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FINAL REPORT

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UNDRAINED STRENGTH STABILITY ANALYSIS FOR WEST PERIMETER DIKE AT THE CRANEY ISLAND DREDGED MATERIAL MANAGEMENT AREA, NORFOLK, VIRGINIA

By: Timothy D. Stark and Thomas A. Williamson
Associate Professor and Graduate Research Assistant
Department of Civil Engineering
University of Illinois at Urbana-Champaign
205 N. Mathews Ave.
Urbana, IL 61801-2397
(217) 333-7394

and

Jack Fowler
Research Civil Engineer
U.S. Army Engineer
Waterways Experiment Station
Vicksburg, MS 39180
(601) 634-2703

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Prepared for U.S. Army Engineer, Waterways Experiment Station
Vicksburg, MS 39180

ABSTRACT

This study investigates the stability of the west perimeter dike at the Craney Island Dredged Material Management Area (CIDMMA) using an undrained strength stability analysis (Mesri, 1983). An undrained strength stability analysis expresses the undrained shear strength in terms of the effective preconsolidation pressure. This allows the undrained shear strength to reflect a strength increase caused by consolidation. The undrained strength stability analysis was used to evaluate the current stability of the west perimeter dike and the possibility of raising the dike. In addition, stability analyses were conducted to determine the effect of installing vertical strip drains on the current stability and the potential for raising the west perimeter dike. Vertical strip drains will accelerate consolidation of the dredged material and underlying marine clay. Consolidation of these materials will result in an increase in soil shear strength and dike stability. In summary, the current (January, 1994) factor of safety of the west perimeter dike is approximately 1.9. As a result, the west perimeter dike and dredged material can be safely (factor of safety greater than 1.3) raised to el. +37.5 ft and +33.5 ft CEMLW, respectively, without the installation of vertical strip drains. The installation of vertical strip drains and 100 percent consolidation of the dredged material and underlying marine clay should allow the west perimeter dike and dredged material to be raised to el. +58 ft and +54 ft CEMLW, respectively. The time required for the strip drains to achieve 100% consolidation depends on the spacing of the drains. Value engineering should be performed to determine the optimal cost and drain spacing for 100% consolidation.

PREFACE

This study investigates the stability of the west perimeter dike at the Craney Island Dredged Material Management Area (CIDMMA) using an undrained strength stability analysis (Mesri, 1983). An undrained strength stability analysis expresses the undrained shear strength in terms of the effective preconsolidation pressure. This allows the undrained shear strength to reflect a strength increase caused by consolidation. The undrained strength stability analysis was used to evaluate the current stability of the west perimeter dike and the possibility of raising the dike. In addition, stability analyses were conducted to determine the effect of installing vertical strip drains on the current stability and the potential for raising the west perimeter dike. Vertical strip drains will accelerate consolidation of the dredged material and underlying marine clay. Consolidation of these materials will result in an increase in soil shear strength and dike stability. In summary, the current (January, 1994) factor of safety of the west perimeter dike is approximately 1.9. As a result, the west perimeter dike and dredged material can be safely (factor of safety greater than 1.3) raised to el. +37.5 ft and +33.5 ft CEMLW, respectively, without the installation of vertical strip drains. The installation of vertical strip drains and 100 percent consolidation of the dredged material and underlying marine clay should allow the west perimeter dike and dredged material to be raised to el. +58 ft and +54 ft CEMLW, respectively. The time required for the strip drains to achieve 100% consolidation depends on the spacing of the drains. Value engineering should be performed to determine the optimal cost and drain spacing for 100% consolidation.

This research was conducted for the U.S. Army Engineer Waterways Experiment Station (WES), located in Vicksburg, Mississippi, and the U.S. Army Engineer District, Norfolk (NAO), located in Norfolk, Virginia, during the period 1 February 1994 to 30 November 1994.

General supervision of the study was carried out by Mr. Sam McGee, NAO, and Dr. Jack Fowler, Geotechnical Laboratory (GL), WES, under the guidance of Mr. Ronn G. Vann, Chief, Dredging Management Branch, and Mr. James N. Thomasson, Chief, Engineering Division, NAO. Technical information was provided by Mr. Matt Byrne, Ms. Yvonne Gibbons, Geotechnical Engineering Section (GES), NAO, and Mr. Dave Pezza, Chief, GES, NAO.

Dr. Timothy D. Stark, Department of Civil Engineering, University of Illinois at Urbana-Champaign, supervised the stability analysis and wrote this report. Mr. Thomas A. Williamson, a Graduate Research Assistant at the University of Illinois at Urbana-Champaign, performed the stability analyses. The study was monitored by Dr. Jack Fowler, WES. General guidance was provided by Mr. W. Milton Myers, Chief, Soil Mechanics Branch, Soil and Rock Mechanics Division (SRMD), Dr. Don C. Banks, Chief, SRMD, and Dr. William F. Marcuson, III, Director, GL. This research was performed under Contract No. DACW39-94-C-0017 between WES and Dr. Timothy D. Stark. The Director of WES at the time of this report is Dr. Robert W. Whalin.

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CONVERSION FACTORS, NON-SI TO SI (METRIC) UNITS OF MEASUREMENT

Non-SI units of measurement can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
acres	4046.873	square meters
cubic yards	0.7645549	cubic meters
feet	0.3048	meters
feet per day	0.3048	meters per day
feet per minute	0.3048	meters per minute
inches	0.0254	meters
pounds (force) per square foot	0.04788026	kilopascals
pounds (mass) per cubic foot	16.01846	kilograms per cubic meter
square feet	0.09290304	square meters
square feet per day	0.09290304	square meters per day
square inches	645.16	square millimeters
tons (force) per square foot	95.76052	kilopascals

UNDRAINED STRENGTH STABILITY ANALYSIS FOR WEST PERIMETER DIKE AT THE CRANEY ISLAND DREDGED MATERIAL MANAGEMENT AREA, NORFOLK, VIRGINIA

PART I: INTRODUCTION

Background

1. The Craney Island Dredged Material Management Area (CIDMMA) is a 2,500 acre site with a storage area of approximately 2,200-acres (Figure 1). Craney Island is located near Norfolk, Virginia, in Portsmouth, Virginia. A vicinity map is shown in Figure 2. Planned in the early 1940's, construction of the CIDMMA began in August 1954 and was completed in January 1957. Craney Island is the long-term placement area for material dredged from the channels and ports in the Newport News/Hampton Roads area.

2. Dredged material has been placed in the management area almost continuously since it was completed in 1957. The original design was for an initial capacity of about 100 million cu yd at an annual dredging rate of 4 to 7 million cu yd. Therefore, the CIDMMA was designed for a service life of approximately 20-years (1957 to 1977) based on an annual dredging rate of 5 million cu yd. Continued dredging in the Norfolk channel has required the capacity of Craney Island to be increased through three major dike raising efforts. The initial raising from el. +8 ft Corps of Engineers Mean Low Water[1] (CEMLW) to el. +17 ft CEMLW occurred around 1969 with the second increase to el. +26 ft CEMLW around 1980. The U.S. Army Engineer District, Norfolk (NAO) is currently raising the perimeter dike system based on recommendations presented by Fowler et al. (1987). The west dike is being raised to el. +34 ft CEMLW but this raising required the placement of a 1,000-foot-wide underwater stability berm along the outer toe of the dike (Figure 3) to ensure stability. The perimeter dike in the northwest corner is being raised to el. +34 ft CEMLW using a dike setback of approximately 450 ft (Figure 4). The north and east perimeter dikes are being raised to el. +40 ft CEMLW with setbacks from the dike perimeter road of 430 and 450 ft, respectively (Figures 5 and 6). Dike setbacks have resulted in approximately 20 to 30 acres of lost storage capacity during each dike raising. Figure 1 shows the location of these dike cross sections.

3. Using plans developed by Palermo et al. (1981), interior dikes were built within Craney Island to create three containment areas (Figure 1) that would improve sedimentation in the compartment being filled and allow the other two compartments to desiccate and consolidate at a faster rate. Desiccation will be accelerated by the removal and/or evaporation of surface water, and will increase the amount of consolidation because the effective unit weight of the soil increases as the pore-water evaporates. Construction of the interior dikes was completed in 1983, and the dredged material management plan (Palermo et al., 1981) was implemented in 1984 starting with the center compartment. The management plan has resulted in each compartment being filled approximately every third year. On the average 4 to 5 million cu yd of dredged material are placed in a compartment each year. This results in an annual increase in dredged material thickness of 3 to 6 feet in the compartment being filled or about 1 to 2 feet over the entire disposal area (Szelest 1991).

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[1]CEMLW is 2 ft (0.6 m) below NGVD 1929, 1972 Adjustment, and 0.65 ft (0.2 m) below MLW (NOS).

4. The Environmental Laboratory at the U.S. Army Engineer Waterways Experiment Station conducted an extensive consolidation and desiccation analysis to predict the remaining service life of the CIDMMA (Palermo and Schaefer 1990). This study utilized the finite strain consolidation microcomputer program PCDDF (Stark 1991; Stark and O'Meara 1991) and revealed that the current capacity of the CIDMMA will be exhausted by the year 2000. As a result, the Norfolk District started investigating new techniques for increasing the storage capacity of Craney Island.

Alternatives for Increasing Storage Capacity at Craney Island

5. The studies by Fowler et al. (1987) showed that the perimeter dikes are at their maximum height. However, if the undrained shear strength of the dredged material and underlying marine clay was increased as a result of consolidation, the perimeter dikes could be raised again. The time required for this consolidation would be substantial and thus would not alleviate the short-term storage problem.

6. An extensive study was conducted by Spigolon and Fowler (1987) on the feasibility of expanding Craney Island. Six expansion configurations (Figure 7) were considered by Spigolon and Fowler (1987). However, in 1991 the Virginia State Legislature ruled that Craney Island could not be expanded or replaced. Therefore, the feasibility of restricting the usage of Craney Island to placement of contaminated dredged material and ocean dumping the remainder of the material was investigated (Myers et al. 1993). The cost of ocean dumping is approximately $7.00 per cu yd whereas placement in Craney Island is approximately $0.90 per cu yd (Szelest 1991). NAO is still dredging at a rate of approximately 5 million cu yd per yr. Therefore, the difference between placement in Craney Island and ocean dumping is approximately $30 million per yr. As a result, additional alternatives for increasing the storage capacity of Craney Island were sought.

7. Recently installed piezometers in the perimeter dikes at Craney Island revealed that large excess pore-water pressures exist in the dredged material and underlying marine clay. It can be seen from Figures 3 through 6 that the excess pore-water pressure levels in February 1991 exceed the ground surface elevation by 20-25 feet in some locations. The dissipation of these excess pore-water pressures would result in substantial consolidation settlement of the perimeter dikes. Consolidation of the marine clay and dredged material would cause a significant increase in the undrained shear strength of these materials and probably allow the perimeter dikes to be substantially raised. This would allow the perimeter dikes to be constructed to higher elevations without setbacks or stability berms. It was also anticipated that large excess pore-water pressures exist in the dredged material and underlying marine clay in the containment area. Dissipation of these excess pore-water pressures would result in substantial consolidation settlement, and thus increased storage capacity.

8. The time required for 90 percent consolidation to occur can be estimated using Terzaghi's one-dimensional consolidation equation (Terzaghi and Peck, 1967):

$$t_{90\%} = \frac{0.848 * H_{dr}^{2}}{C_v} \qquad (1)$$

where H_{dr} is the maximum length of the drainage path and C_v is the vertical coefficient of consolidation. This equation shows that the time required for consolidation is controlled by the coefficient of consolidation, that is, permeability, of the soil and the maximum drainage length that water must travel to exit the soil deposit. Since altering the permeability of a soil in situ is not economically feasible, techniques were sought to decrease the drainage path to accelerate consolidation.

Use of Strip Drains to Increase Storage Capacity

9. Figure 8 shows the generalized subsurface profile at Craney Island. It can be seen that the average surface elevation of the dredged material is approximately +23 ft CEMLW and the thickness of both the dredged material and marine clay is approximately 123 ft. Since the site appears to be doubly drained, the maximum length of the vertical drainage path to either the top surface or the sands underlying the marine clay is 61.5 ft or approximately 62 ft. However, the thickness of the marine clay varies throughout the site. For example, in the north compartment the marine clay is approximately 125 ft thick where an old river channel is located. Therefore, the thickness of both the marine clay and dredged material is approximately 158 ft. The resulting maximum length of vertical drainage path is 79 ft. For illustrative purposes the marine clay will be assumed to be 90 ft thick as shown in Figure 8. Recent piezocone penetration tests (Stark, 1992) in the north compartment suggest that the underlying dense sand and silty sand are permeable. Therefore, the site is assumed to be doubly drained.

10. Figure 9 shows that the installation of vertical strip drains will result in radial or horizontal flow, as well as vertical flow. As a result, the maximum drainage path will be reduced to one-half of the strip drain spacing, that is, 6 ft, instead of one-half of the compressible layer thickness, approximately, 62 ft. This reduction in drainage path is extremely significant since the time rate of consolidation is a function of the length of drainage path squared (Equation 1). Therefore, the installation of vertical strip drains will result in a substantial reduction in the time required to consolidate the dredged material and underlying marine clay. This will yield a rapid increase in consolidation settlement and undrained shear strength of the dredged material and marine clay.

11. It is proposed that the strip drains be installed throughout the placement area and subsequently the perimeter dikes. The strip drains will consolidate the dredged material and underlying marine clay in the placement area, which may permit future development of this site. Installing strip drains in only the perimeter dikes would be less expensive and may also result in more settlement because of the additional surcharge applied by the dikes. The strip drains would accelerate consolidation, and an increase in shear strength, of the marine clay underlying the perimeter dikes and allow the dikes to be constructed to higher elevations. However, the strip drains installed in the perimeter dikes would not consolidate the placement area and thus not reduce the surface elevation of the placement area. NAO is interested in consolidating the placement area because it may create opportunities for future development of the site, and possibly the construction of a new placement area in the Norfolk area. If consolidation of the 2,200 acre disposal area is not economically feasible, strip drains can be installed in only the perimeter dikes.

Purpose

12. The purpose of this study is to evaluate the stability of the existing west perimeter dike using an undrained strength stability analysis. In addition, the effect of installing vertical strip drains in the west perimeter dike on the stability and potential raising of the dike are investigated. Installation of vertical strip drains will accelerate consolidation of the marine clay underlying the west perimeter dike. Only the west perimeter dike is evaluated in this study because Fowler et al. (1987) concluded that the west perimeter dike is the least stable with respect to foundation stability.

Scope

13. This study required the identification of west perimeter dike (the least stable dike with respect to foundation stability) cross sections for conventional limit equilibrium analysis. As part of the evaluation, analyses and determination of factors of safety for several scenarios of the west perimeter dike cross section based on the shear strengths and undrained strength ratios determined from cone penetration tests were conducted. The dike geometry was estimated using aerial and surface surveys and cone penetration test results. In addition, stability analyses were conducted to determine the effect of installing vertical strip drains in the west perimeter dike on the factor of safety.

Objective

14. The objective of this investigation was to determine the current factor of safety of the west perimeter dike at Craney Island and to determine if the west perimeter dike could be raised. In addition, the increase in stability, and thus dike elevation, caused by consolidation of the underlying marine clay will be evaluated. Consolidation can be accelerated by the installation of vertical strip drains.

PART II: VERTICAL STRIP DRAIN TECHNOLOGY

15. In the last 5 to 10 years, prefabricated vertical strip drains have replaced conventional sand drains as the preferred method to accelerate consolidation of soft cohesive soils. This is primarily due to the ease of installation, higher flexibility and reliability, less environmental impact, and reduced cost of the prefabricated strip drains. Most prefabricated strip drains are modeled after the cardboard strip drain developed by Kjellman in 1948 (a and b). Strip drains are band-shaped and have a rectangular cross section of approximately 4 inches wide and 0.15 to 0.20 inches thick. The drain consists of a plastic core with grooves, studs, or channels that is surrounded by a filter fabric. The fabric is most commonly a nonwoven geotextile. The plastic core carries the excess pore-water to the ground surface, and/or the underlying drainage layer, and the filter fabric prevents soil particles from entering the core. Vertical strip drains have been used to accelerate consolidation of soft cohesive soils in many projects throughout the United States, including the recent expansion of the Port of Los Angeles (Jacob et al. 1994), the Seagirt project in Baltimore Harbor (Koerner et al. 1986), the construction of a dredge material containment area in the Delaware River near Wilmington, Delaware (Koerner and Fritzinger 1988 and Fritzinger 1990), and the New Bedford Superfund Site near New Bedford, Massachusetts (Schimelfenyg et al. 1990).

16. Vertical strip drains are easily installed using equipment that exerts a ground pressure as low as 3 to 5 psi. Special low ground pressure (1 to 1.5 psi) equipment was developed to install strip drains in the Craney Island containment test area (Figure 10). It can be seen that the equipment is mounted on pontoons that are 7 feet wide and 35 feet long. The pontoons reduce the ground pressure to approximately 1.3 psi and will provide flotation if the desiccated crust cannot support the strip drain equipment. A test section in the north compartment (Stark and Williamson 1994) of the CIDMMA showed that a well-developed desiccated crust can support this new low ground pressure equipment. The installed cost of strip drains is usually $0.40 to $0.70 per lineal foot depending on the quantity of strip drains installed. In contrast, the installed cost of conventional sand drains is $3.50 to $6.50 per lineal foot. The time required for consolidation of the dredged material and foundation clay is controlled by the spacing of the strip drains. Therefore, value engineering can be used to determine the optimal spacing of the drains to produce a certain increase in settlement, that is, storage capacity, in a specified time.

17. The prefabricated strip drains arrive at the site in large rolls and are installed using a hollow mandrel. The end of the strip drain is threaded down the inside of the mandrel, which must be as long as the depth to which the strip drains are to be installed. At the bottom of the mandrel, the strip drain is threaded through a baseplate and inserted into the mandrel (Figure 11). The baseplate is used to keep the strip drain at the bottom of the mandrel, to prevent soil from entering the mandrel during the insertion process, and to anchor the strip drain at the desired depth as the mandrel is withdrawn. When the mandrel is withdrawn from the ground, the strip drain is cut, and the process is repeated at the next location. This insertion cycle is rapid (1 to 5 minutes depending on insertion depth) and only strip drains, baseplates, and a cutting tool are required.

