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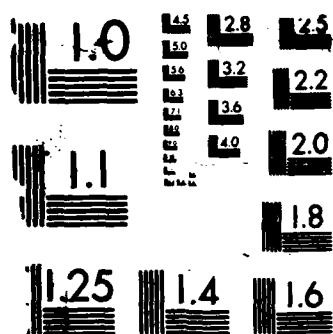
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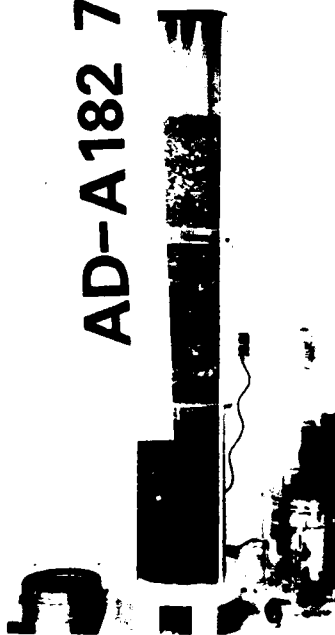


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PERIMETER DIKE STABILITY ANALYSES CRANEY ISLAND DISPOSAL AREA, NORFOLK DISTRICT, NORFOLK, VIRGINIA

by

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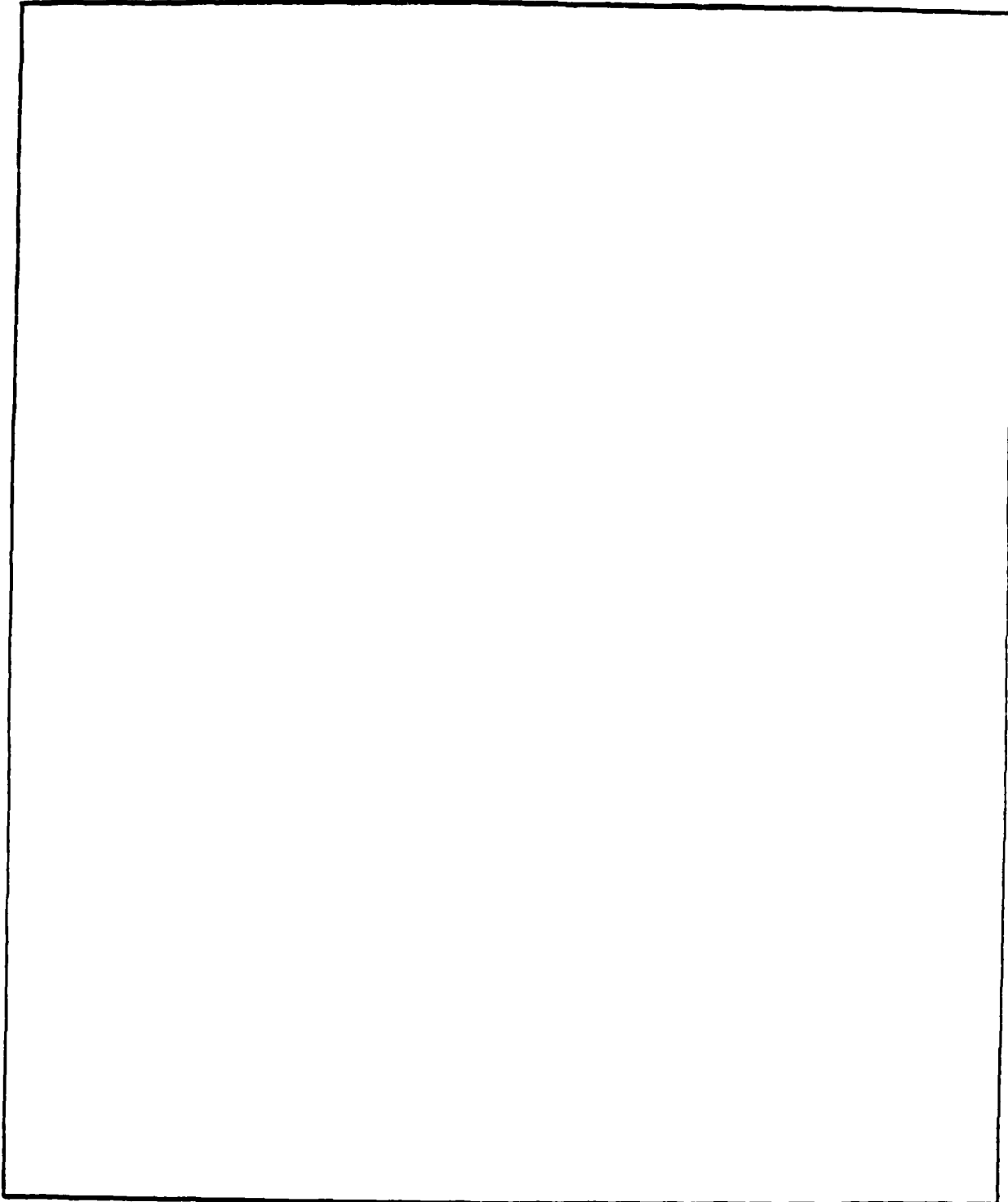
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19. ABSTRACT (Continue on reverse if necessary and identify by block number) This study investigates the stability of existing and proposed perimeter dikes at Craney Island disposal area when the crest elevations are raised to el +34 on the west and el +40 on the east side. With these crest elevations, the dredged material could be raised to el +30 on the west side and el +36 on the east side. Dike configurations were developed for the west leg, east leg, north leg, and northwest corner. The west leg required a 2-ft road berm and a water berm to achieve a safety factor of 1.3. The other sections did not require any additional berms or setbacks. By raising the dike on the inward side using the same slopes, the safety factors were above 1.3. Reinforcement of the raised dikes was found not to be feasible. Dike settlement was evaluated. It was found that the use of wick drains to increase the shear strength of the soft foundation clay was not economically feasible because of the excess depth of the dike cross section. Settlement from raising the east, west, and north perimeter dikes was estimated to be about 4 ft.					
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PREFACE

This publication describes the slope stability analyses and recommended designs for the raised perimeter dikes at the Craney Island disposal area in Norfolk, Virginia.

The investigation was performed by the Geotechnical Laboratory (GL), US Army Engineer Waterways Experiment Station (WES), Vicksburg, Miss., for the Dredging Management Branch of the US Army Engineer District, Norfolk, during the period May 1986 to October 1986.

The report was written by Dr. J. Fowler, Mr. E. V. Edris, Jr., Mr. W. L. Hanks, and Mrs. T. S. Holloway under the general supervision of Mr. G. B. Mitchell, Chief, Engineering Group, Soil Mechanics Division (SMD), Mr. C. L. McAnear, Chief, SMD, and Dr. W. F. Marcuson III, Chief, GL.

COL Allen F. Grum, USA, was the previous Director of WES. COL Dwayne G. Lee, CE, is the present Commander and Director. Dr. Robert W. Whalin is Technical Director.

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CONTENTS

	<u>Page</u>
PREFACE	1
CONVERSION FACTORS, NON-SI TO SI (METRIC)	
UNITS OF MEASUREMENT	3
PART I: INTRODUCTION	4
Background	4
Purpose	4
Scope	5
Objective	5
PART II: PERIMETER DIKES	6
. General Topography	6
Development	6
Description	7
Hydrographical Survey	9
PART III: GEOTECHNICAL INVESTIGATIONS	10
1953 Investigation	10
1971 Investigation	10
1978-1979 Investigations	11
1980 Investigation	11
1981-1983 Investigations	11
PART IV: SLOPE STABILITY ANALYSIS	14
Dike Cross Section for Analysis	14
West Perimeter Dike Stability Analysis	14
East Perimeter Dike Stability Analysis	21
North Perimeter Dike Stability Analysis	22
Northwest Corner Dike Stability Analysis	23
Reinforcement of Dike Sections	23
PART V: ESTIMATED DIKE SUBSIDENCE, RECOMMENDED INSTRUMENTATION, AND DREDGED MATERIAL MANAGEMENT PLAN	24
Subsidence	24
Recommended Instrumentation	25
Dredged Material Management Plan	26
PART VI: CONCLUSIONS AND RECOMMENDATIONS	28
REFERENCES	29
FIGURES: 1-37	

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

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

<u>Multiply</u>	<u>By</u>	<u>To Obtain</u>
acres	4046.873	square metres
cubic yards	0.7645549	cubic metres
feet	0.3048	metres
feet per day	0.3048	metres per day
feet per minute	0.3048	metres per minute
inches	0.0254	metres
pounds (force) per square foot	0.04788026	kilopascals
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
square feet	0.09290304	square metres
square feet per day	0.09290304	square metres per day
square inches	645.16	square millimetres
tons (force) per square foot	95.76052	kilopascals

PERIMETER DIKE STABILITY ANALYSES, CRANEY ISLAND DISPOSAL
AREA, NORFOLK DISTRICT, NORFOLK, VIRGINIA

PART I: INTRODUCTION

Background

1. The Craney Island disposal area is a 2,500-acre confined dredged material containment located near Norfolk, Virginia, in Portsmouth, Virginia. A vicinity map is shown in Figure 1. Plans for construction of the site were developed in the early 1940's. Construction was begun in August 1954 and was completed in January 1957. Craney Island was to provide a long-term disposal area for material dredged from channels and ports in the Hampton Roads area by providing storage capacity and adequate sedimentation of dredged material solids to maintain water quality of the effluent.

2. Dredged material has been placed in the disposal area almost continuously since it was completed in 1957. The initial capacity was estimated to be about 100 million cu yd based on an assumed final elevation of +18 ft mlw. Over 180 million cu yd have been placed in the containment to date and the height of dredged material is at an average elevation of +17 ft mlw. Continual upgrading of the perimeter dike system to +26 ft mlw has led to concern for dike stability, possible failure, loss of dredged material, and limited use of the disposal area. An investigation was initiated by the US Army Engineer District, Norfolk (NAO), with US Army Engineer Waterways Experiment Station (WES) assistance to evaluate (a) the stability of the perimeter dike system at the Craney Island Disposal site and (b) the use of innovative design and construction techniques such as geotextile reinforcement and strip drains for improved foundation strength.

Purpose

3. The purpose of this study is to evaluate the stability of the existing and the final perimeter dike cross section based on subsurface investigations conducted by the NAO.

Scope

4. The study included collection and statistical evaluation of in situ field vane shear strength and conventional laboratory shear strength data and boring log information collected from 1948 to 1983. Innovative procedures for improving the foundation strength and techniques for strengthening the final dike sections were evaluated. This study required the identification of several critical dike cross sections for conventional limit equilibrium analysis. As part of the evaluation, an analysis and determination of factors of safety for each of the cross sections based on the shear strengths determined from the statistical analysis were conducted. The determination of potential failure areas provided input for the location of proposed instrumentation. Dike subsidence was also estimated.

Objective

5. The objective of this investigation was to determine if it is technically feasible to incrementally raise the perimeter dikes at Craney Island to the proposed dike height of el 34 ft mhw and to contain dredged material to a height of el 30 ft mhw.

PART II: PERIMETER DIKES

General Topography

6. The Craney Island disposal area is about 10,000 by 10,500 ft in rectangular trapezoidal shape. A peripheral dike 25 to 30 ft high surrounds the entire disposal area, and two dividing dikes running parallel with the shoreline separate the disposal area into three almost equal areas of about 800 acres each.

7. The southernmost section has been filled at the eastern dike to el +27 ft, the middle of this section to el +19 ft, and areas adjacent to the western dike to an average el +17 ft as of 1986.

8. The middle section is filled at the eastern dike to el +19 ft sloping toward the western dike about el +13 ft in the northwestern corner.

9. The northern section slopes from el +22 ft in the east to +16 ft at the western dike.

Development

10. Beginning from the initial construction, the perimeter dike height has increased in two major efforts. The initial change from el +8 to el +17 occurred around 1969 with the second increase to el +26 around 1980. Usually a stepped or benched dike construction technique had been used to incrementally raise the dikes at Craney Island. Adjacent dried dredged material crust along the dike alignment is generally used to raise the dikes and supplemented as required with truck-hauled coarse-grained material. With projections for more containment volume, studies have been made to provide better dredge management or more containment volume.

11. During a study conducted by Palermo (1981) it was recommended that the Craney Island disposal area be subdivided into three separate containment sites for improved dredged material management. Attempts in the past had been made to construct two interior displacement dikes using end dumped wood debris and sand, and hydraulically placed sand. The interior dikes were designed to create three containment areas that would improve sedimentation in the containment area being used and allow the other two containment areas to dry out. Construction of the interior dikes was completed in 1983, and the dredged

material management plan (Palermo 1981) was implemented in 1984 starting with the center compartment. The dredged material management plan consists of management of surface and ground water through the use of ditches made in the newly placed dredged material to promote the rate of consolidation. The west perimeter dike heights for the center compartment were raised to about el 26, and the division dikes were raised to about el +22 because it was anticipated that dredged material would be stored to about el +17. Filling of the center compartment began in late 1985.

12. A feasibility study by the Norfolk District in 1971 concerning the raising of the perimeter dikes to el +30 was performed. 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. Their proposed section is shown in Figure 2. The location of the dredging work dictated that the inflow points be located along the eastern dike. The natural slope of the fine-grained dredged material is about 5 ft in 10,000 ft which is the approximate distance between the east and west dikes. This required that the elevations of efficient perimeter dikes be tailored to contain the slope. Palermo in his 1981 report recommended that for an average dike el +30 that the east and west dikes be constructed to crest el +32.5 and +27.5, respectively. The north and south dikes would require a crown elevation sloping from east to west.

13. In 1981, a foundation analysis performed by the Norfolk District indicated that the outside slope along the west perimeter dike needs to be constructed to 1:8 for a dike crest el +30 ft. When the west perimeter dike was constructed to el +26 ft 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. It was also recommended that because rapid loading was anticipated the use of geotextiles and strip drains be considered.

Description

East dike

14. Since most of the coarse-grained material is located along the east dike it provides a convenient location for construction material for continued construction of this dike. Because these coarse-grained materials provide a

firm foundation, progressive raisings of the east dike have experienced no stability problems. This section is now benched westward from the perimeter road center line at el +8 ft approximately 250 ft to a crest el +30 ft. Selective placement of coarse-grained material along the east perimeter dike alignment has almost eliminated truck-hauled material for incremental dike increases. Dragline placement of material along the dike alignment is all that has been necessary. A second perimeter access roadway at about el +16 to +18 ft has been constructed in addition to the original roadway at el +8 ft. This second roadway was set back about 200 ft parallel to the original perimeter roadway.

North dike

15. Coarse-grained dredged material sand has accumulated over an extensive area in both the NE and NW corners of the disposal area and along the inside of the north perimeter dike. A second perimeter access roadway was also constructed at about el +16 to +18 ft and set back about 250 ft parallel to the original roadway at el +8 ft. The dike alignment was set back about 400 ft south of the north perimeter roadway in 1981 to accommodate a borrow area adjacent to the roadway. In 1983 it was suggested that this portion of the north perimeter dike be moved to about 250 ft south of the roadway at el +8 ft to increase the dredged material storage capacity of the disposal area.

16. At the present time this dike is being raised with a dragline on 20- by 20-ft wooden mats using excavating dredged material adjacent to and from within the containment area for raising the crest of the dike. There have never been any stability problems along the north perimeter dike.

West dike

17. 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 bearing capacity failure. Incremental dike height 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.

18. After the interior division dikes were completed, subdividing the disposal area into three separate containment areas, the middle area began to

dry out. Tracks made by a Riverine Utility Craft and ditches made by a Gemco Trencher kept surface water drained to collection ditches that exited the site through newly constructed weir boxes. Surface subsidence of the dredged material along the inside of the west perimeter dike was below el +13 ft and the invert elevation of the drainage trenches made by the Gemco ditcher was below el +10 ft. Because the invert elevation of the trenches was below the outlet el +10 ft for the newly constructed weirs, one of the older weir structures had to put back in service for continued site drainage. Continued site drainage caused about 12 to 18 in. of crust to develop along the west perimeter dike. The west perimeter dike was raised in late 1985 to about el +26 ft without the displacement type failure toward the inside of the disposal area as experienced in the past because of improved foundation conditions caused by trenching and dewatering of the site. Continued dike raising will conform to the horizontal and vertical location of the newly constructed weir.

Hydrographical Survey

19. Hydrographic surveys were conducted by the Norfolk District to determine the bottom elevations within the Craney Island 1,000-ft right-of-way adjacent to the perimeter dike surrounding the disposal area. Depth of water at the 1,000-ft right-of-way varied in depth of about 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 appeared to have been caused by fast flowing water currents around the northeast corner of the island. Bottom slopes determined from the hydrographic surveys varied from 1V:30H to 1V:100H.

PART III: GEOTECHNICAL INVESTIGATIONS

20. This section describes the foundation soil conditions below the dredged material deposited within the Craney Island Disposal Area. This section also describes the engineering properties required for the perimeter dike stability analysis.

1953 Investigation

21. An extensive foundation investigation began in 1948 was completed by the Norfolk District in 1953 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, triaxial shear strength, Atterberg limits, specific gravity, and classification tests on the compressible marine clays underlying the site. The 1953 design report identified four major soil zones as listed in Table 1.

Table 1
Major Foundation Soil Zones (Palermo 1981)

Zone	Soil type	Elevation ft mlw		Natural Densities lb/ft ³	
		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

22. During a feasibility study in 1971 for raising the perimeter dikes at the Craney Island Disposal Area, the Norfolk District conducted a

subsurface investigation beneath the main dike. Location of the borings are shown in a plan view of Craney Island Disposal Area in Figure 3. Several of these borings were taken through the dredged material deposit as well as through the main dike.

1978-1979 Investigations

23. Several other foundation investigations made in or near the Craney Island Disposal Area by the Virginia Department of Transportation and others are summarized in studies prepared for the Virginia Port Authority (Dames and Moore 1978, 1979). Most of the borings taken during these investigations were in search of coarse-grained material borrow sources and did not provide information concerning the engineering properties of the fine-grained dredged materials or marine clay foundation materials.

1980 Investigation

24. Three rotary borings were taken under contract for the Norfolk District at three locations on the western portion of the disposal 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 shown in Figure 3 that were usable for dike enlargement.

1981-1983 Investigations

25. Because the storage volume of dredged material needed to accommodate the proposed Norfolk harbor and channel deepening, 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 to 100 ft penetrating a very dense sand. Location of these soil borings are shown in Figure 3.

26. A generalized plot of the major foundation soil zones is shown in Figure 4 and tabulated in Table 1. Field vane shear strength tests and

laboratory triaxial tests were performed during this investigation and the results of these tests are shown plotted in Figures 5, 6, and 7 for the east, north, and west perimeter dikes, respectively. Soil borings for the east, north, and west perimeter dikes are shown in subsurface profiles for each dike in Figures 8, 9, and 10, respectively. The subsurface stratum shown in the plots are the interpretation of the author.

27. Location of the cross sections investigated during the stability analysis are shown in Figure 11. Foundation shear strengths selected for the east, north, and west perimeter dikes are shown in Figures 12, 13, and 14, respectively, for each dike. These widely varying shear strengths were selected using a linear regression analysis of both the laboratory triaxial and field vane shear strength test results shown in Figures 5, 6, and 7.

28. The marine clay layer is a continuous stratum of recent marine sediments which are normally consolidated, i.e., they have never experienced a greater load than their own, present self-load. Therefore, it would be expected that soil unit weight would be lowest (void ratio highest) at the present river bottom (profile line) and would increase (void ratio decrease) with depth. Concomitantly, shear strength should increase with depth. Actual values will vary from point to point as a result of natural variations in sand content within the matrix clay. Tests of samples from the subsurface investigations referenced by Fowler (1986) indicate:

- a. Atterberg Limits: Liquid limit ranges from 40 to 105, with typical values of 60 to 90. Plastic Limit ranges from 25 to 40, with typical values of 30 to 35. Plasticity Index ranges from 15 to 65, with typical values of 30 to 60.
- b. Saturated unit weight ranges from 90 to 105 pcf. Void ratio ranges from about 2.75 near the profile line to about 1.50 at depth. These are void ratio values (density) after compression under the existing dikes. Initially, before compression under the dike body, the void ratio at each depth is much higher.
- c. Values of the compression index, from laboratory consolidation tests of undisturbed samples, ranged from 0.55 to 0.74. A typical value of 0.58 was used in volume estimates by Goforth (1986).
- d. Shear strength is expected to vary from zero at the profile line to a maximum at a depth of -90 ft mlw. Test values of cohesion (1/2 of unconfined compressive strength) from the 1953 investigations (Norfolk District, 1953), before construction of the Craney Island dikes, showed (1) that the upper 10 ft or more could not be sampled because of softness and (2) the rate

of increase of cohesion with depth was about 4 to 5 psf per foot of depth.

PART IV: SLOPE STABILITY ANALYSIS

29. Slope stability analyses conducted during this investigation were performed using the two-dimensional slope stability package UTEXAS2, version 1.205 (CAGE et al. 1986 (Draft)). Of the four analysis procedures available in this program, Spencer's procedure was used for all analyses because it satisfies both force and moment equilibrium. The side force inclination is calculated in the Spencer procedure. The phreatic surface is taken as the river level outside the dike toe and the top of the dredge material behind the dikes. A linear change in the phreatic surface is assumed between these two horizontal segments. Circular shear surfaces were used in all analyses. Searches were performed to determine the critical shear surface. The searching was initiated with the tangent search mode using a final grid spacing of 0.3 ft. These analyses will result in recommended design cross sections for each raised dike section. Also, several assumptions made during the study will be evaluated to show the significance of variation in the selected perimeter.

Dike Cross Section for Analysis

30. The dike cross sections used for analysis were selected based on the largest thickness of the soft foundation clay shown on the soil profiles developed in the geotechnical investigation section of this report. Four cross sections were identified with one in the northwest corner and one each on the west, north, and east dikes as shown in Figure 11. The stations of the cross sections identified were 104+00, 80+00, 45+00, and 80+00 for the northwest corner, west, north, and east dikes, respectively. The shear strength of the soft foundation clay at these cross sections are shown in Figures 12-14. The geometry of the cross sections shown in Figures 15-18 was obtained from survey results and foundation investigations.

