</table>

#### Dry Unit Weight (pcf)

<table>
<thead>
<tr>
<th>Rods per Foot Quality 6'</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
</tr>
<tr>
<td>2</td>
</tr>
<tr>
<td>3</td>
</tr>
</tbody>
</table>

---

Figure A-1    Sheet 3 of 3
# Boring Log P-2

AmerenUE Labadie Power Plant UWL  
Franklin County, Missouri  
CLIENT: Ameren Services

<table>
<thead>
<tr>
<th>DEPTH (FEET)</th>
<th>ELEVATION</th>
<th>WATER TABLE</th>
<th>GRAPHIC LOG</th>
<th>SAMPLE TYPE</th>
<th>PERCENT RECOVERY</th>
<th>MATERIAL DESCRIPTION</th>
<th>DRY UNIT WEIGHT (PCF)</th>
<th>BLOWs PER 6 INCHES</th>
<th>MOISTURE CONTENT</th>
<th>PERCENT BY WEIGHT</th>
<th>SHEAR STRENGTH, tsf</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>465</td>
<td></td>
<td></td>
<td></td>
<td>100</td>
<td>TOPSOIL (5&quot;)</td>
<td>3-3-6</td>
<td>27.3</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>465</td>
<td>94</td>
<td></td>
<td></td>
<td></td>
<td>100</td>
<td>CLAY (CH), dark grayish brown, high plastic, moist, stiff</td>
<td>3-3-3</td>
<td>36.4</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>460</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>100</td>
<td>Becoming firm, with seams of grayish brown silt</td>
<td>3-4-4</td>
<td>14.7</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>455</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>89</td>
<td>SILT (ML), tan, medium-dense, with fine sand, dry</td>
<td>2-1-3</td>
<td>25.9</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>450</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Becoming loose, with traces of iron stains</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>445</td>
<td>15</td>
<td></td>
<td></td>
<td></td>
<td>100</td>
<td>SAND (SP), grayish tan, fine, medium-dense</td>
<td>4-6-7</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>440</td>
<td>20</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Becoming dark gray</td>
<td>1-3-5</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>435</td>
<td>25</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Becoming fine- to medium-grain</td>
<td>2-3-5</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>30</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Becoming fine to coarse</td>
<td>7-7-8</td>
<td></td>
<td></td>
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<td></td>
</tr>
</tbody>
</table>

Boring terminated at 30'-0"

**Driller:** Midwest  
**Method:** CFA/Mud Rotary  
**Type of SPT Hammer:** Automatic  
**Hammer Efficiency (%):**  
**Logged by:** J. Pruett  
**Water Levels:**  
**During Drilling:** 14 FEET  
**Piezometer:** Installed at 30 FEET

---

Figure A-2 Sheet 1 of 1
## Boring Log P-3

### AmerenUE Labadie Power Plant UWL
Franklin County, Missouri

**CLIENT:** Ameren Services

**LOCATION:** N   **ELEVATION:** 467   **DATE DRILLED:** 3-12-2007

**DATUM:** U.S.G.S.

### Material Description

<table>
<thead>
<tr>
<th>Depth (Feet)</th>
<th>Elevation</th>
<th>Graphic Log</th>
<th>Sample Log Type</th>
<th>Percent Recovery</th>
<th>Dry Unit Weight (pcf)</th>
<th>Standard Penetration Test</th>
<th>Moisture Content (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>465</td>
<td></td>
<td></td>
<td></td>
<td>2-3-4</td>
<td>▲</td>
<td>24.5</td>
</tr>
<tr>
<td>5</td>
<td>460</td>
<td></td>
<td></td>
<td></td>
<td>1-3-1</td>
<td>▲</td>
<td>26.6</td>
</tr>
<tr>
<td>10</td>
<td>455</td>
<td></td>
<td></td>
<td></td>
<td>1-1-4</td>
<td>▲</td>
<td>29.9</td>
</tr>
<tr>
<td>15</td>
<td>450</td>
<td></td>
<td></td>
<td></td>
<td>2-3-4</td>
<td>▲</td>
<td>31.4</td>
</tr>
<tr>
<td>20</td>
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<td>1-1-4</td>
<td>▲</td>
<td>29.9</td>
</tr>
<tr>
<td>25</td>
<td>440</td>
<td></td>
<td></td>
<td></td>
<td>2-5-8</td>
<td>▲</td>
<td>26.6</td>
</tr>
<tr>
<td>30</td>
<td>435</td>
<td></td>
<td></td>
<td></td>
<td>5-7-8</td>
<td>▲</td>
<td>29.9</td>
</tr>
<tr>
<td>35</td>
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<td></td>
<td></td>
<td></td>
<td>6-7-7</td>
<td>▲</td>
<td>31.4</td>
</tr>
</tbody>
</table>

**Topsoil (5")**
- CLAY (CH), dark gray, high plastic, with silt seams, moist, stiff
- Silt (ML), grayish tan, with fine sand
- Silty SAND (SP-SM), grayish tan, fine, loose
- With lenses of gray high plastic clay
- With alternating layers of fine silty sand and high plastic clay
- SAND (SP), gray, fine, loose
- Becoming medium-dense

### Water Levels

**DURING DRILLING:** 13.5 FEET

**N BORING DRY AT COMPLETION OF DRILLING**

**AT FEET AFTER HOURS**

**AT FEET AFTER HOURS**

**PIEZOMETER:** INSTALLED AT 30 FEET

---

**DRILLER:** Midwest
**METHOD:** CFA/Mud Rotary
**TYPE OF SPT HAMMER:** Automatic
**HAMMER EFFICIENCY (%):**

**LOGGED BY:** J. Pruett

---

**Figure A-3**  **Sheet 1 of 1**
MATERIAL DESCRIPTION

TOPSOIL (3")
Sandy Silt (ML-SM), tan, loose, wet CLAY (CH), grayish brown, high plastic, moist, soft

Becoming dry and firm

Sandy Silt (ML-SM), tan, medium-dense, moist

SAND (SP), tan, fine, medium-dense with organic clay balls

Becoming tan and gray, fine- to medium-grain, loose

Becoming medium-dense

Boring terminated at 25'-0"

DRILLER: Midwest
METHOD: CFA
TYPE OF SPT HAMMER: Automatic
HAMMER EFFICIENCY (%):
LOGGED BY: J. Pruett

WATER LEVELS:
DURING DRILLING 18.5 FEET
N BORING DRY AT COMPLETION OF DRILLING
AT ______ FEET AFTER ______ HOURS
AT ______ FEET AFTER ______ HOURS
PIEZOMETER: INSTALLED AT ______ FEET
BORING LOG B-5

AmerenUE Labadie Power Plant UWL
Franklin County, Missouri
CLIENT: Ameren Services

LOCATION: N
ELEVATION: 467
DATUM: U.S.G.S.
DATE DRILLED: 3-12-2007

MATERIAL DESCRIPTION

DEPTH (FEET)   ELEVATION   WATER TABLE   GRAPHIC LOG   SAMPLE TYPE   PERCENT RECOVERY   DRY UNIT WEIGHT (pCF)   BLOWING CDR & INCHES   ROCK QUALITY DS.   MOISTURE CONTENT PERCENT BY WEIGHT   SHEAR STRENGTH, tsf

0     465
100

TOPSOIL (4"
Silty CLAY (CL-CH), brown, moderately plastic, stiff, dry

460
78

CLAY (CH), grayish brown, high plastic, stiff, moist

94

With tan silty fine sand laminations

83

SAND (SP), tan, fine, medium-dense with sporatic clay balls

10

Becoming grayish tan, without clay balls

15

72

455

15

89

450

100

445

100

25

440

Boring terminated at 25'-0"

20

40

60

1

2

3

△ QUIZ □ PP □ SV □ TV

STANDARD PENETRATION TEST
N-VALUE (BLOWNS PER LAST FOOT)
○ MOISTURE CONTENT, %
○ % FINES (PASSING #200 SIEVE)

PL LL

DRILLER: Midwest
METHOD: CFA
TYPE OF SPT HAMMER: Automatic
HAMMER EFFICIENCY (%): LOGGED BY: J. Pruett

WATER LEVELS: DURING DRILLING 16.5 FEET
N BOARING DRY AT COMPLETION OF DRILLING
AT _ FEET AFTER _ HOURS
AT _ FEET AFTER _ HOURS
PIEZOMETER: INSTALLED AT _ FEET
BORING LOG B-6

AmerenUE Labadie Power Plant UWL
Franklin County, Missouri
CLIENT: Ameren Services

LOCATION: N
ELEVATION: 467
DATE DRILLED: 3-12-2007

DATA DRILLED: 3-12-2007

MATERIAL DESCRIPTION

<table>
<thead>
<tr>
<th>DEPTH (FEET)</th>
<th>ELEVATION</th>
<th>WATER TABLE</th>
<th>GRAPHIC LOG</th>
<th>SAMPLE RECOVERY</th>
<th>DRY UNIT WEIGHT (SG)</th>
<th>BLOWS PER 6 INCHES</th>
<th>MOISTURE CONTENT PERCENT RECOVERED</th>
<th>SOIL QUALITY INDEX</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>465</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>2-3-3</td>
<td>22.4</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>460</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>3-4-4</td>
<td>23.0</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>455</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>3-5-9</td>
<td>16.6</td>
<td></td>
</tr>
<tr>
<td>15</td>
<td>450</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>6-6-5</td>
<td>11.6</td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>445</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>3-4-5</td>
<td>12.0</td>
<td></td>
</tr>
<tr>
<td>25</td>
<td>440</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1-2-3</td>
<td>13.0</td>
<td></td>
</tr>
<tr>
<td>30</td>
<td>435</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>12.0</td>
<td>13.0</td>
<td></td>
</tr>
</tbody>
</table>

MOISTURE CONTENT, %

PL

MOISTURE CONTENT, %

% FINES (PASSING #200 SIEVE)

SHEAR STRENGTH, t/ft

△ QU/2 □ PP □ SV □ TS

STANDARD PENETRATION TEST

N-VALUE (BLOWS PER LAST FOOT)

1

2

3

PL

WATER LEVELS:

DURING DRILLING 16 FEET

N

BORING DRY AT COMPLETION OF DRILLING

AT FEET AFTER HOURS

AT FEET AFTER HOURS

PIEZOMETER:

INSTALLED AT FEET

File: 20070130A001

Figure A-6 Sheet 1 of 1
<table>
<thead>
<tr>
<th>Depth (Feet)</th>
<th>Material Description</th>
<th>Dry Unit Weight (pcf)</th>
<th>Percent Recovery</th>
<th>Moisture Content, %</th>
<th>% Fines (Passing #200 SIEVE)</th>
<th>Shear Strength, t/sf</th>
</tr>
</thead>
<tbody>
<tr>
<td>35 - 430</td>
<td>Becoming dense with coarse gravel (up to 1&quot;)</td>
<td>17-15-13</td>
<td>78</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>40 - 425</td>
<td>SAND (SP), gray, fine, very dense</td>
<td>10-17-24</td>
<td>100</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>45 - 420</td>
<td></td>
<td></td>
<td>100</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>50 - 415</td>
<td></td>
<td></td>
<td>89</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>60 - 405</td>
<td>SAND &amp; GRAVEL (SP-GP), gray, coarse</td>
<td>9-13-12</td>
<td>94</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>sand and fine gravel, dense</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>70 - 395</td>
<td>With medium gravel (to 3/8&quot;)</td>
<td>11-11-14</td>
<td>100</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
### Boring Log B-7

**AmerenUE Labadie Power Plant UWL**

<table>
<thead>
<tr>
<th>Depth (Feet)</th>
<th>Elevation</th>
<th>Material Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>75</td>
<td>390</td>
<td>Becoming medium-dense</td>
</tr>
<tr>
<td>80</td>
<td>385</td>
<td>Becoming very dense, with black shale fragments</td>
</tr>
<tr>
<td>90</td>
<td>375</td>
<td>Becoming medium-dense</td>
</tr>
<tr>
<td>100</td>
<td>365</td>
<td>LIMESTONE, very weathered, soft</td>
</tr>
<tr>
<td>105</td>
<td>360</td>
<td>Auger Refusal at 104'-6&quot;</td>
</tr>
</tbody>
</table>

**Shear Strength, tps**

- **N-Value (Blows per Last Foot)**
- **Moisture Content, %**
- **% Fines (Passing #200 Sieve)**

**Sonic Velocity**

<table>
<thead>
<tr>
<th>Depth (Feet)</th>
<th>Velocity (ft/sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>75</td>
<td>20</td>
</tr>
<tr>
<td>80</td>
<td>40</td>
</tr>
<tr>
<td>90</td>
<td>60</td>
</tr>
</tbody>
</table>

