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Technical Information Report
Confined Disposal Facility "D"

Prepared for the CDF D Value Engineering Study
19-23 June 2000

EPA Superfund Program

New Bedford Harbor Remedial Design
Upper & Lower Harbor

New Bedford, MA

Prepared By:
US Army Corps of Engineers
New England District
696 Virginia Road
Concord, MA 01742-2751

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TECHNICAL INFORMATION REPORT FOR VALUE ENGINEERING STUDY
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TECHNICAL INFORMATION REPORT FOR VALUE ENGINEERING STUDY
CONFINED DISPOSAL FACILITY 'D'
OPERABLE UNIT #1
NEW BEDFORD HARBOR SUPERFUND SITE
NEW BEDFORD, MA

1. INTRODUCTION

1.1. Project Background.

The New Bedford Harbor Superfund Site is located in Bristol County, MA. The site extends from the shallow northern reaches of the Acushnet River Estuary south through the commercial port of New Bedford Harbor and adjacent areas of Buzzards Bay (Figure 1). The sediments in the harbor are contaminated with high concentrations of many pollutants including PCBs and heavy metals from the industrial and urban development surrounding the harbor.

The United States Environmental Protection Agency (EPA) has selected a remedial action plan for the upper and lower areas of the New Bedford Harbor (NBH). The plan includes removal of approximately 450,000 cubic yards of PCB-contaminated sediment; containment of the sediments in four shoreline confined disposal facilities (CDFs), treatment of water decanted from the sediments, and interim and final capping of the CDFs once filled. Section X of the Record of Decision (ROD) provides a more complete discussion of the remedy. Subsequently, the EPA and U.S. Army Corps of Engineers , New England District (USACE-NAE) entered into an Inter-Agency Agreement in February 1998 which gives NAE responsibility of providing technical assistance to the EPA on this project. In October of 1998, the EPA authorized NAE to perform remedial investigation and design activities associated with the upper and lower New Bedford Harbor cleanup.

In order to perform a number of pre-design and design activities requested by EPA, a team approach between Foster Wheeler Environmental Corporation (FW) and NAE was determined to be the most advantageous means of accomplishing the work. FW has been awarded a task order under NAE's Total Environmental Restoration Contract (TERC) with the intent of establishing a collaborative effort and allowing the project to take advantage of the most qualified individuals and specialists in both organizations to prepare and implement the designs in a cost effective manner. In addition, NAE has acquired the services of Haley & Aldrich, Inc. (H&A) for their geotechnical expertise in the design of CDF "D".

1.2. Site History.

From the 1940s until approximately the 1970s, two electrical capacitor manufacturing plants in the New Bedford area discharged PCB waste either directly into the harbor or indirectly through discharges to the city's sewerage system. In the mid 1970s, as a result of EPA sampling, PCBs were identified in the sediments and the seafood in the New Bedford Harbor area. These previous releases of PCBs into the harbor pose an imminent threat and substantial endangerment to the public health and welfare and the environment. In 1979, the Massachusetts

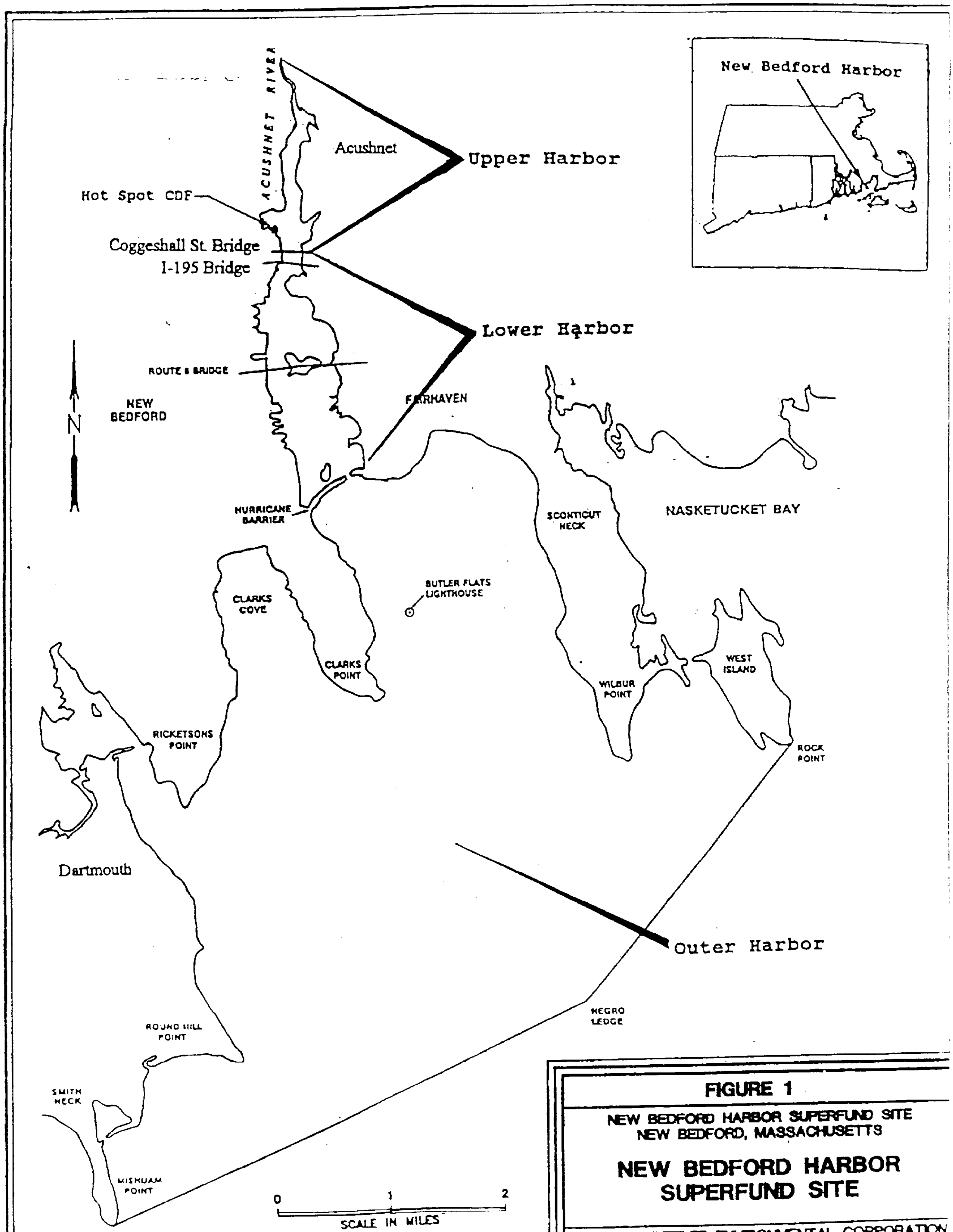

FIGURE 1

NEW BEDFORD HARBOR SUPERFUND SITE
NEW BEDFORD, MASSACHUSETTS

NEW BEDFORD HARBOR SUPERFUND SITE

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Department of Public Health issued regulations prohibiting fishing and lobstering throughout the site due to high levels of PCB contamination ranging from below detection limits to higher than 100,000 parts per million (ppm) in various parts of the harbor. The site was included on the Superfund National Priorities List (NPL) in September 1983. EPA's site-specific investigations were initiated in 1983-1984, and included engineering feasibility studies of alternative dredging methods and disposal of contaminated sediments, pilot dredging and disposal studies to field test different dredging and disposal technologies for the contaminated sediments, and extensive physical and chemical computer modeling of the site. These studies are summarized in more detail in EPA's Administrative Record of the site.

1.3. Project Description.

The selected cleanup remedy as described in the ROD requires the dredging and excavation of approximately 450,000 cubic yards (CY) of PCB contaminated sediments spread over 170 acres of the upper, lower and outer areas of the New Bedford Harbor (Figure 2). The goals of the remedy are to minimize health risks due to the consumption of PCB contaminated seafood and contact with the shoreline sediments and improve the quality of the upper and lower harbor marine ecosystem for the City of New Bedford and the Towns of Acushnet and Fairhaven, Massachusetts.

The project site encompasses the entire upper and lower harbor areas of the New Bedford Harbor. The upper harbor is defined as all intertidal, subtidal, beach combing, wetland/salt marsh and upland areas north of Coggeshall Street and the lower harbor is defined as all areas south of Coggeshall Street Bridge to the New Bedford Hurricane Barrier. This project involves the dredging and excavation of all intertidal, subtidal, upland and wetland/marsh areas of the upper and lower New Bedford Harbor which have PCB contaminated sediments that exceed the cleanup levels established by EPA's September 1998 Record of Decision (ROD). A few areas outside of the New Bedford Hurricane barrier will also be dredged since they exceed the cleanup levels for this project. The contaminated sediments will be disposed of in four Confined Disposal Facilities, identified as CDFs A,B,C, and D, along the New Bedford Harbor shoreline that will be designed and constructed (see Fig. 2).

2. CONFINED DISPOSAL FACILITY (CDF) 'D' DESIGN

2.1. Design Objectives/Criteria.

The preliminary analyses and cell configurations reported in this Information Packet reflect CDF "D" Plan A1 as defined in the report titled "Draft Feasibility Study - Confined Disposal Facility 'D'" Prepared by NAE, dated June 2000. The plan presented herein is generally similar to the other plans discussed in the Draft Feasibility Study. Bulkhead cell diameters described in this report apply to alternatives with cell heights of +10.5 ft NGVD and +13 ft NGVD. Note that those plans presented in the Feasibility Study that consider raising the bulkhead height to +16 ft NGVD will require larger cell diameters.

Confined Disposal Facility (CDF) D will be a near-shore facility located along the west shore of the Acushnet River, south of Coggeshall Street (Figure 3). The facility is located

