DELAWARE SOLID WASTE AUTHORITY

prepared for:

Delaware Solid Waste Authority
1128 S. Bradford Street
Dover, Delaware 19903

Volume 14: Hydrogeologic Assessment Reports II

Cherry Island Landfill Expansion Project Wilmington, Delaware

prepared by:

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Columbia, Maryland 21046

Project No. ME0250
April 2003 -
Revised September 2003
Revised June 2004

HYDROGEOLOGIC ASSESSMENT REPORTS II:

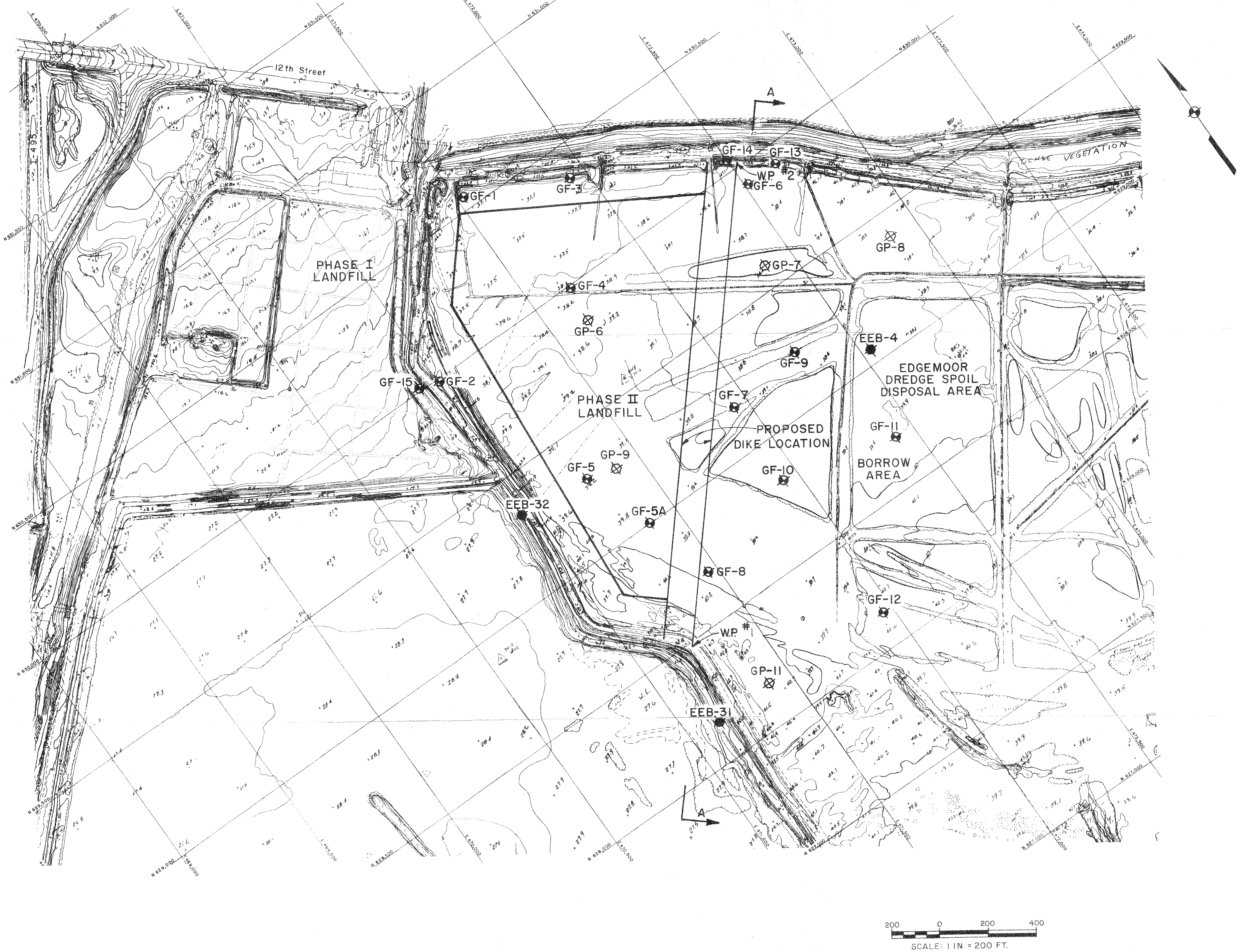
1. Gannett Fleming Environmental Engineers, Inc., "Northern Solid Waste Facility – 2, Phase II Area, Landfill Design Report," prepared for the Delaware Solid Waste Authority, December 1986.

2. Gannett Fleming Engineers, Inc., "Northern Solid Waste Facility-2, Interim Hydrogeology Report, Phase II Landfill," prepared for the Delaware Solid Waste Authority, January 1987.

3. Gannett Fleming, Inc., "Phase III Design Memorandum," prepared for the Delaware Solid Waste Authority, March 1990.

4. Gannett Fleming, Inc., "Phase III, Cherry Island, Wilmington, Delaware, Geotechnical and Hydrogeologic Report," prepared for the Delaware Solid Waste Authority, March 1990 - Revised August 1990.

5. Roy F. Weston, Inc., "Northern Solid Waste Management Center – Cherry Island Landfill, Phase IV Disposal Area, Hydrogeologic, Geotechnical and Landfill Capping Report," prepared for the Delaware Solid Waste Authority, August 1992.



Dredge Spoil Borrow not to be excavated closer than 100 feet to the toe of Proposed Dike.

WORK POINT COORDINATES

W.P. #1	N 628,482.005 E 471,222.028
W.P. #2	N 630,018.330 E 472,527.708

LEGEND

- BORINGS PERFORMED FOR PHASE II LANDFILL.
- BORINGS PERFORMED BY U.S. ARMY CORPS OF ENGINEERS.
- BORINGS PERFORMED BY TERRAQUA ASSOCIATES

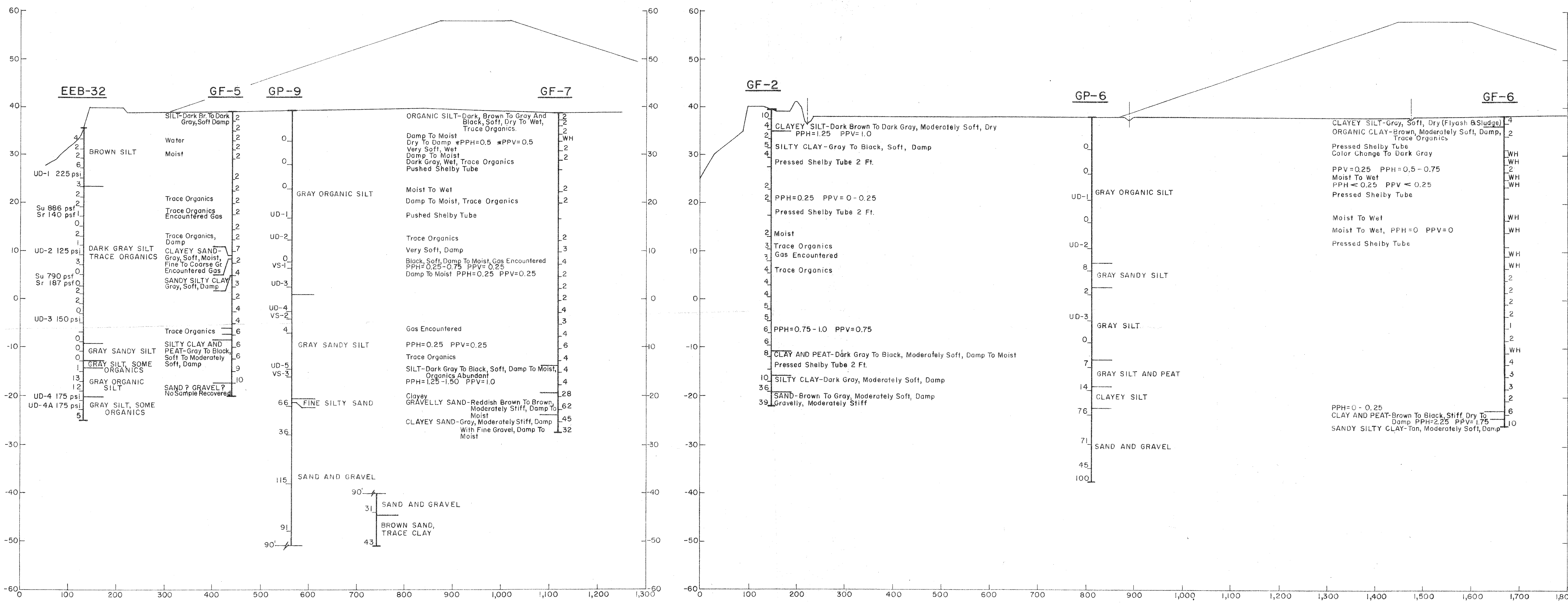
NOTES

Topography as determined by Aerial Photography in 1985. Elevations referenced to National Geodetic Survey Datum Coordinates – Delaware State Plane Coordinates.

PHASE II
NORTHERN SOLID WASTE FACILITY – 2
DELAWARE SOLID WASTE AUTHORITY

SITE PLAN PHASE II LANDFILL AND PROPOSED DIKE LOCATION

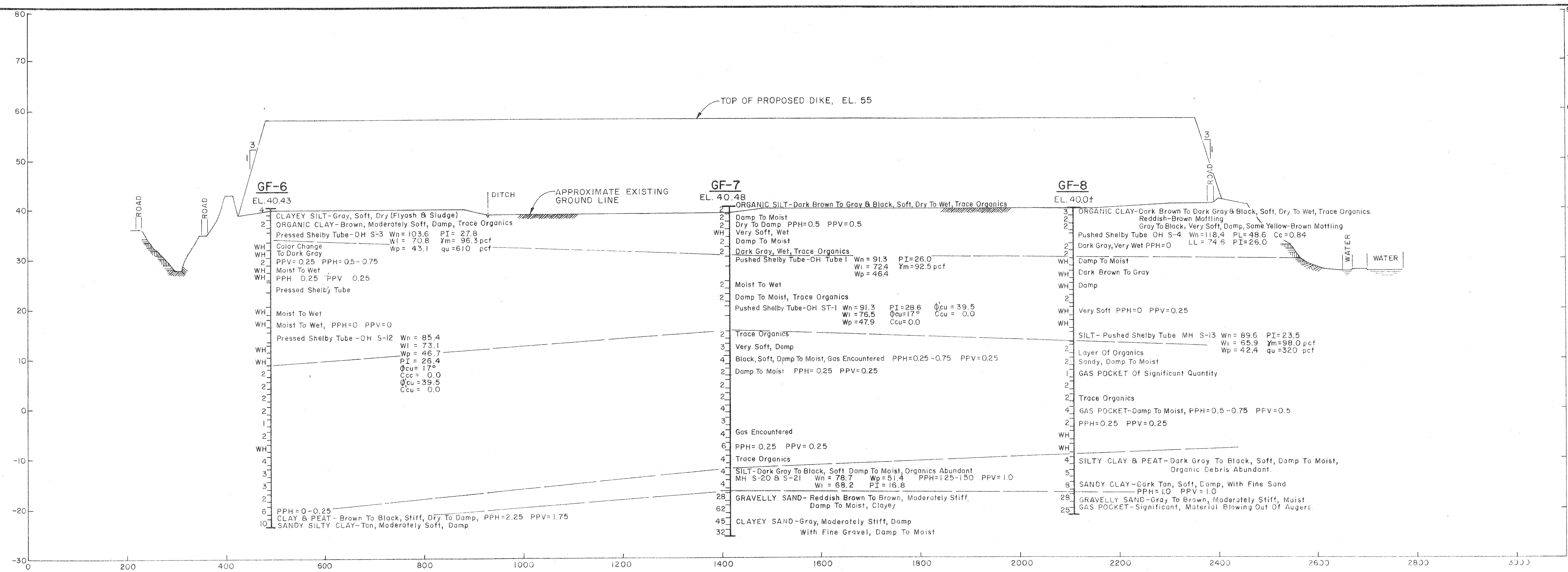
GANNETT FLEMING ENVIRONMENTAL ENGINEERS, INC.
BALTIMORE, MARYLAND OCT. 1986 PLATE I



PHASE II
NORTHERN SOLID WASTE FACILITY - 2
DELAWARE SOLID WASTE AUTHORITY

SOIL PROFILES

GANNETT FLEMING ENVIRONMENTAL ENGINEERS, INC.
BALTIMORE, MARYLAND SEPT. 1986 PLATE III



SECTION A-A

PHASE II
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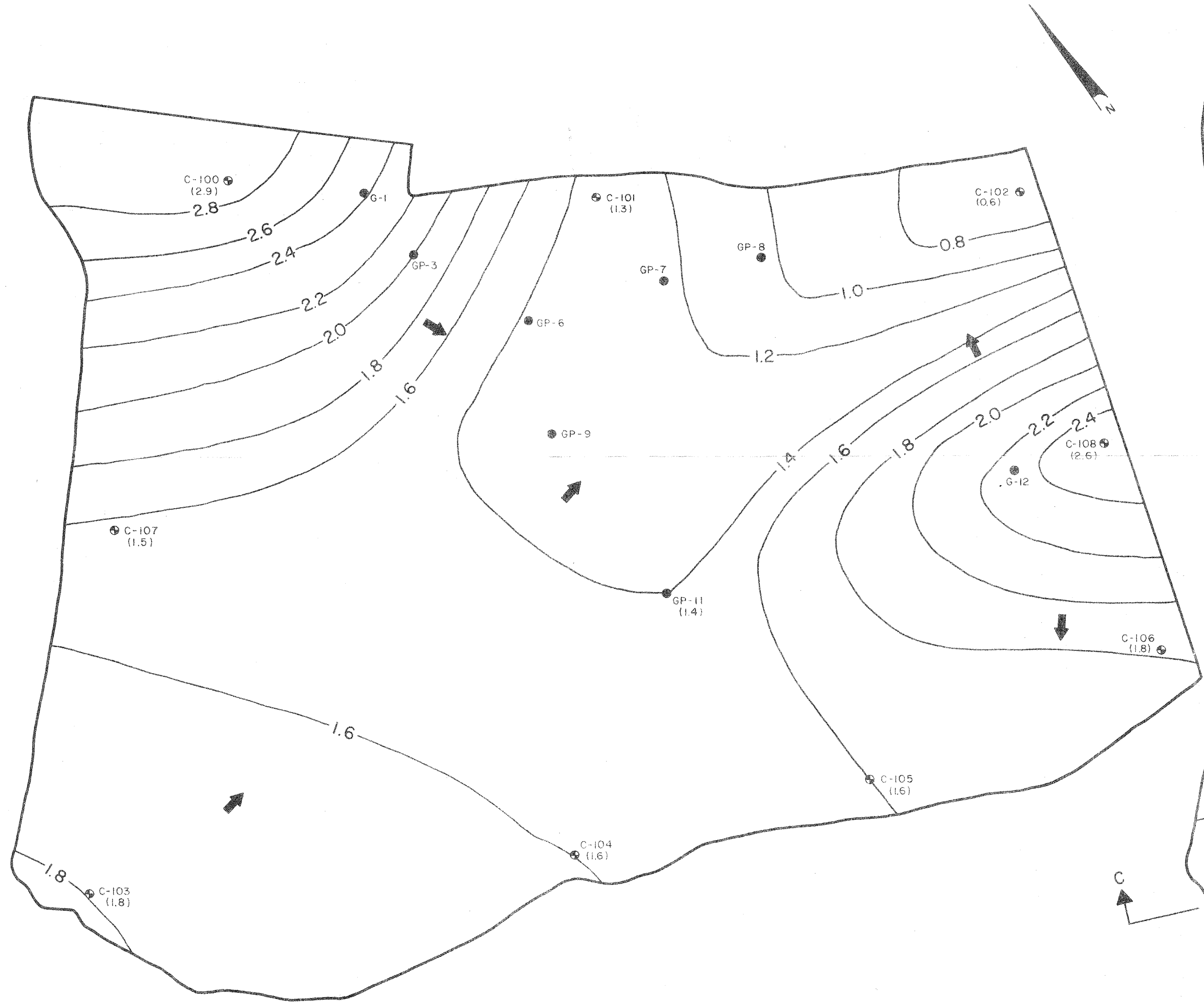
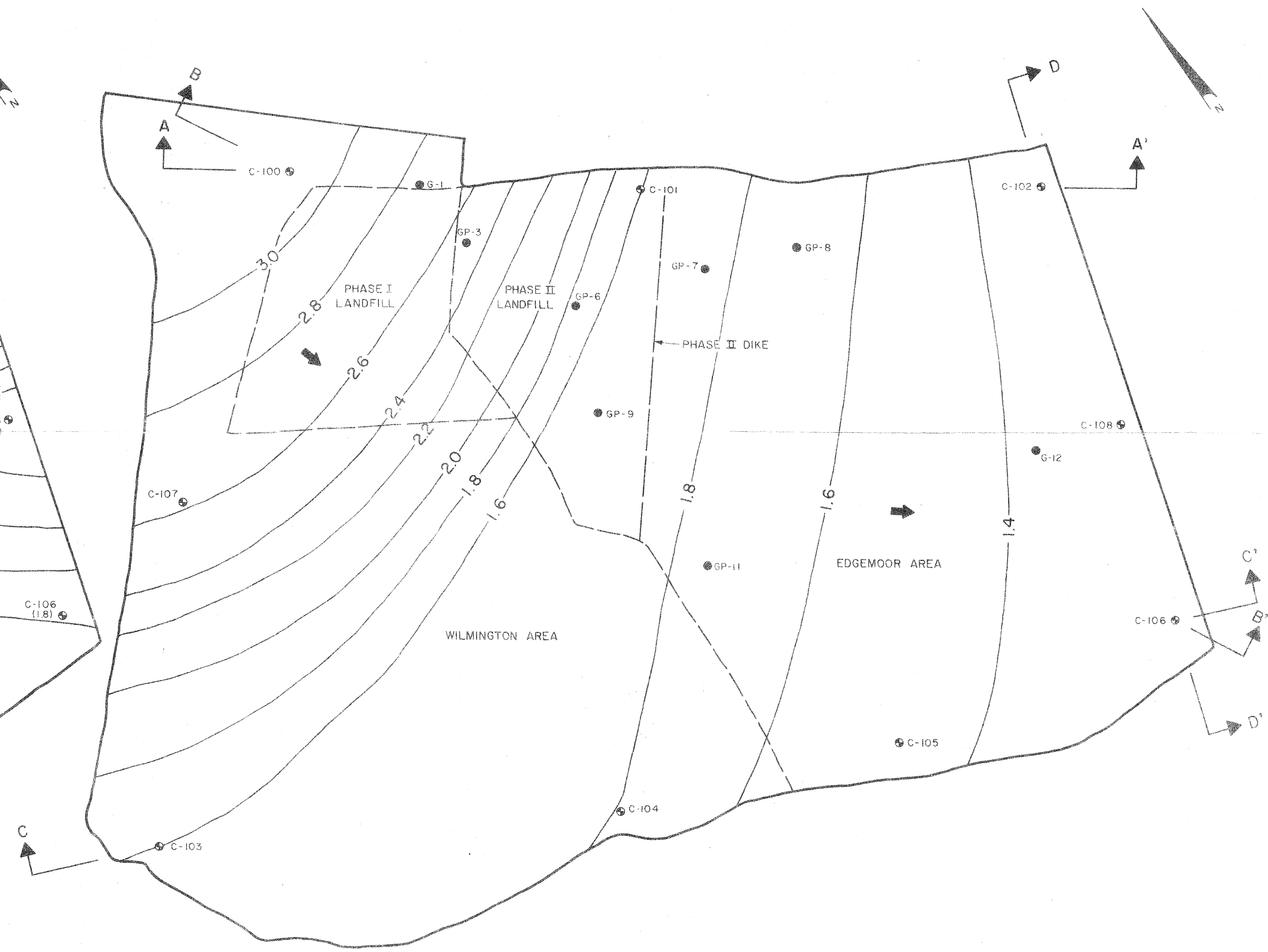
SOIL CROSS SECTION A-A

GANNETT FLEMING ENVIRONMENTAL ENGINEERS, INC.
BALTIMORE, MARYLAND SEPT. 1986 PLATE II



POTENTIOMETRIC SURFACE MAP
COLUMBIA FORMATION
MODIFIED FROM TERRAQUA 1984



PROPOSED POTENTIOMETRIC SURFACE MAP
COLUMBIA / RECENT SEDIMENTS

LEGEND

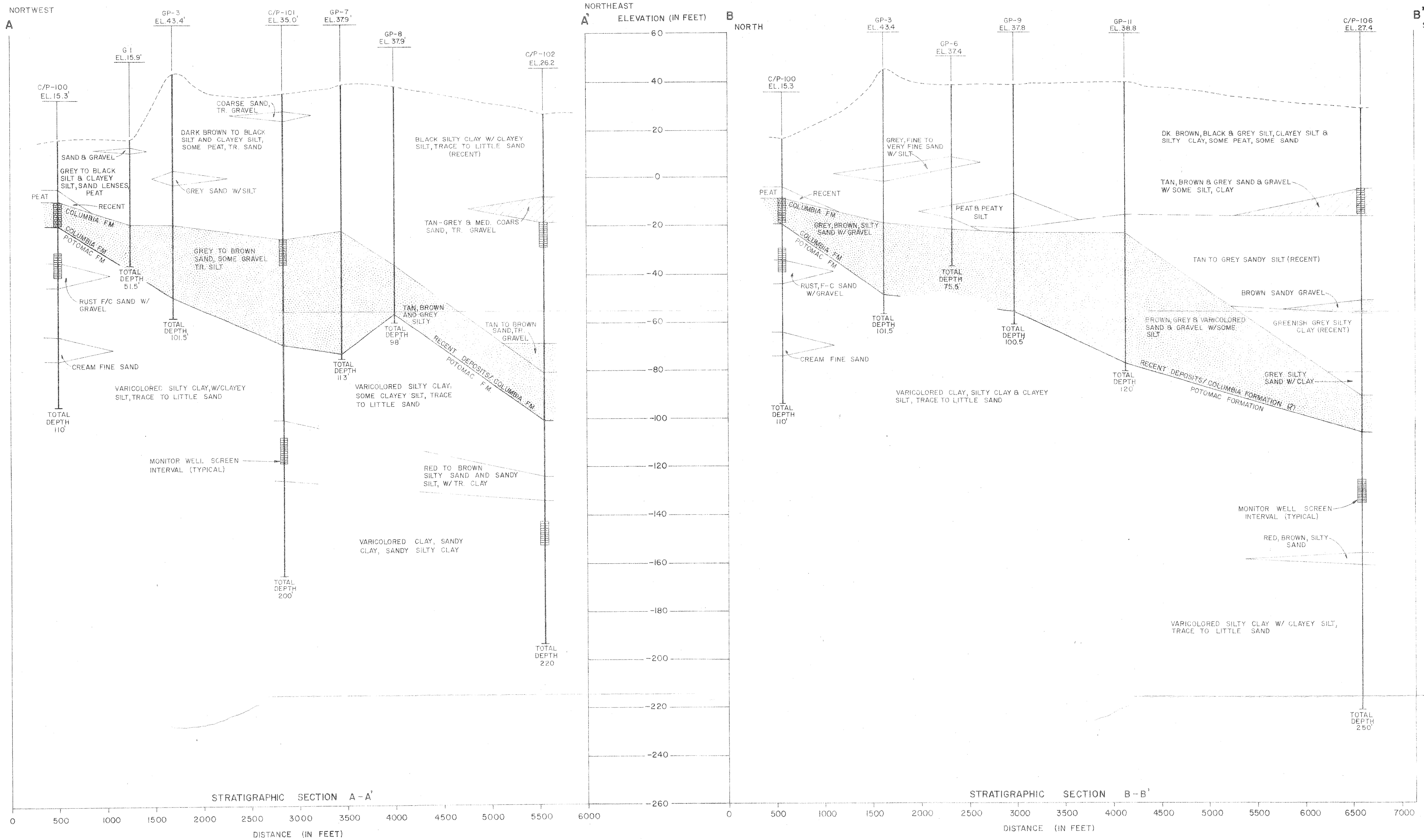
● G-1	GEOTECHNICAL BORING
● GP-3	GEOTECHNICAL BORING WITH DEEP PIEZOMETER
⊗ C-102	COLUMBIA FORMATION MONITORING WELL
A — A'	STRATIGRAPHIC CROSS-SECTION
(2.6)	GROUNDWATER ELEVATION
—1.6—	POTENTIOMETRIC SURFACE CONTOUR (EL. IN FEET
➡	GROUNDWATER FLOW DIRECTION

400 0 400 800
SCALE IN FEET

GANNETT FLEMING ENVIRONMENTAL ENGINEERS, INC.
BALTIMORE, MARYLAND JAN. 1987

PHASE II
NORTHERN SOLID WASTE FACILITY - 2
DELAWARE SOLID WASTE AUTHORITY

PLATE IV



NOTES:

1. SEE PLATE IV FOR CROSS-SECTION LOCATIONS.
2. THESE CROSS-SECTIONS ARE MODIFIED FROM DUFFIELD ASSOC. OCT 1985.
3. STRATIGRAPHIC SECTIONS ARE BASED ON DRILLER'S DESCRIPTIVE LOGS FOR CONDITIONS ENCOUNTERED BY THE TEST BORINGS AND WELL BORINGS, AND STRAIGHT LINE INTERPOLATION OF CONDITIONS BETWEEN BORINGS. ACTUAL CONDITIONS BETWEEN BORINGS ARE UNKNOWN.
4. STRATA TEXTURAL DESCRIPTIONS ARE A GENERALIZATION OF INDIVIDUAL SAMPLE DESCRIPTIONS INDICATED ON THE DRILLER'S DESCRIPTIVE LOGS. FOR LOGS SEE "SITE SUITABILITY REPORT, HYDROGEOLOGIC CONDITIONS AND GEOTECHNICAL EVALUATION OF THE CHERRY ISLAND SITE", PREPARED BY TERRAQUA RESOURCES CORP., DATED 1 FEB. 1984.

KEY

- UPPER SAND ZONE
- LOWER SAND ZONE

PHASE II
NORTHERN SOLID WASTE FACILITY - 2
DELAWARE SOLID WASTE AUTHORITY

GANNETT FLEMING ENVIRONMENTAL ENGINEERS, INC.
BALTIMORE, MARYLAND JAN 1987

PLATE V
STRATIGRAPHIC SECTIONS
A-A' AND B-B'

1. Gannett Fleming Environmental Engineers, Inc., "Northern Solid Waste Facility – 2, Phase II Area, Landfill Design Report," prepared for the Delaware Solid Waste Authority, December 1986.

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Delaware Solid Waste Authority

Northern Solid Waste Facility - 2

Phase II Area

Landfill Design Report

December 1986

-

Submitted by

Gannett Fleming Environmental Engineers, Inc.
Baltimore, Maryland

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1.0 Introduction

This document presents site conditions, solid-waste quantity projections, and design information for development of Phase II of the Northern Solid Waste Facility-2 (NSWF-2) at Cherry Island near Wilmington, Delaware. This landfill area represents the second phase of a program that will eventually involve much of the Cherry Island site. Site suitability and hydrogeologic studies were carried out by the Delaware Solid Waste Authority (DSWA) in 1984. Engineering activities for Phase II, carried out from July through November, 1986, included soil investigations, topographic surveying, soil stability analysis and consolidation testing, and landfill design. This report is organized into several sections:

2.0 Existing Conditions

Phase I landfilling practices, solid waste projections, influence of the Energy Generating Facility (EGF) on solid waste quantities.

3.0 Phase II Geotechnical Investigation

Description of site geology, description of subsurface investigation, laboratory testing results, description of soils.

4.0 Site Hydrogeology

Hydrogeologic setting and groundwater flow patterns, groundwater quality and monitoring.

5.0 Phase II Landfill Design

Description of landfill design approach, leachate management, landfill instrumentation.

6.0 Phase II Landfill Operation

Description of filling sequence, stormwater management during and following filling, landfill instrument monitoring.

2.0 Existing Conditions

2.1 Solid Waste Quantities

Landfilling operations at Phase I of the NSWF-2 began in October, 1985. Deliveries to the site are summarized in Table 2.1 for the six month period ending March, 1986.

Table 2.1

Summary by Source of Solid
Waste for Phase I - NSWF-2

Light industrial wastes	95,000 Tons
Residues from DRP	96,000 Tons
Industrial sludges	3,000 Tons
Miscellaneous other sources	6,000 Tons
Total	200,000 Tons

On a daily basis the Phase I site receives about 900 tons of residue from the Delaware Reclamation Project (DRP) and 750 tons from all other sources Monday through Friday plus 500 tons on Saturday. The weekly total averaged 8750 tons during the first eleven months of operation. A compacted density of 1150 pounds per cubic yard has been achieved for these deposits excluding cover soil.

Deliveries to the site have been substantially greater than projected at the time Phase I was being developed. Initial estimates included 120,000 tons of waste delivered directly to the landfill and 26,000 tons of residues from DRP. To this figure an additional 13,000 tons was added to account for DRP facility outages to give a first-year total of 159,000 tons. Projected actual deliveries for the first year are 221,000 tons of waste delivered directly to the landfill and 234,000 tons of residues from DRP for a first-year total of 455,000 tons.

Solid waste quantities are expected to increase in proportion to a 1.1% annual increase in population in the area. DSWA has developed a waste-to-energy incineration facility that will accept and burn preprocessed solid waste from the DRP as well as unprocessed waste from other sources. This Energy Generation Facility (EGF) entered the start-up and testing phase in the latter part of 1986. When it becomes fully operational, solid waste deliveries to NSWF-2 are expected to decrease from 8,750 tons weekly to 5,200 tons weekly. If the EGF reaches routine operation by March, 1987, solid waste quantities delivered to the landfill are projected to be as shown on Table 2.2.

The Phase I area was estimated to provide capacity for 750,000 tons of solid waste. The projections shown on Table 2.2 indicate this capacity will be reached about midway between March and September 1987.

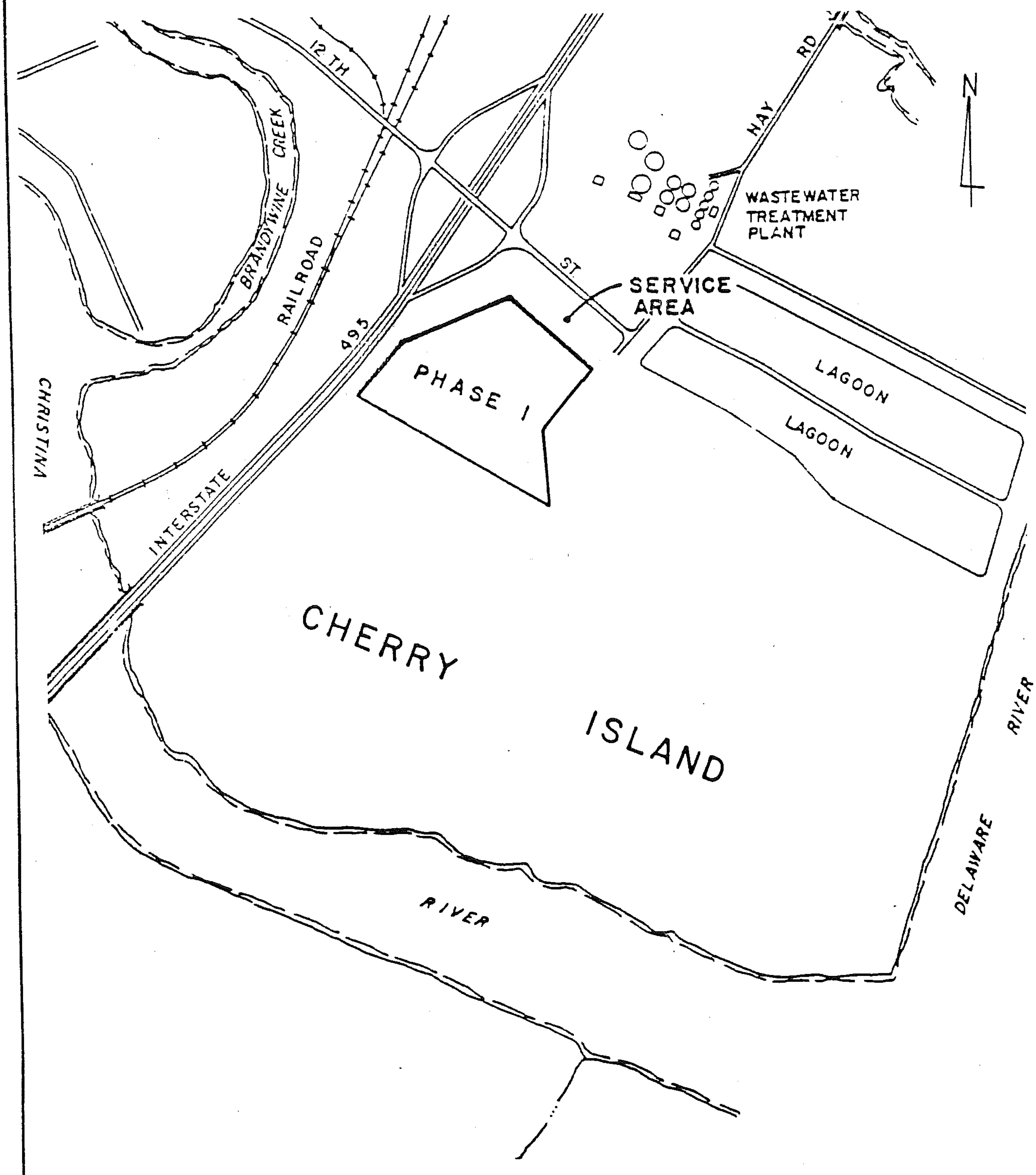
2.2 Phase I Landfilling Practice

Phase I was the first area to be developed for use as a sanitary landfill at the NSWF-2. The U.S. Army Corps of Engineers (COE) has used the site for several decades to dispose of spoil material resulting from Delaware and Christina River dredging operations. Phase I (see Figure 2.1) was established by construction of a dike along its southern edge, to avoid inundation as dredge spoil disposal continued to the south of Phase I. The dredge spoil materials exhibit extremely low permeability, a characteristic that makes the site very suitable for sanitary

TABLE 2.2

Solid Waste Quantities Projected to be Delivered
to NSWF-2

Six-month Period Ending	Tons Solid Waste Delivered	Cumulative Tons Solid Waste
March 1986	220,000	220,000
September 1986	228,000	448,000
March 1987	229,000	677,000
September 1987	135,000	812,000
March 1988	135,000	947,000
September 1988	136,000	1,080,000
March 1989	137,000	1,220,000
September 1989	138,000	1,360,000
March 1990	139,000	1,630,000
September 1990	140,000	1,640,000
March 1991	141,000	1,780,000
September 1991	141,000	1,920,000
March 1992	142,000	2,060,000
September 1992	142,000	2,200,000
March 1993	143,000	2,350,000
September 1993	144,000	2,490,000



DELAWARE SOLID WASTE AUTHORITY
NORTHERN SOLID WASTE FACILITY - 2
PHASE II AREA
LANDFILL DESIGN REPORT
DECEMBER 1986

FIGURE: 2.1
PHASE I AREA

landfilling. The dredge spoils constitute a thick, reliable landfill liner, but their low permeability also causes the soils to be slow-draining and quite weak in resisting shearing forces.

Phase I development included a dewatering system designed to decrease the water content of the upper several feet of dredge spoil. Dewatering trenches were excavated and the water drained from the soils was collected and pumped from the site. The trenches were installed with 1.5 to 1 (horizontal to vertical) side slopes to depths of 10 to 15 feet on 220-foot centers. The trenches were completed in four months and were allowed to stand open for about 8 months. Due to the low permeability of the site soils "mounding" of the soil water between the dewatering trenches occurred and additional excavations were needed in some areas. The dewatering system caused the soil moisture to decrease and improved the workability of the soil appreciably. Upon completion of the dewatering program, trenches were refilled with dredge spoil material and the landfill bottom was graded uniformly to slope from south to north.

A leachate collection system was installed to capture water percolating through the landfill. The design incorporated gravel-filled "french-drain" type laterals for collection and delivery of leachate to 6-inch diameter, smooth-wall polyethylene headers installed in gravel-filled trenches. These header pipes conveyed leachate to 10-inch or 12-inch diameter mains at the perimeter which emptied into a sump in the northernmost corner of the site. From the sump leachate is pumped to a force main which conveys wastewater to the nearby City of Wilmington wastewater treatment plant for disposal.

Phase I was divided into four quadrants of roughly equal size to facilitate filling and access. The boundaries were established by two imaginary lines running southwest to northeast and northwest to southeast through the approximate center of the site. Ten foot lifts were placed first on the northwest and southwest quadrants, followed by a second lift in these quandrants. The northeast and southeast quadrants were then filled first with 10 feet of solid waste, and then with a second 10-foot lift. Two more 10-foot lifts were placed atop the first 20 feet, bringing the Phase I landfill to about 40 feet.

The site was operated as an "area-fill" with six inches of cover soil placed and compacted over the waste at the end of each day. Daily cover materials in use at the site were a Delaware Borrow Type F material or the humus product from DRP. Soil cover requirements were approximately 10% (on a weight basis) of the daily solid waste deliveries to the site or about 850 tons/week based on a minimum acceptable density of about 2500 lb/CY for Delaware Type F borrow. The density of compacted solid waste excluding daily cover was measured quarterly and was found to be about 1150 lb/CY.

During the early phases of landfilling on Phase I, before all of the leachate drains were covered with refuse, provisions were made to segregate stormwater runoff from leachate produced in the active quadrants. These measures were not completely effective, however, and flows conveyed by the leachate system were composed of both leachate and relatively clear rainwater. The approximate maximum flows of record for the site occurred over a nine day period from November 26, 1985 to December 5, 1985 when about one-fourth of the surface area of the site was covered with solid waste. During this period 134,000 gpd (93 gpm) was pumped off-site for treatment. After all leachate lines were covered by solid waste, flows were greatly reduced. Recent records show that an average flow of 5700-6400 gpd (4 to 4.5 gpm) can be expected. Under current conditions the maximum flow extrapolated from the records is 12,000 gpd (8.3 gpm). Estimates of the hydraulic head on the landfill liner made by site personnel during the last week of August, 1986 indicate from zero to 0.3 ft at various locations around the site. Percolation through the dredge spoil underlying Phase I would be inconsequential under this condition.

3.0 Phase II Geotechnical Investigation

3.1 Site Geology

The site is located in the Coastal Plain Physiographic Province, which is characterized by low lying and partially submerged landforms. The materials of the Coastal Plain consist of layers of unconsolidated gravels, sands, silts, and clays. Frequently, these materials are interbedded with interconnected lenses. It is reported that the thickness of the unconsolidated layers in Wilmington, at the contact between the Piedmont Province and Coastal Plains Province, is zero and in the southern portion of the county, it increases to 2,300 feet. At the Cherry Island site the thickness ranges from 95 feet in the northwest corner, to about 220 feet in the southeast corner.

As indicated, The lower unconsolidated layer of sediments overlying the rock is the Potomac Formation. This formation consists of multicolored silts and clays and interbeds of white, gray, and rust colored sands and some gravel. These granular interbeds are typically on the order of 5 to 10 feet thick. The thickness of the Potomac Formation varies from about 7 feet in the northwest corner to about 145 feet in the southeast corner of Cherry Island. The Columbia Formation is typically about 12 feet thick along the western edge of Cherry Island and 45 to 65 feet thick along the eastern edge. This formation generally consists of multicolored sands and gravels with interbeds of silty sand, silty clay, and clayey silt. Overlying the Columbia Formation are recent deposits and thick layers of dredge spoil. These materials are typically silty clays and clayey silts with some organic content and a layer of peat and clay. The overlying dredge spoil is of primary concern for geotechnical considerations for this project and is more fully defined below.

3.2 Subsurface Investigation

A subsurface investigation was conducted in July 1986, including nine conventional soil borings, two electric piezocone penetration tests, and four test pits. These borings are designated with the prefix GF in on Plate I, attached. The four test pits are designated GF-9 through GF-12 and are located east of the proposed dike location. Borings 4 and 5A are electric piezocone penetration tests. The remainder of the borings are conventional soil borings in which standard penetration tests were performed, and split-spoon samples and undisturbed samples were obtained. Records of the borings are appended to this report. Since the purpose of the borings was primarily to investigate the thickness and characteristics of the dredge spoils, the borings were generally terminated when the underlying granular layer was encountered.

Several of the soil borings encountered decomposition gases under pressure at the approximate depth of the underlying granular material upon which COE dredge spoil disposal was commenced. Gas deposits were assumed to exist due to anaerobic decomposition of marsh vegetation. These gas pockets propelled water and granular soil particles as high as 20 feet into the air through the hollow stem auger equipment. Analysis of the ambient air near the borehole indicated that the gases included methane.

Drilling equipment was shut down during these gas discharge events to avoid the danger of explosion. In one case the shutdown required 5 days; a more typical shutdown was 24 hours.

The purpose of the test pits was to obtain samples of the dredge spoil material for compaction tests and to determine the variation in natural water content with depth. The pits were excavated by backhoe to a maximum depth of 9 feet. It is of interest that the pit walls generally remained stable during the excavation. However, they did begin to slough, particularly when water-bearing, coarser grained lenses were encountered.

The results of the subsurface investigation are illustrated by three profiles, shown on Plates II and III, attached. As noted, previous borings performed by others are also included on the profiles. The profiles show the thickness and composition of the underlying materials. From Elevation 40± to Elevation -10± is a layer of dredge spoil material consisting of very weak, clayey silt with sandy lenses and varying amounts of organic material. This layer can be further divided into three layers by properties which reflect their age and environment. From Elevation 40± to Elevation 33±, the soil has an average undrained shear strength (Su) of 250 psf. It is believed that this layer is slightly stronger than the underlying layer, due to drying. The next layer extends from Elevation 33± to Elevation 13±, and exhibits an average Su of 150 psf. From Elevation 13± to Elevation -10±, the material exhibits an average Su of 325 psf. The shear strength testing indicates that the dredge spoil material below the depth of the upper layer where air drying had occurred exhibits, on the average, an increase in strength with depth. This observation was expected since normally consolidated clay deposits typically exhibit a strength proportional to the overburden pressure. Underlying the dredge spoil material is typically a layer of clay and peat which extends from about Elevation -10 to Elevation -15. This is underlain by sand and gravel. The overall thickness of dredge spoil varies from about 55 to 65 feet with the thickness generally increasing in a southeasterly direction.

Water level was monitored in all borings during the drilling program and a piezometer was installed in Boring GF-7. Generally, the water level in the boreholes was 2 to 5 feet below the top of the dredge spoils. This water does not constitute the regional groundwater table, nor does it indicate the presence of perched groundwater. It represents the water held by the saturated, very-poorly-drained dredge spoil material. This tightly-held interstitial water will not provide a pathway for landfill leachate to enter the underlying Columbia formation. To the contrary, the presence of this water attests to the excellent (that is, very low) permeability of the dredge spoil material, a quality that makes the NSWF-2 site very suitable as a sanitary landfill.

3.3 Laboratory Testing

Undisturbed, split-spoon, and bulk samples were collected and subjected to laboratory tests. The tests included classifications, hydrometer analysis, natural water content, Atterberg Limit tests, unconfined compression, triaxial test with pore pressure measurements, consolidation, permeability and compaction tests. A summary of the tests performed, and the results, is shown on Figure 3.1. The complete testing results are appended to this report. The test results show general agreement with tests performed by others. It is noted that the shear strength is somewhat lower than that used for the Phase I design. It is also noted that the strength of materials underlying existing dikes is also greater than that of soils within diked areas. This observation is to be expected due to the strength gain caused by consolidation and the compactive effort used to construct dikes.

Permeability testing of dredge spoil material indicates that the in situ permeability will be less than 1×10^{-7} cm/s. With as much as 50 vertical feet of this material lying between the Phase II area and the underlying aquifer formation, the site may be developed as a sanitary landfill without additional soil or synthetic membrane liner layers.

Boring No.	Sample No.	Sample Depth (ft)	Class	w_n (%)	w_l (%)	w_p (%)	I_p	% Passing No. 200	γ_d (pcf)	γ_m (pcf)	γ_{sat} (pcf)	G_s	Unconfined Compression q_u (ksf)	Triaxial Tests Total ϕ	c(ksf)	Effective ϕ	c(ksf)	Type Test	C_c	Consolidation C_v(inch^2/min) @ 0.5 tsf	e_o	Moisture γ_{dmax} (pcf)	Density Wopt (%)	k (cm/sec)
GF 1	S7	10-12	MH	69.1	62.4	46.3	16.1		58.8	99.4	90.8		0.58						0.67	$1.5X10^{-3}$	2.429			
	S10	20-22	MH	80.8	62.8	44.7	18.1		51.8	93.7	92.2		0.38						0.73	$3.6X10^{-3}$	2.271			
	S15, S16	35-39	ML	33.8	N/P	N/P	--	39.0				2.63												
	S19, S20	45-49	OH	109.0	89.1	N/P	--																	
GF 3A	S5, S6	6-9	MH	59.7	62.5	46.7	15.8																	
	S10, S11	17.5-21.5	MH	78.5	67.6	49.9	17.7																	
	S15, S16	30-34	ML	42.6	46.8	32.8	14.0																	
GF 5	S4, S5	6-10	MH	66.7	53.0	36.5	16.5	94.5				2.61												
	S9, S10	20-24	MH	85.6	62.1	44.2	17.9	100.0				2.40												
GF 6	S3	5-7	OH	103.6	70.8	43.1	27.8	97.5	47.3	96.3	91.9	2.66	0.61											
	S12	25-27	OH	85.4	73.1	46.7	24.4	100.0						17°	0.0	39.5	0.0	$\bar{R}$						
GF 7	Tube 1	10-12	OH	86.7	72.4	46.4	26.0	99.0	49.6	92.5	92.8	2.59							0.64	$3.4X10^{-3}$	2.261			$6.6X10^{-8}$
	ST-1	20-22	OH	91.3	76.5	47.9	28.6	100.0	53.6					17°	0.0	39.5	0.0	$\bar{R}$	0.72	$2.45X10^{-3}$	2.232			$1.1X10^{-3}$
	S20, S21	52.5-56.5	MH	78.7	68.2	51.4	16.8																	
GF 8	S4	5-7	OH	118.4	74.6	48.6	26.0	100.0	50.8	110.9	96.9		0.44						0.84	$3.3X10^{-3}$	2.828			
	S13	25-27	MH	89.6	65.9	42.4	23.5	100.0	51.7	98.0	94.0	2.56	0.32						0.54	$2.25X10^{-3}$	2.101			
GF 9	S-1	0-1.5		82.1																				
	S-2	7-8		96.7																				
GF 10	Bulk	0-2	OH	86.2	76.6	48.4	28.2	100.0														69.5	46.0	
	S-1	2-4		111.8																				
	S-2	6-8		108.2																				
GF 11	Bulk	0-2	OH	82.5	78.7	53.4	25.3	99.0														70.5	44.5	
	S-1	4-5		102.3																				
	S-2	7-8		95.6																				

FIGURE 3.1

SUMMARY OF SOIL TESTING RESULTS

4.0 Site Hydrogeology

4.1 Hydrogeologic Setting

This section of the report represents a review of conditions at the Cherry Island site, carried out to address comments of the Delaware Department of Natural Resources and Environmental Control (DNREC) regarding hydrogeologic interpretations prepared by the DSWA in 1984. These interpretations, presented in the Site Suitability Report - Hydrogeologic and Geotechnical Evaluation of the Cherry Island Site (Terraqua Resources Corporation, January 1984), established potential surfaces in the Columbia formation sediments which indicated groundwater groundwater would flow away from the Delaware and Christina Rivers at the southeast corner of the Cherry Island site (Plate IV).

4.1.1 Previous Investigation

In July and August 1983 a monitoring well installation program was conducted at the site in order to define existing hydrogeologic conditions in partial fulfillment of DNREC regulations. During this program 18 piezometers were installed into the unconsolidated coastal plain sediments which underlie dredge spoil materials emplaced by the Corps of Engineers. In addition to installation of two piezometers at each of nine locations, sediment samples were collected at 5-foot intervals to determine thicknesses and characteristics of the geologic units involved. Periodic measurements of piezometric elevations were carried out for the remainder of 1983. In July of 1985, quarterly monitoring of these same piezometers was commenced and continues to the present. Nine of the eighteen piezometers located in the Columbia/Recent deposits are of primary concern and are shown in on Plate IV, attached. The groundwater surface defined by piezometric elevations taken from the nine piezometers screened in the Potomac Formation, was interpreted to slope without deviation toward the Delaware and Christina Rivers. The Columbia potentiometric surface exhibited the anomalous behavior described above. DNREC expressed concern that differential settlement of piezometers may have occurred, changing the top of casing elevation. Such settlement would decrease the distance from casing top to water table, presenting the invalid impression that groundwater had risen.

DSWA conducted a survey of the Columbia formation piezometers in December 1985 and determined that the little settlement that had occurred would not explain the potentiometric surface anomalies.

4.1.2 Review of Hydrogeologic Information

The Columbia Formation of northern Delaware, as recorded by Jordan (1962), consists primarily of coarse sand, considerable admixture of gravel and cobbles with thin silty layers. Although the basal sediments which overlie the Cretaceous Potomac Formation resemble this description, most of the remaining succession of sediments are considerably finer grained and should be interpreted as recent river sedimentation. Geologic review by Duffield Associates as part of their quarterly monitoring effort, also support this conclusion. The resulting hydrogeologic framework is much refined and is illustrated in the stratigraphic cross sections of on Plates V and IV. Included in these cross sections are data from several geologic test borings and also stratigraphic locations of many of the piezometers installed during July and August of 1983. Because both Potomac and "Columbia" Formation piezometer pairs are located very close to each other they are shown as composite installations.

Interpretation of the post-Potomac Formation sediments as recent river deposits rather than Columbia Formation, explains the lithologic variations shown in the stratigraphic cross sections. If these sediments were truly Columbia Formation, they would be expected to be uniformly more coarse grained and contain less silt or silty clay. Instead they are composed of thick deposits of dark brown to gray silty clay and clayey silt with thinner intervening sand and gravel units. Sedimentation of this type is typical of meandering river systems where sediment type, channel position, and geometry are variable.

Cross Section C-C of on Plate VI illustrates the extent of the sand and gravel units which constitute the permeable zones within the river sediment aquifer. These sand and gravel layers can be divided into lower and upper units which are separated by an inherently less permeable silty unit. The upper sand unit is confined to the eastern area of the site where it forms a westward diminishing wedge. The lower sand unit is more extensive and is connected to underlying sandy sediments which may be true Columbia Formation. These sand units are shown on

Plates V and VI with stippled patterns for the sake of clarity and identification from one cross section to another.

The most important detail to be observed in the cross sections is the placement of piezometers in the recent river sediments. Cross Section C-C best illustrates the physical cause of the anomalous appearance of the potentiometric surface of the previous report. In this section three of four piezometers are placed in the lower sand unit and one (C-106) is placed in the upper sand unit. Adjoining Cross Section D-D shows two additional piezometers (C-102 and C-108) located in the upper sand unit. Because the upper sand unit is isolated from the lower sand unit by a thick sandy silt, the piezometers located in it should be considered as a separate network.

Additional evidence that Piezometers C-102, 106, and 108 should be treated as separate from other installations in the Recent/Columbia sediments if found in data from monitoring of several piezometers during tidal fluctuations on December 30, 1983. On that date two sets of paired piezometers and the Christina River were monitored through a 10-hour period to determine the influence of ocean tides on groundwater levels at the site. Tidal response curves and total tidal fluctuation for C-105, 106 and the Christina River are shown in Figure 4.1. The most obvious difference in these tidal responses is the very limited fluctuation of C-106 as compared to C-105 and the River. Not only is response in C-106 limited in magnitude but is also delayed. Tidal response of C-106 might be expected to be greater than C-105 because C-106 is installed nearer to the

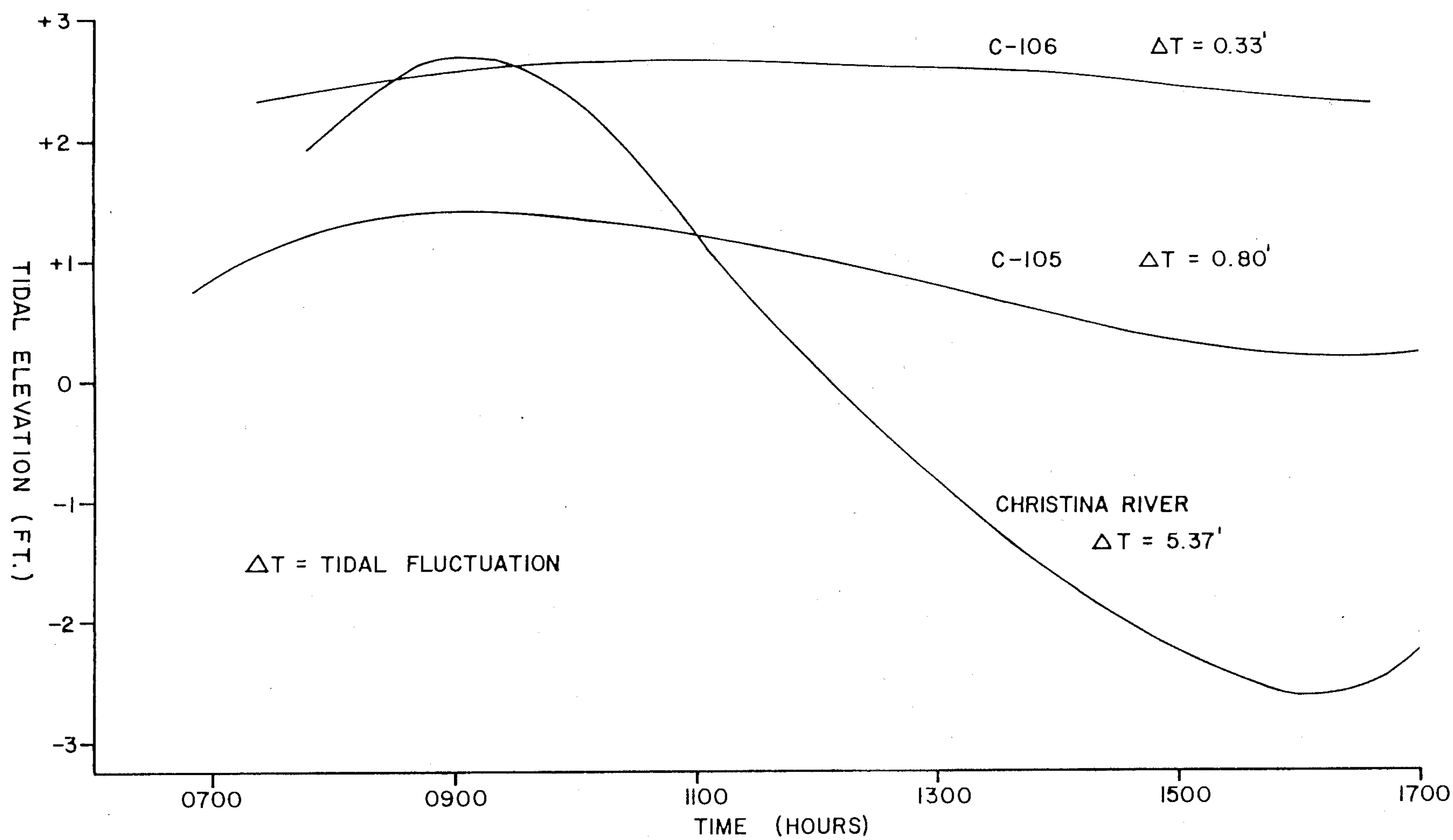


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FIGURE 4.1
GROUNDWATER RESPONSE TO
TO TIDAL FLUCTUATION
DECEMBER 30, 1983

elevations of the Christina River, increasing the likelihood of hydraulic interconnection between the river and upper sand zone. If the upper and and lower sand units were hydraulically interconnected, tidal responses of the two piezometers would be similar. Assuming C-105 and C-106 were functioning properly, it is concluded the units in which these two piezometers are screened are isolated from one another.

4.1.3 Conclusions Regarding Hydrogeologic Setting

The "Columbia formation" mapped by the 1984 Site Suitability Report included considerable quantities of recent river deposits and minor Columbia formation sediments. Within these sediments there are two distinct water bearing units consisting of sand and gravel. Separating the two water bearing units is a thick silt that acts to hydraulically isolate the upper sand and gravel from the lower.

Plate IV illustrates an alternate interpretation of the potentiometric surface observed in the recent and Columbia sediments. This interpretation does not include data from the three piezometers installed in the upper sand zone and is based on average piezometric elevations calculated for data collected between July and December 1983. These averages are based on the assumptions that the piezometeric levels were taken at random, representing all tidal positions in an attempt to filter out tidal positions and precession which were not taken into consideration during data collection. Although minor variations in the flow direction are shown in the figure, the important conclusion is that a relatively flat potentiometric surface exists beneath Cherry Island and that the gradient sharply increases to the west of the site. This is particularly important since the Columbia Formation is more clearly defined along the western edge of the site where the gradient is prominently toward the Delaware and Christina Rivers.

4.2 Prevailing Groundwater Quality

DSWA installed a number of monitoring wells preparatory to commencing landfilling operations on the Phase I site. Water quality data for the Potomac formation, the Columbia formation, and the recent dredge spoil sediments are illustrated on Table 4.1.

TABLE 4.1
Cherry Island Site
Water Quality Data
Mean Concentrations

Constituent	Units	Delaware Limit	Recent and Dredge		Columbia Formation		Potomac Formation	
			Range	Mean	Range	Mean	Range	Mean
Alkalinity, total	mg/l	----	275 - 620	433	95 - 720	391	60 - 98	77
Arsenic	mg/l	0.05	.01 - .05	.027	.01 - 1.4	.21	----	<.002
Chloride	mg/l	250	230 - 520	370	320 - 580	449	120 - 340	243
Chromium, total	mg/l	0.05	<.01 - .04	.022	<.01 - .03	.013	----	<.01
Conductance	mg/l	----	1160 - 1960	1500	1000 - 3050	1609	480 - 980	817
Copper	mg/l	1.0	<.01 - .17	.11	<.01 - .05	.018	<.01 - .01	<.01
Hardness	mg/l	----	350 - 850	559	260 - 1350	579	110 - 385	193
Iron	mg/l	0.3	18.4 - 72	55	32.9 - 102	62.7	.72 - 31.6	8.5
Lead	mg/l	0.05	<.01 - .08	.035	<.01 - .06	.018	----	<.01
Manganese	mg/l	0.05	16 - 80	60	51 - 116	85.3	.77 - 54	18.7
Mercury	ug/l	2.0	----	<.2	----	<.2	----	<.2
Nickel	mg/l	----	----	<.05	----	<.05	----	<.05
Nitrate	mg/l	10	----	<1	----	<1	----	<1
Nitrogen, Ammonia	mg/l	----	.96 - 37	17.0	3.0 - 40.0	21.8	<.05 - 5.1	1.1
Nitrogen, Kjeldahl	mg/l	----	4.8 - 42	86.8	4.4 - 55	27.8	.34 - 4.6	1.1
pH	----	----	6.1 - 6.8	6.3	5.9 - 7.4	6.4	5.7 - 7.5	6.7
Selenium	mg/l	0.10	----	<.005	----	<.005	----	<.005
Sodium	mg/l	----	150 - 298	233	27.8 - 312	214	68 - 224	140
Sulfate	mg/l	250	13 - 430	151	3.0 - 710	109	14 - 240	72
Suspended Solids	mg/l	----	32 - 785	260	17 - 284	144	<5 - 23	8.4
Total Diss. Solis	mg/l	500	890 - 1780	1238	793 - 1750	1263	240 - 735	559
Total Solids	mg/l	----	965 - 1930	1501	886 - 2210	1407	240 - 750	607
Total Organic Carbon	mg/l	----	115 - 298	210	61 - 288	163	23 - 53	28.5
Zinc	mg/l	5	0.07 - 0.48	.22	<.01 - .28	.15	<.01 - .19	.07

From: "Site Suitability Report" submitted to DSWA, January, 1984.

5.0 Phase II Landfill Design

5.1 Site Configuration

The Phase II area constitutes about 43 acres situated as shown in Figure 5.1. An earthen dike constructed by DSWA along the eastern edge of the site will allow the area to be used for landfilling while the COE deposits dredge material to the east of Phase II. During the summer and fall of the 1986 the COE removed about two feet of dried dredge spoil from the Phase II area, using the excavated material to raise the dikes around the disposal area to the east of Phase II. The approximate configuration and topography of the site following COE activity are illustrated in Figure 5.2.

5.2 Landfill Bottom

The landfill bottom was designed to drain from the northern and southern portions of the site to a central leachate header pipe constructed across the site, as shown in Figure 5.3. The bottom was designed to achieve approximately equal cuts and fills; construction would entail cutting in the center portion of the area and pushing excavated material toward the northern and southern edges. Bottom slopes were designed to be fairly gentle. Substantial settlement of the landfill bottom is expected due to consolidation of subgrade soils. This settlement should be most pronounced in the center of the fill, due to heavier surcharging by deeper solid waste compared to areas closer to the edges of the landfill. The differential settlement will accentuate downward slopes toward the collection area, thereby enhancing

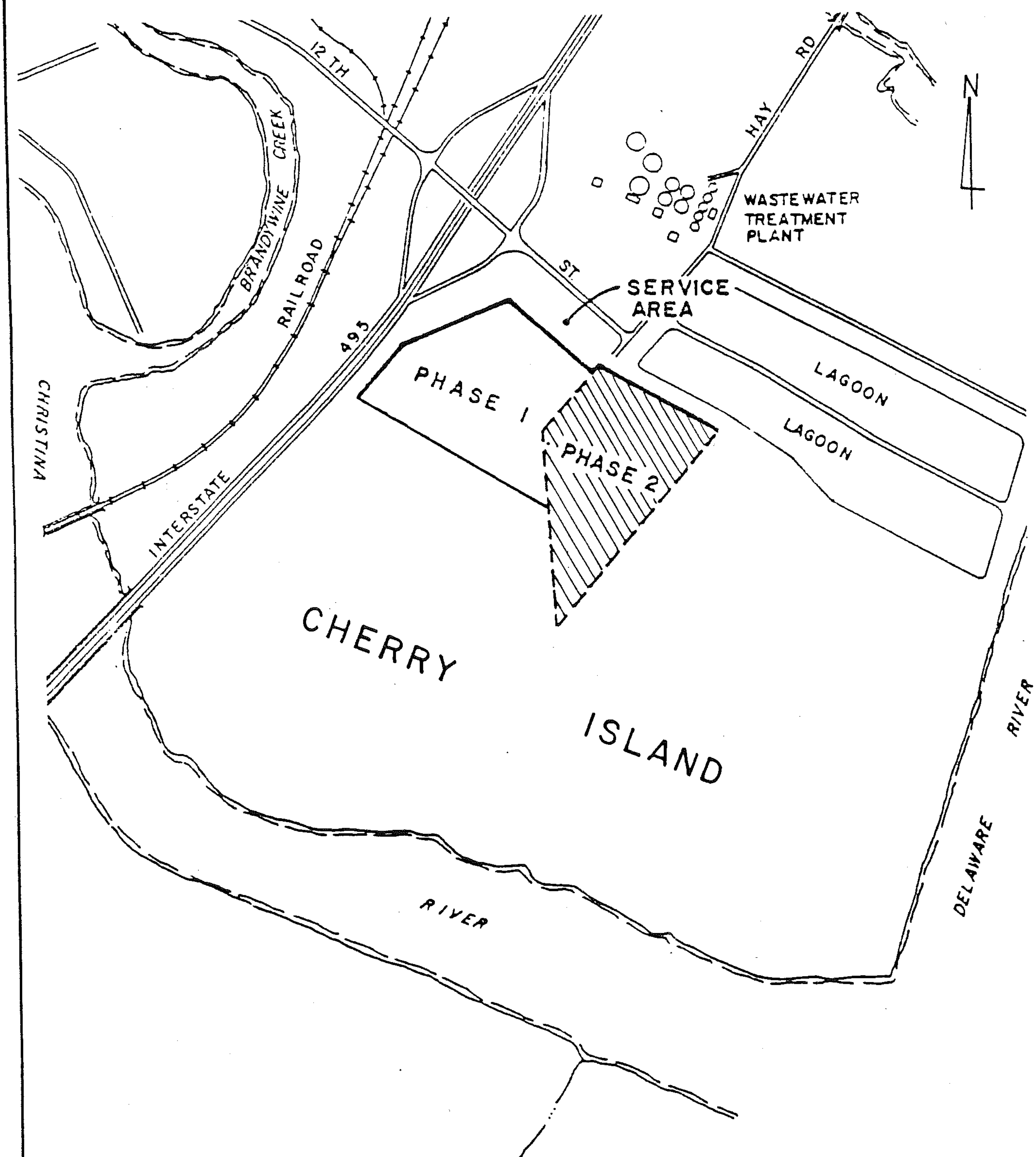

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FIGURE 5.1:
PHASE II AREA

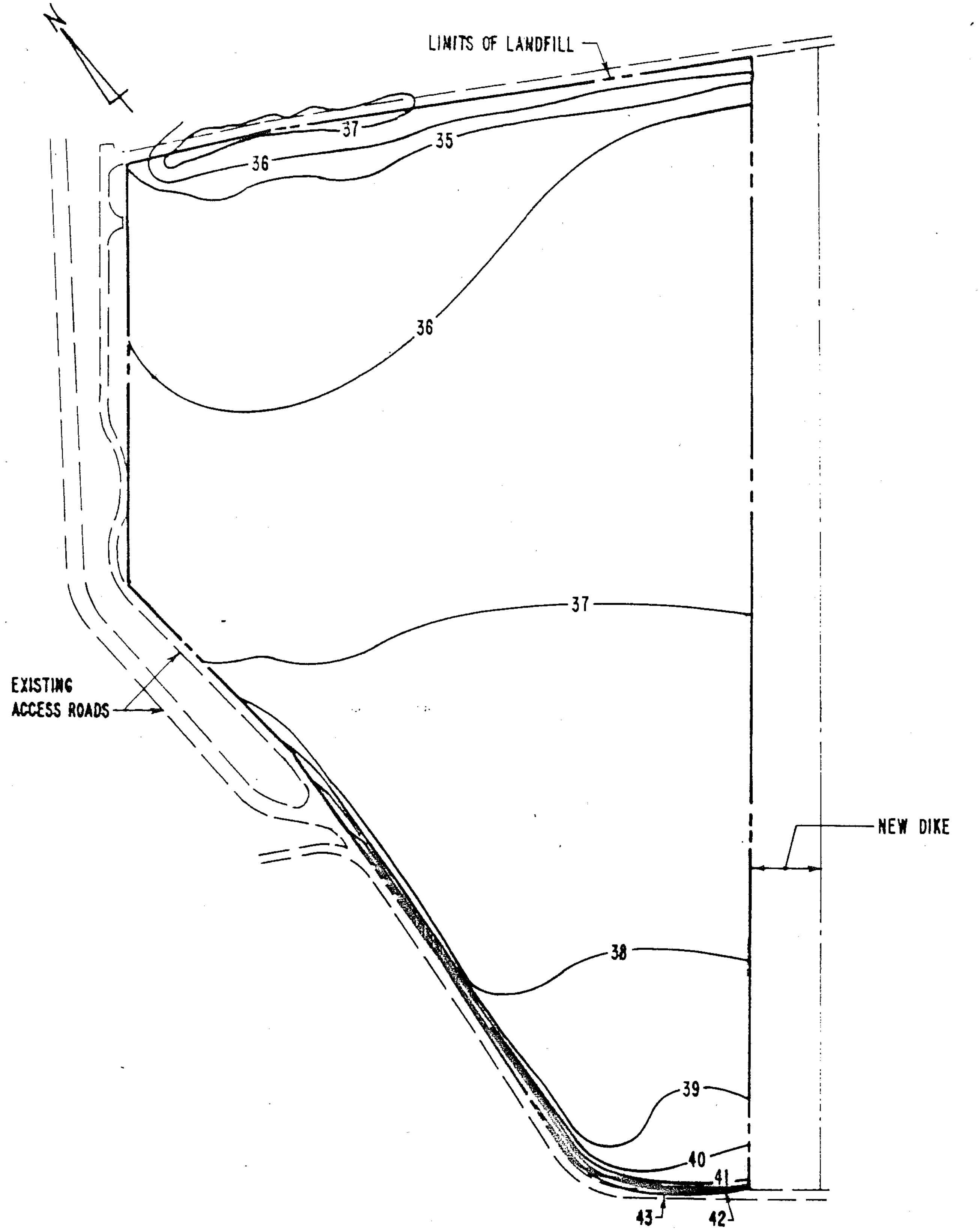

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FIGURE 5.2:
APPROXIMATE CONTOURS FOLLOWING C.O.E. EXCAVATIONS

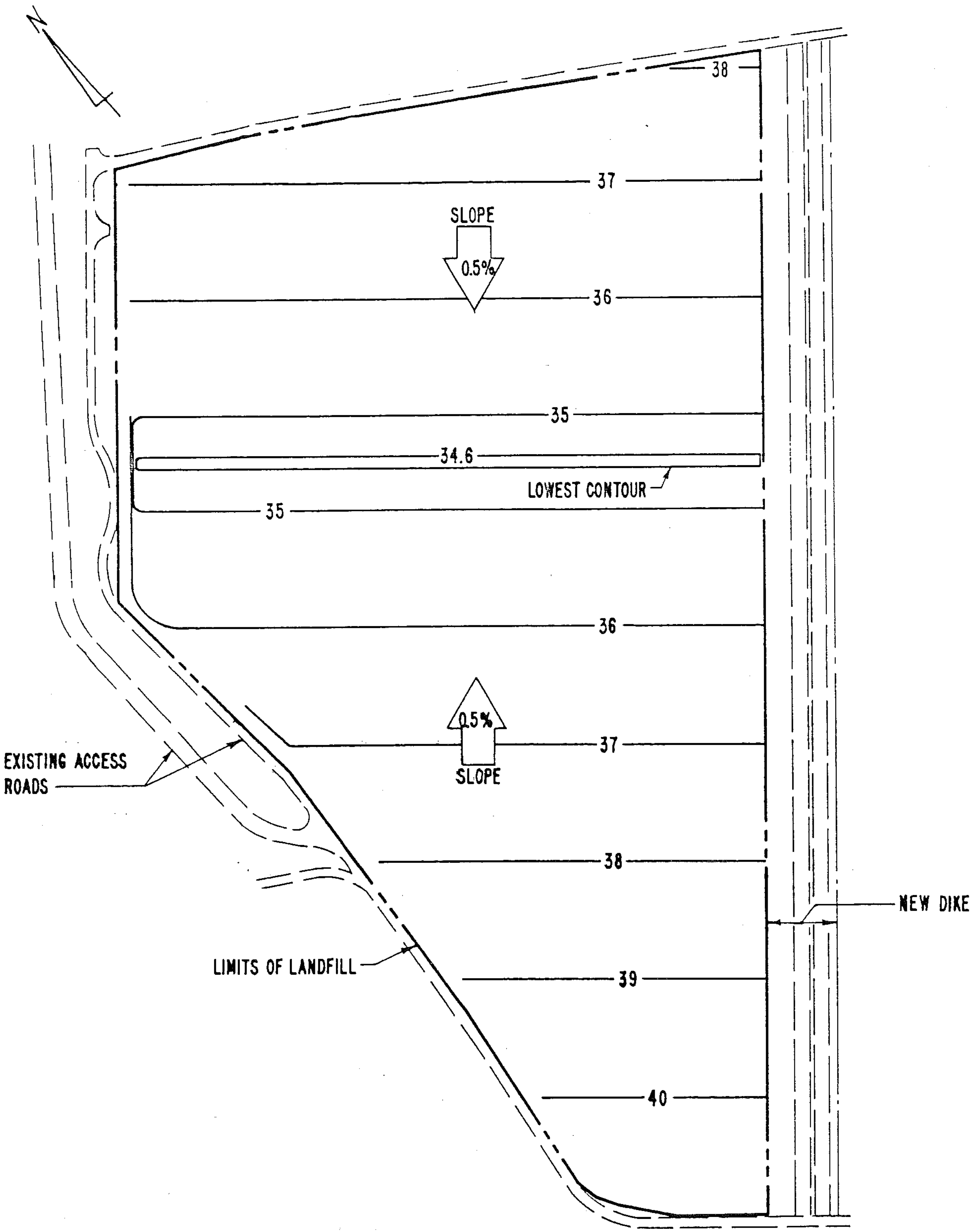

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FIGURE 5.3:
LANDFILL BOTTOM GRADING

conditions for collection and conveying leachate out of the fill.

The Phase II area was resurveyed after the COE concluded its excavation activities (December 1986). As expected, about two feet of material was removed from the site. Analysis of cuts and fills required to grade the landfill bottom confirmed that the bottom as depicted in Figure 5.3 would balance cuts and fills.

5.3 Dewatering Plan

Phase II area soils are very poorly drained. In the time since the final deposition of wet dredge spoil by COE, the upper several feet of spoil have dried to an extent. Below this dried layer, the spoil material is so wet that standing water forms when the spoil is excavated. This ponded water is neither a regional groundwater table nor perched groundwater. It is soil interstitial water that will drain by gravity if an outlet (for instance, the excavation) is provided. The rate of draining for the spoil material will be governed by overburden pressure, soil permeability and the length of flow paths travelled by moisture leaving the site. Increasing the overburden and shortening the flow paths will enhance dewatering. Computations carried out in the early stage of design indicated that, given the extremely low permeability of the dredge spoil, dewatering trenches would provide little enhancement of water flow more than a few feet beyond the edge of the trench. Trenches would have to be spaced closely, and dewatering would be very slow in any event. The analysis indicated that a dewatering system would not be cost-effective for the Phase II site.

5.4 Final Landfill Grading

A preliminary final landfill configuration as shown in Figure 5.4 was selected to provide a basis for landfill design. Stability analysis and stormwater management considerations during final design required certain adjustments to the preliminary configuration. Side slopes were established at 3:1 (horizontal:vertical) to match Phase I practice. Solid waste lifts will be about ten feet thick (nine and one half feet of waste and six inches of daily cover). For the purpose of the preliminary filling plan, an initial height of 80 feet above the landfill was established. Runoff from the finished top of the landfill was to be conveyed down the side slopes in drainage channels to the perimeter of the finished fill. Stormwater was to be conveyed to a drainageway on the north side of the landfill, which discharges to the Delaware River.

The volume of the preliminary fill configuration was calculated to be about 3,500,000 cubic yards.

5.4.1 Predicted Settlement

Calculations of settlement caused by loadings exerted by the preliminary landfill configuration were made, assuming that the underlying dredge material is normally consolidated. It was noted that the laboratory data indicate that some of the dredge may be underconsolidated. This condition was indicated by natural water contents in excess of the material liquid limit and by the Liquidity Index (LI) which is generally in excess of 1.0. Actual settlements may be somewhat in excess of those predicted. It is emphasized that settlement estimates

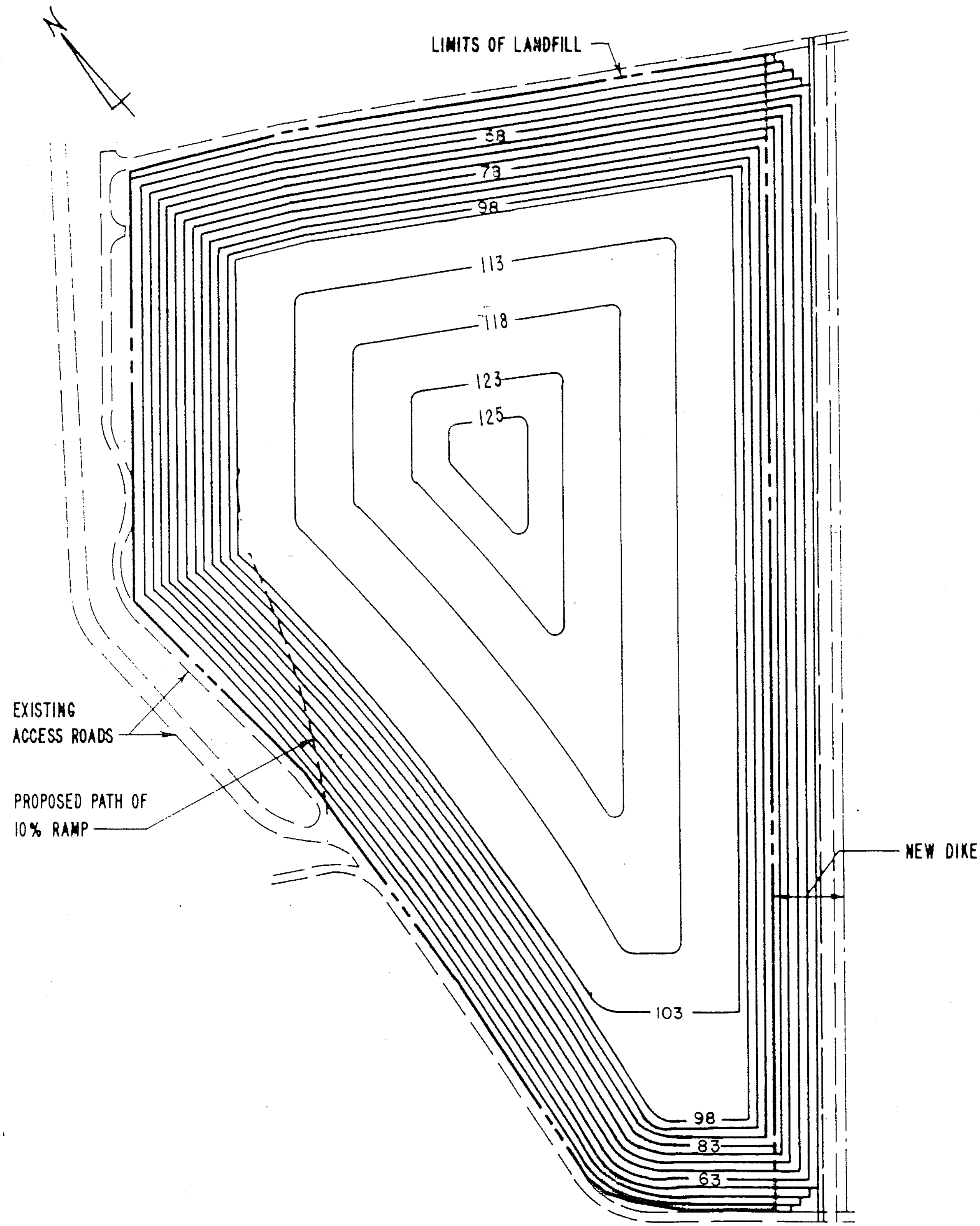

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FIGURE: 5.4
FINAL GRADING PLAN

are difficult to make with soils such as dredge material, particularly because of the variable organic contents. The estimated settlement due to the landfill is 13 feet; it is predicted to occur in about 30 years (99 percent of the primary consolidation). In addition, it is expected another 2 feet of secondary consolidation will occur within 25 years following primary consolidation.

It is noted that the thickness of the compressible layers underlying the site vary by about 10 feet. This variation will cause minimal differential settlement. Variations in soil properties across the site will probably be more significant contributors to differential settlement. Such differential settlement may be on the order of 2 feet from one point to another across the site.

5.4.2 Landfill Stability

Stability analyses focused on existing dike stability and general stability over the site with regard to landfilling sequence and maximum allowable thickness of landfill materials. Of particular concern was the stability of the dike along the northern boundary since large lagoons for the nearby treatment plant are immediately north of the site, and the stability of the western dike in the proximity of the existing southern dike of Phase I and the northern portion of the adjacent dredge disposal area.

Preliminary stability analysis was carried out before consolidation and triaxial testing was completed and was based on a forty-foot high final landfill. This analysis established certain parameters for landfill construction:

1. The toe of the fill would be set back 100 feet from the edges of the existing COE dikes to the north and west of Phase II.

2. The first forty feet of landfill height would be constructed in two 20-foot layers, with the second layer set back 20 feet on the northern and western sides.

Subsequent to these preliminary evaluations, soil testing was completed and the landfill concept was adjusted to provide for an 80-foot height. Stability of several cases was calculated. The cases and their calculated factors of safety are illustrated in Figures 5.5 through 5.7, which depict conditions considered likely in the western side of Phase II. Case I shown in Figure 5.5, would entail a landfill built with a uniform side slope to a height of 80 feet. This configuration is predicted to offer a factor of safety of about 0.95, indicating that a sliding failure would be likely. Case 2, illustrated by Figure 5.6, would involve solid waste filled to 40 feet, rather than 80 feet over the existing ground surface. This configuration is predicted to have a safety factor of 1.08, indicating that sliding failure would be less likely for Case 2 than for Case 1. Of course, Case 2 attains its greater stability at a cost of sacrificing nearly half the solid waste capacity of Case 1. Case 3, as shown in Figure 5.7, represents a compromise that, while not offering as much solid waste capacity as Case 1, would allow DSWA to fill to 80 feet. The setback volume could be filled eventually, after

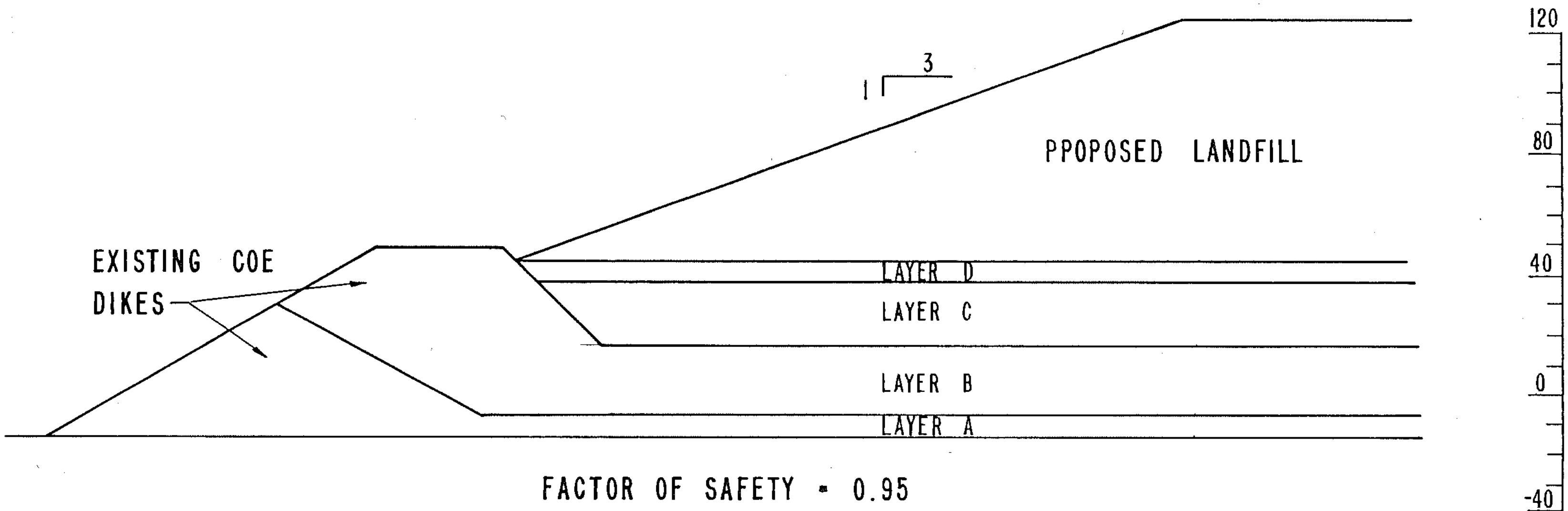

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FIGURE 5.5
SLOPE STABILITY ANALYSIS
CASE I

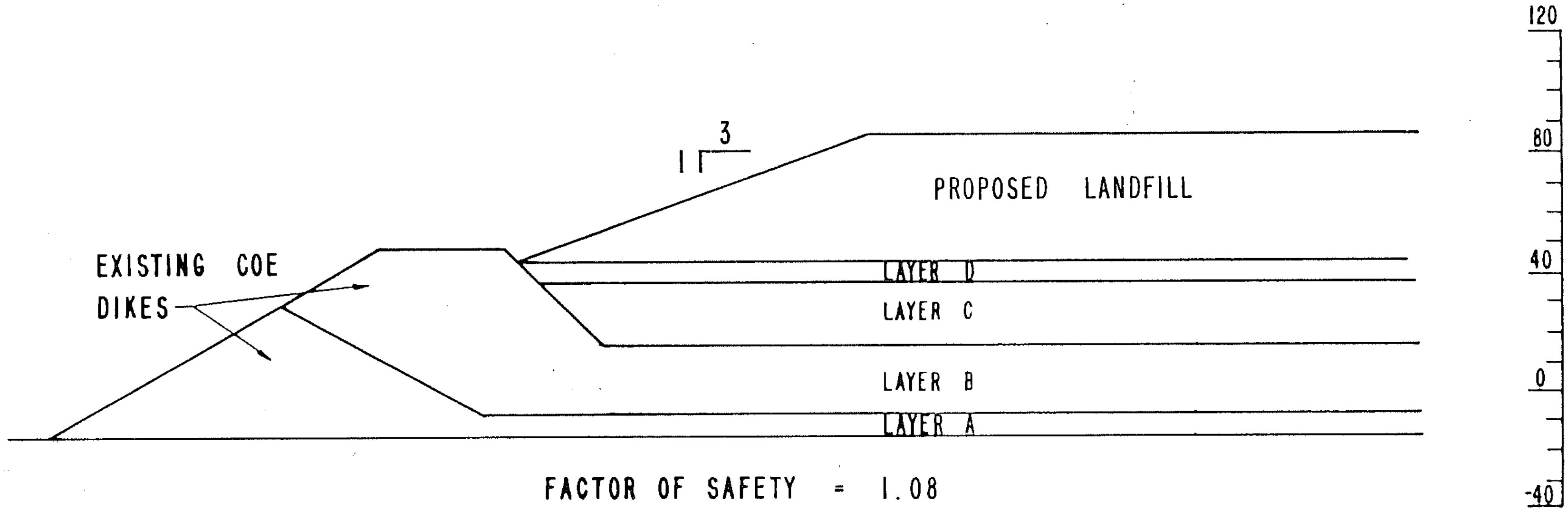

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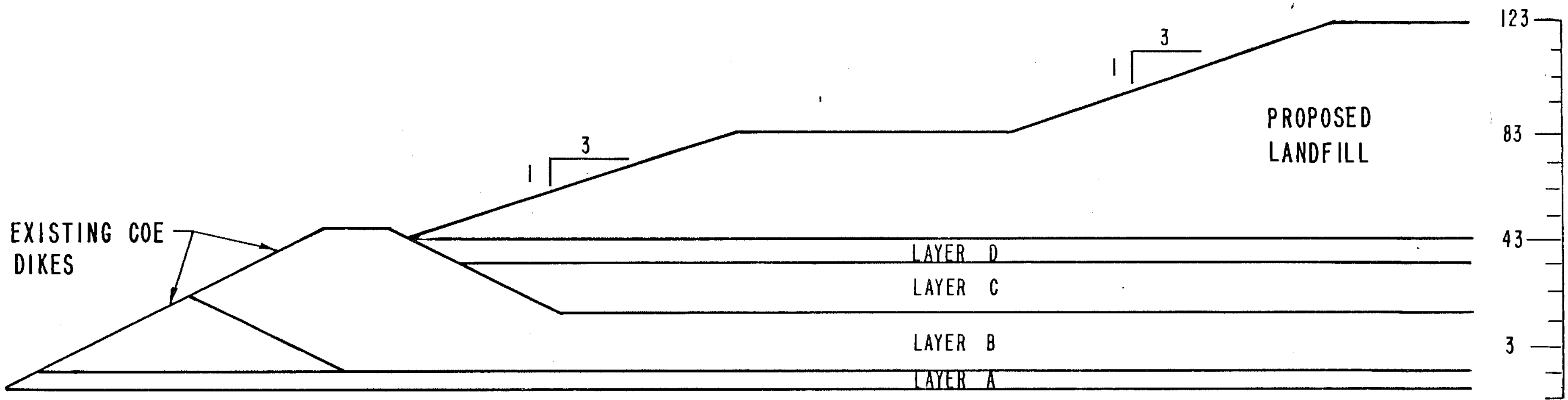
FIGURE 5.6
SLOPE STABILITY ANALYSIS
CASE 2



FACTORS OF SAFETY

40-FOOT SECTION = 1.09
80-FOOT SECTION = 1.1 TO 1.2

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FIGURE 5.7
SLOPE STABILITY ANALYSIS
CASE 3

site soils have consolidated adequately. Case 3 is predicted to have factors of safety of 1.09 and 1.2, respectively, for failure surfaces through the 40-foot portion and the 80-foot portion of the landfill.

On the basis of the stability analyses, Case 3 was selected for the interim landfill configuration along the northern and western sides of the landfill.

5.4.3 Interim Grading Plan

An interim grading plan incorporating the selected side slope configuration for the northern and eastern sides of the landfill is shown in Figure 5.8. Total landfill capacity for the interim situation was computed to be about 500,000 cubic yards less than the capacity of the final grading plan (Figure 5.4).

5.4.4. Geotechnical Reinforcing

The stability analysis described earlier in this section was carried out using several assumptions:

1. The dredge spoil materials from elevation about -14 to about +43 were divided into four strata, with average shear strength of these strata as shown in Figures 5.5 through 5.7.
2. Shear strength of landfill materials was assumed to be 750 pounds per square foot (psf), based on limited values reported in the technical literature.
3. Averaging of soil properties in a specific stratum or structure (such as the existing dikes) entails the

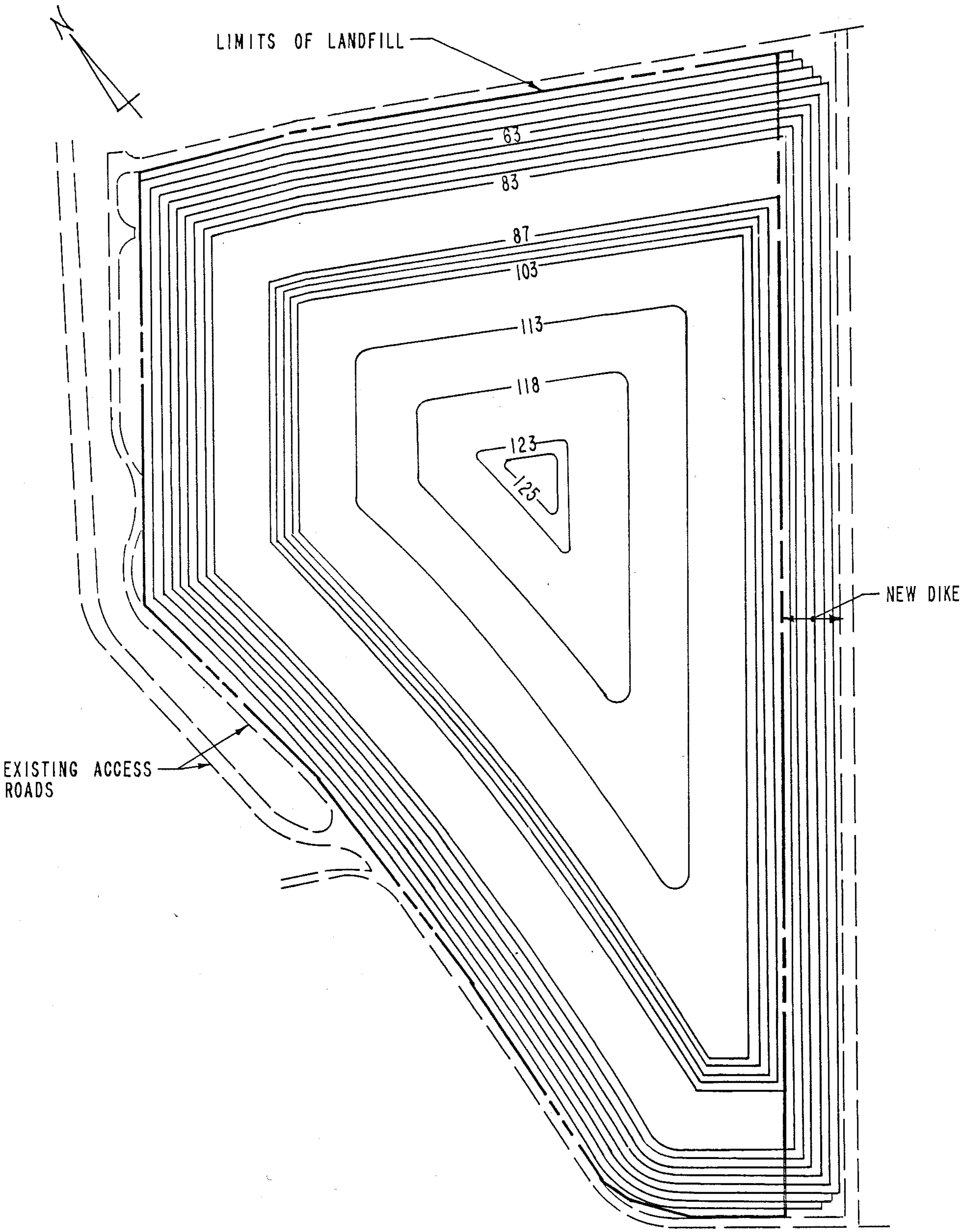

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FIGURE 5.8:
INTERIM GRADING PLAN

assumption that areas of poorer-than-average properties will be limited in extent to the point that the average safety factor accurately predicts overall slope stability.

These assumptions were made of necessity, to allow stability analysis to proceed. In an area of uniform soils, one could be comfortable with this "averaged" approach. The Cherry Island soils are not uniform. Strength of stratified materials depends on time in place, overburden pressure, and time of exposure to the air. Soil characteristics within a stratum vary widely due to different flow velocities at the time dredge spoil materials were placed in different parts of the Cherry Island site. These varying velocities would "sort" deposits by particle size, with larger particles being deposited in a high velocity area and finer particles in a low velocity area.

If failures occur along landfill slopes, they will probably be located in areas where soil characteristics vary substantially from the average. Thus, factor of safety calculated from average conditions must be considered with caution. Soils engineers generally seek safety factors of 1.3 or greater, reflecting their understanding of the effect of localized departures of soil characteristics from the average. Such a factor of safety at NSWF-2, Phase II could be obtained only by limiting landfill height to an extent that would cause the site to be impractical for solid waste disposal. The Phase II soils are so weak that the best situation that would allow reasonable solid waste capacity exhibits a safety factor of less than 1.1.

The marginal factor of safety of the selected slope configuration would be improved by placing a geotechnical reinforcing material on the landfill subgrade in a location that would span the predicted failure zone, as illustrated in Figure 5.9. The tensile strength of the reinforcing material will add to the resistance caused by soil shear strength along the failure surface. A geotechnical grid with tensile strength of 6,000 pounds per foot (Tensar SR3 or Signode TNX 5001) was selected to be placed around the northern and western edges of the landfill. Safety factor for the 40-foot embankment was predicted to improve from less than 1.1 to nearly 1.2. While this measure does not achieve the preferred 1.3 objective, it does provide marked improvement to slope stability, and represents a properly cautious approach to the difficult soils of Phase II.

5.5 Estimated Landfill Capacity and Life

Capacity of the final-graded landfill on the Phase II site (Figure 5.4) was calculated to be about 3.5 million cubic yards. The interim grading plan was calculated to provide 3 million cubic yards of capacity.

Following start-up of the EGF, solid waste deliveries to the landfill are predicted to be:

Waste Description	Weight delivered	Compacted density
Waste directly to NSWF-2	171,000 tons/yr	1150 lb/CY
DRP residues	20,000 tons/yr	1150 lb/CY
EGF residue	80,000 tons/yr	2000 lb/CY

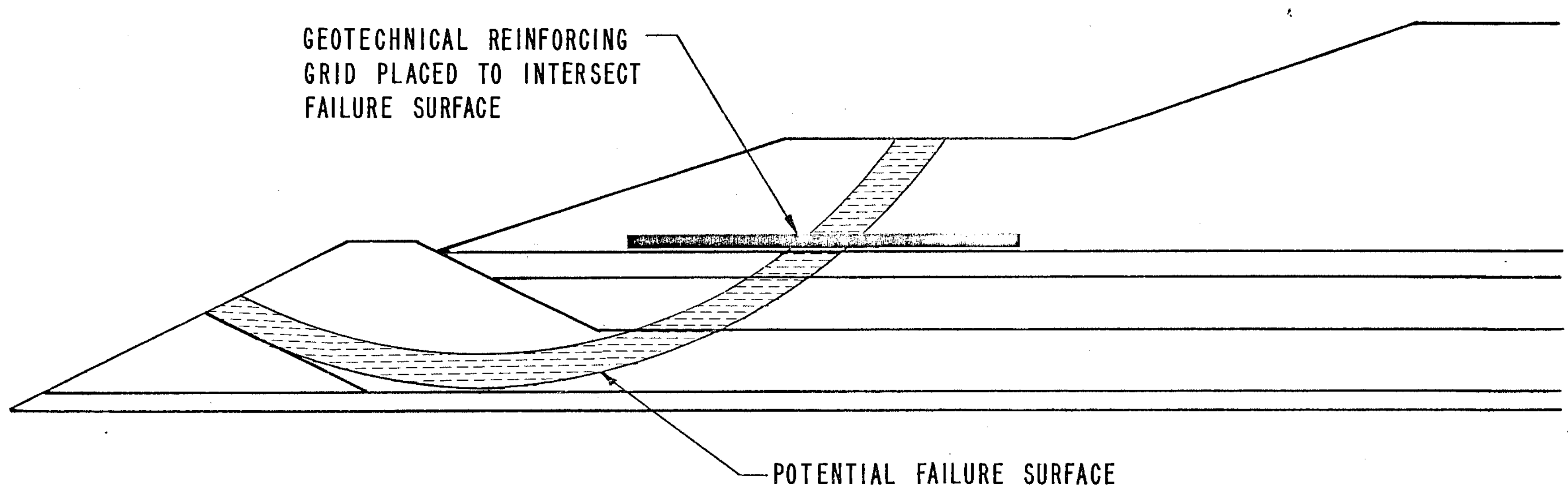

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FIGURE 5.9
GEOTECHNICAL REINFORCING

Aggregate in-place density of the material delivered to the landfill is predicted to be about 1,300 pounds per cubic yards. If DSWA continues its practice of providing daily cover soil in the proportion of 7 percent by volume, solid waste and associated cover will occupy about 8,400 cubic yards weekly. Phase II is predicted to provide about 357 weeks (6.9 years) of landfill life to interim grades.

DSWA may decide to use EGF residue as daily cover. If this practice were acceptable, as much as 500 cubic yards per week of cover soil could be eliminated. This measure would provide additional capacity for solid waste, extending Phase II life an estimated 23 weeks, as well as decreasing the cost of imported cover soil.

5.6 Leachate Management

5.6.1 Leachate Generation

The potential quantity of leachate that may be generated at the NSWF-2 is directly related to the quantity of infiltration into the fill. This quantity was calculated using the "Water Balance Method" as recommended by the U.S. Environmental Protection Agency in Use of the Water Balance Method for Predicting Leachate from Solid Waste Disposal Sites (530/SW-168, Oct. 1975). The computations appended to this report apply to completed and stabilized landfills. Infiltration experienced during operation of the fill will probably be higher.

Long range average monthly values for precipitation and temperature were obtained from Climatological Data, and were based on data collected at Porter Reservoir near the site.

Runoff coefficients of 0.13 for summer months when vegetation is most dense and tends to reduce runoff and 0.17 for winter months when vegetation has less effect were used for the calculations. In the Water Balance Method three fates are considered for the precipitation which does not run off. This moisture, termed infiltration (I), is either returned to the atmosphere through evapotranspiration (AET), retained in the soil pores of the final cover as storage (ST), or percolates (P) through the fill when soil pores are saturated. In the analysis a clay loam soil was chosen for the final cover, both to be conservative and since this soil type most closely resembles the dredge spoil which may ultimately be used for this purpose. From the tables in Instructions and Tables for Computer Potential Evapotranspiration and the Water Balance, by C.W. Thornthwaite and J.R. Mather (Drexel Institute of Technology, Laboratory of Climatology, "Publications in Climatology, Vol. X, No. 3", 3rd Printing, Centerton, New Jersey, 1957) the available soil moisture is 250 mm/m and for the two foot (0.6 m) final cover the resultant storage (ST) is 150 mm. From an empirical relationship given in the reference a value for potential evapotranspiration (PET) is computed and adjusted for the site location. In months when infiltration (I) is greater than potential evapotranspiration (PET) any water not used to replenish depleted storage percolates (P) and AET=PET. When I is less then PET storage is depleted and AET=I+ ST. As shown in the appended calculation, annual average leachate flow from the completed landfill is predicted to be about 19.5 gallons per minute, or 28,000 gallons per day. Actual flows from the completed fill will be influenced by precipitation, generally lagging rainfall events by several days due to the dampening effect of the solid waste.

5.6.2 Leachate Collection System

The leachate collection system was designed to function in a manner similar to the Phase I system. As illustrated by Figure 5.10, leachate will be collected in a series of perforated pipes placed in gravel-filled trenches. The design objective was to maintain little ponding of leachate over the dredge spoil liner, thus controlling migration of leachate into the liner. Measurements by DSWA of hydraulic head over the landfill subgrade in the Phase I area attest to the design approach; a maximum of only several inches of leachate has been observed.

Leachate laterals will consist of 6-inch diameter perforated, corrugated polyethylene pipe placed in coarse aggregate bedding. Leachate lateral trenches will be lined with geotextile fabric to avoid clogging of the leachate conveyance system and to maintain the structural integrity of the coarse aggregate envelope. This feature is important to avoid crushing the polyethylene pipe, since the envelope is intended to carry the weight of overburden and transfer this weight to the subgrade. Laterals will discharge into leachate headers, 8-inch diameter perforated, corrugated polyethylene pipe, installed in trenches in a manner similar to the lateral pipes. Interconnection between laterals and headers will be achieved by terminating lateral pipes where they enter the gravel-filled header trench.

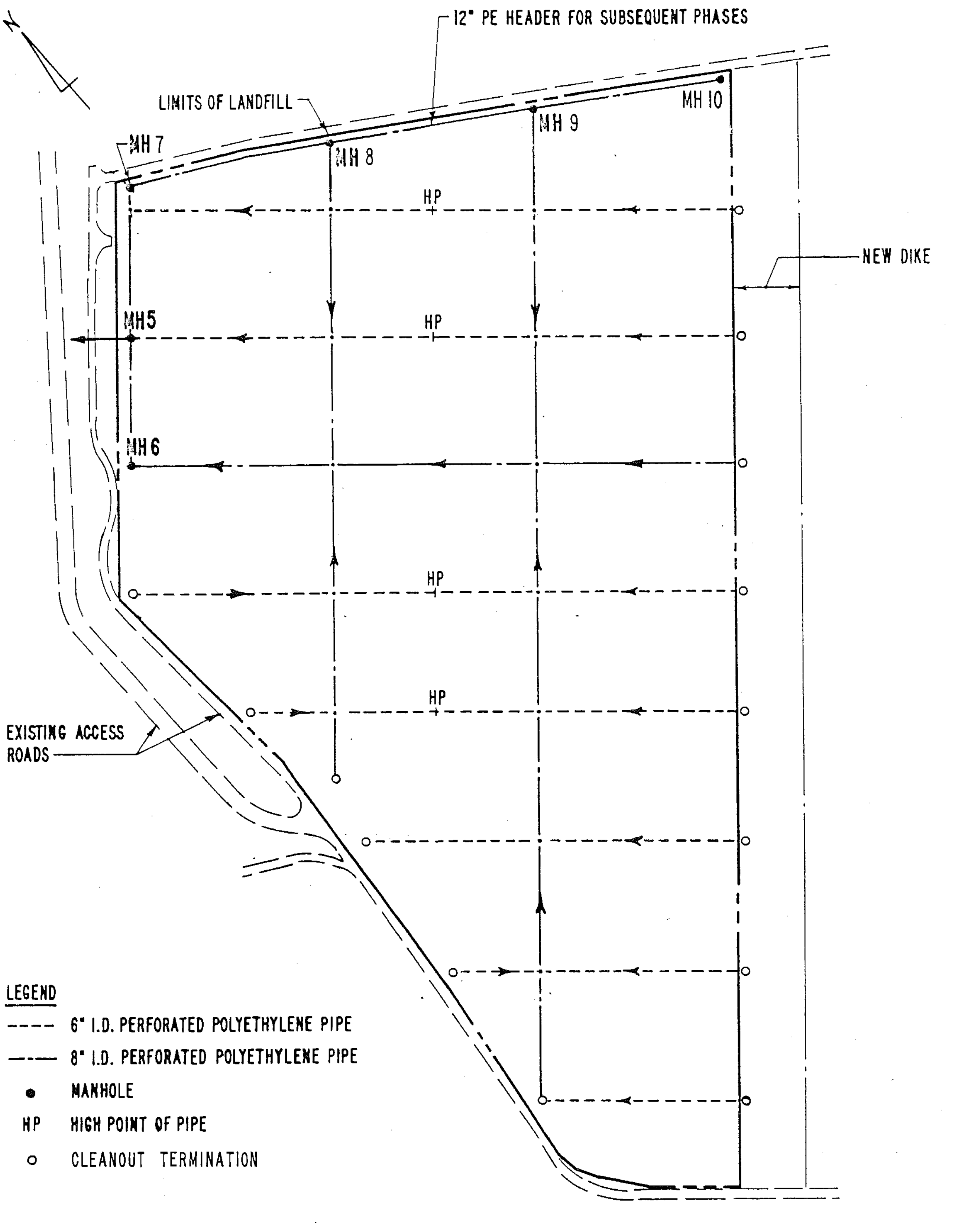

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FIGURE: 5.10
LEACHATE COLLECTION SYSTEM

Leachate from the laterals will flow through the gravel into the headers through the header pipe perforations. This approach will maintain drainage system integrity as the subgrade settles.

Laterals and headers will be terminated near the edges of the landfill in riser pipes fitted with plugs, to allow access for inspection and cleaning. A 12-inch diameter non-perforated corrugated polyethylene sewer will be installed along the northern edge of the site to provide for leachate flows from future phases developed at the site. Phase II leachate will be conveyed to the Phase I leachate system, and thence to the existing leachate pumping station.

The main header which will convey collected leachate out of the fill will be constructed to overcome the effect of subgrade settlement. It will be inclined fairly steeply from the center of the fill toward the western edge. As settlement occurs, the gradient of this pipe will decrease, but it will continue to convey flow out of the fill.

5.7 Landfill Instrumentation

Instrument clusters will be located along the landfill perimeter at locations shown in Figure 5.11. Each cluster will include an inclinometer, a piezometer, and a gas monitoring well. The inclinometer will be used to monitor movement of soils that would indicate possible impending sliding failure of landfill slopes. The piezometer will indicate changes in soil pore water pressure; abrupt changes may signal imminent failure conditions. The gas monitoring well will be used to detect the migration decomposition gases.

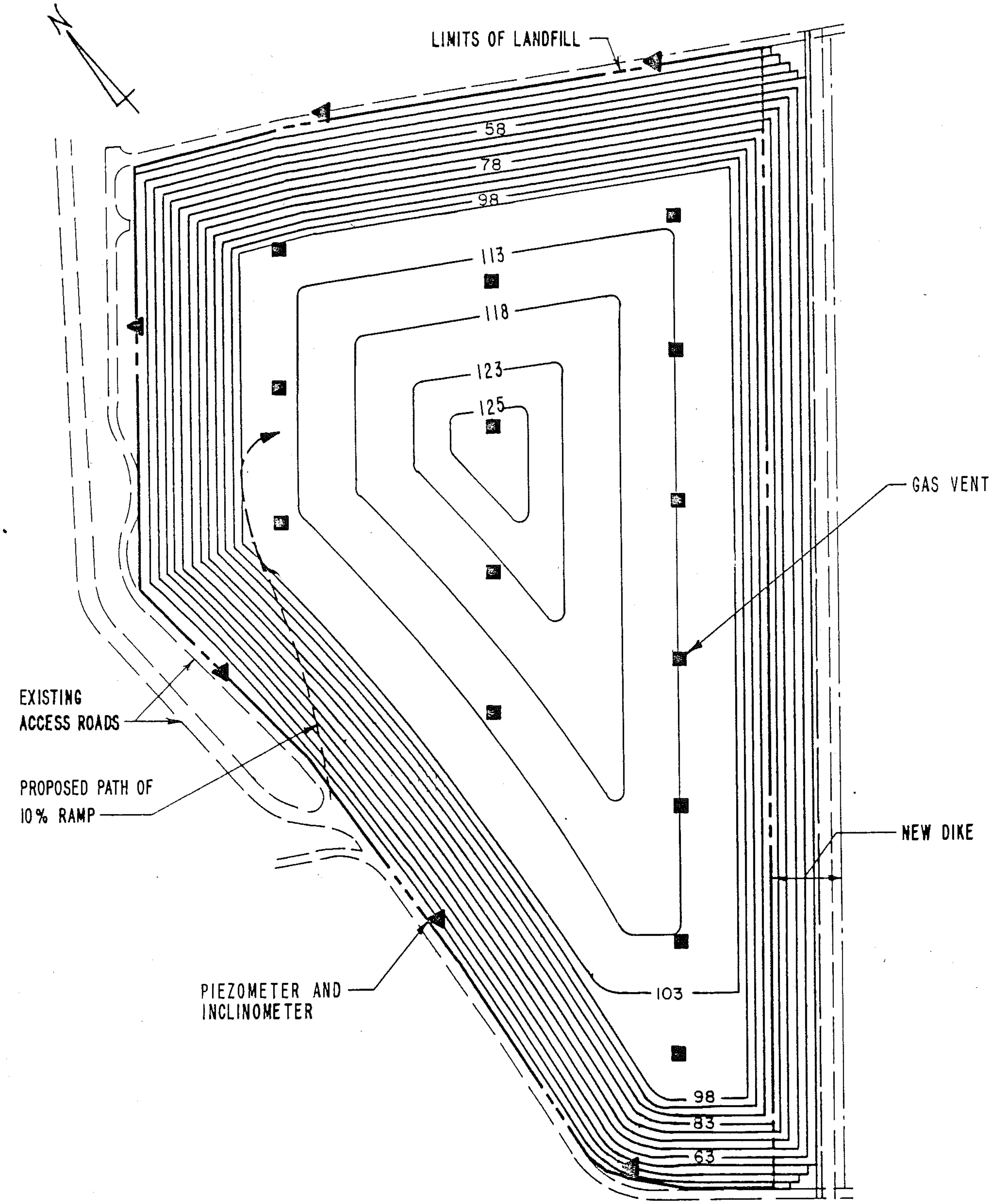

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FIGURE: 5.11
LANDFILL INSTRUMENTATION

Settlement plates will be installed at locations shown in Figure 5.11. The vertical rods connected to these plates will be extended by the landfill operations contractor and periodic elevation readings will be taken as the fill is developed and after it is completed.

Decomposition gas vents will be installed at the locations shown in Figure 5.11. These vents will consist of stone columns extended upward as the fill is developed. A plastic pipe will be installed in each stone column, to serve as a piezometer inside the landfill. These piezometers will be used to measure leachate depth over the liner during and following landfill development. The plastic pipe may be vulnerable to deformation and crushing by forces exerted by shifting and consolidating landfill contents.

6.0 Phase II Landfill Operation

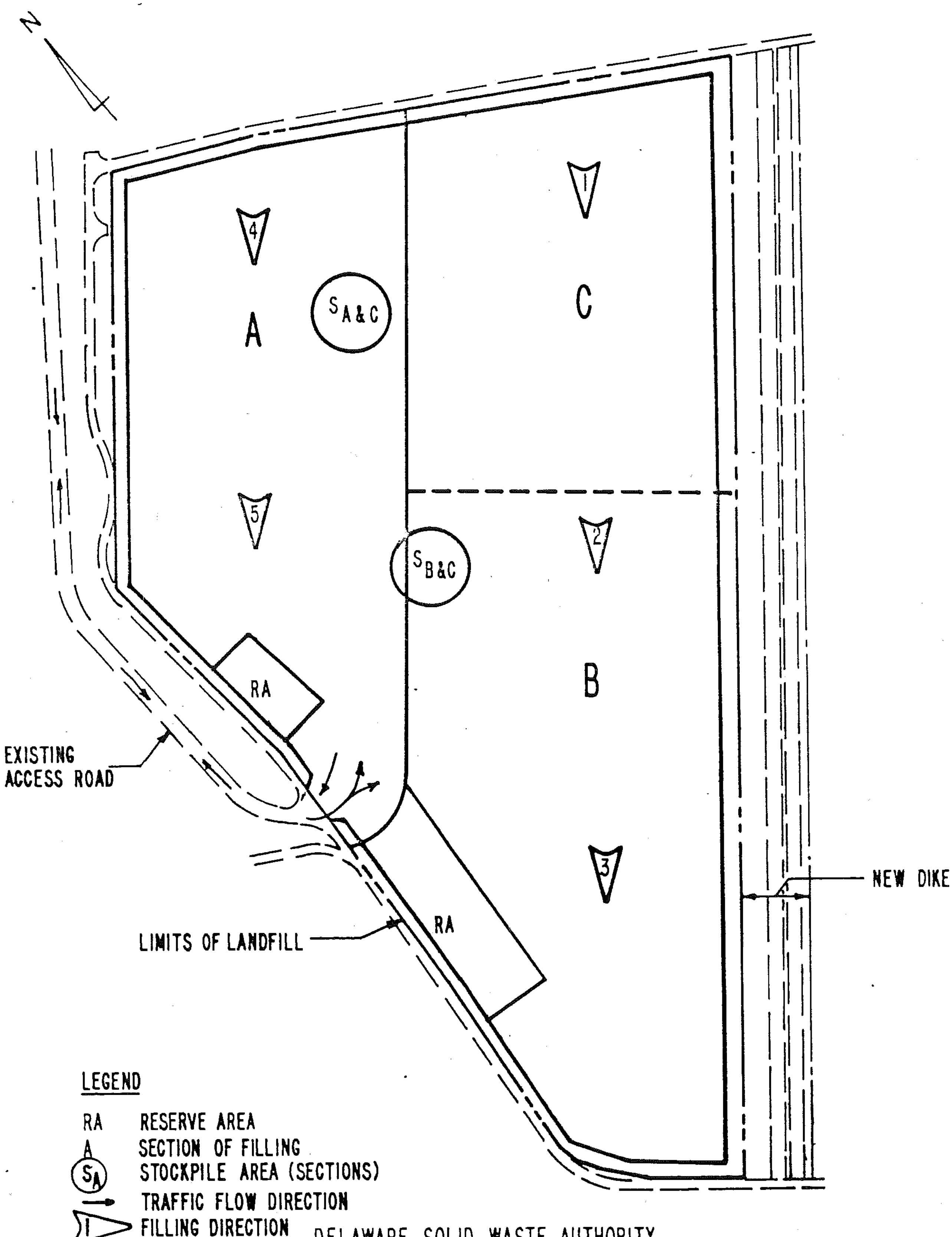
6.1 Sequence of Landfilling

The phase II area will be divided into three sections, designated A, B, and C in Figure 6.1. The landfill bottom and leachate collection system will be prepared for Section A and the first 10-foot lift of solid waste will be placed on this portion of the fill. Section A will be segregated from B and C by an earthen berm, which will divert stormwater on Sections B and C away from A. As the first lift on Section A nears completion, the leachate system in Section B will be connected. Section C will continue to drain to a stormwater collection trench and sump. While the first 10-foot lift is placed on Section B, the Section C leachate collection system will be connected. Finally, the first 10-foot lift will be placed on Section C.

The second 10-foot lift will start in Section C and proceed through B to A, as shown in Figure 6.2. Subsequent lifts will be placed in the same manner until a 40-foot depth is achieved. Final cover will be placed over the 100-foot setback and filling will continue, following the back-and-forth sequence described above.

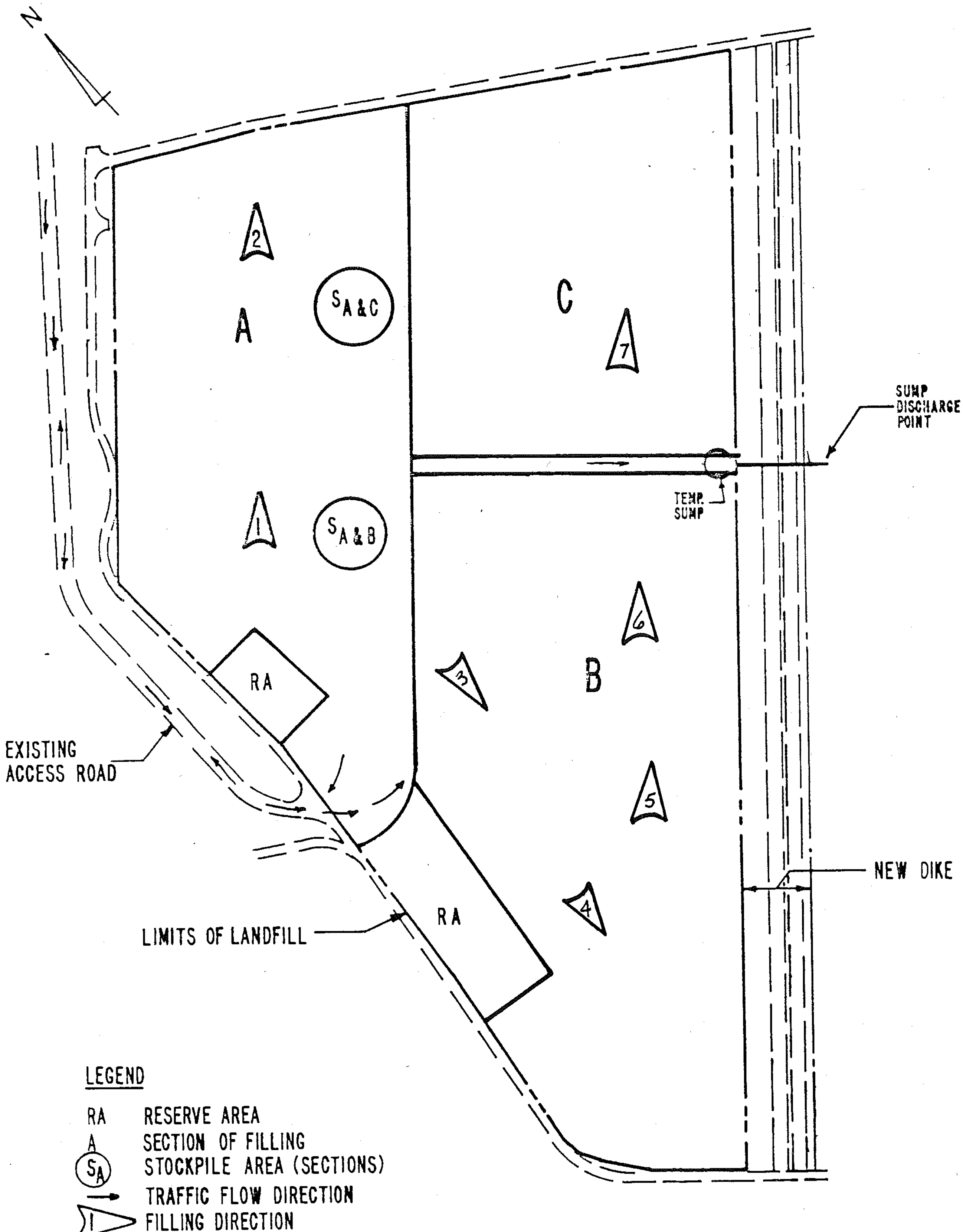
6.2 Stormwater Management

In the very early stages of landfill development, stormwater falling on Sections C and B will be pumped across the new dike, thus avoiding entry of uncontaminated water into the leachate system. After the first lift of solid waste is in place, any



DELAWARE SOLID WASTE AUTHORITY
NORTHERN SOLID WASTE FACILITY – 2
PHASE II AREA
LANDFILL DESIGN REPORT
DECEMBER 1986

FIGURE: 6.1
SEQUENCE OF OPERATION
FIRST LIFT



DELAWARE SOLID WASTE AUTHORITY
NORTHERN SOLID WASTE FACILITY – 2
PHASE II AREA
LANDFILL DESIGN REPORT
DECEMBER 1986

FIGURE: 6.2
SEQUENCE OF OPERATION
SECOND LIFT

precipitation falling inside the limits of filling will be handled by the leachate system.

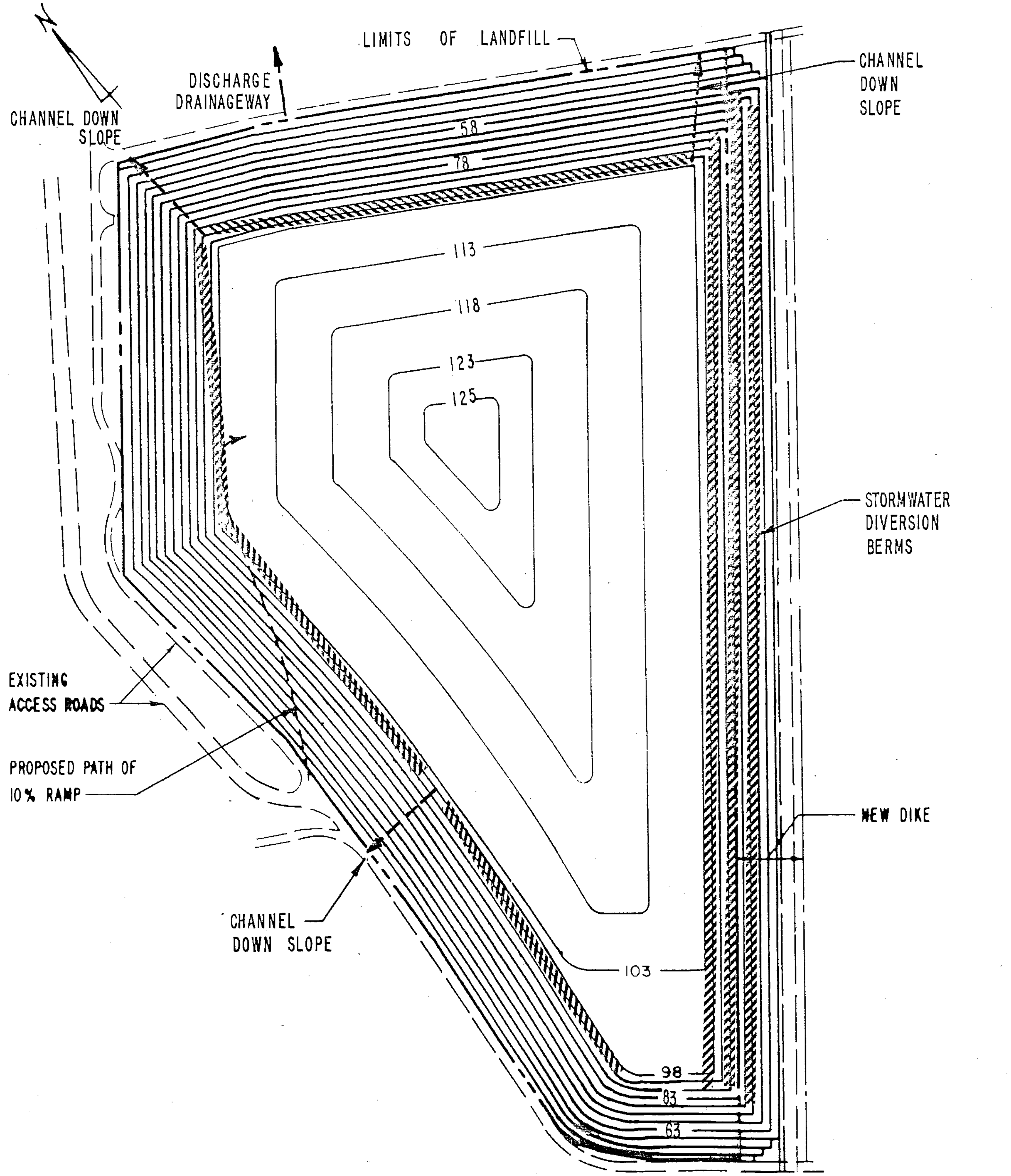
As the landfill is extended upward and finished slopes are established, precipitation on those slopes will run off to the base of the landfill. Drainage ditches along the western/northern side and along the point of contact of the landfill and the new dike on the eastern side will convey stormwater to the northern drainageway which discharges to the Delaware River. Runoff will be directed at the top of finished landfill slopes to protected channels to avoid eroding the landfill cover soil. Long term stormwater management facilities are illustrated in Figure 6.3.

6.3 Instrument Monitoring

Instrument clusters around the landfill and settlement plates and piezometers within the landfill should be monitored on a regular basis as filling progresses. The frequency of such monitoring should be established based on a predicted time span for changes. Monitoring event frequency may be adjusted with experience as filling progresses. Suggested initial frequency of monitoring is:

Perimeter piezometers	Monthly
Perimeter inclinometers	Monthly
Perimeter gas monitors	Every 3 months
Settlement plates	Every 3 months
Gas vent piezometers	Every 3 months

Obviously, perimeter piezometer and inclinometer measurements are considered most important for Phase II. Changes from one month to the next must be monitored to provide early warning of slope stability problems. This early warning could allow DSWA to



DELAWARE SOLID WASTE AUTHORITY
NORTHERN SOLID WASTE FACILITY – 2
PHASE II AREA
LANDFILL DESIGN REPORT
DECEMBER 1986

FIGURE 6.3
STORMWATER MANAGEMENT FACILITIES

modify filling procedures or configuration to avoid slope failures.

APPENDIX A

Records of Subsurface Investigation

Earth Engineering & Sciences, Inc.

BORING LOG

PROJECT Delaware Solid Waste Authority
Northern Solid Waste Facility - 2

BORING No. GF-1
PROJECT No. 86-123

LOCATION OF BORING Wilmington, Delaware

ELEV. ______ DATE: START 7-22-86 FINISH 7-23-86 INSPECTOR ______
HAMMER Wt. 140 HAMMER DROP 7" SPOON OD ______ FOREMAN ______
BORING METHOD HSA ROCK CORE DIA ______ MISC. ______

ELEV.	SOIL DESCRIPTION	DEPTH	BLOWS 6"	No.	TYPE	REC	REMARKS
	Brown moist silty clay w/ grass		1-1-1	1	DS	1.3	Water at 3.0' 24 hrs. 4.4' Backfilled
		3.0	WOH	2	DS	1.3	
	Brown moist silty clay	5	WOH	3	DS	1.0	
			WOH	4	DS	1.0	
			WOH	5	DS	0.3	
			WOH-1	6	DS	1.0	
		10	TUBE	1	PT	1.5	
		15					
		16.5	WOH-1-1	7	DS	1.5	
	Brown moist silty clay w/grass (org)		WOH-WOH-1	8	DS	1.5	
		20	TUBE	2	PT	2.0	
		25	WOH-1-1	9	DS	1.5	
			WOH	10	DS	1.5	
		30	WOH-WOH-1	11	DS	1.5	
		34.0	WOH-1-1	12	DS	1.5	
	Brown gray moist silty clayey sand	35	2-1-2	13	DS	1.5	
			7-6-5	14	DS	1.5	
		39.0					
		40					

LEGEND
DS DRIVEN SPOON
ST SHELBY TUBE
PS PISTON SAMPLE

GROUND WATER
AT COMPLETION ______ CAVED ______
AT ______ HRS. ______ CAVED ______

HSA HOLLOW STEM AUGER
DC DRIVEN CASING
MD MUD DRILLING

NC ROCK CORE

Earth Engineering & Sciences, Inc.

BORING LOG

PROJECT: Delaware Solid Waste Authority
Northern Solid Waste Facility - 2

BORING No. GF-1
PROJECT No. 86-123

LOCATION OF BORING: Wilmington, Delaware

ELEV. ______ DATE: START ______ FINISH ______ INSPECTOR ______
HAMMER Wt. 140 HAMMER DROP 7" SPOON OD ______ FOREMAN ______
BORING METHOD HSA ROCK CORE DIA ______ MISC. ______

ELEV.	SOIL DESCRIPTION	DEPTH	BLOWS 6"	No.	TYPE	REC	REMARKS
	Brown gray moist silty clay		1-2-2	15	DS	1.5	
			2-3-3	16	DS	1.5	
		44.0					
	Black moist silty clay w/grass (org)	45					
			2-2-4	17	DS	1.5	
			2-3-4	18	DS	1.5	
		50					
		51.5	1-2-3	19	DS	1.5	
	Brown moist silty clay						
		54.0	WOH-3-3	20	DS	1.4	
	Brown moist silty sand w/sand & gravel	55					
			2-2-3	21	DS	1.5	
		59.0	11-17-22	22	DS	1.5	
	Bottom of hole 59.0'	60					
		65					
		70					
		75					
		80					

LEGEND
DS DRIVEN SPOON
ST SHELBY TUBE
PS PISTON SAMPLE
RC Rock Core

GROUND WATER
AT COMPLETION ______ CAVED ______
AT ______ HRS. ______ CAVED ______

HSA HOLLOW STEM AUGER
DC DRIVEN CASING
MD MUD DRILLING

Earth Engineering & Sciences, Inc.

BORING LOG

PROJECT Delaware Solid Waste Authority
Northern Solid Waste Facility - 2

BORING No. GF-2 1 of 2
PROJECT No. 86-123

LOCATION OF BORING Wilmington, Delaware

ELEV. ______ DATE: START ______ FINISH ______ INSPECTOR ______
HAMMER Wt. 140 HAMMER DROP 7" SPOON OD ______ FOREMAN M. Ebert
BORING METHOD HSA ROCK CORE DIA ______ MISC. ______

ELEV.	SOIL DESCRIPTION	DEPTH	BLOWS 6"	No.	TYPE	REC	REMARKS
	Brown moist silty clay		2-5-5-5	1	DS	2.0	Water at 7.5' At completion 17.5' backfilled
			2-2-2-2	2	DS	2.0	
		5	1-1-1-1	3	DS	2.0	
			2-2-3-3	4	DS	1.0	
		10	2-2-2-2	5	DS	1.5	
			TUBE	1	PT	1.5	
		15					
			1-1-1	6	DS	1.0	
			1-1-1	7	DS	1.5	
		20					
			TUBE	2	PT	2.0	
		25					
			1-1-1	8	DS	1.5	
			1-1-2	9	DS	1.5	
		30	1-2-1	10	DS	1.5	
			2-2-2	11	DS	1.5	
		35					
			2-2-2	12	DS	1.5	
			2-2-2	13	DS	1.5	
		40					

LEGEND
DS DRIVEN SPOON
ST SHELBY TUBE
PS PISTON SAMPLE
RC Rock Core

GROUND WATER
AT COMPLETION ______ CAVED ______
AT ______ HRS. ______ CAVED ______

HSA HOLLOW STEM AUGER
DC DRIVEN CASING
MD MUD DRILLING

Earth Engineering & Sciences, Inc.

BORING LOG

PROJECT Delaware Solid Waste Authority
Northern Solid Waste Facility - 2

BORING No. GF-2
PROJECT No. 86-123

LOCATION OF BORING Wilmington, Delaware

ELEV. ______ DATE: START ______ FINISH ______ INSPECTOR ______
HAMMER Wt. 140 HAMMER DROP 7" SPOON OD ______ FOREMAN M.Ebert
BORING METHOD HSA ROCK CORE DIA ______ MISC. ______

ELEV.	SOIL DESCRIPTION	DEPTH	BLOWS 6"	No.	TYPE	REC	REMARKS
	Brown moist silty clay	41.5	2-2-3	14	DS	1.5	
	Tan & brown moist silty clay (org)		2-2-3	15	DS	1.5	
		44.0					
	Brown moist silty clay (org)	45					
			3-3-3	16	DS	1.5	
			3-3-3	17	DS	1.5	
		50					
			4-4-4	18	DS	1.5	
		55					
			4-4-6	19	DS	1.5	
		58.5	4-17	20	DS	1.0	
	Brown green moist silty sand	60	22	20A	DS	0.5	
		61.5					
	Bottom of hole 61.5'		5-17-24	21	DS	1.0	
		65					
		70					
		75					
		80					

LEGEND
DS DRIVEN SPOON
ST SHELBY TUBE
PS PISTON SAMPLE
RC ROCK CORE

GROUND WATER
AT COMPLETION ______ CAVED ______
AT ______ HRS. ______ CAVED ______

HSA HOLLOW STEM AUGER
DC DRIVEN CASING
MD MUD DRILLING

Earth Engineering & Sciences, Inc.

BORING LOG

PROJECT Delaware Solid Waste Authority
Northern Solid Waste Facility - 2

BORING No. GF-3
PROJECT No. 86-123

LOCATION OF BORING Wilmington, Delaware

ELEV. ______ DATE: START 7-21-86 FINISH 7-21-86 INSPECTOR ______
HAMMER Wt. 140 HAMMER DROP 7" SPOON OD ______ FOREMAN T. Zeiler
BORING METHOD HSA ROCK CORE DIA ______ MISC. ______

ELEV.	SOIL DESCRIPTION	DEPTH	BLOWS 6"	No.	TYPE	REC	REMARKS
	Brown moist sandy silt w/roots & gravel	3.0	5-7-10	1	DS	1.2	GF-3 Auger refusal 7.5'
			3-3-3	2	DS	1.0	offset hole 20'
	Brown moist sandy silt	4.5	2-2-3	3	DS	1.0	southwest GF-3A
	Brown moist silty clay	6.0	1-1-1	4	DS	1.4	Drove spoon to 9.0'
	Brown moist silty sand & gravel		8-12-40	5	DS	1.3	
		9.0	20-22-23	6	DS	1.2	hole dry & backfilled
	Bottom of hole 9.0'						

5 10 15 20 25 30 35 40

LEGEND
DS DRIVEN SPOON
ST SHELBY TUBE
PS PISTON SAMPLE
RC ROCK CORE

GROUND WATER
AT COMPLETION ______ CAVED ______
AT ______ HRS. ______ CAVED ______

HSA HOLLOW STEM AUGER
DC DRIVEN CASING
MD MUD DRILLING

Earth Engineering & Sciences, Inc.

BORING LOG

PROJECT Delaware Solid Waste Authority
Northern Solid Waste Facility - 2

BORING No. GF-3A 1 of 2

PROJECT No. 86-123

LOCATION OF BORING Wilmington, Delaware

ELEV. ______ DATE: START 7-21-86 FINISH 7-21-86 INSPECTOR ______

HAMMER Wt. 140 HAMMER DROP 7" SPOON OD ______ FOREMAN T. Zeiler

BORING METHOD HSA ROCK CORE DIA ______ MISC. ______

ELEV.	SOIL DESCRIPTION	DEPTH	BLOWS 6"	No.	TYPE	REC	REMARKS
	Brown moist silty clay w/grass	1.5	2-3-3	1	DS	1.2	Water at 15.0' at completion 14.6'
	Brown moist silty clay w/roots		2-2-2	2	DS	1.2	
		5	2-2-2	3	DS	1.2	
		7.5	1-2-2	4	DS	1.2	
	Dark brown moist silty clay		2-1-2	5	DS	1.1	
		10	1-1-1	6	DS	0.0	
			1-1-2	7	DS	1.1	
			TUBE	1	PT	0.8	
		15	1-1-1	9	DS	1.4	
			1-1-1	10	DS	1.5	
		20	1-1-1	11	DS	1.5	
		24.0	1-1-1	12	DS	1.2	
	Dark brown moist silty clay	25	1-2-2	13	DS	1.2	
			1-1-2	14	DS	1.5	
		30	1-1-2	15	DS	1.5	
		34.0	1-WOH-1	16	DS	1.4	
	Brown moist clayey silty sand w/gravel	35	2-2-3	17	DS	1.5	
		39.0	4-4-6	18	DS	1.5	
		40					

LEGEND

DS DRIVEN SPOON

ST SHELBY TUBE

PS PISTON SAMPLE

RC ROCK CORE

GROUND WATER

AT COMPLETION ______ CAVED ______

AT ______ HRS. ______ CAVED ______

HSA HOLLOW STEM AUGER

DC DRIVEN CASING

MD MUD DRILLING

Earth Engineering & Sciences, Inc.

BORING LOG

PROJECT Delaware Solid Waste Authority
Northern Solid Waste Facility - 2

BORING No. GF-3A
PROJECT No. 86-123

LOCATION OF BORING Wilmington, Delaware

ELEV. ______ DATE: START 7-21-86 FINISH 7-21-86 INSPECTOR ______
HAMMER Wt. 140 HAMMER DROP 7" SPOON OD ______ FOREMAN T. Zeiler
BORING METHOD HSA ROCK CORE DIA ______ MISC. ______

ELEV.	SOIL DESCRIPTION	DEPTH	BLOWS 6"	No.	TYPE	REC	REMARKS
	Dark brown black moist silty clay w/grass (org)		2-2-2	19	DS	1.4	
			WOH-2-2-	20	DS	1.5	
		45					
			1-2-3	21	DS	1.5	
		49.0	2-2-3	22	DS	1.5	
	Dark brown black moist silty clay	50					
		51.5	3-3-3	23	DS	1.5	
	Dark brown black moist silty clay w/grass(org)	54.0	2-4-5	24	DS	1.5	
	Brown moist silty claye sand w/gravel	55	2-6-7	25	DS	1.5	
		59.0	8-17-25	26	DS	1.5	
	Bottom of hole 59.0'	60					
		65					
		70					
		75					
		80					

LEGEND
DS DRIVEN SPOON
ST SHELBY TUBE
PS PISTON SAMPLE
RC ROCK CORE

GROUND WATER
AT COMPLETION ______ CAVED ______
AT ______ HRS. ______ CAVED ______

HSA HOLLOW STEM AUGER
DC DRIVEN CASING
MD MUD DRILLING

Earth Engineering & Sciences, Inc.

BORING LOG

PROJECT Delaware Solid Waste Authority
Northern Solid Waste Facility - 2

BORING No. GF-5 1 of 2
PROJECT No. 86-123

LOCATION OF BORING Wilmington, Delaware

ELEV. ________ DATE: START 7-22-86 FINISH 7-22-86 INSPECTOR ________
HAMMER Wt. 140 HAMMER DROP 7" SPOON OD ________ FOREMAN M. Ebert
BORING METHOD HSA ROCK CORE DIA ________ MISC. ________

ELEV.	SOIL DESCRIPTION	DEPTH	BLOWS 6"	No.	TYPE	REC	REMARKS
	Brown moist silty clay w/grass	2.0	1-1-1-2	1	DS	1.5	water at 6.0' backfilled
	Brown moist silty clay		1-1-1-1	2	DS	1.0	
		5	1-1-1-1	3	DS	2.0	
			1-1-1-1	4	DS	2.0	Last sample 57.5-59.0 still in hole poss. gravel
		10	1-1-1-1	5	DS	2.0	
			1-1-1	6	DS	1.5	
		15	1-1-1	7	DS	1.5	
			1-1-1	8	DS	1.5	
		20					
		21.5	1-1-1	9	DS	1.5	
	Brown moist silty clay w/grass (org)		1-1-1	10	DS	1.5	
		25	1-1-1	11	DS	1.5	
		29.0	2-3-4	12	DS	1.5	
	Brown moist silty clay w/sand & gravel	30, 30.5	1	13	DS	0.5	
	Black moist silty clay	31.5	1-1	13A	DS	1.0	
			2-2-2	14	DS	1.5	
	Brown moist silty clay w/sand & gravel	35, 34.0	2-1-2	15	DS	1.5	
	Brown moist silty clay w/grass (org)		1-1-1	16	DS	1.5	
		40					

LEGEND
DS DRIVEN SPOON
ST SHELBY TUBE
PS PISTON SAMPLE
RC ROCK CORE

GROUND WATER
AT COMPLETION ________ CAVED ________
AT ______ HRS. ________ CAVED ________

HSA HOLLOW STEM AUGER
DC DRIVEN CASING
MD MUD DRILLING

Earth Engineering & Sciences, Inc.

BORING LOG

PROJECT Delaware Solid Waste Authority
Northern Solid Waste Facility - 2

BORING No. GF-5
PROJECT No. 86-123

LOCATION OF BORING Wilmington, Delaware

ELEV. ______ DATE: START 7-22-86 FINISH 7-22-86 INSPECTOR ______
HAMMER Wt. 140 HAMMER DROP 7" SPOON OD ______ FOREMAN M. Ebert
BORING METHOD HSA ROCK CORE DIA ______ MISC. ______

ELEV.	SOIL DESCRIPTION	DEPTH	BLOWS 6"	No.	TYPE	REC	REMARKS
	Brown moist silty clay w/grass (org)		2-2-2	17	DS	1.5	
			2-2-2	18	DS	1.5	
		45					
		46.5	2-3-3	19	DS	1.5	
	Black moist silty clay w/grass (org) sand & gravel		2-2-4	20	DS	1.0	
		50	3-3-3	21	DS	1.5	
			4-4-5	22	DS	1.5	
		55	4-5-5	23	DS	1.5	
		59.0					
	Bottom of hole 59.0'	60					
		65					
		70					
		75					
		80					

LEGEND
DS DRIVEN SPOON
ST SHELBY TUBE
PS PISTON SAMPLE
RC ROCK CORE

GROUND WATER
AT COMPLETION ______ CAVED ______
AT ______ HRS. ______ CAVED ______

HSA HOLLOW STEM AUGER
DC DRIVEN CASING
MD MUD DRILLING

Earth Engineering & Sciences, Inc.

BORING LOG

PROJECT Delaware Solid Waste Authority
Northern Solid Waste Facility - 2

BORING No. GF-6 1 of 2
PROJECT No. 86-123

LOCATION OF BORING Wilmington, Delaware

ELEV. ______ DATE: START 7-23-86 FINISH 7-24-86 INSPECTOR ______

HAMMER Wt. 140 HAMMER DROP 7" SPOON OD ______ FOREMAN T. Zeiler

BORING METHOD HSA ROCK CORE DIA ______ MISC. ______

ELEV.	SOIL DESCRIPTION	DEPTH	BLOWS 6"	No.	TYPE	REC	REMARKS
	Black moist silty clay	1.5	2-2-2	1	DS	1.2	water at 6.5' backfilled
	Brown moist silty clay w/grass	4.0	1-1-1	2	DS	1.5	
	Brown moist silty clay	5					
			TUBE	1	PT	1.8	
			WOH	3	DS	1.0	
		10	WOH	4	DS	1.2	
			WOH -1-1	5	DS	1.0	
			WOH	6	DS	1.5	
			WOH	7	DS	1.2	
		15	TUBE	2	PT	2.0	
		20	WOH	8	DS	1.5	
			WOH	9	DS	1.5	
		25	TUBE	3	PT	2.0	
			WOH	10	DS	1.5	
		30	WOH	11	DS	1.5	
			WOH-1-1	12	DS	1.5	
		34.0					
	Dark brown moist silty clay	35	WOH-1-1	13	DS	1.5	
			WOH-1-1	14	DS	1.5	
		40					

LEGEND

DS DRIVEN SPOON
ST SHELBY TUBE
PS PISTON SAMPLE
RC ROCK CORE

GROUND WATER
AT COMPLETION ______ CAVED ______
AT ______ HRS. ______ CAVED ______

HSA HOLLOW STEM AUGER
DC DRIVEN CASING
MD MUD DRILLING

Earth Engineering & Sciences, Inc.

BORING LOG

PROJECT Delaware Solid Waste Authority
Northern Solid Waste Facility - 2

BORING No. GF-6
PROJECT No. 86-123

LOCATION OF BORING Wilmington, Delaware

ELEV. ______ DATE: START 7-23-86 FINISH 7-24-86 INSPECTOR ______
HAMMER Wt. 140 HAMMER DROP 7" SPOON OD ______ FOREMAN T. Zeiler
BORING METHOD HSA ROCK CORE DIA ______ MISC. ______

ELEV.	SOIL DESCRIPTION	DEPTH	BLOWS 6"	No.	TYPE	REC	REMARKS
	Dark brown moist silty clay		1-1-1	15	DS	1.5	
			WOH-WOH-1	16	DS	1.5	
		45	1-1-1	17	DS	1.5	
			WOH	18	DS	1.5	
		50	WOH-2-2-	19	DS	1.5	
		52.0					
	Brwon, black moist silty clay (org)		WOH-1-2	20	DS	1.5	
		55	1-1-2	21	DS	1.5	
			1-1-1	22	DS	1.5	
		60					
		61.5	1-2-4	23	DS	1.5	
	Brown moist silty clay		3-4-6	24	DS	1.5	
		64.0					
	Bottom of hole 64.0'	65					
		70					
		75					
		80					

LEGEND
DS DRIVEN SPOON
ST SHELBY TUBE
PS PISTON SAMPLE
RC ROCK CORE

GROUND WATER
AT COMPLETION ______ CAVED ______
AT ______ HRS. ______ CAVED ______

HSA HOLLOW STEM AUGER
DC DRIVEN CASING
MD MUD DRILLING

Earth Engineering & Sciences, Inc.

BORING LOG

PROJECT Delaware Solid Waste Authority
Northern Solid Waste Facility - 2

BORING No. GF-7 1 of 2
PROJECT No. 86-123

LOCATION OF BORING Wilmington, Delaware

ELEV. ____ DATE: START ____ FINISH ____ INSPECTOR ____
HAMMER Wt. 140 HAMMER DROP 7" SPOON OD ____ FOREMAN M. Ebert
BORING METHOD HSA ROCK CORE DIA ____ MISC. ____

ELEV.	SOIL DESCRIPTION	DEPTH	BLOWS 6"	No.	TYPE	REC	REMARKS
	Brown moist silty clay w/grass		1-1-1-1	1	DS	1.8	Installed well-point 27.0' w/ 3' of stickup
		3.0	1-1-1-1	2	DS	1.5	
	Brown moist silty clay	5	1-1-1-1	3	DS	1.5	hole backfilled
			1-WOH-WOH	4	DS	1.5	
			1-1-1-1	5	DS	1.5	
		10	1-1-1-1	6	DS	1.5	
			TUBE	1	PT	1.8	
		15					
			1-1-1	7	DS	1.5	
			1-1-1	8	DS	1.5	
		20	TUBE	2	PT	2.0	
		25	1-1-1	9	DS	1.5	
			1-1-2	10	DS	1.5	
		30	2-2-2	11	DS	1.5	
		34.0	2-1-1	12	DS	1.5	
	Dark brown moist silty clay	35	1-1-1	13	DS	1.5	
			1-1-1	14	DS	1.5	
		40					

LEGEND
DS DRIVEN SPOON
ST SHELBY TUBE
PS PISTON SAMPLE
RC ROCK CORE

GROUND WATER
AT COMPLETION ____ CAVED ____
AT HRS. ____ CAVED ____

HSA HOLLOW STEM AUGER
DC DRIVEN CASING
MD MUD DRILLING

Earth Engineering & Sciences, Inc.

BORING LOG

PROJECT Delaware Solid Waste Authority
Northern Solid Waste Facility - 2

BORING No. GF-7
PROJECT No. 86-123

LOCATION OF BORING Wilmington, Delaware

ELEV. ______ DATE: START ______ FINISH ______ INSPECTOR ______
HAMMER Wt. 140 HAMMER DROP 7" SPOON OD ______ FOREMAN M. Ebert
BORING METHOD HSA ROCK CORE DIA ______ MISC. ______

ELEV.	SOIL DESCRIPTION	DEPTH	BLOWS 6"	No.	TYPE	REC	REMARKS
	Dark brown moist silty clay		2-2-2	15	DS	1.5	
			2-1-2	16	DS	1.5	
		45					
			2-2-2	17	DS	1.5	
			2-3-3	18	DS	1.5	
		49.0					
	Dark brown moist silty clay w/grass org.	50					
			2-2-2	19	DS	1.5	
			2-2-2-	20	DS	1.5	
		55					
		56.5	2-2-2	21	DS	1.5	
	Tan brown moist sandy silty clay		4-6-10	22	DS	1.5	
		59.0					
	Brown moist silty clayey sand w/gravel	60					
		61.5	22-26-36	23	DS	1.5	
	Dark brown moist sandy silty clay w/gravel		20-22-23	24	DS	1.0	
		65					
		66.1	10-12-20	25	DS	0.5	
	Bottom of hole 66.1'						
		70					
		75					
		80					

LEGEND
DS DRIVEN SPOON
ST SHELBY TUBE
PS PISTON SAMPLE
RC Rock Core

GROUND WATER
AT COMPLETION ______ CAVED ______
AT ______ HRS. ______ CAVED ______

HSA HOLLOW STEM AUGER
DC DRIVEN CASING
MD MUD DRILLING

Earth Engineering & Sciences, Inc.

BORING LOG

PROJECT Delaware Solid Waste Authority
Northern Solid Waste Facility - 2

BORING No. GF-8
PROJECT No. 86-123

LOCATION OF BORING Wilmington, Delaware

ELEV. ______ DATE: START 7-16-86 FINISH 7-18-86 INSPECTOR D. Collins
HAMMER Wt. 140 HAMMER DROP 7" SPOON OD ______ FOREMAN T. Zeiler
BORING METHOD HSA ROCK CORE DIA ______ MISC. ______

ELEV.	SOIL DESCRIPTION	DEPTH	BLOWS 6"	No.	TYPE	REC	REMARKS
	Dark brown moist silty clay	3.0	1-1-2	1	DS	1.2	
			1-1-1	2	DS	1.0	
	Wet gray black moist ~~silty clay~~	~~4.5~~ 5	1-1-1	3	DS	1.0	
	Dark brown moist silty clay	8.5	TUBE	1	PT	1.2	
	Wet blue brown moist silty clay w/occ. organics		1-1-1	4	DS	1.5	
		10	1-1-1	5	DS	1.5	
			WOH-WOH-WOH	6	DS	1.3	
			WOH-WOH WOH	7	DS	1.5	
		15	WOH-1-1	8	DS	1.5	
			1-1-1	9	DS	1.5	
		20	WOH-WOH WOH	10	DS	1.5	
			WOH-WOH	11	DS	1.5	
		25	TUBE	2	PT	2.0	
			1-1-1	12	DS	1.5	
	Wet blue brown moist sandy silty clay w/ occ. organics	30.0	WOH-1-1	13	DS	1.5	
	Blue brown wet moist black silty clay w/ occ. organics	31.5	1-WOH-1	14	DS	1.5	Gas pocket 33.0
		35	2-1-1	15	DS	1.5	
		40	WOH-1-1	16	DS	1.0	Gas pocket 39.0

LEGEND
DS DRIVEN SPOON
ST SHELBY TUBE
PS PISTON SAMPLE
RC ROCK CORE

GROUND WATER
AT COMPLETION ______ CAVED ______
AT ______ HRS. ______ CAVED ______

HSA HOLLOW STEM AUGER
DC DRIVEN CASING
MD MUD DRILLING

Earth Engineering & Sciences, Inc.

BORING LOG

PROJECT Delaware Solid Waste Authority
Northern Solid Waste Facility - 2

BORING No. GF-8
PROJECT No. 86-123

LOCATION OF BORING Wilmington, Delaware

ELEV. ______ DATE: START 7-16-86 FINISH 7-18-86 INSPECTOR D. Collins
HAMMER Wt. 140 HAMMER DROP 7" SPOON OD ______ FOREMAN T. Zeiler
BORING METHOD HSA ROCK CORE DIA ______ MISC. ______

ELEV.	SOIL DESCRIPTION	DEPTH	BLOWS 6"	No.	TYPE	REC	REMARKS
	Blue brown wet moist black silty clay w/ occ. organics		WOH-2-2	17	DS	1.0	
			WOH-WOH-2	18	DS	1.5	
		45	WOH-WOH WOH	19	DS	1.5	
		49.0	WOH-WOH-WOH	20	DS	1.0	
	Blue brown black moist wet silty clay w/larger quantity organics (poss. peat)	50	1-2-2	21	DS	1.5	
		54.0	2-3-3	22	DS	1.5	
	Tan gray moist wet sandy clay	55	4-4-4	23	DS	1.5	
	Dark brown moist sandy gravel	58.0	10-13-15	24	DS	1.5	Gas pocket -59.0 large cavity
		60	6-11-14	25	DS	1.5	
	Bottom of hole 61.5'	61.5					
		65					
		70					
		75					
		80					

LEGEND
DS DRIVEN SPOON
ST SHELBY TUBE
PS PISTON SAMPLE
RC ROCK CORE

GROUND WATER
AT COMPLETION ______ CAVED ______
AT ______ HRS. ______ CAVED ______

HSA HOLLOW STEM AUGER
DC DRIVEN CASING
MD MUD DRILLING

Earth Engineering & Sciences, Inc.

BORING LOG

PROJECT Delaware Solid Waste Authority
Northern Solid Waste Facility - 2

BORING No. GF-15
PROJECT No. 86-123

LOCATION OF BORING Wilmington, Delaware

ELEV. ______ DATE: START 7-24-86 FINISH 7-24-86 INSPECTOR ______
HAMMER Wt. 140 HAMMER DROP 7" SPOON OD ______ FOREMAN T. Zeiler
BORING METHOD HSA ROCK CORE DIA ______ MISC. ______

ELEV.	SOIL DESCRIPTION	DEPTH	BLOWS 6"	No.	TYPE	REC	REMARKS
	Brown moist sitly clay		4-4-6	1	DS	1.2	Dry & backfilled
		4.0	WOH-1-1	2	DS	1.3	
	Dark brown moist silty clay w/grass	5	WOH-1-1	3	DS	1.5	
		8.0	WOH-WOH-1	4	DS	1.0	
	Bottom of hole 8.0'	10					
		15					
		20					
		25					
		30					
		35					
		40					

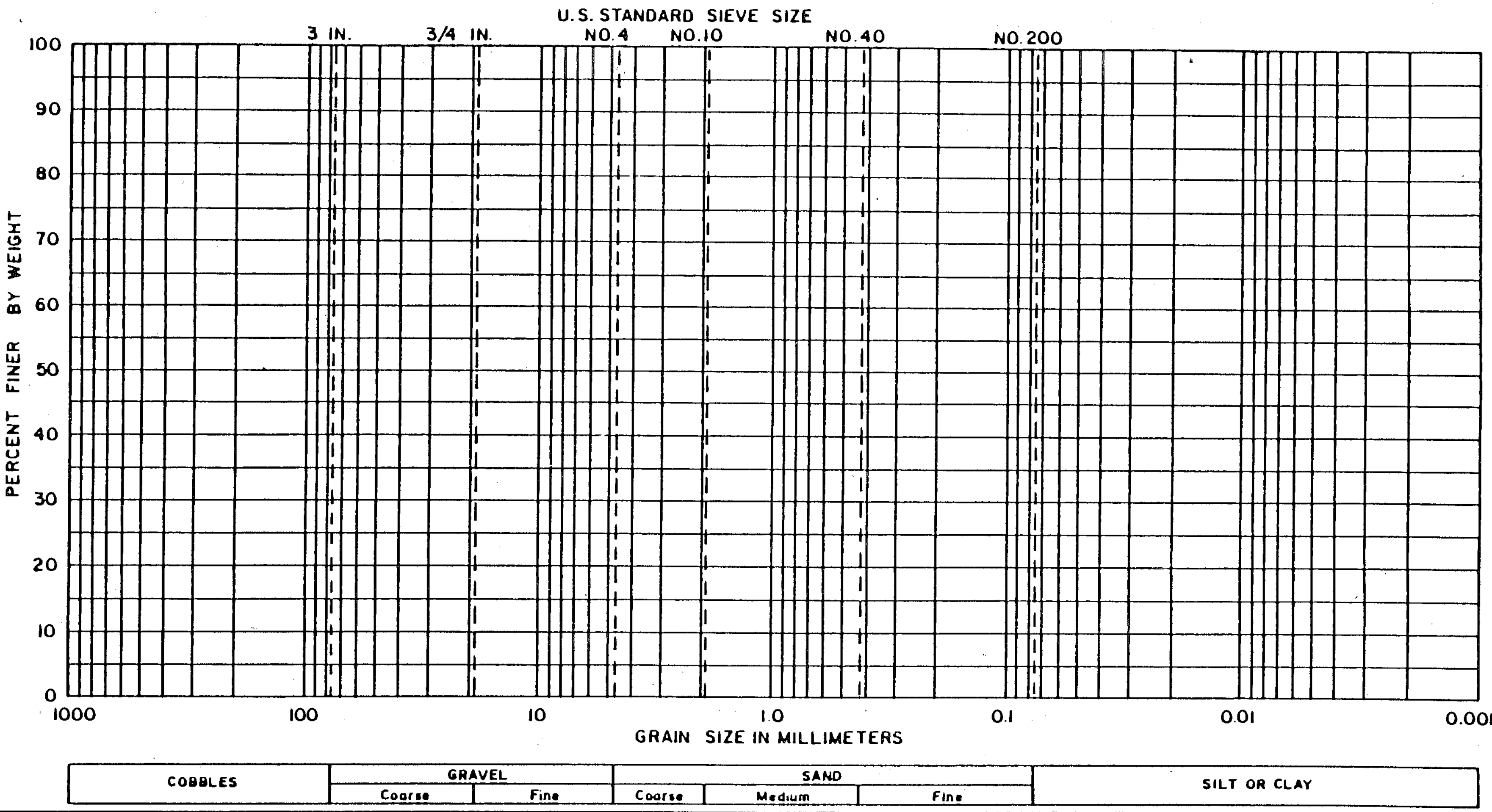
LEGEND
DS DRIVEN SPOON
ST SHELBY TUBE
PS PISTON SAMPLE
RC Rock Core

GROUND WATER
AT COMPLETION ______ CAVED ______
AT HRS. ______ CAVED ______

HSA HOLLOW STEM AUGER
DC DRIVEN CASING
MD MUD DRILLING

APPENDIX B

Laboratory Test Results



Sample No.	Depth	Classification	Nat.WC	LL	PL	PI	Gs
GF-1/ S-7	10.0'-12.0'	MH	69.1	62.4	46.3	16.1	

Description and Comments: Gray Elastic Silt.

CLASSIFICATION TEST-GRADATION CURVES

GANNETT FLEMING GEOTECHNICAL LABORATORY

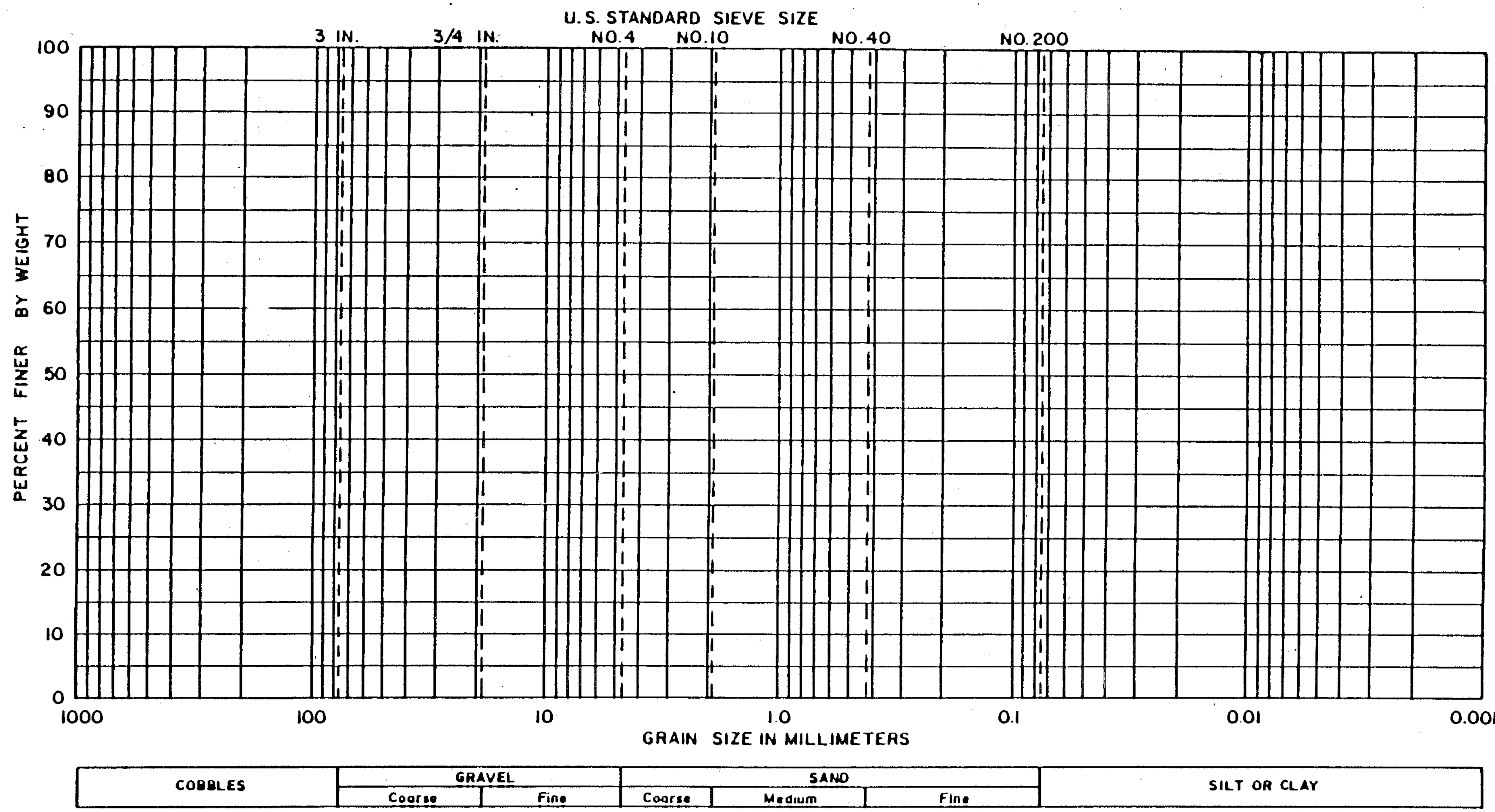
Project: DELAWARE SOLID WASTE FACILITY - 2

Area: WILMINGTON, DE.

Boring No: GF-1 / S-7

Date: 8/20/86

Tested By: Abdolos



Sample No.	Depth	Classification	Nat. WC	LL	PL	PI	Gs
GF-1/S-10	20.0-22.0'	MH	80.8	62.8	44.7	18.1	
			67.9				

Description and Comments: Gray Elastic Silt.

GANNETT FLEMING GEOTECHNICAL LABORATORY

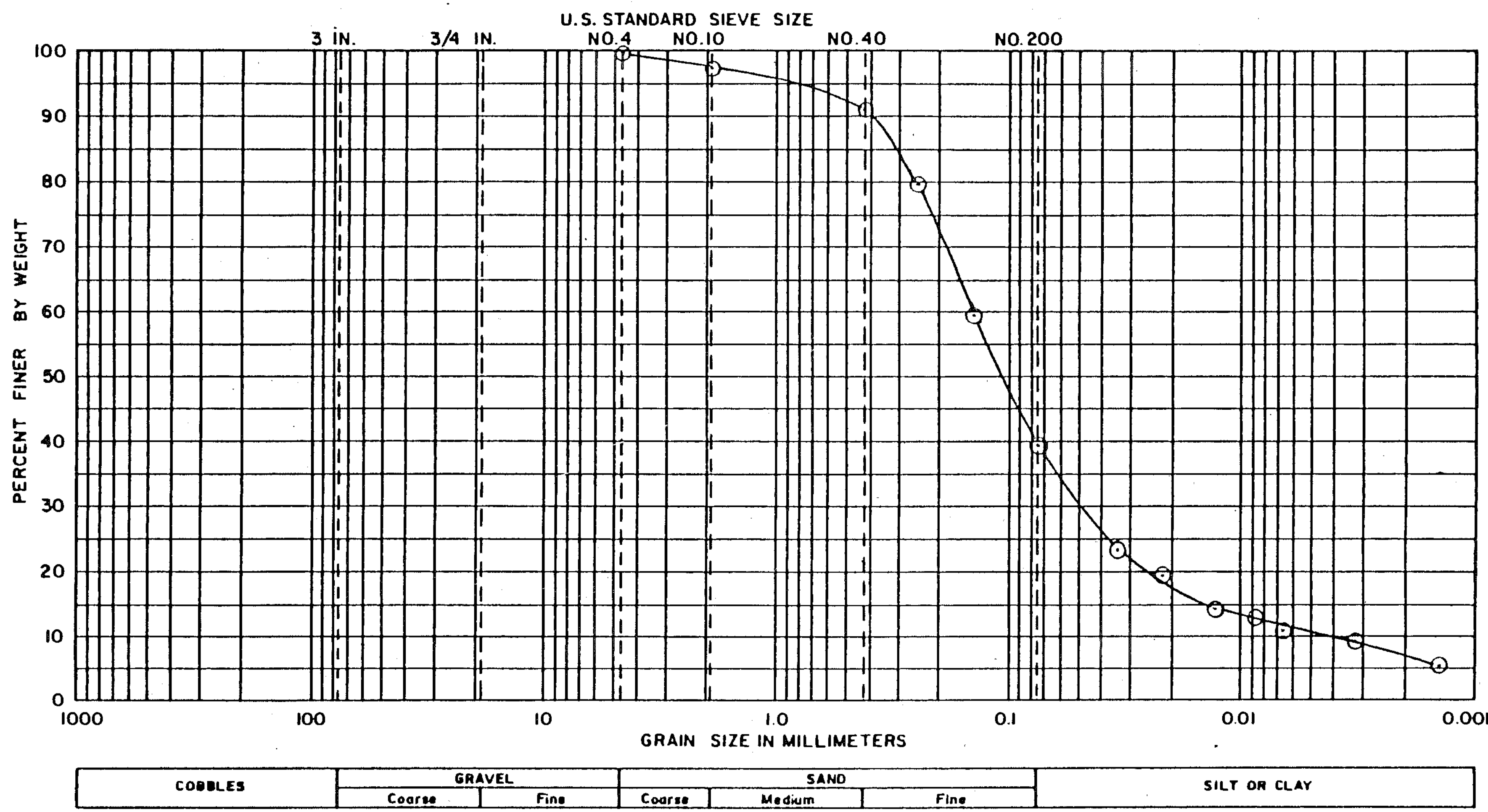
Project: DELAWARE SOLID WASTE FACILITY - 2

Area: WILMINGTON, DE.

Boring No: GF-1 / S-10

Date: 8/20/86 Tested By: Abdolos

CLASSIFICATION TEST-GRADATION CURVES



Sample No.	Depth	Classification	Nat. WC	LL	PL	PI	Gs
GF-1/S-15 & 16	--	ML	33.8	N/P	N/P	--	2.63

Description and Comments: Gray Sandy Silt.

CLASSIFICATION TEST-GRADATION CURVES

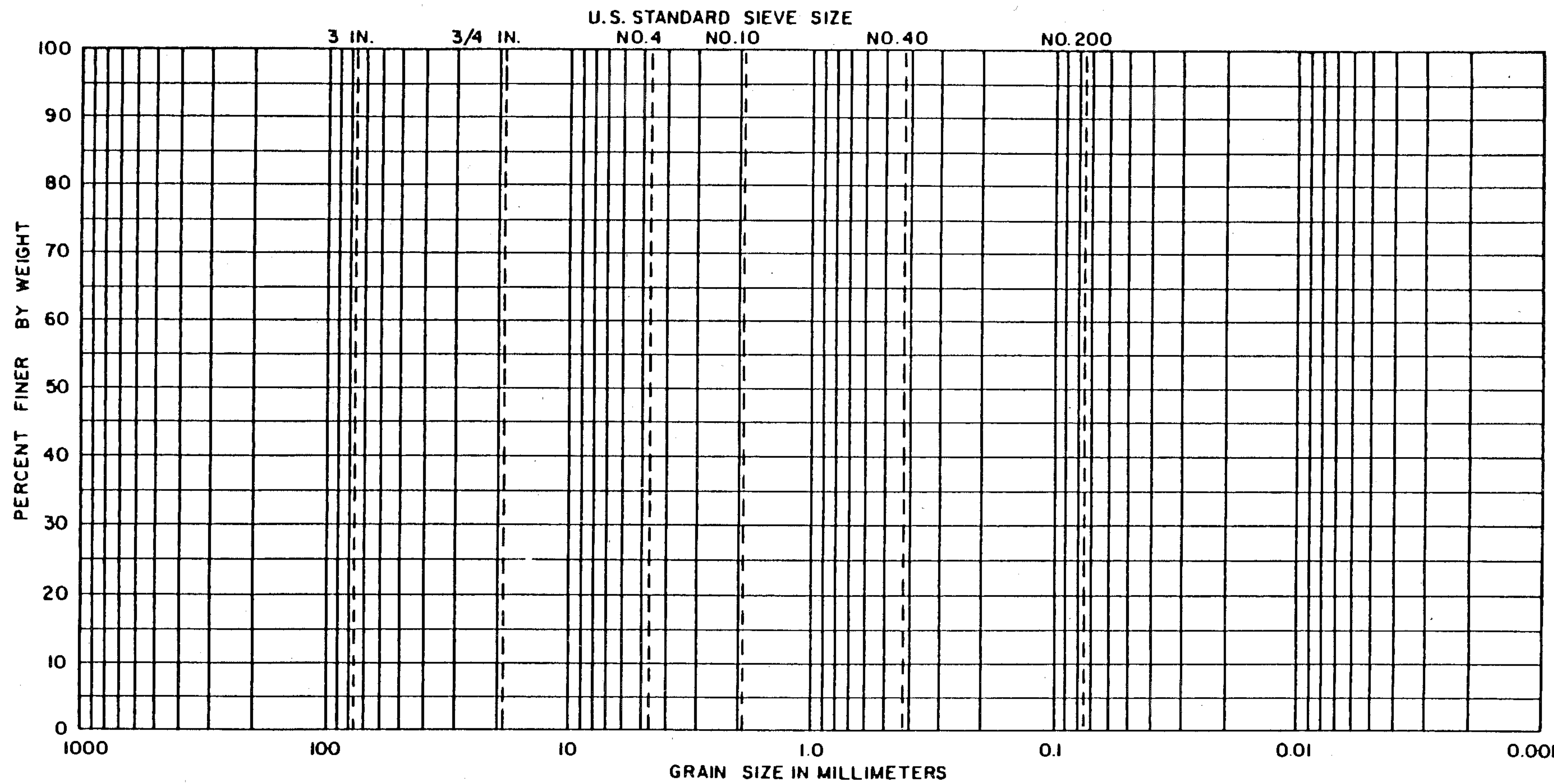
GANNETT FLEMING GEOTECHNICAL LABORATORY

Project: DELAWARE SOLID WASTE FACILITY - 2

Area: Wilmington, DE.

Boring No: GF-1 / S-15 & 16

Date: Aug. 20/86 | Tested By: Abdolos



COBBLES	GRAVEL		SAND			SILT OR CLAY
	Coarse	Fine	Coarse	Medium	Fine	

Sample No.	Depth	Classification	Nat. WC	LL	PL	PI	Gs
GF-1/S-19&	---	OH	109.0	89.1	N/P	--	
20		Oven-dried LL =		67.7			

Description and Comments: 1) Gray Organic Silt.

$$\frac{\text{LL Oven-dried}}{\text{LL Air-dried}} = .752$$

GANNETT FLEMING GEOTECHNICAL LABORATORY

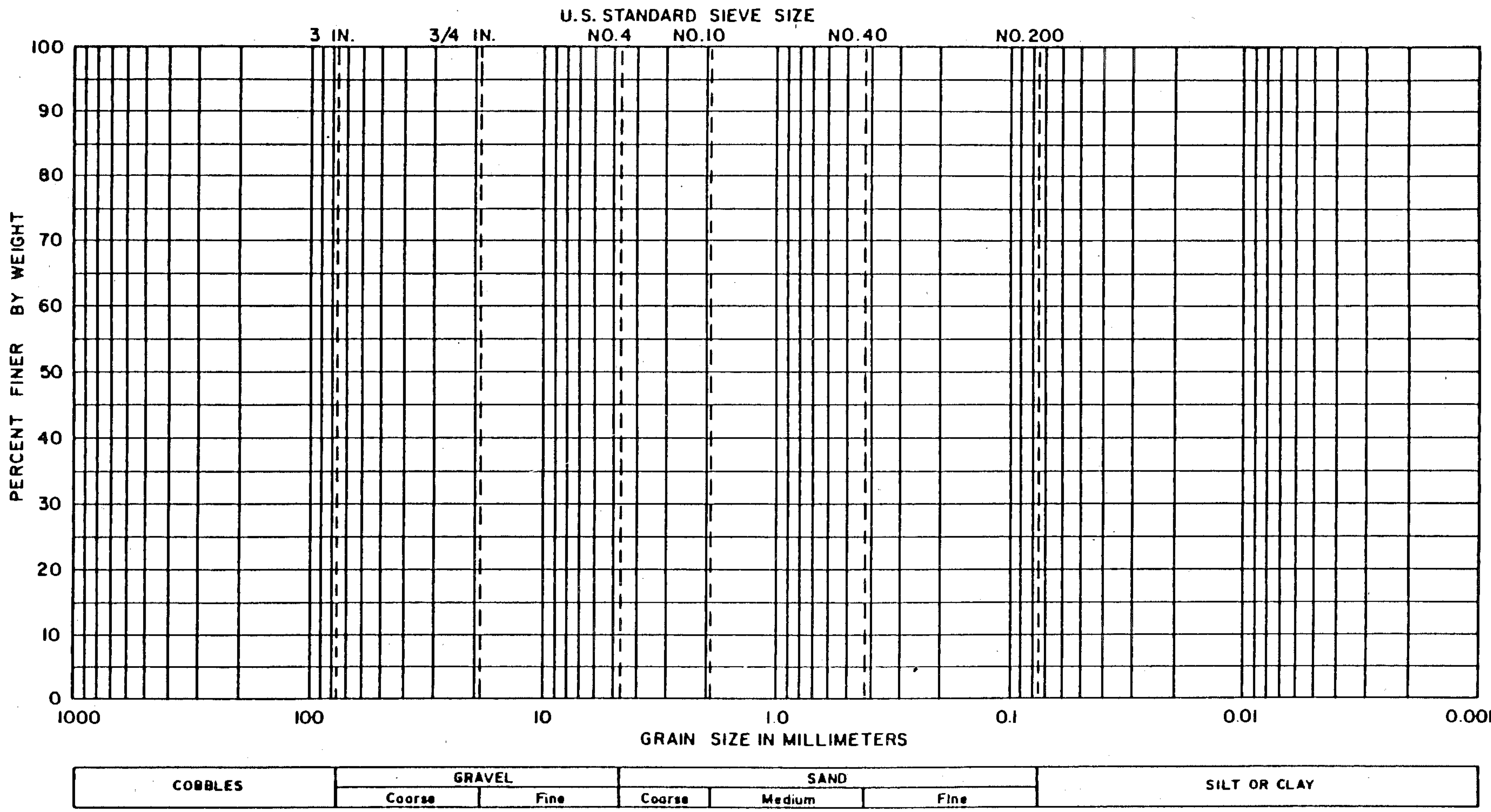
Project: DELAWARE SOLID WASTE FACILITY - 2

Area: WILMINGTON, DE.

Boring No: GF-1 / S-19 & 20

Date: 8/20/86 **Tested By:** Abdolos

CLASSIFICATION TEST-GRADATION CURVES



Sample No.	Depth	Classification	Nat.WC	LL	PL	PI	Gs
GF-3-A /S-5 & 6	---	MH	59.7	62.5	46.7	15.8	--

Description and Comments: Gray Elastic Silt.

