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Volume 2: Geotechnical Characterization Report: Text, Tables, and Figures

Cherry Island Landfill Expansion Project Wilmington, Delaware

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1. INTRODUCTION

1.1 Terms of Reference

This Geotechnical Report presents the results of a geotechnical investigation for the design of the proposed vertical expansion of Cherry Island Landfill (CIL). This Geotechnical Report was undertaken to provide a comprehensive characterization of the current foundation conditions beneath the CIL and provides recommended parameters developed for design of the proposed expansion. This information will be used to support the calculations needed to demonstrate the stability and performance of the CIL under the proposed vertical expansion.

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1.2 Project Background

CIL is located in Wilmington, New Castle County, Delaware and is owned and operated by the Delaware Solid Waste Authority (DSWA) of Dover, Delaware. The CIL, which has been in operation since 1985, has been developed in five phases, identified as Phases I through V. The facility comprises a footprint area of 240 acres. The majority of the waste at the facility is municipal solid waste (MSW); although historically there has been a small amount of industrial waste (i.e., approximately 2 to 4 percent of the total disposed tonnage) placed at the CIL. Currently, portions of Phase I are used for the disposal of asbestos and for composting operations. Phases III through V are presently receiving all of the MSW and part of the non-MSW (i.e., fly ash and similar materials as daily cover). Phases I and II receive only industrial waste (mainly asbestos). DSWA proposes to expand the landfill vertically over the existing landfill footprint to provide additional landfill disposal capacity for at least 22 years.

The CIL was developed adjacent to the Delaware River in an area where thick, natural soft soils and man-deposited dredge exist. Preliminary calculations that consider future waste disposal rates indicate that stability of the soft foundation soils must be carefully considered. As part of the expansion project, DSWA proposes to improve the existing foundation soils to provide a design that is sufficient for the proposed waste placement configurations.

Under the current conditions, the very low hydraulic conductivity of the foundation soils will cause the strength gain of these soils to occur at a very slow rate. As a result, the necessary strength to the resist additional subsurface stresses in the foundation will not be available fast enough and unstable conditions may be prevalent for a long time. Therefore, the strength of the existing foundation soils must be increased and made available sooner to provide a sufficiently stable. The proposed method to improve the foundation strength and increase the rate of strength gain includes a combination of preloading and the installation of prefabricated vertical drains (PVD) installed along the periphery of the landfill (GeoSyntec, 2002a). The design of the PVDs as well as other elements of the design will be supported by this Geotechnical Evaluation Report.

1.3 Objectives of the Geotechnical Evaluation Report

The objectives of the geotechnical investigation are to: (a) collect, analyze, and synthesize geotechnical data related to the site; (b) characterize the materials that will affect the design of the proposed expansion; (c) develop recommended engineering parameters for design; (d) analyze the performance of the existing landfill under the proposed conditions; (e) support engineering analysis to demonstrate that the proposed CIL expansion design has an acceptable stability safety margin; and (f) develop data to be used during procurement of construction contractors.

The materials that will be considered and characterized in this report include the foundation soils and the new and existing waste. The physical and engineering properties of these materials have an important role in the design of the proposed CIL expansion. The soil parameters include soil classification, gradation, unit weight, consistency, plasticity, strength, compressibility, and hydraulic conductivity. The waste is characterized in terms of composition, unit weight, hydraulic conductivity, and strength. The report also presents the liquid conditions in the waste and a characterization of the water contained in the dredge under CIL.

1.4 Report Organization

The report is organized in five Binders. Following this introductory chapter, the remainder of Binder 1, which contains the main text, is organized as follows:

- Chapter 2 provides a general description of site conditions and the overall setting of the project;
- Chapter 3 describes the historic sources of geotechnical and hydrogeological data and presents the procedures used by GeoSyntec to conduct this Geotechnical Investigation;
- Chapter 4 presents a geological characterization that synthesizes the geological information available from historic reports and the newly acquired information;
- Chapter 5 presents a synthesized characterization of the water contained in the dredge under CIL;
- Chapter 6 presents synthesized geotechnical data, discusses results, and provides a characterization of various geotechnical parameters;
- Chapter 7 presents a synthesized engineering characterization of the waste;
- Chapter 8 summarizes relevant parameters that will be used in design; and
- Chapter 9 contains a list of references used in this report.

Appendices include results from the recent (i.e., 2002) investigation conducted by GeoSyntec, as well as the historical results previously reported by others. Appendices are organized in the remaining four binders of this report as follows.

Binder 2 – Appendices A through F include the 2002 and 2003 data developed by GeoSyntec and historic piezometric data.

- Appendix A describes the cone penetration test (CPT) procedures and presents the CPT results for tests conducted during 2002 and 2003;
- Appendix B describes drilling and sampling procedures and presents logs of the recently (2002 and 2003) advanced borings [includes one test with natural gamma, spontaneous potential (SP), and single point resistance (SPR)];

- Appendix C describes vane shear test (VST) procedures and presents the VST results for tests conducted during 2002;
- Appendix D contains descriptions of test procedures and results of the laboratory testing soil program conducted in 2002;
- Appendix E presents the results of the CPT dissipation tests conducted in 2002; and
- Appendix F presents records of piezometric data contained in the current (i.e., throughout December 2001) Geographical Information System (GIS).

Binder 3 – Appendices G through N present historical geotechnical data.

- Appendix G presents logs of historical borings;
- Appendix H contains historical CPT results;
- Appendix I presents historical VST results;
- Appendix J contains historical flat-plate dilatometer test (DMT) results;
- Appendix K contains historical pressuremeter test (PMT) results;
- Appendix L presents results of historical CPT dissipation tests;
- Appendix M presents results of historical geophysical logging and seismic surveys; and
- Appendix N presents historical laboratory testing soil results including:
    - gradation (grain size distribution);
    - unconfined compressive strength tests;
    - triaxial tests;
    - consolidation tests;
    - permeability tests; and
    - compaction tests.

Binder 4 – Appendices O through Q present the waste characterization report.

- Appendix O presents the results of the waste investigation program and identifies the methods used to characterize the engineering properties of the waste;

- Appendix P contains the results of a spectral analysis of surface waves (SASW) survey conducted at the site to help assess the engineering properties of the waste, as well as the potential variability of these properties; and
- Appendix Q presents historical records of waste settlements.

Binder 5 – Appendix R provides an account of the site historical development and includes a compilation of the results of previous testing that has been conducted at the CIL site itself, as well as at the Wilmington Harbor North (WHN) area located adjacent to the CIL. The contents of Appendix R were prepared by Duffield Associates, Inc. of Wilmington, Delaware.

2. SITE DESCRIPTION

2.1 Introduction

In this section, the general site conditions at the CIL site are described to provide a background for the more specific descriptions of the field investigation and characterizations that are presented in subsequent sections of this report. The descriptions provided in this section include the physiographic and geologic setting, a summary of the historic (i.e., pre-dredging) site conditions, the current site conditions, and the history of site development and filling.

2.2 Physiographic Setting

The CIL is located on Cherry Island, a tract of land located entirely in the generally flat Coastal Plain Physiographic province (Coastal Plain) and is bounded by the Delaware and Christina Rivers (Figure 2-1). Locally, a wedge-shaped mass (in a vertical cross section) of geologically unconsolidated clays, silts, sands, and gravels of river, estuary or marine depositional environments characterizes the Coastal Plain. The Coastal Plain is adjacent to the Piedmont Physiographic province. The Piedmont province consists of weathered pre-Cambrian metamorphic rocks (Schenk et al., 2000). The contact between the outcropping Piedmont and Coastal Plain province is called the "Fall Line" (Woodruff and Thomson, 1975; Terraqua, 1984) and lies approximately one mile northwest of the site, as shown in Figure 2-1. Starting from the "fall line," the Coastal Plain thickens vertically to the southeast toward the continental shelf. In the vicinity of the site, the Coastal Plain sediments range in thickness from approximately 95 ft north of the site to 220 ft on the southeast portion of the site (Terraqua, 1984). Much older metamorphic and igneous rocks underlie the Coastal Plain sediments in the vicinity of the site.

2.3 Geologic Setting

The geologic setting at CIL has been described in previous geotechnical and hydrological reports (e.g., Gilbert, 1981; Terraqua, 1984; Gannett Fleming, 1990a and 1990b). This section synthesizes the information presented in these earlier reports.

The following units (four natural and one artificial), listed from bottom to top, comprise the subsurface stratigraphy of Cherry Island:

(i) Wilmington Formation;
(ii) Potomac Formation;
(iii) Columbia Formation;
(iv) alluvium; and
(v) dredge.

The main characteristics of these features as they exist beneath the CIL are described below.

Wilmington Formation: The Wilmington Formation is the lowermost geologic unit at the site and is the crystalline basement rocks consisting locally of metamorphic and igneous rocks (Woodruff and Thomson, 1975). The rock in the upper portion of the Wilmington Formation is heavily weathered and is a weakly consolidated mixture of clay, silt, sand, and rock fragments. Although highly weathered, these materials are competent and relatively incompressible. Approximate depths of the Wilmington Complex range from 70 to 200 ft below the ground surface at Cherry Island (Gilbert, 1981; Terraqua, 1984; Gannett Fleming, 1990a and 1990b).

Potomac Formation: Overlying the Wilmington Formation is the Potomac Formation that consists of well-consolidated sediments, predominantly multicolored clay (Woodruff, 1981). The Potomac Formation is of early to mid Cretaceous age (e.g., 80 to 100 million years old) and is the basal unit of the Coastal Plain sediments. The Potomac Formation was deposited in a low-energy depositional environment (Woodruff, 1977) and resulted in discontinuous channel deposits of clay, silt, and sand. Due to erosion caused by deep streams and subsequent filling with younger granular deposits, interbedded layers of sand occur commonly in the Potomac Formation. It is common to encounter discontinuous (i.e., typically 5- to 10-ft thick) interbedded layers of sand within the Potomac Formation. The Potomac Formation is shaped as a wedge (in a vertical cross section) and thickens downdip generally to the southeast. At the site, the thickness of the Potomac Formation varies between approximately 40 ft near the Christina River and at least 150 ft at the confluence of the Delaware and Christina Rivers. The sand layers in the Potomac Formation are the major aquifer in the region (DNREC, 2003, personal communication).

Columbia Formation: The Columbia Formation consists of poorly sorted, multi-color, fine to coarse sand and gravel with varying amounts of silty sand, silty clay, and clayey silt lenses. The Columbia Formation, also part of the Coastal Plain, was formed in a fluvial environment during the Pleistocene (e.g., approximately 8,000 to 1.8 million years ago) and covered the Potomac Formation with predominantly granular sediments deposited by generally high-energy flows. The contact between the Columbia Formation and the Potomac Formation is defined by an erosional surface (unconformity) (Woodruff, 1977). The Columbia thickens to the southeast from about 10 to 15 ft to the northwest of the site to 60 and 70 ft to the southeast. The top the Columbia slopes gently to the southeast with an average slope of approximately 1 to 2 percent. This feature was probably caused by erosion toward the Delaware and Christina rivers after the last Ice Age, approximately 12,000 years ago, when the sea level was hundreds of feet lower than the present levels. The Columbia Formation maybe a local aquifer and is typically considered the source of recharge of deeper aquifers.

Alluvium: After the last Ice Age, the sea level rose, and a deposit of alluvium derived from low-energy flows overlaid the Columbia formation. The alluvium has been often referenced as "recent alluvium deposits" in previous studies (e.g., Gilbert, 1981; Terraqua, 1984; Gannett Fleming, 1990a and 1990b). The alluvium consists of unconsolidated deposits of Holocene age (e.g., approximately within the last 8,000 years). The alluvium is comprised of clayey silts and silty clays with varying amounts of sand and organic vegetative matter. Occasionally, peat lenses are also encountered in the alluvium. Discontinuous and irregular lenses of coarse sediments, mainly silty to gravelly sand, are encountered within the alluvium at various elevations along the boundary with the Delaware River. These lenses appear to be the remnants of a historic, buried dike constructed to reclaim land from the Delaware River. The alluvium thickness varies approximately between 30 and 40 ft at the site.

Dredge: The dredge consists of sandy silty clay to clayey silt with organic matter in some locations. These materials have been dredged from alluvial sediments lining the Delaware and Christina riverbeds. The dredge is also referred to as "dredge spoils" in previous geotechnical and hydrogeological reports, and as "dredge material" in the *Construction Drawings for Landfill Expansion, Contract S01445NC, Cherry Island Landfill, New Castle County, Delaware*). The dredge thickness varies from about 30 to 40 ft at the site. As with the alluvium, lenses of gravel and sand are encountered within the dredge at various elevations along boundary with the Delaware River. These lenses appear to be the remnants of a historic, sunken dike constructed to reclaim land from the

Delaware River. Original placement of the dredge is discussed in Section 2.6, while the engineering characteristics of the materials relevant to the landfill expansion design regarding above dredge are described in more detail in Chapter 4.

2.4 Site Location

The CIL is a tract of land within the limits of the City of Wilmington, locally known as Cherry Island. Cherry Island was partly reclaimed from the Delaware River and has been used for many years as a dredge disposal area. The center of the CIL is located at latitude 39° 43' 30"N, and longitude 75° 30' 50"W. A general site plan of the CIL is shown on Figure 2-2. CIL is bounded on the northeast by 12th Street and the City of Wilmington's sewage treatment plant, on the southeast by the Delaware River, on the southwest by the Christina River and the Wilmington Harbor North Dredge Disposal Area (operated by the USACE), and on the northwest by Interstate Highway phases, as described earlier, and include an administration building, maintenance buildings, and other miscellaneous structures.

2.5 Original Site Conditions

Cherry Island is adjacent to the confluences of the Christina River and Delaware River on the southeast, and the confluence of the Brandywine Creek and the Christina River on the northwest. Shellpot Creek originally bounded Cherry Island on the north. Historic maps also show that other smaller streams were present in the area before the site was developed as a dredge disposal area. Based on an interpretation of the local depositional and erosional conditions, it is believed that Cherry Island was formed as a result of complex fluvial depositional process that created a highly variable stratigraphy. It is possible that the Brandywine Creek played a prominent role in the deposition at the site based on the orientation of the site with respect to the Brandywine Creek and its drainage pattern.

Originally, the site was described in historic maps dated 1841 to 1883 as a "meadow" toward the west and a "tidal marsh" to the riverside. Therefore, the complex stratigraphy at the site is likely due in part to continuous episodes of emergence and submergence by the tidally influenced river. The upper stratum was defined in early geotechnical studies dating back to 1923 as "mud, sand", from approximately elevation -10 to -50 ft. A soil survey of New Castle County (USDA, 1970) described the surficial

soils on the eastern portion of the Cherry Island tract (most of current Phases III, IV, and V) as a tidal marsh with frequent flooding episodes.

The original Cherry Island tract occupied approximately 900 acres. The area had an approximate elevation of 3 to 8 ft above mean sea level (all elevations reported in this document are referenced to mean sea level) near the Delaware River, and approximately 12 to 15 ft, near the present Interstate Highway 95. Figures 2-3 and 2-4 show the circa-1923 condition of Cherry Island and the historic shoreline of the Delaware River. Superimposed on this aerial figure is the current landfill footprint. Historic records show that original shoreline at the site was different than the current shoreline and that that the 1920's high water line (approximate elevation of 3 ft) was located where the present boundary between Phase II and Phases III, IV, and V, while the low water line (i.e., approximate elevation of 0 ft) was situated roughly 1,200 ft to the east of the high water line. This map confirms that prior to development of the site, the water-line elevation varied over a relatively wide area due to the tide and flat topography.

2.6 General History of Land Use in the Vicinity of the Cherry Island Site

Records indicate that Cherry Island and adjacent areas along the Delaware River have been used for many years as a disposal area for the material dredged from the Delaware and Christina rivers to facilitate navigation at the Port of Wilmington, which is located across the Christina River from the CIL. Historic records indicate that dredging of the Christina River was initiated approximately in 1896. Two major disposal areas were created at Cherry Island: (i) the "Edgemoor" disposal area, on the eastern side of Cherry Island; and (ii) the "Wilmington" disposal area, on the western side. The Wilmington disposal area occupied approximately the same location as the Wilmington Harbor North Dredge Disposal Area where the USACE is currently placing dredge. The dredge was placed using a hydraulic filling process that produced a material with very high water content, extremely soft consistency, and total unit weights as low as 75 to 81 pounds per cubic foot (pcf). Appendix R includes a detailed account of the historic development of the Cherry Island tract by the USACE.

Review of historic records shows that most of the dredge was placed from 1896 until approximately 1927-1930, in the present-day Wilmington Harbor North Dredge Disposal Area, Phases I and II and the western-most portions of Phases III and IV. This

dredge area extended over the 1920's low water shoreline, as shown in Figure 2-4. This historical evolution of the dredge disposal areas is also shown in Figure 2-3.

Subsequent to this initial development, part of the Delaware River channel (i.e., the area east of the 1920's low water line) was reclaimed as a dredge storage area. The majority of this reclaimed area is beneath the present location of Phases III, IV, and V. Historical records indicate that this area was developed sequentially. Key development features are shown in Figure 2-4 and are summarized as follows. An existing timber bulkhead running along the Delaware River east of CIL was extended in 1910 and was referenced as the "Edgemoor training dike." This training dike provided better control over river flow conditions and protected the shoreline. The training dike demarked the eastern boundary of the Edgemoor Disposal Site. The area created between the training dike and the existing shoreline was approximately 300 acres and was subsequently used for dredge disposal. This area corresponds approximately to the eastern portions of Phases III, IV, and V, and other facilities north of the CIL. The training dike was constructed of dredge, gravel, boulders, and blasted rock. This dike is shown in Figure 2-3 and was approximately one mile long, extending apparently between Shellpot Creek and the Christina River. The training dike was approximately 106 ft wide at the base and 6 ft at the top. It apparently had an original elevation of approximately -13 ft at the base and 12 ft at its crest. Due to continued consolidation of the soft river sediments comprising the foundation of the dike, it is estimated that half of the original volume of the training dike currently exists below the original as-built elevation (for further information see Appendix R).

Dikes were subsequently constructed by USACE, in the area to provide additional dredge containment after 1954. The westernmost dike appears to have been constructed immediately inside of the Edgemoor training dike, approximately along the current boundary of Phases I through V. The remaining portions of the perimeter dike constructed at that time coincided approximately with the current boundaries of the CIL. A "baffle dike" was constructed inside the containment structure on the northeast corner of the CIL to allow a longer detention time of the dredge. Apparently, all of these interior dikes were constructed from the dredge, without the benefit of significant preconditioning (e.g., "ditching" to dry surficial soils). It is believed that the common USACE practice for dredge containment at that time considered these dikes as low-strength structures with variable soil conditions that received little engineering control during construction. As shown in Appendix R, typically undrained shear strengths of 350 to 400 pounds per square foot (psf) were used in the USACE slope stability analyses conducted at the time.

There is no conclusive information on the details of how the dredge was placed at the Cherry Island site. It appears from the historical photographs that the dredge was discharged into the disposal area from the south boundary near the Christina River and a sluice structure was situated in the northeast corner of the disposal area as shown in Figure 2-4. Although more coarse-grained deposits would be expected to occur near the dredge discharge and fine-grained sediments would tend to be deposited increasingly farther from this point, the grain-size distribution of samples of dredge obtained by USACE is not appreciably different across the site.

2.7 History of Landfill Development at Cherry Island

The CIL has been developed sequentially in five phases, identified as Phases I, II, III, IV, and V within the previously constructed containment dikes on top of the dredge. Phases I and II were developed through 1987 and 1989, respectively. After their initial development, Phases I and II were subsequently raised to form what was called Phase IA. Phases III through V continue being developed. Phase I and the northern area of Phase II accepted some asbestos as well as municipal solid waste (MSW). However, Phases III through V accepted only MSW. The current (summer of 2002) average top elevations of the waste are approximately: 95 ft in Phase I, 100 ft in Phase II, approximately 60 to 65 ft in Phase III, and 70 to 85 ft in Phases IV and V.

All phases of the CIL have a natural base liner system, not a man-made liner, typical of modern landfills. The original design considered the alluvium to act as a hydraulic barrier because of its inherent low hydraulic conductivity. All phases have a leachate collection system (LCS, or also referred as leachate collection zone in design drawings), although the characteristics and efficiency of the LCS vary greatly from phase to phase. In Phases I and II, the LCS consists of a system of horizontal drainpipes, not all of them necessarily properly connected. In Phase II, there is a system of horizontal drainpipes, which is overlain by a nominal 1-ft thick layer of fill (apparently with a sizeable portion of fines) placed directly on the dredge. In Phases III through V, the LCS consists of a nominal 1-ft thick layer of aggregate and a system of subhorizontal collecting drainpipes.

The interior slopes of the perimeter dike of CIL are lined in Phases III through V to prevent leachate from permeating into the dredge. The northeast portion of Phase III contains a geotextile layer placed between the waste and the top of the dredge. The

geotextile was placed as reinforcement with the objective of increasing stability. The interior slope of the perimeter dike in this area is lined with the 1-ft thick LCS overlying the geotextile and the natural soil liner. The interior slope of Phase IV is lined with the 1-ft thick LCS, a geosynthetic clay liner (GCL) and a geotextile over the natural soil liner. Phase V is lined with a 1-ft thick LCS, a GCL, and natural soil liner. Prior to the development of Phases IV and V, an approximately 100-ft wide zone of prefabricated vertical drains (PVD) was installed under the perimeter dike along the Delaware and Christina Rivers. The PVD zone extended from the boundary of Phases III and IV (on the east end) and to the boundary of Phases V and II (on the south end). The depth of the PVDs reportedly extends down to about 75 to 100 ft below the ground surface. This depth corresponds approximately to the Columbia Formation or into the alluvium above the Columbia Formation.

3. FIELD INVESTIGATION AND LABORATORY TESTING PROGRAM

3.1 Introduction

This section describes the historic geotechnical data that have been collected during previous field investigations at the CIL and the adjacent Wilmington Harbor North Dredge Disposal Area. Also, this initial section concludes with a discussion of the limitations of the historic information in relation to the data needs for design of the CIL expansion. The remainder of the section then describes the geotechnical data that were obtained from GeoSyntec's 2002 investigation, which included both field and laboratory testing programs.

3.2 Historic Data

3.2.1 Overview

3.2.1.1 Historic Data for the Cherry Island Site

The first geotechnical investigation at Cherry Island appears to have been performed by the USACE in 1923. Numerous geotechnical and hydrogeologic studies have been performed during the past twenty years at the CIL. These studies provide valuable hydrogeologic and geotechnical information to support the vertical expansion design. Figure 3-1 presents a timeline of the various geotechnical and hydrogeologic studies conducted at the CIL since 1981, the time period when extensive investigation for purposes of supporting the landfill design commenced. A chronological summary of the reports from these investigations and other related studies is presented in Table 3-1.

The information in these referenced reports was used to prepare the discussion that is presented in the remainder of this section. The information resulting from the field and laboratory tests related to these historic investigations was organized by category (i.e., piezometric data, laboratory data, etc), test type, landfill phase, and chronologically when sufficient information is available. Appendices A through E were organized to present the recent (2002 and 2003) geotechnical data, which are subsequently discussed in Sections 4 and 6. Following these appendices, the historical geotechnical information is provided in Appendices F through N of this report. A summary of all geotechnical characterization data is presented in Section 8.

A geographic information system (GIS) model was developed for DSWA to compile the historic geotechnical information from CIL (Pennoni and SEA, 2001). The GIS model was developed to assist DSWA in reviewing the large body of historic data and tracking the results of the ongoing monitoring program in a user-friendly manner. The ultimate objective of the GIS model and the accompanying database was to facilitate the management of the data used in the planning and performance monitoring of the CIL. The GIS model and the accompanying database were provided by DSWA to GeoSyntec, and they were used in the preparation of this report. Data from the database were reviewed to avoid using incorrect or unrepresentative entries. When historic data were missing from the database, the original source was used, when available. If data were found to be inconclusive, they were not used (e.g., apparent incorrect orientation of the inclinometer lateral deflection). The database provided to GeoSyntec in January 2002 contains geotechnical data obtained up to approximately the third quarter of 2001.

Table 3-2 presents a summary of the number of historic field tests conducted within specific phases of the CIL. As shown in this table, previous field investigations produced geotechnical information from:

- borings;
- cone penetration tests (CPTs) (including mechanical, piezocone, dissipation, and seismic CPTs);
- vane shear tests (VSTs);
- flat-plate dilatometer tests (DMTs);
- pressuremeter tests (PMTs); and
- geophysical logging [natural gamma, spontaneous potential (SP), and single point resistance (SPR)].

In addition, hydrogeologic data were obtained using the following types of test/instruments:

- slug tests;
- well yield tests;
- monitoring wells;

- piezometers; and
- tidal fluctuations.

Additionally, geotechnical monitoring instruments were installed over the years. These instruments included: (i) inclinometers; (ii) total-pressure cells; (iii) settlement plates; and (iv) thermistors.

The following sections describe succinctly the scope of the main geotechnical testing techniques used in CIL and identify potential limitations.

3.2.1.2 Historic Exploratory Borings

Geotechnical borings have been the most commonly used method to investigate subsurface conditions at the CIL. Table 3-3 identifies 135 exploratory borings conducted at the site prior to the current investigation program. In addition, many borings are identified in the GIS as monitoring wells and boreholes for instruments. Figure 3-2 shows a plan view with the location of 83 records. Borings were advanced to retrieve samples for geotechnical soil investigations and to install piezometers. Reproductions of the historic boring logs that were available to GeoSyntec are included in Appendix G. Boring depths range from approximately 10 ft to approximately 250 ft, with an average depth of about 70 ft. Approximately 90 of the historic borings reached the Columbia Formation, 31 reached the Potomac Formation, and nine borings reached weathered bedrock (Gilbert, 1981; Terraqua, 1984). Most borings logs provide both a soil description and the soil penetration resistance, expressed as number of blows per foot (N) as obtained with the Standard Penetration Test (SPT).

3.2.1.3 Historic Cone Penetration Tests

A total of 29 CPT soundings were previously advanced at the site. Table 3-4 presents a summary of location and depth of the 29 CPT soundings conducted at the site prior to this investigation. Figure 3-3 shows a plan view of the CPT locations. Available records of the historic CPTs are included in Appendix H. Twenty-three CPT soundings were performed in 1998 (SEA-10s series) and six were conducted in October 2001 (SEA-100s series). The historic CPT sounding depths varied from approximately 18 ft to approximately 95 ft, with an average depth of about 55 ft.

The CPT records were used to supplement historic borings, and were particularly helpful in defining the subsurface stratigraphy. The CPT results were also used to provide the cone penetration tip resistance, friction resistance, and cone dynamic pore pressure, which were used to correlate undrained shear strength and other soil parameters. Dissipation tests were conducted at selected elevations within the fine-grained deposits (i.e., dredge and alluvium). These tests provide an in-situ estimate of the horizontal hydraulic conductivity (k_h) or the horizontal coefficient of consolidation (c_h). Records of these CPT dissipation tests are included in Appendix L. Four seismic CPTs (i.e., SEA-10, SEA-12, SEA-14, and SEA-19) were advanced to depths ranging from 3 to 100 ft to obtain shear wave velocity via geophysical methods. Records of the historic seismic CPTs are included in Appendix M.

3.2.1.4 Historic Vane Shear Tests

Field vane shear tests (VSTs) were conducted in 24 borings at the site. Table 3-5 presents a summary of the historic VST locations and maximum depths. These locations are shown in the plan view on Figure 3-4. Appendix I presents plots and information on the historic VSTs. The depth of the VSTs ranged from 20 to 120 ft, with an average of 75 ft.

3.2.1.5 Historic Dilatometer Tests and Pressuremeter Tests

Other in-situ tests were utilized to estimate the undrained shear strength of the dredge and alluvium, namely the flat-plate dilatometer test (DMT) and the pressuremeter test (PMT). Table 3-6a and 3-6b presents a summary of the location and depth of the historic DMTs and PMTs performed at the site. Figure 3-5 shows the location of DMTs and PMTs. Records of the DMTs and PMTs are included in Appendices J and K, respectively. The data from the DMTs were used to establish the stratigraphy (soil type classification), the undrained shear strength of dredge, coefficient of lateral earth pressure at rest (K_o), and other soil parameters. Only four PMTs were conducted and the data were inconclusive. Similarly, it is noted that of the eight DMTs conducted, only a few were performed in the dredge.

3.2.1.6 Historic Geophysical Logging

A total of nine geophysical tests including natural gamma, spontaneous potential (SP), and single point resistance (SPR), were performed during 1983 in borings later used as monitoring wells in the Potomac Formation (i.e., borings P-100 through P-108). These records are included in Appendix M contained in Appendix.

3.2.2 Historic Laboratory Tests

An extensive amount of data has been generated from historical soil laboratory tests conducted on CIL foundation soils. The number of tests performed within each phase of the CIL is presented in Table 3-7. Results from these laboratory results are included in Appendix N, which contains results of:

- particle size distribution tests;
- unconfined compressive strength tests;
- triaxial tests;
- consolidation tests;
- permeability tests; and
- compaction tests.

The review of historic data showed that the majority of the data could be used for the site characterization; however, some important laboratory testing data were not properly evaluated, as will be discussed later in this chapter, while some other data were not conclusive. Historic laboratory testing results were compared with the recent laboratory testing results obtained during this investigation. Comparisons and related discussions are presented in Chapter 6.

3.2.3 Historic Geotechnical Monitoring Instruments

Settlement plates and thermistors were installed beginning in 1991, and inclinometers and total pressure cells were at later dates, beginning in 1996. Figure 3-1 summarizes the installation dates of these geotechnical instruments. The locations of the 52 inclinometers, the 42 settlement plates; the 32 total pressure cells, and the 28 thermistors are shown in Figures 3-6, 3-7, and 3-8, respectively. In addition, numerous piezometers were installed along the dike and inside CIL. Data from the piezometers

are discussed in Chapter 5 and are presented in Appendix F. Records of the settlements are presented in Appendix Q.

3.2.4 Historic and Recent Geotechnical Information from USACE

As presented earlier, it appears that the first geotechnical investigation at the CIL site was performed by the USACE in 1923. Subsequently, USACE performed other geotechnical studies at or in close proximity to CIL. Twenty-six SPT borings were performed between 1967 and 1984 at the Edgemoor Disposal Area, with five of them drilled within the limits of CIL, most through the perimeter dikes. Copies of the original USACE logs are included in Appendix R, which describes the historic site development and presents early geotechnical data prior to landfill operations.

The USACE borings included continuous and discontinuous, disturbed (SPT) and undisturbed sampling. VSTs were performed in some of the borings. USACE installed eleven piezometers at the Edgemoor Disposal Area between 1972 and 1974. USACE also performed geotechnical laboratory testing on undisturbed samples. These tests included natural moisture content, unit weight, hydrometer analysis, Atterberg limits, specific gravity, consolidation, unconsolidated-undrained triaxial testing, and consolidated-undrained triaxial testing with pore pressure measurements. Copies of the laboratory testing results are also included in Appendix R.

More recently, the subsurface conditions of the adjacent Wilmington Harbor North Dredge Disposal Area have been studied in support of USACE objectives to increase the disposal capacity of that facility. Duffield Associates (2001a, 2001b) has performed geotechnical studies including drilling of borings, SPT sampling, laboratory testing, and CPTs. These tests were reviewed to provide supplementary geotechnical information to characterize the subsurface conditions at CIL. Overall, test results from adjacent locations and at the same elevations are comparable to those obtained previously by others and by GeoSyntec.

3.2.5 Summary and Limitations of Historic Data

The extensive historic geotechnical data provide abundant information that helps in the overall site and area characterization. The most useful information is included in: (i) boring logs, including SPT blowcount values; (ii) CPT records; (iii) VST results; (iv)

DMT results; (v) settlement records; and (vi) laboratory results, including basic soil parameters and soil shear strength. Specifically, boring logs, CPT and laboratory results were used to assist in the delineation of the thickness and contacts of soil deposits. Likewise, field records and laboratory results provide estimates of soil basic properties and shear strength. Settlement records can be used to assess the compressibility characteristics of the dredge and alluvium.

The historic field investigations that provide the existing information appear to have been conducted according to generally accepted good practice and had apparently appropriate scope for the objectives of previous design efforts. However, the historic information has some limitations in regard to the proposed landfill expansion as described below.

- The SPT data do not provide an accurate stratigraphy where sampling and testing is discontinuous, as it has been commonly done at CIL. As a consequence, thin layers of soft soil, which can have a significant and detrimental impact on stability, may be totally missed by SPTs.

- For the same reasons presented that soft layers may be missed, the contact between alluvium and sand layers may be in error by a few feet. Therefore, the stratigraphy under most of the dike, which will provide the most stability in the proposed design, has not been sufficiently defined during previous investigations to the level of detail to allow a reliable design.

- As the dredge and alluvium deposits are the most critical layers from the stability viewpoint, a thorough characterization of the shear strengths of these layers is essential. SPT blowcount provides a general indication of strength, yet it is not a sufficiently quantitative indicator of the shear strength of the very low strength dredge and alluvium. Unfortunately, the current or any SPT data, which is the most common at the site, is not capable in providing an acceptable characterization of strength in the dredge and alluvium.

- The lack of stratigraphy details from SPT can be (and to some it has been) compensated by the use of the CPTs. However; relatively only ten (10) CPTs were advanced through the dike, the most critical area for the expansion design. The existing CPT information appears to be useful and mostly adequate for the purposes of initial Phase development. However, this information has not produced a well-defined stratigraphy under most of the

dike; therefore, the existing CPT information is not sufficiently to allow a reliable stability assessment of the perimeter dike system.

- Along the boundary of the CIL with the Delaware River, where slope stability appears to be most critical due to the presence of thicker soft deposits, the CPT soundings are spaced approximately at 1,000- to 1,500-ft intervals. Along the north boundary of Phases I and II, where slope movements took place in 1992, CPTs are spaced up to approximately 2,000-ft long intervals. The spacing of these soundings is excessive to develop a detailed and thorough stratigraphy for the proposed vertical design expansion.

- The VST was used numerous times in CIL. However, most tests were performed under the landfill footprint, not along the dike. Therefore, the historic information on the most critical area is limited. Additionally, the interpretation of the undrained shear strength from VSTs was found to be incorrect because soil plasticity parameters, which affect the shear strength interpretation, were not evaluated correctly.

- All of the laboratory data have been reviewed. Most of the historic laboratory results have proved valuable in helping characterize the foundation soil during this study. However, some critical laboratory testing data was incorrectly interpreted or is considered to be inconclusive. Specifically, some results related to Atterberg Limits, shear strength and compressibility were found to be in error or dubious. To compensate for this limitation, new laboratory soil tests were considered necessary for the proposed vertical expansion.

- Historic geotechnical information may not provide conclusive information regarding the effects of strength anisotropy.

To complement the extensive geotechnical data available from the historic investigations and to overcome the shortcomings identified above, a geotechnical investigation was conducted by GeoSyntec as part of this project. A description of this investigation is presented below.

3.3 Geotechnical Information Obtained during this Investigation

3.3.1 Field Investigation for Landfill Expansion

3.3.1.1 Introduction and Objectives

To support the analyses needed for the vertical expansion design, GeoSyntec conducted a field investigation at the CIL with the following objectives:

- obtain more detailed information regarding the stratigraphy along the dike and in the interior of the CIL immediately adjacent to the perimeter dike;
- supplement the historic data regarding the consolidation properties, in particular the coefficient of vertical and horizontal consolidation using field data (i.e., records of settlement, C_c and C_v);
- supplement information on the undrained shear strength profile within the dredge and alluvium;
- assess the strength gain with time in the interior of landfill, under the dikes, and in the existing PVD field;
- assess the extent of organics content and its influence on engineering properties;
- characterize other soil properties that may have a bearing on the various stabilizing techniques that are considered (i.e., shear strength, compressibility properties, saturated unit weight, and angle of internal friction);
- understand the conditions at the waste-LCS-dredge interface;
- characterize the unit weight and shear strength of the waste;
- understand the liquid conditions in the landfill; and
- characterize the dredge water conditions in the soil.

The field study included an extensive geotechnical investigation of the dredge and alluvium at strategic locations across the site. In addition, a component of the investigation focused on assessing waste properties.

The geotechnical investigation included the following types of tests. Specific testing objectives are identified in parenthesis.

- 59 CPTs in soil (stratigraphy and strength profiling, and pore water dissipation tests);
- nine CPTs in the waste (stratigraphy profiling and pore water dissipation tests);
- exploratory borings in soils under the landfill and along the dike (stratigraphy, soil penetration resistance in cohesionless soils, VST in fine-grained soils, dredge pore water level observation, and disturbed and undisturbed sampling for laboratory tests);
- exploratory borings in the waste (waste sampling, liquids observation and monitoring, and waste sampling for laboratory tests);
- large-diameter auger borings in waste (unit weight characterization);
- vane shear tests in selected borings (undrained shear strength);
- spectral analysis of surface waves (SASW) testing in waste and soil (unit weight and shear wave velocity profiling);
- 26 SPTs (related to an ongoing PVD pilot test program to verify stratigraphy, conduct VST, obtain samples, install geotechnical monitoring instruments);
- 18 CPTs (related to PVD pilot test program to verify stratigraphy, measure strength gain in fine-grained soils where PVDs were installed);
- additional VSTs (related to PVD pilot test program; to measure strength gain in fine-grained soils where PVDs were installed); and
- natural gamma, spontaneous potential (SP), and single point resistance (SPR), measurement in one borehole (related to PVD pilot test program, to verify stratigraphy).

Figure 3-9 shows the location of tests and sampling conducted during the GeoSyntec geotechnical investigation. For reference, the location of historic borings and other geotechnical tests conducted prior to this study are also shown.

The following sections describe the activities undertaken during the recent geotechnical investigation. The results of the soil characterization are presented in Appendices A through N. The results of the field activities to characterize waste properties are included in Appendix O. Results of the SASW survey are included in Appendix P. A brief discussion of the general conditions (e.g., depth, location, type of test, etc.) related to the 2002 and 2003 investigation follows.

3.3.1.2 Recent Cone Penetration Tests

A total of 68 CPTs were advanced in the five phases of the CIL (Figure 3-9). The CPT soundings ranged from 70 to 120 ft in depth below the existing ground surface. The location and depth of the CPT soundings are summarized in Table 3-8. Eighteen (18) additional CPTs were performed in relation to an ongoing PVD pilot test program. These additional CPTs are not discussed in this report.

During the period of 28 January through 14 February 2002, 59 CPT soundings were performed at the site. A total of 50 CPTs were advanced along the perimeter of the CIL (i.e., CPT C001-C050). Eight of the remaining nine CPTs, (i.e., C051-C058) were advanced through the waste into the underlying foundation. CPT C059, was advanced in the waste, but did not extend into the underlying foundation.

From 11 April and 16 April 2002, nine CPTs were performed at seven locations within the landfill waste disposal footprint. Tests C060, C062, C067, and C068 penetrated into the underlying soils, while the remaining five (i.e., CPTs C061 and C063-C066) encountered refusal within the waste.

The CPT soundings were performed using an electric piezo-cone of either a 10-cm^2 cross-sectional area and a 150 cm^2 of friction sleeve area, or a 15-cm^2 cross-sectional area and a 225 cm^2 of friction sleeve area. The probe with the 10-cm^2 cross-sectional area was used to increase the sensitivity of measurements in very soft layers. The probes were advanced at approximately 2 cm/sec. The cone tip resistance, sleeve friction, and pore pressure were recorded electronically at approximately 4-in. intervals as the soundings were advanced. Individual logs of the CPT are presented in Appendix A. The plots present the cone tip resistance that is corrected for unequal end area (Q_t) sleeve friction (F_s) friction ratio (R_f), and dynamic pore pressure (u). The plotted values represent local averages that were obtained by averaging measurements within 10-cm long depth intervals. These parameters allowed the user to infer soil types shown on the right side of the logs. These inferences used existing correlations as presented by Robertson and Campanella (1983a), (1983b), (1985), (1988). Other results from CPT and their significance are discussed in Chapters 6 and 7.

3.3.1.3 Recent Exploratory Borings

A total of 26 exploratory borings were drilled in the five phases of the CIL (Figure 3-9) to characterize the subsurface soil conditions, observe liquid elevations, perform VSTs, and retrieve samples. Table 3-9 presents a summary of the location and depth of each boring. Additional 26 SPT were drilled in relation to an ongoing PVD pilot test program. These additional SPTs are not discussed in this report.

Twelve exploratory borings, (i.e., S01 though S12), were drilled near the perimeter of the CIL. Ten of these borings were drilled through the dike while two were drilled along the outside toe of the dike. Boring spacing ranged from approximately 500 to 600 ft. Borings through the dike had depths varying between 120 and 160 ft, which in most cases corresponded to a termination depth within the Columbia Formation.

Fourteen exploratory borings were drilled through the waste in each of the five phases. Two types of borings were drilled:

- Five borings, (i.e., S13 though S17), were drilled through waste and approximately 20 ft below the elevation of the LCS. The focus of these borings was the upper portion of the dredge immediately under the waste. The objective of these boreholes was to obtain undisturbed samples of the dredge for laboratory testing and to perform VSTs in the foundation dredge. The depth ranged between 51.5 and 85 ft below the ground surface.
- Nine additional conventional borings (i.e., S1W-S5W, S5AW, S6W-S7W, and S7AW) were drilled through the waste and a short distance under the leachate collection system (LCS) in seven locations (two were drilled at the same location due to drilling difficulties). The focus of these borings was the waste material itself. The objective was to observe liquid levels within the waste, obtain waste samples for characterization and laboratory testing, inspect the conditions of the waste/LCS layer/dredge interface, and investigate the conditions inside the LCS. The boring depths ranged from 13 to 59.5 ft below ground surface. Four of these borings encountered refusal in the waste; one reached the top of the LCS, and the remaining four penetrated into the dredge, to a maximum depth of approximately 3 ft under the LCS. The findings from these borings related to level of liquids in the waste are presented in Chapter 5. Other findings from these borings are discussed in the waste characterization report included in Appendix O.

- Nine large-diameter borings (bucket auger borings, designated as UW02, UW02A, UW03, UW04, UW05, UW06, UW07, UW08, and UW09) were drilled through the waste to observe standing liquid levels and perform a waste field unit weight determination. The findings from the bucket auger borings related to level of liquids in the waste are presented in Chapter 5. Other findings are discussed in the waste characterization report included in Appendix O.

The drilling method used in the conventional 12 borings outside the waste (i.e., borings S01 though S12) involved the use of drilling fluid below a certain depth to avoid the collapse of the boring and to preserve, to some extent, the overburden pressure of the soil at the sample locations to reduce sample disturbance. For the borings drilled through the dike, dredge water level readings were taken at the time of drilling, when the drilling method allowed. Before drilling fluid was introduced to the borehole, water level readings were taken, as noted in the boring logs. In the conventional borings through waste (i.e., S13-S17 and S1W-S7AW) no drilling fluid was used due to difficulty of maintaining the level of drilling fluid in waste.

Both disturbed (i.e., driven) and undisturbed (i.e., pushed) representative samples were obtained at selected depth intervals in borings. In the borings through the dike, soil samples typically were obtained between depth intervals of 5 ft (in the top 30 ft of each boring) and 10 ft (below a depth of 25 ft). For the disturbed samples, SPT split-spoon samplers were driven while the blow count being recorded to obtain the soil penetration resistance. In the borings drilled through the waste and into the upper dredge, undisturbed samples were obtained using Shelby tubes every 10-ft and 5-ft intervals. In boring S20, which was drilled in the middle of pilot test area 6 (see Figure 3-9), the sampling was continuous between depths of 100 and 130 ft to investigate in detail the location of the alluvium-Columbia Formation contact and the gradation of materials at these elevations.

Recovered soil samples were field classified, sealed, and transported to the soils laboratory for testing. Appendix B contains a description of the procedures utilized in this field investigation and logs of Borings S01 through S17. Results are discussed in subsequent chapters. Boring logs for boreholes S1W through S7W can be found in Appendix O. This information also includes a discussion of the procedures followed for the borings through the waste.

3.3.1.4 Recent Vane Shear Tests

Vane shear tests (VSTs) were conducted in ten of the twelve borings advanced from the dike and in borings advanced through the waste. VSTs were run at selected elevations to obtain the in-situ undrained shear strength of fine-grained soils. The VSTs were performed at two to four locations in the each of the ten borings selected. The depths of the test varied between 15 and 97 ft. The test locations were selected such that silty/clayey soil with no or little sand was encountered based on previously assessed stratigraphy. In the borings advanced through the waste, the VSTs were performed in four or five locations, usually spaced at 5-ft intervals. Table 3-10 presents a summary of vane shear test location and test depth. Results from the VSTs are presented in Chapter 6. Appendix C contains a description of the procedures utilized in this investigation and the results of the VSTs. Additional VSTs were performed in relation to an ongoing PVD pilot test program; however, these additional VST results are not discussed in this report.

3.3.2 Recent Laboratory Tests

Index and engineering properties were performed on selected representative samples to characterize the various soils encountered at the site. Laboratory tests consisted of:

- particle size distribution (with and without hydrometer);
- Atterberg limits;
- organics content via loss on ignition;
- natural moisture content;
- unit weight;
- compressibility via oedometer consolidation tests; and
- shear strength parameters:
  - isotropically consolidated undrained triaxial compression (CIUTC) tests);
  - consolidated direct shear tests (DST) on samples trimmed horizontally and vertically; and
  - direct simple shear tests (DSS).

Representative soil samples recovered from Shelby tubes were used for determination of unit weight, natural moisture content, Atterberg limits, consolidation

tests, and shear strength tests. Samples obtained with the SPT sampler were used for determination of natural moisture content, Atterberg limits (fine-grained soils), and grain size distributions.

The basic soil parameters, such as plastic indices, gradation, unit weight, and water content were determined by a total of 61, 51, 15, and 104 tests, respectively. Additional laboratory tests were conducted during the pilot test program. Compressibility and shear strength were determined in 30 and 27 tests, respectively. A summary of the laboratory test data and the relevant ASTM methods are presented in Table 3-11. The most relevant laboratory results are also noted in the logs of borings included in Appendix B. Results are discussed in Chapter 6.

4. SUBSURFACE CONDITIONS

4.1 Overview

The goal of this chapter is to describe the general subsurface condition of each soil deposit. Specifically, the native soils and bedrock consist, from top to bottom, of: (i) alluvium; (ii) the Columbia Formation; (iii) the Potomac Formation; and (iv) weathered bedrock. In addition, in the interior of Phases II through V, dredge overlays the native soil deposits. Based on the information depicted in the profiles, the subsurface conditions at the site are described, from the ground surface and down, in this chapter.

4.2 Basis for Developing Soil Profiles

The CPTs and soil borings performed during the 2002-2003 study, as well as the historic CPTs obtained by others, were used as the primary means to develop the stratigraphy for the CIL site. The CPT data included the corrected cone tip resistance and the friction ratio. By plotting these two cone parameters side to side at the same elevation, it is possible to identify thin (i.e., approximately 2- to 3-in. thick) strata, based on existing correlations between cone parameters and soil types (Robertson and Campanella, 1983a, 1983b, 1985, 1988).

To facilitate reference of the CPTs along the perimeter dike, a stationing system around the landfill was created, commencing on the northwest side of the site near the entrance (Figure 4-1). Each soil profile along the dike was not more than 2,500 ft long. Using these data, longitudinal profiles of soil stratigraphy were generated along the perimeter dike (Figures 4-2 through 4-8), and along seven cross sections (Figures 4-9 through 4-15), at the locations shown in Figure 4-1. Table 4-1 summarizes the location of the soil profiles. Table 4-1 also shows the number of both historic and recent CPT soundings used to develop each soil profile.

The seven soil profiles along the perimeter (i.e., Figures 4-2 through 4-8) also show profiles of penetration resistance (SPT blowcount) and VST-based undrained shear strength obtained during the 2002-2003 field exploration. The soil descriptions contained in the boring logs were also incorporated in soil profiles whenever possible. Descriptions of stratigraphy and general conditions in each soil deposit, which are presented subsequently, are based on these profiles.

Figures 4-16 through 4-18 present the profiles of penetration resistance obtained from the 2002 field exploration SPT borings along the dike. Specifically, the blow count in the crust of the dredge ranged from 5 to 15, and in the dredge and alluvium from 0 to 5. Larger blow counts were recorded in sand lenses interbedded in the dredge and alluvium. Blow counts in the Columbia Formation ranged typically from 20 to 60, and in the Potomac Formation from 30 to 50. The location of vane shear tests and location of Shelby tube samples is also noted on Figures 4-16 through 4-18.

The stratigraphy presented in the soil profiles is consistent in most locations with those represented in previous reports. In a few locations, where the 2002 information was not sufficient to delineate the stratigraphy, the closest historic information was utilized to supplement the 2002 information.

4.3 General Soil Conditions

4.3.1 Dredge

The dredge is either nonexistent or has a negligible thickness in Phase I (Figure 4-9). Based on topographic information obtained shortly before landfill operations were initiated, it is estimated that the top of dredge is currently approximately at elevation 30 to 35 ft at the center of Cell II (Figure 4-10) and approximately at elevation 38 to 42 ft at the center of Cells III-V (Figure 4-11 through 4-15). The bottom of the dredge could not be easily identified from the underlying alluvium in borings and from CPTs due to the similarities of soil type, color, and consistency of these materials. The bottom of the dredge was estimated from interpretations of historical maps, bathymetric maps, geological descriptions, and site conditions before dredge disposal was initiated (e.g., Figure 2-3). Based on the interpreted elevation location of the alluvium prior to dredging, the dredge thickness is estimated to increase from approximately 20 ft at the center of Cell II to about 30 to 35 ft near the center of cells III through V. The thickness of the dredge and granular materials used as fill to reclaim land is estimated to increase up to 50 to 60 ft southeast of the historic low water line (see Figure 2-3).

The dredge tends to have a stiff crust, possibly due to desiccation. Along the dikes, the crust on top of the dredge is stiffer because of the effects of compaction and/or desiccation. Borings also showed that in some locations the upper layer is composed of fill containing gravel, sandy silt, and common fill. The thickness of the stiff, upper layer along the dike varies approximately between 5 and 10 ft. A dashed

line demarks the boundary of the upper layer in the soil profiles along the perimeter dike (Figures 4-2 through 4-8). In the interior of the landfill, however, the original upper and stiffer portion of dredge could not be identified. The uncorrected SPT blowcounts range typically between 12 and 26 (with occasional blowcounts > 50) in the stiffer crust. CPT cone tip resistance values are on the order of 100 tsf in this layer.

The dredge, which was placed by hydraulic filling, is composed of a dark gray-to-gray, clayey to elastic silt with variable (trace to little) amounts of fine sand. It also contains some mica. In general, the plasticity index (PI) of the silt is high. Average values of the PI are approximately 50 to 55. At some locations, the silt was found to be of medium plasticity (i.e., $10 \leq PI \leq 25$). A more detailed discussion of the distribution of plasticity (aerial and with depth), and the implications of plasticity to other engineering properties included in Chapter 6.

The dredge is generally classified as inorganic, with organic content varying up to 12% (refer to Chapter 6). Figures 4-16 through 4-18 show that organics extend variably within the dredge. In some areas near the western boundary of Phases III-V, organics occur in a few locations (e.g., borings S09 and S10 in Figure 4-17). In other locations, organics extend within most of the dredge (e.g., borings S011 in Figure 4-16, S05 in Figure 4-17, and S12 in Figure 4-18). In addition, observations made in borings as well as CPT results indicate that in some areas the dredge consist of as peat (e.g., Figures 4-6, 4-8, 4-12, and 4-14).

Under the stiff upper layers mentioned above, the consistency of the dredge is soft to very soft. The uncorrected SPT blowcounts in the soft portions of the dredge are low, ranging between 0 and 4. CPT cone tip resistance values in this area are on the order of 10 tsf. Due to consolidation under the load imposed by the waste, the consistency tends to become stiffer with time. However, this effect is not presently pronounced because of the little consolidation that has taken placed so far in the relatively short time elapsed since waste placement. It is expected that the full consolidation and stiffer consistencies in the dredge and underlying alluvium will be achieved over hundred years from to date.

Although still characterized as soft to very soft, the perimeter dike, composed mainly of dredge, is typically stiffer and stronger than the dredge inside CIL. The material of the dike was compacted and subjected to partial desiccation. Based on the review of historic records, it is estimated that the as-compacted total densities (possibly on the order of 100 pcf) were significantly lower than those achieved in modern

engineered fills. In addition, historic records and the variability of encountered conditions suggest that the engineering control was poor during dike compaction.

4.3.2 Alluvium

As the alluvium can be considered the parent material of the dredge in a general case, some of the characteristics described for the dredge are applicable to the alluvium. However, in this report the distinction is made between dredge and alluvium, whenever enough information was available. In particular, efforts were made to delineate the present location of the dredge-alluvium contact. In the previous section, where the features of the dredge deposit were discussed, it was mentioned that historical maps, bathymetric maps, geological descriptions, and site conditions depicted in construction drawings were used to delineate this contact. Additionally, geotechnical information was used to define the alluvium-dredge contact. Although the alluvium is expected to be a more structured material due to the natural sedimentation, it was not always possible to detect significant differences between the two materials. However, some engineering properties of the dredge and alluvium exhibit some differences, as discussed in Chapter 6.

The thickness of the alluvium varies across the site but it tends to be more homogeneous than the dredge. The minimum thickness is approximately 30 ft and occurs under the center of Phase I (see Figure 4-9). At this location, the Columbia Formation is highest and the bottom of the waste is lowest because of the prolonged presence of a relative high load of waste (up to an elevation of 90 ft). The maximum thickness of the alluvium is approximately 40 to 45 ft under the center of Phases III-V. Therefore, at this location, the maximum combined thickness of the dredge and alluvium is about 90 ft. Near the Delaware River, it is believed that the top of the alluvium drops at the historic margin of the river (e.g., Figures 4-14 and 4-15). The levels of the bottom of the river were obtained from historic and recent bathymetric surveys developed for the USACE (Duffield Associates, personal communication, 2002).

The alluvium is composed of soft-to-very soft clayey silt and silty clay, with trace to little amounts of fine, micaceous sand. Typical fines content is 95 percent and higher. Plasticity is mostly high (i.e., an average of 40), with a few instances of medium plasticity (i.e., $10 \leq PI \leq 25$). Although classified as inorganic, the alluvium

contains organics, which occur less frequently than in the dredge (Figures 4-16 through 4-18).

In some areas away from the Delaware River, the alluvium contains a few lenses of sand (e.g., Figure 4-2). Near the Delaware River, the alluvium contains alternating layers of fine-grained and granular materials. The lenses of granular materials comprise fine to coarse, silty, medium dense to very dense sand and some gravel (Figures 4-2 through 4-15). These layers appear to be the remnants of the historic, buried dike constructed to reclaim land from the Delaware River. While sections of the buried dike appear to be in the dredge deposit, a sizeable portion of the dike appears to have settled into the alluvium deposit (Figures 4-14 and 4-15). The thickness of these layers of granular materials ranges from 1 to 12 ft based on CPT soundings. Based on the examination of adjacent CPT results and boreholes, individual lenses of granular materials are typically less than 250 ft and up to 500 ft long, in the direction transversal to the perimeter berm, possibly reflecting the deformed width of the buried dike (Figures 4-14 and 4-15). In the direction longitudinal to the dike, the length of these lenses is commonly less than 400 ft an up to about 700 ft long. Overall, these lenses of granular material appear to be discontinuous. Along the existing perimeter dike of CIL, lenses of granular material are vertically spaced at 5 to 10 ft and are up to 10 to 15 ft thick (e.g., Figure 4-4). Closer to the Delaware River, the lenses occur more frequently and are up to 20 ft thick in some locations (Figure 4-5).

The uncorrected SPT blowcounts in the alluvium are also low, ranging from 0 to 15 (when adjacent sandy layers are not present). CPT cone tip resistance values in this area are on the order of 10 tsf and 20 tsf. Due to its recent deposition, the alluvium deposit is normally consolidated. No layers of harder material (i.e., possibly due to desiccation) are encountered in CIL. Therefore, this deposit is quite compressible. A characterization of compressibility parameters is presented in Chapter 6.

4.3.3 Columbia Formation

Underlying the alluvium is the Columbia Formation, which is a deposit of typically brown to gray, medium to coarse, dense to medium dense sand and also contains silty sand with some gravel in some locations. The typical fines content in this layer is 5 to 10 percent. In some locations, the sand deposit contains interbedded lenses of sandy silt to clayey silt.

The location of the Columbia Formation is important for the design of the landfill expansion because this deposit has high permeability and is a natural candidate to drain the proposed PVDs under the perimeter dike of the CIL. A contour map showing the contact between the recent alluvium and the Columbia Formation is shown on Figure 4-19. The location of the top of the Columbia Formation is discussed in the next paragraph. The following descriptions are based on the stationing system along the perimeter dike that starts at the north corner of Phase I (Figure 4-1), runs from 00+00 to 145+00 in a clockwise fashion, and ends at the start point.

The top of the Columbia Formation is highest under the west of Phase I at approximate elevation –12 ft (approximately Sta. 0+00, Figure 4-2). From this elevation, the top of the Columbia Formation dips gently from the northwest to the southeast, with an average slope of approximately 0.2 to 0.5 percent. At approximate Sta. 38+00 along the northern margin of Phase III (Figure 4-3), the top of the Columbia Formation drops almost 20 ft within a short distance. From this location, the top of the Columbia Formation continues to drop and reaches an approximate elevation of –50 ft under the Delaware River. Along the Delaware River, the top pf the Columbia Formation is more or less at a constant elevation of –50 ft. Continuing the stationing system along the Delaware and Christina rivers in a clockwise fashion, the top of the Columbia Formation rises to the northwest (Figure 4-5), rises abruptly approximately 20 ft (Sta. 92+00, Figure 4-6), and continues rising gently to the northwest to approximate elevation –12 ft (Figures 4-7 and 4-8). The line connecting the 20-ft-drops at Stations 38+00 and 92+00 trends north-south and roughly coincides with the 1920's low water line and the historic boundary of the inland and tidal marsh. This line seems to mark the extent of the Delaware River historic floodplain.

The thickness of the Columbia Formation varies across the site, with the average being approximately 40 to 50 ft. Considering the Columbia Formation as being only the upper, continuous deposit of sand immediately underlying the alluvium (as is commonly interpreted by local geologists), the thickness of the Columbia Formation increases from approximately 10 ft (north of Phase I) to approximately 40 ft near the Delaware River. In locations near the center of CIL, the thickness of the Columbia Formation can be 50 to 60 ft (e.g., Figure 4-4). However, it has been hypothesized that in some areas just outside the property boundaries of the CIL the Columbia Formation is absent (DSWA, personal communication, 2002). Maps dated 1841 indicate that a creek existed adjacent to boring P-107 (southwest of the Phase I, Figure 3-9). The stratigraphy of this boring (see Appendix G for historic boring logs) shows that the

Columbia Formation is absent at this location, possibly due to erosion by the creek shown in historic maps.

In most locations underneath the Columbia Formation, there exist sand deposits that are difficult to distinguish from the Columbia Formation proper because relatively thin (a few inches to a few feet) layers of sandy silt separate these deposits (Figures 4-4 through 4-8). In some cases, the layers of sandy silt are small and discontinuous, while in other they are more extensive. These interbedded layers of sandy silt are thicker and appear more frequently in the southwest corner of CIL (Figure 4-6). The Columbia Formation and the underlying sand deposits appear to be hydraulically connected as evidenced by piezometric data.

The Columbia Formation is dense with uncorrected SPT blowcounts ranging from 25 to over 100, with typical values above 30. CPT cone tip resistance values are commonly greater than 100 tsf. When silty layers are encountered within the Columbia Formation, the blowcounts can be as low as 16. In these layers, the CPT cone tip resistance values can be as low as 40 tsf.

4.3.4 Potomac Formation

The Potomac Formation is locally a deposit of brownish red (often described as varicolored), stiff to hard clayey silt to silty clay, with little to trace of fine sand. The contact of the Columbia and Potomac formations varies in elevation from approximately -70 ft (west of Phase I, Figure 4-2) to approximately –120 ft, under the eastern dike. The thickness of the upper layer of fine-grained soils in the Potomac Formation ranges from approximately 15 to 50 ft. The overall thickness of the Potomac Formation, however, is larger. This trend is more accentuated toward the Delaware River. Figure 4-20 shows the elevation contours of the top of the clay layer in the Potomac Formation.

Layers of coarse sand and fine gravel are commonly encountered as part of the Potomac Formation. As discussed in Chapter 2, it has been hypothesized that the erosion of deep streams and subsequent filling with younger granular sediments created these layers of sand and gravel. Locally, the top of these deposits of granular material in the Potomac Formation occur below an approximate elevation –90 (under Phase I) and dip to an approximate elevations of –150 ft near the Delaware River.

As discussed in Chapter 2, the Potomac Formation was formed in the Cretaceous and was subjected to large overburden, possibly as high as 10 to 20 tons per square foot (tsf) (e.g., Derrenbacher, 1998). As a consequence, the current consistency of the Potomac Formation is very stiff to hard, the material is overconsolidated, and the shear strength is relatively high. The uncorrected SPT blowcounts are high, ranging from 50 to approximately 90. The number of CPT cone tip resistance in the Potomac Formation is more limited. Typical values of the CPT cone tip resistance range between 25 tsf and 90 tsf, with an average of 45 tsf.

4.3.5 Bedrock

As discussed in Chapter 2, the top portion of bedrock consists of heavily weathered rock (saprolite) consisting of a weakly consolidated mixture of clay, silt, sand, and rock fragments and characterized by a dark green color, and crystalline quartzitic rock of the Wilmington Formation. Historic borings indicate that the weathered rock portion is approximately 10 to 15-ft thick. The highest point of the weathered rock occurs at approximate elevation –80 under Phase I (Figure 4-2). However, the location of the top of weathered bedrock cannot be exhaustively defined from historic borings because of inaccuracies in the rock descriptions contained in the boring logs (e.g., borings P-104A and P-113, contained in Appendix G). High values of pH measured in groundwater samples obtained from borings P-113 and P-104A are high and indicate close proximity to the Wilmington complex. The top of the crystalline rock is encountered at approximate elevation –95 under Phase I (Figure 4-2) and as deep as elevation –200 along the Delaware River. Figure 4-21 shows the elevation contours of the top of bedrock.

The water conditions in the waste and the dredge are described in the next chapter.

5. CHARACTERIZATION OF WATER LEVELS IN DREDGE AND OTHER LAYERS

5.1 Overview

The existing effective pressures in a fine-grained soil (e.g., dredge and alluvium) affect numerous aspects of the soil's response including strength and compressibility. As pore pressures affect effective stresses, the knowledge of pore pressures as they vary while the soil is being loaded is necessary. This chapter presents information and characterizes the water levels in the dredge, waste, and underlying layers under the landfill as affected by waste placement. In addition, this chapter presents a characterization of liquid levels in the waste. The characterization of water levels also relates the dredge pore pressures and groundwater with the water levels in the adjacent rivers.

The focus in this chapter is the pore pressure in the dredge and alluvium, which refers to the excess porewater induced in the dredge and alluvium due to landfill construction (i.e., consolidation water). In contrast, groundwater refers to water in the soil formations prior to landfill construction.

The water level characterization in the dredge and alluvium relies heavily on water level data collected between July 1983 to August 2002 from numerous instruments including piezometers and monitoring wells. The characterization of liquid levels in the waste is based on liquid level observations made in the waste in gas monitoring wells and waste bucket auger borings during the 2002-2003 site investigation. River levels are obtained from recent surveys.

The resulting water level characterization has been developed taking into account the existing hydrogeological information developed in previous studies. The next section presents a review of existing hydrogeological information.

5.2 Review of Previous Hydrogeologic Studies

The focus of previous studies was to characterize the groundwater, i.e., the natural groundwater before any waste was placed. Previous studies presented the predominant groundwater flow directions (i.e., mainly in the Columbia and Potomac formations), potentiometric surfaces, horizontal and vertical hydraulic conductivities, discussions on possible interconnections between water-bearing zones, and recharge and discharge

areas for underlying water-bearing zones. Water quality and tidal influence were also evaluated in previous studies. In those studies, no significant efforts were made toward assessing the water levels in the dredge and alluvium due to waste filling.

The review of previous hydrogeologic studies indicates that:

Aquifers and predominant groundwater direction

- Locally, an important aquifer is the Columbia Formation (Sundstrom and Pickett, 1971; Sundstrom et al., 1975; Terraqua, 1984; Woodruff, 1984; Gannett Fleming, 1986, 1990; and Weston, 1992).
- Regionally, the Columbia aquifer is unconfined; however, it acts as a confined aquifer at the CIL site due to the low permeability of the overlying alluvium and the underlying Potomac Formation. The pre-landfill hydraulic conductivity of the dredge and alluvium are low, approximately between $1x10^{-6}$ to $5x10^{-7}$ cm/sec (Gilbert, 1982; DSWA, 1982). With increased consolidation of these layers due to the weight of the placed waste, the hydraulic conductivity of these layers has continued and will continue to decrease.
- The recharge to the site appears to occur from off-site outcrops and subcrop areas to the northwest (Woodruff, 1985), and the predominant flow direction at the site in the Columbia Formation is approximate in a northwest-southeast direction (Weston, 1992).
- This direction is consistent with the structural strike and dip of the Potomac Formation (Terraqua, 1984).
- The Columbia formation serves to recharge aquifers in the Potomac Formation.
- Downward vertical gradients are measured in the Columbia and Potomac Formations (Terraqua, 1984; Gannett Fleming, 1986, Weston, 1992). A few piezometric records show upward gradients for relative short periods of time (on the order of months).
- Another major aquifer occurs in the layers of granular material in the Potomac Formation (Woodruff, 1984). The Potomac Formation performs as a confined aquifer in areas where it contains sand lenses that are enclosed in clays (Terraqua, 1984).

Connection Between Colombia and Potomac Formations and Rivers

- Importantly, the Colombia Formation and the sand layers in the Potomac Formation are hydraulically connected to the Delaware and Christina rivers.
- Hydraulic connection between the rivers and these deposits has been evidenced by tidal influence, as measured along the eastern dike in previous studies (Terraqua, 1984; Gannett Fleming, 1986, 1990a and 1990b; Weston, 1992).
- Results indicate that groundwater levels fluctuate approximately 1.25 ft in response to an approximate 4.5-ft tidal fluctuation in this layer. This response attenuates with increasing distance from the Delaware and Christina rivers.
- Groundwater levels in the Potomac formation fluctuate approximately 1 ft to the same tidal fluctuation.

Connection between layers of granular materials in alluvium and rivers

- Layers of granular material in the alluvium and dredge (above the Columbia Formation) are only partially connected with the Delaware River (Weston, 1992).
- There is no hydraulic connection between these layers of granular soil in the dredge and alluvium and the Columbia Formation itself (Weston, 1992).
- There is not a strong hydraulic connection between the granular layers in the alluvium and dredge (above the Columbia Formation) and the Christina River.
- These interpretations are consistent with observations made during the 2002-2003 field investigation that suggest the following trends:
    1. The upper layers of sand in the dike do not seem to be fully connected to the Delaware River (Figures 4-14 and 4-15);
    2. The layers of granular materials in the lower dredge and alluvium are more frequent and thicker near the Delaware River, suggesting an increased likelihood for connection at this location (e.g., Figures 4-3 through 4-6 and Figures 4-14 and 4-15);
    3. The layers of granular material in the dredge and alluvium alternate with layers of fine-grained soil, which effectively decreases the hydraulic conductivity of the former;
    4. CPT and boring logs in close proximity to each other indicate that the sand layers are continuous for only a few hundred feet;

5. Eight (8) pump tests and three slug tests performed during previous investigations did not assist in defining the extent of these sand lenses;
6. The interbedded layers of granular materials in the dredge and alluvium along the Christina River were observed to be less frequent and thinner than comparable layers along the Delaware River.

5.3 Description of Available Data

5.3.1 General

The sources of information to conduct this water level characterization include piezometric data (the vast majority of the available data), direct water level observations in the waste, and river water level surveys. The description of the available piezometric data is included in the next section. The description of the data of liquid levels in the waste is included in Section 5.5. Duffield Associates (personal communication, 2002) provided river water levels from USACE data.

5.3.2 Description of Piezometric Data

In previous studies conventional piezometers, standing pipes, vibrating-wire piezometers and monitoring wells were installed and have been or are presently in operation at the CIL site. Piezometer records at the site date back from 1983 and continue to date. These instruments are screened in the dredge and alluvium deposits and also in the Columbia and Potomac formations. Only one piezometer was installed in the waste to provide information on slope stability performance near the northwest corner of Phase I. Overall, as numerous instruments were installed across the site, a significant amount of information is available.

A large number of piezometer records were reviewed in this study to characterize the water levels in the dredge and alluvium. The piezometric data had been incorporated into the site GIS model (Pennoni and Schnabel, 2001). However, some relevant data related to the piezometers (e.g., screen elevation, screen length, date of installation) are not available for all piezometers in the GIS model. Also, piezometer records, particularly early records, are missing or incomplete as some instruments were abandoned or destroyed as the landfill was developed. Piezometer records obtained in the last few years appear to be more complete. A summary of the instrument

construction records for the wells reviewed for this study is included in Table 5-1. Figure 5-1 shows the location of the piezometers included in the GIS model.

As the placement of waste increases pore pressures in the dredge and alluvium, it is important to relate pore pressure records with the waste placement rate and schedule at the same location. Settlement records were obtained from settlement plates, which were installed during previous investigations in pairs: one on top of dredge and one on top of waste. The difference between readings in the lower and upper settlement plates gives the cumulative waste compression. The settlement on top of the dredge is obtained directly from the readings in the lower settlement plate. Table 5-2 summarizes the initial elevation and installation date of the settlement plate on top of dredge, and the final elevations as of 2001 on top of dredge and waste. Previously in Chapter 3, Figure 3-7 indicated the settlement plate locations.

The piezometer and monitoring-well data were grouped by phases, instrument clusters, and by soil deposit where instruments were screened. Instruments are grouped along a line of instruments within each phase to facilitate the comparison. For example, in Figure 5-2, Lines 1 and 2 are two instruments clusters in Phase I. For each line, the records of various piezometers are shown in separate figures (e.g., Figures 5-3 and 5-4 correspond to Lines 1 and 2, respectively, in Phase I). Table 5-3 provides a summary of the organization of piezometric data by phase and instrument cluster.

Each of Figures 5-2 through 5-26 contains five panels that present information as follows:

- *Panel I (top left corner).* This panel shows a schematic soil depth profile in the vicinity of the instrument cluster showing the location of the screened depth or interval depth (when available) of each piezometer. When screen elevations were not available from the GIS, this information was obtained from borehole logs or from monitoring reports. The panel also shows instrument ID, depth, and installation date, if available. In some cases, although the piezometer schematic location is shown in Panel I, the piezometric data is missing. Conversely, in some instances the piezometer data is available, but information of the screen location is missing and thereby not shown. Out of approximately 200 piezometer records available in the GIS database, only approximately half contain the complete required amount of information to be plotted.

- *Panel II (top right corner).* This panel shows the time-record of elevations of top and bottom of waste as obtained from settlement plates. The records of the plates placed on top of waste give an indication of the rate at which the waste was placed.

- *Panels III through V (bottom left to right).* These panels show the time-history of piezometer data in different strata. The focus is on the dredge and alluvium, as piezometric data from these deposits are more commonly available. The data from the dredge was classified as upper dredge (above an approximate elevation of 10 ft) and lower dredge (below elevation 10 ft). To relate the piezometer data in dredge with that in the underlying formations, piezometer data from the Columbia and Potomac formations is also shown. To facilitate comparisons, the elevation scale is the same in each of these panels and whenever possible, the same time scale was used.

5.4 Data Analysis

5.4.1 River Levels

River surveys indicate that the high and mean water levels along the Delaware and Christina rivers are approximately at elevations 3.86 and 3.46 ft, respectively (Duffield Associates, personal communication, 2002). It can be inferred that due to the low relief of the Coastal Plain, the levels of surface water in the other nearby waterways (i.e., Brandywine Creek and Shellpot Creek) and in the groundwater are controlled by water levels in the Delaware and Christina rivers. Hydrologic studies predict that the 100-year flood event is at a water surface elevation of 10 ft (Duffield Associates, personal communication, 2002).

5.4.2 Potomac and Columbia Formations

The water levels in the Potomac and Columbia Formation are characterized from data obtained in monitoring wells installed along the perimeter of the entire Cherry Island site (including portions of the USACE site). The piezometers in the Potomac Formation include twelve (12) piezometers: P-100A, P-101/P-101A, P-102/P-102A, P-103, P-104, P-104A, P-105, P-106, P-107, P-108, P-113, and P-116. The piezometers in the Columbia Formation include nineteen (19) piezometers: UC-100, LC-100 (Formerly P-100), C-101/C-101A, UC-102/UC-102A, LC-102, C-103, C-104, C-104A,

C-105, UC-106, C-107, UC-108, C-109, C-110, C-111, C-113, C-114, C-115, and C-116. While most of these piezometers are still in operation, some of them were abandoned and/or replaced. Only those piezometers under the footprint or in close proximity of the CIL were used in the evaluation.

Piezometric records in the Potomac and Columbia formation (e.g., Figures 5-3, 5-4, or 5-6) show that the water levels in these formations were approximately between elevations 2 and 5 ft. During some period of time, pronounced lower potentiometric elevations were recorded in piezometers located near the northern edge of Phase I (e.g., see 1989-1992 records in Figures 5-3 and 5-4) along the I-95 corridor, which reportedly was subjected to periodical drainage (DSWA, personal communication, 2003). Records also show that the potentiometric elevations of the water in the Columbia Formation are the same or slightly higher than those in the Potomac Formation at the same location (e.g., C-100 and P-100 in Figure 5-3).

The existing records show that there is a small elevation differential, up to approximately 2 feet, between the water potentiometric levels measured in Phase I (e.g., Figures 5-3 and 5-4) and those measured near the Delaware River (e.g., Figure 5-18). This observation is consistent with the predominant flow direction and gradient magnitude discussed in previous reports (e.g., Terraqua, 1984). The Environmental Assessment Report (GeoSyntec, 2003a) presents potentiometric maps of the Columbia and Potomac formations.

In some locations near Phase I, the overall potentiometric levels in these formations barely raised due to waste placement. While in other locations (e.g., Phases III and IV), the potentiometric elevation increase was moderate after waste placement. For example, in Phase I (Figures 5-3 and 5-4), the potentiometric elevations hardly increased during the filling period 1985-1987 (of Phase I) and 1989-1992 (filling of Phase IA). In Phase V and IV (Figures 5-13 and 5-18, respectively), the potentiometric elevations increased on the order of one to two feet during the filling period 1997-present. Although these observations were made along the perimeter of the CIL, they suggest that most of the pore pressures induced by waste placement are dissipated once the consolidation water from the alluvium enters the Columbia Formation.

Dredge/Alluvium

The following observations can be made regarding piezometric records in dredge and alluvium in: (i) the interior of the landfill; and (ii) along the perimeter berm.

Interior of the Landfill

- In general, piezometers located in the interior of the landfill and screened in the dredge show systematically higher pore pressures than piezometers under the dike. This can be attributed to the consolidation from recent waste placement
- A significant raise in the piezometric elevations can be observed under the waste in Phases III-V with respect to levels recorded along the dike in nearby piezometers. These high gradients at the toe of the landfill are indicative of a "mounding" effect in the dredge under the landfill.
- The higher elevation of this "mound" of excess pore pressures in the dredge under the interior of the landfill makes water flow down to lower layers and eventually to the sides toward lower elevations.
- The higher topography in the middle of the landfill favor the creation of the "mound" of water in the dredge described above. Near the edges of the landfill, the lower topography causes lower water levels. This effect of the landfill topography is less pronounced in the deeper dredge layers.
- In Phases I and II, piezometers did not show a noticeable response to waste placement because all the piezometers available in these phases are along the perimeter berm, away from the waste surcharge; and (ii) waste placement in these phases was negligible over the observed period.
- In phases undergoing waste placement, a 15-ft to 20-ft thick layer of waste placed over a period of approximately 2 years resulted in an increment of pore pressures of 15 to 40 ft. This is an indication of limited dissipation in this time period.
- Although records of interior piezometers show some correlation with waste placement, the large oscillations in the record do not allow establishing a clear relationship between the magnitude of waste placement and measured excess pore pressures in the dredge.
- These erratic piezometric data over some periods of time suggest the possibility of instrument malfunction, damage, incorrect reading, etc.

Perimeter Berm

- Piezometer readings under the perimeter berm appear to be only slightly influenced by waste placement, as expected.
- In Phase I and II, piezometers in the dredge indicate relatively stable liquid levels under the berm.
- In Phase III and IV, piezometers along the perimeter berm did not show a noticeable response to waste placement.
- In Phase V, most piezometers along the perimeter berm did not show a noticeable response to waste placement, except for piezometers P5-PZ-07B, P5-PZ-08A, P5-PZ-10A, and P5-PZ-10B.
- If no additional load is placed near the dike (e.g., in Phase I and II), the piezometers in this location indicate relatively stable liquid levels, with a slight tendency to decrease over the years.
- The tendency to decrease in piezometric levels is insignificant in Phase I, and more pronounced in Phases II-V. Annual head variations in the piezometers of Phases II-V are on the order of 3 to 5 ft.
- No significant differences of porewater pressures levels inside and outside the PVD area (i.e., under berm along Phases IV and V and outside) are observed upon berm rise. It is believed that this is a limitation of the installed piezometers rather than inefficiency of the PVD installed in this area. It has been thoroughly documented that settlements (and pore pressure dissipation) occurred within a short period of time, on the order of a few weeks.

Information from piezometers in the dredge has been used to develop potentiometric surfaces in the dredge for the period of July through December 2002 (Figures 5-27 to 5-29). Although these potentiometric surfaces are limited because they are only "instantaneous" illustrations of a dynamic and varying process, they provide some insight into the pore pressure distribution in the dredge within the landfill and along the perimeter berm.

The piezometric information in the dredge was divided into three screen elevation intervals to facilitate the interpretation as follows:

- Screen elevation between 20 and 32 ft: Figure 5-27;
- Screen elevation between 0 and 20 ft: Figure 5-28; and
- Screen elevation between -25 and 0 ft: Figure 5-29.

These figures indicate that relatively high piezometric levels in the upper layer of dredge (i.e., screen elevation between 20 and 32 ft) occur around the center of the landfill; increasingly lower water levels occur toward the toe (figure 5-27). The water level differential between the center of Phases III-V and the perimeter is larger for the upper dredge layer (Figure 5-27) and decreases for the lower layers (Figures 5-28 and 5-29). It is noted that the lower layer (Figure 5-29), the zone of higher water elevations is not concentrated at the center of the landfill as with the upper layers. It is believed that a part of the excess pore pressures in the upper dredge is dissipated and drained through the LCS, particularly near the edges of the landfill. The head differences between the upper and lower dredge layers and the alluvium deposit may indicate that water is flowing from the dredge into the alluvium, thereby causing a net head loss between these sections.

5.5 Levels of Liquid in the Waste

5.5.1 General

This section presents a summary of the characterization of the liquid level in the waste. The data presented in this section is summarized from the Waste Characterization Report, which is presented in Appendix O.

The knowledge of the liquid level in the waste is necessary in relation to stability analyses when the waste is included. For example, above the liquid surface, the waste mass exhibits lower total units and the entire frictional shear strength is available as pore pressures are negligible. Conversely, the waste mass below the liquid surface exhibits higher total units and only the effective frictional shear strength is available. Therefore, as higher liquid levels in the waste tend to less stable conditions, it is important to correctly consider liquid levels in the analysis. Another important aspect in the characterization of the level of liquids in the waste that is covered in this section is ascertaining potential perched layers. Additionally, this section provides an evaluation of the potential implications of the level of liquids in the waste in relation to the leachate collection system.

5.5.2 Description of Data

The data to estimate the level of liquids in the waste was obtained from direct water level observations (the majority of the available information), CPT dissipation tests, and one set of historic piezometric measurements in the waste.

The 2002-2003 field investigation program included various observations and tests in relation to the determination of the level of liquids in the waste. The tests and observations (shown in Figure 5-30) included the following:

(i) Nine (9) large diameter bucket auger borings (identified as S1W-S5W, S5AW, S6W-S7W, and S7AW in Figure 3-9, and as designated UW01-UW05, UW05A, UW06-S7W, and S7AW;

(ii) Nine (9) standard borings (identified as S1W-S5W, S5AW, S6W-S7W, and S7AW S1W-S5W, S5AW, S6W-S7W, and S7AW in Figure 5-30);

(iii) Existing 30 gas monitoring wells (locations identified with a prefix EGW in Figure 5-30);

(iv) Existing 21 large-diameter borings converted into gas monitoring wells (locations identified with a prefix OTW in Figure 5-30);

(v) 33 CPT dissipation tests at different elevations in eight (8) CPTs (locations identified with a prefix C0 in Figure 5-30); and

(vi) Review of the only piezometric record screened in waste (i.e., P1_PZ_9).

Liquid elevations in each borehole were measured twice: (i) when first encountered, and (ii) after a few hours (on the order of four hours). The liquid levels observed in the nine bucket auger borings are believed to be representative because they remained open a sufficient length of time and they correlate with readings taken in nearby gas wells, which are assumed to be in equilibrium due to the extended period of time they remained open.

Because there are several gas monitoring wells installed throughout the landfill, of which approximately ten are close to the bottom of the landfill, measurement of the liquid elevation inside these wells was used to provide an indication of the liquid elevation in the waste. In contrast to the extensive piezometric data available from dredge and alluvium, the information from piezometers in the landfill is extremely limited.

Of the nine CPTs conducted in waste, eight locations were chosen to perform dissipation tests to evaluate the nature of the liquids found in the initial bucket auger tests. To determine if the observed liquid were continuous or perched, the following criteria based on the CPT dissipation tests was established. If the equilibrium pore pressures measured with the CPT dissipation tests increased with depth from one test depth to the next ones, this was an indicatión that the liquid was more or less continuous. Conversely, if the CPT-based equilibrium pore pressures changed either from a positive value to nearly zero or are erratic at lower elevations, this would indicate that the standing liquids are perched. On the contrary, pore pressures that increase with depth would indicate that the standing liquid might be the result of a clogged leachate collection system or hydrostatic conditions within perched zone.

It must be recognized, however, that occasionally CPT dissipation tests do not provide precise values of static levels of water/liquids. This limitation is typically caused because the porous stone, which is located behind the cone and allows the measurement of water/liquid pressures, looses saturation when passing through non-saturated materials. Despite this potential limitation, the CPT dissipation tests were used as an indication of the condition of liquids in the waste.

There is only one piezometer screened in waste (i.e., P1_PZ_9), which was installed in 2001 to monitor an incident of large lateral displacements near the northwest boundary of Phase I. The level of standing water provided by this instrument was also reviewed.

5.5.3 Data Analysis

Five cross sections were developed to synthesize the information on liquid levels in the waste. The cross sections are indicated on the plan view (Figure 5-30) and are shown individually in Figures 5-31 through 5-35. The cross section incorporate the levels of liquid obtained from the various sources described above and show the top surface of standing water.

These figures suggest the top surface of the standing liquid varies in elevation considerably across the site. The relatively high levels of liquid observed in Phases I and II (Figures 5-31 and 5-32) are consistent with the fact that there is no operational LCS in these landfill phases. Conversely, the low levels of liquid observed to the east

of Phases III through V (Figures 5-33 through 5-35) are suggestive that the LCS in this area is operating properly and the leachate is been drained at acceptable rates.

Figure 5-36 shows profiles of the measured head of liquid pressure from CPT dissipation tests, which were conducted in all five landfill phases in the waste and the dredge immediately below the LCS. The profiles compare CPT dissipation tests results, the static liquid levels observed in levels in bucket auger borings and gas wells, and the theoretical hydrostatic liquid pressure head distribution between the observed liquid surface and the LCS. Each liquid head plotted along the abscissas in Figure 5-36 are shown at the individual test elevation. The stabilized liquid levels based on dissipation tests are always below the theoretical hydrostatic pressure, as shown in Figure 5-36. Furthermore, the values of the stabilized liquid levels are very small, near zero, or even negative (negative values were plotted as zero). In a few cases near the top of the static liquid level observed in borings, the CPT-based liquid levels tend to increase for a few feet below the liquid surface and then decreases to zero (e.g., CPTs C060 and C061). The observations described above suggest that the liquids in the waste are not continuous but perched.

Figure 5-36 also shows that the pressure head of liquid from CPT dissipation tests measured in the LCS itself (CPT tests C065 and C067), immediately above the LCS (CPT test C060), and immediately below the LCS (CPT test C062) are mostly zero and less than a few feet in one location. This observation also suggests that the head of liquid at the LCS is small overall and that that the LCS is draining properly.

Figure 5-36 also indicates that the pressure head of liquid increases significantly in the dredge. For example, in one point (test C060) located a few feet under the LCS, the static pressure head was measured at 30 ft. Within 10 to 20 ft under the LCS, the static pressure head was measured at 35, 53, and 67 ft. This rapid build-up of dredge pore pressure head below the LCS and further increase with depth is attributed to the ongoing consolidation process in the dredge deposit. While higher pore pressures are expected to occur near the middle of the dredge/alluvium deposit (assuming no intermediate drainage layers), the dissipation of pores pressures at the top and bottom drainage layers (e.g., LCS and Columbia Formation, respectively) takes place in these layers first and propagate to the interior of the compressible deposits.

The data of level of liquids in the waste is summarized in Figure 5-37. The figure shows contours that represent the top surface of saturated waste. Figure 5-34 shows that the elevation of the top surface of saturated waste ranges from 20 to 50 ft in

Phase I, 40 to 50 ft in Phases II and V, 40 to 60 ft in Phases III and IV. These contours also incorporate the only liquid level from a piezometer screened in waste (P1_PZ_9), which indicated a piezometric elevation of about 34 ft in mid 2001. The perched liquid levels in the waste causes the moisture content of the waste to vary considerably with location and depth of within the landfill. Below the waste ground surface, the moisture content tended to be constant or increase gradually with depth; below the surface presented in Figure 5-37, the waste becomes saturated. Appendix O presents a detailed analysis of the variation of the unit weight of the waste between phases and with depth. Chapter 7 presents a summary of the effects of the variable liquid contents on the unit weight in the waste.

6. CHARACTERIZATION OF ENGINEERING SOIL PROPERTIES

6.1 Introduction

The goal of this chapter is to characterize the engineering properties of soils (i.e., dredge, alluvium, Columbia Formation and Potomac Formation) found at the CIL site. The characterization is performed through the evaluation of data from the 2002-2003 field and laboratory testing programs and historic information when applicable.

Specifically, soil index properties such as unit weight, specific gravity, grain size distribution, fines content, natural water content, organics content, Atterberg Limits and Liquidity Index are discussed. Soil engineering properties discussed in this chapter are undrained shear strength, as determined by field VST, CPT and laboratory tests, consolidation parameters, and hydraulic conductivity. In addition, stress history and other geotechnical considerations are discussed.

The values derived from this characterization for soil index and engineering properties are used in the design of the CIL expansion. This Chapter presents the basis for the characterization and recommends values for engineering parameters. Chapter 8 synthesizes the recommended values for these parameters in tables.

6.2 Basic Characterization

Various parameters concerning basic characterization of the soils at CIL have been reviewed and analyzed. The following parameters are presented and discussed:

- Unit Weight
- Specific Gravity
- Grain Size Distribution
- Fines Contents
- Natural Water Content
- Organics Contents
- Atterberg Limits
- Liquidity Index

6.2.1 Unit Weight

For the proposed expanded configuration of CIL, potential critical failure surfaces would go through waste and the soils beneath the landfill including dredge, alluvium, Columbia Formation, and Potomac Formation. The compressible soils (i.e., dredge, alluvium, and possibly the Potomac Formation) encompass a significant depth. Therefore, as the unit weight of these soils affects directly the calculations of stability and settlement, it is important to assess this parameter and obtain mean values (i.e., average values, or μ). In addition, it is necessary to identify and quantify the variability (i.e., standard deviation, σ, or coefficient of variation, COV = σ/μ) of this parameter to allow a reliability analysis of the proposed design. Although the unit weight of the bedrock is not necessary for stability analysis, this parameter is needed to perform site response analysis for the seismic evaluation of the proposed structure.

Results of unit weight are available from various locations and elevations. A relatively large number of unit weight measurements on dredge and alluvium. Fewer data were obtained in the alluvium and the Potomac Formation deposits. The unit weight for the dredge and alluvium was determined from standard laboratory determinations combining determination of in-situ moisture content and dry unit weight according to ASTM D2216 and D2937 (ASTM, 2001). Because the Columbia Formation is granular, the evaluation of the unit weight is based on correlations with the soil penetration resistance, expressed as the corrected (in some cases normalized) blowcount obtained in the SPT, N_{60} (e.g., NAVFAC, 1982; Kulhawy and Mayne, 1990). No direct measurements of the soil unit, either historic or made during this investigation, are available in the Potomac Formation. Therefore, the unit weight of the Potomac Formation is assessed from values estimated in historic reports and existing correlations between soil consistency and unit weights (e.g., NAVFAC, 1982).

Figure 6-1 shows the distribution of the dry unit weights (γ_{dry}) and saturated unit weight (γ_{sat}) of the dredge and alluvium with depth. This figure, which incorporates data obtained during the present field investigation and by others, illustrates the trends and variability of the unit weight in the upper soil deposits. In the dredge, the distributions of the dry and saturated unit weights do not exhibit a clear tendency with depth. Based on 33 measurements, the mean (μ) is 97 pcf, the standard deviation (σ) is 7 pcf, and the coefficient of variation (COV) is 7 percent. The variability of the unit weight among landfill phases was also evaluated. After the unit weight data is separated according to respective landfill phases, no significant variability of the unit weight was observable. For unsaturated conditions above the water level and

considering a lower bound natural moisture content of 50 percent (see section 6.2.6), the unit weight of the dredge is estimated at 85 pcf.

In the alluvium, both dry and total unit weight are essentially uniformly distributed. However, there are fewer data in the alluvium than in the dredge. Based on four measurements, μ = 101 pcf, σ = 3 pcf, and COV = 3% for this material.

The dry unit weight of the Columbia Formation was estimated from normalized blowcounts (see section 6.5 below) to range from 110 to 120 pcf. With an estimated natural water content of 15 percent, the saturated unit weight is approximately 125 pcf. The unit weight of the Potomac Formation is conservatively estimated to be 120 pcf based on consistency descriptions from boring logs and references in the literature (e.g., Derrenbacher, 1998). The weathered bedrock is assumed to occur in a 15-ft thick layer and to have a total unit weigh of 135 pcf. The bedrock is assumed to have a unit weight of 150 pcf. The variability of the unit weights of these materials is estimated from the literature (e.g., Harr, 1987; Kulhawy et al., 1992).

In summary, the recommended values of the average unit weight for design are as follows:

Material	Unit Weight (pcf)		Remarks
	Unsaturated γ_{moist}	Saturated γ_{sat}	
Dredge	85	97	γ_{dry} = 56 pcf
Alluvium	NA	101	γ_{dry} = 64 pcf
Columbia Formation	NA	125	Assumed
Potomac Formation	NA	120	Assumed
Weathered Bedrock	NA	135	Assumed 15- ft thick layer
Bedrock	NA	150	Assumed

Chapter 8 also presents the recommended valued reported above and contains values of calculated and estimated variability for each of these parameters.

6.2.2 Specific Gravity

The specific gravity of soil (G_s) is not needed in most common geotechnical investigations. In this case, the specific gravity of the dredge and alluvium is needed to estimate the initial void ratios in these materials from moisture content values. The intent is to compare systematically the large body of historic consolidation parameters data, which is expressed as a consolidation index (as a function of void ratio), with the consolidation data obtained during this investigation, which is expressed as a modified compression index (as a function of strain). According to the existing GIS database, there are 84 measurements of the specific gravity. There is no significant difference in the specific gravity of the dredge and of the alluvium, with $\mu = 2.61$ for both. In addition, $G_{s\,min} = 2.53$, $G_{s\,max} = 2.74$, and $\sigma = 0.05$. Although some mica (a mineral with relatively low specific gravity) has been observed in the dredge and alluvium, there is no clear indication why the dredge and alluvium exhibit a low G_s as compared to that of other soil components.

6.2.3 Grain Size Distribution

Grain size distributions are plotted for the dredge, alluvium, and the Potomac Formation (Figure 6-2), and the Columbia Formation (Figures and 6-2 and 6-3). Individual grain size distributions were obtained from sieve analysis and hydrometers and are plotted individually in Appendix D. Figure 6-2 shows that the grain size distribution for the dredge and alluvium falls within a relatively narrow range. This is consistent with the understanding that the dredge and alluvium materials are closely related as the dredged materials are alluvial deposits removed from the riverbeds of the Delaware and Christina Rivers. Figure 6-2 also shows that little sand is present in the dredge and alluvium, typically less than 5 percent, if the cases of sandy lenses occurrences are excluded. The encountered sand is predominantly fine. Hydrometer results indicate that the silt and clay fractions (i.e., grain size less than 0.005 mm) are approximately 45 to 50 percent, respectively. Figure 6-2 also illustrates that the Potomac Formation has a higher content of fine sand than the dredge/alluvium.

The grain size distributions for the Columbia Formation were divided in three groups according to the sample elevation (Figure 6-2). The objective of this grouping was to inspect the occurrence of distinct geologic features in each layer by identifying differences in the grain size distributions. The three elevation groups are the upper

layers (between elevation -22 to -45 ft), middle layers (between elevation -45 to -65 ft), and lower layers (between elevation -65 to –125 ft).

The upper group (elevations -22 to -45 ft) exhibits a coarser structure, has up to 20 percent of gravel, comparable percentages of coarse, medium, and fine sand, and approximately only 5 percent of fines. The middle group exhibits variability, having poorly graded to well-graded, gravelly to fine sand. The lower group is predominantly fine sand and exhibits a marked increase of fines content, up to about 30 to 40 percent. In one location near the western edge of Phase V (Boring P5-03-S20, see Figure 3-9), grain size distributions were obtained sampled almost continuously for 25 ft in the sand deposit, from approximate elevations –33 ft (interface of alluvium and sand deposits) down to –58 ft (deposit of clayey silt). Figure 6-3 confirms the trend illustrated in Figure 6-2 that the Columbia Formation is coarser (predominantly medium to coarse sand) in the upper portions than at lower elevations (predominantly fine sand with larger fines contents). The larger coarse fraction in the upper portions suggests that the deposition energy with which the Columbia Formation was formed increased with time. While deeper layers were deposited with a relatively low depositional energy, a higher-energy process deposited larger particles. A consequence of this trend is that the upper portion of the Columbia Formation has a more open matrix and is expected to have a higher hydraulic conductivity than the lower portion, a condition that facilitates the groundwater drainage from the overlying alluvium.

In addition to the granular material in the Columbia Formation described above, layers or lenses of granular material (silty sand) were observed in the dredge and alluvium. These lenses are believed to be remnants of the material used to reclaim part of the Delaware River and to construct the submerged training dike. Grain size distributions for these lenses (Figure 6-4) indicate a mixture of predominantly fine sand and of coarser material. In Boring S07 (drilled at the west corner of Phase I), the granular material encountered at approximate elevations 3 to 4 ft is a well-graded mixture of gravel, sand and fines. Lenses of silt in the Columbia Formation have grain size distributions that vary greatly. While at some locations the features are similar to those of the Potomac Formation, at other locations, the lenses have coarser material. The grain size distributions of the sand lenses in the Potomac Formation have comparable characteristics to those of the lower portions of Columbia Formation, indicating the possibility that some of these lenses may be ancient channels carved into the Potomac Formation.

6.2.4 Fines Contents

A profile showing the variation of fines content (f_c) with depth grouped by landfill phase is presented in Figure 6-5. The fines content in the dredge/alluvium ranges from 90 percent to 99 percent, with an average of 96 percent. The fines content of the Potomac Formation was found to be 75 percent. The sand lenses and layers encountered in the Potomac elevations exhibit an approximate fines content of 30 to 40 percent. The silty sand layers in the dredge/alluvium range in fines content from 10 to 50 percent, with an average of 30 percent. The fines content of the Columbia Formation ranges from approximately 3 to 40 percent, and has an average of 20 percent. Layers of clayey silt in the Columbia Formation vary in fines content from 60 to 99 percent, with an average of 80 percent.

6.2.5 Organics Content

Because the CIL site is underlain by a floodplain and dredge obtained from the riverbeds, the presence of organics soils or relatively high organics in the soil was of consideration. The organics content (OC), as obtained with the loss-on-ignition (LOI) method, and its distribution were investigated extensively at CIL because it is an important parameter that can be an indication of difficult soil conditions. For example, soils with high organic content are generally soft to very soft, highly compressible, and may exhibit very low strength.

Figure 6-6 shows a summary of the data concerning OC in the dredge and the upper portions of the alluvium. The data presented in Figure 6-6 includes results obtained in previous investigations and during the 2002 field study. The OC was measured in various samples obtained predominantly from locations observed in the field to contain organics. Figure 6-6 shows that the OC ranges, in most cases, between 2 percent to 14 percent. Higher OCs (i.e., 10 to 15 percent) were encountered at approximate elevations 5 to –5 ft. Occasionally, very high OCs (i.e., 20 percent and above) were encountered at these elevations. Because this area was historically a marsh where organic matter likely developed and accumulated more profusely, the presence of high OCs at these locations is not surprising. The examination of the distribution of OCs by phase also indicated that a slight increase of OC tends to occur to the east of the site, which, again, is consistent with the historic marsh to the east of the site. Below an approximate elevation of –5 ft, the OC tend to decrease to an average value of 5 percent.

In most places, the dredge and soil under CIL are classified as inorganic, except for very isolated cases, where peaty soil was encountered. A separation between organic and inorganic soil can be established by following the procedures described in the Uniform Soil Classification System (USCS) contained in ASTM method D2487-00 (ASTM, 2000). In this method, the ratio $LL_{dried}/LL_{non\text{-}dried}$ is calculated. LL_{dried} is the liquid limit measured in samples that were dried (oven-or air-dried) prior to plasticity testing, and $LL_{non\text{-}dried}$ is the liquid limit measured in samples whose water content was either reduced or increased (without drying) to the Liquid Limit. Ratios $LL_{dried}/LL_{non\text{-}dried} \leq 0.75$ indicate organic soils. The ratio $LL_{dried}/LL_{non\text{-}dried}$ was determined in four samples of dredge and two samples of alluvium. The calculated ratios varied between 0.70 and 0.85, with smaller ratios tending to occur in the dredge. For the alluvium, the ratio is always greater than 0.75; therefore, the alluvium is classified as inorganic. In the dredge deposit, the ratio approaches the limit between organic and inorganic soils. Therefore, the dredge is considered to be borderline between organic and inorganic.

6.2.6 Natural Water Content

A summary of the variation of natural water (w_n) content with depth is shown in Figure 6-7. Profiles of natural moisture water content separated by landfill phase are presented in Figure 6-8. Based on a population of 32 samples, the average natural water content in the upper portion of the dredge (i.e., elevation +20 to +40 ft) was determined to be 75 percent with a maximum value of approximately 100 percent; the standard deviation and coefficient of variation are σ = 15 percent%, and COV = 20%. The natural water content decreases to approximately 70 percent at the bottom of the dredge deposit (i.e., at approximate elevations 0 to –20 ft) (Figure 6-7). A decrease of the water content in the dredge is apparent; this trend is typical in normally consolidated soils.

Based on seven tests, the natural water content of the alluvium generally ranges from 45-68 percent, with an average of approximately 58 percent, σ = 8%, and COV = 13%. The average natural water content in the Potomac Formation is approximately 22 percent, based on 9 tests. Below elevation -70 ft the natural water content of this formation is low, approximately 19 percent, which reflects the state of overconsolidation of this deposit. As shown in Figure 6-8, the natural water content does not vary significantly from phase to phase in the CIL.

6.2.7 Atterberg Limits

Prior to obtaining the Atterberg Limits (AL) testing [i.e., Liquid Limit (LL) and Plastic Limit (PL) through testing and the Plasticity Index (PI) as PI = LL - PL], a series of preliminary tests were conducted to assess the effect of sample preparation on the ALs. The motivation of this assessment was GeoSyntec's previous experience that the determination of the ALs in soils containing considerable organics can be significantly affected by the sample preparation. Specifically, the objective was to assess the effect on the ALs of either drying (oven-or air-dried) the sample versus reducing or increasing to the target limits (without drying) the water content of the sample prior to plasticity testing. These two methods of sample preparation are referred to the "dry sample" and the "wet sample" methods (ASTM, 2000). The "dry method" is far more common than the "wet sample" method because is simpler and faster. As many engineering properties that are relevant to this project, including strength and compressibility, can be directly related to the ALs, it is important to obtain a precise assessment of the soil plasticity. However, samples that are pre-dried tend to produce lower ALs, because the material is effectively changed as the drying process profoundly affects the organics. Lower ALs correlate with stronger and less compressible soils. Because extensive areas were logged as "with organics" in previous studies and during the 2002 field investigation (e.g., boring logs included in Appendix B and schematically shown in Figures 4-16 through 4-18), this assessment was warranted.

A comparison of the effects between the air-dried sample preparation and the "wet method" confirmed that ALs, in particular the PI, are greatly affected if the soil sample is dried. Results shown in Figure 6-9 indicate that the increase is about 30 percent, 6 percent, and 70 percent for the LL, PL, and PI, respectively, when the wet sample method is used instead of the dry sample method. As mentioned above, this departure is attributed to the great alteration that drying has on the organic matter. Based on this comparison, the remaining of the ALs were obtained using the wet sample method. Results are summarized in the plasticity chart shown in Figure 6-10, which indicates that the average LL in the dredge, alluvium, and Potomac Formation is $\mu = 98$, 79, and 36, respectively. The average PI in the dredge, alluvium, and Potomac Formation is $\mu \doteq 54$, 41, and 17, respectively. Figure 6-8 shows that the profiles of PL and LL of the dredge, alluvium, and Potomac Formation are quite consistent from one landfill phase to the other.

The review of historic reports suggests that the historic plasticity data in CIL appears to have been obtained using the "dry sample," and most likely the oven-dried

method, which is expected to produce a more intense degradation. The significant difference between the historic and 2002 plasticity data (shown in Figure 6-11) gives a strong indication that the "dry method" was presumably used to develop the historic plasticity. Based on this comparison, historic plasticity data were not used in the characterization of the soils at the CIL site presented herewith.

6.2.8 Liquidity Index

Profiles of liquidity index (LI) are presented in Figure 6-12. The plot shows that in the upper portion of the dredge (i.e., above elevation 20 ft-msl), LI varies from approximately 0.4 to 1.1 and has an average of approximately 0.75. Below elevation 20 ft-msl, LI decreases with elevation. Based on these higher values of LI in the upper layers of the dredge, it is expected that this material will tend to exhibit a more pronounced reduction in peak shear strength to remolded values. This aspect is discussed in section 6.3. In the alluvium, LI is more uniform and has an average of 0.5. The Potomac Formation has an average LI of 0.13. In some locations of the Potomac Formation, negative LIs were obtained, which is another indication of the overconsolidation in this deposit.

6.3 Shear Strength

6.3.1 Undrained Shear Strength of Fine Grained Soils

6.3.1.1 Overview

One of the most critical aspects in the analysis and design of the expansion of CIL is the undrained shear strength (S_u) of fine grained-soils. The undrained shear strength was measured and/or estimated by field and laboratory procedures using various techniques during the 2002-2003 investigation. Field methods included the field Vane Shear Tests (VST) and Cone Penetration Tests (CPT). Although a significant amount of Standard Penetration Test (SPT) data was obtained from the site during this investigation, SPT-based data (i.e., penetration resistance or blowcounts) were not to characterize the undrained shear strength of fine-grained soils because they are not reliable for this purpose. Laboratory methods to measure the undrained shear strength included isotropically consolidated undrained triaxial compression tests (CIUTC) and direct simple shear tests (DSS). A limited number of direct shear tests (DS) were performed to assess the effect of strength anisotropy. In addition, consolidation tests

results and plasticity results were used to estimate S_u values indirectly through correlations and verify the laboratory and field measured values. The evaluation of S_u also considers historic field and laboratory data, when applicable.

6.3.1.2 Field Methods

Field Vane Shear Tests (VST)

Vane Shear Tests were performed in previous field investigations (Figure 3-4) and during this investigation (Figure 3-9). In addition, VST result obtained by the USACE in the site (i.e., the CIL site and the existing USACE dredge disposal area) were reviewed and used in this characterization. The logs of 2000 VST results are presented in Appendix C. Historic VST results from the CIL are presented in Appendix I. USACE VST logs are included in Appendix R.

Figure 6-13 presents profiles of corrected peak shear strength obtained from the three sources mentioned above. The VST values were corrected to account for shearing rate using factors [developed by Bjerrum (1972 and 1973) and updated by others (Ladd et al., 1977; Larsson, 1980)] that depend on the plasticity index (PI). For the 2002 data, the correction was based on actual PI at the VST location or PI of the closest sample. Because of the limitation of the historic AL data described in Section 6.2.7, the historic VST results could not be corrected using the historic measured ALs. Instead, the average values obtained during this investigation for the dredge (PI = 54) and alluvium (PI = 41) were applied for these materials.

Figure 6-13 shows that expectedly, S_u increases with depth, as is common in normally consolidated soils. The average S_u at the upper portion of the dredge (under the existing perimeter berm) is approximately 380 psf. At the upper portion of the dredge under the waste, the average S_u is lower, on the order of 250 psf. The current underconsolidation that is taking place under the waste or excessive soil disturbance at the VST location due to upgradient consolidation flow may be reasons that cause this lower strength in the dredge under the waste. Below an elevation of approximately 20 ft, the rate of increase of S_u is approximately 11 psf per foot of depth, as indicated in Figure 6-13. This rate of S_u increase appears to be consistent among most tests and locations.

The normalized undrained shear strength (i.e., S_u/σ'_v, where σ'_v is the effective overburden stress at the tip of the VST) based on historic (from CIL and USACE area) and 2002 VST results are plotted in Figure 6-14. Figure 6-14 shows the S_u/σ'_v values for VSTs performed along the existing perimeter berm (on the left of the Figure), and on the right the S_u/σ'_v values for VSTs carried under the waste (on the right of the Figure). Note that VSTs in boring SEA-100 and SEA-104 were conducted in locations inside the landfill in October 2001. The overburden effective stress under the existing berm was calculated based on the soil depth at the time of testing, the estimated values of the unit weight (described earlier in this chapter), and the dredge water levels (described in Chapter 5) at the time of testing. The effective overburden stress under waste should be considered best estimates; however, it is noted that the uncertainty in these values is not insignificant because of the simultaneous loading and the consolidation-induced water flow from the upper portion of the dredge into the leachate collection system. Normalized shear undrained strength under the waste are low probably because the effective stresses are difficult to estimate due to the non-equilibrium transient conditions.

Table 6-1 presents a summary of the pertinent information to calculate normalized peak undrained shear strengths based on the 2000 VST results. Tables 6-2 and 6-3 present similar information for the historic CIL data and the USACE data, respectively. Note that some VST results were eliminated from the data population because after a careful inspection of the boring and laboratory logs, it was determined that the soil at the VST location was too sandy, was too close to a sand lens, or there was evidence of excessive disturbance to the fine-grained soil. A summary of averages and standard deviations for all VST results is presented in Table 6-4.

Results in Table 6-4 indicate that the weighted average of S_u/σ'_v is $\mu = 0.31$. The variability of S_u/σ'_v values is significant, with and weighted average COV approximately equal to 34%. Considering the slope of the S_u distribution of 11 psf/ft described above (Figure 6-13), considering an average saturated unit weight of the dredge $\gamma_{sat} = 97$ pcf (see Figure 6-1), and assuming the groundwater table only a few feet from the ground surface along the existing berm (see discussions in Chapter 5), a range of the normalized undrained shear strength was obtained between 0.29 to 0.32. This range is similar to that obtained from the analysis included in Tables 6-1 through 6-3.

Figure 6-15 shows the distribution of normalized peak undrained shear strength by locations: (i) outside landfill away from berm; (ii) along the berm with no existing Prefabricated Vertical Drain (PVDs); (iii) along the berm with existing PVDs (i.e., along perimeter of Phases IV and V); and (iv) under the waste. The distribution of S_u/σ'_v is consistent and similar in all areas, except under the waste. Along the berm (with or without PVDs) and away from the berm, the S_u/σ'_v values fall around the average value $S_u/\sigma'_v = 0.31$ presented above. Under the waste, this ratio is significantly smaller. These distributions suggest that outside the waste area, S_u/σ'_v values are comparable and do not seem to be affected significantly by recent landfill loads (outside the PVD area) or by the PVDs. In addition, slightly higher VST results appear to be measured in 2002 than those measured at the same location in the 1998 and 2001 period by the SEA series of borings. However, it is believed that slight strength increase is more related by spatial variability than by strength gain with time.

In lieu of the Bjerrum corrections mentioned above, other methods to correct the field VST readings were utilized as independent verifications. The first method was that developed by Aas et al. (1986). In this method, the correction factor is a decreasing function as the effective overburden (normalized with the atmospheric pressure) increases. Applying this methodology to the 2002 VST records along the berm, it was found that S_u/σ'_v values differ approximately ±30% from the values obtained using the Bjerrum correction. Some of this variability can be explained by the uncertainty associated with the vertical effective stress, particularly because the water levels could not be accurately obtained for each VST location and thus the dredge pore pressure may not be representative. Overall, the mean value of S_u/σ'_v obtained with the Aas et al. (1986) method is, on average, 10% larger than the mean obtained with the Bjerrum factors. The other methods are those in which the field S_u/σ'_v ratio is corrected according to the Atterberg limits. For example, with a correlation that associates S_u/σ'_v to the Plastic Limit (Jamiolkowski et al., 1985), the average S_u/σ'_v ratio is approximately 0.30 for a PI = 54 in the dredge. Another correlation that associates S_u/σ'_v and the Liquid Limit (Larsson, 1980), provides higher values for this ratio. For, the Liquid Limit of the dredge and alluvium at 98 and 79, the average S_u/σ'_v is 0.32 and 0.40, respectively. Although these two alternative methods appear to provide comparable normalized strengths, the Bjerrum factors were used for design, because they seem to provide more conservative estimates of the undrained shear strength than the other approaches.

The VST also provide a measurement of the residual undrained shear strength. This information is valuable because it gives an estimate of the amount that shear strength may decrease due to extremely large deformations (i.e., after the soil is totally remolded). Calculated ratios of residual-to-peak undrained shear strengths ($S_{u\ res}/S_u$) are summarized in Tables 6-1 through 6-4. Figure 6-16 shows the distribution of the residual undrained shear strength, $S_{u\ res}$ and the ratio $S_{u\ res}/S_u$. The ratio $S_{u\ res}/S_u$ varies between 0.22 and 0.35, with an average value of 0.3. The inverse of the $S_{u\ res}/S_u$ ratio can be interpreted as the sensitivity (S_t). Therefore, the sensitivity as obtained from VST results ranges from about to 2.9 and 4.5. These sensitivity values are slightly greater than those obtained using the correlation proposed by Mitchell (1993) that relates sensitivity, Liquidity Index, and the existing overburden pressure (normalized with the atmospheric pressure).

Cone Penetration Tests (CPT)

The VSTs supply valuable information as direct measurements of the undrained shear strength (S_u) in the dredge and alluvium are obtained. However, as shown by Figures 6-13 through 6-16, the results exhibit great variability. Also, as these results do not provide a continuous profile of undrained shear strength, potential changes of strength with depth may not be captured by the VST procedure. Consequently, the variation of S_u from one landfill area to another is only approximate and with not insignificant uncertainty with the VST. To avoid these limitations, the spatial variability of the undrained shear strength of the dredge is investigated using cone penetration test (CPT) results and published correlations. The undrained shear strength of fine-grained soils can be estimated from CPT parameters using two methodologies as follows.

In Method 1, the following empirical correlation is used:

$$S_u = \frac{q_t - \sigma_{vo}}{N_k}$$

where q_t is the corrected cone tip resistance, σ_{vo}= is the total overburden pressure, and N_k, is a factor referred to as the cone factor. The cone factor is empirically obtained by relating the measured cone tip resistance with the undrained shear strengths measured by other means, such as field methods (e.g., VST or DMT) or laboratory procedures (e.g., CIUTC or DSS tests). In this investigation, the calibration is carried out using the VST-based and DSS-based S_u/σ'_v values. For the DSS values (see section 6.3.3), only results from locations along the perimeter berm (i.e., where there is a good degree of

certainty of the dredge piezometric surface position) are selected. The calibration (see Table 6-5) shows that for the existing conditions at the time of testing, the calculated N_k ranges from 12.4 to 18.7 (VST) and 12.7 to 19.9 (DSS). The average N_k from both methods is 15.1. For higher conditions of the dredge water, the N_k increases. Therefore, the assumption of $N_k = 15$ is well supported, on average, by the VST and DSS results. These results are comparable to values reported in the literature that for normally consolidated, soft to very soft soils exhibiting the reported range of plasticity, the cone factor falls between approximately 12 and 15 (Lunne and Klever, 1982; Aas et al., 1986; Lacasse 1988).

