US Army Corps
Of Engineers
Wilmington District

PHASE II DMMP Study
Upper Portion of Wilmington Harbor
Eagle Island Management Plan

Volume II
Embankment Stability Analysis
Eagle Island Confined Disposal Site

Wilmington, North Carolina

Final Report

Prepared by:

URS/Dames & Moore
5301 77 Center Drive, Suite 41
Charlotte, NC 28217
Contract Number DACW54-99-D-0001

May 2001

LIST OF REPORT VOLUMES AND TITLES

VOLUME	TITLE
I	PHASE II DMMP Study
	Upper Portion of Wilmington Harbor
	Eagle Island Management Plan
	Attachment A – Operations Manual
	Attachment B – CD Containing all Volumes and Files in Electronic Format
II	**Embankment Stability Analysis**
III	Data Records of Embankment Stability Analyses
IV	Field Investigation
V	Laboratory Tests
VI	Primary Consolidation, Secondary Compression, and Desiccation of Dredged Fill (PSDDF) Report

VOLUME II

FINAL REPORT
EMBANKMENT STABILITY ANALYSES
EAGLE ISLAND CONFINED DISPOSAL SITE
WILMINGTON, NORTH CAROLINA

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

1.0 INTRODUCTION

1.1 GENERAL

This report was prepared at the request and authorization of the Wilmington District, U.S. Army Corps of Engineers (USACE). The Eagle Island confined disposal facility (CDF) is an approximately 700-acre site located near Wilmington, Brunswick County, North Carolina. The Eagle Island CDF is the primary disposal site for the dredged material from the upper portion of Wilmington Harbor. The site location is depicted on Figure 1.1. An aerial view of the general project vicinity is shown on Figure 1.2.

Vertical expansion of the existing perimeter dikes will be required to allow the CDF to accommodate future periodic disposal of dredged materials. Over the period about 2001 to 2021, the CDF will receive about 30,000,000 cubic yards of dredge spoil, principally fine grained soils associated with maintenance dredging. This report presents the findings of field exploration, laboratory testing, and engineering analyses (hereinafter, “geotechnical studies”) by Dames & Moore at the Eagle Island CDF. The geotechnical studies address the existing stability of perimeter embankments around the CDF, and projected stability of those embankments following vertical expansion of about 15 feet, from about elevation 23 feet msl to about 38 feet msl. Figure 1.3 shows a cross section of the dike with the anticipated raises.

(Note: Unless specifically stated otherwise, all elevations given in this report refer to Mean Sea Level (msl).)

The work reported herein addresses the technical feasibility of geotechnical design for vertical expansion of the perimeter dikes. It is understood that the information presented herein will be used by USACE to support a variety of decision-making regarding the Eagle Island CDF, including:

- the most appropriate methodology for vertical expansion of the existing CDF,
- estimates of the additional containment volume which may be created, and
- estimates of the life expectancy of the Eagle Island CDF.

1.2 CONSTRUCTION FOR DREDGE SPOIL STORAGE

1.2.1 Historical

The Eagle Island CDF is owned by USACE. Historically the site was divided into two cells, a North Cell and a South Cell; however, as part of the recent improvement to the CDF, the north Cell was subdivided into two cells of approximately equal size. Therefore, the existing Eagle Island CDF currently consists of three cells; Cell 1, Cell 2, and Cell 3 with diked in areas of 220, 260 and 260 acres, respectively.

Dikes have been built incrementally over time to facilitate the containment of dredged material as needed. The elevation of the top of the existing dikes varies from about 25 feet msl to 27 feet msl.

1.2.2 Future Use

Based upon review of historical dredge records it is expected that approximately 1,000,000 cubic yards (cy) of dredged material will be delivered to Eagle Island on an annual basis. In addition to this work, USACE intends to dredge approximately 6.6 million cy of new material from reaches in the upper harbor as part of the Wilmington Harbor deepening project. Table 1.1 presents a summary of the expected dredge spoil volumes to be delivered to the Eagle Island CDF. Dames & Moore developed the information presented on Table 1.1 from previous work related to the Dredged Material Management Plan (DMMP) Study for Eagle Island.

Due to the current limited storage capacity and the need to create new capacity, a principal near term objective of the USACE is to establish the greater containment capacity for the Eagle Island CDF. This expansion in capacity must be sufficient to allow for periodic disposal of dredged materials as indicated on Table 1.1. To this end, the existing perimeter dike at elevation 25 feet to 27 feet msl is expected to be raised to elevation 38 feet msl by the year 2018.

Table 1.1 - Summary of Dredging Volumes

Dredge Window	Maintenance Volume (cy)	New Work Volume(cy)	Total Volume (cy)
99-2000	250,000	-	250,000
2000-01	951,000	-	951,000
2001-02	903,900		903,9000
2002-03	951,000	1,875,291	2,826,291
2003-04	1,201,454	1,875,270	3,076,724
2004-05	1,425,000	1,395,800	2,820,800
2005-06	1,417,375	904,825	2,322,200
2006-07	1,053,275	-	1,053,275
2007-08	1,329,445	-	1,329,445
2008-09	1,286,825	-	1,286,825
2009-10	994,400	-	994,400
2010-11	1,046,200	-	1,046,200
2011-12	1,425,200	-	1,425,200
2012-13	1,118,075	-	1,118,075
2013-14	994,400	-	994,400
2014-15	1,258,000	-	1,258,000
2015-16	1,213,400	-	1,213,400
2016-17	1,046,200	-	1,046,200
2017-18	1,269,400		1,269,400
2018-19	1,046200		1,046200

2019-20	1,213,400		1,213,400
2020-2021	1,258,000		1,258,000
Totals	**24,402,149**	**6,051,186**	**30,453,335**

Table 1.1 Continued

1.3 ORGANIZATION OF VOLUME II

This document is organized as follows:

- Section 1 provides a brief introduction and report organization.
- Section 2 provides a summary of the project purpose and its scope, including an abstract of the key technical activities that were undertaken by Dames & Moore.
- Section 3 includes a description of the field exploration, a summary of the currently available site information, and a summary of the current design approach used by USACE in the design of the CDF.
- Section 4 includes a description of the laboratory testing
- Section 5 provides a description of the site, including the subsurface stratigraphy, existing dike geometry, and material properties of the dike materials and the underlying natural marsh soils.
- Section 6 provides a summary of the findings of each element of Dames & Moore's work.
- Section 7 summarizes the work completed and provides recommendation for additional investigation and analyses.
- Section 8 provides references cited in the text.

1.4 LIST OF REPORT VOLUMES AND TITLES

This report is Volume II of a seven-volume submittal that comprises the Eagle Island Confined Disposal Facility Report. The volumes are as follows:

- Volume I is the primary report for this project entitled, "Eagle Island Management Plan". This plan summarizes the data and conclusions contained in Volumes II through VI and is the primary report provided.
- Volume III is a record of the stability calculations used in this Volume II.
- Volume IV contains the results of the field exploration conducted as part of the geotechnical evaluation of Eagle Island. It includes logs of borings, DPT soundings, CPT soundings, pore pressure dissipation tests, field vane shear test data, and the findings of earlier field work by USACE.
- Volume V contains the results of the laboratory tests conducted as part of the geotechnical evaluation of Eagle Island. The laboratory testing includes strength, index and consolidation testing conducted by Dames & Moore, as well as previous work conducted by USACE for this site. Laboratory testing to estimate the rate of consolidated of recently placed dredged sediments (column sedimentation tests) are also included.

- Volume VI is an evaluation of the settlement characteristics of the dredged material and foundation soils based on the computer program "Primary Consolidation, Secondary Settlement and Desiccation of Dredge Fill" (PSDDF).

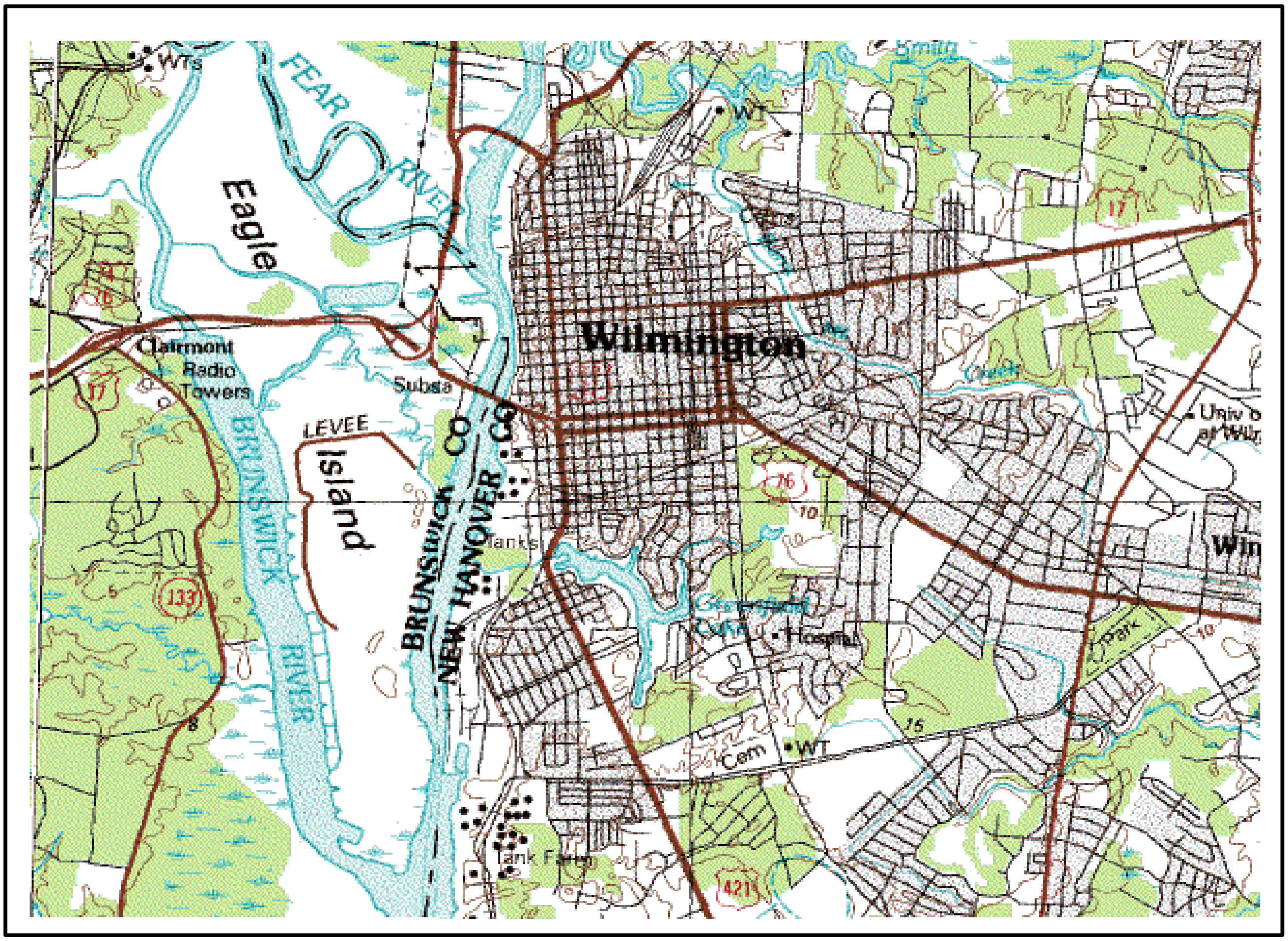
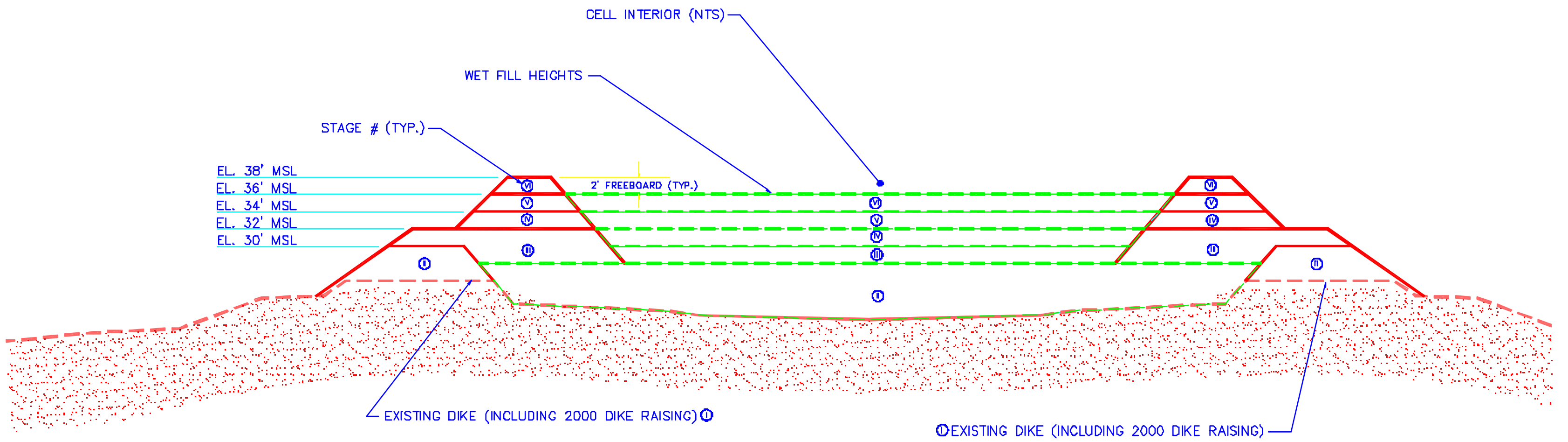


Figure 1.1 - Location Map

Figure 1.2 - Aerial Photo of Site Plan



LEGEND

- = Approximate Groundwater Level
- = SM, SP (Sands – Upper, Middle, Lower)
- = MH/OH (Upper Organic Silt)
- = MH/CH/SP (Lower Clays w/ thin Sand Seams)

SCALE: N.T.S

US ARMY CORPS OF ENGINEERS
WILMINGTON DISTRICT

TITLE: CROSS–SECTION

PROJECT: EMBANKMENT STABILITY ANALYSIS
EAGLE ISLAND MANAGEMENT PLAN

DATE: MAY 2001

Figure 1.3

D&M JOB NO.

2.0 OBJECTIVE AND SCOPE OF THIS WORK

2.1 OBJECTIVE

The objective of the work reported herein is to complete field exploration, laboratory testing, and engineering analyses to:

- evaluate the existing stability of the dikes at the Eagle Island CDF, and
- evaluate alternatives for vertical expansion of those dikes.