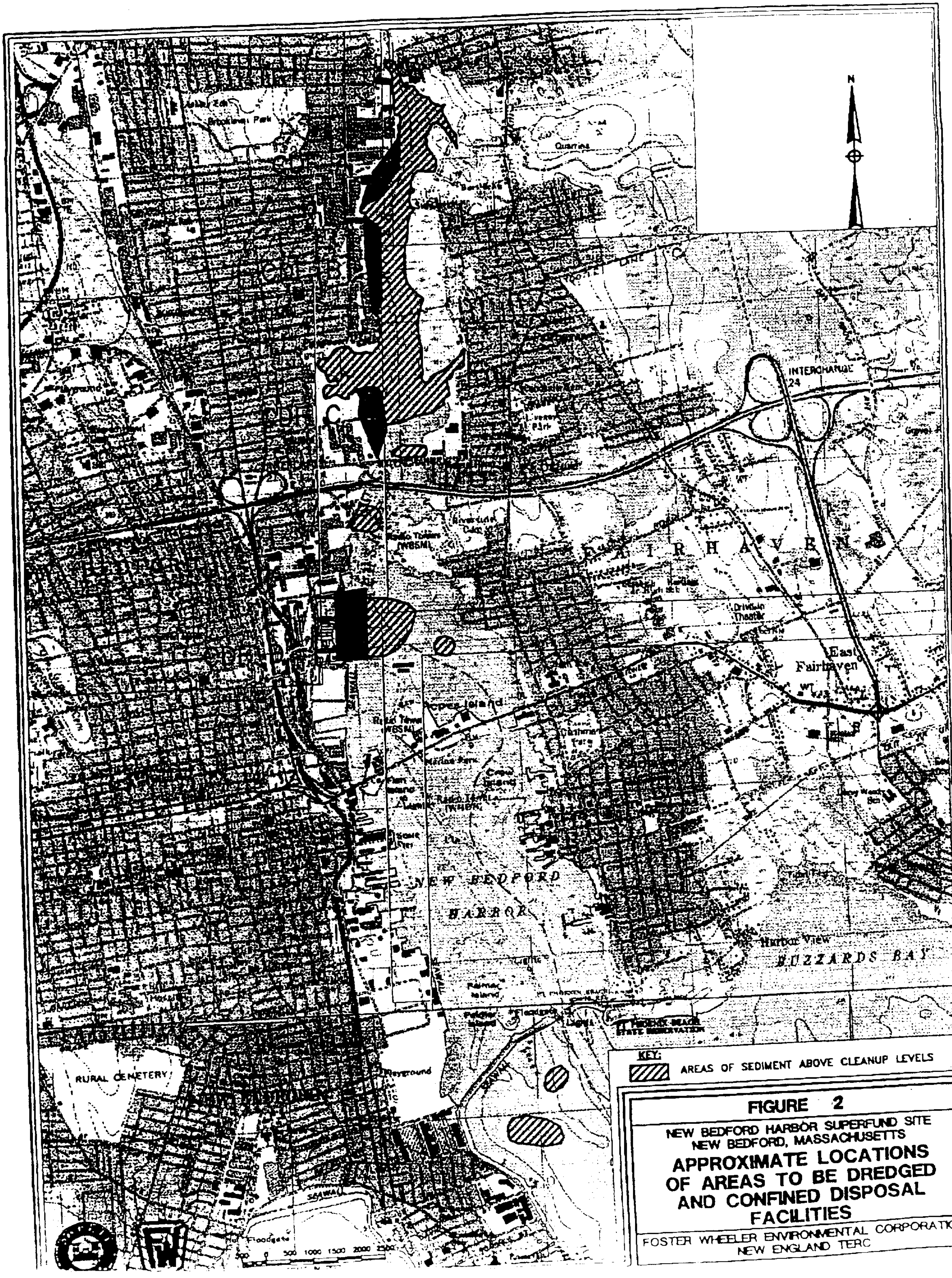

FIGURE 2

NEW BEDFORD HARBOR SUPERFUND SITE
NEW BEDFORD, MASSACHUSETTS

APPROXIMATE LOCATIONS OF AREAS TO BE DREDGED AND CONFINED DISPOSAL FACILITIES

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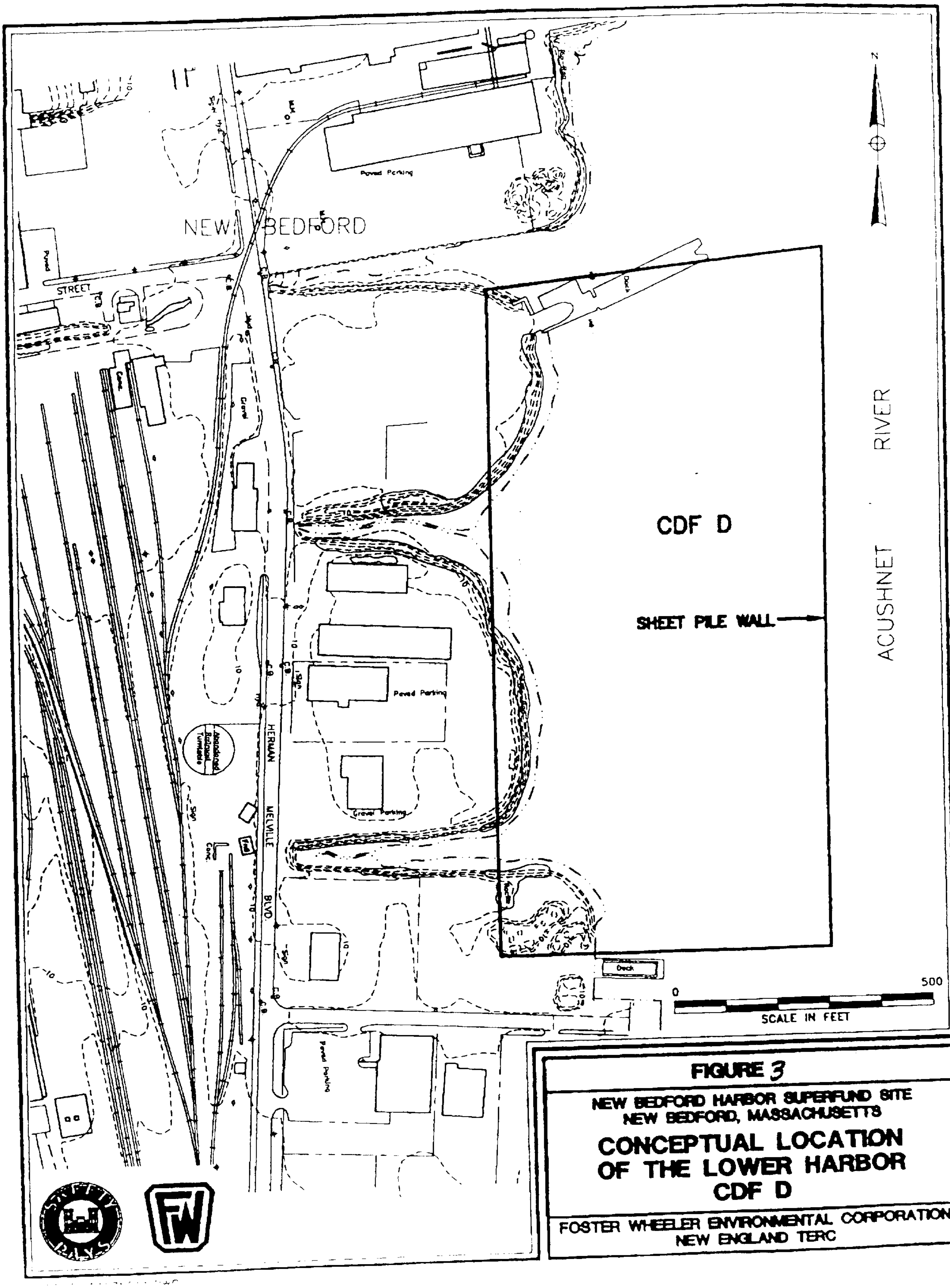

FIGURE 3

NEW BEDFORD HARBOR SUPERFUND SITE
NEW BEDFORD, MASSACHUSETTS

CONCEPTUAL LOCATION OF THE LOWER HARBOR CDF D

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adjacent to several commercial operations including a seafood processing plant, a welding supply facility, and a salvage operation. The shoreline wall of the CDF will be constructed of driven steel sheet piles aligned along the existing shoreline. In accordance with the conceptual design defined in the ROD for OU#1, containment of PCB contaminated sediments will be provided by a combination of the steel sheet pile wall along the shore and cellular bulkhead structure within the harbor, combined with a liner along the interior sides of the facility. As indicated in the ROD for OU#1, the bottom of the facility will not require either a liner or a leachate collection system. Final design will include a relatively impermeable cover system with gas venting and drainage layers, among other components. The facility will be designed for future use as an intermodal port facility in coordination with the City of New Bedford and their design consultant.

The cellular steel sheet-pile bulkhead will consists of a series of complete circular cells connected by shorter arcs. The area within and between the circular cells (within the arcs) will be filled with granular material. Cellular structures depend on the weight and strength of the cell fill material (granular soils or stone) for stability and strength. The flat steel sheet piles only provide confining pressures for the soil (as a tension ring). The primary advantages of circular cells are that each cell is independent of adjacent cells, it can be filled as soon as it is constructed, and it is easier to form by means of templates. Increasing the strength of the cell fill material and the in-situ soils contained within the cell will be necessary to keep the cell sizes reasonable. Locating a reliable source for cell fill material, and assuring that appropriate placement and compaction techniques are specified will have the most significant impact on the required size of the cellular structure. The allowable height of the cellular structure will also affect the required cell size.

The design of CDF "D" will consist of 30%, 90%, and 100% Design submittals for review by EPA, DEP, our TERC Contractor, Foster Wheeler Environmental Corporation, the Corps Center of Expertise and other State and Local Agencies as required. Table 1 provides a list of CDF "D" significant design features, as well as a list of design objectives that will be pursued in development of a final design. These design objectives may be revised as the design of CDF "D" progresses.

Table 1 - CDF "D" Design Criteria - Structural Features

CDF Feature	Design Objectives
Cellular Bulkhead	• Contain contaminated dredge materials; optimize storage capacity • Support final cap • Serve as a working berm during CDF filling and consolidation • Support 800 psf surcharge on backfill for future use needs • Support marine terminal type activities on top of the cells • Withstand river currents, and potential vessel impact
Shoreline Walls	• Contain contaminated dredge materials along the existing shoreline • Support final cap • Support 800 psf surcharge on backfill for future use needs
Cutoff Wall	• Prevent migration of contaminants outside of the CDF area • Extend into bedrock, sealing off any fractures in the rock • Accommodate wall displacements • Repairable if necessary (Monitoring will be required)

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Table 1 - CDF "D" Design Criteria - Structural Features

CDF Feature	Design Objectives
Interior CDF Compartment Walls	• Separate the CDF into multiple compartments to increase efficiencies in CDF filling and consolidation • Support differential loading due to filling compartments independently • Serve as a working berm during CDF filling and consolidation
Cap on Cellular Bulkhead	• Protect cell fill material • Serve as a working platform for Marine Terminal activities • May or may not support a crane for handling cargo
Final CDF Cap	• Seal off CDF from precipitation; Pass runoff away from the CDF • Collect CDF gasses • Accommodate settlement of contaminated dredge material • Support Marine Terminal activities (800 psf surcharge) • Capable of allowing piles to be driven through the cap to support future warehouse type buildings

2.1.1. Design Regulations and Requirements. USACE ERs, EMs, Codes, & Other References used in the design of CDF "D" are listed below:

ER 1110-2-1150 "Engineering and Design for Civil Works Projects", 31 August 1999

ER 1110-2-1806 "Earthquake Design and Evaluation for Civil Works Projects", 31 July 1995

EM 1110-2-2503 "Design of Sheet Pile Cellular Structures", 29 September 1989

EM 1110-2-2504 "Design of Sheet Pile Walls", 31 March 1994

ETL 1110-2-474 "Engineering and Design of Cathodic Protection", 14 July 1995

AISC Manual of Steel Construction, Allowable Stress Design, 9th Edition

ACI 318-99 "Building Code Requirements for Reinforced Concrete" 1999

EPA/600/R-95/051 "RCRA Subtitle D (258) Seismic Design Guidance for Municipal Solid Waste Landfill Facilities" April 1995

310 CMR 19.000 "Landfill Design and Operational Standards"

Technical Report ITL 87-5 "Theoretical Manual for Design of Cellular Sheet Pile Structures" May 1987

Technical Report ITL 90-1 "A Study of the Effects of Differential Loading on Cofferdams" April 1990

Technical Report ITL 91-1 "User's Guide: Computer Program for Design and Analysis of Sheet Pile Walls by Classical Methods (CWALSHT Version 03/02/1998) Including Rowe's Moment Action" October 1991

Technical Report ITL 92-1 "3-D Finite Element Analysis of Sheet Pile Cellular Cofferdams" April 1992

Technical Report ITL 92-11 "The Seismic Design of Waterfront Retaining Structures" November 1992

Technical Report ITL 94-5 "User's Guide: Computer Program for Winkler Soil-Structure Interaction Analysis of Sheet Pile Walls (CWALSSI Version 02/02/1998)"

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Schaaf & Ebeling "Olmsted Lock and Dam, Design/Analysis of Sheet Pile Cellular Retaining Walls" 1995 Corps of Engineer's Structures Conference

Wissmann, Filz, Martin, & Mosher "Supplemental Manual for Design of Sheet Pile Cellular Structures" November 1996

Technical Report ITL 97-1 "User's Manual: Computer Program for the Analysis of Circular Sheet Pile Cellular Structures (CCELL – DOS/Windows Version 1.0 1999/06/15)" September 1997

2.1.2. Applicable or Relevant and Appropriate Requirements (ARARs) .

2.1.2.1. Regulatory Compliance Program.

The New Bedford Harbor Superfund Site is classified as a NPL site requiring remediation by the US Environmental Protection Agency (USEPA) under the Comprehensive Environmental Response, Compensation, and Liability Act of 1980 (CERCLA) and the National Hazardous Oil and Substances Pollution Contingency Plan (NCP). An interagency agreement has been signed between the USEPA and the US Army Corps of Engineers (USACE) granting authority to the USACE for administration of the selected remedy. Foster Wheeler Environmental Corporation (FWENC) will be conducting the Remedial Design for Operable Unit #1 as a TERC contractor to the USACE.