Method 2 estimates S_u using the dynamic excess pore pressure (Δu) that is measured during cone penetration. The Δu value develops over the initial, static pore pressure (u_o) of the soil. The correlation used in Method 2 is:

$$S_u = \frac{\Delta u}{N_u}$$

where N_u is a factor that can be related to soil rigidity, I_r and the Skempton's pore pressure parameter at failure, A_f. Soil rigidity is defined as $I_r = G_o/S_u$, where G_o is the elastic shear modulus. With the soil rigidity varying between 50 and 100 (as computed from stress-strain curves obtained during ICUTC tests) and assuming $A_f = 1$ (which is a typical value for normally consolidated soils), N_u results between 5 and 6. Figure 6-17 compares the undrained shear strength obtained from Methods 1 and 2 using $N_k = 15$ (Method 1) and $N_u = 6$ (Method 2) with selected CPT tests. The two methods can predict similar values in most CPT locations and at the depth intervals where the soft soils were encountered.

Based on the VST and DDS calibration (Method 1), the similitude of results from Methods 1 and 2, and the good comparison with published values, $N_k = 15$ is selected in the remaining of the shear strength characterization with the CPT. One limitation is that in very soft layers, the N_k approach predicts unrealistically low shear strengths, in some cases even less than zero. This might be caused by an underestimation of the initial pore pressure u_0.

Various profiles of undrained shear strength with elevation based on CPT soundings were developed using the 2002 data as well as the historic data, when applicable. Figures 6-18 through 6-24 present S_u profiles from CPTs along the dike. Each of these figures present CPT results grouped by stretches of perimeter berm up to 2,500 ft long, which are identified by their stationing. Figures 6-25 through 6-28 show

the S_u profiles from CPTs under the landfill footprint. Figure 6-25 presents the S_u profiles under Phases I and II, and Figure 6-26 through 6-29 present the S_u profiles under each of the remaining landfill phases.

The following general observations can be made from the CPT-based S_u profiles:

Under Berm (Figures 6-18 through 6-24)

- the S_u profiles are consistent within a given berm stretch (up to 2,500 ft long), which suggests that the spatial variation is not significant for similar load conditions;
- in any of the berm segments, the CPT-based S_u profiles are quite consistent; this observation suggests that the variability of CPT profiles is less that than that from the VSTs;
- the rate of S_u increase with depth is comparable and consistent in the dredge and alluvium;
- a crust of relatively hard material (possibly due to desiccation and compaction) occurs from the ground surface of the existing perimeter berm down to approximate an elevation of 30 ft;
- below this layer, the shear strength drops and exhibits the lowest values (up to about 400 psf) between approximate elevations 30 and 15 ft;
- below this zone, the shear strength increases almost linearly with depth (consistent with normally consolidated deposits) at a rate that ranges between 10 to 12 psf/ft in a few locations up to more than 20 psf/ft in other locations;
- outside the dike, close to the Delaware and Christina Rivers, shear strengths [e.g., CPT 29 and CPT 31 (Figure 6-21)] are lower than those measured under the dike because the dredge at these locations has not been loaded beyond its original deposition overburden pressure as opposed to the dredge under the berm (e.g., compare the left and right charts of Figure 6-21);
- shear strength along the eastern boundary of Phase IV and the eastern and south limits of Phase V are significantly higher than shear strengths in other areas. Although the 2002 CPTs were carried out between 10 to 15 ft outside the existing PVD zone along Phases IV and V, it appears that the PVDs, with average depths between approximately 80 to 95 ft, have had a significant beneficial effect on the strength;

- along the northwest boundary of Phase I between Stations 132+50 and 150+00 (Figure 6-24), the undrained shear strength decreases below the average observed in other location along the berm;

Waste (Figures 6-25 through 6-28)

- the shear strength of the dredge is consistently lower (up to about 250 psf) between elevations 30 and 15 ft than for other areas;
- despite being subjected to lower loads, the S_u values in the dredge and alluvium under the berm are consistently higher than those measured under the landfill;
- Under Phases I and II, the dredge appears to be stronger than in the remaining areas (e.g., see SEA-1 and CPT-51 in Figure 6-25) of the landfill;
- the S_u profiles under the waste are in most cases curved (particularly under Phases III through V), which is an evidence of underconsolidation;
- CPT results can be reasonably well reproduced by various CPT sounding carried out at the same location (e.g., SEA-5 and CPT-68 in Figure 6-27, SEA-7 and CPT-56 in Figure 6-28, etc.);
- When CPTs were performed at the same location, the S_u profiles obtained during 2002 have in most cases higher strengths than the six CPTs of the SEA-100s series (performed in October 2001) and even higher than the 1998 SEA-10s series. This aspect will be discussed in more detail in section 6.3.2; and
- the CPT-based methodology appears to predict unrealistically low values of undrained shear strength for values under approximately 200 psf.

The CPT also allowed the characterization of the undrained shear strength of deeper fine-grained soils, including layers of clayey silt (which occur within the Columbia Formation) and the Potomac Formation. With a cone factor N_k = 15, S_u profiles for these materials are presented in Figure 6-29. In Figure 6-29, S_u for the clayey silt ranges between 2,500 and 5,000 psf. An average of 4,000 psf can be considered. The S_u distribution for Potomac Formation, which in these CPTs starts at approximate elevation –90 ft, has a minimum value of about 2,300 psf and then increases with depth at an approximate rate of 100 psf/foot below its contact with the overlying sand layers. This high rate of increase although consistent with the high degree of overconsolidation of the Potomac Formation, was considered to be too high

for analysis. Instead, this rate, which is related to the S_u/σ'_v ratio, was estimated from correlations, as described below.

The ratio S_u/σ'_v for the Potomac Formation was estimated from correlations with Atterberg limits. For an average PI = 17 for this deposit, correlations (e.g., Kulhawy and Mayne, 1990) give $S_u/\sigma'_v \approx 0.19$ to 0.22 OCR, where OCR is the overconsolidation ratio. Considering conservatively the lower bound and with an OCR of 2, S_u/σ'_v is approximately equal to 0.38. Additionally, based on this estimate and considering a saturated unit weight of 120 pcf, the rate of increase with depth is approximately calculated as 20 psf/ft. Additional information on the undrained shear strength of the Potomac Formation is included in section 6.3.3.1.

6.3.1.3 Laboratory Soil Testing

Shear strength was evaluated using laboratory tests including isotropically consolidated undrained triaxial compression Tests (CIUTC), simple shear tests (DSS), and direct shear (DS) tests. Results of laboratory tests are included in Appendix D. A discussion of results is presented in the next three sections.

Triaxial Tests

Figure 6-30 shows the normalized undrained shear strength from 2002 and historic CIUTC tests on the dredge and alluvium as a function of the consolidation pressure. The normalized undrained shear strength is expressed as S_u/σ'_c, where S_u is the undrained shear strength, defined as $S_u = \sigma_{dev\,f}/2$; $\sigma_{dev\,f}$ is the deviatoric stress at failure, defined as $\sigma_{dev\,f} = (\sigma'_1 - \sigma'_3)_f$; σ'_1 and σ'_3 are the major and minor principal effective stresses at failure, and σ'_c is the consolidation pressure.

The normalized shear strength for low levels of consolidation is high; however, these values should be ignored because the soils in this range of σ'_c is most likely overconsolidated due to the removal of the overburden during sampling. For high levels of consolidation, where the effects of overconsolidation are eliminated, the normalized strength S_u/σ'_c is on average approximately equal to 0.42 for the dredge and alluvium.

Only three triaxial tests were carried out on samples from the Potomac Formation during this investigation. For the high levels of consolidation, expected for this formation, the normalized shear strength S_u/σ'_c is equal to 0.58.

In addition to the above results, some information was made available to GeoSyntec Consultants from a nearby area related to the shear strength of the Potomac Formation. Duffield Associates recovered an undisturbed sample of Potomac Formation during a field investigation in the Port of Wilmington, which is located across the Christina River from the site (Duffield Associates, personal communication, 2003). The sample was obtained with a Shelby sampler from an approximate elevation –116 (i.e., 126 ft below the ground surface) and subsequently subjected to an unconfined compression (UC) test. The UC strength was recorded at 8.4 kips per square foot (ksf). The undrained shear strength derived from these tests is commonly lower than those measured in the CIUC triaxial test. Chen and Kulwhawy (1993) estimated that the UC-based S_u is on average 0.88 of the CIUC-based S_u for moderately overconsolidated (i.e., $3 < OCR \leq 10$) soils. Considering that the Potomac Formation falls in this category, the CIUC-based S_U is estimated to be 4,600 psf.

Direct Simple Shear Tests

A total of 22 direct simples shear (DSS) tests where conducted on undisturbed dredge samples retrieved in Shelby tubes. All DSS samples were anisotropically consolidated (approximately at at-rest conditions) at selected consolidation pressures in a 2.62-in. diameter Geonor-type apparatus before being sheared at a constant shear deformation rate of approximately 0.05 percent per hour. Figure 6-31 presents a summary of DSS results performed at an overconsolidated ratio (OCR) of 1. Figure 6-31 shows that the dredge material mobilizes large strains (between 15 and 25 percent) to reach peak values indicating that the material is highly ductile. Figure 6-32 presents the peak undrained shear strength normalized with consolidation pressure. The figure illustrates there is a trend of the normalized shear strength to decrease with increasing consolidation pressure. While the relatively high values of the normalized shear strengths at a low level of consolidation (i.e., 2,500 psf) may be the result of some overconsolidation effect due to sampling, Figure 6-32 shows that the normalized strength continues to decreases, although at a lower rate, at higher consolidation pressures. Considering that, on average, the critical surfaces for stability analysis are subjected to normal effective stresses ranging from approximately 3,5000 to 10,000 psf, it is reasonable to adopt a mean value S_u/σ'_v of 0.29 from DSS tests.

These results are consistent with typical values obtained from DSS, which tends to give lower estimates of shear strength than other techniques such as VST (which tends to give intermediate values when corrections are adequately applied) and triaxial compression tests (which tend to give the highest estimates).

Direct Shear Tests (Anisotropy Effects)

It is well known that the direct shear test (DST) is not a suitable test to measure the undrained shear strength of fine-grained soils because drainage in these tests is unrestrained and the failure plane is controlled by apparatus geometry rather than by limiting stress. However, during this investigation, the DST was used to assess the potential for strength anisotropy in the dredge that may have arisen from the sedimentation process in which the dredge was placed. It is reasonable to assume that shear strength along in situ horizontal planes would be lower than those along other planes because the sedimentation process may favor preferential horizontal weakness surfaces.

A series of DSTs were conducted on pairs of samples obtained with Shelby tubes from the same location. One of the samples was trimmed vertically (conventional practice) and sheared along the in-situ horizontal plane. The other sample was trimmed horizontally (i.e., axis of DST sample was 90 degrees from axis of Shelby tube) and sheared along the in-situ vertical plane. Despite the testing limitations described above, these limitations should affect to both horizontal and vertical tests, and anisotropic effects, if significant, should to be observed. The strength was measured in the direct shear box apparatus in eight samples (i.e., four pairs) at three horizontal displacement rates: 0.002, 0.02, and 0.2 inches per minute (in./min).

Figure 6-33 shows results of the DSTs at various displacement rates for pairs of samples. It is apparent from figure 6-33 that the shear strength along each direction is quite similar. In addition, the effect of the displacement rate on shear strength appears to be negligible in these tests. This is most likely due to the lack of drainage control during shearing in this test.

Figure 6-34 compares the normalized shear strength with respect to the applied normal stress. As the average of the ratio of $S_{horizontal}/S_{vertical}$ ($S_{horizontal}$ and $S_{vertical}$ are the shear strengths along in situ horizontal and vertical planes, respectively) is close to one, it is suggestive that the anisotropy effects on shear strength are not significant in the dredge material.

6.3.1.4 Summary of Undrained Shear Strength of Fine-Grained Soils

The undrained shear strength for the dredge and alluvium (i.e., the controlling layers for the stability at the CIL facility) was estimated using several methods. The strength to be used for design should be related to the strength that is expected mobilize in the field. As no single field or laboratory technique covers all the stress states that are expected in the field, it has been recommended that an average strength should be used for design (e.g., Mesri, 1989). This average should take into account the predominant state of stress along the entire potential failure surfaces considered for stability. As such, the average should incorporate strength values from (i) triaxial compression tests (under the passive wedge at the toe of the slope); (ii) DSS tests (at the base of the sliding mass); and (iii) triaxial extension tests (under the active wedge at the top of the slope). By following this procedure, and estimating the undrained shear strength in triaxial extension as a fraction of the triaxial compression results, an average of $S_u/\sigma'_v = 0.31$ is calculated. Although no anisotropy effect was observed in DSTs, a more conservative value is considered and a value of $S_u/\sigma'_v = 0.29$ is selected for design. No plane-strain considerations are given to the undrained shear strength of fine-grained soils.

6.3.2 Increase of Undrained Shear Strength of Fine Grained Soils with Time

The shear strength gain with time in fine-grained soils that are loaded is controlled by the soils' ability to drain. While the proposed CIL configuration calls for the use of PVDs to accelerate the consolidation of the dredge and alluvium deposits upon loading and thereby speed the shear strength of these soils, the dredge/alluvium under the waste will not have any added drainage capacity. Therefore, these areas will exhibit a much slower shear strength increase with time. The strength time rate increase was evaluated in Appendix VIb-D.1 of the Engineering Report (GeoSyntec, 2003b) titled "Evaluation of Partially Consolidated Undrained Parameters." In that calculation package, consolidation parameters, field records (settlements), and numerical simulations were used to assess partially consolidated undrained shear strength parameters at various load stages for the dredge under the waste.

An independent estimate of the strength gain is carried out in this section using CPT records by analyzing the cone tip resistance, q_t, at the same location at different times. Although the time span considered in this approach is not sufficiently long to analyze this effect exhaustively, some trends are observable at the seven locations at

analyzed (see Table 6-6). Most of the locations were under the existing waste footprint. The date of the baseline CPT tests is December 1998 and comparisons with CPT data from October 2001 and February 2002 were made.

Figures 6-35 through 6-40 and Table 6-6 indicate that the cone tip resistance gain with time is modest along the dike. Therefore, as no significant increase in total overburden has taken place in this area, the undrained shear strength with time is none to minor. In two of the cases along the berm the cone tip resistance was essentially the same for all times, while in the remaining two cases the increase was modest, less than about 20 % over approximately three years. Under the waste, the increase was modest to significant due to the loading process and consolidation. The higher increases were observed in the areas with more active landfilling. Overall, the strength gain with time appears to be more significant in the upper layers of the dredge near the waste. These observations are comparable to those made previously (SEA, 2001b). In addition, these rates of increase were confirmed and calculated with more accuracy in "Evaluation of Partially Consolidated Undrained Parameters" (Appendix VIb-D.1).

6.3.3 Drained Shear Strength of Fine Grained Soils

A summary of historic and 2002 CIUTC is shown in Figure 6-41. The data consists of the p'_f and q_f points for all available data as well Mohr circles for most of the data. The value p'_f is the effective mean pressure at failure and is defined as $p'_f = (\sigma'_1 + \sigma'_3)_f/2$, where σ'_{1f} and σ'_{3f} are the major and minor principal effective stresses at failure. The value q_f is the shear stress at failure and is defined as $q_f = (\sigma_1 - \sigma_3)_f/2$. Figure 6-41 shows the lower bound, mean, and upper bound Mohr-Coulomb type envelopes, and presents a shear strength envelope developed for analysis.

Figure 6-41 shows that the effective friction angle (ϕ') of the mean envelope is approximately 34 degrees. The effective friction angles of the upper and lower bound envelopes differ up to two degrees from that of the mean envelope. The cohesion intercept varies from 620 to 850 psf. The shear strength envelope selected for analysis is more conservative than the lower bound Mohr-Coulomb envelope for stresses lower than 2000 psf. As shown in Figure 6-41, the shear strength envelope was selected to have no cohesion intercept, higher effective friction angle at low and intermediate normal stress, and an effective friction angle of 34 degrees at high normal stresses.

Figure 6-42 presents the triaxial tests results in terms of effective strength parameters for the three CIUTC tests conducted on the Potomac Formation material. The effective friction angle (ϕ') of the envelope is approximately 17 degrees, while the cohesion intercept is approximately 3,000 psf. Existing correlations based on the Plasticity Index (Mitchell, 1993) predicts an effective friction angle (based on triaxial compression data) of approximately 33 degrees for a fine-grained soil with PI = 17. If plane-strain conditions are further assumed (as is the prevalent case in the slope stability analysis at CIL), the effective friction angle is expected to be approximately 10 percent higher (Wroth, 1984; Kulhawy and Mayne, 1990); therefore $\phi' \approx 36$ degrees.

However, a more conservative approach is adopted for the Potomac Formation based on the following aspects. If a deep failure surface is considered passing through the dredge/alluvium and the Potomac Formation, the kinematic compatibility along the failure surface (i.e., the same slip movement causing the same shear deformation astride the failure surface, equivalent to progressive failure) will mobilize the strengths of these materials differently. For small shear deformations, the strength of the Potomac Formation may be fully mobilized because of the less ductile nature of this overconsolidated material, while the dredge has not achieved its peak yet because it is much more ductile (e.g., as evidenced by the results from CIUTC and DSS tests). For large shear deformations, the strength of the Potomac Formation is not longer at its peak but at a reduced, remolded value, while the dredge achieve its peak strength. To consider this situation, the effective strength of the Potomac Formation must be selected at a remolded value. As no field or laboratory data is available from the CIL, correlations are used to estimate the fully remolded (i.e., residual) effective friction angle (ϕ'_{res}). Correlations by Skempton (1985) and Lambe (1985) indicate that for the conditions of the Potomac Formation [clay fraction (CF, i.e., percentage of clay fraction with grain size less than 2 μm) and high overburden], the residual friction is estimated to be 21 degrees. The cohesion intercept is assumed to be zero.

6.3.4 Shear Strength of Coarse-Grained Soils

The friction angle of coarse-grained soils is commonly estimated from correlations with results from field tests because they are more reliable than laboratory procedures, which tend to provide unreliable results due to sample disturbance. In this study, the shear strength of the Columbia Formations is established based on field measurements including the STP and the CPT. The following two sections discuss these aspects.

6.3.4.1 Shear Strength based on SPT

The shear strength of cohesionless soils is generally represented by a drained, effective angle of internal friction, ϕ', which varies as a function of SPT N-values. To take into account the increase of SPT N-values with increasing overburden, normalized values of the SPT N-values must be considered. Therefore, the SPT records in the sands from boring logs (Appendix B) were reviewed and the normalized effective overburden considered (i.e., σ'_v/p_a, σ'_v is the effective overburden and p_a is the atmospheric pressure). The SPT-values range from about 20 to over 100 blows per foot (bpf) and the estimated normalized overburden varies between 0.5 to about 1.5. Using a chart developed by Schmertmann (1975), an effective friction angle (based on triaxial compression tests) approximately equal to 37 to 40 degrees is obtained (Figure 6-43). Considering that plane-strain will prevail, it is reasonable to adopt a higher degrees friction angle for plane-strain conditions. Based on results compiled by Lee (1970), the effective friction angle is expected to increase between 4 to 8 degrees from triaxial to plane-strain conditions. Based on these considerations, the effective angle of internal friction for the Columbia Formation is conservatively selected at $\phi' = 40$ degrees.

6.3.4.2 Shear Strength based on CPT

The shear strength of cohesionless soils can be also estimated from the cone tip resistance, q_t. As with the SPT, the increase of q_t with overburden must be taken into account, and normalized values of the cone tip resistance must be considered. The CPT q_t values in the sands from CPT logs (Appendix A) were reviewed and normalized with the atmospheric pressure and the effective overburden considered (i.e., σ'_v/p_a). The normalized q_t values range from about 100 to over 250; the estimated normalized overburden varies between 0.5 to about 1.5, as before. Using a chart developed by Schmertmann (1975), an effective friction angle (based on triaxial compression tests) approximately equal to 36 to 42 degrees is obtained (Figure 6-43). Based on the plane-strain considerations, the effective angle of internal friction for the Columbia Formation is confirmed to be at $\phi' = 40$ degrees.

6.4 Stress History

A total of 28 consolidation tests were performed as part of the 2002 laboratory testing program. Figure 6-44 shows a summary of results of consolidation tests performed from samples retrieved along the berm. The figure shows that for the range of preconsolidation pressure in the dredge falls within a relatively narrow range, between approximately 2 and 5 ksf, values which are consistent with the existing overburden at the sample locations. These tests confirm that the dredge under the waste can be considered normally consolidated. One deep sample (S04-SH2 at the bottom of the alluvium at about 81 ft below ground surface (bgs)) and a shallow one (S10-SH2 at about 41 ft bgs in alluvium) exhibited a higher OCR value, on the order of 2. It is noticed that the measured values of modified compression index (slope on the virgin portion of the consolidation curve) are quite consistent.

No consolidation was performed in the Potomac Formation. Based on existing information (e.g., Derrenbacher, 1998), it is estimated the Potomac Formation has a preconsolidation pressure between 10 and 20 ksf.

6.5 Consolidation Parameters

Consolidation parameters such as the compression index (C_c), the secondary consolidation index (C_α), and vertical and horizontal coefficients of consolidation, (C_v and C_h) are important parameters in engineering design. The following section describes methods used to evaluate these parameters and suggests representative values to be used in design. Methods included are: (i) laboratory data, including triaxial tests, (ii) correlations, and (iii) settlement plate data.

6.5.1 Compression Index

Results from the 2002 testing program, which are complemented with historic data, show that the modified coefficient of compression (C_{ce}), which relates strains with changes in the log of normal stress, ranges from 0.16 to 0.33 and has an average of 0.24 in the dredge and alluvium, a slightly lower value at the bottom of the alluvium (Figure 6-45). These values were transformed into the conventional compression index to compare with similar soil conditions. To do this, the in situ void ratio (e_o) was reviewed from historic data and was estimated using conventional soil phase formulas with

average values derived previously. Figure 6-46 shows that the average in-situ e_o of the dredge and alluvium from historic laboratory data is about 2.3, with a standard deviation (σ) equal to 0.2. Using the relationship $e_o = w_n G_s / S_r$ [where w_n is the mean natural moisture content equal to 75 (see Figure 6-7); G_s is the specific gravity (see section 6.2.2); and S_r is the degree of saturation (assumed to be 100 %)], and average e_o = 1.92 is obtained. This value is statistically comparable the mean value obtained from the historic data. With the average void ratio of 2.3, the mean and standard deviation for compression index can be computed as $\mu = 0.8$, $\sigma = 0.2$, respectively. These results are shown in Figure 6-47. As a verification of the reasonability of the results, the range of compression index was related to the PI of the sample and compared with an existing correlation (Kulhawy and Mayne, 1990). Figure 6-48 shows that on average the compression index from CIL falls within average values provided by the correlation.

6.5.2 Coefficient of Secondary Compression

Consolidation tests also provided data for the modified coefficient of secondary compression ($C_{\alpha e}$). Figure 6-49 shows the obtained values as a function of the natural moisture content. The resulting mean (μ) and standard deviation (σ) are 1.6% and 0.5%, respectively. As a verification of the reasonability of the results, the range of calculated coefficient of secondary compression was obtained and compared to plasticity parameters. The $C_{\alpha e}$ values obtained from consolidation tests were used in conjunction with the average void ratio to determine C_α. With these values, the average C_α/C_c ratio generally ranged from 0.03 to 0.08, with an average of 0.7. This is approximately within the range suggested by Mesri and Godlewski (1977) for organic clay silt.

6.5.3 Rate of Consolidation

6.5.3.1 Introduction

The rate of consolidation is expressed through the vertical and horizontal coefficients of consolidation, C_v and C_h. These parameters were measured or estimated from several sources including consolidation tests, correlations, CPT dissipation tests, and inference from settlement data. A description of the different sources is presented below.

6.5.3.2 Coefficient of Consolidation using Triaxial Tests and Correlations

Figure 6-50 shows results of C_v from triaxial consolidation tests (both historic and 2002 data) along with values estimated from NAVFAC (1971) for the historic range of liquid limit. It can be noticed that the spread is significant.

6.5.3.3 Coefficient of Consolidation using Settlement Data

To estimate the vertical coefficient of consolidation, c_v, at the site, data from the historic settlement plates was used. Settlement data is available from some plates existing since 1992. In some areas, this provides almost ten years of data on settlement of the dredge and alluvium under the waste load. The method used to estimate the vertical coefficient of consolidation is that developed by Asaoka (1978). The method consists of using equally spaced settlement observations and plotting the settlement at a given time, t, against the settlement at the prior time step, t_1. The settlement data at CIL is not equally spaced; therefore, to obtain equally spaced data points, linear interpolations were made between observed settlement points. Once the data has been plotted, the slope of a linear best-fit line through the data (B) is calculated. This value is then used to calculate c_v. The original equation used by Asaoka to calculate c_v is derived from a differential equation that takes into account soil strain in three-dimensions. Given the size of CIL and the locations of the settlement plates, the vertical strain will be much larger than strain in the horizontal direction; therefore, the equation for c_v used is based on the Terzaghi theory (1943), which accounts only for vertical strain as follows:

$$c_v = \frac{-4H^2}{\pi^2}\frac{\ln(B)}{\Delta t}$$

where: H is the thickness of the compressible layer; B is the slope of the best-fit line through the historic data; and Δt is the time between each observation point used to fit the line.

The settlement plots are shown in Figures 6-51 through 6-56 for various settlement plates. A summary of the resulting c_v values is shown in Table 6-7. The plots are shown on a log scale for original time intervals along with the top of waste elevation record. In the lower portion of Figures 6-51 through 6-56, the type of plots developed

by Asaoka is also included. In some instances, the c_v calculation is performed for several time periods.

6.5.3.4 Coefficient of Consolidation from CPT Dissipation Tests

Numerous analyses were performed to estimate c_h values from CPT dissipation tests. It was found that this method was not that very reliable, as it tends to give unreasonably large values. The reason of this trend can be attributed to the fact that cone dissipation tests tend to give relatively large values where the soil becomes sandier. In only a few areas the CPT provided reasonable values that helped establish that the ratio of C_h/C_v is typically 1.5 to 2.5.

6.6 Hydraulic Conductivity

A profile of hydraulic conductivity of fine-grained soils is presented in Figure 6-57. Historic data were based on triaxial tests. There were no hydraulic conductivity tests performed during the 2002 investigation. Therefore, a correlation was made utilizing 2002 laboratory results (i.e., consolidation parameters). Based on this correlation, the average hydraulic conductivity of the dredge and alluvium is 1.5×10^{-7} cm/sec. These results are comparable to those obtained by others using Triaxial Permeability Tests.

7. SUMMARY OF WASTE PROPERTIES

7.1 Introduction

This Chapter summarizes the major findings and provides recommendations of the most relevant engineering properties of the waste that will be used in support of design activities for the expansion project. More in-depth information is included in the Waste Characterization Report (WCR) in Appendix O.

The WCR contains the evaluation and interpretation of data and site information related to the physical properties and characteristics of in-place waste at CIL and presents:

(i) a review of historic site information;
(ii) a review of waste properties used in previous reports for CIL;
(iii) procedures and results of field investigations by GeoSyntec and their sub-consultants in relation to waste;
(iv) procedures and results of waste characterization and laboratory analyses; and
(v) a summary of findings, conclusions, and recommendations.

The main waste properties reported herein are the unit weight, shear strength, and compressibility. Unit weight is based on site–specific results. Shear strength and compressibility are also based on site-specific laboratory data values and are supplemented by values published in the literature.

7.2 Field Investigation

7.2.1 Introduction

GeoSyntec and others conducted the field investigations performed for physical characterization of the waste in March and April of 2002. The field investigations consisted of the following activities:

(i) nine (9) large diameter (bucket auger) borings within the waste mass;
(ii) collection of bulk waste samples for characterization studies and laboratory testing;
(iii) nine (9) SPT borings within the waste mass and underlying dredge material;

(iv) eleven (11) Spectral Analysis of Surface Waves (SASW) surveys at the interior and perimeter of landfill;
(v) nine (9) CPT dissipation tests performed within the waste mass; and
(vi) measurement of standing liquid levels in existing gas wells.

7.2.2 Objectives

The objectives of the field activities consisted of:

- evaluation of the structure and physical properties of the waste mass;
- characterization of the spatial variability of waste materials;
- evaluation of standing liquid elevations within the waste mass;
- collection of representative samples of waste for laboratory testing; and
- identification and completion of data gaps to facilitate design activities and address other concerns

7.2.3 Large Diameter Test Boring Program

The large diameter (LD) boring program consisted of nine large diameter (36 in.) bucker auger borings; which were advanced to depths between 22 and 31 ft, and had the following specific objectives:

- obtain measurements of waste unit weight in all phases of CIL;
- visually characterize the waste mass; and
- obtain representative waste samples for on-site compositional classification and laboratory testing.

The final augered depth was dictated by the apparent free-standing liquid encountered at each location. The high water content and cohesionless nature of the waste below the standing liquid level prevented the effective sample removal from the boreholes using the bucker auger. In addition, a representative mass of wet waste could not be transferred into a container because parts of the liquid in the waste was lost from the bucker during transfer or seeped from the container after the waste was placed into it.

7.2.4 Small Diameter Test Borings

The small diameter (SD) test boring program had the following specific objectives:

- visually characterize and photograph the waste to supplement observations from the LD borings;
- obtain undisturbed samples of waste to measure the bulk unit weight; and
- inspect and obtain undisturbed samples at the interfaces of the waste with the leachate collection system (LCS) and the LCS with the dredge to assess the potential for clogging in the LCS.

The SD test boring program consisted of nine 8-in.-diameter borings, which were advanced to depths of 14 – 60 ft. Once the boring had been advanced below the level of saturated waste (estimated from observation of drill cuttings) and/or was close to the estimated depth of the LCS, an attempt to recover SPT or Shelby samples was made. Recovered split-spoons were opened and then sampled and/or photographed. Recovered Shelby tubes were opened, inspected, and sealed if the retrieved material was enough. On completion of drilling, each borehole was backfilled with drill cuttings.

7.2.5 Other Tests

The primary objective of the SASW investigation was to evaluate the spatial variability of waste properties using shear wave velocity. The objective of the CPT dissipation tests was to establish whether liquid levels encountered during the test boring programs were static (connected) or perched (unconnected). The program of measuring liquid levels in existing gas wells was performed to further the data of standing liquid elevations across CIL.

7.3 Engineering Properties

7.3.1 Waste Classification

The waste material, for the most part, was found to be in poor condition, fairly degraded, and difficult to sort. Most of the waste was black and very wet, making it visually indistinguishable. Waste from greater depths was more difficult to recognize

and classify. Some general characteristics of the waste are as follows:

- inert material (generally wet soil or rock) and fines (mostly inert material) generally comprised more than 50 percent of the waste sample by mass;
- paper, the second largest component category, was generally wet and heavy but relatively non-degraded;
- glass was always in small fragments, probably due to waste compaction during filling operations; and
- organics generally consisted of wood, highly degraded compost material, and other yard waste (e.g., leaves).

In general, waste from Phases I through III was more degraded than waste in Phases IV and V.

The waste composition is generally heterogeneous. No unusual waste materials were observed. Fresher, drier, less degraded waste was generally more fibrous than older, wetter, or more degraded waste. The moisture content of excavated waste was the most significant property affecting waste structure, compaction, degradation, composition, and waste unit weight. Waste close to or below the standing liquid level was highly degraded, although degradation was somewhat dependent on waste composition.

7.3.2 Unit Weight

Six methods were used to estimate waste unit weight:

- Method A. From previous reports or published in technical literature;
- Method B. Calculated from settlement plate and total pressure cell data;
- Method C. Estimated from waste placement records and topographic surveys;
- Method D. Calculated from large diameter test borings;
- Method E. Calculated from SPT and Shelby samplers; and
- Method F. Estimated from SASW survey data.

7.3.2.1 Method A: Literature Review

A review of reported values for municipal solid waste unit weight in published literature indicated values between 60 and 90 pcf.

In previous reports for CIL, the bulk unit weight of waste (i.e., landfilled material, including cover soil layers) was generally assumed to be approximately 70 pcf. Previous field investigations by GeoSyntec at similar landfills found the bulk unit weight of waste to vary from 55 to 130 pcf.

A study by GeoSyntec on long-term values of waste unit weight at another DSWA's site based on analysis of waste receipt records and topographic surveys indicated that the waste unit weight increased from 57 to 70 pcf in the 13-year post-closure period at the site.

7.3.2.2 Method B: Settlement Plate and Total Pressure Cell

In this method, the waste unit weight can be calculated from total pressure cells (TPC) and settlement data. Using this method with the historic TPC data from CIL, the calculated unit weight of waste generally appeared to decrease, often to approximately half of its initial value in a four-year period (mid-1997 and October 2001) for which TPC data was available. Some decrease in the average waste unit weight through the entire depth of waste in Phases IV and V may be due to the use of highly compressible, low-density compost mulch as daily cover in recent years. However, it is unlikely that this effect could cause a decrease in unit weight of the magnitude obtained with the TPC data. As conventional experience suggests that waste unit weight generally increases over time due to waste degradation and settlement therefore, the historic TPC data is suspected to be erroneous.

7.3.2.3 Method C: Waste Placement Records and Topographic Surveys

An evaluation of the bulk unit weight of the waste and cover soil materials was made by analyzing waste receipt records and topographic maps from the site for five time periods, from January 1985 (beginning of site operations) to April 2002. The volume between cumulative surveys was calculated from differences in ground surface elevations, which account for increasing landfill volumes resulting from waste

placement. This volume calculation considered settlements of the dredge underlying the waste.

The unit weight in Phases I and II was found to decrease from 64 pcf (January 1985-December 1997) to 59 pcf (January 1985 to April 2002). During the same periods, the unit weight in Phase III/IV/V was found to increase from 43 pcf to 51 pcf. Apparently, the unit weight in the older, thicker Phases I and II is decreasing while it is increasing in the newer, shallower Phases III through V. This effect may be due to the fact that recent waste deposition in Phases I and II has tended to be mainly of asbestos. Asbestos is arranged in flat, pre-packaged units, which are laid out and immediately covered with low-density, compost-rich soil without being compacted. On the other hand, in Phases III through V waste degradation and settlement are more intense, which tended to densify the waste.

7.3.2.4 Method D: Large Diameter Test Borings

This procedure was adapted from the conventional sand-cone method to measure in-situ unit weight of soils (ASTM D 1556). The procedure consisted of weighing the waste removed from each depth interval of the borehole until the borehole had been advanced to that interval's maximum depth. The volume of each LD boring was determined using three alternative volume assumptions a) gravel backfill volume method (the borehole volume was backfilled with well-calibrated gravel and the volume estimated from the weight of the placed gravel), b) no borehole overbore (straight shaft volume), and c) borehole with 16-percent overbore (16 percent increase of straight shaft volume). Unit weights obtained from the gravel backfill volume method are considered to be too high due to the reduction in borehole volume that actually takes place due to waste radial deformation prior to gravel backfilling. The borehole volume calculated from the gravel backfill volume method may, however, provide a useful check on future calculations of waste compressibility and consolidation. Borehole volumes derived using the "no overbore" and "16 percent overbore" assumptions were considered to yield the upper- and lower-bound waste bulk unit weight estimates at each depth interval.

Averaging over the whole site, the bulk unit weight from the LD borings ranged from 31 – 108 pcf, with and an average bulk unit weight of 61 pcf. A general trend of increasing unit weight with depth was evident in most borings.

7.3.2.5 Method E: SPT and Shelby Samplers

The procedure of calculating waste unit weight from SPT and Shelby samplers involved extraction of a cylindrical sample for volume and mass measurement from the samplers. Care was taken when opening sample tubes to avoid disturbing the samples. The top and bottom of the sample were trimmed to ensure that a cylindrical section was obtained. The calculated waste unit weights were 97 pcf (depth of 53 ft) and 76 pcf (depth of 34 ft). The calculated unit weight values represent saturated values because samples were collected below the apparent depth of saturation.

7.3.2.6 Method F: SASW Survey

The SASW method was used to determine shear wave velocity (V_s) profiles inside and around the landfill. The correlation of V_s or the low-strain shear modulus with waste bulk unit weight measured in the top 20 to 30 ft with Method D, allows estimating the waste unit weight at greater depths and then developing a profile of unit weight with depth. The estimated unit weights ranged from 36 pcf (top 8-ft) to 90 pcf (depth of 25 to 36 ft).

7.3.2.7 Summary of Unit Weight

Each of the methods described above used for determination of waste unit weight have limitations, which are discussed in Appendix O. After the methods were ranked in Appendix O in terms of their accuracy and how representative the obtained values were, it was found that the large diameter borehole method (Method D) provided the most acceptable values.

Appendix O presents summary tables providing data derived from all methods. Cross sections showing the distribution of unit weight using these methods were developed. These cross sections contain the apparent depth of saturated waste. The cross-sections are divided into regions and/or depths with different waste unit weight values.

Recommended values were developed for the waste unit weight for existing waste and additional waste to be placed during future operation to be used in design analyses. These values are based on the data summarized in Table 10 of WCR and the distribution of waste unit weight (Figures 7-1 through 7-5 of WCR). The recommended

values are based on the ranking of methods described previously.

Values for the unit weight of existing waste in the CIL are given in Table 7-1. Saturated unit weight values for existing waste are assumed to apply at elevations below the limits of saturated waste shown in Figure 8 of the WCR.

Based on the unit weight values obtained for existing waste in CIL, estimates for unit weight values for future waste placement in the landfill were derived as shown in Table 7-2.

Figure 7-1 summarized the recommended values of unit weight for the dredge.

7.4 Waste Shear Strength and Consolidation

Five laboratory consolidation and direct shear tests were performed on a waste sample collected from a depth of 27 to 29 ft below ground surface and approximately one foot below the observed standing liquid level (i.e., the sample was fully saturated). The samples were reconstituted and consolidated at confining stresses of 800 and 2,000 psf. A summary of the test conditions is presented in Table 7-3. The average modified compression index of the waste (C_{cE}, which relates settlement with strains and log values of normal effective stress) was estimated to be 0.28 (see Appendix O-6).

Recommended values for the waste drained strength parameters (S_w) is the bi-linear envelope with a slope of ϕ'=33° and a minimum shear strength of 500 psf for dry/moist waste, and a linear envelope with a slope of ϕ'=33° for saturated waste.

7.5 Limits of Saturated Waste

7.5.1 Waste Moisture Content and Elevation of Saturated Waste

Moisture content either remained constant or increased with depth. During the LD test boring program, intermittent pockets of high moisture content waste underlain by relatively dry waste material were not generally observed.

7.5.2 Depth to Saturated Waste

The depth to saturated waste was determined in all borings except two in Phase II, which were abandoned before saturated conditions were encountered. From the results, it is apparent that there is the deepest saturated waste occurs in Phase II. Saturated conditions were not encountered in the borings in the central section of Phase III, while there appears to be little saturated waste in Phase V. The elevation of saturated waste was found to be highest near the Phase II/III boundary.

7.5.3 Cone Penetrometer Tests (CPTs)

Only the results of CPT dissipation tests carried out in the waste were presented and discussed in the WCR, because these tests are used for determining the characteristics of standing liquid in the landfill (i.e., whether liquid encountered in the waste is present in one static, connected layer or present in several unconnected, perched layers). Dissipation tests in the waste and underlying dredge material were performed at CPT soundings C060 through C068. It was intended that soundings be pushed through the entire depth of waste in at least one location in each phase of the CIL, ending 20 – 30 ft. into the underlying dredge material. An attempt to perform a dissipation test within the LCS, in order to assess the free draining nature of the LCS at that location, was to be made at each sounding location. At least one dissipation test in the underlying dredge material at each location was attempted.

At most dissipation test depths, negative pore pressures were measured, indicating that standing liquid was either not present or only present below the test depth. Where positive pore pressures were measured in the waste, conditions indicative of static water levels were not encountered. This suggests that, while a great deal of liquid is present in the waste, it exists in unconnected, perched layers and is not truly static. It is interesting to note that the pore pressures measured in the underlying dredge spoil material were significantly higher than those measured in the waste.

7.5.4 Liquid Level Measurement Program

An extensive program of measurement of liquid levels in existing gas wells at CIL was performed. The wells were selected in all Phases of CIL to provide data on static liquid levels in areas where test borings were not advanced, and to confirm measured liquid levels at test boring locations in which static liquid levels were found to be

excessively high. Three types of gas wells were considered as part of this program:

1. Existing gas wells. These are generally long-standing gas vents built up as the height of the landfill increased from interlocking cylindrical 48-in. diameter concrete sections. These gas wells are predominant in Phases I and II.

2. Gas wells installed during March and April 2002 within large diameter bucket auger borings. These gas wells consist of a slotted 6-in. diameter PVC pipe surrounded by coarse aggregate.

3. Borings from the large diameter test boring program finished as gas wells.

As part of this investigation, the present conditions of the LCS were assessed inspected in borings and soundings. One goal of the exploration was to obtain samples at the interfaces of the waste with the LCS and the LCS with the dredge to asses the clogging potential of the LCS. Through a series of nine 8-in diameter borings two samples of the interface were obtained. Due to continued settlement, the stiffness of the waste, and the presence of obstructions, it was difficult to locate the existing leachate collection system.

Where the liquid level measured in a gas well was less than 2 ft above the base of the well, it was assumed that the liquid was trapped gas condensate and not representative of the elevation of standing liquid or saturated conditions in the surrounding waste.

7.5.5 Summary of Limits of Saturated Waste

Relevant data from each LD and SD test boring, CPT sounding, dipped gas well, as well as other relevant sources of data (e.g., CPT dissipation tests, settlement plates, piezometers in waste, and two SASW surveys outside the limits of waste) were used to interpret the limits of saturated waste on a whole-phase basis. Findings from the CPT dissipation tests, small diameter test borings, and well dipping program indicated that the upper liquid encountered in the LD boreholes does not always extend continuously to the LCS. It is believed that perched liquid levels are commonly present within the landfill). All waste located below the depth at which significant free liquid or saturated conditions were observed is considered to be saturated, but it is assumed that little or no

free standing liquid is present in the landfill. For this reason, the term 'standing liquid level' is not used in the WCR.

Phase I. The limits of saturated waste vary from approximately 30 ft on the north side to 40 to 50 ft in the center and south side of the phase. The elevation of dredge is approximately at datum with surface elevation ranging from 40 to 90 ft.

Phase II. The top of dredge is at elevation 20 to 30 ft, the ground surface ranges from approximately 20 to 90 ft, and saturated waste generally exists below 45 to 50 ft. Dissipation tests indicate that the pressure head along the same vertical line but at different elevations is approximately constant at 2 to 3 ft. This suggests that liquid in the central portion of Phase II is either perched or flowing downwards.

Phase III/IV/V (Western Areas). The elevation of dredge is approximately at 40 ft with the ground surface elevation ranging from 45 to 75 ft. Saturated waste is shown to be present up to elevations of 60 to 65 ft in Phases III and V and up to 55 ft in Phase IV. Dissipation tests suggest that these are either perched layers or that downward flow of liquid is taking place.

Phase III/IV/V (Central Areas). The elevation of dredge is approximately at 40 to 45 ft with the surface elevation ranging from 45 to 80 ft. Saturated waste is shown to be present at a few confined locations up to elevations of 60 ft and 50 ft in Phases III and V, respectively. Dissipation tests suggest that perched liquid or saturated waste layers are present.

Phase III/IV/V (Eastern Areas). The elevation of dredge is approximately at 40 to 45 ft with the surface elevation ranging from 45 to 80 ft. As with Section D-D, saturated waste is shown to be present at a few confined locations up to elevations of 50 ft and 55 ft in Phases III and V, respectively.

The limits of saturated waste are presented in Figure 8 of the WCR. All waste located below the surface elevation shown in Figure 8 can be considered to be saturated. However, the limits of saturated waste in the CIL are expected to fluctuate seasonally and by precipitation, and the contours shown in Figure 8 represent existing conditions at the site in April 2002 only.

7.6 Condition of the LCS at the Waste-LCS-Dredge Interface

Attempts to recover Shelby tubes and split-spoon samples at the waste-LCS-dredge interface proved to be troublesome. Only two samples were successfully extracted from this interface.

1. A split-spoon sample recovered from a depth of 24.5 ft. showed that the LCS consisted of poorly graded soil and gravel.

2. A Shelby tube sample pushed from 42 - 44 ft is maintained sealed for later evaluation. The field inspection of the tube did not show evidence of granular soil at the tip of the tube. It is doubtful that the tube contains any of the LCS.

7.7 Calculation of Other Waste Properties

The specific gravity (G_s), void ratio (e), porosity (n), degree of saturation (S) of waste were calculated from sample obtained from the LD borings. Values based on both maximum and minimum bulk unit weight values are given in Table 8 of the WCR.

8. SUMMARY OF RECOMMENDED VALUES

Table 8-1 presents a summary of recommended values to be used for design.

9. REFERENCES

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TABLE 3-1

HISTORIC GEOTECHNICAL REPORTS AND RELATED DOCUMENTS

No.	Date	Title/Description	AUTHOR/ORIGINATOR
1	1981?	Candidate Sites for NSWMC No. 2	Gilbert & Assoc., Inc.
2	4/1/1982	Compressibility and Permeability Characteristics, Dredge Spoil Materials, Cherry Island Site	Woodward-Clyde
3	1984	Site Suitability Report Hydrogeologic and Geotechnical Evaluation of the CI site Vols I and 2	Duffield/Terraqua
4	1986	Northern Solid Waste Facility-2, Phase II Area LF Design Report	Gannett Fleming
5	1990	Northern Solid Waste Facility-2, Phase III Area LF Design Report Geotechnical and Hydrogeological Report	Gannett Fleming
6	1990	DSWA NSWMC-2 Cherry Island, Phase III Stability Analysis	Gannett Fleming
7	1990a	Northern Solid Waste Facility-2, Phase IA Geotechnical Report	Gannett Fleming
8	6/1/1991	DSWA NSWMC-2 Cherry Island, Phase III Environmental Assessment	Gannett Fleming
9	1992	Northern Solid Waste Management Center CILF Phase IV Disposal Area Hydrogeol. and Geotechnical Report	Weston
10	3/14/1995	Northern Solid Waste Management Center CILF Phase V Disposal Area Hydrogeologic and Geotechnical Report	Weston
11	1995	Field Exploration on Edgemoore Disposal Area	Duffield
12	Jun-95	Waste Lift Evaluation	Gannett Fleming
13	6/12/1995	Unitech Drilling Logs and Drawing	Unitech Drilling Co.
14	Feb-96	Summary Evaluation Phase I Through V, Northern Solid Waste Mgt., Center-2	Duffield
15	Jun-96	Summary Report for Monitoring Well Inst., Abandonment, Redevelopment and Repair at Pigeon Point and CILs	MARCOR
16	11/6/1996	Environmental Risk Assessment Survey	Versar, Inc.
17	1996	Settlement Data During Berm Construction in Phase V	DSWA
18	8/28/1997	Phase V, Subcell V-1 First Lift Landfilling Sequence	DSWA
19	9/9/1997	Letter to establish directives for future Lift 2 MSW LF activities	Weston
20	12/15/1997	Accelerated Filling of Phases IV and V (Weston's position regarding accelerated filling of Phase IV)	Weston
21	2/1/1998	DSWA NSWMC-2 Cherry Island, Phase III Stability Analysis	Gannett Fleming
22	1998	DSWA NSWMC-2 Cherry Island, Phase IV Stability Analysis	Gannett Fleming
23	1998a	Quarterly Geotechnical Monitoring Report for the Northern Solid Waste Management Center at CIL	Gannett Fleming
24	3/24/1998	Technical Evaluation of Settlement Plate/Total Stress Cell Data Phase IV and V Disposal Areas	Weston
25	12/14/1998	Approval SWA-98/14 to continue use of remaining capacity of Phase 1A for asbestos disposal	DNREC
26	1998	Phase III Stability Analysis	Gannett Fleming
27	5/17/1999	Investigations into Failure of Portugal Landfill memorandum	Schnabel
28	6/21/1999	Composite Stability Analysis - revised report for geotechnical evaluation	Schnabel
29	7/18/1999	Review of Geotechnical Engineering Reports for the Cherry Island Landfill	J.M. Duncan
30	8/25/1999	Response to questions from July 18, 1999 report from J.M. Duncan	J.M. Duncan
31	9/28/1999	Composite Stability Analysis - Comments based on review of Dr. Duncan's report	Gannet Fleming
32	10/12/1999	Composite Stability Analysis Letter discussing receipt of Schnabel report	Duffield
33	11/23/1999	Memo CIL Phases IV and V - Prepared in Response to Schnabel Eng Assoc and J.M. Duncan Comments	DSWA
34	6/1/2000	DSWA, CI, Plan of Operation	DSWA
35	1/17/2000	Composite Stability Analysis NSWMC-2	Schnabel
36	4/30/2001	Permit SW-01/01, NSWMC-2	DNREC
37	5/29/2001	Compliance Issues at Cherry Island Landfill and Pigeon Point Landfill	DNREC
38	6/1/2001	DSWA Request for Proposals for Design and Construction Inspection Services at (CIL)	DSWA
39	6/1/2001	Subsurface Investigation, Wilm. Harbour N., Dredged Material Confined Disposal Facility	Duffield
40	6/14/2001	Monitoring Well Installation Report (CIL), NSWMC-2)	SAIC
41	7/01	Quarterly Geotechnical Monitoring Report for NSWMC at CIL	Schnabel
42	10/2001	Quarterly Geotechnical Monitoring Report for N. Solid Waste Mgt. At Cherry Island	Schnabel
43	11/01	Quarterly Geotechnical Monitoring Report for NSWMC at CIL	Schnabel
44	12/10/2001	W.O. 4276.GL, Supplemental Subsurface Exploration, Wilm. Harbor N. Disposal Area	Duffield
45	12/19/2001	In Situ Testing NSWMC CIL	Schnabel
46	12/21/2001	GIS Geotechnical Info. Users Guide (version 1.0)	Pennoni/Schnabel

TABLE 3-2

SUMMARY OF HISTORICAL GEOTECHNICAL/HYDROGEOLOGICAL FIELD TESTS

Test / Data Record	Number of Available Records					
	Phase					Total[1]
	I	II	III	IV	V	
Borings	35	17	26	17	32	135
CPT	3	4	6	5	7	25
CPT Pore Press. Dissipation	1	2	1	1	3	8
Dilatometer	1	0	5	1	1	8
Inclinometers	10	8	12	9	13	52
Monitoring Wells	4	2	2	2	8	18
Piezometers	29	19	41	29	54	172
Pressuremeter	0	3	0	1	0	4
Settlement Plates	7	9	9	6	10	41
Seismic CPT	1	1	1	0	1	4
Slug Tests	0	0	0	1	0	3
Thermistors	0	0	8	8	12	28
Total Pressure Cells	0	0	10	10	12	32
Vane Shear	8	9	5	2	4	28
Well Yield	1	0	4	1	1	8

Note: (1) In some cases, the cumulative number of available tests in the five phases does not coincide with the total number of available tests because some tests could not be assigned to a specific phase. In such cases, the number of tests were tallied in the total.

TABLE 3-3

HISTORIC EXPLORATORY BORINGS [1]

No	Phase	Boring Name	GIS Model Designation	DATE	Easting	Northing	Ground Surface Elevation (ft)	Maximum Depth [2] (ft)	Groundwater Elevation [2] (ft)
1	I	a-1	P1_TB_A1	1983	630,399	470,821	38	40.5	-
2		a-2	P1_TB_A2	1983	630,940	470,750	22	45.5	-
3		a-3a	P1_TB_A3A	1983	630,834	470,167	29.6	16	-
4		a-4a	P1_TB_A4A	1983	630,040	470,820	23.6	19	-
5		a-5	P1_TB_A5	1983	630,051	470,399	21.7	45	-
6		a-6	P1_TB_A6	1983	630,382	469,950	29.6	53	-
7		a-7	P1_TB_A7	1983	629,701	470,892	23.7	50	-
8		gf-15	P1_TB_GF15	1986	630,064	470,977	35	8	-
9		gf-200	P1_TB_GF200	1990	631,095	470,902	12	37	-
10		gf-201	P1_TB_GF201	1990	630,853	471,157	24	42.5	-
11		gf-202	P1_TB_GF202	1990	631,165	470,633	17.9	22	-
12		gf-203	P1_TB_GF203	1990	630,593	471,277	22.6	39.5	-
13		gf-204	P1_TB_GF204	1990	630,712	471,045	48.6	28	-
14		gf-204-a	P1_TB_GF204A	1990	630,715	471,050	48.6	46.5	-
15		gf-205	P1_TB_GF205	1990	630,921	470,640	44.6	7	-
16		gf-205-a	P1_TB_GF205A	1990	630,925	470,645	44.6	46	-
17		gf-206	P1_TB_GF206	1990	630,822	470,233	22	42	-
18		gf-207	P1_TB_GF207	1990	630,277	471,092	26.3	42	-
19		gf-208	P1_TB_GF208	1990	630,462	470,191	49.2	46.5	-
20		gf-208a	P1_TB_GF208A	1990	630,467	470,196	49.2	51	-
21		gf-209	P1_TB_GF209	1990	630,156	470,589	54.7	51	-
22		gf-209-a	P1_TB_GF209A	1990	630,160	470,595	54.7	37	-
23		gf-210	P1_TB_GF210	1990	629,881	471,033	30.3	52	-
24		gf-211	P1_TB_GF211	1990	630,489	469,808	23.2	42	-
25		p-1	P1_TB_P1	1981	630,149	470,933	31.9	194	0.9
26	II	eeb-32	P2_TB_EEB-32	-	629,411	471,033	34	-	-
27		gf-1	P2_TB_GF1	1986	630,588	471,529	37	59	34
28		gf-2	P2_TB_GF2	1986	630,033	471,058	32	61.5	24
29		gf-3	P2_TB_GF3	1986	630,399	471,918	38	9	-
30		gf-3a	P2_TB_GF3A	1986	630,399	471,918	37.3	59	22.3

TABLE 3-3
HISTORIC EXPLORATORY BORINGS [1]
(continued)

No	Phase	Boring Name	GIS Model Designation	DATE	Easting	Northing	Ground Surface Elevation (ft)	Maximum Depth [2] (ft)	Groundwater Elevation [2] (ft)
31		gf-5	P2_TB_GF5	1986	629,376	471,321	39.2	59	33.2
32		gp-6	P2_TB_GP6	1983	629,838	471,714	34.4	75.5	-
33		gp-6a	P2_TB_GP6A	1983	629,827	471,702	37.3	30	-
34		eeb-4	P3_TB_EEB-4	-	629,149	472,505	39.2	-	-
35		gf-101	P3_TB_GF101	1989	629,957	472,739	42	75	-3
36		gf-102	P3_TB_GF102	1989	629,426	473,346	30	70.5	-
37	III	gf-103	P3_TB_GF103	1989	629,124	474,047	42	50	35
38		gf-104	P3_TB_GF104	1989	628,879	474,039	45	35	-
39		gf-104a	P3_TB_GF104A	1989	628,879	474,039	45	37	-
40		gf-105	P3_TB_GF105	1989	629,489	472,750	47	35	-
41		gf-106	P3_TB_GF106	1989	629,045	473,471	47	55	-
42		gf-107	P3_TB_GF107	1989	628,732	474,039	44	60	-
43		gf-108	P3_TB_GF108	1989	629,375	472,086	42	51	-
44		gf-109	P3_TB_GF109	1989	628,959	472,988	44	55.5	-
45		gf-110	P3_TB_GF110	1989	628,516	473,997	43	65.5	28
46		gf-6	P3_TB_GF6	1986	629,979	472,477	38.5	64	32.5
47		gf-7	P3_TB_GF7	1986	629,279	471,940	39.1	66.1	-
48		gf-9	P3_TB_GF9	1986	629,320	472,254	38.8	8	31.8
49		gp-8a	P3_TB_GP8A	1986	629,381	472,913	34	25	34
50	IV	gf-10	P4_TB_GF10	1986	628,920	471,946	39.6	8	-
51		gf-11	P4_TB_GF11	1986	628,799	472,404	39.6	8	-
52		gf-12	P4_TB_GF12	1986	628,257	471,978	40.2	9	-
53		gf-8	P4_TB_GF8	1986	628,794	471,503	40	61.5	-
54		tb-1	P4_TB_TB1	1992	628,703	472,138	47.8	83.5	33.8
55		tb-10	P4_TB_TB10	1992	629,072	471,639	47.8	70	39.8
56		tb-2	P4_TB_TB2	1992	628,345	473,091	47.7	84.5	39.7
57		tb-3	P4_TB_TB3	1992	628,276	472,507	47.8	84	39.8
58		tb-4	P4_TB_TB4	1992	627,899	473,369	48.1	73.5	40.1

TABLE 3-3
HISTORIC EXPLORATORY BORINGS [1]
(continued)

No	Phase	Boring Name	GIS Model Designation	DATE	Easting	Northing	Ground Surface Elevation (ft)	Maximum Depth [2] (ft)	Groundwater Elevation [2] (ft)
59		tb-5	P4_TB_TB5	1992	628,593	471,254	42.5	75.5	-
60		tb-7	P4_TB_TB7	1992	627,578	473,889	26.6	125	9.6
61		tb-7a	P4_TB_TB7A	1992	627,567	473,877	-	-	-
62		tb-8	P4_TB_TB8	1992	627,670	473,264	48.8	81.5	36.8
63		tb-9	P4_TB_TB9	1992	627,770	473,860	53.1	107.5	43.1
64		ab-1	P5_TB_AB1	1983	627,064	472,012	39.1	15	-
65		ab-2	P5_TB_AB2	1983	626,935	472,279	39.1	15	-
66		ab-3	P5_TB_AB3	1983	626,682	472,610	39	15	-
67		ab-4	P5_TB_AB4	1983	627,941	468,783	25.5	15	-
68		ab-5	P5_TB_AB5	1983	627,875	469,185	23.3	15	-
69		ab-6	P5_TB_AB6	1983	627,825	469,503	26.9	15	-
70		eeb-31	P5_TB_EEB-31	-	628,283	471,215	38	-	-
71		p-2	P5_TB_P2	1981	628,155	471,189	35.1	170	4.1
72		tb-11	P5_TB_TB11	1993	627,768	471,638	46.2	81.6	-
73	V	tb-12	P5_TB_TB12	1993	627,419	472,524	47.8	86.6	37.8
74		tb-13	P5_TB_TB13	1993	627,112	473,101	47.8	101.5	29.8
75		tb-14	P5_TB_TB14	1993	627,092	472,070	47.3	86.5	37.3
76		tb-15	P5_TB_TB15	1993	626,463	473,003	48.3	101.5	34.3
77		tb-16	P5_TB_TB16	1993	627,714	471,252	41.4	75.5	27.4
78		tb-17	P5_TB_TB17	1993	627,014	471,323	23.8	92	8.8
79		tb-18	P5_TB_TB18	1993	626,724	471,867	42.6	90.5	28.6
80		tb-19	P5_TB_TB19	1993	626,206	472,525	29.3	76.5	20.3
81		tb-20	P5_TB_TB20	1993	626,293	472,985	43.3	95.5	11.3
82		tb-21	P5_TB_TB21	1993	626,219	473,359	27.6	85.5	22.6
83		tb-22	P5_TB_TB22	1993	626,696	473,446	50.6	110.5	-13.4

Notes: (1) Data available in GIS.
(2) Information not available if not included.

TABLE 3-4

HISTORIC CPT SOUNDINGS [1]

No.	Phase	Sounding Name	GIS Model Designation[1]	Date	Northing	Easting	Ground Surface Elevation (ft)	Maximum Depth (ft)
1	I	SEA-1	P1_CPT_SEA-1	1998	630,102	470,544	89	109
2		SEA-9	P1_CPT_SEA-9	1998	631,193	470,633	18	36
3		SEA-19	P1_CPT_SEA-19	1998	630,407	469,417	22.5	44
4	II	gf-14	P2_CPT_GF14	-	630,106	472,458	47.5	23
5		gf-4	P2_CPT_GF4	-	630,037	471,676	38	63
6		gf-5a	P2_CPT_GF5A	-	629,087	471,433	39.8	32
7		SEA-2	P2_CPT_SEA-2	1998	629,480	471,601	96	118
8		SEA-10	P2_CPT_SEA-10	1998	630,223	472,217	48.75	75
9		SEA-17	P2_CPT_SEA-17	1998	628,621	471,133	43	64
10		SEA-18	P2_CPT_SEA-18	1998	629,490	470,944	38	61
11	III	gf-13	P3_CPT_GF13	-	629,984	472,610	46	23
12		SEA-3	P3_CPT_SEA-3	1998	629,483	472,519	65	94
13		SEA-4	P3_CPT_SEA-4	1998	628,665	473,497	64	125
14		SEA-11	P3_CPT_SEA-11	1998	629,242	473,400	37.5	75
15		SEA-12	P3_CPT_SEA-12	1998	628,827	474,276	28	84
16	IV	SEA-5	P4_CPT_SEA-5	1998	628,543	471,961	70	95
17		SEA-6	P4_CPT_SEA-6	1998	627,678	473,397	70	82
18		SEA-20	P4_CPT_SEA-20	1998	628,334	472,367	70	108
19	V	SEA-7	P5_CPT_SEA-7	1998	627,394	471,948	55	89
20		SEA-8	P5_CPT_SEA-8	1998	627,056	472,509	42.5	96
21		SEA-14	P5_CPT_SEA-14	1998	626,820	473,136	48	101
22		SEA-15	P5_CPT_SEA-15	1998	626,643	472,190	45	89
23		SEA-16	P5_CPT_SEA-16	1998	627,692	471,269	52.5	84
24	III- V	SEA-100	NA	2001	Same as SEA-3[2]		78	106
25		SEA-101	NA	2001	Same as SEA-4[2]		62	131
26		SEA-102	NA	2001	Same as SEA-5[2]		72	101
27		SEA-103	NA	2001	Same as SEA-6[2]		76	150
28		SEA-104	NA	2001	Same as SEA-7[2]		73	110
29		SEA-105	NA	2001	Same as SEA-15[2]		43.6	105

Notes: (1) Data available in GIS, except for SEA-100 though SEA-105.
(2) Ground elevation at time of test may differ from those of previous CPT soundings with the same coordinates.

TABLE 3-5

HISTORIC VANE SHEAR TESTS

No.	Phase	GIS Model Designation	Date	Northing	Easting	Ground Surface Elevation (ft)	Maximum Depth (ft)
1	I	P1_VS_A2A	1983	630,941	470,750	14.1	20
2		P1_VS_A3	1983	630,836	470,168	29.6	N/A
3		P1_VS_A4	1983	630,041	470,821	23.6	N/A
4		P1_VS_A5A	1983	630,049	470,398	22.2	N/A
5		P1_VS_G1	1983	630,947	471,374	15.9	51.5
6		P1_VS_G2	1983	630,559	471,119	20.1	45
7	II	P1_VS_GP3	1983	630,498	471,369	43.4	101.5
8		P2_VS_GP3A	1983	630,498	471,369	43.4	26.9
9		P2_VS_GP4	1983	630,160	472,469	34.4	110.5
10		P2_VS_GP4A	1983	630,150	472,458	34.6	20
11		P2_VS_GP5	1983	630,112	472,403	44	118.5
12		P2_VS_GP5A	1983	630,106	472,393	43.7	30.8
13		P2_VS_GP9	1983	629,236	471,439	39	100.5
14		P2_VS_GP9A	1983	629,234	471,438	39	26.5
15	III	P3_VS_GP7	1983	629,589	472,431	37.9	113
16		P3_VS_GP7A	1983	629,579	472,417	34	26.5
17		P3_VS_GP8	1983	629,387	472,925	37.9	98
18		P3_VS_SEA-12	1998	628,832	474,281	25	86.5
19	IV	P4_VS_GF4-1	1998	627,541	473,745	50.3	108.5
20		P4_VS_GF4-2	1998	627,489	473,916	27.8	91
21		P4_VS_GP12	1983	627,626	473,402	32.5	120
22	V	P5_VS_G10	1983	628,913	469,842	27.4	81.5
23		P5_VS_GP11	1983	628,175	471,461	34	120
24		P5_VS_SEA-14	1998	626,815	473,131	50	86.5

TABLE 3-6A

HISTORIC DILATOMETER TESTS

No.	Phase	GIS Model Designation	Date	Northing	Easting	Ground Surface Elevation (ft)	Maximum Depth (ft)
1	P1	P1_DMT_SEA-19	1998	630,412	469,422	22.5	42.6
2	P3	P3_DMT_GF111	1989	629,470	472,735	47	50
3	P3	P3_DMT_GF112A	1989	629,023	473,459	47	47
4	P3	P3_DMT_GF113	1989	629,450	472,300	42	50.5
5	P3	P3_DMT_GF114	1989	628,944	472,974	44	40
6	P3	P3_DMT_SEA-12	1998	628,837	474,286	28	81.6
7	P4	P4_DMT_SEA_13	1998	627,503	473,801	45	54.1
8	P5	P5_DMT_SEA-14	1998	626,810	473,127	48.33	82

TABLE 3-6B

HISTORIC PRESSUREMETER TESTS

No.	Phase	GIS Model Designation	Date	Northing	Easting	Ground Surface Elevation (ft)	Depth of Test (ft)
1	P1	P4_PMT_SEA-6	1998	627,678	473,397	70	39.4
2	P2	P2_PMT_SEA-18	1998	629,490	470,944	38	19.7
3	P2	P2_PMT_SEA-18	1998	629,490	470,944	38	29.5
4	P2	P2_PMT_SEA-18	1998	629,490	470,944	38	42.9

TABLE 3-7

SUMMARY OF HISTORIC SOIL LABORATORY TESTING DATA

Test	Number of Tests (1,2)					
	Total	Phase				
		I	II	III	IV	V
Atterberg Limits	58	58				
Specific Gravity	13	0	2	2	5	4
Unit Weight	146	146				
Organic Content	15	6	0	4	5	0
Ash Content	11	0	0	0	5	6
Unconfined Compression Test	42	20	7	9	4	2
Triaxial Tests	34	0	1	3	21	9
Consolidation Tests	43	18	5	12	5	3
Hydraulic Conductivity	36	13	4	11	4	4

Note: (1) = Based on GIS database.
(2) = Data is based on historical references listed in Table 3-1.

TABLE 3-8

2002 CPT INFORMATION

No.	CPT Designation	Phase	Area: Waste or Dike	Easting (ft)	Northing (ft)	Depth (ft)	Surveyed Ground Elevation (ft)	Final CPT Tip Elevation (ft)
1	P1-02-C001	I	D	471,365.33	630,647.14	125	21.0	-104.0
2	P2-02-C002	II	D	472,088.31	630,342.49	70	40.7	-29.4
3	P3-02-C003	III	D	472,623.77	629,897.16	125	46.9	-78.2
4	P3-02-C004	III	D	472,961.55	629,580.24	70	45.6	-24.4
5	P3-02-C005	III	D	473,312.68	629,282.85	125	42.7	-82.3
6	P3-02-C006	III	D	473,646.72	629,037.59	70	45.7	-24.3
7	P3-02-C007	III	D	473,793.84	628,961.96	125	47.2	-77.8
8	P3-02-C008	III	D	474,122.56	628,795.09	125	41.6	-78.9
9	P3-02-C009	III	D	474,312.89	628,918.32	150	23.9	-126.1
10	P3-02-C010	III	D	474,233.83	628,375.56	70	17.9	-57.3
11	P3-02-C011	III	D	474,025.04	628,320.85	125	43.1	-81.9
12	P3-02-C012	III	D	474,171.14	628,149.21	150	11.0	-139.0
13	P4-02-C013	IV	D	473,929.23	628,036.67	70	46.0	-24.0
14	P4-02-C014	IV	D	474,064.27	627,937.15	70	12.4	-57.6
15	P4-02-C015	IV	D	473,852.92	627,843.08	125	47.5	-77.5
16	P4-02-C016	IV	D	474,011.11	627,603.22	125	14.7	-110.3
17	P4-02-C017	IV	D	473,757.65	627,633.50	125	49.6	-75.4
18	P4-02-C018	IV	D	473,663.61	627,343.99	125	49.0	-76.0
19	P4-02-C019	IV	D	473,800.47	627,146.77	150	17.2	-132.9
20	P5-02-C020	V	D	473,546.46	627,130.88	125	47.6	-77.4
21	P5-02-C021	V	D	473,567.85	626,942.58	70	45.3	-24.7
22	P5-02-C022	V	D	473,368.89	626,978.19	70	44.6	-25.4
23	P5-02-C023	V	D	473,381.42	626,713.51	125	44.4	-80.6
24	P5-02-C024	V	D	473,218.08	626,859.75	125	47.6	-77.4
25	P5-02-C025	V	D	473,001.52	626,458.38	70	45.0	-25.0
26	P5-02-C026	V	D	472,902.00	626,596.97	125	50.6	-74.4
27	P5-02-C027	V	D	472,735.08	626,444.28	125	51.7	-73.3
28	P5-02-C028	V	D	472,546.78	626,469.22	70	44.9	-19.5
29	P5-02-C029	V	D	472,454.61	626,336.20	150	30.6	-119.4
30	P5-02-C030	V	D	472,390.43	626,554.54	125	45.5	-75.2

TABLE 3-8
2002 CPT INFORMATION
(continued)

No.	CPT Designation	Phase	Area: Waste or Dike	Easting (ft)	Northing (ft)	Depth (ft)	Surveyed Ground Elevation (ft)	Final CPT Tip Elevation (ft)
31	P5-02-C031	V	D	472,013.17	626,544.51	125	22.2	-102.8
32	P5-02-C032	V	D	472,171.90	626,673.64	125	45.5	-75.1
33	P5-02-C033	V	D	471,985.35	626,774.96	70	45.5	-21.0
34	P5-02-C034	V	D	471,784.58	626,876.29	125	46.8	-78.2
35	P5-02-C035	V	D	471,580.30	627,026.63	125	46.2	-74.8
36	P5-02-C036	V	D	471,466.57	627,147.47	70	47.2	-22.8
37	P5-02-C037	V	D	471,358.16	627,309.16	70	48.4	-21.6
38	P5-02-C038	V	D	471,260.40	627,460.18	125	47.7	-77.3
39	P5-02-C039	V	D	471,208.74	627,849.21	125	41.3	-83.7
40	P5-02-C040	V	D	471,217.54	628,053.49	70	41.5	-28.5
41	P5-02-C041	V	D	471,242.29	628,359.01	125	44.0	-81.0
42	P5-02-C042	V	D	471,256.45	628,479.79	70	51.3	-18.7
43	P2-02-C043	II	D	470,975.67	629,021.27	125	43.6	-85.4
44	P1-02-C044	I	D	470,218.56	629,916.94	70	40.0	-30.0
45	P1-02-C045	I	D	469,925.33	630,636.11	70	20.5	-49.6
46	P1-02-C046	I	D	470,408.34	631,069.64	125	18.1	-106.9
47	P1-02-C0047	I	D	470,950.46	630,958.75	75	50.0	-25.0
48	P1-02-C0048	I	D	470,165.00	630,810.37	70	20.0	-50.0
49	P1-02-C0049	I	D	469,769.31	630,179.11	70	38.0	-32.0
50	P1-02-C0050	I	D	470,562.28	629,713.18	70	37.0	-33.0
51	P2-02-C0051	II	W	471,561.48	630,301.60	90	80.0	-10.0
52	P3-02-C0052	III	W	473,888.52	628,600.84	85	59.0	-26.0
53	P4-02-C0053	IV	W	473,570.00	627,950.00	90	68.9	-27.0
54	P5-02-C0054	V	W	473,126.20	627,192.84	85	73.6	-22.0
55	P2-02-C0055	II	W	471,563.00	630,298.00	90	80.0	-10.0
56	P5-02-C0056	V	W	471,973.73	627,319.81	90	74.0	-16.0
57	P5-02-C0057	V	W	471,507.04	627,640.24	90	72.0	-18.0
58	P3-02-C0058	III	W	472,424.44	629,533.39	90	75.0	-15.0
59	P1-02-C0059	I	W	470,517.63	630,158.01	95	90.0	-5.0
60	P5-02-C0060	V	W	472,380	627,760	64.6	83.8	19.2
61	P5-02-C0061	V	W	471,780	627,775	26.4	78.9	52.5

TABLE 3-8
2002 CPT INFORMATION
(continued)

No.	CPT Designation	Phase	Area: Waste or Dike	Easting (ft)	Northing (ft)	Depth (ft)	Surveyed Ground Elevation (ft)	Final CPT Tip Elevation (ft)
62	P3-02-C0062	III	W	472,282	629,212	69.2	77.2	8.0
63	P1-02-C0063	I	W	470,240	630,281	19.2	87.5	68.3
64	P1-02-C0064	I	W	470,240	630,281	33.0	87.5	54.5
65	P2-02-C0065	II	W	471,615	629,487	63.3	93.1	29.8
66	P2-02-C0066	II	W	471,615	629,487	43.5	93.1	49.6
67	P3-02-C0067	III	W	472,943	629,025	59.2	63.3	4.1
68	P4-02-C0068	IV	W	471,935	628,598	71.2	82.6	11.4

TABLE 3-9

2002 EXPLORATORY BORING INFORMATION

No.	Exploratory Boring Designation	Phase	Easting (ft)	Northing (ft)	Ground Elevation (ft)	ID of Nearest CPT	Max. Depth (ft)
1	P3-02-S01	III	474,312.89	628,918.32	24.0	CPT 9	120
2	P3-02-S02	III	474,171.14	628,149.21	5.0	CPT 12	160
3	P4-02-S03	IV	473,710.63	627,488.745	48.0	Between CPT 17 and 18	120
4	P5-02-S04	V	473,218.08	626,859.75	37.0	CPT 24	160
5	P5-02-S05	V	471,217.54	628,053.49	47.5	CPT 40	160
6	P1-02-S06	I	470,944	629,490	44.0	Between CPT 43 and 50	120
7	P1-02-S07	I	469,417	630,407	22.5	CPT 49	140
8	P1-02-S08	I	470,408.34	631,069.64	20.0	CPT 46	120
9	P3-02-S09	III	472,623.77	629,897.16	47.5	CPT 3	160
10	P5-02-S10	V	471,985.35	626,774.96	48.0	CPT 33	160
11	P1-02-S11	I	471,365.33	630,647.14	22.5	CPT 1	120
12	P3-02-S12	III	473,646.72	629,037.59	45.5	CPT 6	120
13	P3-02-S13	III	473,537.45	628,559.92	69.8	Between CPT 67 and 11	55
14	P4-02-S14	IV	473,582.21	627,993.50	75.2	CPT 53	56
15	P5-02-S15	V	473,176.26	627,218.29	73.2	CPT 54	55
16	P5-02-S16	V	471,925.67	627,328.01	73.5	CPT 56	51.7
17	P2-02-S17	II	471,666.55	629,417.19	91.4	CPT 65 and 66	85

Note: Exploratory Borings S01 through S012 were performed along the dike, while S013 through S017 were performed through the waste.

TABLE 3-10

2002 VANE SHEAR TEST INFORMATION

No.	Phase	SPT Boring Designation	Test No.	Ground Elevation (ft)	Depth (ft)	VS Test Elevation (ft)	Soil Layer
1	III	P3-02-S01	1	23.9	14.7	9.2	Dredge
2			2		24.7	-0.8	Alluvium
3			3		44.7	-20.8	Alluvium
4			4		75.7	-51.8	
5	III	P3-02-S02	1	11	24.6	-13.6	Alluvium
6			2		54.7	-43.7	
7	IV	P4-02-S03	1	49.0	24.7	24.3	Dredge
8			2		46.7	2.3	Dredge
9			3		96.2	-47.2	
10	V	P5-02-S04	1	47.6	34.7	12.9	Dredge
11			2		84.7	-37.1	Alluvium
12	V	P5-02-S05	1	41.5	14.7	26.8	Dredge
13			2		34.7	6.8	Dredge
14			3		59.7	-18.2	Alluvium
15	I	P1-02-S08	1	18.1	24.7	-6.6	Alluvium
16			2		76.5	-58.4	
17	III	P3-02-S09	1	46.9	24.7	22.15	Dredge
18			2		46.7	0.15	Dredge
19			3		70.2	-23.35	Alluvium
20	V	P5-02-S10	1	49.0	19.7	29.32	Dredge
21			2		47.7	1.32	Dredge
22			3		64.7	-15.68	Alluvium
23	I	P1-02-S11	1	21.0	24.7	-3.7	Alluvium
24			2		34.7	-13.7	Alluvium
25	III	P3-02-S12	1	45.7	23.7	22	Dredge
26			2		56.7	-11	Alluvium
27			3		79.7	-34	Alluvium

TABLE 3-10

2002 VANE SHEAR TEST INFORMATION
(Continued)

No.	Phase	SPT Boring Designation	Test No.	Ground Elevation (ft)	Depth (ft)	VS Test Elevation (ft)	Soil Layer
28	III	P3-02-S13	1	69.8	34.7	35.1	Dredge
29			2		39.7	30.1	Dredge
30			3		46.7	23.1	Dredge
31			4		51.7	18.1	Dredge
32	IV	P4-02-S14	1	75.2	30.7	44.5	Waste
33			2		38.7	36.5	Dredge
34			3		43.7	31.5	Dredge
35			4		48.7	26.5	Dredge
36			5		53.7	21.5	Dredge
37	V	P5-02-S015	1	73.2	40.2	33.0	Dredge
38			2		44.7	28.5	Dredge
39			3		49.7	23.5	Dredge
40			4		54.7	18.5	Dredge
41	V	P5-02-S016	1	73.5	36.7	36.8	Dredge
42			2		41.7	31.8	Dredge
43			3		46.7	26.8	Dredge
44			4		51.7	21.8	Dredge
45	II	P2-02-S017	1	91.4	65.7	25.7	Dredge
46			2		70.7	20.7	Dredge
47			3		76.7	14.7	Dredge
48			4		81.7	9.7	Dredge

TABLE 3-11

2002 LABORATORY TESTS

Test	ASTM Designation	Number of Tests
Atterberg Limits	D 4318	61
Sieve Analysis w/Hydrometer	D 5084	24
Sieve Analysis w/o Hydrometer	D 422	27
Natural Water Content	D 2216	104
Loss On Ignition	D 2863	31
Sample Unit Weight	D 2937	15
One-Dimensional Consolidation	D 2435	28 Vertically Trimmed Samples 2 Horizontally Trimmed Samples
Triaxial "R" Test (Consolidated Undrained with Pore Pressure Measurements)	D 4767	13
Direct Shear	D 3080	4 Vertically Trimmed Samples 4 Horizontally Trimmed Samples
Direct Simple Shear	D 6528	10

TABLE 4-1

SUMMARY OF HISTORIC AND 2002 CPT DATA ALONG DIKE

Figure No.	Station		Zone	Phase(s)	No. of Existing CPTs	No. of New CPTs
	Start	Finish				
4-2	0	25+00	Dike	I, II	1	3
4-3	25+00	50+00	Dike & Toe	III	2	7
4-4	50+00	75+00	Dike	III to V	1	10
4-5	50+00	75+00	Toe	III to V	1	8
4-6	75+00	100+00	Dike & Toe	V	2	12
4-7	100+00	125+00	Dike	I, II, V	2	6
4-8	125+00	150+00	Dike	I	2	4

TABLE 5-1

SUMMARY OF PIEZOMETER INFORMATION

PHASE I PIEZOMETERS

NAME	Phase	Line	Formation	Date Installed	Elevations (ft)			
					Ground Surface	Screen Top	Screen Bottom	Sandpack Bottom
P1_IC_04B	P1	1	Dredge	Apr-91				
P1_IC_06A	P1	1	Dredge	Apr-91				
P1_IC_06B	P1	1	Dredge	Apr-91				
P1_IC_07A	P1	1	Dredge	Apr-91				
P1_IC_07B	P1	1	Dredge	Apr-91				
P1_IC_08A	P1	1	Dredge	Apr-91				
P1_IC_08B	P1	1	Dredge	Apr-91				
P1_MW_C100	P1	1	Columbia	Aug-83	15.6	-9.4	-19.4	-26.4
P1_MW_P100	PI	1	Potomac	Aug-83	15.3	-30.7	-40.7	-40.7
P1_PZ_0P4	P1	1	Columbia	1985				
P1_IC_01A	P1	2	Dredge	Apr-91				
PI_IC_01B	P1	2	Dredge	Apr-91				
P1_IC_01C	P1	2	Dredge	Nov-96	20.7	7.3	5.3	3.9
P1_IC_02A	P1	2	Dredge	Apr-91				
P1_IC_02C	P1	2	Dredge					
P1_IC_0P3	P1	2	Dredge	1985				
P1_IC_P3A	P1	2	Dredge	1985				
P1_MW_C107	P1	2	Alluvial	Aug-83	10.3	-5.7	-4.7	-9.7
P1_MW_P107	P1	2	Columbia	Aug-83	9.6	-34.4	-40.4	-40.4

TABLE 5-1

SUMMARY OF PIEZOMETER INFORMATION
(continued)

PHASE II PIEZOMETERS

Name	Phase	Line	Formation	Date Installed	Elevations (ft)			
					Ground Surface	Screen Top	Screen Bottom	Sandpack Bottom
P2_MW_LC101A	P2	1	Columbia	Feb-96	51	-22	-33	-33
P2_MW_C101	P2	1	Columbia	Aug-83	35.2	-24.8	-34.8	-36.8
P2_MW_P101	P2	1	Potomac	Aug-83	46.9	-93.6	-103.6	-103.6
P2_MW_P101A	P2	1	Potomac	Mar-96	47	-108	-118	-122
P2_PZ_01A	P2	1	Dredge					
P2_PZ_01B	P2	1	Dredge					
P2_PZ_01C	P2	1	Dredge	Sep-90				
P2_PZ_01D	P2	1	Dredge	Sep-90				
P2_PZ_02A	P2	1	Dredge					
P2_PZ_02B	P2	1	Dredge					
P2_PZ_07A	P2	1	Dredge	Oct-90				
P2_PZ_07B	P2	1	Dredge	Oct-90				
P2_PZ_08A	P2	1	Dredge	Jan-91				
P2_PZ_08B	P2	1	Dredge	Jan-91				
P2_PZ_09A	P2	1	Dredge	Oct-90				
P2_PZ_09B	P2	1	Dredge	Oct-90				
P2_MW_LC111	P2	2						
P2_MW_LC113	P2	2	Alluvial	Feb-96	40.8	-24.2	-34.2	-34.2
P2_PZ_06A	P2	2	Dredge					
P2_PZ_06B	P2	2	Dredge					
P2_PZ_12A	P2	2	Dredge	Oct-96	40.9	28.7	30.7	28
P2_PZ_12B	P2	2	Dredge	Oct-96	41			1
P2_PZ_12C	P2	2	Dredge	Mar-97	40	6	8	5.8

TABLE 5-1

SUMMARY OF PIEZOMETER INFORMATION
(continued)

PHASE III PIEZOMETERS

Name	Phase	Line	Formation	Date Installed	Elevations (ft)			
					Ground Surface	Screen Top	Screen Bottom	Sandpack Bottom
P3_PZ_01A	P3	1						
P3_PZ_01B	P3	1						
P3_PZ_02A	P3	1						
P3_PZ_02B	P3	1						
P3_PZ_101A	P3	1						
P3_PZ_101B	P3	1						
P3_PZ_102B	P3	1						
P3_PZ_013	P3	2						
P3_PZ_03A	P3	2						
P3_PZ_03B	P3	2						
P3_PZ_04A	P3	2						
P3_PZ_04B	P3	2						
P3_PZ_102A	P3	2						
P3_PZ_103A	P3	2						
P3_PZ_103B	P3	2						
P3_PZ_103D	P3	2						
P3_PZ_103E	P3	2						
P3_IC_005	P3	3						
P3_IC_05A	P3	3						
P3_PZ_05A	P3	3						
P3_PZ_05B	P3	3						
P3_PZ_104A	P3	3						
P3_PZ_104B	P3	3						
P3_PZ_104D	P3	3						
P3_PZ_104E	P3	3						
P3_PZ_06A	P3	4						
P3_PZ_06B	P3	4						
P3_PZ_07A	P3	4						
P3_PZ_07B	P3	4						
P3_PZ_105A	P3	4						
P3_PZ_105B	P3	4						
P3_PZ_105D	P3	4						
P3_MW_C102	P3	5	Alluvial	Jul-83	26.3	-19.7	-29.7	-43.7
P3_MW_C102A	P3	5	Alluvial	Jan-96	28	-7	-17	-17
P3_MW_P102	P3	5	Potomac	Jul-83	26	-129	-139	-139
P3_MW_P102A	P3	5		Feb-96	113	-42	-52	-54
P3_PZ_012	P3	5						
P3_PZ_08A	P3	5						
P3_PZ_08B	P3	5						
P3_PZ_09A	P3	5						
P3_PZ_09B	P3	5						
P3_PZ_10A	P3	5						
P3_PZ_10B	P3	5						
P3_PZ_11A	P3	5						
P3_PZ_11B	P3	5						

TABLE 5-1

SUMMARY OF PIEZOMETER INFORMATION
(continued)

PHASE IV PIEZOMETERS

Name	Phase	Line	Formation	Date Installed	Elevations (ft)			
					Ground Surface	Screen Top	Screen Bottom	Sandpack Bottom
P4_PZ_023A	P4	1	Dredge	Jan-94	46.8	25.8	23.8	22.8
P4_PZ_023B	P4	1	Dredge	Jan-94	46.2	7.2	5.2	1.2
P4_PZ_023C	P4	1	Alluvial	Jan-94	47	-13	-15	-19
P4_PZ_024A	P4	1	Dredge	Jan-94	46.7	25.7	23.7	22.7
P4_PZ_024B	P4	1	Dredge	Jan-94	46.9	6.9	4.9	-0.1
P4_PZ_024C	P4	1	Alluvial	Jan-94	46.5	-12	-14	-18.5
P4_PZ_025A	P4	2	Dredge	Jul-94	45.7	23.2	21.2	19.7
P4_PZ_025B	P4	2	Dredge	Jul-94	45.7	4.7	2.7	0.7
P4_PZ_025C	P4	2	Alluvial		45.6	-12.6	-14.6	-15.9
P4_PZ_026A	P4	2	Dredge	Jul-94	47	28	26	24
P4_PZ_026B	P4	2	Dredge	Jul-94	46.5	10	8	6.5
P4_PZ_026C	P4	2	Alluvial	Jul-94	46.5	-5	-7	-8.5
P4_PZ_002	P4	3	Alluvial	Oct-93	32	3	-7	-8
P4_PZ_004	P4	3	Dredge	Jan-94	52.3	37.8	27.8	24.8
P4_PZ_007	P4	3	Dredge	Jan-94	48	24	14	13
P4_PZ_011	P4	3	Dredge	Jan-94	52	38	28	27
P4_PZ_014	P4	3	Dredge	Jan-94	47	24	14	13
P4_PZ_017	P4	3	Dredge	Jan-94	51	38	28	27
P4_PZ_021	P4	3	Dredge	Oct-93	46	24	14	13
P4_PZ_001	P4	4	Dredge	Jan-94	47	24	14	13
P4_PZ_027	P4	4	Columbia	Oct-93	36.7	-33.8	-35.8	-36.8
P4_PZ_005	P4	4	Dredge	Oct-93	37	14	4	1
P4_PZ_008	P4	4	Alluvial	Oct-93	30.5	3	-7	-8
P4_PZ_028	P4	4	Columbia	Oct-93	39	-37	-39	-40
P4_PZ_012	P4	4	Dredge	Oct-93	37.8	14	4	2.8
P4_PZ_015	P4	4	Alluvial	Oct-93	31	3	-7	-8
P4_PZ_019	P4	4	Dredge	Nov-96	37.3	14	4	3
P4_PZ_029	P4	4	Columbia	Nov-96	40.37	-37.63	-39.63	-40.63
P4_PZ_022	P4	4	Alluvial	Nov-96	31	3	-7	-11

TABLE 5-1

SUMMARY OF PIEZOMETER INFORMATION
(continued)

PHASE V PIEZOMETERS

Name	Phase	Line	Formation	Date Installed	Elevations (ft)			
					Ground Surface	Screen Top	Screen Bottom	Sandpack Bottom
P5_PP_01A	P5	1	Dredge					
P5_PP_01B	P5	1	Dredge					
P5_PP_01C	P5	1	Dredge					
P5_PZ_14A	P5	1	Dredge	Sep-96	46.4	37	27	25.9
P5_PZ_14B	P5	1	Dredge	Sep-96	46.9	17.8	7.8	5.9
P5_PZ_14C	P5	1	Alluvial	Sep-96	47.5	-3	-13	-14.5
P5_PP_02A	P5	2	Dredge					
P5_PP_02B	P5	2	Dredge					
P5_PP_02C	P5	2	Dredge					
P5_PP_06A	P5	2	Dredge					
P5_PP_06B	P5	2	Dredge					
P5_PP_06C	P5	2	Dredge					
P5_PP_09A	P5	3						
P5_PP_09B	P5	3						
P5_PP_09C	P5	3						
P5_PP_10A	P5	3						
P5_PP_10B	P5	3						
P5_PZ_15A	P5	3	Dredge	Dec-96	47.1	33.1	23.1	22.1
P5_PZ_15B	P5	3	Dredge	Dec-96	47.2	7.8	-2.2	-3.3
P5_PZ_15C	P5	3	Alluvial	Dec-96	47.5	-17.2	-27.2	-28.5
P5_PP_03A	P5	4						
P5_PP_03B	P5	4						
P5_PP_03C	P5	4						
P5_PZ_10A	P5	4		Oct-96	49	16	6	5
P5_PZ_10B	P5	4		Oct-96	49	-4	-14	-15
P5_PZ_09B	P5	4		Sep-96	48	16	6	5
P5_PZ_11A	P5	4		Sep-96	23	16	6	5
P5_PZ_11B	P5	4		Sep-96	23	-1	-11	-12
P5_PZ_08A	P5	4		Oct-96	47	11	1	0
P5_PZ_08B	P5	4		Oct-96	47	-12	-22	-23
P5_PZ_12A	P5	4		Sep-96	22.5	-1	-11	-12.5
P5_PZ_12B	P5	4		Sep-96	22.5	-18	-28	-29.5
P5_PZ_07A	P5	5		Nov-96	47	32.5	22.5	21.5
P5_PZ_07B	P5	5		Nov-96	47	9.5	-0.5	-1.5
P5_PZ_06A	P5	5		Nov-96	46	7.2	-2.8	-4
P5_PZ_06B	P5	5		Nov-96	46	-18	-28	-29
P5_PZ_05A	P5	5		Nov-96	46.8	32.1	22.1	20.8
P5_PZ_05B	P5	5		Nov-96	47	7	-3	-4
P5_PZ_13A	P5	5		Sep-96	26.2	15	5	3.2
P5_PZ_13B	P5	5		Sep-96	25.3	-3	-13	-14.2
P5_PZ_09A	P5	5		Sep-96	48.2	36	26	24.7
P5_PZ_01A	P5	6		Nov-96	51	32	22	30
P5_PZ_01C	P5	6						
P5_PZ_02A	P5	6		Nov-96	52	7	-3	-4

TABLE 5-1

SUMMARY OF PIEZOMETER INFORMATION
(continued)

Name	Phase	Line	Formation	Date Installed	Elevations (ft)			
					Ground Surface	Screen Top	Screen Bottom	Sandpack Bottom
P5_PZ_02B	P5	6		Nov-96	52	-18	-28	-29
P5_PZ_04A	P5	6		Nov-96	52	7	-3	-4
P5_PZ_04B	P5	6		Oct-96	52	-18	-28	-28
P5_MW_P106	P5	7		Jul-83	28	-127	-137	-137
P5_MW_UC106	P5	7		Jul-83	27.6	-6.4	-16.4	-32.4
P5_PP_10C	P5	7						
P5_PZ_03A	P5	7		Nov-96	41	32	22	21
P5_PZ_03B	P5	7		Nov-96	41	7	-3	-4

TABLE 5-2

SUMMARY OF SETTLEMENT PLATE INFORMATION

Settlement Plate	Initial Dredge Spoils		Final Dredge Spoils		Final Waste		
	Elevation	Date	Elevation	Date	Elevation	Depth	Date
P1_SP_1AA	7.1	15-Dec-92	2.0	1-Oct-01	59.7	57.7	1-Oct-01
P1_SP_1BA	11.4	15-Dec-92	-4.3	1-Oct-01	71.5	75.8	1-Oct-01
P1_SP_3	15.6	15-Dec-92	14.0	1-Oct-01	26.9	12.9	1-Oct-01
P1_SP_4	17.3	15-Dec-92	16.1	1-Oct-01	22.5	6.4	1-Oct-01
P1_SP_5	21.3	15-Dec-92	15.1	1-Oct-01	58.1	43.0	1-Oct-01
P1_SP_6	25.6	15-Dec-92	7.4	1-Oct-01	91	83.6	1-Oct-01
P1_SP_7	24.8	15-Dec-92	6.6	1-Oct-01	90.3	83.7	1-Oct-01
P2_SP_1	34.5	14-Apr-94	29.6	1-Oct-01	74.9	45.3	1-Oct-01
P2_SP_2	35.4	14-Apr-94	31.3	1-Oct-01	77.1	45.8	1-Oct-01
P2_SP_4	33.7	14-Apr-94	22.8	1-Oct-01	78.8	56.0	1-Oct-01
P2_SP_5	33.2	14-Apr-94	21.3	1-Oct-01	92.4	71.1	1-Oct-01
P2_SP_7	33.6	14-Apr-94	14.6	1-Oct-01	88.2	73.6	1-Oct-01
P2_SP_8	33.0	14-Apr-94	24.6	1-Oct-01	93.5	68.9	1-Oct-01
P2_SP_10	35.2	14-Apr-94	30.0	1-Oct-01	80.8	50.8	1-Oct-01
P2_SP_13	36.2	14-Apr-94	28.7	1-Oct-01	84.2	55.5	1-Oct-01
P2_SP_14	36.7	14-Apr-94	33.2	1-Oct-01	77.1	43.9	1-Oct-01
P3_SP_1	44.9	1-Feb-94	40.1	1-Oct-01	78.8	38.7	1-Oct-01
P3_SP_2	46.4	1-Feb-94	41.1	1-Oct-01	76.8	35.7	1-Oct-01
P3_SP_4	45.5	1-Feb-94	41.4	24-Apr-01	70.2	28.8	24-Apr-01
P3_SP_5	45.4	1-Feb-94	40.4	1-Oct-01	78.1	37.7	1-Oct-01
P3_SP_8	43.6	1-Feb-94	39.7	1-Oct-01	63.3	23.6	1-Oct-01
P3_SP_11	45.0	1-Feb-94	39.7	1-Oct-01	56.6	16.9	1-Oct-01
P3_SP_11A	44.7	15-Aug-94	41.49	21-Apr-98	62.5	21.0	2-Jan-98
P3_SP_12	44.7	1-Feb-94	40.6	1-Dec-01	61.1	20.5	1-Oct-01
P3_SP_13	43.6	1-Feb-94	39.5	1-Oct-01	57.8	18.3	1-Oct-01
P3_SP_14	43.6	1-Feb-94	41.2	1-Oct-01	59.8	18.6	1-Oct-01
P4_SP_1	46.65	4-Apr-97	42.05	1-Oct-01	78.8	36.8	1-Oct-01
P4_SP_2	47.9	4-Apr-97	45.1	8-Aug-01	75.1	30	8-Aug-01
P4_SP_5	45.9	4-Apr-97	43.5	1-Oct-01	61.5	18	1-Oct-01
P4_SP_6	48.9	4-Apr-97	46.3	1-Oct-01	76.6	30.3	1-Oct-01
P4_SP_7	47.15	4-Apr-97	43.85	1-Oct-01	60.2	16.35	1-Oct-01
P4_SP_8	47.4	4-Apr-97	43.1	1-Oct-01	62.8	19.7	1-Oct-01
P5_SP_1	48.13	31-Jul-97	42.03	1-Oct-01	79.6	37.57	1-Oct-01
P5_SP_2	48.12	4-Apr-97	43.52	1-Oct-01	72.8	29.28	1-Oct-01
P5_SP_3	48.88	31-Jul-97	45.88	1-Oct-01	67.8	21.92	1-Oct-01
P5_SP_4	49.09	4-Apr-97	46.69	1-Oct-01	72.2	25.51	1-Oct-01
P5_SP_5	48.28	31-Jul-97	41.48	1-Oct-01	70.9	29.42	1-Oct-01
P5_SP_6	48.28	17-Oct-97	45.58	1-Oct-01	75.9	30.32	1-Oct-01
P5_SP_7	48.07	17-Oct-97	44.47	1-Oct-01	69.9	25.43	1-Oct-01
P5_SP_8	47.84	31-Jul-97	44.94	1-Oct-01	73.4	28.46	1-Oct-01
P5_SP_9	48.21	31-Jul-97	43.41	1-Oct-01	71.5	28.09	1-Oct-01
P5_SP_10	48.22	17-Oct-97	45.0	1-Oct-01	74.2	29.18	1-Oct-01

TABLE 5-3

ORGANIZATION OF PIEZOMETERS BY CLUSTERS

Phase	Feature	Figure No.
I	Plan View	5-3
	Lines 1 and 2	5-4 and 5-5
II	Plan View	5-6
	Lines 1 and 2	5-7 and 5-8
III	Plan View	5-9
	Lines 1 to 5	5-10 to 5-14
IV	Plan View	5-15
	Lines 1 to 4	5-16 to 5-19
V	Plan View	5-20
	Lines 1 to 7	5-21 to 5-27

TABLE 6-1

2002 VST-BASED UNDRAINED SHEAR STRENGTH (DREDGE AND ALLUVIUM UNDER PERIMETER BERM)

VST No.	Ground Elevation (ft)	Depth (ft)	Test Elevation (ft)	Measured		Keep Peak Value?	PI (%)	Bjerrum Correction Factor μ	Corrected Peak $S_u = \mu \times S_u(FV)$ (psf)	Estimated σ_v' (psf)	S_u/σ_v'	$S_{u\,res}/S_u$
				Peak S_u (FV) (psf)	Residual $S_{u\,res}$ (FV) (psf)							
S01	23.9	14.7	9.2	321	58	Y	53	0.76	244	1,031	0.24	0.18
		24.7	-0.8	630	221	Y	53	0.76	479	1,307	0.37	0.35
		44.7	-20.8	836	221	Y	40	0.83	691	1,964	0.35	0.26
		75.7	-51.8	987	151	Y	21	1.00	990	2,970	0.33	-
S02	11	24.6	-13.6	786	70	Y	40	0.83	650	1,059	0.61	-
		54.7	-43.7	1,216	469	N	-	-	-	-	-	0.39
S03	49	24.7	24.3	391	122	Y	53	0.76	297	1,615	0.18	0.31
		46.7	2.3	456	NA	Y	53	0.76	346	2,226	0.16	-
		96.2	-47.2	783	342	Y	21	1.00	785	3,830	0.21	0.44
S04	41.5	34.7	6.8	332	NA	Y	53	0.76	252	2,134	0.12	-
		84.7	-43.2	2,795	2,279	N	-	-	-	-	-	-
S05	41.5	14.7	26.8	480	49	Y	53	0.76	365	1,328	0.27	-
		34.7	6.8	923	58	Y	53	0.76	701	1,891	0.37	-
		59.7	-18.2	1,510	462	Y	40	0.83	1248	2,680	0.47	0.31
S08	18.1	24.7	-6.6	652	55	Y	40	0.83	539	1,336	0.40	0.08
		76.5	-58.4	3,133	1,972	N	-	-	-	-	-	-
S09	46.85	24.7	22.2	737	90	Y	53	0.76	560	1,606	0.35	0.12
		46.7	0.2	980	105	Y	53	0.76	745	2,217	0.34	-
		70.2	-23.4	2,775	1,302	N	-	-	-	-	-	-
S010	45.5	19.7	25.8	698	235	Y	53	0.76	530	1,479	0.36	0.34
		47.7	-2.2	2,040	675	N	-	-	-	-	-	-
		64.7	-19.2	1,308	869	Y	40	0.83	1081	2,821	0.38	0.66
S011	21	24.7	-3.7	245	28	N	-	-	-	-	-	-
		34.7	-13.7	2,236	410	N	-	-	-	-	-	0.18
S012	45.7	23.7	22.0	458	46	Y	53	0.76	348	1,588	0.22	-
		56.7	-11.0	634	23	Y	40	0.83	524	2,552	0.21	0.04
		79.7	-34.0	1,123	1,063	Y	40	0.83	928	3,400	0.27	0.95
										Averages	0.31	0.33

Notes: Some peak values are reject due to 1) too close to sand, 2) interbedded sand layers, 3) too low (possibly, excessive disturbance), 4) too high (possibly, sandy soil).

TABLE 6-1

2002 VST-BASED UNDRAINED SHEAR STRENGTH (DREDGE AND ALLUVIUM UNDER PERIMETER BERM) (Continued)

VST No.	Ground Elevation (ft)	Depth (ft)	Test Elevation (ft)	Measured		Keep Peak Value?	PI [1] (%)	Bjerrum Correction Factor μ	Corrected Peak $S_u = \mu \times S_u(FV)$ (psf)	Measured $S_{u\,res}/S_u$
				Peak S_u (FV) (psf)	Residual $S_{u\,res}$ (FV) (psf)					
S13	69.77	34.7	35.07	377	102	Y	53	0.76	286	0.27
		39.7	30.07	328	91	Y	53	0.76	249	0.28
		46.7	23.07	374	21	Y	53	0.76	284	0.06
		51.7	18.07	164	67	N	-	-	-	-
S014	75.21	38.7	36.51	240	55	Y	53	0.76	182	0.23
		43.7	31.51	193	173	Y	53	0.76	147	0.90
		48.7	26.51	514	49	Y	53	0.76	390	0.10
		53.7	21.51	362	67	Y	53	0.76	275	0.19
S15	49	40.2	33.01	120	102	N	-	-	-	-
		44.7	28.51	316	52	Y	53	0.76	240	0.16
		49.7	23.51	233	126	Y	53	0.76	177	0.54
		54.7	18.51	333	158	Y	53	0.76	253	0.47
S16	41.5	36.7	36.8	269	233	Y	53	0.76	204	0.87
		41.7	31.8	176	149	Y	53	0.76	134	0.85
		46.7	26.8	146	102	N	-	-	-	-
		51.7	21.8	258	137	Y	53	0.76	196	0.53
S17	41.5	65.7	25.74	543	88	Y	53	0.76	413	0.16
		70.7	20.74	667	132	Y	53	0.76	507	0.20
		76.7	14.74	348	108	Y	53	0.76	264	0.31
		81.7	9.74	959	155	Y	53	0.76	729	0.16
									Average	0.37

Notes: 1) Assumed from dredge 2002 Atterberg limits. Some peak values are reject due to 1) too close to sand, 2) interbedded sand layers, 3) too low (possibly, excessive disturbance), 4) too high (possibly, sandy soil).

TABLE 6-2

HISTORIC CIL VST-BASED UNDRAINED SHEAR STRENGTH (DREDGE AND ALLUVIUM UNDER PERIMETER BERM)

VST No.	Ground Elevation (ft)	Depth (ft)	Test Elevation (ft)	Measured		Keep Peak Value?	PI (%)	Bjerrum Correction Factor μ	Corrected Peak $S_u = \mu \times S_u(FV)$(psf)	Estimated σ_v' (psf)	S_u/σ_v'	Measured $S_{u\,res}/S_u$
				Peak S_u (FV) (psf)	Residual $S_{u\,res}$ (FV) (psf)							
G1	15.9	13.5	2.4	408	NA	Y	54	0.76	308	889	0.35	-
G2	20.9	18.5	2.4	408	NA	Y	54	0.76	308	1,082	0.28	-
G3A	43.4	18.5	24.9	340	40	Y	54	0.76	257	1,550	0.17	0.12
G3A	43.4	25	18.4	634	40	Y	54	0.76	479	1,801	0.27	-
G4	34.4	18.5	15.9	566	NA	Y	54	0.76	428	915	0.47	-
G4A	34.6	18.5	15.9	385	NA	Y	54	0.76	291	685	0.42	-
GP5	44	33.5	10.5	227	NA	Y	54	0.76	171	1,721	0.10	-
GP5A	41.4	18.5	22.9	272	NA	Y	54	0.76	205	1,039	0.20	-
GP6A	27.8	20.5	7.3	295	NA	Y	54	0.76	223	1,207	0.18	-
GP7	33.9	37.7	-3.8	861	NA	Y	54	0.76	650	1,360	0.48	-
GP7A	34	16.5	17.5	326	NA	Y	54	0.76	246	571	0.43	-
GP7A	34	26.5	7.5	521	NA	Y	54	0.76	394	2,581	0.15	-
GP8	34	33	1	462	102	Y	54	0.76	349	1,178	0.30	0.22
GP8	34	43	-9	553	91	Y	54	0.76	418	3,685	0.11	0.16
GP9	31.8	33	-1.2	417	NA	Y	41	0.82	342	1,187	0.29	-
GP9A	31.8	16.5	15.3	281	NA	Y	54	0.76	212	571	0.37	-
GP9A	31.8	26.5	5.3	521	NA	Y	54	0.76	394	936	0.42	-
A2	14.1	12	2.1	299	NA	Y	54	0.76	226	1,212	0.19	-
A2	14.1	27	-12.9	568	NA	Y	41	0.82	466	1,797	0.26	-
A2A	14.1	15.5	-1.4	1,394	NA	Y	54	0.76	1,053	-	-	-
A3	12.6	27.5	-14.9	1,593	NA	N	-	-	-	-	-	-
A4	23.6	24.5	-0.9	1,344	NA	N	-	-	-	-	-	-
SEA-12	25	11	14	470	50	Y	54	0.76	355	1,067	0.33	-
SEA-12	25	20	5	630	150	Y	54	0.76	476	1,940	0.25	0.24
SEA-12	25	25	0	860	250	Y	54	0.76	650	2,238	0.29	0.29
SEA-12	25	45	-20	990	140	Y	41	0.82	812	2,970	0.27	-
SEA-12	25	64	-39	2,250	1040	Y	41	0.82	1,846	3,703	0.50	0.46
SEA-14	50	20	30	840	140	Y	54	0.76	635	1,940	0.33	0.17
SEA-14	50	25	25	1,050	300	Y	54	0.76	793	2,425	0.33	0.29
SEA-14	50	35	15	1,160	240	Y	54	0.76	876	3,395	0.26	0.21
SEA-14	50	40	10	1,300	370	Y	54	0.76	982	3,880	0.25	0.28
SEA-14	50	45	5	1,280	200	Y	54	0.76	967	4,365	0.22	0.16
SEA-14	50	60	-10	1,480	120	Y	41	0.82	1,214	5,236	0.23	-
SEA-14	50	70	-20	1,770	400	Y	41	0.82	1,452	5,622	0.26	0.23
										Averages	0.29	0.24

Notes: Some peak values are reject due to 1) too close to sand, 2) interbedded sand layers, 3) too low (possibly, excessive disturbance), and 4) too high (possibly, sandy soil).

TABLE 6-2

HISTORIC CIL VST-BASED UNDRAINED SHEAR STRENGTH (DREDGE AND ALLUVIUM UNDER WASTE) (Continued)

VST	Ground Elevation	Depth	Test Elevation	Measured Peak S_u (FV)	Measured Residual $S_{u\,res}$ (FV)	Keep Peak Value?	PI	Correction Factor μ	Corrected Peak $S_u = \mu \times S_u$ (FV)	Measured $S_{u\,res}/S_u$
	(ft)	(ft)	(ft)	(psf)	(psf)		(%)		(psf)	
SEA-100	78	43.5	34.5	370	90	Y	54	0.76	280	0.24
		48.5	29.5	1,090	90	N	-	-	-	-
		59.5	18.5	400	160	Y	54	0.76	302	0.40
		64.5	13.5	780	140	Y	54	0.76	589	0.18
		70	8	820	230	Y	54	0.76	619	0.28
SEA-104	73	45	28	650	60	Y	54	0.76	491	0.09
		50.5	22.5	990	120	Y	54	0.76	748	0.12
		54.5	18.5	1,170	110	Y	54	0.76	884	0.09
		60.5	12.5	790	NA	Y	54	0.76	597	-
		70	3	1,100	90	Y	54	0.76	831	0.08
		75	-2	510	230	Y	54	0.76	385	0.45
									Average	0.22

TABLE 6-3

HISTORIC USACOE VST-BASED UNDRAINED SHEAR STRENGTH (DREDGE AND ALLUVIUM UNDER PERIMETER BERM)

VST No.	Ground Elevation (ft)	Depth (ft)	Test Elevation (ft)	Measured		Keep Peak Value?	PI (%)	Bjerrum Correction Factor μ	Corrected Peak $S_u = \mu \times S_u(FV)$ (psf)	Estimated σ_v' (psf)	S_u/σ_v'	Measured $S_{u\,res}/S_u$
				Peak S_u (FV) (psf)	Residual $S_{u\,res}$ (FV) (psf)							
EBB-27	47.2	10.2	37	1,018	212	Y	54	0.76	774	969	-	0.21
EBB-17	37.5	10.5	27	650	170	Y	54	0.76	494	998	-	0.26
		27.5	10	780	210	Y	54	0.76	593	1,864	0.42	0.27
		40.5	-3	NA	400	-	-	-	-	-	-	-
		60.5	-23	900	330	Y	41	0.82	684	2,940	0.31	0.37
EBB-28	47.1	16.1	31	404	130	Y	54	0.76	307	1,155	0.35	0.32
		27.1	20	643	155	Y	54	0.76	489	1,514	0.42	0.24
		36.1	11	668	195	Y	54	0.76	508	1,807	0.37	0.29
EBB-18	37.5	10.5	27	500	100	Y	54	0.76	380	1,310	0.38	0.20
		26	11.5	800	110	Y	54	0.76	608	1,815	0.44	0.14
EBB-29	49.5	10	39.5	749	155	N	-	-	-	-	-	0.21
		20	29.5	1,881	163	N	-	-	-	-	-	0.09
EBB-19	34	10	24	300	90	Y	54	0.76	228	1,075	0.28	0.30
		25	9	700	130	Y	54	0.76	532	1,564	0.45	0.19
EBB-30	46.5	32	14.5	1,002	350	Y	54	0.76	762	2,229	0.45	0.35
EBB-31	52.6	16.6	36	1,725	466	N	-	-	-	-	-	0.27
		28.6	24	1,377	130	N	-	-	-	-	-	0.09
		44.6	8	1,212	186	Y	54	0.76	921	2,552	0.47	0.15
EBB-32	48	16	32	886	140	N	-	-	-	-	-	0.16
		24	24	790	187	Y	54	0.76	600	1,781	0.44	0.24
		48	0	818	NA	Y	54	0.76	622	2,563	0.32	-
EBB-22	39	15	24	600	NA	Y	54	0.76	456	1,550	0.39	-
		30	9	500	NA	Y	54	0.76	380	2,039	0.25	-
EBB-25	47.6	13.6	34	692	130	Y	54	0.76	526	1,417	0.49	0.19
		29.6	18	945	179	Y	54	0.76	718	1,938	0.49	0.19
		39.6	8	1,385	326	N	-	-	-	-	-	0.24
EBB-21	32	10	22	200	175	Y	54	0.76	152	950	0.21	0.88
		25	7	210	200	Y	54	0.76	160	1,439	0.15	0.95
		40	-8	300	50	Y	54	0.76	228	1,928	0.16	0.17
EBB-26	47.8	10	37.8	668	155	N	-	-	-	-	-	0.23
		27.8	20	619	119	Y	54	0.76	470	1,474	0.42	0.19

TABLE 6-3

HISTORIC USACOE VST-BASED UNDRAINED SHEAR STRENGTH (DREDGE AND ALLUVIUM UNDER PERIMETER BERM) (Continued)

VST No.	Ground Elevation (ft)	Depth (ft)	Test Elevation (ft)	Measured		Keep Peak Value?	PI (%)	Bjerrum Correction Factor μ	Corrected Peak $S_u = \mu \times S_u(FV)$(psf)	Estimated σ_v' (psf)	Su/σ_v'	Measured $S_{u\ res}/S_u$
				Peak S_u (FV) (psf)	Residual $S_{u\ res}$ (FV) (psf)							
EBB-23	35	11	24	438	188	N	-	-	-	-	-	0.43
		16	19	306	57	Y	54	0.76	233	915	0.33	0.19
		21	14	578	123	Y	54	0.76	439	1,078	0.54	0.21
		31	4	171	NA	N	-	-	-	-	-	-
		36	-1	661	31	Y	54	0.76	502	1,567	0.42	0.05
		41	-6	551	105	Y	54	0.76	419	1,730	0.32	0.19
		46	-11	853	311	Y	54	0.76	648	1,893	0.45	0.36
		51	-16	840	451	Y	41	0.82	638	2,056	0.41	0.54
EBB-20	33	10	23	190	2	Y	54	0.76	144	1,137	0.17	-
		25	8	500	420	Y	54	0.76	380	1,626	0.31	0.84
		40	-7	500	450	Y	54	0.76	380	2,115	0.24	0.90
EBB-24	33	11	22	228	88	Y	54	0.76	173	758	0.30	0.39
		16	17	175	9	Y	54	0.76	133	921	0.19	-
		21	12	166	NA	N	-	-	-	-	-	-
		26	7	424	197	Y	54	0.76	322	1,247	0.34	0.46
		31	2	298	219	Y	54	0.76	226	1,410	0.21	0.73
		36	-3	153	26	N	-	-	-	-	-	0.17
		41	-8	455	61	Y	54	0.76	346	1,736	0.26	0.13
		46	-13	503	NA	Y	41	0.82	382	1,899	0.26	-
		51	-18	481	66	Y	41	0.82	366	2,062	0.23	0.14
EBB-16	33	10	23	220	50	Y	54	0.76	167	1,012	0.22	0.23
		26	7	400	180	Y	54	0.76	304	1,534	0.26	0.45
		42	-9	170	NA	N	-	-	-	-	-	-
										Averages	0.34	0.31

Notes: Some peak values are reject due to 1) too close to sand, 2) interbedded sand layers, 3) too low (possibly, excessive disturbance), and 4) too high (possibly, sandy soil)

TABLE 6-4

SUMMARY OF VST-BASED UNDRAINED SHEAR STRENGTH RESULTS (UNDER BERM)

Parameter	Database	S_u/σ_v' (1)	$S_{u\ res}/S_u$
Mean, μ	2002	0.31	0.33
	CIL	0.29	0.24
	USACE	0.34	0.31
	Weighted Average	**0.31**	**0.30**
Standard Deviation, σ	2002	0.12	0.24
	CIL	0.10	0.09
	USACE	0.11	0.22
	Weighted Average	**0.11**	**0.20**
COV, σ / μ × 100	2002	38	73
	CIL	36	39
	USACE	31	72
	Weighted Average	**34**	**66**
Points	2002	20	14
	CIL	31	12
	USACE	39	44
	Total	**90**	**70**

Note: (1) Based on corrected values.