West Perimeter Dike Stability Analysis

31. The dike section for analysis sta 80+00 is located in the lower half of the north compartment where dredged material has recently been placed. Before dredging began in 1985, the dredged material surface adjacent to the

west dike had an average elevation of about +13 ft. The dredged material elevation is presently about +17. The first scenario, investigating the existing conditions, consisted of the dike at el +26 and the dredged material at el +17. The safety factor for this case was 1.20. The next scenarios increased the dredged material to el +19 and +22. The safety factors decreased from 1.20 to 1.16 at the higher dredged material elevation. The results of these analyses are tabulated in Table 2 with the critical circles shown in Figure 19. These safety factors are above 1.0 but below a design value of 1.3. Thus, some of the berms described for the recommended enlarged dike section could be started to increase the safety factor to 1.3.

Table 2
Effects of Dredged Material Elevation on Safety Factor

Stab. Run	Dike Crest El	Dredged Material El	Results of Circular Search					Safety Factor
			X	Y	Radius	Tang. El	Side Force Incl	
1	26	17	25.5	145.9	199.7	-53.8	3.42	1.20
2	26	19	26.2	147.7	200.6	-52.9	3.49	1.18
3	26	22	26.7	153.6	204.5	-50.9	3.63	1.16
4	34	22	58.3	195.0	253.2	-58.2	3.86	0.99
5	34	30	60.6	210.0	264.1	-54.1	4.09	0.95

32. The next scenarios considered raising the containment dike to el +34 with the dredge material at two elevations. The first case involving the raised dike considered the dredged material at el +22. The resulting safety factor is below 1.0 with a value of 0.99. This result indicates that some or part of the berms necessary for stability when the dredge material is increased to el +30 should be in place before the dike is raised. When the analysis was performed with the dredged material up to el +30, the safety factor dropped to 0.95. The results of these calculations are shown in Table 2 as runs 4 and 5. The critical circles are shown in Figure 20.

33. The results from the analysis with the dike height at el +34 and the dredged material to el +30 indicate that additional resisting force is

necessary. Three possible solutions were investigated. These are a road berm, a water berm, and a bench or setback distance for the new dike. Each of these potential alternatives will be investigated separately to show the influence and then combined for the recommended design.

34. The existing roadway at el +8 is about 58 ft wide. The road berm consists of a mass of sandy material that increases the road elevation. A slope of 2H:1V was used for the outboard or west side of the berm. The effectiveness of the road berm was evaluated by increasing the berm elevation in 2-ft increments. Figure 21 illustrates how the berms fit in the current configuration and indicates the volume of material contained in each berm. The safety factors increase from 0.95 for the current roadway to 1.01 for an 8-ft berm (top of berm at el +16). The results are listed in Table 3 with the critical circles shown in Figure 22. A plot of safety factor versus berm elevation is shown in Figure 23. The safety factor increased as the berm elevation increased with a change in the rate of increase at +12. The center coordinates for the critical shear surfaces moved outward (lower X value) and upward (larger Y value). This movement tends to counter balance the additional resisting forces generated by the berm.

Table 3
Effect of Road Berm Thickness on Safety Factor

Stab. Run	Thickness Road Berm	Elevation of Road Berm	Results of Circular Search					Safety Factor
			X	Y	Radius	Tang. El	Side Force Incl	
5	0	8	60.6	210.0	264.1	-54.1	4.09	0.95
6	2	18	55.2	219.6	275.2	-55.6	4.05	0.97
7	4	12	48.6	231.0	288.1	-57.1	4.01	0.99
8	6	14	43.2	237.0	295.6	-58.6	3.98	1.00
9	8	16	37.2	246.0	305.5	-59.5	3.94	1.01

35. A water berm is a mass of sand placed at the toe of the dike to increase the stability. This sand material is assumed to displace all the soft bay bottom sediments. There are several variables that are involved in the water berm design. A general schematic of a water berm, illustrating the

terminology is shown in Figure 24. The three variables are the inboard elevation, outboard elevation, and the top width. Three combinations of elevations were used. They are el 0 for both the inboard and outboard values, el 0 for the outboard and el +3 for the inboard value, and el +3 for the outboard and el +6 for the inboard value. For each combination of elevations, three top widths of 100, 300, and 500 ft were evaluated. The results of these computations are shown in Table 4. The critical circles for the berms with el +3 to +6 are shown in Figure 25. There is a significant increase in safety factor when the top width increased to 100 ft. A small increase occurs with an increase from 100 to 300 ft. However, no increase occurs when the berm width is increased to 500 ft. Figure 25 shows that a minimum width of 100 to 150 ft is needed. However, a berm top width of 300 ft was selected for further study

Table 4
Effect of Water Berm on Safety Factor

Stab. Run	Water Berm			Results of Circular Search					
	Outboard El	Top Width	Inboard El	X	Y	Radius	Tang. El	Side Force Incl	Safety Factor
5	-	-	-	60.6	210.0	264.1	-54.1	4.09	0.95
10	0	100	0	65.7	202.2	254.8	-52.6	4.07	0.96
11	0	300	0	66.0	202.0	255.0	-53.0	4.05	0.96
12	0	500	0	65.7	202.2	254.8	-52.6	4.07	0.96
13	0	100	3	50.3	223.4	282.4	-59.0	3.56	1.04
14	0	300	3	68.8	195.8	249.1	-53.3	3.41	1.07
15	0	500	3	71.7	191.8	244.2	-52.4	3.39	1.08
16	3	100	6	66.0	196.8	252.9	-56.1	2.90	1.18
17	3	300	6	81.7	167.2	219.6	-52.4	2.79	1.20
18	3	500	6	82.0	166.9	219.3	-52.4	2.78	1.20
19	0	300	4	70.8	191.5	244.8	-53.3	3.21	1.11
20	3	300	4	77.7	182.0	233.0	-51.0	3.19	1.12
21	0	300	5	74.0	185.7	238.7	-53.0	3.02	1.15
22	3	300	5	80.9	174.8	225.8	-51.0	2.99	1.16
23	0	300	6	77.7	178.3	231.0	-52.7	2.83	1.19

so that a margin against erosion of the berm is provided. Several additional elevation pairs were evaluated for 300-ft-wide berms. These results are also shown in Table 4. The volume of the various berms are listed in Table 5. The safety factor increased more for the water berm than for the road berm, but the volume of material in the water berm is 16 to 70 times larger than the road berm. A plot of the water berm volume versus safety factor is shown in Figure 26. This plot shows that the safety factor is dependent upon the inboard elevation and not the outboard elevation.

Table 5
Water Berm Volumes

Outboard El	Water Berm		Volume Per Foot of Dike (cy ³)	Total Volume* (cy ³) x 10 ⁶
	Top Width	Inboard El		
0	300	0	309.0	3.09
0	300	3	325.7	3.26
3	300	6	395.4	3.95
0	300	4	331.2	3.31
3	300	4	384.3	3.84
0	300	5	336.8	3.37
3	300	5	389.9	3.90
0	300	6	342.3	3.42
0	300	7	347.9	3.48

* Used 10,000 ft for dike length.

36. A setback distance is the horizontal distance from the outboard crest of the el +26 dike to the outboard toe of the el +34 dike. This setback was varied from 0 to 60 ft in intervals of 20 ft. The results are shown in Table 6. A plot of safety factor versus setback distance is shown in Figure 27. Only small increase in safety factor occurs when the dike is setback.

37. Based on the above results, none of the three individual solutions for increasing the resisting force resulted in the safety factor being above 1.3. Thus, combinations of road berm, water berm, and setback will be

Table 6
Effect of West Dike Setback on Safety Factor

Stab. Run	Setback Distance ft	Results of Circular Search					Safety Factor
		X	Y	Radius	Tang. El	Side Force Incl	
5	0	60.6	210.0	264.1	-54.1	4.09	0.95
24	20	66.4	231.6	287.5	-55.9	3.92	0.96
25	40	72.7	248.2	307.0	-58.8	3.75	0.98
26	60	77.6	270.9	331.8	-60.9	3.59	1.00

considered. When combining these three potential solutions it must be noted that there is not a unique solution. The recommended design will be closest to a safety factor of 1.3 and minimize the material in the various berms. The first combination consisted of a water berm 300 ft wide with el +3 to +6 ft and a road berm. The road berm varied from el +16 to el +12. The safety factor for the el +16 berm was 1.34 reducing to 1.29 for the el +12 berm. Notice that the changes in the safety factor calculated individually for each the three solutions can only serve as a guide and can not be added together. Water berms were then combined with a setback distance of 60 ft. The safety factor for a water berm from el +3 to +6 and 300 ft wide was 1.24. When a water berm with el 0 to +3 was considered, the safety factor decreased to 1.19. Because of other constraints such as weirs and reductions in size, setbacks larger than 60 ft were not considered. The optimum design was then a matter of selecting the best water berm and road berm. Four additional combinations were considered. The recommended design consists of a water berm 300 ft wide with el 0 to +7 ft and a road berm to el +10 ft. The safety factor for this combination is 1.30. The results and total volumes for all combinations considered are shown in Table 7. Figure 28 shows the recommended section and the critical circle.

38. There are several assumptions that were made in the course of the analysis. These will be reviewed and examined to determine the significance of the selected value. An assumption concerning the water berm is that the sand material will totally displace the soft bay bottom sediments. This should occur. Pockets of soft material of sufficient size to cause problems

Table 7
Effect of Water Berm and Road Berm Combinations on Safety Factor

Stab. Run	Water Berm El/ Width	Road Berm Elev	Volume Per Foot of Dike	Results of Circular Search					
				X	Y	Radius	Tang. El	Side Force Incl	Safety Factor
27	3-6/300	16	419.4	45.4	226.8	288.4	-61.6	2.79	1.34
28	3-6/300	12	405.6	65.2	198.0	254.4	-56.4	2.82	1.29
29*	3-6/300	8	395.4	102.4	222.1	280.3	-58.2	2.57	1.24
30*	0-3/300	16	366.3	55.8	305.3	373.3	-68.0	2.96	1.19
31	0-6/300	14	359.0	60.0	295.3	365.0	-69.7	2.61	1.31
32	0-7/300	10	352.6	83.5	253.0	317.7	-64.7	2.45	1.30**
33	0-7/300	12	358.1	70.1	276.7	344.6	-67.9	2.49	1.33
34	0-5/300	16	360.8	52.1	310.2	380.8	-70.6	2.71	1.28

* Includes 60-ft setback.

** Recommended design.

should not occur due to the slope of the sand lense under the existing dikes providing a good base for the displacement of the soft material.

39. The shear strength of the foundation clay was selected based on numerous tests (see Figure 7). There is a large degree of scatter in the data. To evaluate the sensitivity of the selected shear strength, values plus and minus 10 psf, 25 psf, and 50 psf of the selected cohesion parameter were used in the analysis of the recommended design. The cohesion value is assigned to the profile describing the top of the clay. All analyses used the same rate of increase for the cohesion. Figure 29 illustrates the relationship between cohesion and the safety factor.

40. The last assumption concerns the side force inclination. For the recommended west dike section, the side force inclination calculated by the Spencer's procedure was nearly horizontal, indicating that the Simplified Bishop procedure would generate about the same results. However, the Modified Swedish force equilibrium procedure with the Corps of Engineers side force assumption of the average outer slope would generate safety factors larger than the rigorous procedure. Spencer's procedure determines the side force inclination that satisfies both moment and force equilibrium. The Modified

Swedish method where different side force inclinations were specified was used to show the influence of the side force inclination. A plot of the results is shown in Figure 30. The side force inclination in the Modified Swedish procedure must be specified by the user. With the surface geometry broken up into many slopes, the proper side force inclination is difficult to determine. Thus, since Spencer's procedure calculates the side force inclination, this method should be used. The side force inclination from Spencer's method can then be used in the Modified Swedish procedure to perform a hand check.

East Perimeter Dike Stability Analysis

41. The dike section for analysis at sta 80+00 along the east perimeter dike is located in the lower half of the north compartment opposite the west dike cross section. A stability analysis was conducted for the raised dike conditions where the dike crest elevation was +40 and the dredged material elevation was +36. The dike crest was set back about 450 ft from the center line of the east perimeter road. The raised dike began at the crest of the existing dike and increased on a slope of 4H:1V. The results of this analysis are plotted in Figure 31. A satisfactory safety factor of 1.40 was obtained for this condition. The search was begun so that the circle passed through the raised dike and moved to the final location during the search process.

42. The practice of reclaiming sandy dredged material by building an outward offset in the perimeter dike has been utilized in the past for this dike section. During an inspection of east dike topographic surveys conducted by the Norfolk District, it was found that a short section of dike about 400 ft long was moved east about 200 ft during dredging activities in early 1985. For the raised dike configuration, this section was analyzed to determine if this practice would cause stability problems. In the analysis it was assumed that the raised dike would begin at the crest of the existing dike and contain no additional setback. The section analyzed consisted of the same foundation cross section as used for sta 80+00. A factor of safety of 0.89 was calculated for this section. This indicates that failure would occur for this configuration. The plot of the critical circle is shown in Figure 32.

43. The difference in the safety factors of the two sections analyzed for the east dike is the distance the dike is set back from the perimeter road

Table 8
Effect of East Dike Setback on Safety Factors

Stab. Run	Setback Distance ft	Results of Circular Search					Safety Factor
		X	Y	Radius	Tang. El	Side Force Incl	
E1	450	-10.5	155.8	245.6	-89.9	1.24	1.40
E2	400	-146.7	522.8	612.6	-89.8	2.08	1.22
E3	350	-126.8	403.9	493.7	-89.8	2.21	1.09
E4	300	-103.2	313.3	403.1	-89.8	2.29	0.98
E5	250	-75.1	253.7	343.5	-89.9	2.27	0.89

center line. To evaluate the setback effect, three intermediate setbacks were considered. These setback distances from the center line of the perimeter road were 300, 350, and 400 ft. The results of these analyses are shown in Table 8 with the resulting safety factors plotted against the setback distance on Figure 33. These results indicate that a setback distance of less than 420 ft will cause the safety factor to drop below 1.3. If a smaller distance is required, a decision can be made based on the results shown in Figure 33. A trade-off between the volume of material available versus the volume needed for a berm would be necessary for those setbacks that generate a safety factor less than 1.0.

North Perimeter Dike Stability Analysis

44. The dike cross section for investigation was located at sta 45+00 on the north perimeter dike. This section is set back about 420 ft from the center line of the perimeter road. The dike crest elevation of +40 with a dredged material elevation of +36 was used in the analysis. The safety factor of 1.28 was calculated. The results of this analysis are shown in Figure 34. The safety factor is close enough to 1.3 that it would not be necessary to construct a berm for additional stability.

45. As along the east dike, there is a small length of dike that is closer to the perimeter road. In this case the dike is 120 ft closer. The setback from the center line of the perimeter road is about 300 ft for this

case. The same foundation cross section was used in this analysis. The safety factor was calculated to be 0.99. This represents an unsafe condition. Since the full setback dike is barely acceptable, outward offsets should not be allowed along this leg of the perimeter dike unless additional stability berms are provided. The results of this analysis are plotted in Figure 35.

Northwest Corner Dike Stability Analysis

46. The dike cross section in the northwest corner located at sta 104+00 was selected for analysis because of the steepness of the slopes within the 1,000-ft right-of-way. Also this area is the transition area for those dikes that require berms to those that do not. Contours from hydrographic investigations conducted by the Norfolk District indicated el -25 located about 450 ft north of the corner. The stability analysis considered the raised crest to be at el +34 with the dredged material at el +30. A satisfactory factor of safety of 1.53 was calculated for this location. The results are plotted in Figure 36. Two reasons for this high a safety factor are the long flat berm resulting from previous dike crest setbacks and the considerable amount of sand found in this area. It is recommended that the dike crest, el +34, be constructed to accommodate the weir at this location because of its relatively high stability.

Reinforcement of Dike Sections

47. The original scope of this project was to investigate reinforcement of the enlarged dike sections. Once the recommended sections shown in Figures 28, 31, 34, and 36 were determined, reinforcement could be considered. The easiest location to install reinforcement would be between the existing dike with crest el +26 and the raised portion. Evaluation of these figures indicates that the critical circles were outside that portion of the raised dike where reinforcement could be installed. Thus, reinforcement of the raised portion of the dike to eliminate a portion of the driving force is not applicable for the perimeter dikes.

PART V: ESTIMATED DIKE SUBSIDENCE, RECOMMENDED INSTRUMENTATION,
AND DREDGED MATERIAL MANAGEMENT PLAN

Subsidence

48. Dike subsidence has continued to occur along the perimeter dike alignment since the Craney Island disposal area was completed in 1957. During design and prior to construction it was estimated that dike settlement would exceed 7 ft. Dike subsidence includes a combination of settlement caused by consolidation and displacement of the dike foundation caused by bearing capacity failure and long-term plastic flow of the soft foundation material. An attempt was made during construction to minimize foundation bearing failure by placement of fill material over a very large area in thin lifts of dredged sand using hydraulic dredging with a floating discharge pipe. The floating discharge pipe was controlled by use of a baffle plate positioned at the end of the pipe. The discharge pipe was allowed to swing over a very large arc by manipulation of the angle of the traffic plate that was placed in line with the flow from the discharge pipe. Prior to use of this placement technique large displacement failure occurred in the soft foundation material. Foundation failures along the west perimeter dike (sta 38+00) were as deep as el -60 ft or about 50 ft below the original bottom, -10 ft. After the swinging discharge pipe technique was implemented along with careful monitoring of the subsurface profile, no major failures occurred during construction. Because of continual dike subsidence and loss of survey control points caused by continued placement of fill material to maintain dike height, monitoring of dike subsidence has been hampered.

49. The average depth of the top of the foundation or bay bottom in 1953 before construction of the east, north, and west perimeter dikes was about -10 to -12 ft. The east perimeter dike foundation has subsided to an average elevation of about -25 to -27 ft or an average depth of about 17 ft because of the loads from dredged sand placed along the dike alignment. The base of the north perimeter dike has subsided to an average elevation of about -20 to -27 ft or an average depth of about 13 ft. This was also caused by large quantities of sand fill placed along the north dike. If the failed section at sta 38+00 is included in the base of west perimeter dike, elevation varied from -15 to -17 ft or an average subsidence of about 11 ft since it was

originally constructed. Utilizing the foundation consolidation test results performed by the Norfolk District, it was estimated that an incremental increase in dike height of 10 ft will cause the north and east perimeter foundations to consolidate an additional depth of about 4 ft. Increasing the west perimeter dikes another 8 ft will also cause the foundation materials to consolidate about 4 ft. Incremental raising of the perimeter dikes has been a continuing process at Craney Island over the past 30 years, and it is not quite understood what percentage of the foundation materials are being squeezed out. There has been no discernible increase in foundation shear strength since initial construction.

50. Several innovative construction techniques have been considered such as "wick drains" and geotextile reinforcement. Because the soft materials in the dike foundation cross section are too deep, it was decided that the use of "wick drains" would not be economically feasible. Too much of the "wick drain" would be left in the dike section causing the cost of the project to not be cost effective. The use of geotextiles in incremental dike raising will not have an effect on the overall dike stability because the fabric does not intercept the potential failure plane in the slope stability analysis.

Recommended Instrumentation

51. It is recommended that several settlement rods be installed along the perimeter dike alignment at locations that would be easily accessible to monitor and protected from damage by construction equipment. These settlement rods may be deep-seated cast-in-place in concrete to a depth about 20 ft in the existing dike. A safe location would be at the intersection of the dike outward slope and the edge of the original perimeter road, but a preferred location would be at the height fill as shown in Figure 37. Attempts should be made to align these monuments along an assumed baseline for ease of electronic survey monitoring. Surface monuments for visual observation would be desired along the dike crest and toe, but because of the constant hauling, dumping, and upgrading the dike crest elevation for increased dredged material storage capacity, this type of monuments may have a short life span.

52. It is also recommended that a vertical slope inclinometer tube and at least four piezometers be installed at various depths in the vicinity of the settlement rods (see Figure 37). The vertical slope inclinometers should

be installed to about el -100 ft or about 5 to 10 ft into the dense sand found at about el -90 ft. The piezometer should be evenly spaced within the original marine clay deposits between the base of the perimeter dikes and the dense sand at el -90 ft. Observation of pore water pressure in the piezometers would provide a warning to the Norfolk District when the effective shear stress in the foundation materials changed, thus effecting the factor of safety of the perimeter dike. Changes in pore pressure would also indicate whether or not the dike is simply floating or the dike is consolidating the foundation materials. Two to three piezometers located within the dike fill would help monitor and define the phreatic surface. The vertical slope inclinometer tubes would determine if the dike and foundation were consolidating vertically or were spreading laterally, or if the foundation material was being squeezed out by long-term creep and plastic flow.

Dredged Material Management Plan

53. Incremental raising of the dike heights along the east and north perimeter dikes has historically been by dragline excavating dewatered dredged material from within dredged material disposal area along the dike alignment. It is anticipated that this technique and the quality of fill material will improve because of implementation of the dredged material management plan. Selective placement of the dredged material discharge pipe along these dike alignments have in the past minimized the placement distances and provided areas for borrow sources for truck hauling dredged sand to the west perimeter dike. Historically, fill material for raising the west perimeter dike has been excavated and truck-hauled from the northwest corner or from the north side of the disposal area. Since completion of the interior division dikes, fill material has been excavated and truck-hauled from borrow sources located at the east end of these interior dikes.

54. Implementation of the dredged material management plan will improve the foundation conditions for incremental dike raising along the west perimeter dike. The use of draglines on mats on the dried crust can supplement the amount of fill material that is now being truck-hauled long distances.

55. Sand materials will be required for an 8-ft-high berm constructed on top of the road and for the berm adjacent to the west perimeter dike.

Volume requirements on the 10,000-ft-long road berm will be about 274,000 cu yd, and about 2.5 million cu yd will be required for the 300-ft-wide berm placed in the water adjacent to the west perimeter berm. These materials may be truck-hauled or hydraulically placed depending on which method is the most convenient or economical.

PART VI: CONCLUSIONS AND RECOMMENDATIONS

56. It was concluded that it is technically feasible to raise the west perimeter dike to el +34 and dredged material to el +30 ft with a factor of safety of 1.3 (Figure 28). It was also concluded that it was technically feasible to raise the east and north perimeter dikes to el +40 ft and dredged material to el +36 ft with a factor of safety of 1.4 and 1.28, respectively (Figures 31 and 34). The northwest corner can also be raised to el +34 and dredged material to el +30 ft without the factor of safety dropping below 1.5 (Figure 36).

57. It is recommended that a 300-ft-wide water berm be constructed on the west dike starting at el 0 ft and intercepting the existing dike at el +7 ft and a road berm be constructed to el +10 or 12 ft above the existing roadway. This scenario would not require changing the existing slope of 8H:1V. It was concluded that the east, north, and northwest dikes would not require construction of berms if the present setback distances and slopes are maintained.