18. If strip drains are installed throughout the placement area, it is anticipated that strip drains will be installed in one compartment while the other compartments are used for placement

and desiccation. After the strip drains accelerate consolidation in the first compartment, this compartment will be used for placement while strip drains are installed in another compartment and the third compartment undergoes desiccation to support the strip drain equipment. Installation of strip drains will continue until strip drains have been installed in all three compartments. If strip drains are installed in only the perimeter dikes, it is anticipated that strip drains will be installed first in the west perimeter dike.

PART III: PERIMETER DIKES

General Topography

19. The Craney Island Dredged Material Management Area is approximately 10,000 ft by 10,500 ft in a rectangular shape (Figure 1). A peripheral dike 25 to 30 ft high surrounds the entire management area, and two dividing dikes running parallel with the shoreline and north perimeter dike separate the facility into three nearly equal areas of approximately 800 acres each. The southernmost compartment has been filled at the eastern dike to el. +27 ft CEMLW, the middle of this compartment to el. +19 ft CEMLW, and areas adjacent to the western dike to an average el. +17 ft CEMLW as of 1994. The middle compartment is filled at the eastern dike to el. +19 ft CEMLW sloping toward the western dike to about el. +13 ft CEMLW in the northwestern corner of the middle compartment. The northern compartment slopes from el. +22 ft CEMLW at the east dike to el. +16 ft CEMLW at the western dike.

Development

20. Beginning from the initial construction, the perimeter dike height has increased in three major efforts. The initial change from el. +8 to el. +17 CEMLW occurred in 1969 with the second increase to el. +26 CEMLW in 1980. Usually a stepped or benched dike construction technique is used to incrementally raise the dikes at Craney Island. Desiccated dredged material along the dike alignment is generally used to raise the dikes and supplemented as required with truck-hauled coarse-grained material.

21. Since 1971 several investigations have been conducted to study the possibility of raising the perimeter dikes at the CIDMMA. In 1971 the Norfolk District performed a feasibility study for raising the perimeter dikes to el. +30 CEMLW (Norfolk District 1971). The results, based on geotechnical data from 1971 borings, indicated that a bench or setback of about 1,000 ft was needed for the west perimeter dike. In 1982, a foundation analysis performed by the Norfolk District indicated that the outside slope along the west perimeter dike needed to be constructed at 1V:8H for a dike crest el. +30 ft CEMLW (Byrne 1982). When the west perimeter dike was constructed to el. +26 ft CEMLW in 1985 and the possibility of deepening the channels at Norfolk was considered, it was recommended that a thorough analysis be conducted for the main retaining dikes at Craney Island.

Description of Perimeter Dikes

East dike

22. Since most of the coarse-grained dredged material is located along the east dike, it provides a convenient location for construction material and continued construction of this dike. Because this coarse-grained dredged material exhibits a large shear strength, progressive raising of the east dike has not caused any stability problems. However, a dike setback of approximately 450 ft was required to raise the east dike from el. +8 ft to el. +26 ft CEMLW. A dragline has been used to excavate and place coarse-grained dredged material along the dike alignment. The stability of this dike was not evaluated during this investigation.

North dike

23. Coarse-grained dredged material has also accumulated over an extensive area in both the northeast and northwest corners of the management area and along the inside of the north perimeter dike. The dike alignment was setback approximately 420 ft south of the north perimeter road to accommodate a borrow area adjacent to the roadway and to increase stability during dike raising to el. +30 ft CEMLW. There have never been any stability problems along the north perimeter dike. As a result, the stability analyses described herein concentrates on the west perimeter dike and the north dike was not investigated.

West dike

24. Because of the continuously wet condition of fine-grained dredged material adjacent to the west perimeter dike, it has been virtually impossible to construct a benched dike section without a bearing capacity failure. Incremental dike raising has historically been achieved by displacing sand fill into the containment area adjacent to the existing perimeter dike. Coarse-grained dredged sand is truck-hauled and end-dumped on the slope, and a dozer pushes the sand up the slope and into the disposal area creating a large mud wave as the weight of the sand displaces the soft fine-grained dredged material.

25. After the interior division dikes were completed, subdividing the management area into three separate containment areas, the middle area began to dry out. Continued site drainage caused about 6 to 12 in. of desiccated crust to develop along the west perimeter dike. The west perimeter dike was raised in late 1985 to about el. +26 ft CEMLW without the displacement type failure toward the inside of the containment area as experienced in the past because of improved foundation conditions caused by trenching and desiccation of the fine-grained material.

Hydrographical Survey

26. Hydrographic surveys were conducted by the Norfolk District to determine the bottom elevations within the CIDMMA 1,000-ft right-of-way adjacent to the perimeter dike surrounding the management area. Depth of water at the 1,000-ft right-of-way varied from 9 to 12 ft corresponding fairly close to the values found on the National Oceanic and Atmospheric Administration charts for the Hampton Roads Area. The only difference in the bottom topographic features was found near the northwest corner where bottom depths were found to be over 25 ft deep at a distance of about 500 ft from the corner. This trough was noted in 1979 and appears to have been caused by fast flowing water currents around the northwest corner of the island. Bottom slopes determined from the hydrographic surveys varied from 1V:30H to 1V:100H. This information was used to create an underwater stability berm along the west perimeter dike as suggested by Fowler et al. (1987). The stability berm is 1,000 ft wide and the top of the berm corresponds to approximately the mean low water line.

PART IV: GEOTECHNICAL INVESTIGATIONS

27. This section describes the geotechnical investigations of the foundation soil conditions below the west perimeter dike and dredged material deposited within the north compartment of the Craney Island Dredged Material Management Area. The engineering properties used in the stability analyses of the west perimeter dike are also described.

1948 Investigation

28. An extensive foundation investigation was initiated in 1948 and was completed by the Norfolk District in 1949 during the design phases of this project and prior to the beginning of construction. A total of 11 undisturbed sample borings and a large number of general investigative borings were conducted. Laboratory tests on undisturbed samples consisted of several consolidation tests, undrained triaxial compression tests, Atterberg limits, specific gravity, and classification tests on the compressible marine clays underlying the site. The General Design Memorandum (U.S. Army 1949) identified four major soil zones underlying the current placement area (Table 1).

Table 1. Major Foundation Soil Zones (Palermo et al. 1981)

		Elevation (ft, cemlw)		Natural Unit Weights (lb/cu ft)	
Zone	Soil Type	From	To	Dry	Submerged
A	Grey marine clay	-10	-30	48.8	29.3
B	Grey marine clay	-30	-60	49.7	30.1
C	Marine clay, some silt	-60	-90	57.1	34.3
D	Clay and silt, some sand	-90	-110	60.3	39.9
Below D	Hard compact sand	Below	-110	----	----

1971 Investigation

29. During a 1971 feasibility study for raising the perimeter dikes at the CIDMMA, the Norfolk District conducted a subsurface investigation beneath the main perimeter dike. Several of these borings were taken through the dredged material deposit as well as through the perimeter dike. These borings verified the foundation soil zones shown in Table 1.

1978-1979 Investigations

30. Several other foundation investigations made in or near the CIDMMA by the Virginia Department of Transportation and others are summarized in studies prepared for the Virginia Port Authority (Dames and Moore 1978). Most of the borings taken during these investigations were in search of coarse-grained material borrow sources and do not provide information concerning the engineering properties of the fine-grained dredged material or foundation materials.

1980 Investigation

31. Three rotary borings were augered under contract for the Norfolk District at three locations on the western portion of the management area in April 1980. A shallow floating barge mounted drill rig was used during this investigation. A number of consolidation and permeability tests were conducted on the fine-grained dredged material samples obtained. Eighteen additional borings were conducted in August 1980 (Pezza and Byrne 1980) to define the quality and volume of the coarse-grained materials at the inflow points that were usable for dike enlargement.

1981-1983 Investigations

32. Because of the large dredged material storage volume required to accommodate the proposed Norfolk harbor and channel deepening, the Norfolk District initiated a subsurface investigation of the main perimeter dike in 1981 that was completed in 1983. Seven borings were drilled in 1981 and 20 borings in 1983 to a depth of about 90 ft to 100 ft. These borings penetrated the very dense sand underlying the marine clay. In addition to the borings, field vane shear strength tests and laboratory triaxial compression tests were performed during this investigation. The field vane test results were used by Fowler et al. (1987) to estimate the stability of the west perimeter dike. Since the dredged material and foundation clay are continuously undergoing consolidation, the shear strength of these materials is continuously increasing. Therefore, the results of the 1981-1983 vane shear tests represent the undrained shear strength in 1981-1983. This has led to a conservative estimate of dike stability in subsequent years. Due to the constant increase in shear strength, Stark and Fowler (1994) recommended the use of an undrained strength stability analysis to investigate the stability of the west perimeter dike. This analysis expresses the undrained shear strength, Su, of the dredged material and marine clay in terms of the effective preconsolidation pressure, p', of these materials. This analysis is described in more detail in paragraph 38.

1991 Investigation

33. Based on recommendations by Stark and Fowler (1994) piezometers were installed in 1991 in the perimeter dikes at the CIDMMA to investigate the degree of consolidation of the dredged material and underlying marine clay. In February, 1991 the piezometers revealed that large excess pore-water pressures exist in the dredged material and marine clay underlying the perimeter dikes. It should be noted that the Stark and Fowler (1994) report is later than the 1991 peizometer installation so that the report could include the piezometric measurements. It can be seen from Figure 3 that the excess pore-water pressure level beneath the west perimeter dike exceeds the dike surface elevation by approximately 20-25 feet. Stark and Fowler (1994) concluded that the dissipation of these excess pore-water pressures would result in substantial consolidation settlement, and thus increased shear strength of the dredged material and marine clay. This increase in shear strength should allow the perimeter dikes to be constructed to higher elevations without setbacks or stability berms. If the excess pore-water pressures in the dredged material and marine clay underlying the containment area are dissipated, a substantial amount of

consolidation settlement will occur, which would result in an increase in storage capacity. In summary, dissipation of these excess pore pressures could result in a large increase in service life of the CIDMMA.

1992 Investigation

34. In 1992 a subsurface investigation was conducted in the north compartment at the location of the first strip drain test section. The strip drain test section is located in the north compartment and described by Stark and Williamson (1994). The subsurface investigation was conducted before prefabricated strip drains were installed to aid interpretation of the consolidation settlements. A plan view of Craney Island and the location of the strip drain test section are presented in Figure 12. Cone and piezocone penetration tests (CPT) were conducted to define the soil stratigraphy below the test section area. These test results were also used to estimate the magnitude and variability of the undrained shear strength with depth. The existing undrained shear strength profile in the test section was estimated using a technique that utilizes the tip resistance from cone penetration tests and the following equation (Lunne and Kleven, 1981):

$$Su = \frac{q_c - \sigma_{vo}}{N_k} \qquad (2)$$

where q_c is the cone tip resistance, σ_{vo} is the total overburden pressure, and N_k is an empirical cone factor. Empirical correlations of N_k have been developed using the results of field vane shear tests (Lunne and Kleven 1981 and Meigh 1987) and unconsolidated-undrained triaxial compression tests (Stark and Delashaw 1990). To differentiate the unconsolidated-undrained triaxial mode of shear from the field vane shear, Stark and Delashaw (1990) denoted their cone factor N_{kuu}. It should be noted that the field vane shear strengths were corrected using Bjerrum's (1972) correction factor. Both correlations utilize plasticity index (PI) to estimate values of cone factor.

35. Table 2 presents the index properties of the marine clay underlying the CIDMMA. The statistical values of the index properties were determined from the results of 135 tests from 1949 to 1992. Since the dredged material is similar to the foundation clay the same index properties are used to characterize both deposits.

36. The value of N_k based on 1981-1983 field vane shear tests was estimated to range from 10 to 15 for a PI of 41. The value of N_{kuu}, which is based on unconsolidated-undrained triaxial tests, ranges from 8 to 14 for a PI of 41 (Stark and Delashaw 1990). Therefore, a representative value of N_k equal to 12 was utilized in the analysis. This is also an average value of N_{kuu} for 20 sites studied by Stark and Delashaw (1990). Figure 13 presents the variation of undrained shear strength with depth using N_k equal to 12. Each data point corresponds to a calculation of Su using Equation (2), the appropriate total stress, and a value of N_k equal to 12.

37. Figure 13 presents the variation in Su with depth estimated from cone penetration test results. It can be seen that the dredged material contains many sand and/or silt seams. As a result, the cone penetration tip resistance, and thus undrained shear strength, varies significantly with depth. The dredged material is probably undergoing self-weight consolidation and the excess pore-water pressures are being dissipated by the sand/silt seams. Based on this conclusion, the

majority of the consolidation settlement measured in the strip drain test section is attributed to consolidation of the marine clay. The dredged material appears to be undergoing self-weight consolidation and acting as a surcharge for the marine clay. A constant value of undrained shear strength was used to model the dredged material in the stability analysis of the west perimeter dike.

Table 2. Summary of Index Properties of Foundation Soil (after Ishibashi et al. 1993)

	Liquid Limit	Plastic Limit	Plasticity Index	Clay Size Fraction (%)	Specific Gravity of Solids
AVERAGE	70.7	29.3	41.4	94.4	2.71
STANDARD DEVIATION	14.7	4.88	12.3	7.25	0.04
COEFFICIENT OF VARIATION	21%	17%	30%	4%	2%

38. Ladd and Foott (1974) and Mesri (1975) showed that the undrained shear strength can be normalized with respect to the effective preconsolidation pressure. The resulting ratio Su/p', is termed the undrained strength ratio. These researchers showed that the undrained strength ratio is usually constant for a particular deposit. Therefore, one of the main objectives of the 1991 subsurface investigation was to estimate the value of the Su/p' ratio of the marine clay. The increase in preconsolidation pressure, caused by consolidation of the normally consolidated dredged material and marine clay, can be used to estimate the increase in Su. The new value of Su is estimated by multiplying the undrained strength ratio by the new value of preconsolidation pressure. The revised value of Su can then be used in a stability analysis. This stability analysis was termed an undrained strength stability analysis by Mesri (1983) and subsequently an undrained strength analysis by Ladd (1991). Therefore, the object of subsequent subsurface investigations is to measure the preconsolidation pressure profile with depth and then estimate the new Su profile.

39. From Figure 13 the marine clay appears to be under-consolidated. This is evident by the approximately linear increase in Su with depth. The linear increase in Su with depth indicates that the preconsolidation pressure increases linearly with depth, and thus the deposit is normally-consolidated for the current effective stress. The range of 1992 Su values in the marine clay corresponds to an undrained strength ratio of 0.22 to 0.25. This is based on effective stresses estimated from piezocone test results. Piezocone dissipation tests were used to estimate the pore-water pressure, and thus effective stress. The effective vertical stress was assumed to be equal to the preconsolidation pressure. Stark and Williamson (1994) describe the piezocone test results. It should be noted that an undrained strength ratio of 0.22 is recommended by Mesri (1989) for stability analyses.

40. The range of Su values after 100% consolidation were estimated using undrained strength ratios of 0.22 to 0.25, the effective vertical stress corresponding to the current overburden, and 100% consolidation. It can be seen that the marine clay is significantly under-consolidated. In addition, it also appears that the sand underlying the marine clay is free-draining

because the values of Su increase near the bottom of the marine clay. In fact, the measured value of Su near the bottom of the marine clay is approximately equal to the Su value estimated for the effective stress at 100% consolidation (Figure 13).

1993 Investigation

41. In 1993 Su values were obtained from cone penetration tests conducted through the west perimeter dike near the cross section being evaluated (CPT test 93PC-12) in the stability analysis. The undrained shear strength profile for the west perimeter dike is presented in Figure 14 and the values of Su were obtained using Equation (2) and N_k equal to 12. Values of Su were estimated using an undrained strength ratio of 0.22 and 0.25 and the effective vertical stress under the west perimeter dike after 100% consolidation. It can be seen that the marine clay is also significantly under-consolidated under the west perimeter dike. Consolidating the marine clay would result in a substantial increase in Su and thus increased stability of the west perimeter dike. It also appears that the sand underlying the marine clay is free-draining because the values of Su increase near the bottom of the marine clay layer.

42. A comparison of Figures 13 and 14 provides an insight to the difference in the undrained shear strength of the marine clay under the containment area and the west perimeter dike. It can be seen that the undrained shear strength in the upper 25 ft of the marine clay is substantially larger under the west perimeter dike. This is probably caused by the sand dike acting as a surcharge and drain. The surcharge induces a larger effective stress and the sand dissipates the excess pore-water pressures in the upper portion of the marine clay. Under the test section the dredged material does not allow substantial drainage of the excess pore-water pressures, which results in lower values of Su in the upper portion of the marine clay. However, from a depth of 65 to 115 ft the marine clay under the west dike exhibits lower values of Su than under the containment area. Below a depth of 115 ft, the undrained shear strength increases significantly due to the drainage provided by the underlying sand layer. This comparison suggests that the marine clay from a depth 65 to 115 ft will control the stability of the west perimeter dike. This will be evident from the subsequent stability analyses, which reveal that the critical slip surfaces are located between these depths. If vertical strip drains are installed through the west perimeter dike, it is imperative that the drains extend to a depth that is approximately 10 ft above the dense sand layer. This corresponds to a depth of at least 115 ft from the top of CPT location 93PC-12. The elevation of the dike at CPT 93PC-12 is +12 ft CEMLW, therefore strip drains should extend below approximately to el. -100 ft CEMLW. In other locations along the west perimeter dike, the strip drains should extend to at least a depth below which the marine clay starts to exhibit a strength increase due to the underlying dense sand. Cone penetration test results suggest that this depth is approximately 10 ft above the free-draining dense sand layer. This should result in an increase in west perimeter dike stability and reduce the quantity of strip drains required.

* Weak zone. Check inclinometer

Note not all that necessary to fully anchor.

PART V: SLOPE STABILITY ANALYSIS

43. Slope stability analyses conducted during this investigation were performed using the two-dimensional slope stability microcomputer program UTEXAS2, version 2 (Edris and Wright 1987). Spencer's (1967) stability method as coded in UTEXAS2 was used for all analyses because it satisfies all force and moment equilibrium. The side force inclination is calculated in the Spencer procedure to produce equilibrium. The phreatic surface is taken as the harbor level outside the west perimeter dike toe and the top of the dredge material inside of the dike. A linear variation in the phreatic surface is assumed between these two horizontal segments. Circular sliding surfaces were used in all analyses. Searches were performed to determine the critical sliding surface for each task.