CERCLA response actions are exempted by law from the requirement to obtain Federal, State or local permits related to any activities conducted completely "on-site." It is the policy of the USEPA (and the Department of the Army) to assure all activities conducted on site are protective of human health and the environment, and the requirement to meet (or waive) the substantive provisions of permitting regulations that are applicable or relevant and appropriate requirements (ARARs).

The Final Regulatory Compliance Plan (RCP) prepared by FWENC, dated July 1999, identified the ARARs and applicable regulatory requirements for the remedial design of Operable Unit #1 of the New Bedford Harbor Superfund Site based on the following Project Decision Documents:

- The USEPA Record of Decision (ROD), dated September 1998;
- The Foster Wheeler Remedial Design Work Plan; and
- The USACE Project Management Plan (PMP).

The RCP included an expanded definition of "on-site" along with the identified ARARs. Any work in areas deemed to be "off-site" will be conducted in accordance with both administrative and substantive requirements of applicable Federal, State, and local regulations.

2.1.2.2. ARARs.

The ARARs are provided in Table 8 of the ROD and reproduced as Appendix A in the RCP. The ARARs are presented in tabular format with the last column stating the "Actions to be Taken to Attain ARARS". These actions are directly incorporated into the CDF "D" project designs and work plans. However, the following ARARs involve additional consultation with

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appropriate Federal and State regulatory agencies and local interests relative to natural resources in order to obtain full attainment:

2.1.2.2.1 Action Specific ARARs "Medium/Authority" - "Actions to be Taken to Attain ARARS": "*CWA, Section 404, Dredge and Fill Activities*" – "EPA finds that the remedy is the least damaging alternative to remediating the Harbor. Dredging of sediments and filling CDFs will be implemented so as to minimize to the maximum extent possible any adverse environmental impacts through engineering controls such as type of dredge used, rate of dredging, varying target cleanup levels in wetlands, and salt marsh revegetation."

2.1.2.2.2 Location Specific ARARs "Medium/Authority" - "Actions to be Taken to Attain ARARS" – Federal: "*Wetland Protection – Executive Order 11990*" - "This is the best practical alternative for remediating the Harbor. The Agency will minimize the destruction, loss and degradation of wetlands as much as possible given the extent and location of contaminated sediment. Where ever possible, higher target cleanup levels were set in wetlands to minimize destruction. Replanting of dredged wetlands will occur."

"*Fish and Wildlife Coordination Act*" – "Appropriate agencies will be consulted prior to implementation to find ways to minimize adverse effects to fish and wildlife from harbor dredging and from construction and maintenance of CDFs."

"*Endangered Species Act*" – "EPA will consult with appropriate agencies to consider measures for remedial activities affecting the identified feeding grounds for roseate tern."

2.1.2.2.3 Location Specific ARARs "Medium/Authority" - "Actions to be Taken to Attain ARARS" – Massachusetts: "*Wetlands Protection Act*" – "Best available measures will be used to minimize adverse effects on identified resource areas and associated 100 foot buffer zones during design and implementation of remedy. Dredged marshes will be replanted. DMF will be consulted for activities affecting fish and shellfish habitat."

The specific Performance Standard and Mitigation Method proposed for the CDF construction activities pursuant to the Massachusetts Wetland Protection Act Regulations (310 CMR 10) are provided in detailed tabular format in Section 3.8.10, Wetlands Protection Requirements, in the RCP. If project design cannot meet the performance standards, then either mitigation methods or evaluating a variance of the requirement will be employed.

"*Administration of Waterways Licenses Law*" – Temporary unavoidable impacts to public access rights to water and to water dependent users will occur. Alternate access will be available. CDFs will be designed to accommodate future uses, subject to institutional controls, such as parks, sport fields, and in designated port areas, marinas."

The specific Performance Standard and Mitigation Method proposed for the CDF construction activities pursuant to the Massachusetts Waterways Regulations (310 CMR 9) are provided in detailed tabular format in Section 3.8.11, Waterways Regulations, in the RCP. The CDF construction project should comply with substantive requirements. If the performance standards cannot be met, then mitigation methods may be considered.

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Note that there is some overlap between the performance standards for the two Massachusetts ARARs, e.g. regarding the design and timing of work activities relative to anadromous and catadromous fish runs, shellfish, and related resources.

In summary, various commitments were made in the RCP based on the aforementioned Project Decision Documents relative to consultation of remediation activities with the appropriate Federal and State regulatory agencies and local interests. Coordination of the CDF designs and the dredging/excavation and wetland restoration plans with appropriate regulatory agencies and local interests was tasked to the USACE and is required based on commitments made in the ROD and RCP. Accordingly, USACE will conduct coordination of the CDF "D" design with the appropriate Federal, State, and Local Regulatory Agencies during the design review process.

2.2. Pre-Design Activities.

2.2.1. Real Estate. TBD

2.2.2. Hydrographic and Topographic Surveys.

The New England District survey unit performed hydrographic surveys of the upper and lower harbor and portions of the outer harbor areas during February and March 1999. Topographic surveys of the project area were performed by the James E. Sewall Company under contract to Foster Wheeler Environmental Company, the TERC for New England District. The topographic surveys were developed using aerial photogrammetric mapping with horizontal and vertical control provided by conventional ground surveys. Additional topographic surveys of wetland areas slated for excavation and restoration will be undertaken using conventional ground methods.

2.2.3. CDF "D" Exploration Program.

An extensive geotechnical investigation of the CDF "D" site was developed by NAE and performed by Foster-Wheeler during the summer and fall of 1999. The investigation included borings and wells drilled on land and from a barge in the harbor. In-situ testing, including borehole permeability testing, packer testing, and field vane shear testing, was conducted in some of the borings. Other subsurface investigation work included onshore and offshore geophysical surveys. The on-shore geophysical survey was conducted to locate utilities, obstructions, and/or historic relics that may interfere with the proposed construction. The offshore geophysical survey was conducted along the alignment of the proposed cellular bulkhead to better delineate the bedrock surface elevation.

Based on the results of the exploration tests, as well as the investigation of additional bulkhead configurations, additional geotechnical investigations will be performed in the Summer of 2000. These investigations are currently being scoped out, but are expected to include additional foundation borings, in-situ and laboratory strength testing, chemical analysis of samples taken at depth to characterize weak materials to be excavated from the bulkhead alignment, and marine geophysical surveys.

2.2.3.1. Phase 1 Geotechnical Investigation and Laboratory Testing (Fall 1999). Figure A1 in Appendix A shows the location of all the borings and wells advanced as part of this work as well as the approximate location of previous explorations by others. The boring program consisted of 17 offshore borings along the alignment of the proposed cellular bulkhead, 7 borings along the proposed alignment of the cantilever sheet pile wall located on the land side of the CDF, 7 borings in the interior of the CDF, and 2 borings in the channel for evaluation of dredging conditions. Boring spacing along the perimeter of the proposed structure was approximately every 150 feet. In addition, groundwater monitoring and observation wells were installed in the CDF "D" area to obtain environmental and hydrogeologic data. Total soil and rock boring footage for all of the explorations at CDF "D" is estimated at 1,600 linear feet. Both undisturbed and split-spoon samples were collected. Refer to Appendix A for additional information regarding the boring program.

2.2.3.1.1 In-Situ Testing. Based on a limited number of Borehole Permeability Tests, the coefficient of permeability for the organic soil deposits ranges from 4.91 x 10^{-6} to 6.1 x 10^{-4} cm/sec, the permeability for the glaciolacustrine soils ranged from 1 x 10^{-4} to 2.5 x 10^{-3} cm/sec, permeability of the glaciofluvial sands from 1 x 10^{-4} to 8 x 10^{-2} cm/sec.

The in-situ undrained shear strengths, undisturbed and remolded, as estimated from the Field Vane Shear Tests (FVSTs) ranged from approximately 0 to 160 psf with and average of about 40 psf.

Bedrock permeability measured in the Constant Head Permeability and Packer tests varies widely. The results of the bedrock permeability tests are discussed in more detail in Appendix A.

2.2.3.1.2 Laboratory Testing Program. A geotechnical laboratory-testing program was performed to assist in classification of soils and estimation of engineering parameters necessary for design of the CDF. Index testing, including moisture content, specific gravity, particle size, Atterberg limits, and organic content determinations, were conducted on samples from the organic stratum. Triaxial compression tests, one-dimensional consolidation tests, and flexible wall permeability tests were performed on undisturbed shelby tube samples of the organic clay. Particle size analysis were conducted on samples of the marine sand, glaciolacustrine, glaciofluvial, and glacial till deposits. One glaciolacustrine clay sample was tested for plasticity. Laboratory testing was performed by GeoTesting Express, Inc. (GTX) of Boxborough, Massachusetts.

The results of the geotechnical laboratory index testing as well as the consolidation and strength testing are summarized in Tables A-1 and A-2 in Appendix A. The gradations of the granular deposits are presented as Figures A-5 through A-11.

2.2.3.1.3 Column Settling Tests. Dredge material testing was performed by Soil Technology of Bainbridge Island, WA. In their report entitled "Dredge Material Testing, New

Bedford Harbor", Soil Technology presents the results of five column settling tests run on samples of different dredge slurry concentrations.

The concentrations tested were 0.6, 1.6, 3.6, 6.8, and 19.8 percent solids by volume. The settlement and consolidation behavior during the tests varied with concentration. This data was only recently received and has not yet been fully incorporated into the design.

2.2.3.2. Geophysical Investigation.

2.2.3.2.1 Onshore Geophysical Investigation. To investigate for subsurface utilities and obstructions, Foster Wheeler performed a geophysical investigation of the shoreline of the CDF "D" site. Their work is presented in a report dated February, 2000 entitled "Draft Report of On-Land Geophysical Surveys: Electromagnetics and Ground Penetrating Radar, Area "D", New Bedford Harbor Superfund Site, Operable Unit #1, New Bedford, Massachusetts.

The conclusions of the onshore geophysics investigation (as stated in the geophysics report) are that the following features exist on site:

- A buried reinforced concrete foundation or pad in the north end of the site.
- Numerous electric and/or pipe lines related to former industrial activity, including at least one of significant size (possibly a large pipe or buried sheet pile wall) which cuts across an area very close to the future CDF (from Marine Hydraulics to Taco Metals);
- Several areas that contain a large quantity of buried debris which could cause difficulty in future trenching, sheet pile driving and other construction activities; and
- Numerous individual potential buried objects, which may be hazardous to subsurface investigations or construction operations.