CLASSIFICATION TEST-GRADATION CURVES

GANNETT FLEMING GEOTECHNICAL LABORATORY

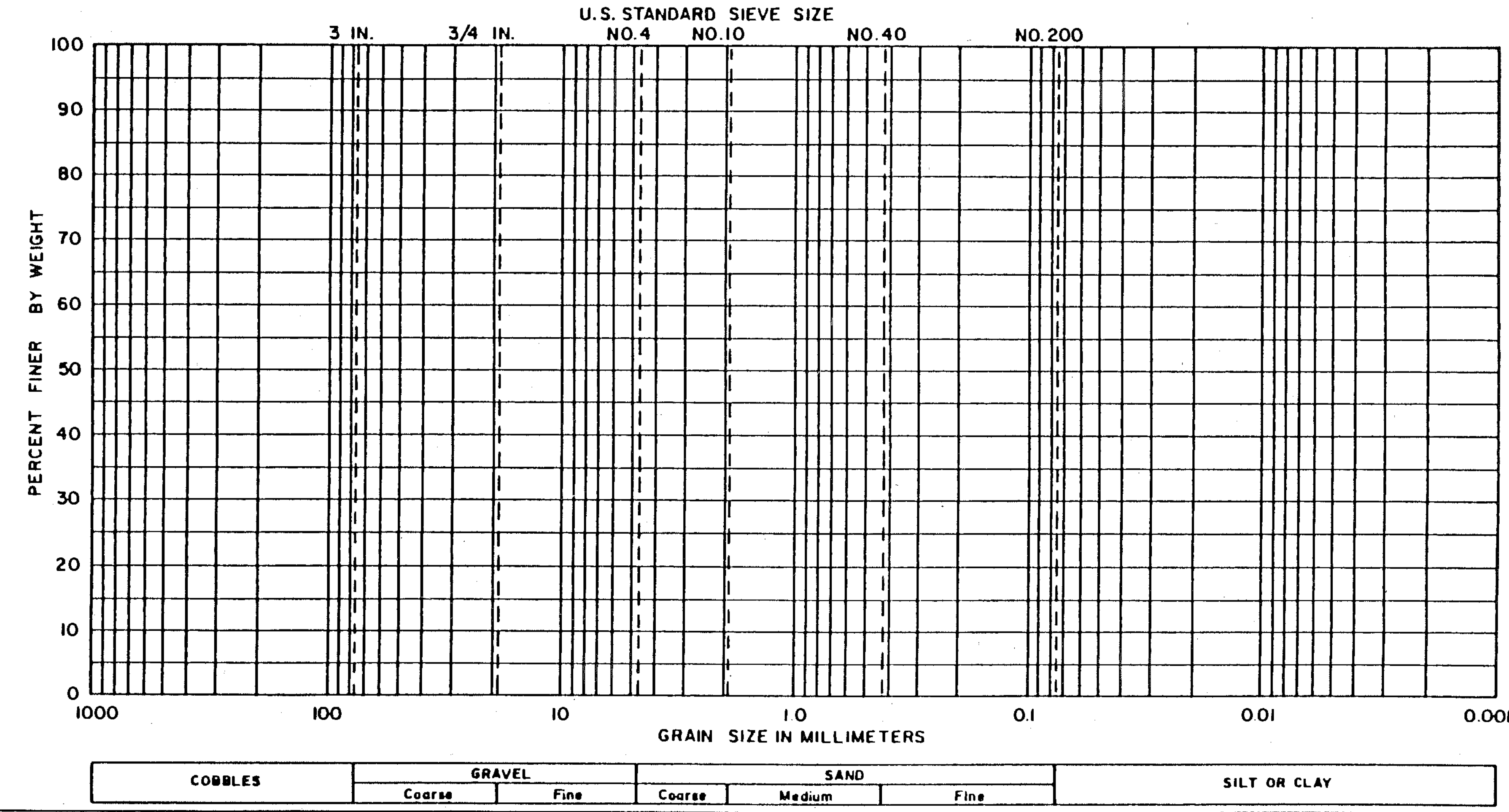
Project: DELAWARE SOLID WASTE FACILITY - 2

Area: WILMIMGTON, DE.

Boring No: GF-3-A / S - 5 & 6.

Date: 8/20/86

Tested By: Abdolos



Sample No.	Depth	Classification	Nat.WC	LL	PL	PI	Gs
GF-3-A/ S-10 & 11	--	MH	78.5	67.6	49.9	17.7	--

Description and Comments: Gray Elastic Silt.

GANNETT FLEMING GEOTECHNICAL LABORATORY

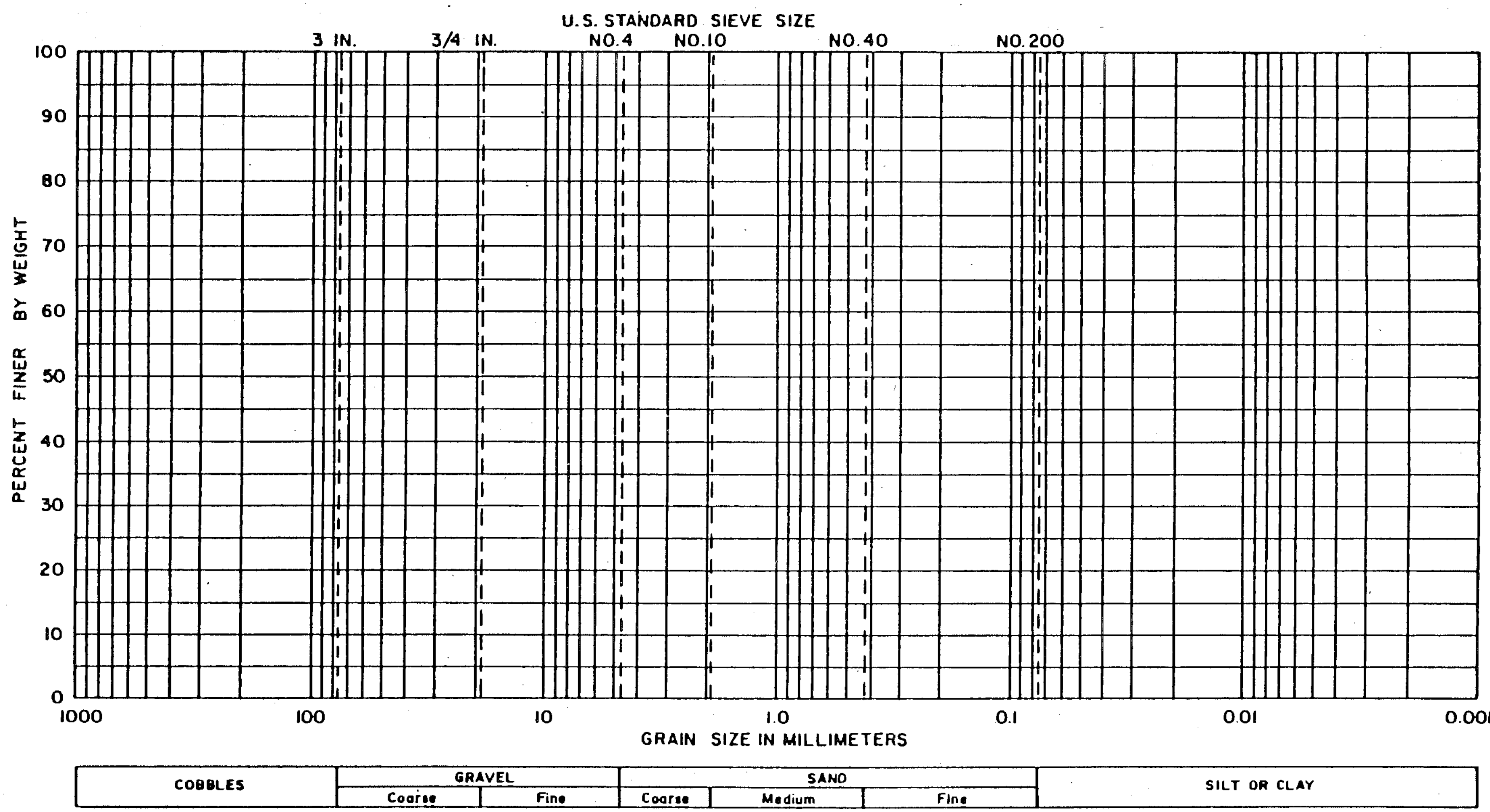
Project: DELAWARE SOLID WASTE FACILITY - 2

Area: WILMINGTON, DE.

Boring No: GF-3-A / S - 10 & 11.

Date: 8/20/86 | Tested By: Abdolos

CLASSIFICATION TEST-GRADATION CURVES



Sample No.	Depth	Classification	Nat.WC	LL	PL	PI	Gs
GF-3-A /S- 15 & 16	--	ML	42.6	46.8	32.8	14.0	---

Description and Comments: Gray Silt.

CLASSIFICATION TEST–GRADATION CURVES

GANNETT FLEMING GEOTECHNICAL LABORATORY

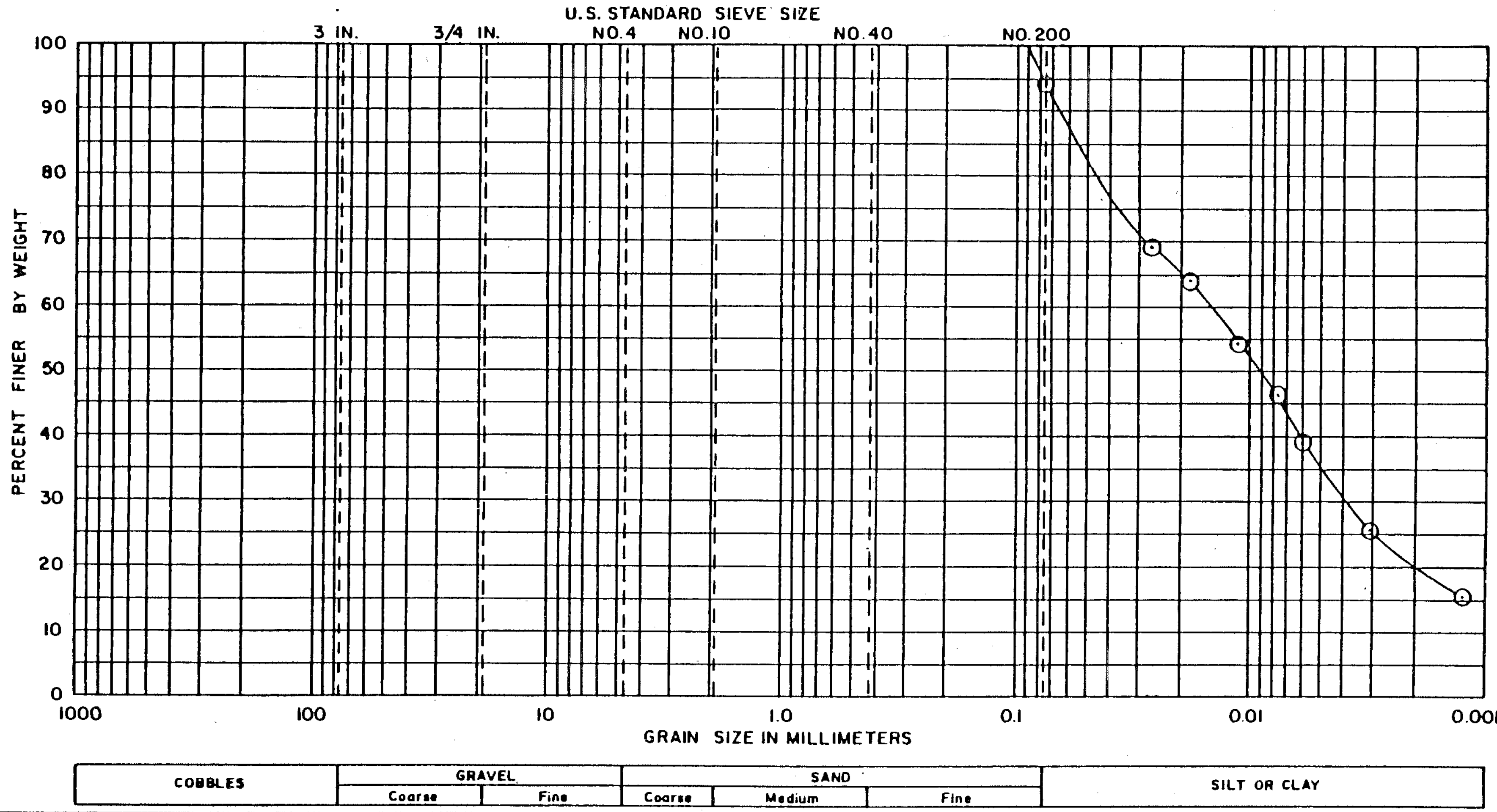
Project: DELAWARE SOLID WASTE FACILITY - 2

Area: WILMINGTON,DE.

Boring No: GF-3-A / S - 15 & 16.

Date: 8/20/86

Tested By: Abdolos



Sample No.	Depth	Classification	Nat. WC	LL	PL	PI	Gs
GF-5/S-4-5	5.0' - 10.0'	MH	66.7	53.0	36.5	16.5	2.61

Description and Comments: Gray Elastic Silt.

CLASSIFICATION TEST-GRADATION CURVES

GANNETT FLEMING GEOTECHNICAL LABORATORY

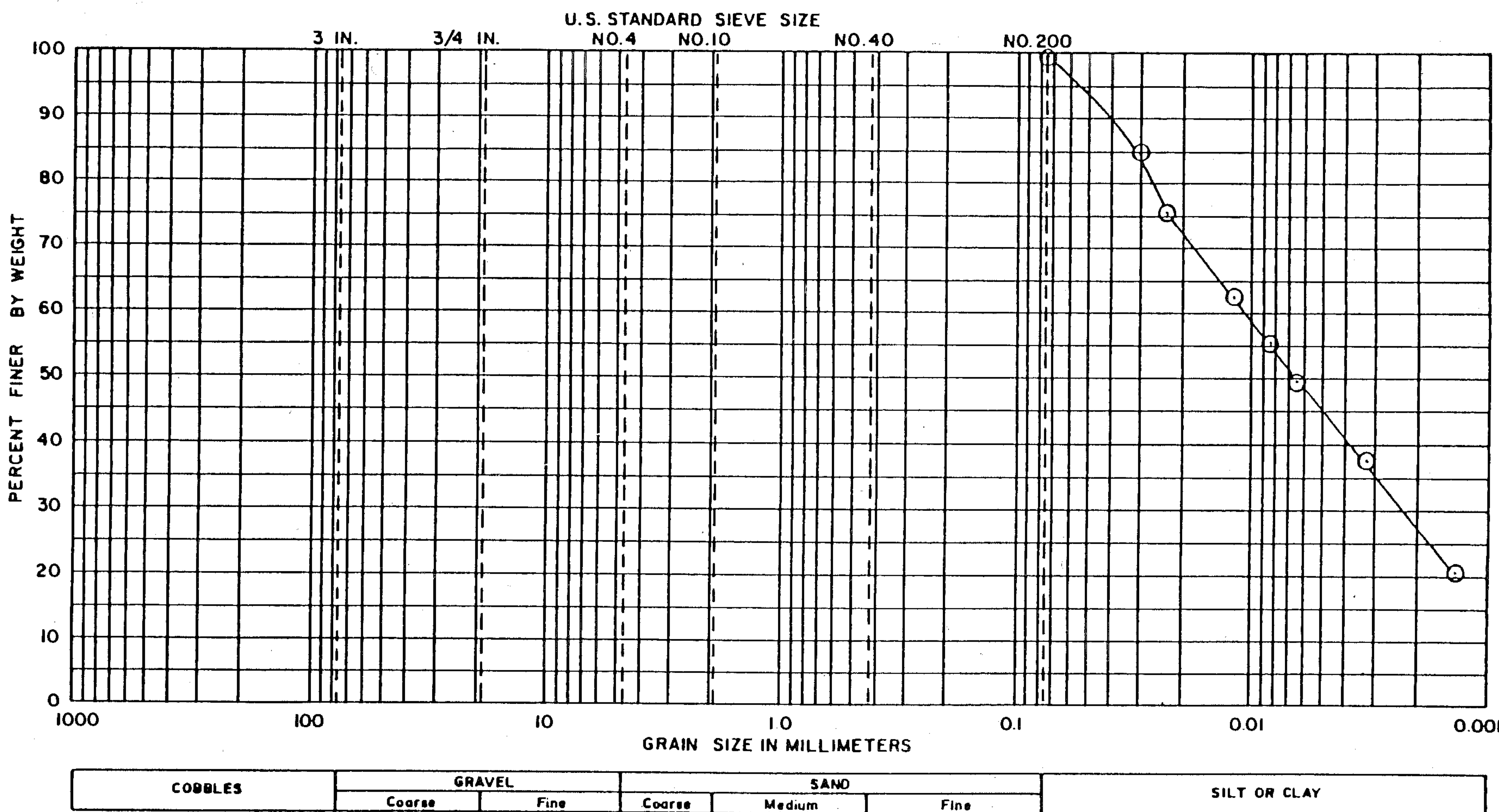
Project: DELAWARE SOLID WASTE FACILITY - 2

Area: WILMINGTON, DE.

Boring No: GF-5 / S-4 & 5

Date: 8/20/86

Tested By: Abdolos



Sample No.	Depth	Classification	Nat. WC	LL	PL	PI	Gs
GF-5/S-9&10	20.0'-24.0'	MH	85.6	62.1	44.2	17.9	2.40

Description and Comments: Gray Elastic Silt.

CLASSIFICATION TEST-GRADATION CURVES

GANNETT FLEMING GEOTECHNICAL LABORATORY

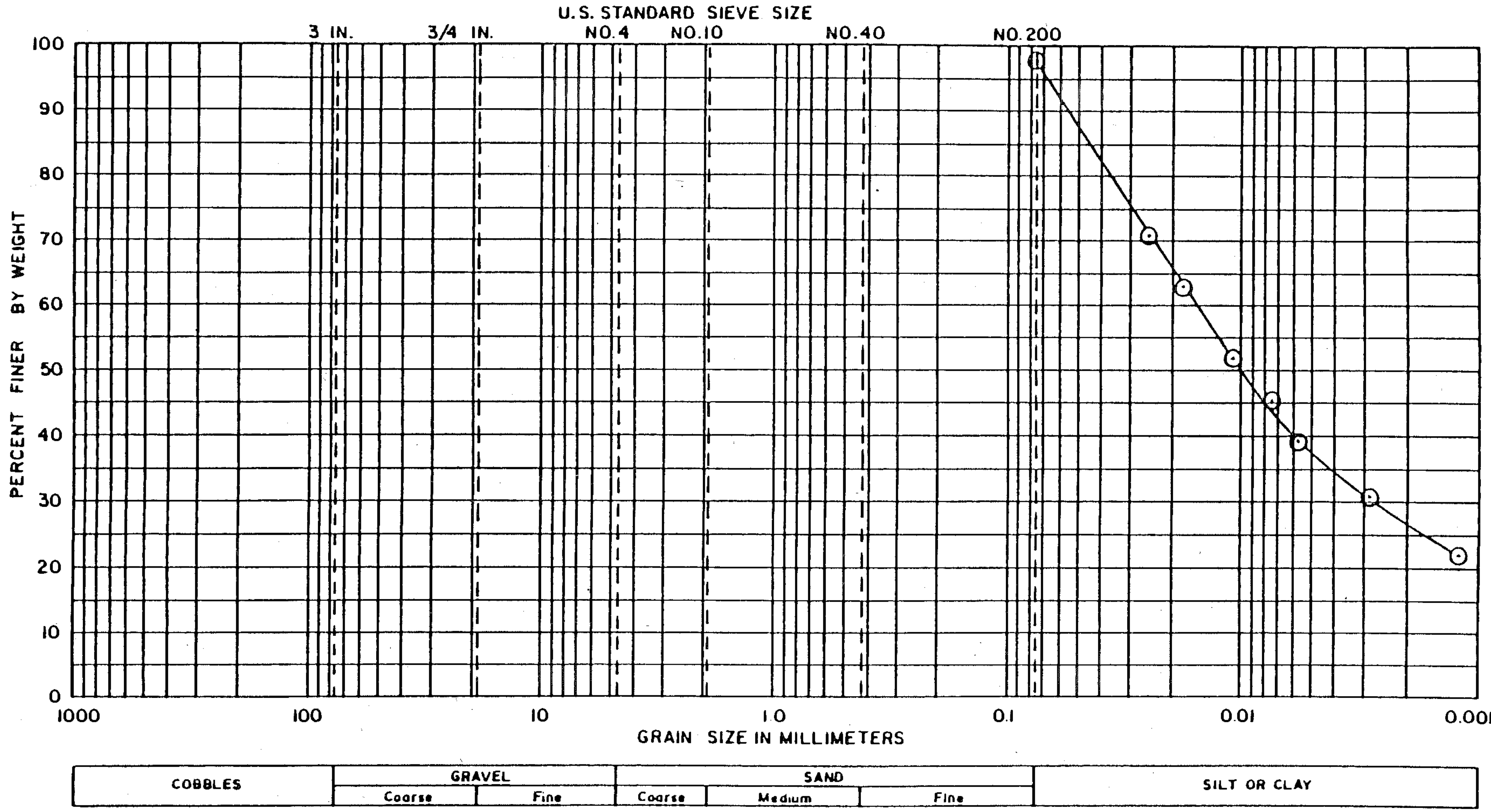
Project: DELAWARE SOLID WASTE FACILITY - 2

Area: WILMINGTON, DE.

Boring No: GF-5 / S-9 & 10.

Date: 8/20/86

Tested By: Abdolos



Sample No.	Depth	Classification	Nat. WC	LL	PL	PI	Gs
GF-6/ S-3	5- 7'	OH	103.6	70.8	43.1	27.8	2.66
		oven-dried		47.3			

Description and Comments: Gray organic clay ;soil had a slightly fibrous texture

GANNETT FLEMING GEOTECHNICAL LABORATORY

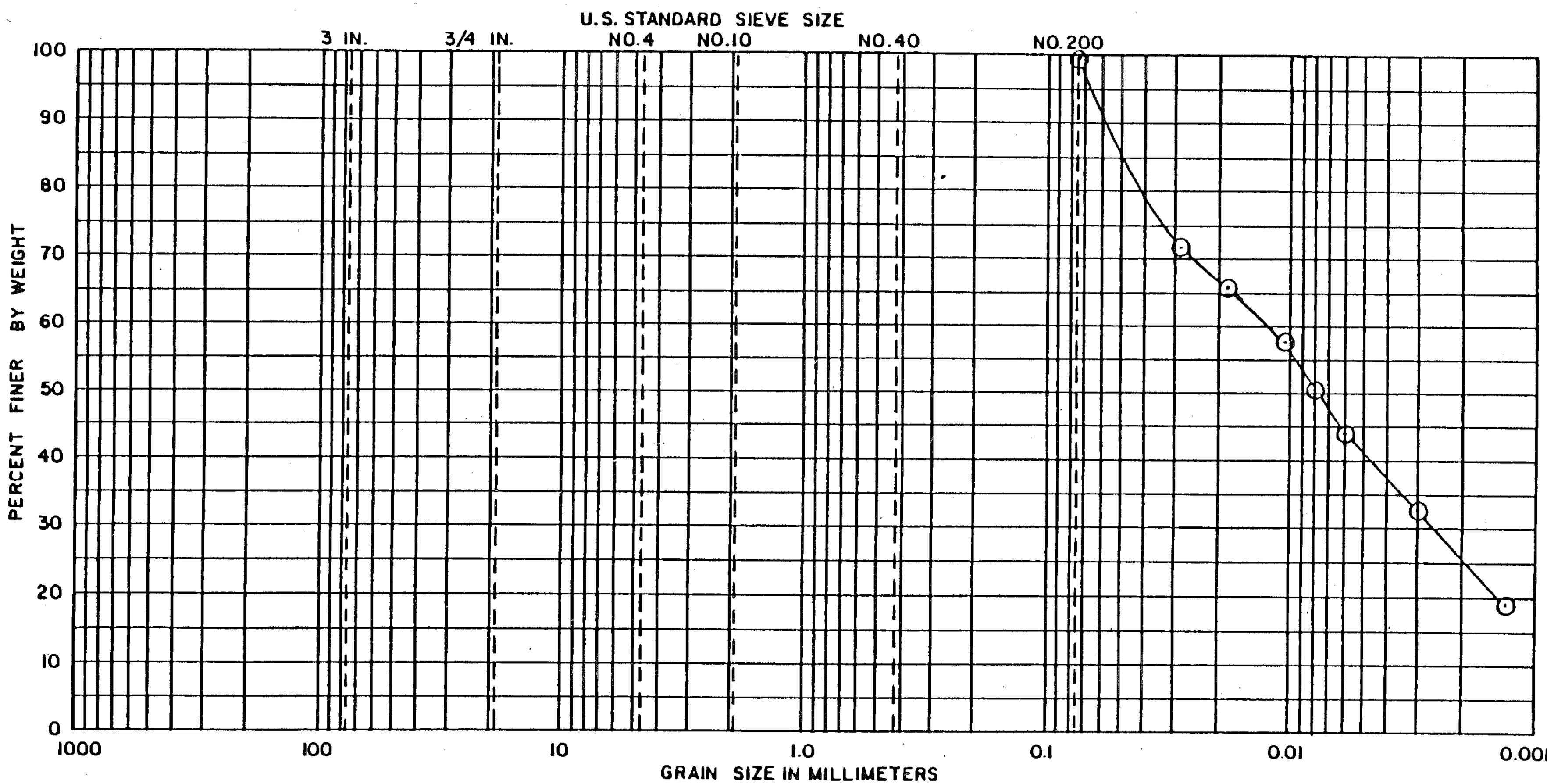
Project: DELAWARE SOLID WASTE FACILITY-2

Area: WILMINGTON, DELAWARE

Boring No: GF-6/ S-3

Date: 8/ 86 Tested By: AB

CLASSIFICATION TEST-GRADATION CURVES



COBBLES	GRAVEL		SAND			SILT OR CLAY
	Coarse	Fine	Coarse	Medium	Fine	

Sample No.	Depth	Classification	Nat. WC	LL	PL	PI	Gs
GF-6/ S-12		OH	85.4	73.1	46.7	26.4	

Description and Comments: Gray organic clay; soil had a slighly fibrous texture

GANNETT FLEMING GEOTECHNICAL LABORATORY

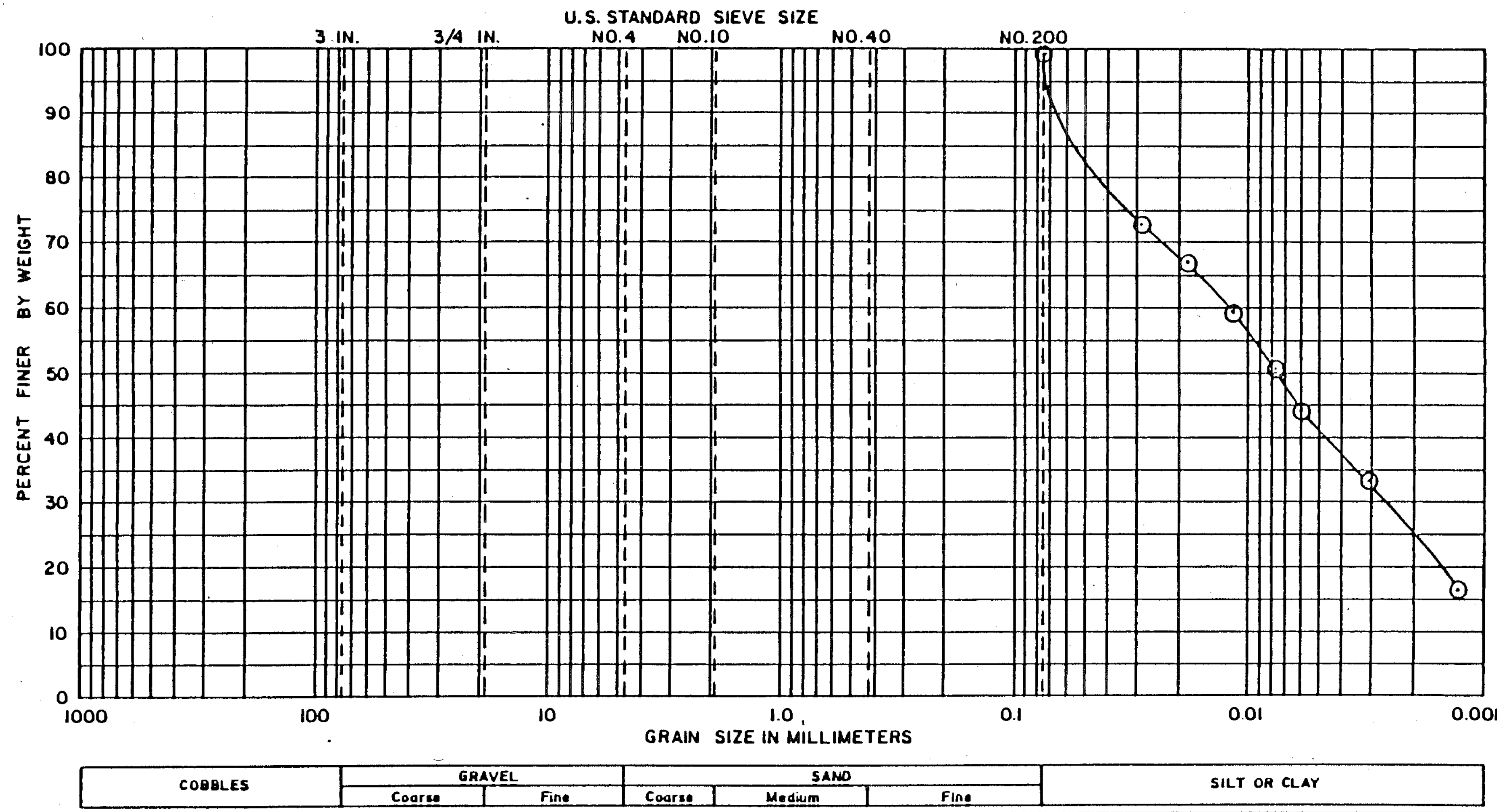
Project: DELAWARE SOLID WASTE FACILITY-2

Area: WILMINGTON, DELAWARE

Boring No: GF-6/ S-12

Date: 8/ 86 **Tested By:** AB

CLASSIFICATION TEST-GRADATION CURVES



Sample No.	Depth	Classification	Nat. WC	LL	PL	PI	Gs
GF-7/ ST-1	20- 22'	OH	91.3	76.5	47.9	28.6	
		oven-dried		53.4			

Description and Comments: Gray organic clay ; soil had a slightly fibrous texture

GANNETT FLEMING GEOTECHNICAL LABORATORY

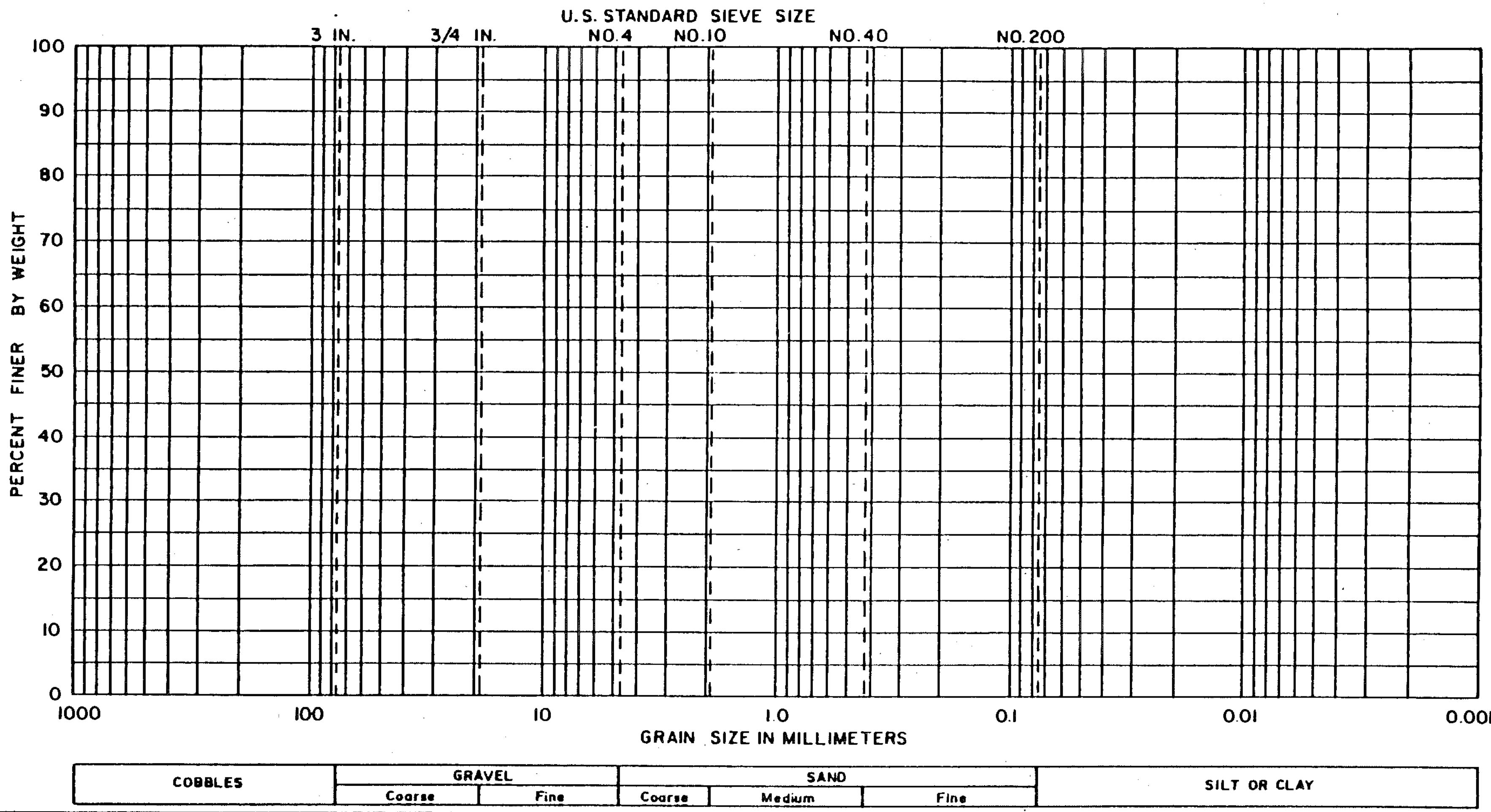
Project: DELAWARE SOLID WASTE FACILITY-2

Area: WILMINGTON, DELAWARE

Boring No: GF-7/ ST-1

Date: 8/ 86 Tested By: AB

CLASSIFICATION TEST-GRADATION CURVES



Sample No.	Depth	Classification	Nat. WC	LL	PL	PI	Gs
GF-7/S-20	---	MH	78.7	68.2	51.4	16.8	---
& 21		LL Oven-dried = 52.6					

Description and Comments: Dark Gray Elastic Silt.

$$\frac{\text{LL Oven-dried}}{\text{LL Air-dried}} = .772$$

CLASSIFICATION TEST-GRADATION CURVES

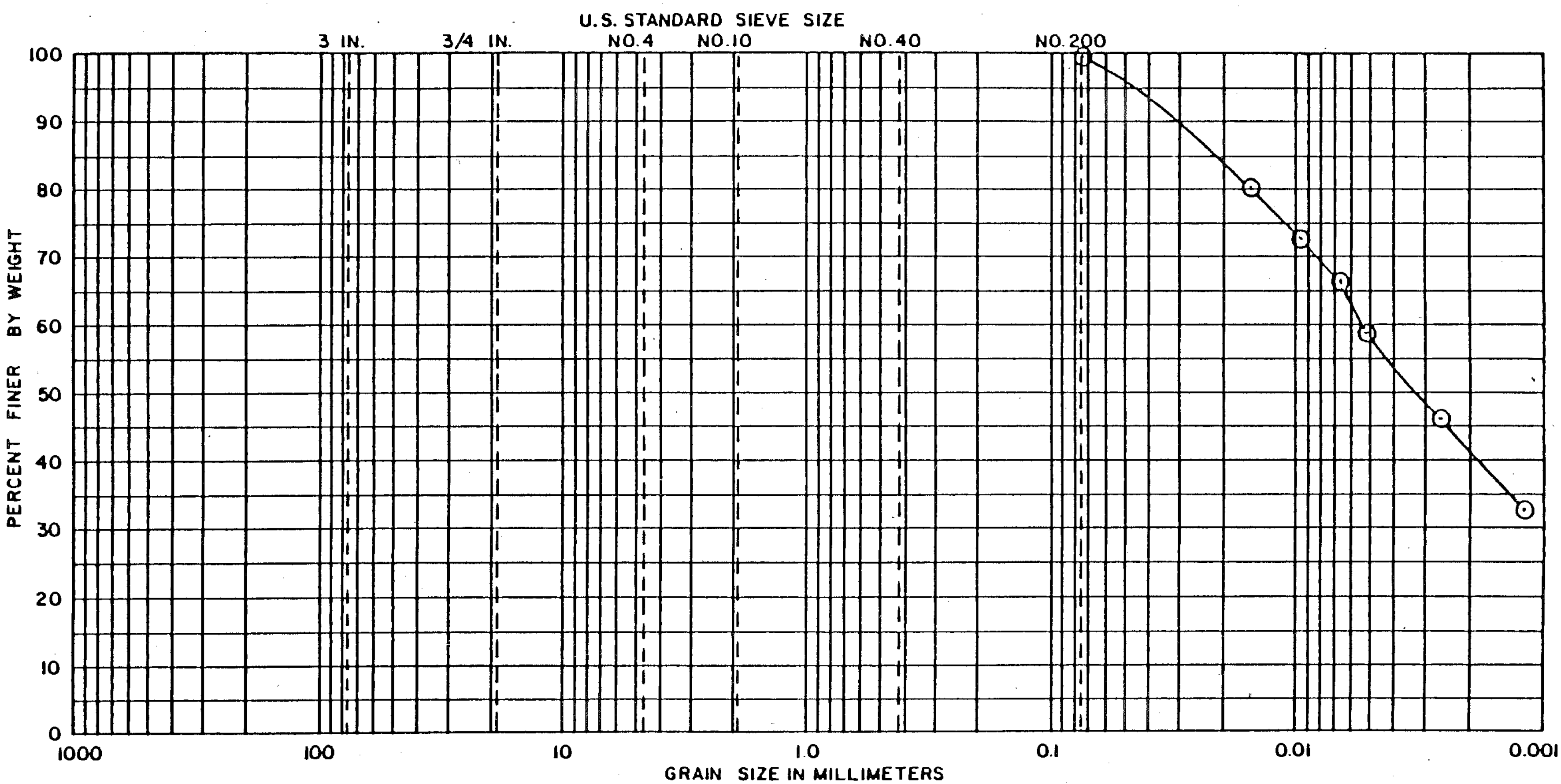
GANNETT FLEMING GEOTECHNICAL LABORATORY

Project: DELAWARE SOLID WASTE FACILITY - 2

Area: WILMINGTON, DE.

Boring No: GF-7 / S-20 & 21.

Date: 8/20/86 **Tested By:** Abdolos



COBBLES	GRAVEL		SAND			SILT OR CLAY
	Coarse	Fine	Coarse	Medium	Fine	

Sample No.	Depth	Classification	Nat. WC	LL	PL	PI	Gs
GF-8/ S-4	5- 7'	OH	118.4	74.6	48.6	26.0	

Description and Comments: Gray organic clay; soil had a slightly fibrous texture

CLASSIFICATION TEST-GRADATION CURVES

GANNETT FLEMING GEOTECHNICAL LABORATORY

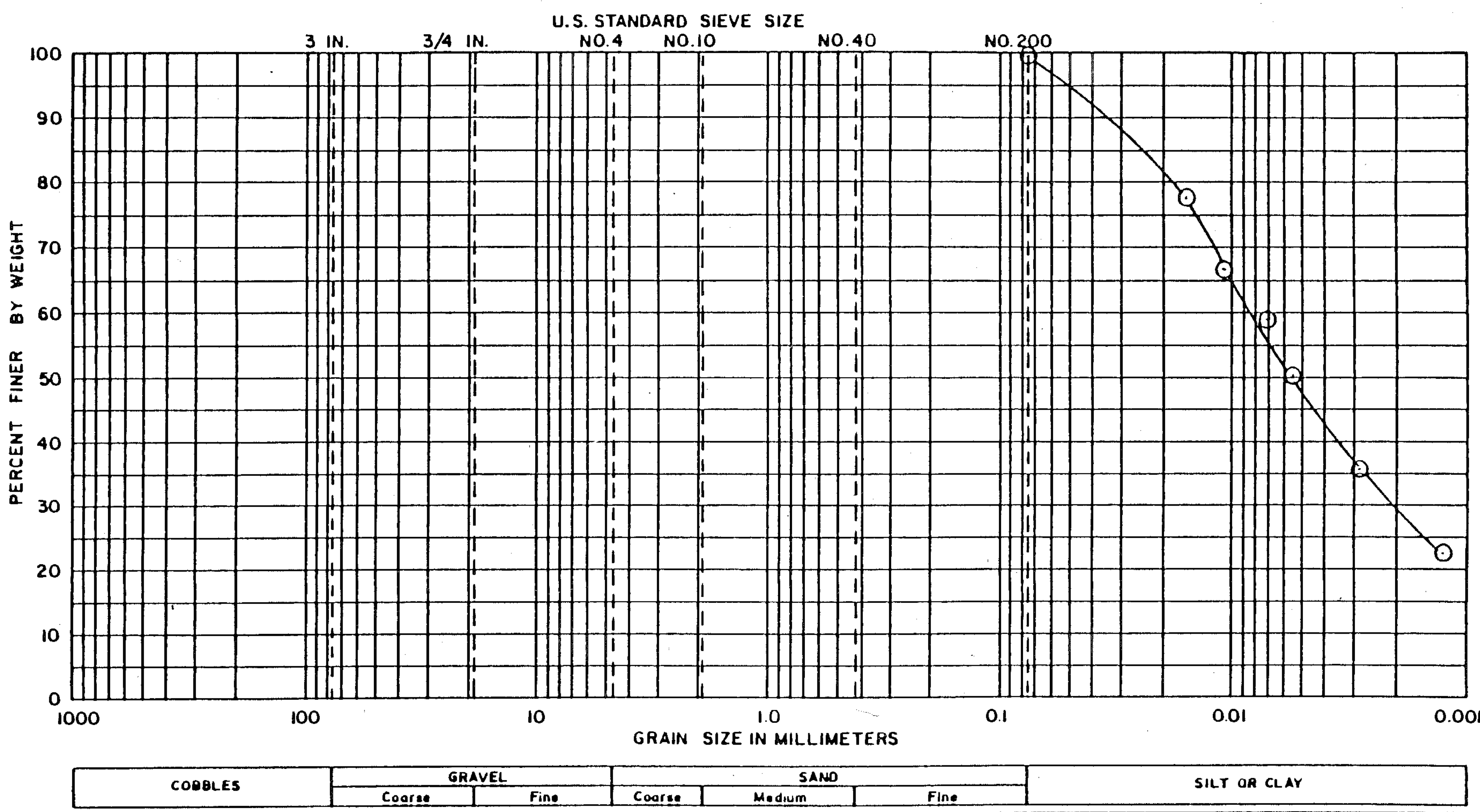
Project: DELAWARE SOLID WASTE FACILITY-2

Area: WILMINGTON, DELAWARE

Boring No: GF-8/ S-4

Date: 8/ 86

Tested By: AB



Sample No.	Depth	Classification	Nat. WC	LL	PL	PI	Gs
GF-8/S-13	25.0'-27.0'	MH	89.6	65.9	42.4	23.5	2.56
		LL Oven-dried =		54.1			

Description and Comments: Gray Elastic Silt. $\frac{\text{LL Oven-dried}}{\text{LL Air-dried}} = .795$

CLASSIFICATION TEST–GRADATION CURVES

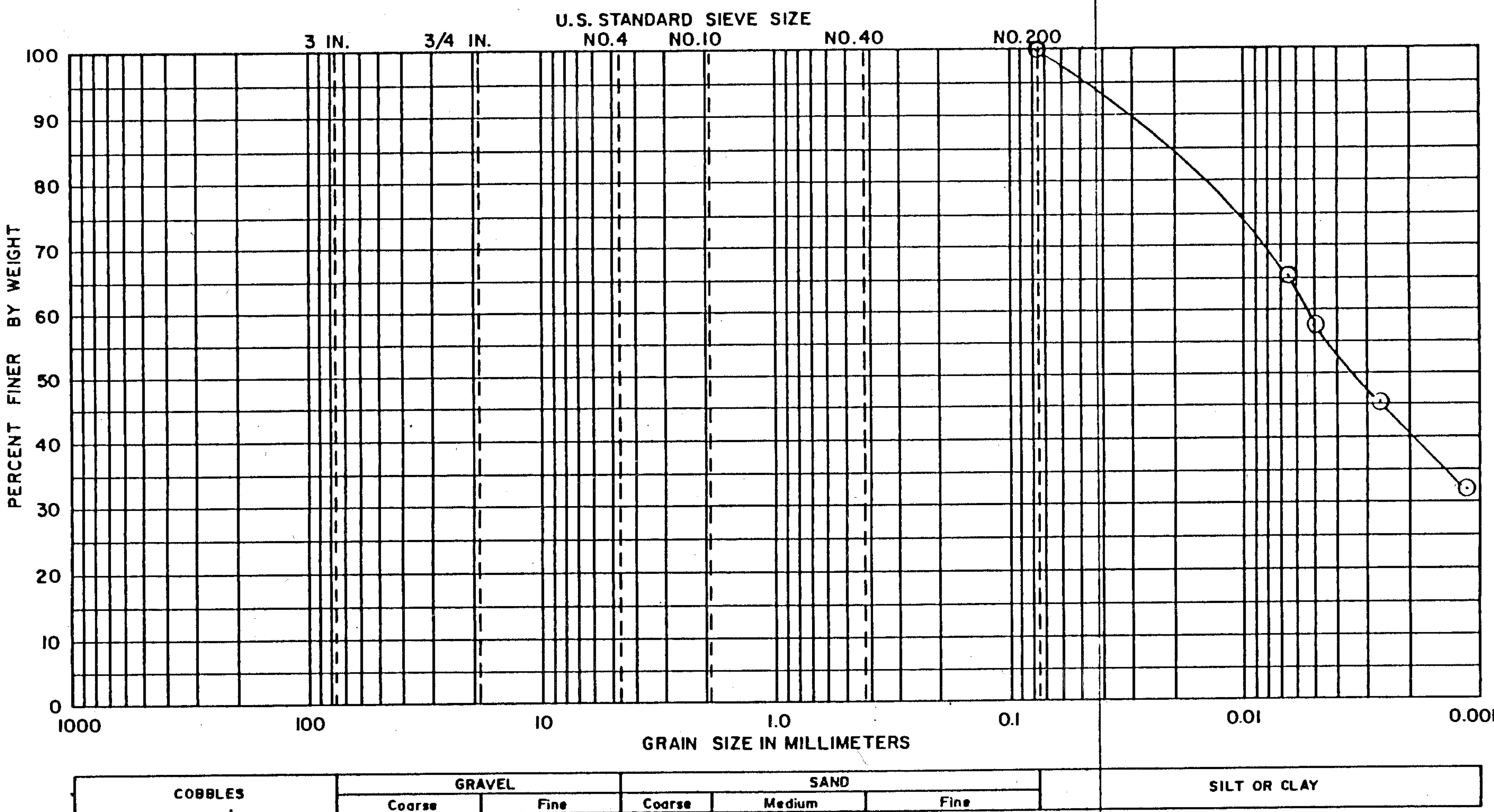
GANNETT FLEMING GEOTECHNICAL LABORATORY

Project: DELAWARE SOLID WASTE FACILITY - 2

Area: WILMINGTON, DE.

Boring No: GF-8 / S-13

Date: 8/11/86 **Tested By:** Abdolos



Sample No.	Depth	Classification	Nat. WC	LL	PL	PI	Gs
Bulk	0-2'	OH	86.2	76.6	48.4	28.2	--
		LL Oven-dried		54.3			

Description and Comments: Gray Organic Silt. $\frac{\text{LL Oven-dried}}{\text{LL Air-dried}} = 0.709$

CLASSIFICATION TEST-GRADATION CURVES

GANNETT FLEMING GEOTECHNICAL LABORATORY

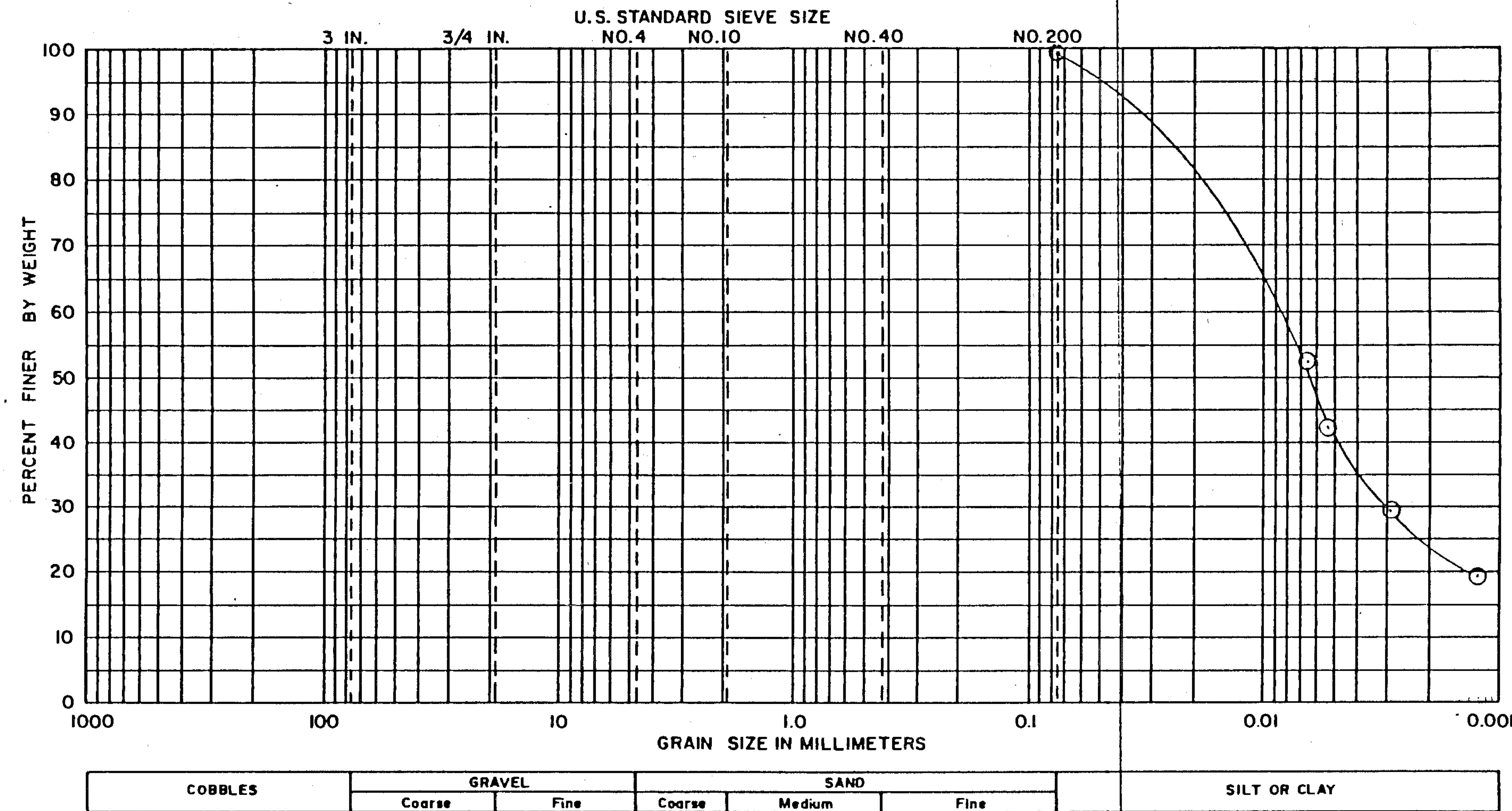
Project: DELAWARE SOLID WASTE FACILITY-2

Area: WILMINGTON, DELAWARE

Boring No: GF-10

Date: 9/ 86

Tested By: AB



Sample No.	Depth	Classification	Nat. WC	LL	PL	PI	Gs
Bulk	0- 2'	OH	82.5	78.7	53.4	25.3	--
		LL Oven-dried		57.3			

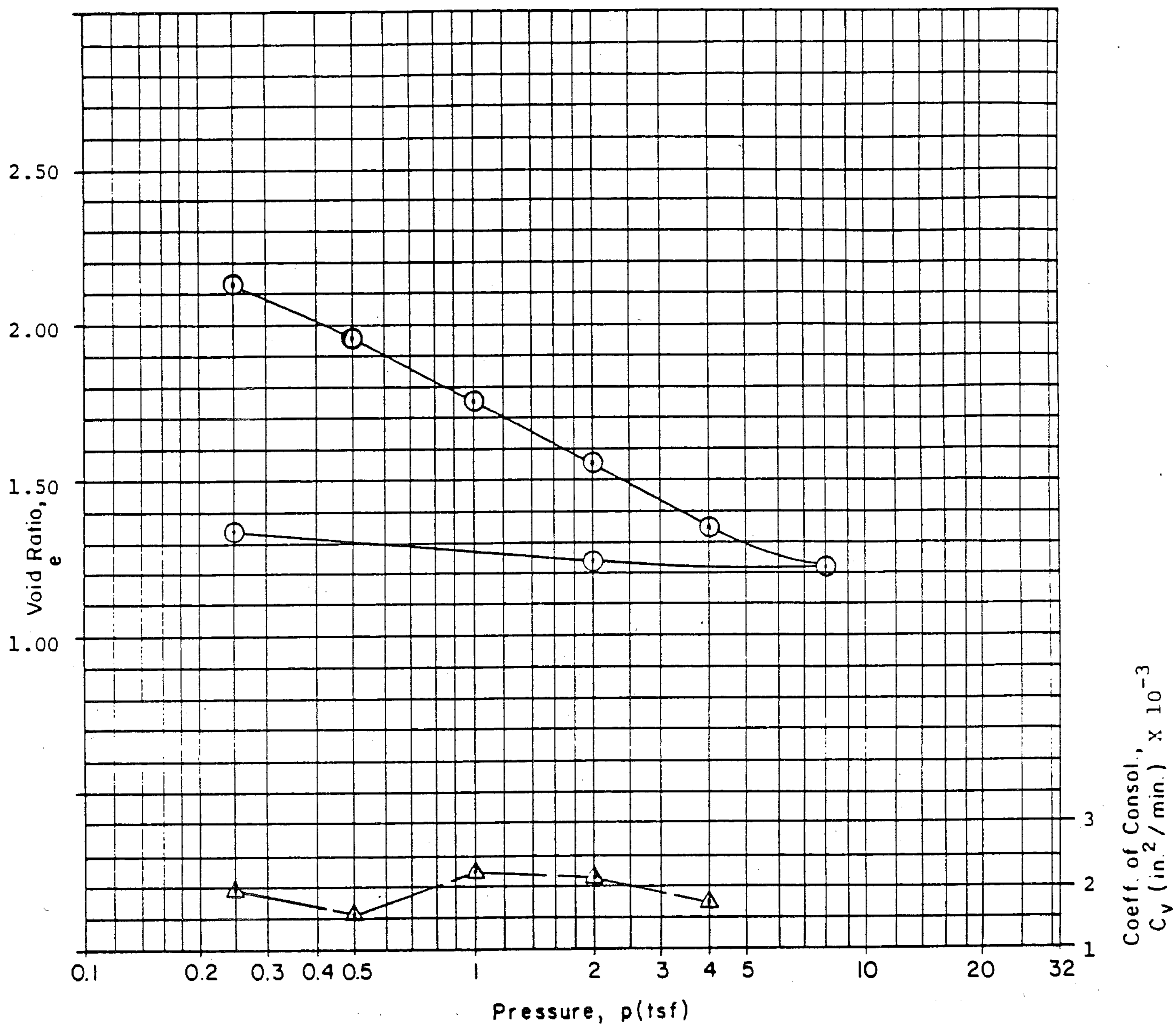
Description and Comments: Gray Organic Silt.

$$\frac{\text{LL Oven-dried}}{\text{LL Air-dried}} = 0.728$$

CLASSIFICATION TEST-GRADATION CURVES

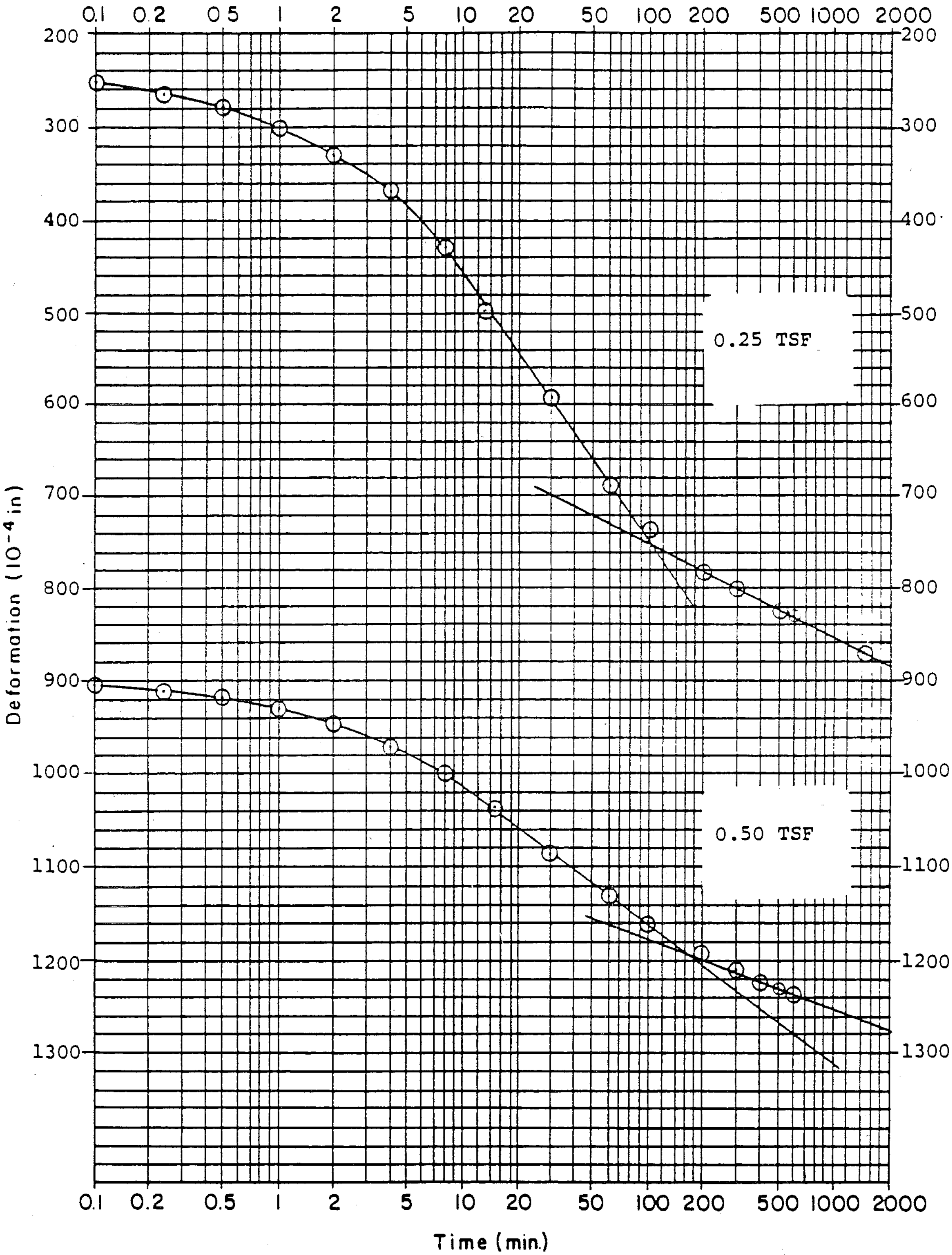
GANNETT FLEMING GEOTECHNICAL LABORATORY

Project:	DELAWARE SOLID WASTE FACILITY
Area:	WILMINGTON, DELAWARE
Boring No:	GF-11
Date:	9/ 86
Tested By:	AB



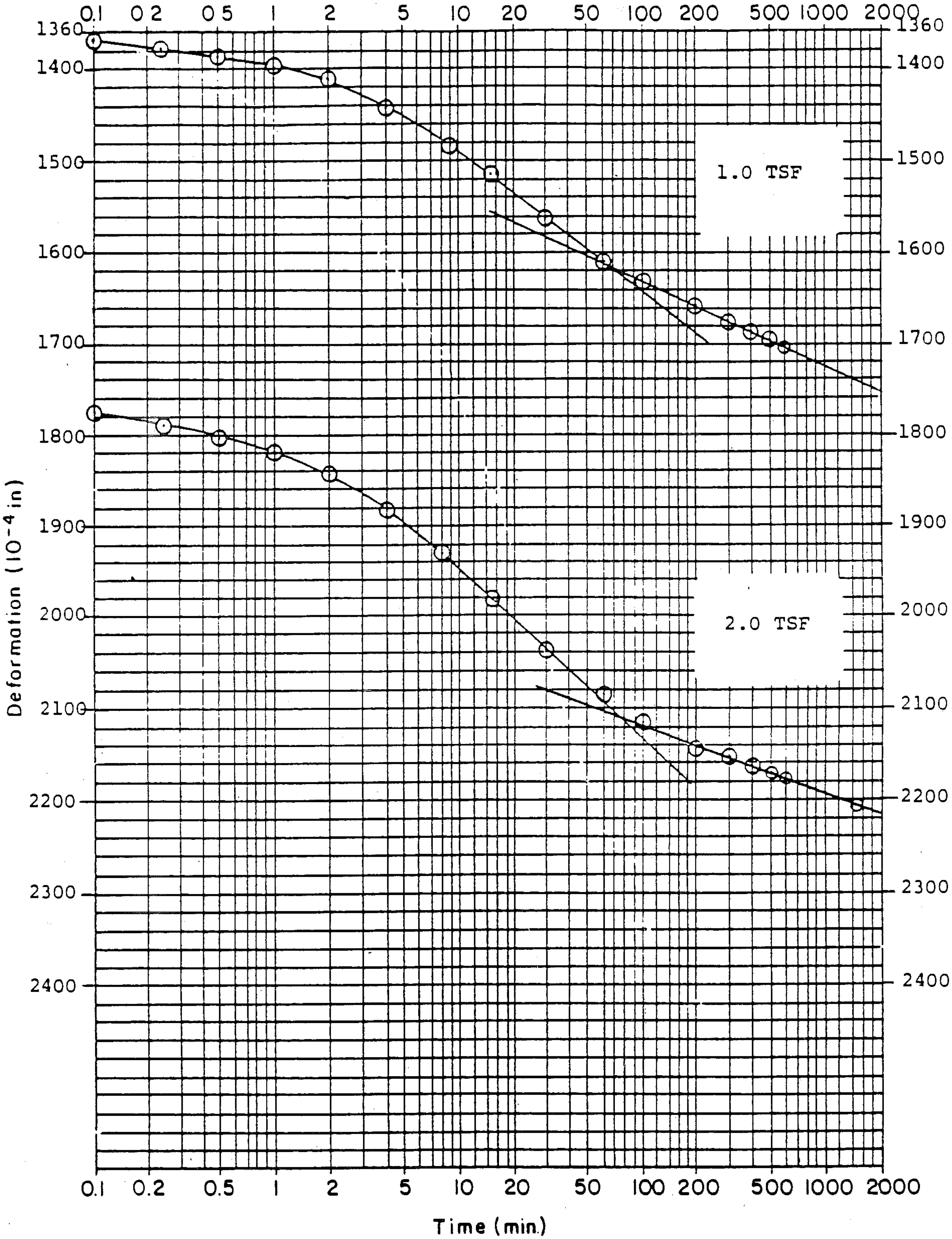
Type of Specimen	Shelby			Before Test			After Test	
Dia	2.50 in	H_T	0.75 in	Water Content	w_o	82.3	w_f	52.4
Compression Index		C_c	0.67	Void Ratio	e_o	2.429	e_f	1.341
Classification	MH			Saturation	S_o	96	S_f	99
w_l	62.4	I_p	16.1	Project	DSWF NORTHERN FACILITY-2			
w_p	46.3	LI	2.2	Boring No	GF-1	Sample No	S-7	
Remarks	Exceeded machine travel limits at 8 TSF.			Depth	10- 12'	Date	8/ 86	

GANNETT FLEMING GEOTECHNICAL LABORATORY
CONSOLIDATION TEST REPORT



CONSOLIDATION TEST-TIME CURVES			
Project	DELAWARE SOLID WASTE FACILITY - 2	Boring No	GF-1
Sample No S-7	Depth 10.0' - 12.0'	Date	Aug. 1986

GANNETT FLEMING GEOTECHNICAL LABORATORY

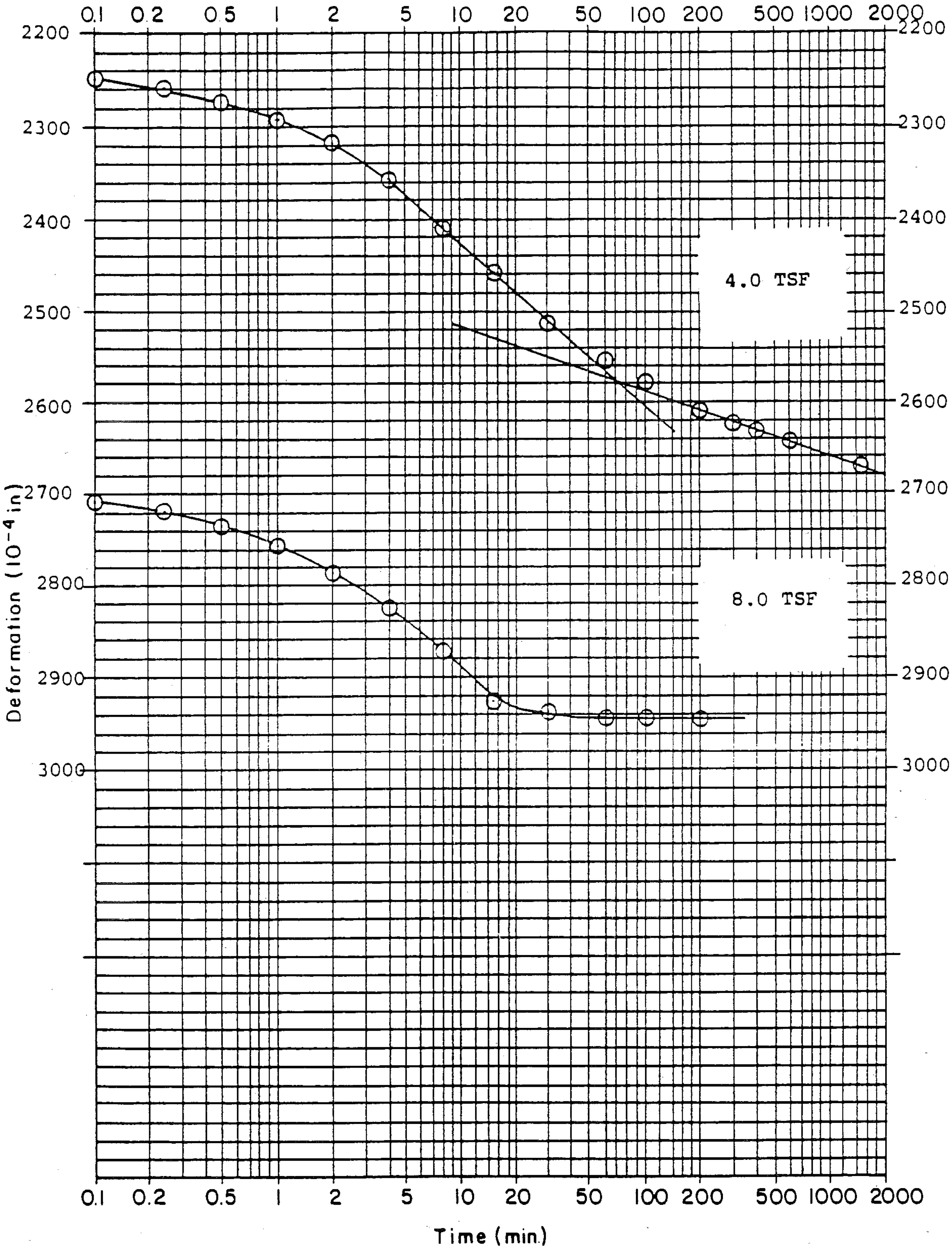


CONSOLIDATION TEST-TIME CURVES

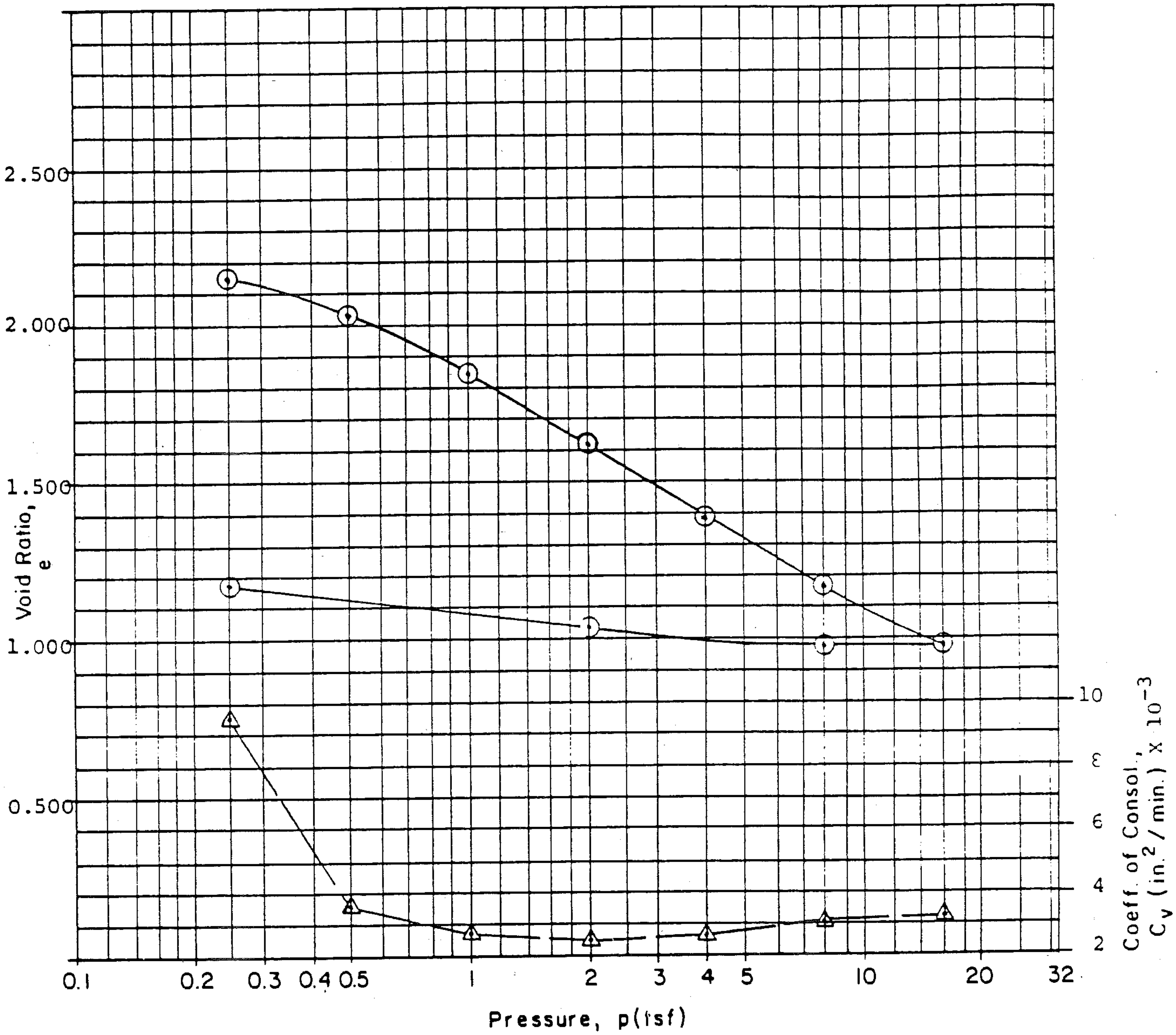
Project DELAWARE SOLID WASTE FACILITY - 2 Boring No GF-1

Sample No S-7 Depth 10.0' - 12.0' Date Aug. 1986

GANNETT FLEMING GEOTECHNICAL LABORATORY

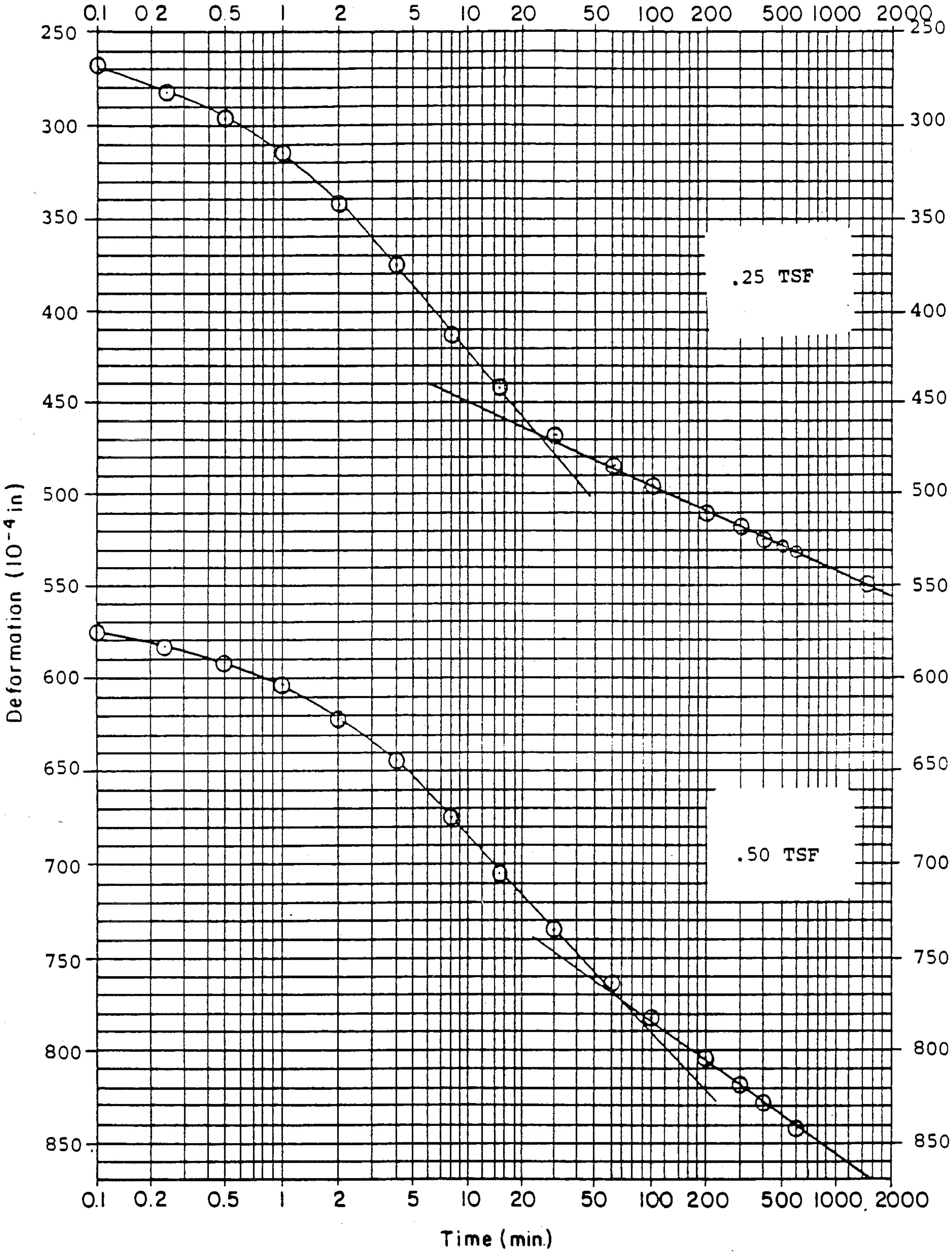


CONSOLIDATION TEST-TIME CURVES			
Project DELAWARE SOLID WASTE FACILITY - 2		Boring No GF-1	
Sample No S-7	Depth 10.0' - 12.0'	Date Aug. 1986	
GANNETT FLEMING GEOTECHNICAL LABORATORY			



Type of Specimen	Shelby			Before Test			After Test	
Dia	2.50 in	H_T	0.75 in	Water Content	w_o	79.0	w_f	46.5
Compression Index		C_c	0.73	Void Ratio	e_o	2.271	e_f	1.178
Classification		MH		Saturation	S_o	97.6	S_f	100
w_l	62.8	I_p	18.1	Project	DSWA NORTHERN FACILITY-2			
w_p	44.7	LI	1.9	Boring No	GF-1		Sample No	S-10
Remarks	Results questionable beyond 8 TSF due to specimen's extreme compressibility. Exceeded machine travel limits at 32 TSF.			Depth	20- 22'		Date	8/ 86

GANNETT FLEMING GEOTECHNICAL LABORATORY
CONSOLIDATION TEST REPORT

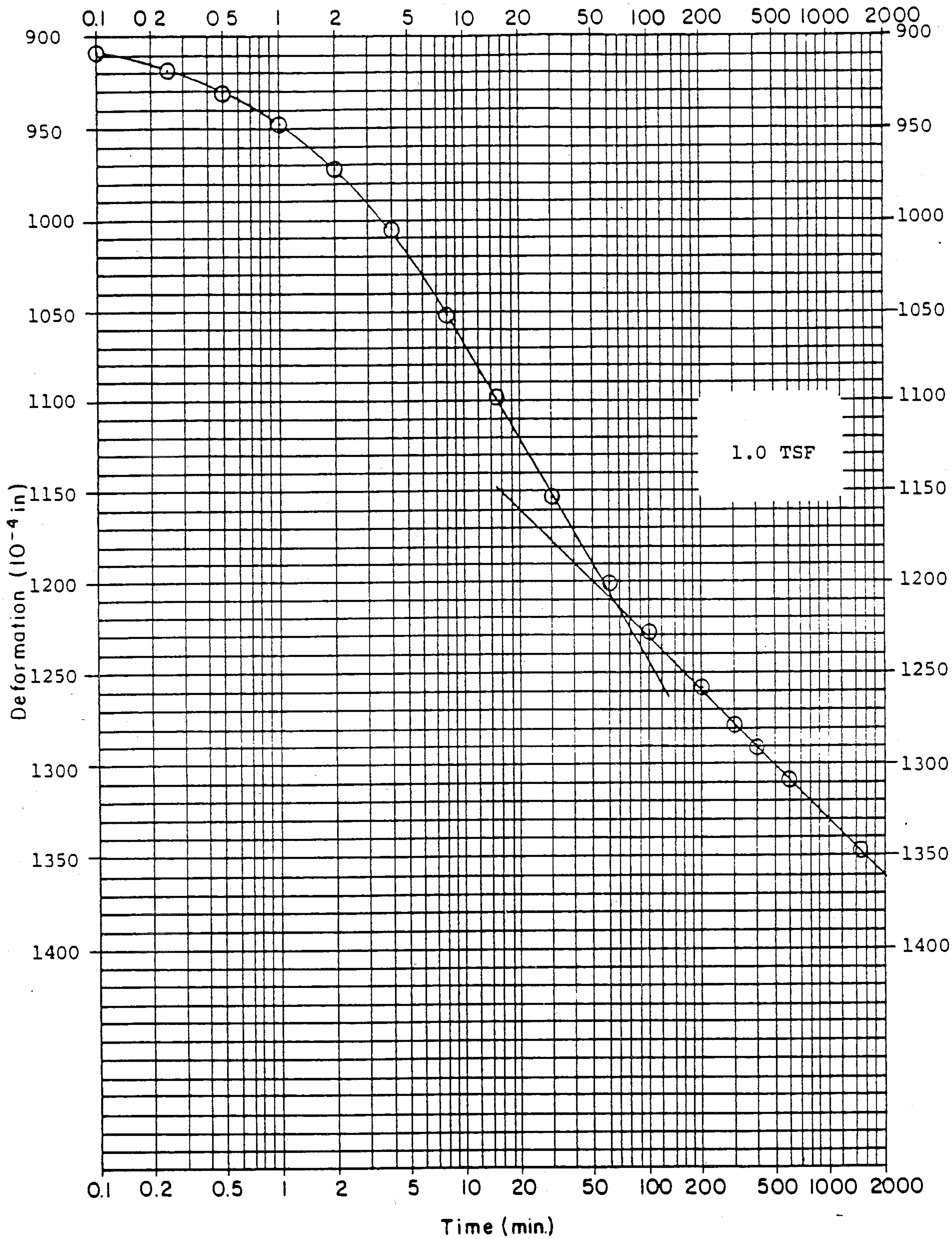


CONSOLIDATION TEST-TIME CURVES

Project DELAWARE SOLID WASTE FACILITY - 2 Boring No GF - 1

Sample No S-10 Depth 20.0' - 22.0' Date Aug. 1986

GANNETT FLEMING GEOTECHNICAL LABORATORY

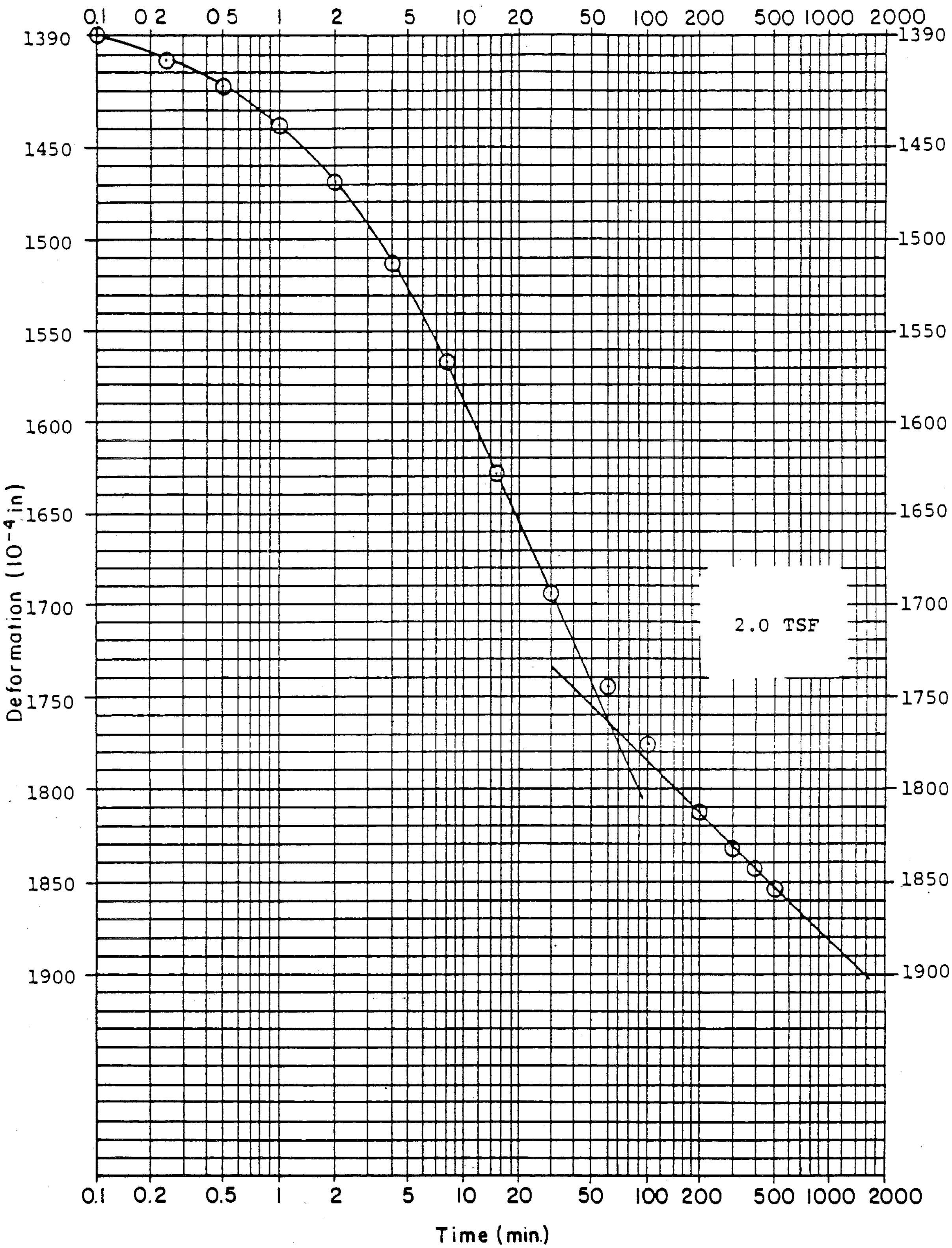


CONSOLIDATION TEST-TIME CURVES

Project DELAWARE SOLID WASTE FACILITY - 2 Boring No GF-1

Sample No S-10 Depth 20.0' - 22.0' Date Aug. 1986

GANNETT FLEMING GEOTECHNICAL LABORATORY

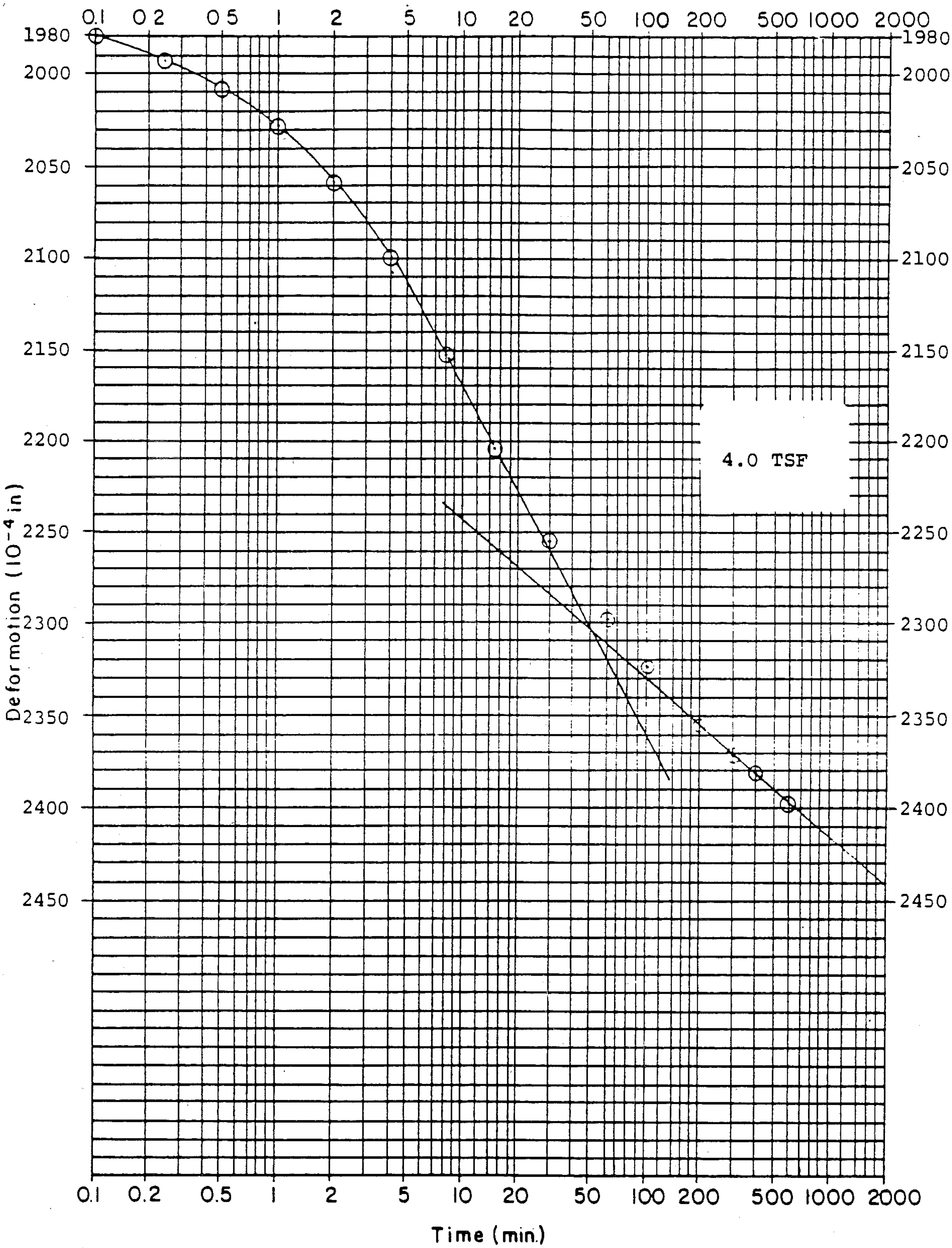


CONSOLIDATION TEST-TIME CURVES

Project DELAWARE SOLID WASTE FACILITY - 2 Boring No GF-1

Sample No S-10 Depth 20.0' - 22.0' Date Aug. 1986

GANNETT FLEMING GEOTECHNICAL LABORATORY

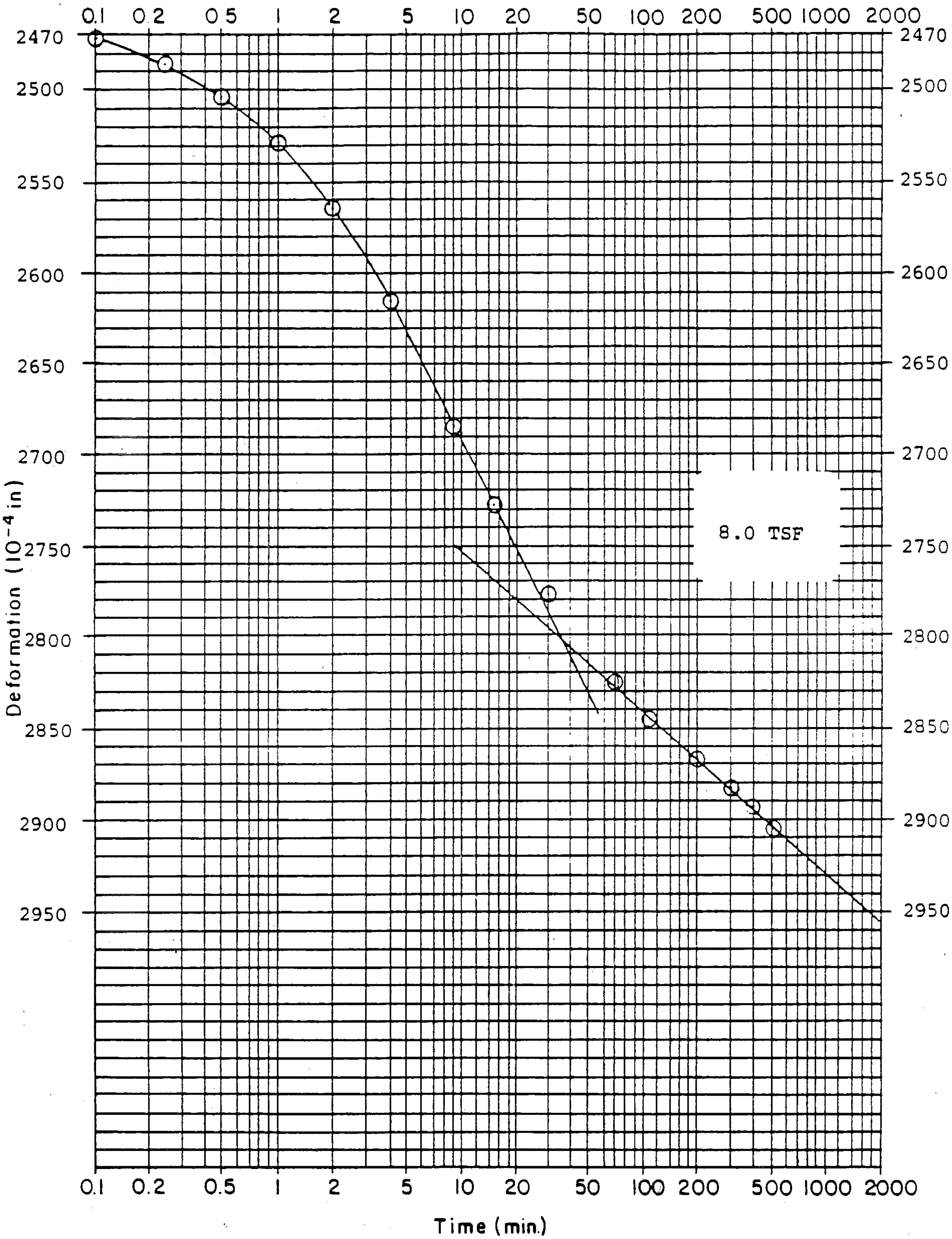


CONSOLIDATION TEST-TIME CURVES

Project DELAWARE SOLID WASTE FACILITY - 2 Boring No GF-1

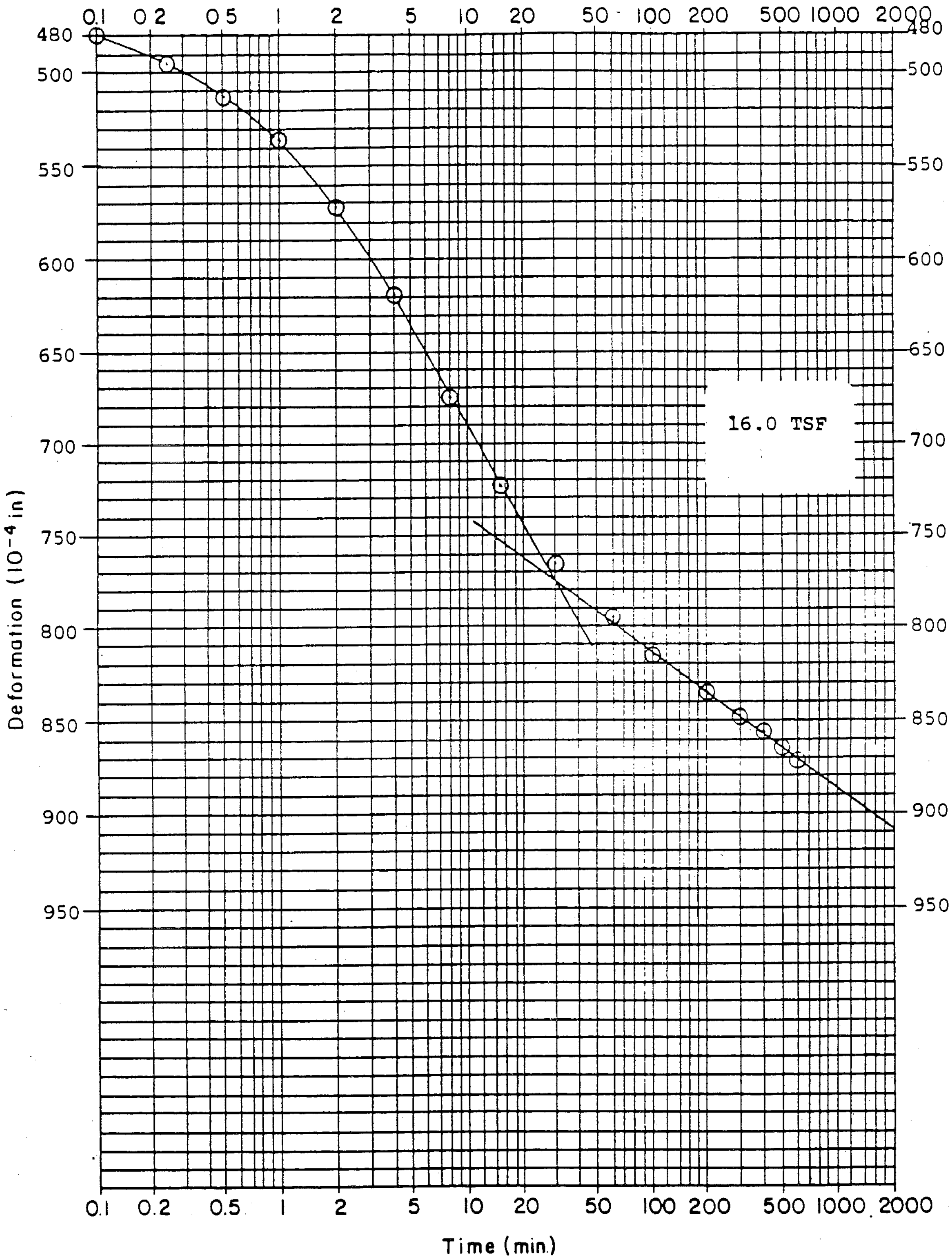
Sample No S-10 Depth 20.0' - 22.0' Date Aug. 1986

GANNETT FLEMING GEOTECHNICAL LABORATORY



CONSOLIDATION TEST-TIME CURVES			
Project DELAWARE SOLID WASTE FACILITY - 2		Boring No GF-1	
Sample No S-10	Depth 20.0' - 22.0'	Date Aug. 1986	

GANNETT FLEMING GEOTECHNICAL LABORATORY

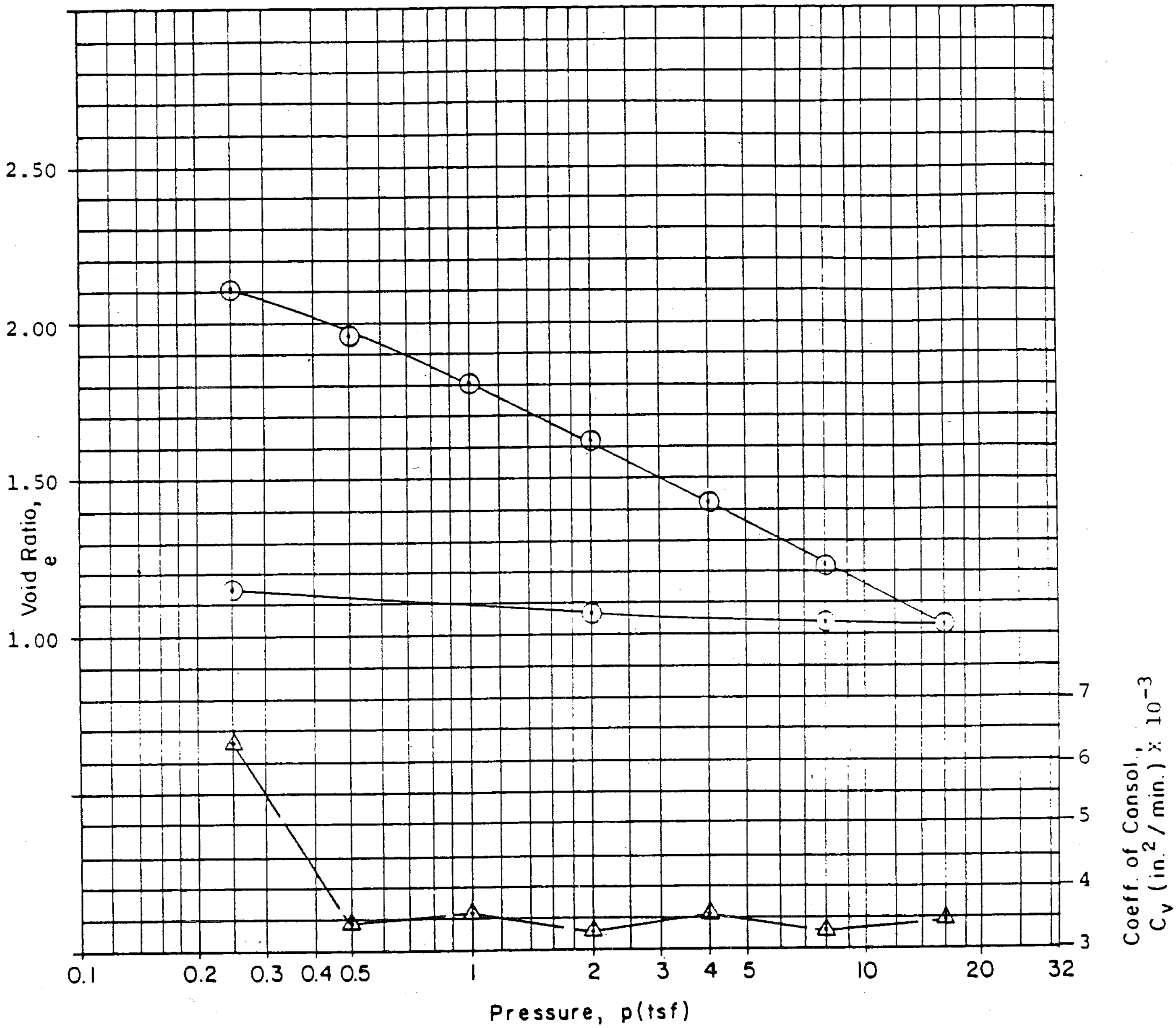


CONSOLIDATION TEST-TIME CURVES

Project DELAWARE SOLID WASTE FACILITY - 2 Boring No GF-1

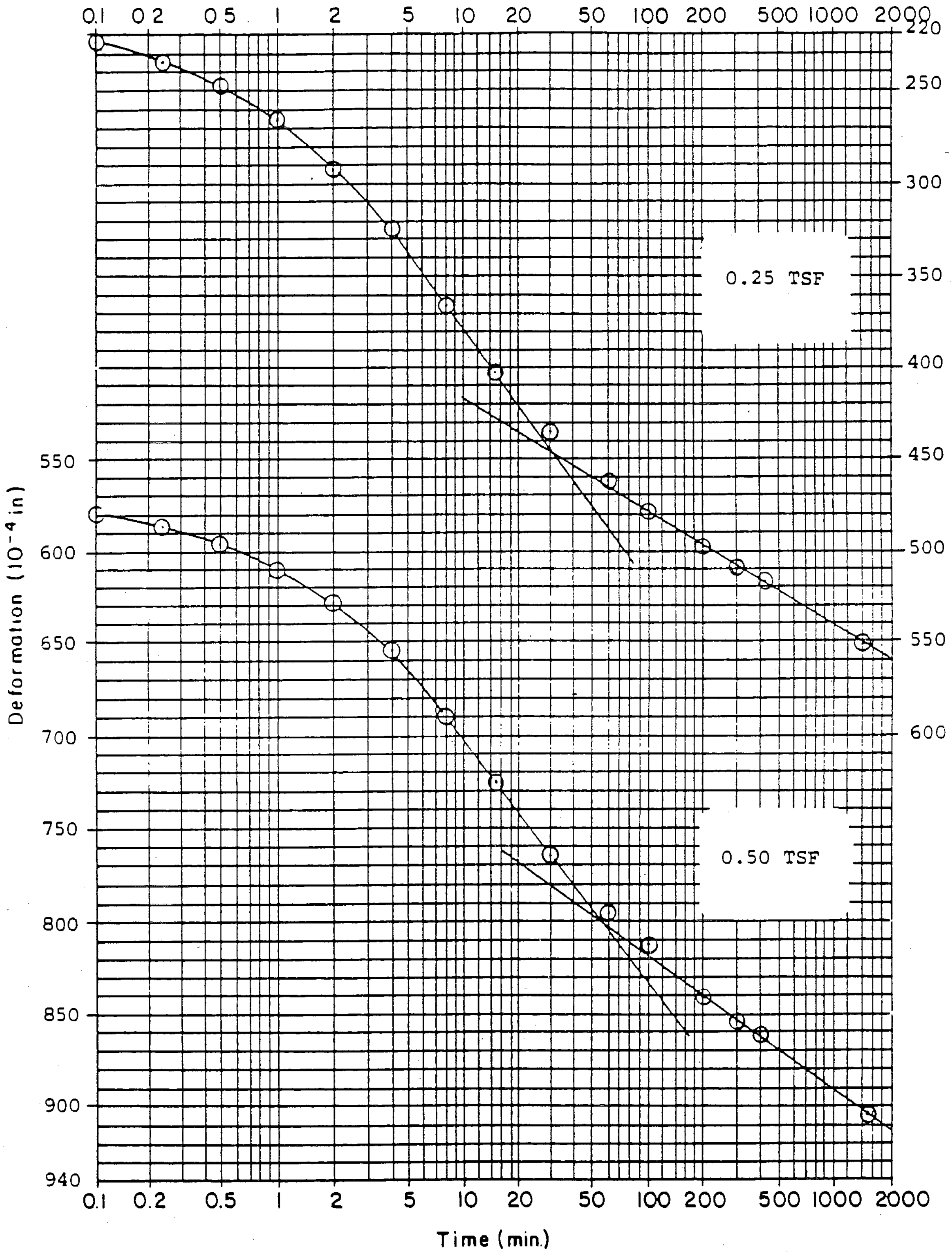
Sample No S-10 Depth 20.0' - 22.0' Date Aug. 1986

GANNETT FLEMING GEOTECHNICAL LABORATORY



Type of Specimen	Shelby				Before Test		After Test
Dia 2.50 in	H_T	0.75 in	Water Content	w_o	84.9	w_f	45.3
Compression Index	C_c	0.64	Void Ratio	e_o	2.261	e_f	1.150
Classification	OH		Saturation	S_o	99	S_f	100
w_l 72.4	I_p	26.0	Project DSWA NORTHERN FACILITY-2				
w_p 46.4	LI	1.5	Boring No	GF-7		Sample No	Tube #1
Remarks Exceeded machine			Depth	10- 12'		Date	8/ 86
travel limits at 32 TSF.							

GANNETT FLEMING GEOTECHNICAL LABORATORY
CONSOLIDATION TEST REPORT

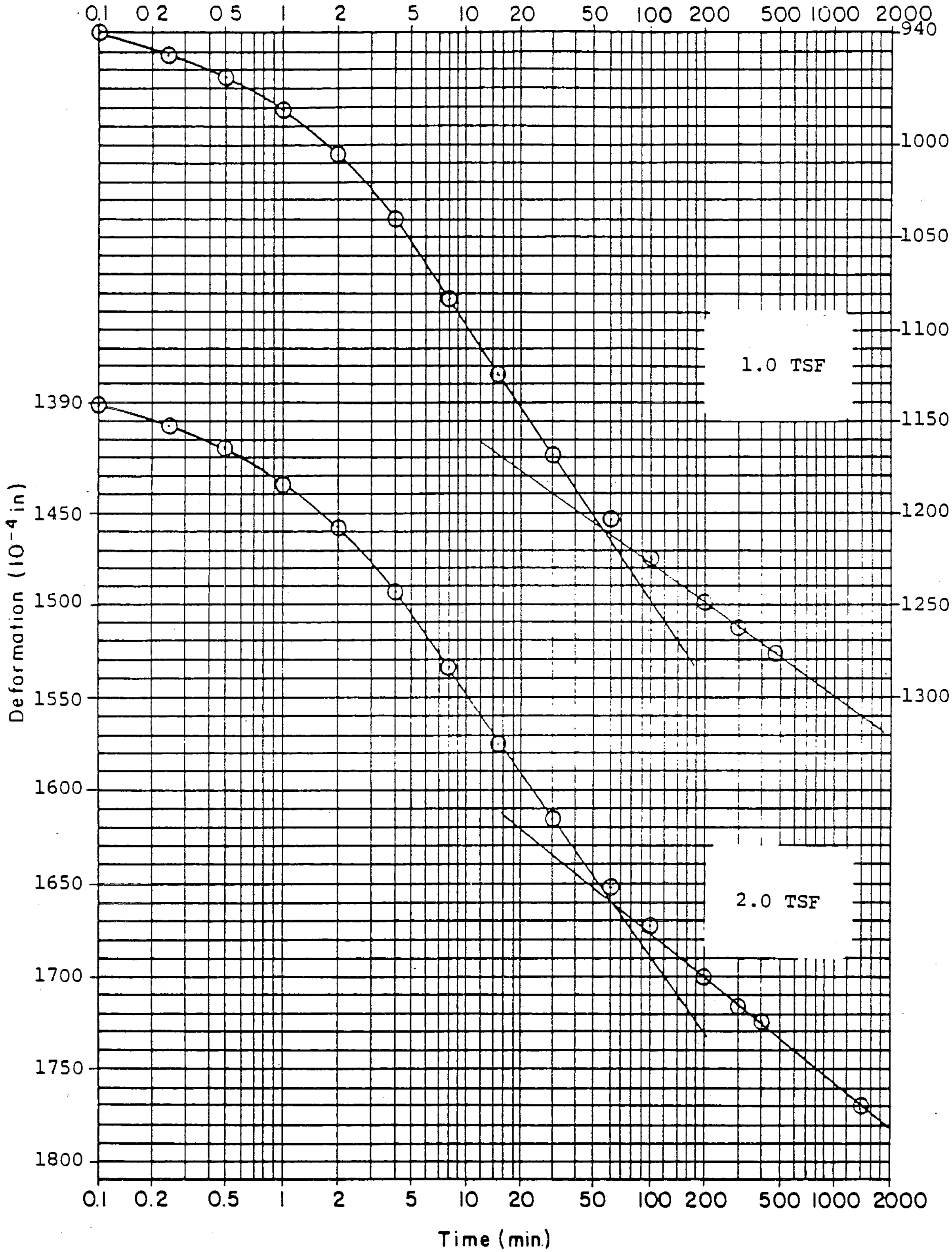


CONSOLIDATION TEST-TIME CURVES

Project DELAWARE SOLID WASTE FACILITY - 2 Boring No GF - 7

Sample No Tube - 1 Depth 10.0' - 12.0 ' Date Aug. 1986

GANNETT FLEMING GEOTECHNICAL LABORATORY

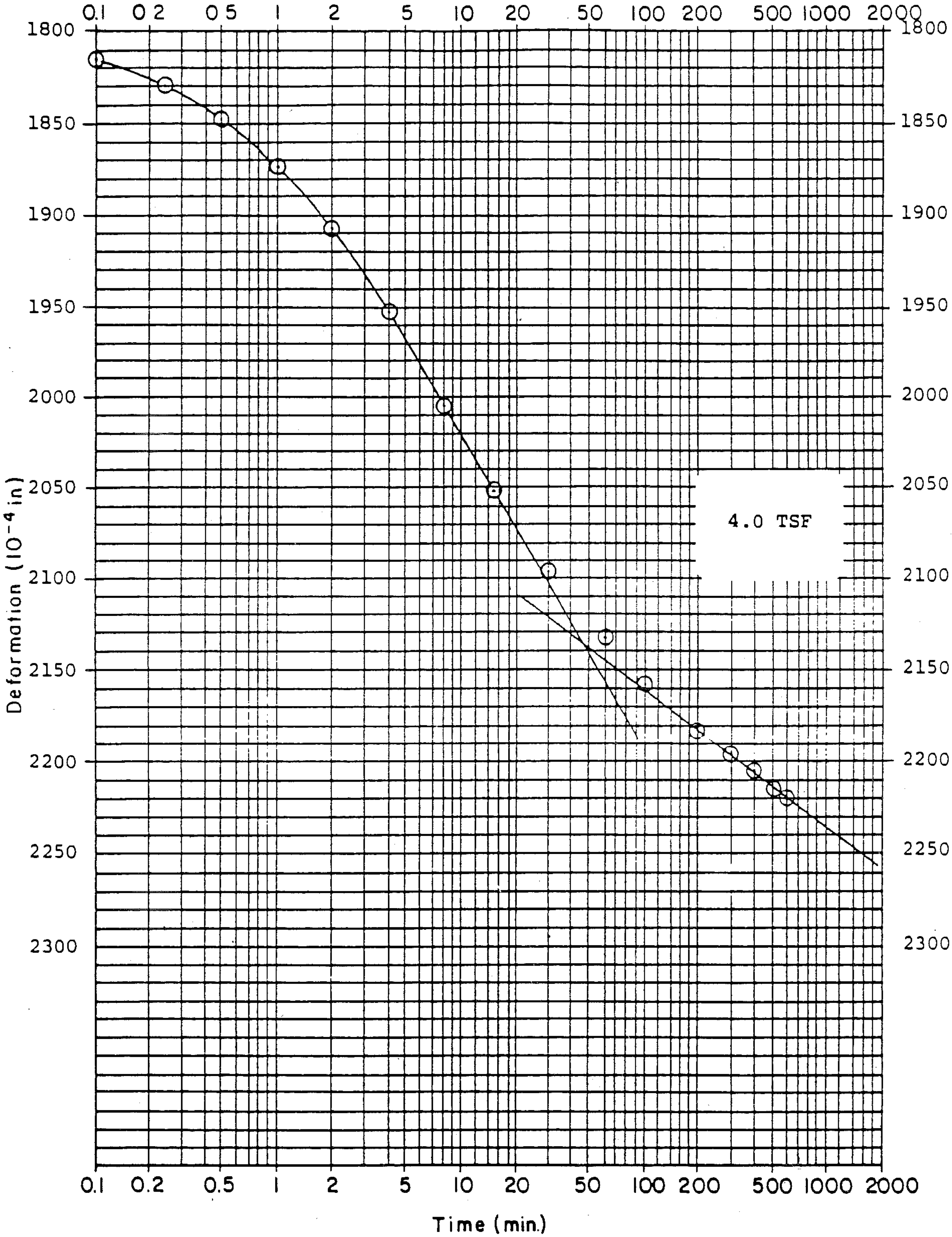


CONSOLIDATION TEST-TIME CURVES

Project DELAWARE SOLID WASTE FACILITY - 2 Boring No GF - 7

Sample No Tube - 1 Depth 10.0' - 12.0' Date Aug. 1986

GANNETT FLEMING GEOTECHNICAL LABORATORY

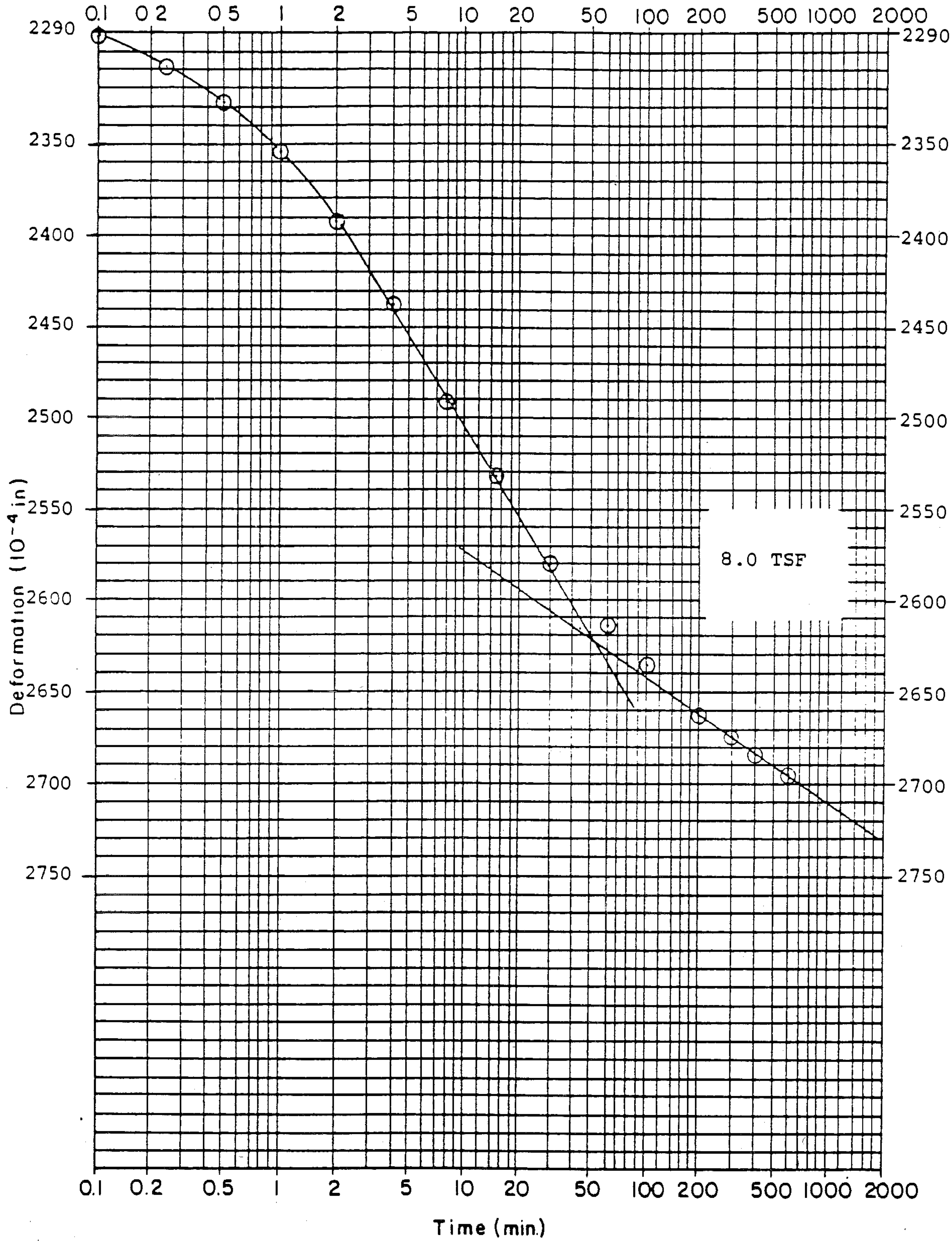


CONSOLIDATION TEST-TIME CURVES

Project DELAWARE SOLID WASTE FACILITY - 2 Boring No GF-7

Sample No Tube - 1 Depth 10.0' - 12.0' Date Aug. 1986

GANNETT FLEMING GEOTECHNICAL LABORATORY

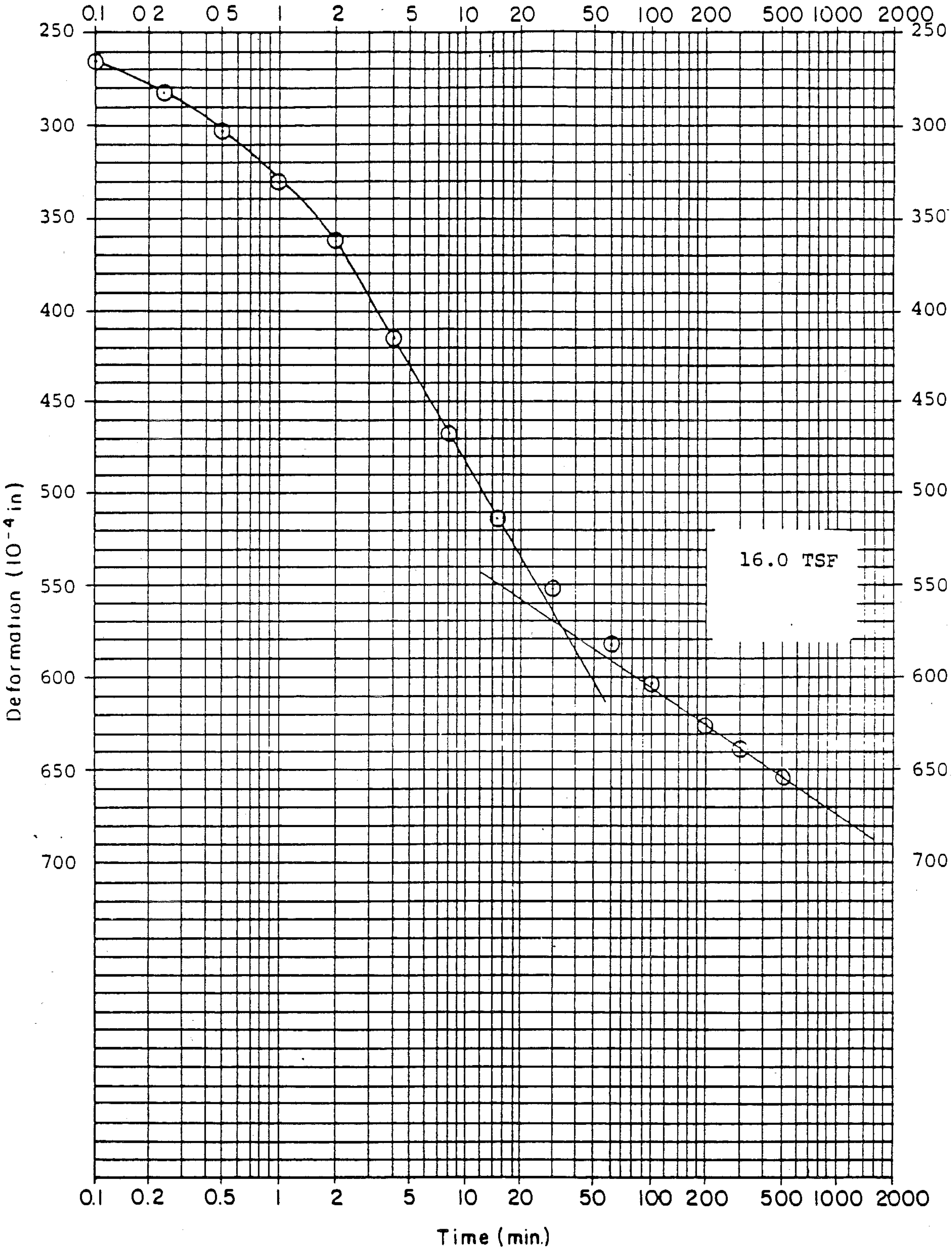


CONSOLIDATION TEST-TIME CURVES

Project DELAWARE SOLID WASTE FACILITY - 2 Boring No GF-7

Sample No Tube - 1 Depth 10.0' - 12.0' Date Aug. 1986

GANNETT FLEMING GEOTECHNICAL LABORATORY

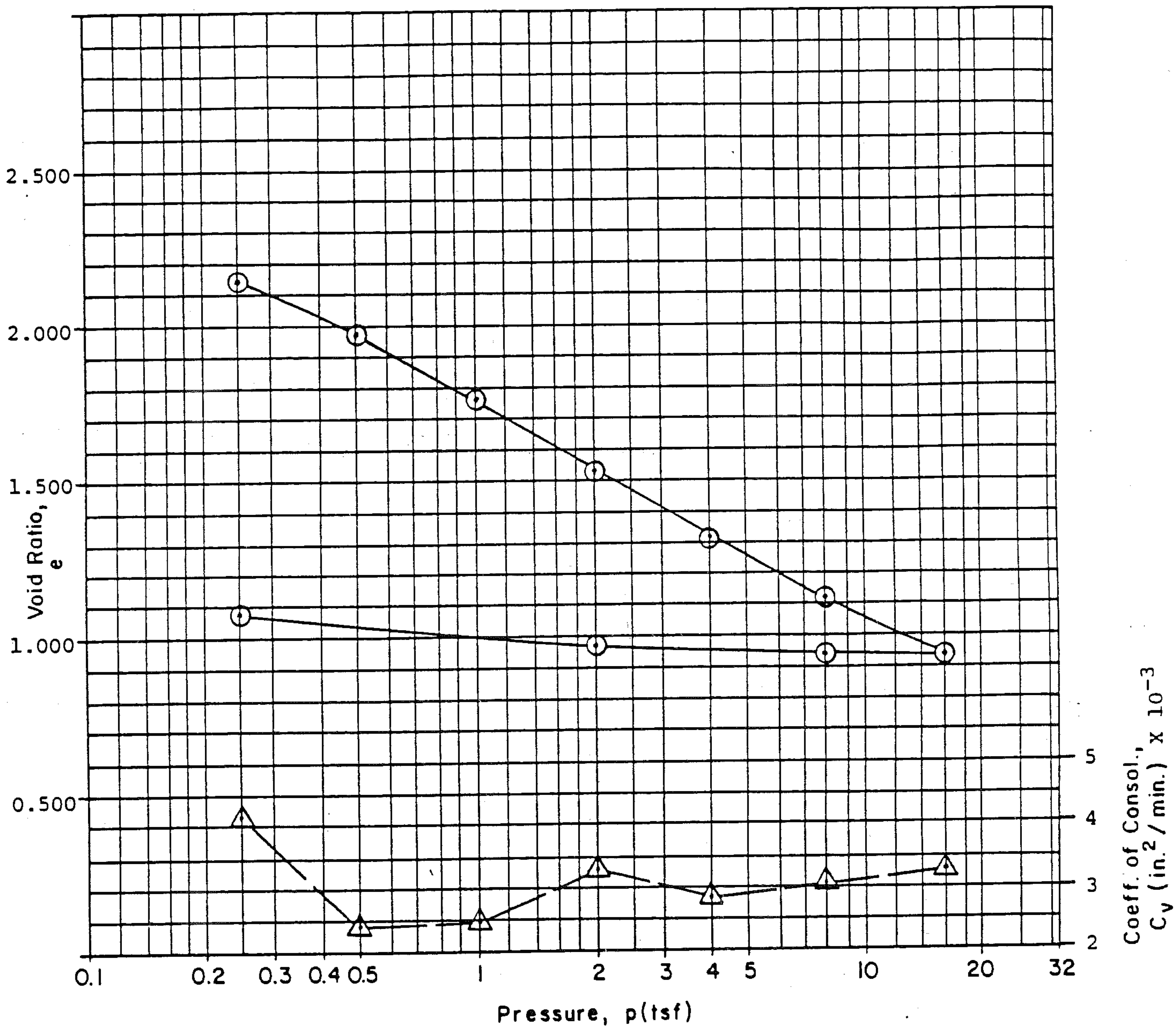


CONSOLIDATION TEST-TIME CURVES

Project DELAWARE SOLID WASTE FACILITY - 2 Boring No GF-7

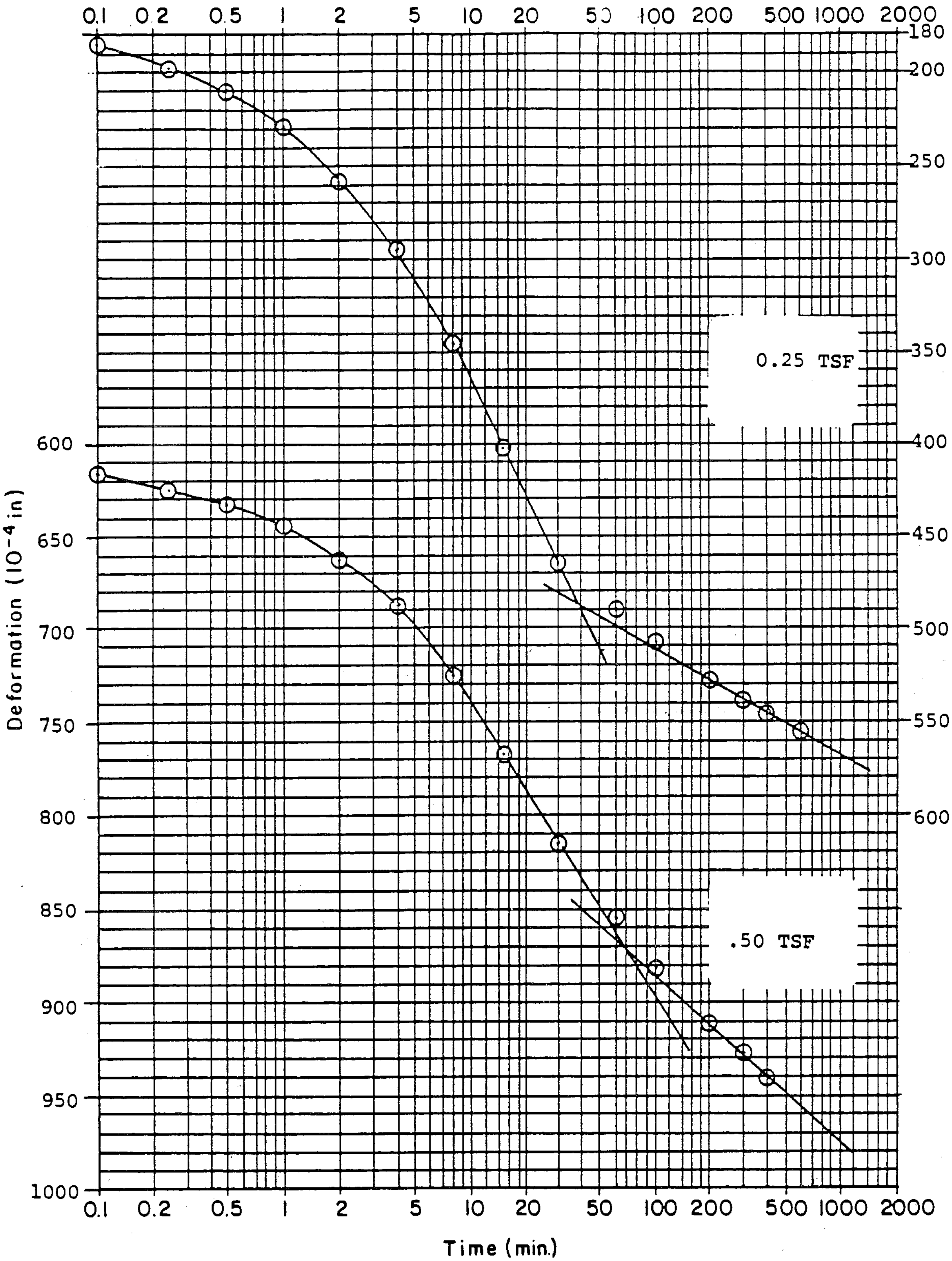
Sample No Tube - 1 Depth 10.0' - 12.0' Date Aug. 1986

GANNETT FLEMING GEOTECHNICAL LABORATORY



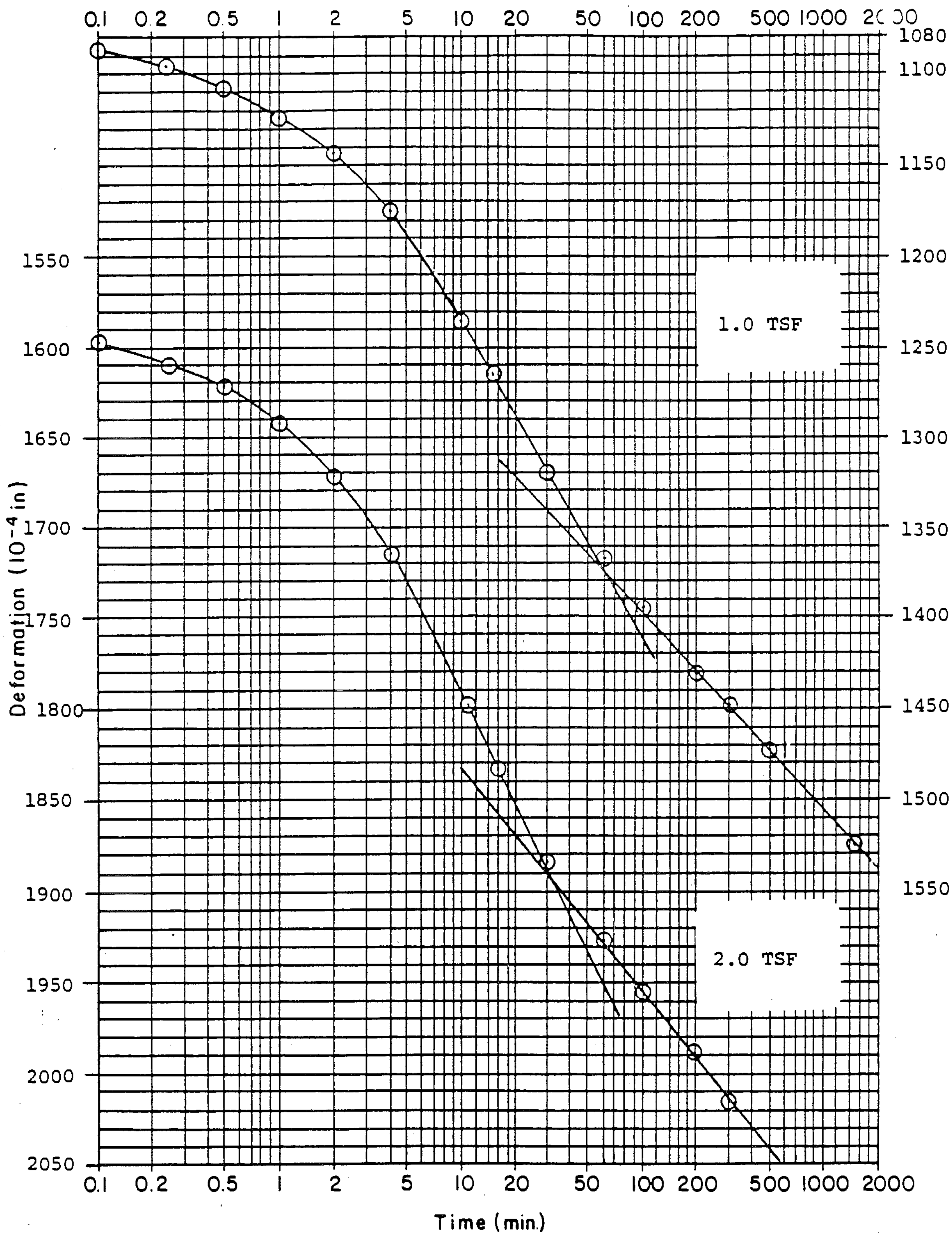
Type of Specimen	Shelby		Before Test			After Test	
Dia 2.50 in	H_T 0.75 in		Water Content	w_o	88.4	w_f	43.4
Compression Index	C_c	0.72	Void Ratio	e_o	2.322	e_f	1.085
Classification	OH		Saturation	S_o	97.1	S_f	100
w_l 76.5	I_p 28.6		Project	DSWA NORTHERN FACILITY-2			
w_p 47.9	LI 1.4		Boring No	GF-7		Sample No	ST-1
Remarks Results questionable beyond 8 TSF due to specimen's extreme compressibility. Exceeded machine travel limits at 32 TSF.			Depth	20-22'		Date	9/ 86

GANNETT FLEMING GEOTECHNICAL LABORATORY
CONSOLIDATION TEST REPORT



CONSOLIDATION TEST-TIME CURVES			
Project	DELAWARE SOLID WASTE FACILITY - 2	Boring No	GF-7
Sample No ST-1	Depth 20.0' - 22.0'	Date	Aug. 1986

GANNETT FLEMING GEOTECHNICAL LABORATORY

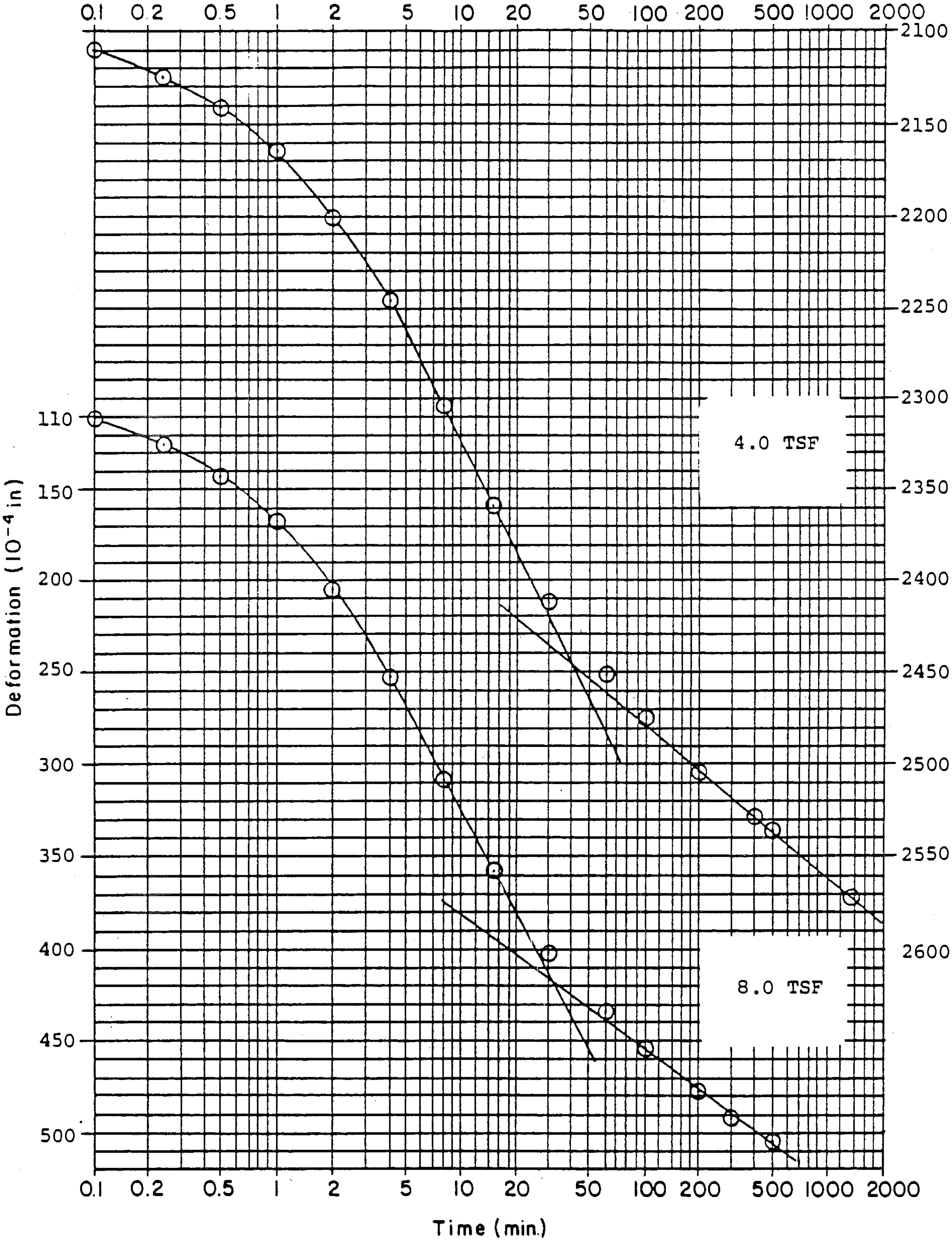


CONSOLIDATION TEST-TIME CURVES

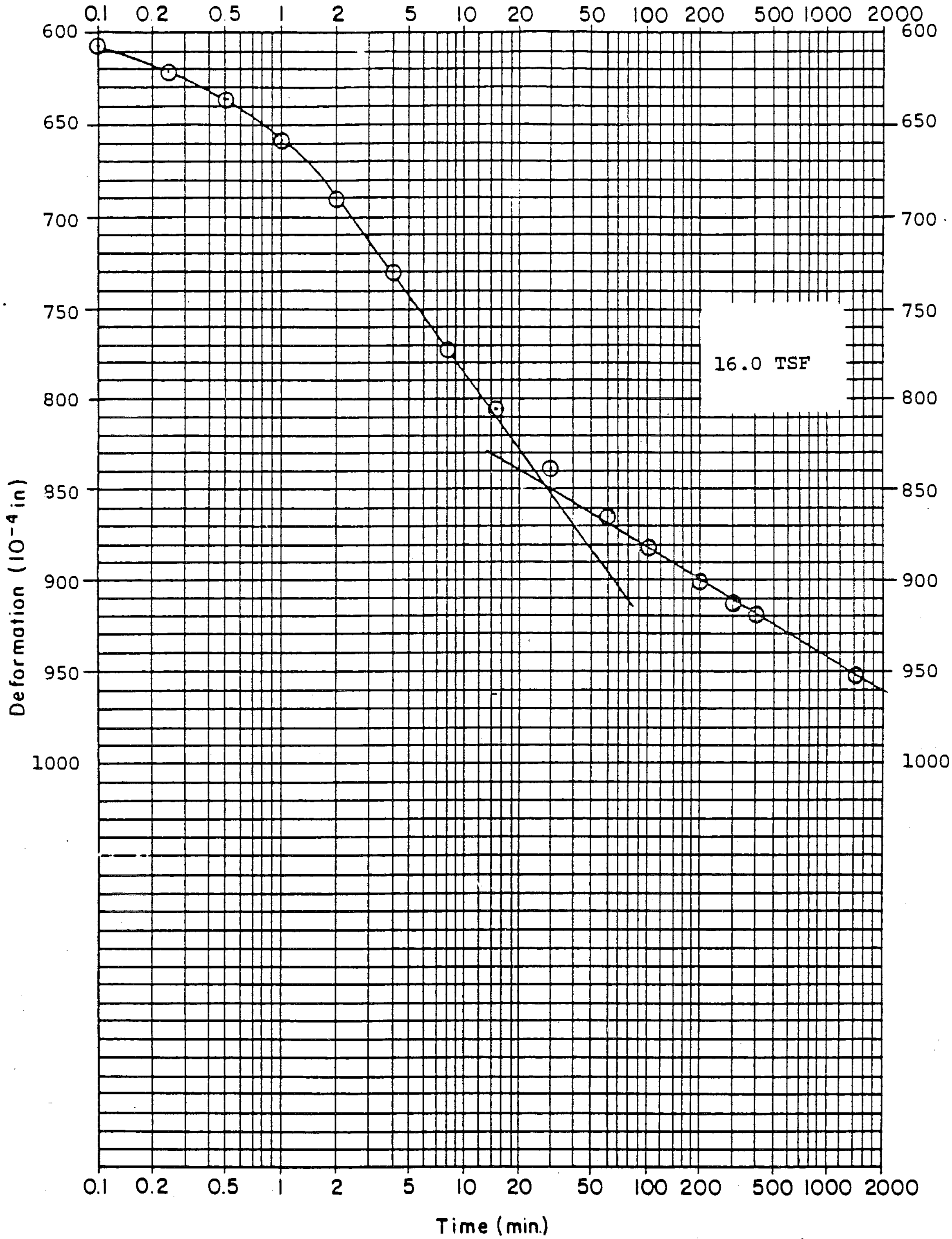
Project DELAWARE SOLID WASTE FACILITY - 2 Boring No GF-7

Sample No ST-1 Depth 20.0' - 22.0' Date Aug. 1986

GANNETT FLEMING GEOTECHNICAL LABORATORY



CONSOLIDATION TEST-TIME CURVES		
Project DELAWARE SOLID WASTE FACILITY - 2		Boring No GF-7
Sample No ST-1	Depth 20.0' - 22.0'	Date Aug. 1986
GANNETT FLEMING GEOTECHNICAL LABORATORY		

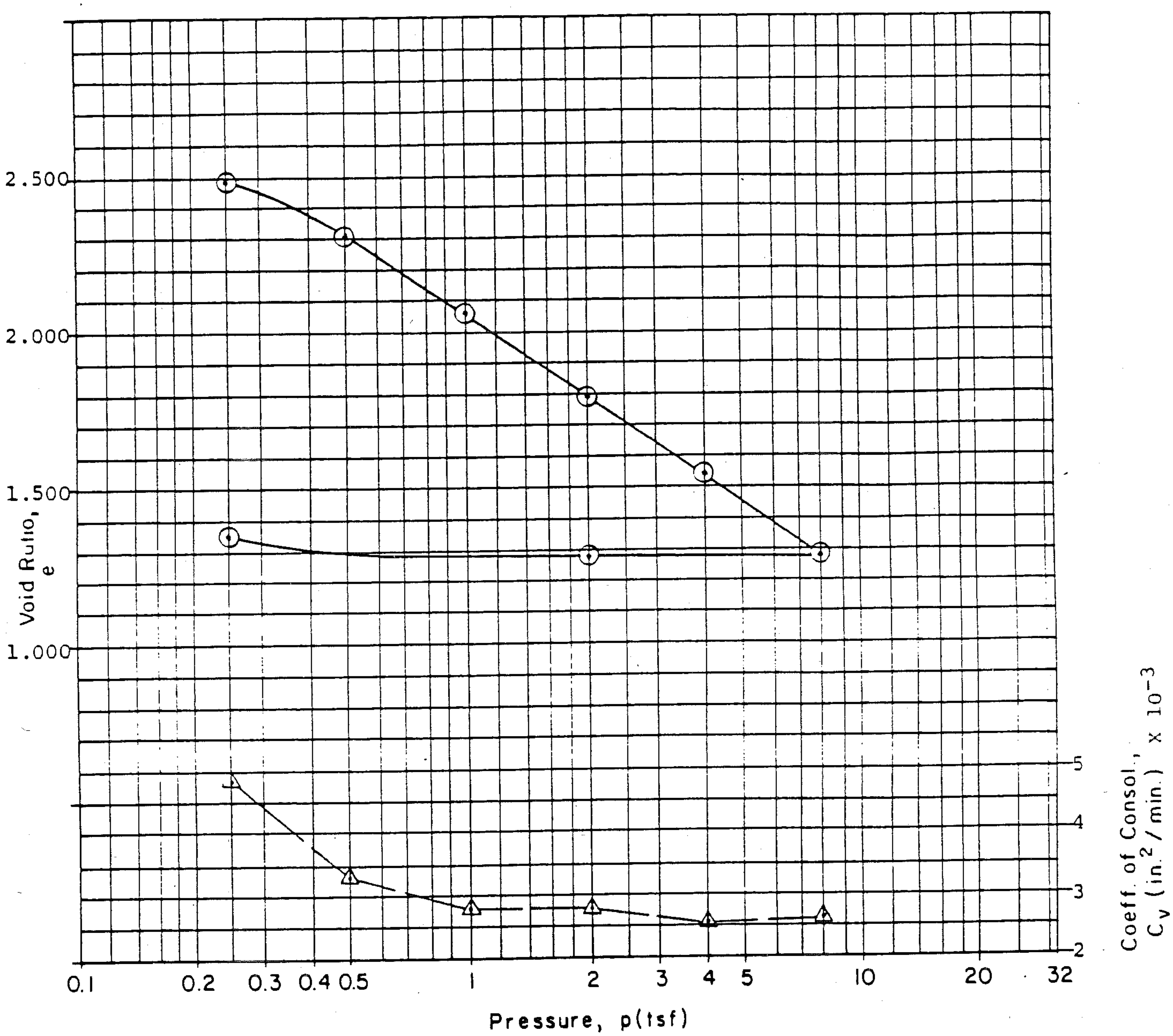


CONSOLIDATION TEST-TIME CURVES

Project DELAWARE SOLID WASTE FACILITY - 2 Boring No GF-7

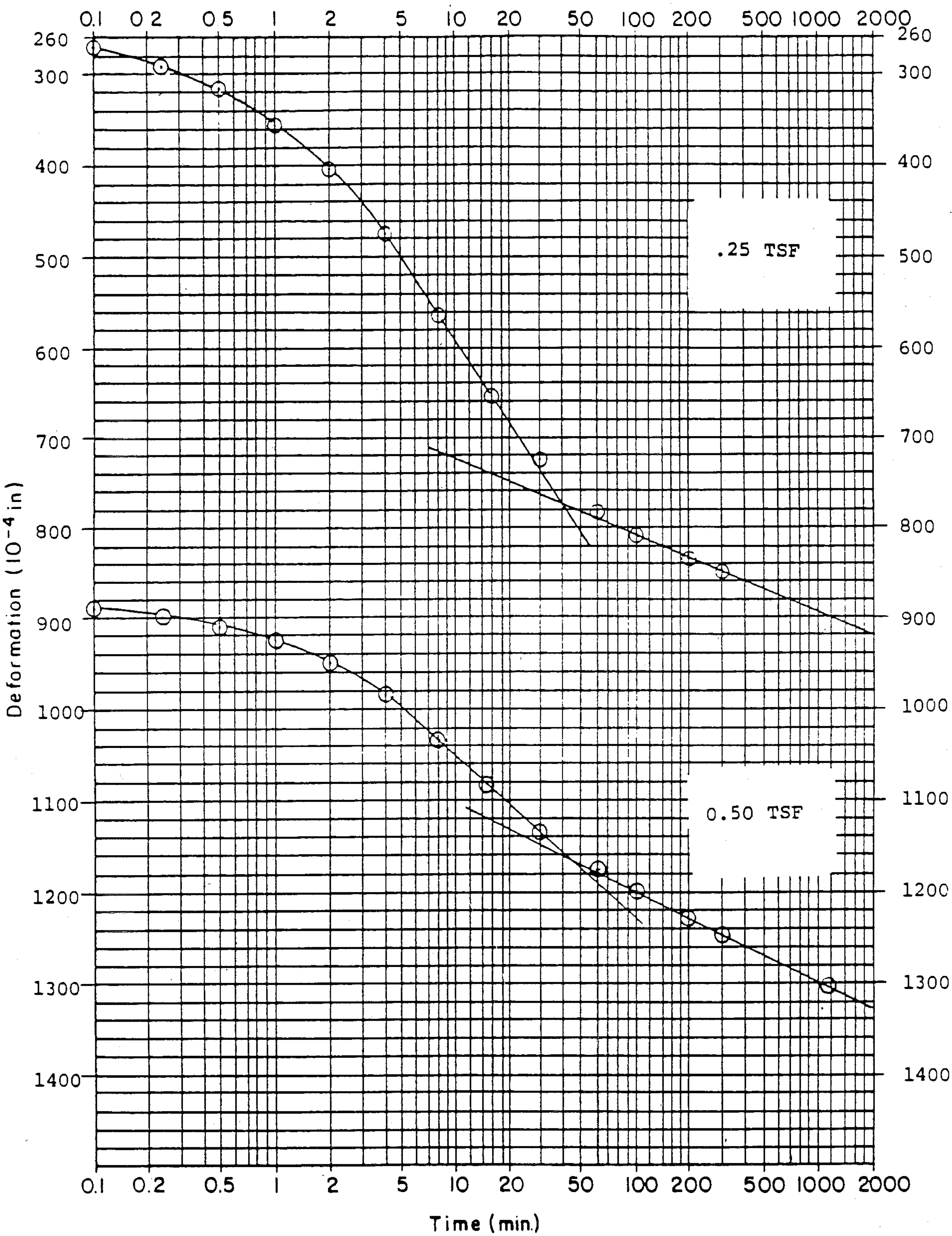
Sample No ST-1 Depth 20.0' - 22.0' Date Aug. 1986

GANNETT FLEMING GEOTECHNICAL LABORATORY



Type of Specimen	Shelby			Before Test			After Test	
Dia	2.50 in	H_T	0.75 in	Water Content	w_o	95.2	w_f	52.8
Compression Index		C_c	0.84	Void Ratio	e_o	2.828	e_f	1.365
Classification			OH	Saturation	S_o	95.4	S_f	98.6
w_l	74.6	I_p	26.0	Project	DSWA NORTHERN FACILITY-2			
w_p	48.6	LI	1.8	Boring No	GF-8	Sample No	S-4	
Remarks Exceeded machine travel				Depth	5- 7'	Date	8/ 86	
limits at 16 TSF.								

GANNETT FLEMING GEOTECHNICAL LABORATORY
CONSOLIDATION TEST REPORT

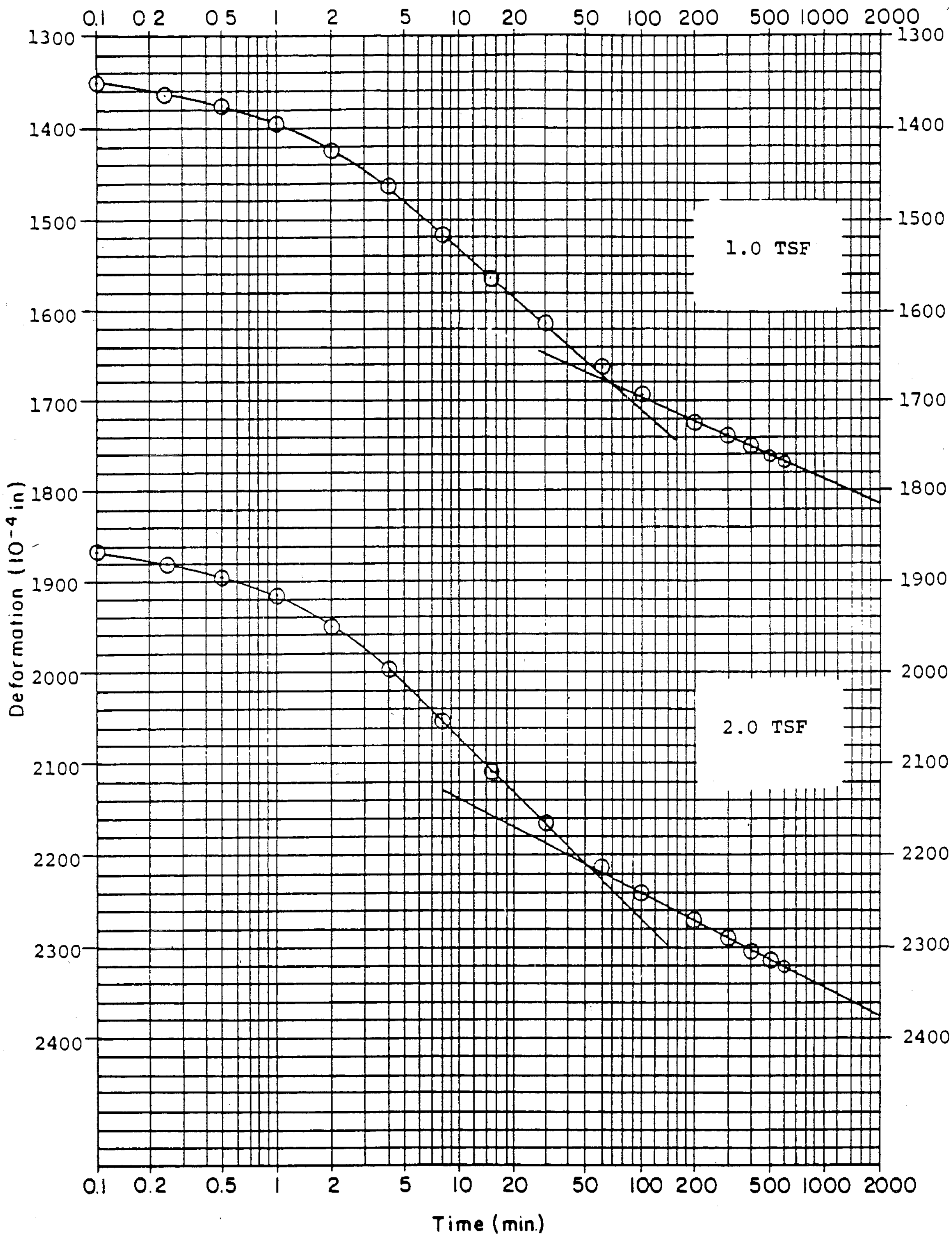


CONSOLIDATION TEST-TIME CURVES

Project DELAWARE SOLID WASTE FACILITY - 2 Boring No GF-8

Sample No S-4 Depth 5.0' - 7.0' Date Aug. 1986

GANNETT FLEMING GEOTECHNICAL LABORATORY

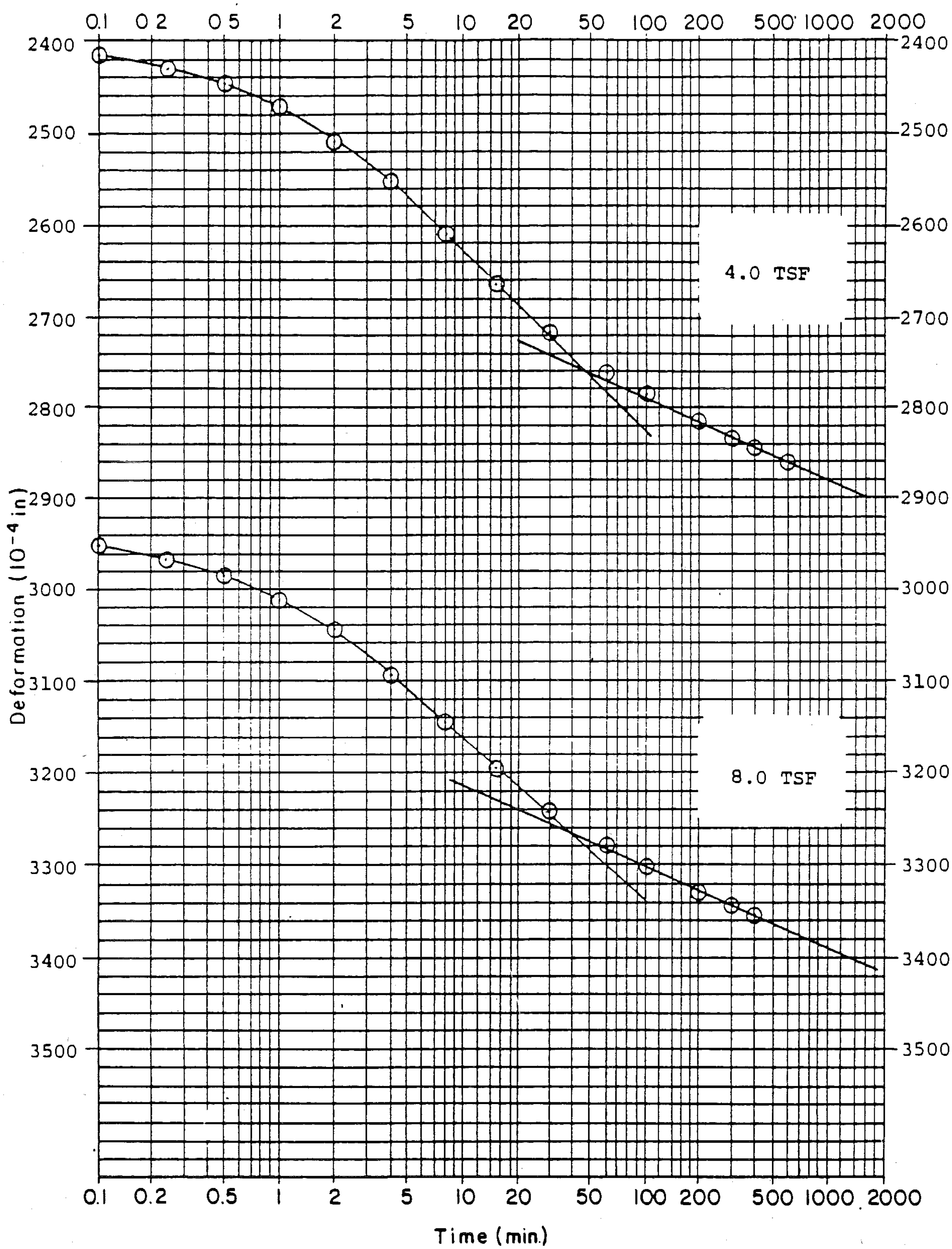


CONSOLIDATION TEST-TIME CURVES

Project DELAWARE SOLID WASTE FACILITY - 2 Boring No GF-8

Sample No S-4 Depth 5.0' - 7.0' Date Aug. 1986

GANNETT FLEMING GEOTECHNICAL LABORATORY

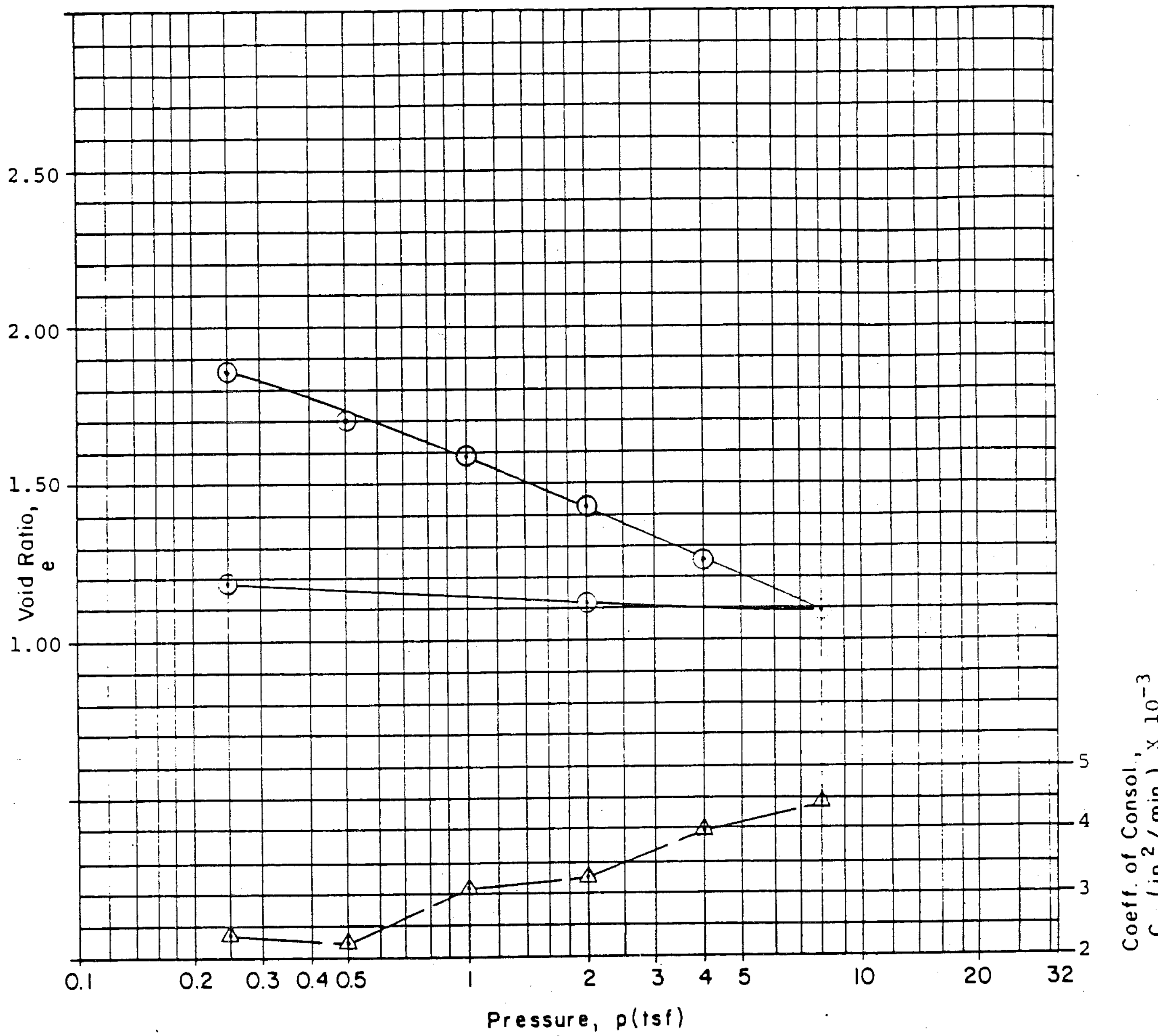


CONSOLIDATION TEST-TIME CURVES

Project DELAWARE SOLID WASTE FACILITY - 2 Boring No GF-8

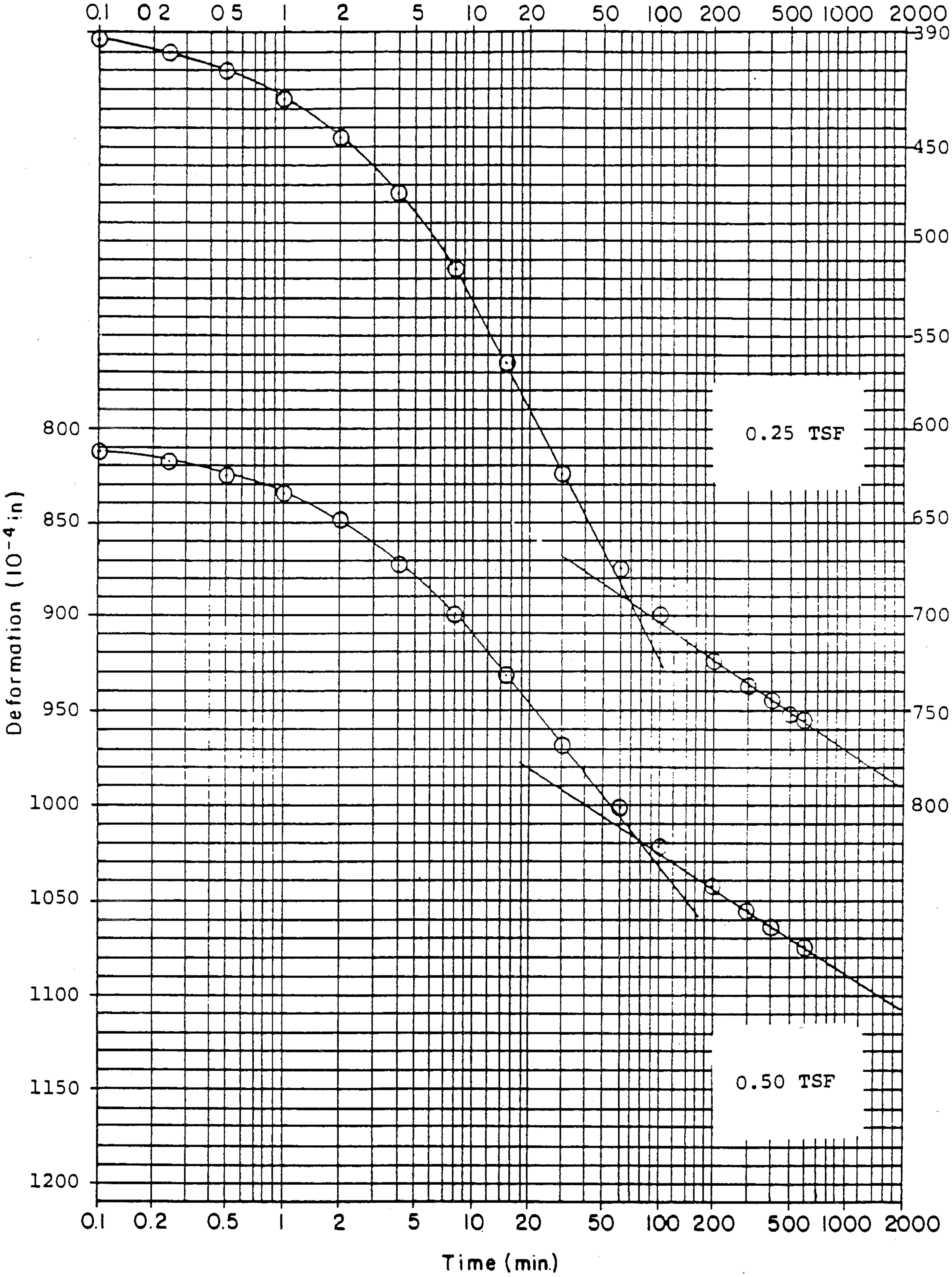
Sample No S-4 Depth 5.0' - 7.0' Date Aug. 1986

GANNETT FLEMING GEOTECHNICAL LABORATORY

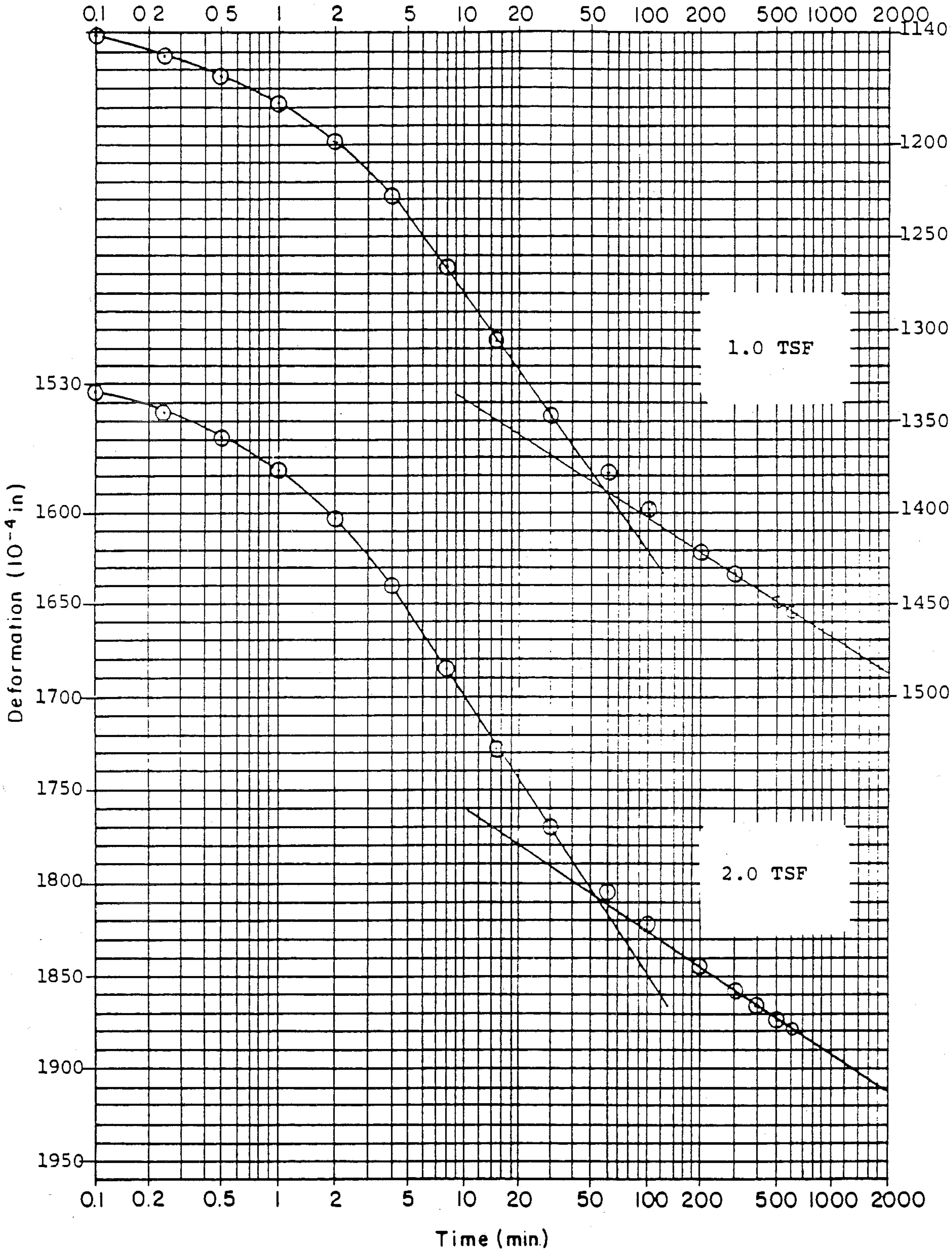


Type of Specimen	Shelby			Before Test		After Test	
Dia 2.50 in		H_T 0.75 in		Water Content	w_o 82.7	w_f	46.5
Compression Index		C_c 0.54		Void Ratio	e_o 2.101	e_f	1.196
Classification		MH		Saturation	S_o 97	S_f	100
w_l	65.9	I_p	23.5	Project DSWA NORTHERN FACILITY-2			
w_p	42.4	LI	1.7	Boring No	GF-8	Sample No	S-13
Remarks Exceeded machine limits at 16 TSF.				Depth	25- 27'	Date	8/ 86

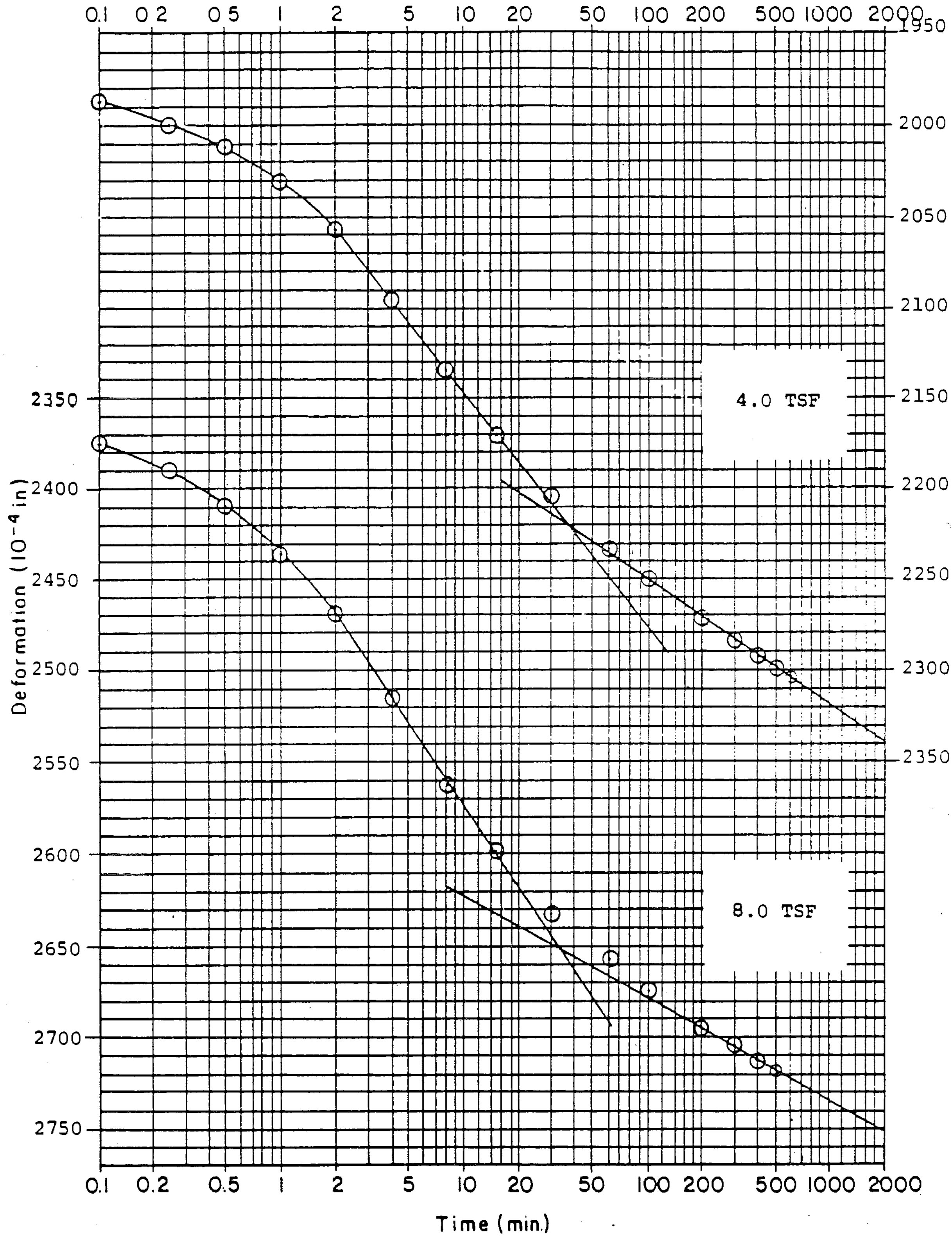
GANNETT FLEMING GEOTECHNICAL LABORATORY
CONSOLIDATION TEST REPORT



CONSOLIDATION TEST-TIME CURVES			
Project	DELAWARE SOLID WASTE FACILITY - 2	Boring No	GF-8
Sample No	S-13	Depth 25.0' - 27.0'	Date Aug. 1986
GANNETT FLEMING GEOTECHNICAL LABORATORY			



CONSOLIDATION TEST-TIME CURVES
Project DELAWARE SOLID WASTE FACILITY - 2 Boring No GF-8
Sample No S-13 Depth 25.0' - 27.0' Date Aug. 1986
GANNETT FLEMING GEOTECHNICAL LABORATORY



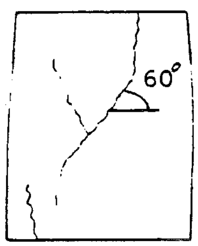
CONSOLIDATION TEST-TIME CURVES

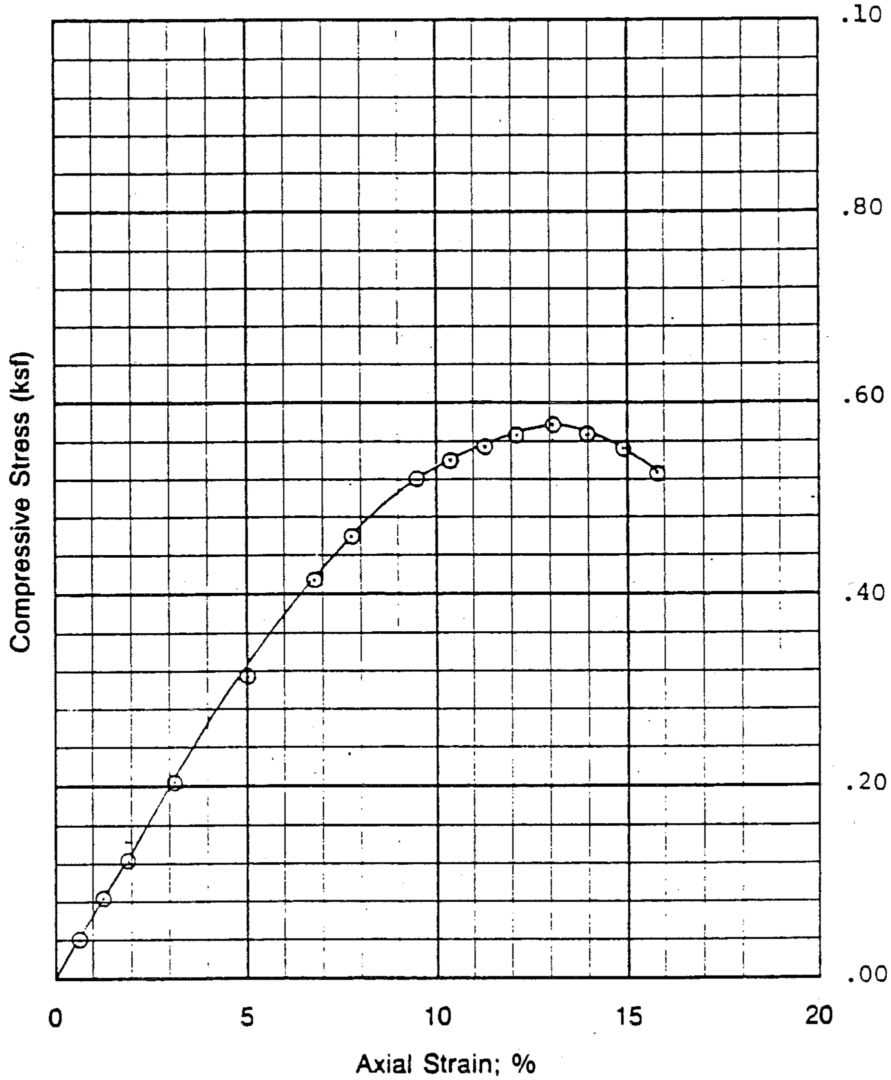
Project DELAWARE SOLID WASTE FACILITY - 2 Boring No GF-8

Sample No S-13 Depth 25.0' - 27.0' Date Aug. 1986

GANNETT FLEMING GEOTECHNICAL LABORATORY

Failure Sketches





Test no			1			
Type of specimen			Shelby			
Initial	Water content	w_o	62.0 %	%	%	%
	Void ratio	e_o	1.55			
	Saturation	S_o	100 %	%	%	%
	Dry density, lb/cu ft	γ_d	58.8			
Time to failure, min		t_f	11.5			
Unconfined compressive strength, ksf		q_u	.58			
Initial specimen diameter, in		D_o	2.815			
Initial specimen height, in		H_o	5.569			

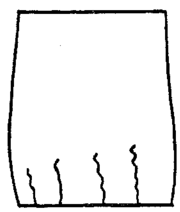
UNCONFINED COMPRESSION TEST REPORT

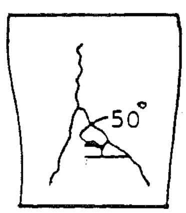
Project DELAWARE SOLID WASTE FACILITY - 2 Boring No GF-1

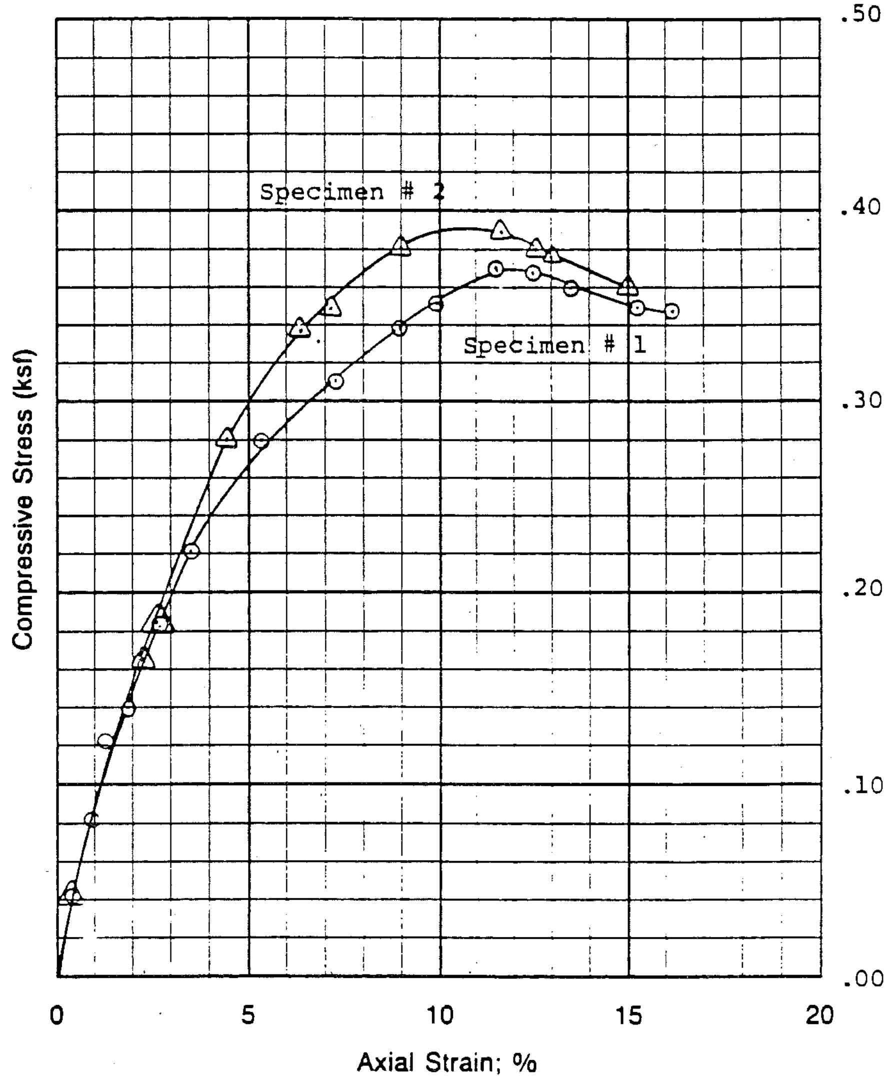
Sample No S-7 Depth 10.0' - 12.0' Date 8/6/86

GANNETT FLEMING GEOTECHNICAL LABORATORY

Failure Sketches







Test no			1	2		
Type of specimen			Shelby	Shelby		
Initial	Water content	w_o	67.9 %	80.8 %	%	%
	Void ratio	e_o	1.69	1.94		
	Saturation	S_o	96.4 %	100 %	%	%
	Dry density, lb/cu ft	γ_d	51.8	51.2		
Time to failure, min		t_f	8.5	6.6		
Unconfined compressive strength, ksf		q_u	.37	.39		
Initial specimen diameter, in		D_o	2.818	2.799		
Initial specimen height, in		H_o	5.557	5.553		

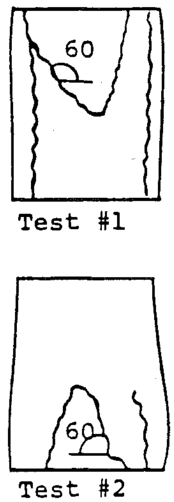
UNCONFINED COMPRESSION TEST REPORT

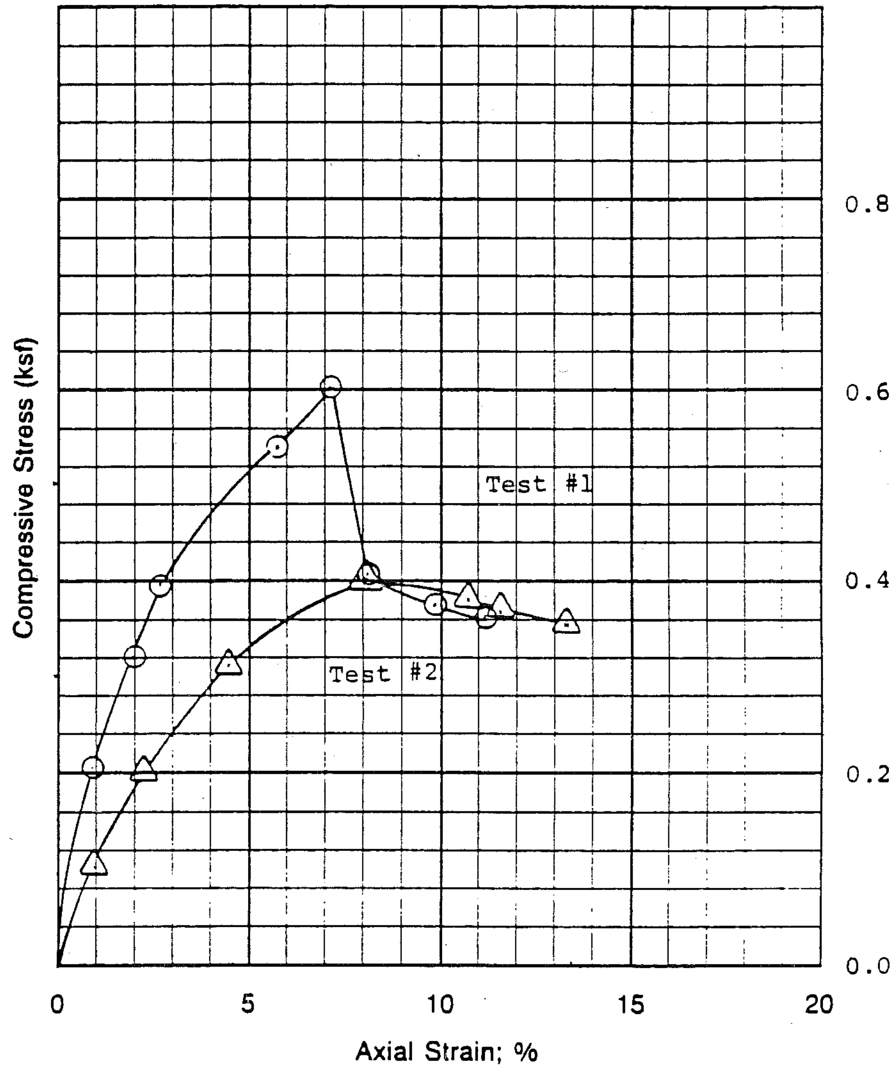
Project DELAWARE SOLID WASTE FACILITY - 2 Boring No GF-1

Sample No S-10 Depth 20.0' - 22.0' Date 8/7/86

GANNETT FLEMING GEOTECHNICAL LABORATORY

Failure Sketches





Test no			1	2		
Type of specimen			Shelby	Shelby		
Initial	Water content	w_o	85.3%	83.1%	%	%
	Void ratio	e_o	2.508	2.337		
	Saturation	S_o	90.4%	94.5%	%	%
	Dry density, lb/cu ft	γ_d	47.3	49.8		
Time to failure, min		t_f	3.6	4.6		
Unconfined compressive strength, ksf		q_u	0.61	0.38		
Initial specimen diameter, in		D_o	2.84	2.83		
Initial specimen height, in		H_o	6.35	5.58		

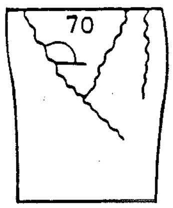
UNCONFINED COMPRESSION TEST REPORT

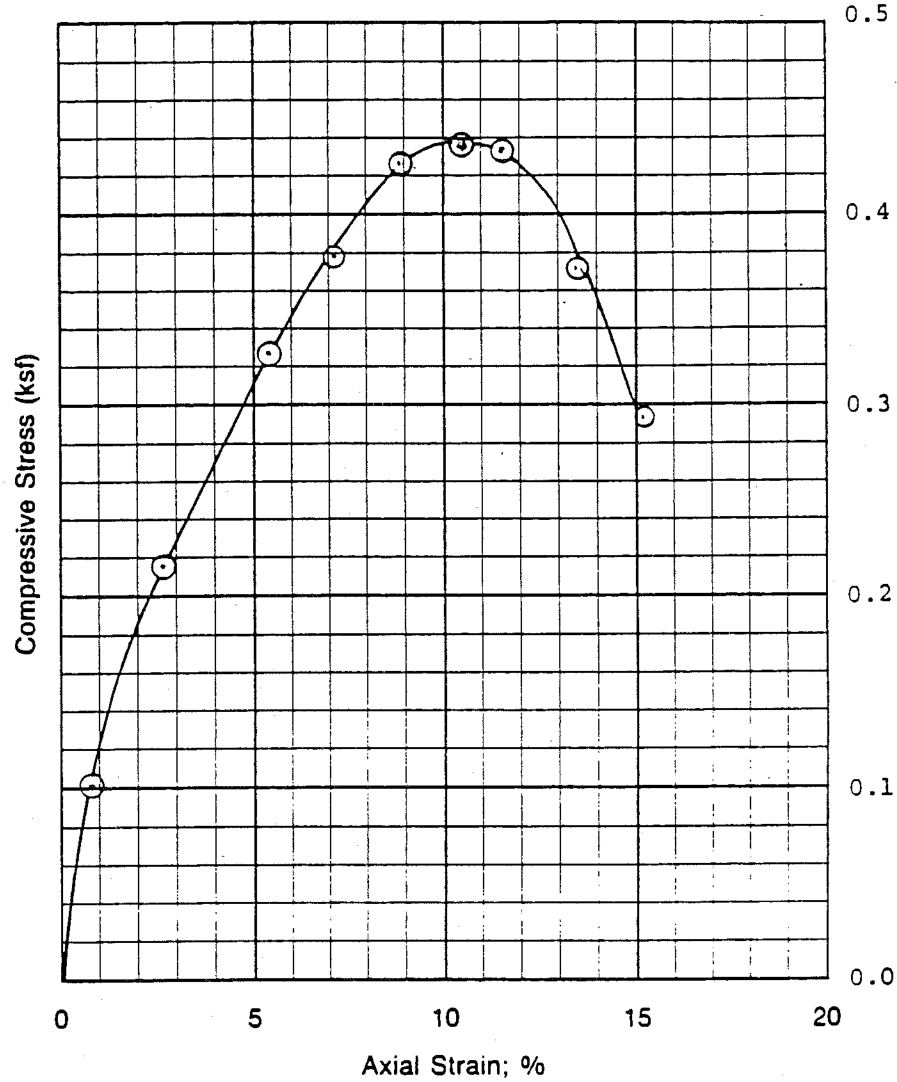
Project DSWA NORTHERN FACILITY -2 Boring No GF-6

Sample No S-3 Depth 5- 7' Date 8/ 86

GANNETT FLEMING GEOTECHNICAL LABORATORY

Failure Sketches





Test no			1			
Type of specimen			Shelby			
Initial	Water content	w_o	82.1 %	%	%	%
	Void ratio	e_o	2.317			
	Saturation	S_o	95.5 %	%	%	%
	Dry density, lb/cu ft	γ_d	50.8			
Time to failure, min		t_f	6.1			
Unconfined compressive strength, ksf		q_u	0.44			
Initial specimen diameter, in		D_o	2.85			
Initial specimen height, in		H_o	6.36			

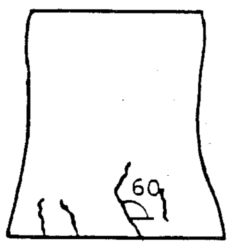
UNCONFINED COMPRESSION TEST REPORT

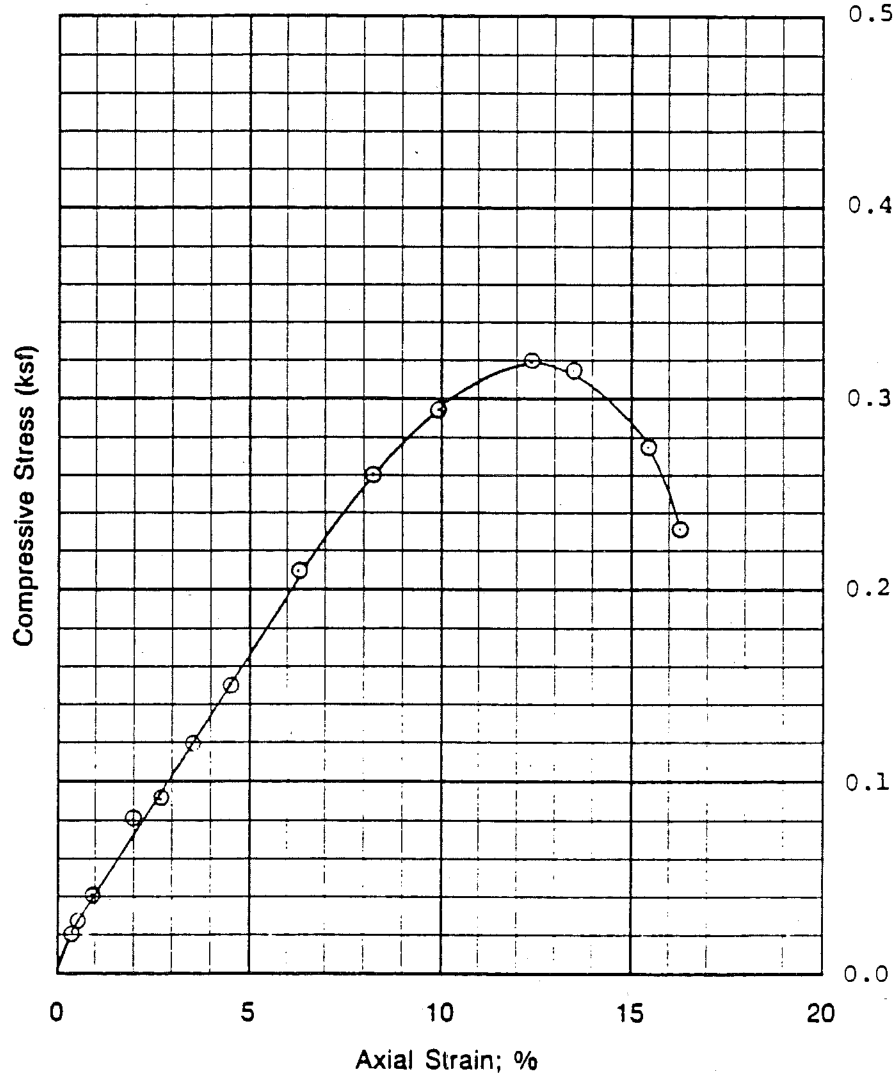
Project DSWA NORTHERN FACILITY- 2 Boring No GF-8

Sample No S-4 Depth 5- 7' Date 8/ 86

GANNETT FLEMING GEOTECHNICAL LABORATORY

Failure Sketches





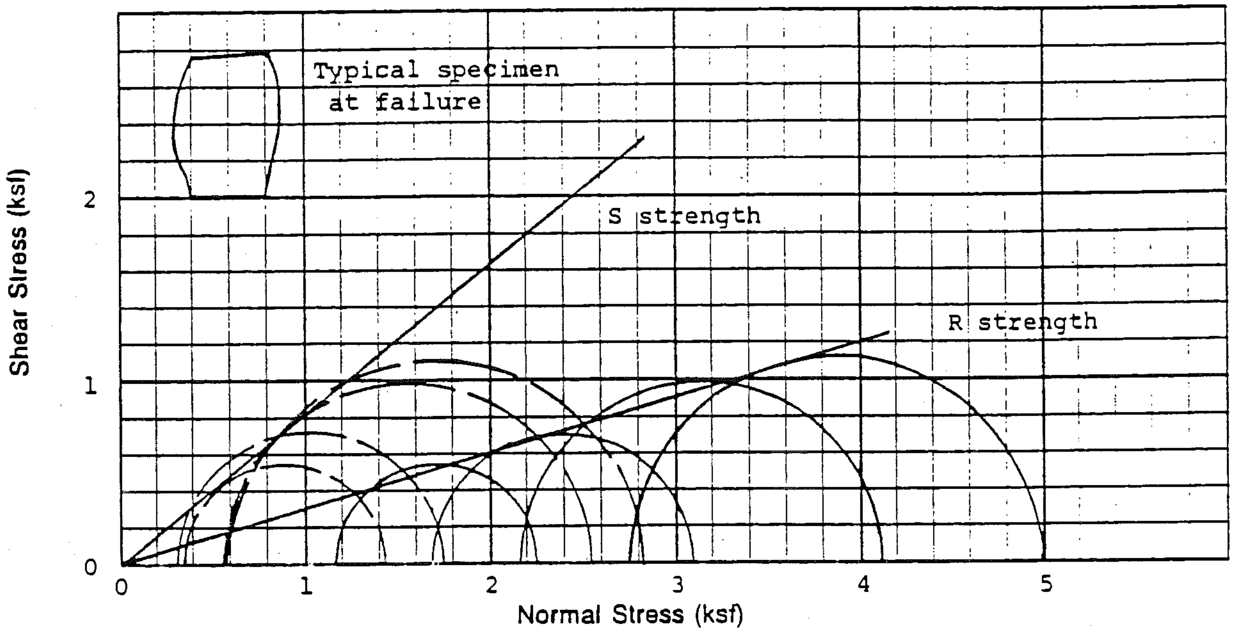
Test no			1			
Type of specimen			Shelby			
Initial	Water content	w_o	78.5 %	%	%	%
	Void ratio	e_o	2.100			
	Saturation	S_o	96.1 %	%	%	%
	Dry density, lb/cu ft	γ_d	51.7			
Time to failure, min		t_f	10.2			
Unconfined compressive strength, ksf		q_u	0.32			
Initial specimen diameter, in		D_o	2.843			
Initial specimen height, in		H_o	5.512			

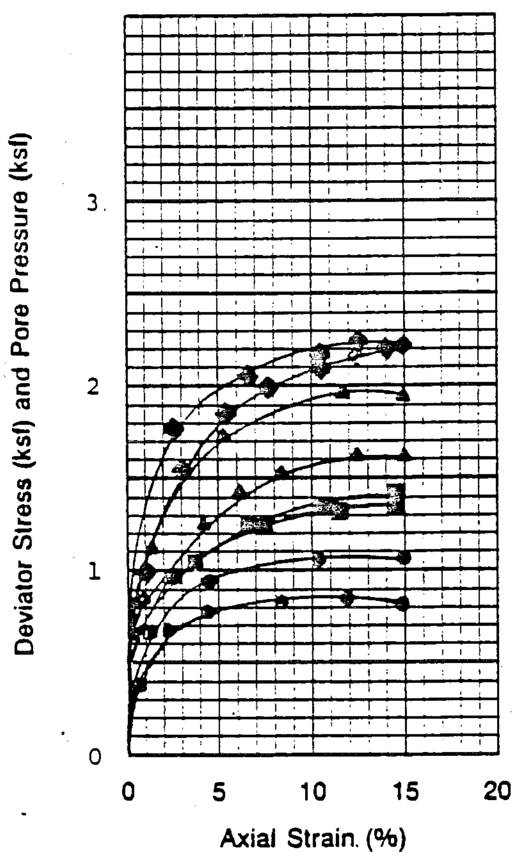
UNCONFINED COMPRESSION TEST REPORT

Project DELAWARE SOLID WASTE FACILITY - 2 Boring No GF-8

Sample No S-13 Depth 25.0' - 27.0' Date Aug. 1986

GANNETT FLEMING GEOTECHNICAL LABORATORY





Specimen No			1	2	3	4
Diameter (in)		D_o	2.85	2.83	2.82	2.83
Height (in)		H_o	5.49	5.54	5.49	5.60
Initial	Water Content (%)	w_o	91.3	91.3	91.3	85.4
	Dry Density (pcf)	γ_{do}	47.4	47.3	47.3	43.2
	Void Ratio	e_o	2.357	2.368	2.347	2.302
	Saturation (%)	S_o	98.8	98.3	99.2	94.5
Before Test	Water Content (%)	w_c	74.4	70.8	69.3	65.0
	Dry Density (pcf)	γ_{dc}	55.0	56.8	57.5	59.8
	Void Ratio	e_c	1.890	1.806	1.766	1.644
	Saturation (%)	S_c	100	100	100	100
	Back Pressure (ksf)	μ_B	6.29	4.20	9.63	6.02
Total Minor Princ Stress (ksf)		σ_3	1.17	1.68	2.15	2.74
Maximum Deviator Stress (ksf)		P/A	1.07	1.41	1.97	2.24
Time Max Deviator Stress(min)		t_f	120	120	120	120
Total Major Princ Stress (ksf)		σ_1	2.24	3.09	4.12	4.98
Pore Pressure at Max Deviator Stress (ksf)		μ	0.83	1.35	1.60	2.17
Eff Minor Princ Stress (ksf)		σ_3'	0.34	0.32	0.55	0.57
Eff Major Princ Stress (ksf)		σ_1'	1.41	1.73	2.52	2.81

Remarks: Specimens produced a bulging failure.