44. A number of stability analyses were conducted to investigate the accuracy of previous stability investigations, evaluate current (1994) stability of the west perimeter dike, address the importance of 1992-1993 undrained shear strength values on west perimeter dike stability, and address Norfolk District stability concerns. In addition, the effect of installing vertical strip drains through the west perimeter dike and the construction of a bird sanctuary berm near the toe of the west dike on the factor of safety, and thus ability to raise the dike, were investigated. The results of each analysis or task are described in the following sections.

Task 1 - Verify Previous Stability Analyses

Ignore, only a check.

45. Six stability analyses reported by Fowler et al. (1987) were re-analyzed to ensure agreement between previous and current stability analyses (Table 3). Stability analyses 1, 2, 3, 4, 5 and 32 reported by Fowler et al. (1987) were conducted during this study using the same geometries, soil shear strengths, and analysis procedure (Spencer's Method) to ensure proper data input and execution of the slope stability microcomputer program UTEXAS2. These six analyses correspond to different elevations of the west perimeter dike and dredged material in the CIDMMA. The elevations of the dike crest and dredged material are presented in Table 3 and the dike cross sections are presented in Figures 15 through 20.

46. The factors of safety reported by Fowler et al. (1987) for these six cases are presented in Table 3 under the heading "Fowler et al. (1987) Safety Factor." The factors of safety for the same analyses conducted during this study are also presented in Table 3 under the heading "IL FS with Fowler et al. (1987) Strengths." It can be seen that there is excellent agreement between these factors of safety, and thus accurate modeling of the dike geometry, soil shear strengths, and analysis procedure.

A check

Task 2 - Stability Analyses Using 1992 Shear Strengths

Ignore, no longer relevant.

47. The six scenarios analyzed in Task 1 were re-evaluated using updated soil shear strength parameters for the dredged material and marine clay. The updated shear strength parameters were estimated from 1992 cone penetration test (CPT) results obtained at the site of the first strip drain test section in the north compartment (Figure 13). This information was extrapolated to the west perimeter dike to allow preliminary stability analyses to be conducted.

To accurately estimate the shear strength of the materials under the west perimeter dike, cone penetration tests through the dike were proposed during this analysis and conducted in 1993. The resulting 1993 shear strength parameters for the west perimeter dike are presented in Figure 14 and the corresponding stability analyses are described in Task 4. A comparison of the factor of safety for this task (Task 2) and Task 4 can be used to estimate the increase in undrained shear strength from 1992 to 1993, the difference in undrained shear strength between the containment area and under the west perimeter dike, and the resulting effect on dike stability.

48. The relationship between undrained shear strength and depth based on the 1992 CPT information at the test section is presented in Figure 13. The undrained shear strength (Su) was calculated using Equation 2 and a value of N_k equal to 12. Included in Figure 13 are Su relationships that correspond to undrained shear strengths ratios (Su/p') of 0.22 and 0.25. These values of Su/p' were used in the stability analyses and were estimated from the cone penetration data and Mesri (1989). The undrained shear strength of the dredged material varied considerably, probably due to sand lenses in the dredged fill. It was decided to assume a constant value of Su equal to 210 psf for the dredged fill in this task.

49. Stability analyses were conducted for Su/p' ratios of 0.22 and 0.25 for the six stability analyses described in Task 1. The results are summarized under "IL FS with 1992 Strengths" in Table 3. The shear strengths corresponding to an Su/p' of 0.22 did not vary significantly from the shear strength parameters used by Fowler et al. (1987), and thus the factors of safety are similar to those reported by Fowler et al. (1987). However, an Su/p' ratio of 0.25 yielded factors of safety that are 3 to 5% higher than those reported by Fowler et al. (1987). It should be noted again that these shear strengths were estimated from CPT results at the strip drain test section and extrapolated to the west perimeter dike.

Task 3 - Effect of Vertical Strip Drains Ignore

50. Analysis number 32 was re-analyzed to determine the effect of dredged fill and marine clay consolidation on the factor of safety of this cross-section (Figure 20). This dike geometry corresponds to raising the west dike to el. +34 ft CEMLW and the dredged material to el. +30 ft CEMLW. Consolidation of the dredged material and marine clay would be accelerated using vertical strip drains. The values of Su used in the analysis were estimated using an Su/p' ratio of 0.22 and 0.25 and the effective vertical stress after 100% consolidation. The relationships between undrained shear strength and depth after 100% consolidation are presented in Figure 13. Comparing the calculated profiles of Su in Figure 13, before and after strip drain installation, provides an insight to the degree of underconsolidation that currently exists in the dredged material and marine clay. If 100% consolidation is achieved, the undrained shear strength should increase by 40 to 50%.

51. The results of these stability analyses are presented in Table 3 under the heading "IL FS with Strip Drain Strengths." The increase in shear strength caused by consolidation of the dredged material and marine clay results in an increase in the factor of safety from 1.29 to 2.05 for an Su/p' ratio of 0.22, while the factor of safety increased from 1.35 to 2.12 for an Su/p' ratio of 0.25. In summary, consolidating the dredged material and marine clay will result in a substantial

increase in undrained shear strength (Figure 13), and thus factor of safety of the west perimeter dike.

Task 4 - Stability Analyses Using 1993 Shear Strengths

52. Analysis number 32 (Figure 21) was re-analyzed using the 1993 values of Su for the dredged material and marine clay. The 1993 Su values were obtained from cone penetration test results obtained in the west perimeter dike near the cross-section being evaluated (CPT test 93PC-12). The undrained shear strength profile for the west perimeter dike is presented in Figure 14 and the values of Su were obtained using Equation (2) and a value of N_k equal to 12. The shear strength parameters for the sand dikes were also revised using the cone penetration data from CPT 93PC-12. The drained friction angle, ϕ', for the starter dike was estimated to be 30 degrees, which is the same value used by Fowler et al. (1987). However, the remainder of the dike material was assigned a drained friction angle of 40 degrees based on cone penetration test 93PC-12, instead of 30 degrees as reported by Fowler et al. (1987). The effective cohesion, c', was assumed to be zero for all of the sands. These revised shear strength parameters for the dredged material, marine clay, and sand dikes (Figure 21) are used for the analyses in Tasks 4 through 8, and are considered to be representative of the current (1993) in-situ strengths at the location of CPT 93PC-12 in the west perimeter dike.

? $\frac{0.10}{1.28} = 0.078 \sim 8\%$

Repeat. *

53. The factor of safety for the stability analysis 32 and 1993 shear strength parameters is 1.38 (Table 4). Comparing the factors of safety for stability analysis 32 in Table 3 (1.28) and 1.38, it may be concluded that the use of an undrained strength stability analysis and 1993 cone penetration test results in an increase in factor of safety of approximately 10%. This increase is attributed to the higher undrained shear strengths estimated from the cone penetration test results instead of the 1981-1983 undrained triaxial and field vane shear test results used by Fowler et al. (1987).

Task 5 - Effect of Proposed Bird Sanctuary

Ignore.

54. A proposed "bird sanctuary" (an above water stability berm) was superimposed on the cross-section used in Task 4 (Figure 21) to determine the effect of constructing this berm on the computed factor of safety. This berm was suggested by the Norfolk District to create a bird habitat and efficiently use the sand that is proposed to be dredged in the Norfolk area. It can be seen from Table 4 that the "bird sanctuary" berm increases the factor of safety from 1.38 to 1.66. Therefore, the proposed "bird sanctuary" berm will have a positive effect on the stability of the west perimeter dike and provide a beneficial placement alternative for the proposed dredging of sand in the Norfolk area.

Task 6 - Stability of 1994 West Perimeter Dike Geometry

55. Stability analysis "a" (Table 5) was conducted using the January, 1994 west perimeter dike geometry at Station 75+67, provided by the Norfolk District, to evaluate the current (January, 1994) stability of the west perimeter dike. Figure 22 presents the January, 1994 dike geometry. A comparison of Figures 21 and 22 illustrates the difference in dike geometry from

analysis number 32 in Fowler et al. (1987) and the January, 1994 geometry at Station 75 +67. It can be seen that the January, 1994 elevation of the west perimeter dike is only 23.2 ft (Figure 22) instead of 34 ft (Figure 21). The shear strength parameters used in analysis "a" are presented in Figure 14 and described in Task 4. These values of Su represent the best estimate of the in-situ strengths at this location in 1993. With these updated shear strengths and geometry, stability analysis "a" represents the best estimate of the present (January, 1994) stability of the west perimeter dike at Station 75+67. The computed factor of safety is 1.91 (Table 5), which suggests that the west perimeter is stable against rotational failure in January, 1994. Field observations (no signs of distress) also suggest that the dike is stable. In summary, improved estimates of soil shear strength and dike geometry resulted in a substantial increase in the estimated factor of safety. Therefore, it appears possible to raise the west perimeter dike and increase the storage capacity of the CIDMMA without installing vertical strip drains.

1993 values

Current Jan 94

Task 7 - Raising of West Perimeter Dike Without Vertical Strip Drains

56. Stability analysis "b" (Table 5) was conducted to determine the maximum elevation that the west perimeter dike can be raised and maintain a factor of safety greater than or equal to 1.3 without installing vertical strip drains. The shear strength parameters used in analysis "b" are presented in Figure 14 and described in Task 4. The geometry corresponding to a factor of safety of 1.3 is presented in Figure 23. It can be seen from Figure 23 that the proposed dike raising follows a slope similar to the existing west perimeter dike. To maintain a factor of safety greater than or equal to 1.3, it is concluded that the maximum dike elevation is +33 feet CEMLW and the maximum dredged material elevation is +29 feet CEMLW (Table 5).

Lower than Fowler's because not all of berm present see next p.

Task 8 - Comparison of 1987 and 1994 Dike Raising Recommendations

57. Stability analysis 32 conducted by Fowler et al. (1987) concluded that the maximum dike and dredged material elevations could be +34 feet and +30 feet CEMLW, respectively. The maximum elevations of the dike and dredged fill (+33 feet and +29 feet CEMLW) calculated during this study are slightly lower than those reported by Fowler et al. (1987). This is attributed to the absence of two berms that were incorporated into the cross-section analyzed by Fowler et al. (1987). The missing berms are shown in Figure 23. These berms were proposed by Fowler et al. (1987) to increase stability, however, the January, 1994 survey clearly shows that this material is not in the cross-section.

58. Stability analysis "c" (Table 5) was conducted to determine the effect of the missing berm on the factor of safety. Therefore, stability analysis "c" utilized the same geometry as stability analysis 32 conducted by Fowler et al. (1987). The only difference between this analysis is that the shear strength parameters were obtained from CPT 93PC-12 data. The shear strength parameters used in analysis "c" are presented in Figure 14 and described in Task 4. The 1993 shear strength parameters result in an increase in factor of safety from 1.28 to 1.38. This corresponds to a maximum dike and dredged material elevation of +34 feet and +30 feet CEMLW, respectively. Therefore, the absence of these berms has a 5 to 10% effect on the factor of safety because the critical slip circle passes through the outer berm (Figure 23). In summary, the shear strength parameters estimated from CPT 93PC-12 resulted in an increase in the factor of

safety of approximately 10% from the Fowler et al. (1987) study. In addition, the current west perimeter dike can be safely raised (factor of safety greater than 1.3) to el. +33 ft CEMLW and the dredged material can be raised to +29 ft CEMLW under the current (January, 1994) conditions without the additional berms proposed by Fowler et al. (1987) present. If the additional berms proposed by Fowler et al. (1987) are placed, the dike and dredged material can be raised to +37.5 ft and +33.5 ft CEMLW, respectively and will have a factor of safety equal to 1.3. The results of this analysis are presented as Stability Analysis "d" in Table 5.

Task 9 - Effect of Vertical Strip Drains on 1994 Dike Geometry

59. The 1994 west perimeter dike geometry (Analysis "a") was re-analyzed to determine the effect of consolidation of the dredged material and marine clay. Consolidation could be accelerated by the installation of strip drains through the west perimeter dike. The undrained shear strength values corresponding to 100% consolidation of the dredged material and marine clay were estimated using an Su/p' ratio of 0.22 and 0.25 and the effective stress corresponding to the dike geometry after 100% consolidation. Stability analyses "e" and "f" were conducted with Su values corresponding to Su/p' ratios of 0.22 and 0.25, respectively. The results of the analyses are presented in Table 5. The installation of strip drains, and the subsequent consolidation of the dredged material and marine clay, results in factors of safety of 2.43 and 2.48 for Su/p' values of 0.22 and 0.25, respectively. These factors of safety are significantly greater than 1.91, which corresponds to the January, 1994 condition (Analysis "a"). Therefore, strip drain installation will cause a significant increase in soil shear strength and dike stability. This increase will allow the dike to be raised as discussed in Task 10.

Task 10 - Effect of Vertical Strip Drains on Raised Dike Geometry ignore

60. The effect of installing strip drains, and thus consolidating the dredged material and marine clay, on the west dike geometry corresponding to crest and dredged material elevations of +33 feet and +29 feet CEMLW (Analysis "b") was also evaluated. This geometry corresponds to the maximum dredged material and marine clay elevation for a factor of safety greater than or equal to 1.3 with 1993 shear strength values. The undrained shear strength corresponding to 100% consolidation of the dredged material and marine clay were estimated using an Su/p' ratio of 0.22 and 0.25, and the effective stresses corresponding to the dike geometry after 100% consolidation. Stability analyses "g" and "h" were conducted using Su/p' ratios of 0.22 and 0.25, respectively (Table 5). It can be seen that the installation of vertical strip drains in the perimeter dike, and thus 100% consolidation of the dredged material and marine clay, will increase the factor of safety from 1.3 (1993 shear strengths) to 1.92 and 2.15 for Su/p' ratios of 0.22 and 0.25, respectively. Therefore, installation of strip drains will allow the west perimeter dike to be raised significantly higher than the proposed elevation of +33 ft CEMLW as discussed in Task 11.

Task 11 - Raising of West Perimeter Dike After Strip Drain Installation

61. Stability analysis "i" was conducted to determine the maximum elevation that the west perimeter dike could be raised and maintain a factor of safety greater than or equal to 1.3

after strip drain installation. This analysis was conducted using shear strengths that correspond to 100% consolidation. Stability analysis "i" was conducted using an Su/p' ratio of 0.22 (Table 5).

62. The geometry corresponding to a factor of safety of 1.3 for an undrained strength ratio of 0.22 is presented in Figure 24. It can be seen from Figure 24 that the proposed dike raising follows a slope similar to the existing west perimeter dike. To maintain a factor of safety greater than or equal to 1.3, it is concluded that the maximum dike elevation is +58 feet CEMLW and the maximum dredged fill elevation is +54 feet CEMLW. These elevations are significantly higher than the +33 and +29 ft CEMLW, respectively, previously reported. The large increase in maximum dike elevation is attributed to a large increase in the radius of the critical circle. A longer critical slip radius forces the critical slip surface farther into the containment area, which reduces the driving moment and increases the amount of shear resistance mobilized along the length of the failure circle. This is evident from a comparison of the critical slip circles in Figures 23 and 24.

63. Stability analysis "j" was conducted to determine the maximum elevation that the west perimeter dike can be raised and maintain a factor of safety greater than or equal to 1.3 after strip drain installation using an Su/p' ratio of 0.25 and 100% consolidation. The geometry corresponding to a factor of safety of 1.3 for an undrained strength ratio of 0.25 is presented in Figure 25. It can be seen from Figure 25 that the proposed dike raising follows a slope similar to the existing west perimeter dike. To maintain a factor of safety greater than or equal to 1.3, it is concluded that the maximum dike elevation is +66 feet CEMLW and the maximum dredged fill elevation is +62 feet CEMLW.

64. In summary, the installation of vertical strip drains will cause a substantial increase in shear strength of the dredged material and marine clay underlying the west perimeter dike. This should allow the west dike to be raised to an elevation of +58 to +66 ft CEMLW and substantially increase the service life of the CIDMMA. The time required for the strip drains to achieve 100% consolidation depends on the spacing of the drains. Value engineering should be performed to determine the optimal cost and drain spacing for 100% consolidation.

Figure 14
24
Table 5

PART V: CONCLUSIONS AND RECOMMENDATIONS

65. This study investigated the stability of the existing west perimeter dike at the Craney Island Dredged Material Management Area using an undrained strength stability analysis. An undrained stability analysis expresses the undrained strength in terms of the effective preconsolidation pressure. This allows the undrained strength to reflect a strength increase caused by consolidation. The undrained strength stability analysis was used to evaluate the current stability of the west perimeter dike and the possibility of raising the dike. In addition, stability analyses were conducted to determine the effect of installing vertical strip drains on the stability and potential raising of the dike. Vertical strip drains will accelerate consolidation of the dredged material and underlying marine clay. Consolidation of these materials will result in an increase in shear strength and stability.

66. Stability analyses of the west perimeter dike revealed that the critical slip surfaces are tangent to an elevation of approximately -100 ft CEMLW at Station 75+67 (Figure 1) on the west perimeter dike. Below this elevation the marine clay shear strength increases due to the dissipation of excess pore-water pressures by the underlying dense sand. As a result, the critical slip circles do not extend below this depth. Therefore, at Station 75+67, the vertical strip drains need to extend to a depth approximately 10 ft above the dense sand. This corresponds to el. -100 ft CEMLW or a depth of 115 ft from the top of CPT location 93PC-12. The elevation of the dike at CPT 93PC-12 is +12 ft CEMLW. Cone penetration tests should be conducted along the west perimeter dike to identify the optimal depth that strip drains should be installed along the dike.

67. The factor of safety of the west perimeter dike at Station 75+67 is 1.91. This factor of safety reflects the January, 1994 geometry and 1993 values of undrained shear strength. Therefore, it was concluded that it is technically feasible to raise the west perimeter dike without installing strip drains. Stability analyses showed that the west dike and dredged material can be raised to elevation +33 and +29 ft CEMLW, respectively, using the geometry shown in Figure 23 and still exhibit a factor of safety of 1.3. If the two berms proposed by Fowler et al. (1987) and shown in Figure 23 are placed, the dike and dredged material can be raised to elevations +37.5 and +33.5 ft CEMLW, respectively, and exhibit a factor of safety of 1.3.

68. If vertical strip drains are installed, the undrained shear strength of the dredged material and marine clay should increase, and thus increase the factor of safety at Station 75+67 to approximately 2.5. A factor of safety of 2.5 is based on January, 1994 dike geometry and 1993 values of undrained shear strength. This increase in shear strength will allow the west perimeter dike to be raised to an elevation of at least +58 ft CEMLW after 100% consolidation is achieved in the dredged material and marine clay. This raising should substantially increase the service life of the Craney Island Dredged Material Management Area. The time required for the strip drains to achieve 100% consolidation depends on the spacing of the drains. Value engineering should be performed to determine the optimal cost and drain spacing for 100% consolidation.