58. It was concluded that the use of "wick drains" to increase foundation shear strength was not economically feasible because of excessive depth of dike cross section. If the dike height requirements increase, a reassessment of the use "wick drain" should be conducted based on field trials along the perimeter dike. It was also concluded that the use of geotextile reinforcement in the raised dike sections would not increase the factor of safety because the potential failure plane would not intercept the fabric.

59. Settlement for the east, west, and north perimeter dikes was estimated to be about 4 ft for each dike, respectively.

60. Construction of a road berm on the west perimeter dike is presently being constructed by truck-hauling dredged sand from nearby sand sources to cover the 3-ft-diam weir pipes that cross the road. Covering these weir pipes will more than satisfy the 2-ft berm required. Construction of the water berm may be accomplished by selective placement of dredged sand in thin lifts.

61. Continued and successful implementation of the Craney Island dredged material management plan will provide not only improved foundation conditions as the fine-grained dredged material forms a dried crust but also a good source of fill material for continued incremental dike raising and a fairly impermeable dike.

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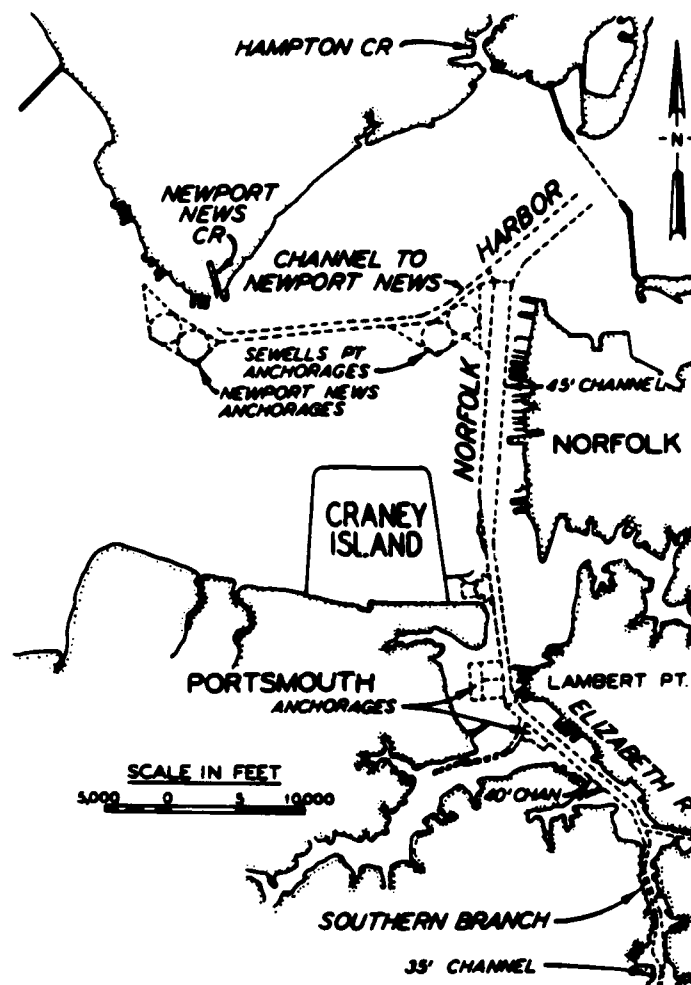
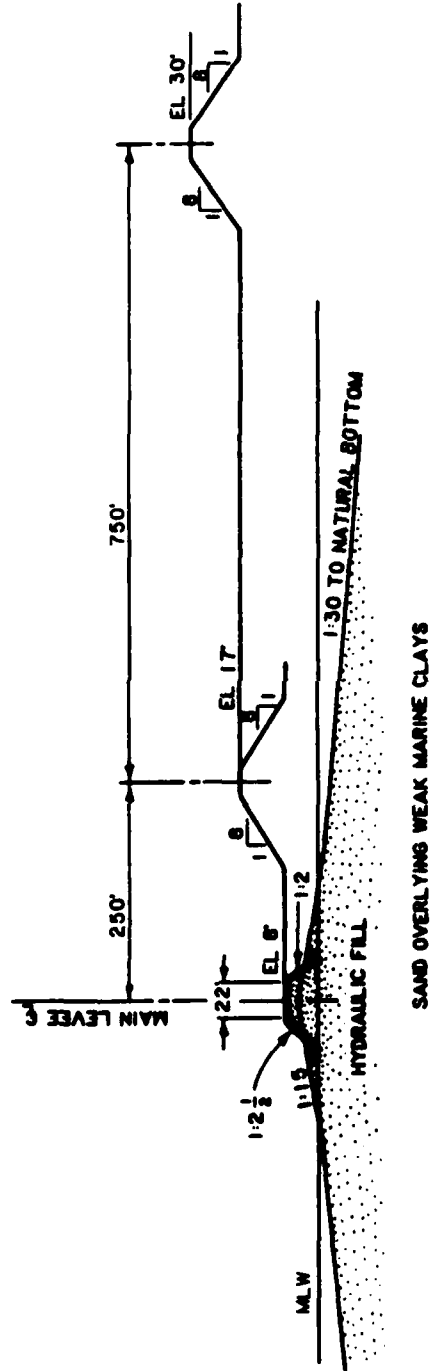


Figure 1. Project location



TYPICAL SECTION THROUGH LEVEE (NOT TO SCALE)

Figure 2. Proposed benched cross-section for west perimeter dike by Norfolk District