2.2 SCOPE

Dames & Moore has performed geotechnical engineering studies with the broad scope of work identified below:

- executed a geotechnical investigation, completing field exploration and laboratory testing sufficient to characterize the occurrence of subsurface materials and the mechanical characteristics of those materials;
- provided geotechnical evaluations of existing dike stability (including perimeter and cross dikes),
- completed geotechnical engineering analyses to support recommendations for future vertical dike expansion;
- defined requirements for monitoring dike stability as the dikes are built; and,
- summarized all data, findings and design recommendations in a technical report submitted to USACE.

The scope of the field exploration and laboratory testing are summarized in Sections 3.0 and 4.0, respectively. The scope of the geotechnical evaluations of dike stability is discussed in Section 6.0. Requirements for monitoring dike stability as the dikes are raised are discussed in Section 6.0

3.0 GEOTECHNICAL FIELD EXPLORATION

3.1 WORK BY OTHERS AS A BASIS FOR THIS WORK

3.1.1 Unreviewed 1970's Work By USACE

Snipes 1999 reports that a subsurface investigation was conducted at Eagle Island in the mid-1970's by the Wilmington District's Foundation and Materials Section. The work included engineering borings, laboratory testing, and geotechnical analysis. A test fill embankment was constructed near the southwest end of the South Cell using borrow from the disposal area. The embankment was raised until intentional failures occurred. It was concluded that a first-stage embankment could be constructed alongside the existing dike (with 3:1 side slopes) using local surficial borrow materials.

Dames & Moore has not reviewed the above-described work. The work resulted in the suggestion that a stepped-in dike concept could be built. Long-term dike height was not addressed.

3.1.2 USACE 1999

Twenty-two borings were taken at Eagle Island in 1996-98 for dike design purposes. Seven of these (EI-96-1 through EI-96-7) were taken in April-May 1996 to obtain information for the design of both the North and South Cell Dikes. However, the subsequent technical analysis that followed focused upon the South Cell design only. The work noted that that several failures have occurred in the past resulting in quantities of dredged material flowing into the marsh and river.

The above studies by USACE (USACE, 1999) concluded that:

- *"the height of the dikes have reached a maximum whereby the foundation cannot safely support any more load in some areas;"*
- *that stability analyses were limited by a lack of data upon which to base analyses.*

Dames & Moore peer reviewed USACE 1999 (Dames & Moore 2000) and provided the below-listed conclusions with respect to embankment stability: (i) that there is insufficient data (i.e., subsurface information, *in-situ* testing, and laboratory data, etc) to conclude that dike design to elevation +32 feet msl would be safe (i.e., long term factor of safety of 1.3 or greater), a finding consistent with the USACE findings; and (ii) that USACE 1999's general approach to design of the dikes is technically sound, utilizing a sequence of analyses both appropriate to the problem at hand and consistent with industry practice.

Based upon the above, Dames & Moore 2000 recommended that USACE develop additional information regarding the occurrence of subsurface materials and the mechanical characteristics of those materials, believing that elimination of these design uncertainties could enable development of more cost effective embankment designs.

3.2 OBJECTIVES OF THIS FIELD EXPLORATION

An extensive geotechnical field exploration program was undertaken during June-August 2000. The field exploration was undertaken with two objectives, namely:

- to establish the mechanical characteristics (strength and compressibility) of the soil, particularly the softer fine grained soils which profoundly affect embankment performance; and;
- to establish the occurrence of soils beneath the embankments.

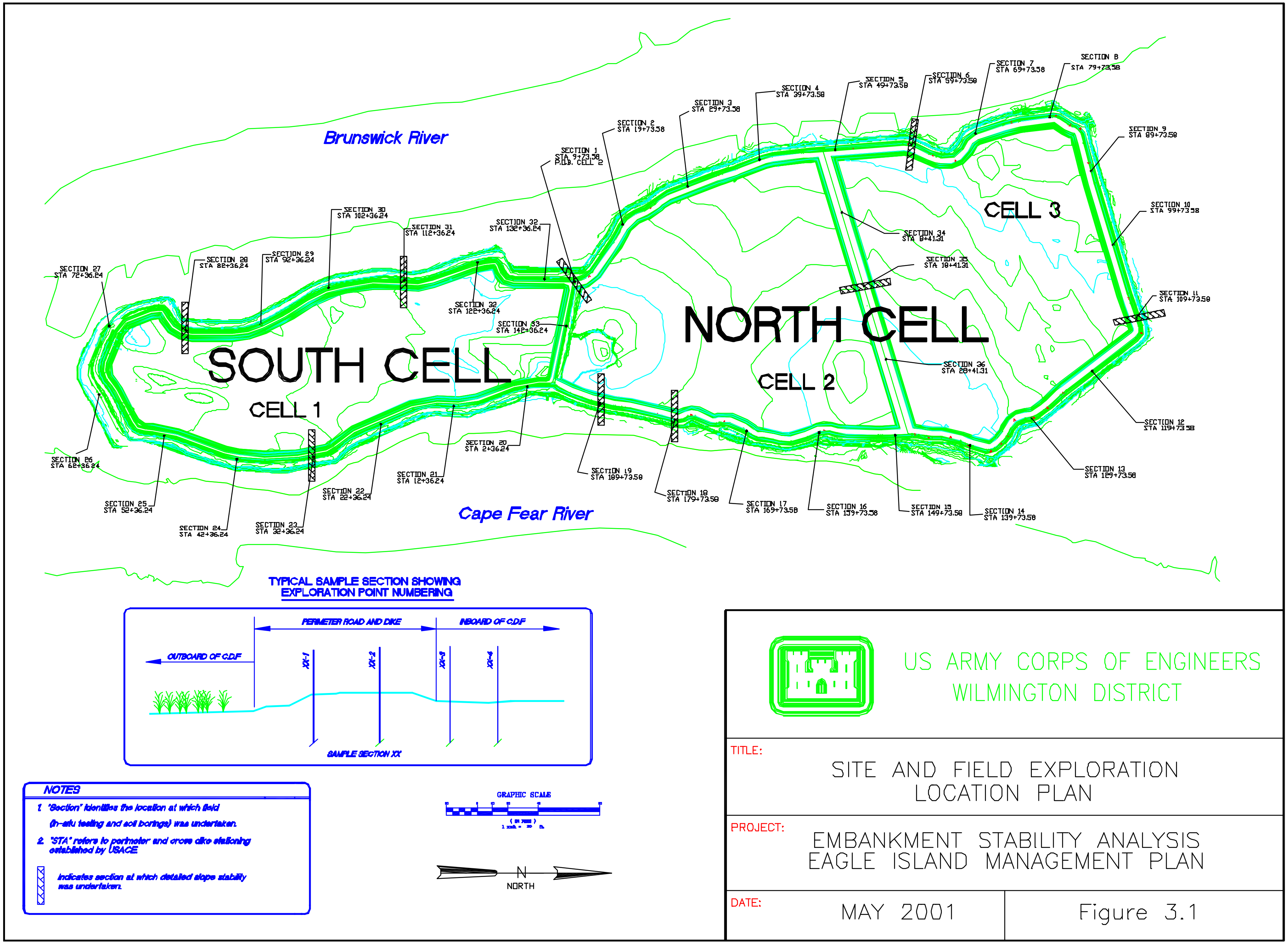
3.3 ORGANIZATION OF THE FIELD WORK

3.3.1 Ordering and Numbering the Exploration Points

At the outset of the field exploration, relatively little subsurface information existed. Earlier analyses by USACE (USACE, 1999) determined that unless it could be demonstrated that the subsurface materials held more strength, that there would be little chance to raise the dikes to the heights required by the planned dredging and that a new CDF would have to be developed.

The embankments at the CDF are nearly seven miles in length, and known to be constructed on soft ground. For the purposes of planning the field exploration, the following was undertaken: (i) stationing along the alignment of the dikes was established, using survey points established by USACE; and (ii) exploration "sections" (36 in total) were established at approximately 1,000 foot intervals.

Figure 3.1 presents a view of the site as it was viewed for the purposes of planning the field exploration. As may be seen from the key to this figure, borings and soundings (either DMT, CPT, or both) were undertaken at regular locations on each section. For example, CPT Sounding 2-1 would reference a sounding completed at Section 2 in the 1 position- the location most outboard of the section. CPT Sounding 2-3 would reference a sounding at the top center of the embankment.



Brunswick River
Cape Fear River
SOUTH CELL
CELL 1
NORTH CELL
CELL 2
CELL 3
SECTION 1 STA 9+73.58 P.I.D. CELL 2
SECTION 2 STA 19+73.58
SECTION 3 STA 29+73.58
SECTION 4 STA 39+73.58
SECTION 5 STA 49+73.58
SECTION 6 STA 59+73.58
SECTION 7 STA 69+73.58
SECTION 8 STA 79+73.58
SECTION 9 STA 89+73.58
SECTION 10 STA 99+73.58
SECTION 11 STA 109+73.58
SECTION 12 STA 119+73.58
SECTION 13 STA 129+73.58
SECTION 14 STA 139+73.58
SECTION 15 STA 149+73.58
SECTION 16 STA 159+73.58
SECTION 17 STA 169+73.58
SECTION 18 STA 179+73.58
SECTION 19 STA 189+73.58
SECTION 20 STA 2+36.24
SECTION 21 STA 12+36.24
SECTION 22 STA 22+36.24
SECTION 23 STA 32+36.24
SECTION 24 STA 42+36.24
SECTION 25 STA 52+36.24
SECTION 26 STA 62+36.24
SECTION 27 STA 72+36.24
SECTION 28 STA 82+36.24
SECTION 29 STA 92+36.24
SECTION 30 STA 102+36.24
SECTION 31 STA 112+36.24
SECTION 32 STA 122+36.24
SECTION 32 STA 132+36.24
SECTION 33 STA 142+36.24
SECTION 34 STA 8+41.31
SECTION 35 STA 18+41.31
SECTION 36 STA 28+41.31
TYPICAL SAMPLE SECTION SHOWING EXPLORATION POINT NUMBERING
PERIMETER ROAD AND DIKE
INBOARD OF CDF
OUTBOARD OF CDF
XX-1
XX-2
XX-3
XX-4
SAMPLE SECTION XX
NOTES
1. "Section" identifies the location at which field (in-situ testing and soil borings) was undertaken.
2. "STA" refers to perimeter and cross dike stationing established by USACE
indicates section at which detailed slope stability was undertaken.
GRAPHIC SCALE
NORTH
US ARMY CORPS OF ENGINEERS
WILMINGTON DISTRICT
TITLE:
SITE AND FIELD EXPLORATION LOCATION PLAN
PROJECT:
EMBANKMENT STABILITY ANALYSIS EAGLE ISLAND MANAGEMENT PLAN
DATE:
MAY 2001
Figure 3.1

3.3.2 Sequence of the Field Work

The field exploration program included Cone Penetration Test (CPT) soundings, Standard Penetration Test (SPT) borings, Marchetti flat plate dilatometer (DMT) soundings, field vane shear (FVS) tests, disturbed soil sampling, undisturbed (Shelby tube) sampling, and completion of pore pressure dissipation tests. The exploration was conducted with the following general sequence of work:

- CPT soundings have a well-documented record of effectiveness in evaluating subsurface stratigraphy and were undertaken first to provide an overall indication of subsurface conditions, and, secondly, to provide an indication of the strength and compressibility of the subsurface materials. Based upon the indications of the CPT soundings, stratigraphic profiles (comparing CPT signatures of cone tip resistance, q_c, sleeve friction, f_s, and dynamic pore pressure, u, from sounding to sounding) were established about the site. These profiles were used to identify the thickness and elevations of identifiable layers of soil across the site. Table 3.1 (following page) provides the locations and depths of all CPT's that were taken.
- With the indications of the subsurface profiles based on the CPT results, a focused program of SPT borings, DMT soundings, FVS tests, disturbed soil sampling, Shelby tube sampling, and pore pressure dissipation tests was established. This latter program was intended to be focused at completing *in situ* testing and obtaining samples for laboratory testing such that the mechanical characteristics of the various strata could be developed. Table 3.2 summarizes the scope of SPT borings, DMT soundings, FVS tests, disturbed soil sampling, Shelby tube sampling, and pore pressure dissipation tests which were planned based upon the indications of the CPT soundings.

As may be seen from review of Table 3.1, a large number of CPT soundings were completed. A total of 7,807 lineal feet of subsurface were explored in 149 separate CPT soundings.

Table 3.1 - Locations and Depths of CPT's Soundings Taken

Station	Location	Depth (Feet)	Station	Location	Depth (Feet)	Station	Location	Depth (Feet)
9+73.58	1-1	55	19+73.58	2-1	52	29+73.58	3-1	52
	1-2	61		2-2	61		3-2	58
	1-3	61		2-3	48		3-3	48
	1-4	60		2-4	48		3-4	48
	1-5	55						
39+73.58	4-1	51	49+73.58	5-1	58	59+73.58	6-1	56
	4-2	62		5-2	46		6-2	50
	4-3	48		5-3	49		6-3	50
	4-4	51		5-4	44		6-4	52
69+73.58	7-1	45	79+73.58	8-1	53	89+ 73.58	9-1	45
	7-2	48		8-2	47		9-2	49
	7-3	40		8-3	44		9-3	45
	7-4	45		8-4	42		9-4	36
	7-5	36					9-5	35
99+73.58	10-1	51	109+73.58	11-1	44	119+73.58	12-1	44
	10-2	46		11-2	40		12-2	43
	10-3	54		11-3	44		12-3	45
	10-3A	59		11-4	41		12-4	38
	10-4	52		11-5	42		12-5	38
	10-5	55						
129+73.58	13-1	47	139+73.58	14-1	44	149+73.58	15-1	50
	13-2	50		14-2	44		15-2	58
	13-3	45		14-3	44		15-3	58
	13-3A	45		14-4	44		15-4	52
	13-4	38		14-5	44		15-5	50
	13-5	41						
159+73.58	16-1	68	169+73.58	17-1	58	179+73.58	18-1	66
	16-2	41		17-2	55		18-2	59
	16-3	68		17-3	65		18-3	65
	16-4	52		17-4	62		18-4	62
				17-5	57		18-5	60
189+73.58	19-1	64	2+36.24	20-1	53	12+36.24	21-1	51
	19-2	65		20-2	53		21-2	51
	19-3	51		20-3	61		21-3	61
	19-4	47		20-4	54			
	19-5	47		20-5	47			

Table 3.1 (continued) Locations and Depths of CPT's Soundings Taken

Station	Location	Depth (Feet)	Station	Location	Depth (Feet)	Station	Location	Depth (Feet)
22+36.24	22-1	57	32+36.24	23-1	65	42+36.24	24-1	61
	22-2	56		23-2	66		24-2	42
	22-3	64		23-3	69		24-3	69
52+36.24	25-1	58	62+36.24	26-1	58	72+36.24	27-1	53
	25-2	55		26-2	58		27-2	53
	25-3	61		26-3	63		27-3	64
82+36.24	28-1	57	92+36.24	29-1	55	102+36.24	30-1	51
	28-2	62		29-2	55		30-2	55
	28-2A	53		29-3	61		30-3	64
	28-3	64						
112+36.24	31-1	51	122+36.24	32-1	45	142+36.24	33-1	66
	31-2	51		32-2	46		33-2	49
	31-3	58		32-3	60		33-3	52
							33-4	52
							33-5	57
							33-5A	42
8+41.31	34-1	49	18+41.31	35-1	55	28+41.31	36-1	51
	34-2	64		35-1A	40		36-2	57
	34-3	42		35-2	69		36-3	53
				35-2A	36			
				35-3	46			

