HALEY & ALDRICH, INC.
6500 Rockside Road
Suite 200
Cleveland, OH 44131
216.739.0555

MEMORANDUM

5 June 2020
Original – 16 October 2016
File No. 129638-007

SUBJECT: History of Construction – Revised
Associated Electric Cooperative, Inc.
New Madrid Power Plant – Pond 004
New Madrid, MO

Haley & Aldrich, Inc. (Haley & Aldrich) has assisted Associated Electric Cooperative, Inc. (AECI) with the compilation of this history of construction in accordance with §257.73(c)(1) for the existing coal combustion residuals (CCR) surface impoundment known as Pond 004 at the New Madrid Power Plant (NMPP). This document addresses the requirements of the US Environmental Protection Agency's (EPA's) Hazardous and Solid Waste Management System; Disposal of Coal Combustion Residuals from Electric Utilities, 40 CFR Part 257 (CCR Rule) effective 19 October 2015 including subsequent revisions. To the extent feasible, AECI has provided documentation supporting the history of construction. Information on the history of construction of Pond 004 is presented in the following sections.

§257.73(c)(1)(i): The name and address of the person(s) owning or operating the CCR unit; the name associated with the CCR unit; and the identification number of the CCR unit if one has been assigned by the state.

Owner: Associated Electric Cooperative, Inc.
2814 South Golden Avenue
P.O. Box 754
Springfield, Missouri 65807

Name of CCR Unit: Pond 004 (Referred to as "Slag Pond 2 Impoundment" and "Slag Dewatering Pond" in the past)

§257.73(c)(1)(ii): The location of the CCR unit identified on the most recent U.S. Geological Survey (USGS) 7 ½ minute or 15 minute topographic quadrangle map, or a topographic map of equivalent scale if a USGS map is not available.

Latitude: 36°30'52"
Longitude: 89°33'34"
The general location of the facility is provided in Appendix A.

<u>§257.73(c)(1)(iii):</u> A statement of the purpose for which the CCR unit is being used.

Pond 004 is used for settling and temporary wet storage of bottom ash and boiler slag sluiced from the NMPP. This unit is primarily used when maintenance and repair of sluice lines to Pond 003 occur. The Unit is also used to stage CCR for beneficial use purposes.

<u>§257.73(c)(1)(iv):</u> The name and size in acres of the watershed within which the CCR unit is located.

USGS Watershed Name: Little River Ditches Watershed 08020204
Unit-specific Watershed Area: 10 acres

The watershed area, which includes only the impoundment area itself, is based on the most recent site topography, provided by AECI and conducted by Pictometry International Corporation between October 4-8, 2014. It should be noted that the drainage area was determined as part of the Inflow Flood Control System Plan required by §257.83 of the CCR Rule which is provided under separate cover.

<u>§257.73(c)(1)(v)</u>: A description of the physical and engineering properties of the foundation and abutment materials on which the CCR unit is constructed.

Supplemental investigation of engineering properties was conducted under structural stability analyses of §257.73 which were provided under separate cover to support CCR Rule compliance.

AECI more recently located supplemental information in its files related to Pond 004. Historic boring logs, laboratory testing, and summary of soil test results from the Pond 004 footprint and the borrow source from which the CCR unit is constructed are discussed in "Geotechnical Investigation Slag Processing and Loadout Facility New Madrid Power Plant" prepared by Bendy Engineering and dated November 1983 (Appendix B).

<u>§257.73(c)(1)(vi)</u>: A statement of the type, size, range, and physical and engineering properties of the materials used in constructing each zone or stage of the CCR unit; the method of site preparation and construction of each zone of the CCR unit; and the approximate dates of construction of each successive stage of construction of the CCR unit.

The type, size, range, and physical engineering properties of the materials of each zone of the Pond 004 were discussed on pages 2 and 4-6 of "Final Report, Round 7 Dam Assessment" by GZA GeoEnvironmental, Inc., dated 3 June 2011, from which an excerpt is provided as Appendix C.

AECI more recently located supplemental information in its files related to Pond 004. Historic boring logs, laboratory testing, and summary of soil test results from the Unit footprint and the borrow source from which the CCR unit is constructed are discussed in "Geotechnical Investigation Slag Processing and Loadout Facility New Madrid Power Plant" prepared by Bendy Engineering and dated November 1983 (Appendix B). Additional historic boring logs from the Unit footprint are located in "Contract Documents for Slag Loadout Facility" prepared by Barttelbort, Rhutasel and Associates, Inc. dated July 1979 (Appendix D).

Pond bottom design is discussed in "Slag Loadout Facility" prepared by AECI dated 1 December 1983 (Appendix E). AECI has not located construction records as relates to the actual permeability of constructed soils. As-built information on the method of site preparation and construction of Pond 004 is not readily available. Pond 004 was constructed circa 1984.

§257.73(c)(1)(vii): At a scale that details engineering structures and appurtenances relevant to the design, construction, operation and maintenance of the CCR unit, detailed dimensional drawings of the CCR unit, including a plan view and cross sections of the length and width of the CCR unit, showing all zones, foundation improvements, drainage provisions, spillways, diversion ditches, outlets, instrument locations, and slope protection, in addition to the normal operating pool surface elevation and the maximum pool surface elevation following peak discharge from the inflow design flood, the expected maximum depth of CCR within the CCR surface impoundment, and any identifiable natural or manmade features that could adversely affect operation of the CCR unit due to malfunction or mis-operation.

AECI more recently located supplemental information in its files related to Pond 004. Detailed dimensional drawings that represent the CCR Unit are attached in Appendix F.

§257.73(c)(1)(viii): a description of the type, purpose, and location of existing instrumentation.

Pond 004 does not have existing instrumentation.

§257.73(c)(1)(ix): area-capacity curves for the CCR unit.

Original design area-capacity curves for Pond 004 are not readily available. It should be noted that updated area-capacity curves for the impoundment were developed as part of the Inflow Flood Control System Plan required by §257.83 of the CCR Rule which were provided under separate cover.

§257.73(c)(1)(x): a description of each spillway and diversion design features and capacities and calculations used in their determination.

Discharges from the impoundment flow to a concrete drop outlet structure with concrete stoplogs. It should be noted that updated calculations for the outlet structure were developed as part of the Inflow Flood Control System Plan required by §257.83 of the CCR Rule which were provided under separate cover. No emergency spillway exists on the unit.

§257.73(c)(1)(xi): The construction specifications and provisions for surveillance, maintenance, and repair of the CCR unit.

Information on the construction specifications and provisions for surveillance, maintenance, and repair of the CCR unit are not readily available.

§257.73(c)(1)(xii): any record or knowledge of structural instability of the CCR unit.

There are no records or knowledge of structural instability associated with Pond 004.

§257.73(c)(2): Changes to the history of construction. If there is a significant change to any information compiled under paragraph (c)(1) of this section, the owner or operator of the CCR unit must update the relevant information and place it in the facility's operating record as required by §257.105(f)(9).

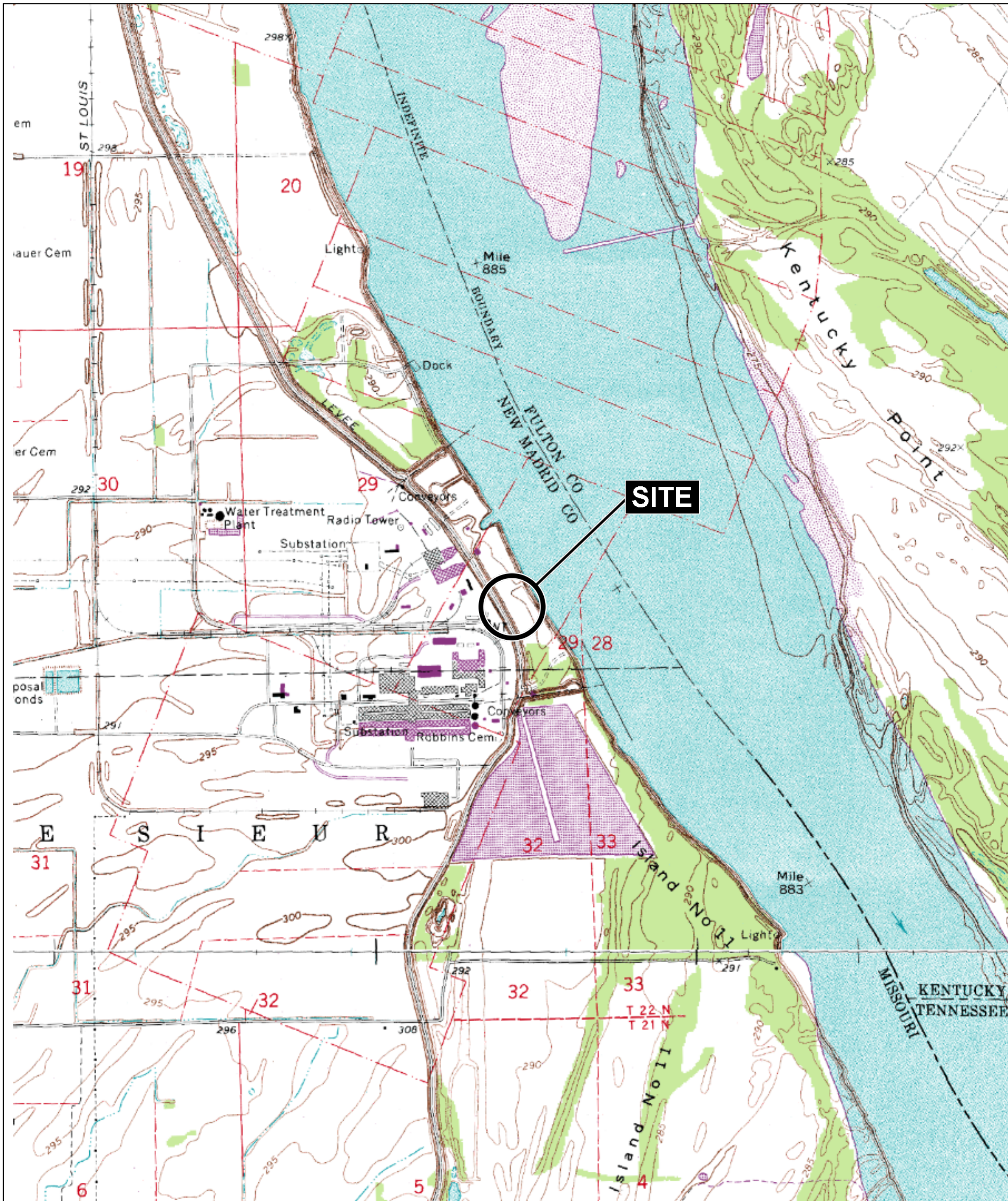
The History of Construction will be amended if conditions change that substantially affect the written plan in effect.

A record of amendments to the document will be tracked below. The latest version of the History of Construction will be noted on the first page of the Plan.

Revision	**Date**	**Description of Changes Made**
0	16 October 2016	Initial Submittal
1	5 June 2020	Description of Unit updated. Based on Owner locating additional supplemental information, 257.73(c)(1)(v), 257.73(c)(1)(vi), and 257.73(c)(1)(vii) were updated. Appendices B, D, E, and F were added.

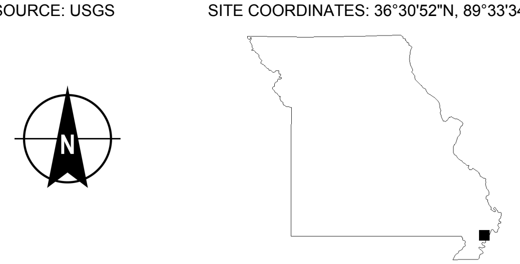
APPENDIX A
Site Locus

HALEY ALDRICH



MAP SOURCE: USGS

SITE COORDINATES: 36°30'52"N, 89°33'34"W



HALEY ALDRICH

ASSOCIATED ELECTRIC COOPERATIVE, INC.
POND 004 - NEW MADRID POWER PLANT
MARSTON, MISSOURI

PROJECT LOCUS

APPROXIMATE SCALE: 1 IN = 2000 FT
FEBRUARY 2020

FIGURE 1

40616_1_LOCUS.PDF

APPENDIX B
Geotechnical Investigation Slag Processing and Loadout Facility
New Madrid Power Plant
By Bendy Engineering, Dated November 1983

Geotechnical Investigation Slag Processing and Loadout Facility New Madrid Power Plant New Madrid, Missouri

Associated Electric Cooperative, Inc.
2814 S. Golden, P.O. Box 754
Springfield, Missouri 65801

Bendy Engineering
4260 Shoreline Drive
Earth City, Missouri 63045

November 1983

SHANNON & WILSON, INC. M-695

CONSULTANTS
William L. Shannon, P.E.
Stanley D. Wilson, P.E.

SHANNON & WILSON, INC.
Geotechnical Consultants
Suite 276 • 11500 Olive Boulevard • St. Louis, Missouri 63141-7126 • Telephone (314) 872-8170

November 18, 1983 M-695

Bendy Engineering
4260 Shoreline Drive
Earth City, Missouri 63045

Attention: Mr. Thomas J. Marshall, P.E.

REPORT OF GEOTECHNICAL INVESTIGATION
SLAG PROCESSING AND LOADOUT FACILITY
NEW MADRID POWER PLANT
NEW MADRID, MISSOURI

Gentlemen:

Submitted herewith is our final report of a geotechnical investigation for the slag processing and loadout facility to be constructed at the Associated Electric Cooperative, Inc., New Madrid Power Plant. A draft of the report was issued on October 27, 1983. The scope of our services was outlined in our proposal dated August 8, 1983. A supplementary proposal dated August 12, 1983, outlined a program of river soundings in the vicinity of the barge loadout facility. Acceptance of these proposals and formal authorization for the project was accomplished through a contract with Bendy Engineering.

We have appreciated this opportunity to be of service to you on the project. We also wish to express our appreciation for the assistance given by Associated Electric personnel during the field studies, particularly Mr. Russell A. Rice.

M. Mike Alizadeh, P.E.
Senior Vice President and
Central Regional Director

J. Ronald Salley, P.E.
Vice President

Christopher B. Groves, P.E.
Associate

Seattle • Portland • Spokane • Fairbanks • Anchorage • St. Louis

Bendy Engineering
November 18, 1983
Page 2

If you have any questions concerning our report or if we can be of other service to you, please call us.

Very truly yours,

SHANNON & WILSON, INC.

J. Ronald Salley

J. Ronald Salley, P.E.
Vice President

WJG:mjw

Copies submitted (11)

Bendy Engineering	3
Associated Electric Cooperative, Inc.	8

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ABSTRACT

A geotechnical investigation has been accomplished for the proposed slag processing and loadout facility at the New Madrid Power Plant, located adjacent to the Mississippi River south of New Madrid, Missouri. The facility will include a diked process area where slag will be dewatered by draining, and a retention pond to clarify the effluent. The dewatered slag will be carried to a barge loadout facility by an overhead conveyor. The loadout facility will consist of three sheet-pile mooring cells and two mooring dolphins.

The site is centered in an area of active seismicity, having experienced a series of extremely severe earthquakes early in the Nineteenth Century. Subsurface conditions consist of eight to twelve feet of clay soils underlain by deep alluvial sands. The upper sands are only medium dense and would be susceptible to liquefaction under moderate earthquake events.

The process area and surrounding dike will be constructed with a two-foot thick compacted clay liner to reduce infiltration into the underlying sands. The clay will be obtained from the retention pond excavation, supplemented as necessary by material from a nearby stockpile of fine-grained soils. Because of their relatively high permeability, the stockpiled soils may be used only in less critical areas such as the outer portion of the dike.

The risk of bottom heave in the lower portions of the pond was evaluated and found to be low providing that there is a head loss of one to two feet between the river level and the base of the underlying natural clay.

The effect of the processing area on the adjacent section of the existing river levee has been studied, considering stability, seepage, and settlement. Our analyses indicate that

the new facility should have no significant detrimental effect on the existing levee.

The dewatered slag will be fed to the conveyor system through a hopper located within the process area. A sheet-pile retaining wall will be used to retain an elevated earthern ramp adjacent to the hopper. All onshore structures including the pond outfall tower, the hopper, and the overhead conveyor should be supported on low-capacity piles to reduce the potential for damage due to loss of soil-supporting capability (liquefaction) during earthquakes. The piles should penetrate to at least El 265, several feet into relatively dense soils. Alternate recommendations for pile types include precast concrete, steel pipe and pressure grouted, augered piles. Very light elements of the conveyor system which can tolerate substantial differential settlement may be supported on spread footings high in the dike fill.

One thirty-foot and two sixteen-foot diameter mooring cells will be constructed for the barge loadout facility. The cells will extend approximately sixty-five feet above the river bottom. Our analyses indicate that the cells will be stable under maximum river stages, but that the loads resulting from a barge which became lodged broadside to the sixteen-foot diameter cells, or the impact of a runaway barge could overstress the supporting soils and result in tipping or leaning of the cell. These occurrences are considered too remote for design consideration. The larger central cell appears to be stable under the most severe high water conditions.

Two mooring dolphins will be installed between the sheet-pile cells. The dolphins will be made up of one vertical 30-inch diameter steel pipe and two battered 20-inch diameter pipe piles.

REPORT OF GEOTECHNICAL INVESTIGATION
SLAG PROCESSING AND LOADOUT FACILITY
NEW MADRID POWER PLANT
NEW MADRID, MISSOURI

INTRODUCTION

Project Description

The project under study consists of a facility which will be used to dewater and convey coal slag generated at the Associated Electric Cooperative, Inc. (AECI) New Madrid Power Plant. The plant is located in the Bootheel area of southeast Missouri on the west bank of the Mississippi River, approximately five miles south of the town of New Madrid.

Slag is presently flushed from the coal-fired boilers with water and the slurry is piped to a settling pond located about 2,500 feet south of the plant, crossing property belonging to the adjacent Noranda Aluminum Company facility. The planned new facility will consist of a diked area adjacent to the plant approximately 1100 feet by 400 feet in plan dimensions. The slurry will be deposited at the north end and permitted to drain. The fluid and suspended fine particles will flow to a retention pond located in the south portion of the diked area. After clarification, the water will be released to the adjacent river through an outlet structure.

The drained slag, consisting primarily of fine to coarse sand-sized particles, will be placed into a hopper using a rubber-tired front end loader. The hopper will feed a continuous belt conveyor system which will extend about 440 feet out to a barge loadout facility. This facility will consist of a central sheet-pile cell 30 feet in diameter flanked on either side by mooring dolphins and 16-foot diameter sheet-pile cells. The layout of the facility is shown on Plate 1.

Purpose and Scope

The purpose of our study was to investigate subsurface conditions at the site as a basis for recommendations concerning site preparation and earthwork for the pond and storage areas, together with foundation recommendations for the conveyor system and barge loadout facility. Our specific scope of work for each portion of the project is given below:

Barge Loadout Facility

- Review available boring logs, data on channel geometry, and construction records of existing structures.
- Conduct underwater soundings in the loadout area using a continuous reading fathometer.
- Prepare general recommendations for design and construction of mooring cells and dolphins based on the existing data base.

Conveyor Facility

- Drill one 40-foot boring at the hopper location.
- Prepare recommendations for foundation design for the hopper and any intermediate conveyor supports.
- Provide recommendations for sheet-pile retaining walls adjacent to the hopper.

Retention Pond

- Drill eight borings to depths of 10 to 20 feet in proposed dike areas, at the proposed pump structure, and in the pond interior.
- Determine the thickness of clay soils overlying sand and gravel.
- Conduct permeability tests on undisturbed clay samples and clay compacted to differing amounts. Perform Atterberg limits and strength testing of clay soils and grain size analyses of granular soils.

- Conduct engineering studies to develop dike design criteria considering underseepage, stability, seepage through the dikes, and slope protection. Also determine thickness and degree of compaction for impervious blanketing of pond area.

- Evaluate a nearby stockpile as a source of fine-grained borrow.

- Assess the effect of the facility on the adjacent existing levee system, including seepage, stability and settlement considerations.

SITE INVESTIGATION

Review of Available Data

Logs of borings performed during earlier investigations at the plant were provided by AECI. The information was reviewed and pertinent data plotted to supplement the field explorations undertaken during the present investigation. Information concerning the variability of soil conditions in off-shore borings was reviewed in detail to assess the need for over-water borings in the planned barge loading area.

Drilling Program

A total of nine borings was drilled within the retention pond and storage area, ranging in depth from 10 to 40 feet. Drilling was accomplished by Wabash Drilling Company of St. Louis under subcontract to Shannon & Wilson, Inc. An engineer from our firm was present throughout the drilling program to locate the borings, observe drilling procedures, assist in sampling, and maintain a detailed log of each boring. The location of each boring is shown on Plate 1.

Standard Penetration Test (SPT) samples and three-inch diameter thin-wall tube samples were taken at selected intervals in the borings. The samples were examined and then sealed in jars or tubes and transported to our St. Louis laboratory for further examination and testing. A discussion of drilling and sampling techniques and detailed logs of the borings are presented in Appendix A. Generalized profiles of the subsurface soil conditions across the slag processing area and through the existing levee are shown on Plates 2 and 3.

River Soundings

A series of soundings was conducted in the vicinity of the planned barge loadout facility to provide information on bottom elevations for use in positioning the facility. A base line was established across the north portion of the processing area and a skew line (Line A) was surveyed along the shoreline, extending downstream from the vicinity of the cooling water outlet for Unit Two. Seven lines of river soundings were extended outward from the shore, referenced to points established along Line A. The soundings were conducted by subcontract to the Okie Moore Company using sonar equipment operated from a work boat. An engineer from our firm was present to assist in the sounding operation. A discussion of the sounding survey is given in Appendix A, together with a tabulation of the results. Approximate contours of the river bottom, developed from the sounding data, are given on Plate 1.

Borrow Area Sampling

Surface samples of material were obtained from a stockpile of fine-grained soil which had been excavated from the adjacent Noranda Aluminum Plant. The stockpile lies at the southwest corner of the present settling pond. The samples were returned to our laboratory for additional examination and testing.

LABORATORY TESTING

Soil samples obtained from the test borings and the stockpile area were returned to our laboratory for examination, classification, and testing. The laboratory program consisted of index tests such as moisture content and Atterberg limits to establish basic properties and aid in the classification of the soils. Selected samples were then tested to determine the strength and permeability characteristics of the soils. Results of the laboratory tests are summarized and a brief description of test procedures is provided in Appendix B.

SITE CONDITIONS

Physiographic and Geologic Setting

The site is located near the northern limits of the Mississippi Embayment, which is a part of the Central Gulf Coastal Plain. This portion of the embayment is known as the Southeast Missouri Lowlands. The embayment is a broad, flat plain with a nominal southward slope, bounded by outcrops of Paleozoic rock which form uplands to the east and west. The boundary is characterized by a prominent escarpment known as the fall-line, located about 50 miles northwest of the site at its nearest point. Within the northern limits of the embayment lie two important topographic features, Crowley's Ridge and the Benton Hills. The closest approach of these upland remnants, which protrude through the recent deposits of the embayment, is about 30 miles from the site.

Within the embayment area, the Mississippi River and tributary streams formed deeply eroded valleys during glacial stages when large sheets of ice spread over the continents, resulting in an effective lowering of sea level by as much as 450 feet below its present-day elevation. This lowering of the base level enabled these rivers to erode downward into the Tertiary deposits, developing entrenched valleys. With the rise in sea level subsequent to glacial times, these entrenched valleys have been filled with alluvium. Typically the embayment is underlain by granular alluvium with a surface cover of silty clays. The alluvium generally grades coarser with depth, extending to depths of about 175 feet in the vicinity of the site. The alluvium is underlain by Tertiary and Cretaceous materials which are moderately indurated, varying from clays and marls to siltstone, sandstone, lignites and glauconitic sands.

Seismicity

The site is approximately centered in the New Madrid Fault Zone, an area which is the chief source of seismic activity in the central United States. The New Madrid earthquake series of 1811 and 1812 had a maximum estimated body wave magnitude of 7.4, with a maximum Modified Mercalli Intensity of XI-XII. The events were felt along the East Coast with Intensity IV-V from New York to Georgia. Intensity VII-VIII effects were noted in St. Louis. Subsequent earthquakes in 1843 and 1895 had body wave magnitudes of 6.0 and 6.2, respectively.

The high level of seismic activity in the New Madrid region has been attributed to the presence of the fault zone, consisting of a series of faults trending approximately parallel to the Mississippi Valley. The faults are covered by the recent alluvium of the embayment. The area is presently the subject of an on-going New Madrid Seismotectonic Study sponsored by the Nuclear Regulatory Commission.

Surface Features

The planned processing area is located on a level field between the existing levee and the bank of the river. At the time of our investigation, the south portion of the field was planted in soybeans and the remainder was overgrown with weeds. A drainage ditch oriented perpendicular to the river bisects the field. Just north of the ditch, an active stockpile of coal slag is maintained.

The river bank in the vicinity of the plant slopes downward at an inclination of approximately 1V on 3H, becoming somewhat flatter below El 260 to 270. The bank is covered with a concrete revetment consisting of rectangular slabs of concrete, approximately 1 ft by 4 ft in plan dimension, connected by heavy wire mesh. The revetment extends riverward to at least El 255; it is supplemented by riprap on the upper portion of the bank and at other locations along the shore within the project area. The concrete is somewhat broken up, but in most areas remains functional because of the mesh.

Results of the river soundings indicate the river bed in the vicinity of the proposed barge loading facility slopes downward at an inclination of about 1V on 4H to 1V on 7H out to approximately El 235, where it becomes flat in the vicinity of the existing mooring cells. The south portion of the area has a fairly broad terrace between El 240 and El 245, as shown on Plate 1. These contours should be considered only as an approximation of the river bed to give a general idea of its topography. It is not intended for accurate calculations of pile length or other precise determinations.

Subsurface Conditions

The soils encountered beneath the processing area consist of a surface layer of silty clay underlain by a silty sand unit extending to great depths. The base of the upper clay was encountered at depths of seven to fourteen feet in the retention pond area; borings at the north end of the process area encountered this level at a depth of five feet. Typically the thickness of the clay increased from north to south. The clays are generally stiff to very stiff in the upper extent, grading to medium stiff with depth. They are generally of high plasticity with liquid limits ranging from 50 to 80. Thin lenses and layers of low-plasticity silts are found within the clay. Laboratory tests on samples of the clay soils indicate that these materials are relatively impervious, with permeabilities generally varying between 10^{-7} and 10^{-9} cm/sec. A sample of silty clay from the lower portion of the unit in Boring 6 had a permeability of 2×10^{-6} cm/sec.

Boring 1 encountered coal slag mixed with silty clay extending to a depth of about 8 feet, nearly to the base of the clay layer. Boring 9, drilled approximately 100 feet away to check its horizontal extent, encountered no slag or other fill. Consequently, it is believed that the zone of slag is limited in extent.

The unit of sand which underlies the clay is an alluvial deposit having lenses and layers ranging from fractions of an inch to several feet in thickness. The horizontal extent of the stratification is variable, probably discontinuous across the site. At some locations a transition zone between the two units was found consisting of interlayered silty clay, clayey silt and silt with lenses of sand. Borings made during this investigation and previous studies indicate that the alluvium grades coarser with depth. The upper portion consists of silty fine sand with lenses and layers of coarse silt. In Boring 4 the unit consisted of silty fine sand to a depth of 25 feet, grading slightly cleaner with medium sand below this level. However, a review of earlier boring logs indicates that this stratum generally contains coarse sand and gravel below a depth of 30 to 40 feet. A review of the SPT N-value or blow count from the present investigation and earlier studies indicates that, in general, the N-value of the materials range from 7 to 25 to a depth of approximately 20 to 25 feet, indicating a medium dense condition. Below this level the sand increases in density with N-values of 20 to 40.

The soils observed near the surface of the Noranda stockpile consist of very silty clays to clayey silts which contain a minor to moderate amount of crushed rock up to two inches in maximum dimension. These soils have relatively high permeabilities (10^{-5} cm/sec), with correspondingly low liquid limits.

The elevation of groundwater at the site is directly related to the level of the adjacent river. The groundwater elevation in Boring 4 stabilized at approximately El 258, two or three feet above the level of the river. Isolated seepage from perched zones of water were occasionally encountered in some of the borings.

ANALYSES AND DESIGN RECOMMENDATIONS

Slag Processing Facility

Retention Pond Base. The results of our laboratory permeability tests indicate that the near-surface clay materials which underlie the processing area have low inherent permeabilities, less than 10^{-7} cm/sec. This material will be suitable for use in forming an impermeable pond base and perimeter dike, provided that the upper two-foot zone of native soil which will form the pond base and lower portion of the sides is over-excavated, conditioned to a moisture content greater than optimum, and recompacted to a specified density. Details are given in the section on construction recommendations. The purpose of this reworking of the pond materials is to minimize the effects of any shrinkage cracks or other discontinuities in the existing clay zone.

The material excavated from the pond may be used as fill to form the dikes and the elevated northern portion of the processing area. Our laboratory tests on samples of the Noranda stockpile indicate that it is gravelly, and composed of low-plasticity silts and silty clays. The permeability of these soils when compacted to a level corresponding to approximately 90 percent of the maximum dry density (ASTM D-698), at a moisture content near the optimum for compaction, was about 10^{-5} cm/sec, significantly higher than the on-site clays.