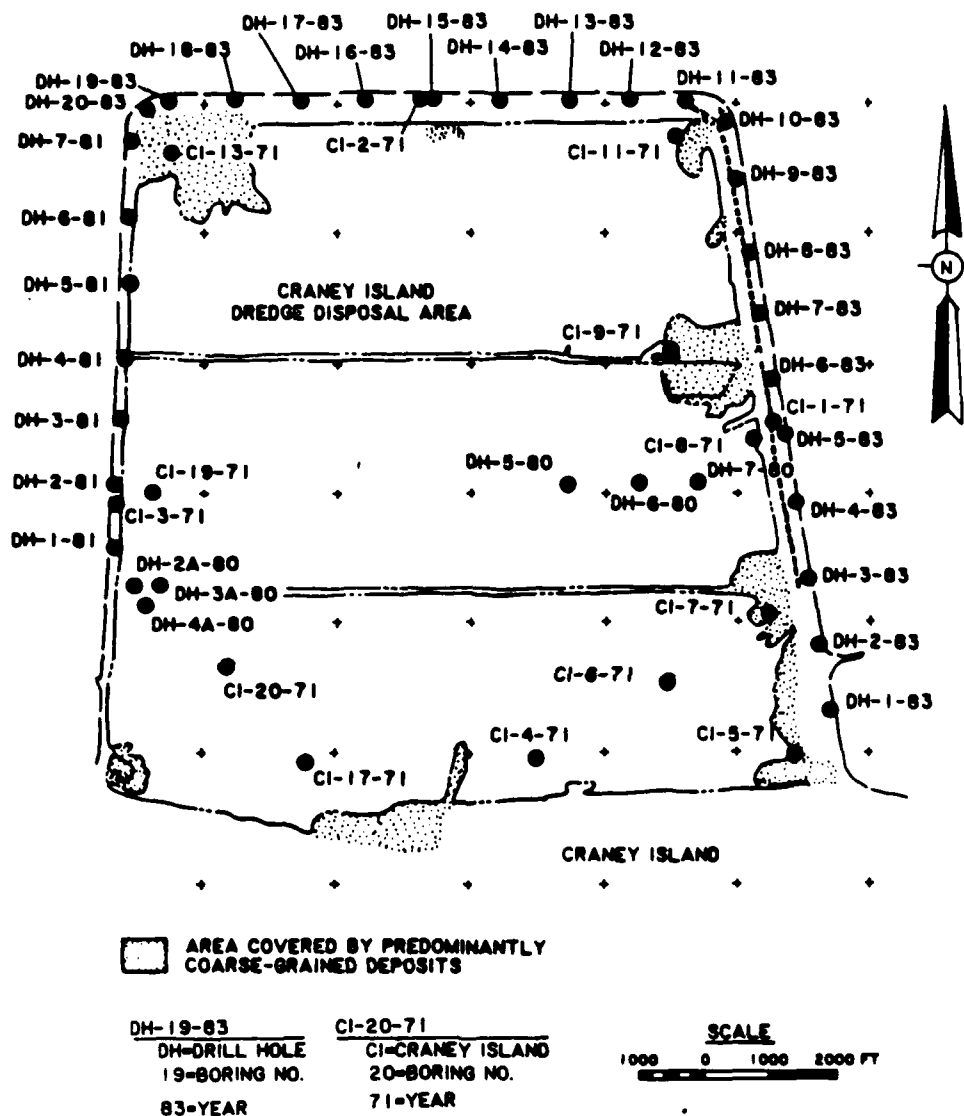


Figure 3. Boring plan and coarse grain material at inflow points

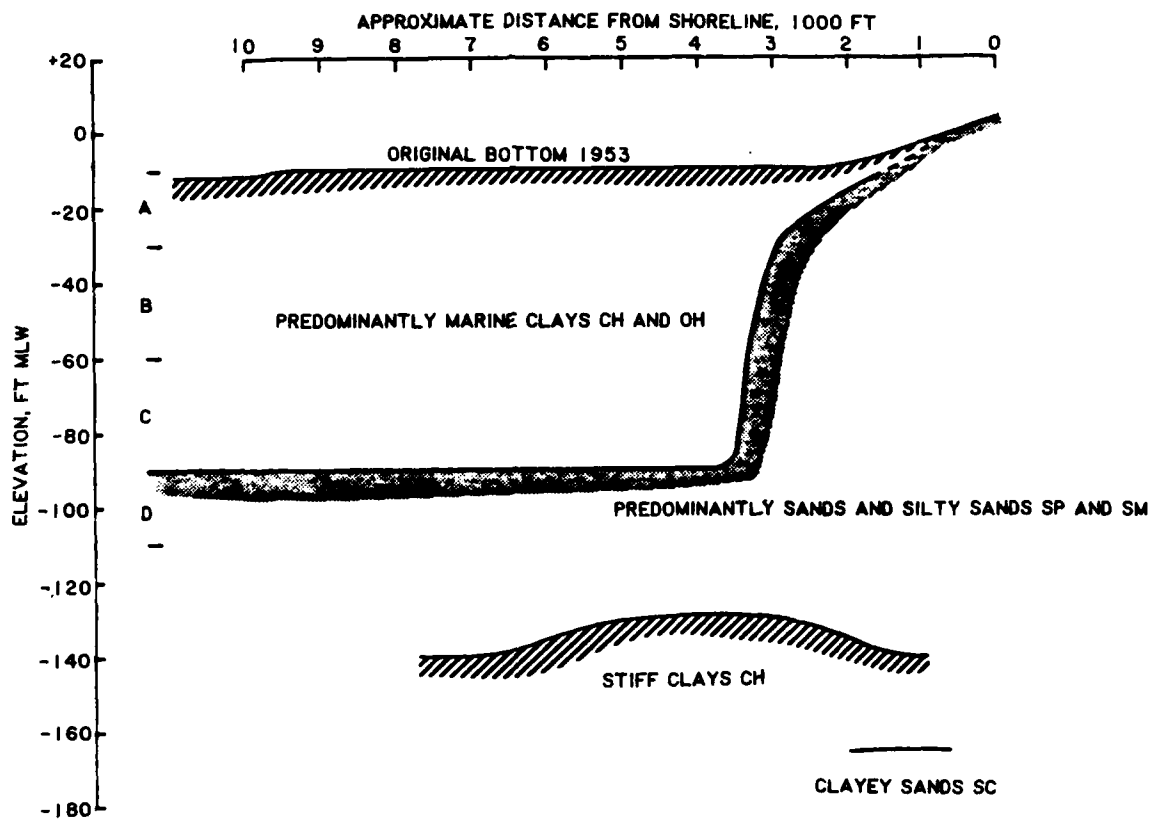


Figure 4. Generalized foundation conditions

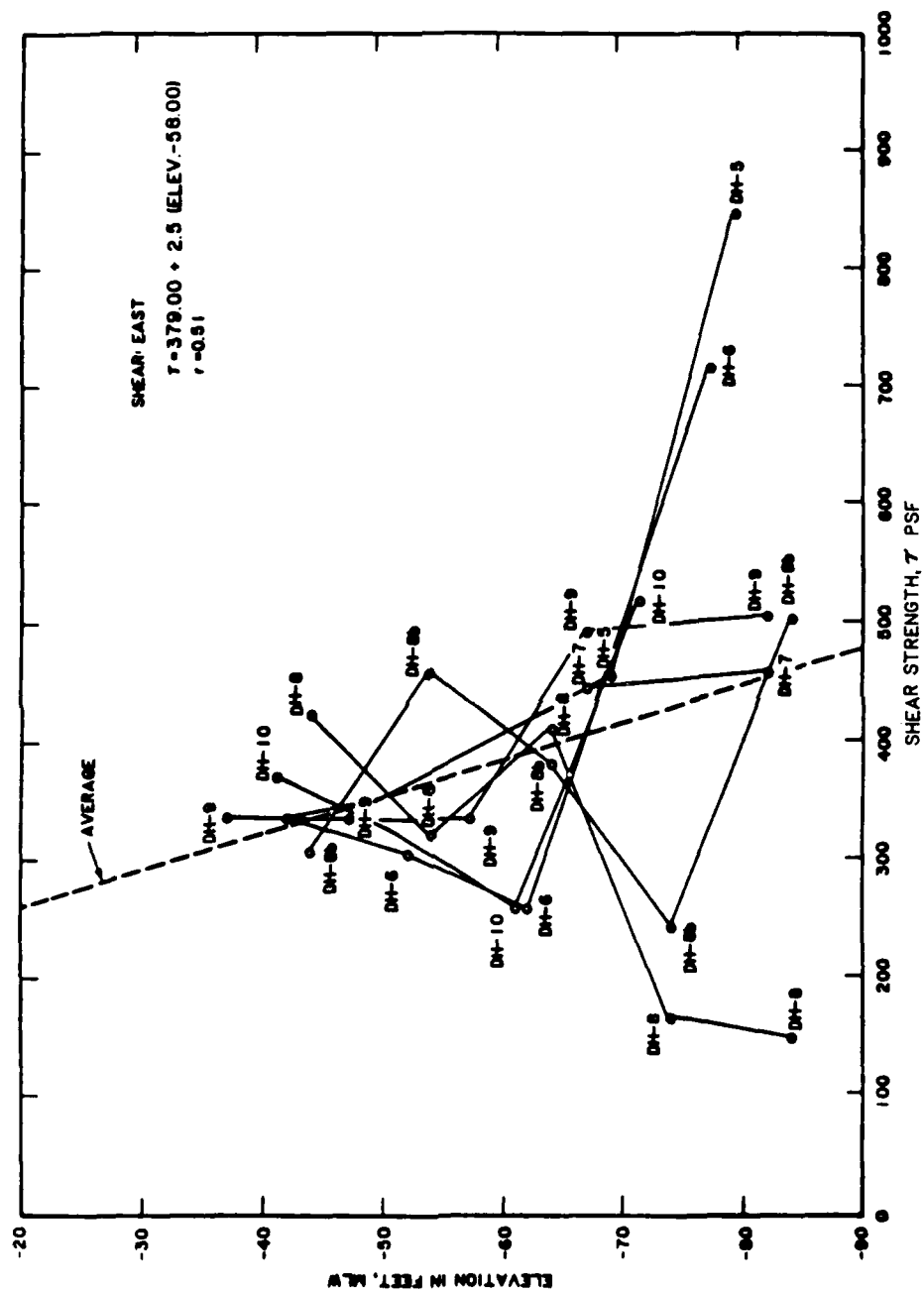


Figure 5. Shear strength versus depth-East

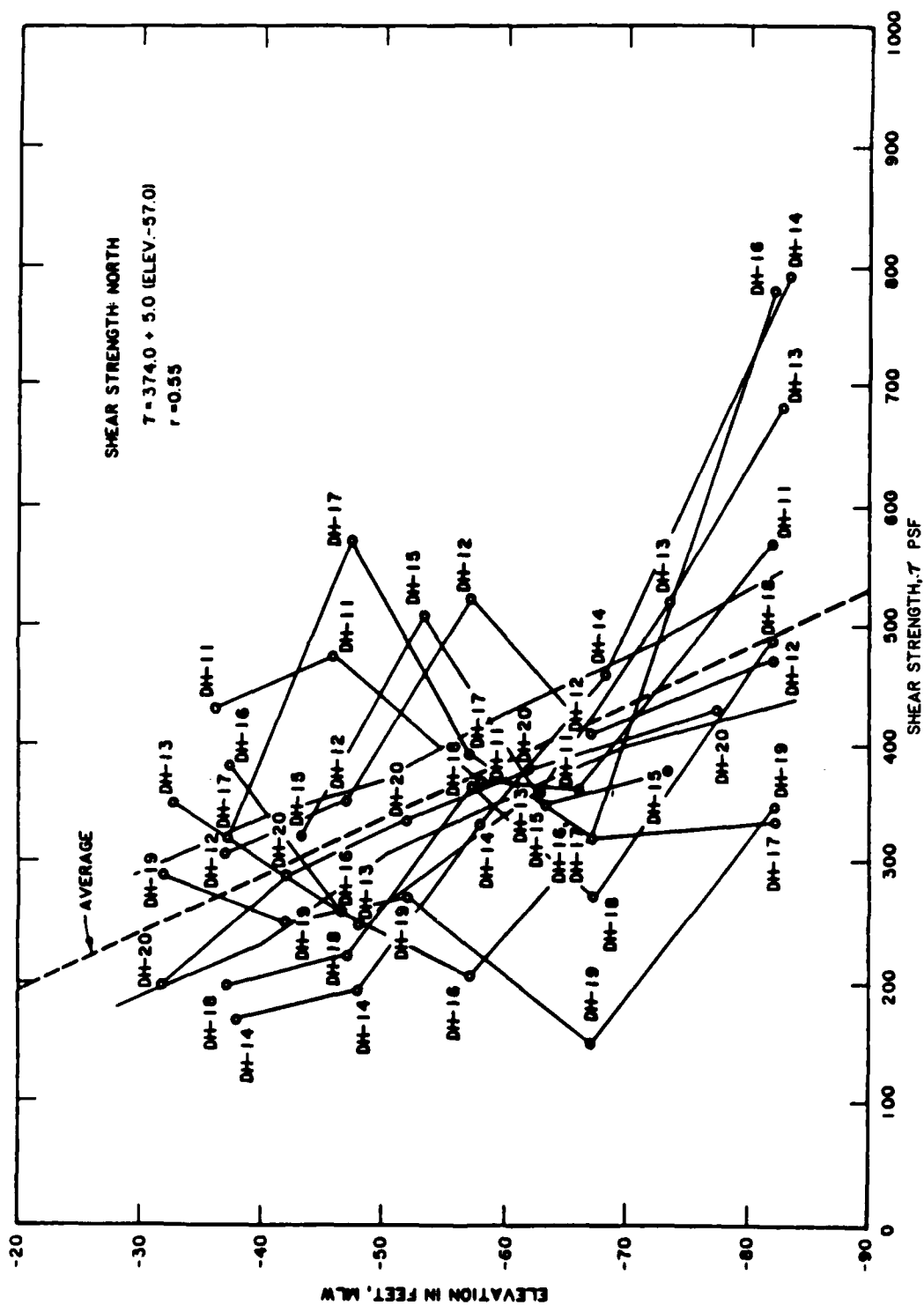


Figure 6. Shear strength versus depth-North

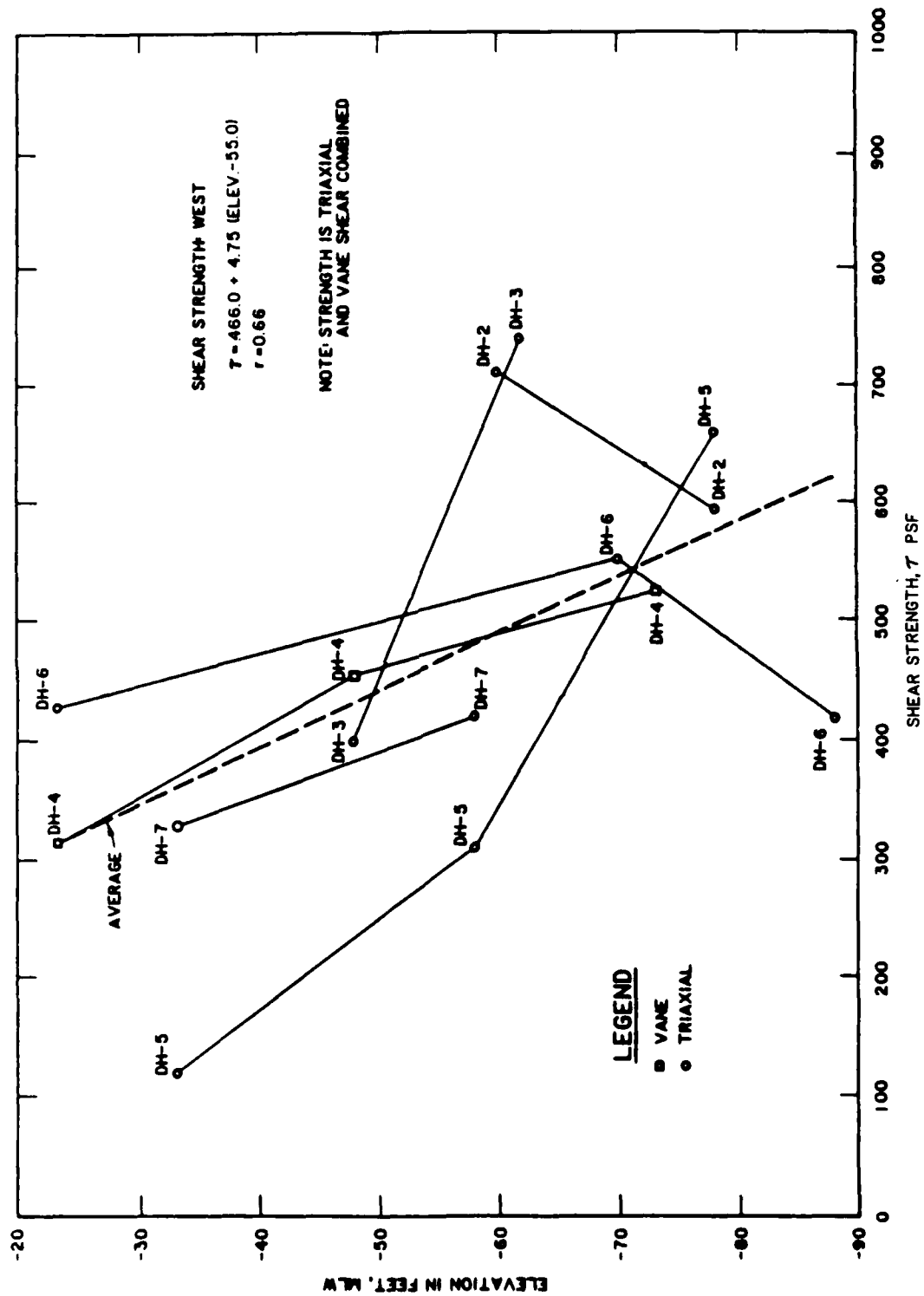


Figure 7. Shear strength versus depth-West

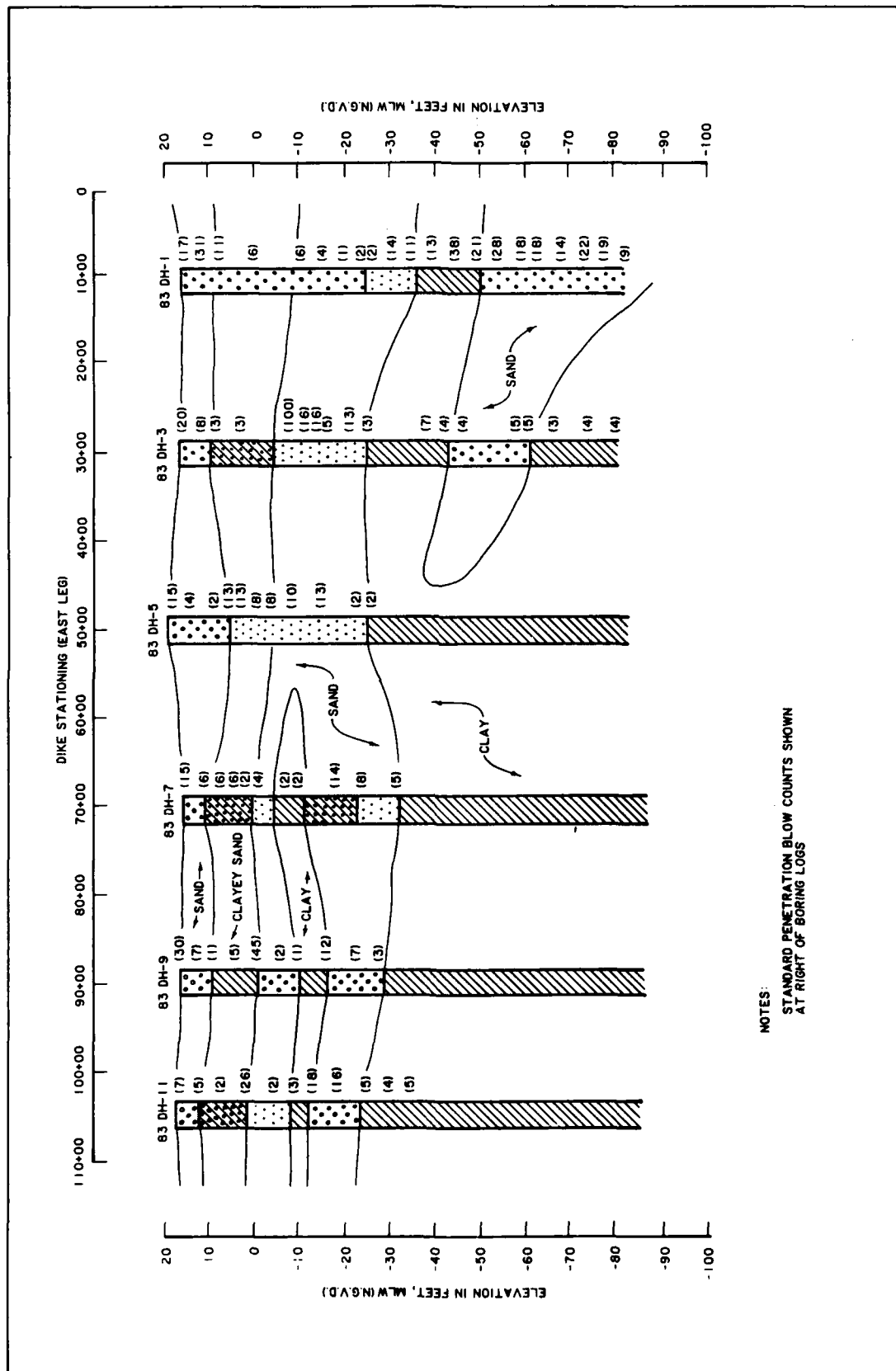


Figure 8. Subsurface profile A-A-east leg

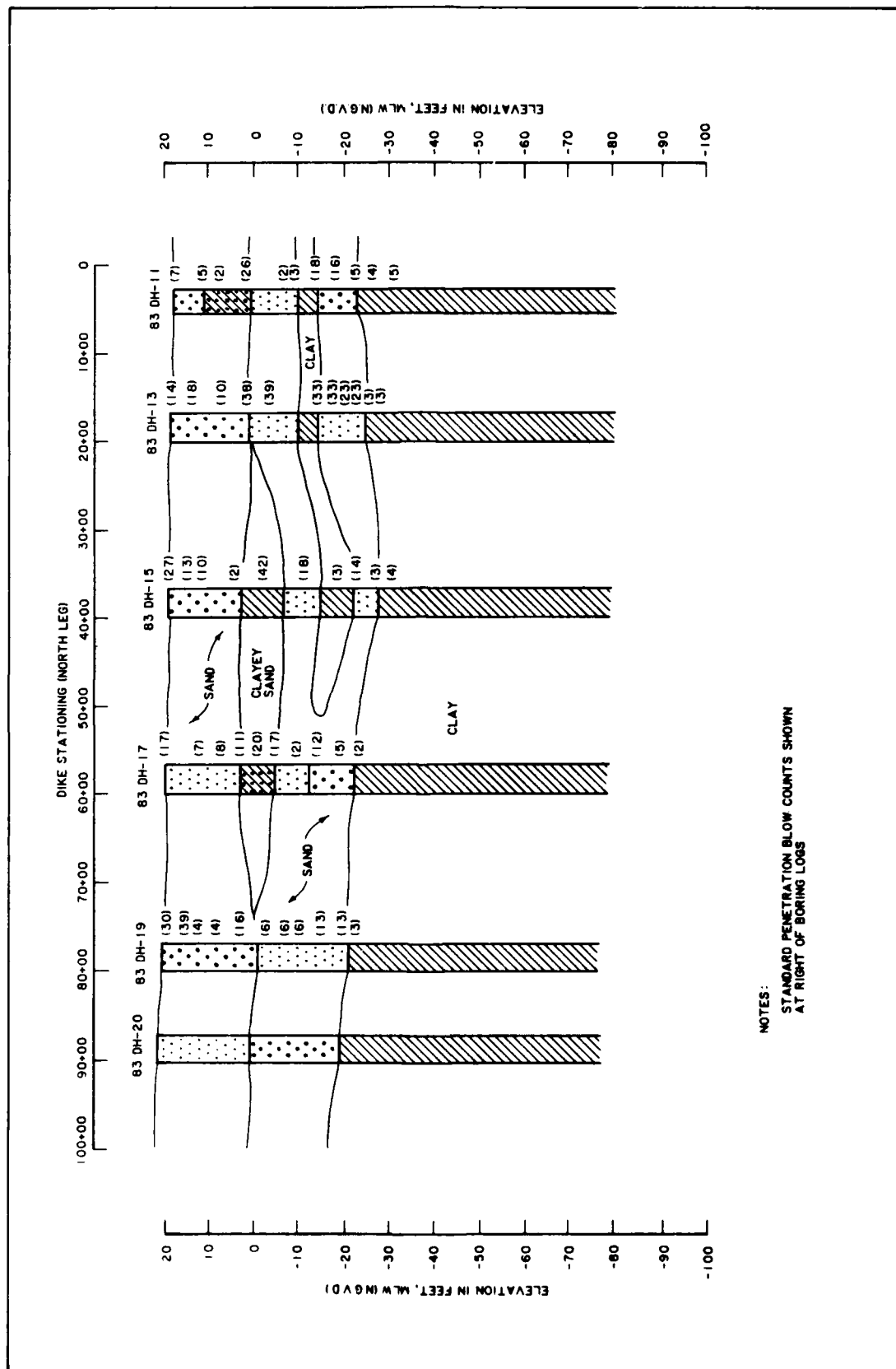


Figure 9. Subsurface profile B-B-north leg

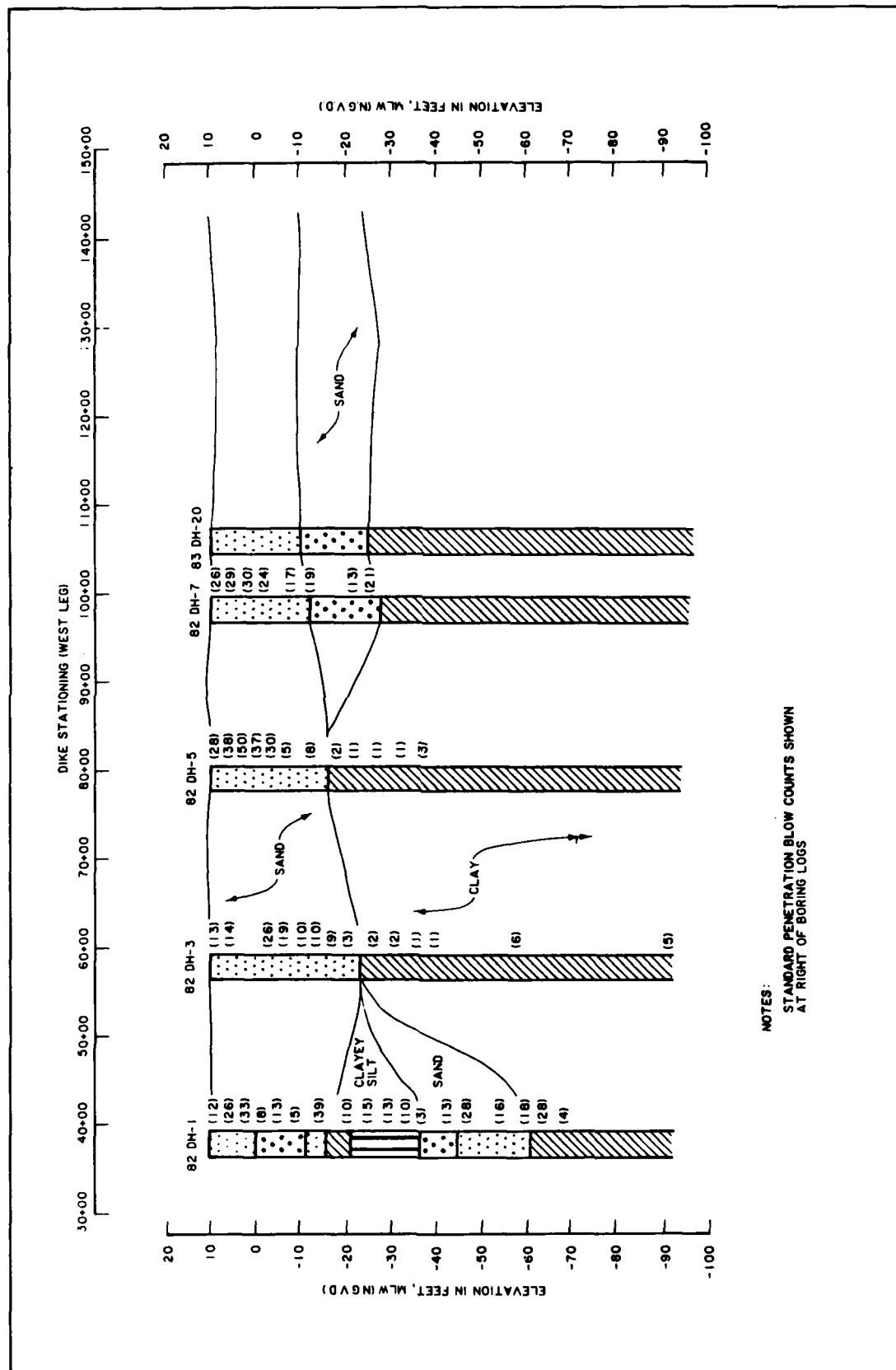


Figure 10. Subsurface profile C-C-west leg

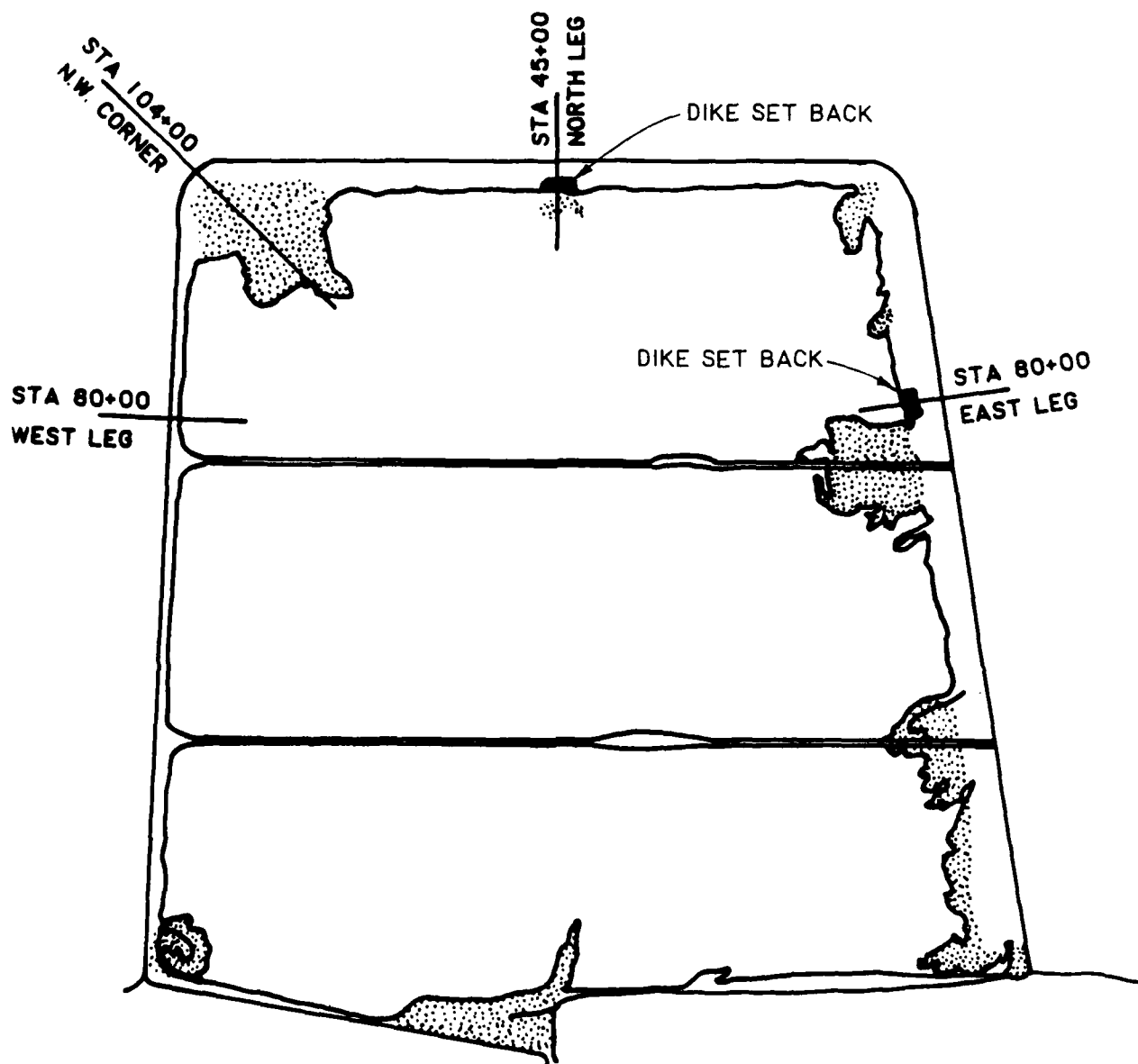


Figure 11. Location of cross section investigated during dike stability analysis

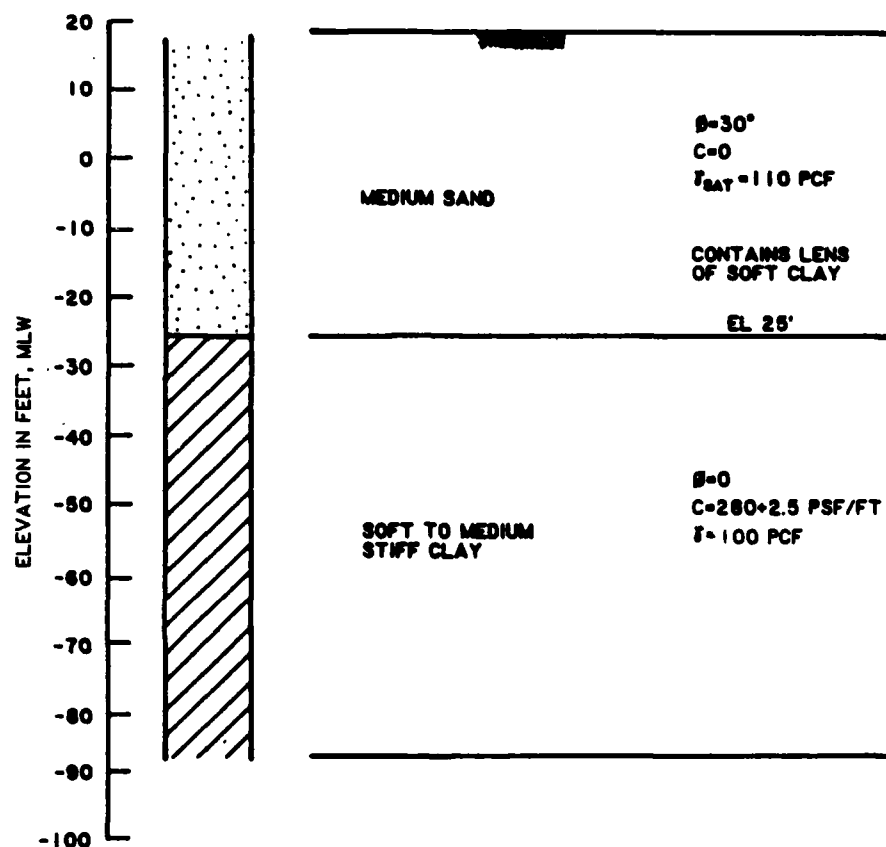


Figure 12. Generalized foundation shear strength profile for east dike stability analysis, sta 80+00

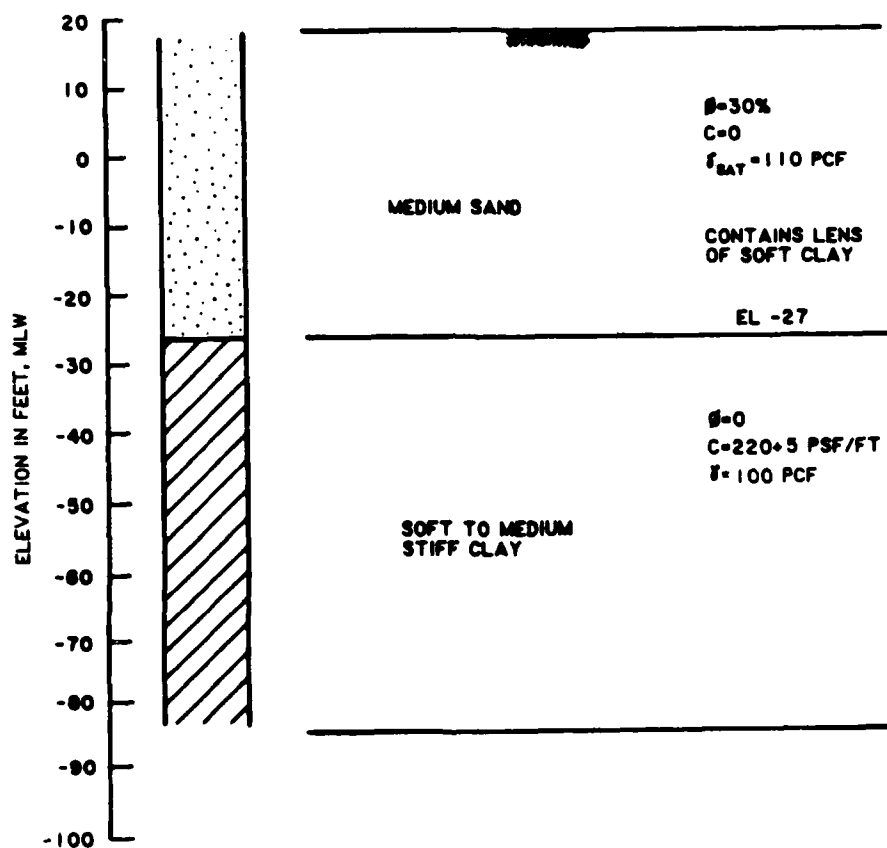


Figure 13. Generalized foundation shear strength profile for north dike stability analysis, north sta 45+00