Table 3.2 - Summary of the SPT, Field Vane Shear, Dilatometer, and Pore Pressure Dissipation Testing Undertaken Based Upon the Indications of the Cone Penetrometer Soundings

Area	Sample	Station	SPT	Depth (feet)	FVS	Depth (feet)	DMT	Depth (feet)	CPT with PPD	Depth (feet)
5	18-3	179+73	x	30			x	60		
5	17-1	169+73	x	15			x	50		
6	22-2	22+36	x	20	x	20				
6	22-3	22+36	x	23			x	40		
7	26-2	62+36	x	18	x	30	x	55		
1	2-2	19+73			x	25	x	55		
2	5-2	49+73	x	20	x	20	x	45		
11	35-1	18+41	x	15	x	20	x	40	x	30-40
11	35-2	18+41			x	18	x	40	x	35-45
10	33-5	142+36	x	15	x	15	x	40	x	35-45
8	28-2	82+36			x	20	x	55	x	45-55
9	30-1	102+36	x	10	x	22	x	50		
4	13-3	129+73	x	18			x	40	x	30
3	10-3	99+73			x	30	x	40	x	20
TOTAL			10	184	10	220	13	610	6	

1. SPT refers to Standard Penetration Test boring after ASTM D 1586. Undisturbed samples were recovered from SPT borings by pushing 3-inch diameter Shelby tubes, after ASTM D 1587
2. “FVS” refers to “field vane shear” test after ASTM D 2573
3. “DMT” refers to “Marchetti flat plate dilatometer test” sounding after ASTM D18.02.10 (1986)
4. “CPT” refers to static “cone penetrometer test” sounding, after ASTM D 3441
5. “CPT with PP” refers to “cone penetrometer test, with pore pressure dissipation”

3.4 DESCRIPTION OF THE FIELD EXPLORATION

3.4.1 Duration

The field exploration was undertaken over the June-August 2000.

3.4.2 Key Subcontractors

The engineering borings and field vane shear tests were completed by Mid-Atlantic Testing, Inc. under surveillance of Dames & Moore. The CPT, DMT, and pore pressure dissipation testing was provided by Fugro Geosciences, Inc.

3.4.3 Soil Borings

A total of 11 soil borings were performed at various locations throughout the site. All borings were performed using hollow stem auger drilling techniques by an all terrain drill rig to advance the borehole. Borings ranged in depth from 10 to 45 feet below existing grade, drilling an aggregate of 184 lineal feet of borings. Both disturbed and undisturbed samples were obtained from the borings, which were then used to test various material properties in the laboratory. All borings were backfilled to the surface upon completion. Boring logs are provided in Volume IV of this report.

Undisturbed samples of the fine-grained materials were recovered from engineering borings in thin wall (Shelby tube) samplers for laboratory testing to evaluate strength, stress history, and compressibility (including time rates of compressibility). A total of 28 undisturbed samples were recovered at 11 boring locations, as is summarized on Table 3.3. Note that three of the samples had low recovery and were unusable (see the note to Table 3.3).

Table 3.3 - Summary of Shelby Tube Samples

Area	Boring Location	Station Reference	Shelby Samples	Depth Interval (ft)
5	17-1	169+73	2	15-20
6	22-3	22+36	2	23-28
7	26-2	62+36	3	25-35
1	2-2	19+73	3	35-45
2	5-2	49+73	3	28-38
11	35-1	18+41	3	2-15
10	33-5	142+36	3	25-35
8	28-2	82+36	3	12-22
9	30-1	102+36	3	12-22
9	30-1	102+36	1	30-33
9	30-1	102+36	1	40-43
4	13-3	129+73	1	20-25
		Total	28	

Note: The following samples had low recovery and were unusable: Boring 5-2 at 33' to 35', Boring 5-2 at 36' to 38', and Boring 26-2 at 33' to 35'

3.4.4 Static Cone Penetrometer Soundings:

Measurements of *in-situ* soil strength were obtained by completing static cone penetrometer test (CPT) soundings to depths of 30 to 45 feet bgs. The soundings were performed using a cone penetrometer equipped with a friction mantle and equipped to record tip pore pressure measurement. The CPT soundings utilized equipment and methods conforming to those described in ASTM D 3441. The CPT was utilized to provide a continuous record of the subsurface soil types, and to provide an *in-situ* measurement of the strength and compressibility of the soils.

Exploratory sections of CPT soundings were placed at 1,000-foot centers along the dike alignment at all practical locations. Results are presented in Volume IV of this report.

3.4.5 Pore Pressure Dissipation Tests

Pore pressure dissipation tests pore pressure dissipation tests were performed to evaluate soil permeability by the measurement of pore pressure decay with time. These tests are being used primarily to provide an estimate of horizontal permeability. One-dimensional consolidation tests were utilized to estimate vertical permeability. The pore pressure dissipation test data are reported in Volume IV, Field Data.

3.4.6 Dilatometer Testing

Dilatometer (DMT) soundings were completed using a Marchetti dilatometer in general accordance with ASTM D18.02.10 (1986), Suggested Method for Performing the Flat Dilatometer Test. The DMT soundings were performed both to provide *in-situ* measurements of soil strength and to support evaluation of stratigraphy. The results of the DMT soundings are presented in Volume IV, Field Data.

3.4.7 Field Vane Shear Tests

Both *in situ* Field Vane Shear Tests (ASTM D 2573) and Laboratory Miniature Vane Shear Tests (ASTM 4648) were performed. The combination of the two tests offers a fast and economical method to support the data being developed by the other tests. The Field Vane Shear data is reported in Volume IV, Field Data, and the Laboratory Miniature Vane Shear Test results are presented in Volume V, Laboratory Tests.

4.0 LABORATORY TESTING AND MECHANICAL SOIL PROPERTIES

4.1 GENERAL

4.1.1 Description of the Soil Layers

The laboratory-testing program was established upon completion of the field exploration. The testing was focused toward establishing the strength and compressibility of those soils which would be key to the performance of embankments.

When the laboratory-testing program was established, the subsurface materials were generalized to occur in five strata, as summarized on Table 4.1.

Table 4.1 - Generalized Subsurface Conditions Used in Planning Laboratory Testing

Layer	Depth (feet, bgs)		Description
	From	To	
1	0	4	Upper Sand (SP): Loose fine sand (dike sand)
2	4	18	Upper Organic Silt (MH): soft to medium stiff, highly plastic organic silt (embankment fill and original marsh surface)
3	18	24	Middle Sands (SP): Brown fine to medium sand (medium dense) interlayered with silt and clay
4	24	38	Lower Fines (CH with MH and SP): interlayered highly plastic medium stiff silt and clay, with lenses of medium dense fine sand
5	38	60	Lower Sands : medium dense to dense fine to medium sand with lenses of highly plastic medium stiff silt and clay

Key to the performance of the dikes is Layer 2 Upper Organic Silt. These soils were shown by the field vane shear, DMT and CPT to locally be very soft. In consideration of the importance of these soils to embankment performance, laboratory testing focused on establishing the strength and compressibility of these soils. Results of all laboratory testing are included in Volume V.

4.1.2 Summary of the Testing to Determine Strength and Compressibility

The testing undertaken to establish strength and compressibility is summarized on Table 4.2 (following page).

Table 4.2 - Testing to Determine Strength and Compressibility

Layer	Boring	Depth (feet, bgs)	Consolidation ASTM D2435	Direct Shear ASTM D3080	Laboratory Vane Shear ASTMD4648	'R' Test ASTM D2435
2	28-2	12-14			1	1
2		16-18	1	1		
2		20-22				1
2	30-1	12-14			1	
2		16-18	1	1		
2		20-22		1	1	1
4		41-43	1		1	
4	33-5	25-27			1	1
4		33-35	1		1	
2	17-1	15-17		1	1	
2		18-20	1	1		1
2		8-10			1	
2		13-15		1	1	
4	5-2	28-30	1		1	
2	13-3	22-24			1	
4	2-2	35-37			1	
4		39-41			1	
4		43-45	1		1	
4	26-2	25-27			1	
4	22-3	23-25	1		1	
		Total	**7**	**6**	**16**	**5**

4.1.3 Summary of the Index Testing

In order to confirm field classifications and obtain data needed to estimate mechanical soil properties from published correlations, testing to estimate index properties was undertaken. The results are summarized in Table 4.3. Like the strength testing, in consideration of the importance of the Layer 2 soils to embankment performance, laboratory testing focused on index characteristics of these soils.

Table 4.3 - Summary of Index Testing

Layer	Boring	Depth (feet)	Moisture/ Density	Atterberg Limits	Grain size with Hydrometer
2	28-2	12-14	1	1	1
2		16-18	1	1	
2		20-22	1	1	1
2	30-1	12-14	1	1	
2		16-18	1	1	1
2		20-22	1	1	1
4		41-43	1	1	
4	33-5	25-27	1	1	1
4		33-35	1	1	1
2	17-1	15-17	1	1	
2		18-20	1	1	1
2		8-10	1	1	
4	5-2	28-30	1		
2	13-3	22-24	1	1	1
4	2-2	35-37	1	1	
4		39-41	1	1	
4		43-45	1	1	1
4	26-2	25-27	1	1	
4	22-3	23-25	1	1	1

4.2 INDEX PROPERTIES

Tables 4.4 and 4.5 summarize the results of plasticity and moisture data for Layer 2 and Layer 4 soils.

Table 4.4 - Summary of the Index Testing of the Layer 2 Upper Organic Silts

Location	Boring	Depth	Dry Density (lb/ft^3)	Moisture Content (%)	Liquid Limit	Plasticity Index	USCS
30	1	18	46	90	96	39	CH/MH
30	1	17	51	78	96	39	CH/MH
28	2	17	37	125	143	79	CH/MH
17	1	19	47	90	101	46	CH/MH
22	3	24	59	65	83	42	CH/MH
35	1	9	NM	NM	102	42	CH/MH
35	1	14	32	150	159	73	CH/MH
30	1	21	40	112	129	71	CH/MH
30	1	13	32	148	96	47	CH/MH
28	2	21	35	148	143	81	CH/MH
26	2	26	NM	NM	109	60	CH/MH
29	2	13	NM	NM	148	89	CH/MH
22	3	24	NM	NM	83	42	CH/MH
13	3	23	NM	NM	111	52	CH/MH
17	1	16	46	95	124	58	CH/MH

"NM" indicates "not measured"

Table 4.5 - Summary of the Index Testing of the Layer 4 Lower Clays

Location	Boring	Depth	Dry Density (lb/ft^3)	Moisture Content (%)	Liquid Limit	Plasticity Index	USCS
33	5	34	39	112	141	75	CH/MH
30	1	42	27	169	131	65	CH/MH
30	1	43	NM	NM	131	65	CH/MH
2	2	40	NM	NM	182	73	CH/MH
2	2	36	NM	NM	137	69	CH/MH

"NM" indicates "not measured"

As may be seen from review of the above tables, the Layer 2 and Layer 4 soils exhibit variable plasticity. Though both may be classified as "OH" by the Unified Soils Classification System", the Layer 4 soils have characteristically greater plasticity (both as liquid limit and plasticity index) than the Layer 2 soils.

Mechanical and hydrometer analyses after ASTM D 422 were undertaken on soils representative of Layers 2, 3, and 4. The hydrometer analyses indicate that both the Layer 2 and Layer 4 soils include a large amount of clay-sized materials, typically in excess of 40% particles by weight finer than 2µ.

4.3 STRESS HISTORY AND CONSOLIDATION CHARACTERISTICS

4.3.1 General

A total of seven (7) one-dimensional consolidation tests after ASTM D 2435 were performed in order to estimate the consolidation characteristics of the encountered cohesive soils. These tests were performed on undisturbed samples taken from the Layer 2 upper organic silt and Layer 4 lower fines.

4.3.2 Consolidation Characteristics

The strain-based compression ratio ($CR = (C_c)/1+e_0$) values for the Layer 2 organic silt range from 0.24 to 0.54, and from 0.28 to 0.36 for the Layer 4 lower fines.

The coefficient of consolidation (c_v) was calculated for several loading increments of each consolidation test. Values of c_v ranged from 0.54 square feet per day (ft^2/day) at a load of 800 pounds to 0.01 ft^2/ day at a load of 6,400 pounds for the Layer 2 upper organic silt. For the Layer 4 lower fines, C_v values ranged from 0.18 ft^2/ day at a load of 1,600 psf to 0.02 ft^2/ day at a load of 4,800 psf.

Table 4.6 - Summary of the Consolidation Characteristics of the Layer 2 and Layer 4 Soils

Boring	Location	Depth/ Layer	Cv for load increments (ft2/day)				Compression Ratio
			800 psf	1600 psf	3200 psf	6400 psf	C=Cc/(1+eo)
33	5	34/4	nm	0.18	0.03	0.02	0.28
30	1	42/4	nm	0.30	0.07	0.04	0.33
30	1	17/2	0.54	0.26	0.1	0.07	0.30
28	2	17/2	nm	0.02	0.01	0.01	0.30
5	2	29/4	nm	0.31	0.05	0.02	0.36
17	1	19/2	0.60	0.59	0.04	0.02	0.54
22	3	24/2	nm	0.57	0.1	0.07	0.24

"nm" indicates "not measured"

4.3.3 Stress History

The results of the one-dimensional consolidation tests were also used to estimate the stress history of the *in-situ* soils. First the pre-consolidation pressure (P_C') was calculated using a construction technique first developed by Cassagrande. Using a calculated effective *in-situ* stress and the pre-consolidation pressure an over-consolidation ratio (OCR) can be estimated. Table 4.6 summarizes these data.

Table 4.7 - Summary of the Stress History of the Layer 2 and Layer 4 Soils

Boring	Location	Layer	Depth	Initial Void Ratio	Moisture (%)	Dry Density (pcf)	Maximum Past Pressure (psf)	Overburden (psf)
33	5	4	34	3.35	112	39	1600	1800
30	1	4	42	5.33	169	27	1600	2500
30	1	2	17	2.29	78	51	4500	1600
28	2	2	17	3.59	125	37	1200	1200
5	2	4	29	5.7	333	25	1200	1200
17	1	2	19	2.56	90	47	3300	1600
22	3	2	24	1.87	65	59	1800	1800

As may be seen from review of Table 4.7, individual consolidation test data provide somewhat confusing results. In particular, Boring 30-1 (42 feet) indicates that the Layer 4 material is substantially underconsolidated. Boring 30-1 (17 feet) indicates the Layer 2 soil is heavily over consolidated. The remainder of the data indicates that the Layer 2 and Layer 4 soils are likely normally consolidated under their current vertical effective stress. Dames & Moore believes that, for the purpose of general analyses, the Layer 2 and Layer 4 soils are indeed normally consolidated.