If sufficient clay is not available from the excavation, carefully selected portions of the Noranda stockpile may be used to form the exterior half of the dike and that portion of the dike which lies above El 295. These portions of the processing area fill are least critical with regard to seepage into the underlying sand aquifer. If necessary, the uppermost portion of the elevated discharge area could be made up of the stockpile material.

The potential for bottom heave in the deeper portion of the pond was evaluated by determining the thickness of the underlying clay layer based on our borings and then calculating the differential water head that could be resisted with a safety factor of 1.2. The results are as follows:

Location	Min. Clay Thickness, ft.	Allowable Differential Head, ft. (FS = 1.2)
Within 200'of river bank	9	6.7
Beyond 200' of river bank	7	5.2

With the pond at its operating level of El 294, the differential water level with the river at its 100-year flood level of El 300.5 is 6.5 feet. However, the net pressure at the base of the clay layer is expected to be somewhat less than the differential head between pond and river because of the distance from the river, the short duration of the peak river elevation, and time lags in transfer of piezometric pressure through the sand/silt material beneath the clay.

Within 200 feet of the river, as shown in the above table, the allowable differential head exceeds the 6.5-foot difference between operating pond elevation and the 100-year river level. Beyond 200 feet from the river, the clay layer can safely withstand a 5.2-foot differential head. If a one- to two-foot head loss is allowed between the river and bottom of the clay layer, our calculations indicate that this area of the pond will satisfactorily resist bottom heave.

Considering the infrequency of the 100-year storm and the probable lag in piezometric response between the high river level and the hydrostatic pressure at the base of the clay, the risk of heave is considered to be low. If heaving does occur, it should be localized, and could be repaired by draining the pond during normal water levels. Provided that the pond level does not drop below El 294 during floods, the risk of bottom heave appears to be acceptable as long as the Owner is prepared to make the necessary shutdowns and repairs if a heave occurs.

We suggest that as an operating policy, the pond level be raised whenever the river stage is predicted to rise above the operating level (El 294) to limit the differential head to five feet. This would be accomplished by placing additional stop-logs in the outlet structure so that inflow from the plant will effect the rise. This level should not severely impact the hopper and conveyor equipment or operations. The purpose would be to reduce any risk of heave or detrimental seepage. If overtopping is expected, the pond should be flooded to reduce or eliminate erosion on the interior slope of the dike.

<u>Trafficability</u>. The clayey soils to be used for the pond base and the elevated discharge area are relatively unstable when subjected to loads under wet or saturated conditions. As an example, the California Bearing Ratio (CBR) of a clay typically ranges from two to five, whereas the CBR of a sand would generally be from 20 to 40.

We expect that the surface of the discharge area will rut and weave severely under repeated passes of the front end loader. We suggest that the upper 30 inches of high traffic areas such as the ramp to the hopper be made up of slag or other granular material which is free from fines. Experience during initial operations with the front end loader will dictate how much of the coal slag must remain in place above the base of the pond in order to maintain trafficability. We anticipate that at least 18 to 24 inches must remain in place.

A similar situation will occur when removing sediment from the retention pond. The saturated clays will be easily disrupted under even moderately heavy equipment loads. Careful consideration must be given to the selection of equipment to use during cleanout. Special, wide tread-low bearing pressure dozers may be necessary to feed material to a backhoe or dragline for removal. It is extremely doubtful that a conventional front end loader could operate within the pond area without disrupting the integrity of the compacted clay base.

Dike Stability. Present plans call for the perimeter of the process area to be surrounded by a dike with its crest at El 301, approximately seven to ten feet above the surrounding grade. The dike will have a crest width of fifteen feet and side slopes of 1V on 3H, as shown on Plate 2. Where the process area lies adjacent to the existing levee, a ten-foot wide berm will be installed along the base of the levee side slope up to El 302. The surface of the berm will be faced with riprap within the operating level of the pond.

Our analysis of the stability of this dike system, assuming construction as recommended in subsequent paragraphs, indicates a factor of safety of at least 5.0 against rotational slides, under normal pond operating conditions. Additional unbalanced water forces will act on the dike during flood stages of the river which do not overtop the dike. Because of the low permeability of the clays and silts, it is doubtful that steady-state seepage conditions within the dikes would be established during the time frame in which the unbalanced water forces would act. However, even under these conditions, the embankments should have a factor of safety of at least 2.0. As noted previously, The factor of safety against bottom heave is about 1.2 during extreme high river stages.

Because the underlying sand stratum will act as a drain beneath the surface clay layer, the predominant direction of seepage from the pond during normal river stages will be downward rather than laterally away from the pond limits. Because of this and the fact that the operating level is approximately the same as the adjacent ground surface on the pond exterior, only minor seepage, if any, is expected to occur at the exterior toe of the dikes.

Placement of the dike fill is expected to result in settlements of 1.5 to 2.0 inches. Most of this settlement should occur rapidly as the fill is placed. Post-construction settlement should be less than one-half inch.

The on-site clays and in particular the low-plasticity silts from the Noranda stockpile are susceptible to erosion. We recommend that the side slopes of the dikes be seeded and fertilized to establish a functional ground cover as soon as possible after construction.

<u>Effect of Planned Process Area on Existing Levee</u>

We have analyzed the effect of the proposed pond and elevated drainage area on the existing levee system. The sections on Plate 3 illustrate the relationship between the cuts and fills involved in the proposed process area and the existing levee. A stability analysis of the levee was made using a conservatively assumed undrained shear strength value of 1000 pounds per square foot. The analysis indicates that critical potential failure surfaces on the landward side of the levee are not significantly affected by the planned facility.

A potential failure surface on the river side of the levee has a factor of safety of about 3.5 under present conditions. If the toe of the existing slope is extended from El 294 to El 289 to simulate pond excavation and water is ponded against the slope up to El 294, the factor of safety is reduced only slightly, to about 3.4. In actuality, the excavation for the pond will be maintained 30 feet away from the toe of the levee and the ten-foot wide berm will be added. The overall stability of the levee under normal conditions should remain at its original level or be increased by the planned process area.

During high water stages when the pond dike is overtopped, the pond would have no effect on levee stability. As flood waters recede, the adverse effects of sudden drawdown seepage conditions imposed on the outboard levee slope would be lessened by the planned berm.

Seepage analyses were made assuming the pond excavation would penetrate through the upper clay into the underlying silty sand. Our borings indicate that such an occurrence would be very unlikely, as in all cases the top of the sand was encountered at elevations well below the base of the pond. The purpose of the analysis is to check the potential for piping or erosion of soil fines when a differential head of water is imposed on the system. Lane's Method was used to evaluate a creep ratio (R_c) as follows:

$$R_c = \frac{H/3}{h}$$

Where H is the horizontal flow path length
and h is the differential head

The most critical combination of flow path and head occur for a flow from the pond to a drainage ditch at El 292 on the opposite side of the levee about 200 feet away. The two-foot differential head gives a ratio of 33, which is approximately four times greater than the recommended minimum for silty sand of 8.5. A similar analysis using Bligh's Method indicates a factor of about 5 between the actual ratio and the minimum recommended value for safety against piping.

We have analyzed the settlement which should occur at those points where the proposed dikes abut the levee. As indicated in a previous section, the compression of soil immediately beneath the dike is estimated to be approximately two inches. The settlement of the crest of the levee under the influence of the dike is estimated to be substantially less, because this portion of the levee will be somewhat removed from the influence of the new load. We estimate that the settlement of the levee crest in the immediate vicinity of the dike will be less than 1/2 inch.

Based on the analyses summarized above and our engineering judgment, we believe that the new facility will have no significant detrimental effect on the existing levee system.

<u>Pond Outfall</u>. The foundation level for the concrete outfall tower is El 284.5. The soils immediately beneath this level consist of medium stiff clays underlain by medium dense sand and silt. The total load of the structure could be supported by these soils while undergoing tolerable settlements. However, the upper sands are susceptible to liquefaction under only moderate earthquake loads which could be expected within the useful life of the facility. The loss of foundation support would probably involve toppling of the structure and drainage of pond contents. Therefore, we recommend that the structure be supported on low-capacity piles which extend below the zone of the easily liquefiable soils. Recommended pile types include 10-inch and 12-inch square precast concrete, 12-3/4-inch diameter closed end steel pipe, and 12-inch diameter pressure-grouted augered piles. Allowable capacity-embedment relations for these piles are given on Plate 4. We have not included a factor for densification of the upper sands during driving since the piles are in small groups which are widely spread.

Conveyor Facility

Hopper Foundation. The hopper will consist of a metal bin having maximum dimensions of approximately 10 feet by 15 feet in plan, which will receive slag at El 312 and discharge it onto a continuous belt conveyor. The floor of the hopper structure slopes to the west, with a minimum level at El 297.

The conveyor system will extend eastward to the barge loading facility about 440 feet away. The conveyor will have a maximum span of approximately 225 feet extending from the edge of the river bank out to the central mooring cell. Both the hopper structure and the conveyor system impose only moderate foundation loads which could be supported by the stiff upper clays with minor to moderate settlement. However, as with the outfall structure, the underlying sands are relatively loose and susceptible to liquefaction under moderate seismic loads. Consequently, we recommend that these structures also be supported on lightly loaded piles in accordance with the recommendations on Plate 4. The on-shore portion of the conveyor system between the hopper and the crest of the dike is supported about three feet above grade on pairs of short columns or piers. Each set of columns is independent of the others. These lightly loaded columns (less than 5 kips per pair) may be supported on shallow footings founded about two to three feet below grade, with provision for substantial movements to permit realignment of the conveyor belt idlers.

Retaining Wall. The hopper structure will be flanked by a sheet-pile retaining wall supporting a six-foot high elevated platform from which the front end loader will feed the hopper. The sheet pile system may be designed using the soil parameters given on Plate 5. Design methods are given in "U.S. Steel Sheet Piling Design Manual". Methods for modeling the wheel loads imposed by the front end loader are given on page 15 of that manual. Seismic effects can be accounted for by multiplying the weight of the active pressure wedge by the ratio of the earthquake acceleration to the acceleration of gravity. We recommend that a ratio of 0.2 be used, with the force acting at a level above the base of the wall equal to 0.55 times the free-standing wall height. These recommendations assume that a drain system is installed behind the wall and that free draining granular material such as coarse sand or slag is used for a minimum horizontal distance of four feet behind the wall.

Barge Loadout Facility

Mooring Cells. Three sheet-pile mooring cells are planned for the unloading facility. A 30-foot diameter central cell will support the conveyor with 16-foot diameter cells located about 300 feet upstream and downstream at the ends of the loadout facility. The cells will be installed in the river bottom at approximate El 243 to 248; the sheets will be 100 feet long with the cell top at El 309, giving a tip elevation of about 209.

Analyses of the cells were made for extreme loading conditions, assuming a maximum flood at El 301.5 and a current velocity of ten feet per second. Barge impact velocities were assumed at 0.5 feet per second. Our analyses indicate that the stability of the 16-foot diameter cell is satisfactory when the barges are in their normal orientation parallel to the facility during these high water conditions. If a barge were to become lodged against the cell when broadside to the current, or if the cell were struck by a runaway barge, the foundation soils would be overstressed and tilting might occur. The low probability of these occurrences does not warrant consideration as a design event, provided the Owner is willing to accept the remote risk of damage. The stability of the 30-foot diameter cell appears to be satisfactory under the most severe high water conditions.

Mooring Dolphins. Two mooring dolphins will be installed between the mooring cells at the unloading facility. Each dolphin will consist of a vertical 30-inch diameter steel pipe pile and two 20-inch pipe piles driven at a four on twelve batter. The batter piles will form a 60 degree angle in the horizontal plane, centered at right angles to the long axis of the pier. The piles will be driven open-ended and then filled with sand.

The lateral capacity of the dolphins can be calculated on the basis of uplift and compression capacities of the individual piles. The allowable capacity of these members for a range of penetrations is given on Plate 6. The uplift capacity of the 30-inch pile does not include the self weight of the pile and the interior sand fill. An allowance is included in these design charts for jetting of the piles down to a level no closer than 10 feet of the final tip elevation.

CONSTRUCTION RECOMMENDATIONS

Site Preparation

Topsoil in the processing area should be stripped and wasted away from the site or stockpiled for use in planting areas. The stockpile of slag should be removed from the area. The slag-clay fill zone encountered in Boring 1 should be excavated to a depth of at least three feet below final grade. The material should be wasted off-site.

Following site preparation, the processing area should be overexcavated to a depth of 18 inches below final grade. The exposed material should then be scarified to a depth of six inches and recompacted in accordance with the following section. The overexcavated material should be carefully examined by experienced geotechnical personnel to determine its suitability for use in the compacted clay pond liner. The base of the excavation should also be examined prior to compaction to check for sand zones.

Fill Placement and Compaction

Prior to compaction, all fill should be conditioned by drying or addition of water as necessary to achieve a moisture content between the optimum as determined by ASTM D-698 and three percentage points above this value. The material should be thoroughly mixed by a disc or other approved means of positive agitation to achieve a uniform moisture content. The fill material should be placed in uniform, level layers having a maximum loose thickness of six inches. The material should be compacted using a sheepsfoot roller to at least 90 percent of the maximum dry density as determined by the Standard Proctor compaction test procedure (ASTM D-698). If additional fill must be brought from the Noranda stockpile, the pile should be explored by excavating a series of test pits to identify the most suitable soils for inclusion in the drainage platform and dikes.

Pile Installation

Driven Piles. Piles are to be driven to the penetration which corresponds to the required design capacity as given on Plates 4 and 6. The piles should be driven with hammers which develop energies no greater than the maximum ratings given below. Hammers developing lower energy may be quite adequate. No penetration resistance criteria have been given as the tip bearing has been conservatively estimated. If refusal or pile damage occurs near the required penetration, a reanalysis of soil-pile-hammer conditions should be made to determine the most appropriate adjustment.

Pile Type		Maximum Hammer Energy (ft.-lb.)
10" square concrete		19,000
12" square concrete		26,000
12-3/4" steel pipe	(closed end)	26,000
20" steel pipe	(open end)	32,000
30" steel pipe	(open end)	42,000

Vibratory hammers having mass and frequency matched to the selected pile type and length may also be used. On-shore piles would have to be predulled through the clay and proof-driven with an impact hammer to assure that the required end-bearing had been developed.

The offshore piles may be jetted if necessary, provided that the tip of the jet is maintained at least ten feet above the final tip elevation. Proper hammer and pile cushioning must be maintained at all times. Concrete piles should not be driven with full hammer energy until sufficient soil resistance is encountered to prevent the development of excessive tensile forces within the pile.

Pile installation should be observed by experienced geotechnical personnel who will maintain driving records and confirm that installation procedures and tip elevations are in accordance with design.

<u>Pressure-Grouted Augered Piles</u>. These piles should be installed by a competent contractor who has demonstrated previous experience in their installation. The auger should be advanced into the ground in such a fashion as to minimize excessive rotation and removal of soil which would loosen the material adjacent to the grout column. The auger must be withdrawn from the hole with great care to insure that positive grout pressure is continuously maintained at the tip. The grout pump should be of the positive displacement type which is equipped with a working stroke-counter and pressure gage. It should be capable of delivering a maximum pressure of 350 psi at the pump outlet.

The installation should be continuously observed by a member of our staff who would document the soil type, depth of penetration, grout volumes, and pumping pressures for each pile.

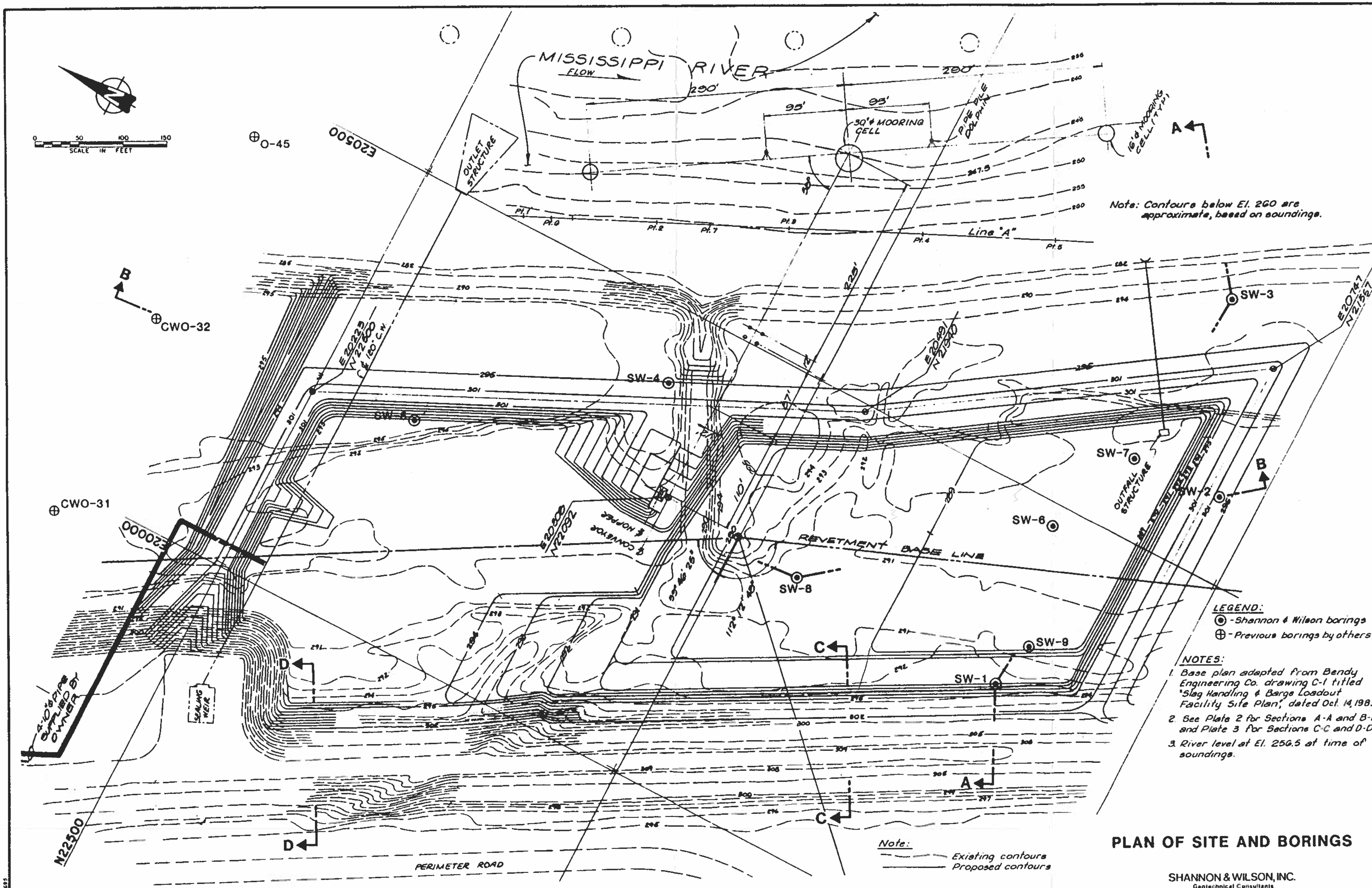
LIMITATIONS OF REPORT

The analyses, conclusions, and recommendations contained in this report are based on the site conditions described herein and further assume that the exploratory borings are representative of the subsurface conditions throughout the site, i.e., the subsurface conditions everywhere are not significantly different from those disclosed by the borings. If, during construction, subsurface conditions different from those encountered in the exploratory borings are observed or appear to be present beneath excavations, we should be advised at once so that we can review these conditions and reconsider our recommendations where necessary.

If there is a substantial lapse of time from the submittal of this report and the start of work at the site, or if conditions have changed due to natural causes or construction operations at or adjacent to the site, we recommend that this report be reviewed to determine the applicability of conclusions and recommendations considering the changed conditions and time lapse.

We recommend that we be retained to review those portions of the plans and specifications which pertain to foundations and earthwork to determine if they are consistent with our recommendations. In addition, we are available to observe construction, particularly construction of foundations, site grading, and earthwork. We would also be available to make such other field observations as may be necessary.

This report was prepared for the exclusive use of the owner, architect, and engineer for evaluating the design of the structure as it relates to the geotechnical aspects discussed herein. It should be made available to prospective contractors for information on factual data only and not as a warranty of subsurface conditions included in this report. Unanticipated soil conditions are commonly encountered and cannot be fully determined by taking soil samples from the borings. Such unexpected conditions may require that additional expense be made to attain a properly constructed project. Therefore, some contingency fund is recommended to accommodate these potential extra costs.



NOTES:

1. Base plan adapted from Bendy Engineering Co. drawing C-1 titled "Slag Handling & Barge Loadout Facility Site Plan," dated Oct. 14, 1983.
2. See Plate 2 for Sections A-A and B-B, and Plate 3 for Sections C-C and D-D.
3. River level at El. 256.5 at time of soundings.

PLAN OF SITE AND BORINGS

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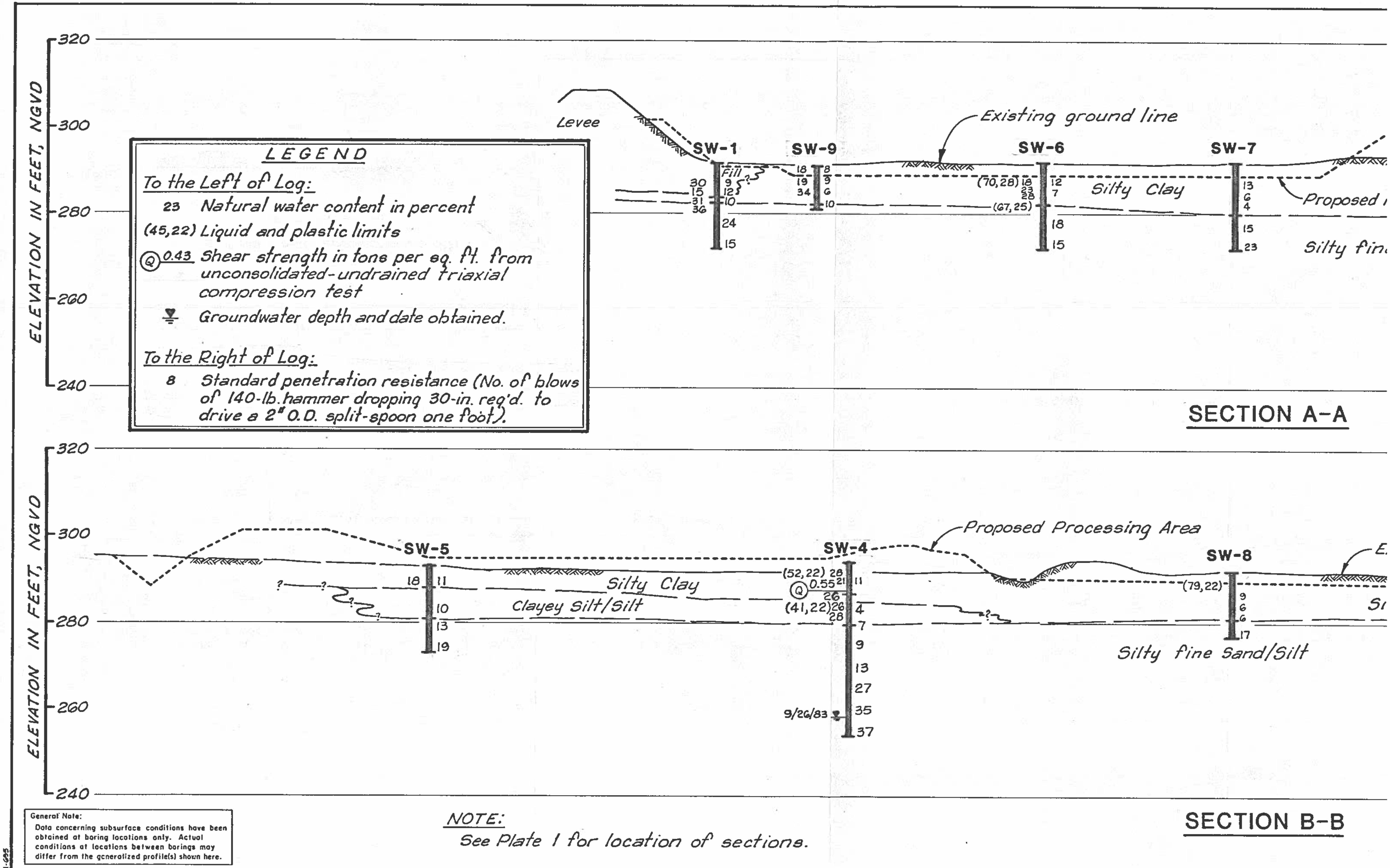
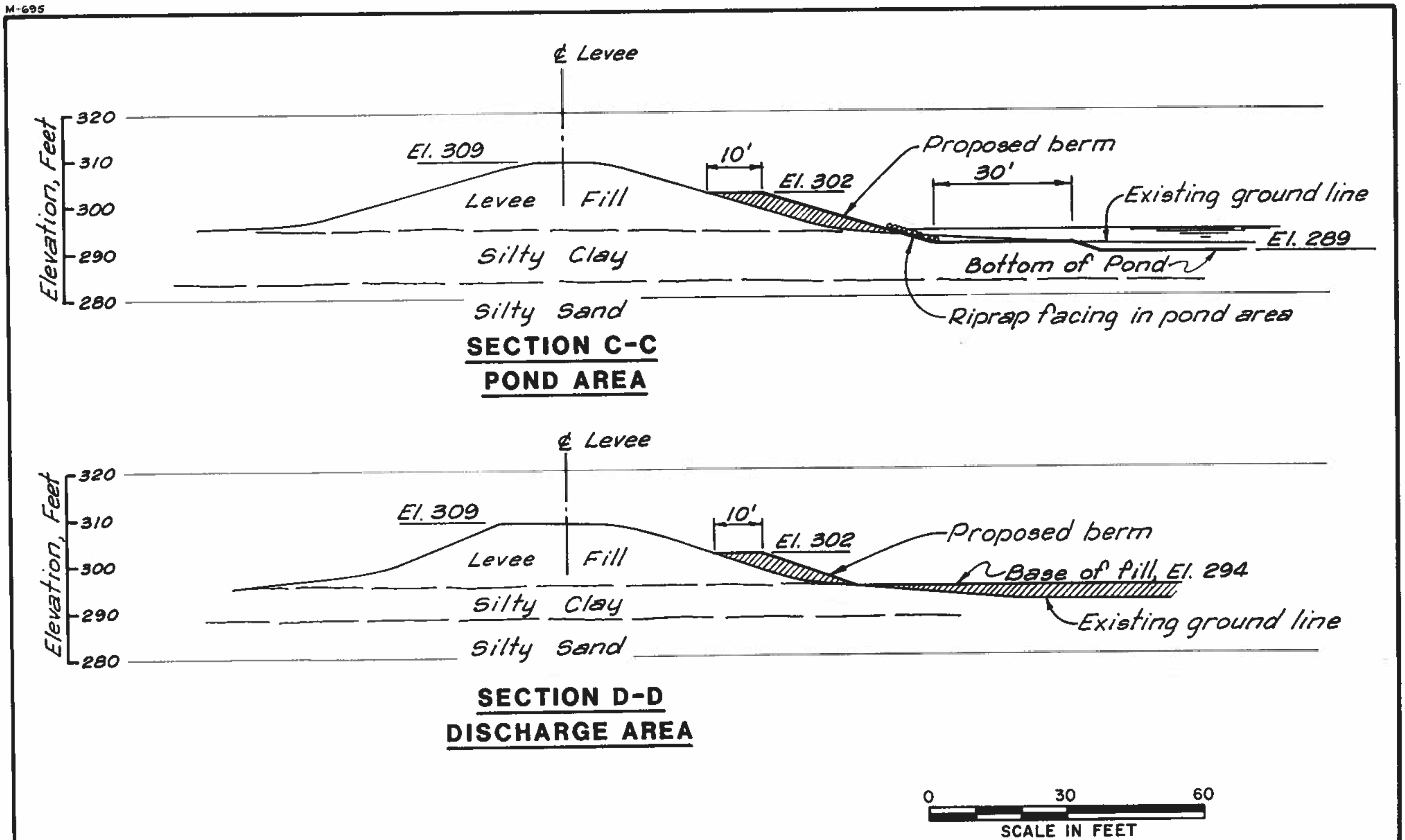
PLATE 1


M-695
ELEVATION IN FEET, NGVD
320
300
280
260
240
LEGEND
To the Left of Log:
23 Natural water content in percent
(45,22) Liquid and plastic limits
Q 0.43 Shear strength in tons per sq. ft. from unconsolidated-undrained triaxial compression test
Groundwater depth and date obtained.
To the Right of Log:
8 Standard penetration resistance (No. of blows of 140-lb. hammer dropping 30-in. req'd. to drive a 2" O.D. split-spoon one foot).
Levee
Existing ground line
SW-1
SW-9
SW-6
SW-7
Fill
Silty Clay
Proposed
Silty fin
(70,28)
(67,25)
SECTION A-A
SW-5
SW-4
SW-8
Proposed Processing Area
Silty Clay
Clayey Silt/Silt
Silty fine Sand/Silt
(52,22)
Q 0.55
(41,22)
(79,22)
9/26/83
SECTION B-B
General Note:
Data concerning subsurface conditions have been obtained at boring locations only. Actual conditions at locations between borings may differ from the generalized profile(s) shown here.
NOTE:
See Plate 1 for location of sections.