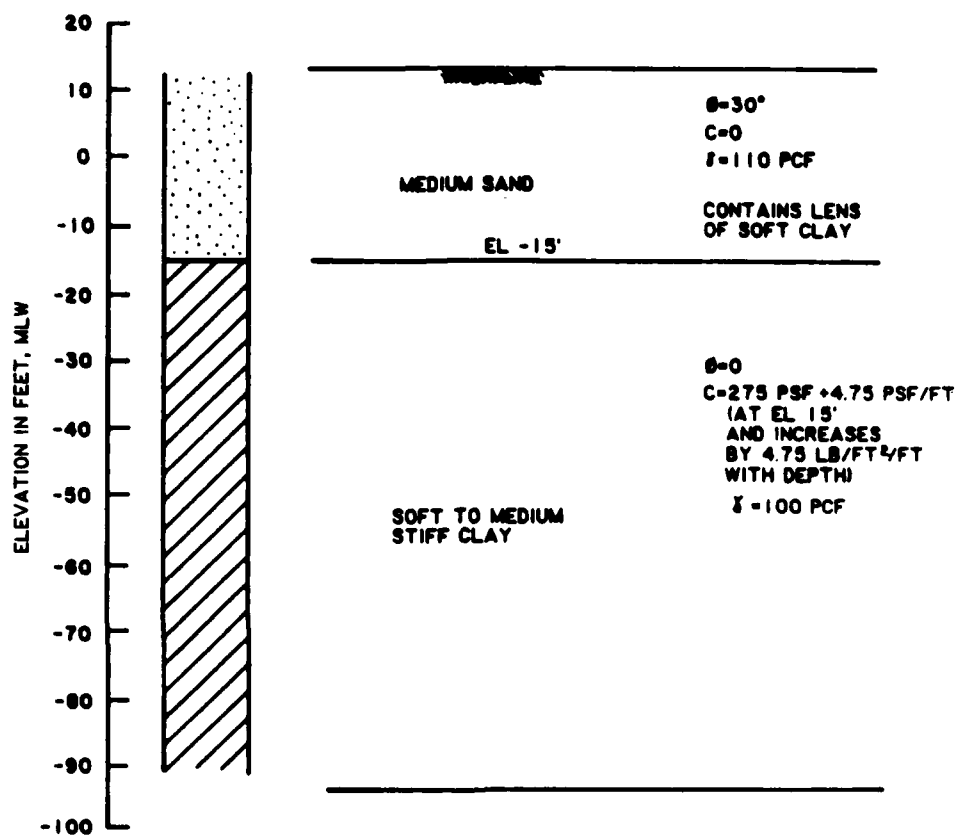


Figure 14. Generalized foundation shear strength profile for west dikes stability analysis, northwest corner sta 80+00

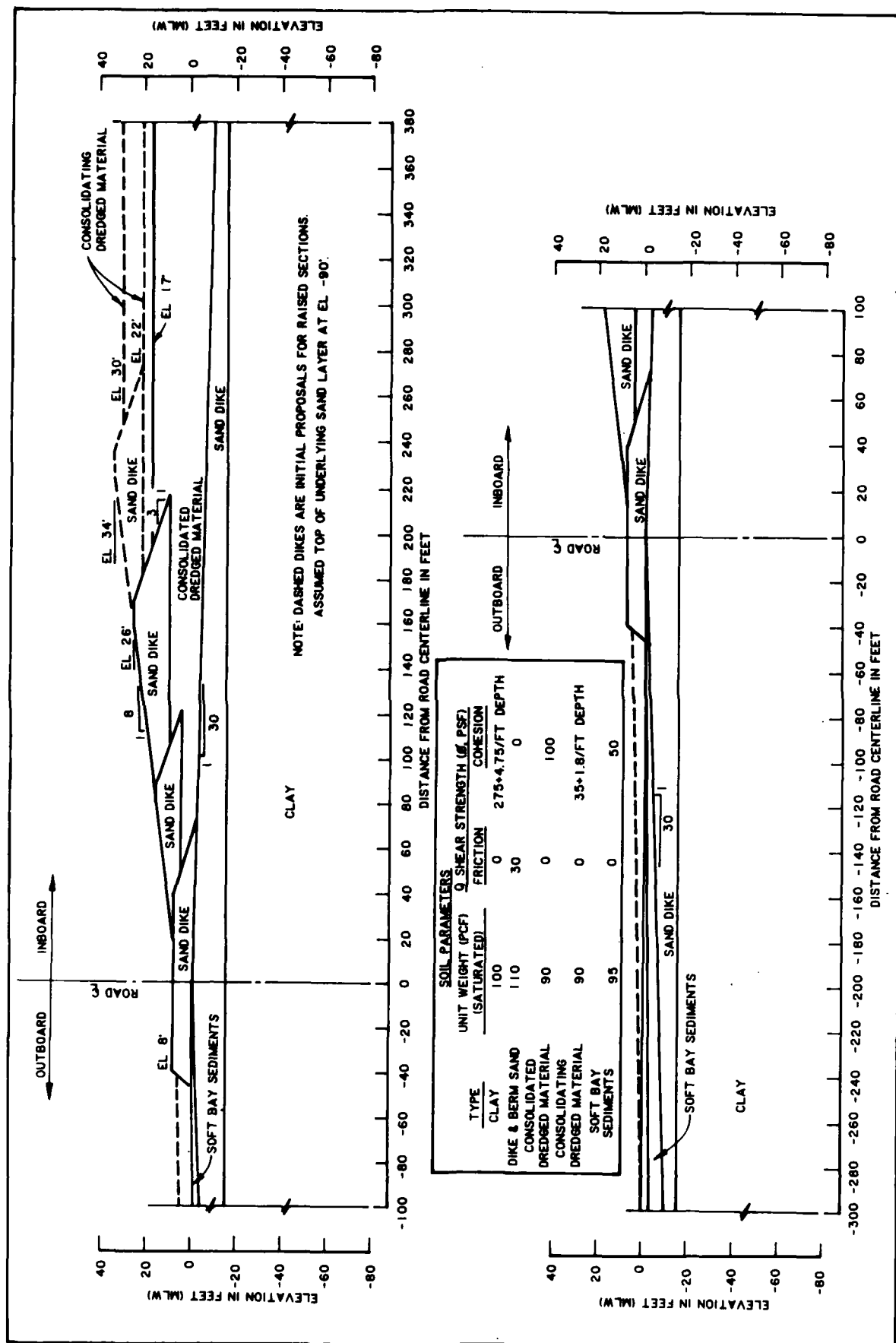


Figure 15. Generalized cross-section, west perimeter dike

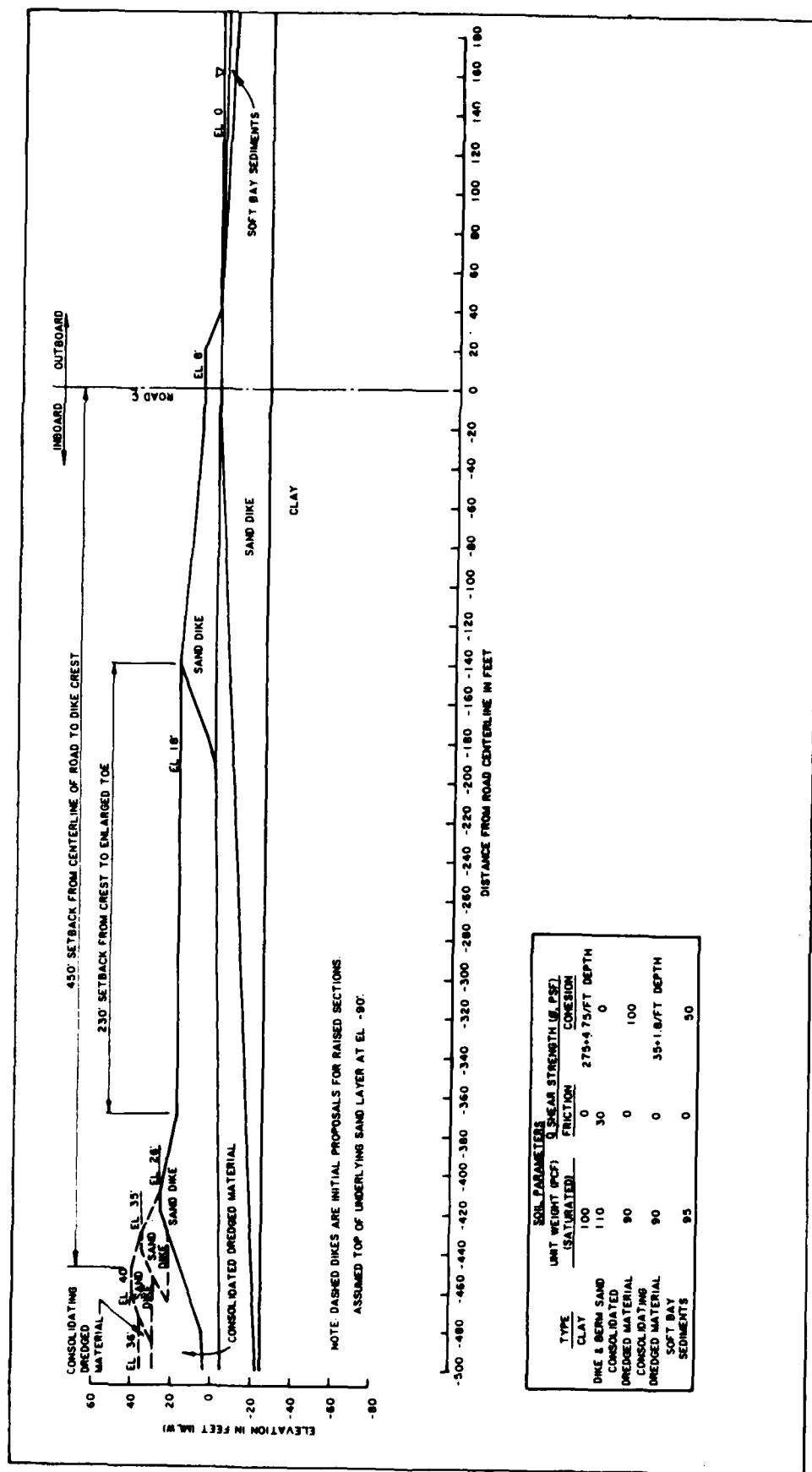


Figure 16. Generalized cross-section, east perimeter dike

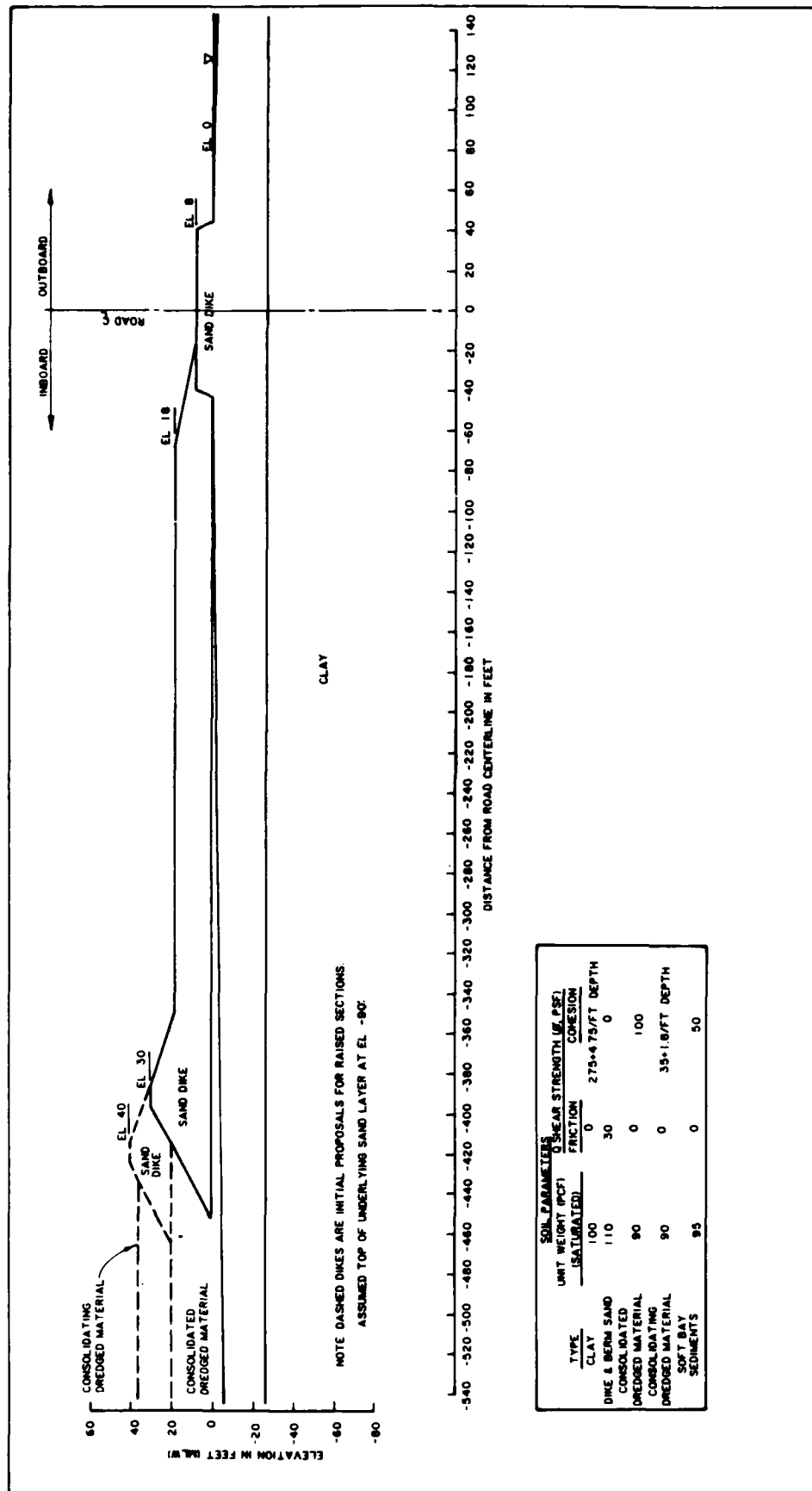


Figure 17. Generalized cross-section, north perimeter dike

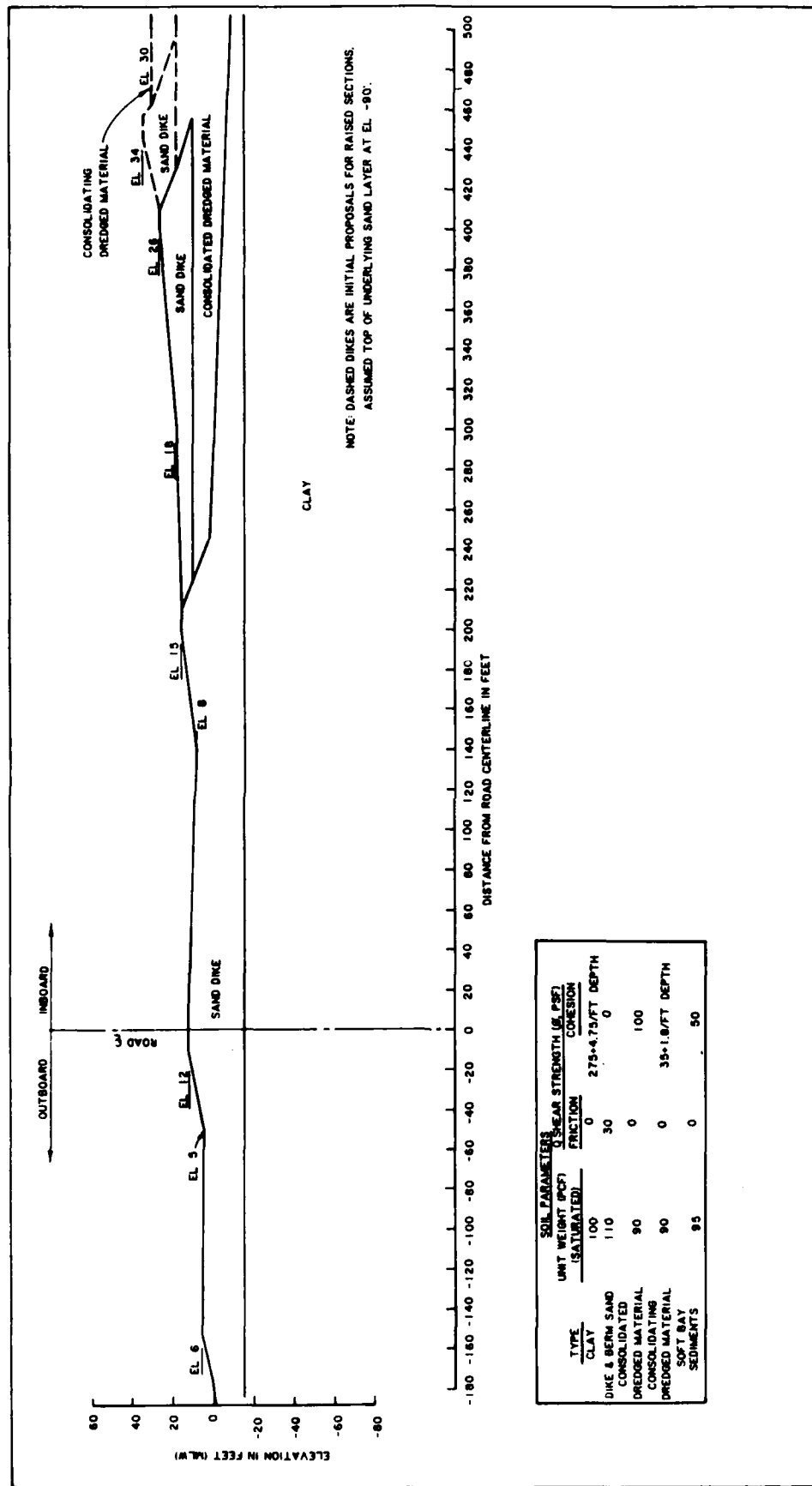


Figure 18. Generalized cross-section, northwest corner perimeter dike

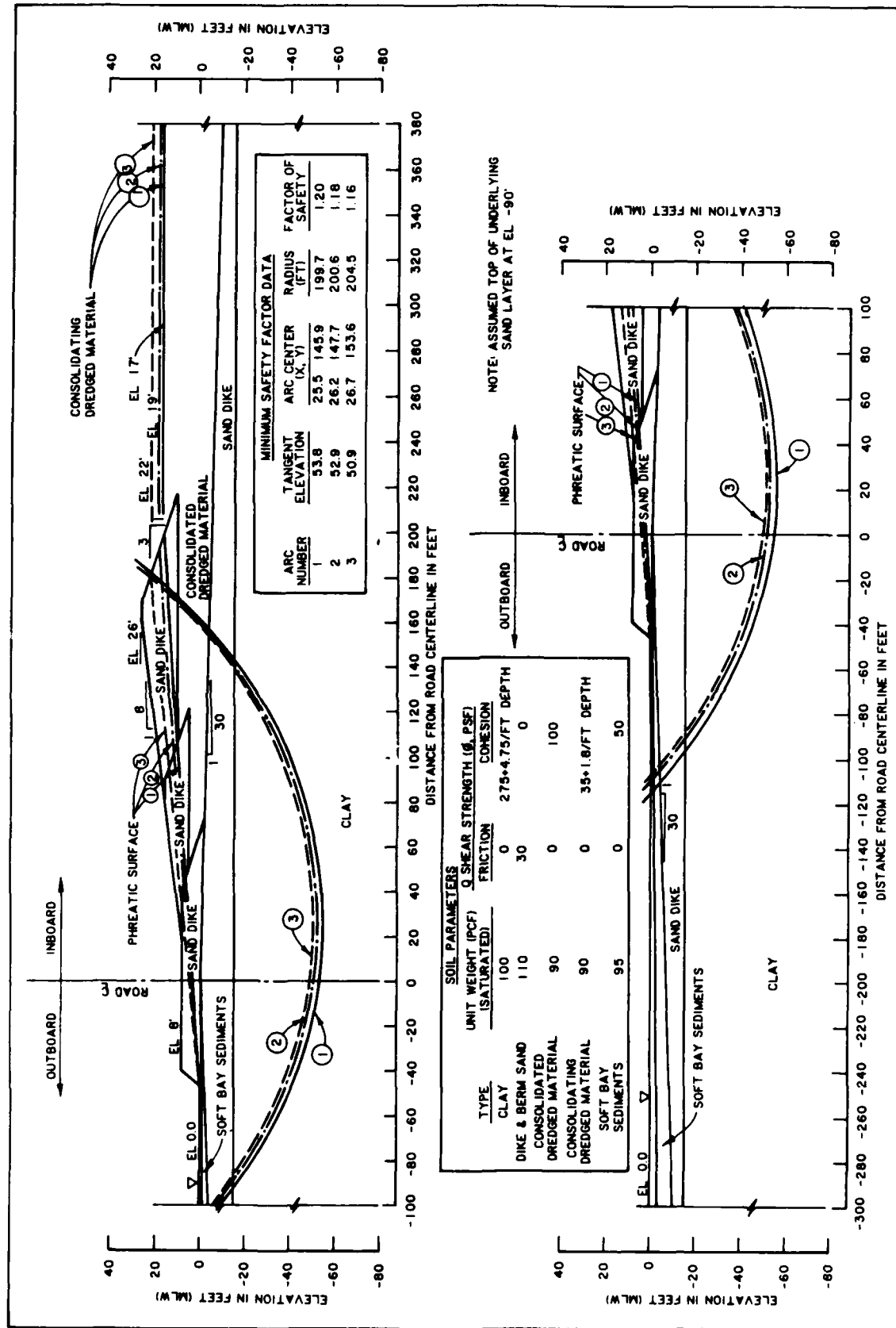


Figure 19. Critical circles for various dredged material elevations using existing west dike configuration

