4. RECOMMENDED PLAN

4.1 INTRODUCTION

Chapter 3 described the process for screening Harbor placement options, the selection of a DMCF at Masonville as the preferred option, and the selection of the recommended plan for a facility at Masonville. This chapter describes the recommended Dredged Material Containment Facility (DMCF) at Masonville: the existing site conditions/design criteria, site design, construction plan, and mitigation projects associated with the DMCF.

The costs, quantities, and site characteristics presented in this section may differ from those presented in the Masonville alternatives analysis (Chapter 3). Further studies were completed for the recommended plan, and the changes in costs, quantities, and site characteristics are due to the subsequent findings from the greater level of study.

4.2 OVERVIEW OF RECOMMENDED PLAN

Dredged Material Containment Facility

The recommended plan to meet the immediate Harbor placement need is the preferred alternative from the alternatives analysis (Chapter 3). This alternative (3-C-10) consists of final feasibility alignment (FFA) 3 with a cofferdam instead of a rock dam and an initial dike height of +10 ft MLLW (Figure 4-1). This alternative was selected as the recommended plan based upon the options screening process and the analysis of possible alternatives. Chapter 3 provides a detailed explanation of the recommended plan selection process.

The recommended plan is to construct a DMCF with a 141-acre footprint (affecting 130 acres of existing tidal open water, 10 acres of existing upland area, and 1 acre of existing wetland vegetation) DMCF in the Patapsco River with a containment structure composed of the following structural components, which are described in detail in Section 4.4 and shown in Figure 4-1:

- Beach Dike
- Armored Dike
- Cofferdam
- Onshore Dike

The initial elevation of the containment structure would be +10 feet (ft) mean lower low water (MLLW). A berthing area would be constructed along the cofferdam section.

The project includes the Wet Basin, which is located on the eastern portion of the site. A rock dike would close this area off from the Patapsco River, and the Wet Basin would be used to increase the capacity of the proposed Masonville DMCF. The initial elevation of the rock dike would be +8 ft MLLW. The material for filling the Wet Basin would be excavated from within the Masonville DMCF footprint, and placed into the Wet Basin.

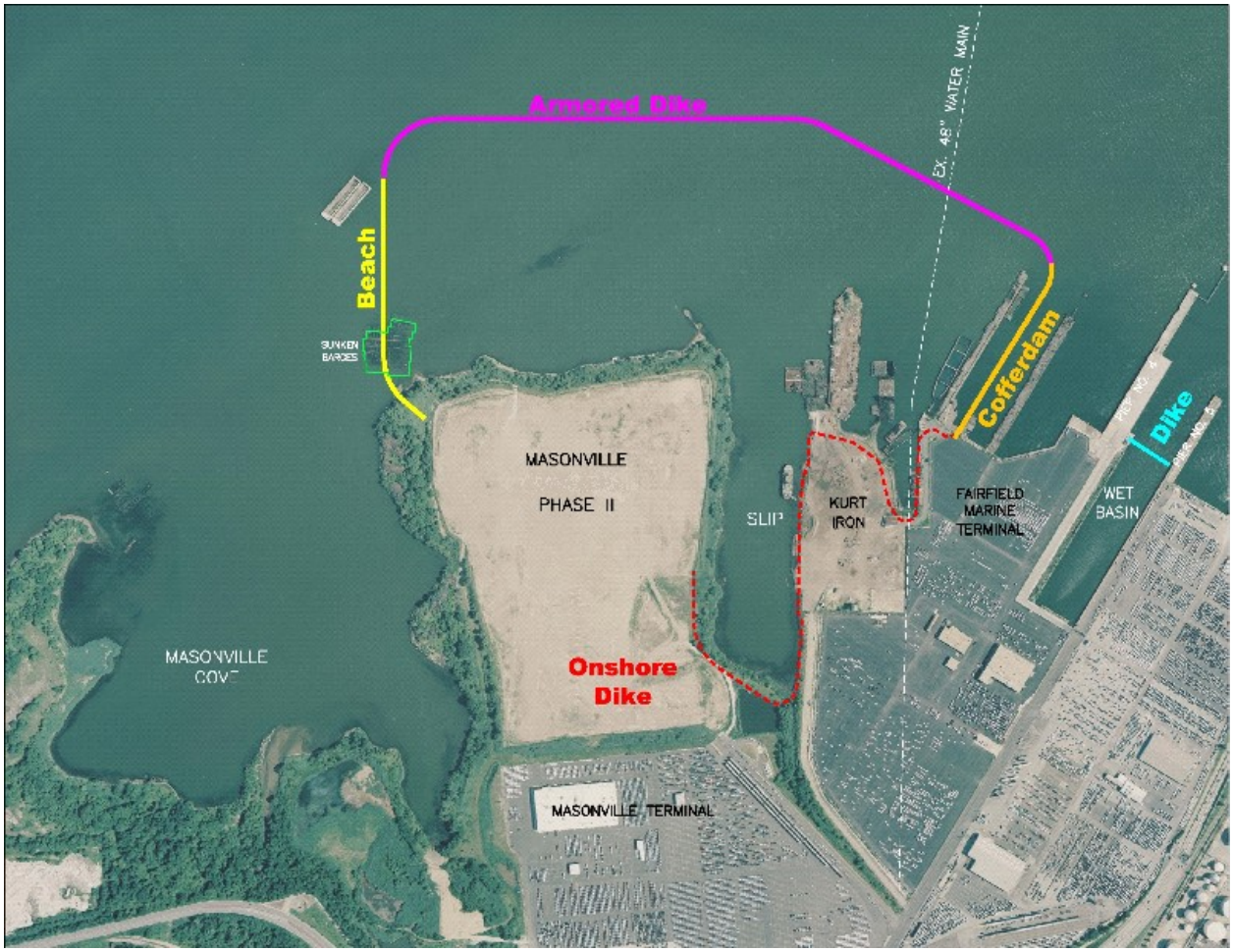


Figure 4-1. Containment Structure Segments of the Recommended Plan (Alternative 3-C-10)

On-site sand and stiff clay, as well as offsite materials, would be used for the construction of the beach and armored dike sections of the containment structure. The use of on-site materials is important to the project because it decreases the cost of obtaining construction materials and increases placement capacity at the site. Overburden material (soft silts and clays) overlying the on-site borrow materials or underlying the footprint of the containment structure would be removed and transported to the Hart-Miller Island (HMI) DMCF for placement prior to mining the sand borrow.

4.2.1 Affected Area

The footprint area is considered to be the area covered by the proposed project, including the dike. The total project footprint is 141 acres. The affected area as per Code of Maryland Regulations (COMAR) is the open water affected at the mean high water (MHW) mark along the proposed structure. The placement facility, including the 6-acre Wet Basin, would encompass approximately 130 acres of open water, the existing and affected upland areas comprise 10 acres of the project footprint, the existing and affected vegetated wetland areas comprise of 1 acre of

the project footprint, therefore, affecting a total of 141 acres from this project. See Section 4.2.2.2 for a more detailed description of the areas used to describe the alignment.

4.2.2 General Site Characteristics

Table 4-1 displays the primary site characteristics for the recommended plan (preferred alternative),including the capacity in million cubic yards (mcy), and the list below describes the characteristics.

Table 4-1. Preferred Alternative 3-C-10 Characteristics

Site Characteristic	Quantity
Dredged Material Placement Capacity (mcy)	16
Anticipated Annual Usage (mcy)	0.5 - 1.0
Effective Area (acres)	101
Footprint Area (acres)	141
Affected Tidal Open Water Area (acres)	130
Affected Upland Area (acres)	10
Affected Wetland Vegetation Area (acres)	1
Site Life (years)	20

4.2.2.1 Dredged Material Placement Capacity, Annual Placement Capacity, and Site Usage

Dredged material placement capacity is defined as the total volume of dredged material (*in-situ* volume) the site can hold when the placed material has reached a steady state of consolidation and the final design surface elevation. This value is calculated using the air space volume available within the site and making assumptions as to the properties of the dredged material placed within the site. Annual placement capacity is the volume of dredged material that can be placed annually within the site, such that the site does not experience overloading conditions. The annual quantity of dredged material placed at the site is anticipated to be approximately 0.5 to 1.0 mcy. This annual volume of placement exceeds the optimum annual placement (annual placement capacity) is driven by a bulked 3 ft lift (USACE 2001b). Exceeding the optimum annual placement would be necessary to accommodate Harbor needs and depending on its extent, may result in decreased total site capacity. The overloading proposed for the proposed Masonville DMCF is not expected to significantly decrease overall site capacity. The average annual site usage is anticipated to be 0.8 mcy, based on current placement projections. It is anticipated that any total capacity reductions due to the projected overloading may be recovered over time through site management techniques and resting (little to no placement) the site for one or more years after future options are brought online. If no other sites are brought online, overloading may cause reductions in the overall site capacity of the proposed Masonville DMCF.