PROFILES THROUGH EXISTING LEVEE

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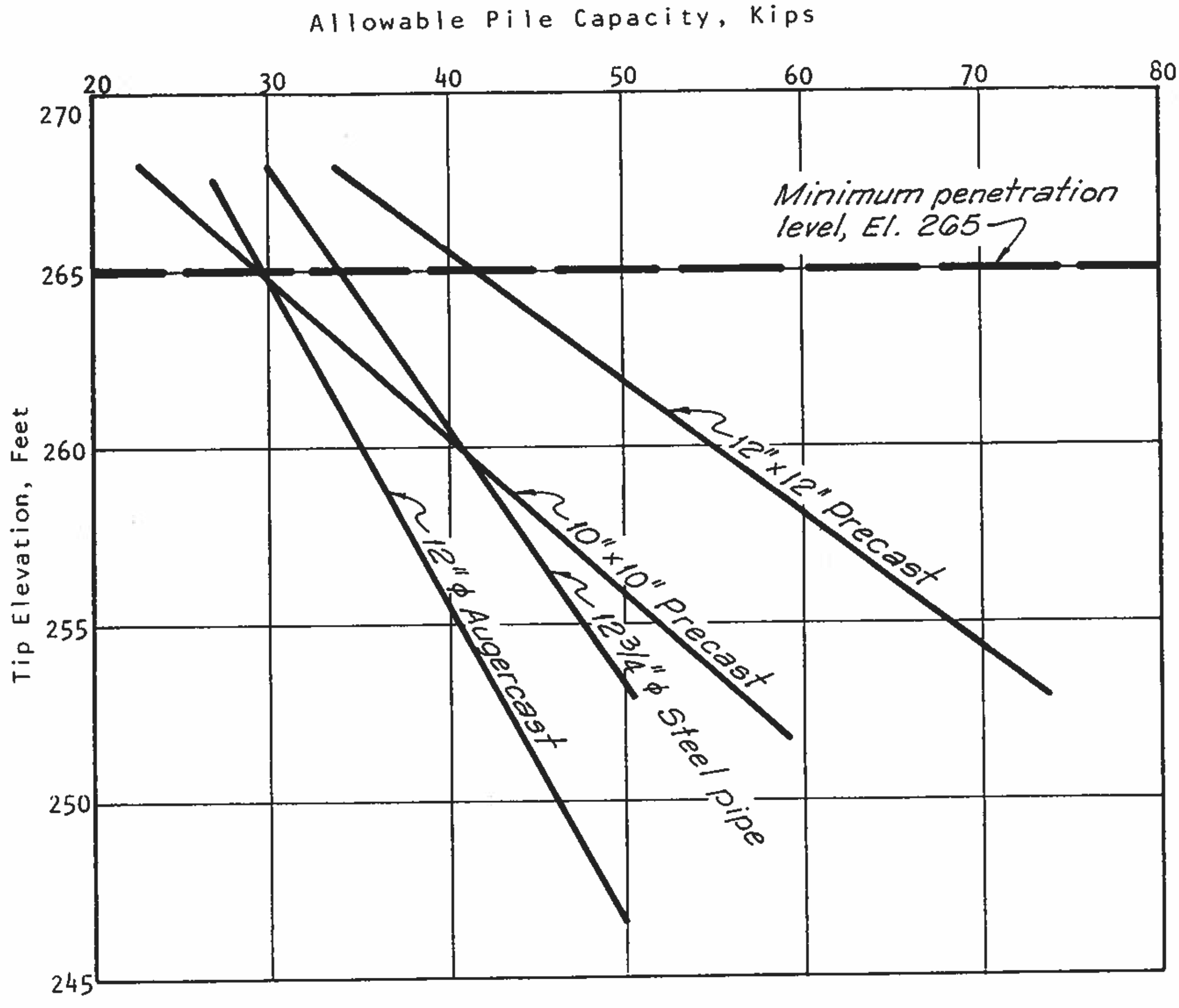

Note: All piles should penetrate below EL 265 regardless of design capacity to account for potential liquefaction of upper sands.

PILE CAPACITY VS TIP ELEVATION ONSHORE STRUCTURES

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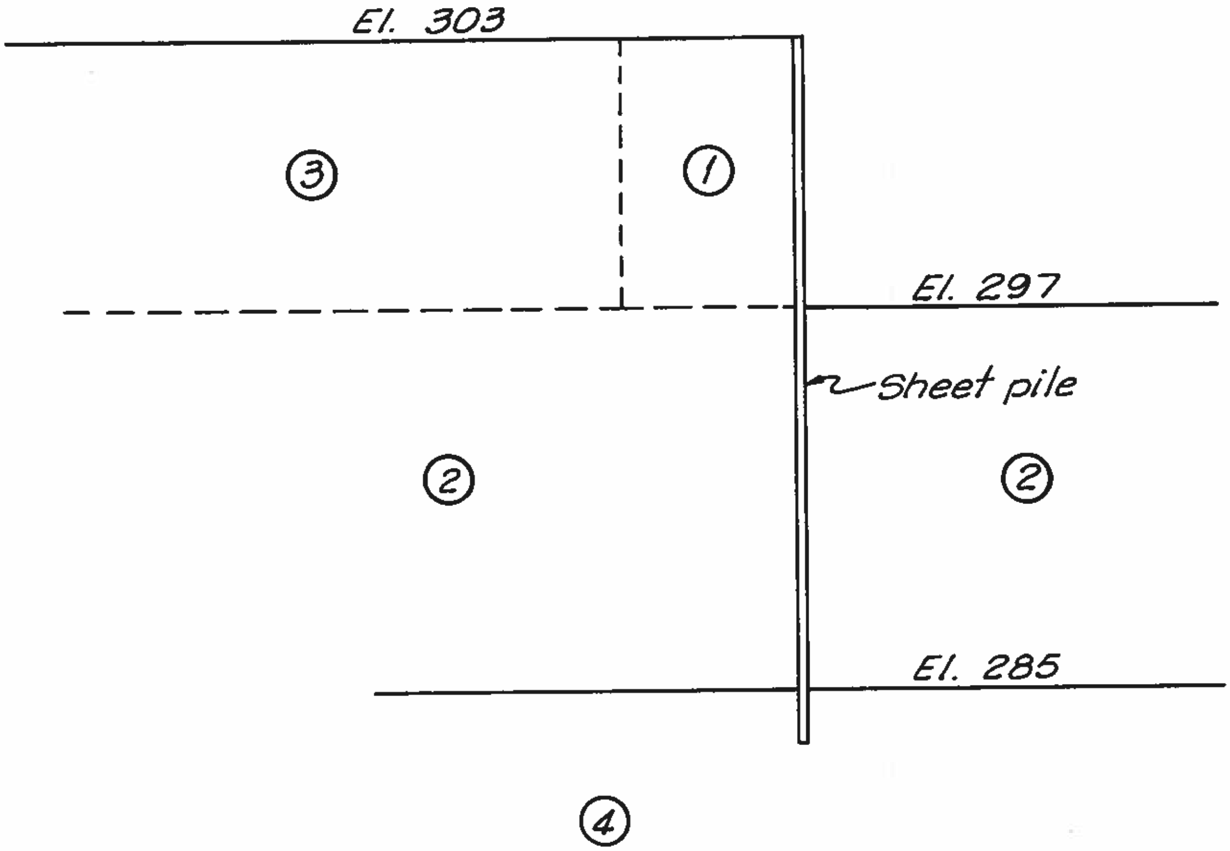

Materials:

(1) Slag (Wall should be backfilled for a distance of at least 4' from the wall with slag)

$c = 0$ $\phi = 34°$ $\gamma = 105$ pcf

(2) Silty Clay

$c = 1000$ psf $\phi = 0°$ $\gamma = 110$ pcf

(3) Fill - Silty Clay

$c = 1000$ psf $\phi = 0°$ $\gamma = 120$ pcf

(4) Silty Sand

liquefiable - assumes no support

SOIL PARAMETERS FOR DESIGN OF SHEET PILE RETAINING WALL

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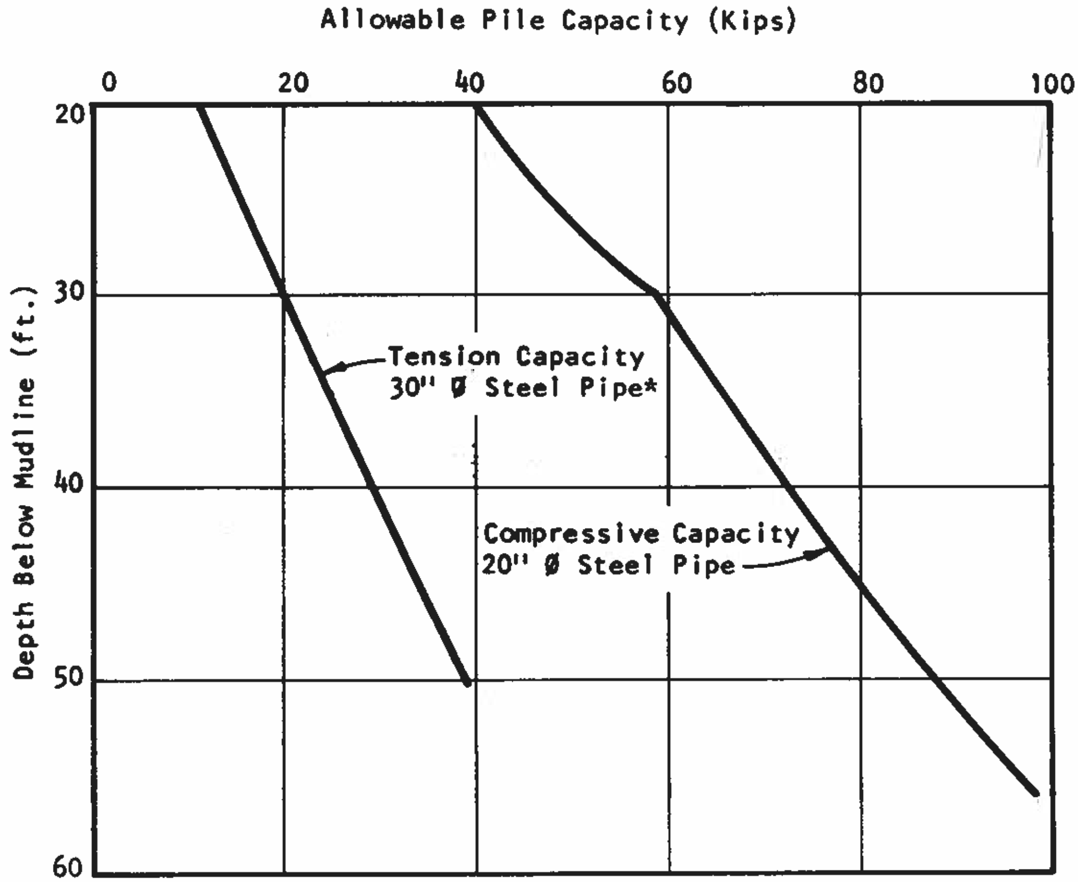
M-G95



* Does Not Include Weight of Pile.

PILE CAPACITY VERSUS DEPTH OF PENETRATION OFFSHORE STRUCTURES

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APPENDIX A

FIELD INVESTIGATION

APPENDIX A

FIELD INVESTIGATION

General

The field investigation was accomplished between September 7 and September 15, 1983. The field studies consisted of a program of soil borings in the process area, a brief inspection of a proposed borrow area, and a program of river soundings in the vicinity of the barge loading facility. A base line and a skew line parallel to the river were established to provide a means of control for the river soundings.

Borings

A total of nine borings was drilled in the process area ranging in depth from 10 to 40 feet. The borings were accomplished by Wabash Drilling Company of St. Louis under subcontract to Shannon & Wilson, Inc. An engineer from our firm was present throughout the drilling program to locate the borings, observe drilling procedures, assist in sampling, and maintain a detailed log of each boring. The locations shown on Plate 1 were established by pacing and measuring from existing property lines and physical features. Elevations at the borings were estimated from a topographic plan provided by AECI.

The borings were drilled with a truck-mounted Central Mining Equipment (CME) Model 55 drill rig. The borings were advanced using a four-inch diameter continuous solid flight auger. Boring 4, which was extended to 40 feet below the surface, was drilled to a depth of 13.5 feet using the solid auger. Below this level, rotary wash methods were used with a tri-cone drill bit. Casing was set to a depth of 8.5 feet and the bore hole was stabilized by the addition of an organic drilling fluid (Revert).

Standard Penetration Test (SPT) samples were taken at 2.5 to 5.0 foot intervals in the borings. In this test, a standard two-inch O.D. split spoon sampler is driven into the soil using a 140 pound weight falling a distance of 30 inches. The number of blows required to drive the sampler 18 inches is recorded in six-inch increments. The number of blows required for the last 12 inches of penetration is termed the Standard Penetration Test blow count or N-value. This value is reported on the boring logs, Plates A-1 through A-10. The soils retained in the SPT sampler were classified, placed in airtight glass jars, and returned to our St. Louis laboratory.

Relatively undisturbed samples of the soils were obtained by hydraulically pushing three-inch O.D. thin-walled steel tubes below the base of the advancing boring. The ends of the tubes were examined, sealed with wax, and transported along with the jar samples to our laboratory for more detailed inspection and testing.

A standpipe piezometer was installed in Boring 4 with its tip at 38.5 feet below the surface. The piezometer consisted of two-inch inside diameter Schedule 40 PVC pipe with external couplings. The lower four feet of the pipe was slotted using a hacksaw. Piezometer readings taken by our engineer during the course of the field program indicated a relatively high water table. This was attributed to the high viscosity of the organic drilling fluid. Subsequent readings taken by AECI personnel after the drilling fluid had "reverted" or lost its viscous characteristics, indicated a water level approximately 35 feet below the surface.

River Soundings

A series of soundings was conducted in the vicinity of the planned barge loading facility to provide general information on the configuration of the river bed in this area. The soundings were referenced to a survey line established along the shoreline which was tied to the plant coordinate system. Seven points were established along the line; offset stakes were set at approximate right angles to the axis of the river. The points and offset stakes were used to maintain approximate alignment as the soundings were made.

The soundings were conducted by subcontract to the Okie Moore Company using a Halliburton depth finder which was mounted on the side of a 26-foot work boat. An engineer from our firm was present to assist in the sounding operation. Horizontal control for the soundings was maintained by direct measurement from the shoreline using a hemp line which was marked at 20-foot intervals.

The accuracy of the survey was limited by the strong current of the river, coupled with the brisk wind which blew during the survey. These influences made it difficult to maintain the lateral alignment of the work boat while moving from the vicinity of the existing mooring cells back to the shore as the sounding equipment was operated. Variations in lateral alignment were estimated to be on the order of as much as 20 or 25 feet upstream and downstream. Attempts were made to account for this variation when plotting the data to draw the contours given on Plate 1. The effects of sag and stretch of the hemp line balanced to some degree. A measurement from the skew line to the existing cells was checked using an aerial photograph of the site; the two measurements differed by approximately six feet.

The intent of the survey was to provide a general approximation of the configuration of the river bed. These data should not be used for accurate determinations of pile lengths or other precise measurements. The river bed elevations are considered to be accurate within one foot. A tabulation of the sounding data is given on Table A-1.

Borrow Area Sampling

Surface samples of material from the Noranda stockpile were obtained during a brief inspection of the area. One sample was obtained from the sides of a gulley which been eroded at the north end of the stockpile. Another sample was taken from the east central portion of the stockpile, from an area which contained no crushed rock.

TABLE A-1

SUMMARY OF RIVER SOUNDINGS

LINE 1			LINE 2			LINE 3		
OFFSET FROM LINE A (ft)	DEPTH (ft)	ELEV. (ft)	OFFSET FROM LINE A (ft)	DEPTH (ft)	ELEV. (ft)	OFFSET FROM LINE A (ft)	DEPTH (ft)	ELEV. (ft)
170.0	22.5	234.0	190.0	20.5	236.0	195.0	25.0	231.5
150.0	22.5	234.0	180.0	22.5	234.0	180.0	22.0	234.5
130.0	16.5	240.0	160.0	23.2	233.3	160.0	17.0	239.5
125.0	14.2	242.3	140.0	22.0	234.5	140.0	15.2	241.3
110.0	14.0	242.5	120.0	15.5	241.0	120.0	13.3	243.2
90.0	11.0	245.5	100.0	12.0	244.5	100.0	11.7	244.8
70.0	8.5	248.0	80.0	10.0	246.5	80.0	10.2	246.3
50.0	6.5	250.0	60.0	7.5	249.0	60.0	8.0	248.5
34.5	0.0	256.5	40.0	4.5	252.0	40.0	5.0	251.5
			25.5	0.0	256.5	16.5	0.0	256.5

TABLE A-1

SUMMARY OF RIVER SOUNDINGS

OFFSET FROM LINE A (ft)	DEPTH (ft)	ELEV. (ft)	OFFSET FROM LINE A (ft)	DEPTH (ft)	ELEV. (ft)	OFFSET FROM LINE A (ft)	DEPTH (ft)	ELEV. (ft)	OFFSET FROM LINE A (ft)	DEPTH (ft)	ELEV. (ft)
225.0	23.0	233.5	250.0	24.7	231.8	200.0	23.0	233.5	212.0	20.5	236.0
205.0	22.5	234.0	230.0	25.2	231.3	180.0	22.4	234.1	200.0	20.0	236.5
192.0	22.0	234.5	220.0	25.7	230.8	160.0	21.7	234.8	180.0	20.2	236.3
185.0	19.0	237.5	210.0	24.0	232.5	143.0	22.0	234.5	160.0	20.7	235.8
165.0	17.1	239.4	190.0	19.2	237.3	140.0	21.3	235.2	140.0	20.8	235.7
145.0	16.8	239.7	170.0	15.8	240.7	120.0	19.0	237.5	120.0	15.9	240.6
125.0	16.0	240.5	150.0	13.0	243.5	100.0	13.8	242.7	100.0	12.0	244.5
105.0	13.5	243.0	130.0	12.0	244.5	80.0	10.3	246.2	80.0	10.0	246.5
85.0	12.0	244.5	110.0	8.7	247.8	72.0	8.8	247.7	60.0	7.2	249.3
65.0	8.0	248.5	90.0	7.0	249.5	60.0	8.1	248.4	40.0	4.5	252.0
45.0	3.5	253.0	70.0	4.1	252.4	40.0	5.4	251.1	22.5	0.0	256.5
32.0	0.0	256.5	55.5	0.0	256.5	23.5	0.0	256.5			

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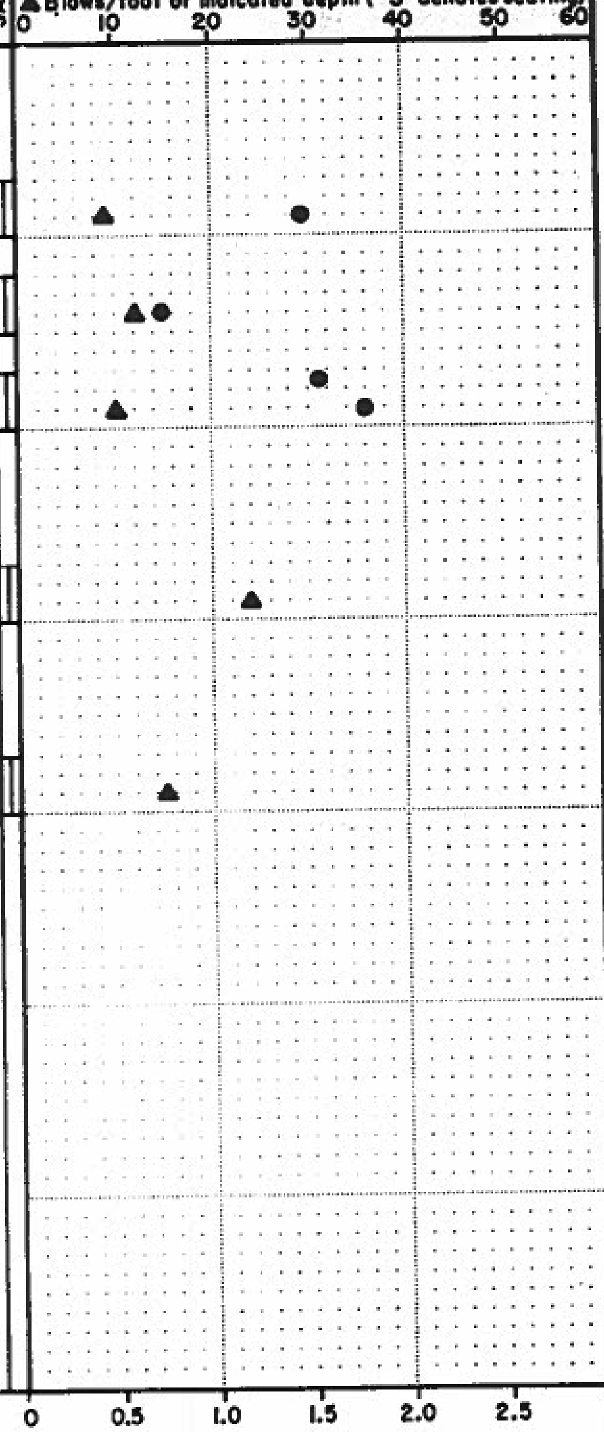
Surface Elevation & Datum: 292± MSL

Completion Depth: 20.0 ft **Completion Date:** 09/08/83

DEPTH IN FEET	DESCRIPTION OF MATERIAL	DEPTH IN FEET	SAMPLES
0–3.0	Loose to medium dense, black COAL and COAL SLAG mixed with SILTY CLAY (FILL).	3.0	
3.0–8.0	Stiff, brown to gray-brown, silty CLAY with weed particles and sand-size fragments of COAL SLAG (FILL).	8.0	
8.0–9.5	Stiff, gray mottled brown, silty CLAY	9.5	
9.5–20.0	Medium dense, brown, silty fine SAND with lenses and layers (6" - 12") of fine sandy SILT	20.0	
	Boring terminated at 20.0 ft. No water encountered during drilling.		

ATTERBERG LIMITS & WATER CONTENT DATA
Natural Water Content, %
Plastic Limit ├──●──┤ Liquid Limit

STANDARD PENETRATION RESISTANCE
(140-lb. weight / 30-in. drop)
▲ Blows/foot or indicated depth ("S" denotes seating)



SHEAR STRENGTH IN TONS PER SQ. FT. FROM INDICATED TEST

LEGEND

SAMPLER TYPES

- 3" O.D. Thin-wall tube
- 2" O.D. Split-spoon
- Rock core barrel
- * Sample not recovered
- %REC / %RQD
- ▽ Water level and date observed

SHEAR STRENGTH DATA

SYMBOL	TEST METHOD
○	Unconfined compression
△	Unconsolidated - undrained triaxial compression
□	Consolidated - undrained triaxial compression
⊕	Field vane
◇	Torvane
◆	Pocket penetrometer

NOTE: The stratification lines represent the approximate boundaries between soil types and the transition may be gradual.

LOG OF BORING SW-1

SHANNON & WILSON, INC.
GEOTECHNICAL CONSULTANTS

PLATE A-1

M-695

Surface Elevation & Datum: 292± MSL

Completion Depth: 20.0 ft Completion Date: 09/08/83

DEPTH IN FEET	DESCRIPTION OF MATERIAL	DEPTH IN FEET
0–11	Medium stiff to stiff, gray mottled brown, silty CLAY - with layers and lenses of silt and sandy silt.	11.0
11–20	Medium dense to dense, grayish brown, silty fine SAND interlayered with SILT. - becoming more silty below 19 ft.	20.0
20	Boring terminated at 20.0 ft. No water encountered during drilling.	

ATTERBERG LIMITS & WATER CONTENT DATA

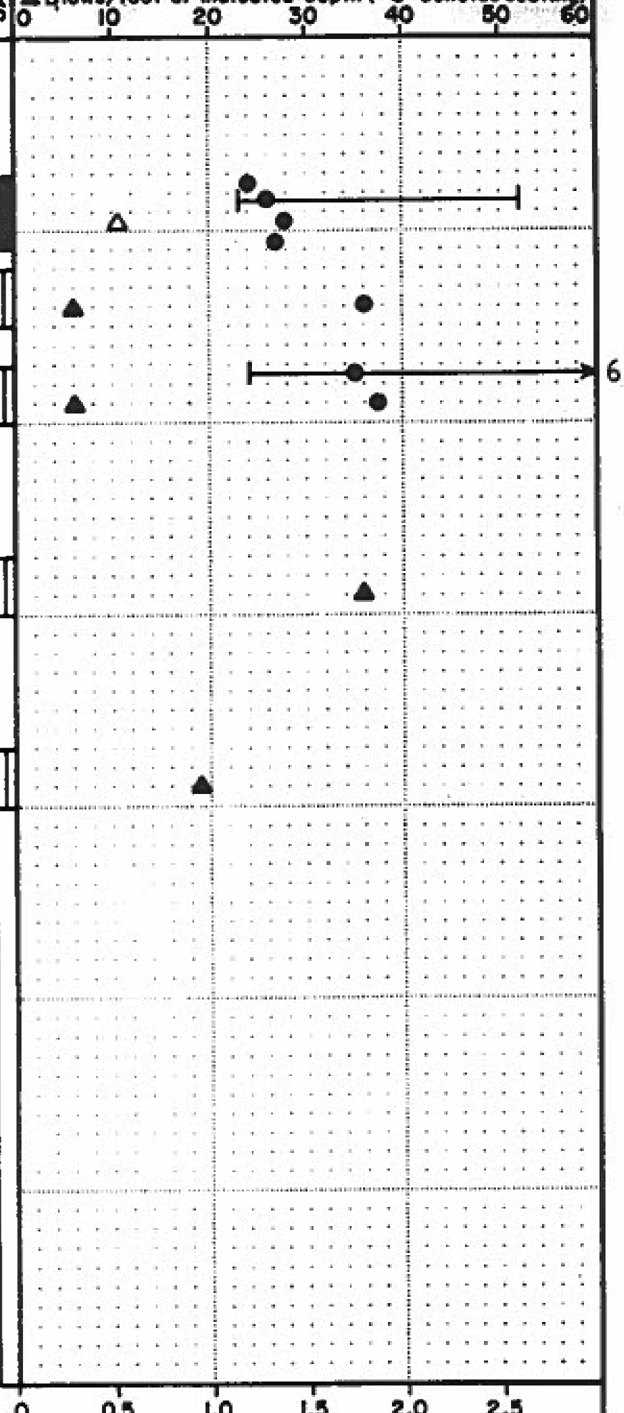
Natural Water Content, %

Plastic Limit ├──●──┤ Liquid Limit

STANDARD PENETRATION RESISTANCE

(140-lb. weight / 30-in. drop)

▲ Blows/foot or indicated depth ("S" denotes seating)



SHEAR STRENGTH IN TONS PER SQ. FT. FROM INDICATED TEST

LEGEND

SAMPLER TYPES

- 3" O.D. Thin-wall tube
- 2" O.D. Split-spoon
- Rock core barrel
- * Sample not recovered
- %REC / %RQD
- ▽ Water level and date observed

SHEAR STRENGTH DATA

SYMBOL	TEST METHOD
○	Unconfined compression
△	Unconsolidated - undrained triaxial compression
□	Consolidated - undrained triaxial compression
⊕	Field vane
◇	Torvane
◆	Pocket penetrometer

NOTE: The stratification lines represent the approximate boundaries between soil types and the transition may be gradual.