TABLE 6-5

SUMMARY OF CPT N_k CALIBRATION

CPT		Boring							VST		DSS					
No.	q_t(1) (psf)	No.	Surf. Grnd Elev. (ft)	Test Depth (ft)	Test Elevation (ft)	Total Vert. Stress(2)(3) σ_v (psf)	Pore Pressure (psf)	Effective Vert. Stress σ'_v (psf)	S_u(4) (psf)	N_k	Test No.	σ'_c(5) (psf)	S_u (psf)	S_u/σ'_c	Correct S_u for actual σ'_v (psf)	N_k
C069	20,267	S-19	45.5	36	9.5	3,300 (3,395)(6)	374 (967)	2,926 (2,428)	907 (753)	18.7 (22.4)	4	2,500	921	0.37	1,078 (894)	15.7 (18.9)
												5,000	1,454	0.29	851 (706)	19.9 (23.9)
C071	17,200	S-18	45.5	47	-1.5	4,400 (4,495)	1,061 (1,654)	3,339 (2,841)	12.4 (14.4)	12.4 (14.4)	10	2,500	755	0.30	1,008 (858)	12.7 (14.8)
											11	5,000	1,498	0.30	1,000 (851)	12.8 (14.9)
									Average	**15.5 (18.4)**					Average(7)	**14.6 (16.9)**

Notes:
(1) q_t is the corrected cone tip resistance.
(2) Depth to dredge water was assumed to be 30 ft (elevation 15.5 ft) at the time of CPT testing based on piezometric data; however, a depth up to 20 ft (elevation 25 ft) was considered.
(3) Unsaturated and saturated unit weights of dredge/alluvium are assumed as γ_{uns} = 90 pcf and γ_{sat} - 100 pcf.
(4) $Su/\sigma_v' = 0.31$ is assumed for the VST-based shear strengths.
(5) σ_c' is the effective consolidation pressure during DSS test.
(6) Values in parenthesis for lower estimate of depth to dredge water.
(7) A weighted average based on the difference between the actual σ'_v and σ_c' was obtained.

TABLE 6-6

EVALUATION OF CPT-BASED UNDRAINED SHEAR STRENGTH INCREASE WITH TIME

Case	Figure	Location	Baseline Test (1)	Comparisons			
				October 2001		February 2002	
				Test	Average CPT-q_t increase (1)	Test	Average CPT-q_t increase (1)
1	6-35	Berm	SEA-10	NA	NA	C-2	No increase
2	6-36		SEA-14	NA	NA	C-24	No increase
3	6-37		SEA-17	NA	NA	C-42	0 to 10 %
4	6-37		SEA-17	NA	NA	C-43	10 to 20 %
5	6-38	Waste	SEA-8	NA	NA	C-55	Large
6	6-39		SEA-7	SEA-104	5 to 50 %	C-56	5 to 50 %
7	6-40		SEA-3	SEA-100	5 to 25 %	C-58	0 to 50 %

Notes: (1) Baseline CPTs were conducted in December 1998.
(2) Increase is in the dredge, i.e., within the area with the lowest initial q_t down to an approximate elevation of 0.

TABLE 6-7

SUMMARY OF C_V VALUES FROM SETTLEMENT PLATE DATA

Phase	Settlement Plate	C_v(ft^2/day)
I	SP-111	0.065 0282
I	SP-1BA	0.013 0.312
I	SP-3	0:761
I	SP-5	0.204 0.423
I	SP-6	0.232
I	SP-7	0.140
II	SP-2	0.194
II	SP-5	0.153
II	SP-7	0.510
II	SP-10	0.412
III	SP-8	0.908

TABLE 7-1

SUMMARY OF UNIT WEIGHTS FOR EXISTING WASTE IN CIL

Existing Phase	Area	Bulk Unit Weight of Dry/Moist Waste (pcf)	Saturated Unit Weight of Waste (pcf)
I	Asbestos disposal areas. Other areas, excluding areas with clinker disposal. Areas with clinker disposal.	30 60 – 85 120	95 (average)(1)
II	ALL	40 – 75	75 – 100
III	ALL	45 – 70	75 – 100
IV	Top lift near Phase II/III boundary. Other areas and depths.	30 40 – 75	75 – 110
V	ALL	45 – 75	85 – 95

Note: (1) Recommended values for Phase I are less reliable than for other phases.

TABLE 7-2

RECOMMENDED UNIT WEIGHT VALUES TO BE USED FOR FUTURE WASTE PLACEMENT IN CIL

Depth of Waste (ft)	Moist Unit Weight (pcf)
0 – 20	60
20 - 100	75
100 – 150	80
150 – 200	85
> 200	90

Note: Saturated Unit Weight = 100 pcf.

TABLE 7-3

SUMMARY OF WASTE SHEAR STRENGTH

Test ID	Confining Stress (psf)	Shear Rate (in/min)	Avg. Friction Angle (°)
1A 1B	800 2,000	0.3	36.0
2A 2B	80 2,000	0.04	34.0
3B	2,000	0.006	37.4

TABLE 8-1

SUMMARY OF SOIL AND WASTE PARAMETERS FOR DESIGN

Parameter	Symbol	Units	Deposit	Phase(s)	Number of Points	Mean	Standard Deviation	Min	Max
Basic									
Dry Unit Weight	δ_D	pcf	Dredge	1,3,4,5	35	56	7	45.3	73
			Alluvium	1,3,4,5	4	64.2	1.9	62	66.5
Moist Unit Weight	δ_M	pcf	Dredge	1,3,4,5	33	97	7	85.3	108
			Alluvium	1,3,4,5	4	101	3	97.4	104
Natural Water Content	w_n	%	Dredge	1,3,4,5	31	74	15	43.3	99.2
			Alluvium	1,3,4,5	7	58	8	44.7	67.7
			Potomac	1,3,4,5	10	24	8	18.4	40
			All	1	9	43	31	11	86
			All	3	19	62	22	19	85
			All	4	9	70	21	34	92
Organic Content	OC	%	All	1,3,4,5	35	9	3	4	14
			Dredge	1,3,4,5	30	10	2	6	14
			Alluvium	1,3,4,5	3	7	4	4	12
			-	1	8	9	3	4	14
			-	3	11	10	3	5	13
			-	4	8	8	2	6	12
			-	5	8	9.5	3	6	14
Void Ratio	e_o	-	Dredge	1,2,3,4	10	2.3	0.22	2.1	2.8
Saturated Unit Weight Existing Waste	γ_{sat}	pcf		1		95			
				2				75	100
				3				75	100
				4				75	110
				5				85	95

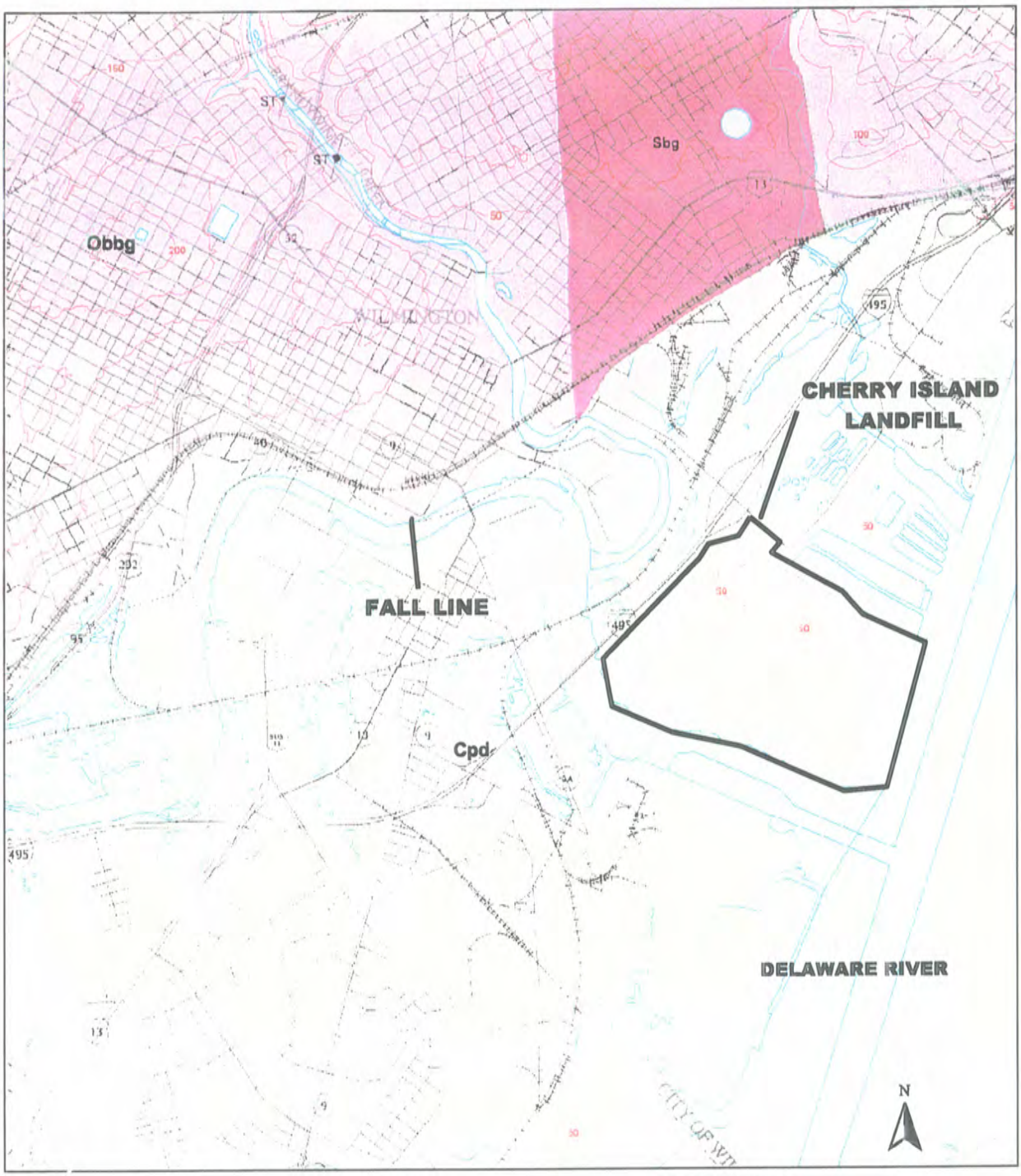
TABLE 8-1
SUMMARY OF SOIL AND WASTE PARAMETERS FOR DESIGN
(Continued)

Parameter	Symbol	Units	Deposit	Phase(s)	Number of Points	Mean	Standard Deviation	Min	Max
Plasticity									
Plastic Limit	PL	-	Dredge	1,3,4,5	32	44	6	32	55
		-	Alluvium	1,3,4,5	8	35	10	15	47
		-	Potomac	1,3,4,5	10	20	5	13	27
		-	Dredge	1,3,4,5	32	98	19	51	126
		-	Alluvium	1,3,4,5	8	72	22	24	96
		-	Potomac	1,3,4,5	10	39	15	27	68
		-	Dredge	1,3,4,5	32	54	14	19	73
		-	Alluvium	1,3,4,5	8	40	9	23	49
		-	Potomac	1,3,4,5	10	19	12	10	42
		-	All	1	10	30	12	15	52
		-	All	3	18	37	13	13	55
		-	All	4	9	43	7.5	26	53
		-	All	5	17	38	10	18	48
Liquid Limit	LL	%	All	5	16	59	27	18	99
		-	All	1	10	61	35	24	123
		-	All	3	18	84	32	27	124
		-	All	4	9	97	18	68	126
		-	All	5	17	77	28	28	108
Plastic Index	PI	-	All	1	9	33	23	9	71
		-	All	3	18	47	19	10	71
		-	All	4	9	54	12	38	73
		-	All	5	17	39	18	10	62
Liquidity Index	LI	-	Dredge	1,3,4,5	31	0.62	0.3	0.02	1.13
		-	Alluvium	1,3,4,5	7	0.5	0.2	0.16	0.72
		-	Potomac	1,3,4,5	10	0.15	0.17	0.03	0.42

TABLE 8-1 (Continued)

SUMMARY OF SOIL AND WASTE PARAMETERS FOR DESIGN

Parameter	Symbol	Units	Deposit	Phase(s)	Number of Points	Mean	Standard Deviation	Min	Max
Strength									
Normalized Shear Strength	$S_u(FV)/s'_v$		All	1,3,4,5		0.31	0.12		
Consolidation									
Compression Index	C_c		All	1,3,4,5	28	0.8	0.2	0.6	1.1
			All	1	4	0.78	0.16	0.59	0.96
			All	3	13	0.81	0.17	0.53	1.1
			All	4	5	0.85	0.086	0.79	0.99
			All	5	6	0.88	0.16	0.69	1.1
Secondary Compression Index	$C_{\alpha c}$	%	All	1,3,4,5	26	1.6	0.5	0.63	2.7
		%	All	1	4	1.7	0.7	0.75	2.45
		%	All	3	11	1.9	0.54	1	2.67
		%	All	4	5	1.7	0.08	1.6	1.8
		%	All	5	6	1.3	0.56	0.63	1.95



1000 0 1000 Feet

LEGEND

COASTAL PLAIN PROVINCE

Cpd - Coastal Plain Sediments

Holocene Deposits
Columbia Formation - Pleistocene Sediments
Potomac Formation - Cretaceous Sediments

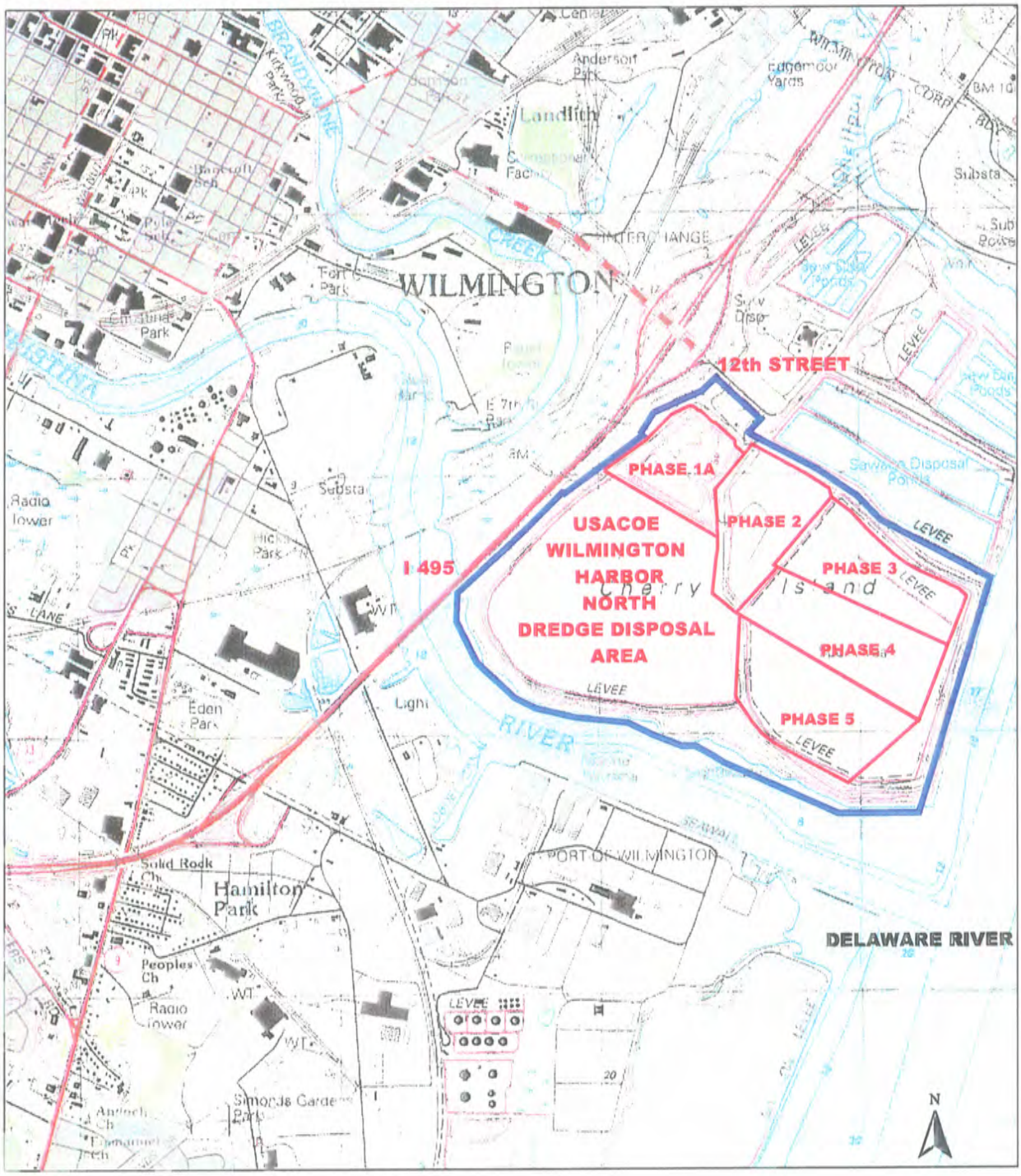
PIEDMONT PROVINCE

Sbg - Bringhurst Gabbro, Wilmington Complex, Silurian

Obbg - Brandywine Blue Gneiss, Wilmington Complex, Cambro-Ordivician

SOURCE: "Bedrock Geologic Map of the Piedmont of Delaware and Adjacent Pennsylvania" Schenck, et al. 2000

GEOSYNTEC CONSULTANTS 10015 Old Columbia Road, Suite A-200 Columbia, Maryland 21046	GENERALIZED GEOLOGIC MAP CHERRY ISLAND LANDFILL WILMINGTON, DELAWARE	FIGURE 2-1 PROJECT - ME0250



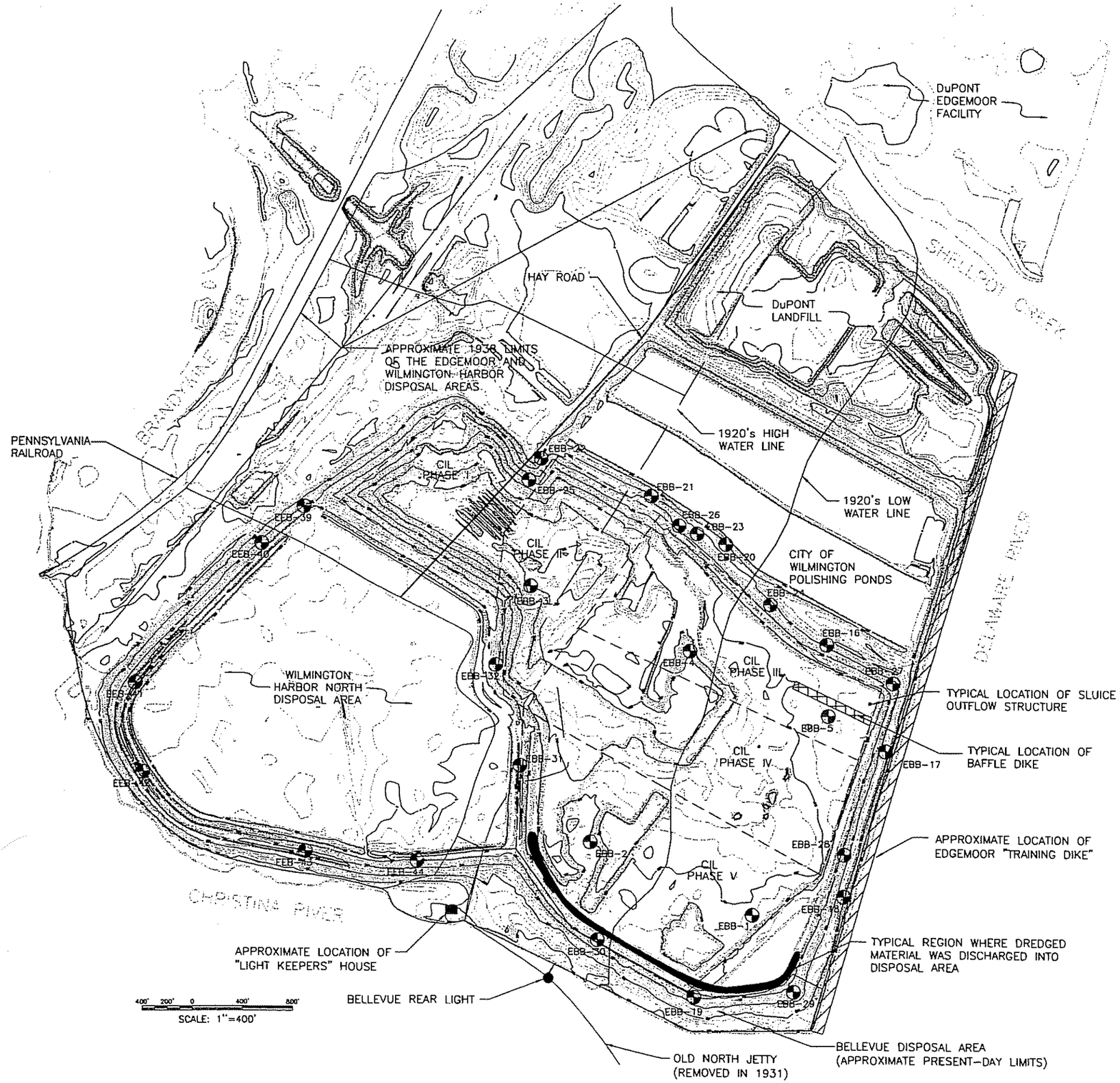
2000 0 2000 Feet

LEGEND

PHASE BOUNDARY
PROPERTY BOUNDARY

SOURCE: USGS 7.5-Minute Topographic Map
Wilmington, DE-NJ Quadrangle
1993

GEOSYNTEC CONSULTANTS 10015 Old Columbia Road, Suite A-200 Columbia, Maryland 21046	GENERAL SITE PLAN CHERRY ISLAND LANDFILL WILMINGTON, DELAWARE	FIGURE 2-2 PROJECT- ME0250



ESTIMATED DELAWARE STATE PLANE COORDINATES (NAD 83)

BORING	NORTHING	EASTING
EBB-1	627068	629331
EBB-2	627640	628036
EBB-3	629668	627556
EBB-4	629158	628833
EBB-5	628643	629936
EBB-16	629208	629926
EBB-17	628365	630396
EBB-18	627208	630061
EBB-19	626413	628866
EBB-20	630008	629118
EBB-21	630388	628523
EBB-22	630683	627643
EBB-23	630089	628890
EBB-24	629528	629475
EBB-25	630513	627541
EBB-26	630153	628746
EBB-27	628903	630456
EBB-28	627543	630066
EBB-29	626458	629661
EBB-30	626683	628091
EBB-31	628243	627471
EBB-32	629403	627281
EEB-39	630302	625753
EEB-40	630005	625416
EEB-41	628897	624407
EEB-42	628189	624466
EEB-43	627560	625763
EEB-44	627488	626656

NORTH

LEGEND

- CHERRY ISLAND LANDFILL PHASE BOUNDARIES
- ORIGINAL USACE DISPOSAL AREA BOUNDARIES
- APPROXIMATE CURRENT BELLEVUE DISPOSAL AREA BOUNDARIES
- EDGEMOOR TRAINING DIKE
- BAFFLE DIKE

SOURCE: DUFFIELD ASSOCIATES, 2002.

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COLUMBIA, MARYLAND

PROJECT: LANDFILL EXPANSION PROJECT CHERRY ISLAND LANDFILL		
TITLE: LOCATION PLAN EDGEMOOR AND WILMINGTON HARBOR DREDGED MATERIAL DISPOSAL AREAS		
SCALE:	NTS	SHEET NO: 2-3
PROJ. NO.:	ME0250	
DATE:		
FILE NO.:	0250F219	

CIL Photo.grf

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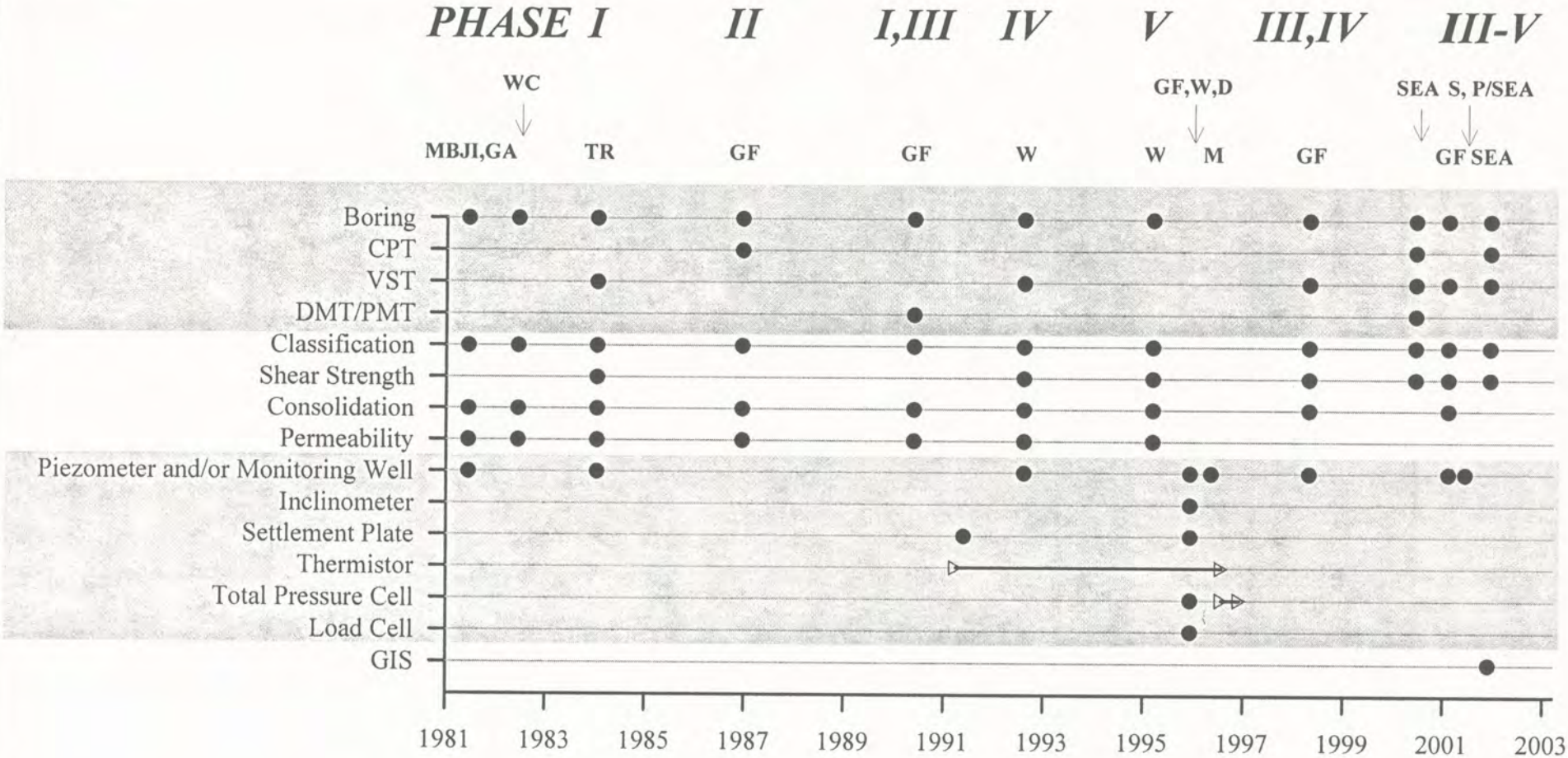
Columbia, Maryland 21046

Historic Photograph of Cherry Island

FIGURE 2-4

PROJECT ME0250
Cherry Island



D: Duffield Associates
GA: Gilbert Associates
GF: Gannett Fleming
MBJI: M. Baker, Jr. Inc.
SAIC: Science Applications Intrnt. Corp.
SEA: Schnabel Engineering Associates
TR: Terraqua Resources
W: Weston
WC: Woodward Clyde

Timeline3.grf

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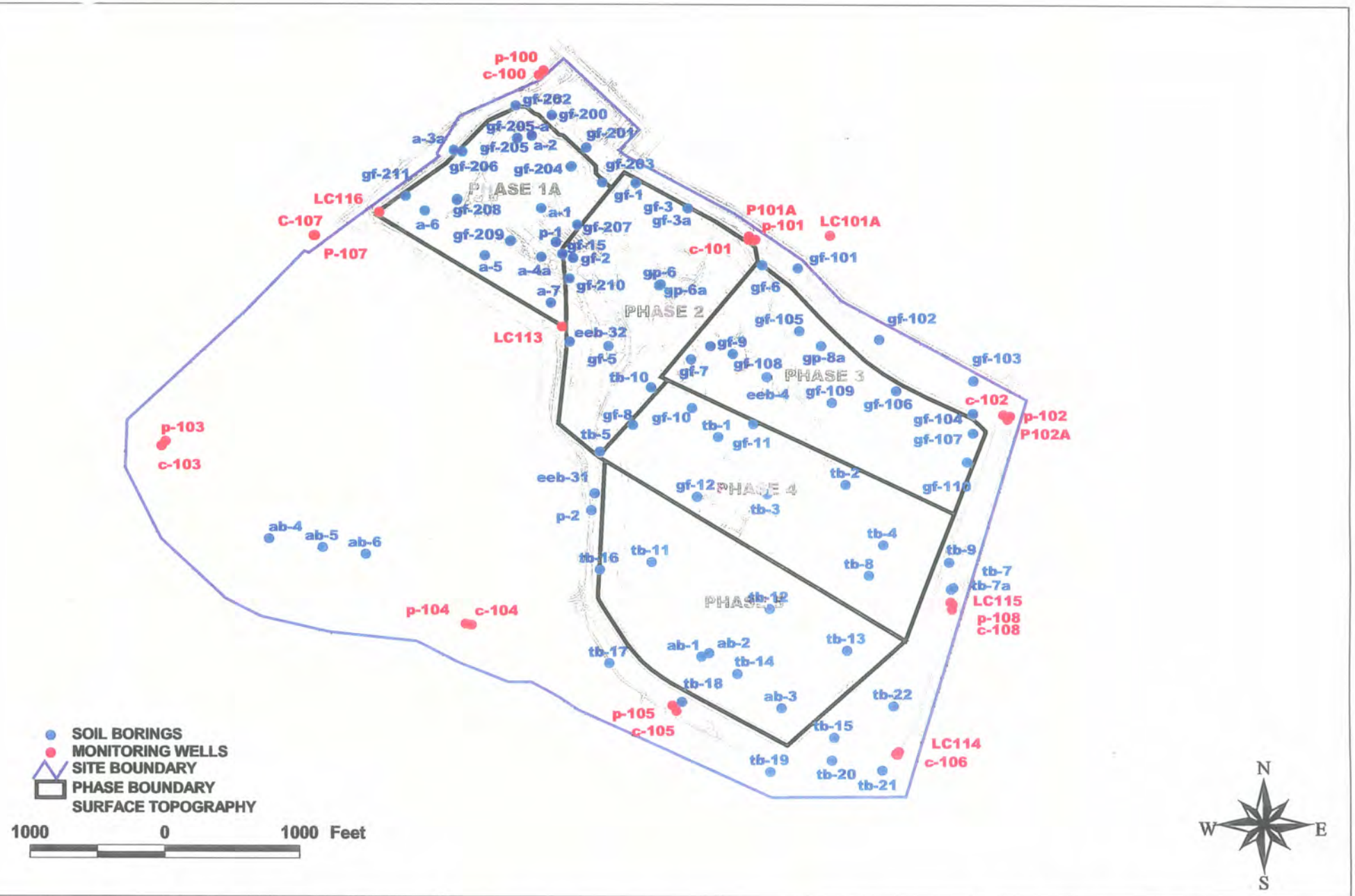
Columbia, Maryland 21046

Timeline of Hydrogeologic and Geotechnical Studies

FIGURE 3-1

PROJECT: ME0250 Cherry Island



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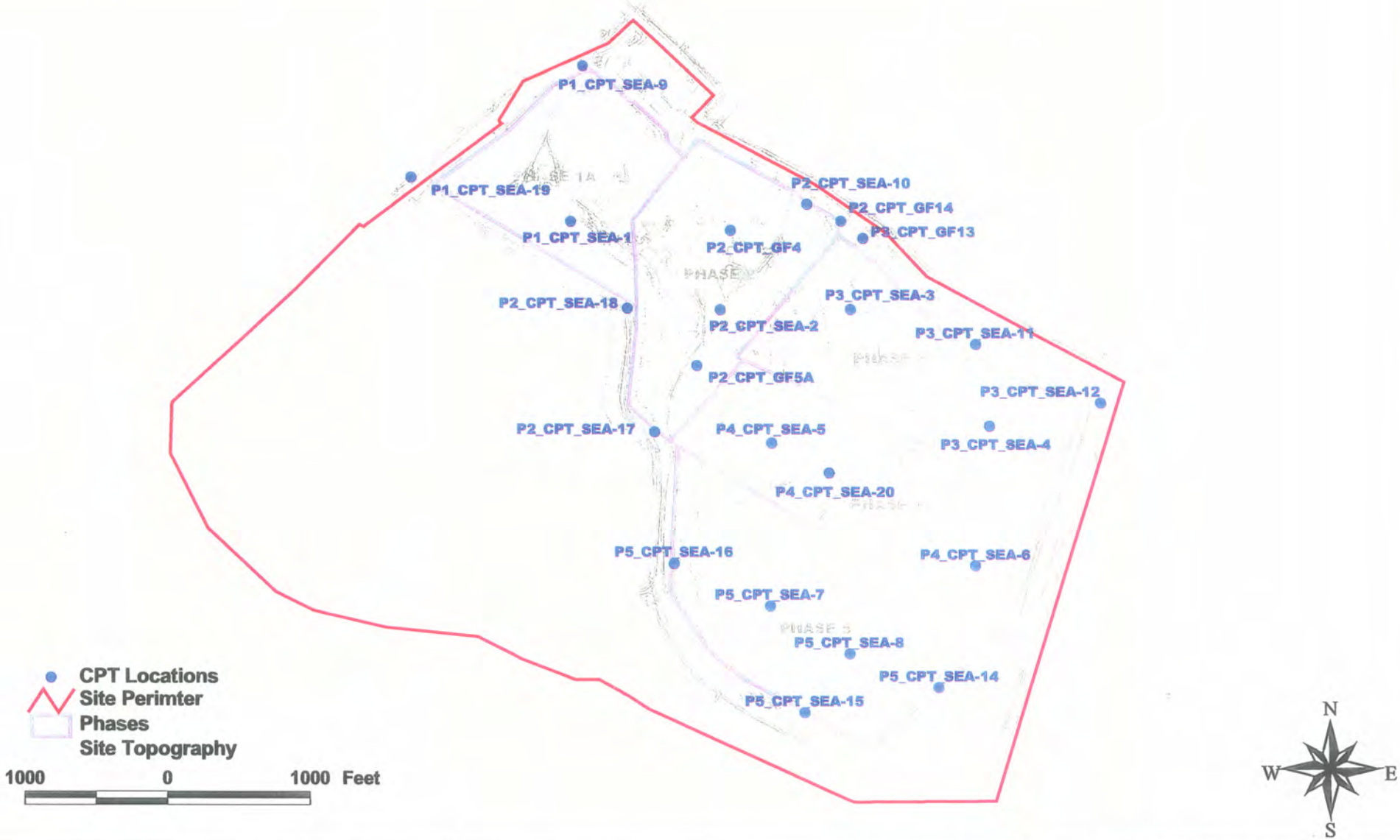
PLAN VIEW OF HISTORIC SOIL BORINGS AND MONITORING WELLS

FIGURE 3-2

PROJECT: ME0250



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PLAN VIEW OF HISTORIC CONE PENETROMETER TESTS

FIGURE 3-3

PROJECT: ME0250

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PLAN VIEW OF HISTORIC VANE SHEAR TESTS

FIGURE 3-4

PROJECT: ME0250

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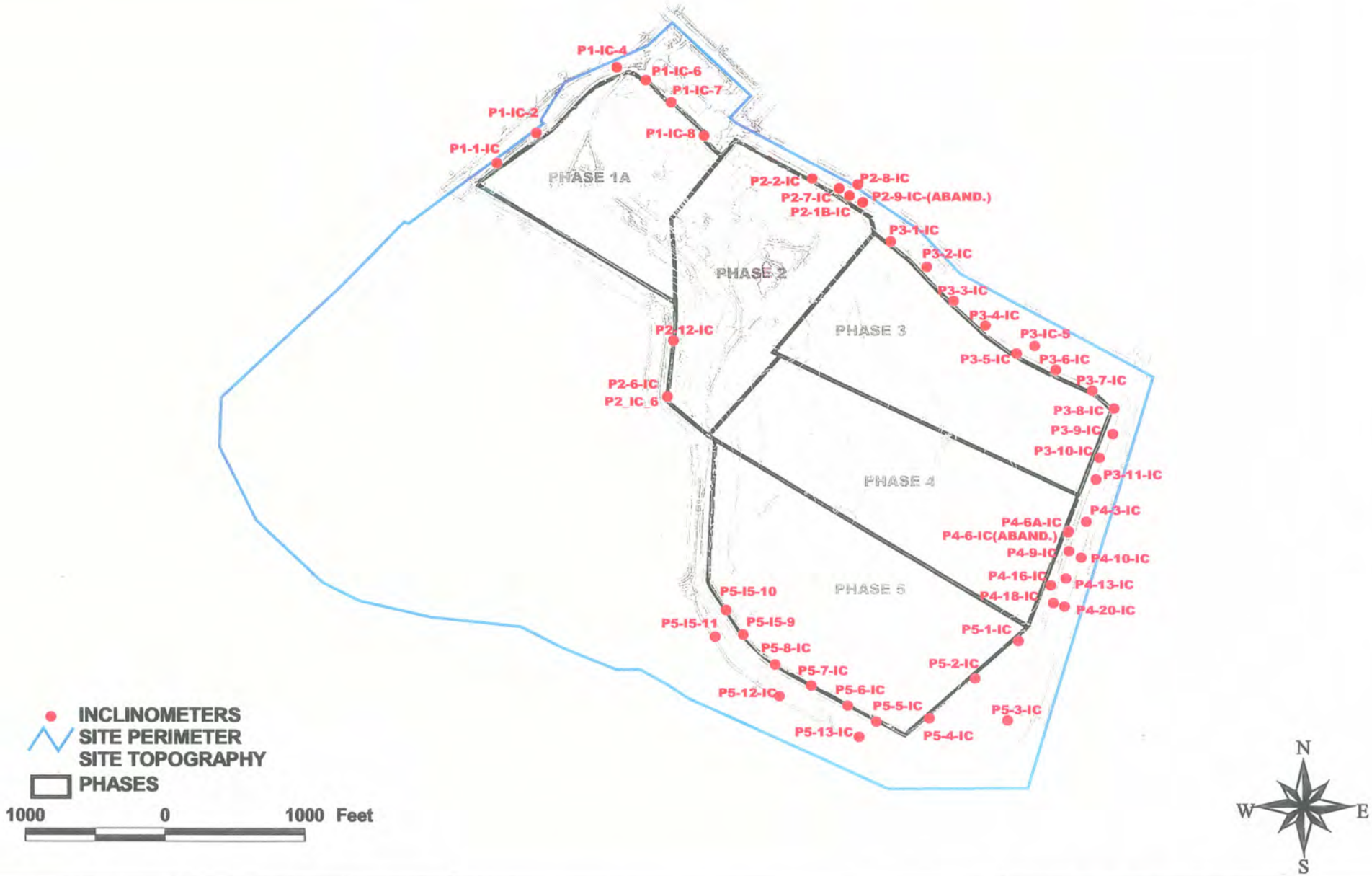
Columbia, Maryland 21046

PLAN VIEW OF HISTORIC DILATOMETER AND PRESSUREMETER TESTS

FIGURE 3-5

PROJECT: ME0250



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PLAN VIEW OF HISTORIC INCLINOMETERS

FIGURE 3-6

PROJECT: ME0250

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PLAN VIEW OF HISTORIC SETTLEMENT PLATES

FIGURE 3-7

PROJECT: ME0250

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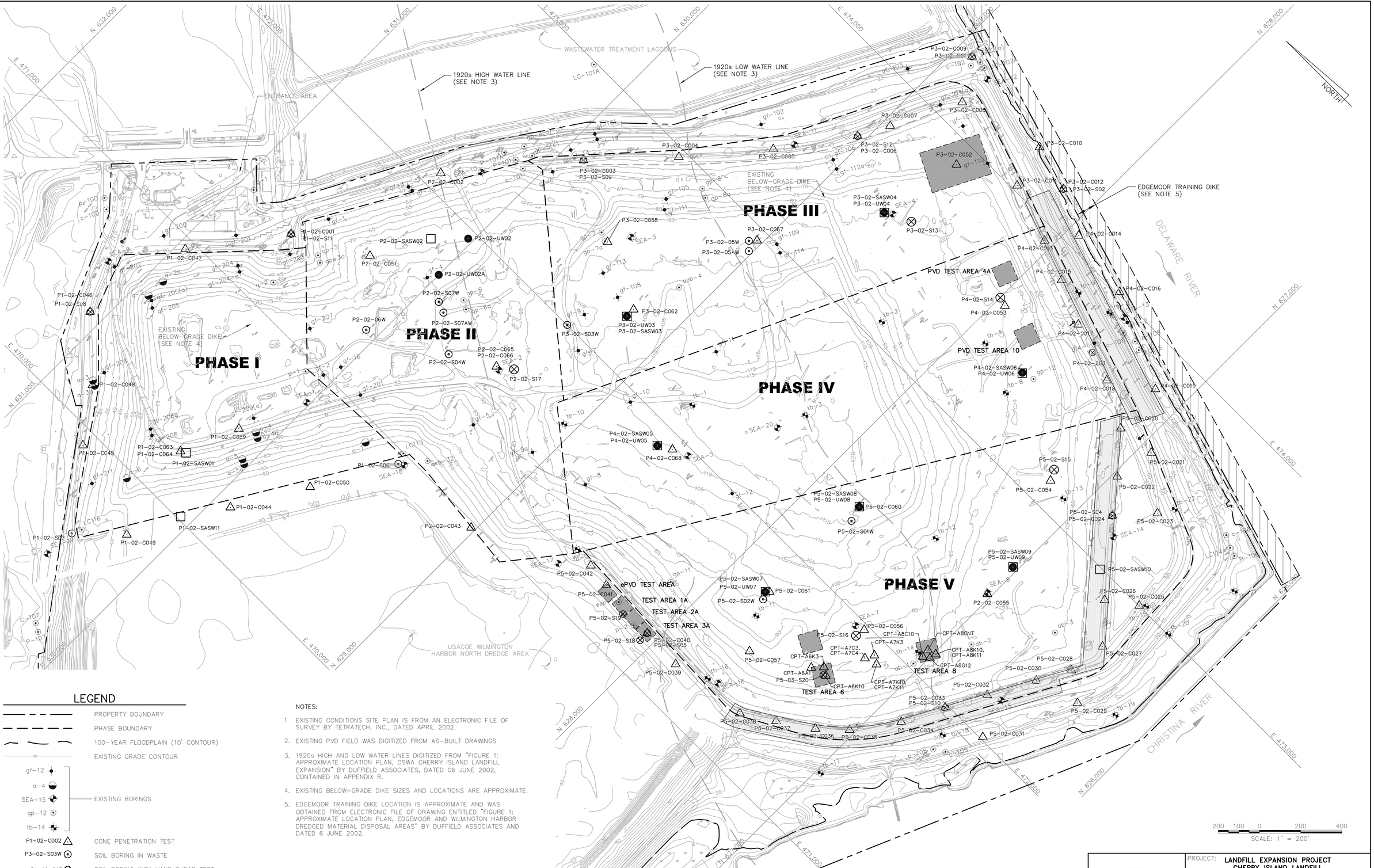
Columbia, Maryland 21046

PLAN VIEW OF HISTORIC THERMISTORS

FIGURE 3-8

PROJECT: ME0250



LEGEND

- PROPERTY BOUNDARY
- PHASE BOUNDARY
- 100-YEAR FLOODPLAIN (10' CONTOUR)
- EXISTING GRADE CONTOUR
- gf-12, a-4, SEA-15, gp-12, tb-14 — EXISTING BORINGS
- P1-02-C002 — CONE PENETRATION TEST
- P3-02-S03W — SOIL BORING IN WASTE
- P1-02-S07 — SOIL BORING WITH VANE SHEAR TEST
- P4_02_SASW06 — SASW SURVEY
- P3_02_UW04_NEW — BUCKET AUGER (IN WASTE)
- EXISTING PVD AREA (SEE NOTE 2)
- PVD TEST AREA

NOTES:

1. EXISTING CONDITIONS SITE PLAN IS FROM AN ELECTRONIC FILE OF SURVEY BY TETRATECH, INC., DATED APRIL 2002.
2. EXISTING PVD FIELD WAS DIGITIZED FROM AS-BUILT DRAWINGS.
3. 1920s HIGH AND LOW WATER LINES DIGITIZED FROM "FIGURE 1: APPROXIMATE LOCATION PLAN, DSWA CHERRY ISLAND LANDFILL EXPANSION" BY DUFFIELD ASSOCIATES, DATED 06 JUNE 2002, CONTAINED IN APPENDIX R.
4. EXISTING BELOW-GRADE DIKE SIZES AND LOCATIONS ARE APPROXIMATE.
5. EDGEMOOR TRAINING DIKE LOCATION IS APPROXIMATE AND WAS OBTAINED FROM ELECTRONIC FILE OF DRAWING ENTITLED "FIGURE 1: APPROXIMATE LOCATION PLAN, EDGEMOOR AND WILMINGTON HARBOR DREDGED MATERIAL DISPOSAL AREAS" BY DUFFIELD ASSOCIATES AND DATED 6 JUNE 2002.

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COLUMBIA, MARYLAND

PROJECT: LANDFILL EXPANSION PROJECT CHERRY ISLAND LANDFILL		
TITLE: FIELD EXPLORATION PLAN		
SCALE:	1" = 200'	SHEET NO:
PROJ. NO.:	ME0250	3–9
DATE:	JAN 2003	
FILE NO.:	0250F705	

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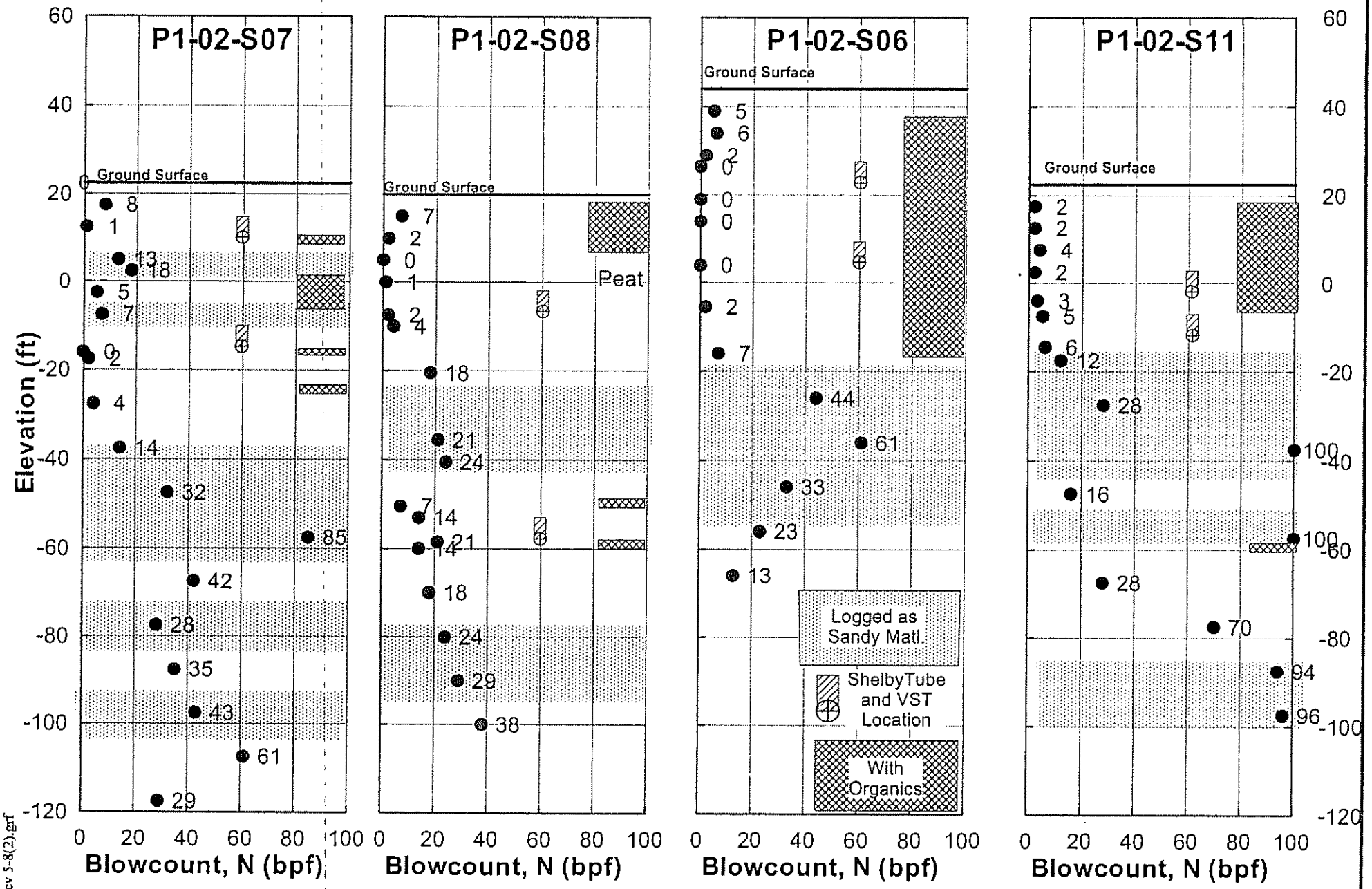

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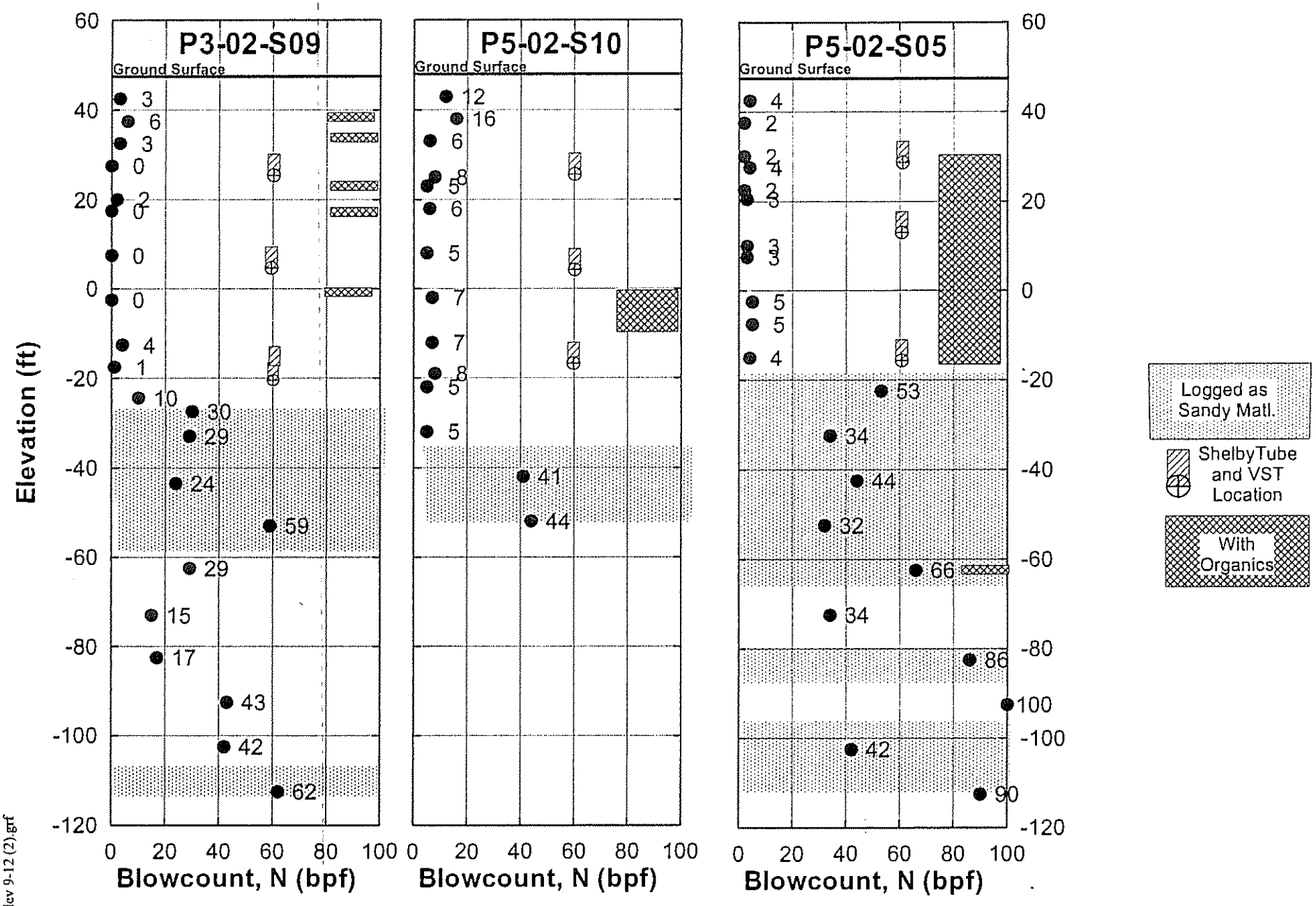
SPT-based stratigraphy along dike Phase I

FIGURE 4-16

PROJECT: ME0250 Cherry Island

Blowcount vs. Elev 5-8(2).grf



Blowcount vs. Elev 9-12 (2).grf

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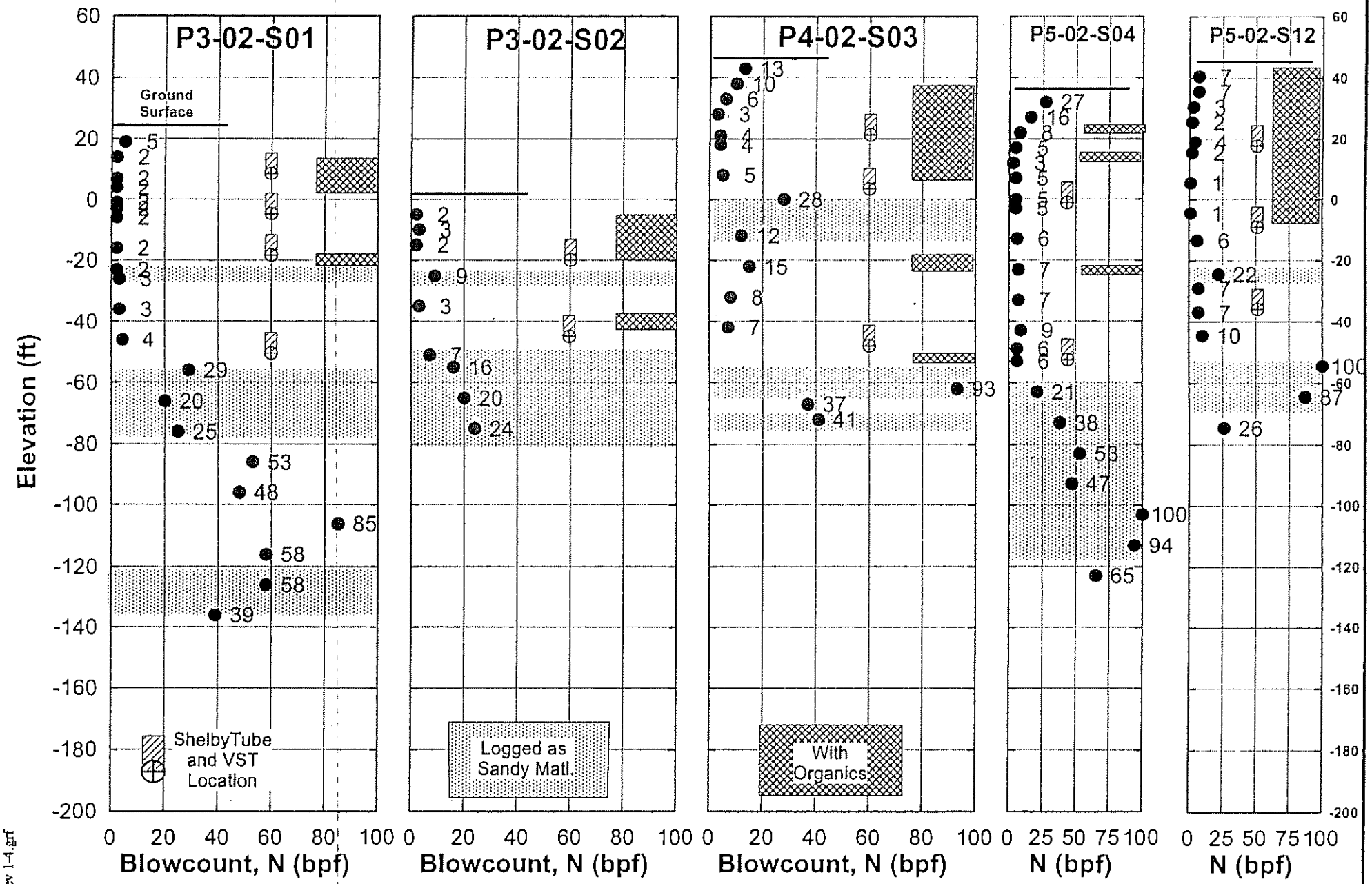
SPT-based stratigraphy along dike Phases III - V (west boundary)

FIGURE 4-17

PROJECT: ME0250 Cherry Island



Blowcount vs. Elev 1-4.grf

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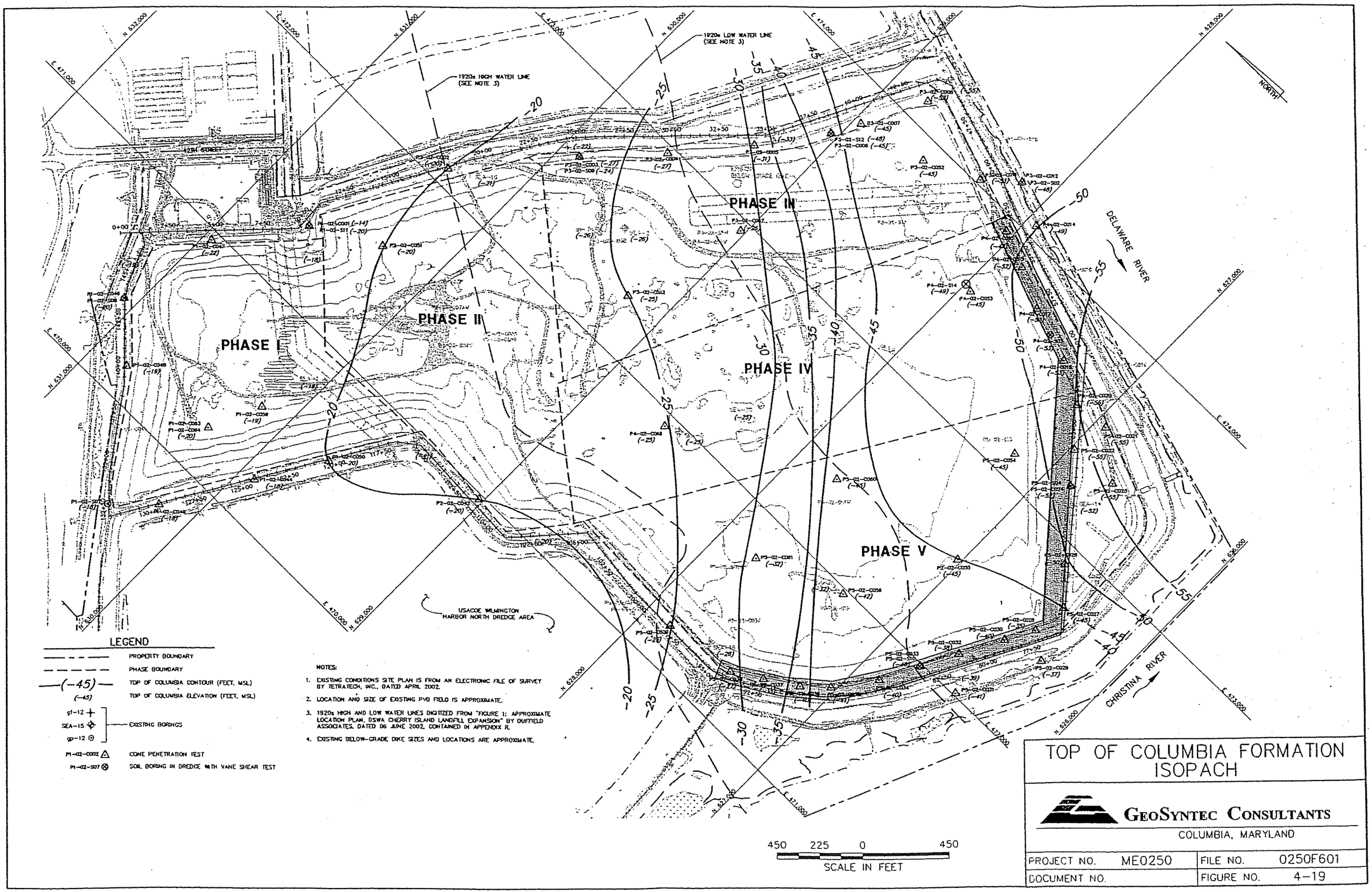
SPT-based stratigraphy along dike
Phases III - V (east boundary)

FIGURE 4-18

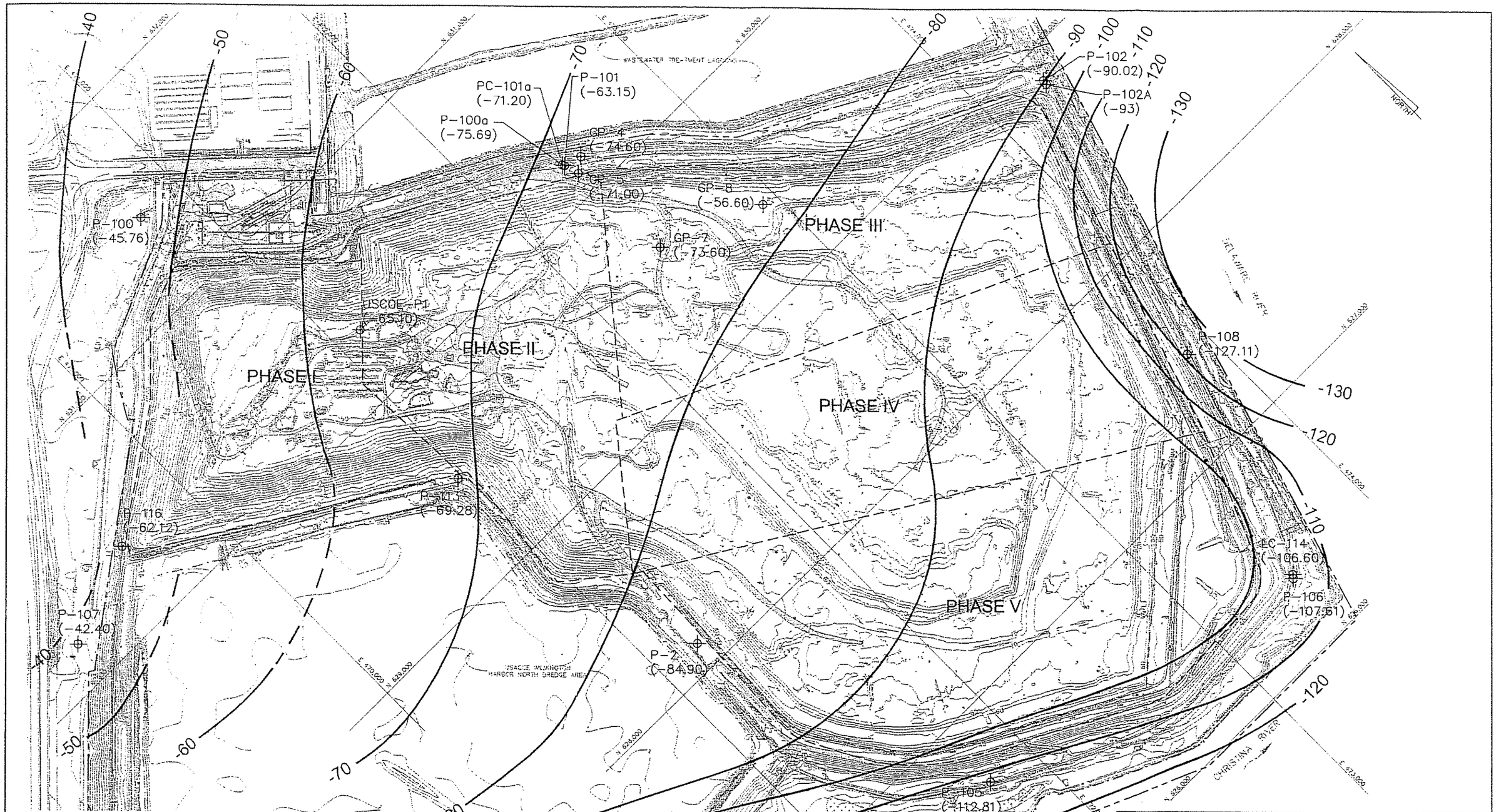
PROJECT: ME0250 Cherry Island



TOP OF COLUMBIA FORMATION ISOPACH

GEOSYNTEC CONSULTANTS
COLUMBIA, MARYLAND

PROJECT NO.	ME0250	FILE NO.	0250F601
DOCUMENT NO.		FIGURE NO.	4-19



LEGEND

— — — —	PROPERTY BOUNDARY
— — — —	PHASE BOUNDARY
—110—	TOP OF POTOMAC FORMATION CONTOUR (FT-MSL) (SEE NOTES 2 AND 3)
—40—	EXISTING GRADE CONTOUR (FT-MSL)
P-101 (-63.15)	EXISTING BORING ADVANCED TO BEDROCK (ELEVATION OF TOP OF POTOMAC (FT. MSL))

NOTES:

1. EXISTING CONDITIONS SITE PLAN IS FROM AN ELECTRONIC FILE OF SURVEY BY TETRATECH, INC., DATED APRIL 2002.
2. THE POTOMAC FORMATION IS COMPOSED OF INTERBEDDED LAYERS OF CLAYEY SILT AND SAND.
3. THE ELEVATION CONTOURS SHOWN CORRESPOND TO THE CLAY IN THE POTOMAC FORMATION USUALLY DESCRIBED AS A REDDISH OR ORANGE, VERY STIFF TO HARD, CLAYEY SILT TO SILTY CLAY. OTHER CLAYEY MATERIALS THAT MAY BE PART OF THE POTOMAC FORMATION AT HIGHER ELEVATIONS BUT WITH DIFFERENT DESCRIPTIONS THAN THOSE PRESENTED ABOVE ARE NOT SHOWN.

TOP OF POTOMAC FORMATION ("CLAY") CONTOUR MAP

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COLUMBIA, MARYLAND

PROJECT NO.	ME0250	FILE NO.	0250F1004
DOCUMENT NO.		FIGURE NO.	4-20

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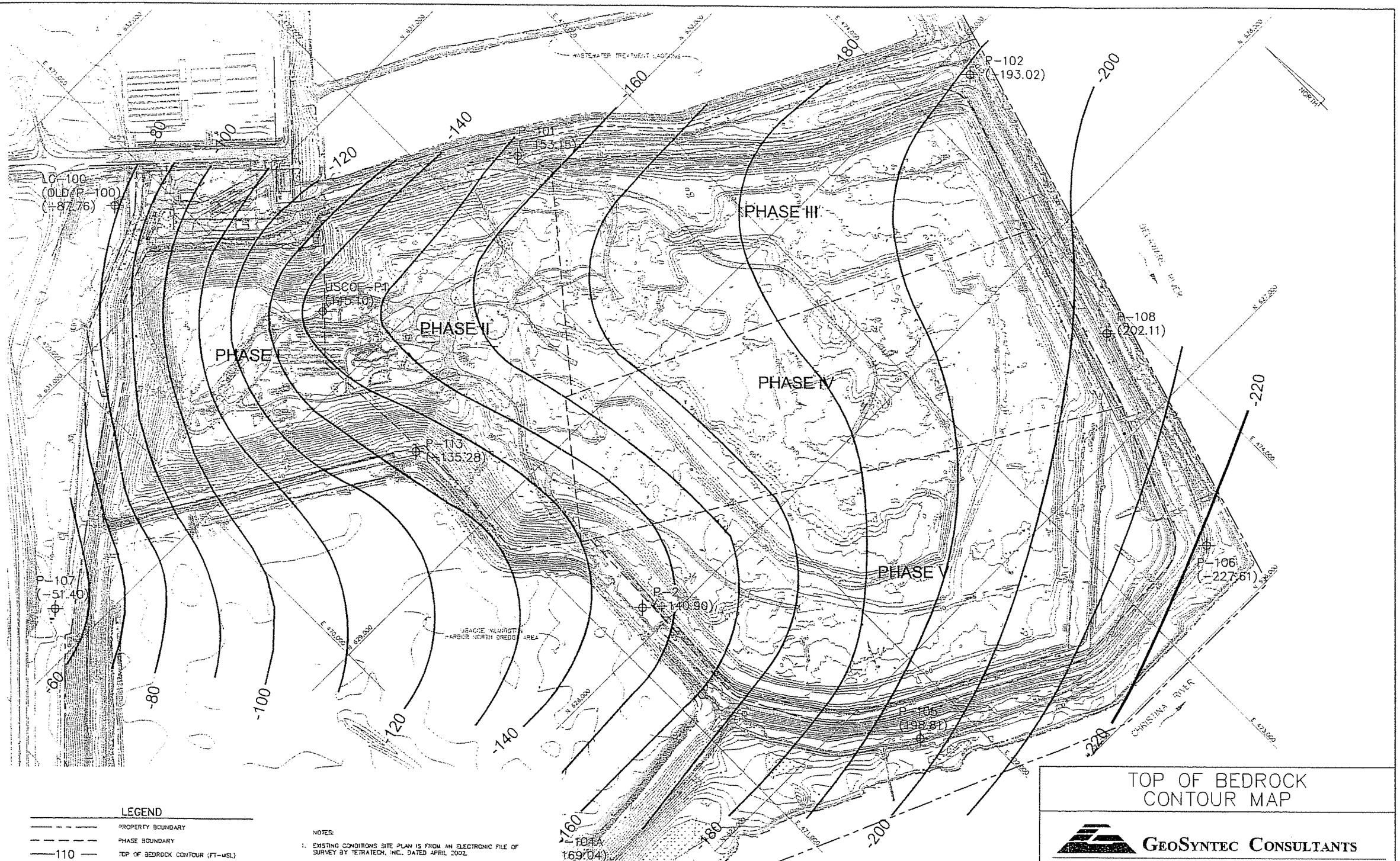


LEGEND

— — — —	PROPERTY BOUNDARY
— — — —	PHASE BOUNDARY
—110—	TOP OF BEDROCK CONTOUR (FT-MSL)
—80—	EXISTING GRADE CONTOUR (FT-MSL)
P-101 (-193.02)	EXISTING BORING ADVANCED TO BEDROCK ELEVATION OF TOP OF BEDROCK (FT. MSL)

NOTES:

1. EXISTING CONDITIONS SITE PLAN IS FROM AN ELECTRONIC FILE OF SURVEY BY TETRATECH, INC. DATED APRIL 2002.

450 225 0 450

SCALE: 1" = 450'

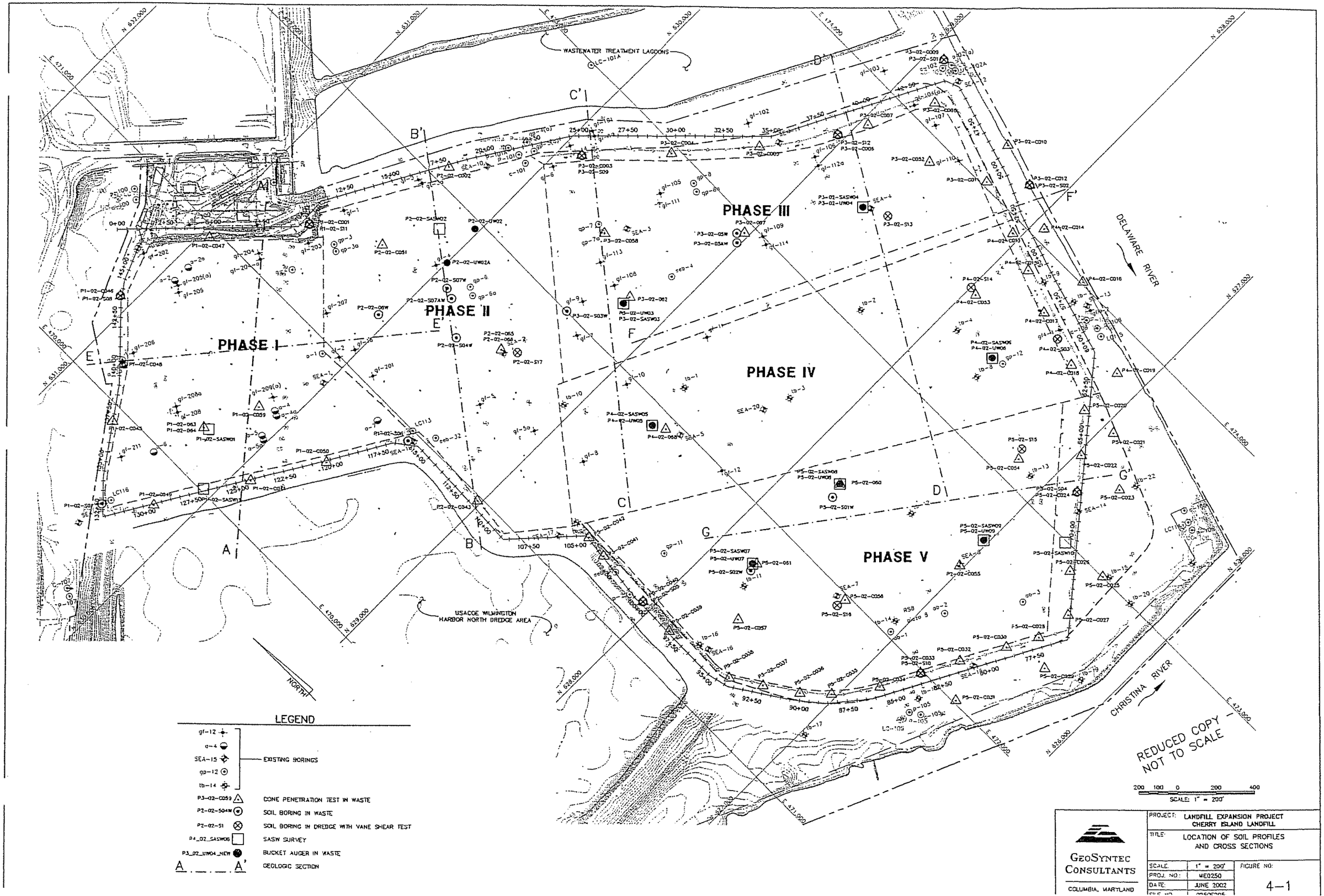
TOP OF BEDROCK CONTOUR MAP

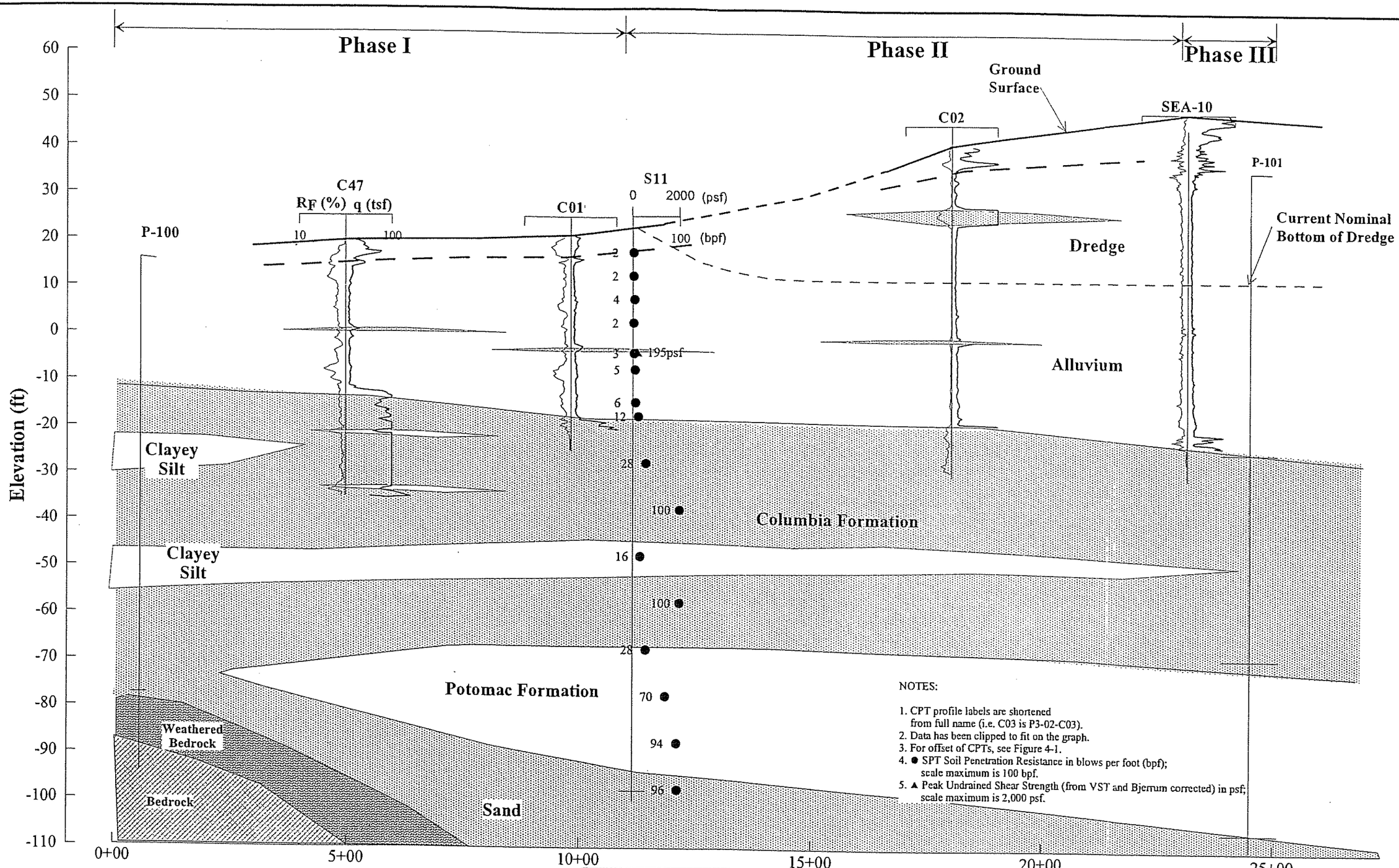
GEOSYNTEC CONSULTANTS

COLUMBIA, MARYLAND

PROJECT NO.	ME0250	FILE NO.	C250F1003
DOCUMENT NO.		FIGURE NO.	4-21

P:\cadd\0250\0250100\figures\0250F1003.DWG, Layout1, 9/4/2003 6:13:22 PM, geosyntec consultants, inc. (jpc)


WASTEWATER TREATMENT LAGOONS
PHASE I
PHASE II
PHASE III
PHASE IV
PHASE V
DELAWARE RIVER
CHRISTINA RIVER
USACOE WILMINGTON HARBOR NORTH DREDGE AREA
NORTH
REDUCED COPY NOT TO SCALE
SCALE: 1" = 200'
LEGEND
EXISTING BORINGS
CONE PENETRATION TEST IN WASTE
SOIL BORING IN WASTE
SOIL BORING IN DREDGE WITH VANE SHEAR TEST
SASW SURVEY
BUCKET AUGER IN WASTE
GEOLOGIC SECTION
GEOSYNTEC CONSULTANTS
COLUMBIA, MARYLAND
PROJECT: LANDFILL EXPANSION PROJECT CHERRY ISLAND LANDFILL
TITLE: LOCATION OF SOIL PROFILES AND CROSS SECTIONS
SCALE: 1" = 200'
PROJ. NO.: ME0250
DATE: JUNE 2002
FIGURE NO: 4-1



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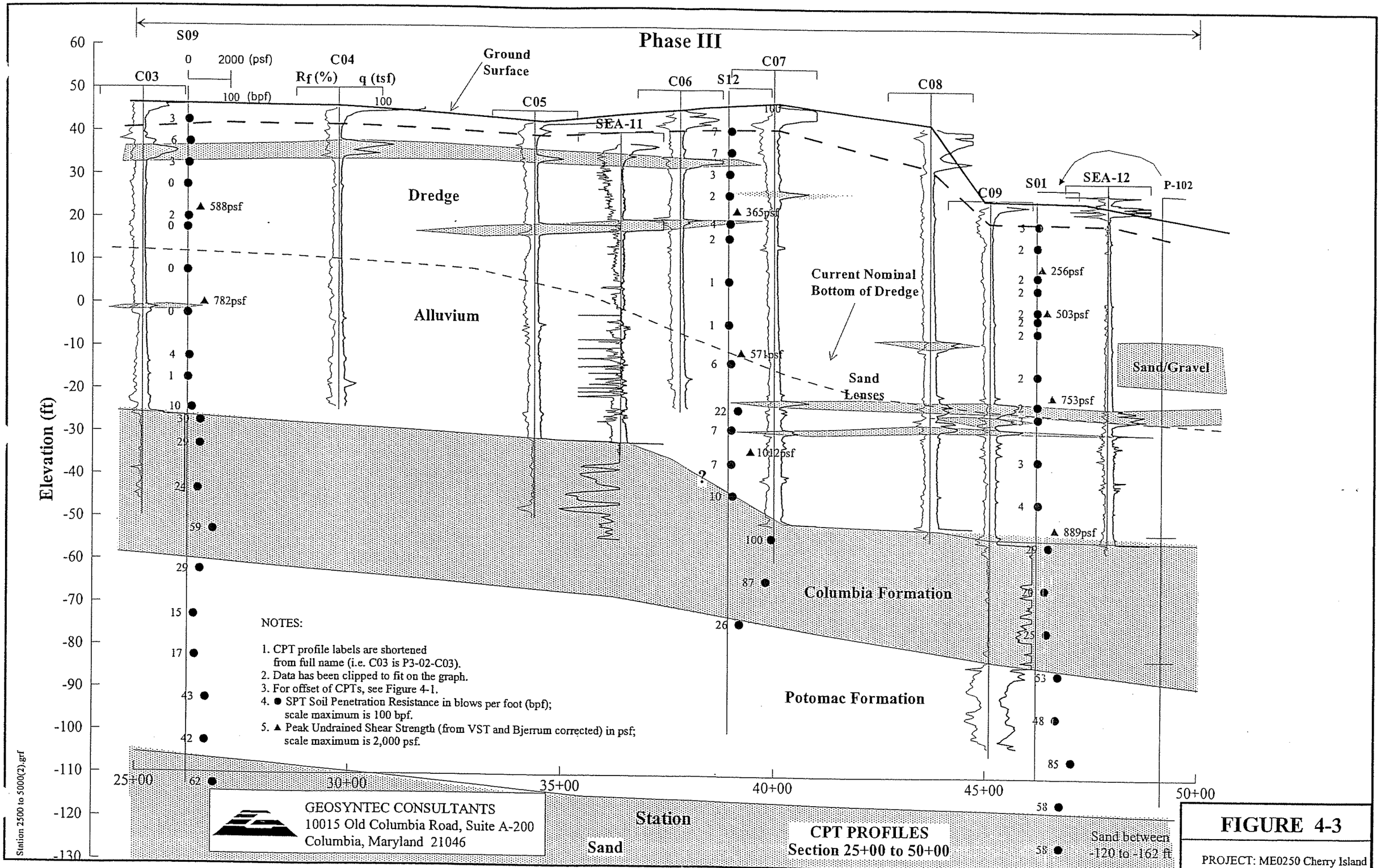
CPT PROFILES
Station 0 to 25+00

FIGURE 4-2

PROJECT: ME0250 Cherry Island

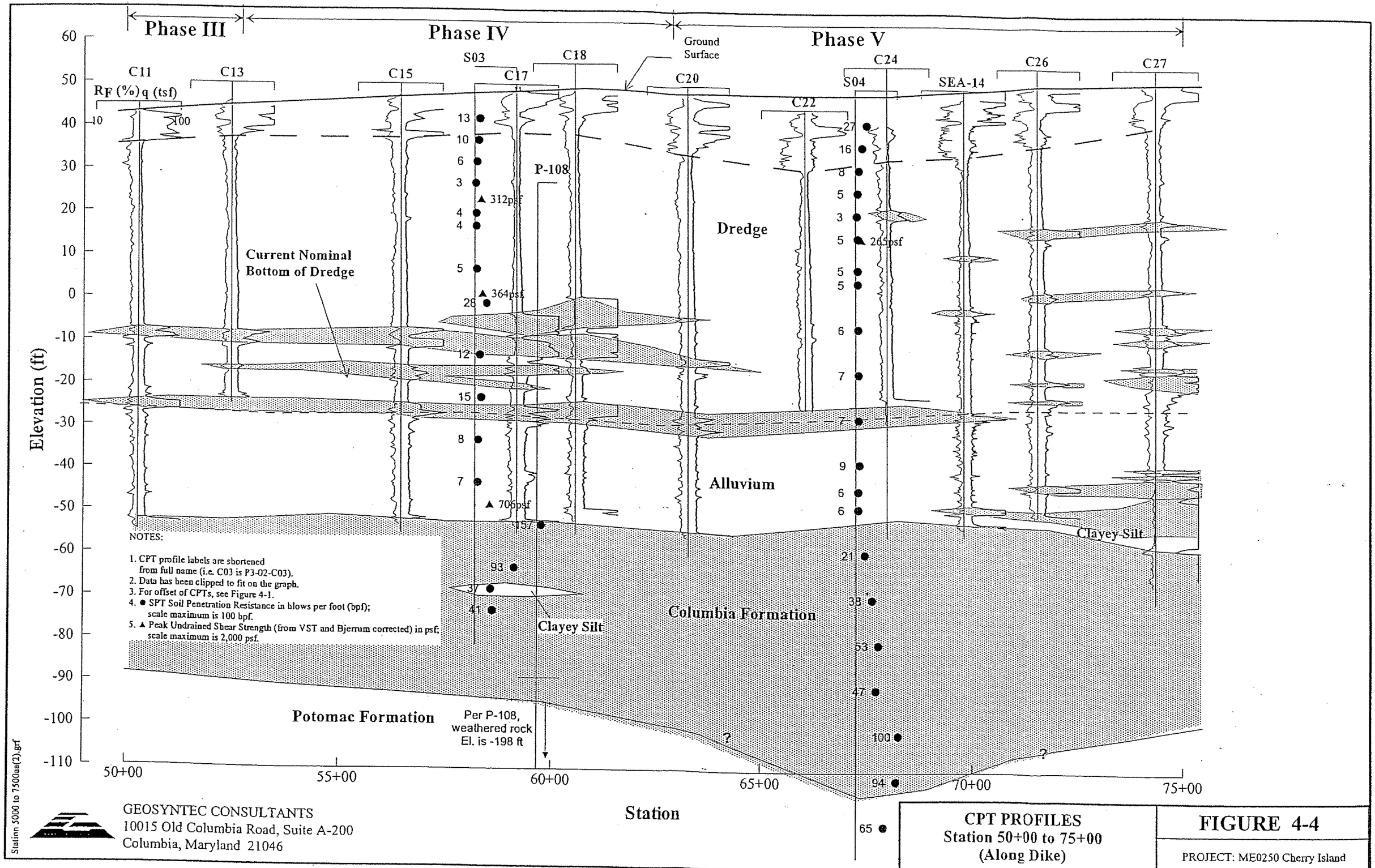
Station 0 to ...(2).grf



CPT PROFILES
Section 25+00 to 50+00

FIGURE 4-3

PROJECT: ME0250 Cherry Island



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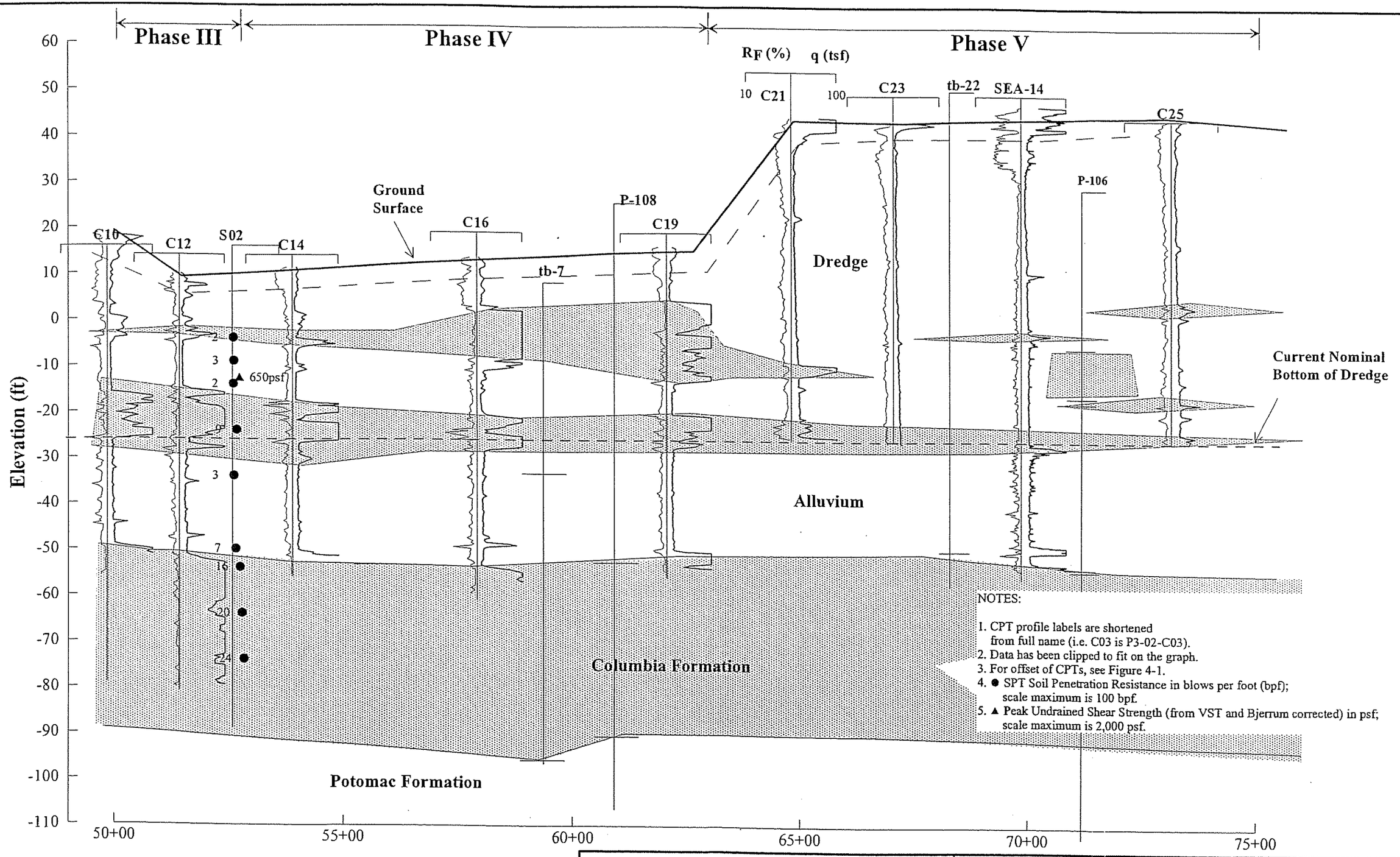
Columbia, Maryland 21046

CPT PROFILES
Station 50+00 to 75+00
(Along Dike)

FIGURE 4-4

PROJECT: ME0250 Cherry Island



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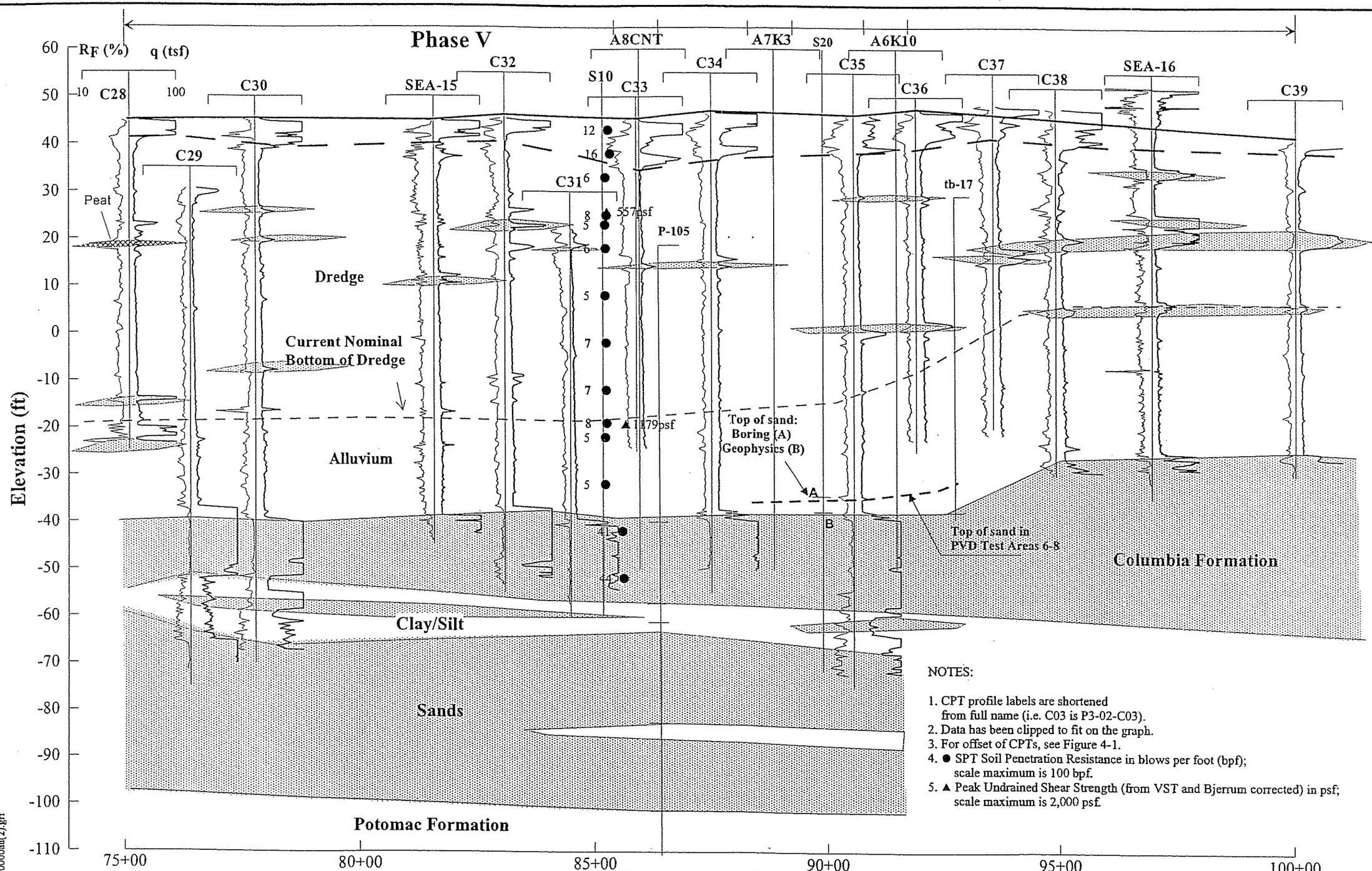
Columbia, Maryland 21046

CPT PROFILES
Station 50+00 to 75+00
(Along Toe)

FIGURE 4-5

PROJECT: ME0250 Cherry Island

Station 5000 to 7500bb(2).grf



NOTES:

1. CPT profile labels are shortened from full name (i.e. C03 is P3-02-C03).
2. Data has been clipped to fit on the graph.
3. For offset of CPTs, see Figure 4-1.
4. ● SPT Soil Penetration Resistance in blows per foot (bpf); scale maximum is 100 bpf.
5. ▲ Peak Undrained Shear Strength (from VST and Bjerrum corrected) in psf; scale maximum is 2,000 psf.

Station 7500 to 10000an(2).grf

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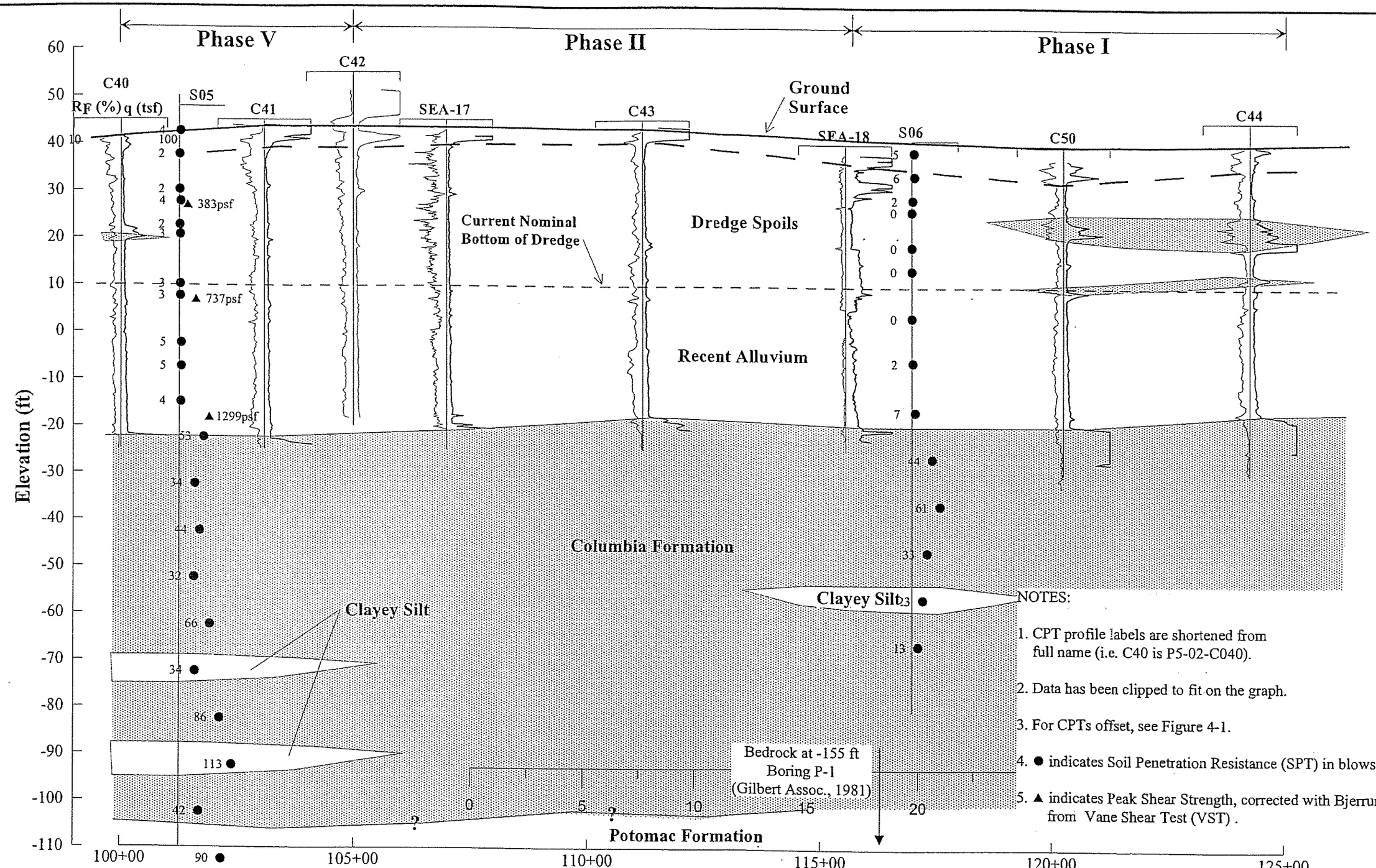
Columbia, Maryland 21046

CPT PROFILES
Station 75+00 to 100+00

FIGURE 4-6

PROJECT: ME0250 Cherry Island



NOTES:

1. CPT profile labels are shortened from full name (i.e. C40 is P5-02-C040).
2. Data has been clipped to fit on the graph.
3. For CPTs offset, see Figure 4-1.
4. ● indicates Soil Penetration Resistance (SPT) in blows per foot.
5. ▲ indicates Peak Shear Strength, corrected with Bjerrum method, from Vane Shear Test (VST).

Station 10000 to 12500aa(2).grf

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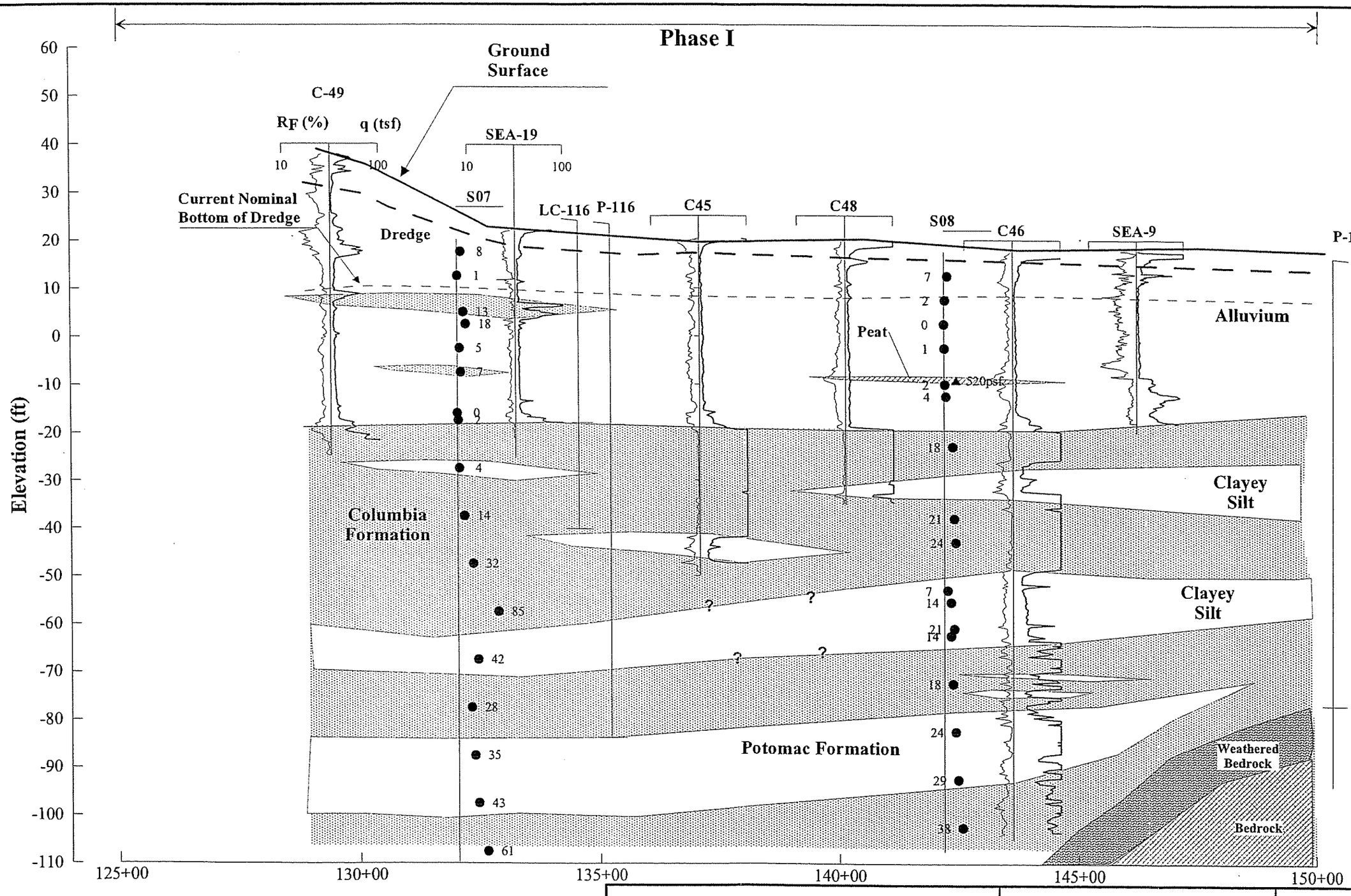
Columbia, Maryland 21046

CPT PROFILES
Station 100+00 to 125+00

FIGURE 4-7

PROJECT: ME0250 Cherry Island



Station 12500 to 15000(2).grf

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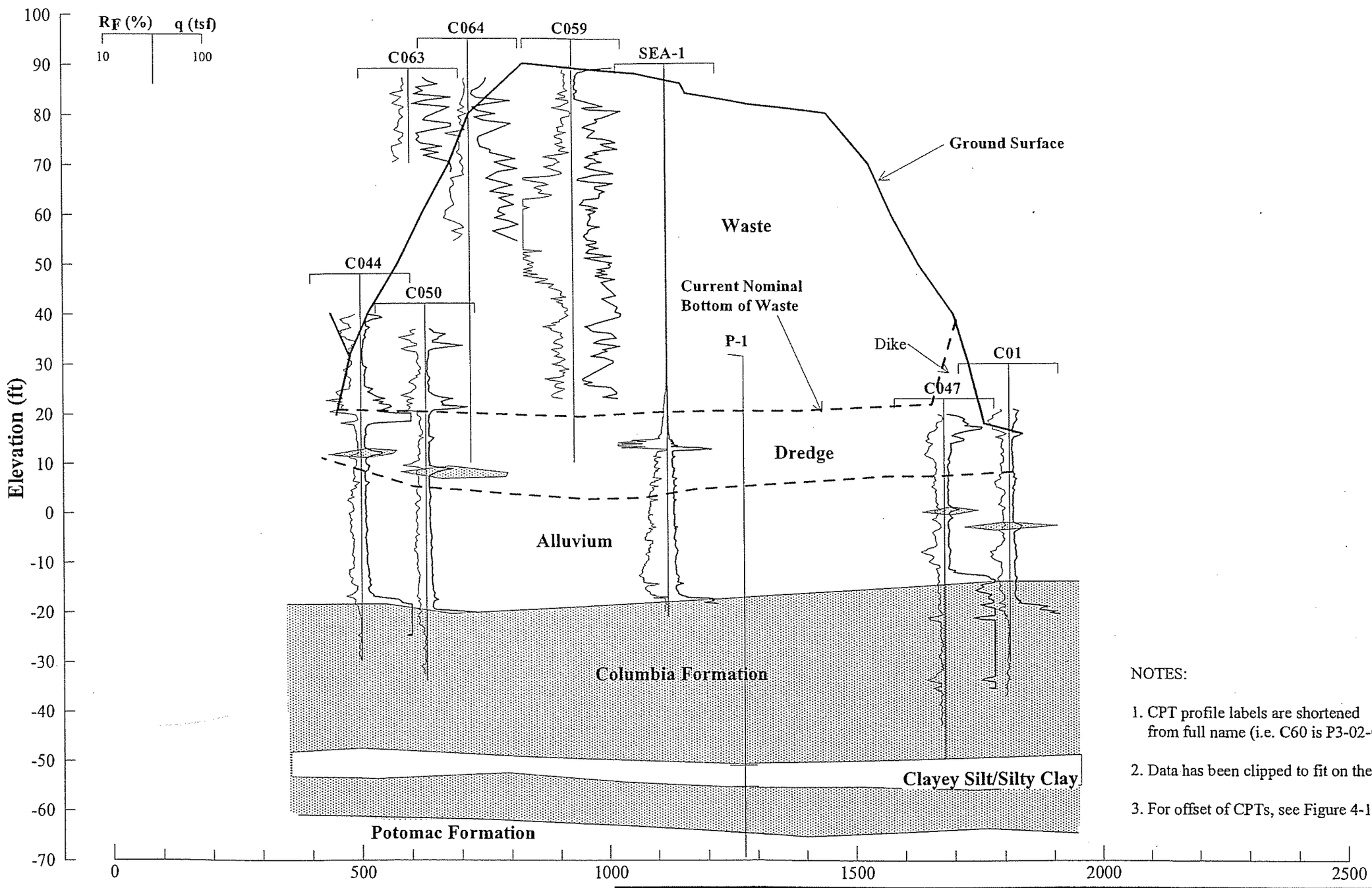
Columbia, Maryland 21046

CPT PROFILES
Station 125+00 to 150+00

FIGURE 4-8

PROJECT: ME0250 Cherry Island



NOTES:

1. CPT profile labels are shortened from full name (i.e. C60 is P3-02-C60).
2. Data has been clipped to fit on the graph.
3. For offset of CPTs, see Figure 4-1.

Section AA.grf

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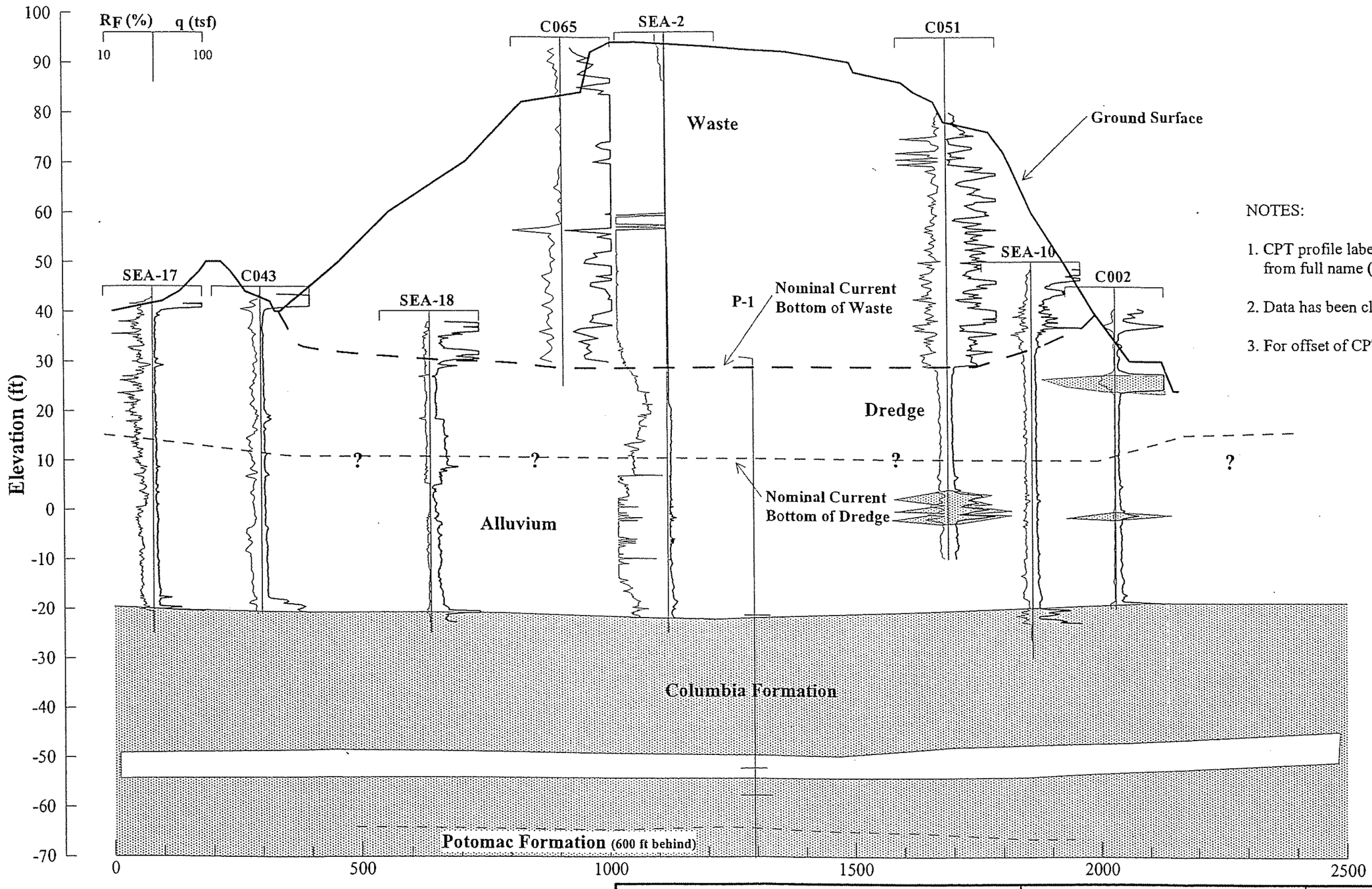
CPT PROFILES
Section A-A

FIGURE 4-9

PROJECT: ME0250 Cherry Island



NOTES:

1. CPT profile labels are shortened from full name (i.e. C060 is P3-02-C060).
2. Data has been clipped to fit on the graph.
3. For offset of CPTs, see Figure 4-1.