4.2.2.2 Site Area

Two areas are used to describe an alignment. The first is site effective area, which is the average area within the inside slope of the containment dike and is used to determine the average annual capacity of the site. The second is site footprint area, which is the area encompassed within the outer toe of the containment dike and defines the total area impacted by the site (river bottom

and upland areas). The MHW and river bottom areas are also listed in Table 4-1, as MHW is the area used to determine the area of tidal wetland impact and river bottom helps define impact to benthic organisms.

4.2.2.3 Site Life

The life of the site is determined by dividing the *average annual site usage* into the *dredged material placement capacity*. This value is critical for long term planning of dredged material placement.

4.2.3 Mitigation

Mitigation would be required for the filling of 130 acres of tidal open water, affecting 1 acre of vegetated wetlands, and constructing the facility over 10 acres of upland within the Chesapeake Bay Critical Area buffer. Approximately 0.4 acres of SAV would also be affected by the recommended plan. Mitigation projects are beneficial to the environment and the community surrounding Masonville and are an integral part of the overall project. The mitigation projects would focus on improving the area adjacent to the DMCF known as Masonville Cove, along with supplemental projects in other areas. The mitigation projects are described in detail in Chapter 6.

4.2.4 Schedule

It is anticipated that the DMCF could be operational by the Fall of 2008, and that the majority of the mitigation projects could be completed by 2009. Detailed schedules for the construction and implementation of the recommended plan are presented in Chapter 7 and Appendix G.

4.2.5 Project Cost

The current estimated cost for initial construction of the recommended alternative, including pre-dredging but exclusive of demolition/relocation of existing infrastructure and mitigation, is estimated at $54 million. The range of costs for demolition/relocation of existing infrastructure and mitigation is estimated at $29 million, for a total initial cost of $83 million. The total project cost is estimated to be $246 million, which translates to approximately $15 per cubic yard of capacity for 16 mcy of material. It should be noted that $122 million of the total cost ($246 million) is dredging transportation and placement of the dredged material in the DMCF. Approximately $5 to $10 million would be required for the remediation and relocation of derelict vessels. In addition, $12.5 million of the total cost of the project will be required for mitigation. Section 4.10 describes the initial project cost, along with the total costs over the life of the project. Table 4-3 in Section 4.10 provides a breakdown of project costs by line item. A cost analysis is provided in Appendix F.

4.3 EXISTING CONDITIONS AND DESIGN CRITERIA

4.3.1 Hydrodynamics

Information on site hydrology and hydrodynamics not specific to the recommended plan is available in Section 2.1.3.

4.3.2 Geotechnical Conditions

Boring, probing, and vane shear data were collected to determine existing geotechnical conditions and design criteria for the site. Each data set is described and analyzed in this section.

4.3.2.1 Borings

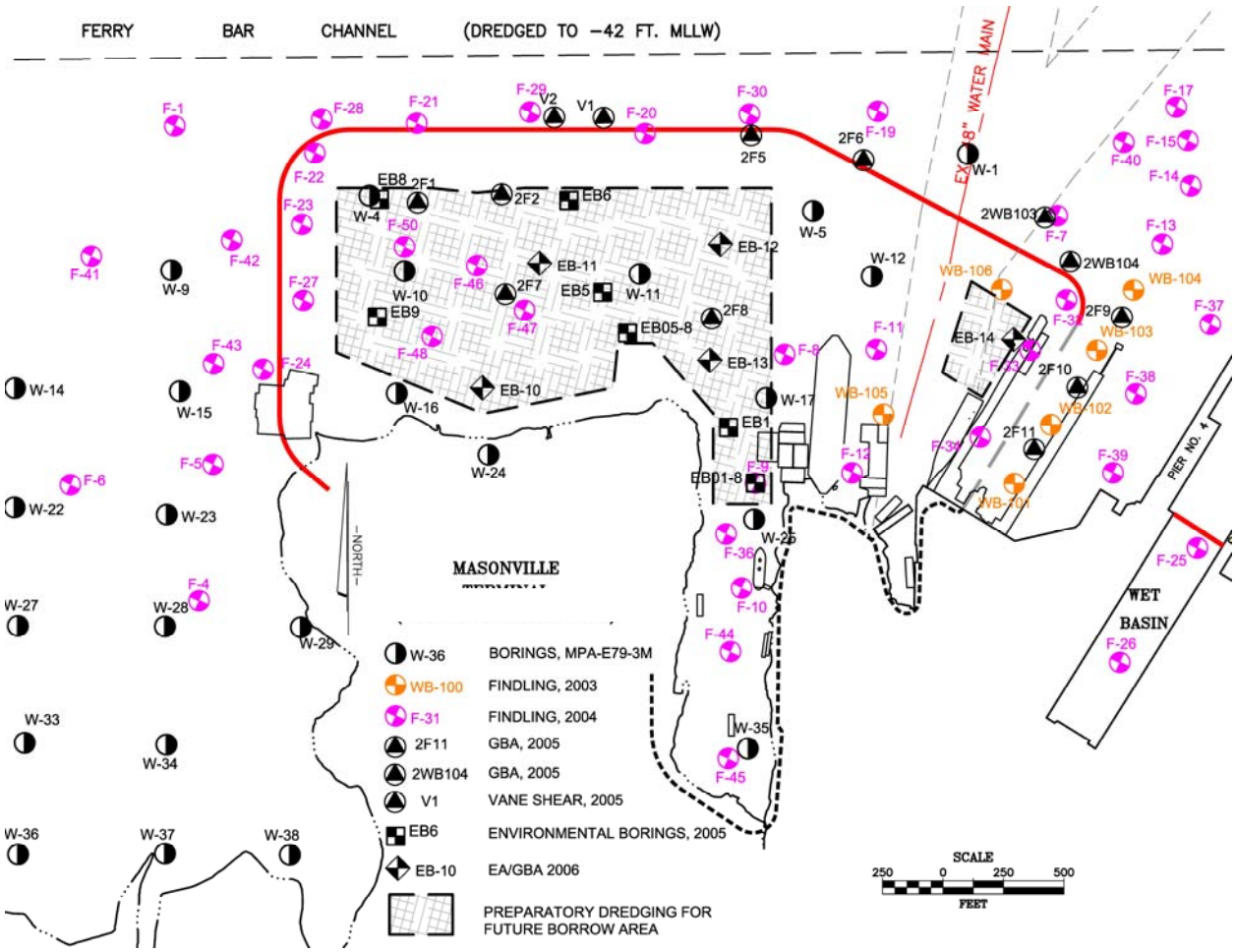
Boring data for this report came from a combination of historical data provided by Maryland Port Administration (MPA) and U.S. Army Corps of Engineers (USACE) – Baltimore District, and from data collected during this study. Figure 4-2 shows the borings evaluated during the conceptual reconnaissance and State feasibility-level study phases.

At each boring location, standard penetration tests were performed; in-situ vane shear tests were performed; and split spoon and Shelby tube samples were collected. Laboratory testing was performed on the collected samples to evaluate their geotechnical design characteristics.

The borings provide material descriptions at discrete depths, allow for the generation of soil profiles and design criteria, and provide a base from which to make quantity estimates. Boring locations are provided in Figure 4-2. Figures 4-3 through 4-5 depict soil profiles along the containment structure alignment generated from the borings displayed in Figure 4-2. Refer to Findling 2005 for a detailed analysis of the geotechnical data.