4.4 STRENGTH PROPERTIES

4.4.1 General

Soil strength parameters have been evaluated based on laboratory test results, *in-situ* testing and standard correlations between soil index parameters and previously published research data. An important concept in the analysis of an embankment constructed over soft foundation soils is the difference between drained and undrained behavior of various soils during shear and how strength can vary with time as a result of changes in pore pressure within the soil and the effective stresses which act between soil particles. Whether a soil layer behaves as drained or undrained depends on its internal drainage conditions or permeability, the drainage conditions at its boundary and the rate at which shear load is applied.

4.4.2 Drained Strength Parameters

The Layer 1 soils are, of course, unsaturated. Relatively free-draining, sandy soils of Layers 3 and 5 will exhibit only drained behavior under the relatively slow loading rate applied as the dredge spoil dike is built. The drained shear strength of a sandy soil is measured as an angle of internal friction, Ø. At the Eagle Island site the upper, middle and lower sand layers behave as purely frictional or fully drained materials due to their relatively high *in-situ* permeability.

In consideration of the characteristics of these three layers, it is believed that data obtained from the CPT and DMT soundings best characterizes their mechanical characteristics.

Layer 1 Upper Sands

A friction angle Ø = 34 degrees was selected for upper sands based primarily on direct measurements from the DMT soundings, as well as our experience with similar material on other projects.

Layer 3: Middle Sands and Silts

The middle sands classify as clayey fine to medium sands. The sands are of medium dense to dense consistency , as characterized by cone tip resistance (q_c) in the range of 50 tons per square foot (tsf) to 75 tsf. Published correlations for CPT and DMT data with these sands indicate a range of Ø' values from 35 degrees.

Layer 5: Lower Sands

The lower sands typically classify as dense to very dense clean to clayey fine sands (SPT N typically over 30 blows per foot) with occasional zones of high plasticity clay. The strata are characterized by high cone tip resistance (q_c) in excess of 50 tsf. Published correlations for CPT and DMT data with these sands indicate a range of Ø' values from 34 to 38 degrees. An average value of Ø = 38 degrees is considered as representative of *in-situ* strength for the lower sand stratum.

4.4.3 Undrained Soil Strength Parameters

The undrained shear strength (c_u) of a cohesive soil is not a fundamental soil property but varies with boundary conditions, rate of loading, effective confining or overburden stress level, and past stress history. For undrained conditions to exist in the field, load must be applied to the soil sufficiently rapidly so that drainage and volume change of the soil cannot take place. In comparison to drained conditions, the undrained shear strength represents a lower bound limit to clay shear strength assuming that soil pore pressures and effective stresses are known or can be adequately predicted at any time immediately prior to a load increment being applied to the soil.

The undrained shear strength (c_u) is commonly normalized with respect to vertical effective overburden stress (p_o') at the depth where soil strength is measured. Published correlations by Jamiolkowski and Mesri for normally consolidated clays indicate c_u/p_o' to be 0.22for clays with moderate to high plasticity where the undrained shear strength c_u corresponds to the value measured by the direct simple shear test. There is a generally observed trend of increasing c_u/p_o' ratio with increasing plasticity index (PI). Data published by Jardine and Hight based on back analyses of embankment failures on soft clay foundation soils confirms this trend with c_u/p_o' around 0.30 for high plasticity clays.

The undrained shear strength of the upper organic silt and the sandy lean clay can also be estimated from piezocone data using a correlation developed by Mayne and Chen (1993). This correlation uses the following formula to estimate undrained shear strength (c_u)

$$c_u = \frac{q_t - u_{bt}}{N_{qu}}$$

where:

q_t is the cone tip resistance,
u_{bt} is the pore pressure measured behind the tip, and
N_{qu} is the cone-bearing factor.

N_q was set equal to 8.5. This data is presented in Appendix B.

<u>Layer 2Upper Organic Silts</u>

Based upon the indications of the laboratory testing, the Layer 2 soils are considered to be normally consolidated. Field vane shear and laboratory vane shear data indicate an average undrained shear strength of about 400 pounds per square foot (psf), as summarized on the following Tables 4.8 and 4.9

Table 4.8 - Summary of Field Vane Shear Tests, Layer 2

Boring	Depth (Feet)	Peak Shear Strength (pounds per sq. foot)	Remolded Strength (psf)	Sensitivity (Undist/Remolded)
2-2	25	471.2	157.1	3.0
5-2	20	366.5	117.8	3.1
10-3	30	392.7	222.5	1.8
22-2	20	523.6	209.4	2.5
26-2	30	589.0	65.5	9.0
28-2	20	431.9	104.7	4.1
30-1	22	248.7	157.1	1.6
35-1	20	248.7	104.7	2.4
35-2(2)	19	733.0	170.2	4.3
33-5	15	196.3	52.4	3.7
Average		**420.16**	**136.14**	**3.1**

Table 4.9 - Summary of Laboratory Vane Shear Tests, Layer 2

Boring	Depth (feet)	Laboratory Vane Shear Strength (psf)
13-3	23	265
17-1	16	344
22-3	24	649
26-2	26	212
28-2	13	546
30-1	13	678
30-1	21	339
33-5	26	215
35-1	9	387
35-1	14	141
	Average	**378**

As may be seen from review of the individual test data, the Layer 2 soils include a wide range of soil strengths. The field vane shear strength data range from about 250 psf to 600 psf, while the laboratory vane shear strengths range from about 140 psf to 680 psf. These data are consistent with a wide range of measured shear strength for Layer 2 from both the CPT and DMT soundings.

The direct shear test data suggest an undrained shear strength of about 400 psf and an angle of friction of about 8 degrees.

Assuming an average stress ratio of about 0.25, as suggested by the previously discussed published data, the Layer 2 clays might be expected to have undrained shear strength in the range 400 psf to 600 psf..

In consideration of available data, Dames & Moore has utilized the following relationship for undrained strengths in the Layer 2 upper organic silts:

- Layer 2 soils consolidated beneath the existing embankments: c = 400 psf Ø= 5 degrees
- Layer 2 outside the existing embankments: c = 300 psf Ø= 5 degrees

Layer 4 Lower Organic Silts

The lower organic silts are similar in nature to the Layer 2 upper organic silts, though of greater strength and greater plasticity. Table 4.10 summarizes the laboratory vane shear test data for Layer 4.

Table 4.10 - Summary of Laboratory Vane Shear Tests, Layer 4

Boring	Depth (feet)	Laboratory Vane Shear Strength (psf)
30-1	42	574
33-5	34	333
2-2	40	664
5-2	29	753
	Average	**581**

The above data are generally consistent with that developed for Layer 2, with greater shear strengths developing from greater consolidation pressures.

The Layer 4 soils are shown by the CPT and DMT soundings to often include finely layered strata of clays, silts and sands. In consideration of the above data and the indications of the CPT and DMT soundings, the following was conservatively chosen to describe the strength of the Layer 4 soils:

c = 400 psf Ø = 15 degrees

4.4.4 Soil Permeability Characteristics

The permeability characteristics of the in-situ soils were estimated in two ways:

- by a total of 11 dissipation tests using the CPT; and
- calculated from consolidation test data.

CPT dissipation tests provide less reliable estimates of vertical permeability. Dissipation tests are a measure more of horizontal rather than vertical permeability. The permeability of the Layer 2 and Layer 4 soils may be estimated to be about 5E-8 cm/sec based upon on laboratory measured values obtained from one-dimensional consolidation tests.

4.4.5 Compaction Characteristics of Surficial Embankment Soils

Six (6) compaction (Standard Proctor, ASTM D698 94) tests were run on bulk samples collected from the existing embankments. The samples were taken from zero (0) to three (3) feet below existing grade.

The soils recovered were characteristically fine grained, classified as MH/OH by the USCS. The maximum dry density ranged from 101pounds per cubic foot (pcf) to 54 pcf. The optimum moisture content ranged from 19 to 63 percent. The percent material passing the U.S. No. 200 sieve ranged from 31 percent to 97 percent.

These tests are summarized on Table 4.11 (following page). Results of individual tests can be found in Appendix B, Laboratory Test Results (Index Tests).

Table 4. 11 - Summary of Compaction Test Data of Existing Embankment Soils (ASTM D 698-94)

Location	Depth (feet)	USCS	Liquidity LL	Liquidity PI	Natural Moisture	Percent P#200	Opt. Dry Density	Optimum Moisture (%)
Cell 3, East Side	2	MH	84	47	37	63.3	101	19
Cell 1, West Side	1	MH	173	89	109	97	58	63
Cell 2, East Side	3	SM	NP	NP	5	31	94	23
Cell 2, West Side	2	MH	151	68	62	96	54	69
Cell 3, West Side	2	MH	139	63	89	95	69	47
Cell 1, East Side	1	MH	156	70	86	97	61	52

Notes:

1. “P#200” indicates “percent by weight finer than the US #200 sieve”
2. “Opt Dry Density” indicates “Optimum Dry Density” in units of pounds per cubic foot

5.0 SITE CHARACTERIZATION

5.1 REGIONAL GEOLOGY

5.1.1 General

Eagle Island is located in the extreme southeastern part of the Coastal Plain Physiographic Province. The region is characterized by relatively flat, low, topography with many wetland features, including salt marshes and tidal flats. To the west of the site, the Coastal Plain is drained by the Cape Fear River and its tributaries, which are estuarine in the lower reaches. In the vicinity of Eagle Island, drainage is by streams that flow directly to the Atlantic Ocean forming bays and broad estuaries. The regional shoreline is characterized by sand beaches and a chain of barrier islands that is separated from the mainland by narrow, shallow sounds.

The region is underlain by a wedge of sedimentary deposits consisting of sand, clay, limestone, and various combinations of these lithologies. The sediments thicken in a southeasterly direction attaining a thickness of approximately 1,500 feet near the coast. The sediments lie on crystalline metamorphic and igneous bedrock consisting of schist, gneiss, granite, and metamorphosed volcanics. The sediments, which range in age from Cretaceous to Pleistocene, can be classified into several stratigraphic units or formations. From oldest to youngest, these stratigraphic units are known to include the following:

- Black Creek Formation;
- Peedee Formation;
- Castle Hayne Formation;
- undifferentiated Late Tertiary deposits; and,
- undifferentiated surficial deposits.

The following subsections briefly discuss these formations.

5.1.2 Black Creek Formation

Approximately 400 feet of the Late Cretaceous Black Creek Formation unconformably overlies the crystalline bedrock (Bain, 1970) near Wilmington. The Black Creek Formation is complexly interbedded and consists primarily of layers of gray, fine- to medium-grained sand and dark gray clay. In its upper section, the Black Creek Formation is lithologically similar to, and difficult to distinguish from, the overlying Peedee Formation described below.

5.1.3 Peedee Formation

Approximately 700 feet of the Cretaceous Peedee Formation lies conformably on the Black Creek Formation. The Peedee Formation consists of interbedded sand, silt, and clay layers, many of that are dense and well indurated. The sand is typically fine-grained, glauconitic, and dark green to gray in color. Many of the sand layers are calcareous and some are argillaceous. The clay layers are dark and are also typically calcareous.

5.1.4 Castle Hayne Formation

Overlying the Peedee Formation is the Castle Hayne Limestone of Tertiary age. According to LeGrand and Brown (1955), the Castle Hayne Limestone is reported to be of Middle to Late Eocene age. Bain (1970) reports that in some locations Tertiary sediments older than the Castle Hayne Formation are present; however, no such Early Tertiary deposits are present in the area. The lack of Early Tertiary deposits is attributed to the positive effects of the Cape Fear Arch (Bain, 1970), which produced a period of erosion or non-deposition prior to the late Eocene. As a result, the Castle Hayne Limestone unconformably overlies the channeled and eroded upper surface of the Peedee Formation.

Bain (1970) describes four separate facies that can be part of the Castle Hayne Formation, although the thickness and distribution of these facies varies markedly with location. The four facies are: a basal sandy shell conglomerate containing reworked material from the Peedee Formation; a glauconite shell limestone; a dense, chalk-white, siliceous limestone locally known as “cap rock;” and a cream to light green, glauconite-bearing reworked shell hash. Not all of these facies are present in every location, and the total thickness of the Castle Hayne Formation can range from 0 to 150 feet within New Hanover County, and is expected to be thin in the vicinity of the site.

5.1.5 Undifferentiated Late Tertiary Deposits

Undifferentiated deposits of Late Tertiary Age consisting of phosphatic sand, silt, clay, and limestone overlie the Castle Hayne Formation in the eastern part of New Hanover County. In the northeastern quadrant, the deposits include a dark gray to blue-gray clay layer interbedded with 10 to 20 feet of fine- to medium-grained sand that is, itself, overlain by 5 to 20 feet of dense blue or gray clay. Farther south, near Carolina Beach, the undifferentiated Late Tertiary deposits are approximately 75 feet thick and consist primarily of marl interbedded with light green to dark gray silty clay containing thin shell beds. The lower, silty part of this unit continues to thicken toward the south where it is overlain by light gray to olive green clay and shell beds (Bain, 1970).

5.1.6 Undifferentiated Surficial Deposits

Undifferentiated surficial deposits of varying origin and age (post Miocene to Holocene) generally blanket the region. This unconsolidated sedimentary unit includes Pamlico and Chowan terrace deposits and consists of sand, clay, and marl that range from 0 to 70 feet in aggregate thickness. In the vicinity of the site, the surficial deposits likely lie on the undifferentiated Late Tertiary deposits. Farther west, the surficial unit rests on the Castle Hayne Formation or directly on the Peedee Formation where the Castle Hayne Formation is absent.

5.2 SURFACE CONDITIONS

The site and surrounding area is relatively flat, typically exists as marsh areas with an elevation of approximately +4 feet msl to +8 msl. Existing grades of the perimeter dikes typically range from +23 to +25 feet msl. The average interior elevations of Cells 1, 2, and 3 are +16 feet msl, +17 feet msl, and +17 feet msl, feet respectively.

5.3 SUBSURFACE CONDITIONS AT THE SITE

5.3.1 Correlation of Subsurface Materials

The subsurface data obtained in this work was obtained with two objectives, namely:

- to establish the mechanical characteristics (strength and compressibility) of the soil, particularly the softer fine grained soils which profoundly affect embankment performance; and
- to establish the occurrence of soils beneath the embankments.