LOG OF BORING SW-2

SHANNON & WILSON, INC.
GEOTECHNICAL CONSULTANTS

PLATE A-2

M-695

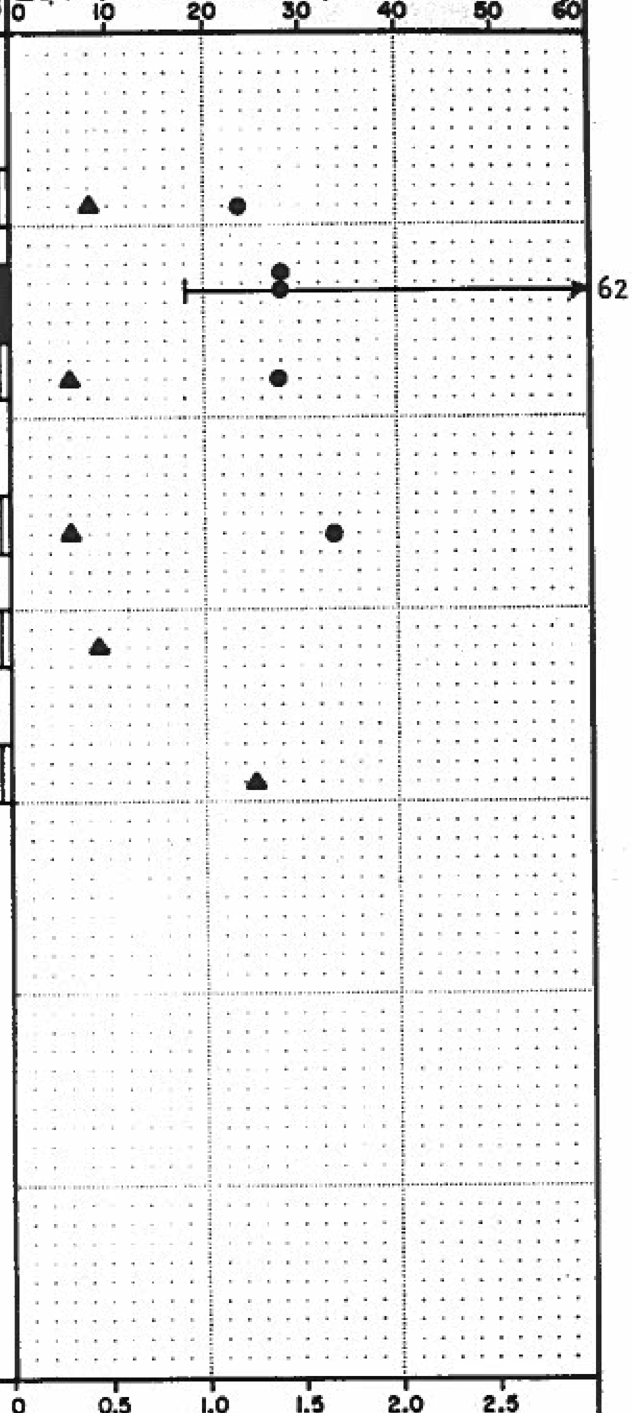
Surface Elevation & Datum: 294± MSL

Completion Depth: 20.0 ft Completion Date: 09/08/83

DEPTH IN FEET	DESCRIPTION OF MATERIAL	DEPTH IN FEET
	Brown, silty CLAY	0.9
5	Medium stiff to stiff, brown mottled gray, silty CLAY with occasional layers of silt. - increasing silt content	
10		14.0
15	Medium dense, brown, silty fine SAND to SILT	20.0
20	Boring terminated at 20.0 ft. No water encountered during drilling.	
25		
30		

ATTERBERG LIMITS & WATER CONTENT DATA
Natural Water Content, %
Plastic Limit ⊢—●—⊣ Liquid Limit

STANDARD PENETRATION RESISTANCE
(140-lb. weight / 30-in. drop)
▲ Blows/foot or indicated depth ("S" denotes seating)



SHEAR STRENGTH IN TONS PER SQ. FT. FROM INDICATED TEST

LEGEND

SAMPLER TYPES

- 3" O.D. Thin-wall tube
- 2" O.D. Split-spoon
- Rock core barrel
- * Sample not recovered
- %REC / %RQD
- ▽ Water level and date observed

SHEAR STRENGTH DATA

SYMBOL	TEST METHOD
○	Unconfined compression
△	Unconsolidated - undrained triaxial compression
□	Consolidated - undrained triaxial compression
⊕	Field vane
◇	Torvane
◆	Pocket penetrometer

NOTE: The stratification lines represent the approximate boundaries between soil types and the transition may be gradual.

LOG OF BORING SW-3

SHANNON & WILSON, INC.
GEOTECHNICAL CONSULTANTS

PLATE A-3

M-695

Surface Elevation & Datum: 294± MSL

Completion Depth: 40.0 ft Completion Date: 09/08/83

DEPTH IN FEET	DESCRIPTION OF MATERIAL	DEPTH IN FEET
	Brown, silty CLAY with root fragments and organic material	0.6
5	Stiff, brown to gray mottled, silty CLAY	7.5
10	Medium stiff, gray mottled brown, interlayered silty CLAY and clayey SILT to SILT.	14.5
15, 20, 25, 30	Medium dense to dense, brown, silty fine SAND - with fine to medium sand below 23 ft	

ATTERBERG LIMITS & WATER CONTENT DATA

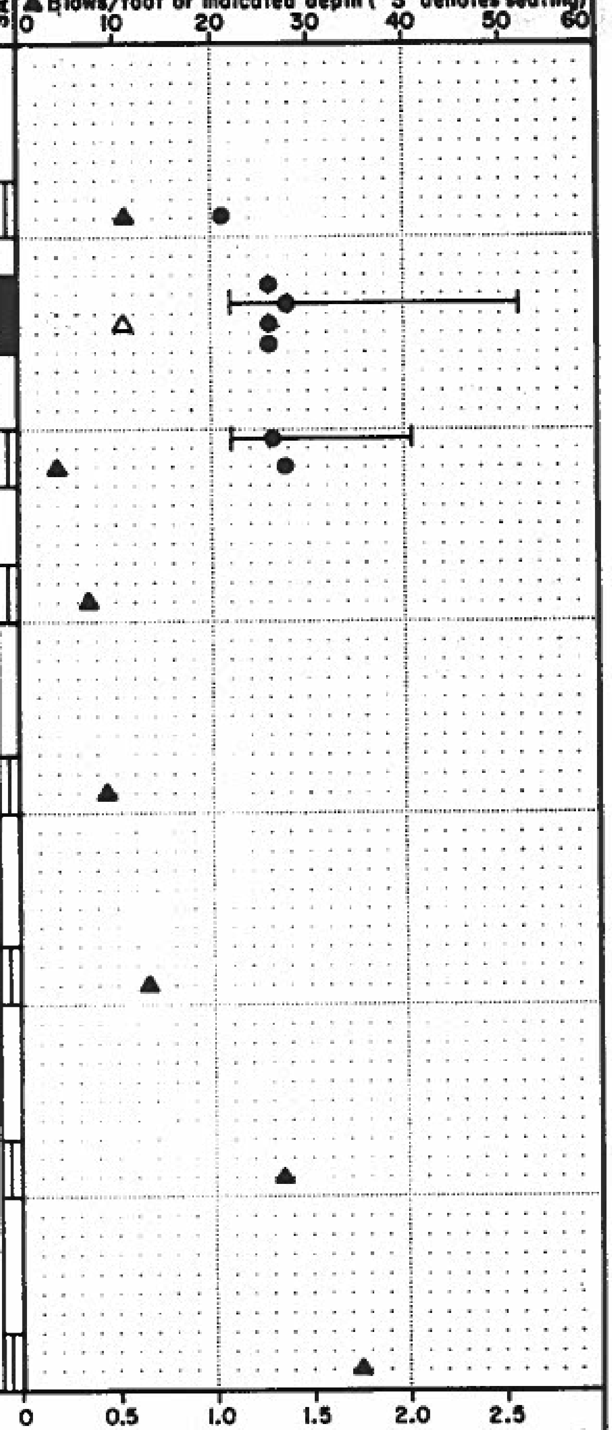
Natural Water Content, %

Plastic Limit ⊢—●—⊣ Liquid Limit

STANDARD PENETRATION RESISTANCE

(140-lb. weight / 30-in. drop)

▲ Blows/foot or indicated depth ("S" denotes seating)



SHEAR STRENGTH IN TONS PER SQ. FT. FROM INDICATED TEST

LEGEND

SAMPLER TYPES

- 3" O.D. Thin-wall tube
- 2" O.D. Split-spoon
- Rock core barrel
- * Sample not recovered
- %REC / %RQD
- ∇ Water level and date observed

SHEAR STRENGTH DATA

SYMBOL	TEST METHOD
○	Unconfined compression
△	Unconsolidated - undrained triaxial compression
□	Consolidated - undrained triaxial compression
⊕	Field vane
◇	Torvane
◆	Pocket penetrometer

NOTE: The stratification lines represent the approximate boundaries between soil types and the transition may be gradual.

LOG OF BORING SW-4

SHANNON & WILSON, INC.
GEOTECHNICAL CONSULTANTS

PLATE A-4

M-695

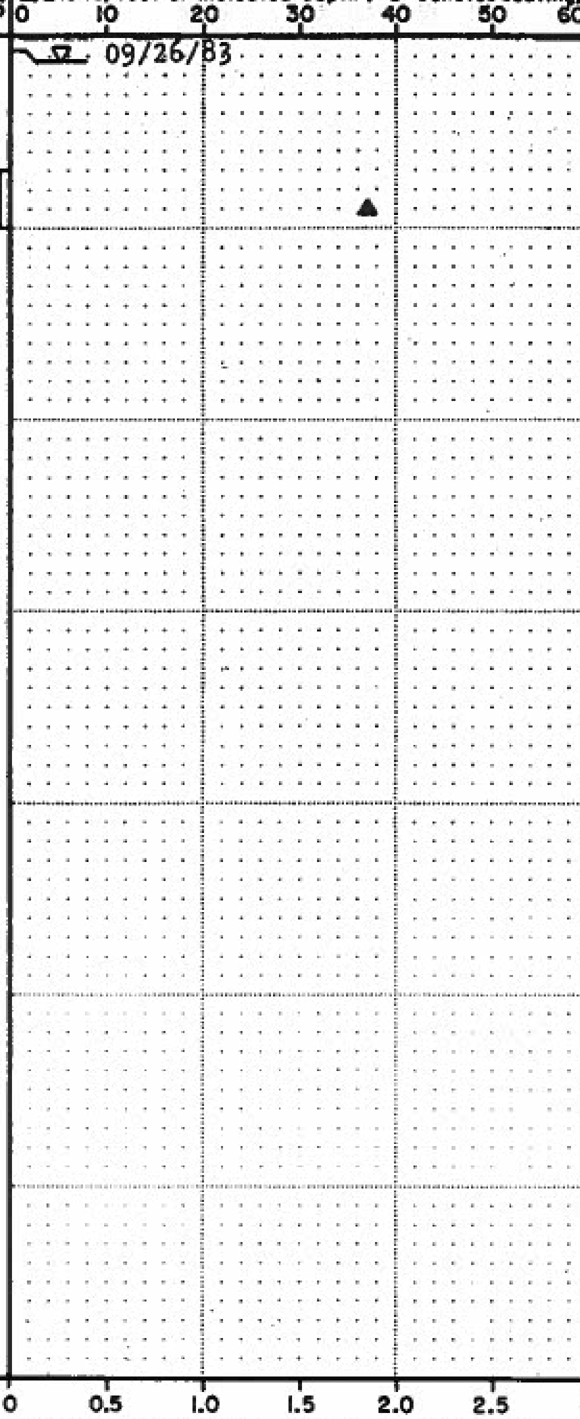
Surface Elevation & Datum: [illegible]

Completion Depth: 40.0 ft Completion Date: 09/08/83

DEPTH IN FEET	DESCRIPTION OF MATERIAL	DEPTH IN FEET	SAMPLES
	Dense, brown, silty fine to medium SAND [illegible]	[illegible]	
40		40.0	
	Boring terminated at 40.0 ft. 2" piezometer installed to depth of 38.1 ft.		

ATTERBERG LIMITS & WATER CONTENT DATA
Natural Water Content, %
Plastic Limit ⊢—●—⊣ Liquid Limit

STANDARD PENETRATION RESISTANCE
(140-lb. weight / 30-in. drop)
▲ Blows/foot or indicated depth ("S" denotes seating)



SHEAR STRENGTH IN TONS PER SQ. FT. FROM INDICATED TEST

LEGEND

SAMPLER TYPES

- 3" O.D. Thin-wall tube
- 2" O.D. Split-spoon
- Rock core barrel
- * Sample not recovered
- %REC / %RQD
- ▽ Water level and date observed

SHEAR STRENGTH DATA

SYMBOL	TEST METHOD
○	Unconfined compression
△	Unconsolidated - undrained triaxial compression
□	Consolidated - undrained triaxial compression
⊕	Field vane
◇	Torvane
◆	Pocket penetrometer

NOTE: The stratification lines represent the approximate boundaries between soil types and the transition may be gradual.

LOG OF BORING SW-4 (Cont'd.)

SHANNON & WILSON, INC.
GEOTECHNICAL CONSULTANTS

W-695

Surface Elevation & Datum: 293± MSL

Completion Depth: 20.0 ft Completion Date: 09/09/83

DEPTH IN FEET	DESCRIPTION OF MATERIAL	DEPTH IN FEET
	Brown, silty CLAY with occasional roots.	0.5
	Stiff, gray mottled brown, fine sandy, silty CLAY.	
5	Stiff, gray mottled brown, fine sandy SILT to SILT with layers of silty fine SAND	5.0
10		
	- increasing sand	
15		
20	Boring terminated at 20.0 ft. No water encountered during drilling	20.0
25		
30		

ATTERBERG LIMITS & WATER CONTENT DATA

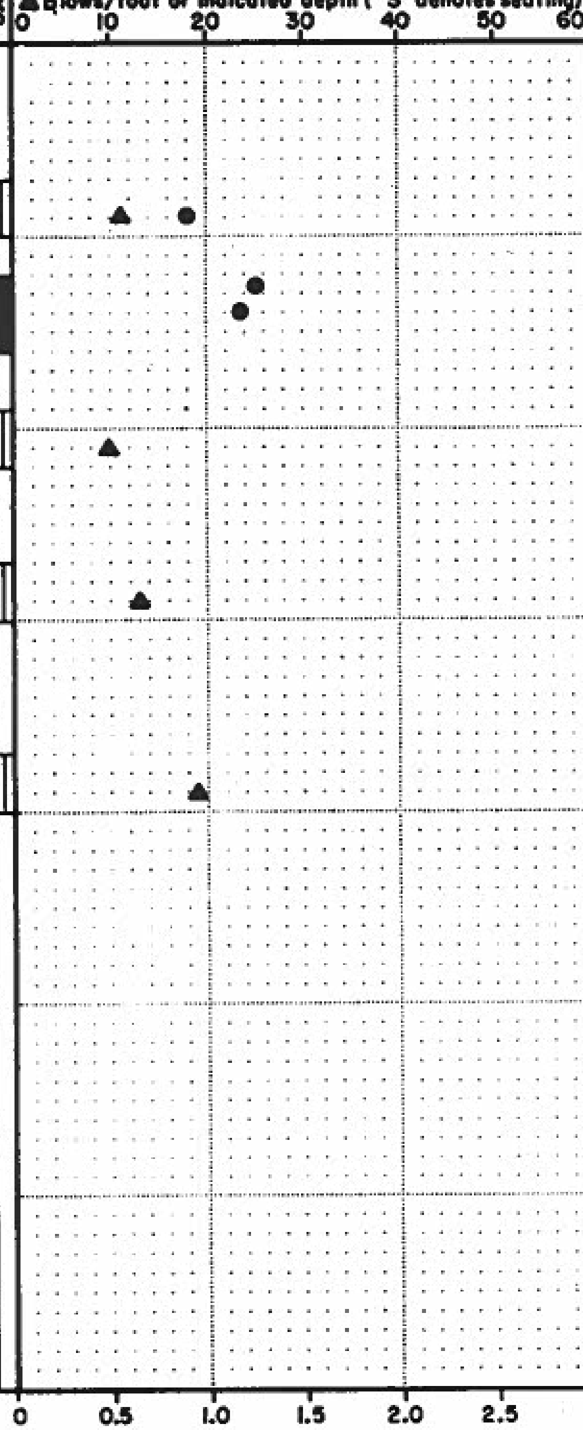
Natural Water Content, %

Plastic Limit ├──●──┤ Liquid Limit

STANDARD PENETRATION RESISTANCE

(140-lb. weight / 30-in. drop)

▲ Blows/foot or indicated depth ("S" denotes seating)



SHEAR STRENGTH IN TONS PER SQ. FT. FROM INDICATED TEST

L E G E N D

SAMPLER TYPES

- 3" O.D. Thin-wall tube
- 2" O.D. Split-spoon
- Rock core barrel
- * Sample not recovered
- %REC / %RQD
- ▽ Water level and date observed

SHEAR STRENGTH DATA

SYMBOL	TEST METHOD
○	Unconfined compression
△	Unconsolidated - undrained triaxial compression
□	Consolidated - undrained triaxial compression
⊕	Field vane
◇	Torvane
◆	Pocket penetrometer

NOTE: The stratification lines represent the approximate boundaries between soil types and the transition may be gradual.

LOG OF BORING SW-5

SHANNON & WILSON, INC.
GEOTECHNICAL CONSULTANTS

M-695

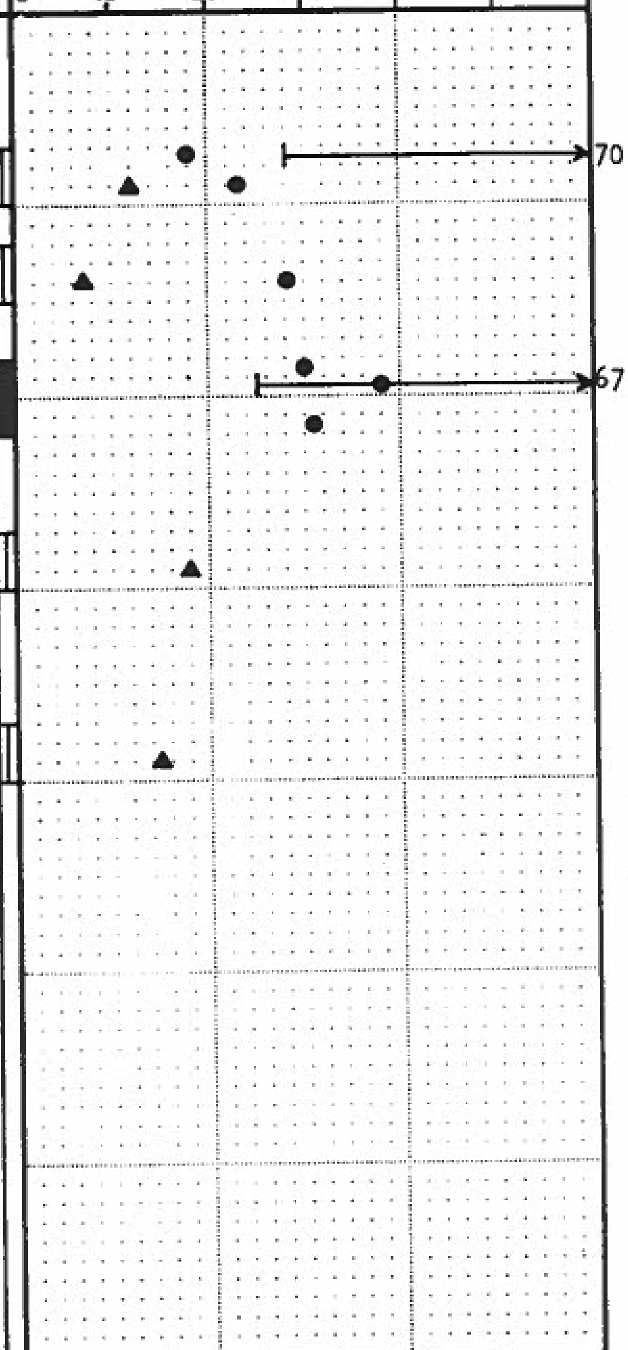
Surface Elevation & Datum: 292± MSL

Completion Depth: 20.0 ft Completion Date: 09/09/83

DEPTH IN FEET	DESCRIPTION OF MATERIAL	DEPTH IN FEET	SAMPLES
	Loose, brown, silty CLAY (plowed)	0.5	
	Stiff, gray mottled brown, silty CLAY.		
5	- grading to medium stiff		
10	Medium dense, gray to gray mottled brown, silty fine SAND interlayered with coarse SILT and silty CLAY.	9.8	
15			
20	Boring terminated at 20.0 ft. No water encountered during drilling.	20.0	
25			
30			

ATTERBERG LIMITS & WATER CONTENT DATA
Natural Water Content, %
Plastic Limit ├──●──┤ Liquid Limit

STANDARD PENETRATION RESISTANCE
(140-lb. weight / 30-in. drop)
▲ Blows/foot or indicated depth ("S" denotes seating)
0 10 20 30 40 50 60



0 0.5 1.0 1.5 2.0 2.5
SHEAR STRENGTH IN TONS PER SQ. FT. FROM INDICATED TEST

LEGEND

SAMPLER TYPES

- 3" O.D. Thin-wall tube
- 2" O.D. Split-spoon
- Rock core barrel
- * Sample not recovered
- %REC / %RQD
- Water level and date observed

SHEAR STRENGTH DATA

SYMBOL	TEST METHOD
○	Unconfined compression
△	Unconsolidated - undrained triaxial compression
□	Consolidated - undrained triaxial compression
⊕	Field vane
◇	Torvane
◆	Pocket penetrometer

NOTE: The stratification lines represent the approximate boundaries between soil types and the transition may be gradual.

LOG OF BORING SW-6

SHANNON & WILSON, INC.
GEOTECHNICAL CONSULTANTS

M-695

Surface Elevation & Datum: 292± MSL

Completion Depth: 20.0 ft Completion Date: 09/09/83

DEPTH IN FEET	DESCRIPTION OF MATERIAL	DEPTH IN FEET	SAMPLES
	Brown, silty CLAY (plowed)	0.5	
	Very stiff to hard, gray very silty CLAY.	4.0	
5	Very stiff, gray mottled brown, silty CLAY		
10	- becoming more silty with sand below 11 ft.	13.0	
15	Medium dense, gray mottled brown, silty fine SAND to SILT.	20.0	
20	Boring terminated at 20.0 ft. No water encountered during drilling.		
25			
30			

ATTERBERG LIMITS & WATER CONTENT DATA

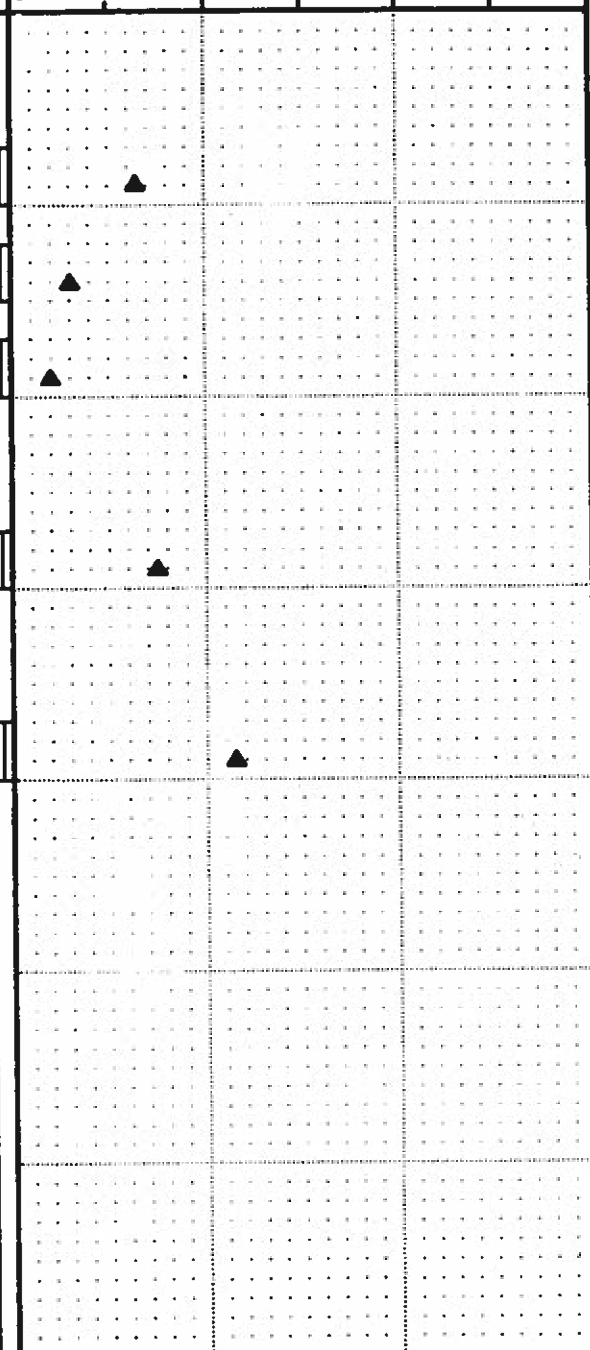
Natural Water Content, %

Plastic Limit ├──●──┤ Liquid Limit

STANDARD PENETRATION RESISTANCE

(140-lb. weight / 30-in. drop)

▲ Blows/foot or indicated depth ("S" denotes seating)



SHEAR STRENGTH IN TONS PER SQ. FT. FROM INDICATED TEST

LEGEND

SAMPLER TYPES

- 3" O.D. Thin-wall tube
- 2" O.D. Split-spoon
- Rock core barrel
- * Sample not recovered
- %REC / %RQD
- ∇ Water level and date observed

SHEAR STRENGTH DATA

SYMBOL	TEST METHOD
○	Unconfined compression
△	Unconsolidated - undrained triaxial compression
□	Consolidated - undrained triaxial compression
⊕	Field vane
◇	Torvane
◆	Pocket penetrometer

NOTE: The stratification lines represent the approximate boundaries between soil types and the transition may be gradual.

LOG OF BORING SW-7

SHANNON & WILSON, INC.
GEOTECHNICAL CONSULTANTS

M-695

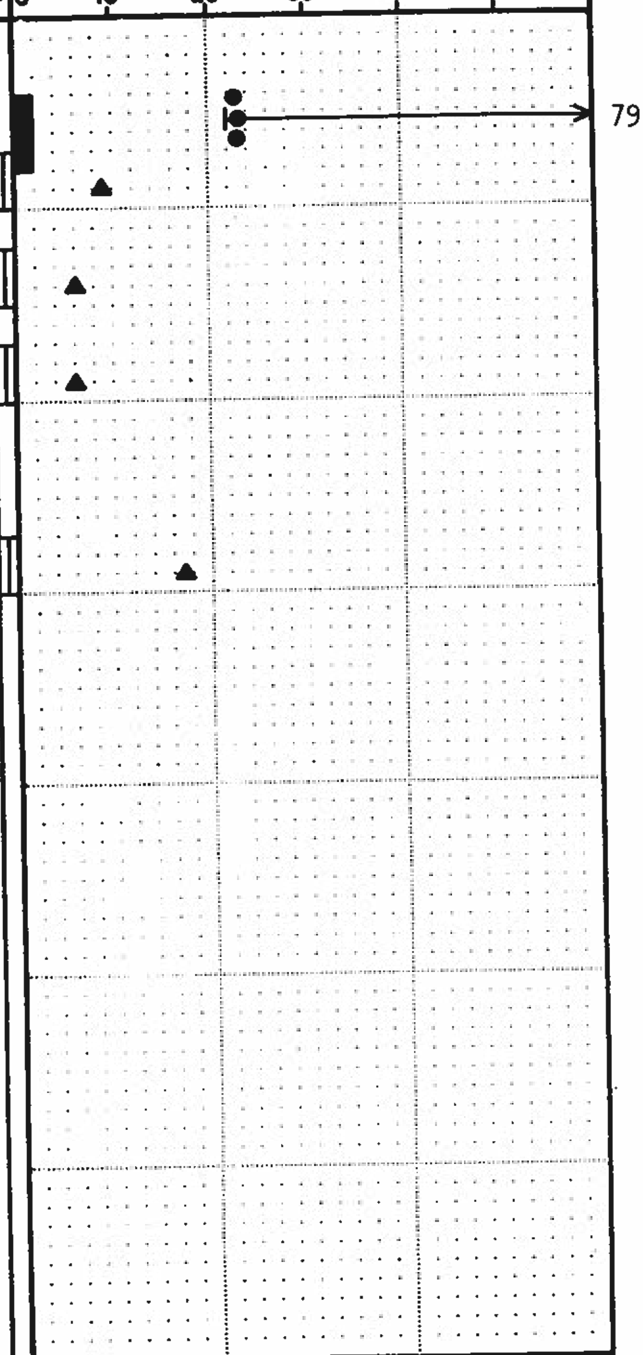
Surface Elevation & Datum: 292± MSL

Completion Depth: 15.0 ft Completion Date: 09/09/83

DEPTH IN FEET	DESCRIPTION OF MATERIAL	DEPTH IN FEET
	Brown, silty CLAY (plowed)	0.5
	Hard, light brown CLAY	4.0
5	Medium stiff, gray mottled brown, silty CLAY	
10		11.0
	Medium dense, brown, interlayered silty fine SAND and SILT	15.0
15	Boring terminated at 15.0 ft. No water encountered during drilling.	
20	3" thin-wall sample taken 3' N of completed boring.	
25		
30		

ATTERBERG LIMITS & WATER CONTENT DATA
Natural Water Content, %
Plastic Limit —●— Liquid Limit

STANDARD PENETRATION RESISTANCE
(140-lb. weight / 30-in. drop)
▲ Blows/foot or indicated depth ("S" denotes seating)



SHEAR STRENGTH IN TONS PER SQ. FT. FROM INDICATED TEST

LEGEND

SAMPLER TYPES
- 3" O.D. Thin-wall tube
- 2" O.D. Split-spoon
- Rock core barrel
- * Sample not recovered
- %REC / %RQD
- ▽ Water level and date observed

SHEAR STRENGTH DATA

SYMBOL	TEST METHOD
○	Unconfined compression
△	Unconsolidated - undrained triaxial compression
□	Consolidated - undrained triaxial compression
⊕	Field vane
◇	Torvane
◆	Pocket penetrometer

NOTE: The stratification lines represent the approximate boundaries between soil types and the transition may be gradual.