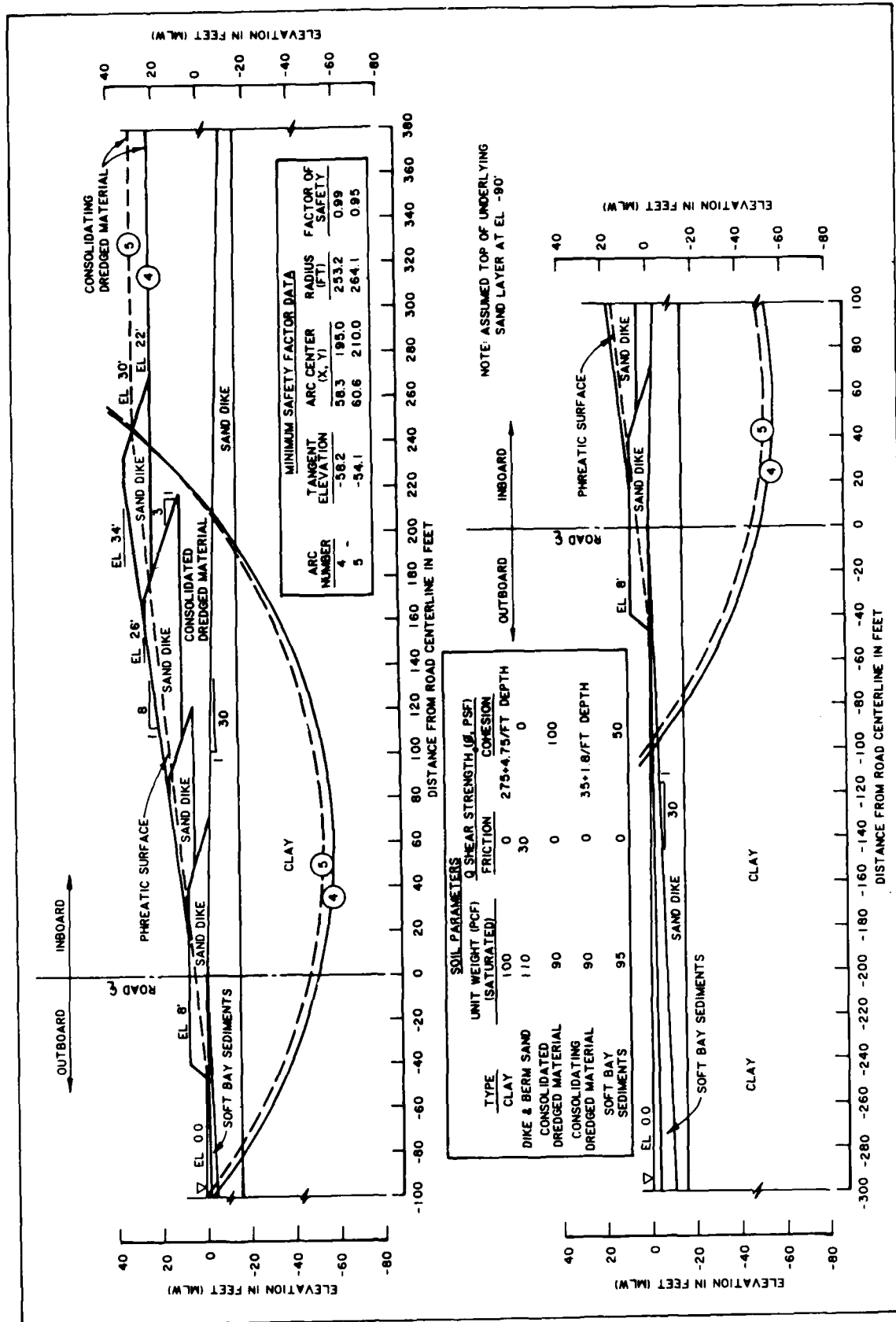


Figure 20. Critical circles for raised west dike with no berms

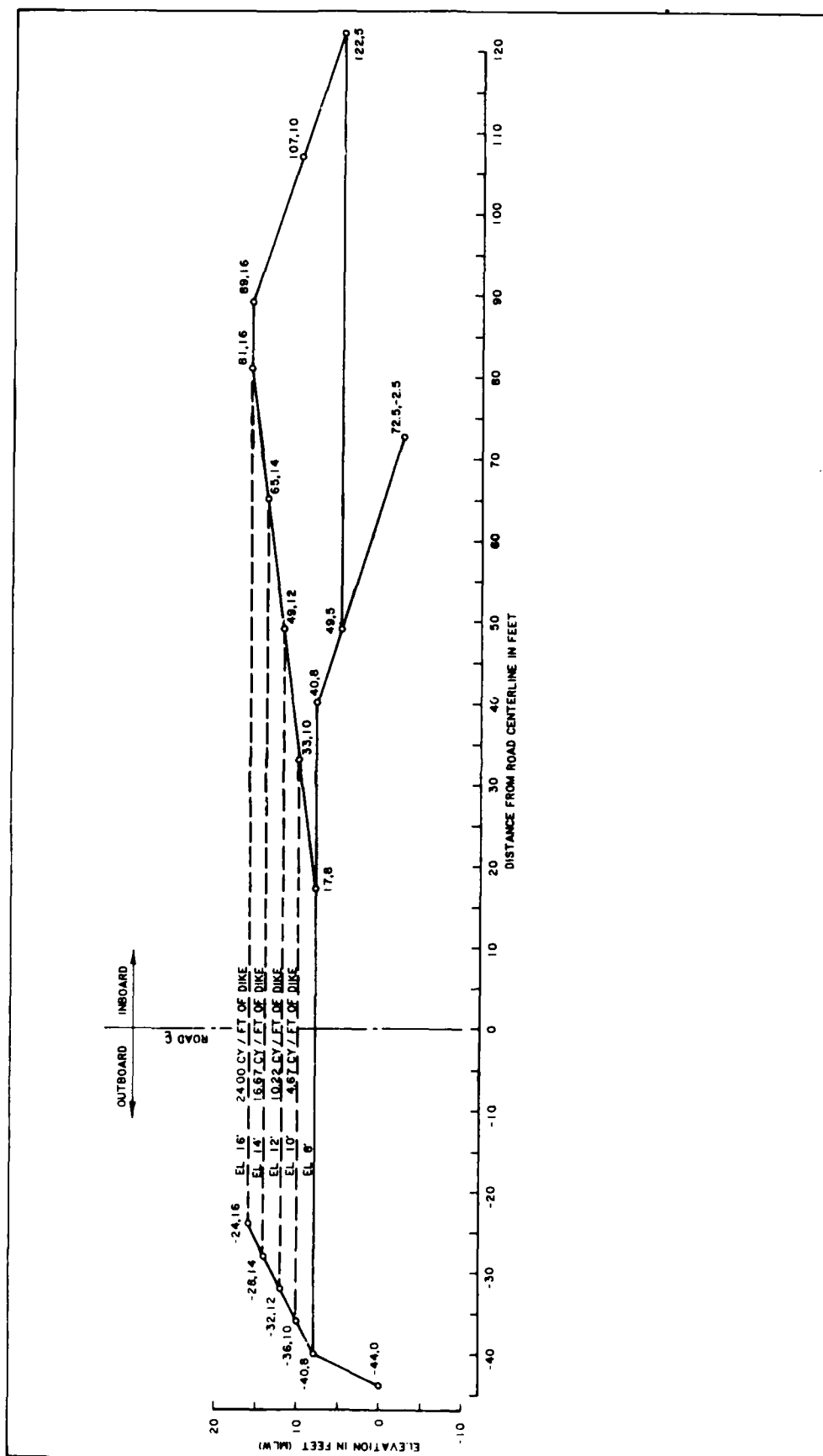


Figure 21. West dike road berm configuration

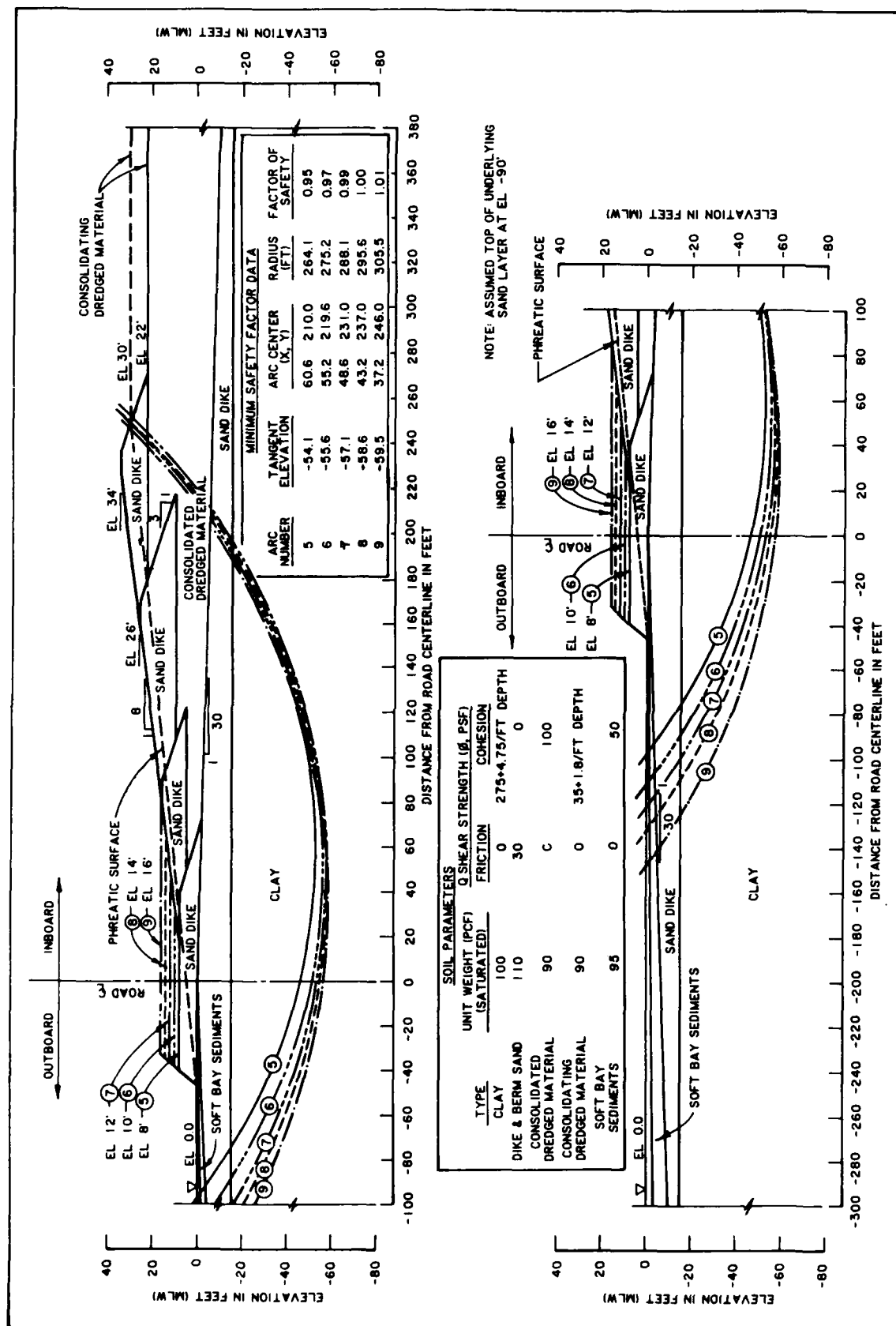


Figure 22. Critical circles for raised west dike with road berms

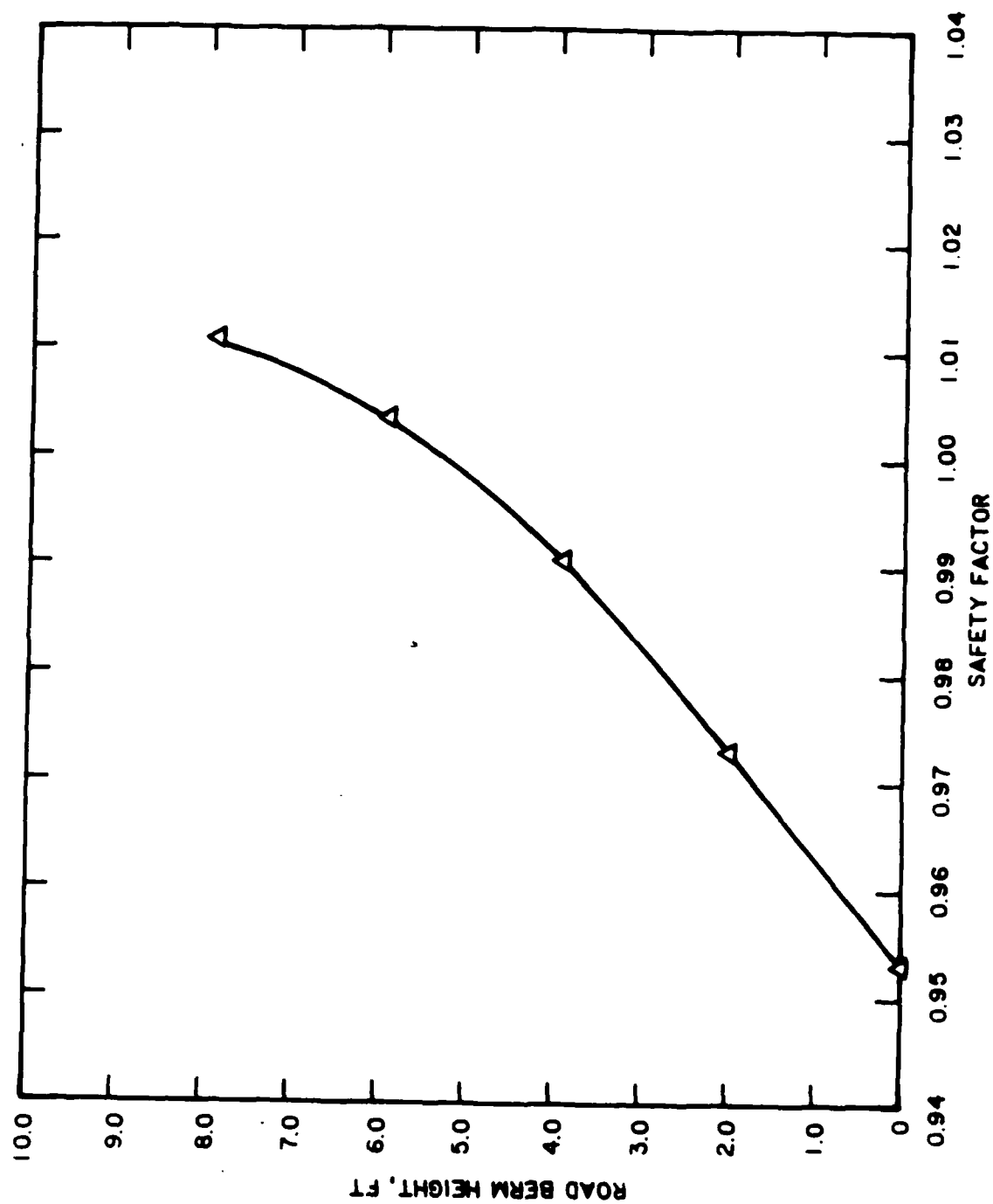


Figure 23. West dike road berm height versus safety factor

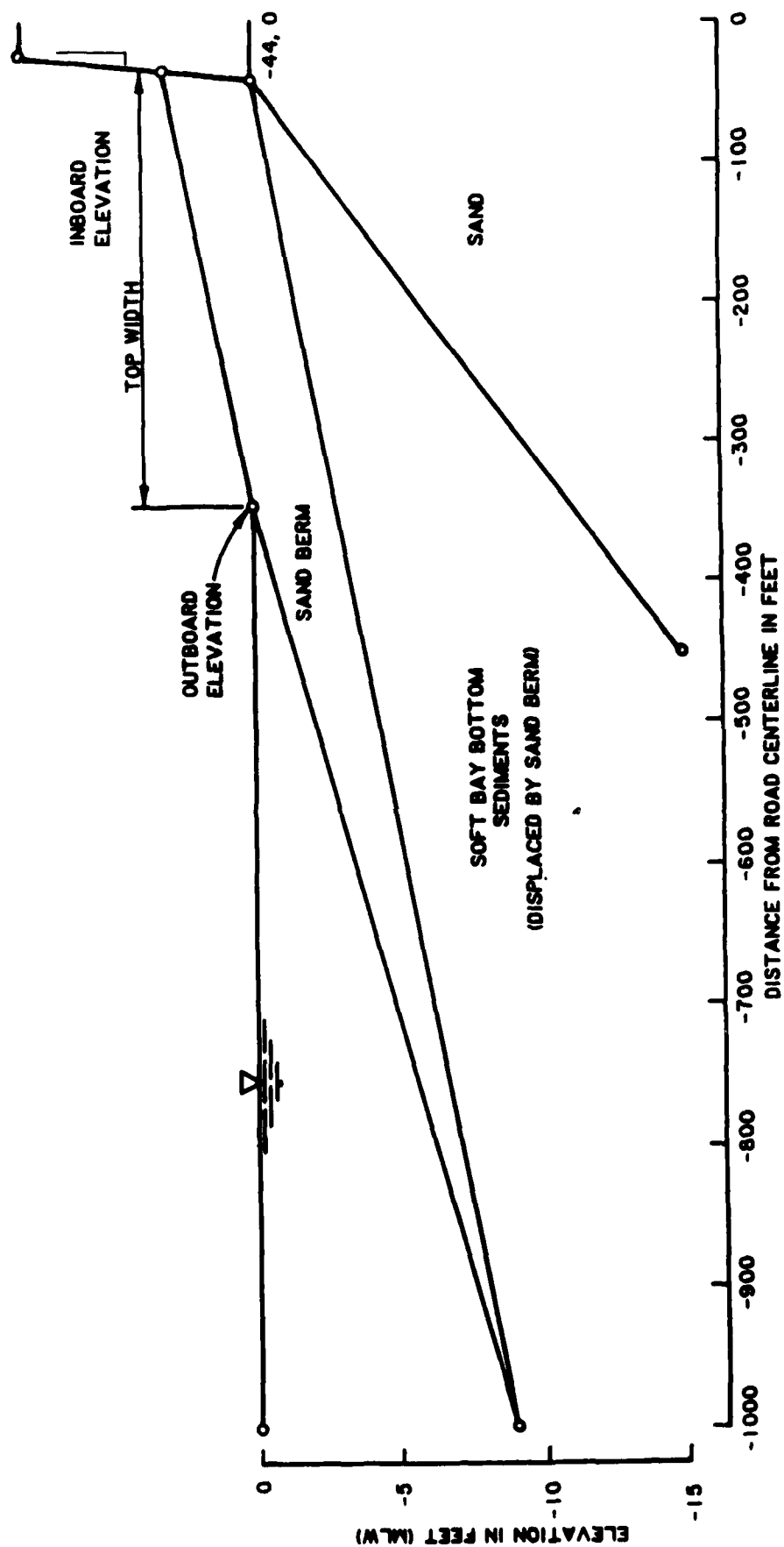


Figure 24. General schematic of a water berm illustrating terminology

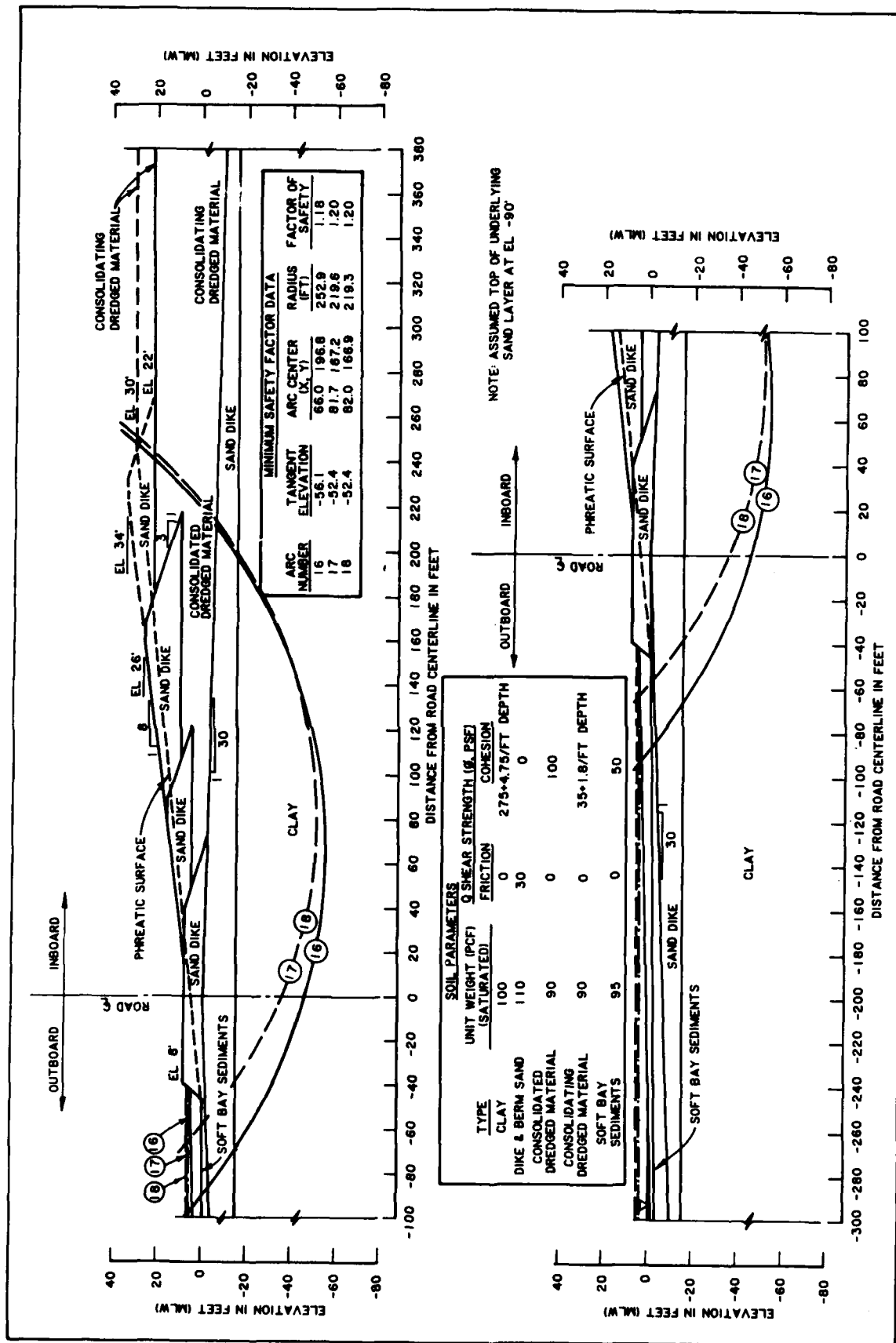


Figure 25. Critical circles for raised west dike with water berm elevations of +3 and +6