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Table 3. - Results of Stability Analyses Conducted in Tasks 1 through 3

Fowler et al. (1987) Stability Run	Dike Crest Elevation (ft)	Dredged Material Elevation (ft)	Geometry Comments	Critical Circle Center X (ft)	Critical Circle Center Y (ft)	Critical Circle Radius (ft)	Circle Tangent Elevation (ft)	Side Force Inclination (degrees)	Fowler et al. (1987) Safety Factor	II. FS with Fowler et al. (1987) Strengths	Strength Parameters from First Strip Drain Test Section: Su/p' = 0.22, II. FS with 1992 Strengths	Su/p' = 0.22, II. FS with Strip Drain Strengths	Su/p' = 0.25, II. FS with 1992 Strengths	Su/p' = 0.25, II. FS with Strip Drain Strengths
1	26	17	No Berm	25.5	145.9	199.7	-53.8	3.42	1.20	1.20	1.18		1.23	
2	26	19	No Berm	26.2	147.7	200.6	-52.9	3.49	1.18	1.19	1.17		1.21	
3	26	22	No Berm	26.7	153.6	204.5	-50.9	3.63	1.16	1.17	1.14		1.18	
4	34	22	No Berm	58.3	195	253.2	-58.2	3.86	0.99	1.03	1.04		1.09	
5	34	30	No Berm	60.6	210	264.1	-54.1	4.09	0.95	1.00	1.01		1.05	
32	34	30	Berm	83.5	253	317.7	-64.7	2.45	1.28*	1.28	1.29	2.05	1.35	2.12

* Corrected from FS = 1.3 because wrong strength was assigned to soft bay bottom sediments

Table 4. - Results of Stability Analyses Conducted in Tasks 4 and 5

Dike Crest Elevation (ft)	Dredged Material Elevation (ft)	Strength Comments	Geometry Comments	Critical Circle Center X (ft)	Critical Circle Center Y (ft)	Critical Circle Radius (ft)	Circle Tangent Elevation (ft)	Side Force Incl. (degrees)	Factor of Safety
34	30	1993 Strengths	Analysis 32 Geometry	62.75	260	339.2	-79.2	1.9	1.38
34	30	1993 Strengths	Analysis 32 Geometry with "Bird Sanctuary" Berm	108.5	163.5	242.45	-79.0	1.44	1.66

Table 5. - Results of Stability Analyses Conducted in Tasks 6 through 11

Stability Run	Dike Crest Elevation (ft)	Dredged Material Elevation (ft)	Analysis Comments	Critical Circle Center X (ft)	Critical Circle Center Y (ft)	Critical Circle Radius (ft)	Circle Tangent Elevation (ft)	Side Force Inclination (degrees)	Strength Parameters from West Perimeter Dike — Factor of Safety: January, 1994 Geometry	Strip Drains and Su/p' = 0.22	Strip Drains and Su/p' = 0.25
a	23.2	19	1994 Geometry & Shear Strengths	60.50	120.25	207.50	-87.25	1.17	1.91	----	----
b	33	29	Raise Dike to FS = 1.3	102.75	165.50	253.50	-88.00	1.73	1.30	----	----
c	34	30	1993 Strengths and Analysis 32 Geometry	62.75	260.00	339.20	-79.20	1.90	1.38	----	----
d	37.5	33.5	1993 Strengths and Raised Analysis 32 Geometry	94.50	322.50	401.95	-79.45	1.95	1.30	----	----
e	23.2	19	1994 Geometry & Strip Drain Strengths (.22)	70.00	138.75	198.50	-59.75	2.29	----	2.43	----
f	23.2	19	1994 Geometry & Strip Drain Strengths (.25)	71.00	141.75	198.50	-56.75	2.64	----	----	2.48
g	33	29	Maximum Geometry & Strip Drain Strengths (.22)	107.75	195.25	257.75	-62.50	3.24	----	1.92	----
h	33	29	Maximum Geometry & Strip Drain Strengths (.25)	105.25	207.25	266.00	-58.75	3.51	----	----	2.15
i	58	54	100% Consolidation & Raise Dike to FS = 1.3	191.75	443.45	514.75	-71.30	4.16	----	1.30	----
j	66	62	100% Consolidation & Raise Dike to FS = 1.3	222.50	579.95	651.25	-71.30	4.42	----	----	1.30

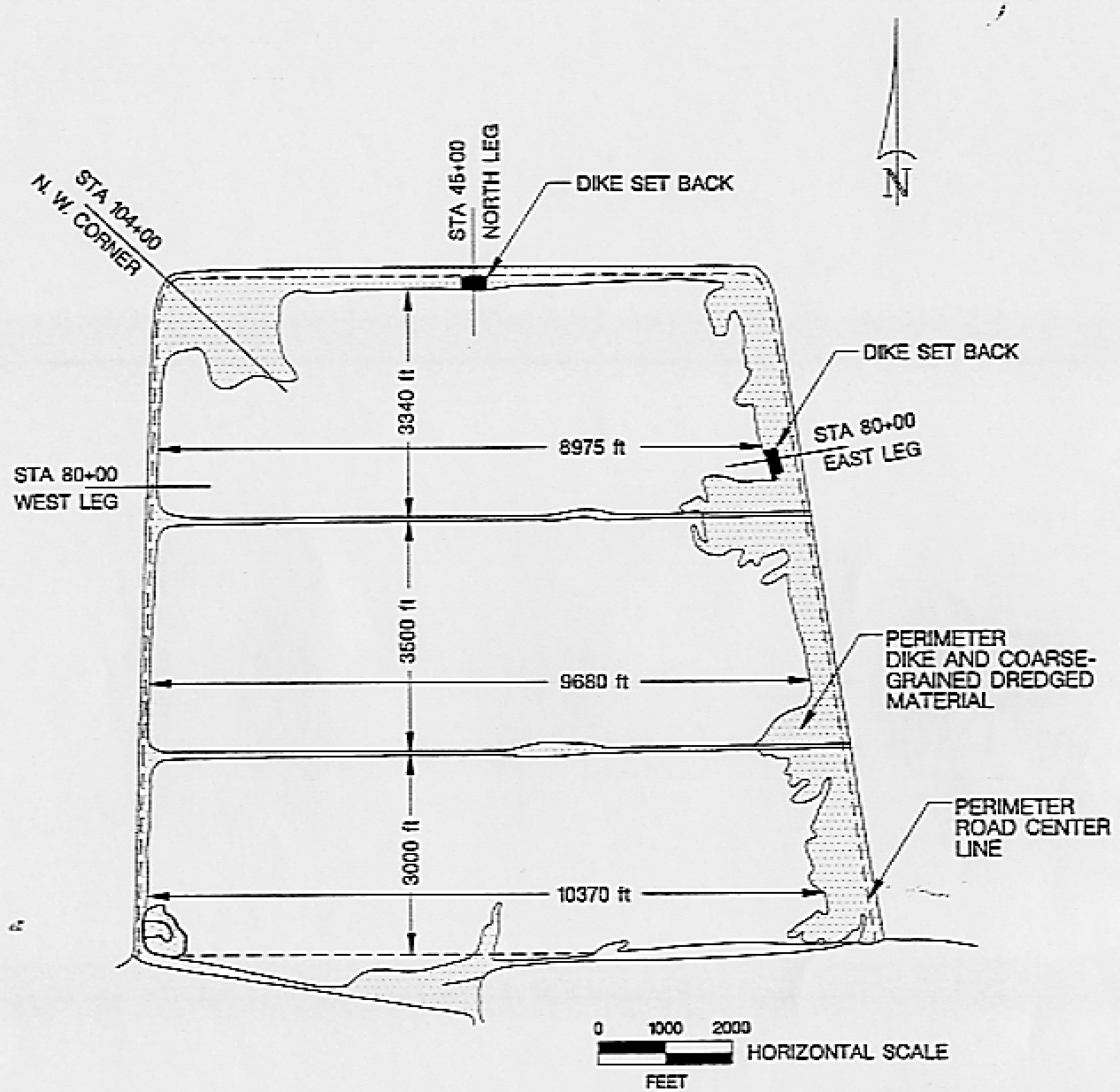


Figure 1. Plan View of Craney Island and Location of Dike Cross Section

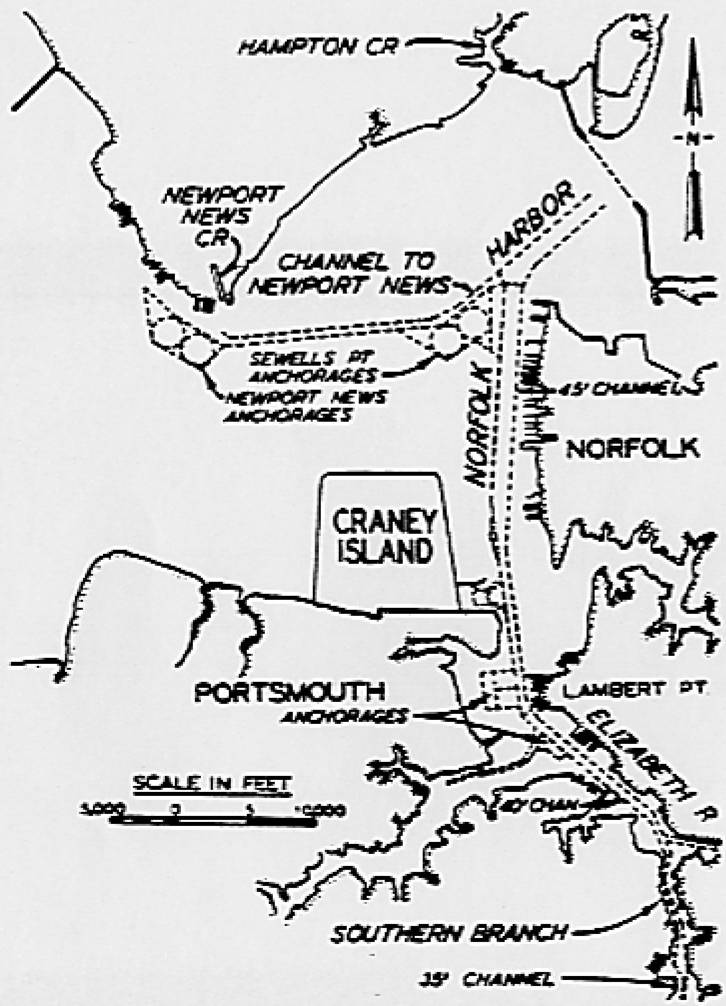


Figure 2. Project Location (from Fowler et al. 1987)

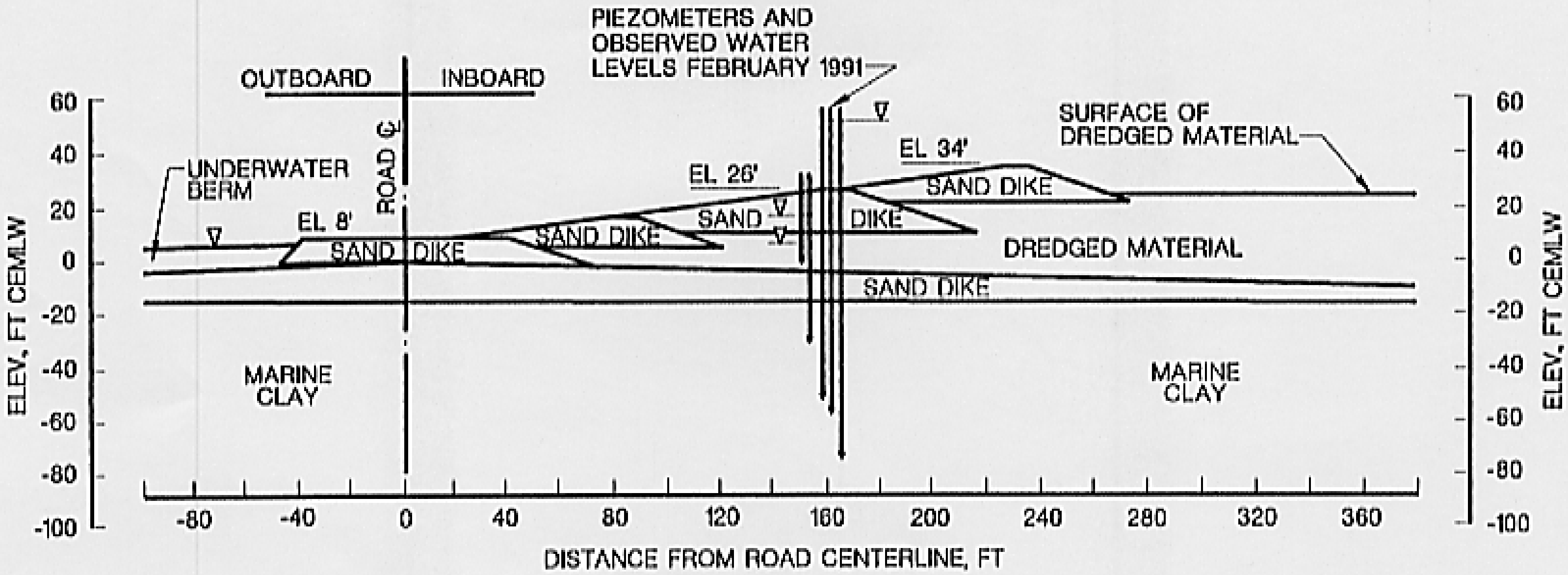


Figure 3. Generalized Cross-Section, West Perimeter Dike
(from Fowler et al. 1987)

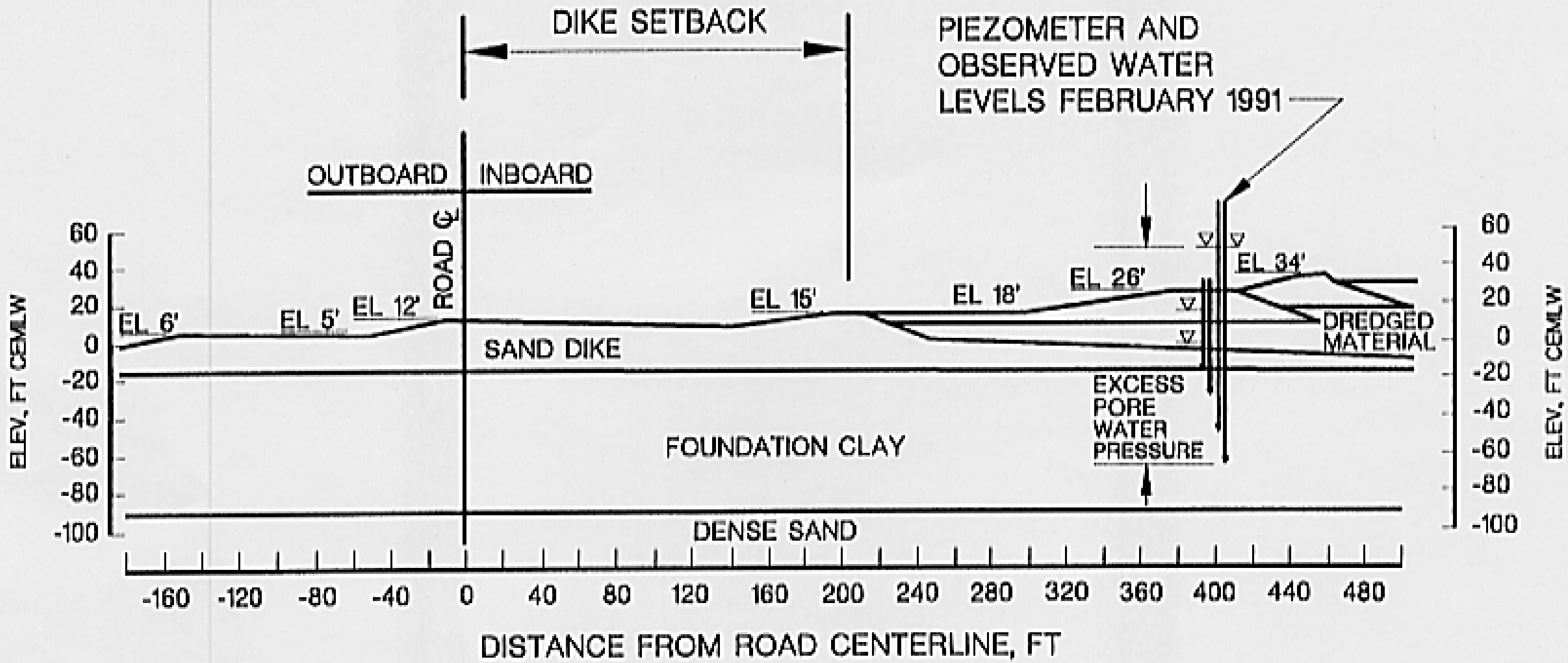


Figure 4. Generalized Cross-Section, Northwest Corner Perimeter Dike (from Fowler et al. 1987)