More detailed information about the methods and results of the on-land geophysical survey are available in the above referenced report.

2.2.3.2.2 Marine Geophysical Investigation. When it became apparent that the bulkhead structure might need to be founded on rock and that the bedrock surface elevation varied significantly between boring locations, it was decided to undertake an offshore geophysical investigation. Primary goals of the marine geophysical investigation were to determine the top of bedrock elevation and to look for boulders or other large obstructions above the bedrock.

Foster Wheeler and their subcontractors performed the marine geophysical investigation. Their work is presented in a draft report dated February, 2000 entitled "Report of Marine Geophysical Surveys: Uniboom and Seismic Refraction, New Bedford Harbor Superfund Site, Operable Unit #1, New Bedford, Massachusetts". Figure A-12 summarizes the results of the geophysical survey in the form of a bedrock surface contour map. (This figure is taken from a draft report, and is subject to revision). From the contour map, it can be seen that the bedrock surface elevation varies widely along the alignment with the deepest rock at approximately El. -90 ft NGVD and the shallowest at approximately El. -36 ft NGVD. The difference in the bedrock elevation varies as much as 47 ft from one side of the cell wall to the other (Note that the bulkhead alignment shown on this map is an earlier alignment that has since been superseded). Such variation in rock elevation makes founding the cellular bulkhead on the bedrock surface very problematic.

Foster Wheeler geophysicists also identified what they have termed "low velocity zones" which could be interpreted as nested boulders, highly fractured rock, or some other material which has a seismic velocity lower than hard rock but higher than soil. One of these zones is in the vicinity of boring FD-5, in the northeast corner of the site. These "low velocity zones" are a potential obstruction for sheeting if it is desired to drive sheeting to good quality bedrock. Even if sufficient bearing for the sheeting can be obtained on the surface of this low velocity material, the permeability of the low velocity material is likely to be high and seepage underneath the bulkhead could be difficult to control.

Upon review of the marine geophysics report, it was judged prudent to alter the alignment of the bulkhead wall. The southern wall was moved approximately 30 feet further south to avoid the rock valley that is on the northern side of the bulkhead, (Line 13 on the contour plan). For similar reasons, it was decided to round the corners of the bulkhead wall (this also saves wall length). Rounding the northeast corner moves the bulkhead away from the low velocity zone identified in that area. This realignment is important even if the bulkhead sheeting terminates at some fixed elevation in the overburden, as is currently proposed, because it will reduce the amount of soil improvement necessary and make construction of a cutoff wall less expensive.

A more detailed description of the procedures used and the results of the survey are available in the above referenced report.

2.2.4. Generalized Subsurface Profile.

Subsurface conditions vary across the project site both onshore and offshore. Major soil deposits encountered during the subsurface exploration program are listed below. This sequence reflects the typical order of occurrence of the geologic units below the ground surface. However, at specific locations, one or more units may be absent and the order of occurrence of soil units may vary. The marine sand and glaciolacustrine deposits, for instance, are interbedded in some of the borings.

- Fill (onshore only)
- Organic Soil Deposits (mostly Organic Clay)
- Marine Sand Deposits (also called Estuary Deposits)
- Glaciolacustrine Silt and Clay Deposits
- Glaciofluvial Sand Deposits
- Glacial Till Deposits
- Bedrock

Subsurface profiles of the centerline of the north and east bulkhead walls are presented as Figures A2 and A3, respectively. A profile along the shoreline is presented as Figure A4. These profiles show approximate strata boundaries, grouping the marine sand and glaciolacustrine strata together and the glaciofluvial and glacial till strata together. Tables A1 and A2 summarize the subsurface conditions encountered at each boring location. Table A3 summarizes the average groundwater elevation encountered in the monitoring wells and observation wells. Subsurface conditions were observed only at the boring locations. The strata boundary lines on the profiles are based on interpolation between borings and are shown only to provide visual continuity. The actual strata boundaries between borings may vary substantially from the lines shown on the profiles.

2.2.4.1. Offshore Soil Deposits. Gradation and other telltale features have been used to help identify the different soil strata, particularly the different sand deposits. The glacial till deposits are very similar to the glaciofluvial and in some area are largely indistinguishable. Typical descriptions of the soil units and bedrock encountered in the offshore subsurface explorations are described below, beginning at the harbor mudline.

ORGANIC SOIL DEPOSITS: Organic clay was encountered at most of the offshore boring locations and ranged from 7 to 16.5 feet in depth. Of 28 samples of organic soil tested for gradation, plasticity and organic content, 17 have been classified as organic clay (OH), one has been classified as organic silt (OL), eight have been classified as clayey sand (SC), and two have been classified as silty sand (SM). The clayey sand samples had 37 to 46 percent by weight passing the No. 200 sieve. The silty sand samples had 26 to 37 percent passing the No. 200 sieve. Soils in the organic stratum are generally dark gray to black, and have an organic odor.

Dark brown fibrous peat was encountered in at least five of the offshore borings. The peat stratum ranges from 1 to 5 feet in thickness and is fibrous in texture and dark brown in color.

The proposed cellular bulkhead roughly follows a previously dredged channel along the east side of the CDF. The organic soil thickness along the east side is was approximately 7- to 16.5-feet thick with the bottom of the strata at approximately El. –30. The "organic soil thickness" referred to here includes the organic clay, peat, organic clayey sand, and organic silty sand encountered in the organic stratum.

At boring locations FD-11 and FD-12 organic soil was not encountered at all. Along the northern wall of the CDF, the organic clay thickness varies between 1.5 and 11.5 with bottom sloping up from El. –30 on the east to about El. –20 near the shoreline. Borings FD-3 and FD-4, advanced in an area along the north wall that apparently was previously dredged, encountered only 1 foot and 2.5 feet of organic soil, respectively.

Generally, fill was encountered onshore only. However, granular soil was encountered overlying the organic clay at one offshore boring location (FD-24) which was advanced in the CSO out-fall channel. It is likely that the fill material at this location has been deposited over time as a result of storm water flow through the CSO. Cinder fragments were encountered in Boring FD-1 at a depth of 11.5 feet (the bottom of the organic clay strata). Shreds of polyethylene plastic were noted in the sample and wash cuttings at 19.5 feet in Boring FD-31 which corresponds to the bottom of the organic stratum at this location. These last two instances suggest that the organic soils at these locations have been recently deposited or pushed into place as a mudwave.

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MARINE SAND DEPOSITS: Fine sands and fine silty sands were encountered in most of the offshore borings below the organic silt deposits. These deposits are interbedded with the glaciolacustrine deposits in some locations. The results of five laboratory gradation tests on marine sand suggest that this deposit is between 47 and 97 percent fine sand with 4 to 29 percent silt. The marine sands are generally loose to medium dense and often contain shell fragments.

GLACIOLACUSTRINE SILT AND CLAY DEPOSITS: The glaciolacustrine deposits generally underlie and are sometimes interbedded with the marine sand deposits. This strata is mostly silt, although layers of lean clay (1 to 5 feet thick) were encountered within the strata at a number of locations. Standard Penetration Test N-values are generally less than 5 blows per foot, particularly where the lean clay is present. Fines content of the glasciolacustrine samples tested range from 50 to 82 percent passing the number 200 sieve size. The deepest known glaciolacustrine silt and clay deposits along the alignment extend down to El.-50. Silt was encountered at El –62 in Boring FD-36 which is approximately 100 feet east of the current alignment.

GLACIOFLUVIAL SAND DEPOSITS: Poorly graded sands with various amounts of gravel underlay the marine sand and glaciolacustrine deposits. These sands are most likely glaciofluvial in origin. They are generally loose to medium dense and have less than 10% silt. Where the glaciofluvial soils are gravely and contain cobbles, they are often indistinguishable from glacial till deposits.

GLACIAL TILL: Glacial till was encountered below the glaciofluvial deposits in some of offshore borings. Glacial till was only clearly identified in a few of the offshore borings on the basis of increased silt and gravel content over the glaciofluvial sand deposits. In some cases, a "washed" glacial till deposit was suspected, i.e. a till leached of much of its original silt content.

BEDROCK: Bedrock was cored in almost all of the offshore borings. The rock consists of slightly to very weathered gneiss. RQD, noted on the boring logs, ranged from very poor to very good. In some locations the bedrock surface is highly fractured or overlain by boulders. Core barrel drops of up to one foot, were noted on a number of boring logs. A core barrel drop occurs when the core barrel exits a boulder or encounters a fracture in the bedrock. The occurrence of boulders and fractured bedrock in some of the early borings precipitated a change in the length of the rock core runs to 20 feet from 10 feet.

2.2.4.2. <u>Onshore Soil Deposits</u>. The onshore section of the CDF "D" site between Herman Melville Boulevard and the existing shore line was originally part of the harbor but was filled in the 1970's. The onshore soil deposits are similar to the offshore deposits and are described in full in Appendix A. A short history land creation on site and a description of the fill stratum (which was found on land only) is presented below.

2.2.4.2.1 <u>History Of Site Filling</u>: Historic USGS topographic maps show that the original colonial shoreline was more than 1000 feet west of its current location in the CDF-D area. An 1885 map shows the shoreline to the east of the currently existing railroad mainline. A 1949 map shows the railroad yard constructed on filled land with the shoreline at the edge of the railroad yard.

A plan dated June 1966 accompanying a petition for land filling shows the shoreline at the edge of a "proposed 60 foot access road" that is now Herman Melville Boulevard. This plan shows the proposed filling of southern two thirds of the land area of the current CDF "D". The filling was to take place by constructing a series of containment dikes and then filling the contained area. The existing 24-inch diameter and 72-inch wide combined sewers were to be extended out to the new shoreline. Obviously the CSO extension was not implemented because the current site has three channels cut through it. The proposed shoreline on the June 1966 plan looks like it is at approximately the location of the current shoreline. The plan does not show proposed filling the northern third of the CDF "D" land area. The CSO currently running along Hervey Tichon Avenue apparently was rerouted to it current location from the CDF "D" site as part of the work proposed in June 1966.

Conversations at the site with a workman at the Packer property (south side of the site) suggest that dredge material was buried in a lined pit on the property when the anchored sheet pile bulkhead wall was constructed in that area. A review of city records will be included in the Geotechnical Data Report, to be prepared separately.

2.2.4.2.2 Urban Fill. The surficial fill strata in the onshore borings ranges from 8 to 23 feet in thickness. The fill is mostly granular in nature and consists of poorly graded sands with varying amounts of silts and gravels. Other components include brick, concrete, wire, and wood. In at least one instance, organic soils were encountered in the fill stratum, which suggests a mixing of both natural and fill materials when the fill was placed. The bottom of the fill appears to be at between El. 0 ft NGVD and El. –5 ft NGVD, with the exception of boring FD-23 where the fill extends to El. -13 ft NGVD. Note that distinguishing whether a granular fill deposit is fill or a natural deposit can be very difficult if the soil does not have man made artifacts mixed in with it. The fill, therefore, could extend deeper than the boring logs suggest.

Many obstructions were encountered in the fill. The drill rig was able to advance through these obstructions with some difficulty. The obstructions could be construction debris, buried riprap, or waste rock. In some areas the obstructions appeared to be nested.