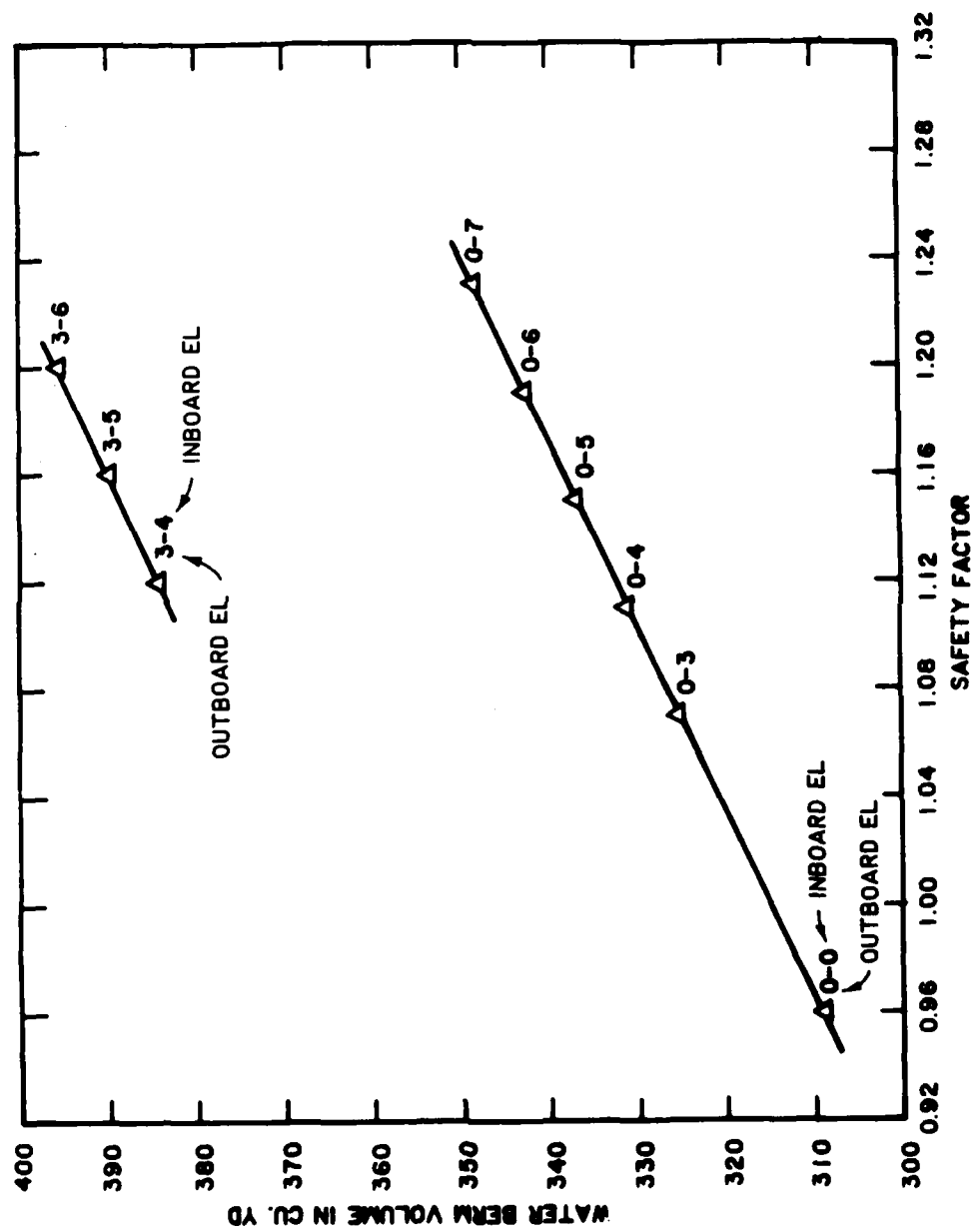


Figure 26. West dike water berm volume versus safety factor

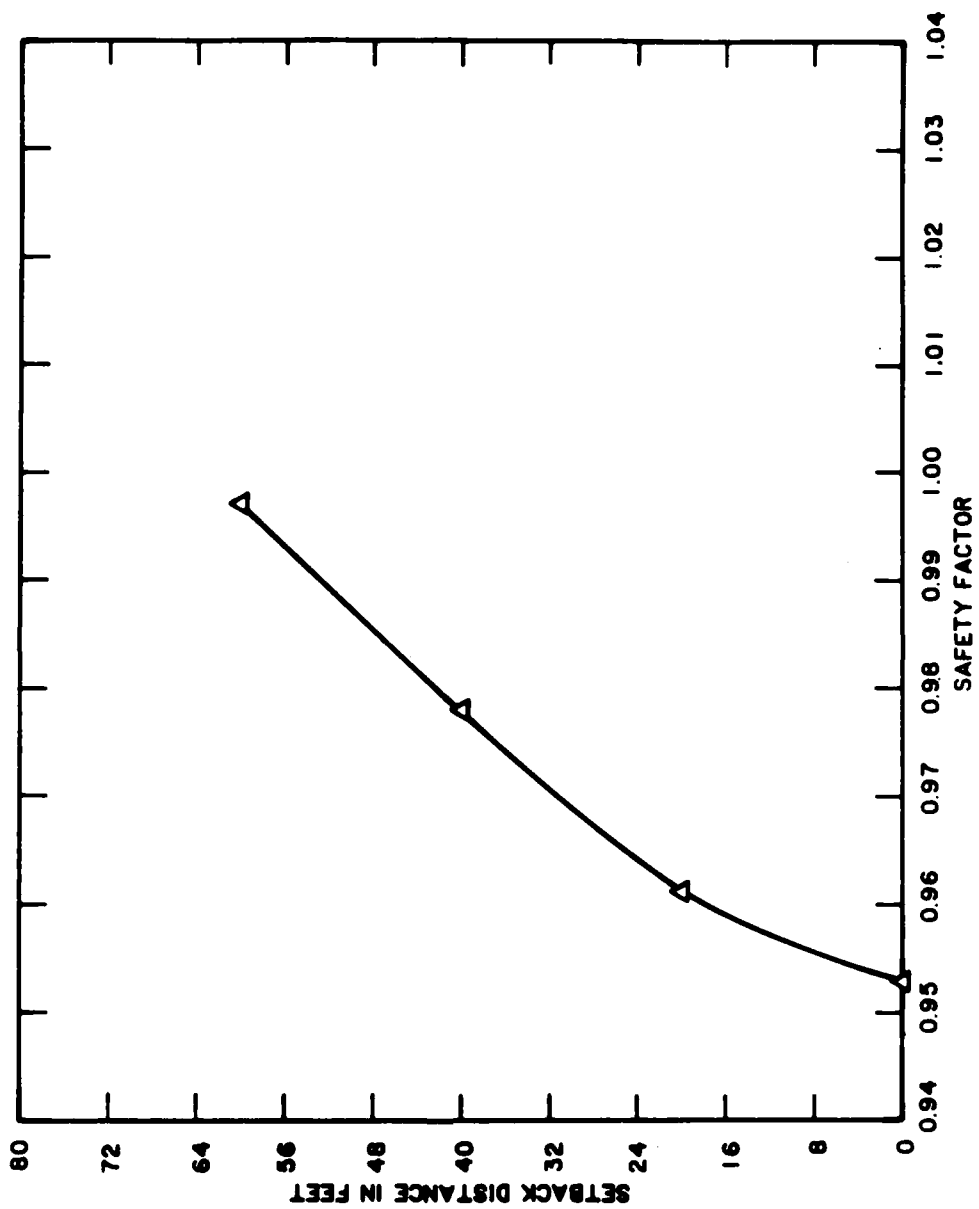


Figure 27. West dike setback versus safety factor

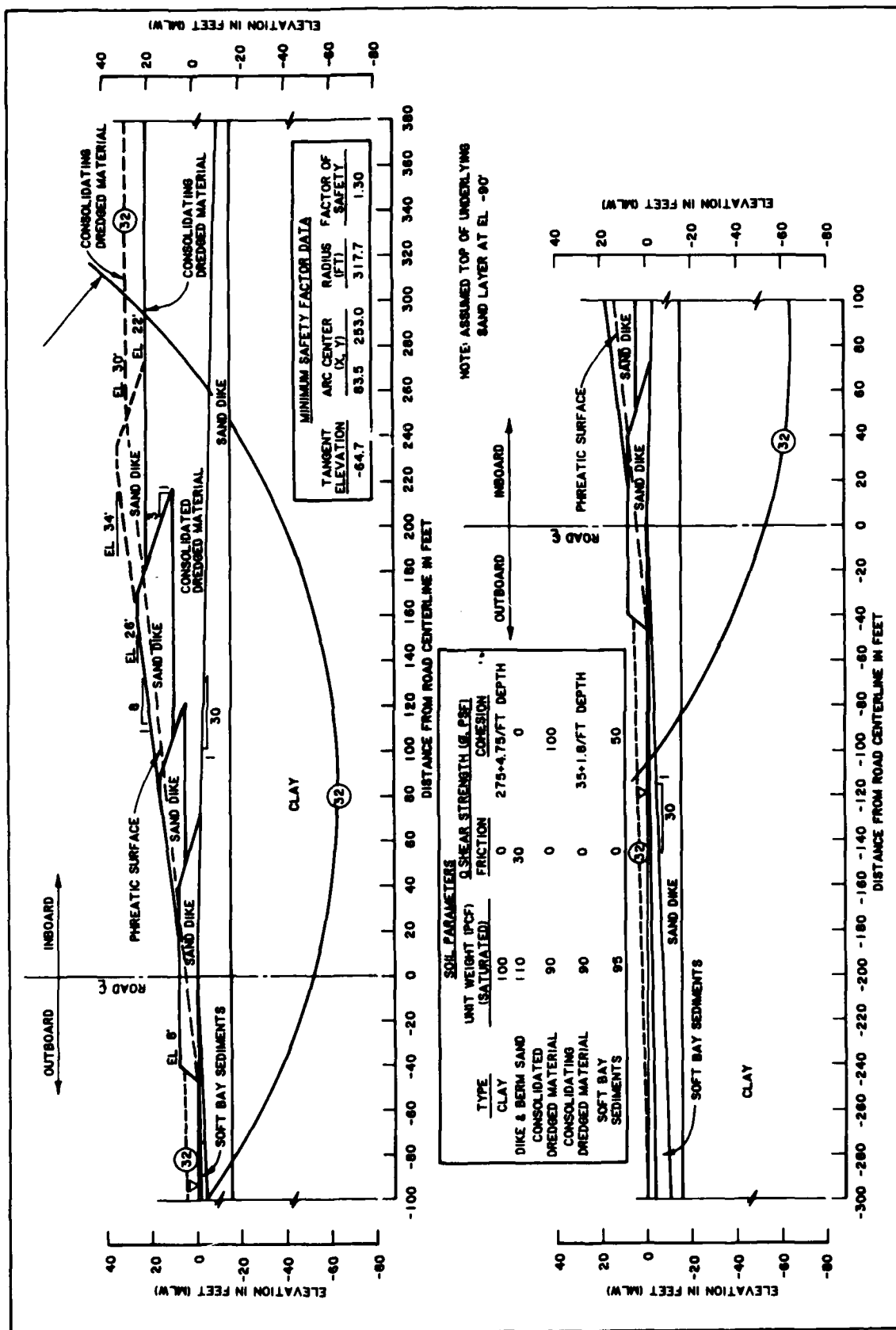


Figure 28. Recommended raised dike cross-section for the west perimeter dike

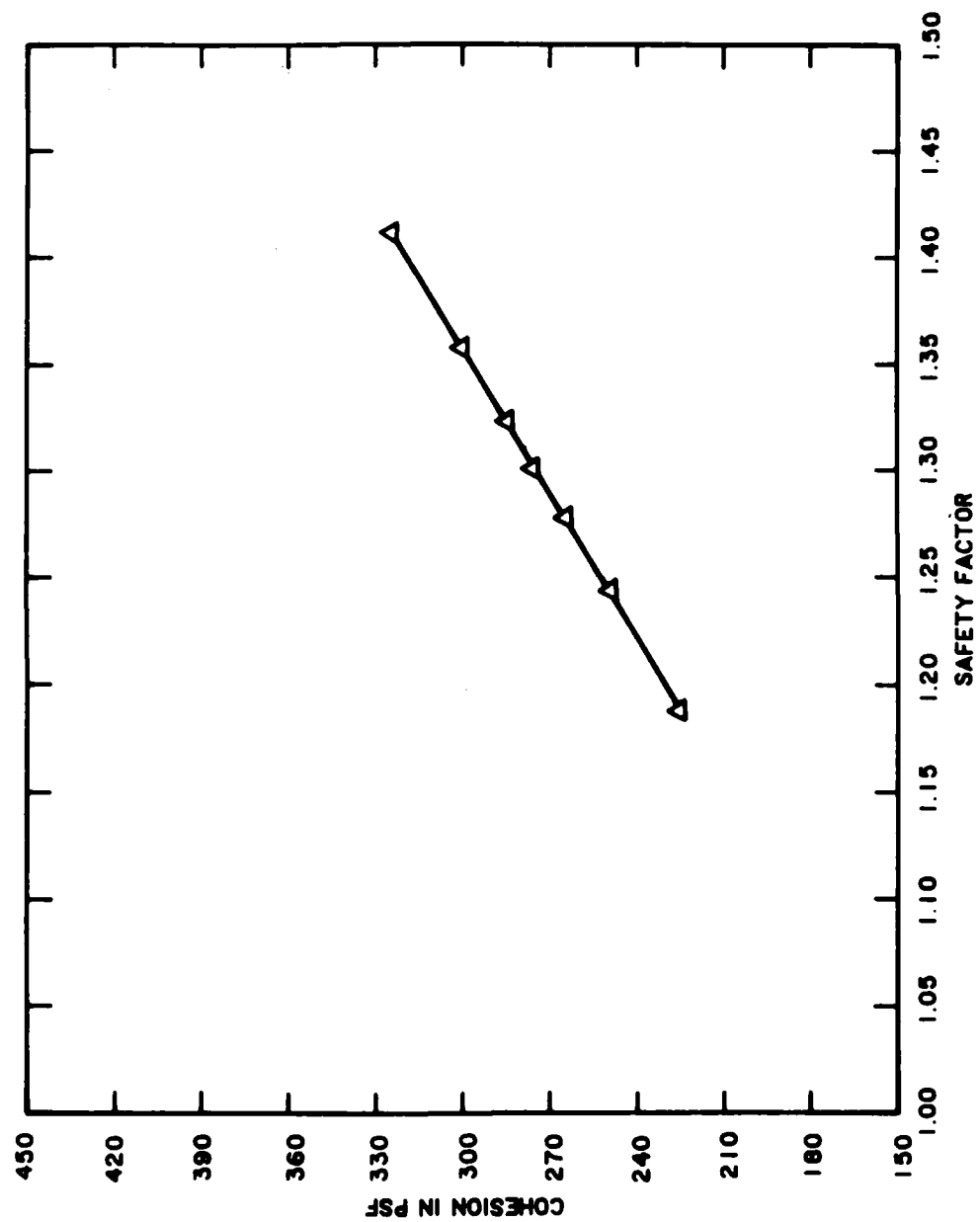


Figure 29. Cohesion of foundation clay under west dike versus safety factor

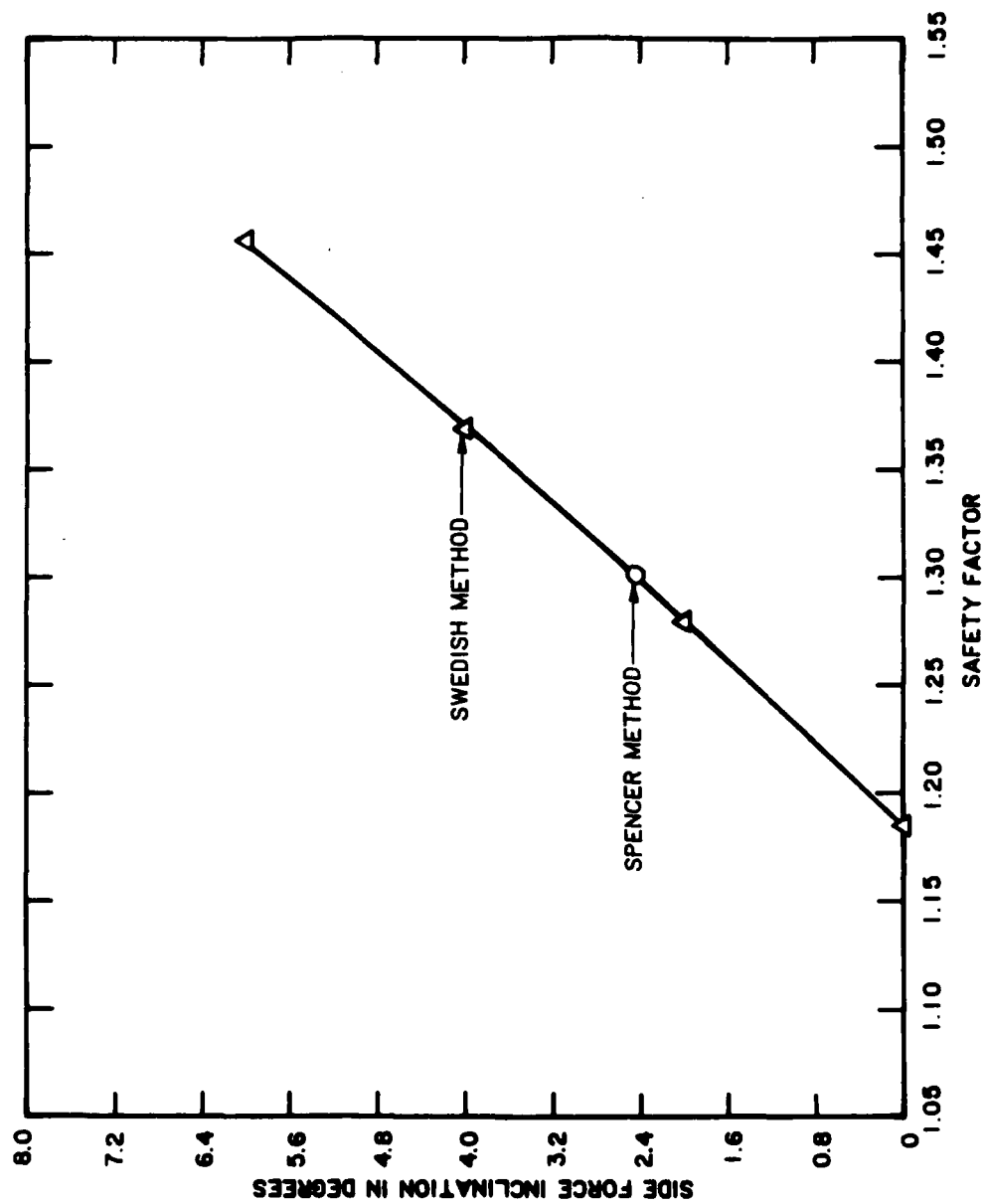


Figure 30. Side force inclination for recommended west dike section versus safety factor