**Sample Recovery**

- **Percent Recovery**
  - 100
  - 89
  - 78

**Sample Type**

- **WATER TABLE**
- **GRAPHIC LOG**

---

Date: 2007/11/24

Figure Sheet 3 of 3
# Boring Log B-8

**AmerenUE Labadie Power Plant UWL**  
Franklin County, Missouri

**CLIENT**: Ameren Services  
**LOCATION**: N  
**ELEVATION**: 468  
**DATE DRILLED**: 3-9-2007  
**DATUM**: U.S.G.S.

<table>
<thead>
<tr>
<th>DEPTH (FEET)</th>
<th>ELEVATION</th>
<th>WATER TABLE</th>
<th>GRAPHIC LOG</th>
<th>SAMPLE TYPE</th>
<th>PERCENT RECOVERY</th>
<th>MATERIAL DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>465</td>
<td></td>
<td>83</td>
<td></td>
<td></td>
<td>TOPSOIL (4&quot;)</td>
</tr>
<tr>
<td>5</td>
<td>460</td>
<td></td>
<td>67</td>
<td></td>
<td></td>
<td>Sandy SILT (ML-SM), gray and tan, loose, with seams of grayish brown high plastic clay, dry</td>
</tr>
<tr>
<td>10</td>
<td>455</td>
<td></td>
<td>89</td>
<td></td>
<td></td>
<td>Becoming medium-dense, without clay seams, with fine sand laminations</td>
</tr>
<tr>
<td>15</td>
<td>450</td>
<td></td>
<td>94</td>
<td></td>
<td></td>
<td>SAND (SP), tan, fine, medium-dense, with sporatic clay balls</td>
</tr>
<tr>
<td>20</td>
<td>445</td>
<td></td>
<td>78</td>
<td></td>
<td></td>
<td>Becoming grayish brown, loose</td>
</tr>
<tr>
<td>25</td>
<td>440</td>
<td></td>
<td>94</td>
<td></td>
<td></td>
<td>Becoming very loose</td>
</tr>
<tr>
<td>30</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Boring terminated at 20'-0&quot;</td>
</tr>
</tbody>
</table>

**DRILLER**: Midwest  
**METHOD**: CFA  
**TYPE OF SPT HAMMER**: Automatic  
**HAMMER EFFICIENCY (%)**:  
**LOGGED BY**: J. Pruett

<table>
<thead>
<tr>
<th>DEPTH (FEET)</th>
<th>ELEVATION</th>
<th>WATER TABLE</th>
<th>GRAPHIC LOG</th>
<th>SAMPLE TYPE</th>
<th>PERCENT RECOVERY</th>
<th>MATERIAL DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>465</td>
<td></td>
<td>83</td>
<td></td>
<td></td>
<td>TOPSOIL (4&quot;)</td>
</tr>
<tr>
<td>5</td>
<td>460</td>
<td></td>
<td>67</td>
<td></td>
<td></td>
<td>Sandy SILT (ML-SM), gray and tan, loose, with seams of grayish brown high plastic clay, dry</td>
</tr>
<tr>
<td>10</td>
<td>455</td>
<td></td>
<td>89</td>
<td></td>
<td></td>
<td>Becoming medium-dense, without clay seams, with fine sand laminations</td>
</tr>
<tr>
<td>15</td>
<td>450</td>
<td></td>
<td>94</td>
<td></td>
<td></td>
<td>SAND (SP), tan, fine, medium-dense, with sporatic clay balls</td>
</tr>
<tr>
<td>20</td>
<td>445</td>
<td></td>
<td>78</td>
<td></td>
<td></td>
<td>Becoming grayish brown, loose</td>
</tr>
<tr>
<td>25</td>
<td>440</td>
<td></td>
<td>94</td>
<td></td>
<td></td>
<td>Becoming very loose</td>
</tr>
<tr>
<td>30</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Boring terminated at 20'-0&quot;</td>
</tr>
</tbody>
</table>

**DRILLER**: Midwest  
**METHOD**: CFA  
**TYPE OF SPT HAMMER**: Automatic  
**HAMMER EFFICIENCY (%)**:  
**LOGGED BY**: J. Pruett

**WATER LEVELS**: During Drilling 17.5 Feet

**PIEZOMETER**: Installed at ____ Feet

**Figure A-8** Sheet 1 of 1
### KEY TO BORING LOGS

**KEY TO SOIL SYMBOLS**

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td><strong>TOPSOIL</strong></td>
</tr>
<tr>
<td></td>
<td>Topsoil</td>
</tr>
<tr>
<td></td>
<td>Silty SAND or Sandy SILT (SM)</td>
</tr>
<tr>
<td></td>
<td>Poorly-graded SAND (SP)</td>
</tr>
<tr>
<td></td>
<td>Poorly-graded SAND &amp; GRAVEL (GP)</td>
</tr>
<tr>
<td></td>
<td>High plastic CLAY (CH)</td>
</tr>
<tr>
<td></td>
<td>Inorganic, non-plastic SILT (ML)</td>
</tr>
<tr>
<td></td>
<td>Medium to high plastic CLAY</td>
</tr>
<tr>
<td></td>
<td>Low plastic SILT CLAY (CL)</td>
</tr>
<tr>
<td></td>
<td>Very Weathered LIMESTONE</td>
</tr>
</tbody>
</table>

**SOIL SAMPLERS**

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>2-in. O.D. Split-Spoon</td>
</tr>
<tr>
<td></td>
<td>3-in. O.D. Shelby Tube</td>
</tr>
</tbody>
</table>

**MISCELLANEOUS SYMBOLS**

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Water table during drilling</td>
</tr>
<tr>
<td></td>
<td>Boring continues</td>
</tr>
</tbody>
</table>

### Notes:

1. Borings were drilled March 9 - 14, 2007, by Midwest Drilling, Inc. The borings were advanced using continuous flight augers (CFA) to below the water table, and then with mud rotary drilling techniques using Bentonite slurry.

2. Boring locations were selected and located by Reitz and Jens, Inc.

3. Borings were logged in the field by a Reitz & Jens' soils technician based upon the recovered samples, cuttings and drilling characteristics. Samples were transported to Reitz & Jens' lab for testing. Field logs were revised, if needed, based upon laboratory classification and testing.

4. Stratification lines shown on the log represent approximate soil boundaries; actual changes in strata may be gradual or occur between samples.

5. Piezometers were installed in Borings labeled P-1, P-2, and P-3.

Figure A-0
APPENDIX 5

Well Log Record Data
<table>
<thead>
<tr>
<th>log no. 24461</th>
<th>owner J. Kopasky</th>
</tr>
</thead>
<tbody>
<tr>
<td>county Franklin</td>
<td>farm</td>
</tr>
<tr>
<td>driller Shadami</td>
<td>date 6-8-66</td>
</tr>
<tr>
<td>elev.lund</td>
<td>prod. 591</td>
</tr>
<tr>
<td>44 m</td>
<td>2 e</td>
</tr>
<tr>
<td>logged by K. Anderson</td>
<td></td>
</tr>
</tbody>
</table>

- Remarks: 57' of 2 1/4" crq.
| REF NO | 00066803 |
| DATE RECEIVED | 11/14/1991 |
| CR NO | |
| STATE CERT NO | |
| APPROVED DATE | |
| A023115 |
| CHECK NO | 26228 |
| DATE ENTERED | |
| PHASE 1 PHASE 2 PHASE 3 | |
| 11/21/1991 01/01/1000 12/28/2005 |
| ROUTE | PLT / PCD |
| REVENUE NO | 661637 |

**INFORMATION SUPPLIED BY WELL OR PUMP INSTALLATION CONTRACTOR**

- OWNER NAME: ST ALBANS COUNTRY CLUB
- TELEPHONE (OPTIONAL): 000-742-4654
- OWNER ADDRESS: ST ALBANS RD
- CITY: ST ALBANS
- STATE: MO
- ZIP: 63073

**PROPOSED USE OF WELL**

- Water Supply for Irrigation (capable of producing more than 70 gpm to surface)
- Unconsolidated Material Well
- Water Supply for a High-Capacity Well capable of producing more than 70 gpm to surface - get casing depth from GSRAD before start
- Open Loop Heat Pump
- Supply Well
- Return Well
- Water Supply to a Public Facility (convenience store, restaurant, church, business, condo, mobile home park, rural or urban water supply)

**CONTACT THE DNR REGIONAL OFFICE TO GET INSTRUCTIONS FOR WATER SUPPLY TO A PUBLIC FACILITY**

**CASING DETAILS**

<table>
<thead>
<tr>
<th>CASING LENGTH</th>
<th>O.D. OF CASING</th>
<th>DIAMETER OF DRILL HOLE</th>
</tr>
</thead>
<tbody>
<tr>
<td>16.0' FT.</td>
<td>12.0' IN.</td>
<td></td>
</tr>
<tr>
<td>32.0' IN.</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

- POSITION OF GROUT SEAL: BOTTOM
- FULL LENGTH:
- TOP:
- CASING MATERIAL:
  - X STEEL
  - PLASTIC
  - CONCRETE
- CASING GROUT MATERIAL:
  - CEMENT:
    - TYPE 1: SLURRY
    - Hi-EARLY:
    - CHIPS
    - GRANULAR
    - PELLETS
  - BENTONITE:
    - SLURRY
    - CHIPS
    - GRANULAR
    - PELLETS
- METHOD OF GROUT INSTALLATION:
  - OPEN HOLE
  - TRENIE
- PRESSURE GROUT:
  - THROUGH CASING
  - THROUGH TRENIE
- DRILLING SUSPENDED:
  - NO
  - YES 0 HRS

**LINER DETAILS**

<table>
<thead>
<tr>
<th>LENGTH</th>
<th>O.D. OF LINER</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0' FT.</td>
<td>0.0' IN.</td>
</tr>
</tbody>
</table>

- LINER MATERIAL:
  - STEEL
  - PLASTIC
- POSITION OF SEAL:
  - FULL LENGTH
  - BOTTOM
  - TOP
- LINER GROUT MATERIAL:
  - CEMENT:
    - TYPE 1: SLURRY
    - Hi-EARLY:
    - CHIPS
    - GRANULAR
    - PELLETS
  - BENTONITE:
    - SLURRY
    - CHIPS
    - GRANULAR
    - PELLETS
- METHOD OF GROUT INSTALLATION:
  - OPEN HOLE
  - TRENIE
- PRESSURE GROUT:
  - THROUGH CASING
  - THROUGH TRENIE
- LINER USED TO:
  - HOLD BACK FORMATION
  - SEAL OUT UNSIREEABLE AQUIFER CONDITIONS
  - PREVENT RUST
  - ABANDONED WELL ON SITE?
    - X YES
    - PLUGGED?
    - YES

**LOCATION OF WELL**

- DEPTH TO FIRST GROUNDWATER: 0.0 FEET
- PUMP RATE: 1000.0 GPM
- DEPTH TO BEDROCK: 0.0 FEET
- WHOLE COMPLETION DATE: 06/19/1991
- PUMP INSTALLATION DATE:
- PUMP INST. DEPTH: 01.0 FEET
- PUMP DET. DEPTH: 01.0 FEET
- PUMP INSTALLATION DATE:
- PUMP INST. DEPTH: 01.0 FEET
- PUMP DET. DEPTH: 01.0 FEET
- PUMP INSTALLATION DATE:
- LOCATION OF WELL:
  - LATT: 38° 30' 28.2"
  - LONG: 90° 46' 54.2"
  - COUNTY: FRANKLIN

**OTHER INFORMATION OR LOCATION DATA (OPTIONAL)**

- DEPTH FROM TO:
- FORMATION DESCRIPTION:
- ELEVATION:
- LEGAL LOCATION:
- AREA:
- OTHER INFORMATION:

**I HEREBY CERTIFY THE WELL/PUMP INFORMATION DESCRIBED HEREIN IS TRUE AND ACCURATE**

<table>
<thead>
<tr>
<th>PRIMARY CONTRACTOR SIGNATURE</th>
<th>PERMIT NUMBER</th>
<th>DATE</th>
</tr>
</thead>
<tbody>
<tr>
<td>DANIEL FLYNN</td>
<td>001120</td>
<td></td>
</tr>
<tr>
<td>WELL DRILLER SIGNATURE</td>
<td>PERMIT NUMBER</td>
<td>DATE</td>
</tr>
<tr>
<td>DANIEL FLYNN</td>
<td>001120</td>
<td></td>
</tr>
<tr>
<td>PUMP INSTALLER SIGNATURE</td>
<td>PERMIT NUMBER</td>
<td>DATE</td>
</tr>
<tr>
<td>DANIEL FLYNN</td>
<td>001120</td>
<td></td>
</tr>
<tr>
<td>APPRENTICE DRILLER SIGNATURE</td>
<td>PERMIT NUMBER</td>
<td>DATE</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>APPRENTICE PUMP SIGNATURE</td>
<td>PERMIT NUMBER</td>
<td>DATE</td>
</tr>
</tbody>
</table>
MISSOURI DEPARTMENT OF NATURAL RESOURCES
DIVISION OF ENVIRONMENTAL QUALITY
(573) 368-2165
HIGH YIELD AND PUBLIC WELL RECORD AND PUMP INFORMATION DATA

INFORMATION SUPPLIED BY WELL OR PUMP INSTALLATION CONTRACTOR
OWNER NAME
CAPITOL SAND
TELEPHONE (OPTIONAL)
673-634-3020
DNR VARIANCE NUMBER

OWNER ADDRESS
700 MOKANE RD BOX 104990
CITY
JEFFERSON CITY
STATE
MO
ZIP
65109
ADDRESS OF WELL (DIFFERENT THAN ABOVE)
CITY
STATE
ZIP

PROPOSED USE OF WELL
SEE BACK OF FORM FOR WELL CLASSIFICATIONS

- [X] Water Supply for Irrigation (capable of producing more than 70 gpm to surface)
- Unconsolidated Material Well
- Bedrock Well
- Water Supply for a High-Capacity Well capable of producing more than 70 gpm to surface - get casing depth from GSRAD before start
- Open Loop Heat Pump
- Supply Well
- Return Well
- Water Supply to a Public Facility (convenience store, restaurant, church, business, condo, mobile home park, rural or urban water supply)

CONTACT THE DNR REGIONAL OFFICE to get instructions for water supply to a PUBLIC FACILITY

CASING DETAILS
CASING LENGTH
DIAMETER OF CASING
O.D. OF CASING
36.0 IN.
DIA. OF CASING
30.0 IN.
POSITION OF GROUT SEAL
BOTTOM
FULL LENGTH
PLASTIC

CASING GROUT MATERIAL
CEMENT
BENTONITE
HI-EARLY
CHIPS
GRANULAR
PELLETS
METHOD OF GROUT INSTALLATION
OPEN HOLE
TREME
PRESSURE GROUT
THROUGH CASING
DRILLING SUSPENDED

NO. OF SACKS USED 0.0
POUNDS PER SACK 84

LINER DETAILS
LENGTH
O.D. OF LINER
6.0 FT.
LINER MATERIAL
STEEL
PLASTIC
POSITION OF SEAL
FULL LENGTH
BOTTOM

LINER GROUT MATERIAL
CEMENT
BENTONITE
HI-EARLY
CHIPS
GRANULAR
PELLETS
METHOD OF GROUT INSTALLATION
GRAVITY
POS. DISPLACEMENT
TREME
LINER USED TO:
HOLD BACK FORMATION
SEAL OUT UNDESIRABLE AQUIFER CONDITIONS
PREVENT RUST

NO. OF SACKS USED 0.0
POUNDS PER SACK

LOCATION OF WELL
LAT.
0 0 60
LONG.
0 0 60
COUNTY
FRANKLIN

Please be aware that we do not guarantee the accuracy of the data. It is submitted to us by a third party and has not been filed verified.