Section BB.grf

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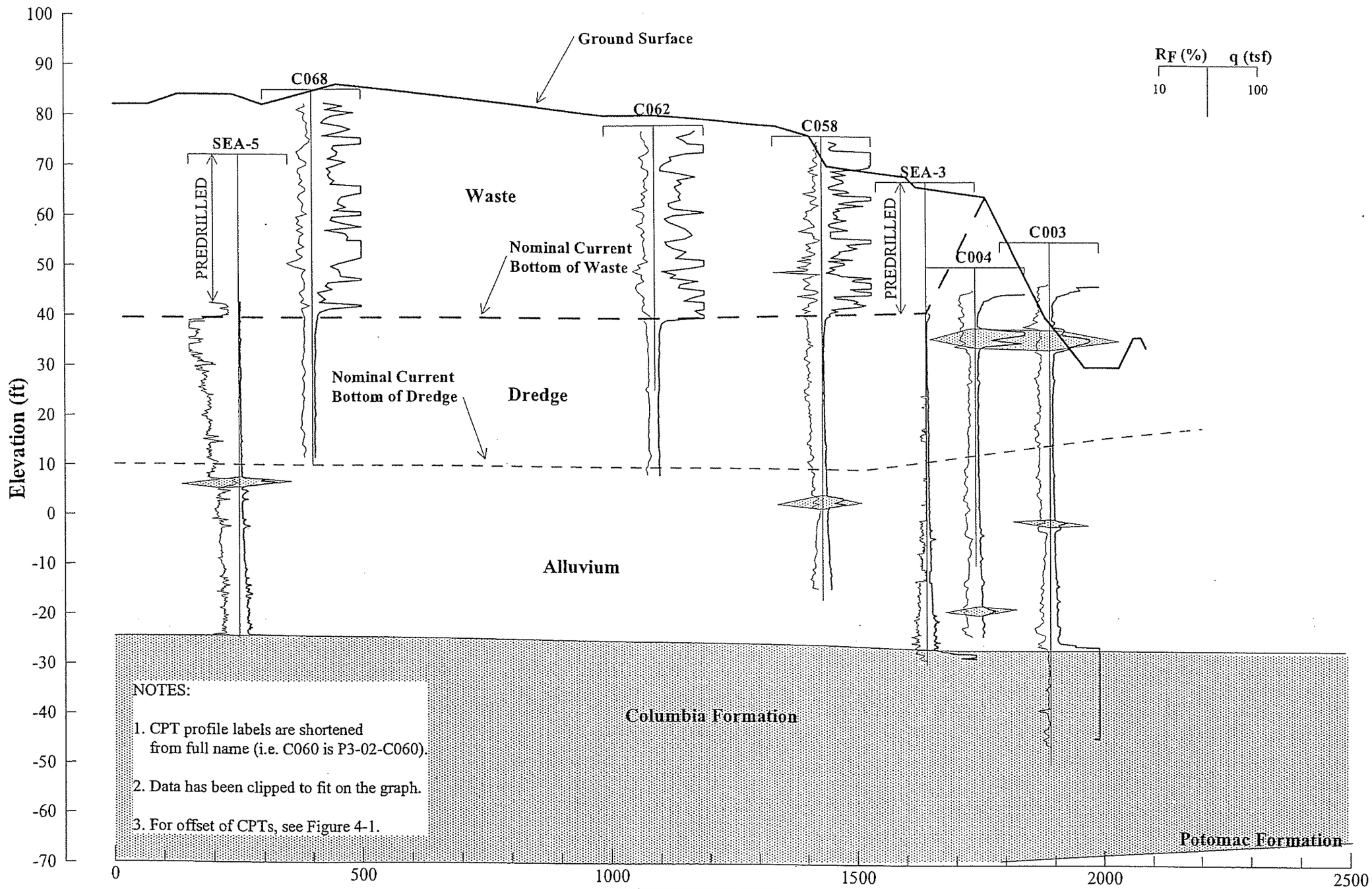
Columbia, Maryland 21046

CPT PROFILES
Section B-B

FIGURE 4-10

PROJECT: ME0250 Cherry Island



NOTES:

1. CPT profile labels are shortened from full name (i.e. C060 is P3-02-C060).
2. Data has been clipped to fit on the graph.
3. For offset of CPTs, see Figure 4-1.

Section CC.grf 2/21/03

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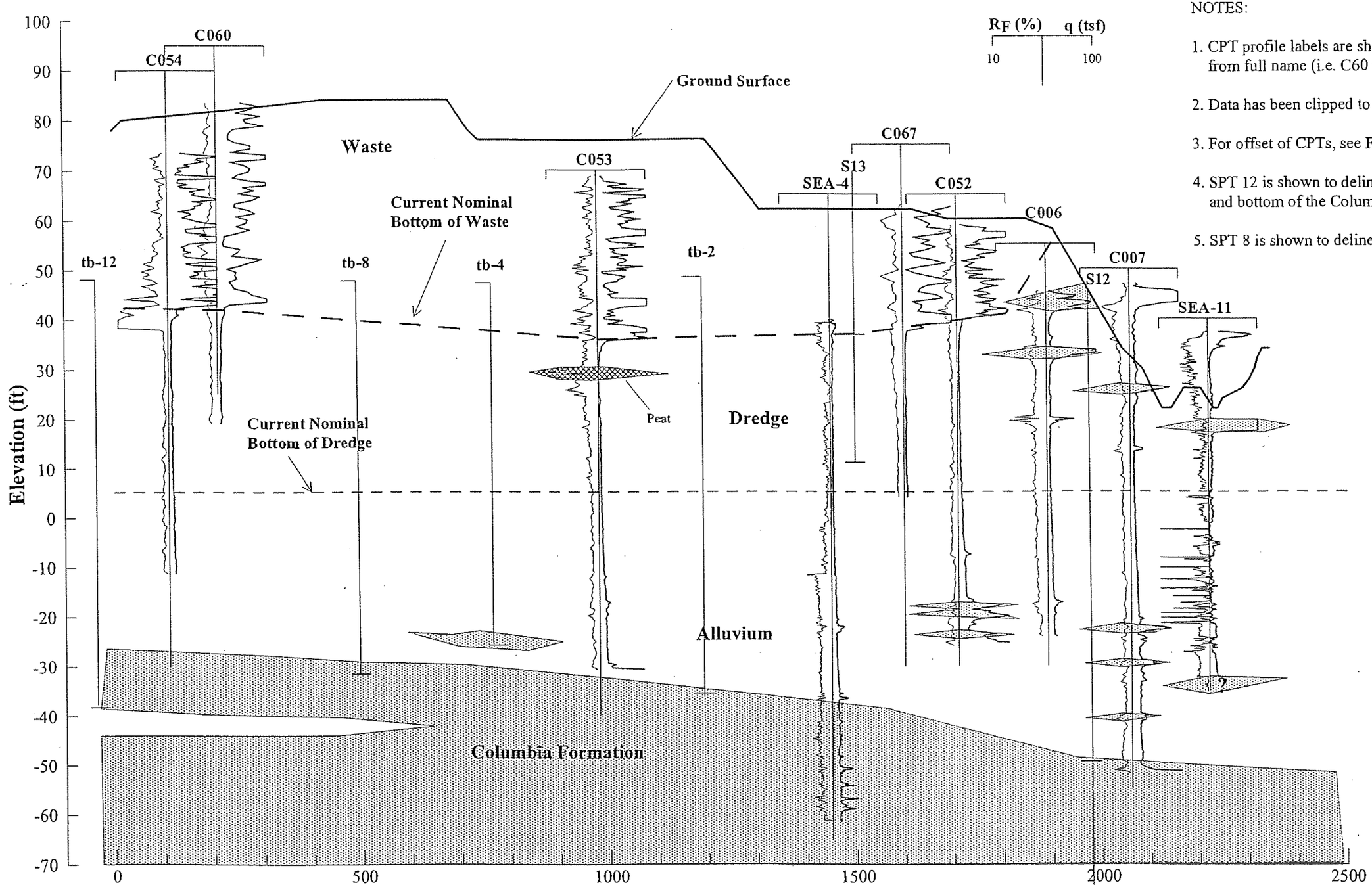
CPT PROFILES
Section C-C

FIGURE 4-11

PROJECT: ME0250 Cherry Island



NOTES:

1. CPT profile labels are shortened from full name (i.e. C60 is P3-02-C60).
2. Data has been clipped to fit on the graph.
3. For offset of CPTs, see Figure 4-1.
4. SPT 12 is shown to delineate the top and bottom of the Columbia Formation.
5. SPT 8 is shown to delineate the bottom of MSW.

Section DD.grf

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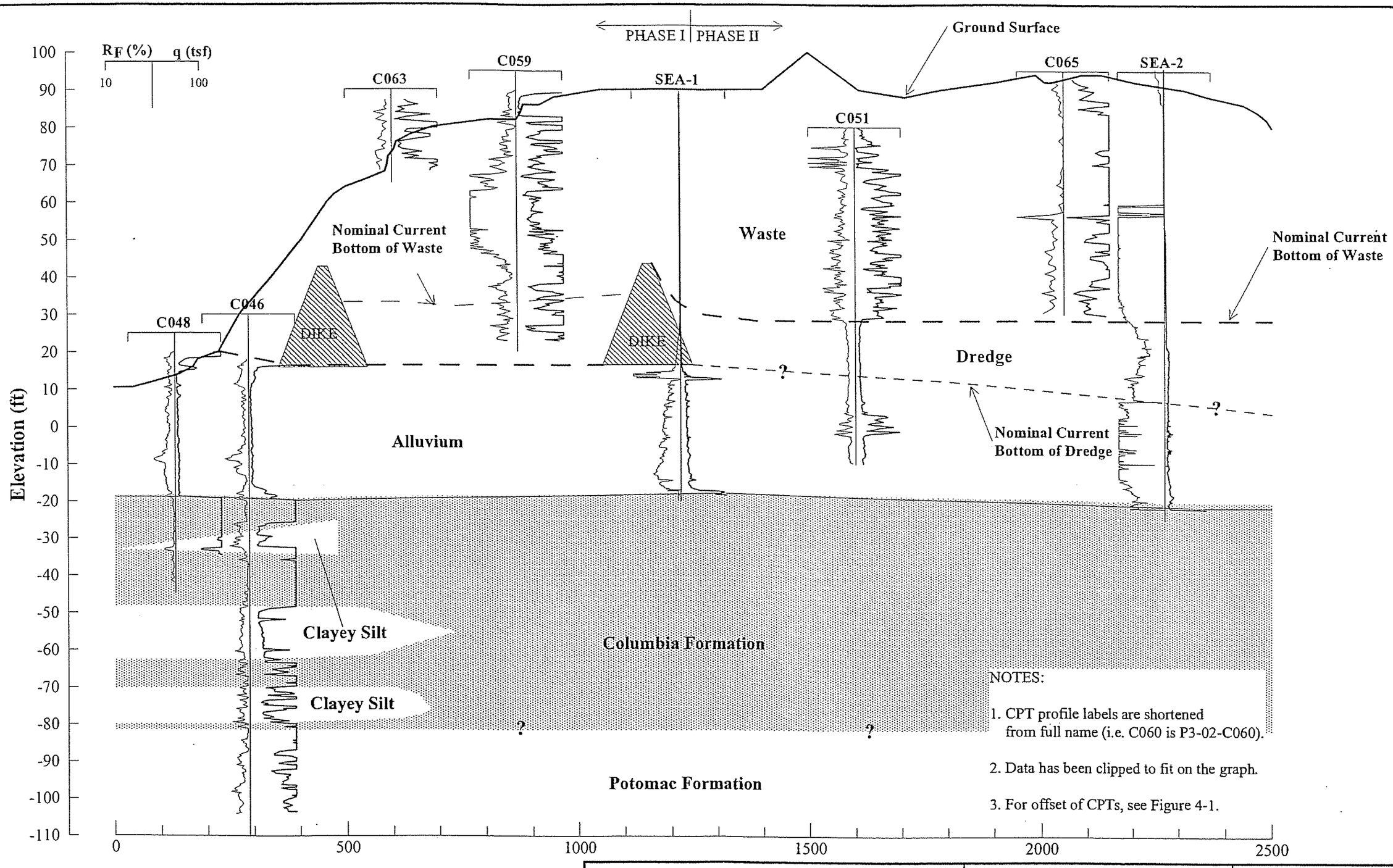
Columbia, Maryland 21046

CPT PROFILES
Section D-D

FIGURE 4-12

PROJECT: ME0250 Cherry Island



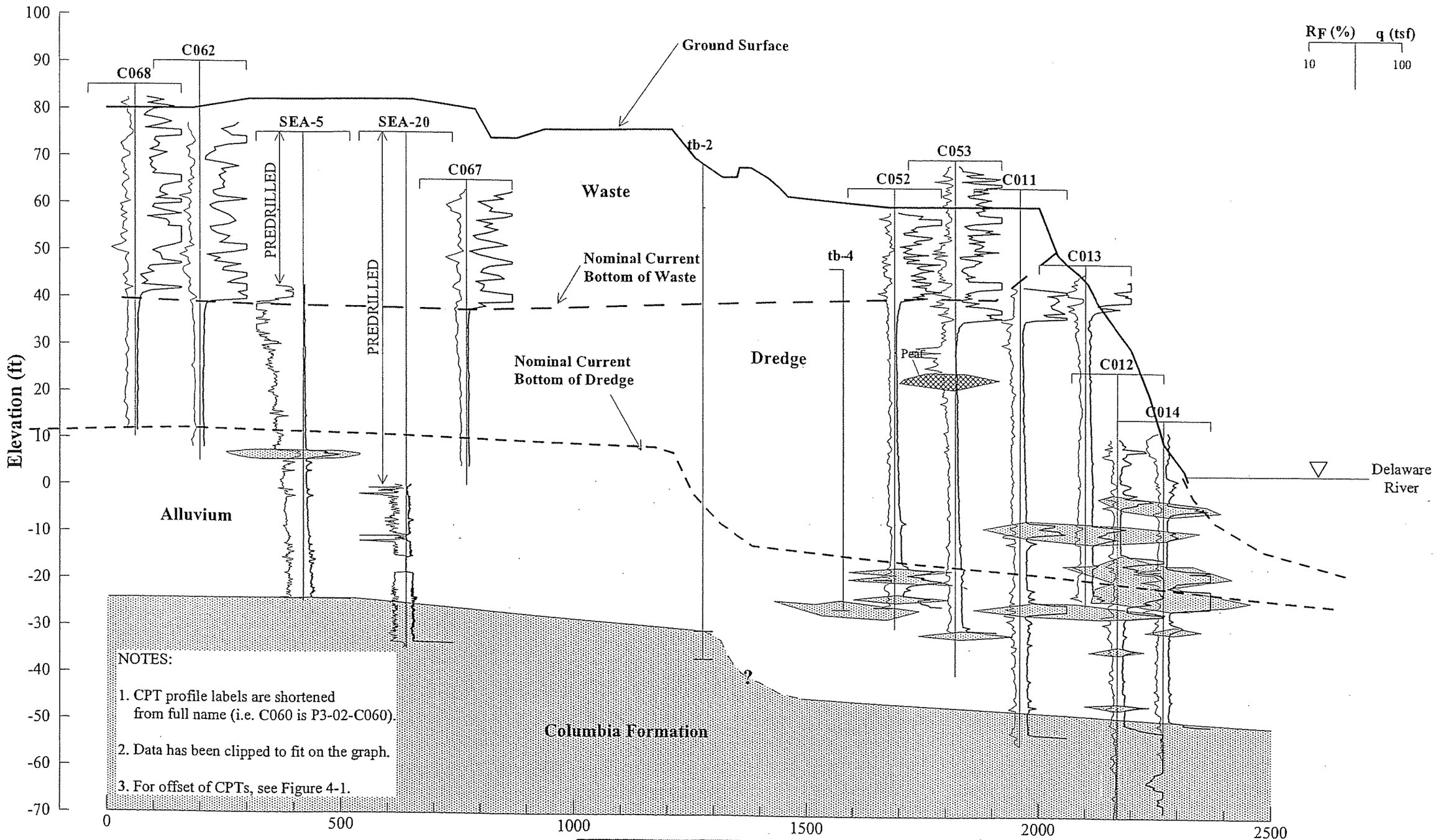
Section EE.grf

GEOSYNTEC CONSULTANTS
10015 Old Columbia Road, Suite A-200
Columbia, Maryland 21046

CPT PROFILES
Section E-E

FIGURE 4-13

PROJECT: ME0250 Cherry Island



Section FF.grf

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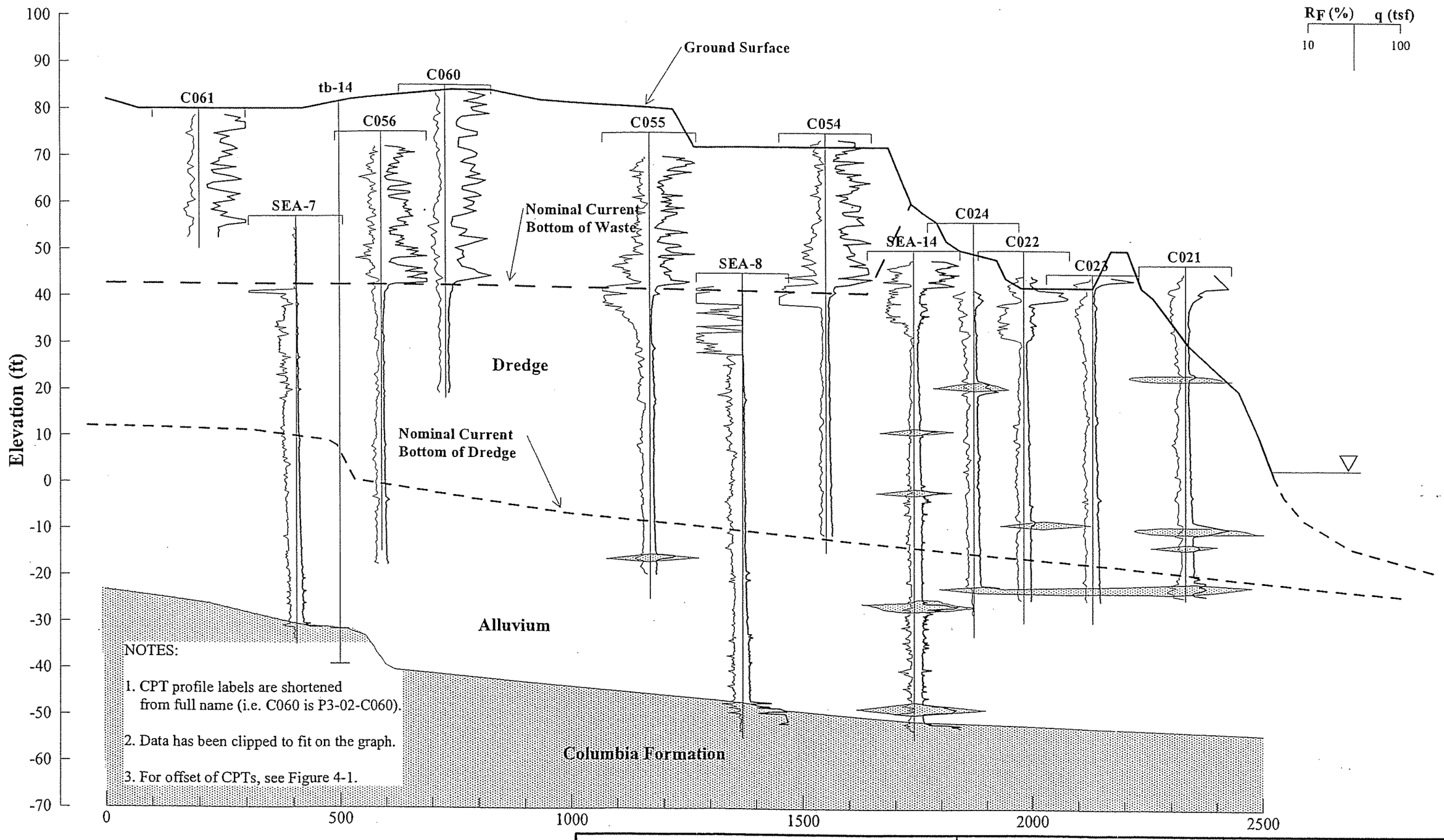
Columbia, Maryland 21046

CPT PROFILES
Section F-F

FIGURE 4-14

PROJECT: ME0250 Cherry Island



Section GG.grf

GEOSYNTEC CONSULTANTS
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Columbia, Maryland 21046

CPT PROFILES
Section G-G

FIGURE 4-15

PROJECT: ME0250 Cherry Island

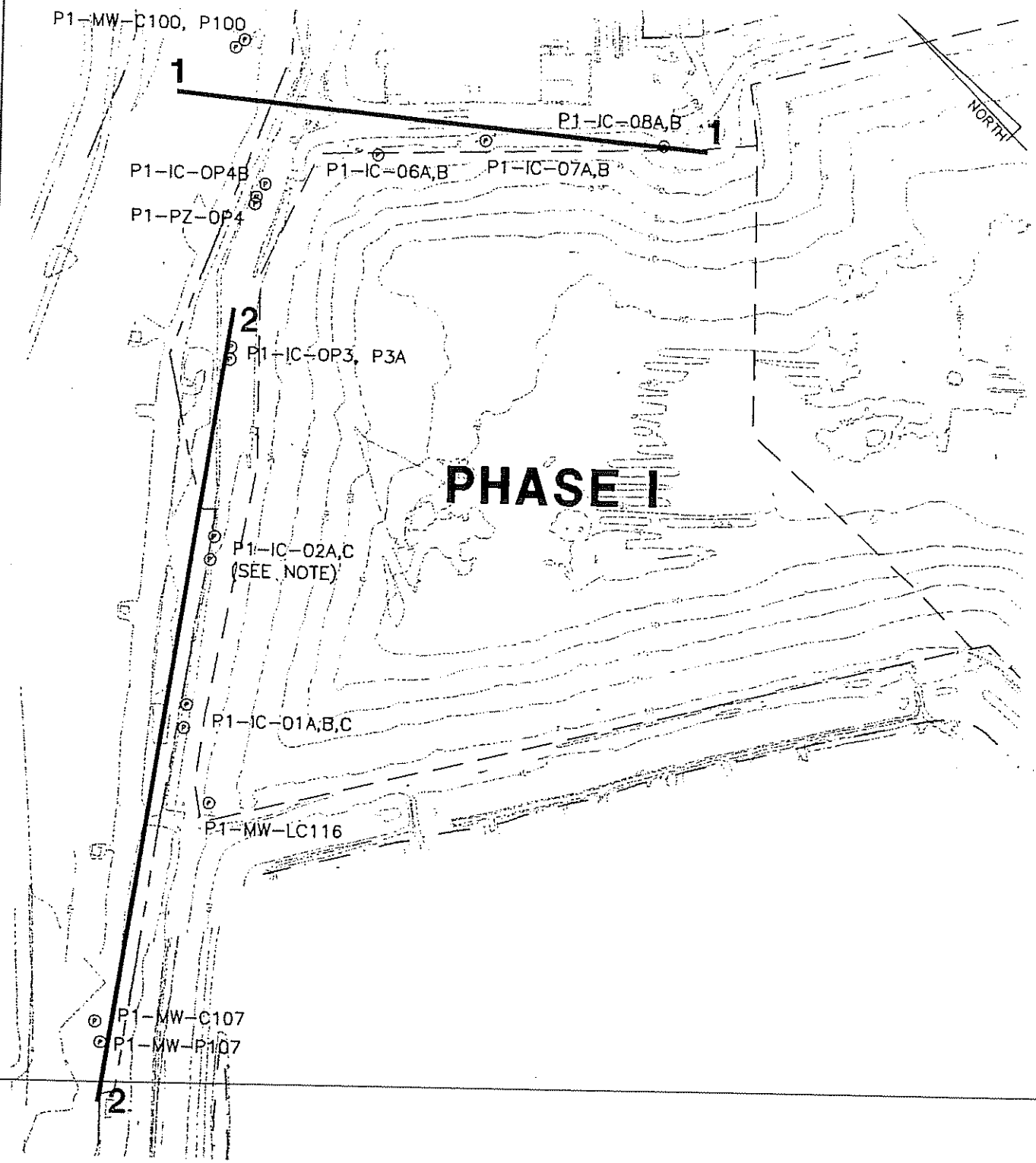
INSTRUMENT LOCATIONS
10015 Old Columbia Road, Suite A-200
Columbia, Maryland 21046

PLAN VIEW OF HISTORIC PIEZOMETERS

FIGURE 5-1

PROJECT: ME0250

PLAN VIEW OF PIEZOMETRIC CLUSTERS IN PHASE I



EXISTING GRADE CONTOUR

PHASE BOUNDARY

EXISTING PIEZOMETER

NOT TO SCALE

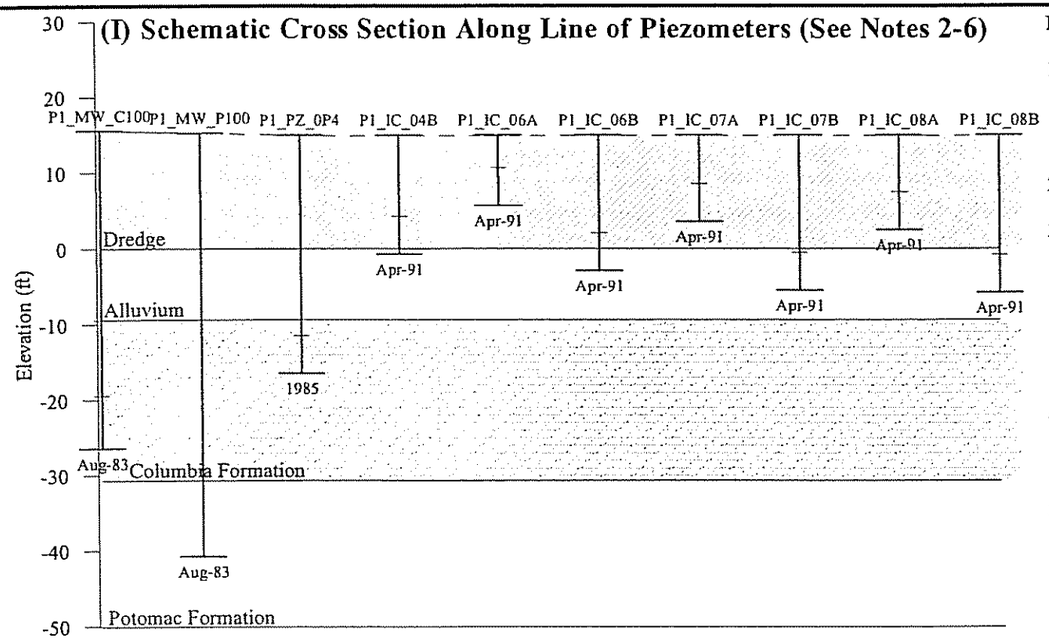
NOTE:

TO SIMPLIFY NOTATION, PIEZOMETERS LOCATED CLOSE TO EACH OTHER WITH SIMILAR LABELS HAVE BEEN CLUSTERED. FOR INSTANCE, P1-IC-02A,C INDICATES P1-IC-02A AND P1-IC-02C.

GEOSYNTEC CONSULTANTS

COLUMBIA, MARYLAND

FIGURE NO.	5-2
PROJECT NO.	ME0250
DOCUMENT NO.	
FILE NO.	0250F514

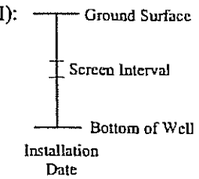


NOTES:

1. Data labeling system (as in GIS):
   P1_IC_07B: Phase 1 Piezometer
   P1_MW_C100: Phase 1 Monitoring Well
   P1_SP_3: Phase 1 Settlement Plate

2. No horizontal scale in (I).

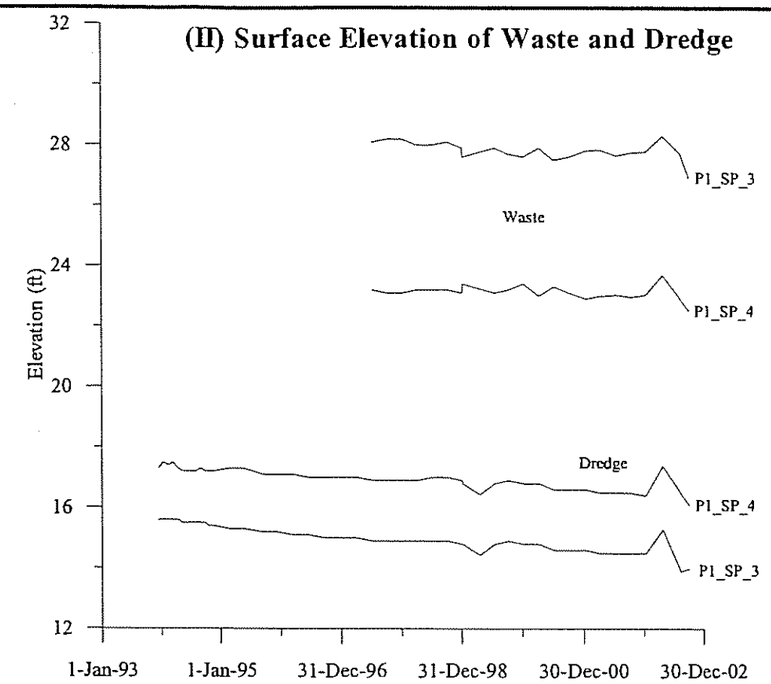
3. Key for (I):

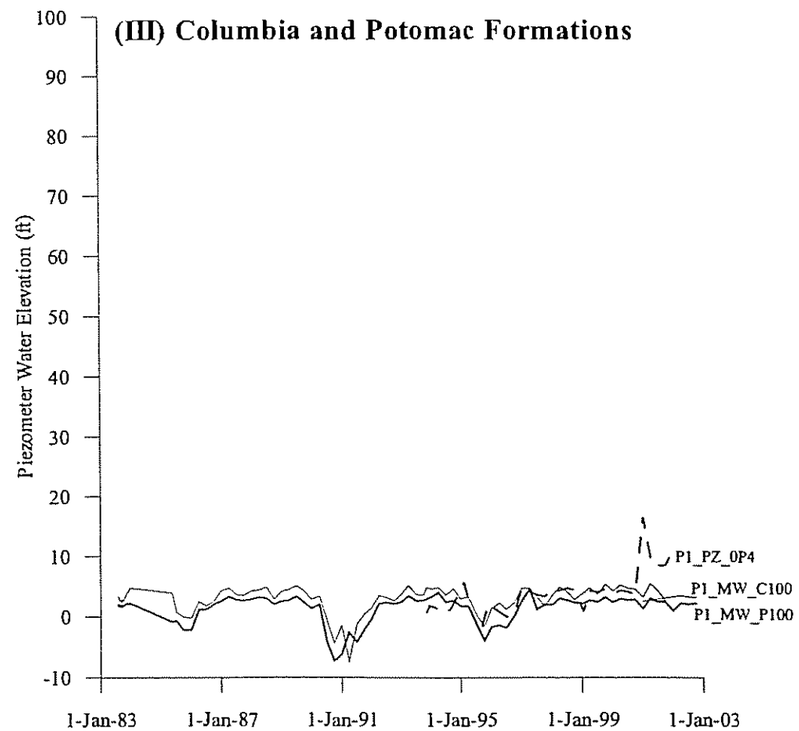


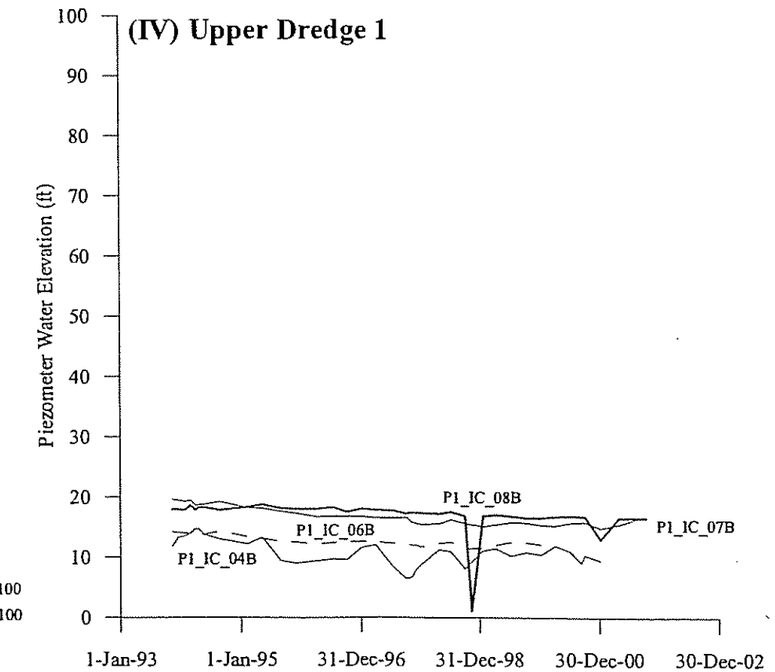
4. Elevation of geologic strata shown in (I) are approximate and are estimated from information obtained when P1_MW_C100 and P1_MW_P100 were installed.

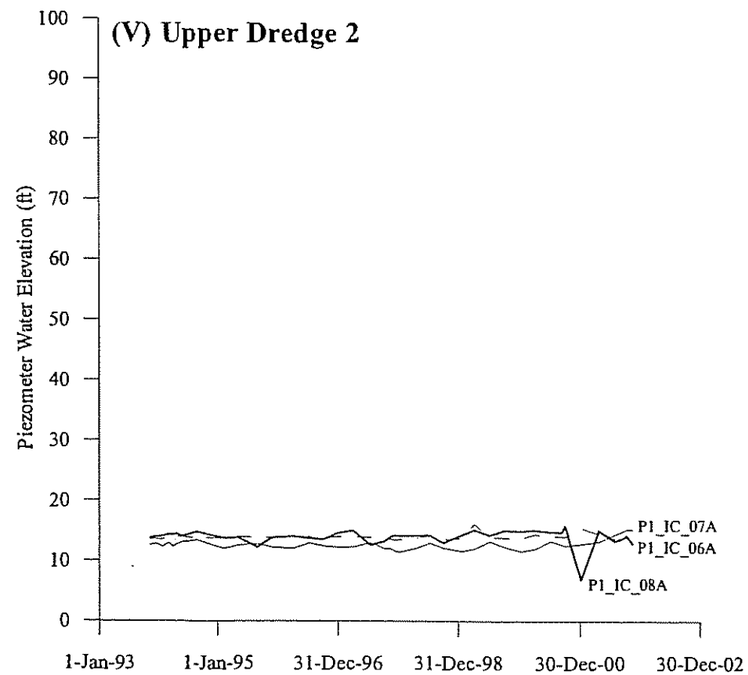
5. Limited installation data is available for P1_PZ_0P4, P1_IC_04B, P1_IC_06A, P1_IC_06B, P1_IC_07A, P1_IC_07B, P1_IC_08A, and P1_IC_08B; ground surface and bottom of boring elevations are estimated.

6. Lines of piezometers are shown in Figure 5-2.









NOTE: Date scale is different in (III), (IV), and (V).

Phase 1-1(2).grf

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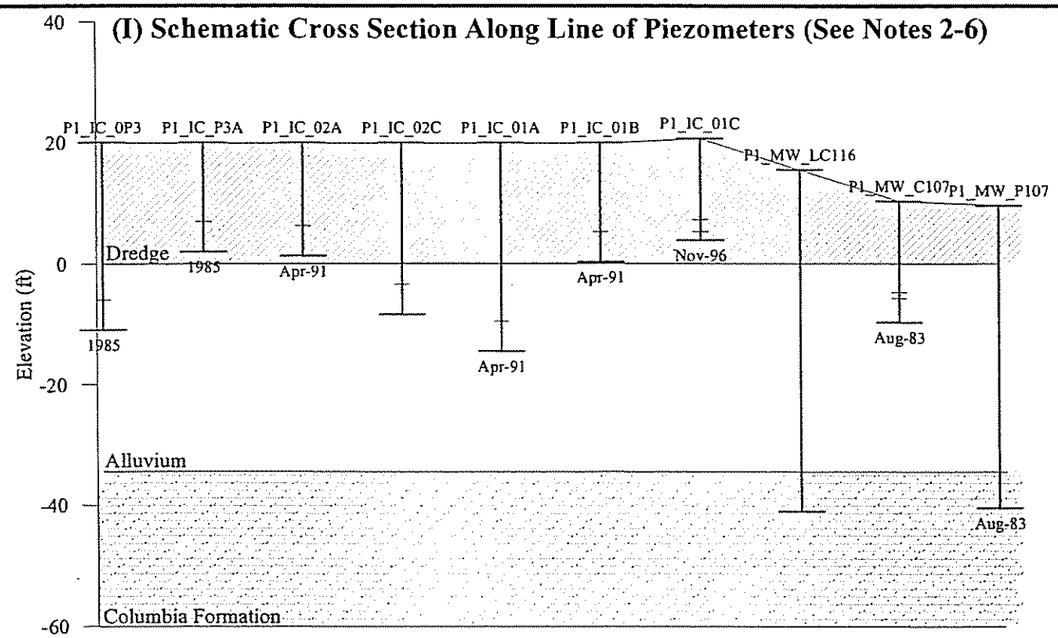
Piezometer Data
Phase 1, Line 1

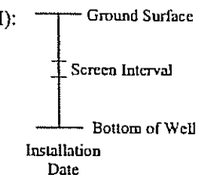
FIGURE 5-3

PROJECT: ME0250 Cherry Island

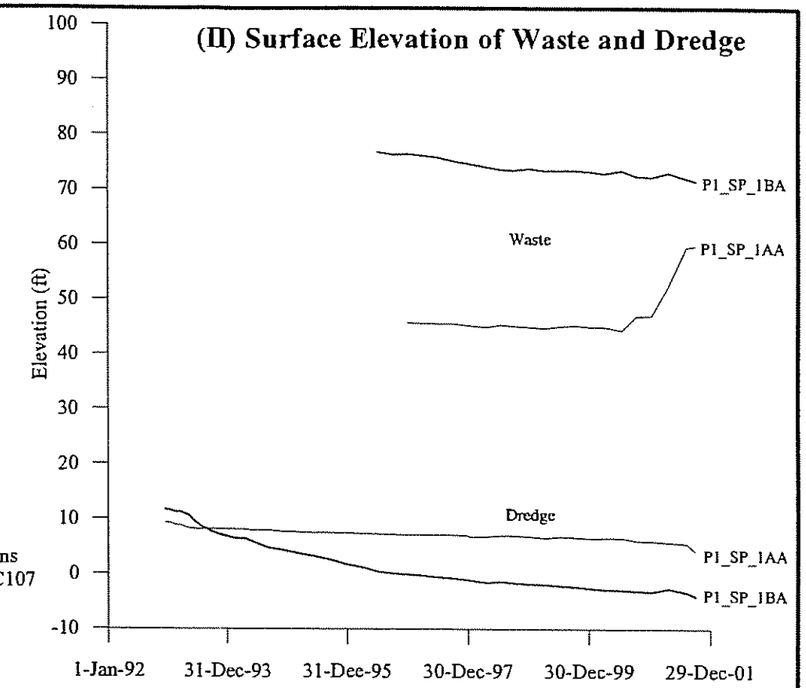


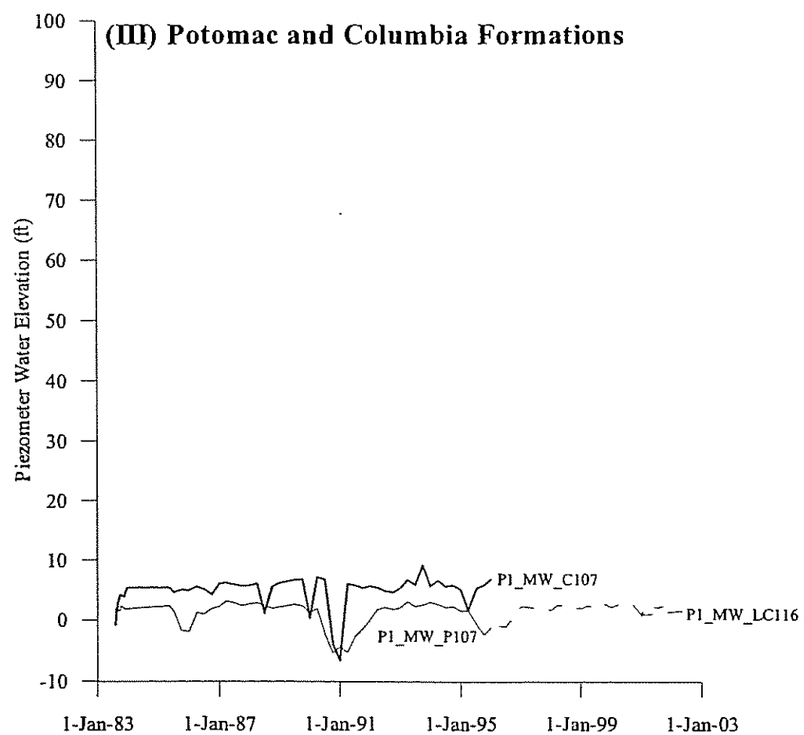
NOTES:

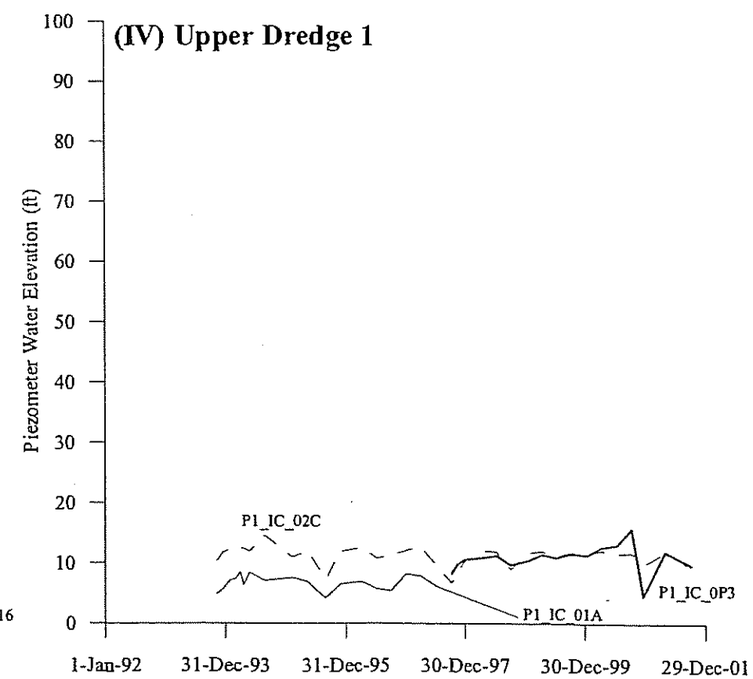
1. Data labeling system (as in GIS):
   P1_IC_01C: Phase 1 Piezometer
   P1_MW_C107: Phase 1 Monitoring Well
   P1_SP_1AA: Phase 1 Settlement Plate
2. No horizontal scale in (I).
3. Key for (I):

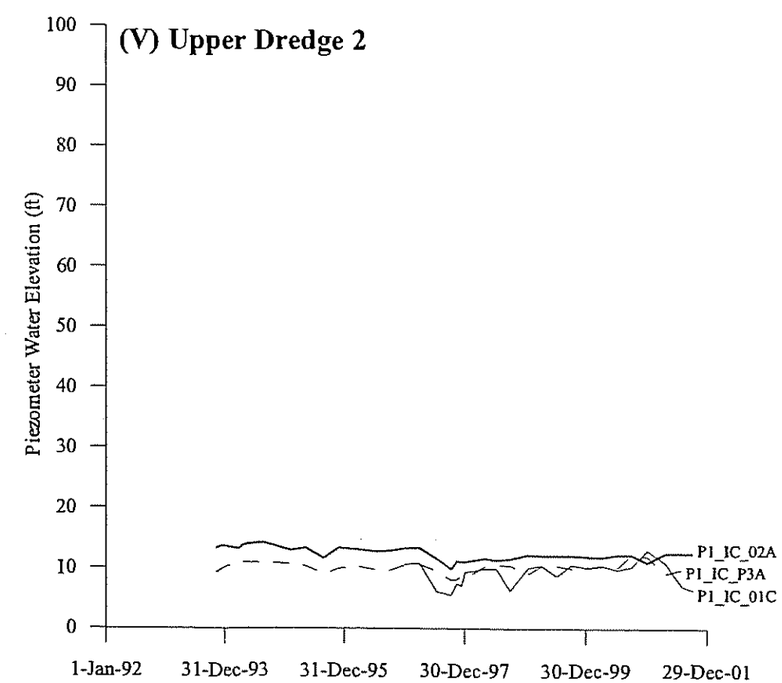


4. Elevation of geologic strata shown in (I) are approximate and are estimated from information obtained when P1_MW_C107 and P1_MW_P107 were installed.
5. Limited installation data is available for P1_IC_0P3, P1_IC_P3A, P1_IC_02A, P1_IC_01A, P1_IC_01B, and P1_IC_01C; ground surface and bottom of boring elevations are estimated. Water level data is available for P1_MW_C107 for the year 1983 but is not shown on this Figure.
6. Lines of piezometers are shown in Figure 5-2.









NOTE: Date scale is different in (III), (IV), and (V).

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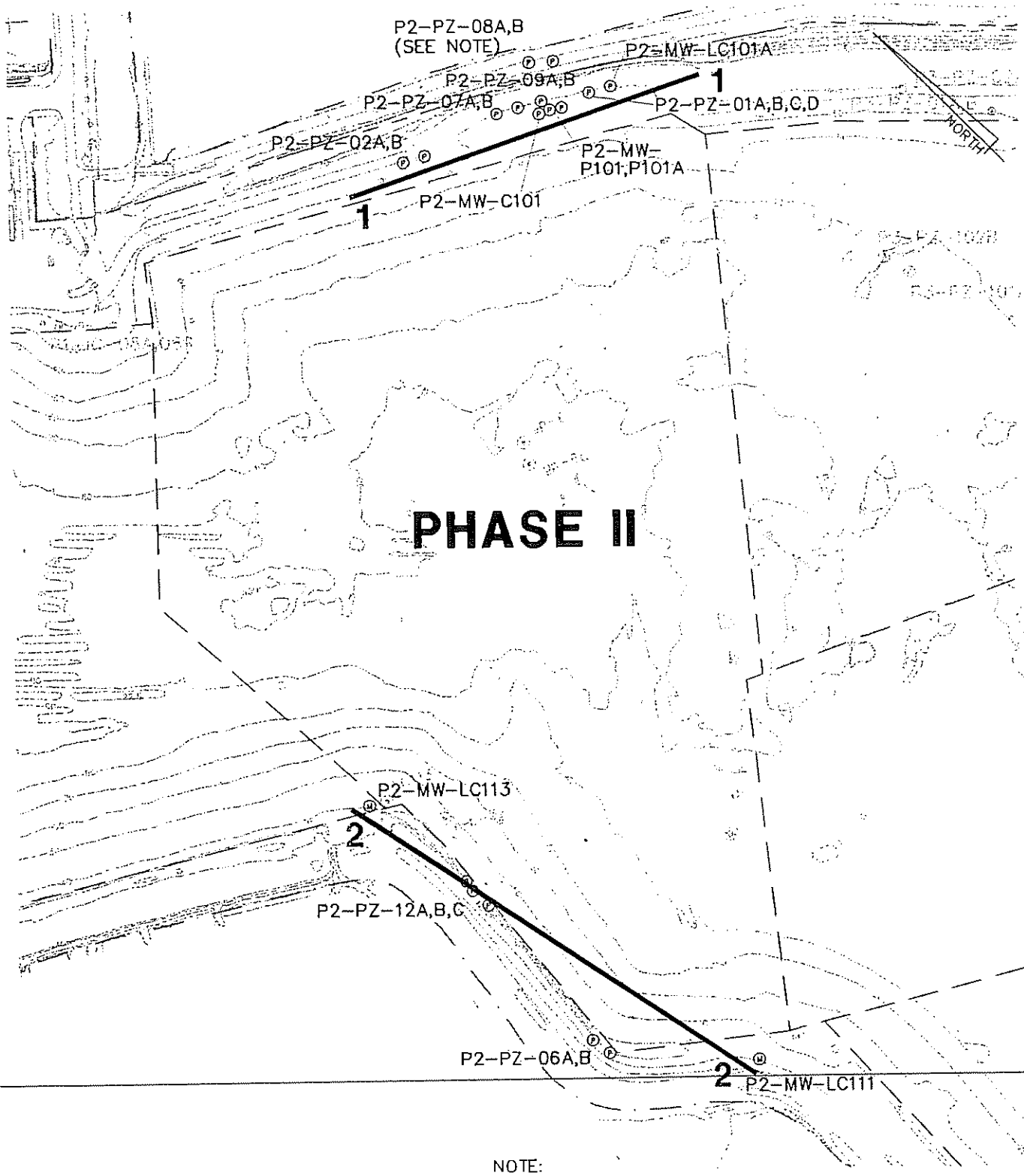
Piezometer Data
Phase 1, Line 2

FIGURE 5-4

PROJECT: ME0250 Cherry Island

Phase 1-2(2).gif

PLAN VIEW OF PIEZOMETRIC CLUSTERS IN PHASE II



EXISTING GRADE CONTOUR

PHASE BOUNDARY

EXISTING PIEZOMETER

NOTE:

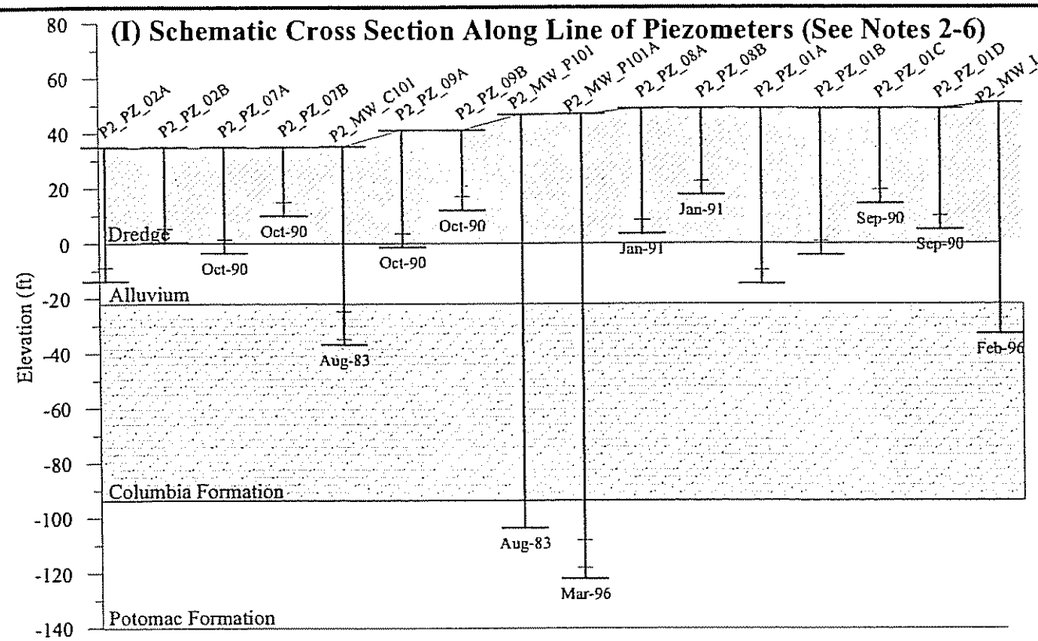
TO SIMPLIFY NOTATION, PIEZOMETERS LOCATED CLOSE TO EACH OTHER WITH SIMILAR LABELS HAVE BEEN CLUSTERED. FOR INSTANCE, P2-PZ-08A,B INDICATES P2-PZ-08A AND P2-PZ-08B.

NOT TO SCALE

GEOSYNTEC CONSULTANTS

COLUMBIA, MARYLAND

FIGURE NO	5-5
PROJECT NO	ME0250
DOCUMENT NO.	
FILE NO.	0250F513

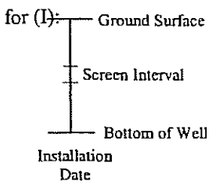


NOTES:

1. Data labeling system (as in GIS):
   P3_PZ_01A: Phase 3 Piezometer
   P3_SP_4: Phase 3 Settlement Plate

2. No horizontal scale in (I).

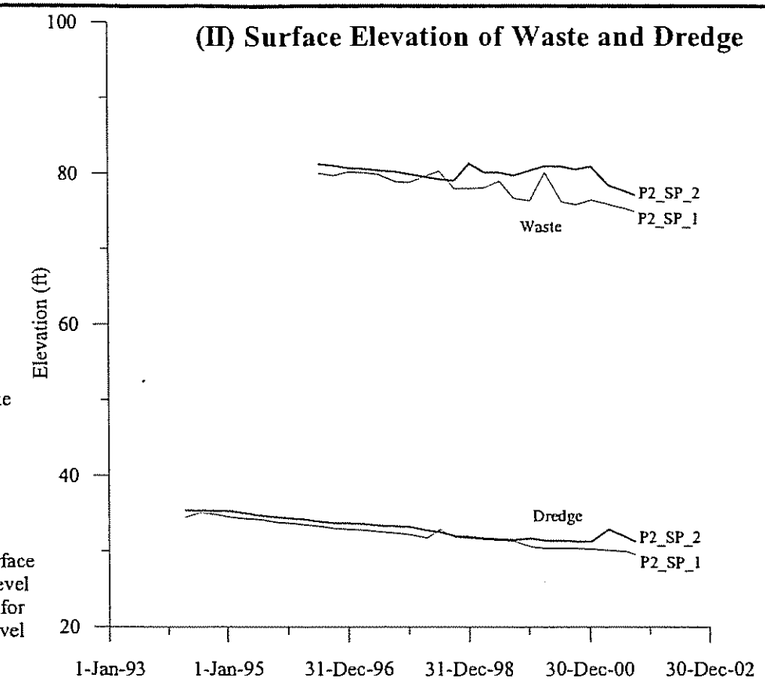
3. Key for (I):

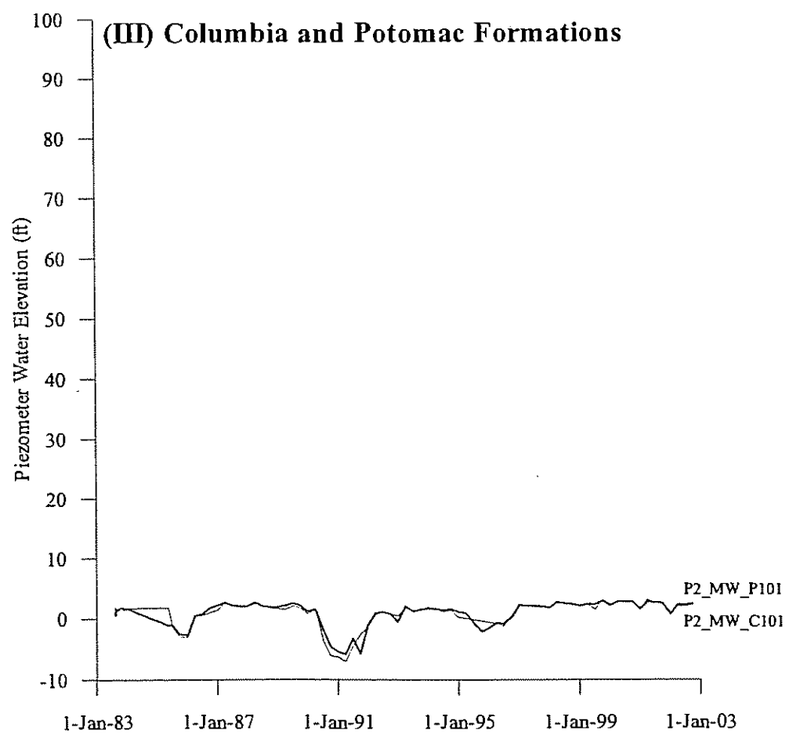


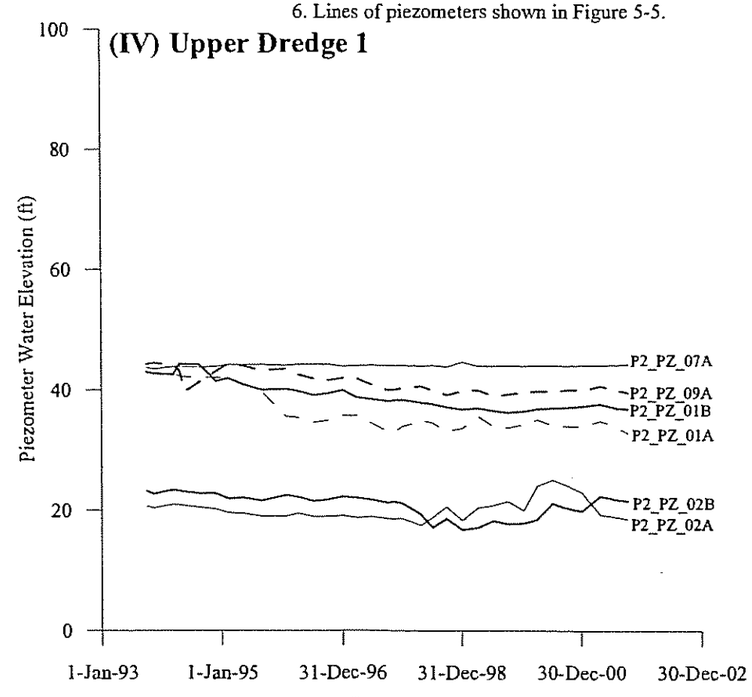
4. Elevation of geologic strata shown in (I) are approximate and are estimated from information obtained when P2_MW_C101 and P2_MW_P101 were installed.

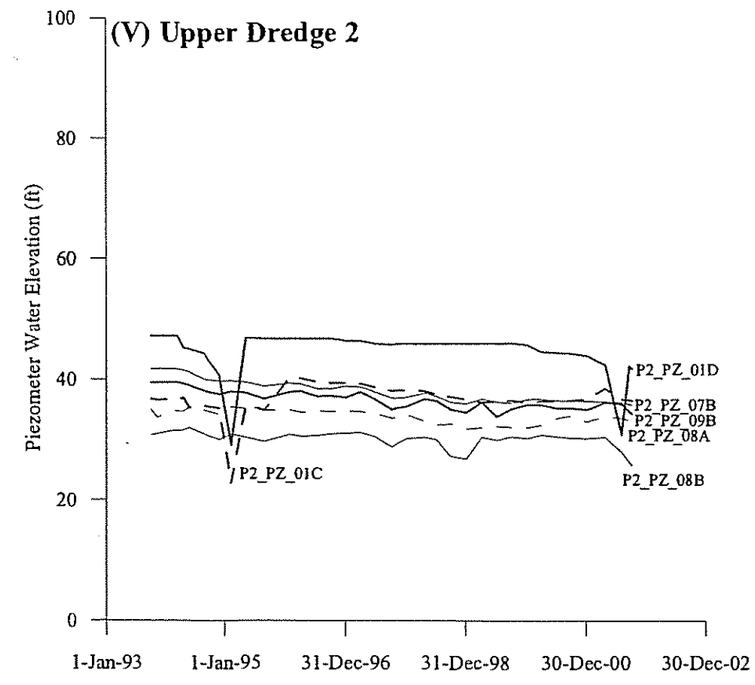
5. Limited installation data is available for P2_PZ_02A, P2_PZ_02B, P2_PZ_07A, P2_PZ_07B, P2_PZ_08A, P2_PZ_08B, P2_PZ_01A, P2_PZ_01B, P2_PZ_01C, P2_PZ_01D, P2_PZ_09A, and P2_PZ_09B; ground surface and bottom of boring elevations are estimated. Water level data is available for P2_MW_C101 and P2_MW_P101 for the year 1983 but is not shown on this Figure. Water level data is not available for P2_MW_P101A.

6. Lines of piezometers shown in Figure 5-5.









Phase 2-1(2).grf

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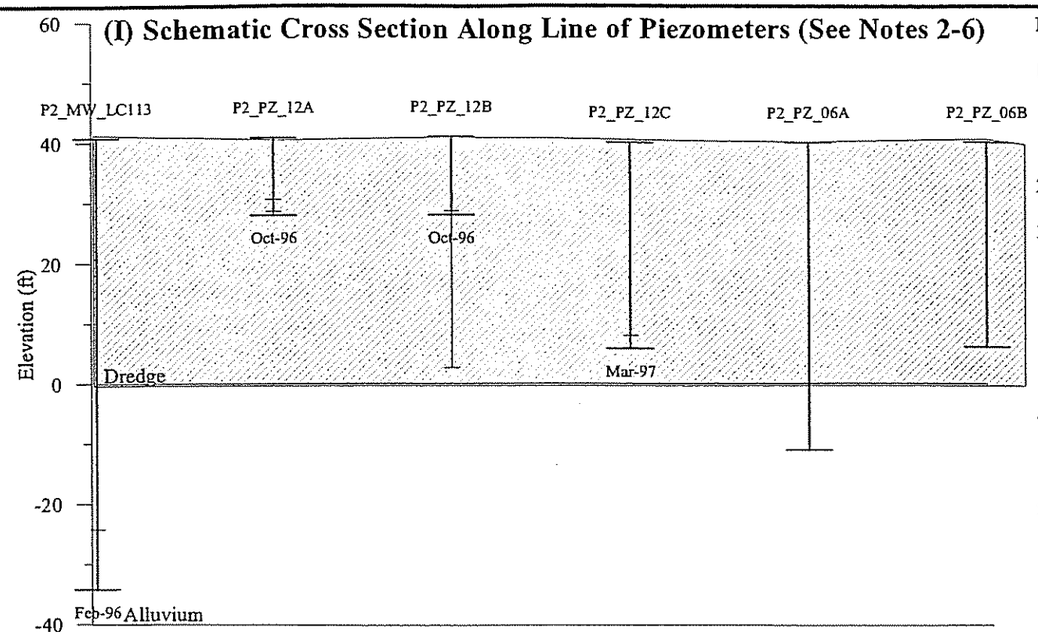
Columbia, Maryland 21046

Piezometer Data
Phase 2, Line 1

FIGURE 5-6

PROJECT: ME0250 Cherry Island

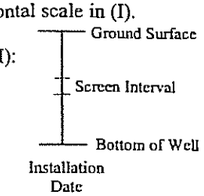


NOTES:

1. Data labeling system (as in GIS):
   P2_PZ_12A: Phase 2 Piezometer
   P2_MW_LC113: Phase 2 Monitoring Well
   P2_SP_10: Phase 2 Settlement Plate

2. No horizontal scale in (I).

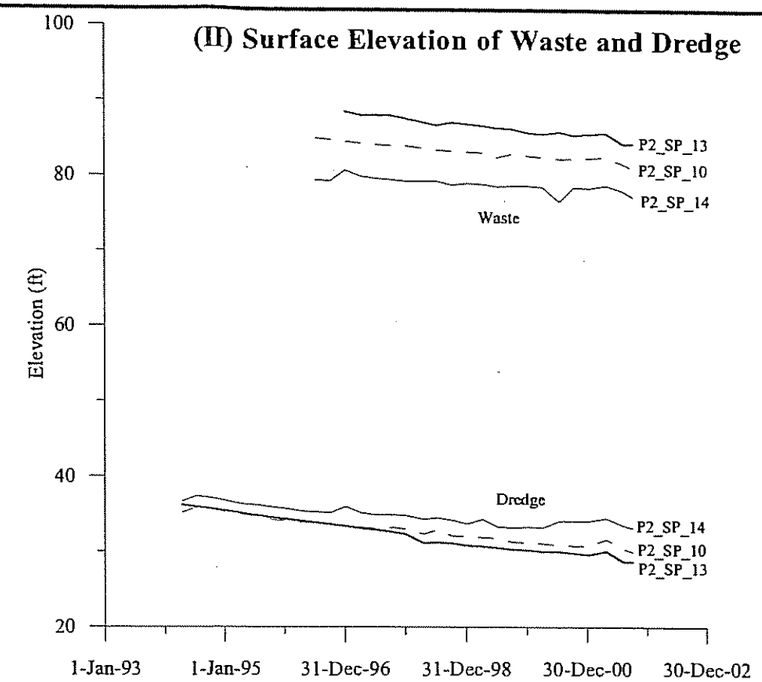
3. Key for (I):

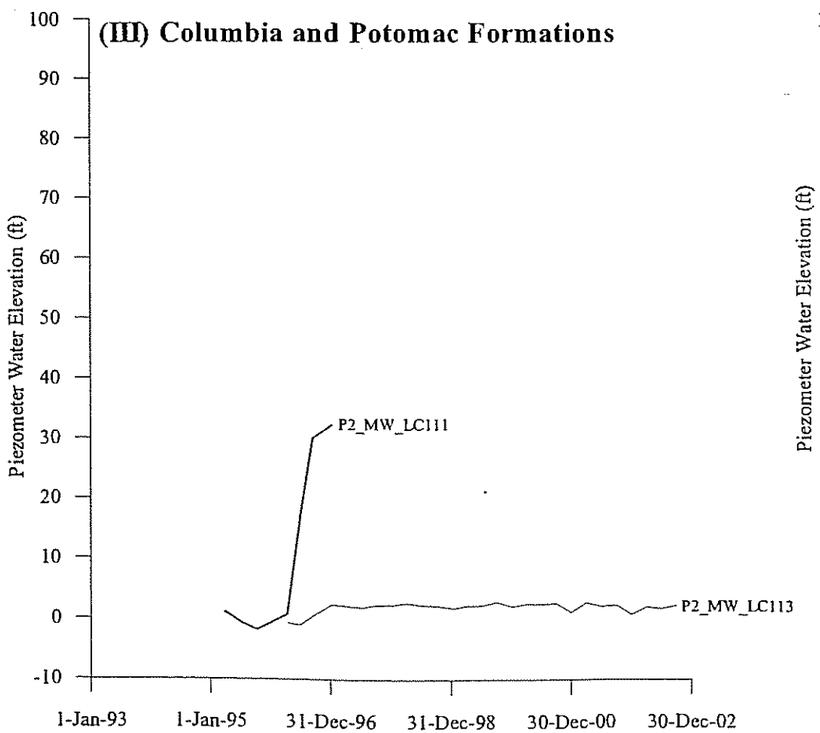


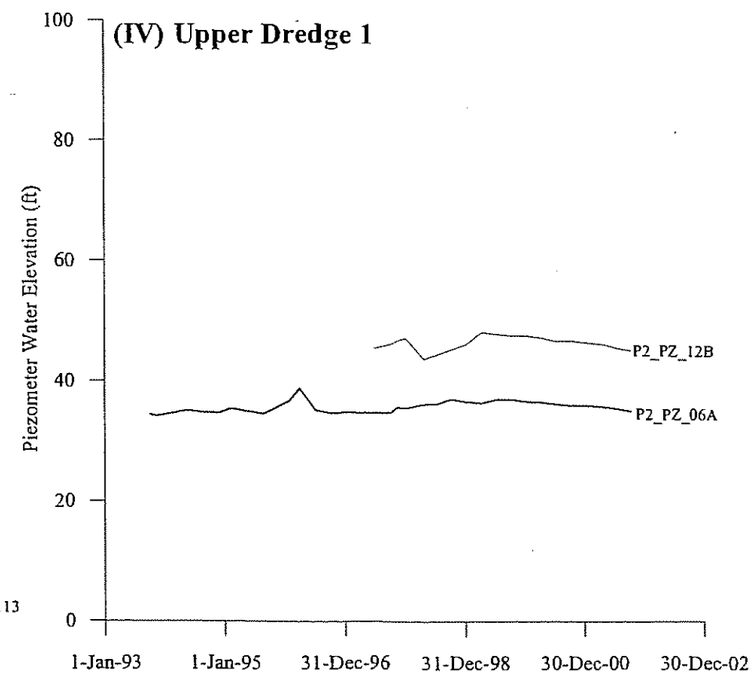
4. Elevation of geologic strata shown in (I) are approximate and are estimated from information obtained when the wells in (I) were installed.

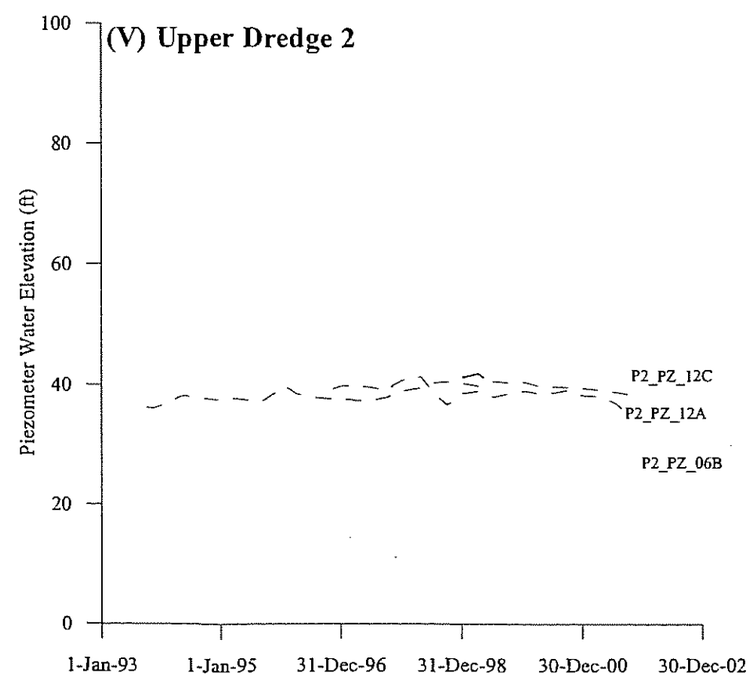
5. Installation data is not available for P2_PZ_06A and P2_PZ_06B; however, water level data is shown in (IV). Water level data is not available for P2_MW_LC113. Neither installation data nor water level data are available for P2_MW_LC111.

6. Lines of piezometers shown in Figure 5-5.









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Columbia, Maryland 21046

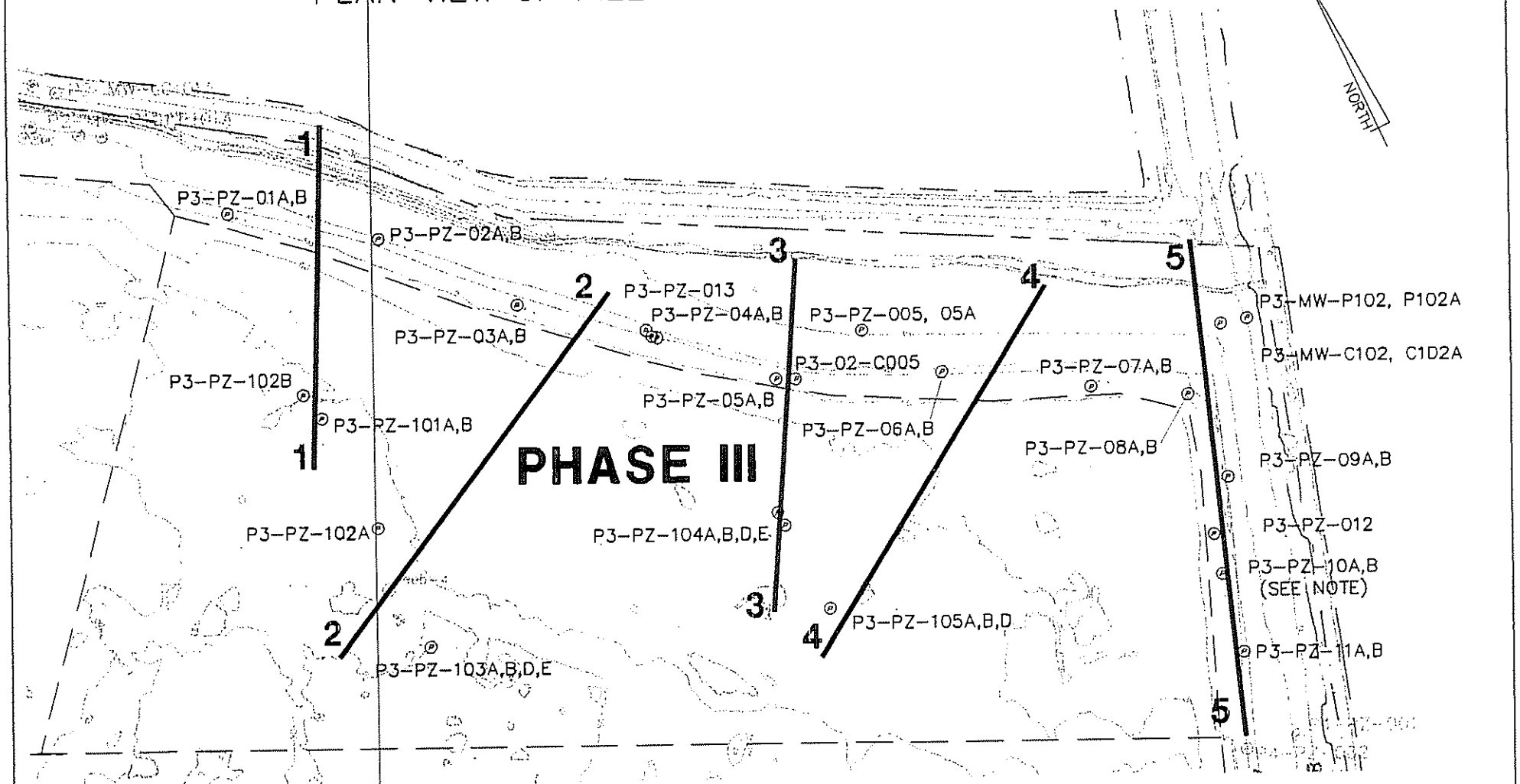
Piezometer Data
Phase 2, Line 2

FIGURE 5-7

PROJECT: ME0250 Cherry Island

Phase 2-2(2).grf

PLAN VIEW OF PIEZOMETRIC CLUSTERS IN PHASE III



EXISTING GRADE CONTOUR

PHASE BOUNDARY

EXISTING PIEZOMETER

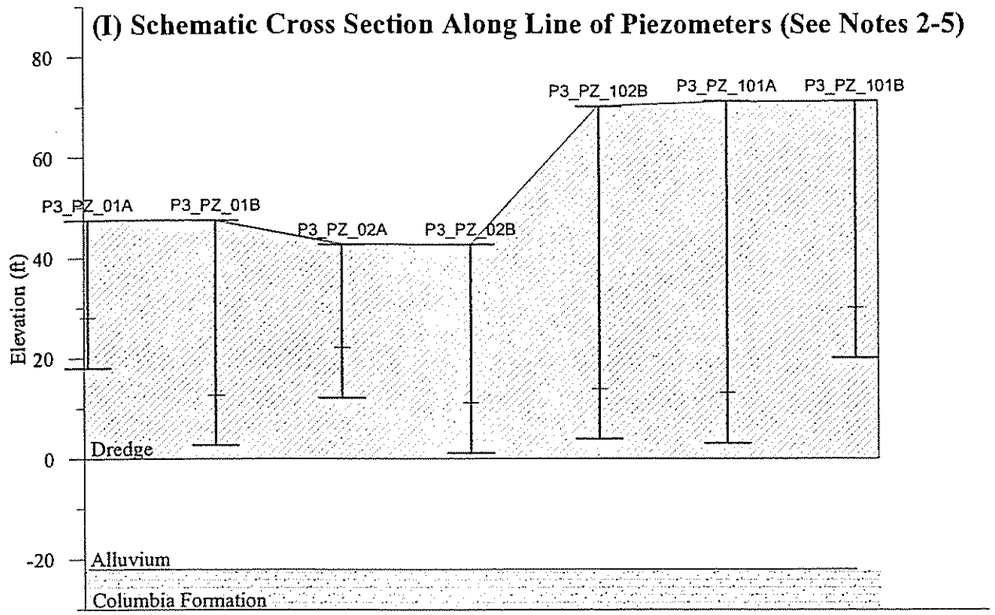
NOTE:

TO SIMPLIFY NOTATION, PIEZOMETERS LOCATED CLOSE TO EACH OTHER WITH SIMILAR LABELS HAVE BEEN CLUSTERED. FOR INSTANCE, P3-PZ-10A,B INDICATES P3-PZ-10A AND P3-PZ-10B.

NOT TO SCALE

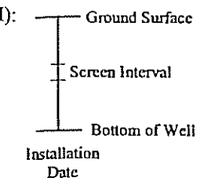
GEOSYNTEC CONSULTANTS
COLUMBIA, MARYLAND

FIGURE NO.	5-8
PROJECT NO.	ME0250
DOCUMENT NO.	
FILE NO.	0250F516

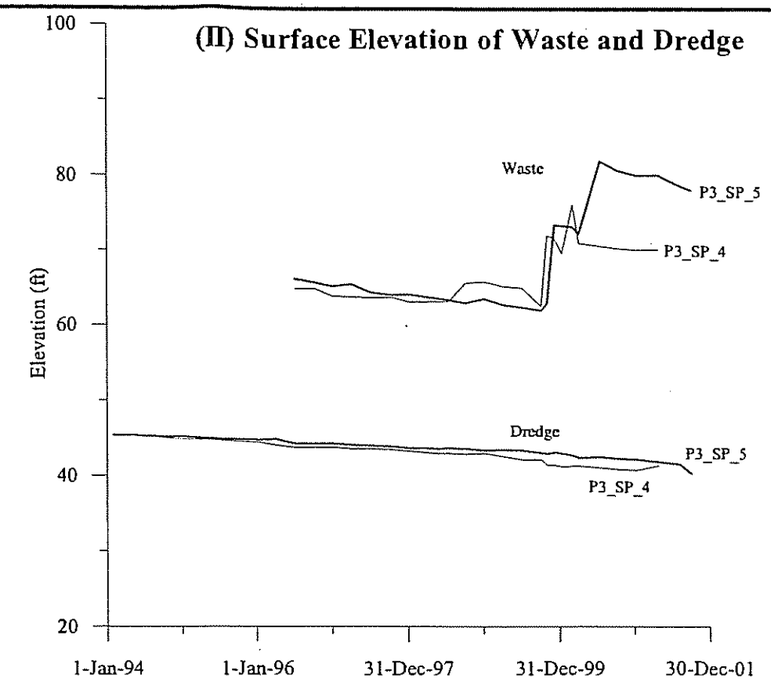


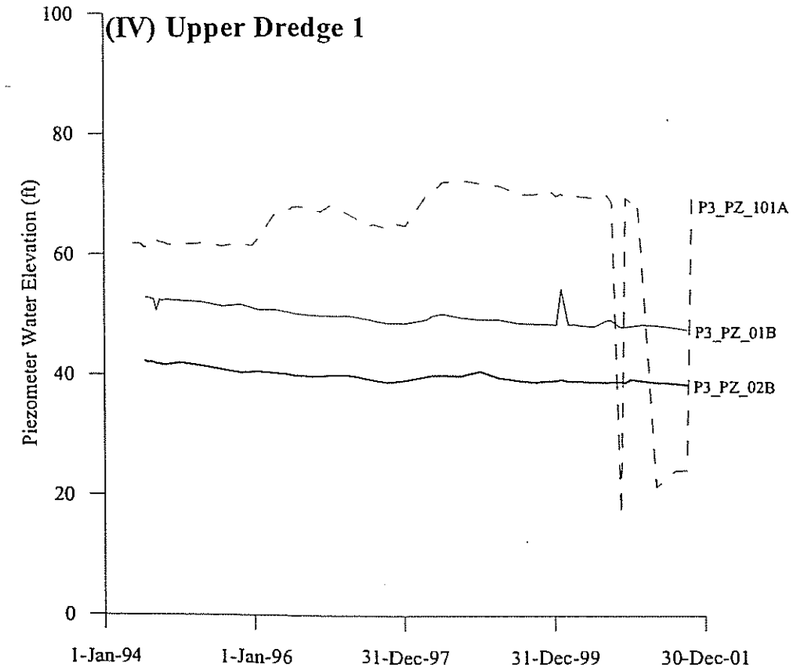
NOTES:

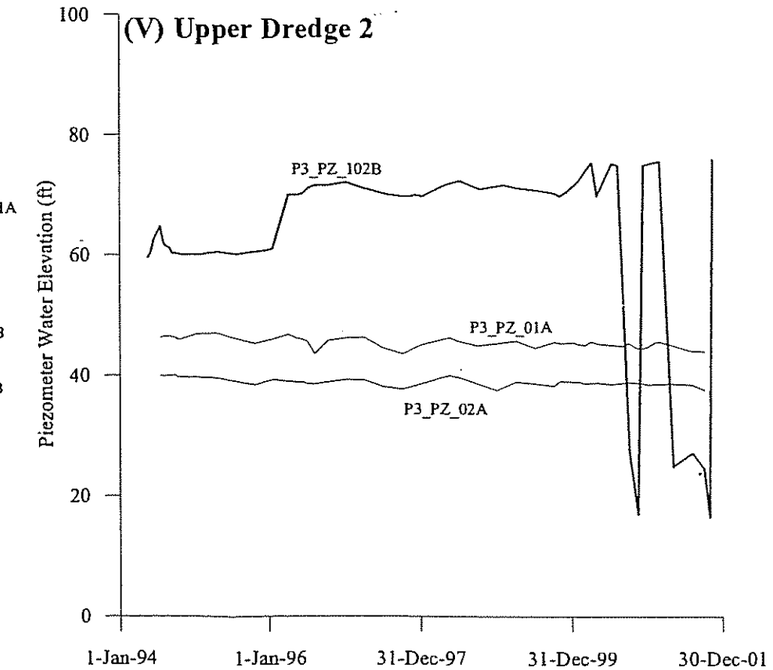
1. Data labeling system (as in GIS):
   P3_PZ_01A: Phase 3 Piezometer
   P3_SP_4: Phase 3 Settlement Plate
2. No horizontal scale in (I).
3. Key for (I):



4. Elevation of geologic strata shown in (I) are approximate and are estimated from data obtained when P2_MW_P101 and P2_MW_C101 were installed.
5. Only limited installation data is available for all of the wells with water level data shown on this Figure; ground surface and bottom of boring elevations are estimated.
6. Lines of piezometers shown in Figure 5-8.







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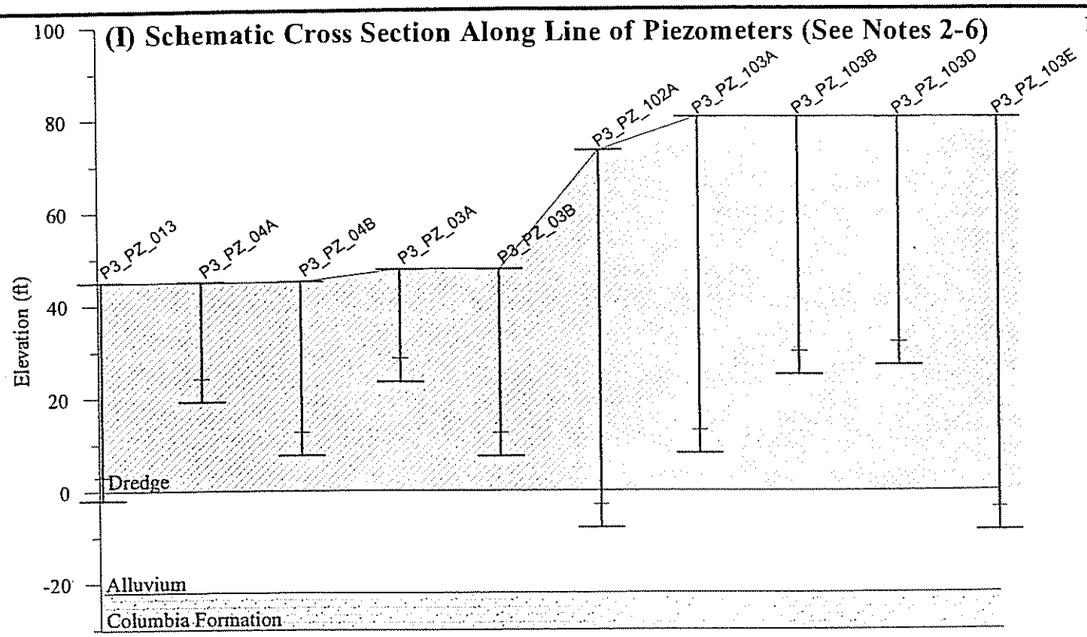
Columbia, Maryland 21046

Piezometer Data
Phase 3, Line 1

FIGURE 5-9

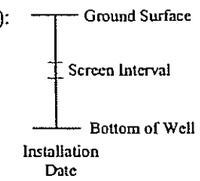
PROJECT: ME0250 Cherry Island

Phase 3-1(2).grf

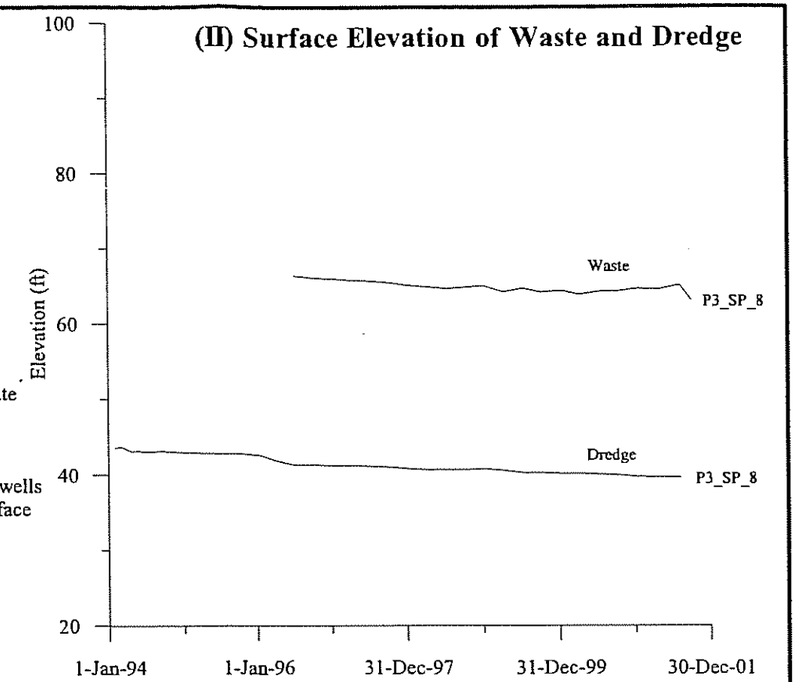


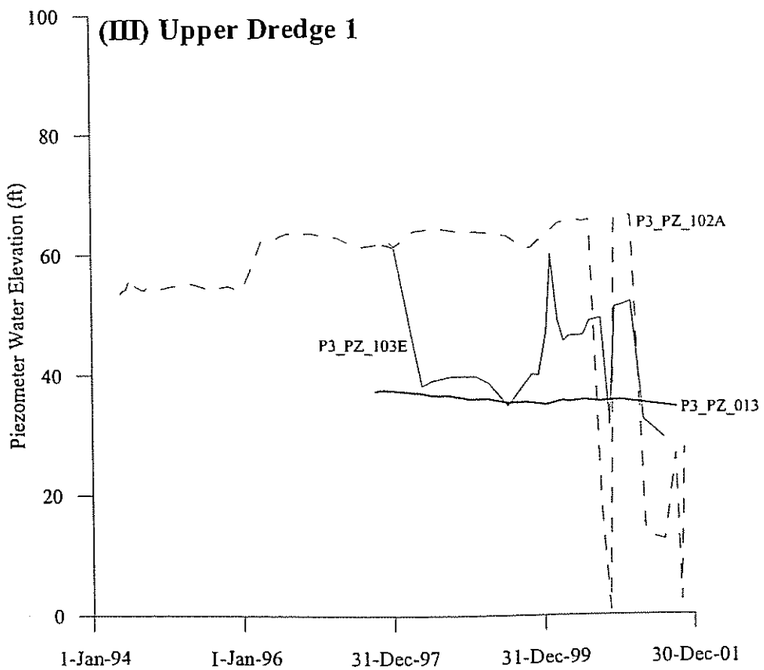
NOTES:

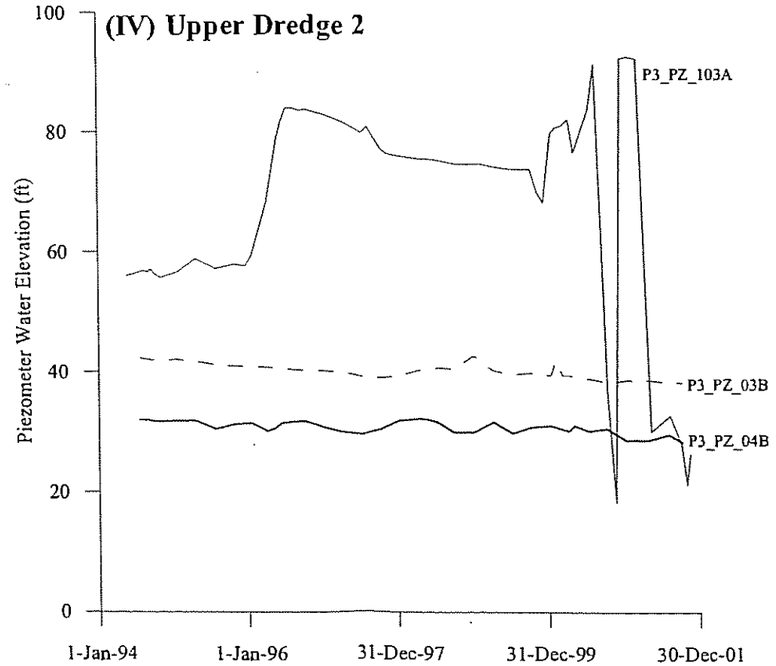
1. Data labeling system (as in GIS):
   P3_PZ_03A: Phase 3 Piezometer
   P3_SP_8: Phase 3 Settlement Plate
2. No horizontal scale in (I).
3. Key for (I):

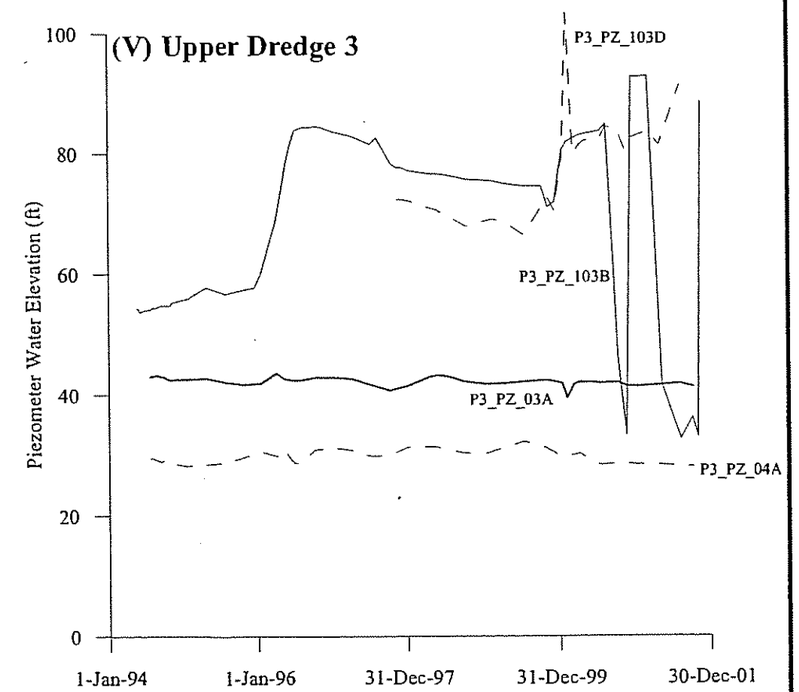


4. Elevation of geologic strata shown in (I) are approximate and are estimated from information obtained when P2_MW_P101 and P2_MW_C101 were installed.
5. Only limited installation data is available for all of the wells with water level data shown on this Figure; ground surface and bottom of boring elevations are estimated.
6. Lines of piezometers shown in Figure 5-8.









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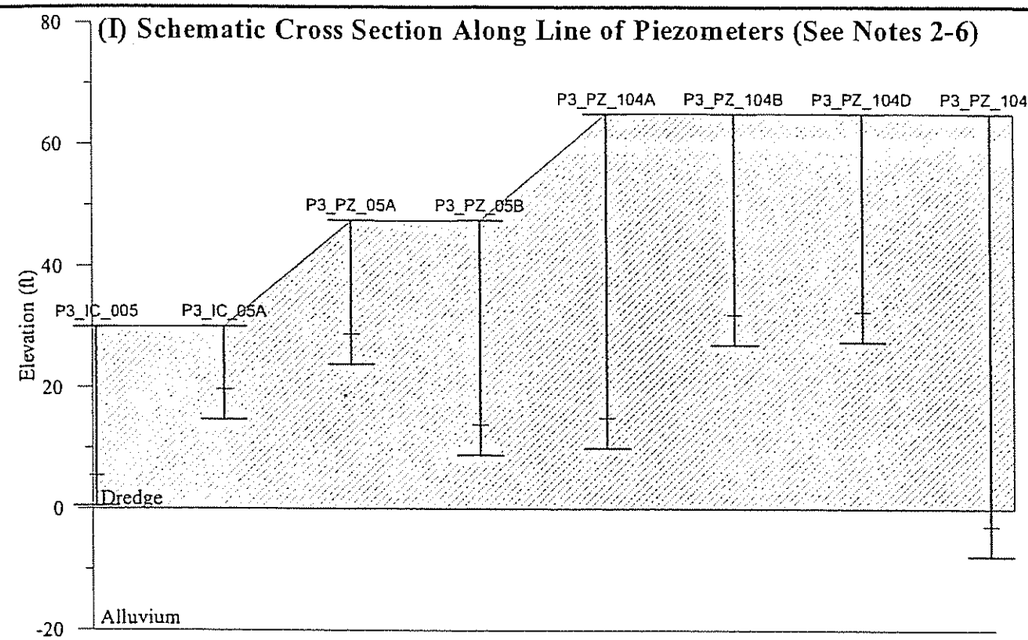
Piezometer Data
Phase 3, Line 2

FIGURE 5-10

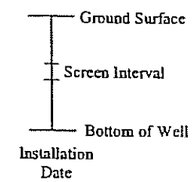
PROJECT: ME0250 Cherry Island

Phase 3-2(2)

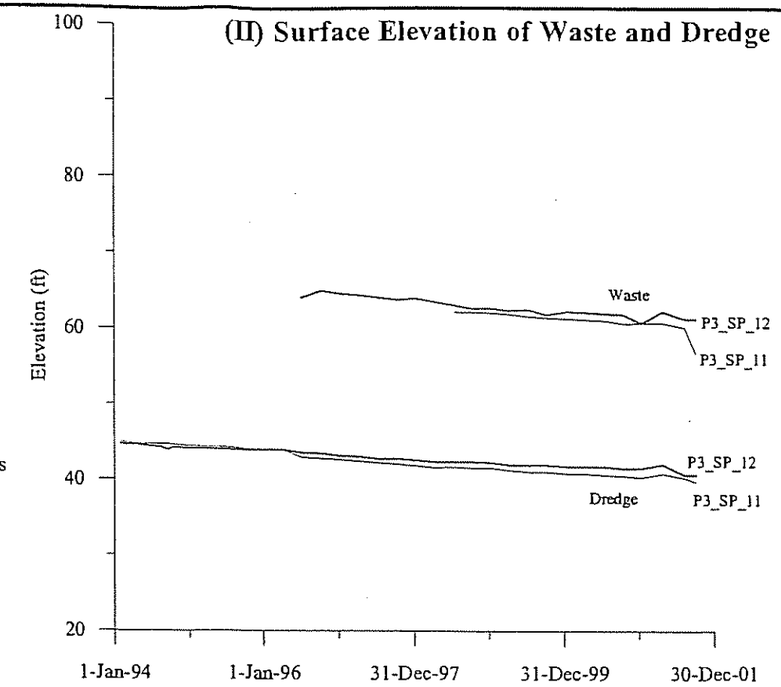


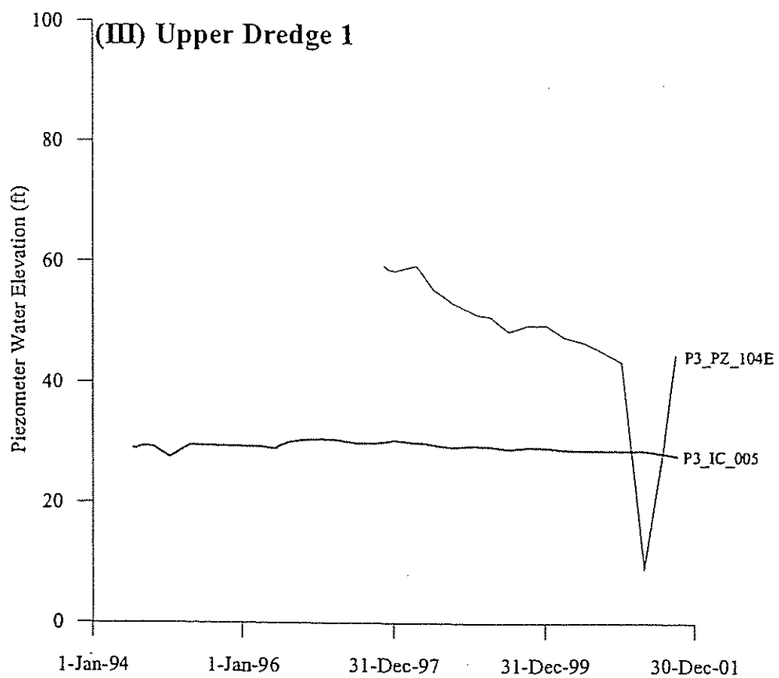
NOTES:

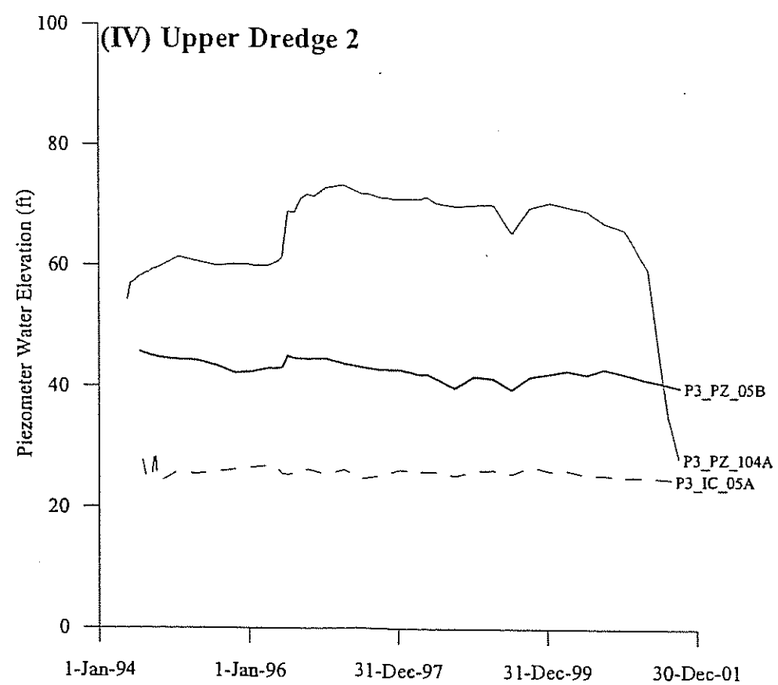
1. Data labeling system (as in GIS):
   P3_PZ_05A: Phase 3 Piezometer
   P3_SP_11: Phase 3 Settlement Plate
2. No horizontal scale in (I).
3. Key for (I):

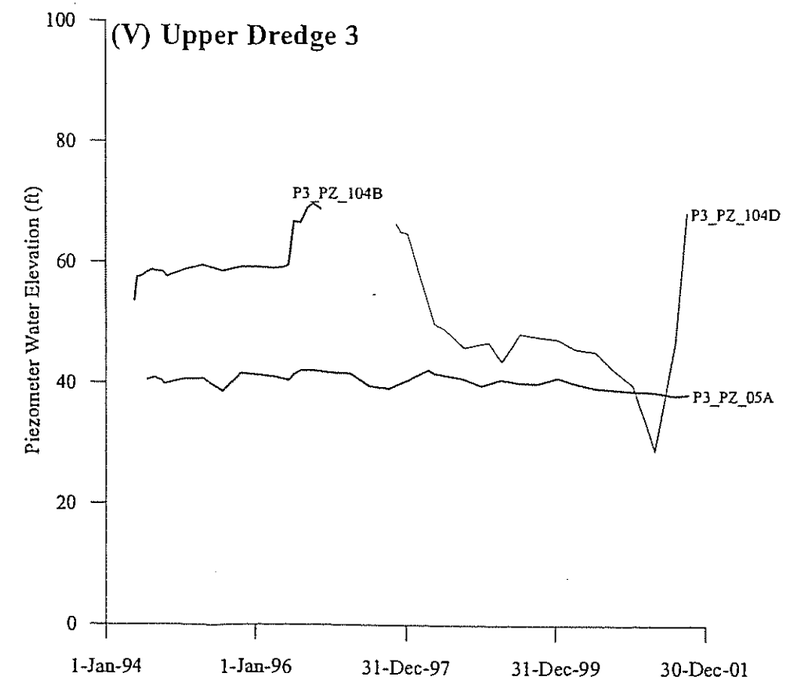


4. Elevation of geologic strata shown in (I) are approximate and are obtained from nearby boring P3_02_S012.
5. Only limited installation data is available for all of the wells with water level data shown on this Figure; ground surface and bottom of boring elevations are estimated.
6. Lines of piezometers shown in Figure 5-8.









Phase 3-3(2).grf

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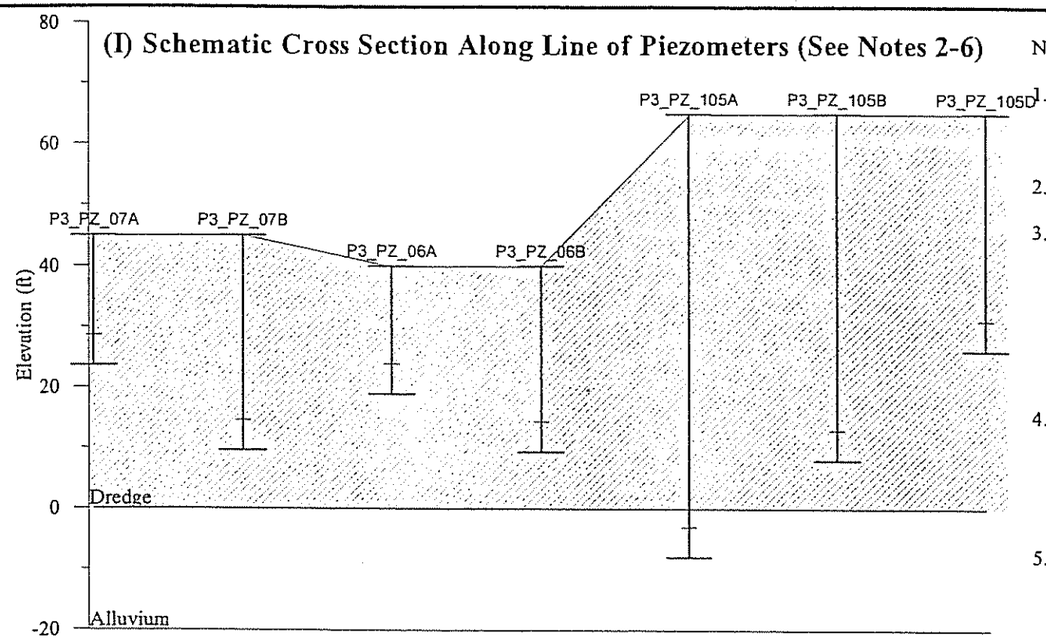
Piezometer Data
Phase 3, Line 3

FIGURE 5-11

PROJECT: ME0250 Cherry Island

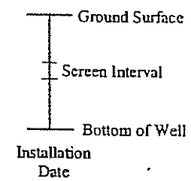


NOTES:

1. Data labeling system (as in GIS):
   P3_PZ_06A: Phase 3 Piezometer
   P3_SP_12: Phase 3 Settlement Plate

2. No horizontal scale in (I).

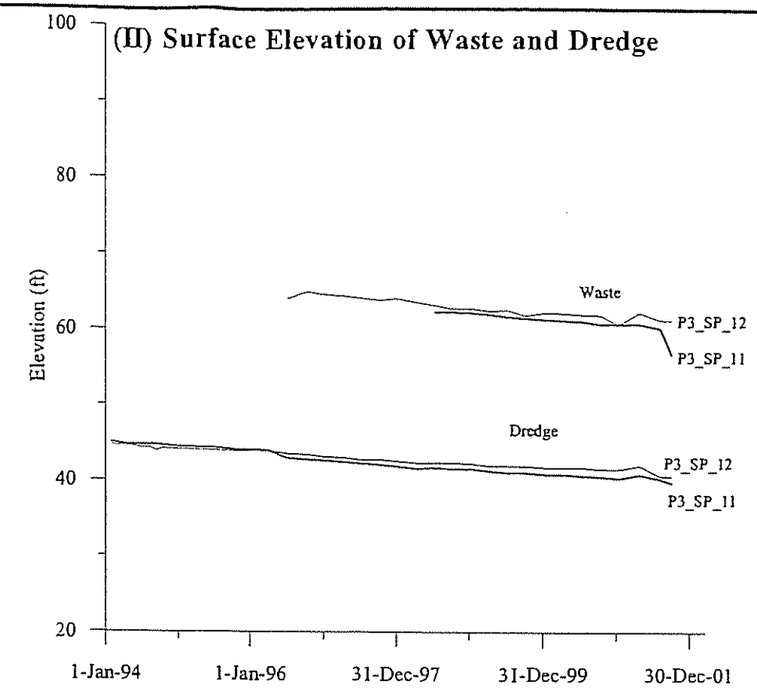
3. Key for (I):

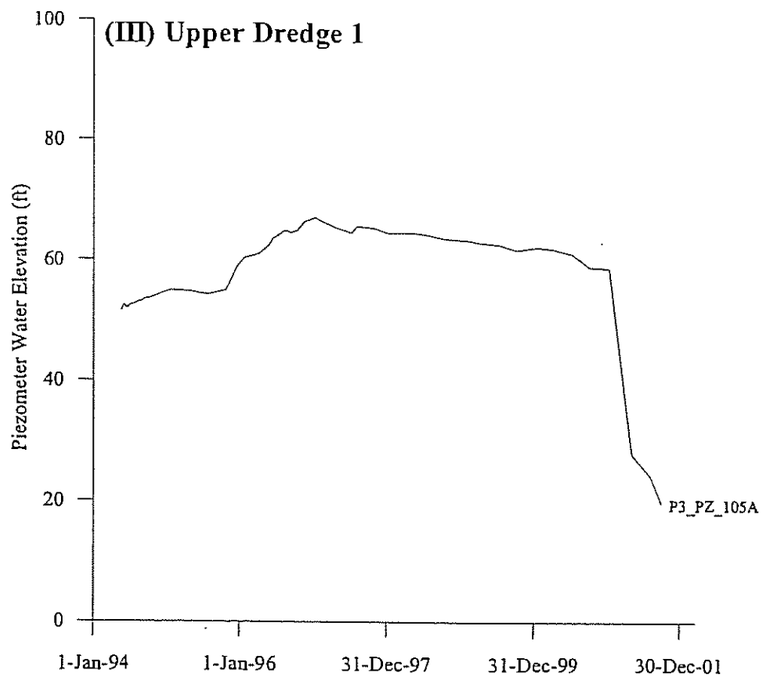


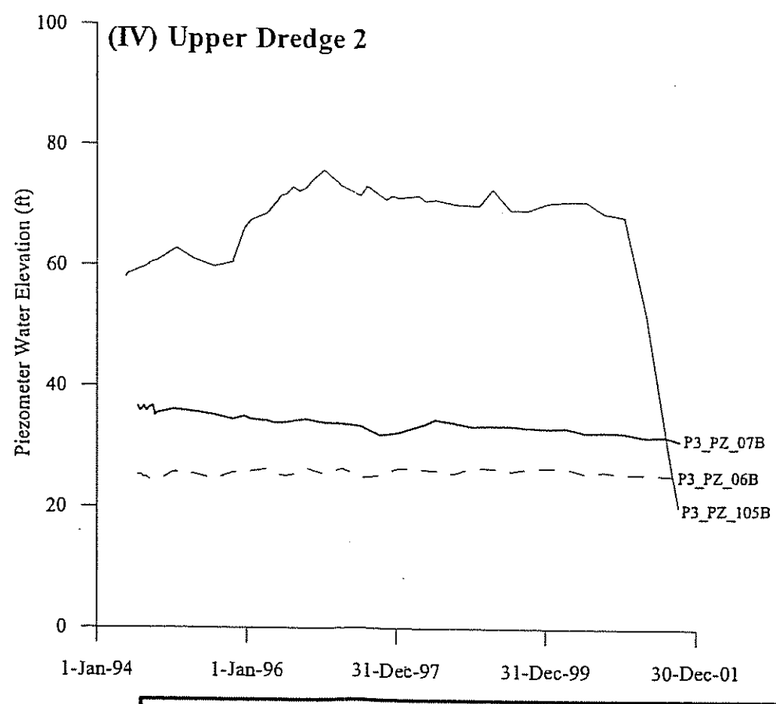
4. Elevation of geologic strata shown in (I) are approximate and are obtained from nearby boring SEA-12. Elevation of line between Columbia Formation and Potomac Formation is estimated from information in borehole log for P3_MW_P102.

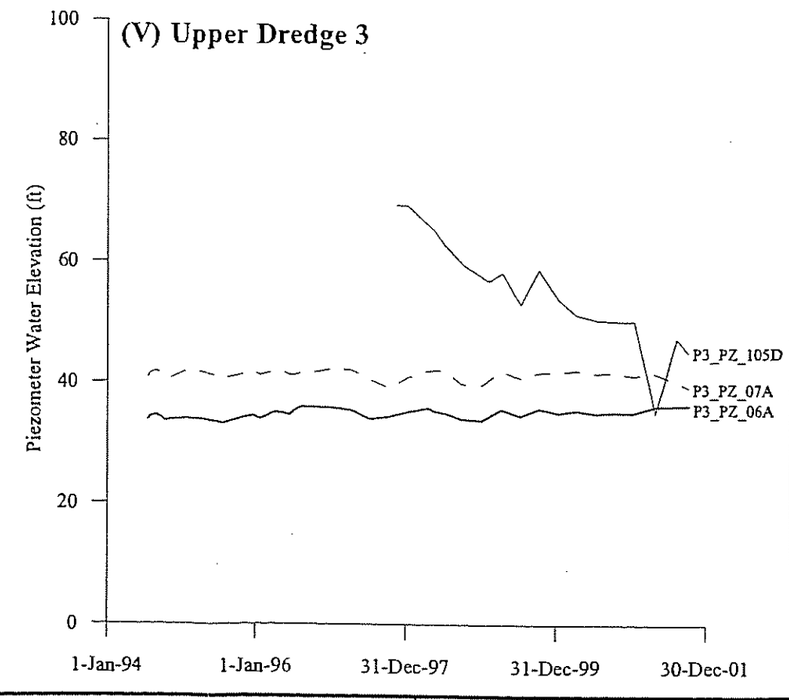
5. Only limited installation data is available for all of the wells with water level data shown on this Figure; ground surface and bottom of boring elevations are estimated.

