DEAD SHIP ANCHORAGE

COASTAL ENGINEERING INVESTIGATION

RECONNAISSANCE STUDY

DRAFT REPORT

MAY 14, 2002

MOFFATT & NICHOL
ENGINEERS

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1. INTRODUCTION

1.1 Purpose and Objectives

The purpose of this Coastal Engineering Investigation report (CEI) is to present a preliminary coastal engineering analysis of three options for confined disposal of dredged material at Dead Ship Anchorage, located within Curtis Bay north of Marley Neck. This preliminary assessment includes a literature search of coastal data and a review of environmental, geotechnical and dredging engineering studies conducted for the site.

The objectives of this study include:

- Analysis of site bathymetry, water levels and wind conditions
- Hindcasting of offshore and nearshore waves at the project site
- Determination of dike design parameters

1.2 Project Scope

The scope of this project consists of preparing preliminary reconnaissance studies on the potential for Dead Ship Anchorage to be used as a site for confined disposal of dredged material. This study includes use of available data on bathymetry, water levels, and wind conditions to hindcast waves for the site. Wave conditions are used to prepare the conceptual design for a dike that would be used to contain dredged material. The design parameters evaluated for this study include: alignment location, crest height, structure slope and armor stone size.

1.3 Project Description

This project provides information as to the feasibility of the concept developed for Dead Ship Anchorage for confined disposal of dredged material and whether further evaluation of any of the concepts is warranted. Included in this report are relevant bathymetric, wind, water level and geotechnical data for evaluation of wave height and dike construction requirements.

Waves are hindcast based upon wind data and results from previous studies of storm-induced water levels in the Chesapeake Bay. Offshore and nearshore waves are hindcast for the appropriate winds along the proposed dike alignment. Based upon the hindcast waves, the dike crest height and armor stone size are designed. This report presents the proposed dike alignment and a typical cross-section.

2. SITE CONDITIONS

2.1 General

Dead Ship Anchorage is being studied as a potential site for confined disposal of dredged material. Its ultimate use would be for upland habitat creation. The site is located within Curtis Bay, north of Marley Neck. Dead Ship Anchorage is located in Baltimore City, one and one-half miles north of the southern corner of the city, at approximately 39° 13' N latitude and 76° 34' W latitude (Maryland State Plane Coordinates N 565,000 E 1,440,000) as shown in Figure 2-1.

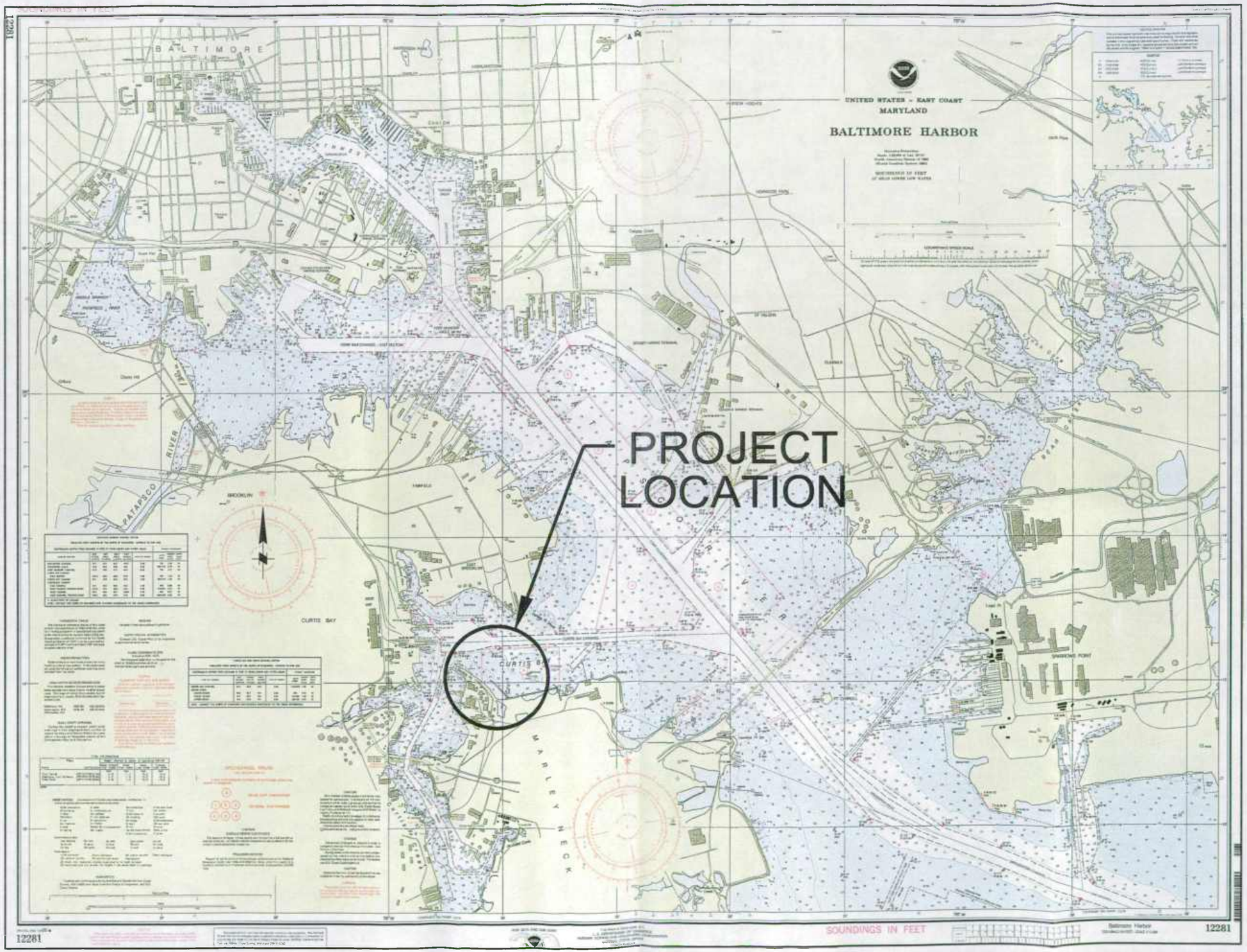
Site conditions germane to project design include bathymetry and topography, wind conditions, water levels, wave conditions, tidal currents, and site soil characteristics. A discussion of each of these factors is presented in the following paragraphs.

2.2 Bathymetry and Topography

Hydrographic data were obtained from NOAA charts 12278 and 12281. Vertical and horizontal data are referenced to Mean Lower Low Water (MLLW) based on the 1960 to 1978 tidal epoch, and the Maryland State Plane, North American Datum of 1983. The proposed dike alignment and existing bathymetry in the area of Dead Ship Anchorage are presented in Figure 2-2. Water depths within the site area vary between –2 to –17 ft MLLW. The maximum water depth where the exposed dike would be constructed is about –17 ft MLLW. Water depths in Curtis Bay Channel, approximately 500 feet north of the study area, are approximately –50 ft MLLW.

2.3 Wind Conditions

Annual extreme windspeed data from the National Oceanic and Atmospheric Administration, National Climatic Data Center (NOAA, NCDC) for Baltimore-Washington International (BWI) Airport, for the period 1951 through 1982, were used in estimating wind conditions for this study (NOS 1982 and NCDC 1994). The BWI data are presented in Table 2-1 as fastest mile winds which are defined as the highest recorded wind speeds that last long enough to travel one mile during a 24 hour recording period. For example, a fastest mile wind speed of 60 miles per hour would have a duration of 60 seconds, a fastest mile wind speed of 50 miles per hour would have a duration of 72 seconds, etc. The wind data presented in Table 2-1 were used to develop



0 4000 8000 12000 FEET

FIGURE 2-1
PROJECT LOCATION

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windspeed-return period relationships based on a Type I (Gumbel) distribution for eight directions, namely: north (N), northeast (NE), east (E), southeast (SE), south (S), southwest (SW), west (W) and northwest (NW). Return period is defined as the average time between wind events which equal or exceed a given value. The specific return periods examined were 5, 10, 15, 20, 25, 30, 35, 40, 50 and 100 years.

Table 2-1
Annual Extreme Wind Speed Per Direction
for Baltimore-Washington International (BWI) Airport, 1951-1982
Fastest Mile Wind Speed (mph)

Year	North	Northeast	East	Southeast	South	Southwest	West	Northwest	All Directions
1951	24	41	27	34	39	29	42	46	46
1952	66	25	47	66	41	66	46	43	66
1953	20	28	22	27	34	39	47	43	47
1954	31	27	22	60	28	39	57	44	60
1955	21	43	29	28	43	53	40	43	53
1956	29	34	25	24	28	34	56	40	56
1957	29	53	35	33	33	30	46	46	53
1958	30	52	25	33	37	43	40	43	52
1959	28	26	20	27	23	38	46	43	46
1960	26	38	28	27	25	35	40	53	53
1961	45	28	28	29	24	70	41	54	70
1962	56	41	28	17	25	36	42	61	61
1963	38	32	18	34	25	28	44	60	60
1964	34	31	23	24	47	23	48	61	61
1965	36	26	28	34	36	54	44	44	54
1966	32	25	29	24	47	43	50	48	50
1967	30	29	25	39	27	46	53	43	53
1968	45	30	36	26	19	45	48	50	50
1969	28	21	20	34	26	45	45	53	53
1970	28	28	18	21	39	34	48	60	60
1971	31	45	26	18	21	41	39	58	58
1972	28	25	35	26	20	41	41	41	41
1973	40	26	26	38	26	35	49	33	49
1974	32	23	46	29	33	33	45	41	46
1975	40	26	21	24	25	38	54	45	54
1976	31	18	20	28	32	28	45	54	54
1977	32	31	19	28	26	25	49	48	49
1978	39	28	36	28	19	52	33	45	52
1979	32	25	27	36	32	32	45	47	47
1980	33	27	18	32	20	32	45	50	50
1981	24	24	19	26	23	28	41	42	42
1982	31	20	23	23	29	34	40	48	48

Note: Data adjusted to 10 meter height.