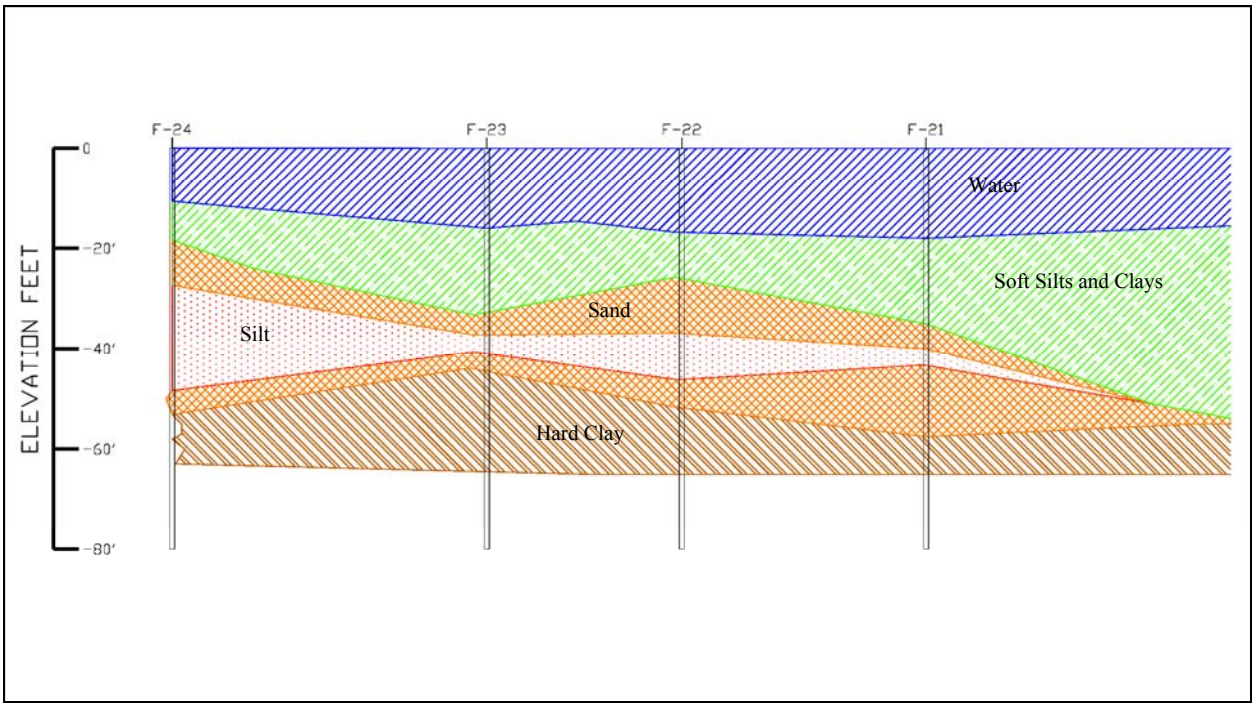
4.3.2.2 Probes

The primary goal of the probing investigation, described in Masonville Probing Report (GBA 2005), was to define the soft material-firm bottom interface. Figure 4-6 shows the locations of the 620 soil probes taken for this investigation. A condition hydrographic survey, performed in April 2005, provided the water depth to the top of soft material. The probing investigation provided a basis for the generation of a Triangulated Irregular Network (TIN) model for the bottom of this material, or the soft material-firm bottom interface. Figure 4-7 displays the contours of the TIN surface of the soft material-firm bottom interface. The probing data set helps to determine boring locations, dredging grades and quantities, and construction parameters.



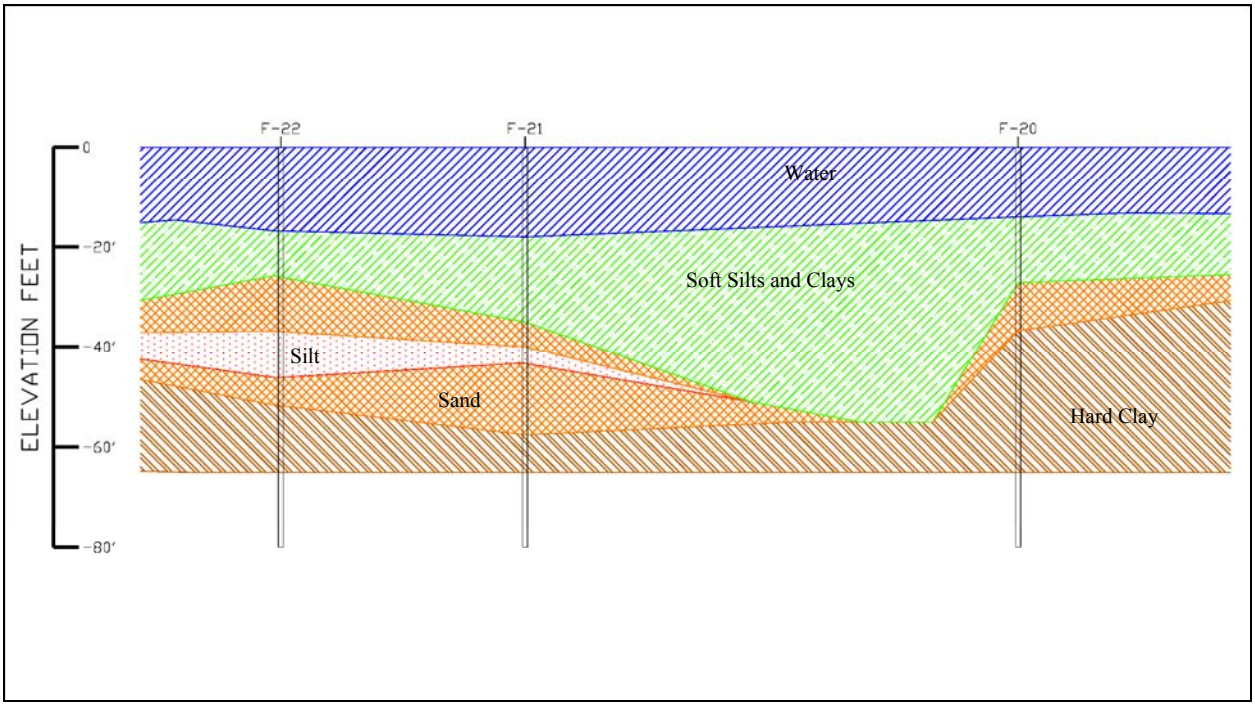
Adapted from Findling (2005)
Note: Environmental boring sites EB1and EB9 from 2005 were also sampled in 2006

Figure 4-2. Boring Locations



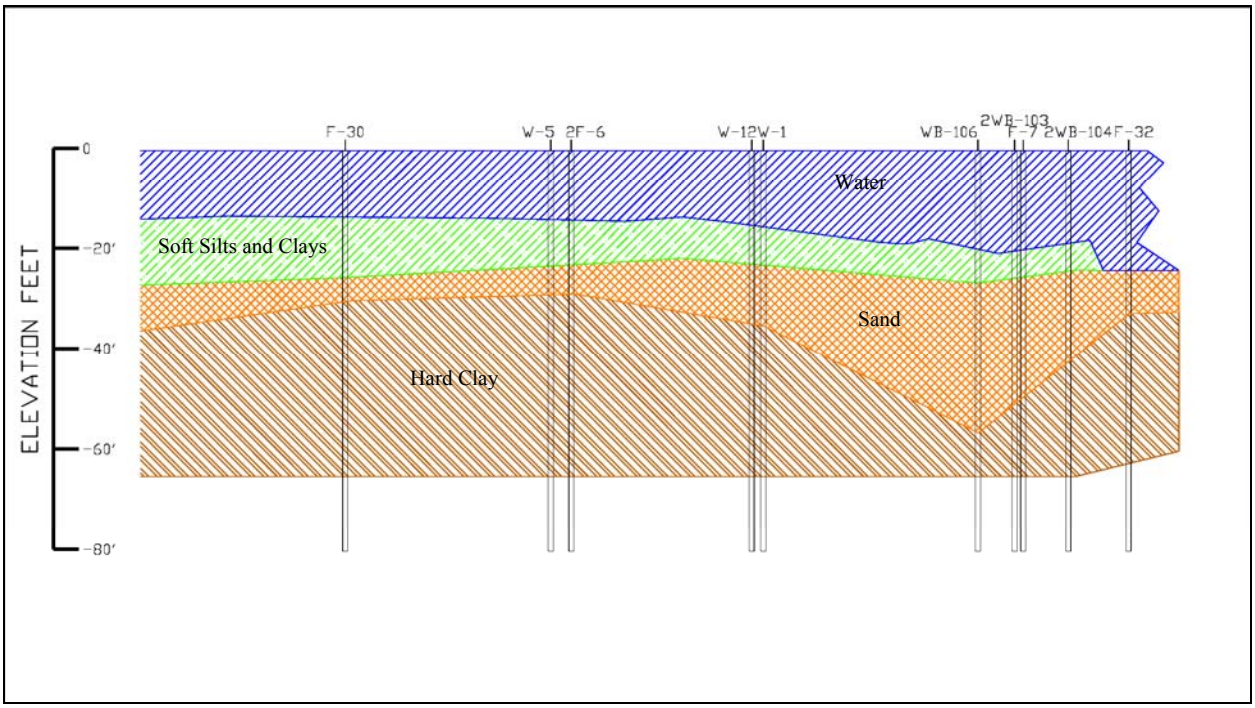
Adapted from Findling (2005)