6. Lines of piezometers shown in Figure 5-8.









Phase 3-4(2).grf

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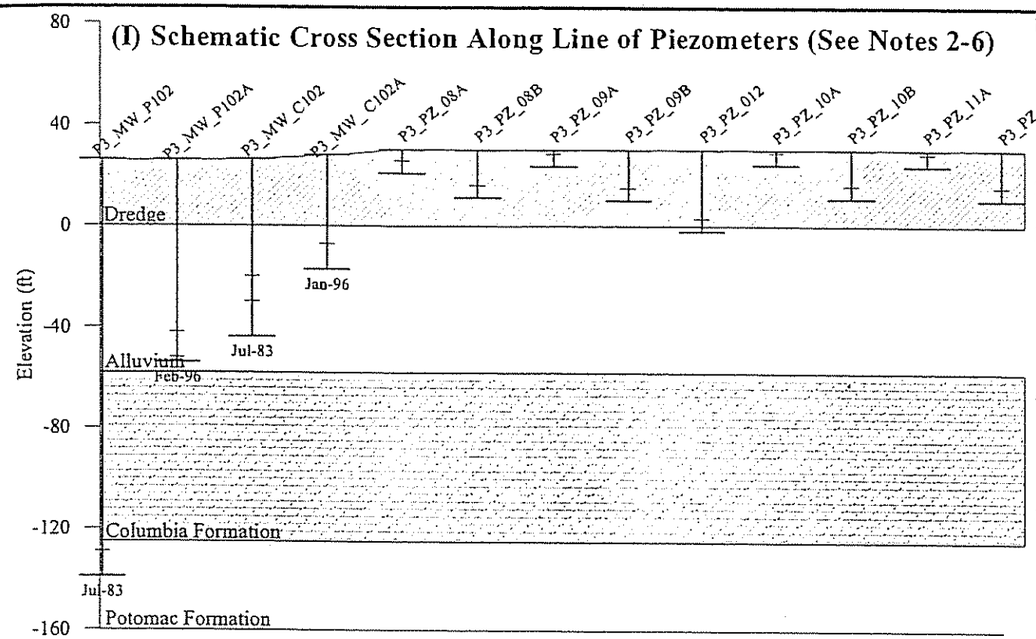
Piezometer Data
Phase 3, Line 4

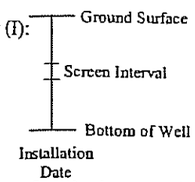
FIGURE 5-12

PROJECT: ME0250 Cherry Island

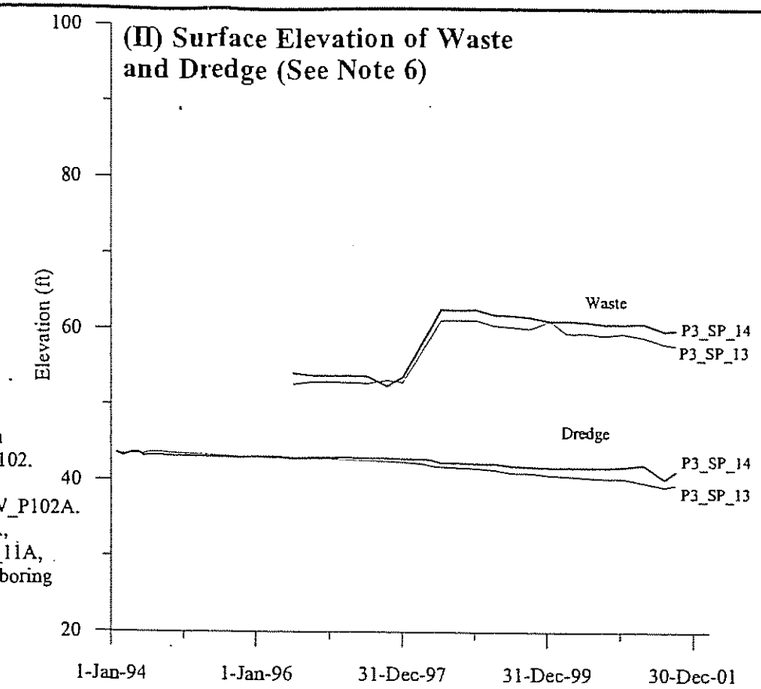


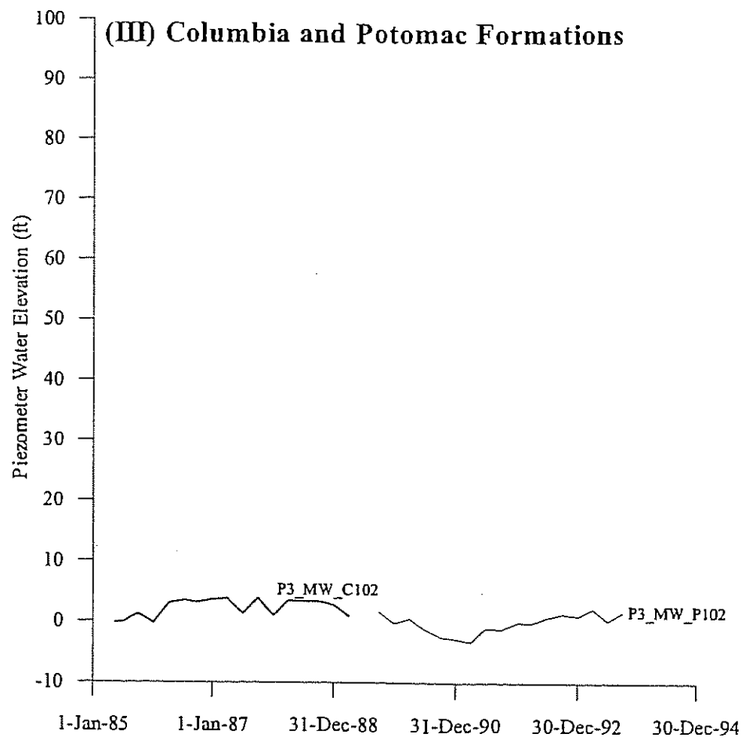
NOTES:

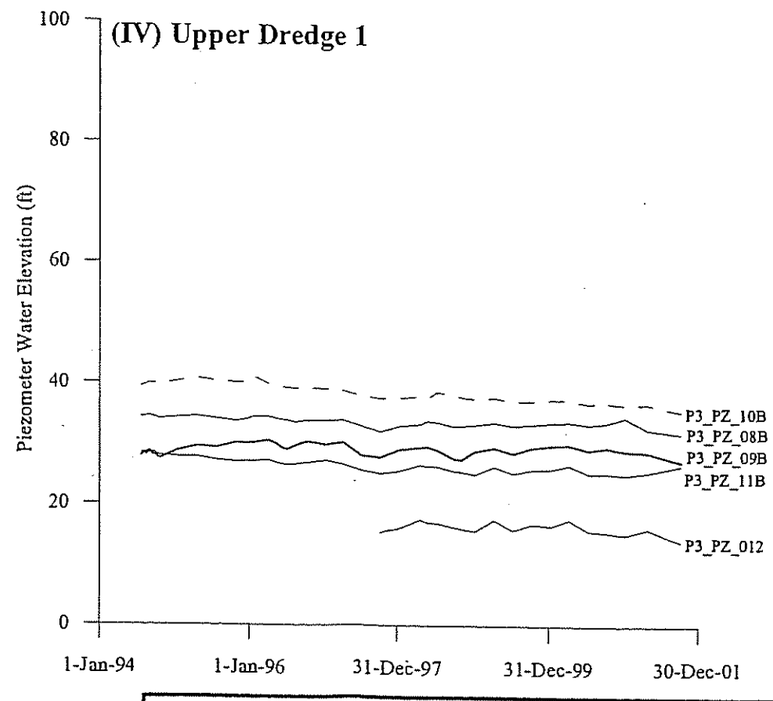
1. Data labeling system (as in GIS):
   P3_PZ_08A: Phase 3 Piezometer
   P3_MW_C102: Phase 3 Monitoring Well
   P3_SP_13: Phase 3 Settlement Plate
2. No horizontal scale in (I).
3. Key for (I):

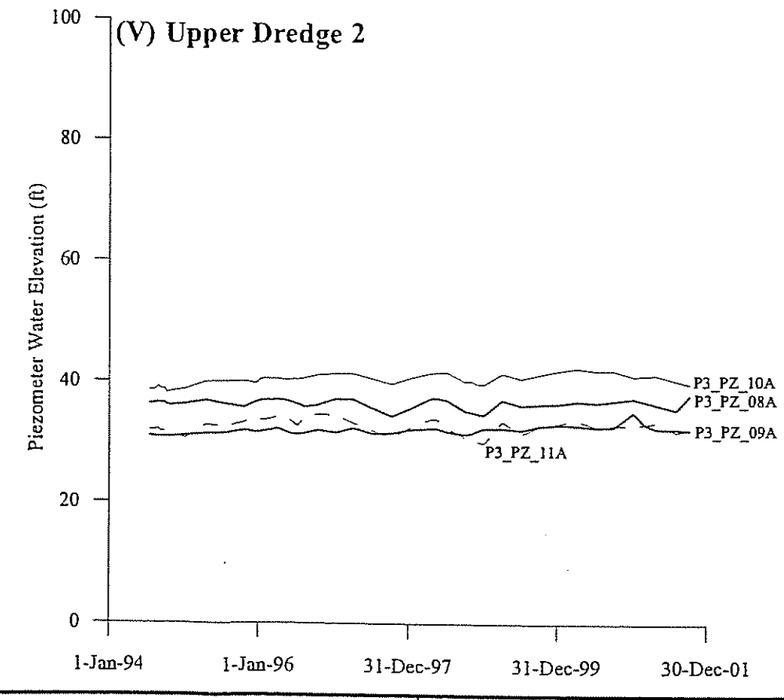


4. Elevation of geologic strata shown in (I) are approximate and are obtained from nearby boring SEA-12. Elevation of line between Columbia Formation and Potomac Formation is estimated from information in borehole log for P3_MW_P102.
5. There is no water level data for P3_MW_C102A and P3_MW_P102A. There is limited installation data for piezometers P3_PZ_08A, P3_PZ_08B P3_PZ_09B, P3_PZ_10A, P3_PZ_10B, P3_PZ_11A, P3_PZ_11B, and P3_PZ_012; ground surface and bottom of boring elevations are estimated.
6. Lines of piezometers shown in Figure 5-8.









NOTE: Time scale is different between (III), (IV), and (V).

Phase 3-5(2).grf

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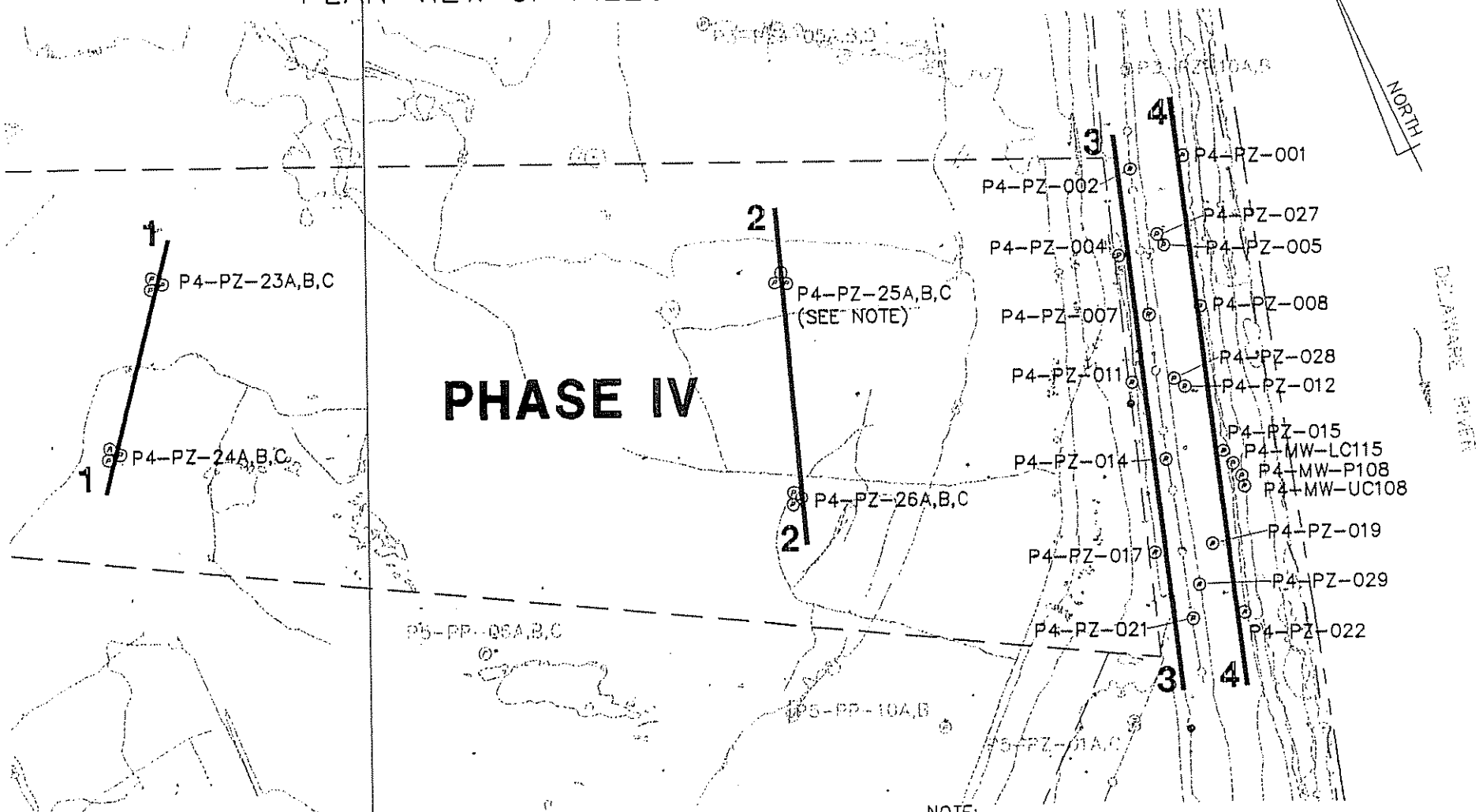
Piezometer Data
Phase 3, Line 5

FIGURE 5-13

PROJECT: ME0250 Cherry Island

PLAN VIEW OF PIEZOMETRIC CLUSTERS IN PHASE IV



EXISTING GRADE CONTOUR

PHASE BOUNDARY

EXISTING PIEZOMETER

NOTE:

TO SIMPLIFY NOTATION, PIEZOMETERS LOCATED CLOSE TO EACH OTHER WITH SIMILAR LABELS HAVE BEEN CLUSTERED. FOR INSTANCE, P4-PZ-25A,B,C INDICATES P4-PZ-25A, P4-PZ-25B AND P4-PZ-25C.

NOT TO SCALE

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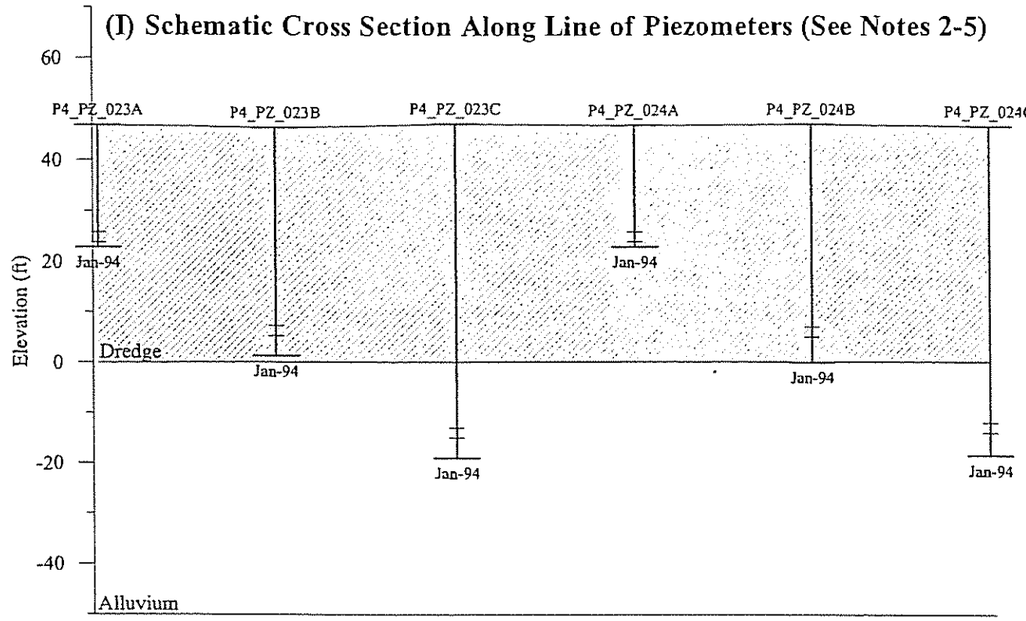
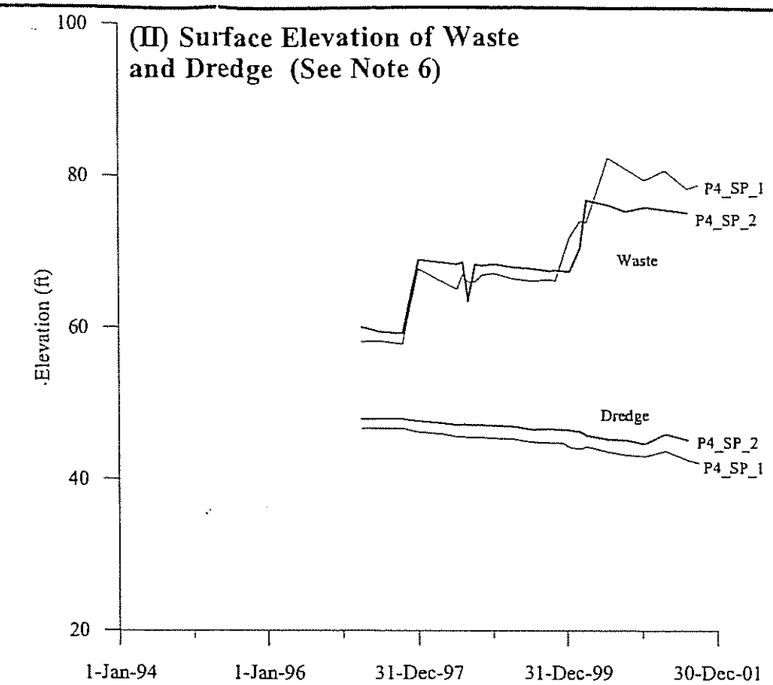
COLUMBIA, MARYLAND

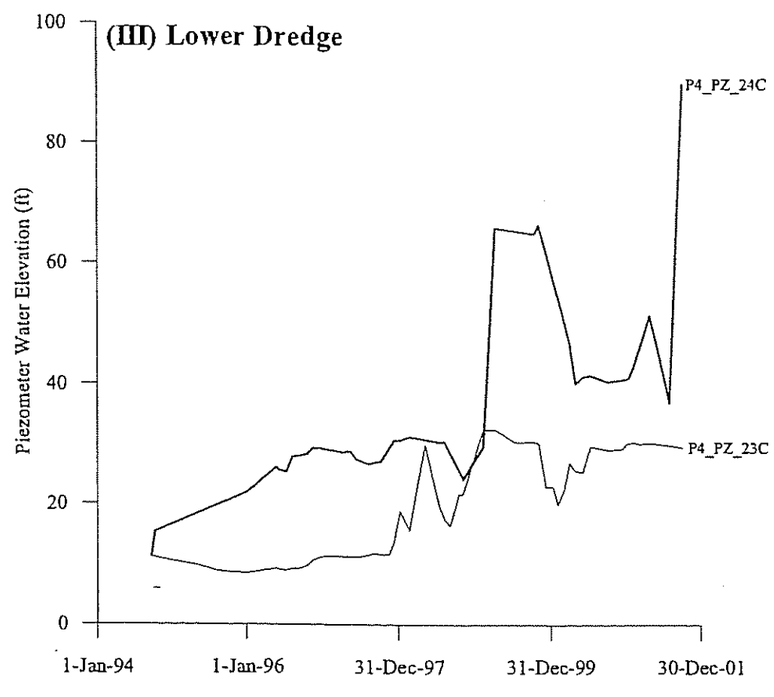
FIGURE NO.	5-14
PROJECT NO.	ME0250
DOCUMENT NO.	
FILE NO.	0250F517

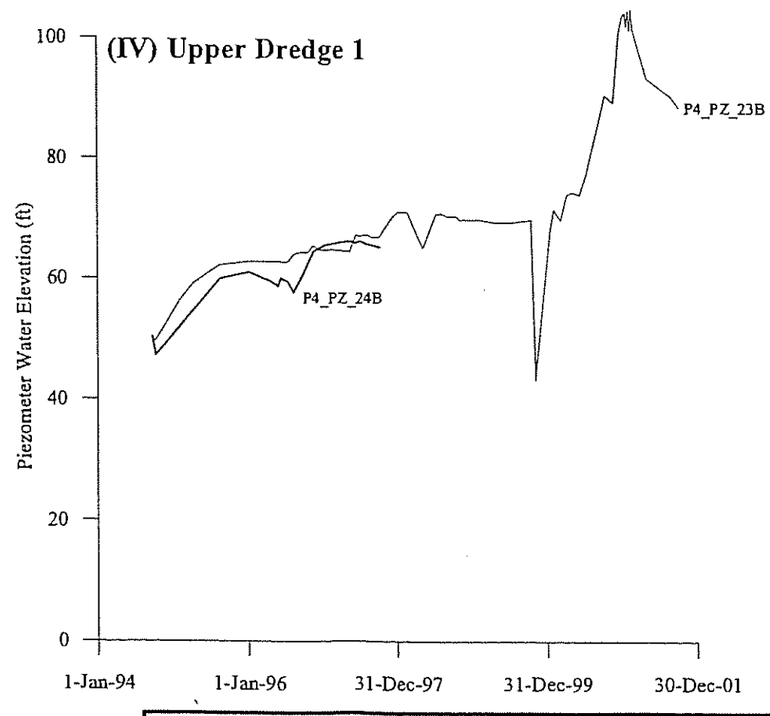


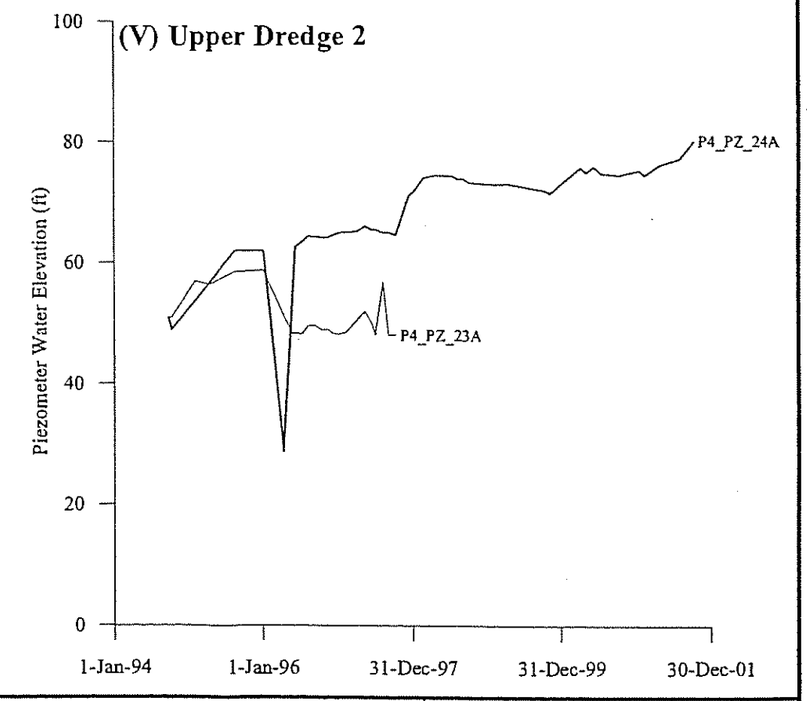
NOTES:

1. Data labeling system (as in GIS):
   P4_PZ_023A: Phase 4 Piezometer
   P4_SP_1: Phase 4 Settlement Plate
2. No horizontal scale in (I).
3. Key for (I): Ground Surface; Screen Interval; Bottom of Well; Installation Date
4. Elevation of geologic strata shown in (I) are approximate and are obtained from nearby boring tb-1.
5. Lines of piezometers shown in Figure 5-14.
6. Data point in dashed portion of dredge surface elevation lines in (II) was obtained from average surface elevation of dredge when piezometers in (I) were installed.









Phase 4-1(2).grf

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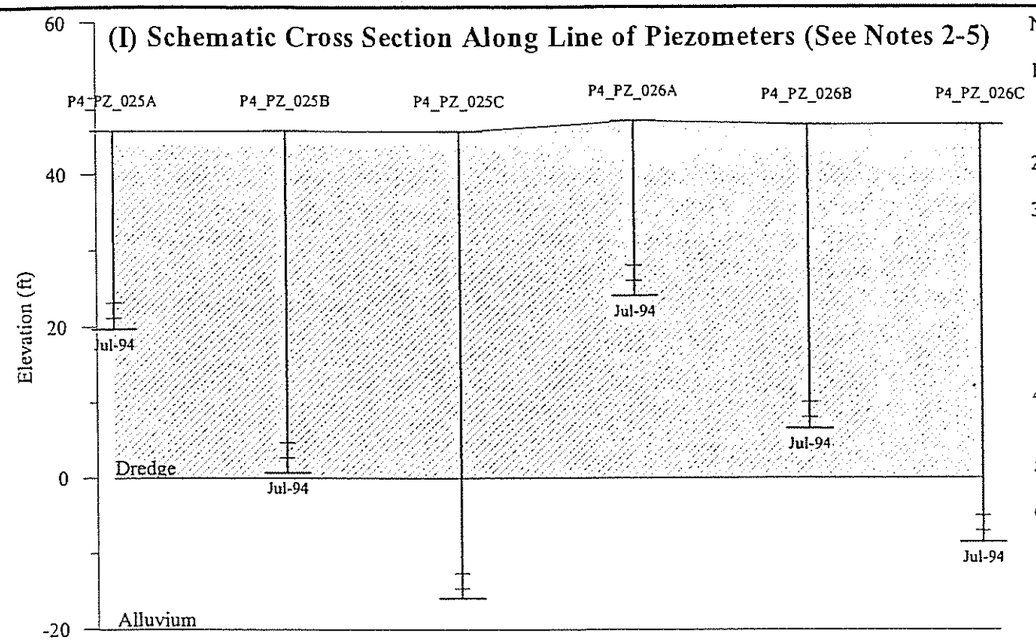
Piezometer Data
Phase 4, Line 1

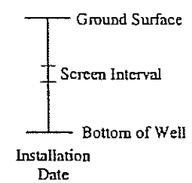
FIGURE 5-15

PROJECT: ME0250 Cherry Island

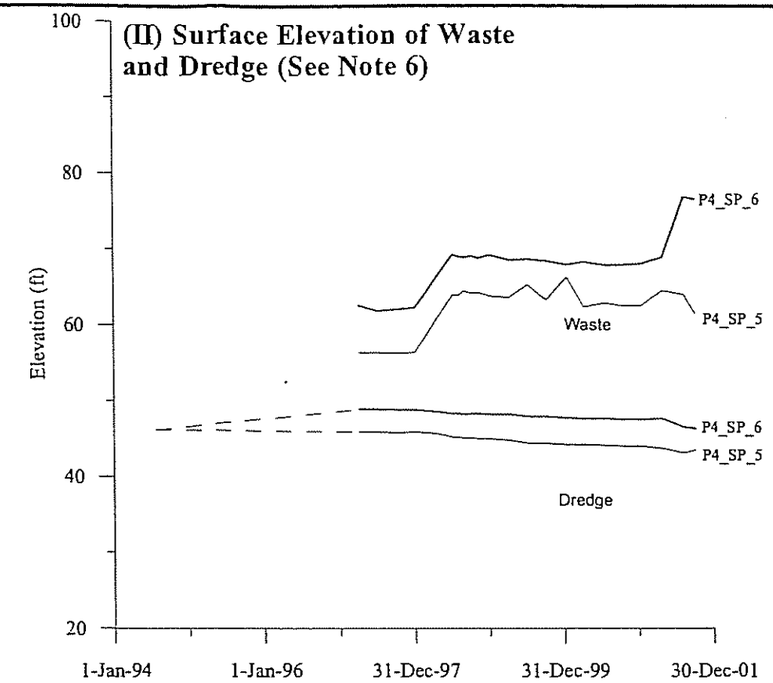


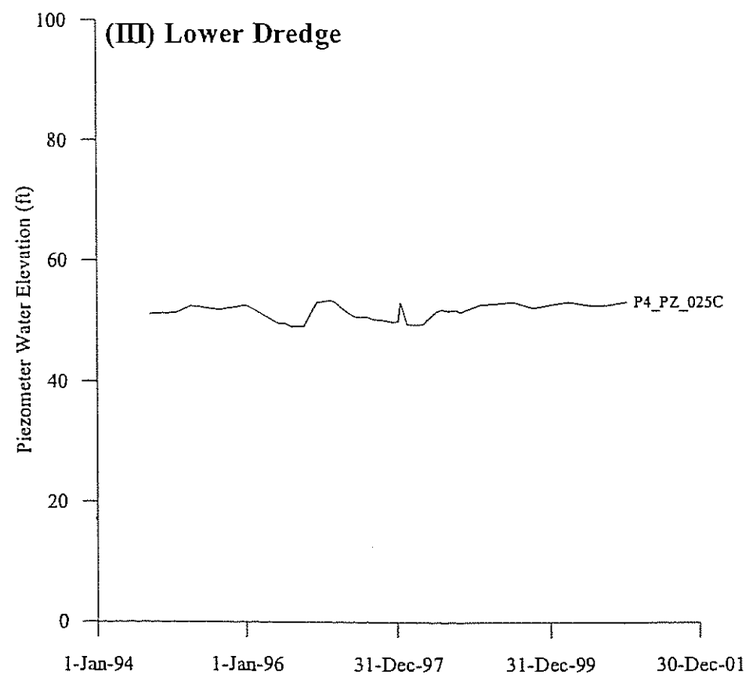
NOTES:

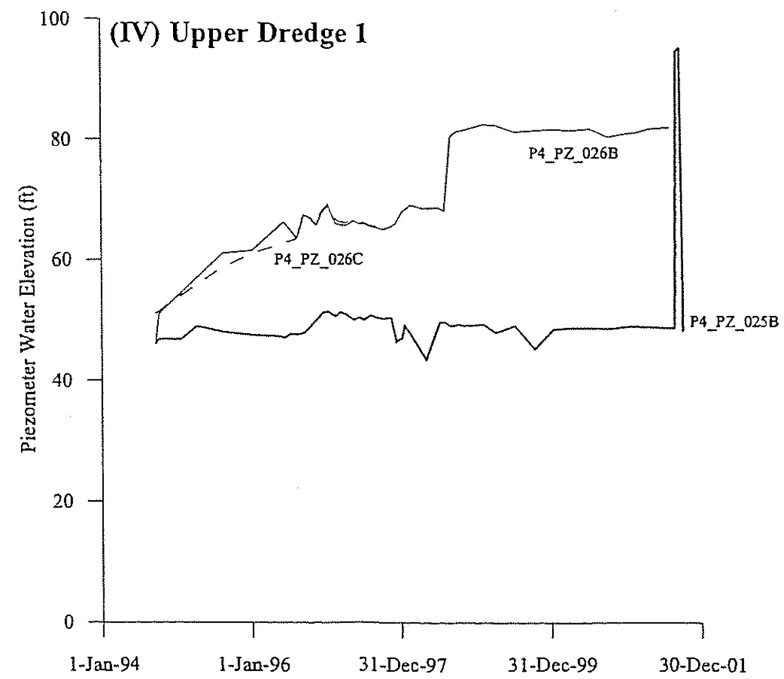
1. Data labeling system (as in GIS):
   P4_PZ_025A: Phase 4 Piezometer
   P4_SP_5: Phase 4 Settlement Plate
2. No horizontal scale in (I).
3. Key for (I):

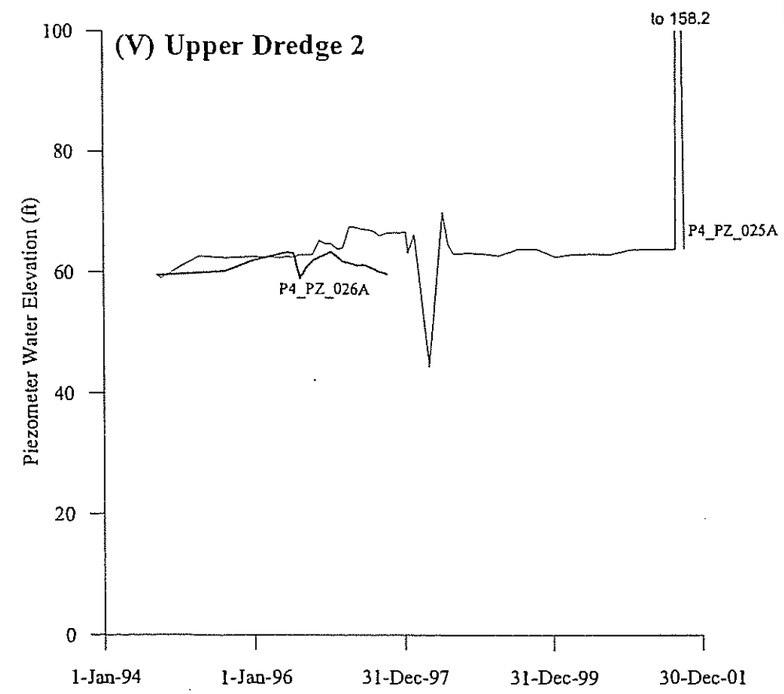


4. Elevation of geologic strata shown in (I) are approximate and are obtained from nearby boring tb-2.
5. Lines of piezometers shown in Figure 5-14.
6. Data point in dashed portion of dredge surface elevation lines in (II) was obtained from average surface elevation of dredge when piezometers in (I) were installed.









Phase 4-2(2).grf

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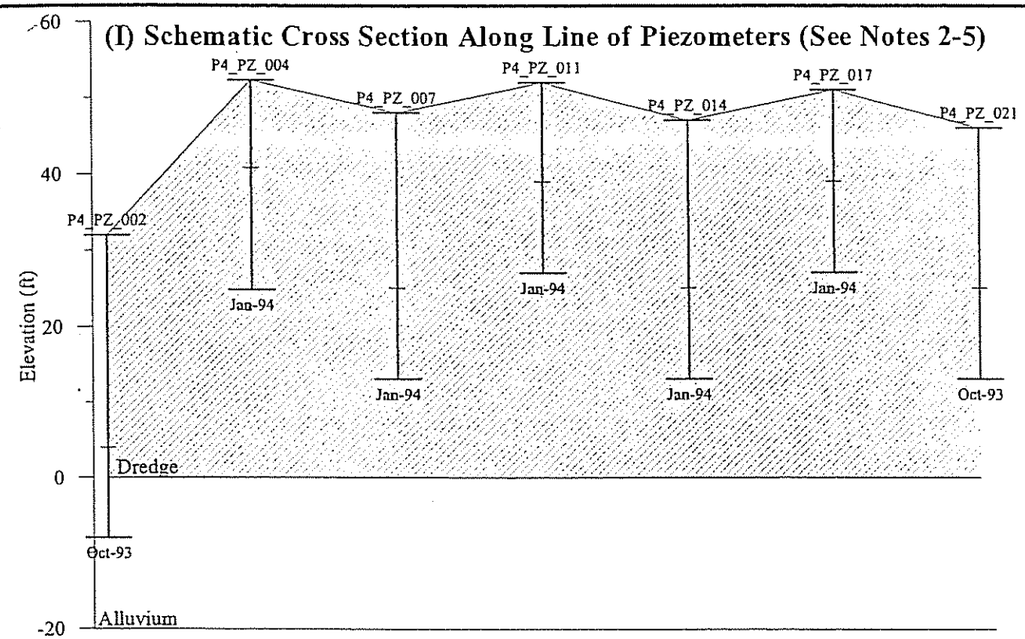
Piezometer Data
Phase 4, Line 2

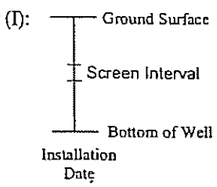
FIGURE 5-16

PROJECT: ME0250 Cherry Island

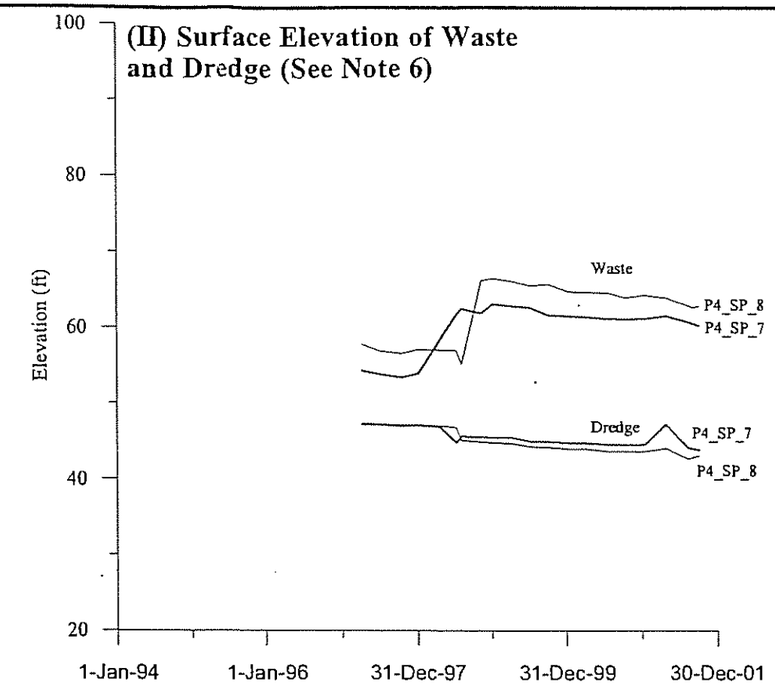


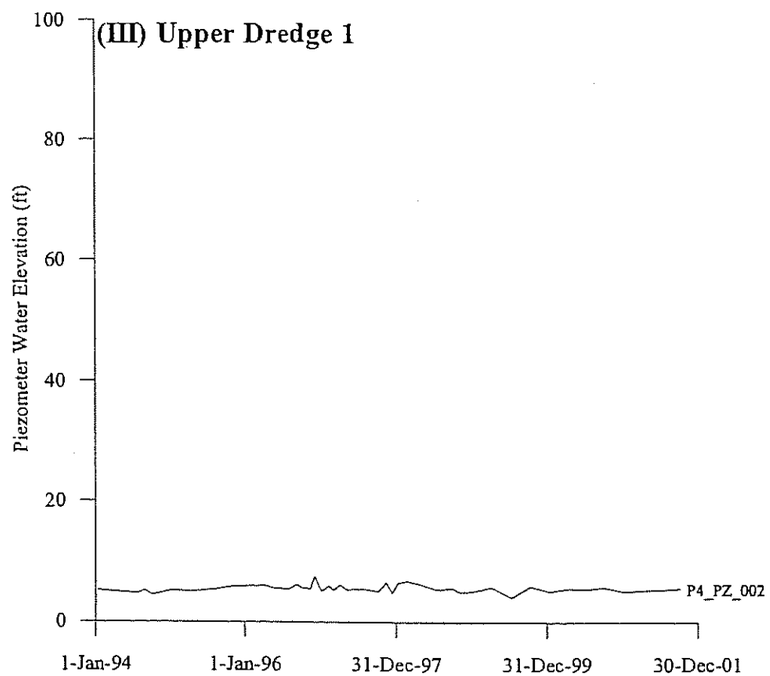
NOTES:

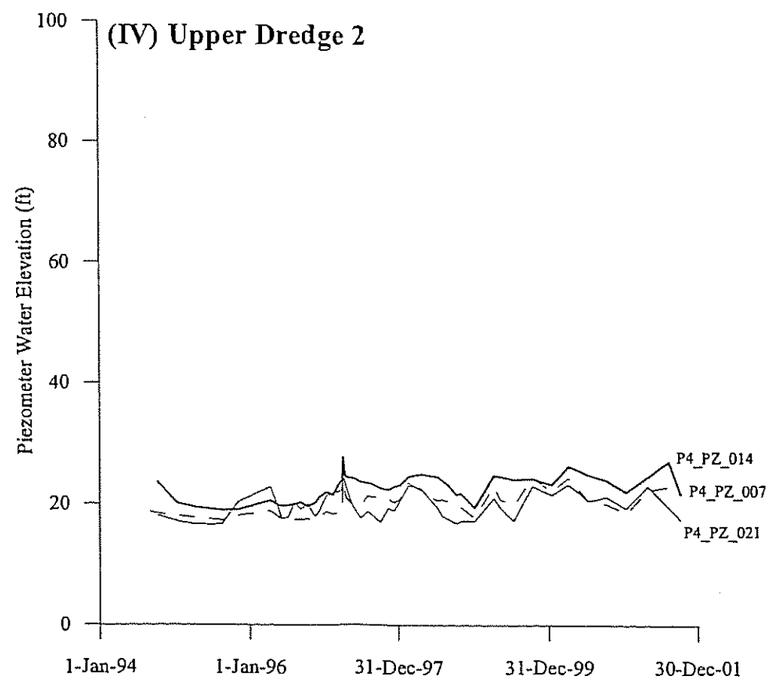
1. Data labeling system (as in GIS):
   P4_PZ_002: Phase 4 Piezometer
   P4_SP_7: Phase 4 Settlement Plate
2. No horizontal scale in (I).
3. Key for (I):

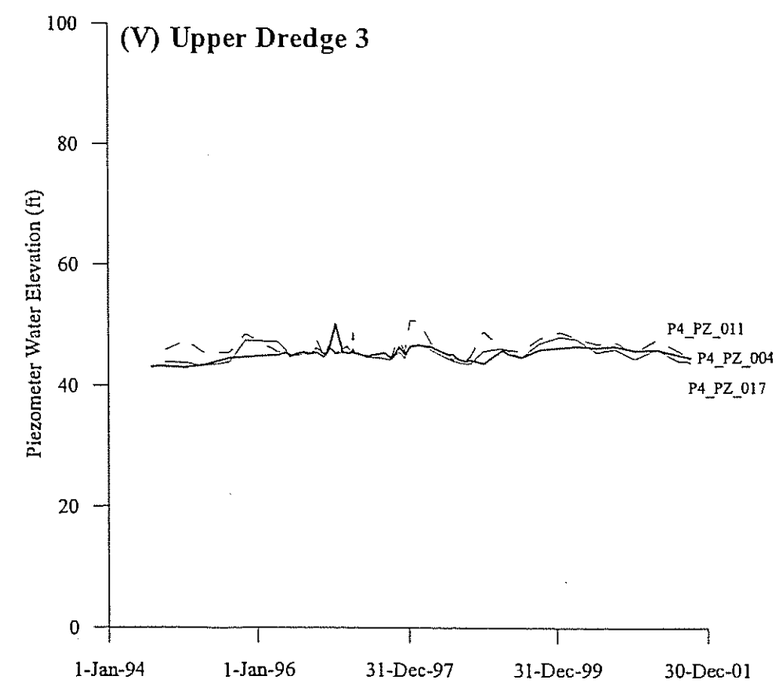


4. Elevation of geologic strata shown in (I) are approximate and are obtained from nearby boring tb-9.
5. Lines of piezometers shown in Figure 5-14.
6. Data points in dashed portion of dredge surface elevation lines in (II) was obtained from average surface elevation of dredge when piezometers in (I) were installed.









Phase 4-3.grf

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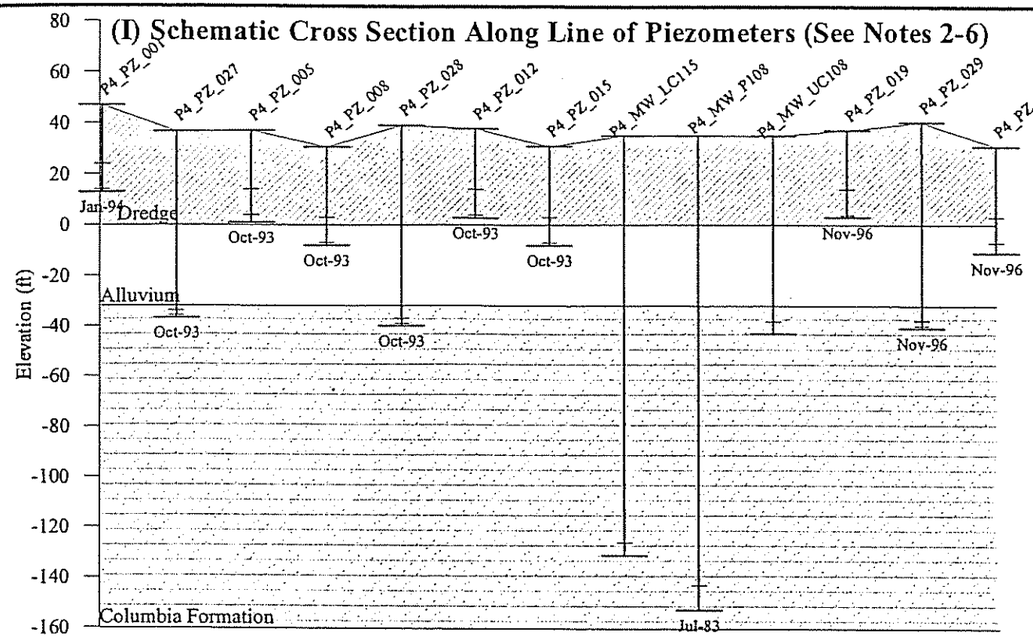
Piezometer Data
Phase 4, Line 3

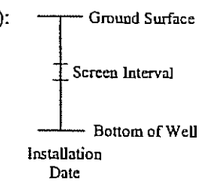
FIGURE 5-17

PROJECT: ME0250 Cherry Island

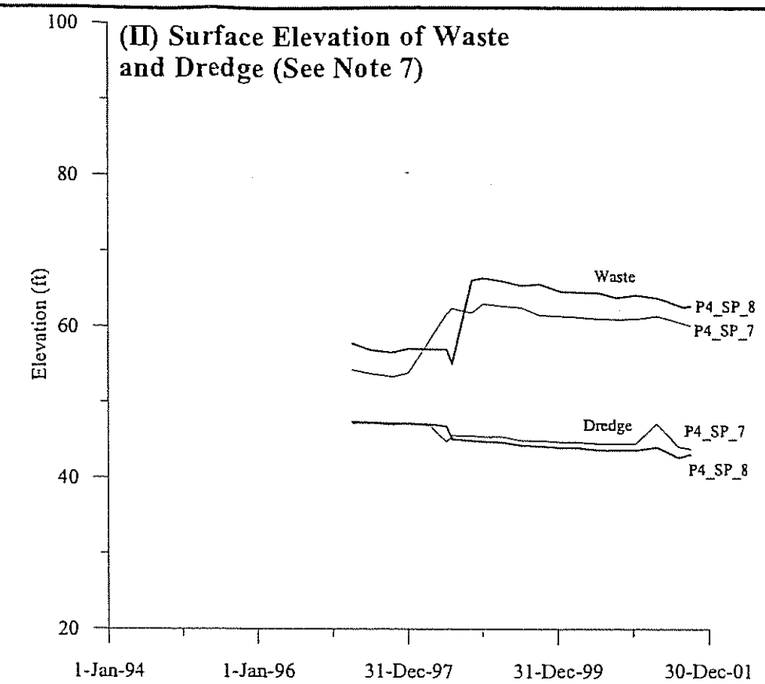


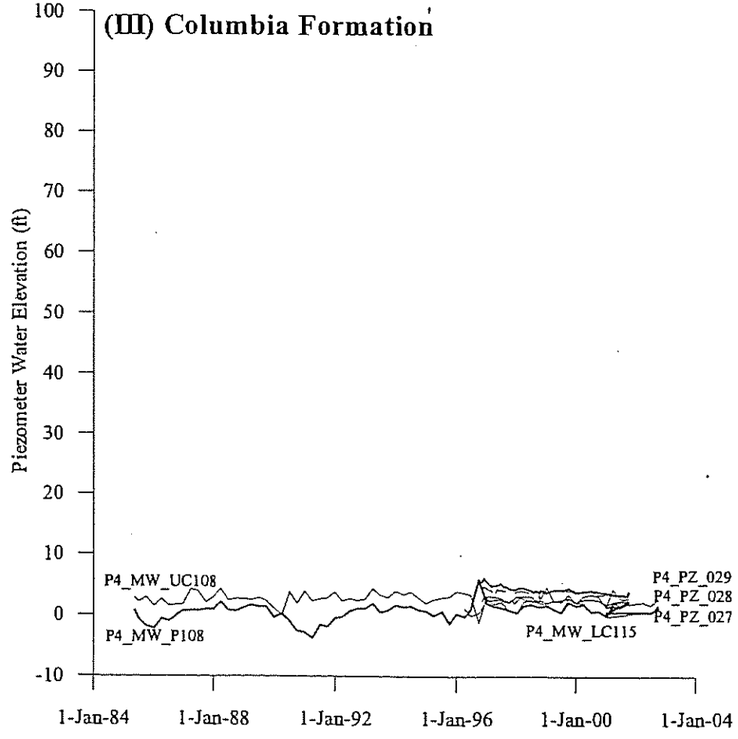
NOTES:

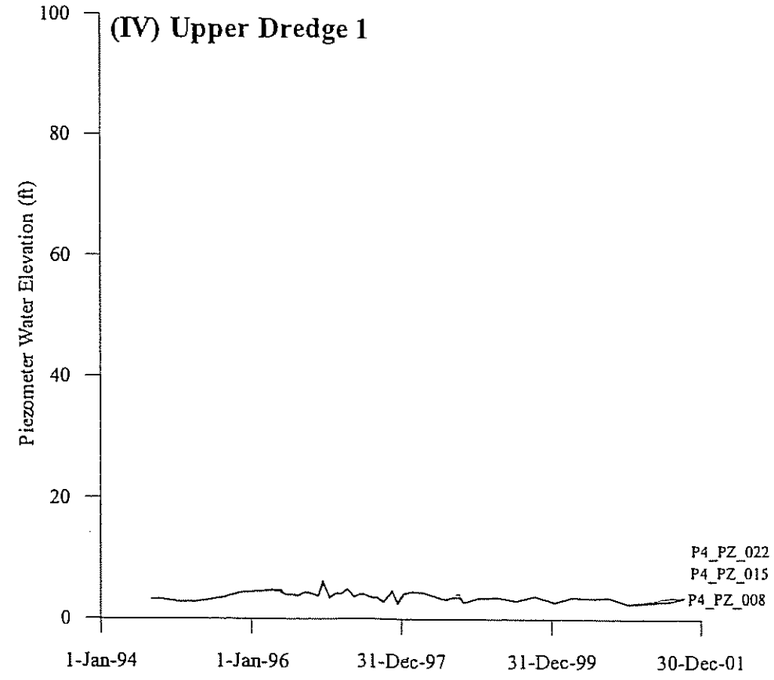
1. Data labeling system (as in GIS):
   P4_PZ_001: Phase 4 Piezometer
   P4_SP_8: Phase 4 Settlement Plate
2. No horizontal scale in (I).
3. Key for (I):

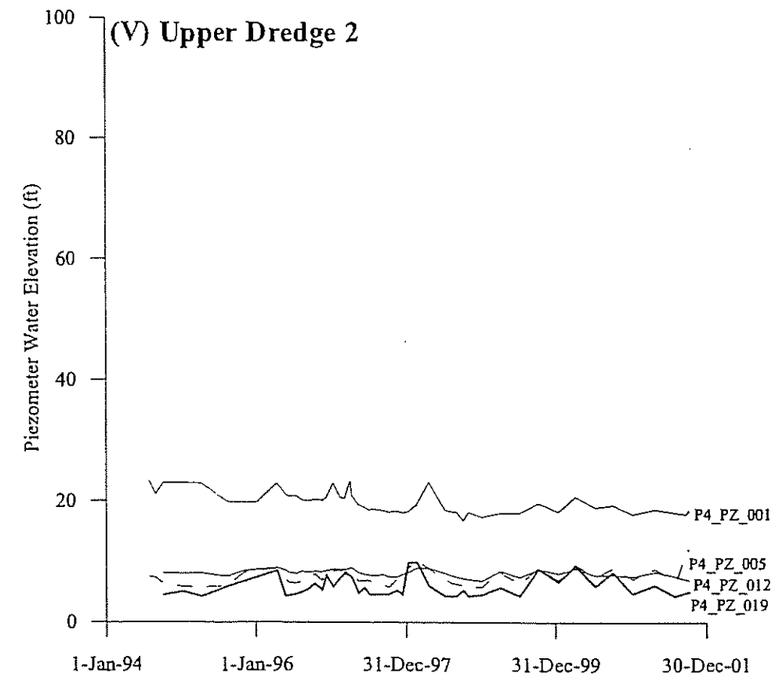


4. Elevation of geologic strata shown in (I) are approximate and are obtained from nearby boring tb-7.
5. Lines of piezometers sbown in Figure 5-14.
6. Additional water level data for 1983 is available along this line.
7. Data points in dashed portion of dredge surface elevation lines in (II) were obtained from average surface elevation of dredge when piezometers in (I) were installed.









Phase 4-4.grf(2)

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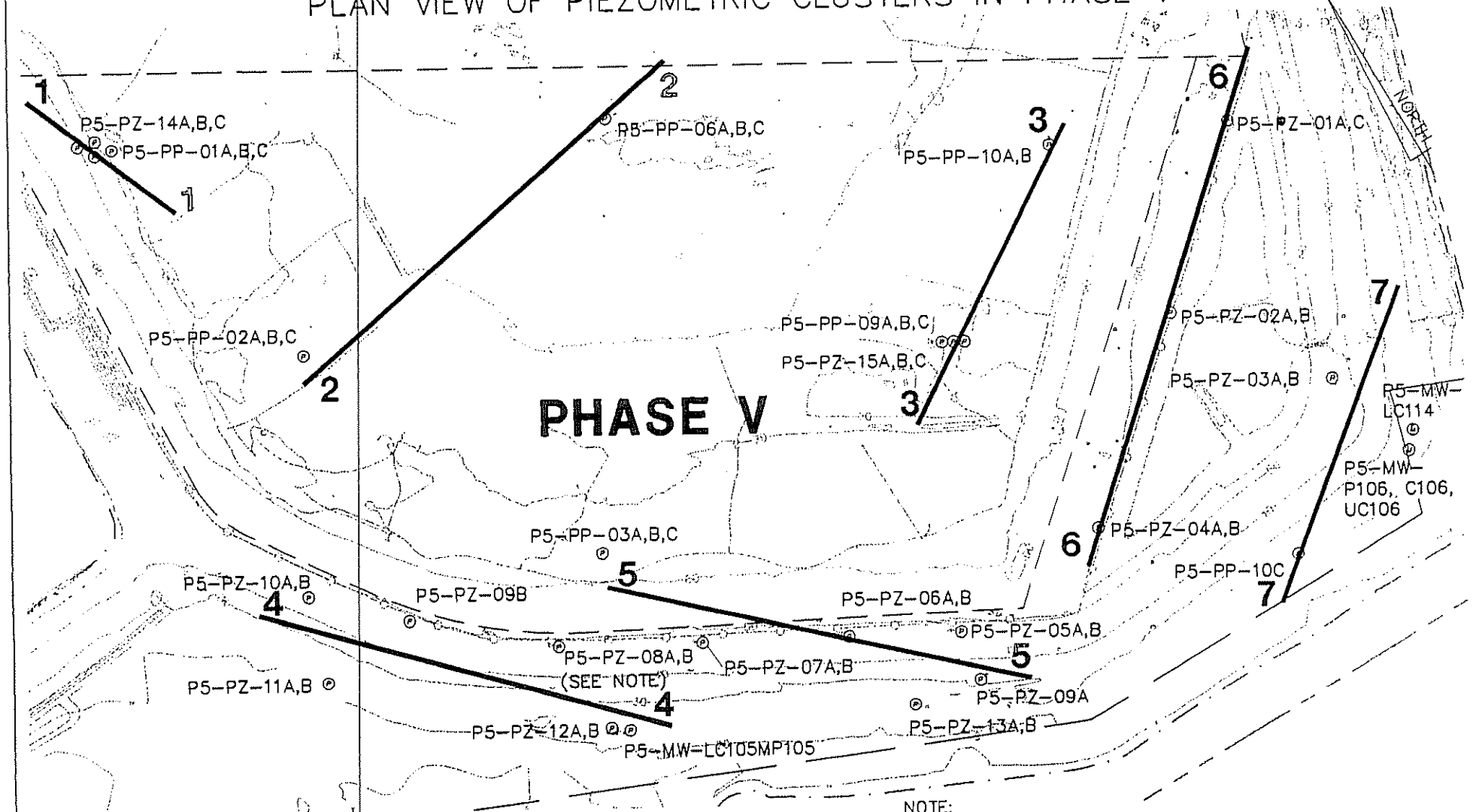
Piezometer Data
Phase 4, Line 4

FIGURE 5-18

PROJECT: ME0250 Cherry Island

PLAN VIEW OF PIEZOMETRIC CLUSTERS IN PHASE V



EXISTING GRADE CONTOUR

PHASE BOUNDARY

EXISTING PIEZOMETER

NOTE:

TO SIMPLIFY NOTATION, PIEZOMETERS LOCATED CLOSE TO EACH OTHER WITH SIMILAR LABELS HAVE BEEN CLUSTERED. FOR INSTANCE, P5-PZ-08A,B INDICATES P5-PZ-08A AND P5-PZ-08B.

NOT TO SCALE

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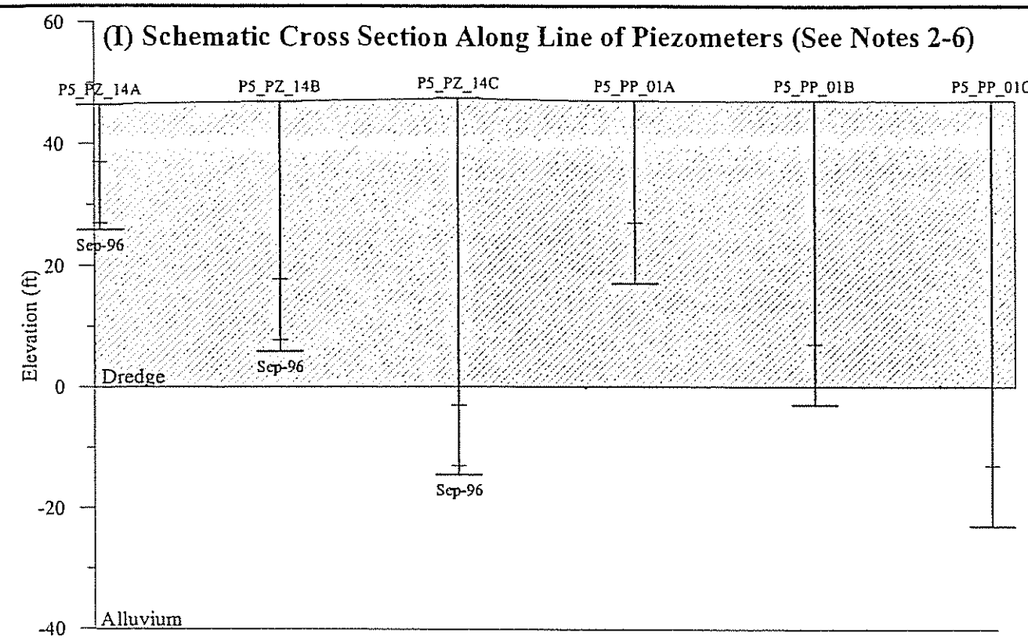
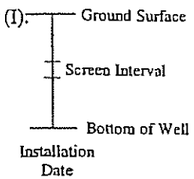
FIGURE NO.	5-19
PROJECT NO.	ME0250
DOCUMENT NO.	
FILE NO.	0250F518

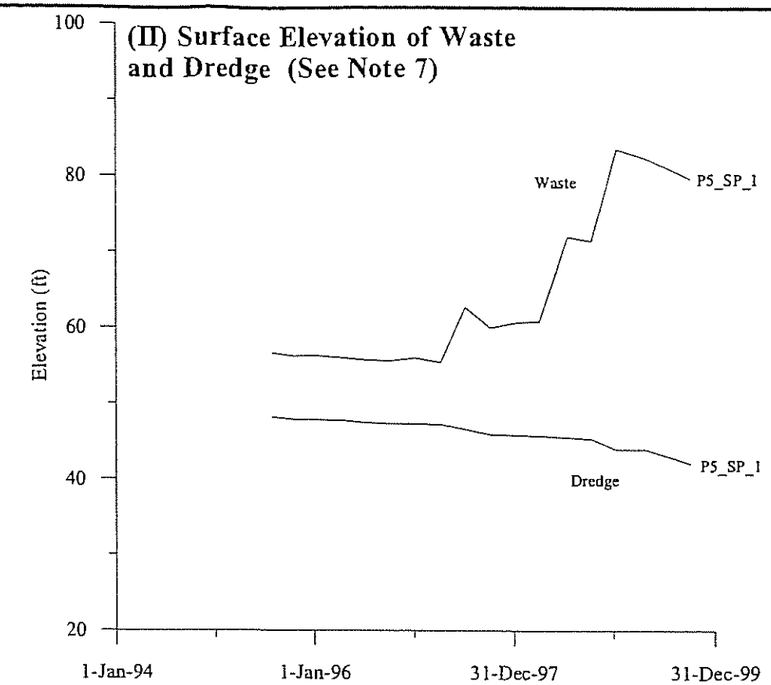


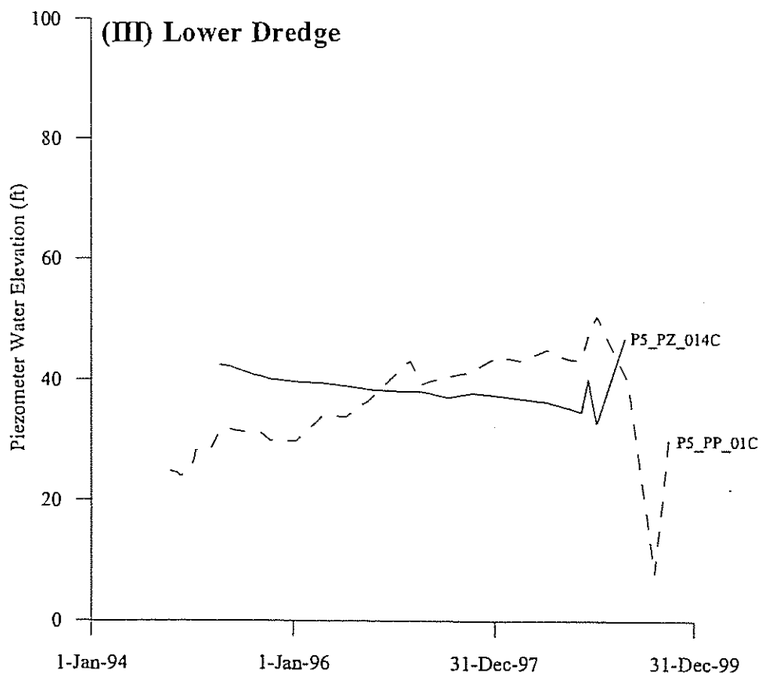
NOTES:

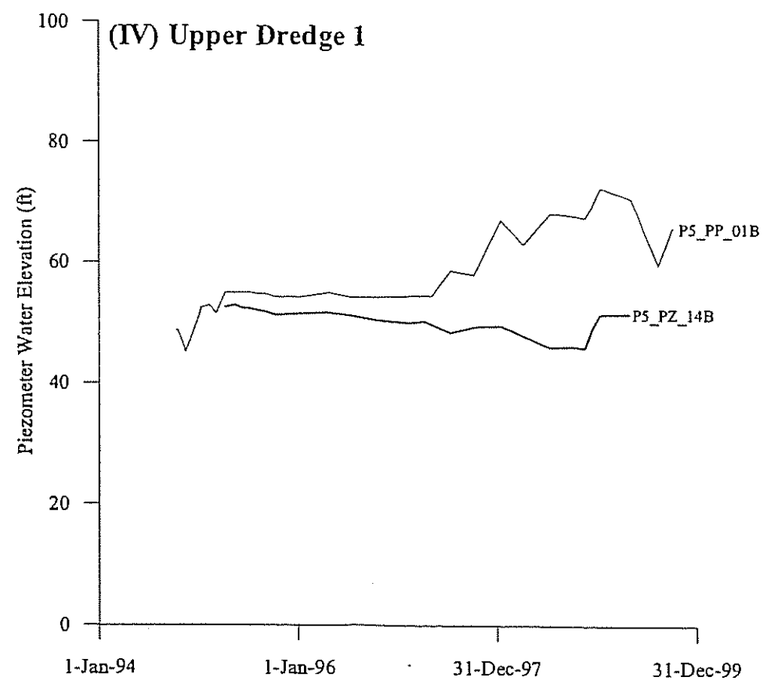
1. Data labeling system (as in GIS):
   P5_PZ_014A: Phase 5 Piezometer
   P5_SP_1: Phase 5 Settlement Plate
2. No horizontal scale in (I).
3. Key for (I):

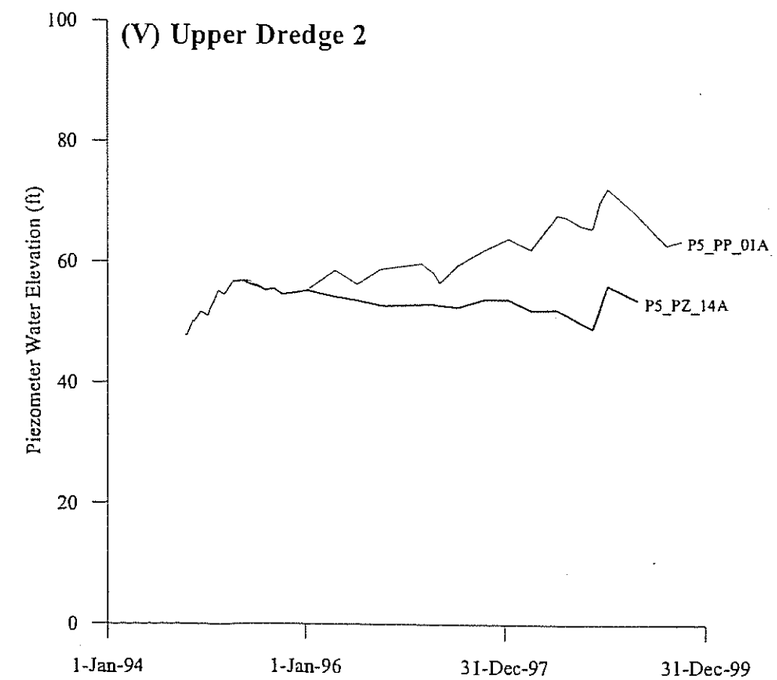


4. Elevation of geologic strata shown in (I) are approximate and are obtained from nearby boring tb-5.
5. Lines of piezometers shown in Figure 5-19.
6. Installation data was not available for wells P5_PP_01A, P5_PP_01B, and P5_PP_01C; however, data for these wells is shown in (III), (IV), and (V).
7. Data point in dashed portion of dredge surface elevation lines in (II) was obtained from average surface elevation of dredge when piezometers in (I) were installed.









Phase 5-1(2).grf

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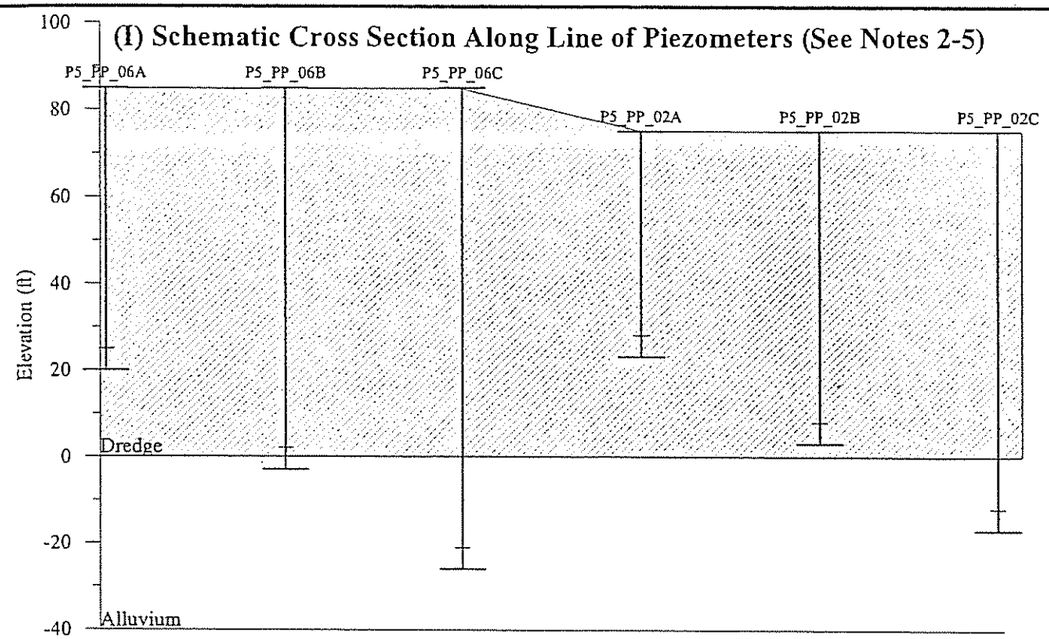
Piezometer Data
Phase 5, Line 1

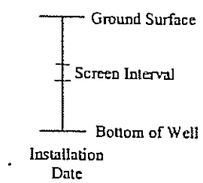
FIGURE 5-20

PROJECT: ME0250 Cherry Island

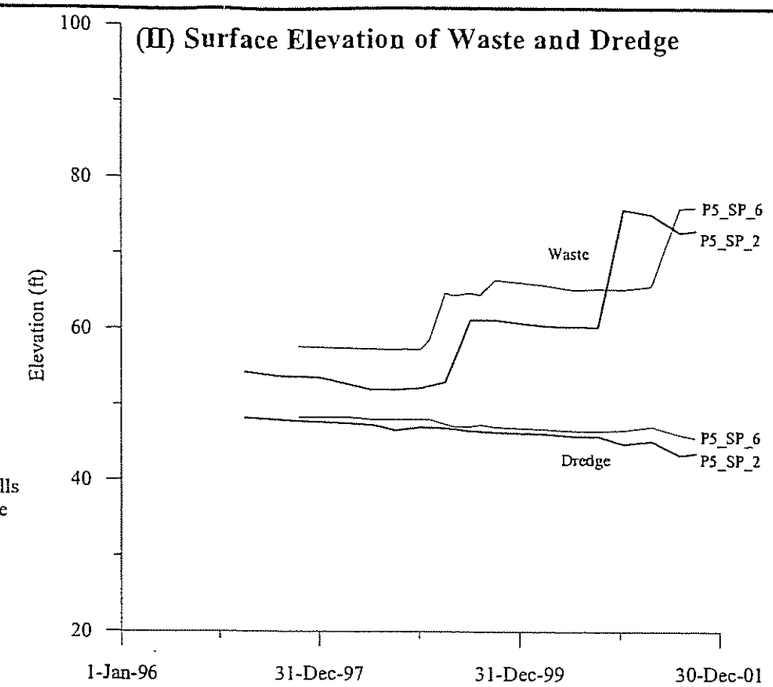


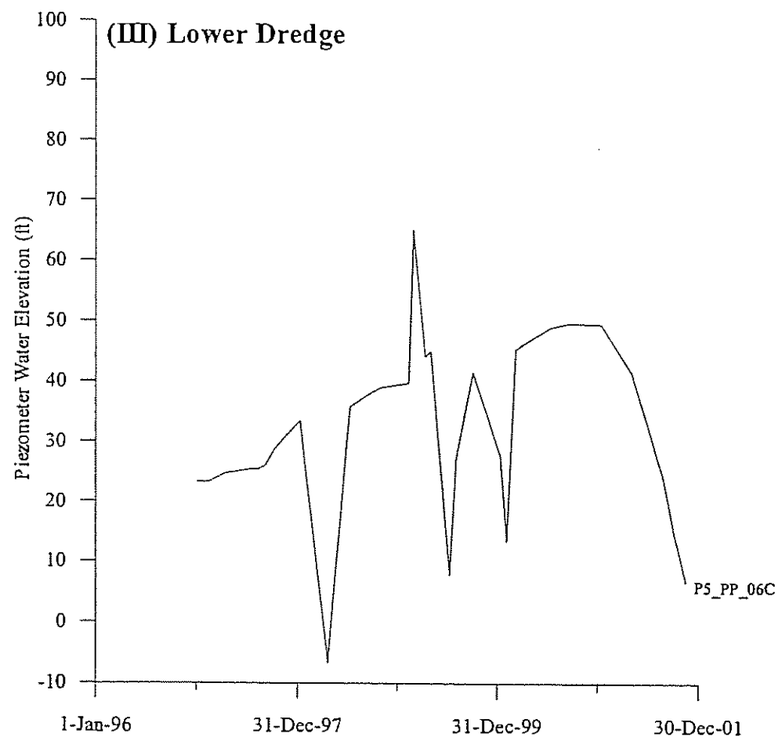
NOTES:

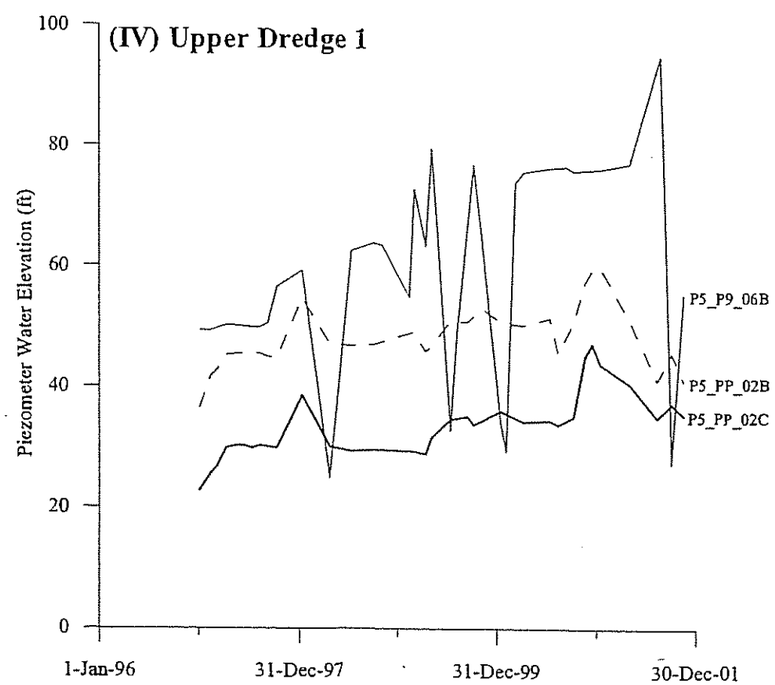
1. Data labeling system (as in GIS):
   P5_PZ_06A: Phase 5 Piezometer
   P5_SP_2: Phase 5 Settlement Plate
2. No horizontal scale in (I).
3. Key for (I):

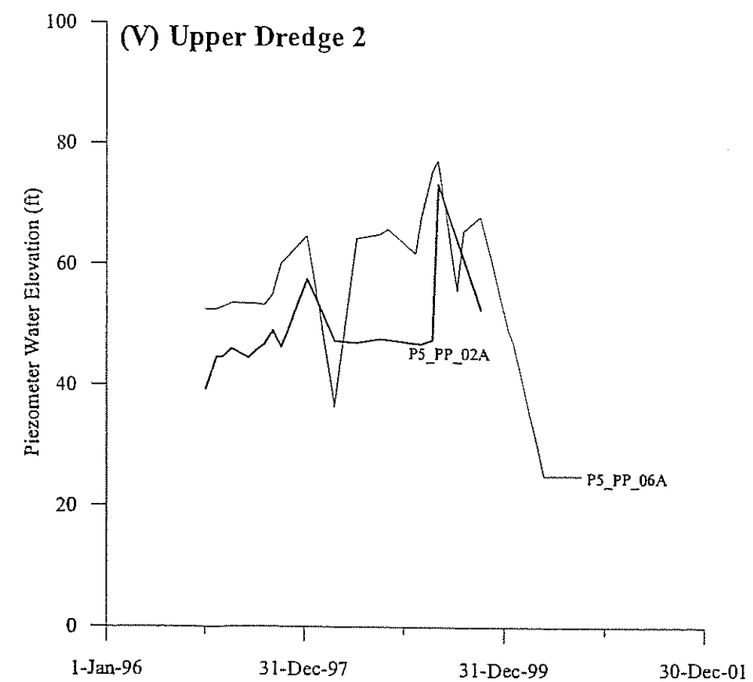


4. Elevation of geologic strata shown in (I) are approximate and are obtained from nearby boring tb-11.
5. Only limited installation data is available for all of the wells with water level data shown on this Figure; ground surface and bottom of boring elevations are estimated.
6. Lines of piezometers shown in Figure 5-19.









Phase 5-2(2).grf

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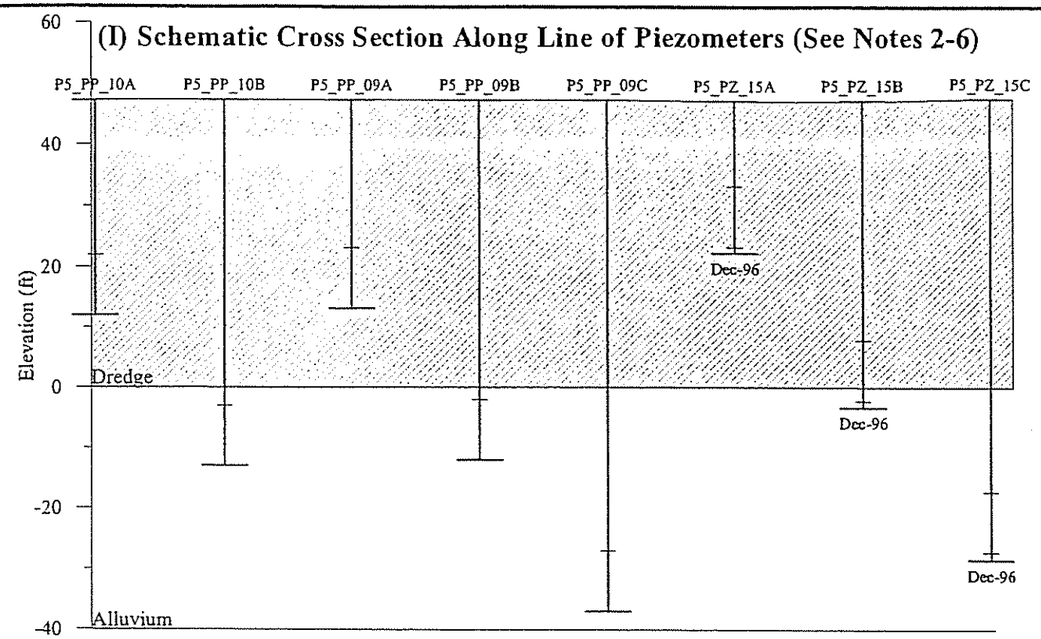
Piezometer Data
Phase 5, Line 2

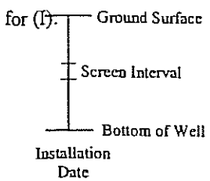
FIGURE 5-21

PROJECT: ME0250 Cherry Island

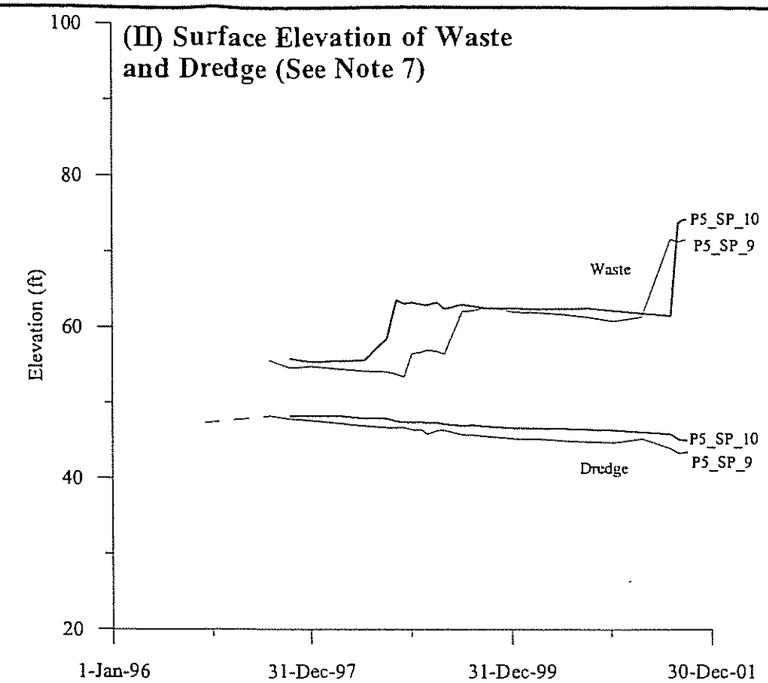


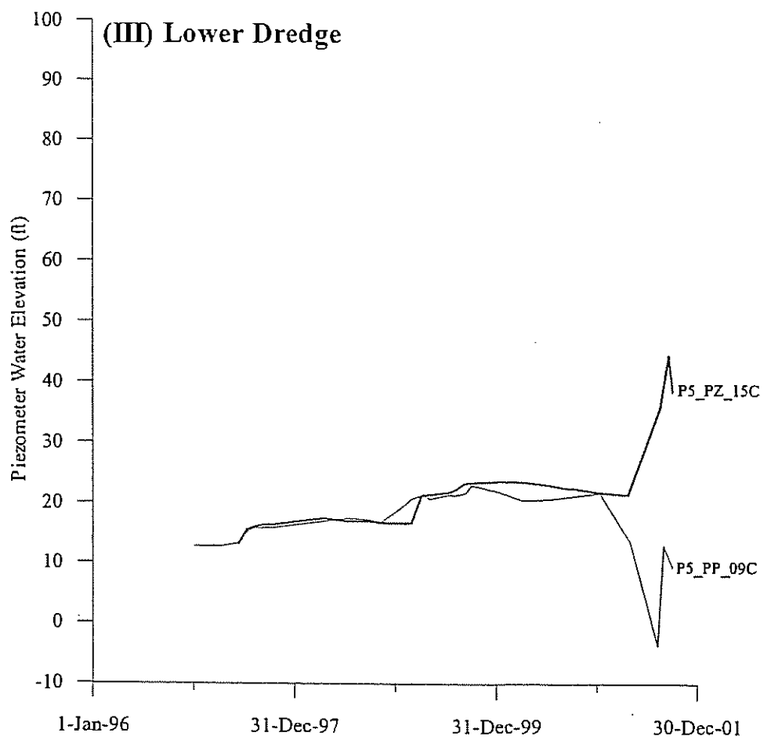
NOTES:

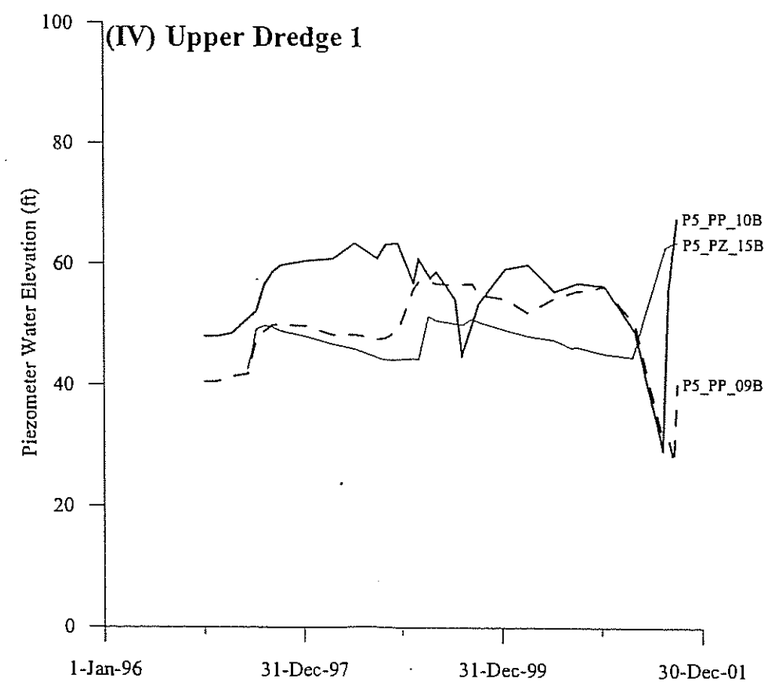
1. Data labeling system (as in GIS):
   P5_PZ_15A: Phase 5 Piezometer
   P5_SP_9: Phase 5 Settlement Plate
2. No horizontal scale in (I).
3. Key for (I):

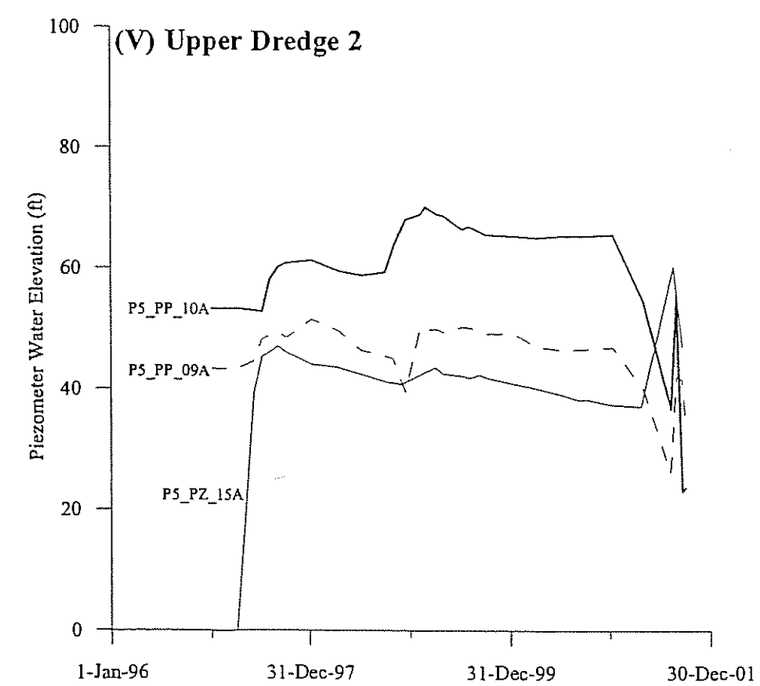


4. Elevation of geologic strata shown in (I) are approximate and are obtained from nearby boring tb-13.
5. Lines of piezometers shown in Figure 5-19.
6. Only limited well installation data is available for wells P5_PP_10A P5_PP_10B, P5_PP_09A, P5_PP_09B, and P5_PP_09C; ground surface and bottom of boring elevations are estimated.
7. Data point in dasbed portion of dredge surface elevation lines in (II) was obtained from average surface elevation of dredge when piezometers in (I) were installed.









Phase 5-3(2).grf

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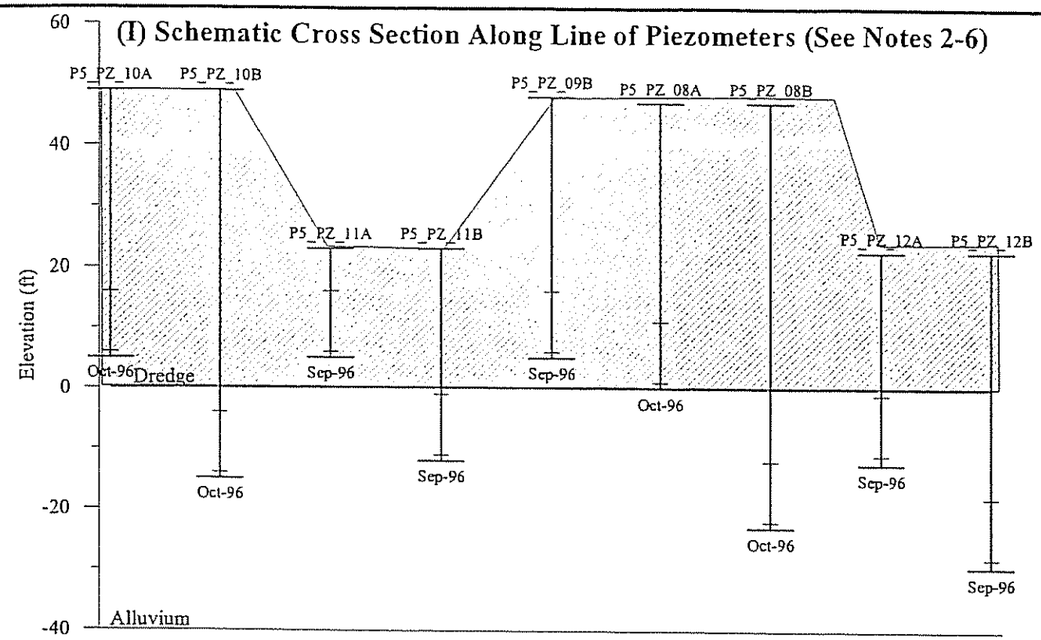
Piezometer Data
Phase 5, Line 3

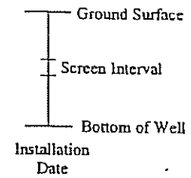
FIGURE 5-22

PROJECT: ME0250 Cherry Island

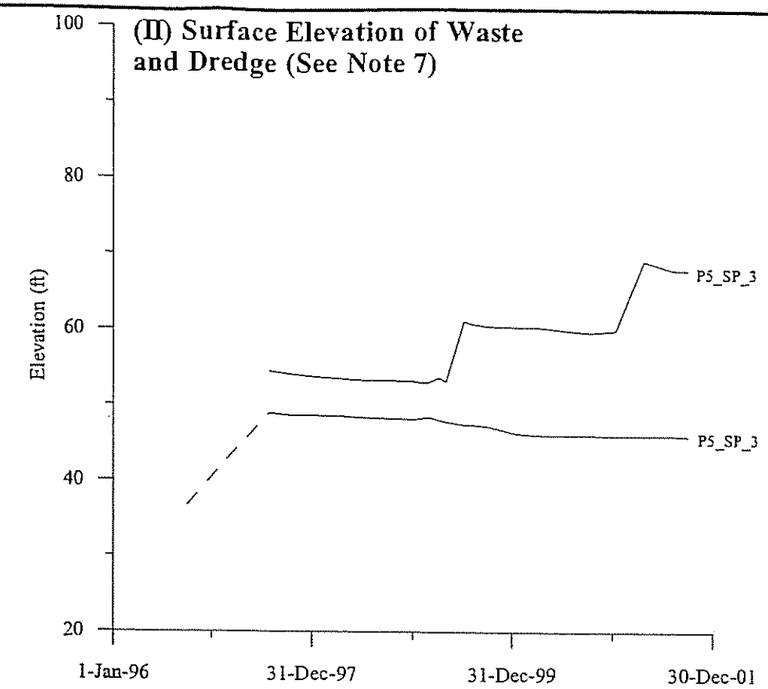


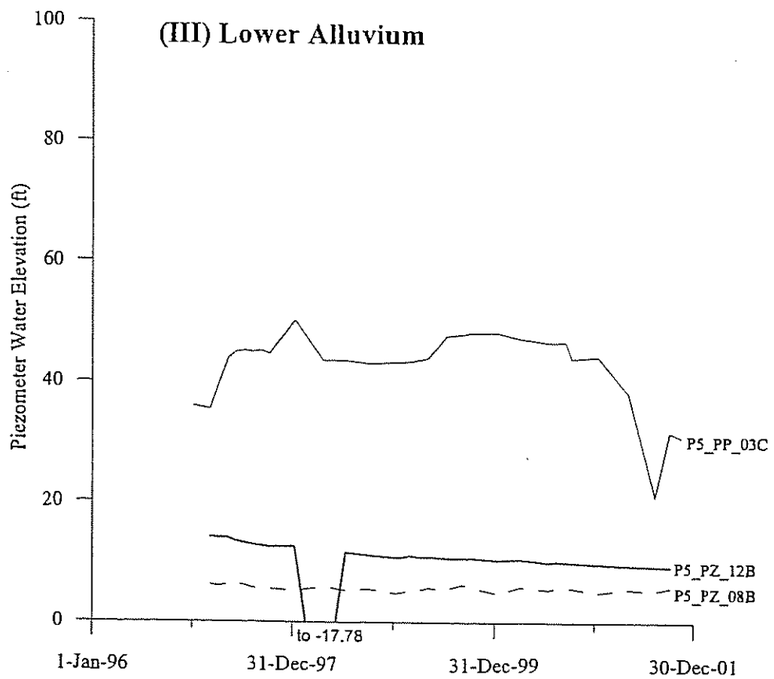
NOTES:

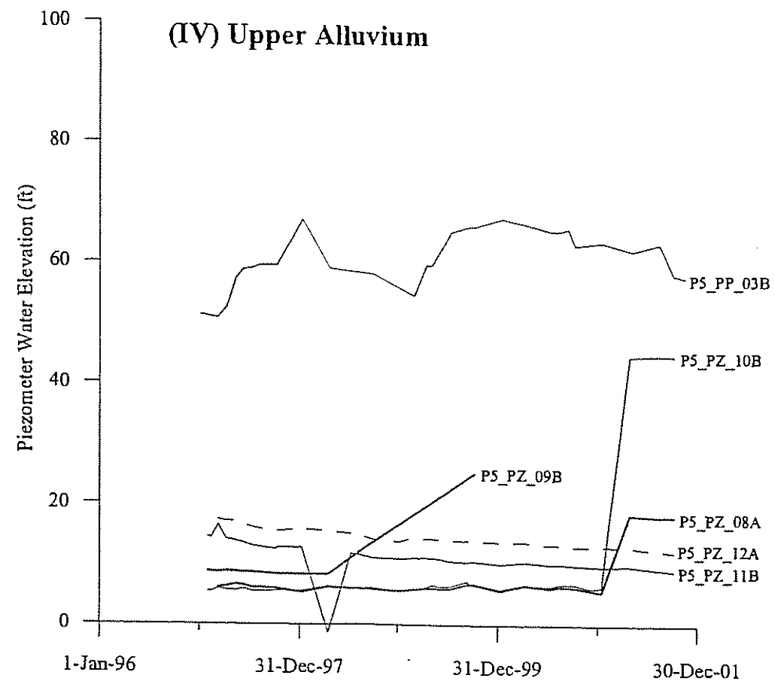
1. Data labeling system (as in GIS):
   P5_PZ_10A: Phase 5 Piezometer
   P5_SP_3: Phase 5 Settlement Plate
2. No horizontal scale in (I).
3. Key for (I):

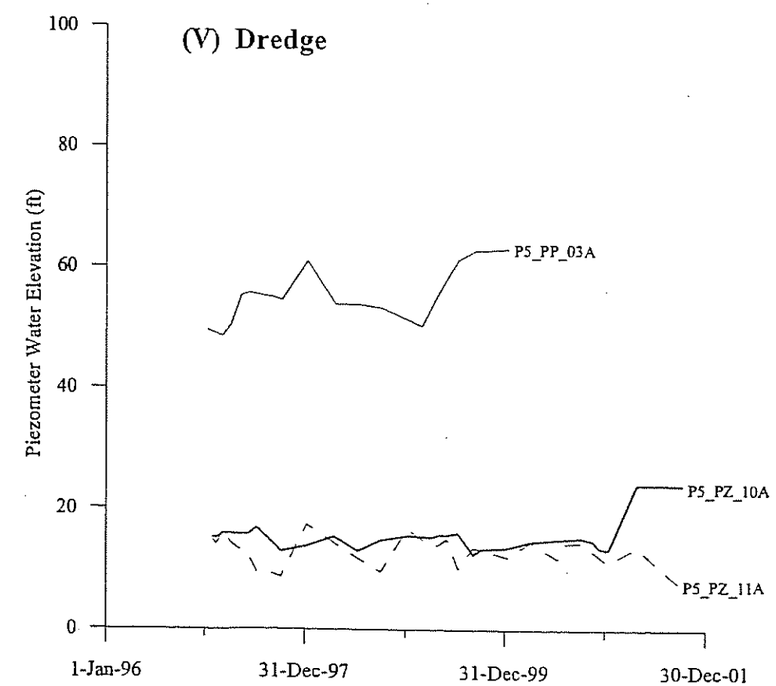


4. Elevation of geologic strata shown in (I) are approximate and are obtained from nearby boring tb-17.
5. Lines of piezometers shown in Figure 5-19.
6. Installation data was not available for wells P5_PP_03A, P5_PP_03B, and P5_PP_03C; however, water level data is still shown in (III), (IV), and (V).
7. Data point in dashed portion of dredge surface elevation line in (II) was obtained from average surface elevation of dredge when piezometers in (I) were installed.









Phase 5-4(2).grf

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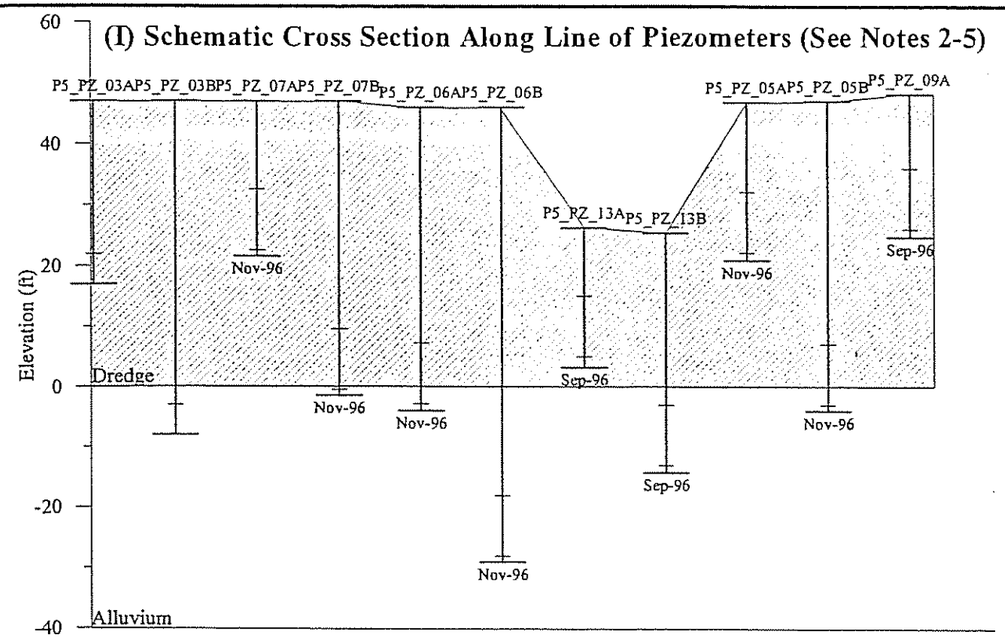
Piezometer Data
Phase 5, Line 4

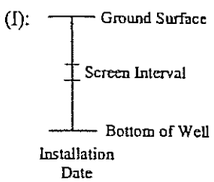
FIGURE 5-23

PROJECT: ME0250 Cherry Island

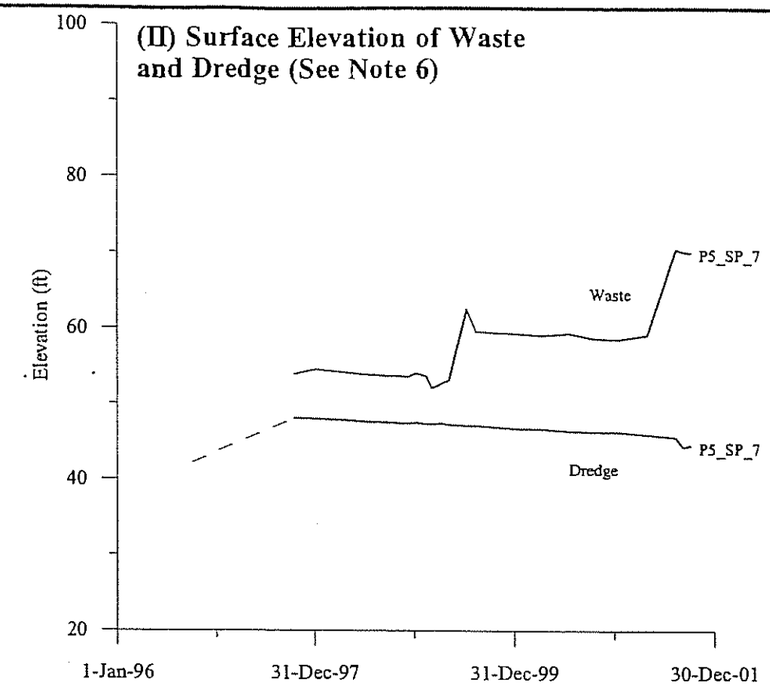


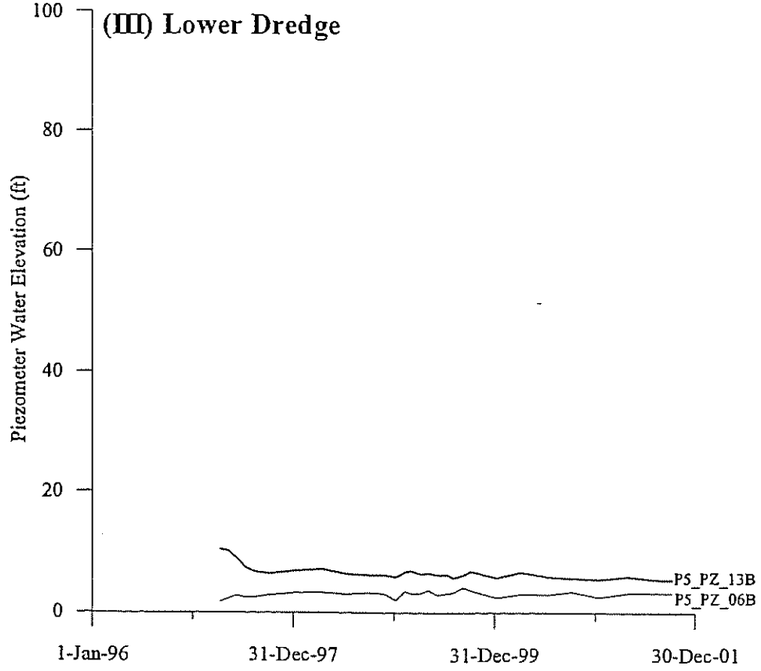
NOTES:

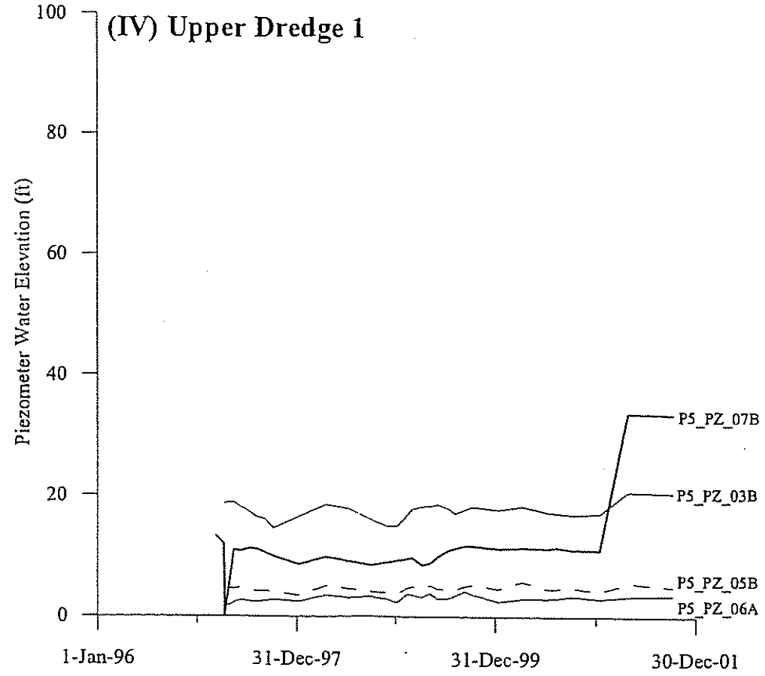
1. Data labeling system (as in GIS):
   P5_PZ_07A: Phase 5 Piezometer
   P5_SP_2: Phase 5 Settlement Plate
2. No horizontal scale in (I).
3. Key for (I):

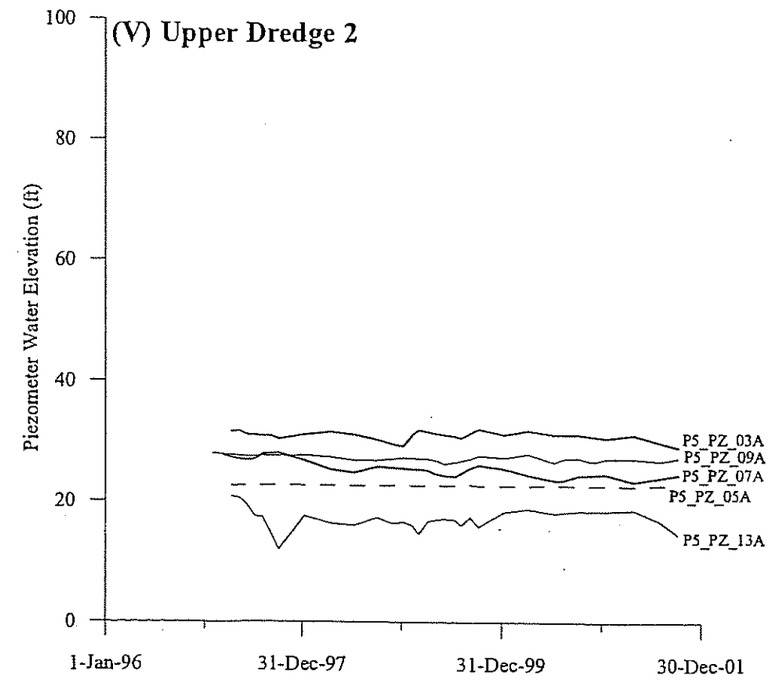


4. Elevation of geologic strata shown in (I) are approximate and are obtained from nearby boring tb-18.
5. Lines of piezometers shown in Figure 5-19.
6. Data point in dashed portion of dredge surface elevation line in (II) was obtained from average surface elevation of dredge when piezometers in (I) were installed.









Phase 5-5(2).grf

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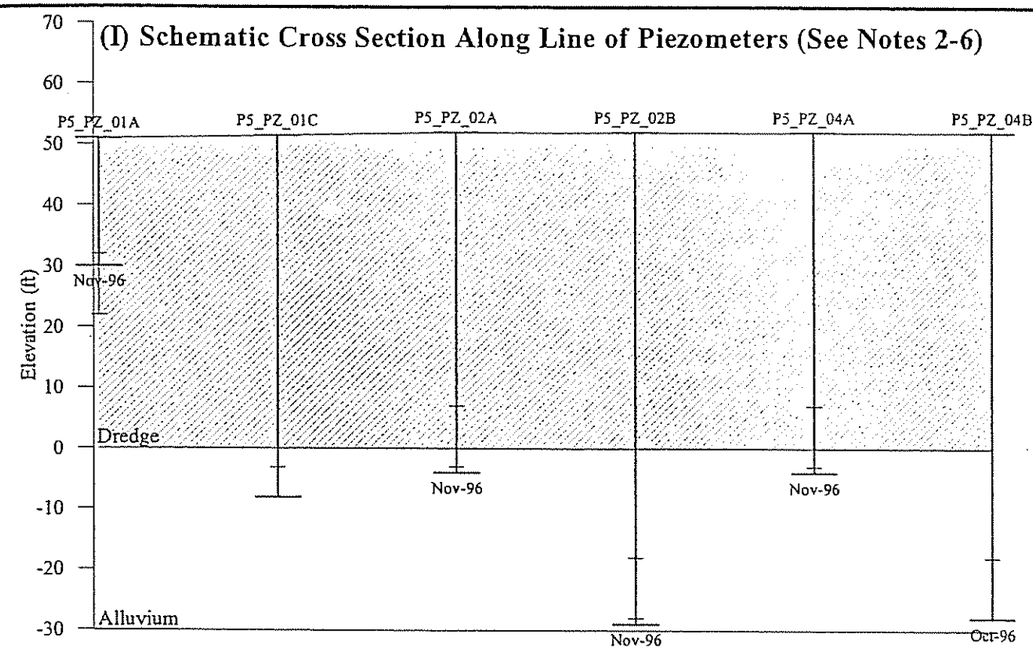
Piezometer Data
Phase 5, Line 5

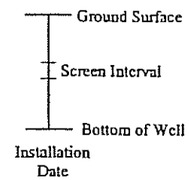
FIGURE 5-24

PROJECT: ME0250 Cherry Island

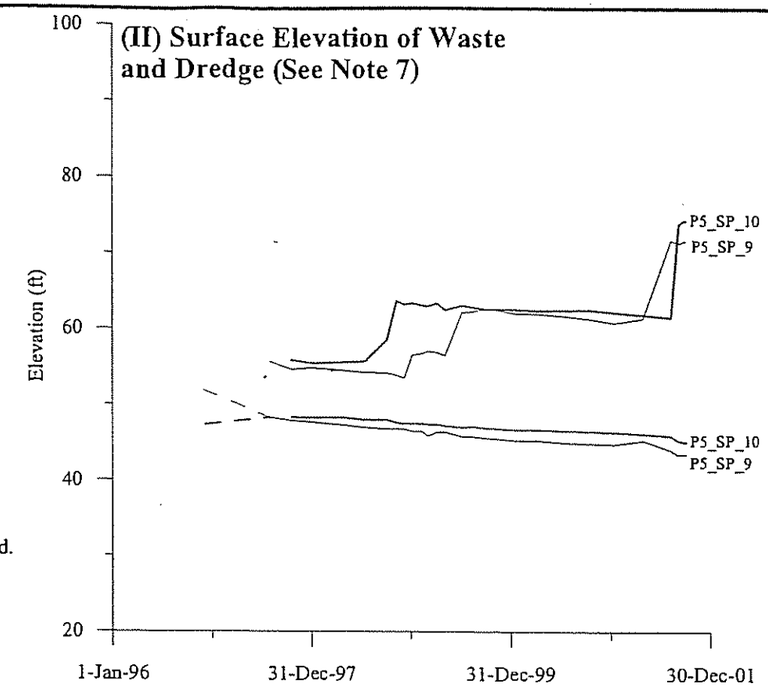


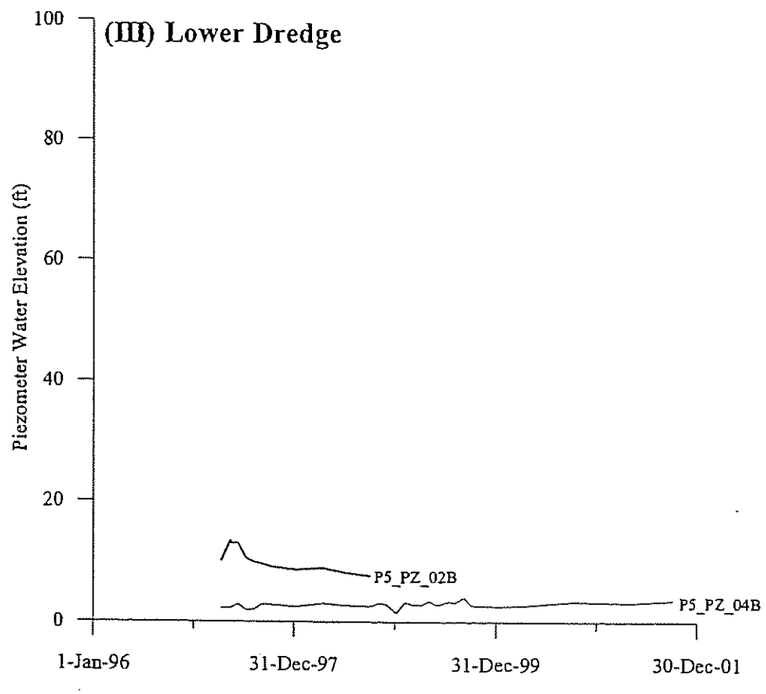
NOTES:

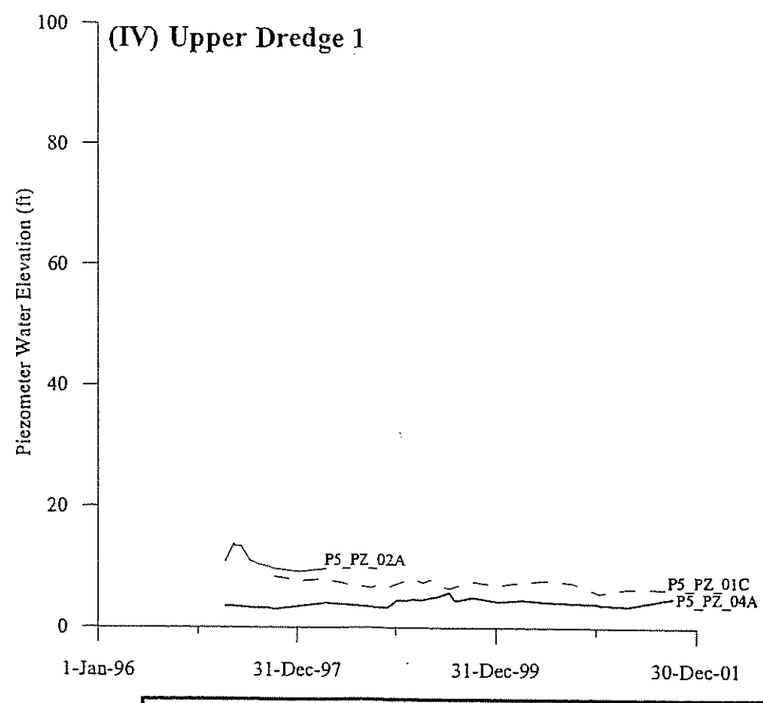
1. Data labeling system (as in GIS):
   P5_PZ_01A: Phase 5 Piezometer
   P5_SP_9: Phase 5 Settlement Plate
2. No horizontal scale in (I).
3. Key for (I):

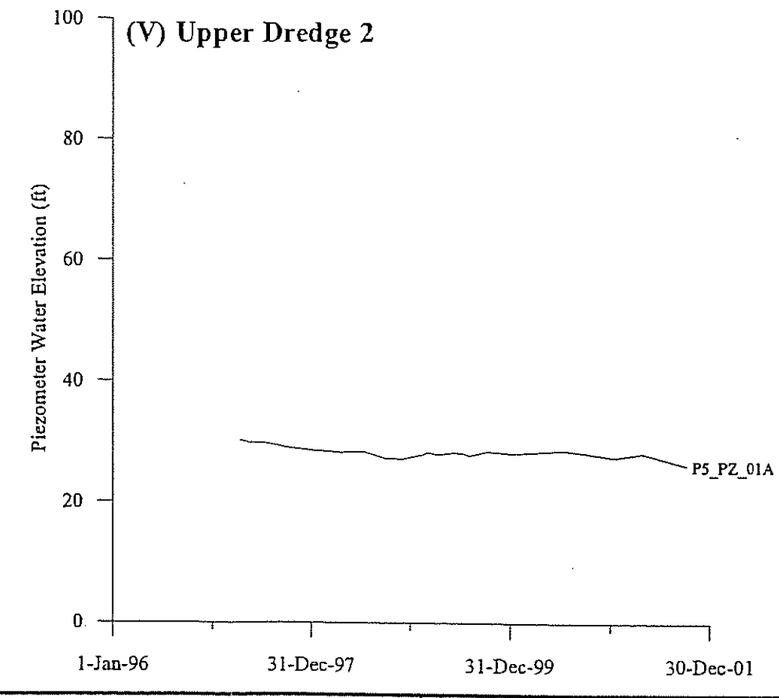


4. Elevation of geologic strata shown in (I) are approximate and are obtained from nearby boring SEA-14.
5. Lines of piezometers shown in Figure 5-19.
6. Limited installation data is available for well P5_PZ_01C; ground surface and bottom of boring elevations are estimated.
7. Data point in dashed portion of dredge surface elevation lines in (II) was obtained from average surface elevation of dredge when piezometers in (I) were installed.