Project DSWA NORTHERN FACILITY-2		Boring No GF-6/ -7	Sample No ST-2/ -1
Type of Test: CU with pore pressure measurements			Sample Type: Shelby
c = 0.0 ksf	ϕ = 17.0 °	c' = 0.0 ksf	ϕ' = 39.5 °

GANNETT FLEMING GEOTECHNICAL LABORATORY
TRIAXIAL COMPRESSION TEST REPORT

GANNETT FLEMING GEOTECHNICAL LABORATORY

WATER CONTENT SUMMARY SHEET

PROJECT DSWA NORTHERN FACILITY-2

BORING	SAMPLE	DEPTH (ft)	WATER CONTENT (%)
GF-9	S-1	0- 1.5	82.1
GF-9	S-2	7- 8	96.7
GF-10	Bulk	0- 2	86.2
GF-10	S-1	2- 4	111.8
GF-10	S-2	6- 8	108.2
GF-11	Bulk	0- 2	82.5
GF-11	S-1	4- 5	102.3
GF-11	S-2	7- 8	95.6

GANNETT FLEMING GEOTECHNICAL LABORATORY

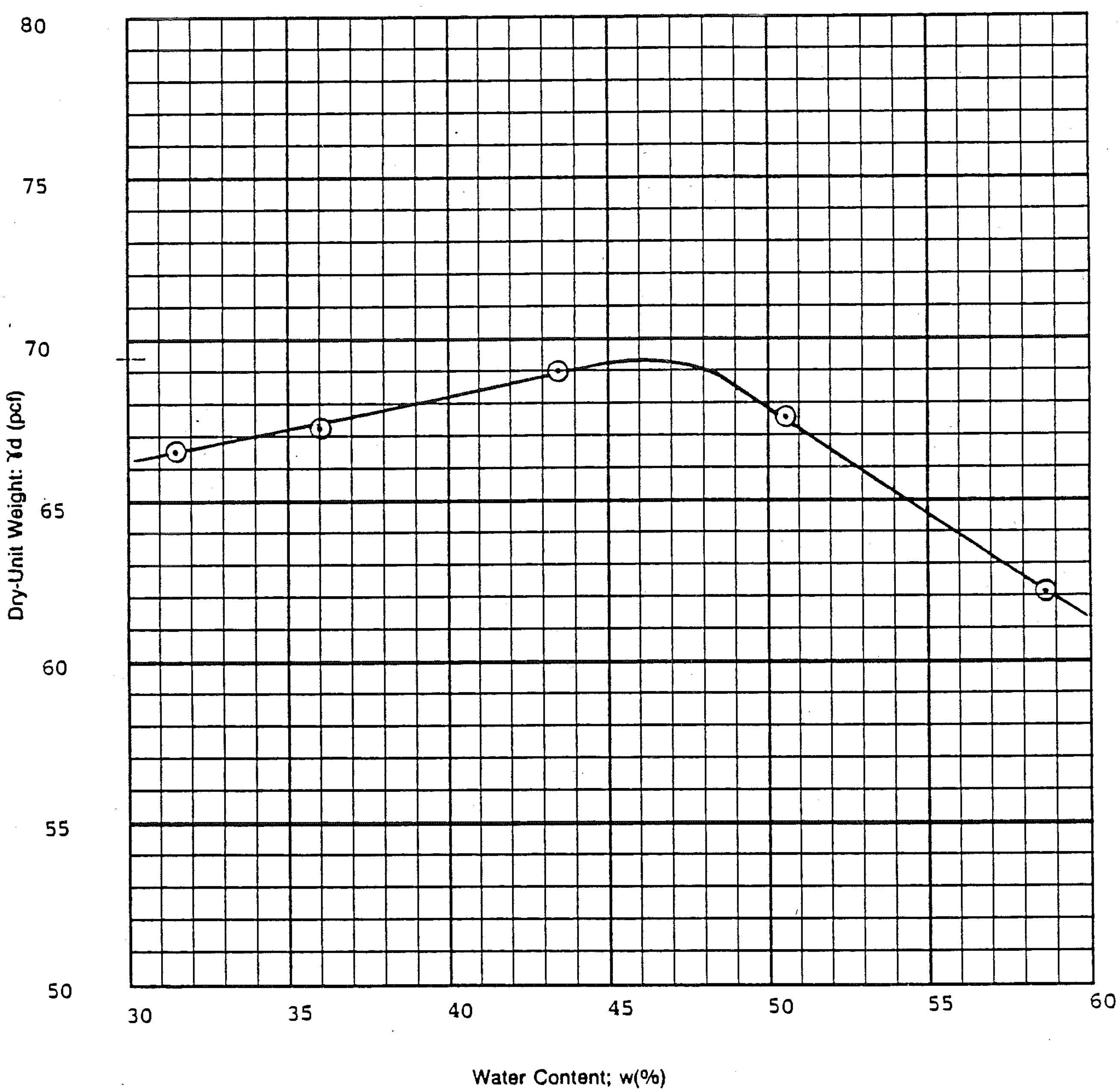
COMPACTION TEST

Project DSWA NORTHERN FACILITY-2 Job No. ______

Boring No. GF-10 Sample No. Bag Depth 0- 2'

Description of Soil Gray organic silt

Test Method ASTM D698- Method A Date of Testing 9/ 86



GANNETT FLEMING GEOTECHNICAL LABORATORY

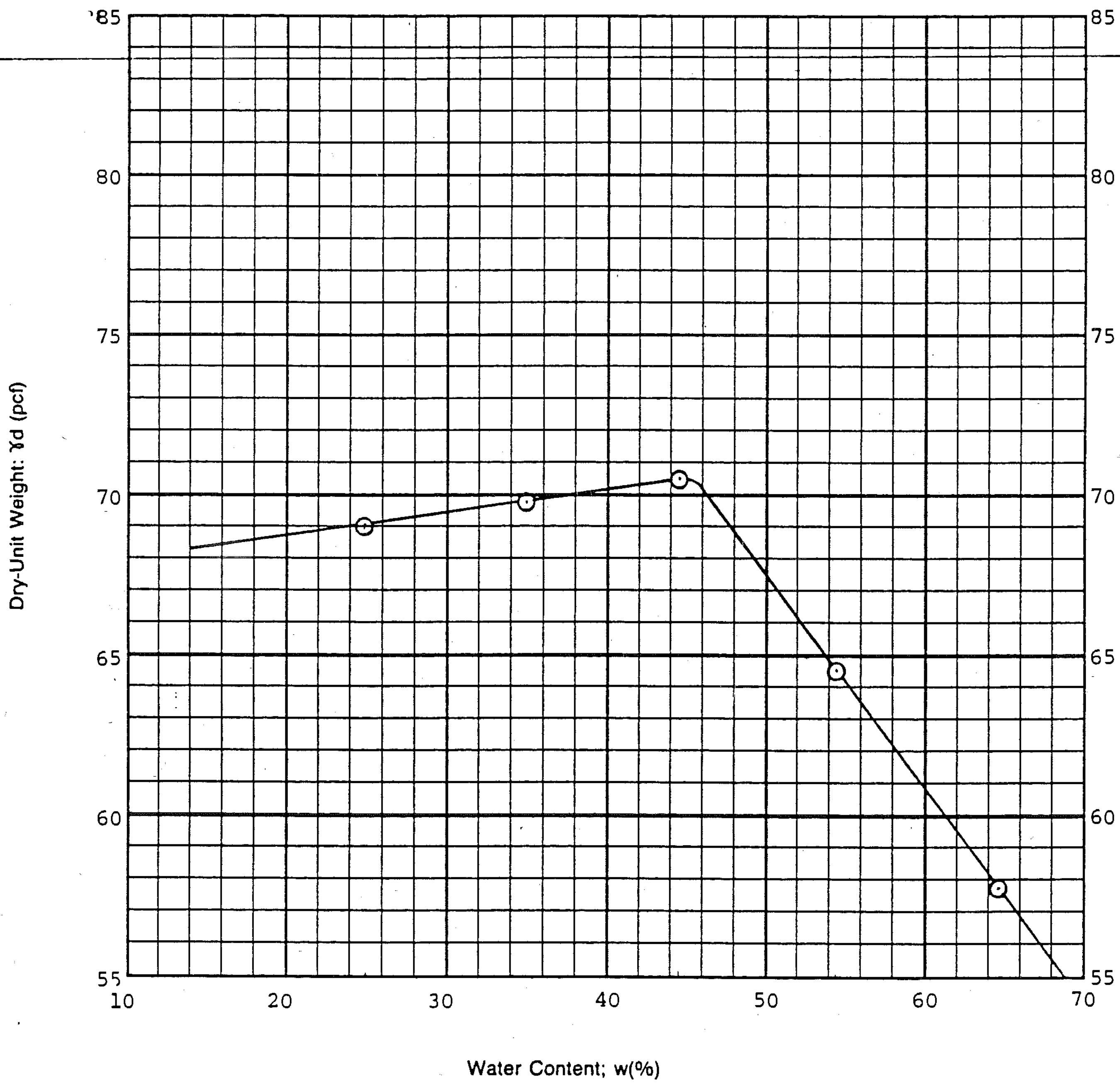
COMPACTION TEST

Project DELAWARE SOLID WASTE FACILITY - 2 Job No.

Boring No. GF-11 Sample No. Bulk Depth 0.0'- 2.0'

Description of Soil

Test Method ASTM - D - 698 Method A Date of Testing 9/9/86

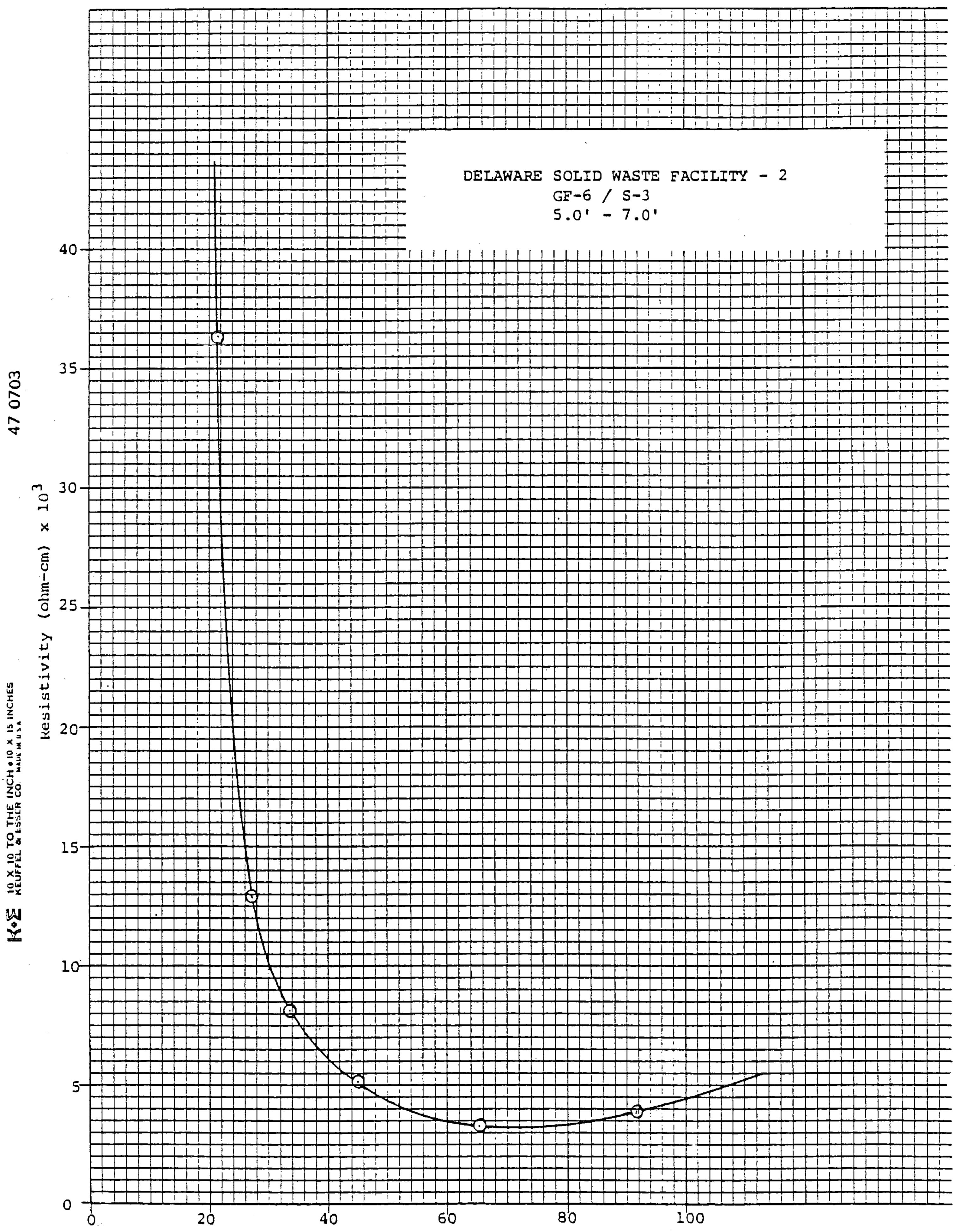


GANNETT FLEMING GEOTECHNICAL LABORATORY

PERMEABILITY SUMMARY SHEET

PROJECT DSWA NORTHERN FACILITY-2

BORING	SAMPLE	DEPTH (ft)	SAMPLE TYPE	WATER CONTENT (%)	DRY DENSITY (pcf)	PERMEABILITY (cm/sec)
GF-6	S-3	5-7	Shelby	105.4	42.8	6.6×10^{-8}
GF-7	ST-1	20-22	Shelby	78.7	53.6	1.1×10^{-7}


DELAWARE SOLID WASTE FACILITY - 2
GF-6 / S-3
5.0' - 7.0'
Resistivity (ohm-cm) x 10^3
0
5
10
15
20
25
30
35
40
0
20
40
60
80
100
47 0703
K•E 10 X 10 TO THE INCH • 10 X 15 INCHES
KEUFFEL & ESSER CO.

GANNETT - MCCREATH LABORATORIES
ANALYTICAL AND CONSULTING CHEMISTS
P.O. BOX 1963
HARRISBURG, PA 17105
TELEPHONE (717) 763-7211

LABORATORY NO. 024163001 DATE - SEPTEMBER 5, 1986

GF GEOTECHNICAL ENGINEERS

SAMPLE ID NUMBER - 862445

SAMPLE DESIGNATION - DELAWARE SOLID WASTE FAC - 2

DATE RECEIVED - AUG 25,1986

THE RESULTS OF ANALYSES PERFORMED ON THE ABOVE REFERENCED SAMPLE ARE AS FOLLOWS:

ANALYSIS PERFORMED	RESULT
PH	6.1

RESPECTFULLY SUBMITTED,

GANNETT - MC CREATH LABORATORIES

Max E. Snavely

MAX E. SNAVELY, SUPERVISOR

APPENDIX C

Water Balance Computations

Water Balance Method of Leachate Generation

Parameter	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec	Annual
	Months												
Temp, C	0	0.9	5.3	11.3	16.9	21.9	24.3	23.4	19.9	14.0	7.6	1.5	
Heat index	0.00	0.07	1.09	3.40	6.21	9.17	10.7	10.1	7.94	4.69	1.87	0.16	55.45
Unadj. PET, mm	0.00	1.30	15.0	42.7	74.4	106	123	117	93.2	57.4	24.7	2.64	
PET, mm	0	1	15	47	89	133	151	134	97	53	21	2	
P, mm	76	69	97	93	94	89	118	115	85	72	98	87	
$C_{r/o}$	0.17	0.17	0.17	0.17	0.17	0.13	0.13	0.13	0.13	0.13	0.13	0.13	
Runoff, mm	13	12	16	16	16	12	15	15	11	9	13	11	
I, mm	63	57	81	77	78	77	103	100	74	63	85	76	
I-PET	63	56	66	30	-11	-56	-48	-34	-23	10	64	74	+191
Neg (I-PET)				0	-11	-67	-115	-149	-172	-142	-32	0	
ST, mm	150	150	150	150	139	95	69	54	47	57	121	150	
ΔST, mm	0	0	0	0	-11	-44	-26	-15	-7	10	64	29	
AET, mm	0	1	15	47	89	121	129	115	81	53	21	2	
PERC	63	55	66	30	0	0	0	0	0	0	0	45	259

*The parameters are as follows: PET, potential evapotranspiration; P, precipitation; $C_{r/o}$, surface runoff coefficient; I, infiltration; ST, storage; ΔST, change in storage; AET, actual evapotranspiration; PERC, percolation.

2. Gannett Fleming Environmental Engineers, Inc., "Northern Solid Waste Facility – 2, Interim Hydrogeology Report, Phase II Landfill," prepared for the Delaware Solid Waste Authority, December 1986.

DELAWARE SOLID WASTE AUTHORITY
NORTHERN SOLID WASTE FACILITY-2
INTERIM HYDROGEOLOGY REPORT
PHASE II LANDFILL

This report presents a review of hydrogeologic conditions at the Delaware Solid Waste Authority's Northern Solid Waste Facility-2 (NSWF-2 site) located on Cherry Island, Wilmington, Delaware. The present review was prompted by a need to address concerns of the State of Delaware, Department of Natural Resources and Environmental Control (DNREC), regarding the hydrogeology of the site. These concerns are focused around apparent anomalies in the potentiometric surface in the Columbia Formation sediments and the need for a reevaluation of data and piezometer installations at the site. The potentiometric surface in the Columbia Formation sediments as reported in January 1984, by Terraqua Resources Corporation, in Site Suitability Report-Hydrogeologic and Geotechnical Evaluation of the Cherry Island Site, indicates that groundwater flow at the southeast corner of the site may be away from the Delaware and Christina Rivers (see Figure 1). Considerable hydrogeologic review reveals that existing data may not support this conclusion.

Previous Work

In July and August 1983 a monitoring well installation program was conducted at the site in order to define existing hydrogeologic conditions in partial fulfillment of DNREC regulations. During this program 18 piezometers were installed into the unconsolidated coastal plain sediments which underlie dredge spoil materials emplaced by the Corps of Engineers. In addition to installation of two piezometers at each of nine locations, sediment samples were collected at 5-foot intervals to determine thicknesses and characteristics of the geologic units involved. Piezometer placement and geologic variability of the sediments will be discussed in a later section of this report. Periodic measurements of piezometric elevations were carried out for the remainder of 1983. In July of 1985, quarterly monitoring of these same piezometers was commenced and continues to the present. Piezometers located in the Columbia/Recent deposits are of primary concern and are shown in Figure 1. The groundwater surface defined by piezometric elevations taken in the Potomac Formation,slopes without deviation toward the Delaware and Christina Rivers as expected and will not be discussed further.

Site Geology

The NSWF-2 site is located approximately 1-mile east of the Fall Line which separates the Atlantic Coastal Plain physiographic province from the adjacent Piedmont Province. Coastal Plain sediments at the site range in thickness from 95 feet at the northernmost corner to greater than 220 feet at the southeast corner. The oldest Coastal Plain sediments at the site are Early Cretaceous in age (represented by Potomac Formation) and are overlain by sediments of Quaternary age (represented by Columbia Formation). The Upper Cretaceous and Tertiary sections are absent at the site due either to erosion or nondeposition.

The Cretaceous sediments at the site consist predominantly of variegated red, gray, yellow, and white silts and clays containing interbedded white, gray and rust-colored sands and some gravels.

Unconformably overlying the Cretaceous strata are sediments of Quaternary and Holocene or recent age that are dominated by dark brown to black silts and silty clays. These silts consist of both natural river deposits and compositionally similar man-made dredge spoil deposits. Peat deposits in a few boreholes indicate that the boundary between natural and man-made deposits exists at or near sea level. These recent sediments are thickest at the site's eastern edge and also contain thin discontinuous lenses of sand and gravel.

Underlying the recent river deposits and dredge spoil materials are tan, brown, and gray silty sands with some gravel that are part of the Columbia Formation. This sandy unit reaches its maximum thickness of about 60 feet at the interior of the site, thinning toward the Fall Line and also toward the confluence of the Delaware and Christina Rivers. According to Jordan 1962: "The Columbia Formation of northern Delaware may be described as yellow to dark reddish-brown, mostly coarse, moderately sorted, quartz sand, with a considerable admixture of gravel and commonly containing cobbles and, in some places, boulders. Thin silts may be present but are uncommon. It is generally cross-bedded but the bedding may be contorted. Limonite is common as thin ledges and as a stain or coating on other particles. A variety of lithic types is recognizable among the larger fragments, including especially quartzose sandstone and quartzite, some of which contain Paleozoic fossils". This description stands in obvious distinction from the finer grained recent river deposits and dredge spoil material.

Piezometer Installations

As mentioned previously, 18 piezometers were installed to monitor groundwater flow at the site. Nine of these are placed in the Potomac Formation and the remaining nine are located in both the recent and Columbia Formation sediments.

Because the unconsolidated sediments at Cherry Island are prone to consolidation, a primary concern expressed by DNREC is that differential piezometer settlement may have taken place during the monitoring period. Such settlement would change the piezometer elevations, produce incorrect water level data, and result in an anomalous potentiometric surface. The lowering of a piezometer through downward settlement would change the reference point (usually the casing top) used for measuring the depth to the water table. A downward change in the casing top reference point effectively decreases the distance to the water table giving the impression that the water table has risen.

Although it is possible that piezometers may settle independently from the material in which they are installed, it is far more likely that any changes taking place in piezometer elevations would also be reflected in ground surface elevation changes.

As indicated by two independent elevation surveys at piezometer locations; changes in ground surface elevations between July-August 1983 and December 1985 have been very small. (See Appendix A for ground surface elevation changes). With the notable exceptions of Piezometer Locations C-104, P-104 where the casings were extended about 5.5 feet, ground surface elevation changes are minimal and may represent erosional/depositional changes in the land surface and not ground settlement.

If piezometer settlement has not taken place and water level measurements are correct, an alternative explanation for the anomalous appearance of the Columbia Formation potentiometric surface (Figure 1, modified from Terraqua 1984) is necessary. Such an explanation is found in review of site geology..

Geologic Review

The Columbia Formation of northern Delaware, as recorded by Jordan (1962), consists primarily of coarse sand, considerable admixture of gravel and cobbles with thin silty layers. Although the basal sediments which overlie the Cretaceous Potomac Formation resemble this description, most of the remaining succession of sediments are considerably finer grained and should be interpreted as recent river sedimentation. Geologic review by Duffield Associates as part of their quarterly monitoring effort, also support this conclusion. The resulting hydrogeologic framework is much refined and is illustrated in the stratigraphic cross sections of Figure 3 and 4. Included in these cross sections are data from several geologic test borings and also stratigraphic locations of many of the piezometers installed during July and August of 1983 (discussed in previous section). Because both Potomac and "Columbia" Formation piezometer pairs are located very close to each other they are shown as composite installations.

Interpretation of the post-Potomac Formation sediments as recent river deposits rather than Columbia Formation, explains the lithologic variations shown in the stratigraphic cross sections of Figures 3 and 4. If these sediments were truly Columbia Formation, they would be expected to be uniformly more coarse grained and contain less silt or silty clay. Instead they are composed of thick deposits of dark brown to gray silty clay and clayey silt with thinner intervening sand and gravel units. Sedimentation of this type is typical of meandering river systems where sediment type, channel position, and geometry are variable.

Cross Section C-C of Figure 4 illustrates the extent of the sand and gravel units which constitute the permeable zones within the river sediment aquifer. These sand and gravel layers can be divided into lower and upper units which are separated by an inherently less permeable silty unit. The upper sand unit is confined to the eastern area of the site where it forms a westward diminishing wedge. The lower sand unit is more extensive and is connected to underlying sandy sediments which may be true Columbia Formation (see Figure 4). These sand units are shown in Figures 3 and 4 with stippled patterns for the sake of clarity and identification from one cross section to another.

The most important detail to be observed in the cross sections is the placement of piezometers in the recent river sediments. Cross Section C-C best illustrates the physical cause of the anomalous appearance of the potentiometric surface of the previous report. In this section three of four piezometers are placed in the lower sand unit and one (C-106) is placed in the upper sand unit. Adjoining Cross Section D-D shows two additional piezometers (C-102 and C-108) located in the upper san unit. Because the upper sand unit is isolated from the lower sand unit by a thick sandy silty, the piezometers located in it should be considered as a separate network.

Additional evidence that Piezometers C-102, 106, and 108 should be treated as separate from other installations in the Recent/Columbia sediments if found in data from monitoring of several piezometers during tidal fluctuations on December 30, 1983. On that date two sets of paired piezometers and the Christina River were monitored through a 10-hour period to determine the influence of ocean tides on groundwater levels at the site. The two Potomac Formation piezometers of the tidal study will not be discussed further because those installed in Recent/Columbia sediments are of primary concern in this review.

The important difference between the two Recent/Columbia piezometers of the tidal study is that they are installed in different sand zones within the sediments. C-105 is installed in the lower sand unit and C-106 in the upper. If the upper and lower sand units were hydraulically interconnected then the tidal responses of these two piezometers would be expected to be similar. However, this is not the case and assuming that both C-105 and C-106 were functioning properly, it appears that the two units are isolated from each other. Tidal response curves and total tidal fluctuation for C-105, 106, and the Christina River are shown in Figure 5.

The most obvious difference in these tidal responses is the very limited fluctuation of C-106 as compared to C-105 and the River. Not only is response in C-106 limited in magnitude but is is also delayed. In actuality tidal response of C-106 might be expected to be greater than C-105 because C-106 is installed nearer to the elevations of the Christina River, increasing the likelihood of hydraulic interconnection between the river and upper sand zone.

Conclusions

Review of geologic and hydrogeologic data reveals that the Columbia Formation of the Terraqua Report of 1984 actually includes considerable quantities of recent river deposits and minor Columbia Formation sediments. Within these sediments there are two distinct water bearing units consisting of sand and gravel. Separating the two water bearing units is a thick silt that acts to hydraulically isolate the upper sand and gravel from the lower. Hydraulic isolation of the two water bearing units is further supported by differing responses with regard to tidal fluctuations.

Review of piezometer placement particularly in light of refined stratigraphic interpretation indicates that Piezometers C-102, C-106, and C-108 are placed within the isolated upper sand zone and should not be considered as part of the network of other Recent/Columbia piezometers.

Additionally, two ground elevation surveys conducted in 1983 and 1985, indicate that ground settlement has been insignificant at the piezometer locations and that a resurvey of piezometer installations is not necessary.

Finally, Figure 2 is forwarded as a possible interpretation of the potentiometric surface of the Recent/Columbia sediments. This interpretation does not include data from the three piezometers installed in the upper sand zone and is based on average piezometric elevations calculated for data collected between July and December 1983. These averages are based on the assumptions that the piezometric levels were taken at random, representing all tidal positions in an attempt to filter out tidal position and precession which were not taken into consideration during data collection. Although minor variations in the flow direction are shown in the figure, the important conclusion is that a relatively flat potentiometric surface exists beneath Cherry Island and that the gradient sharply increases to the west of the site. This is particularly important since the Columbia Formation is more clearly defined along the western edge of the site where the gradient is prominently toward the Delaware and Christina Rivers.

GROUND SURFACE ELEVATIONS

Well No.	July or August 1983	December 1985	Δ Elevation
C-100	15.6	15.4	-0.2
P-100	15.3	15.2	-0.1
C-101	35.2	35.2	0.0
P-101	35.0	34.7	-0.3
C-102	26.3	26.5	+0.2
P-102	26.2	26.2	0.0
C-103	11.1	12.3	+1.2*
P-103	12.1	12.5	+0.4
C-104	10.2	15.8	+5.6**
P-104	10.2	15.6	+5.4
C-105	21.4	21.6	+0.2
P-105	21.4	21.6	+0.2
C-106	27.6	27.2	-0.4
P-106	27.4	27.6	+0.2
C-107	10.3	10.2	-0.1
P-107	9.9	9.8	-0.1
C-108	27.2	27.3	+0.1
P-108	27.1	27.3	+0.2
C-109		38.7	
C-110		31.6	

* Data is questionable
** Piezometer extended

APPENDIX A

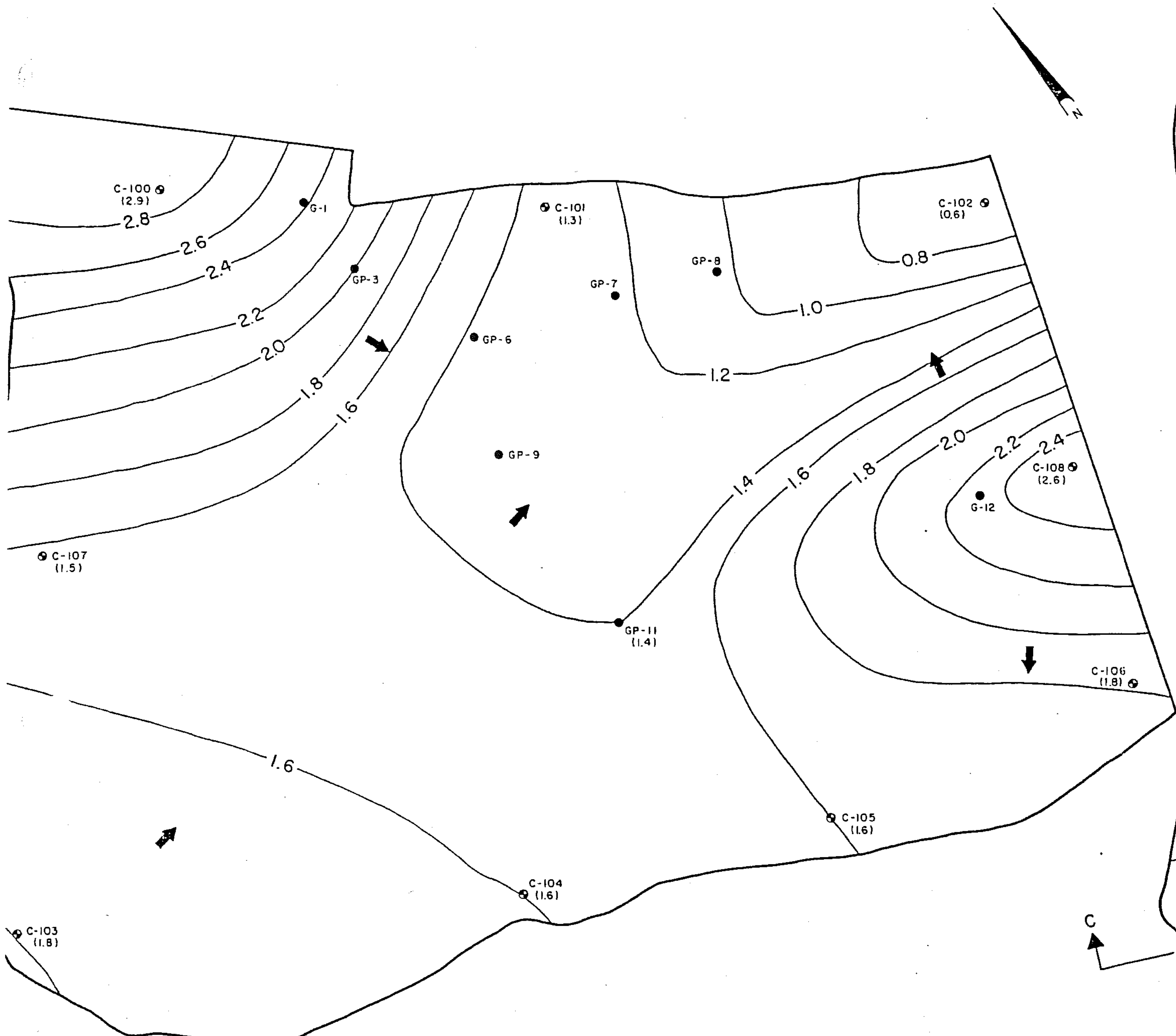


FIGURE 1

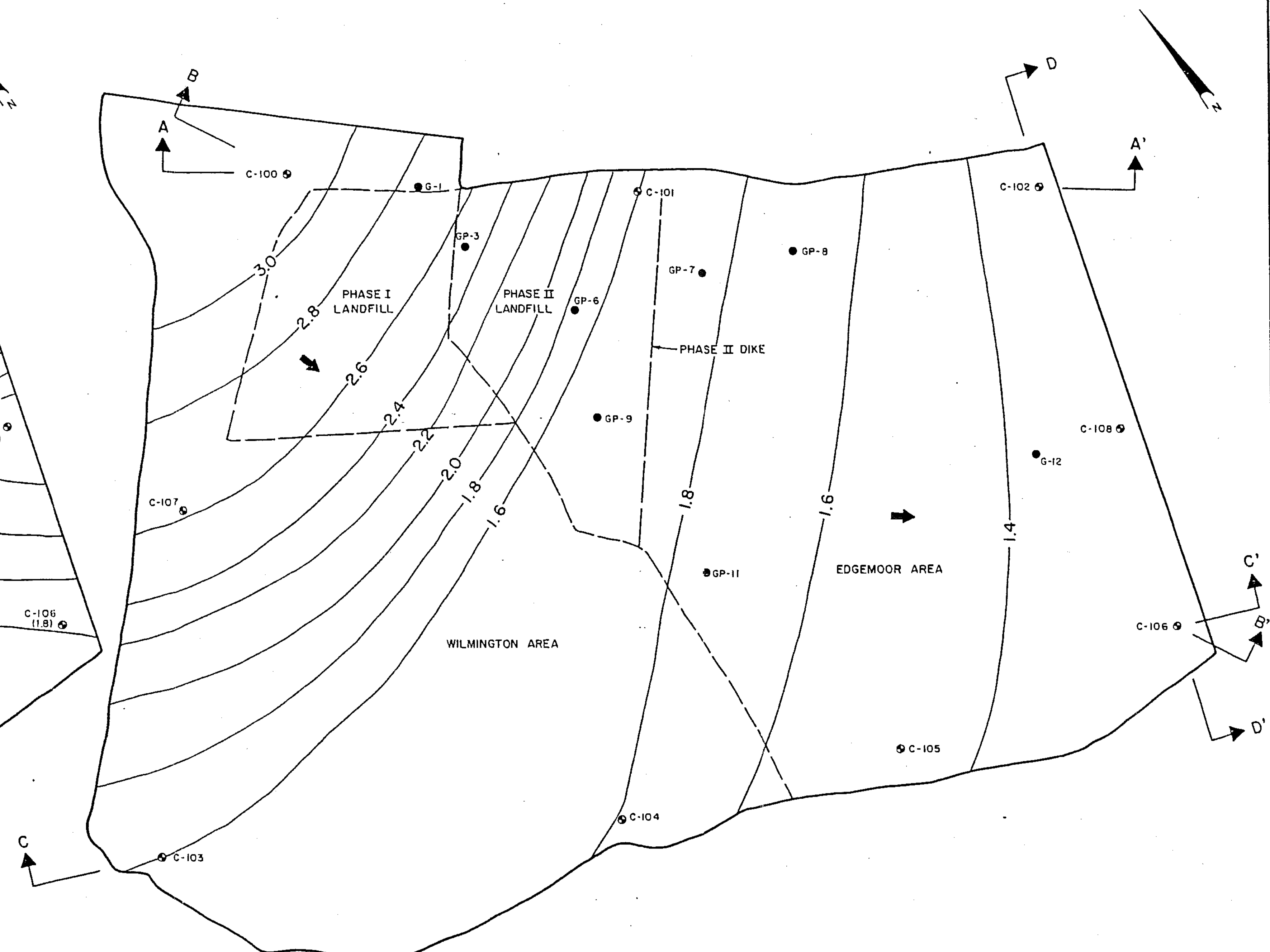


FIGURE 2

LEGEND

GEOTECHNICAL BORING

GEOTECHNICAL BORING WITH DEEP PIEZOMETER

COLUMBIA FORMATION MONITORING WELL

STRATIGRAPHIC CROSS-SECTION

GROUNDWATER ELEVATION

P[illegible]ETRIC SURFACE CONTOUR ([illegible]ET)

GROUNDWATER FLOW DIRECTION

400 0 400 800

SCALE IN FEET

NOTE: FIGURE I MODIFIED FROM TERRAQUA 1984.

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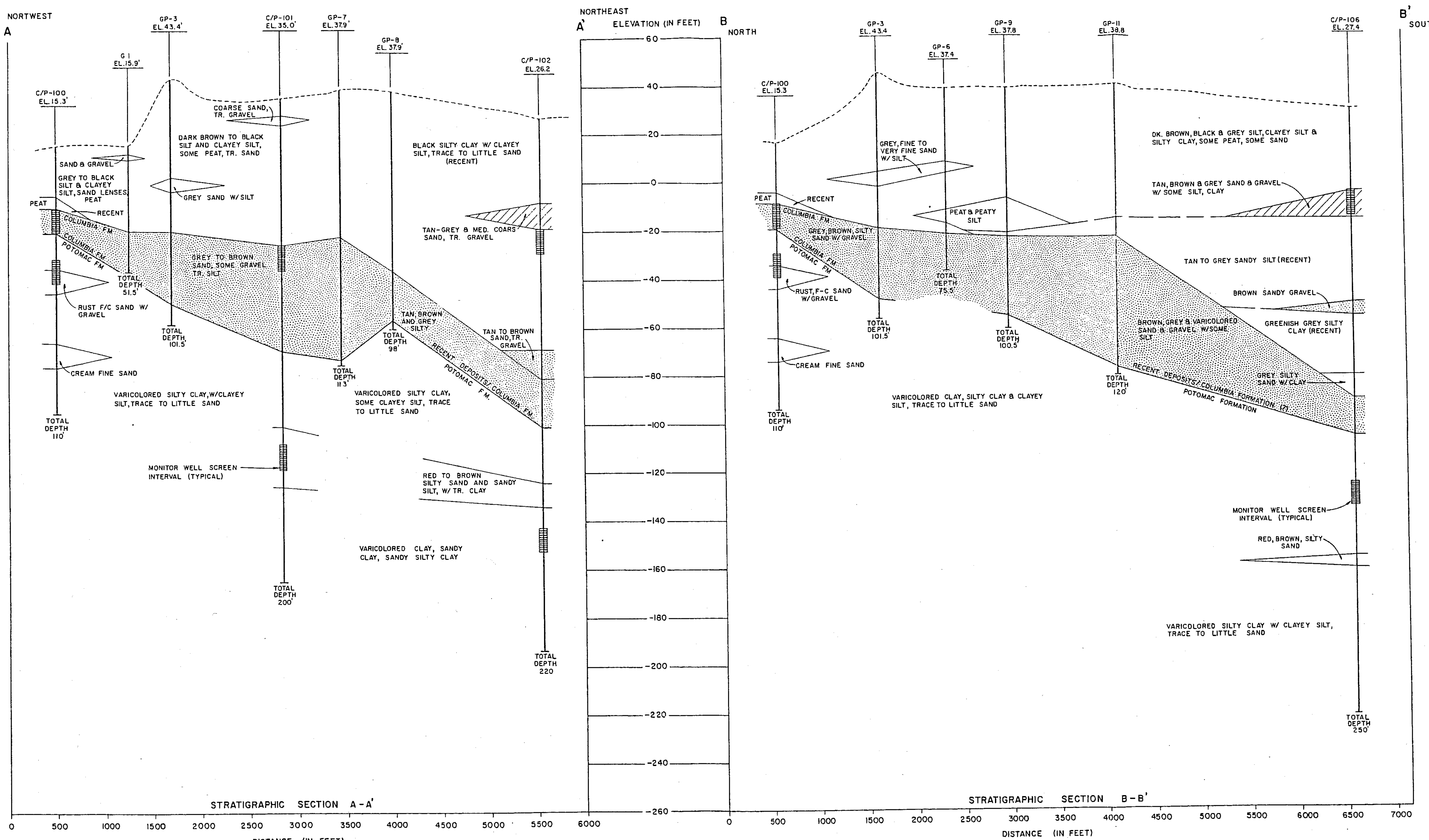
BALTIMORE, MARYLAND JAN. 1987

PHASE II
NORTHERN SOLID WASTE FACILITY - 2
DELAWARE SOLID WASTE AUTHORITY

FIGURE I POTENTIOMETRIC SURFACE MAP COLUMBIA FM.

FIGURE 2 PROPOSED POTENTIOMETRIC SURFACE MAP COLUMBIA/RECENT SEDIMENTS



NOTES:
1. SEE FIGURE 2 FOR CROSS-SECTION LOCATIONS.
2. THESE CROSS-SECTIONS ARE MODIFIED FROM DUFFIELD ASSOC. OCT. 1985.
3. STRATIGRAPHIC SECTIONS ARE BASED ON DRILLER'S DESCRIPTIVE LOGS FOR CONDITIONS ENCOUNTERED BY THE TEST BORINGS AND WELL BORINGS, AND STRAIGHT LINE INTERPOLATION OF CONDITIONS BETWEEN BORINGS. ACTUAL CONDITIONS BETWEEN BORINGS ARE UNKNOWN.
4. STRATA TEXTURAL DESCRIPTIONS ARE A GENERALIZATION OF INDIVIDUAL SAMPLE DESCRIPTIONS INDICATED ON THE DRILLERS DESCRIPTIVE LOGS. FOR LOGS SEE "SITE SUITABILITY REPORT, HYDROGEOLOGIC CONDITIONS AND GEOTECHNICAL EVALUATION OF THE CHERRY ISLAND SITE", PREPARED BY TERRAQUA RESOURCES CORP. DATED 1 FEB. 1984.

KEY
- UPPER SAND ZONE
- LOWER SAND ZONE

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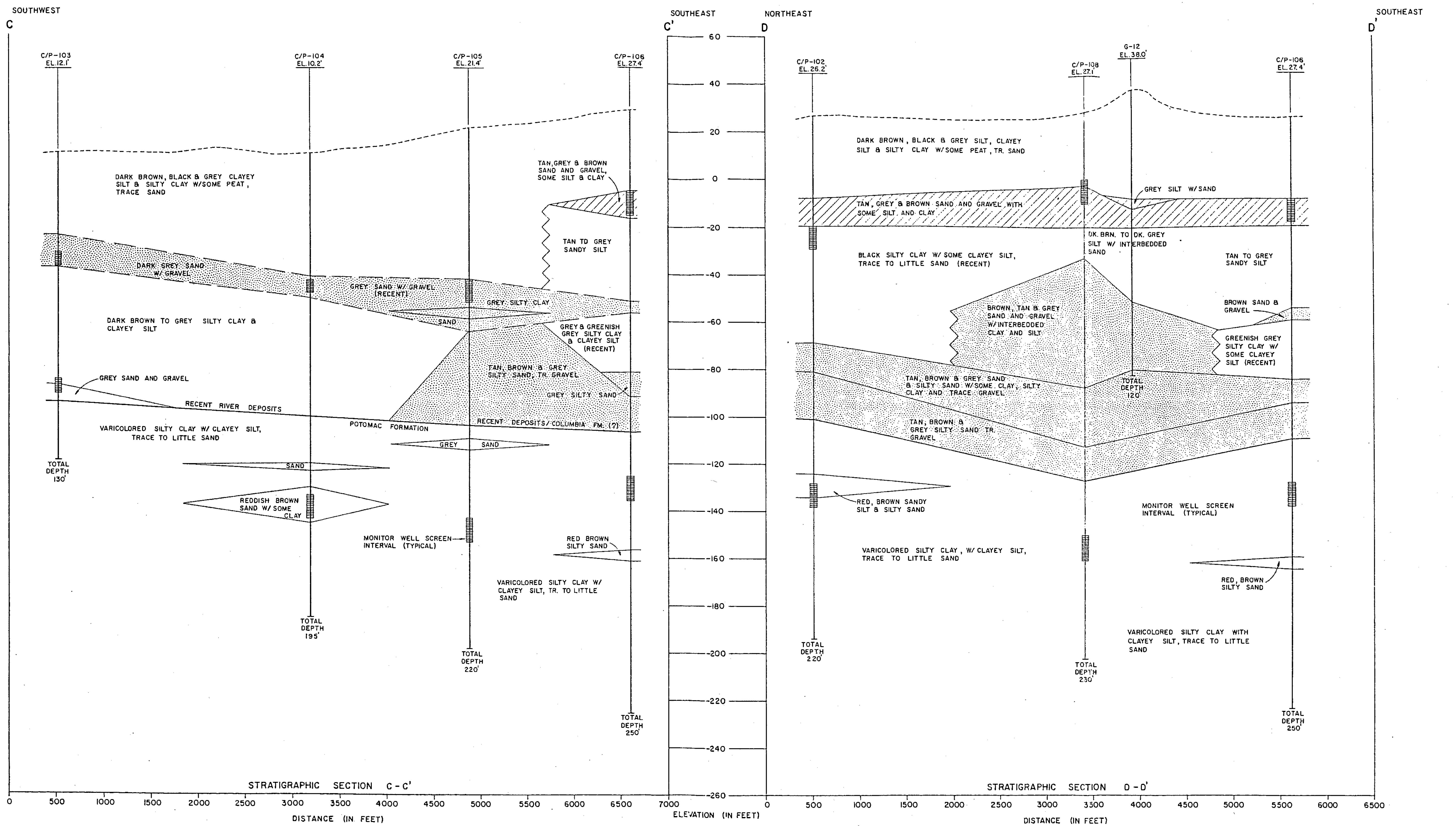
PHASE II
NORTHERN SOLID WASTE FACILITY - DELAWARE SOLID WASTE AUTHORIT

FIGURE 3
STRATIGRAPHIC SECTIONS
A-A' AND B-B'



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KEY

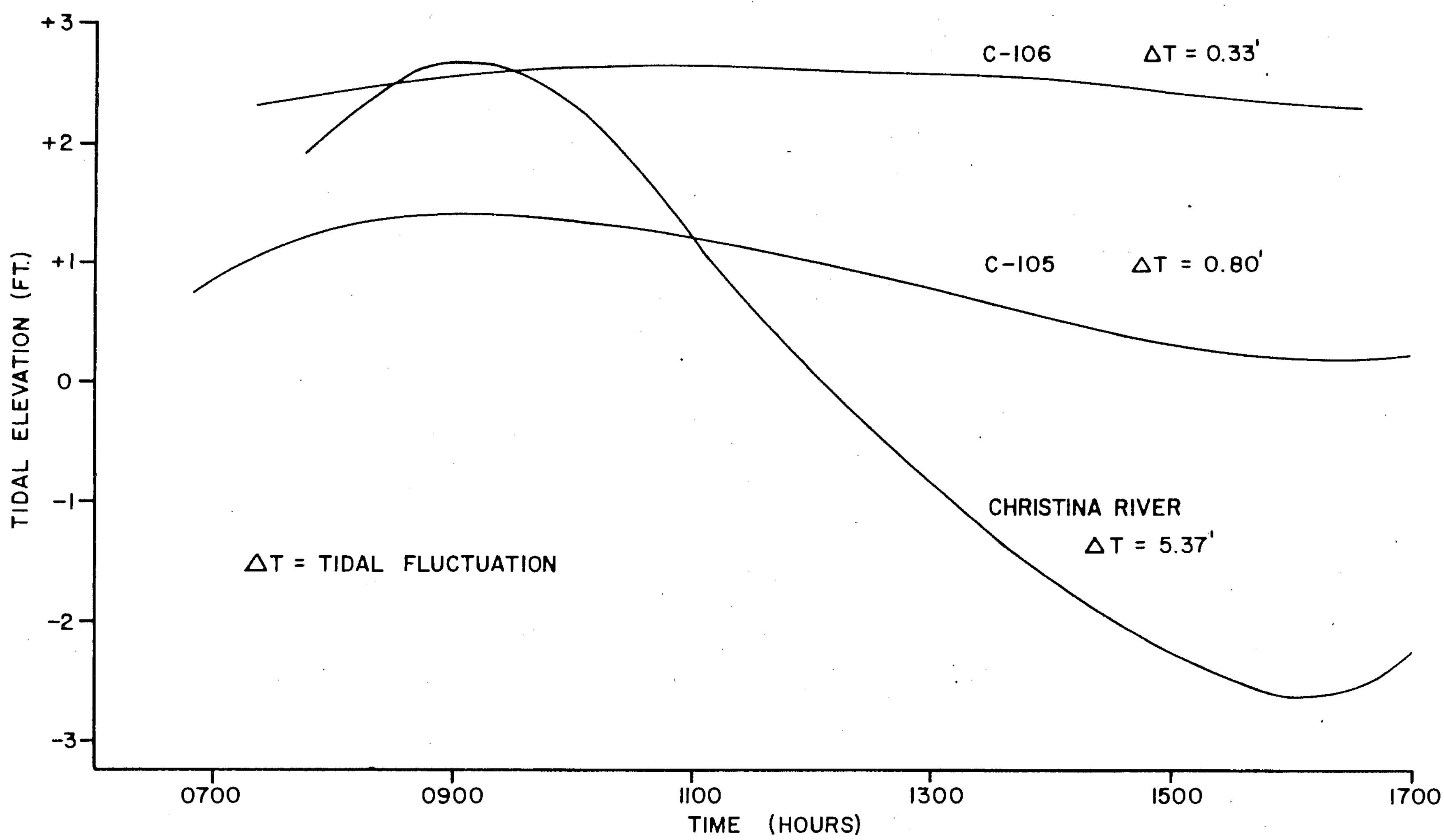
UPPER SAND ZONE

LOWER SAND ZONE

GANNETT FLEMING ENVIRONMENTAL ENGINEERS, INC.
BALTIMORE, MARYLAND JAN. 1987

PHASE II
NORTHERN SOLID WASTE FACILITY - DELAWARE SOLID WASTE AUTHORIT

FIGURE 4
STRATIGRAPHIC SECTIONS
C-C' AND D-D'



GROUNDWATER RESPONSE TO TIDAL FLUCTUATION
DECEMBER 30, 1983

FIGURE 5

3. Gannett Fleming, Inc., "Phase III Design Memorandum," prepared for the Delaware Solid Waste Authority, March 1990.

DELAWARE SOLID WASTE AUTHORITY

NORTHERN SOLID WASTE FACILITY – 2
PHASE III
CHERRY ISLAND
WILMINGTON, DELAWARE

PHASE III DESIGN MEMORANDUM

MARCH 1990

Prepared by
Gannett Fleming, Inc.
King of Prussia, Pennsylvania

DELAWARE SOLID WASTE AUTHORITY

NORTHERN SOLID WASTE FACILITY - 2

PHASE III

CHERRY ISLAND

WILMINGTON, DELAWARE

PHASE III DESIGN MEMORANDUM

MARCH 1990

GANNETT FLEMING, INC.

KING OF PRUSSIA, PENNSYLVANIA

TABLE OF CONTENTS

1.0 Introduction

This report presents a proposal for the development of Phase III of the Northern Solid Waste Facility-2 (NSWF-2) at Cherry Island in Wilmington, Delaware. Cherry Island is used as a dredge disposal site by the Army Corps of Engineers (COE) and the top 58-69 feet of site soil is composed of dredge spoil. The proposal for landfilling Phase III is based on a consideration of existing operations at Phase I and II adjacent to this new landfill area. Pertinent data have been summarized and were used to project the remaining life of Phase II as well as the life of the proposed new landfill area. In order to develop design parameters for the landfill, a geotechnical investigation was conducted at the site. The results are presented in a separate Geotechnical and Hydrogeologic Report, which includes a soils stability analysis.

This proposal includes an estimate of the site's bottom contours which are expected following the US Army Corps of Engineers' (COE) dredge disposal program and a bottom grading plan designed to minimize site work required for bottom preparation. A leachate collection system is proposed to capture percolation through the landfilled material. A preliminary finished grading plan has been established to provide for satisfactory capacity of Phase III and to provide drainage for stormwater runoff. A tentative schedule has also been set to achieve a March 1991 date for commencement of filling in Phase III.

2.0 Waste Disposal

2.1 Existing Waste Disposal Rates

Landfilling operations at Phase I of the Northern Solid Waste Facility at Cherry Island began in October 1985. Phase II operations began in July 1987. Annual solid waste disposal rates to Cherry Island are summarized in Table 1. This Table shows that deliveries to the landfill from October through December of 1985 were 86,149 tons and were 398,739 tons in calendar year 1988. Deliveries to the landfill in calendar year 1989 were 460,278 tons.

TABLE 1

3/22/90

PROJECTED WASTE TONNAGE TO NORTHERN
SOLID WASTE FACILITY AND CHERRY ISLAND

			SOLID WASTE PROJECTIONS			
YEAR	POPULATION (1)	% Annual Increase	TONS TO NSWF	% Annual Increase	TONS TO CHERRY ISLAND (2)	CUMULATIVE TONS TO CHERRY ISLAND
1981			366798			
1982			369849	0.83		
1983			363280	-1.78		
1984			389133	7.12		
1985	414000		446521	14.75	86149	86149
1986	418700	1.14	460511	3.13	458522	544671
1987	423700	1.19	490840	6.59	444000	988671
1988	428950	1.24	533455	8.68	398739	1387410
1989	434500	1.29	581614	9.00	460278	1847688
1990 *	440300	1.33	615871	6.90	518371	2366059
1991 *	445790	1.25	652146	6.90	554646	2920705
1992 *	451280	1.23	690557	6.90	593057	3513762
1993 *	456770	1.22	731231	6.90	633731	4147493
1994 *	462260	1.20	774301	6.90	676801	4824294
1995 *	467750	1.19	819907	6.90	722407	5546701
1996 *	471130	0.72	868199	6.90	770699	6317400
1997 *	474510	0.72	919336	6.90	821836	7139237
1998 *	477890	0.71	973485	6.90	875985	8015222
1999 *	481270	0.71	1030824	6.90	933324	8948545
2000 *	484650	0.70	1091539	6.90	994039	9942584

* Projected figure

(1) Data obtained from New Castle County Planning Commission

(2) Expected tonnage to Cherry Island assumes operation of the EGF at 150,000 tpy with 65% waste reduction.

The average amount of waste disposed at Cherry Island between January 1989 and December 1989 was 38,358 tons per month with a maximum monthly rate of 48,435 tons in May 1989. Recent daily disposal rates are approximately 1,800 tons per day, Monday through Friday, and 600-700 tons per day on Saturdays.

Waste types and tonnages delivered to the site are summarized below for the period January 1, 1989 to December 31, 1989:

Waste Category	Tons Disposed	Estimated Compacted Density (lbs/cy)
Light Industrial/Commercial	302,600	1,100
DRP Residues/RDF	49,548	1,000
Industrial Sludges	5,000	2,000
EGF Ash	46,179	2,000
Asbestos	4,570	500
Municipal Sewage Sludge	17,381	1,900
Municipal Solid Waste	35,000	1,100
Total	460,278	

Based on the above figures, an average compacted density of 1,150 pounds per cubic yard (excluding soil cover) has been achieved in Phase II during calendar year 1989.

2.2 Waste Projections

Deliveries to the site have been greater than projected at the time Phase II was developed. Projections from the Design Memorandum for Phase II for waste disposed for the year ending March 1989 were 326,000 tons; the actual amount disposed during this period was 400,499 tons. Table 1 illustrates the cumulative tons disposed at Cherry Island since 1985. For the period October 1985 through December 1989, a total of 1,847,688 tons of waste have been disposed at Cherry Island. Phase I capacity was estimated at 750,000 tons and Phase II capacity was estimated at 1,949,220 tons for a total capacity of 2,699,220 tons. As of December 1989, an estimated 851,532 tons of capacity remains at Phase II. (Phase I was filled in mid-1987.)

Typically, the quantity of solid waste for disposal is expected to increase in proportion to the population increase in the area. However as shown in Table 1, the projected population increase was 1.1 to 1.2% per year between 1985 and 1989, while the waste tonnage increase to the NSWF was between 3 to 15% per year during the same time. Table 1 shows the projected waste volumes to be disposed at the NSWF during the next ten years assuming that the annual tonnage will increase at a rate of 6.9%, which is the average annual increase of waste delivered to Cherry Island between 1986 and 1989. Based on visual observations of the types of deliveries to the landfill, the high rate of municipal waste generation is believed to be a result of an increase in the light industrial/commercial waste fraction rather than the residential component.

The Energy Generation Facility (EGF) began operations at the end of 1986. The maximum capacity of the EGF is 175,000 TPY based on a 600 TPD throughput and 80% availability. The actual throughput from September 1988 through August 1989 was 140,475 tons. A total of 49,113 tons of ash was generated during this time period and was landfilled. Solid waste projections for disposal of ash at Cherry Island, as shown in Table 1, were made assuming that EGF operations will remain constant at an average 150,000 TPY with an average weight reduction of 65%.

Assuming Phase II continues to be filled at an average compacted density of 1,150 lbs/cy; a total annual tonnage increase of 6.9% per year is delivered to the NSWF; and the EGF maintains a throughput rate of 150,000 TPY; Phase III will be required between February and March 1991. The Authority is currently considering options for filling Phase II without the one-hundred foot setback proposed for the start of the second set of four ten-foot lifts. Without the setback, Phase II capacity will be extended and Phase III will be required between March 1991 and June 1991.

3.0 Landfilling Practices for Phase I and II

3.1 Pertinent Features of Phase I Design

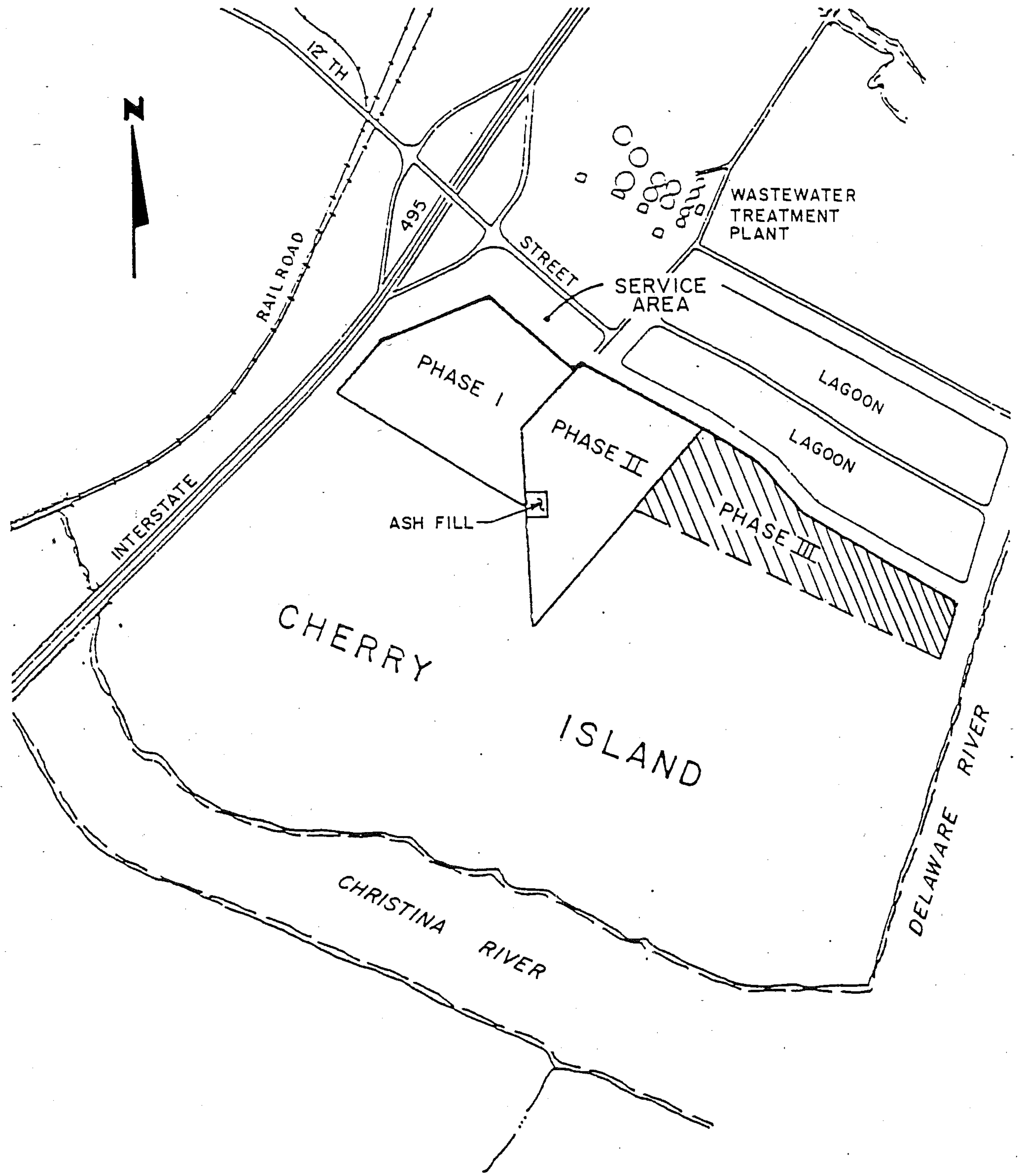
Figure 1 shows the approximate location and boundaries of Phases I, II and III. In Phase I, a dewatering system was used to improve the strength and workability of the soil by lowering the water level in the top few feet of soil.

Dewatering trenches were excavated and the water which was drained from the soils was collected and pumped from the site. Due to the low permeability of the soils, "mounding" of the interstitial water in the soil occurred between the dewatering trenches and additional excavations were needed in some areas.

The leachate collection system incorporated gravel-filled "french-drain" type laterals for collection and delivery of leachate to 6 inch polyethylene headers installed in gravel-filled trenches. Gravity flow in the header conveys leachate to 10 or 12 inch mains at the perimeter, which empty into a sump at the Cherry Island pumping station located at the northernmost corner of the site. From this station, leachate is pumped to the City of Wilmington Wastewater Treatment Plant for treatment.

The approximate maximum leachate flow rate of record for Phase I was 144,000 gpd during a one week period in 1985 when none of the surface area of the site was covered with refuse. The maximum flows observed with one lift of solid waste in place were 57,000 gpd. The leachate flow rate predicted for Phase I upon closure by the Water Balance Method was 23,000 gpd.

Gas vents constructed for Phase I are four foot diameter concrete pipes filled with stone. A PVC perforated pipe in the center of the concrete pipe extends down to the bottom of the landfill.



DELAWARE SOLID WASTE AUTHORITY
NORTHERN SOLID WASTE FACILITY – 2
PHASE III AREA
LANDFILL DESIGN MEMO

FIGURE 1
PHASE III AREA

3.2 Pertinent Features of Phase II Design

Phase II encompasses approximately 43 acres and is bounded on the west and north by dikes constructed of dredge spoils by the COE. An earthen dike was constructed along the eastern boundary and joined the west and north dikes in order to segregate the landfill area from COE dredge disposal activity. The landfill bottom was designed to drain from the northern and southern portions of the site to a central leachate header pipe constructed across the site.

The configuration selected for Phase II includes a total of eight ten-foot lifts with reinforcement at the bottom of the first lift. Following the completion of the first set of four ten-foot lifts, a second set of four ten-foot lifts was designed to begin with the toe of slope set back 100 feet from the top of the first four ten-foot lifts. A synthetic reinforcing grid was placed on the landfill bottom prior to filling to reinforce the areas around the northern and eastern edges of the landfill.

The leachate collection system was designed to function in a manner similar to the Phase I system. Phase II leachate is conveyed to the Phase I system. Average leachate generation rates for Phase I and II combined are 20,000 to 30,000 gpd. Flows of 117,000 gpd were observed during 1987 when none of the Phase II surface area was covered with refuse. Flows of 106,000 to 163,000 gpd were observed during the summer of 1989 due to wet weather.

The Phase II area was constructed in three sections; A, B, and C. The Landfill bottom and leachate collection system was prepared for Section A and the first 10-foot lift of waste was placed on this portion. A berm was constructed between Section A and Sections B and C so that stormwater could be diverted away from the leachate collection system during the early stages of filling. Leachate collection systems for Areas B and C were installed and connected just prior to the commencement of filling.

As shown in Figure 1, EGF ash is being landfilled in a separate area in Section A of Phase II.

4.0 Description of Phase III Site

4.1 Site Configuration

Phase III comprises approximately 41 acres and is bounded on the north and east by dikes constructed of dredge spoils by the COE. Phase III is bounded on the west by a dike constructed as part of the Phase II landfill construction.

A baffle dike constructed of dredge spoil by the COE runs in an east-west direction through the center of the Phase III area. A sluice and drainage pipe located on the eastern dike serve to drain the entire eastern dredge disposal area during dredge deposition. Figure 2 shows the expected bottom contours after the COE dredge deposition is completed in June 1990.

4.2 Site Soils

A detailed investigation of Phase III site soils was undertaken by Gannett Fleming Engineers and the results are presented in the Geotechnical and Hydrogeological Report.

4.3 Groundwater Conditions

A description of Phase III groundwater conditions is presented in the Geotechnical and Hydrogeological Report.

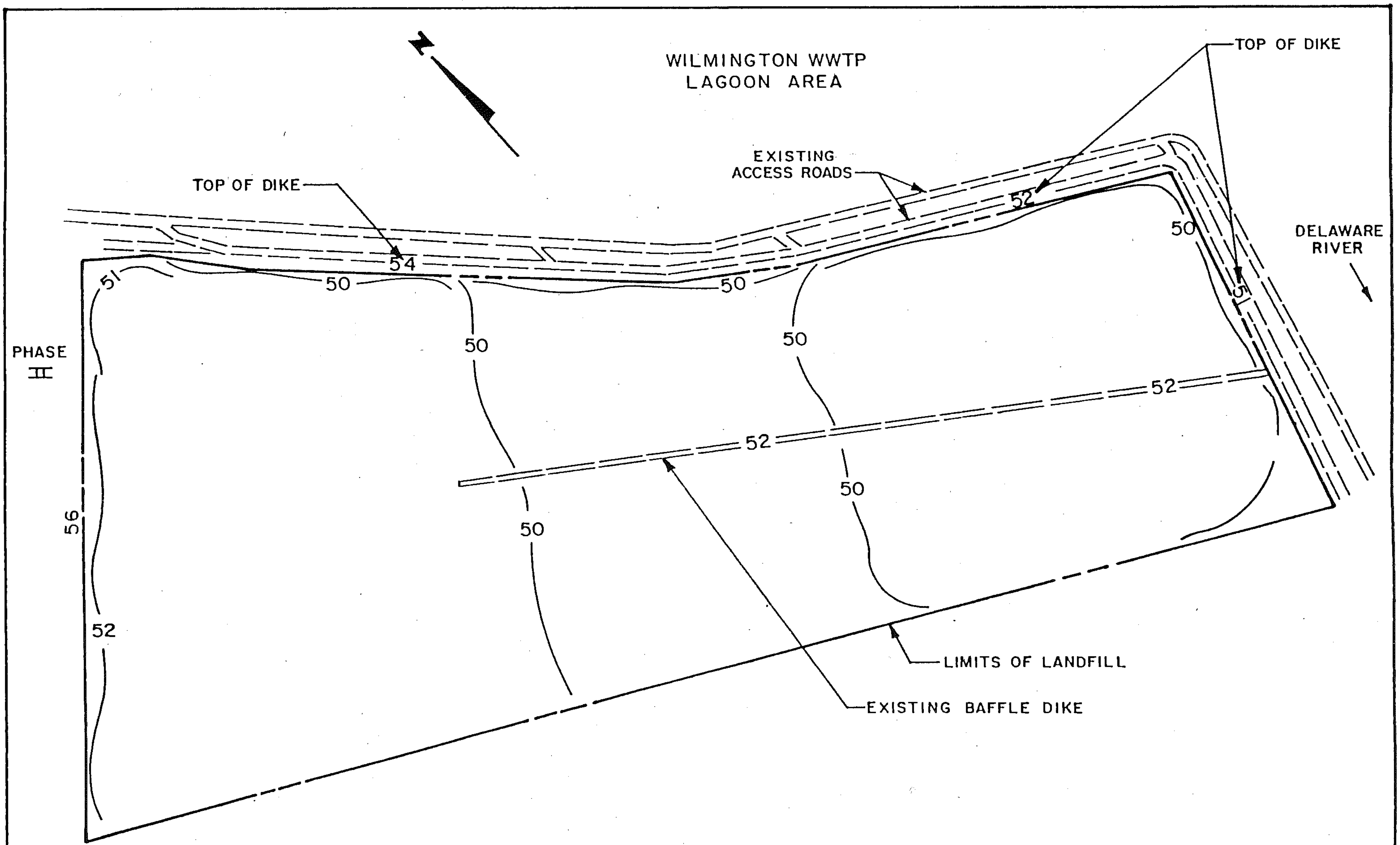


FIGURE 2

EXPECTED BOTTOM CONTOURS
AFTER COE DREDGING

SCALE: 1" = 250'

5.0 Phase III Landfilling Plan

5.1 Corps of Engineers' Activities

The COE is currently using the eastern portion of Cherry Island (Edgemoor Area) for dredge deposition. The most recent dredging was conducted in August 1989. The COE plans to initiate dredging on March 15, 1990 and continue until June 1990. This dredge deposition is expected to result in the addition of three to five feet of dredge spoil in the Phase III area. As a result of the Corps' activities, a separate investigation for dewatering or drying the Phase III site is underway.

Following the next dredging scheduled for March 1990, the COE plans to use the western portion of the Cherry Island site (Wilmington Area) for dredge deposition. Therefore, as discussed with DNREC, a dike separating the southern edge of Phase III from the remaining dredge disposal area will not be needed. Section 5.8 discusses the stormwater management practices which will be used to segregate Phase III from the remainder of the Edgemoor Area. The existing baffle dike and sluice in the Phase III area must be removed as part of the Phase III construction.

5.2 Landfill Bottom

Because COE dredge deposition was delayed until August 1989, the topography in the Phase III area could not be surveyed. The topography for this design memorandum has therefore been assumed to be approximately five feet above grades measured during the hydrogeological investigation. The assumed contours are not likely to coincide with elevations left after the COE dredging activity expected to take place in March of 1990.

The site must be resurveyed, following the completion of dredging activities, to establish an accurate starting point for site preparation.

Observations of field conditions following the COE dredge deposition indicated that the entire site was covered with a few feet of water. Four months after the dredge deposition, observations of field conditions indicated that a thin, dry crust was present at the edges of the site and standing water was present in the center of the site. Areas where standing water exists over the dredge spoil will not support construction equipment, even after ponded water is removed. The actual condition of the site bottom will not be known for a number of months following the completion of the March 1990 dredging.

The proposed landfill bottom grading plan is shown in Figure 3. To achieve proper leachate drainage, the bottom slopes from the south and north to the center of the site. Bottom slopes are proposed to be approximately 1.5 percent.

Consolidation of subgrade soils is expected to cause substantial settlement of the landfill bottom. Settlement will be most pronounced in the center of the site because the greater amounts of solid waste will cause heavier surcharging compared to areas closer to the edges of the landfill. This differential settlement will accentuate downward slopes towards the center of the landfill such that they will exceed 2% following settlement. Therefore, the bottom grading plan has been designed to take advantage of this settlement, provide for a balanced site for cut and fill using all on-site materials, and still provide adequate leachate collection.

5.3 Dewatering Plan

As with Phase II soils, Phase III soils are very poorly drained. The COE is scheduled to deposit three to five feet of dredge material at the site between March 15 and June 15, 1990. Unless measures are taken to dewater the site, soils will not be stable enough for construction until at least twenty-four months after deposition is completed.

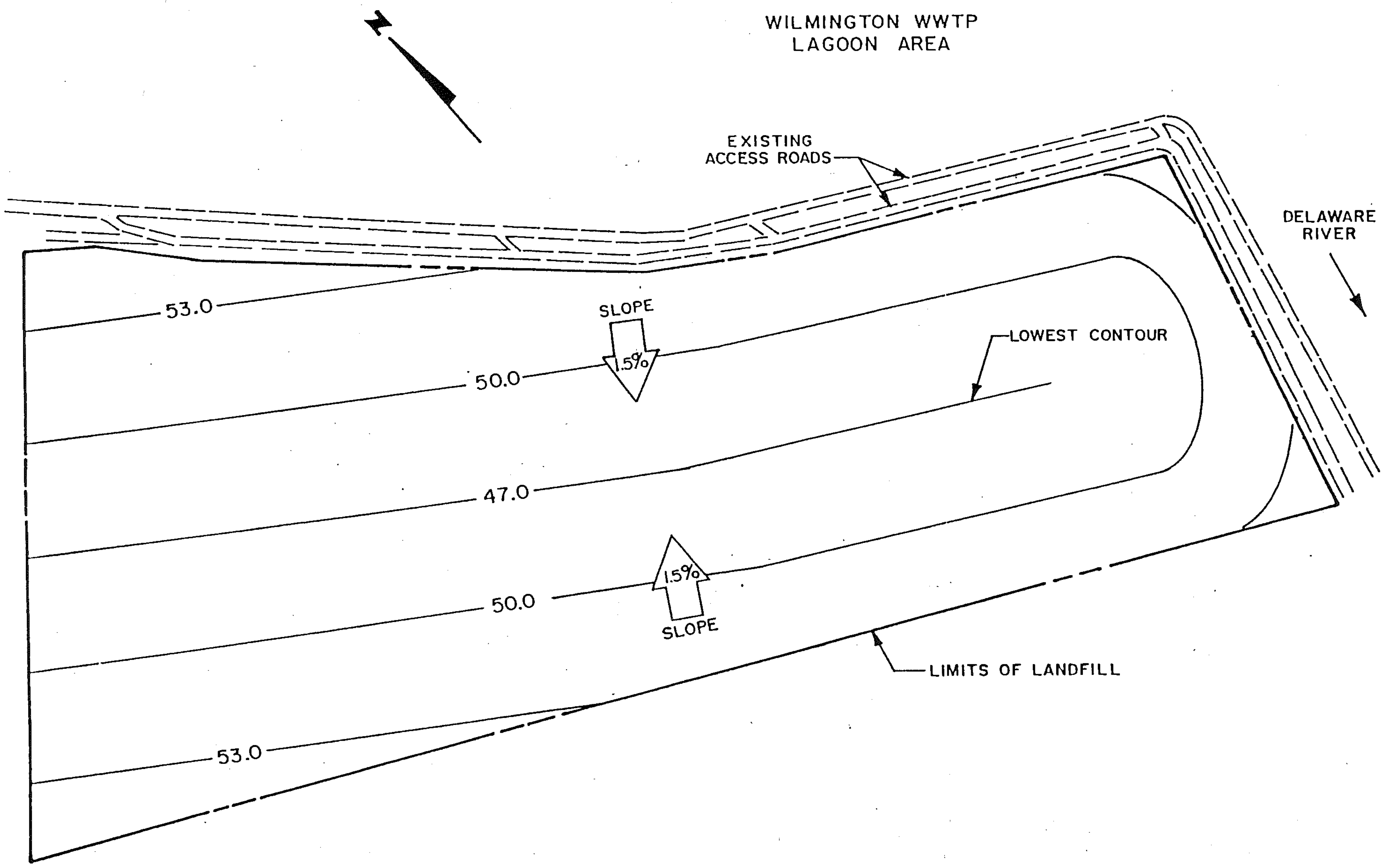


FIGURE 3

LANDFILL BOTTOM GRADING PLAN

SCALE: 1"=250'

The characteristics of the dredge spoil are such that standing water forms when the spoil is excavated. This standing water is neither a regional ground water table nor perched groundwater, but rather it is interstitial water in the soil that will drain by gravity if an outlet is provided. The rate of draining is governed by the overburden pressure and the length of the water's flow path. Therefore, either increasing the amount of overburden or decreasing the flow path will result in a more rapid dewatering rate.

One option for enhancing water removal at the site is to install a drainage material on the soil surface prior to the initiation of the March 15, 1990 dredge deposition. Various woven and non-woven geotextiles and preformed plastic drainage nets were evaluated to determine the drainage capacity. Calculations performed, using laboratory-derived consolidation parameters, indicated that the time required for the upper five feet of dredge spoils to reach normal consolidation would be reduced from approximately two years to six months with the insertion of this draininage material.

Plans, specifications and cost estimates were prepared for the installation of this system; however, the Corps of Engineers did not allow the Authority access to the Phase III site in order to proceed with installation.

Depending on the site conditions following the completion of the dredge deposition, additional dewatering methods may be needed in order to meet the proposed construction schedule, or the Authority may decide to pursue other landfilling measures.

5.4 Preliminary Landfill Configuration

Figure 4 shows the preliminary Phase III fill configuration. Side slopes were established at 3:1 (horizontal: vertical) to match the practice in Phases I and II. Solid waste lifts will be about ten feet thick, consisting of nine and one-half feet of waste and six inches of daily cover. The height of the landfill is about 80 feet above the landfill bottom. A 100 foot setback at the top of the first 40 feet has been included on the northern, eastern and southern sides of the landfill to provide for the necessary stability. Figure 4 also illustrates the location of the ash fill area.

Runoff from the finished top of the landfill will be conveyed down the side slopes in drainage channels to the perimeter of the finished fill area on the north side of the landfill. As described in Section 5.8, a sediment basin will be located in the northeastern portion of the Cherry Island site.

Figure 4 also illustrates the proposed access roads to the site. Access to the Phase III area will be along the northern edge of the site. The existing access to the landfill at the small load collection area will be rerouted to provide entry to the Phase III site along the boundary of Phase II and III.

5.5 Estimated Capacity

The volume of the fill configuration shown in Figure 4 is about 2,708,000 cubic yards. Assuming that 1 foot of sand (65,760 cubic yards) will be added to the base of the landfill as part of the drainage system, the total volume of the landfill available for waste and cover material is 2,642,240 cubic yards.

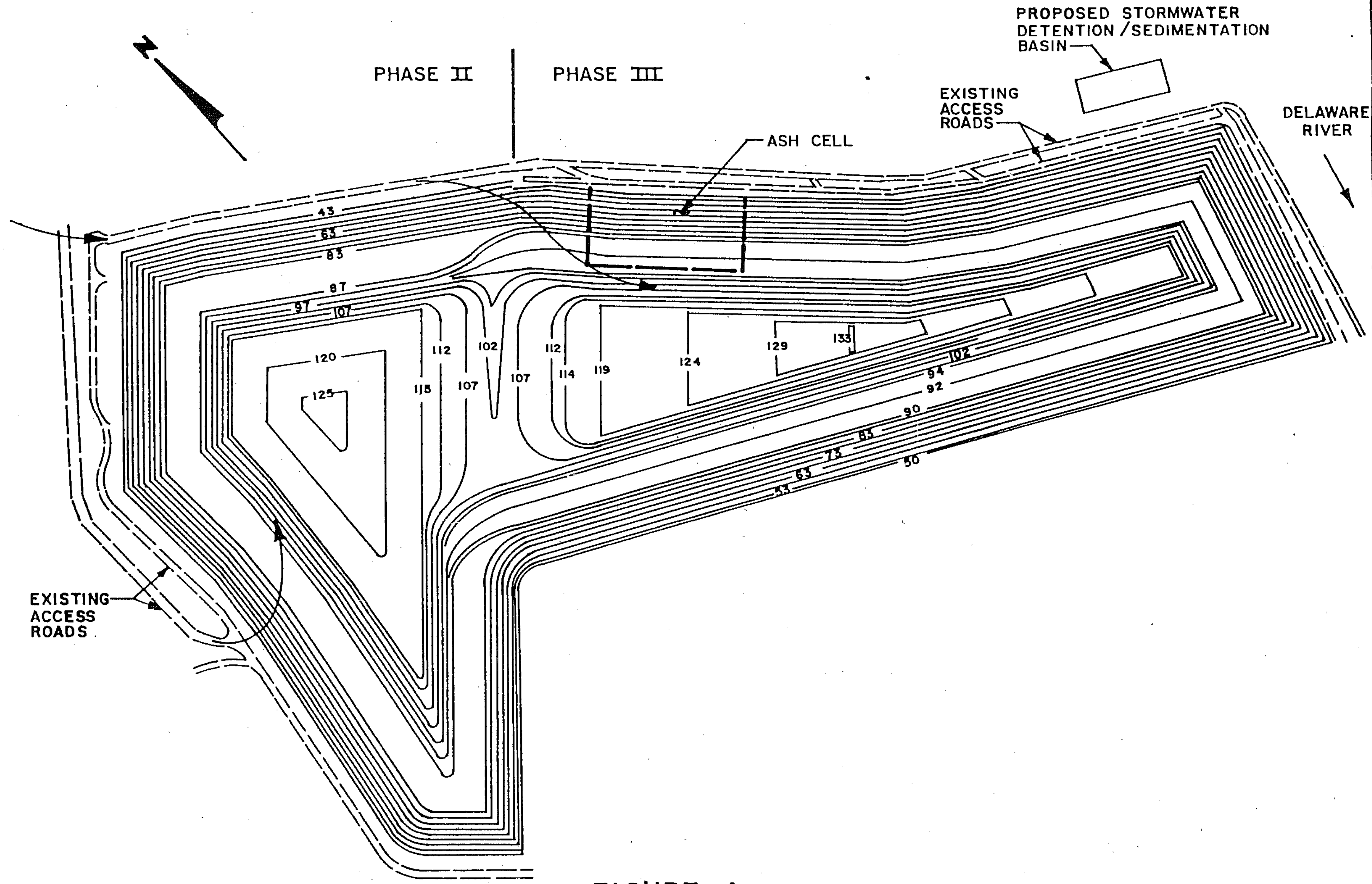


FIGURE 4

EXISTING PHASE II LANDFILL AND
PROPOSED PHASE III LANDFILL
CONFIGURATION

SCALE: 1" = 400'

The average compacted density of waste in Phase II currently is 1,150 lbs. per cubic yard. Assuming a compacted density of 2,500 lbs. per cubic yard for cover material; 5% by volume of daily cover and 4.5 feet of final cover; approximately 146,677 tons (117,342 cubic yards) of cover soil will be needed. The remaining capacity of Phase III for solid waste, is therefore about 2,103,292 cubic yards or 1,209,393 tons. Assuming delivery rates of 11,035 tons/week, during the period of 1991 to 1992, Phase III lifetime is estimated at 110 weeks, or 2.1 years.

Assuming continued operations of the EGF at 150,000 TPY with 65% weight reduction, the average weekly delivery of EGF ash will be 1,010 tons per week (1,010 cubic yards per week assuming a compaction density of 2,000 lbs/cubic yard). The volume of the ash cell is estimated at 126,198 cubic yards. Assuming a 1 foot sand layer, 5% by volume of daily cover and 4.5 feet of final cover, the ash cell capacity is estimated at 125 weeks, or 2.4 years.

5.6 Leachate Management and Collection

Leachate generation rates for the Phase I and II presently average between 20,000 to 30,000 gpd. As stated in Section 3.0, the quantity of leachate generated for Phase II, as predicted by the U.S. Environmental Protection Agency's "Water Balance Method", was 27,000 gpd. Using the same method for Phase I yields 23,000 gpd, for a total for Phase I and II of 50,000 gpd.

Leachate generation rates for Phase III were estimated using the Hydrologic Evaluation of Landfill Performance (HELP) model, Version 2. The average annual rate of leachate generation for Phase III is estimated to be 31,000 gpd. The maximum daily leachate generation rate is predicted to be 50,600 gpd.

The layout of the Phase III leachate collection system is shown in Figure 5. The ash cell has been designed with a separate leachate collection system. The systems will consist of perforated, corrugated six-inch lateral pipes and eight inch header pipes in geotextile wrapped, gravel filled trenches below grade. One foot of sand will be placed on top of the collection systems. The systems have been designed to minimize head pressure and minimize ponding over the underlying dredge spoil liner. Collection pipes are designed to drain by gravity to a sump system. Manholes and cleanouts are located along the perimeter of the site to allow for proper maintenance of the systems. A temporary flow reducing orifice plate will be installed between the Phase II and Phase III leachate collection systems to reduce peak flows generated during placement of the first lift in Phase III.

5.7 Landfill Capping

The proposed landfill configuration offers a large surface area for infiltration of stormwater. The fairly gentle top slopes (two percent) are predicted to convey about 15 percent of precipitation off the top of the fill. The 85 percent that is predicted to infiltrate will generate an average annual leachate flow of about 30 million gallons. Most of this leachate can be avoided if the top of the landfill is capped with a barrier to liquid such as a clay or a synthetic membrane. A cap would entail about 40 acres and could be expected to preclude about 90 percent of precipitation from infiltrating into the fill depending on the system selected.

The proposed cap system for Phase III will be evaluated and presented in a separate technical memorandum.

5.8 Surface Water Management

During the first stages of site preparation, an earthen berm will be constructed along the southern edge of Phase III and along the eastern edge of Cell A to keep stormwater from the remainder of the Edgemoor area from entering the active filling area. The staging of filling is shown in Figure 6.

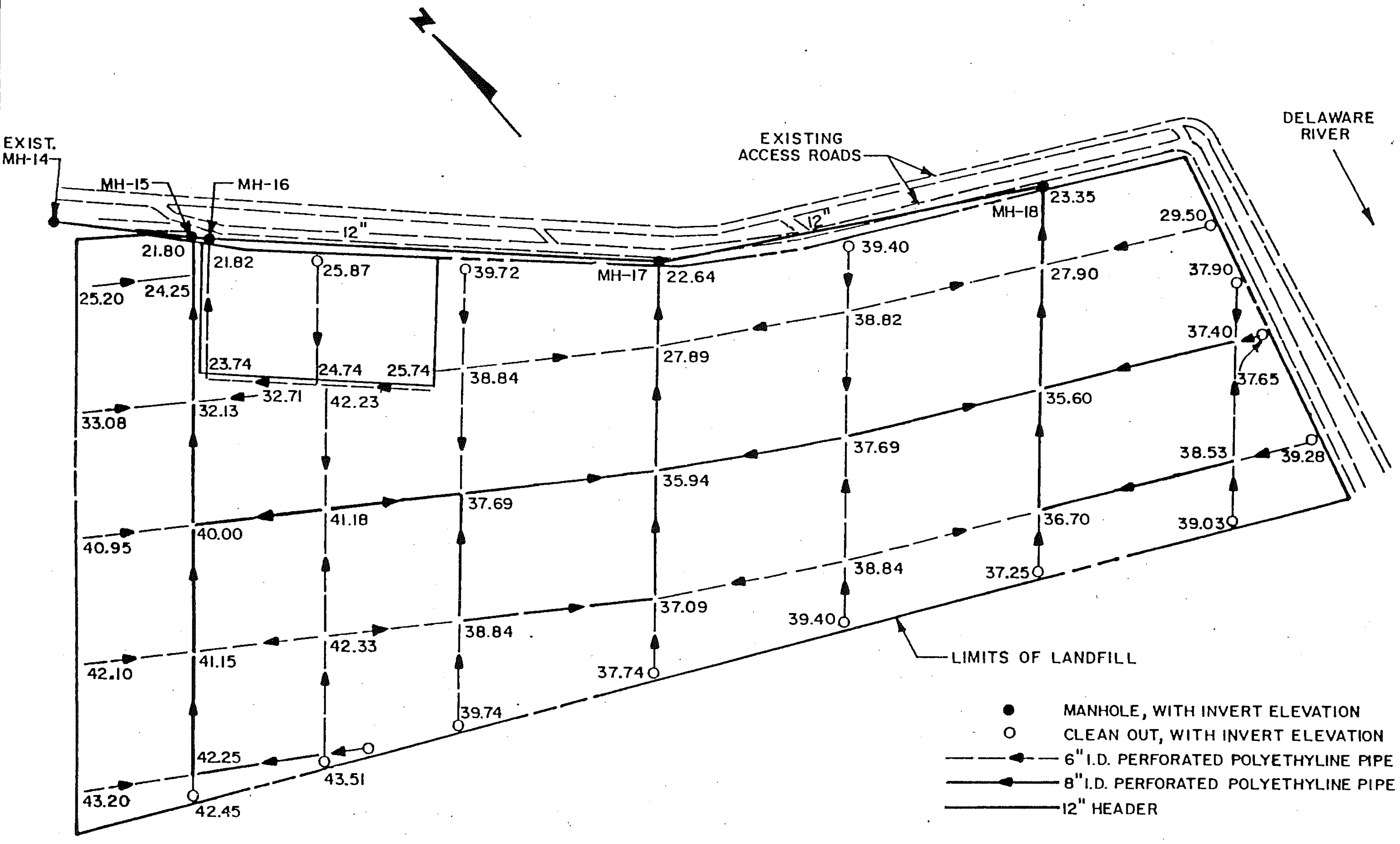


FIGURE 5

PROPOSED LEACHATE COLLECTION SYSTEM

SCALE: 1"=250'

Because of these berms, stormwater falling on Cells B and C will also be kept out of the leachate collection system until solid waste placement begins.

Stormwater runoff from the site during bottom preparation of Cell A will be directed over the berm and out of the existing sluice along the eastern side of the landfill. Similarly, as Cell B is developed, stormwater will be directed to Cell C and out of the existing sluice. Prior to the installation of the leachate collection system in Cell C, a stormwater detention/sediment basin sized for a 2-hour ten-year storm (approximately 72,000 ft^3) will be constructed in the northeastern area of Cherry Island.

After the entire landfill area has been filled with the first ten-foot lift, stormwater runoff from the landfill will be directed down the side slopes of the fill around the entire perimeter of the landfill. This runoff will be directed into perimeter drainage swales and conveyed on the east and west sides of the fill into the northern drainageway and into the stormwater detention/sediment basin.

5.9 Landfill Operation

In situ dredge spoils will act as an impermeable liner for the landfill. A dewatering system may need to be installed, depending on the results of the landfilling feasibility study in progress.

The landfill bottom will be graded to a uniform 1.5% slope. Grading will begin at the lowest contour and proceed up-slope in both directions. Excess soil from bottom grading and COE dedging activity will be stockpiled at the far eastern end of the site until it is needed for berm construction. A leachate collection system will be installed according to Figure 5 only for the Ash Cell and Cell A in order to minimize inflow of stormwater to the leachate system.

After the leachate collection system is completed, a sand drainage layer will be placed and graded to maintain approximately one foot thickness over the site. As outlined in the Geotechnical and Hydrogeological Report, geotechnical reinforcing materials will be placed within the sand layer as shown in Figure 2.2 of the Hydrogeological Report.

Materials to be landfilled will be placed according to the sequence depicted in Figures 6 and 7. Prior to completion of the first lift in Cell A, the leachate collection system for Cell B will be completed. In a similar fashion, the leachate collection system for Cell C will be completed prior to completion of the first lift in Cell B. Access to the site during landfilling will be over roads on existing dikes as shown in Figure 4. The second lift for the entire site will be filled according to Figure 7. Since the Ash Cell will be filled at a rate of approximately 1/16 that of the Cells A, B and C, it has been sized accordingly. After the first two lifts are in place, final cover soil placed on the eastern side of Phase II will be displaced and the third lift of Phase III will directly abut the eastern side of Phase II.

5.10 Sequence and Schedule of Site Preparation Activities

The following sequence is proposed for Phase III landfill site preparation:

1. Sediment control facilities. A berm will be constructed along the southern perimeter of Phase III to divert stormwater to non active areas of the Edgemoor site.

2. Bottom grading. Depending on the condition of the site following COE deposition, bottom grading will be carried out by appropriate excavation and filling. If accessible, the entire site will be graded. The baffle dike will be removed at this stage.

3. Stormwater diversion berm. An earthen diversion berm separating Cell A from Cells B and C will be constructed. Stormwater from Cell A will be allowed to settle in Cells B and C prior to its discharge through the existing sluice gate on a temporary basis until the stormwater basin is constructed.

4. Leachate conveyance system. New gravity sewers from existing Manhole No. 14 to new Manholes No. 15 and 16 will be constructed.

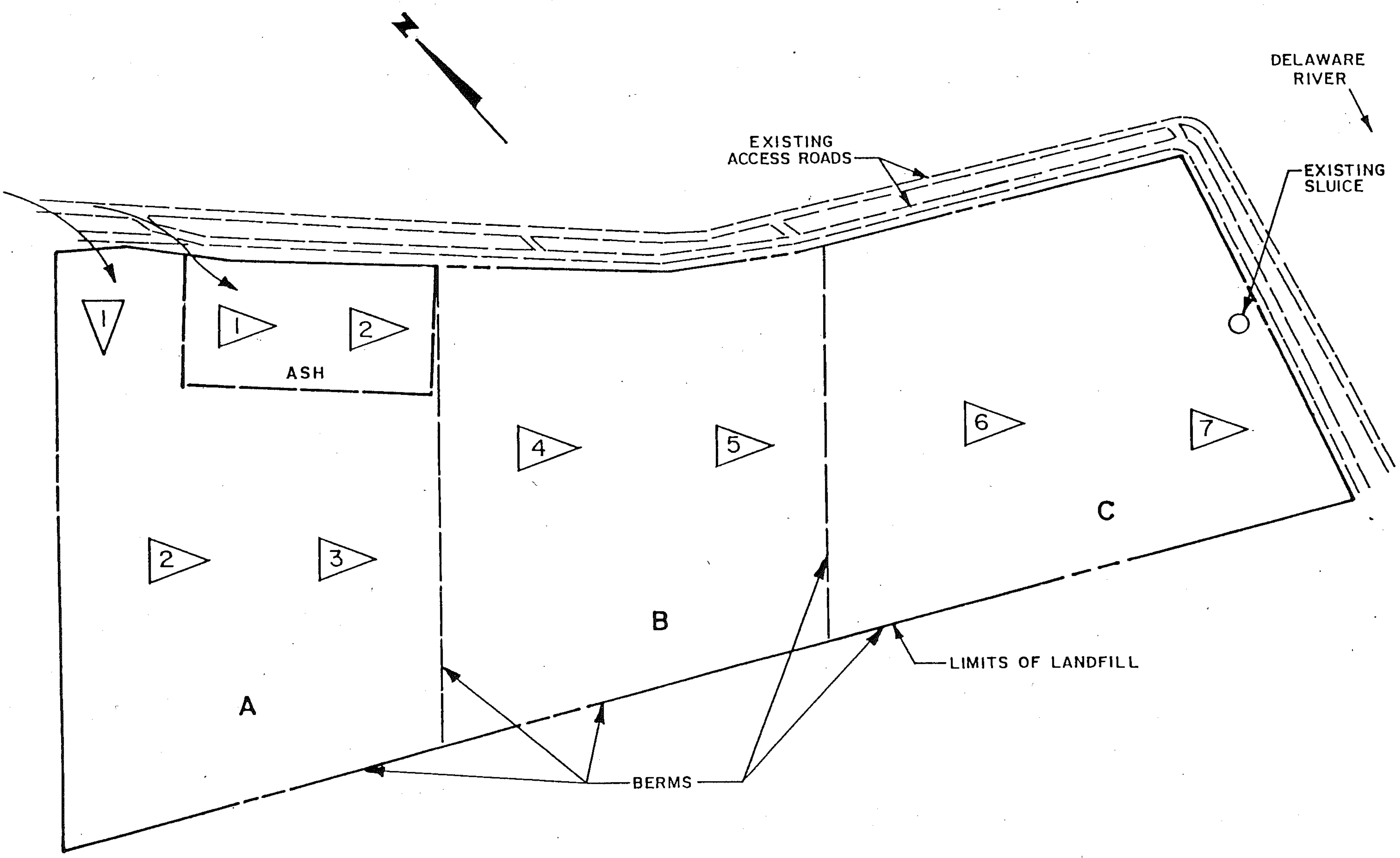


FIGURE 6

SEQUENCE OF OPERATION
FIRST LIFT

SCALE: 1"=250'

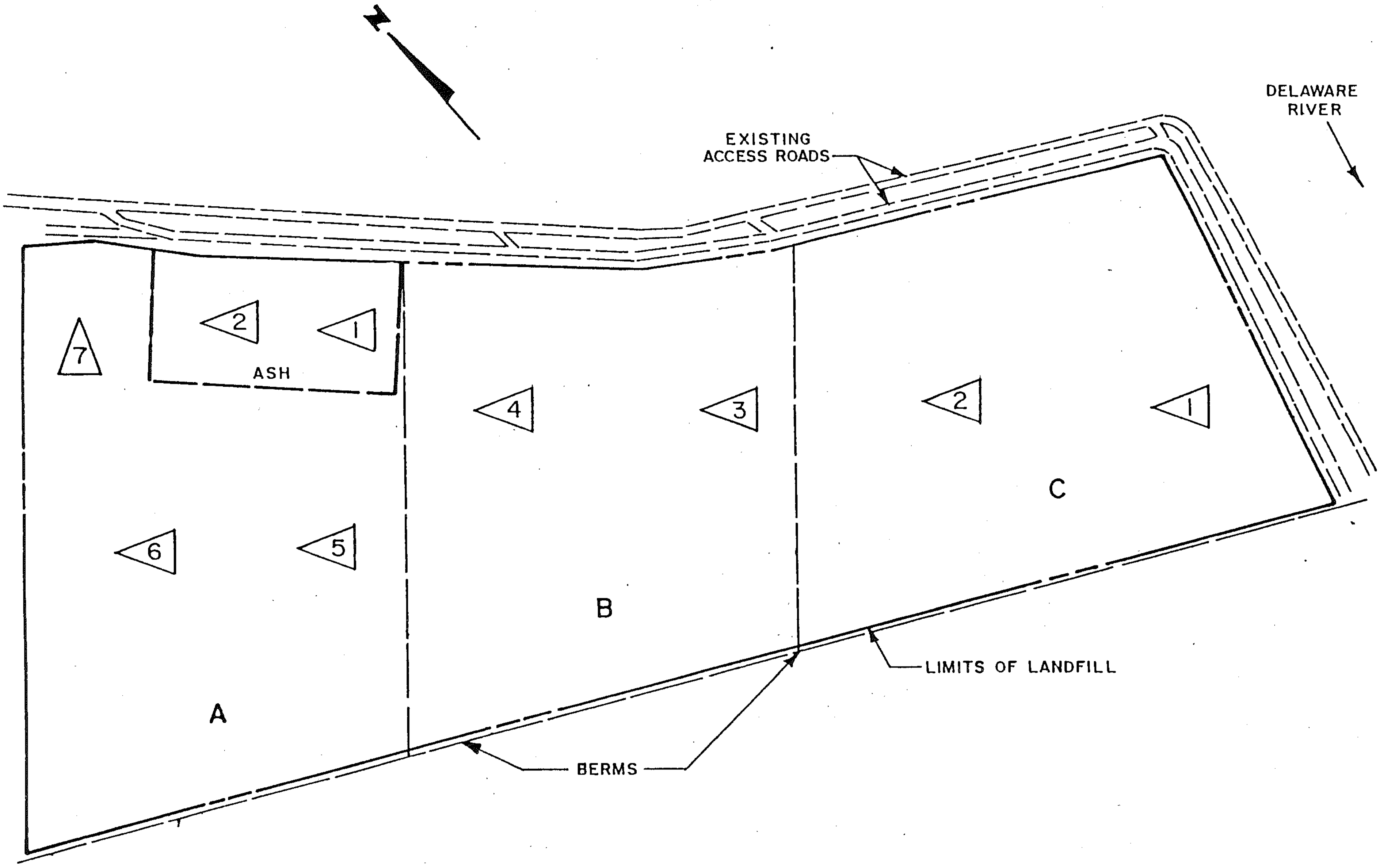


FIGURE 7

SEQUENCE OF OPERATION
SECOND LIFT

SCALE: 1"=250'

5. Cell A - leachate collection system. The western portion of the leachate collection system, including the Ash Cell, will be installed. Collection pipes will be placed in their gravel bedding and connected to Manholes No. 15 and 16. Geotextile reinforcement and sand will be replaced in Cell A.

6. Solid waste disposal. The first lift of solid waste and/or ash will be placed in Cell A.

7. Cell B - leachate collection system. The portion of the leachate collection system in Cell B will be installed in a manner similar to Cell A. Laterals and headers will be connected to establish a complete system and geotextile reinforcement and sand will be placed.

8. Solid waste disposal. The first lift of solid waste will be placed in Cell B.

9. Sediment basin - Prior to the start of installation of the leachate collection system in Cell C, the sluce gate will be removed and a stormwater basin will be constructed in the northeastern section of the Cherry Island site.

10. Cell C - leachate collection system. The portion of the leachate collection system in Cell C will be installed in a manner similar to Cells A and B. Laterals and headers will be connected to establish a complete system and geotextile reinforcement and sand will be placed.

11. Solid Waste Disposal. The first lift of solid waste will be placed in Cell C.

Establishing a realistic schedule for the work requires an understanding of the probable duration of the construction tasks involved in preparing the site. Practical production rates for one construction unit (such as one dozer, one backhoe, one pipelaying crew, etc.) on the tasks at NSWF would be:

Task	Average Production Rate for One Unit
Stormwater basin	125 CY per day
Bottom grading	375 CY per day
Leachate system installation	300 LF per day
Placing fabric and drainage layer	15,000 SF per day

The schedule shown in Figure 8 illustrates the timing, duration and number of construction units to bring Cell A on line. A number of points should be recognized:

1. The schedule depends on good weather. Wet conditions will hamper or prohibit most of the construction tasks.

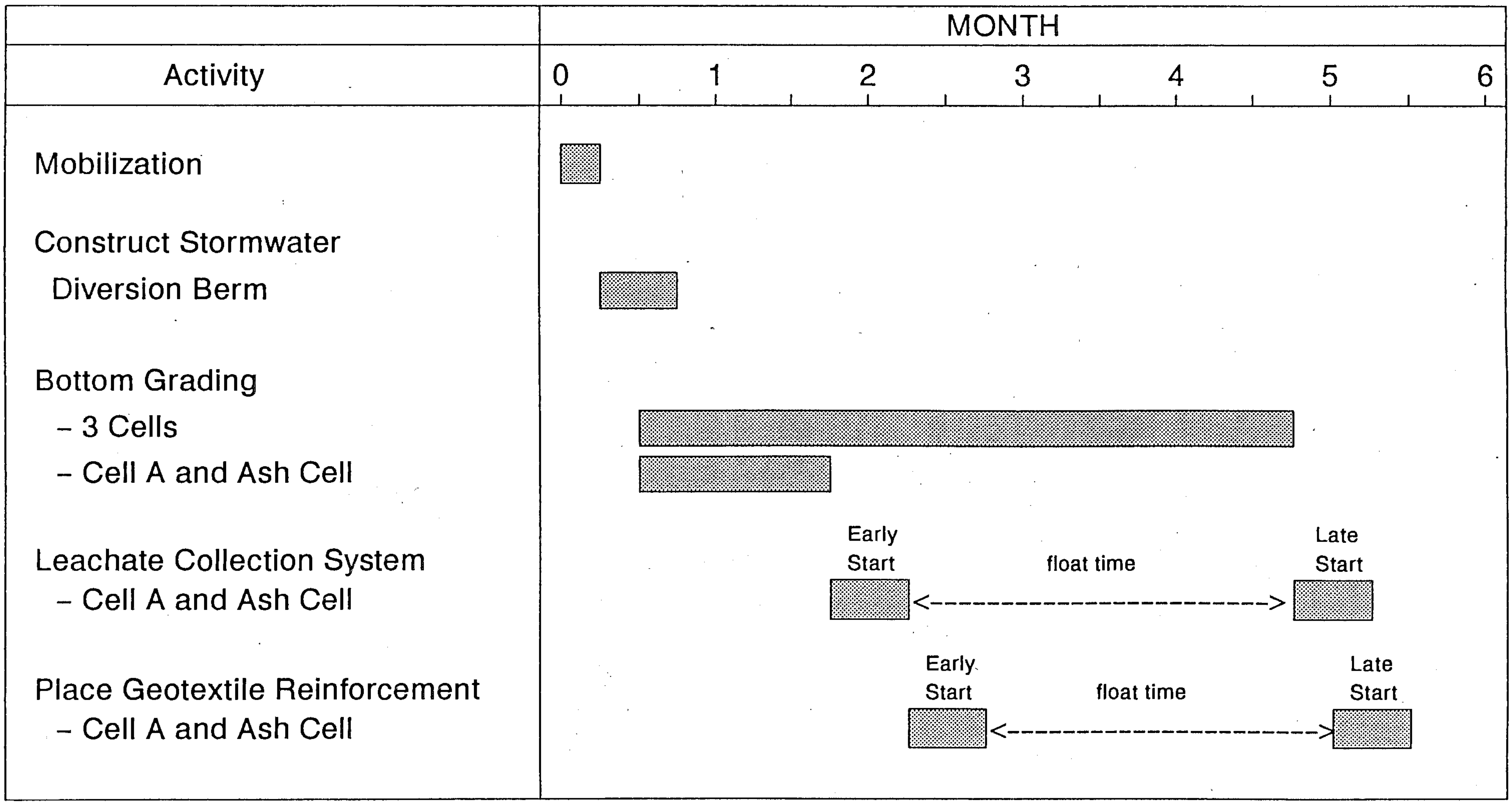
2. The duration of the bottom grading task is based on an assumption that site conditions following COE deposition in the Phase III area will allow construction equipment to work on the site, and that the excavation in the central portion of the fill will not encounter wet conditions. If soils are too wet to work on, off site material will have to be placed on the Cell A area, and the bottom elevation will be raised. This task will require additional time particularly if equipment is required to travel any distance to a borrow or stockpile area.

3. Depending on the outcome of the landfilling feasibility study, Phase III soils may need to be stabilized prior to any site construction. The schedule does not reflect the additional time required to dewater the site. If Phase III is required to be available for landfilling earlier than anticipated, the schedule can be shortened by grading the bottom of only Cell A and the Ash Cell.

FIGURE 8

PHASE III

PRELIMINARY SITE PREPARATION SCHEDULE
FOR PLACING CELL A AND ASH CELL IN OPERATION



4. **Gannett Fleming, Inc., "Phase III, Cherry Island, Wilmington, Delaware, Geotechnical and Hydrogeologic Report," prepared for the Delaware Solid Waste Authority, March 1990 - Revised August 1990.**

FILE COPY

DELAWARE SOLID WASTE AUTHORITY

NORTHERN SOLID WASTE FACILITY - 2
PHASE III
CHERRY ISLAND
WILMINGTON, DELAWARE

GEOTECHNICAL AND HYDROGEOLOGIC REPORT

MARCH 1990

REVISED AUGUST 1990

STATE OF DELAWARE
F. James Knight
Reg. No. 113
REGISTERED PROFESSIONAL GEOLOGIST

Prepared By

Gannett Fleming, Inc.
Harrisburg, Pennsylvania

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GEOTECHNICAL AND HYDROGEOLOGIC
REPORT

PROJECT DESCRIPTION

This geotechnical and hydrogeologic report presents the results of the investigations performed for the design and construction of the Phase III Landfill at the Northern Solid Waste Facility - 2 of the Delaware Solid Waste Authority (DSWA). The project site and landfill are located on Cherry Island, which is currently in joint use by the Philadelphia District of the U.S. Army Corps of Engineers for the deposition of dredge spoil materials and DSWA for solid waste disposal. As shown on Figure 1.1, the site for Phase III is located on the Edgemoor dredge disposal area of Cherry Island, east of the Phase II Landfill.

The main design elements of the Phase III Landfill are described below:

Stability Analysis - The North and East boundaries of the Phase III area are existing dikes previously constructed by the Corps of Engineers for containment of the dredge spoil. Solid waste for this phase will be placed adjacent to these dikes which required stability analyses of the dikes as well as the general landfill area.

Settlement - Deposition of the solid waste will cause settlement of the underlying dredge spoil. The settlement magnitude and rate was estimated for evaluation of strength gains due to consolidation and to properly slope leachate collection systems and final covers.

Permeability - DNREC regulations require a minimum 5-foot thick natural soil liner with a permeability less than or equal to $1x10^{-7}$ cm/sec.

Hydrogeologic Setting - Groundwater quality, piezometric levels, and groundwater gradients were investigated at the site.

In 1984, Terraqua Resources Corporation prepared a hydrogeologic and geotechnical report which addressed the characteristics of the entire Cherry Island site for landfill development. In addition Gannett Fleming prepared a second report addressing the Phase II expansion. Since these reports provided data with applicable information to the Phase III site, this data has been incorporated with the data obtained from the present investigation. In addition, the Corps of Engineers has also performed subsurface investigations at the site. The results of these investigations were also considered in the Phase III design.

The topography of the site is constantly changing due to current landfill operations, settlement of the dredge spoil, and earthwork activities by the Corps of Engineers. Throughout this report, topography as determined by a survey performed by Winward Associates in October, 1989 and aerial photography from 1985 supplied by Delaware Solid Waste Authority has been used. For purposes of this report, this information is considered sufficiently accurate for the required analyses.

SITE GEOLOGY

The site is located in the Coastal Plain Physiographic Province, which is characterized by low lying and partially submerged landforms. The materials of the Coastal Plain consist of layers of unconsolidated gravels, sands, silts, and clays. Frequently, these materials are interbedded with interconnected lenses. It is reported that the thickness of the unconsolidated layers in Wilmington, at the contact between the Piedmont Province and Coastal Plains Province, is 0 feet and in the southern portion of the county it increases to 2,300 feet. At the Cherry Island site the thickness ranges from 95 feet in the northwest corner, to about 220 feet in the southeast corner.

A generalized cross-section of the site geology is shown in Figure 1.2. As indicated, the lower unconsolidated layer overlying the rock is the Potomac Formation. This consists of multicolored silts and clays and interbeds of white, gray, and rust colored sands and some gravel. These granular interbeds are typically on the order of 5 to 10 feet thick. The thickness of the Potomac Formation varies from about 7 feet in the northwest corner to about 145 feet in the southeast corner of Cherry Island. The Columbia Formation is typically about 12 feet thick along the western edge of Cherry Island and 45 to 65 feet thick along the eastern edge. This formation generally consists of multicolored sands and gravels with interbeds of silty sand, silty clay, and clayey silt. Overlying the Columbia Formation are recent deposits and thick layers of dredge spoil. These materials are typically silty clays and clayey silts with some organic content and a layer of peat and clay. The overlying dredge spoil is of primary concern for geotechnical considerations for this project and is more fully defined below.

SUBSURFACE INVESTIGATION

In order to better estimate the soil parameters required for analyses, a subsurface investigation was conducted from October, 1989 through January, 1990. This investigation included ten Standard Penetration Test (SPT) borings and four dilatometer borings. The SPT borings were designated as GF-101 through GF-110. The dilatometer borings were designated as GF-111 through GF-114. The locations of these borings are shown on Figure 1.1 and 1.1A. Dilatometer borings GF-111 through GF-114 were located adjacent to SPT borings GF-106 through GF-109, respectively. SPT boring GF-101, 102, 103, 107, and 110 were performed along the access road at the outside toe of the containment dikes using a truck mounted rig. SPT borings GF-104, 105, 106, 108, and 109 as well as the four dilatometer borings were performed within the containment dikes using a tripod set-up mounted on a platform which was fabricated out of six inch thick styrofoam and 3/4 inch plywood. Sampling in the SPT borings was continuous for the first ten feet and at five foot intervals thereafter. Split-spoon samples, as well as undisturbed samples, were acquired in the SPT borings for laboratory testing. Copies of the driller's logs for these borings are attached in Appendix A of this report.

Cross-sections were developed incorporating the information acquired in this investigation as well as previous investigations at the site. A total of three sections were developed and are shown in Figures 1.3 through 1.5. As seen in these cross-sections, the dredge spoil deposit varies from 58 feet in thickness at the eastern limit of the Phase III site to 69 feet at the western limit.

LABORATORY TESTING

Undisturbed, split-spoon and bulk sample were collected during the subsurface investigation and subjected to laboratory testing in Gannett Fleming's Geotechnical Laboratory located in Harrisburg, PA. The laboratory testing included soil classifications, natural moisture contents, Atterberg limit determinations, unconfined compression tests, consolidation tests, water permeability tests, and standard compaction tests. The results of these tests have been tabulated and are presented in Table 1. A complete set of the test results is presented in Appendix B. In general, the test results agree with those found during the Phase II testing. For comparison, the Phase II test results are also attached as Table 2.

STABILITY ANALYSIS

The proposed site for Phase III was evaluated to determine the adequacy of the landfill configuration. The evaluation included the stability of the landfill/dike slope arrangement and, in particular, the stability of the slopes with regard to excessive deformation or collapse along an assumed failure surface.

The area of Phase III determined to be of particular concern regarding stability of the dike and landfill slopes occurs along the northern boundary of the site. Adjacent to this boundary are large lagoons, or finishing ponds, utilized by the nearby waste water treatment plant. It is believed that the most severe consequences would be realized with a slope stability failure in this area. As a result, a cross-section was developed which modeled the conditions which exist at the northern boundary of the site. This section was then evaluated for slope stability. For a more complete analysis of overall stability, cross-sections were developed along the east (Delaware River) side and south (COE Edgemoor Disposal Area) side of the site and these sections were also investigated for stability in order to refine the final design requirements of Phase III.

The stability analyses were performed in order to determine both the stability of the existing dikes as well as the stability of the overall site as landfilling progresses. The stability of the site was evaluated for the following solid waste thicknesses and configurations: 0 feet, 20 feet, 40 feet, 60 feet with the top 20 feet setback 100 feet, 80 feet with the top 40 feet setback 100 feet, and 80 feet present for one year with the top 40 feet setback 100 feet. Figure 2.2 shows the final landfill configuration with 3:1 (horizontal:vertical) side slopes and a landfill height of 80 feet with the described setback.

In the initial analysis along the northern boundary of the site for 0 feet of landfill, the underlying dredge spoil material was separated into five distinct layers with specific soil parameters as shown on Figure 2.1. Two of these layers, Layers 1 and 4, are directly under the existing dike and access roads. These layers are of considerably higher strength than the other three layers shown within the containment dikes because of soil compaction and consolidation due to the weight of the dikes. The initial strength parameters of each layer are also shown on Figure 2.1. It is important to note that the entire stability analysis was performed based on the assumption that either some method was employed to stabilize the most recent dredge spoil deposit and any future deposits, or that sufficient time was permitted for these spoils to consolidate to the strength of the underlying dredge material, which has a cohesion of approximately 190 psf at an elevation ten feet below the existing surface.

Strength gains in the dredge layers, resulting from the placement of solid waste material and corresponding consolidation of the dredge spoil, was estimated based on the established relationship, $Su=0.25P_o$, where Su is the soil strength, or cohesion, and P_o is the effective overburden pressure. The incremental increases in strength were added to the initial strength values for the three interior layers, numbers 2, 3, and 5, directly under the landfill. These increased dredge spoil properties for the incremental increases in solid waste thickness are shown in Table 3. The initial strengths were determined from unconfined compression tests conducted on Phase III undisturbed samples, pocket penetrometer readings taken on split-spoon samples during Phase III drilling operations, and laboratory tests conducted during previous phases.

The various landfill scenarios, mentioned above, were analyzed with respect to slope stability through the use of a computer software package called "TENSL01". The program, developed and distributed by The Tensar Corporation, is a version of "STABL6", (Purdue University), modified to incorporate geogrids into the analysis of slopes.

Slope stability analysis involves the quantification of the possibility of slope failure. This quantification was achieved through limit equilibrium methods, in which a factor-of-safety was calculated for a specified landfill scenario. In this specific analysis, a circular failure surface is assumed in which Coulomb's failure criterion will be satisfied along a circular arc. The factor-of-safety for a specified circular failure surface is determined by calculating the shearing resistance, or stress, required for equilibrium and dividing it by the available shear strength of the soil. The simplified Bishop method of slices is employed to obtain the factor-of-safety. In this method, the circular failure surface is divided into vertical slices and the forces on each slice are evaluated through limit equilibrium methods to produce a factor-of-safety for the given failure surface. This procedure is repeated for many circular failure surfaces until the one with the lowest factor-of-safety, the critical failure surface, is found. Typically, the factor-of-safety for slope stability should be 1.3 or greater. However, due to the very weak nature of the dredge spoil materials present at this site, it is not practical to achieve a factor-of-safety of 1.3 in the early life of the landfill. In the Phase II analysis, a factor-of-safety equal to 1.1 was used.

In an attempt to improve the factor-of-safety for a specified slope scenario, a geotechnical reinforcing material, or geogrid, was placed within the slope configuration so that it intersects the critical failure surface. The geogrids provide additional resisting forces by virtue of their inherent tensile strength capacity and, therefore, increased factors-of-safety. For these analyses, an equivalent geogrid tensile strength of 20,000 lbs/ft. was employed at the base of the landfill to provide increased stability of the landfill slopes. To obtain an equivalent geogrid tensile strength of 20,000 lbs/ft., five layers of Tensar's UX-1700 geogrid can be used. As an alternative, two layers of Matrex 240 and one layer of Matrex 120, both manufactured by The Reinforced Earth Company, can be utilized to achieve the necessary tensile strength.

The results from the slope stability analysis along the northern boundary, are presented in Table 4. Four cases were evaluated in which combinations of solid waste height, solid waste strength, and geogrid reinforcement were varied. Case I considers solid waste strength parameters of γ=43 pcf, c=750 psf and ϕ=0° with no geogrid reinforcement for the six landfill increments described above. Case II is similar to Case I except the solid waste properties are γ=43 pcf, c=200 psf and ϕ=10°. In Case III, the six landfill increments are analyzed with solid waste properties of γ=43 pcf, c=750 psf and ϕ=0° with an equivalent geogrid tensile strength of 20,000 lbs/ft. Finally, Case IV is similar to Case III with the solid waste strength parameters adjusted to γ=43 pcf, c=200 psf and ϕ=10°. The critical failure surface (circular arc), along with its corresponding factor-of-safety, is illustrated for each landfill scenario of Case III and Case IV on Figures 2.3 through 2.12.

The factors-of-safety for the four cases analyzed ranged from 0.89 for the weaker solid waste strength parameters, no geogrid reinforcement and 40 feet of landfill to 1.3 for the stronger solid waste material, 20,000 lbs/ft. equivalent geogrid tensile strength and 20 feet of solid waste material. The variation in solid waste strength parameters represents possible upper and lower values and, therefore, is used in sensitivity analyses to define upper and lower bounds regarding factors-of-safety. The solid waste strength parameters utilized as upper and lower bounds in these analyses are within the range of values cited in the literature (Dvinoff and Munion, 1986, Oweiss and Khera, 1986). The stronger solid waste material values were γ=43 pcf, c=750 psf and ϕ=0° while the weaker refuse material values were γ=43 pcf, c=200 psf and ϕ=10°. In the cases with no reinforcement, Cases I and II, the factors-of-safety for the lower strength solid waste material are generally below 1.0, whereas the cases with the higher strength solid waste material had factors-of-safety greater than 1.1. With the addition of geogrids, the factors-of-safety for the weaker and stronger solid waste strength parameters are approximately 1.12 and 1.26, respectively.

Similar conditions for cases with and without geogrid tensile reinforcement were analyzed to determine the benefits of the geogrids. In every case, the inclusion of geogrid reinforcement facilitated increased factors-of-safety. The effect of the reinforcement was less pronounced as the landfill height and soil strength properties increased.

A cross-section along the eastern boundary of Phase III, adjacent to the Delaware River, was also developed for slope stability analyses. This section, illustrated on Figure 2.14, consists of a single dike structure with a 4:1 horizontal:vertical exterior slope. The sand layer between the dredge spoil and solid waste contains geogrid layers with a total equivalent tensile strength of 20,000 lbs/ft. Factors-of-safety for the various construction phases are shown in Table 5. The average factor-of-safety for the configurations with the weaker solid waste properties, c=200 psf and ϕ=10°, is 1.11, whereas the average value for the cases with the stronger waste strength, c=750 psf and ϕ=0°, is 1.22.

For completeness, a cross-section along the southern boundary of Phase III, next to the COE Edgemoor Disposal Area, was analyzed. This section, shown on Figure 2.13, depicts the solid waste compacted on the dredge spoil material with a sand layer and geogrids (20,000 lbs/ft. equivalent tensile strength) separating them. Table 6 contains the factors-of-safety for the various phases of construction. The average factor-of-safety for the cases with the weaker refuse strength is 1.34 while the corresponding average value for the cases with the higher refuse strength is 1.53.

To examine the effect of time on the landfill stability, the factor-of-safety was determined for 80 feet of solid waste material just after final placement and also after the refuse material was in place for one year. The long-term strength of the geogrid reinforcement should be considered in the stability analysis to account for the effects of creep under long-term sustained loading, reinforcement damage associated with the construction operations, and long-term reinforcement durability. In the cases with 80 feet of refuse in place for one year, the equivalent geogrid tensile strength was reduced from 20,000 lbs/ft. to 15,000 lbs/ft. to account for this time-dependent phenomenon. The strength of 15,000 lbs/ft. represents the long-term design strength of 5 layers of Tensar's UX-1700 geogrid. The value of 15,000 lbs/ft. was used for the long-term design strength because it was the most conservative of the long-term design strengths which were obtained. In all cases, even when the geogrid tensile strength was reduced to long-term strength, the factor-of-safety increased, indicating that consolidation of the dredge materials provided increased strengths and greater stability.

Generally, these stability analyses did not result in a factor-of-safety greater than the value of 1.3 typically preferred. The factors-of-safety with an equivalent geogrid tensile strength of 20,000 lbs/ft. ranged from just above 1.0 to just below 1.3 for the various stages of landfilling and solid waste strength parameters. These values may be considered acceptable because the slope stability failure which could result in this case would probably be slow in nature with ample warning signs. To compensate for the lower factors-of-safety, a monitoring program should be implemented in which the soil parameters are measured at regular intervals as the landfill is constructed. The monitoring program should consist of the installation of inclinometers, piezometers, and settlements plates. Readings should be obtained and soil parameters determined after the placement of each ten-foot lift of refuse material and again immediately before placement of the next ten foot lift. Re-evaluation of the landfill stability using the information gained from the monitoring program to predict soil parameters will confirm the parameters estimated in the initial design and the validity of their corresponding factors-of-safety.

From this stability analysis it was determined that the solid waste material could be placed to a height of 40 feet (approximately Elevation 91) at a three horizontal to one vertical (3:1) slope. At 40 feet, the landfill should be set back 100 feet horizontally and continued at the 3:1 slope for 40 additional feet, to approximately Elevation 131. Geogrid reinforcement which has an equivalent tensile strength of at least 20,000 lbs/ft. should be placed at the base of the landfill within the sand drainage layer as shown on Figure 2.2 The setback of the landfill, the increased strength of the dredge spoil material due to consolidation resulting from the increased overburden pressures, (landfill loads), and the use of geogrid reinforcement are necessary to provide an adequate factor-of-safety in the stability analysis.

SETTLEMENT

Calculations of settlement caused by landfilling activities were performed. A landfill configuration consisting of an 80 foot high landfill constructed with 3 horizontal to 1 vertical slopes was assumed in the analysis (See Figure 2.2). A configuration was assumed without the setback shown in Figure 2.2, as required for stability concerns, because it is assumed that this setback will eventually be filled with refuse. Settlement calculations were performed assuming that the underlying dredge spoil strata is normally consolidated, including the most recent dredge spoil deposit and any future deposits. It is important to note that in many instances the underlying dredge spoils are actually underconsolidated. This condition is indicated by a natural water content in excess of the material's liquid limit (i.e., liquidity index in excess of 1). Referring to Table 1, this is the case in all but a few of the samples which were tested. The end result of this is that the material has not yet come to equilibrium under its own weight, thus settlement estimates assuming a normally consolidated stratum may slightly underestimate the total settlement. This difference is not expected to be in excess of 1 foot. The estimated settlement due to an 80 foot high landfill is 11.6 feet in the central portion of the landfill and tapers to 0 feet at the edge. Ninety percent of this settlement is expected to occur over a period of 70 years.

An important note is that the thickness of the dredge spoil strata across the site varies by approximately 11 feet. This fact, combined with the varying organic content of the underlying dredge spoil, will cause some differential settlement to occur across the site. The magnitude of this differential settlement is not expected to exceed 2 feet. A schematic profile showing the initial subgrade and the subgrade after 100 percent consolidation is presented in Figure 2.15.

PERMEABILITY

Currently, the dredge spoil stratum is either underconsolidated or normally consolidated under the present loading conditions. When solid waste material is added to the site, the overburden pressure is increased, excess pore water pressures are induced, and the entire dredge spoil stratum becomes underconsolidated. The magnitude of the excess pore pressures which are developed is equivalent to the increase in overburden pressure. Excess pore pressures can be expressed in equivalent feet of water head. One foot of water head is equal to a pressure of 62.4 psf. Each 10 foot layer of solid waste (γ=43 pcf) provides an increase in overburden pressure of 430 psf. This increase in overburden pressure corresponds to an equal increase in pore water pressure which can be expressed as 430/62.4 feet or 6.9 feet. Therefore, each 10-foot lift of solid waste induces an increase in pore water pressure of 6.9 ft.

The addition of the 1-foot sand drainage layer induces an excess pore water pressure of approximately 2 feet (125/62.4). As a result, after the drainage layer is placed and the first lift of solid waste has been added, an excess pore pressure of 8.9 feet exists. This excess pore pressure dissipates slowly over time due to the consolidation process. These pore pressures are dissipated by "squeezing" water out of the dredge spoils. As water is "squeezed" out due to the consolidation process, the water follows a path of least resistance. In other words, water flows in the direction of decreasing pressure or gradient. Figure 3.01, showing excess pore water pressure distributions, has been provided in order to aid in explaining the dissipation of these excess pore water pressures. Immediately after load is applied, the excess pore pressures are equal throughout the dredge spoil strata as seen on Figure A. Since the dredge spoils at the site are freely drained at both the top and bottom of the stratum, (doubly drained), the pore pressures begin to dissipate at each of the freely drained surfaces. This produces a pressure distribution similar to the distribution presented as Figure B. The pressure at the drained surface quickly dissipates to zero while the pressures within the dredge spoil strata dissipate very slowly over time. Thus, the excess pore pressures are greatest at the center of the dredge spoils and decrease in the direction of each of the drained surfaces. In other words, a pressure gradient exists up towards the surface in the upper half of the dredge spoils and down in the direction of the sand and gravel layer in the lower half of the dredge spoils. The magnitude of this gradient is equal to the change in pressure (expressed in feet of water) divided by the length of the flow path. Since the direction of the gradient is towards the surface in the upper half of the spoils, the direction of flow is, in fact, up towards the surface in this region. No downward flow of leachate could occur unless the head of leachate within the cell exceeded the excess pore water pressure. Therefore, after the initial lift of solid waste has been placed, the head of leachate within the cell would have to be greater than 8.9 feet to produce any downward migration of leachate. As additional lifts are placed, the excess pore water pressure is again increased.

Using consolidation data to calculate the rate at which pore pressures will dissipate, and assuming a 6-month period between lifts, average excess pore pressures of the dredge spoil stratum were calculated after the placement of subsequent refuse lifts. These values of excess pore water pressure are tabulated below in equivalent feet of water.

Solid Waste Height (feet)	Excess Pore Water Pressure (feet of water)
10	8.9
20	13.62
30	18.70
40	23.30
50	27.60
60	31.42
70	34.90
80	38.15

It is noted that as long as the gradient is upward or zero, downward flow in the upper half of the dredge spoil strata does not occur. Any downward flow does not begin until the head of leachate within the landfill exceeds the excess pore pressures in the spoils. Calculations show that the average excess pore pressure is greater than 5 feet of water head until approximately 60 years after 80 feet of solid waste has been in place. This means that for any flow to occur, even after 60 years, the leachate head would have to be greater than 5 feet. As these excess pore pressures dissipate, the dredge spoils consolidate and the void ratio decreases, thereby decreasing the coefficient of permeability.

DNREC requirements for natural soil liners state that a minimum 5-foot thick layer of material exists with a water permeability less than or equal to $1x10^{-7}$ cm/sec. This layer is to begin at the elevation of the leachate collection system. However, these regulations are unclear as to what cell pressure or gradient should exist when performing the laboratory triaxial permeability tests. It would be logical for the conditions in the laboratory to model the anticipated conditions in the field.

Laboratory permeability tests were performed on both undisturbed and remolded samples in a triaxial permeability device at various consolidation pressures which modeled anticipated field conditions. Permeability tests were run at consolidation pressures which modeled the following landfill heights: 10 feet, 20 feet, 40 feet, 60 feet and 80 feet. These solid waste heights correspond to the following consolidation pressures at the base of the landfill: 550 psf, 980 psf, 1840 psf, 2,700 psf, and 3,560 psf, respectively. These pressures were calculated based on a 1-foot sand drainage layer with a unit weight of 120 pcf and a unit weight of 43 pcf for the solid waste material. In general, the results of these tests show that, at some point during the landfilling sequence, a consolidation pressure is reached at which the permeability of the dredge spoils is less than $1x10^{-7}$ cm/sec. This degree of impermeability is reached when the dredge material becomes normally consolidated under the corresponding consolidation pressure.

Figure 3.02 shows a plot of permeability versus consolidation pressures, shown as equivalent solid waste heights. As can be seen, in all cases it is estimated that the permeability will become less than $1x10^{-7}$ cm/sec. at some time during landfilling activities. In most cases the required permeability is reached at pressures equivalent to less than 40 feet of solid waste material. In particular, all of the undisturbed samples shown achieved the required degree of impermeability at pressures equivalent to less than 35 feet of landfill height. Another important point is that the entire dredge spoil stratum has a permeability less than or equal to $1x10^{-6}$ cm/sec. in its present state. As stated previously, the dredge spoils vary from 58 feet to 69 feet in thickness. Therefore, in its present state, the dredge spoil strata is equivalent to at least 5.8 to 6.9 feet of material with a permeability of $1x10^{-7}$ cm/sec. This is neglecting the increase in dredge spoil thickness due to the COE's deposition in the spring of 1990. This equivalent thickness of material with a permeability less than $1x10^{-7}$ cm/sec. will increase as the consolidation process continues until the entire strata has a permeability less than $1x10^{-7}$ cm/sec. In fact, Figure 3.02 shows that at some point the dredge spoils will approach a permeability of $1x10^{-8}$ cm/sec. or less.

In general, the hydraulic scenario at the site is ideally suited to prevent migration of leachate from the site. Although, initially the permeability of the dredge spoils is approximately $1x10^{-6}$ cm/sec., 70 feet of material with this hydraulic conductivity will exist, providing for an equivalent thickness of 7 feet of material with a hydraulic conductivity less than or equal to $1x10^{-7}$ cm/sec. In addition, an upward gradient will be induced from the increase in overburden pressure and flow of pore water will occur in the upward direction preventing any leachate from migrating through the dredge spoils. As the pore water escapes, the excess pore pressures are reduced and thus, the gradient is reduced. However as these pore pressures dissipate, the permeability of the dredge spoils steadily decreases. In fact, it will take over 75 years for the pore pressures to dissipate. During this time, the permeability of the dredge spoils will have been reduced to approximately $2x10^{-8}$ cm/sec. Figure 3.03 was developed using laboratory consolidation data to predict the time for dissipation of excess pore pressures and laboratory permeability test results. This figure provides graphic representations of the magnitude of the excess pore water pressures, in equivalent feet of water, vs. time and permeability vs. time. As can be seen, during the time in which it takes for the dredge spoils to reach a permeability less than $1x10^{-7}$ cm/sec., an upward gradient exists and no migration of leachate will occur.

The permeabilities shown on Figure 3.03 represent an average permeability over the first 15 feet of dredge spoils. In order to better represent the changes in permeability with time, Figures 3.03A through 3.03E have been provided. These figures are representations of the anticipated permeability zones in the dredge spoils as well as the magnitude of the excess pore pressure gradients at the following times after landfilling commences: 0 years, 1.5 years, 6 years, 11 years, and 80 years. These figures show both thickness of the anticipated zones with permeabilities less than $1x10^{-7}$ cm/sec. as well as calculations of an equivalent thickness of material with a permeability less than $1x10^{-7}$ cm/sec. Also shown on the figures is the upward pressure expressed in equivalent feet of water. In all cases, the magnitude of this pressure exceeds 2.6 feet of water head. Thus, even after 80 years, a leachate head greater than 2.6 feet would need to exist to provide for any downward migration of leachate. At that time, the permeability of the entire 70-foot dredge spoil stratum will be approximately $2x10^{-8}$ cm/sec.

Taking a closer look at the permeability results on Table 1, it is seen that in four of the permeability trials the samples had a permeability greater than the maximum permeability of $1x10^{-7}$ cm/sec. In one trial, the remolded bulk sample of GF-106, the test was stopped at a pressure equal to only 19.3 feet of refuse material. Using Figure 3.02, it is estimated that this sample would have reached the desired degree of impermeability at a pressure equivalent to 61 feet of refuse. In the other 3 cases, the samples contained some unique characteristic to explain these higher permeabilities. These 3 cases are not plotted on Figure 3.02. In sample U-1, GF-106, the material consisted of a non-plastic black silt with sand with 25 percent of the particles greater than the 200 sieve. This sample, with higher sand content and non-plastic characteristics, in all likelihood produced the higher permeability. The classification and non-plastic characteristics of this sample are not typical at the site. In fact, another sample from the same undisturbed sample classified as an MH, elastic silt, with only 2 percent of the particles greater in size than the number 200 sieve.

Sample U-1 of boring GF-105 contained a fractured structure which was most likely due to the sample undergoing numerous wet, dry cycles. These cycles cause small, high-strength nodules to form with relatively large voids or fractures between adjacent nodules. This highly fractured structure caused the permeability to be two orders of magnitude higher than other samples with similar characteristics. The permeability of this sample would have been reduced at the time that a consolidation pressure was reached, which would break down (remold) these high strength nodules.

Sample U-1 of GF-110 had a very high permeability, 1.1×10^{-4} cm/sec. In order to investigate the reason for this high permeability, the sample was cut into sections in the laboratory. A cylindrically shaped open region was encountered which was oriented vertically in the sample and extended through the entire length of the sample. This open cylinder appeared to be a decaying Phragmite and probably caused the high permeability. It is predicted that as the Phragmite continued to decay and the increase in overburden pressure caused the open region to become smaller, the permeability would be significantly reduced. In each of these three cases, the higher permeability can be explained and the attributes are not considered to be common at the site. One sample appeared to be a small sand lense while the other two samples appeared to be characteristic of surface layers. Since the COE generally deposits five feet (plus or minus) of dredge spoil material, it is very unlikely that these surface layer discontinuities are hydraulically connected. It is also noted that the effects of these surface layer discontinuities will become less and less pronounced as the overburden pressure is increased and these discontinuities are closed or healed. In general, it appears that the site contains approximately 58 feet to 69 feet of material which will achieve a permeability of less than 1×10^{-7} cm/sec. at some point during landfill activities and, in most cases, a short time after the first lift has been placed. During the time required for the permeability of the dredge spoils to reach 1×10^{-7} cm/sec., an upward gradient exists and no downward flow will occur.

HYDROGEOLOGIC SETTING

The Columbia Formation of northern Delaware, as recorded by Jordan (1962), consists primarily of coarse sand and considerable admixtures of gravel and cobbles with thin silty layers. Although the basal sediments which overlie the Cretaceous Potomac Formation resemble this description, most of the remaining succession of sediments are considerably finer grained and should be interpreted as a separate unit of recent river sediments. Geologic review by Duffield Associates, as part of their quarterly monitoring effort, also support this conclusion. The resulting hydrogeologic framework, which is based on data from several geologic test borings and stratigraphic locations of many of the piezometers installed during July and August of 1983, is illustrated in the stratigraphic cross-sections of Figures 3.1 and 3.2. Because both Potomac and Columbia Formation piezometer pairs are located in close proximity to each other, they are shown as composite installations.

Interpretation of the post-Potomac Formation sediments as recent river deposits, separate from the Columbia Formation, was used to explain the lithologic variations shown in the stratigraphic cross-sections. Sediments of truly Columbia Formation would be expected to be more coarse grained and contain less silt or silty clay; however, the sediments were composed of thick deposits of dark brown to gray silty clay and clayey silt with thinner intervening sand and gravel units. Sediments of this texture are typical of meandering river systems where sediment type, channel position, and geometry are variable.

Cross-Section C-C' of Figure 3.2 illustrates the extent of the sand and gravel units which constitute the permeable zones within the river sediment aquifer. These sand and gravel layers are divided into the lower and the upper units which are separated by an inherently less permeable silty unit. The upper sand unit is apparently confined to the eastern area of the site with diminishing thickness westward. The lower sand unit is more extensive and is bordered by an underlying sand unit which may be the true Columbia Formation (see Figure 3.2). These sand units are shown in Figures 3.1 and 3.2 with stippled patterns for the sake of clarity and identification from one cross-section to another.

The most important detail to be observed in the cross-sections is the placement of the piezometers in the recent river sediments. In Cross-Section C-C' three of four piezometers are placed in the lower sand unit and one (C-106) is placed in the upper sand unit. Adjoining Cross-Section D-D' shows two additional piezometers (C-102 and C-108) located outside the lower sand unit. Because the upper sand unit is isolated from the lower sand unit by a thick sandy silt, the piezometer readings taken from these meters should be identified as having originated from a separate unit.

Additional evidence that Piezometers C-102, 106, and 108 should be treated as separate from other meters in the Recent/Columbia sediments is found in data collected over 10 hours from two piezometers and the Christina River during tidal fluctuations on December 30, 1983, to determine the influence of ocean tides on groundwater levels at the site. The important distinction between the two Recent/Columbia piezometers of the tidal study is that they are located in different sand zones within the sediments. C-105 is located in the lower sand unit and C-106 in the upper. If the upper and lower sand units were hydraulically interconnected, then the tidal responses of these two piezometers would be expected to be similar. However, this is not the case. Not only is response in C-106 limited in magnitude as compared to C-105, but it is also delayed. In actuality, tidal response of C-106 might be expected to be greater than C-105 because C-106 is installed nearer to the elevations of the Christina River, increasing the likelihood of hydraulic interconnection between the river and upper sand zone.

Therefore, it appears that the two units are isolated from each other. Tidal response curves and total tidal fluctuation for C-105, 106, and the Christina River are shown in Figure 3.3.

Finally, data collected from C-101, C-102, C-104 and C-106 in January, 1990, further indicate the uniqueness of the units from which the readings were taken, supporting the more complex hydrogeologic framework developed using additional geologic information. Data loggers were used to measure water levels in these four wells every fifteen minutes over a 48-hour period. The resulting plot, showing water level fluctuations with time is shown on Figure 3.4. The potentiometric readings for C-106 suggest that this is a perched unit, hydrologically isolated from the Columbia Formation. The anomalous results from C-102, apparently due to tidal fluctuations, indicates that this piezometer is located in a separate unit which possesses drastically different hydraulic characteristics. C-101 and C-104, which are located in the Columbia Formation, show a decrease in the potentiometric surface of the Columbia Formation in the direction of the point of convergence between the Delaware and Christina Rivers. C-102 and C-106, which are located outside the

Columbia Formation, have not been used for defining the location of the potentiometric surface.

Review of geologic and hydrogeologic data reveals that the Columbia Formation of the Terraqua Report of 1984 actually includes considerable quantities of recent river deposits and minor Columbia Formation sediments. Within these sediments, there are two distinct water bearing units which are separated by a thick silt that acts to hydraulically isolate the upper sand and gravel from the lower unit. Hydraulic isolation of the two water bearing units is further supported by the two studies which showed differing piezometric responses with regard to tidal fluctuations.

Review of piezometer placement, using the more complex stratigraphic interpretation recently developed, indicates that Piezometers C-102, C-106, and C-108 are located outside the lower sand zone and should not be considered as part of the network of other Recent/Columbia piezometers.

Additionally, two ground elevation surveys conducted in 1983 and 1985, indicate that ground settlement has been insignificant at the piezometer locations and that a resurvey of piezometer installations is not necessary.

Finally, Figure 3.5 shows a possible interpretation of the potentiometric surface of the Recent/Columbia sediments. This interpretation does not include data from the three piezometers installed in the upper sand zone and is based on average piezometric elevations calculated for data collected between July and December 1983. The data collected in January, 1990 also supports this interpretation. These averages are based on the assumptions that the piezometric levels were taken at random, representing all tidal positions in an attempt to filter out tidal position and precession which were not taken into consideration during data collection. Although minor variations in the flow direction are shown in the figure, the important conclusion is that a relatively flat potentiometric surface exists beneath Cherry Island and that the gradient sharply increases to the west of the site. This is particularly important since the Columbia Formation is more clearly defined along the western edge of the site where the gradient is prominently toward the Delaware and Christina Rivers.

ADDENDUM NO. 1
MAY 9, 1990

TO THE GEOTECHNICAL AND HYDROGEOLOGIC REPORT FOR

DELAWARE SOLID WASTE AUTHORITY
NSWF-2 PHASE III

This Addendum is made part of the above-noted report and shall be taken into account in the review of the report and the subsequent design of Phase III.

A meeting was held between representatives of DSWA, DNREC, and Gannett Fleming on April 9, 1990 in Dover, Delaware at the office of DSWA. During this meeting DNREC requested the following laboratory test data:

1. Soil pH Analyses,
2. Cation Exchange Capacity Tests and,
3. Leachate Compatibility Test.

The results of each of these tests are now available and are attached to this addendum. A discussion of the results of each test follows:

Soil pH

Four soil samples were analyzed for pH by Gannett Fleming's Environmental Laboratory. The pH of the samples ranged from 7.10 to 7.25 units, with an average value of 7.19 units. As is seen from the results, the dredge spoils are very near the neutral pH of 7.0 units.

Cation Exchange Capacity

The cation exchange capacity (CEC) is a measure of the extent to which the cations in a soil can be reversibly replaced by salt solutions and acids. In the case of a natural soil liner for a municipal solid waste landfill, the CEC is a measure of the soils ability to remove heavy metals which may be present in the leachate and replace those heavy metals with cations which naturally occur in the soil. CEC is typically expressed in meq. (milliequivalents) per 100 grams of soil.

Five samples obtained during the Phase III subsurface investigation were submitted to Geochemical Testing, located in Somerset, Pennsylvania, in order to determine the CEC of the dredge spoils underlying the Phase III area. The CEC of the dredge spoils varied over a limited range, 23.2 meq/100g to 39.9 meq/100g, and varied little with depth. The average value of the CEC was 28.4 meq/100g. These values are within the range of values expected for clay minerals. For example, the CEC of kaolonite typically ranges from 5 to 10 meq/100g (at pH=7.0) while the CEC of montmorillonite typically ranges from

50 to 100 meq/100g (at pH=7.0). The CEC of soils generally increases with increasing pH since a greater amount of excess OH^- ions are available for bonding with cations.

Leachate Compatibility Test

In order to determine any changes in the permeability of the dredge spoils when exposed to leachate, a compatibility test was performed in the Gannett Fleming Geotechnical Laboratory. Leachate for the test was obtained from the leachate pump station at the Cherry Island site.

The sample used for the test was a remolded bulk sample taken at the surface in the location of Boring GF-108. The initial permeability of the sample, using distilled water as the permeant, was 1.5×10^{-8} cm/sec. The permeability of the sample after exchanging two pore volumes with leachate was 1.7×10^{-8} cm/sec. The results indicate that little or no change in permeability occurs when the sample is exposed to leachate.

The results of each of these tests have been attached to this addendum.

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TABLES

Table 1 (Continued)

oring No.	Sample No.	Sample Depth (ft)	Class.	Wn (%)	Wl (%)	Wp (%)	Ip (%)	% Passing No. 200	γd (pcf)	γm (pcf)	γsat (pcf)	Gs	Unconfined qu (Ksf)	Cc	Cons. Cv in^2/min	e_o	γd (pcf)	Wopt (%)	K* (cm/sec)
F-105	U-2	18.5-20.5	MH	77.2	75.5	45.1	30.4	99		89.2				0.79	0.011	2.350			
F-105	Bulk	0-1.0	MH	52.0	71.2	49.5	21.7	99	63.2										$2.0x10^{-6}$ (remolded)
GF-105	Bulk	0-1.0	MH	46.7	71.2	49.5	21.7	99	70.8										$2.4x10^{-8}$ (remolded)
GF-106 (permeability)	U-1	8.0-10.0	ML	69.7		N/P			55.3							2.047	72	36.5	$1.3x10^{-5}$
GF-106 (consolidation)	U-1	8.0-10.0	MH	95.7	72.0	40.4	31.6							0.97	0.003	2.781			
GF-106	S-2, 3	2.0-6.0	MH	103.6	70.2	46.2	24.0	98											
GF-106	S-10, 11	33.5-40.0	MH	90.8	72.3	48.2	24.1	99											
GF-106	S-13, 14	48.5-55.0	MH	70.1	61.8	41.8	20.0	99											
GF-106	Bulk	0-1.0	MH	52.0	53.2	38.1	15.1	88	67.0										$2.7x10^{-7}$ (remolded)
GF-107	U-1	20.0-22.5	MH	83.0	66.3	43.3	23.0	96	48.9	89.5									$5.3x10^{-8}$
GF-107	S-5, 6	8.0-15.0	ML	47.1		N/P		84											
GF-107	S-9, 10	23.5-30.0	ML	48.5		N/P		90											
GF-107	S-11, 12	33.5-40.0	MH	75.8	68.7	47.4	21.3	98											
GF-107	S-14, 16	48.0-55.0	MH	64.0	61.4	41.9	19.5	98											
GF-107	U-2	55.0-57.0	ML	66.5	49.1	32.9	16.2		66.5	97.4				0.53	0.003	1.883			
GF-108	U-2	15.0-17.0	MH	86.3	59.4	40.3	19.1		86.3	92.1				0.8	0.0050	2.409			
GF-108	U-1	8.0-10.0	MH	93.4	67.8	42.1	25.7			98									
GF-108	Bulk	0-1.0	MH	54.1	55.6	38.3	17.3	99	64.6								79.8	34	$1.4x10^{-7}$ (remolded)

Table 1 (cont'd.)

Boring No.	Sample No.	Sample Depth (ft)	Class.	Wn (%)	Wl (%)	Wp (%)	Ip (%)	% Passing No. 200	γd (pcf)	γm (pcf)	γsat (pcf)	Gs	Unconfined qu (Ksf)	Cc	Cons. Cv in²/min	e_o	γd (pcf)	Wopt (%)	K* (cm/sec)
GF-108	Bulk	0-1.0	MH	42.2	55.6	38.3	17.3	99	75.0										2.9×10^{-8} (remolded)
GF-108	U-4	49.0-51.0	MH	67.9	55.0	44.0	11.0	99.8	62.0	104.1			0.64						
GF-108	Bulk	0-1.0	MH	44.3	55.6	38.3	17.3		74.0										1.9×10^{-8} (remolded)
GF-108	U-3	28.5-30.5	MH	86.7	63.0	54.0	9.0	99.5	53.5	99.8			0.46						
GF-109	U-3	37.0-39.0	MH	79.5	84.0	49.0	35.0	99.3	58.7	105.3			0.66						
GF-109	U-1	14.0-16.0	ML	35.2	37.7	26.3	11.4	78	49.8	91.7									5.2×10^{-8}
GF-109	U-2	17.0-19.0	MH	84.1	56.8	35.8	21.0	96		90.6									
GF-110	U-1	10.0-12.0	MH	62.5	67.1	45.4	21.7		61.8	100.4									1.1×10^{-4}
GF-110	U-2	25.0-27.0	MH	78.7	53.3	35.8	17.5	70	52.8				0.26			2.249			
GF-110	S-16, 17	53.5-60.0	ML	53.2	47.4	31.6	15.8	93											
GF-110	S-12, 13	33.5	MH	78.5	68.8	45.2	23.6	99											
GF-110	S-3, 5	4.0-10.0	MH	49.9	56.1	37.4	18.7	62											

* See permeability summary sheets for pressures at which permeability results are shown (Appendix B).