LOG OF BORING SW-8

SHANNON & WILSON, INC.
GEOTECHNICAL CONSULTANTS

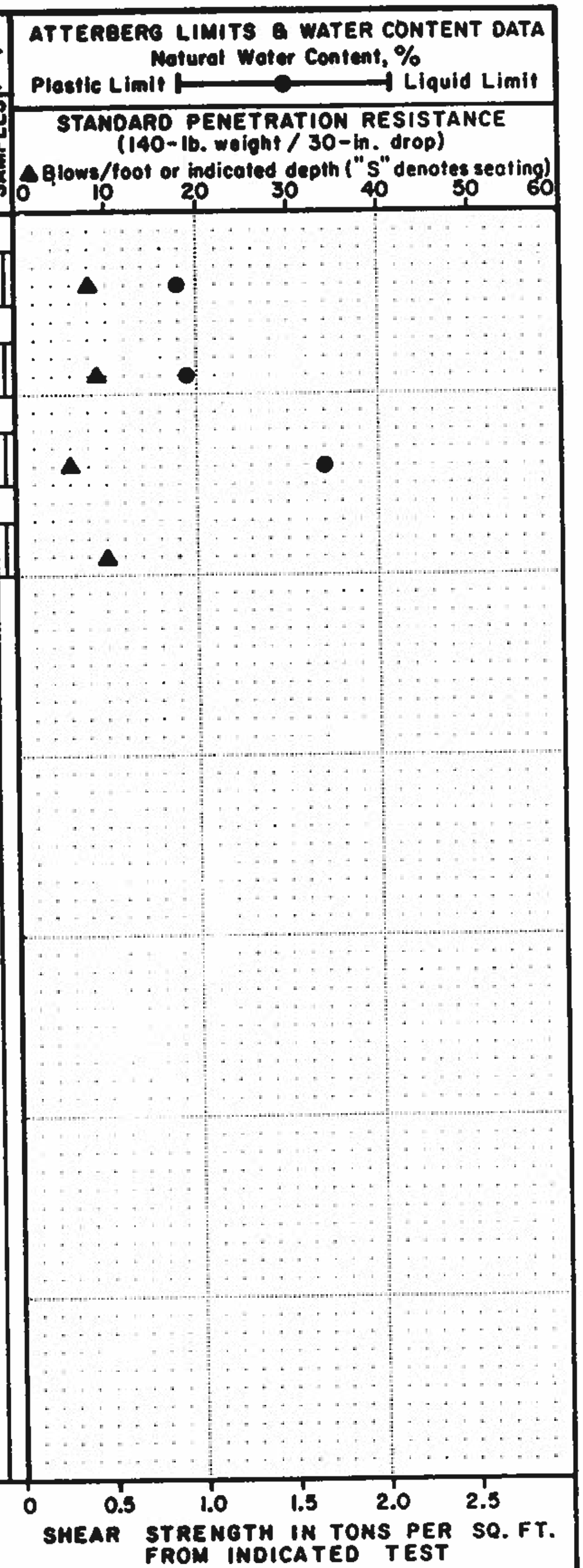
PLATE A-9

M-695

Surface Elevation & Datum: 291± MSL

Completion Depth: 10.0 ft Completion Date: 09/09/83

DEPTH IN FEET	DESCRIPTION OF MATERIAL	DEPTH IN FEET	SAMPLES
	Light brown, silty CLAY with roots	0.5	
	Medium stiff to stiff, gray mottled brown, silty CLAY with a layer of very silty CLAY at 3 ft.		
5			
	Medium dense, grayish brown, inter-layered silty fine SAND to SILT	9.0	
10		10.0	
	Boring terminated at 10.0 ft. No water encountered during drilling		
15			
20			
25			
30			



LEGEND

SAMPLER TYPES

- 3" O.D. Thin-wall tube
- 2" O.D. Split-spoon
- Rock core barrel
- * Sample not recovered
- %REC / %RQD
- ▽ Water level and date observed

SHEAR STRENGTH DATA

SYMBOL	TEST METHOD
○	Unconfined compression
△	Unconsolidated - undrained triaxial compression
□	Consolidated - undrained triaxial compression
⊕	Field vane
◇	Torvane
◆	Pocket penetrometer

NOTE: The stratification lines represent the approximate boundaries between soil types and the transition may be gradual.

LOG OF BORING SW-9

SHANNON & WILSON, INC.
GEOTECHNICAL CONSULTANTS

PLATE A-10

APPENDIX B

LABORATORY TESTING

APPENDIX B

LABORATORY TESTING

General

Samples transported from the site were examined in our laboratory to check the field classification. Tube samples were extruded and portions were retained for possible testing. A laboratory test program was conducted on the samples, including water content determinations, Atterberg limits tests, grain size analyses, density determinations, unconsolidated undrained triaxial compression tests, and permeability determinations. Compaction tests were performed on the bulk samples from the Noranda stockpile.

Classification Tests

Water Content. Each sample of cohesive soil was tested for its natural water content as an aid in soil classification and evaluation of soil properties. Water contents, reported as a percentage of the dry weight of the soil, are plotted on the boring logs presented in Appendix A.

Atterberg Limits. Atterberg limits were performed on selected samples obtained from the borings to assist in classification of the soil and as an indication of engineering properties. Standard ASTM procedures were used in the performance of these tests. Results of the determinations are presented on the boring logs.

Density Tests. Density measurements were performed on samples which were selected for triaxial compression tests and permeability tests, to assist in evaluating test results and to provide data for analyses. In this test, the dimensions and weight of the undisturbed test specimens are carefully determined after they were extruded from Shelby tubes, and the data are used to calculate the wet unit weight of the specimens. Moisture data are then used to determine the dry density. The densities are listed in Table B-1, Summary of Test Results.

Grain-Size Characteristics. Wash sieve analyses were performed on selected samples of the soils to determine the grain-size distribution and fines content of these materials. The results of these analyses are shown graphically on Plate B-1.

Triaxial Compression Tests

Two unconsolidated, undrained triaxial compression tests were performed on samples from the pump area and the hopper area. These tests are used to evaluate the shear strength parameters of soils under undrained conditions. In this test, the soil specimen is encased in a rubber membrane and placed in a triaxial chamber. A confining pressure is imposed within the chamber. No drainage is allowed to take place within the sample. After the confining pressure is applied, the specimen is tested to failure by increasing the axial load at a constant strain rate. The undrained shear strengths of the samples as obtained from these tests are given on the boring logs.

Permeability Tests

These tests were performed in the triaxial chamber by encasing the specimen in a rubber membrane and imposing a preselected confining pressure within the chamber. A hydraulic gradient is then imposed between the top and base of the sample by applying a differential water pressure across the sample and then measuring the volume of water which flows through the sample over measured time intervals. The results of the permeability determinations are given in Table B-1.

Compaction Tests

Two compaction tests were performed on samples from the Noranda stockpile. The tests were performed in accordance with ASTM D 1557 (Modified Proctor) procedures. The results are shown on Plates B-2 and B-3.

TABLE B-I

SUMMARY OF SOIL TEST RESULTS

BORING NO.	SAMPLE NO.	DEPTH	CLASSIFICATION	WATER CONTENT %	ATTERBERG LIMITS			SHEAR STRENGTH		UNIT DRY WEIGHT LB/CU FT	CONSOLIDATION	MECHANICAL ANALYSIS	REMARKS
					LL	PL	PI	STRESS TON/SQ FT	STRAIN %				
SW 2	1	3.5- 5.5	Stiff, gray mottled brown, CLAY	26	52	23	29						
				28				0.53	22.11	91.5			Triaxial compression
				28						89.9			k = 8.4 x 10^{-8} cm/sec
	3	8.5-10.0	Stiff, gray mottled brown slightly sandy CLAY	35	61	24	37						
												*	
SW 3	2	6.0- 8.0	Medium stiff, brown mottled gray, CLAY	28	62	18	44						
				29						92.3			k = 6.1 x 10^{-9} cm/sec
SW 4	2	6.0- 8.0	Stiff, gray mottled brown, CLAY	28	52	22	30						
				26				0.54	14.46	95.5			Triaxial compression
				26						93.6			k = 3.5 x 10^{-8} cm/sec
	3	10.0-11.5	Medium stiff, gray mottled brown, silty CLAY	27	41	22	19						
	7	28.5-30.0	Medium dense, brown, silty, fine to medium SAND									*	
SW 6	1	3.5- 5.0	Stiff, gray mottled brown, CLAY	18	70	28	42						
	3	9.0-11.0	Medium stiff, gray mottled brown, CLAY	38	67	25	42						

* SEE ATTACHED CURVE SHEETS

TABLE B-I

SUMMARY OF SOIL TEST RESULTS

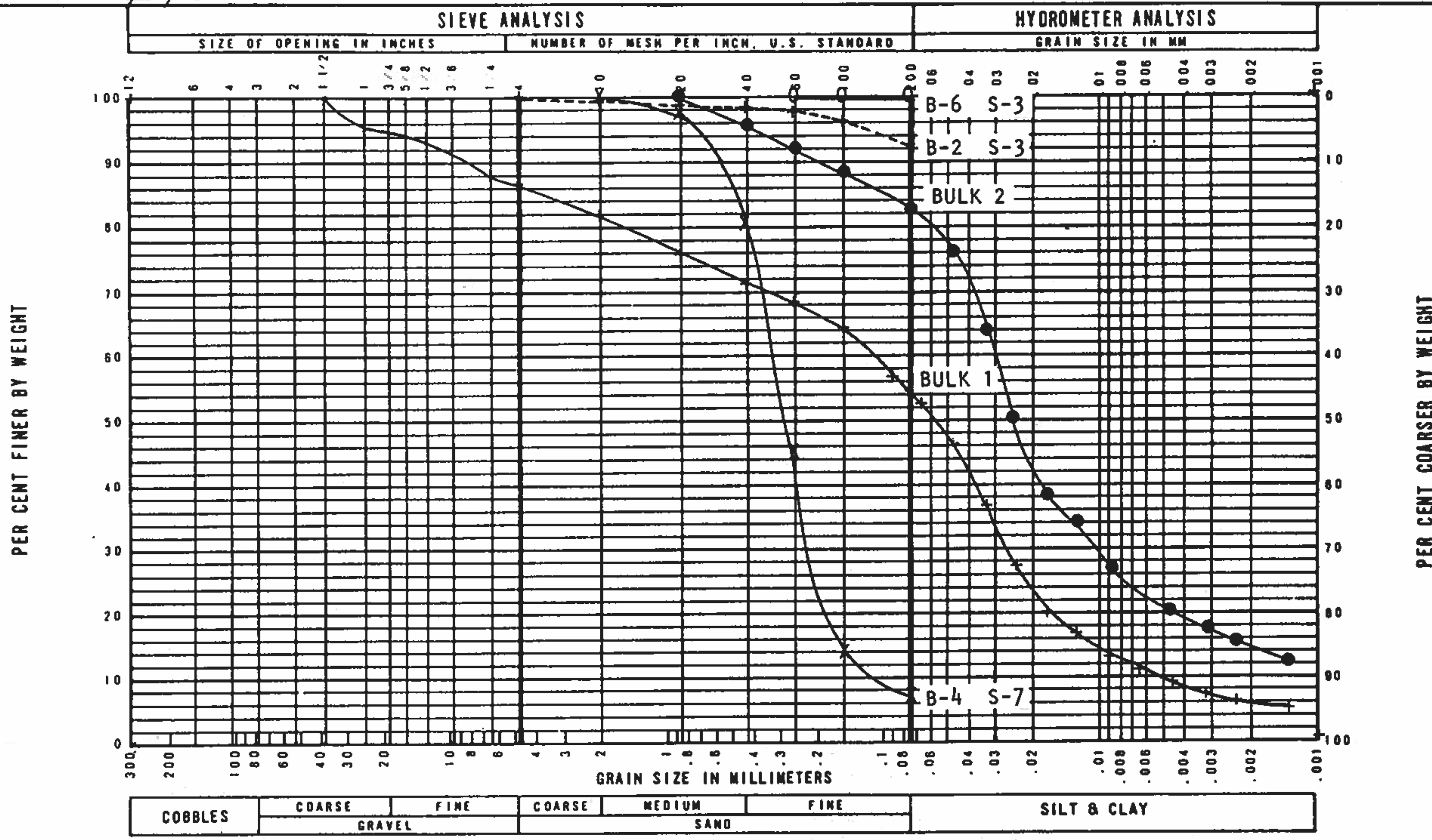
BORING NO.	SAMPLE NO.	DEPTH	CLASSIFICATION	WATER CONTENT %	ATTERBERG LIMITS LL	PL	PI	SHEAR STRENGTH STRESS TON/SQ FT	STRAIN %	UNIT DRY WEIGHT LB/CU FT	CONSOLIDATION	MECHANICAL ANALYSIS	REMARKS
SW 6	3	9.0-11.0	Medium stiff, gray mottled brown, CLAY									*	
				33						87.7			$k = 2.1 \times 10^{-6}$ cm/sec
SW 8	1A	2.0- 4.0	Hard, brown, CLAY	25	79	22	57						
				22						102.5			$k = 3.9 \times 10^{-9}$ cm/sec
BULK #1			Brown, gravelly, sandy, silty, CLAY/clayey SILT	2	23	18	5						
												*	Combined analysis
				10						113.8			$k = 2.1 \times 10^{-5}$ cm/sec
													Compaction *
BULK #2			Brown, sandy, very silty CLAY	9	34	23	11						
												*	Combined analysis
				12						95.2			$k = 8.8 \times 10^{-6}$ cm/sec
													Compaction *

* SEE ATTACHED CURVE SHEETS

Drawn AMC Checked______ Approved______ Revised______ Approved______

Date 9/23/83 Date______ Date______ Date______ Date______

M-695



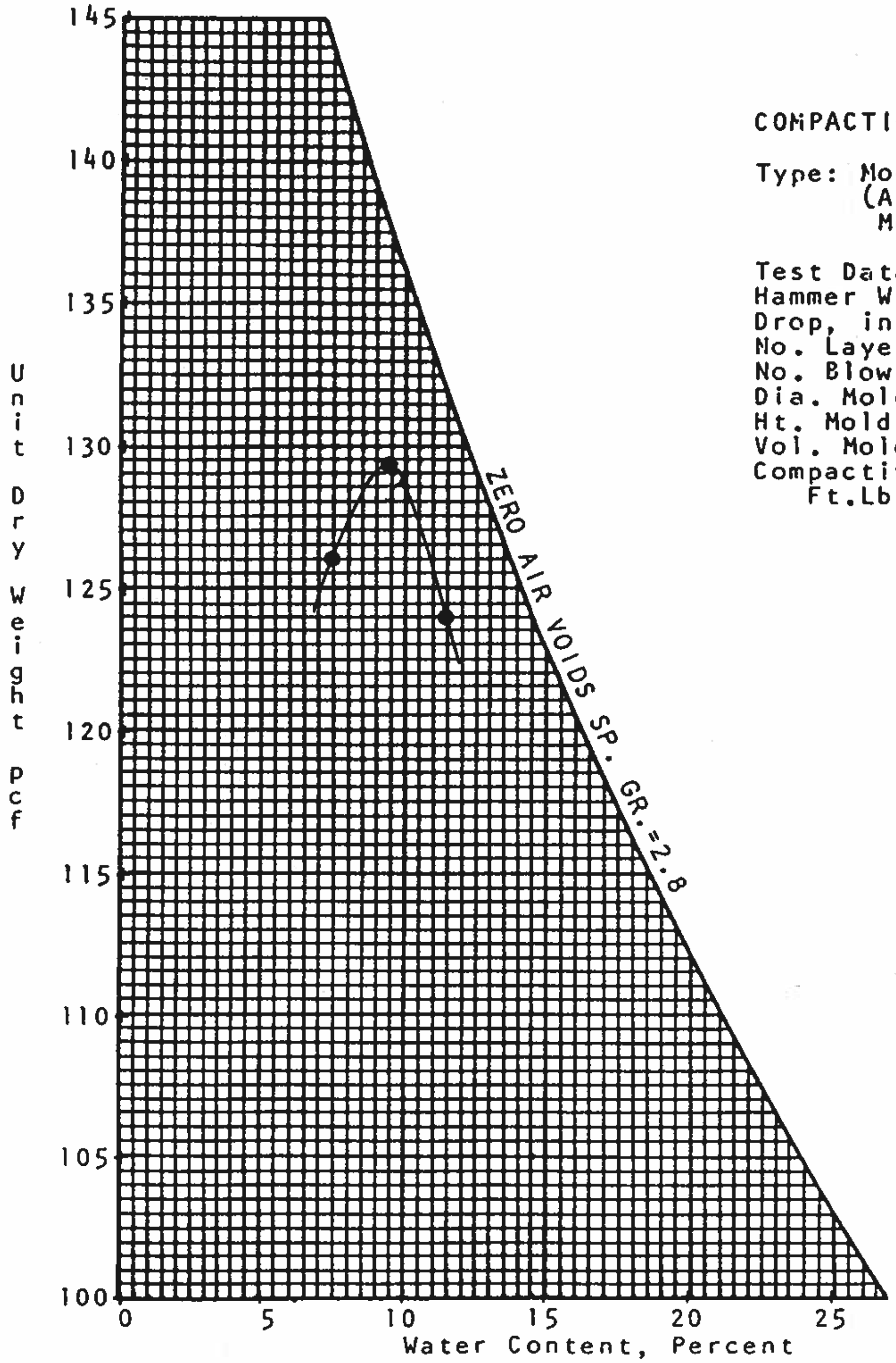
SAMPLE NO.	DEPTH-FT.	U.S.C.	CLASSIFICATION	G_s	NAT. W.C. %	LL	PL	PI
B-2 S-3	8.5-10.0	CH	Gray-brown, slightly sandy CLAY		35	61	24	37
B-4 S-7	28.5-30	SP-SM	Brown, slightly silty, fine to medium SAND					
B-6 S-3	9.0-11.0	CH	Gray-brown, silty CLAY		38	67	25	42
BULK 1		CL-ML	Brown, gravelly, sandy, silty CLAY/clayey SILT		2	23	18	5
BULK 2		CL	Brown, sandy, very silty CLAY		9	34	23	11

GRAIN SIZE CLASSIFICATION

SHANNON & WILSON, INC.
GEOTECHNICAL CONSULTANTS

PLATE B-1

M-695



COMPACTION TEST

Type: Modified AASHO
(ASTM D-1557-74)
Method C

Test Data	
Hammer Wt., lbs.	10.0
Drop, in.	18
No. Layers	5
No. Blows/Layer	25
Dia. Mold, in.	3.99
Ht. Mold, in.	4.58
Vol. Mold, cu.ft.	0.0331
Compactive Effort Ft.Lbs./Cu.Ft.	56300

Maximum Dry Density	129.4 pcf
Optimum Water Content	9.5 %
Natural Water Content	2.7 %

Classification:

Brown gravelly, sandy, silty
CLAY/clayey SILT

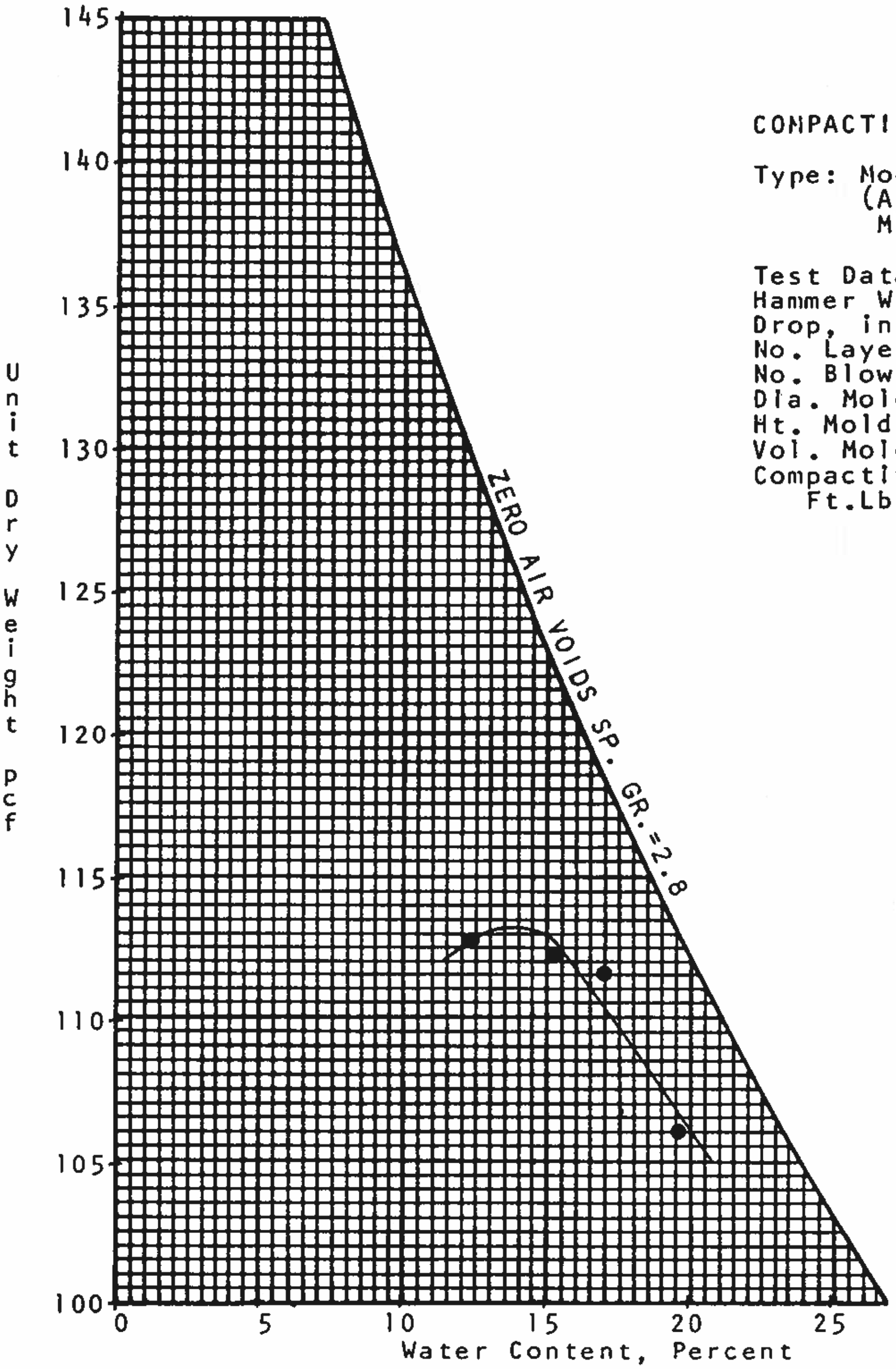
LL=23 PL=18 PI=5

COMPACTION TEST

Sample S- 1

Shannon & Wilson, Inc.
Geotechnical Consultants

Drawn AMC Date 9/21/83
Checked Date
Approved Date
Revised Date
Approved Date



M-695

COMPACTION TEST

Type: Modified AASHO
(ASTM D-1557-74)
Method C

Test Data	
Hammer Wt., lbs.	10.0
Drop, in.	18
No. Layers	5
No. Blows/Layer	25
Dia. Mold, in.	3.99
Ht. Mold, in.	4.58
Vol. Mold, cu.ft.	0.0331
Compactive Effort Ft.Lbs./Cu.Ft.	56300

Maximum Dry Density 112.8 pcf
Optimum Water Content 12.5 %
Natural Water Content 8.6 %

Classification:

Brown, sandy, very silty CLAY

LL=34 PL=23 PI=11

COMPACTION TEST

Sample S- 2

Shannon & Wilson, Inc.
Geotechnical Consultants

Drawn AMC Date 9/21/83 Checked ___ Date ___ Approved ___ Date ___ Revised ___ Date ___ Approved ___ Date ___

APPENDIX C
Excerpts from:
Final Report, Round 7 Dam Assessment, Associated Electric Cooperative, Inc.
New Madrid Power Plant, Ash Pond 1 & 2 and Slag Pond 1 &2 Impoundments
By GZA Environmental, Dated June 2011

FINAL REPORT
ROUND 7 DAM ASSESSMENT
ASSOCIATED ELECTRIC COOPERATIVE, INC.
NEW MADRID POWER PLANT
ASH POND 1 & 2 AND SLAG POND 1 & 2 IMPOUNDMENTS
NEW MADRID COUNTY, MISSOURI

June 3, 2011

PREPARED FOR:

U.S. Environmental Protection Agency
1200 Pennsylvania Avenue, NW
Washington, DC 20460

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19500 Victor Parkway, Suite 300
Livonia, MI 48152
GZA File No. 01.0170142.20

1.2 Description of Project

1.2.1 Location

The NMPP is located about three miles east of the city of Marston in New Madrid County, Missouri. The Site is accessible from the west via State Highway EE and from the north and south from Levee Road. The NMPP CCW impoundments are located near the power plant, which is located at latitude 36° 30' 56" North and longitude 89° 33' 47" West. A Site locus of the impoundments and surrounding area is shown in **Figure 1**. An aerial photograph of the impoundments and surrounding area is provided as **Figure 2**. The impoundments can be accessed by vehicles from earthen access roads from the NMPP.

1.2.2 Owner/Caretaker

The CCW impoundments are owned and operated by AECI.

	Dam Owner/Caretaker
Name	Associated Electric Cooperative, Inc. New Madrid Power Plant
Mailing Address	2814 S. Golden, P.O. Box 754
City, State, Zip	Springfield, Missouri 65801-0754
Contact	Duane Highley, P.E.
Title	Director, Power Production
E-Mail	duanehighley@aeci.org
Daytime Phone	(573) 643-2211
Emergency Phone	911 / (573) 379-0451 (Yard Superintendent Cell)

1.2.3 Purpose of the Impoundments

The NMPP is a two-unit coal-fired power plant, with a maximum generating capacity of approximately 1200 Megawatts. Unit 1 was constructed in 1972 while Unit 2 was constructed in 1977. Four earthen embankment CCW impoundments known as Ash Pond 1 (AP1) Impoundment, Slag Pond 1 (SP1) Impoundment, Ash Pond 2 (AP2) Impoundment, and Slag Pond 2 (SP2) Impoundment were constructed for the purpose of storing CCW waste and discharging plant wastewater.

The AP1 Impoundment and SP1 Impoundment were constructed in 1972 and function as sedimentation and storage basins for fly ash and boiler slag, respectively.[2] The SP2 Impoundment was constructed in 1984 and functions as a sedimentation and storage basin for boiler slag. The AP2 Impoundment was constructed in 1994 and functions as a sedimentation and storage basin for fly ash. The impoundments are located outside (on the river side) of the Mississippi River levee system. The top of embankment elevation of the AP1 Impoundment,

[2] Information regarding the materials received by each impoundment is based on the March 24, 2009 "Response to Request for Information Under Section 104(e) of the Comprehensive Environmental Response, Compensation, and Liability Act," from AECI to EPA.

the embankment slopes. To maintain positive downward pressure, water can be pumped from the MUW Pond to the AP2 Impoundment via two 18-inch diameter pipelines located on the downstream slope of the eastern embankment. The transfer pumps are manually controlled in the Pump Control Building on the crest of the eastern embankment.

The AP2 Impoundment embankments were designed with 3H:1V upstream and downstream slopes without rip-rap or other protection against wave action erosion[11]. The downstream slope of the western and southern embankments was generally designed to be vegetated with grass. There are three groundwater monitoring wells (P-6 through P-8) located along the eastern and southern embankments of the AP2 Impoundment.

1.2.7 Description of the Slag Pond 2 Impoundment and Appurtenances

Based on information provided by the NMPP personnel, the SP2 Impoundment was designed by Burns and McDonnell of Kansas City, Missouri. No construction documentation was available for the impoundment but a survey drawing from December of 2005 was provided by AECI. The following description of the SP2 Impoundment is based on the available survey drawing, the March 24, 2009 Response, and information provided by NMPP personnel.

The SP2 Impoundment is located east of the NMPP and north of the AP1 Impoundment and the SP1 Impoundment as shown on **Figure 2**. Most of the northern portion of the SP2 Impoundment is filled with fly ash that has settled in-place or has been stockpiled in the impoundment. The northern portion of the impoundment is used as a processing area for recovered ash. Water and fly ash are discharged into the SP2 Impoundment via four pipelines located on the northern portion of the impoundment. The discharged water and ash flow through an approximately 3 foot deep channel into an ash delta that is maintained by removal of settled fly ash. Clarified water from the SP2 Impoundment is discharged to the Mississippi River through a decant structure located near the southeast portion of the impoundment. The pond water elevation is maintained by stop logs in the decant structure. The ash that is removed from the channel is dewatered and stockpiled in the SP2 Impoundment as shown on **Figure 3** until it is recycled or transported to the dry ash landfill.

The SP2 Impoundment consists of an earthfill embankment with a crest length of approximately 3,000 feet and a general height (from the lowest toe elevation to the crest of the impoundment) of approximately 20 feet. The impoundment is unlined and the embankments were constructed from native silty clays. The impoundment has a surface area of approximately 4 acres at a water level elevation of 299 feet MSL and the stockpiled ash occupies approximately 18 percent of the storage capacity. A gravel access road is present on the crest of the impoundment. The crest elevation of the impoundment is approximately 302 feet MSL which appears to be below the elevation of the Mississippi River levee system.[12] Based on information provided by NMPP, the impoundment has not experienced damage from flooding of the Mississippi River.