DEPT. FROM TO FORMATION DESCRIPTION
0 3 FILL
3 7 SILT
7 70 CRSE SND:SH @ 70

DEPTH TO FIRST GROUNDWATER
DEPT. TO BEDROCK
TOTAL DEPTH
0.0 FEET
70.0 FEET

PUMP RATE
1800.0 GPM
WELL YIELD
2000.0 GPM
STATIC WATER LEVEL
10.0 FEET
WELL COMPLETION DATE
03/20/2004

OTHER INFORMATION OR LOCATION DATA (OPTIONAL)

I HEREBY CERTIFY THE WELL/PUMP INFORMATION DESCRIBED HEREIN IS TRUE AND ACCURATE

PRIMARY CONTRACTOR SIGNATURE
GARY SISK
PERMIT NUMBER
000032
DATE

WELL DRILLER SIGNATURE
GARY SISK
PERMIT NUMBER
000032
DATE

PUMP INSTALLER SIGNATURE
GARY SISK
PERMIT NUMBER
000032
DATE

APPRENTICE DRILLER SIGNATURE
PERMIT NUMBER
DATE
APPRENTICE PUMP SIGNATURE
PERMIT NUMBER
DATE
MISSOURI DEPARTMENT OF NATURAL RESOURCES
DIVISION OF ENVIRONMENTAL QUALITY
(573) 368-2165
HIGH YIELD AND PUBLIC WELL RECORD
AND PUMP INFORMATION DATA

REF NO 00236293 DATE RECEIVED 05/23/2000
CR NO
STATE CERT NO APPROVED DATE
A080861 07/25/2000 CHECK NO. 39457
DATE ENTERED PHASE 1 PHASE 2 PHASE 3
05/31/2000 01/01/2000 07/25/2000 ROUTE PCD
REVENUE NO. 052400

INFORMATION SUPPLIED BY WELL OR PUMP INSTALLATION CONTRACTOR
OWNER NAME CHARLES WEDEMIER
TELEPHONE (OPTIONAL)
DNR VARIANCE NUMBER
CASING DEPTH NUMBER
Applicable only if casing depth or variance were obtained from DNR

OWNER ADDRESS
1566 FIDDLE RD
CITY LABADIE
STATE MO
ZIP 63055

ADDRESS OF WELL (IF DIFFERENT THAN ABOVE)
CITY
STATE MO
ZIP

PROPOSED USE OF WELL SEE BACK OF FORM FOR WELL CLASSIFICATIONS
[X] Water Supply for Irrigation (capable of producing more than 70 gpm to surface)
[ ] Unconsolidated Material Well
[ ] Bedrock Well
[ ] Water Supply for a High-Capacity Well capable of producing more than 70 gpm to surface - get casing depth from GSRAD before start
[ ] Open Loop Heat Pump
[ ] Water Supply to a Public Facility (convenience store, restaurant, church, business, condo, mobile home park, rural or urban water supply)

CONTACT THE DNR REGIONAL OFFICE to get instructions for water supply to a PUBLIC FACILITY

CASING DETAILS
CASING LENGTH O.D. OF CASING DIAMETER OF DRILL HOLE
62 FT. 6.0 IN. 10.63 IN.
POSITION OF GROUT SEAL [ ] BOTTOM [ ] FULL LENGTH [ ] TOP CASING MATERIAL [ ] STEEL [ ] PLASTIC [ ] CONCRETE

CASING GROUT MATERIAL
[ ] CEMENT [ ] BENTONITE [ ] CHIPS [ ] GRANULAR [ ] PELLETS
[ ] Hi-EARLY [ ] BURNT [ ] GRANULAR [ ] PELLETS
METHOD OF GROUT INSTALLATION [ ] POS. DISPLACEMENT [ ] TOTALLY DISPLACEMENT PRESSURE GROUT THROUGH TREMIE [ ] NO
[ ] DRILLING SUSPENDED [ ] PIVOT DRILLING [ ] OTHER

NO. OF SACKS USED 110 POUNDS PER SACK 50

LINER DETAILS
LENGTH O.D. OF LINER LINER MATERIAL POSITION OF SEAL [ ] FULL LENGTH [ ] BOTTOM [ ] TOP
0 FT. 0.0 IN. [ ] STEEL [ ] PLASTIC

LINER GROUT MATERIAL
[ ] CEMENT [ ] BENTONITE [ ] CHIPS [ ] GRANULAR [ ] PELLETS
[ ] Hi-EARLY [ ] BURNT [ ] GRANULAR [ ] PELLETS
METHOD OF GROUT INSTALLATION [ ] POS. DISPLACEMENT [ ] TOTALLY DISPLACEMENT LINER USED TO:
[ ] HOLD BACK FORMATION [ ] SEAL OUT UNDESIRABLE AQUIFER CONDITIONS
[ ] PREVENT RUST [ ] OTHER

NO. OF SACKS USED 0.0 POUNDS PER SACK

LOCATION OF WELL
LAT. 38° 12.4' LONG. 90° 46.4'
COUNTY FRANKLIN

Please be aware that we do not guarantee the accuracy of the data. It is submitted to us by a third party and has not been field verified.

DEPT TO FIRST GROUNDWATER 0.0 FEET PUMP RATE 20.0 GPM
WELL YIELD 75.0 GPM PUMP SET DEPTH 43.0 FEET
STATIC WATER LEVEL 24.0 FEET PUMP INSTALLATION DATE
WELL COMPLETION DATE 05/18/2000 pump info required this record or on pump card

DEPT FROM TO FORMATION DESCRIPTION (OPTIONAL) LEGAL LOCATION (OPTIONAL) AREA A1

520 FT. SEC. 22 TWN. 44 RNG. 2 E C DATA REQ'D

OTHER INFORMATION OR LOCATION DATA (OPTIONAL)

I HEREBY CERTIFY THE WELL/PUMP INFORMATION DESCRIBED HEREIN IS TRUE AND ACCURATE

PRIMARY CONTRACTOR SIGNATURE PERMIT NUMBER DATE

WELL DRILLER SIGNATURE PERMIT NUMBER DATE

RON HEATH 001587

PUMP INSTALLER SIGNATURE PERMIT NUMBER DATE

JAMES WEBER 001049

APPRENTICE DRILLER SIGNATURE PERMIT NUMBER DATE

APPRENTICE PUMP SIGNATURE PERMIT NUMBER DATE

DEPTH TO BEDROCK 0.0 FEET
TOTAL DEPTH 62.0 FEET

COPY
APPENDIX 6

Standard for Piezocone Penetration Testing of Soils
ASTM D 5778-07
Standard Test Method for 
Electronic Friction Cone and Piezocone Penetration Testing 
of Soils

This standard is issued under the fixed designation D 5778; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (e) indicates an editorial change since the last revision or reapproval.

1. Scope

1.1 This test method covers the procedure for determining the point resistance during penetration of a conical-shaped penetrometer as it is advanced into subsurface soils at a steady rate.

1.2 This test method is also used to determine the frictional resistance of a cylindrical sleeve located behind the conical point as it is advanced through subsurface soils at a steady rate.

1.3 This test method applies to friction-cone penetrometers of the electric and electronic type. Field tests using mechanical-type penetrometers are covered elsewhere by Test Method D 3441.

1.4 This test method can be used to determine porewater pressures developed during the penetration, thus termed piezocone. Porewater pressure dissipation, after a push, can also be monitored for correlation to time rate of consolidation and permeability.

1.5 Additional sensors, such as inclinometer, seismic geophones, resistivity, electrical conductivity, dielectric, and temperature sensors, may be included in the penetrometer to provide useful information. The use of an inclinometer is highly recommended since it will provide information on potentially damaging situations during the sounding process.

1.6 Cone penetration test data can be used to interpret subsurface stratigraphy, and through use of site specific correlations, they can provide data on engineering properties of soils intended for use in design and construction of earthworks and foundations for structures.

1.7 The values stated in SI units are to be regarded as standard. Within Section 13 on Calculations, SI units are considered the standard. Other commonly used units such as the inch-pound system are shown in brackets. The various data reported should be displayed in mutually compatible units as agreed to by the client or user. Cone tip projected area is commonly referred to in square centimetres for convenience. The values stated in each system are not equivalents; therefore, each system must be used independently of the other.

Norm 1—This test method does not include hydraulic or pneumatic penetrometers. However, many of the procedural requirements herein could apply to those penetrometers. Also, offshore/marine CPT systems may have procedural differences because of the difficulties of testing in those environments (for example, tidal variations, salt water, waves). Mechanical CPT systems are covered under Test Method D 3441.

1.8 This standard does not purport to address all of the safety concerns, if any, associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.

2. Referenced Documents

2.1 ASTM Standards:

2.1.1 D 653 Terminology Relating to Soil, Rock, and Contained Fluids

2.1.2 D 3441 Test Method for Mechanical Cone Penetration Tests of Soil

2.1.3 D 3740 Practice for Minimum Requirements for Agencies Engaged in the Testing and/or Inspection of Soil and Rock as Used in Engineering Design and Construction

2.1.4 E 4 Practices for Force Verification of Testing Machines

3. Terminology

3.1 Definitions:

3.1.1 Definitions are in accordance with Terminology Convention (D 653).

3.2 Definitions of Terms Specific to This Standard:

3.2.1 apparent load transfer—apparent resistance measured on either the cone or friction sleeve of an electronic cone penetrometer while that element is in a no-load condition but the other element is loaded. Apparent load transfer is the sum of cross talk, subtraction error, and mechanical load transfer.

3.2.2 baseline—a set of zero load readings, expressed in terms of apparent resistance, that are used as reference values during performance of testing and calibration.

3.2.3 cone tip—the conical point of a cone penetrometer on which the end bearing component of penetration resistance is developed. The cone has a 60° apex angle, a diameter of 35.7
mm, and a corresponding projected (horizontal plane) surface area or cone base area of 10 cm². Also, enlarged cones of 43.7 mm diameter (base area = 15 cm²) are utilized.

3.2.4 cone penetration test—a series of penetration readings performed at one location over the entire vertical depth when using a cone penetrometer. Also referred to as a cone sounding.

3.2.5 cone penetrometer—a penetrometer in which the leading end of the penetrometer tip is a conical point designed for penetrating soil and for measuring the end-bearing component of penetration resistance.

3.2.6 cone resistance, qc—the measured end-bearing component of penetration resistance. The resistance to penetration developed on the cone is equal to the vertical force applied to the cone divided by the cone base area.

3.2.7 corrected total cone resistance, qt—tip resistance corrected for water pressure acting behind the tip (see 13.2.1). Correction for water pressure requires measuring water pressures with a piezoecone element positioned behind the tip at location u₂. (See section 3.2.6). The correction results in estimated total tip resistance, qₜ.

3.2.8 cross talk—an apparent load transfer between the cone and the friction sleeve caused by interference between the separate signal channels.

3.2.9 electronic cone penetrometer—a friction cone penetrometer that uses force transducers, such as strain gauge load cells, built into a non-telescoping penetrometer tip for measuring, within the penetrometer tip, the components of penetration resistance.

3.2.10 electronic piezoecone penetrometer—an electronic cone penetrometer equipped with a low volume fluid chamber, porous element, and pressure transducer for determination of porewater pressure at the porous element soil interface.

3.2.11 end bearing resistance—same as cone resistance or tip resistance, qc.

3.2.12 equilibrium pore water pressure, u₀—at rest water pressure at depth of interest. Same as hydrostatic pressure (see Terminology D 653).

3.2.13 excess pore water pressure, Δu—the difference between porewater pressure measured as the penetration occurs (u), and estimated equilibrium porewater pressure (u₀), or: Δu = (u - u₀). Excess porewater pressure can either be positive or negative for shoulder position filters.

3.2.14 friction cone penetrometer—a cone penetrometer with the capability of measuring the friction component of penetration resistance.

3.2.15 friction ratio, Rf—the ratio of friction sleeve resistance, fₛ, to cone resistance, qc, measured at where the middle of the friction sleeve and cone point are at the same depth, expressed as a percentage.

3.2.16 friction reducer—a narrow local protruberance on the outside of the push rod surface, placed at a certain distance above the penetrometer tip, that is provided to reduce the total side friction on the push rods and allow for greater penetration depths for a given push capacity.