TABLES

Table 1

Boring No.	Sample No.	Sample Depth (ft)	Class.	Wn (%)	Wl (%)	Wp (%)	Ip (%)	% Passing No. 200	γd (pcf)	γm (pcf)	γsat (pcf)	Gs	Unconfined qu (Ksf)	Cc	Cons. Cv in^2/min	e_o	γd (pcf)	Wopt (%)	K* (cm/sec)
GF-101	S-3, 4, 5	4.0-10.0	MH	44.8	50.6	39.1	11.5	92											
GF-101	S-7, 8	18.5-25.0	MH	74.9	60.6	41.9	18.7	98											
GF-101	S-11, 12	38.5-45.0	MH	76.0	63.2	43.2	20.0	98.5											
GF-101	S-16, 17	63.5-70.0	ML	47.5	42.2	26.8	15.4	93											
GF-102	S-6, 7	13.5-20.0	MH	69.7	57.6	39.1	18.5	96											
GF-102	S-9, 10	28.5-35.0	MH	92.8	66.3	46.2	20.1	98											
GF-102	S-14, 15	48.5-55.0	ML	63.0	46.5	33.0	13.5	99											
GF-102	U-1	4.0-6.0	ML	43.7	37.7	31.0	6.7	75	58.	85.3									
GF-102	U-2	35.0-37.0	MH	73.7	69.3	47.8	21.5	97		94.3				0.55	0.0025	2.105			
GF-103	S-2, 3, 4	2.0-8.0	ML	47.8	----	N/P	----	83											
GF-103	S-8, 9	23.5-30.0	ML	65.8	48.2	34.0	14.2	97											
GF-103	U-1	15.0-17.0	ML	94.2	43.7	43.5	0.2	88	48.5				0.19			2.552			
GF-103	U-2	45.0-47.0	ML	61.4	47.4	33.5	13.9	95	63.6				0.46			1.698			
GF-104	S-1, 2, 3	0-6.0	MH	141.0	75.2	49.0	26.2	98											
GF-104	S-8, 9	23.5-30.0	MH	95.2	68.4	47.1	21.3	98											
GF-104	U-2	16.5-18.5	ML	55.1	----	N/P	----	82	59.8	92.0			0.86	0.30	0.15	1.872			
GF-104	U-4	35.0-37.0	MH	94.2	74.1	48.0	26.1	100	46.3	91.8			0.28			2.705			
GF-104	U-1	6.5-8.5	MH	50.6	65.9	40.9	25.0	95	70.2	105.7									$9.8x10^{-8}$
GF-104	Bulk	0-1.0	MH		79.9	54.6	25.3	99											$6.3x10^{-8}$ (remolded)
GF-105	U-1	5.0-7.0	MH	70.5	69.5	42.2	27.3	98	53.8	91.7						2.113			$7.2x10^{-5}$

Boring No.	Sample No.	Sample Depth (ft)	Class	w_n (%)	w_l (%)	w_p (%)	I_p	% Passing No. 200	γ_d (pcf)	γ_m (pcf)	γ_{sat} (pcf)	G_s	Unconfined Compression q_u (ksf)	Triaxial Tests Total ϕ	Total c(ksf)	Effective ϕ	Effective c(ksf)	Type Test	C_c	Consolidation C_v(inch^2/min) @ 0.5 tsf	e_o	Moisture γ_{dmax} (pcf)	Density W_{opt} (%)	k (cm/sec)
GF 1	S7	10-12	MH	69.1	62.4	46.3	16.1		58.8	99.4	90.8		0.58						0.67	$1.5X10^{-3}$	2.429			
	S10	20-22	MH	80.8	62.8	44.7	18.1		51.8	93.7	92.2		0.38						0.73	$3.6X10^{-3}$	2.271			
	S15, S16	35-39	ML	33.8	N/P	N/P	--	39.0				2.63												
	S19, S20	45-49	OH	109.0	89.1	N/P	--																	
GF 3A	S5, S6	6-9	MH	59.7	62.5	46.7	15.8																	
	S10, S11	17.5-21.5	MH	78.5	67.6	49.9	17.7																	
	S15, S16	30-34	ML	42.6	46.8	32.8	14.0																	
GF 5	S4, S5	6-10	MH	66.7	53.0	36.5	16.5	94.5				2.61												
	S9, S10	20-24	MH	85.6	62.1	44.2	17.9	100.0				2.40												
GF 6	S3	5-7	OH	103.6	70.8	43.1	27.8	97.5	47.3	96.3	91.9	2.66	0.61											
	S12	25-27	OH	85.4	73.1	46.7	24.4	100.0						17°	0.0	39.5	0.0	$\bar{R}$						
GF 7	Tube 1	10-12	OH	86.7	72.4	46.4	26.0	99.0	49.6	92.5	92.8	2.59							0.64	$3.4X10^{-3}$	2.261			$6.6X10^{-8}$
	ST-1	20-22	OH	91.3	76.5	47.9	28.6	100.0	53.6					17°	0.0	39.5	0.0	$\bar{R}$	0.72	$2.45X10^{-3}$	2.232			$1.1X10^{-7}$
	S20, S21	52.5-56.5	MH	78.7	68.2	51.4	16.8																	
GF 8	S4	5-7	OH	118.4	74.6	48.6	26.0	100.0	50.8	110.9	96.9		0.44						0.84	$3.3X10^{-3}$	2.828			
	S13	25-27	MH	89.6	65.9	42.4	23.5	100.0	51.7	98.0	94.0	2.56	0.32						0.54	$2.25X10^{-3}$	2.101			
GF 9	S-1	0-1.5		82.1																				
	S-2	7-8		96.7																				
GF 10	Bulk	0-2	*	86.2	76.6	48.4	28.2	100.0														69.5	46.0	
	S-1	2-4		118.8																				
	S-2	6-8		108.2																				
GF 11	Bulk	0-2	*	82.5	78.7	53.4	25.3	99.0																
	S-1	4-5		102.3																				
	S-2	7-8		95.6																				

TABLE 2

PHASE II LABORATORY DATA SUMMARY

TABLE 3

Increased Soil Properties
(Assuming One Layer of Dredge Spoil, Doubly-Drained)

Initial Conditions:

Soil #	γ(pcf)	c (psf)
1	105	625
2	100	407
3	93	225
4	110	750
5	93	188

40' Landfill:
(20 ft present for 1 year)

Soil #	γ(pcf)	c (psf)
1	105	625
2	100	437
3	95	255
4	110	750
5	93	218

60' Landfill:
(40 ft present for 1 year)

Soil #	γ(pcf)	c (psf)
1	105	625
2	100	494
3	97	312
4	110	750
5	95	275

80' Landfill:
(60 ft present for 1 year)

Soil #	γ(pcf)	c (psf)
1	105	625
2	103	577
3	100	395
4	110	750
5	97	358

Landfill 1 year after completion
(80' present for 1 year)

Soil #	γ(pcf)	c (psf)
1	107	687
2	107	687
3	103	505
4	110	750
5	100	468

Table 4
Northern Cross-Section

CASE I
Landfill Properties c=750 pcf, $\phi=0^\circ$
No Geogrid

Landfill	Factor-of-Safety
0 ft	1.19
20 ft	1.30
40 ft	1.14
60 ft (100' offset)	1.17
80 ft (100' offset)	1.14
80 ft (after 1 yr)	1.24

CASE II
Landfill Properties c=200 pcf, $\phi=10^\circ$
No Geogrid

Landfill	Factor-of-Safety
0 ft	1.19
20 ft	0.94
40 ft	0.89
60 ft (100' offset)	0.97
80 ft (100' offset)	1.02
80 ft (after 1 yr)	1.13

CASE III
Landfill Properties c=750 pcf, $\phi=0^\circ$
Geogrid Tensile Strength of 20,000 lbs/ft. at the base of the landfill

Landfill	Factor-of-Safety
0 ft *	1.19
20 ft *	1.30
40 ft	1.25
60 ft (100' offset)	1.27
80 ft (100' offset)	1.19
80 ft (after 1 yr)	1.28

CASE IV
Landfill Properties c=200 pcf, $\phi=10^\circ$
Geogrid Tensile Strength of 20,000 lbs/ft at the base of the landfill

Landfill	Factor-of-Safety
0 ft*	1.19
20 ft*	1.13
40 ft	1.06
60 ft (100' offset)	1.15
80 ft (100' offset)	1.08
80 ft (after 1 yr)	1.18

*Geogrid Tensile Strength for these cases was equal to 16,000 lbs/ft

Table 5
Southern Cross-Section

CASE III
Landfill Properties c=750 pcf, ϕ=0°
Geogrid Tensile Strength of
20,000 lbs/ft. at the base of the
landfill

Landfill	Factor-of-Safety
0 ft	9.23
20 ft	2.16
40 ft	1.42
60 ft (100' offset)	1.39
80 ft (100' offset)	1.27
80 ft (after 1 yr)	1.42

CASE IV
Landfill Properties c=200 pcf, ϕ=10°
Geogrid Tensile Strength of
20,000 lbs/ft at the base of the
landfill

Landfill	Factor-of-Safety
0 ft	9.23
20 ft	1.86
40 ft	1.14
60 ft (100' offset)	1.23
80 ft (100' offset)	1.14
80 ft (after 1 yr)	1.30

Table 6
Eastern Cross-Section

CASE III
Landfill Properties c=750 pcf, $\phi=0^\circ$
Geogrid Tensile Strength of
20,000 lbs/ft. at the base of the landfill

Landfill	Factor-of-Safety
0 ft	1.22
20 ft	1.29
40 ft	1.18
60 ft (100' offset)	1.23
80 ft (100' offset)	1.18
80 ft (after 1 yr)	1.24

CASE IV
Landfill Properties c=200 pcf, $\phi=10^\circ$
Geogrid Tensile Strength of
20,000 lbs/ft at the base of the landfill

Landfill	Factor-of-Safety
0 ft	1.22
20 ft	1.18
40 ft	1.07
60 ft (100' offset)	1.12
80 ft (100' offset)	1.07
80 ft (after 1 yr)	1.13

FIGURES

NORTHERN SOLID WASTE FACILITY-2

MONITOR WELL AND FIELD INSTRUMENTATION
LOCATION COORDINATES AND REFERENCE ELEVATIONS

Groundwater Monitor Well	Location Coordinates (1) North	Location Coordinates (1) East	Reference Elevation - N.G.V.D. (Feet) (2) (3) Approx. Grd. Sfc.	Top of Casing	Screen Interval
C-100	631,474.9	470,751.9	15.4	18.3	-9.4 to -19.4
C-101	630,194.5	472,365.6	35.2	38.05	-24.8 to -34.8
C-102	628,860.6	474,291.8	26.5	28.9	-29.7 to -39.7
C-103	628,642.3	467,859.8	12.3	13.4	-10.9 to -15.9
C-104	627,285.7	470,194.2	15.8	19.1	-42.8 to -47.8
C-105	626,680.6	471,741.5	21.6	24.3	-40.6 to -50.6
C-106	626,323.8	473,448.4	27.2	30.0	-6.6 to -16.6
C-107	630,213.3	469,133.1	10.2	13.45	-5.7 to -9.7
C-108	627,436.0	473,866.7	27.3	29.7	-0.8 to -10.8
C-109	629,945.5	470,236.6	30.7	36.05	-15.4 to -25.4
C-110	629,885.4	470,946.6	31.6	35.0	-21.3 to -31.3
P-100	631,436.1	470,763.3	15.2	18.65	-30.7 to -40.7
P-101	630,184.2	472,382.8	34.7	37.4	-108.0 to -118.0
P-102	628,859.8	474,300.6	26.2	27.85	-120.8 to -130.8
P-103	628,661.2	467,861.5	12.5	14.25	-79.9 to -89.9
P-104	627,285.3	470,173.9	15.6	18.0	-131.8 to -141.8
P-105	626,676.5	471,759.8	21.6	24.0	-142.6 to -152.6
P-106	626,318.9	473,471.6	27.6	29.85	-127.6 to -137.6
P-107	630,202.4	469,113.5	9.8	11.85	-30.1 to -40.1
P-108	627,451.8	473,871.2	27.3	30.05	-142.9 to -152.9
Gas Monitor Well					
G-1	630,535.4	469,802.7	22.0	24.85	18.7 to 13.7
G-2	630,677.6	470,000.8	19.7	22.15	16.6 to 10.6
G-3	630,912.1	470,283.1	17.5	19.90	14.1 to 9.1
G-4	631,176.1	470,766.8	15.5	17.7	13.1 to 3.1
G-5	631,126.1	470,766.6	16.1	18.9	11.4 to 1.4
G-6	630,920.0	470,907.0	16.1	18.8	11.2 to 1.2
G-7	630,821.6	471,085.8	16.6	19.08	9.6 to 4.6
G-8	630,713.8	471,208.3	17.3	20.45	15.0 to 11.0
3/4 Inch Piezometer					
P-1	630,496.9	469,724.0	20.8	23.5	-6.2 to -8.2
P-1A	630,495.0	469,721.4	20.5	23.65	3.0 to 1.0
P-1B	630,716.4	469,718.3	20.4	21.7	13.9 to 11.81
P-2	630,716.4	470,000.4	19.8	22.05	-4.8 to -6.8
P-2A	630,714.7	469,998.4	18.8	22.85	8.2 to 6.2
P-3	630,982.6	470,304.2	18.4	22.3	-3.9 to -5.9
P-3A	630,979.6	470,301.5	14.3	21.8	9.0 to 7.8
P-4	631,180.3	470,336.9	14.8	20.2	-9.5 to -11.5
2 Inch Observation Well					
P-5	629,208.6	473,571.7	28.7	31.75	5.5 to 0.5
P-5A	629,210.5	473,568.7	28.8	31.75	19.7 to 14.7
P-6	629,167.9	473,531.6	34.1	37.0	10.8 to 5.8
P-6A	629,169.0	473,529.1	34.2	36.8	24.0 to 19.0
P-7	627,943.7	474,008.4	26.2	29.55	7.7 to 2.7
P-7A	627,941.8	474,000.4	26.1	29.25	19.3 to 14.3
P-8	626,753.5	473,654.6	26.6	29.4	12.5 to 7.5
P-8A	626,751.2	473,654.2	26.6	29.45	22.6 to 17.6
P-9	626,382.5	472,450.2	34.1	37.8	13.8 to 8.8
P-9A	626,380.9	472,461.1	34.3	37.6	24.8 to 19.8
P-10	626,834.4	471,676.7	27.6	31.35	-8.2 to -13.2
P-10A	626,836.8	471,673.0	27.4	30.4	10.2 to 5.5
P-11	627,358.8	470,225.0	27.7	30.55	-11.5 to -16.5
P-11A	627,358.6	470,221.9	28.0	30.7	9.5 to 4.5
P-12	628,243.6	468,083.4	28.1	31.35	-8.1 to -13.1
P-12A	628,246.3	468,081.2	28.0	31.45	11.9 to 6.9
P-13	630,656.8	470,955.0	39.6	42.65	16.1 to 11.1

Inclinometer				Casing Elevation-NGVD (Feet) Top	Bottom
IC-1	630,504.1	469,721.2	18.5	20.15	-29.5
IC-2	630,717.9	470,002.8	19.1	21.1	-33.9
IC-3	630,982.1	470,280.9	13.2	17.8	-32.8
IC-4	631,190.9	470,561.9	14.9	16.45	-31.6
IC-5	629,196.0	473,588.8	29.3	33.40	-56.7

NOTES:

1) Location coordinates for groundwater monitor wells are those reported on Figure 2-1, Cherry Island Site Suitability Report, prepared by Terraqua Resources Corporation and dated January 1984. Location coordinates for: groundwater monitor wells C-109 & C-110, gas monitor wells, 3/4 inch piezometers, 2 inch observation wells and inclinometers, are based on the December 1985 survey, performed by VanDemark and Lynch.

2) Top of casing and approximate ground surface elevations are based on the December 1985 survey, performed by VanDemark and Lynch.

3) Screen interval elevations for groundwater monitor wells are those reported in Appendix A, Monitoring Well Logs, Cherry Island Site Suitability Report, prepared by Terraqua Resources Corporation and dated January 1984. Well screen interval elevations for: groundwater monitor wells C-109 & C-110, gas monitor wells, 3/4 inch piezometers, and 2 inch observation wells, are based on field measurements, performed by Duffield Associates, Inc. and referenced to reported top of casing elevations.

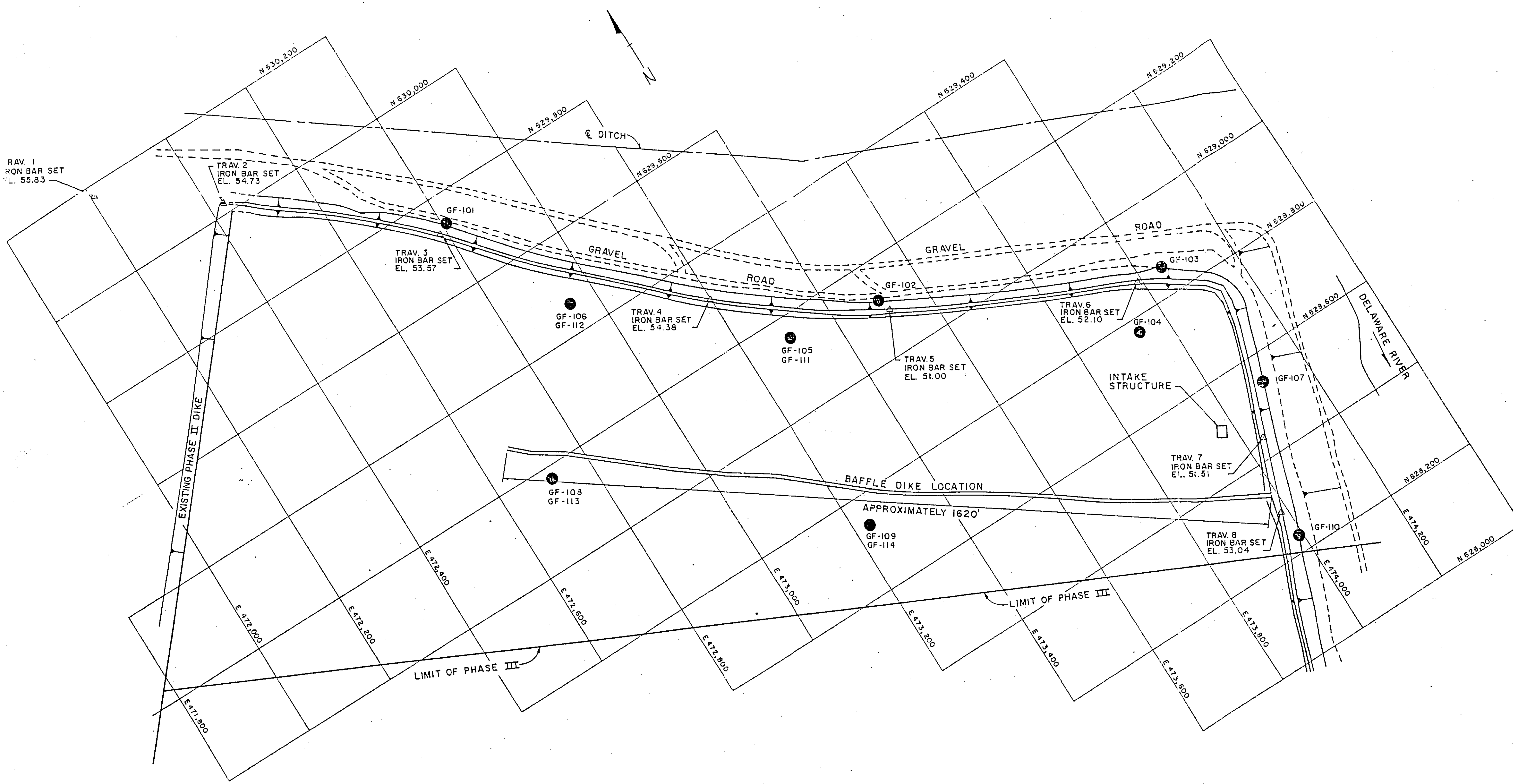
KEY:

- P-104 GROUNDWATER MONITOR WELL (4"DIA.)
- G-1 LANDFILL GAS MONITOR WELL (4"DIA.)
- P-1 PIEZOMETER (3/4"DIA.)
- P-6 GROUNDWATER OBSERVATION WELL (2"DIA.)
- IC-1 SLOPE INCLINOMETER
- Existing Borings
- Proposed Borings

PHASE III
NORTHERN SOLID WASTE FACILITY - 2
DELAWARE SOLID WASTE AUTHORITY

SITE LOCATION, DIKE LOCATION EXISTING AND PROPOSED BORING LOCATIONS

GANNETT FLEMING ENVIRONMENTAL ENGINEERS, INC.
BALTIMORE, MARYLAND AUG. 1989 FIGURE - 1



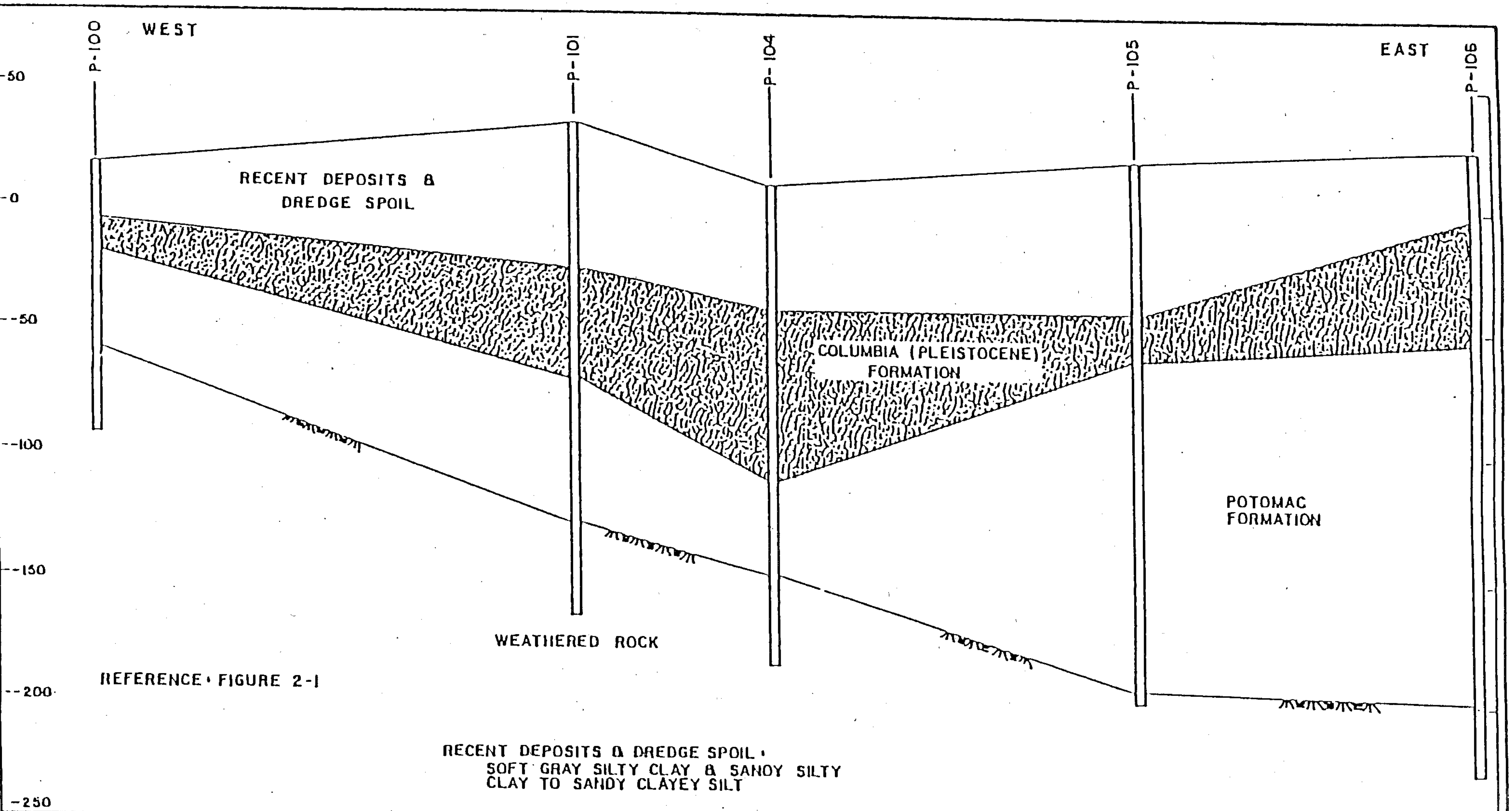
GF-101 THRU GF-110 INDICATE SPT BORINGS.
GF-111 THRU GF-114 INDICATE DILATOMETER TEST BORINGS.

SCALE
1"=100' 50 0 100

PHASE III
NORTHERN SOLID WASTE FACILITY - 2
DELAWARE SOLID WASTE AUTHORITY

BORING LOCATION PLAN

GANNETT FLEMING, INC.
HARRISBURG, PENNSYLVANIA
JANUARY 19



RECENT DEPOSITS & DREDGE SPOIL:
SOFT GRAY SILTY CLAY & SANDY SILTY CLAY TO SANDY CLAYEY SILT

COLUMBIA (PLEISTOCENE) FORMATION
MULTICOLOR FINE TO COARSE SAND AND GRAVEL WITH SILTY SAND, SILTY CLAY AND CLAYEY SILT LAYERS

POTOMAC FORMATION:
RED, GRAY, GREEN, CREAM CLAY, FINE SAND, SILT, SILTY SAND, SANDY SILT

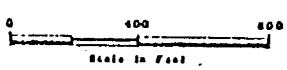
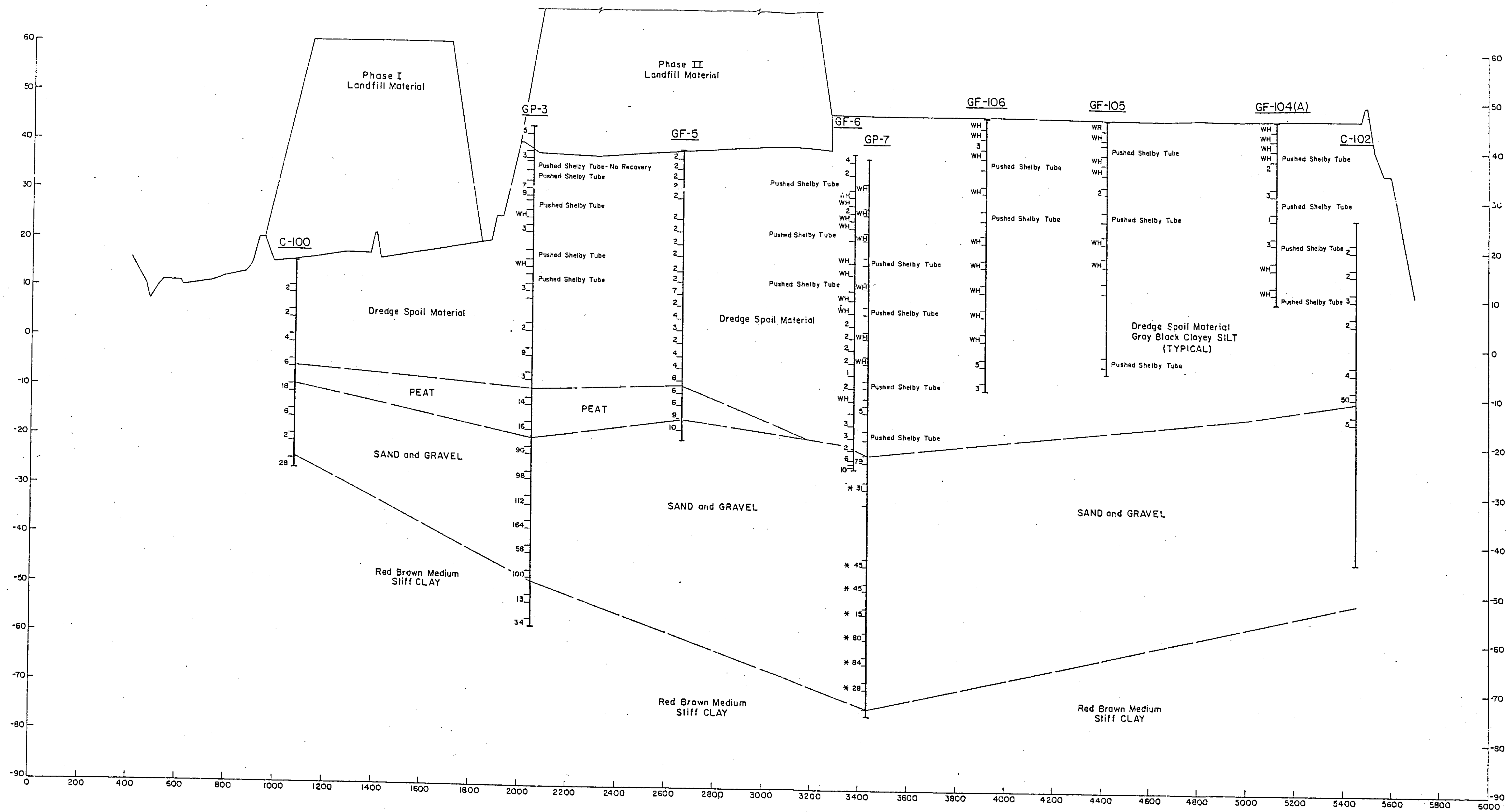


FIGURE 1.2
GEOLOGIC CROSS-SECTION A-A
CHERRY ISLAND



SECTION E-E

SCALES

200 0 200 400

HORIZ: 1 IN. = 200 FT.

10 0 10 20

VERT: 1 IN. = 10 FT.

LEGEND

BORINGS DESIGNATED AS "GP" PERFORMED BY TERRAQUA ASSOCIATES.
BORINGS DESIGNATED AS "GF" PERFORMED FOR PHASE II LANDFILL.
BORINGS DESIGNATED AS "GF 101" TO "GF 110" PERFORMED FOR PHASE III LANDFILL.

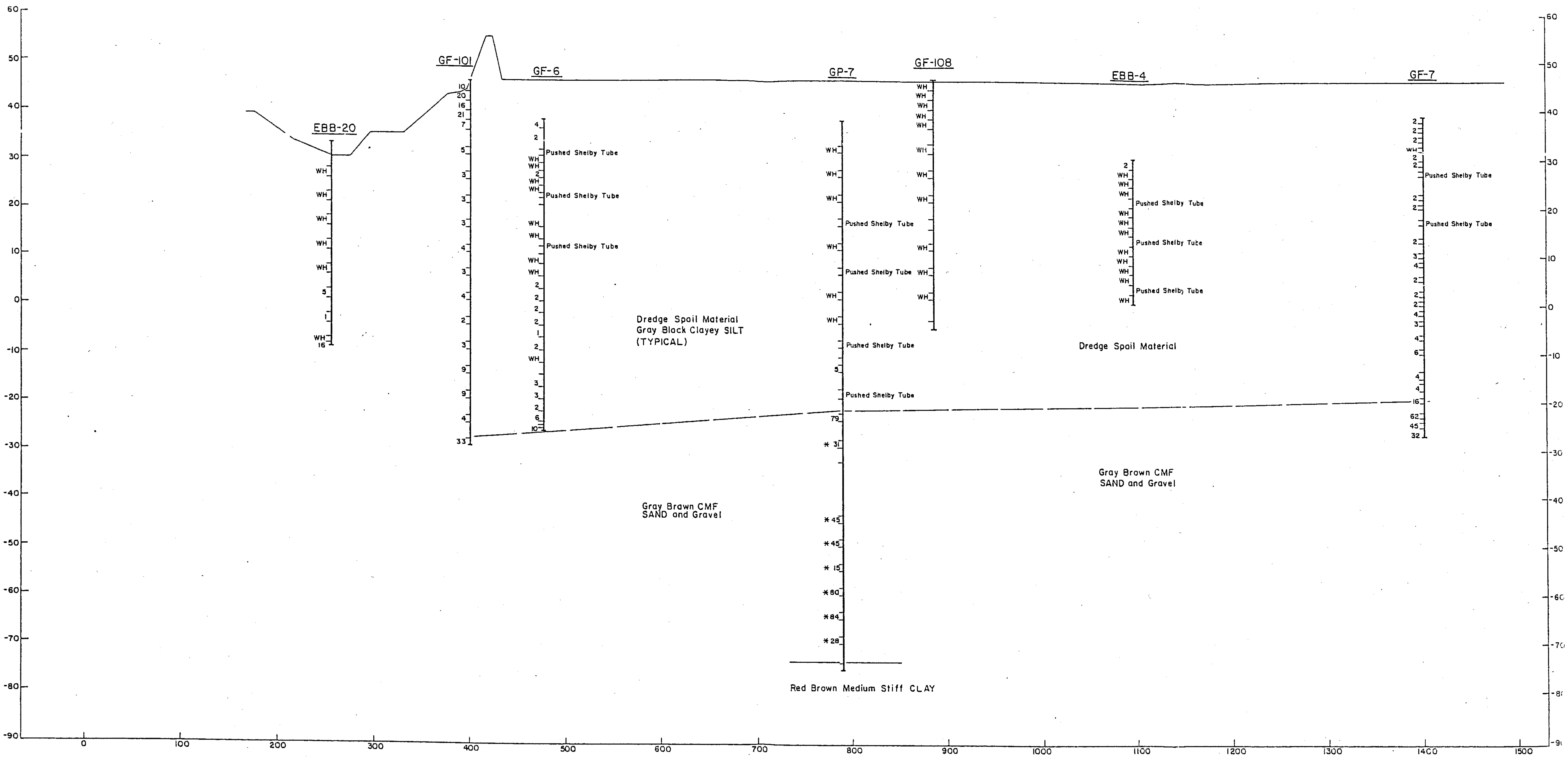
NOTES

- WH = WEIGHT OF HAMMER
- ELEVATIONS SHOWN ARE AS OF FEBRUARY 1990.
- * = 300 LB. HAMMER USED
- 2 = INDICATES THE NUMBER OF BLOWS PER ONE FOOT OF PENETRATION

PHASE III
NORTHERN SOLID WASTE FACILITY - 2
DELAWARE SOLID WASTE AUTHORITY

SOIL PROFILE
SECTION E-E

GANNETT FLEMING, INC.
HARRISBURG, PENNSYLVANIA FEB 1990 FIGURE 13



SECTION F-F

SCALES

HORIZ.: 1 IN. = 50 FT.

VERT.: 1 IN. = 10 FT.

LEGEND

BORINGS DESIGNATED AS "GP" PERFORMED BY TERRAQUA ASSOCIATES.
BORINGS DESIGNATED AS "GF-1" TO "GF-14" PERFORMED FOR PHASE II LANDFILL.
BORINGS DESIGNATED AS "GF-101" TO "GF-110" PERFORMED FOR PHASE III LANDFILL.
BORINGS DESIGNATED AS "EBB" PERFORMED FOR THE U.S. ARMY C.O.E.

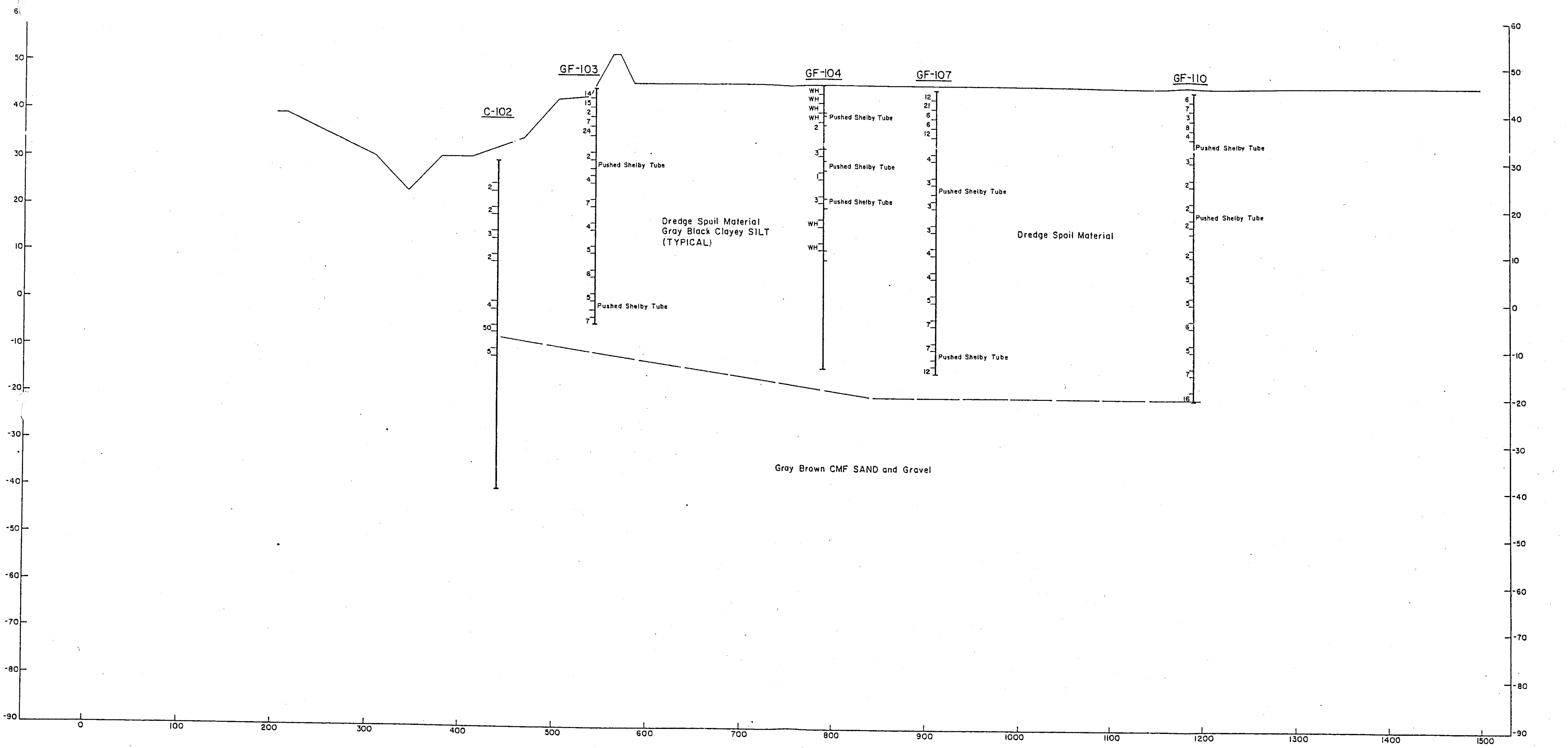
NOTES

- 2 = INDICATES THE NUMBER OF BLOWS PER ONE FOOT OF PENETRATION
- ELEVATIONS SHOWN WERE AS OF OCTOBER 1989 AND MAY NOT REPRESENT PRESENT CONDITIONS AT THE SITE.
- * = 300 LB. HAMMER USED
- WH = WEIGHT OF HAMMER

PHASE III
NORTHERN SOLID WASTE FACIL
DELAWARE SOLID WASTE AUTI

SOIL PROFILE
SECTION F-F

GANNETT FLEMING, INC.
HARRISBURG, PENNSYLVANIA FEB. 199



SECTION G-G

SCALES

HORIZ.: 1 IN. = 50 FT.

VERT.: 1 IN. = 10 FT.

LEGEND

BORINGS DESIGNATED AS "C" PERFORMED BY TERRAQUA ASSOCIATES.
BORINGS DESIGNATED AS "GF-101" TO "GF-110" PERFORMED FOR PHASE III LANDFILL.

NOTES

- ELEVATIONS SHOWN WERE AS OF OCTOBER 1989 AND MAY NOT REPRESENT PRESENT CONDITIONS AT THE SITE.
- WH = WEIGHT OF HAMMER
- 2 = INDICATES THE NUMBER OF BLOWS PER ONE FOOT OF PENETRATION

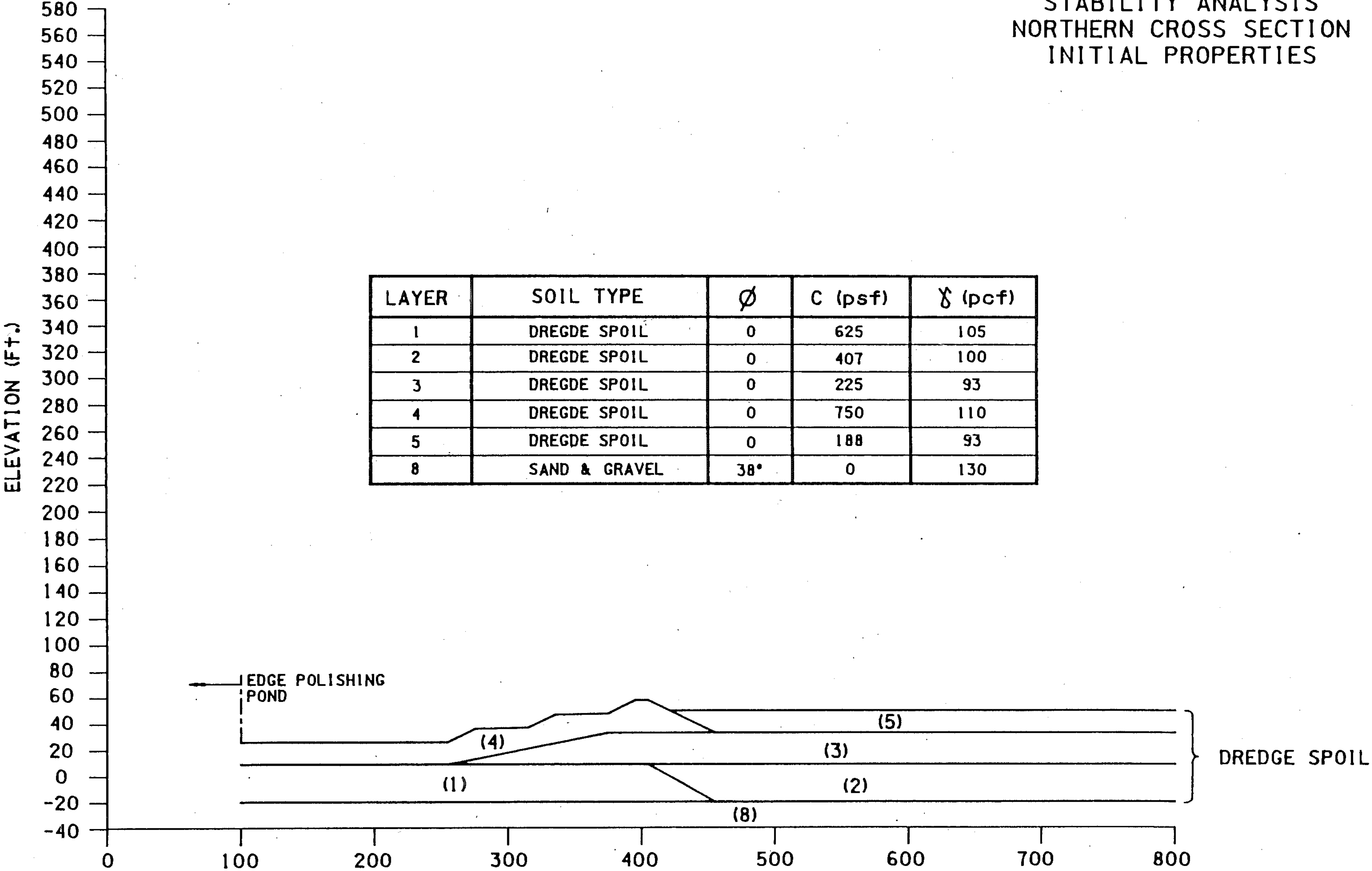
PHASE III
NORTHERN SOLID WASTE FACILITY - 2
DELAWARE SOLID WASTE AUTHORITY

SOIL PROFILE
SECTION G-G

GANNETT FLEMING, INC.

STABILITY ANALYSIS
NORTHERN CROSS SECTION
INITIAL PROPERTIES

LAYER	SOIL TYPE	Ø	C (psf)	γ (pcf)
1	DREGDE SPOIL	0	625	105
2	DREGDE SPOIL	0	407	100
3	DREGDE SPOIL	0	225	93
4	DREGDE SPOIL	0	750	110
5	DREGDE SPOIL	0	188	93
8	SAND & GRAVEL	38°	0	130



SCALE: 1" = 100'

FIGURE 2.1
GANNETT FLEMING, INC.
MARCH 1990

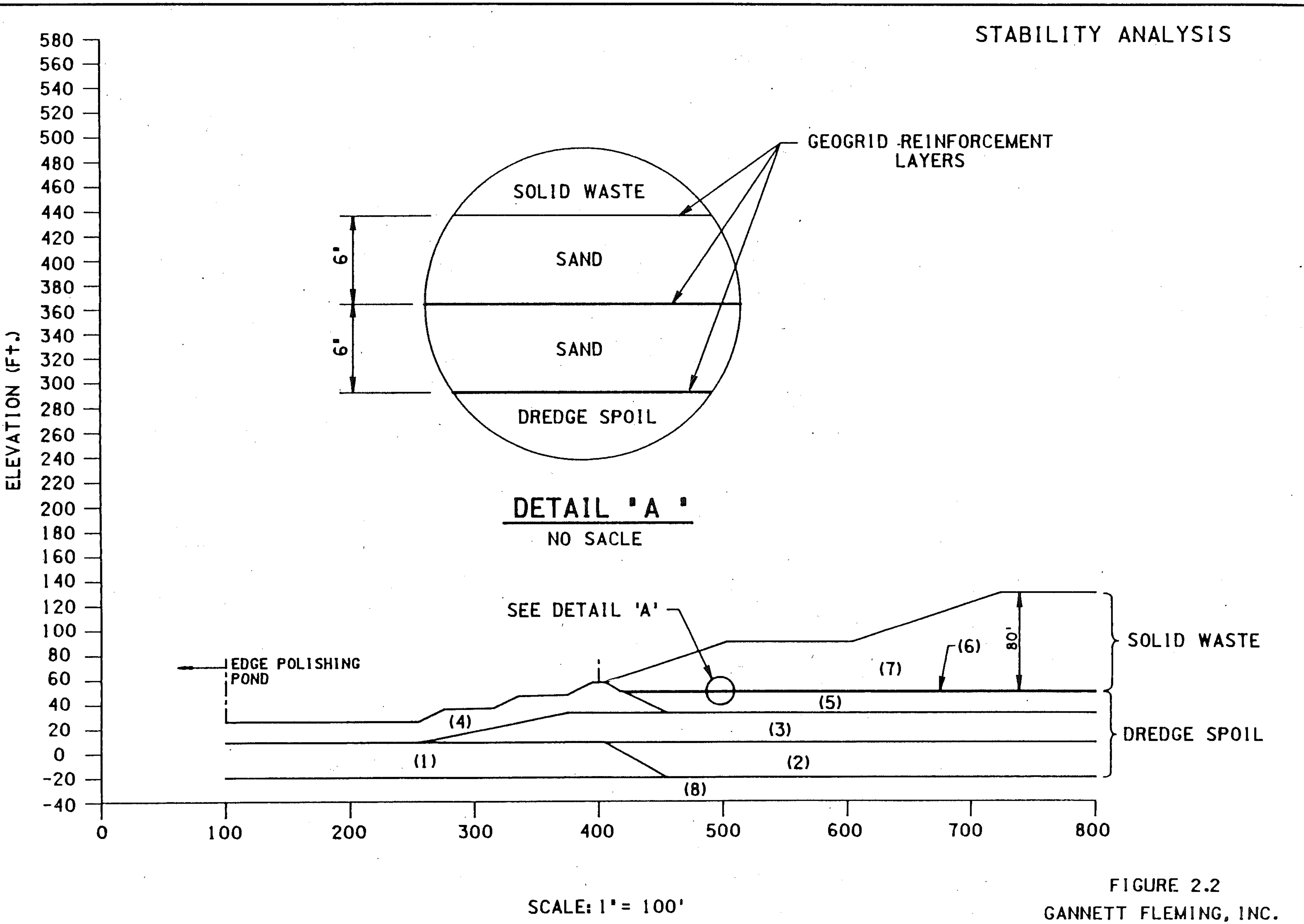
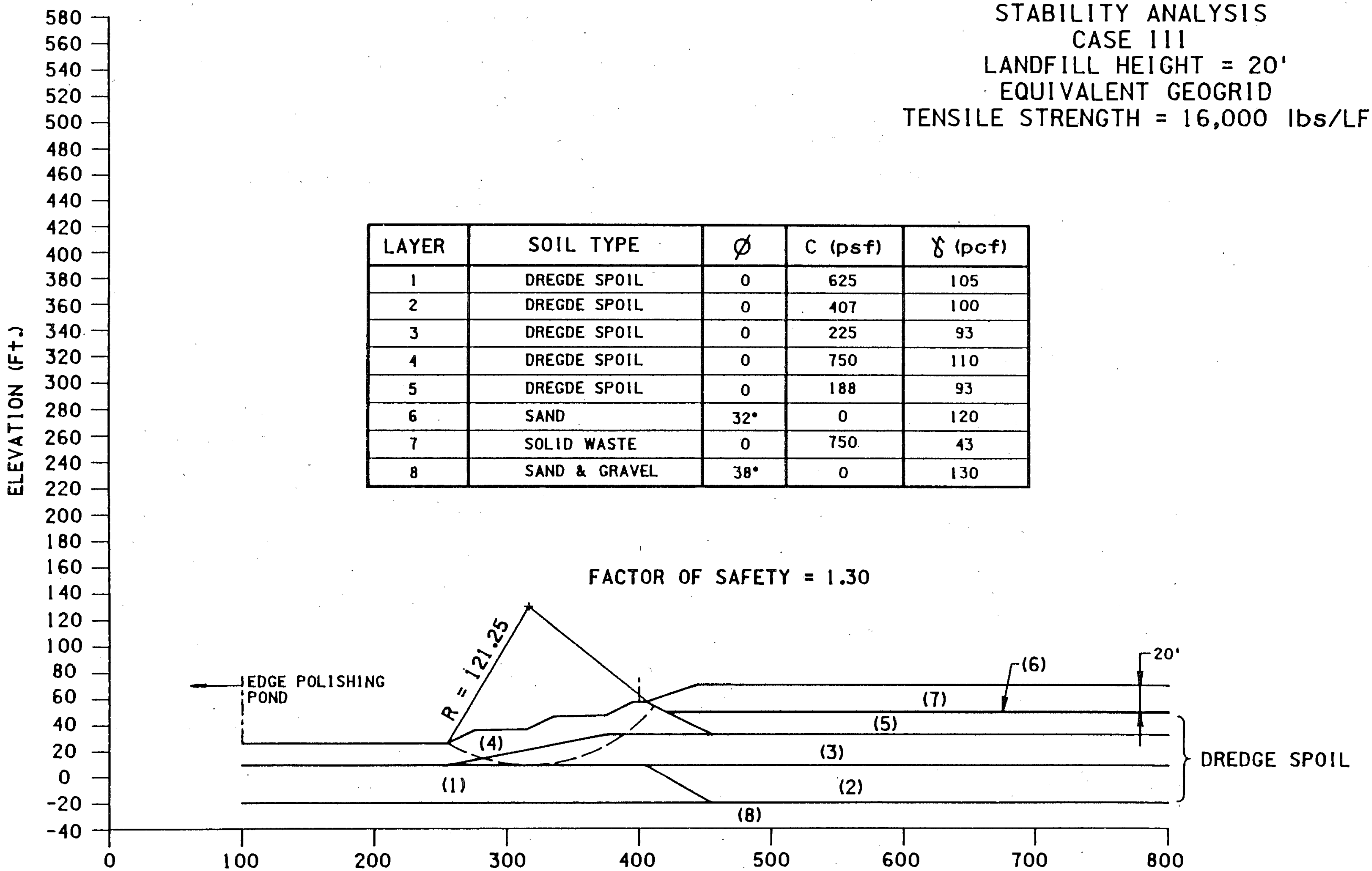


FIGURE 2.2
GANNETT FLEMING, INC.
MARCH 1990

STABILITY ANALYSIS
CASE III
LANDFILL HEIGHT = 20'
EQUIVALENT GEOGRID
TENSILE STRENGTH = 16,000 lbs/LF

LAYER	SOIL TYPE	Ø	C (psf)	γ (pcf)
1	DREGDE SPOIL	0	625	105
2	DREGDE SPOIL	0	407	100
3	DREGDE SPOIL	0	225	93
4	DREGDE SPOIL	0	750	110
5	DREGDE SPOIL	0	188	93
6	SAND	32°	0	120
7	SOLID WASTE	0	750	43
8	SAND & GRAVEL	38°	0	130

FACTOR OF SAFETY = 1.30

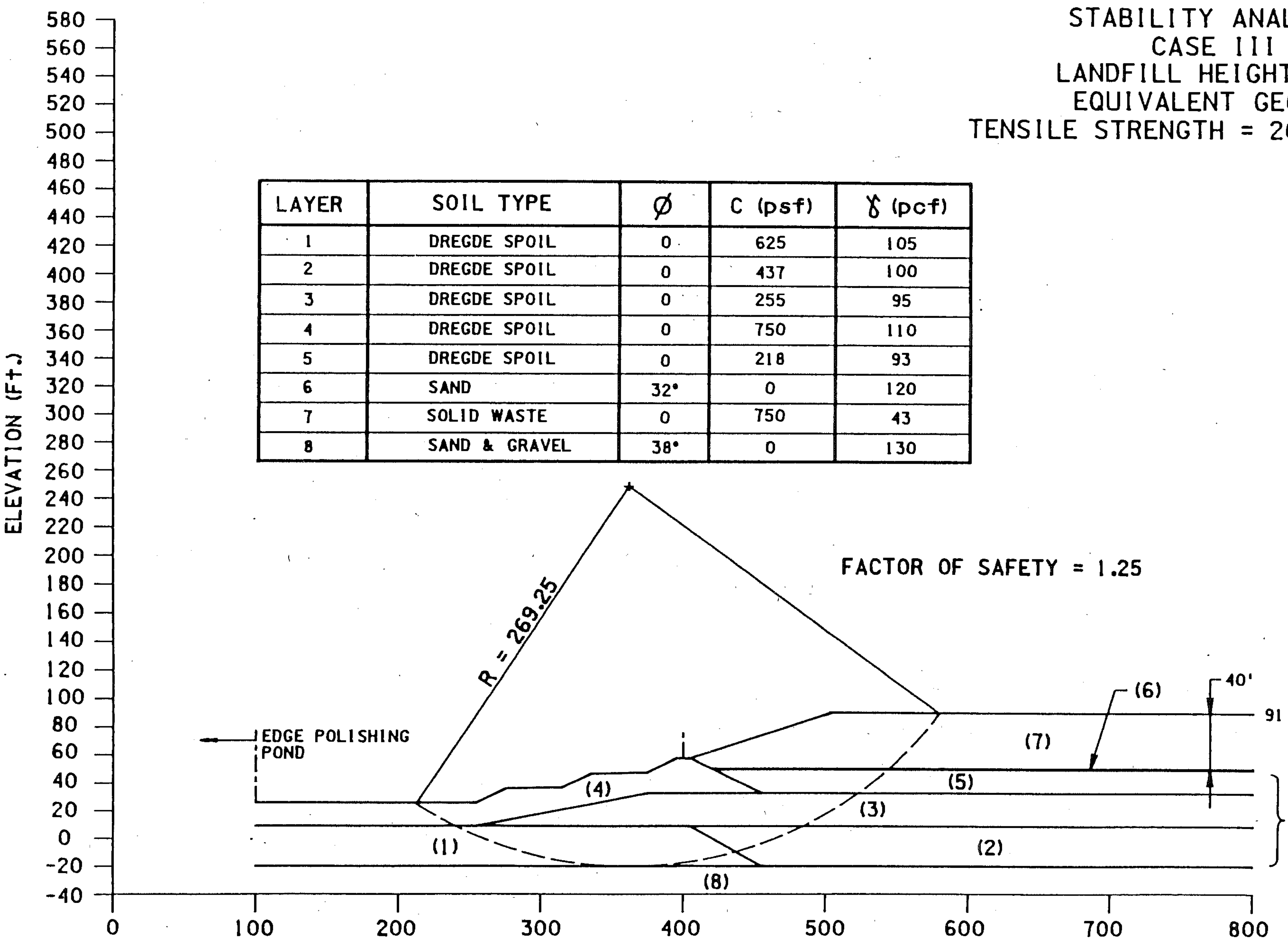


SCALE: 1" = 100'

FIGURE 2.3
GANNETT FLEMING, INC.
MARCH 1990

STABILITY ANALYSIS
CASE III
LANDFILL HEIGHT = 40'
EQUIVALENT GEOGRID
TENSILE STRENGTH = 20,000 lbs/LF

LAYER	SOIL TYPE	Ø	C (psf)	γ (pcf)
1	DREGDE SPOIL	0	625	105
2	DREGDE SPOIL	0	437	100
3	DREGDE SPOIL	0	255	95
4	DREGDE SPOIL	0	750	110
5	DREGDE SPOIL	0	218	93
6	SAND	32°	0	120
7	SOLID WASTE	0	750	43
8	SAND & GRAVEL	38°	0	130

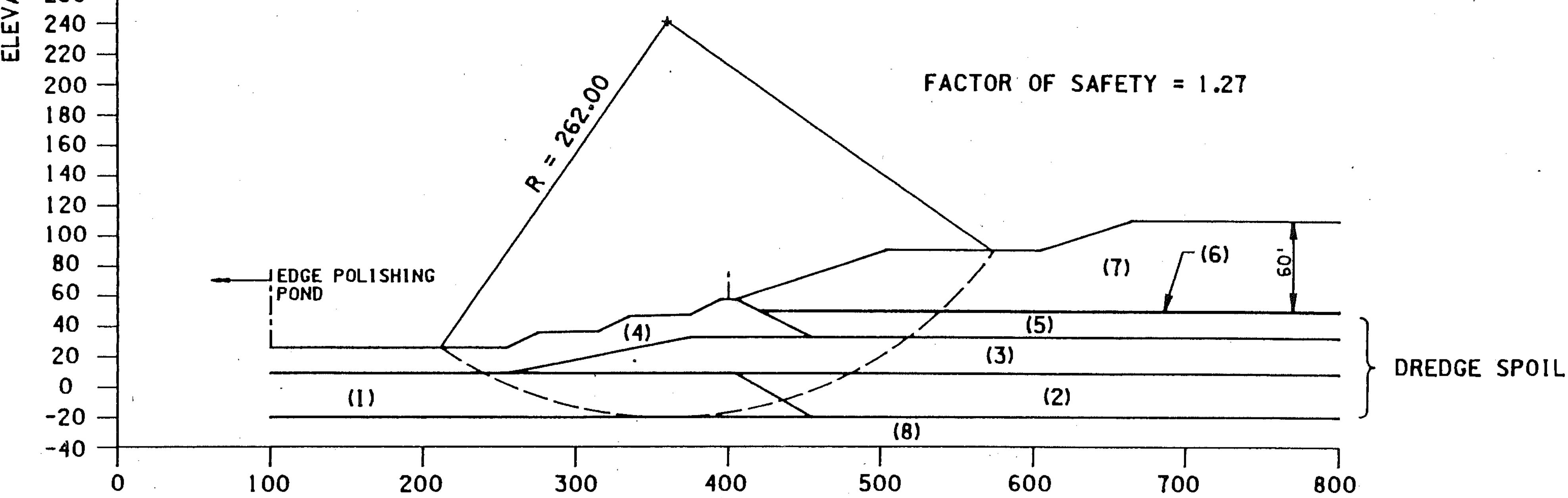


SCALE: 1" = 100'

FIGURE 2.4
GANNETT FLEMING, INC.
MARCH 1990

STABILITY ANALYSIS
CASE III
LANDFILL HEIGHT = 60'
EQUIVALENT GEOGRID
TENSILE STRENGTH = 20,000 lbs/LF

LAYER	SOIL TYPE	Ø	C (psf)	γ (pcf)
1	DREGDE SPOIL	0	625	105
2	DREGDE SPOIL	0	494	100
3	DREGDE SPOIL	0	312	97
4	DREGDE SPOIL	0	750	110
5	DREGDE SPOIL	0	275	95
6	SAND	32°	0	120
7	SOLID WASTE	0	750	43
8	SAND & GRAVEL	38°	0	130

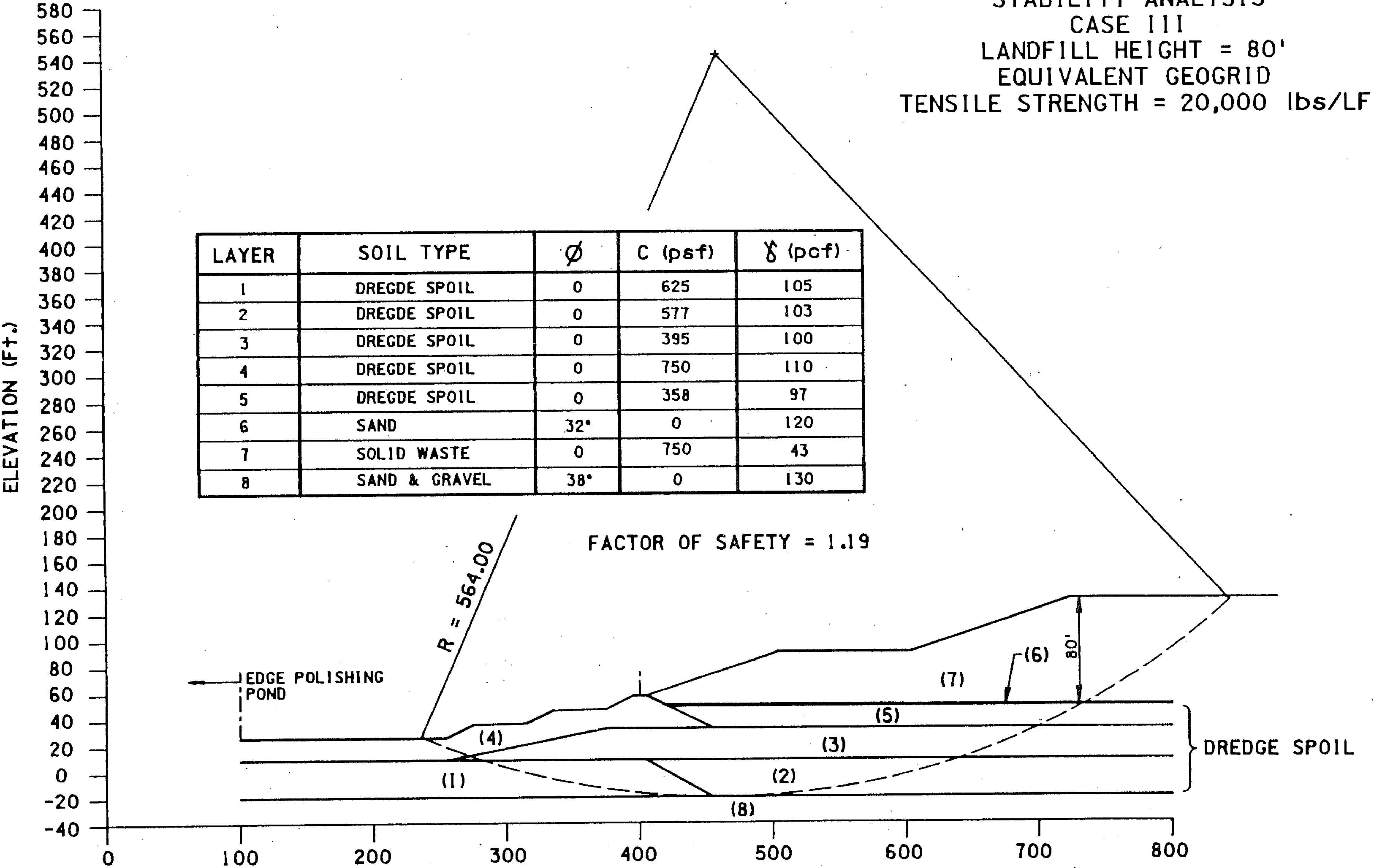


SCALE: 1" = 100'

FIGURE 2.5
GANNETT FLEMING, INC.
MARCH 1990

STABILITY ANALYSIS
CASE III
LANDFILL HEIGHT = 80'
EQUIVALENT GEOGRID
TENSILE STRENGTH = 20,000 lbs/LF

LAYER	SOIL TYPE	Ø	C (psf)	γ (pcf)
1	DREGDE SPOIL	0	625	105
2	DREGDE SPOIL	0	577	103
3	DREGDE SPOIL	0	395	100
4	DREGDE SPOIL	0	750	110
5	DREGDE SPOIL	0	358	97
6	SAND	32°	0	120
7	SOLID WASTE	0	750	43
8	SAND & GRAVEL	38°	0	130

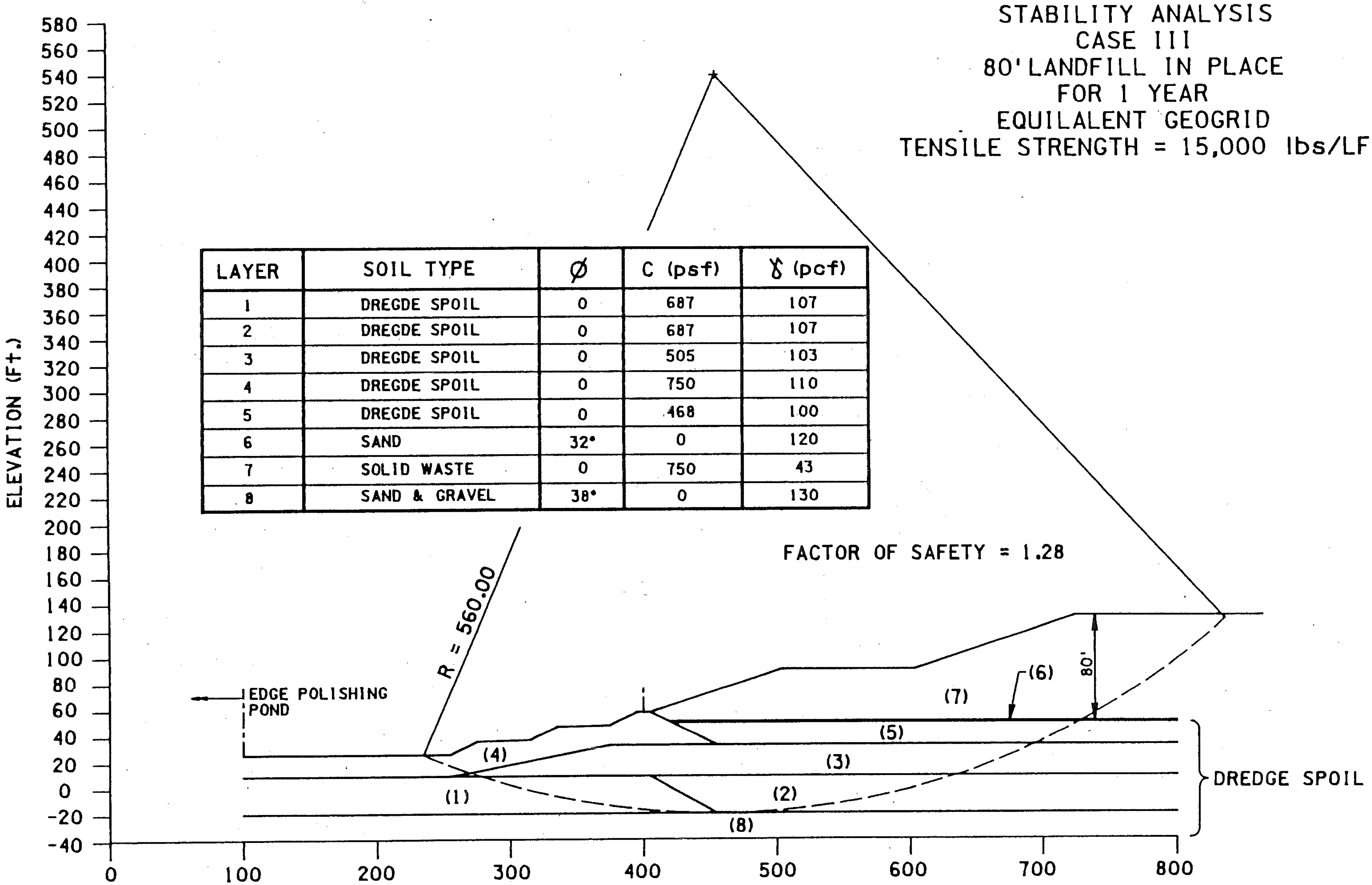


SCALE: 1" = 100'

FIGURE 2.6
GANNETT FLEMING, INC.
MARCH 1990

STABILITY ANALYSIS
CASE III
80' LANDFILL IN PLACE
FOR 1 YEAR
EQUILALENT GEOGRID
TENSILE STRENGTH = 15,000 lbs/LF

LAYER	SOIL TYPE	Ø	C (psf)	γ (pcf)
1	DREGDE SPOIL	0	687	107
2	DREGDE SPOIL	0	687	107
3	DREGDE SPOIL	0	505	103
4	DREGDE SPOIL	0	750	110
5	DREGDE SPOIL	0	468	100
6	SAND	32°	0	120
7	SOLID WASTE	0	750	43
8	SAND & GRAVEL	38°	0	130

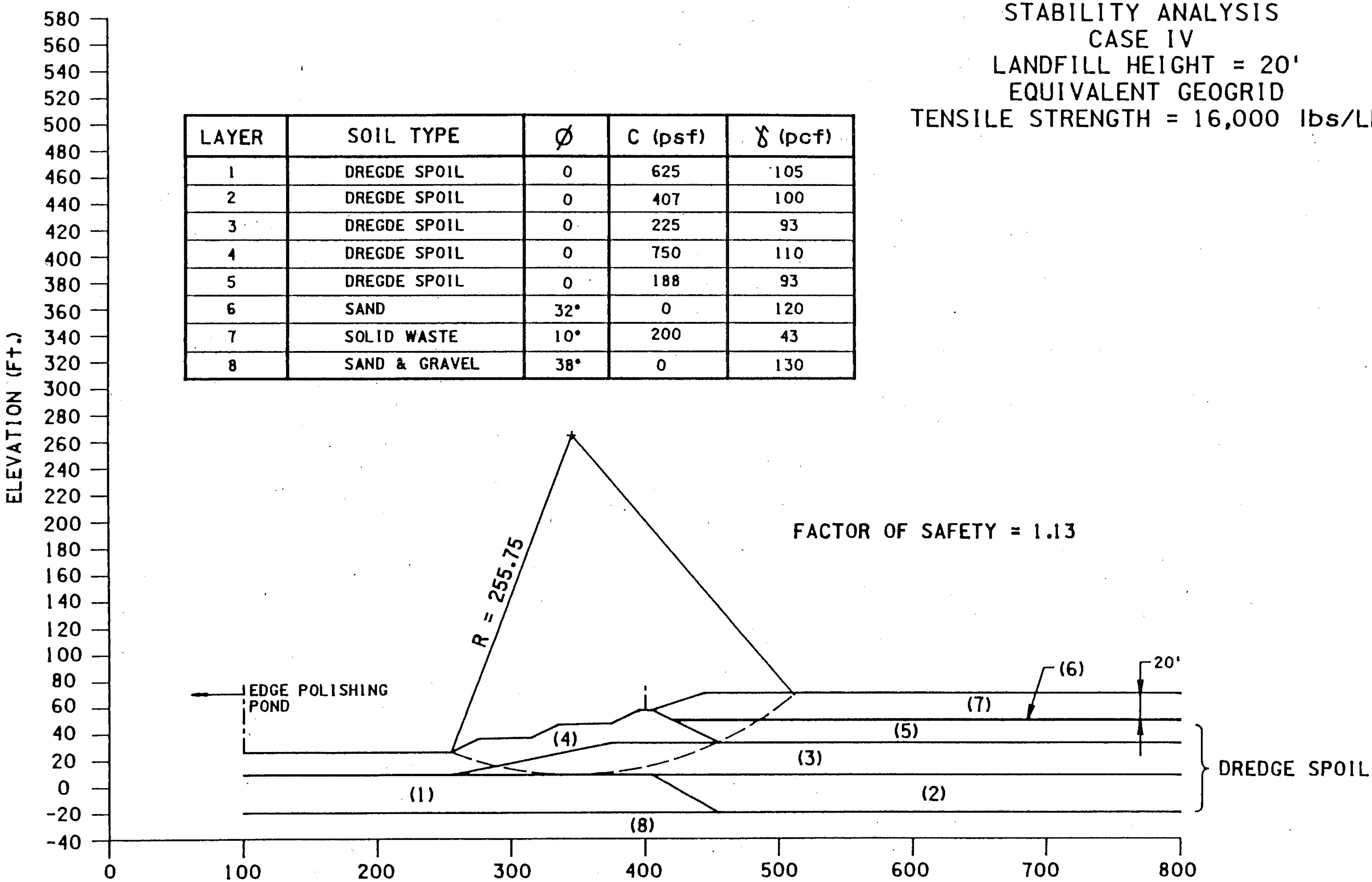


SCALE: 1" = 100'

FIGURE 2.7
GANNETT FLEMING, INC.
MARCH 1990

STABILITY ANALYSIS
CASE IV
LANDFILL HEIGHT = 20'
EQUIVALENT GEOGRID
TENSILE STRENGTH = 16,000 lbs/LF

LAYER	SOIL TYPE	Ø	C (psf)	γ (pcf)
1	DREGDE SPOIL	0	625	105
2	DREGDE SPOIL	0	407	100
3	DREGDE SPOIL	0	225	93
4	DREGDE SPOIL	0	750	110
5	DREGDE SPOIL	0	188	93
6	SAND	32°	0	120
7	SOLID WASTE	10°	200	43
8	SAND & GRAVEL	38°	0	130

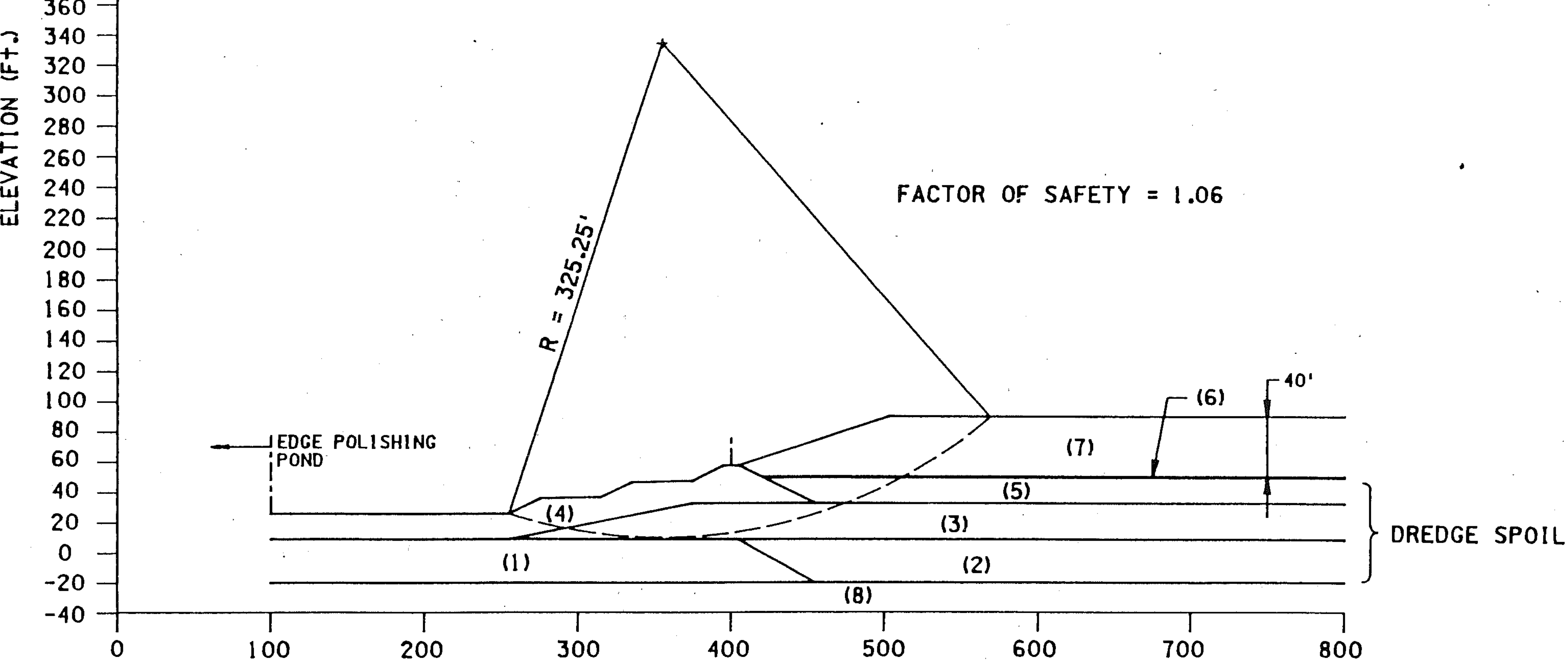


SCALE: 1" = 100'

FIGURE 2.8
GANNETT FLEMING, INC.
MARCH 1990

STABILITY ANALYSIS
CASE IV
LANDFILL HEIGHT = 40'
EQUIVALENT GEOGRID
TENSILE STRENGTH = 20,000 lbs/LF

LAYER	SOIL TYPE	Ø	C (psf)	γ (pcf)
1	DREGDE SPOIL	0	625	105
2	DREGDE SPOIL	0	437	100
3	DREGDE SPOIL	0	255	95
4	DREGDE SPOIL	0	750	110
5	DREGDE SPOIL	0	218	93
6	SAND	32°	0	120
7	SOLID WASTE	10°	200	43
8	SAND & GRAVEL	38°	0	130

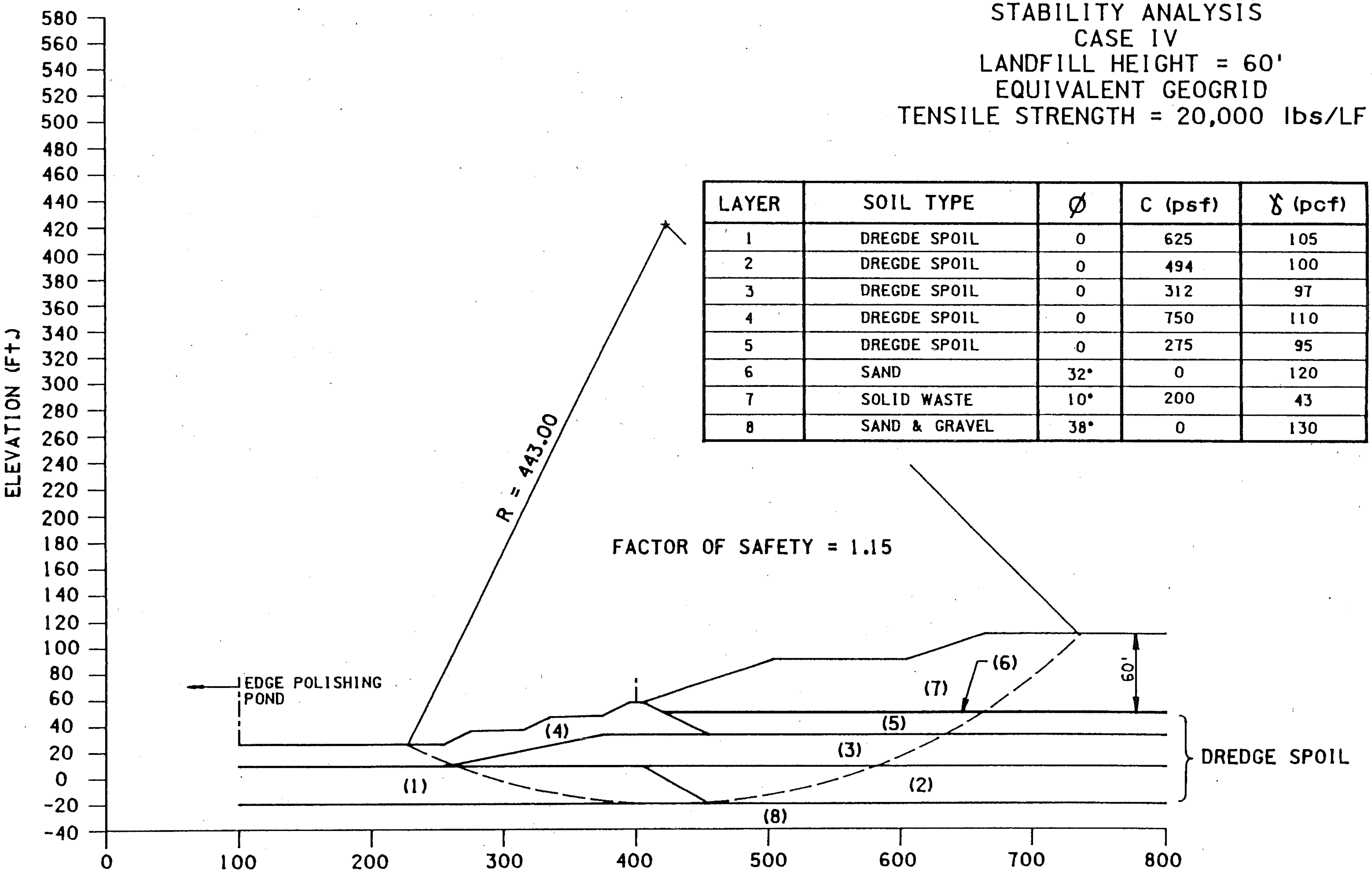


SCALE: 1" = 100'

FIGURE 2.9
GANNETT FLEMING, INC.
MARCH 1990

STABILITY ANALYSIS
CASE IV
LANDFILL HEIGHT = 60'
EQUIVALENT GEOGRID
TENSILE STRENGTH = 20,000 lbs/LF

LAYER	SOIL TYPE	Ø	C (psf)	γ (pcf)
1	DREGDE SPOIL	0	625	105
2	DREGDE SPOIL	0	494	100
3	DREGDE SPOIL	0	312	97
4	DREGDE SPOIL	0	750	110
5	DREGDE SPOIL	0	275	95
6	SAND	32°	0	120
7	SOLID WASTE	10°	200	43
8	SAND & GRAVEL	38°	0	130

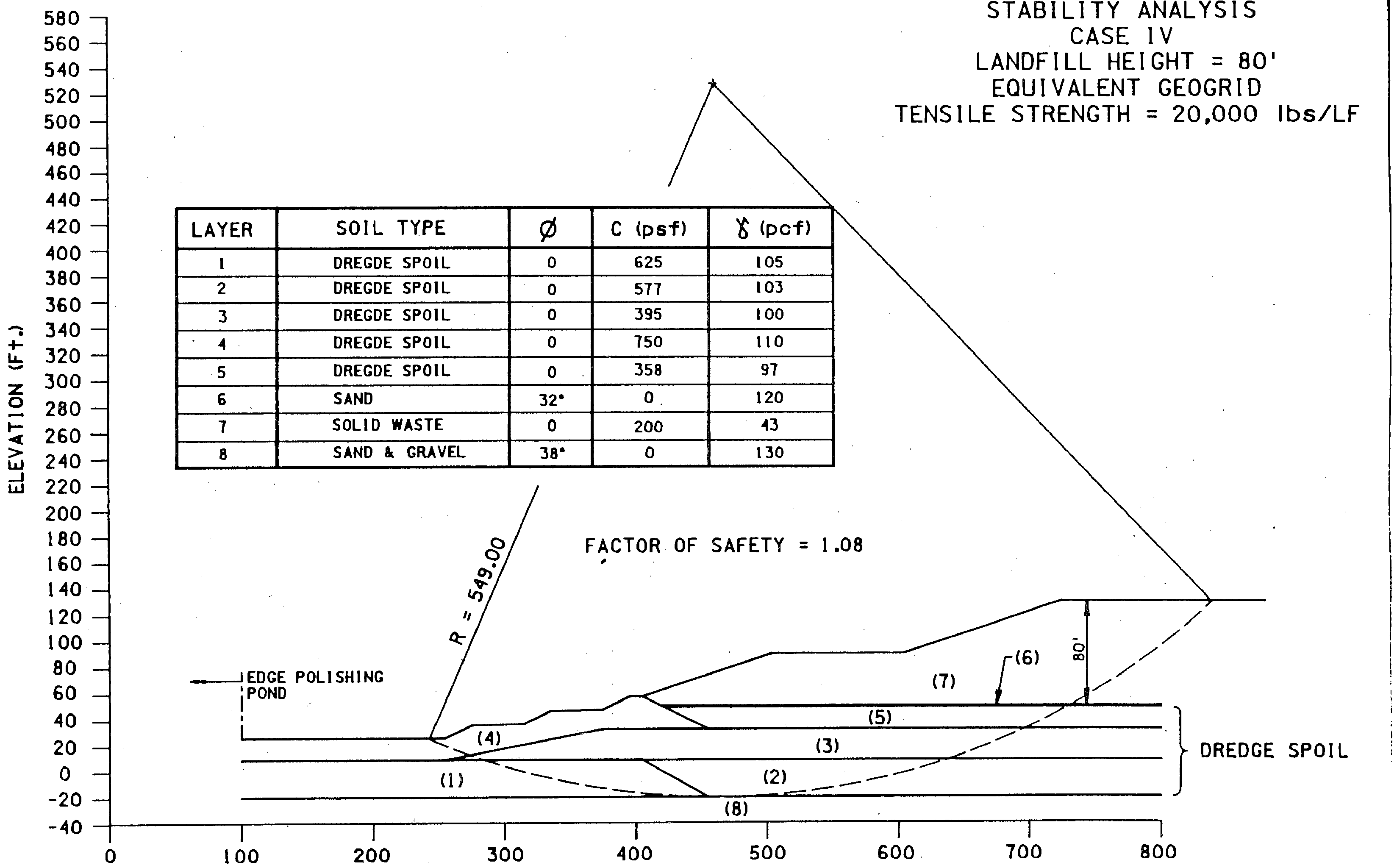


SCALE: 1" = 100'

FIGURE 2.10
GANNETT FLEMING, INC.
MARCH 1990

STABILITY ANALYSIS
CASE IV
LANDFILL HEIGHT = 80'
EQUIVALENT GEOGRID
TENSILE STRENGTH = 20,000 lbs/LF

LAYER	SOIL TYPE	Ø	C (psf)	γ (pcf)
1	DREGDE SPOIL	0	625	105
2	DREGDE SPOIL	0	577	103
3	DREGDE SPOIL	0	395	100
4	DREGDE SPOIL	0	750	110
5	DREGDE SPOIL	0	358	97
6	SAND	32°	0	120
7	SOLID WASTE	0	200	43
8	SAND & GRAVEL	38°	0	130

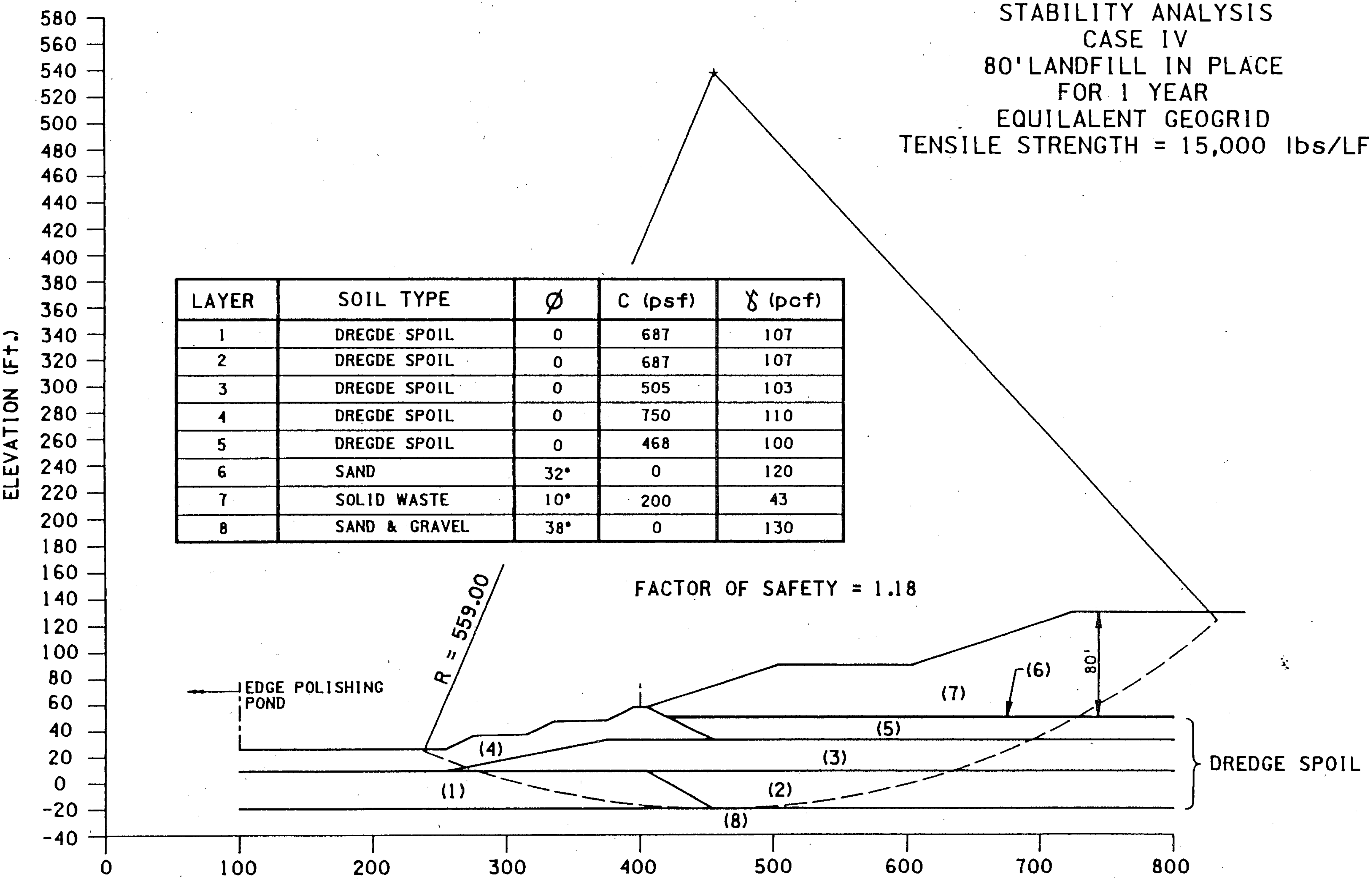


SCALE: 1" = 100'

FIGURE 2.11
GANNETT FLEMING, INC.
MARCH 1990

STABILITY ANALYSIS
CASE IV
80'LANDFILL IN PLACE
FOR 1 YEAR
EQUILALENT GEOGRID
TENSILE STRENGTH = 15,000 lbs/LF

LAYER	SOIL TYPE	Ø	C (psf)	γ (pcf)
1	DREGDE SPOIL	0	687	107
2	DREGDE SPOIL	0	687	107
3	DREGDE SPOIL	0	505	103
4	DREGDE SPOIL	0	750	110
5	DREGDE SPOIL	0	468	100
6	SAND	32°	0	120
7	SOLID WASTE	10°	200	43
8	SAND & GRAVEL	38°	0	130

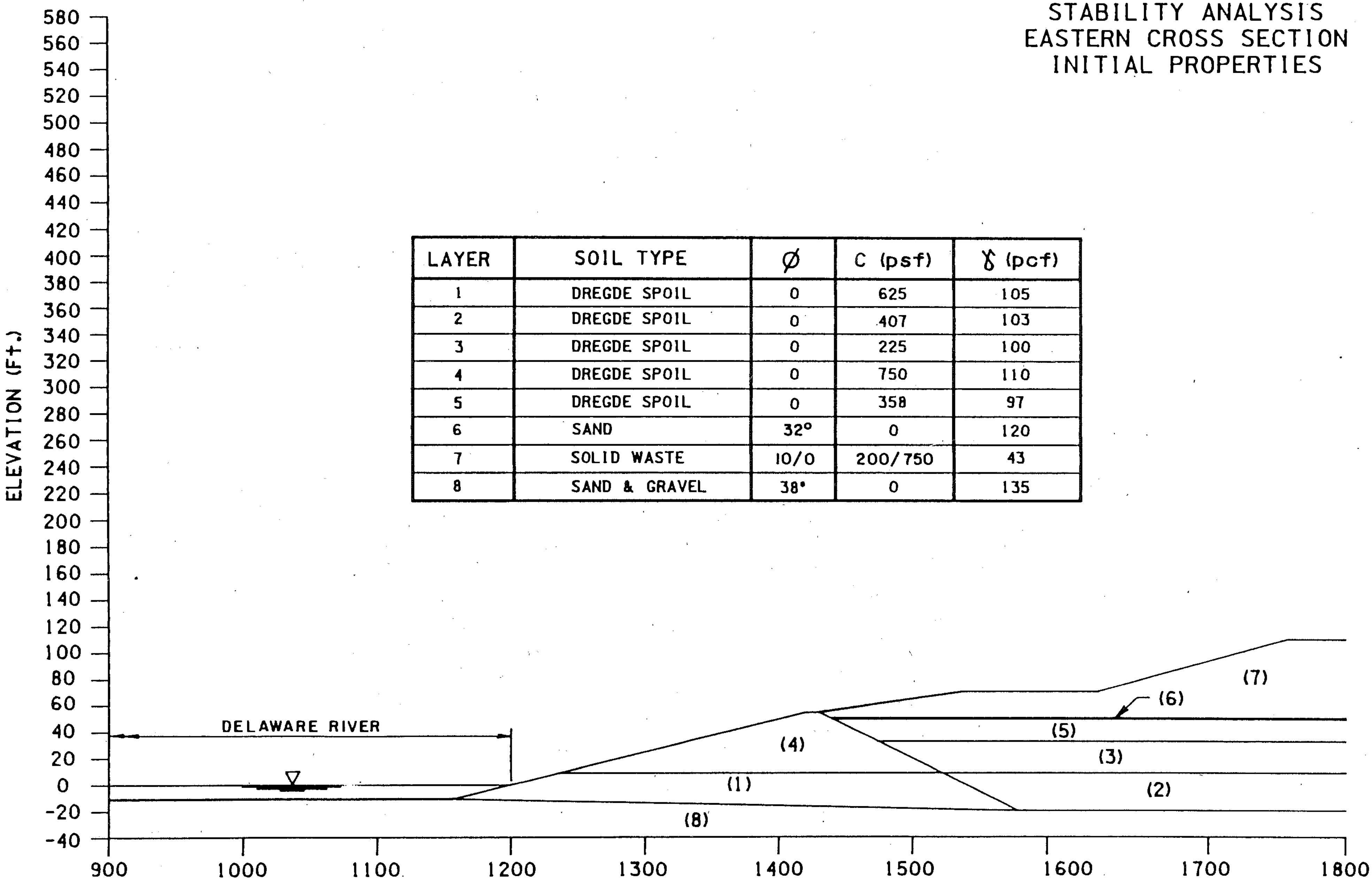


SCALE: 1" = 100'

FIGURE 2.12
GANNETT FLEMING, INC.
MARCH 1990

STABILITY ANALYSIS
EASTERN CROSS SECTION
INITIAL PROPERTIES

LAYER	SOIL TYPE	Ø	C (psf)	γ (pcf)
1	DREGDE SPOIL	0	625	105
2	DREGDE SPOIL	0	407	103
3	DREGDE SPOIL	0	225	100
4	DREGDE SPOIL	0	750	110
5	DREGDE SPOIL	0	358	97
6	SAND	32°	0	120
7	SOLID WASTE	10/0	200/750	43
8	SAND & GRAVEL	38°	0	135

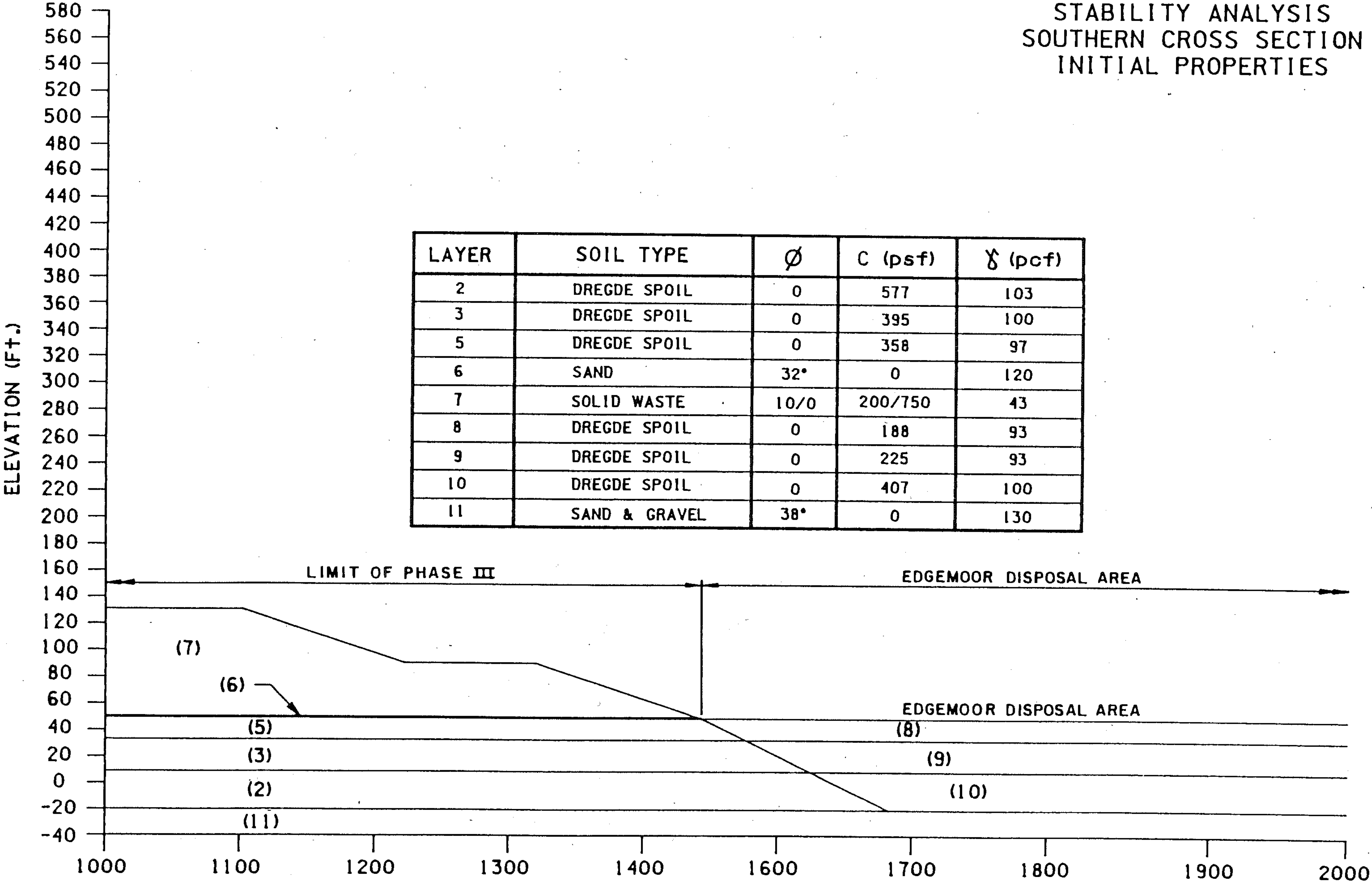


SCALE: 1" = 100'

FIGURE 2.13
GANNETT FLEMING, INC.
MARCH 1990

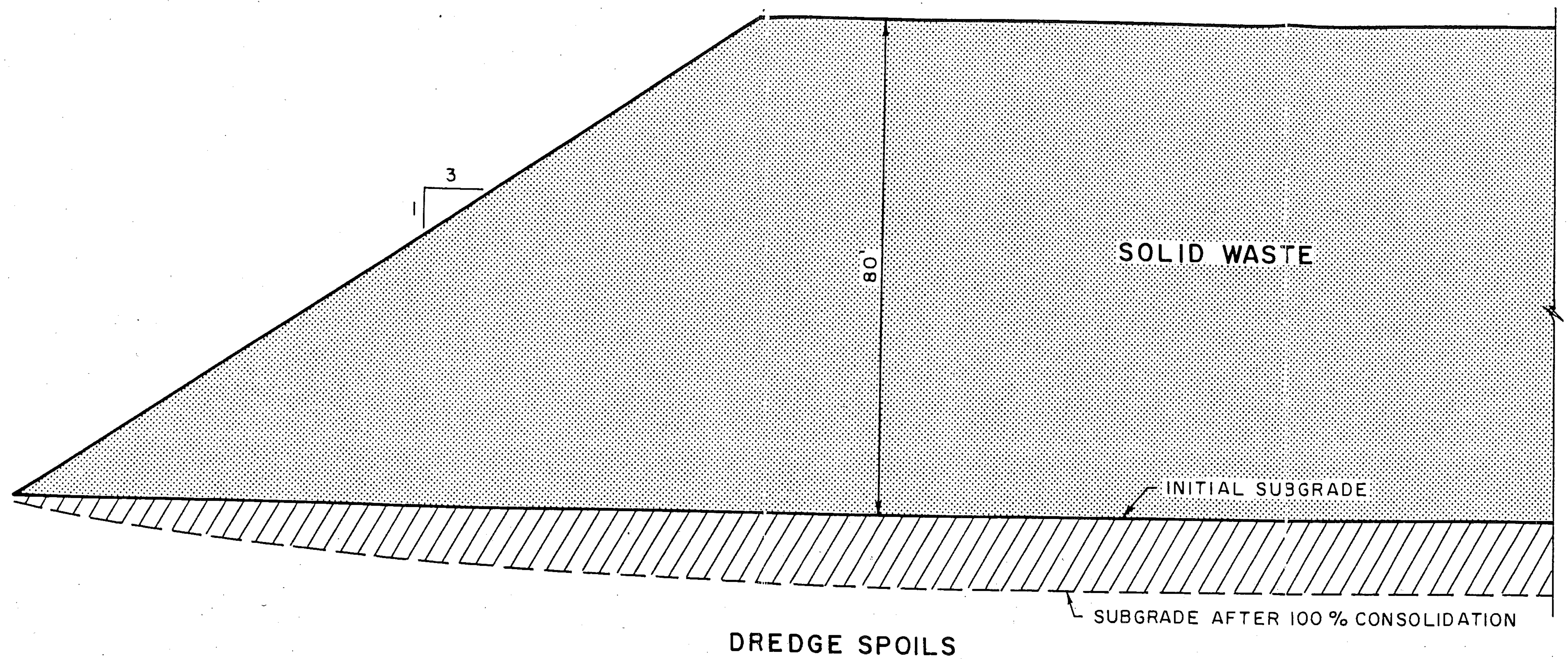
STABILITY ANALYSIS
SOUTHERN CROSS SECTION
INITIAL PROPERTIES

LAYER	SOIL TYPE	Ø	C (psf)	γ (pcf)
2	DREGDE SPOIL	0	577	103
3	DREGDE SPOIL	0	395	100
5	DREGDE SPOIL	0	358	97
6	SAND	32°	0	120
7	SOLID WASTE	10/0	200/750	43
8	DREGDE SPOIL	0	188	93
9	DREGDE SPOIL	0	225	93
10	DREGDE SPOIL	0	407	100
11	SAND & GRAVEL	38°	0	130

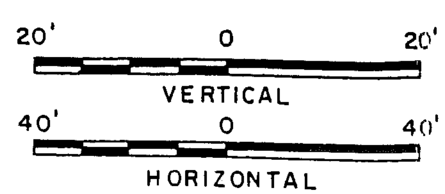


SCALE: 1" = 100'

FIGURE 2.14
GANNETT FLEMING, INC.
MARCH 1990



SCALE



PHASE III
NORTHERN SOLID WASTE FACILITY – 2
DELAWARE SOLID WASTE AUTHORITY

SETTLEMENT PROFILE

GANNETT FLEMING, INC.
HARRISBURG, PENNSYLVANIA

MARCH 1990

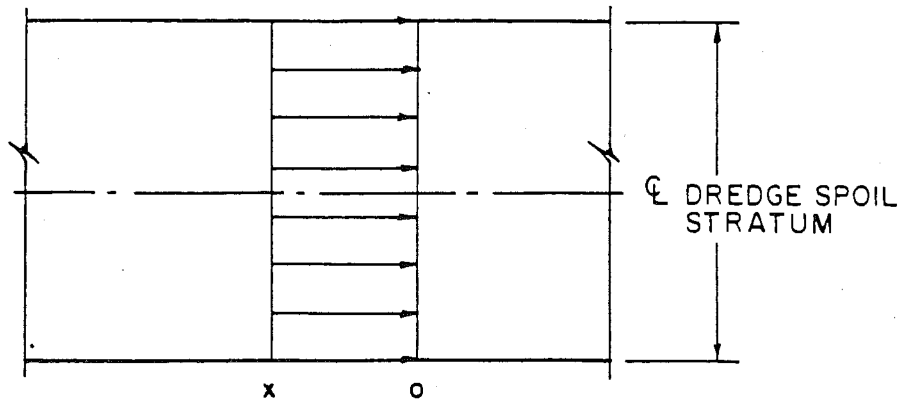


FIGURE A: INITIAL EXCESS PORE WATER PRESSURE DISTRIBUTION IMMEDIATELY AFTER PLACEMENT OF LOAD.

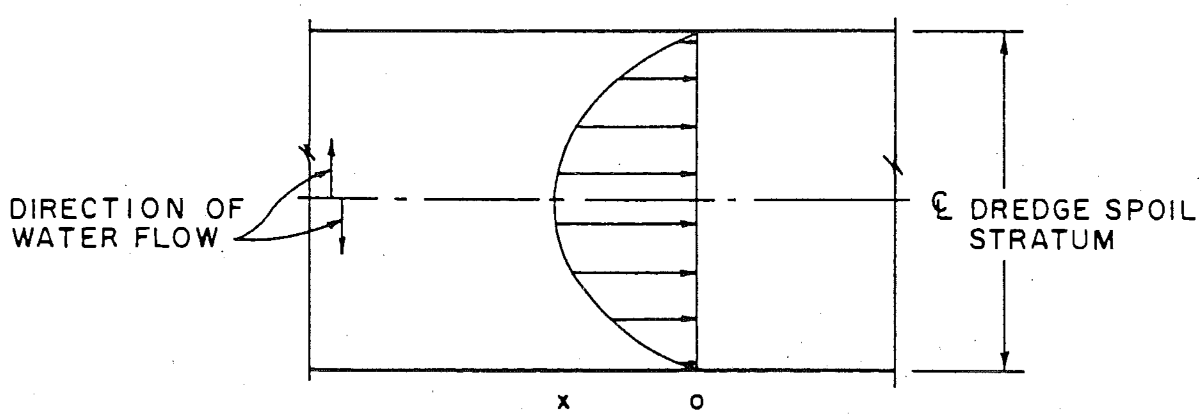
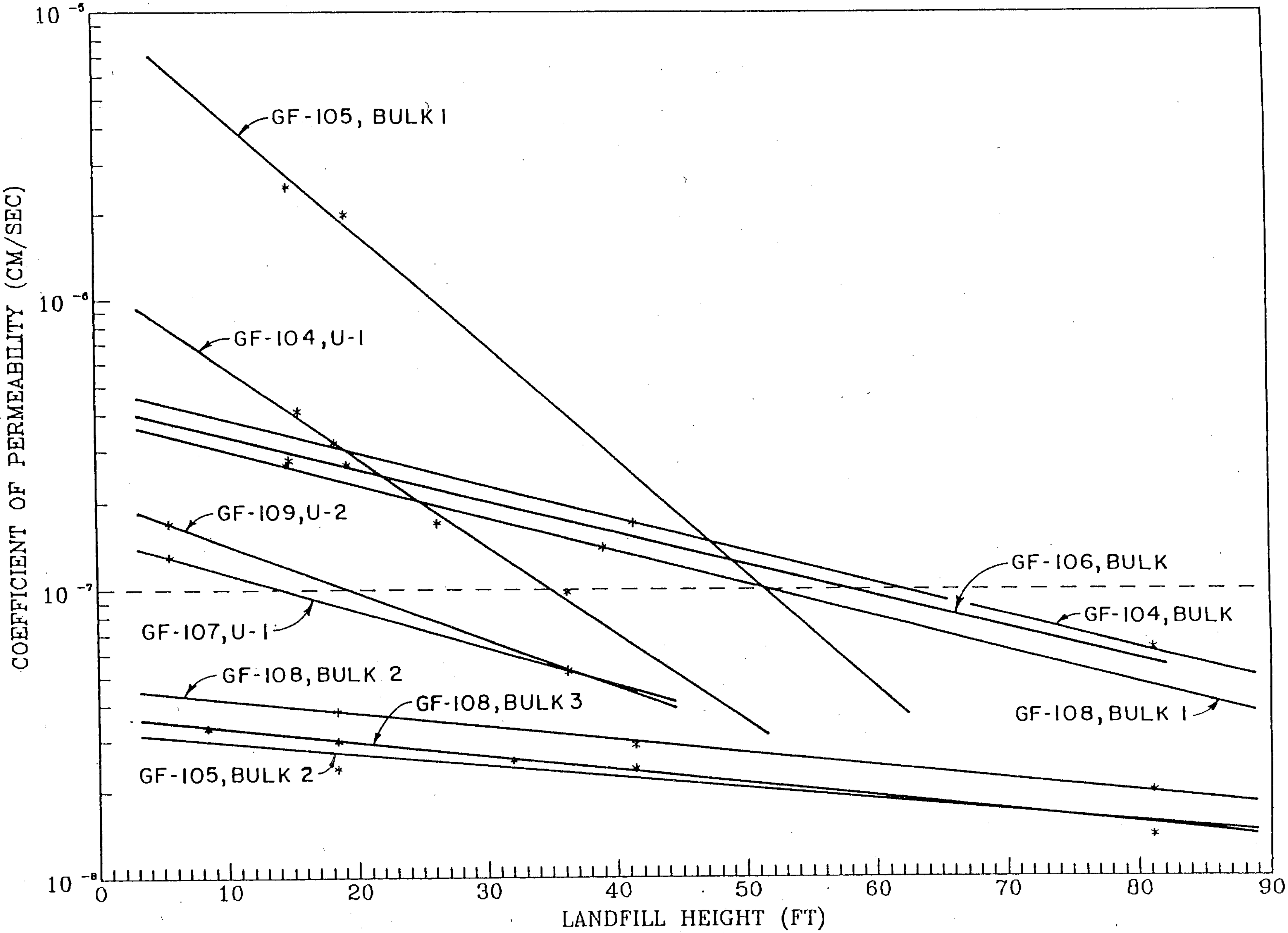


FIGURE B: EXCESS PORE WATER PRESSURE AFTER SOME TIME, t, FOR A DOUBLY DRAINED STRATUM.

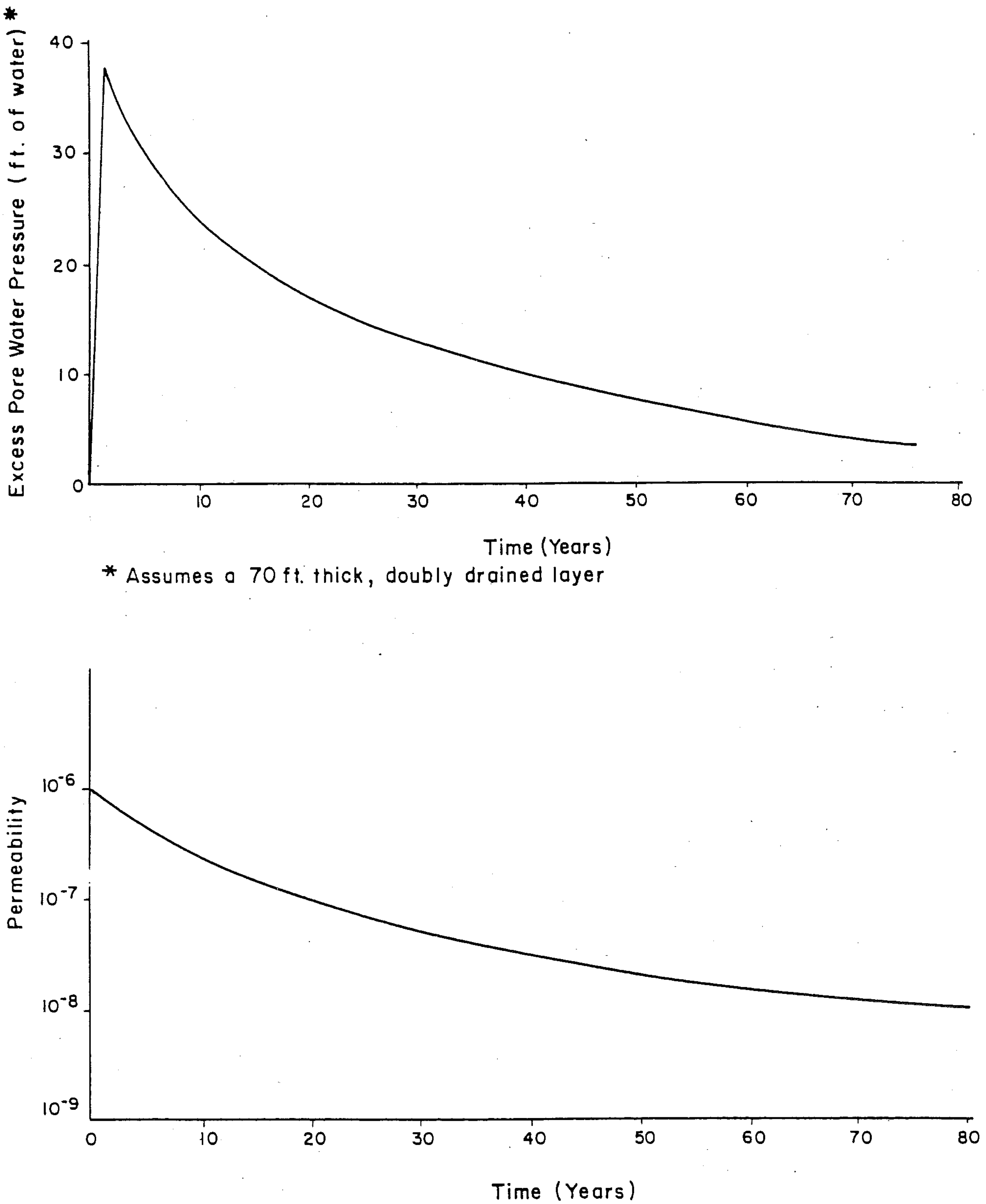
EXCESS PORE WATER PRESSURE DISTRIBUTIONS

FIGURE 3.01

PERMEABILITY VS. LANDFILL HEIGHT



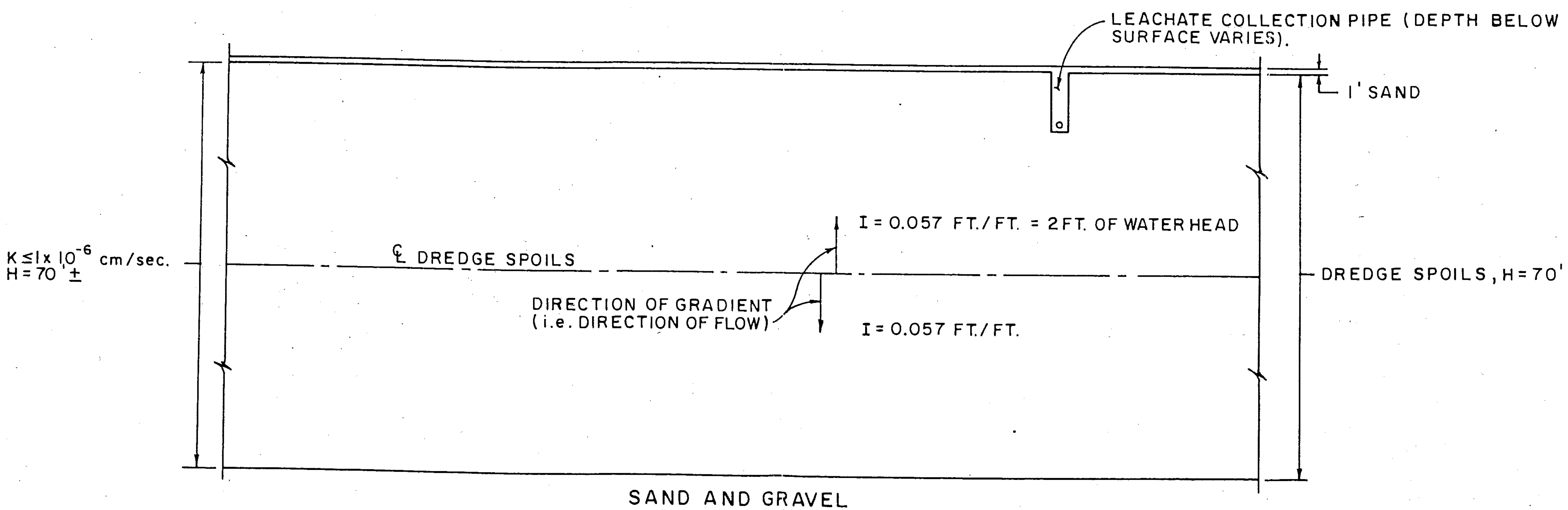
PERM VS CONSOLIDATION PRESSURE



UPWARD GRADIENT AND PERMEABILITY VS. TIME

Figure 3.03

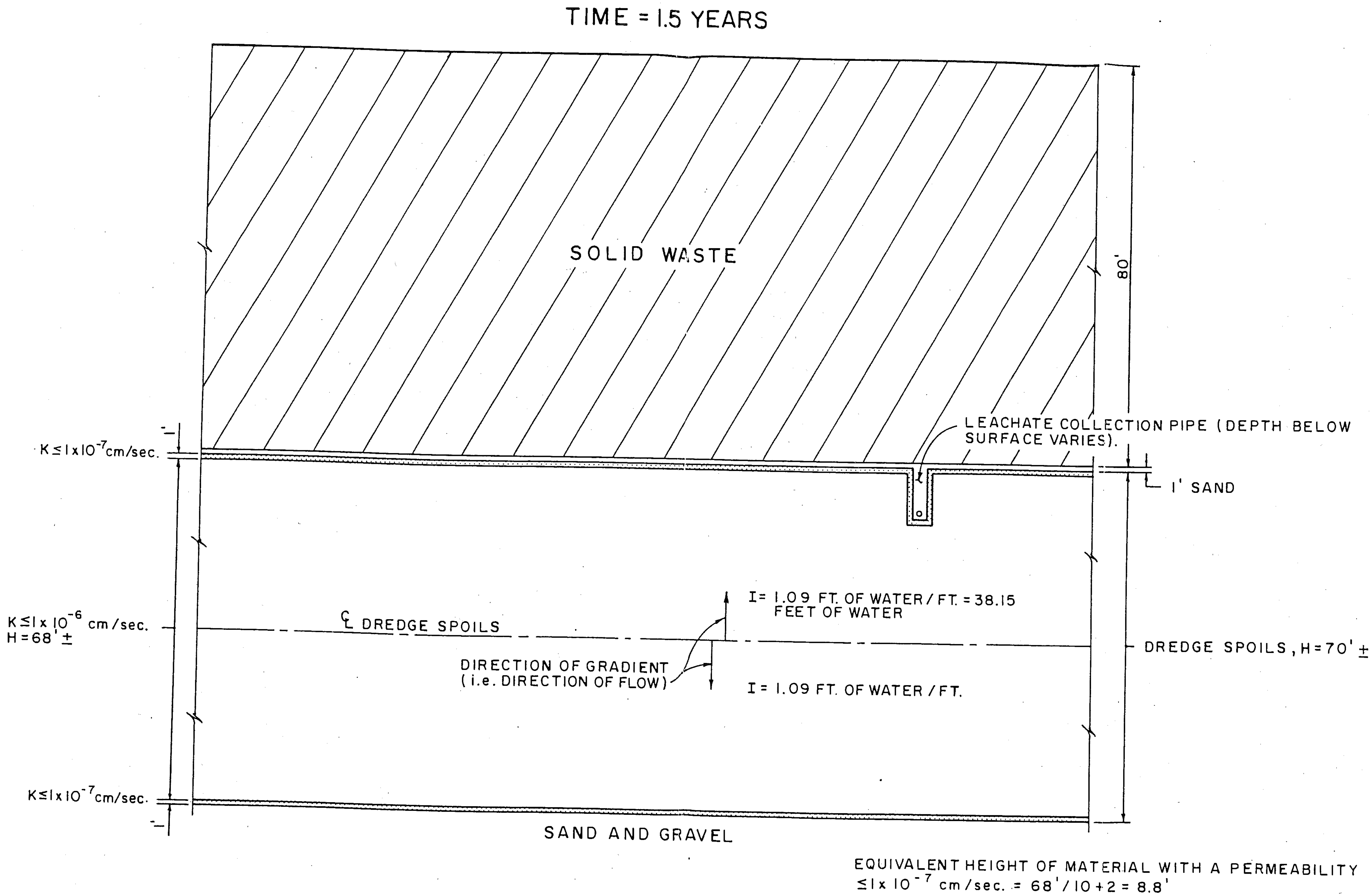
TIME = 0 YEARS



EQUIVALENT HEIGHT OF MATERIAL WITH A PERMEABILITY ≤ 1 x 10^{-7} cm/sec. = 70'/10 = 7.0'

CHANGE IN PERMEABILITY OF DREDGE SPOIL STRATUM

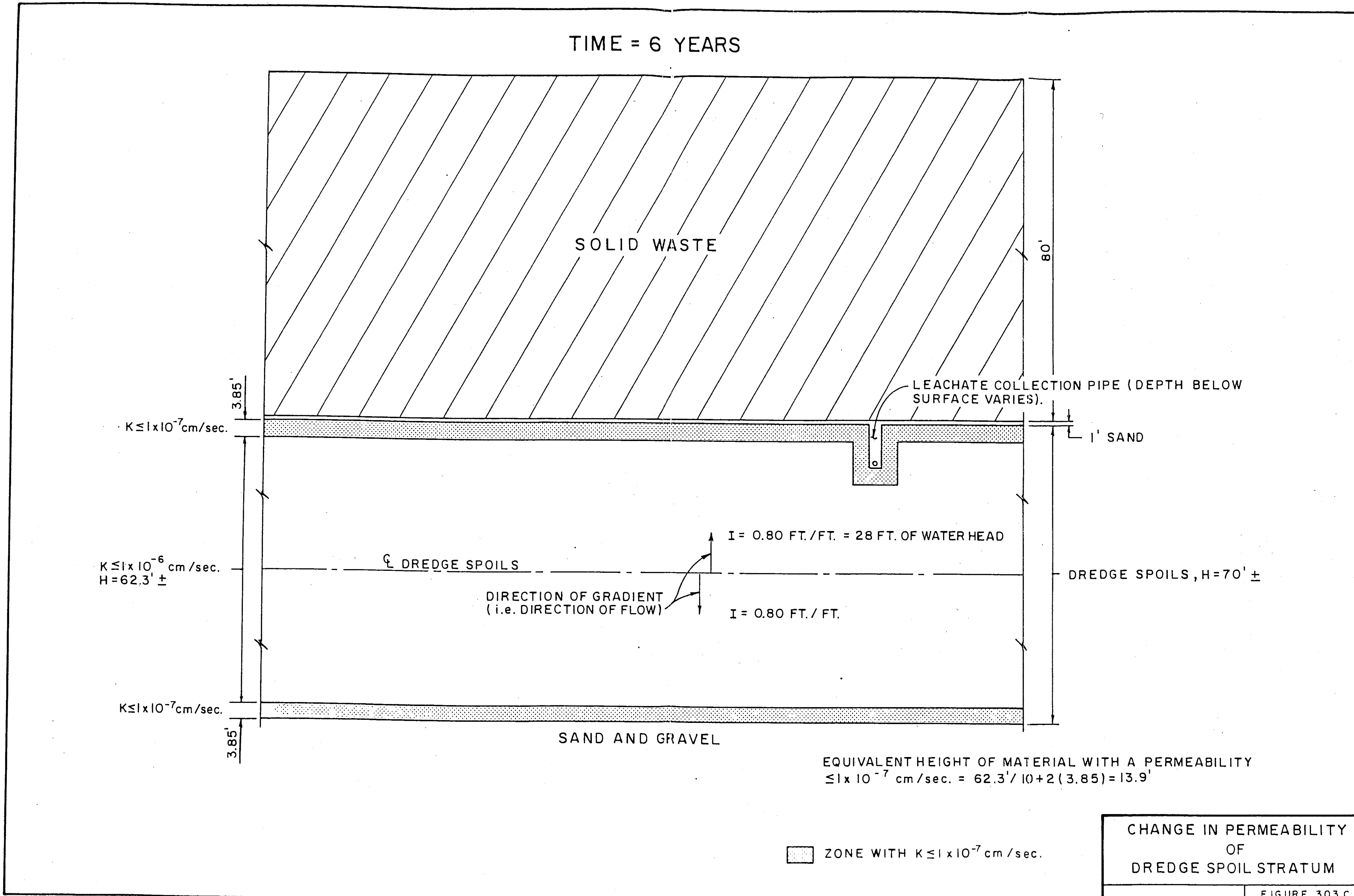
FIGURE 3.03 A



ZONE WITH $K \leq 1 \times 10^{-7}$ cm/sec.

CHANGE IN PERMEABILITY OF DREDGE SPOIL STRATUM

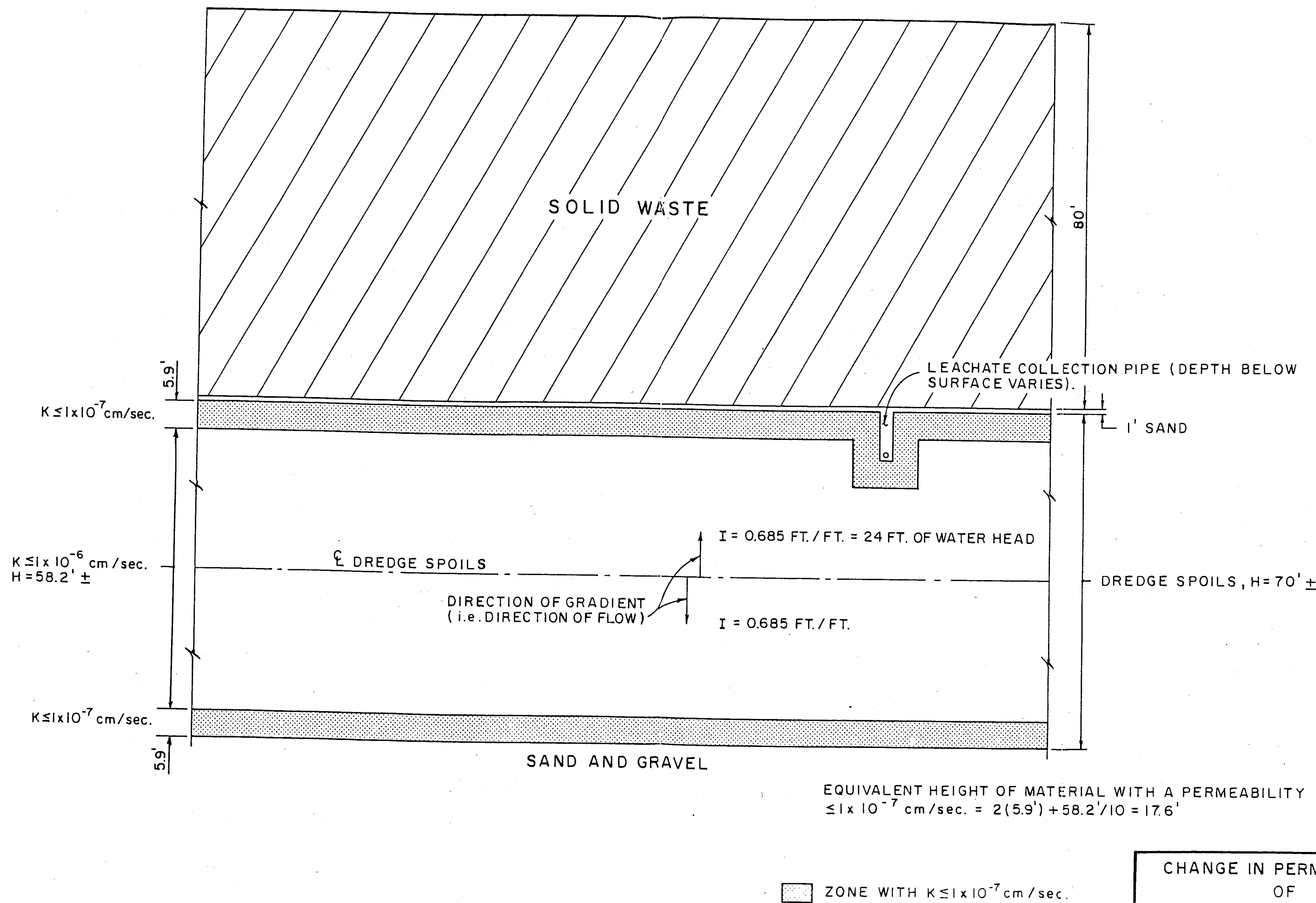
FIGURE 303 B



CHANGE IN PERMEABILITY OF DREDGE SPOIL STRATUM

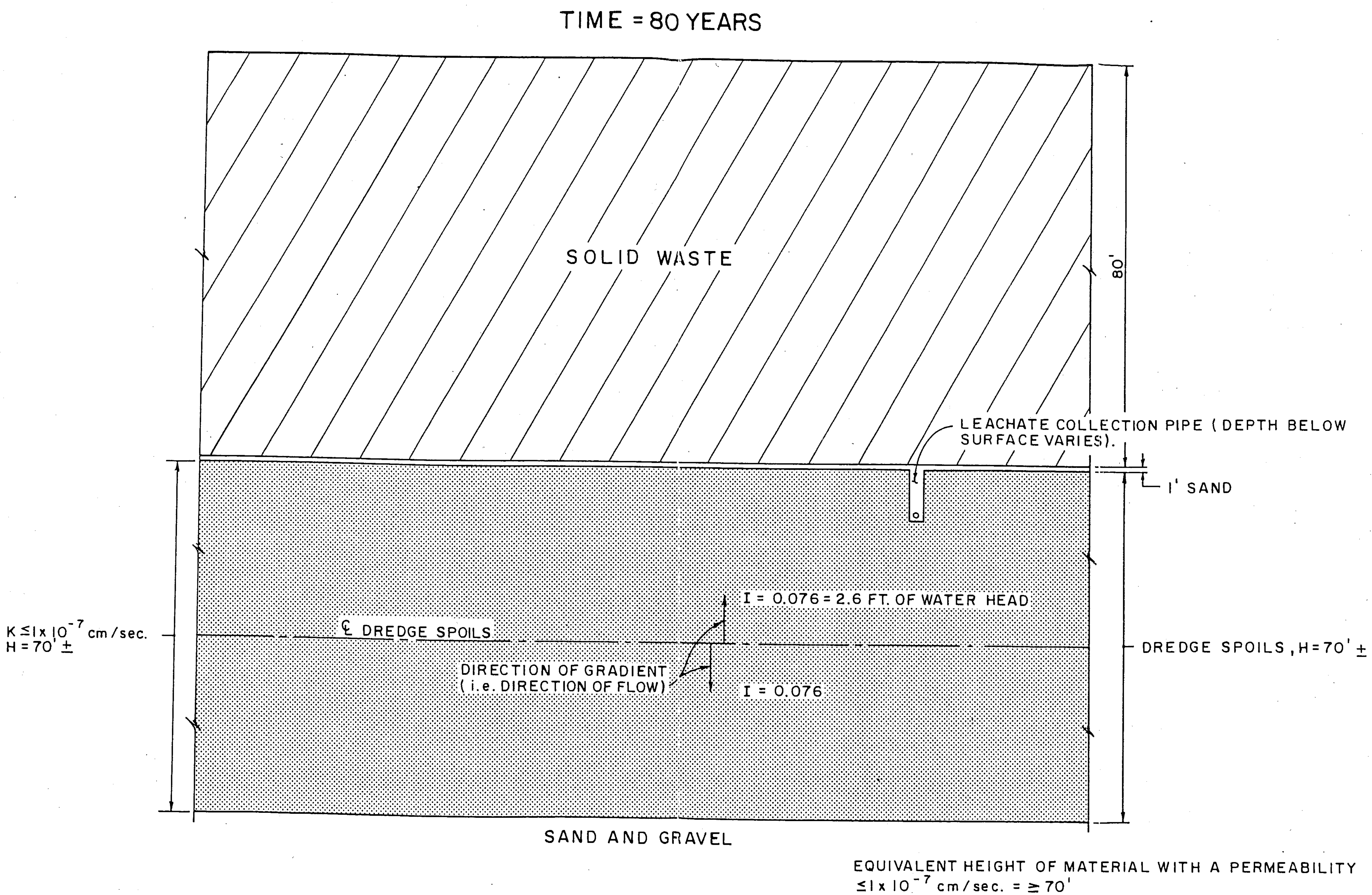
FIGURE 303 C

TIME = 11 YEARS



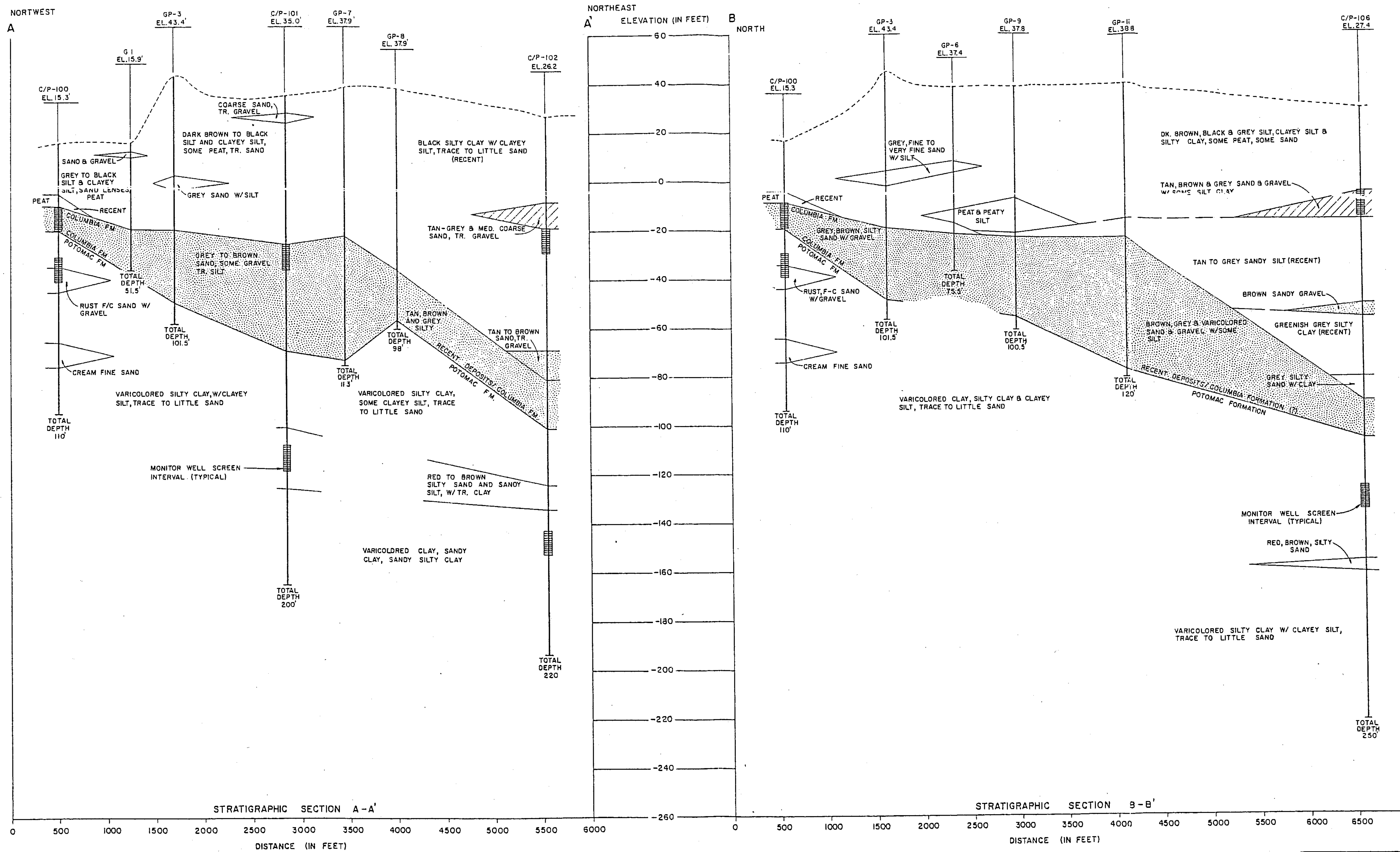
CHANGE IN PERMEABILITY
OF
DREDGE SPOIL STRATUM

FIGURE 3.03 D



CHANGE IN PERMEABILITY OF DREDGE SPOIL STRATUM

FIGURE 303 E



KEY

- UPPER SAND ZONE
- LOWER SAND ZONE

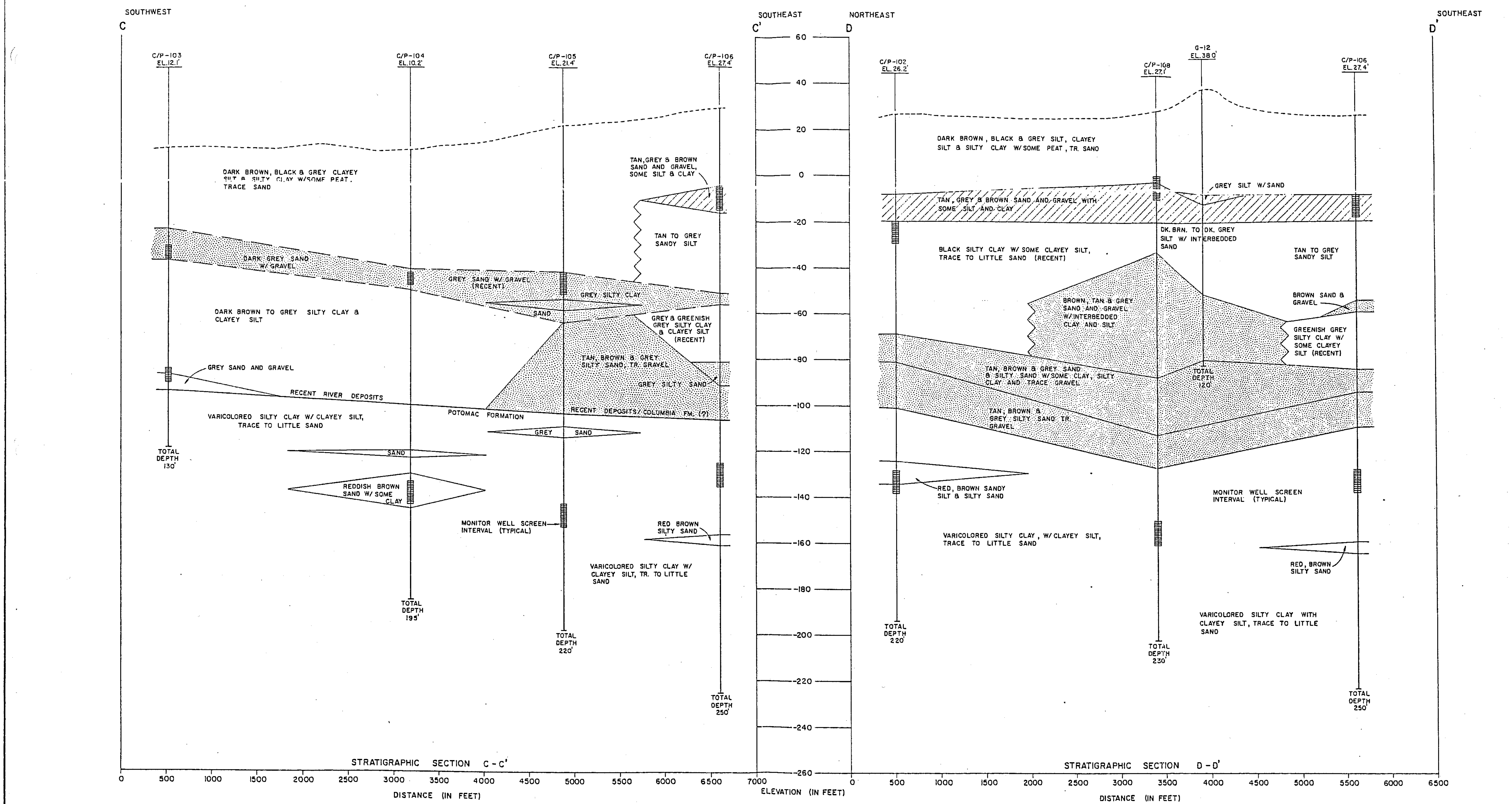
NOTES:

1. SEE FIGURE 3.5 FOR CROSS-SECTION LOCATIONS.
2. THESE CROSS-SECTIONS ARE MODIFIED FROM DUFFIELD ASSOC. OCT. 1985.
3. STRATIGRAPHIC SECTIONS ARE BASED ON DRILLER'S DESCRIPTIVE LOGS FOR CONDITIONS ENCOUNTERED BY THE TEST BORINGS AND WELL BORINGS, AND STRAIGHT LINE INTERPOLATION OF CONDITIONS BETWEEN BORINGS. ACTUAL CONDITIONS BETWEEN BORINGS ARE UNKNOWN.
4. STRATA TEXTURAL DESCRIPTIONS ARE A GENERALIZATION OF INDIVIDUAL SAMPLE DESCRIPTIONS INDICATED ON THE DRILLER'S DESCRIPTIVE LOGS. FOR LOGS SEE "SITE SUITABILITY REPORT, HYDROGEOLOGIC CONDITIONS AND GEOTECHNICAL EVALUATION OF THE CHERRY ISLAND SITE" PREPARED BY TERRAQUA RESOURCES CORP. DATED 1 FEB 1984
5. THIS INFORMATION TAKEN FROM 1986 PHASE II LANDFILL DESIGN REPORT PREPARED BY GANNETT FLEMING.
6. ELEVATIONS SHOWN ARE AS OF SEPTEMBER, 1983 AND MAY NOT REPRESENT PRESENT CONDITIONS AT THE SITE

GANNETT FLEMING, INC.
HARRISBURG, PENNSLVANIA FEB. 1990

PHASE III
NORTHERN SOLID WASTE FACILITY - 2
DELAWARE SOLID WASTE AUTHORITY

FIGURE 3.1
STRATIGRAPHIC SECTIONS
A-A' AND B-B'



KEY

☐ UPPER SAND ZONE

☐ LOWER SAND ZONE

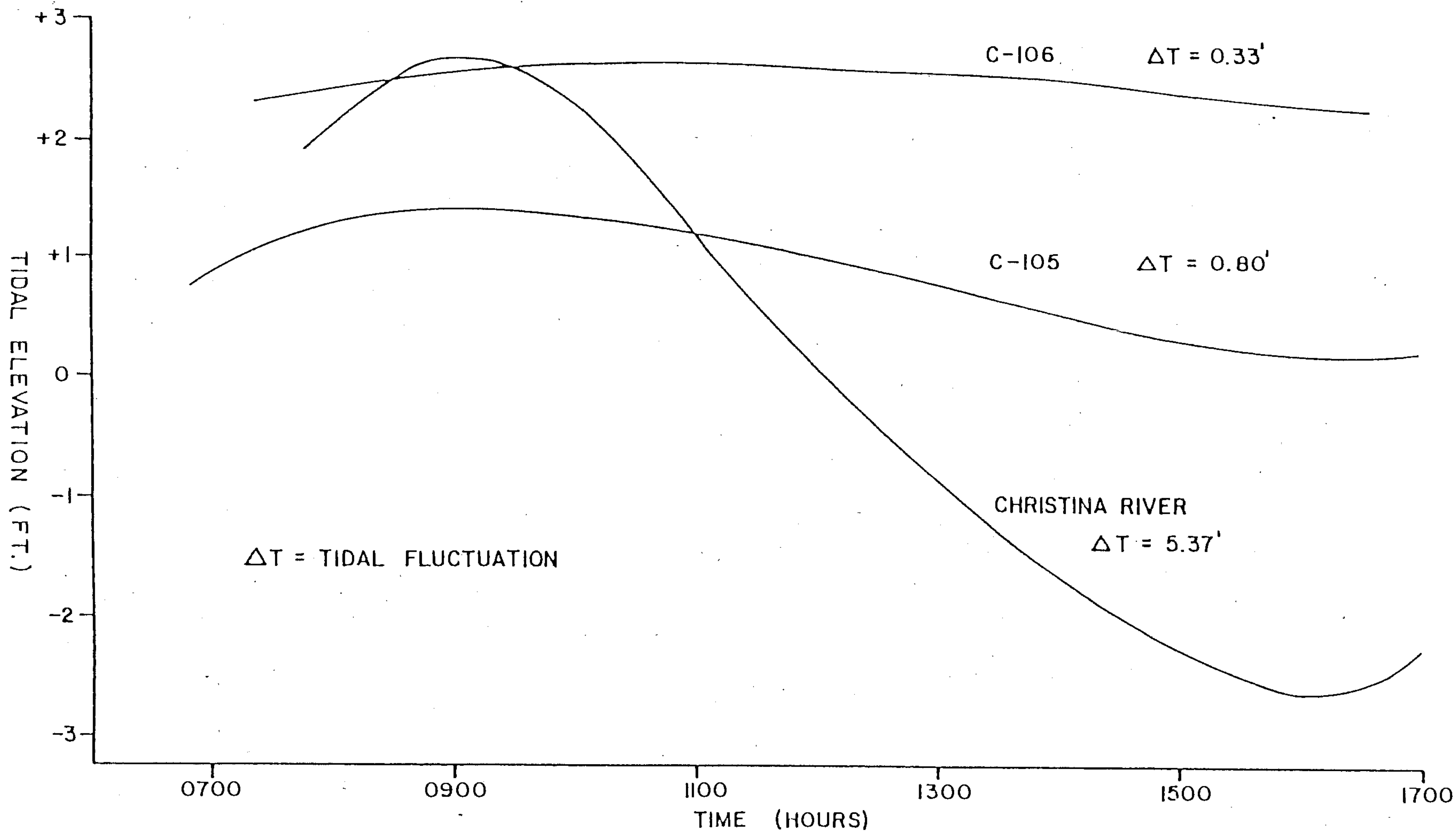
NOTES:

1. SEE FIGURE 3.5 FOR CROSS-SECTION LOCATIONS.
2. THESE CROSS-SECTIONS ARE MODIFIED FROM DUFFIELD ASSOC. OCT. 1985.
3. STRATIGRAPHIC SECTIONS ARE BASED ON DRILLER'S DESCRIPTIVE LOGS FOR CONDITIONS ENCOUNTERED BY THE TEST BORINGS AND WELL BORINGS, AND STRAIGHT LINE INTERPOLATION OF CONDITIONS BETWEEN BORINGS. ACTUAL CONDITIONS BETWEEN BORINGS ARE UNKNOWN.
4. STRATA TEXTURAL DESCRIPTIONS ARE A GENERALIZATION OF INDIVIDUAL SAMPLE DESCRIPTIONS INDICATED ON THE DRILLER'S DESCRIPTIVE LOGS. FOR LOGS SEE "SITE SUITABILITY REPORT, HYDROGEOLOGIC CONDITIONS AND GEOTECHNICAL EVALUATION OF THE CHERRY ISLAND SITE", PREPARED BY TERRAQUA RESOURCES CORP., DATED 1 FEB. 1984.
5. THIS INFORMATION TAKEN FROM 1986 PHASE II LANDFILL DESIGN REPORT PREPARED BY GANNETT FLEMING.
6. ELEVATIONS SHOWN ARE AS OF SEPTEMBER, 1983 AND MAY NOT REPRESENT PRESENT CONDITIONS AT THE SITE.

GANNETT FLEMING, INC.
HARRISBURG, PENNSYLVANIA FEB. 1990

PHASE III
NORTHERN SOLID WASTE FACILITY
DELAWARE SOLID WASTE AUTHORI

FIGURE 3.2
STRATIGRAPHIC SECTIONS
C-C' AND D-D'



DELAWARE SOLID WASTE AUTHORITY
NORTHERN SOLID WASTE FACILITY – 2
PHASE II AREA
LANDFILL DESIGN REPORT
DECEMBER 1986

FIGURE 3.3
GROUNDWATER RESPONSE TO
TO TIDAL FLUCTUATION
DECEMBER 30, 1983

POTENTIOMETRIC ELEVATION VERSUS ELAPSED TIME
COLUMBIA FORMATION GROUND WATER LEVEL STUDY
DECEMBER 1989

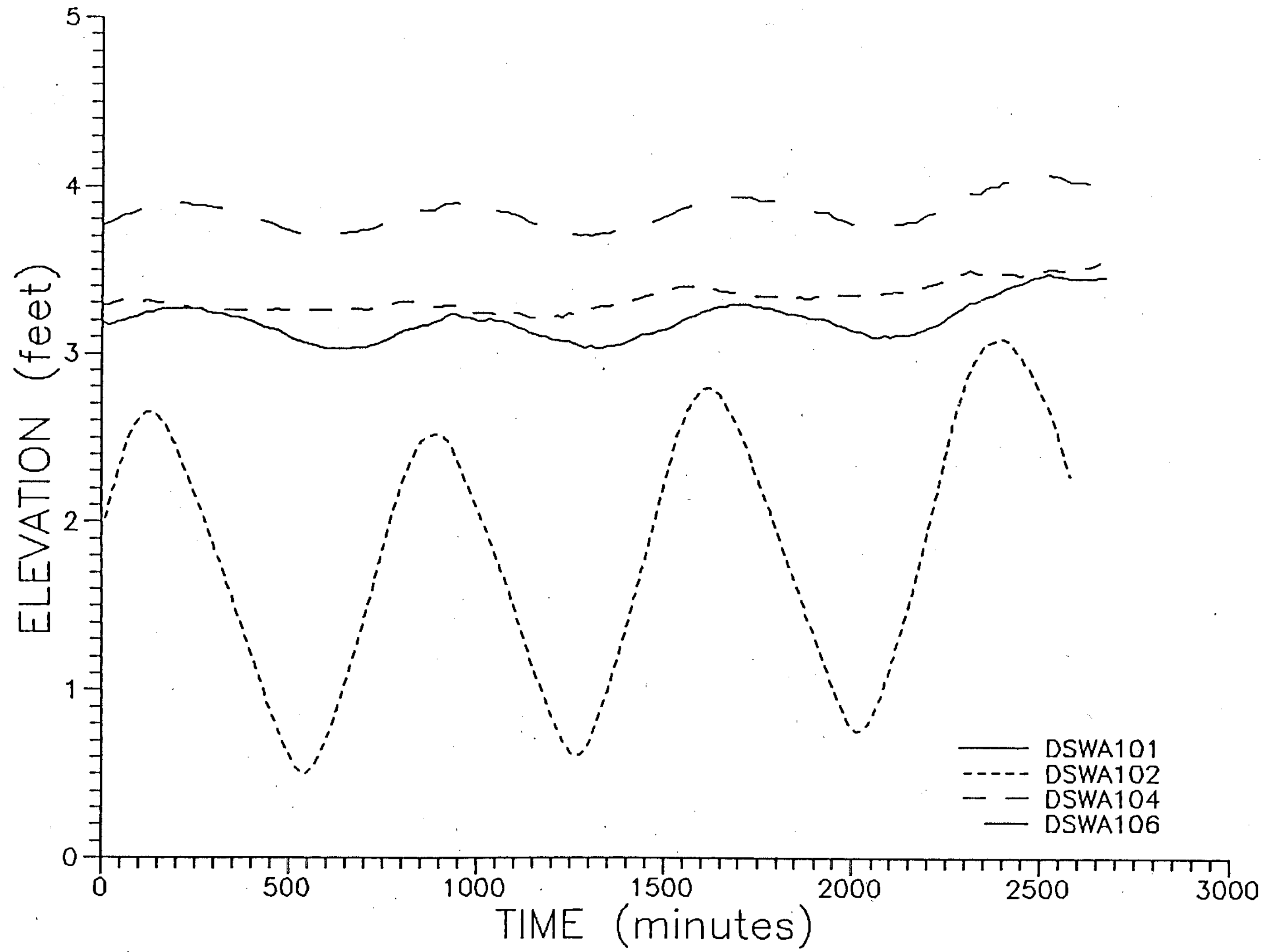
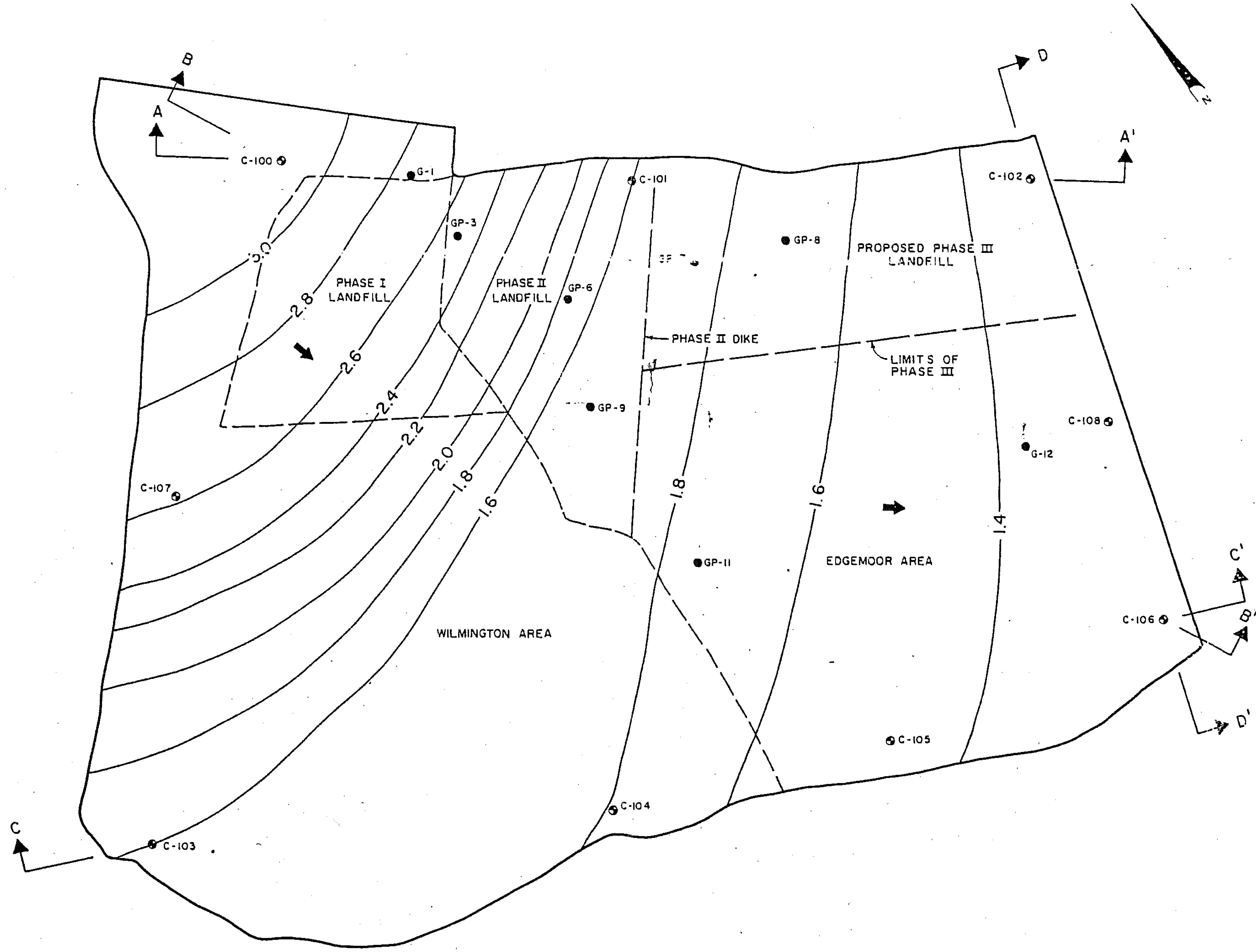


FIGURE 3.4



LEGEND

● G-1	GEOTECHNICAL BORING
● GP-3	GEOTECHNICAL BORING WITH DEEP PIEZOMETER
⊗ C-102	COLUMBIA FORMATION MONITORING WELL
A — A'	STRATIGRAPHIC CROSS-SECTION
(2.6)	GROUNDWATER ELEVATION
—1.6—	POTENTIOMETRIC SURFACE CONTOUR (EL. IN FEET)
	GROUNDWATER FLOW DIRECTION

400 0 400 800
SCALE IN FEET

NOTE: - THIS INFORMATION TAKEN FROM COLUMBIA FM. 1986 PHASE II DESIGN REPORT, PREPARED BY GANNETT FLEMING, INC.

PHASE III
NORTHERN SOLID WASTE FACILITY - 2
DELAWARE SOLID WASTE AUTHORITY

POTENTIOMETRIC SURFACE MAP
COLUMBIA / RECENT SEDIMENTS

GANNETT FLEMING, INC.
HARRISBURG, PENNSYLVANIA FEB. 1990 FIGURE 3.5

APPENDIX A

Driller's Boring Logs

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BORING LOG (1 of 2)

PROJECT Cherry Island - Phase I

BORING No. GF-101

PROJECT No. 89-275

LOCATION OF BORING

ELEV. DATE: START 10-12-89 FINISH 10-12-89 INSPECTOR

HAMMER 140 lbs. HAMMER DROP 30 in. SPOON OD 2 in. FOREMAN Mike Kalandros

BORING METHOD HSA ROCK CORE DIA MISC.

ELEV.	SOIL DESCRIPTION	DEPTH	BLOWS 6"	No.	TYPE	REC	REMARKS
	Brown silt with mica		2-2-8-11	1	DS		Water encountered at 45.0 ft.
			5-7-13-14	2	DS		
	Grey silty clay	5	4-5-11-11	3	DS		
			8-11-10-14	4	DS		
			4-4-3-4	5	DS		
		10					
		15	2-2-3	6	DS		
		20	WOH-2-1	7	DS		
		25	WOH-2-1	8	DS		
		30	WOH-1-2	9	DS		
		35	1-1-3	10	DS		
		40	WOH-WOH-3	11	DS		

LEGEND
DS DRIVEN SPOON
ST SHELBY TUBE
PS PISTON SAMPLE
RC ROCK CORE

GROUND WATER
AT COMPLETION 45.0' CAVED 18.0'
AT HRS. CAVED

HSA HOLLOW STEM AUGER
DC DRIVEN CASING
MD MUD DRILLING

National Foundation Engineering, Inc. NFE

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BORING LOG

PROJECT Cherry Island - Phase I

BORING No. GF-101

PROJECT No. 89-275

LOCATION OF BORING

ELEV. DATE: START 10-12-89 FINISH 10-12-89 INSPECTOR

HAMMER 140 lbs. HAMMER DROP 30 in. SPOON OD 2 in. FOREMAN Mike Kalandros

BORING METHOD HSA ROCK CORE DIA MISC.

ELEV.	SOIL DESCRIPTION	DEPTH	BLOWS 6"	No.	TYPE	REC	REMARKS
	Grey silty clay						
		45	1-1-3	12	DS		
		50	1-1-1	13	DS		
		55	1-1-2	14	DS		
		60	2-3-6	15	DS		
		65	3-4-5	16	DS		
	Gray coarse silty sand with some gravel	70	2-2-2	17	DS		
		75	10-14-15	18	DS		
	Bottom of Boring at 75.0 feet.	80					

LEGEND
DS DRIVEN SPOON
ST SHELBY TUBE
PS PISTON SAMPLE

GROUND WATER
AT COMPLETION 45.0' CAVED 18.0'
AT HRS. CAVED

HSA HOLLOW STEM AUGER
DC DRIVEN CASING
MD MUD DRILLING

National Foundation Engineering, Inc. NFE

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BORING LOG

PROJECT Cherry Island - Phase I

BORING No. GF-102

PROJECT No. 89-275

LOCATION OF BORING

ELEV. DATE: START 10-10-89 FINISH 10-10-89 INSPECTOR

HAMMER 140 lbs. HAMMER DROP 30 in. SPOON OD 2 in. FOREMAN Mike/Taylor

BORING METHOD HSA ROCK CORE DIA MISC.

ELEV.	SOIL DESCRIPTION	DEPTH	BLOWS 6"	No.	TYPE	REC	REMARKS
	Dark brown silty sandy clay		1-3-6-8	1	DS		
	Gray silty clay		10-13-16-18	2	DS		
		5		3	PT		
	Black silty clay		2-2-2-3	4	DS		
		10	4-2-1-1	5	DS		
		15	WOH-1-1	6	DS		
		20	WOH-WOH-1	7	DS		
		25	2-1-1	8	DS		
		30	WOH-WOH-1	9	DS		
		35	WOH-WOH-WOH	10	DS		
				11	ST		
	Black silty clay	40	WOH-2-2	12	DS		

LEGEND
DS DRIVEN SPOON
ST SHELBY TUBE
PS PISTON SAMPLE
RC ROCK CORE

GROUND WATER
AT COMPLETION ______ CAVED ______
AT ______ HRS. ______ CAVED ______

HSA HOLLOW STEM AUGER
DC DRIVEN CASING
MD MUD DRILLING

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BORING LOG

PROJECT Cherry Island - Phase I

BORING No. GF-102

PROJECT No. 89-275

LOCATION OF BORING

ELEV. DATE: START 10-10-89 FINISH 10-10-89 INSPECTOR

HAMMER 140 lbs. HAMMER DROP 30 in. SPOON OD 2 in. FOREMAN Mike/Taylor

BORING METHOD HSA ROCK CORE DIA MISC.

ELEV.	SOIL DESCRIPTION	DEPTH	BLOWS 6"	No.	TYPE	REC	REMARKS
	Black silty clay	45	2-2-2	13	DS		
	Grey-black silty sandy clay	50	2-2-3	14	DS		
		55	3-3-4	15	DS		
		60	4-3-5	16	DS		
	Grey sandy silt with gravel		3-3-6	17	DS		
	Grey sandy silty clay	65					
		70	7-4-6	18	DS		
	Bottom of Boring at 70.5 feet.	75					
		80					

LEGEND
DS DRIVEN SPOON
ST SHELBY TUBE
PS PISTON SAMPLE

GROUND WATER
AT COMPLETION ____ CAVED ____
AT ____ HRS. ____ CAVED ____

HSA HOLLOW STEM AUGER
DC DRIVEN CASING
MD MUD DRILLING

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BORING LOG

PROJECT Cherry Island - Phase I

BORING No. GF-103

PROJECT No. 89-275

LOCATION OF BORING

ELEV. DATE: START 10-9-89 FINISH 10-9-89 INSPECTOR

HAMMER 140 lbs. HAMMER DROP 30 in. SPOON OD 2 in. FOREMAN Mike/Taylor

BORING METHOD HSA ROCK CORE DIA MISC.

ELEV.	SOIL DESCRIPTION	DEPTH	BLOWS 6"	No.	TYPE	REC	REMARKS
	Brown clayey silt		4-6-8-10	1	DS		Water encountered at 7.0 ft.
	Gray silt		9-12-11-9	2	DS		
	Moist gray silt	5	1-1-1-2	3	DS		
			3-3-4-9	4	DS		
	Gray silt	10	9-11-13-12	5	DS		
		15	WOH-1-1	6	DS		
				7	ST		
	Gray clayey silt	20	2-2-2	8	DS		
		25	3-3-4	9	DS		
		30	2-2-2	10	DS		
		35	2-2-3	11	DS		
		40	2-3-3	12	DS		

LEGEND
DS DRIVEN SPOON
ST SHELBY TUBE
PS PISTON SAMPLE
RC ROCK CORE

GROUND WATER
AT COMPLETION 7.0' CAVED 7.7'
AT HRS. CAVED

HSA HOLLOW STEM AUGER
DC DRIVEN CASING
MD MUD DRILLING

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BORING LOG

PROJECT Cherry Island - Phase I

BORING No. GF-103

PROJECT No. 89-275

LOCATION OF BORING

ELEV. DATE: START 10-9-89 FINISH 10-9-89 INSPECTOR

HAMMER 140 lbs. HAMMER DROP 30 in. SPOON OD 2 in. FOREMAN Mike/Taylor

BORING METHOD HSA ROCK CORE DIA MISC.

ELEV.	SOIL DESCRIPTION	DEPTH	BLOWS 6".	No.	TYPE	REC	REMARKS
				13	ST		Only 3" recovery tube sample discarded
	Gray silt	45	2-2-3	14	DS		
				15	ST		
	Gray silt	50	3-3-4	16	DS		
	Bottom of Boring at 50.0 feet.	55					
		60					
		65					
		70					
		75					
		80					

LEGEND
DS DRIVEN SPOON
ST SHELBY TUBE
PS PISTON SAMPLE

GROUND WATER
AT COMPLETION 7.0' CAVED 7.7'
AT HRS. CAVED

HSA HOLLOW STEM AUGER
DC DRIVEN CASING
MD MUD DRILLING

National Foundation Engineering, Inc. NFE

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BORING LOG

PROJECT Cherry Island - Phase II BORING No. GF-104

Gannett Fleming PROJECT No. 89-2350

LOCATION OF BORING

ELEV. DATE: START 11-8-89 FINISH 11-9-89 INSPECTOR

HAMMER 140 lb. HAMMER DROP 30 in. SPOON OD 2 in. FOREMAN M. Kalandro

BORING METHOD Tripod ROCK CORE DIA MISC.

ELEV.	SOIL DESCRIPTION	DEPTH	BLOWS 6"	No.	TYPE	REC	REMARKS
	Dredge spoils, wet (grey black clayey silt)		WOH	1			
			WOH	2			
		5	WOH	3			
			WOH	4			
		10	WOH-1-1-3	5			
		15	2-2-1	6			
		20	1-1-WOH	7			
		25	WOH-1-2	8			
		30	WOH-WOH-WOH	9			
		35	WOH.	10			
	Bottom of hole 35.0'						
		40					

LEGEND
DS DRIVEN SPOON
ST SHELBY TUBE
PS PISTON SAMPLE

GROUND WATER
AT COMPLETION CAVED
AT HRS CAVED

HSA HOLLOW STEM AUGER
DC DRIVEN CASING
MD MUD DRILLING

National Foundation Engineering, Inc. NFE

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BORING LOG

PROJECT Cherry Island - Phase II
Gannett Fleming

BORING No. GF-104A
PROJECT No. 89-2350

LOCATION OF BORING

ELEV. DATE: START 11-9-89 FINISH 11-9-89 INSPECTOR

HAMMER 140 lb. HAMMER DROP 30 in. SPOON OD 2 in. FOREMAN M. Kalandros

BORING METHOD Tripod ROCK CORE DIA MISC.

ELEV.	SOIL DESCRIPTION	DEPTH	BLOWS 6"	No.	TYPE	REC	REMARKS
		5					
			SHELBY TUBE	U-1		13"/ 24"	
		10					
		15					
			SHELBY TUBE	U-2		100%	
		20					
			SHELBY TUBE	U-3		18"/ 24"	
		25					
		30					
		35	SHELBY TUBE	U-4		100%	
		40					

LEGEND
DS DRIVEN SPOON
ST SHELBY TUBE

GROUND WATER
AT COMPLETION ________ CAVED ________

HSA HOLLOW STEM AUGER
DC DRIVEN CASING

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BORING LOG

PROJECT Cherry Island - Phase II
Gannett Fleming
BORING No. GF-105
PROJECT No. 89-2350
LOCATION OF BORING
ELEV. DATE: START ~~11-15-89~~ FINISH ~~11-15-89~~ INSPECTOR
HAMMER 140 lb. HAMMER DROP 30 in. SPOON OD 2 in. FOREMAN M. Kalandro
BORING METHOD Tripod ROCK CORE DIA MISC.

ELEV.	SOIL DESCRIPTION	DEPTH	BLOWS 6"	No.	TYPE	REC	REMARKS
	Dredge spoils, wet (grey black clayey silt)		WOR-WOR	1			
			WOH-WOH	2			
		5	SHELBY TUBE	U-1		100%	
			WOR-WOH	4			
		10	WOH-WOH	5			
		15	WOH-1-1	6			
		20	SHELBY TUBE	U-2		100%	
		25	1/18"	8			
		30	WOH-WOH	9			
				10			
	Bottom of hole 35.0'	35					
		40					

LEGEND
DS DRIVEN SPOON
ST SHELBY TUBE
PS PISTON SAMPLE

GROUND WATER
AT COMPLETION ______ CAVED ______
AT ______ HRS. ______ CAVED ______

HSA HOLLOW STEM AUGER
DC DRIVEN CASING
MD MUD DRILLING

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BORING LOG

PROJECT Cherry Island - Phase II
Gannett Fleming

BORING No. GF-106
PROJECT No. 89-2350

LOCATION OF BORING

ELEV. DATE: START 12-6-89 FINISH 12-6-89 INSPECTOR

HAMMER 140 lb. HAMMER DROP 30 in. SPOON OD 2 in. FOREMAN M. Kalandros

BORING METHOD Tripod ROCK CORE DIA MISC.

ELEV.	SOIL DESCRIPTION	DEPTH	BLOWS 6"	No.	TYPE	REC	REMARKS
	Dark brown silty clay		WOH	1		1.5	
			WOH	2		0.5	
		5	1-1-2-2	3		1.0	
			WOH/2	4		1.5	
	Dark brown clay silt	10	SHELBY TUBE	5	PT	2.0	
	Dark brown clay organic silt	15	WOH	6		1.5	
		20	WOH	7	PT	2.0	
	Brown to black organic silt	25	WOH	8		1.5	
	Moist silty clay w/ tr. of organic material	30	WOH	9		1.5	
		35	WOH	10		1.5	
		40	WOH	11		1.5	

LEGEND
DS DRIVEN SPOON
ST SHELBY TUBE

GROUND WATER
AT COMPLETION ______ CAVED ______

HSA HOLLOW STEM AUGER
DC DRIVEN CASING

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BORING LOG

PROJECT Cherry Island - Phase II
Gannett Fleming

BORING No. GF-106 pg 2 o:
PROJECT No. 89-2350

LOCATION OF BORING

ELEV. DATE: START 12-6-89 FINISH 12-6-89 INSPECTOR

HAMMER 140 lb. HAMMER DROP 30 in. SPOON OD 2 in. FOREMAN M. Kalandros

BORING METHOD Tripod ROCK CORE DIA MISC.

ELEV.	SOIL DESCRIPTION	DEPTH	BLOWS 6"	No.	TYPE	REC	REMARKS
	Moist silty clay w/ tr. of organic material						
		45	1/WOH	12		1.5	
		50	1-2-3	13			
		55	1-1-2	14			
	Bottom of hole 55.0'						
		60					
		65					
		70					
		75					
		80					

LEGEND
DS DRIVEN SPOON
ST SHELBY TUBE

GROUND WATER
AT COMPLETION CAVED
AT HRS CAVED

HSA HOLLOW STEM AUGER
DC DRIVEN CASING
MD MUD DRILLING

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BORING LOG

PROJECT Cherry Island - Phase I

BORING No. GF-107

PROJECT No. 89-275

LOCATION OF BORING

ELEV. DATE: START 10-11-89 FINISH 10-11-89 INSPECTOR

HAMMER 140 lbs. HAMMER DROP 30 in. SPOON OD 2 in. FOREMAN Taylor

BORING METHOD HSA ROCK CORE DIA MISC.

ELEV.	SOIL DESCRIPTION	DEPTH	BLOWS 6"	No.	TYPE	REC	REMARKS
	Tan sandy silt		3-3-9-16	1	DS		
	Grey silty clay		8-12-9-13	2	DS		
		5	2-3-3-4	3	DS		
			4-3-3-7	4	DS		
		10	3-5-7-8	5	DS		
		15	2-2-2	6	DS		
		20	1-1-2	7	DS		
				8	ST		
	Grey silty clay	25	2-1-2	9	DS		
		30	WOH-1-2	10	DS		
		35	WOH-2-2	11	DS		
		40	1-2-2	12	DS		

LEGEND
DS DRIVEN SPOON
ST SHELBY TUBE
PS PISTON SAMPLE

GROUND WATER
AT COMPLETION ______ CAVED ______
AT ______ HRS. ______ CAVED ______

HSA HOLLOW STEM AUGER
DC DRIVEN CASING
MD MUD DRILLING

National Foundation Engineering, Inc.

3401 CARLINS PARK DRIVE • BALTIMORE, MARYLAND 21215

BORING LOG

PROJECT Cherry Island - Phase I

BORING No. GF-107

PROJECT No. 89-275

LOCATION OF BORING

ELEV. DATE: START 10-11-89 FINISH 10-11-89 INSPECTOR

HAMMER 140 lbs. HAMMER DROP 30 in. SPOON OD 2 in. FOREMAN Taylor

BORING METHOD HSA ROCK CORE DIA MISC.

ELEV.	SOIL DESCRIPTION	DEPTH	BLOWS 6"	No.	TYPE	REC	REMARKS
	Grey silty clay						
		45	2-2-3	13	DS		
		50	3-3-4	14	DS		
		55	3-4-3	15	DS		
				16	ST		
		60	5-6-6	17	DS		
	Bottom of Boring at 60.0 feet.	65					
		70					
		75					
		80					

LEGEND
DS DRIVEN SPOON
ST SHELBY TUBE
PS PISTON SAMPLE

GROUND WATER
AT COMPLETION ______ CAVED ______
AT ______ HRS. ______ CAVED ______

HSA HOLLOW STEM AUGER
DC DRIVEN CASING
MD MUD DRILLING

National Foundation Engineering, Inc. NFE

3401 CARLINS PARK DRIVE • BALTIMORE, MARYLAND 21215

BORING LOG

PROJECT Cherry Island - Phase II
Gannett Fleming

BORING No. GF-108
PROJECT No. 89-2350

LOCATION OF BORING

ELEV. DATE: START 12-27-89 FINISH 12-27-89 INSPECTOR

HAMMER 140 lb. HAMMER DROP 30 in. SPOON OD 2 in. FOREMAN M. Kalandro

BORING METHOD Tripod ROCK CORE DIA MISC.

ELEV.	SOIL DESCRIPTION	DEPTH	BLOWS 6"	No.	TYPE	REC	REMARKS
	Saturated black silt w/ tr. of clay		WOH	1			
			WOH	2		0.8	
		5	WOH	3		1.0	
			WOH	4		1.0	
	Black clay silt	10	SHELBY TUBE	5	PT	1.0	
		15	SHELBY TUBE	6	PT	2.0	
		20	WOH	7			
	Organic silty clay	25	WOH	8		1.3	
		30	SHELBY TUBE	9	PT	2.0	
		35	WOH	10		1.5	
		40	WOH	11		1.5	

LEGEND
DS DRIVEN SPOON
ST SHELBY TUBE

GROUND WATER
AT COMPLETION CAVED

HSA HOLLOW STEM AUGER
DC DRIVEN CASING

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BORING LOG

PROJECT Cherry Island - Phase II
Gannett Fleming

BORING No. GF-108 pg 2 o

PROJECT No. 89-2350

LOCATION OF BORING

ELEV.

DATE: START 12-27-89 FINISH 12-27-89 INSPECTOR

HAMMER 140 lb. HAMMER DROP 30 in. SPOON OD 2 in. FOREMAN M. Kalandros

BORING METHOD Tripod ROCK CORE DIA MISC.

ELEV.	SOIL DESCRIPTION	DEPTH	BLOWS 6"	No.	TYPE	REC	REMARKS
	Organic silty clay						
		45	WOH	12		1.5	
		50	SHELBY TUBE	13			
	Bottom of hole 51.0'						
		55					
		60					
		65					
		70					
		75					
		80					

LEGEND
DS DRIVEN SPOON
ST SHELBY TUBE
PS PISTON SAMPLE

GROUND WATER
AT COMPLETION CAVED
AT HRS CAVED

HSA HOLLOW STEM AUGER
DC DRIVEN CASING
MD MUD DRILLING

National Foundation Engineering, Inc. NFE
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BORING LOG

PROJECT Cherry Island - Phase II
Gannett Fleming

BORING No. GF-109
PROJECT No. 89-2350

LOCATION OF BORING

ELEV. DATE: START FINISH INSPECTOR

HAMMER 140 lb. HAMMER DROP 30 in. SPOON OD 2 in. FOREMAN M. Kalandros

BORING METHOD Tripod ROCK CORE DIA MISC.

ELEV.	SOIL DESCRIPTION	DEPTH	BLOWS 6"	No.	TYPE	REC	REMARKS
	Black saturated silt		WOH	1		0.2	
			WOH	2		0	
		5	WOH	3		0	
			WOH	4		0	
			WOH	5		0.3	
		10					
	Black silt						
			SHELBY TUBE	6		1.5	
		15					
			SHELBY TUBE	7		2.0	
		20					
	Black silt w/some organic material	25	WOH	8		1.0	
		30	WOH	9		1.0	
		35	WOH	10		1.2	
		40	WOH	11		2.0	

LEGEND
DS DRIVEN SPOON
ST SHELBY TUBE

GROUND WATER
AT COMPLETION CAVED

HSA HOLLOW STEM AUGER
DC DRIVEN CASING

National Foundation Engineering, Inc.

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BORING LOG

PROJECT Cherry Island - Phase II
Gannett Fleming

BORING No. GF-109 pg 2 o

PROJECT No. 89-2350

LOCATION OF BORING

ELEV. DATE: START FINISH INSPECTOR

HAMMER 140 lb. HAMMER DROP 30 in. SPOON OD 2 in. FOREMAN M. Kalandros

BORING METHOD Tripod ROCK CORE DIA MISC.

ELEV.	SOIL DESCRIPTION	DEPTH	BLOWS 6"	No.	TYPE	REC	REMARKS
	Black silt w/some organic material						
		45	WOH	12		1.5	
		50	WOH	13		1.1	
		55	TUBE	14		2.0	
	Bottom of hole 55.5'						
		60					
		65					
		70					
		75					
		80					

LEGEND
DS DRIVEN SPOON
ST SHELBY TUBE
PS PISTON SAMPLE

GROUND WATER
AT COMPLETION CAVED
AT HRS CAVED

HSA HOLLOW STEM AUGER
DC DRIVEN CASING
MD MUD DRILLING

National Foundation Engineering, Inc. NFE

3401 CARLINS PARK DRIVE • BALTIMORE, MARYLAND 21215

BORING LOG

PROJECT Cherry Island - Phase I

BORING No. GF-110

PROJECT No. 89-275

LOCATION OF BORING

ELEV. DATE: START 10-10-89 FINISH 10-10-89 INSPECTOR

HAMMER 140 lbs. HAMMER DROP 30 in. SPOON OD 2 in. FOREMAN Mike/Taylor

BORING METHOD HSA ROCK CORE DIA MISC.

ELEV.	SOIL DESCRIPTION	DEPTH	BLOWS 6"	No.	TYPE	REC	REMARKS
	Brown clayey silt		2-2-4-4	1	DS		Water encountered at 15.0 ft.
			4-4-3-4	2	DS		
	Grey clayey silt	5	1-1-2-1	3	DS		
			4-4-4-3	4	DS		
		10	1-2-2-2	5	DS		
				6	ST		
	Grey clayey silt	15	WOH-1-2	7	DS		
		20	1-1-1	8	DS		
		25	1-1-1	9	DS		
				10	ST		
		30	1-1-1	11	DS		
		35	1-1-1	12	DS		
		40	2-2-3	13	DS		

LEGEND
DS DRIVEN SPOON
ST SHELBY TUBE
PS PISTON SAMPLE

GROUND WATER
AT COMPLETION 15.0' CAVED 18.0'
AT HRS. CAVED

HSA HOLLOW STEM AUGER
DC DRIVEN CASING
MD MUD DRILLING

National Foundation Engineering, Inc. NFE

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BORING LOG

PROJECT Cherry Island - Phase I

BORING No. GF-110

PROJECT No. 89-275

LOCATION OF BORING

ELEV. DATE: START 10-10-89 FINISH 10-10-89 INSPECTOR

HAMMER 140 lbs. HAMMER DROP 30 in. SPOON OD 2 in. FOREMAN Mike/Taylor

BORING METHOD HSA ROCK CORE DIA MISC.

ELEV.	SOIL DESCRIPTION	DEPTH	BLOWS 6"	No.	TYPE	REC	REMARKS
	Grey clayey silt						
		45	2-2-3	14	DS		
		50	2-3-3	15	DS		
		55	3-2-3	16	DS		
		60	5-4-3	17	DS		
	Grey sand		4-8-8-8	18	DS		
	Bottom of Boring at 65.5 feet.	65					
		70					
		75					
		80					

LEGEND
DS DRIVEN SPOON
ST SHELBY TUBE
PS PISTON SAMPLE
RC ROCK CORE

GROUND WATER
AT COMPLETION 15.0' CAVED 18.0'
AT HRS. CAVED

HSA HOLLOW STEM AUGER
DC DRIVEN CASING
MD MUD DRILLING

EARTH ENGINEERING AND SCIENCES, INC.

TEST NO. GF - 111

FILE NAME: GANNETT-FLEMING
FILE NUMBER: JOB NUM 89-275

RECORD OF DILATOMETER TEST NO. GF - 111
USING DATA REDUCTION PROCEDURES IN MARCHETTI (ASCE,J-GED,MARCH 80)
KO IN SANDS DETERMINED USING SCHMERTMANN METHOD (1983)

LOCATION: CHERRY ISLAND
PERFORMED - DATE: 12-4-89
BY: AL MYERS

CALIBRATION INFORMATION:
DA= .20 BARS DB= 1.00 BARS ZM= .00 BARS ZW= 1.00 METERS

1 BAR = 1.019 KG/CM2 = 1.044 TSF = 14.51 PSI ANALYSIS USES H20 UNIT WEIGHT = 1.000 T/M3

Z (M)	THRUST (KG)	A (BAR)	B (BAR)	ED (BAR)	ID	KD	UO (BAR)	GAMMA (T/M3)	SV (BAR)	PC (BAR)	OCR	KO	CU (BAR)	PHI (DEG)	M (BAR)	SOIL TYPE
*****	******	*****	*****	*****	*****	*****	******	******	******	*****	*****	*****	*****	*****	******	************
1.52		.30	1.30	-7.	-.45		.051	PO1 =	.51	PO =	.50	P1 =	.30			QUESTIONABLE
3.05		.40	1.50	-3.	-.25		.201	PO1 =	.60	PO =	.60	P1 =	.50			QUESTIONABLE
4.57		.90	2.20	4.	.14	7.55	.350	1.500	.099	.78	7.94	1.54	.114		8.1	MUD
6.10		1.50	3.40	26.	.63	6.43	.500	1.600	.181	1.12	6.18	1.38	.172		52.2	CLAYEY SILT
7.62		1.60	3.20	15.	.37	4.18	.650	1.600	.271	.85	3.15	1.02	.149		23.4	SILTY CLAY
9.15		2.00	3.40	7.	.15	3.94	.800	1.500	.353	1.02	2.87	.97	.181		11.3	MUD
10.67		2.20	3.30	-3.	-.07		.949	PO1 =	2.40	PO =	2.40	P1 =	2.30			QUESTIONABLE
12.20		2.20	3.70	11.	.24	2.56	1.099	1.500	.503	.74	1.47	.68	.150		12.0	MUD
13.72		2.50	4.00	11.	.22	2.49	1.248	1.500	.578	.81	1.41	.67	.167		11.7	MUD
15.24		2.80	4.60	22.	.40	2.38	1.397	1.600	.660	.87	1.32	.64	.181		22.5	SILTY CLAY

END OF SOUNDING

EARTH ENGINEERING AND SCIENCES, INC. TEST NO. GF-112A

FILE NAME: GANNETT-FLEMING

FILE NUMBER: 89-275

RECORD OF DILATOMETER TEST NO. GF-112A

USING DATA REDUCTION PROCEDURES IN MARCHETTI (ASCE,J-GED,MARCH 80)

KO IN SANDS DETERMINED USING SCHMERTMANN METHOD (1983)

LOCATION: CHERRY ISLAND DISPOSAL AREA, DEL.