The SP2 Impoundment embankments appeared to be designed with 4 horizontal on 1 vertical (4H:1V) upstream slopes and 2.5 horizontal on 1 vertical (2.5H:1V) downstream

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[11] Slopes based on Geotechnology, Inc. report "Global Stability Evaluation, Slag Pond 1 And Ash Pond 2, AECI New Madrid Power Generating Facility, New Madrid County, Missouri", dated July 31, 2009

[12] Elevations of all impoundments are estimated from topographic contours provided in AECI drawing titled: "Fly Ash Pond Improvement Plan View W/Contours", dated 1989.

slopes without rip-rap or other protection against wave action erosion on the upstream slopes[13]. The downstream slopes were generally designed to be vegetated with grass. There are no survey monuments or other instrumentation associated with this impoundment.

1.2.8 Operations and Maintenance

The impoundments are operated and maintained by NMPP personnel. Operation of the impoundments includes operation of the stop logs in the SP2 Impoundment decant structure, as well as removal of settled ash from the AP1 Impoundment and the SP2 Impoundment. Maintenance of the impoundments includes regular (annual) mowing of the applicable downstream slopes.

Operation and maintenance of the NMPP facility, including the impoundments, is regulated by the EPA under the NPDES Permit No. MO-0001171. Based on the March 24, 2009 Response, the State of Missouri does not perform inspections of the impoundments or regulate the impoundments.

The NMPP personnel monitor the impoundments according to a series of informal and written protocols. These protocols include:

- Informal observation of the impoundment embankments during normal operations at impoundments; and,
- Semi-annual inspection of the impoundments by NMPP personnel.

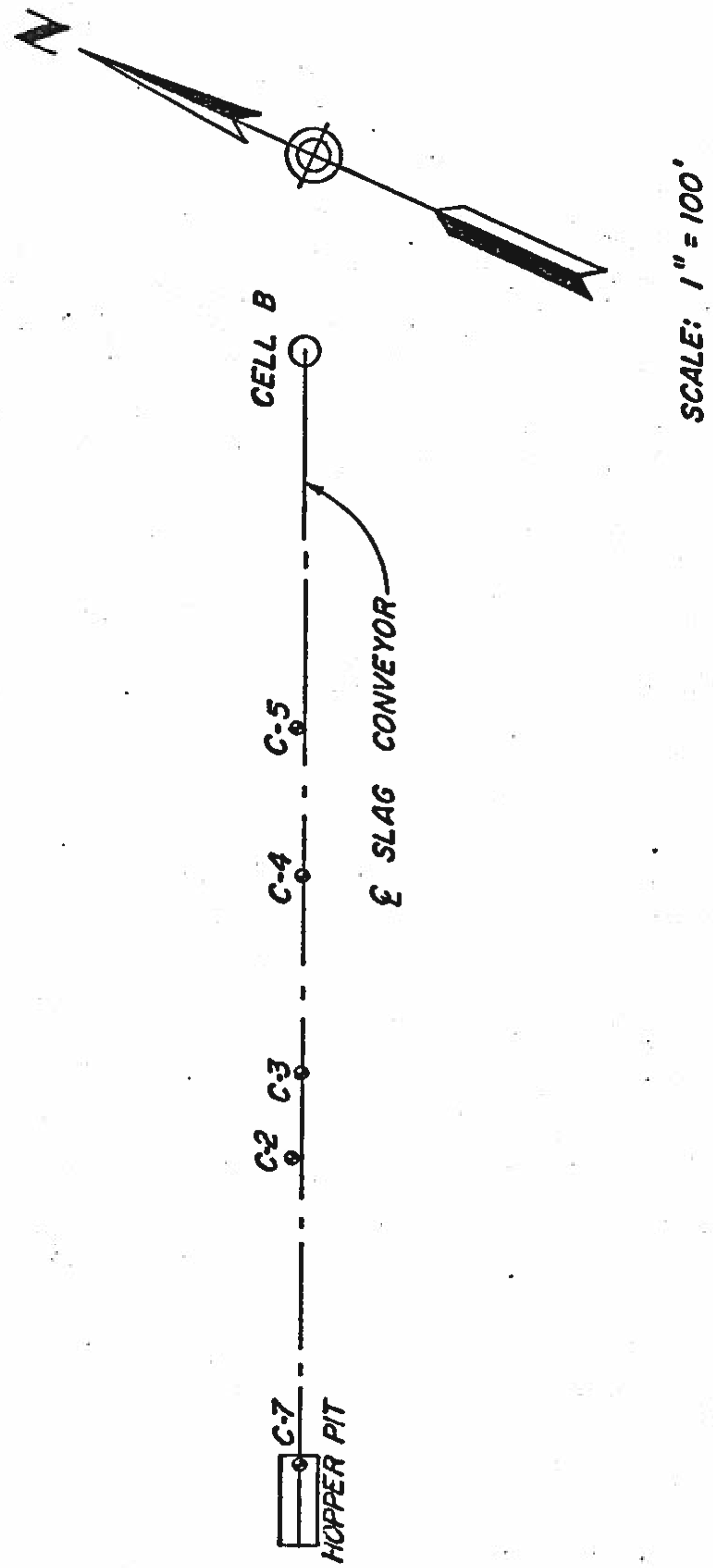
1.2.9 Size Classification

For the purposes of this EPA-mandated inspection, the size of the impoundments was based on U. S. Army Corps of Engineers (COE) criteria. Based on the maximum height of 12 feet and a storage volume of approximately 570 acre-feet, it is GZA's opinion that the AP1 Impoundment is considered a **Small** sized structure. Based on the maximum height of 20 feet and a storage volume of 14 acre-feet, it is GZA's opinion that the SP2 Impoundment is also classified as a **Small** sized structure. According to guidelines established by the U.S. Army COE, dams with a storage volume less than 1,000 acre-feet and/or a height less than 40 feet are classified as Small sized structures. The maximum impoundment height and storage volume was based on information provided by the NMPP.

Based on the maximum height of 20 feet and a storage volume of 1,137 acre-feet, it is GZA's opinion that the SP1 Impoundment is considered an **Intermediate** sized structure. Based on the maximum height of 20 feet and a storage volume of 1,351 acre-feet, it is GZA's opinion that the AP2 Impoundment is considered an **Intermediate** sized structure. According to guidelines established by the U.S. Army COE, dams with a storage volume between 1,000 and 50,000 acre-feet and/or a height between 40 and 100 feet are classified as Intermediate sized structures. The maximum impoundment height and storage volume was based on information provided by the NMPP.

[13] Slopes estimated by GZA from survey drawing generated by Smith & Co. titled "Pond at Outfall 004" and dated December 30, 2005.

APPENDIX D
Excerpts from:
Contract Documents for Slag Loadout Facility
Barttelbort, Rhutasel and Associates, Inc., Dated July 1979



BARTTELBORT, RHUTASEL & ASSOC., INC.
ENGINEERS - ARCHITECTS - SURVEYORS
FREEBURG, ILLINOIS

BORING LOCATION SKETCH

RECORD OF SUBSURFACE EXPLORATION

PROJECT New Madrid Power Plant BORING C - 2

JOHN MATHES & ASSOCIATES, INC.

LOGGED BY Schaefer

PIEZOMETER Yes

DRILLING METHOD Hollow Auger

SURFACE ELEVATION 292.5

DEPTH (FT.)	SAMPLE NUMBER	SAMPLE TYPE	NOTES	DESCRIPTION OF MATERIAL	BLOWS	DRY UNIT WEIGHT PCF
				Brownish-Gr. Si. CLAY w/Med. SAND & Gravel Fill		
	1	3T	Adv. 30" Rec. 14"	Intermixed Gr. & Dk. Gr. CLAY w/ Oxi. Spots & Stains, Fi. SAND & Si. Pockets Fill		
5	2	SS		Gr. Br. Cl. SILT w/Oxi. Spots & Stains, Fi. Sand & Mica Fill	7-12	
				Br. Gr. Si. CLAY w/Med. SAND & Grav., Fill		
	3	3T	Adv. 30" Rec. 27"	Br. Fi. SAND w/Oxi. Stains & Mica -Silty @ 7.5'		
10	4	SS		Grayish-Brown Silty CLAY w/ Fine Sand	5-5	
	5	SS		Brown Fine SAND -Light Gray Below 13.5'	8-14	
15	6	SS			10-12	
	7	SS			10-10	
				Gr. Br. Cl. SILT w/Fi. Sa. & Mica		
20	8a 8b	SS		Grayish-Brown Fine SAND -w/Mica @ 20' -Gr. Medium To Fine Below 23.5'	8-11	
25	9	SS			14-20	
30	10	SS		T. O. B.	14-25	

SHEAR STRENGTH, TSF: SV △ — QP/2 □ — QU/2 ○ (0, 0.5, 1.0, 1.5, 2.0, 2.5)

PL + — WATER ● CONTENT, % — LL + (0, 20, 40, 60, 80, 100)

GROUND WATER DEPTH AT COMPLETION Dry AFTER ______ ______ AFTER ______ ______

SCALE 1" = 5'

JOHN MATHES & ASSOCIATES, INC.

RECORD OF SUBSURFACE EXPLORATION

JOHN MATHES & ASSOCIATES, INC.

PROJECT New Madrid Power Plant BORING C - 3

LOGGED BY Schaefer
PIEZOMETER
DRILLING METHOD Hollow Auger

DEPTH (FT.)	SAMPLE NUMBER	SAMPLE TYPE	NOTES	DESCRIPTION OF MATERIAL (SURFACE ELEVATION 293.3)	BLOWS	DRY UNIT WEIGHT PCF
	1	3T	Adv. 30" Rec. 18"	Intermixed Gr., Brownish Gray & Dk. Gr., Si. CLAY & CLAY w/Fine Sand, Oxi. Spots & Stains Gravel, Silt Pockets, Silt Lenses		
5	2	SS		Fill	3-4	
	3	3T	Adv. 30" Rec. 25"	Grayish-Light Brown Silty Fine SAND -Brownish-Gray Below 8.5'		
10	4	SS			4-5	
	5	SS		-w/Clay Pockets From 13.5' To 20.0'	9-12	
15	6	SS			6-8	
	7	SS			5-6	
20	8	SS			8-10	
				-Medium To Fine Below 23.5'		
25	9	SS			10-14	
30	10	SS		T. O. B.	13-16	

SHEAR STRENGTH, TSF: SV △, QP/2 ○, QU/2 ○ — 0, 0.5, 1.0, 1.5, 2.0, 2.5

PL + WATER ● CONTENT, % + LL — 0, 20, 40, 60, 80, 100

GROUND WATER DEPTH AT COMPLETION D.C.I. 22.0' AFTER ______ AFTER ______

SCALE 1" = 5'

JOHN MATHES & ASSOCIATES, INC.

RECORD OF SUBSURFACE EXPLORATION

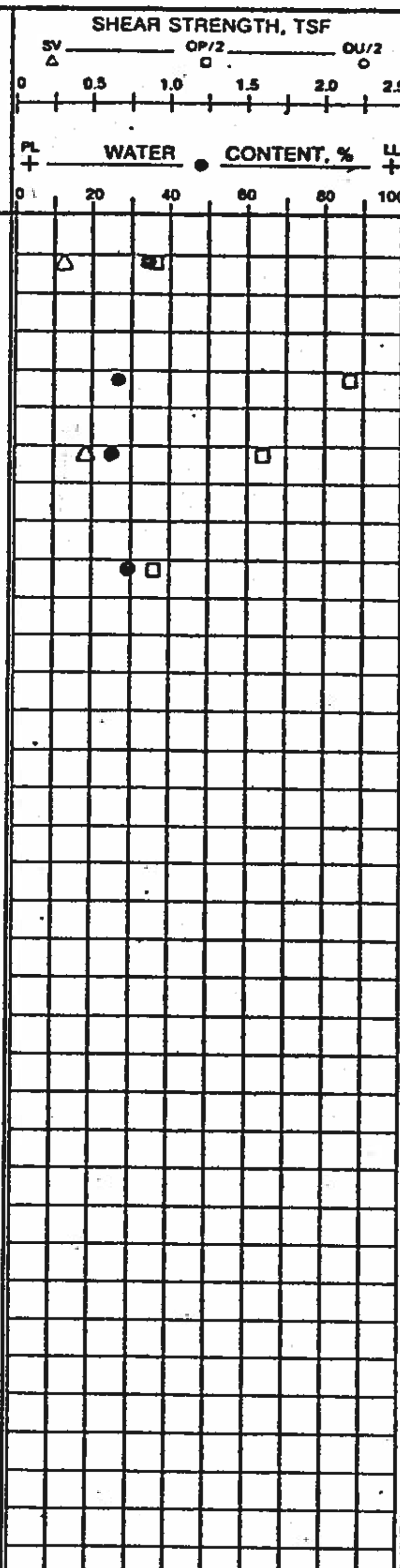
JOHN MATHES & ASSOCIATES, INC.

PROJECT New Madrid Power Plant BORING C - 4

DRILLED BY Simoncini
LOGGED BY Schaefer
DATE DRILLED 12/1/78
PIEZOMETER None
CONTRACT 555-78
DRILLING METHOD Hollow Auger

SURFACE ELEVATION 292.4

DEPTH (FT.)	SAMPLE NUMBER	SAMPLE TYPE	NOTES	DESCRIPTION OF MATERIAL	BLOWS	DRY UNIT WEIGHT PCF
				Intermixed Gr. CLAY & Sa. CLAY w/Oxi. Spots & Gravel, Fill		
	1	3T	Adv.:30" Rec. 18"	Gray CLAY w/Oxi. Spots -Mottled Gray & Brown Below 2.5'		87
5	2	SS			6-8	
			Adv. 30"	Gr. & Br. CLAY w/Si., Oxi. Spots & Stains & Fine Sand		97
	3	3T	Rec. 14"	Br. Sandy Silty CLAY w/Oxi. Spots & Stains		
10	4	SS		Gr. & Br. CLAY w/Sandy Lenses, Si. & Oxidized Spots	5-8	
	5	3T	Adv. 30" Rec. 18"	Light Brown Fine SAND -w/Traces Of Clay @ 11.5'		
15	6A 6B	SS		Gr. Micaseous Fine Sandy Clayey SILT	5-6	
	7	SS		Light Brown Fine SAND	7-8	
20	8	SS			9-10	
25	9A 9B	SS			6-14	
				Br. Sandy Clayey SILT -Gr., Clayey w/Traces Of Fine Sand @ 25.0'		
				Brown Fine To Medium SAND		
30	10	SS		T. O. B.		



GROUND WATER DEPTH AT COMPLETION 20' AFTER ______ AFTER ______

SCALE 1" = 5'

JOHN MATHES & ASSOCIATES, INC.

RECORD OF SUBSURFACE EXPLORATION

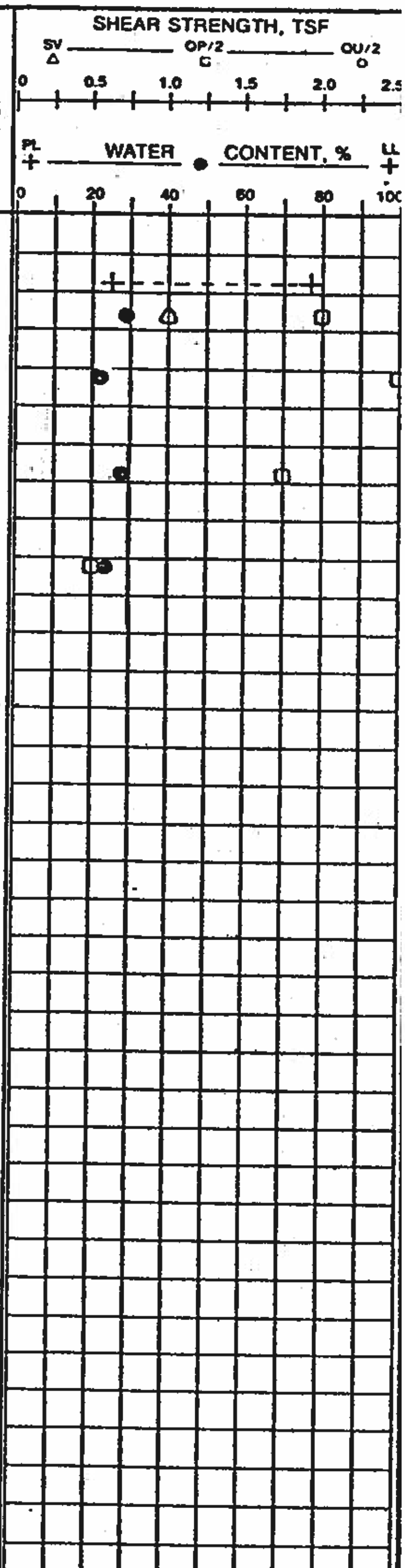
JOHN MATHES & ASSOCIATES, INC.

PROJECT: New Madrid Power Plant   BORING: C - 5

CONTRACT: 535-78
DRILLED BY: Simoncini
LOGGED BY: Schaefer
DATE DRILLED: 12/4/78
PIEZOMETER: Yes
DRILLING METHOD: Hollow Auger

SURFACE ELEVATION: 293.0

DEPTH (FT.)	SAMPLE NUMBER	SAMPLE TYPE	NOTES	DESCRIPTION OF MATERIAL	BLOWS	DRY UNIT WEIGHT PCF
	1	3T	Adv.30" Rec.12"	Intermixed Brown & Gray CLAY w/Oxi. Spots & Stains, Sand, Gravel, Silt Pockets, Fine Roots & Silt Lenses, Fill		
5	2	SS			9-17	
	3	3T	Adv.30" Rec.10"	Gr., Dk.Gr. & Br. Silty CLAY w/ Oxi. Spots & Stains & Silt Pockets, Fill		
10	4	SS			4-4	
	5	3T	Adv.30" Rec.10"	Brown Fine SAND -Light Gray Silty Below 13.5'		
15	6	SS			7-6	
	7	SS			8-16	
20	8	SS		Light Gray & Gray Silty CLAY w/Fine Sand	5-8	
				Gray Medium To Fine SAND		
25	9	SS			4-4	
30	10	SS			15-14	
				T. O. B.		



GROUND WATER DEPTH AT COMPLETION: Dry   AFTER ______ ______   AFTER ______ ______

SCALE 1" = 5'

JOHN MATHES & ASSOCIATES, INC.

RECORD OF SUBSURFACE EXPLORATION

JOHN MATHES & ASSOCIATES, INC.

PROJECT New Madrid Power Plant BORING C - 7

DRILLED BY ... LOGGED BY Schaefer

DATE DRILLED ... PIEZOMETER None

CONTRACT ... DRILLING METHOD Hollow Auger

DEPTH (FT.)	SAMPLE NUMBER	SAMPLE TYPE	NOTES	DESCRIPTION OF MATERIAL (SURFACE ELEVATION 310.0 est.)	BLOWS	DRY UNIT WEIGHT PCF
	1	SS		SLAG	3-5	
5				Intermixed Brownish-Gr. Gray, Dk. Gr. & Br., CLAY, Si. CLAY, SLAG, Fi. SAND, & Sandy CLAY w/Gravel, Oxi. Spots, Med. & Coarse SAND, Mica, Wood Fragments, Black Spots Fine SAND & Rocks Fill		
	2	SS				
10	3	3T	Adv. 30" Rec. 24"			
	4A 4B	SS			9-10	
	5	3T	Adv. 8" Rec. 8"			
15	6	SS		ROCK T. O. B.	32 - 50-3"	

SHEAR STRENGTH, TSF: SV △, QP/2 □, QU/2 ○ — 0, 0.5, 1.0, 1.5, 2.0, 2.5

PL + WATER ● CONTENT, % LL + — 0, 20, 40, 60, 80, 100

GROUND WATER DEPTH AT COMPLETION Dry AFTER ____ AFTER ____

SCALE 1" = 5'

JOHN MATHES & ASSOCIATES, INC.

NOTATION USED ON RECORD OF SUBSURFACE EXPLORATION

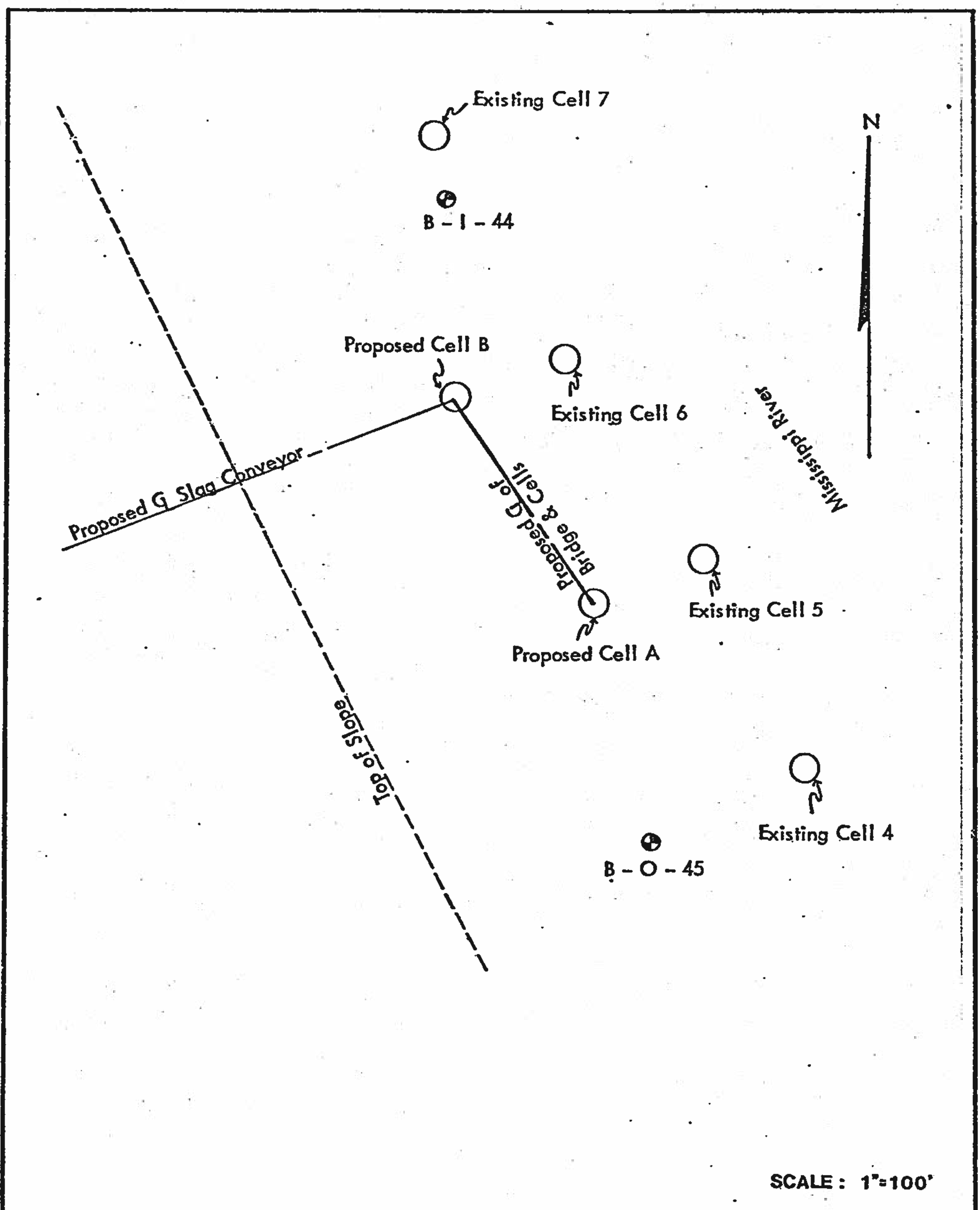
BLOWS

5-7	Each integer under the heading BLOWS refers to the number of impacts of a 140 pound hammer falling a distance of 30 inches to cause a standard split barrel sampler (1-3/8" I.D., 2" O.D.) to penetrate a distance of 6 inches. The pair of integers shown are for the last 12 inches of a total of 18 inches of penetration, the sum of which is the "N" value. The number of impacts for the first 6 inches of penetration, which is known as the seating drive, is regarded as unreliable and is not shown.
20-60/2"	The first integer refers to the number of impacts as described above to cause a standard split barrel sampler to penetrate a distance of 6 inches. The second integer refers to the number of impacts required to cause the same sampler to penetrate an additional indicated number of inches. The number of impacts for the first 6 inches of penetration is not shown.
60/2"	The first integer refers to the number of impacts required to cause the sampler to penetrate the indicated number of inches. The number of impacts for the first 6 inches of penetration is not shown.
X 60/2"	X indicates Seating Drive could not be achieved. Total penetration of the sampler is the indicated number of inches.
WR	WR indicates that the sampler penetrated under the static loading of the weight of the drill rod.
WH	WH indicates that the sampler penetrated under the static loading of the weight of the drill rod and the weight of the 140 pound hammer and its appurtenances.

NOTATION USED ON RECORD OF SUBSURFACE EXPLORATION

SS	-	Standard Split Barrel Sampler 1-3/8" I.D., 2" O.D.
3T	-	Thin Walled Tube Sampler 3" O.D.
PS	-	Piston Sampler, Using 3" O.D. Thin Walled Tube
WS	-	Wash Sample
NX	-	2-1/8" Diamond Rock Core
AS	-	Auger Sample
HA	-	Hollow Auger
FA	-	Flight Auger
WB	-	Wash Boring
TC	-	Tri-Cone Rotary Rock Bit
$\underline{\nabla}$	-	Piezometric Level
WCI	-	Wet Cave In
DCI	-	Dry Cave In

REC	-	Recovered
Q	-	Unconsolidated Undrained Triaxial Compressive Strength
QU	-	Unconfined Compressive Strength
Qp	-	Calibrated Penetrometer
SV	-	Shear Vane
G_s	-	Specific Gravity
LL	-	Liquid Limit
PL	-	Plastic Limit
PI	-	Plasticity Index
TOB	-	Termination of Boring
AR	-	Auger Refusal - Unable To Advance Augers
SSR	-	Split Barrel Sampler Refusal Less Than 1" Penetration For 50 BLOWS



BARTTELBORT, RHUTASEL & ASSOC., INC.
ENGINEERS - ARCHITECTS - SURVEYORS
FREEBURG, ILLINOIS

95

LAYNE-WESTERN COMPANY, INC.