3.2.17 friction sleeve—an isolated cylindrical sleeve section on a penetrometer tip upon which the friction component of penetration resistance develops. The friction sleeve has a surface area of 150 cm² for 10-cm² cone tips or 225 cm² for 15-cm² tips.

3.2.18 friction sleeve resistance, fₛ—the friction component of penetration resistance developed on a friction sleeve, equal to the shear force applied to the friction sleeve divided by its surface area.

3.2.19 FSO—abbreviation for full-scale output. The output of an electronic force transducer when loaded to 100 % rated capacity.

3.2.20 local side friction—same as friction sleeve resistance, fₛ (see 3.2.18).

3.2.21 penetration resistance measuring system—a measuring system that provides the means for transmitting information from the penetrometer tip and displaying the data at the surface where it can be seen or recorded.

3.2.22 penetrometer—an apparatus consisting of a series of cylindrical push rods with a terminal body (end section), called the penetrometer tip, and measuring devices for determination of the components of penetration resistance.

3.2.23 penetrometer tip—the terminal body (end section) of the penetrometer which contains the active elements that sense the components of penetration resistance. The penetrometer tip may include additional electronic instrumentation for signal conditioning and amplification.

3.2.24 piezoecone—same as electronic piezoecone penetrometer (see 3.2.10).

3.2.25 piezoecone porewater pressure, u—fluid pressure measured using the piezoecone penetration test.

3.2.26 piezoecone porewater pressure measurement location: u₁, u₂, u₃—fluid pressure measured by the piezoecone penetrometer at specific locations on the penetrometer as follows (1):³ u₁—porous filter location on the midface or tip of the cone, u₂—porous filter location at the shoulder position behind the cone tip (standard location) and, u₃—porous filter location behind the friction sleeve.

3.2.27 porewater pressure—total porewater pressure magnitude measured during penetration (same as 3.2.25 above).

3.2.28 porewater pressure ratio parameter, Bₛ—the ratio of excess porewater pressure at the standard measurement location Δu₂, to corrected total cone resistance qₜ, minus the total vertical overburden stress, σvo (see Eq 10).

3.2.29 push rods—the thick-walled tubes or rods used to advance the penetrometer tip.

3.2.30 sleeve friction, shear, and friction resistance—same as friction sleeve resistance.

3.2.31 subtraction error—an apparent load transfer from the cone to the friction sleeve of a subtraction type electronic cone penetrometer caused by minor voltage differences in response to load between the two strain element cells.

3.3 Abbreviations:

3.3.1 CPT—abbreviation for the cone penetration test.

³ The boldface numbers given in parentheses refer to a list of references at the end of the text.
3.3.2 *PCPT* or *CPμ*—abbreviation for piezocone penetration test (note: symbol "μ" added for porewater pressure measurements).

3.3.3 *CPTμ*—abbreviation for the piezocone penetration test with dissipation phases of porewater pressures (μ).

3.3.4 *SCPμ*—abbreviation for seismic piezocone test (includes one or more geophones to allow downhole geophysical wave velocity measurements).

3.3.5 *RCPμ*—abbreviation for resistivity piezocone (includes electrical conductivity or resistivity module).

4. Summary of Test Method

4.1 A penetrometer tip with a conical point having a 60° apex angle and a cone base area of 10 or 15 cm² is advanced through the soil at a constant rate of 20 mm/s. The force on the conical point (cone) required to penetrate the soil is measured by electrical methods, at a minimum of every 50 mm of penetration. Improved resolution may often be obtained at 20- or 10-mm interval readings. Stress is calculated by dividing the measured force (total cone force) by the cone base area to obtain cone resistance, *q*.<br>

4.2 A friction sleeve is present on the penetrometer immediately behind the cone tip, and the force exerted on the friction sleeve is measured by electrical methods at a minimum of every 50 mm of penetration. Stress is calculated by dividing the measured axial force by the surface area of the friction sleeve to determine sleeve resistance, *f*<sub>s</sub>.

4.3 Most modern penetrometers are capable of registering pore water pressure induced during advancement of the penetrometer tip using an electronic pressure transducer. These penetrometers are called "piezocones." The piezocone is advanced at a rate of 20 mm/s, and readings are taken at a minimum of every 50 mm of penetration. The dissipation of either positive or negative excess porewater pressure can be monitored by stopping penetration, unloading the push rod, and recording porewater pressure as a function of time. When porewater pressure becomes constant it is measuring the equilibrium value (designated *u*<sub>0</sub>) or piezometric level at that depth.

5. Significance and Use

5.1 Tests performed using this test method provide a detailed record of cone resistance which is useful for evaluation of site stratigraphy, homogeneity and depth to firm layers, voids or cavities, and other discontinuities. The use of a friction sleeve and porewater pressure element can provide an estimate of soil classification, and correlations with engineering properties of soils. When properly performed at suitable sites, the test provides a rapid means for determining subsurface conditions.

5.2 This test method provides data used for estimating engineering properties of soil intended to help with the design and construction of earthworks, the foundations for structures, and the behavior of soils under static and dynamic loads.

5.3 This method tests the soil in-situ and soil samples are not obtained. The interpretation of the results from this test method provides estimates of the types of soil penetrated. Engineers may obtain soil samples from parallel borings for correlation purposes but prior information or experience may preclude the need for borings.

6. Interferences

6.1 Refusal, deflection, or damage to the penetrometer may occur in coarse grained soil deposits with maximum particle sizes that approach or exceed the diameter of the cone.

6.2 Partially lithified and lithified deposits may cause refusal, deflection, or damage to the penetrometer.

6.3 Standard push rods can be damaged or broken under extreme loadings. The amount of force that push rods are able to sustain is a function of the unrestrained length of the rods and the weak links in the push rod-penetrometer tip string such as push rod joints and push rod-penetrometer tip connections. The force at which rods may break is a function of the equipment configuration and ground conditions during penetration. Excessive rod deflection is the most common cause for rod breakage.

7. Apparatus

7.1 *Friction Cone Penetrometer*—The penetrometer tip should meet requirements as given below and in 10.1. In a conventional friction-type cone penetrometer, the forces at the cone tip and friction sleeve are measured by two load cells within the penetrometer. Either independent load cells or subtraction-type penetrometers are acceptable for use (Fig. 1).

7.1.1 In the subtraction-type penetrometer, the cone and sleeve both produce compressive forces on the load cells. The load cells are joined together in such a manner that the cell nearest the cone (the "C" cell in Fig. 1b) measures the compressive force on the cone while the second cell (the "C + S" cell in Fig. 1b) measures the sum of the compressive forces on both the cone and friction sleeve. The compressive force from the friction sleeve portion is computed then by subtraction. This cone design is common in industry because of its rugged design. This design forms the basis for minimum performance requirements for electronic penetrometers.

7.1.1.1 Alternative designs have separate and non-independent load cells separate for tip and sleeve. For instance, in Fig. 1a, the cone penetrometer tip produces a compression force on the cone load cell (the "C" cell in Fig. 1a) while the friction sleeve produces a tensile force on the independent friction sleeve load cell (the "S" cell). Designs are also available where both the tip and sleeve load cells are independent and operate in compression (2). These penetrometer designs result in a higher degree of accuracy in friction sleeve measurement, however, may be more susceptible to damage under extreme loading conditions.

7.1.1.2 Typical general purpose cone penetrometers are manufactured to full scale outputs (FSO) equivalent to net loads of 10 to 20 tons. Often, weak soils are the most critical in an investigation program, and in some cases, very accurate friction sleeve data may be required. To gain better resolution, the FSO can be lowered or the independent type penetrometer design can be selected. A low FSO subtraction cone may provide more accurate data than a standard FSO independent type cone depending on such factors as system design and thermal compensation. If the FSO is lowered, this may place electrical components at risk if overloaded in stronger soils.
Expensive preboring efforts may be required to avoid damage in these cases. The selection of penetrometer type and resolution should consider such factors as practicality, availability, calibration requirements, cost, risk of damage, and preboring requirements.

7.1.1.3 The user or client should select the cone design requirements by consulting with experienced users or manufacturers. The need for a specific cone design depends on the design data requirements outlined in the exploration program.

7.1.1.4 Regardless of penetrometer type, the friction sleeve load cell system must operate in such a way that the system is sensitive to only shear stresses applied to the friction sleeve and not to normal stresses.

7.1.2 Cone—Nominal dimensions, with manufacturing and operating tolerances, for the cone are shown on Fig. 2. The cone has a diameter \( d = 35.7 \text{ mm} \), projected base area \( A_p = 1000 \text{ mm}^2 \), + 2%–5% with an apex angle of 60°. A cylindrical extension, \( h_2 \), of 5 mm should be located behind the base of the cone to protect the outer edges of the cone base from excessive wear. The 10 cm² cone is considered the reference standard for which results of other penetrometers with proportionally scaled dimensions can be compared.

7.1.2.1 In certain cases, it may be desirable to increase the cone diameter in order to add room for sensors or increase ruggedness of the penetrometer. The standard increase is to a base diameter of 43.7 mm which provides a projected cone base area of 1500 mm² while maintaining a 60° apex angle. Nominal dimensions, with manufacturing and operating tolerances for the 15 cm² cone, are shown in Fig. 2, based on the international guides (3).

7.1.2.2 The cone is made of high strength steel of a type and hardness suitable to resist wear due to abrasion by soil. Cone tips which have worn to the operating tolerance shown in Fig. 2 should be replaced. Piezocone tips should be replaced when the tip has worn appreciably (as shown) and the height of the cylindrical extension has reduced considerably (as shown).

Note 2—In some applications it may be desirable to scale the cone diameter down to a smaller projected area. Cone penetrometers with 5 cm² projected area find use in the field applications and even smaller sizes (1 cm²) are used in the laboratory for research purposes. These cones should be designed with dimensions scaled in direct proportion to standard 10-cm² penetrometers. In thinly layered soils, the diameter affects how accurately the layers may be sensed. Smaller diameter cones may sense thinner layers more accurately than larger cones. If there are questions as to the effect of scaling the penetrometer to either larger or smaller size, results can be compared in the field to the 10-cm² penetrometer for soils under consideration. This is because the 10-cm² cone is considered the reference penetrometer for field testing.

7.1.3 Friction Sleeve—The outside diameter of the manufactured friction sleeve and the operating diameter are equal to the diameter of the base of the cone with a tolerance of +0.35 mm and −0.0 mm. The friction sleeve is made from high strength steel of a type and hardness to resist wear due to abrasion by soil. Chrome-plated steel is not recommended due to differing frictional behavior. The surface area of the friction sleeve is 150 cm² ± 2%, for a 10-cm² cone. If the cone base area is increased to 15 cm², as provided for in 7.1.2.1, the surface area of the friction sleeve should be adjusted proportionally, with the same length to diameter ratio as the 10-cm² cone. With the 15-cm² tip, a sleeve area of 225 cm² is similar in scale.

7.1.3.1 The top diameter of the sleeve must not be smaller than the bottom diameter or significantly lower sleeve resistance will occur. During testing, the top and bottom of the sleeve should be periodically checked for wear with a micrometer. Normally, the top of the sleeve will wear faster than the bottom.

7.1.3.2 Friction sleeves must be designed with equal end areas which are exposed to water pressures (2, 3, 4, 5, 6). This will remove the tendency for unbalanced end forces to act on the sleeve. Sleeve design must be checked in accordance with A1.7 to ensure proper response.
7.1.4 Gap—The gap (annular space) between the cylindrical extension of the cone base and the other elements of the penetrometer tip should be kept to the minimum necessary for operation of the sensing devices and should be designed and constructed in such a way to prevent the entry of soil particles. Gap requirements apply to the gaps at either end of the friction sleeve and to other elements of the penetrometer tip.

7.1.4.1 The gap between the cylindrical extension of the cone base and other elements of the penetrometer tip, \( e_{c} \), must not be larger than 5 mm for the friction cone penetrometer.

7.1.4.2 If a seal is placed in the gap, it should be properly designed and manufactured to prevent entry of soil particles into the penetrometer tip. It must have a deformability at least two orders of magnitude greater than the material comprising the load transferring components of the sensing devices in order to prevent load transfer from the tip to the sleeve.

7.1.4.3 Filter Element in the Gap—If a filter element for a piezocone is placed in the gap between cone and sleeve the sum of the height of cylindrical extension, \( h_{c} \), plus element thickness filling the gap, \( e_{c} \), can range from 8 to 20 mm (see 7.1.8 for explanation).

7.1.5 Diameter Requirements—The friction sleeve should be situated within 5 to 15 mm behind the base of the cone tip. The annular spaces and seals between the friction sleeve and other portions of the penetrometer tip must conform to the same specifications as described in 7.1.4. Changes in the diameter of the penetrometer body above the friction sleeve should be such that tip or sleeve measurements are not influenced by increases in diameter. International reference test procedures require that the penetrometer body have the same diameter as the cone for the complete length of the penetrometer body (3, 7, 8).

7.1.5.1 For some penetrometer designs, it may be desirable to increase the diameter of the penetrometer body to house additional sensors or reduce friction along push rods. These diameter changes are acceptable if they do not have significant influence on tip and sleeve data. If there is question regarding a specific design with diameter increases, comparison studies can be made to a penetrometer with constant diameter. Information on diameters of the complete penetrometer body should be reported.

Note 3—The effects caused by diameter changes of the penetrometer on tip and sleeve resistance are dependent on the magnitude of diameter increase and location on the penetrometer body. Most practitioners feel that diameter increases equivalent to addition of a friction reducer with area increases of 15 to 20% should be restricted to a location at least eight to ten cone diameters behind the friction sleeve.