A review of the wind speed data indicate that during the 32-year period from 1951 through 1982, six wind events exceeded 60 miles per hour (mph). In order to quantify the frequency of various wind events, statistical analyses of the wind data were performed. These analyses consisted of fitting external statistical distributions through the annual extreme windspeeds for each of the wind directions. The wind statistics for each direction (design wind speeds) are presented in Table 2-2 in terms of fastest mile windspeeds for various return periods. For example Table 2-2 shows that the design windspeeds for a 25-year return period storm range from 47 mph for the east direction to 70 mph for the southwest direction. The design windspeeds presented in Table 2-2 have been used to estimate design wave conditions for the proposed project site in Section 2.5 of this report.

Table 2-2
Design Wind Speeds per Direction and Return Period (mph)

Return Period	Direction							
	N	NE	E	SE	S	SW	W	NW
5	40	37	32	37	36	47	50	54
10	48	44	38	45	43	56	54	59
15	52	48	41	50	47	61	56	62
20	56	52	45	55	51	67	59	65
25	59	55	47	58	54	70	60	67
30	62	57	49	61	56	73	61	68
35	64	60	51	63	58	76	62	70
40	66	62	53	65	60	78	63	71
50	69	66	55	69	63	82	64	73
100	81	76	65	82	74	97	69	81

2.4 Water Levels

Normal water level variations in the upper Chesapeake Bay are generally dominated by astronomical tides, although wind effects and freshwater discharge can be important. Extreme water levels, on the other hand, are dictated by storm tides.

2.4.1 Astronomical Tides

Astronomical tides in the upper Chesapeake Bay are semi-diurnal. The mean tide level is 0.8 ft above MLLW; the mean tidal range is 1.1 ft and the spring tidal range is 1.7 ft (NOS 1997).

Tidal datum near the study area reported from National Ocean Service are presented in Table 2-3. The difference in elevation between MLLW and National Geodetic Vertical Datum (NGVD) is approximately 0.28 ft for Fort McHenry. MLLW will serve as the datum for this project.

Table 2-3
Astronomical Tidal Datum Characteristics for Baltimore (Fort McHenry) (ft, MLLW)

Tidal Datum	Baltimore
Mean Higher High Water (MHHW)	1.66
Mean High Water (MHW)	1.35
Mean Tide Level (MTL)	0.80
National Geodetic Vertical Datum (NGVD)	0.28
Mean Low Water (MLW)	0.24
Mean Lower Low Water (MLLW)	0.00

2.4.2 Storm Surge

Design water levels for the study site area are dominated by storm effects (i.e. storm surge and wave setup) in combination with astronomical tides. Storm surge is a temporary rise in water level generated either by large-scale extra-tropical storms, known as northeasters, or by hurricanes. The rise in water level results from wind action, the low pressure of the storm disturbance and the Coriolis force. Wave setup is a term used to describe the rise in water level due to wave breaking. Specifically, wave setup is a change in momentum which attends the breaking of waves propagating towards shore results in a surf zone force that raises water levels at the shoreline. A comprehensive evaluation of storm-induced water levels for several Chesapeake Bay locations has been conducted by the Virginia Institute of Marine Science (1978) as part of the Federal Flood Insurance Program. Results of this study, summarized in Table 2-4, were used to generate the water-level vs. return period curve presented in Figure 2-3, which provides water levels in feet above MLLW for various return periods. Data in Figure 2-3 are for the closest station location for the project site, which is Baltimore (Fort McHenry). The Fort McHenry station in Baltimore is located at 39° 16' north latitude and 76° 34.7' west longitude, about 3.1 miles north of Dead Ship Anchorage. Figure 2-3 indicates that the storm tide elevation

for a 25-year return period at Baltimore is 5.4 ft MLLW and the 100-year water level is 8.4 ft MLLW.

Table 2-4
Storm Induced Water Level
(ft, NGVD)

Return Period	Water Level
10	4.1
50	6.8
100	8.1
500	10.7

2.5 Wave Conditions

Dead Ship Anchorage is sheltered from wind-generated waves from the south, southeast and southwest. The proposed alignment is exposed to wind generated waves from the west, northwest, north and east. The greatest fetch distances, to which the site is exposed, are from the northeast and east directions (Figure 2-4). Wind-generated wave calculations were completed for the west, northwest, north, northeast and east directions.

In accordance with procedures recommended by the U.S. Army Corps of Engineers (USACE), Shore Protection Manual (SPM) (USACE 1984), a radially averaged fetch distance was computed for each direction. The radially averaged fetch distances for the W, NW, N, NE and E directions for the Dead Ship Anchorage site are shown in Figure 2-4. Table 2-5 presents radially averaged fetch distances and mean water depths corresponding to the W, NW, N, NE, and E directions. Wave conditions were hindcast along each fetch direction for the design winds presented in Table 2-2 (adjusted appropriately for duration) and the water levels presented in Figure 2-3. Specifically, waves were hindcast for the five directional design windspeeds (i.e. the design windspeeds computed for each individual direction) using methods published in the SPM (USACE 1984). Wave hindcast results are presented in Tables 2-6 and 2-7. The significant wave height (H_s) is shown in Table 2-6 and the peak spectral wave period (T_p) is shown in Table 2-7. Polar plots for H_s and T_p are shown in Figures 2-5 and 2-6, respectively. These figures present a summary of H_s and T_p that graphically show the directions from which the highest waves and longest periods approach the site.

Table 2-5
Mean Radial Fetch Distances and Mean Water Depth for Radially-Averaged Fetch Used for Wave Hindcasting – Dead Ship Anchorage

Direction	Mean Fetch Distance (Miles)	Mean Water Depth (ft, MLLW)
West	0.4	14.6
Northwest	0.7	27.5
North	0.6	23.9
Northeast	2.6	26.3
East	2.5	22.6

Table 2-6
Offshore Significant Wave Heights (ft)
Dead Ship Anchorage

Return Period (years)	West	Northwest	North	Northeast	East
5	1.2	1.6	1.1	1.9	1.6
10	1.3	1.8	1.3	2.3	1.9
15	1.4	1.9	1.4	2.6	2.1
20	1.5	2.0	1.6	2.8	2.3
25	1.5	2.1	1.7	3.0	2.4
30	1.5	2.1	1.8	3.1	2.5
35	1.5	2.2	1.8	3.3	2.7
40	1.6	2.2	1.9	3.4	2.8
50	1.6	2.3	2.0	3.6	2.9
100	1.7	2.6	2.4	4.2	3.5

Table 2-7
Peak Spectral Wave Period (sec)
Dead Ship Anchorage

Return Period (years)	West	Northwest	North	Northeast	East
5	1.6	2.0	1.7	2.5	2.3
10	1.7	2.0	1.8	2.7	2.5
15	1.7	2.1	1.8	2.8	2.6
20	1.7	2.1	1.9	2.9	2.7
25	1.8	2.1	1.9	2.9	2.7
30	1.8	2.2	2.0	3.0	2.8
35	1.8	2.2	2.0	3.0	2.8
40	1.8	2.2	2.0	3.1	2.8
50	1.8	2.2	2.1	3.1	2.9
100	1.9	2.3	2.2	3.3	3.1