Figure 4-3. Western Soil Profile



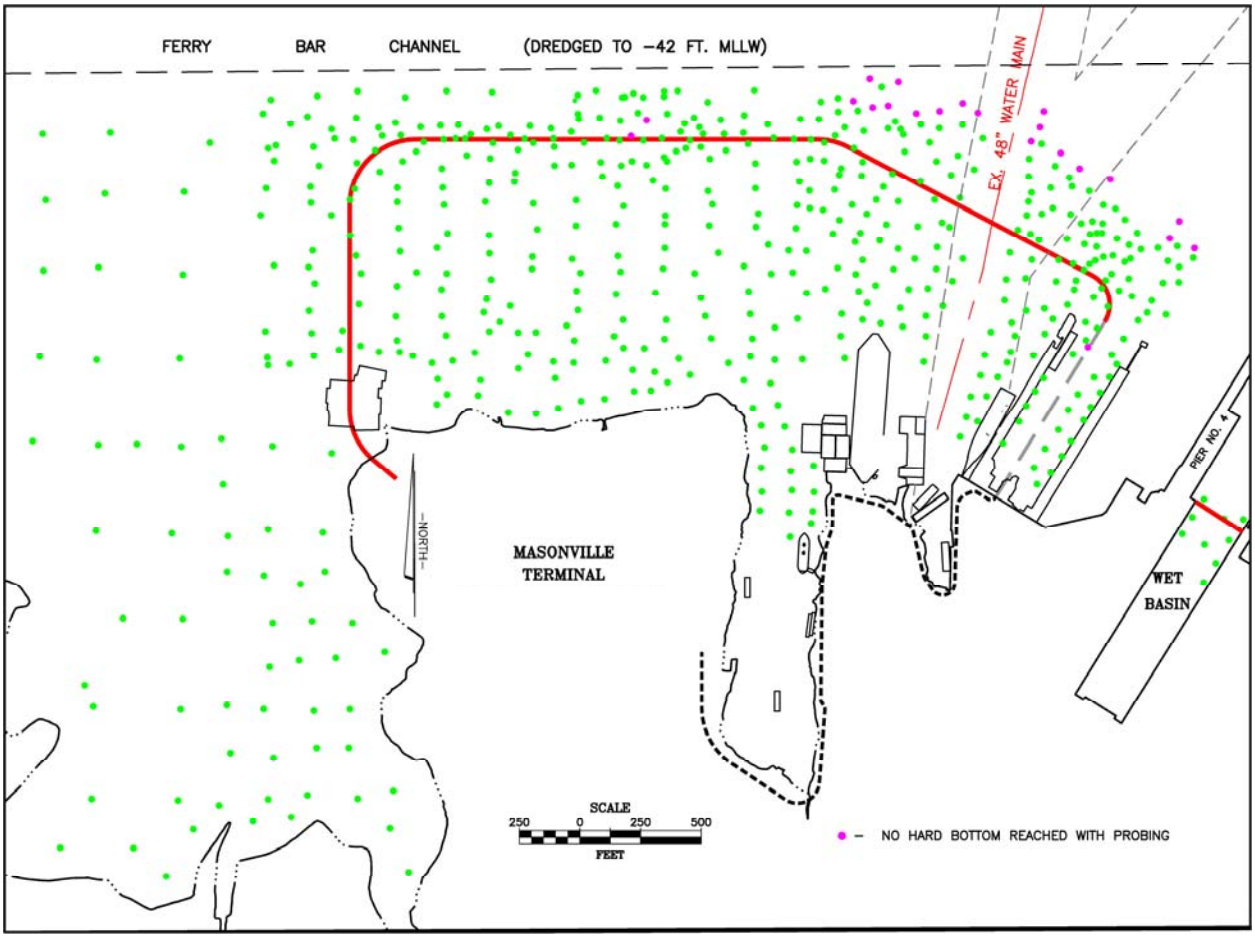
Adapted from Findling (2005)

Figure 4-4. Northern Soil Profile



Adapted from Findling (2005)

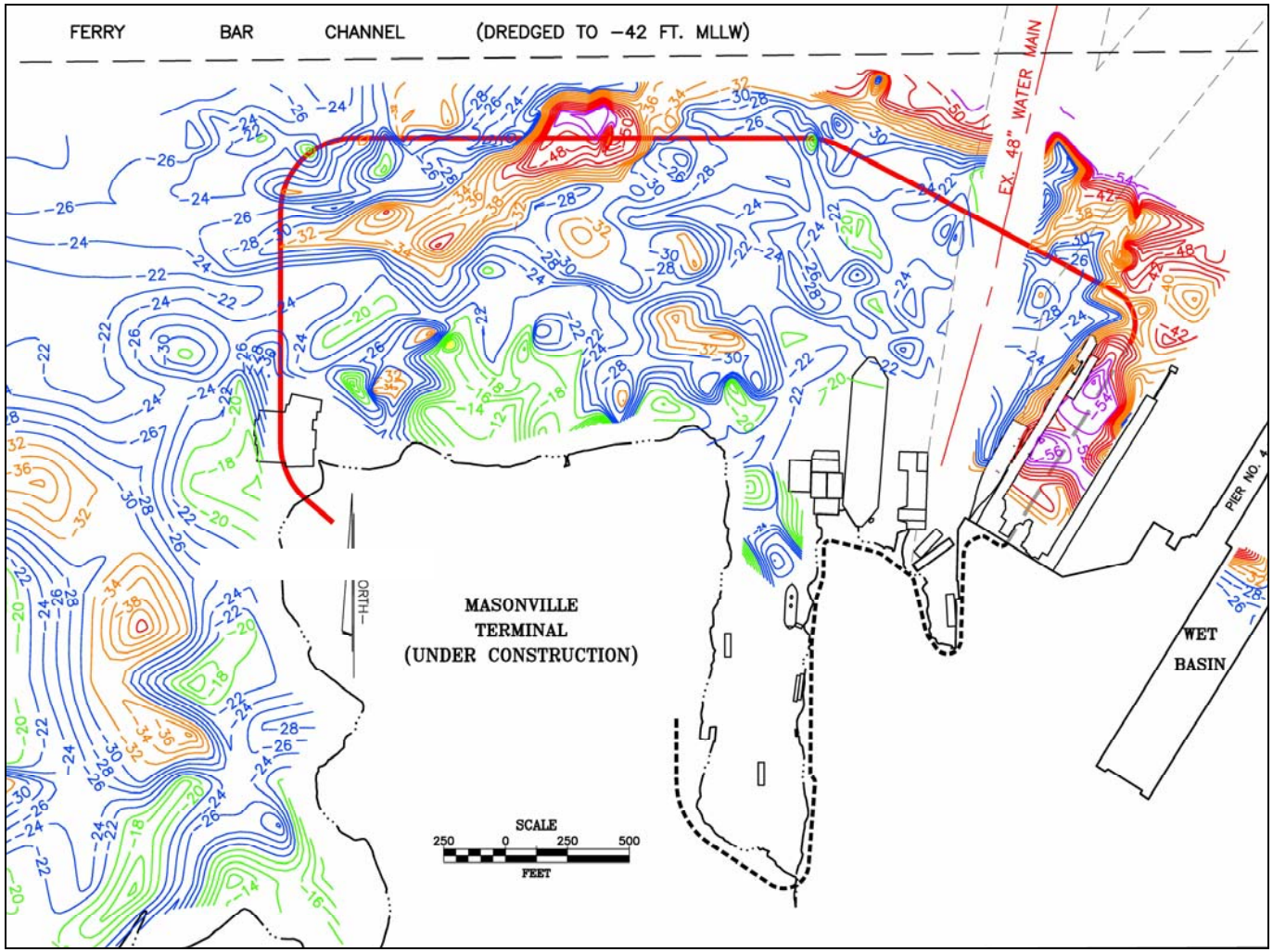
Figure 4-5. Eastern Soil Profile



Source: GBA 2005

Figure 4-6. Probe Locations

Notes: The green and pink dots represent locations where probes were taken at the site. The green dots indicate that the probe met refusal, and pink dots indicate that the maximum depth of the probe (50 ft below the waterline) was reached, without encountering refusal. A total of 620 probes were taken at the site.



Source: GBA 2005

Figure 4-7. Topography of Soft Material/Firm Bottom Interface

Notes: The colored contour lines represent depths of bathymetry. The green contours show depths of 0 to -20 ft, the blue contours show depths of -22 to -30 ft, the orange contours show depths of -32 to -40 ft, the red contours show depths of -42 to -50 ft, and the purple contours show depths of -52 ft and deeper.