The laboratory testing is primarily directed at establishing the mechanical characteristics, as is the extensive suite of *in situ* testing. The *in situ* testing was also directed toward establishing site stratigraphy. These soundings were supplemented by borings performed, in part, to assist in the interpretation of the CPT soundings. To this end, 149 CPT soundings were completed (sounding a total of about 7,800 lineal feet). Detail regarding the borings and soundings is presented in Section 3.0, as well as Appendix A

CPT soundings have a well-documented record of effectiveness in evaluating subsurface stratigraphy. The stratigraphic profiles being used in ongoing engineering analyses were developed based on the CPT results. CPT signatures (comparing cone tip resistance, q_c, sleeve friction, f_s, and dynamic pore pressure, u, from sounding to sounding) have been employed to identify the thickness and elevations of identifiable layers of soil.

As is discussed in Section 3.0, borings and soundings were taken at regular sections along the alignment of the embankments, such that well correlated descriptions of the embankments could be established. A total of 36 sections were established along the alignment of the embankments.

In general, the boundary between five identifiable soil layers or “zones” are well defined, though the thickness of the identified layers or zones varies slightly across the site. The layers are described in the following subsections (in order from ground surface down), as follows:

- Layer 1 – Upper Sands
- Layer 2 - Upper Organic Silt
- Layer 3 – Middle Sands
- Layer 4 – Lower Organic Silts
- Layer 5 - Lower Sands

5.3.2 Near Surface Soils

Based on the available boring and sounding data as well as our understanding of geomorphologic processes in the project vicinity, the near surface stratigraphy is generally consistent throughout the site. Cross sections of subsurface conditions under the CDF dikes were developed for each of the 36 sections explored. For the purposes of geotechnical analyses, nine (9) sections considered representative of the different areas of the site were analyzed for varying conditions of stability. These nine sections are depicted on Figures 5.1 through 5.9, and discussed in more detail (with respect to stability analyses) in Section 6.0.

As distinguished by the CPT and DMT soundings, the near subsurface materials may be considered to consist of five strata. The boundary between layers is generally well defined, though the thickness of each layer varies slightly across the site. The layers are described in the following sections in the order from ground surface down.

Layer 1 – Upper Sands

A one to 15 foot thick layer of loose to dense silty fine sands to sandy silts material overlies most of the embankment area. The thickness of the surficial sand generally decreases in thickness from the crest of the dike to the outboard and inboard sides of the dike as the ground surface elevation drops. The layer averages about six feet in thickness.

The surficial sands generally consist of silty fine sand (SM) to clayey and sandy silt (ML to OL). In general, this layer represents embankment soils. Judging from lower tip resistance to cone penetrometer soundings, low SPT blow counts and low shear strength as indicated by the flat plate dilatometer tests, these soils were sometimes apparently placed with little engineering control.

The CPT signature in these fine-grained soils is also evidenced by a lack of dynamic pore pressures, with locally negative pore pressures.

Layer 2 - Upper Organic Silt

A typically 7 to 16 foot thick layer of highly plastic organic silt (MH) underlies the surficial sands and was present in all borings and piezocone soundings throughout the site. The layer averages about 16 feet in thickness, and was of negligible in some soundings (for examples, the soundings in Section 20

SPT N values were generally in the weight of hammer (WOH) to 2 bpf range. Atterberg limits of this material are typically in the range of 80 to 110 for the liquid limit and 50 to 80 for the plasticity index. The interface between the Layer 1 material and the Layer 2 Upper Organic Silt is marked by a declining and relatively smooth signature of cone tip resistance and sleeve friction with depth.

The Layer 2 Upper Organic Silt generally represents the marsh soil that was original ground at the site. The CPT signature in these fine grained soils is also evidenced by relatively large excess positive dynamic pore pressures.

Layer 3 – Middle Sands

A 3 to 10 foot thick layer of silty fine sand (SM) with lenses of silt and clay underlies the upper organic silt. SPT N values in the range of 4 to 6 were encountered in the middle sands.

The transition from the upper organic silt to the middle sands is characterized by the pore pressure response tracking more closely to the hydrostatic pore pressure line, and an increase in cone tip resistance.

Layer 4 – Lower Silts

A 5 to 15 foot layer of organic silts (MH), with interlayers of fine sand (SP) and clay (CL to CH) occurs below the middle sands layer. This layer's SPT N values generally range from 5 to 10 bpf. Atterberg limits for this layer generally range from 40 to 50 for the liquid limit and 20 to 30 for the plastic index.

Similar to the Layer 2 upper organic silts, excess pore pressures within this layer become large; though the cone tip resistance and sleeve friction for this layer are larger than that for the Layer 2 soil, indicating that this soil layer is stronger than the marsh soil.

Layer 6 - Lower Sands

A dense sand with some silt (SM) to clayey sand (SC) layer underlies the middle clay layer. This layer is characterized by refusal blowcounts for the SPT N values. SPT N blowcounts for this layer are on the order of 10 to 25. The layer is readily identified by high cone tip resistance, and pore pressures which approach hydrostatic.

5.4 GROUNDWATER CONDITIONS

The surficial water table at the site generally ranges from approximately 10 to 14 feet below ground surface near the center of the dikes, to near the ground surface within the CDF and near the outboard edges.

6.0 GEOTECHNICAL ANALYSES

6.1 SLOPE STABILITY

6.1.1 Method of Analysis

The slope stability analyses were performed using the computer program WINSTABL. WINSTABL calculates the factors of safety against slope failure using a limiting equilibrium method assuming two-dimensional, plane strain conditions. The program is capable of handling heterogeneous soil profiles, anisotropic soil strength parameters, excess pore water pressure due to shear, and static ground water and surface water. The analytical methods available for use in the program include the simplified and modified Bishop methods (Bishop, 1955) applicable for circular-shaped failure surfaces; the simplified Janbu method (Janbu, 1954) applicable for failure surfaces of a general shape; and the Spencer method (Spencer, 1967) which is applicable for any surface. The program includes a trial and error search routine to evaluate numerous potential slip surfaces for any given profile and soil conditions and outputs data for the ten most critical (i.e. lowest factor of safety) for each case.

Dames & Moore selected Bishop's and Spencer's method for wedge shaped surfaces in this study. Differences between the many alternative methods of limiting equilibrium analyses are largely due to varying hypotheses regarding the location and direction of internal forces within the sliding soil mass. Spencer's method was selected for analysis of wedge shaped surfaces as the method is more rigorous than other alternatives because it solves equations of equilibrium for both moments and forces. Use of such methods for non-circular slip surfaces is recommended by Lambe and Whitman (1969).

Studies by Espinoza et. al. (1992) have shown that variations in the factors of safety calculated for the same slip surface but by differing methods is typically minimal. The assumption inherent in all limiting equilibrium methods that the soil is at limiting equilibrium with a constant factor of safety along the entire slip surface is probably more critical. Limiting equilibrium analysis methods currently in use do not adequately model progressive failure mechanisms that can occur in materials of widely dissimilar stress-strain characteristics. However, Spencer's method satisfies all conditions of equilibrium and the minimum factor of safety computed by it is considered by Duncan (1992) to be within 12 percent of that computed by other analyses of similar capability and within 6 percent of what may fairly be considered the correct answer.

Analyses were performed assuming drained behavior in the sandy soils of Layers 1, 2, and 3 and undrained behavior in the fine grained soils of Layer2 and Layer 4. Strength parameters for various soil layers used in the analyses are previously described in Section 5.

In the undrained analyses, no allowance was made for increasing undrained shear strength in the clay due to increased effective stresses as the perimeter dike is raised as multi-stage load conditions occur. As will be shown, the embankments can be shown to be stable with minimal increases in soil strength beyond that already existing.

To account for the different subsurface and embankment conditions, different stratigraphies were developed for nine separate locations about the CDF. Each is considered to be representative of the conditions around it. The stratigraphies chosen were selected with the following criteria:

- it was representative of that portion of the embankments; and
- within its area it was deemed critical because it displayed the greatest thickness of Layer 2 upper organic silts along with the smallest thickness of surficial sands.

The upper organic silt soils represent the weakest layer within the dredge spoil dike foundation and their strength and geometry typically defines the most critical potential failure surface and corresponding minimum factor of safety.

6.1.2 Slope Geometry and Drainage Conditions

At Eagle Island, the overall slopes of the existing dike at present are typically about 3:1 (horizontal: vertical) on the inboard slopes and 4:1 to one on the outboard slopes, shown graphically in Figure 6.1 through Figure 6.9. The Layer 2 and Layer 4 soils now behave in a fully consolidated, drained condition. For the events of dike raising, these soils will have this minimum strength, plus additional strength as the soil drain under the loads of fill used to raise the dikes. As discussed subsequently in this report, higher perimeter dike levels than the +38 feet msl now planned may also be attainable if substantiated based on monitoring of the dikes' performance during its construction. Section 6.6 addresses this consideration.

For each stability analysis, Dames & Moore assumed a variety of piezometric surfaces, each commensurate with a stage of dike raising and the associated level of dredged material in the impoundment. These assumptions are shown in the detailed backup of the stability analysis in Volume III.

6.2 MINIMUM FACTOR OF SAFETY

Ultimately, the steepness of the perimeter dikes' side slopes and the maximum height of the dikes are governed by the factor of safety that is selected as the appropriate basis of design in this instance. There are no building code requirements or other prescriptive design standards for this type of waste disposal structure imposed by the State of North Carolina or federal regulatory authorities which would fix either the maximum slope angle or the minimum factor of safety. Consequently, there is a need to decide the minimum appropriate factor of safety to be used as one of the design criteria.

The factor of safety used for design purposes is a composite factor that should reflect the following:

- uncertainties in the engineering behavior of the soil materials;
- the uncertainties as to the accuracy of the analytical method of predicting the slope stability; and,
- the degree of effective engineering control over the building of the dike in compliance with the design.

Embankments on soft clay soils usually pass through a time during construction in which the lowest factor of safety is due to an undrained condition in the underlying clay and a later time when the computed factor of safety must reflect a drained condition, which is usually higher. In such instances, it is appropriate to consider the two different analyses and the two different factors of safety in establishing the design slope and the rate of filling.

At the Eagle Island site, the foundation soils are weaker during raising of the dike than they will be after this loading ceases. Consequently, with all other factors equal, the factor of safety will be lower during the dike raising than later. To maximize the CDF capacity, it is thus necessary that the factor of safety during the dike raising be as low as is prudent, with the expectation that the factor of safety will increase after filling with dredge spoil is completed. However, the lower the design factor of safety, the more necessary it becomes to monitor the soil's response to the dike filling and to reassess frequently the operating factor of safety. On the contrary, if conditions do not permit monitoring and periodic analysis, then prudence requires a higher factor of safety to assure that the dike does not fail.

With the stipulation that the recommended geotechnical field monitoring and the periodic reassessment of stability described in Section 6.4 will be carried out and that the dike's growth rate or slope angle will be modified if necessary, Dames & Moore recommends that the factor(s) of safety used as the basis of design be as follows:

During the raising of the perimeter dikes:

- to prevent yielding of the soft foundation soils, the true factor of safety actually operating in the field should not be less than 1.1; and
- to account for uncertainties in the ability of modern stability analyses to forecast accurately the driving and resisting forces that determine the stability of the overall mass, the minimum factor of safety during the most critical period should be increased about 10 percent (Lambe and Whitman, 1969; Duncan, 1992); and
- to reflect the risk that the geotechnical data used in the analyses do not accurately reflect the geotechnical conditions at a given location, the factor of safety should be increased about 10 percent; which results in a minimum operating short-term factor of safety during dike filling of 1.30.

After embankment raising is completed to +38 feet msl (i.e., the perimeter dike is topped out) and there are no monitoring programs or reassessments of stability, the computed long-term factor of safety should be 1.5 or greater.

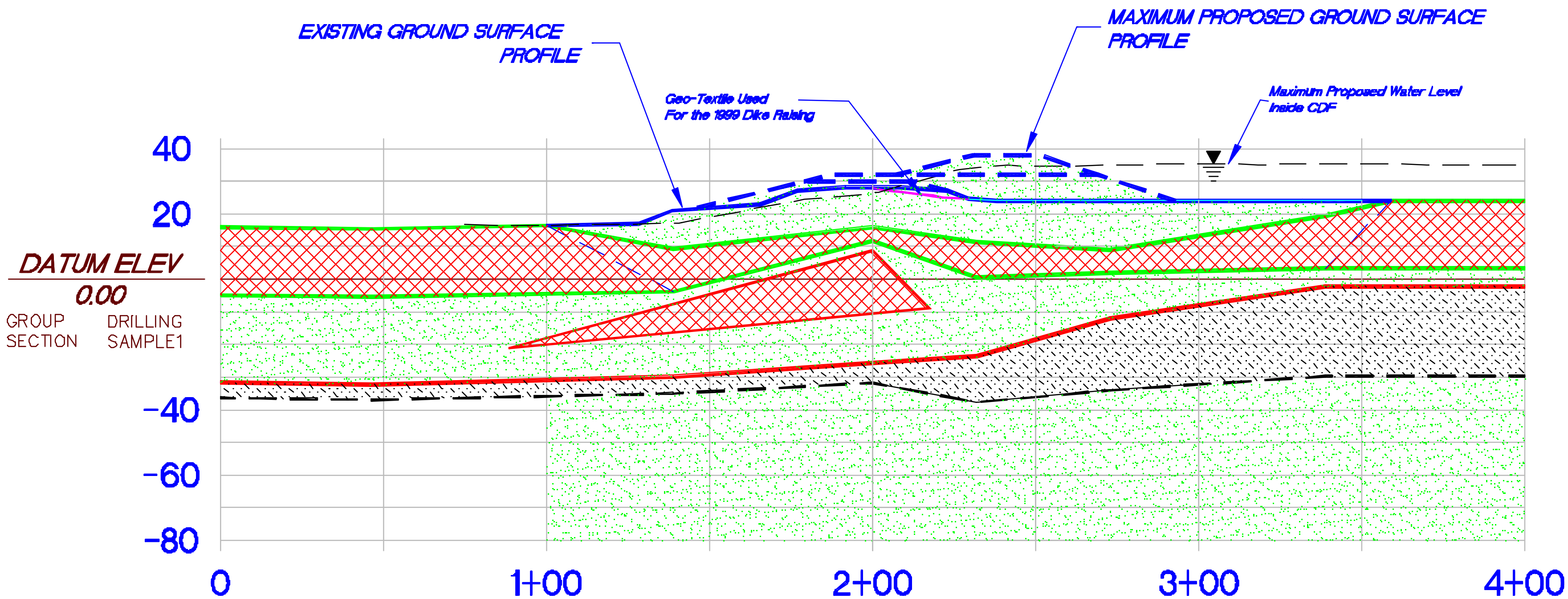
6.3 STABILITY ANALYSIS RESULTS

6.3.1 General

Typical cross sections for the CDF were developed using the aforementioned factors of safety as guidelines. These cross sections are included as Figures 6.1through 6.9.