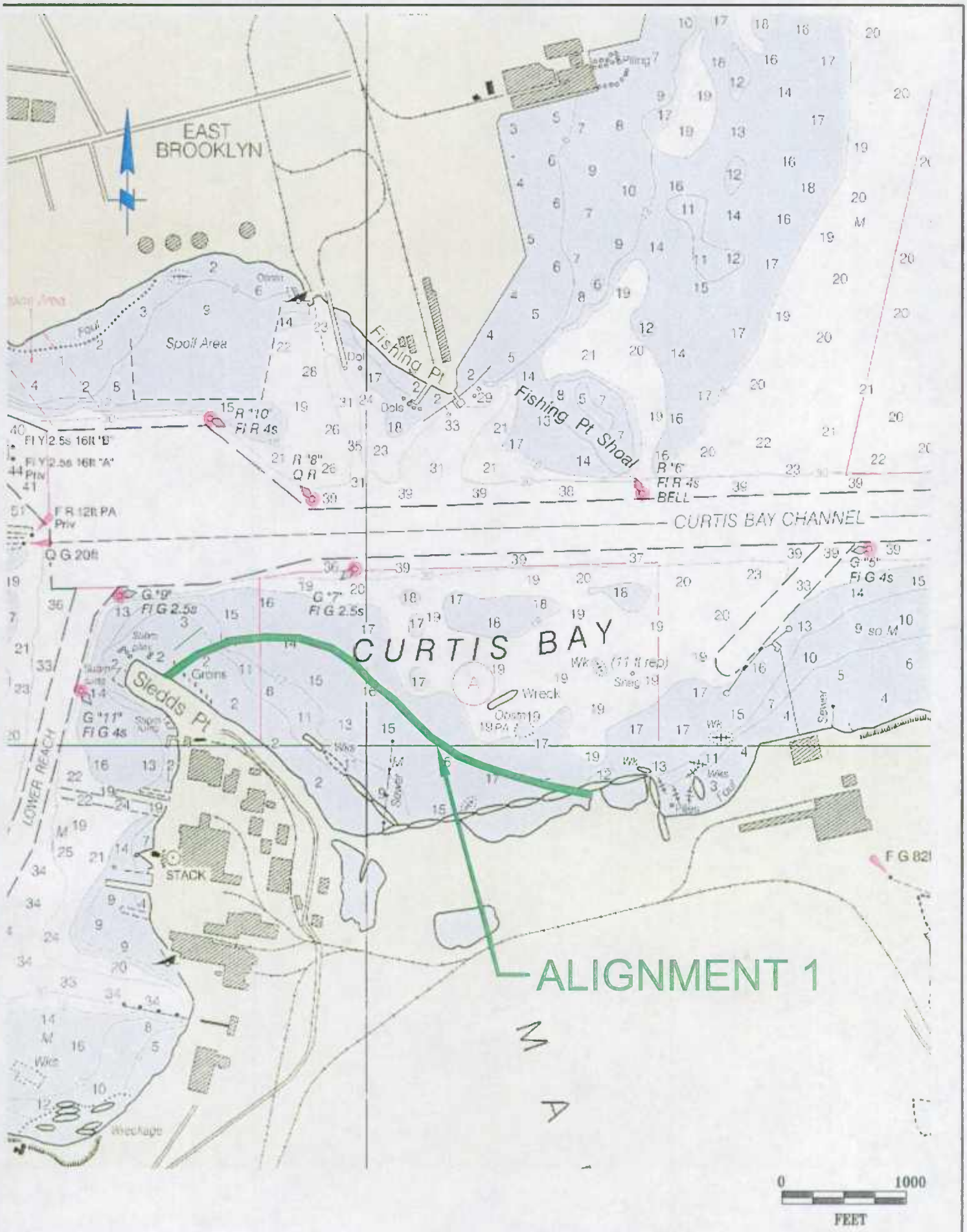

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FIGURE 2-2
POTENTIAL DIKE ALIGNMENT

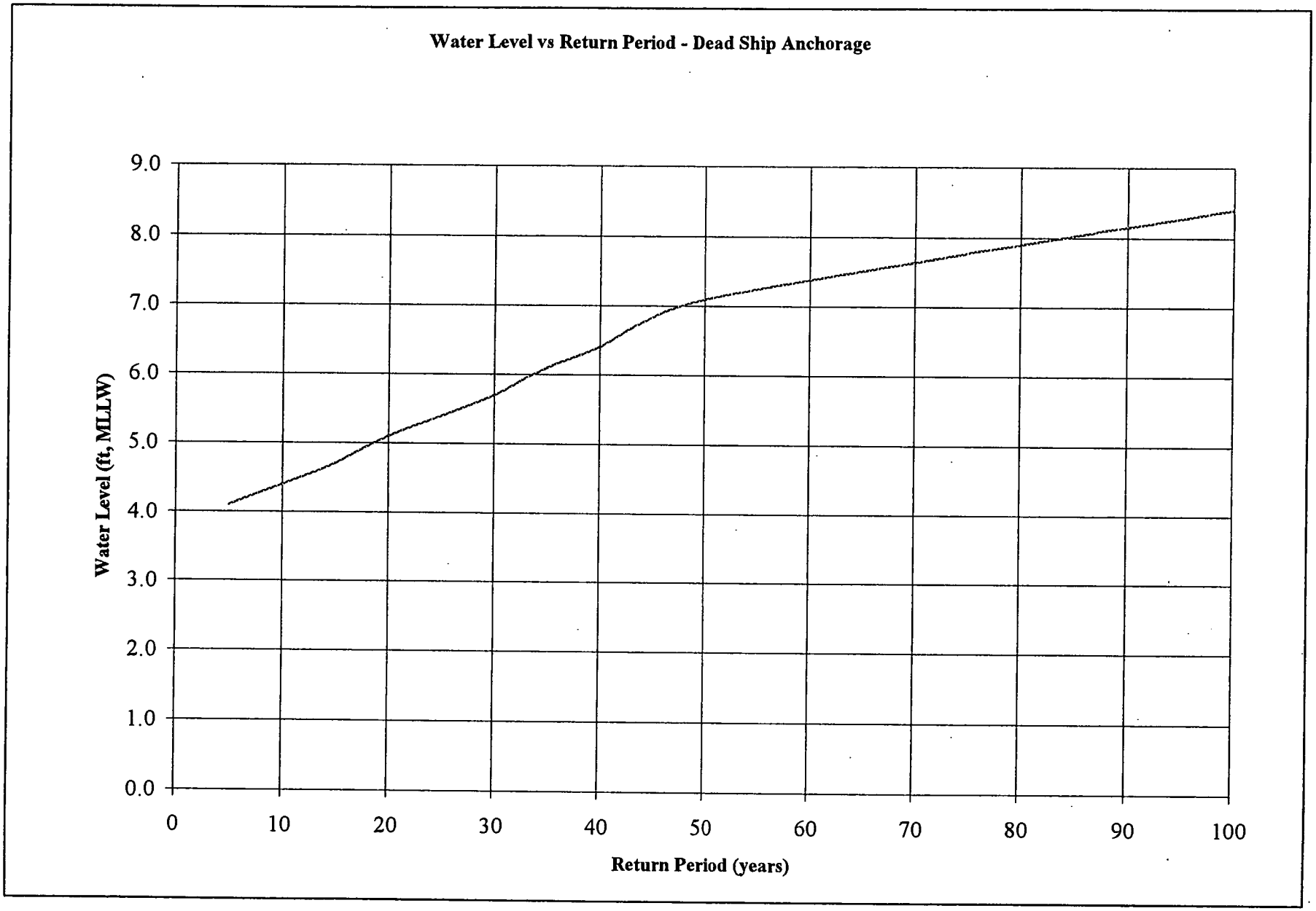
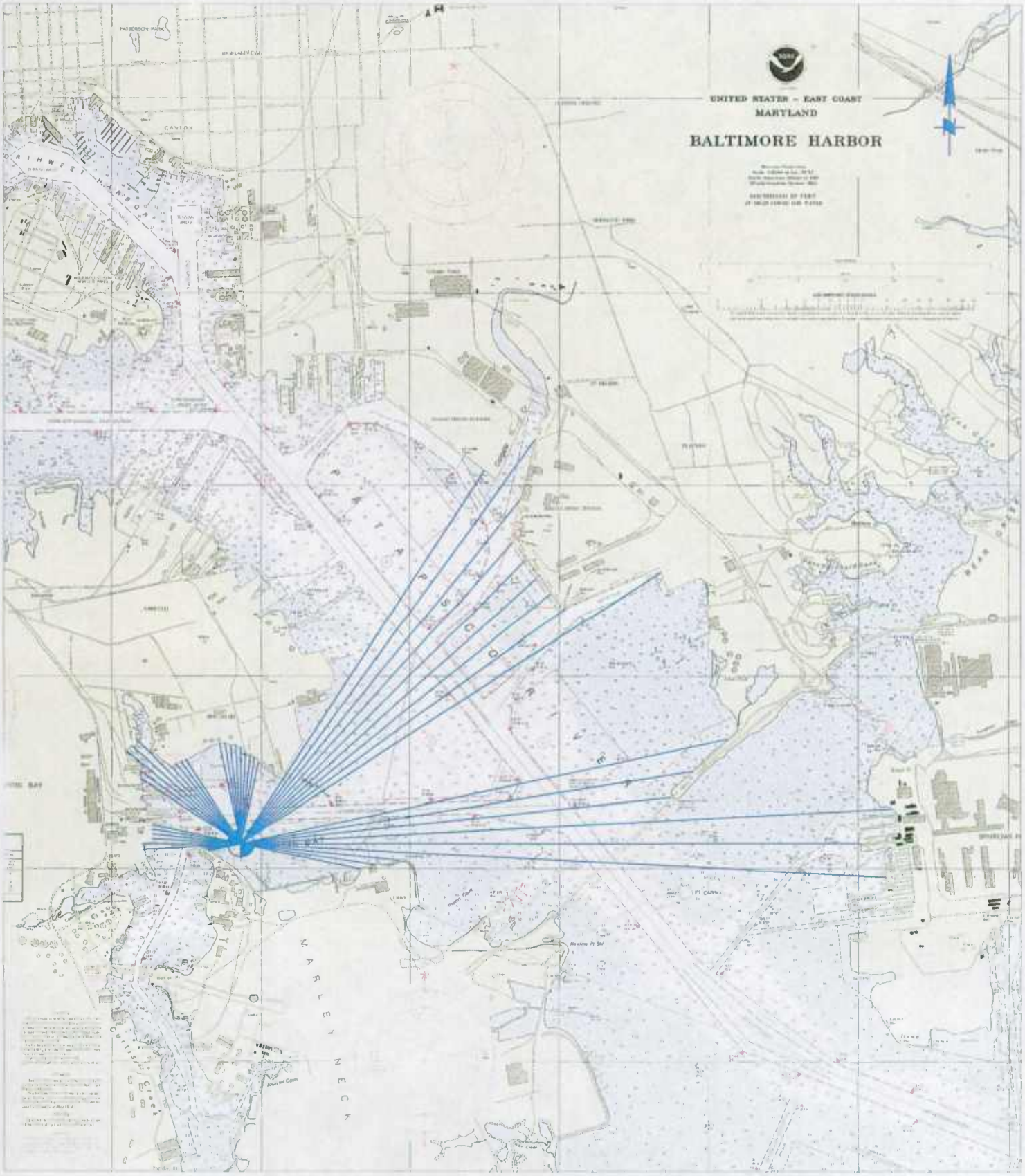


Figure 2-3. Design Water Levels (ft, MLLW) for Dead Ship Anchorage.