2.2.4.2.3 Environmental Contamination In The Fill. Mention is made on the boring logs of possible environmental contamination in the fill in the following borings:

Boring	Depth	Description
FD-19	9 feet	Oily odor
FD-19	14 feet	Oily odor
FD-22	14 feet	Heavy sheen, strong petroleum odor, black
FD-23	14.5 feet	Asphalt like odor, oily sheen, black, PID = 36 ppm
CSO-D1	4 feet	Trace asphalt
CSO-D1	6 feet	Trace asphalt
CSO-D1	10 feet	Noticeable petroleum odor

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2.3. Development of CDF Configuration.

2.3.1. Bulkhead Design.

The primary structural feature of CDF-D is the cellular sheet pile bulkhead comprising the North, South, and East walls of the CDF. The main function of the bulkhead is to contain contaminated organic dredge material and to support the final CDF cap. The bulkhead must also support all loads associated with the proposed marine terminal facility, and shall accommodate a ship berth directly adjacent to the structure dredged down to El. -30 ft MLLW (-31.4 ft NGVD). The final top elevation of the bulkhead will be at +10.5 ft NGVD. The top of the cap for CDF "D" will also be at the final grade of +10.5 ft NGVD. For this design submission, the bulkhead was analyzed for static loading only. Dynamic (seismic) loading will be incorporated into future design efforts.

2.3.1.1. Alternative Wall Types. A study was undertaken in April 1999 to identify feasible sheet pile wall types for construction of CDF-D, and to recommend the most suitable wall type for implementation in the final design. Four sheet pile wall options were evaluated for strength, stability, and constructability considerations. The study concluded that, given the poor foundation soil conditions and the high lateral loads applied by the contained dredge materials, only a cellular sheet pile bulkhead was feasible for construction of CDF-D. The Feasibility Report was finalized on 10 May 1999, and is reproduced as Appendix B.

2.3.1.2. Cellular Sheet Pile Bulkhead. A cellular sheet pile bulkhead consists of a series of complete circular cells connected by shorter arcs. The area within the circular cells, and between the cells (within the arcs) is then filled with granular fill material. Cellular structures depend on the weight and strength of the cell fill material (granular soils or stone) for stability and strength. The cell fill material may be compacted as necessary to gain the strength required for stability. The flat steel sheet piles only provide confining pressures for the soil (as a tension ring). The primary advantages of circular cells are that each cell is independent of adjacent cells, it can be filled as soon as the piles are driven, and it is easier to form by means of templates. From a constructability perspective, we have determined that an 88' diameter cell is the maximum that could reasonably be constructed. This conclusion was drawn from the experience of other Corps Districts and experts at the Corps Waterways Experiment Station.

2.3.1.3. Bulkhead Loading /Lateral Earth Pressures. It is envisioned that the cellular bulkhead structure will be subjected to a number of load conditions during its service life. Of the following load cases, the 'End of Construction' Case (Load Case D) has been assumed, for now, to control the design. It has been assumed that the loading in Cases A through C listed below can be restricted as required so that they do not control the design. If it is desired to accelerate consolidation more than can be accommodated based on these restrictions, then the calculated bulkhead size based on the 'End of Construction' Case (Load Case D), may need to be increased. Seismic loading information has not yet been developed. It may be necessary to modify the bulkhead design based on the seismic analysis.

- Load Case A - Dewatering. After the cellular bulkhead and cutoff wall have been constructed and prior to dredge slurry placement, the interior of the CDF will be de-

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watered to the extent possible so that the existing clean seawater will not be mixed with the contaminated slurry and require processing at the water treatment plant. This is an incidental load case and the maximum drawdown will be selected so as to not control the design. See Figure S-1A

- Load Case B - Maximum CDF Filling. The maximum height of filling will largely determine the short term storage capacity of the CDF. In this load case the undrained shear strength of the dredge material is zero and no surcharge load has yet been applied. Currently it is assumed that the maximum height of filling is 0.5 feet below the top of the bulkhead. See Figure S-1B

- Load Case C - Maximum Surcharge Loading In order to accelerate consolidation and improve dredge fill bearing capacity, it will be necessary to pre-load the dredge fill material. Currently the City of New Bedford is requesting that a proposed port facility be designed for a uniform surcharge load of 800 psf. To reduce long term consolidation settlement to near zero, the required pre-load is normally in excess of the desired service load. See Figure S-1C.

- Load Case D - End of Construction Loading. Consolidated organic fill (contaminated dredge material) is assumed to have achieved an undrained shear strength of 400 psf under a 5.5 foot thick soil cap and a uniform surcharge load of 800 psf. Excess pore water pressures are assumed to have fully dissipated under the applied loading. Seismic loading will be applied to this load case for the 90% design. See Figure S-1D

- Load Case E - Long Term Loading. A design issue that has not yet been addressed but needs to be considered is the long-term creep of the backfill soils. Creep of backfill behind retaining walls can increase the lateral earth pressure with time, in some cases effectively reducing the undrained shear strength to zero. Long term creep may or may not be an important factor for this design and will be addressed for the 60% on-board review. Seismic loading will be applied to this load case for the 90% design.

2.3.1.4. Bulkhead Analysis and Design. Analysis and design of the cellular bulkhead is performed in accordance with EM 1110-2-2503, and considering input from the numerous applicable references listed in Section 2.1.1. For this report, only static load cases were considered. The computer program CCELL (Ref. Technical Report ITL 97-1) was used to analyze stability of the various cell sizes and load case combinations.

In accordance with EM 1110-2-2503, the cellular sheet pile bulkhead was checked for both internal and external stability. Internal stability refers to the strength of the cellular structure (cell fill material) to withstand the design loads without excessive deformations or complete failure. Checks for internal stability include the following potential failure mechanisms:

- Bursting of Cell Interlocks
- Vertical Slip Along the Center Plane of the Cell Fill
- Horizontal Shear in the Cell Fill

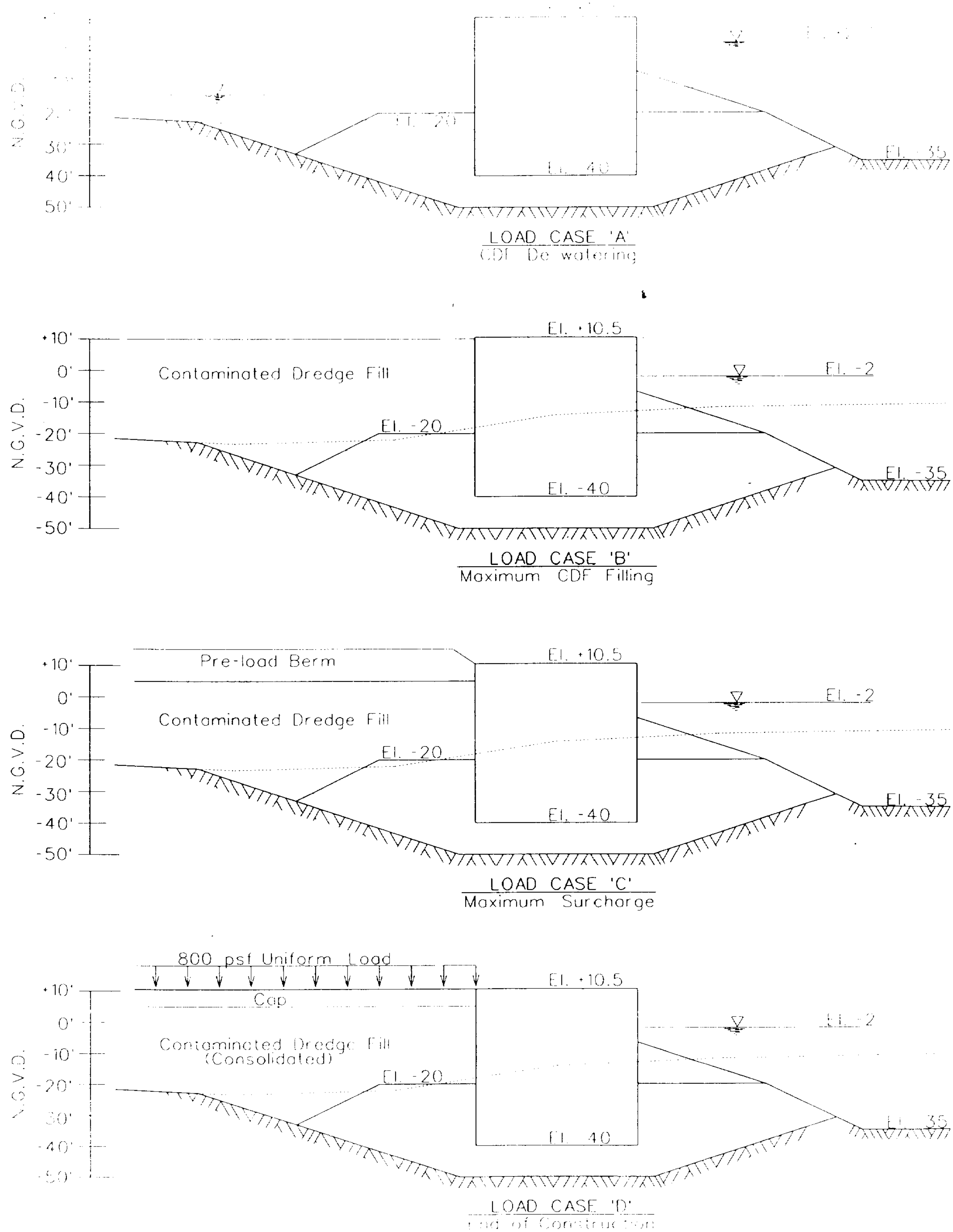

Figure S-1, Load Cases

- Pull-Out of the Loaded Side Sheeting
- Penetration of the Unloaded Side Sheeting
- External stability refers to the ability of the size and weight of the cell structure in combination with the resisting capacity of the surrounding soils to counteract the design loads without excessive deformations or complete failure. Checks for external stability include the following potential failure mechanisms:
    - Bearing Capacity of the Foundation Materials
    - Sliding of the Cell Structure

Design criteria as it relates to appropriate factors of safety for each potential mode of failure is contained in Table 4-4 of EM 1110-2-2503.

2.3.1.4.1 Geotechnical Concerns. The in-situ soils consist of organics, silts, clays, and poorly graded sands. The presence of these materials below and on the resisting side of the structure create sliding and bearing problems, weakening the structure intolerably.

To counter the effects of these weak in-situ soils, three methods were considered. The first is to remove weak soils and replace them (as necessary) with engineered fills. The second is to improve those soils in place. However, improvement methods for silts and clays (such as stone columns) are quite expensive and quality assurance would prove a challenge. The organics are essentially unimprovable and, in all cases, will need to be removed from within/beneath the structure. The third method is to construct berms in front of and behind the bulkhead to decrease loading on the wall and to increase the passive resistance in front of the bulkhead.

The bedrock surface varies greatly in elevation throughout the site, ranging in elevation from –40 ft NGVD to as deep as –90 ft NGVD. In some locations the slope of the bedrock surface is as steep as 1V:3H. The design team has not yet been able to identify any documented cases of cellular structures founded on irregular sloping rock foundations. Major concerns with these conditions include: the increased potential to drive the sheets out of interlock, and the loss of 'ring beam' effect at the pile tips. These concerns coupled with the lack of experience with cellular structures on steeply sloped rock foundations have lead to the decision not to found the cells on rock (even where it is shallow) but to analyze the structures as on a soil foundation. This decision, however, creates other problems in the area of sliding stability, particularly where the bedrock is high.