Phase 5-6(2).grf

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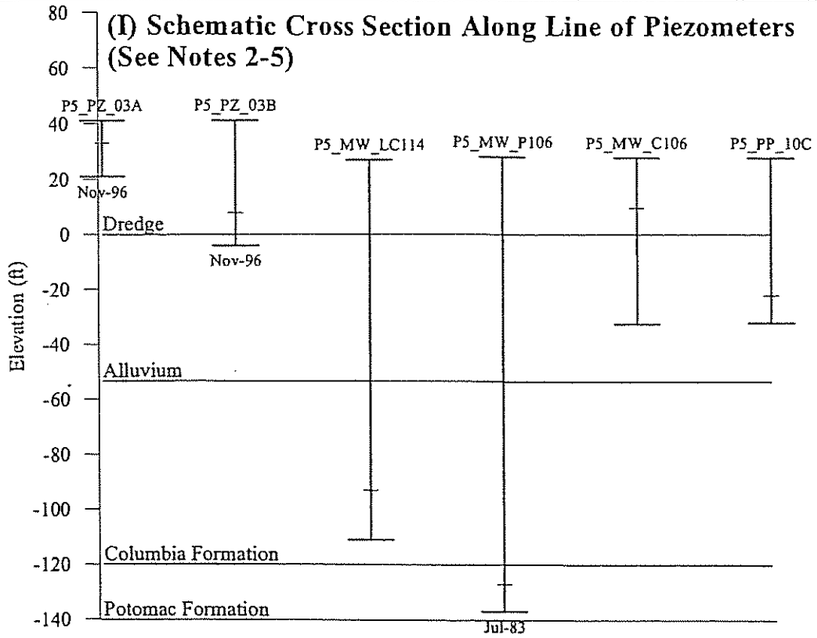
Piezometer Data
Phase 5, Line 6

FIGURE 5-25

PROJECT: ME0250 Cherry Island

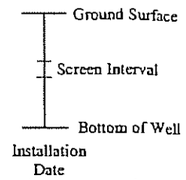


NOTES:

1. Data labeling system (as in GIS):
   P5_PZ_03A: Phase 5 Piezometer
   P5_MW_P106: Phase 5 Monitoring Well
   P5_SP_9: Phase 5 Settlement Plate

2. No horizontal scale in (I).

3. Key for (I):



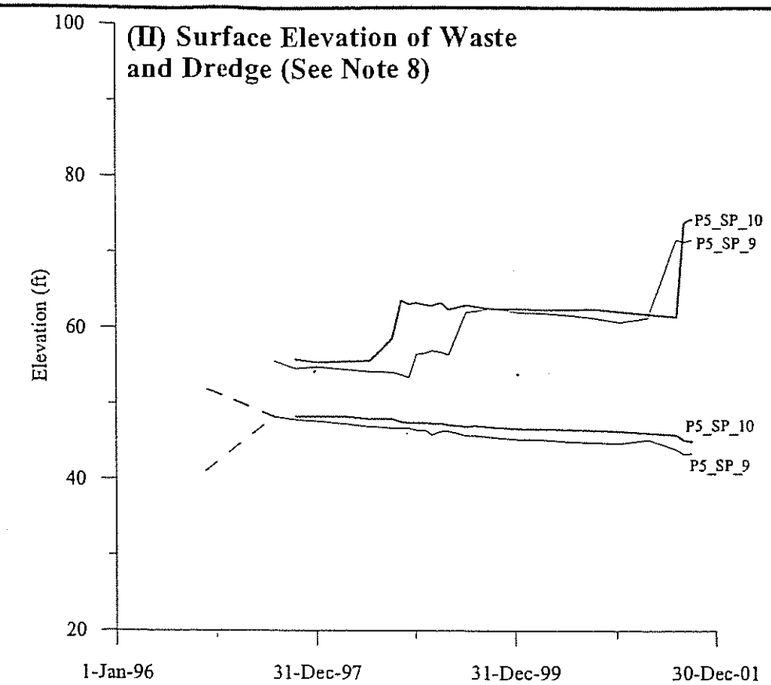
4. Elevation of geologic strata shown in (I) are approximate and are obtained from nearby boring TB-21. Line between Columbia Formation and Potomac Formation is estimated from boring information when P5_MW_P106 was installed.

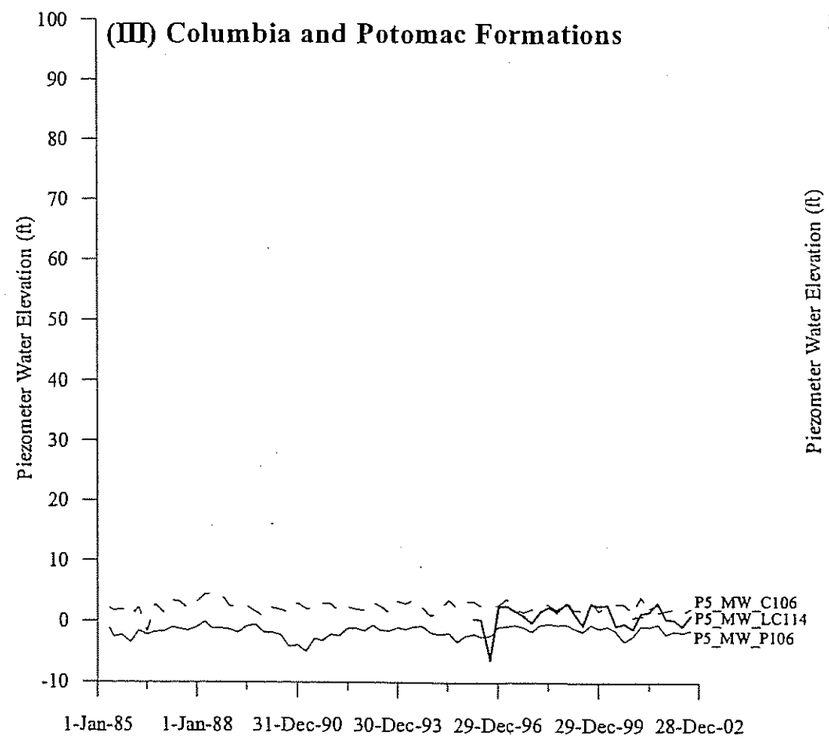
5. Lines of piezometers shown in Figure 5-19.

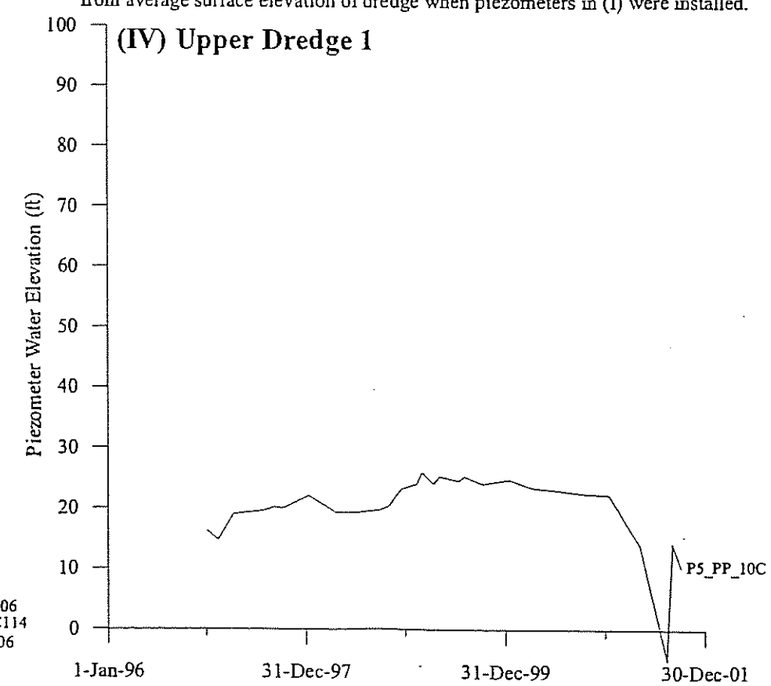
6. Water level data for P5_MW_P106 and P5_MW_UC106 is available for the year 1983 but is not shown on this Figure.

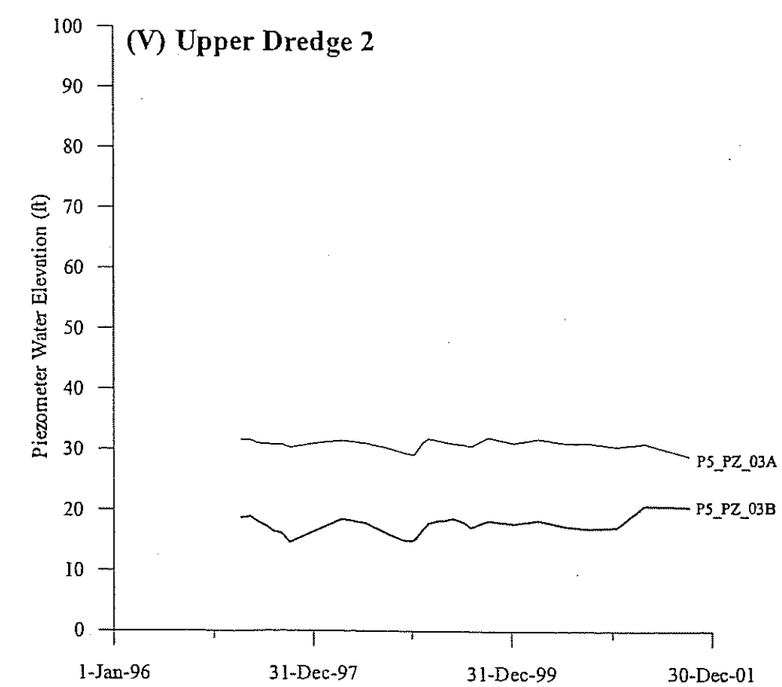
7. Installation data is not available for P5_PP_10C; however, water level data is still shown in (III).

8. Data point in dashed portion of dredge surface elevation lines in (II) was obtained from average surface elevation of dredge when piezometers in (I) were installed.









Phase 5-7(2).grf

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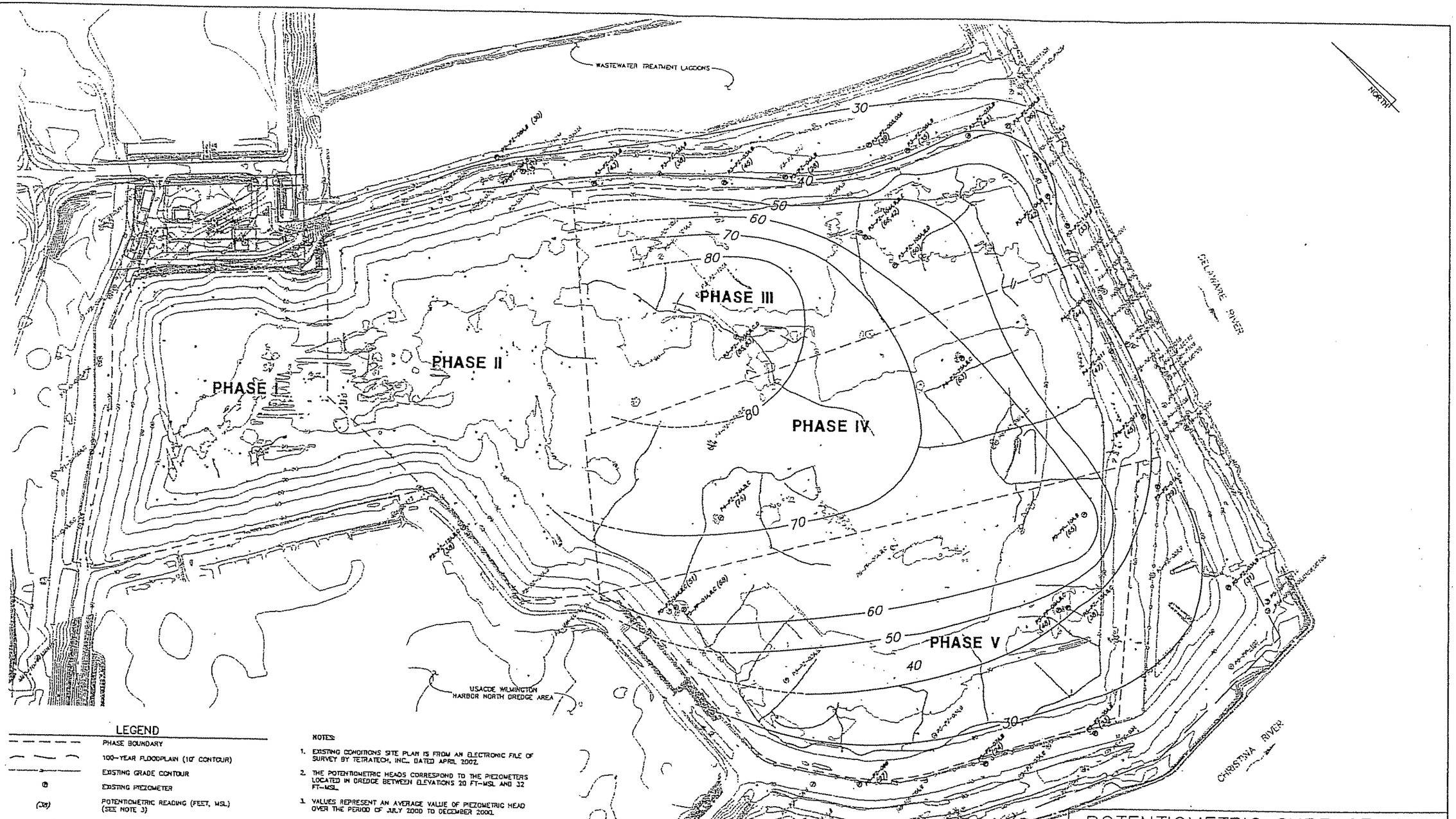
Piezometer Data
Phase 5, Line 7

FIGURE 5-26

PROJECT: ME0250 Cherry Island



LEGEND

- PHASE BOUNDARY
- 100-YEAR FLOODPLAIN (10' CONTOUR)
- EXISTING GRADE CONTOUR
- EXISTING PIEZOMETER
- (30) POTENTIOMETRIC READING (FEET, MSL) (SEE NOTE 3)
- 40 POTENTIOMETRIC CONTOUR (FEET, MSL) (SEE NOTES 2 AND 3)
- 80 INFERRED POTENTIOMETRIC CONTOUR (FEET, MSL) (SEE NOTES 2 AND 3)

NOTES:

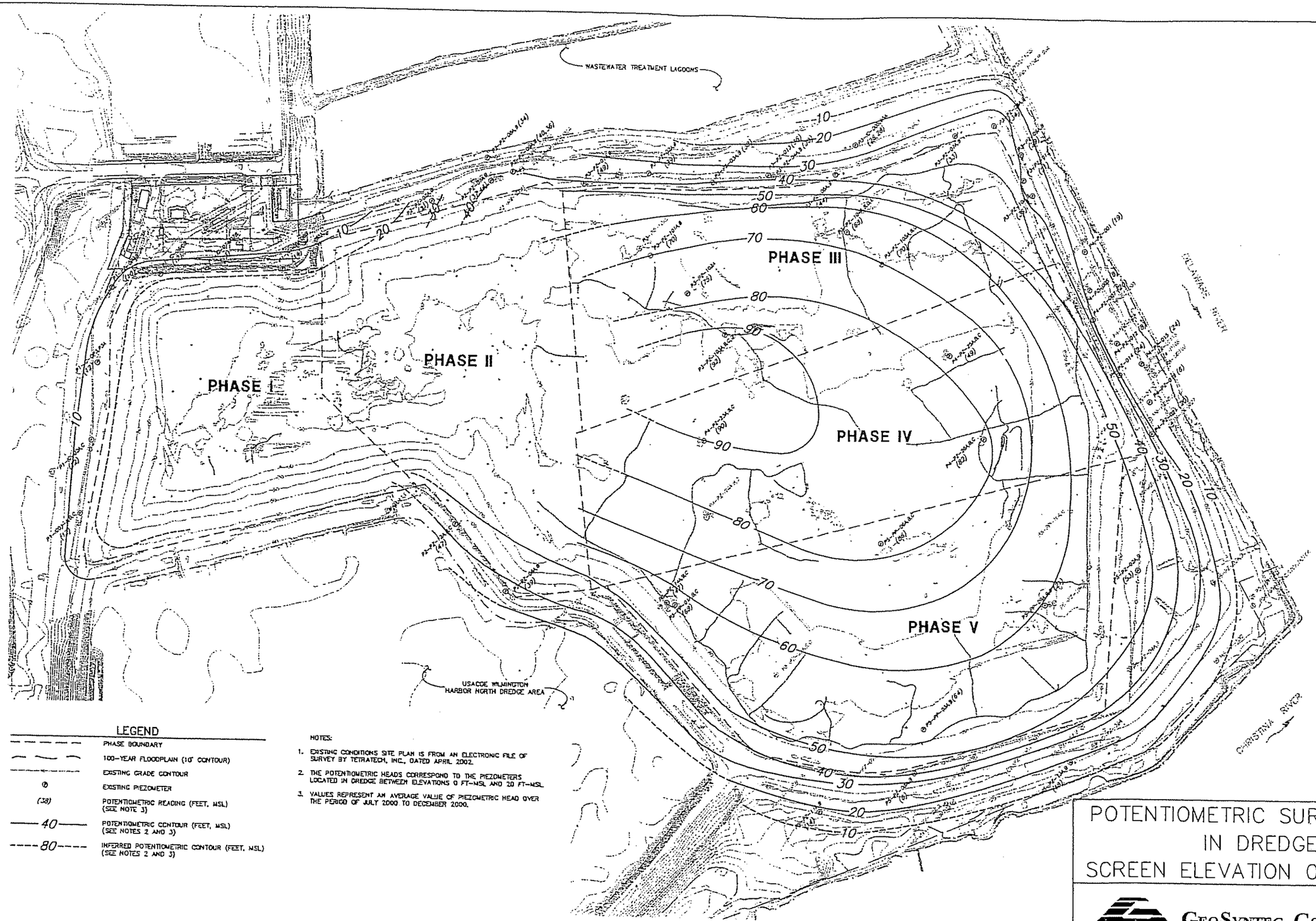
1. EXISTING CONDITIONS SITE PLAN IS FROM AN ELECTRONIC FILE OF SURVEY BY TETRATECH, INC., DATED APRIL 2002.
2. THE POTENTIOMETRIC HEADS CORRESPOND TO THE PIEZOMETERS LOCATED IN DREDGE BETWEEN ELEVATIONS 20 FT-MSL AND 32 FT-MSL.
3. VALUES REPRESENT AN AVERAGE VALUE OF PIEZOMETRIC HEAD OVER THE PERIOD OF JULY 2000 TO DECEMBER 2000.

POTENTIOMETRIC SURFACE MAP IN DREDGE SCREEN ELEVATION 20 TO 32 FT

GEOSYNTEC CONSULTANTS
COLUMBIA, MARYLAND

PROJECT NO.	ME0250	FILE NO.	0250F511A
DOCUMENT NO.		FIGURE NO.	5-27

P:\cadd\0250\0250F511A.DWG Wed Mar 26 08:46:28 2003 GeoSyntec Consultants, Inc. (JimD)



LEGEND

— — — —	PHASE BOUNDARY
— — — —	100-YEAR FLOODPLAIN (10' CONTOUR)
— — — —	EXISTING GRADE CONTOUR
⊕	EXISTING PIEZOMETER
(38)	POTENTIOMETRIC READING (FEET, MSL) (SEE NOTE 3)
—— 40 ——	POTENTIOMETRIC CONTOUR (FEET, MSL) (SEE NOTES 2 AND 3)
- - - 80 - - -	INFERRED POTENTIOMETRIC CONTOUR (FEET, MSL) (SEE NOTES 2 AND 3)

NOTES:

1. EXISTING CONDITIONS SITE PLAN IS FROM AN ELECTRONIC FILE OF SURVEY BY TETRATECH, INC., DATED APRIL 2002.
2. THE POTENTIOMETRIC HEADS CORRESPOND TO THE PIEZOMETERS LOCATED IN DREDGE BETWEEN ELEVATIONS 0 FT-MSL AND 20 FT-MSL.
3. VALUES REPRESENT AN AVERAGE VALUE OF PIEZOMETRIC HEAD OVER THE PERIOD OF JULY 2000 TO DECEMBER 2000.

450 225 0 450
SCALE IN FEET

POTENTIOMETRIC SURFACE MAP IN DREDGE SCREEN ELEVATION 0 TO 20 FT

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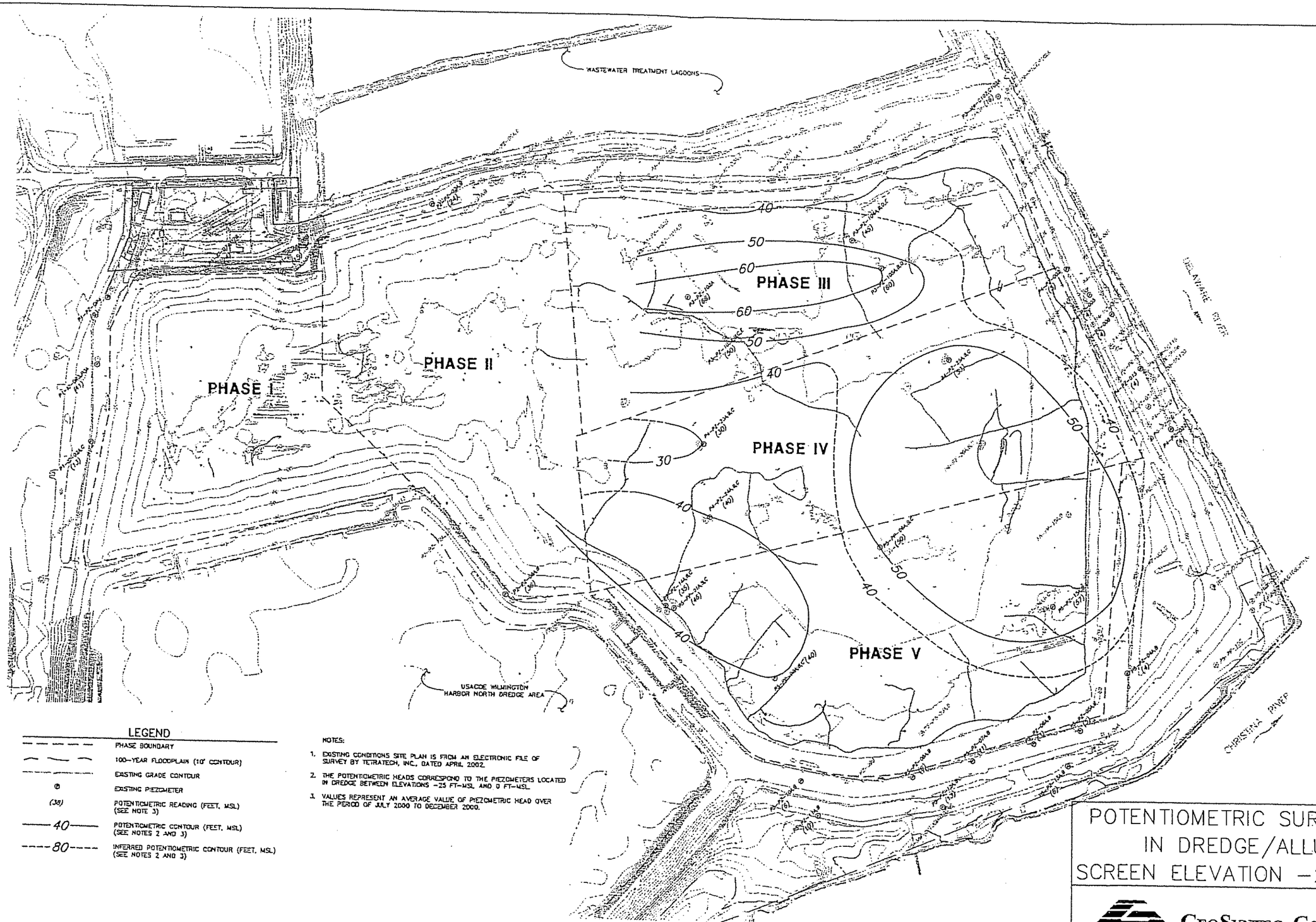
COLUMBIA, MARYLAND

[illegible]	ME0250	[illegible]	0250F511B
[illegible]		[illegible]	5-28



LEGEND

- PHASE BOUNDARY
- 100-YEAR FLOODPLAIN (10' CONTOUR)
- EXISTING GRADE CONTOUR
- EXISTING PIEZOMETER
- (38) POTENTIOMETRIC READING (FEET, MSL) (SEE NOTE 3)
- 40 POTENTIOMETRIC CONTOUR (FEET, MSL) (SEE NOTES 2 AND 3)
- 80 INFERRED POTENTIOMETRIC CONTOUR (FEET, MSL) (SEE NOTES 2 AND 3)

NOTES:

1. EXISTING CONDITIONS SITE PLAN IS FROM AN ELECTRONIC FILE OF SURVEY BY TETRATECH, INC., DATED APRIL 2002.
2. THE POTENTIOMETRIC HEADS CORRESPOND TO THE PIEZOMETERS LOCATED IN DREDGE BETWEEN ELEVATIONS -25 FT-MSL AND 0 FT-MSL.
3. VALUES REPRESENT AN AVERAGE VALUE OF PIEZOMETRIC HEAD OVER THE PERIOD OF JULY 2000 TO DECEMBER 2000.

450 225 0 450
SCALE IN FEET

POTENTIOMETRIC SURFACE MAP IN DREDGE/ALLUVIUM SCREEN ELEVATION -25 TO 0 FT

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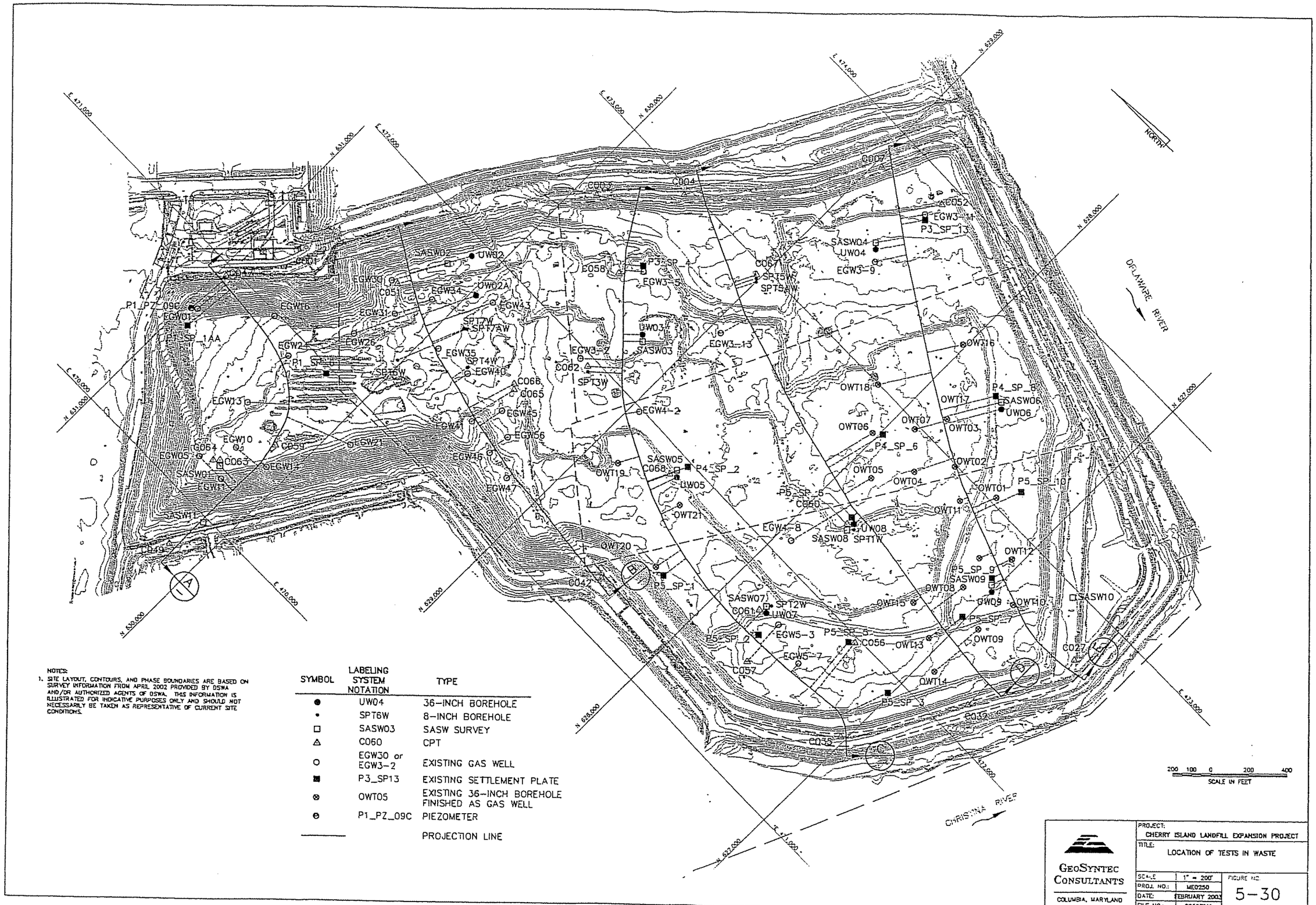
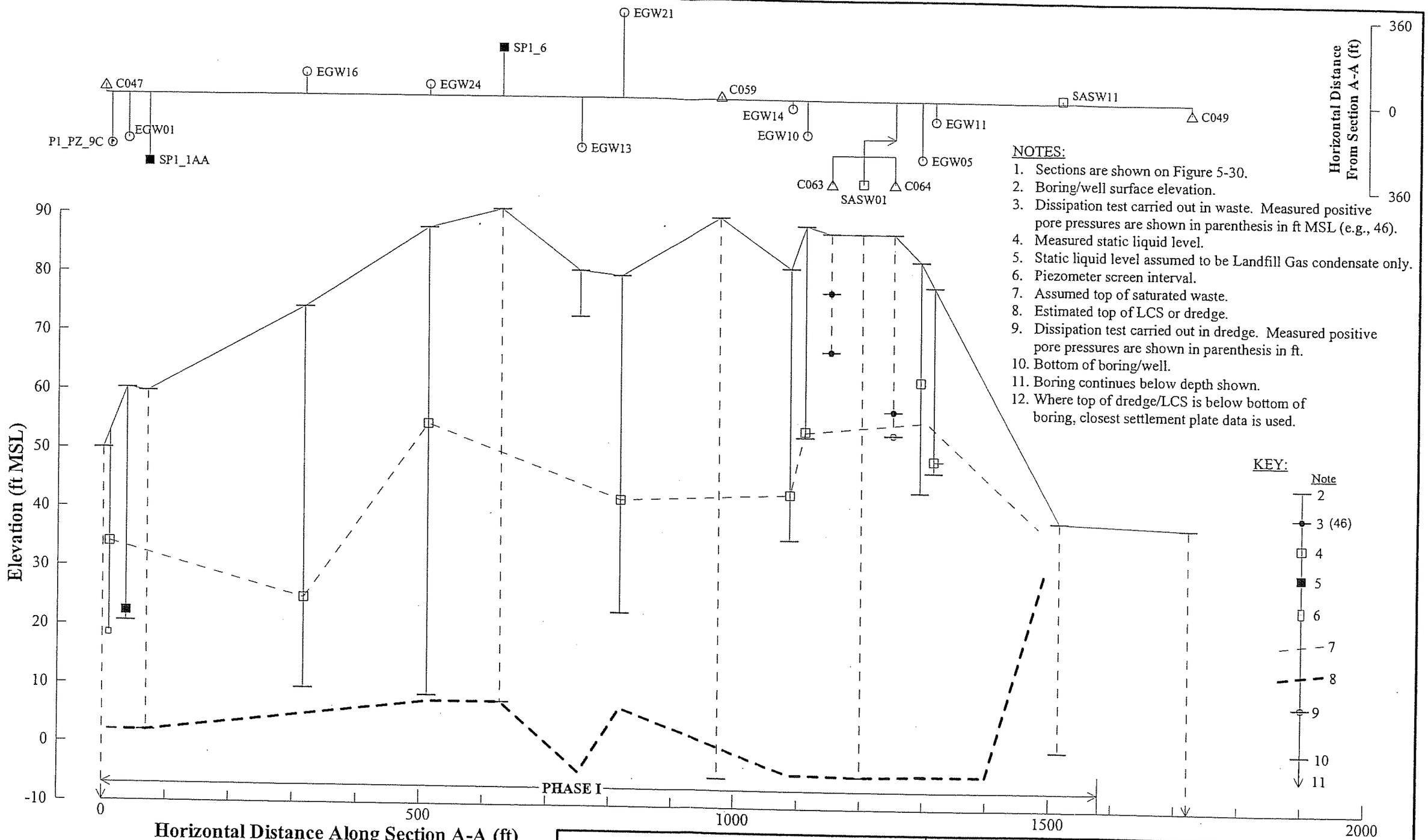
PROJECT NO.	ME0250	FILE NO.	0250F511C
DATE		DATE	5-29


NORTH
DELAWARE RIVER
CHRISTINA RIVER
SCALE IN FEET
200 100 0 200 400
SYMBOL
LABELING SYSTEM NOTATION
TYPE
UW04
36-INCH BOREHOLE
SPT6W
8-INCH BOREHOLE
SASW03
SASW SURVEY
C060
CPT
EGW30 or EGW3-2
EXISTING GAS WELL
P3_SP13
EXISTING SETTLEMENT PLATE
OWT05
EXISTING 36-INCH BOREHOLE FINISHED AS GAS WELL
P1_PZ_09C
PIEZOMETER
PROJECTION LINE
NOTES:
1. SITE LAYOUT, CONTOURS, AND PHASE BOUNDARIES ARE BASED ON SURVEY INFORMATION FROM APRIL 2002 PROVIDED BY DSWA AND/OR AUTHORIZED AGENTS OF DSWA. THIS INFORMATION IS ILLUSTRATED FOR INDICATIVE PURPOSES ONLY AND SHOULD NOT NECESSARILY BE TAKEN AS REPRESENTATIVE OF CURRENT SITE CONDITIONS.
GEOSYNTEC CONSULTANTS
COLUMBIA, MARYLAND
PROJECT: CHERRY ISLAND LANDFILL EXPANSION PROJECT
TITLE: LOCATION OF TESTS IN WASTE
SCALE 1" = 200'
PROJ. NO.: ME0250
DATE: FEBRUARY 2003
FILE NO.: 0250F811
FIGURE NO. 5-30



NOTES:

1. Sections are shown on Figure 5-30.
2. Boring/well surface elevation.
3. Dissipation test carried out in waste. Measured positive pore pressures are shown in parenthesis in ft MSL (e.g., 46).
4. Measured static liquid level.
5. Static liquid level assumed to be Landfill Gas condensate only.
6. Piezometer screen interval.
7. Assumed top of saturated waste.
8. Estimated top of LCS or dredge.
9. Dissipation test carried out in dredge. Measured positive pore pressures are shown in parenthesis in ft.
10. Bottom of boring/well.
11. Boring continues below depth shown.
12. Where top of dredge/LCS is below bottom of boring, closest settlement plate data is used.

P:/.../Field Exploration I/Settlement_Density/Section AA Water(2).grf 3/25/03

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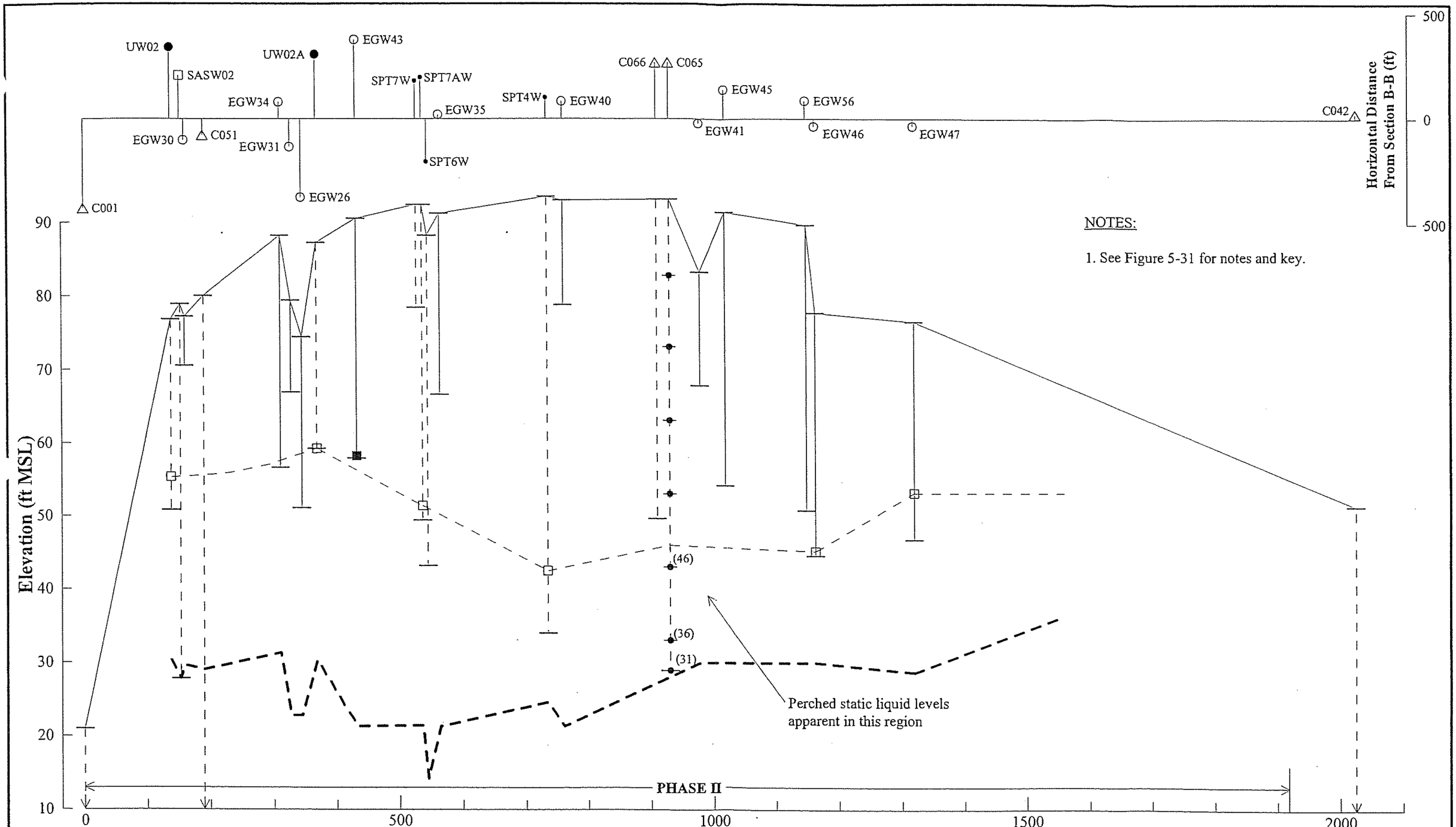
PROFILE OF SATURATED WASTE ELEVATIONS IN LANDFILL
Section A-A

FIGURE 5-31

PROJECT: ME0250 Cherry Island



P:/.../Field Investigation I/Settlement_Density/Section BB Water.grf 3/25/03

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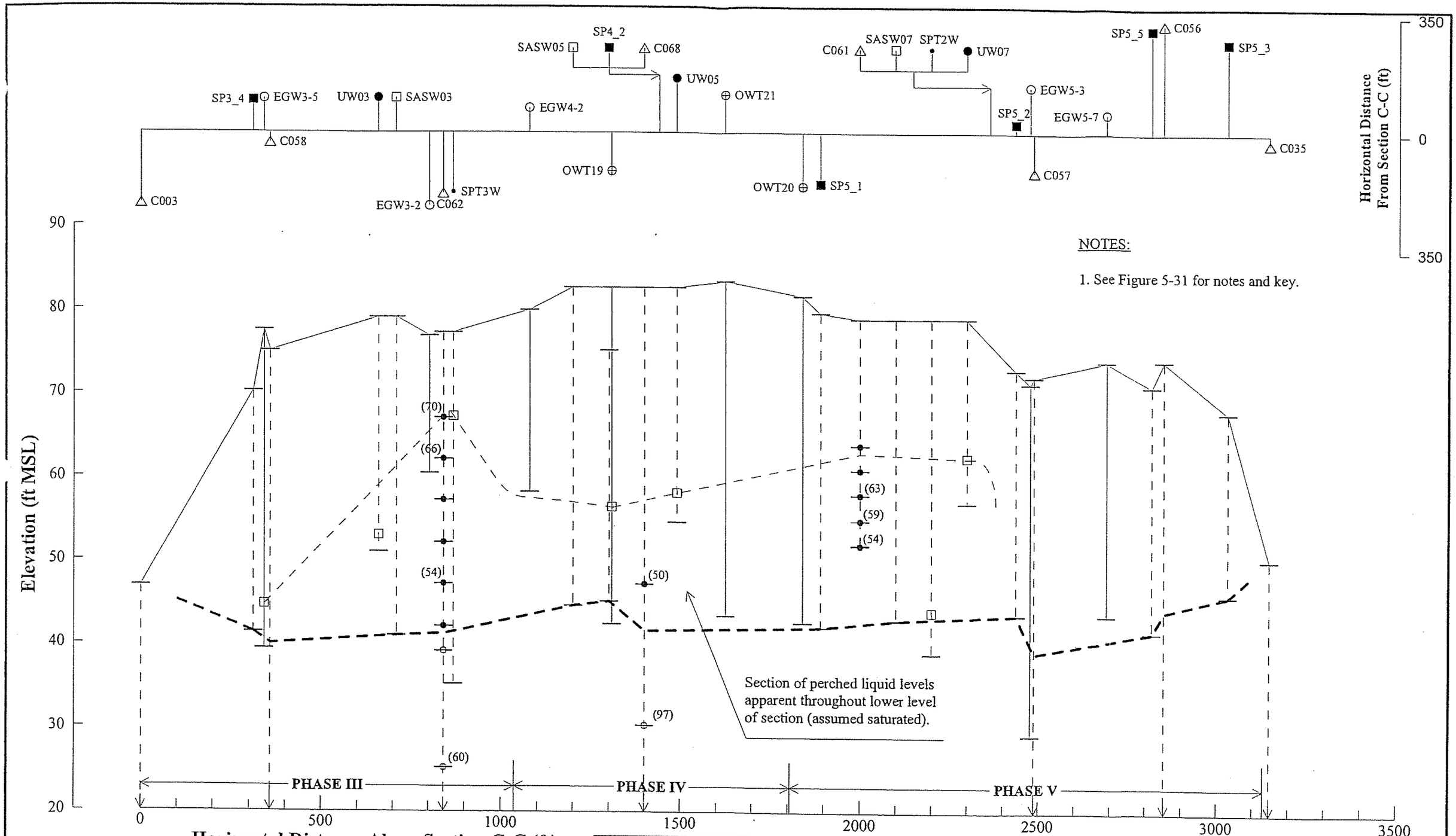
PROFILE OF SATURATED WASTE ELEVATIONS IN LANDFILL
Section B-B

FIGURE 5-32

PROJECT: ME0250 Cherry Island



P:/.../Field Exploration I/Settlement_Density/Section CC Water.grf 3/25/03

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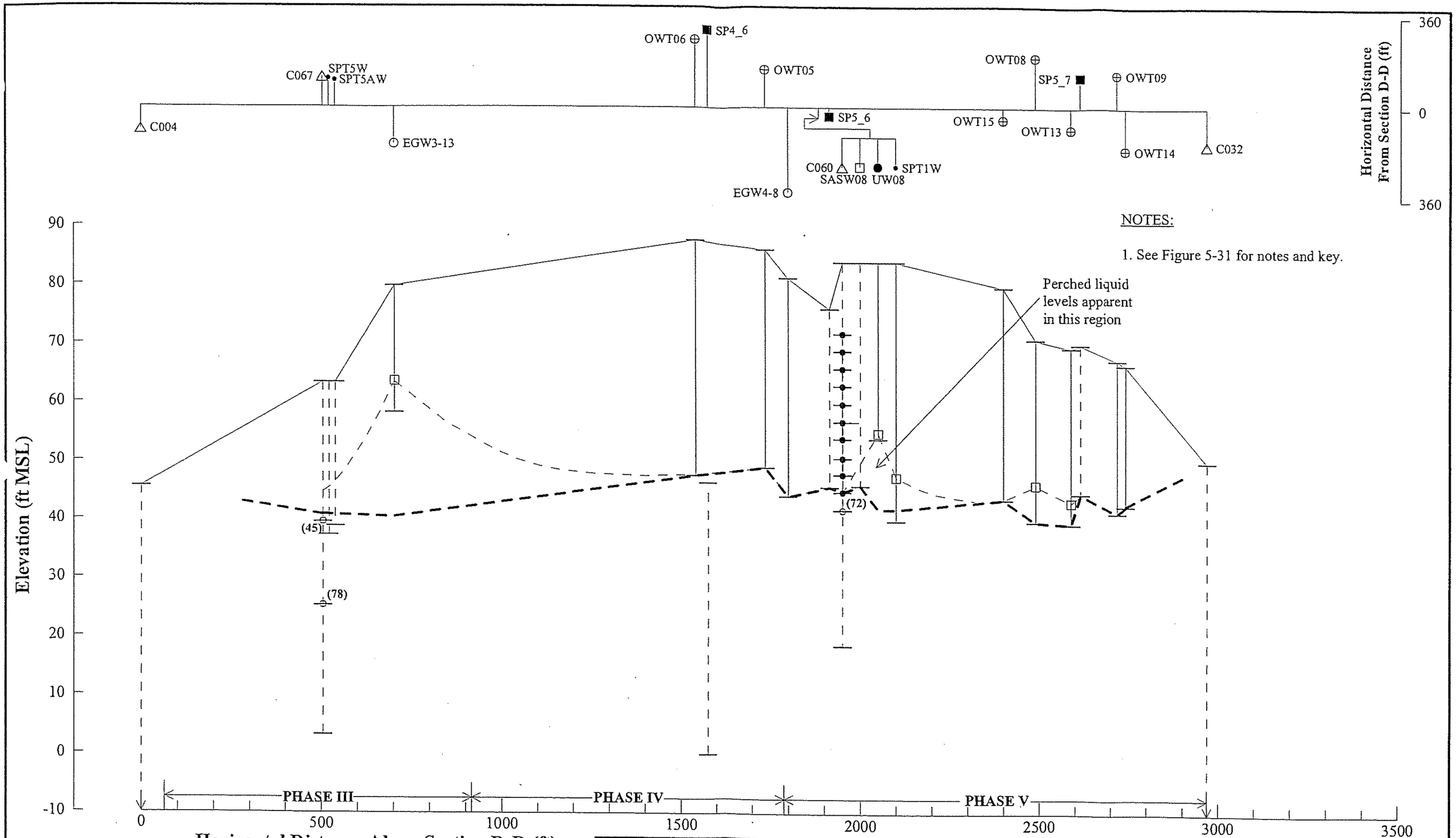
PROFILE OF SATURATED WASTE ELEVATIONS IN LANDFILL
Section C-C

FIGURE 5-33

PROJECT: ME0250 Cherry Island



P:/.../Field Exploration I/Settlement_Density/Section DD Water.grf 3/25/03

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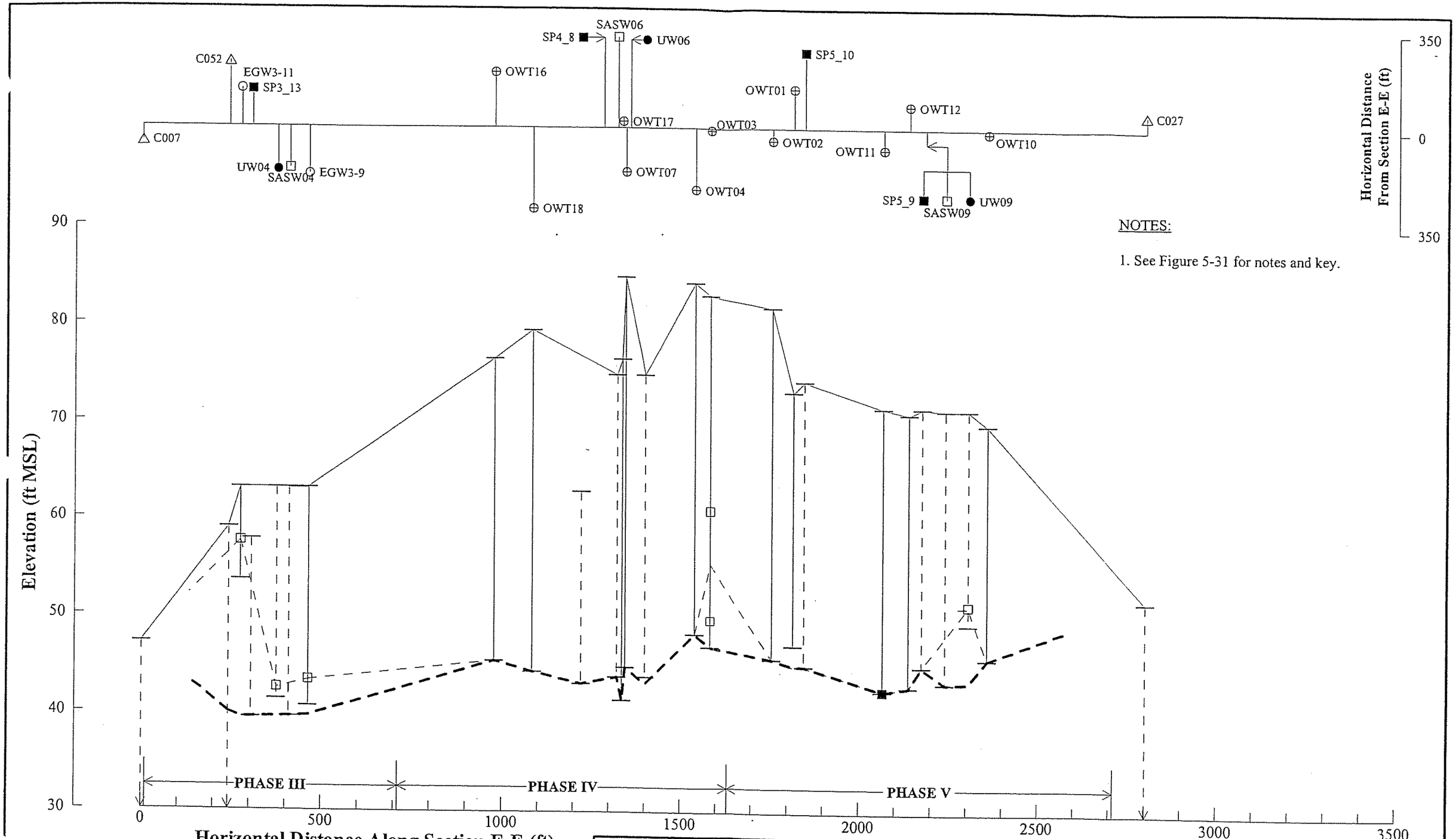
Columbia, Maryland 21046

PROFILE OF SATURATED WASTE ELEVATIONS IN LANDFILL
Section D-D

FIGURE 5-34

PROJECT: ME0250 Cherry Island



NOTES:

1. See Figure 5-31 for notes and key.

P:/.../Field Exploration I/Settlement_Density/Section EE Water.grf 3/25/03

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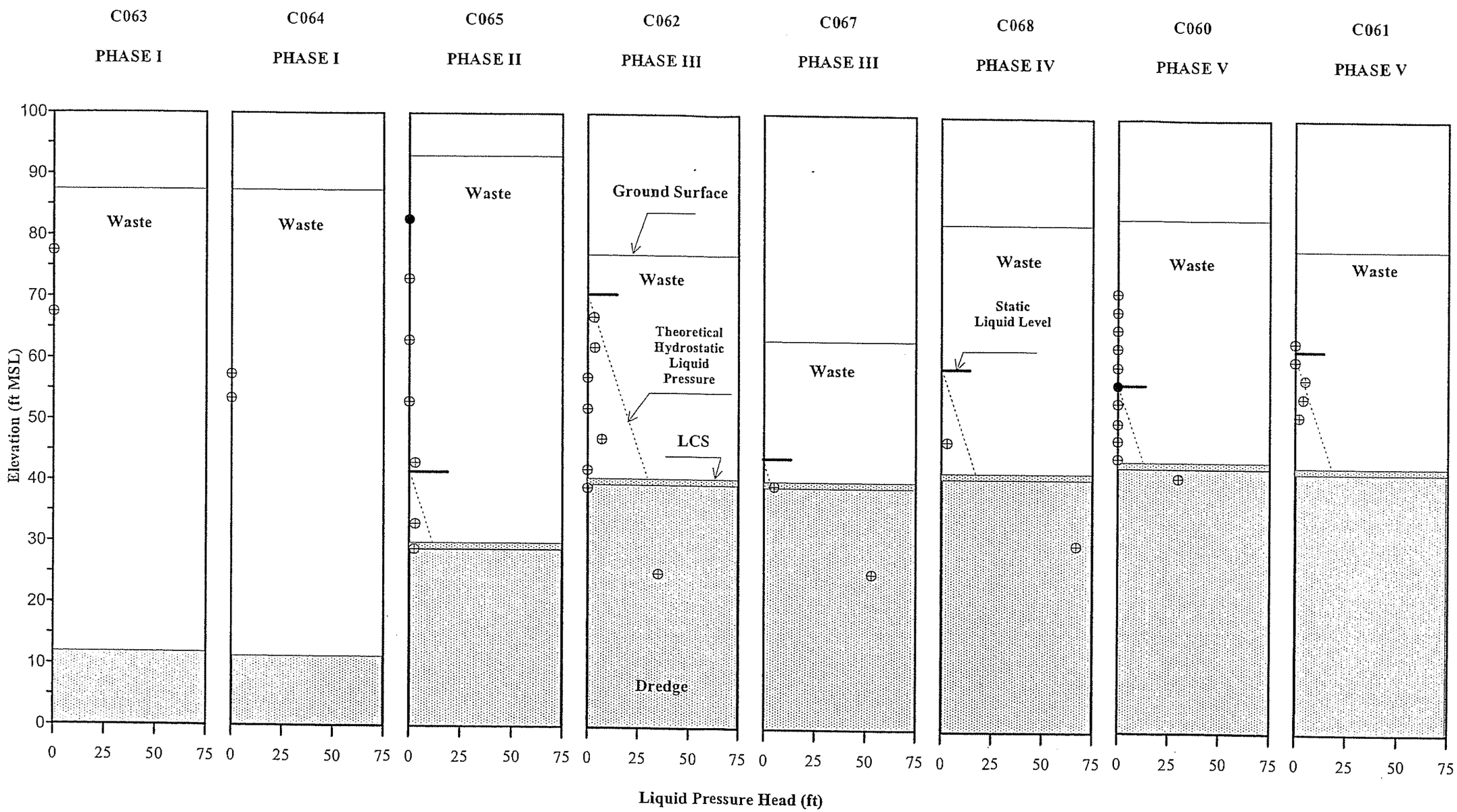
PROFILE OF SATURATED WASTE ELEVATIONS IN LANDFILL
Section E-E

FIGURE 5-35

PROJECT: ME0250 Cherry Island



Static Liquid Level Observed in Bucket Auger Borings/Wells

Stabilized Liquid Pressure from CPT Dissipation Test

Measured Zero Liquid Pressure

NOTE: Except as noted, dissipation tests at depths with reported pore pressure of zero actually recorded negative pore pressures.

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CPT DISSIPATION TEST RESULTS

FIGURE 5-36

PROJECT: ME0250 Cherry Island

P:/Field Exploration II/CPT/CPT Dissipation.grf 3/25/03

WASTEWATER TREATMENT LAGOONS

NORTH

PHASE I

PHASE II

PHASE III

PHASE IV

PHASE V

USACOE WILMINGTON HARBOR NORTH DREDGE AREA

DELAWARE RIVER

CHRISTINA RIVER

LEGEND

	PROPERTY BOUNDARY
	CELL BOUNDARY
50	SURFACE ELEVATION CONTOUR
50	SATURATED WASTE CONTOUR
40	INFERRED SATURATED WASTE CONTOUR
45	SAMPLE LOCATION (MEASURED VALUE)
45	MEASURED TOP ELEVATION OF SATURATED WASTE
	SAMPLE LOCATION (INFERRED VALUE)
(45)	INFERRED TOP ELEVATION OF SATURATED WASTE (SEE NOTE 3)

NOTES:

1. SITE LAYOUT, GROUND SURFACE CONTOURS, AND PHASE BOUNDARIES ARE BASED ON SURVEY INFORMATION FROM APRIL 2002 PROVIDED BY DSWA AND/OR AUTHORIZED AGENTS OF DSWA. THIS INFORMATION IS ILLUSTRATED FOR INDICATIVE PURPOSES ONLY AND SHOULD NOT BE TAKEN AS REPRESENTATIVE OF CURRENT SITE CONDITIONS.
2. ALL DEPTHS AND ELEVATIONS ARE REPORTED IN FEET.
3. ESTIMATED ELEVATIONS OF SATURATED WASTE ARE OBTAINED FROM CROSS-SECTIONS SHOWN IN FIGURES 6-1 THROUGH 6-5. ESTIMATED ELEVATIONS ARE USED FOR SAMPLE LOCATIONS WHERE SATURATED WASTE LEVELS WERE BELOW THE BASE OF BORINGS OR GAS WELLS.
4. SURFACES SHOWN ARE DERIVED FROM HIGHEST ELEVATIONS OF SATURATED WASTE REPORTED. PERCHED (I.E., UNCONNECTED) LIQUID LEVELS MAY BE PRESENT BELOW THESE LEVELS, ESPECIALLY WHERE REGIONS OF SATURATED WASTE ARE SHOWN ABOVE 50 FEET MSL.

450 225 0 450

SCALE IN FEET

TOP SURFACE OF SATURATED WASTE

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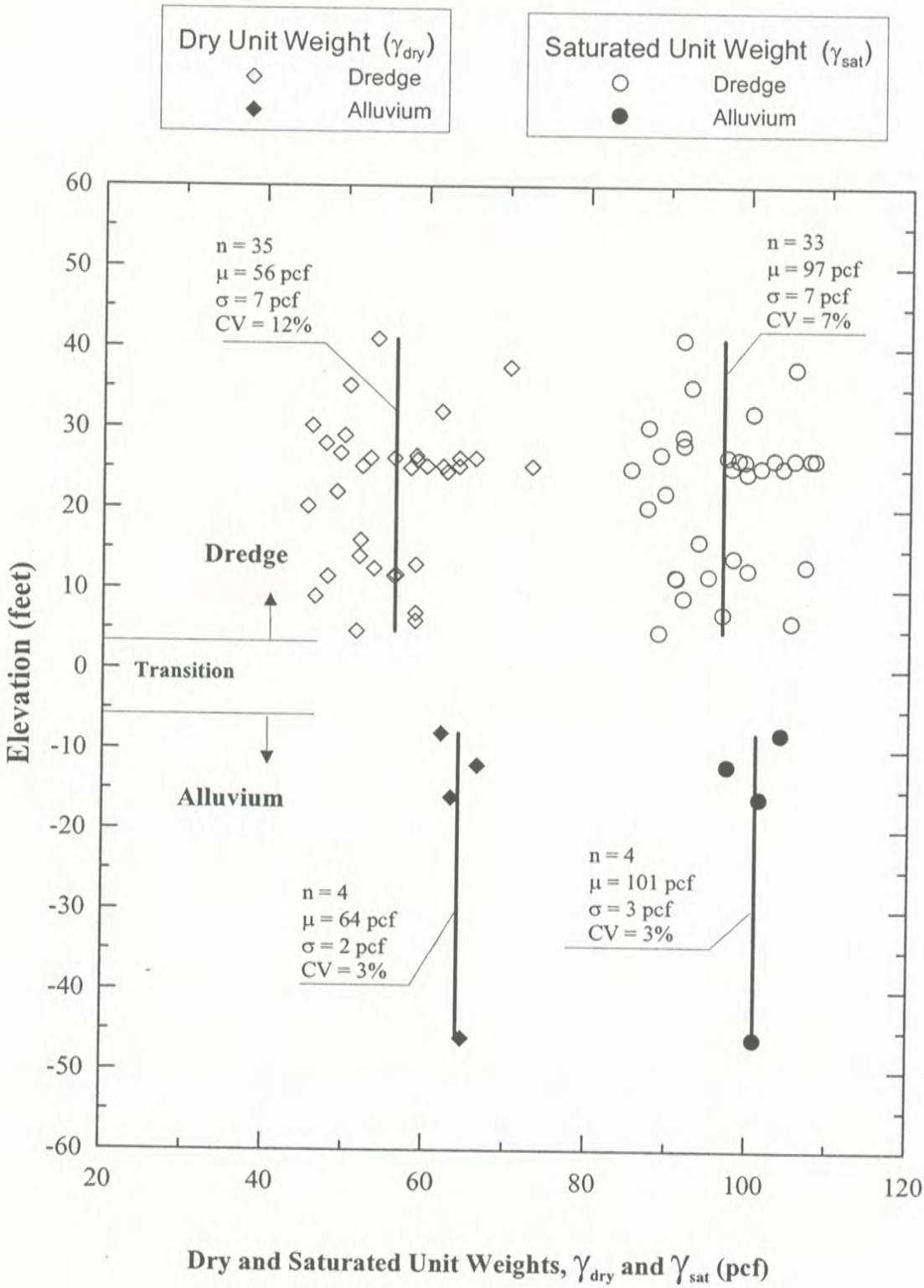
COLUMBIA, MARYLAND

PROJECT NO	ME0250	FILE NO	0250F515
DOCUMENT NO.		FIGURE NO.	5-37



gammas w depth(2).grf

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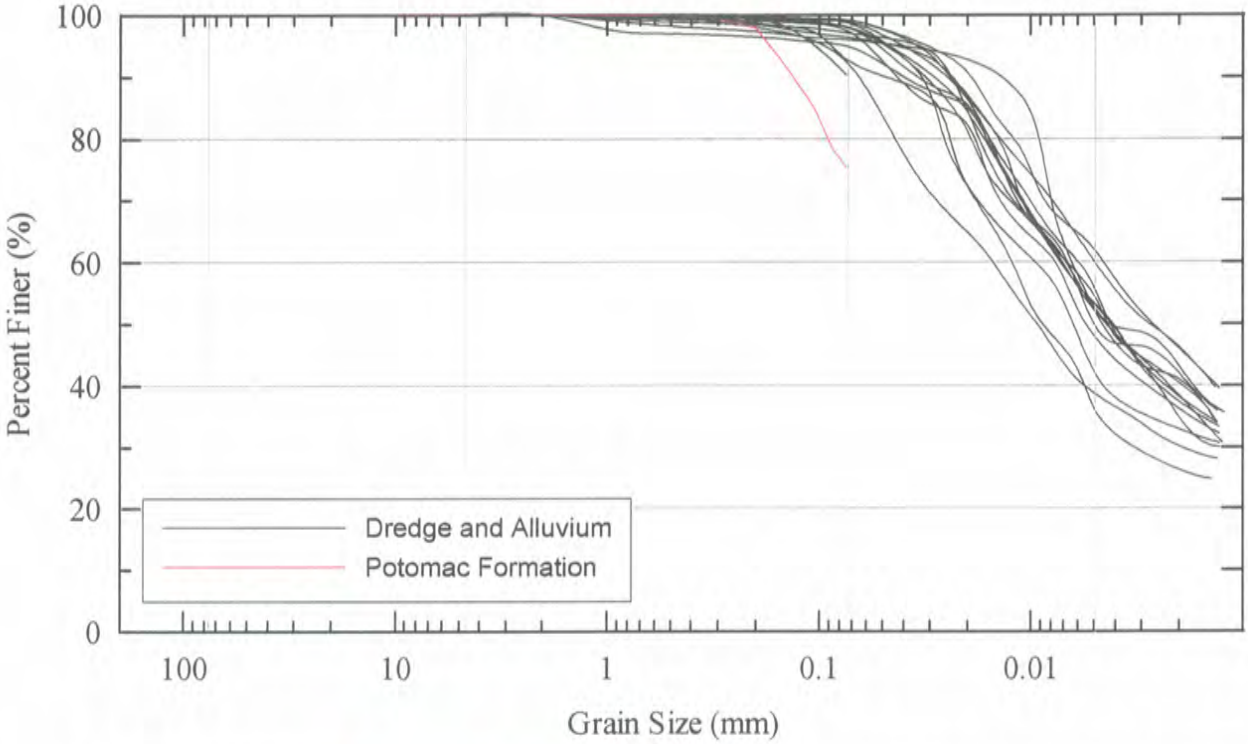
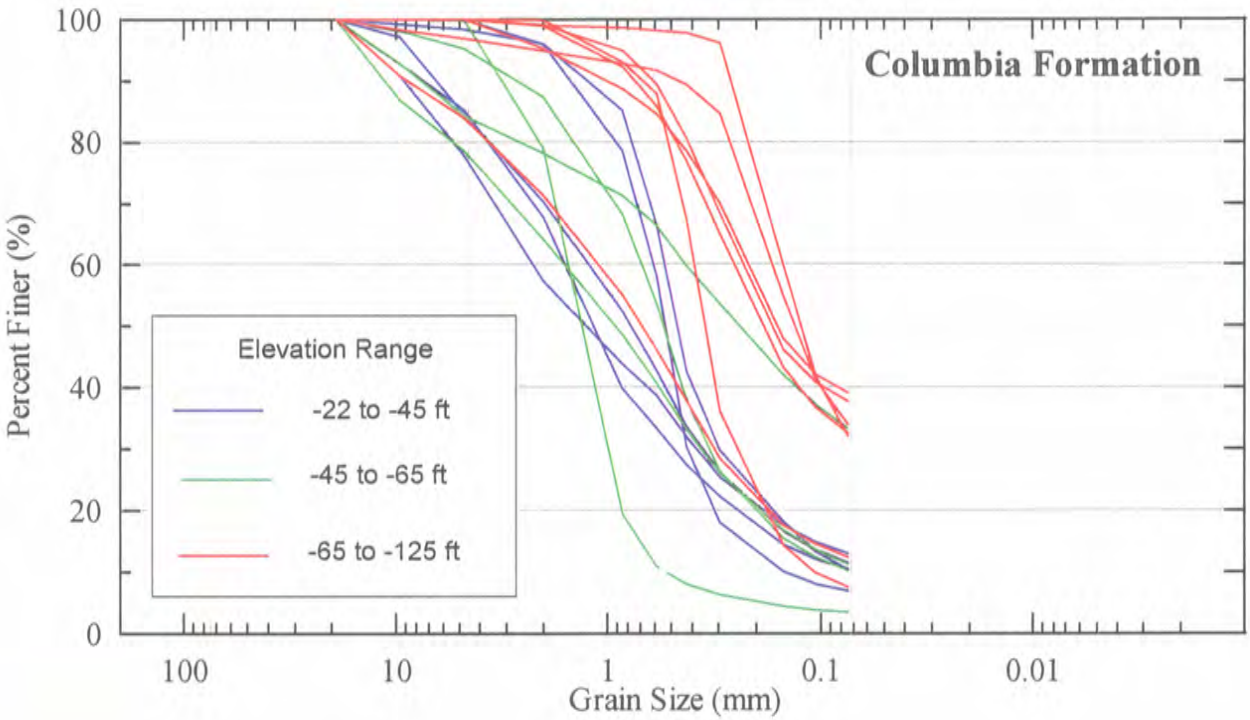
Profile of Saturated and Dry Unit Weight of Fine-Grained Soils

FIGURE 6-1

PROJECT: ME0250
Cherry Island

Cobbles	Gravel		Sand			Fines (Silt and Clay)
	Coarse	Fine	Coarse	Medium	Fine	
Size (mm)	19	4.75	2.0	0.425	0.075	
Sieve No.		#4	#10	#20#40	#60 #200	





Grain Size Dredge-Col-Pot.grf 3/27/03

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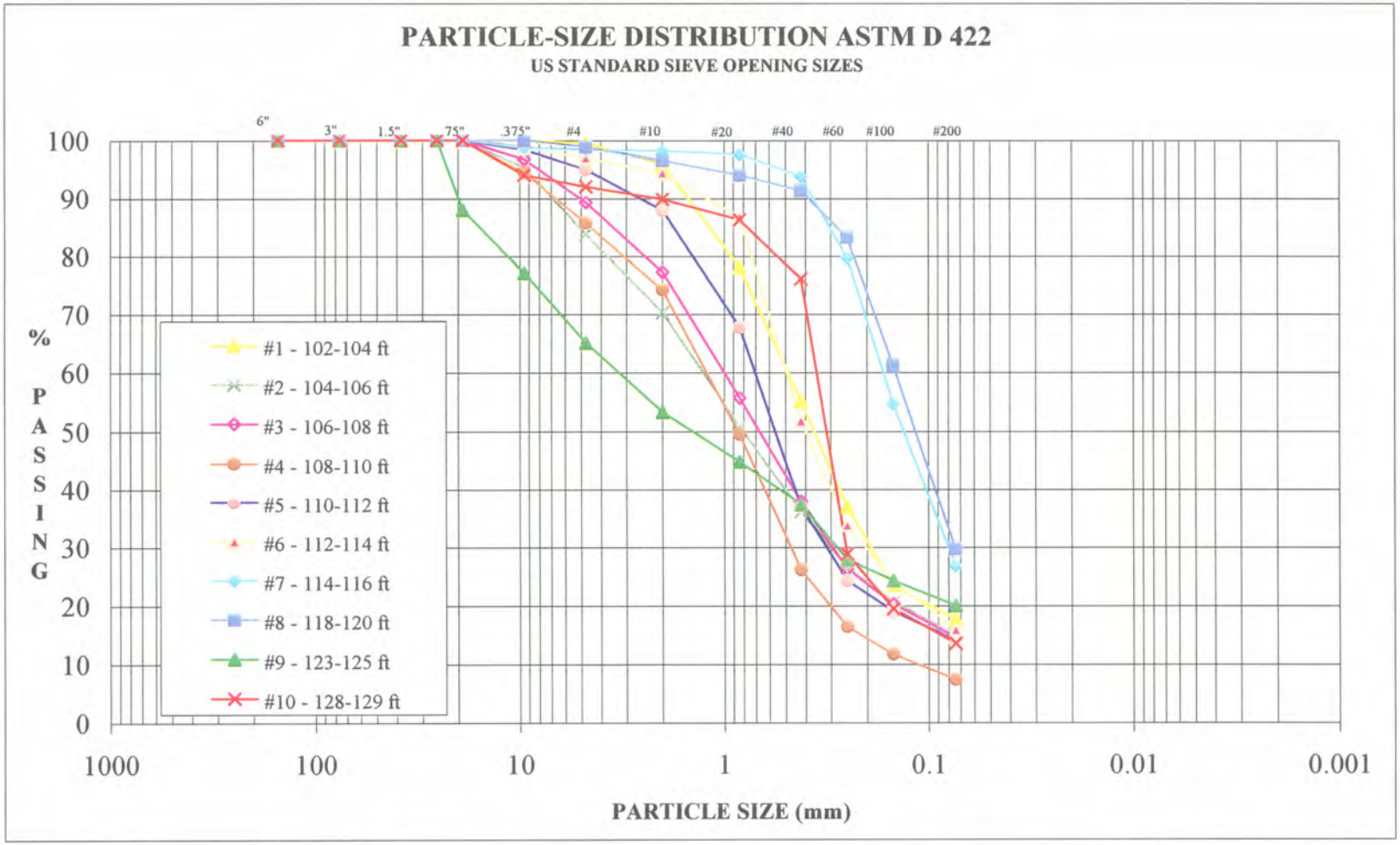
Particle Size Distribution of Dredge, Alluvium and Potomac Formation

FIGURE 6-2

PROJECT: ME0250
Cherry Island



	Coarse	Fine	Coar.	Med	Fine	Silt	Clay
COBBLE	GRAVEL		SAND			FINES	

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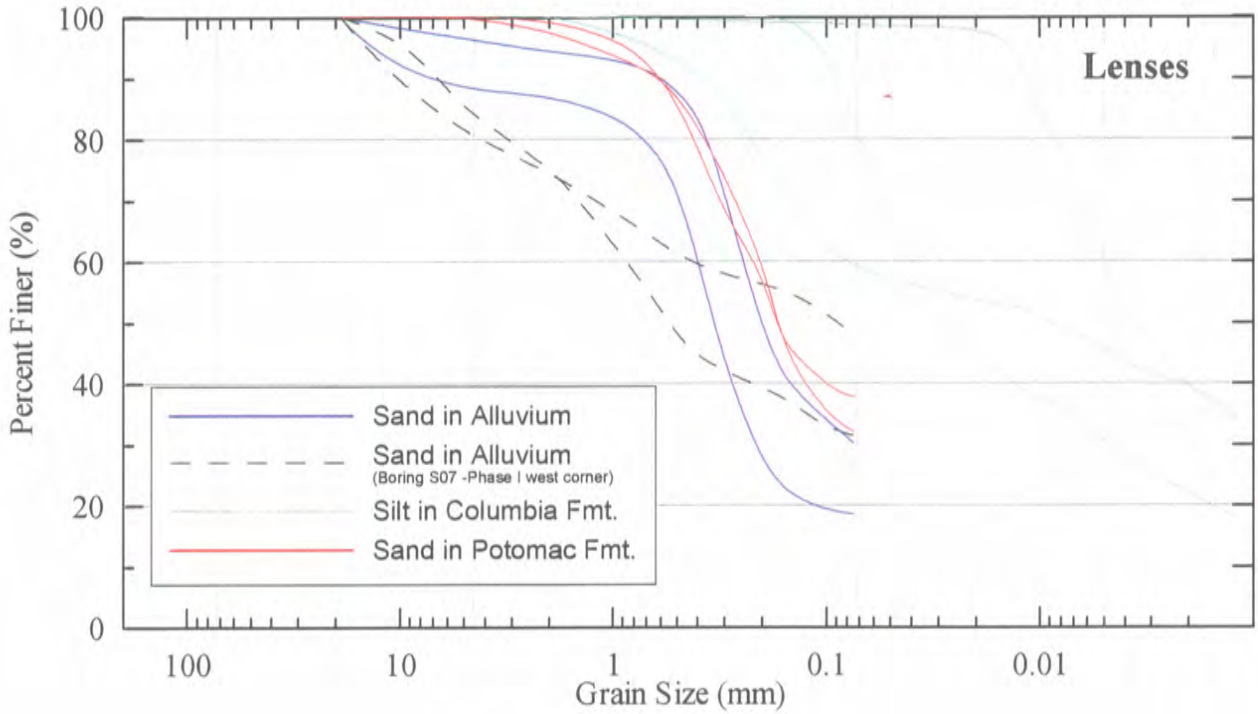
Grain Size Distribution
Columbia Formation
Boring P5-03-S20 (continuous sampling)

Figure 6-3

PROJECT: ME0250
Cherry Island

summary of sieve in Area 6.xls

Cobbles	Gravel		Sand			Fines (Silt and Clay)
	Coarse	Fine	Coarse	Medium	Fine	
Size (mm)	19	4.75	2.0	0.425	0.075	
Sieve No.		#4	#10	#20 #40	#60 #200	



Grain Size - lenses.grf 3/27/03

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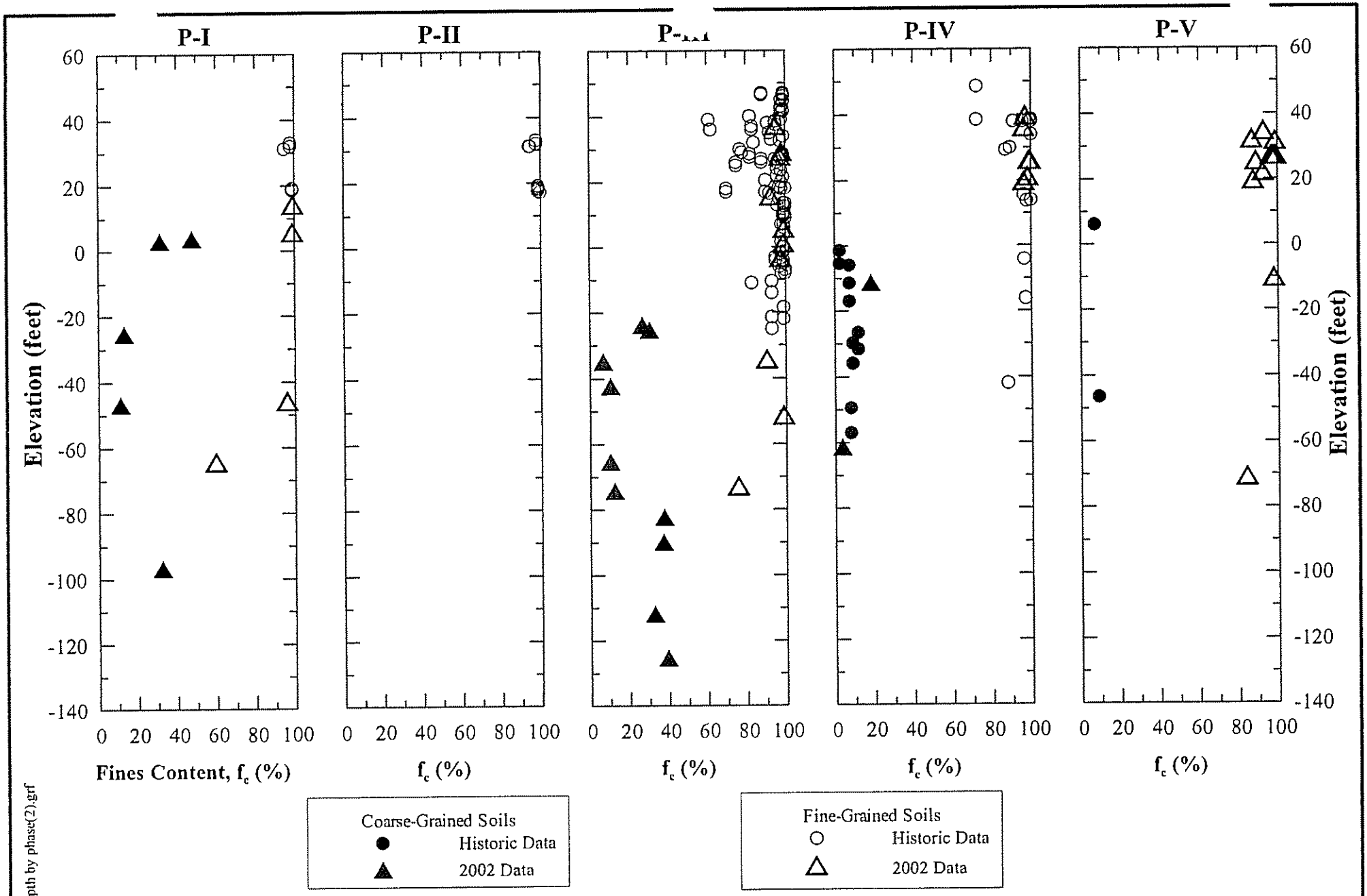
Particle Size Distribution of Materials in Lenses

FIGURE 6-4

PROJECT: ME0250
Cherry Island



Fines content w depth by phase(2).grf

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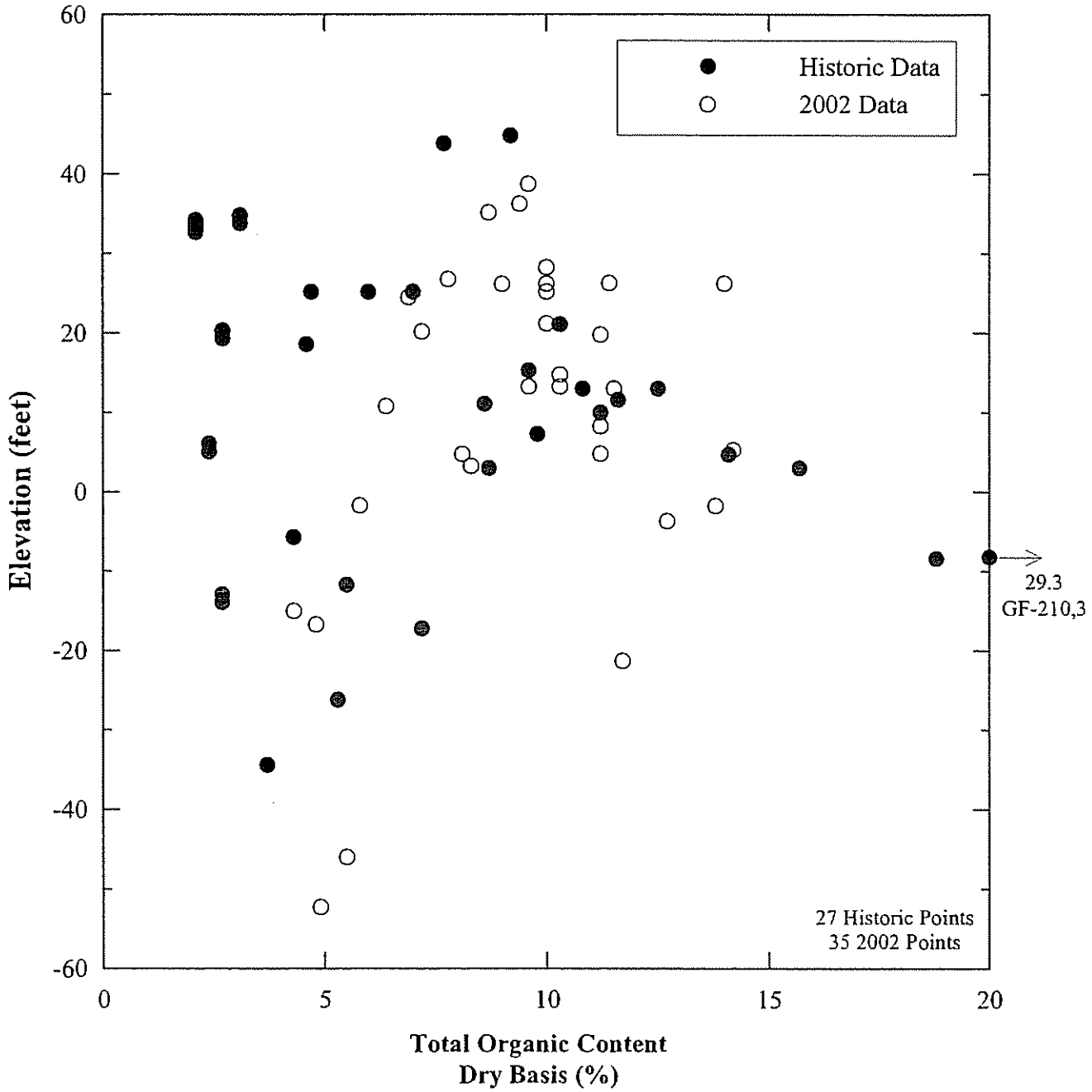
Profiles of Fines Content by Phase

FIGURE 6-5

PROJECT: ME0250 Cherry Island



Note: peaty samples excluded

Organic Content w Depth(2).grf

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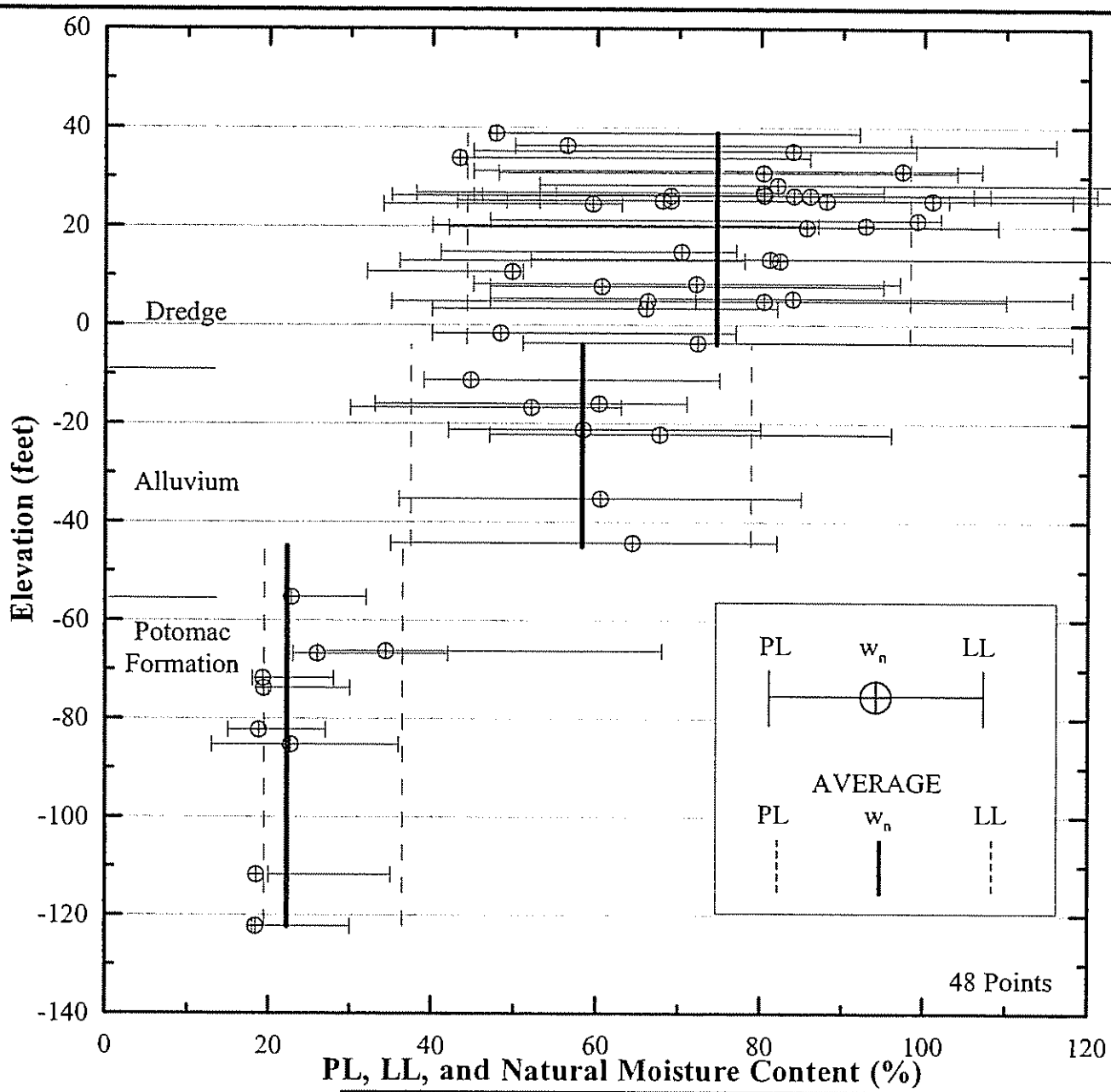
Profile of Organic Content by Elevation

FIGURE 6-6

PROJECT: ME0250
Cherry Island



Material		PL	w_n	LL	PI
Dredge	μ	44	75	98	54
	σ	6	15	19	14
	COV	13	20	19	25
	n	32			
Alluvium	μ	37	58	79	41
	σ	6	8	11	7
	COV	15	13	13	16
	n	7			
Potomac	μ	19	22	36	17
	σ	4	5	13	10
	COV	21	24	35	61
	n	9			

NOTE: μ = Mean; σ = Standard Deviation; COV = Coefficient of Variation (%); n = Number of Points

all Atterberg w depth(2).grf

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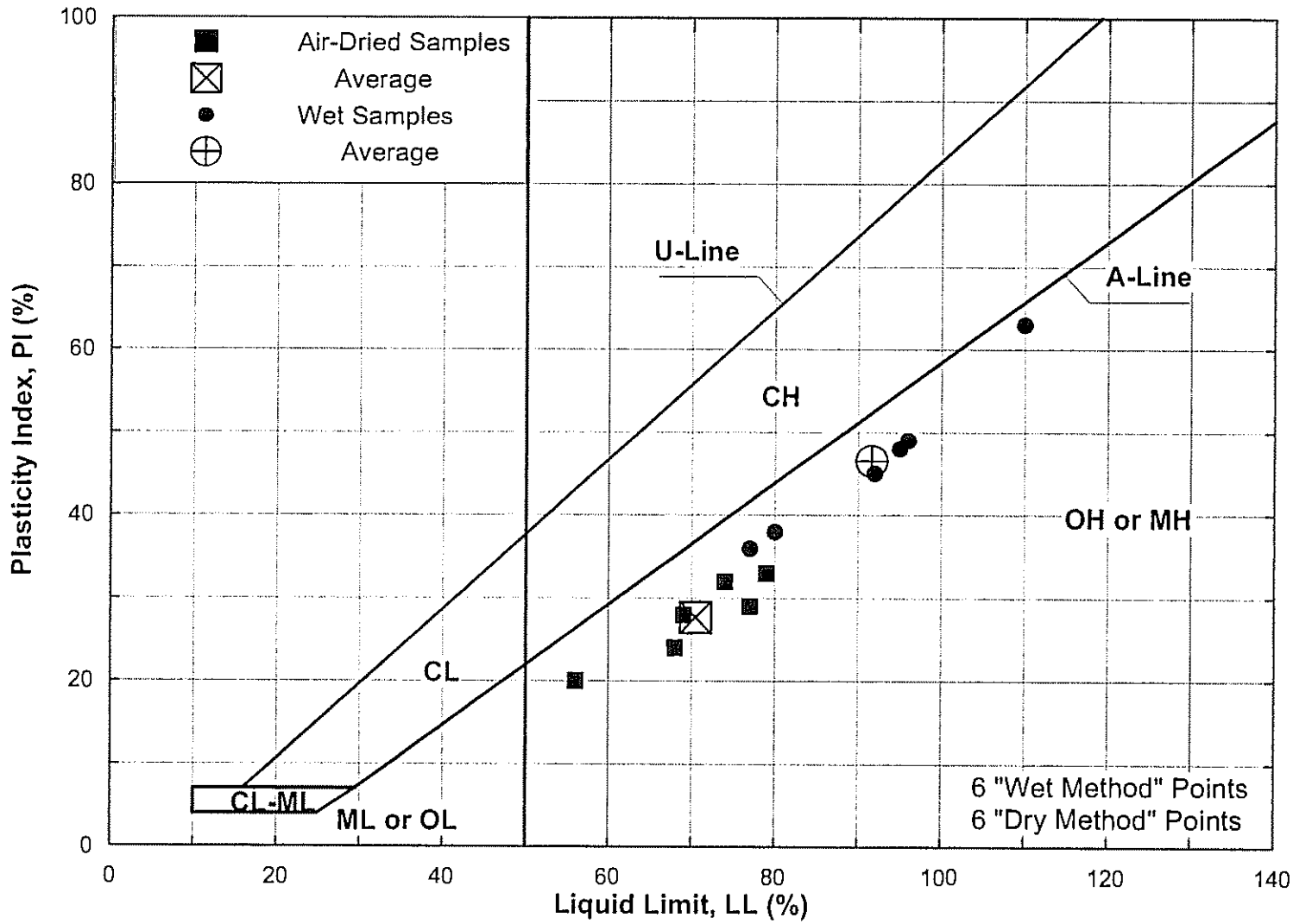
Profiles of Plastic Limit, Natural Moisture Content, and Liquid Limit

FIGURE 6-7

PROJECT: ME0250
Cherry Island



Plasticity Chart for Benchmark Atterberg.grf 3/27/03

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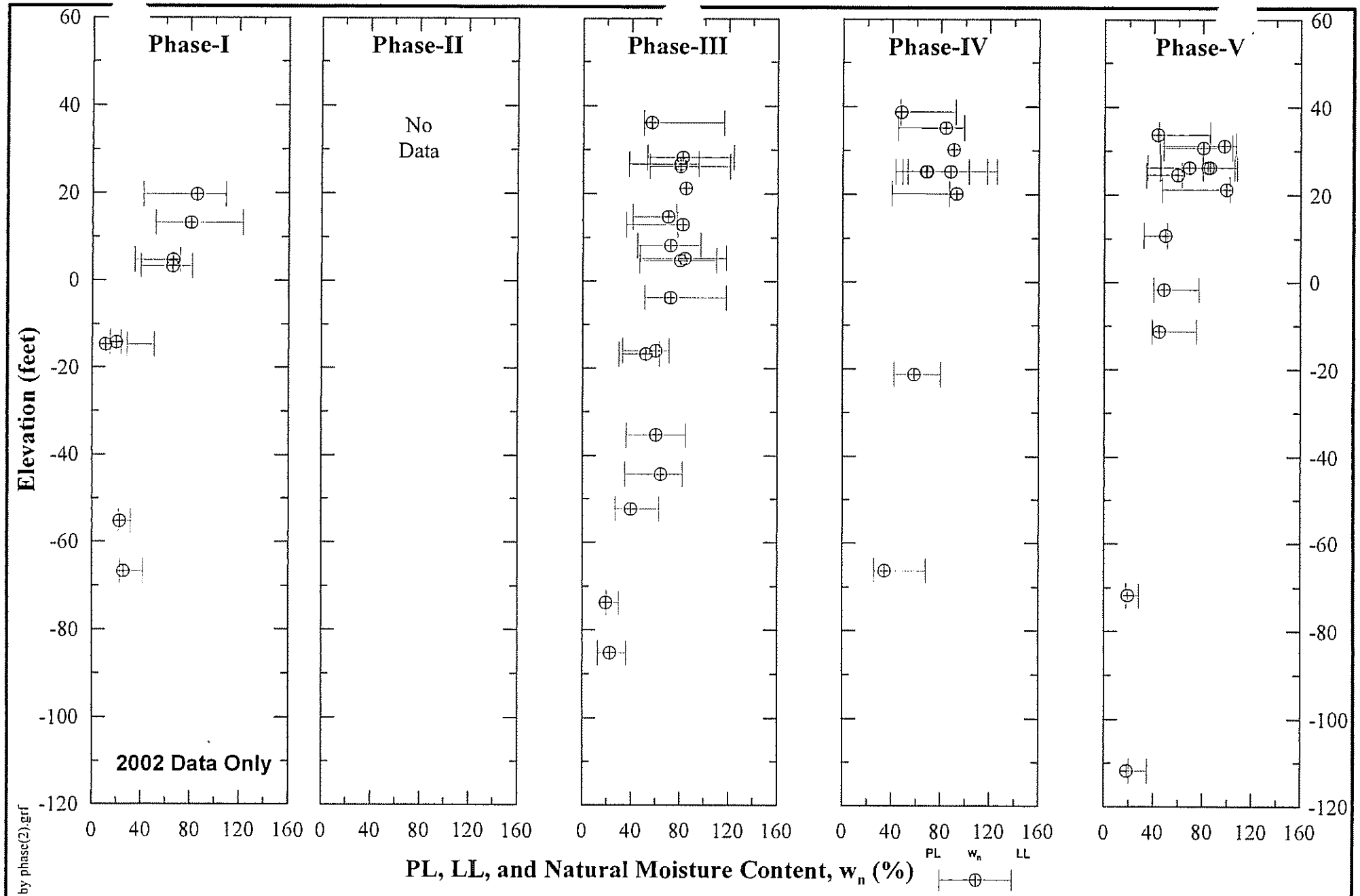
Comparison of Atterberg Limits based on Dry and Wet Sample Preparation

FIGURE 6-9

PROJECT ME0250
Cherry Island



Plasticity w depth by phase(2).grf

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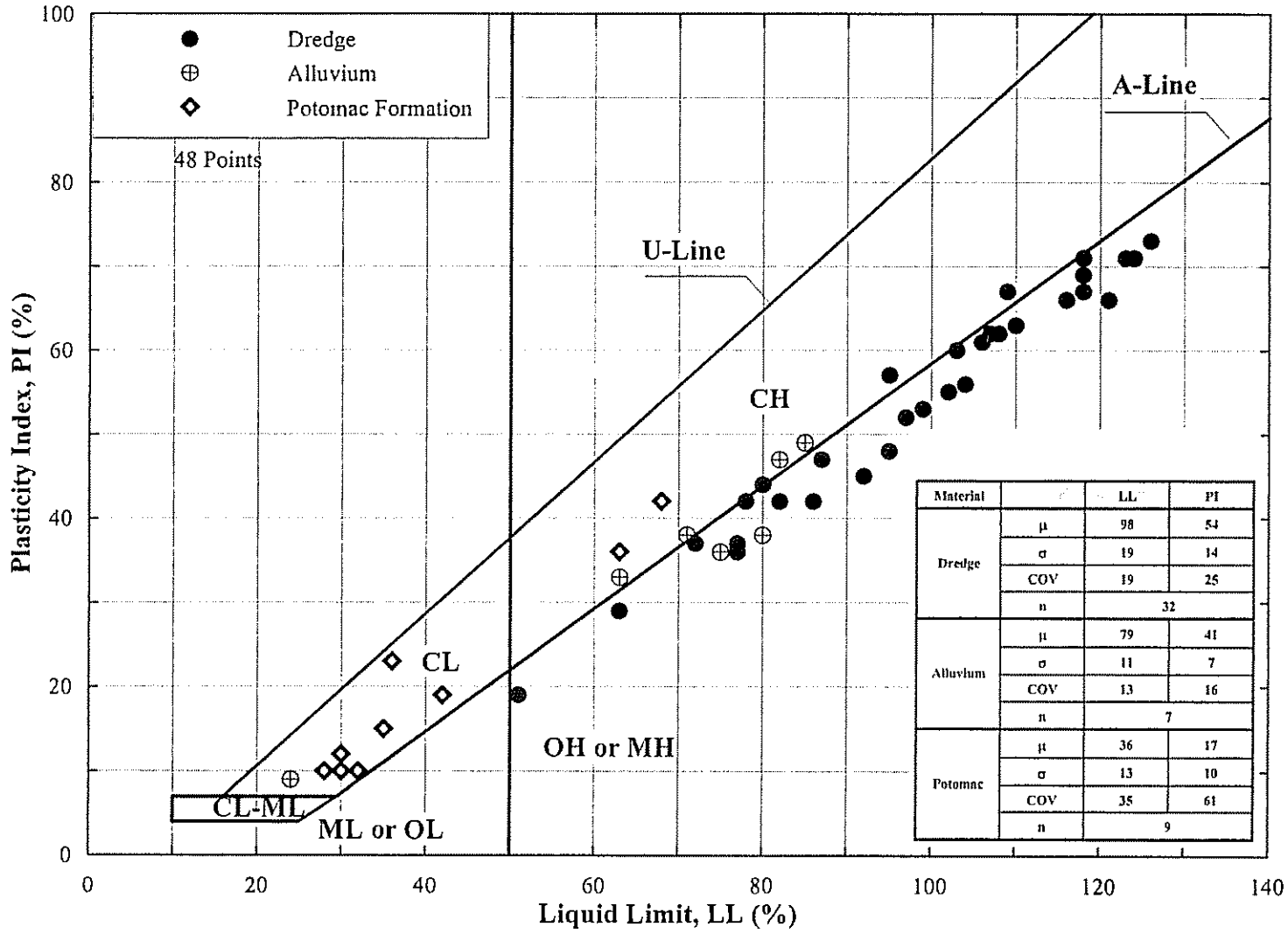
Profiles of Plastic Limit, Liquid Limit, and Natural Moisture Content by Phase

FIGURE 6-8

PROJECT: ME0250 Cherry Island



Material		LL	PI
Dredge	μ	98	54
	σ	19	14
	COV	19	25
	n	32	
Alluvium	μ	79	41
	σ	11	7
	COV	13	16
	n	7	
Potomac	μ	36	17
	σ	13	10
	COV	35	61
	n	9	

NOTE: μ = Mean; σ = Standard Deviation; COV = Coefficient of Variation (%); n = Number of Points

Peaty soil not included

Plasticity Chart(2).grf

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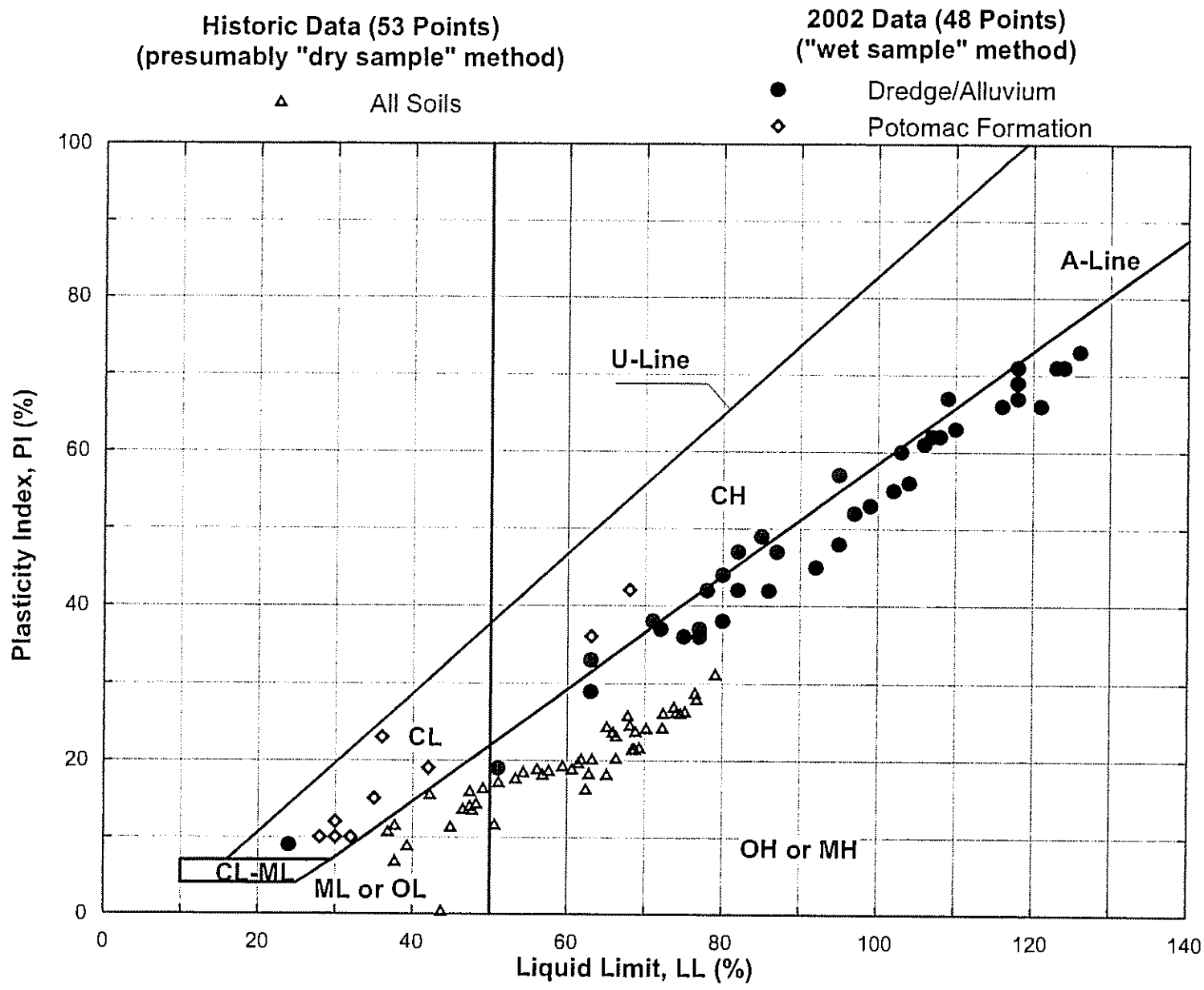
Atterberg Limits
2002 Data

FIGURE 6-10

PROJECT: ME0250
Cherry Island



Plasticity Chart-Historic vs 2002.grf 3/28/03

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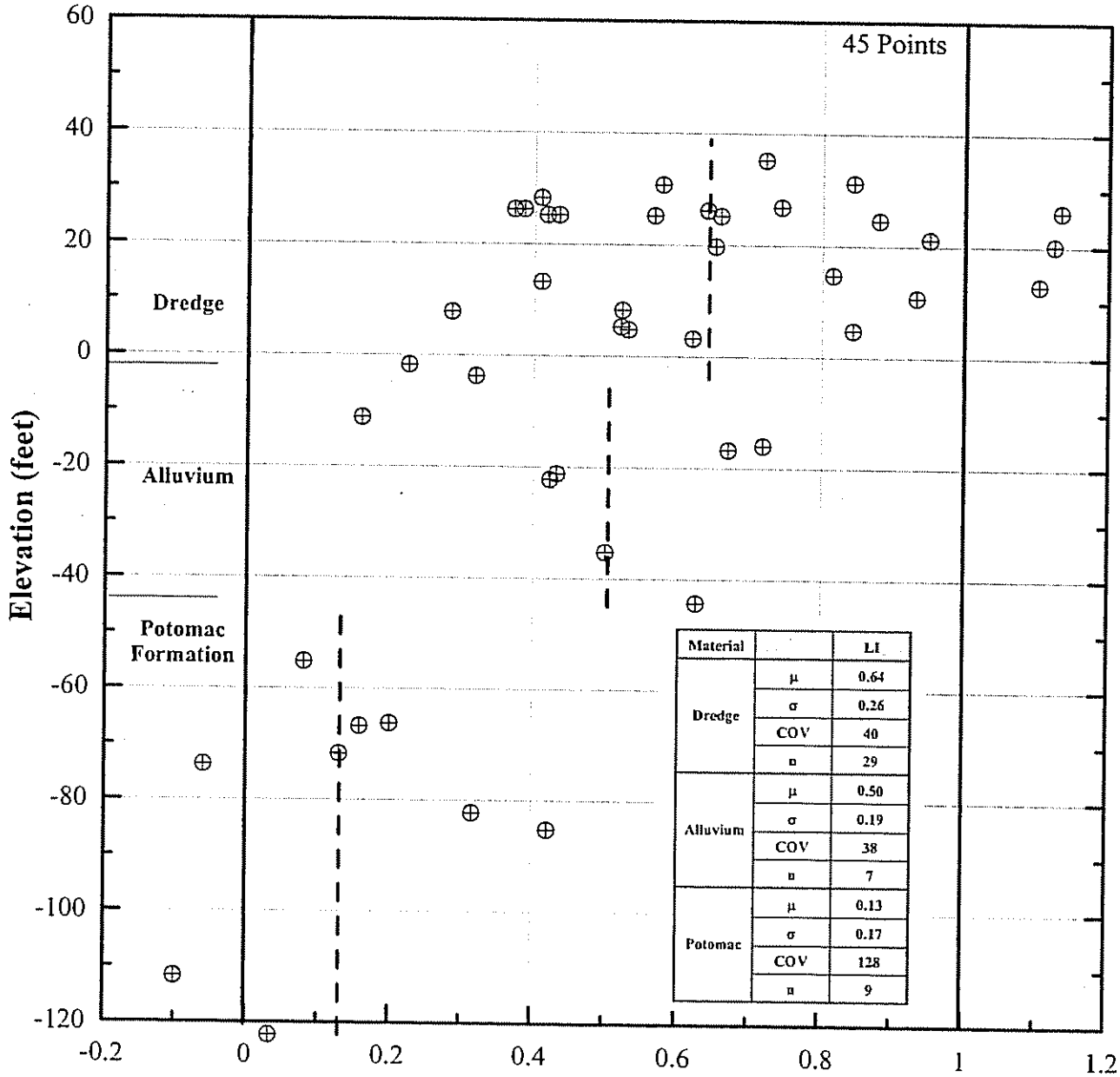
Comparison of Atterberg Limits from Historic and 2003 Data

FIGURE 6-11

PROJECT ME0250
Cherry Island



Material		LI
Dredge	μ	0.64
	σ	0.26
	COV	40
	n	29
Alluvium	μ	0.50
	σ	0.19
	COV	38
	n	7
Potomac	μ	0.13
	σ	0.17
	COV	128
	n	9

$$\text{Liquidity Index} = \frac{w_n - PL}{LL - PL}$$

NOTE: μ = Mean; σ = Standard Deviation; COV = Coefficient of Variation (%); n = Number of Points

Liquidity w elevation(2).grf

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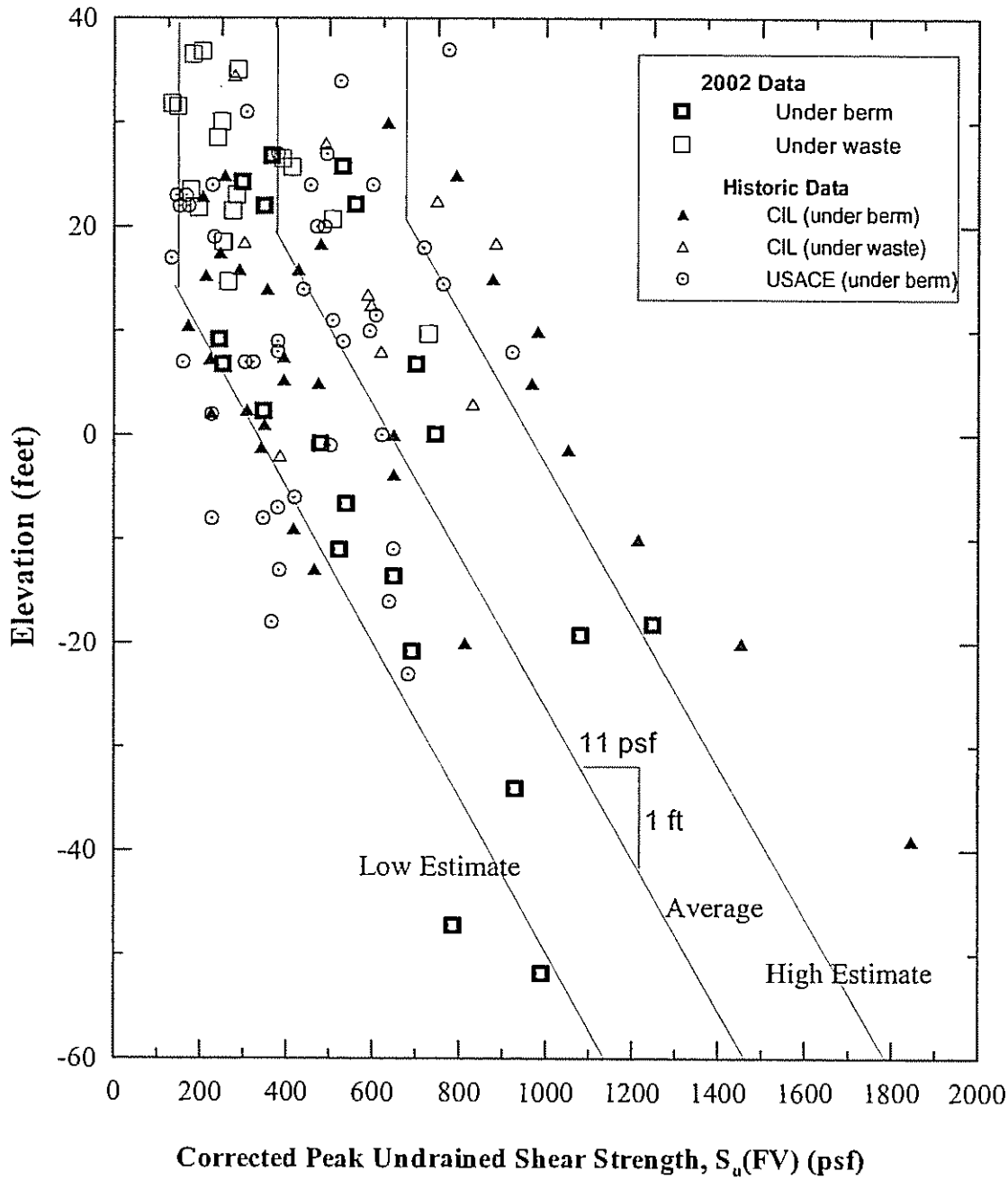
Profile of Liquidity Index

FIGURE 6-12

PROJECT: ME0250
Cherry Island



Su by elevation(2).grf

NOTE: Historic data has been corrected with a Plastic Index (PI) of 54 in the Dredge and PI = 41 in the Alluvium (approximately below elevation -5 ft).