0 3000
FEET

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FIGURE 2-4
RADIALLY AVERAGED
FETCH LINES

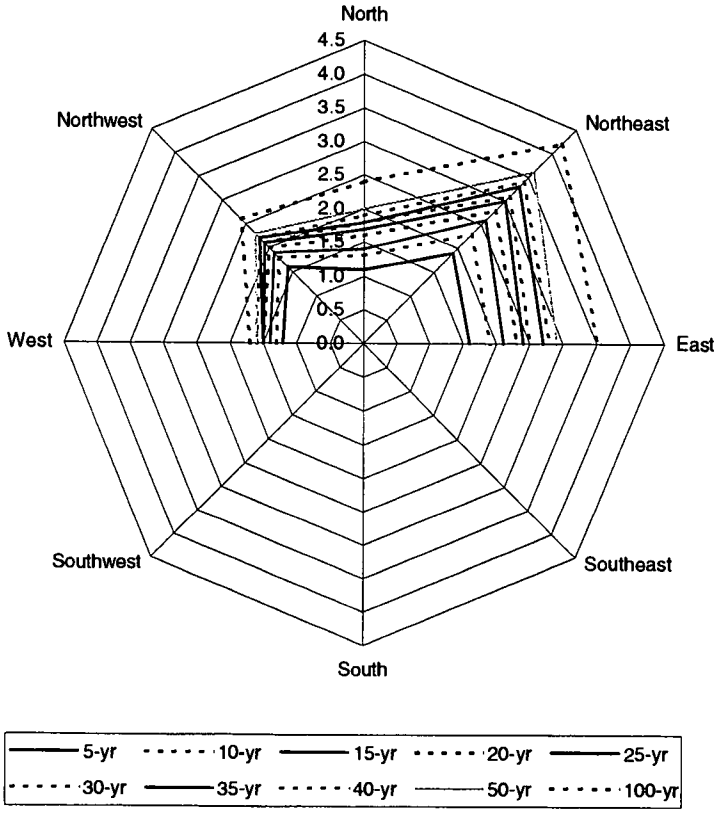

Figure 2-5. Offshore Significant Wave Heights (ft) for Dead Ship Anchorage

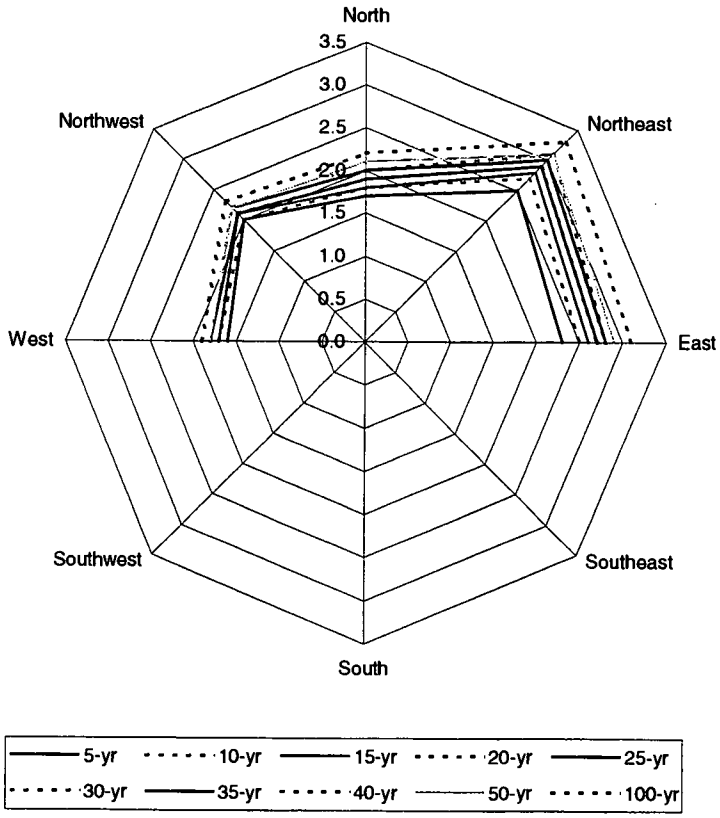

Figure 2-6. Peak Spectral Wave Periods (sec) for Dead Ship Anchorage

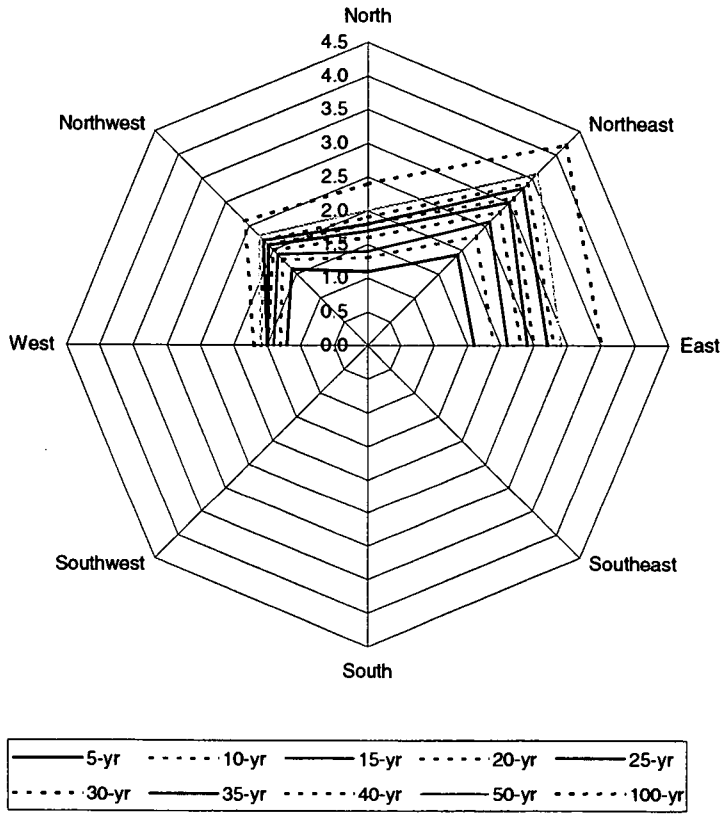

Figure 2-7. Nearshore Significant Wave Heights (ft) for Dead Ship Anchorage.

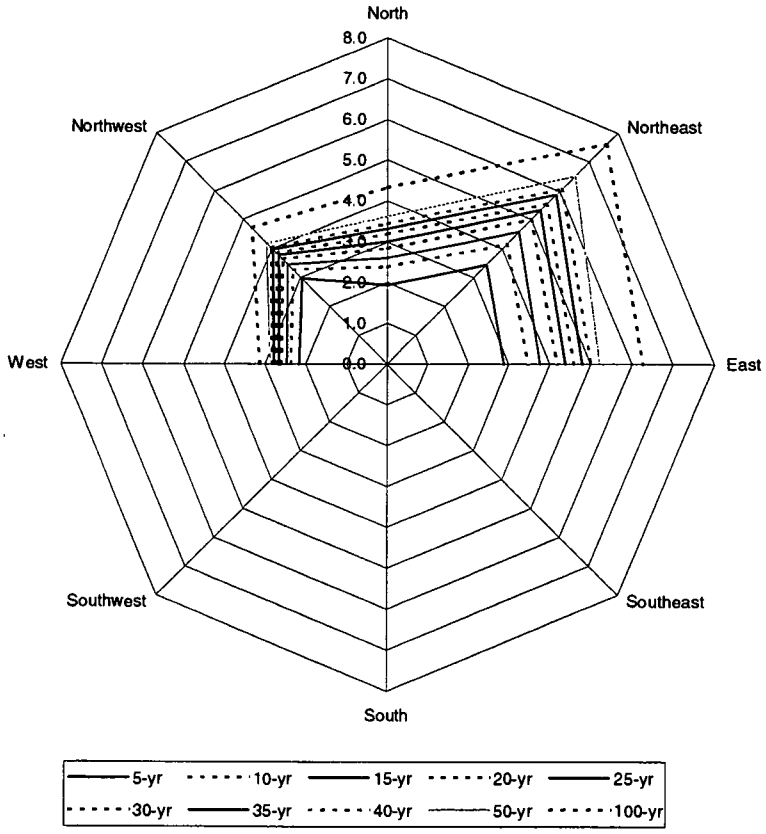

Figure 2-8. Nearshore Maximum Wave Heights (ft) for Dead Ship Anchorage.

A sea state is normally composed of a spectrum of waves with varying heights and periods which may range from relatively long waves to short ripples. In order to summarize the spectral characteristics of a sea state it is customary to represent that wave spectrum in terms of a distribution of wave energy over a range of wave periods. Having made this distribution, known as a wave spectrum, it is convenient to represent the wave spectrum by a single representative wave height and period. The wave conditions reported above are the significant wave height, H_s, and the peak spectral wave period, T_p. The significant wave height, H_s, is defined as the average of the highest one-third of the waves in the spectrum. Depending on the duration of the storm condition represented by the wave spectrum, maximum wave heights may be as high as 1.8 to 2 times the significant wave height. The peak spectral period, T_p, is the wave period, which corresponds to the maximum wave energy level in the wave spectrum.