1010 West 39th Street
Kansas City, Missouri

DATE 4/10/69 BORING NO. I-44

CONTRACT City of New Madrid

ADDRESS

CITY AND STATE New Madrid, Missouri

DRILLER J. Harper

SURFACE ELEV. 239.9 Approx. RIG KC54

WATER LEVEL CASING USED 57' - 10" / 54' - 6"

ABBREVIATIONS: A.O.—Auger Only; S.A.—Sample Auger; W.B.—Wash Bore; R.B.—Rock Bit; S.S.—Split Spoon; S.T.—Shelby Tube; S.V.—Shear Vane; C.W. Core Water; C.A.—Core Air

DEPTH From	DEPTH To	METHOD	PENETRATION RECORD Hydraulic Pressure	PENETRATION RECORD Number of Blows	CORE RECOVERY	SAMPLE DESCRIPTION COLOR—MATERIAL—MOISTURE—CLAY CONSISTENCY SAND DENSITY
0'0"	45'0"	WB				Open Hole
45'0"	49'6"	WB				Gray med. to coarse sand, some clay, wet, med. dense
49'6"	50'0"	WB				Concrete, broken
50'0"	55'0"	WB				Gray med. to coarse sand, tr. gravel, wet, dense
55'0"	56'6"	SS	D1	4/4/5	6"	Gray med. to coarse sand, tr. gravel, wet, med. dense
56'6"	60'0"	WB				Same
60'0"	61'6"	SS	D2	10/6 9	6"	Same
61'6"	65'0"	WB				Same
65'0"	66'6"	SS	D3	11/11 12	8"	Same
66'6"	70'0"	WB				Same
70'0"	71'6"	SS	D4	16/15 15	8"	Same
71'6"	75'0"	WB				Same
75'0"	76'6"	SS	D5	10/13 14	6"	Same
76'6"	80'0"	WB				Same
80'0"	81'6"	SS	D6	11/15/ 19	8"	Gray med. to coarse, tr. fine sand, gravel, med. dense
81'6"	85'0"	WB				Same
85'0"	86'6"	SS	D7	15/17 12	3"	Same
86'6"	92'6"	WB				Same

DRILLER TO INDICATE: TIME AND DATE HOLE COMPLETED

TIME AND DATE WATER LEVEL RECORDED

3318 10M 2-69

00200-11

7ø

LAYNE-WESTERN COMPANY, INC.
1010 West 39th Street
Kansas City, Missouri

DATE 4/10/69 BORING NO. I-44

CONTRACT City of New Madrid

ADDRESS

CITY AND STATE New Madrid, Missouri

DRILLER J. Harper

SURFACE ELEV. 239.9 Approx. RIG KC52

WATER LEVEL CASING USED

ABBREVIATIONS: A.O.—Auger Only; S.A.—Sample Auger; W.B.—Wash Bore; R.B.—Rock Bit; S.S.—Split Spoon; S.T.—Shelby Tube; S.V.—Shear Vane; C.W. Core Water; C.A.—Core Air

Depth From	Depth To	Method	Penetration Record: Hydraulic Pressure	Penetration Record: Number of Blows	Core Recovery	Sample Description (Color—Material—Moisture—Clay Consistency / Sand Density)
92'6"	94'0"	SS	D8	6/7/9		Gray med. to coarse sand, wet, med.dense, tr. gravel,
94'0"	97'6"	WB				Same
97'6"	99'0"	SS	D9	15/15/17	8"	Same dense
99'0"	101'6"	PS	S10		0	Same
101'5"	102'6"	WB				Same
102'6"	104'0"	SS	S11		10"	Same
104'0"	105'6"	PS	S12		18"	Same
105'6"	107'6"	WB				Gray brown med. to coarse sand, V.dense, some med. fine
107'6"	109'0"	SS	D13	16/41 62	10"	Same
109'0"	112'6"	WB				Same few boulders and gravel
112'6"	114'0"	SS	D14	9/75 refusal	6"	Same tr. lignite, V. dense
117'0"	117'6"	WB				Same
117'6"	119'0"	SS	D15	4/12 40	8"	Same
119'0"	122'6"	WB				Gray brown coarse to V. coarse sand, some med. & med. fine, few boulders, V. dense
122'6"	124'0"	SS	D16	18/31 32	8"	Gray brown med. to coarse sand, dense some V.
124'0"	126'0"	WB				Same
126'0"	127'6"	SS	D17	21/40 50 ref.	8"	Same
127'6"	Total depth					

DRILLER TO INDICATE: TIME AND DATE HOLE COMPLETED

TIME AND DATE WATER LEVEL RECORDED

Form 3919 10M 3-69

77

LAYNE-WESTERN COMPANY, INC.
1010 West 39th Street
Kansas City, Missouri

DRILLER Harper

SURFACE ELEV. 244.5 Approx RIG KC54

DATE 4/8/69 BORING NO. O -45

CONTRACT City of New Madrid

ADDRESS

CITY AND STATE New Madrid, Missouri

WATER LEVEL CASING USED 46.7' - 10" 49.2' - 6'

ABBREVIATIONS: A.O.—Auger Only; S.A.—Sample Auger; W.B.—Wash Bore; R.B.—Rock Bit; S.S.—Split Spoon; S.T.—Shelby Tube; S.V.—Shear Vane; C. W. Core Water; C.A.—Core Air

DEPTH From	DEPTH To	METHOD	PENETRATION RECORD Hydraulic Pressure	PENETRATION RECORD Number of Blows	CORE RECOVERY	SAMPLE DESCRIPTION COLOR—MATERIAL—MOISTURE—CLAY CONSISTENCY SAND DENSITY
0'0"	38'0"					Water and open hole
38'0"	48'0"	WB				Gray brown med. to coarse, V. coarse sa loose
48'0"	49'6"	SS	D1	9/6/7	2"	Same
49'6"	53'0"	WB				Same
53'0"	54'6"	SS	D2	8/9/10	8"	Same tr. lignite
54'6"	58'0"	WB				Same
58'0"	59'6"	SS	D3	10/15/16	12"	Same
59'6"	61'0"	WB				Same
61'0"	62'6"	SS	D4	18/19/ 22	8"	Same dense
62'6"	65'0"	WB				Same
65'0"	66'6"	SS	D5	16/18/27	18"	Gray silty fine sand, dense, tr.lignite
66'6"	69'0"	PS	S6		24"	Same
69'0"	70'0"	WB				Same
70'0"	71'6"	SS	D7	9/9/12		Gray fine to med. sand, med. dense
71'6"	75'0"	WB				Same
75'0"	76'6"	SS	D8	10/14/ 15	10"	Gray med. to coarse sand, tr. fine, med. dense
76'6"	80'0"	WB				Same
80'0"	82'6"	PS	S9			Same

DRILLER TO INDICATE: TIME AND DATE HOLE COMPLETED

TIME AND DATE WATER LEVEL RECORDED

3918 10M 3-69

00200-13

7 8

LAYNE-WESTERN COMPANY, INC.
1010 West 39th Street
Kansas City, Missouri

DRILLER McLaughlin

SURFACE ELEV. 244.5 Approx. KC54

DATE 4/8/69 BORING NO. 0-45

CONTRACT City of New Madrid

ADDRESS

CITY AND STATE New Madrid, Missouri

WATER LEVEL ________ CASING USED ________

Page 2

ABBREVIATIONS: A.O.—Auger Only; S.A.—Sample Auger; W.B.—Wash Bore; R.B.—Rock Bit; S.S.—Split Spoon; S.T.—Shelby Tube; S.V.—Shear Vane; C.W. Core Water; C.A.—Core Air

DEPTH From	DEPTH To	METHOD	PENETRATION RECORD Hydraulic Pressure	PENETRATION RECORD Number of Blows	CORE RECOVERY	SAMPLE DESCRIPTION COLOR—MATERIAL—MOISTURE—CLAY CONSISTENCY SAND DENSITY
82'0"	84'0"	WB				Gray med. to fine sand, med. dense
84'0"	85'0"	WB				Coarse gravel
85'0"	86'6"	SS	D10	12/10/11		Same
86'6"	90'0"	WB				Same
90'0"	91'6"	SS	D11	9/26/20		Same Med. sand @ 91'0 med. dense
91'6"	95'0"	WB				Gray med. sand, dense
95'0"	96'6"	SS	D12	15/17/30		Same
96'6"	98'0"	WB				Same
98'0"	101'6"	WB				Limestone
101'6"	102'0"	WB				Gravel
102'0"	103'6"	SS	D13	13/30/23		Gray gravel, coarse sand, dense
103'6"	107'0"	WB				Same
107'0"	108'6"	SS	D14	35-48 Ref.		Same very dense
108'6"	112'0"	WB				Gray coarse & med. sand, tr. gravel, wet, dense
112'0"	113'0"	SS	D15	29/37/55	10"	Same
113'0"	117'0"	WB				Same
117'0"	118'6"	SS	D16	15/19/38	6"	Same
118'6"	Total depth					

DRILLER TO INDICATE: TIME AND DATE HOLE COMPLETED ________
TIME AND DATE WATER LEVEL RECORDED ________

3918 10M 3-69

APPENDIX E
Slag Loadout Facility
By AECI, Dated 1 December 1983

associated electric cooperative, inc.

2814 S. Golden, P.O. Box 754
Springfield, Missouri 65801
417-881-1204

December 1, 1983

Missouri Department of
Natural Resources
948 Lester Street
Poplar Bluff, MO 63901

RE: Slag Loadout Facility
EP.NO.8001.3
EN.NO.2120

Attention: Mr. James A. Burris

Gentlemen:

Associated Electric Cooperative, Inc., herein requests a permit to construct ash handling facilities at our New Madrid Power Plant located in the St. Jude Industrial Park on the Mississippi River, 5 miles south of New Madrid, Missouri. Enclosed is our permit application dated December 1, 1983, and our application fee of $25.

The facility will be used for the treatment and handling of boiler bottom ash (slag) generated as a waste by-product during the combustion of coal in the power plant. The facility will include a diked process area where slag will be dewatered by draining, and a retention pond to clarify the effluent. The effluent will discharge to the river through an outfall structure. Additional facilities will be provided to load the dewatered slag onto barges for transportation off the plant site. These additional facilities will consist of a loading hopper, a conveyor system, and barge mooring structures. Drawing No. C-1, enclosed, provides an overall site layout of the facilities. Drawings No. M-1, M-2, and M-3 provide additional detail.

Slag will be hydraulically transported (sluiced), using Mississippi River water as the transport medium, into the process area from the coal-fired boilers via four (4) 10-inch diameter pipelines. The upper portion of the process area will be relatively flat to allow the slag particles to settle near the pipe discharge. The water level in the process area will be maintained below the level of the upper area so that only the lower portion of the process area will retain water. The lower portion will provide retention time for the reduction of the concentration of suspended solids to meet effluent limitations.

The clarified effluent will be discharged from the process area through an overflow weir-type outfall structure. Concrete stop logs will be used to maintain water level. A 24-inch diameter pipeline will exit near the bottom of the outfall structure, travel under the ash retaining dike, and discharge into the river. A flap gate will be provided at the discharge to prevent backflow into the process area. Drawing No. S-5, enclosed, details the outfall structure.

The dewatered slag will be reclaimed from the upper portion of the process area and stockpiled using front-end loaders. The stockpiled "dry" material will be periodically loaded into the hopper, dumped onto the conveyor belt, transported to the river mooring facilities, and loaded directly into barges.

These facilities will replace existing facilities being used for handling of the bottom ash. The existing facilities consist of a dewatering plant and ash pond. The new facilities will provide a more cost-effective method for the processing of slag for market.

As stated above, the material being sluiced into the process area will be boiler bottom ash. This material is granular with the approximate appearance of medium sand with black, glass-like particles. A very small amount (approximately 0.7%) of economizer ash will also be present. Economizer ash is also a granular material, but with much smaller particles than the bottom ash, and has a brownish color.

Earlier this year, samples of boiler slag and boiler slag sluice water with economizer ash present were collected and analyzed. A sample of Mississippi River water was also analyzed to establish background concentrations. The analysis conducted on the slag utilized the Toxic Extraction Procedure (TEP) as described in Section 10 CSR 25-4.010, inclusive of sulfate and exclusive of TEP organics. Parameters analyzed in the Mississippi River and slag sluice water samples were pH, the TEP heavy metals, and sulfate. Results of the analysis, enclosed, indicate the concentrations of metals and sulfates in the slag and sluice water to be below TEP criteria and drinking water standards. We note that although the instrument detection limit for lead, selenium and mercury was higher than the drinking water standards, we believe the concentration of these metals would also be low. We have based this on an analysis of the ash pond discharge water submitted last year for renewal of the New Madrid NPDES permit and the physical nature of these metals during the coal combustion process. These results also indicate that the sluice water pH is within effluent limitations.

In order to insure that the concentration of suspended solids in the pond effluent will be within applicable effluent limitations, additional testing and calculations were performed. Enclosed is a copy of the slag settling analysis prepared by Bendy Engineering Co. This analysis confirms that the effluent limitation can be met. Note that a laboratory settling test, the results of which are included in the analysis, shows that a retention time of 24 hours will lead to a concentration of 2 mg/l in the effluent. The pond to be constructed will have a retention time of 24 hours based on maximum sluice water flows and a 10-year, 24-hour rainfall event. These results are consistent with utility industry practices. Experience with similar facilities has shown that slag particles settle immediately, and fly ash particles settle within 7 to 8 hours.

Concentrations of oil and grease in the sluice water have characteristically been extremely low and were not analyzed in any testing procedure.

The design criteria for construction of the facilities have been established in a geotechnical investigation prepared by Shannon & Wilson, Geotechnical Consultants. A copy of their report is enclosed. This report describes the measures that will be taken to construct the facilities in accordance with the regulations in 10 CSR 20-8.200. You will note that the pond bottom will be over-excavated and recompacted to insure its integrity and will have a permeability of 10^{-7} cm/sec or less.

The retaining dikes will be of sufficient height to protect against a 100-year flood event and will be protected from erosion by seeding on the interior slopes and rip-rap on the exterior slopes.

We have attempted to comply with the applicable regulations in the design of these facilities and will look forward to your response.

If you have any comments or questions, please advise.

Very truly yours,

Doug Kent

Doug Kent
Mechanical Engineer

DK:lh

Enclosures

cc: B. T. Stone
W. T. Lawler
J. G. Rekus

FOR AGENCY USE ONLY
APPLICATION NUMBER
MO -
DATE RECEIVED

FORM A - APPLICATION FOR DISCHARGE PERMIT - ALL APPLICANTS

DO NOT ATTEMPT TO COMPLETE THIS FORM BEFORE READING THE ACCOMPANYING INSTRUCTIONS

MISSOURI DEPARTMENT OF NATURAL RESOURCES - DIVISION OF ENVIRONMENTAL QUALITY
P. O. Box 1368
Jefferson City, Missouri 65102

1.10 Construction permit application X. A $25.00 filing fee must accompany each application for a construction permit.

1.20 Operating permit application ________. A $75.00 filing fee must accompany each application for an operating permit.

Filing fees must be in the form of check, bank draft, or money order, payable to the State of Missouri. Cash will not be accepted.

2.10 Name of Facility: New Madrid Power Plant

2.20 Facility Address: St. Jude Industrial Park, Marston, Missouri
Street / City / State / Zip Code

2.30 This facility is now in operation under Missouri Operating Permit Number 0001171

2.40 This is a new facility and was constructed under Missouri Construction Permit Number ________. (Complete only if this facility does not have an operating permit.)

2.50 Owner Name: Associated Electric Cooperative, Inc. Phone: (417) 881-1204

Address: 2814 S. Golden, Springfield, MO 65801
Street / City / State / Zip Code

2.60 Operating Authority Name: Same

Address: ________
Street / City / State / Zip Code

2.70 Facility Contact Name Bill Lawler Phone (314) 643-2211

Title Plant Manager

2.80 Additional forms necessary to complete this application:

a. Does your facility receive and treat basically domestic waste:

_____ yes (complete form B) X no

b. Is your facility a manufacturing, commercial, mining or silviculture waste treatment facility: X yes (complete form C and answer c of this subpar _____ no

c. Is your facility considered a "primary industry" under EPA guidelines:
X yes (complete forms C & D) _____ no
(Forms C & D not applicable for this construction permit.)

2.90 I certify that I am familiar with the information contained in the application, that to the best of my knowledge and belief such information is true, complete and accurate, and if granted this permit, I agree to abide by the Missouri Clean Water Law and all rules, regulations, orders and decisions, subject to any legitimate appeal available to applicant under the Missouri Clean Water Law, of the Missouri Clean Water Commission.

_____ Date: 12/1/83
Applicant's Signature (see instructions)

MMTL Analytical Services

206 South Keene Street
Columbia, MO 65201
314/874-8080

1000 Lake Saint Louis Blvd. #248
Lake Saint Louis, MO 63367
314/625-1047

Client: Associated Electric Cooperative

Attention: Jerry Bindell

Customer P.O. Number: 29036

Date Reported: May 26, 1983

Date Sampled: May 6, 1983

Date Received: May 12, 1983

MMTL I.D. Number: 16550

REPORT OF ANALYSIS

Parameter	River Water	Bottom Ash Sluice
Total Arsenic mg/l	≤0.05	≤0.05
Total Barium mg/l	0.8	0.9
Total Cadmium mg/l	≤0.01	≤0.01
Total Chromium mg/l	≤0.02	≤0.02
Total Lead mg/l	≤0.1	≤0.1
Total Mercury mg/l	≤0.0005	≤0.0005
Total Selenium mg/l	≤0.002	≤0.002
Total Silver mg/l	≤0.01	≤0.01
Sulfate mg/l as $SO_4^{=}$	37	37
ph*	7.70	7.32

RECEIVED
MAY 27 1983

Comments: *Field measurement by client.

Approved: [signature]

MMTL
Analytical Services

206 South Keene Street
Columbia, MO 65201
314/874-8080

1000 Lake Saint Louis Blvd. #248
Lake Saint Louis, MO 63367
314/625-1047

CLIENT: Associated Electric Cooperative

ATTENTION: Jerry Bindell

CUSTOMER P.O. NUMBER: 29036

DATE REPORTED: May 26, 1983

DATE SAMPLED: May 6, 1983

DATE RECEIVED: May 12, 1983

MMTL I.D. NUMBER: 16500

REPORT OF ANALYSIS

Parameter	Boiler Slag
Toxic Extraction Procedure	
Arsenic mg/l	$\leq$0.005
Barium mg/l	0.5
Cadmium mg/l	$\leq$0.01
Chromium mg/l	$\leq$0.02
Lead mg/l	$\leq$0.1
Mercury mg/l	$\leq$0.0005
Selenium mg/l	$\leq$0.002
Silver mg/l	$\leq$0.01
Sulfate mg/l as $SO_4^{=}$	$\leq$5

RECEIVED
MAY 27 1983

COMMENTS:

APPROVED: [signature]

Page 1 of 1

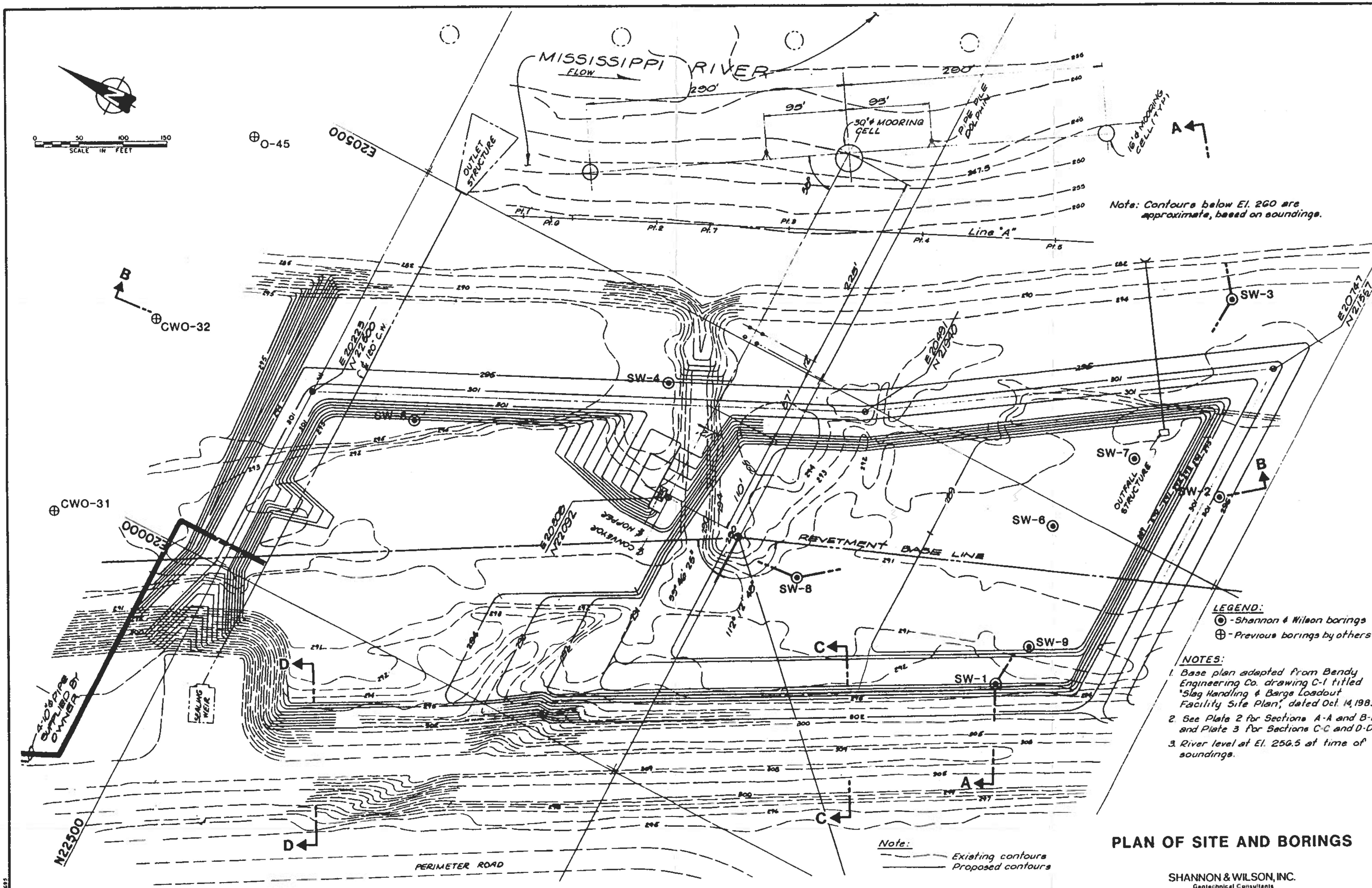
ATTACHMENT

Parameter	TEP Limits mg/l	Public Drinking Water Supply Standards, mg/l	Instrument Detection Limit, mg/l	Boiler Slag mg/l	Mississippi River Water mg/l	Boiler Slag Sluice Water mg/l	Ash Pond Discharge Water (NPDES Permi Application) mg/l
Arsenic	5.0	.05	.005	≤ dl[1]	≤ dl	≤ dl	.001
Barium	100.0	1.0	.1	.5	.8	.9	< .1
Cadmium	1.0	.01	.01	≤ dl	≤ dl	≤ dl	< .005
Chromium	5.0	.05	.02	≤ dl	≤ dl	≤ dl	.005
Lead	5.0	.05	.1	≤ dl	≤ dl	≤ dl	.001
Mercury	.2	.00005	.0005	≤ dl	≤ dl	≤ dl	.002
Selenium	1.0	.01	.02	≤ dl	≤ dl	≤ dl	.002
Silver	5.0	.05	.01	≤ dl	≤ dl	≤ dl	.0003
pH	≤ or ≥ 12.5	6-9	--	--	7.70	7.32	6.7
Sulfate	--	250	5	≤ dl	37	37	190

[1] dl refers to detection limit

JEB:ph
5/26/83

APPENDIX F
Detailed Dimensional Drawings



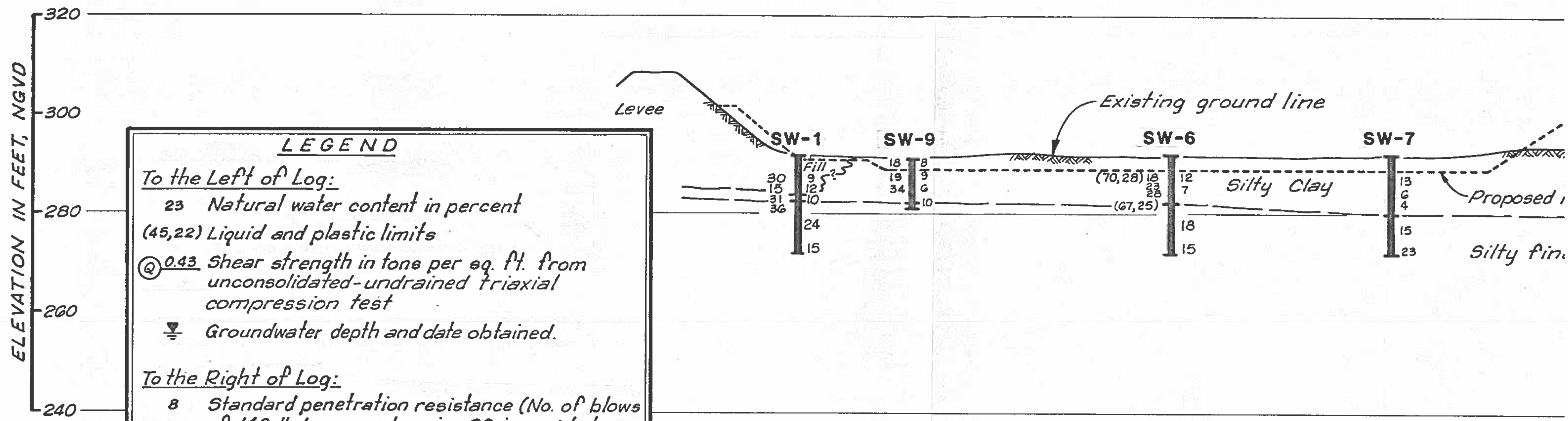
PLAN OF SITE AND BORINGS

SHANNON & WILSON, INC.
Geotechnical Consultants

LEGEND:
- ⊙ - Shannon & Wilson borings
- ⊕ - Previous borings by others

NOTES:
1. Base plan adapted from Bendy Engineering Co. drawing C-1 titled "Slag Handling & Barge Loadout Facility Site Plan," dated Oct. 14, 1983.
2. See Plate 2 for Sections A-A and B-B, and Plate 3 for Sections C-C and D-D.
3. River level at El. 256.5 at time of soundings.

PLATE 1



LEGEND

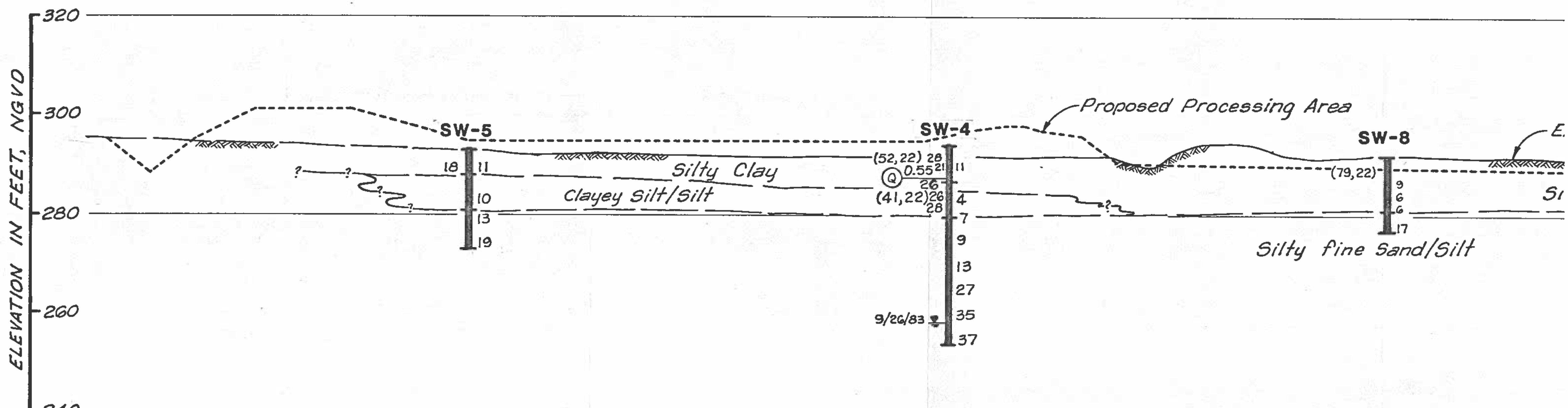
To the Left of Log:

- 23 — Natural water content in percent
- (45,22) — Liquid and plastic limits
- Q 0.43 — Shear strength in tons per sq. ft. from unconsolidated-undrained triaxial compression test
- ▼ — Groundwater depth and date obtained.

To the Right of Log:

- 8 — Standard penetration resistance (No. of blows of 140-lb. hammer dropping 30-in. req'd. to drive a 2" O.D. split-spoon one foot).

SECTION A-A

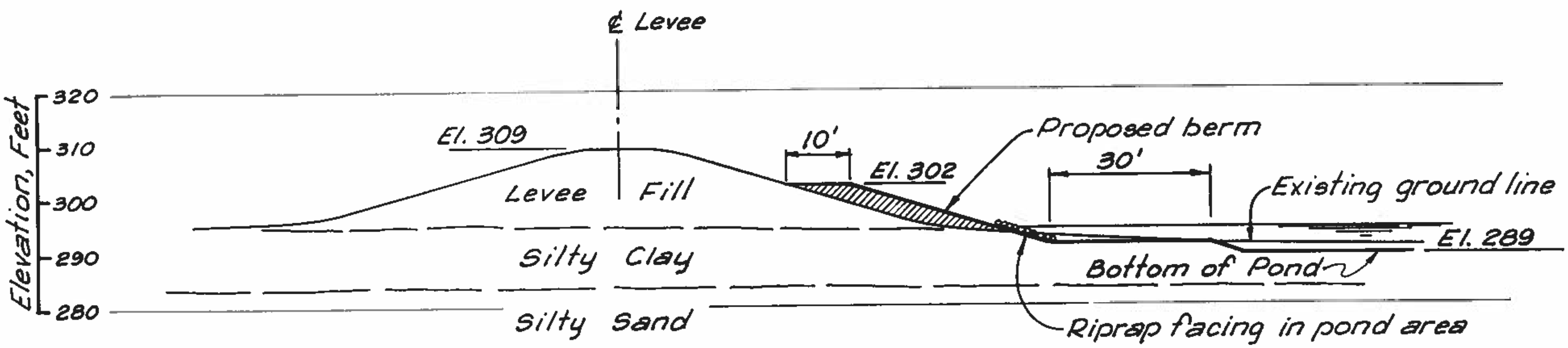


General Note:
Data concerning subsurface conditions have been obtained at boring locations only. Actual conditions at locations between borings may differ from the generalized profile(s) shown here.

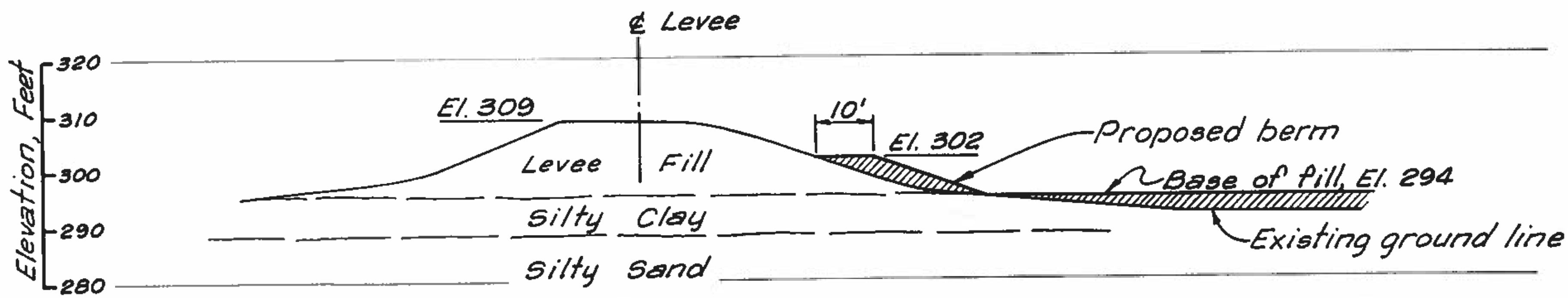
M-695

NOTE:
See Plate I for location of sections.