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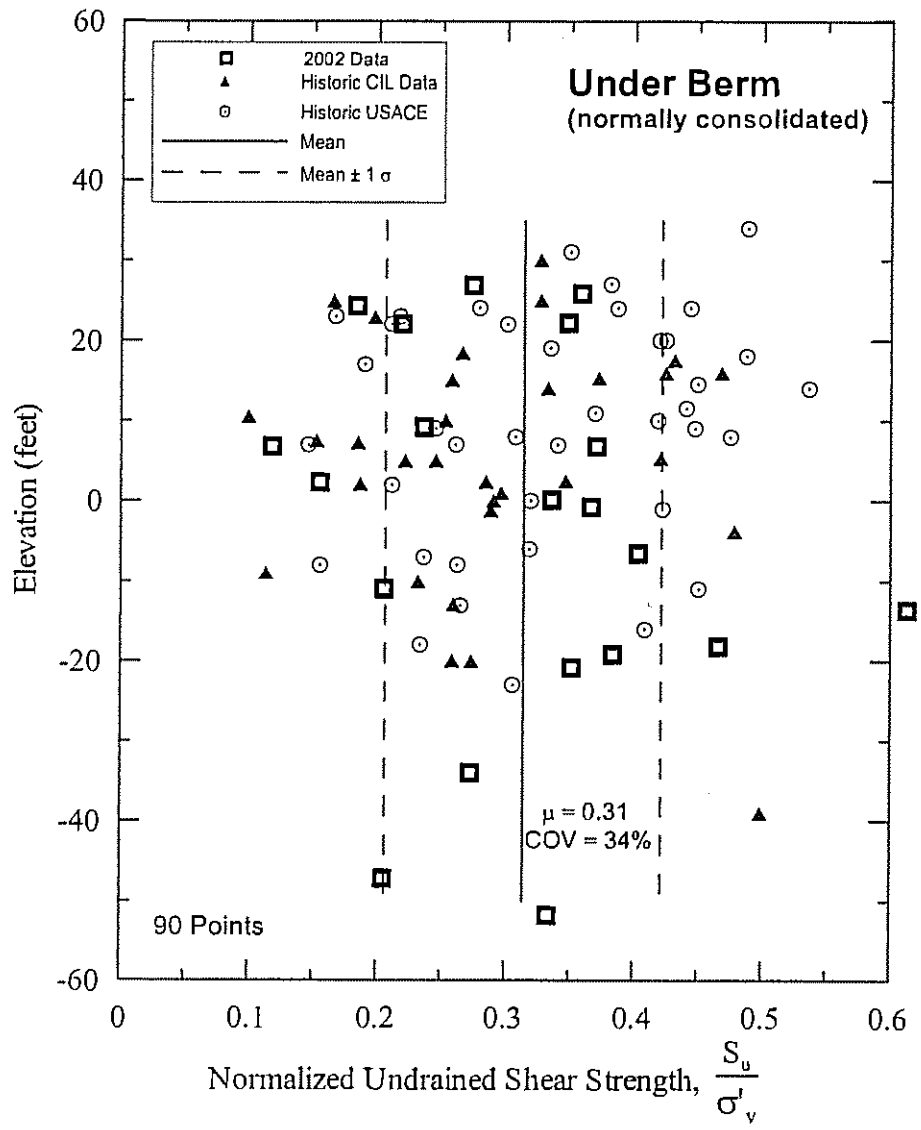
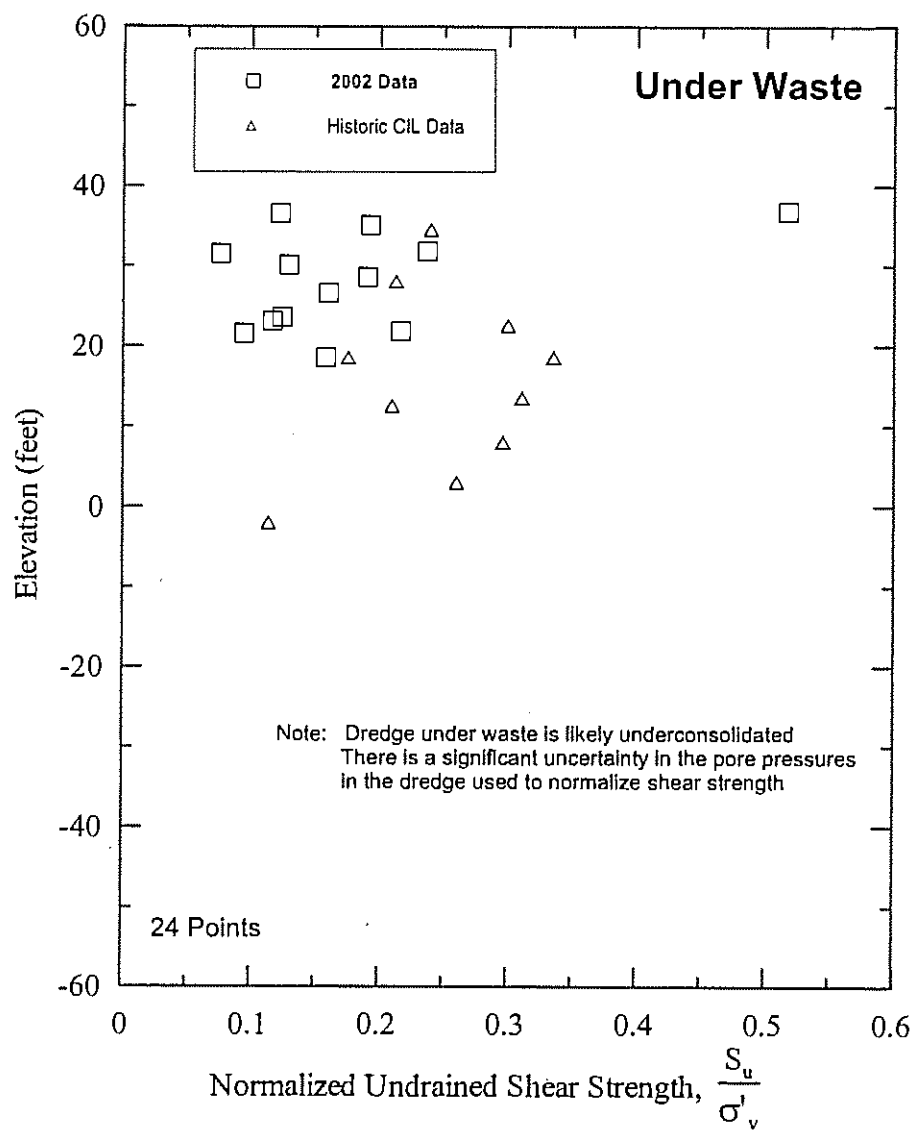
Profile of VST-based Undrained Shear Strength

FIGURE 6-13

PROJECT: ME0250
Cherry Island





Su (FV) over sigma vs elevation4.grf

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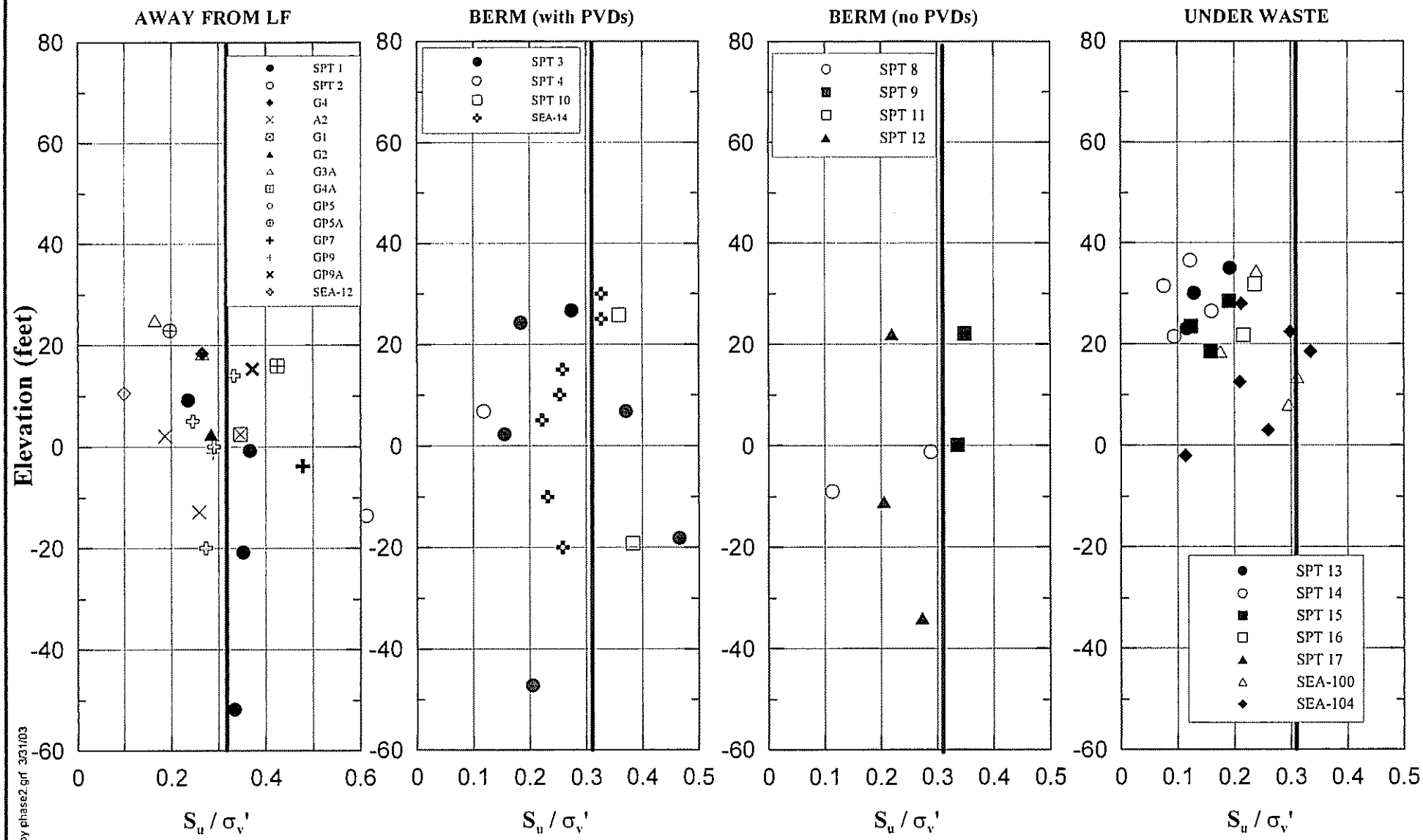
Profiles of VST-based Normalized Undrained Shear Strength of Dredge and Alluvium

FIGURE 6-14

PROJECT: ME0250
Cherry Island



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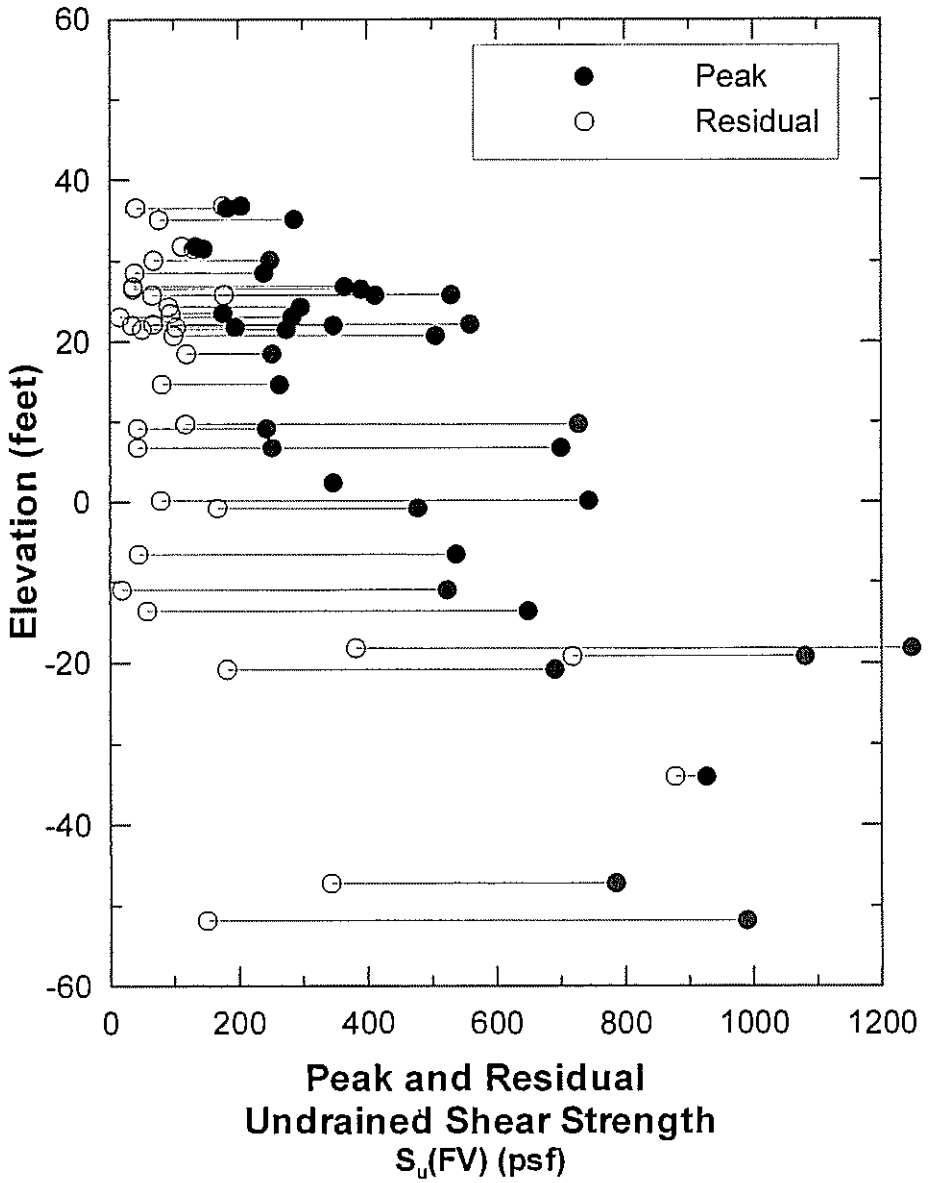
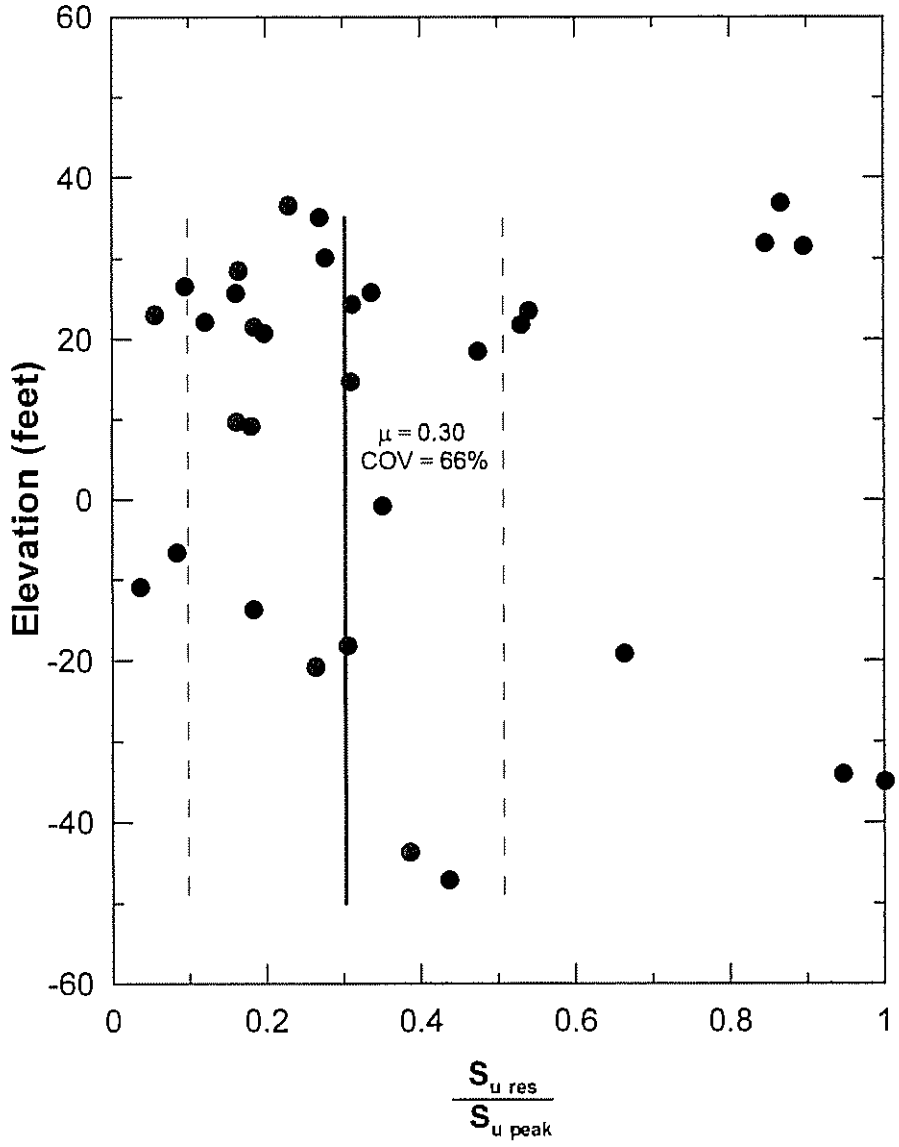
Profiles of VST-based Vane Tests S_u / σ_v' by Location

FIGURE 6-15

PROJECT: ME0250 Cherry Island





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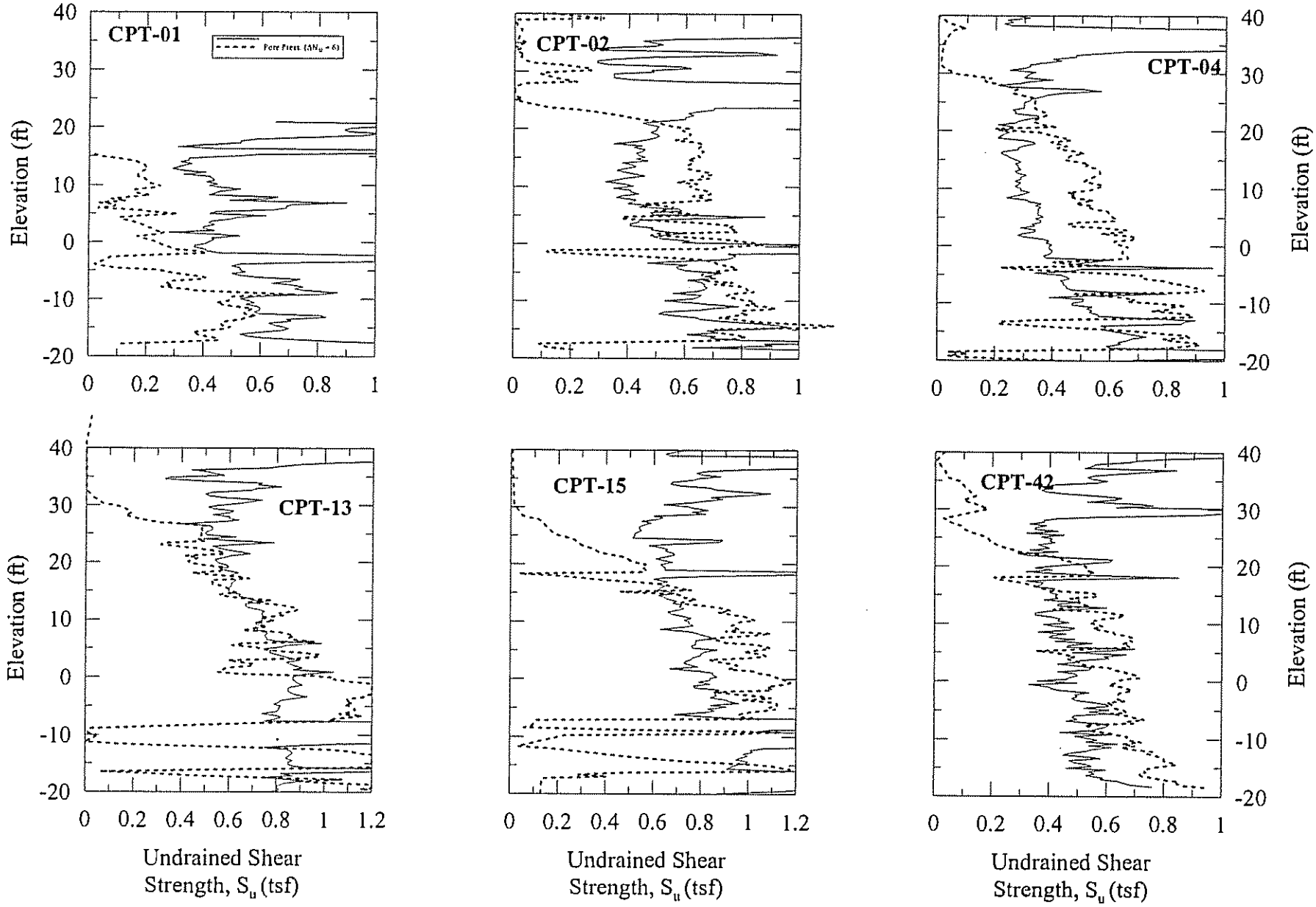
Profile of VST-based Residual Undrained Shear Strength

FIGURE 6-16

PROJECT: ME0250 Cherry Island

Note: N_k = 15 and N_u = 6 in all plots



su vs pp(2).grf

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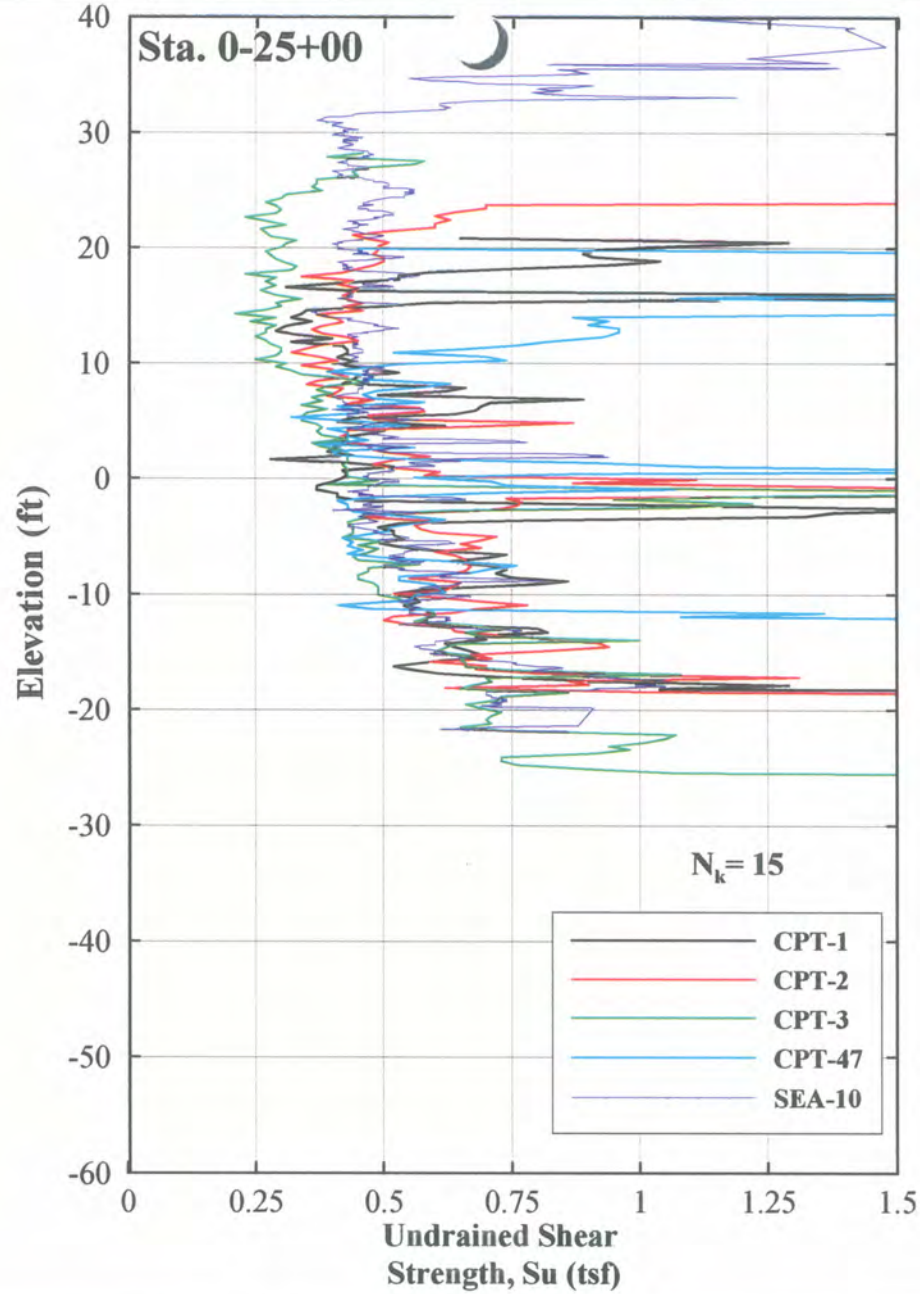
Comparison of Undrained Shear Strength with CPT N_k and N_u parameters

FIGURE 6-17

PROJECT: ME0250 Cherry Island



CPT Su - 0 2500(2).grf

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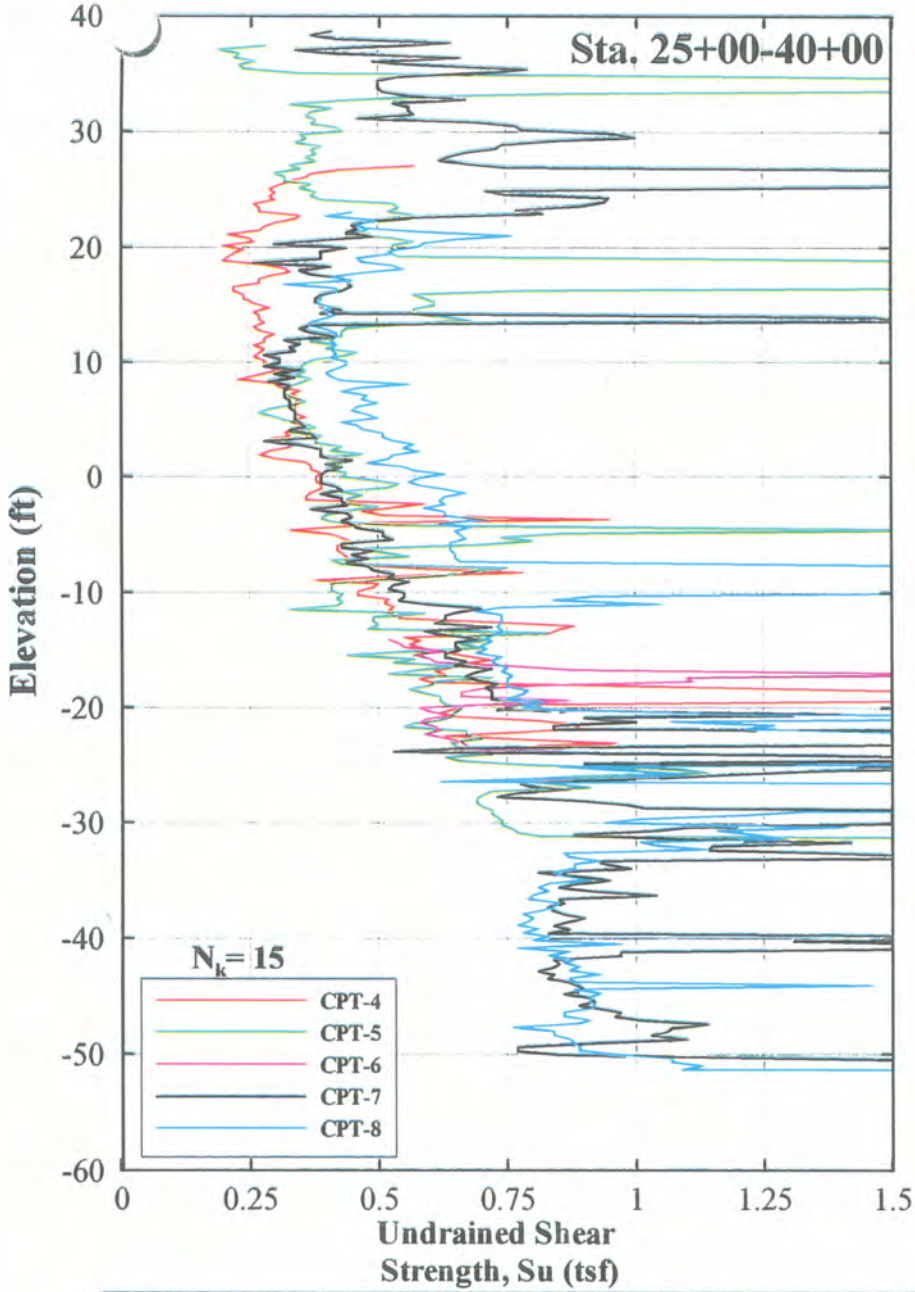
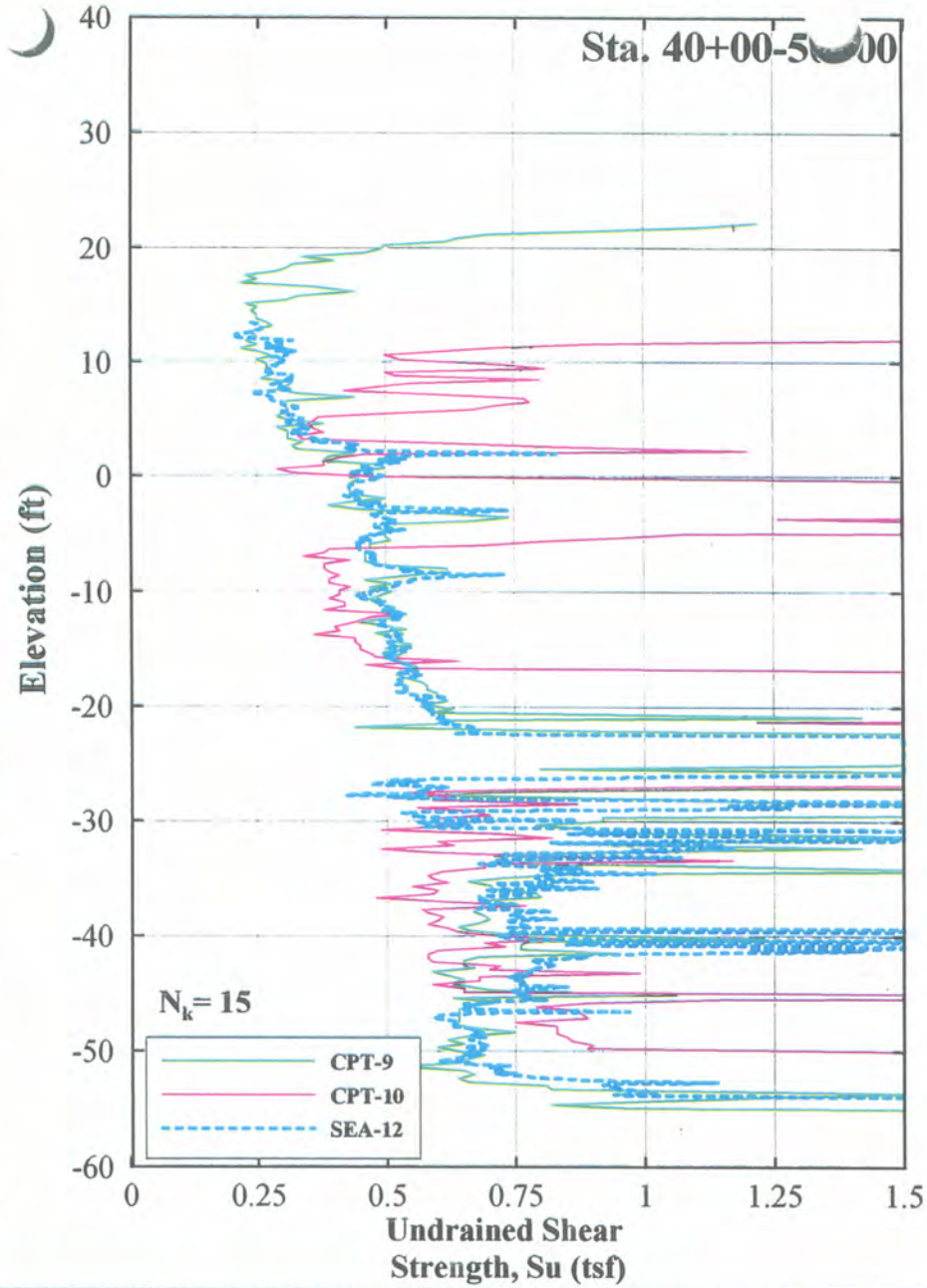
Profiles of Undrained Shear Strength with CPT Sta. 0-25+00

FIGURE 6-18

PROJECT: ME0250 Cherry Island





CPT Su - 2500 5000(2).grf

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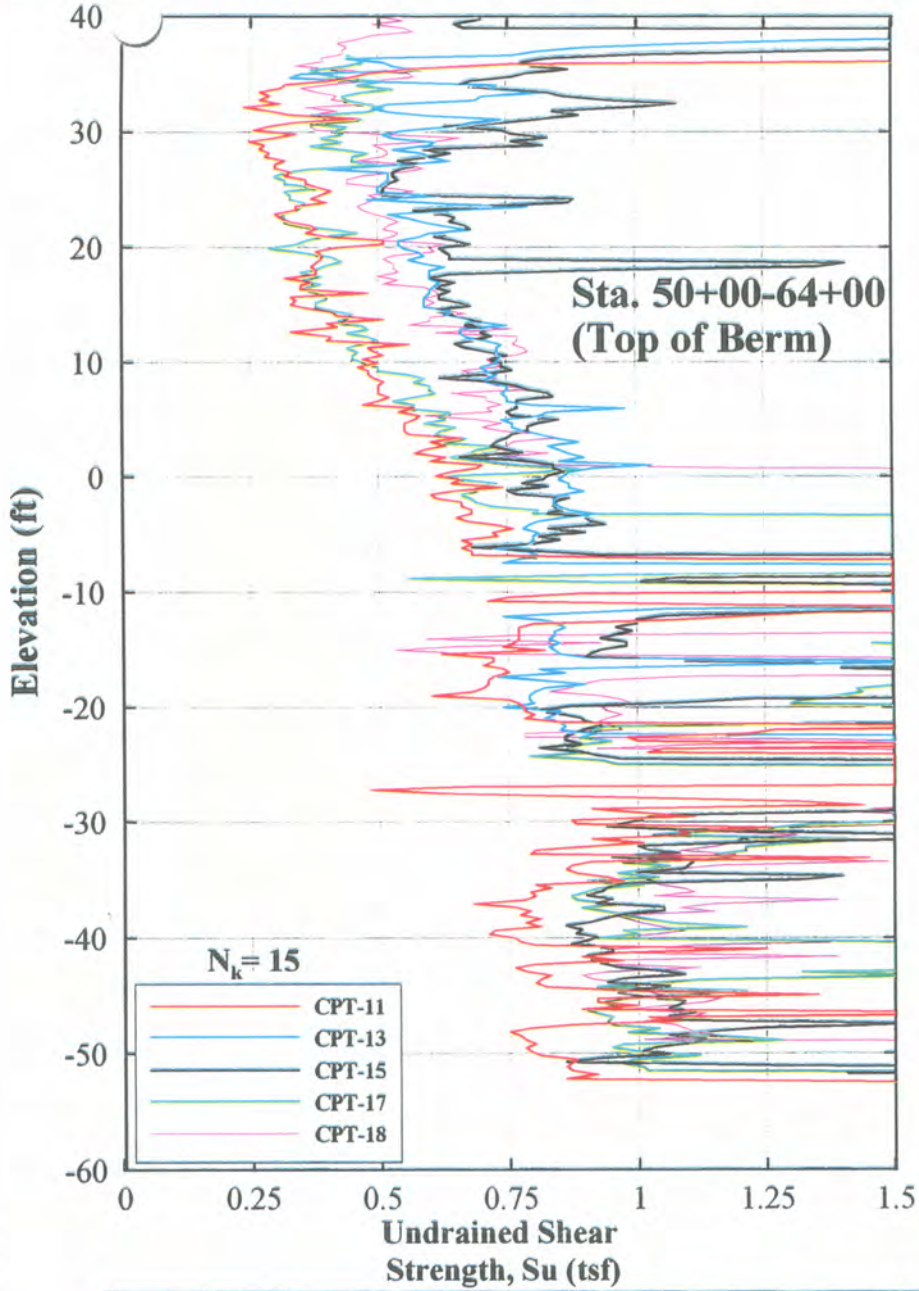
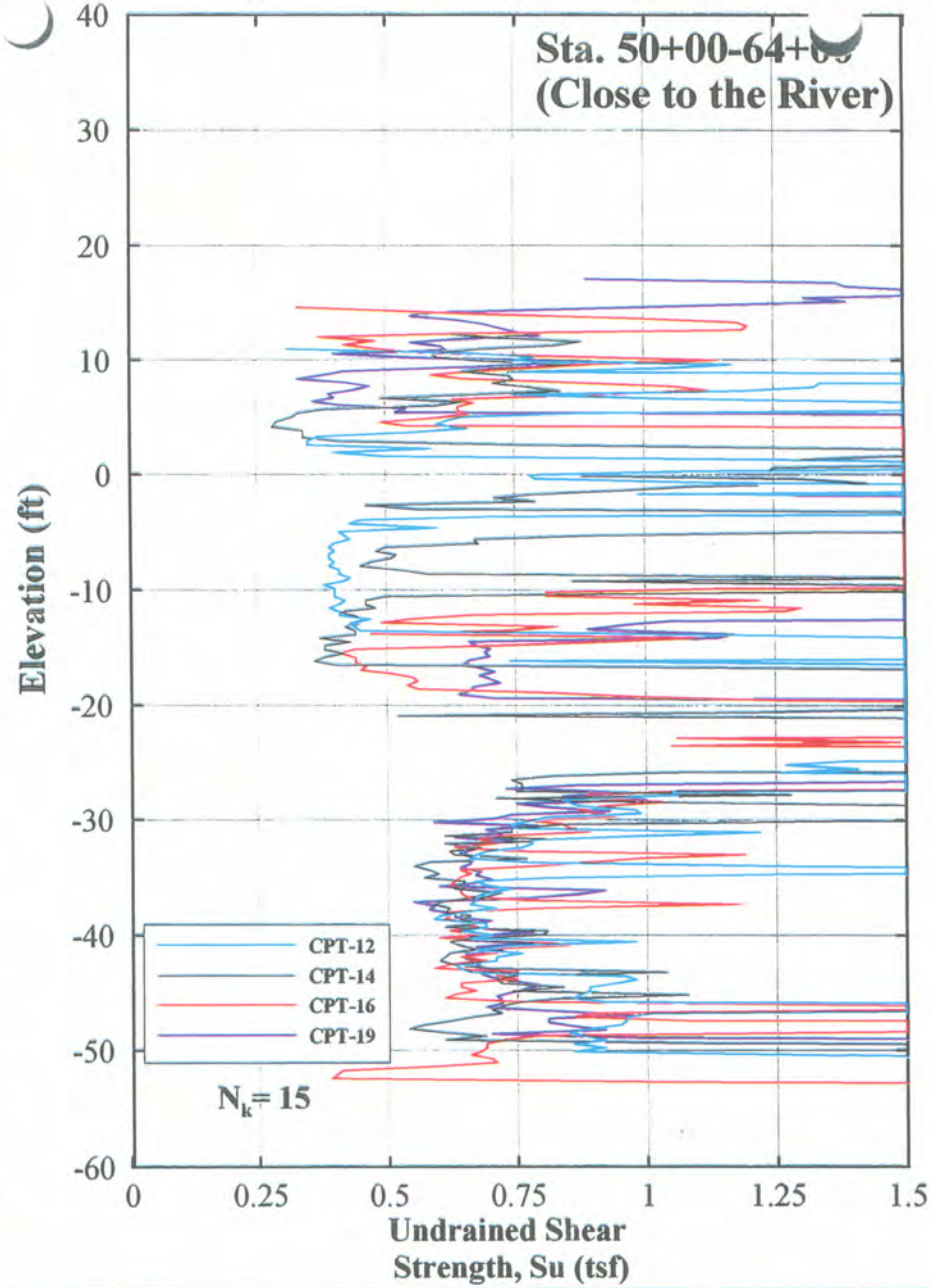
Profiles of Undrained Shear Strength with CPT Sta. 25+00-50+00

FIGURE 6-19

PROJECT: ME0250 Cherry Island





CPT Su - 5000 6400(2).grf

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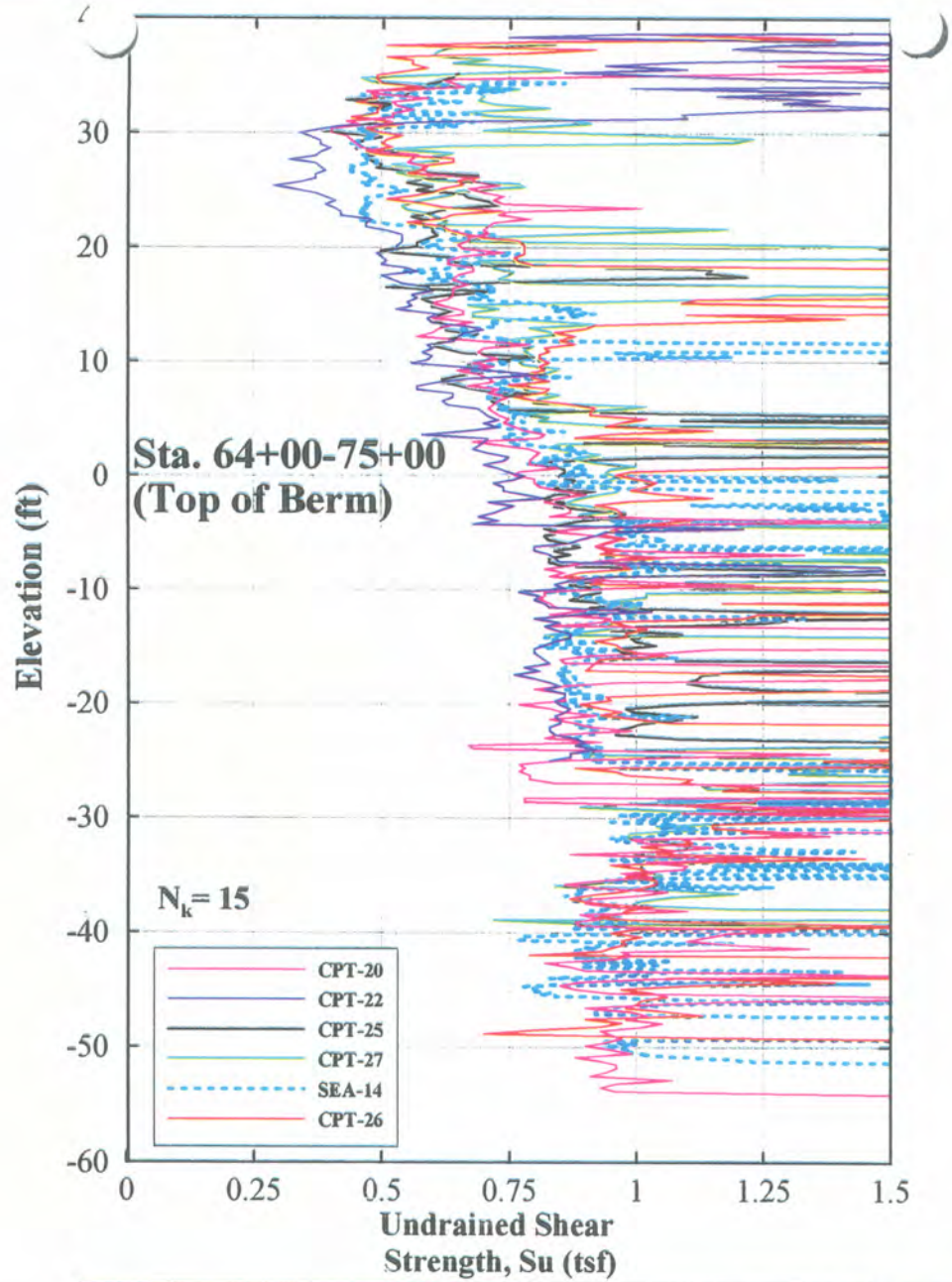
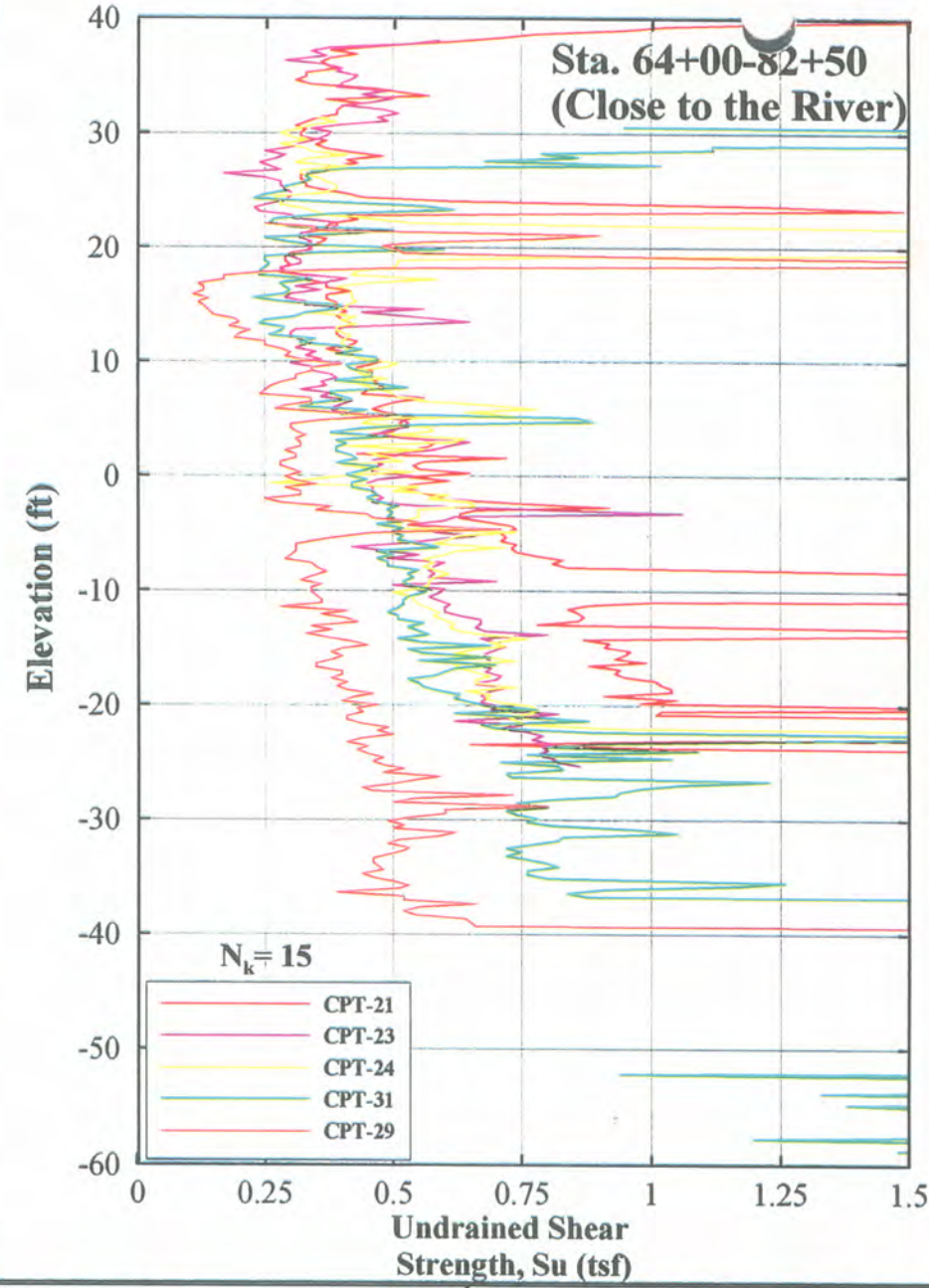
Profiles of Undrained Shear Strength with CPT Sta. 50+00-64+00

FIGURE 6-20

PROJECT: ME0250 Cherry Island





CPT Su - 6400 7500(2).grf

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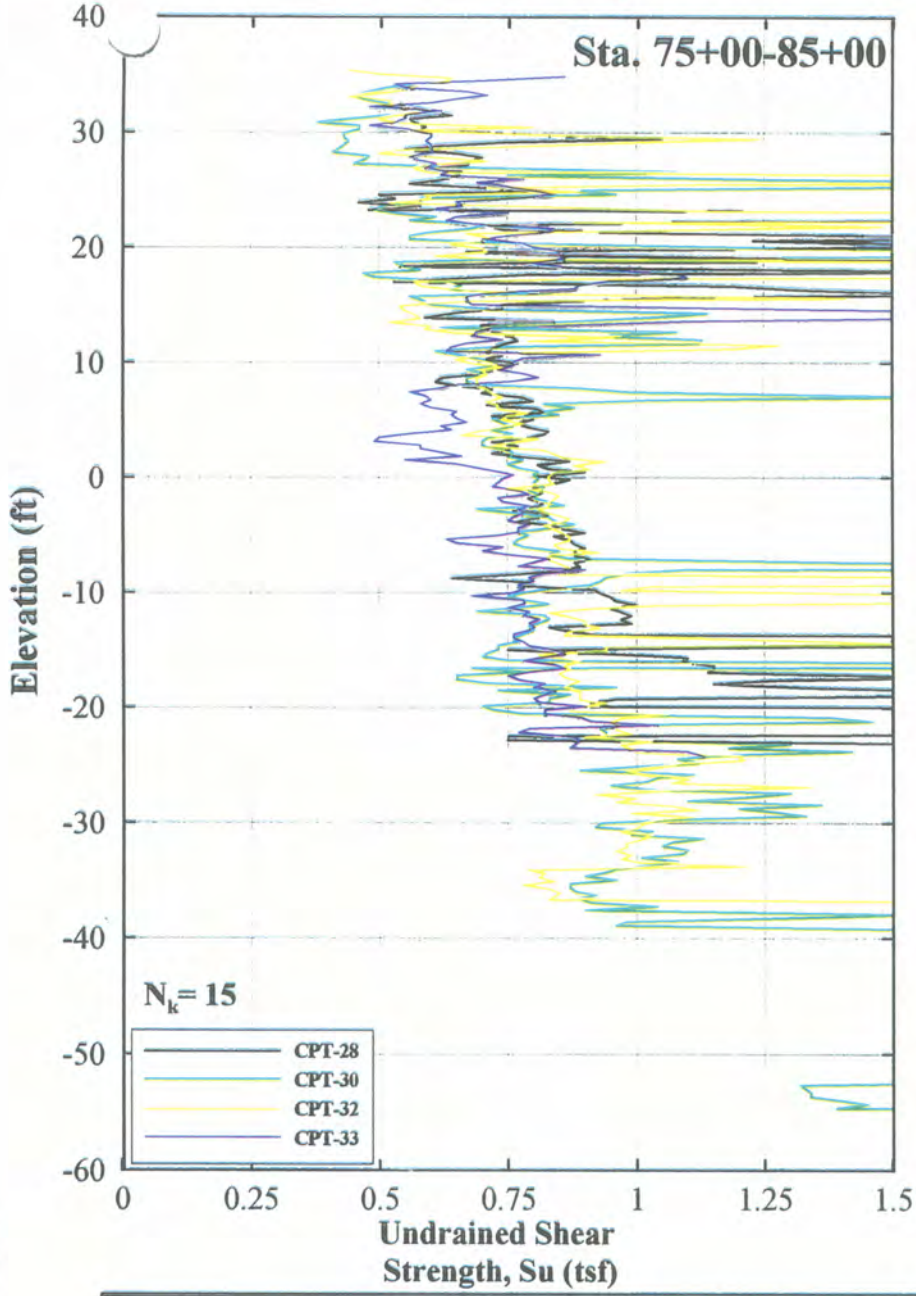
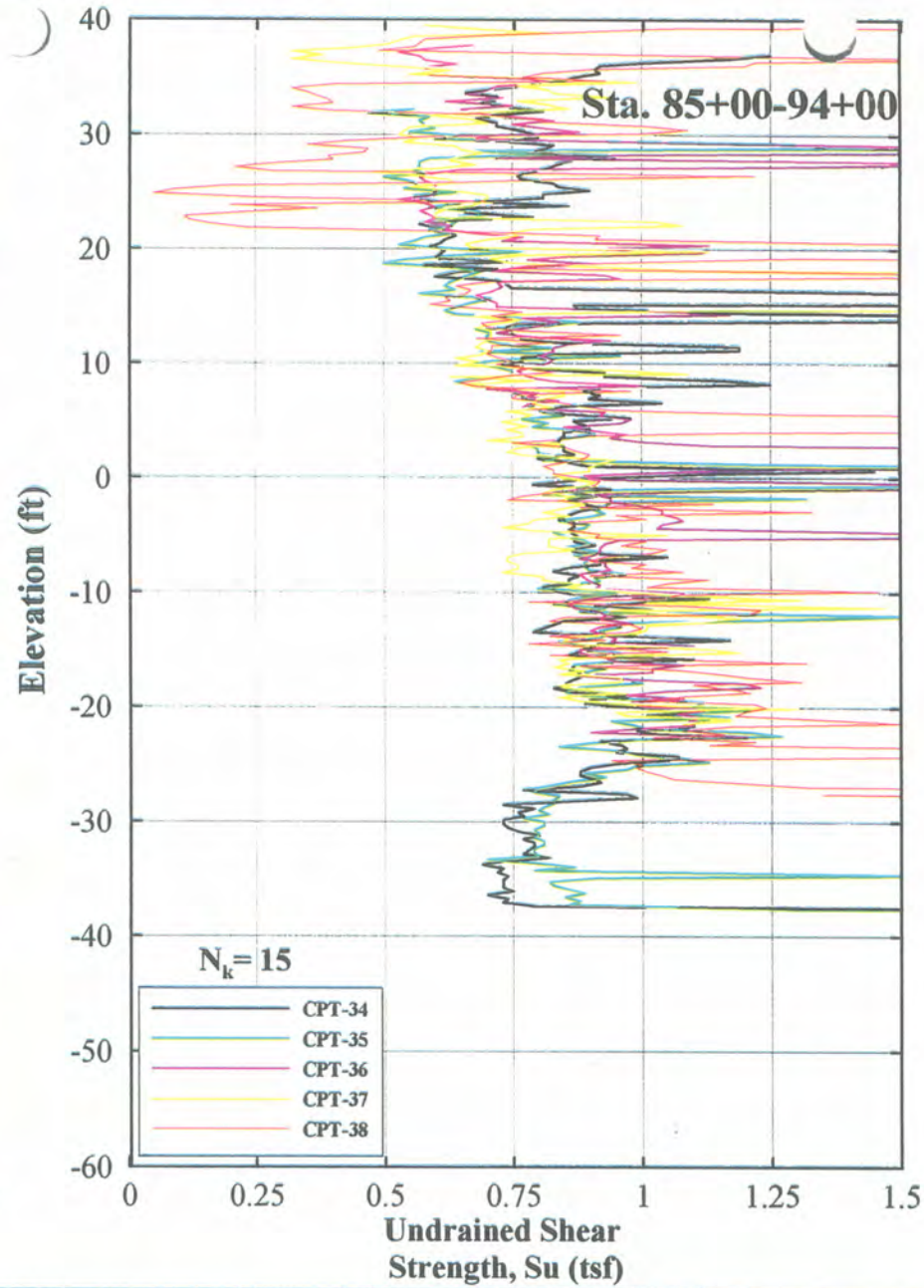
Profiles of Undrained Shear Strength with CPT Sta. 64+00-75+00

FIGURE 6-21

PROJECT: ME0250 Cherry Island





CPT Su - 7500 9400(2).grf

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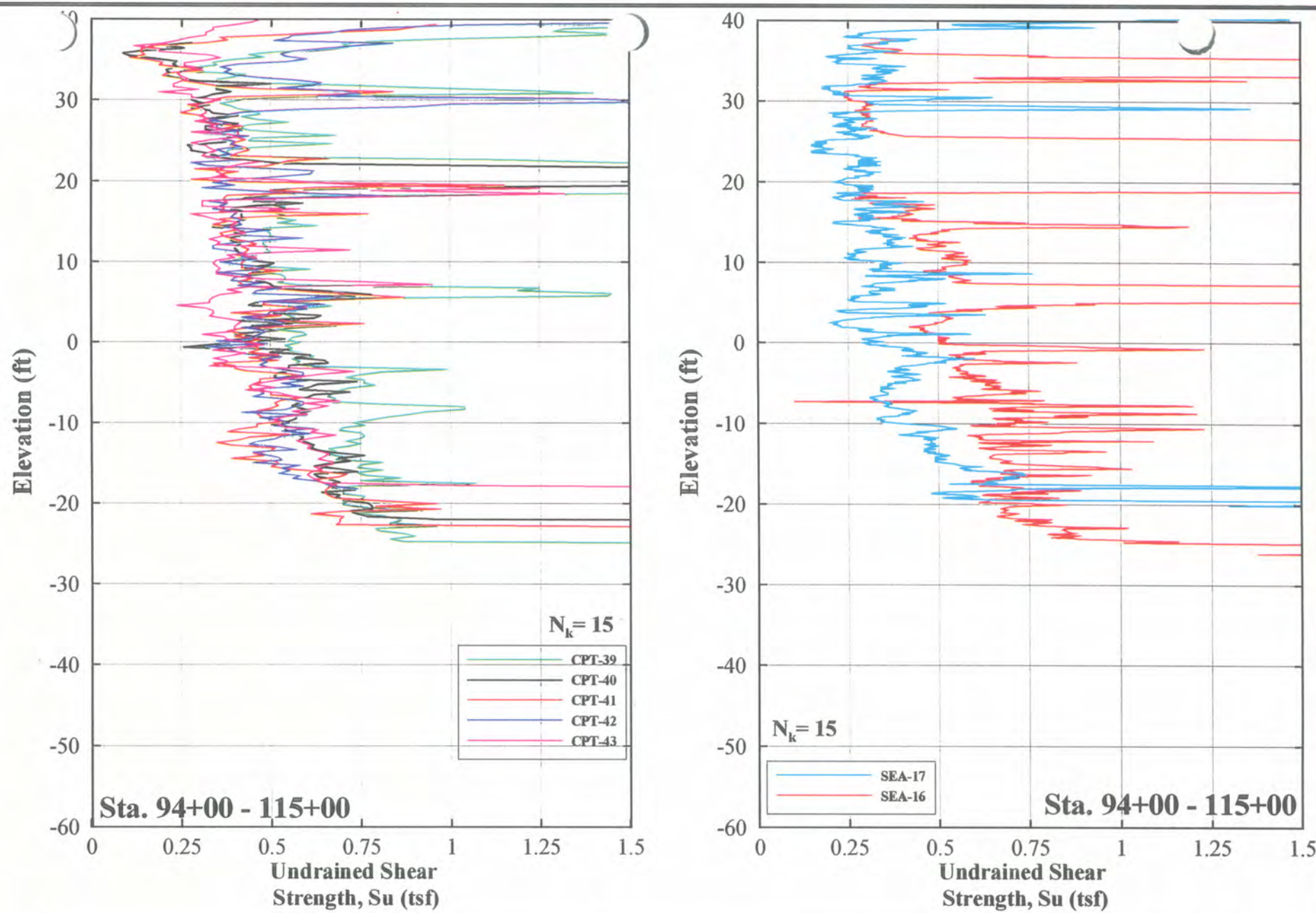
Profiles of Undrained Shear Strength with CPT Sta. 75+00-93+00

FIGURE 6-22

PROJECT: ME0250 Cherry Island



CPT Su - 9300 10000.grf

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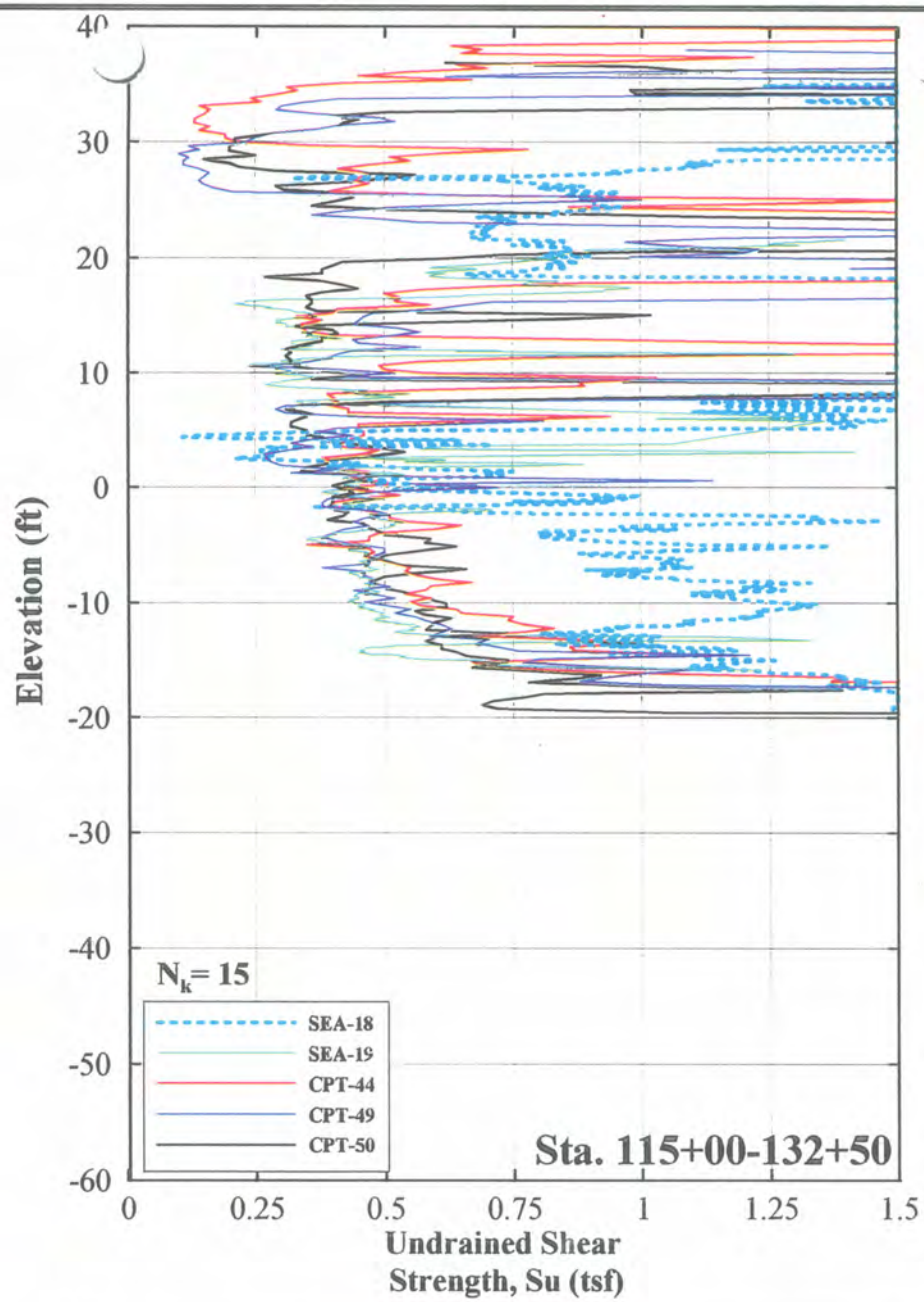
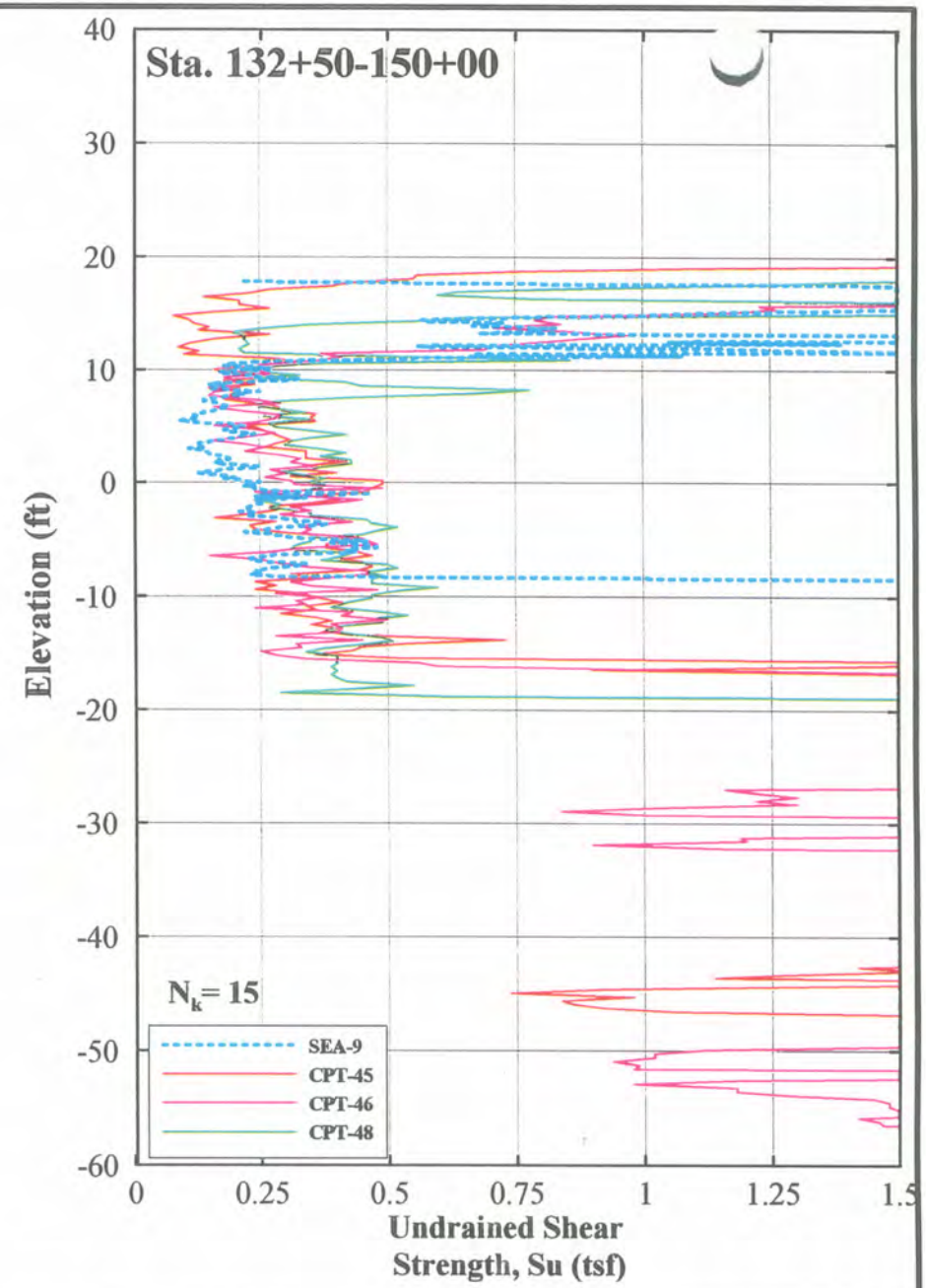
Profiles of Undrained Shear Strength with CPT Sta. 94+00-100+00

FIGURE 6-23

PROJECT: ME0250 Cherry Island





CPT Su -10000 12500(2).grf

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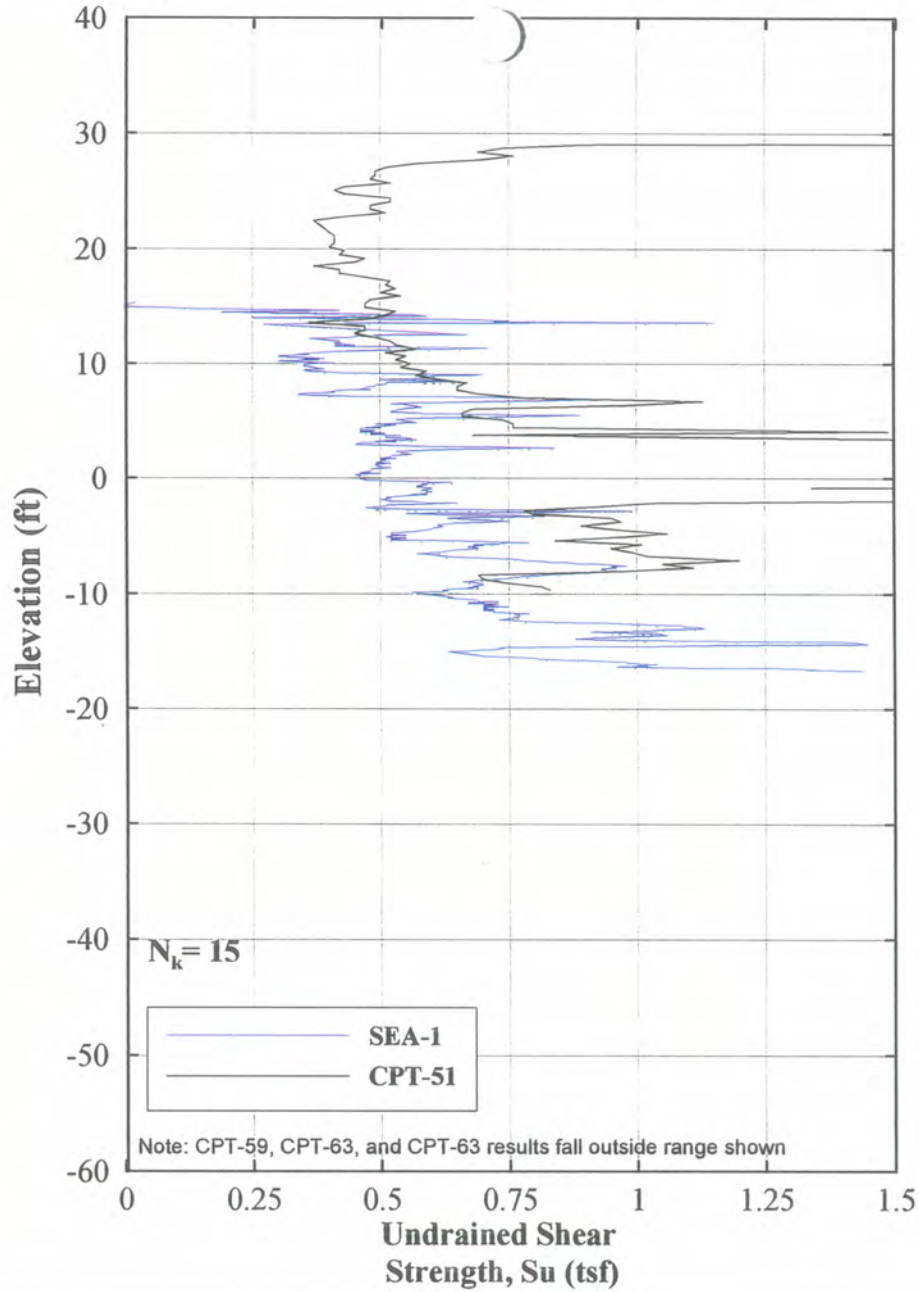
Profiles of Undrained Shear Strength with CPT Sta. 115+00-150+00

FIGURE 6-24

PROJECT: ME0250 Cherry Island



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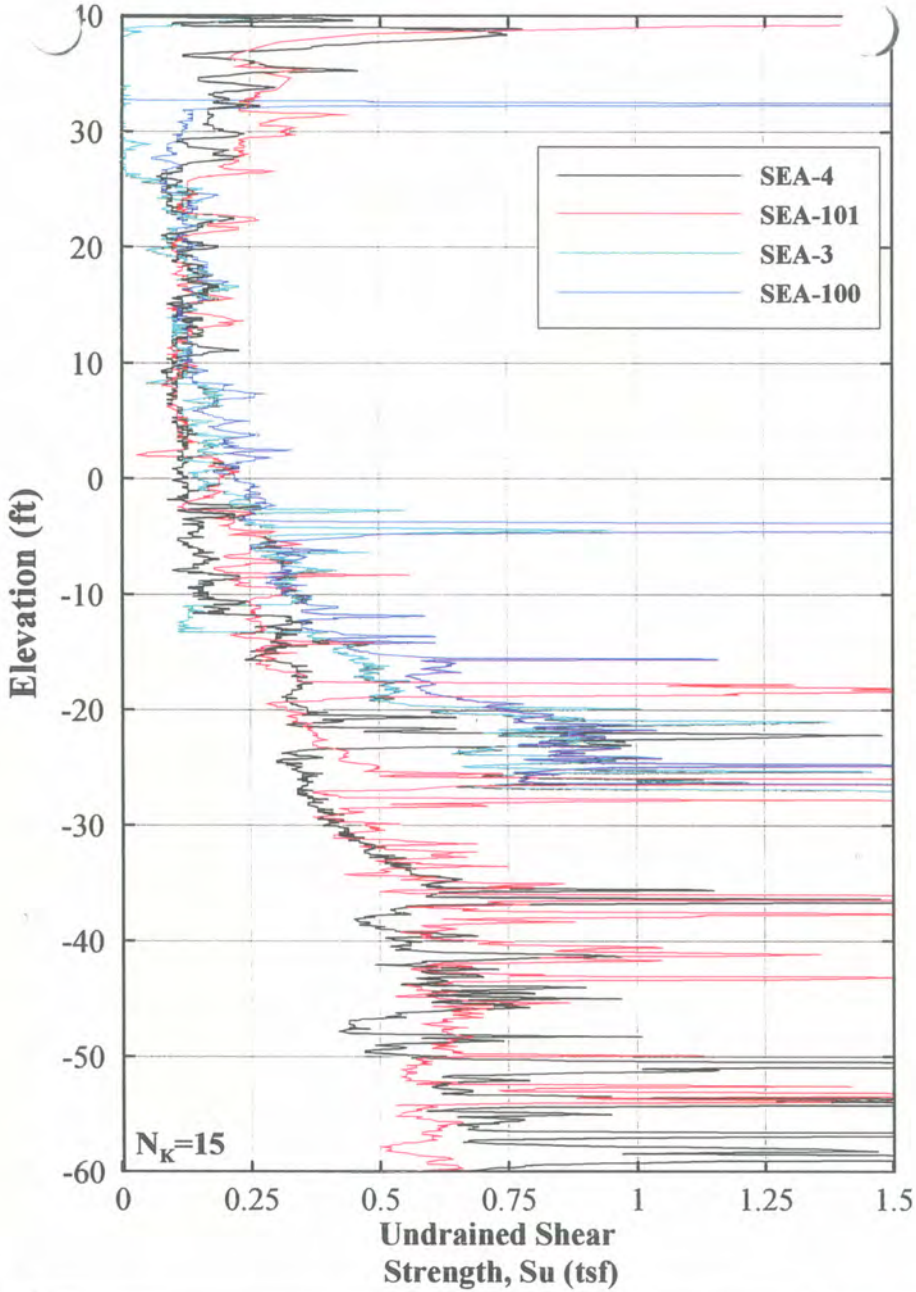
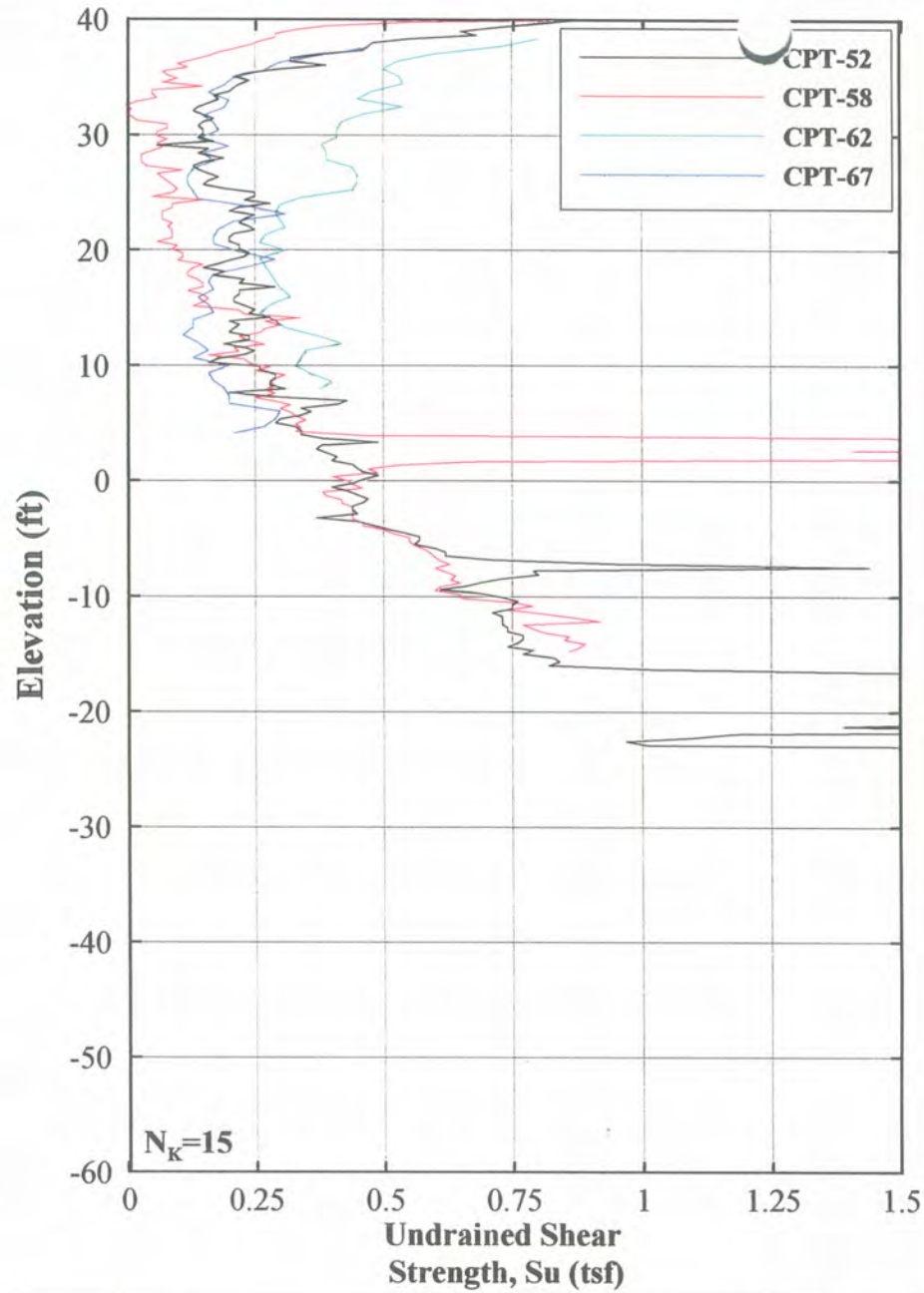
Profiles of Undrained Shear Strength with CPT Phases I and II

FIGURE 6-25

PROJECT: ME0250 Cherry Island

CPT Phases 1 and 2.grf 4/2/03





CPT Phase 3 Plot(2).grf

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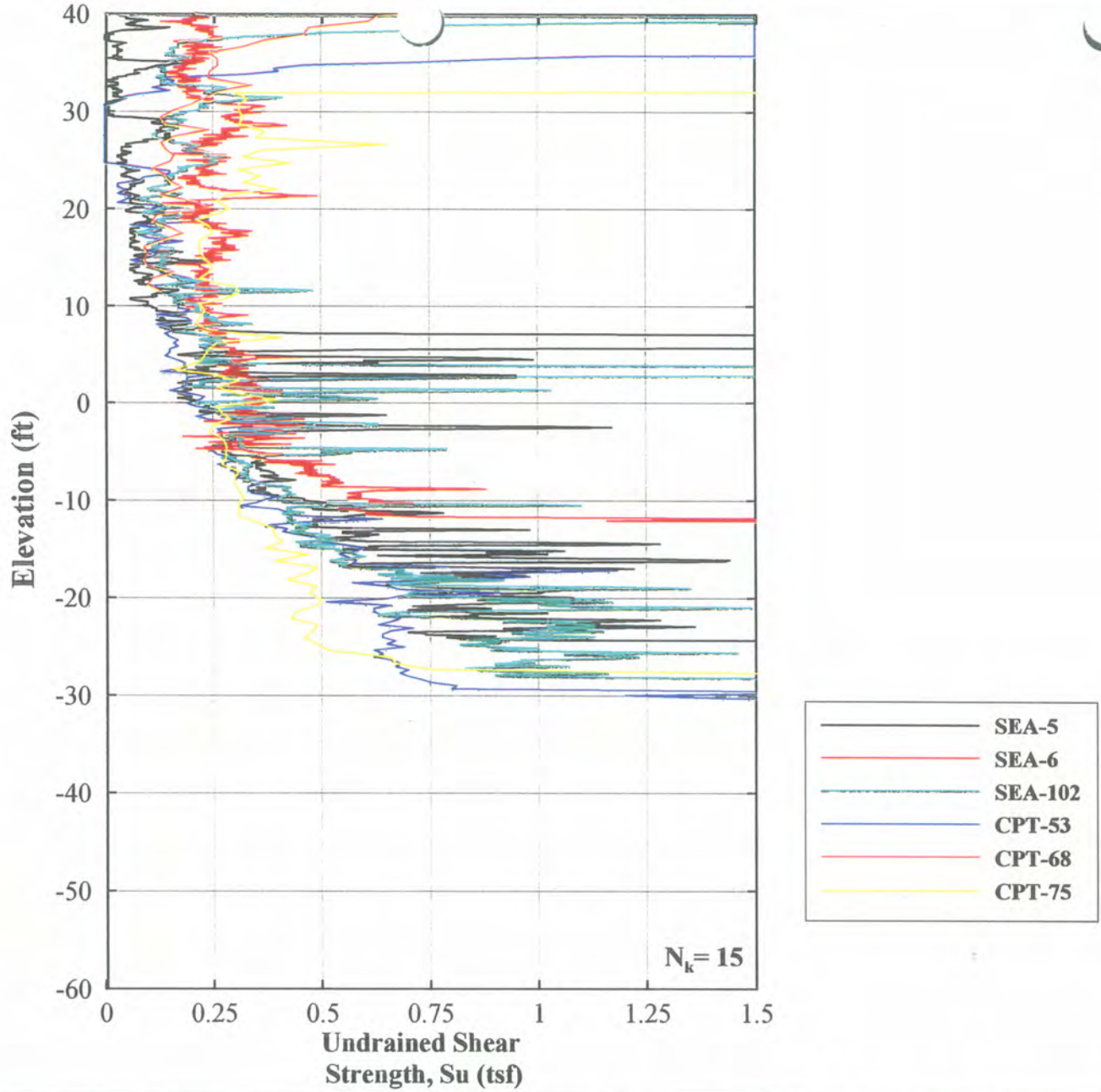
Profiles of Undrained Shear Strength with CPT Phase III

FIGURE 6-26

PROJECT: ME0250 Cherry Island



CPT Phase 4(2).grf

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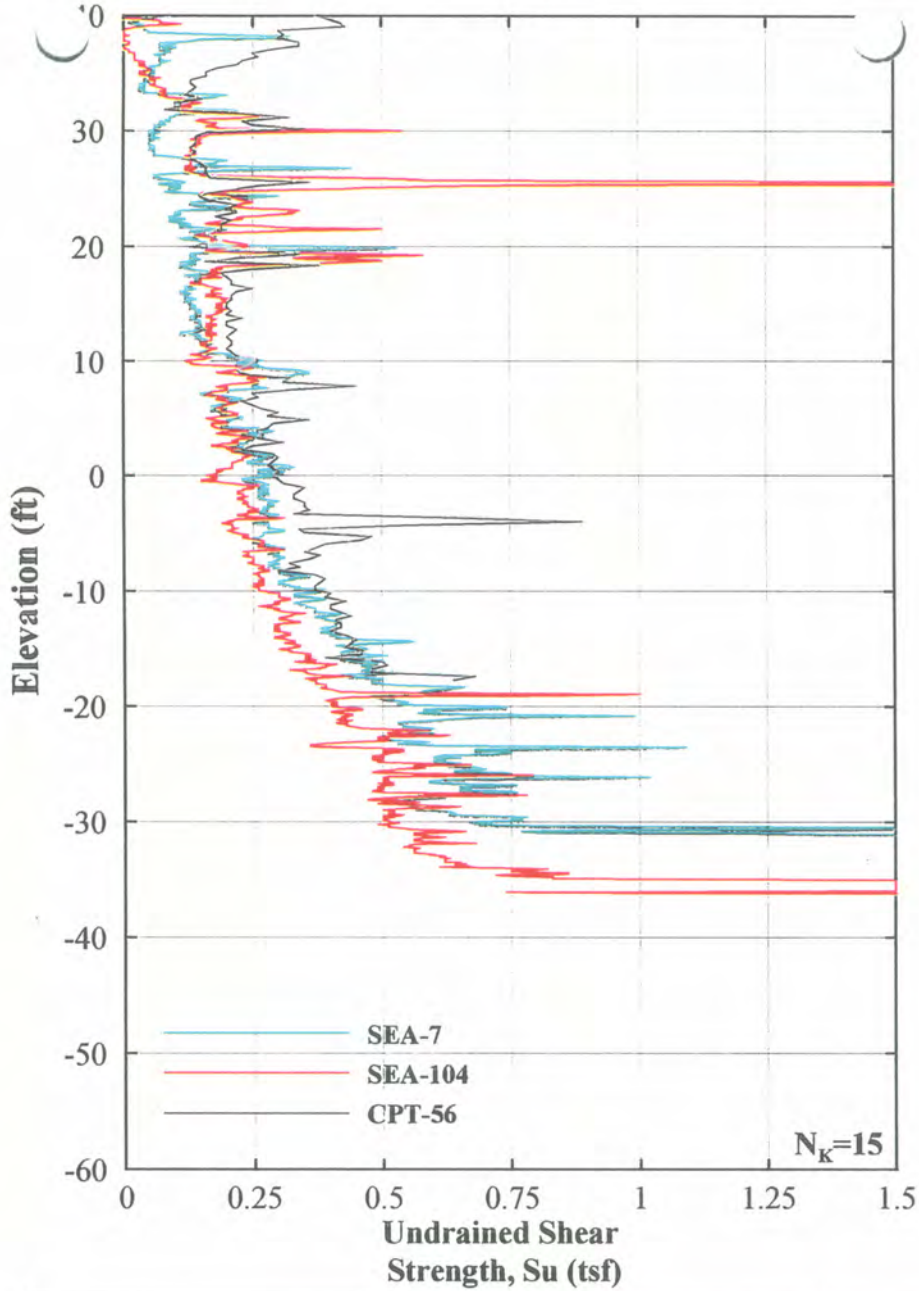
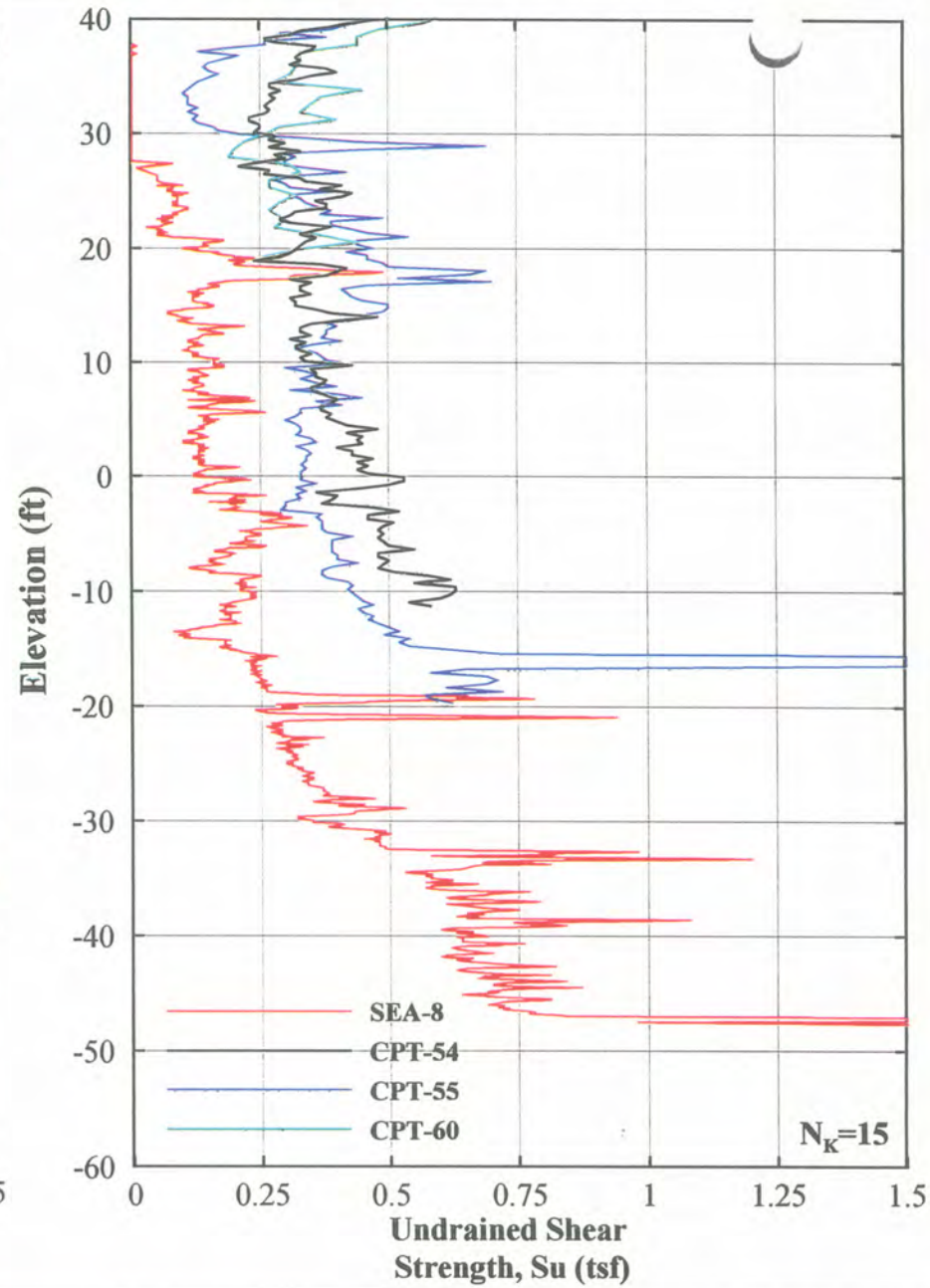
Profiles of Undrained Shear Strength with CPT Phase IV

FIGURE 6-27

PROJECT: ME0250 Cherry Island





CPT Phase 5 Plots(2).grf

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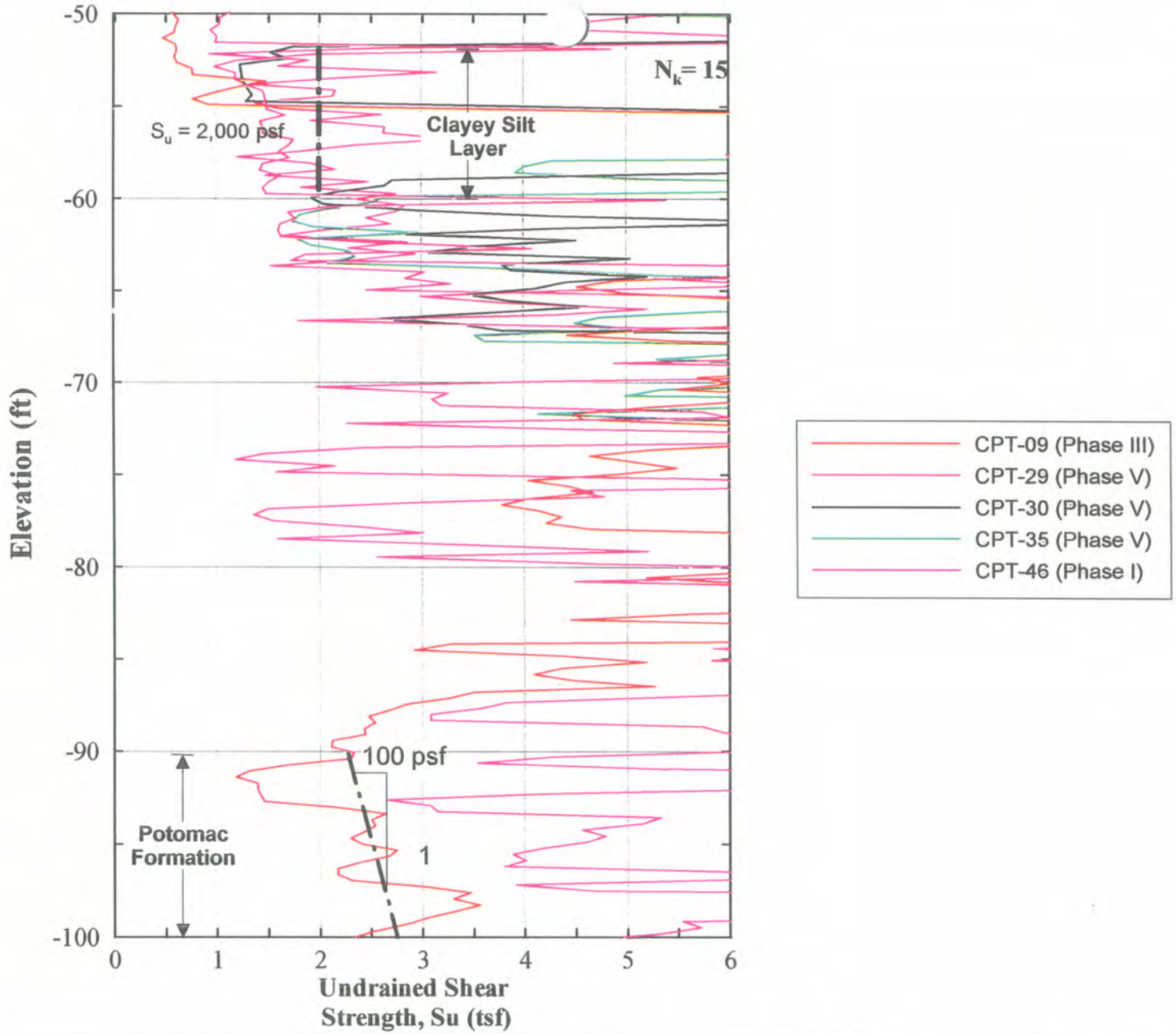
Profiles of Undrained Shear Strength with CPT Phase V

FIGURE 6-28

PROJECT: ME0250 Cherry Island



CPT Su - Potomac.grf 4/2/03

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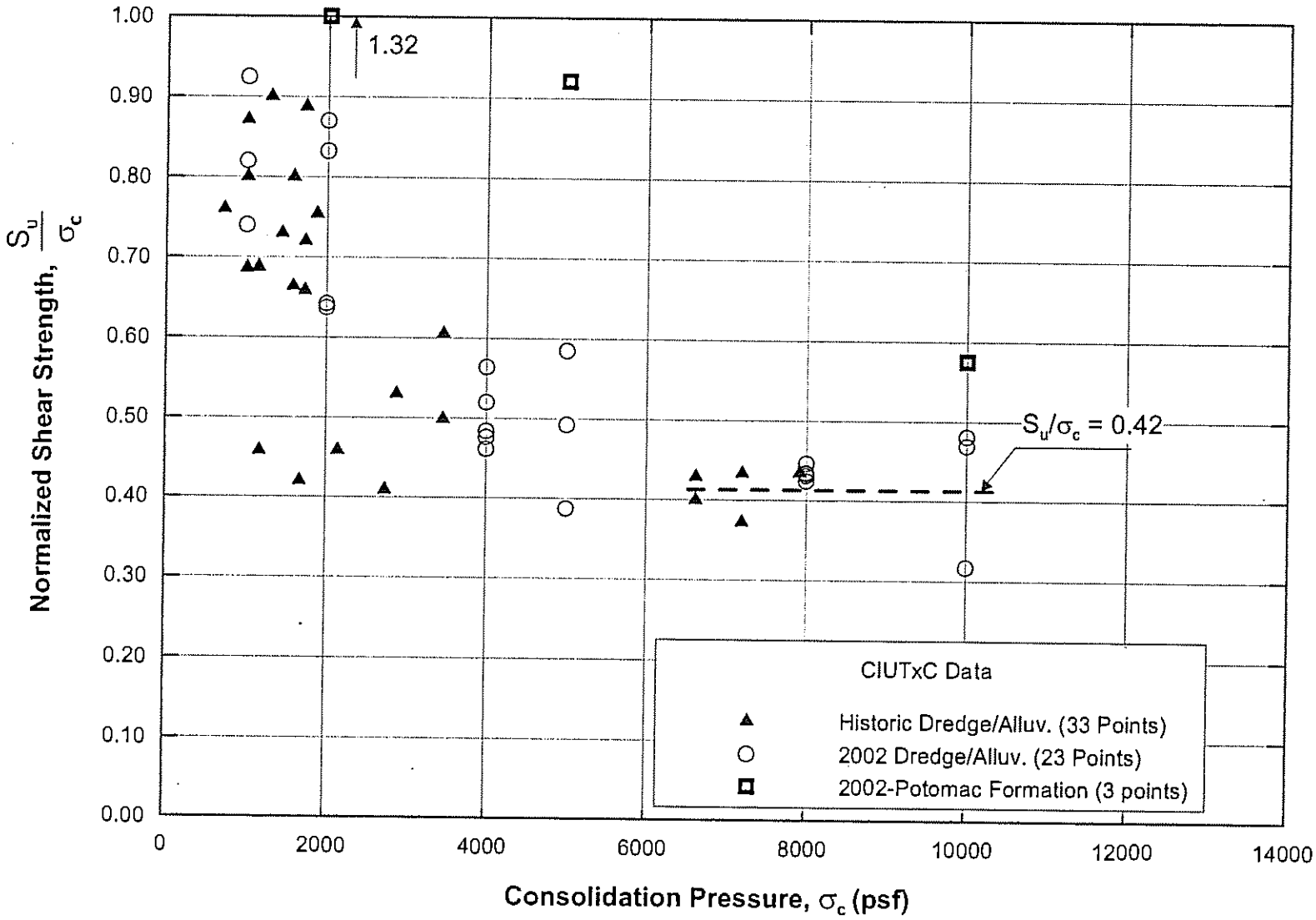
Undrained Shear Strength with CPT Clayey Silt Layer and Potomac Formation

FIGURE 6-29

PROJECT: ME0250 Cherry Island



2002 Triaxial - Normalized Su Strength.grf 4/3/03

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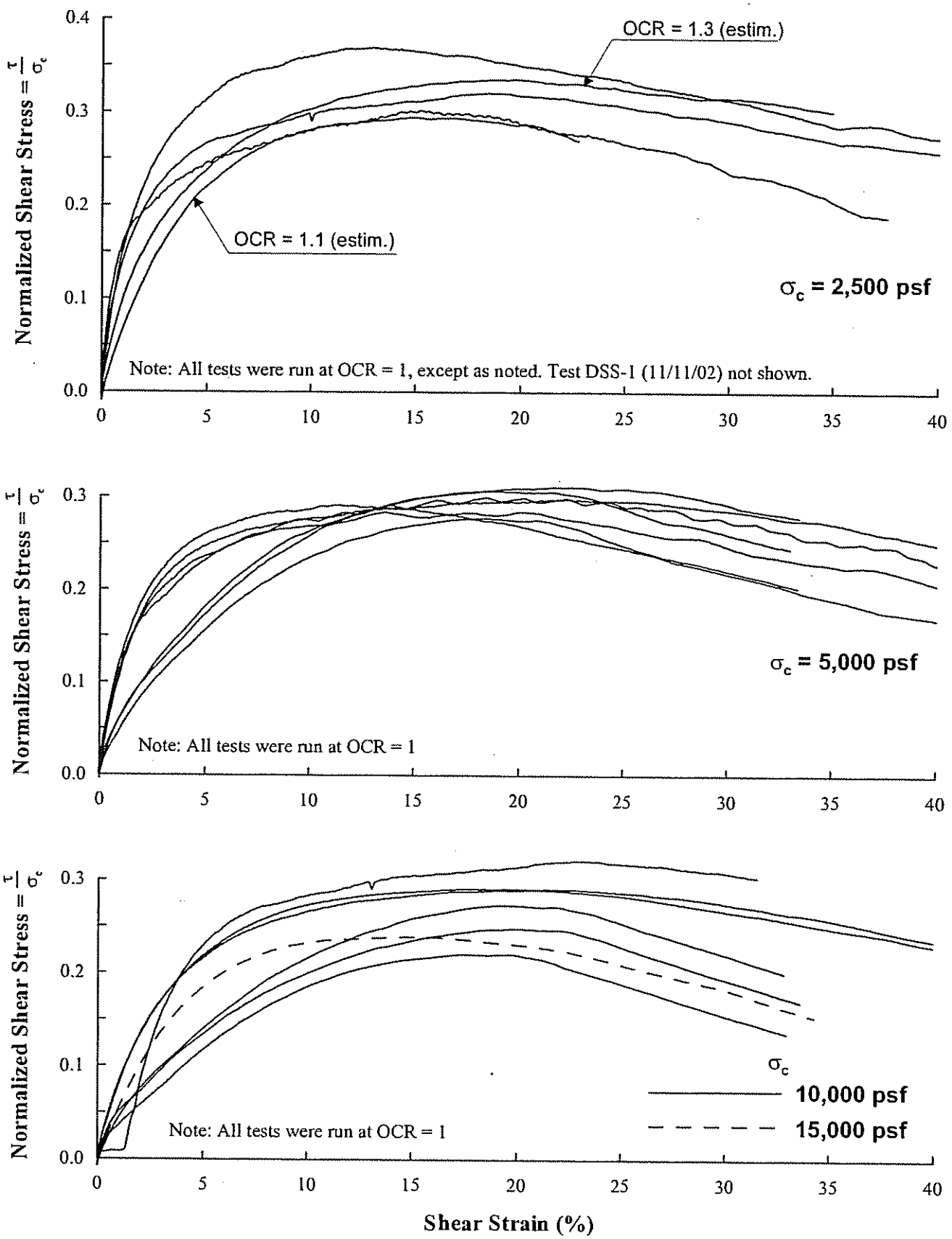
Undrained Shear Strength from CIU Triaxial Tests Dredge/Alluvium and Potomac Formation

FIGURE 6-30

PROJECT ME0250
Cherry Island



Summary of DSS (all).grf 4/3/03

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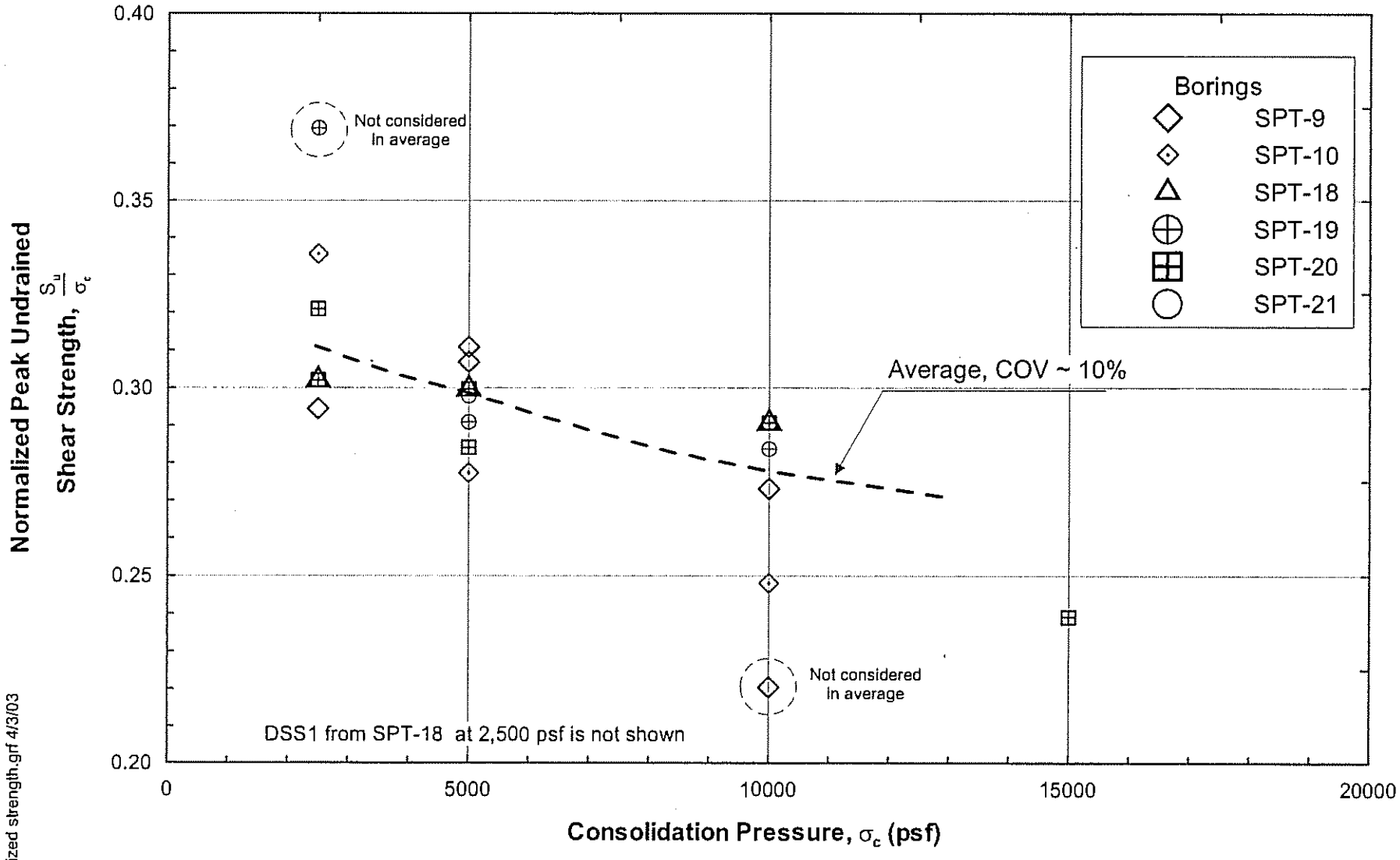
Direct Simple Shear Tests
Normalized Shear Stress vs. Shear Strain

FIGURE 6-31

PROJECT ME0250
Cherry Island



Summary of normalized strength.grf 4/3/03

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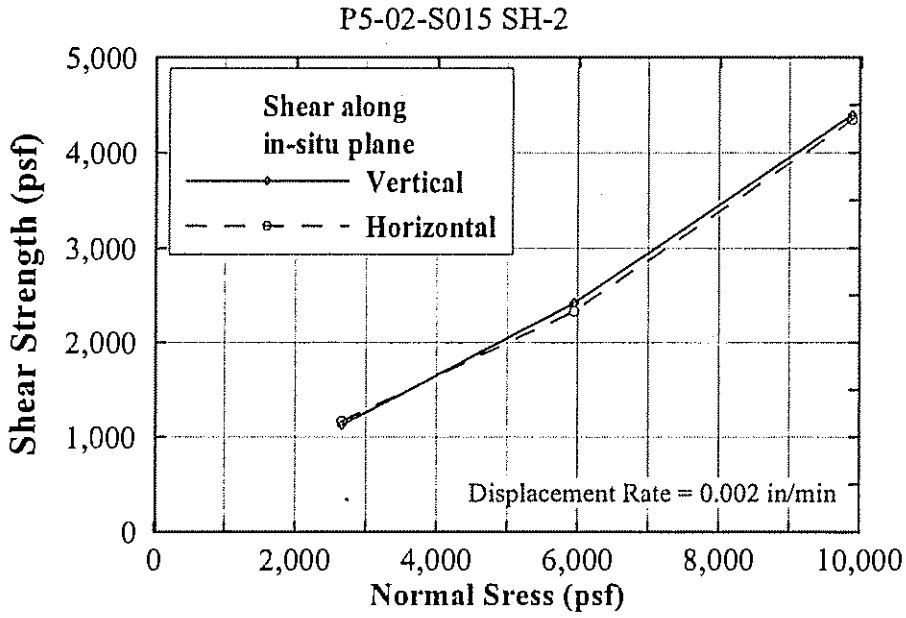
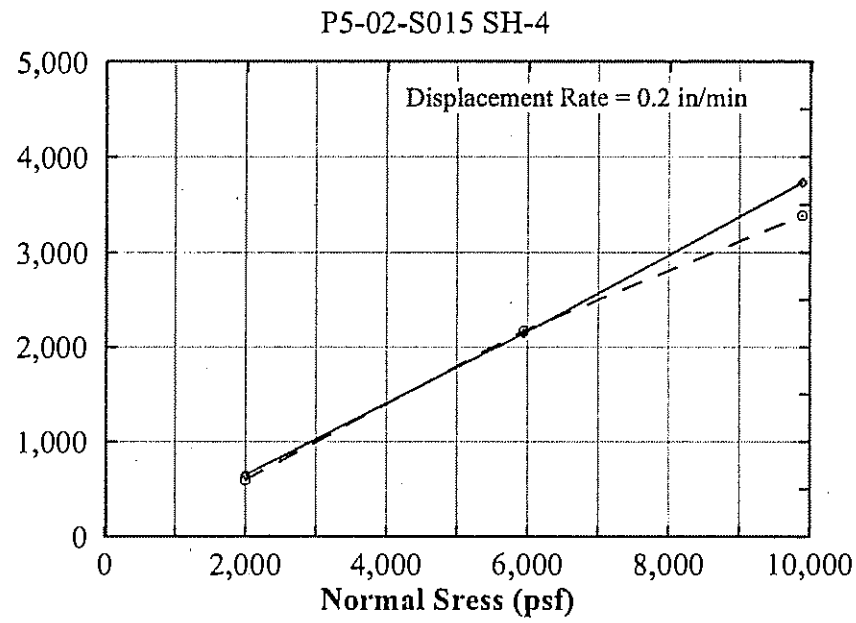
Columbia, Maryland 21046

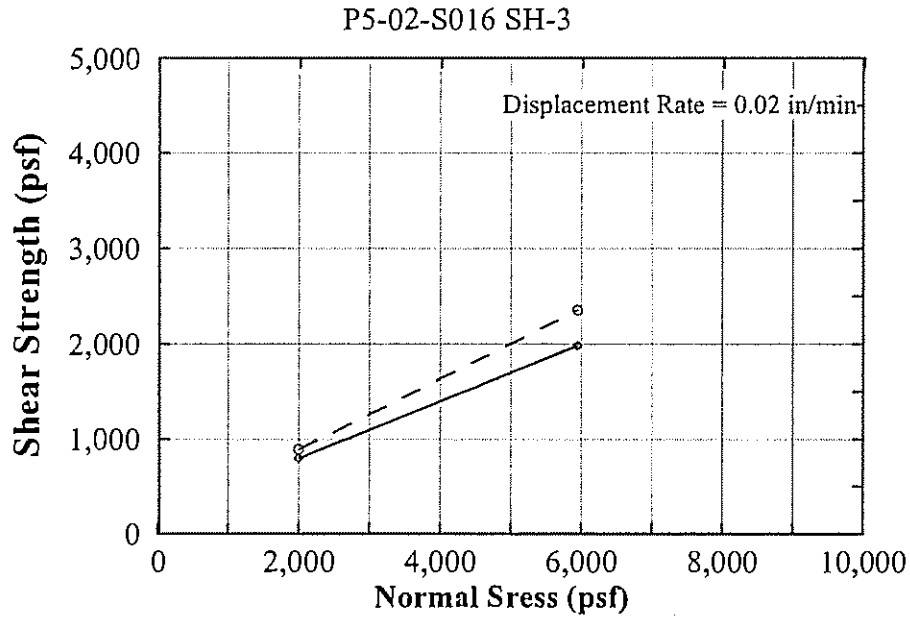
Normalized Peak Undrained Shear Strength from Direct Simple Shear Tests