For Dead Ship Anchorage, the highest waves are estimated to approach from the northeast direction. The 100-year return period wave for this direction has a significant height (H_s) of 4.2 ft and peak spectral period (T_p) of 3.3 seconds. The 35-year return period significant wave height (H_s) is 3.3 ft, and the peak spectral wave period (T_p) is 3.0 seconds.

The random wave analyses of Goda (1985) were used to examine whether the offshore waves would break prior to reaching the dikes. The first step in examining wave conditions for a given bottom elevation and water level is to compute the total water depth from which the maximum breaking wave height can be determined. This breaker depth, h_b, is the sum of the selected water elevation above MLLW and the bottom elevation below MLLW. The maximum breaker height which can be supported in the resulting water depth is computed using the following formulae published in the SPM (USACE 1984):

$$h_b = \frac{H_b}{B_w - A_w \frac{H_b}{gT^2}}$$

$$B_w = \frac{1.56}{(1 + \exp(-19.5m)}$$

$$A_w = 43.75\,(1 - e^{-19m})$$

Where:
H_b = breaking wave height at the outer edge of the surf zone
m = tangent of beach slope
h_b = breaker depth
g = acceleration due to gravity
T = spectral wave period
A_w, B_w = Breaking wave height parameters (w referring to wave)

The solution to the above equation will provide an estimate of the maximum breaker height to which the structure is subjected for a given total water depth. Goda's analyses require the estimate of an equivalent offshore significant wave height (also referred to as the equivalent unrefracted wave height) which is computed from the maximum breaking wave height and the linear shoaling coefficient in accordance with the following equations:

$$H_s \approx \frac{H_b}{1.8}$$

$$H_o' = \frac{H_s}{K_s}$$

$$K_s = \sqrt{\frac{1}{\tanh\left(2\pi \frac{h_b}{L}\right)\left[1 + \frac{\frac{4\pi h_b}{L}}{\sinh\left(4\pi \frac{h_b}{L}\right)}\right]}}$$

$$L = \frac{gT^2}{2\pi} \tan h\ (2\pi \frac{h_b}{L})$$

Where:
H_s = approximate significant wave height at breaking
H_O' = equivalent unrefracted deepwater significant wave height
K_s = shoaling coefficient
L = local wave length

The H_{max} values are computed using the following equations published by Goda (1985):

$$H_{max} = 0.8 k_s H_O \quad for \frac{h}{L_O} \geq 0.2$$

and

$$H_{max} = MIN\,[(\beta_o * H_o' + \beta_1 * h), \beta_{max} * H_o', 1.8 K_S H_o'] \quad for \frac{h}{L_o} < 0.2$$

Where:

h = water depth

L_O = deepwater wave length

$$\beta_o * = 0.052 \left(\frac{H_o'}{L_o}\right)^{-0.38} e^{20.0m^{1.5}}$$

$$\beta_{max} * = MAX\,(1.65, 0.53 \left(\frac{H_o'}{L_o}\right)^{-0.29} e^{2.4m}$$

$$\beta_1 * = 0.63 e^{3.8m}$$

Similar equations are available for computing H_s, and the results are used to compute the nearshore significant and maximum wave heights. The results of the analysis show that the proposed dike alignment will be located in water having depths too large to allow for breaking of the waves prior to reaching the structure. Table 2-8 and Figure 2-7 present the nearshore significant wave heights at Dead Ship Anchorage for the proposed Alignment. Table 2-9 and Figure 2-8 present the nearshore maximum wave heights at Dead Ship Anchorage. Comparison of Tables 2-6 and 2-8 shows that the nearshore significant wave heights at Dead Ship Anchorage are the same as the offshore significant wave height. The nearshore significant wave heights from the northeast are 3.3 and 4.2 ft for a 35-year storm and 100-year storm, respectively. Table 2-9 indicates that for Dead Ship Anchorage, the maximum waves from the northeast are 5.9 and 7.6 ft, for the 35-year and 100-year storms, respectively.

Table 2-8
Nearshore Significant Wave Heights (ft)
Dead Ship Anchorage

Return Period (years)	West	Northwest	North	Northeast	East
5	1.2	1.6	1.1	1.9	1.6
10	1.3	1.8	1.3	2.3	1.9
15	1.4	1.9	1.4	2.6	2.1
20	1.5	2.0	1.6	2.8	2.3
25	1.5	2.1	1.7	3.0	2.4
30	1.5	2.1	1.8	3.1	2.5
35	1.5	2.2	1.8	3.3	2.7
40	1.6	2.2	1.9	3.4	2.8
50	1.6	2.3	2.0	3.6	2.9
100	1.7	2.6	2.4	4.2	3.5

Table 2-9
Nearshore Maximum Wave Heights (ft)
Dead Ship Anchorage

Return Period (years)	West	Northwest	North	Northeast	East
5	2.2	3.0	1.9	3.5	2.9
10	2.4	3.3	2.4	4.2	3.5
15	2.5	3.5	2.6	4.6	3.8
20	2.6	3.7	2.8	5.0	4.2
25	2.7	3.8	3.0	5.3	4.4
30	2.7	3.9	3.2	5.6	4.6
35	2.8	4.0	3.3	5.9	4.8
40	2.8	4.0	3.4	6.1	5.0
50	2.9	4.2	3.6	6.5	5.2
100	3.1	4.7	4.3	7.6	6.3

2.6 Currents

Tidal current velocities in Curtis Bay are typically weak and variable (NOS 1996). Currents are not considered to be important for shore protection design at this project site.

2.7 Soil Characteristics

An evaluation of the soil characteristics at the project site was performed by Engineering Consultation Construction Remediation, Inc. (E2CR 2002). The evaluation included performing soil borings, preparing soil boring profiles, identifying soil strata thickness, location and characteristics, and conducting a preliminary slope stability analysis. Results of the preliminary study indicate that the underlying soil is soft clays that would require strengthening prior to dike construction.

3. DIKE CONSTRUCTION

3.1 Introduction

The principal components of a coastal protection dike include:

- Toe Protection
- Protective Revetment
- Berm (if included)
- Upper Slope
- Crest Area and Roadway
- Dike Core

Toe protection is normally an integral part of the revetment structure and is designed to prevent that structural component from undermining as a result of wave and/or current-induced scour. The protective revetment serves to hold the dike core in place and is often comprised of several layers of rock armoring. A berm may or may not be included in the dike cross section. Where included, a berm can be used to limit wave runup and overtopping. The berm can also be used to minimize the armoring requirements for the revetment and upper slope of the dike. Roadways are often included on dikes in order to provide access for operations and repairs to the dikes.

The dike geometry used for this preliminary study is comprised of toe protection, a rubble mound revetment (i.e. the side slope), a horizontal crest with a crushed stone roadway and a core constructed of sand. One of the more important variables of the dike design is the side slope which, together with the crest height, is generally dictated by soil conditions and dike construction methodologies. Based on the analyses performed for prior projects, and the geotechnical analysis performed for this project, the dike design has been determined to have an outer slope of three horizontal to one vertical (3:1) and an inner side slope of five horizontal to one vertical (5:1).

3.2 Dike Design Life

The design life selected for the containment dikes is an important factor in the overall planning. It should be noted that project life for dike design is different than the life capacity of the site for storing dredged material. The former pertains to the life expectancy and costs of the containment dikes and is treated in this section of the report whereas the latter pertains to the period of time it takes to fill the dredged material placement site.

Previously, USACE would stipulate a project life of 50 years (ER-1110-2-1407 "Hydraulic Design of Coastal Shore Protection Projects"). This has now been superseded by the revised ER-1110-2-1407 (November 1990) which dictates that a fuller range of alternatives be studied to account for differences in cost of repair, periodic replacements and rehabilitation. The 50-year project life is consistent with the nature of routine coastal and hydraulic engineering projects, which are designed to protect large areas of rural and urban infrastructure against flooding and/or wave-induced damages. Furthermore, such projects are normally justified on the basis of a rigorous and codified economic analysis, which assures that the project benefits exceed project costs. The most rational means for selecting the project design life at Dead Ship Anchorage, however, is on the basis of economics (i.e. project costs and cost effectiveness). This approach was used for the design of the Poplar Island Restoration Project dikes (GBA-M&N JV 1995), and has been used in selecting the design return periods for this project.

3.3 Dike Design Values

The dikes must be designed for a given level of hydrodynamic design conditions including winds, waves, water levels, and currents. Design conditions can be stipulated in terms of levels of risk and/or in terms of statistical return period. These two factors are related to one another and the project life through the following formula:

$$R = 1 - (1 - \frac{1}{RP})^{L}$$

Where: R = risk or probability that a given condition will be equaled or exceeded

L = project life in years

RP = return period in years

The previous USACE criteria stipulate that a project should be designed for an event that has a 50% risk during a 50 year project life. Manipulation of the above formula will show that these criteria correspond to a return period of 73 years. Stated simply, the return period is the average time intervals between events of a similar magnitude. For example, a 73-year design wave would be a wave that occurs an average of once every 73 years. For this study, an optimization analysis similar to that used for Poplar Island was performed. The results indicate that a 35-year return period is optimal for design of the dikes at Dead Ship Anchorage, which is comparable to the results obtained for the Poplar Island project.