The separate sections were analyzed for stability by either "outboard" failure (failure toward the marsh/river) or "inboard" failure (failure toward the contained dredge spoils) under three conditions:

- Condition I- Embankments Raised to +30 feet msl.
- Condition II- Embankments Raised to +32 feet msl.
- Condition III- Embankments Raised to +38 feet msl.



LEGEND

- = Approximate Groundwater Level
- = SM, SP (Sands - Upper, Middle, Lower)
- = MH/OH (Upper Organic Silt)
- = MH/CH/SP (Lower Clays w/ thin Sand Seams)

SCALE: N.T.S

US ARMY CORPS OF ENGINEERS
WILMINGTON DISTRICT

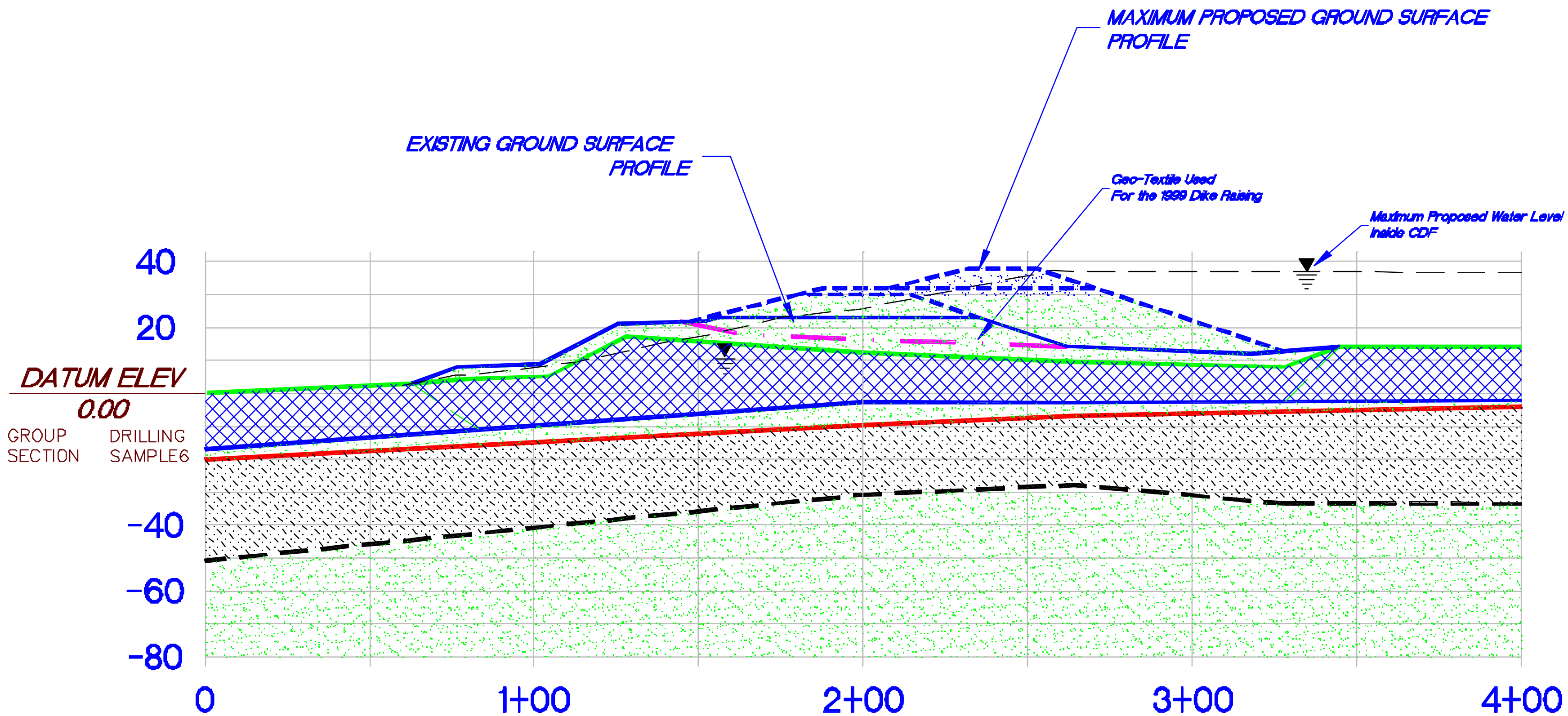
TITLE: CROSS-SECTION-01 (SECTION 28)

PROJECT: EMBANKMENT STABILITY ANALYSIS
EAGLE ISLAND MANAGEMENT PLAN

DATE: MAY 2001

Figure 6.1

D&M JOB NO.



LEGEND

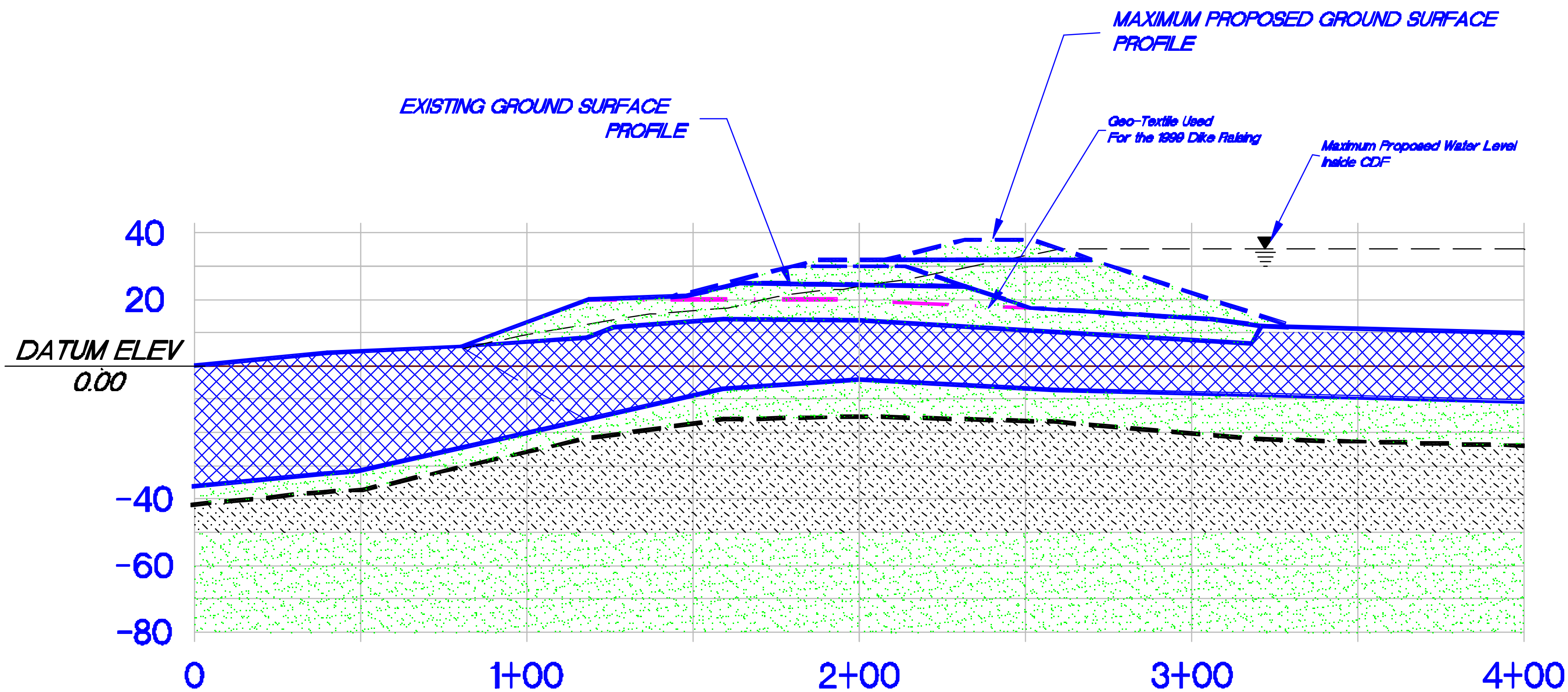
- Approximate Groundwater Level
- SM, SP (Sands - Upper, Middle, Lower)
- MH/OH (Upper Organic Silt)
- MH/CH/SP (Lower Clays w/ thin Sand Seams)

SCALE: N.T.S

D&M JOB NO.

US ARMY CORPS OF ENGINEERS
WILMINGTON DISTRICT

TITLE:	CROSS-SECTION: SECTION 06
PROJECT:	EMBANKMENT STABILITY ANALYSIS EAGLE ISLAND MANAGEMENT PLAN
DATE: MAY 2001	Figure 6.2



LEGEND

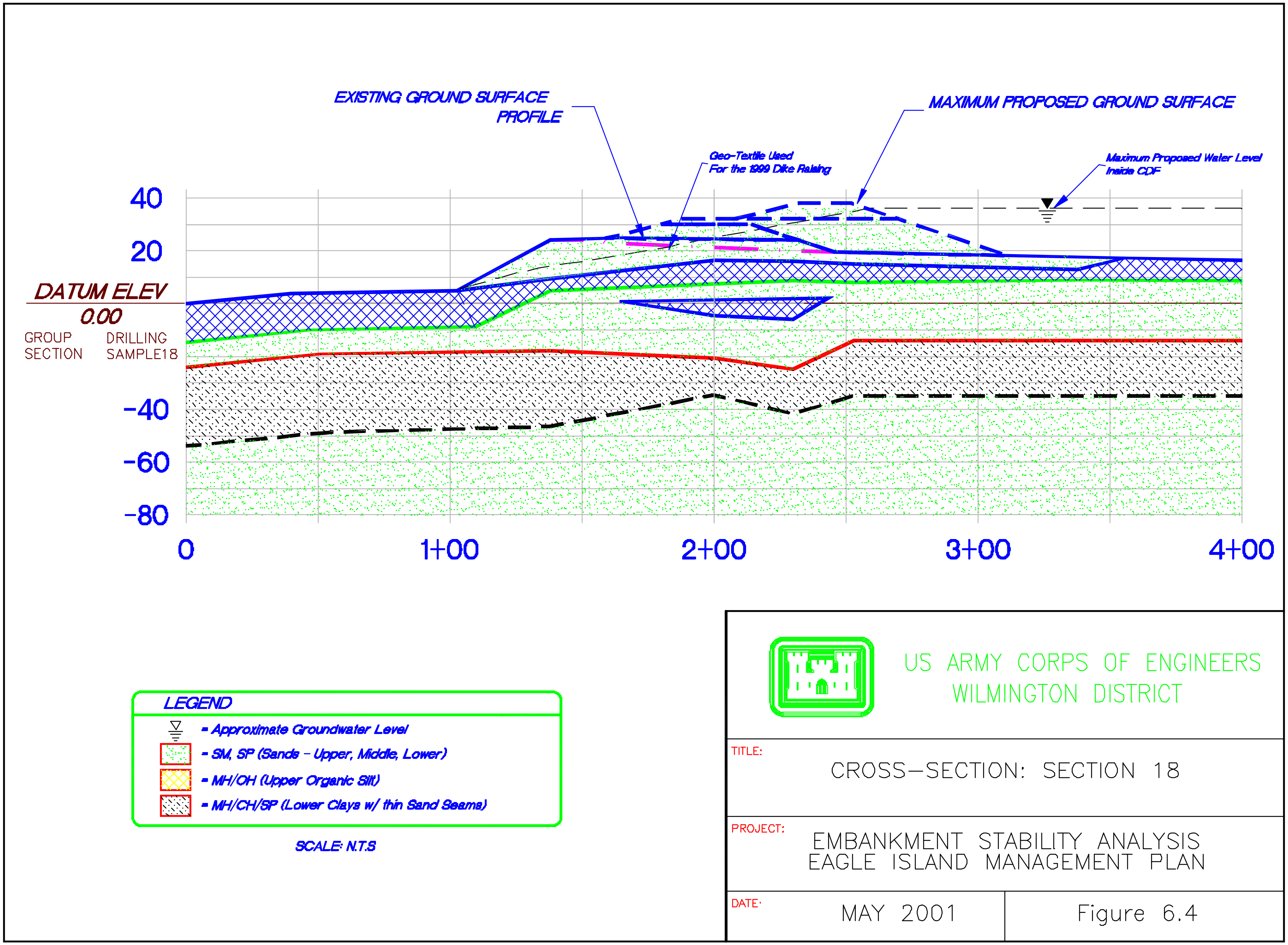
- = Approximate Groundwater Level
- = SM, SP (Sands - Upper, Middle, Lower)
- = MH/OH (Upper Organic Silt)
- = MH/CH/SP (Lower Clays w/ thin Sand Seams)

SCALE: N.T.S

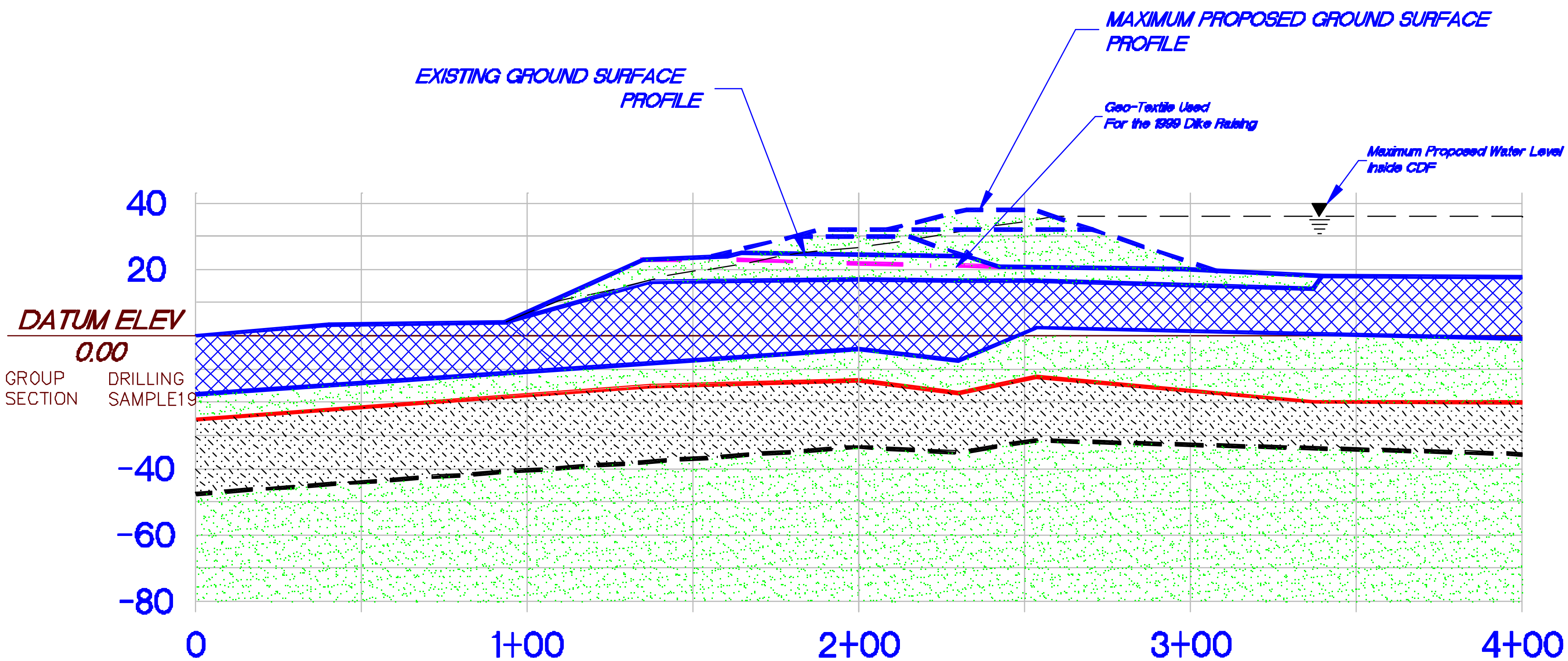
US ARMY CORPS OF ENGINEERS
WILMINGTON DISTRICT

TITLE:	CROSS-SECTION: SECTION 11
PROJECT:	EMBANKMENT STABILITY ANALYSIS EAGLE ISLAND MANAGEMENT PLAN
DATE: MAY 2001	Figure 6.3

D&M JOB NO.