7.1.6 The axis of the cone, the friction sleeve (if included), and the body of the penetrometer tip must be coincident.

7.1.7 Force Sensing Devices—The typical force sensing device is a strain gauge load cell that contains temperature compensated bonded strain gages. The configuration and location of strain gages should be such that measurements are not influenced by possible eccentricity of loading.

7.1.8 Electronic Piezocone Penetrometer—A piezocone penetrometer can contain porous filter element(s), pressure transducer(s), and fluid filled ports connecting the elements to the transducer to measure pore water pressure. Fig. 3 shows the common design types used in practice including: 10-cm² friction-type, type 1 and type 2 piezocone, and 15-cm² size. The standard penetrometer should be the type 2 piezocone with filter located at the shoulder (both 10-cm² and 15-cm²) to allow correction of tip resistances. The electric friction penetrometer without porewater transducers can be used in soils with minor porewater pressure development, such as clean sands, granular soils, as well as soils and fills well above the groundwater table. The type 1 with face filter element finds use in fissured geomaterials and materials prone to desaturation, as well as dissipation readings. Numerous design and configuration aspects can affect the measurement of dynamic water pressures. Variables such as the element location, design and volume of ports, and the type and degree of saturation of the fluids.
cavitation of the element fluid system and resaturation lag time, depth and saturation of soil during testing all affect the dynamic porewater pressure measured during testing and dissipation tests of dynamic pressures (1, 6). It is beyond the scope of the procedure to address all of these variables. As a minimum, complete information should be reported as to the design, configuration, and the preparation of the piezocone system that is used for the particular sounding.

7.1.8.1 Measurement of hydrostatic water pressures during pauses in testing are more straightforward. The presence of air entrained in the system only affects dynamic response. In high permeability soils (that is, clean sands), hydrostatic pressures will equalize within seconds or minutes. In low permeability materials such as high plasticity clays, equalization can take many hours. If the goal of the exploration program is only to acquire hydrostatic pressures in sands, some of the preparation procedures for dynamic pressure measuring can be relaxed, such as deairing fluids.

7.1.8.2 The porewater pressure measurement locations of the porous element are limited to the face or tip of the cone, $u_1$, directly behind the cylindrical extension of the base of the cone, $u_2$, or behind the sleeve, $u_3$. Some penetrometers used for research purposes may have multiple measurement locations.

7.1.8.3 There are several advantages to locating the porous element immediately behind the tip of the cone in location $u_2$, primarily the required correction of measured $q_t$ to total tip stress, $q_t$, as detailed extensively (1-6). Also, the element is less subject to damage and abrasion, as well as fewer compressibility effects (1, 6). Elements located in the $u_2$ location may be subject to cavitation at shallow depths in dense sands because the zone behind the height of cylindrical extension is a zone of dilation in drained soils. Similar response can occur in stiff fissured clays and crusts (1). Porewater pressure measurements obtained at the $u_1$ face location are more effective for compressibility determinations and layer detection, particularly in fissured soils, but are more subject to wear (9). At the $u_2$ location, a minimum 2-mm cylindrical extension of the cone tip ($h_t$) should be maintained for protection of the cone. Typical filter element thickness at all locations in the horizontal plane ranges from 5 to 10 mm.

7.1.8.4 The miniature diaphragm-type electronic pressure transducer is normally housed near the tip of the cone. For dynamic pressure measurements, the filter and ports are filled with deaired fluid to measure dynamic porewater pressure response. The volume of connecting ports to the transducer should be minimized to facilitate dynamic pressure response. These electronic transducers are normally very reliable, accurate, and linear in response. The transducer shall have a precision of at least ±14 kPa (±2 psi). The porewater pressure transducer must meet requirements given in 10.2.
7.1.8.5 Element—The element is a fine porous filter made from plastic, sintered steel or bronze, or ceramic. Typical pore size is between 20 to 200 microns (6, 9). Different materials have different advantages. Smearing of metallic element openings by hard soil grains may reduce dynamic response of the system, thus normally not used for face elements but best suited for shoulder filter positions. Ceramic elements are very brittle and may crack when loaded, but perform well on the cone face as they reduce compressibility concerns. Polypropylene plastic elements are most commonly used in practice, particularly at the shoulder. Plastic filters (as high-density polyethylene, HDPE, or high-density polypropylene, HDPP) may be inappropriate for environmental type CPTs where contaminant detection is sought. Typically, the filter element is wedged at the tip or midface (\(u_1\)) location, or located at the shoulder in the gap immediately above the cone extension (designated \(u_2\)) location. At these locations, it is important to design the penetrometer such that compression of the filter elements is minimized.

7.1.8.6 Fluids for Saturation—Glycerine, or alternatively silicone oil, is most often used for deairing elements for dynamic response. These stiff viscous oils have less tendency to cavitate, although cavitation may be controlled by the effective pore size of the element mounting surfaces. Water can be used for the fluid if the entire sounding will be submerged, or if dynamic response is not important. The fluids are deaired using procedures described in 11.2.

7.2 Measuring System—The signals from the penetrometer transducers are to be displayed at the surface during testing as a continuously updated plot against depth. The data are also to be recorded electronically for subsequent processing. Electronic recording shall be digital and use at least twelve bit (one part in 4096) resolution in the analog to digital conversion, although 16-bit resolution and higher may be preferable in very soft ground. Either magnetic (disk or tape) or optical (disk) non-volatile storage may be used. In analog systems, the temperature stability and accuracy of the A-to-D converter shall be such that the overall cone-transmission-recording system complies with calibration requirements set forth in the annex.

7.2.1 Use of analog systems is acceptable but the system resolution may be lower than requirements in the annex and Section 10. Use of an analog recorder as a supplement to digital system is advantageous because it can provide system backup.

Note 4—Depending upon the equipment, data stored digitally on magnetic drives, tapes, floppy disks, or other media are often used. The data files should include project, location, operator, and data format information (for example, channel, units, corrected or uncorrected, etc.) so that the data can be understood when reading the file with a text editor.

7.3 Push Rods—Steel rods are required having a cross sectional area adequate to sustain, without buckling, the thrust required to advance the penetrometer tip. For penetrometers using electrical cables, the cable is prestressed through the rods prior to testing. Push rods are supplied in 1-meter lengths. The push rods must be secured together to bear against each other at the joints and form a rigid-jointed string of push rods. The deviation of push rod alignment from a straight axis should be held to a minimum, especially in the push rods near the penetrometer tip, to avoid excessive directional penetrometer drift. Generally, when a 1-m long push rod is subjected to a permanent circular bending resulting in 1 to 2 mm of center axis rod shortening, the push rod should be discarded. This corresponds to a horizontal deflection of 2 to 3 mm at the center of bending. The locations of push rods in the string should be varied periodically to avoid permanent curvature.

7.3.1 For the 10-cm² penetrometer, standard 20-metric ton high tensile strength steel push rods are 36-mm outside diameter, 16-mm inside diameter, and have a mass per unit length of 6.65 kg/m. For 15-cm² penetrometers, the test may be pushed with 44.5-mm outside diameter rods or with standard rods used for the 10-cm² penetrometer.

7.4 Friction Reducer—Friction reducers are normally used on the push rods to reduce rod friction. If a friction reducer is used, it should be located on the push rods no closer than 0.5 m behind the face of the cone. Friction reducers, that increase push rod outside diameter by approximately 25 %, are typically used for 10-cm² cones. If a 15-cm² penetrometer is advanced with 36-mm push rods there may be no need for friction reducers since the penetrometer itself will open a larger hole. The type, size, amount, and location of friction reducer(s) used during testing must be reported.

7.5 Thrust Machine and Reaction—The thrust machine will provide a continuous stroke, preferably over a distance greater than 1 m. The thrust machine should be capable of adjusting push direction through the use of a leveling system such that push initiates in a vertical orientation. The machine must advance the penetrometer tip and push rods at a smooth, constant rate (see 12.1.2) while the magnitude of thrust can fluctuate. The thrust machine must be anchored or ballasted, or both, so that it provides the necessary reaction for the penetrometer and does not move relative to the soil surface during thrust.

Note 5—Cone penetration soundings usually require thrust capabilities ranging from 100 to 200 kN (11 to 22 tons) for full capacity. High mass ballasted vehicles can cause soil surface deformations which may affect penetrometer resistance(s) measured in near surface layers. Anchored or ballasted vehicles, or both, may induce changes in ground surface reference level. If these conditions are evident, they should be noted in reports.

7.6 Other Sensing Devices—Other sensing devices can be included in the penetrometer body to provide additional information during the sounding. These instruments are normally read at the same continuous rate as tip, sleeve, and porewater pressure sensors, or alternatively, during pauses in the push (often at 1-m rod breaks). Typical sensors are inclinometer, temperature, resistivity (or its reciprocal, electrical conductivity), or seismic sensors, such as geophones that can be used to obtain downhole shear wave velocity. These sensors should be calibrated if their use is critical to the investigation program. The use of an inclinometer is highly recommended since it will provide information on potentially damaging situations during the sounding process. An inclinometer can provide a useful depth reliability check because it provides information on verticality. The configuration and methods of operating such sensors should be reported.
8. Reagents and Materials

8.1 O-Ring Compound—A petroleum or silicon compound for facilitating seals with O-rings. Use of silicon compounds may impede repair of strain gages if the strain gauge surface is exposed to the compound.

8.2 Glycerine, or CH₃OH(CH₂OH)₂, for use in porewater pressure measurement systems. Approximately 95% pure glycerine can be procured from most drug stores.

8.3 Silicone Oil (or fluid), for use in porewater pressure measurement systems. This material is available in varying viscosities ranging from 1400 to 10 000 CP.

Note 6—Detailed comparisons and discussions on the use of these fluids can be found elsewhere (6, 9).

9. Hazards

9.1 Technical Precautions—General:

9.1.1 Use of penetrometer components that do not meet required tolerances or show visible signs of non-symmetric wear can result in erroneous penetration resistance data.

9.1.2 The application of thrust in excess of rated capacity of the equipment can result in damage to equipment (see Section 6).

9.1.3 A cone sounding must not be performed any closer than 25 borehole diameters from any existing unbackfilled or uncased bore hole.

9.1.4 When performing cone penetration testing in prebored holes, an estimate of the depth below the prebored depth which is disturbed by drilling, should be made and penetration resistance data obtained in this zone should be noted. Usually, this depth of disturbance is assumed to be equal to at least three borehole diameters.

9.1.5 Significant bending of the push rods can influence penetration resistance data. The use of a tubular rod guide is recommended at the base of the thrust machine and also in prebored holes to help prevent push rod bending.

9.1.6 Push rods not meeting requirements of 7.3 may result in excessive directional penetrometer drift and possibly unreliable penetration resistance values.

9.1.7 Passing through or alongside obstructions may deflect the penetrometer and induce directional drift. Note any indications of encountering such obstructions, such as gravel, and be alert for possible subsequent improper penetrometer tip operation.

9.1.8 If the proper rate of advance of the penetrometer is not maintained for the entire stroke through the measurement interval, penetration resistance data will be erroneous.

9.2 Technical Precautions—Electronic Friction Cone Penetrometer:

9.2.1 Failure of O-ring seals can result in damage to or inaccurate readings from electronic transducers. The O-ring seals should be inspected regularly, after each sounding, for overall condition, cleanliness and watertightness.

9.2.2 Soil ingress between different elements of a penetrometer tip can result in unreliable data. Specifically, soil ingress will detrimentally affect sleeve resistance data. Seals should be inspected after each sounding, maintained regularly, and replaced when necessary. If very accurate sleeve resistance data is required, it is recommended to clean all seals after each sounding.

9.2.3 Electronic cone penetrometer tips should be temperature compensated. If extreme temperatures outside of the range established in A1.3.3 are to be encountered, the penetrometer should be checked for the required temperature range to establish that they can meet the calibration requirements. Also, harsh environments may severely affect the data acquisition system of power supplies, notebook or field computers, and other electronics.

9.2.4 If the shift in baseline reading after extracting the penetrometer tip from the soil is so large that the conditions of accuracy as defined in 10.1.2.1 are no longer met, penetration resistance data should be noted as unreliable. If baseline readings do not conform to allowable limits established by accuracy requirements in 10.1.2.1, the penetrometer tip must be repaired, and recalibrated or replaced.

9.2.5 Electronic friction cone penetrometers having unequal end areas on their friction sleeves can yield erroneous friction readings because of dynamic porewater pressures acting unevenly on the sleeve (2, 3, 4, 6). Friction sleeve design should be checked in accordance with A1.7 to ensure balanced response. The response is also dependent on location of water seals. If O-ring water seals are damaged during testing, and sleeve data appear affected, the sounding data should be noted as unreliable and the seals should be repaired.

9.3 Piezocone Penetrometer—The electronic piezocone penetrometer tip measures pore water pressures on the exterior of the penetrometer tip by transferring the pressure through a de-aired fluid system to a pressure transducer in the interior of the tip. For proper dynamic response, the measurement system (consisting of fluid ports and porous element) must be completely saturated prior to testing. Entrained air must be removed from the fluid-filled system or porewater pressure fluctuation during penetrometer tip advancement will be incorrect due to response lag from compression of air bubbles (see 11.2, 12.3.2, and 12.3.3). For soundings where dynamic response is important, the prepared filter elements should be replaced after every sounding.