3.4 Geotechnical Factors

The main geotechnical factors that should be evaluated in the design of the habitat restoration dikes (Pilarczk 1990) are:

- Macro-instability of slopes due to failure along circular or straight sliding surfaces
- Settlements and horizontal deformations due to the self weight of the structure
- Micro-instability of slopes caused by groundwater seepage out of the slope face
- Piping or internal erosion due to seepage flow underneath the structure
- Liquefaction caused by erosion (flow down the side slopes) or by cyclic loading wave actions or earthquakes
- Erosion of revetments at the outer slopes (or underwater slopes) due to unstable filters or local failure of top layer elements

The phenomena most germane to the overall planning of the dike designs are: (1) slope stability which dictates maximum allowable combinations of side slopes and structure heights and (2) settlement which influences the initial and final crest elevation of the dike. The geotechnical assessment indicates that significant improvements to the foundation would be needed to construct dikes at the Dead Ship Anchorage Site. For this report it is assumed that the foundation has been strengthened to permit an outer structure slope of three horizontal to one vertical (3:1). Wave runup, overtopping, armor stone sizing and toe scour protection are evaluated for a 3:1 side slope. It is noted that this side slope is the same as that used for the

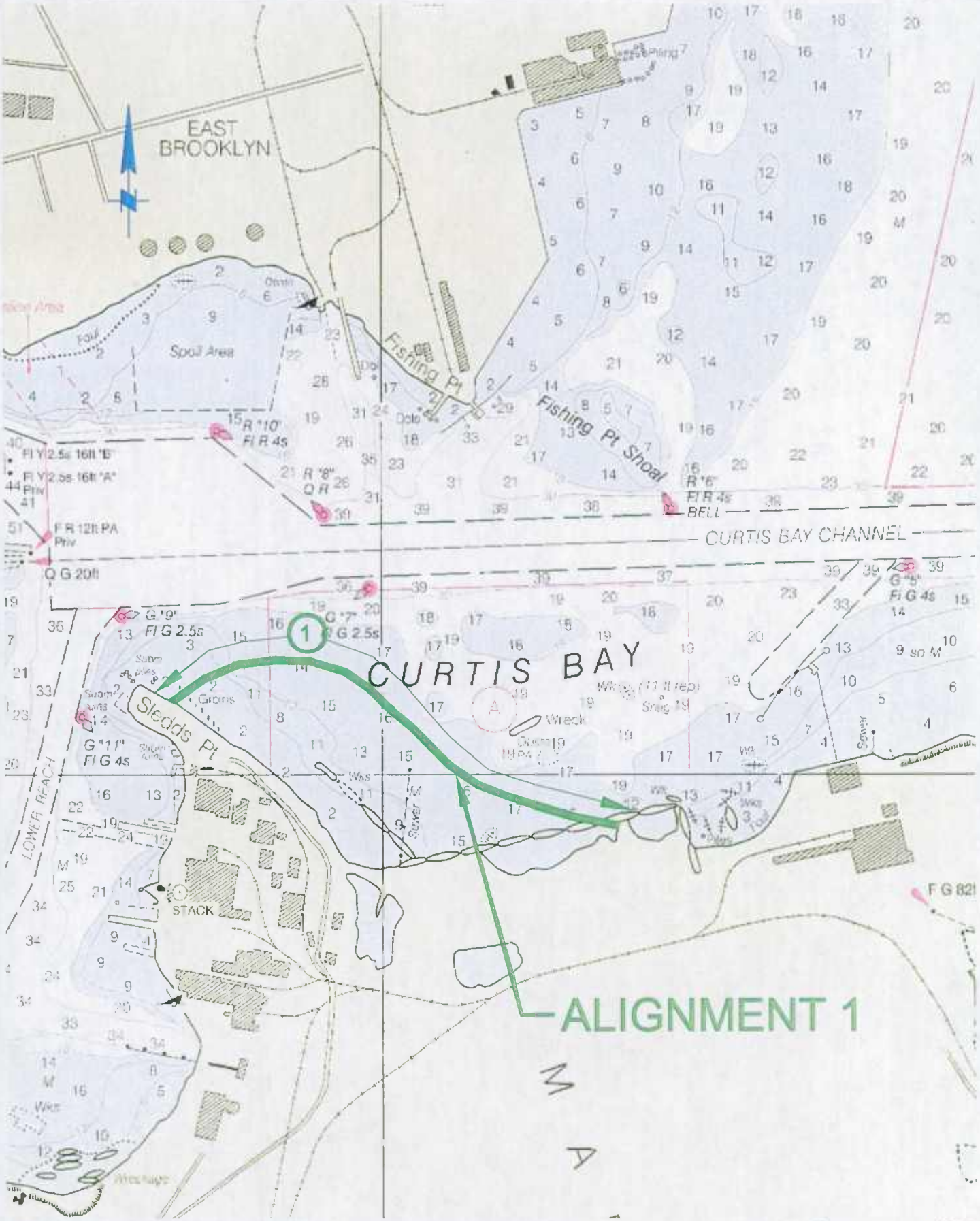

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FIGURE 3-3
DIKE SECTION
LOCATION

majority of the dike at Hart-Miller Island Dredged Containment Facility (HMI DMCF) and Poplar Island.

3.5 Dike Height - Wave Runup and Overtopping

One of the primary functions of the containment dikes is to protect the interior of the diked placement area against the adverse effects of high water and waves. If a high level of protection is required, the structure should have a height well above the maximum level of wave runup during storm surges. Typically, this requires setting high crest elevations for the structure. However, if some overtopping is allowed based on the nature of the site (i.e., wetlands), the design requirement can be evaluated in terms of allowable overtopping. The design then is based on maintaining the structural integrity of the dikes themselves with minimal concern for protecting the interior.

The level of protection against high water and wave attack has been defined as the return period of the storm event that balances initial dike construction capital costs with long-term operations and maintenance costs needed to repair the dike as a result of destruction from wave runup and/or overtopping waves. Wave runup, and more importantly, overtopping computations allow an objective means for evaluating the level of protection (i.e. allowable overtopping) offered by various dike height and armor protection combinations. In addition, wave overtopping computations provide a rational means for evaluating the relative risk of dike breaching and subsequent failure.

Wave runup is commonly evaluated on the basis of the composite-slope runup method outlined in the Shore Protection Manual (SPM) (USACE 1984). This approach has been critically reviewed by FEMA (1988) who found that the composite slope method provides a valid method for estimating the *mean* runup value in random waves but was lacking in its ability to predict *extreme* values of wave runup. The mean runup values computed using the FEMA composite-slope runup model are generally on the order of 2 to 4 ft above the still water level under extreme conditions (e.g. 50 to 100 year storms). Low or insignificant wave overtopping discharge values are normally computed on the basis of the mean wave runup values.

Dutch engineers have long appreciated the need to consider wave runup levels higher than the mean values in design applications and have generally used the 2% exceedence runup value to select the heights of dunes and coastal dikes. Van der Meer (1992) published the following formulae for computing the 2% runup for seawalls and dikes:

$$R_{2\%} = 1.5\,\gamma_f\,\gamma_h\,\xi_p$$

$$Maximum = 3.0\,\gamma_f\,\gamma_h$$

$$\xi_p = \tan\frac{\alpha}{\sqrt{S_p}}$$

$$S_p = \frac{H_s}{\frac{g}{2\pi}T_p^{\,2}}$$

Where:

$R_{2\%}$ = 2% wave runup (wave runup exceeded only 2% of the time during a storm)

γ_f = influence factor for roughness

γ_h = influence factor for shallow water

ξ_p = breaker parameter (surf similarity parameter) based on equivalent slope

α = angle of beach and or structure slope

S_p = wave steepness

H_s = significant wave height, average of highest one-third

g = acceleration of gravity

T_p = peak spectral wave period

Van der Meer's formulae are based on an extensive series of physical model tests including several full scale tests for 3:1 slopes.

The influence of roughness based on Van der Meer (1992) is summarized as follows:

Surface Covering	Influence Factor (γ_f)
One Layer of Rock	0.55 - 0.60
Two or More Rock Layers	0.50 - 0.55

A value of 0.55 was used for the present work.

While wave runup is an important overall indicator of the protection offered by coastal dikes, wave overtopping is judged to be a more objective and rational method for estimating the level of wave protection for the present work. Van der Meer (1992) presents the following formula for estimating the mean wave overtopping on coastal structures subject to random waves:

$$\frac{q}{\sqrt{gH_s^{\,3}}} = 8.10^{-5} \exp\left[3.1\frac{R_{u2\%} - R_c}{H_s}\right]$$

Where: q = mean wave overtopping discharge per unit width

R_c = dike crest freeboard (height of structure above still water)

$R_{u2\%}$ = 2% wave runup

The reliability of the above equation can be given by assuming that $log(q/\sqrt{gH_s^3})$ has a normal distribution with a variation coefficient (the ratio of the standard deviation to the mean value) of 0.11. Reliability bands can then be calculated for various practical values of mean overtopping discharges. The 90% confidence bands have been used for the purposes of this report.