EXISTING GROUND SURFACE PROFILE
MAXIMUM PROPOSED GROUND SURFACE
Geo-Textile Used For the 1999 Dike Raising
Maximum Proposed Water Level Inside CDF
40
20
DATUM ELEV 0.00
GROUP SECTION
DRILLING SAMPLE18
-40
-60
-80
0
1+00
2+00
3+00
4+00
LEGEND
- Approximate Groundwater Level
- SM, SP (Sands - Upper, Middle, Lower)
- MH/OH (Upper Organic Silt)
- MH/CH/SP (Lower Clays w/ thin Sand Seams)
SCALE: N.T.S
US ARMY CORPS OF ENGINEERS
WILMINGTON DISTRICT
TITLE:
CROSS-SECTION: SECTION 18
PROJECT:
EMBANKMENT STABILITY ANALYSIS
EAGLE ISLAND MANAGEMENT PLAN
DATE:
MAY 2001
Figure 6.4

D&M JOB NO.



LEGEND

- Approximate Groundwater Level
- SM, SP (Sands - Upper, Middle, Lower)
- MH/OH (Upper Organic Silt)
- MH/CH/SP (Lower Clays w/ thin Sand Seams)

SCALE: N.T.S

US ARMY CORPS OF ENGINEERS
WILMINGTON DISTRICT

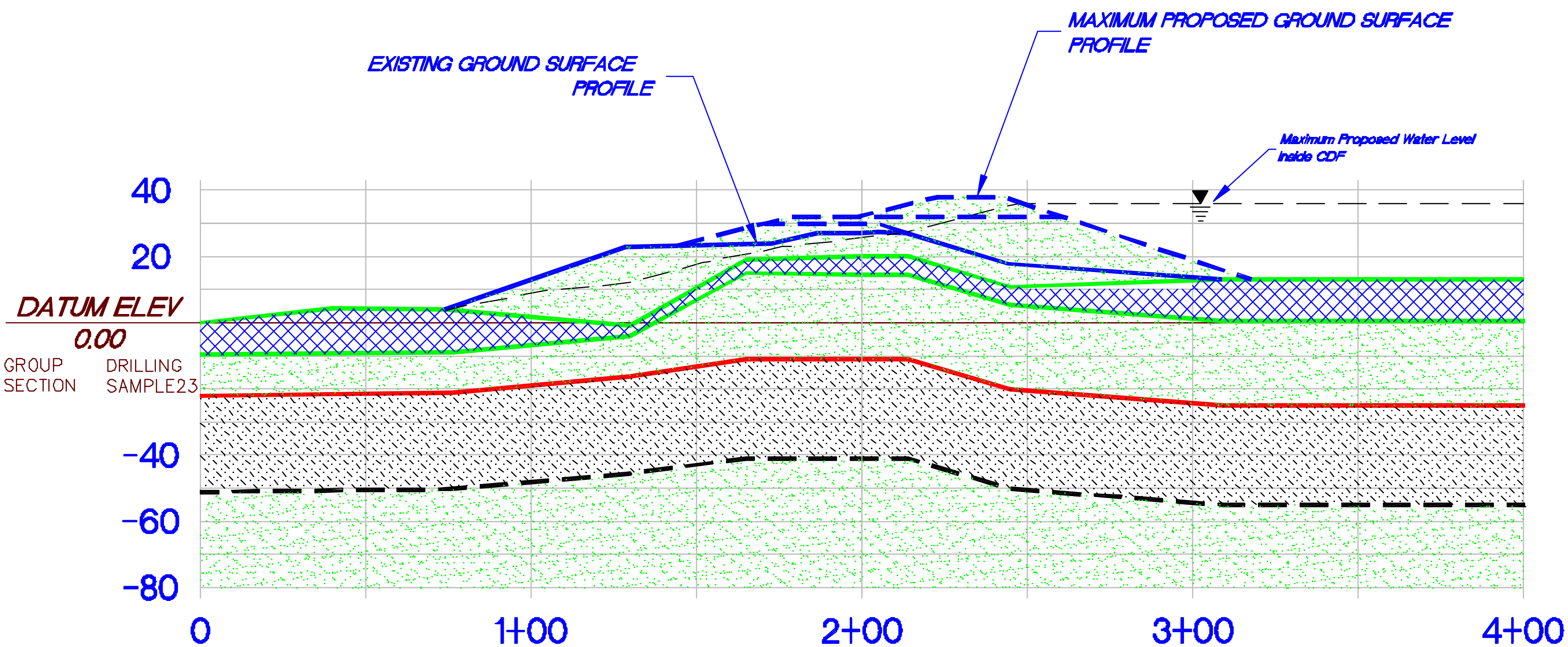
TITLE: CROSS-SECTION: SECTION 19

PROJECT: EMBANKMENT STABILITY ANALYSIS
EAGLE ISLAND MANAGEMENT PLAN

DATE: MAY 2001

Figure 6.5

D&M JOB NO.



SCALE: N.T.S

US ARMY CORPS OF ENGINEERS
WILMINGTON DISTRICT

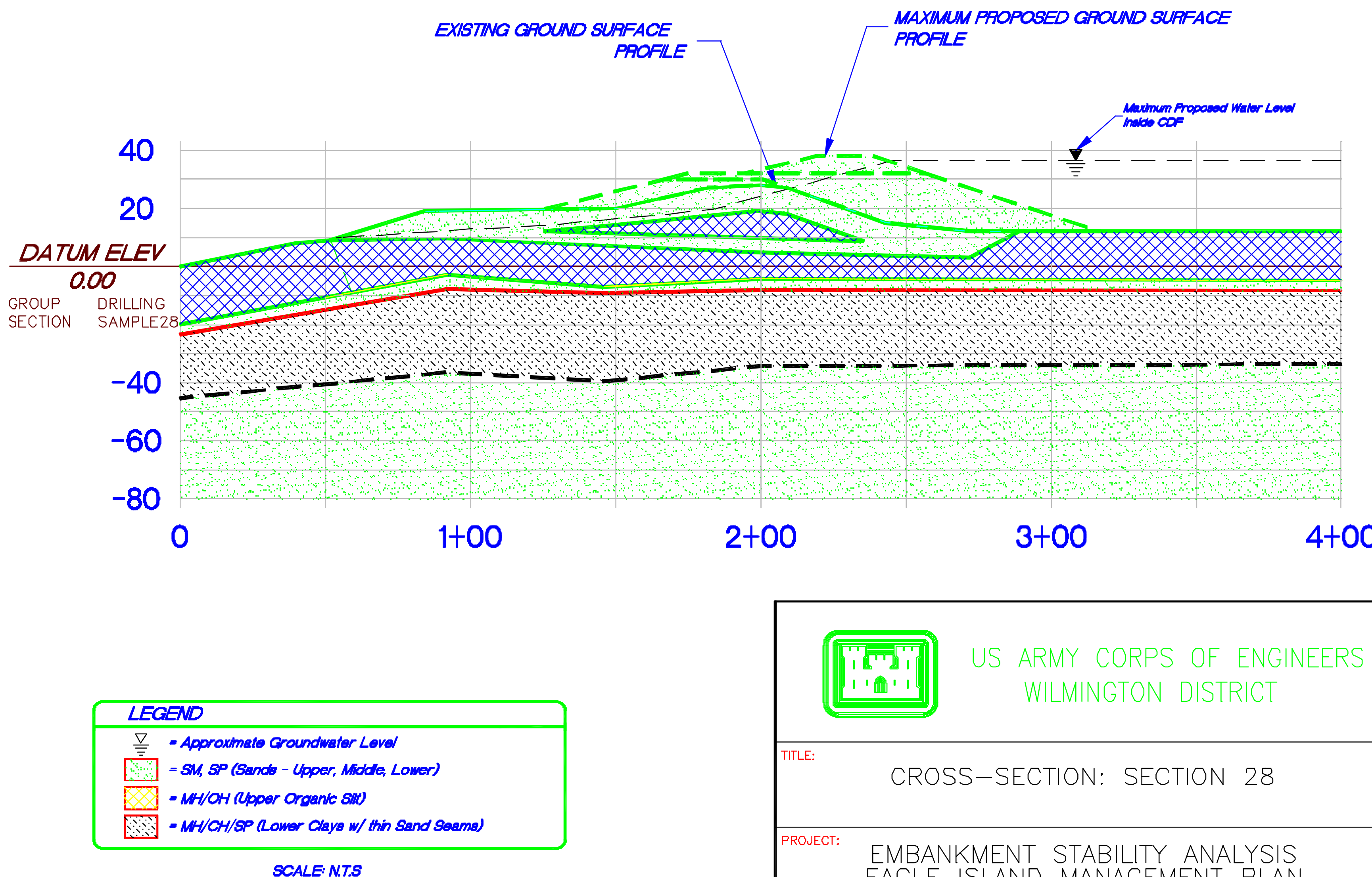
TITLE: CROSS-SECTION: SECTION 23

PROJECT: EMBANKMENT STABILITY ANALYSIS
EAGLE ISLAND MANAGEMENT PLAN

DATE: MAY 2001

Figure 6.6

D&M JOB NO.



US ARMY CORPS OF ENGINEERS
WILMINGTON DISTRICT

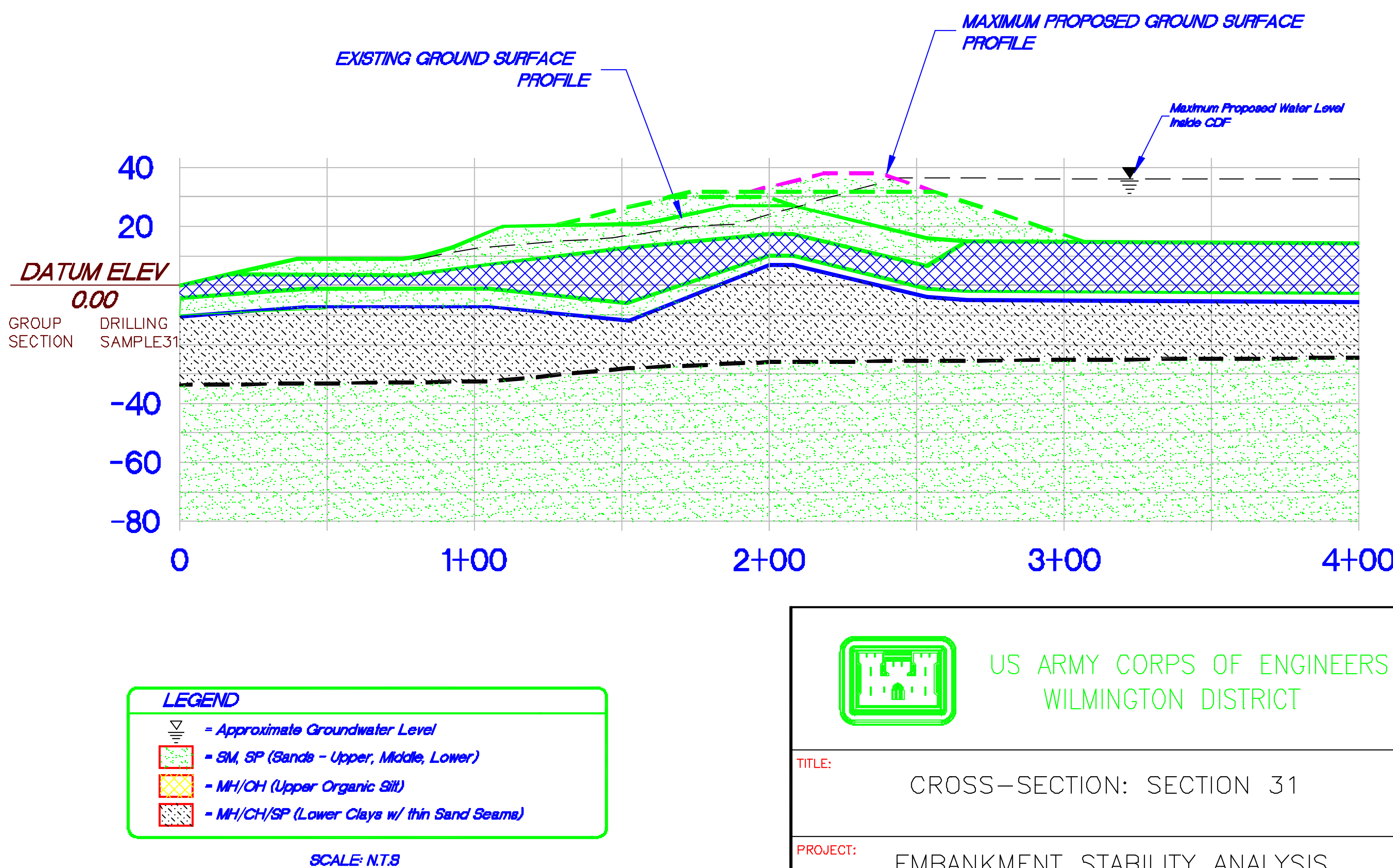
TITLE: CROSS-SECTION: SECTION 28

PROJECT: EMBANKMENT STABILITY ANALYSIS
EAGLE ISLAND MANAGEMENT PLAN

DATE: MAY 2001

Figure 6.7

D&M JOB NO.



D&M JOB NO.