10. Calibration and Standardization

10.1 Electronic Friction Cone Penetrometers:

10.1.1 The requirements for newly manufactured or repaired cone penetrometers are of importance. Newly manufactured or repaired electronic cone penetrometers are to be checked to meet the minimum calibration requirements described in the annex. These calibrations include load tests, thermal tests, and mechanical tests for effects of imbalanced hydrostatic forces. Calibration procedures and requirements given in the annex are for subtraction-type cone penetrometers. Calibration requirements for independent-type cone penetrometers should equal or exceed those requirements. The calibration records must be certified as correct by a registered professional engineer or other responsible engineer with knowledge and experience in materials testing for quality assurance. Applied forces or masses must be traceable to calibration standard forces or masses retained by the National Institute of Standards and Technology (NIST), formerly the
National Bureau of Standards. For description of calibration terms and methods for calibrating, refer to the annex.

10.1.2 Baseline Readings—Baseline or zero-load readings for both cone and friction sleeve load cells and porewater pressure transducers must be taken before and after each sounding. The baseline reading is a reliable indicator of output stability, temperature-induced apparent load, soil ingress, internal friction, threshold sensitivity, and unknown loading during zero setting. Take the initial baseline reading after warming electrical circuits according to the manufacturer’s instructions, generally for 15 to 30 min, and in a temperature environment as close as possible to that of the material to be sounded. If temperature is of concern, immerse the penetrometer tip in a bucket of fresh tap water, or insert the penetrometer tip in the ground while electrically warming circuits to stabilize its temperature and then extracted for rapid determination of initial baseline. After a sounding is completed, take a final baseline. The change in initial and final baseline values should not exceed 2% FSO for the cone tip, sleeve, and pressure transducer.

10.1.2.1 Maintain a continuous record of initial and final baselines during production testing. After each sounding, compare the final baseline to the initial baseline for agreement within the tolerances noted above. In some cases during heavy production testing where the cone is not disassembled and cleaned after each sounding, the initial baseline for the next sounding can serve as the final baseline to the previous sounding as long as agreement is within allowable limits.

10.1.2.2 If the post sounding baseline shift exceeds above criteria, inspect the cone for damage by inspecting the tip and checking to see that the sleeve can be rotated by hand. If there is apparent damage, replace parts as required. Clean the cone and allow temperatures to equalize to presupposing conditions, and obtain a new baseline. If this value agrees with the initial baseline within the above criteria, a load range calibration check is not required. If the pre and post baselines are still not within the above criteria then it is likely that the shift was caused by an obstacle or obstruction and linearity should be checked with a load range calibration.

10.1.2.3 If the baseline shift still exceeds the above criteria, perform a load range calibration as described in 10.1.2.1. If the cone load cell baseline shift exceeds 2% FSO, the cone is likely damaged and will not meet load range criteria in 10.1.2.2. Sleeve load cell baseline shifts for subtraction-type penetrometers usually can exceed 2% FSO and still meet load range criteria.

10.1.2.4 Report data for the sounding where unacceptable baseline shift occurs as unreliable. In some cases it may be obvious where the damage occurred and data prior to that point may be considered reliable. The location where obvious damage occurred should be clearly noted in reports.

10.1.3 Penetrometer Wear and Usage

10.1.3.1 For penetrometers used regularly during production, periodic load range checks should be performed. The inspection period can be based on production footage such as once every linear 3000 m (approx. 10⁴ linear feet) of soundings. If field load range equipment is not available, the penetrometer may be checked in the laboratory at the end of a project.

10.1.3.2 For penetrometers that are used infrequently, a periodic check may be based on time period, such as once every year. If a penetrometer has not been used for a long period of time, checking it before use is advisable.

10.1.3.3 For projects requiring a high level of quality assurance, it may be required to do load range checks before and after the project.

10.1.3.4 Load range calibrations are required if the initial and final baselines for a sounding do not meet requirements given in 10.1.2.1.

10.1.3.5 Records documenting the history of an individual penetrometer should be maintained for evaluation of performance.

10.2 Porewater Pressure Transducer—Calibrate newly manufactured or repaired transducers in accordance with requirements in the annex. During production, the transducer should be calibrated at regularly scheduled intervals and whenever linear performance is suspect. The reference gauge can be a Bourdon tube pressure gauge, or electronic pressure transducer that is calibrated annually to NIST traceable loading device (dead weight testing apparatus).

10.2.1 Prior to testing, baseline values or initial zeroing of the transducer is performed on the porewater pressure transducer at ambient air pressures at the surface. Maintain records as to the baseline values for the transducer in similar fashion to those for tip and sleeve resistance. If significant changes in baseline values occur, normally 1 to 2% FSO, perform load range tests to check for possible damage and nonlinear response.

10.3 Calibrations of Other Sensing Devices—Calibration data for other sensors in the penetrometer body may require calibrations using procedures similar to those given in the annex for load cells and pressure transducers. The need for calibration depends on the requirements of the individual investigation program. For noncritical programs, the occurrence of reasonable readings may be sufficient. In critical programs, it may be necessary to load the sensor through the range of interest with reference standards to ensure accurate readings.

11. Conditioning

11.1 Power electronic cone penetrometer and data acquisition systems for a minimum time period to stabilize electric circuits before performing soundings. Power the system to manufacturer’s recommendations prior to obtaining reference baselines. For most electronic systems this time period is 15 to 30 min.

11.2 Electronic piezocone penetrometer soundings require special preparation of the transmitting fluid and porous elements such that entrained air is removed from the system. For soundings where dynamic response is important, replace the prepared filter elements and the ports flushed after every sounding. Some of the techniques discussed below have been successful for preparation of elements. Regardless of the techniques used, report the equipment and methods.
11.2.1 Field or laboratory tests can be performed to evaluate assembled system response, if desired. Place the cone tip and element in a pressurized chamber and subject to rapid pressure change. Compare the response of the system to the applied pressure changes and if responses match, the system is properly prepared.

11.2.2 Place elements in a pure glycerine or silicone oil bath under a vacuum of at least 90 % of one atmosphere (~90 kPa). Maintain vacuum until air bubble generation is reduced to a minimum. Application of ultrasonic vibration and low heat (T < 50°C) will assist in removal of air. Generally with use of combined vacuum, ultrasonic vibration, and low heat, filter elements can be deaired in about 4 h, although it is best to allow for 24 h to ensure best performance. Results will depend upon the viscosity of the fluid and pore size of the filter element.

11.2.3 Elements can be prepared in water by boiling the elements while submerged in water for at least 4 h, although damage may result from prolonged exposure in this approach (1).

11.2.4 Other Suitable Means—Report other techniques, such as commercially-purchased pre-saturated filter elements that are available, or grease-filled slot (2, 5).

11.2.5 Storage—Store prepared elements submerged in the prepared fluid until ready for use. Fill the containers and evacuate during storage. Allowable storage length depends on the fluid. If elements are prepared in water they must be deaired again one day after containers are opened and exposed to air. Elements stored in glycerine or silicone may be stored for longer periods, up to several months, after storage containers have been exposed to air.

12. Procedure

12.1 General Requirements:

12.1.1 Prior to beginning a sounding, perform site surveys to ensure hazards such as overhead and underground utilities will not be encountered. Position the thrust machine over the location of the sounding, and lower leveling jacks to raise the machine mass off the suspension system. Set the hydraulic rams of the penetrometer thrust system to as near vertical as possible. The axis of the push rods must coincide with the thrust direction.

12.1.2 Set the hydraulic ram feed rate to advance the penetrometer at a rate of 20 ± 5 mm/s for all electronic cone penetrometers. This rate must be maintained during the entire stroke during downward advance of the rods while taking readings.

12.1.3 Check push rods for straightness and permanent bending (See Section 7.3). Push rods are assembled and tightened by hand, but care must be taken and threads may need cleaning to ensure that the shoulders are tightly butted to prevent damage to the push rods. For electronic cone penetrometers using cables, the cable is prestrung through the push rods. Add friction reducer to the string of push rods as required, usually the first push rod behind the penetrometer tip and other rods as required.

12.1.4 Inspect penetrometer tips before and after soundings for damage, soil ingress, and wear. In very soft and sensitive soils where accurate sleeve data is required, dismantle electronic cone penetrometer tips and friction sleeves after each sounding to clean and lubricate as required. If damage is found after a sounding, note and record this information on the sounding data record or report.

12.2 Friction Cone Penetrometers:

12.2.1 Power up the penetrometer tip and data acquisition system according to the manufacturer's recommendations, typically 15 to 30 min, prior to use.

12.2.2 Obtain an initial baseline reading for the penetrometer in an unloaded condition at a temperature as close as possible to ground conditions. Obtain baseline readings with the penetrometer tip hanging freely in air or in water, out of direct sunlight. Compare baseline readings with the previous baseline reading for the requirements given in 10.1.2.1. If thermal stability needs to be assured, immerse the penetrometer tip in water at temperature close to ground; or perform an initial short penetration test hole, stop penetration and allow the penetrometer tip to reach soil temperature, and withdraw the penetrometer.

12.2.3 Measure the depth at which readings were taken with an accuracy of at least ±100 mm from the ground surface.

12.2.4 Determine the cone resistance and friction sleeve resistance, continuously with depth, and record the data at intervals of depth not exceeding 50 mm.

12.2.5 During the progress of sounding, monitor tip and sleeve forces continuously for signs of proper operations. It is helpful to monitor other indicators such as ram pressure or inclination to ensure that damage may not occur if highly resistant layers or obstructions are encountered. Inclination is a particularly useful indicator of imminent danger to the system (see 12.4).

12.2.6 At the end of a sounding, extract the penetrometer tip, obtain a final set of baseline readings with the penetrometer tip hanging freely in air or in water, and check them against the initial baseline. Record initial and final baselines on all documents related to the sounding.

12.3 Electronic Piezcone Penetrometers:

12.3.1 Power up the penetrometer tip and data acquisition system according to the manufacturer's recommendations, typically 15 to 30 min, prior to use.

12.3.2 Assemble the piezoelements with all fluid chambers submerged in the de-aired medium used to prepare the elements. Flush all confined areas with fluid to remove air bubbles. Tighten the cone tip to effectively seal the flat surfaces. For water fluid systems, protect the assembled system from evaporation by enclosing the porous element inside a fluid-filled plastic bag or cap sealed to the penetrometer tip.

12.3.3 If unsaturated soil is first penetrated and it is desired to obtain accurate dynamic porewater pressure response once below the ground water, it may be necessary to pre bore or sound a pilot hole to the water table. In many cases, the piezoelement fluid system may cavitate during penetration through unsaturated soil or in dilating sand layers below the water table and this can adversely affect dynamic response. As the cone is advanced deeper, the saturation levels may recover as air bubbles are driven back into solution according to Boyles Law. Evaluation of proper interpretation of dynamic response requires experience (1, 6). Pre-punching or pre-boring with a
two-level phase approach to soundings may help alleviate desaturation problems.

12.3.4 Record baseline readings with the penetrometer tip hanging freely in air, or in water, out of direct sunlight. Compare baseline readings with reference baseline readings for requirements given in 10.1.2.1 and 10.2. A baseline for the porewater pressure transducer is obtained immediately after assembly to avoid evaporation effects. If evaporation is a problem, temporarily immerse the penetrometer in a bucket of water until ready for baseline. Do not obtain transducer baselines with protective caps or covers in place as these may induce pressure in the system. Note the pressure from the pressure transducer to see if it is a reasonable value for the equipment and assembly technique used.

12.3.5 Follow procedures similar to electric friction cone in 12.2.4-12.2.6 with the addition of recording porewater pressure readings.

12.3.6 Dissipation Tests—If dissipation tests are to be conducted during progress of the sounding, penetration is temporarily stopped at the location of interest. If porewater pressures are measured at the \( u_2 \) or \( u_0 \) locations, it is common practice to release the force on the push rods. If porewater pressures are measured at the midface location \( u_1 \), maintain the force on the push rods. Record porewater pressure versus time during conduct of the dissipation test. Monitor pressures until equilibrium porewater pressure is reached or 50 % of the initial excess porewater pressure has dissipated. In fine graded soils of very low conductivity, very long times may be required to reach the 50 % dissipation. Depending on the requirements of the program, and any concern of friction buildup on the push rods, dissipation testing may be terminated prior to reaching the 50 % level. Report dissipation test data as a record of porewater pressure versus time, or more commonly, \( u \) versus logarithm of time.

12.3.7 Hydrostatic Porewater Condition:
If full dissipation are carried out, then the porewater transducer will eventually record the hydrostatic condition, thus providing an evaluation of the position of the groundwater table or phreatic surface.

12.4 Penetrometer Operation and Data Interpretation-Guidelines:
12.4.1 Directional Drift of Penetrometer:
12.4.1.1 The penetrometer may drift directionally from vertical alignment. Large deviations in inclination can create nonuniform loading and result in unreliable penetration resistance data. Reduce drift by accurately setting thrust alignment and using push rods which meet tolerances given in 7.3.

12.4.1.2 Passing through or alongside obstructions such as boulders, cobbles, coarse gravel, soil concretions, thin rock layers, or inclined dense layers will deflect the penetrometer tip and induce drifting. Note and record any indication of encountering such obstructions, and be alert for possible subsequent improper penetrometer tip operations as a sign of serious directional drift.

12.4.1.3 Penetrometer inclination is typically monitored in cone penetrometers. Impose limitations on inclination in the system to prevent damage to push rods and non-symmetric loading of the penetrometer tip. Generally, a 5° change in inclination over 1 m of penetration can impose detrimental push rod bending. Total drift of over 12° in 10 m of penetration imposes non-symmetric loading and possible unreliable penetration resistance data.

12.4.2 Push Rod Addition Interruptions—Short duration interruptions in the penetration rate during addition of each new push rod can affect initial cone and friction sleeve readings at the beginning of the next push. If necessary, note and record the depths at which push rods are added and where long pauses may have affected initial startup resistances.