The above overtopping formula provides a means for computing wave overtopping on dikes of various geometries (i.e. structure slopes, slope breaks and crest elevations). In order to evaluate the level of protection offered by a given dike configuration, it is necessary to establish limiting values of allowable overtopping. Critical or allowable overtopping discharges have been published by the United Kingdom (UK) Construction Industry Research and Information Association (CIRIA) and the Netherlands Centre for Civil Engineering Research and Codes

(CUR) (CIRIA/CUR 1991). Similar values have also been published by Goda (1985). The Goda allowable overtopping values have been used in this study and are summarized below:

Structure Type	Surface Armoring	Overtopping Rate (Liters/m·s)
Type I: Coastal Dike	Concrete on front slope, soil on crown and back slope	5
Type II: Coastal Dike	Concrete on front slope and crown, soil on back slope	20
Type III: Coastal Dike	Concrete on front slope, crown and back	50
Type IV: Revetment	No pavement on ground	50
Type V: Revetment	Pavement on ground	200

Overtopping computations from Van der Meer's formula were used to develop required crest elevations for construction of a dike with no armor stone on the crest or back slope. The results for dikes having a 3:1 side slope are summarized in Table 3-1 and the corresponding polar plot shown in Figure 3-1.

As shown in Table 3-1, required crest elevations for Dead Ship Anchorage are highest for dikes exposed to waves from the northeast, and range from about 4.6 ft MLLW for a 5-year storm to about 11.1 ft MLLW for a 100-year event. The lowest required crest elevations for Dead Ship Anchorage are for a dike exposed to waves from the west, and range from about 3.8 ft to 8.0 ft.

Table 3-1
Required Crest Elevation (ft)
Dead Ship Anchorage

Return Period (years)	West	Northwest	North	Northeast	East
5	3.8	4.1	3.9	4.6	4.4
10	4.2	4.5	4.3	5.3	5.0
15	4.6	4.9	4.7	5.8	5.4
20	4.9	5.3	5.1	6.4	5.9
25	5.3	5.7	5.4	6.9	6.4
30	5.6	6.1	5.8	7.3	6.8
35	6.0	6.5	6.2	7.8	7.2
40	6.3	6.8	6.6	8.3	7.7
50	7.0	7.5	7.3	9.2	8.5
100	8.0	9.0	8.8	11.1	10.3

3.6 Armor Stone

There are a number of methodologies available for determining armor stone requirements for dike revetments subject to wave attack. A commonly used method is based on the Hudson equation published in the SPM (USACE 1984):

$$W = \frac{\gamma_r H^3}{K_D (S_r - 1)^3 \cot(\theta)}$$

Where: W = weight of armor stone

γ_r = unit weight of the armor rock (taken as 165 pounds per cubic foot)

H = wave height to which the structure is exposed

K_D = stability coefficient

$S_r = \gamma_r / \gamma_w$

γ_w = unit weight of water (taken as 64 pounds per cubic foot)

θ = angle of structure slope

The dikes at Dead Ship Anchorage will be located in relatively deep water (–14 to –17 ft MLLW), and will be exposed to a wave spectrum characterized by non-breaking waves. The wave height used in the above equation depends on whether one is evaluating breaking or non-breaking waves. According to the SPM (USACE 1984), an H_{10} wave height, which is equal to 1.27 times the significant wave height (H_s), is used for the non-breaking wave height while the maximum depth limited wave height is used for breaking waves.

Previous studies have shown that use of the Hudson formula results in oversized armor stone, and a more appropriate method is to use procedures published by Van der Meer (1988). Van der Meer's equations for sizing armor stone subject to shallow water random waves are as follows:

For Plunging Waves:

$$\frac{H_{2\%}}{\Delta} = 8.7\ P^{0.18} \left(\frac{S}{\sqrt{N}}\right)^{0.2} \xi_m^{-0.5}$$

For Surging Waves:

$$\frac{H_2\%}{\Delta} D_{n50} = 1.4\ P^{-0.13} \left(\frac{S}{\sqrt{N}}\right)^{0.2} \xi_m^{P}$$

The waves are of the surging types when:

$$\xi_{mc} \geq 6.2\ P^{0.31} \sqrt{\tan\theta}^{\,1/(P+0.5)}$$

Where:

$H_{2\%}$ = two percent exceedence wave height

Δ = S_r

D_{n50} = mean nominal diameter of the stone = $(W/S_r)^{1/3}$

S = structural damage level taken as 2 for 0-5% damage

N = number of waves in the storm (a value of 7000 was used)

P = structure permeability (taken as 0.1 which is typical of a revetment structure with an armor layer, under or filter layer and an impermeable core)

ξ_m = surf similarity parameter

The surf similarity parameter is defined as:

$$\xi_m = \frac{\tan(\theta)}{\sqrt{\frac{2\pi H_s}{gT_p^2}}}$$

Where: H_s = significant wave height

T_p = peak spectral wave period

The methodology presented by Van der Meer is judged to be most applicable because it is based on random wave conditions. The guidance stone sizing presented in the SPM (USACE 1984) are based on monochromatic (i.e. single sine wave) wave conditions. Accordingly, the Van der Meer methodology will be used as the basis for preliminary dike design. It has been incorporated into the USACE's Automated Coastal Engineering System (ACES) and has been recommended in lieu of the Hudson Equation in the USACE's EM-1614 Design of Coastal Revetments, Seawalls and Bulkheads. Computations were made using Van der Meer's equations for each wave exposure direction.

Table 3-2 and the corresponding polar plot in Figure 3-2 show the results for stone sizes for a 3:1 side slope for the proposed dike at Dead Ship Anchorage. For Dead Ship Anchorage, required stone sizes for dike sections exposed to waves from the northeast range from 140 pounds for a 5-year return period to 1200 pounds for a 100-year return period.

The above armor stone requirements assume that the armor layer for the dike revetments will consist of two layers of placed rock. This is the normal design practice prescribed in the Shore Protection Manual.

Table 3-2
Individual Armor Stone Weight (pounds)
Dead Ship Anchorage

Return Period (years)	West	Northwest	North	Northeast	East
5	30	70	20	140	80
10	30	90	40	230	140
15	40	100	50	300	170
20	40	120	60	390	230
25	40	130	70	460	260
30	50	140	80	510	300
35	50	150	90	600	340
40	50	160	100	660	380
50	60	170	110	800	430
100	70	240	190	1200	710

3.7 Scour Protection

Toe scour protection is the supplemental armoring of the bottom surface fronting a structure that prevents wave energy from scouring and undercutting it. Factors that affect the severity of toe scour include wave breaking, wave runup and rundown, wave reflection and grain size distribution of the beach or bottom materials. Toe stability is essential because failure of the toe will generally lead to failure throughout the entire structure. Toe scour is a complex process and specific design guidance has not been developed. Some general guidelines, however, have been suggested.

A berm toe apron has been selected for the project for the following reasons: (1) construction costs for a berm toe are generally lower than for a buried toe, (2) higher quantities of sediment can be suspended during excavation and construction of a buried toe, and (3) the water depth associated with this project is better served by using a berm toe.

3.8 Underlayers and Filters

Revetments are normally constructed with an armor layer and one or more underlayers. Revetments often have two layers of armor and a thin underlayer overlying a geotextile built upon a core of sand or clay. Small particles beneath the geotextile should not be washed through the fabric and the underlayer stones should not be washed through the armor.

The SPM (1984) recommends that an underlayer stone range from 1/10 to 1/15 of the armor weight. This results in a relatively large underlayer that has two advantages. First, a large underlayer permits surface interlocking with the armor. Second, a large underlayer gives a more permeable structure and therefore has an influence on the stability of the armor layer. For the dike underlayer design, the SPM criteria are recommended.

3.9 Dike Cross Sections

Figure 3-3 presents the conceptual dike alignment for the Dead Ship Anchorage proposed project. The dike cross-section has been developed for the alignment based on the northeast wave exposure direction. Figure 3-4 presents the typical dike cross section along the alignment. The primary characteristics of the dike design are:

- Designs are based on 35-year return period storm conditions
- Designs incorporate a 3:1 side slope
- Dike heights are based on allowable overtopping for an unarmored crest and an allowance for settlement
- Stone sizes are computed using the Van der Meer method
- Above grade toe protection is used
- Core is constructed using sand
- A crushed stone roadway having a width of 20 ft is located on the structure crest.

Figure 3-4 shows the dike section for Dead Ship Anchorage with a crest of +8.0 ft MLLW, and includes two layers of 600 pound armor stone, two layers of 60 pound underlayer stone overlaying a geotextile that separates the stone revetment from the dike core.