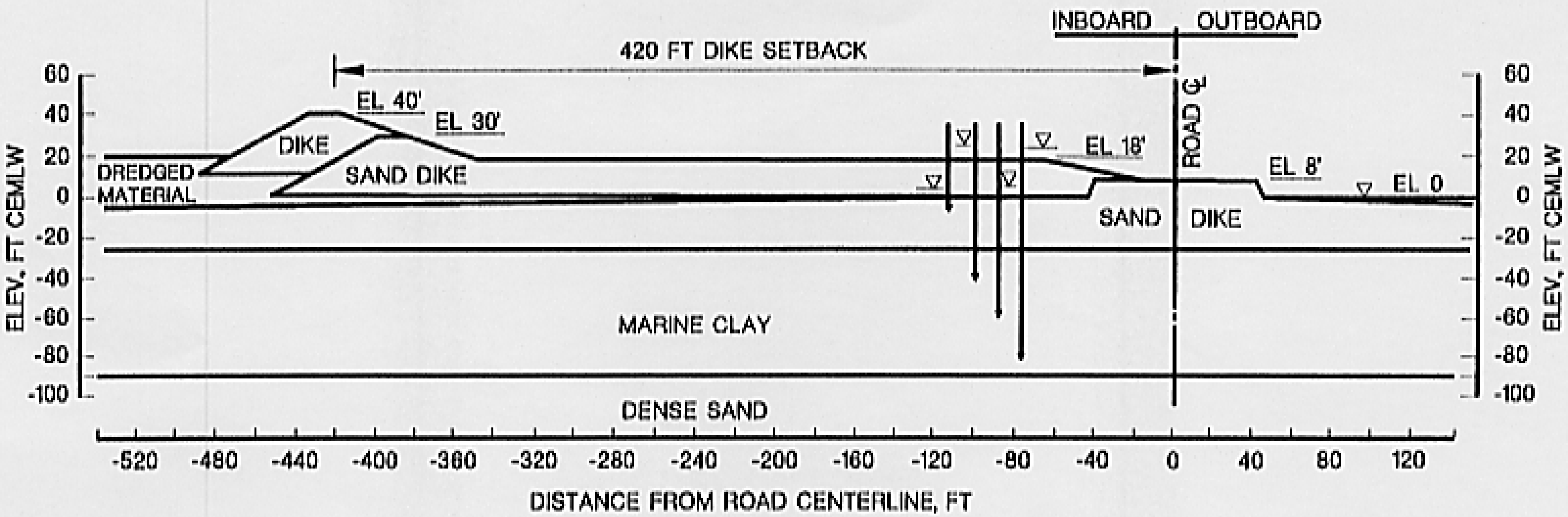


Figure 5. Generalized Cross-Section, North Perimeter Dike (from Fowler et al. 1987)

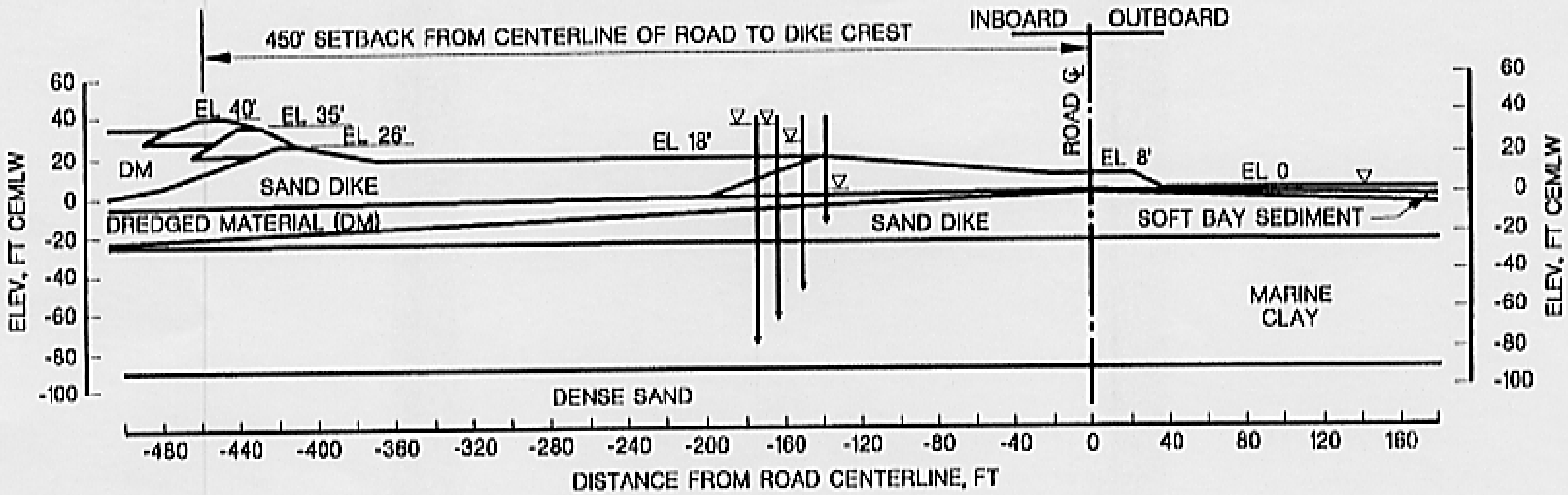


Figure 6. Generalized Cross-Section, East Perimeter Dike
(from Fowler et al. 1987)

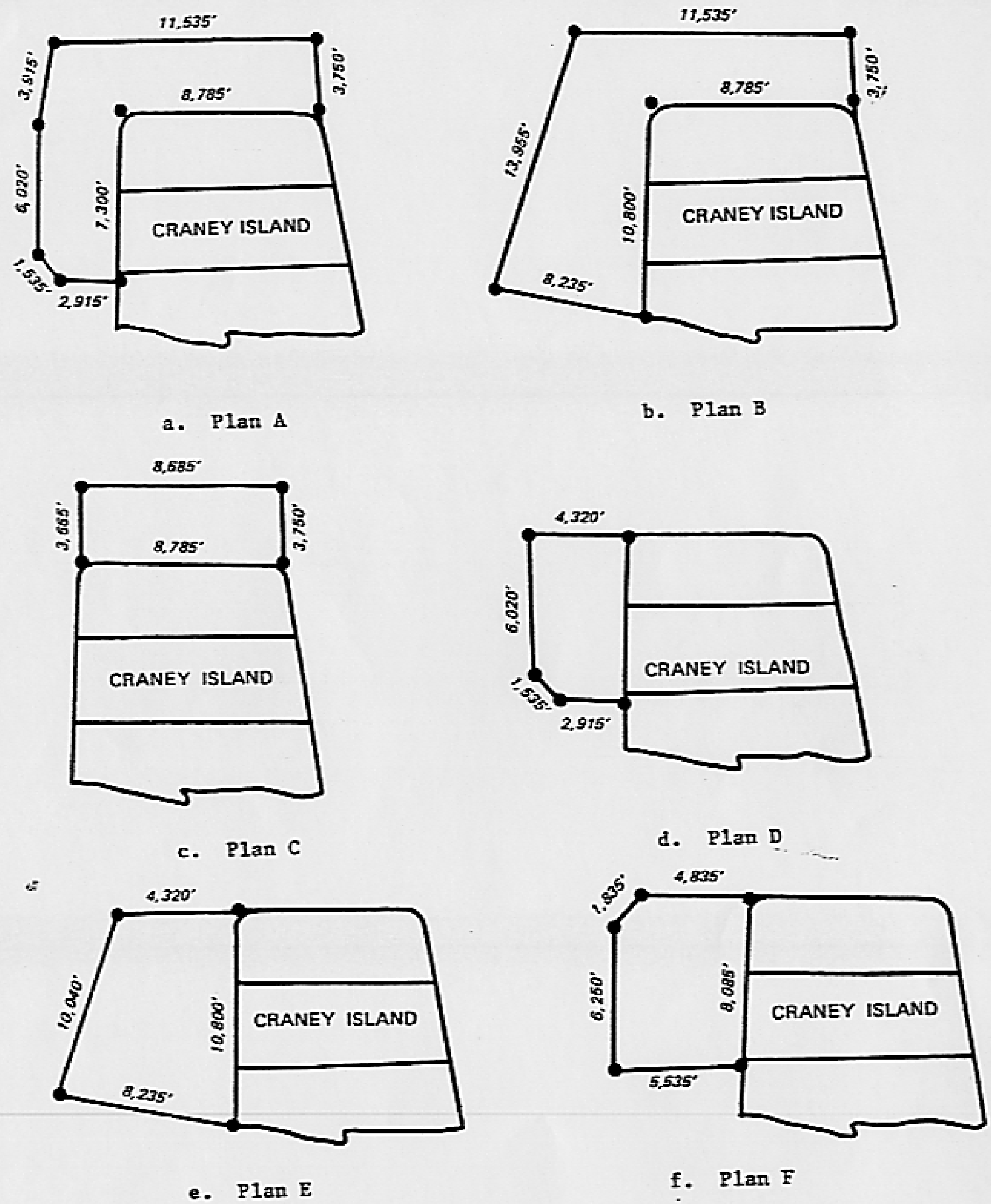


Figure 7. Dimensions of Craney Island Enlargement Alternatives (from Spigolon and Fowler 1987)

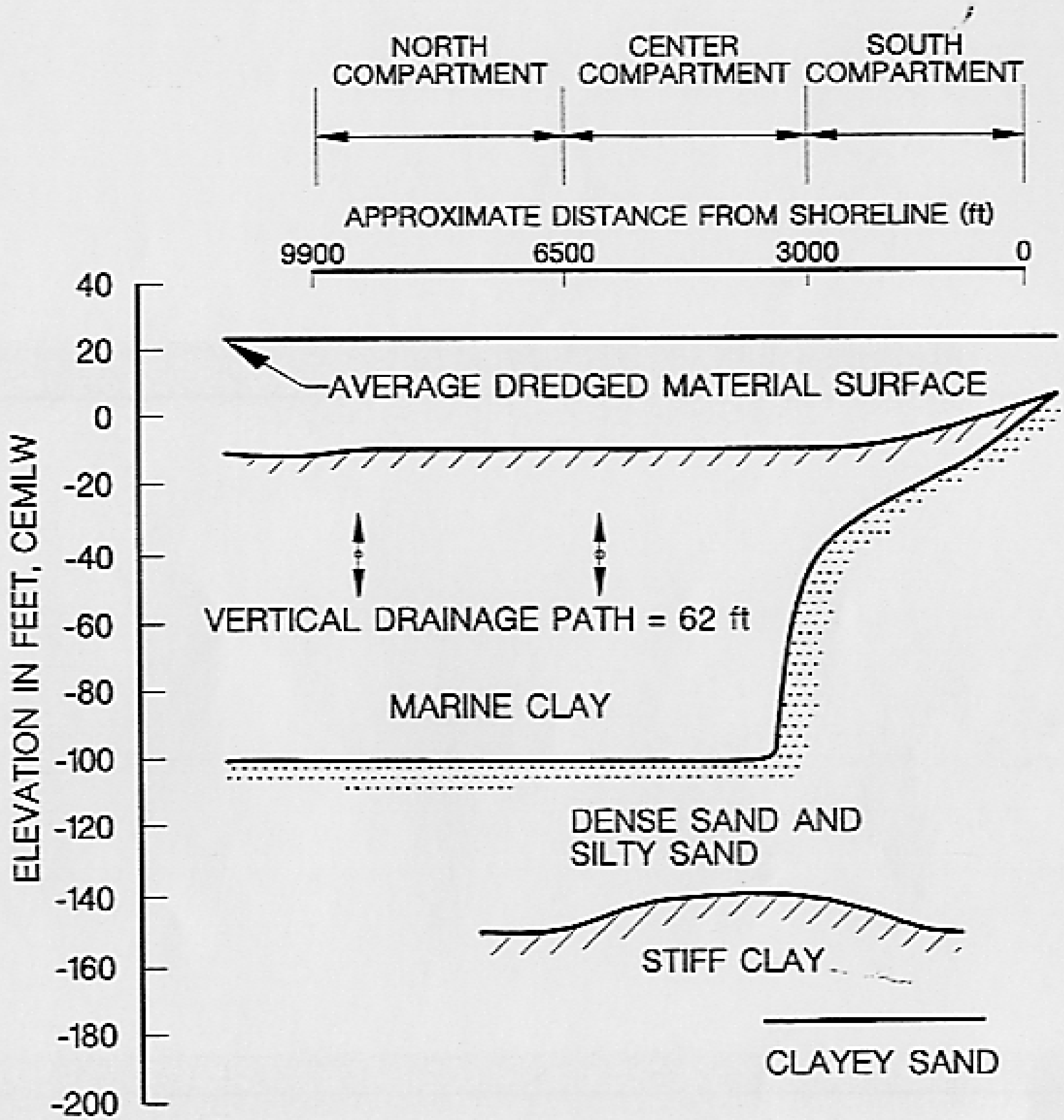


Figure 8. Generalized Subsurface Profile at Craney Island

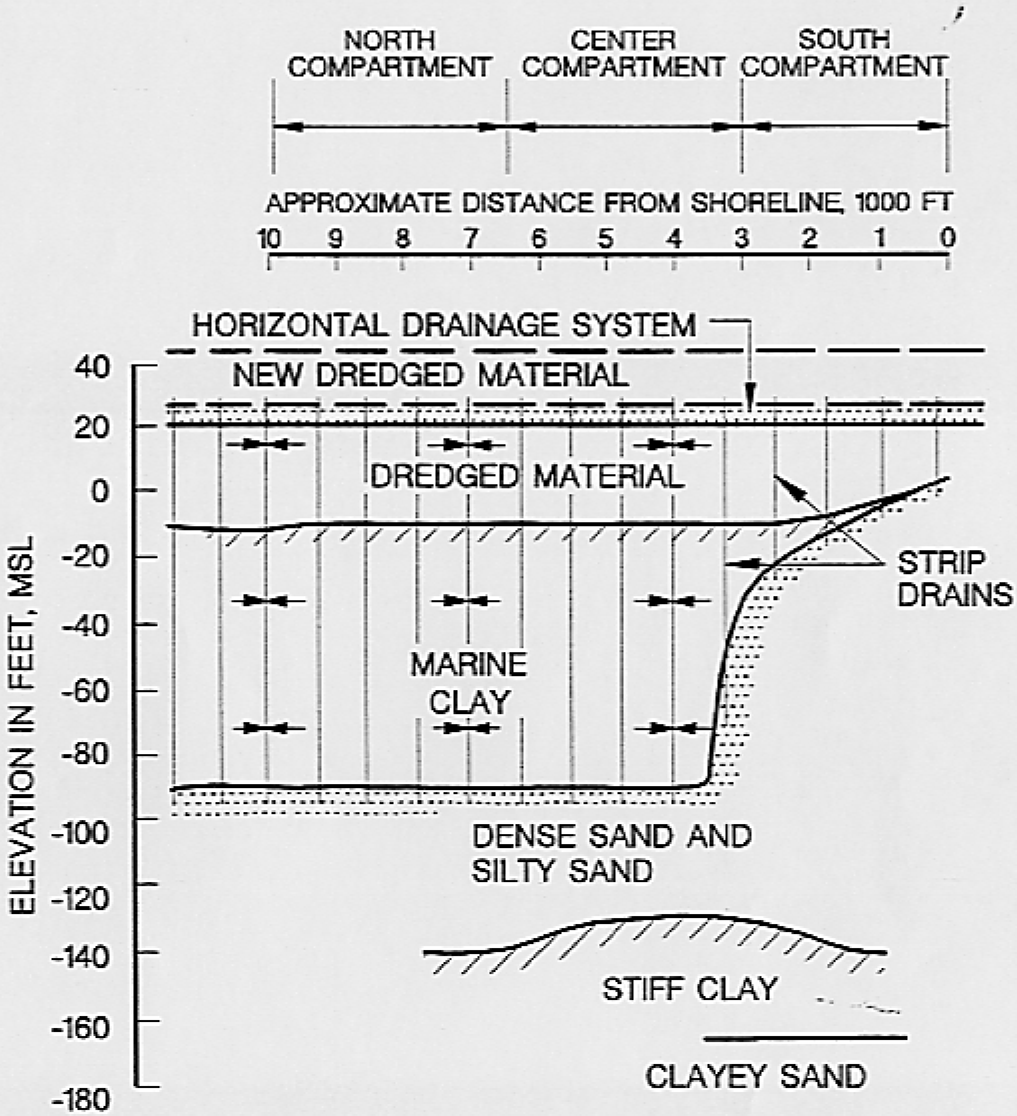


Figure 9. Radial Drainage Pattern Using Vertical Strip Drains

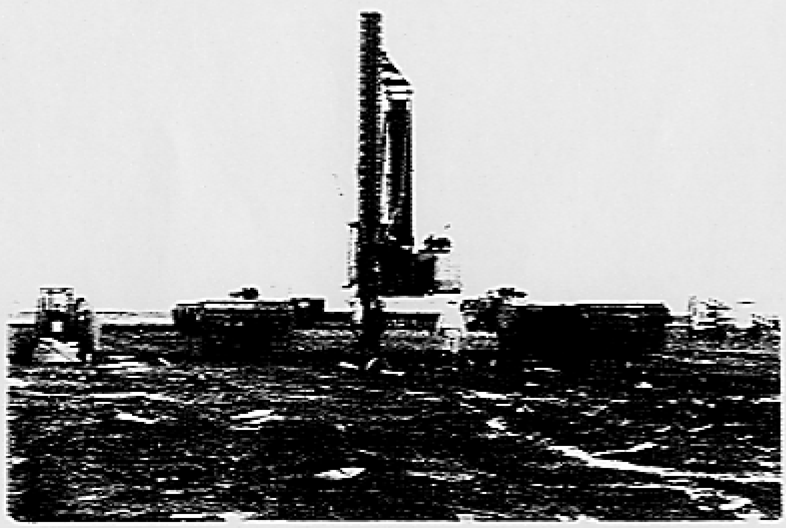


Figure 10. Strip Drain Installation Equipment

Figure 11. Strip Drain Installation Procedure

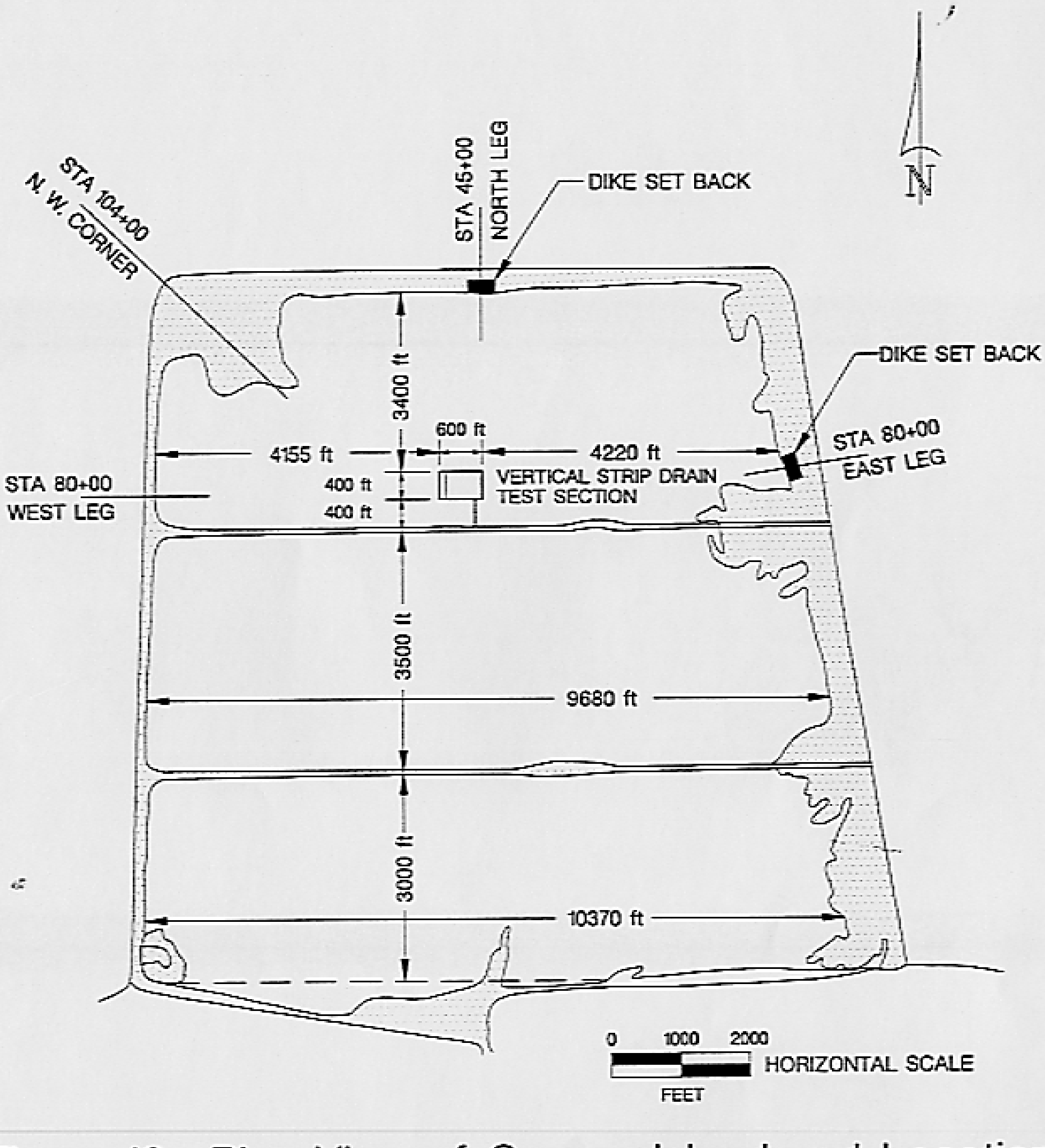


Figure 12. Plan View of Craney Island and Location of Vertical Strip Drain Test Section