PERFORMED - DATE: 08 DEC 1989

BY: GRANT SLOAN

CALIBRATION INFORMATION:

DA= .20 BARS DB= 1.00 BARS ZM= .00 BARS ZW= 8.00 METERS

1 BAR = 1.019 KG/CM2 = 1.044 TSF = 14.51 PSI ANALYSIS USES H2O UNIT WEIGHT = 1.000 T/M3

Z (M)	THRUST (KG)	A (BAR)	B (BAR)	ED (BAR)	ID	KD	UO (BAR)	GAMMA (T/M3)	SV (BAR)	PC (BAR)	OCR	KO	CU (BAR)	PHI (DEG)	M (BAR)	SOIL TYPE
1.22		.80	2.20	7.	.21	6.19	.000	1.500	.160	.93	5.82	1.35	.144		14.6	MUD
1.52		.80	2.00	0.	.00	4.90	.000	1.500	.204	.83	4.04	1.14	.138		.0	MUD
3.05		.80	1.60	-14.	-.40		.000	P01 =	1.02	P0 =	1.00	P1 =	.60			QUESTIONABLE
4.57		1.00	2.00	-7.	-.17		.000	P01 =	1.21	P0 =	1.20	P1 =	1.00			QUESTIONABLE
6.10		1.20	16.00	496.	19.83	.76	.000	1.800	.946	.26	.28	-.75		44.3	421.2	SAND
7.62		1.80	2.60	-14.	-.20		.000	P01 =	2.02	P0 =	2.00	P1 =	1.60			QUESTIONABLE
9.15		1.80	2.80	-7.	-.11		.113	P01 =	2.01	P0 =	2.00	P1 =	1.80			QUESTIONABLE
10.67		2.20	3.00	-14.	-.19		.262	P01 =	2.42	P0 =	2.40	P1 =	2.00			QUESTIONABLE
12.20		2.40	3.00	-21.	-.27		.412	P01 =	2.63	P0 =	2.60	P1 =	2.00			QUESTIONABLE
13.72		3.00	4.00	-7.	-.08		.561	P01 =	3.21	P0 =	3.20	P1 =	3.00			QUESTIONABLE
14.33		2.80	3.90	-3.	-.04		.621	P01 =	3.00	P0 =	3.00	P1 =	2.90			QUESTIONABLE
.00		.00	.00	-42.	-6.00		.000	P01 =	.26	P0 =	.20	P1 =	-1.00			QUESTIONABLE

END OF SOUNDING

EARTH ENGINEERING AND SCIENCES, INC. TEST NO. GF-113

FILE NAME: GANNETT-FLEMING

FILE NUMBER: 89-2/5

RECORD OF DILATOMETER TEST NO. GF-113

USING DATA REDUCTION PROCEDURES IN MARCHETTI (ASCE,J-GED,MARCH 80)

KO IN SANDS DETERMINED USING SCHMERTMANN METHOD (1983)

LOCATION: CHERRY ISLAND DISPOSAL AREA, DEL.

PERFORMED - DATE: 20 DEC 1989

BY: GRANT SLOAN

CALIBRATION INFORMATION:

DA= .10 BARS DB= .30 BARS ZM= .00 BARS ZW= 8.00 METERS

1 BAR = 1.019 KG/CM2 = 1.044 TSF = 14.51 PSI ANALYSIS USES H20 UNIT WEIGHT = 1.000 T/M3

Z (M)	THRUST (KG)	A (BAR)	B (BAR)	ED (BAR)	ID	KD	UO (BAR)	GAMMA (T/M3)	SV (BAR)	PC (BAR)	OCR	KO	CU (BAR)	PHI (DEG)	M (BAR)	SOIL TYPE
*****	******	*****	*****	*****	*****	*****	******	******	******	*****	*****	*****	*****	*****	******	************
3.05		.20	.80	7.	.72	.00	.000	1.500	******	.00	.00	-.60	.002		6.2	MUD
4.57		.80	1.40	7.	.24	.00	.000	1.500	******	.00	.00	-.59	.007		6.2	MUD
6.10		1.20	2.00	15.	.33	.00	.000	1.600	******	.00	.00	-.59	.011		12.4	CLAY
7.62		1.60	2.20	7.	.12	.00	.000	1.500	******	.00	.00	-.59	.016		6.2	MUD
9.15		1.60	2.20	7.	.13	.00	.113	1.500	******	.00	.00	-.59	.014		6.2	MUD
10.67		2.00	2.60	7.	.11	.00	.262	1.500	******	.00	.00	-.59	.017		6.2	MUD
12.20		2.20	2.80	7.	.11	.00	.412	1.500	******	.00	.00	-.59	.018		6.2	MUD
13.72		2.60	3.20	7.	.10	.00	.561	1.500	******	.00	.00	-.59	.021		6.2	MUD
15.24		3.60	6.80	102.	1.03	.00	.711	1.700	******	.01	.00	-.59			86.7	SILT
.00		.00	.00	-14.	-4.00		.000	P01 =	.12	P0 =	.10	P1 =	-.30			QUESTIONABLE

END OF SOUNDING

1

TEST NO. GF - 114

EARTH ENGINEERING AND SCIENCES, INC.
FILE NAME: GANNETT-FLEMING
FILE NUMBER: 89-275

RECORD OF DILATOMETER TEST NO. GF - 114
USING DATA REDUCTION PROCEDURES IN MARCHETTI (ASCE,J-GED,MARCH 80)
KO IN SANDS DETERMINED USING SCHMERTMANN METHOD (1983)

LOCATION: CHERRY ISLAND DISPOSAL AREA, DEL.
PERFORMED - DATE: 05 JAN 1990
BY: GRANT SLOAN

CALIBRATION INFORMATION:
DA= .20 BARS DB= .90 BARS ZM= .00 BARS ZW= 8.00 METERS

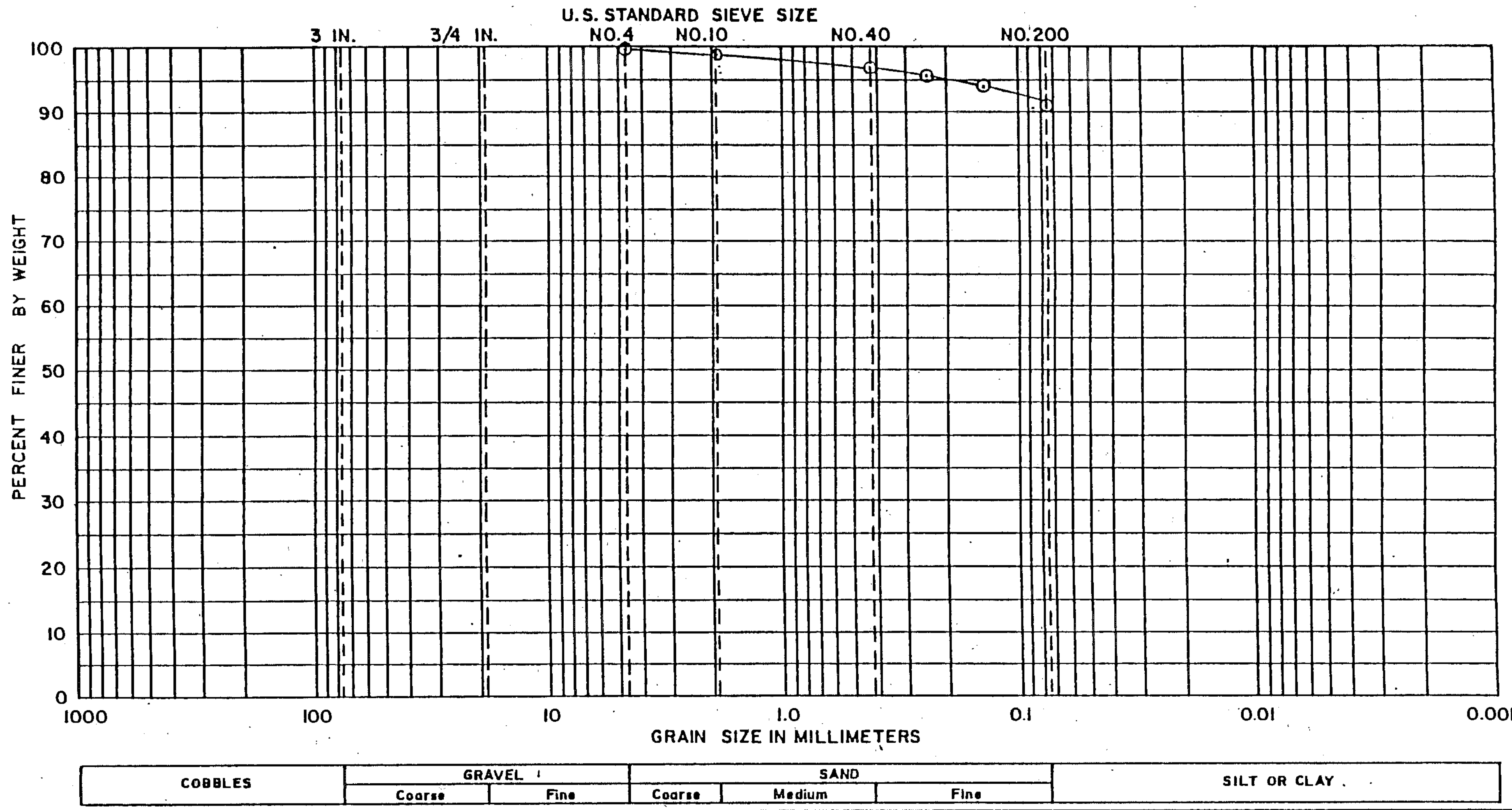
1 BAR = 1.019 KG/CM2 = 1.044 TSF = 14.51 PSI ANALYSIS USES H2O UNIT WEIGHT = 1.000 T/M3

Z (M)	THRUST (KG)	A (BAR)	B (BAR)	ED (BAR)	ID	KD	UO (BAR)	GAMMA (T/M3)	SV (BAR)	PC (BAR)	OCR	KO	CU (BAR)	PHI (DEG)	M (BAR)	SOIL TYPE
1.22		.30	1.40	0.	.00		.000	PO1 =	.50	PO =	.50	P1 =	.50			QUESTIONABLE
1.52		.30	1.40	0.	.00		.000	PO1 =	.50	PO =	.50	P1 =	.50			QUESTIONABLE
3.05		.60	1.40	-10.	-.38		.000	PO1 =	.81	PO =	.80	P1 =	.50			QUESTIONABLE
4.57		1.00	1.80	-10.	-.26		.000	PO1 =	1.21	PO =	1.20	P1 =	.90			QUESTIONABLE
6.10		1.20	2.20	-3.	-.07		.000	PO1 =	1.40	PO =	1.40	P1 =	1.30			QUESTIONABLE
7.62		1.40	2.60	4.	.07	1.68	.000	1.500	.942	.73	.77	.46	.168		3.1	MUD
9.15		1.40	2.40	-3.	-.07		.113	PO1 =	1.80	PO =	1.60	P1 =	1.50			QUESTIONABLE
10.67		2.20	3.20	-3.	-.05		.262	PO1 =	2.40	PO =	2.40	P1 =	2.30			QUESTIONABLE
12.20		2.20	3.80	18.	.27	1.60	.412	1.600	1.227	.87	.71	.43	.204		15.5	CLAY
.00		.00	.00	-38.	-5.50		.000	PO1 =	.25	PO =	.20	P1 =	-.90			QUESTIONABLE

END OF SOUNDING

LAB DATA

CLASSIFICATIONS



Sample No.	Depth	Classification	Nat.WC	LL	PL	PI	Gs
S3,S4,S5	4.0'-10.0'	MH	44.8	50.6	39.1	11.5	–

Description and Comments: Gray Elastic Silt

CLASSIFICATION TEST – GRADATION CURVES

GANNETT FLEMING GEOTECHNICAL LABORATORY

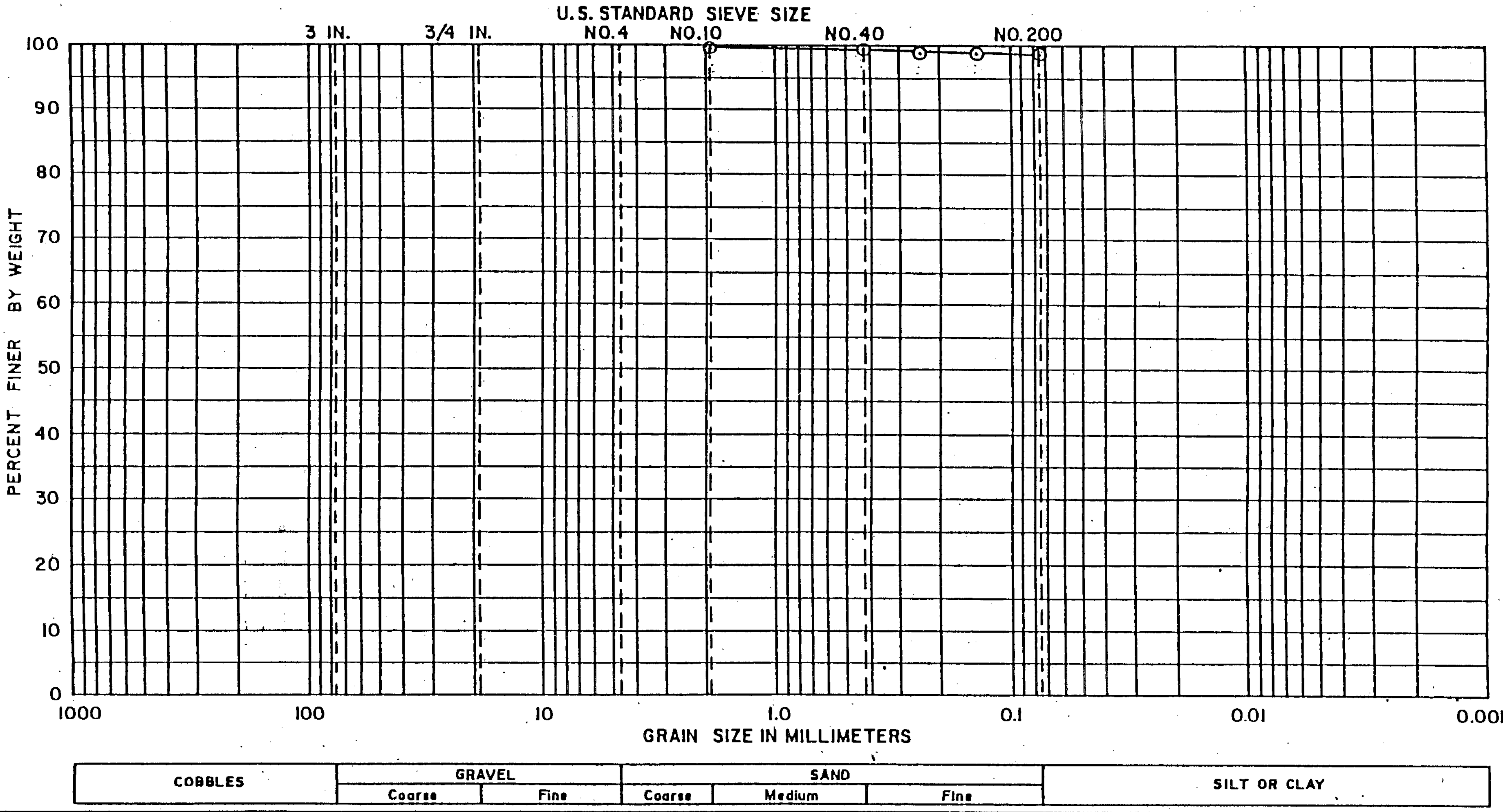
Project: DSWA NSWF - 2 PHASE III

Area: WILMINGTON , DE

Boring No: GF-101

Date: 10/27/89

Tested By: DKN



Sample No.	Depth	Classification	Nat. WC	LL	PL	PI	Gs
S7,S8	18.5'-25.0'	MH	74.9	60.6	41.9	18.7	-

Description and Comments: Gray Elastic Silt

CLASSIFICATION TEST-GRADATION CURVES

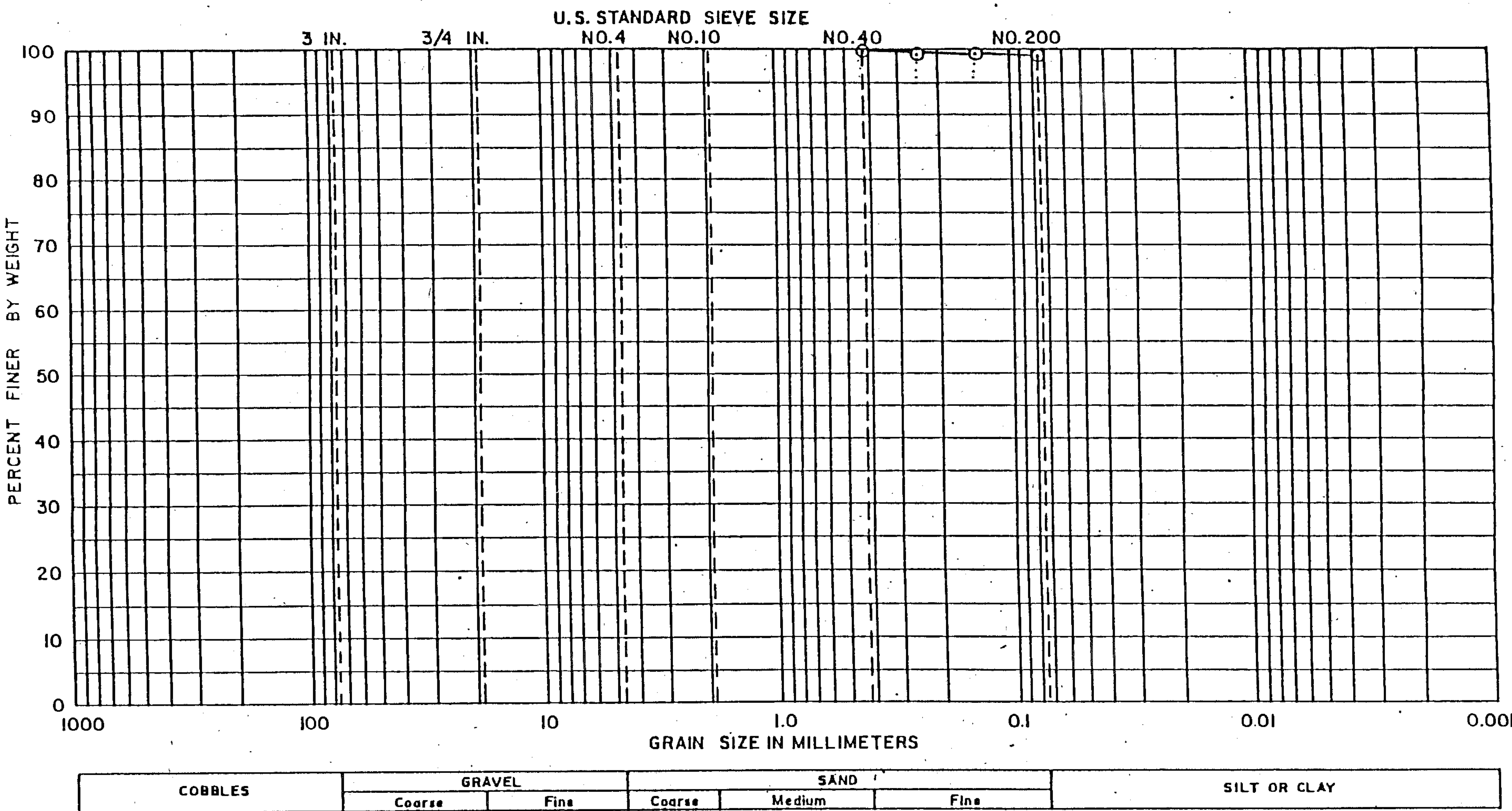
GANNETT FLEMING GEOTECHNICAL LABORATORY

Project: DSWA NSWF - 2 PHASE III

Area: WILMINGTON , DE

Boring No: GF-101

Date: 10/27/89 **Tested By:** DKN



Sample No.	Depth	Classification	Nat.WC	LL	PL	PI	Gs
S11,S12	38.5'-45.0'	MH	76.0	63.2	43.2	20.0	-

Description and Comments: Gray Elastic Silt

GANNETT FLEMING GEOTECHNICAL LABORATORY

Project: DSWA NSWF - 2 PHASE III

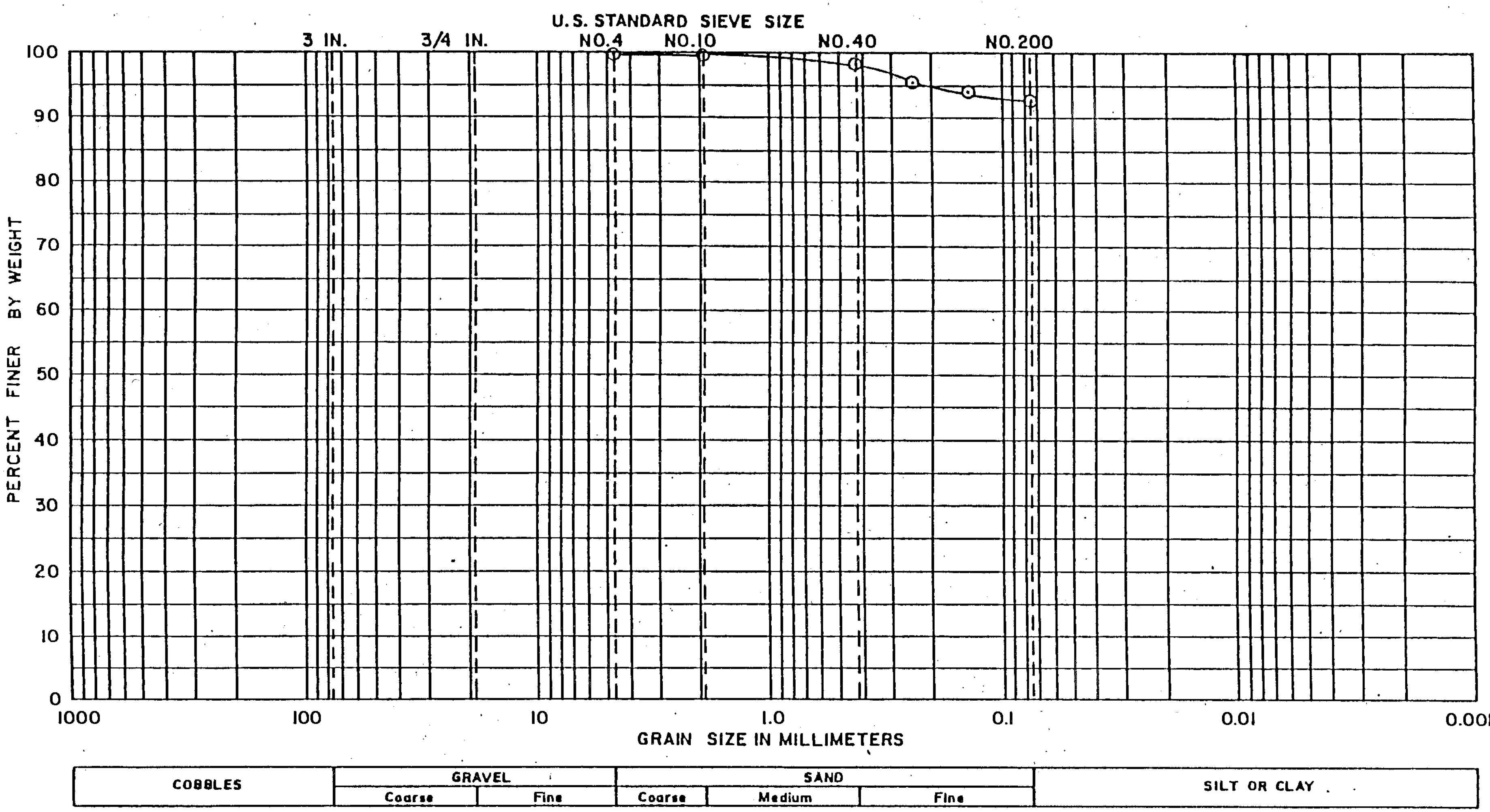
Area: WILMINGTON , DE

Boring No: GF-101

Date: 10/27/89

Tested By: DKN

CLASSIFICATION TEST-GRADATION CURVES



Sample No.	Depth	Classification	Nat.WC	LL	PL	PI	Gs
S16,S17	63.5'-70.0'	ML	47.5	42.2	26.8	15.4	–

Description and Comments: Gray Silt

GANNETT FLEMING GEOTECHNICAL LABORATORY

Project: DSWA NSWF – 2 PHASE III

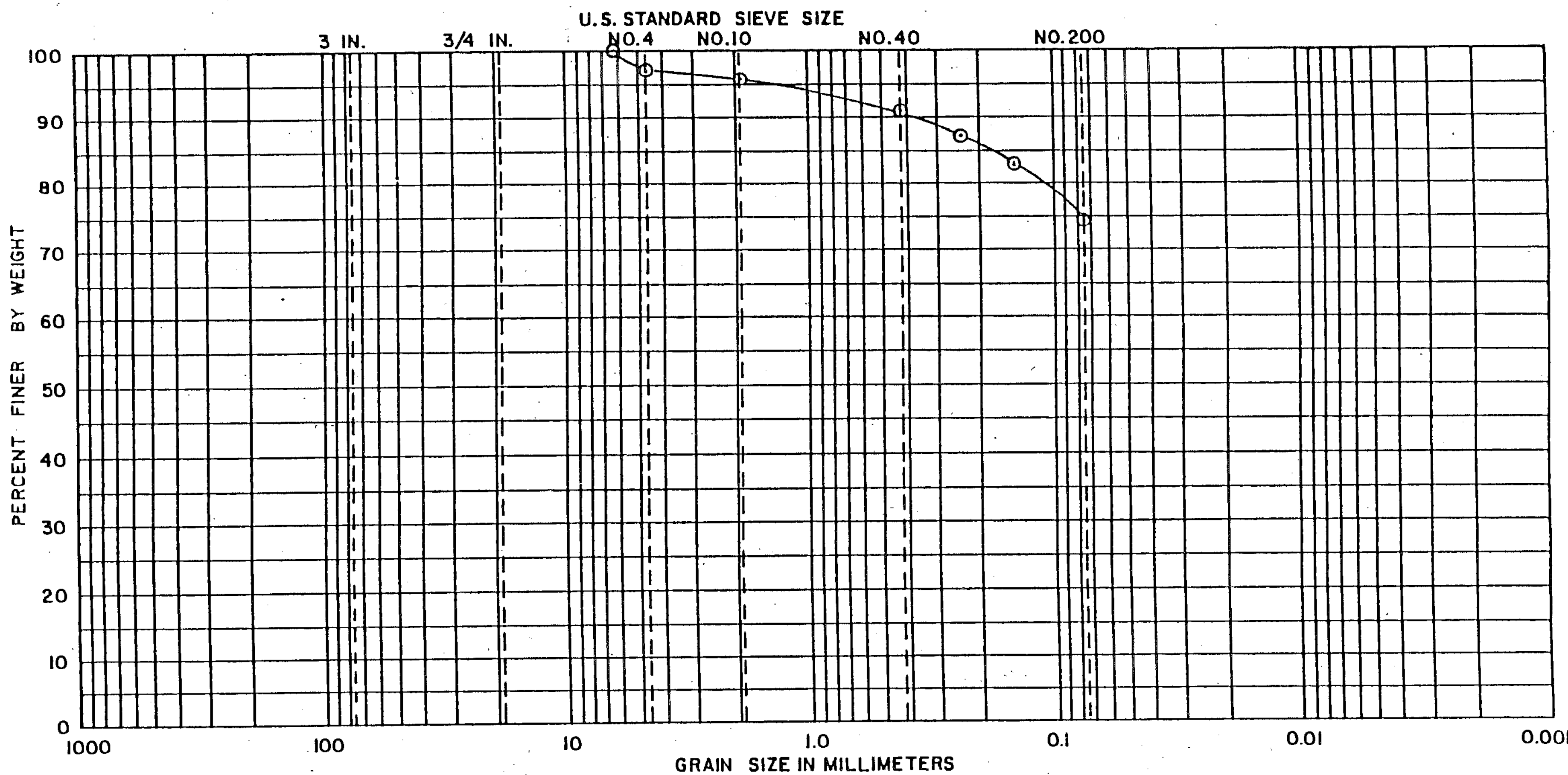
Area: WILMINGTON , DE.

Boring No: GF-101

Date: 10/27/89

Tested By: DKN

CLASSIFICATION TEST – GRADATION CURVES



COBBLES	GRAVEL		SAND			SILT OR CLAY
	Coarse	Fine	Coarse	Medium	Fine	

Sample No.	Depth	Classification	Nat.WC	LL	PL	PI	Gs
U-1	4.0'-6.0'	ML	43.7	37.7	31.0	6.7	-

Description and Comments: Gray Silt With Sand / Max. Grain Size 0.25"
Sample No. 1 Dry Density = 54pcf/W Content=49.2%
Sample No. 2 Dry Density = 62pcf/W Content=44.5%

CLASSIFICATION TEST-GRADATION CURVES

GANNETT FLEMING GEOTECHNICAL LABORATORY

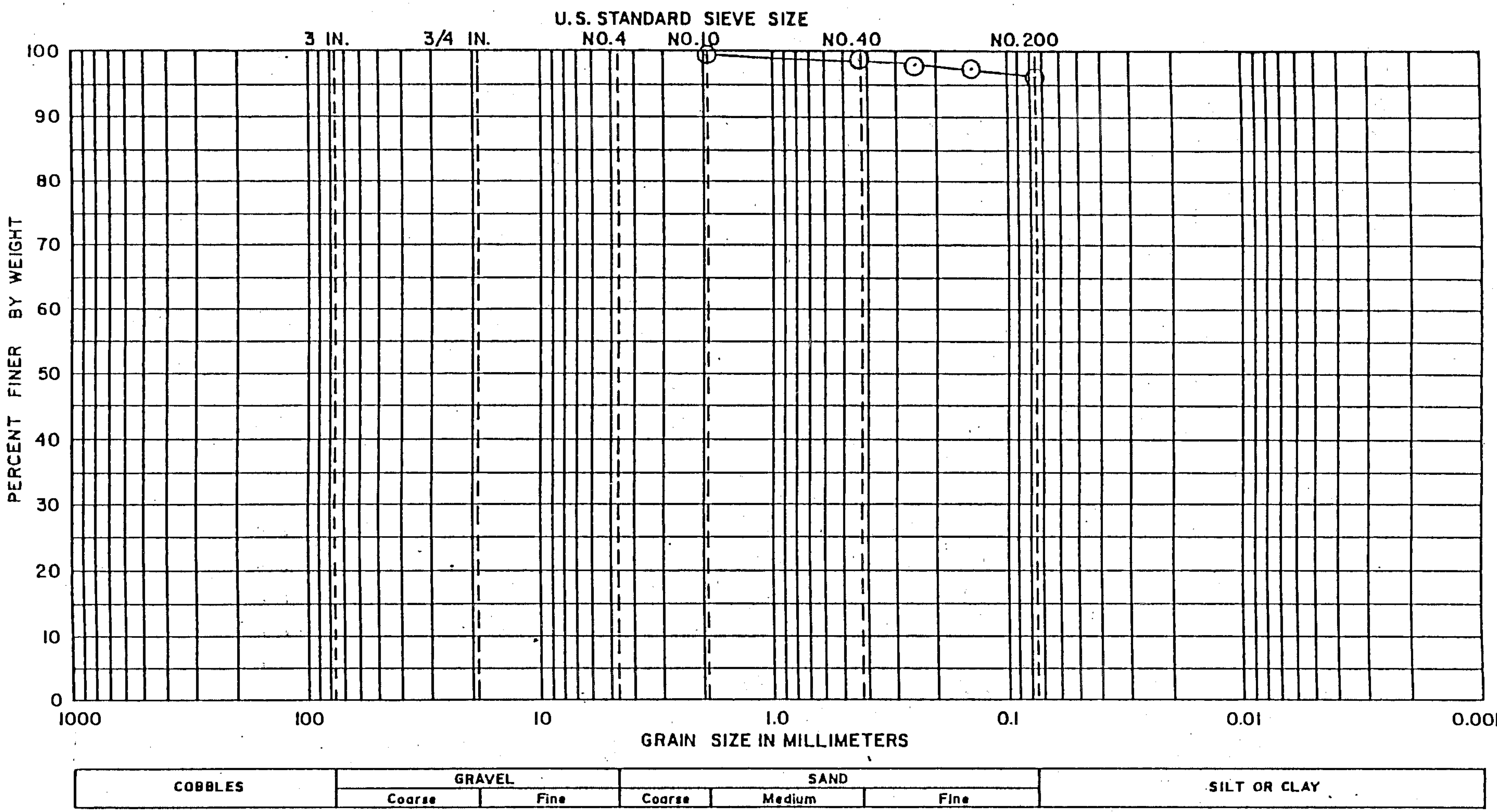
Project: DSWA NSWF - 2 PHASE III

Area: WILMINGTON , DE

Boring No: GF-102

Date: 11/3/89

Tested By: DKN



Sample No.	Depth	Classification	Nat.WC	LL	PL	PI	Gs
GF-102/S6,7	13.5'-20.0'	MH	69.7	57.6	39.1	18.5	--

Description and Comments: Gray Elastic Silt

GANNETT FLEMING GEOTECHNICAL LABORATORY

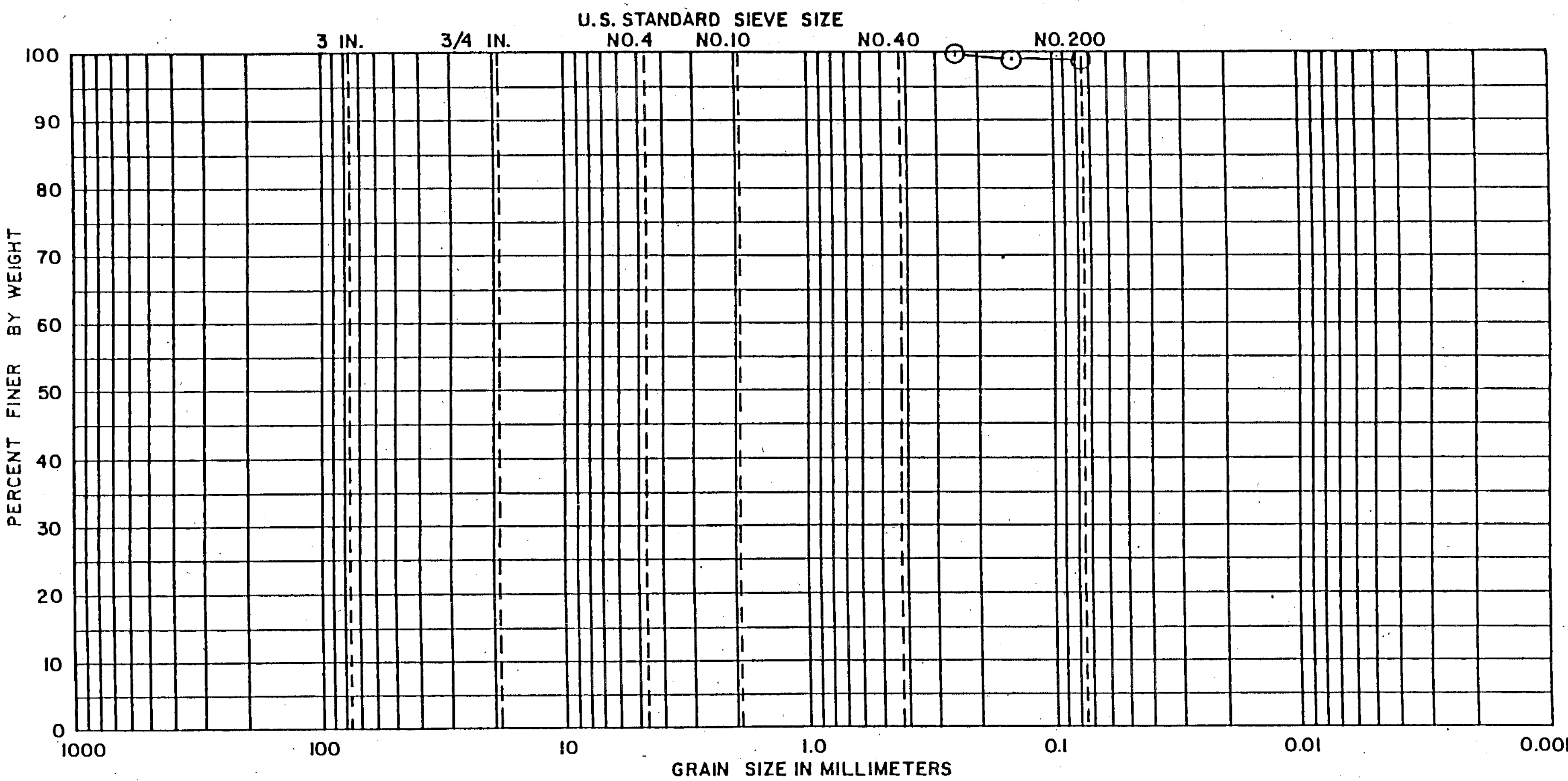
Project: DSWA NSWA - 2 PHASE III

Area: WILMINGTON, DE

Boring No: GF-102 / S-6 & S-7

Date: Oct. 27,1989 | Tested By: K.A.Abdolos

CLASSIFICATION TEST-GRADATION CURVES



COBBLES	GRAVEL		SAND			SILT OR CLAY
	Coarse	Fine	Coarse	Medium	Fine	

Sample No.	Depth	Classification	Nat.WC	LL	PL	PI	Gs
GF-102/S-9	28.5'-35.0'	MH	92.8	66.3	46.2	20.1	--
S-10							

Description and Comments: Gray Elastic Silt

GANNETT FLEMING GEOTECHNICAL LABORATORY

Project: DSWA NSWF - 2 PHASE III

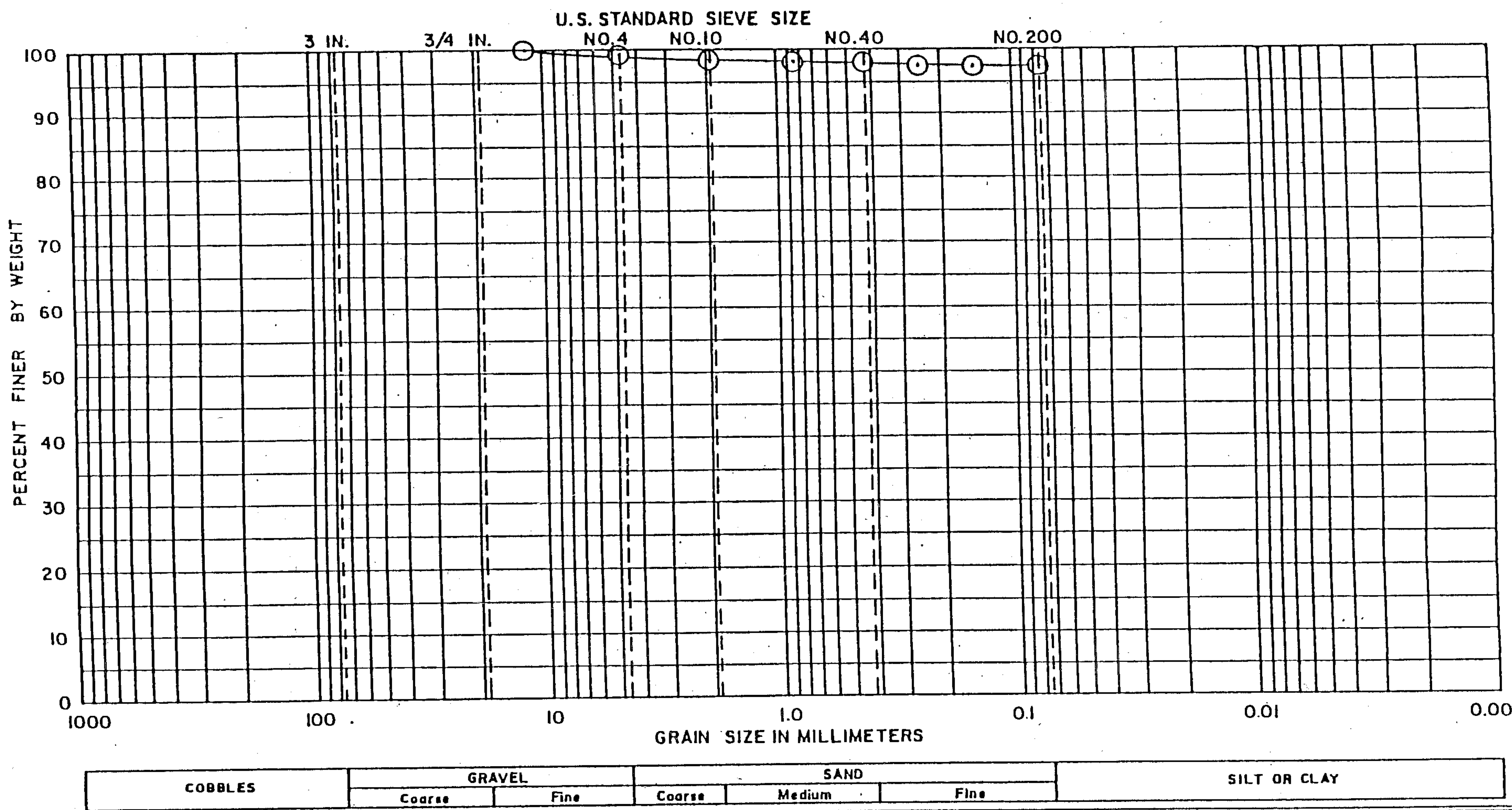
Area: WILMINGTON, DE.

Boring No: GF-102 / S-9 & S-10

Date: Oct. 27,1989

Tested By: K.A. Abdolos

CLASSIFICATION TEST-GRADATION CURVES



Sample No.	Depth	Classification	Nat. WC	LL	PL	PI	Gs
U-2	35'-37'	MH	91.2	69.3	47.8	21.5	--
		(LL OVEN DRYED)	=	58.8			

Description and Comments:

1) BROWN-ELASTIC SILT.
2) LL OD / LL AD = .85

GANNETT FLEMING GEOTECHNICAL LABORATORY

Project: DSWA, NSWF, PHASE III

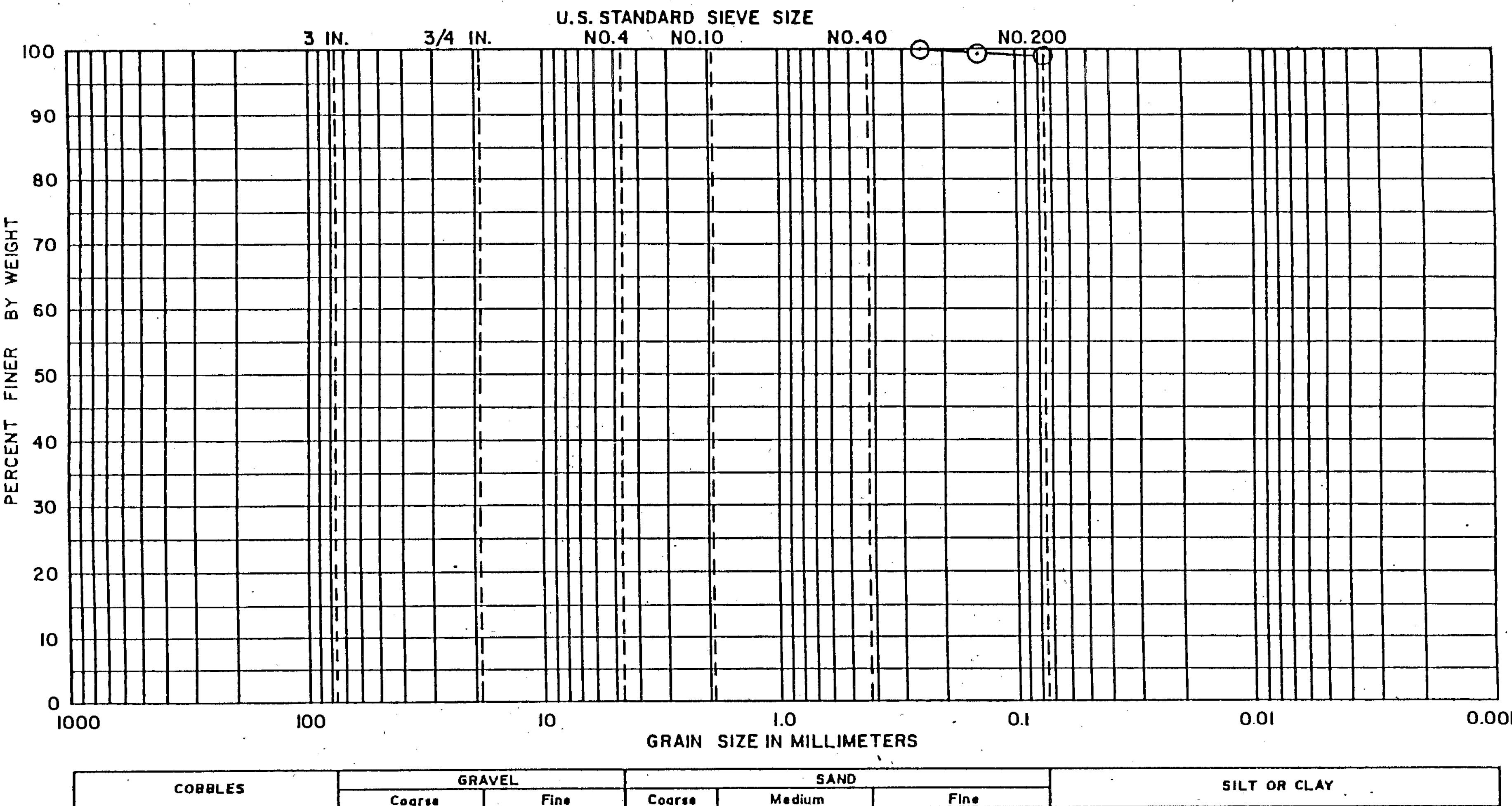
Area: WILMINGTON, DE

Boring No: GF-102

Date: 2/1/90

Tested By: KLM

CLASSIFICATION TEST-GRADATION CURVES



Sample No.	Depth	Classification	Nat.WC	LL	PL	PI	Gs
GF-102/S-14	48.5'-55.0'	ML	63.0	46.5	33.0	13.5	--
S-15							

Description and Comments: Gray Silt

CLASSIFICATION TEST - GRADATION CURVES

GANNETT FLEMING GEOTECHNICAL LABORATORY

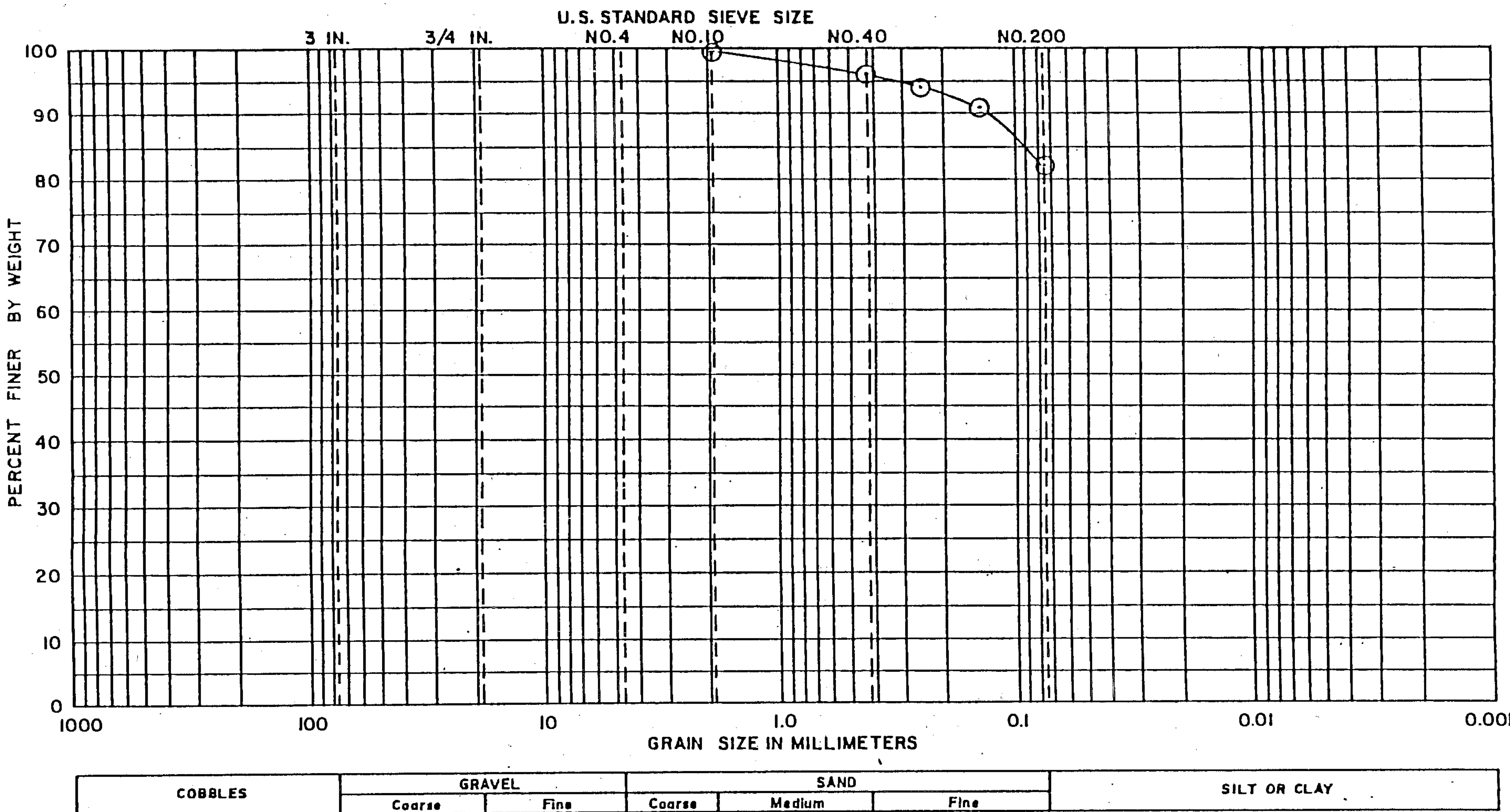
Project: DSWA NSWF - 2 PHASE III

Area: WILMINGTON, DE.

Boring No: GF-102 / S-14 & S-15

Date: Oct: 27,1989

Tested By: K.A.Abdolos



Sample No.	Depth	Classification	Nat.WC	LL	PL	PI	Gs
GF-103/S-2, S-3 & S-4	2.0' - 8.0'	ML	47.8	--	N/P	--	--

Description and Comments: Gray Silt with Sand

CLASSIFICATION TEST-GRADATION CURVES

GANNETT FLEMING GEOTECHNICAL LABORATORY

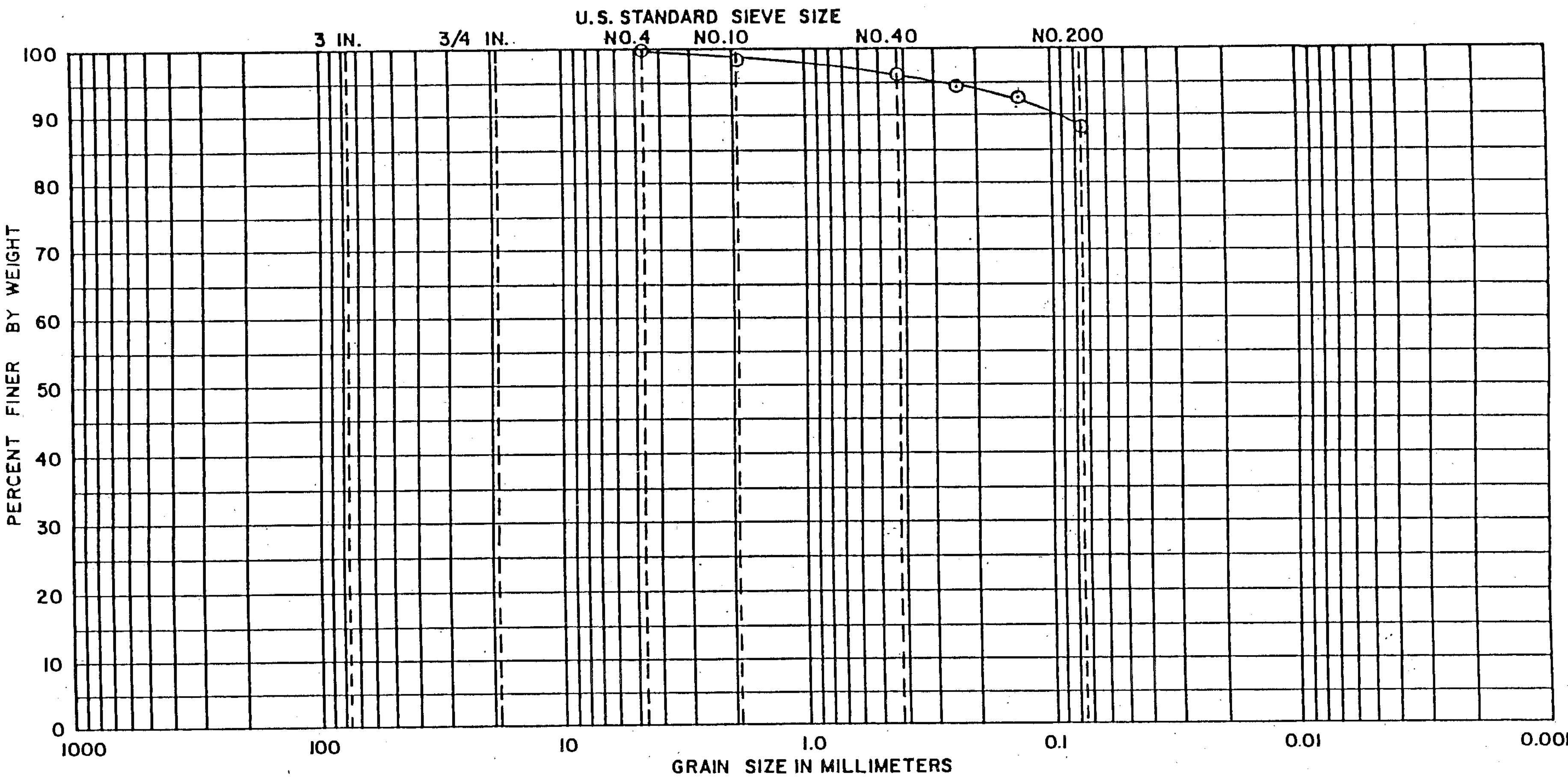
Project: DSWA NSWF - 2 PHASE III

Area: WILMINGTON, DE.

Boring No: GF-103 / S-2 , S-3 & S-4

Date: Oct. 27,1989

Tested By: K.A. Abdolos



COBBLES	GRAVEL		SAND			SILT OR CLAY
	Coarse	Fine	Coarse	Medium	Fine	

Sample No.	Depth	Classification	Nat.WC	LL	PL	PI	Gs
U-1	15.0'-17.0'	ML	94.2	43.7	43.5	0.2	–

Description and Comments: Gray Silt

CLASSIFICATION TEST-GRADATION CURVES

GANNETT FLEMING GEOTECHNICAL LABORATORY

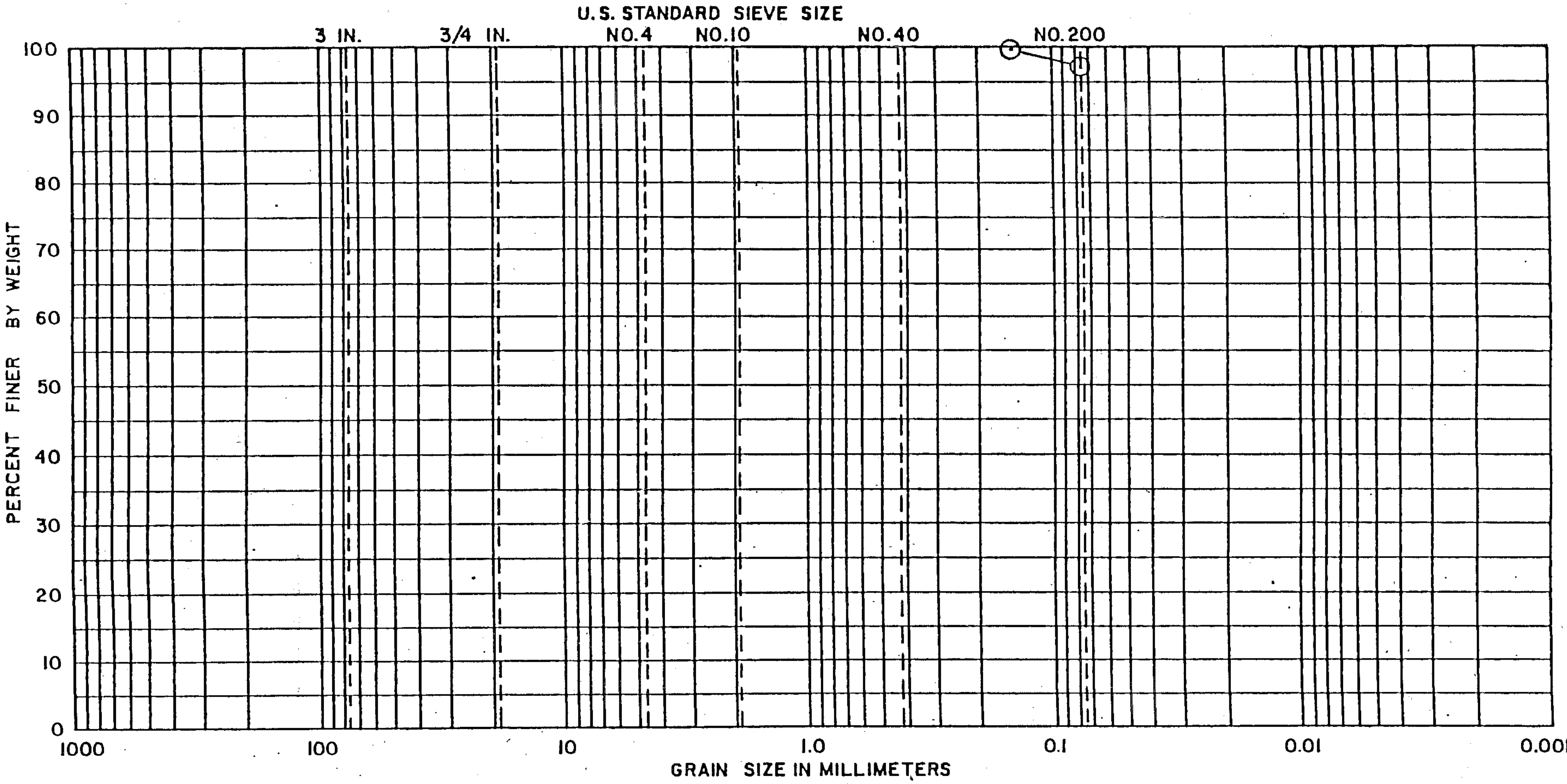
Project: DSWA NSWF - 2 PHASE III

Area: WILMINGTON , DE

Boring No: GF-103

Date: 11/3/89

Tested By: DKN



COBBLES	GRAVEL		SAND			SILT OR CLAY
	Coarse	Fine	Coarse	Medium	Fine	

Sample No.	Depth	Classification	Nat.WC	LL	PL	PI	Gs
GF-103/S-8,	23.5'-30.0'	ML	65.8	48.2	34.0	14.2	--
S-9							

Description and Comments: Brown Silt

CLASSIFICATION TEST-GRADATION CURVES

GANNETT FLEMING GEOTECHNICAL LABORATORY

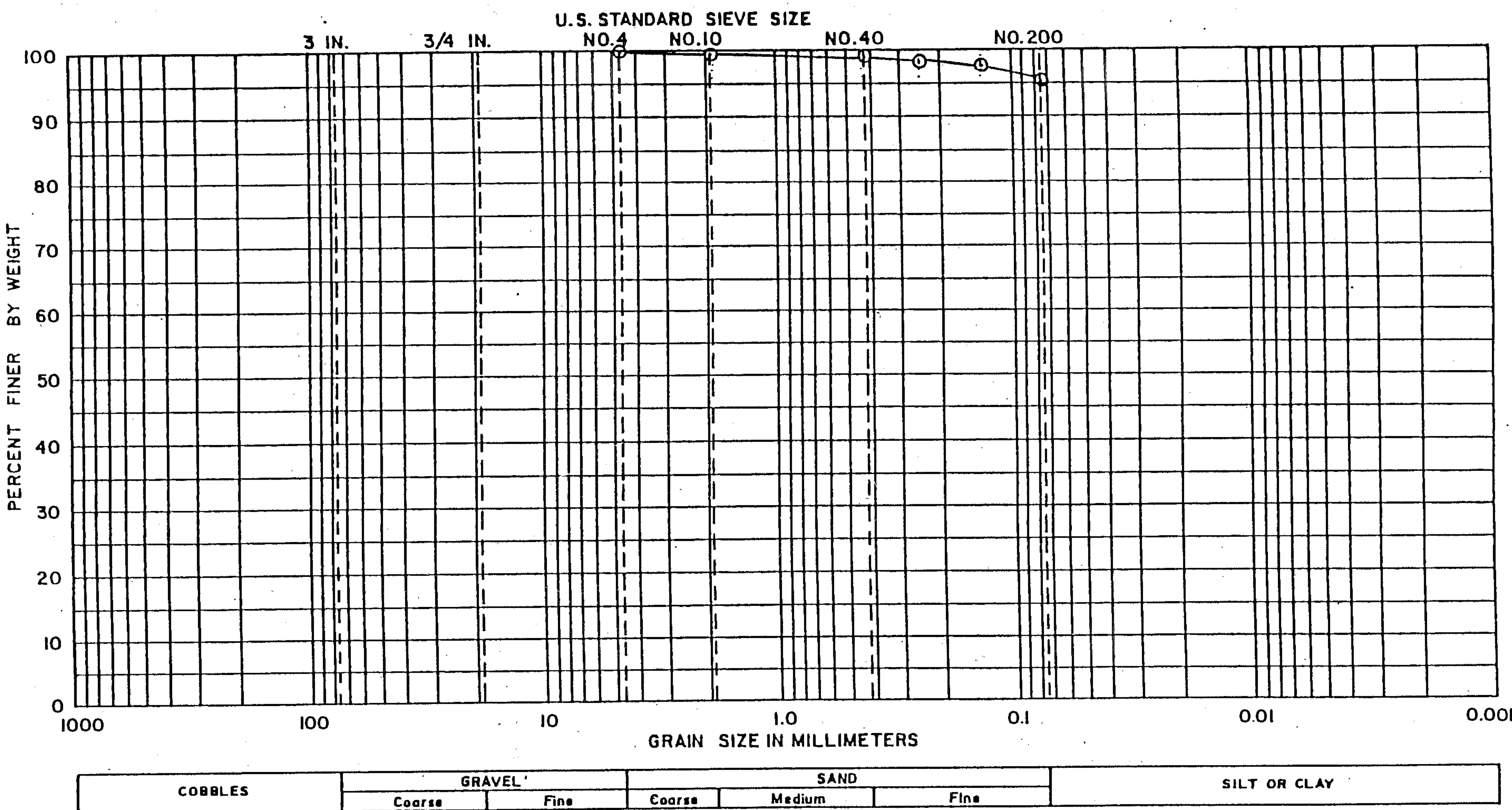
Project: DSWA NSWF - 2 PHASE III

Area: WILMINGTON, DE.

Boring No: GF-103 / S-8 & S-9

Date: Oct. 27, 1989

Tested By: K.A. Abdolos



Sample No.	Depth	Classification	Nat. WC	LL	PL	PI	Gs
U-2	45.0'-47.0'	ML	61.4	47.4	33.5	13.9	-

Description and Comments: Gray Silt

CLASSIFICATION TEST-GRADATION CURVES

GANNETT FLEMING GEOTECHNICAL LABORATORY

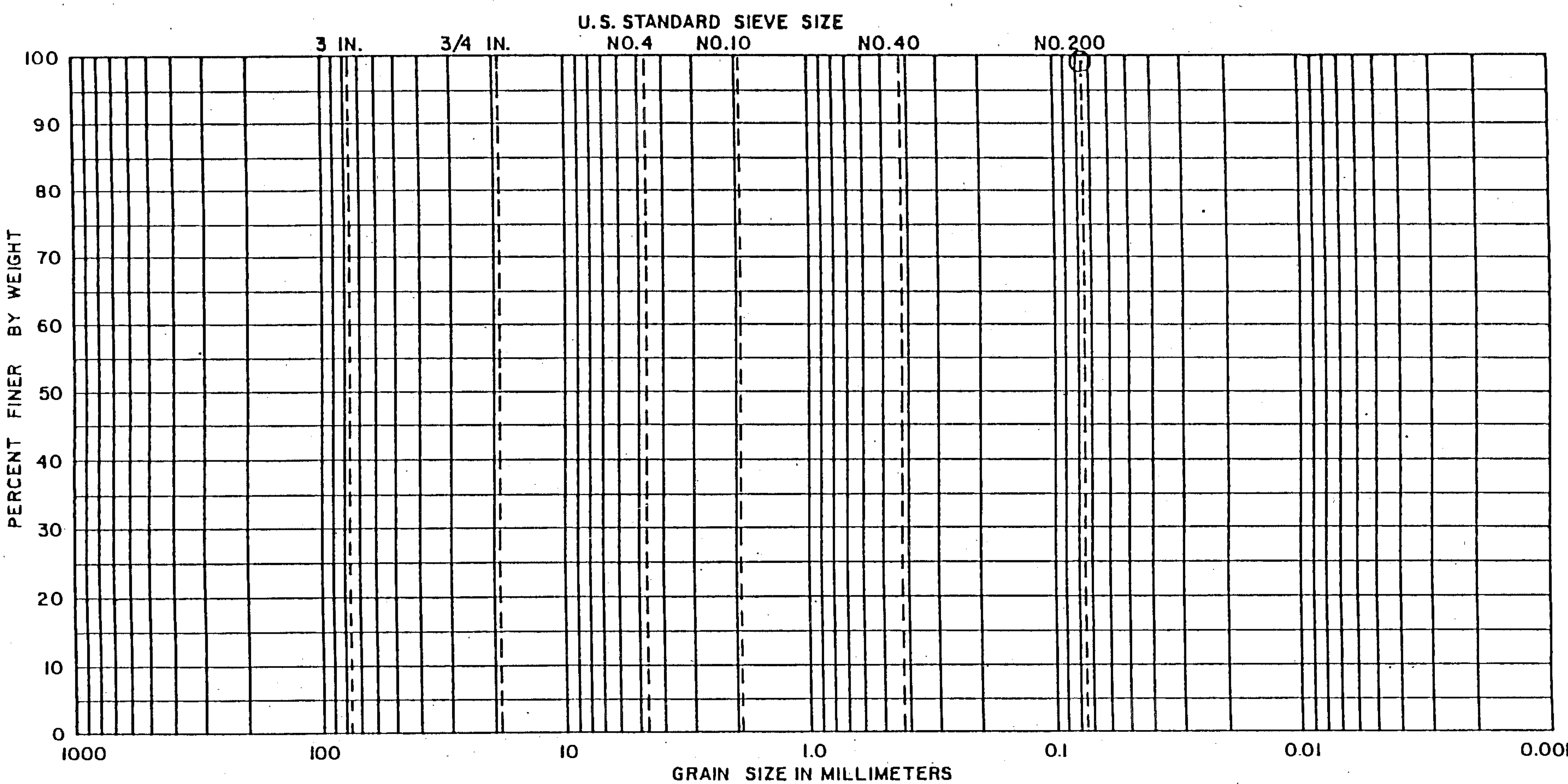
Project: DSWA NSWF - 2 PHASE III

Area: WILMINGTON , DE

Boring No: GF-103

Date: 11/3/89

Tested By: DKN



COBBLES	GRAVEL		SAND			SILT OR CLAY
	Coarse	Fine	Coarse	Medium	Fine	

Sample No.	Depth	Classification	Nat.WC	LL	PL	PI	Gs
BAG	0 - 1'	MH		79.9	54.6	25.3	--
		(LL OVEN DRYED)	=	66.3			

Description and Comments:
1) GRAYISH BLACK-ELASTIC SILT.
2) LL OD / LL AD = .83

CLASSIFICATION TEST-GRADATION CURVES

GANNETT FLEMING GEOTECHNICAL LABORATORY

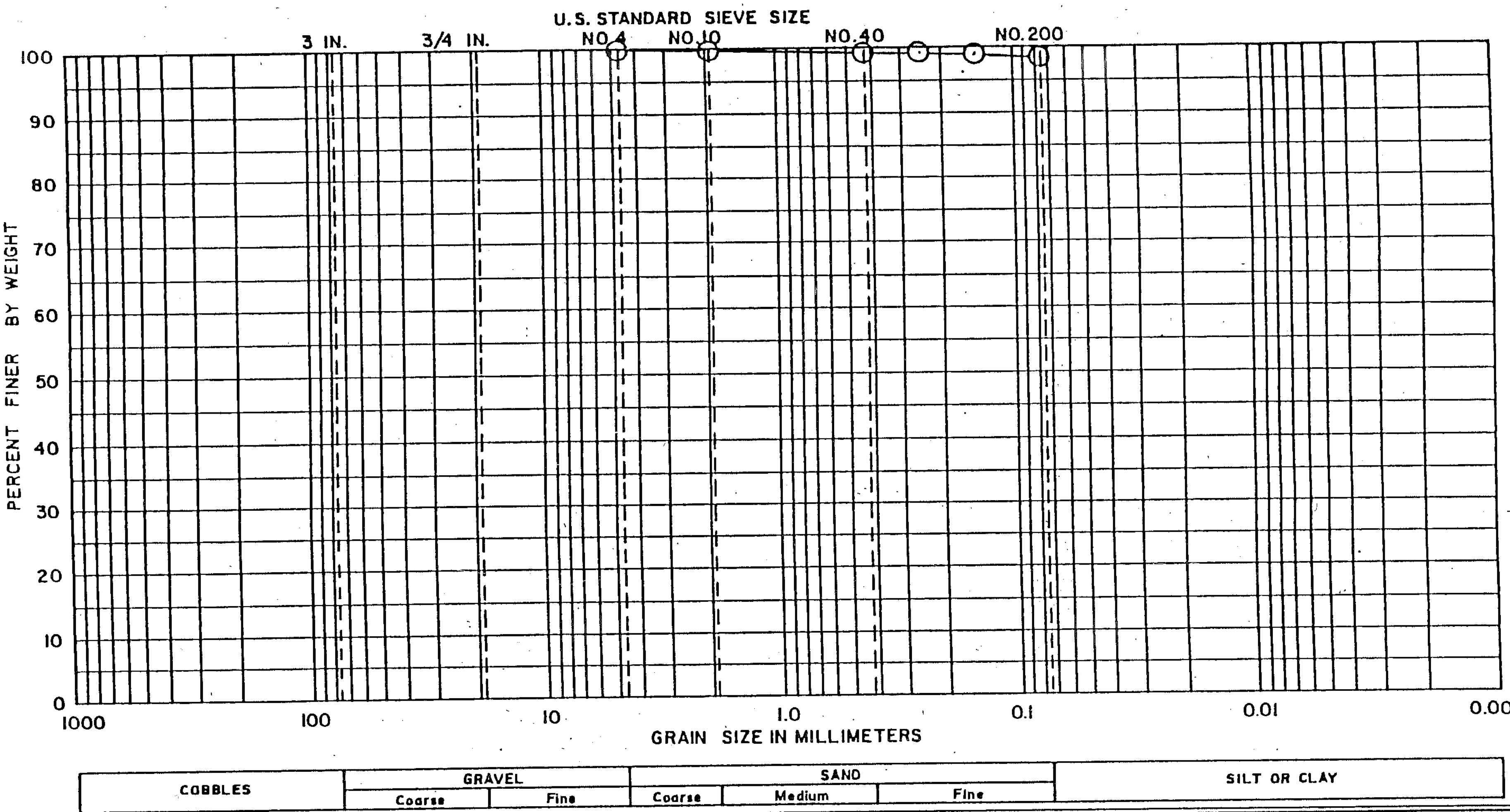
Project: DSWA, NSWF, PHASE III

Area: WILMINGTON, DE

Boring No: GF-104

Date: 2/16/90

Tested By: KLM



Sample No.	Depth	Classification	Nat. WC	LL	PL	PI	Gs
S1,2,3	0-6'	MH	141.0	75.2	49.0	26.2	-

Description and Comments: DK. GRAYISH BROWN-ELASTIC SILT.

CLASSIFICATION TEST-GRADATION CURVES

GANNETT FLEMING GEOTECHNICAL LABORATORY

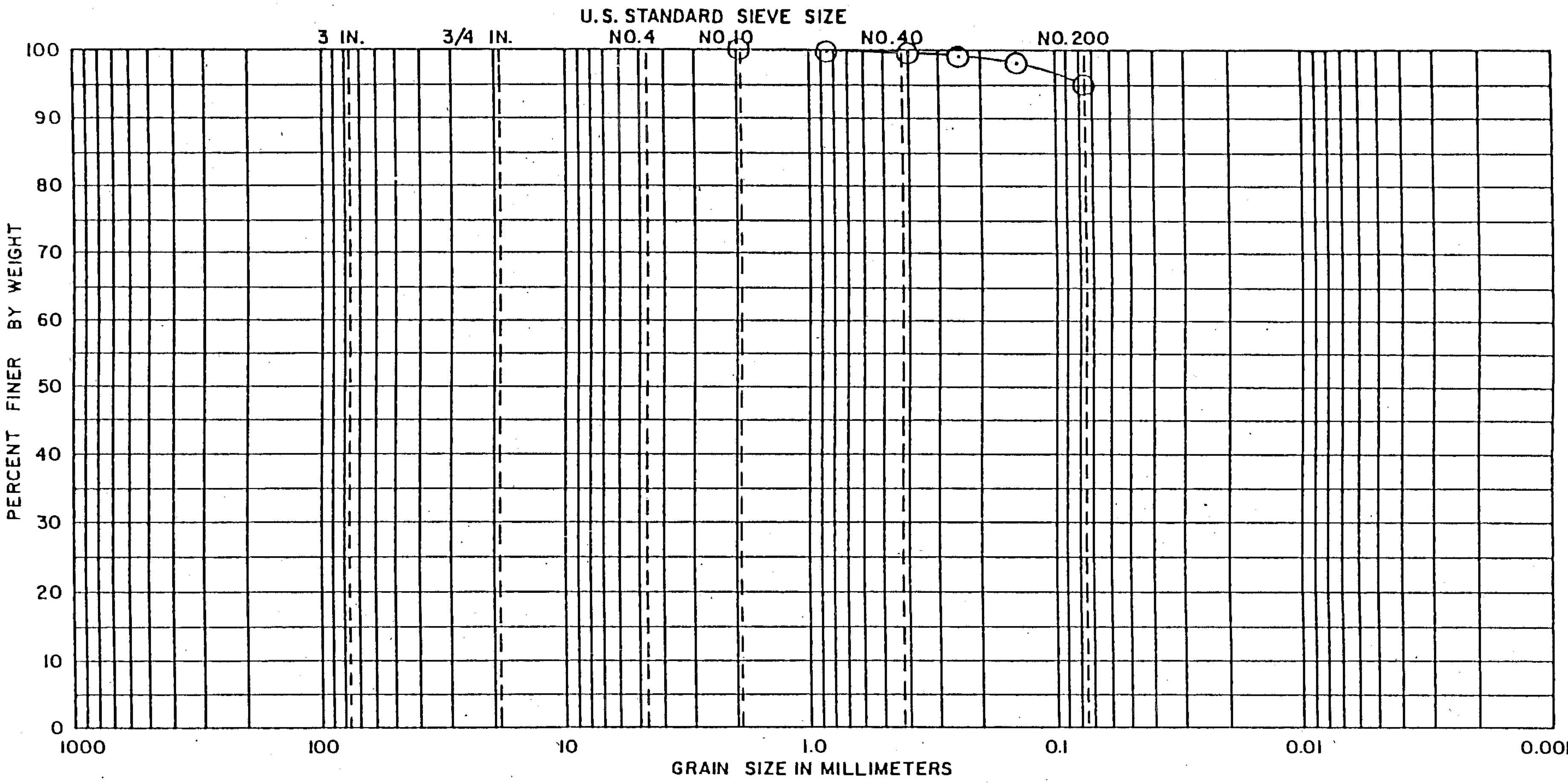
Project: DSWA NSWF-2 PHASE III

Area: WILMINGTON, DE

Boring No: GF-104

Date: 12/13/89

Tested By: KLM



COBBLES	GRAVEL		SAND			SILT OR CLAY
	Coarse	Fine	Coarse	Medium	Fine	

Sample No.	Depth	Classification	Nat.WC	LL	PL	PI	Gs
U-1	6.5-8.5'	MH		65.9	40.9	25.0	--
		(LL OVEN DRYED)	=	52.0			

Description and Comments:
1) BROWN-ELASTIC SILT.
2) LL OD/LL AD = .79

CLASSIFICATION TEST-GRADATION CURVES

GANNETT FLEMING GEOTECHNICAL LABORATORY

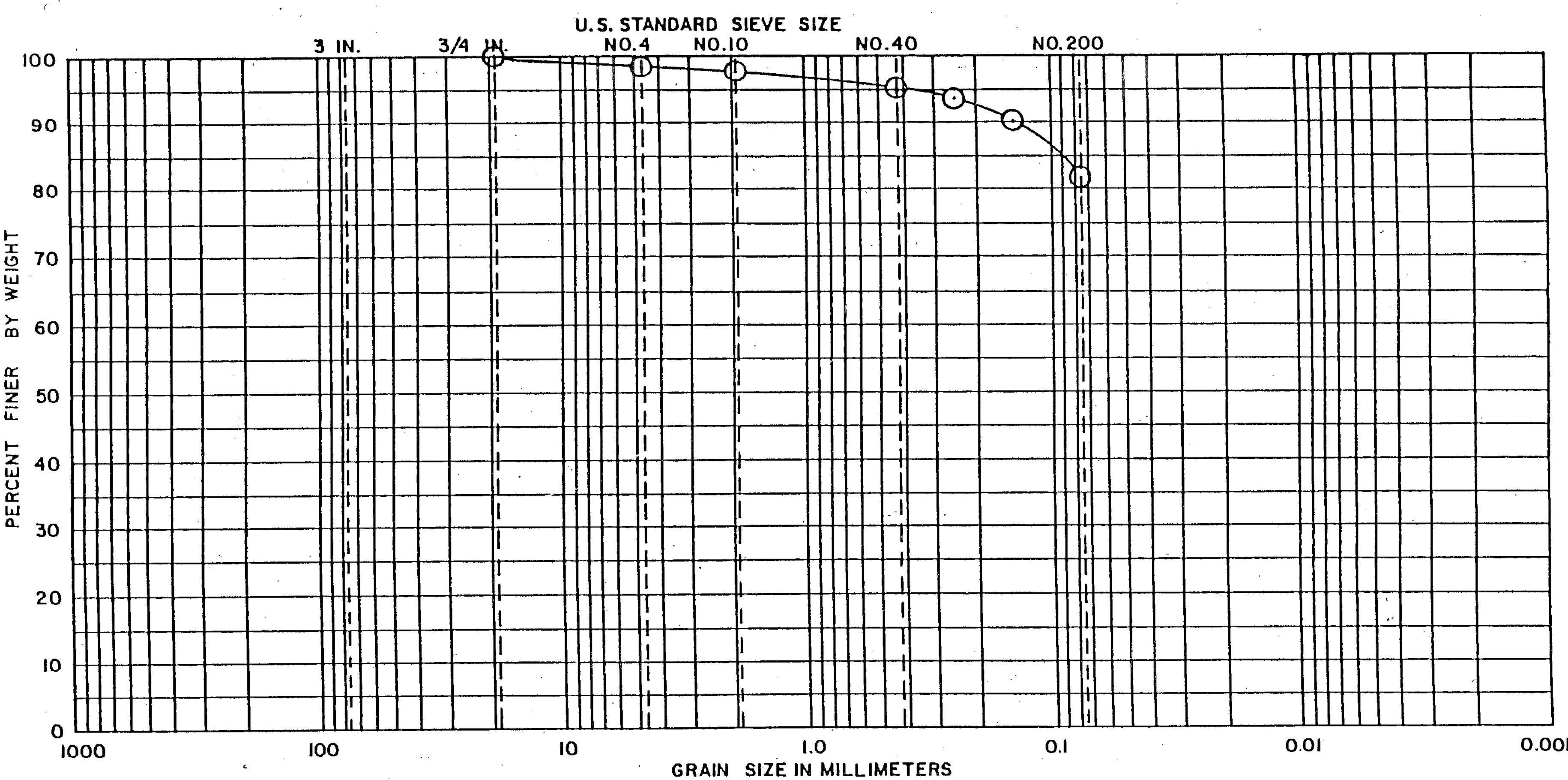
Project: DSWA, NSWF, PHASE III

Area: WILMINGTON, DE

Boring No: GF-104

Date: 1/24/90

Tested By: KLM



COBBLES	GRAVEL		SAND			SILT OR CLAY
	Coarse	Fine	Coarse	Medium	Fine	

Sample No.	Depth	Classification	Nat. WC	LL	PL	PI	Gs
U-2	16.5'-18.5	ML	55.1	--	N/P	--	-

Description and Comments: GRAY-SILT WITH SAND.

GANNETT FLEMING GEOTECHNICAL LABORATORY

Project: DSWA NSWF-2 PHASE III

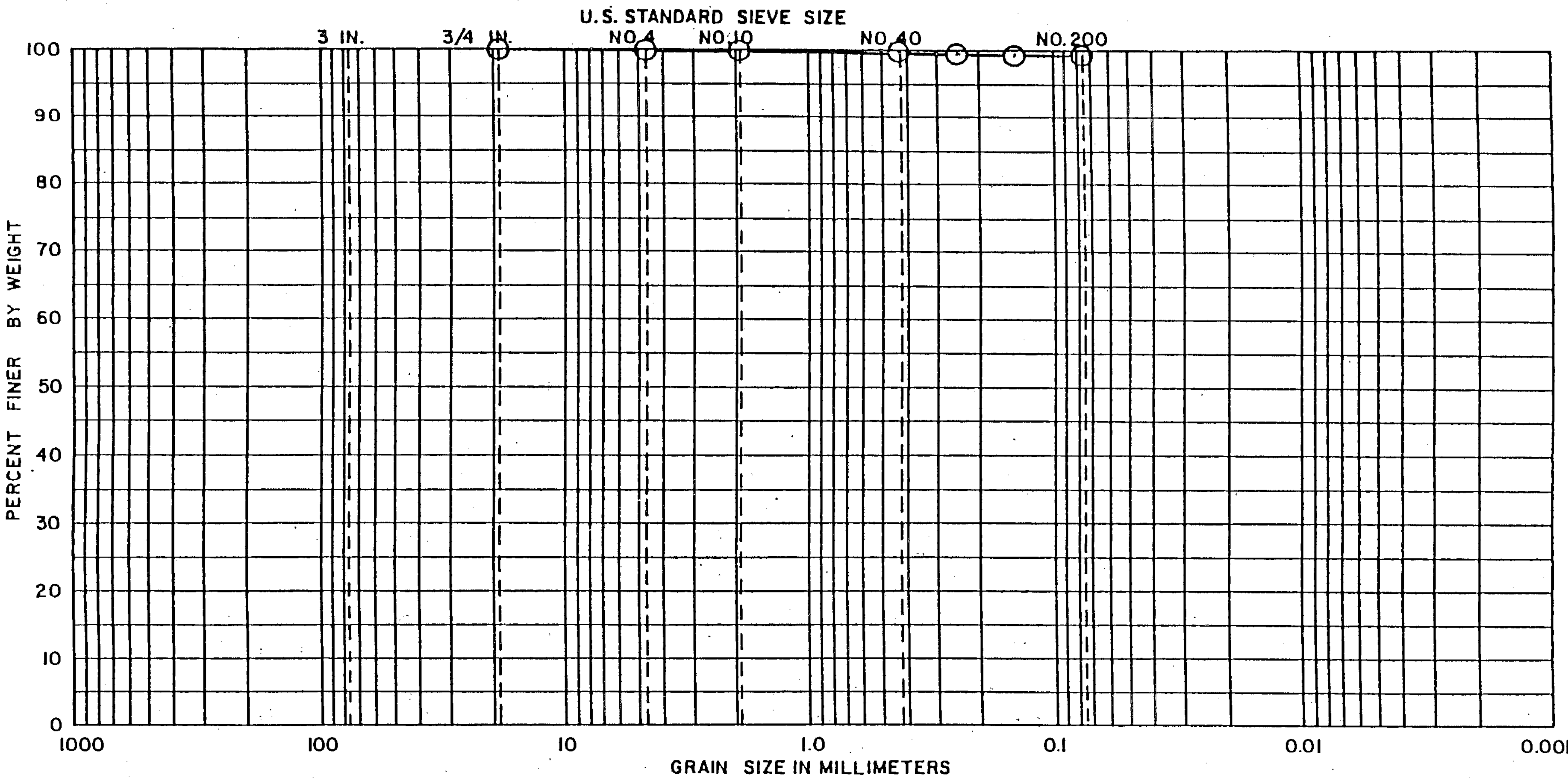
Area: WILMINGTON, DE

Boring No: GF-104A

Date: 12/13/89

Tested By: KLM

CLASSIFICATION TEST-GRADATION CURVES



COBBLES	GRAVEL		SAND			SILT OR CLAY
	Coarse	Fine	Coarse	Medium	Fine	

Sample No.	Depth	Classification	Nat. WC	LL	PL	PI	Gs
S8,9	23.5'-30'	MH	95.2	68.4	47.1	21.3	--

Description and Comments: DK. GRAYISH BROWN-ELASTIC SILT.

CLASSIFICATION TEST-GRADATION CURVES

GANNETT FLEMING GEOTECHNICAL LABORATORY

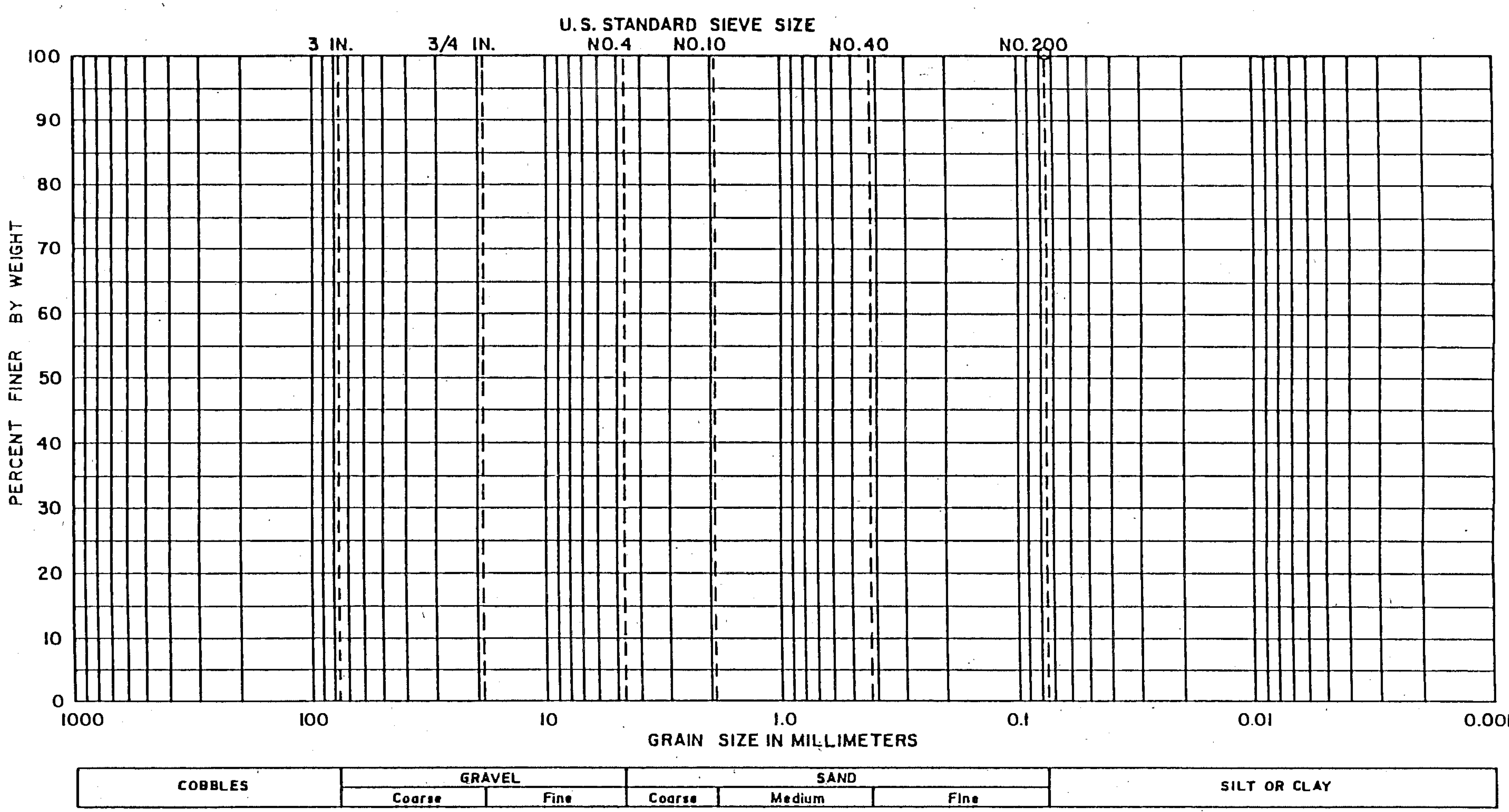
Project: DSWA NSWF-2 PHASE III

Area: WILMINGTON, DE

Boring No: GF-104

Date: 12/13/89

Tested By: KLM



Sample No.	Depth	Classification	Nat.WC	LL	PL	PI	Gs
U-4	35.0'-37.0'	MH	94.2	74.1	48.0	26.1	-

Description and Comments: Gray Elastic Silt

GANNETT FLEMING GEOTECHNICAL LABORATORY

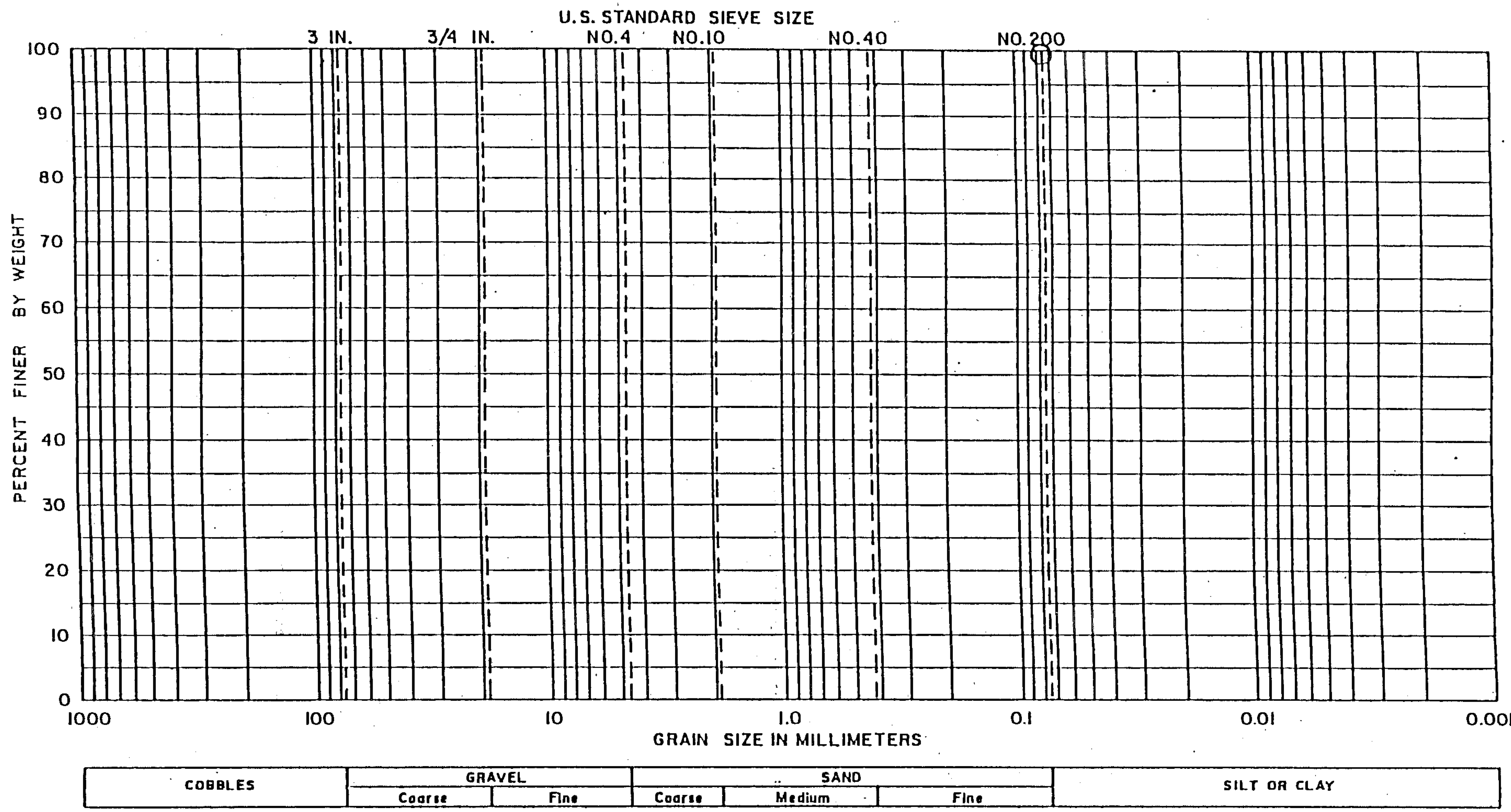
Project: DSWA NSWF - 2 PHASE III

Area: WILMINGTON , DE.

Boring No: GF - 104A

Date: 11/27/89 Tested By: DKN

CLASSIFICATION TEST-GRADATION CURVES



Sample No.	Depth	Classification	Nat.WC	LL	PL	PI	Gs
BAG	0 - 1'	MH		71.2	49.5	21.7	---
		(LL OVEN DRYED)	=	59.8			

Description and Comments:
1) BROWN-ELASTIC SILT.
2) LL OD / LL AD = .84

GANNETT FLEMING GEOTECHNICAL LABORATORY

Project: DSWA, NSWF, PHASE III

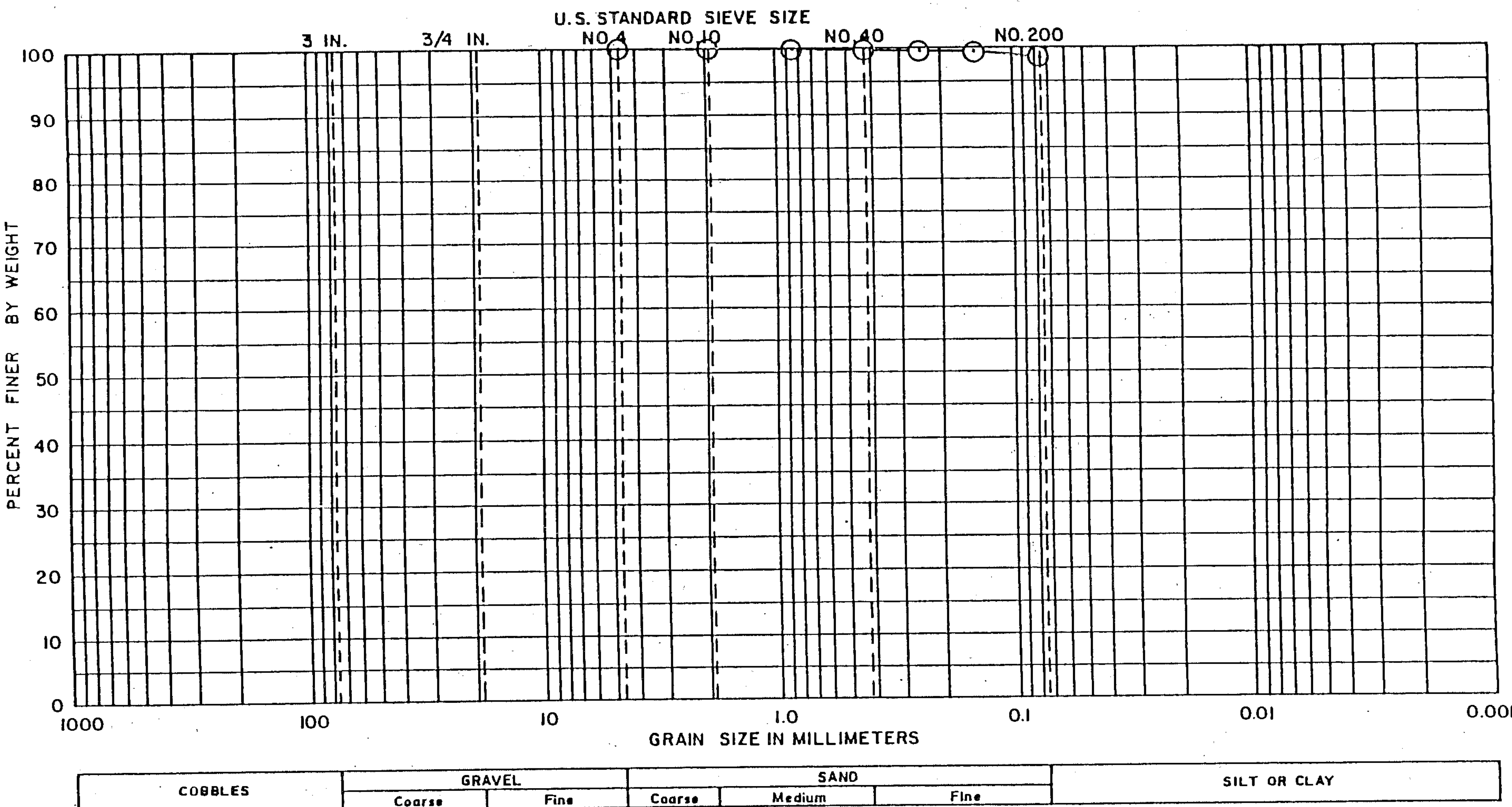
Area: WILMINGTON, DE

Boring No: GF-105

Date: 2/9/90

Tested By: KLM

CLASSIFICATION TEST-GRADATION CURVES



Sample No.	Depth	Classification	Nat.WC	LL	PL	PI	Gs
U-1	5 - 7'	MH		69.5	42.2	27.3	---
		(LL OVEN DRYED)	=	51.8			

Description and Comments:
1) BROWN-ELASTIC SILT.
2) LL OD/LL AD = .75

CLASSIFICATION TEST-GRADATION CURVES

GANNETT FLEMING GEOTECHNICAL LABORATORY

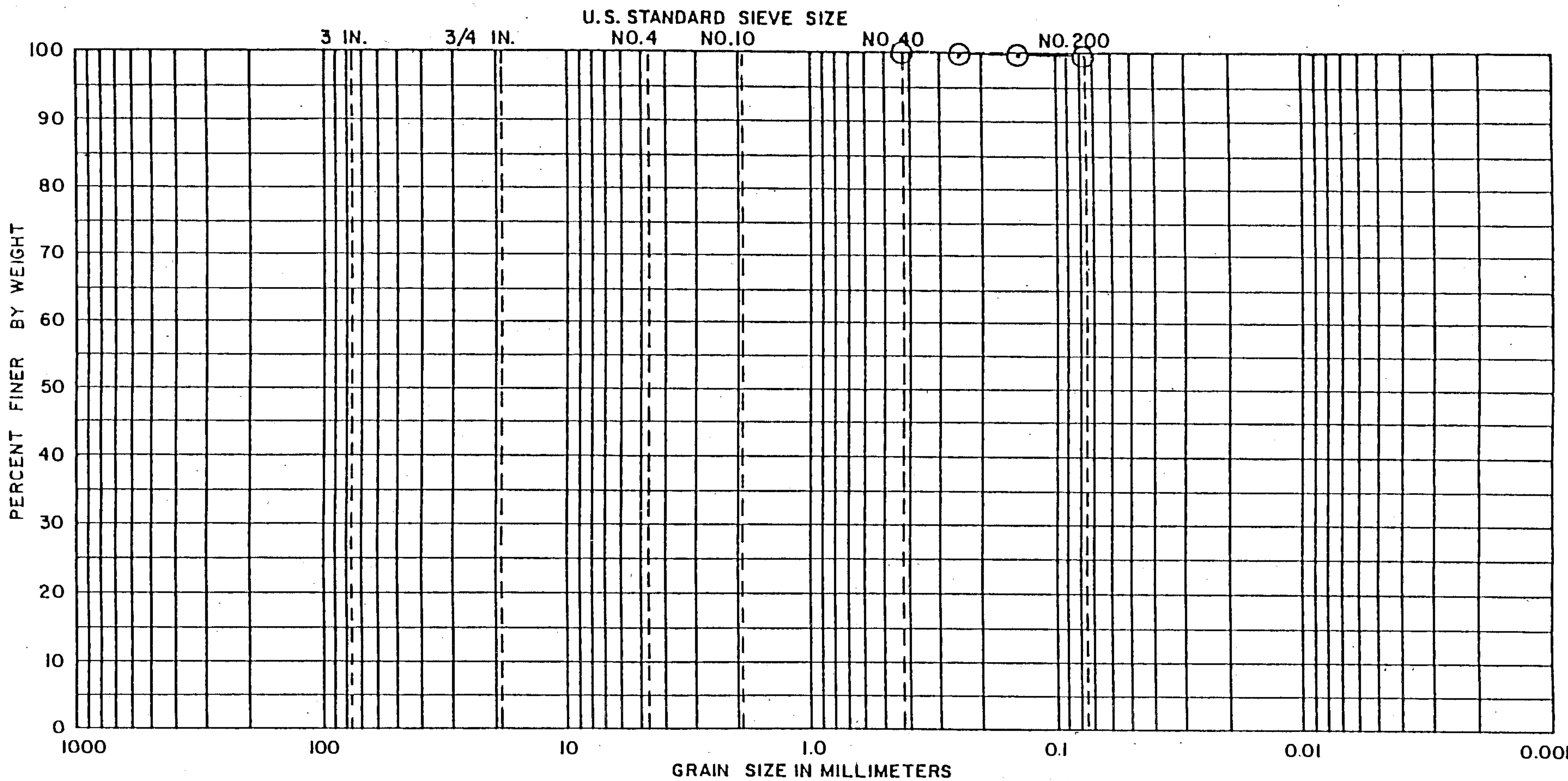
Project: DSWA, NSWF, PHASE III

Area: WILMINGTON, DE

Boring No: GF-105

Date: 1/24/89

Tested By: KLM



COBBLES	GRAVEL		SAND			SILT OR CLAY
	Coarse	Fine	Coarse	Medium	Fine	

Sample No.	Depth	Classification	Nat. WC	LL	PL	PI	Gs
U-2	18.5-20.5'	MH		75.5	45.1	30.4	-
		(LL OVEN DRYED) =		58.8			

Description and Comments:
1) BROWN-ELASTIC SILT.
2) LL OD/LL AD = .78

CLASSIFICATION TEST-GRADATION CURVES

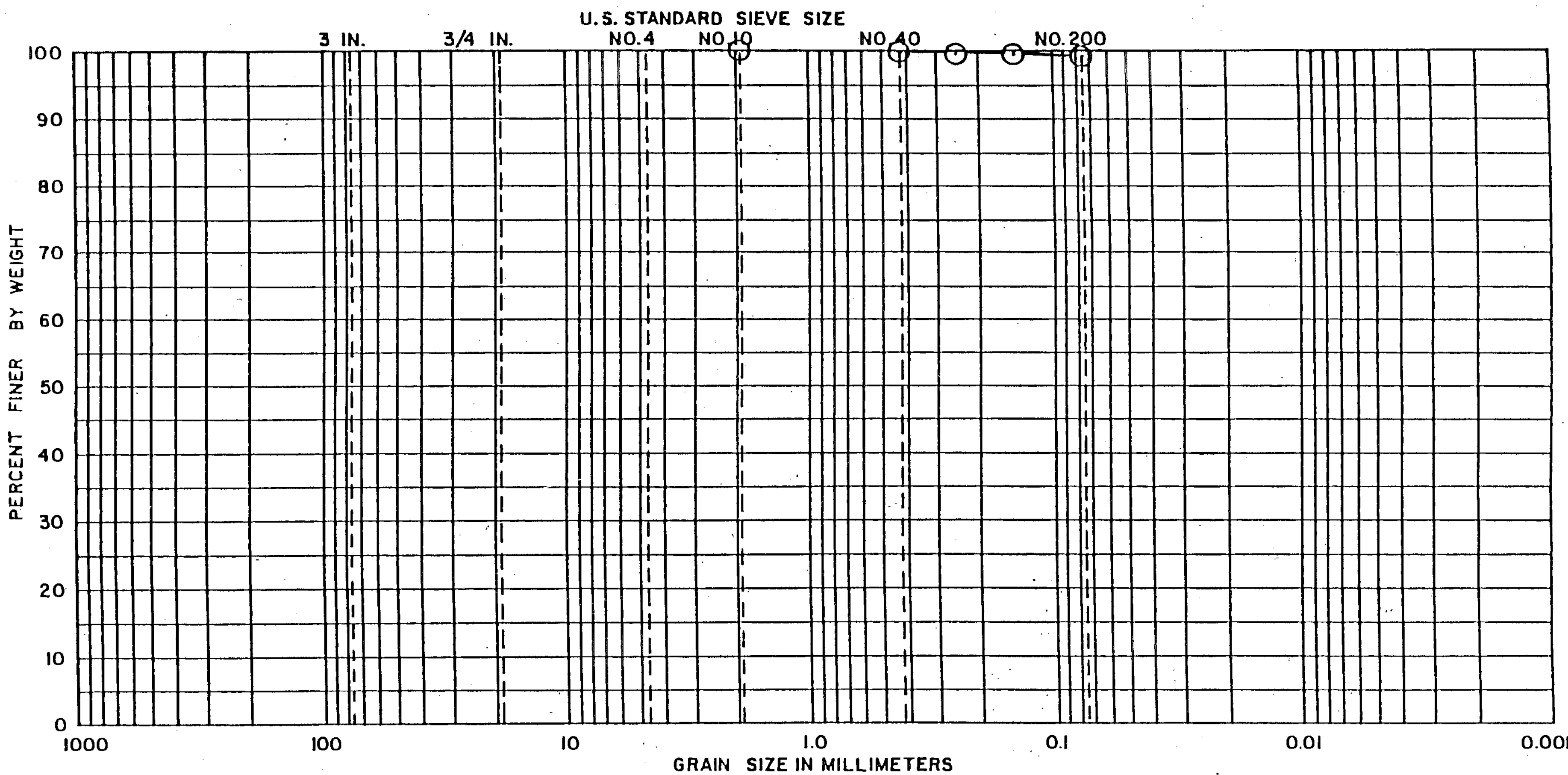
GANNETT FLEMING GEOTECHNICAL LABORATORY

Project: DSWA,NSWF, PHASE III

Area: WILMINGTON, DE

Boring No: GF-105

Date: 1/24/90 | Tested By: KLM



COBBLES	GRAVEL		SAND			SILT OR CLAY
	Coarse	Fine	Coarse	Medium	Fine	

Sample No.	Depth	Classification	Nat.WC	LL	PL	PI	Gs
S2,3	2' - 6'	MH	103.6	70.2	46.2	24.0	--

Description and Comments: DK. GRAYISH BROWN-ELASTIC SILT.

CLASSIFICATION TEST-GRADATION CURVES

GANNETT FLEMING GEOTECHNICAL LABORATORY

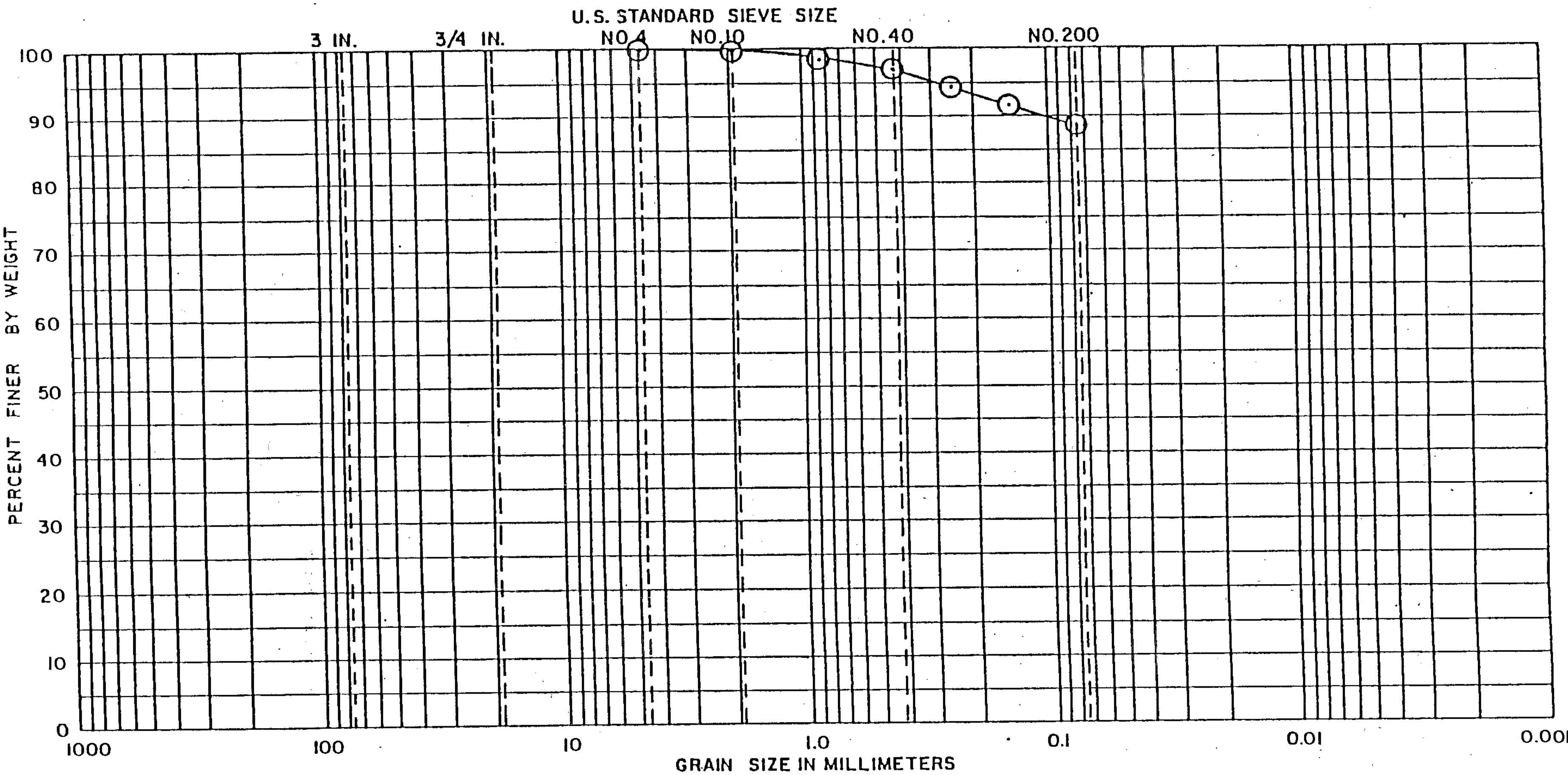
Project: DSWA,NSWF, PHASE III

Area: WILMINGTON, DE

Boring No: GF-106

Date: 12/19/89

Tested By: KLM



COBBLES	GRAVEL		SAND			SILT OR CLAY
	Coarse	Fine	Coarse	Medium	Fine	

Sample No.	Depth	Classification	Nat.WC	LL	PL	PI	Gs
BAG	0 - 1'	MH		53.2	38.1	15.1	-
		(LL OVEN DRYED)	=	44.7			

Description and Comments:
1) BROWN-ELASTIC SILT.
2) LL OD / LL AD = .84

GANNETT FLEMING GEOTECHNICAL LABORATORY

Project: DSWA, NSWF, PHASE III

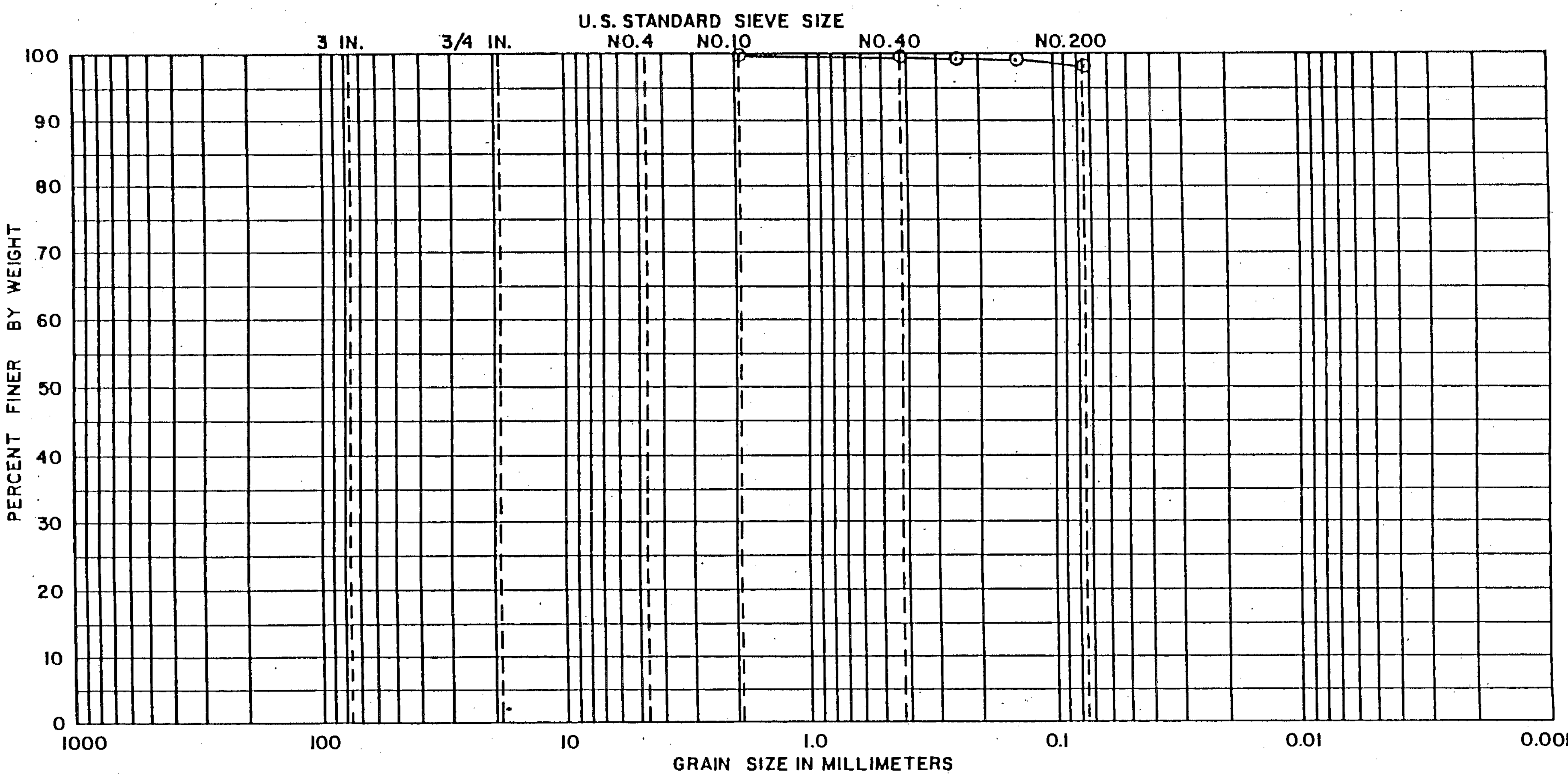
Area: WILMINGTON, DE

Boring No: GF-106

Date: 2/9/90

Tested By: KLM

CLASSIFICATION TEST-GRADATION CURVES



COBBLES	GRAVEL		SAND			SILT OR CLAY
	Coarse	Fine	Coarse	Medium	Fine	

Sample No.	Depth	Classification	Nat.WC	L.L.	PL	PI	Gs
U-1	8.0'-10.0'	MH	-	72.0	40.4	31.6	-

Description and Comments: Gray Elastic Silt

GANNETT FLEMING GEOTECHNICAL LABORATORY

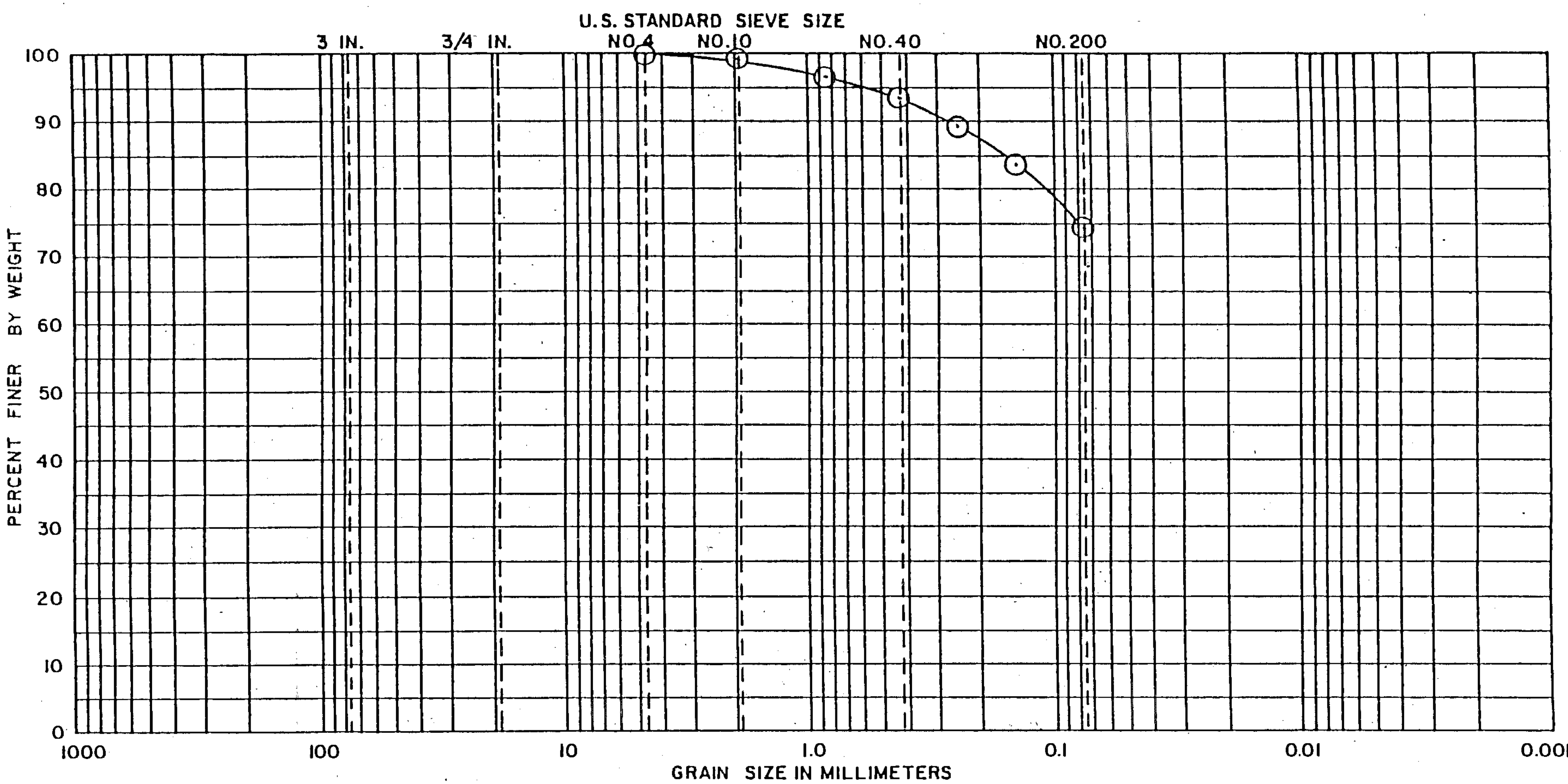
Project: DSWA NSWF PHASE III

Area: WILMINGTON , DE

Boring No: GF - 106

Date: 12/15/89 Tested By: DKN

CLASSIFICATION TEST—GRADATION CURVES



COBBLES	GRAVEL		SAND			SILT OR CLAY
	Coarse	Fine	Coarse	Medium	Fine	

Sample No.	Depth	Classification	Nat. WC	LL	PL	PI	Gs
U-1	8' - 10'	ML		-	N/P	-	---
(PERM.)							

Description and Comments: 1) BLACK-SILT WITH SAND.

GANNETT FLEMING GEOTECHNICAL LABORATORY

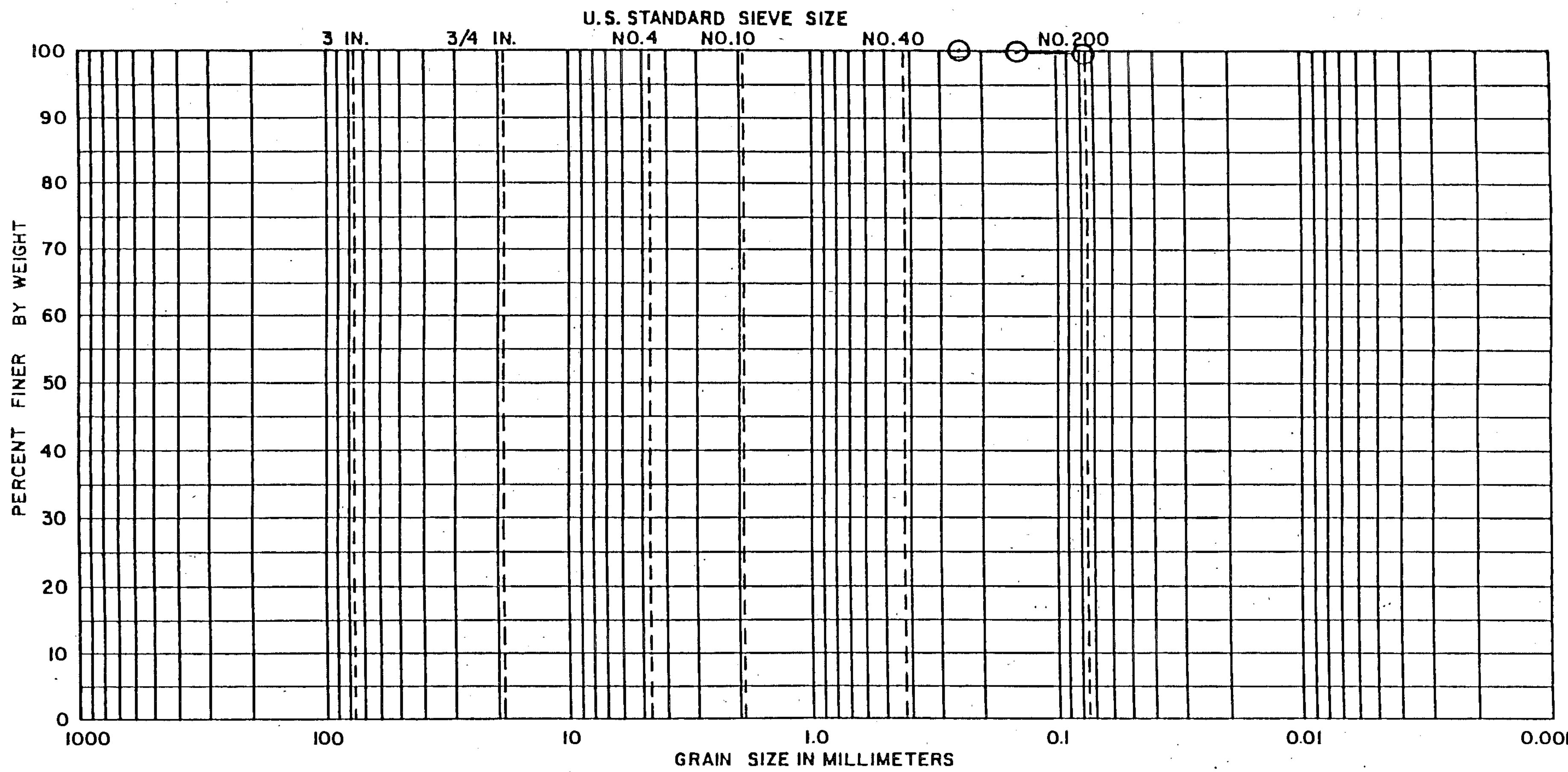
Project: DSWA, NSWF, PHASE III

Area: WILMINGTON, DE

Boring No: GF-106

Date: 1/24/90 Tested By: KLM

CLASSIFICATION TEST-GRADATION CURVES



COBBLES	GRAVEL		SAND			SILT OR CLAY
	Coarse	Fine	Coarse	Medium	Fine	

Sample No.	Depth	Classification	Nat.WC	LL	PL	PI	Gs
S10,11	33.5'-40'	MH	90.8	72.3	48.2	24.1	--

Description and Comments: DK. GRAYISH BROWN-ELASTIC SILT.

CLASSIFICATION TEST-GRADATION CURVES

GANNETT FLEMING GEOTECHNICAL LABORATORY

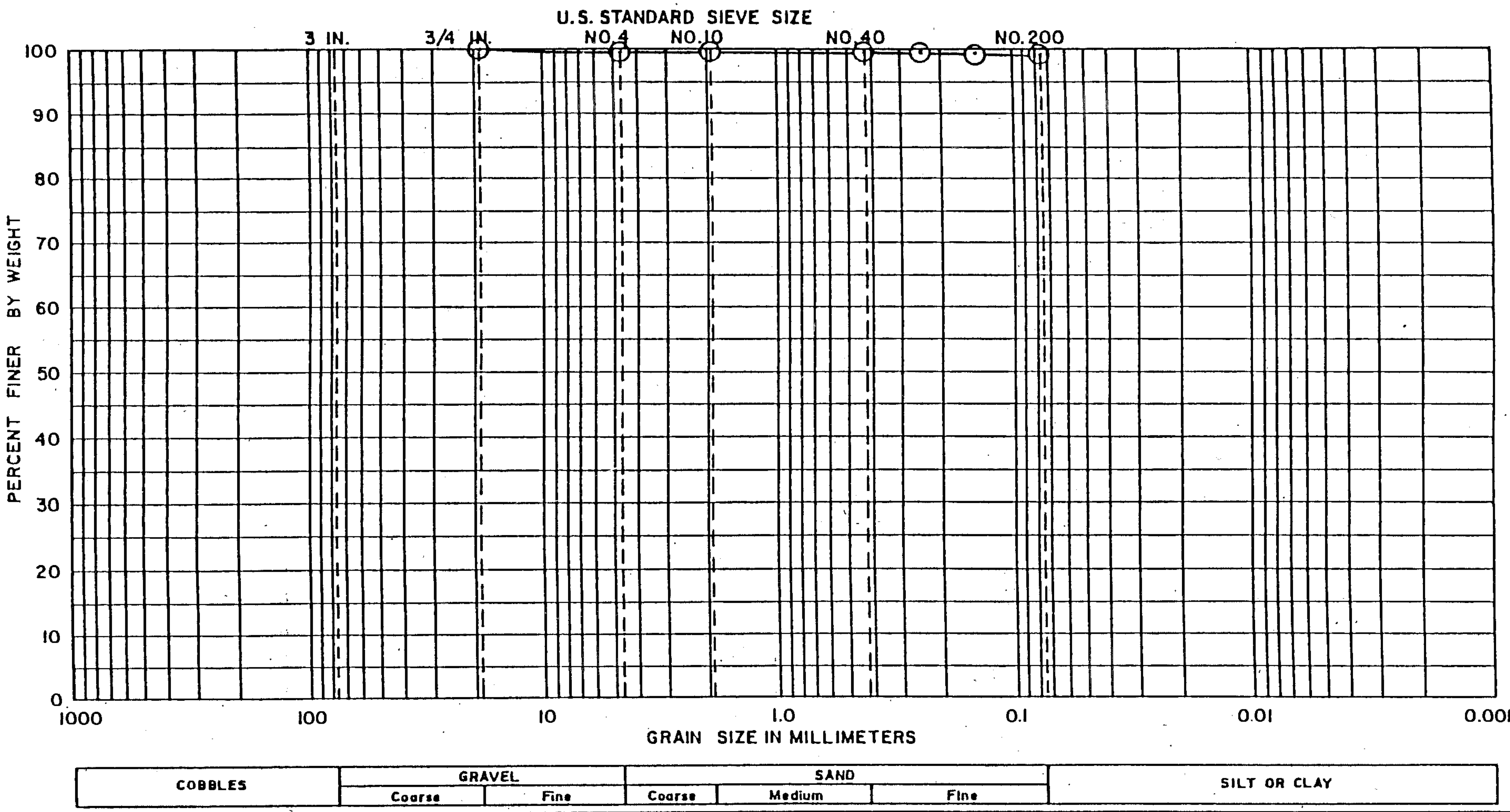
Project: DSWA, NSWF, PHASE III

Area: WILMINGTON, DE

Boring No: GF-106

Date: 12/19/89

Tested By: KLM



Sample No.	Depth	Classification	Nat.WC	LL	PL	PI	Gs
S13,14	48.5'-55'	MH	70.1	61.8	41.8	20.0	--

Description and Comments: DK. GRAYISH BROWN-ELASTIC SILT.

CLASSIFICATION TEST-GRADATION CURVES

GANNETT FLEMING GEOTECHNICAL LABORATORY

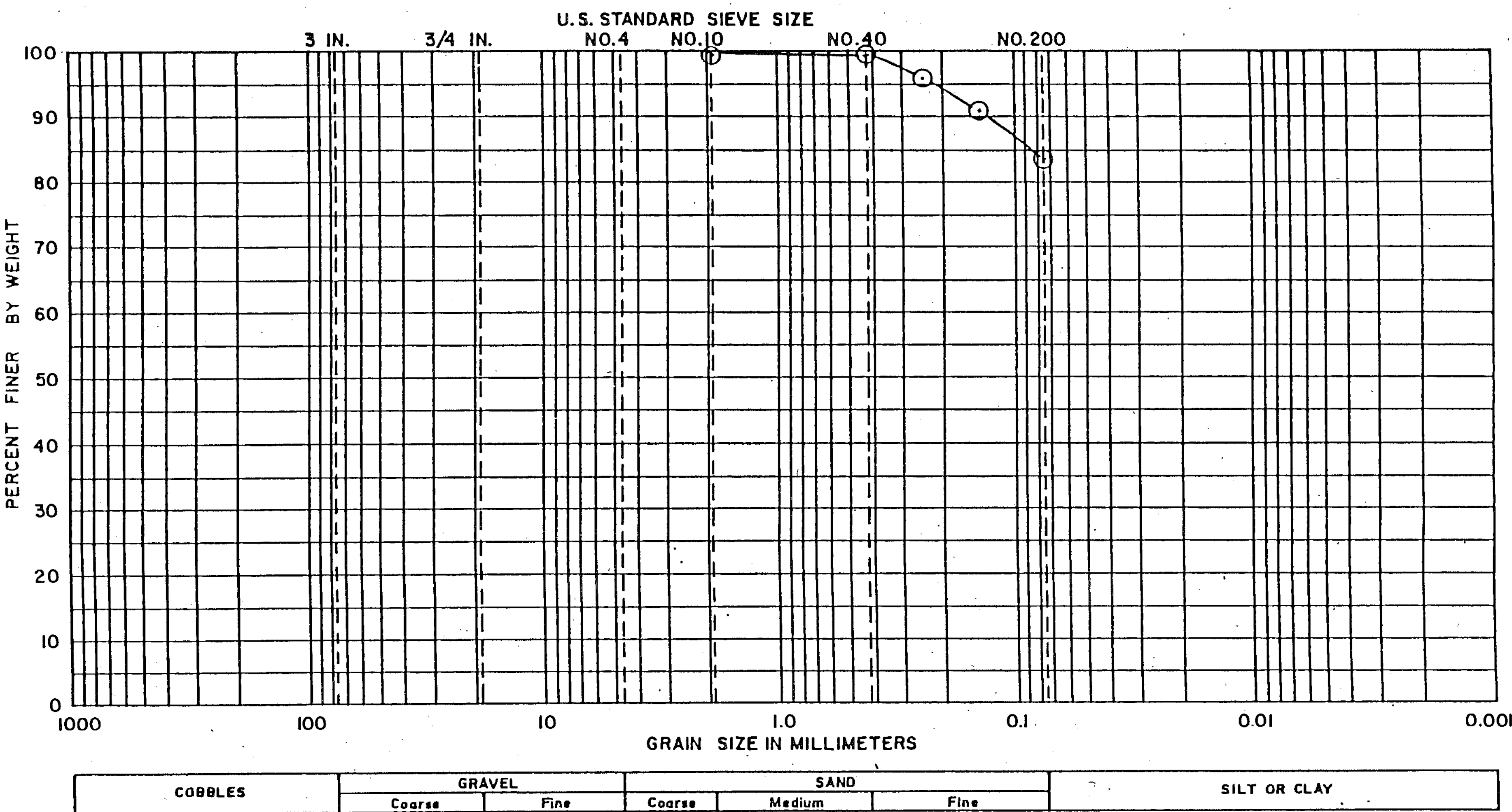
Project: DSWA, NSWF, PHASE III

Area: WILMINGTON, DE

Boring No: GF-106

Date: 12/19/89

Tested By: KLM



Sample No.	Depth	Classification	Nat.WC	LL	PL	PI	Gs
GF-107/S-5 ,	8.0'-15.0'	ML	47.1	--	N/P	--	--
S-6							

Description and Comments: Gray Silt with Sand

GANNETT FLEMING GEOTECHNICAL LABORATORY

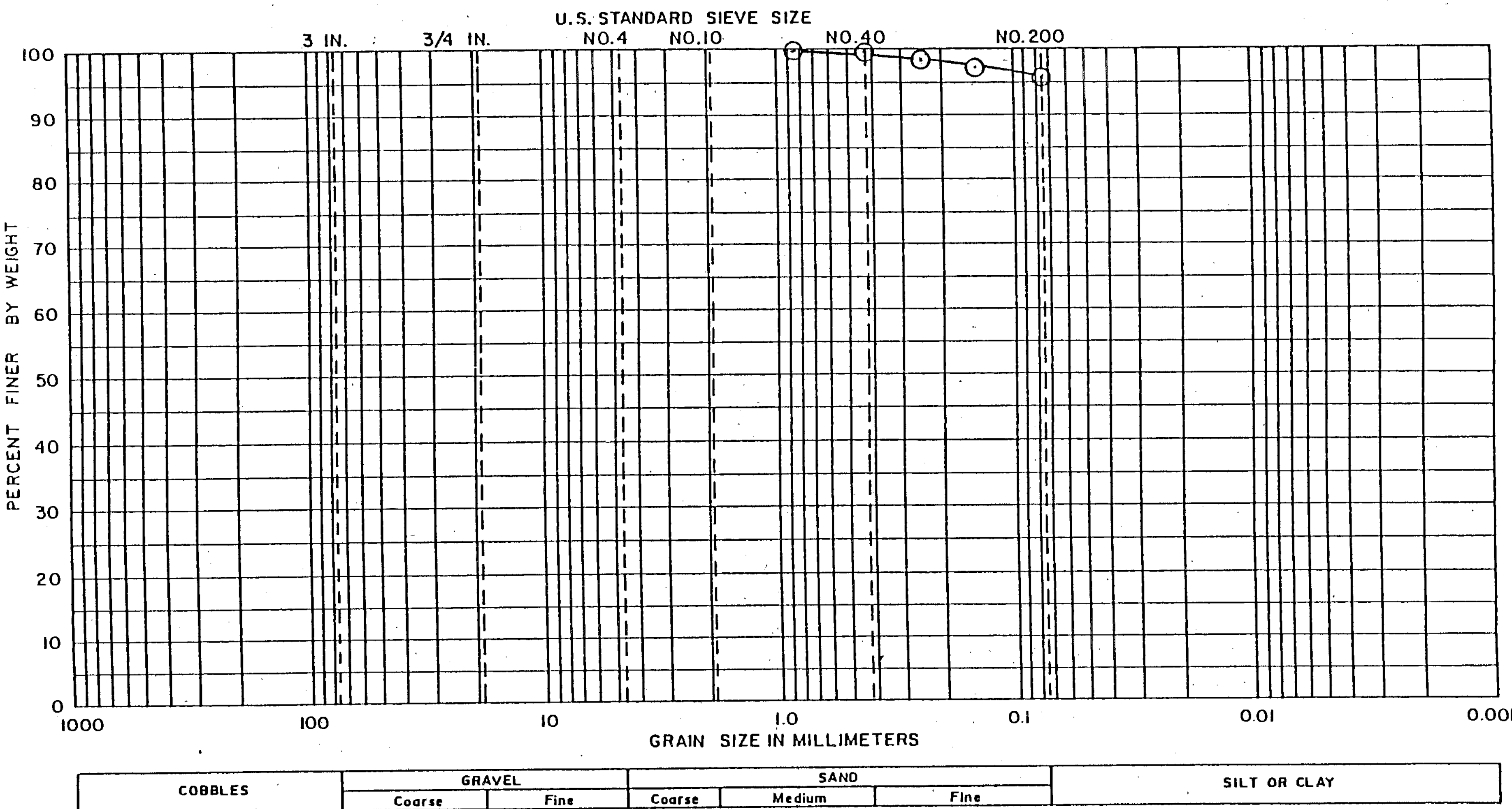
Project: DSWA NSWF - 2 PHASE III

Area: WILMINGTON, DE.

Boring No: GF-107 / S-5 & S-6

Date: Oct. 27, 1989 | Tested By: K.A. Abdolos

CLASSIFICATION TEST-GRADATION CURVES



Sample No.	Depth	Classification	Nat. WC	LL	PL	PI	Gs
GF-107/U-1	20.0'-22.5'	MH	83.0	66.3	43.3	23.0	—
		LL ovendried =		56.3			

Description and Comments:
1) Grey elastic silt
2) LL ovendried/LL not dried = 0.85

CLASSIFICATION TEST-GRADATION CURVES

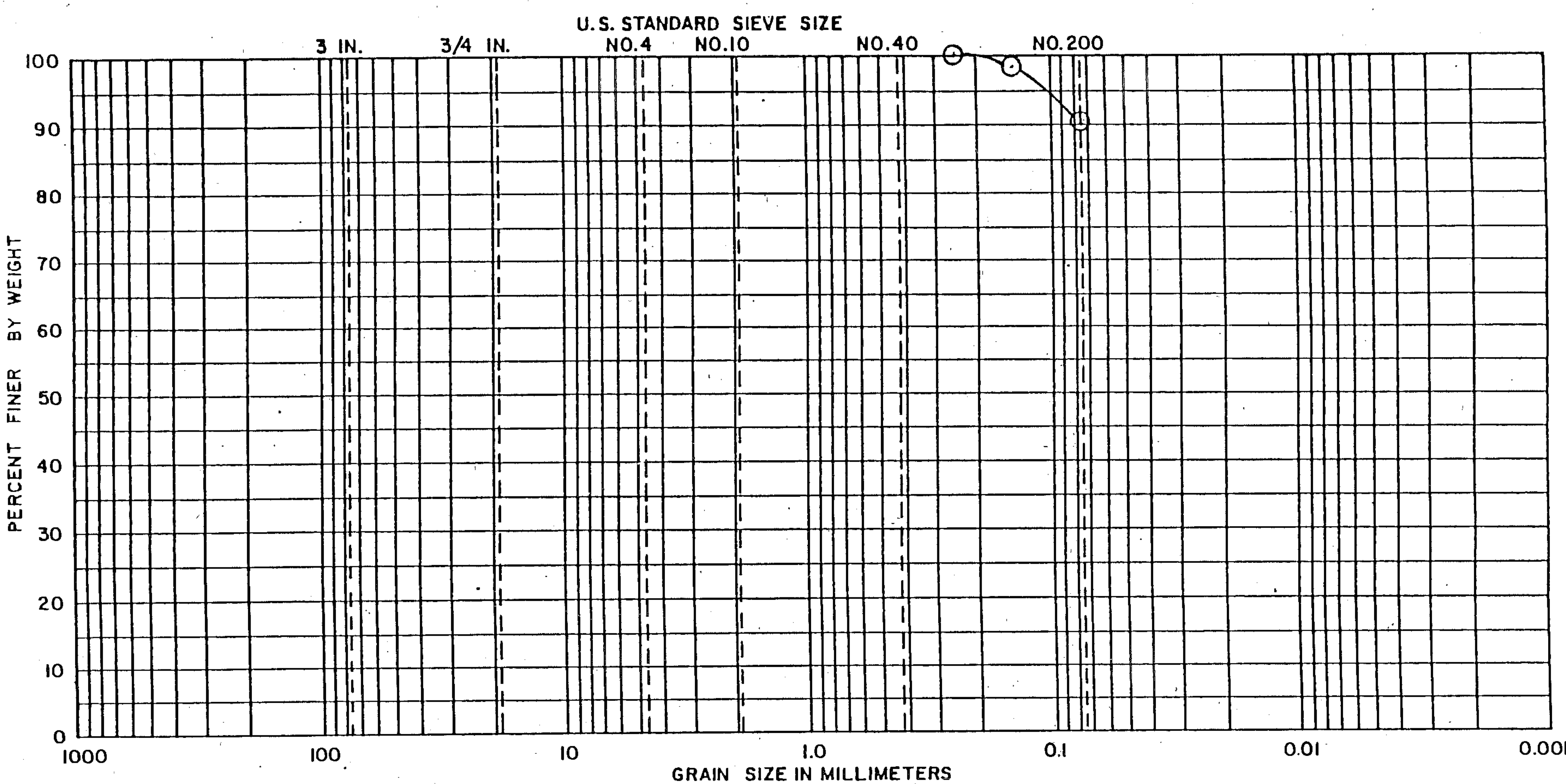
GANNETT FLEMING GEOTECHNICAL LABORATORY

Project: DSWA , NSWF , PHASE III

Area: WILMINGTON, DE.

Boring No: GF-107 / U-1

Date: Jan. 29, 1990 . Tested By: K.A. Abdolos



COBBLES	GRAVEL		SAND			SILT OR CLAY
	Coarse	Fine	Coarse	Medium	Fine	

Sample No.	Depth	Classification	Nat. WC	LL	PL	PI	Gs
GF-107/S-9	23.5'-30.0'	ML	48.5	--	N/P	--	--
S-10							

Description and Comments: Gray Silt

CLASSIFICATION TEST-GRADATION CURVES

GANNETT FLEMING GEOTECHNICAL LABORATORY

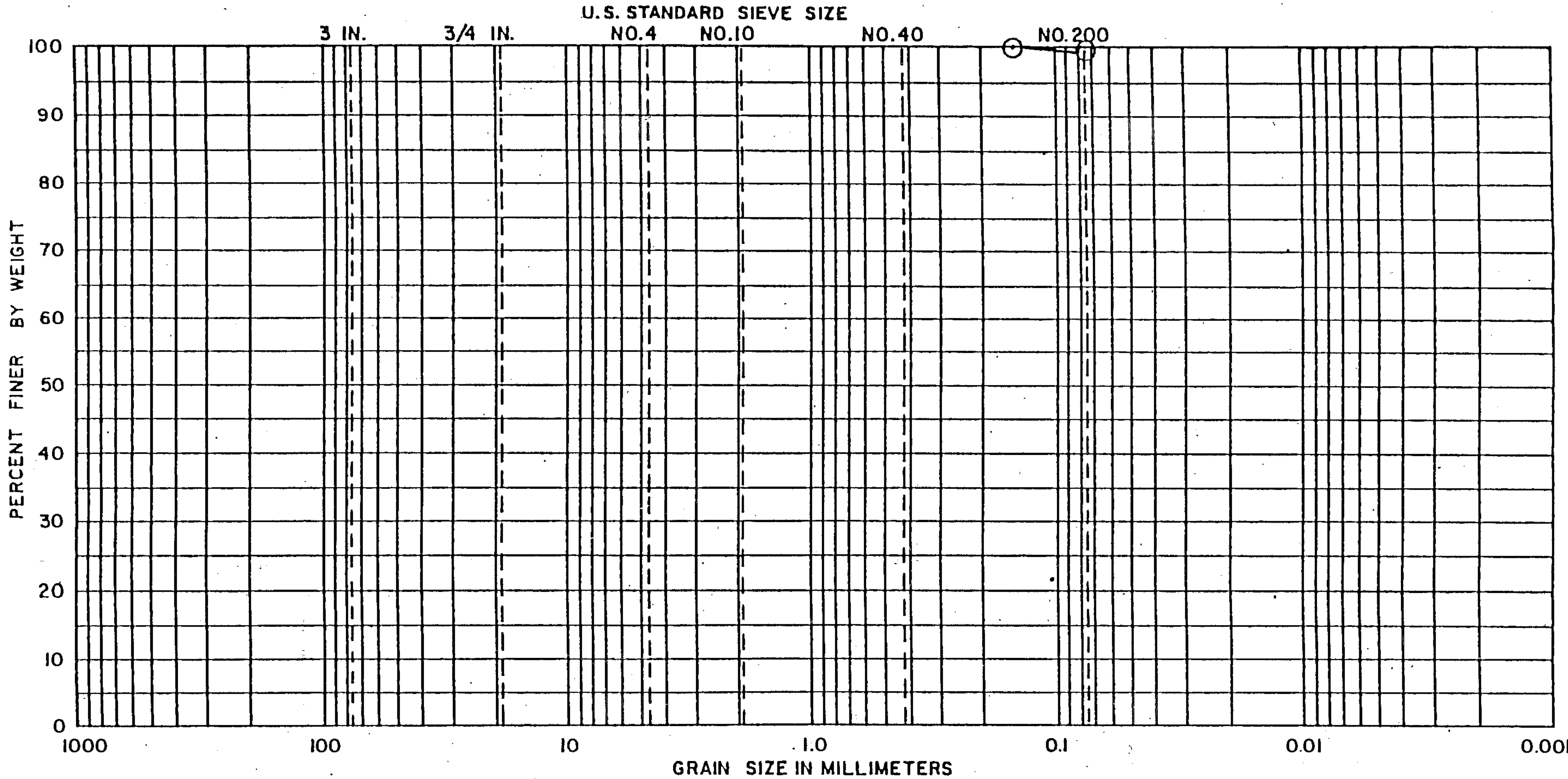
Project: DSWA NSWF - 2 PHASE III

Area: WILMINGTON, DE.

Boring No: GF-107 / S-9 & S-10

Date: Oct. 27,1989

Tested By: K.A. Abdolos



COBBLES	GRAVEL		SAND			SILT OR CLAY
	Coarse	Fine	Coarse	Medium	Fine	

Sample No.	Depth	Classification	Nat.WC	LL	PL	PI	Gs
GF-107/S-11	33.5'-40.0'	MH	75.8	68.7	47.4	21.3	--
S-12							

Description and Comments: Gray Elastic Silt

CLASSIFICATION TEST—GRADATION CURVES

GANNETT FLEMING GEOTECHNICAL LABORATORY

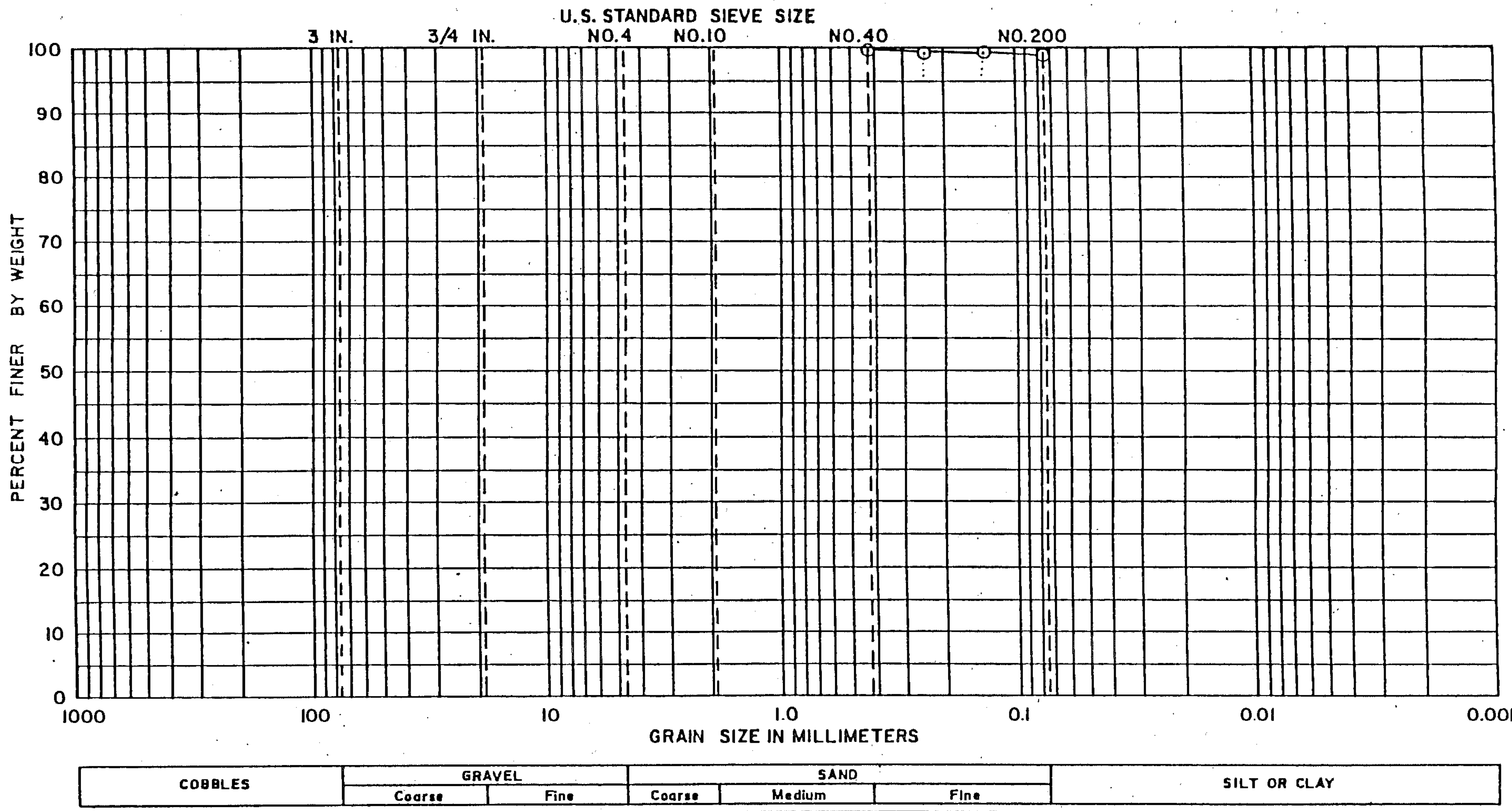
Project: DSWA NSWF - 2 PHASE III

Area: WILMINGTON, DE.

Boring No: GF-107 / S-11 & S-12

Date: Oct. 27, 1989

Tested By: K.A.Abdolos



Sample No.	Depth	Classification	Nat. WC	LL	PL	PI	Gs
S14	48.0'-50.0'	MH	64.0	61.4	41.9	19.5	-
S16	53.5'-55.0'						

Description and Comments: Gray Elastic Silt

CLASSIFICATION TEST-GRADATION CURVES

GANNETT FLEMING GEOTECHNICAL LABORATORY

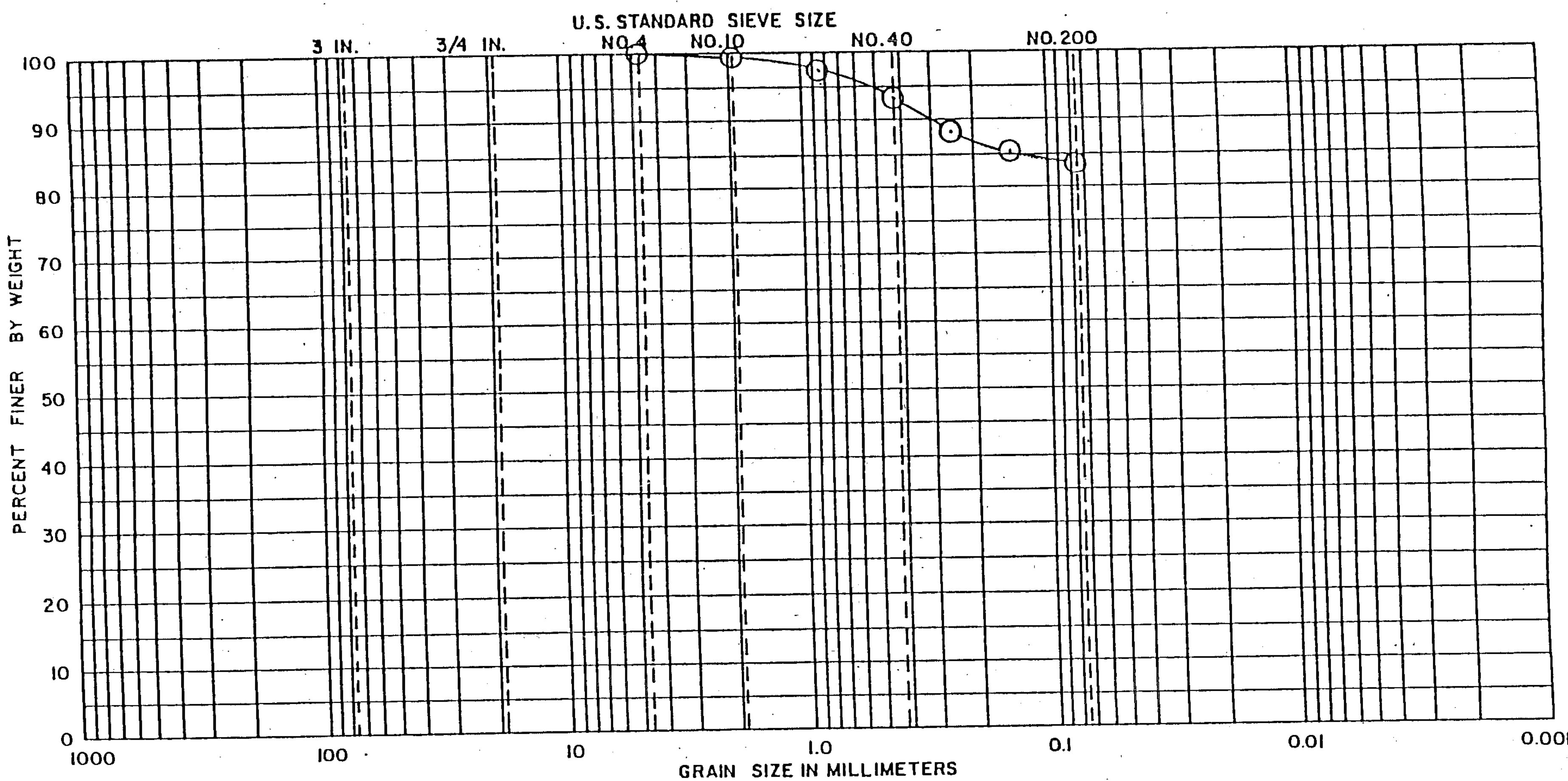
Project: DSWA NSWF - 2 PHASE III

Area: WILMINGTON , DE

Boring No: GF-107

Date: 10/27/89

Tested By: DKN



COBBLES	GRAVEL		SAND			SILT OR CLAY
	Coarse	Fine	Coarse	Medium	Fine	

Sample No.	Depth	Classification	Nat.WC	L.L.	PL	PI	Gs
U-2	55'-57'	ML	55.1	49.1	32.9	16.2	-
		(LL OVEN DRYED)	=	41.6			

Description and Comments:

1) BROWN-SILT WITH SAND.
2) LL OD / LL AD = .85

GANNETT FLEMING GEOTECHNICAL LABORATORY

Project: DSWA, NSWF, PHASE III

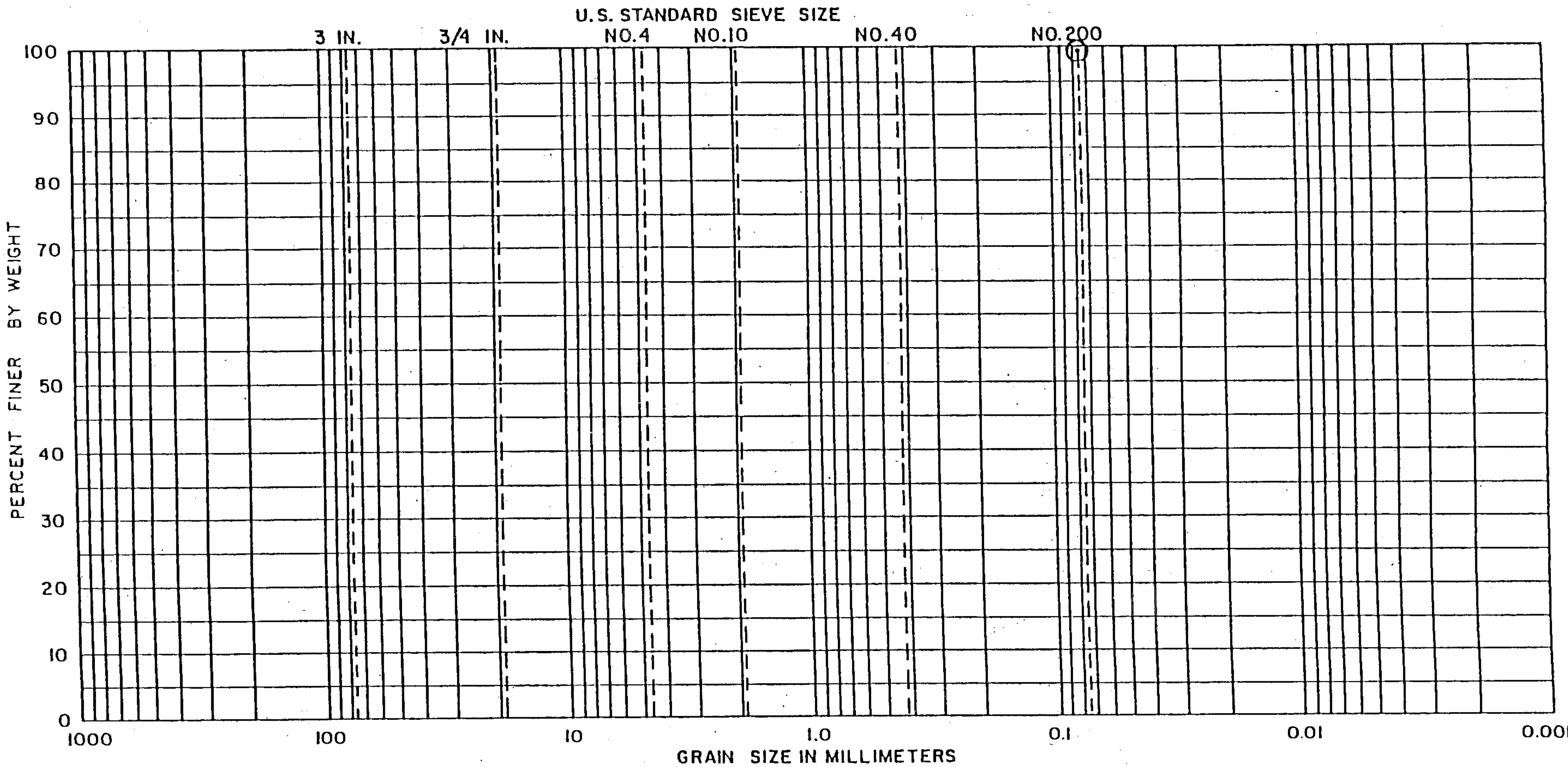
Area: WILMINGTON, DE

Boring No: GF-107

Date: 2/1/90

Tested By: KLM

CLASSIFICATION TEST-GRADATION CURVES



COBBLES	GRAVEL		SAND			SILT OR CLAY
	Coarse	Fine	Coarse	Medium	Fine	

Sample No.	Depth	Classification	Nat. WC	LL	PL	PI	Gs
BAG	0 - 1'	MH		55.6	38.3	17.3	--
		(LL OVEN DRYED)	=	45.8			

Description and Comments:
1) BROWN-ELASTIC SILT.
2) LL OD / LL AD = .82

GANNETT FLEMING GEOTECHNICAL LABORATORY

Project: DSWA, NSWF, PHASE III

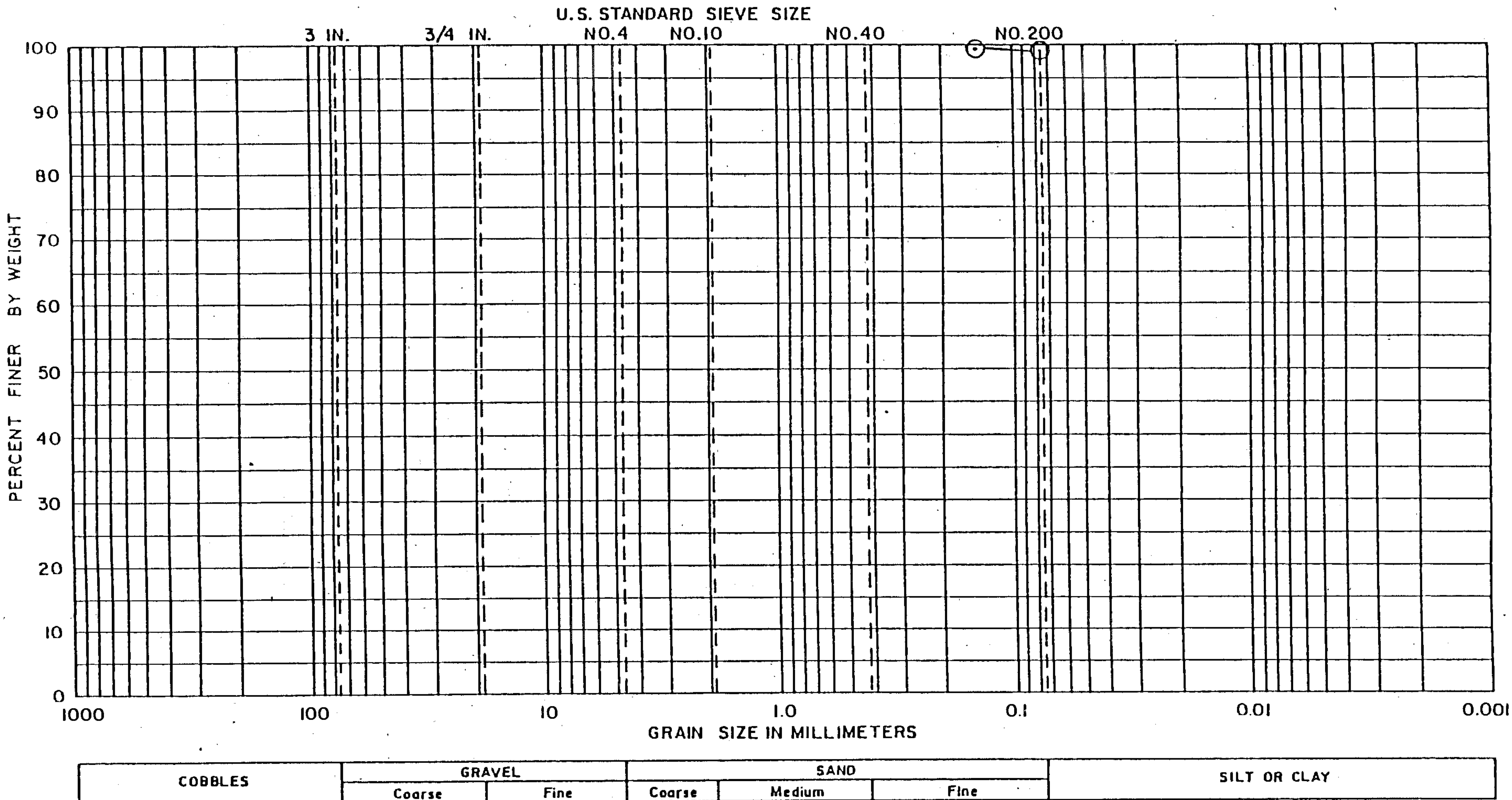
Area: WILMINGTON, DE

Boring No: GF-108

Date: 2/9/90

Tested By: KLM

CLASSIFICATION TEST-GRADATION CURVES



Sample No.	Depth	Classification	Nat. WC	LL	PL	PI	Gs
GF-108/U-1	8.0'-10.0'	MH	93.4	67.8	42.1	25.7	--
		LL ovendried =		55.9			

Description and Comments:
1) Grey elastic silt
2) LL ovendried / LL not dried = 0.82

GANNETT FLEMING GEOTECHNICAL LABORATORY

Project: DSWA , NSWF, PHASE III

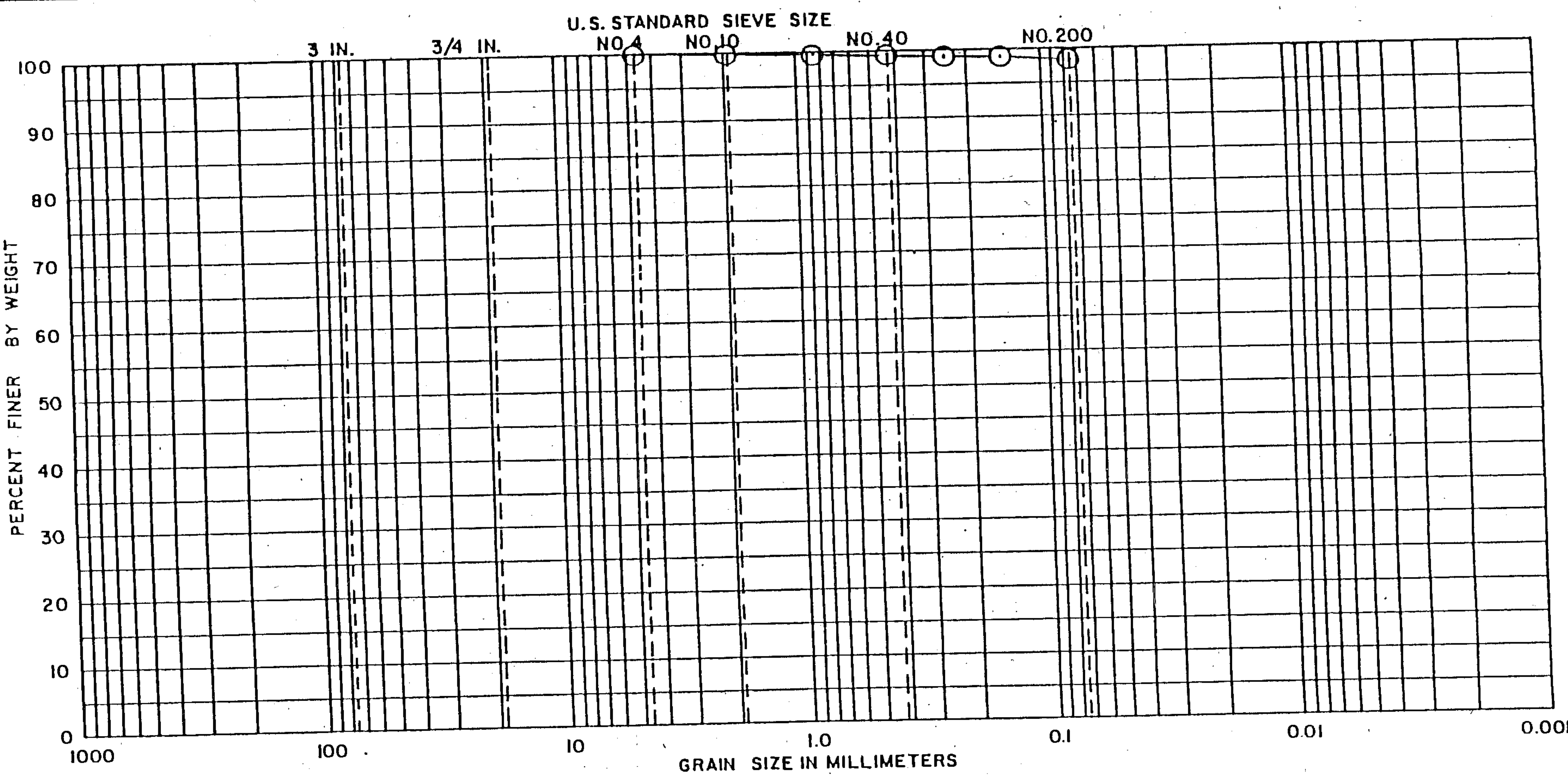
Area: WILMINGTON,DE.

Boring No: GF-108 / U-1

Date: Jan, 29, 1990

Tested By: K.A.Abdolos

CLASSIFICATION TEST-GRADATION CURVES



COBBLES	GRAVEL		SAND			SILT OR CLAY
	Coarse	Fine	Coarse	Medium	Fine	

Sample No.	Depth	Classification	Nat.WC	LL	PL	PI	Gs
U-2	15'-17'	MH	75.0	59.4	40.3	19.1	---
		(LL OVEN DRYED)	=	49.5			

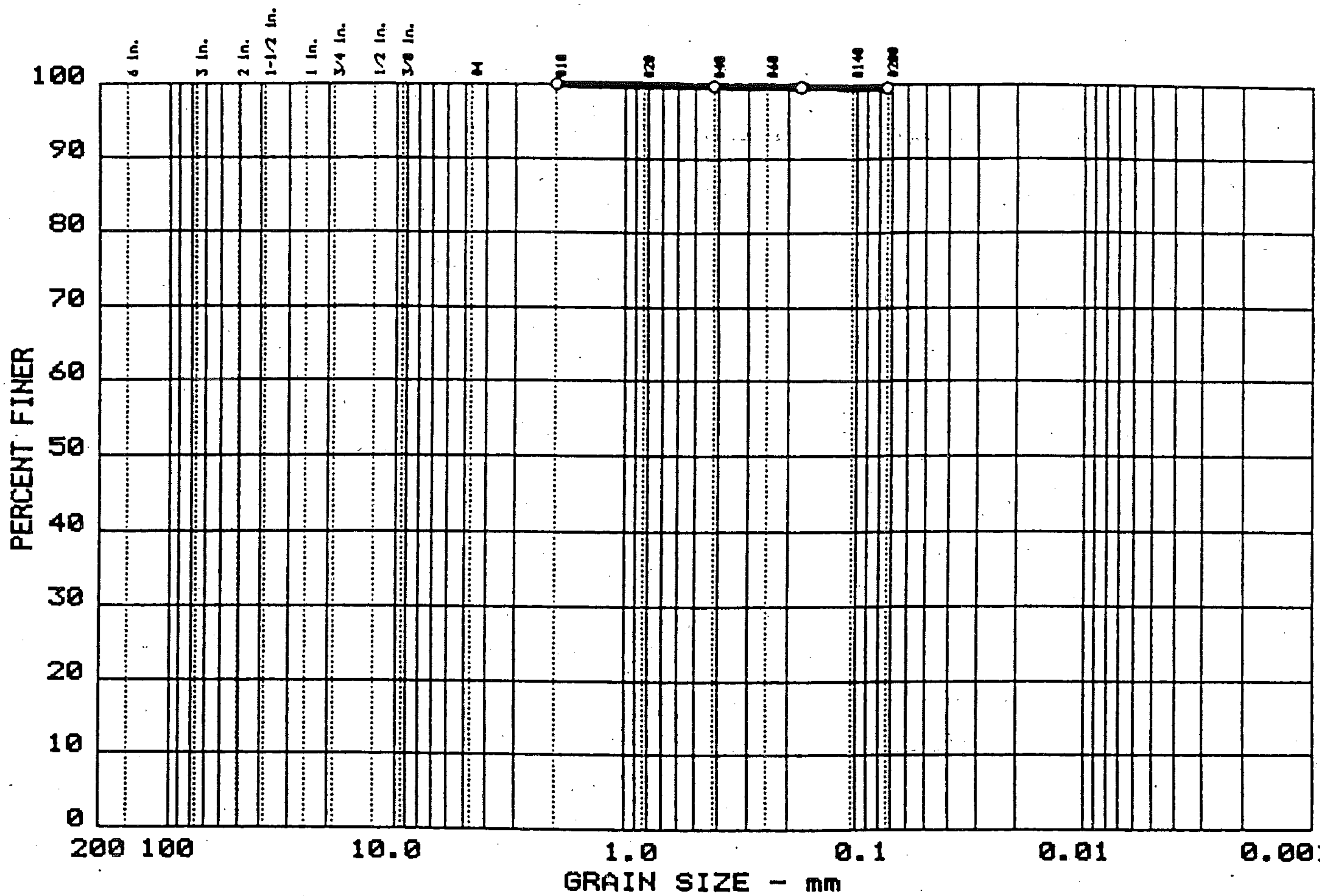
GANNETT FLEMING GEOTECHNICAL LABORATORY

Project: DSWA, NSWF, PHASE III

Area: WILMINGTON DE

Description and 1) BROWN PLASTIC SILT

GRAIN SIZE DISTRIBUTION TEST REPORT



	%+75mm	% GRAVEL	% SAND	% SILT	% CLAY
o	0.0	0.0	0.5	99.5	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
o	63	9								

MATERIAL DESCRIPTION	USCS	AASHTO
o Dark Grey Clayey Silt	MH	

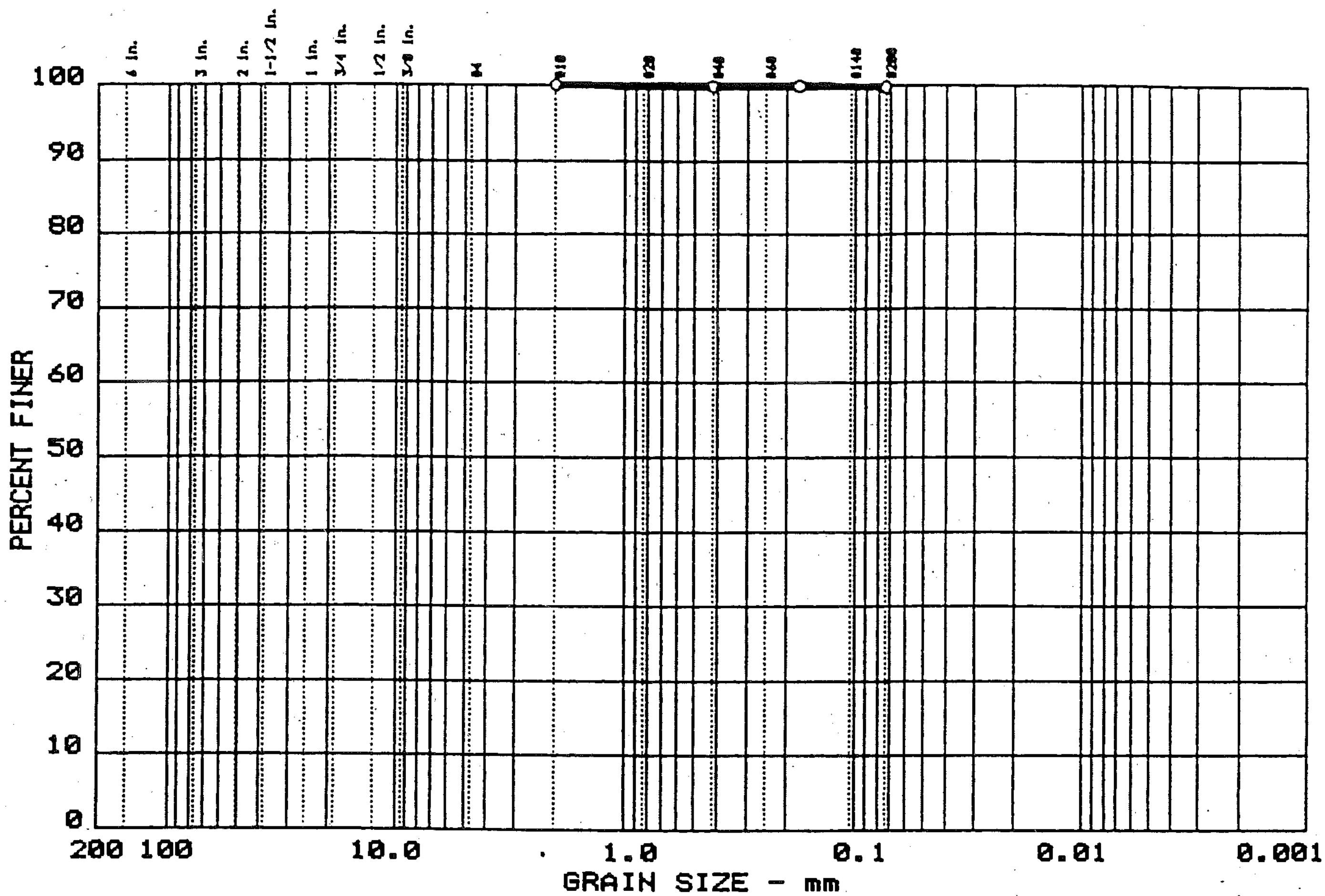
Project No.: 5411
Project: DSWA - Phase III
o Location: Boring: GF-108; S: U-3; 28.5'-30.5'

Date: 1-29-90

GRAIN SIZE DISTRIBUTION TEST REPORT

Remarks:
Project Location:
Wilmington, Delaware
Client:
Gannett-Fleming

GRAIN SIZE DISTRIBUTION TEST REPORT



	%+75mm	% GRAVEL	% SAND	% SILT	% CLAY
○	0.0	0.0	0.2	99.8	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	55	11								

MATERIAL DESCRIPTION	USCS	AASHTO
○ Dark Grey Clayey Silt	MH	

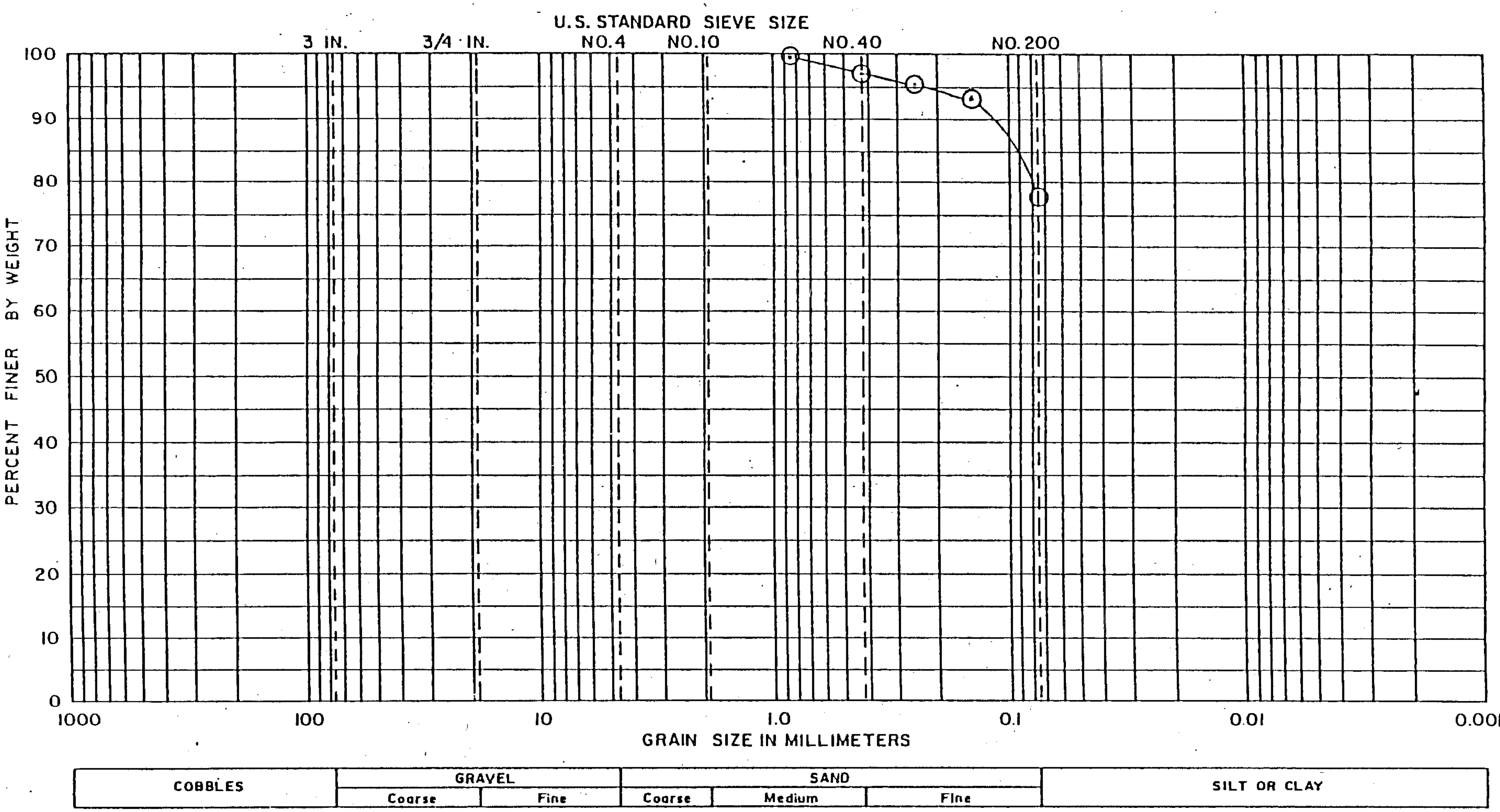
Project No.: 5411
Project: DSWA - Phase III
○ Location: Boring: GF-108; S: U-4; 49.0'- 51.0'

Date: 1-29-90

GRAIN SIZE DISTRIBUTION TEST REPORT
F. T. KITLINSKI & ASSOCIATES, INC.