12.4.3 Piezocone Porewater Pressure Dissipation Interruptions—Porewater pressure dissipation studies, for which soundings are stopped and rod load is released for varying time durations, can affect the initial cone, friction sleeve, and dynamic porewater pressure readings at resumption of cone penetration. If dissipation tests are performed, be aware of possible rebound effects on initial excess porewater pressures. Note and record the depth and duration for which dissipation values are taken.

12.4.4 Interruptions Due to Obstructions—If obstructions are encountered and normal advance of the sounding is stopped to bore through the obstructions, obtain further penetration resistance data only after the penetrometer tip has passed through the estimated zone of disturbance due to drilling. As an alternative, readings may be continued without first making the additional penetration and the disturbed zone evaluated from these data. Note and record the depth and thickness of obstructions and disturbed zones in areas where obstructions are drilled through.

12.4.5 Excessive Thrust Capacity—If excessive thrust pressure begins to impede the progress of the sounding, it may be necessary to withdraw and change friction reducers. Alternatively, sometimes friction may be reduced by withdrawing the penetrometer and rods up to one third to one half of the penetration depth and then repushing to depth at which the friction caused stopping. Continue collection of sounding data from the point of stopping. Note and record the delay time and depths to which the penetrometer was moved. Long delays and pauses may cause buildup of friction on the rods. Hold delays to the minimum required to perform dissipation tests or equipment repairs.

12.4.5.1 If a high resistance layer is encountered, and the hydraulic thrust machine is physically moved during penetration, terminate the sounding. Another indicator of reaching thrust capacity is the rebound of rods after they are released. The magnitude of rebound depends on the flexibility of the thrust machine and the push rods. An operator must become familiar with the safe deflection of the system and decide when excessive deflections are being reached.

12.4.6 Unusual Occurrences—As data are recorded, it is important to note unusual occurrences in testing. When penetrating gravels, it is important to note “crunching” sounds that may occur when particle size and percentage of coarse particles begin to influence penetration. Note and report all occurrences of coarse gravels.

12.5 Withdrawal:
12.5.1 Withdraw the push rods and penetrometer tip as soon as possible after attaining complete sounding depth.
12.5.2 Upon complete withdrawal of the penetrometer, inspect the penetrometer tip for proper operation. The friction sleeve should be able to be rotated through 360° by hand without detectable binding.

12.5.3 Record baseline readings with the penetrometer tip hanging freely in air, or in water, out of direct sunlight. Compare baseline readings with initial baseline reading for requirements given in 10.1.2.1.

12.6 Hole Closure—In certain cases, it may be prudent or required by state law or specifications, that the cone hole be filled, sealed, or grouted and closed after the sounding is completed. For example, in complex groundwater regimes, hole closure should be made to protect the water aquifer. Details on various methods for hole closure are provided elsewhere (10).

13. Calculation

13.1 Friction Cone Penetrometers—Most electronic cone penetrometers in use at the present time measure a change in voltage across a strain gauge element to determine change in length of the strain element. Using known constitutive relationships between stress and strain for the element, the applied force may be determined for the cone or friction sleeve. The applied force may then be converted to stresses using the basic equations given in 13.2 and 13.3. Since there are a wide variety of additional, optional measurements currently being obtained with electronic cone penetrometers and new ones being continually developed, it is beyond the scope of this procedure to detail the makeup, adjustments, and calculations for these optional measurements.

13.2 Cone Resistance, \( q_c \)—Required:

\[
q_c = Q_c / A_c
\]

where:
\( q_c \) = cone resistance MPa (for example, ton/ft\(^2\), kg/cm\(^2\), or bar),
\( Q_c \) = force on cone kN (for example, ton or kg), and
\( A_c \) = cone base area, typically 10 cm\(^2\), or 15 cm\(^2\).

13.2.1 Corrected Total Cone Resistance (Required)—Calculation of corrected total cone resistance requires measurement of porewater pressures measured at the shoulder in the \( u_2 \) position.

\[
q_t = q_c + u_2 (1 - a_n)
\]

where:
\( q_t \) = corrected total cone resistance, MPa (ton/ft\(^2\), kg/cm\(^2\), bar, or suitable units for stress),
\( u_2 \) = porewater pressure generated immediately behind the cone tip, kPa (for example, tsf, kg/cm\(^2\), bar, or suitable units for pressure), and
\( a_n \) = net area ratio (see A1.7).

13.2.1.1 The correction to total cone resistance is particularly important when porewater pressures are generated during penetration (for example, saturated clays, silts, and soils with appreciable fines). Generally, the correction is not so significant for CPTs in clean sands, dense to hard materials, and dry soils. The correction is due to porewater pressures acting on opposing sides of both the face and joint annulus of the cone tip (1, 2, 4, 6).

Note 7—In all cases, the total value \( q_t \) should be used, substituted for (or both) \( q_c \), wherever possible. In no cases should \( q_c \) be backdetermined from \( q_t \) for use in equations, charts, formulas, or other purposes. It is always a forward procedure with corrected total \( q_t \) to be preferred.

13.2.1.2 Empirical adjustment factors based on select soil types have been developed for some pressure elements in the \( u_1 \) position, however these are not reliable. On a site-by-site basis, a relationship between \( u_1 \) and \( u_2 \) may be possible.

13.3 Friction Sleeve Resistance, \( f_s \)—Required:

\[
f_s = Q_s / A_s
\]

where:
\( f_s \) = friction sleeve resistance kPa (ton/ft\(^2\), kg/cm\(^2\), bar, or suitable units for stress),
\( Q_s \) = force on friction sleeve kN (ton, kg, or suitable units for force), and
\( A_s \) = area of friction sleeve, typically 150 cm\(^2\) for 10-cm\(^2\) tip, or 200 to 300 cm\(^2\) for larger 15-cm\(^2\) cones.

Note 8—A corrected sleeve friction resistance may also be obtained (\( f_s \)) yet this requires both \( u_2 \) and \( u_1 \) measurements simultaneously (2, 3, 4, 6). Thus, the raw \( f_s \) has been accepted for practical reasons. A simplified correction has been adopted by selected organizations (for example, (6)).

13.4 Friction Ratio, \( R_f \)—Optional:

\[
R_f = (f_s / q_t) \times 100
\]

where:
\( R_f \) = friction ratio, \%,
\( f_s \) = friction sleeve resistance kPa (ton/ft\(^2\), kg/cm\(^2\), bar, or suitable units for stress),
\( q_c \) = cone resistance kPa (ton/ft\(^2\), kg/cm\(^2\), bar, or suitable units for force), and
100 = conversion from decimal to percent.

13.4.1 Determination of the friction ratio requires obtaining a cone resistance and friction sleeve resistance at the same point in the soil mass. The point of the cone is taken as the reference depth. Typically, a previous cone tip resistance reading at friction sleeve midpoint depth is used for the calculations. For the 10-cm\(^2\) penetrometer, the standard offset is 100 mm. If an offset other than midheight is used it must be reported.

Note 9—In some cases, if readings are compared at the same point in a soil mass which has alternating layers of soft and hard materials erratic friction ratio data will be generated. This is because cone resistance is sensed, to varying degrees, ahead of the cone. The erratic data may not be representative of soils actually present.

Note 10—The friction sleeve resistance and friction ratio obtained from the mechanical friction cone penetrometers will differ considerably from values obtained from electronic friction cone penetrometers. When using soil classification charts that use \( R_f \) and \( q_c \), it is important to use charts based on correlations for the type of penetrometer being used.

13.5 Porewater Pressure Data:

13.5.1 SI metric units for reporting porewater pressure data are kPa.

13.5.2 Conversion of Measured Porewater Pressures to Equivalent Height of Water—Optional—If it is desired to display porewater pressure in equivalent height of water, convert the dynamic or static water pressures to height by dividing pressure by the unit weight of freshwater, \( \gamma_w \approx 9.8 \) kN/m\(^3\)(62.4 lb/ft\(^3\)). For salt water, use \( \gamma_w = 10.0 \) kN/m\(^3\)(64.0 lb/ft\(^3\)).
13.5.3 Estimate of Equilibrium Porewater Pressure (Hydrostatic Porewater Pressure)—Excess porewater pressure can only be calculated by knowing equilibrium pore water pressure, \(u_e\) (see 3.2.14). The equilibrium water pressure can be measured by dissipation test or estimated by calculation as follows (see Terminology D 653):

\[ u_e = \text{estimated equilibrium water pressure} = h_w \cdot \gamma_w \]  \(5\)

In saturated soils below the groundwater level, the hydrostatic case is obtained from:

\[ u_e = (z - z_w) \gamma_w \]  \(6\)

For soils above the groundwater table that are saturated due to full capillarity, Eq 6 is also applicable. For dry soils above the groundwater table, it is commonly adopted that \(u_e = 0\). In partially-saturated soils (vadose zone), there can be great transient variations and variability in the \(u_e\) profile.

where:

- \(h_w\) = height of water, m (or feet), evaluate from site conditions,
- \(\gamma_w\) = unit weight of (fresh) water = 9.8 kN/m\(^3\) (or 62.4 lbs/ft\(^3\)),
- \(z\) = depth of interest (m or feet),
- \(z_w\) = depth to the groundwater table (phreatic surface).

In layered soils with multiple perched aquifers the assumption of a single height of water may be in error.

13.6 Normalized CPT Measurements—In the latest soil behavior classification charts and CPT interpretation methods, normalized readings for cone tip resistance, sleeve friction, and porewater pressure are utilized \((z, 4, 11, 1\) ), as defined below.

13.6.1 Normalized cone tip resistance:

\[ Q = \frac{(q_i - \sigma_{vo})}{\sigma_{vo}'} \]  \(7\)

13.6.2 Normalized Porewater Pressure Parameter, \(B_2\)—

This parameter is normally calculated with the shoulder porewater pressure measurement (location immediately behind the cone tip), designated \(u_2\).

\[ B_2 = \frac{\Delta u}{q_i - \sigma_{vo}} \]  \(8\)

13.6.3 Normalized friction ratio:

\[ F = \frac{f_i}{q_i - \sigma_{vo}} \]  \(9\)

where:

- \(\Delta u\) = excess pore water pressure \((u_2 - u_e)\) (see 3.2.13),
- \(u_e\) = estimated equilibrium pore water pressure, or hydrostatic pressure (see 13.5.3),
- \(\sigma_{vo}\) = total vertical overburden stress, and
- \(\sigma_{vo}'\) = effective overburden stress \(= \sigma_{vo} - u_e\).

The total overburden stress is calculated:

\[ \sigma_{vo} = \Sigma (\gamma_h \Delta z) \]  \(10\)

where:

- \(\Delta z_i\) = layer thickness, and
- \(\gamma_h\) = total soil unit weight for layer.

14. Report

14.1 Report the following information:

14.1.1 General—Each sounding log should provide as a minimum:

14.1.1.1 Operator name,
sounding. Calibration records for the porewater pressure transducers are required as given in 10.2. If the project requires calibrations of other sensors they should also be submitted in final reports.

14.4 Graphs—Every report of friction cone penetration sounding is to include a cone tip resistance plot, \( q_t \) MPa, or preferably total cone tip resistance, \( q \) MPa (or \( \text{ton/ft}^2 \), \( \text{kg/cm}^2 \), bar, or other acceptable units of stress) with depth below ground surface \( m \) (ft), friction sleeve resistance, \( f_s \) kPa (ton/ft², kg/cm², bar, or other acceptable units of stress), and friction ratio, \( R_f \) (%), on the same plot. (See Fig. 4 and Fig. 5 for example plots.) As a minimum, the plot should provide general information as outlined in 14.1. Electronic piezocene penetrometer soundings should provide an additional plot of porewater pressure kPa (or lb/ft², kg/cm², bar, or other acceptable units of pressure) versus depth, \( m \) (ft). Porewater readings can be plotted as pressures, or alternatively, the pressure may be converted to equivalent heights of water (that is, \( h_w = u_w/\gamma_w \)).

14.4.1 Symbols \( q_t \) and \( f_s \) for tip and sleeve resistance are accepted by the International Society for Soil Mechanics and Geotechnical Engineering (1, 2, 3, 7).

14.4.2 For uniform presentation of data, the vertical axis (ordinate) should display depth and the horizontal axis (abscissa) should display the test values. There are many preferences in plotting such that uniform plotting scales and presentation will not be required.

15. Precision and Bias

15.1 Precision—There are little direct data on the precision of this test method, in particular because of the natural variability of the ground. Committee D-18 is actively seeking comparative studies. Judging from observed repeatability in approximate uniform deposits, persons familiar with this test estimate its precision as follows:

15.1.1 Cone Resistance—Provided that compensation is made for unequal area effects as described in 13.2.1, a standard deviation of approximately 2 % FSO (that is, comparable to the basic electromechanical combined accuracy, nonlinearity, and hysteresis).

15.1.2 Sleeve Friction—Subtraction Cones—Standard deviation of 15 % FSO.

15.1.3 Sleeve Friction—Independent Cones—Standard deviation of 5 % FSO.

15.1.4 Dynamic Porewater Pressure—Strongly dependent upon operational procedures and adequacy of saturation as described in 11.2. When carefully carried out a standard deviation of 2 % FSO can be obtained.

15.2 Bias—This test method has no bias because the values determined can be defined only in terms of this test method.

Note 11—Jefferyes and Davies (11) report \( q_t \), repeatability of the two different soundings in compact clean sand using two different cones by the same manufacturer. Approximately 50 % of the data lay within 8 % of the average of the two tests, and 90 % of the data lay within 15 % of the average. In this trial the transducers (that conformed to the requirements in A1.5) were loaded to between one tenth and one fifth of their rated FSO, so confirming a standard deviation of better than 2 % FSO.