4.3.2.3 Vane Shear Testing

The probing data provided the interface between soft overburden material and the underlying hard material. The general site design calls for stripping of the overburden material to expose the hard material. In several locations along the sand dike alignment, an interface was not found. The consensus among the design team was that in the places where hard bottom was not found, the shear strength of the material through which the probe was penetrating increased with depth. Thus, vane shear testing was performed at discrete intervals through the soil profile to determine if and at what depth the material reached a shear strength sufficient to support the proposed dike section.

Vane shear testing was performed along the sand dike portions of the containment structure alignment. Figure 4-8 shows the locations of the vane shear tests.

In-situ field vane shear tests were conducted in accordance with American Society for Testing Materials (ASTM) D-2573. A six inch long vane with a vane diameter of 2 ⅜ inches was used. The torque was measured using a calibrated torque wrench with an arm length of 12 inches. Appendix B of Findling 2005 contains the results of the vane shear testing.

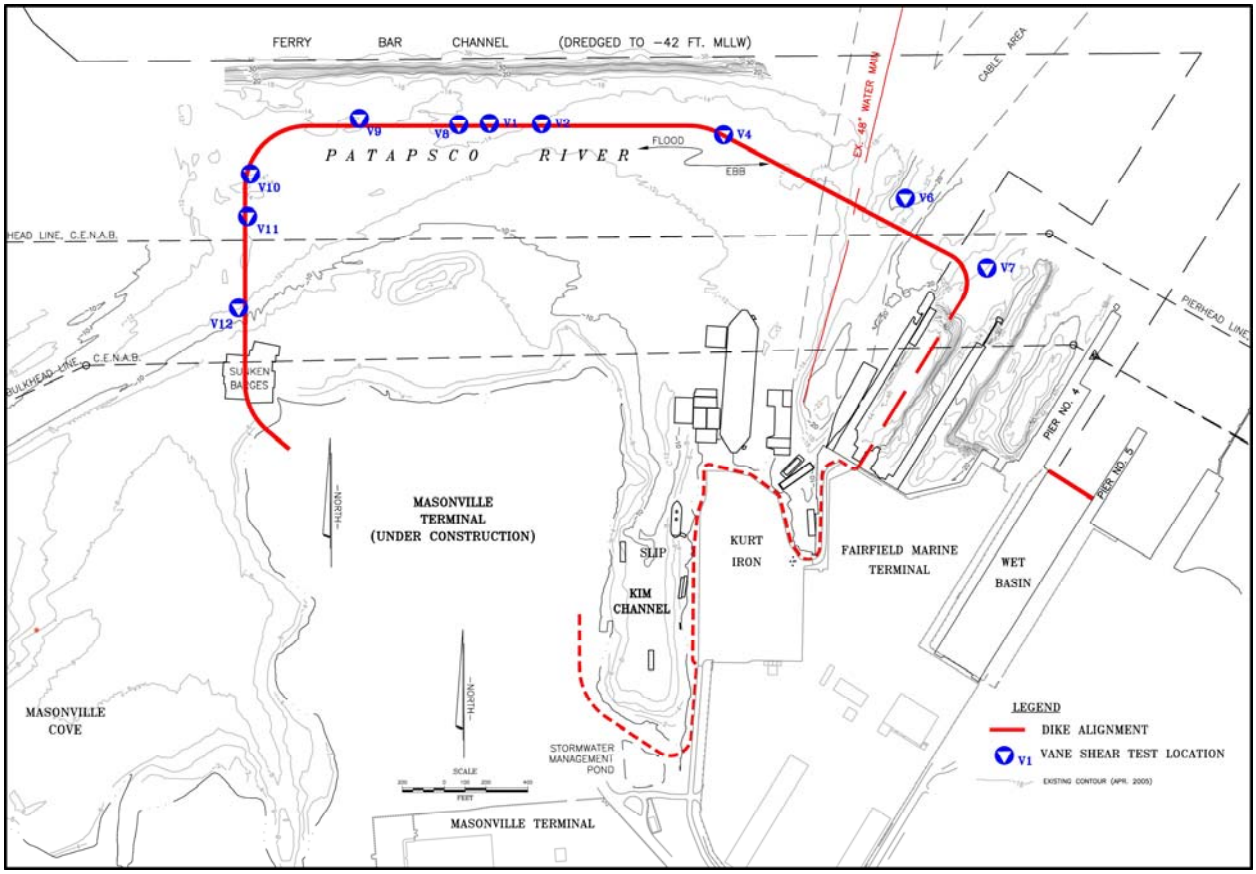
Vane shear testing results were used to help define the *in-situ* shear strength of the material through the soil profile along the dike alignment. The shear strength was used to help determine the undercutting depths necessary along the dike alignment.

4.4 SITE DESIGN

Design of the general site layout and containment structure was developed for the alternatives analysis, and allowed comparison of the varying alternatives. Following the selection of the recommended plan (Figure 4-1) a more detailed level of design was performed.

The three main elements of the design are: 1) the borrow area within the site footprint, 2) the containment structure, and 3) the Wet Basin rock dike. Each of these elements require the removal of overburden material from the site.

The designs of the containment structure and Wet Basin rock dike determine the quantity of construction materials needed and thus the extent and type of the borrow within the site. Similarly, the quantity and type of construction material available within the site influence the design of the containment structure. Thus, an iterative process (where a design is selected, the materials balance is checked, and the design is modified) was used to develop the containment structure design. This section describes the design of the containment structure and the borrow area within the site, as well as the removal of overburden materials and its part in the design of each element.



Adapted from Findling 2005

Figure 4-8. Vane Shear Testing Locations

4.4.1 Containment Structure and Wet Basin Retention Structure

As previously stated in Section 4.2, there would be four sections to the containment structure: a cofferdam, an armored sand dike, a sand dike with a beach, and an onshore earthen dike. These containment structures are discussed in further detail in this section. The Wet Basin retention structure consists of a rock dike section.

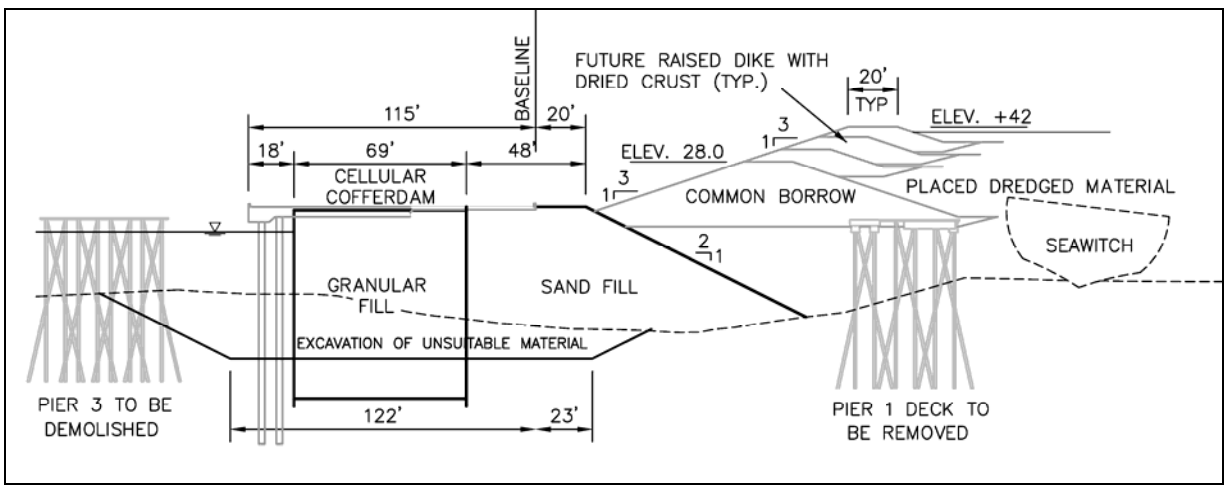
Removal of the materials geotechnically unsuitable for construction (overburden materials) under each section located in the water is required. For the beach and armored dike sections, the design goal was to minimize the removal of overburden material while constructing a dike meeting stability requirements. The same goal was used for removal of unsuitable material prior to the construction of the sand berm behind the cofferdams and under the Wet Basin rock dike. Due to structural design requirements, all of the overburden materials have to be removed from beneath the cofferdam cells. The total, *in-situ* volume of overburden to be removed from under the containment structure and Wet Basin retention structure is 0.6 mcy.