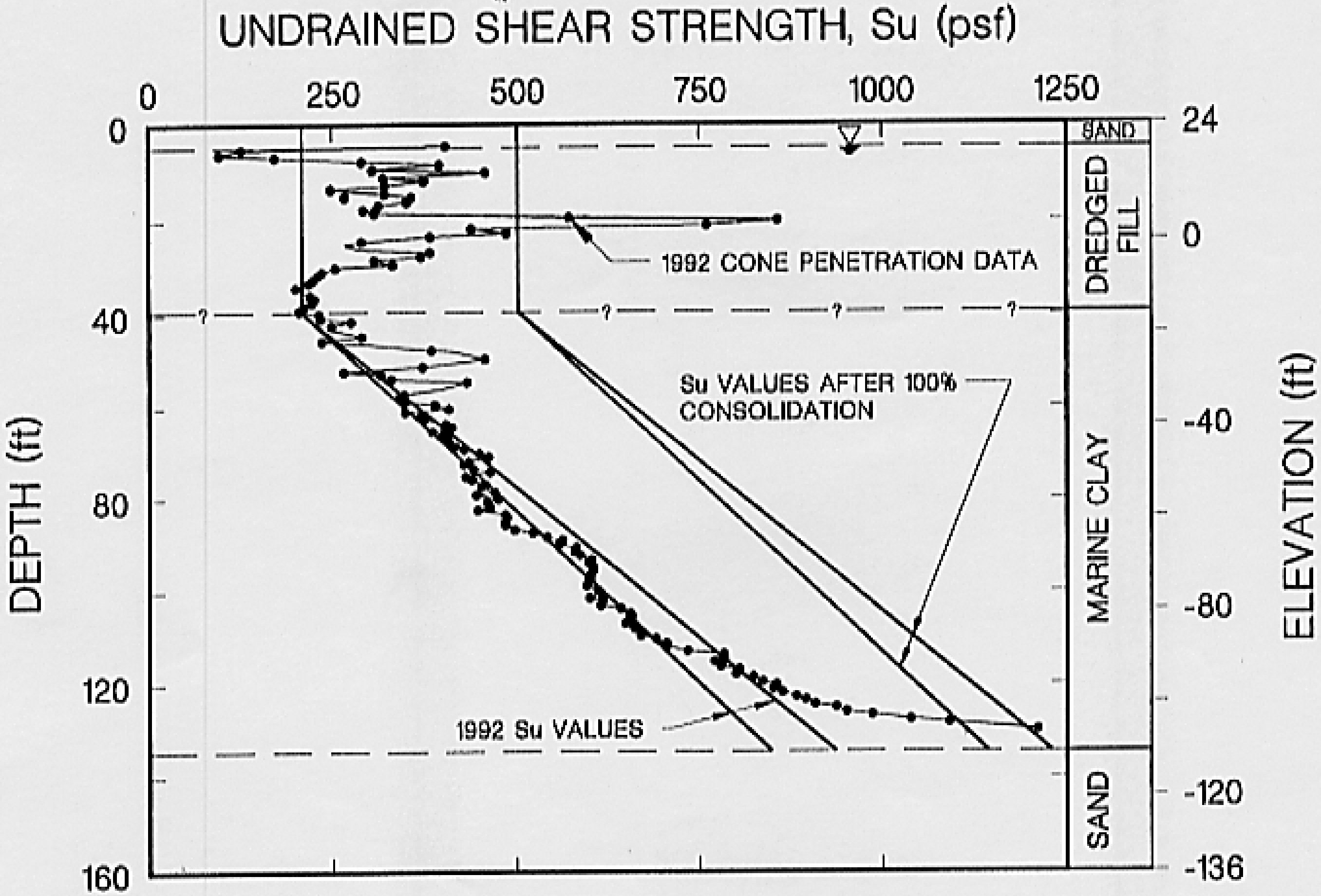


Figure 13. Undrained Shear Strength Profile Below Craney Island Strip Drain Test Section

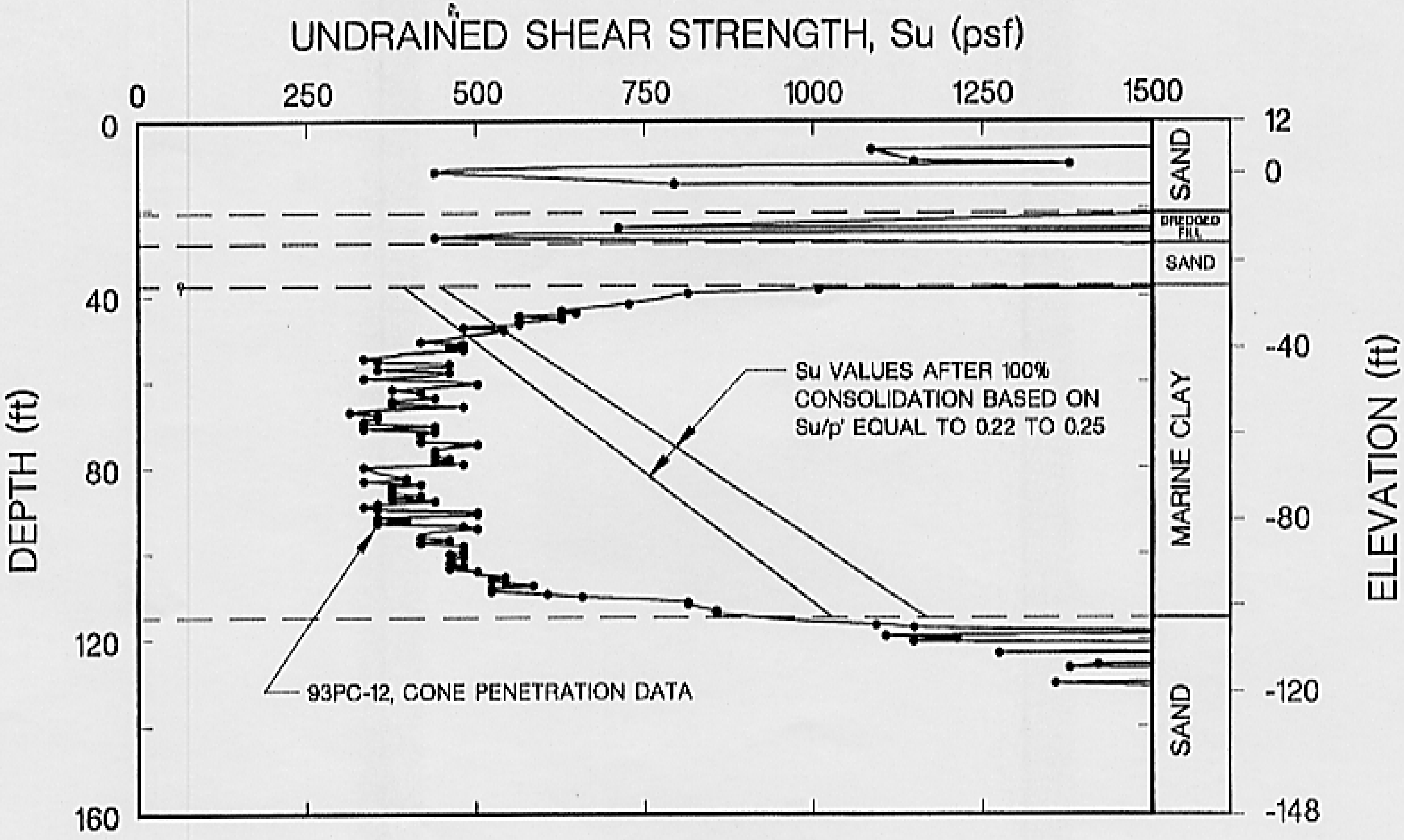
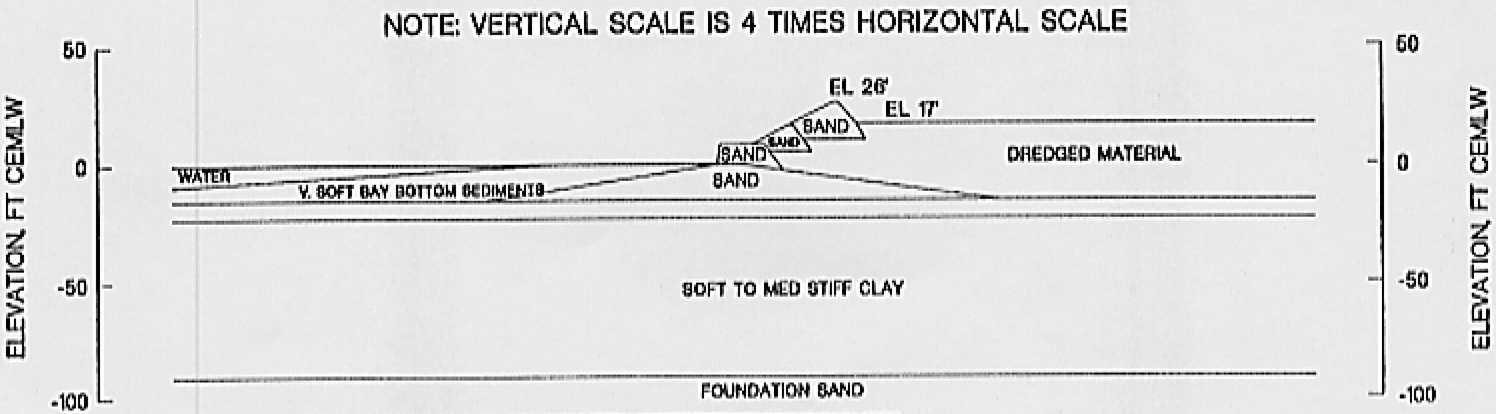


Figure 14. Undrained Shear Strength Profile Below Craney Island West Perimeter Dike



TYPE	SOIL PARAMETERS UNIT WEIGHT (PCF) (SATURATED)	Q SHEAR STRENGTH (Ø, PSF) FRICTION	COHESION
CLAY	100	0	275+4.75/FT DEPTH
DIKE & BERM SAND	110	30	0
CONSOLIDATED DREDGED MATERIAL	90	0	100
CONSOLIDATING DREDGED MATERIAL	90	0	35+18/FT DEPTH
SOFT BAY SEDIMENTS	95	0	50

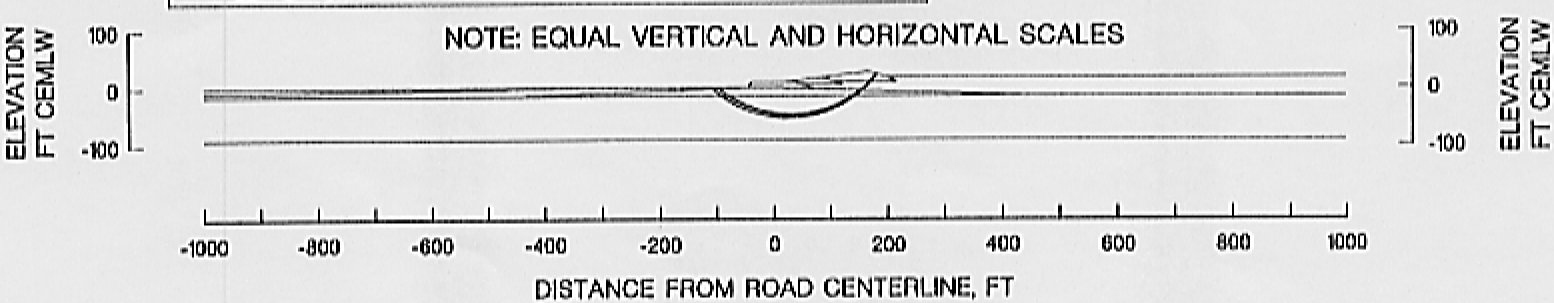
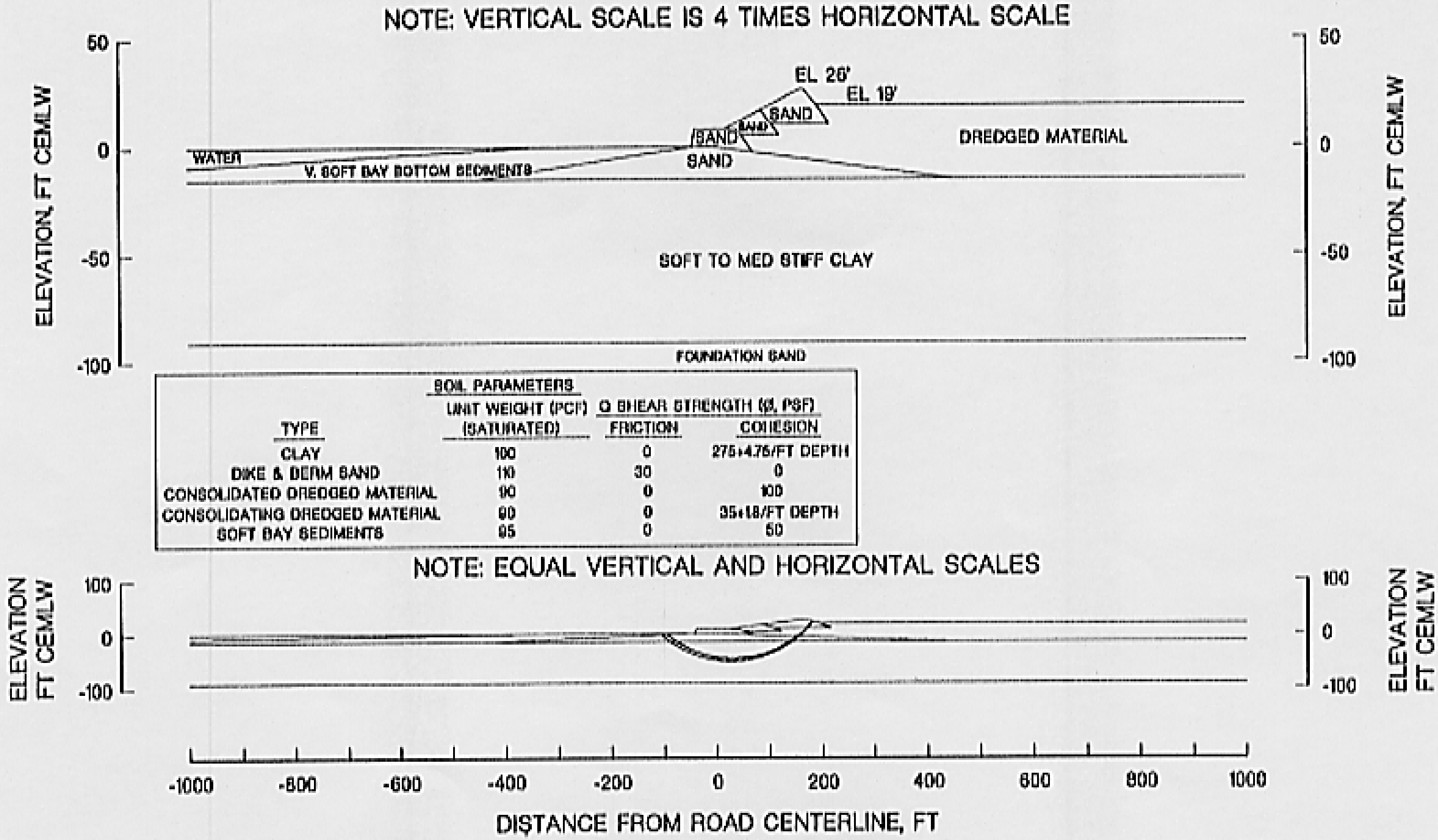