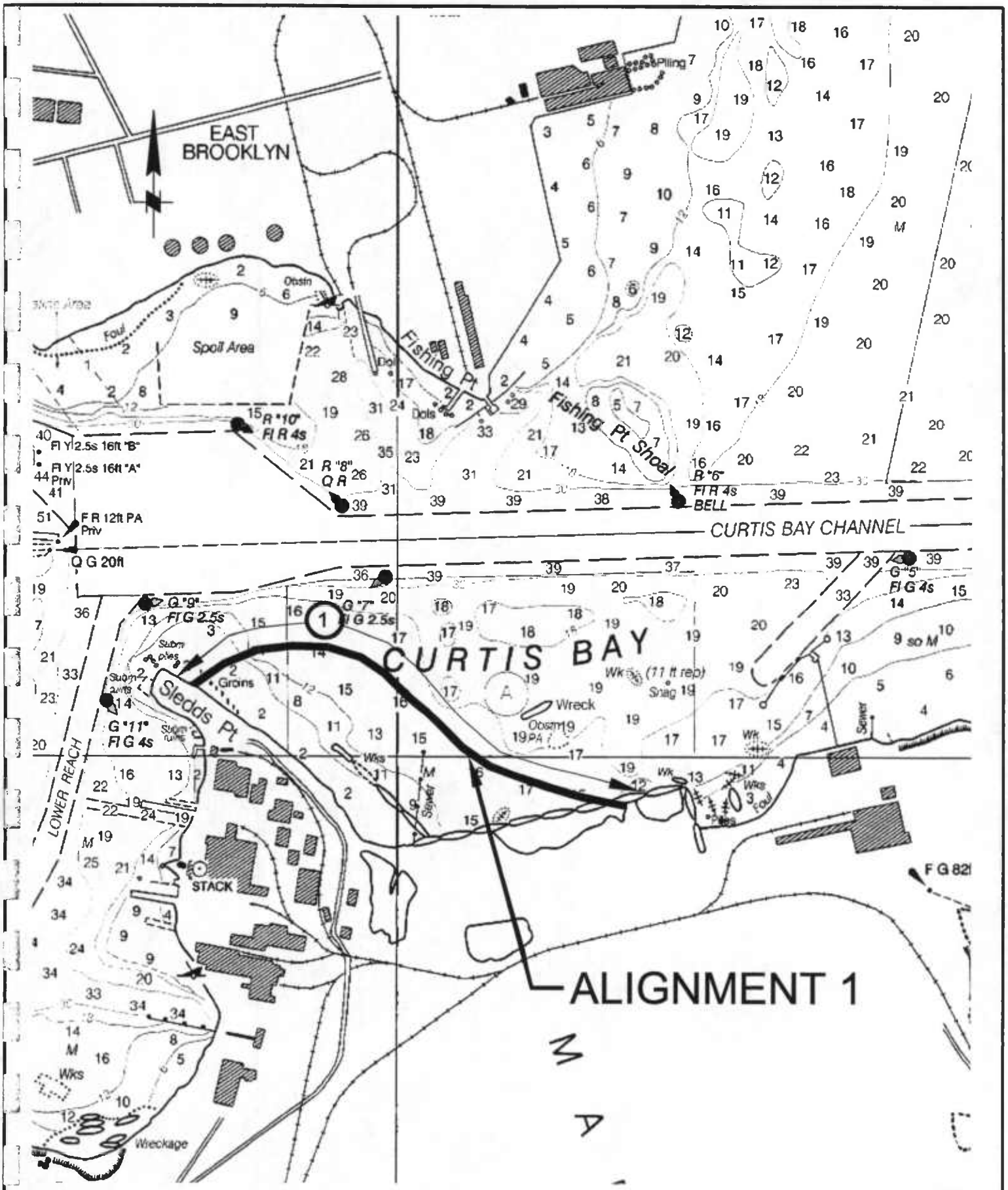

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FIGURE 3-3
DIKE SECTION
LOCATION

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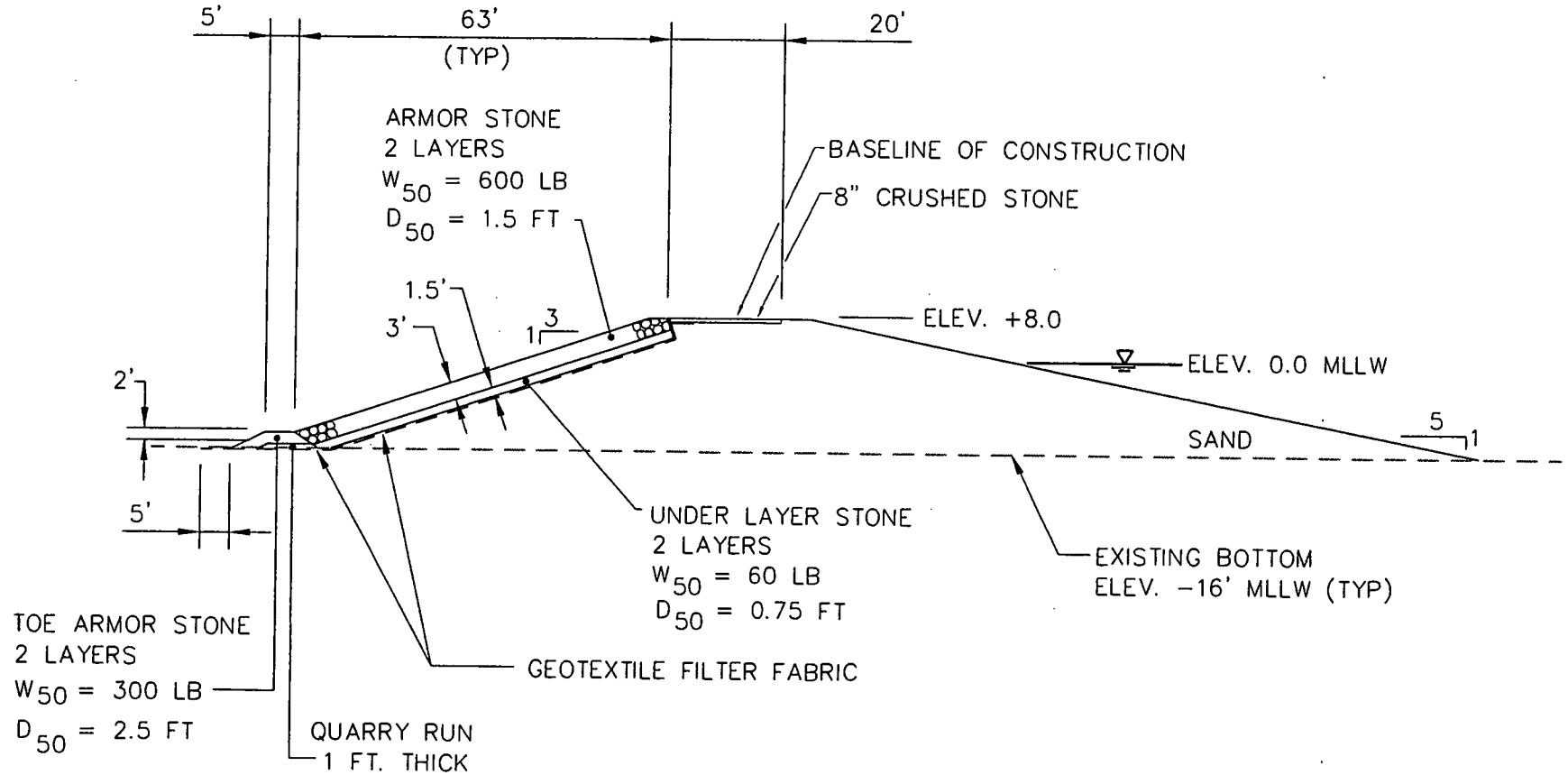
FIGURE 3-4
TYPICAL
DIKE SECTION



0
FEET
30

TYPICAL DIKE SECTION No. 1

1"=30'-0"

4. CONCLUSIONS AND RECOMMENDATIONS

4.1 Site Conditions

This Coastal Engineering Investigation report provides information on the Dead Ship Anchorage site being considered for confined disposal of dredged material. Its ultimate use would be for upland habitat creation. This report addresses evaluation of existing available data pertaining to environmental site conditions and coastal engineering aspects for the design of the diked enclosure.

Water depths in the area where the dikes would be located range from –2 to –17 ft MLLW, with an average depth along the exterior dikes of –14 ft MLLW.

Design winds for the site are developed from data collected at Baltimore-Washington International (BWI) Airport. Design wind speeds are calculated for return periods ranging from 5 to 100 years for eight wind directions including the direction with the longest fetch (northeast).

Normal water levels at the site are dictated by astronomical tides. Mean tide level is 0.8 ft above MLLW. Design water levels for the project area are dominated by storm surge which for a 100-year return period can be as high as 8.4 ft above MLLW.

The highest waves for the site approach from the northeast direction. Dead Ship Anchorage is sheltered from wind generated waves from the south, southeast and southwest. Predicted peak spectral wave period and significant offshore, significant nearshore and maximum nearshore wave heights for the northeast direction for the 5-year storm are 2.5 seconds, 1.9 ft, 1.9 ft, and 3.3 ft, respectively. Predicted peak spectral wave period and significant offshore, significant nearshore and maximum nearshore wave heights for the northwest direction for the 35-year storm are 3.0 seconds, 3.3 ft, 3.3 ft and 5.9 ft, respectively. Predicted peak spectral wave period and significant offshore, significant nearshore and maximum nearshore wave heights for the northeast direction for the 100-year and storm are 3.3 seconds, 4.2 ft, 4.2 ft and 7.6 ft, respectively.

Currents in the project area are weak to variable and are not considered critical to design the shore protection.

Results of the preliminary study by E2CR indicate that the underlying soil is soft clays. The geotechnical assessment indicates that significant improvements to the foundation would be needed to construct dikes.

4.2 Coastal Protection Dike Design

A preliminary cross-section was developed for the containment dike. The dike design is based upon a 35-year return period. Dike height is based on allowable overtopping for an unarmored crest and an allowance for settlement. Stone size is computed using the Van der Meer method. The designs incorporate 3:1 side slope, above grade toe protection, a core constructed of sand, and a crushed stone roadway on the structure crest.

The dike section has been developed for Dead Ship Anchorage (refer to Figure 3-4). The Dike Section for Dead Ship Anchorage has a crest of +8.0 ft MLLW, and includes two layers of 600 pound armor stone, two layers of 60 pound underlayer stone overlaying a geotextile that separates the stone revetment from the dike core.

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