Remarks:
Project Location:
Wilmington, Delaware
Client:
Gannett-Fleming

Figure No.2



Sample No.	Depth	Classification	Nat. WC	LL	PL	PI	Gs
GF-109/U-1	14.0'-16.0'	ML	35.2	37.7	26.3	11.4	--
		LL ovendried =		31.0			

Description and Comments:
1) Grey silt with sand
2) LL ovendried / LL not dried = 0.82

GANNETT FLEMING GEOTECHNICAL LABORATORY

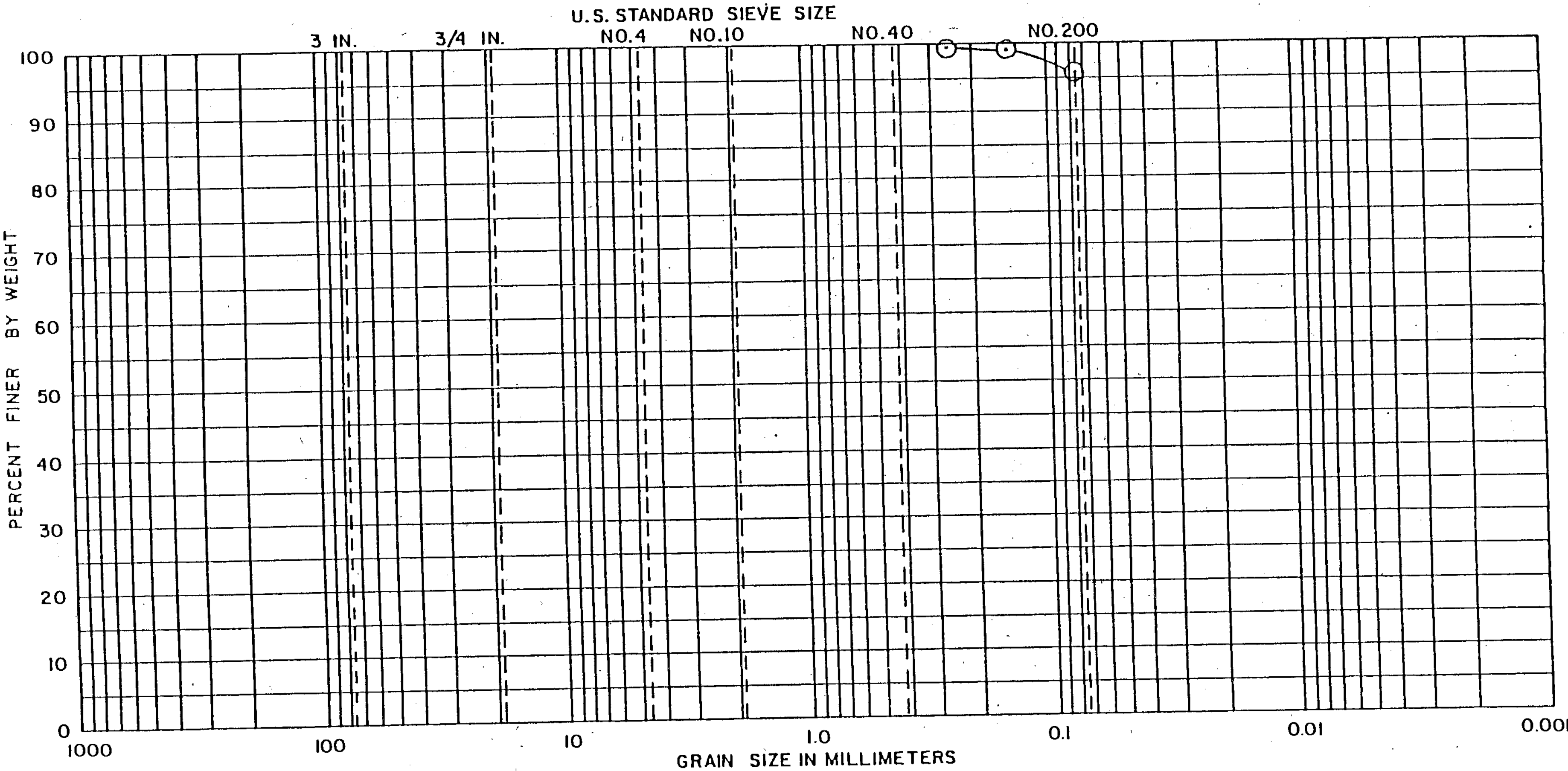
Project: DSWA / NSWF , PHASE III

Area: WILMINGTON, DE.

Boring No: GF-109 / U-1 (Bottom)

Date: Jan. 29, 1990 | Tested By: K.A.Abdolos

CLASSIFICATION TEST-GRADATION CURVES



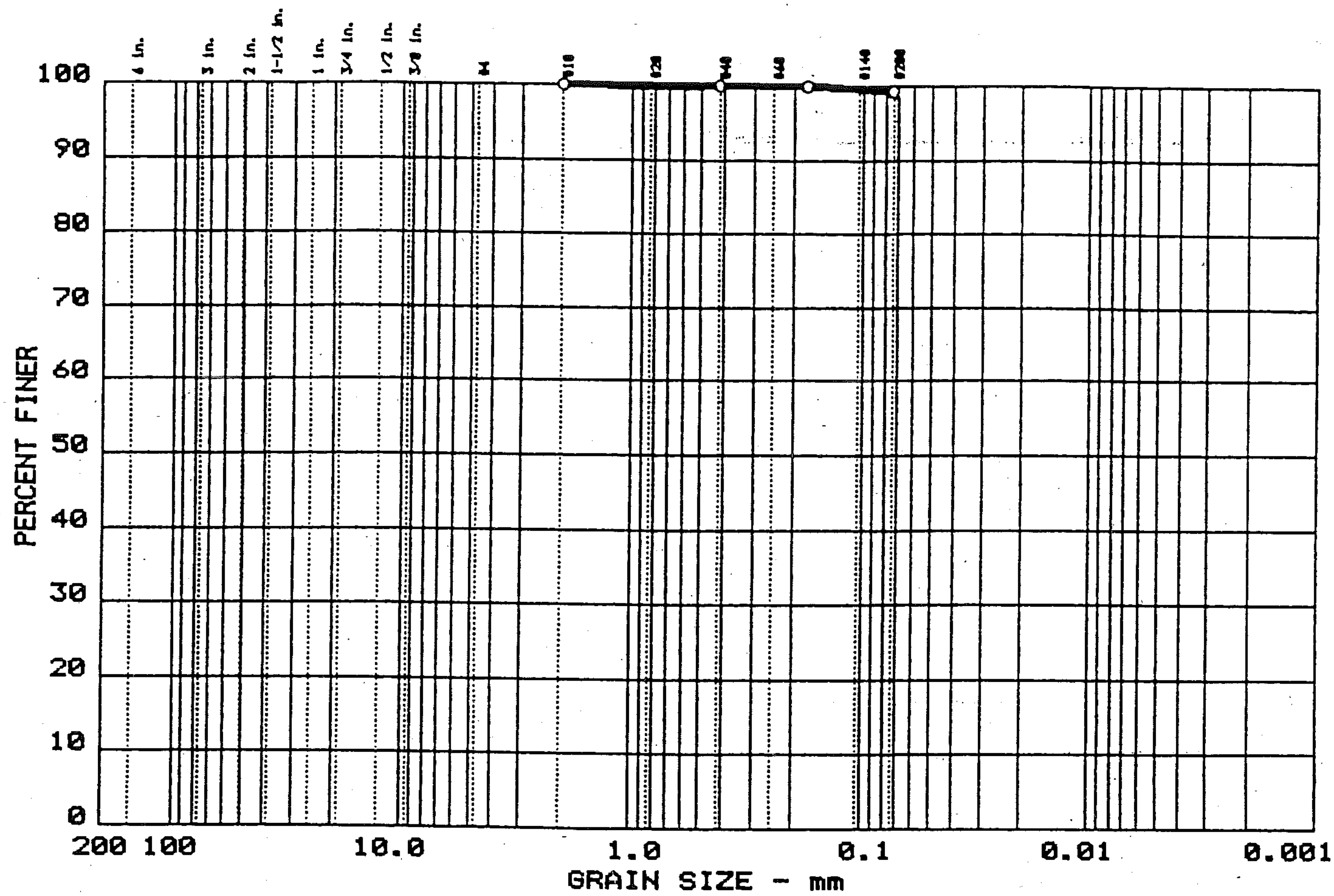
COBBLES	GRAVEL		SAND			SILT OR CLAY
	Coarse	Fine	Coarse	Medium	Fine	

Sample No.	Depth	Classification	Nat.WC	LL	PL	PI	Gs
GF-109/U-2	17.0'-19.0'	MH		56.8	35.8	21.0	--
		LL ovendried =		49.5			

GANNETT FLEMING GEOTECHNICAL LABORATORY

Project: DSWA, NSWF , PHASE III

GRAIN SIZE DISTRIBUTION TEST REPORT



	%+75mm	% GRAVEL	% SAND	% SILT	% CLAY
o	0.0	0.0	0.7	99.3	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
o	84	35								

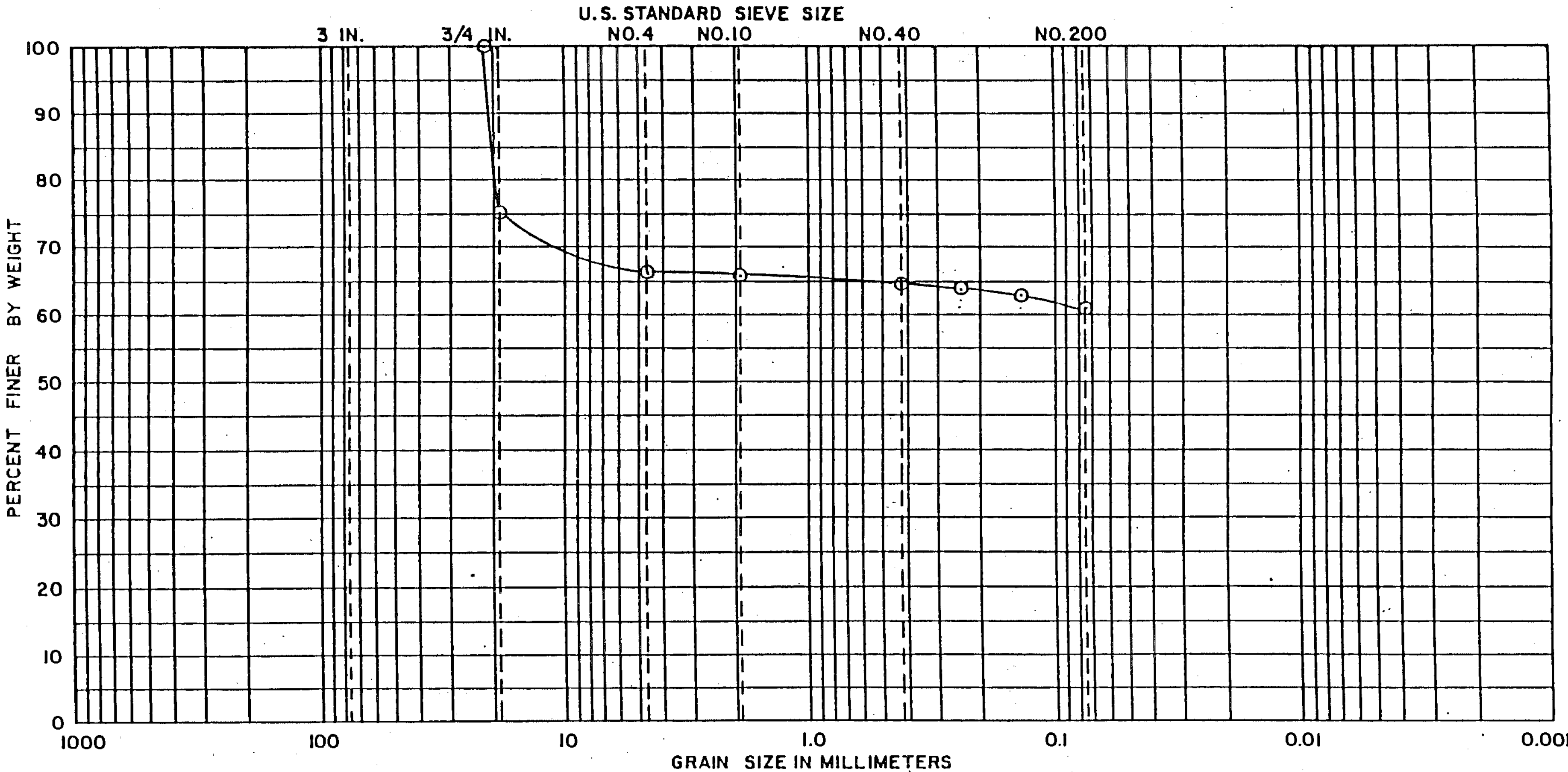
MATERIAL DESCRIPTION	USCS	AASHTO
o Dark Grey Clayey Silt	MH	

Project No.: 5411
Project: DSWA - Phase III
o Location: Boring: GF-109; S: U-3; 37.0'- 39.0'

Date: 1-29-90

Remarks:
Project Location:
Wilmington, Delaware
Client:
Gannett-Fleming

GRAIN SIZE DISTRIBUTION TEST REPORT



COBBLES	GRAVEL		SAND			SILT OR CLAY
	Coarse	Fine	Coarse	Medium	Fine	

Sample No.	Depth	Classification	Nat. WC	LL	PL	PI	Gs
S3	4.0'-6.0'	MH	49.9	56.1	37.4	18.7	-
S5	8.0'-10.0'						

Description and Comments: Gray Gravelly Elastic Silt
Max. Grain Size 0.85"

CLASSIFICATION TEST-GRADATION CURVES

GANNETT FLEMING GEOTECHNICAL LABORATORY

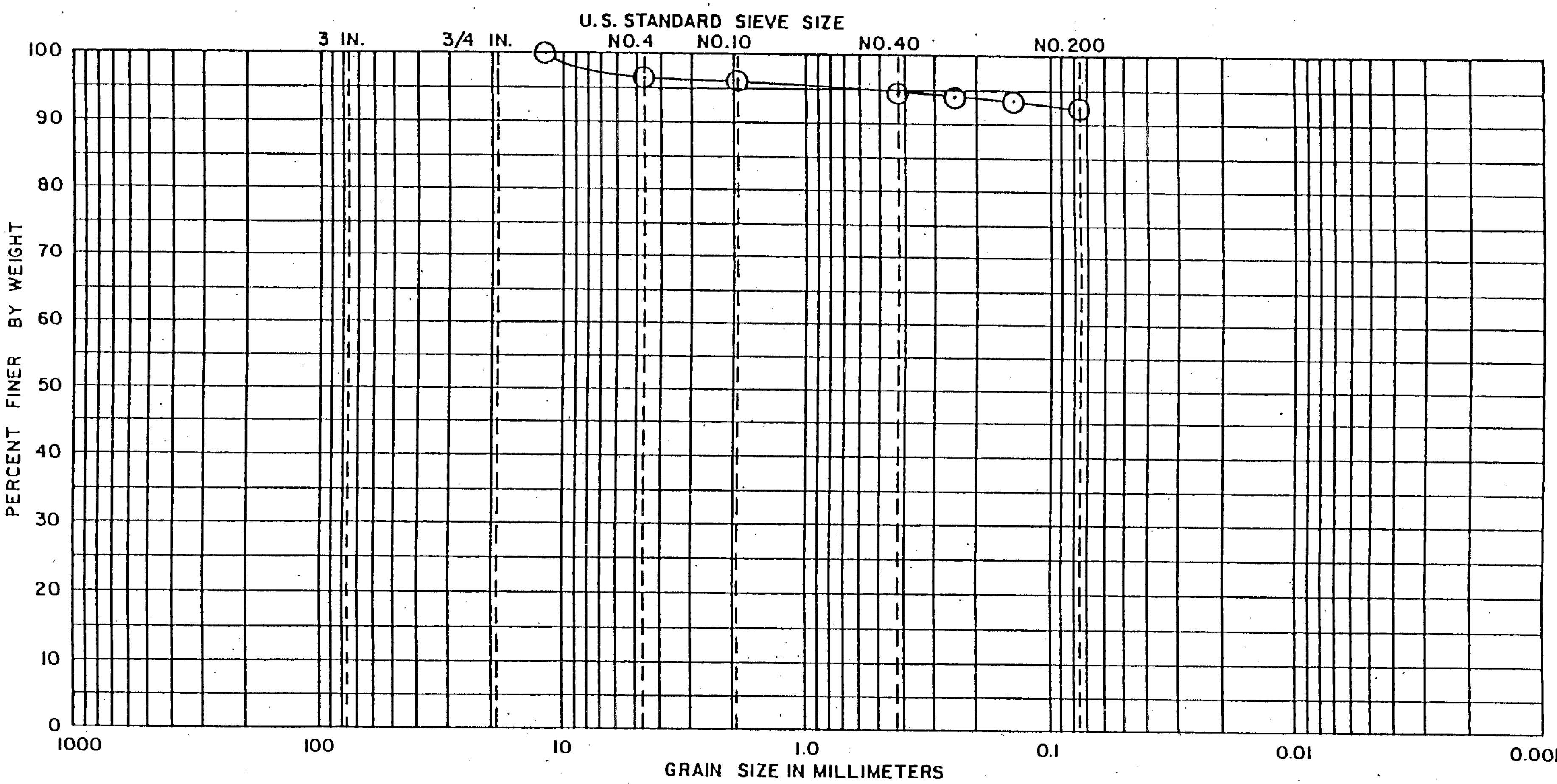
Project: DSWA NSWF - 2 PHASE III

Area: WILMINGTON, DE.

Boring No: GF-110

Date: 10/27/89

Tested By: DKN



COBBLES	GRAVEL		SAND			SILT OR CLAY
	Coarse	Fine	Coarse	Medium	Fine	

Sample No.	Depth	Classification	Nat.WC	LL	PL	PI	Gs
U-1	10 - 12'	MH		67.1	45.4	21.7	--
		(LL OVEN DRYED)	=	55.9			

Description and Comments:
1) BROWN-ELASTIC SILT.
2) LL OD/LL AD = .83

CLASSIFICATION TEST-GRADATION CURVES

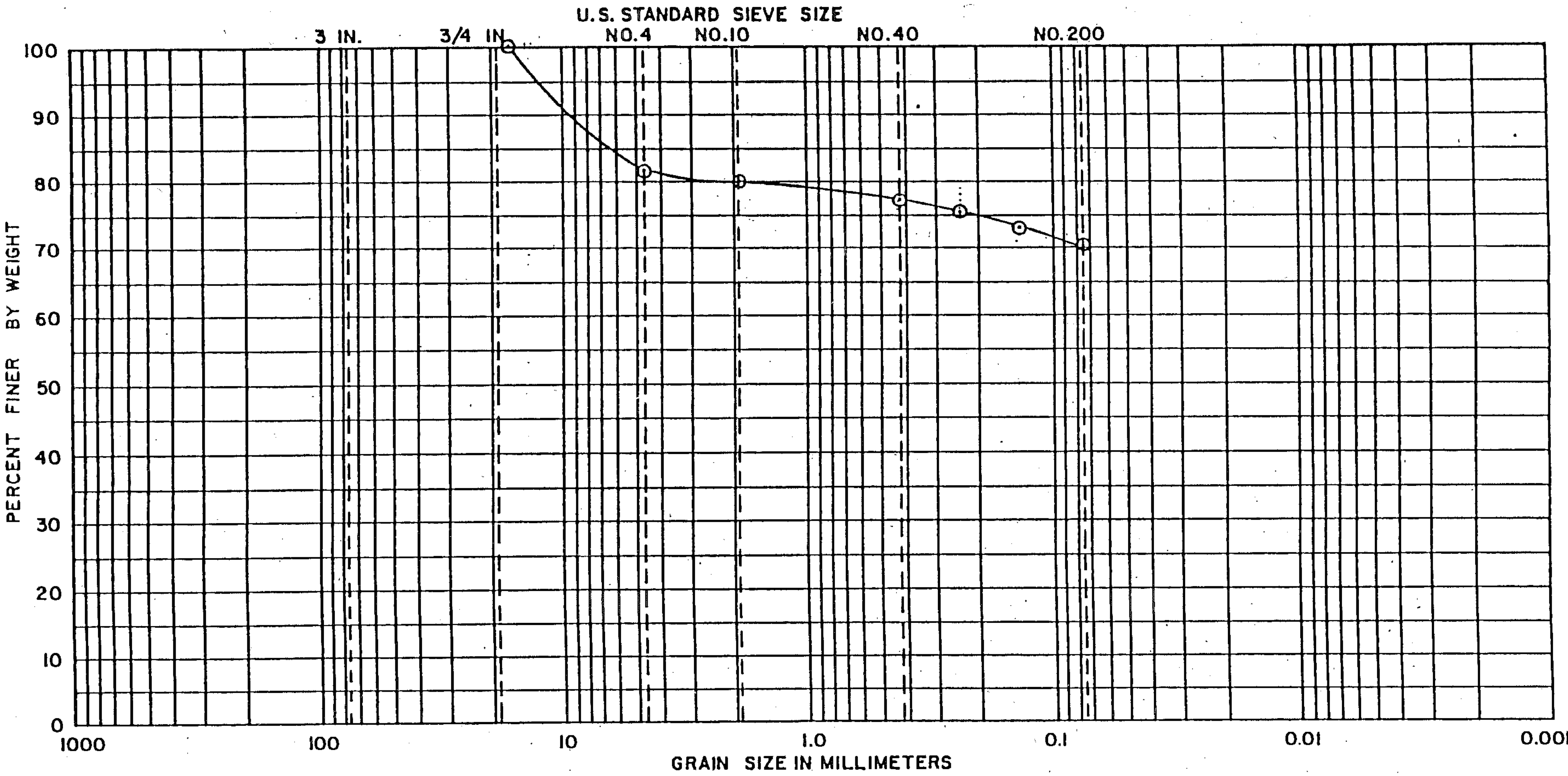
GANNETT FLEMING GEOTECHNICAL LABORATORY

Project: DSWA, NSWF, PHASE III

Area: WILMINGTON, DE

Boring No: GF-110

Date: 1/24/90 | Tested By: KLM



COBBLES	GRAVEL		SAND			SILT OR CLAY
	Coarse	Fine	Coarse	Medium	Fine	

Sample No.	Depth	Classification	Nat.WC	LL	PL	PI	Gs
U-2	25.0'-27.0'	MH	70.1	53.3	35.8	17.5	-

Description and Comments: Gray Gravelly Elastic Silt
Max. Grain Size 0.70"

GANNETT FLEMING GEOTECHNICAL LABORATORY

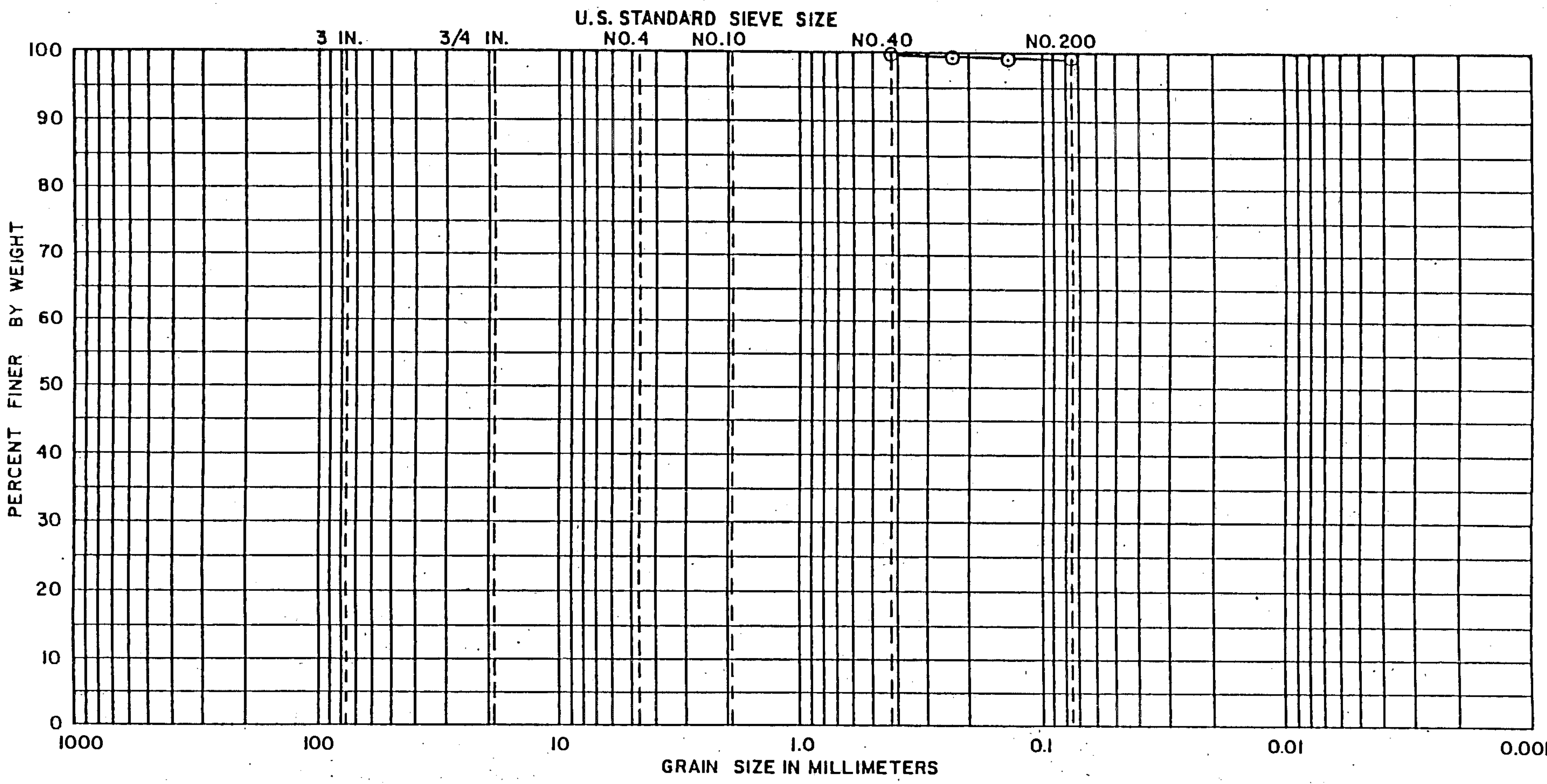
Project: DSWA NSWF - 2 PHASE III

Area: WILMINGTON , DE

Boring No: GF-110

Date: 11/3/89 Tested By: DKN

CLASSIFICATION TEST-GRADATION CURVES



COBBLES	GRAVEL		SAND			SILT OR CLAY
	Coarse	Fine	Coarse	Medium	Fine	

Sample No.	Depth	Classification	Nat. WC	LL	PL	PI	Gs
S12,S13	33.5'-40.0'	MH	78.5	68.8	45.2	23.6	-

Description and Comments: Gray Elastic Silt

CLASSIFICATION TEST-GRADATION CURVES

GANNETT FLEMING GEOTECHNICAL LABORATORY

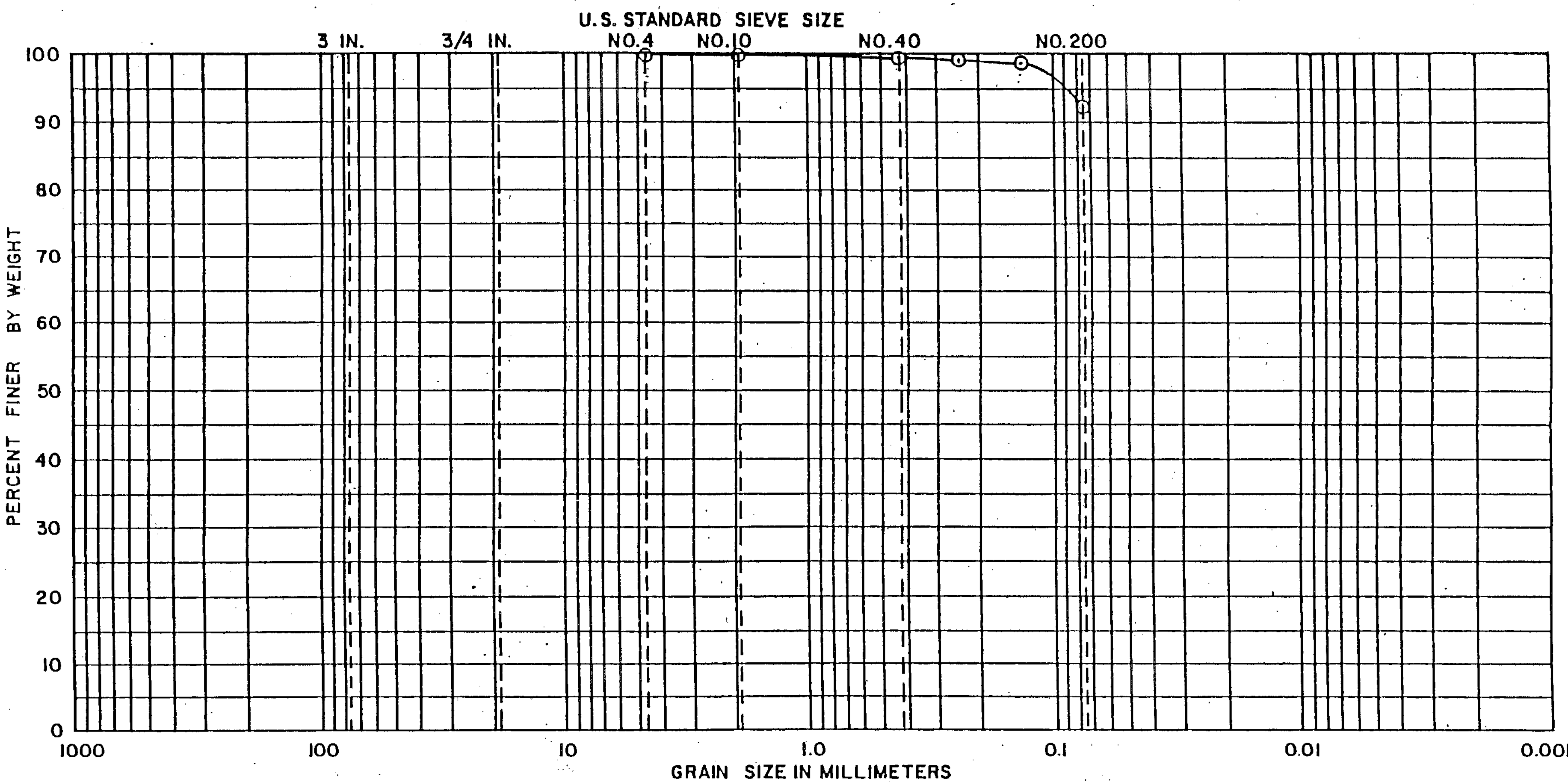
Project: DSWA NSWF - 2 PHASE III

Area: WILMINGTON , DE

Boring No: GF-110

Date: 10/27/89

Tested By: DKN



COBBLES	GRAVEL		SAND			SILT OR CLAY
	Coarse	Fine	Coarse	Medium	Fine	

Sample No.	Depth	Classification	Nat.WC	LL	PL	PI	Gs
S16,S17	53.5'-60.0'	ML	53.2	47.4	31.6	15.8	–

Description and Comments: Gray Silt

GANNETT FLEMING GEOTECHNICAL LABORATORY

Project: DSWA NSWF – 2 PHASE III

Area: WILMINGTON , DE

Boring No: GF-110

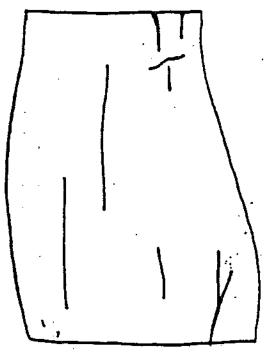
Date: 10/27/89

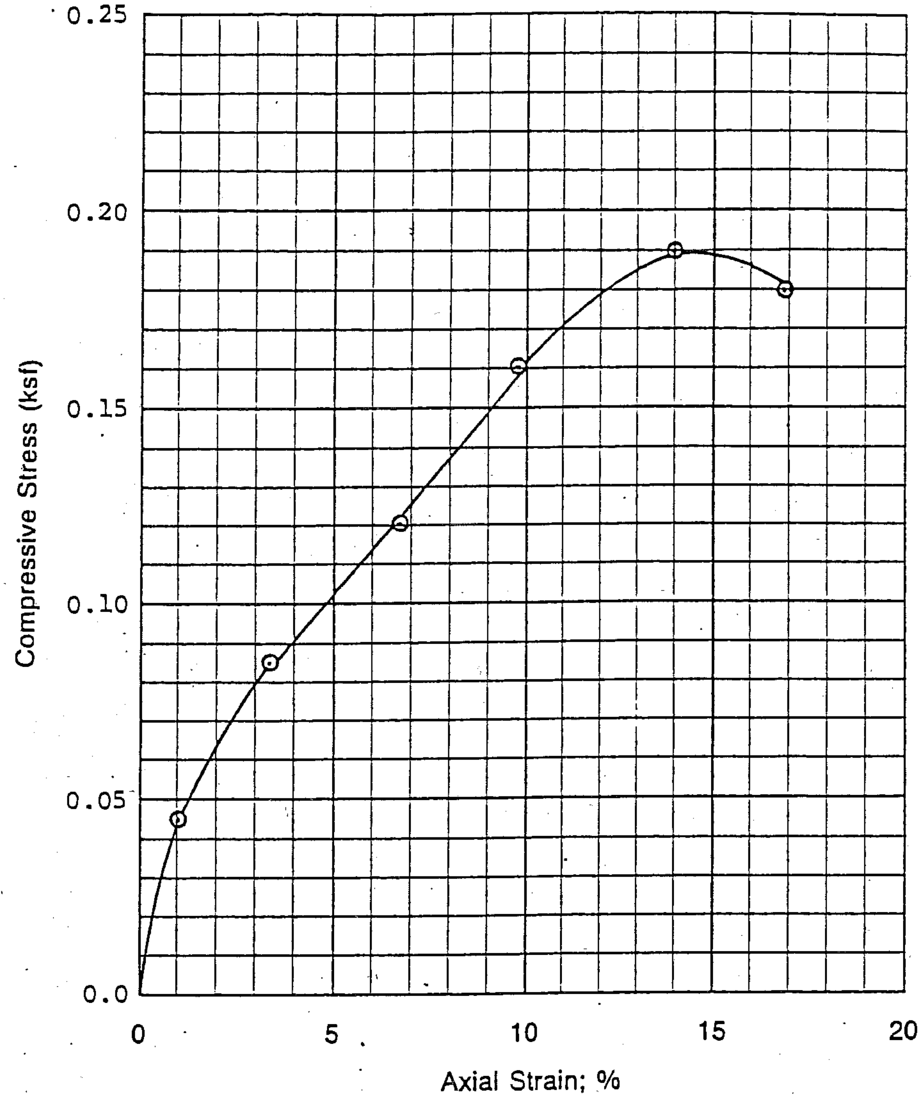
Tested By: DKN

CLASSIFICATION TEST – GRADATION CURVES

UNCONFINED COMPRESSION TESTS

Failure Sketches





Test no			1			
Type of specimen			Shelby			
Initial	Water content	w_o	97.8 %	%	%	%
	Void ratio	e_o	2.552			
	Saturation	S_o	100.0 %	%	%	%
	Dry density, lb/cu ft	γ_d	48.5			
Time to failure, min		t_f	7			
Unconfined compressive strength, ksf		q_u	0.19			
Initial specimen diameter, in		D_o	2.762"			
Initial specimen height, in		H_o	5.664"			

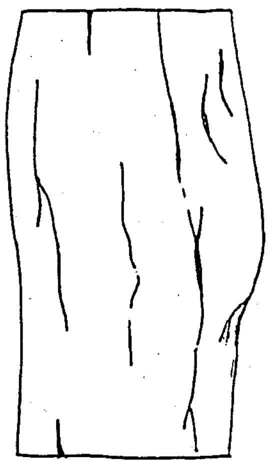
UNCONFINED COMPRESSION TEST REPORT

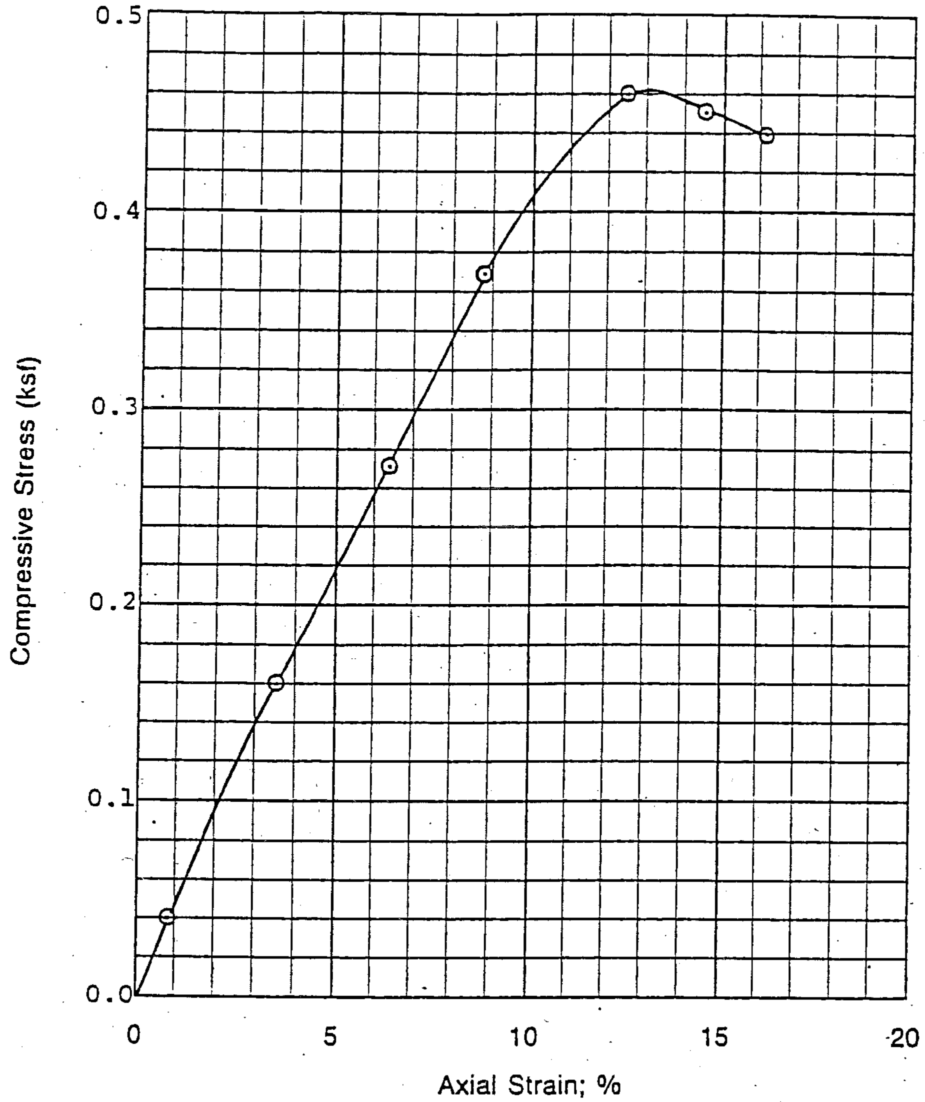
Project DSWA NSWF - 2 PHASE III Boring No GF-103

Sample No U-1 Depth 15.0'-17.0' Date 11/3/89

GANNETT FLEMING GEOTECHNICAL LABORATORY

Failure Sketches





Test no			1			
Type of specimen			Shelby			
Initial	Water content	w_o	56.5 %	%	%	%
	Void ratio	e_o	1.698			
	Saturation	S_o	91.5 %	%	%	%
	Dry density, lb/cu ft	γ_d	63.6			
Time to failure, min		t_f	11			
Unconfined compressive strength, ksf		q_u	0.46			
Initial specimen diameter, in		D_o	2.848"			
Initial specimen height, in		H_o	5.582"			

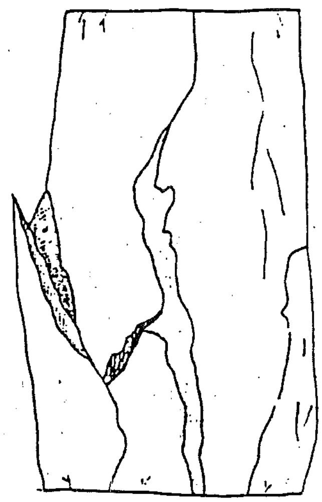
UNCONFINED COMPRESSION TEST REPORT

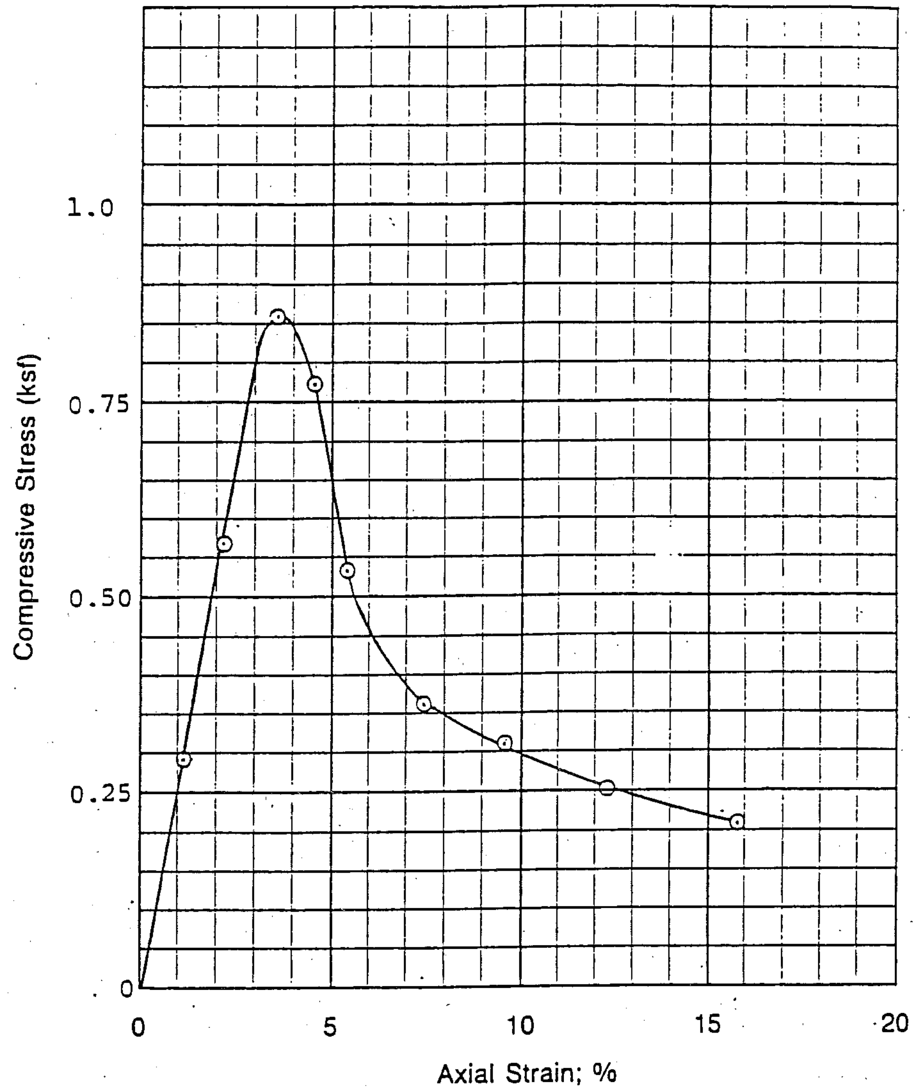
Project DSWA NSWF - 2 PHASE III Boring No GF-103

Sample No U-2 Depth 45.0'-47.0' Date 11/3/89

GANNETT FLEMING GEOTECHNICAL LABORATORY

Failure Sketches

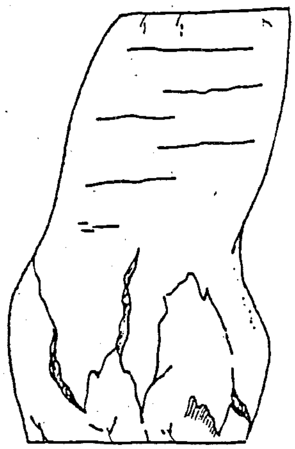


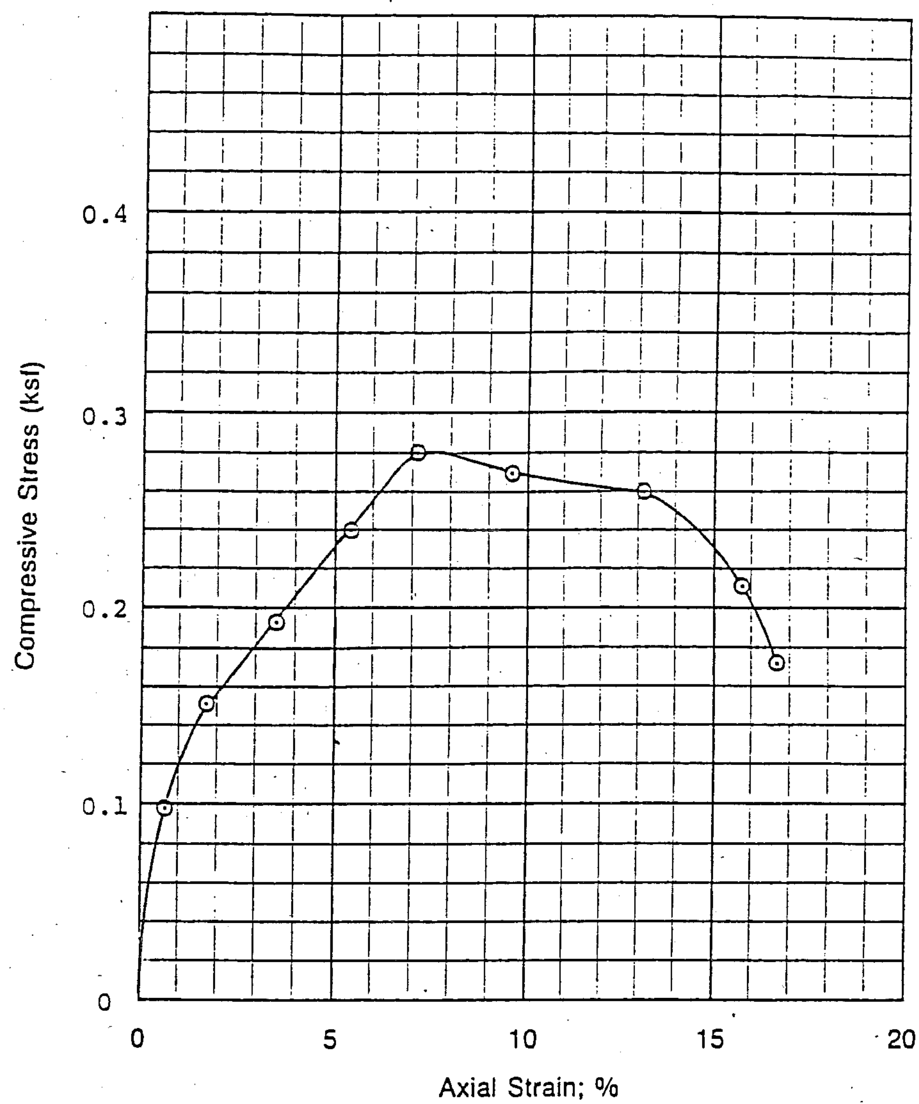


Test no			1			
Type of specimen			Shelby			
Initial	Water content	w_o	53.8 %	%	%	%
	Void ratio	e_o	1.872			
	Saturation	S_o	79.0 %	%	%	%
	Dry density, lb/cu ft	γ_d	59.8			
Time to failure, min		t_f	2.0			
Unconfined compressive strength, ksf		q_u	0.86			
Initial specimen diameter, in		D_o	2.827"			
Initial specimen height, in		H_o	5.669"			

UNCONFINED COMPRESSION TEST REPORT	
Project DSWA NSWF - 2 PHASE III	Boring No GF - 104A
Sample No U - 2 Depth 16.5'-18.5'	Date 11/22/89
GANNETT FLEMING GEOTECHNICAL LABORATORY	

Failure Sketches





Test no			1			
Type of specimen			Shelby			
Initial	Water content	w_o	98.3 %	%	%	%
	Void ratio	e_o	2.705			
	Saturation	S_o	100 %	%	%	%
	Dry density, lb/cu ft	γ_d	46.3			
Time to failure, min		t_f	3.5			
Unconfined compressive strength, ksf		q_u	0.28			
Initial specimen diameter, in		D_o	1.820"			
Initial specimen height, in		H_o	5.770"			

UNCONFINED COMPRESSION TEST REPORT

Project DSWA NSWF - 2 PHASE III Boring No GF - 104A

Sample No U - 4 Depth 35.0'-37.0' Date 11/27/89

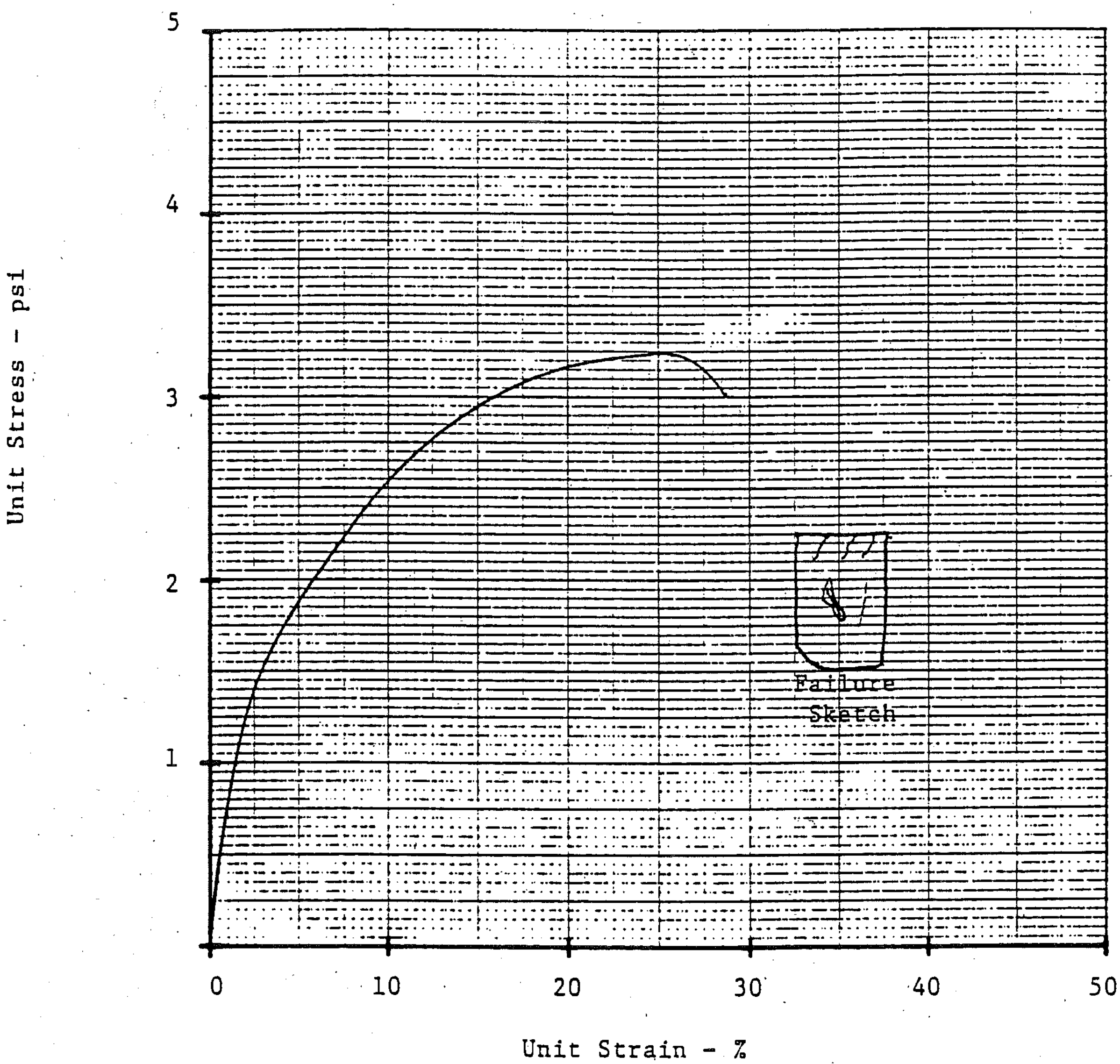
GANNETT FLEMING GEOTECHNICAL LABORATORY

UNCONFINED COMPRESSION TEST
ASTM DESIGNATION: D 2166

DSWA, NSWF-2, Phase III
WILMINGTON, DELAWARE

Boring No.: GF-108 Sample Type: Undisturbed
Depth: 28.5' - 30.5' Specimen Size: 1.75" dia.
3.50" hgt.

Maximum Compressive Stress: 3.23 p.s.i.
Failure Strain: 25.7%
Wet Density: 99.8 p.c.f
Moisture Content: 86.7%
Dry Density: 53.5 p.c.f.



January 1990

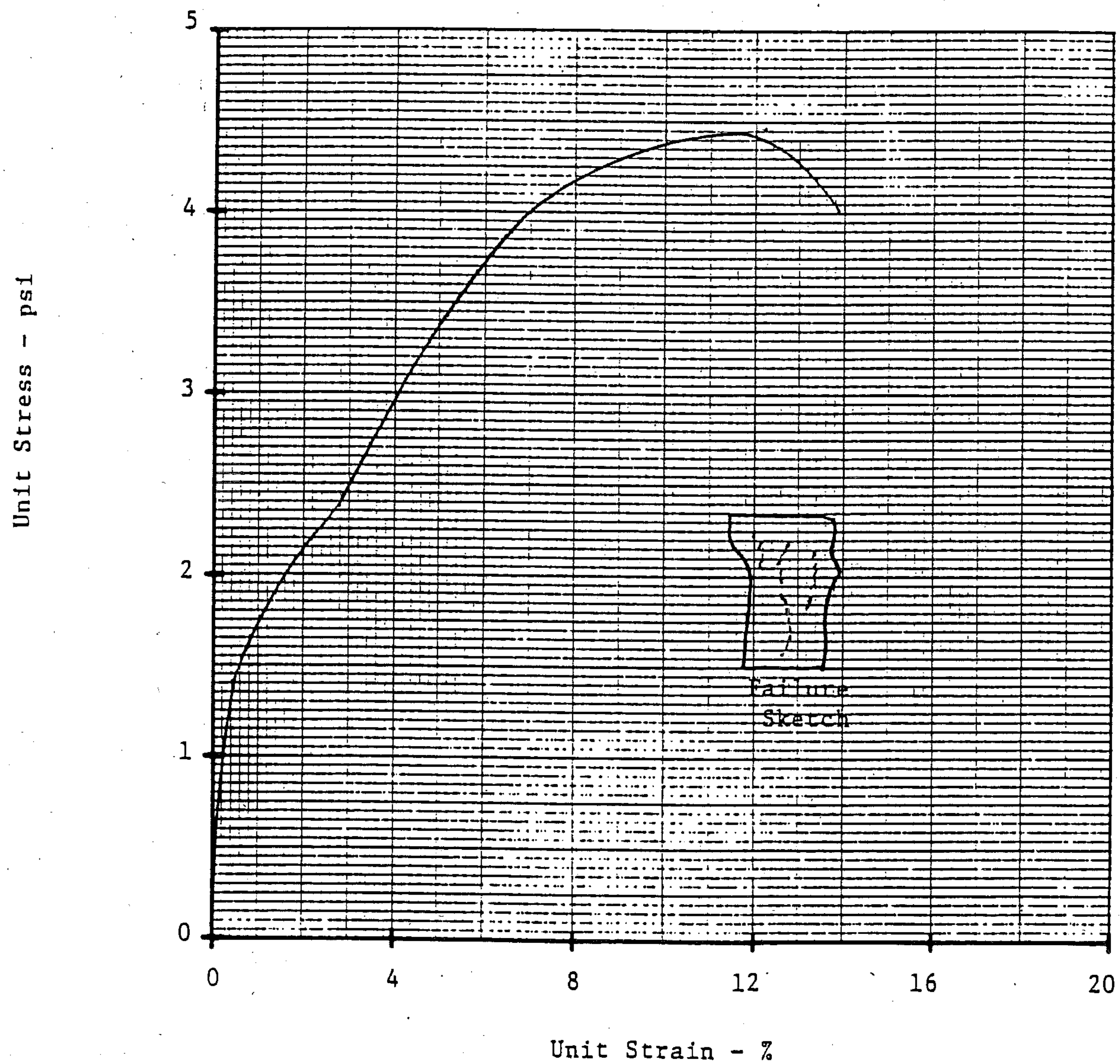
F. T. Kitlinski & Associates, Inc.
Harrisburg, Pennsylvania

UNCONFINED COMPRESSION TEST
ASTM DESIGNATION: D 2166

DSWA, NSWF-2, Phase III
WILMINGTON, DELAWARE

Boring No.: GF-108 Sample Type: Undisturbed
Sample Depth: 49.0'-51.0' Specimen Size: 1.75" dia.
3.50" hgt.

Maximum Compressive Strength: 4.44 p.s.i.
Failure Strain: 11.4%
Wet Density: 105.7 p.c.f.
Moisture Content: 67.9%
Dry Density: 63.0 p.c.f.



January 1990

F. T. Kitlinski & Associates, Inc.
Harrisburg, Pennsylvania

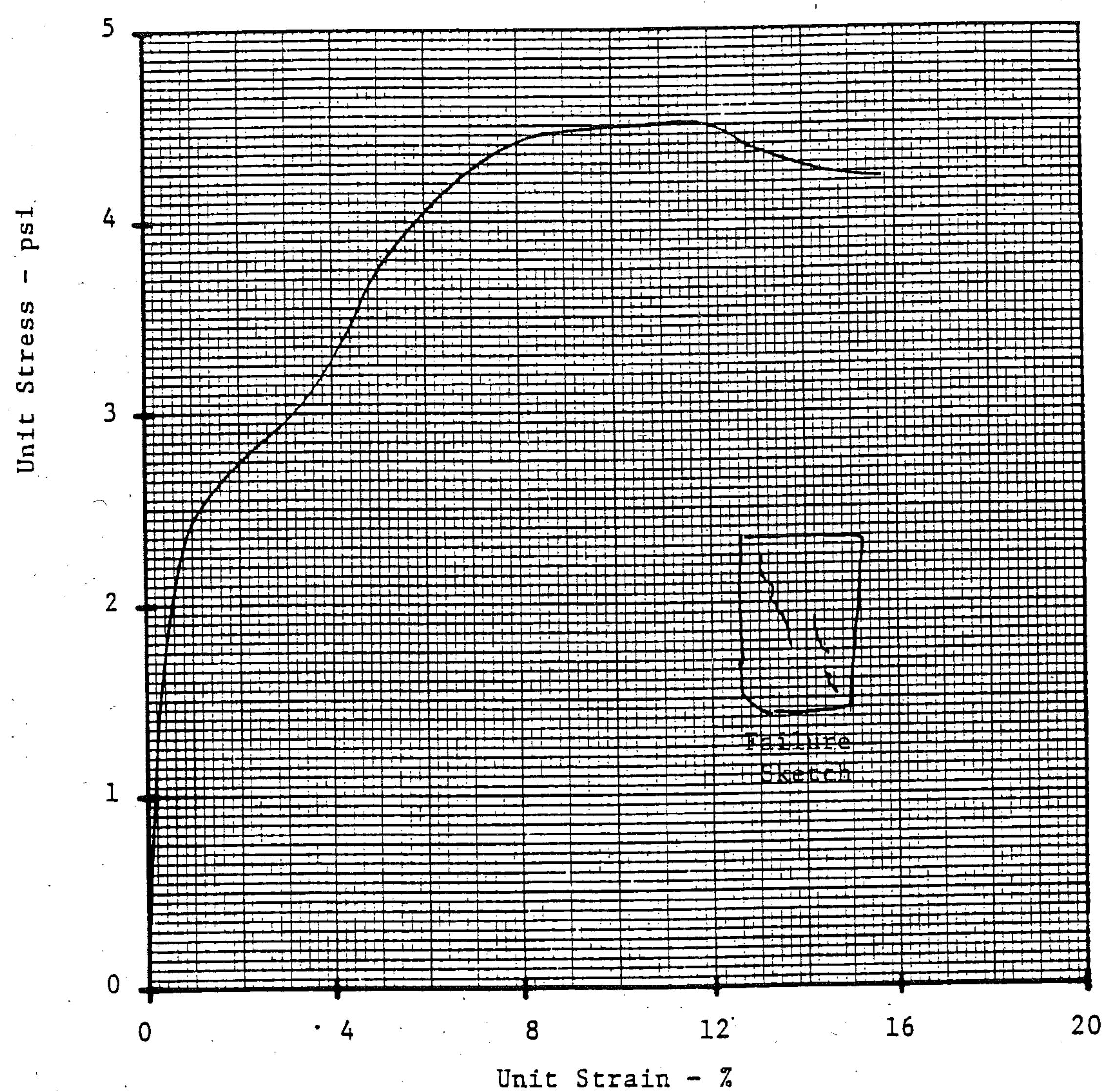
UNCONFINED COMPRESSION TEST
ASTM DESIGNATION: D 2166

DSWA, NSWF-2, Phase III
WILMINGTON, DELAWARE

Boring No.: GF-109
Sample Depth: 37.0'-39.0'

Sample Type: Undisturbed
Specimen Size: 1.75" dia.
3.50" hgt.

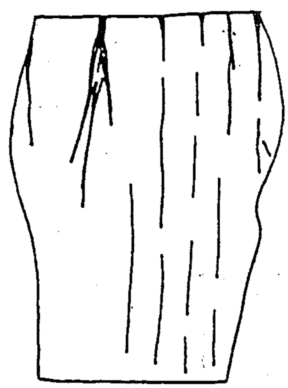
Maximum Compressive Stress: 4.55 p.s.i.
Failure Strain: 11.7%
Wet Density: 105.3 p.c.f.
Moisture Content: 79.5%
Dry Density: 58.7 p.c.f.

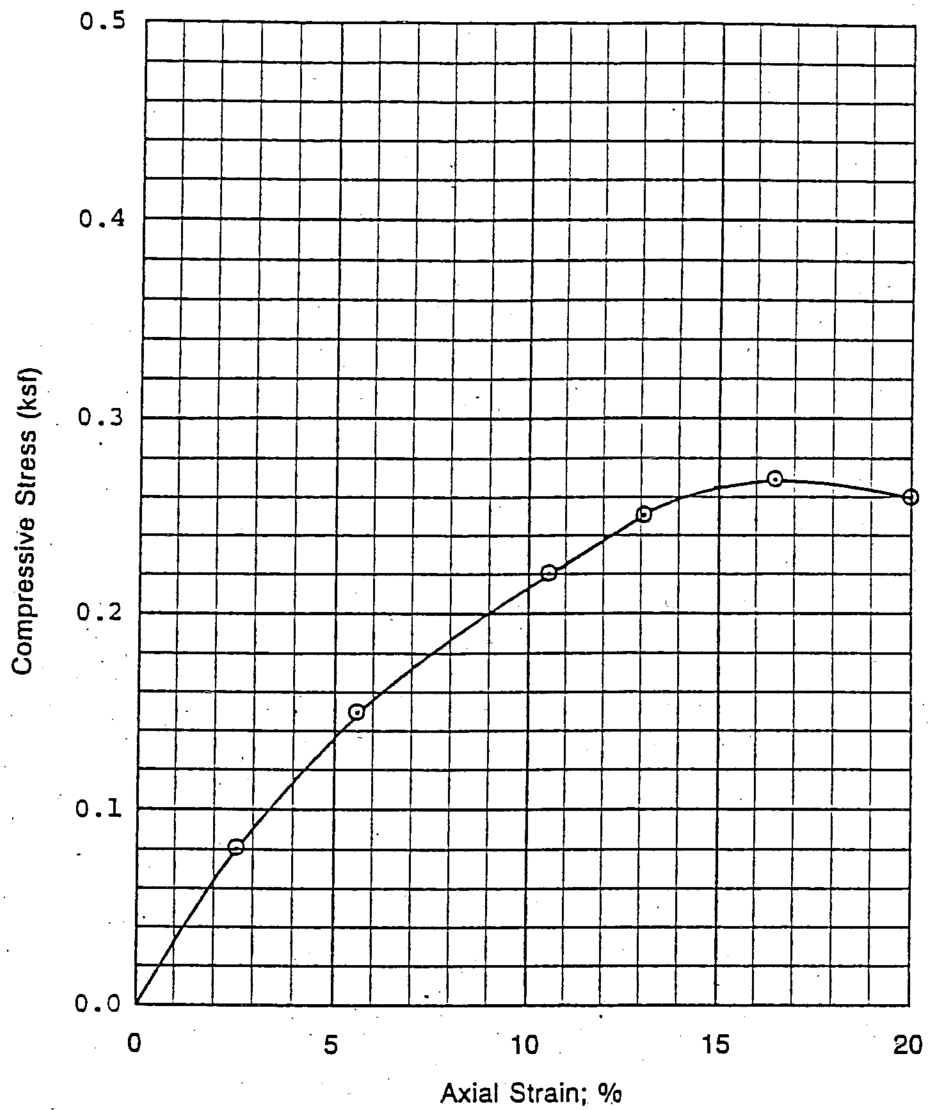


January 1990

F. T. Kitlinski & Associates, Inc.
Harrisburg, Pennsylvania

Failure Sketches





Test no			1			
Type of specimen			Shelby			
Initial	Water content	w_o	78.7 %	%	%	%
	Void ratio	e_o	2.249			
	Saturation	S_o	96.0 %	%	%	%
	Dry density, lb/cu ft	γ_d	52.8			
Time to failure, min		t_f	17			
Unconfined compressive strength, ksf		q_u	0.26			
Initial specimen diameter, in		D_o	2.838"			
Initial specimen height, in		H_o	5.742"			

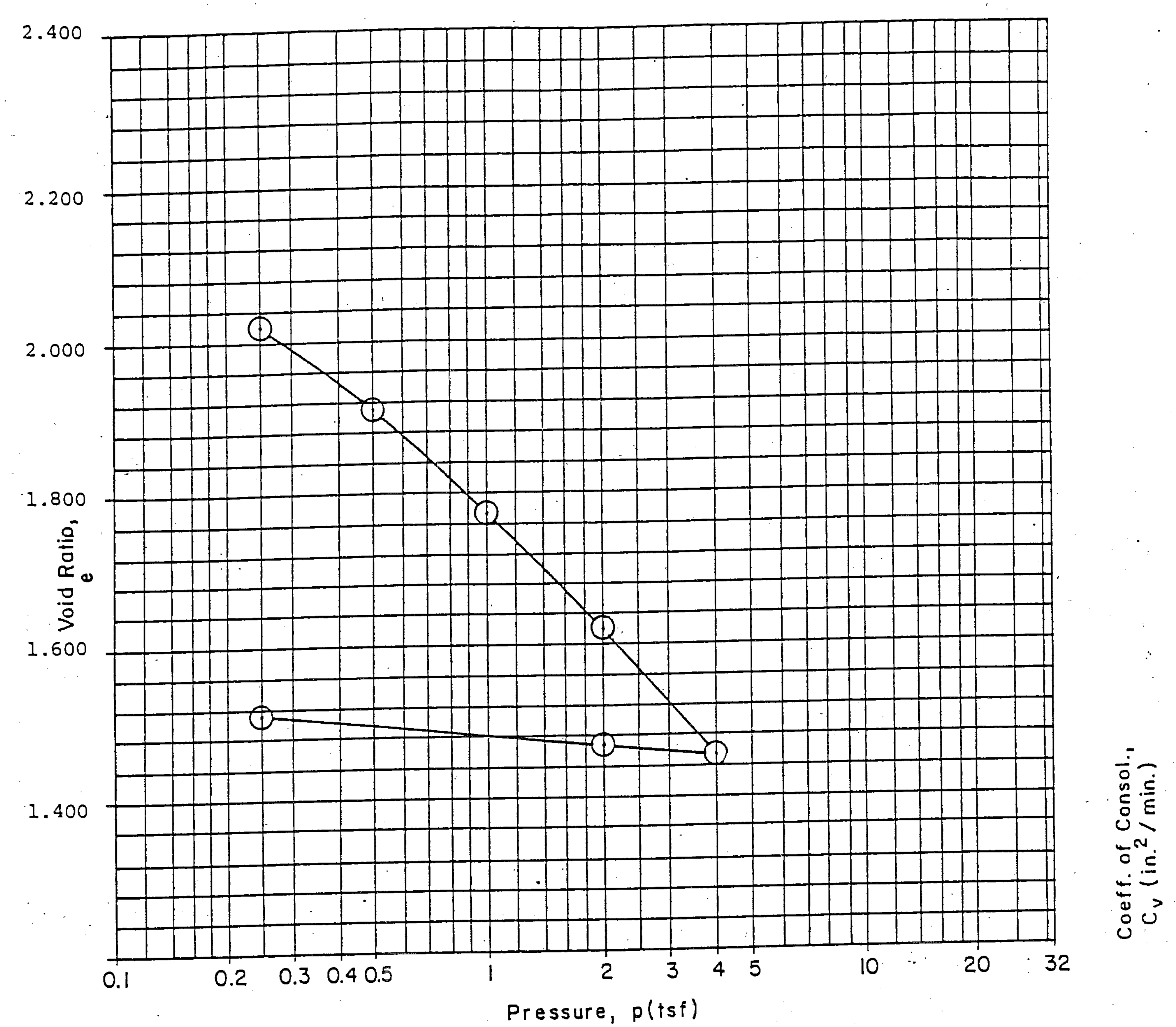
UNCONFINED COMPRESSION TEST REPORT

Project DSWA NSWF - 2 PHASE III Boring No GF-110

Sample No U-2 Depth 25.0'-27.0' Date 11/3/89

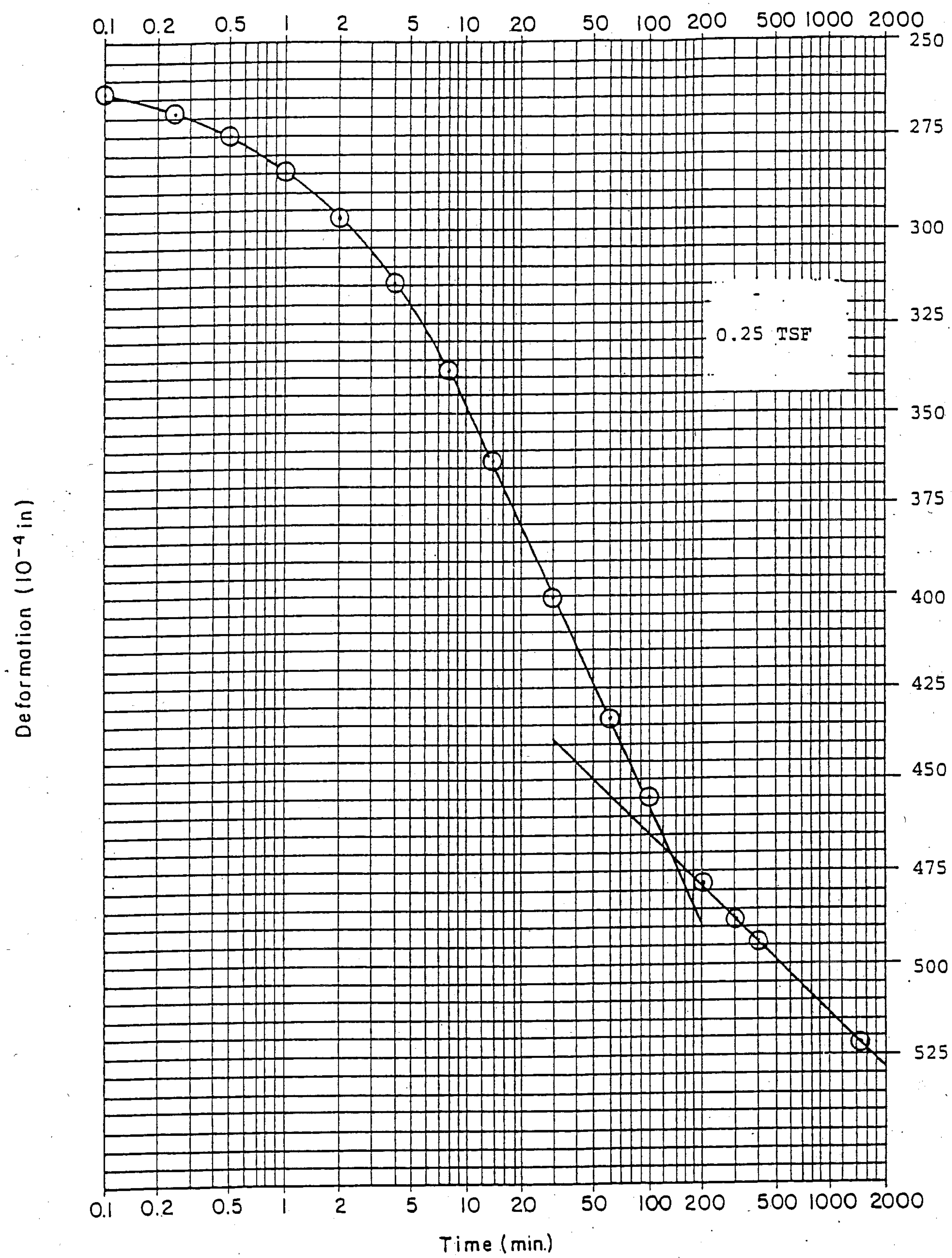
GANNETT FLEMING GEOTECHNICAL LABORATORY

CONSOLIDATION TEST RESULTS



Type of Specimen SHELBY TUBE				Before Test				After Test	
Dia	2.50 in	H_T	1.00 in	Water Content	w_o	73.7		w_f	55.8
Compression Index		C_c	0.55	Void Ratio	e_o	2.105		e_f	1.508
Classification BRN.-ELASTIC SILT.				Saturation	S_o	94.5		S_f	100.0
w_l 69.3		I_p 21.5		Project DSWA, NSWF, PHASE III					
w_p 47.8		LI 1.2		Boring No GF-102				Sample No U-2	
Remarks WET DENSITY = 94.3 pcf				Depth 35'- 37'				Date 2/5/90	

GANNETT FLEMING GEOTECHNICAL LABORATORY
CONSOLIDATION TEST REPORT

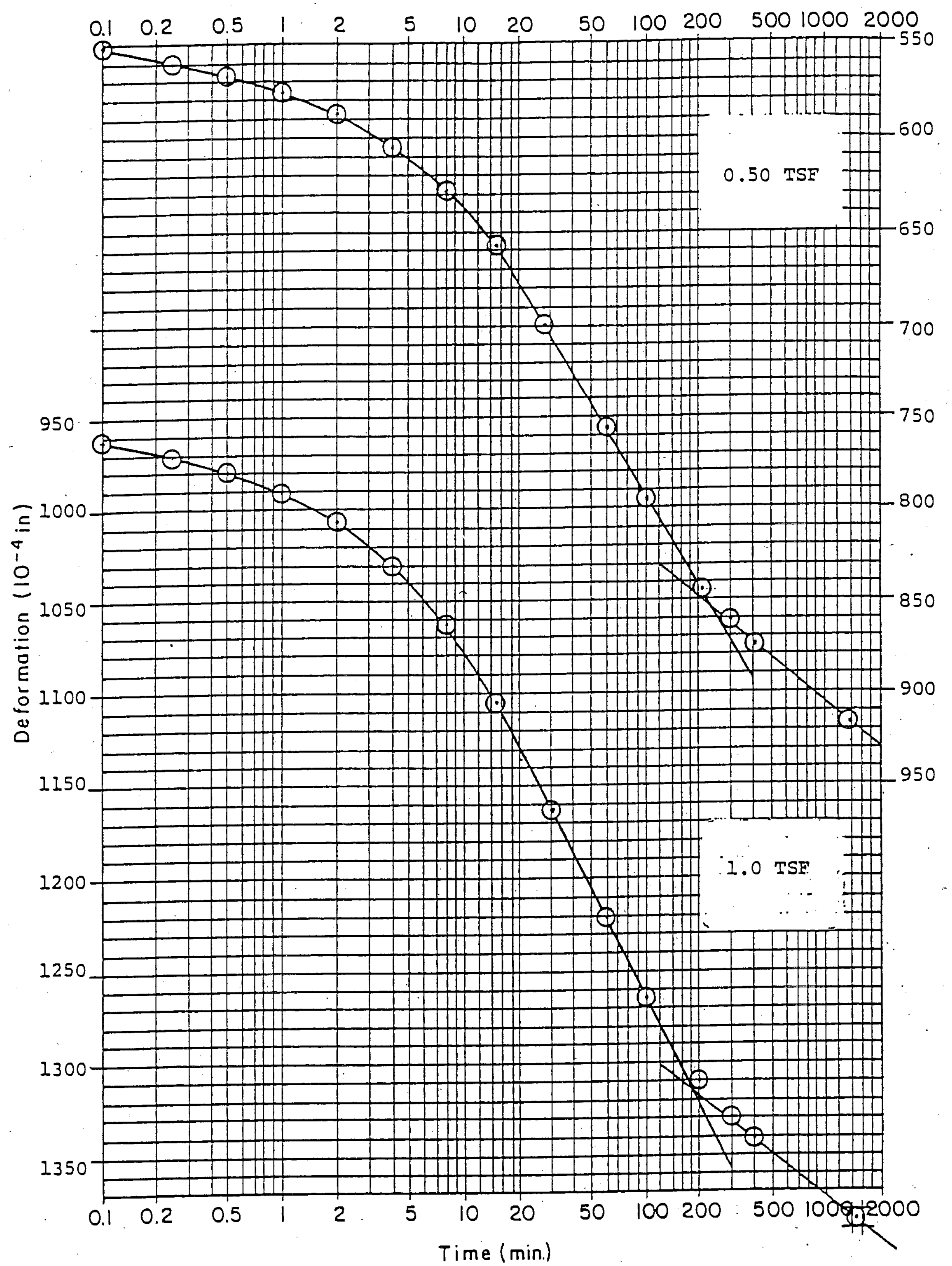


CONSOLIDATION TEST-TIME CURVES

Project DSWA, NSWF, PHASE III Boring No GF-102

Sample No U-2 Depth 35 - 37' Date 1/29/90

GANNETT FLEMING GEOTECHNICAL LABORATORY

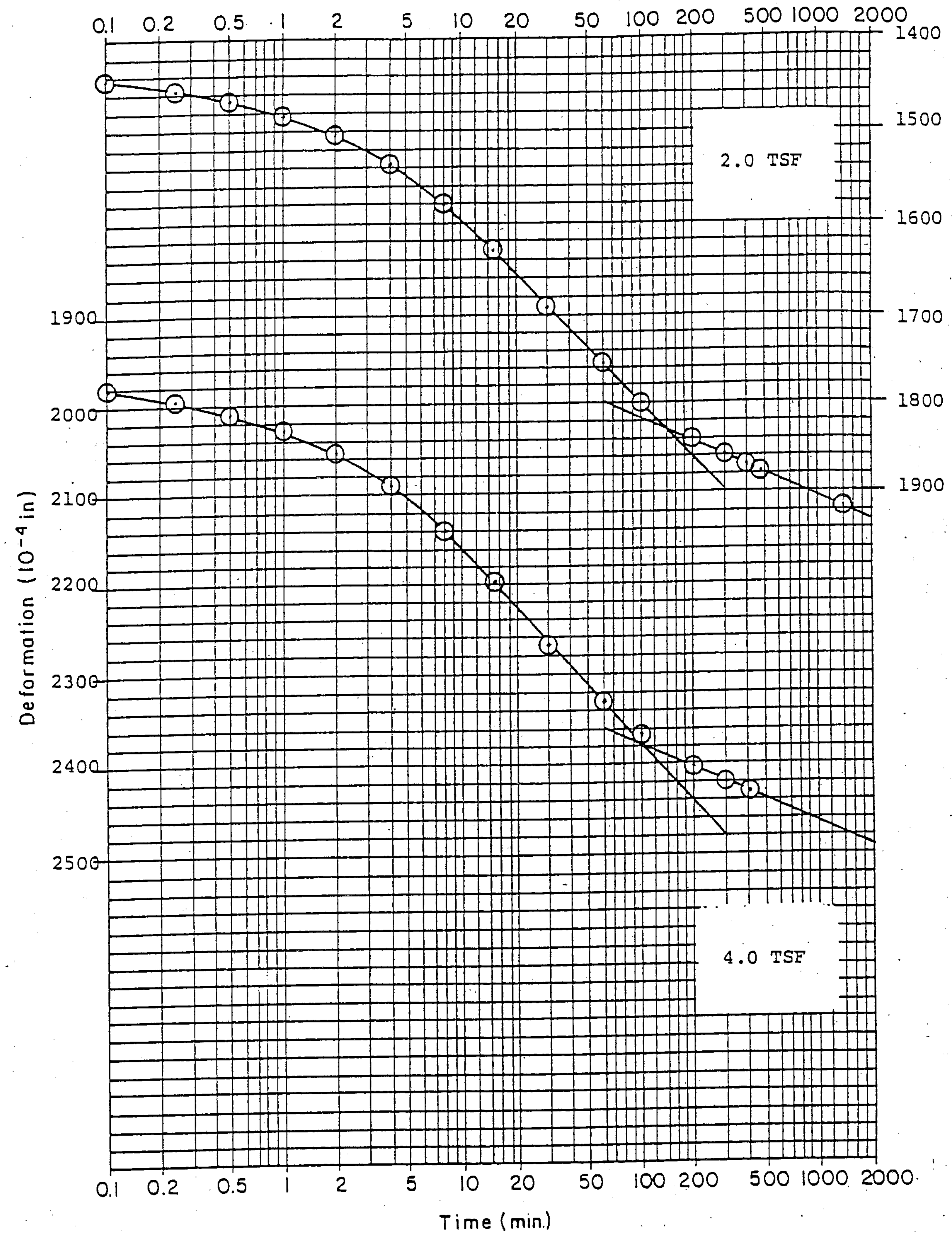


CONSOLIDATION TEST-TIME CURVES

Project DSWA, NSWF, PHASE III Boring No GF-102

Sample No U-2 Depth 35 - 37' Date 1/29/90

GANNETT FLEMING GEOTECHNICAL LABORATORY

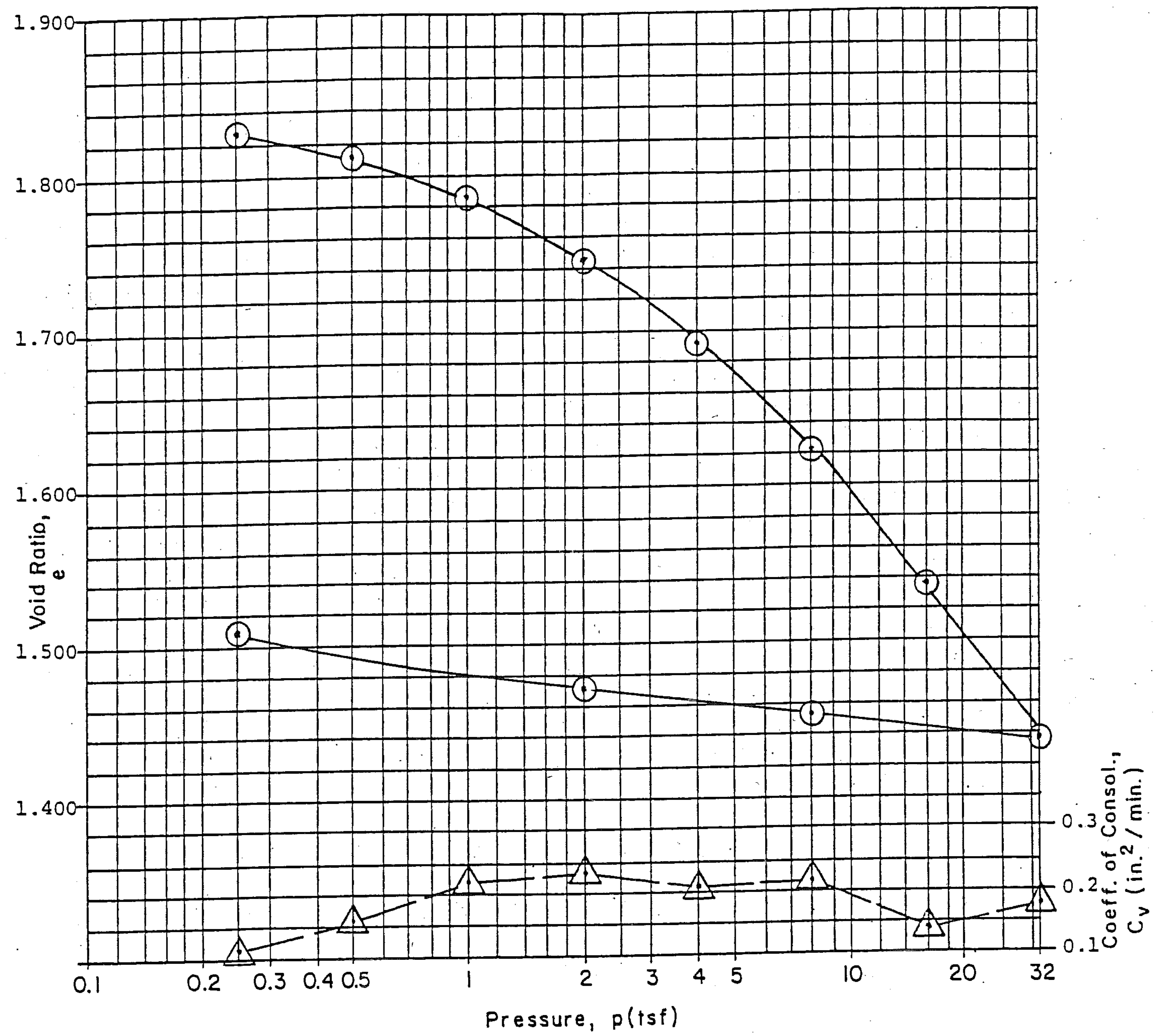


CONSOLIDATION TEST-TIME CURVES

Project DSWA, NSWF, PHASE III Boring No GF-102

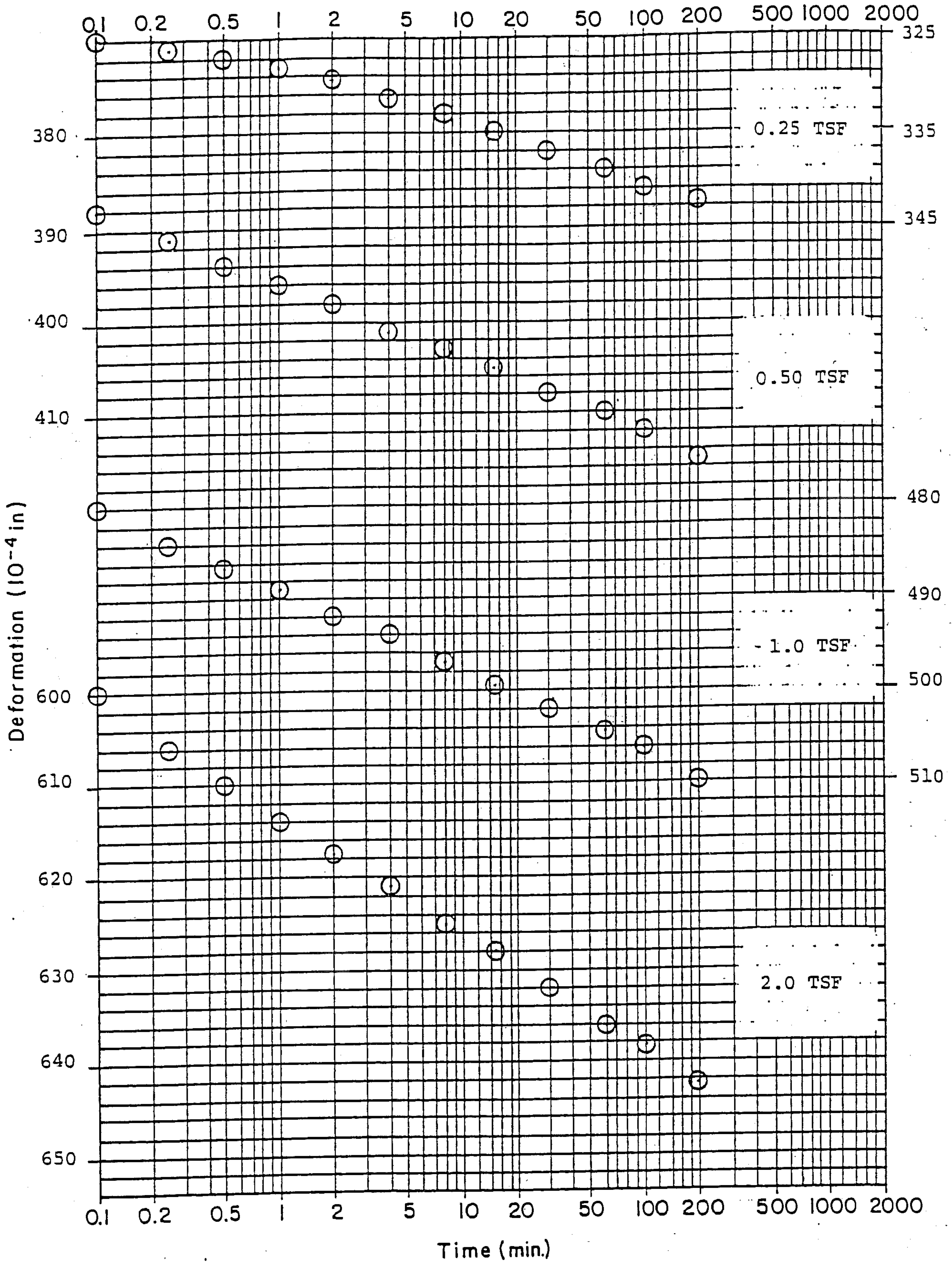
Sample No U-2 Depth 35 - 37' Date 1/29/90

GANNETT FLEMING GEOTECHNICAL LABORATORY



Type of Specimen	Shelby			Before Test			After Test	
Dia	2.50 in	H_T	0.75 in	Water Content	w_o	53.6	w_f	46.9
Compression Index		C_c	0.30	Void Ratio	e_o	1.855	e_f	1.509
Classification	ML			Saturation	S_o	78.0	S_f	85.5
w_l		I_p	NP	Project	DSWA NSWF-2 PHASE III			
w_p		LI		Boring No	GF-104A	Sample No	U-2	
Remarks	Wet Density= 90.6pcf			Depth	16.5-18.5'	Date	12/89	

GANNETT FLEMING GEOTECHNICAL LABORATORY
CONSOLIDATION TEST REPORT

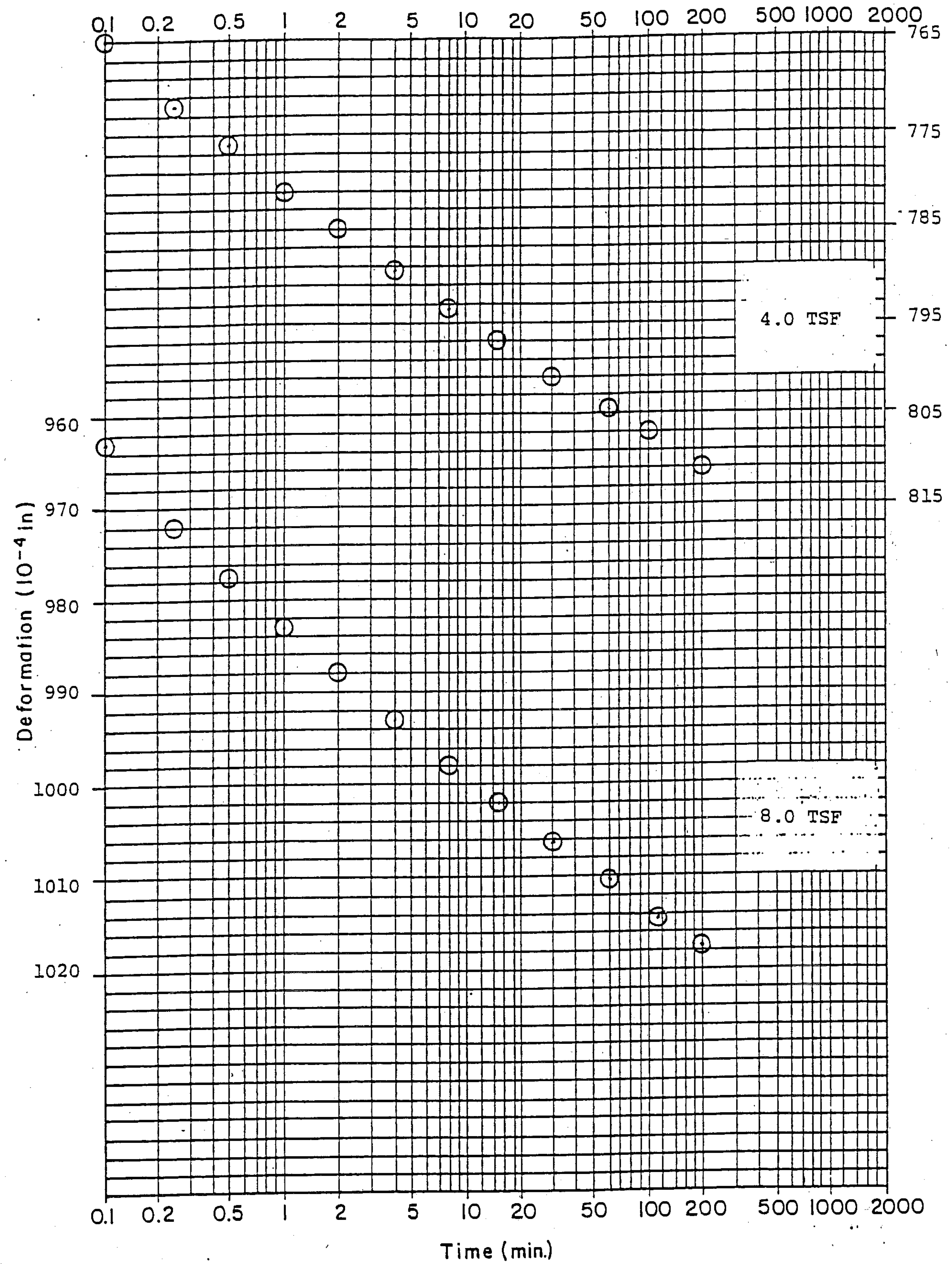


CONSOLIDATION TEST-TIME CURVES

Project DSWA NSWF-2 PHASE III Boring No GF-104A

Sample No U-2 Depth 16.5'-18.5' Date 12/13/89

GANNETT FLEMING GEOTECHNICAL LABORATORY

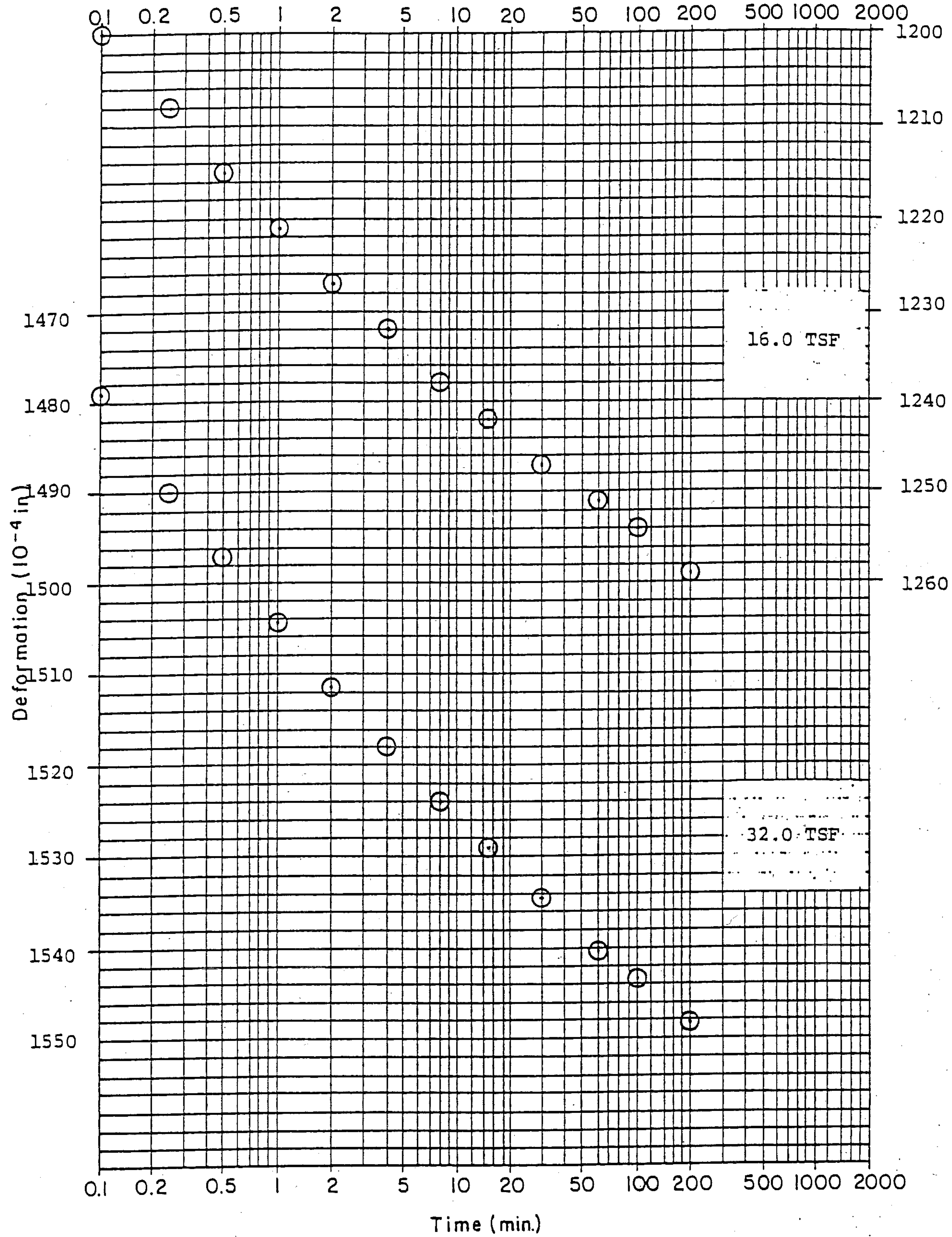


CONSOLIDATION TEST-TIME CURVES

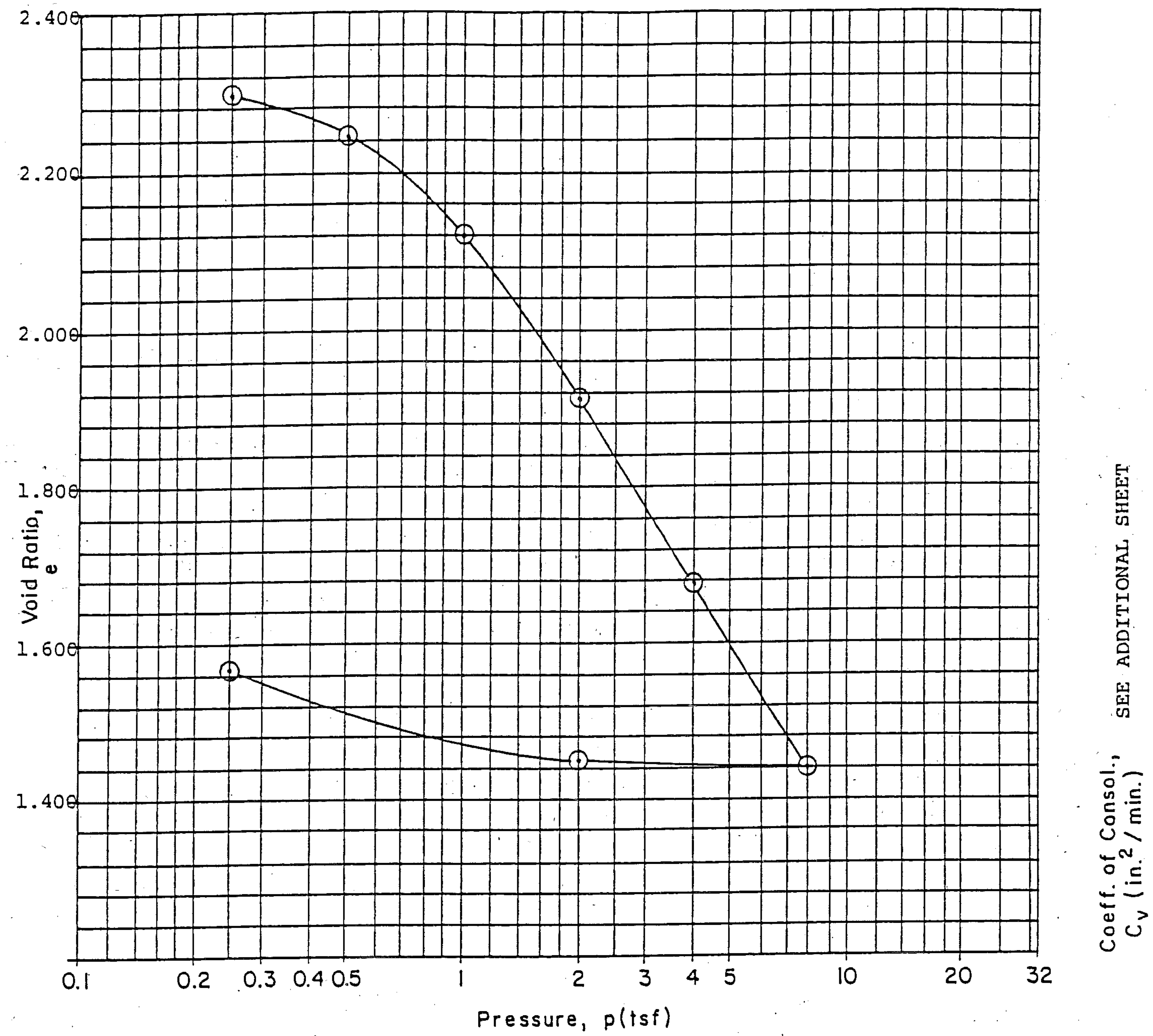
Project DSWA NSWF-2 PHASE III Boring No GF-104A

Sample No U-2 Depth 16.5'-18.5' Date 12/13/89

GANNETT FLEMING GEOTECHNICAL LABORATORY

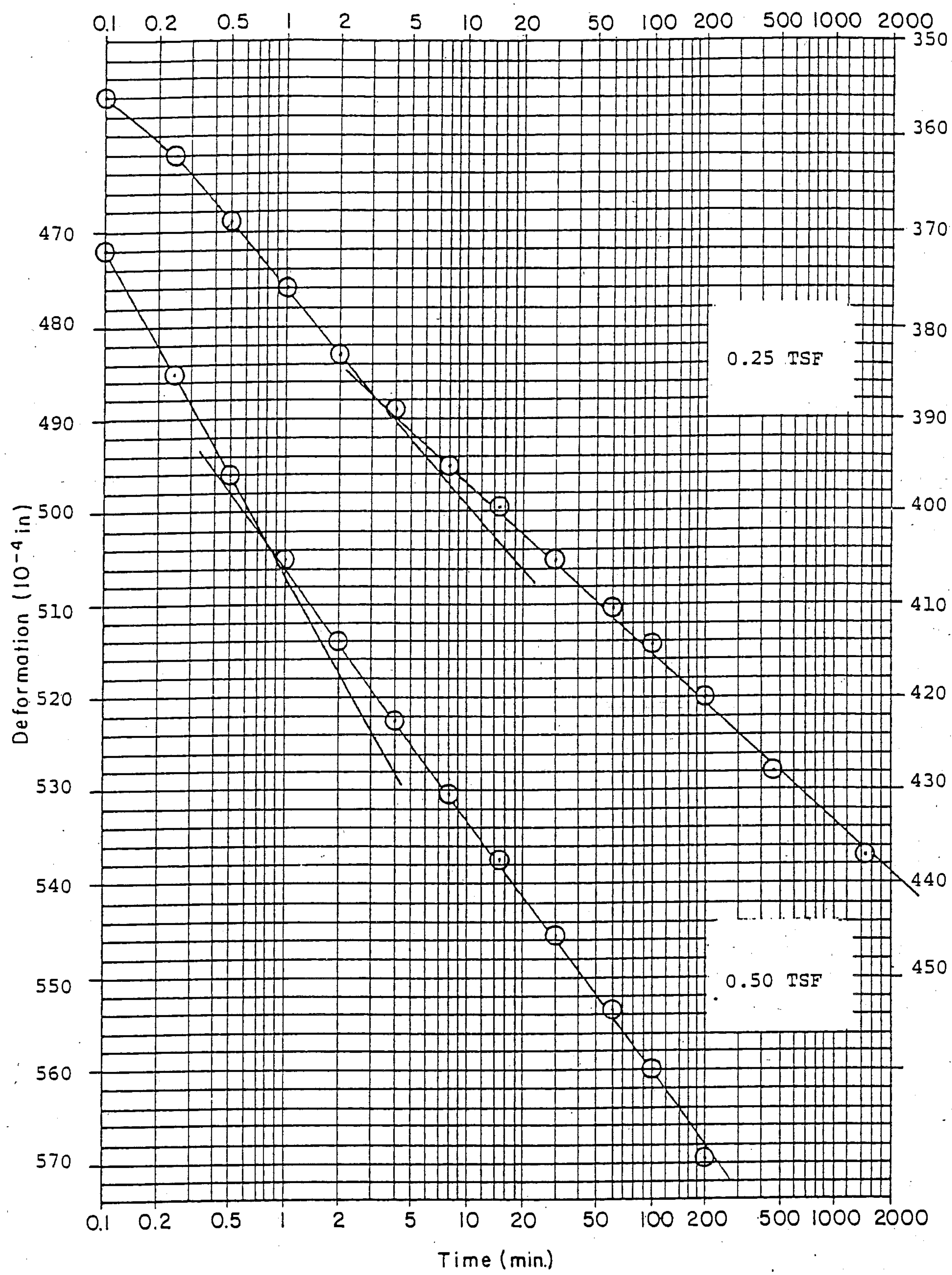


CONSOLIDATION TEST-TIME CURVES			
Project DSWA NSWF-2 PHASE III			Boring No GF-104A
Sample No U-2	Depth 16.5'-18.5'		Date 12/13/89
GANNETT FLEMING GEOTECHNICAL LABORATORY			

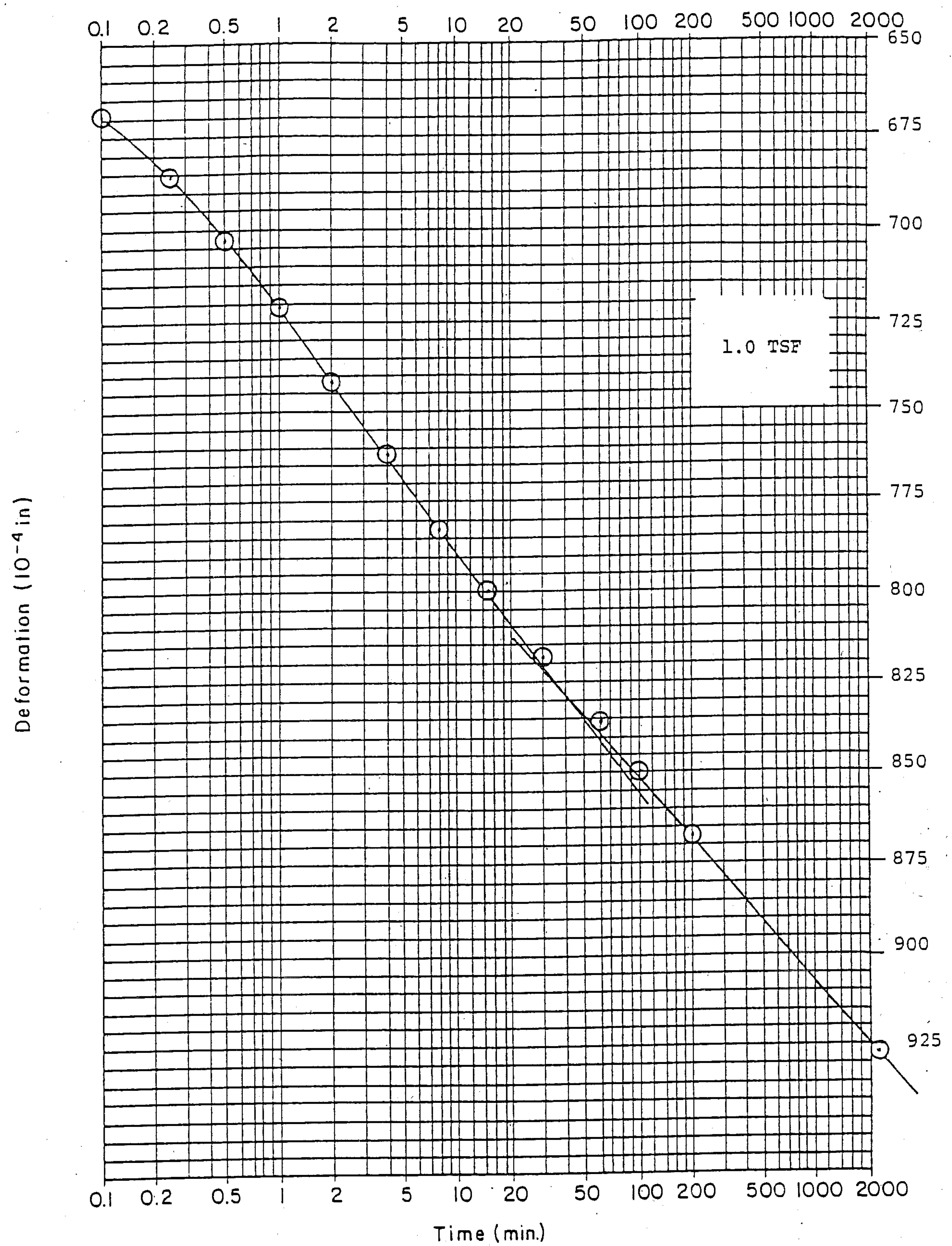


Type of Specimen	Shelby			Before Test			After Test	
Dia	2.50 in	H_T	0.75 in	Water Content	w_o	77.2	w_f	58.0
Compression Index		C_c	0.79	Void Ratio	e_o	2.350	e_f	1.567
Classification	MH			Saturation	S_o	88.7	S_f	100
w_l	75.5	I_p	30.4	Project	DSWA,NSWF,PHASE III			
w_p	45.1	LI	1.1	Boring No	GF-105	Sample No	U-2	
Remarks	Wet density=89.2pcf			Depth	18.5-20.5'	Date	1/90	

GANNETT FLEMING GEOTECHNICAL LABORATORY
CONSOLIDATION TEST REPORT



CONSOLIDATION TEST-TIME CURVES			
Project DSWA, NSWF, PHASE III			Boring No GF-105
Sample No U-2	Depth 18.5'-20.5'		Date 1/24/90
GANNETT FLEMING GEOTECHNICAL LABORATORY			

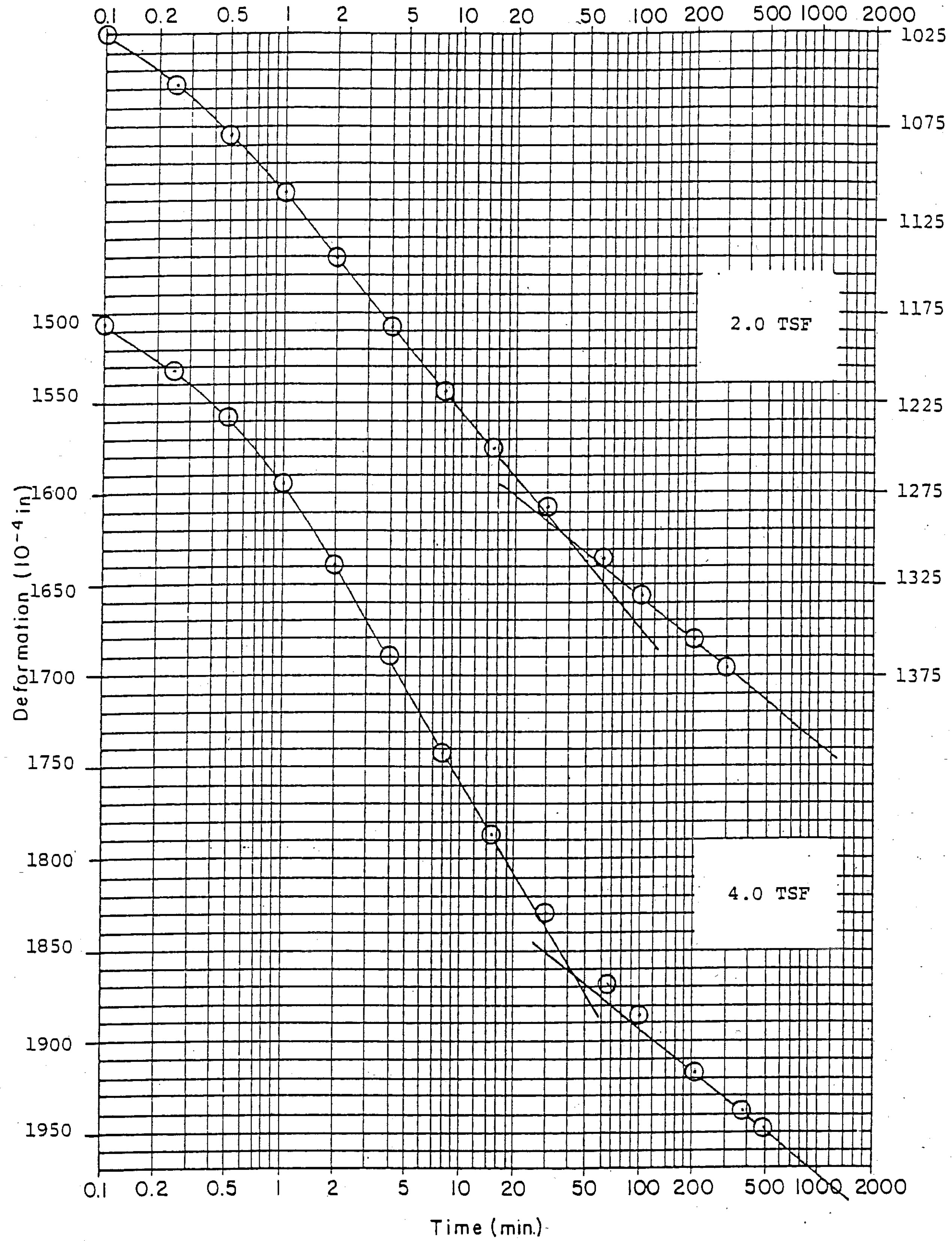


CONSOLIDATION TEST-TIME CURVES

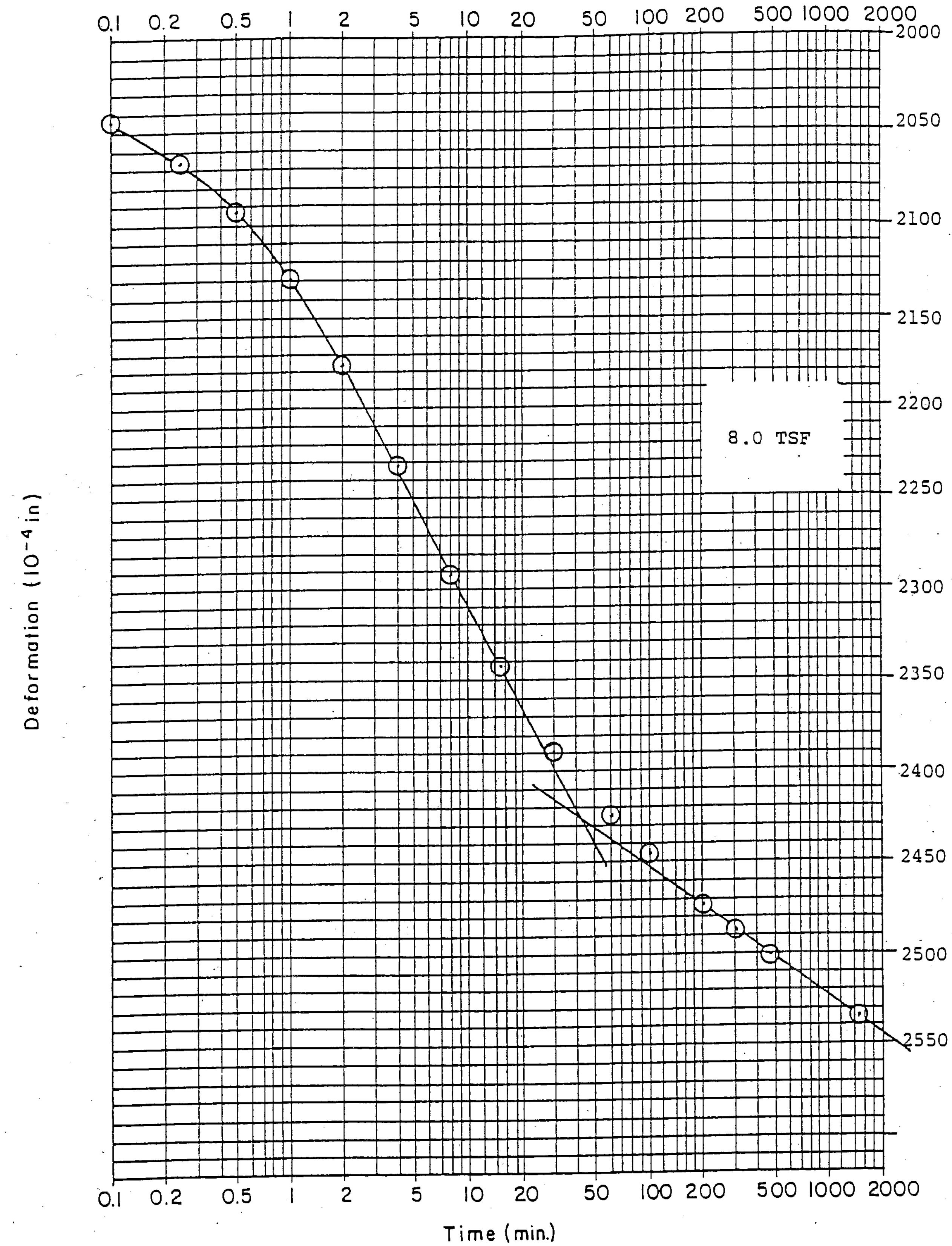
Project DSWA, NSWF, PHASE III Boring No GF-105

Sample No U-2 Depth 18.5'-20.5' Date 1/24/90

GANNETT FLEMING GEOTECHNICAL LABORATORY



CONSOLIDATION TEST-TIME CURVES
Project ' DSWA, NSWF, PHASE III Boring No GF-105
Sample No U-2 Depth 18.5'-20.5' Date 1/24/90
GANNETT FLEMING GEOTECHNICAL LABORATORY

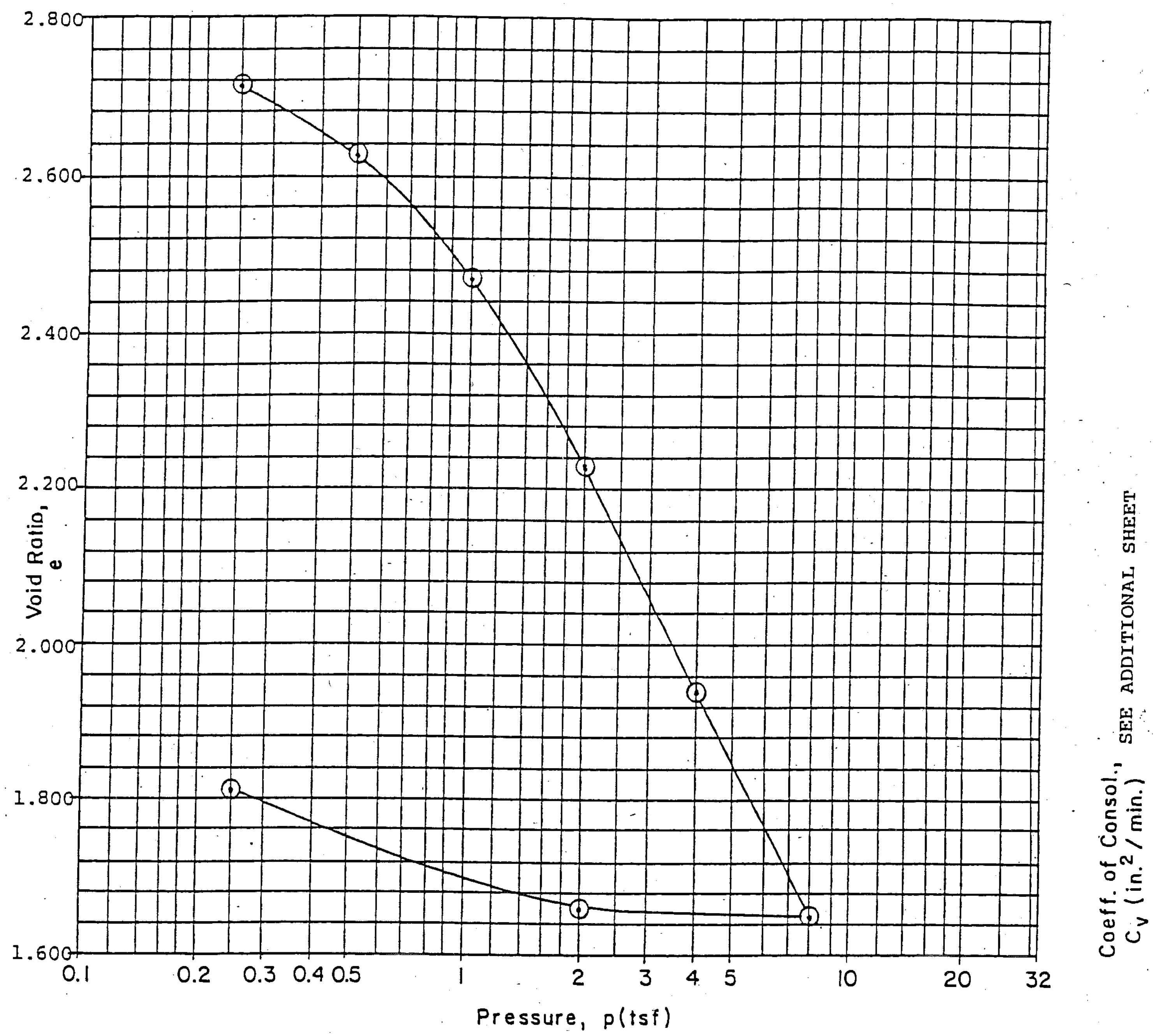


CONSOLIDATION TEST-TIME CURVES

Project ______________________ Boring No ________

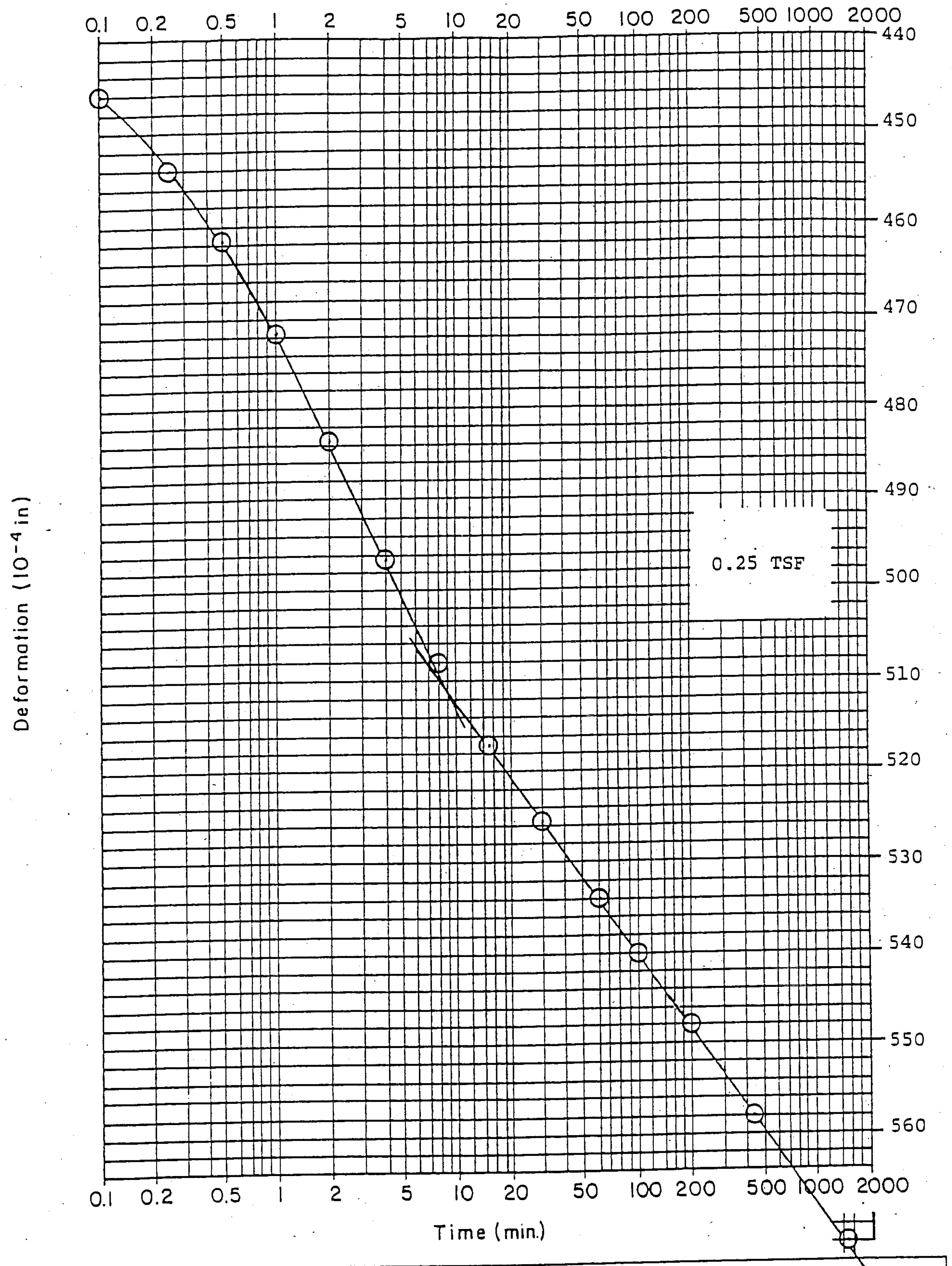
Sample No ____________ Depth ____________ Date ____________

GANNETT FLEMING GEOTECHNICAL LABORATORY



Type of Specimen	Shelby			Before Test			After Test	
Dia	2.50 in	H_T	0.75 in	Water Content	w_o	95.7	w_f	67.0
Compression Index		C_c	0.97	Void Ratio	e_o	2.781	e_f	1.809
Classification	MH			Saturation	S_o	92.9	S_f	100
w_l 72.0		I_p 31.6		Project	DSWA,NSWF,PHASE III			
w_p 40.4		LI 1.8		Boring No	GF-106		Sample No	U-1
Remarks	Wet density=87.3pcf			Depth	8-10'		Date	1/90

GANNETT FLEMING GEOTECHNICAL LABORATORY
CONSOLIDATION TEST REPORT

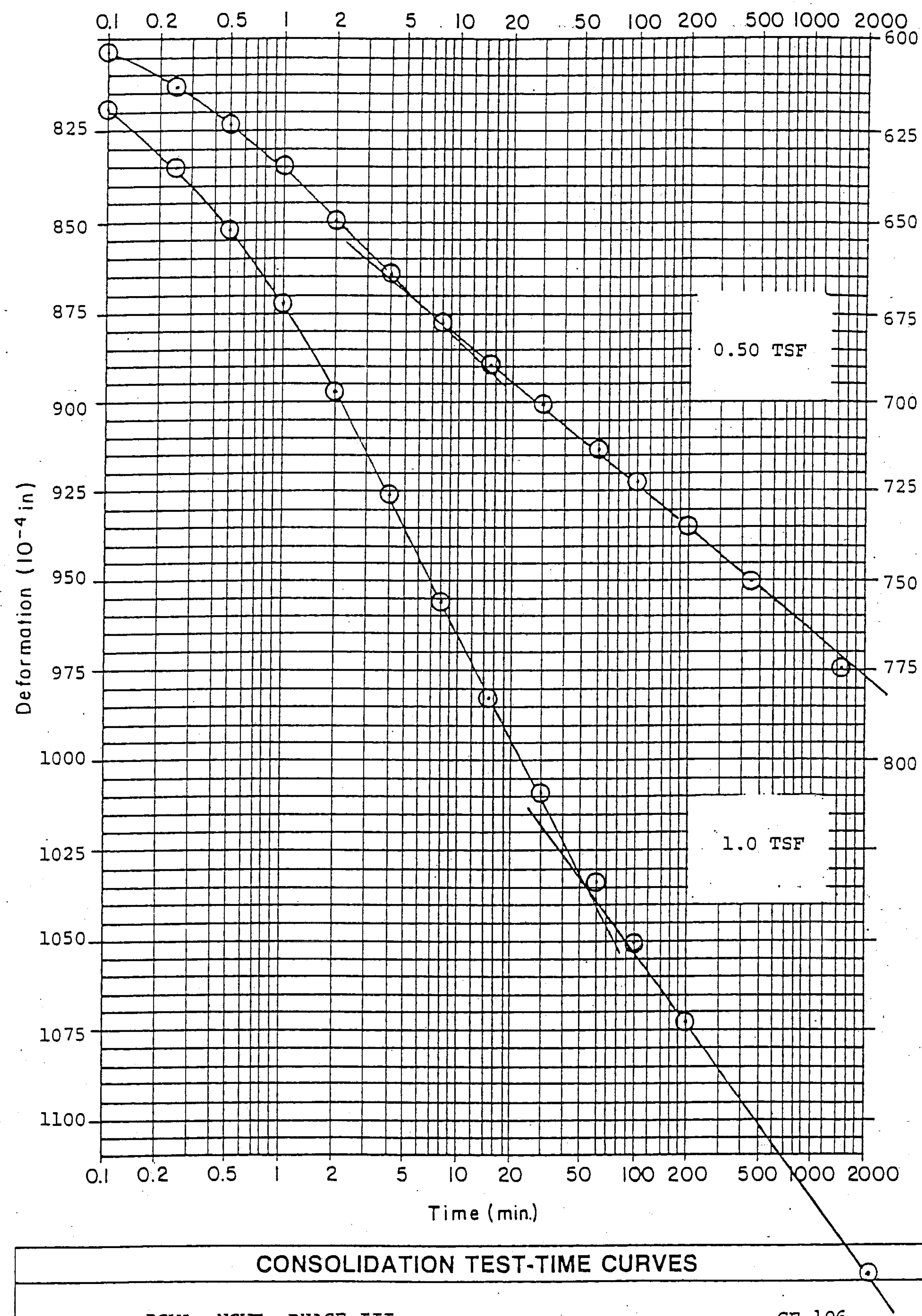


CONSOLIDATION TEST-TIME CURVES

Project DSWA, NSWF, PHASE III Boring No GF-106

Sample No U-1 Depth 8' - 10' Date 1/24/90

GANNETT FLEMING GEOTECHNICAL LABORATORY

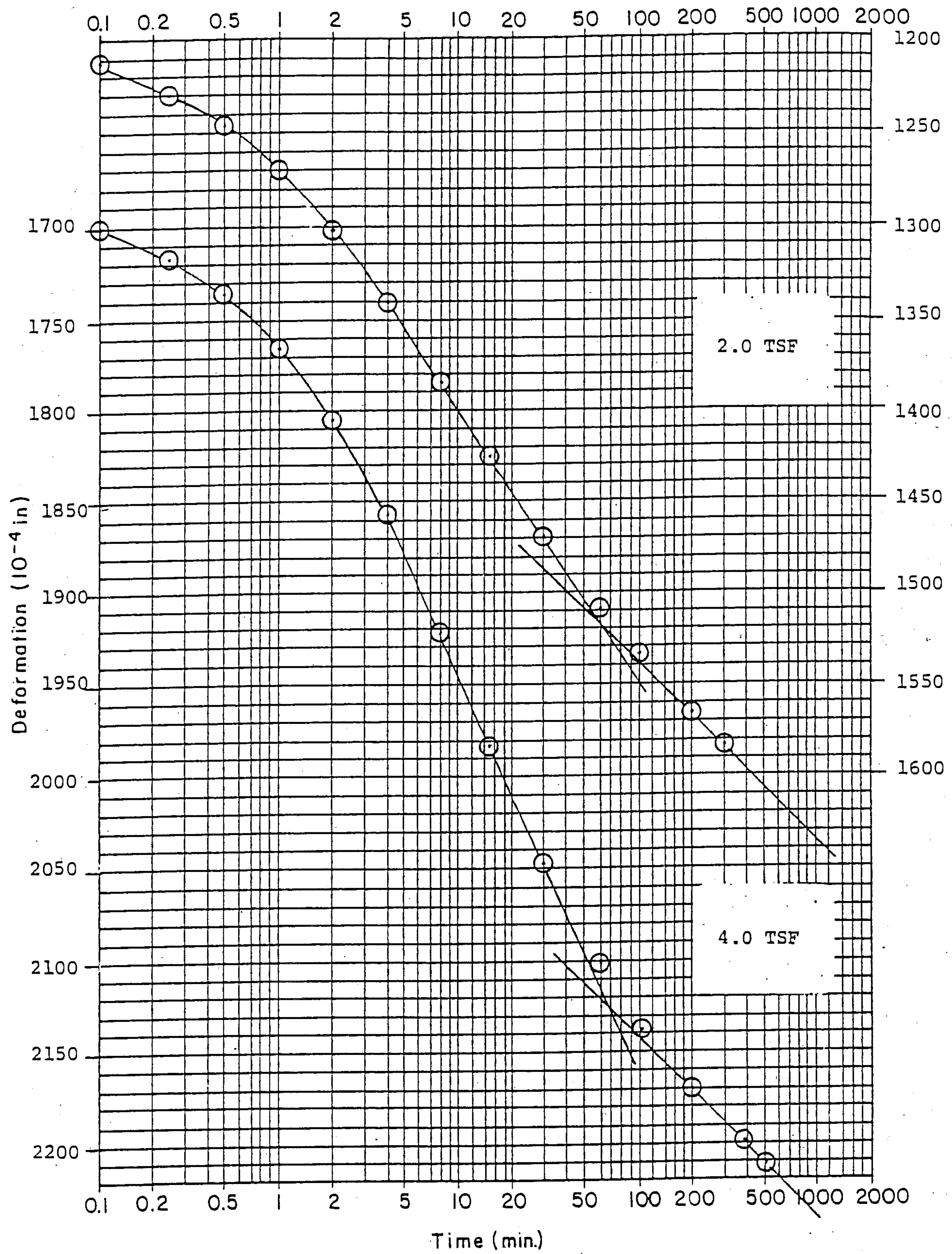


CONSOLIDATION TEST-TIME CURVES

Project DSWA, NSWF, PHASE III Boring No GF-106

Sample No U-1 Depth 8' - 10' Date 1/24/90

GANNETT FLEMING GEOTECHNICAL LABORATORY

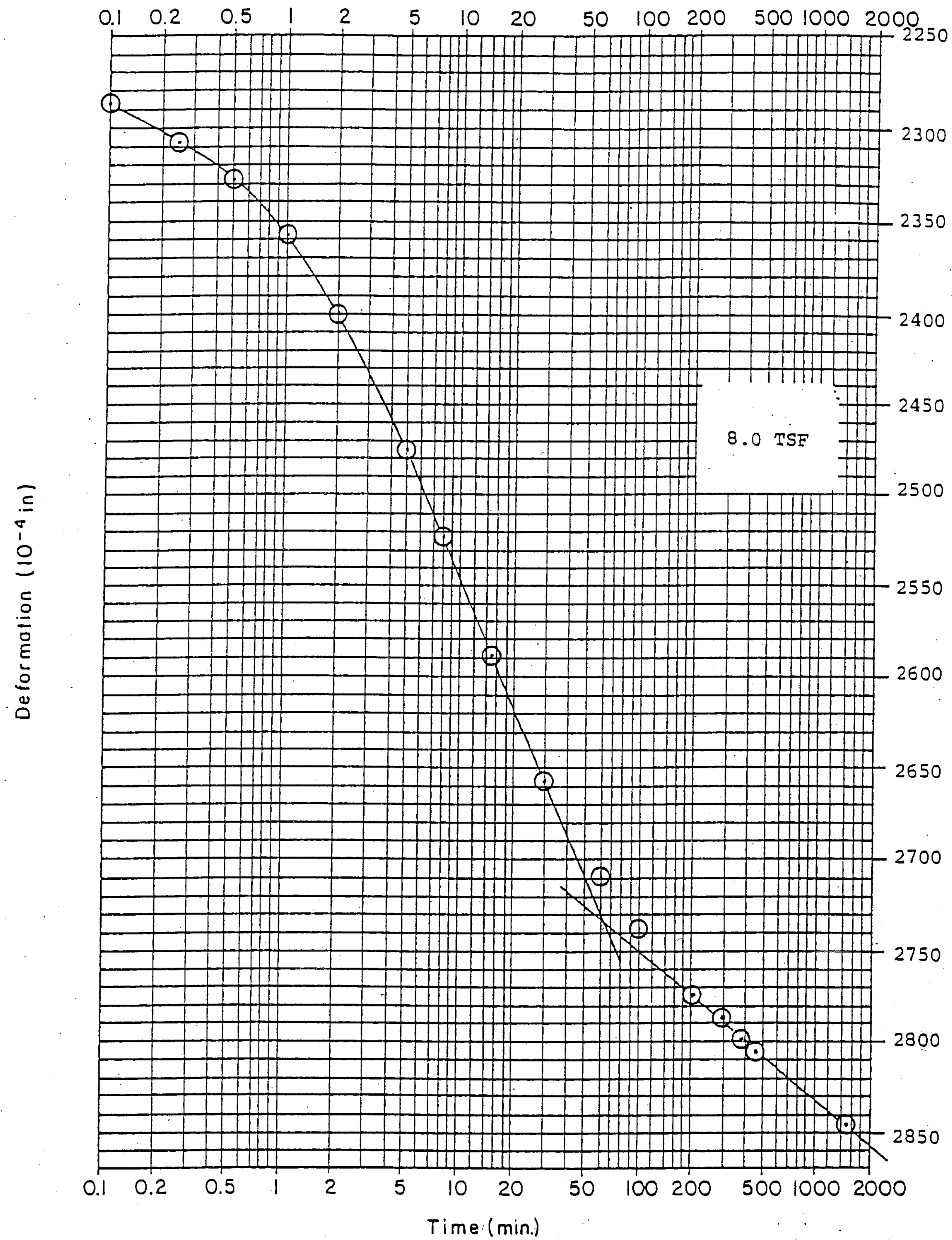


CONSOLIDATION TEST-TIME CURVES

Project ______________________ Boring No ________

Sample No __________ Depth __________ Date __________

GANNETT FLEMING GEOTECHNICAL LABORATORY



CONSOLIDATION TEST-TIME CURVES	
Project DSWA, NSWF, PHASE III	Boring No GF-106
Sample No U-1 Depth 8' - 10'	Date 1/24/90
GANNETT FLEMING GEOTECHNICAL LABORATORY	

MOISTURE DENSITY RELATIONSHIPS

GANNETT FLEMING GEOTECHNICAL LABORATORY

PERMEABILITY SUMMARY SHEET

PROJECT DSWA, NSWF, PHASE III

BOREHOLE GF-104

SAMPLE U-1

DEPTH (FT) 6.5-8.5'

UNIFIED CLASSIFICATION MH

LIQUID LIMIT 65.9

PLASTIC LIMIT 40.9

PLASTICITY INDEX 25.0

SPECIMEN DATA

	BEFORE CONSOLIDATION	AFTER CONSOLIDATION
DIAMETER (IN)	2.82	2.75
HEIGHT (IN)	4.14	4.04
WATER CONTENT (%)	50.6	45.1
VOID RATIO	1.403	1.218
SATURATION (%)	97.4	100
DRY DENSITY (PCF)	70.2	76.0

EFFECTIVE CONSOLIDATION PRESSURE (PSI) Consolidated the same specimen under the four pressures listed below.

PERMEABILITY DATA

CONSOL PRESSURE (KSF)	0.36	0.79	1.25	1.68
CELL PRESSURE (PSI)	53.0	56.0	59.0	62.0
BACK PRESSURE				
AT BOTTOM OF SPECIMEN (PSI)	51.3	51.3	51.3	55.3
AT TOP OF SPECIMEN (PSI)	50.5	50.5	50.3	50.3
HYDRAULIC GRADIENT	6.2	6.7	6.8	34
PERMEABILITY (CM/SEC)	3.5×10^{-6}	4.1×10^{-7}	1.7×10^{-7}	9.8×10^{-8}

GANNETT FLEMING GEOTECHNICAL LABORATORY

PERMEABILITY SUMMARY SHEET

PROJECT DSWA, NSWF, PHASE III

BOREHOLE	GF-105
SAMPLE	U-1
DEPTH (FT)	5-7'

UNIFIED CLASSIFICATION	MH
LIQUID LIMIT	69.5
PLASTIC LIMIT	42.2
PLASTICITY INDEX	27.3

SPECIMEN DATA

	BEFORE CONSOLIDATION	AFTER CONSOLIDATION
DIAMETER (IN)	2.84	2.76
HEIGHT (IN)	4.20	4.11
WATER CONTENT (%)	70.5	71.8
VOID RATIO	2.133	1.940
SATURATION (%)	89.2	100
DRY DENSITY (PCF)	53.8	57.3

EFFECTIVE CONSOLIDATION PRESSURE (PSI) Consolidated the same specimen under the three pressures listed below.

PERMEABILITY DATA

CONSOL PRESSURE(KSF)	0.36	0.79	1.25
CELL PRESSURE (PSI)	53.0	56.0	59.0
BACK PRESSURE			
AT BOTTOM OF SPECIMEN (PSI)	51.3	51.2	51.2
AT TOP OF SPECIMEN (PSI)	50.5	50.5	50.3
HYDRAULIC GRADIENT	5.3	4.8	5.5
PERMEABILITY (CM/SEC)	8.6×10^{-5}	6.7×10^{-5}	7.2×10^{-5}

Note: High permeability due to the fractured structure of the specimen. The fractures are most likely to be the result of numerous wet, dry cycles.

GANNETT FLEMING GEOTECHNICAL LABORATORY

PERMEABILITY SUMMARY SHEET

PROJECT DSWA,NSWF,PHASE III

BOREHOLE GF-106

SAMPLE U-1

DEPTH (FT) 8-10'

UNIFIED CLASSIFICATION ML

LIQUID LIMIT

PLASTIC LIMIT N/P

PLASTICITY INDEX

SPECIMEN DATA /Specimen #1

	BEFORE CONSOLIDATION	AFTER FOURTH CONSOLIDATION
DIAMETER (IN)	2.77	2.76
HEIGHT (IN)	3.74	3.74
WATER CONTENT (%)	69.7	75.3
VOID RATIO	2.047	2.033
SATURATION (%)	91.9	100
DRY DENSITY (PCF)	55.3	55.6

EFFECTIVE CONSOLIDATION PRESSURE (PSI) Consolidated same specimen under the four pressures listed below.

PERMEABILITY DATA

CONSOL PRESSURE (KSF)	0.16	0.45	0.82	1.81
CELL PRESSURE (PSI)	61.8	68.8	76.8	88.8
BACK PRESSURE				
AT BOTTOM OF SPECIMEN (PSI)	60.7	65.9	70.9	75.9
AT TOP OF SPECIMEN (PSI)	61.2	66.6	73.5	78.4
HYDRAULIC GRADIENT	3.7	5.2	19	19
PERMEABILITY (CM/SEC)	2.0×10^{-5}	1.8×10^{-5}	1.5×10^{-5}	1.3 X 1C

GANNETT FLEMING GEOTECHNICAL LABORATORY

PERMEABILITY SUMMARY SHEET

PROJECT DSWA,NSWF,PHASE III

BOREHOLE	GF-107
SAMPLE	U-1
DEPTH (FT)	20-22

UNIFIED CLASSIFICATION	MH
LIQUID LIMIT	66.3
PLASTIC LIMIT	43.3
PLASTICITY INDEX	23.0

SPECIMEN DATA

	BEFORE CONSOLIDATION	AFTER CONSOLIDATION
DIAMETER (IN)	2.90	2.65
HEIGHT (IN)	4.36	4.00
WATER CONTENT (%)	83.0	60.7
VOID RATIO	2.449	1.639
SATURATION (%)	91.5	100
DRY DENSITY (PCF)	48.9	63.9

EFFECTIVE CONSOLIDATION PRESSURE (PSI) Consolidated the same specimen under the two pressures listed below.

PERMEABILITY DATA

CONSOL PRESSURE (KSF)	0.36	1.68
CELL PRESSURE (PSI)	53.0	62.0
BACK PRESSURE		
AT BOTTOM OF SPECIMEN (PSI)	51.3	56.3
AT TOP OF SPECIMEN (PSI)	50.3	50.3
HYDRAULIC GRADIENT	6.6	41
PERMEABILITY (CM/SEC)	1.3×10^{-7}	5.3×10^{-8}

GANNETT FLEMING GEOTECHNICAL LABORATORY

PERMEABILITY SUMMARY SHEET

PROJECT DSWA,NSWF,PHASE III

BOREHOLE	GF-109
SAMPLE	U-1
DEPTH (FT)	14-16

UNIFIED CLASSIFICATION	MH
LIQUID LIMIT	
PLASTIC LIMIT	
PLASTICITY INDEX	

SPECIMEN DATA

	BEFORE CONSOLIDATION	AFTER CONSOLIDATION
DIAMETER (IN)	2.88	2.65
HEIGHT (IN)	4.07	3.76
WATER CONTENT (%)	84.1	61.2
VOID RATIO	2.382	1.654
SATURATION (%)	95.3	100
DRY DENSITY (PCF)	49.8	63.6

EFFECTIVE CONSOLIDATION PRESSURE (PSI) Consolidated the same specimen under the two pressures listed below.

PERMEABILITY DATA

CONSOL PRESSURE (KSF)	0.36	1.68
CELL PRESSURE (PSI)	53.0	62.0
BACK PRESSURE		
AT BOTTOM OF SPECIMEN (PSI)	51.3	56.3
AT TOP OF SPECIMEN (PSI)	50.3	50.4
HYDRAULIC GRADIENT	7.0	44
PERMEABILITY (CM/SEC)	1.7×10^{-7}	5.2×10^{-8}

GANNETT FLEMING GEOTECHNICAL LABORATORY

PERMEABILITY SUMMARY SHEET

PROJECT DSWA, NSWF, PHASE III

BOREHOLE GF-110

SAMPLE U-1

DEPTH (FT) 10-12'

UNIFIED CLASSIFICATION MH

LIQUID LIMIT 67.1

PLASTIC LIMIT 45.4

PLASTICITY INDEX 21.7

SPECIMEN DATA

	BEFORE CONSOLIDATION	AFTER CONSOLIDATION
DIAMETER (IN)	2.84	2.78
HEIGHT (IN)	4.00	3.90
WATER CONTENT (%)	62.5	57.5
VOID RATIO	1.727	1.552
SATURATION (%)	97.7	100
DRY DENSITY (PCF)	61.8	66.1

EFFECTIVE CONSOLIDATION PRESSURE (PSI) Consolidated the same specimen under the three pressures listed below.

PERMEABILITY DATA

CONSOL PRESSURE (KSF)	0.36	0.79	1.25
CELL PRESSURE (PSI)	53.0	56.0	59.0
BACK PRESSURE			
AT BOTTOM OF SPECIMEN (PSI)	51.2	51.2	51.2
AT TOP OF SPECIMEN (PSI)	50.5	50.5	50.3
HYDRAULIC GRADIENT	4.8	5.5	5.9
PERMEABILITY (CM/SEC)	2.5×10^{-4}	1.5×10^{-4}	1.1×10^{-4}

Note: Specimen contained a cylindrically shaped stained region. This stained cylinder had a diameter of about ¼ to ½ inches, and was oriented vertically in the sample. The stained cylinder probably represents a piece of decayed phragmites. Its open structure very probably produced the high permeabilities.