The containment structure is initially being constructed to an elevation of +10 ft MLLW. The current plan is to raise the structure to +28 ft MLLW using common borrow and incrementally from +28 to +42 ft MLLW using dried dredged material. The dike would temporarily be at an elevation of +42 MLLW and graded to a final elevation of +36 ft MLLW. The raisings are

represented in the Figures 4-9 thru 4-12, which show the typical cross-sections for each section of the containment structure. The cross-sections of the future raisings include 4 ft of displacement of the dredged material below the raised dikes accounting for the potential of the underlying dredged material to not gain sufficient strength to support the construction. This report assumes that drying schedules and operations at the site will allow a sufficient amount of dried dredged material to be generated onsite for the incremental dike raisings to a temporary elevation of +42 ft MLLW and a final elevation of +36 ft MLLW. This is reflected in the cost estimate for dike raisings. Should a sufficient amount not be generated, offsite borrow would be required and dike raising costs would increase.

4.4.1.1 Cofferdam (Berth Area)

The typical cross-section of the cofferdam is shown in Figure 4-9. Cellular cofferdams constructed of steel sheet pile serve as the retention system and are later incorporated into the wharf structure. The cofferdam cells are 69 ft in diameter, and would be filled with clean granular fill from a licensed, upland offsite source. Stratum I material would be removed by pre-dredging prior to cell construction, both within the cell footprint and inboard of the cells. A sand berm with a 48 ft wide crest would be placed directly inboard of the cells to reduce active earth pressures on the cells, exerted by future placed dredged material. The sand berm would be constructed to +10 ft MLLW with a future raising using common borrow to +28 ft MLLW and subsequent raisings using dredged material with a temporary elevation of +42 ft MLLW. The dike would be graded into the site, which has an anticipated final elevation of +36 ft MLLW.



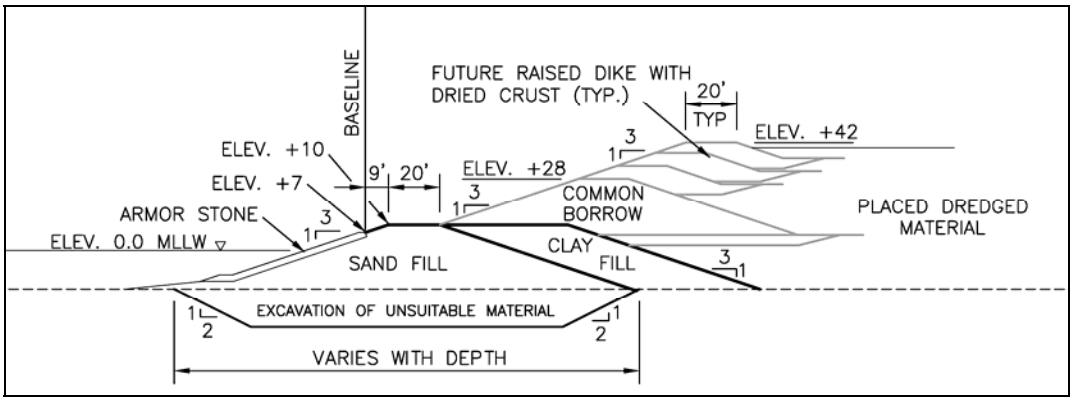
Note: Dimensions shown are subject to change, pending further design and investigations.

Figure 4-9. Typical Cofferdam Cross-Section

4.4.1.2 Armored Containment Dike

The typical armored sand dike cross-section (Figure 4-10) shows the geometry of unsuitable material excavation and sand backfill. Two to one slopes would form the toe of the cut to the river bottom, as described in Findling 2005. The initial dike would be +10 ft MLLW with a second dike raising using common borrow to +28 ft MLLW and subsequent raisings using

dredged material with a temporary elevation of +42 ft MLLW. The dike would be graded into the site, which has an anticipated final elevation of +36 ft MLLW.



Note: Dimensions shown are subject to change, pending further design and investigations.

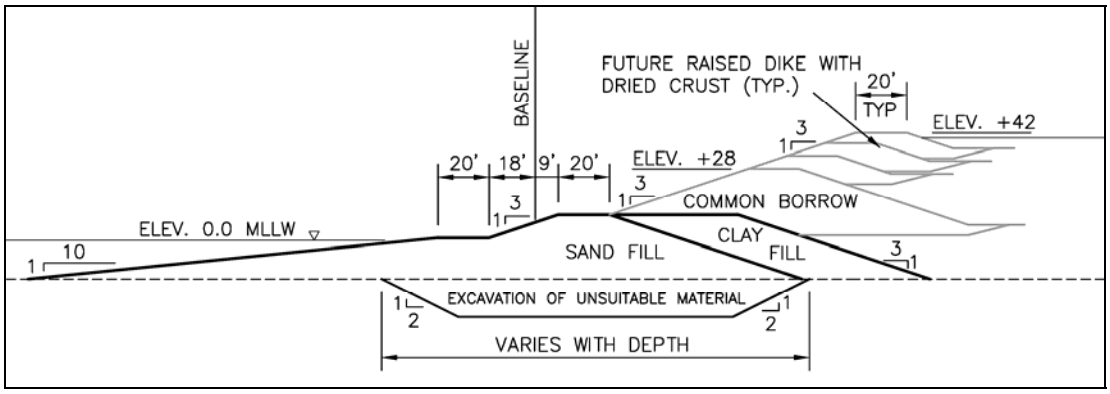
Figure 4-10. Typical Armored Sand Dike Cross-Section

The rock protection of the sand dike includes toe and slope armament up to elevation +7 ft MLLW. Geotextile fabric and a layer of quarry run stone underlie the armor. The armament would be a 2.5 ft thick layer of 250 pound (lb) stone.

The armored containment dike would include a leachate barrier with a permeability of $5x10^{-6}$ centimeters (cm) per second. The type of leachate barrier has not yet been determined, and would be evaluated in the design phase and subject to the approval of regulatory agencies. The currently anticipated barrier type is a non-woven geomembrane liner.

4.4.1.3 Beach Containment Dike

The western portion of the containment structure would consist of a +10 ft MLLW sand dike fronted with a sand beach. The cross-section of the dike would be the same as for the armored dike, except a sand beach would replace the armament. The beach provides environmental benefits, and would consist of a 20 ft wide berm at +1 ft in elevation and a 10:1 slope into the water (Figure 4-11). Hydrodynamic and sedimentation modeling indicates that any beach erosion would be minimal. The initial dike would be +10 ft MLLW with a second dike raising using common borrow to +28 ft MLLW and subsequent raisings using dredged material with a temporary elevation of +42 ft MLLW. The dike would be graded into the site, which has an anticipated final elevation of +36 ft MLLW.



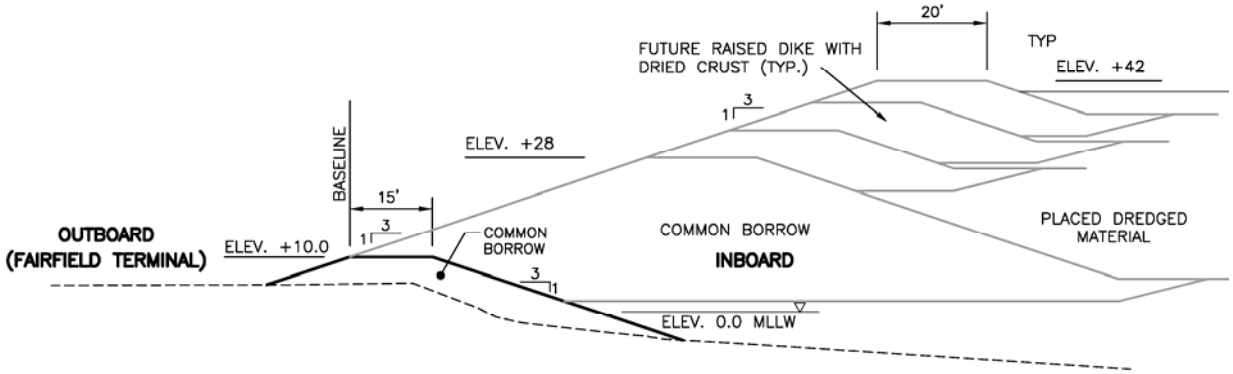
Note: Dimensions shown are subject to change, pending further design and investigations.

Figure 4-11. Typical Beach Dike Cross-Section

The beach containment dike would include a leachate barrier with a permeability of $5x10^{-6}$ cm per second. The type of leachate barrier has not yet been determined, and would be evaluated in the design phase and subject to the approval of regulatory agencies. The currently anticipated barrier type is a non-woven geomembrane liner.

4.4.1.4 Onshore Dike

The onshore segment's design structure consists of a sand dike that would be constructed along the existing shoreline (see Figure 4-12). The initial elevation of the dike would be +10 ft MLLW. A dike raising to +28 ft MLLW using common borrow is anticipated, along with future raisings to a temporary height of +42 ft MLLW using dredged material. The dike would be graded into the site, which has an anticipated final elevation of +36 ft MLLW.