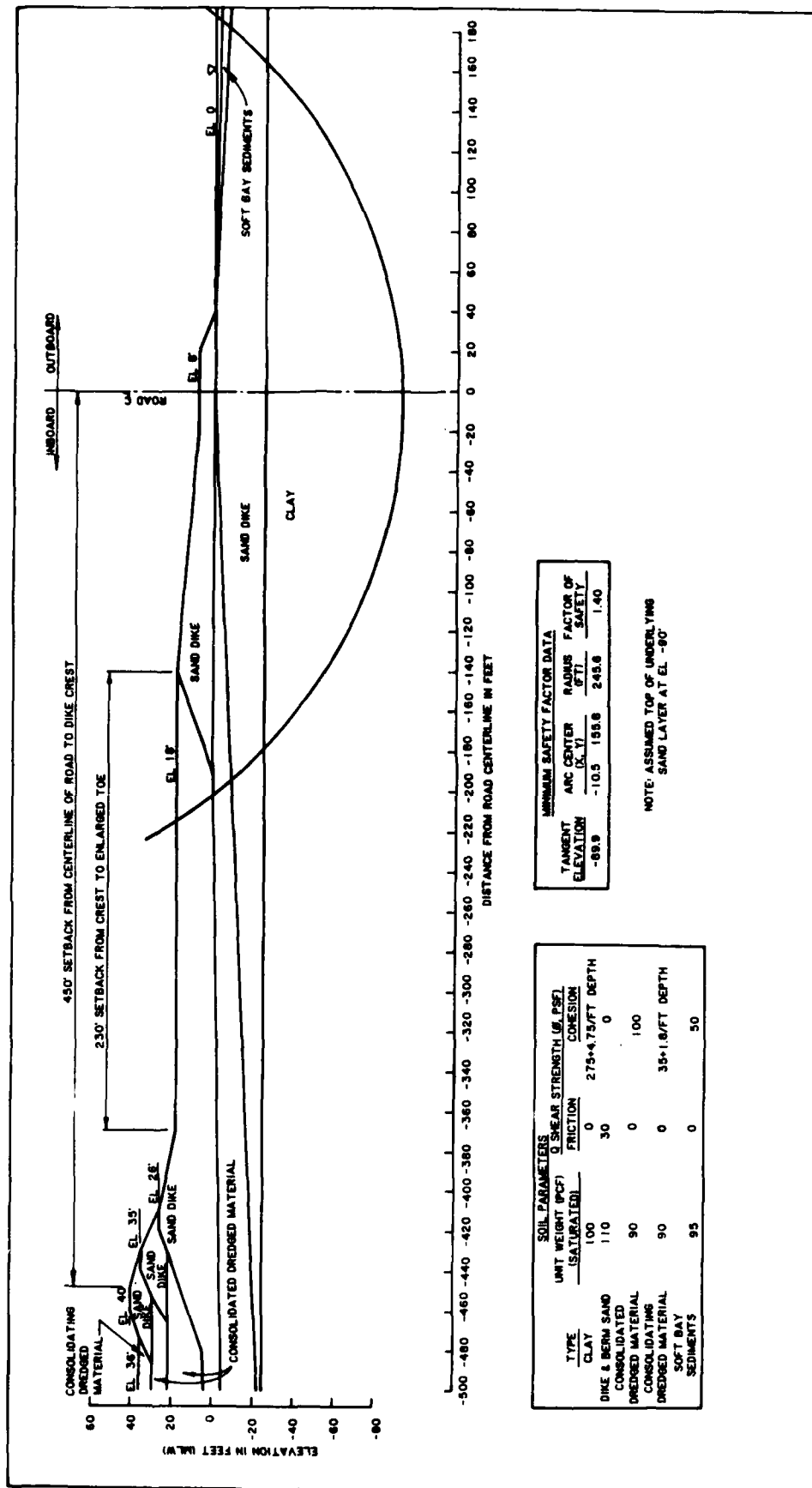


Figure 31. Recommended section for full setback of raised east dike

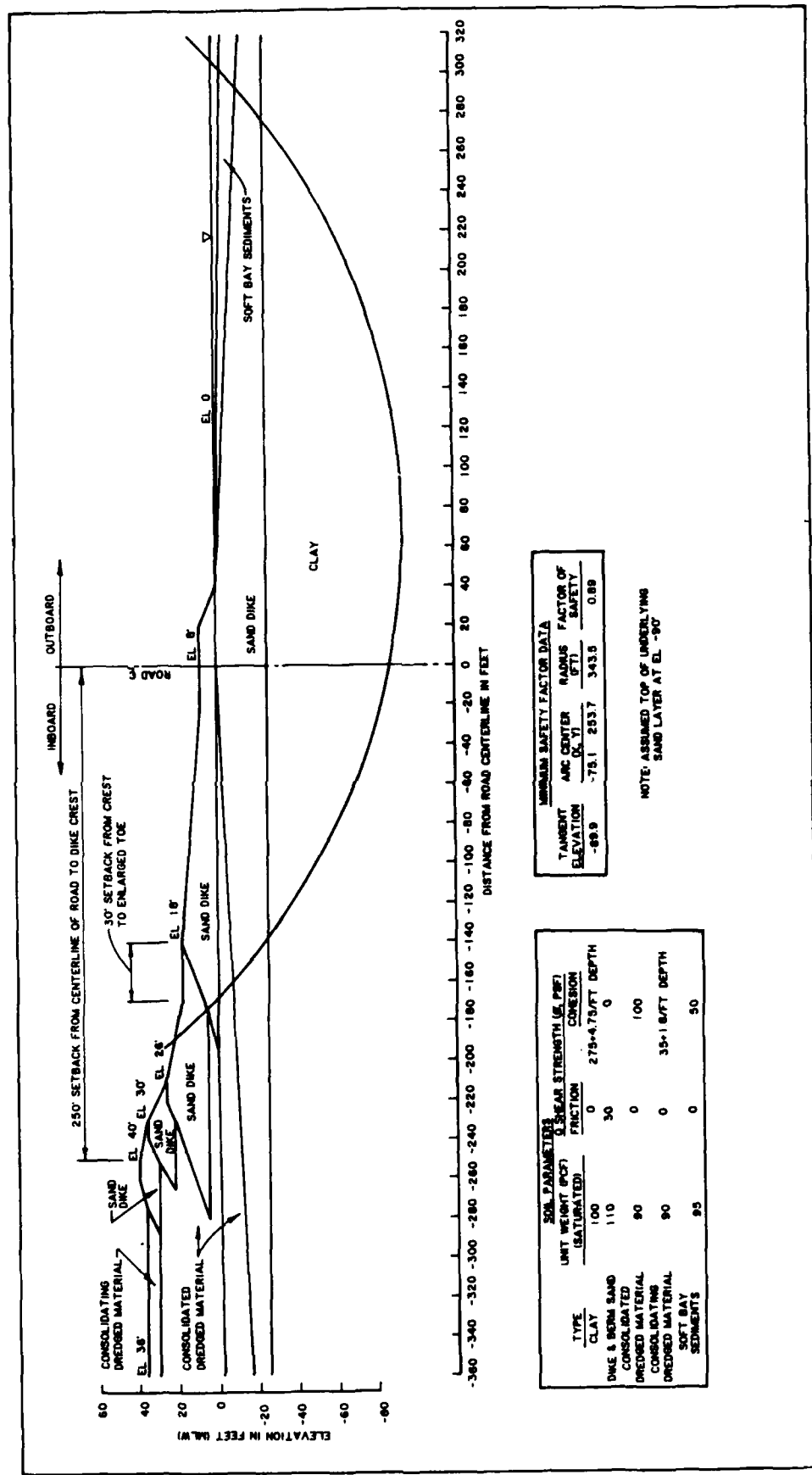


Figure 32. Critical circle for partial setback of raised east dike

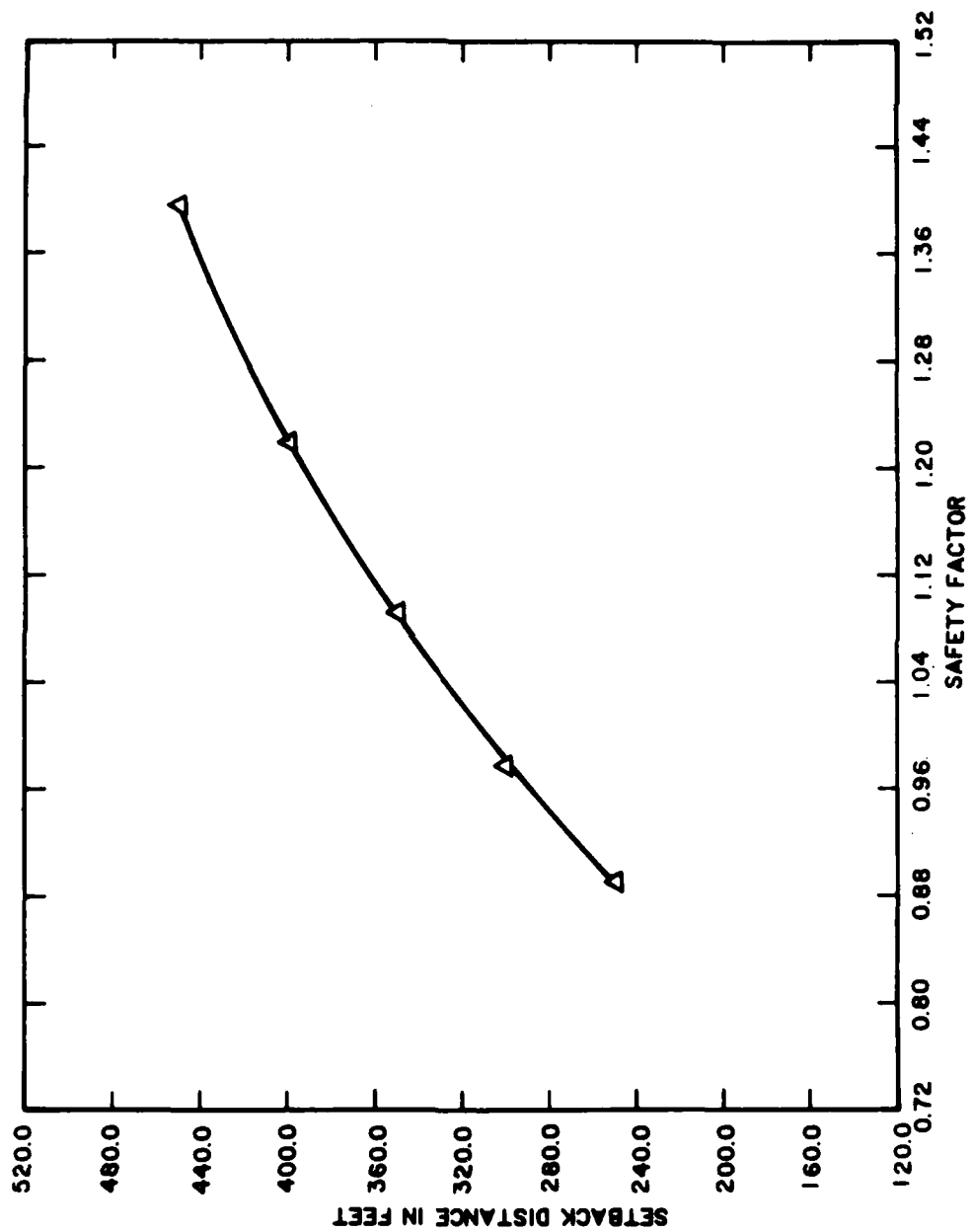


Figure 33. East dike setback distance versus safety factor

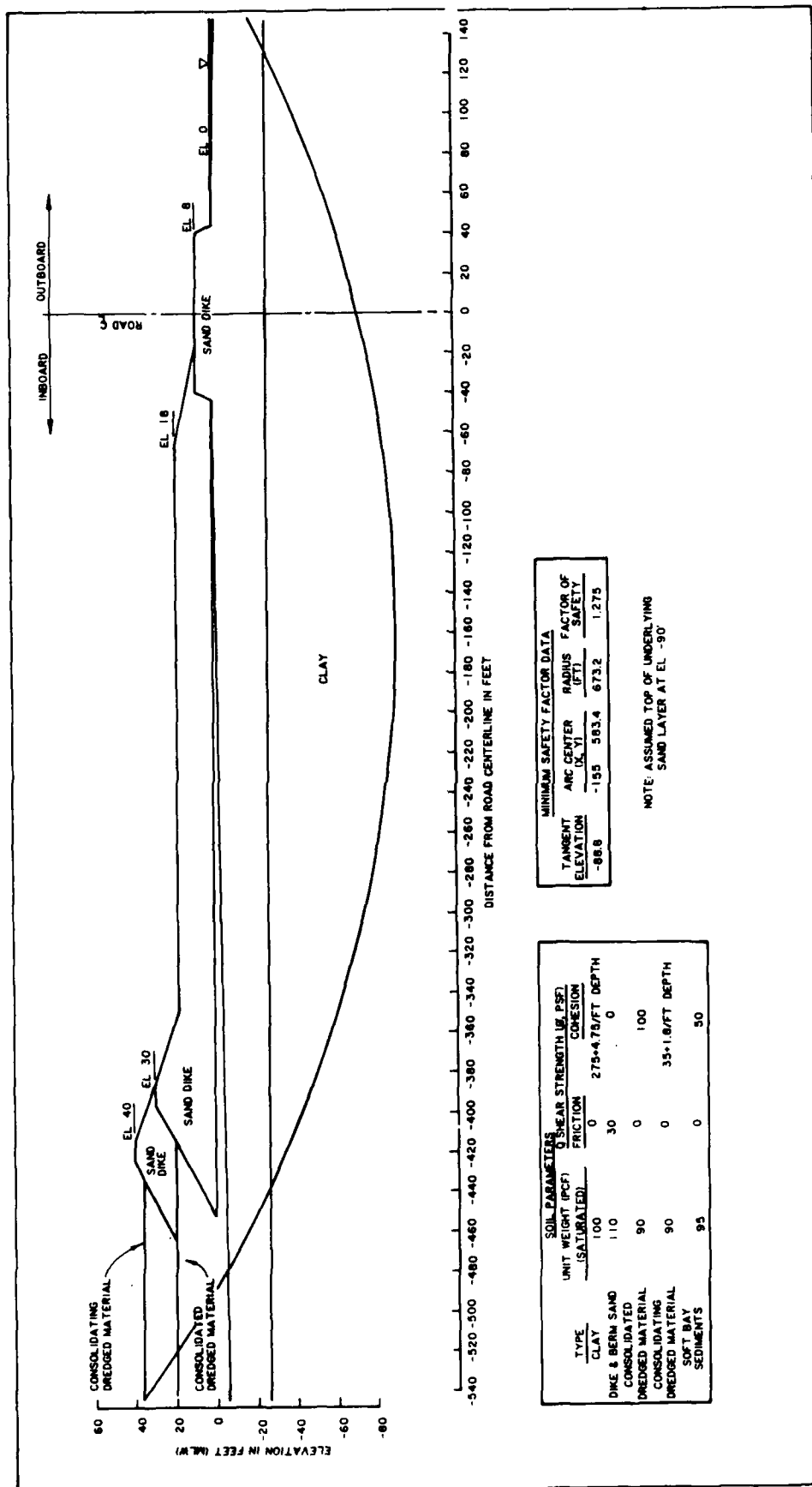


Figure 34. Recommended section for full setback of raised north dike

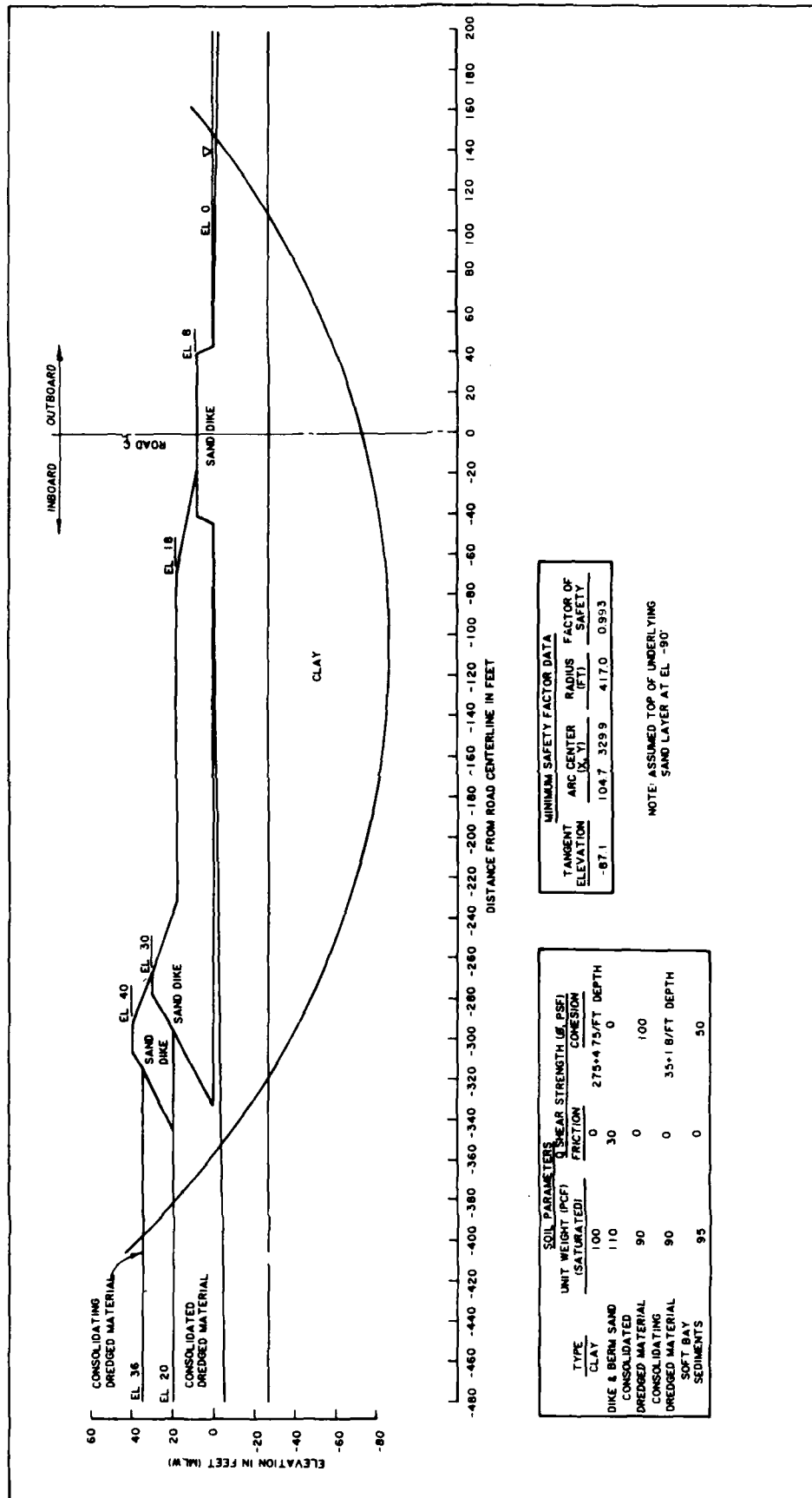


Figure 35. Critical circle for partial setback of raised north dike

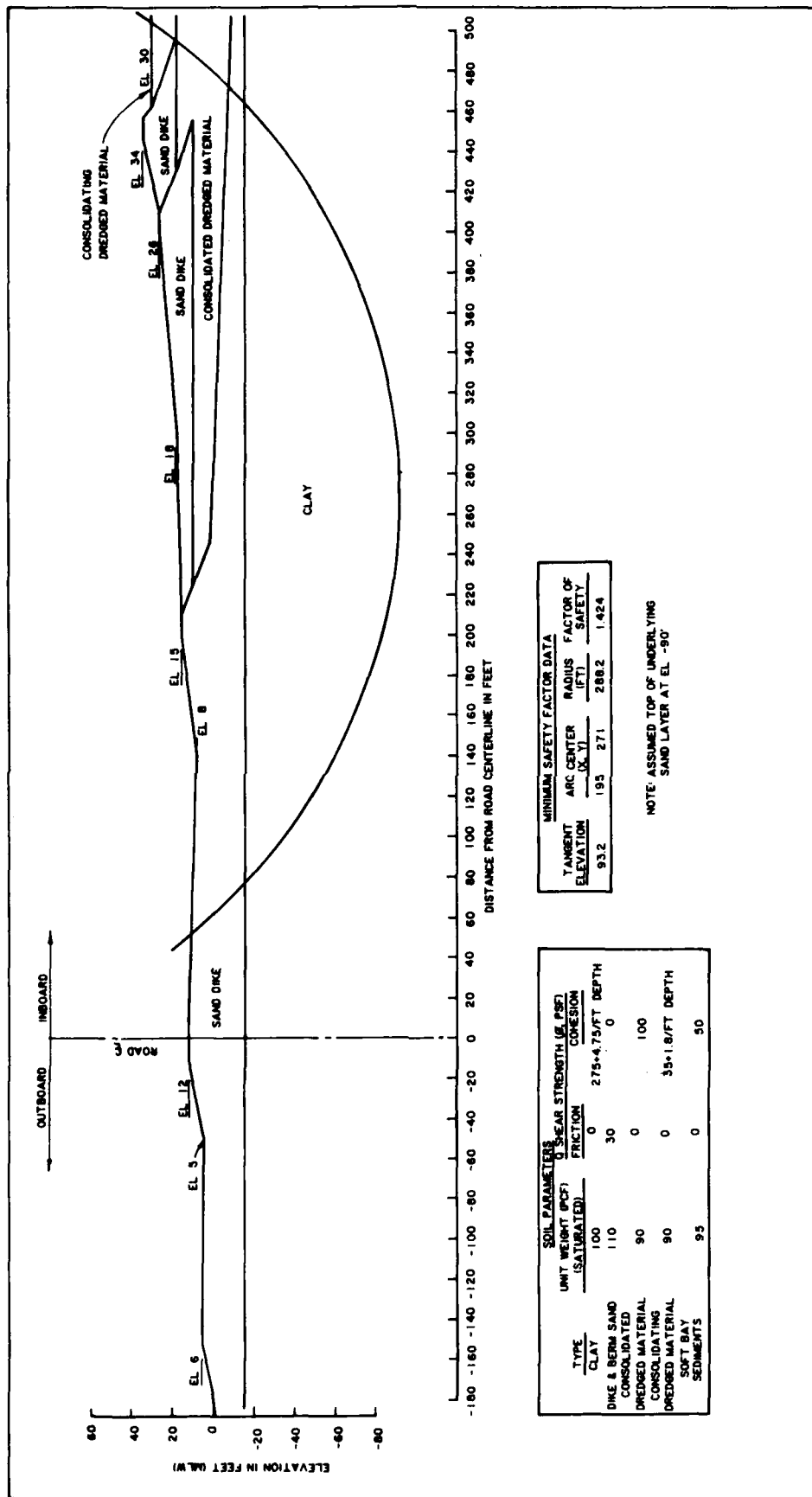


Figure 36. Recommended section for raised northwest corner dike

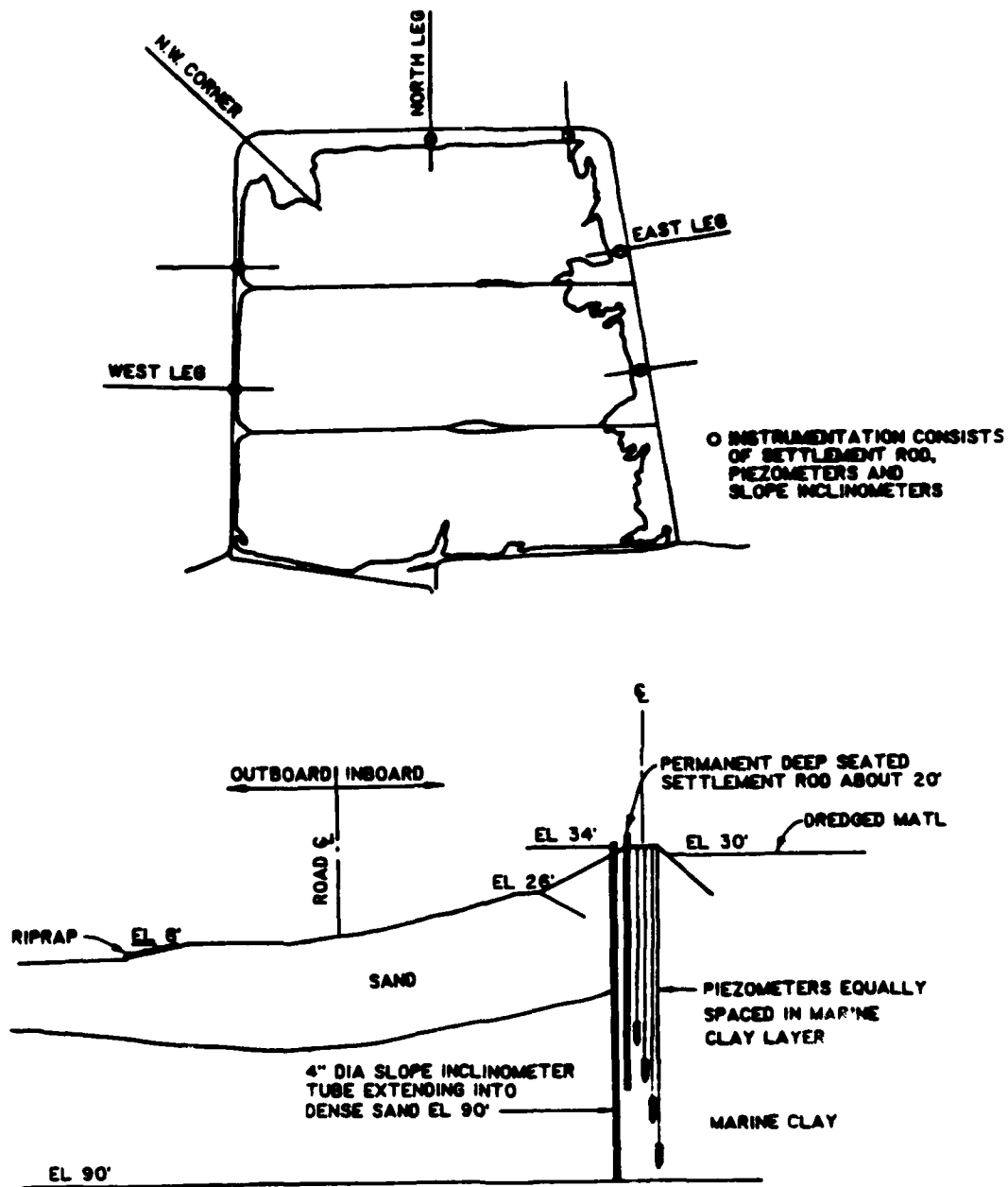


Figure 37. Proposed location for instrumentation

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8-87

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