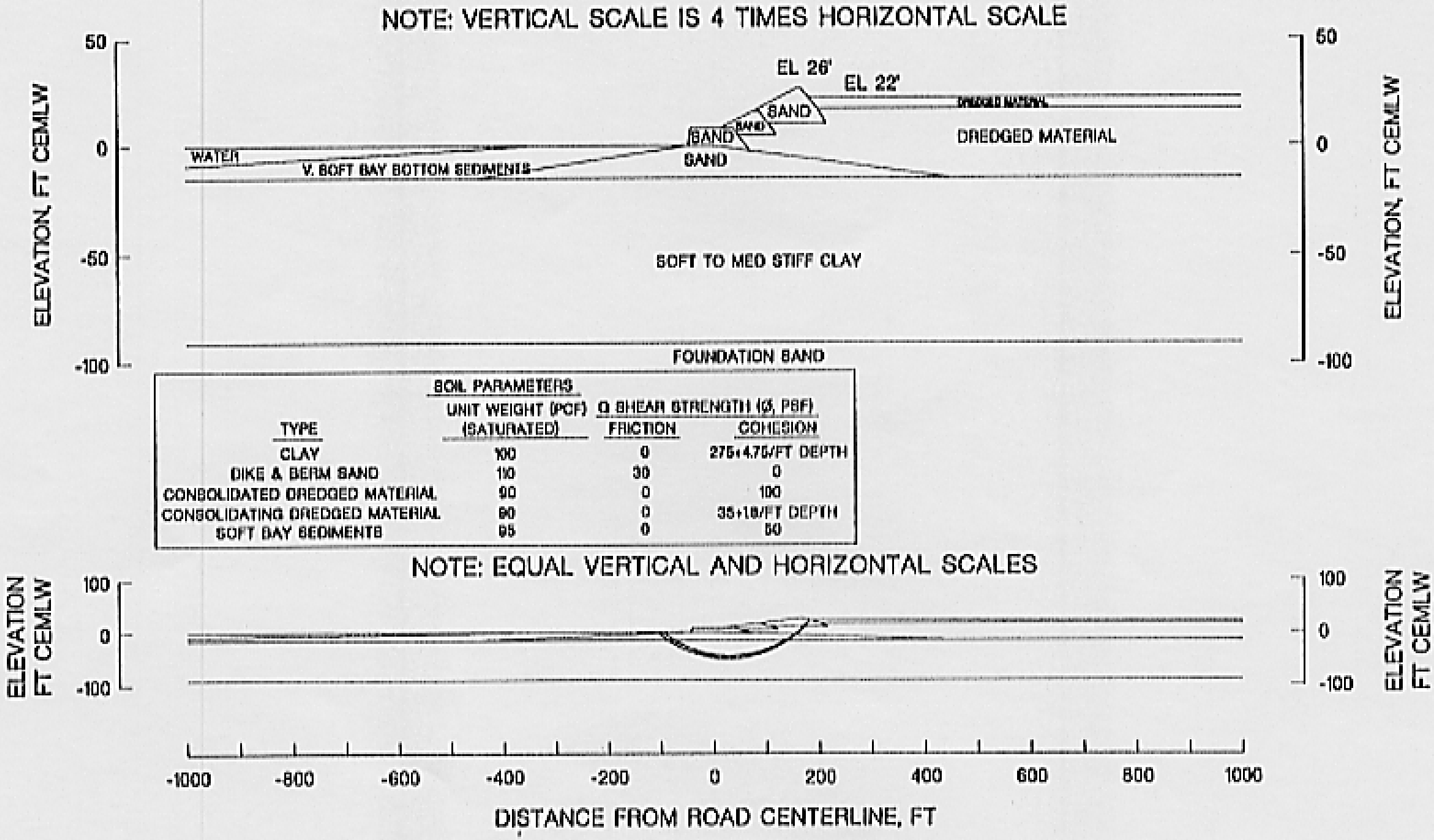
Figure 15. Geometry of Stability Analysis 1 from Fowler et al. (1987)



SOIL PARAMETERS

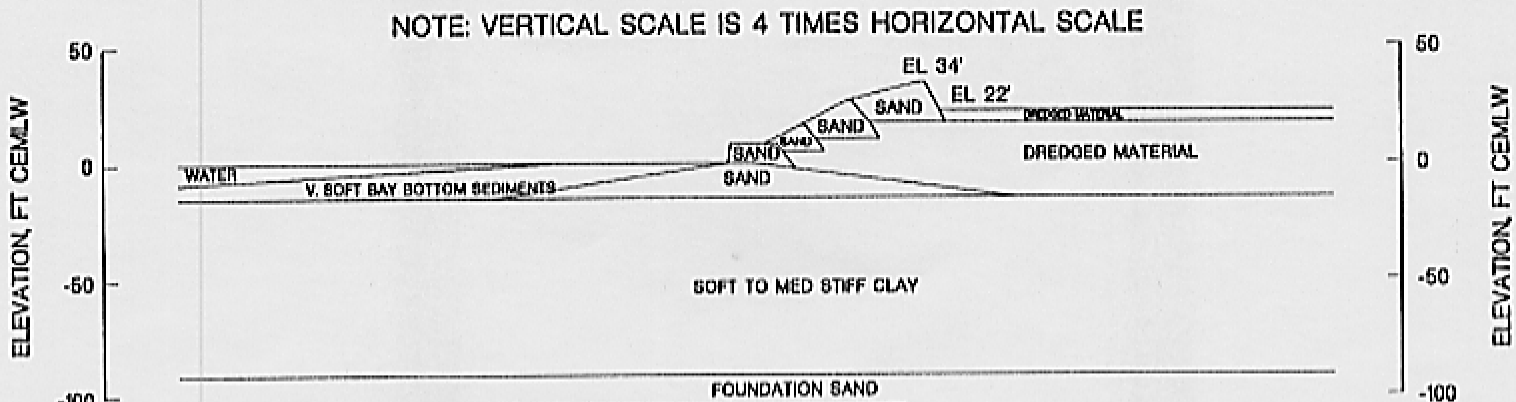
TYPE	UNIT WEIGHT (PCF) (SATURATED)	Q SHEAR STRENGTH (Ø, PSF) FRICTION	COHESION
CLAY	100	0	275+4.75/FT DEPTH
DIKE & BERM SAND	110	30	0
CONSOLIDATED DREDGED MATERIAL	90	0	100
CONSOLIDATING DREDGED MATERIAL	90	0	35+18/FT DEPTH
SOFT BAY SEDIMENTS	95	0	50

Figure 16. Geometry of Stability Analysis 2 from Fowler et al. (1987)



TYPE	SOIL PARAMETERS: UNIT WEIGHT (PCF) (SATURATED)	Q SHEAR STRENGTH (Ø, PSF): FRICTION	COHESION
CLAY	100	0	275+4.75/FT DEPTH
DIKE & BERM SAND	110	30	0
CONSOLIDATED DREDGED MATERIAL	90	0	100
CONSOLIDATING DREDGED MATERIAL	90	0	35+18/FT DEPTH
SOFT BAY SEDIMENTS	95	0	50

Figure 17. Geometry of Stability Analysis 3 from Fowler et al. (1987)



SOIL PARAMETERS

TYPE	UNIT WEIGHT (PCF) (SATURATED)	Q SHEAR STRENGTH (Ø, PSF) FRICTION	COHESION
CLAY	100	0	275+4.75/FT DEPTH
DIKE & BERM SAND	110	30	0
CONSOLIDATED DREDGED MATERIAL	90	0	100
CONSOLIDATING DREDGED MATERIAL	90	0	35+18/FT DEPTH
SOFT BAY SEDIMENTS	95	0	50

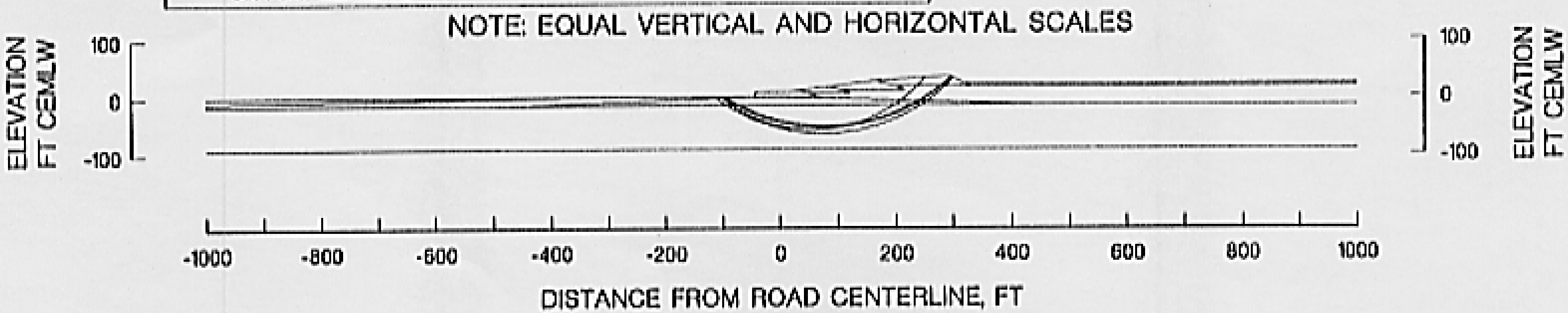
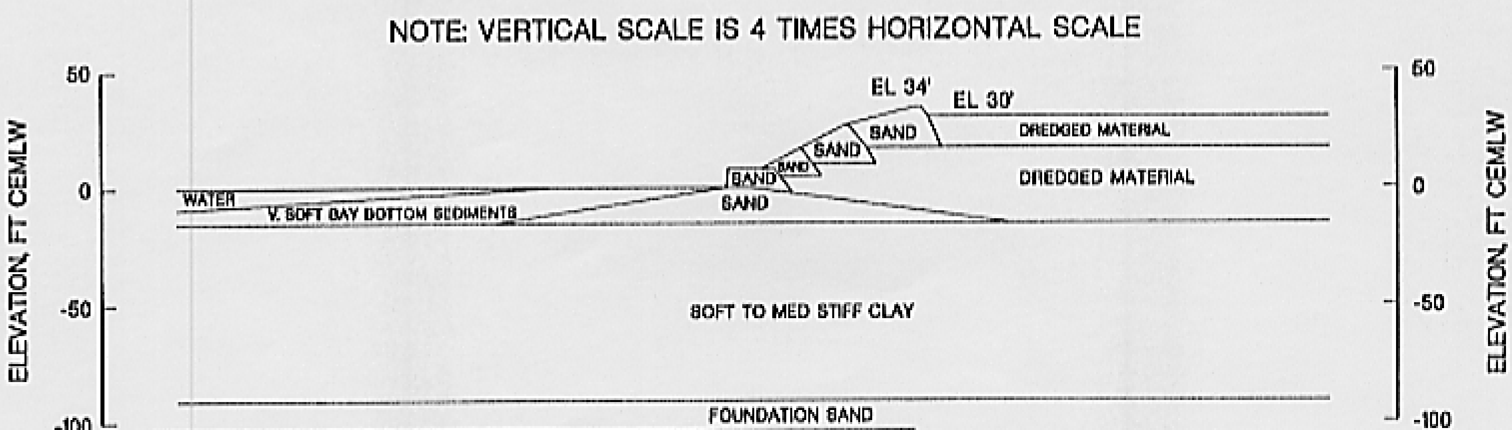


Figure 18. Geometry of Stability Analysis 4 from Fowler et al. (1987)



SOIL PARAMETERS

TYPE	UNIT WEIGHT (PCF) (SATURATED)	Q SHEAR STRENGTH (Ø, PSF) FRICTION	COHESION
CLAY	100	0	275+4.75/FT DEPTH
DIKE & BERM SAND	110	30	0
CONSOLIDATED DREDGED MATERIAL	90	0	100
CONSOLIDATING DREDGED MATERIAL	90	0	35+18/FT DEPTH
SOFT BAY SEDIMENTS	95	0	50

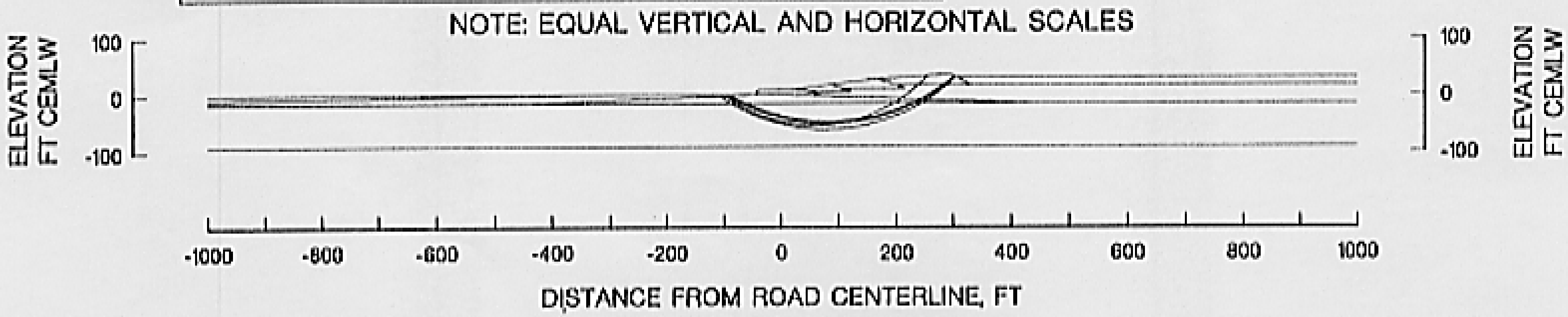
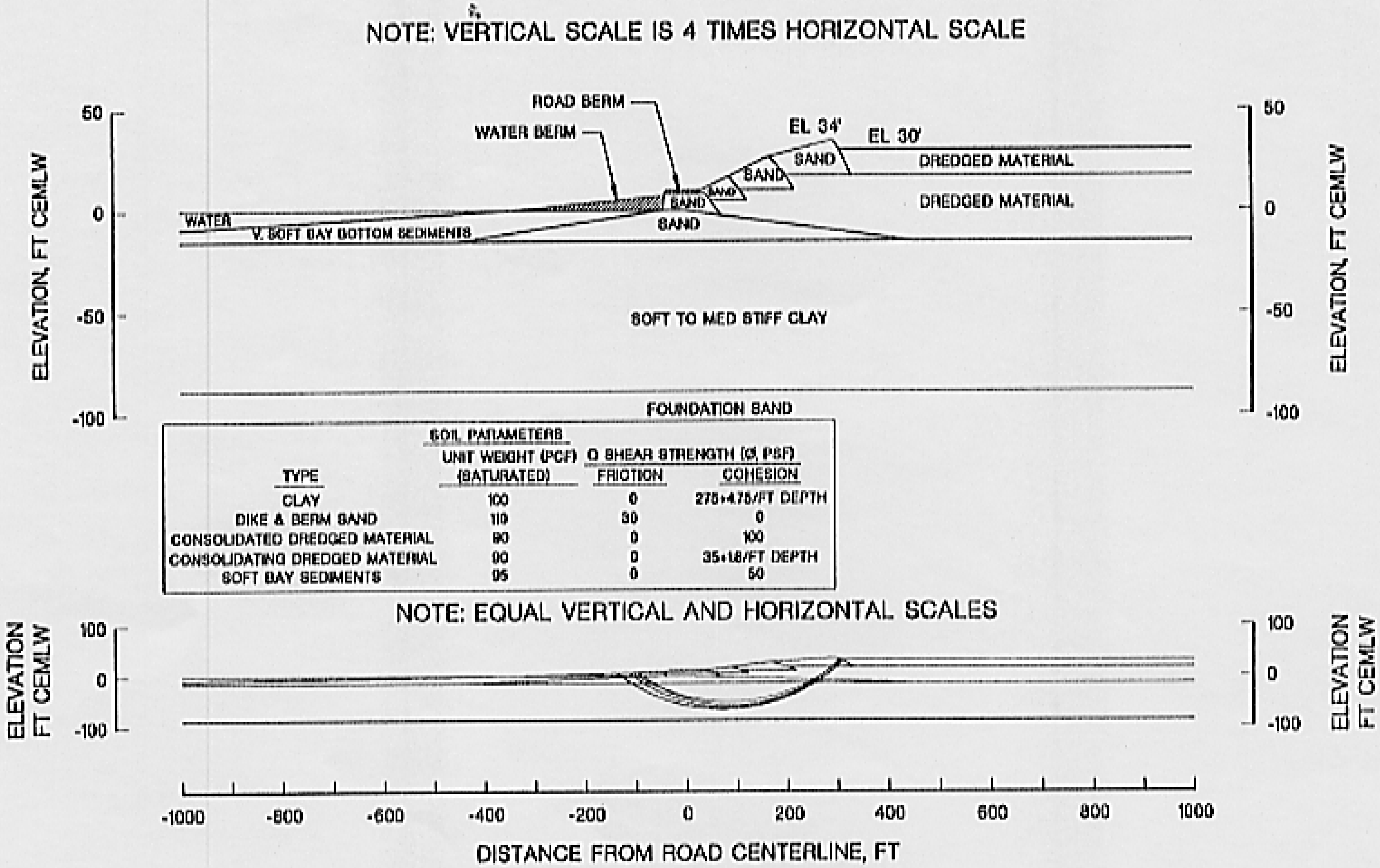


Figure 19. Geometry of Stability Analysis 5 from Fowler et al. (1987)



SOIL PARAMETERS

TYPE	UNIT WEIGHT (PCF) (SATURATED)	Q SHEAR STRENGTH (Ø, PSF) FRICTION	COHESION
CLAY	100	0	275+4.75/FT DEPTH
DIKE & BERM SAND	110	30	0
CONSOLIDATED DREDGED MATERIAL	90	0	100
CONSOLIDATING DREDGED MATERIAL	90	0	35+18/FT DEPTH
SOFT BAY SEDIMENTS	95	0	50

Figure 20. Geometry of Stability Analysis 32 from Fowler et al. (1987)