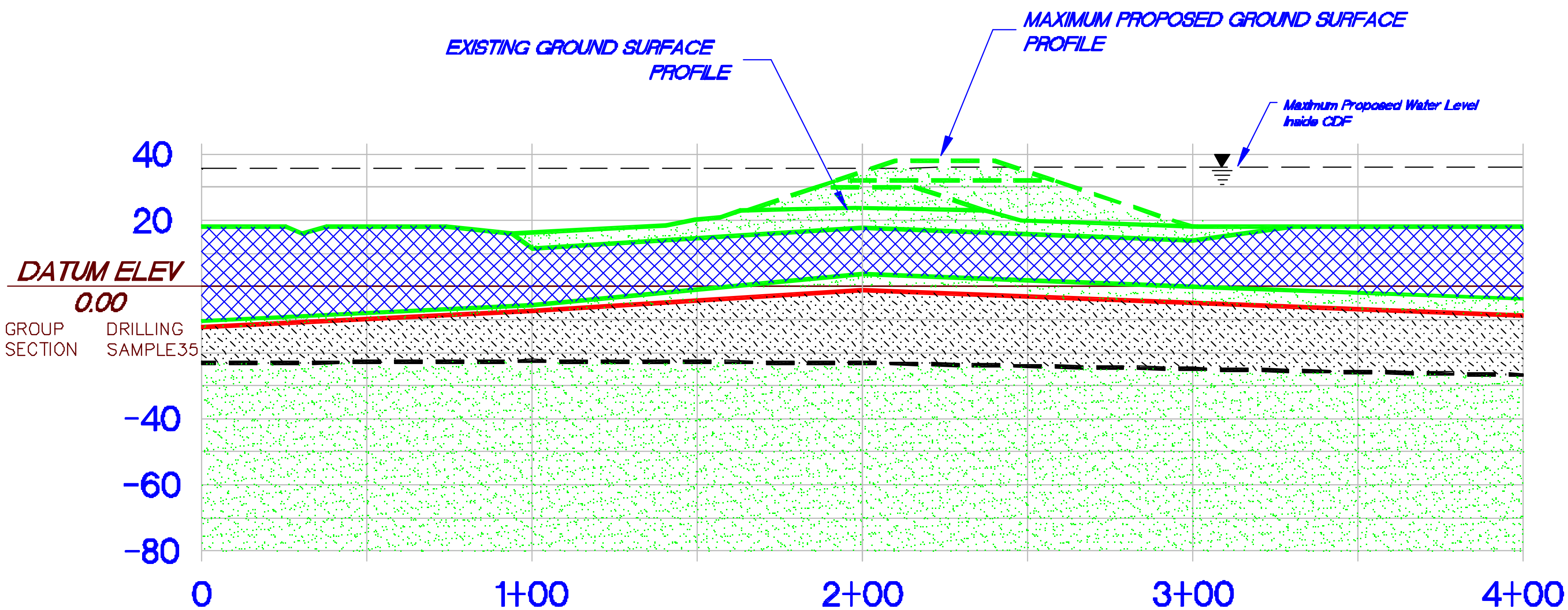
US ARMY CORPS OF ENGINEERS
WILMINGTON DISTRICT

TITLE: CROSS-SECTION: SECTION 31

PROJECT: EMBANKMENT STABILITY ANALYSIS
EAGLE ISLAND MANAGEMENT PLAN

DATE: APRIL 2001

Figure 6.8



LEGEND

- Approximate Groundwater Level
- SM, SP (Sands - Upper, Middle, Lower)
- MH/OH (Upper Organic Silt)
- MH/CH/SP (Lower Clays w/ thin Sand Seams)

SCALE: N.T.S

US ARMY CORPS OF ENGINEERS
WILMINGTON DISTRICT

TITLE: CROSS-SECTION: SECTION 35

PROJECT: EMBANKMENT STABILITY ANALYSIS
EAGLE ISLAND MANAGEMENT PLAN

DATE: MAY 2001

Figure 6.9

D&M JOB NO.

6.3.2 Summary of Embankment Stability

Table 6.1 summarizes the stability of the analyzed embankment sections for Conditions I, II and III.

Table 6.1 Summary of Stability Analyses

Section	Embankment Elevation (feet, msl)	Minimum Short Term Factor of Safety	
		Outboard Failure	Inboard Failure
Section 1	30	1.91	2.70
	32	1.72	2.63
	38	1.58	2.31
Section 6	30	1.47	2.31
	32	1.50	2.17
	38	1.41	1.87
Section 11	30	1.40	2.61
	32	1.31	2.44
	38	1.29	1.89
Section 18	30	1.72	2.56
	32	2.54	2.55
	38	1.42	2.07
Section 19	30	1.33	2.62
	32	1.34	2.34
	38	1.24	2.03
Section 23	30	1.45	2.67
	32	1.51	3.03
	38	1.63	2.65
Section 28	30	2.68	2.35
	32	2.35	2.19
	38	1.85	1.94
Section 31	30	1.86	3.71
	32	1.87	3.09
	38	1.48	2.55
Section 35	30	2.22	2.66
	32	2.06	2.34
	38	1.47	2.16

As may be seen from the above, the sections evaluated meet the minimum factor of safety (F = 1.3) criteria, with the exception of Section 11 (outboard). The consequence of this low factor of safety is discussed subsequently. Minor modifications to Section 11 may be required to meet the minimum recommended factor of safety criteria. Because of the conservative soil strengths used in design, the embankments may be raise to + 30 feet msl and thence to +38 feet msl over a relatively short period of time- perhaps as short as 4-5 years between raisings.

Again, the conclusion that the sections in Table 6.1- considered representative of the entire perimeter dike system- meet the minimum factor of safety (F = 1.3) criteria is made on the explicit condition that regular monitoring and instrumentation demonstrates that the clay foundation soils remain stable during dike construction, with a minimum operating factor of safety in excess of 1.3 against slope failure. After closure of the dike and regular monitoring ceases, the factor of safety should exceed 1.5.

The stability evaluated by Dames & Moore is subject to modification during the lifetime of the dike dependent upon the observed behavior of the embankments and foundation soils. Improved drainage and pore pressure conditions better than assumed in our analysis may give rise to allowance for steeper side slopes and quicker construction to +38 feet msl, whereas poorer drainage and excessive dike displacements may require flatter side slopes or limiting the rate at which dikes may be raised..

Results of the stability analysis for Conditions I, II and III for dike levels at +30 feet msl, +32 feet msl and +38 feet msl are presented in Volumes III, Record of Stability Analysis. The data in Volume III shows critical failure surfaces and computer program inputs and outputs.

6.3.3 Stability at "Section 11"

As is shown on Table 6.1 the stability of the embankment in the vicinity of Section 11 is lower than that recommended for Condition III. This lower stability is due to the fact that the geotechnical exploration disclosed different conditions outboard at the section than at other locations, creating deeper seated failure.

The computed stability at Section 11 is only marginally lower than the recommended Factor of Safety of 1.3. If desired, the stability of Section 11 may be improved to an acceptable factor of safety in any of several ways, for example:

- move the vertical expansion inboard of that now planned;
- utilize ground improvement technology (for example, wick drains) to increase the strength of the Layer 2 soils; and/or
- utilize structural enhancements (for example, sheet piles) to locally increase the apparent strength of the soils.

The simplest solution is to move the vertical expansion inboard of that now planned combining the movement with more careful monitoring of the performance of the embankments in this area.

6.4 SIDE SLOPE INSTRUMENTATION AND MONITORING REQUIREMENTS

6.4.1 Survey Data

Heaving or excessive horizontal movement of soils at the toe of a slope is usually the first indication of instability of an embankment on soft ground. One of the easiest and most economical methods of identifying this condition is to monitor permanent survey monuments located near the toe of the dike. We recommend installing such monuments a maximum of 1,000 feet apart around the dike, as close to the dike toe as is practical. In addition, vertical settlement monitors or gages should be installed beneath the dike crest which are anchored in the Layer 5 foundation soils. These monitors would be monitored periodically to measure the dike foundation settlements at any future time.

The location and elevation of each monument should be surveyed at initial installation and then periodically thereafter, with the frequency of readings dependent on the amount of deposition activity in a given area. For example, during the early stages of soil deposition associated with vertical expansion, readings should be taken on a weekly basis. In addition to the monuments, survey data should include updated elevations of dike crests so that the rate of vertical dike growth can be evaluated. Settlements and deformations should be measured to an accuracy of one-quarter inch. The cross-section should be surveyed at least annually or after each five foot rise in vertical height, whichever occurs first, so that the actual geometry can be verified for subsequent stability analysis. The locations of survey cross-sections should be representative of the entire perimeter embankment and preferably coincide with locations of other survey and other instrumentation.

Survey data should be reviewed by an experienced geotechnical engineer who is familiar with soil conditions at the site and methodology used to evaluate embankment stability.

6.4.2 Inclinometers

We recommend utilizing inclinometers installed through the Layer 2 upper organic silts and founded in the Layer 5 Lower Sands to support survey efforts to monitor soil movements during dike expansion. The Layer 5 Lower Sands will be relatively incompressible under the proposed embankment loads, and will reliably provide a fixed foundation for inclinometers.

It is expected that some lateral squeezing of the Layer 2 soils will occur, and inclinometers will provide important data regarding response of the Layer 2 soils.

Conceptually, two levels of inclinometers would be installed at each of 8 to 10 sections about the perimeter embankments. One inclinometer would be installed on the outboard slope of the embankment and a second near the toe of the slope, but still on the embankment.

As with the survey monuments, inclinometer readings will be required on a weekly during critical times construction and at least monthly thereafter as the data dictates. Engineering evaluation of the data will be necessary to assess the import of the movements to stability analyses

6.4.3 Perimeter Dike Construction and Fill Compaction

Perimeter dikes may be raised in multiple stages:

- to a maximum grade of +30 feet msl in a single construction step; and
- dikes may be then raised up to a maximum elevation of +38 feet msl, given a wider base and done in increments over three additional stages.

In each phase, embankment/dike settlement and movement should be monitored as described in Section 6.4.2. An experienced geotechnical engineer should review dike performance and settlement data at each stage, and further construction of the dikes should continue only following satisfactory observed settlement behavior.

Suitable fill for perimeter dike construction may consist of the dried plastic silty and clayey materials which are now in the CDF. These soils should be placed in vertical lifts not exceeding twelve (12) inches thick and compacted to approximately 90 percent of the maximum dry density attained by the Standard Proctor ASTM D-698 Compaction Test. This compaction requirement is recommended as the best compromise between embankment soil strength and trafficability requirements and limiting the weight of new fill (with lighter weight new fill associated with diminished compaction effort). However, the soils have proven themselves to be hard to compact to a specific ASTM standard, and a detailed compaction testing program is not recommended.

All slopes should be sodded or seeded as a minimum to protect against soil erosion. Special erosion control measures may be necessary for the protection of the dikes.

6.5 FOUNDATION SETTLEMENT

Analyses were performed to evaluate the potential settlements of the dredge spoil dike site as a result of its construction. The analysis consisted of manual calculations using a methodology by Hough ("Compressibility as the Basis for Soil Bearing Value", ASCE Proceedings, August 1959). It is estimated that total settlements under the maximum perimeter dike crest (elevation +38 feet msl) in the range of 2 to 4 feet can be expected. These results do not address compression of the dredge spoil material, only the settlement of the dike foundation subgrade soils. Consolidation/compression of the dredge spoil material is the subject of Volume VI, Primary Consolidation, Secondary Compression, and Desiccation of Dredged Fill (PSDDF) Report.

7.0 CONCLUSIONS AND RECOMMENDATIONS

The geotechnical studies described in this report were performed for the purpose of evaluating the potential to raise the existing perimeter dikes at the USACE's Eagle Island confined disposal facility (CDF).

The geotechnical design of the dredge spoil storage expansion facility is recommended to include the following major features which have been discussed in greater detail in the body of this report.

7.1 SIDE SLOPE STABILITY

Dames & Moore has evaluated the dike side slope stability using limit equilibrium methods based on the laboratory and field test data developed to support those analyses, as well as anecdotal historical data related to the performance of the existing dikes.

For design purposes, a maximum overall side slope of 4:1 horizontal:vertical is recommended for the perimeter dikes. This recommendation is made on the explicit condition that regular monitoring of survey instrumentation demonstrates that the highly plastic organic silt (OH) foundation soils (referenced in this report as "Layer 2" soils) remain stable during embankment construction. Stability analyses utilizing conservative soil strength parameters indicates that this construction can be accomplished with a minimum short term factor of safety of about 1.3 against slope failure. After completion of the vertical dike expansion and regular monitoring ceases, the factor of safety should exceed 1.5. As long as these minimum factors of safety are maintained, Dames & Moore does not consider that excessive strains or local yielding within the upper clay will occur

The embankment height at elevation +38 feet msl may be reached in four stages, first raising embankments from it existing level to +30 feet msl, and then incrementally raising dikes +34 feet msl, +36 feet msl and finally to +38 feet msl (with each stage followed by monitoring). This recommendation is subject to modification during actual raising of the perimeter dikes, dependent upon the observed behavior of the perimeter dikes and foundation soils. Soil strength, soil drainage and pore pressure conditions better than assumed in our analysis would give rise to a steeper recommended side slopes and a quicker time between vertical expansion efforts; whereas weaker soils, poorer drainage and too fast a rate of dike construction may require flatter side slopes or other modifications to the dike design or CDF operating procedures.

7.2 INSTRUMENTATION AND MONITORING REQUIREMENTS

A slope instrumentation and monitoring program is recommended by Dames & Moore to assure that the minimum necessary operating factors of safety can be maintained throughout the operating life of the dike. Such instrumentation should typically consist of survey monuments, settlement gages, and inclinometers spaced generally at 1,000 foot intervals around the dike perimeter. Survey cross-section data should also be obtained at regular intervals to verify the side slope profile actually constructed. All of this data should be regularly reviewed by an experienced geotechnical engineer who is familiar with the site conditions and design methodology for the project. The recommended minimum frequency of monitoring is approximately monthly, but this frequency may be subject to change if dike construction methods or instrumentation data indicate that more frequent monitoring is necessary in a localized area or for a specific time duration.

7.3 FOUNDATION SETTLEMENT

Analyses of consolidation settlement estimate a range of settlement under the projected dike crest elevation of +38 feet msl from 2 to 4 feet. This estimate varies largely due to the varying thickness of compressible soils present beneath the site, and will occur largely due to compression of the Layer 2 Upper Organic Silt. The bulk of the remaining settlement will occur within the Layer 4 Middle Organic Silt.

Soil deformation and settlement is anticipated to be reasonably uniform beneath the dike crest but will likely reflect local variations in thickness of the compressible Layer 2 and Layer 4 soils within the perimeter dike foundation. Soil deformations beyond the embankment toe in the vicinity of the exterior containment dike are anticipated to be minimal (less than one to three inches).

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