2.3.1.5. Optional Bulkhead Configurations. Numerous bulkhead configurations have been analyzed with varying cell heights, diameters, berm configurations, cell fill material strengths, improved in-situ material strengths, and combinations thereof. Given the composition and strength of the existing soils, and the methods available to address these concerns, the following optional bulkhead configurations are developed and are considered adequate considering static loading only:

1) Pre-dredge all of the organics and any silts above El. –30. After driving piles down to El. –50 (unless limited by bedrock elevation), excavate silts from within the cell down to El. –40. Construct a berm on both the active and passive sides of the cellular bulkhead.

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This option would require an 88' diameter cell, and all cell fill material would need to be compacted. This option minimizes the quantity of pre-dredging. See Figure S-2.

2). Pre-dredge all of the organics and silts down to suitable material. Replace the pre-dredged material with suitable granular fill and compact. Drive piles down to El. –40 and fill with compacted granular materials. Construct a berm on both the active and passive sides of the bulkhead. This option would require a 50' diameter cell. This option requires a significant amount of pre-dredging. See Figure S-3.

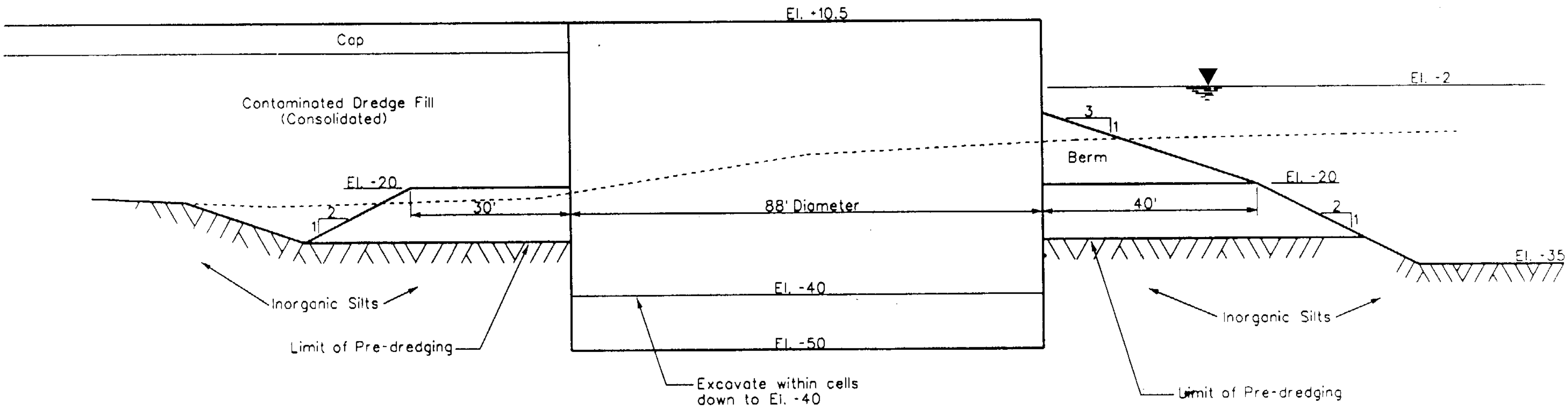
3) Pre-dredge all of the organics and silts down to suitable material. Replace the pre-dredged material with suitable granular fill and compact. Drive piles down to El. –50 (unless limited by bedrock elevation) and fill with compacted granular materials. This option would require an 88' diameter cell. No berms are constructed. This option requires a significant amount of pre-dredging. See Figure S-4.

Using the computer program CCELL, the 3 optional bulkhead configurations presented above have been analyzed for the static load cases B, C & D. These bulkhead configurations were analyzed considering the bedrock at shallow depth (El. –40 ft NGVD) which limited the depth to which piles could be driven, and where rock is deeper (deeper than El. –50ft NGVD). It should be noted that configuration 2 only requires the piles to be driven to El. –40 ft NGVD and therefore the location of the bedrock surface becomes unimportant for that configuration. The resulting factors of safety computed for each potential mode of failure are listed in Table 2.

Table 2 - CCELL Computed Factors of Safety

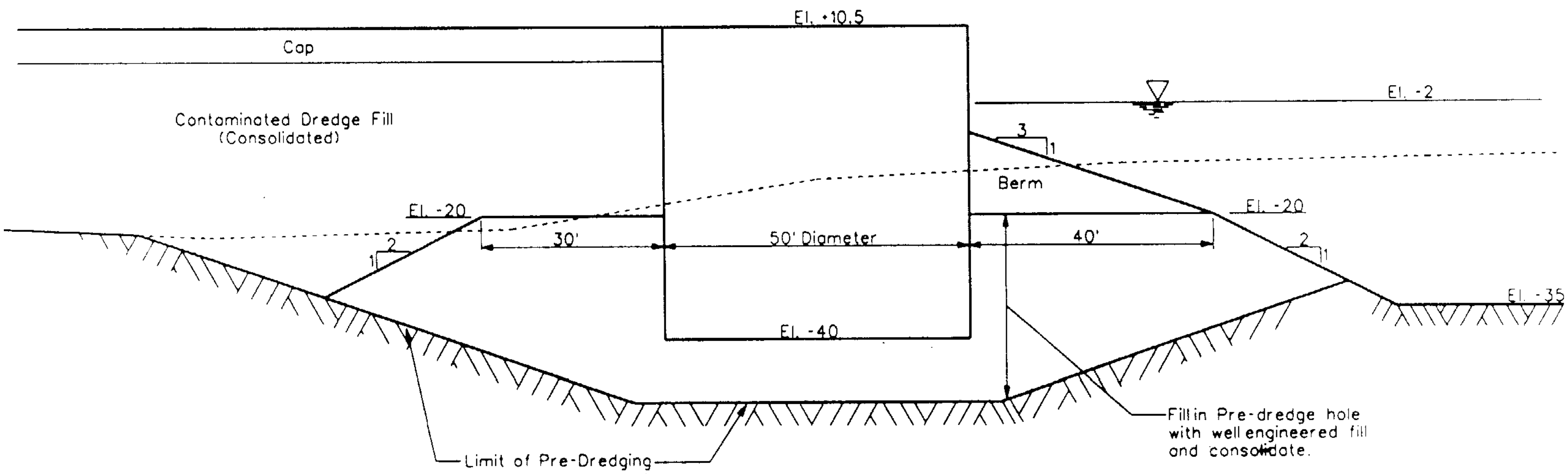
		CCELL Computed Factors of Safety						
Bulkhead Configuration	Load Case	Bursting	Vert. Slip	Horiz. Shear	Pull-out	Penetration	Bearing	Sliding
1	B	2.77	1.53	4.98	1.29	2.51	*	2.56
	C						*	
	D	3.06	1.57	4.43	1.35	2.61	*	2.20
2	B	6.18	2.02	4.50	1.87	14.99	*	2.76
	C	6.97	1.62	3.66	1.54	14.85	*	2.58
	D	7.06	2.11	4.77	2.12	14.26	*	2.70
3	B	1.84	1.13	1.63	0.92	1.55	*	2.21
	C						*	
	D	1.84	1.59	2.30	1.41	1.55	*	2.49
Required Safety Factor For 'Normal' Loading, per EM 1110-2-2503		2.0	1.5	1.5	1.5	1.5	2.0	1.5

* Bearing factors of safety were not calculated for this report due to insufficient information regarding the bearing capacity of the foundation materials.



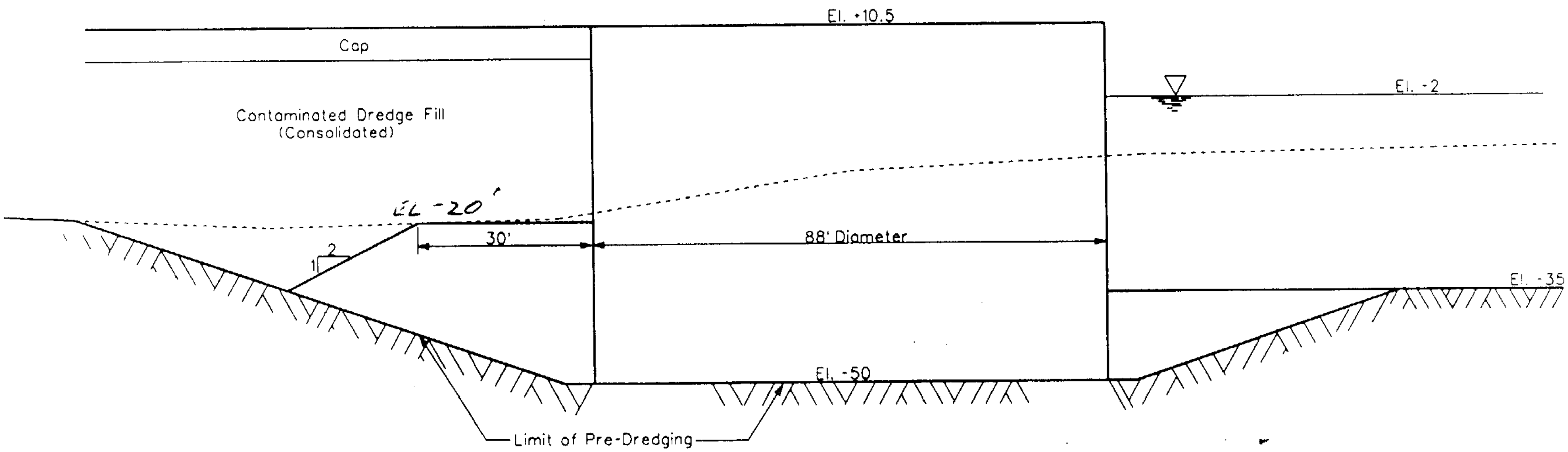
BULKHEAD CONFIGURATION * 1
CDF-D Plan A

Figure S-2



BULKHEAD CONFIGURATION # 2
CDF-D Plan A

Figure S-3



BULKHEAD CONFIGURATION * 3
CDF-D Plan A

Figure S-4

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The factors of safety against 'Pull-Out' for bulkhead configurations 1 and 3 as presented in Table 2 are below what is prescribed in EM 1110-2-2503. Additionally, the safety factors against 'Penetration' for sections where the bedrock is high were well below the prescribed value of 1.5 in the EM. However, Parts VII and VIII of Technical Report ITL 87-5 indicate that no failures by the Pull-Out or Penetration modes have been reported in model studies or in the field, and that the need to design against them "cannot be well established".

2.3.1.6. Conclusion. Each of the three bulkhead configurations will perform acceptably under static loading. However, they each have characteristics which impact constructability, construction cost, and restrictions on future use. These impacts must be evaluated before the best bulkhead configuration can be decided upon. Table 3 presents a summary comparing the advantages and disadvantages of each.

Table 3 - Bulkhead Configurations

Bulkhead Configuration	**Advantages**	**Disadvantages**
1	• Minimizes the quantity of pre-dredging • City's crane system will need to be located on a pier in front of the bulkhead. This keeps the two structures separated which is beneficial when considering seismic performance. The pier will also serve to protect the bulkhead from vessel impact.	• Large diameter (88') cells are required. • Water level within the cells will need to be held at El. +10' NGVD to accommodate excavation within the cells down to –40'. • Berm in front of the bulkhead will require the City to build a pier in order to have a deep draft (-30 ft MLLW) berth. • This configuration only marginally passes the factor of safety for vertical slip in the cell fill material. • In-situ silts below El. –30 ft will need to reach the drained condition before the bulkhead can be loaded. • It is likely that liquefaction will be a major concern when seismic loading is introduced. This could force the design into removing all of the silts (configurations 2 or 3).
2	• Small diameter (50') cells • City's crane system will need to be located on a pier in front of the bulkhead. This keeps the two structures separated which is beneficial when considering seismic performance. The pier will also serve to protect the bulkhead from vessel impact. • This option has the greatest potential to withstand seismic loading. • This option will be adequate where the	• Large quantity of pre-dredging is required. • The pre-dredged area will need to be filled in with well engineered granular material and will need to be compacted. • Berm in front of the bulkhead will require the City to build a pier in order to have a deep draft (-30 ft MLLW) berth.