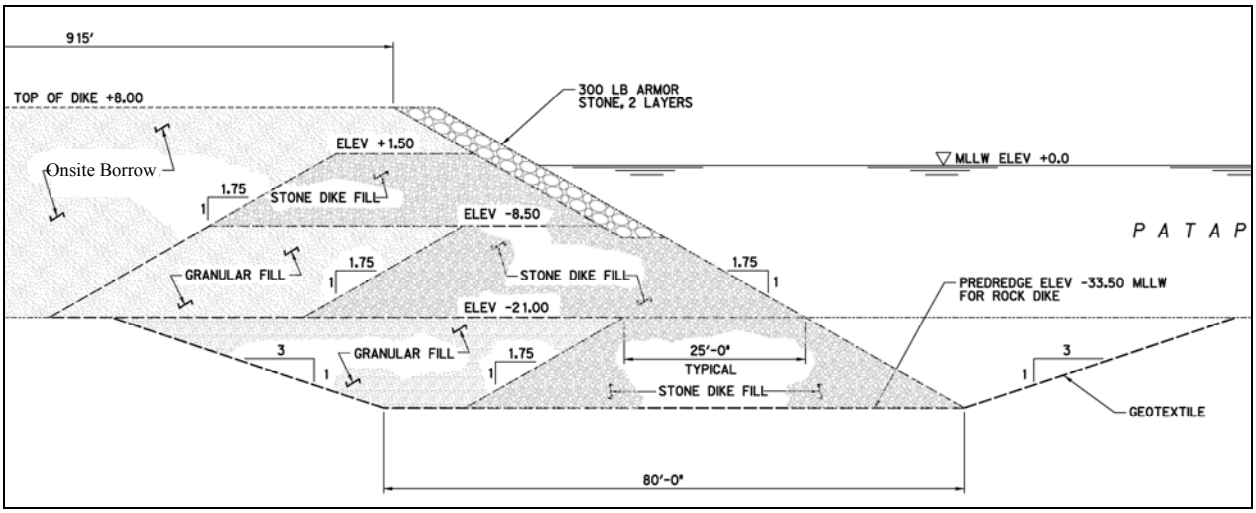
Note: Dimensions shown are subject to change, pending further design and investigations.

Figure 4-12. Typical Onshore Dike Cross-Section

4.4.1.5 Wet Basin Rock Dike

The retention structure for the Wet Basin is a rock dike. The dike has an outside slope of 1.75:1 [horizontal to vertical ratio (H:V)], and would be built in three lifts (Figure 4-13). The dike

would be armored with two layers of 300 lb stone from –10 ft MLLW to its crest at +8 ft MLLW. Undercutting of approximately 12.5 ft of overburden material is required to expose geotechnically suitable foundation material. A reinforcement geotextile may be deployed inboard of the dike to minimize the formation of mud waves.



Note: Dimensions shown are subject to change, pending further design and investigations.

Figure 4-13. Typical Wet Basin Rock Dike Cross-Section

4.4.1.6 Design Quantities

Quantities for construction materials are estimated based on the designs presented in this section. Table 4-2 summarizes the estimated construction material quantities for the containment structure. A breakdown of the quantities for each design section is presented in Appendix G

Table 4-2. Construction and Excavation Quantities

Construction Material	Quantity
Sand (mcy)	1.5
Clay (mcy)	0.4
Stone (tons)	78,000
Geotextile(sy)	94,000
Overburden Removal (mcy)	0.6

4.4.2 Borrow Area

The geotechnical investigation allowed for an estimation of the volume and type of construction material available for borrow from within the site footprint. This section describes the types of materials suitable for construction, identifies the boundaries of the borrow area, and quantifies the volumes and types of materials available. Offsite upland borrow sources may be required should on-site materials be determined to be insufficient. These upland borrow sources would be permitted and approved for in-water placement.

4.4.2.1 Geotechnically Suitable Construction Materials

Sand with a fines content of less than 30 percent is the preferred borrow material for dike construction. Plastic clay with a relatively low moisture content capable of forming "clay balls" when hydraulically pumped is also considered suitable for dike construction. In either case, geotechnically unsuitable materials, referred to as overburden, typically soft silts and clays, frequently overlie the borrow source. This overburden must be stripped off to expose the borrow source and then disposed of at the HMI DMCF. The estimated *in-situ* volume of overburden material to be removed from overtop of the borrow area is approximately 1.1 mcy.

4.4.2.2 Available Borrow

Figure 4-14 shows the plan location of the borrow areas inside the dikes. To avoid slope stability issues, the borrow area is limited by an offset of 65 ft from the containment structure. Sections A and B taken from Figure 4-14 depict the subsurface strata within the borrow area as shown in Figure 4-15. Stratum I is the soft silts and clays (overburden). Stratum II is comprised of medium dense to dense sands, and Stratum III is comprised of stiff to hard clay. The percent fines found within the sand in Stratum II can vary considerably, but is generally less than 30 percent. The clay in Stratum III is stiff to hard and it is anticipated that it will form clay balls during excavation through hydraulic dredging. Further description of the Strata and their suitability for borrow are found in Findling 2005. In calculating borrow quantities, elevation -60 ft MLLW was assumed as the limiting depth of excavation. The estimated volume of sand available is approximately 1.5 mcy, and the estimated volume of stiff clay available is approximately 0.6 mcy.