16. Keywords

16.1 cone penetration test; cone penetrometer; explorations; field test; friction resistance; geotechnical test; in-situ testing; penetration tests; penetrometer; piezocene; point resistance; porewater pressures; resistance; sleeve friction; soil investigations
ANNEX

(Mandatory Information)

A1. CALIBRATION REQUIREMENTS ON NEWLY MANUFACTURED OR REPAIRED ELECTRONIC FRICTION CONE AND PIEZOCONE PENETROMETERS

A1.1 Introduction:

A1.1.1 This annex describes procedures and requirements for calibrating electronic cone penetrometers. The evaluation of cone penetrometer calibration as described in this annex is a quality assurance standard for newly manufactured and repaired penetrometer tips. Many of the standards may be impractical to evaluate under field operating conditions. Therefore, determination of these calibration errors for any individual penetrometer tip should be performed in a laboratory environment under ideal conditions by the manufacturer or other qualified personnel with necessary knowledge, experience, and facilities.

A1.1.2 The electronic cone penetrometer is a delicate instrument subjected to severe field conditions. Proper use of such an instrument requires detailed calibration after manufacture and continuous field calibrations. Years of cone penetrometer design and performance experience have resulted in refined cone designs and calibration procedures which make the electronic cone penetrometer a highly reliable instrument. Reports of these experiences form the basis for requirements in this annex (1, 2, 3, 9).

A1.1.3 The required calibration tolerances developed in this annex are based on subtraction type electronic cone penetrometers. These penetrometers are more robust than electronic cone penetrometers with independent tip and sleeve load cells and are the most widely used design. The subtraction type penetrometer, however, has less precision due to the subtraction process (3, 9). As a result, calibration tolerances given here are considered maximum values and requirements for more sensitive cone penetrometers imply smaller tolerances having greater precision. The calibration process consists of loading the penetrometer tip with reference forces and pressures and then comparing measured output to the reference.

A1.1.4 Calibrations in the laboratory environment should be performed with the complete penetrometer system to be used in the field. The same make and model computer, cable, signal conditioning system, and penetrometer to be used in the field shall be calibrated in the laboratory. Depending on the components of the system some components may be substituted with acceptable replacements. Each individual penetrometer must be tested over a range of loads to assure adequate performance.

A1.2 Terms Related to Force Transducer Calibrations:

A1.2.1 Fig. A1.1 is a graphical depiction of terms related to transducer calibrations and defines the concepts of zero-load error, nonlinearity, hysteresis, and calibration error (2, 8).

A1.2.2 To evaluate several of these values, the FSO (full scale output) of the penetrometer tip is needed. The manufacturer shall provide full scale output information for the system. Cone penetrometer tips usually are available in nominal...
A1.3.3 Thermal Stability—For assurance of thermal stability, evaluate a particular design of a newly manufactured cone under a range of temperature conditions. Newly manufactured penetrometer tips are first cycled to a minimum of 80 % of FSO five times at room temperature, to remove any residual nonlinearity. After cycling, establish an initial reference baseline value at room temperature after the cone has been electrically powered for about 30 min. To evaluate thermal stability, stabilize the penetrometer tip at temperatures of 10 and 30°C and new baseline values are obtained. The change in baseline values must be ± 1.0 % FSO of either cone or friction sleeve resistances.

A1.4 Load Range Calibration:

A1.4.1 Calibrate newly manufactured or repaired cone penetrometers over a range of loads after production or repair. Load test the cone penetrometer system in a universal testing machine or specially designed cone penetrometer calibration device capable of independently loading the cone and friction sleeve. If a universal testing machine is used, a calibration certificate (current within the last year) in accordance with Practice E 4 must be available. If a cone calibration apparatus is used, it should also have a calibration document current within the last year. The calibration document shows that applied forces or masses are traceable to standard forces or masses retained by the National Institute of Standards and Technology. The universal testing machine or cone calibration devices must be capable of loading the penetrometer tip to 100 % FSO.

A1.4.2 Selection of loading steps and maximum loading varies depending on need and application. Select the load steps and maximum load to cover the range of interest and not necessarily the maximum capacity of the cone. Some calibrations stress more frequent load steps at lower loads to evaluate weaker materials. Selection of more frequent lower load steps may result in higher levels of calibration error since the best fit line is more influenced by the low range data.

A1.4.3 Perform the loading after the cone is subjected to five cycles of compressive loading and reference baselines, or internal zeroing, have been obtained at room temperature. The penetrometer is loaded in a minimum of six increments at forces equivalent to 0, 2, 5, 10, 25, 50, and 75 % FSO. At each increment of force, record both cone and sleeve resistances. Compute the actual cone tip resistance by dividing the applied force by the cone base area. The friction sleeve resistance is taken as the corresponding axial force over the sleeve area. Determine the “best fit straight line” by linear regression of applied force and measured output. The linearity is the difference between measured cone resistance and best-straight line cone resistance divided by the cone FSO. Evaluate hysteresis by comparing the difference between cone resistance at the same level of applied force in loading and unloading and dividing by cone FSO. Calculate calibration error by taking the difference between the best-fit-straight line cone resistance and actual cone resistance and dividing by the actual cone resistance. Calibration error can become larger with smaller measured outputs and, therefore, it is not evaluated at loadings equivalent to less than 20 % of cone FSO.
A1.4.3.1 When calibrating the penetrometer, monitor the friction sleeve resistance to evaluate apparent load transfer. With a subtraction-type electronic cone penetrometer tip, the apparent friction sleeve resistance is caused by electrical subtraction error, crosstalk, and any load transferred mechanically to the sleeve. With a cone, that provides for independent cone and sleeve measurements, apparent friction sleeve resistances are caused by electrical crosstalk and mechanical load transfer. Apparent load transfer must be less than 1.5% of FSO of the friction sleeve (1000 kPa).

A1.4.3.2 Maximum nonlinearity should be 0.2%, maximum calibration error should be 0.5%, and maximum apparent load transfer should be 1.2%. For this calibration, the zero load error was zero. Hysteresis was not evaluated in this example because the testing machine was incapable of producing the exact same force on the loading and unloading steps.

A1.4.4 For calibration of the friction sleeve element, apply the forces in seven increments at 0, 2, 5, 10, 25, 50, and 75% of FSO. Nonlinearity, hysteresis, and calibration error are evaluated in the same manner as calibrations for the cone tip reading. During friction sleeve calibration, monitor cone tip resistance to evaluate apparent load transfer that was not apparent in this calibration.

A1.5 Force Transducer Calibration Requirements:

A1.5.1 Calibration requirements developed for electronic cone penetrometers are based on past experience with subtraction-type electronic cone penetrometers and, as a result of this experience, represent the minimum precision requirement of electronic cone penetrometers. In cases where a higher level of precision is required, stricter calibration requirements would be required. Newly manufactured or repaired electronic cone penetrometers are required to meet the following requirements:

<table>
<thead>
<tr>
<th>Calibration Parameter</th>
<th>Element</th>
<th>Requirement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Zero-load error</td>
<td>Tip and sleeve</td>
<td>≤ ± 0.5% FSO</td>
</tr>
<tr>
<td>Zero-load thermal stability</td>
<td>Cone tip and sleeve</td>
<td>≤ ± 1.0% FSO</td>
</tr>
<tr>
<td>Nonlinearity</td>
<td>Cone tip</td>
<td>≤ ± 0.5% FSO</td>
</tr>
<tr>
<td></td>
<td>Sleeve</td>
<td>≤ ± 1.0% FSO</td>
</tr>
<tr>
<td>Hysteresis</td>
<td>Tip and sleeve</td>
<td>≤ ± 1.0% FSO</td>
</tr>
<tr>
<td>Calibration error</td>
<td>Cone tip</td>
<td>≤ ± 1.5% MO at &gt;20% FSO</td>
</tr>
<tr>
<td></td>
<td>Sleeve</td>
<td>≤ ± 1.0% MO at &gt;20% FSO</td>
</tr>
<tr>
<td>Apparent load</td>
<td>While loading cone tip</td>
<td>≤ ± 1.5% FSO of cone tip</td>
</tr>
<tr>
<td></td>
<td>While loading sleeve</td>
<td>≤ ± 0.5% FSO of cone tip</td>
</tr>
</tbody>
</table>

A1.6 Pressure Transducer Calibrations:

A1.6.1 Newly manufactured or repaired pressure transducers shall be supplied with a load range calibration provided by the manufacturer. The load range calibration shall consist of a minimum of six points of loading at least 75% of FSO. The applied pressures shall be traceable to reference forces maintained by NIST. The calibration shall meet the manufacturer's stated tolerances. Minimum requirements are linearity better than 1% of FSO and zero load error less than ±7 kPa (±1.0 lb/in²).

A1.6.2 The transducer shall be subjected to regular periodic inspection meeting requirements in A1.6.1.

A1.7 Correction of Tip and Sleeve Areas:

**Fig. A1.2 Determination of Net Area Ratio (a_n) for Corrections of Cone Tip Resistances (4)**

```latex
\[ d_t = \text{diameter geometry (as shown)} \]
\[ t_s = \text{thickness of friction sleeve} \]
\[ u_t = \text{measured porewater pressure} \]
\[ q_c = \text{measured cone tip resistance} \]
\[ f_s = \text{measured sleeve friction} \]
\[ q_t = \text{total cone tip resistance} \]
\[ f_t = \text{total sleeve resistance} \]
\[ a_n = \text{tip net area ratio from triaxial test} \]
\[ b_n = \text{sleeve net ratio from triaxial test} \]
\[ h_s = \text{height of sleeve} \]

Sleeve Friction:
\[ f_t = f_s - (\pi d_t^2 u_t + \pi d_s^2 u_2)/(\pi d_c h_s) \]
\[ f_t \approx f_s - b_n u_2 \]

Tip Resistance:
\[ q_t = q_c + (1-a_n)u_2 \]
A1.7.1 The conceptual regions where water pressures can act on the cone tip and sleeve elements are shown in Fig. A1.2. Water pressure that acts behind the cone tip will reduce measured cone resistance, \( q_c \), by the magnitude of water pressure acting on unequal areas of the tip geometry. It is therefore advantageous to use a penetrometer having a net area ratio \( a_n = 0.80 \) in order to minimize the effect of the correction (1, 2). Water pressure may also act on both ends of the sleeve, resulting in an imbalance of forces if the sleeve is not designed with equal effective end areas. The water pressures acting on the ends of the sleeve are not just a function of geometry, they are also a function of the location of water seals. Water pressures during penetration are not often measured at both ends of the sleeve (that is, simultaneous \( u_2 \) and \( u_3 \)) so a correction is not normally made for \( f_e \) (3).

A1.7.2 Equal end area friction sleeves should be required for use and should be designed by the manufacturer. The best method for evaluating sleeve imbalance is to seal the penetrometer in a pressure chamber and apply forces to measure the sleeve resistance after zeroing the system. Manufacturers should perform this check for a particular design to assure minimal imbalance.

A1.7.3 In order to calculate the corrected total cone resistance, \( q_c \), as shown in 13.2.1, it will be necessary to determine the area ratio of the cone. The penetrometer can be enclosed in a sealed pressure vessel (for example, triaxial cell) and water pressures should be applied as shown in the example in Fig. A1.3. The net area ratio is then used in computing the corrected total tip resistance.

A1.8 Other Calibrations—Other sensors such as inclination, temperature, etc. may require calibration depending on the requirements of the investigation. Perform such calibrations using similar techniques given in this annex or by other reference procedures. Report such calibrations when required.

A1.9 Documentation of Calibrations:

A1.9.1 Laboratory calibration documents consisting of a short report on the equipment and methods of testing, along with tables and figures similar to those in this annex, are required for the following occurrences:

A1.9.1.1 When new penetrometer tips are received, and
A1.9.1.2 When damaged penetrometer tips are repaired.

A1.9.2 The report must be certified by a registered professional engineer or other responsible engineer with knowledge and experience in materials testing for quality assurance. Calibration documents are retained on file by the offices responsible for performing soundings and should be updated at required intervals. For contract soundings, calibration documents should be obtained prior to contract acceptance and after testing on unaltered equipment.

A1.9.3 If the electronic cone penetrometer meets the field calibration requirements given in 10.1.3, it is only necessary to
adjust the penetrometer tip to the laboratory requirements on a yearly basis. Cone penetrometers should be calibrated using laboratory procedures prior to use on each new project, but they do not need to meet calibration tolerances as required for newly manufactured cones.

REFERENCES


SUMMARY OF CHANGES

Committee D18 has identified the location of selected changes to this standard since the last issue (D 5778 – 95 (2000)) that may impact the use of this standard. (Approved November 1, 2007.)

(1) New references added.
(2) Excess porewater pressure definition corrected in 3.2.13.
(3) Fig. 2 reference citation updated.
(4) Revised Fig. 3.
(5) Normalized cone tip resistance added to 13.
(6) Generally overall improvement in many graphs with newer figures that show better detailing and annotation.
(7) Fig. 1 includes three basic cone penetrometer designs (rather than older figure showing only two designs), that is, compression-, tension-, and subtraction-types.
(8) Fig. A1.1 and Fig. A1.1 have been replaced with newer figures to show the pressurization calibration.
(9) Section 12.6 on hole closure has been added.
(10) Use of capital U for porewater pressures is replaced with small lowercase u in 7.1.8.5.
(11) Penetrometer gap has now been labeled as e in 7.1.4.3.
(12) Added reference to Practice D 3740.
(13) Common stress and pressure values have been mentioned.
(14) Numerous general cleanup and correction of grammatical and spelling errors, too numerous to mention here.

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