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Table 3 - Bulkhead Configurations

Bulkhead Configuration	Advantages	Disadvantages
	bedrock surface is up to El. –40' NGVD.	
3	• No berm in front of the bulkhead will allow for a deep draft (-30' MLLW) berth adjacent to the bulkhead.	• Large diameter (88') cells are required. • Large quantity of pre-dredging is required. • The pre-dredged area will need to be filled in with well engineered granular material and will need to be compacted. • This configuration only marginally passes the factor of safety for vertical slip in the cell fill material.

Of the three options, bulkhead configuration number 2 appears most promising. The major obstacle with this option is the quantity of foundation pre-dredging required, and the amount of compaction of new soil material that will need to take place.

2.3.1.7. Corrosion Protection. The steel sheet pile bulkhead will be designed for a 100 year life. The following corrosion protection methods are being considered to prevent the steel sheet pile cells from corroding:

- Marine grade steel
- Thicker sheet pile sections
- Epoxy coatings (coal-tar or other)
- Impressed Cathodic protection

The design team is planning to contract the corrosion protection design to the Corps of Engineers' Construction Engineering Research Laboratory (CERL).

2.3.2. Other Structural Features.

In addition to the cellular sheet pile bulkhead, there are several other structural features required for construction of CDF-D. They are: the shoreline walls comprising the west limit of the CDF, the interior CDF compartment walls, and the cap on top of the cellular bulkhead.

2.3.2.1. Shoreline Walls. The shoreline walls on the west side of the CDF will likely consist of a single line of cantilever sheet piles driven into suitable material. These walls have not yet been designed as of the date of this report. The height of retained backfill is anticipated to be very small (approximately 3 to 4 feet). The wall will serve more as a seepage cutoff than a retaining structure. It is anticipated that the alignment of these walls will need to be pre-excavated prior to driving the sheet piles due to the presence of randomly dumped rubble and building debris. The shoreline walls will be designed in accordance with EM 1110-2-2504.

2.3.2.2. Interior CDF Compartment Walls. The interior CDF compartment walls serve to split the CDF into two or three separate compartments. Under this configuration, one compartment may be filled and begin consolidating while the other two compartments are being filled. Compartmentalizing the CDF will help to minimize the amount of down time for the dredge. The interior CDF compartment walls have not yet been designed. The loading on these walls will be temporary and will consist of one compartment being filled to a maximum elevation (not yet determined) with the adjacent compartment de-watered. The interior CDF compartment walls will be designed in accordance with EM 1110-2-2504.

2.3.2.3. Cap on Cellular Bulkhead. The cap on the cellular bulkhead will be designed to carry loading commensurate with a marine port facility. It is not anticipated that the cap will carry a crane system due to the berm in front of the bulkhead necessitating the construction of a pier to support the crane system closer to deep water. The concrete cap will be designed and detailed to accommodate the types of bulkhead movement anticipated.

2.3.3. Seismic Criteria.

All structural features of CDF "D" will be designed in accordance with EPA guidance (Ref. EPA/600/R-95/051) and Massachusetts DEP guidance (Ref. 310 CMR 19.000) as well as the Corps of Engineers guidance contained in ER 1110-2-1806. The cellular bulkhead and other permanent structural features of CDF "D" will be designed using the two earthquake ground motions as defined in section 2.3.3.1. below. Each structural feature will be assigned performance objectives, as outlined in section 2.3.3.2. below, in association with each design ground motion. Seismic loading will be applied to load cases D & E as defined above.

2.3.3.1. Design Earthquakes and Ground Motions. For this design report, the seismic ground motions are defined only. The actual ground motions (Peak Ground Acceleration, Velocity, Displacement, etc.) for each event will be determined prior to the 60% on board review and will be incorporated into the 90% design of CDF-D. At this point it is not anticipated that a site-specific seismic study will be required. However, this could change if it is determined that seismic loading controls the design (Ref. ER 1110-2-1806.5.h.2).

2.3.3.1.1 Maximum Design Earthquake (MDE): The MDE, as defined in ER 1110-2-1806, is the maximum level of ground motion for which a structure is designed or evaluated. Both the EPA guidance (Ref. EPA/600/R-95/051 Part 258.14) and the MA-DEP guidance (Ref. 310 CMR 19.038.2.c.6) require containment structures to be designed to resist the maximum horizontal acceleration associated with a seismic event having a 10% probability of being exceeded in 250 years. This corresponds to a 2,373 year return period.

In reference to ER 1110-2-1806 table B-1, because the MDE is less than the maximum Credible Earthquake, the conclusion is drawn that EPA and MA-DEP have determined that the seismic hazard potential for solid waste landfill facilities is 'Significant' but not 'High'. For design purposes, the MDE is considered an 'Extreme' loading condition, and appropriate safety factors will be applied.

2.3.3.1.2 Operating Basis Earthquake (OBE). The OBE, as defined in ER 1110-2-1806, is an earthquake that can reasonably be expected to occur (50% probability of occurring) within the service life of the project (100 year design life). This corresponds to a return period of 144

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years. For design purposes, the OBE is considered an 'Unusual' loading condition, and appropriate safety factors will be applied.

2.3.3.2. Seismic Performance Objectives. Each permanent structural feature will be designed to meet specific performance objectives associated with each design event. For instance, structures should essentially respond elastically to the OBE ground motions with no disruption to service. Conversely, structures may be allowed to respond inelastically to the MDE ground motions which may result in structural damage and limited disruption to services, but the structure should not collapse or endanger lives. Table 4 lists Seismic Performance Objectives for each major structural feature of CDF-D with regards to the MDE and OBE ground motions.

Table 4 - Seismic Performance Objectives

Feature:	OBE Ground Motions	MDE Ground Motions
Cellular Bulkhead	Minor displacements will be allowed. The cellular structure shall respond elastically, with no permanent damage. Overall stability shall be maintained.	The cellular structure must maintain containment of the contaminated sediments. Large displacements will be allowed so long as overturning, sliding, or bearing failure does not occur.
Cap on Cellular Bulkhead	Failure of the concrete cap in localized areas due to bulkhead displacements will be allowed.	Complete failure of the concrete cap due to bulkhead displacements will be allowed. The cap should be fairly easy to replace if necessary.
Shoreline Walls	Minor displacements will be allowed. The shoreline walls shall respond elastically, with no permanent damage. Overall stability shall be maintained.	The shoreline walls must maintain containment of the contaminated sediments. Large displacements will be allowed so long as overturning, sliding, or bearing failure does not occur.
Interior CDF Compartment Walls	The interior CDF compartment walls are considered temporary and will not be designed in accordance with seismic criteria.	The interior CDF compartment walls are considered temporary and will not be designed in accordance with seismic criteria.

The seismic performance objectives listed in table 4 above consider only the containment function of those CDF features. It is likely that the proposed marine terminal facility structures (crane, buildings, railway, etc.) will require more stringent seismic performance criteria in terms of allowable displacements. It must be understood that the placement of certain features on or behind the cellular bulkhead could force the design into more stringent performance objectives, and thus increase the cost. For example, if a crane system for offloading container ships were to be located on top of the cellular bulkhead, then displacements of the bulkhead due to MDE ground motions would likely need to be limited to avoid irreparable damage to the crane system.

2.3.4. Hydraulic Considerations. The Corps of Engineers developed a two dimensional numerical hydrodynamic and transport model (RMA-2V) for New Bedford Harbor during the Engineering Feasibility Study (EFS). The New England District (NAE) with WES support has reactivated the model on a PC computer under the Surface Water Modeling System (SMS), and will use it to make hydrodynamic predictions of currents near shoreline structures. NAE will

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then make further modifications to the model to evaluate various engineering plans and their impact on water levels, circulation patterns, and resulting currents within the harbor. Preliminary analyses have been completed with this model to aid design efforts. These analyses are present in a report titled "New Bedford Harbor Hydrodynamic Modeling", April 2000. As stated in the report the results should be considered preliminary and additional simulations and analyses will be conducted as design efforts proceed.

2.3.5. Foundation Design Alternatives.

2.3.5.1. Geologic Cross Sections.

The bulkhead loading cases representing the different stages of construction are described in Section 2.3.1.3, and bulkhead configurations are described in Section 2.3.1.5 above. Using Load Case D, the End of Construction Case, as a presumed worst case loading in combination with Bulkhead Configuration 1, analysis of bulkhead construction at three different geologic cross sections was undertaken. The geologic cross sections selected represent variations in, existing harbor bottom profiles along the alignment, in bedrock elevation (and slope), and in differing overburden soil conditions. Figures A-13, A-14, and A-15, show the assumed overburden profiles and the selected engineering soil properties for each profile.

Bulkhead Configurations 2 and 3 discussed in Section 2.3.1.5 assume the pre-excavation of unsuitable materials and the replacement of these materials with granular fill. Both the existing granular fluvial deposits and placed fill are assumed to be densified by vibrocompaction or another deep compaction technique. Since over-excavation creates a more uniform geologic cross section, only one soil profile has been assumed for analyses of these bulkhead options. The bulkhead sheeting is driven to a uniform elevation above the highest bedrock. The only difference between Configurations 2 and 3 is the presence of the exterior berm. Figures A-16 show the selected engineering properties for these sections.

2.3.5.2. Soil Improvement. The fluvial deposits present at the site are sufficiently loose to warrant concern that they could liquefy during a seismic event. The design is not yet far enough along yet to determine how serious a threat of liquefaction exists and if soil improvement will be necessary. Of the soil improvement techniques available, vibroflotation, Terra Probe, or resonance compaction techniques appear to be the most cost-effective of the methods available. The gradation of the fluvial deposits compares well with the typical range of gradations for which these methods of soil densification have been used successfully. Soil improvement of the cell and berm fill is considered desirable and cost effective since densification improves the strength and load carrying capacity of the bulkhead, resulting in a smaller bulkhead cross section. The zone of soil improvement of the existing fluvial deposits and the granular fill berm is approximately 110 feet wide in cross section and would extend from the top of the berm to bedrock. The fill within the cellular bulkhead would also be improved.

2.3.5.3. Lateral Earth Pressures. The driving forces acting on the completed cellular bulkhead include:

- Differential hydrostatic pressure;

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- Lateral load from a very weak consolidating organic clay dredge material;
- Vertical surcharge load of the soil cap, or post construction surcharge which results in even higher lateral loads in the organic clay backfill.

Basic to calculation of the bulkhead loading is the assessment of the strength and unit weight of the consolidated dredge fill at each construction stage. The clay strength (S_u), after excess pore water pressure dissipation has been assumed to be 0.2 times the effective vertical stress, which for Load Case D is 400 PSF. In Load Case C (Maximum surcharge loading) it is assumed that the strength gain during consolidation is offset by excess pore water pressures and the net result is $S_u = 0$. In a condition of zero shear strength, the clay essentially acts as heavy water and exerts full hydrostatic lateral pressure on the wall. The assumed unit weight of the dredge fill is therefore very important. This unit weight varies as consolidation occurs and has been assumed to be 110 pcf for Load Case D.