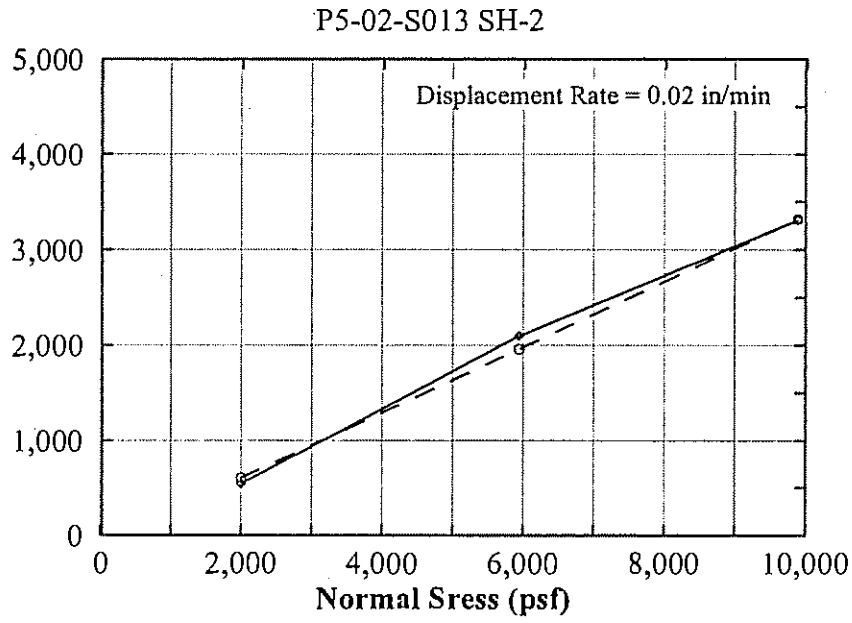
FIGURE 6-32

PROJECT ME0250
Cherry Island









Direct Shear.grf

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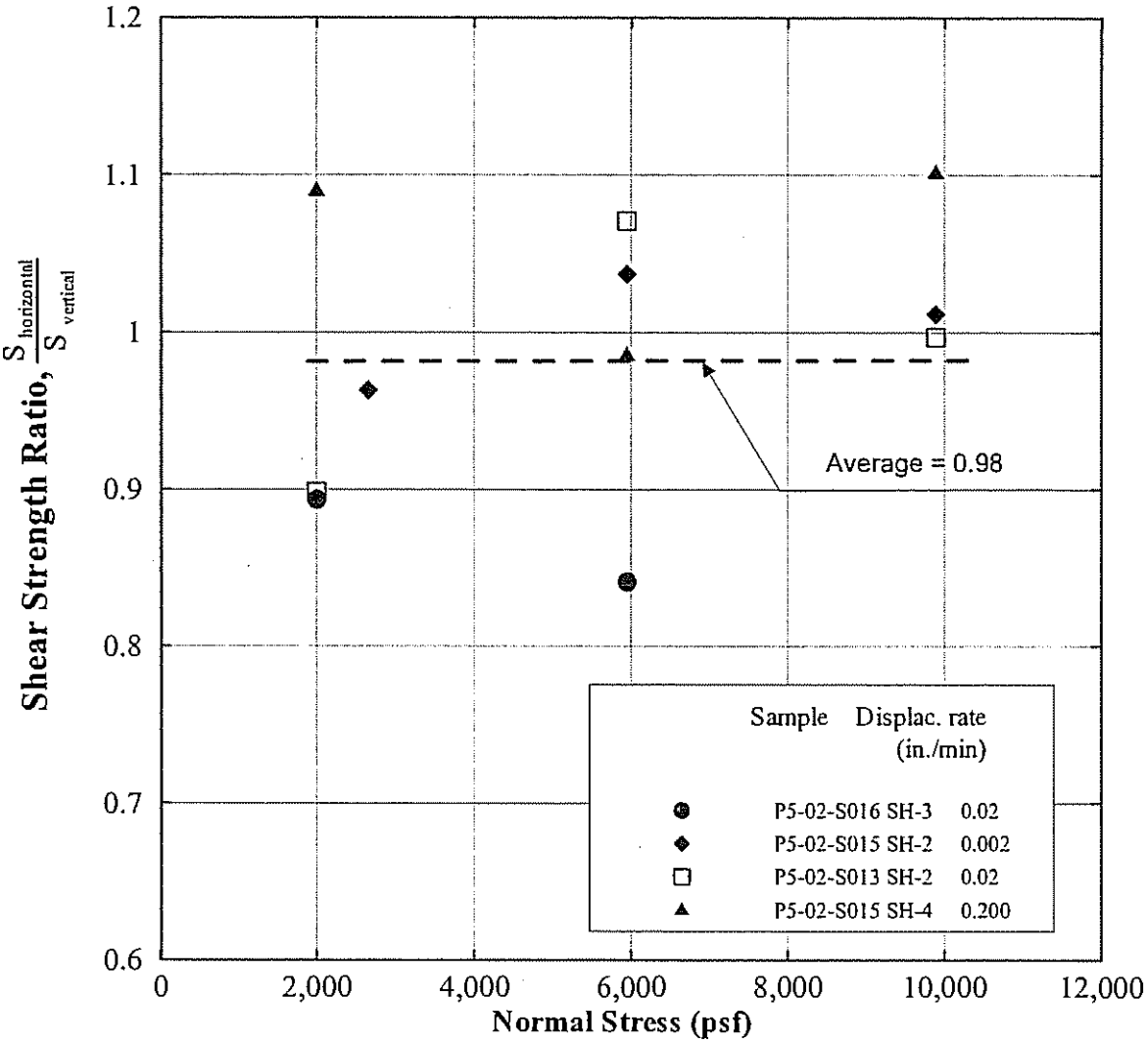
Direct Shear Tests
Shear Strength vs. Normal Stress

FIGURE 6-33

PROJECT: ME0250 Cherry Island



$S_{horizontal}$ = shear strength along an in situ horizontal plane
$S_{vertical}$ = shear strength along an in situ vertical plane

Direct Shear2.grf

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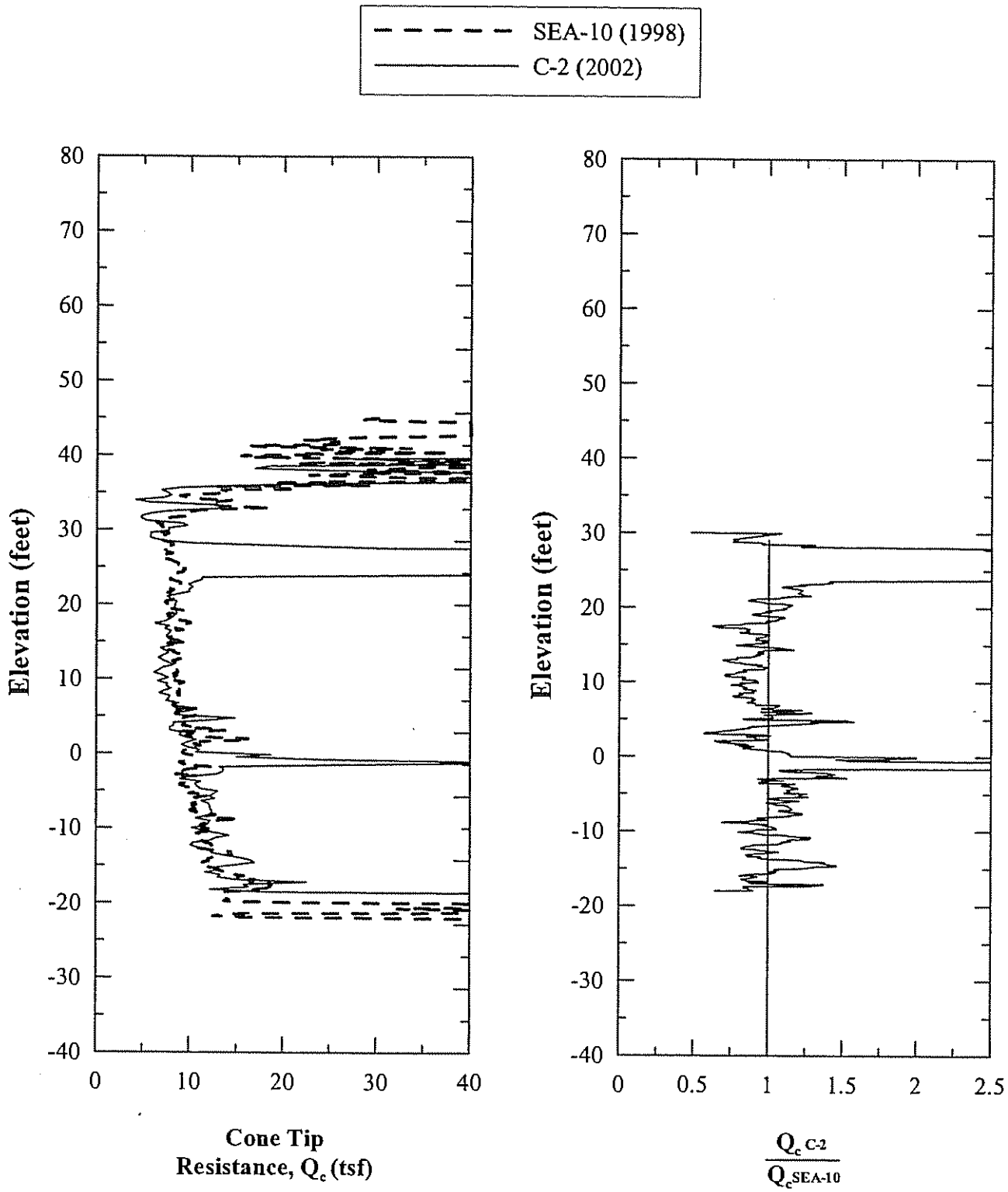
Shear Stress Ratio vs. Normal Stress in Direct Shear Tests

FIGURE 6-34

PROJECT: ME0250
Cherry Island



cpt time effect 9.grf

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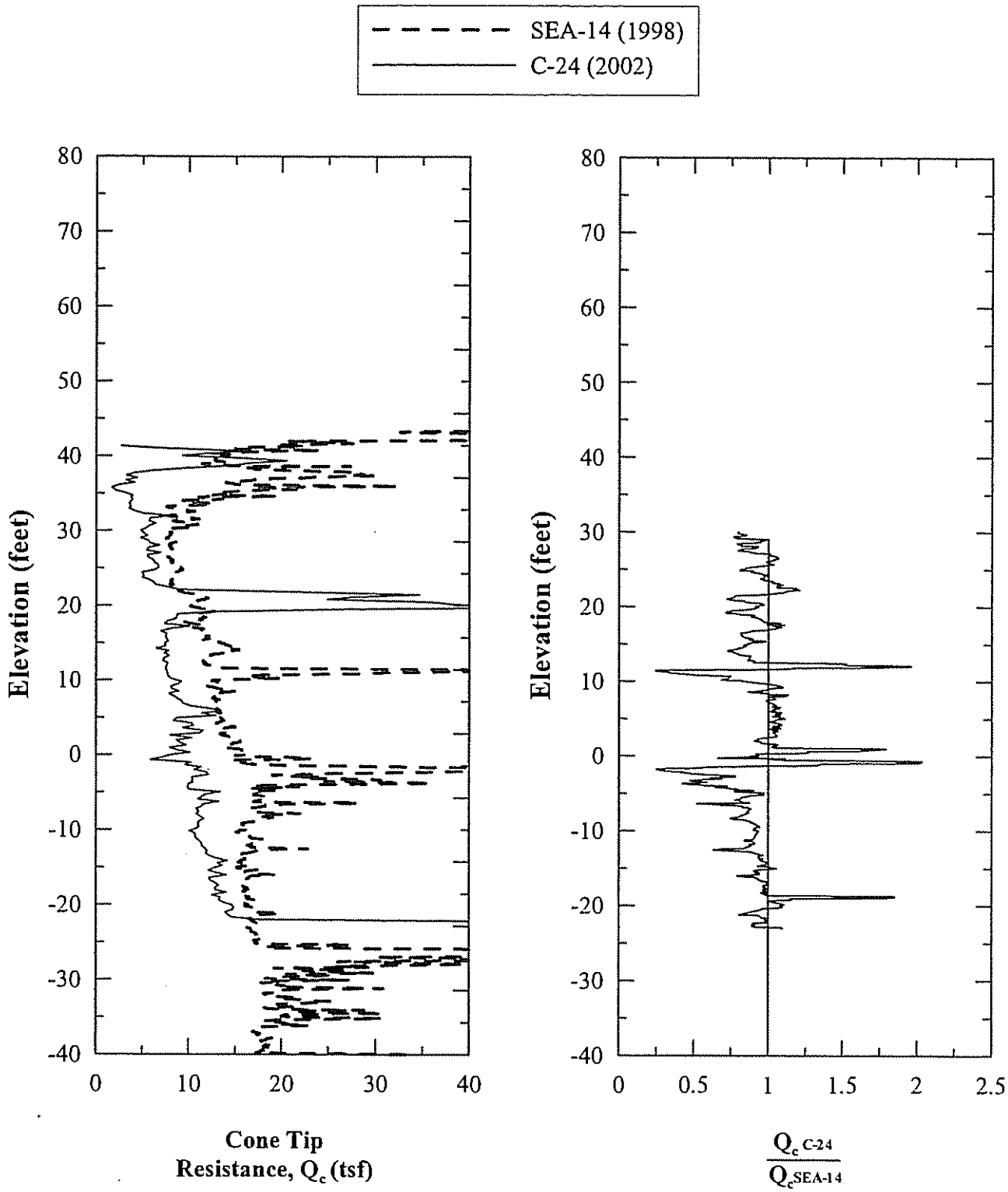
CPT Strength Gain with Time - Along Berm

FIGURE 6-35

PROJECT: ME0250
Cherry Island



cpt time effect 8.grf

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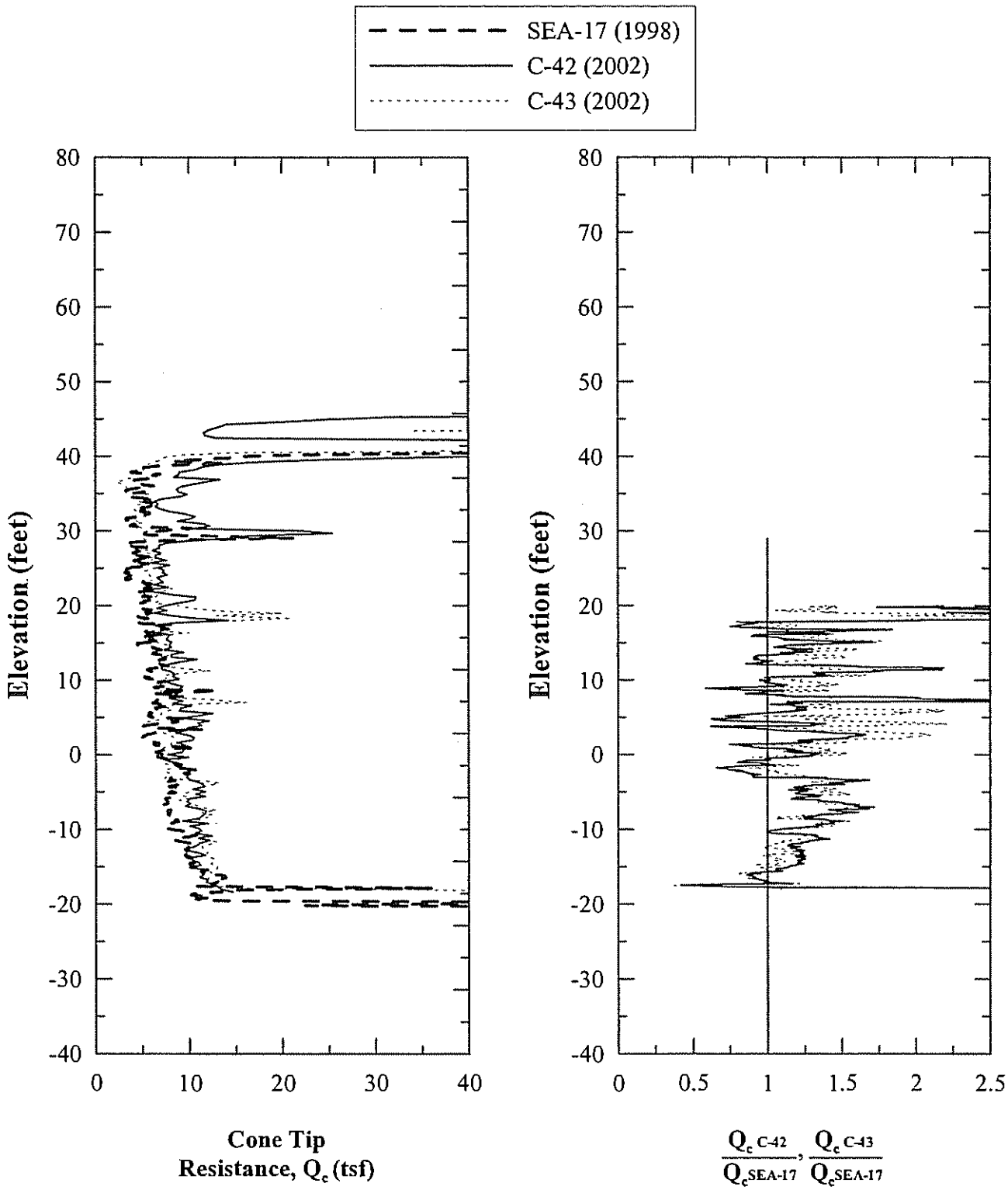
CPT Strength Gain with Time - Along Berm

FIGURE 6-36

PROJECT: ME0250
Cherry Island



cpt time effect 10.grf

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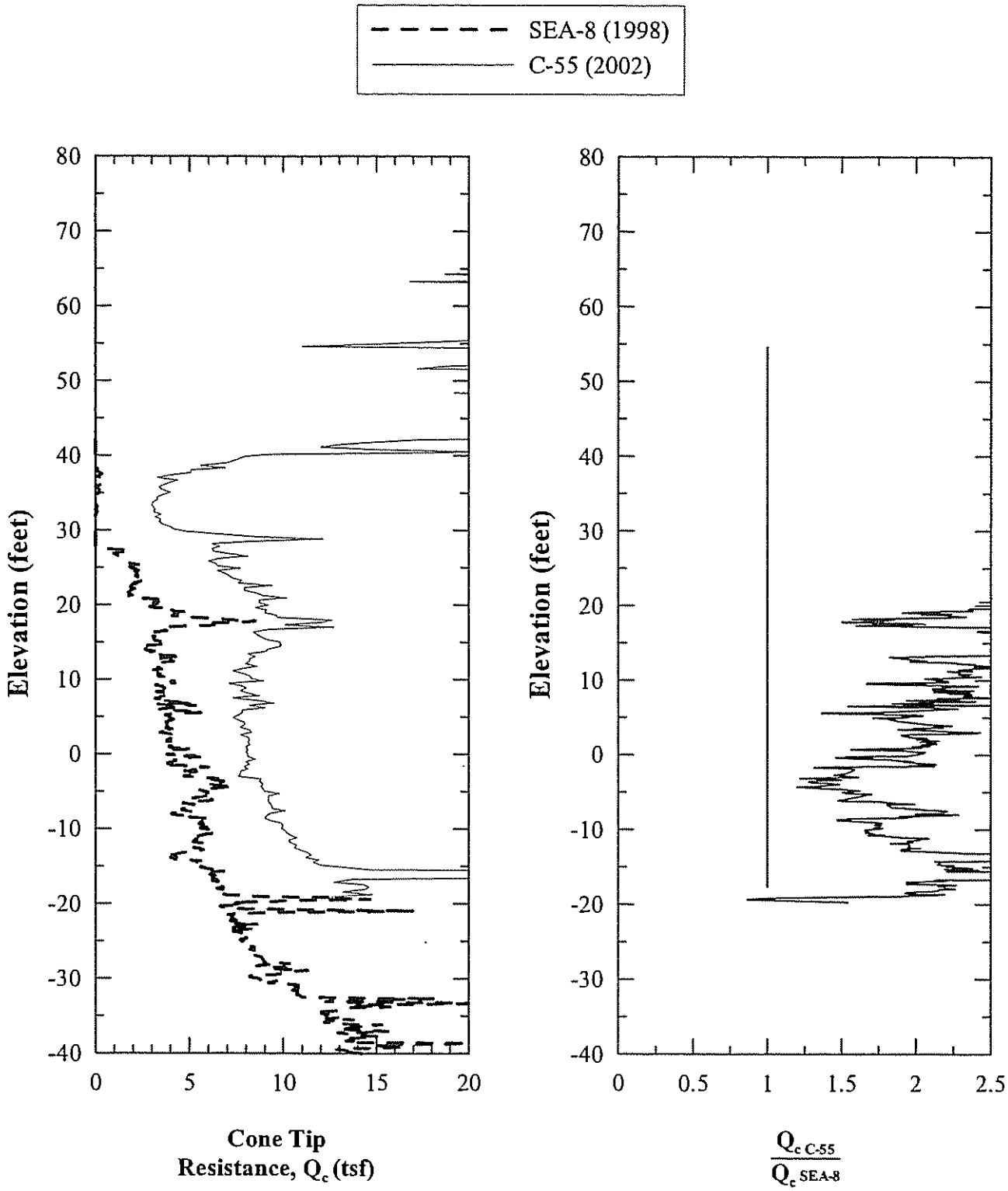
CPT Strength Gain with Time - Along Berm

FIGURE 6-37

PROJECT: ME0250
Cherry Island



cpt time effect 2.grf

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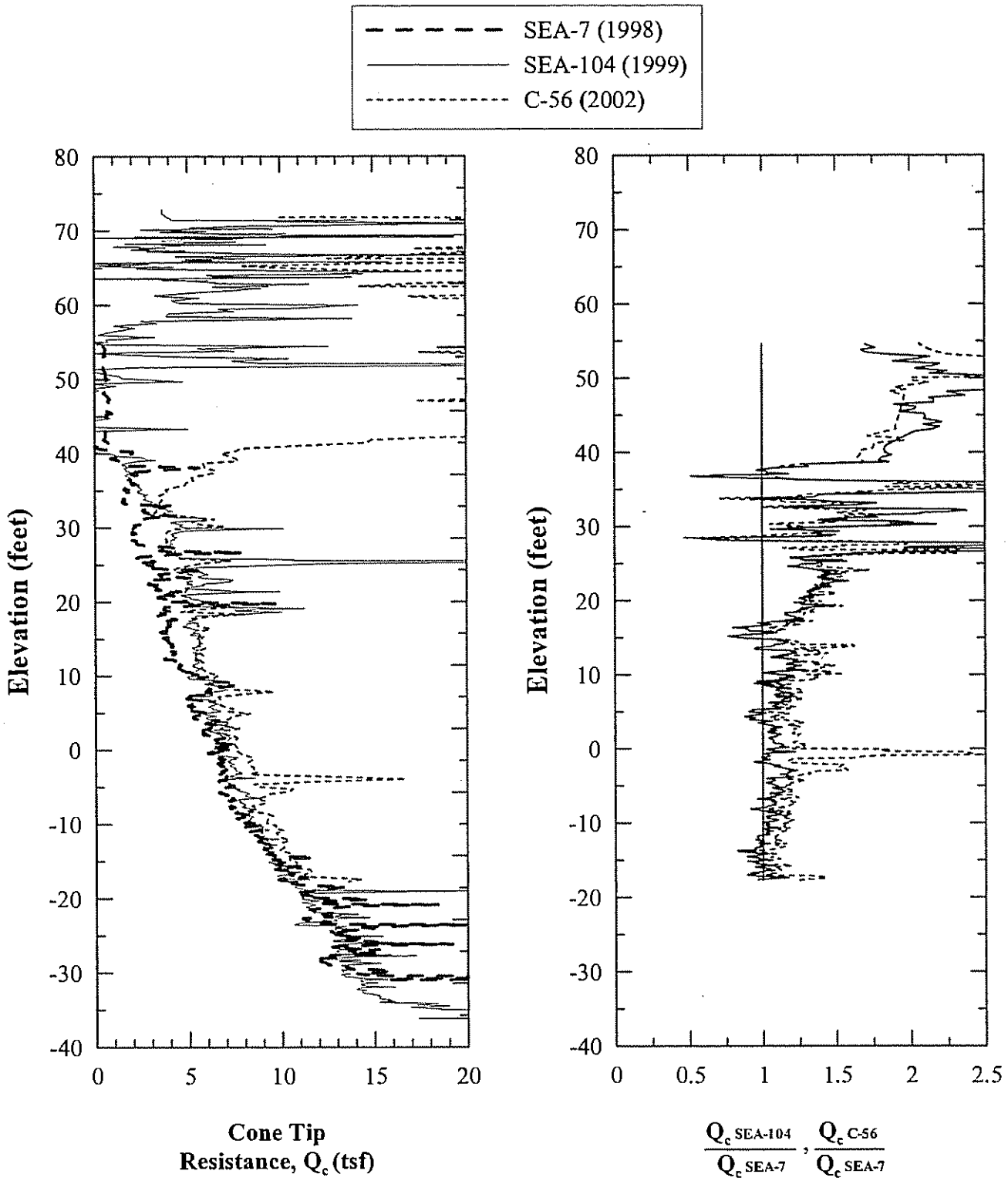
CPT Strength Gain with Time - Under Waste

FIGURE 6-38

PROJECT: ME0250
Cherry Island



cpt time effect 1.grf

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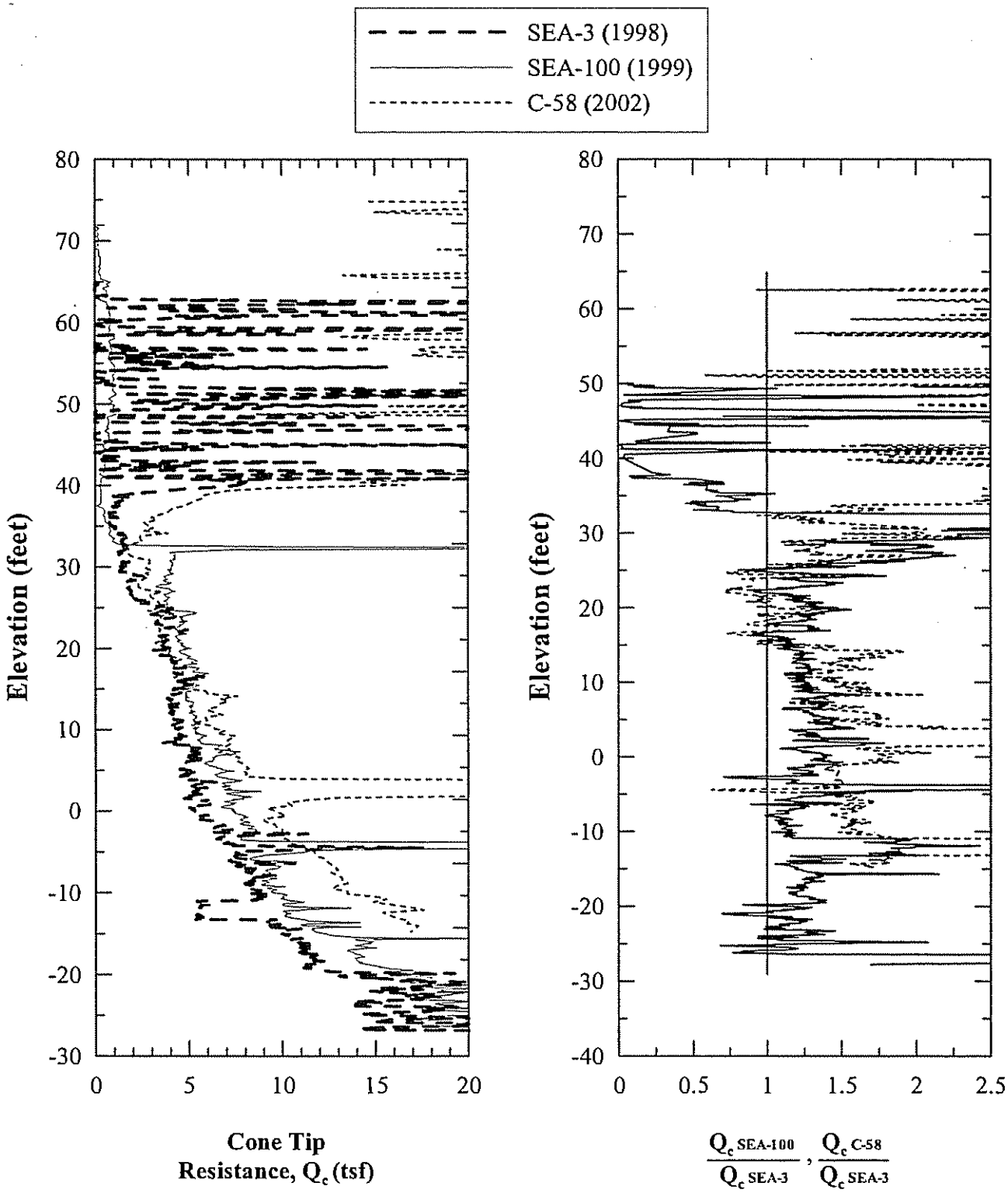
CPT Strength Gain with Time - Under Waste

FIGURE 6-39

PROJECT: ME0250
Cherry Island



cpt time effect 3.grf 4/3/03

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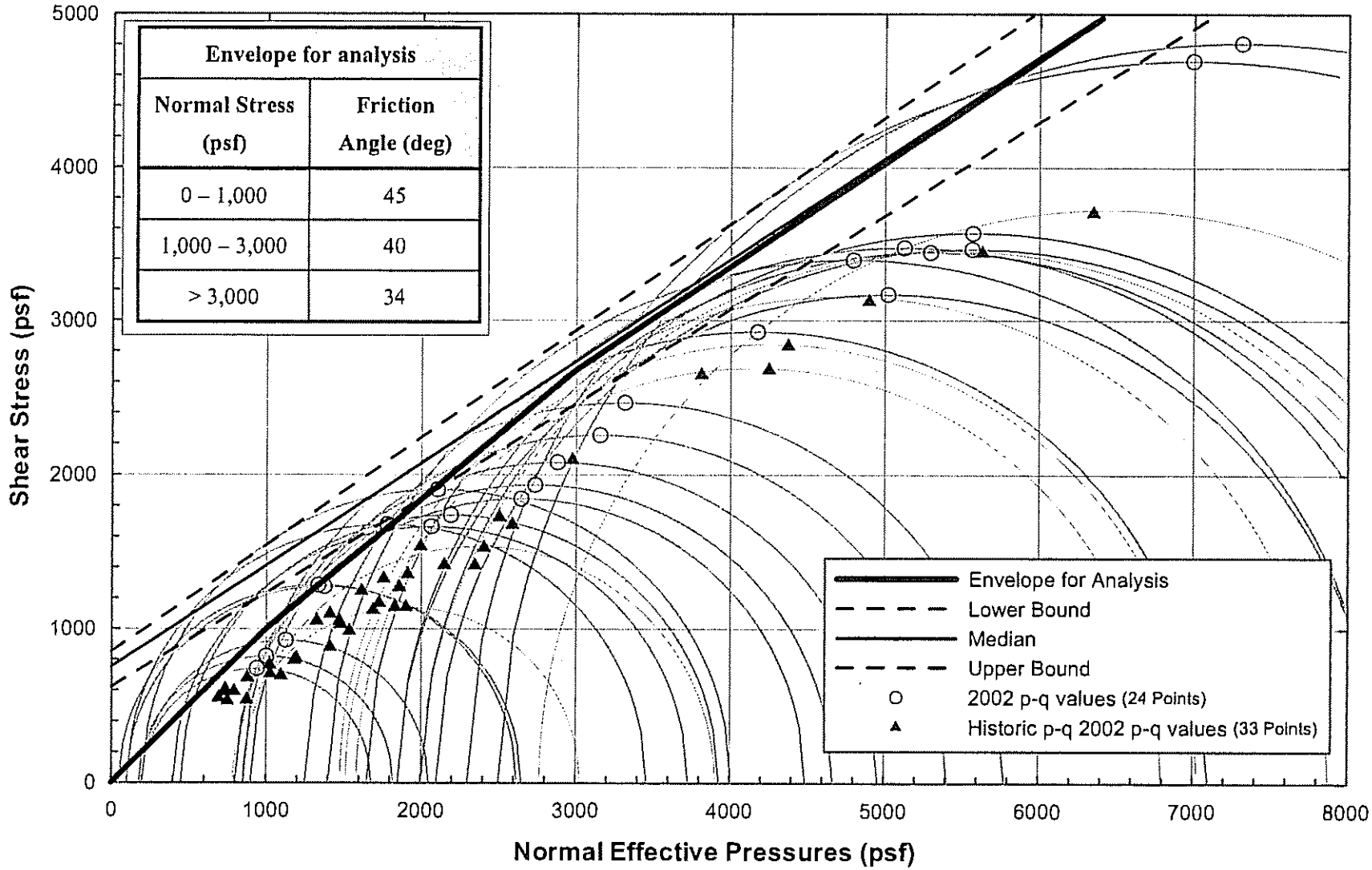
CPT Strength Gain with Time - Under Waste

FIGURE 6-40

PROJECT: ME0250
Cherry Island



Note: some circles are not drawn for clarity

2002 Triaxial - Effective Strength(2).grf 4/3/03

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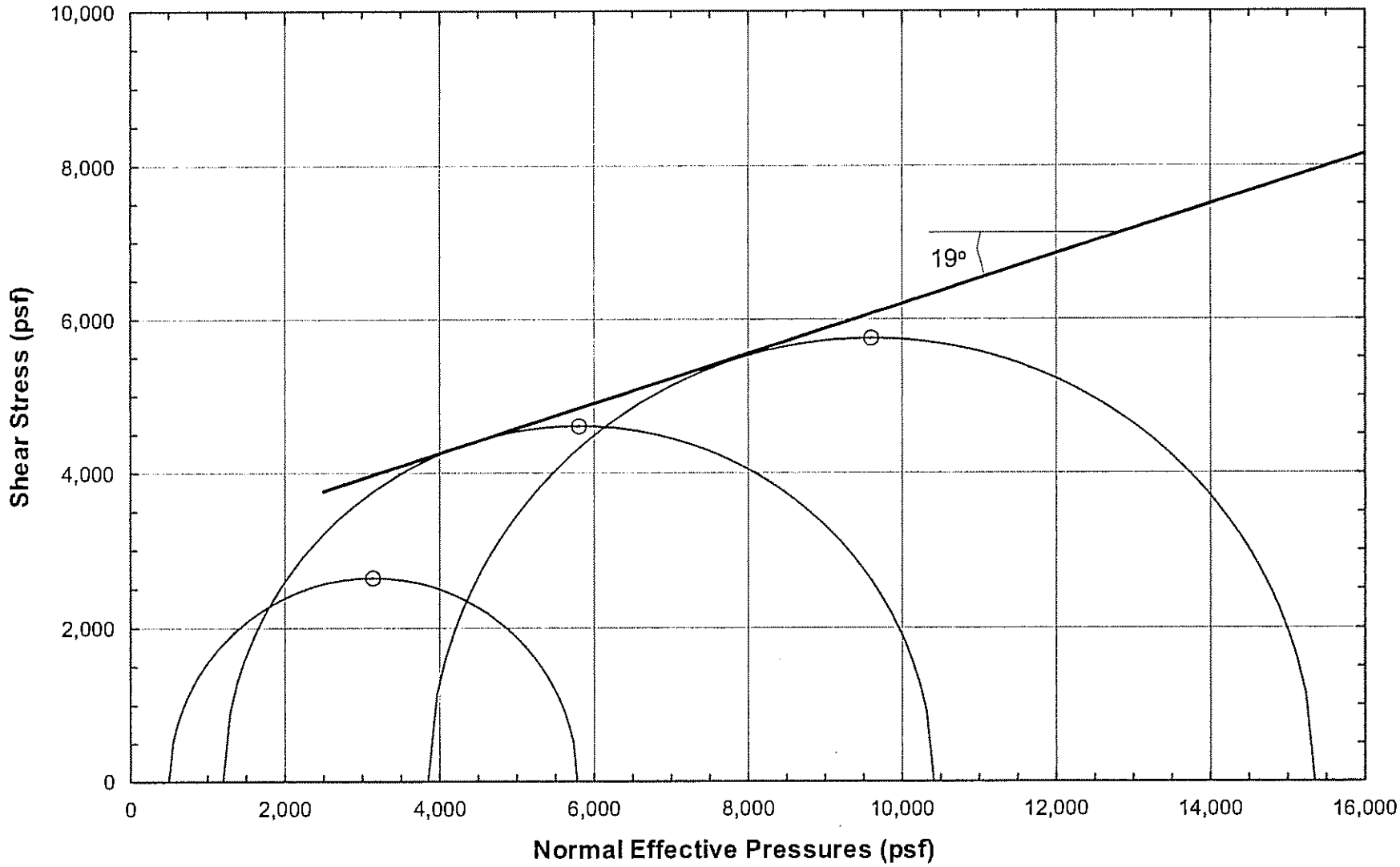
Effective Shear Strength from CIU Triaxial Compression Tests Dredge and Alluvium

FIGURE 6-41

PROJECT ME0250
Cherry Island



2002 Triaxial - Effective Strength - Potomac.grf 4/3/03

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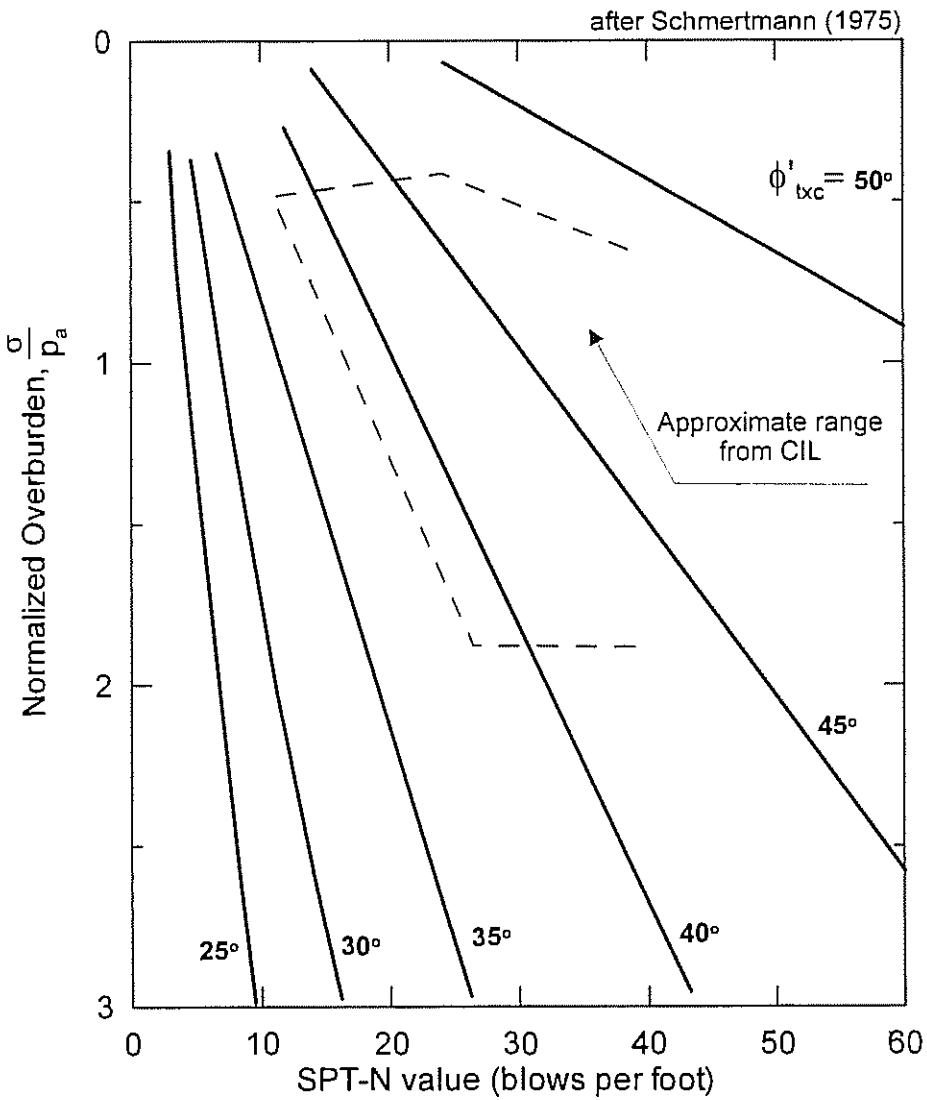
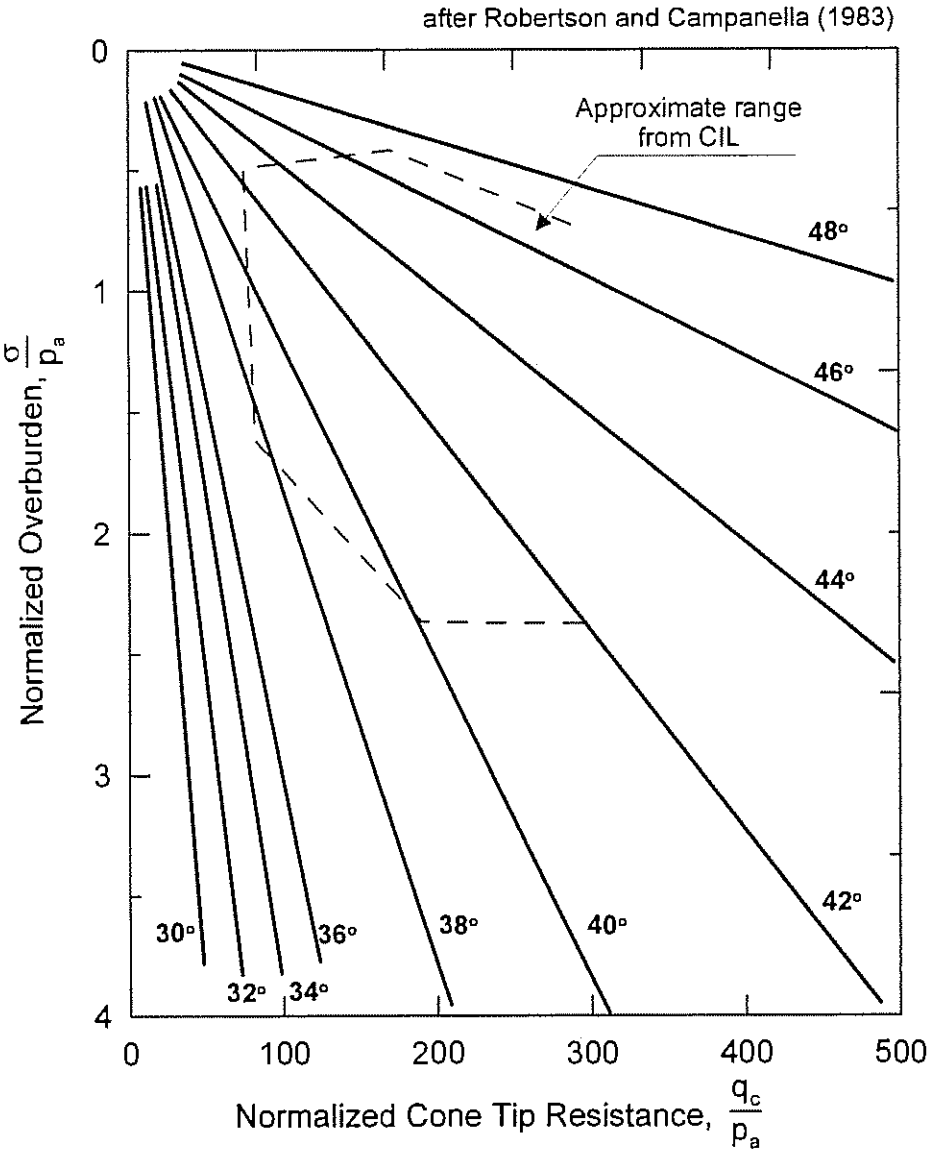
Effective Shear Strength from CIU Triaxial Compression Tests Potomac Formation

FIGURE 6-42

PROJECT ME0250
Cherry Island





ϕ'_{txc} = triaxial compression-based effective friction angle

p_a = atmospheric pressure

Drained parametrs for Sand.grf 4/3/03

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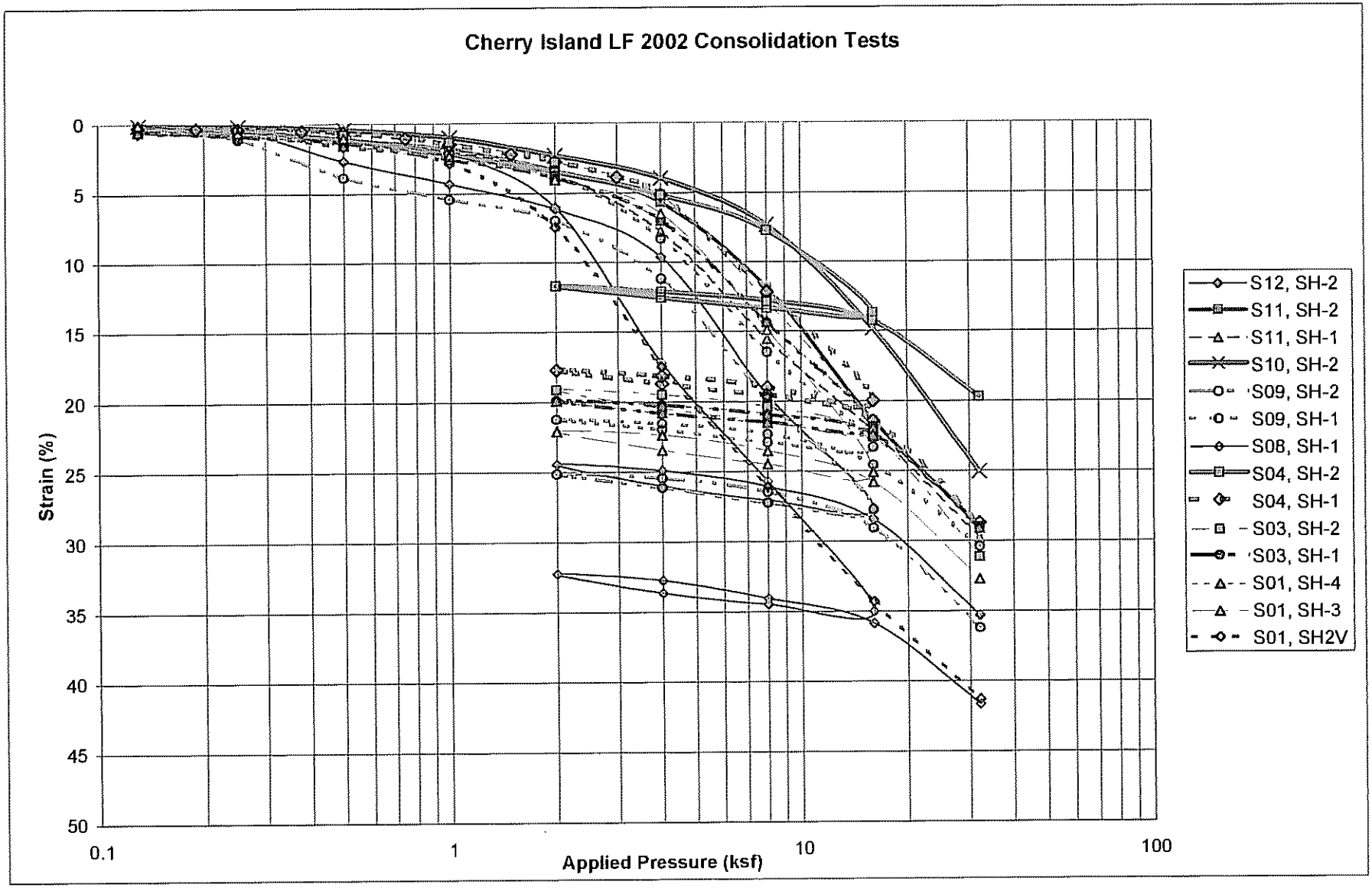
Drained Shear Strength from SPT and CPT Columbia Formation

FIGURE 6-43

PROJECT ME0250
Cherry Island



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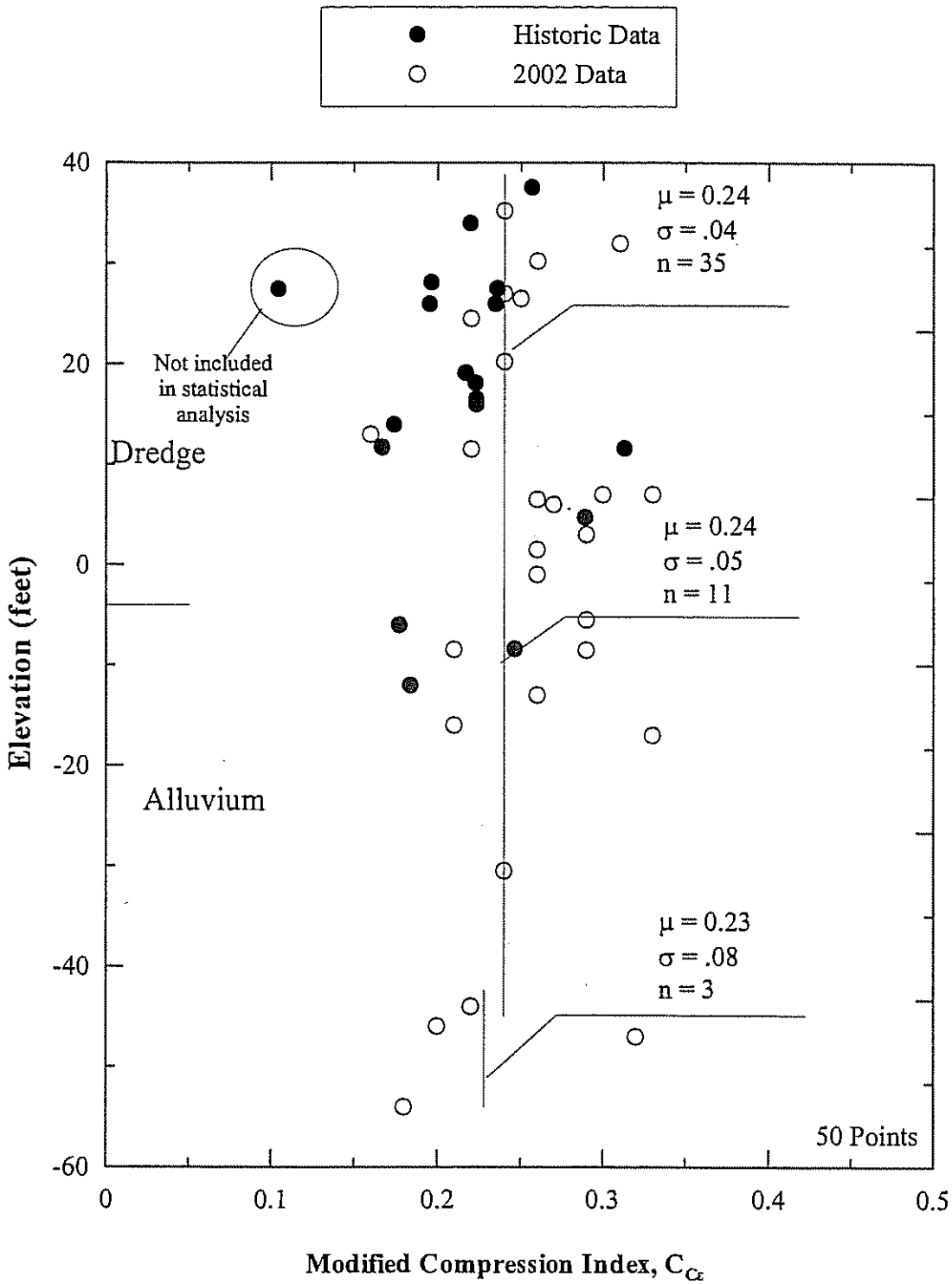
Columbia, Maryland 21046

Summary of 2002 Consolidation Tests

FIGURE 6-44

PROJECT: ME0250
Cherry Island

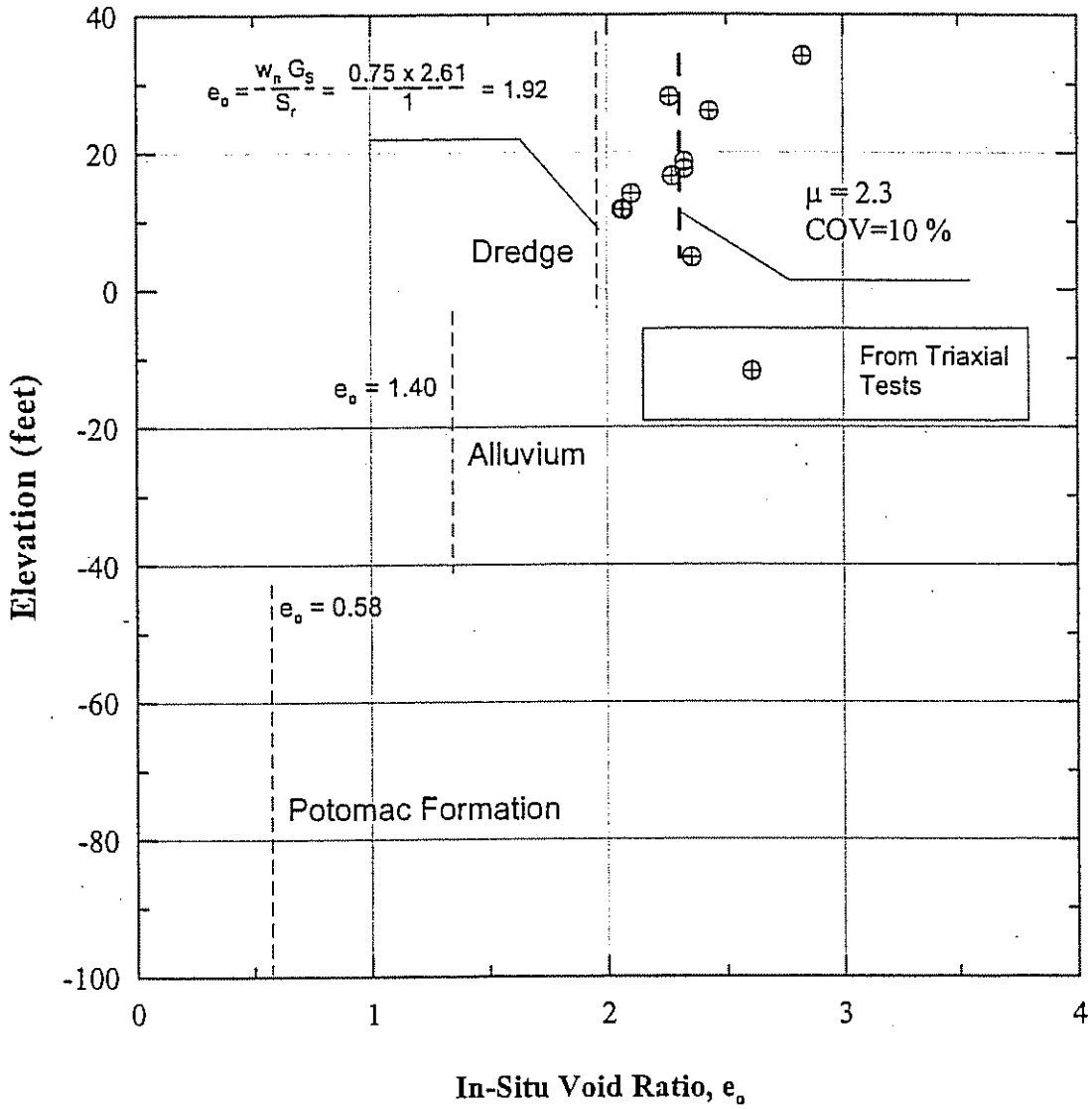
all consolidation tests 4/4/03



Note: μ = average; σ = standard deviation; and
n = number of data points used in statistical anaylsis.

Cc w depth(2).grf

GEOSYNTEC CONSULTANTS 10015 Old Columbia Road, Suite A-200 Columbia, Maryland 21046	**Profile of Compression Index of Fine-Grained Soils**	**FIGURE 6-45** PROJECT: ME0247 Cherry Island



Note: from historic data

eo w depth.grf

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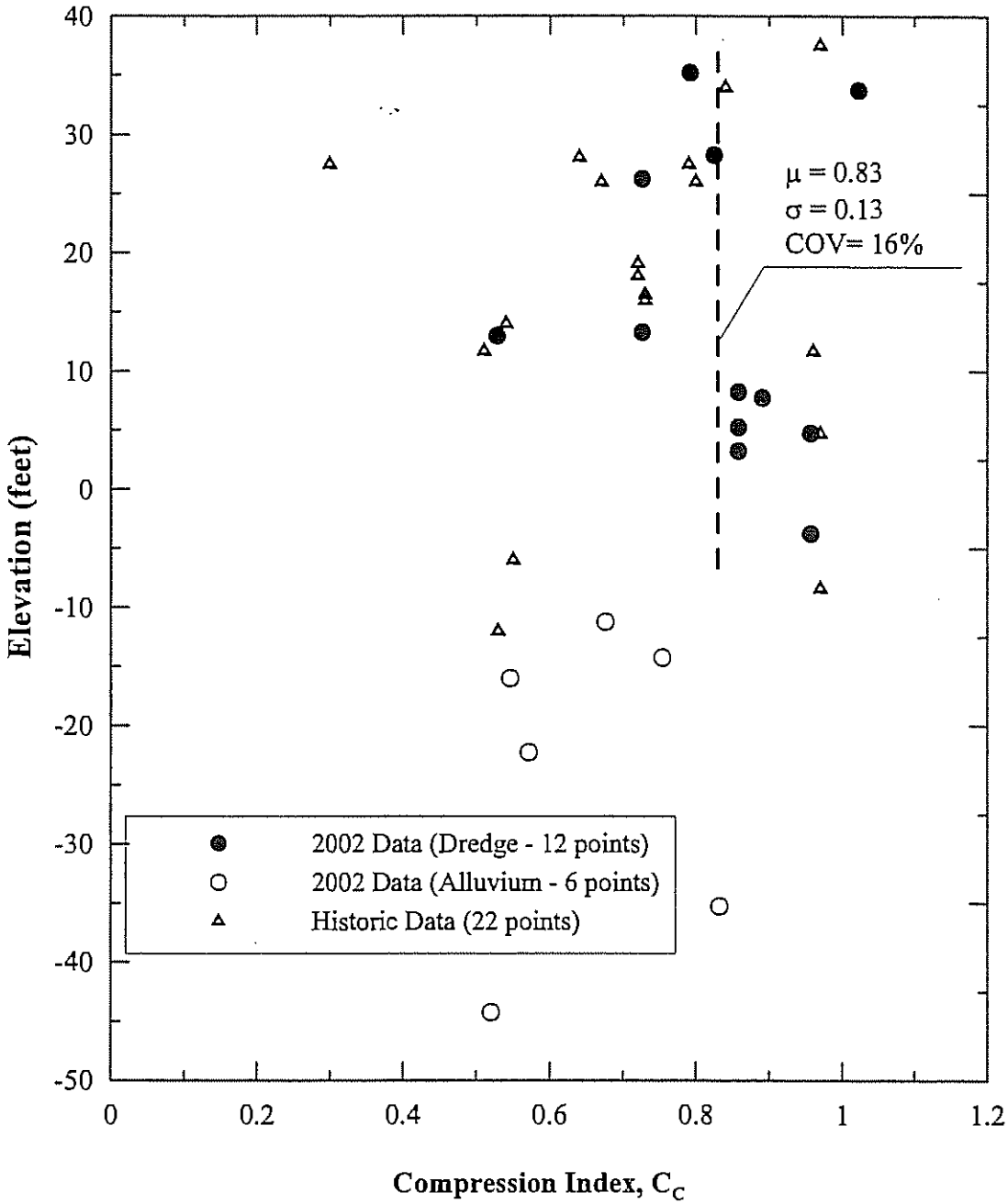
Profile of In-Situ Void Ratio in Dredge

FIGURE 6-46

PROJECT: ME0250
Cherry Island



Cc w depth(2).grf

Note: computed from the modified compression index and $e_0 = 2.3$.

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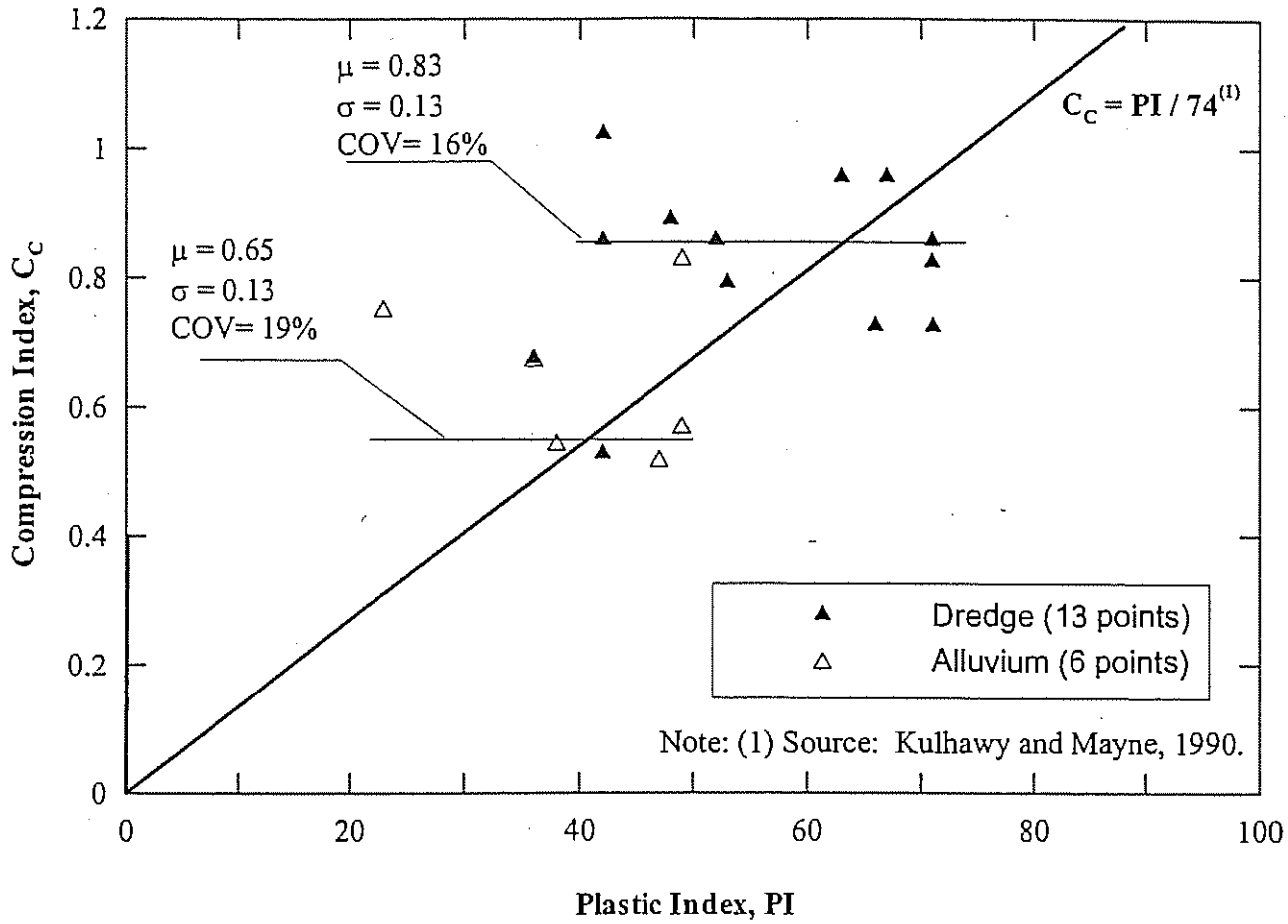
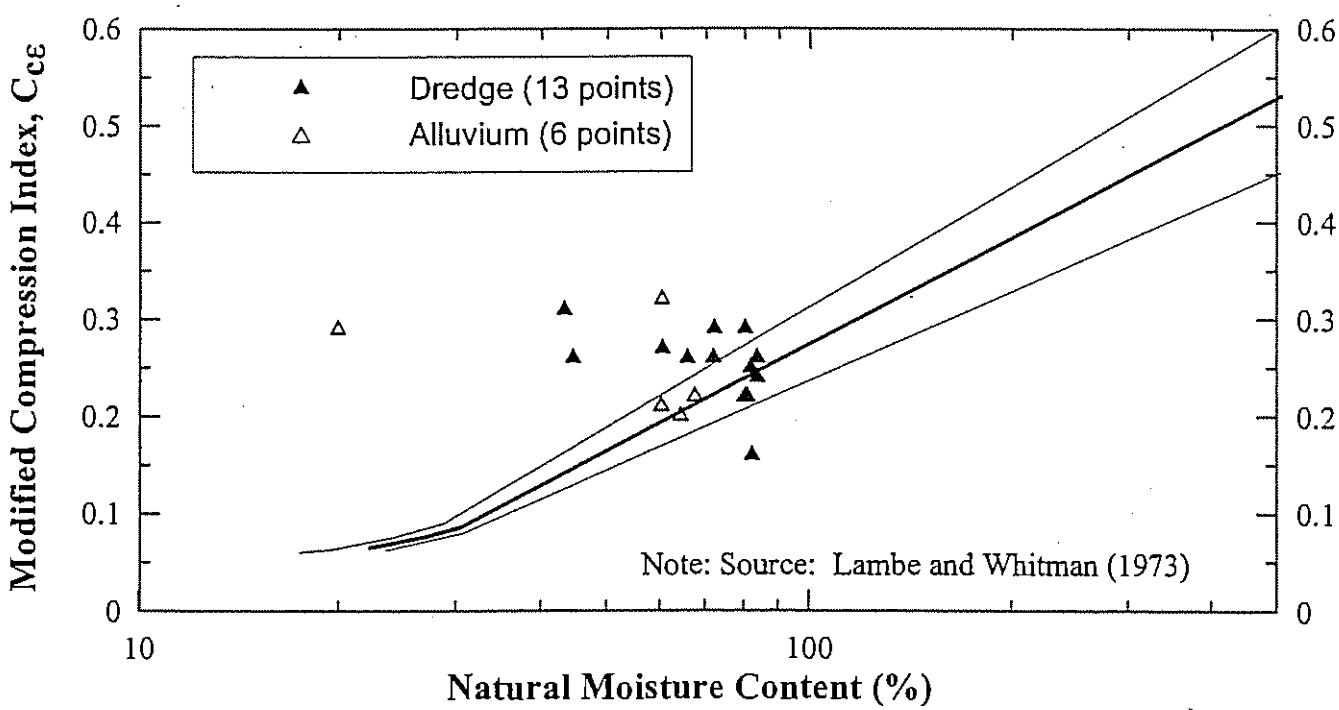
Profile of Compression Index of Fine-Grained Soils

FIGURE 6-47

PROJECT: ME0247
Cherry Island





Cc VS. Wc(2).grf 4/4/03

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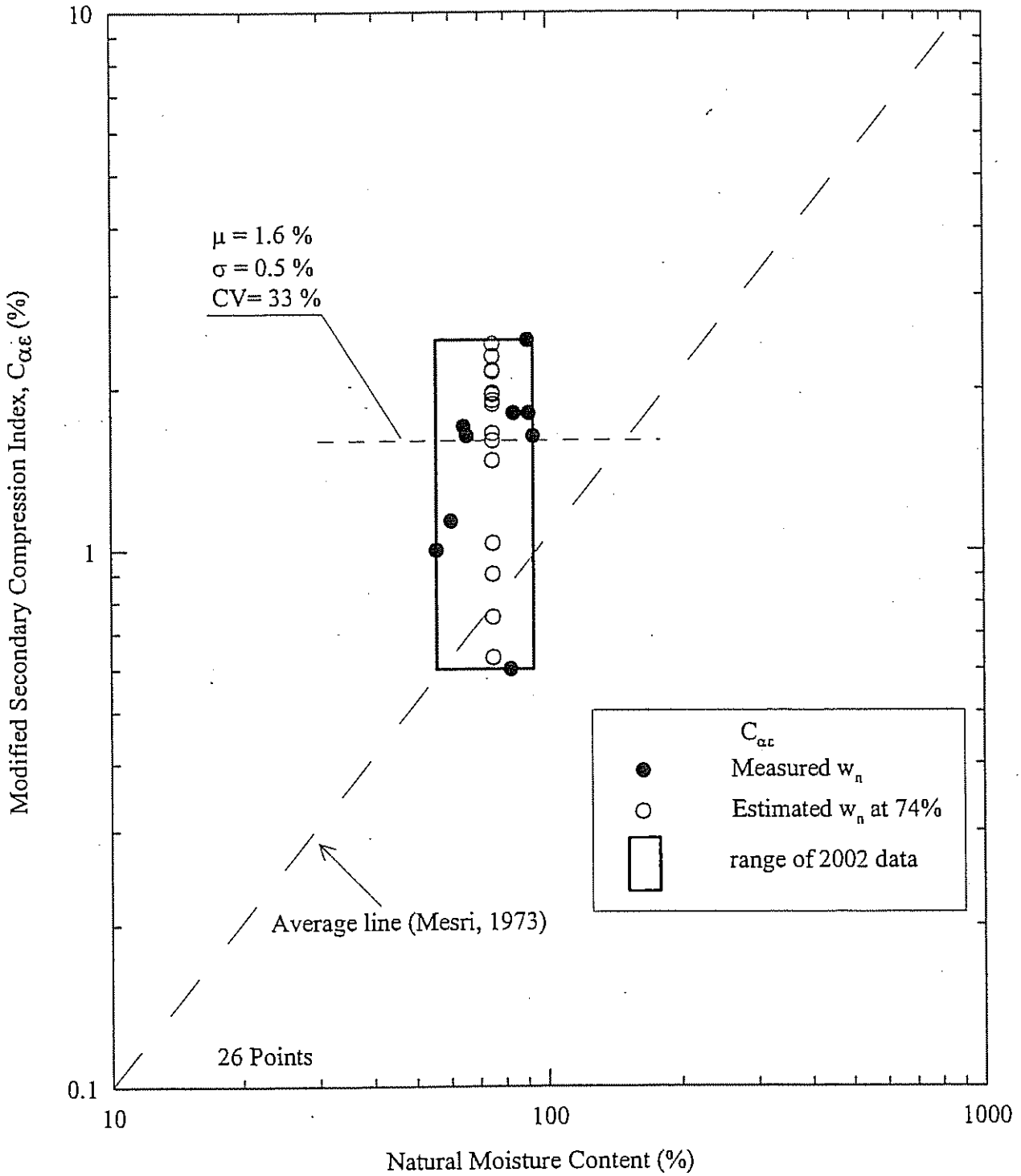
Compression Index vs. Natural Moisture Content

FIGURE 6-48

PROJECT: ME0250
Cherry Island



NOTE: μ = Mean; σ = Standard Deviation; CF = Coefficient of Variation (%)

Cae vs Wc.grf

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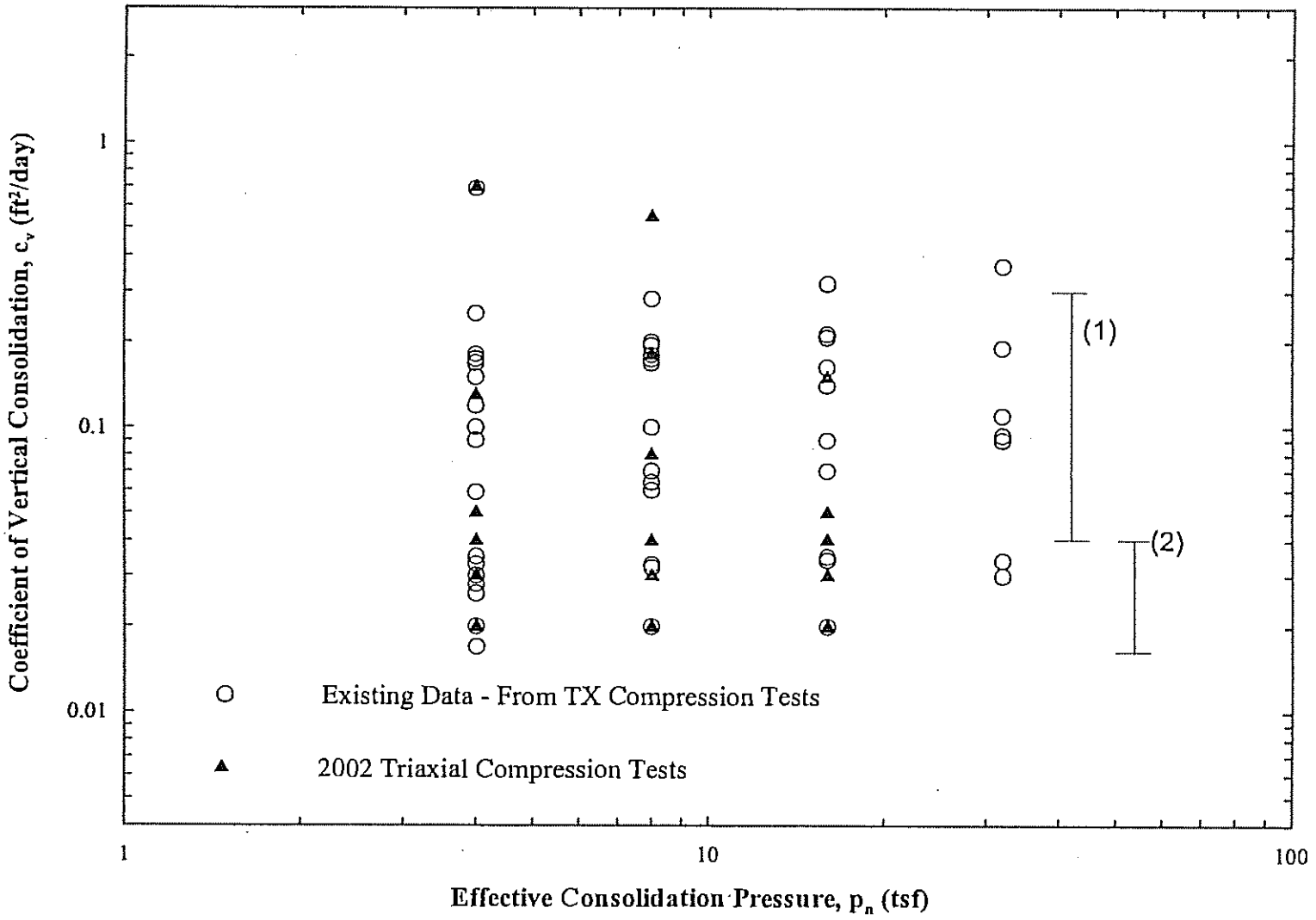
Secondary Compression Index vs. Natural Moisture Content

FIGURE 6-49

PROJECT: ME0250
Cherry Island



NOTES:
(1) Values for cv used in previous reports (i.e. 0.04 - 0.3 ft^2/day)
(2) Typical range for c_v obtained from NAVFAC (1971) for LL=98

combined consolidation data.grf

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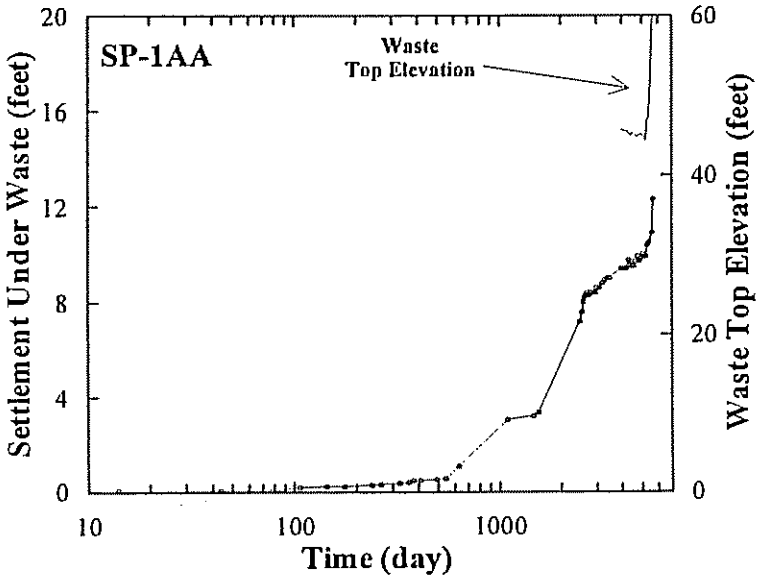
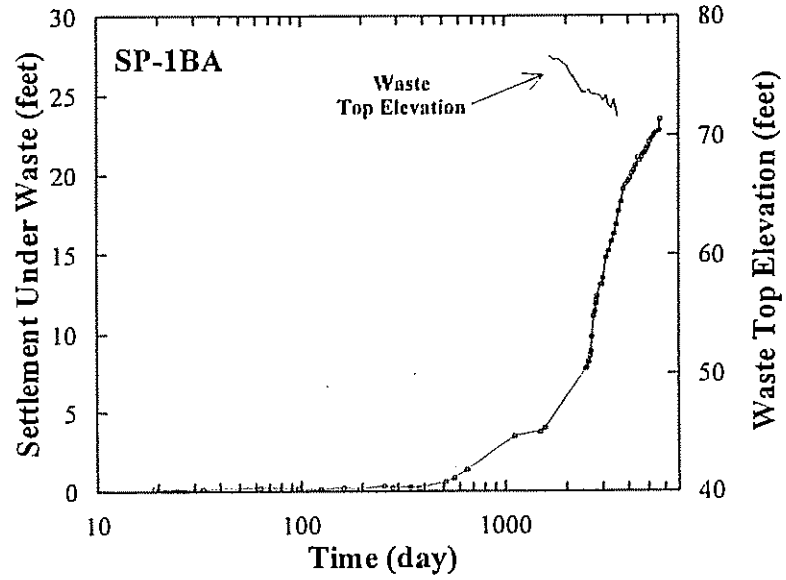
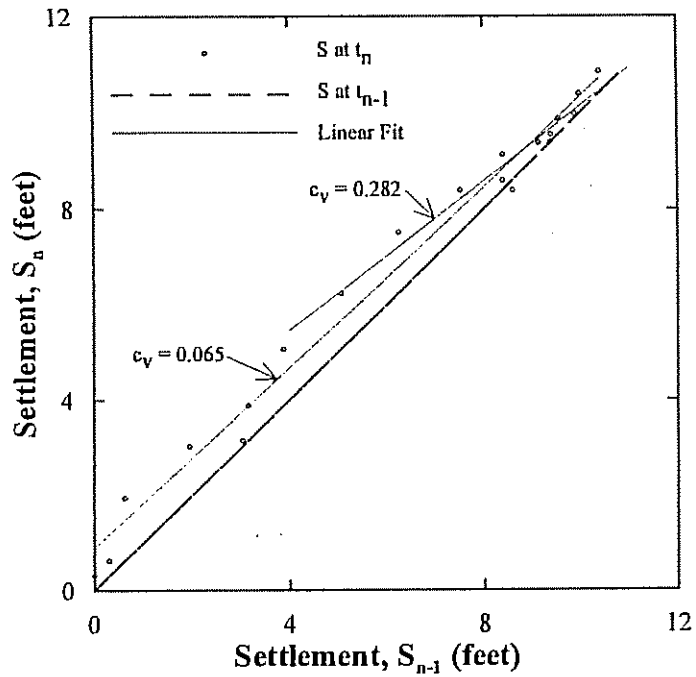
Coefficient of Consolidation from Triaxial Tests

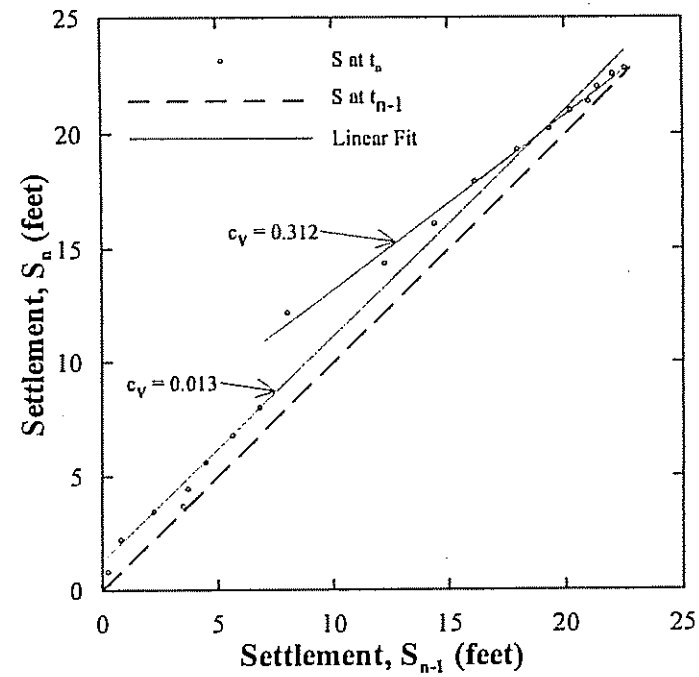
FIGURE 6-50

PROJECT: ME0250
Cherry Island









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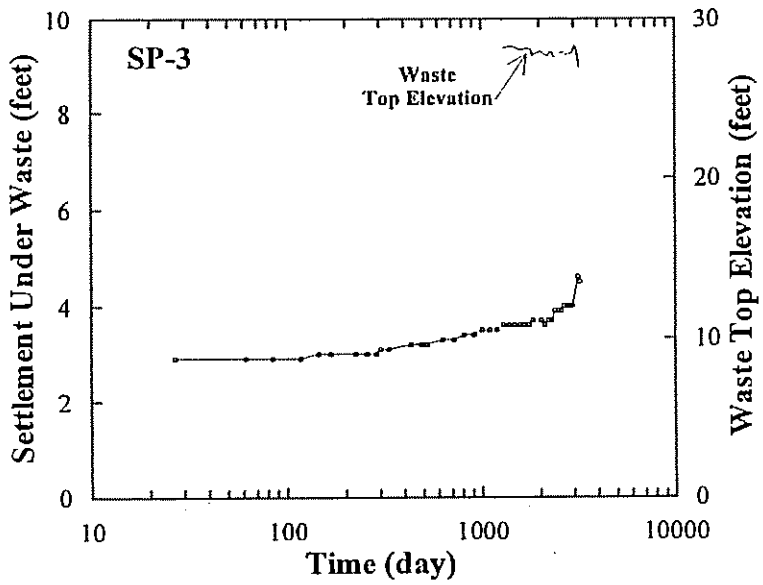
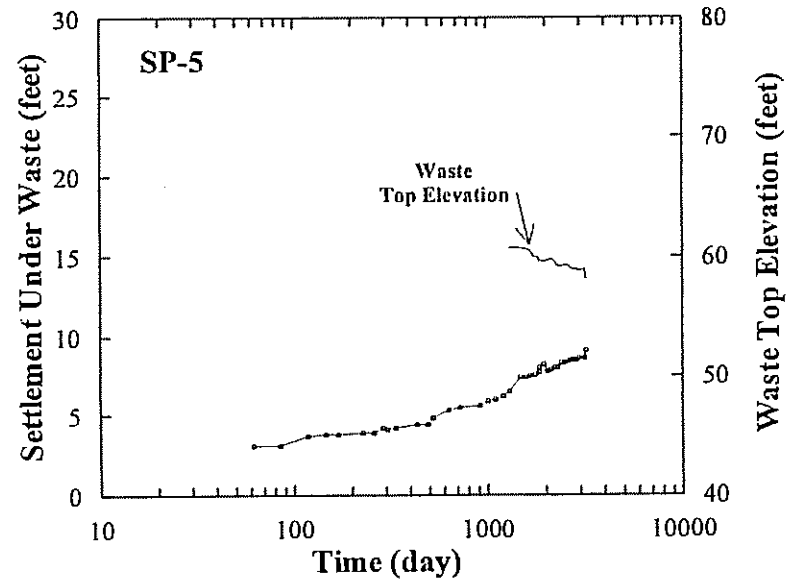
Estimation of Field Coefficient of Consolidation Phase I - SP-1AA and SP-1BA

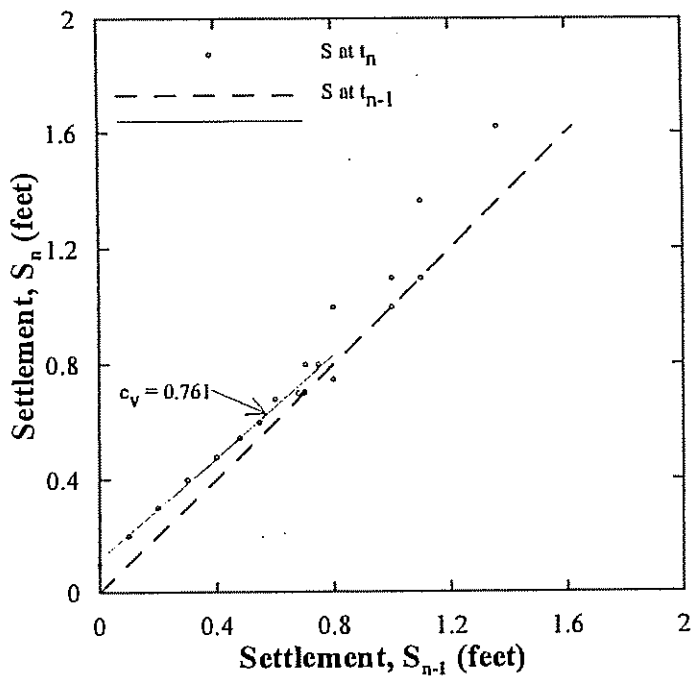
FIGURE 6-51

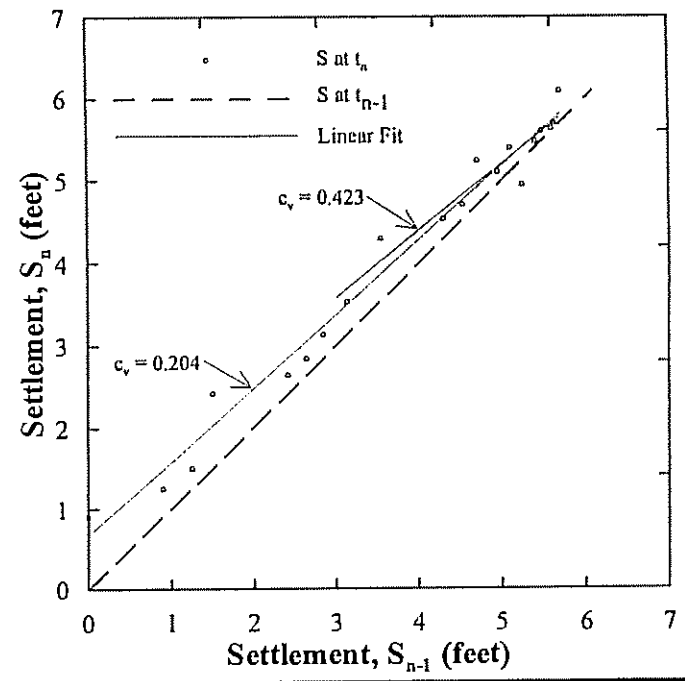
PROJECT: ME0250
Cherry Island

Phase 1_1AA&1BA.grf









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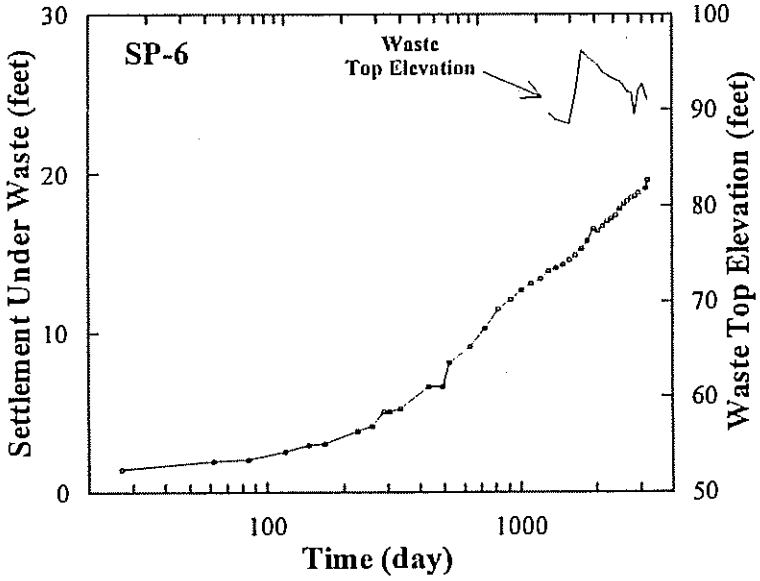
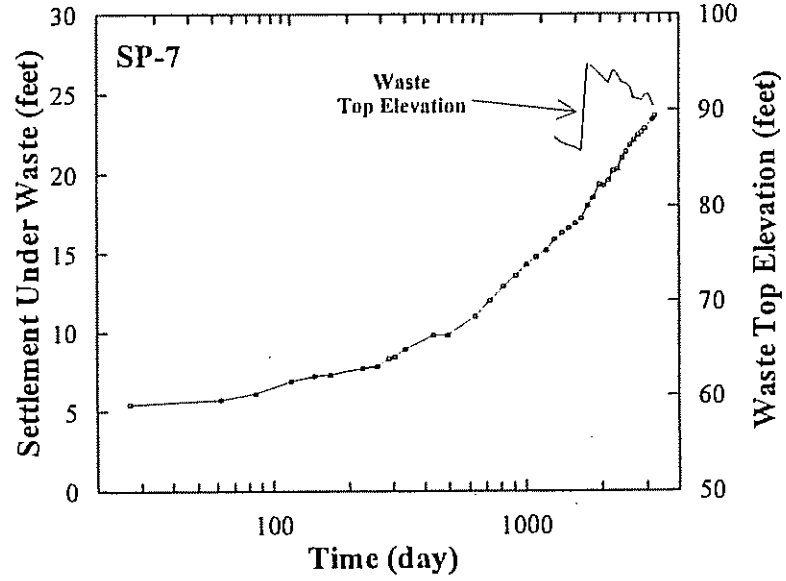
Estimation of Field Coefficient of Consolidation Phase I - SP-3 and SP-5

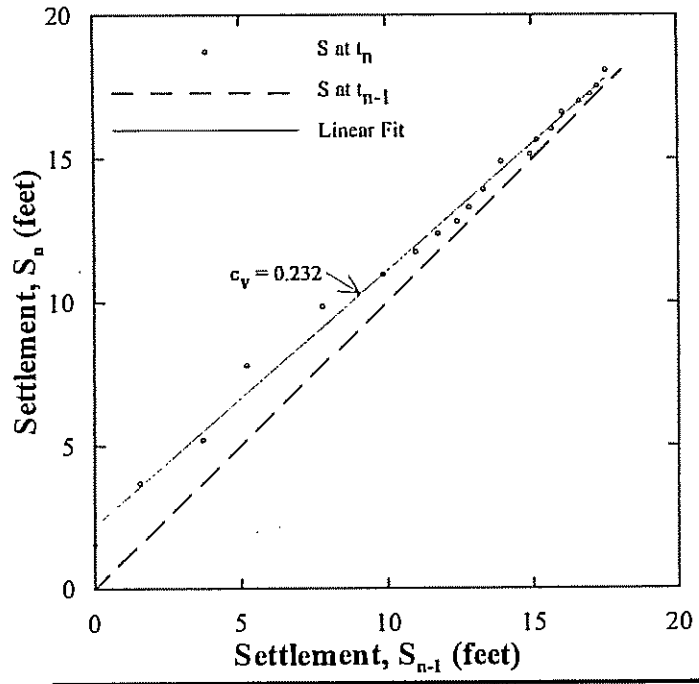
FIGURE 6-52

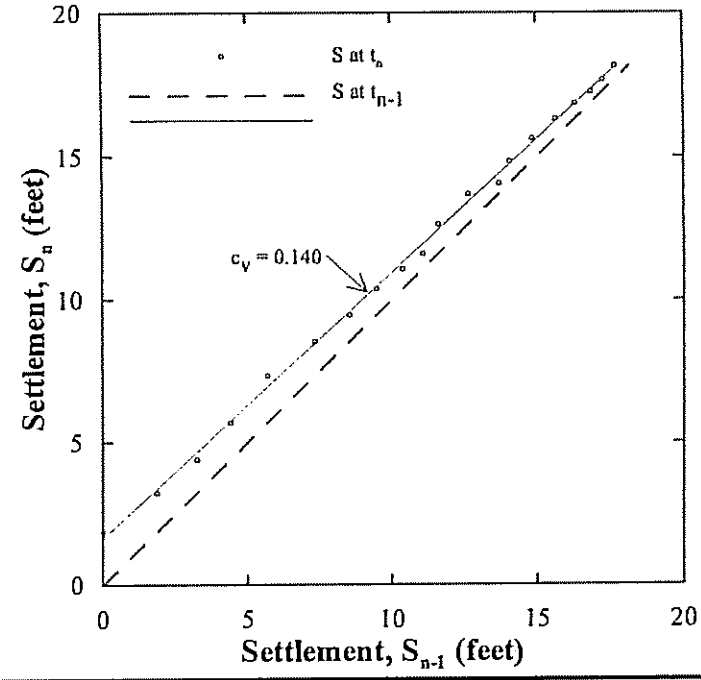
PROJECT: ME0250
Cherry Island

Phase 1_3&5.grf









Phase 1_6&7.grf

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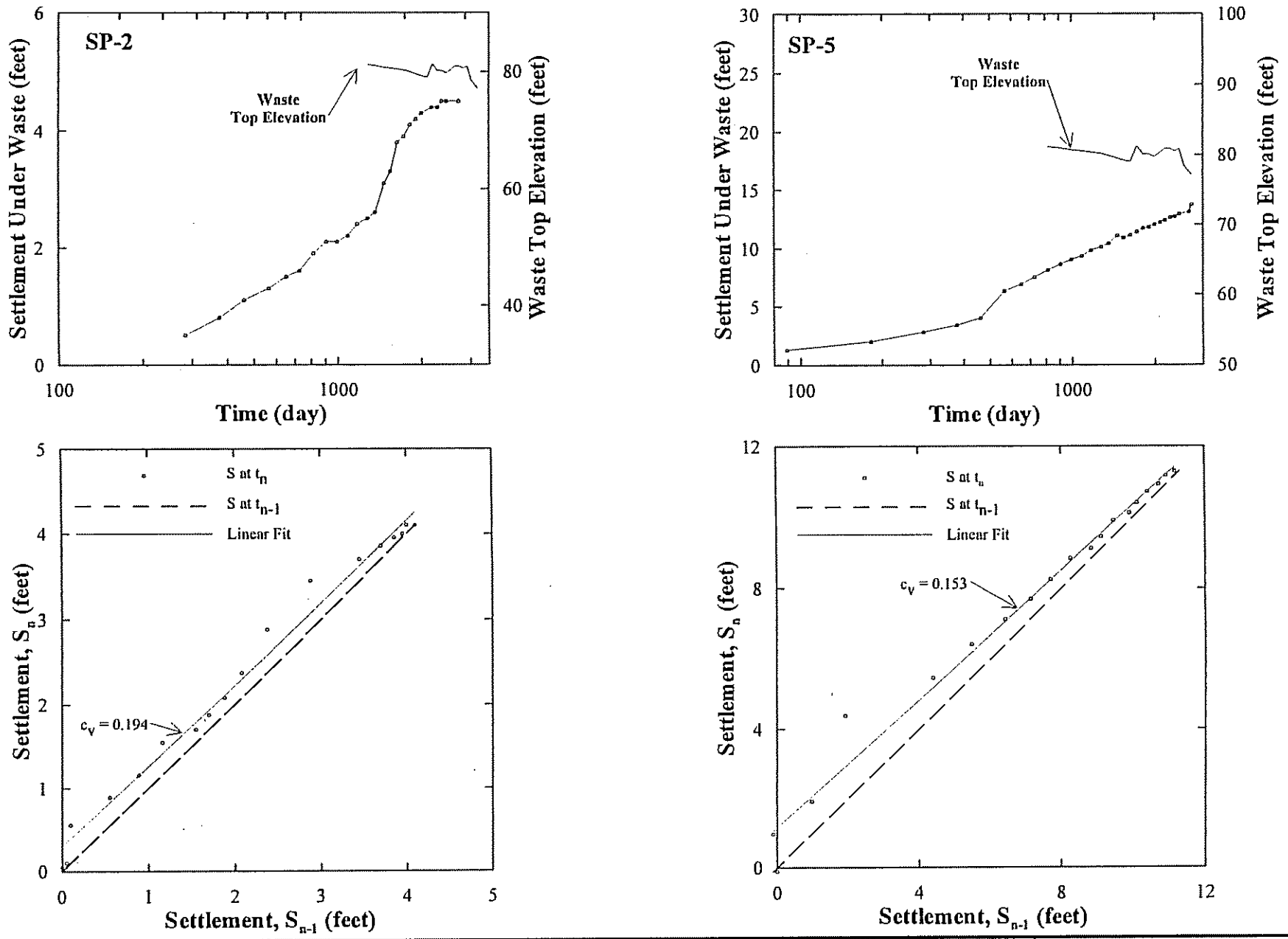
Estimation of Field Coefficient of Consolidation Phase I - SP-6 and SP-7

FIGURE 6-53

PROJECT: ME0250
Cherry Island



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Estimation of Field Coefficient of Consolidation Phase II - SP-2 and SP-5

FIGURE 6-54

PROJECT: ME0250
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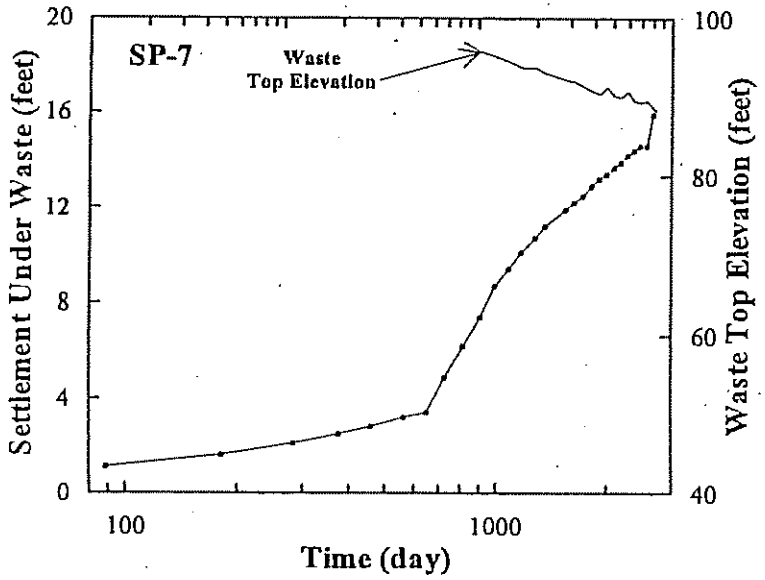
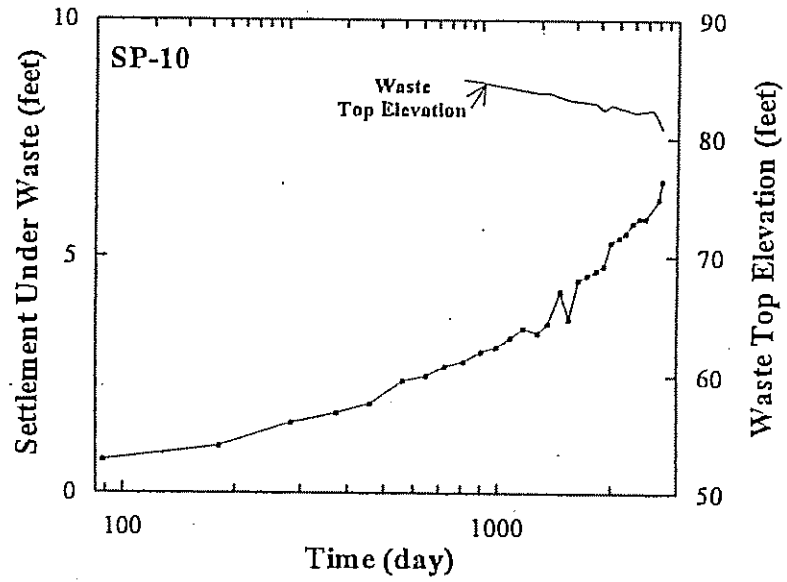
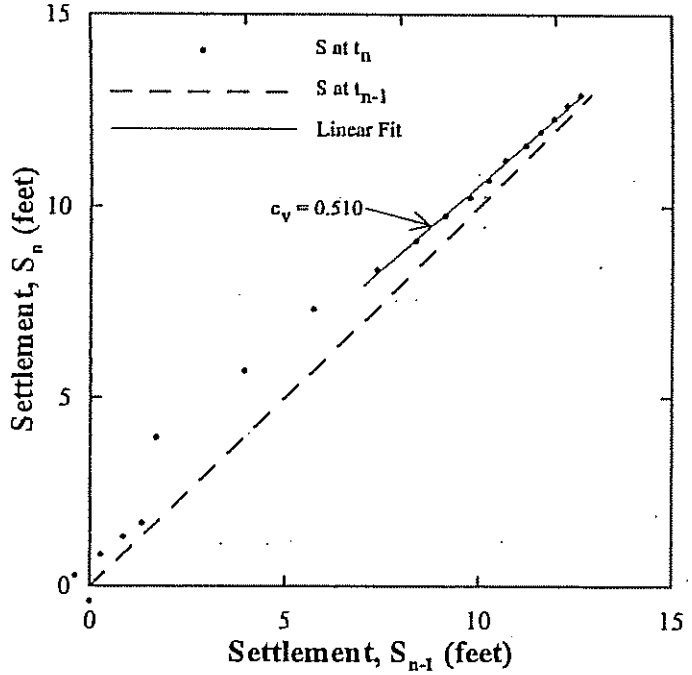
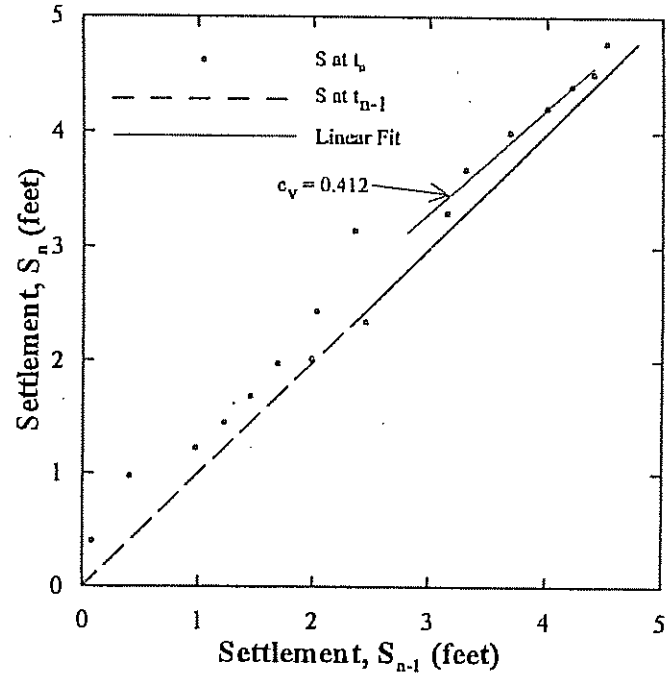
Phase 2_2&5.grf









Phase 2_7&10.grf

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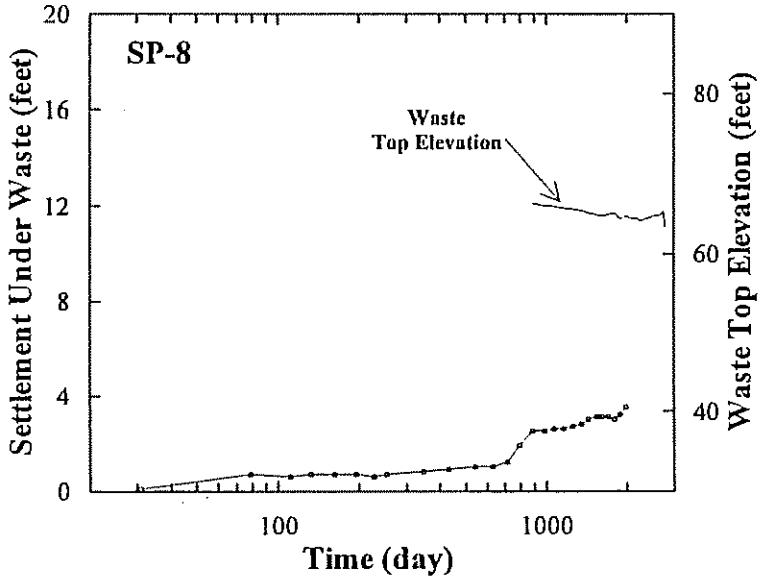
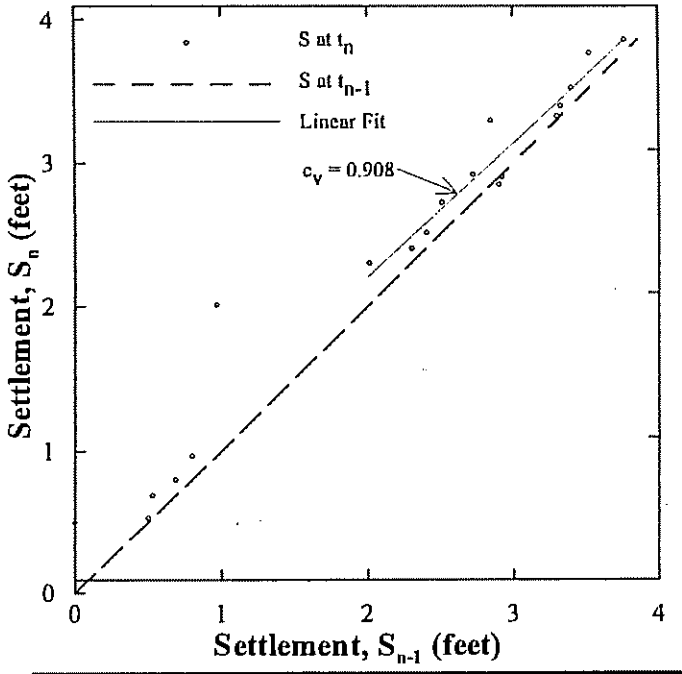
Estimation of Field Coefficient of Consolidation Phase II - SP-7 and SP-10

FIGURE 6-55

PROJECT: ME0250
Cherry Island





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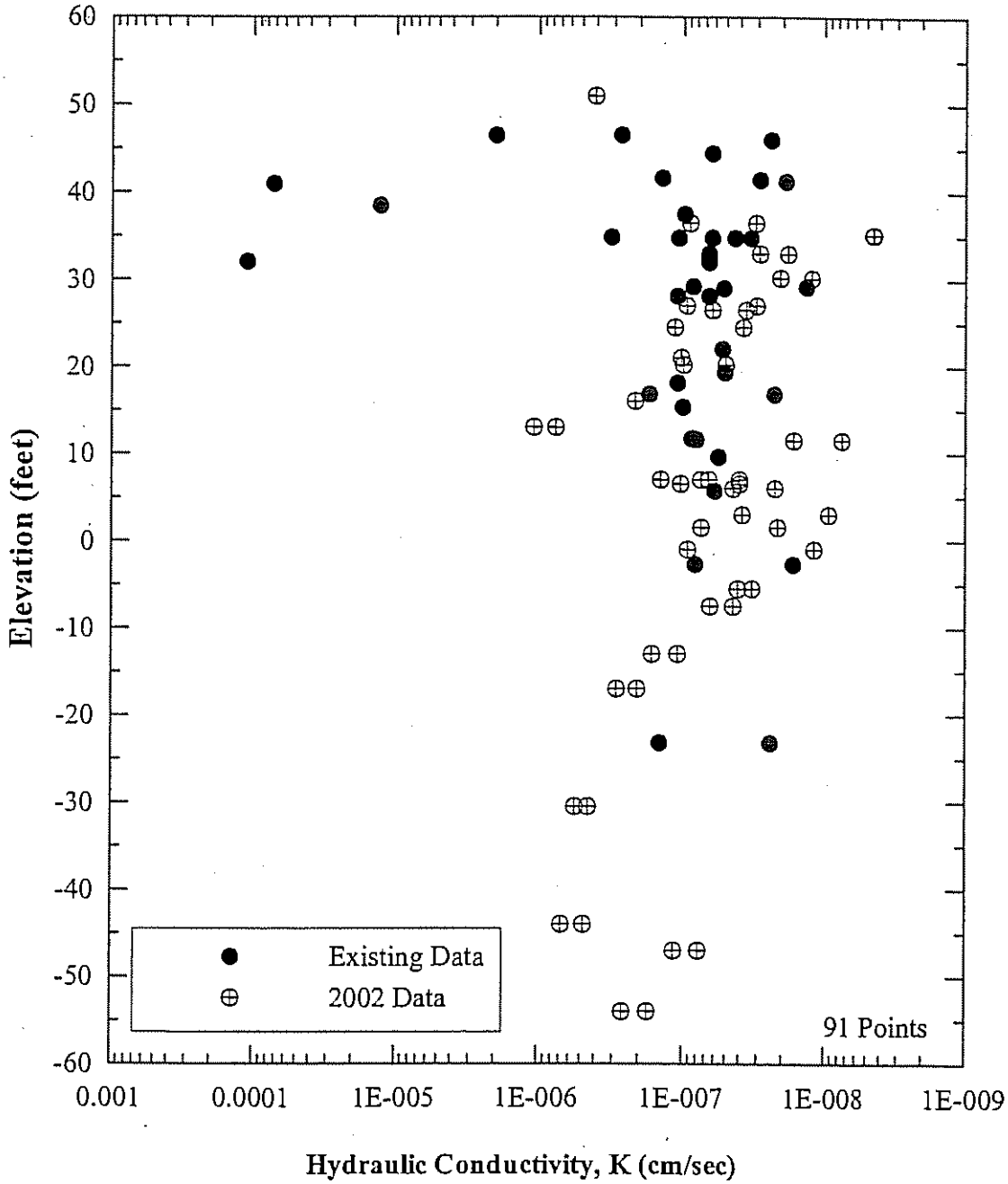
Estimation of Field Coefficient of Consolidation Phase III - SP-8

FIGURE 6-56

PROJECT: ME0250
Cherry Island

Phase 3-8.grf



Hydraulic Conductivity w depth.grf

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Profile of Hydraulic Conductivity of Fine-Grained Soils

FIGURE 6-57

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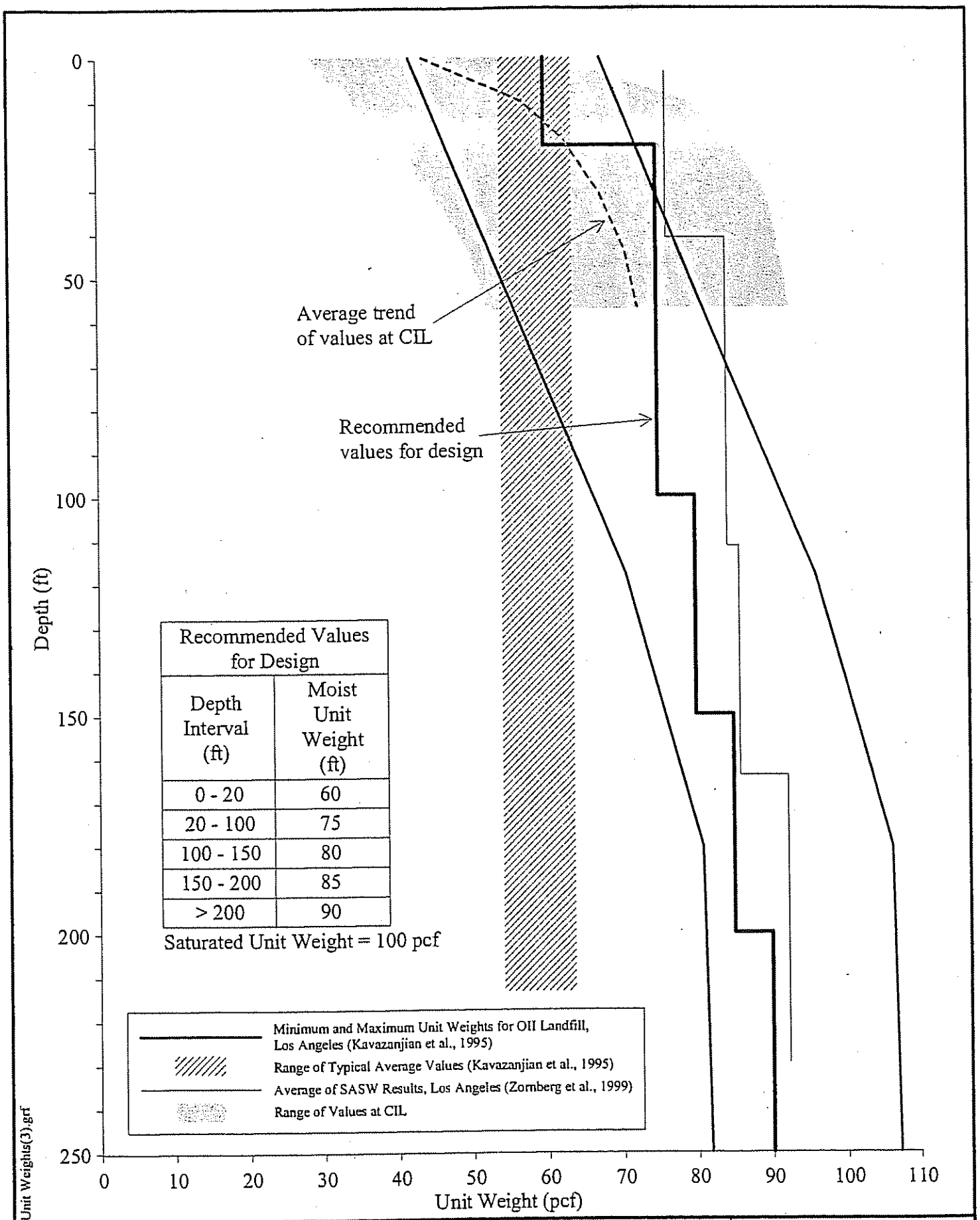

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Recommended Waste Unit Weight Variation with Depth

FIGURE 7-1

PROJECT: ME0250
Cherry Island