GANNETT FLEMING GEOTECHNICAL LABORATORY

PERMEABILITY SUMMARY SHEET

PROJECT DSWA, NSWF, PHASE III

BOREHOLE GF-104

SAMPLE Bag

DEPTH (FT) 0-1

UNIFIED CLASSIFICATION MH

LIQUID LIMIT 79.9

PLASTIC LIMIT 54.6

PLASTICITY INDEX 25.3

SPECIMEN DATA

	BEFORE CONSOLIDATION	AFTER CONSOLIDATION
DIAMETER (IN)	4.00	3.89
HEIGHT (IN)	4.60	4.47
WATER CONTENT (%)	48.6	48.4
VOID RATIO	1.510	1.308
SATURATION (%)	86.7	100
DRY DENSITY (PCF)	67.2	73.0

EFFECTIVE CONSOLIDATION PRESSURE (PSI) (SEE BELOW)

PERMEABILITY DATA

CONSOL PRESSURE (KSF)	0.91	1.90	3.61
CELL PRESSURE (PSI)	47.0	59.0	76.0
BACK PRESSURE			
AT BOTTOM OF SPECIMEN (PSI)	42.2	51.3	60.3
AT TOP OF SPECIMEN (PSI)	40.4	45.4	50.4
HYDRAULIC GRADIENT	11	36	61
PERMEABILITY (CM/SEC)	3.2×10^{-7}	1.7×10^{-7}	6.3×10^{-8}

GANNETT FLEMING GEOTECHNICAL LABORATORY

PERMEABILITY SUMMARY SHEET

PROJECT DELAWARE SOLID WASTE AUTHORITY, NSWF, PHASE III

BOREHOLE GF-105

SAMPLE BAG

DEPTH (FT) 0'-1'

UNIFIED CLASSIFICATION	MH
LIQUID LIMIT	71.2
PLASTIC LIMIT	49.5
PLASTICITY INDEX	21.7

SPECIMEN DATA

	BEFORE CONSOLIDATION	AFTER CONSOLIDATION
DIAMETER (IN)	4.00	3.91
HEIGHT (IN)	3.32	3.25
WATER CONTENT (%)	52.0	55.5
VOID RATIO	1.668	1.490
SATURATION (%)	84.2	100
DRY DENSITY (PCF)	63.2	67.2

EFFECTIVE CONSOLIDATION PRESSURE (PSI) (SEE BELOW)

PERMEABILITY DATA

CON. PRESSURE (KSF)	0.76	0.95
CELL PRESSURE (PSI)	76.0	83.0
BACK PRESSURE		
AT BOTTOM OF SPECIMEN (PSI)	72.2	78.1
AT TOP OF SPECIMEN (PSI)	70.7	76.4
HYDRAULIC GRADIENT	16	15
PERMEABILITY (CM/SEC)	2.5×10^{-6}	2.0×10^{-6}

GANNETT FLEMING GEOTECHNICAL LABORATORY

PERMEABILITY SUMMARY SHEET

PROJECT DSWA, NSWF, PHASE III

BOREHOLE GF-105

SAMPLE Bag

DEPTH (FT) 0-1

UNIFIED CLASSIFICATION MH

LIQUID LIMIT 71.2

PLASTIC LIMIT 49.5

PLASTICITY INDEX 21.7

SPECIMEN DATA

	BEFORE CONSOLIDATION	AFTER CONSOLIDATION
DIAMETER (IN)	4.01	3.85
HEIGHT (IN)	4.60	4.42
WATER CONTENT (%)	46.7	41.4
VOID RATIO	1.380	1.119
SATURATION (%)	91.4	100
DRY DENSITY (PCF)	70.8	79.5

EFFECTIVE CONSOLIDATION PRESSURE (PSI) (SEE BELOW)

PERMEABILITY DATA

CONSOL PRESSURE (KSF)	0.91	1.90	3.61
CELL PRESSURE (PSI)	47.0	59.0	76.0
BACK PRESSURE			
AT BOTTOM OF SPECIMEN (PSI)	42.3	51.3	60.3
AT TOP OF SPECIMEN (PSI)	40.3	45.3	50.4
HYDRAULIC GRADIENT	12	37	62
PERMEABILITY (CM/SEC)	2.4×10^{-8}	2.4×10^{-8}	1.4×10^{-8}

GANNETT FLEMING GEOTECHNICAL LABORATORY

PERMEABILITY SUMMARY SHEET

PROJECT DELAWARE SOLID WASTE AUTHORITY, NSWF, PHASE III

BOREHOLE	GF-106
SAMPLE	BAG
DEPTH (FT)	0'-1'

UNIFIED CLASSIFICATION	MH
LIQUID LIMIT	53.2
PLASTIC LIMIT	38.1
PLASTICITY INDEX	15.1

SPECIMEN DATA

	BEFORE CONSOLIDATION	AFTER CONSOLIDATION
DIAMETER (IN)	4.02	4.00
HEIGHT (IN)	3.89	3.87
WATER CONTENT (%)	52.0	55.1
VOID RATIO	1.518	1.488
SATURATION (%)	92.5	100
DRY DENSITY (PCF)	67.0	67.7

EFFECTIVE CONSOLIDATION PRESSURE (PSI) (SEE BELOW)

PERMEABILITY DATA

CON. PRESSURE (KSF)	0.76	0.95
CELL PRESSURE (PSI)	76.0	83.0
BACK PRESSURE		
AT BOTTOM OF SPECIMEN (PSI)	72.2	78.0
AT TOP OF SPECIMEN (PSI)	70.7	76.4
HYDRAULIC GRADIENT	14	13
PERMEABILITY (CM/SEC)	2.8×10^{-7}	2.7×10^{-7}

GANNETT FLEMING GEOTECHNICAL LABORATORY

PERMEABILITY SUMMARY SHEET

PROJECT DELAWARE SOLID WASTE AUTHORITY, NSWF, PHASE III

BOREHOLE GF-108

SAMPLE BAG

DEPTH (FT) 0'-1'

UNIFIED CLASSIFICATION	MH
LIQUID LIMIT	55.6
PLASTIC LIMIT	38.3
PLASTICITY INDEX	17.3

SPECIMEN DATA

	BEFORE CONSOLIDATION	AFTER CONSOLIDATION
DIAMETER (IN)	4.00	3.79
HEIGHT (IN)	4.56	4.32
WATER CONTENT (%)	54.1	44.8
VOID RATIO	1.608	1.210
SATURATION (%)	90.8	100
DRY DENSITY (PCF)	64.6	76.3

EFFECTIVE CONSOLIDATION PRESSURE (PSI) (SEE BELOW)

PERMEABILITY DATA

CON. PRESSURE (KSF)	0.75	1.80
CELL PRESSURE (PSI)	76.0	89.0
BACK PRESSURE		
AT BOTTOM OF SPECIMEN (PSI)	72.3	78.4
AT TOP OF SPECIMEN (PSI)	70.8	76.4
HYDRAULIC GRADIENT	12	12
PERMEABILITY (CM/SEC)	2.7×10^{-7}	1.4×10^{-7}

GANNETT FLEMING GEOTECHNICAL LABORATORY

PERMEABILITY SUMMARY SHEET

PROJECT DSWA, NSWF, PHASE III

BOREHOLE	GF-108
SAMPLE	Bag
DEPTH (FT)	0-1

UNIFIED CLASSIFICATION	MH
LIQUID LIMIT	55.6
PLASTIC LIMIT	38.3
PLASTICITY INDEX	17.3

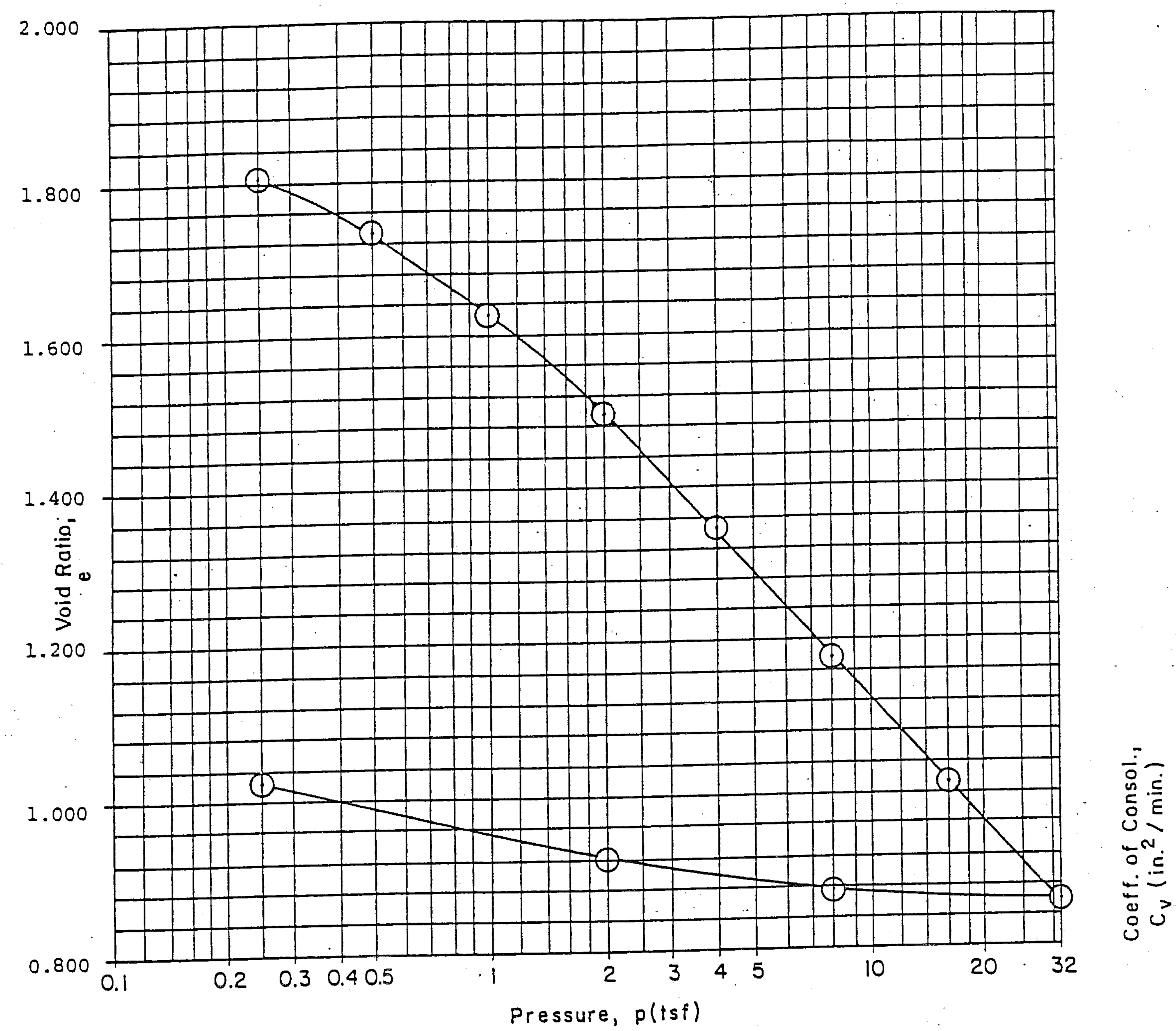
SPECIMEN DATA

	BEFORE CONSOLIDATION	AFTER CONSOLIDATION
DIAMETER (IN)	4.01	3.76
HEIGHT (IN)	4.57	4.29
WATER CONTENT (%)	42.2	31.8
VOID RATIO	1.247	0.857
SATURATION (%)	91.3	100
DRY DENSITY (PCF)	75.0	90.8

EFFECTIVE CONSOLIDATION PRESSURE (PSI) (SEE BELOW)

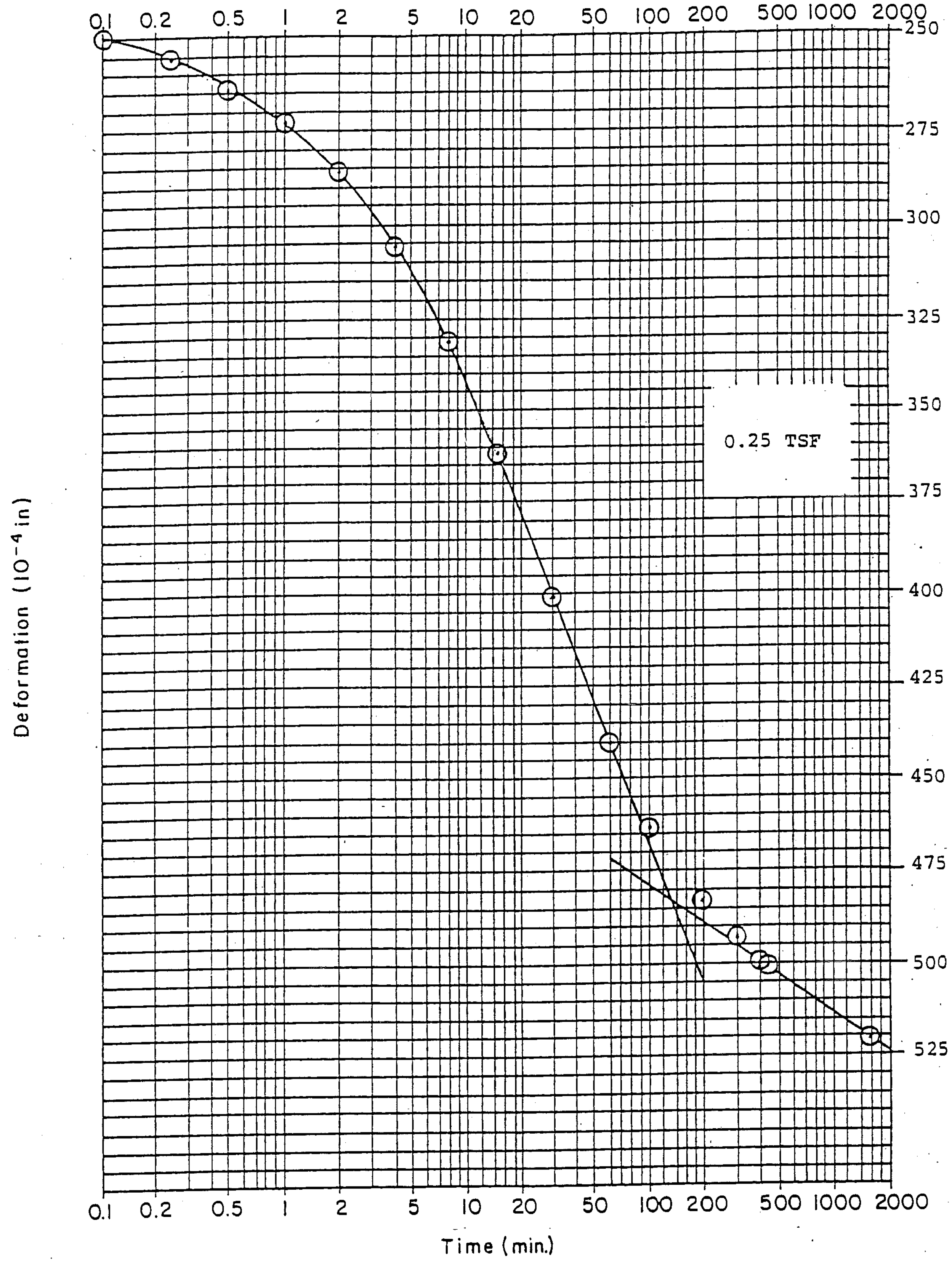
PERMEABILITY DATA

CONSOL PRESSURE (KSF)	0.91	1.90	3.61
CELL PRESSURE (PSI)	47.0	59.0	76.0
BACK PRESSURE			
AT BOTTOM OF SPECIMEN (PSI)	42.3	51.3	60.3
AT TOP OF SPECIMEN (PSI)	40.4	45.3	50.3
HYDRAULIC GRADIENT	12	38	64
PERMEABILITY (CM/SEC)	3.8×10^{-8}	2.9×10^{-8}	2.0×10^{-8}

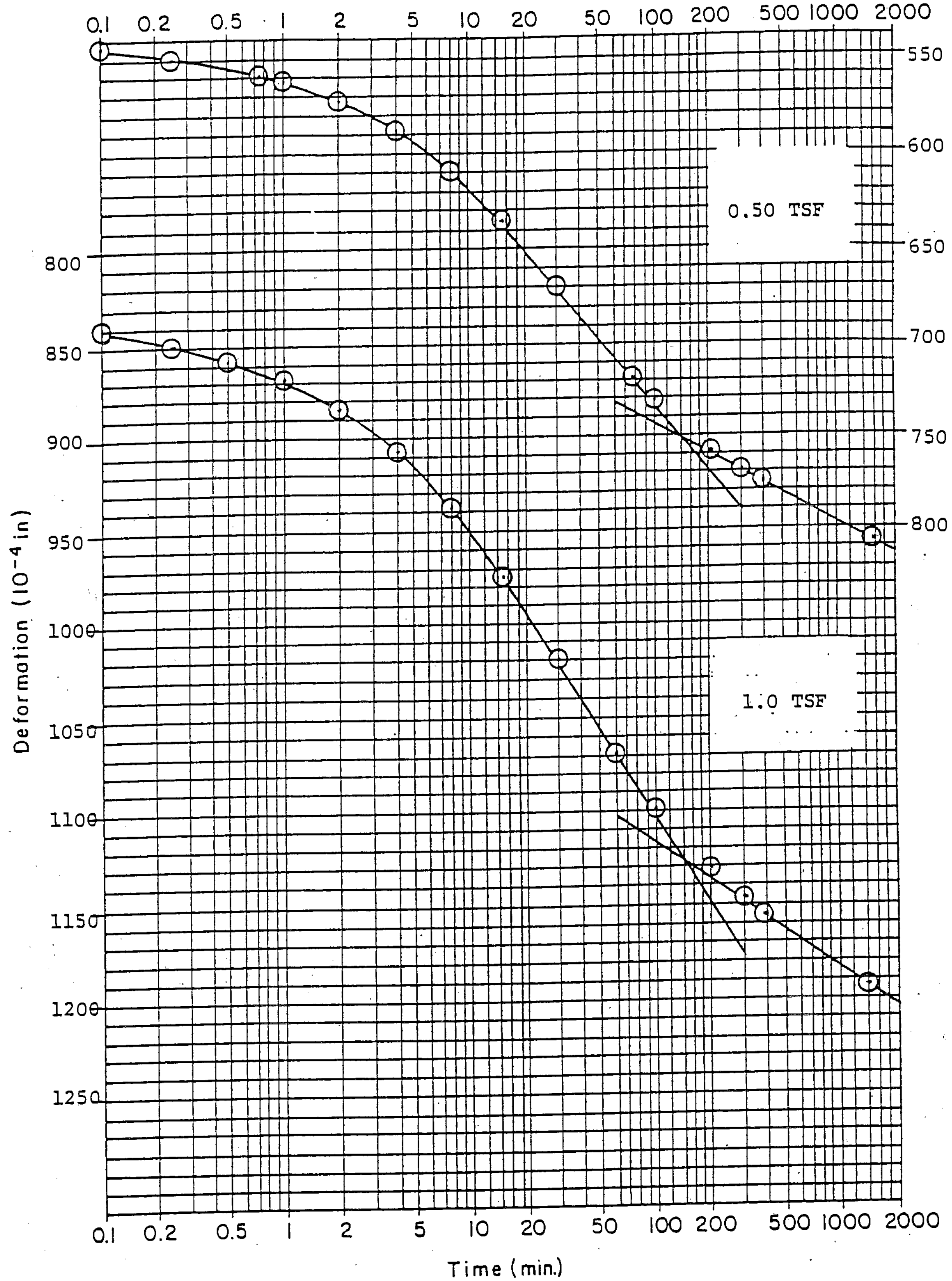


Type of Specimen	SHELBY TUBE			Before Test			After Test	
Dia	2.50 in	H_T	1.00 in	Water Content	w_o	66.5	w_f	38.0
Compression Index		C_c	0.53	Void Ratio	e_o	1.883	e_f	1.026
Classification	BRN.-SILT WITH SAND			Saturation	S_o	95.4	S_f	100.0
w_l	49.1	I_p	16.2	Project	DSWA, NSWF, PHASE III			
w_p	32.9	LI	2.1	Boring No	GF-107	Sample No	U-2	
Remarks	WET DENSITY = 97.4 pcf			Depth	55' - 57'	Date	2/5/90	

GANNETT FLEMING GEOTECHNICAL LABORATORY
CONSOLIDATION TEST REPORT



CONSOLIDATION TEST-TIME CURVES	
Project DSWA, NSWF, PHASE III	Boring No GF-107
Sample No U-2 Depth 55 - 57'	Date 1/29/90
GANNETT FLEMING GEOTECHNICAL LABORATORY	

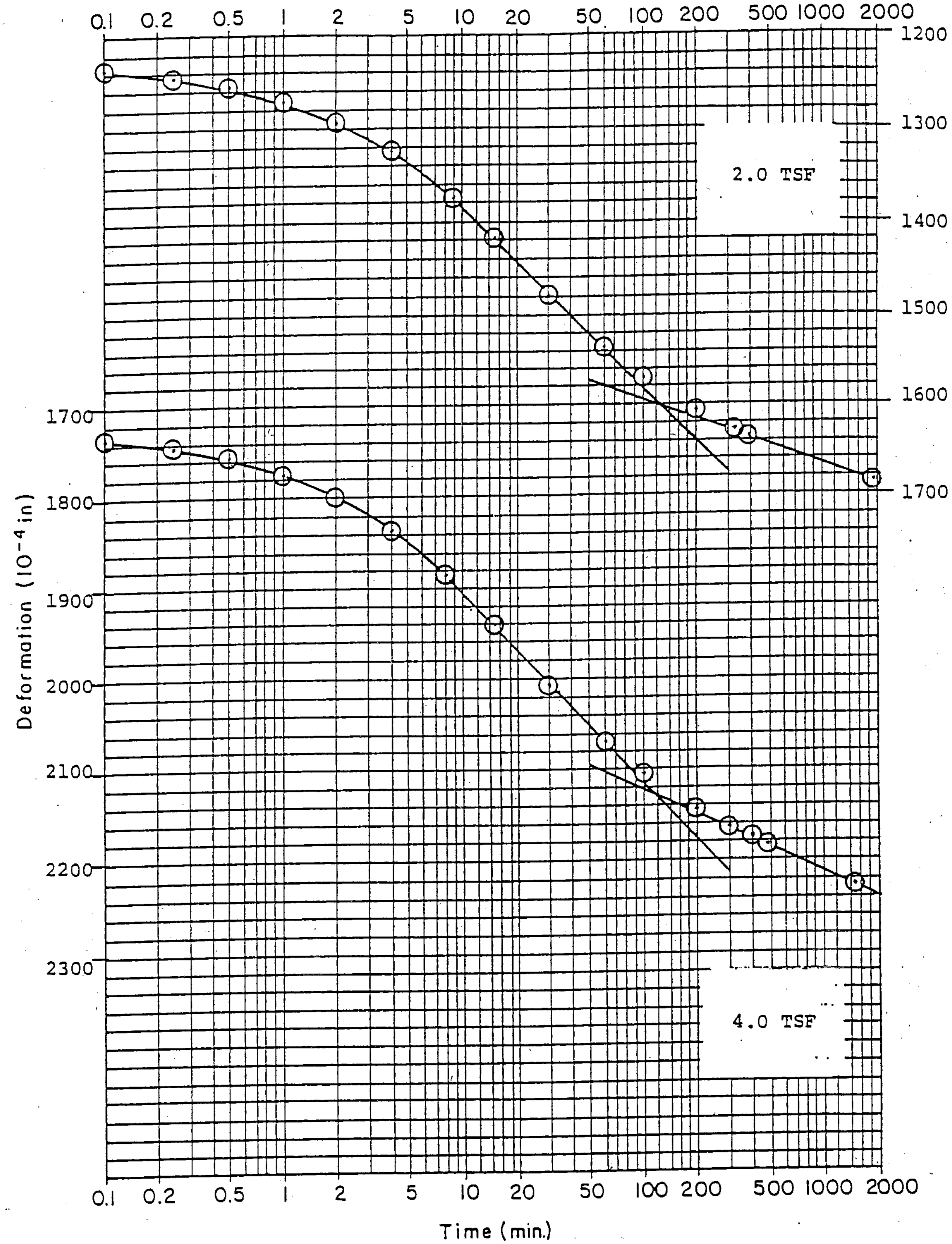


CONSOLIDATION TEST-TIME CURVES

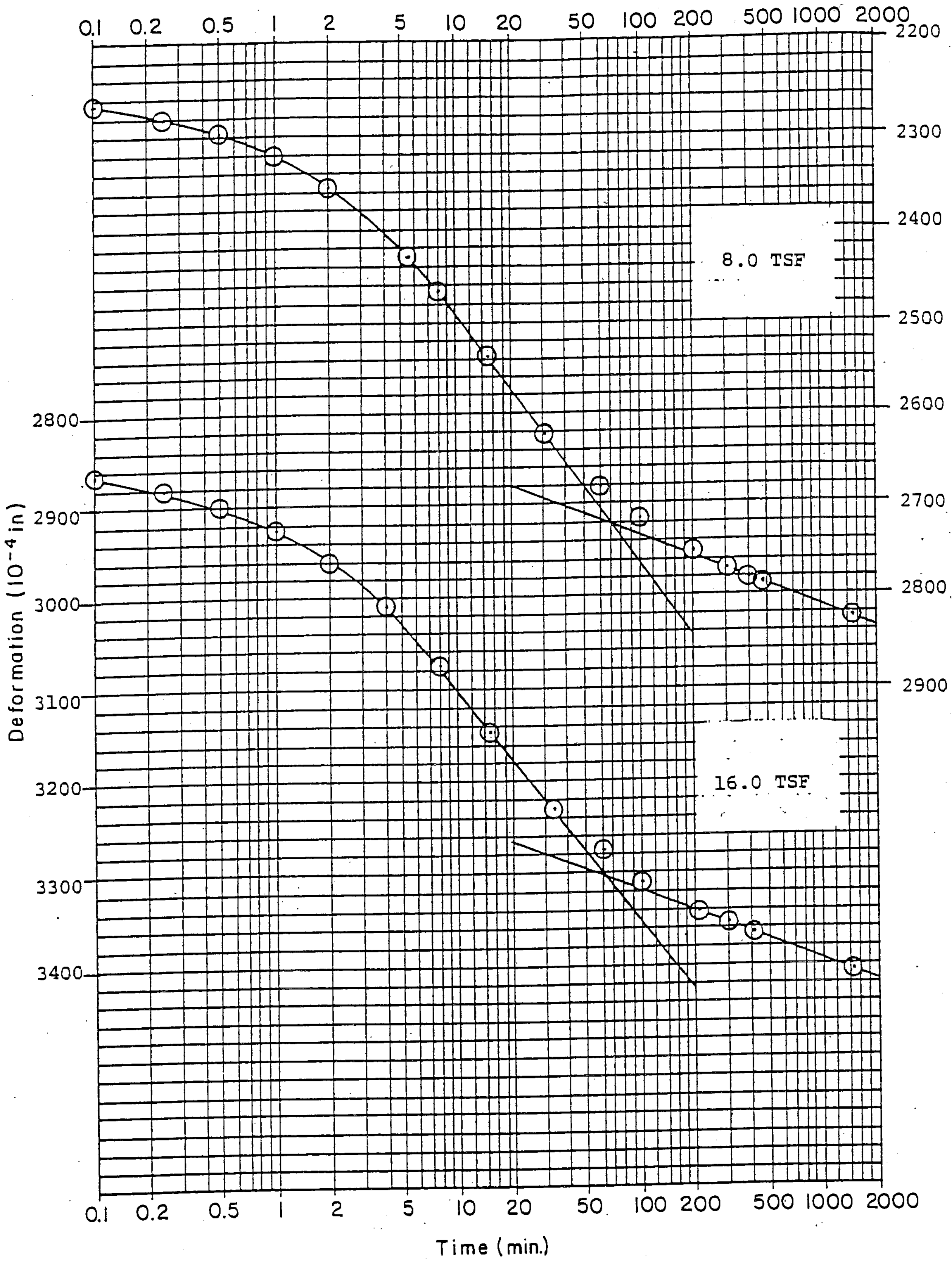
Project DSWA, NSWF, PHASE III Boring No GF-107

Sample No U-2 Depth 55 - 57' Date 1/29/90

GANNETT FLEMING GEOTECHNICAL LABORATORY



CONSOLIDATION TEST-TIME CURVES			
Project DSWA, NSWF, PHASE III		Boring No GF-107	
Sample No U-2	Depth 55 - 57'	Date 1/29/90	
GANNETT FLEMING GEOTECHNICAL LABORATORY			

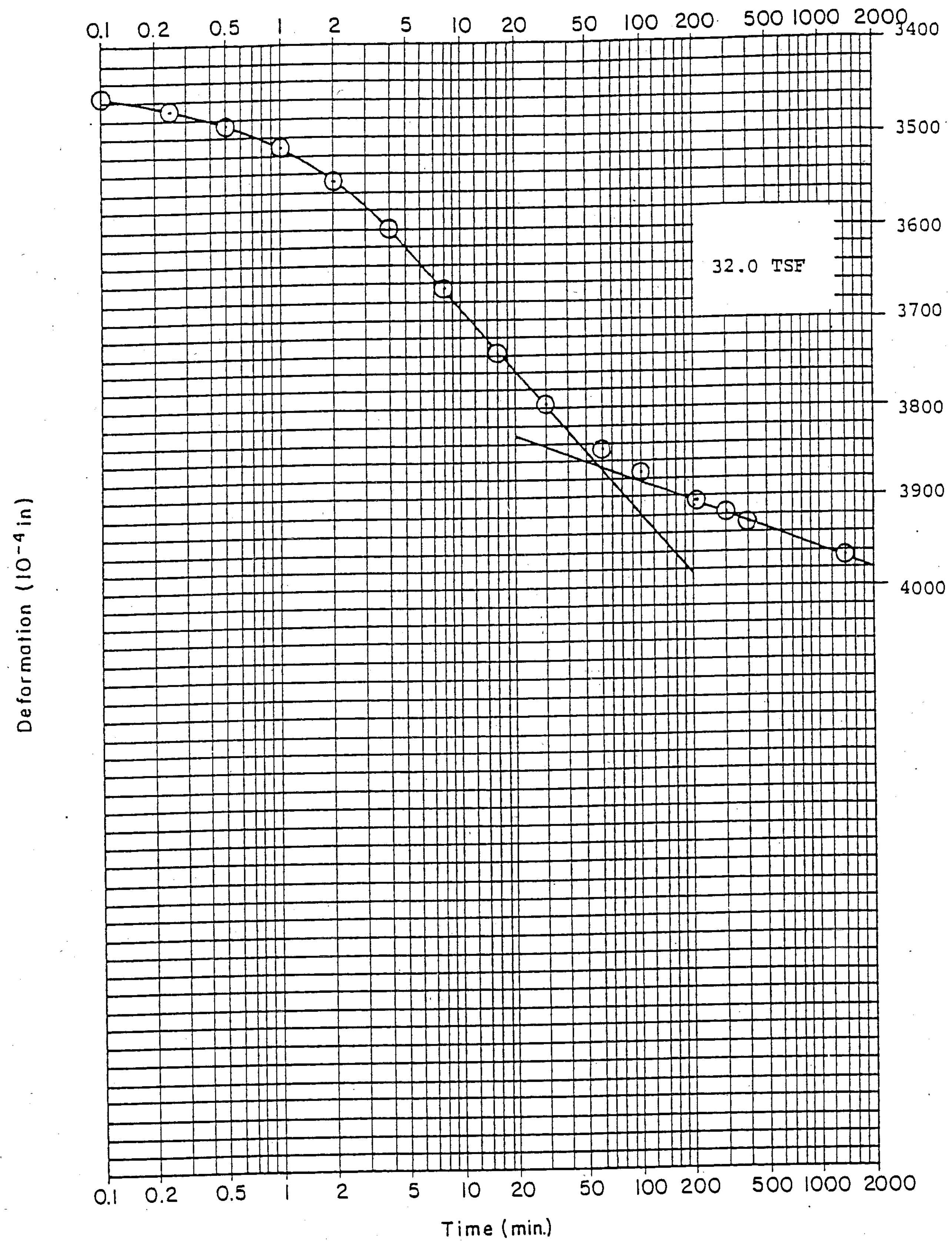


CONSOLIDATION TEST-TIME CURVES

Project DSWA, NSWF, PHASE III Boring No GF-107

Sample No U-2 Depth 55 - 57' Date 1/29/90

GANNETT FLEMING GEOTECHNICAL LABORATORY

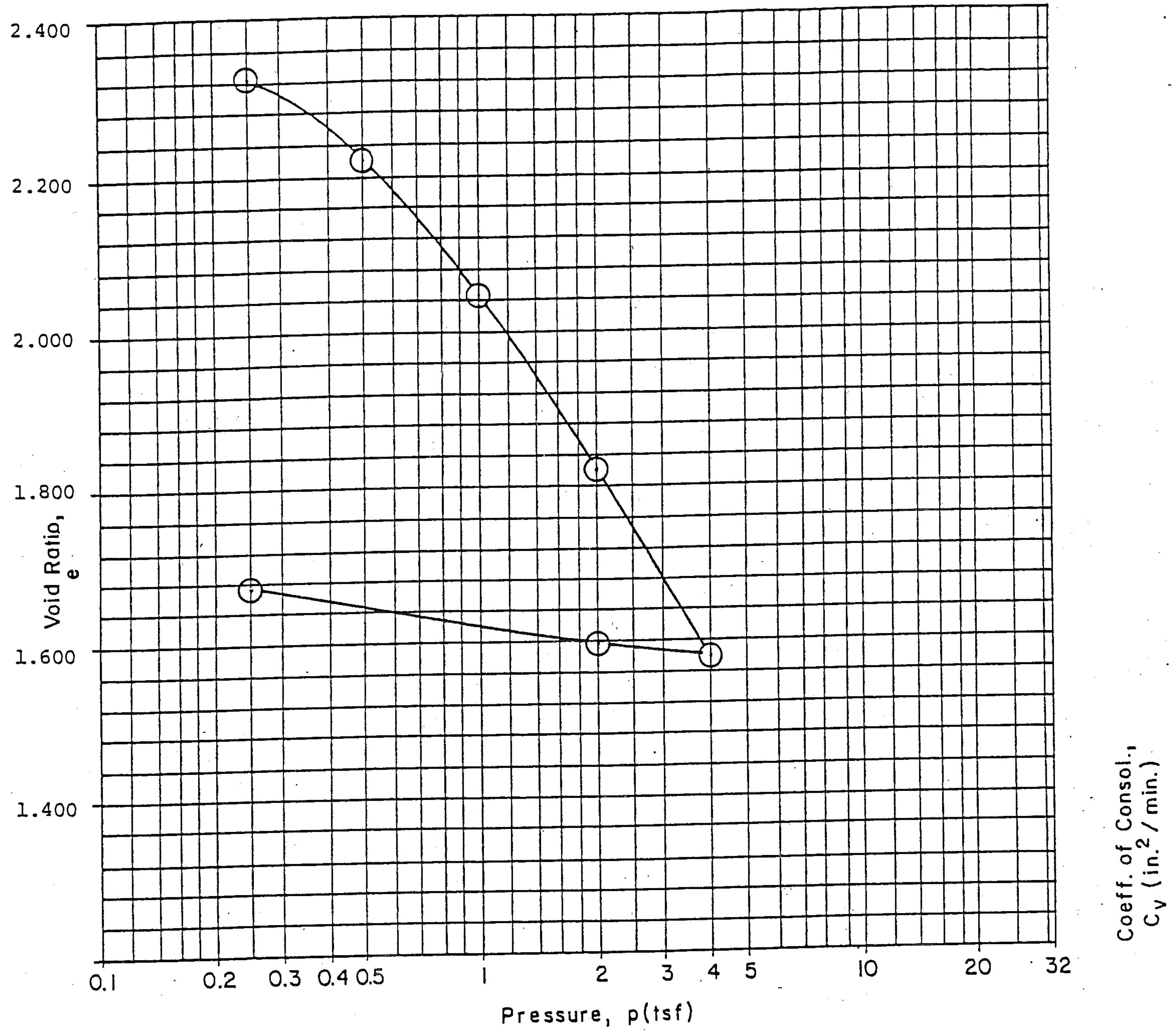


CONSOLIDATION TEST-TIME CURVES

Project DSWA, NSWF, PHASE III Boring No GF-107

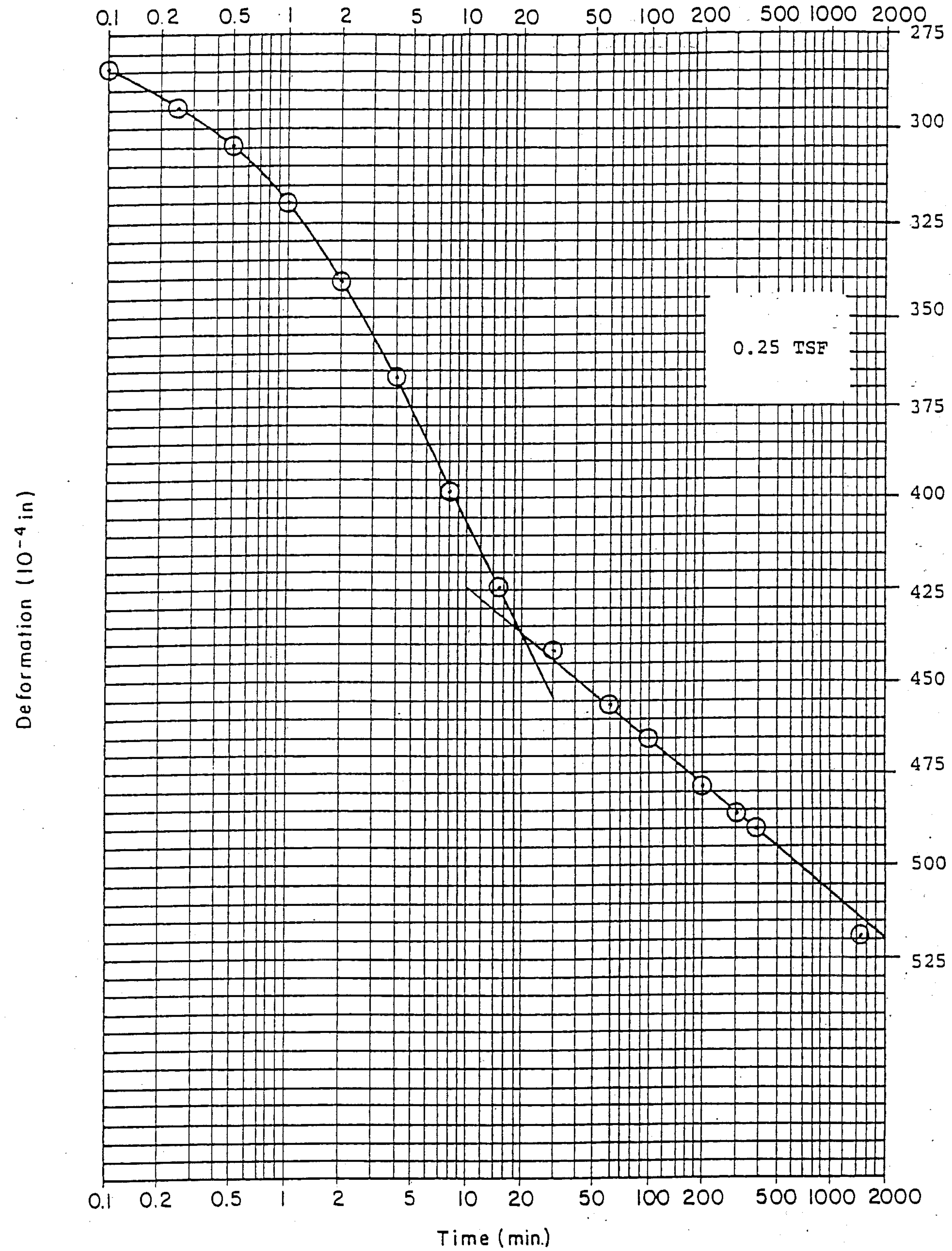
Sample No U-2 Depth 55 - 57' Date 1/29/90

GANNETT FLEMING GEOTECHNICAL LABORATORY



Type of Specimen – SHELBY TUBE				Before Test			After Test	
Dia	2.50 in	H_T	1.00 in	Water Content	w_o	86.3	w_f	61.8
Compression Index		C_c	0.80	Void Ratio	e_o	2.409	e_f	1.670
Classification BRN.-ELASTIC SILT				Saturation	S_o	96.6	S_f	100.0
w_l 59.4		I_p 19.1		Project DSWA, NSWF, PHASE III				
w_p 40.3		LI 2.4		Boring No GF-108			Sample No U-2	
Remarks WET DENSITY = 92.1 pcf				Depth 15' - 17'			Date 2/5/90	

GANNETT FLEMING GEOTECHNICAL LABORATORY
CONSOLIDATION TEST REPORT

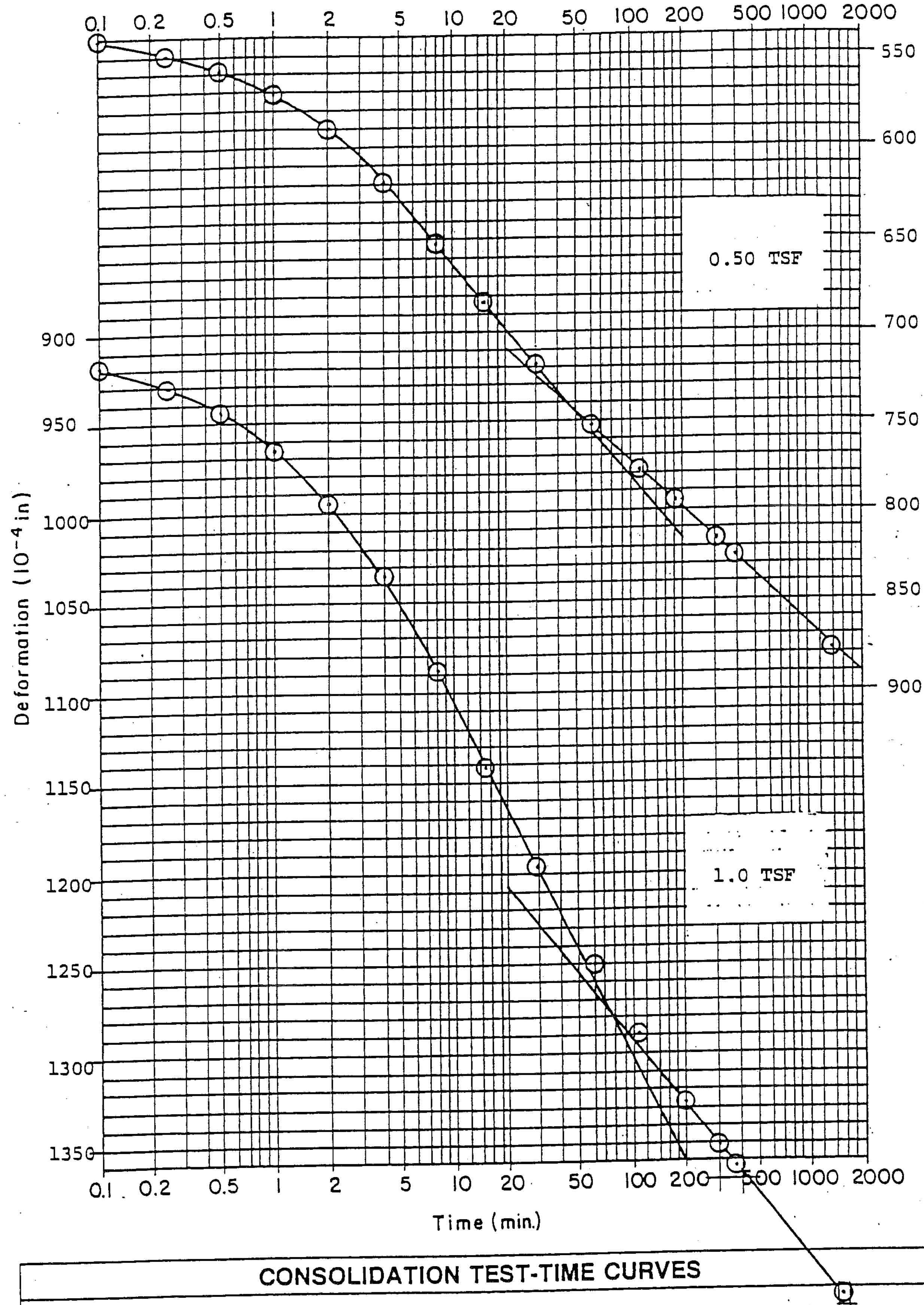


CONSOLIDATION TEST-TIME CURVES

Project DSWA, NSWF, PHASE III Boring No GF-108

Sample No U-2 Depth 15 - 17' Date 1/29/90

GANNETT FLEMING GEOTECHNICAL LABORATORY

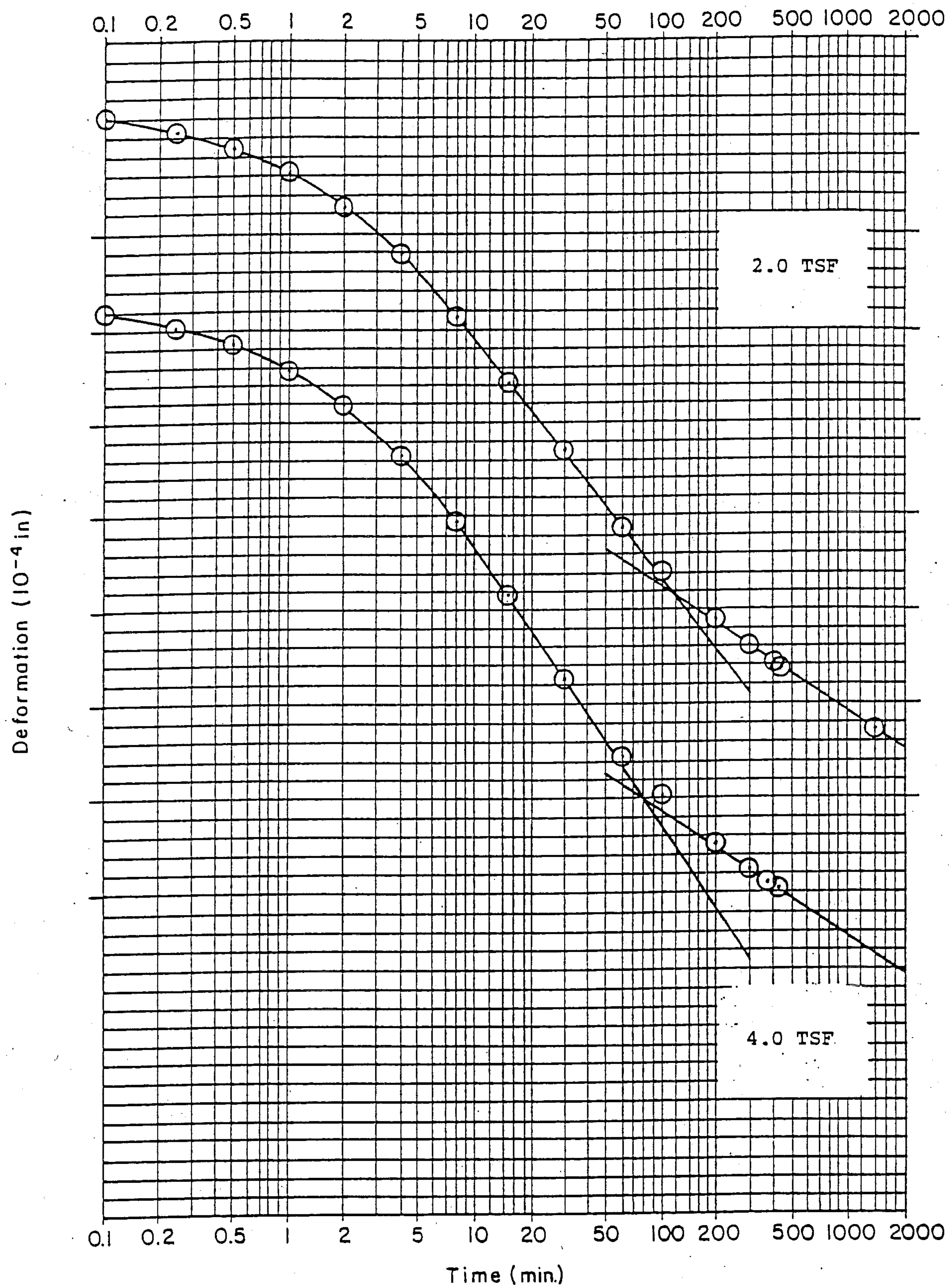


CONSOLIDATION TEST-TIME CURVES

Project DSWA, NSWF, PHASE III Boring No GF-108

Sample No U-2 Depth 15 - 17' Date 1/29/90

GANNETT FLEMING GEOTECHNICAL LABORATORY



CONSOLIDATION TEST-TIME CURVES

Project ______________________ Boring No ________

Sample No ____________ Depth ____________ Date ____________

GANNETT FLEMING GEOTECHNICAL LABORATORY

GANNETT FLEMING GEOTECHNICAL LABORATORY

CONSOLIDATION TEST REPORT

Coefficient of Consolidation, c_v

Project DSWA, NSWF, PHASE III Date 1/90

Boring	GF-105	GF-106	
Sample	U-2	U-1	
Depth	18.5-20.5'	8-10'	
Load (tsf)			
¼	0.057 in^2/min	0.025 in^2/min	
½	0.12 "	0.030 "	
1	0.013 "	0.0077 "	
2	0.011 "	0.0043 "	
4	0.069 "	0.0030 "	
8	0.0051 "	0.0025 "	
16			
32			

GANNETT FLEMING GEOTECHNICAL LABORATORY

CONSOLIDATION TEST REPORT

Coefficient of Consolidation, c_v

Project DSWA, NSWF, PHASE III Date 2/5/90

Boring	GF-102	GF-108	GF-107
Sample	U-2	U-2	U-2
Depth	35' - 37'	15' - 17'	55' - 57'
Load (tsf)			
¼	0.0037 in^2/min	0.017 in^2/min	0.0033 in^2/min
½	0.0016 "	0.0081 "	0.0026 "
1	0.0019 "	0.0052 "	0.0027 "
2	0.0025 "	0.0034 "	0.0029 "
4	0.0030 "	0.0033 "	0.0030 "
8			0.0033 "
16			0.0035 "
32			0.0034 "

PERMEABILITY TEST RESULTS

GANNETT FLEMING GEOTECHNICAL LABORATORY

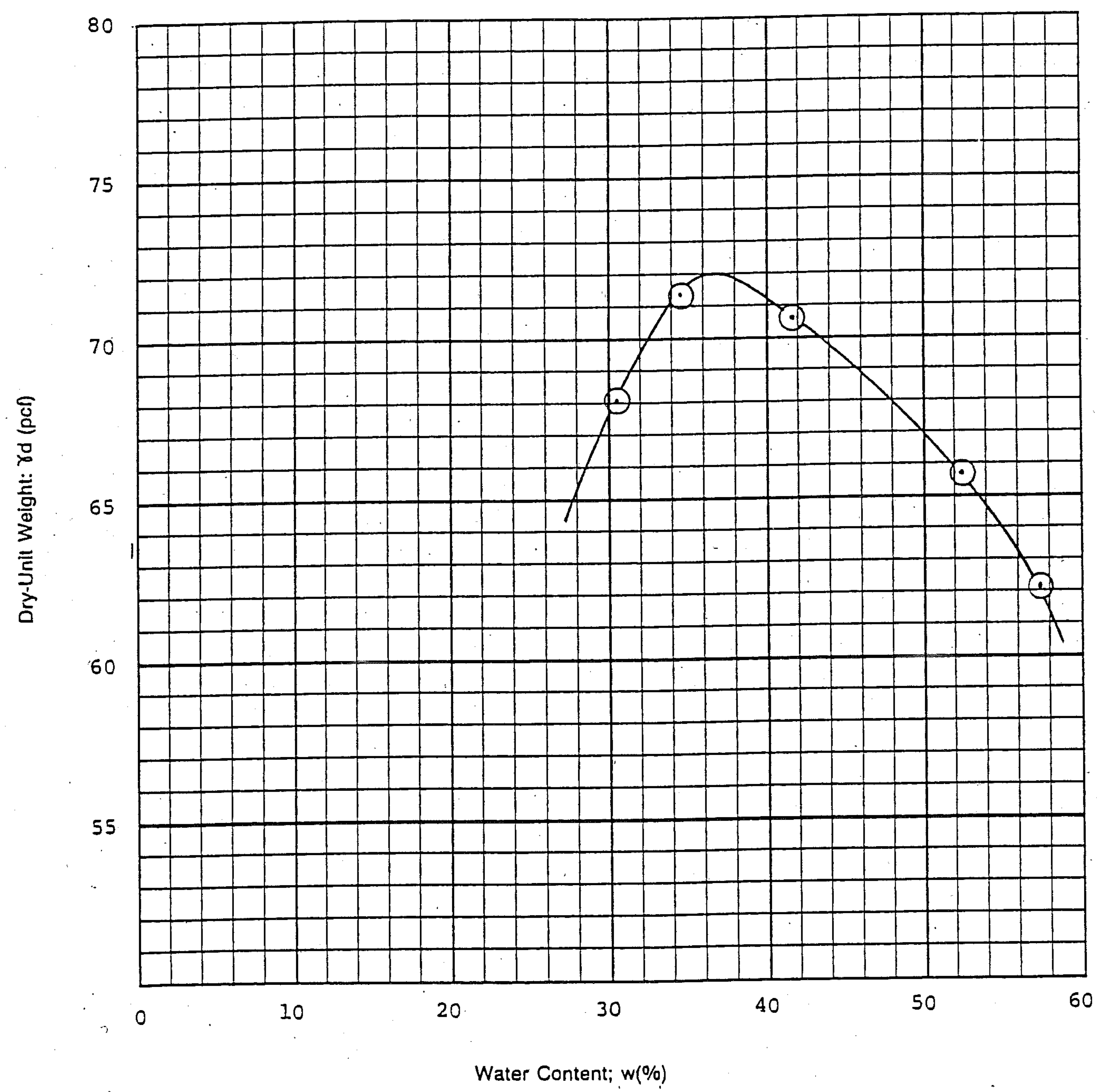
COMPACTION TEST

Project DSWA, NSWF, PHASE III Job No.

Boring No. GF-105 Sample No. Bag Depth 0-1'

Description of Soil MH

Test Method ASTM D 698, Method A Date of Testing 2/ 90



GANNETT FLEMING GEOTECHNICAL LABORATORY

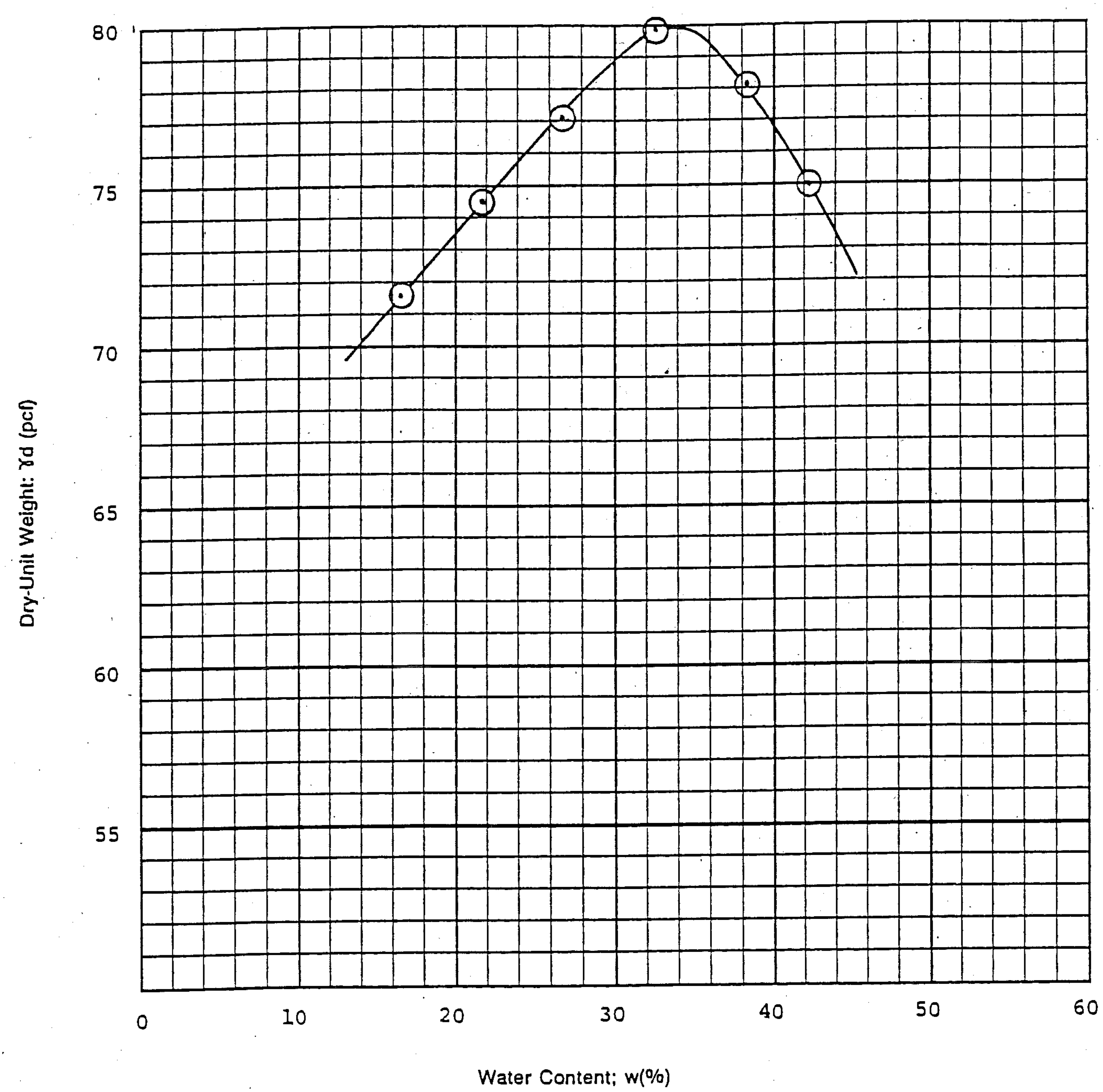
COMPACTION TEST

Project: DSWA, NSWF, PHASE III   Job No.

Boring No. GF-108   Sample No. Bag   Depth 0-1'

Description of Soil: MH

Test Method: ASTM D 698, Method A   Date of Testing 2/90



GANNETT FLEMING GEOTECHNICAL LABORATORY

ORGANIC CONTENT SUMMARY SHEET

PROJECT DELAWARE SOLID WASTE AUTHORITY, NSWF, PHASE III

BORING NO.	SAMPLE NO.	DEPTH (ft)	ORGANIC CONTENT (%)
GF-104	BAG	0'-1'	10.5
GF-105	BAG	0'-1'	10.6
GF-106	BAG	0'-1'	7.6
GF-108	BAG	0'-1'	7.5

UNDISTURBED SAMPLE LOGS

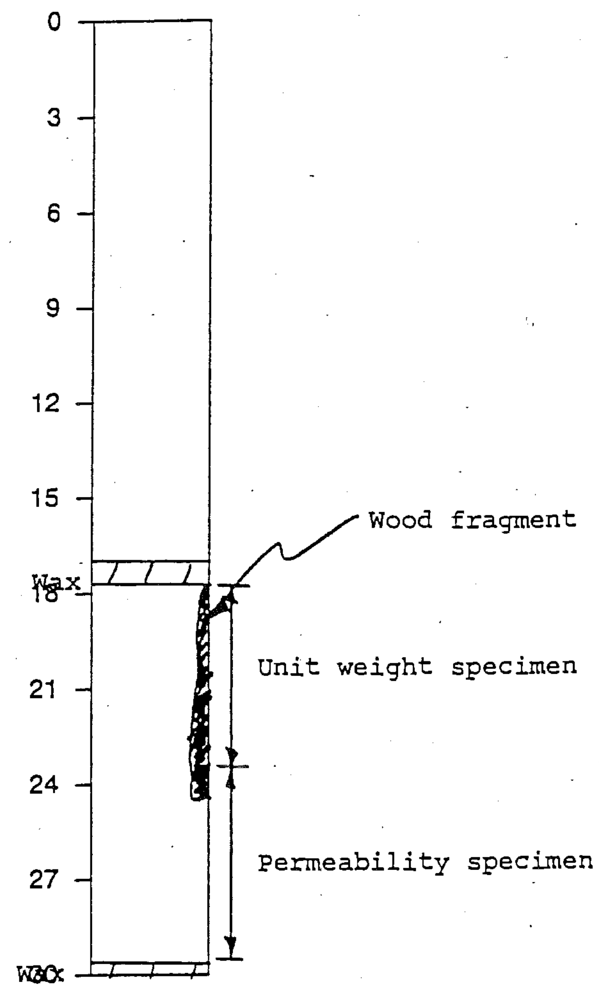
GANNETT FLEMING GEOTECHNICAL LABORATORY

UNDISTURBED SAMPLE LOG

Project DSWA,NSWF,PHASE III Job No ______

Boring No GF-104 Sample No U-1 Depth 6.5-8.5'

Sample Type Shelby Sampler Dia 3" Sampler Length 30"



Comments: Sample had a petroleum odor

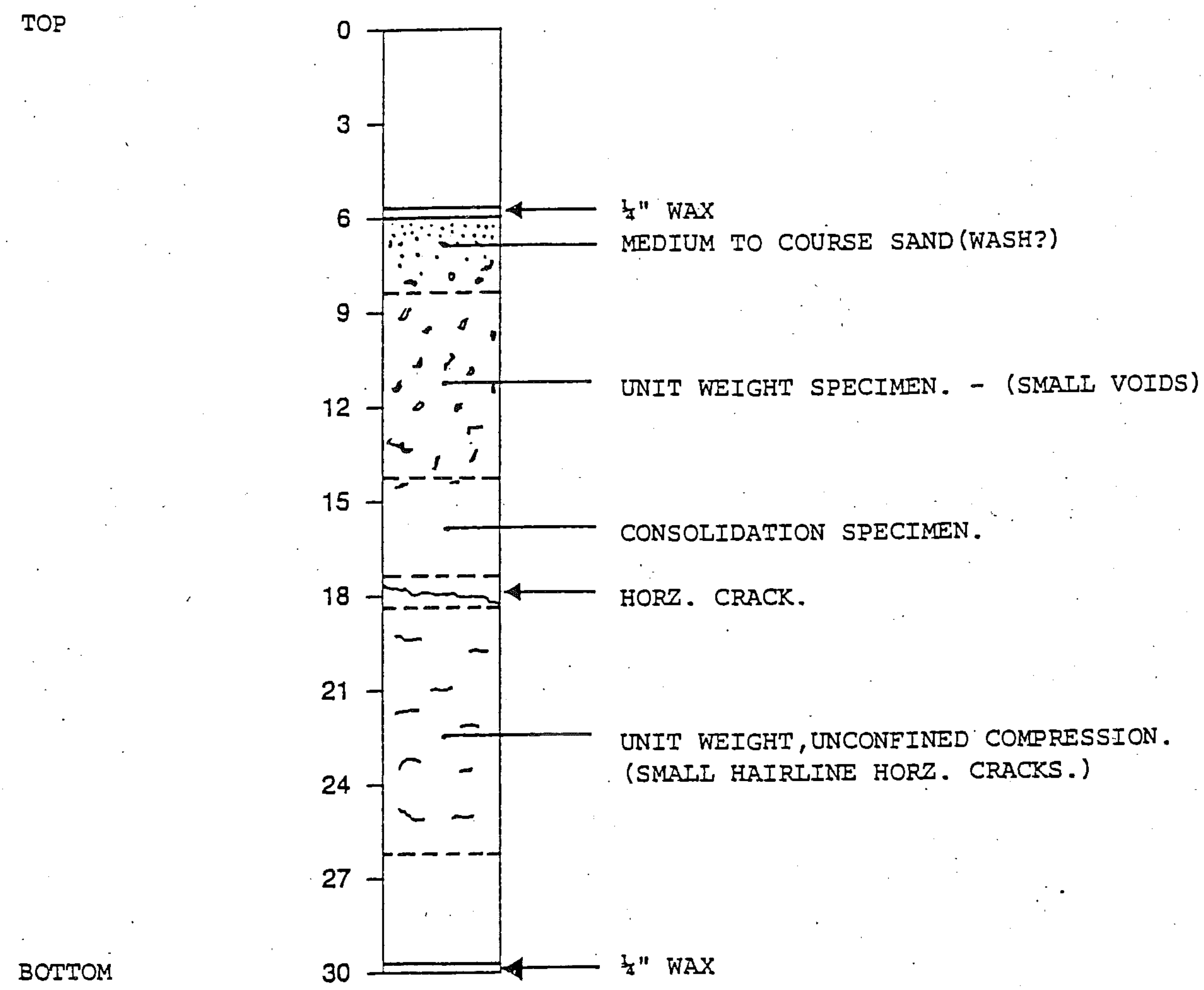
GANNETT FLEMING GEOTECHNICAL LABORATORY

UNDISTURBED SAMPLE LOG

Project DSWA NSWF-2 PHASE III Job No 26680.020

Boring No GF-104A Sample No U-2 Depth 16.5' - 18.5'

Sample Type SHELBY Sampler Dia 3" Sampler Length 30"



Comments: MAJORITY OF TUBE WAS MIXED AND USED FOR CLASSIFICATION.

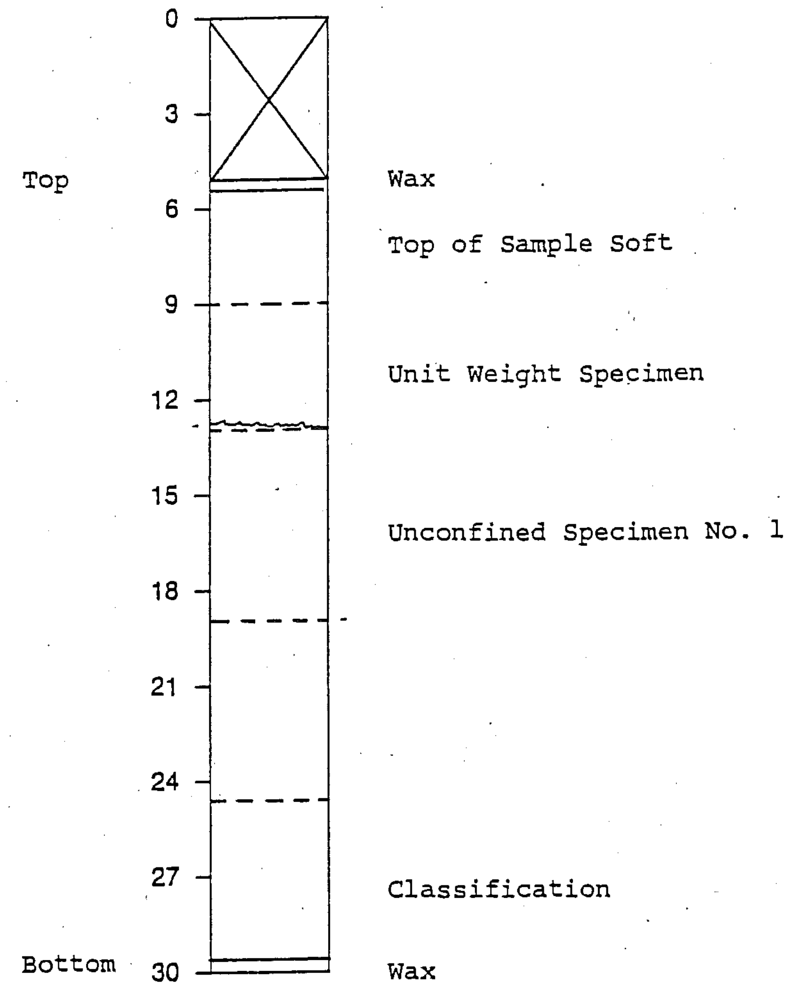
GANNETT FLEMING GEOTECHNICAL LABORATORY

UNDISTURBED SAMPLE LOG

Project: DSWA NSWF - 2 PHASE III — Job No: 26680.020

Boring No: GF - 104A — Sample No: U - 4 — Depth: 35.0'-37.0'

Sample Type: SHELBY — Sampler Dia: 2.0" — Sampler Length: 30.0"



Comments: Gray Elastic Silt , Medium Soft

Unit Weight = 85.8 pcf (Wet).

Wn @ 94.2 = 44.2 pcf (Dry).

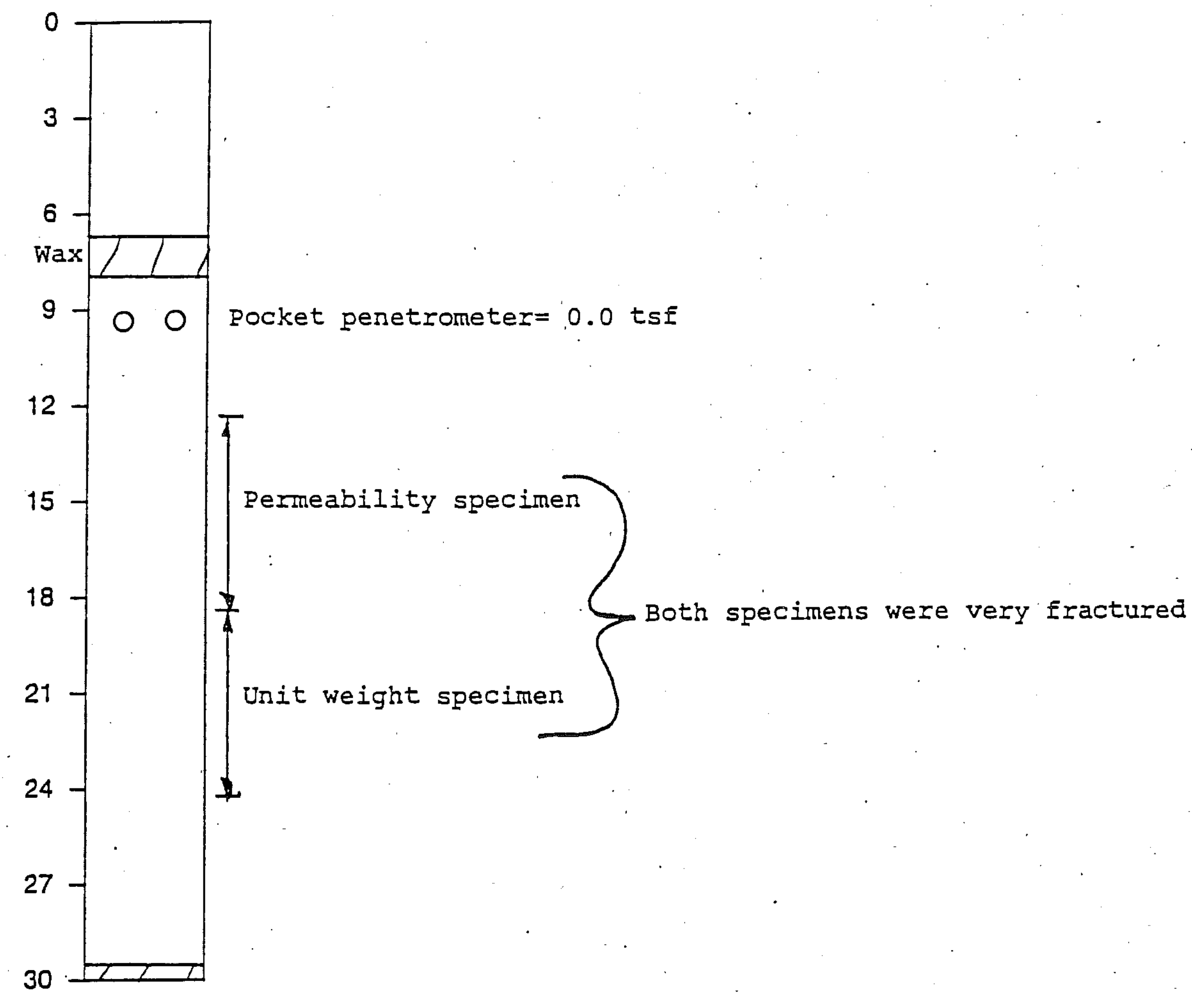
GANNETT FLEMING GEOTECHNICAL LABORATORY

UNDISTURBED SAMPLE LOG

Project DSWA,NSWF,PHASE III Job No

Boring No GF-105 Sample No U-1 Depth 5-7'

Sample Type Shelby Sampler Dia 3" Sampler Length 30"



Comments: Sample had a petroleum odor

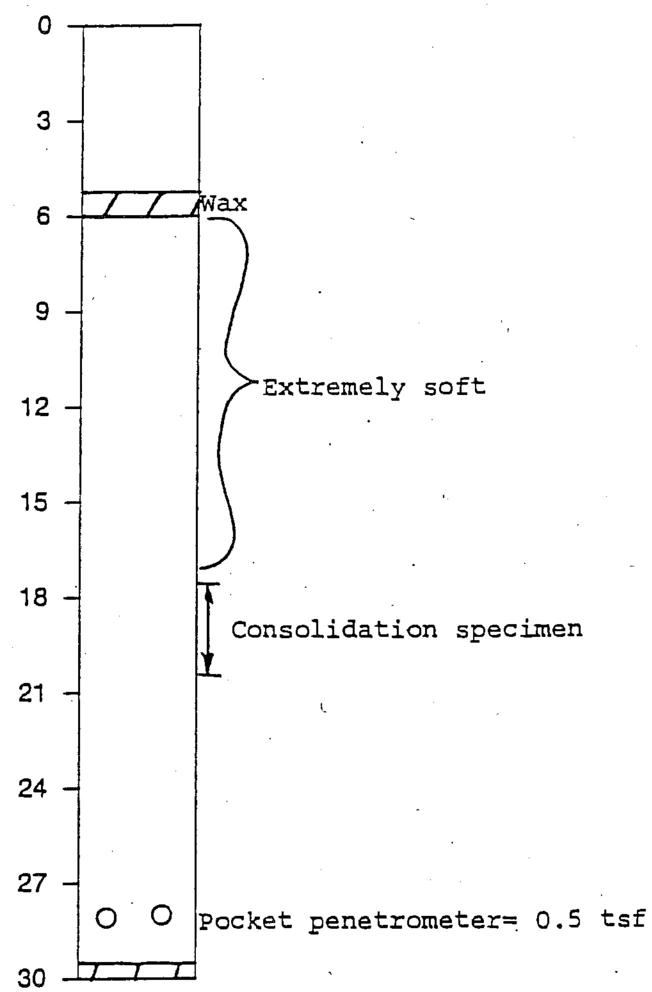
GANNETT FLEMING GEOTECHNICAL LABORATORY

UNDISTURBED SAMPLE LOG

Project DSWA,NSWF,PHASE III Job No

Boring No GF-105 Sample No U-2 Depth 18.5-20.5'

Sample Type Shelby Sampler Dia 3" Sampler Length 30"



Comments:

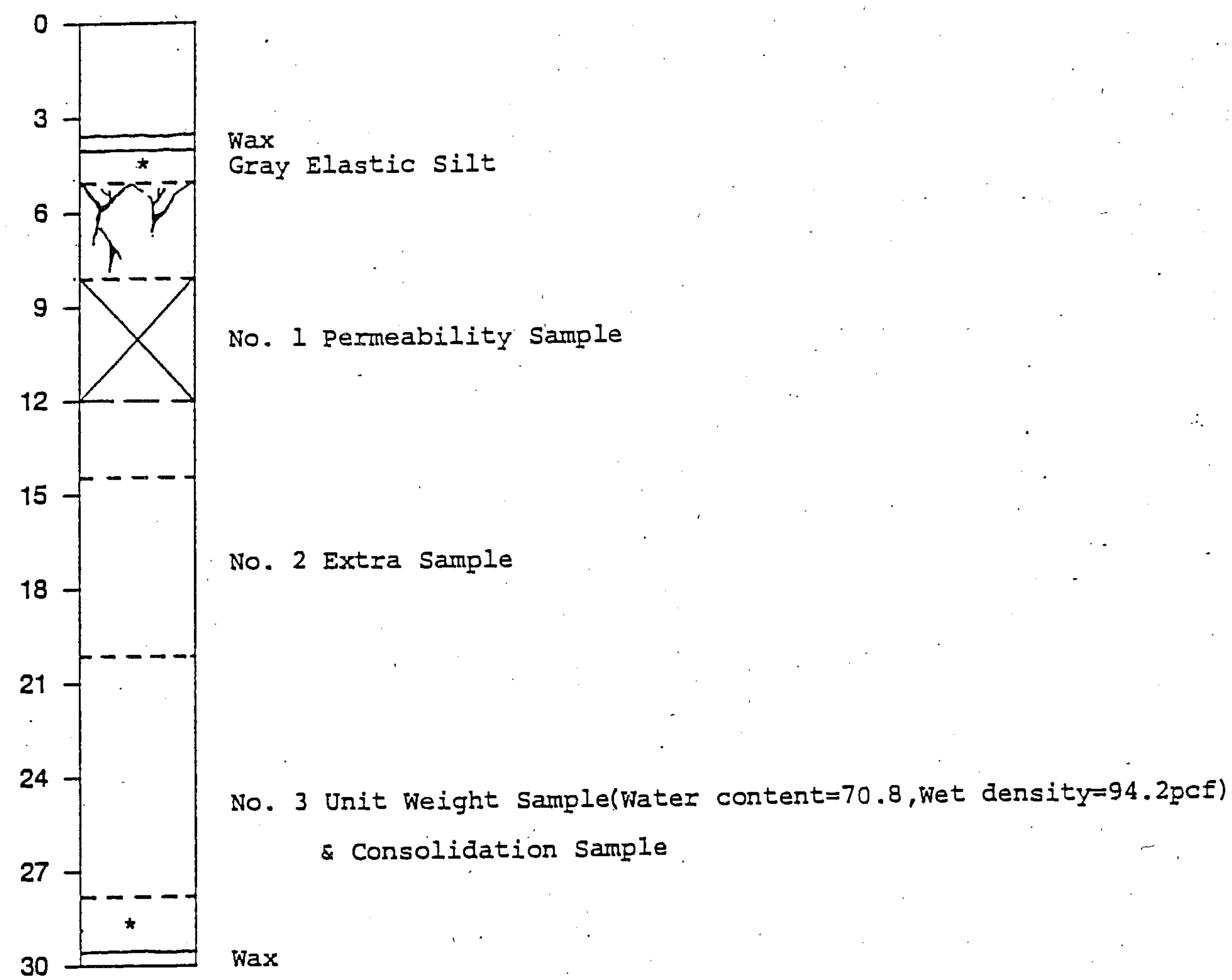
GANNETT FLEMING GEOTECHNICAL LABORATORY

UNDISTURBED SAMPLE LOG

Project DSWA , NSWF PHASE III Job No 26680.020

Boring No GF - 106 Sample No U-1 Depth 8.0'-10.0'

Sample Type SHELBY Sampler Dia 3-INCH Sampler Length 30-INCH



Comments: * Took About 2-Inches Of Soil Out Of Top And Bottom Of Tube For Initial Classification. Sample had a petroleum odor.

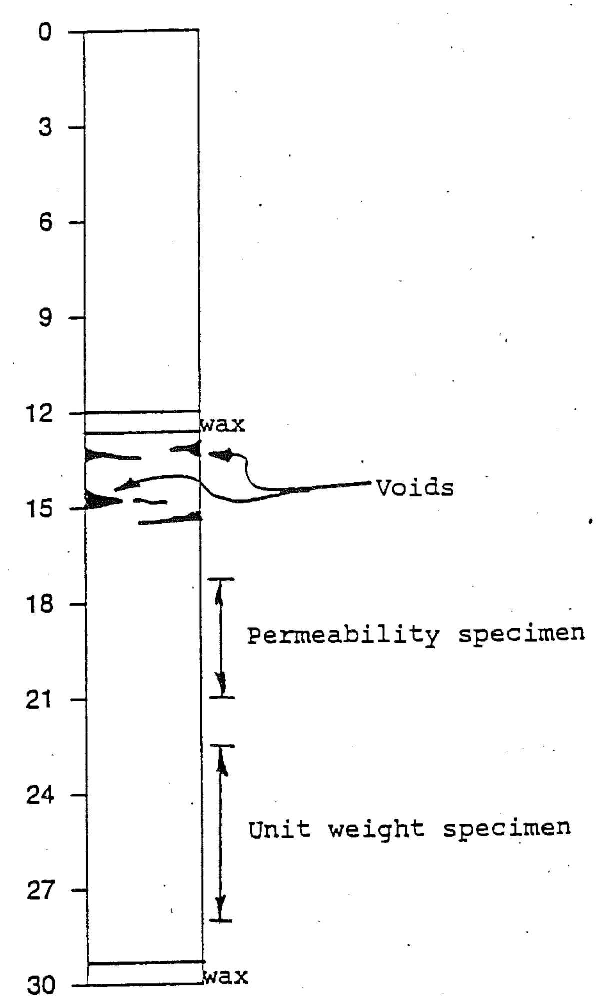
GANNETT FLEMING GEOTECHNICAL LABORATORY

UNDISTURBED SAMPLE LOG

Project DSWA,NSWF,PHASE III Job No ______

Boring No GF-107 Sample No U-1 Depth 20-22'

Sample Type Shelby Sampler Dia 3" Sampler Length 30"



Comments: Voids at the top were probably formed when the tube wobbled as it was pushed.

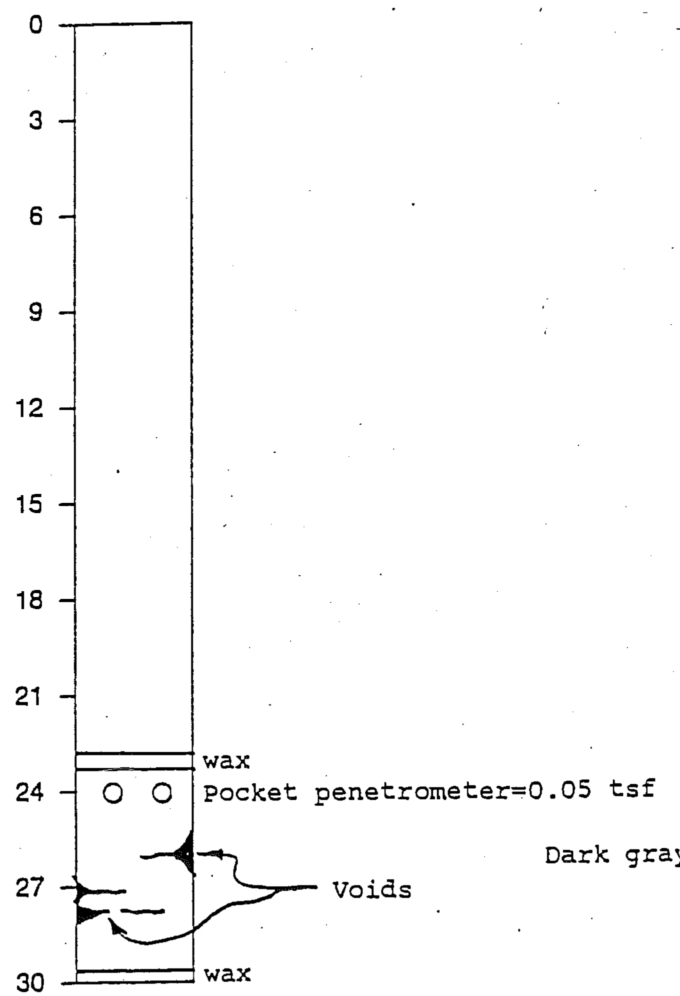
GANNETT FLEMING GEOTECHNICAL LABORATORY

UNDISTURBED SAMPLE LOG

Project DSWA,NSWF,PHASE III Job No

Boring No GF-108 Sample No U-1 Depth 8-10'

Sample Type Shelby Sampler Dia 3" Sampler Length 30"



Dark gray clayey silt

Comments: Sample was very disturbed, voids were probably formed when the tube wobbled as it was pushed.

Organic odor

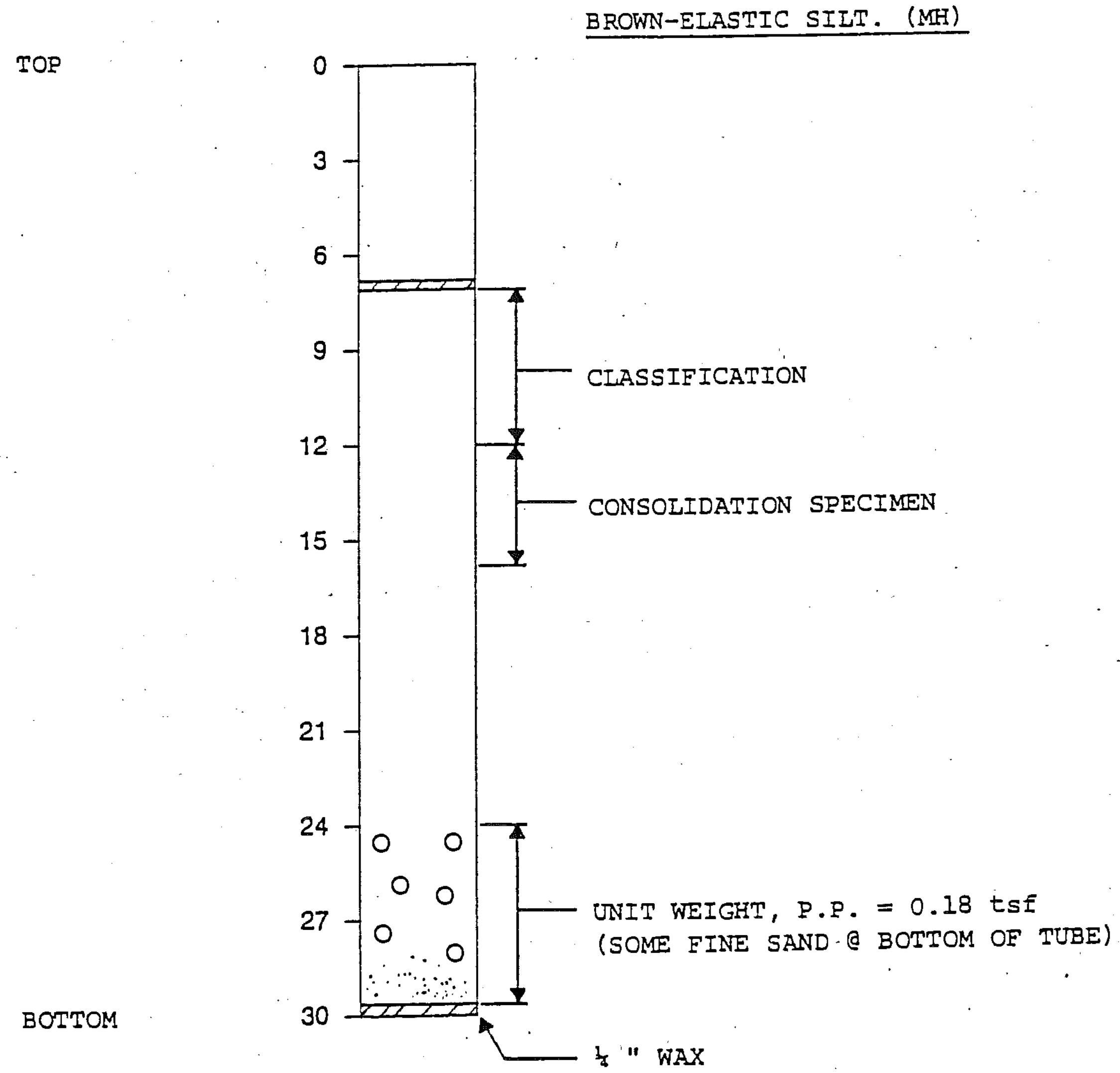
GANNETT FLEMING GEOTECHNICAL LABORATORY

UNDISTURBED SAMPLE LOG

Project DSWA, NSWF, PHASE III Job No 26680.020

Boring No GF-108 Sample No U-2 Depth 15'-17'

Sample Type SHELBY TUBE Sampler Dia 3" Sampler Length 30"



Comments: ______

F. T. KITLINSKI & ASSOCIATES

RECORD OF UNDISTURBED SAMPLE

Project 90-01-5411 Date January 1990 Recorded By D. Matafka & M. Updyke

Project Location DSWA, NSWF-2, Phase III, Wilmington, Delaware

Boring No. GF-108 Sample No. U-3 Depth 28.5' to 30.5'

Type Sample Shelby Size 1.75" diameter

Condition Recovery = 17.0"

From	To	Description
28.50'	28.92'	Extremely soft dark grey clayey silt
28.92'	29.34'	Very soft dark grey clayey silt
29.34'	29.92'	Soft dark grey clayey silt

Remarks Unconfined Compression Sample - 29.40' to 29.69'

Wet Unit Weight - 103.8 p.c.f.

Moisture Content - 86.7%

Dry Unit Weight - 55.6 p.c.f.

F. T. KITLINSKI & ASSOCIATES

RECORD OF UNDISTURBED SAMPLE

Project 90-01-5411 Date January 1990 Recorded By D. Matafka & M. Updyke

Project Location DSWA, NSWF-2, Phase III, Wilmington, Delaware

Boring No. GF-108 Sample No. U-4 Depth 49.0' to 51.0'

Type Sample Shelby Size 1.75" diameter

Condition Recovery = 24.0"

From	To	Description
49.00'	49.50'	Extremely soft dark grey clayey silt
49.50'	50.39'	Very soft dark grey clayey silt
50.39'	51.00'	Soft dark grey clayey silt

Remarks Unconfined Compression Sample - 50.60' to 50.89'

Wet Unit Weight - 104.1 p.c.f.

Moisture Content - 67.9%

Dry Unit Weight - 62.0 p.c.f.

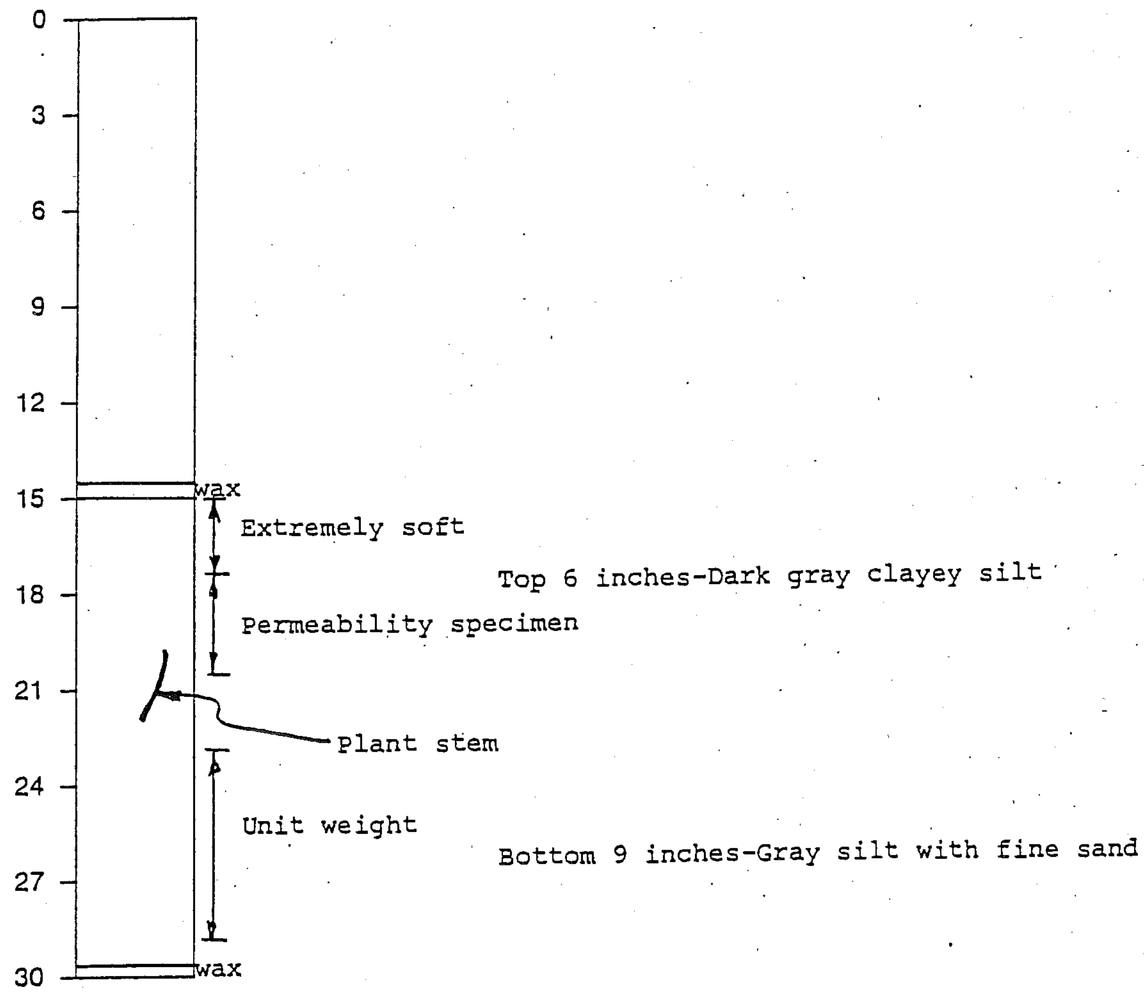
GANNETT FLEMING GEOTECHNICAL LABORATORY

UNDISTURBED SAMPLE LOG

Project DSWA,NSWF,PHASE III Job No ____________

Boring No GF-109 Sample No U-1 Depth 14-16'

Sample Type Shelby Sampler Dia 3" Sampler Length 30"



Comments: ____________

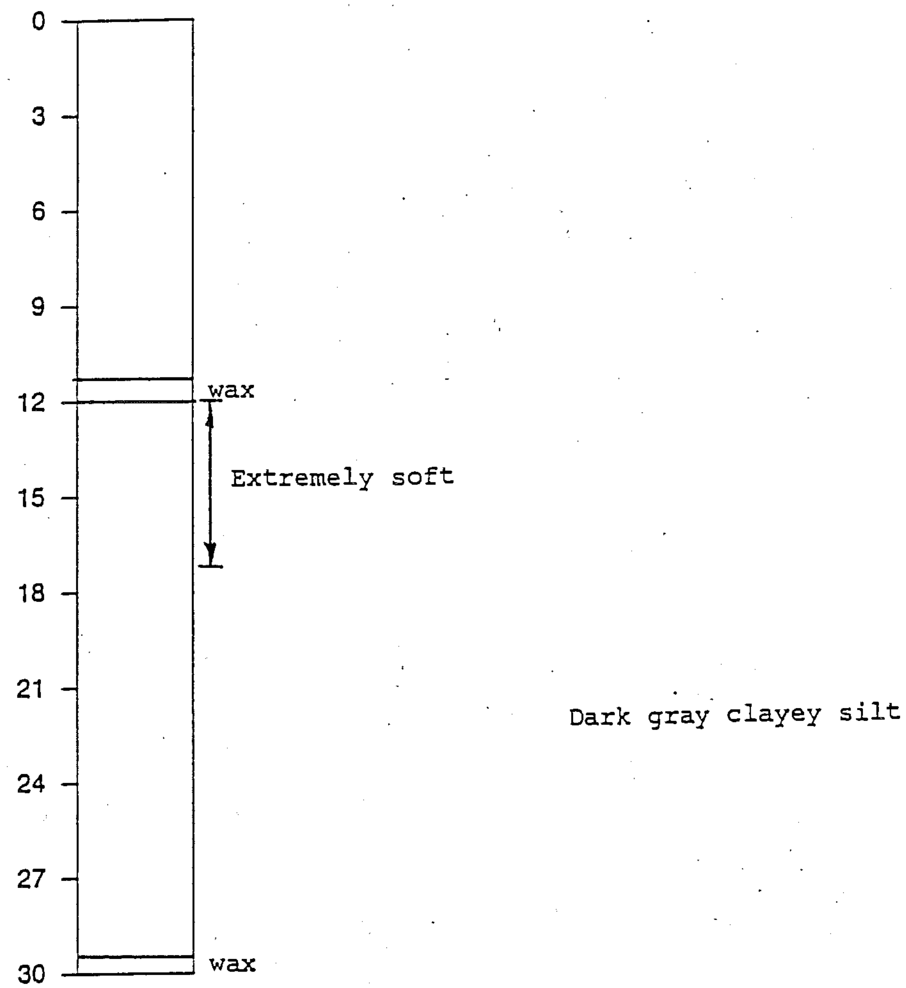
GANNETT FLEMING GEOTECHNICAL LABORATORY

UNDISTURBED SAMPLE LOG

Project DSWA,NSWF,PHASE III Job No ____

Boring No GF-109 Sample No U-2 Depth 17-19'

Sample Type Shelby Sampler Dia 3" Sampler Length 30"



Comments: ____

F. T. KITLINSKI & ASSOCIATES

RECORD OF UNDISTURBED SAMPLE

Project 90-01-5411 Date January 1990 Recorded By D. Matafka & M. Updyke

Project Location DSWA, NSWF-2, Phase III, Wilmington, Delaware

Boring No. GF-109 Sample No. U-3 Depth 37.0' to 39.0'

Type Sample Shelby Size 1.75" diameter

Condition Recovery = 19.0"

From	To	Description
37.00'	37.50'	Soft dark grey clayey silt
37.50'	38.00'	Slightly soft dark grey clayey silt
38.0'	38.59'	Firm dark grey clayey silt

Remarks Unconfined Compression Sample - 38.10' to 38.39'

Wet Unit Weight - 105.3 p.c.f.

Moisture Content - 79.5%

Dry Unit Weight - 58.7 p.c.f.

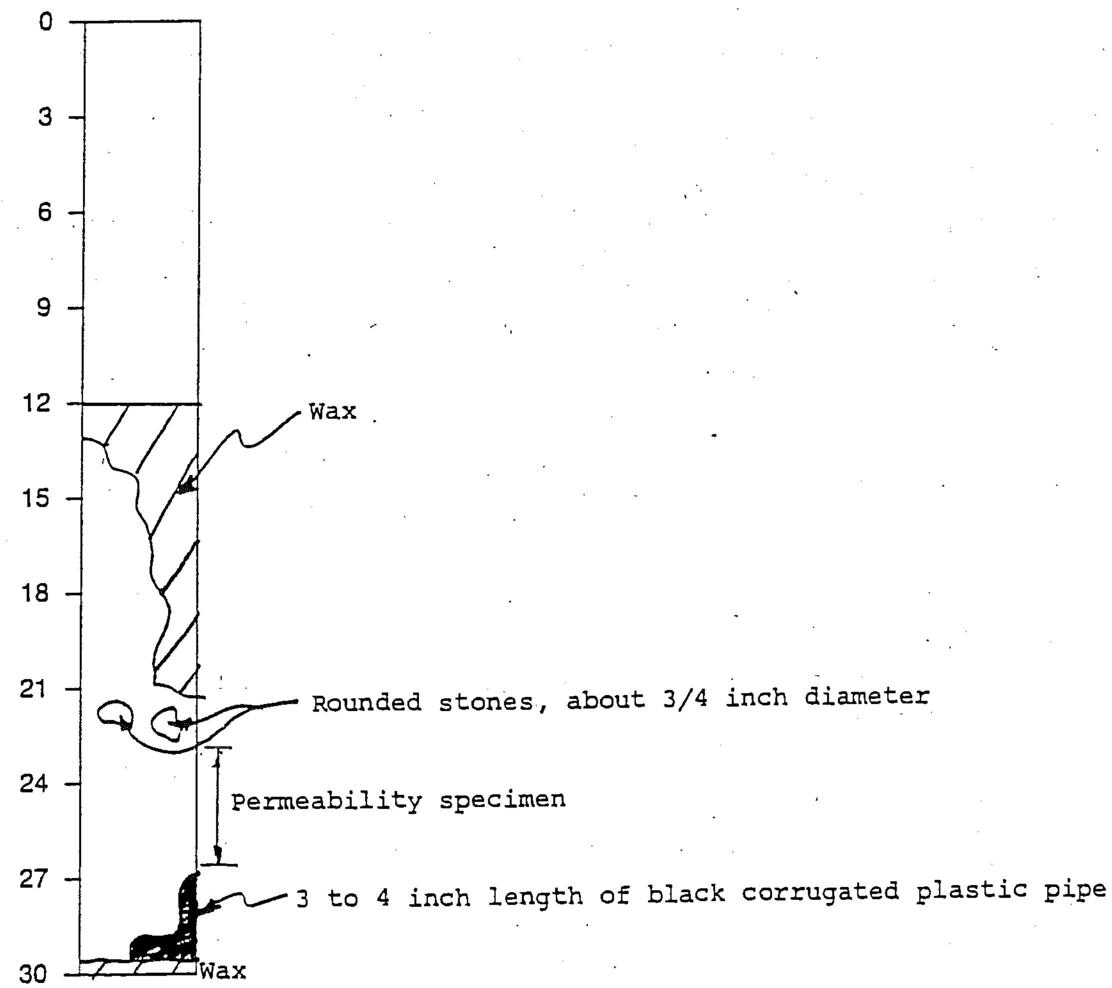
GANNETT FLEMING GEOTECHNICAL LABORATORY

UNDISTURBED SAMPLE LOG

Project DSWA,NSWF,PHASE III Job No ____

Boring No GF-110 Sample No U-1 Depth 10-12'

Sample Type Shelby Sampler Dia 3" Sampler Length 30"



Comments: Sample had an organic odor

GANNETT FLEMING, INC.
ENVIRONMENTAL LABORATORY
209 SENATE AVENUE
CAMP HILL, PA 17011
(717)763-7211
PA DER Certification No. 22-133

Delaware Solid Waste Landfill
c/o Gannett Fleming, Inc.
Room 300, N
Attn: Trent Dreese
GF Job Number: 26680.024

Client Number: 1145
Project Number: 9063
Sample Number: 25310
Date Received: 04/11/90
Time Received: 11:00
Discard Date: 04/30/90

LABORATORY ANALYSIS REPORT
April 16, 1990

Sample Identification: A-1
Date Collected: 04/10/90 Time: 11:00 Collected By: SP

ANALYSIS	RESULTS	UNITS	
pH	7.25	pH Units	

Gannett Fleming, Inc.

David W. Lane, Laboratory Manager

GANNETT FLEMING, INC.
ENVIRONMENTAL LABORATORY
209 SENATE AVENUE
CAMP HILL, PA 17011
(717)763-7211
PA DER Certification No. 22-133

Delaware Solid Waste Landfill
c/o Gannett Fleming, Inc.
Room 300, N
Attn: Trent Dreese
GF Job Number: 26680.024

Client Number: 1145
Project Number: 9063
Sample Number: 25311
Date Received: 04/11/90
Time Received: 11:00
Discard Date: 04/30/90

L A B O R A T O R Y A N A L Y S I S R E P O R T
April 16, 1990

Sample Identification: **A-2**
Date Collected: 04/10/90 Time: 14:00 Collected By: SP

ANALYSIS	RESULTS	UNITS	
pH	7.10	pH Units	

Gannett Fleming, Inc.

David W. Lane, Laboratory Manager

GANNETT FLEMING, INC.
ENVIRONMENTAL LABORATORY
209 SENATE AVENUE
CAMP HILL, PA 17011
(717)763-7211
PA DER Certification No. 22-133

Delaware Solid Waste Landfill
c/o Gannett Fleming, Inc.
Room 300, N
Attn: Trent Dreese
GF Job Number: 26680.024

Client Number: 1145
Project Number: 9063
Sample Number: 25312
Date Received: 04/11/90
Time Received: 11:00
Discard Date: 04/30/90

LABORATORY ANALYSIS REPORT

April 16, 1990

Sample Identification: C-1
Date Collected: 04/10/90 Time: 14:00 Collected By: SP

ANALYSIS	RESULTS	UNITS	
pH	7.16	pH Units	

Gannett Fleming, Inc.

David W. Lane, Laboratory Manager

GANNETT FLEMING, INC.
ENVIRONMENTAL LABORATORY
209 SENATE AVENUE
CAMP HILL, PA 17011
(717)763-7211
PA DER Certification No. 22-133

Delaware Solid Waste Landfill c/o Gannett Fleming, Inc. Room 300, N Attn: Trent Dreese GF Job Number: 26680.024	Client Number: 1145 Project Number: 9063 Sample Number: 25313 Date Received: 04/11/90 Time Received: 11:00 Discard Date: 04/30/90

LABORATORY ANALYSIS REPORT

April 16, 1990

Sample Identification: C-2
Date Collected: 04/10/90 Time: 14:00 Collected By: SP

ANALYSIS	RESULTS	UNITS	
pH	7.23	pH Units	

Gannett Fleming, Inc.

David W. Lane, Laboratory Manager

GEOCHEMICAL TESTING
ENERGY AND ENVIRONMENTAL ANALYSIS

R.D. 2, BOX 124
Somerset, Pennsylvania 15501
Phone: (814) 445-6666 or 443-1671

REPORT OF ANALYSIS

Client: Gannett Fleming, Inc.

Lab # 90-E13917

Sampled by: Client

Sampling date: 04/02/1990

Analysis completed: 04/24/1990

Description : GF-105 S-6 13.5-15.0

Analyte	Result	Units	Test Method
Calcium	13.6	meq/100g	
Cation Exch Capacity	39.9	meq./100g	
Lime Requirement	12600	lbs/ac CaCO3	
Magnesium	10.5	meq/100g	
Potassium	0.7	meq/100g	
SMP Buffer pH	5.7	su	
pH, Solid	5.1	su	EPA 9045

Forrest E. Walker

Forrest E. Walker,
Director of Technical Services

GEOCHEMICAL TESTING
ENERGY AND ENVIRONMENTAL ANALYSIS

R.D. 2, BOX 124
Somerset, Pennsylvania 15501
Phone: (814) 445-6666 or 443-1671

REPORT OF ANALYSIS

Client: Gannett Fleming, Inc.   Lab # 90-E13918

Sampled by: Client

Sampling date: 04/02/1990

Analysis completed: 04/24/1990

Description : GF-105 S-9 28.0-29.5

Analyte	Result	Units	Test Method
Calcium	8.0	meq/100g	
'ation Exch Capacity	26.4	meq./100g	
'ime Requirement	9600	lbs/ac CaCO3	
agnesium	6.4	meq/100g	
Potassium	0.6	meq/100g	
SMP Buffer pH	6.0	su	
pH, Solid	5.5	su	EPA 9045

Forrest E. Walker

Forrest E. Walker,
Director of Technical Services

GEOCHEMICAL TESTING
ENERGY AND ENVIRONMENTAL ANALYSIS

R.D. 2, BOX 124
Somerset, Pennsylvania 15501
Phone: (814) 445-6666 or 443-1671

REPORT OF ANALYSIS

Client: Gannett Fleming, Inc.   Lab # 90-E13919

Sampled by: Client

Sampling date: 04/02/1990

Analysis completed: 04/24/1990

Description : GF-108 S-2 4.0-6.0

Analyte	Result	Units	Test Method
Calcium	11.0	meq/100g	
Cation Exch Capacity	25.9	meq./100g	
Lime Requirement	4400	lbs/ac CaCO3	
Magnesium	9.0	meq/100g	
Potassium	0.6	meq/100g	
SMP Buffer pH	6.5	su	
pH, Solid	5.9	su	EPA 9045

Forrest E. Walker

Forrest E. Walker,
Director of Technical Services

GEOCHEMICAL TESTING
ENERGY AND ENVIRONMENTAL ANALYSIS

R.D. 2, BOX 124
Somerset, Pennsylvania 15501
Phone: (814) 445-6666 or 443-1671

REPORT OF ANALYSIS

Client: Gannett Fleming, Inc.　　Lab # 90-E13920

Sampled by: Client

Sampling date: 04/02/1990

Analysis completed: 04/24/1990

Description : GF-112 S-4 6.0-8.0

Analyte	Result	Units	Test Method
Calcium	9.9	meq/100g	
ation Exch Capacity	27.0	meq./100g	
ime Requirement	7500	lbs/ac CaCO3	
agnesium	7.4	meq/100g	
Potassium	0.7	meq/100g	
SMP Buffer pH	6.2	su	
pH, Solid	5.7	su	EPA 9045

Forrest E. Walker

Forrest E. Walker,
Director of Technical Services

GEOCHEMICAL TESTING

ENERGY AND ENVIRONMENTAL ANALYSIS

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Somerset, Pennsylvania 15501
Phone: (814) 445-6666 or 443-1671

REPORT OF ANALYSIS

Client: Gannett Fleming, Inc. Lab # 90-E13921

Sampled by: Client

Sampling date: 04/02/1990

Analysis completed: 04/24/1990

Description : GF-112 S-12 43.5-45.0

Analyte	Result	Units	Test Method
Calcium	9.6	meq/100g	
Cation Exch Capacity	23.2	meq./100g	
Lime Requirement	5400	lbs/ac CaCO3	
Magnesium	6.5	meq/100g	
Potassium	0.6	meq/100g	
SMP Buffer pH	6.4	su	
pH, Solid	6.0	su	EPA 9045

Forrest E. Walker

Forrest E. Walker,
Director of Technical Services

GANNETT FLEMING GEOTECHNICAL LABORATORY

COMPATIBILITY SUMMARY SHEET

PROJECT DSWA,NSWF,PHASE III DATE 4/ 90

BORING GF-108 SAMPLE Bag DEPTH 0-1'

SPECIMEN DATA

Unified Classification MH

Liquid Limit 55.6

Plastic Limit 38.3

Water Content 40.3 %

Dry Density 80.7 pcf

Void Ratio 1.088

Saturation 100 %

Diameter 2.8 inch/ Height 2.2 inch

TEST CONDITIONS

Consolidation Pressure 1.50 ksf

Cell Pressure 78.8 psi

Back Pressure

At the bottom of specimen 78.3 psi

At the top of specimen 68.3 psi

Hydraulic Gradient 128

TEST RESULTS

Initial Permeability 1.5×10^{-8} cm/sec

Final Permeability 1.7×10^{-8} cm/sec

Notes: (1) The initial permeability is reported using distilled water as the permeant.

(2) The final permeability is reported after exchanging two pore volumes with leachate.

5. Roy F. Weston, Inc., "Northern Solid Waste Management Center – Cherry Island Landfill, Phase IV Disposal Area, Hydrogeologic, Geotechnical and Landfill Capping Report," prepared for the Delaware Solid Waste Authority, August 1992.

FILE COPY Set # 08

Delaware Solid Waste Authority
Dover, Delaware

Northern Solid Waste Management Center-
Cherry Island Landfill
Phase IV Disposal Area

Hydrogeologic, Geotechnical and Landfill Capping Report

Final Report

August 1992

C-696

WESTON
MANAGERS DESIGNERS/CONSULTANTS

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DELAWARE SOLID WASTE AUTHORITY
NORTHERN SOLID WASTE MANAGEMENT
CENTER - CHERRY ISLAND LANDFILL
PHASE IV DISPOSAL AREA

HYDROGEOLOGIC, GEOTECHNICAL, AND
LANDFILL CAPPING REPORT

August, 1992

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SECTION 1

1.0 INTRODUCTION

This report presents the results of the Hydrogeologic and Geotechnical Investigation conducted by Roy F. Weston, Inc. (WESTON) for the Delaware Solid Waste Authority's (DSWA) Cherry Island Landfill Disposal Area IV (Phase IV). The results of the Hydrogeologic and Geotechnical Investigation are presented in Sections 2 and 3, respectively. This report also includes (in Section 4) an evaluation of final landfill capping systems and provides a recommended final cap section.

The purpose of the Hydrogeologic and Geotechnical Investigation was to obtain additional information/data and perform the analyses/evaluations required for the Phase IV design and permit. A significant existing data base for the Cherry Island landfill exists from the investigation of previous phases (Phase I, IA, II, and III). The scope of the Hydrogeologic and Geotechnical Field Investigation was outlined in the "Hydrogeologic Work and Groundwater Sampling Plans" (Work Plan) dated March 1992. This Work Plan was submitted to Delaware Department of Natural Resources and Environmental Control (DNREC). A meeting was held at DNREC's offices on 26 March 1992 to discuss the Work Plan. DNREC requested that the groundwater analysis for Appendix I metals include both filtered and unfiltered samples (the Work Plan specified only unfiltered samples). Both filtered and unfiltered samples were taken and the results are presented herein. The other request by DNREC was to provide a means of monitoring pore water pressures in the dredge spoils/recent deposits during landfill operations to confirm the predicted increase in pore water pressures and upward pressure gradient during the design life of the landfill. WESTON is further investigating possible monitoring instrumentation and will make recommendations to DSWA for inclusion in the construction specifications under operations monitoring equipment to be installed during Phase IV construction.

Field work began on 6 April 1992 and was completed 4 May 1992.

The Hydrogeologic and Geotechnical Investigation for Phase IV was performed in accordance with the Work Plan except for a few modifications. These modifications include:

- Deletion of piezometer P-14 and the termination at 60 ft. of test boring TB-10, located along the berm between Phases IV and II, due to prolonged methane venting from this borehole. Methane concentrations at the borehole were near or at 100% of the Lower Explosive Limit (LEL) even after venting for several days. This modification did not compromise the design effort because sufficient previous and new subsurface data is available for the design analysis.

- Relocation of TB-7 (P-17) to the existing Phase IV perimeter berm adjacent to the Delaware River. This boring/piezometer location was relocated based on subsurface conditions encountered during this investigation which indicated greater horizontal variation in subsurface conditions along the berm than within the proposed footprint.

- Deletion of TB-6 since the subsurface conditions within the dredge spoils/recent deposits were found not to vary greatly horizontally. In addition, a previous boring was located in the vicinity of TB-6 and provided subsurface data for this area.

These modifications were discussed with DNREC by telephone and in a letter dated 5 May 1992. These modifications were approved by DNREC through correspondence dated 26 May 1992.

The proposed Phase IV disposal area of the Northern Solid Waste Management Center Cherry Island Landfill is located south of Phase III as shown on Figure 1-1. Phase IV is bounded on the north by the proposed Phase III disposal area; by the perimeter dike along the Delaware River on the east; by the Phase II disposal area on the west; and, by the proposed Phase IV/V separation berm on the south (that begins at the southernmost point of Phase II). The proposed separation berm along the southern perimeter of Phase IV will separate it from proposed future expansions (Phase V). The Phase IV disposal area will include the use of the dredge spoils as a natural soil liner and a constructed leachate collection system similar to that planned for Phase III.

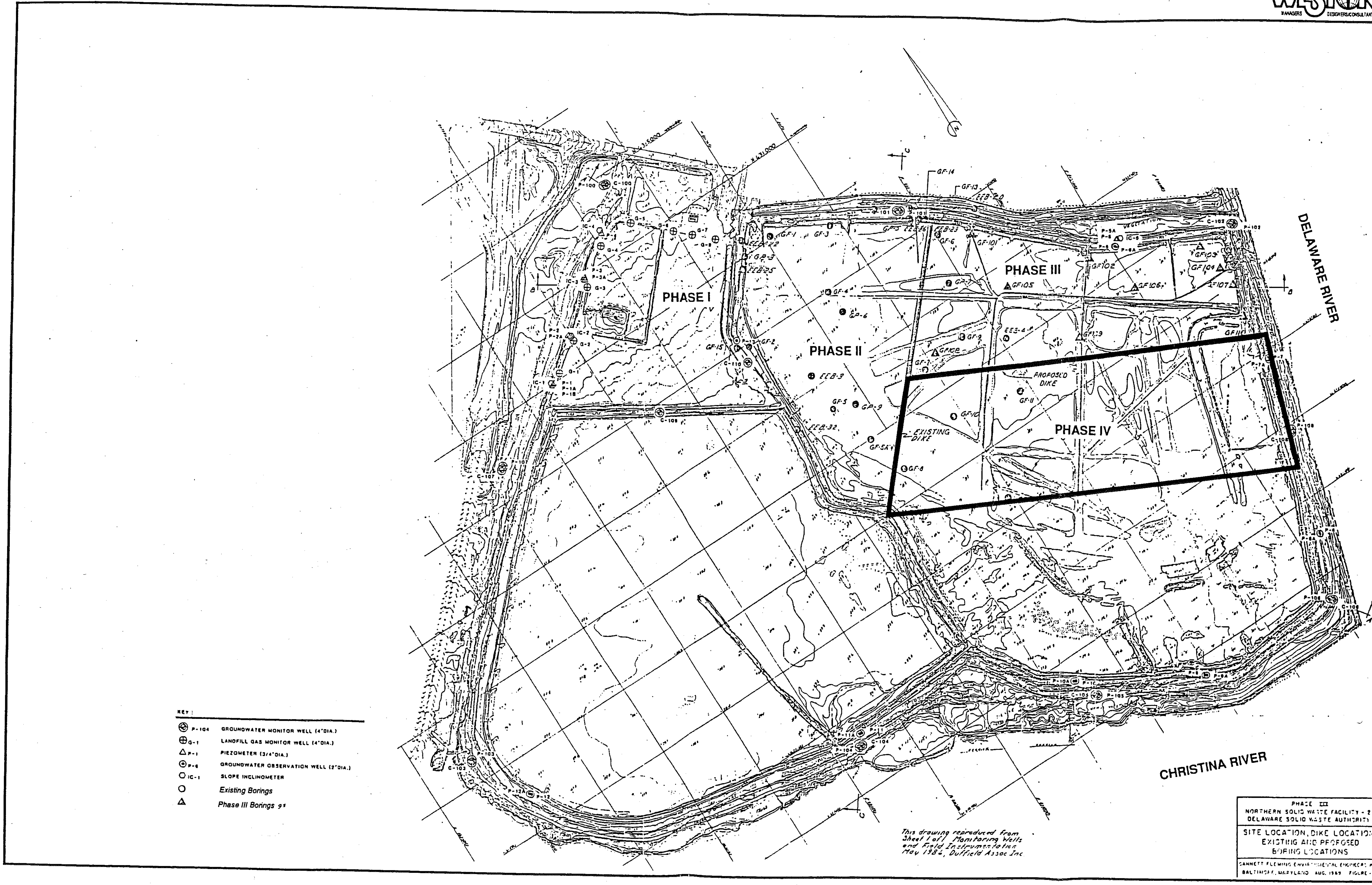

FIGURE 1-1 LOCATION OF PROPOSED PHASE IV DISPOSAL AREA

SECTION 2
HYDROGEOLOGIC INVESTIGATION/ANALYSIS

2.1 GENERAL SITE GEOLOGY AND REVIEW OF EXISTING DATA

2.1.1 General Site Geology

The Cherry Island Landfill site is situated at the confluence of the Christina River and the Delaware River Estuary. The site is bounded to the south by the Christina River and on the east by the Delaware River. The site was once predominantly marshland, but has been extensively filled with dredge spoils from adjacent waterways by the Corps of Engineers.

The site lies on the Coastal Plain physiographic province, several thousand feet southeast of the approximate boundary ("fall line") with the Piedmont physiographic province. The Coastal Plain physiographic province in the area is generally flat and its geology is characterized by layers of unconsolidated clays, silts, sands, and gravels of river, estuary, or marine depositional environments. The coastal plain sediments in the site vicinity are underlain by much older metamorphic and igneous rocks (Woodruff and Thompson, 1975).

The hydrogeology of the Cherry Island site is controlled by four major lithologic units. The lowermost and oldest of these units consists of metamorphic and igneous rocks of the Wilmington Complex (Woodruff and Thompson, 1975). Rocks in the upper part of this sequence are heavily weathered, resulting in a weakly consolidated mixture of clay, silt, sand, and rock fragments. Overlying rocks of the Wilmington Complex are weakly consolidated sediments of the Cretaceous age Potomac Formation. Sediments of the Potomac Formation were deposited in a fluvial environment which resulted in discontinuous channel deposit layers of clay, sand, and silt. Overlying the Potomac Formation are sediments of the Pleistocene Columbia Formation. The unconsolidated sediments of the Columbia Formation are generally coarser than those of the Potomac Formation, and consist of gravels, sands, and silts which are separated from the Potomac Formation by an erosional surface (Sundstrom, et al., 1975; Woodruff and Thompson, 1975; Woodruff, 1985).

Although the Columbia Formation is generally at or near the surface in much of eastern and southern New Castle County, at the site it is overlain by significant accumulations of clay and silt of recent (Holocene) age. Some thin, discontinuous isolated coarse zones, deposited by the Delaware and Christina Rivers, exist within these recent accumulations of clays and silt. Overlying the recent deposits are dredge spoils of similar lithology deposited following dredging of the Delaware and Christina Rivers by the U.S. Army Corps of Engineers. The predominant lithology of both the dredge spoils/recent deposits is that of fine to very fine-grained silts and clays with occasional thin, discontinuous lenses of sand or gravel (Terraqua, 1984; Gannett-Fleming, 1986; Gannett-Fleming, 1990).

2.1.2 Review of Existing Hydrogeologic Data

The hydrogeologic characteristics of the aforementioned lithologic units vary considerably. The igneous and metamorphic rocks of the Wilmington Complex do yield significant quantities of water in some parts of New Castle County, but are not considered a viable water supply aquifer in the site vicinity (Sundstrom and Pickett, 1971; Sundstrom et al., 1975; Woodruff, 1985). Sediments of the overlying Potomac Formation yield significant quantities of potable water within several miles of the site (Terraqua, 1984). However, in the site vicinity, the upper Potomac Formation sand, the most important water-bearing zone in the site vicinity, is thought to be absent (Woodruff, 1985). Water in the Potomac Formation generally exists under confined conditions (Woodruff, 1985). Studies by Terraqua (1984) and Gannett-Fleming (1986; 1990) suggest that although the thick upper Potomac water-bearing zone may be absent in the study area, permeable zones in the Potomac Formation do yield water of marginally potable quality at the site. The quantity of water from such zones is unknown. Water-bearing zones in the Potomac Formation are generally separated from the overlying Columbia Formation by thick clays and silts normally considered to be effective confining layers (Sundstrom and Pickett, 1971; Sundstrom et al., 1975; Woodruff, 1975).

Although water in the Columbia Formation occurs under water table conditions in much of New Castle County, investigations by Terraqua (1984), Gannett-Fleming (1986), and Gannett-Fleming (1990) suggest that the thick (58-70 ft thick) accumulations of overlying recent sediments and dredge spoils create confined conditions in the Columbia water-bearing zone at and near the Cherry Island Landfill. A lens of reworked Columbia sediment occurs within deposits of recent age above the top of the Columbia Formation along the eastern edge of the site. This lens of higher permeable sediment is coarse-grained but contains considerable silt. This lens "interfingers" into the dredge spoils and recent sediments and pinches out beneath the Phase IV expansion area. Water level data indicate it is hydraulically distinct from the Columbia Formation. The wedge is described in the logs for monitor wells C-102, C-106, and C-108 and corresponds to the coarse "sand lens" lying above the Columbia described in Gannett-Fleming (1990).

Interstitial water levels within the dredge spoils/recent deposits materials vary depending on filling and dewatering activities. Filling by either additional dredge spoils or landfill development creates a temporary increase in pore water pressure due to the fine-grained nature of these materials. The dredge spoils/recent deposits comprise the natural liner system for the Cherry Island Landfill. These deposits are of a low permeability and are therefore not considered water-bearing zones but function as the barrier liner between the landfill and the underlying water-bearing zones.

Water levels within the previously investigated water-bearing zones indicated that groundwater flowed predominantly from northwest to southeast in the confined water-bearing zones (Gannett-Fleming, 1986; 1990). The Columbia, and the thin, discontinuous, coarse zones above the Columbia, exhibited sinusoidal water level fluctuations described by Terraqua (1984) and Gannett-Fleming (1986; 1990). These were described as being related to tidal fluctuations in the Delaware and Christina Rivers, which exhibit regular tidal fluctuations. Terraqua (1984) suggested that the tidal response shown in the confined water-bearing zones demonstrated a direct hydraulic connection between these water-bearing zones. Gannett-Fleming (1986; 1990) used the amplitude of tidally related water level fluctuation to contradict Terraqua (1984) by suggesting that the Columbia and the coarse zone above it (defined by wells C-102, C-108, and C-106) were hydraulically distinct.

The dredge spoils/recent deposit materials possesses a very low permeability and therefore do not allow for the rapid dissipation of excess pore water pressures. When additional load is applied to the fine-grained dredge spoils/recent deposits during landfill operation, pore water pressures will increase. The resultant rise in the potentiometric surface within the dredge spoils/recent deposits does not represent an elevated "water-table" when landfill surcharge loads are applied, but rather a temporary increase in excess pore water pressures during consolidation of these materials. Therefore, the depth at the top of a significant water-bearing zone should be considered the appropriate measure of depth to water.

The recharge and discharge characteristics of the hydrogeologic system in the site vicinity are dominated by the Delaware and Christina River embayments and the Piedmont physiographic province to the northwest. Woodruff (1985) indicated that both the Potomac and Columbia aquifers receive recharge waters from off-site outcrop and subcrop areas northwest of the Cherry Island facility. Gannett-Fleming (1986) reported that on-site recharge and discharge occur only to the recent sediments and dredge spoil materials. Groundwater discharge from the confined water-bearing zones underlying the Cherry Island site may occur, but the locations are not known. In general, the base of the Delaware and Christina Rivers would be appropriate discharge locations for the confined water-bearing zones. Vertical gradients measured at well pairs at the site suggest that over most of the site a downward vertical gradient exists between the Columbia and Potomac formations (Terraqua, 1984; Gannett-Fleming, 1986; 1990).

Sampling of groundwater from the Potomac Formation and the water-bearing zones of recent geologic age has shown that groundwater quality varies widely. Evaluations of water quality analyses originally reported by Terraqua (1984) determined that although groundwater from the Potomac Formation was of marginally potable quality, groundwater from the Columbia and the shallower confined water-bearing zone was generally nonpotable. These groundwater quality results were inferred to represent background (pre-landfill) water quality conditions. Groundwater samples from the Columbia and from the confined water-bearing zone above the Columbia exhibited relatively high levels of alkalinity and chloride (although within potable limits for both), specific conductance, hardness, iron, sodium, sulfate, and dissolved solids, among other analytes. High levels of total organic carbon in groundwater samples collected from the Columbia Formation and the confined water-bearing zone above the Columbia, as well as the high alkalinity and dissolved solids values, suggest that this groundwater is under reducing conditions, thus allowing metals to solubilize.

2.1.3 Additional Hydrogeologic Data Requirements for Phase IV

The review of existing hydrogeologic data for the Cherry Island site indicated the need for evaluation of the following items for the purpose of a hydrogeologic evaluation for the Phase IV design and permitting:

- What are the current flow directions in the Columbia and Potomac water-bearing zones?

- What are the probable average horizontal groundwater flow rates within the materials underlying the Phase IV area?

- What is the current quality of groundwater below the site?

The Work Plan (March 1992) defined the specific field activities and laboratory testing required to address these items for the hydrogeologic evaluation of Phase IV. The following section presents a brief discussion on the installation of the piezometers and monitor wells and the groundwater sampling activities. Section 2.3 presents the results of the Hydrogeologic Investigations that address the above items.

2.2 SUMMARY OF HYDROGEOLOGIC FIELD INVESTIGATION ACTIVITIES

2.2.1 Piezometer and Monitor Well Installation

To evaluate the hydrogeologic conditions below the Phase IV disposal area, three piezometers and two monitor wells were installed. The additional monitor wells were used in conjunction with existing monitor wells to determine groundwater flow direction and velocity, aquifer permeability, and groundwater quality. The piezometers were installed to provide information on interstitial water levels in the dredge spoils/recent deposits for stability analysis. All piezometers not required for future monitoring will be removed and properly abandoned prior to construction of Phase III and Phase IV. Methods to monitor pore waters pressure in the dredge spoils/recent deposits during the development of Phase IV will be evaluated during the design process. This may include the use of piezometers within the Phase IV footprint. A recommended method of monitoring excess pore water pressure will be presented to DSWA for review with the final design. All piezometers installed for Phase IV were screened within the dredge spoils/recent deposits.

The new monitor wells were designated to be screened within the Columbia Formation. Monitor wells will serve as groundwater quality monitoring points for the Phase IV design and permitting, and will function as temporary (in the case of C-112) and long term (C-111) points to monitor groundwater quality in conjunction with well clusters already installed along the perimeter dike. Monitor well C-112 will be removed, properly abandoned and grouted when Phase IV construction begins.

The locations of piezometers P-15, P-16, and P-17 and monitor wells C-111 and C-112 are shown on Drawing G-100. At the proposed P-14 piezometer location, methane gas production precluded the construction of a piezometer as discussed in Section 1.

Piezometers and wells were constructed using the standard construction protocols detailed in Section 4-2 of the Work Plan. Installation of the piezometers and monitor wells was in accordance with the Work Plan, however, standard well screens were used instead of "casagrande" type piezometers as described in the Work Plan. Piezometer construction logs are provided in Appendix B. Soil borings installation is described in Section 3.0 of this report. Soil borings were advanced using mud-rotary and using hollow-stem auger techniques. After the field geotechnical engineer recorded the complete lithostratigraphic log from each well and piezometer, the supervising professional geologist, responsible for piezometer and monitor well installation, determined the proper screen placement and well construction details. Well screens consisted of 4 inch diameter Schedule 40 PVC with a 0.010-inch slot, perforated screen connected by flush-jointed and threaded coupling to Schedule 40 PVC riser pipe. Clean, well-sorted silica sand with a medium grain size of 0.75 millimeters was then used to backfill the annular space between the well screen and the borehole to a height of at least 2 feet (P-15 and P-16) and as much as 7 feet (C-112) above the top of the screen.

Newly installed monitor wells were developed by air surging to remove sediment and allow for the later collection of clean groundwater samples. All monitor well drilling and development equipment was decontaminated prior to use in each well by washing with a nonphosphate detergent and then rinsing with potable water. All monitor well subsurface development equipment was decontaminated in this manner. Development time for each well was approximately 4 hours. The installation and development of monitor wells occurred 2 weeks prior to monitor well sampling to allow for the curing of the grout and bentonite seals.

2.2.2 Groundwater Sampling and Analysis

To evaluate groundwater quality in the significant coarse water-bearing zones below the site, groundwater samples were collected between 30 April and 5 May 1992 and were analyzed for significant groundwater quality parameters. A combination of 20 new and existing wells were sampled once for those parameters identified in the current DNREC permit requirements for other phases of the facility. In addition to these parameters, groundwater samples collected from nine Columbia Formation designated wells were also analyzed for 15 metals and the 47 organic compounds listed in Appendix I of the RCRA Subtitle D regulations. The following wells were sampled for groundwater quality:

- Designated Columbia Formation Wells - The two new monitor wells (C-111 and C-112) and existing wells C-100, C-101, C-102, C-103 (dredge spoils and recent deposits), C-104, C-105, C-106, C-107 (dredge spoils and recent deposits), C-108 and C-109. Previously abandoned monitor well C-110 was not sampled.

- Designated Potomac Formation Wells - Existing wells P-100, P-101, P-102, P-103, P-105, P-106, P-107, and P-108.

Groundwater samples were collected by qualified and experienced WESTON personnel following the standard protocols for decontamination and sample collection detailed in the Work Plan. Actual field procedures and equipment used that were not specified or detailed in the Work Plan are listed below:

- The bailers used to collect groundwater samples were of the disposable Teflon® variety. These bailers arrived in pre-sterilized, "sample-ready" condition in separate protective sheaths, thereby precluding the need for decontamination. One bailer was used to sample each well and was disposed-of afterwards.

- During purging, measurements were made in the field of the specific conductivity, temperature, pH, and salinity of the purged water. Although a minimum of three volumes were purged from each well, additional water was purged from some wells until these parameters showed minimal change with further pumping. One low-yield well (C-107 in dredge spoils/recent deposits) was purged until dry then permitted to recover before sampling.

- Samples were collected and analyzed for total iron and total manganese at each location. In addition, soluble iron and soluble manganese samples were collected at all locations sampled for RCRA Subtitle D, Appendix I Metals. Soluble metals samples were first collected in clean, non-preserved bottles. This water was then field-filtered with a 45 micron disposable filter. This filtered water was then decanted into the appropriately pre-marked and preserved sample bottles.

QA/QC samples were collected and analyzed to verify field decontamination procedures, and to document potential sample container and laboratory contamination. These QA/QC samples included:

- Two field decontamination blanks collected from monitor wells C-100 and C-112 by rinsing the probe end of the water level indicator with de-ionized water and containerizing this water in a set of bottles of the same type as those used for groundwater sampling. The water level indicator probe was utilized in lieu of the sampling bailer because the bailers were pre-sterilized prior to use.

- Trip blanks were submitted for laboratory analysis for the batches that contained samples to be analyzed for VOCs. These blanks were only analyzed for VOCs, and originated at WESTON's Lionville Laboratory.

- Duplicate samples of groundwater were collected from four wells, including one from P-100 (which actually monitors groundwater in the Columbia Formation) and one each from the Columbia Formation wells C-100, C-106, and C-112. These samples were used for verification of analysis results. Duplicate samples were collected using the same procedures followed for collection of the initial well water sample and were stored, shipped, and analyzed in the same manner.

- Matrix spike and matrix spike duplicate samples were collected and analyzed at WESTON's Lionville Laboratory from the sample containers submitted by the field sampling team. The results of this analysis were used to evaluate the accuracy/efficiency of the laboratory instrumentation in terms of "percent recovery" of selected spiked, non-target parameters.

- A potable water blank was collected by containerizing water from the NSWF-2 potable supply used for decontamination water. This sample was collected in a manner similar to those outlined above. The sample was analyzed for all standard parameters with the exception of the RCRA Subtitle D Appendix I Metals and RCRA Subtitle D Appendix I Organics. Soluble as well as total iron and total manganese parameters were also analyzed from this sample.

All field personnel followed the EPA chain-of-custody procedures as discussed in the Work Plan to ensure the integrity of all samples.

Using the methods listed in Tables 5-1, 5-2a, 5-2b and 5-3 of the Work Plan, groundwater samples were analyzed for parameters required by the current DNREC permit for other landfill phases. Groundwater samples from 9 Columbia wells were analyzed for the 15 metals and 47 organic compounds listed in Appendix I of the RCRA Subtitle D regulations. The analyses performed are listed in the analytical data summary tables in Section 2.3. All samples were analyzed at WESTON's Lionville laboratories. These laboratories have established a rigorous quality assurance plan to ensure an accurate reporting of sample analysis results. This quality assurance plan (as presented in the Work Plan) was followed for all analyses of groundwater samples collected on this project.

2.2.3 Water Level Monitoring

Water levels in new and existing monitor wells were monitored to evaluate natural groundwater flow directions, horizontal and vertical hydraulic gradients and, in the case of the piezometers, to obtain information on interstitial water levels in the dredge spoils/recent deposits for stability analysis. Two separate types of water level monitoring were conducted. First, water levels in new and existing site monitor wells were recorded twice for comparison to earlier investigations. Water levels were collected first on 29 April 1992 and again on 5 May 1992 for the purpose of constructing piezometric maps of the Potomac and Columbia Formations below the Site.

Second, cyclic water level changes were recorded in two wells (C-104, and C-106) to help determine aquifer interconnections between the Columbia Formation and the Delaware and Christina Rivers. WESTON could not monitor tidal fluctuations in monitor well C-105 as intended because of heavy traffic from construction equipment involved in the berm work around the adjacent Corps of Engineers dredge spoil area. Suspected tidal fluctuations were evaluated over a 48-hour period.

The groundwater sampling team recorded water level measurements for purposes of piezometric mapping. Water levels were collected with a hand-held water level indicator using the procedures detailed in the Work Plan. The location and elevation of the new piezometers and monitor wells were surveyed by VanDemark & Lynch, Inc. Due to potential settlement of existing monitor wells, the elevation of the top of casing of the existing wells was re-surveyed in June 1992. This survey information is provided on Drawing G-100 and the well logs (see Appendix B).

Water level measurements collected to evaluate tidal influences and aquifer interconnections were collected using an electronic data logger and pressure transducer system supplemented by spot checks with a hand-held water level indicator.

The results of water level monitoring was used to develop piezometric maps of each water-bearing zone for which a reliable piezometric surface could be determined. These maps are discussed in subsection 2.3.

2.2.4 In Situ Permeability Testing

Rising head slug tests of both proposed new monitor wells C-111 and C-112 and existing monitor well C-108 were completed to determine average groundwater seepage velocities. These points were selected to measure the in situ-hydraulic conductivities of the Columbia Formation, and a possible isolated coarse zone above the Columbia. Slug tests were not performed on the dredge spoils/recent deposits due to their very low permeabilities. Undisturbed samples were taken of these soils and flex-wall triaxial permeability tests were performed in the laboratory on these samples. The results of the laboratory testing are discussed in Section 3. The results of the slug tests were used to estimate horizontal groundwater seepage velocities in the coarser materials underlying the dredge spoils/recent deposits. The geochemical characteristics of the dredge spoils/recent deposits were also evaluated with regard to potential contaminant transport in support of the Subtitle D performance standards and are presented in Section 3.

2.3 RESULTS OF HYDROGEOLOGIC INVESTIGATION

2.3.1 Water Level Interpretation

Analyses of borehole lithologies and water levels indicated that coarse grained materials of the Columbia and Potomac Formations comprise separate hydrogeologic units. In addition, discontinuous sands above the Columbia appear to be hydrostratigraphically separate from the Columbia. The dredge spoils/recent deposits are not believed to be a significant water-bearing unit at the site since in situ permeabilities between 10^{-6} to 10^{-8} cm/s have been measured from laboratory tests of this material. Monitor wells C-103 and C-107 are believed to be screened within this surficial unit. A summary of water levels collected on 29 April and 5 May 1992, as well as total depth soundings for all existing and newly constructed monitor wells and piezometers are presented in Table 2-1.

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TABLE 2-1

WATER LEVELS, POTENTIOMETRIC SURFACE ELEVATIONS, AND TOTAL MEASURED WELL DEPTHS FOR MONITOR WELLS AT THE CHERRY ISLAND FACILITY

Well	Monitored Formation	Elevation Top of Casing (feet)	Depth to Water Measurements 29 April 1992 (TIC)*	Depth to Water Measurements 05 May 1992 (TIC)*	Piezometric Surface Elevations above mean sea level 05 May 1992	Total Measured Depth During Sampling (TIC)*
C-100	Columbia	18.44	14.16	14.45	3.99	38.03
C-101	Columbia	50.07	48.39	48.67	1.4	85.58
C-102	Isol. Sand Lens	28.95	25.12	27.11	1.84	55.87
C-103	Dredge Soils/Recent	13.13	11.22	10.55	2.58	50.03
C-104	Columbia	(1)	15.44	15.41	2.59	67.38
C-105	Columbia	24.15	22.54	22.01	2.14	74.95
C-106	Isol. Sand Lens	30.06	27.27	27.48	2.58	47.09
C-107	Dredge Spoils/Recent	13.17	10.10	14.38	-1.21	22.80
C-108	Isol. Sand Lens	29.68	26.02	26.61	3.07	40.33
C-109	Columbia	(2)	34.61	34.90	2.26	67.77
C-111	Columbia	46.05	43.00	44.10	1.95	80.25
C-112	Columbia/Isolated Confined Unit	51.57	35.00	35.20	16.37	79.25
P-100	Columbia	18.27	15.71	15.96	2.31	59.14
P-101	Potomac	49.35	47.72	47.75	1.6	168.35
P-102	Potomac	27.98	27.20	26.11	1.87	177.93
P-103	Potomac	14.28	12.51	11.93	2.31	105.16
P-104	Potomac	17.57	Well Damaged	Well Damaged	Well Damaged	Well Damaged
P-105	Potomac	23.81	22.81	22.61	1.2	177.21
P-106	Potomac	29.54	29.83	30.50	-0.96	168.31
P-107	Columbia	12.60	10.17	10.53	2.07	53.27
P-108	Potomac	30.03	29.40	28.70	1.33	182.71
Piezometer						
P-15	Dredge Spoils/Recent	52.40	3.78	3.9	48.50	29.11 (stick-up 4.7 ft)
P-16	Dredge Spoils/Recent	53.86	4.93	5.01	48.85	22.26 (stick-up 6.3 ft)
P-17	Dredge Spoils/Recent	28.03	Piezometer Construction not complete	20.63	7.40	45.47 (stick-up 1.4 ft)

* All measurements recorded from top of innermost casing (TIC).
(1) Not resurveyed.
(2) Damaged at time of survey.

2.3.1.1 Columbia Formation

The Columbia Formation is monitored by wells: C-100, C-101, C-104, C-105, C-109, C-111, and C-112. Monitor wells P-100 and P-107 are designated Potomac wells, but are suspected to be more closely associated with the Columbia Formation. Well C-112 was screened in an isolated coarser lens above the Columbia.

Across the site, the potentiometric surface in the Columbia is slightly above sea level. Net groundwater flow is towards the Delaware River (see Figure 2-1). However, the Columbia Formation is at least in part affected by tidal influence. Section 2.3.1.3 presents an analysis of tidal influence at the site. This influence is believed to be responsible for the "reversal" in groundwater flow (away from the Delaware River) as indicated by the higher water levels measured at well C-105. The equipotential lines shown on Figure 2-1 show the effect of tidal influence on Columbia formation water levels. This well is located within a few tens of yards from the Christina River.

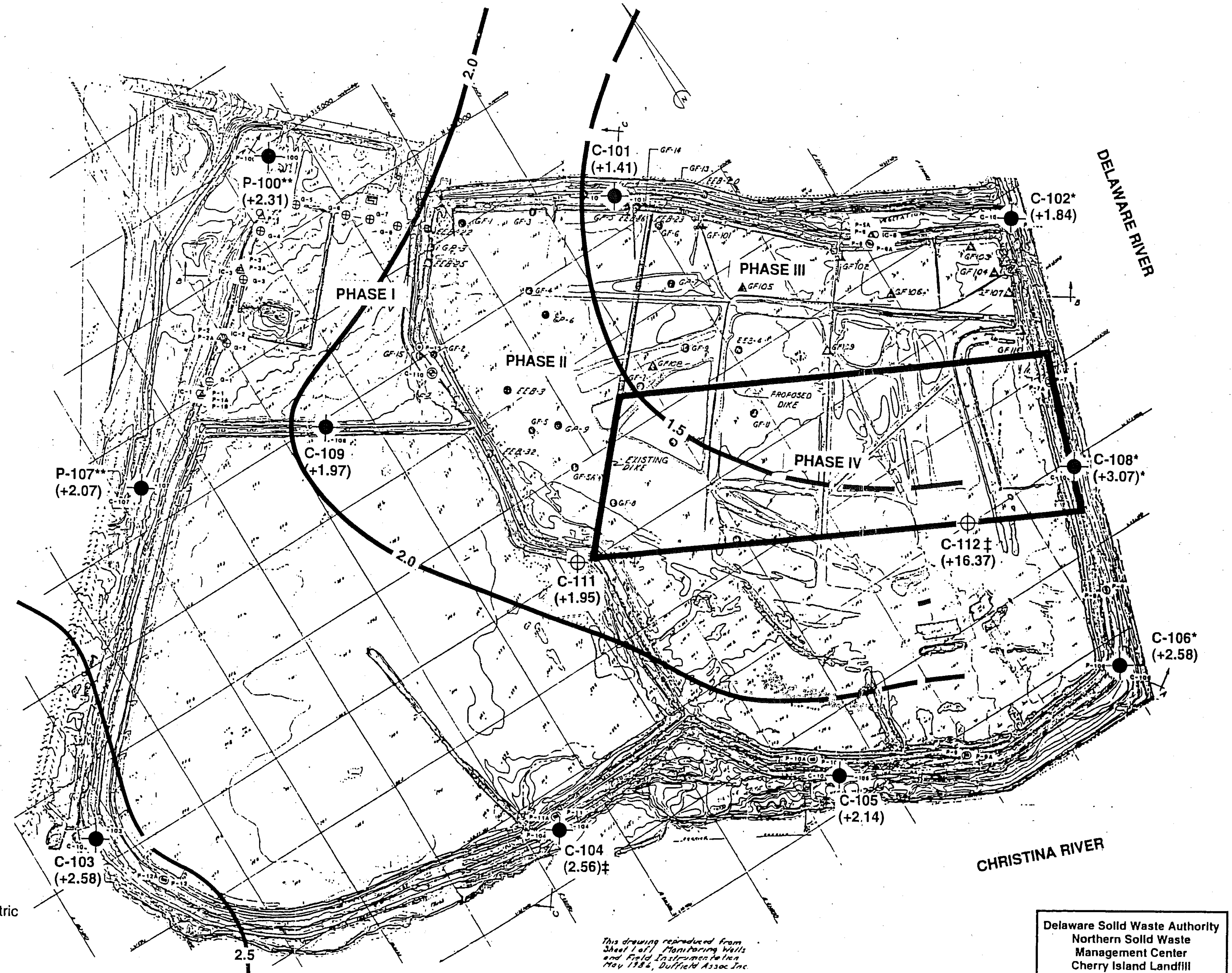
Because short-term tidally-related variation of groundwater levels reciprocates about an average water level slightly above sea level, it is expected that perennial groundwater flow in the Columbia is towards the Delaware River.

The coarser lens encountered within the screen depth of wells C-102, C-106, C-108 corresponds to a hydrostratigraphically distinct unit overlying the Columbia formation. Water levels in these wells are above those in the Columbia Formation. Also, stratigraphic information from these wells and WESTON's TB-7 indicates that there is 10 to 15 feet of recent deposits separating this shallow coarser lens from the underlying Columbia. For these reasons, water levels in those wells were not used to construct the piezometric surface for the Columbia Formation shown in Figure 2-1. Because these three wells lie approximately on a line, no piezometric surface map could be constructed for this shallower coarse lens. Water levels in C-102, C-106, and C-108 are above the top of the shallower coarse lens indicating that it is under confined conditions. The information available suggests that this lens is also of limited extent under the Phase IV expansion area as shown by the absence of this lens in C-112.

2.3.1.2 Potomac Formation

Groundwater in the Potomac Formation is monitored by wells: P-101, P-102, P-103, P-104, P-105, P-106, and P-108. Well P-104 was found to be damaged and could not be used to monitor the Potomac formation water levels. The DSWA is in the process of repairing this well.

Across the site, the potentiometric surface of the Potomac varies between several feet above mean sea level (MSL) and just over one foot below MSL (see Figure 2-2). A slight downward gradient exists between the Columbia and Potomac Formations in most areas



LEGEND

C-101 Existing Columbia Formation Monitor Well

C-111 New WESTON Monitor Well

(+1.91) Water Level Elevation (ft.)

* Columbia Formation Designated Monitor Wells but Actually Screened in the Coarse Grained Isolated Lense Above the Columbia Formation - Not Used for Potentiometric Surface Contours Shown

** Potomac Formation Disignated Monitor Wells Suspected of Being Screened in Columbia Formation - Used for Potentiometric Surface Contours Shown

‡ Possible Confined Isolated Unit Above Columbia Formation - Not Used for Potentiometric Surface Contours Shown

Delaware Solid Waste Authority
Northern Solid Waste Management Center
Cherry Island Landfill
Phase IV Disposal Area

FIGURE 2-1 POTENTIOMETRIC SURFACE FOR THE COLUMBIA FORMATION

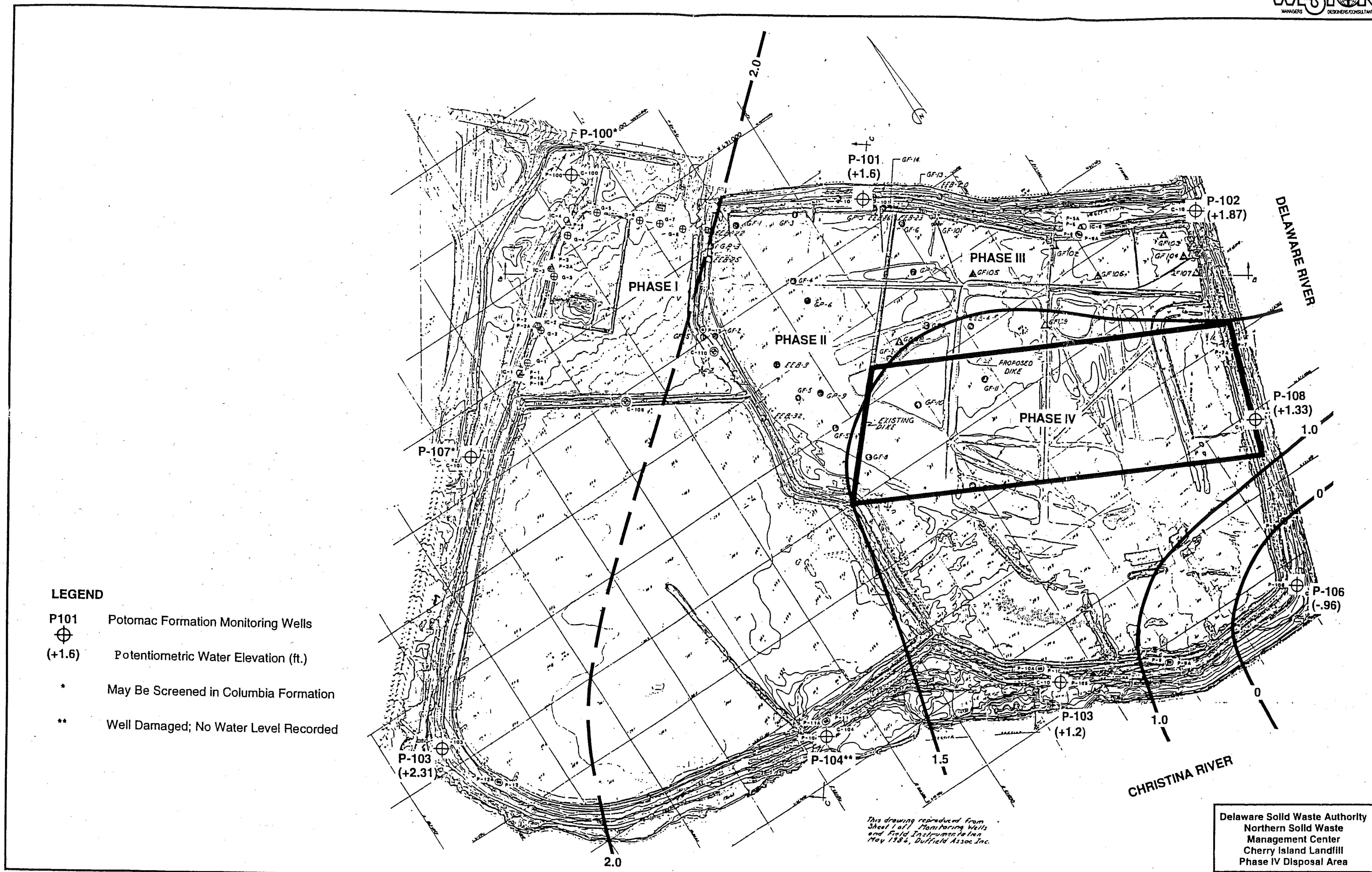


FIGURE 2-2 POTENTIOMETRIC SURFACE FOR THE

across the site. On the average, less than one foot separates the potentiometric surface of these two formations. The gradient is slightly upward in the well pair C-101/P-101. Net groundwater flow in the Potomac is generally eastward, towards the Delaware River.

2.3.1.3 Tidal Influence Study

One well completed within the Columbia Formation (C-104) and one screened across the overlying coarse lens (C-106) were monitored to determine whether a hydraulic connection exists between the Columbia Formation, the isolated coarse lens above the Columbia, and adjacent rivers. Data were collected over a 48-hour period (more than two tidal cycles) between 28 April and 1 May 1992. No rainfall was known to have occurred over the duration of this study. Results of the WESTON tidal study are presented on Figure 2-3 (C-104), and Figure 2-4 (C-106).

Water levels were compared to tide tables published by the National Oceanic and Atmospheric Administration (NOAA) for the period of investigation. Expected tides were generated from data for the reference station at Reedy Point, on the Delaware River. To arrive at expected high and low tide times for the Christina River entrance, 51 minutes was added to the expected Reedy Point high tides and 1 hour and 15 minutes was added to each corresponding low tide as per NOAA instructions.

Data analysis suggests that the shallower coarse lens above the Columbia Formation is in partial hydraulic connection with the Delaware River. Results from C-104 suggest less of a hydraulic connection to the Christina River than indicated in previous studies. The data further suggest no hydraulic connection between the coarser unit above the Columbia Formation and the Columbia itself. These conclusions are based on the following observations:

- The C-106 water level data display regular sinusoidal fluctuations. These fluctuations correspond with the expected tides for the area. As shown on Figure 2-4, an increase in water level of approximately 0.5 feet (maximum upward deflection in water level) occurred roughly ten hours after high tide. Conversely, a nearly equal decrease in water level was noted approximately every eight hours and fifteen minutes after a low tide. This well is completed in the coarser unit above the Columbia formation.

- Results from the WESTON tidal study for C-104 suggested only a slight potential hydraulic connection with the Christina River. The very different tidal responses in C-104 and C-106 indicate that these wells are screened in distinct, isolated units. The results of tidal studies by Gannet-Fleming (1990) on C-101, C-105 (Columbia Formation) and C-102, C-104, C-106 (coarse unit above Columbia) indicated similar results, i.e. the data collected showed that the two units are isolated from each other.

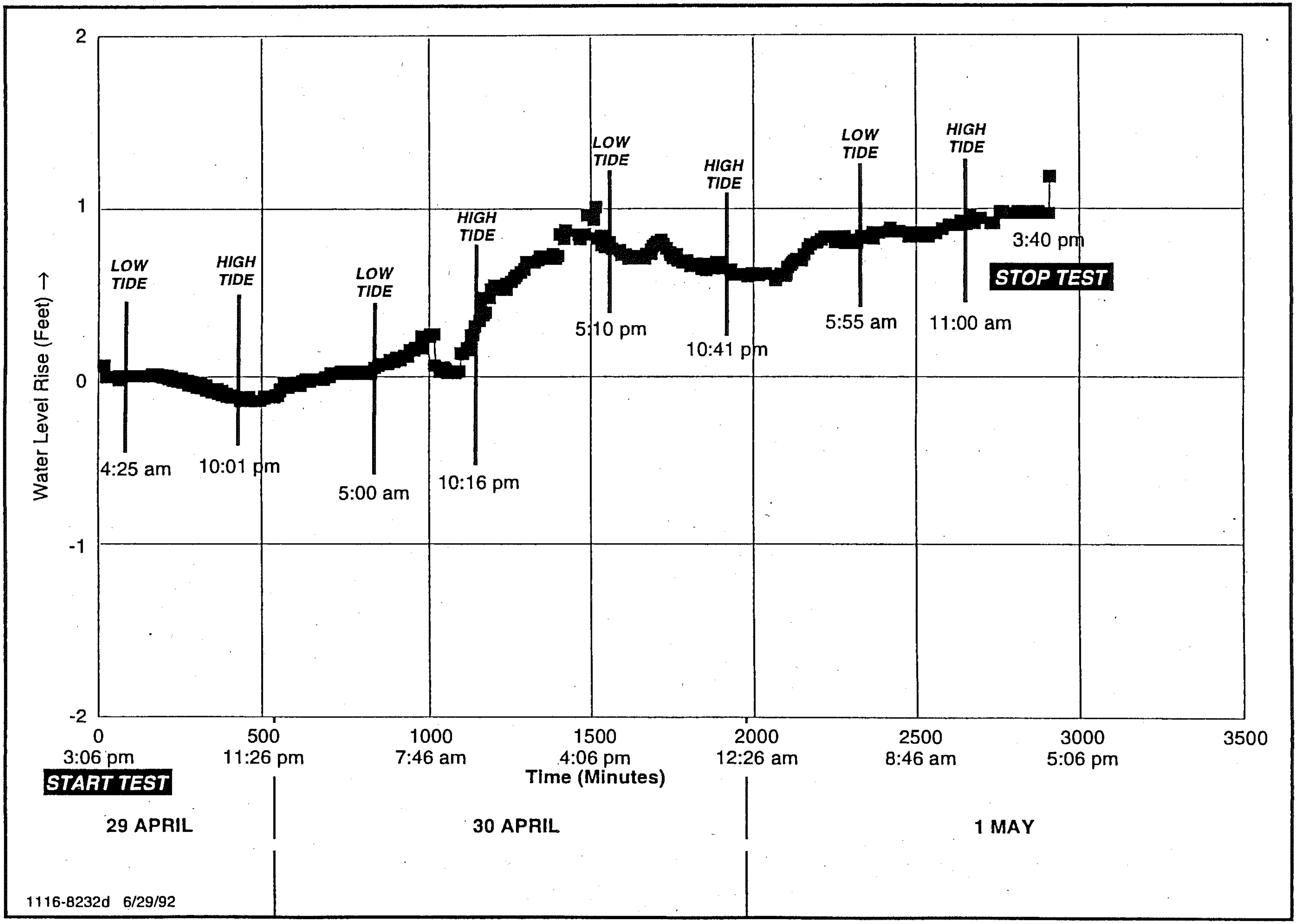


FIGURE 2-3 TIDAL STUDY RESULTS FOR MONITOR WELL C-104

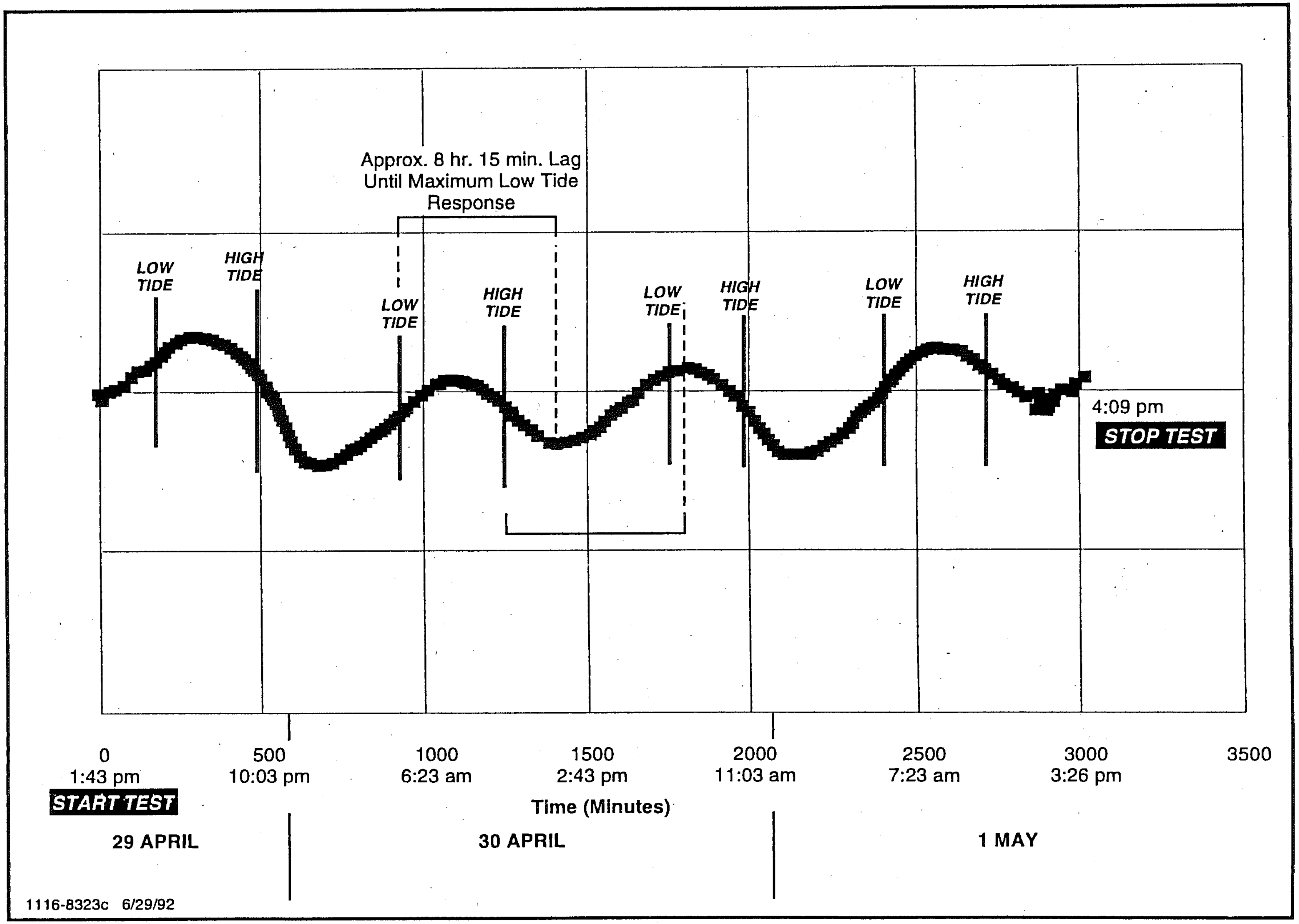


FIGURE 2-4 TIDAL STUDY RESULTS FOR MONITOR WELL C-106

Analysis of water level amplitudes shown in the WESTON and Gannett-Fleming (1990) studies as well as the WESTON potentiometric surface maps suggests that tidal influence is greater in those wells screened in the coarse unit overlying the Columbia near the Delaware and Christina Rivers. A staff gauge will be installed on the river bank adjacent to Phase IV to establish river stages during operations.

2.3.1.4 In Situ Permeability Testing

Slug tests were conducted on the wells C-108, C-111, and C-112 on 5 May, 1992 to obtain a better understanding of hydraulic properties of the coarser units underlying the Phase IV area. Rising head slug tests were used because this method provides the best representation of in-situ hydraulic properties. At each location, well screens were completely saturated. Data from the slug tests were used to derive hydraulic conductivity and seepage velocity values for these locations. These calculated hydraulic conductivities, hydraulic gradient, and seepage velocity are summarized in Table 2-2. The results indicate C-112 was screened in a fine-grained edge of the Columbia Formation that may be an isolated confined unit. Slug test analyses and the calculations used to derive hydraulic conductivity values are located in Appendix B.

Rising head data was analyzed using the Bouwer and Rice method (Bouwer and Rice, 1986) for the determination of hydraulic conductivity. This method tests the hydraulic conductivity of porous media close to the borehole.

Hydraulic conductivities calculated for C-108 and C-111 are similar, 2.6 X 10^{-2} cm/sec and 2.77 x 10^{-2} cm/sec, respectively. These results are expected for clean to slightly silty sands (Freeze and Cherry, 1979). This is supported by the lithologies reported in boreholes logs for the well screen intervals of these wells. At both slug test locations, a return to pretest equilibrium conditions occurred within approximately one minute or less after slug removal. These results suggest that the Columbia, is highly permeable.

Results of the C-112 slug test analysis indicated a marked decrease in hydraulic conductivity. The slug test was run for 53 minutes before equilibrium conditions were achieved. A hydraulic conductivity of 6.1 X 10^{-4} cm/sec was calculated at this well. This value falls within the range for hydraulic conductivity values for silt (Freeze and Cherry, 1979). This value deviates from other calculated conductivity values by two orders of magnitude and is appropriate for the mixture of fine and coarse materials encountered in the screened interval.

The reduced permeability in C-112 suggests that it is approximately at the eastern edge of the Columbia formation. The unusually high water levels recorded in this well suggest that low permeabilities may not allow the well to accurately represent water levels in the Columbia formation or represent an isolated confined lens/unit of the Columbia formation.

TABLE 2-2

HYDRAULIC CONDUCTIVITY AND SEEPAGE VELOCITIES FOR SELECTED COLUMBIA FORMATION WELLS

		Seepage Velocity(v)		
Monitor Well	**Hydraulic Conductivity(k) (cm/sec)**	**Hydraulic Gradient***	**Effective Porosity n****	**Velocity (cm/sec)**
C-108	2.6×10^{-2}	1.4×10^{-3}	.15	2.4×10^{-4}
C-111	2.77×10^{-2}	3.3×10^{-4}	.15	6.09×10^{-5}
C-112	6.1×10^{-4}	9.2×10^{-4}	.15	3.74×10^{-6}

* Hydraulic gradient = change in head per distance; the gradients at C-108, C-111, and C-112 were estimated from Columbia formation piezometric mapping (Figure 2-1).

**An n value of .15 approximates average expected effective porosities for Columbia Formation sand and gravels in Northern Delaware (K. Woodruff, personal communication, 1992).

Hydraulic conductivity values as determined by slug test results were used to determine seepage velocities for each of the slug test locations under static groundwater flow conditions. Seepage velocity is considered an approximation of the average advective groundwater transport rate for a conservative, non-retarded ionic species. Seepage velocities were calculated using the following equation:

$$\bar{V} = \frac{Ki}{n} \tag{1}$$

$\bar{V}$ = Seepage velocity (cm/sec).
K = Hydraulic conductivity (cm/sec).
i = Hydraulic gradient determined from the piezometric maps shown in Figures 2-1 and 2-2
n = Effective porosity of formation. An effective porosity value of .15 was used for all calculations (K. Woodruff, personal communication, May 1992).

Table 2-2 summarizes the results of seepage velocity calculations using this equation. The highest seepage velocity calculated was for C-108. The seepage velocity for C-111, 6.09×10^{-5} cm/sec is intermediate between C-108 (7.4×10^{-4} cm/sec) and C-112 (3.74×10^{-6} cm/sec). It should be noted that the seepage velocities calculated herein only provide an approximation of the rate of groundwater flow beneath the site because the effects upon groundwater gradient resulting from tidal influence cannot be determined precisely.

2.3.2 Groundwater Quality

All, site monitor wells with the exception of P-104 (inner casing broken off below grade) were sampled between 30 April and 4 May 1992 to determine the groundwater quality of the coarse, water-producing strata below the Cherry Island Facility. Groundwater quality parameters measured in the field prior to sampling are presented on Tables 2-3A and 2-3B.

Each Columbia and Potomac Formation well was sampled for the inorganic parameters specified in Table 2-4A. In addition, selected wells in the Columbia Formation and in the coarse unit above the Columbia Formation (C-100, C-101, C-104, C-105, C-106, C-107, C-108, C-111, and C-112) were also sampled and analyzed for RCRA Subtitle D, Appendix I Metals (Soluble; filtered) and Organic Compounds. Total iron and manganese concentrations were also analyzed. The results of analysis of groundwater for the parameters are presented in Tables 2-4B and 2-4C. Where possible, these results were compared to Safe Drinking Water Act Maximum Contaminant Levels (MCL) drinking water standards and the concentrations listed in Terraqua (1984) which provided baseline, natural groundwater quality for the Columbia and Potomac Formations prior to the start of landfill activities at the site.

TABLE 2-3A
COLUMBIA FORMATION
GROUNDWATER QUALITY PARAMETERS FROM FIELD MEASUREMENTS

Well	Temperature Deg. C	Specific Conductance (umhos)	Salinity (ppm)	pH	Comments
C-100	13.5	950	500	6.2	
C-101	14.5	1150	1000	6.4	
C-102	15.0	230	1900	6.9	Bentonite in well (on pump) in sample
C-103	15.0	1900	1500	6.7	
C-104	14.0	1890	1500	6.9	
C-105	14.0	1200	800	6.7	
C-106	15.0	1850	1500	6.9	Petroleum odor
C-107	17.0	1550	1000	6.3	
C-108	17.0	2200	1800	6.6	Strong petroleum odor
C-109	13.0	1000	500	6.7	
C-111	15.0	1170	900	6.1	
C-112	15.5	1710	1500	7.5	Strong petroleum odor

Low	13.0	230	500	6.1
Maximum	17.0	2200	1900	7.5
Mean	14.8	1400	1200	

ppm = Parts per million
Values in table represent final volume observations.

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TABLE 2-3B
POTOMAC FORMATION
GROUNDWATER QUALITY PARAMETERS FROM FIELD MEASUREMENTS

Well	Temperature Deg. C	Specific Conductance (umhos)	Salinity (ppm)	pH	Comments
P-100	13.5	950	500	6.1	
P-101	15.0	400	0.0	7.7	
P-102	15.5	82	500	7.6	
P-103	15.0	1400	800	6.8	
P-104	NS	NS	NS	NS	Well casing broken
P-105	14.5	780	500	7.0	
P-106	16.0	800	500	7.2	
P-107	15.5	1250	1000	6.5	
P-108	19.5	1300	1000	6.3	

Low	13.5	82	0.0	6.1
Maximum	19.5	1400	1000	7.7
Mean	15.6	870.25	600	

ppm = Parts per million
NS = Not sampled
Values in table represent final volume observations.

TABLE 2-4A

Summary of Groundwater Quality Analyses
Inorganic Parameters

Well I.D.	Water-Producing Unit	Analysis Parameters*												
		Sp. Conductance (umho/cm)	TDS (mg/L)	TOC (mg/L)	Chloride (mg/L)	pH	COD (mg/L)	Iron (total) (µg/L)	Alkalinity (mg/L)	Manganese (total) (µg/L)	Nitrate-Nitrogen (mg-N/L)	Ammonia-Nitrogen (mg/L)	Sulfate (mg/L)	Chromium VI (mg/L)
C-100	Columbia	1,120	652	15.0	260	6.1	53.7	58,600	190	6,160	--	2.2	65.0	--
C-100 (dup)	Columbia	1,120	658	16.1	271	6.2	56.9	60,200	195	6,240	--	2.2	39.1	--
C-100 (EB)	Columbia	1.1	--	--	--	6.3	--	--	--	--	--	--	--	--
C-101	Columbia	913	530	28.9	203	6.5	53.7	35,100	190	1,750	.57	5.6	5.5	--
C-102	Columbia	3,100	1,710	172	566	6.6	240	72,100	760	784	--	65.9	--	NA
C-103	Dredge Spoils/ Recent	2,580	1,290	582	430	6.4	63.2	35,100	654	581	0.14	48.7	--	NA
C-104	Columbia	2,430	1,400	43.7	501	6.5	61.6	19,200	475	4,050	.22	39.8	107	--
C-105	Columbia	1,650	875	32	365	6.8	151	6,070	330	241	.10	28.0	2.5	--
C-106	Columbia	2,700	1,560	108	554	6.6	216	53,200	692	1,020	.12	59.3	21.8	--
C-106 (dup)	Columbia	2,560	1,540	119	549	6.6	216	52,000	690	1,020	.28	58.8	--	--
C-107	Dredge Spoils/ Recent	1,630	33.0	149	234	6.3	579	128,000	560	3,630	.22	27.5	4.6	0.23
C-108	Columbia	2,710	1,720	85.5	543	6.7	281	26,400	850	8,710	.10	39.1	2.5	--
C-109	Columbia	1,200	748	30.1	276	6.6	53.7	34,300	290	1,180	.28	4.7	2.8	--
C-111	Columbia	1,490	828	13.3	384	7.2	37.8	2,030	190	595	.10	4.5	17.7	--
C-112	Columbia	2,370	1,430	101	361	6.9	232	30,000	850	1,400	.10	56.1	2.5	--
C-112 (dup)	Columbia	2,280	1,360	106	380	6.8	232	32,900	860	1,490	.1	57.0	2.8	--
MCL	--	--	--	--	--	6.5-8.5	--	--	--	--	10	--	--	0.1

TABLE 2-4B

Summary of Groundwater Quality Analysis
RCRA Subtitle D,
Appendix I, Metals

RCRA Subtitle D Appendix I Metal Analyte (µg/L)	C-100			C-101	C-104	C-105	C-106		C-107	C-108	C-111	C-112			Potable Water Blk	MCL (ug/L)
	Routine	Duplicate	Equipment Blank				Routine	Duplicate				Routine	Duplicate	Equipment Blank		
Antimony (total)	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	10
Arsenic (total)	--	--	--	--	--	--	429	404	--	--	--	511	493	--	--	50
Barium (total)	--	--	--	--	--	257	253	247	--	--	--	--	--	--	--	1000
Beryllium (total)	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	1
Cadmium (total)	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	5
Cobalt (total)	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Chromium (total)	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	0.1
Copper (total)	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	1300
Iron (soluble)	60,700	62,700	--	32,100	18,200	1930	59,200	57,900	107,000	31,300	1,490	36,300	33,200	--	--	--
Lead (total)	--	--	--	---	5.1	5.8	--	4.7	--	--	--	3.2	4.1	--	--	50
Manganese (soluble)	6,570	6,780	--	1630	3770	223	1100	1090	3,590	1,490	525	1500	1450	--	--	--
Nickel (total)	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	100
Selenium (total)	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	50
Silver (total)	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Thallium (total)	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	2
Vanadium (total)	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Zinc (total)	160	162	--	21.9	50.1	21.6	23.2	23.6	--	--	--	--	--	--	--	--

TABLE 2-4C

Summary of Groundwater Quality Analysis
RCRA Subtitle D, Appendix I, Organics

Volatile Organic Compound (µg/L)	C-100				C-101	C-104	C-105	C-106		C-107	C-108	C-111	C-112				
Common Name	Routine	Duplicate	Equipment Blank	Trip Blank				Routine	Duplicate				Routine	Duplicate	Equipment Blank	Trip Blank	M C L (ug/L)
Acetone	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Acrylonitrile	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Benzene	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	5
Bromochloromethane	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Bromodichloromethane	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Bromoform; Tribromothane	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Carbon Disulfide	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Carbon tetrachloride	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	5
Chlorobenzene	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	100
Chloroethane; Ethyl chloride	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Chloroform; Trichloromethane	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Dibromochloromethane; Chlorodibromomethane	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
1,2-Dibromo-3-chloropropane; DSCP	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
1,2-Dibromoethene; Ethlyene dibromide; EDB	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
o-Dichlorobenzene; 1,2-Dichlorobenzene	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	600
p-Dichlorobenzene; 1,4-Dichlorobenzene	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	75
trans-1,4-Dichloro-2-butane	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
1,1-Dichloroethane; Ethylene chloride	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
1,2-Dichloroethane; Ethylene dichloride	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	5
1,1-Dichloroetheylene; 1,1-Dichloroethene; Vinylidene chloride	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	7
cis-1,2-Dichloroethylene; cic-1,2-Dichloroethene	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	70
trans-1,2-Dichloroethylene; trans-1,2-Dichloroethene	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	100
1,2-Dichloropropane; Propylene dichloride	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	5
cis-1,3-Dichloropropene	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
trans-1,3-Dichloropropene	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Ethlybenzene	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	700

TABLE 2-4C (continued)

Volatile Organic Compound (µg/L)	C-100				C-101	C-104	C-105	C-106		C-107	C-108	C-111	C-112				
Common Name	Routine	Duplicate	Equipment Blank	Trip Blank				Routine	Duplicate				Routine	Duplicate	Equipment Blank	Trip Blank	M C L (ug/L)
2-Hexanone; Methyl butyl ketone	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Methyl bromide; Bromomethane	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Methyl chloride; Chloromethane	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Methylene bromide; Dibromomethane	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Methylene chloride; Dichloromethane	--	--	--	--	--	--	--	--	--	--	--	--	--	--	7	8	--
Methyl ethyl ketone; MEK/2-Butanone	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Methyl iodide; iodomethane	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
4-Methyl-2-pentanone; Methyl isobutyl ketone	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Styrene	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
1,1,1,2-Tetrachloroethane	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
1,1,2,2-Tetrachloroethane	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Tetrachloroethylene; Tetrachloroethene; Perchloroethylene	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	5
Toluene	--	--	1 J	--	--	--	--	--	--	--	--	--	--	--	--	--	1000
1,1,1-Trichloroethane; Methylchloroform	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	200
1,1,2-Trichloroethane	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Trichloroethylene; Trichloroethene	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	5
Trichlorofluoromethane; CFC-11	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
1,2,3-Trichloropropane	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Vinyl acetate	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Vinyl chloride	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	2
Xylenes	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	100

-- Parameter not detected in sample.

J = Value detected in sample below reporting limit.

Sampling of groundwater from the Potomac Formation and the younger water-producing zones has shown that groundwater quality varies widely. Water quality analyses originally reported by Terraqua (1984) found that although groundwater from the Potomac Formation was generally of potable quality, groundwater from the Columbia and the shallower confined water-producing zone were generally nonpotable. These groundwater quality results represented background (pre-landfill) water quality conditions. Groundwater samples from the Columbia and from the lens of coarse deposits above the Columbia (C-102, C-106, and C-108) exhibited relatively high levels of alkalinity, chloride, high specific conductance, hardness, iron, sodium, sulfate, and dissolved solids, among other analytes. Chloride, sulfate, and dissolved solids concentrations exceeded several generally accepted potable limits at most shallower wells. High levels of total organic carbon in groundwater samples collected from the Columbia Formation and the isolated coarse unit, as well as the high alkalinity and dissolved solids values, suggest that these groundwaters are under reducing conditions (with a resultant oxygen deficit) thus allowing metals to solubilize. The results of current groundwater quality analyses confirmed these earlier results.

2.3.2.1 Inorganic Parameters

Columbia and Potomac Formation monitor wells at the site were sampled then analyzed for the inorganic parameters presented on Table 2-4A. Wells C-103 and C-107 were determined to be screened in the dredge spoils and recent deposits (see Section 2.3.1). For this reason, groundwater sample analysis results from these wells are not considered indicative of the Columbia Formation.

Recent analysis results indicated that groundwater quality in both the Columbia and Potomac Formations is poor. Analysis also suggests that Potomac Formation water quality is only slightly better than in the overlying Columbia.

The results of laboratory testing approximately correspond with the results of measurements of field parameters collected during sampling. These measurements are summarized on tables 2-3A and 2-3B for the Columbia and Potomac Formations, respectively.

Groundwater quality data collected by WESTON was compared to that collected by Terraqua (1984) for these parameters. In general, and with few exceptions, levels of inorganic parameters in groundwater at the site have remained consistent with results reported in Terraqua (1984). Groundwater in most wells was of low quality.

2.3.2.2 RCRA Metals

Columbia Formation wells C-100, C-101, C-104, C105, C-107, C-111, and C-112 and wells screened in the isolated coarse unit above the Columbia Formation (C-102, C-106, and C-108) were sampled and analyzed for RCRA Appendix I, Subtitle D soluble metals. The results of analysis are presented in Table 2-4B. All accessible wells were also analyzed for total iron and manganese (see Table 2-4A).

Results of analysis indicated that groundwater in both the Columbia and Potomac Formations is very high in both total and soluble iron and manganese. These levels indicate that groundwater in both formations is of poor quality. WESTON data was within ranges for iron and manganese previously reported in Terraqua (1984). Columbia Formation groundwater had higher concentrations of metals, than did the underlying Potomac.

The comparison of total iron and manganese concentrations with soluble (filtered samples) concentrations of these metals showed little significant difference. Comparable levels of iron and manganese in total and soluble form suggests that total and soluble concentrations of the other tested metals would also be comparable. For this reason, field filtration of metals should allow the collection of representative groundwater samples, and is recommended for future sampling events if metals are analyzed.

The only other RCRA Appendix I, Subtitle D soluble metals detected above detection limits were: arsenic, barium, chromium VI, lead, and zinc. With the exception of arsenic, these metals were all well below Federal MCL. Arsenic was detected in wells C-106 and C-112 at levels between 0.49 mg/L and 0.51 mg/L. All metals, arsenic included, were within the expected concentration ranges described in Terraqua (1984). These wells are considered to be hydraulically distinct from the Columbia Formation as discussed previously. The arsenic, chromium, or lead detected in some groundwater samples from wells completed in recent deposits and Columbia along the Delaware/Christina Rivers suggest that leached water from dredges spoils may be the origin of these RCRA metals.

A very low concentration of chromium VI (0.0023 mg/L) was reported in C-107. This well was determined to be screened within the overlying dredge spoils/recent deposits. Because chromium VI was detected in C-107 only, and at a very low concentration, it is suggested that this metal exists in trace quantities (background) related to the fine grain dredge spoils/recent deposits.

2.3.2.3 RCRA Organics

No significant organic compounds were detected in groundwater samples from site monitor wells. Columbia Formation wells C-100, C-101, C-104, C-105, C-107, C-111, and C-112 and isolated coarse unit wells C-102, C-106 and C-108 were sampled and analyzed for RCRA Appendix I, Subtitle D Volatile Organics. Results of analysis are presented in Table 2-4C. Toluene, probably unrelated to on-site sources, was detected at a concentration below method quantification limits and is probably a false reading. The common laboratory solvent methylene chloride was detected in two samples at concentrations less then 10 times their detected concentration in blank samples. For this reason, it is not considered present in natural groundwater. The petroleum odor reported in C-112 probably represents natural organic decay by-products.

2.4 SUMMARY OF HYDROGEOLOGIC CHARACTERISTICS

The investigation of the Phase IV Disposal Area included assessments of site hydrogeology and groundwater quality. The investigation of hydrogeologic characteristics showed that the Potomac, Columbia, and shallower coarse units were confined by thick overlying silts/clays, and were hydraulically distinct. The Potomac and shallower coarse unit were found to be discontinuous.

Flow directions were found to be generally southeastward, confirming earlier investigations. Both the Columbia and the shallower coarser unit experienced water level fluctuations related to tidal effects which created transient reversals in flow direction. Recharge to coarser units is predominantly upgradient and off-site, while discharge areas probably include the Delaware River. Vertical hydraulic gradients are slightly downward at Columbia/Potomac well pairs. Hydraulic conductivities in the Columbia Formation were found to be consistent with silty sands.

Sampling of groundwater from the Potomac Formation, the Columbia Formation, and the shallower coarser unit has shown that groundwater quality varies widely. Evaluations of water quality analyses originally reported by Terraqua (1984) determined that although groundwater from the Potomac Formation was generally of potable quality, groundwater from the Columbia and the shallower confined lens was nonpotable. Groundwater samples from the Columbia and from the shallower coarser unit above the Columbia exhibited relatively high (non-potable) levels of many analytes. High levels of total organic carbon in groundwater samples collected from the Columbia formation and the confided water-bearing zone above the Columbia, as well as the high alkalinity and dissolved solids values, suggest that this groundwater is under reducing conditions (with a resultant oxygen deficit) thus allowing metals to solubilize.

Stratigraphic data suggest that there is no continuous water-bearing unit containing groundwater of potable quality below the Phase IV expansion area. The Potomac and the shallow coarser unit were discontinuous below the Phase IV expansion area. Groundwater quality from all three units was poor, and was non-potable from the shallow coarser unit. These groundwater quality conditions were interpreted to represent pre-landfill conditions.

Analysis and comparison of recent and baseline (1984) groundwater quality data indicated that the waters of the Columbia and shallow coarser units are non potable. Data further suggests that the quality of present site groundwater has not deteriorated since landfill operations commenced.

SECTION 3
GEOTECHNICAL INVESTIGATION/ANALYSIS

3.0 INTRODUCTION/LIMITATIONS

The geotechnical investigation and analysis study reported herein was completed by WESTON for DSWA in conjunction with the design and construction of the Phase IV Disposal Area at the Cherry Island landfill facility in Wilmington, Delaware. Figure 3-1 (Regional Location Plan) shows the location of the project site.

Limitations

The analyses and recommendations provided herein were developed from the data obtained from the test borings and wellpoints completed by WESTON at this site. This data defines subsurface conditions at these specific locations and only at the particular time the work was performed. The nature and extent of variations between the test boring and wellpoint results and other site conditions may not become evident until construction. If variations become evident, they shall be brought to the attention of WESTON immediately, as it may be necessary to re-evaluate the contents of this report to account for actual conditions encountered and/or new data which may arise. The analyses and design conclusions of this report are also based on the premise of competent field engineering and inspection during landfill construction.

The investigation and analyses discussed herein were performed in accordance with generally accepted geotechnical engineering principles, methodologies, and practices. No other warranty, expressed or implied, is made. In addition, WESTON is not responsible for the independent conclusions, opinions, or recommendations made by others based on the contents of this report.

3.1 PROJECT DESCRIPTION

The Phase IV site will be located within the Edgemoor Area of Cherry Island. Phase IV will be bounded by the existing Phase II Landfill to the west, the Phase III Landfill to the north, and an existing earthen perimeter dike to the east which separates the Edgemoor area from the adjacent Delaware River (see Figure 3-2: Site and Boring Location Plan). The southern boundary of Phase IV will extend from the southernmost point of Phase II in a southeasterly direction and approximately parallel to the southern Phase III separation dike until it intersects the existing Delaware River perimeter dike. The Phase IV site will encompass approximately 50 acres of landfill area.

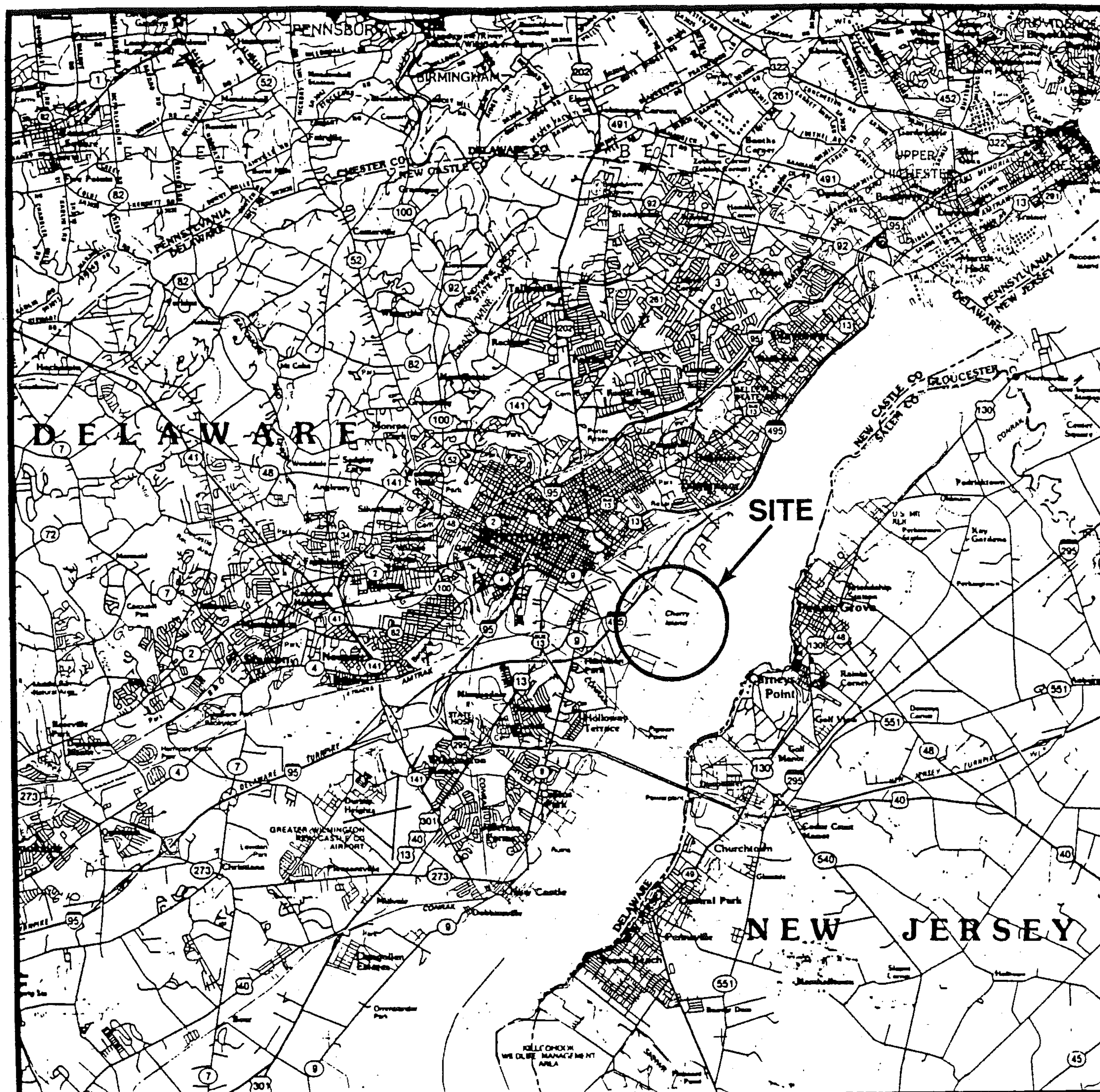


FIGURE 3-1
REGIONAL LOCATION PLAN

DSWA PHASE IV LANDFILL CHERRY ISLAND FACILITY WILMINGTON, DELAWARE	ROY F. WESTON, INC. WESTON MANAGERS DESIGNERS/CONSULTANTS				
	DRAWN A. DELTUFFO	DATE 7/22/92	DES. ENG.	DATE	W. O. NO. 2477-03-01
	CHECKED		APPROVED		DWG. NO.

Figure 3-2

3.2 SITE CONDITIONS

The site of the proposed Phase IV landfill footprint consists of the impounded dredge spoils within the Edgemoor disposal area. These materials are generally soft, wet, weak, and compressible, but typically exhibit a stronger crust during warmer weather months due to surficial drying. The ground surface elevation of the dredge spoils is approximately +48 feet within the Phase IV area as determined from the topographic site base map prepared by Vandemark & Lynch, Inc., February 1992.

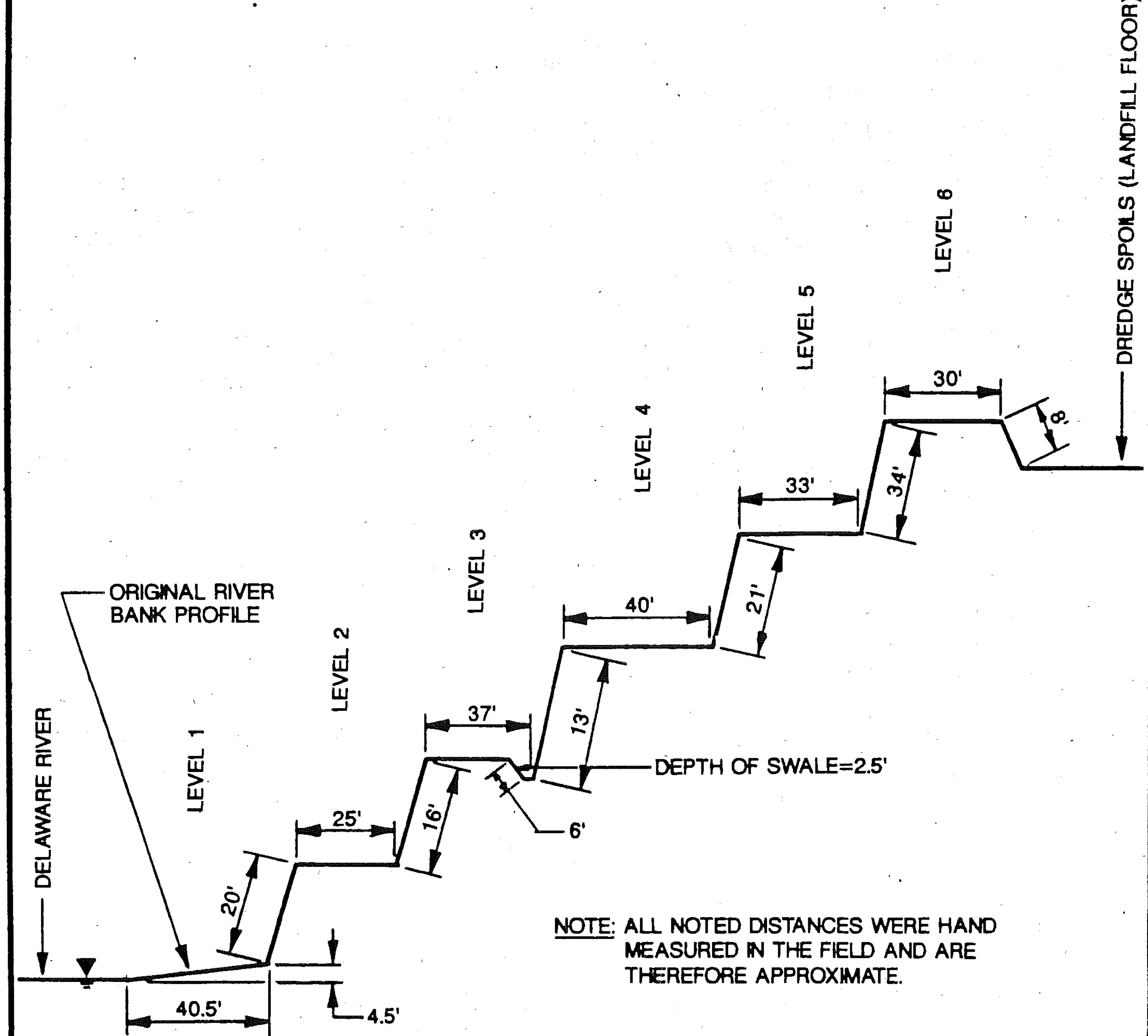
An earthen perimeter dike physically separates the Delaware River from the Edgemoor disposal area on the eastern end of the site within the Phase IV footprint. The top elevation of this dike is approximately +53 feet, that is, approximately 5 feet higher than the surface elevation of the impounded dredge spoils. The outslope of this dike within the Phase IV area includes 4 man-made benches as shown on Figure 3-3. The intermediate sloping faces of this dike are inclined at slopes up to approximately 1H:1V in steepness.

A second earthen dike physically separates the western end of the Phase IV footprint from the active Phase II disposal area. Landfilling activities within the Phase II area have reached the approximate design top elevation of 128 feet. However, settlement has reduced the present top elevation of the landfill to less than this value.

3.3 SITE GEOLOGY

The Cherry Island Landfill site is situated at the confluence of the Christina River and the Delaware River. The site is bounded to the south by the Christina River and on the east by the Delaware River. The site was once predominantly marshland, but has been extensively filled with dredge spoils from adjacent waterways by the Corps of Engineers. The dredge spoils are retained by earthen dikes which separate these materials from the adjacent Christina and Delaware Rivers.

The site lies on the Coastal Plain physiographic province, several thousand feet southeast of the approximate boundary ("fall line") with the Piedmont physiographic province. The Coastal Plain physiographic province in the area is generally flat and its geology is characterized by layers of unconsolidated clays, silts, sands, and gravels of river, estuary, or marine depositional environments. The coastal plain sediments in the vicinity of the site are underlain by much older metamorphic and igneous rocks.



NOTE: ALL NOTED DISTANCES WERE HAND MEASURED IN THE FIELD AND ARE THEREFORE APPROXIMATE.

FIGURE 3-3
CROSS SECTION OF DELAWARE RIVER
PERIMETER DIKE FACING SOUTH

DSWA PHASE IV LANDFILL CHERRY ISLAND FACILITY WILMINGTON, DELAWARE	ROY F. WESTON, INC. WESTON MANAGERS DESIGNERS/CONSULTANTS				
	DRAWN A. DELTUFFO	DATE 6/29/92	DES. ENG.	DATE	W. O. NO. 2477-03-01
	CHECKED		APPROVED		DWG. NO.

The geology of the Cherry Island site is controlled by four major lithologic units. The lowermost and oldest of these units consists of metamorphic and igneous rocks of the Wilmington Complex (Woodruff and Thompson, 1975). Rocks in the upper part of this sequence are heavily weathered, resulting in a weakly consolidated mixture of clay, silt, sand, and rock fragments. Overlying rocks of the Wilmington Complex are weakly consolidated sediments of the Cretaceous age Potomac Formation. Sediments of the Potomac Formation were deposited in a marine environment which resulted in broad, relatively continuous layers of clay, sand, and silt (Sundstrom, et al., 1975). Overlying the Potomac Formation are sediments of the Pleistocene Columbia Formation. The unconsolidated sediments of the Columbia Formation are generally coarser than those of the Potomac Formation, and consist of gravels, sands, and silts which are separated from the Potomac Formation by an erosional surface (Sundstrom, et al., 1975; Woodruff and Thompson, 1975; Woodruff, 1985). Although the Columbia Formation is generally at or near the ground surface in much of eastern and southern New Castle County, at the Phase IV site it is overlain by significant accumulations of clay and silt with some interbedded thin, discontinuous sand lenses. These materials were deposited by the Delaware River and its tributary, the Christina River through alluvial action. These fine-grained recent deposits were thickened by dredge spoils of similar lithology deposited following dredging of the Delaware and Christina Rivers. The predominant lithology of both the recent deposits and dredge spoils is that of fine to very fine-grained silts and clays with occasional thin discontinuous lenses of sand or gravel (Terraqua, 1984; Gannett-Fleming, 1986; Gannett-Fleming, 1990). Based on an approximate Delaware River water surface elevation of +4 feet as measured during the most recent topographic survey of the site, the elevation of the contact between the dredge spoils and underlying recent deposits can be reasonably assumed to be approximately +4 to +10 feet across the Phase IV footprint.

3.4 SUBSURFACE INVESTIGATION

Subsurface conditions at the project site were investigated by completing a total of ten (10) test borings at the locations shown on Figure 3-2, Site and Boring Location Plan. These borings are numbered TB-1 through TB-5, TB-7, TB-7A and TB-8 through TB-10 on Figure 3-2. During the course of the drilling work, when it became apparent that subsurface conditions within borings internal to the landfill footprint were reasonably uniform and consistent, it was decided to delete Boring TB-6 originally planned at a location along the southern boundary of Phase IV. In addition, Boring TB-7, also originally planned along the Phase IV southern boundary, was relocated to a location along a lower bench of the Delaware River perimeter dike to supplement the data obtained from TB-9 which was completed from the top of this dike. Also, an offset boring to TB-7, i.e. TB-7A, was completed for the purpose of obtaining additional "undisturbed" Shelby Tube samples of the fine-grained dike soils for subsequent laboratory testing.

The boring locations were originally positioned in the field by WESTON to provide uniform coverage of the landfill footprint as well as to provide the most direct and least complicated access to these locations for the drill rigs (e.g. by minimizing the number of existing dewatering trenches which the rigs had to cross to access these locations). The completed borings were subsequently surveyed by Vandemark & Lynch, Inc. so that accurate locations and ground surface elevations could be determined. The surveyed boring locations are reflected on Figure 3-2. The surveyed ground surface elevations are presented on the boring logs presented as Appendix C of this report.

The test borings were completed by the Walton Corporation of Newark, Delaware under the direct technical supervision of WESTON. The borings were drilled between April 6 and April 30, 1992. The borings were completed using two All Terrain Vehicles (ATV's). Ancillary equipment (i.e. a small dozer and wooden mats) was also necessary to assist the rigs in crossing existing dewatering trenches.

Each boring was initially advanced using hollow stem augers to a depth that was typically below the saturated zone within the dredge spoils (i.e. generally 8 to 13 feet below the existing ground surface). This allowed the depth of free water within this saturated zone to be measured within the augers. At this point, the drilling technique was converted to mud rotary as a means to partially suppress methane emissions which were emerging from the boreholes from assumed biodegradation of subsurface organic deposits.

All borings which were completed within the proposed landfill footprint were advanced until the fine-grained dredge spoils and recent deposits were fully penetrated and dense underlying granular deposits were encountered. The two deep Delaware River dike borings (TB-7 and TB-9) were also advanced until the interbedded fine and coarse grained soils which comprise both the dike and the underlying natural deposits were fully penetrated and deeper dense granular soils were encountered.

During the drilling work, representative samples of the encountered coarse and fine-grained soils were collected from the borings using the procedures of ASTM D-1586, "The Standard Penetration Test." This procedure requires that samples be obtained using a 2-inch O.D. split-barrel sampler which is driven 18 inches by a 140 pound hammer which freely falls a distance of 30 inches. The number of hammer blows which is required to drive the sampler during the interval from 6 to 18 inches, or fraction thereof, is reported on the test boring logs as the "N" value. In many instances, standard penetration testing within the very soft and weak fine-grained dredge spoils or underlying recent deposits resulted in a full 18 inches of split-spoon penetration into these materials from the weight of the rods (WOR) or the weight of the 140 pound hammer (WOH). In these instances, the "N" value is recorded as WOR or WOH.

"Undisturbed" Shelby Tubes samples of the fine-grained dredge spoils, underlying recent deposits and the Delaware River perimeter dike soils were also obtained using the procedures of ASTM D-1587, "Thin-Walled Tube Sampling of Soils." A total of 34 tubes were obtained during the drilling work. All tubes were trimmed, waxed, capped, sealed and transported in accordance with acceptable procedures so as to maintain the integrity of these samples.

The undrained cohesive shear strength (c_u) of encountered fine-grained dredge spoils, alluvial recent deposits and Delaware River perimeter dike soils was also measured directly in the field at selected locations and depths using the following procedures:

1. Pocket Penetrometer Testing - These tests were completed on all recovered split-spoon samples of the fine-grained soils. This test measures the unconfined compressive strength (q_u) of soil from which the undrained cohesion may be calculated (i.e. $c_u = q_u/2$).

2. Torvane Testing - These tests were completed on the exposed soils at the bottom of all recovered Shelby Tube samples of the fine-grained materials. This test measures the undrained cohesive shear strength of the soil directly.

3. In Situ Vane Shear Testing - These tests were completed directly within the in situ fine-grained soils at preselected depths within the boreholes. The testing procedure consists of attaching a vane shear of standardized geometry to the drill rods, and lowering this instrument to the bottom of the borehole, after which it is pushed to full embeddment within the underlying soils. A torque wrench is then used to rotate the vane until a shear failure is created between soils internal and external to the vane. The measured value of torque is input into a governing equation to calculate the undrained cohesive shear strength. In situ vane shear tests were performed in accordance with the standardized procedures of ASTM D-2573, "Field Vane Shear Test in Cohesive Soil."