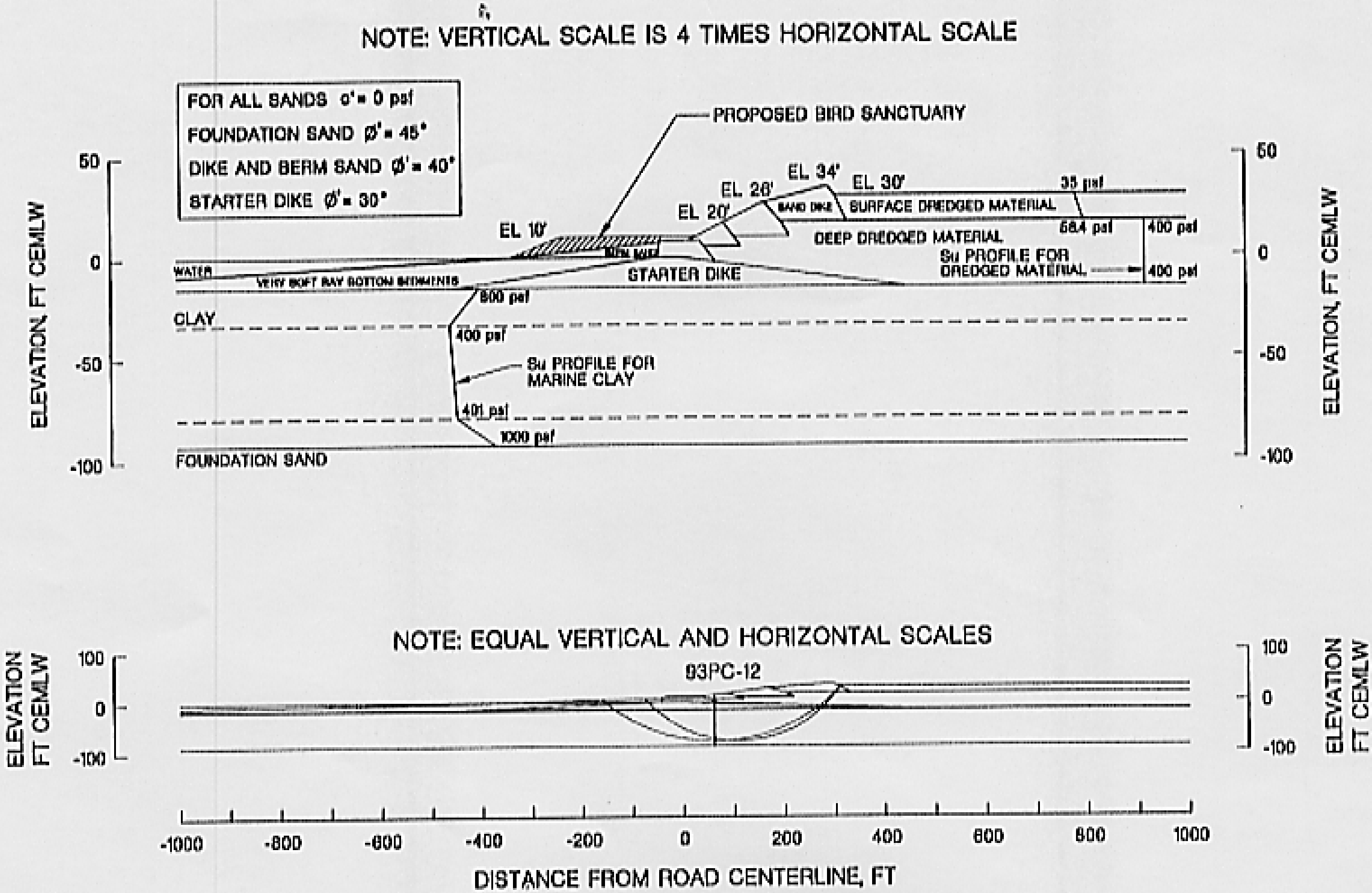


Figure 21. 1994 West Perimeter Dike Geometry and Shear Strengths and Proposed "Bird Sanctuary" Berm

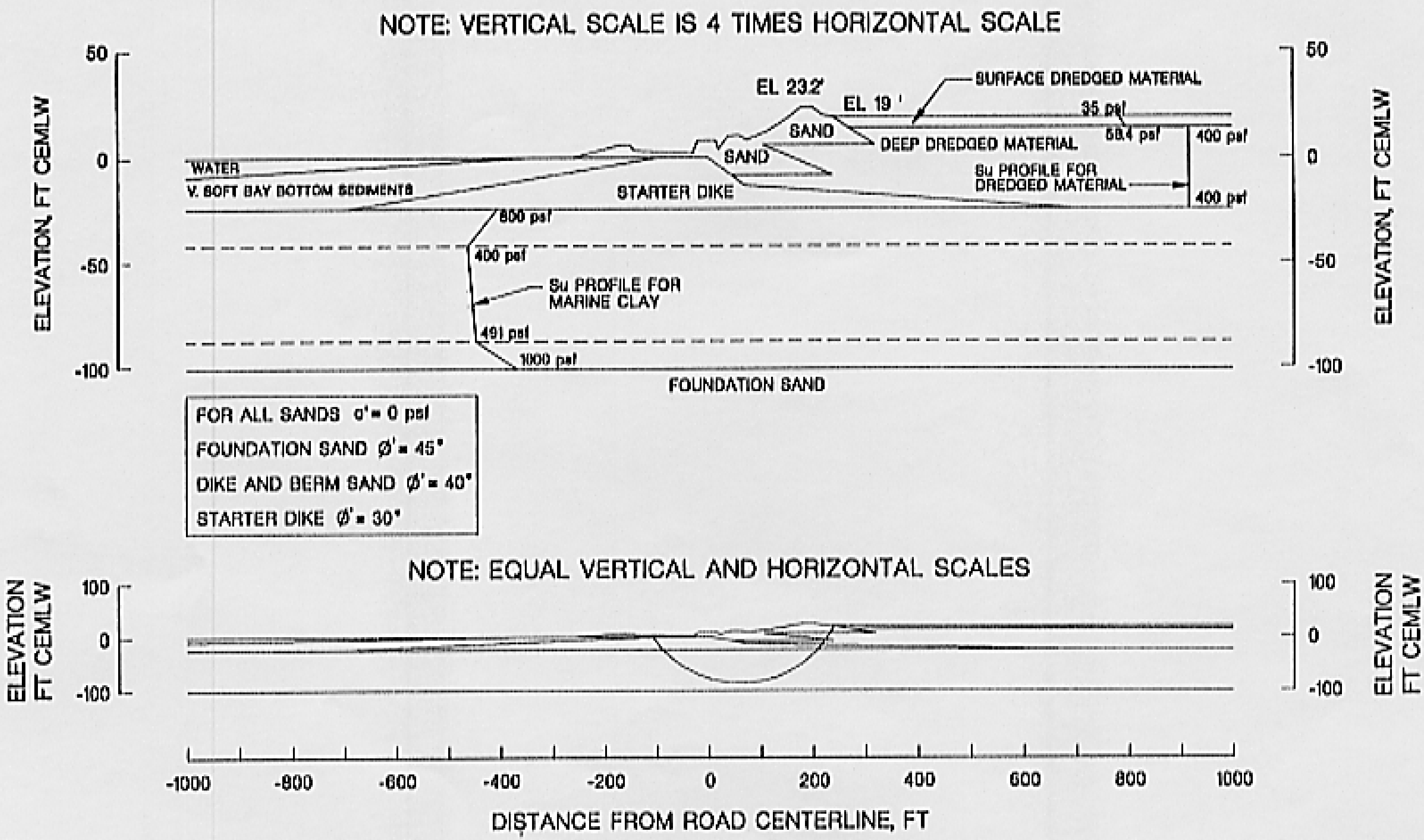


Figure 22. January, 1994 Geometry for Stability Analysis "a"

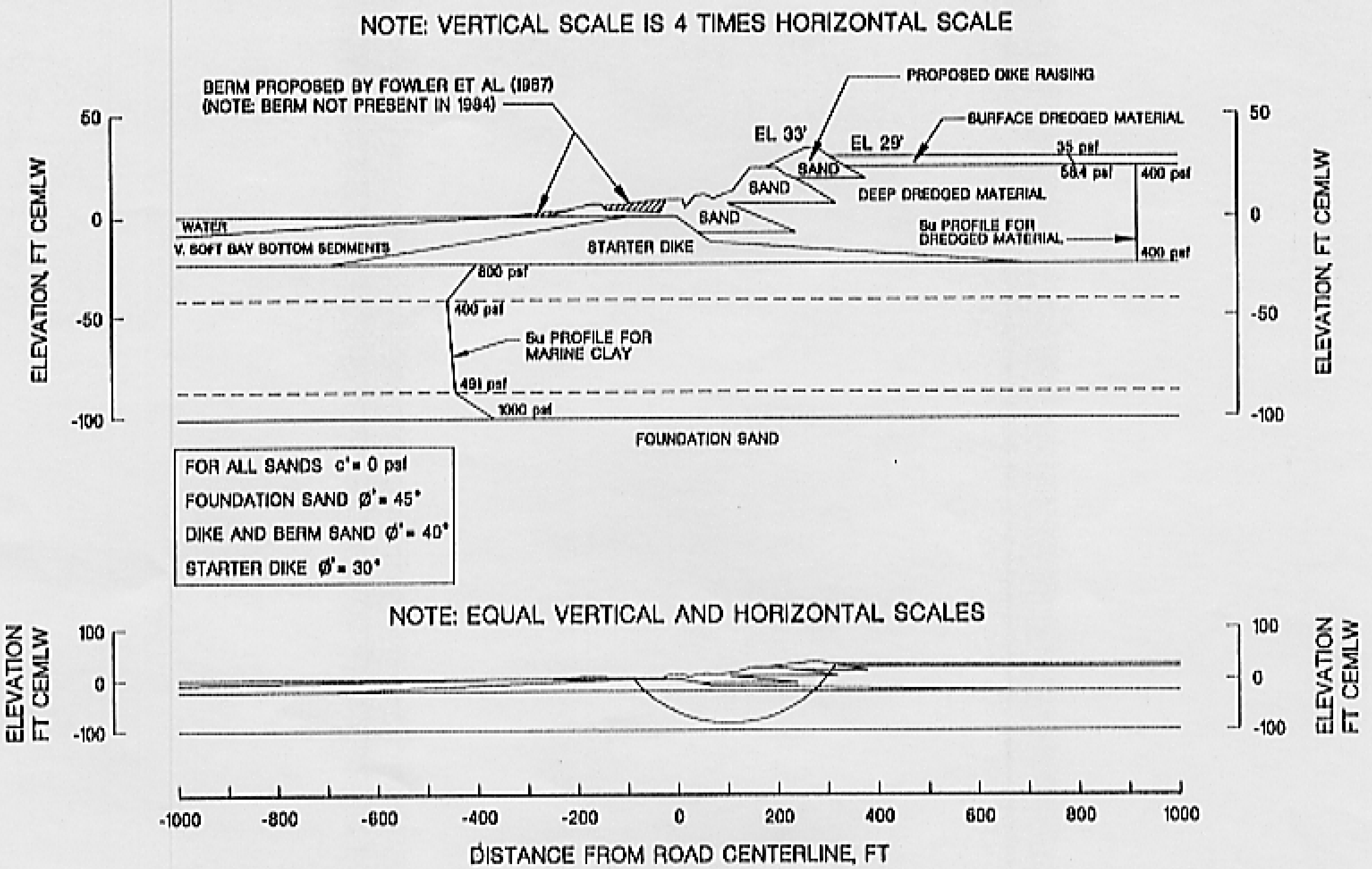


Figure 23. Geometry of Raised West Perimeter Dike to Reduce Factor of Safety to 1.3

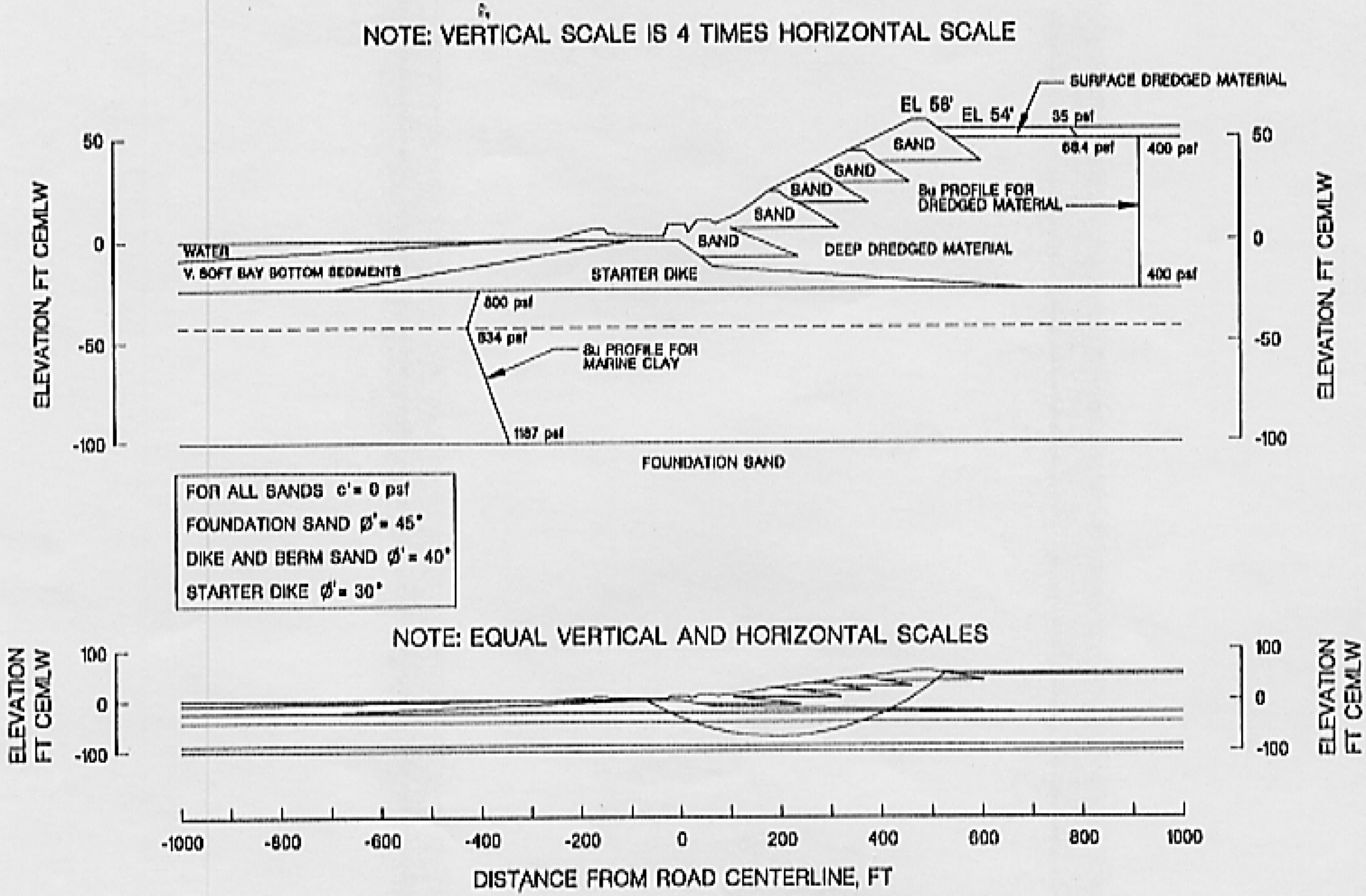


Figure 24. Geometry of Raised West Perimeter Dike at 100% Consolidation to Reduce Factor of Safety to 1.3 (Su/p' = 0.22)

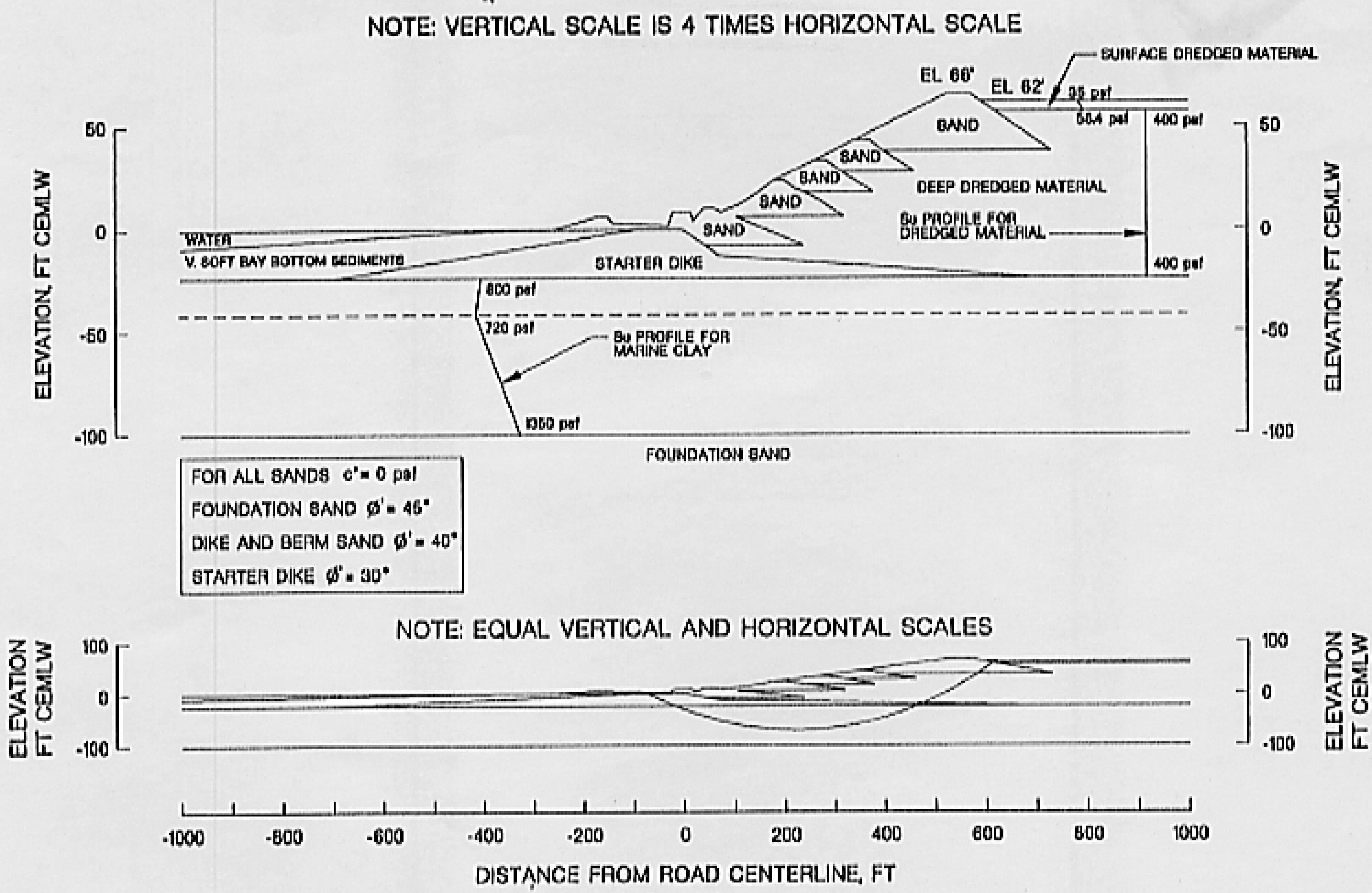


Figure 25. Geometry of Raised West Perimeter Dike at 100% Consolidation to Reduce Factor of Safety to 1.3 (Su/p' = 0.22)

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