SECTION B-B



SECTION C-C
POND AREA

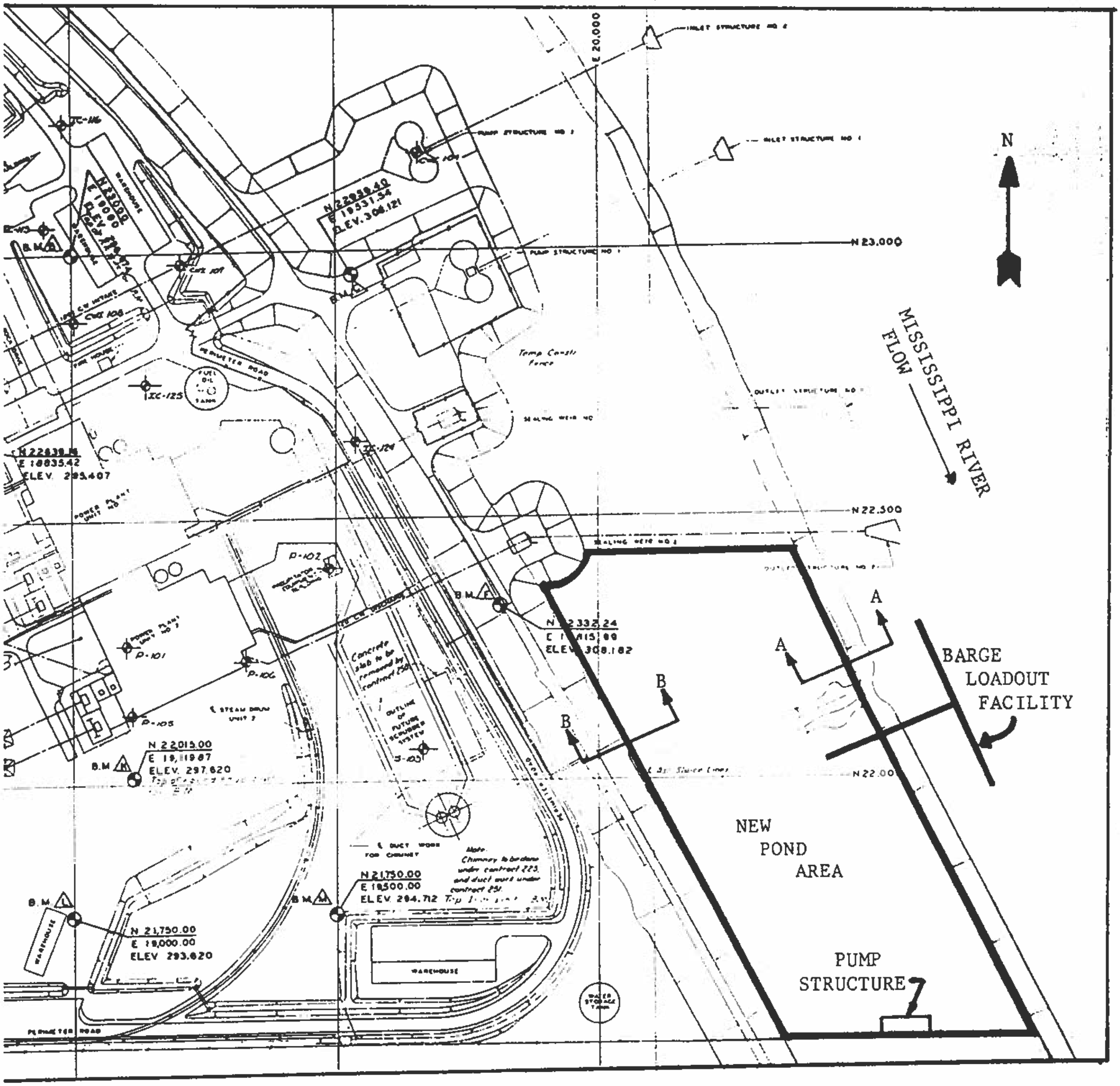


SECTION D-D
DISCHARGE AREA

0 30 60
SCALE IN FEET

PROFILES THROUGH EXISTING LEVEE

SHANNON & WILSON, INC.
Geotechnical Consultants



PLAN VIEW

PROPOSED ASH POND

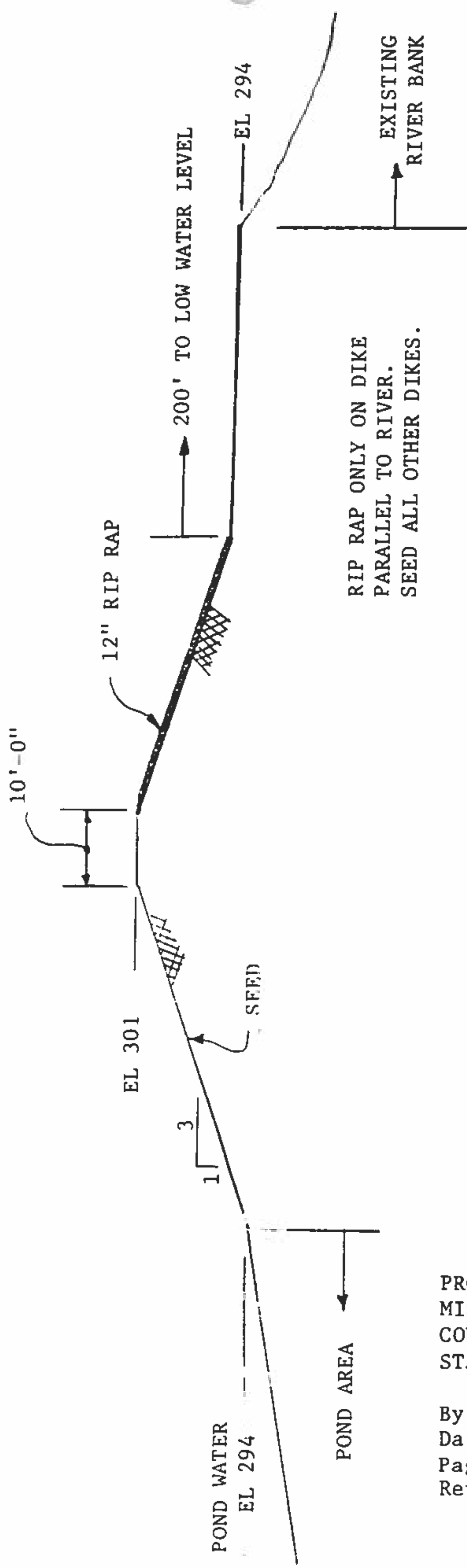
NOTES:

1) FOR BARGE LOADING FACILITIES DETAILS SEE PAGE 5 of 7.

2) ALL ADJACENT PROPERTY IS ST. JUDE INDUSTRIAL PARK, OWNED BY CITY OF NEW MADRID, MISSOURI.

PROPOSED ASH HANDLING FACILITIES ON
MISSISSIPPI RIVER AT APPROXIMATE MILE 884
COUNTY OF NEW MADRID
STATE OF MISSOURI

By Associated Electric Cooperative, Inc.
Date: December 15, 1982
Page 2 of 8



SECTION A-A

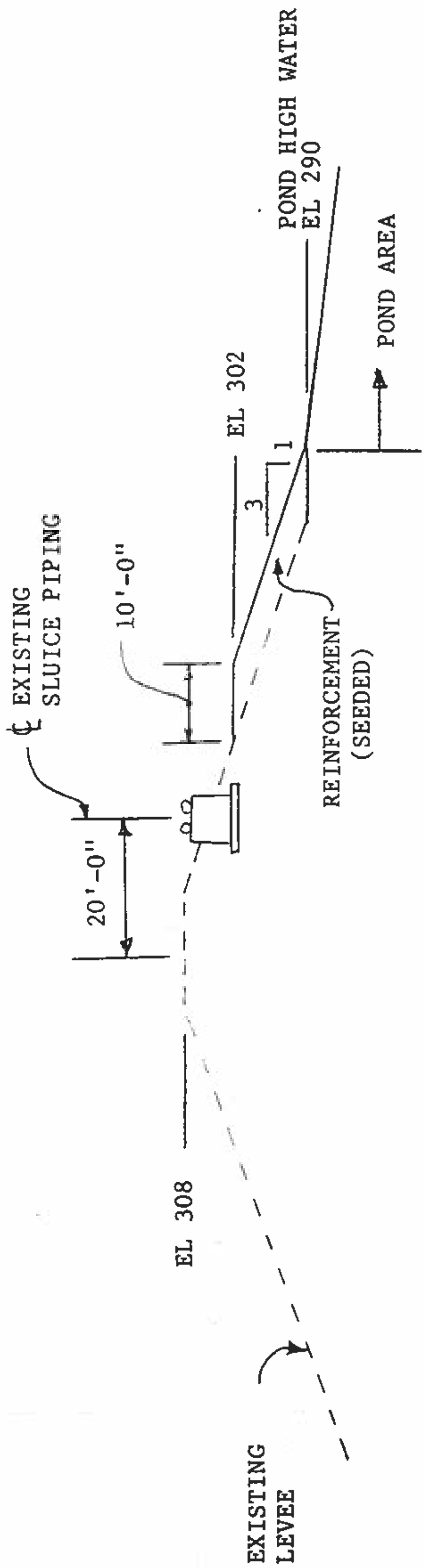
ASH RETAINING DIKE

SCALE: 1" = 25'

NOTE:
LOW WATER LEVEL = EL 254.9
100 YR. FLOOD LEVEL = EL 300.5
OBTAINED FROM HYDRAULIC BRANCH OF
ENGINEERING DIVISION

PROPOSED ASH HANDLING FACILITIES ON
MISSISSIPPI RIVER AT APPROXIMATE MILE 884
COUNTY OF NEW MADRID
STATE OF MISSOURI

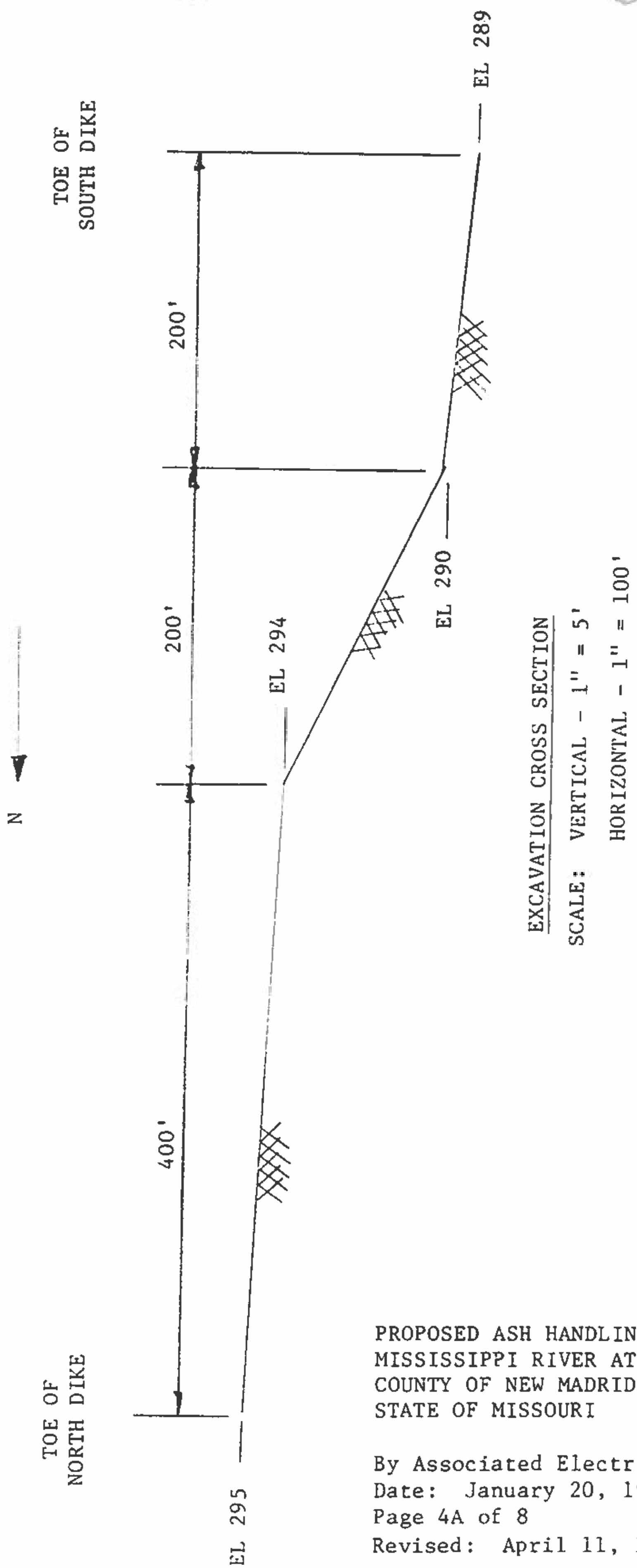
By Associated Electric Cooperative, Inc.
Date: December 15, 1982
Page 3 of 8
Revised: April 11, 1983



SECTION B-B
REINFORCEMENT OF EXISTING LEVEE
SCALE: 1" = 25'

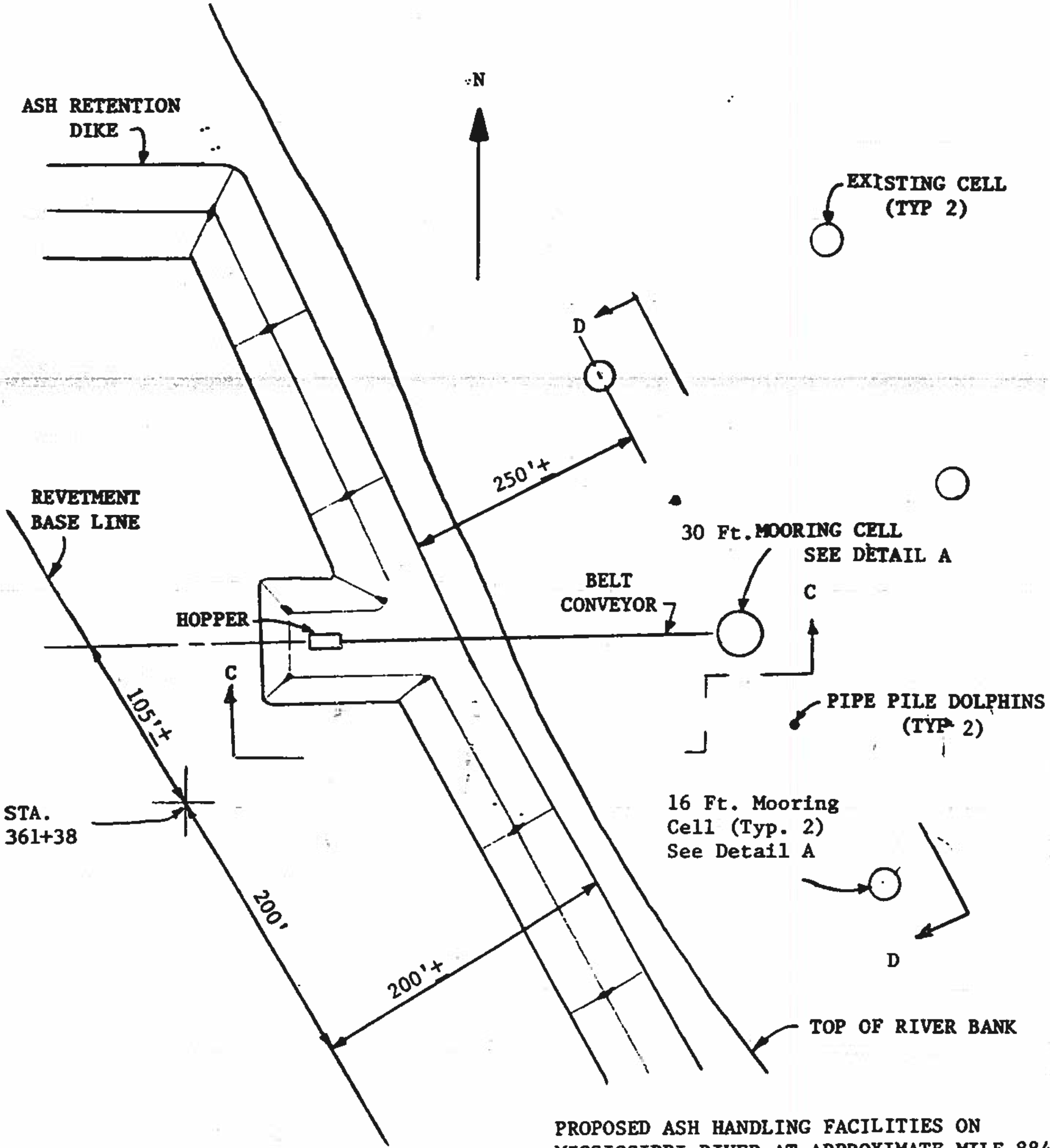
PROPOSED ASH HANDLING FACILITIES ON
MISSISSIPPI RIVER AT APPROXIMATE MILE 884
COUNTY OF NEW MADRID
STATE OF MISSOURI

By Associated Electric Cooperative, Inc.
Date: December 15, 1982



PROPOSED ASH HANDLING FACILITIES ON
MISSISSIPPI RIVER AT APPROXIMATE MILE 884
COUNTY OF NEW MADRID
STATE OF MISSOURI

By Associated Electric Cooperative, Inc.
Date: January 20, 1983
Page 4A of 8
Revised: April 11, 1983



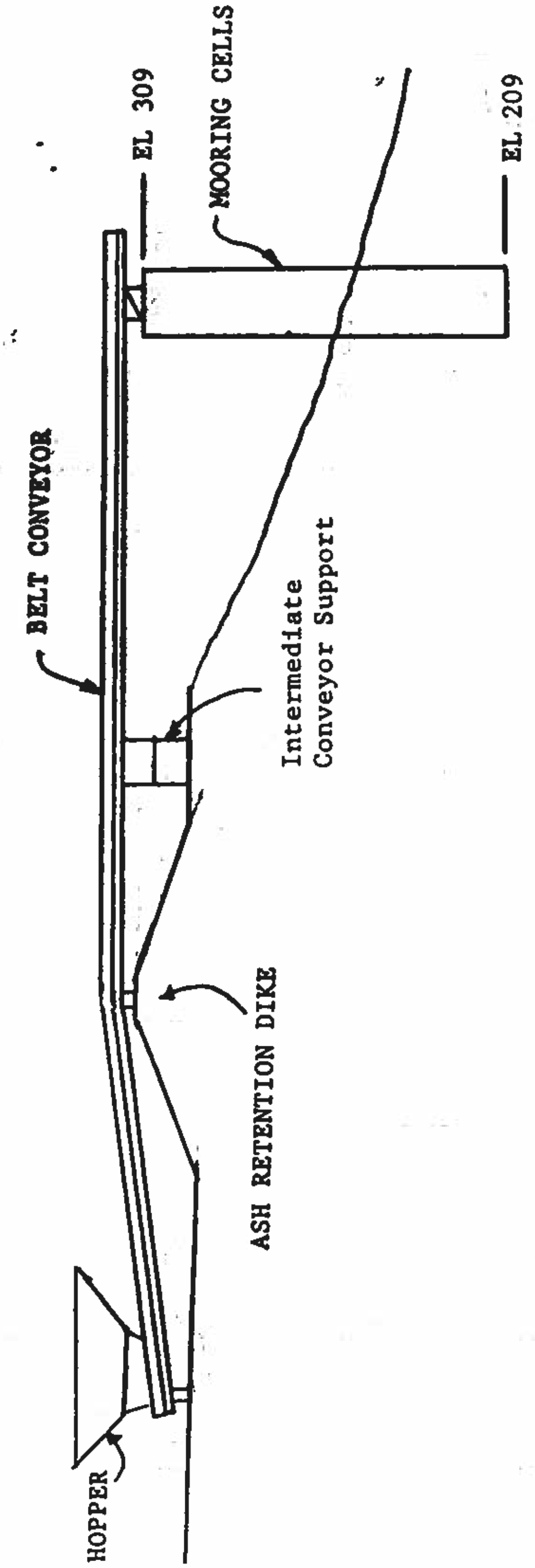
PLAN VIEW

PROPOSED BARGE LOADOUT CONVEYOR

SCALE: 1" = 100'

PROPOSED ASH HANDLING FACILITIES ON
MISSISSIPPI RIVER AT APPROXIMATE MILE 884
COUNTY OF NEW MADRID
STATE OF MISSOURI

By Associated Electric Cooperative, Inc.
Date: April 2, 1984
Page 2 of 5



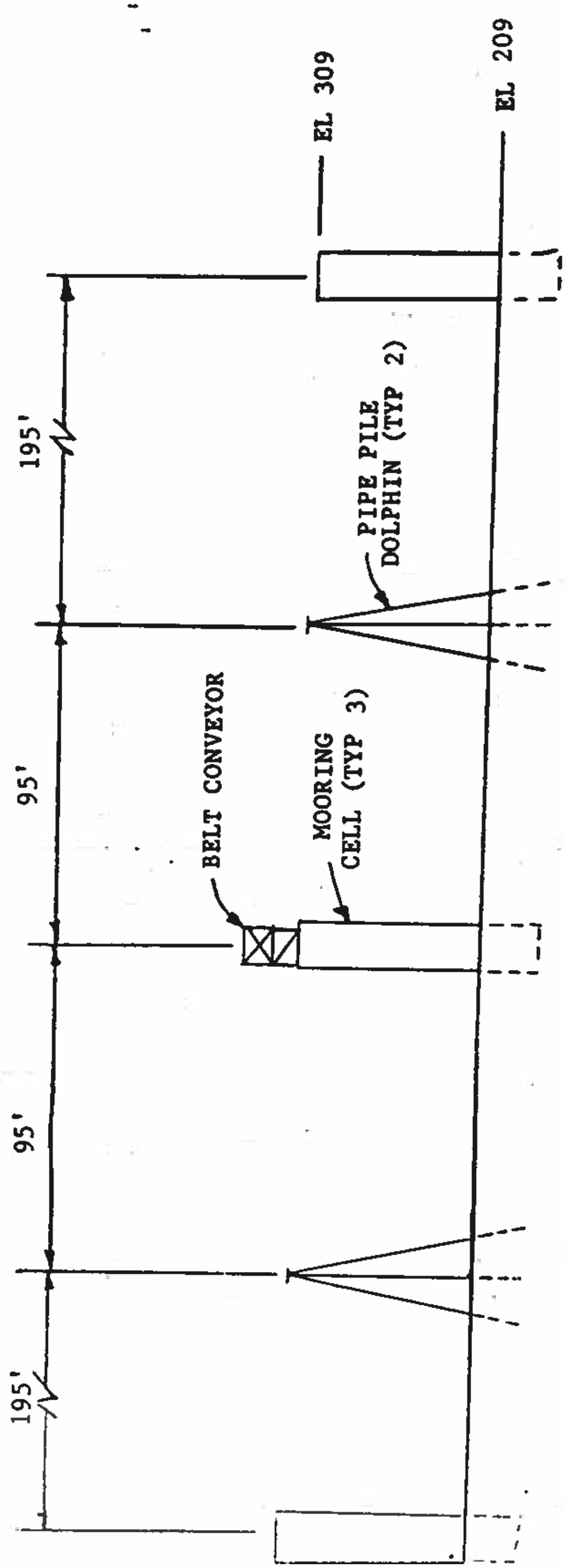
SECTION C-C

PROPOSED BARGE LOADOUT CONVEYOR

SCALE: 1" = 50'

PROPOSED ASH HANDLING FACILITIES ON
MISSISSIPPI RIVER AT APPROXIMATE MILE 884
COUNTY OF NEW MADRID
STATE OF MISSOURI

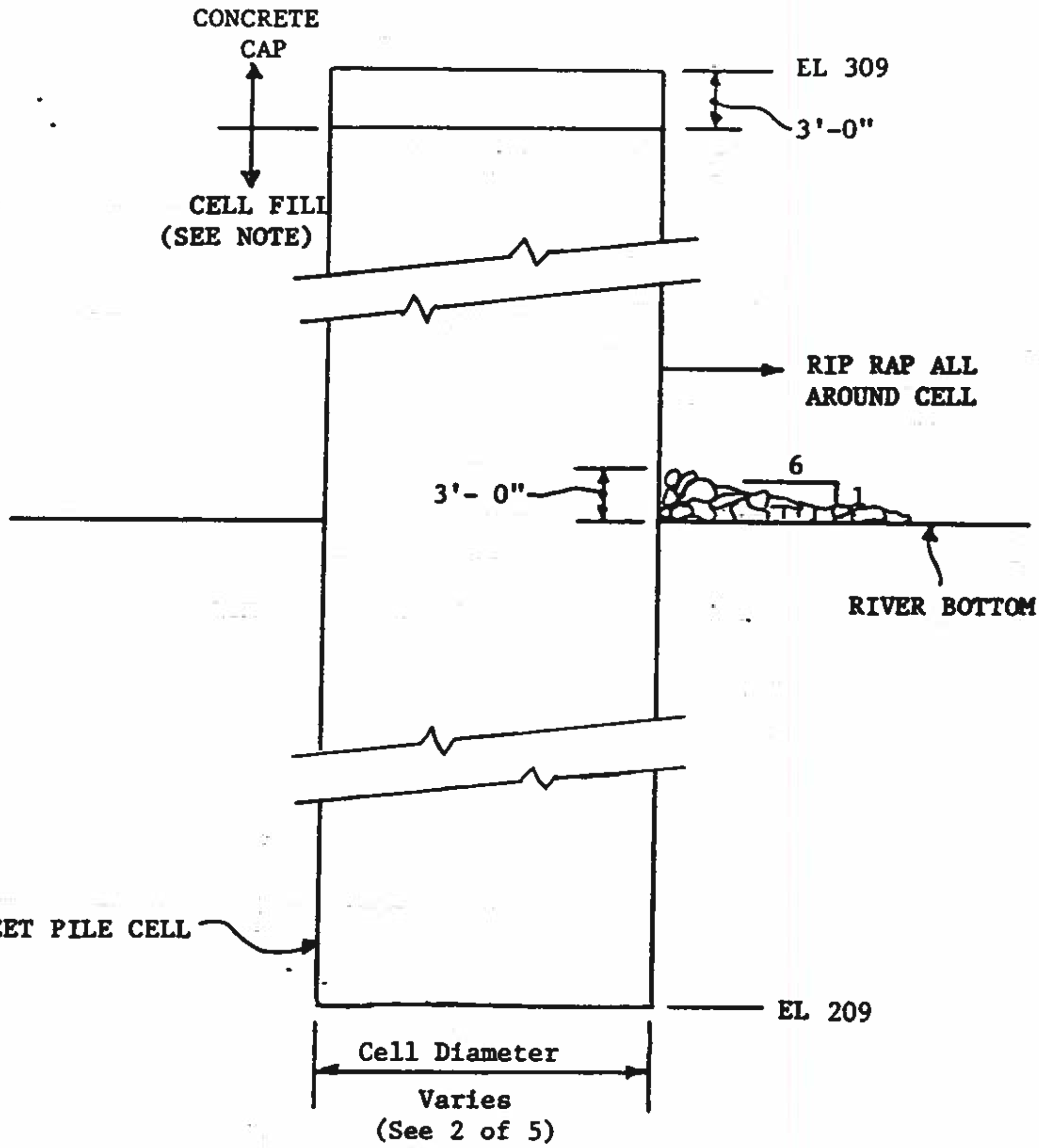
By Associated Electric Cooperative, Inc.
Date: April 2, 1984
Page 3 of 5



SECTION D-D
RIVER STRUCTURES

PROPOSED ASH HANDLING FACILITIES ON
MISSISSIPPI RIVER AT APPROXIMATE MILE 884
COUNTY OF NEW MADRID
STATE OF MISSOURI

By Associated Electric Cooperative, Inc.
Date: April 2, 1984
Page 4 of 5



DETAIL A

CELL STRUCTURE

NOTE:
CELL FILL TO BE CLEAN, WELL GRADED STONE AGGREGATE, WITH A MAXIMUM SIZE NOT TO EXCEED 6 INCHES. SANDSTONE, SHALE, OR OTHER MATERIAL WHICH WOULD DISINTEGRATE UNDER THE ELEMENTS WILL NOT BE USED.

PROPOSED ASH HANDLING FACILITIES ON
MISSISSIPPI RIVER AT APPROXIMATE MILE 884
COUNTY OF NEW MADRID
STATE OF MISSOURI

By Associated Electric Cooperative, Inc.
Date: April 2, 1984