Wellpoints were also installed in a select number of boreholes for the purpose of obtaining long-term measurements of the depth to water within the saturated zone of the fine-grained soils. These wellpoints were installed in boreholes TB-1, TB-2, and TB-7, and generally extended to depths of approximately 10 feet below the encountered water level to allow for possible fluctuations of this level. The wellpoints consist of 1½ inch PVC solid wall pipe with a 5 foot well screen and end cap. Following installation of the wellpoint, the borehole annulus was backfilled with a coarse grained sand which was gradation compatible with the well screen. The remainder of the borehole was subsequently backfilled to the ground surface with an impervious cement/bentonite grout mix. This same grout mix was used to initially backfill the deep boreholes to the level of the much shallower wellpoints before their installation. Two monitoring wells were also installed within boreholes TB-5 and TB-8. These boreholes were screened within the sands and gravels of the granular deposits which underlie the fine-grained soils at the site. Details of this monitoring well program are discussed in Section 2 of this report.

All remaining boreholes which were not retrofitted with a wellpoint or monitoring well were fully grouted to the ground surface following their completion using the same impervious cement/bentonite grout mix discussed above. This construction activity represents an environmental safeguard in that it will prevent future landfill leachate from migrating through the boreholes to the underlying granular soils.

3.5 SUBSURFACE CONDITIONS

The subsurface conditions encountered at the site are depicted on both the individual boring logs presented in Appendix C, and Subsurface Profiles A-A, B-B and C-C presented as Figures 3-4 through 3-6 of this report. Stratigraphic descriptions, split-spoon sampler "blow counts" and Standard Penetration Resistance (i.e. "N") values of the various soil strata encountered during the drilling work are presented on the boring logs and subsurface profiles, as well as the stratigraphic interface depths and elevations, and the groundwater levels as measured during the drilling program. The classification of soils from within the various strata encountered during the drilling work as determined from the procedures of the Unified Soil Classification System (USCS) are documented in the results of the laboratory physical properties testing program presented in Appendix D of this report.

The Phase IV landfill site is underlain by approximately 75 to 80 feet of fine-grained soils. The upper approximately one-half of this stratum thickness (i.e. approximately 40 feet) is believed to consist of dredge spoils which were pumped to deposition within the diked Edgemoor Area by periodic dredging of Delaware River bottom sediments by the U.S. Army Corps of Engineers. These materials are classified as ML (low plasticity clayey silt) or MH (high plasticity clayey silt) soils based on their plasticity properties as determined by the Unified Soil Classification System (see Figure 3-7). These materials were generally noted to be very soft, compressible and of very low in situ shear strength as noted by typical "N" values of WOR and WOH. Pocket penetrometer, torvane and in situ vane shear testing results were also indicative of the very low in situ shear strength of these soils.

The lower approximately one-half of the fine-grained soil stratum thickness is believed to be of natural alluvial depositional history. These materials are designated as "recent deposits" on geologic maps of the area. Based on their plasticity properties, these materials are also classified as ML and MH soils according to the USCS as shown on Figure 3-8. These materials were also noted to be soft, compressible and of low shear strength.

Figure 3-4

Figure 3-5

Figure 3-6

PLASTICITY CHART

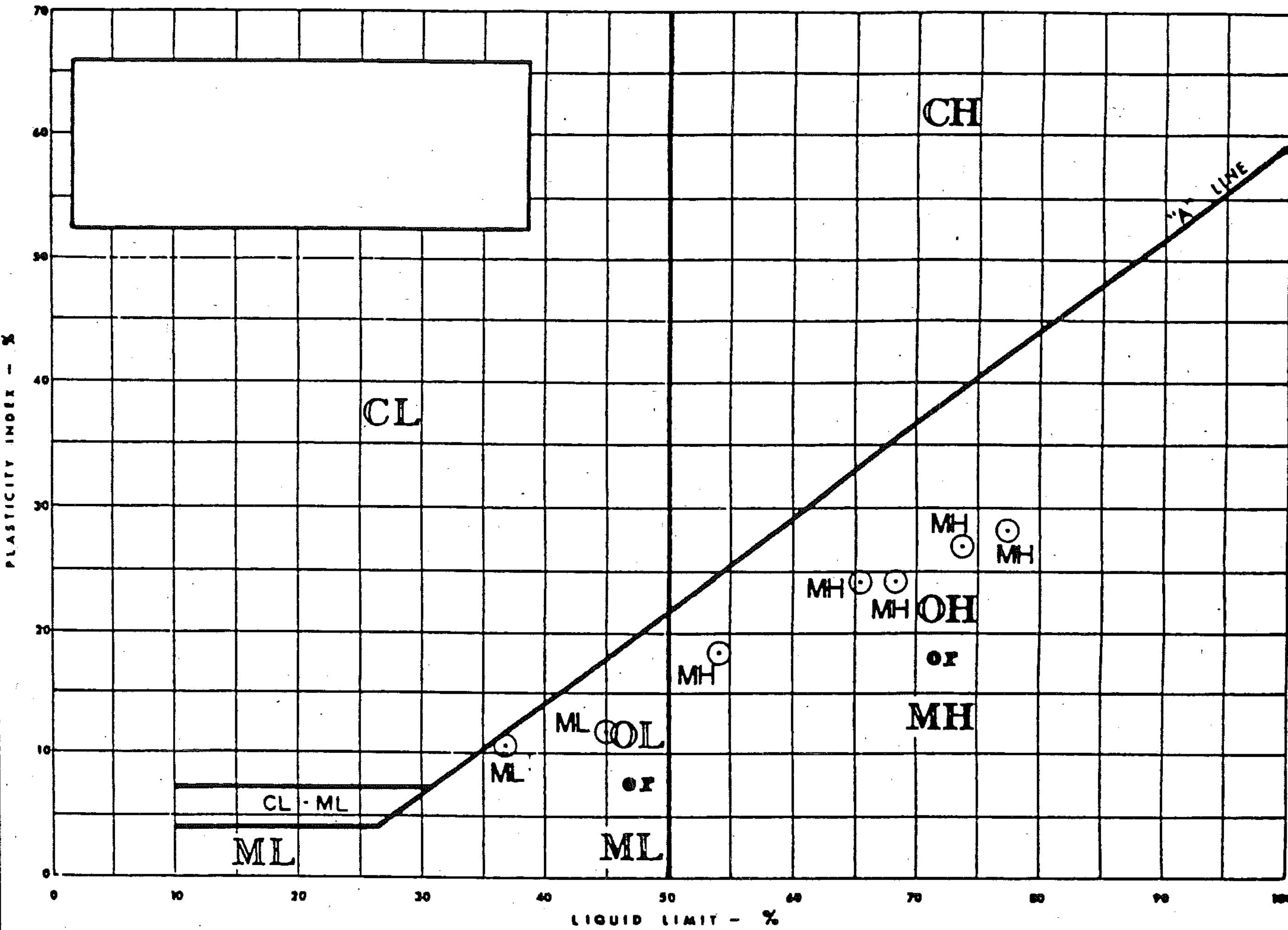


FIGURE 3-7
PLASTICITY OF DREDGE SPOILS

DSWA
PHASE IV LANDFILL
CHERRY ISLAND FACILITY
WILMINGTON, DELAWARE

ROY F. WESTON, INC.
WESTON
MANAGERS DESIGNERS/CONSULTANTS

DRAWN	DATE	DES. ENG.	DATE	W. O. NO.
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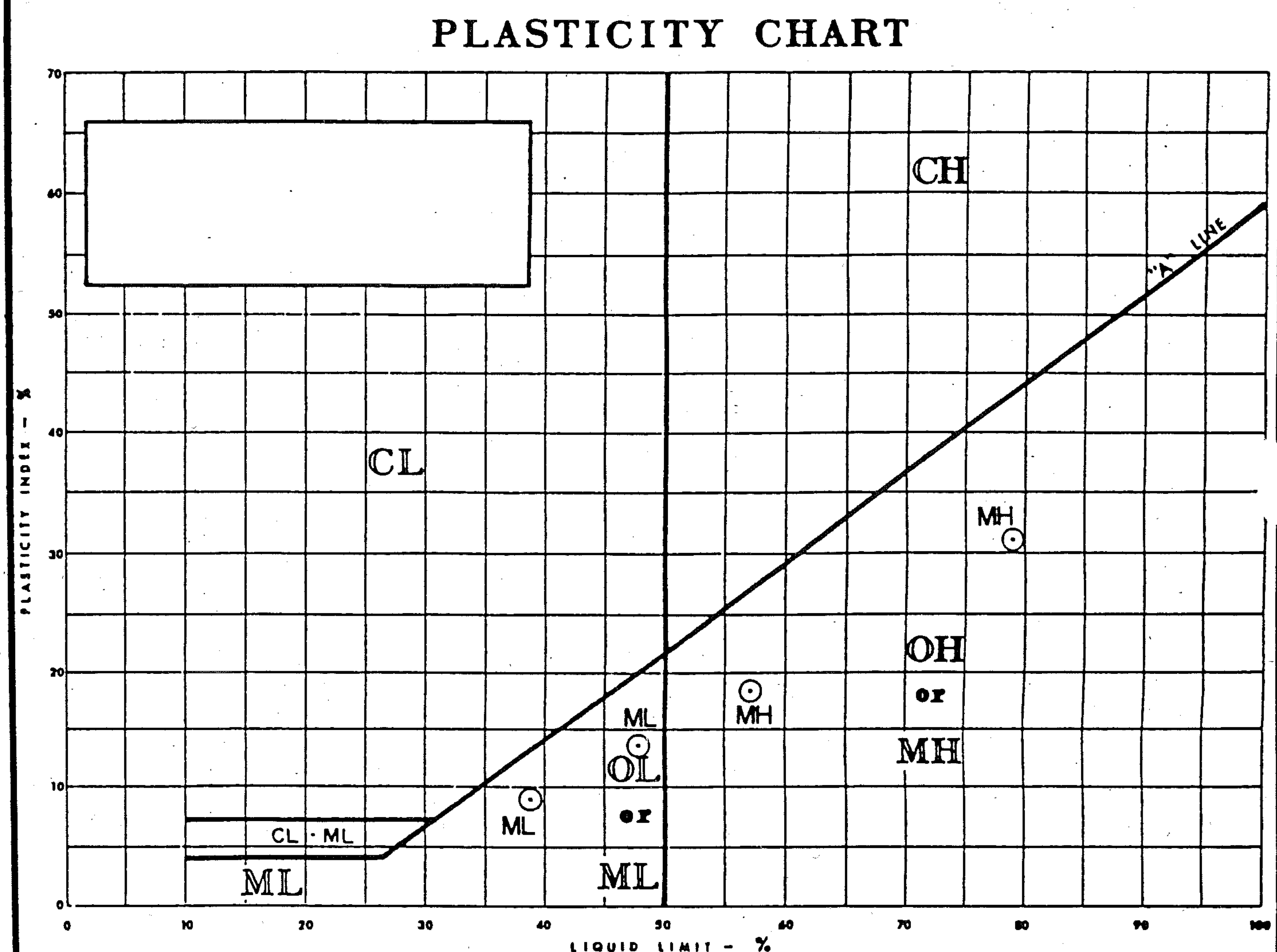

FIGURE 3-8
PLASTICITY OF RECENT DEPOSITS

DSWA PHASE IV LANDFILL CHERRY ISLAND FACILITY WILMINGTON, DELAWARE	ROY F. WESTON, INC. WESTON MANAGERS DESIGNERS/CONSULTANTS				
	DRAWN A. DELTUFFO	DATE 6/29/92	DES. ENG.	DATE	W. O. NO. 2477-03-01
	CHECKED		APPROVED		DWG. NO.

However, the shear strength of these soils was noted to be considerably higher than the overlying dredge spoils because of the effects of densification and strength gain due to a greater overburden. "N" values within these soils were WOR and WOH near the top of this substratum and generally increased to approximately 5 to 9 blows per foot near the bottom of the substratum. Pocket penetrometer, torvane and in situ vane shear tests were also indicative of higher shear strengths within these materials as compared to the overlying dredge spoils.

The soils of the underlying granular deposits generally consisted of coarse to fine sands and gravels with silt inclusions. These soils were noted to be of considerably higher shear strength than the overlying fine-grained soils as evidenced by "N" values which generally ranged between 20 and 60 blows per foot. These materials extended to the depth of exploration of the test borings completed within the landfill footprint.

The soils which comprise the Delaware River separation dike were investigated by deep borings TB-7 and TB-9. The dike soils are noted to be fill materials where placed above the original elevations of the riverbank deposits at this location. (These elevations are assumed to be approximately +4 to +10 feet based on an existing Delaware River water surface elevation of approximately +4 feet). These fill soils were noted to be fine-grained clayey silts comparable in composition to the dredge spoils. "N" values were typically in the range of 2 to 6 blows per foot. These soils were underlain by interbedded finer grained natural alluvial soils of varying composition, ranging from clayey silts to silty medium to fine sands. These finer-grained natural alluvial deposits extended to bottom elevations of - 49.4 feet in TB-7 and -52.4 feet in TB-9, at which elevations much stronger, more competent coarser grained soils were encountered. These deposits were typically of higher shear strength than the overlying fill materials as evidenced by "N" values which typically ranged from 7 to 11 blows per foot.

It should also be noted than an interbedded substratum of coarse to medium sand to coarse to fine gravel was encountered within both borings TB-7 and TB-9 at top elevations ranging from -0.9 to -6.9 feet, and bottom elevations ranging from -19.9 to -20.4 feet. These materials were noted to be higher in shear strength than the finer grained alluvial soils which both overlie and underlie these materials as evidenced by "N" values which typically ranged from 10 to 20 blows per foot.

Saturated soil conditions were encountered within the dredge spoils/recent deposits at depths ranging from 8 to 13 feet below existing ground surface in the borings which underlie the proposed landfill footprint. These depths correspond to elevations which range from approximately +35 to +40 feet. It is believed that this water is simply river water which was carried by pumping along with the dredged river bottom sediments to disposal within the diked Edgemoor area.

A water level elevation +9.6 feet was measured in the wellpoint of boring TB-7 on 5/1/92. This wellpoint reading is consistent with assumed riverbank elevations ranging between +4 and +10 feet before dike construction began as noted above.

3.6 LABORATORY TESTING PROGRAM

A laboratory physical properties testing program was completed by WESTON's Geotechnical Laboratory in Lionville, PA on selected representative split-spoon samples of the encountered coarse and fine-grained soil strata. The program included completion of the following tests:

1. Natural Moisture Content (ASTM D-2216)
2. Atterberg Limits (ASTM D-4318)
3. Grain Size Distribution by Mechanical Sieve Analysis (ASTM D-422)
4. Grain Size Distribution by Mechanical Sieve and Hydrometer Analysis (ASTM D-422)
5. Specific Gravity (ASTM D-854)
6. Organic Content (ASTM D-2974)

These test results were used to classify soils according to the Unified Soil Classification System (i.e. ASTM D-2487). They were also used to define stratigraphical continuity and serve as indices to soil behavior.

An engineering properties testing program was also completed on selected representative Shelby Tubes samples of the encountered fine-grained soil strata (i.e. dredge spoils, recent deposits, dike fill and underlying natural alluvial soils). These tests consisted of the following:

1. Triaxial Permeability Testing using DSWA Cherry Island landfill leachate as the permeant (EPA Method 9100): These tests were completed on Shelby Tube samples of the dredge spoils and recent deposits which directly underlie the proposed landfill footprint, and which will be evaluated as a natural soil liner for this facility.

2. Consolidation Testing (ASTM D-2435): These tests were completed on Shelby Tube samples of the dredge spoils and recent deposits which directly underlie the proposed landfill footprint, and which will therefore undergo settlement under the proposed landfill loadings.

3. Triaxial Shear Strength Testing: These tests were completed on Shelby Tube samples of both the dredge spoils/recent deposits which underlie the landfill footprint as well as the fine-grained dike fill and underlying natural alluvial soils.

Both the undrained and drained shear strength parameters of these soils were measured in Unconsolidated Undrained (UU) and Consolidated-Undrained (with pore pressure measurements) ($\overline{CIU}$) triaxial compression tests. (ASTM D-2850 and D-4767 respectively).

4. Compaction Testing: A Modified Proctor Compaction Test (ASTM D-1557) was completed on a bulk sample of the dredge spoils to determine the moisture/density relationship for these materials. These test results provide information regarding the potential use of excavated quantities of these materials as compacted fill to construct internal separation dikes within the landfill footprint.

5. Geochemical Analyses: These tests consisted of pH and Cation Exchange Capacity (CEC) and were completed on the dredge spoils to determine the contaminant attenuation capabilities of these materials if used as a natural soil liner. Column leaching tests were also performed using a flexible wall permeameter with influent and effluent leachate analyzed for TOC. All chemical testing followed standard EPA procedures.

The results of the laboratory testing program are presented in Appendix D of this report.

3.7 ENGINEERING ANALYSES

Detailed engineering analyses were also completed as part of the Phase IV landfill design. These included bearing capacity, settlement, natural soil liner permeability and geochemical analysis, and slope stability. The results of these analyses are presented as follows along with the technical assumptions utilized in the analyses. A complete set of engineering calculations and computer program input and output data which detail and document these analyses will be maintained in WESTON's project files.

3.7.1 Technical Assumptions

The following technical assumptions were used in completing the geotechnical analyses discussed below.

1. The entire Edgemoor disposal area which encompasses Phases III, IV and V will be landfilled as a single operating unit. That is, the combined area of Phases III, IV and V (approximately 165 acres) will be filled with a single lift of municipal solid waste before subsequent lifts are placed within this three Phase area in a similar manner. In addition, the same direction of fill placement will be used for each MSW lift so as to maximize the time interval between placement of two successive lifts of waste at any given point within the landfill footprint.

2. The initial rate at which the municipal solid waste will be placed within the landfill was assumed to be 350,000 tons/year for the first year of operation. An increase of 5% of the previous years fill rate was assumed for each successive year as reported to WESTON by DSWA.

3. Each lift of waste was assumed to be 10 feet in total thickness. This conservatively includes 9 feet of waste and one foot of daily soil cover (i.e. volumetric proportions of 90% waste/10% cover soil). (In reality, daily cover typically varies from 6-inches to 1-foot in thickness.) A total of 10 lifts of waste (i.e. 100 feet of landfill height) was evaluated.

4. The waste was assumed to be placed to a 3H:1V sloping configuration without benching around the perimeter of Phases III, IV, and V with the exception of the western end of Phases III and IV where an overfill against the existing sideslope of Phase II will be completed.

5. The initial placement density of the waste was assumed to be 43 pcf for calculation of the time intervals required to place the known volumes of the various waste lifts.

6. Based on assumptions 1 through 5 above, a total active landfill life of 29.8 years was calculated.

7. The density of the waste was significantly increased above the initial placement density of 43 pcf for use in completing bearing capacity, settlement and slope stability analyses for the following reasons:

   a. The wetting of the waste by infiltrating rainwater which will dramatically increase its weight. This phenomena is especially important for paper and yard wastes which readily absorb water and which constitute approximately 51% of a typical MSW fill.[7]

   b. Biodegradation of the organic constituents of the waste. This process creates a soft, weak, sludge-like mass which compresses under the weight of overlying materials, thereby densifying the mass in the process.

   c. Settlement of the non-biodegradable constituents of the waste mass under the weight of overlying lifts of this material (i.e. void ratio reduction) which results in a densification of this component of the mass.

   Based on a review of 16 references in which the in-place density of MSW was quoted, a representative value of 65 pcf was selected for this parameter. When weighted with the daily cover soil at an assumed field density of 120 pcf in 9 to 1 proportions, the density of the composite MSW/soil cover mass was determined to be 70 pcf for use in the various engineering analyses discussed subsequently.

3.7.2 Bearing Capacity Analysis

Undrained and drained bearing capacity analyses were completed to determine if the low shear strength dredge spoils and recent deposits can safely support the proposed landfill loadings. The Terzaghi bearing capacity equation was used to quantify this evaluation. These analyses are discussed as follows:

3.7.2.1 Undrained Analysis

The undrained bearing capacity analysis was completed for each 10 foot MSW lift thickness up to a total landfill height of 100 feet. Strength gain during the time interval between when a lift of waste was placed at a given location within the Phases III, IV and V footprint and a subsequent waste lift was placed at this same location was incorporated into the analysis as discussed subsequently. The Factor of Safety (FS) against a potential bearing capacity failure under undrained soil shear strength conditions for a given MSW lift was defined as follows:

$$\text{FS(undrained)} = q_{ult}/q_{actual} \tag{2}$$

In this equation, q_{ult} represents the ultimate bearing capacity of the dredge spoils/recent deposits assuming these soils have strength gained through consolidation from the placement of underlying lifts of waste. The value of q_{actual} is the total landfill loading including the weight of the given lift being evaluated.

Under undrained soil shear strength conditions, the values of undrained cohesion (c_u) and undrained angle of internal fraction (ϕ_u) are used in the analysis. For saturated or nearly saturated low permeability fine-grained soils, ϕ_u is zero. This was confirmed by the laboratory unconsolidated-undrained (UU) triaxial shear strength test results. For an applied loading at the ground surface of the supporting soils under a $\phi_u = 0°$ condition, the Terzaghi bearing capacity equation reduces to the following:

$$q_{ult} = c_u N_c s_c \tag{3}$$

This equation was used to determine the undrained bearing capacity of the dredge spoils/recent deposits at the time of initial placement of each of the 10 MSW lift loadings.

The undrained cohesive strength utilized in this analysis for each MSW lift was determined as follows:

$$c_u = c_{uo} + (\Delta\bar{\sigma}) \operatorname{Tan} \bar{\phi} \qquad (4)$$

in which:

c_u = the undrained cohesive shear strength of the fine-grained soil stratum under the new lift loading condition.

c_{uo} = the undrained cohesive shear strength of the fine-grained soil stratum under the previous lift loading condition.

$\Delta\bar{\sigma}$ = change in vertical effective stress which resulted from the previous lift loading condition.

$\bar{\phi}$ = effective angle of internal friction for the fine-grained soils which was conservatively assumed to be 30° based on $\overline{\text{CIU}}$ laboratory test results as well as correlations with soil physical properties.

The change in vertical effective stress ($\Delta\bar{\sigma}$) was calculated as the change in vertical total stress from the previous lift loading reduced by the average excess pore water pressure which exists within the fine-grained soil stratum thickness due to incomplete consolidation of these materials at the time the next lift loading is placed. This latter parameter ($u_e(t)$) was determined from the average degree of consolidation (U) of the stratum at this time as determined from the following expression:

$$u_e(t) = (1 - U)\, u_{ei} \qquad (5)$$

The value of initial excess pore water pressure (u_{ei}) was assumed to be equal to the lift loading immediately following placement of the lift. Conventional time rate of settlement analysis procedures were used to calculate the average degree of consolidation of the stratum for the time interval between placement of a given MSW lift and a subsequent MSW lift at this same location at any point within the landfill footprint. The value of c_v used in this analysis was .030 in^2/min. This value represents an average of the minimum c_v values as measured from the consolidation tests within the stress range represented by the landfill loadings. It is noted that the calculated average degree of consolidation achieved within the fine-grained soil stratum during the referenced time intervals pertinent to the 10 MSW lifts varied from 54.9% to 52.9%. It is evident from these figures that drained conditions within the fine-grained soil stratum never occurs during active filling of the landfill. Therefore, undrained bearing capacity governs the supporting capabilities of these fine-grained soils during the active life of the landfill.

Two figures were prepared which illustrate the increase in the undrained cohesion of the dredge spoils (Figure 3-9) and recent deposits (Figure 3-10) as a function of the 10 MSW waste lifts. The initial values of undrained cohesion shown on these plots (i.e. 175 psf for the dredge spoils and 540 psf for the recent deposits) represent the in situ average values of this parameter as measured by field and laboratory testing. The selection of these values from the field and laboratory data base is discussed in Section 3.7.5.3 of this report. Figure 3-11 was also prepared to illustrate the strength gain in both of these substrata as a function of time.

The N_c factor for $\phi_u = 0°$ is 5.7 based on the Terzaghi theory. This value assumes that the same undrained cohesive shear strength exists within the entire thickness of the fine-grained soil stratum. However, as discussed previously, both the field and laboratory data suggest that the surficial dredge spoils are significantly lower in shear strength than the underlying recent deposits. Therefore, a procedure by Reddy et al[8] was used which allows this strength differential between substrata to be quantitatively incorporated into the bearing capacity analysis through determination of an N_c' factor which is substituted for N_c in the undrained bearing capacity equation. For the case of a higher cohesive shear strength soil underlying a surficial soil of lower cohesive shear strength, the N_c' factor is greater than N_c thereby increasing the value of q_{ult}. This analysis yielded N_c' factors which ranged from 9.8 for the first lift of waste to 5.8 for the tenth and final lift of waste.

The s_c factor in the Terzaghi bearing capacity equation is a shape factor which is determined from the geometry of the applied loading. This parameter can range between 1.0 and 1.3. A conservative value of 1.0 was selected for use in the undrained bearing capacity analysis.

The results of the undrained bearing capacity analysis are presented as Figure 3-12. Note from this figure that the minimum calculated Factor of Safety value is 1.78 for the tenth and final lift of waste. This value is greater than the minimum acceptable Factor of Safety of 1.5 for dams, fills and embankment as recommended in the geotechnical literature[9].

3.7.2.2 Drained Analysis

As discussed above, the undrained bearing capacity of the fine-grained soil stratum governs the supporting capabilities of these soils as the landfill is being raised to its final design height. Approximately 22.4 years following completion of the tenth and final MSW lift, the fine-grained soil stratum will have fully dissipated excess pore water pressures induced by the landfill loadings. At and beyond this time, the shear strength and bearing capacity

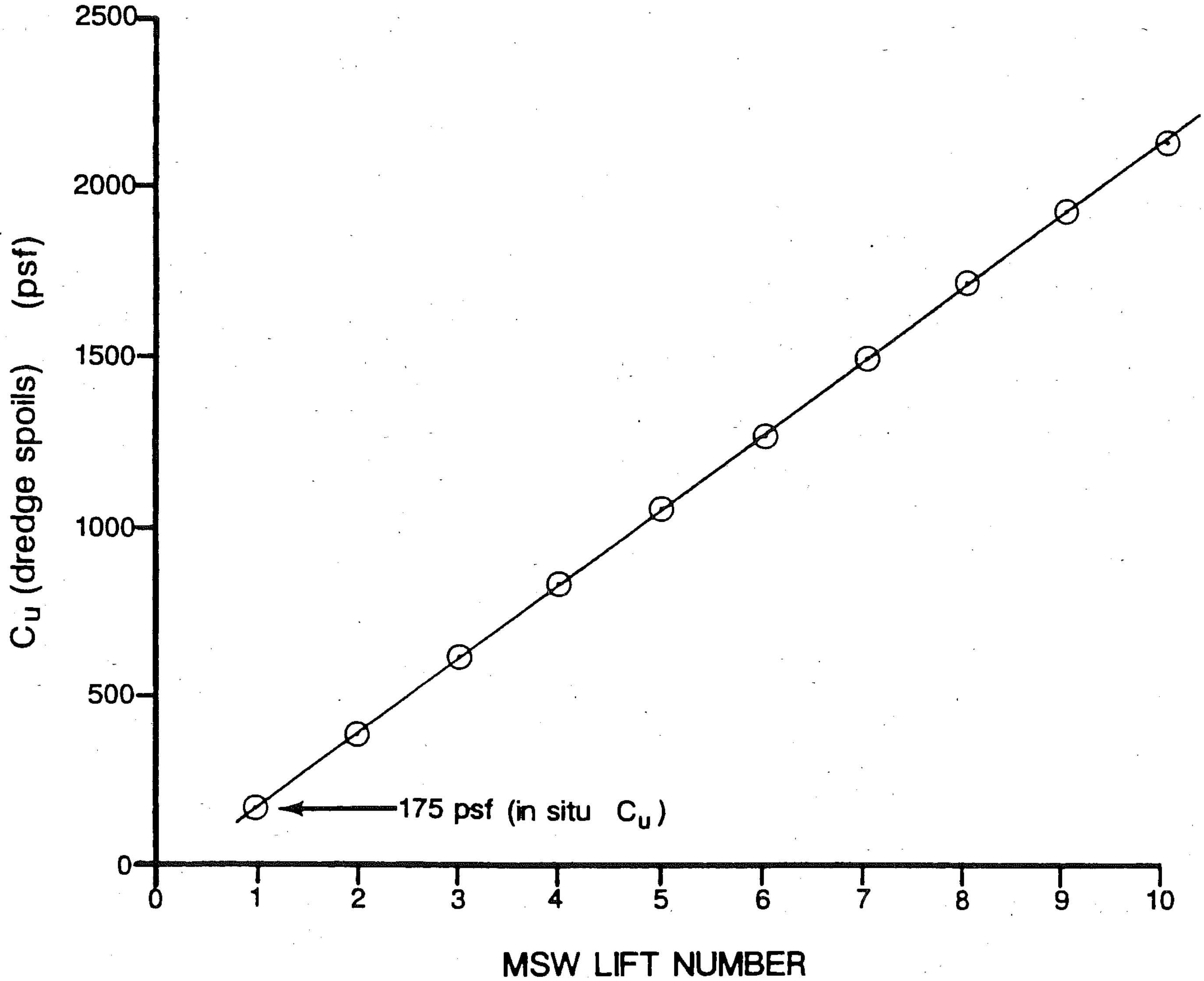

FIGURE 3-9
STRENGTH GAIN OF DREDGE SPOILS
AS A FUNCTION OF MSW LIFT NUMBER

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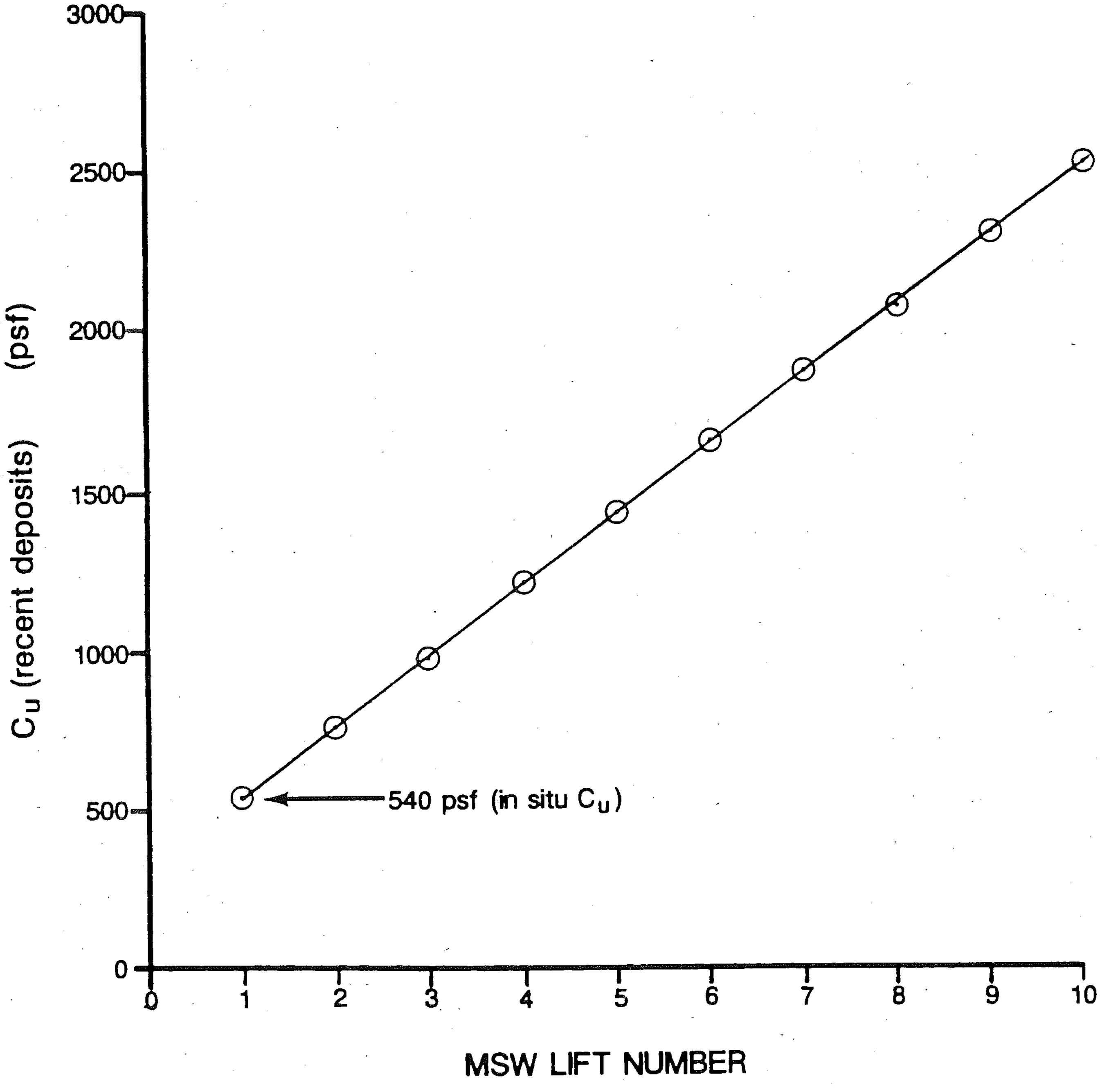

FIGURE 3-10
STRENGTH GAIN OF RECENT DEPOSITS
AS A FUNCTION OF MSW LIFT NUMBER

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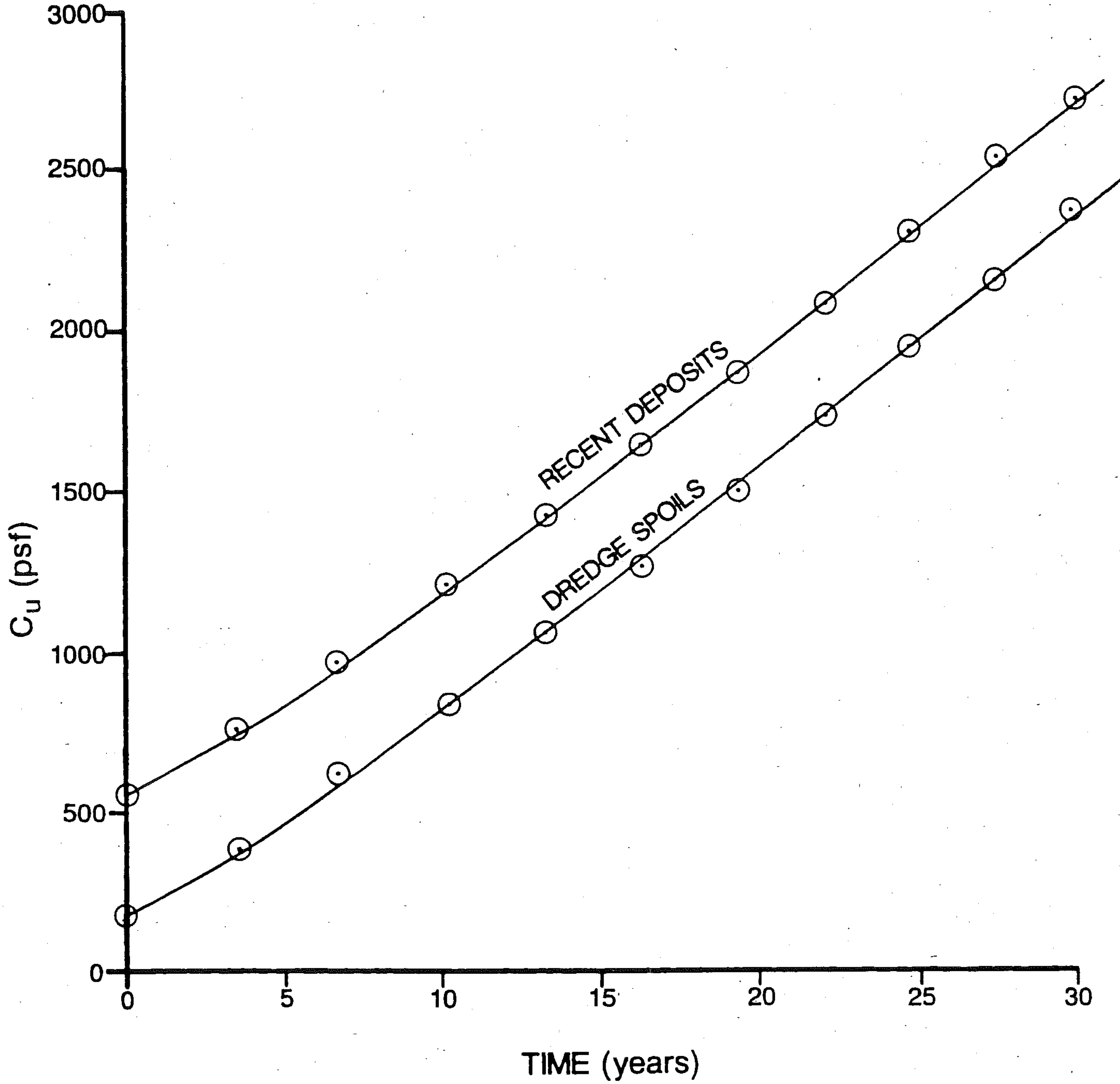

FIGURE 3-11
STRENGTH GAIN OF DREDGE SPOILS AND RECENT DEPOSITS AS A FUNCTION OF TIME

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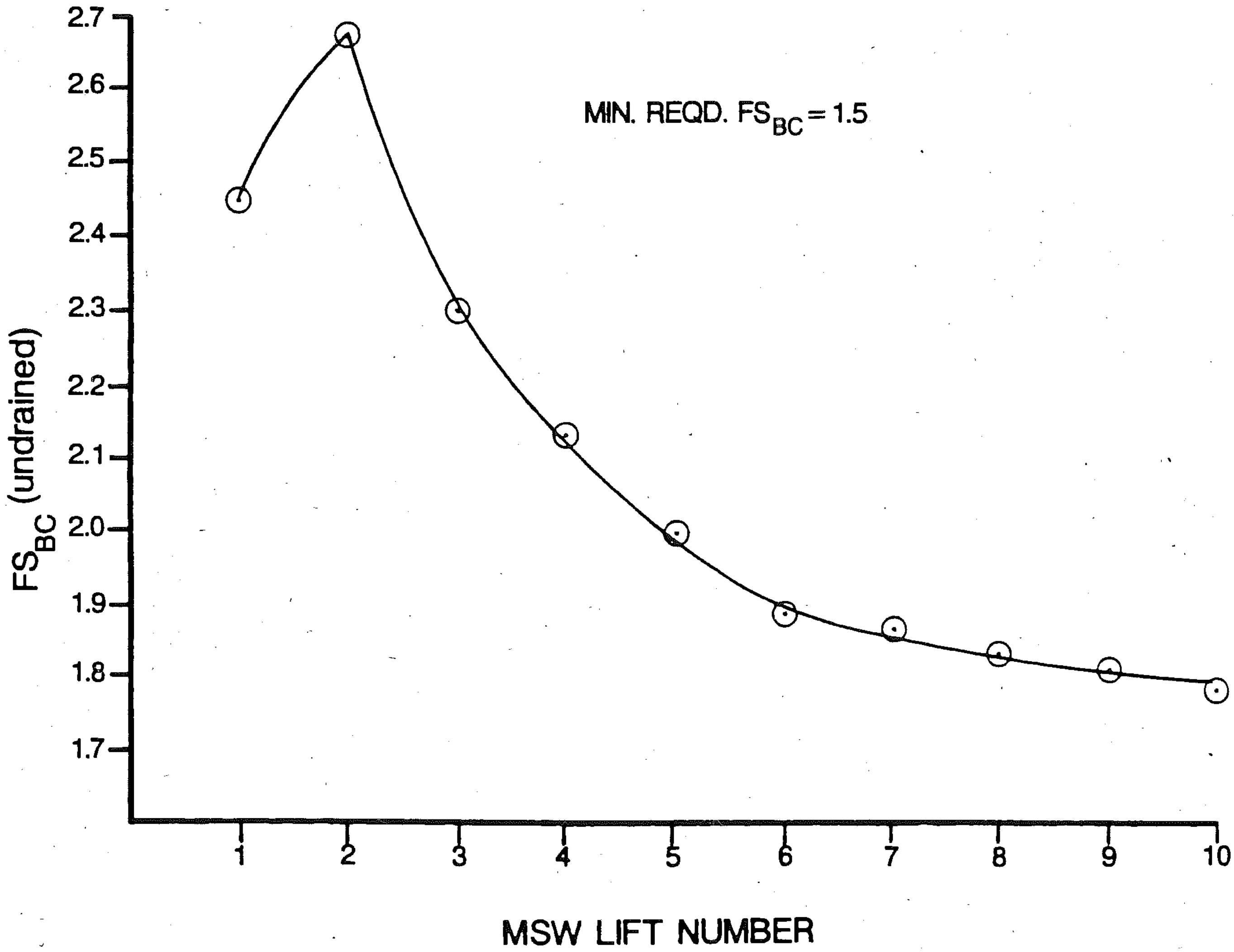

FIGURE 3-12
RESULTS OF UNDRAINED
BEARING CAPACITY ANALYSIS

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of these soils will be governed by the drained soil shear strength parameters of these materials. Therefore, a drained bearing capacity analysis was completed to evaluate the long-term capabilities of these soils to support the finished landfill loadings. The factor of safety (FS) against a potential bearing capacity failure under drained soil shear strength conditions was defined as follows:

$$FS = q_{ult}/q_{actual} \tag{6}$$

In this equation, q_{ult} is the ultimate bearing capacity of the soil as determined using drained soil shear strength parameters while q_{actual} is the landfill loading under the full finished landfill height condition.

Under drained soil shear strength conditions, the effective stress shear strength parameters ($\overline{c}$, $\overline{\phi}$) of the supporting soils are used in the analysis. These values were assumed to be $\overline{c}$ = 134 psf and $\overline{\phi}$ = 30° based on $\overline{\text{CIU}}$ triaxial shear strength testing of Shelby Tube samples of these materials. These values were then conservatively reduced using the following expressions:

$$\overline{c}_L = 2/3\,\overline{c} \tag{7}$$
$$\overline{\phi}_L = \text{Tan}^{-1}\,(2/3\ \text{Tan}\,\overline{\phi}) \tag{8}$$

This reduction accounts for the probability that local, rather than general shear failure conditions (i.e. considerable vertical soil movement occurs before edge bulging) would most likely develop in these soils in the event of a bearing capacity failure.

Under drained soil shear strength conditions, the Terzaghi bearing capacity equation appropriate to an applied loading at the ground surface of the supporting soils becomes:

$$q_{ult} = \overline{c}\,N_{cL}\,s_c + \tfrac{1}{2}\,\gamma B N_{\gamma L}\,s_\gamma\,r_\gamma \tag{9}$$

The bearing capacity factors N_{cL} and $N_{\gamma L}$ are a function of $\overline{\phi}$ and were determined from tabulated values of these parameters pertinent to the local shear failure assumption. The loading shape factors, s_c and s_γ, can vary between 1.0 and 1.3 and 0.8 and 1.0 respectively, depending on the length to width ratio of the loading geometry. Conservative values of s_c = 1.0 and s_γ = 0.8 were used in the analysis. The least lateral dimension of the landfill footprint (B) was assumed to be 2,700 feet. This value was reduced by the r_γ factor appropriate for use in the case of large loading footprints. The value of this factor was calculated to be .337. Finally, a soil unit weight of 85 pcf as determined from the results of the consolidation tests was used in this calculation.

Based on this analysis, a factor of safety of 9.0 against long-term bearing capacity failure under drained soil shear strength conditions for the finished landfill loading was determined. This value is significantly greater than the recommended minimum value of 3.0 as determined by conventional geotechnical engineering practice.

3.7.3 Settlement Analysis

A settlement analysis was completed to estimate total and differential settlements which will occur within the fine-grained dredge spoils/recent deposits which underlie the landfill footprint as a result of the landfill loadings. The total settlement (ΔH_{TOT}) within these materials was calculated from the following general settlement equation:

$$\Delta H_{TOT} = \Delta H_{uc} + \Delta H_{pc} + \Delta H_{sc} \qquad (10)$$

in which:

ΔH_{uc} = settlement resulting from the underconsolidated condition of the dredge spoils/recent deposits at the present time.
ΔH_{pc} = settlement resulting from the landfill loadings (i.e. primary consolidation).
ΔH_{sc} = settlement resulting from secondary compression within these substrata.

A total of four (4) consolidation tests were completed to define the input parameters for the settlement analysis. The complete data sets from these consolidation tests are presented in Appendix D.

The stratigraphic profile used in the settlement analysis to calculate ΔH_{uc} and ΔH_{pc} is presented as Figure 3-13. Note that the 80 foot fine-grained soil stratum thickness was subdivided into three substrata for this analysis. Within each of these substrata, the soil parameters which influence settlement were assumed to be constant and numerically equal to those values determined from the consolidation test completed within the respective substratum thickness. Based on a comparison of the preconsolidation pressure, as determined by the Casagrande graphical construction completed on each of the three laboratory e/log p curves, with the value of vertical effective overburden pressure at the Shelby Tube depth, all three substrata were determined to be slightly to moderately underconsolidated. Appropriate procedures for underconsolidated soils as shown in Figure 3-14 were then used to construct the field e/log p curves from the laboratory e/log p curves. The field compression index (C_{cf}) was subsequently determined as the slope of the virgin compression limb of these curves. The settlement due to combined underconsolidation and

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▼ - Depth to saturated dredge spoils. (This varied from 8 to 13 feet and was conservatively assumed to be 8 feet.)

γ_{sat} - Saturated unit weight values as determined from the consolidation test data.

e_o - Initial void ratio values as determined from the consolidation test data.

C_{cf} - Compression index values as determined from the field e/log p curve as graphically constructed from the laboratory e/log p curve using appropriate procedures for underconsolidated soils (i.e. soils in which $P_c < \bar{\sigma}_o$).

H - Thickness of compressible soil stratum. (This includes entire thickness of dredge spoils and recent deposits strata, and varies from 75 to 80 feet and was conservatively assumed to be 80 feet.)

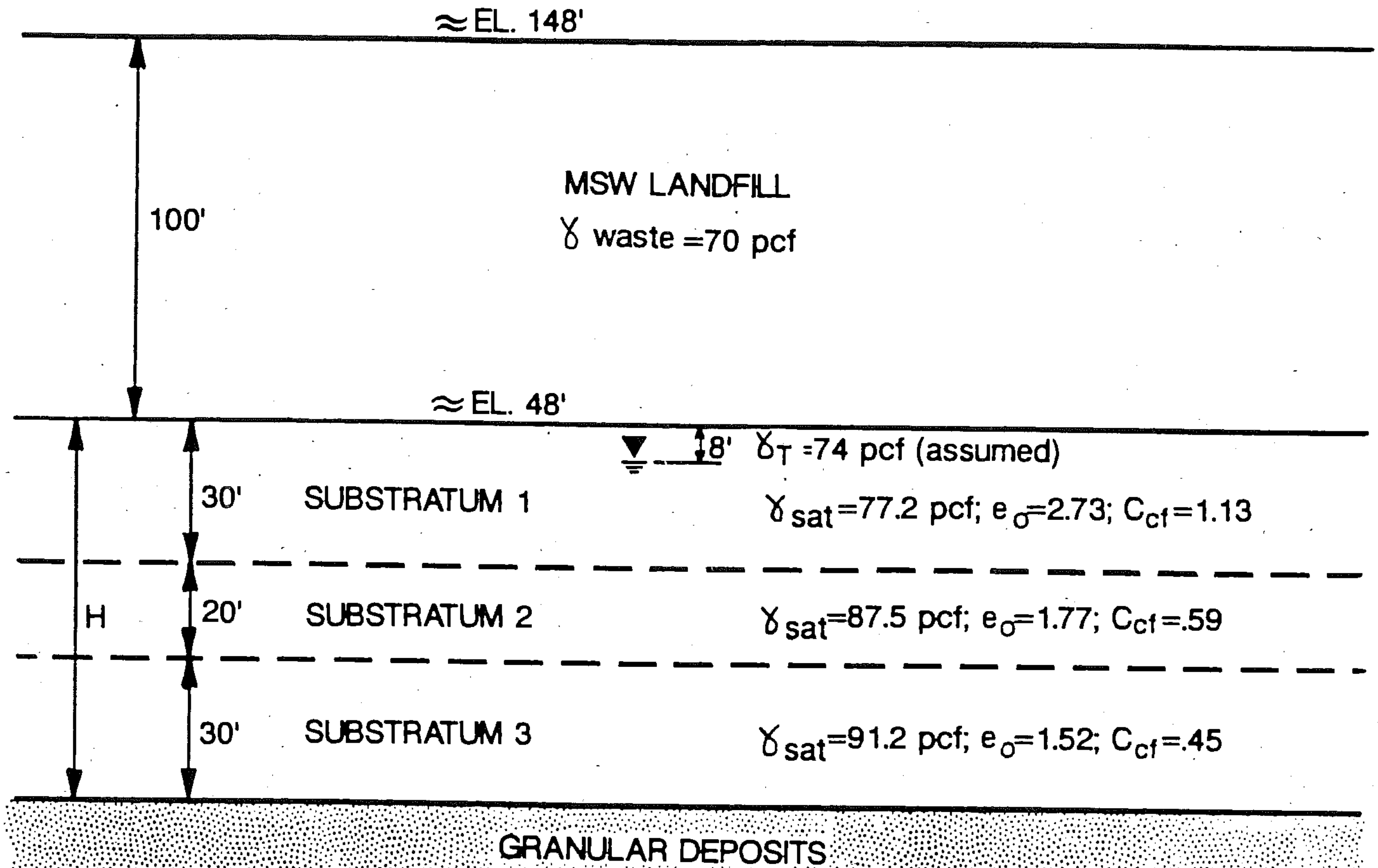

FIGURE 3-13
ASSUMED STRATIGRAPHIC PROFILE
FOR SETTLEMENT ANALYSIS

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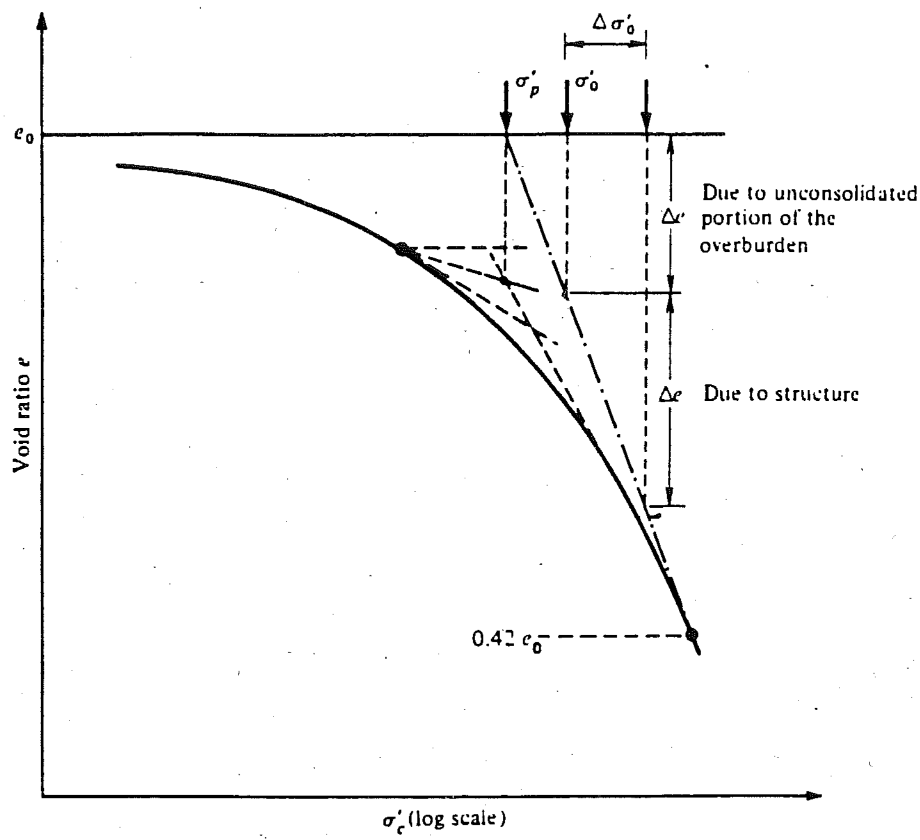

FIGURE 3-14
COMPRESSIBILITY OF
UNDERCONSOLIDATED COHESIVE SOIL

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primary consolidation effects resulting from the assumed 100 foot high landfill loading was then determined for each of the substrata from the following expression.

$$\Delta H_{uc} + \Delta H_{pc} = \frac{HC_{cf}}{1+e_o} \log \left(\frac{\bar{\sigma}_o + \Delta\sigma}{p_c}\right) \tag{11}$$

This analysis yielded a total settlement due to underconsolidation and primary consolidation effects of 17.9 feet.

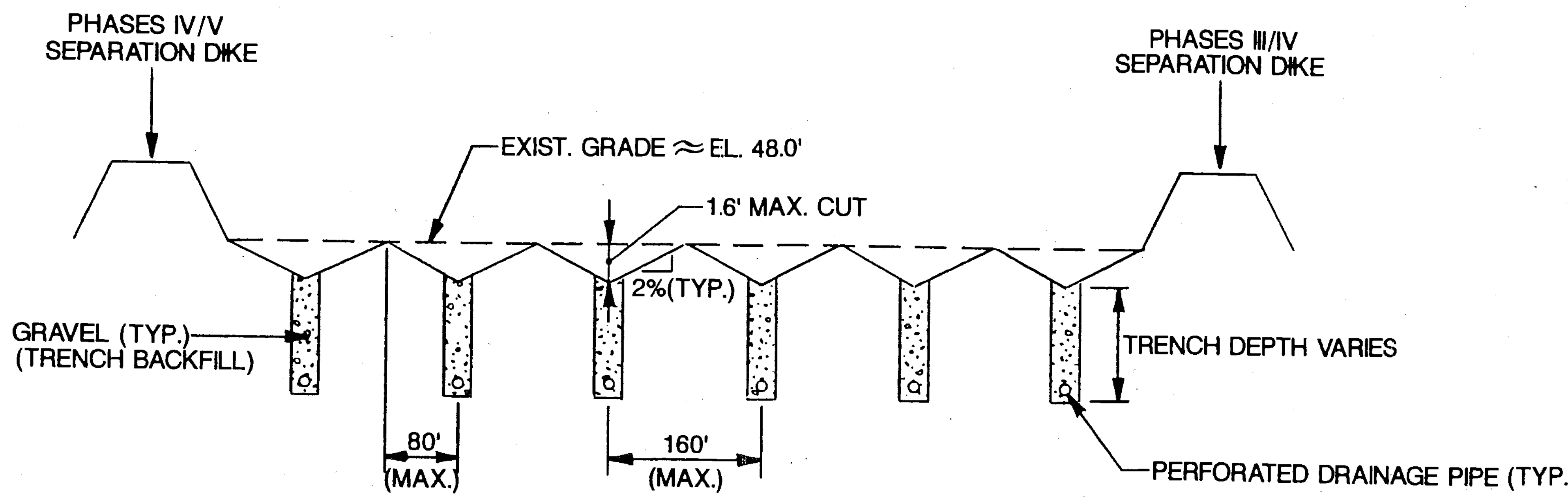
Secondary compression was also computed for these fine-grained soils using the following expression:

$$\Delta H_{sc} = \frac{HC_{\alpha}}{1+e_p} \log (t_f/t_i) \tag{12}$$

The value of C_{α} (i.e. coefficient of secondary compression) was determined from the results of the consolidation tests using the Dial Reading/Log time plots for the stress range pertinent to the landfill construction. This value was determined to be .0267. The time interval over which secondary compression was assumed to occur was from 1 month (t_i) to 62.8 years (t_f). This latter value represents the time at which the 30 year post-closure landfill monitoring period is assumed to end, and includes 29.8 years of active life, an assumed 3 year time period to cap the three phase landfill, and 30 years for post-closure monitoring. Based on this analysis, a total of 1.9 feet of secondary compression settlement has been estimated over the referenced time interval.

Based on the above discussions, 19.8 feet of total settlement is anticipated from the proposed landfill construction. To set landfill floor and leachate collection pipe grades, it is necessary to predict what percentage of this total settlement can be differential in nature. A procedure by Bjerrum as documented in Lambe and Whitman[10] was used for this purpose. Based on this procedure, it has been determined that the maximum potential differential settlement at the site is 8 feet. Based on this value, the following design criteria have been established to compensate for the differential settlement potential of the site to the extent possible:

1. As illustrated in Figure 3-15, a sawtooth configuration consisting of 6 depressions will be constructed across the width of the Phase IV landfill footprint. These depressions will be sloped at 2% grades (i.e. DNREC controlling slope criterion) from highpoints near existing site grades (approximately elevation +48 feet) to lowpoints at which leachate collection pipes (laterals) will be located. These leachate collection laterals will be constructed within gravel filled trenches which will increase in depth as the pipes traverse the cell to their perimeter endpoints. This 6-depression sawtooth



DESIGN "SAWTOOTH" CONFIGURATION
OF LANDFILL BOTTOM

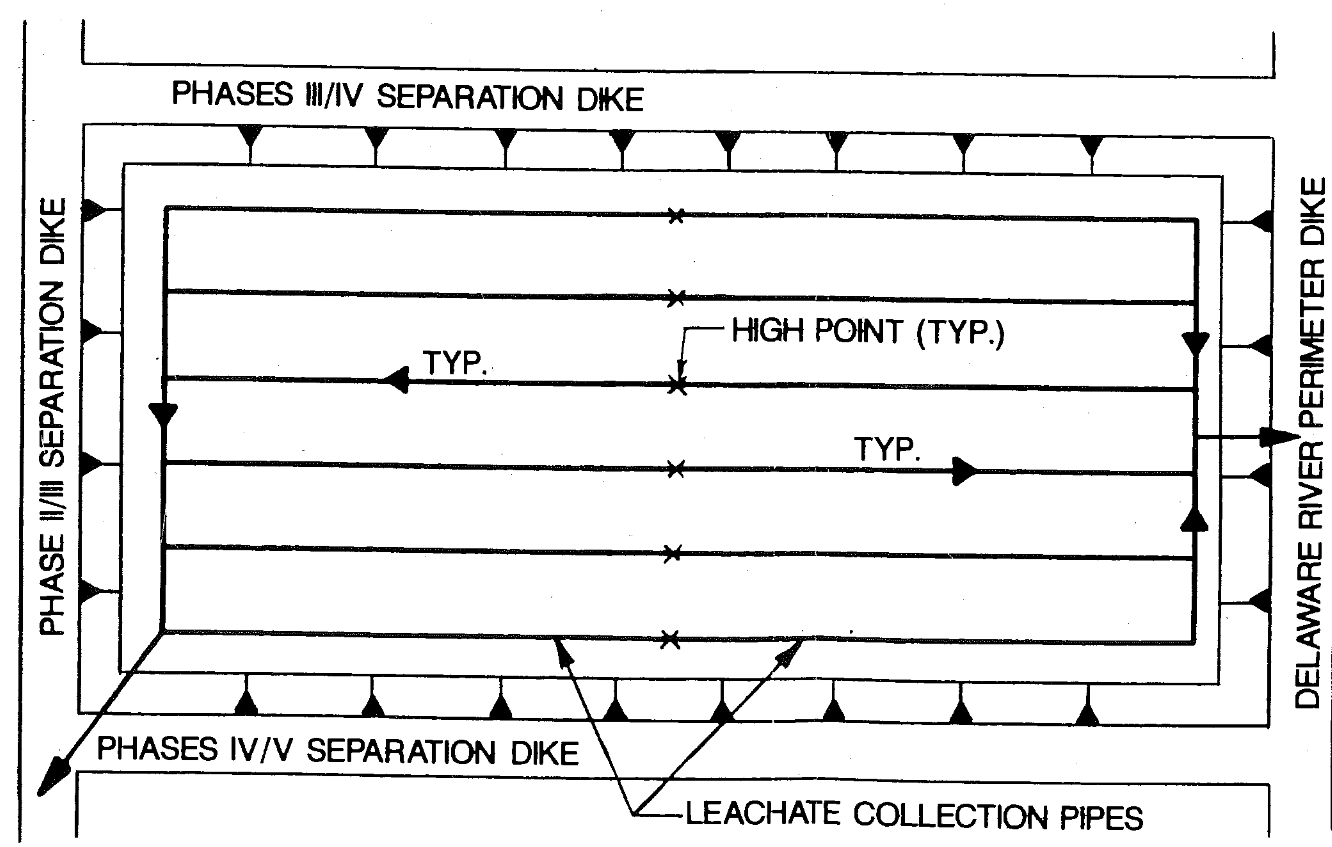


FIGURE 3-15
PHASE IV LEACHATE COLLECTION SYSTEM:
PLAN AND DETAILS

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configuration will result in a maximum spacing of 80 feet between a sawtooth highpoint and lowpoint as shown on Figure 3-15. This pipe spacing is much less than conventional practice would dictate for the case of stable subgrade conditions, and is believed necessary to provide more lowpoint pipe runs to which leachate can flow in the event that differential settlement significantly alters the original design configuration of the landfill floor.

2. As also illustrated in Figure 3-15, the Phase IV landfill footprint will drain to sumps in two directions (east and west). This will allow collected leachate to be removed from the cells on both the eastern and western sides of the site by a system of sloping leachate collection laterals. This will reduce the required length of these sloping laterals. This in turn will allow this piping to be set at steeper inclinations as necessary to accommodate the differential settlement potential of the site, while, at the same time, will result in constructable trench depths at the lateral endpoints.

3. As illustrated in Figure 3-16, the leachate collection laterals will be set at a minimum longitudinal inclination of 1% at locations behind the top of slope of the 3H:1V face of the landfill. In the event that 8 feet of differential settlement occurs between the high point and lowpoint of this pipe run, the initial 1% design longtidual grade will be reduced to a 0.2% grade, which is still sufficient to transmit leachate to the lowpoint of this run. At the top of slope of the 3H:1V landfill face, the piping inclination will be increased to a minimum of 3% to accommodate a greater differential settlement potential at these locations where the landfill height varies.

4. To facilitate effective leachate flow from the floor of the landfill to the collection pipes, a 12-inch thick granular soil drainage blanket of at least 1 x 10^{-2} cm/sec permeability will be placed atop the entire Phase IV footprint. (This granular soil layer is also extremely important to the design from the perspective of natural soil liner permeability as discussed in Section 3.7.4.1 of this report.) To reduce the probability of this granular soil layer becoming discontinuous with time due to differential settlement, a woven geotextile will initially be placed over the graded landfill bottom before placing the overlying granular soil. This geotextile will also help to reinforce the underlying weak dredge spoils, thereby reducing the probability of bearing capacity and/or slope failures occurring along the leading edge of the waste filling operations. The geotextile will also tend to somewhat equalize total settlements, thereby reducing the potential for significant differential settlements occurring. It will also prevent the granular soil particles from being driven into and

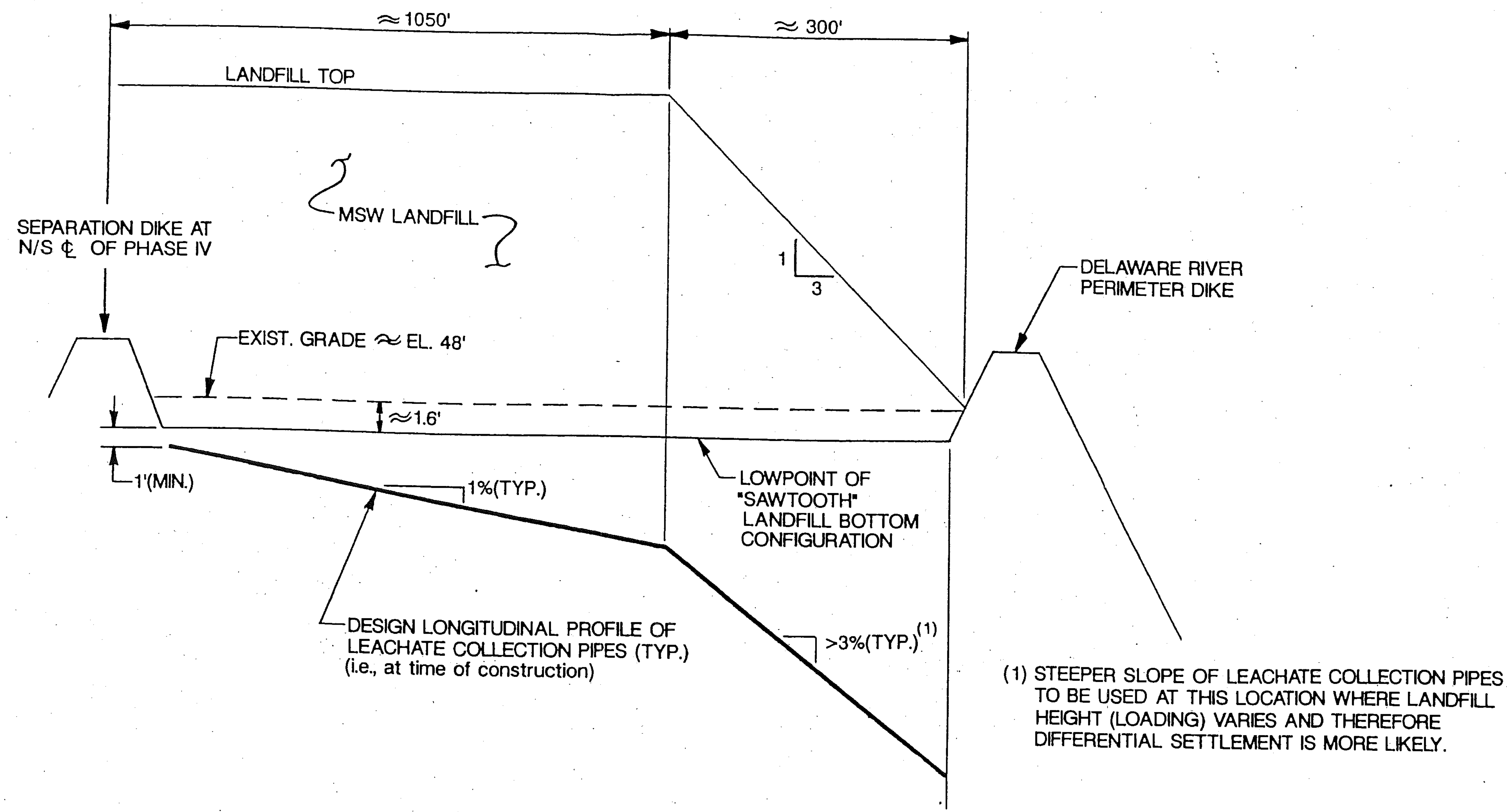

FIGURE 3-16
PHASE IV LEACHATE COLLECTION SYSTEM: LONGITUDINAL PROFILE OF LEACHATE COLLECTION PIPES

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mixed with the underlying dredge spoils by construction equipment loads and/or future overlying waste overburden, thereby increasing the probability of maintaining the continuous nature and design thickness of this layer. Finally, its reinforcement capabilities will also facilitate more efficient fill placement across the landfill footprint during filling of the first lift of waste.

3.7.4 Natural Soil Liner

The approximate 80 foot thickness of the dredge spoils/recent deposits stratum is proposed for use as a natural soil liner for leachate containment at this site. This decision will be impacted by the permeability and geochemical properties of these materials as discussed below.

3.7.4.1 Permeability

It is proposed to construct the Phase IV landfill using the fine-grained site soils (dredge spoils/recent deposits) as a natural soil liner. The DNREC MSW regulations permit this design if there is at least 5 feet of thickness of these materials within which the vertical permeability is less than or equal to 1 x 10^{-7} cm/sec. To investigate this, WESTON completed a series of four (4) flexible wall permeability tests on four undisturbed Shelby Tube samples of these soils obtained from borings which underlie the proposed landfill footprint. These Shelby Tubes were selected at varying depths within the fine-grained stratum thickness, ranging from 14 feet to 62 feet below the existing ground surface. The permeability tests were completed using MSW landfill leachate from the active cell of the Cherry Island landfill as the permeant. The procedures of EPA Test Method 9100, "Saturated Hydraulic Conductivity, Saturated Leachate Conductivity and Intrinsic Permeability" were used to complete these tests.

The results of the permeability testing program are presented in Table 3-1. The complete laboratory data package is presented in Appendix D. The value of $\bar{\sigma}_{3i}$ shown in this Table is the initial confining pressure under which the sample was fully consolidated before the equilibrium permeability value ($K_{\bar{\sigma}3i}$) was measured. The value of $\bar{\sigma}_{3i}$ was calculated as the horizontal effective stress at the middepth of the respective Shelby Tube under the existing overburden conditions. It is therefore evident that the value of $K_{\bar{\sigma}3i}$ represents the in situ permeability of the fine-grained soils at the Shelby Tube depth before the proposed landfill loadings are applied.

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TABLE 3-1

PERMEABILITY TEST RESULTS

TEST NO.	BORING NO.	SHELBY TUBE DEPTH (feet)	SUBSTRATUM(1)	$\bar{\sigma}_{3i}$ (psi)	$K_{\bar{\sigma}3i}$ (cm/sec)	$\bar{\sigma}_{3f}$ (psi)	$K_{\bar{\sigma}3f}$ (cm/sec)
1	TB-1	13-15	Dredge Spoils	4.3	2.21×10^{-7}	25.5	3.36×10^{-8}
2	TB-3	27.5-29.5	Dredge Spoils	6.6	2.85×10^{-7}	28.0	5.07×10^{-8}
3	TB-4	42-44	Recent Deposits	8.8	3.80×10^{-8}	30.1	1.38×10^{-8}
4	TB-4	61-63	Recent Deposits	11.9	2.32×10^{-8}	33.2	1.20×10^{-8}

(1) Assumes that the top 40 feet of the fine-grained soil stratigraphy beneath the landfill footprint is dredge spoils with deeper materials consisting of recent deposits.

The value of $\overline{\sigma}_{3f}$ shown in Table 3-1 is the final confining pressure under which the sample was fully consolidated before the equilibrium permeability value ($K_{\overline{\sigma}3f}$) was measured. The value of $\overline{\sigma}_{3f}$ was calculated as the horizontal effective stress at the middepth of the respective Shelby Tube under the final overburden conditions (i.e. with the full landfill height in place). It is therefore evident that the value of $K_{\overline{\sigma}3f}$ represents the permeability of the fine-grained soils after the full landfill loading has been applied and the material has fully consolidated and therefore densified in response to this loading.

An inspection of the results of Table 3-1 indicates the following:

1. The vertical permeability of the dredge spoils under the existing overburden conditions is approximately 2 to 3 times greater than the maximum regulated value of 1×10^{-7} cm/sec.

2. The vertical permeability of the recent deposits under the existing overburden conditions is less than the maximum regulated value of 1×10^{-7} cm/sec.

3. The vertical permeability of both the dredge spoils and the recent deposits under the final overburden conditions (i.e. full landfill height in place) is less than the maximum regulated value of 1×10^{-7} cm/sec.

Based on WESTON's review of the DNREC regulations, it is our interpretation that the term "natural liner" is meant to connote low permeability soils which directly underlie the leachate collection system of the landfill. Based on this definition, it is possible that the regulatory agencies may consider the dredge spoils alone and not the combined thickness of the dredge spoils and the geotechnically similar recent deposits as the "natural liner" for regulatory purposes. Based on this interpretation, the dredge spoils do not initially satisfy the "natural liner" requirements of the DNREC regulations since they do not possess a permeability of less than 1×10^{-7} cm/sec in their existing density condition. However, as discussed previously in the bearing capacity section of this report, at no time during active filling of the landfill will drained conditions (i.e. hydrostatic pore water pressure conditions) be attained within the dredge spoils/recent deposits stratum thickness. Instead, excess pore water pressures generated from the various waste lift loadings will exist within these fine-grained soils at all times until approximately 22.4 years after the tenth and final waste lift has been placed. Only after this time will excess pore water pressures induced by the landfill loadings be fully dissipated with hydrostatic pore water pressures re-established in the saturated zone of the fine-grained soil stratum. The average excess pore water pressure (u_e) within the dredge spoils/recent deposits stratum thickness is tabulated as a function of both MSW lift number and time in Table 3-2. These excess pore water pressures will induce flow to drainage boundaries as the stratum consolidates under the landfill loadings. In this regard, as discussed previously, the WESTON design will incorporate a 12-inch thick granular soil drainage blanket of at least 1×10^{-2} cm/sec permeability over the entire

TABLE 3-2

CALCULATED AVERAGE EXCESS PORE WATER PRESSURES

LIFT #	TIME TO COMPLETE (YEARS)	U(%)	u_e(psf)	h_L(ft)
1	3.46	54.9	315	4.77
2	3.36	54.8	316	4.79
3	3.25	54.7	317	4.80
4	3.14	54.6	318	4.82
5	3.04	54.5	319	4.83
6	2.93	54.2	321	4.86
7	2.81	53.9	323	4.89
8	2.71	53.7	324	4.91
9	2.59	53.4	326	4.94
10	2.47	----	----	----

U: Average degree of consolidation achieved within the 80 foot thickness of fine-grained soils at the time at which the subject lift is completed.

u_e: Average excess pore water pressure which exists within the fine-grained soils at the time at which the subject lift is completed (calculated from equation 5).

h_L: Leachate head which is equivalent to u_e assuming the unit weight of leachate is 66 pcf.

Phase IV landfill footprint. In addition to facilitating effective flow of leachate to the leachate collection piping system, this drainage blanket also represents a drainage medium atop the dredge spoils which will readily accept pore water which is being driven vertically upward from the upper half of the consolidating fine-grained soil stratum thickness. Therefore, upward flow of water from the dredge spoils into the drainage blanket will be induced by the consolidation process. A piezometer monitoring system will be installed as part of the Phase IV landfill construction to monitor excess pore water pressure in the dredge spoils.

The average excess pore water pressures within the consolidating fine-grained soil stratum at the time at which a given lift of waste is completed have also been converted to an equivalent head of leachate (h_L) as shown in Table 3-2 using the expression.

$$h_L = u_e/\gamma_L \tag{13}$$

In this equation, the unit weight of leachate (γ_L) was assumed to be 66 pcf, that is, slightly greater than water to account for particulates in the liquid. As noted in this Table, the equivalent upward leachate head is at least 4.77 feet during active filling of the landfill. In addition, the selected landfill bottom slope (i.e. 2%) and leachate collection system piping configuration and spacing will limit the maximum leachate head buildup to 7.5 inches based on an equation determined by Giroud[11]. (Note that this head buildup is less than the design thickness of the granular soil drainage blanket so as to permit effective flow of leachate to collection pipes.) Therefore, the pressure head with which the leachate is being driven vertically downward is, at all times during active filling of the landfill, significantly less than the pressure head of the pore water which is being driven vertically upward from the upper half of the consolidating fine-grained soil stratum. Therefore, downward vertical migration of leachate cannot occur during active filling of the landfill. Even in the instance in which differential settlement may reduce the "sawtooth" surface slope from 2% to a flat condition (i.e. 0%), the maximum estimated leachate head buildup would be 1.5 feet. This value is still significantly less than the minimum upward pore water pressure head (4.77 feet) induced by the consolidating soil stratum, thereby also precluding downward vertical migration of leachate under this condition. Only after the final landfill height has been achieved and all excess pore water pressures have been fully dissipated can downward vertical migration of leachate occur. At this time, as shown in Table 3-1, the entire 80 foot dredge spoils/recent deposits stratum thickness has achieved a permeability of less than 1×10^{-7} cm/sec. Therefore, upon completion of active filling of the Phase IV landfill only after which downward vertical migration of leachate can begin, the 80 foot thickness of the fine-grained soil stratum at this site is providing 16 times the thickness of natural soil liner required by the DNREC regulations (i.e. 5 feet).

It is also important to note that the natural alluvial recent deposits which underlie and directly contact the dredge spoils, and which are "geotechnically similar" to these materials in their physical properties, possess a permeability of less than $1x10^{-7}$ cm/sec under existing overburden conditions. This is believed to be a direct result of the recent deposits having achieved a greater density condition with time from both a longer geologic history at this site as well as a greater existing overburden pressure as compared to the more recent overlying dredge spoils. The existing low permeability of this approximately 40 foot thick recent deposits substratum represents an added environmental safeguard against potential groundwater contamination of the underlying granular deposits during active filling of the landfill.

The effects of permeability reduction as a function of increasing landfill height within the near surface dredge spoils was also investigated. In this regard, a fifth flexible wall permeability test was completed on a Shelby Tube sample of the dredge spoils obtained from Boring TB-2 at a depth of 13.5 to 15.5 feet below existing ground surface. This test determined the equilibrium vertical permeability value of these soils when fully consolidated under confining pressures which correspond to 20, 40, 60 and 80 feet of landfill height using Cherry Island MSW leachate as the permeant. These four data points were subsequently compared to a fifth permeability value determined from an undisturbed dredge spoils sample obtained from Boring TB-1 at a similar depth of 13 to 15 feet below ground surface. This sample was initially consolidated to a confining pressure which corresponds to the in situ overburden condition at this depth, and therefore models the case of no landfill loading. These five data points were subsequently plotted as shown in Figure 3-17. As is evident from this Figure, a permeability of significantly less than $1x10^{-7}$ cm/sec is achieved in the near surface dredge spoils following partial consolidation of the fine-grained soil stratum at a very early stage during the active life of the landfill. This is well before the time period necessary to complete active filling of the landfill is complete, during which, as noted above, vertical downward migration of leachate cannot occur.

3.7.4.2 Geochemical Properties of the Dredge Spoils/Recent Deposits in Support of the Subtitle D Approval Process

The results of the permeability tests presented in the previous subsection indicate that the proposed natural soil liner consisting of the dredge spoils/recent deposits meets the current DNREC Solid Waste Regulations (March 1990). Initially, prior to waste placement, the dredge spoils possess a permeability which is slightly greater than 1×10^{-7} cm/sec. However, once solid waste is placed atop these materials, an upward vertical head of water due to excess pore water pressures will be created as the soils consolidate under the landfill loadings. As landfilling continues, the consolidation process reduces the permeability of these materials to less than 1×10^{-7} cm/sec. Excess pore water pressures are not totally dissipated until long after the active life of the landfill, thereby establishing an upward flow

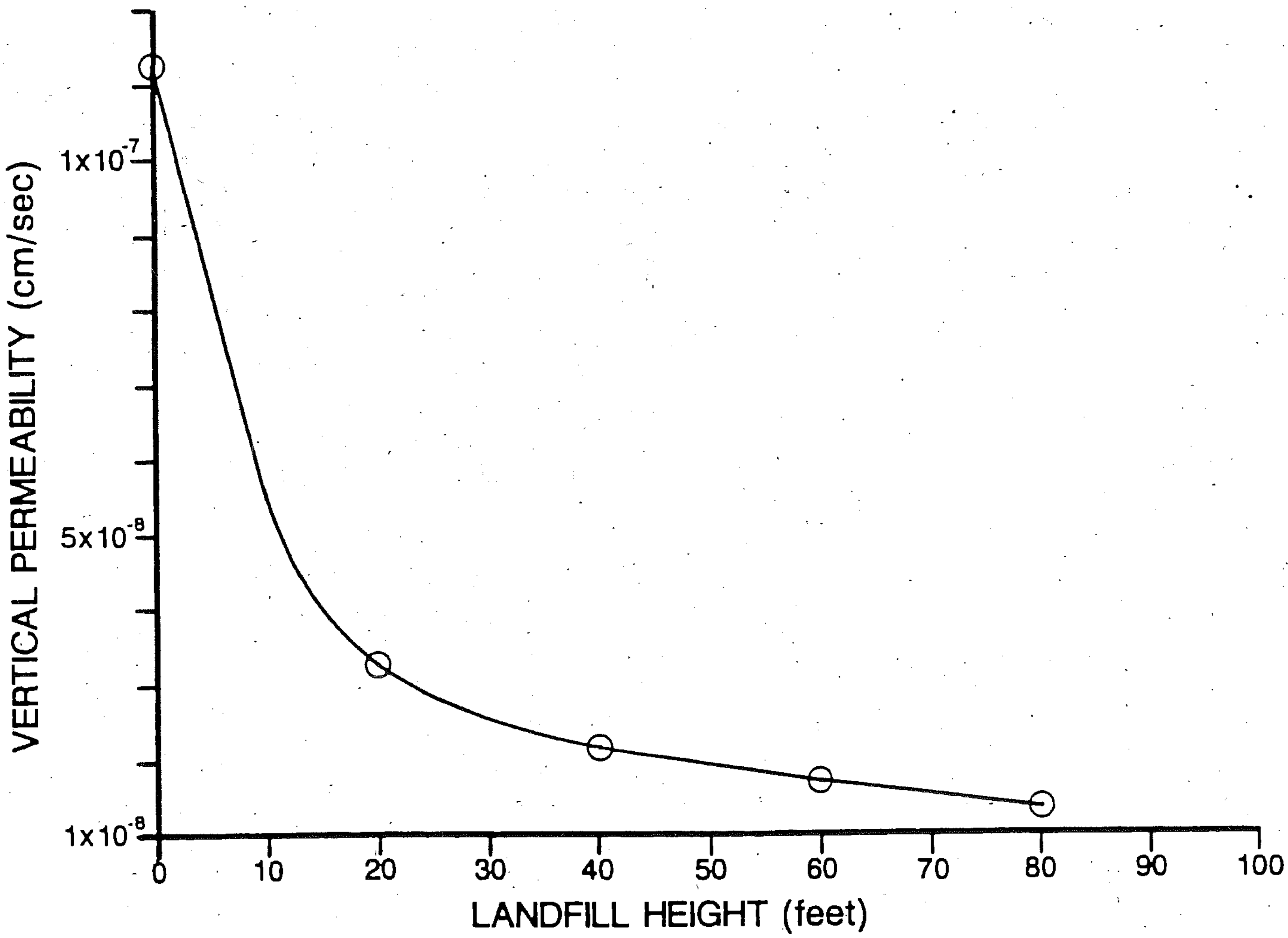

FIGURE 3-17
RELATIONSHIP OF VERTICAL
PERMEABILITY AND LANDFILL HEIGHT

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gradient from the dredge spoils into the leachate collection system during this time, and, in the process, prohibiting the downward migration of contaminant-carrying leachate.

The recently enacted federal RCRA Subtitle D regulations (October 1991) require that the lining system of all new solid waste landfills consist of a composite liner or an alternative design that meets the performance standards outlined in these regulations. Due to the unique subsurface conditions at Cherry Island (soft subgrade, significant anticipated differential settlements, positive upward flow gradients within the natural soil liner, etc.), an effective composite liner system is not deemed feasible. Subtitle D allows for alternative lining systems if it can be demonstrated that the design meets the regulated performance criteria. The performance criteria requires that the lining system provide secure containment such that the regulatory concentrations (based on Federal Drinking Water Standards) of selected target analytes not be exceeded at a selected compliance point within the designated critical water bearing unit below the site. The critical water bearing unit that would most probably be monitored for this purpose is the Potomac Formation, since the quality of the groundwater within the Columbia Formation (only other significant water bearing zone above the Potomac) is poor (non-potable) and is not used as a drinking water source. The compliance point has not been delineated at this time. As has been discussed previously, a positive upward gradient will be established in the dredge spoils until well after the operating life of the landfill. At this time, leachate concentrations will be much lower than during the active life of the landfill. These factors are therefore believed to render the hydraulic characteristics of Cherry Island favorable to the development of the Phase IV landfill using an alternative lining system design under the Subtitle D regulations by meeting the intent of the performance criteria.

In order to provide further data to support the performance of the proposed alternative design to the Subtitle D regulations, a series of geochemical tests were performed. The purpose of these tests was to provide data on the geochemical properties of the dredge spoils/recent deposits which define the ability of these materials to adsorb, retard and retain potential migration of contaminants. The geochemical testing which was completed included pH and Cation Exchange Capacity (CEC) on dredge spoils/recent deposits samples and Total Organic Carbon (TOC) on Cherry Island leachate which was permeated through samples of these soils. The results of these tests are summarized on Table 3-3. The results of the CEC tests indicate that the dredge spoils/recent deposits possess a CEC ranging from 22.1 to 37.1 meq/100g. These values are consistent with published values for clayey silts, silty loams, and low plasticity silts. CEC quantifies one of the geochemical properties of a soil that effect the absorption and retention of positively charged contaminants such as heavy metals. At the CEC values determined for the dredge spoils/recent deposits, adsorption of metals can be expected. In addition, the measured pH values also favor retention of metals within the dredge spoils/recent deposits.

TABLE 3-3

RESULTS OF GEOCHEMICAL TESTS ON DREDGE SPOILS/RECENT DEPOSITS

Boring No.	Sample No.	Depth (ft.)	CEC meq/100g	TOC %	pH in H_2O Standard pH Units	pH in $CaCl_2$ Standard pH Units
TB-1	ST-2	13-15	22.1	3.1	6.78	6.57
TB-3	ST-3	27.5-29.5	29.7	2.7	6.39	6.24
TB-4	ST-2	42-44	30.0	2.4	6.34	6.27
TB-4	ST-3	61-63	37.1	2.7	6.93	6.87
TB-2	ST-1	13.5-15.5	35.8	2.1	6.92	6.95

In order to further evaluate the organic compound retardation and retention properties of the dredge spoils/recent deposits, a flexible-wall column leach study is being performed. Using the flexible-wall permeameter apparatus, leachate from the existing landfill phases at Cherry Island was permeated through a 2-foot thick sample of the dredge spoils/recent deposits with a sample of effluent collected at different pore volume intervals (1/8, 1/2, and 1) and analyzed for TOC. Indicate a TOC concentration range of 98.2 to 119 mg/L compared to a concentration of 340 to 670 mg/L in the leachate itself. This preliminary data indicates favorable organic compound retardation and retention characteristics of the proposed natural liner system materials. The additional results of the column leach test will be provided under separate cover once available.

These geochemical test results indicate that, in addition to the favorable hydrogeologic conditions and low permeability of the dredge spoils/recent deposits, the proposed natural liner system materials will also adsorb and retard the migration of both metals and organic compounds. This data can also be used to model the potential migration of these compounds to a compliance point if required under the Subtitle D State program approval process.

3.7.5 Slope Stability Analysis

The geotechnical and topographic data provided on both the soil boring logs and the topographic survey drawing of the Phase IV area were analyzed for the purpose of developing a critical cross section through the Delaware River separation dike for slope stability analysis. The location of the selected cross section is presented on Figure 3-2 while the developed cross section is presented as Figure 3-18.

The slope stability analysis was completed using the computer program STABL2 which utilizes the Modified Bishop Method of Slices procedure to calculate the minimum Factor of Safety value for the slope. This procedure defines a large number of potential circular arc failure surfaces through the slope and calculates the Factor of Safety of each failure surface by dividing the failure mass into an appropriate number of slices. The overall moment and force equilibrium for each slice is then statically analyzed for both short term (end of construction) and long term stability conditions. The generated Factor of Safety values for these two conditions are then searched by the computer for the minimum values. These values are compared with minimum acceptable values for both short term (i.e. undrained) and long term (i.e. drained) soil shear strength conditions within the slope. In this manner, the Delaware River perimeter dike outslope which will be surcharged through the Phase IV landfill construction can be evaluated for potential slope instability. It should

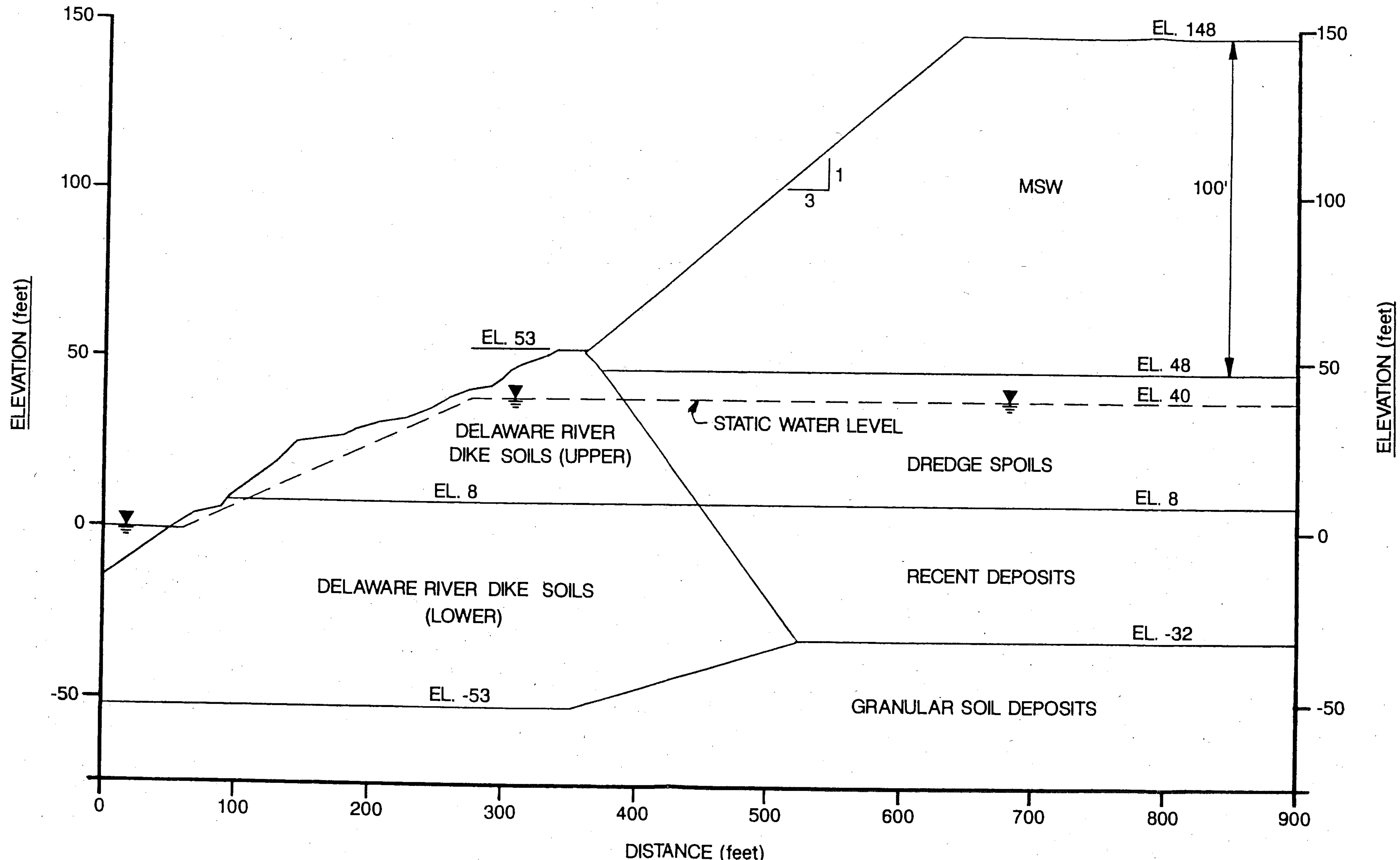

FIGURE 3-18
LANDFILL CROSS SECTION
FOR SLOPE STABILITY ANALYSIS

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be noted that this analysis can predict the likelihood of failure (via the computed Factor of Safety values) as well as the location of the critical failure surface. However, it cannot predict the duration of the failure should it occur. That is, it cannot differentiate between a sudden failure and one that will gradually occur over a long time period as a result of "creep" movements.

The remainder of this report section discusses in detail the referenced slope stability analysis. A discussion of the Modified Bishop Method of Slices computer program (STABL2) used to complete these slope stability analyses is presented in Appendix E of this report. Results (i.e. computer input and output data) of the various slope stability analyses which were completed are presented in Appendix F.

3.7.5.1 Subsurface Stratigraphy

Based on the available subsurface information, a total of five (5) subsurface strata were identified for slope stability analysis as shown on Figure 3-18. These include the following:

a. Dredge Spoils: These materials include the upper approximately 40 feet of the fine-grained soil stratum thickness which exists behind the Delaware River perimeter dike. These soils were assumed to extend from the ground surface (elev. 48.0 feet) to elevation +8.0 feet for this analysis. These materials are Delaware River bed soils which were placed atop natural river bank materials at a location behind the perimeter dike by dredging and pumping operations. Because of the pumping operations used to place these very wet, slurry-like sediments, these materials were found to be very soft, weak and compressible in their in situ condition.

b. Recent Deposits: These materials include the bottom approximately 40 feet of the fine-grained soil stratum thickness which exists behind the Delaware River perimeter dike. These soils were assumed to extend from elevation +8.0 feet to elevation -32.0 feet for this analysis. These materials are believed to be natural river bank deposits which existed at this location before the perimeter dike was constructed and dredge spoils were placed. They are geotechnically similar to the overlying dredge spoils. Since these materials have a much longer geologic history at this site, and therefore had the benefit of consolidation/strength gain under both self weight and the weight of the overlying dredge spoils, the shear strength of these soils is considerably greater, and the compressibility significantly lower, than the overlying dredge spoils. This is illustrated on Figure 3-19 which plots values of in situ void ratio (e_0), dry and total unit weight (γ_D and γ_T), natural moisture content (w), degree of saturation (S) and compression index (C_c) as a function of depth. As is evident from this Figure, values of in situ void ratio, water content and compression index decrease with depth, while dry and total unit weight increase with depth. These trends are

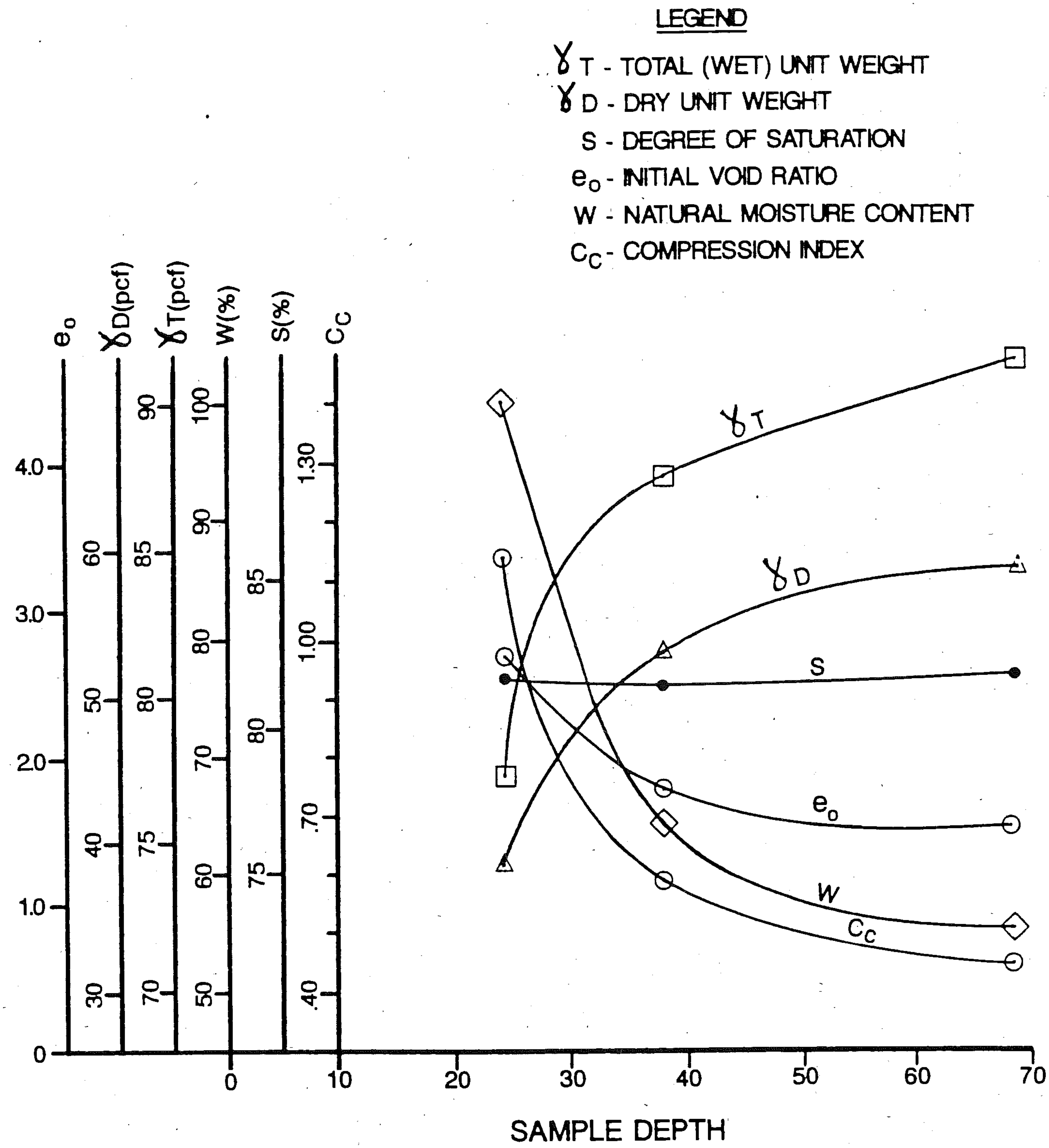

FIGURE 3-19
MEASURED SOIL PHYSICAL AND ENGINEERING PROPERTIES AS A FUNCTION OF DEPTH

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indicative of the greater shear strength of the deeper soils within the 80-foot thick fine-grained soil stratum.

c. Delaware River Dike Soils (Upper): These materials include the upper approximately 45 feet of the Delaware River separation dike thickness. These soils were assumed to extend from elevation +53.0 feet (top of dike) to elevation +8.0 feet for this analysis. These materials are generally fine-grained fill soils with granular inclusions which were placed by earthmoving operations to the geometrical configuration of the dike. As evidenced by the low "N" values of these materials as determined by split spoon sampling, it is believed that the dike fill placement operations were not stringently controlled. It is also believed that the dike was raised in five (5) stages as evidenced by the presence of an equal number of benches (including the top of dike bench) on the outside slope of this structure as shown on Figure 3-3.

d. Delaware River Dike Soils (Lower): These materials included natural fine-grained river bank deposits which lie between the dike fill soils and underlying granular soil deposits. These soils were assumed to extend from elevation +8.0 feet to elevation -53.0 feet (as determined from the test boring data) from the Delaware River to a location beneath the center line of the top bench of the dike. From this location, the bottom of this layer was assumed to increase in elevation to -32.0 feet as shown on Figure 3-18.

e. Granular Deposits: Sands and gravels underlie the recent deposits as well as the lower Delaware River dike soils. The assumed top elevations of this stratum correspond to the bottom elevations of the two overlying strata as discussed above. These granular soil deposits were noted to be of significantly greater shear strength than the overlying fine-grained soils as evidenced by much higher "N" values.

3.7.5.2 Assumed Landfill Geometry

Based on the results of the bearing capacity analysis discussed previously, it was determined that the dredge spoils/recent deposits can safety support up to 100 feet of landfill loadings. Therefore, this height of landfill was also assumed in the slope stability analysis. Based on conventional landfill design practice, the inclination of this landfill mass was assumed to be 3H:1V. No benches were assumed in the slope stability analysis. Therefore, the configuration of the waste slope was assumed to extend from the top of the Delaware River perimeter dike (i.e. elevation +53 feet) to the 100 foot high landfill top elevation of +148.0 feet at this 3H:1V inclination as shown on Figure 3-18.

3.7.5.3 Soil Properties

Soil properties, including unit weight, cohesion, and angle of internal friction, are required as input to the STABL2 slope stability computer program for each soil layer encountered. The selected values of these parameters for use in the undrained and drained stability analyses are presented in Table 3-4. The values of unit weight for the four fine-grained soil strata were directly measured during laboratory shear strength and/or consolidation testing of these materials. The undrained cohesion (i.e. c_u) of the dredge spoils and recent deposits was selected from UU laboratory tests completed on these materials as well as field measurements of this parameter using the testing procedures discussed in Section 3-4 of this report. In particular, in situ values of c_u for these materials were determined from "best fit" estimates of the available data base for both of these soil strata as shown on Figure 3-20. Determination of the linear regression lines was followed by averaging of the endpoints of the distributions to determine the average undrained cohesion of the stratum. Because of the low permeability, fine-grained nature of the dredge spoils/recent deposits, ϕ_u was assumed to be zero for the undrained stability analysis. Table 3-4 also indicates that the Delaware River dike soils were assigned drained shear strength parameters for the undrained stability analysis. As discussed subsequently, this is believed to be a reasonable assumption in light of the fact that special provisions will be incorporated into the Phase IV design (e.g. construction of an internal drainage system within the dike) to safeguard against the development of excess pore water pressures within the dike during active filling of the landfill. Therefore, drained conditions can be reasonably assumed to be maintained within these materials during the application of landfill loadings.

The effective stress shear strength parameters ($\bar{c}$, $\bar{\phi}$) for the four fine-grained soil strata which were used in the drained stability analysis were conservatively selected from the $\overline{\text{CIU}}$ database for these materials. These values were further conservatively adjusted based on correlations with measured soil physical properties (e.g. $\bar{\phi}$ as a function of PI), as appropriate. Even though drained cohesive strength ($\bar{c}$) was measured in all but one of the $\overline{\text{CIU}}$ tests completed on these materials, its value was conservatively neglected in the drained stability analysis.

An internal friction angle of 38° was assigned to the granular deposits for both the drained and undrained stability analyses. This value was determined from correlations of this parameter to the average "N" value of this stratum. A unit weight of 130 pcf was also assumed for these materials based on the average "N" value of this stratum.

TABLE 3-4

ASSUMED SOIL SHEAR STRENGTH PARAMETERS

		UNDRAINED SHEAR STRENGTH PARAMETERS		DRAINED SHEAR STRENGTH PARAMETERS	
SOIL	SOIL UNIT WEIGHT γ_T (pcf)	c_u (psf)	ϕ_u	$\bar{c}$ (psf)	$\bar{\phi}$
Dredge Spoils	80.0	175(1)	0	0	30°
Recent Deposits	90.0	540(1)	0	0	30°
Delaware River Dike Soils (Upper)	97.0	0	34°	0	34°
Delaware River Dike Soils (Lower)	101.5	0	34°	0	34°
Granular Deposits	130.0	0	38°	0	38°

(1) Assumed in situ values of c_u which were used only for the first lift loading condition. Strength gained values of c_u were used for subsequent lift loading conditions as discussed subsequently.

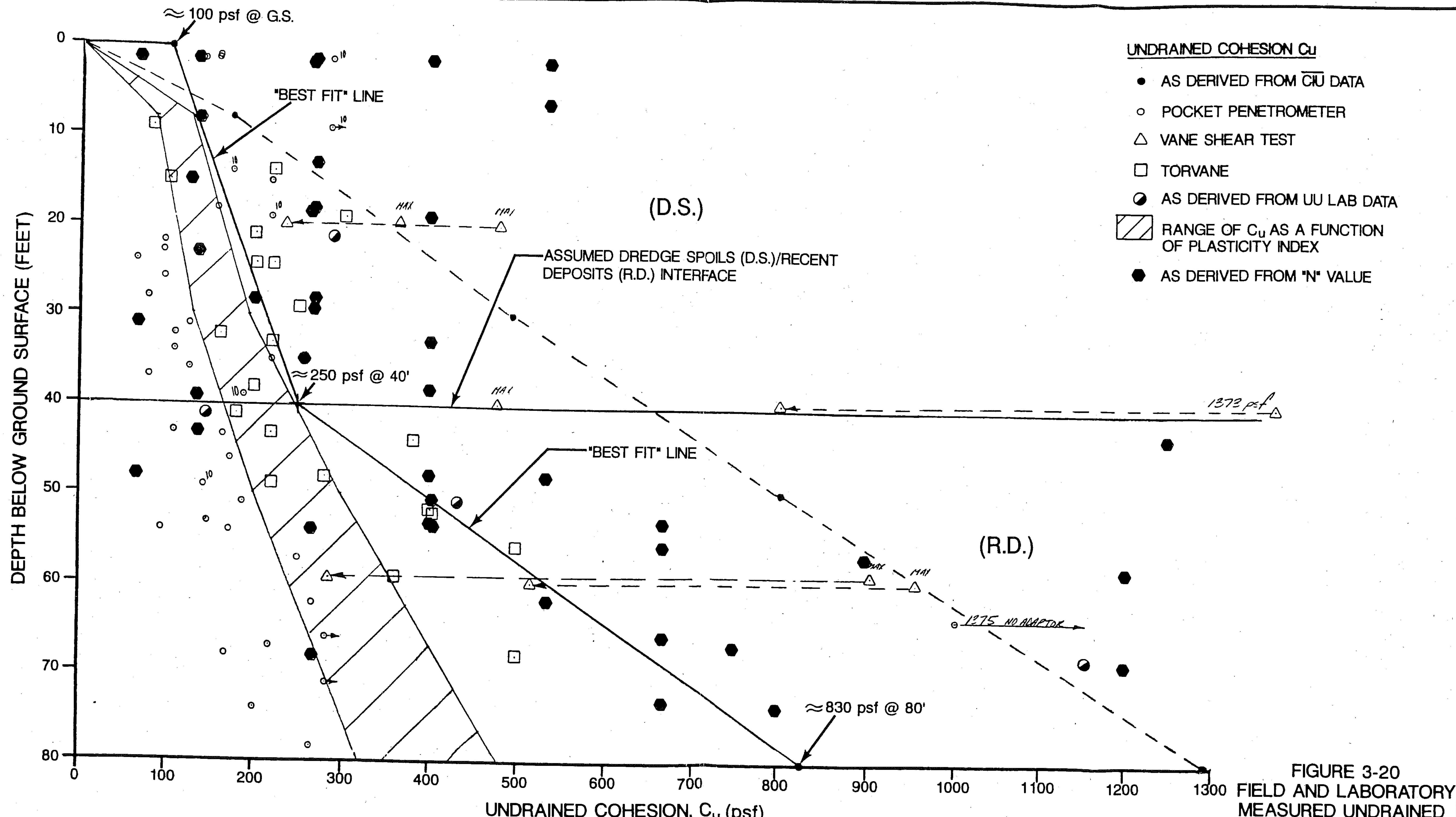

FIGURE 3-20 FIELD AND LABORATORY MEASURED UNDRAINED COHESION TEST RESULTS

TITLE: FIELD AND LABORATORY MEASURED UNDRAINED COHESION TEST RESULTS

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3.7.5.4 Waste Properties

Undrained and drained shear strength properties of the municipal solid waste (MSW) were required as input data to the slope stability analysis. The undrained properties of the waste correspond to the case of biodegraded, sludge-like material representative of older landfilled MSW. In this instance, the waste has high cohesive shear strength with low to zero internal friction similar to a soft clay. Drained shear strength properties of MSW correspond to the case of fresh, recently placed waste in which internal friction between the various waste components is prevalent. Numerical values of cohesion and internal friction for the MSW under undrained and drained shear strength conditions were determined from Figure 3-21. In particular, the undrained properties were selected as the conservative end of the cohesion range for the $\phi = 0°$ condition. The drained properties were selected as the conservative end of the internal friction range for the c = "minimum value" condition. In summary, these values are:

Undrained Waste

$c_u = 1450$ psf
$\phi_u = 0°$

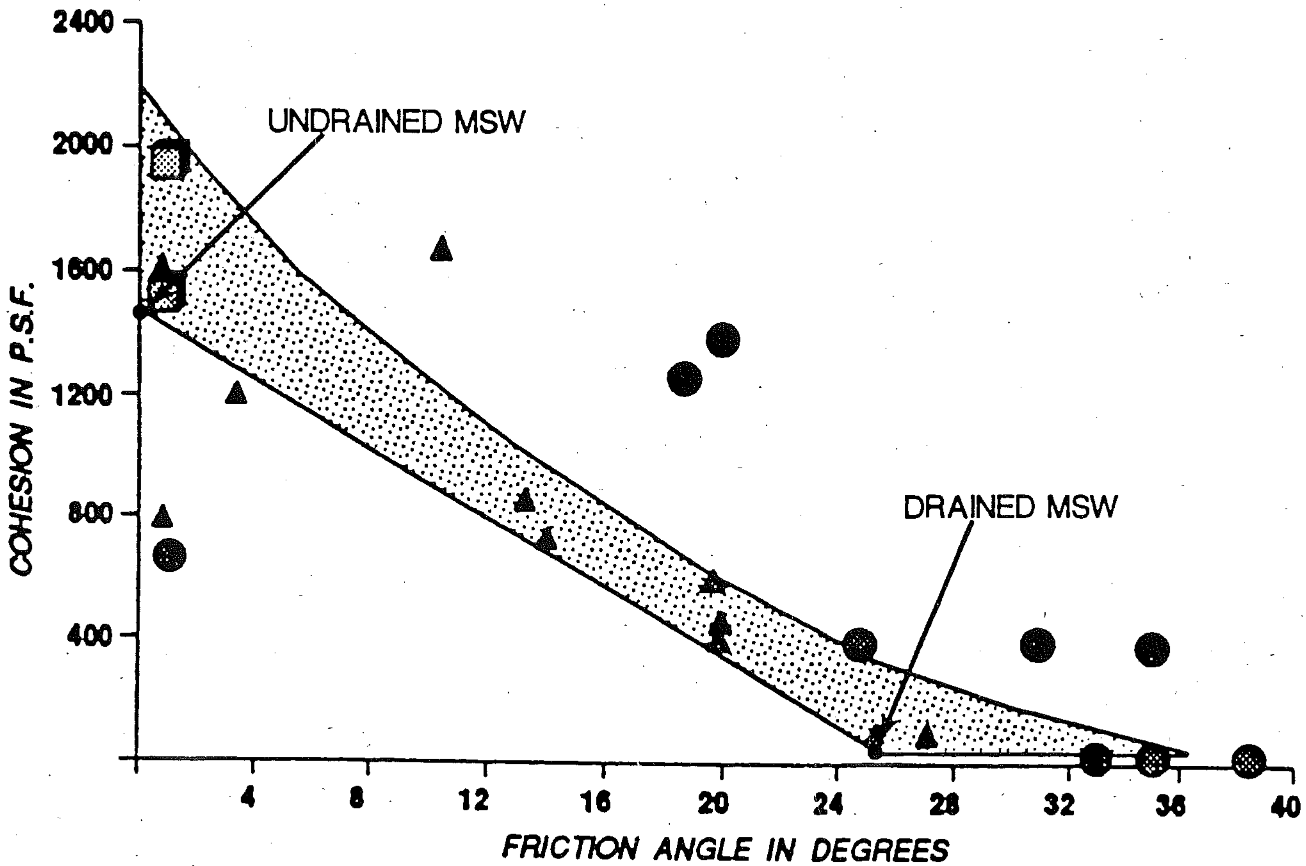
Drained Waste

$\overline{c} = 100$ psf
$\overline{\phi} = 25.5°$

In addition, as previously discussed, the unit weight of the composite MSW/daily cover material was assumed to be 70 pcf.

3.7.5.5 Static Water Level

The location of the static water level within the fine grained dredge spoils/recent deposits as well as the soils which comprise the Delaware River perimeter dike was also required as input to the slope stability analysis. This water level was believed to be created by free, excess water recovered by the dredging operations which was impounded behind the perimeter dike along with the dredge spoils. This free water saturated the dredge spoils to a depth which generally ranged from 8 to 12 feet below the existing ground surface as measured in the test borings completed internal to the landfill footprint. Therefore, a conservative value of 8 foot depth, which corresponds to an elevation of +40.0 feet, was selected as the static water level within the dredge spoils. This elevation was extended through the adjacent dike soils until this horizontal surface intersected an inclined surface



Summary Plot for All Tests with Recommended Parameters Shaded

REF: SINGH S. & MURPHY B., "A CRITICAL EXAMINATION OF THE STRENGTH AND STABILITY OF SANITARY LANDFILLS."

FIGURE 3-21
SHEAR STRENGTH PARAMETERS
FOR MUNICIPAL SOLID WASTE

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which originated at the Delaware River water surface (assumed to be elevation 0 feet for this analysis). The slope of this inclined water surface was determined using a seepage analysis procedure for determining the geometry of a phreatic surface within an earth dam. The final geometry of the assumed static water level used in the slope stability analysis is presented on Figure 3-18.

3.7.5.6 Seismic Effects

The effects of a seismic event were also evaluated in the slope stability analysis based on a site location in Seismic Zone 1 as shown on Figure 3-22. The horizontal seismic coefficient for the site was conservatively selected to be ".07g". The vertical seismic coefficient was selected to be one-half of the horizontal coefficient (i.e. ".035 g") in accordance with acceptable geotechnical engineering practice(12). These coefficients were applied in destabilizing directions (i.e. horizontally away from the slope and vertically upward) so as to reduce the calculated minimum Factor of Safety for the slope during the seismic event.

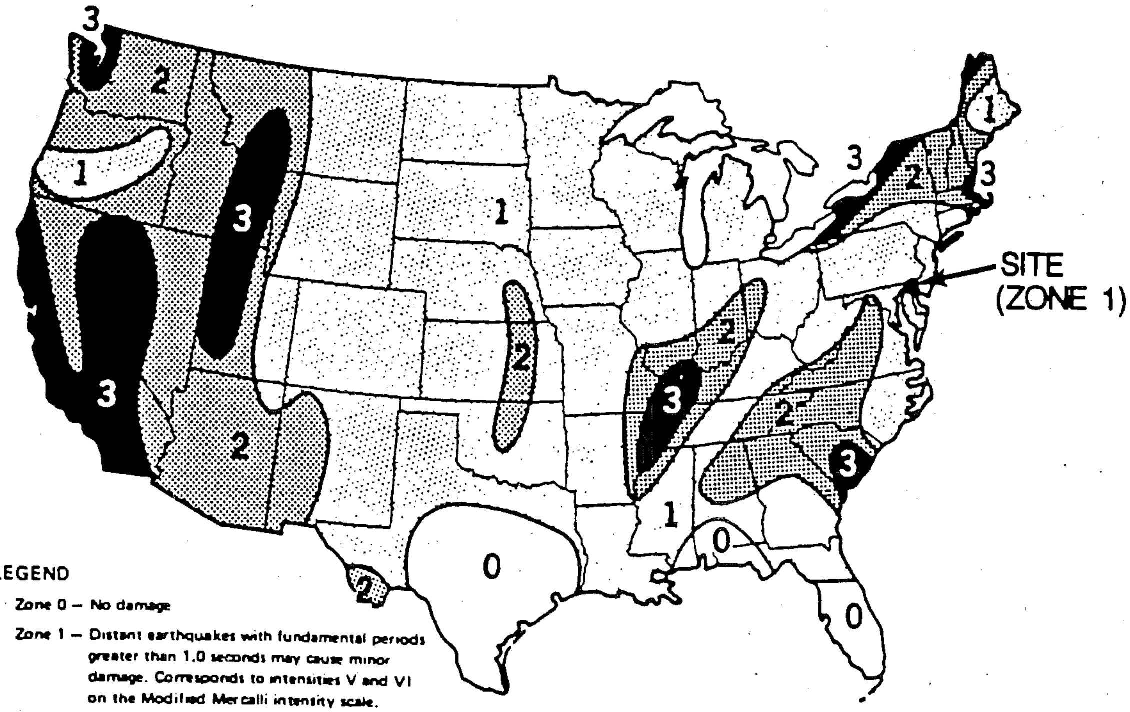
3.7.5.7 Undrained and Drained Analyses

Slope stability analyses assuming that both drained and undrained soil shear strength conditions govern the behavior of the dredge spoils/recent deposits were completed. These are discussed in the following subsections of this report.

3.7.5.7.1 Drained Analysis

The drained soil shear strength parameters used in this analysis are presented in Table 3-4. These values are based on assumed effective stress conditions within the stratigraphic profile. Six landfill height configurations were considered. These include 10, 30, 50, 70, 90 and 100 feet. Both undrained and drained shear strength properties were assumed for the waste mass for each landfill height configuration. Both seismic and non-seismic conditions were also considered for each case. This resulted in a total of 24 cases being completed to define the drained stability condition of the slope during and after active filling of the landfill.

The assumed loading condition for the landfill at any given plan location within the three Phase landfill footprint is shown on Figure 3-23. As is evident from an inspection of this Figure, a given landfill lift loading of 10 foot thickness (i.e. 700 psf) is assumed to be applied instantaneously at any given point within the footprint, followed by the lapse of a certain time period during which this loading remains constant until the next lift is placed at this



LEGEND

Zone 0 — No damage

Zone 1 — Distant earthquakes with fundamental periods greater than 1.0 seconds may cause minor damage. Corresponds to intensities V and VI on the Modified Mercalli intensity scale.

Zone 2 — Moderate damage; corresponds to intensity VII on the Modified Mercalli intensity scale.

Zone 3 — Major damage; corresponds to intensity VIII and higher on the Modified Mercalli intensity scale.

Seismic zone map of continental United States (After Algermissen, 1969)

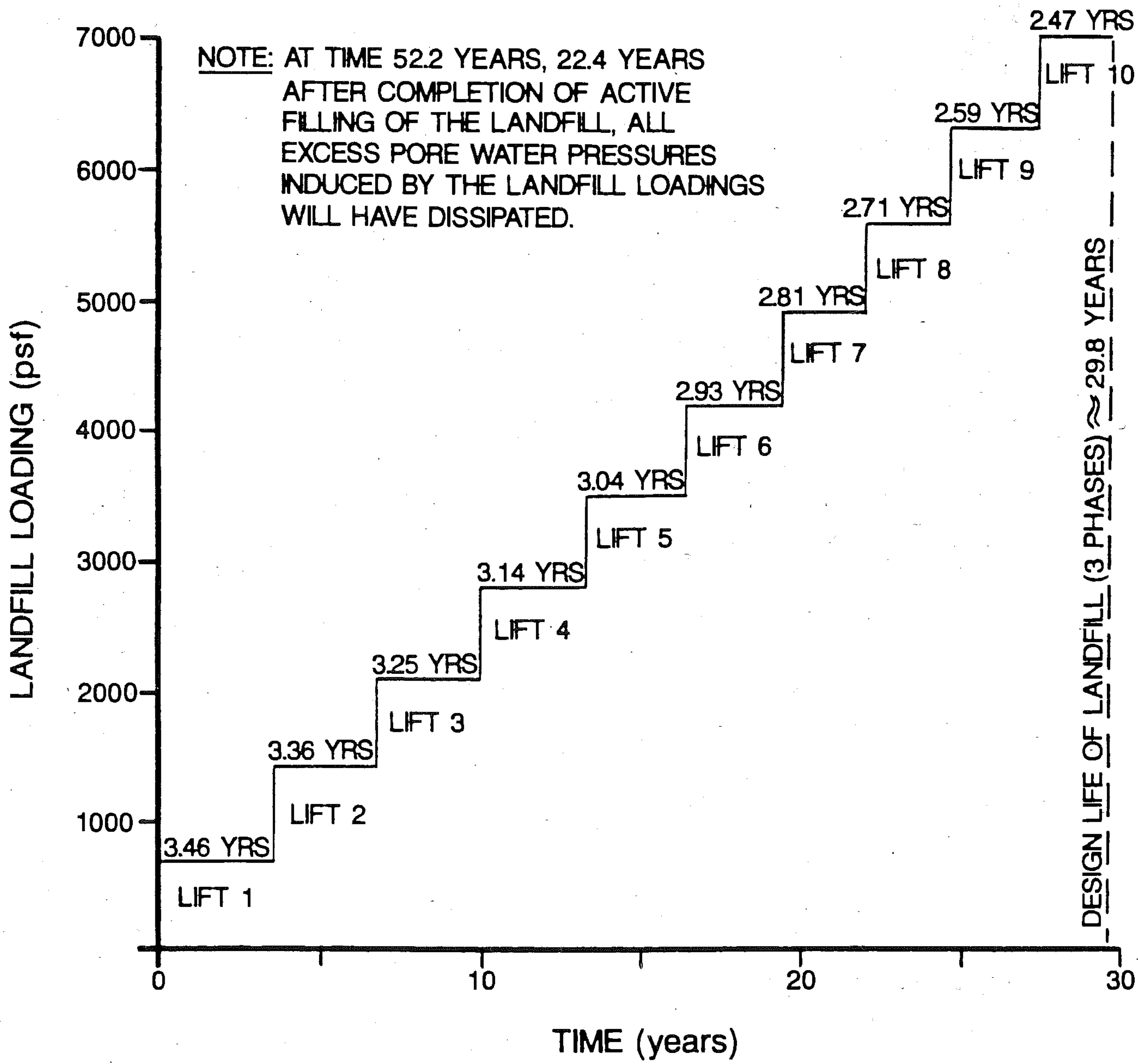
Table 2.2 Seismic Coefficients Corresponding to Each Zone.

ZONE	INTENSITY OF MODIFIED MERCALLI SCALE	AVERAGE SEISMIC COEFFICIENT	REMARK
0	—	0	No damage
1	V and VI	0.03 to 0.07	Minor damage
2	VII	0.13	Moderate damage
3	VII and higher	0.27	Major damage

REF: HUANG Y.H., "STABILITY ANALYSIS OF EARTH SLOPES."

FIGURE 3-22
SEISMIC COEFFICIENTS
FOR SLOPE STABILITY ANALYSIS

DSWA PHASE IV LANDFILL CHERRY ISLAND FACILITY WILMINGTON, DELAWARE	ROY F. WESTON, INC. WESTON MANAGERS DESIGNERS/CONSULTANTS				
	DRAWN A. DELTUFFO	DATE 7/24/92	DES. ENG.	DATE	W. O. NO. 2477-03-01
	CHECKED		APPROVED		DWG. NO.



NOTE:
LOADING DIAGRAM ASSUMES 70 pcf UNIT WEIGHT FOR LANDFILLED MSW AND A LIFT HEIGHT OF 10 FEET.

FIGURE 3-23
RELATIONSHIP OF LANDFILL LOADING AND TIME

DSWA PHASE IV LANDFILL CHERRY ISLAND FACILITY WILMINGTON, DELAWARE	ROY F. WESTON, INC. WESTON MANAGERS DESIGNERS/CONSULTANTS				
	DRAWN A. DELTUFFO	DATE 7/24/92	DES. ENG.	DATE	W. O. NO. 2477-03-01
	CHECKED		APPROVED		DWG. NO.

same location. The referenced time periods were calculated from the volumes of the 10 lifts and the placement rate at which these lifts will be landfilled. These time periods are also shown on Figure 3-23.

As discussed in Sections 3.7.2.1 and 3.7.4.1 of this report, a time rate of settlement analysis has shown that the dredge spoils/recent deposits stratum thickness will not reach full consolidation during any of the various time intervals required to place the 10 waste lifts. Therefore, excess pore water pressures will exist within this stratum at all times during active filling of the landfill. These excess pore water pressures must therefore be incorporated into the drained stability analysis since their effect is to reduce the drained shear strength of the soil and, therefore, the minimum Factor of Safety of the slope. The method by which this effect was incorporated into the analysis was to increase the static water level elevation of +40.0 feet to some higher elevation which is a function of the average excess pore water pressure which exists within the fine-grained soil stratum at the time the designated MSW lift for which stability is being evaluated is placed. This increased water level is designated as the "stressed static water level" for purposes of this discussion. For example, based on Figure 3-23, at the time at which the third MSW lift (i.e. 30 foot landfill height condition) is instaneously placed at a given plan location, the first waste lift has been in place a total of 3.46 years while the second waste lift has been in place a total of 3.36 years at this same location. Therefore, the "stressed static water level" head increase (Δh_{TOT}) at this time is calculated as follows:

$$\Delta h_{TOT} = (1 - U_1)\ \Delta p_1/\gamma_w + (1 - U_2)\ \Delta p_2/\gamma_w + (1 - U_3)\ \Delta p_3/\gamma_w \tag{13}$$

in which:

$\Delta p_1 = \Delta p_2 = \Delta p_3$ = the loading of the first, second and third MSW lift (i.e. 700 psf)

γ_w = unit weight of water

U_1, U_2, U_3 = average degree of consolidation of the fine-grained soil stratum based on time periods of 3.46, 3.36 and 0 years respectively as calculated from conventional consolidation theory assuming double drainage conditions. (Double drainage of excess pore water pressures from the fine-grained soil stratum is assured by the placement of a high permeability granular soil drainage blanket atop the dredge spoils as discussed in Sections 3.7.3 and 3.7.4.1 of this report). The value of the coefficient of consolidation (c_v) used in this analysis was .012 in^2/min. This value represents the minimum, and therefore mostconservative value of this parameter as measured in the consolidation tests within the stress range represented by the landfill loadings.

The values of Δh_{TOT} were then simply added to the elevation of the static water level (+40.0 feet) to determine the "stressed static water level elevation" (SSWLE). These values, along with the corresponding landfill top elevations, are presented in Table 3-5. The geometry of the "stressed static water level" for each analyzed landfill height condition was assumed to be horizontal and equal to the SSWLE beneath the top of the landfill, where the waste height is constant, followed by a linear decrease of this parameter to elevation +40.0 feet beneath the center line of the top bench of the Delaware River perimeter dike where the waste height is zero.

The results of the drained slope stability analysis are presented in Table 3-6. This Table presents the calculated minimum Factor of Safety values for the various conditions of waste height, SSWLE, waste shear strength (drained or undrained) and seismic conditions discussed previously (24 total cases). This Table also notes that the minimum required Factor of Safety values for drained stability analyses under seismic and non-seismic conditions are 1.0 and 1.5, respectively, as presented in the 1977 Federal Register. Based on these criteria, all 24 cases considered in the drained stability analysis resulted in acceptable Factor of Safety values.

Computer input and output data which documents each of the 24 cases is presented in Appendix F. The output data includes a graphical plot of the slope cross section which shows the landfilled mass, the SSWLE, and the assumed subsurface stratigraphy. These plots also indicate the locations of the 10 most critical failure surfaces identified by the computer search as well as the location of the most critical failure surface which is identified by hatch marks. The minimum Factor of Safety value which corresponds to this failure surface is also shown on these output plots. Note also that these plots indicate that a very extensive search routine consisting of approximately 400 circles was completed for each of the 24 cases which were analyzed. Figure 3-24 has been prepared to indicate the extensive and thorough nature of this search for the 100-foot high landfill, undrained waste, non-seismic case. It is evident from this Figure that circles of varying radii and center points which define the orientation of virtually every conceivable failure surface are selected by the search routine.

It is also apparent from an inspection of the computer output plots that all of the critical failure surfaces for the drained slope stability analysis lie completely within the Delaware River separation dike. Therefore, under drained soil shear strength conditions, the stability of the landfill mass is controlled by the drained shear strength and internal stability of the dike soils rather than the drained strength of the fine-grained dredge spoils/recent deposits

TABLE 3-5

STRESSED STATIC WATER LEVEL ELEVATIONS (SSWLE)

Landfill Height (feet)	Landfill Top Elevation (feet)	Δh_{TOT} (feet)	SSWLE (feet)
10	58	11.2	51.2
30	78	24.3	64.3
50	98	33.0	73.0
70	118	39.4	79.4
90	138	44.6	84.6
100	148	46.9	86.9

TABLE 3-6

RESULTS OF DRAINED (EFFECTIVE STRESS) SLOPE STABILITY ANALYSIS[1]

		Undrained Waste Shear Strength				Drained Waste Shear Strength			
Waste Height (feet)	Stressed Static Water Level Elev. (SSWLE) (feet)	Non-Seismic	Status[2]	Seismic	Status[3]	Non-Seismic	Status[2]	Seismic	Status[3]
10	51.2	1.597	OK	1.100	OK	1.597	OK	1.100	OK
30	64.3	1.596	OK	1.100	OK	1.596	OK	1.100	OK
50	73.0	1.602	OK	1.103	OK	1.602	OK	1.103	OK
70	79.4	1.601	OK	1.089	OK	1.627	OK	1.099	OK
90	84.6	1.601	OK	1.103	OK	1.601	OK	1.103	OK
100	86.9	1.601	OK	1.103	OK	1.601	OK	1.103	OK

(1)Table presents minimum calculated Factor of Safety values for the various waste height, SSWLE, waste shear strength and seismic conditions noted.

(2)Based on minimum required Factor of Safety of 1.5 as defined in the 1977 Federal Register.

(3)Based on minimum required Factor of Safety of 1.0 as defined in the 1977 Federal Register.

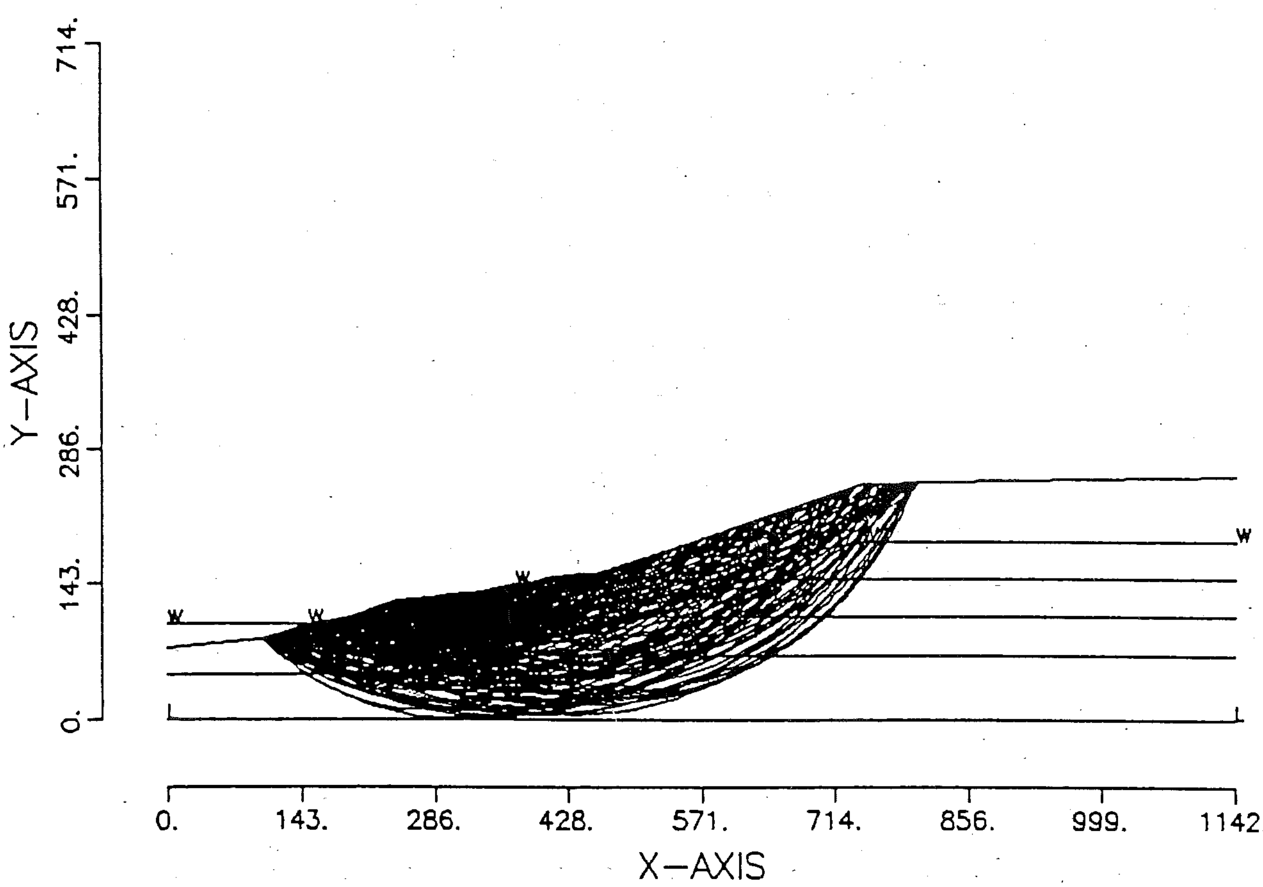


FIGURE 3-24
SLOPE STABILITY ANALYSIS:
SEARCH ROUTINE

DSWA PHASE IV LANDFILL CHERRY ISLAND FACILITY WILMINGTON, DELAWARE	ROY F. WESTON, INC. WESTON MANAGERS DESIGNERS/CONSULTANTS				
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which directly underlie the landfill footprint.

It is also noted that the minimum required Factor of Safety values for all 24 cases of the drained slope stability analysis have been achieved without the use of high strength, high modulus reinforcing geosynthetics on the landfill floor along the eastern perimeter of the landfill footprint. Therefore, it is concluded that no reinforcing geosynthetics of this nature are required for the Phase IV design based on the drained stability analysis.

3.7.5.7.2 Undrained Analysis

The soil shear strength parameters used in this analysis are presented in Table 3-4. Note from this Table that undrained shear strength conditions (i.e., c_u, $\phi_u = 0^o$) were assumed to exist within the dredge spoils and recent deposits for this analysis. This is realistic since these soils directly underline the various landfill lift loadings which can be assumed to be applied very rapidly over what can be a fairly small footprint representative of a potential three dimensional failure surface. Note also from this Table that drained shear strength parameters ($\overline{c}$, $\overline{\phi}$) have been assumed for the Delaware River dike soils. This is believed to be a reasonable assumption based on the fact that the majority of the soils which comprise the dike do not directly underlie the landfill lift loadings, but instead are situated adjacent to these loadings, thereby resulting in significant lateral attenuation of the stresses caused by the loadings before they reach these soils. Therefore, the magnitude of laterally attenuated stresses which create excess pore water pressures within the fine-grained soils which comprise the dike can, at any elevation, be assumed to be significantly less than vertical stresses induced by the landfill loadings directly beneath the landfill footprint at this same elevation. However, to insure the validity of this critical assumption, it is believed necessary to install a pore water pressure relief/control/monitoring system within the dike soils so as to rapidly dissipate excess pore water pressures which may be generated by the waste lift loadings. This feature of the Phase IV design will be conceptually discussed in the Conclusions section of this report.

The same six landfill height configurations assumed in the drained stability analysis were also evaluated in the undrained analysis (i.e. 10, 30, 50, 70, 90 and 100 feet). Both undrained and drained shear strength properties were assumed for the waste mass for each landfill height configuration. Both seismic and non-seismic conditions were also considered for each case. This resulted in a total of 24 cases being analyzed to define the undrained stability condition of the slope during active filling of the landfill. Since drained soil shear strength conditions were assumed to exist within the Delaware River perimeter dike soils for this analysis, the "stressed static water level elevation" discussed in the previous section of the report was also assumed for the analysis.

Strength gain will occur within the dredge spoils/recent deposits as these samples consolidate in response to the various waste lift loadings. This will increase the value of undrained cohesion (c_u) which will control the undrained stability behavior of these soils immediately following placement of the next lift of waste. Quantification of this strength gain phenomenon was discussed in Section 3.7.2.1 of this report. Numerical values of strength gained undrained cohesion which were used in the analysis are presented in Table 3-7. This Table also presents the results of the undrained slope stability analysis in the form of calculated minimum Factor of Safety values for the various conditions of waste height, waste shear strength (drained or undrained), and seismic conditions discussed previously (24 cases total). This Table also notes that the minimum required Factor of Safety values for undrained stability analyses under seismic and non-seismic conditions are 1.0 and 1.3, respectively, as presented in the 1977 Federal Register. Based on these criteria, all 24 cases considered in the undrained stability analysis resulted in acceptable Factor of Safety values.

Computer input and output data which documents each of the 24 cases of the undrained stability analysis is presented in Appendix F. The output data includes a graphical plot of the slope cross section which shows the landfilled mass, the SSWLE, and the assumed subsurface stratigraphy. These plots also indicate the locations of the 10 most critical failure surfaces identified by the computer search as well as the location of the most critical failure surface which is identified by hatch marks. The minimum Factor of Safety value which corresponds to this failure surface is also shown on these output plots.

It is evident from an inspection of the computer output plots that all of the critical failure surfaces for the undrained slope stability analysis also lie completely within the Delaware River separation dike. Therefore, under undrained stability conditions, the stability of the landfill mass is again controlled by the drained shear strength and internal stability of the dike soils rather than the undrained strength of the fine-grained dredge spoils/recent deposits which directly underlie the landfill footprint. It is also evident that the minimum required Factor of Safety values for all 24 cases of the undrained slope stability analysis have been achieved without the use of high strength, high modulus reinforcing geosynthetics on the landfill floor along the eastern perimeter of the landfill footprint. Therefore, it is concluded that no reinforcing geosynthetics of this nature are required for the Phase IV design. It is also noted that the critical failure circles and resulting minimum Factor of Safety values for each of the 24 cases of both the drained and undrained slope stability analyses are essentially the same since, as discussed above, the stability of the dike was assumed to be governed by the same drained shear strength parameters for both analyses.

TABLE 3-7

RESULTS OF UNDRAINED (TOTAL STRESS) SLOPE STABILITY ANALYSIS[(1)]

Waste Height (feet)	Undrained Cohesion C_u (psf)		Stressed Static Water Level Elev (SSWLE) (feet)	Undrained Waste Shear Strength Properties				Drained Waste Shear Strength Properties			
	Dredge Spoils	Recent Deposits		Non-Seismic	Status[(2)]	Seismic	Status[(3)]	Non-Seismic	Status[(2)]	Seismic	Status[(3)]
10	175	540	51.2	1.597	OK	1.100	OK	1.597	OK	1.100	OK
30	619	984	64.3	1.596	OK	1.100	OK	1.596	OK	1.100	OK
50	1061	1426	73.0	1.602	OK	1.103	OK	1.602	OK	1.103	OK
70	1500	1865	79.4	1.601	OK	1.089	OK	1.601	OK	1.089	OK
90	1935	2300	84.6	1.601	OK	1.103	OK	1.601	OK	1.103	OK
100	2151	2516	86.9	1.601	OK	1.103	OK	1.601	OK	1.103	OK

(1)Table presents minimum calculated Factor of Safety values for the various waste height, undrained cohesion, SSWLE, waste shear strength and seismic conditions noted.

(2)Based on minimum required Factor of Safety of 1.3 as defined in the 1977 Federal Register.

(3)Based on minimum required Factor of Safety of 1.0 as defined in the 1977 Federal Register.

3.8 CONCLUSIONS

Based on the results of the geotechnical investigation/analysis of the Cherry Island Phase IV landfill site, the following has been concluded:

1. The Phase IV landfill may be safely constructed from the inside top of slope of the Delaware River perimeter dike (i.e. at approximately elevation +53 feet) at a maximum 3H:1V inclination without benching to the design top elevation of this slope as discussed subsequently.

2. A maximum landfill height of 100 feet (elevation +148 feet) was used in the bearing capacity and slope stability analyses. This elevation was assumed constant beyond the top of slope of the 3H:1V waste face in these analyses. In reality, the landfill top must be sloped for drainage purposes. A value of 2% will be used in the WESTON design for this purpose. This will result in internal portions of the Phase IV landfill footprint extending above elevation +148 feet. Therefore, it is necessary to limit the elevation of the top of the 3H:1V landfill slope to less than +148 feet so as to preclude the 2% sloping landfill top from exceeding elevation +148 feet within all potential slope failure surfaces which are realistically feasible at this site. Based on this criterion, the elevation of the top of the 3H:1V slope will be set at +140.0 feet (i.e. 92 feet of landfill height at this location). Based on this proposed geometry of the landfilled mass, elevation +148 feet will not be exceeded until a horizontal distance of 676 feet behind the toe of slope of the fill. As is evident from Figure 3-24, this is well beyond the distance at which potential slope failures are feasible.

   The noted geometry will also result in a landfill crown elevation of approximately 168 feet (i.e. 120 feet of maximum landfill height) near the north/south centerline of the landfill footprint. Since this exceeds the 100 feet of landfill height assumed in the bearing capacity analysis, the undrained analysis was re-evaluated for this condition. A Factor of Safety reduction from 1.78 to 1.56 was calculated, however, this value still exceeds the minimum acceptable value of 1.5 noted previously. Therefore, it is concluded that the landfill may be raised to this maximum top elevation near the center of the Phase IV footprint.

3. No high-strength, high-modulus reinforcing geosynthetics are necessary on the floor of the landfill along the eastern perimeter of the site since the shear strength and internal stability of the soils of the Delaware River perimeter dike, and not that of the dredge spoils and underlying recent deposits, controls the stability of the landfill mass.

4. To preclude the development of excess pore water pressures within the Delaware River perimeter dike during active filling of the landfill, a pore water pressure relief, control,

and monitoring system will be necessary as part of the design and construction of Phase IV. The details of this design will be presented in the construction plans and specifications. At the present time, it is envisioned that this system will consist of the following:

a. Installation of lines of wick drains on the outslope benches and possibly the sloping faces of the Delaware River perimeter dike within the plan boundaries of the Phase IV site. The spacing, depth and number of these drains will be determined during final design of this system. A near surface drainage system (e.g. french drains) to accept and discharge subsurface water exiting from the wick drains will also constitute part of this design.

b. Controlled preloading of the outslope benches of the perimeter dike with common borrow materials as part of the specified landfill construction activities to be completed before waste placement begins. This process will facilitate internal drainage, consolidation and strength gain of these materials in a very controlled manner. The rate of application of these preloading lifts will be controlled by a system of installed piezometers which will monitor the dissipation of excess pore water pressures (through the wick drains) induced by the various preload lift loadings.

c. Continued maintenance of the wick drain pore water pressure relief system during active filling of the landfill as well as continuous monitoring of the dissipation of excess pore water pressures within the dike during this time using the installed piezometer system.

5. To design for the significant differential settlement potential of the site (to the extent possible), the features of the landfill floor grading plan and leachate collection system presented in Figures 3-15 and 3-16 will be incorporated into the design.

6. To permit effective leachate transmission to the leachate collection piping system, as well as to preclude leachate head buildup greater than the upward pore water pressures induced by the dredge spoils consolidation process, it is necessary to install a 12-inch thick granular soil drainage blanket of at least $1x10^{-2}$ cm/sec permeability over the entire Phase IV landfill footprint. In addition, in order to maintain the continuity and thickness of this drainage blanket, a reinforcement geotextile will need to be placed beneath the sand layer over the entire Phase IV footprint. This geotextile will also help safeguard against excessive localized differential settlement as well as bearing capacity failures near the working face of the waste, thereby facilitating more efficient landfill operations during placement of the first waste lift. The required properties of this geotextile will be determined as part of final design of this facility.

7. Based on the results of the permeability and geochemical testing programs and the technical analyses of Sections 3.7.4.1 and 3.7.4.2 of this report, it is concluded that the dredge spoils/recent deposits will satisfy the DNREC requirements of a natural soil liner at this site.

8. An operations plan must be developed for landfilling the three-phase footprint which conforms to technical assumptions 1 through 4 of Section 3.7.1 of this report. In this regard, it is critical that landfilling of each waste lift progress across the three-phase footprint in a rigidly defined and directionally similar manner so as to maximize the time interval between placement of various waste lifts at any given location within the landfill footprint. It is also critical that the amount of waste placed along the eastern perimeter of the site within any given time interval be minimized to avoid loadings of large areal footprint at this location from excessively surcharging the Delaware River perimeter dike. In this regard, an east-to-west filling operation across the landfill footprint of approximately 160 foot width (i.e. the width of a "sawtooth" depression) is believed reasonable for this purpose.

SECTION 4

LANDFILL CAPPING REPORT

The purpose of this report is to evaluate alternative capping systems for the final cover to be installed atop the Phase IV landfill when final grades are reached. This capping system report includes the evaluation of various available materials for use in a final cover system that meets the applicable regulations.

Based on discussions with DSWA, several types of materials are available locally for use in a final cover system. These materials include:

- **Sludge-Flyash Mixture -** Municipal Sludge from the Wilmington Waste Water Treatment Plant (WWTP) is stabilized with bituminous coal ash from Delmarva Power & Light (DP&L) and lime by VFL Technology Corp. (VFL Tech) for use as structural fill and could be used as a capping material. Approximately 150,000 cu. yds. of this material is stockpiled within a mile from the Cherry Island Landfill.

- **Dredge Spoils-** The existing dredge spoils in Phase IV and the expansion area to the south could be used as a capping material if excess material is available after landfill grading. Previous and current geotechnical testing of this material indicate, however, that the material possesses a significantly higher natural moisture content as compared to its optimum moisture content and would require extensive drying prior to use as cover material.

- **Flyash-** Flyash from the DP&L plant just north of Cherry Island is available as a capping material. This material is used to stabilize sludge as mentioned above to produce a potential capping material. The use of flyash alone and as an admixture with dredge spoils was investigated by Terraqua Resources Corp. as part of their Site Suitability Report, Hydrogeologic and Geotechnical Evaluation of the Cherry Island Site dated January 1984.

- **Digester Discharge (DD)-** Sludge from the Wilmington WWTP is mixed with select organic solid waste and co-composted at the DRP digesters to produce a marketable topsoil/soil amendment product after screening. Both screened and unscreened DD are available as capping materials.

In addition to these materials, the following potential capping materials were also evaluated in order to develop a conceptual final cover system that meets the applicable regulations:

- **Offsite Clay**- Offsite clay sources were evaluated for use as a low permeability layer. Physical properties, availability, cost, and comparison with other materials were evaluated.

- **Geosynthetics**- Geomembranes were evaluated for use as a low permeability layer, geocomposites/geonets for use as a drainage layer, and geotextiles for use as a protective cover layer were all evaluated as potential capping materials.

The focus of this report is on the final capping system for the Phase IV landfill. It is understood, however, that the installation of the final cover system over Phases III, IV, and future expansions (Phase V) will likely occur simultaneously since these phases will be developed concurrently. The final cover system for Phase IV will therefore be the same system that will be installed on the other adjacent phases.

This report also does not include an evaluation of potential daily and intermediate cover materials, because it has been reported that sufficient material is available for this use. The material currently being used as daily and intermediate cover at Cherry Island is the Digester Discharge described above. DSWA is not at this time looking into alternative materials for daily and intermediate cover. A limited amount of offsite sandy soils is currently used for truck access areas within the active phases.

The following sections of this report will discuss applicable regulations, provide further data on the potential capping materials listed above, and provide an evaluation of these materials and alternative final cover systems. The primary issues that are used for the material and system evaluations include availability, cost, conformance with applicable regulations (both in terms of chemical and physical properties), constructability, and performance.

4.1 APPLICABLE REGULATIONS

Section 5.H. entitled "Capping System" of Delaware's Regulations Governing Solid Waste (February 1, 1991) require the following components for a final capping system:

- **A final grading layer on the waste**, consisting of at least six inches of soil, to attain the final slope and provide a stable base for construction of subsequent system components. Daily and intermediate cover may be used for this purpose.

- **An impermeable layer**, consisting of at least a 20 mil geomembrane underlain by a geotextile, or a 24 inch layer of clay having a maximum hydraulic conductivity of 1×10^{-7} cm/sec, or of an equivalent depth material having a hydraulic conductivity less than 1×10^{-7} cm/sec.

- **A final cover**, consisting of 18 inches of soil to provide rooting depth and moisture for plant growth, and a 6 inch layer of topsoil or other approved material to support the proposed vegetation.

The new federal Subtitle D regulations promulgated in October 1991 require the final cover system to consist of an 18 inch cover layer, possessing a hydraulic conductivity less than or equal to the landfill liner system, overlain by a 6 inch vegetative layer. Although Delaware is not yet an approved state, the requirements under the Delaware Solid Waste Regulations may be approved by the US EPA and therefore will be used for the development and evaluation of the capping system for the Phase IV landfill.

4.2 EVALUATION OF CAPPING MATERIALS

This section presents an evaluation of the available capping materials listed in Section 4.0 with regard to the following criteria:

- Availability- Is the material available locally or within an economical distance of Cherry Island at quantities that are estimated for Phase IV and the phases to be closed concurrently (Phases III and V)? Are suppliers subject to potential interruptions in providing adequate supply of the material to DSWA, and are there any environmental/regulatory issues at the supply site?

- Cost- How does the material, transportation, and installation costs of the material compare to other materials to be used in the capping component being evaluated?

- Conformance with DNREC Regulations- Does the capping material meet the requirements both in terms of chemical and physical (where applicable) properties under Section 5.H. of the Solid Waste Regulations?

- Constructability- Will the material be difficult to install and work with under expected field conditions? Will the material require a specialty contractor or require extensive and costly drying, stabilization, or other special prehandling prior to installation?

- Performance- Will the material perform to the requirements specified in the regulations and per the intended design? Is slope stability a concern and how will it perform under critical loading and stress conditions?

Potential capping materials will be evaluated under each final cover system component required in the Delaware regulations beginning from the base of the cover system in the order presented in Section 4.1.

4.2.1 Final Grading Layer on the Waste

The final grading layer atop the waste may consist of daily and intermediate cover material per the Delaware Solid Waste Regulations. Currently, DD is used for daily and intermediate cover. This material is readily available and alternative materials are not being investigated by DSWA at this time.

4.2.2 Impermeable Layer

Per Section 5.H. of the Delaware Solid Waste Regulations, the impermeable layer shall consist of a minimum 20 mil geomembrane underlain by a geotextile, or a 24 inch layer of 1×10^{-7} cm/sec clay, or an equivalent material. Stabilized sludge, dredge spoils, flyash, DD, offsite clay, and geosynthetics will be considered and evaluated for use as an impermeable layer based on the evaluation criteria.

Sludge-Flyash Mixture (Stabilized Sludge)

Approximately 161,350 cu. yds. of material will be required for the final cover impermeable layer on Phase IV. There is an existing stockpile of stabilized sludge at the Wilmington WWTP of approximately 150,000 cu. yds. and additional quantities can be produced with readily available sludge and flyash. Analytical data on the stabilized sludge provided by VFL Tech (see Appendix G) indicates that the material does not demonstrate hazardous characteristics based on the results of the Toxicity Characteristics Leaching Procedure (TCLP) test. Physical property data indicate, however, that the stabilized sludge possesses an average compacted hydraulic conductivity of 1.68×10^{-5} cm/sec (see Appendix G), approximately two orders of magnitude higher than the required value for this cap component.

WESTON further investigated the possible use of this material due to its availability and relatively low cost (approximately \$2.50-3.00/cu. yd. delivered). Bentonite was added at 6 and 10 percent of total dry weight to the stabilized sludge to improve (lower) its permeability properties. Samples of the stabilized sludge were sent to Bowser-Morner Testing Laboratories in Dayton, OH and a series of flexible-wall triaxial permeability tests were run on the bentonite amended sludge-flyash mixture. The results of these tests (see Appendix G) indicated that at 6% bentonite amendment the permeability of the stabilized sludge decreased to 1.1×10^{-5} cm/sec. At 10% bentonite amendment the permeability decreased to 8.1×10^{-6} cm/sec, still well above the required 1×10^{-7} cm/sec value. Based on these results, it appears that even with bentonite amendment the stabilized sludge will not likely meet the regulated permeability requirements for this component. Furthermore, bentonite addition will also increase the cost of this material. It is also not known if a permeability of 1×10^{-7} cm/sec could even be achieved at higher bentonite amendment rates. In addition, it would be very difficult to continually provide a uniform mix that would achieve the necessary permeability without significantly increasing the cost of producing this material. At an amendment rate of 15% by dry weight at an estimated cost of $.19/lb of bentonite and a combined cost of $5.50/cu. yd. for bentonite addition and installation, the cost per cu. yd. of stabilized sludge would increase to approximately $10.50/cu. yd. This will be compared to the cost of offsite clay and a geomembrane impermeable layer component.

Dredge Spoils

Although the in situ dredge spoils to be used as the liner system will achieve a hydraulic conductivity of 1×10^{-7} cm/sec following consolidation of this material due to the landfill loadings, the use of dredge spoils as a capping material is limited due to its high moisture content. The natural moisture content of the dredge spoils ranges from 120% to 126% compared to its optimum moisture content of 46%. To achieve a remolded hydraulic conductivity of 1×10^{-7} cm/sec, the moisture content of the dredge spoils would have to be lowered to close to the optimum moisture content. This would require significant drying out of these materials, necessitating extensive area and reworking to aerate the dredge spoils prior to installation. This may also not be feasible given the time required to fully dry out these materials as compared to the normal time period between significant rainfall events.

A study was conducted by Terraqua in 1983 on the stabilization of dredge spoils with flyash from DP&L and lime. The results of this study are summarized in Appendix D-7 of Terraqua's Site Suitability Study dated January 1984 (portions of this summary are provided in Appendix G of this report). Various ratios of flyash and lime were added to dredge spoil samples and the effect on the mixture's moisture content and dry density were measured. The results of the study indicated that although the flyash and lime addition did lower the moisture content of the dredge spoils up to 40%, the reduction in moisture content was not sufficient enough to significantly improve the workability of the material which would require a reduction of 70-80% in the moisture content. It is therefore concluded that the use of dredge spoils for the impermeable layer is not feasible given the physical characteristics of this material.

Flyash

The use of flyash alone for the impermeable layer is not considered feasible based on permeability tests on remolded samples of this material from DP&L which indicate an average value of 3.2×10^{-5} cm/sec. In addition, the use of flyash has workability concerns with regard to its silty fine gradation in that when dry, it behaves like a dust and when wet, it cannot be readily graded or compacted.

Digester Discharge (DD)

It is assumed that DD possesses similar physical properties as the stabilized sludge, although a lower strength and higher permeability can be expected since no flyash and lime is added to the DD. Since the stabilized sludge does not meet the permeability requirement for this cap component, the use of DD for the impermeable layer is not considered feasible.

Offsite Clay

Two potential offsite sources of clay were investigated for this report. The first source is located 25 miles south of Wilmington at the intersection of I-95 and Rt. 40. The contact for this source is Cary Ahl (717-295-7292) with Eastern Minerals & Chemical Co. Physical property data on the clay from this source is provided in Appendix G. The average remolded permeability of this clay is 5.6 x 10^{-8} cm/sec, which meets the required permeability of 1 x 10^{-7} cm/sec. This clay is currently being used for a capping system at the Dupont complex north of Cherry Island according to Cary Ahl. The existing quantity of this material is approximately 3 million tons and they are currently mining it at a rate of 250,000 tons/year. This source therefore will be expired in approximately six years. An agreement with Eastern Mineral could however, be negotiated to reserve a portion of this stockpile for future use at Cherry Island. The estimated cost of this material, installed, is approximately $12/cu.yd. based on a quoted delivered cost of $5.30/ton.

The in-place moisture content of this material and requirements for air drying prior to installation are not known at the present time. The source site is over 8000 acres and is likely to have adequate area to dry out the clay if required. This would however increase the cost of the material. According to Eastern Minerals, no environmental issues or permits are pending at the site and the access roads are in place to remove the clay from the site. Based on the available data, this material appears to meet the requirements for the impermeable layer. The cost of this material is also comparable to the stabilized sludge amended with bentonite.

A second source of clay was investigated near York, PA. The contact for this source is Charles Gearhart (717-665-2248) with IMC, Inc. Physical property data on this clay is presented in Appendix G. The results indicate this material will also meet the permeability requirements. Estimated delivery costs from this source to Cherry Island would be approximately $15.50/ton, significantly greater than (nearly triple) that of the Eastern Minerals site south of Wilmington.

Based on the available data, the use of offsite clay is a feasible option for the impermeable layer. Cap stability issues are evaluated in Section 4.4

Geosynthetics

Section 5.H. of the Delaware Solid Waste Regulations requires a minimum 20 mil geomembrane underlain by a geotextile as one of the options for the impermeable layer. The recommended geomembrane for use as a capping material on a municipal waste landfill is a very low density polyethylene (VLDPE) geomembrane. VLDPE geomembranes are more frequently being used for landfill closure applications because of their lower modulus as compared to "stiffer" high density polyethylene (HDPE) geomembranes. The lower modulus allows for greater elongation prior to reaching yield strength. This is an important property for cap materials on municipal landfills which can undergo significant differential settlement. Textured VLDPE geomembranes are also available for use on sideslopes where a higher interface friction angle is required to prevent slippage between the geomembrane and the underlying geotextile. PVC and Hypalon/geomembranes have also been used for municipal solid waste landfill closures, however their usage has decreased significantly over recent years. Fusion welding techniques are more common among the VLDPE geomembranes and hence these materials can provide greater assurance of seam integrity during installation as well as in the long term.

VLDPE geomembranes are readily available from the major polyethylene liner manufacturers at costs competitive to other liner materials (PVC, Hypalon, etc.). The approximate installed cost of a smooth 40 mil VLDPE geomembrane is about $4.30/sq. yd. The estimated cost of the underlying geotextile (10-12 oz/sq yd nonwoven needlepunched fabric) is about $1.60/sq. yd., for a total installed cost of the geosynthetic impermeable layer of approximately $6.20/sq. yd. Considering textured 40 mil VLDPE geomembrane and an underlying geotextile, this unit rate would be about $6.70/sq. yd. For landfill closure applications, nonwoven geotextiles are typically used rather than woven geotextiles since the nonwoven geotextiles have more desirable properties/functions such as transmissivity, permeability, cushioning for the geomembrane, and interface friction (stability) when used in conjunction with textured geomembranes on sideslopes.

VLDPE geomembranes are installed and seamed using standard liner installation and fusion/extrusion welding techniques. Specifications and Quality Assurance Plans can be developed to assure proper installation and seam integrity. It is recommended that a minimum thickness of 30 to 40 mils be used for VLDPE geomembranes due to the risks associated with "burn-through" when attempting to fusion weld geomembranes of lesser thicknesses. Cap stability concerns are evaluated in Section 4.3. Based on the available data, a geosynthetic component consisting of a VLDPE geomembrane underlain by a protective nonwoven geotextile is a feasible option for the impermeable layer of the final cover system.

4.2.3 Final Cover

According to Section 5.H. of the Delaware Solid Waste Regulations, the final cover system shall include an 18 inch soil cover layer to provide rooting depth and moisture for plant growth, and a 6 inch topsoil or equivalent vegetated layer. Unlike the impermeable layer, the 18 inch cover is not required to meet any restrictive permeability requirements. Less costly and readily available materials for use in this layer include stabilized sludge, flyash, and dredge spoils. Flyash alone has workability concerns with regard to its silty fine grained gradation as discussed previously. Flyash and dredge spoils mixtures are not recommended based on the results of the evaluation of these materials by Terraqua. If dredge spoils in the area south of Phases I and II could be adequately dewatered, the use of dredge spoils could be an option. However, current efforts in Phases III and IV suggest that the production of a large quantity of acceptable material would be very difficult.

The existing stockpile of stabilized sludge at the Wilmington WWTP is approximately 150,000 cu. yd. which is greater than the volume needed for the 18 inch layer for Phase IV (approximately 121,000 cu. yds.). The materials to produce additional stabilized sludge are readily available from the finishing ponds at the WWTP (sludge) and DP&L power plant (flyash). The estimated installation cost of the stabilized sludge is approximately \$5.50-6.00/cu yd, which is very competitive with offsite sources of common borrow which are estimated to be between \$5.00 and \$7.00/cu. yd. The stabilized sludge material has been approved previously by DNREC for use at the Cherry Island Landfill for separation berm construction and perimeter dike reinforcement. The material is soil-like (due to flyash content) and should be readily workable. This material has also been used on selected highway embankment projects in Delaware. WESTON obtained samples of the stabilized sludge from VFL Tech and sent selected samples to Bowser-Morner Testing Laboratory for triaxial shear strength testing. The shear strength data is discussed along with the cap stability issues in Section 4.3.

Since the primary function of the topsoil layer is to provide for the establishment of an adequate vegetative cover, the most readily available and best suited material for this function is the DD. The use of DD at Cherry Island has been approved by DNREC and is currently being used for daily and intermediate cover. The use of DD in the final cover system complements DSWA's DRP as it provides an environmentally sound reuse of a solid waste.

The results of chemical analysis for heavy metals on the Fairgrow product (screened DD) are provided in Appendix G. These results have previously been submitted to DNREC and meet their regulatory requirements for heavy metal and PCB content (see Appendix G). The nutrient properties of Fairgrow are also provided in Appendix G as listed on the promotional brochure for this product. Based on visual inspection of both the DD and Fairgrow by WESTON and discussion with DNREC on these materials, it is recommended that the final cover be composed of 5 inches of DD overlain by approximately 1 inch of the Fairgrow. Although the DD provides adequate physical and nutrient properties to establish a vegetative cover, large pieces of solid waste present in the unscreened DD may effect the establishment of a uniform grass cover and provide a less aesthetic option to the screened Fairgrow. This recommended cross section has already undergone field testing by DSWA as part of a study on producing marketable sod using these recycled materials. A letter discussing the suitability of this recommended final cover system from Dr. W. H. Mitchell of the University of Delaware, who was involved in the sod study, is included in Appendix G. The results of the study showed that a high quality vegetated cover was established after two years. Greenhouse studies on the DD also suggest the acceptability of this material as landfill cover. Stability issues will be addressed in Section 4.3.

Considering the cost for topsoil materials, discussions with Rebecca Roe of the DSWA have concluded that the Fairgrow product could be delivered to the Cherry Island Landfill at a cost of about $2.75/cu. yd. The Fairgrow product is sold commercially at $4.50/cu. yd. It is expected that offsite sources of topsoil would typically cost approximately $7.00 to $10.00/cu. yd. for delivery to the site.

4.3 EVALUATION OF RECOMMENDED COVER SYSTEMS

Based on the above evaluation of the available capping materials, two alternatives of the final capping system have been developed and are presented on Figures 4-1 and 4-2. Alternative 1 (Figure 4-1) consists of an impermeable layer composed of 24 inches of compacted clay and an overlying protective cover layer composed of 18 inches of stabilized sludge overlain by 5 inches of DD and 1 inch of Fairgrow. Alternative 2 (Figure 4-2) consists of an impermeable layer composed of a 30 to 40 mil VLDPE geomembrane underlain by a nonwoven geotextile, and overlain by the same protective cover as discussed above for Alternative 1. (It may be necessary to physically separate the VLDPE geomembrane from the overlying stabilized sludge by placement of a nonwoven needlepunched geotextile between these two materials. This will also help protect the geomembrane from puncture, tear, and abrasive forces generated by placement of the overlying stabilized sludge. It should also improve the sliding resistance of the final cover system as a result of its greater interface friction angle (as compared to smooth VLDPE) when in contact with clay like materials.) The potential need for a drainage layer within the final cover system, is presented on Figure 4-2 for Alternative 2. This drainage layer may be required to ensure stability of the final cover system should Alternative 2 be selected and detailed during final design. The HELP Model was used to determine the need for this drainage layer (as discussed under the Performance section below). The evaluation of these

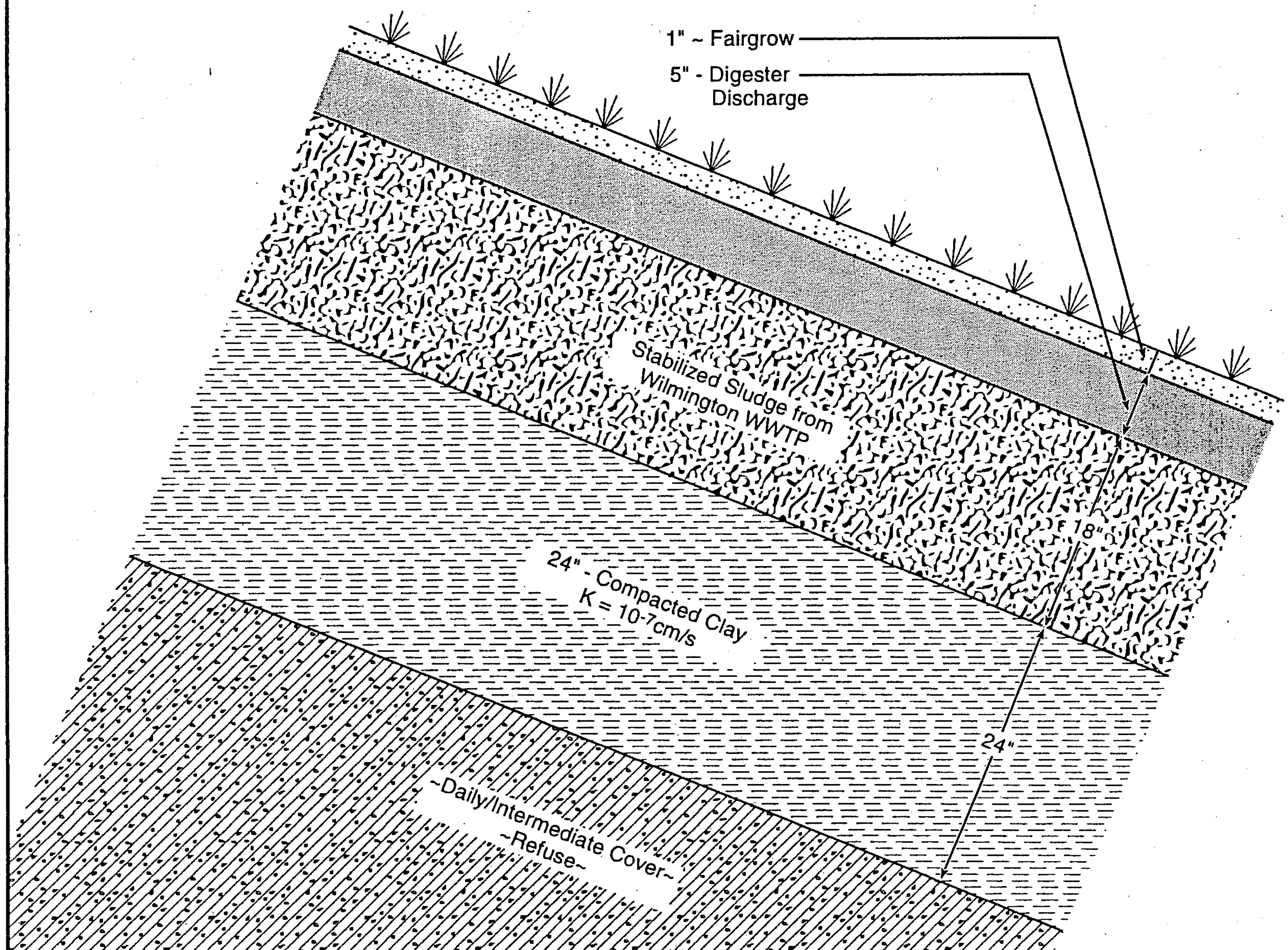

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FIGURE 4-1 CONCEPTUAL FINAL LANDFILL CAP SYSTEM - PHASE IV ALTERNATIVE 1

WESTON
MANAGERS DESIGNERS/CONSULTANTS

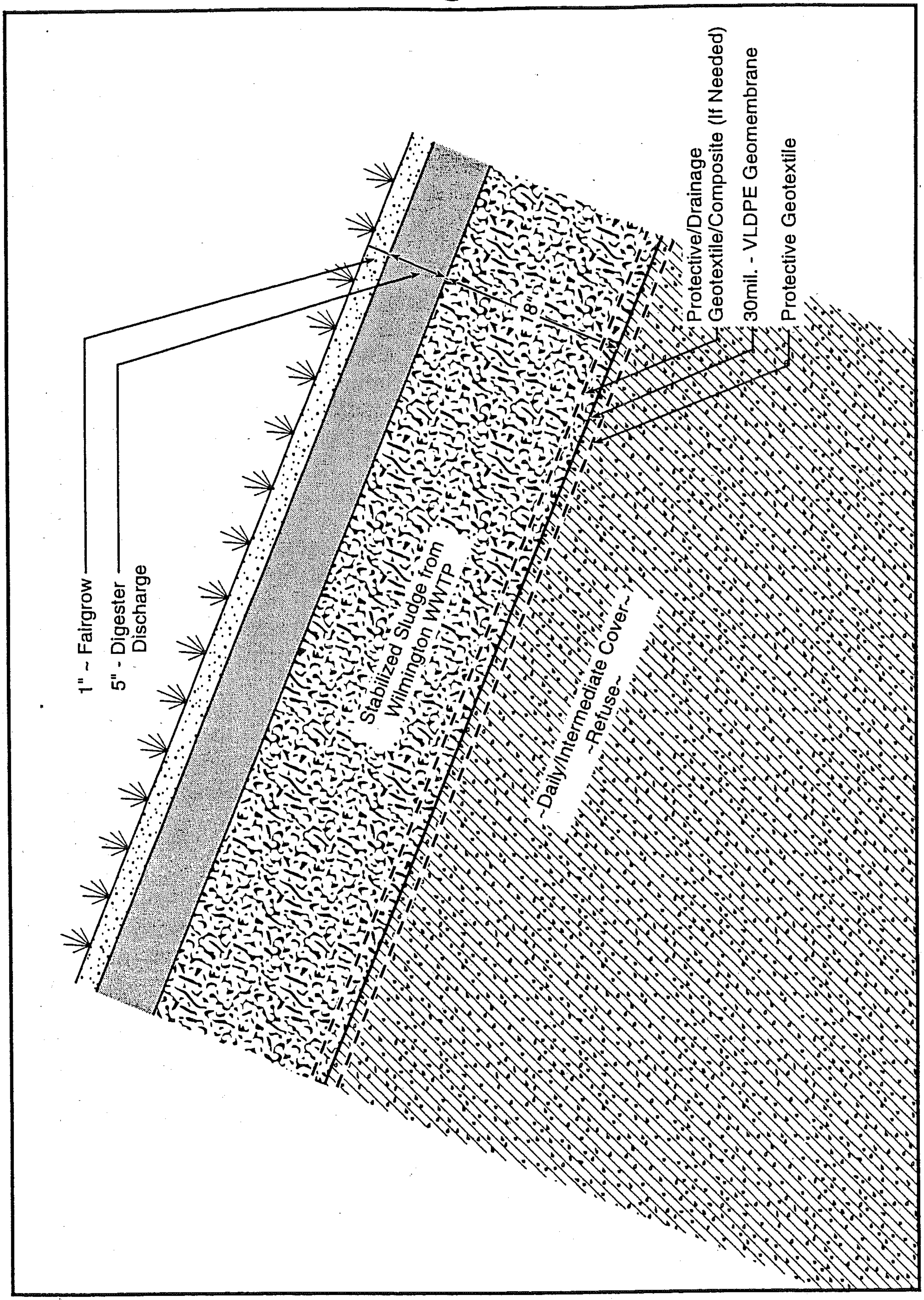

FIGURE 4-2 CONCEPTUAL FINAL LANDFILL CAP SYSTEM - PHASE IV ALTERNATIVE 2

two alternatives is based on the criteria listed in Section 4.2. Since both alternatives consist of the same materials in the protective cover layer, the following evaluation will focus on the impermeable layer of the two alternatives.

Availability

Two sources of clay that meet the required permeability of 1 x 10^{-7} cm/sec have been identified, however, the availability of the quantities necessary for Phases III, IV, and V when the landfill closes cannot be assured at this time. Clay sources can also be contracted for future use to assure that quantities are available at closure. VLDPE geomembranes are available from several major liner manufacturers and could be considered more readily available than clay sources within an economical distance from Cherry Island.

Cost

The estimated cost of the 24 inch compacted clay impermeable layer is approximately $12.00/cu. yd. or $1,940,000 for the 50 acre Phase IV Disposal Area. The estimated cost for smooth 40 mil VLDPE geomembrane underlain by a nonwoven geotextile is approximately $6.20/sq. yd. or approximately $1,500,000 for a 50 acre landfill. These cost estimates are preliminary and are for conceptual design purposes only. They are presented here for approximate comparisons between the two alternatives.

Conformance with DNREC Regulations

Both alternatives meet the requirements of Section 5.H. of the Delaware Solid Waste Regulations.

Constructability

Depending on the moisture content of the clay source, adjustment in the moisture content may be required to achieve the specified compaction, permeability, and strength properties of this material. If the clay source is too wet, the material will require drying out which may be very difficult if construction occurs during wet seasons. High plasticity clays often achieve the required inplace permeability when compacted slightly wet of optimum, however these materials are also susceptible to dessiccation cracking during dry weather which can significantly increase its inplace permeability. Compaction of clays on slopes is not a major concern for Phase IV which has maximum slopes of 3H:1V, although a greater number of passes may be required by the roller to achieve the required density and permeability.

The installation of the VLDPE geomembrane will require a specialty contractor. As mentioned previously, specifications and quality assurance procedures need to be developed to assure proper installation and seaming. Geomembrane installation may be restricted during very hot and cold weather.

Performance

Compacted clay caps are susceptible to desiccation cracking over time which will increase the inplace permeability and affect the long term performance of the cover system. Differential settlement of the landfill solid waste is a concern for both the compacted clay and the VLDPE geomembrane systems. Compacted clay can be repaired by filling in depressions caused by differential settlement of waste with additional clay and final cover material. If a depression results in tensioning of the geomembrane beyond its allowable yield strength, the integrity of the membrane could be impacted, resulting in repairs needed for both the geomembrane and underlying/overlying geosynthetics. The use of a VLDPE has been recommended to allow for a greater allowable yield strength to address these concerns.

Stability issues were investigated for both alternatives. Both an undrained and drained condition using the infinite slope analysis was performed for both alternatives for the sideslopes of the landfill (3H:1V slopes). Triaxial strength data was obtained from Eastern Minerals for the compacted clay layer. Shear strength parameters for the stabilized sludge was obtained from both unconsolidated undrained (UU) and consolidated undrained with pore water pressure measurements (CIU) triaxial compression tests on samples provided by VFL Tech. Laboratory results are provided in Appendix G. The HELP Model was also used to evaluate the performance of the proposed final cover system alternatives. In terms of minimizing infiltration through the final cover system, both Alternative 1 and Alternative 2 yield comparable results. HELP computer runs are provided for Alternative 1 only in Appendix G. The results also indicate that, with no drainage layer between the impermeable layer and the final cover, the entire final cover layer of Alternatives 1 and 2 may, at times, become completely saturated, thereby developing undrained shear strength conditions within these materials.

The results of the stability analysis for Alternative 1 indicates an acceptable Factor of Safety for the undrained and drained cases, assuming no drainage layer. The results of the stability analysis for Alternative 2 evaluating the critical interface angle between a smooth VLDPE geomembrane and the stabilized sludge indicates an unacceptable Factor of Safety under saturated conditions, even when a drainage layer is provided directly above the geomembrane. These preliminary results indicate that, on the sideslopes of the landfill, stability is a concern with Alternative 2. Therefore, a textured VLDPE geomembrane with an overlying drainage layer will be required to provide an acceptable Factor of Safety.

Detailed analyses will be performed during final design to determine the required minimum interface friction angle needed to ensure stability on the 3H:1V sideslopes. It is believed that a textured VLDPE geomembrane and a drainage composite (a drainage net sandwiched between two nonwoven needlepunched geotextiles) may be an acceptable design and adequate for stability if tolerable amounts of tension can be carried by the geosynthetic components within the final cover system. Also, an alternative design consisting of a textured VLDPE geomembrane overlain by 12 inches of drainage sand may also provide adequate stability. These evaluations will be made during final design so that a stable cap is provided.

4.4 CONCLUSIONS

Considering the evaluation criteria presented above for the DSWA Phase IV final cover system alternatives, the following components are recommended for incorporation into the design of this facility:

- Topsoil - 1" inch of DSWA's Fairgrow Product underlain by 5 inches of Digester Discharge.

- Final (Protective) Cover - 18" of stabilized sludge from the Wilmington WWTP.

- Impermeable Layer - 24 inches of low-permeability off-site clay or a 30-mil (minimum) VLDPE geomembrane underlain by a nonwoven geotextile. A nonwoven geotextile may also be required atop the VLDPE geomembrane pending further evaluation. The VLDPE geomembrane option is recommended for the final cover system on the top of the landfill and would include a smooth VLDPE geomembrane. Either option, clay or geomembrane, may be appropriate for the sideslope final cover system. The geomembrane option for the sideslopes would include textured VLDPE.

- Drainage Layer - 12 inches of sand or a drainage composite consisting of a drainage net sandwiched between two nonwoven needlepunched geotextiles. This layer would be used only on the sideslopes with the textured VLDPE geomembrane option.

For the topsoil and final cover soil layers, offsite materials cannot compete with the availability and cost of the Fairgrow, DD, and stabilized sludge materials accessible to DSWA. In addition, these materials have been previously approved by DNREC for DSWA landfill applications.

Two alternative materials are recommended for the impermeable layer depending on the application with respect to the final cover being constructed on the flatter slopes at the top of the landfill or on the steeper 3:1 sideslopes. Textured VLDPE geomembrane will be required on the sideslopes to provide adequate stability should this material be selected instead of offsite clay. In addition, this option will require a sand or drainage composite drainage layer in conjunction with the textured geomembrane to further enhance stability. The additional cost associated with this drainage layer, approximately $6.20/sq. yd. installed, combined with the previously mentioned unit cost of $6.70/sq. yd. for the geomembrane and underlying geotextile, results in a total approximate installed cost of $12.90/ sq. yd. for this component (i.e., impermeable layer) of the final cover system on the landfill sideslopes. Comparatively, the use of offsite clay on the landfill sideslopes will cost approximately $12.00/cu. yd. installed. As indicated above, stability and cost will govern the selection of these materials. These criteria should be further evaluated and revisited during final design of the final cover system. The final cover design must also meet requirements of RCRA Subtitle D and DNREC requirements which must be consulted during the final design phase.

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