2.3.6. Cutoff Wall.

It is likely that some type of cutoff wall will be required, however, the design is not far enough along at this point to make any recommendation. The issues related to cutoff wall design are summarized below:

1) Soil Profile. The sand and silty sand marine sand deposits, interbedded, in some locations, with glaciolacustrine silt and clay deposits, currently exist below the organic soils. Glaciofluvial sands underlie the marine sand and glaciolacustrine deposits. While the marine deposits are of moderately high permeability, the underlying glaciofluvial sands are of very high permeability.

2) Incomplete Organic Clay Bottom Liner. The assumptions on which the original liner design was based are not valid. As originally proposed, the existing organic stratum would provide a bottom liner for the CDF and the sides of the CDF would be lined with a geomembrane down to the top of the organic stratum. However, two borings along the bulkhead alignment did not encounter organic clay at the mudline and two other borings encountered only very thin organic clay layers on the harbor bottom. Large areas with little or no organic clay at the surface are suspected along the southern wall of the bulkhead and in the northwestern corner of the site (after the sunken barges are removed). The proposed pre-dredging will excavate not only the organic stratum but marine and glaciolacustrine stratum as well. These soils will be removed within 30 to 50 feet of the inside face of the bulkhead and will be replaced with free draining granular fill. The organic soil in the passive berm area will also be removed due to berm construction or future dredging.

3) Geomembrane Liner. Placing a geomembrane liner on the curved surface of a cellular bulkhead is considered impractical.

4) Interlock Sealant. The steel bulkhead sheeting will have ball and socket type interlocks. This type of interlock will likely have a relatively high permeability even when the interlocks are in tension. The possibility of using a coal tar epoxy or swelling interlock sealant was discussed with a sheeting distributor. The use of sealant was not recommended because of

the time it takes to drive a cellular bulkhead and because of driveability concerns. The bulkhead itself will be filled with free draining soils.

5) Stability. The cellular bulkhead calculations conducted to date have assumed a no seepage condition. If there is a high seepage exit gradient on the harbor side of bulkhead, this will tend to reduce the passive resistance and will have to be taken into account in the calculations.

6) Fate and Transport Modeling. The need for a bulkhead cutoff wall will be assessed on the basis of bulkhead stability and fate and transport modeling of contaminants.

7) Driving Bulkhead Sheets to Rock. With the currently proposed alignment, the average distance below El. –40 ft NGVD to the top of rock is approximately 25 ft. The deepest distance between El. –40 ft NGVD and rock is 45 ft. The new alignment was chosen to stay clear as possible of deep rock areas and areas that may have boulders. The sheeting could be shop welded together in pairs to reduce permeability of the interlocks by one half. Driving to rock could be problematic and does not address the permeability issue of the upper bulkhead. However it will be explored as one of the cutoff wall possibilities.

8) Straight Steel Sheet Pile Wall. A cut-off wall consisting of a straight Z section steel sheet pile wall driven adjacent to the inside face of the cellular bulkhead structure is considered a possibility. The sheeting would be shop welded together in pairs prior to driving and would use a coal tar epoxy sealant in the non-welded interlocks. The wall would have to be anchored to the cellular bulkhead at about El.-2 ft NGVD for stability. The space between the wall and the cellular bulkhead could be filled with concrete in lifts to create a relatively impermeable boundary against the bulkhead between the top of the inside berm and the top of the CDF. The wall would be driven to refusal on rock. The walls Z shaped sheets would drive relatively straight through hard driving conditions which is an advantage over driving the flat section bulkhead sheets.

9) Jet Grout Soil Cement or Slurry/Concrete Wall. Construction of a Jet Grout or Slurry wall through the center of the bulkhead is considered the option that will provide the greatest hydraulic barrier to seepage.

10) Bedrock Grouting. It may be necessary to grout the bedrock below a cutoff wall if there is excessive seepage through the rock. It is likely that many areas will require no grouting while other areas will require extensive grouting. Since it may not be necessary to have a leak proof cutoff wall, bedrock grouting may not be required at all.

2.3.7. Pre-Construction Dredging of CDF Footprint.

2.3.7.1. Total Removal Option: Pre-Construction Dredging for removal of organic clay and inorganic clay and silt along the bulkhead alignment should occur in the following sequence. This corresponds to bulkhead configuration 2 as discussed in Section 2.3.1.5:

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- Remove and de-sand the contaminated sediments from the dredge area along the alignment. Dispose of these sediments in CDF-C or an offsite landfill.
- Dredge the "clean" organic stratum material. Dredge to approximately El. –30 ft NGVD.
- Dredge to either El. -40 ft NGVD to El. -50 ft NGVD as required to remove inorganic clay and silt soils.
- Upon confirmation that the require dredge depth had been achieved and that all of the inorganic clays and silts have been removed from the dredge area, backfill the dredge excavation with free draining granular fill to El. –20 ft NGVD.

2.3.7.2. Partial Removal Option - The following sequence outlines the process required to remove the organic stratum and some of the inorganic clays and silts from within the cellular bulkhead. This corresponds to bulkhead configuration 1 as discussed in Section 2.3.1.5:

- Remove and de-sand the contaminated sediments from the dredge area along the alignment. Dispose of these sediments in CDF-C or an offsite landfill.
- Dredge the "clean" organic stratum down to El. -30.
- Install the cellular bulkhead. With the inside of the constructed cell filled to the top with water to equalized pressure, excavate within the cell to remove the inorganic silt and clay down to El. -40. Fill the bulkhead with granular cell fill.
- Excavate to the bottom of the organic stratum in proposed berm area.

2.3.7.3. Estimated Dredge and Excavation Quantities. Estimated pre-construction dredging excavation volumes for the bulkhead wall are based on a 50 foot diameter cell and the layout shown in the current plan. They do not include any pre-dredging that may be necessary for any sheetpile divider walls within the CDF.

Contaminated organic soil:	16,100 CY	(top 2 feet of organics in contaminated areas)
Assumed Uncontaminated organic soil:	187,600 CY	
Other soils to be removed:	135,000 CY	(SM, ML, CL below organic layer)

Total	338,700 CY	

2.3.7.4. Environmental Sampling/Sediment Chemical Analytical Data. Table 5 below shows sediment sampling data in the vicinity of CDF "D". Data points within a radius of approximately 1000 ft from the limits of the CDF are shown. Sediment samples from three different programs are included.

Currently, no chemical data is available for organics and silts below 4 ft depth. Additional sampling and testing is planned for Summer 2000 to characterize these materials that are planned to be removed.

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Table 5 - New Bedford Harbor Sediment Sampling Data in the Vicinity of CDF "D"

			PCB Contamination (ppm)			
Sample ID	Northing	Easting	Value 0-1ft	Depth 1-2ft	Depth 2-3ft	Value 3-4ft
183	2698596	815002	171.60	137.16	0.01	
311	2698590	815190	0.63	0.00		
af239	2698570	814821	80.60	0.00	0.00	
184	2698399	814705	81.42	0.09	0.02	
312	2698300	815325	0.00	0.00		
af228	2698168	815355.2	5.79	11.80	0.00	
af243	2698164	814747.9	6.00	0.00	37.00	
313	2698035	815700	0.85	0.00		
206416	2698028	814453.8	13.72			
206316	2698028	815603.8	7.92			
af245	2697662	815282.7	8.30	0.00	0.00	
317	2697550	816675	0.75	0.00		
318	2697400	816800	11.76	0.33		
ac308	2697354	814677.4	8.21	0.98		
185	2697304	814999	51.72	83.10	0.02	
af264	2697159	815741.7	15.50	0.00	0.00	
af248	2697156	815286.2	5.10	0.09	0.00	
af247	2697152	814754.7	66.00	0.00	0.00	
af804	2697149	814299.2	118.00	0.00	0.00	
207417	2697032	815028.8	80.53			
207416	2697032	815028.8	89.81			
186	2696806	815195	0.25	0.00	0.00	
af805	2696646	814758.2	94.40	0.00	0.00	
af284	2696643	814302.6	67.00	0.00	0.00	
af286	2696548	815290.3	29.00	0.00	0.00	
af287	2696450	815670.6	40.00	0.00	0.00	
190	2696397	814794	2.71	0.00	0.01	
Af290	2696045	815673.4	44.79	0.60	0.08	
Af314	2696040	814914.2	1.50	0.00	0.00	
207516	2696036	815603.8	14.09			
Af289	2695941	815294.5	0.55	0.00	0.00	
Af393	2695639	815524.3	1.39	0.00	0.00	
Af315	2695636	815068.8	2.60	0.00	0.00	

Note: All samples within an approximate 1000 ft radius of CDF "D" are listed.

Sample ID Legend:

Af239 Feasibility Report

206416 Long Term Monitoring

183 Fall 1999 Sediment Sampling Program

2.3.7.5. Contaminated Material – Handling, Temporary Storage, Disposal. The contaminated organic clay stratum soils should be de-sanded and disposed of in CDF "C" or an off site landfill.

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2.3.7.6. Clean Material – Handling, Disposal, Temporary Storage, Re-Use.

- Handling – It is anticipated that the clean organic material could be dredged using the same dredge equipment proposed for the contaminated dredging, since this equipment will be onsite. Dredging of the inorganic silts and clays may be more economical with a different dredge type.

- Offsite Organic Soil Disposal - For off-site disposal of the "clean" organic clay, the clay must be dewatered to meet paint filter test shipping requirements. The organics will be dewatered by placing the dredge clay in a temporary containment area for air-drying. If environmentally acceptable, supernatant water from the sediments will be discharged directly into the harbor without treatment. A substantial containment area with several internal chambers will be required to dewater the sediments in this fashion. The reduction in volume and weight of the dredge sediments will reduce landfill-tipping fees. As an alternative, the sediment could be placed in a taller containment area in one lift and then allowed to develop a desiccated crust. Installation of wick drains and preloading would then likely be required to dewater the sediments. A second alternative is to use Filter Press technology to dewater the sediments.

- Onsite Organic Soil Disposal - Disposal in the immediate project area, such as at the railroad yard west of Herman Melville Boulevard or Marsh Island across the harbor on the Fairhaven side, would most likely require the construction of a facility with perimeter and interior unlined containment berms. The desanded dredged material would be placed in approximately one-foot lifts into the area and allowed to dry. Trenching techniques could be used to accelerate drying. To avoid delaying the dredging operation, the disposal area would have to be large and have enough separate chambers for effective air-drying. In its final consolidated condition, the desiccated clay should be shallow enough that building foundation excavations could penetrate to more suitable bearing soils if required. It is possible that provided it meets regulatory requirements, some of the organics may be able to be reused to restore intertidal wetlands that will be dredged as part of the remedial action plan.

- Processing of Non-Organic Soils - The in-situ inorganic silt and clay is interbedded with silty and poorly graded sand. As a rough estimate 50 percent of the soil dredged to remove the inorganic silt and clay is fine sand. If the granular soil is separated out from the silt and clay it can be used in the construction of drainage layers for the CDF cap or elsewhere on site. It may even be suitable as berm or cell fill.

2.4. Impacts on Existing Navigation Channel.

The current CDF bulkhead alignment runs through an existing 100 ft wide navigation channel.

2.4.1. Current Channel Traffic usage and Impacts. Additional information TBD.

2.4.2. Future Channel Traffic USACE and Impacts. TBD.