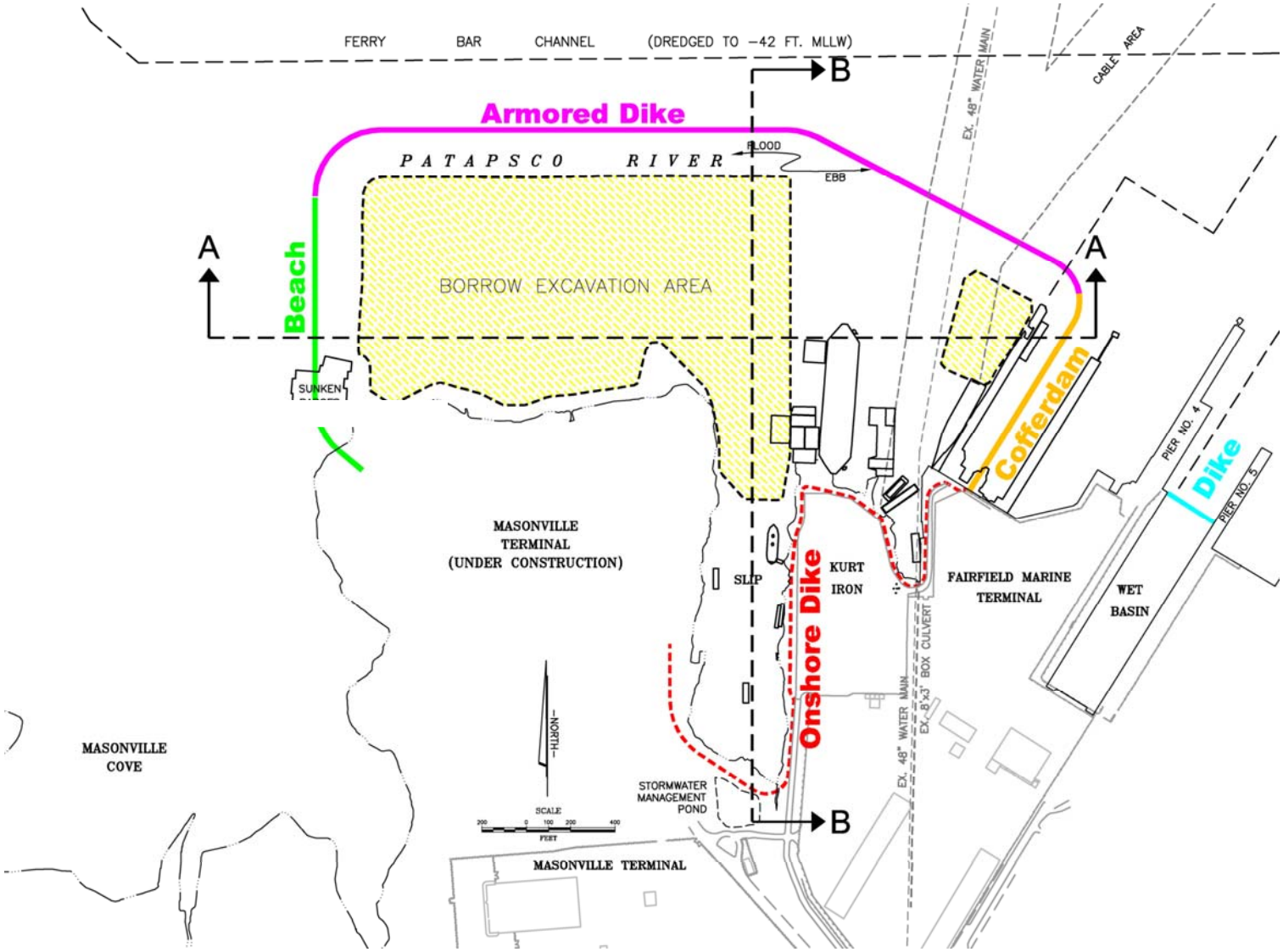


Figure 4-14. Borrow Area Plan View

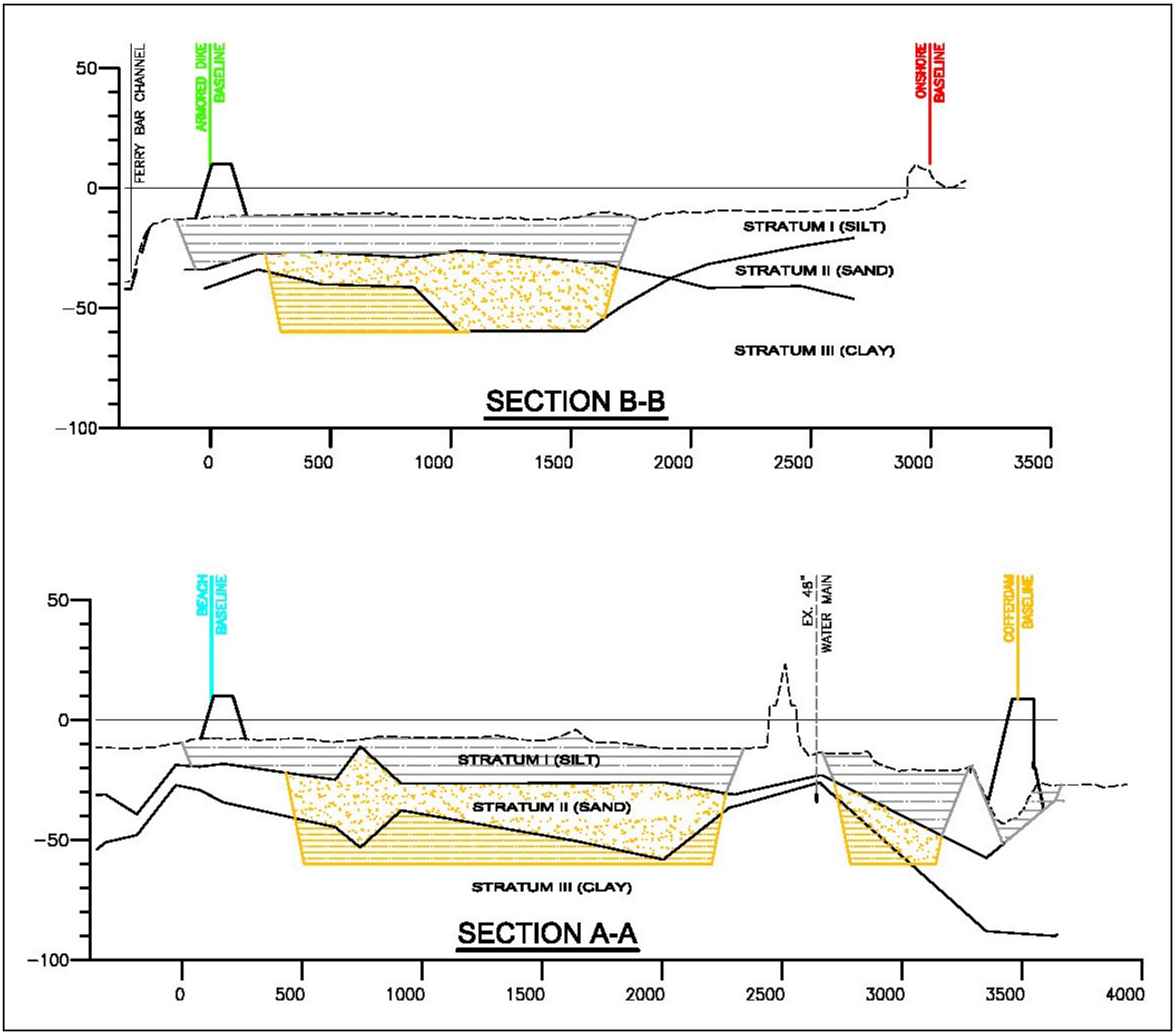


Figure 4-15. Borrow Area Cross-Sections

East Borrow Area

The east borrow area covers 2.3 acres west of existing Pier 1 in about 20 feet of water. About 5 feet [(20,000 cubic yards (cy)] of soft clay and silt will be stripped to about -25 MLLW during preparatory dredging. Below the clay & silt is about 70,000 cy of sand for dike fill to the assumed maximum digging depth of -60 MLLW. The resulting 40 feet deep borrow hole will have 2:1 side slopes.

West Borrow Area

The west borrow area covers 39 acres of Patapsco River bottom with water depths in the 7 to 13 feet range. In the center of the borrow area is a 1.3 acre mound with water depths ranging from 7 feet to 2 feet below MLLW. The preparatory dredging depths in the west borrow area vary between about 10 to 20 feet. Removal of about 950,000 cy of soft material may be accomplished by using a lightweight bucket to dig the soft material and minimize the removal of dense material suitable for dike fill.

Borrow material from the west borrow area for dike fill is stratified in many layers with lenses of sand, gravel, silt, soft clay and stiff clay. Stratification is more evident along the west perimeter and in the north west corner of the borrow area, with some stratification along the east side and less in the middle. Just below the soft material removed during preparatory dredging is about 990,000 cy of sand and gravel in thicknesses ranging from 5 feet to 45 feet, with an average of about ten feet. Below the sand and gravel are lenses of silts, soft clays, and stiff clays providing about 100,000 cy of clay borrow material. Another 490,000 cy of sand and gravel lies below the stratified lenses and below that most of the borings drilled in this area stop within a layer of very stiff red clay. Digging to the assumed maximum depth of –60 MLLW will produce about 400,000 cy of red clay balls.

4.5 DMCF CONSTRUCTION

A substantial site preparatory phase is required prior to the construction of the DMCF. This section describes the activities and methods necessary for both the preparatory phase and the construction of the containment structure. The implementation and schedules for completion of the preparatory phase and construction of the DMCF are described in Chapter 7.

4.5.1 Preparatory Phase

This phase includes the relocation/demolition of existing infrastructure and preparatory dredging within the proposed footprint to remove materials geotechnically unsuitable for use in construction.

4.5.1.1 Relocation/Demolition of Existing Infrastructure

Relocation and demolition of the existing infrastructure includes remediation of 25 derelict vessels (Section 7.3) and abandoned structures and relocation of utilities that would be impacted by the DMCF construction.

Demolition Of Piers 1, 2 and 3 Fairfield Marine Terminal

Two existing piers, Pier 1 and Pier 3, would be impacted by the construction of the DMCF. Pier 3 is a dilapidated pier structure (outside of the proposed footprint) that would need to be removed to allow dredging of a channel providing access to the cofferdam berth area. The pier would be completely demolished and transported to an appropriate disposal facility.

A remnant of the former Pier 2 exists within the Masonville footprint. This dilapidated structure would be removed where necessary to accommodate the construction of the cofferdams. Concrete portions of Pier 2 may be disposed of within the footprint of the DMCF; however, wooden portions of the Pier that are removed would be disposed of offsite at an appropriate disposal facility.

Abandoned Pier 1 is located at the western end of Fairfield Marine Terminal. Pier 1 would be partially demolished by removing the concrete deck and leaving the piles in place. The concrete deck may be disposed of within the footprint of the DMCF. Any portions of Piers 1 and 2 that are placed within the footprint of the DMCF would have a negligible impact on site capacity.

Pier demolition would not impact wetlands, thus no mitigation for this work is proposed.

Phase I, Baltimore City Storm Drain Relocation

An 8 ft by 3 ft box culvert (existing outfall) drains into the small cove between Kurt Iron and Pier 1 (Figure 4-16). This outfall and storm drain would need to be relocated to allow for construction of the DMCF. Phase I of this relocation would involve placement of the new landside storm drain culvert and construction of a new outfall. The new outfall would be relocated to the east of the DMCF (temporary outfall) where a new nine ft by eight ft box culvert would be connected to it (Figure 4-16). However, in this phase, the new storm drain and outfall would not be tied into the existing storm drain.

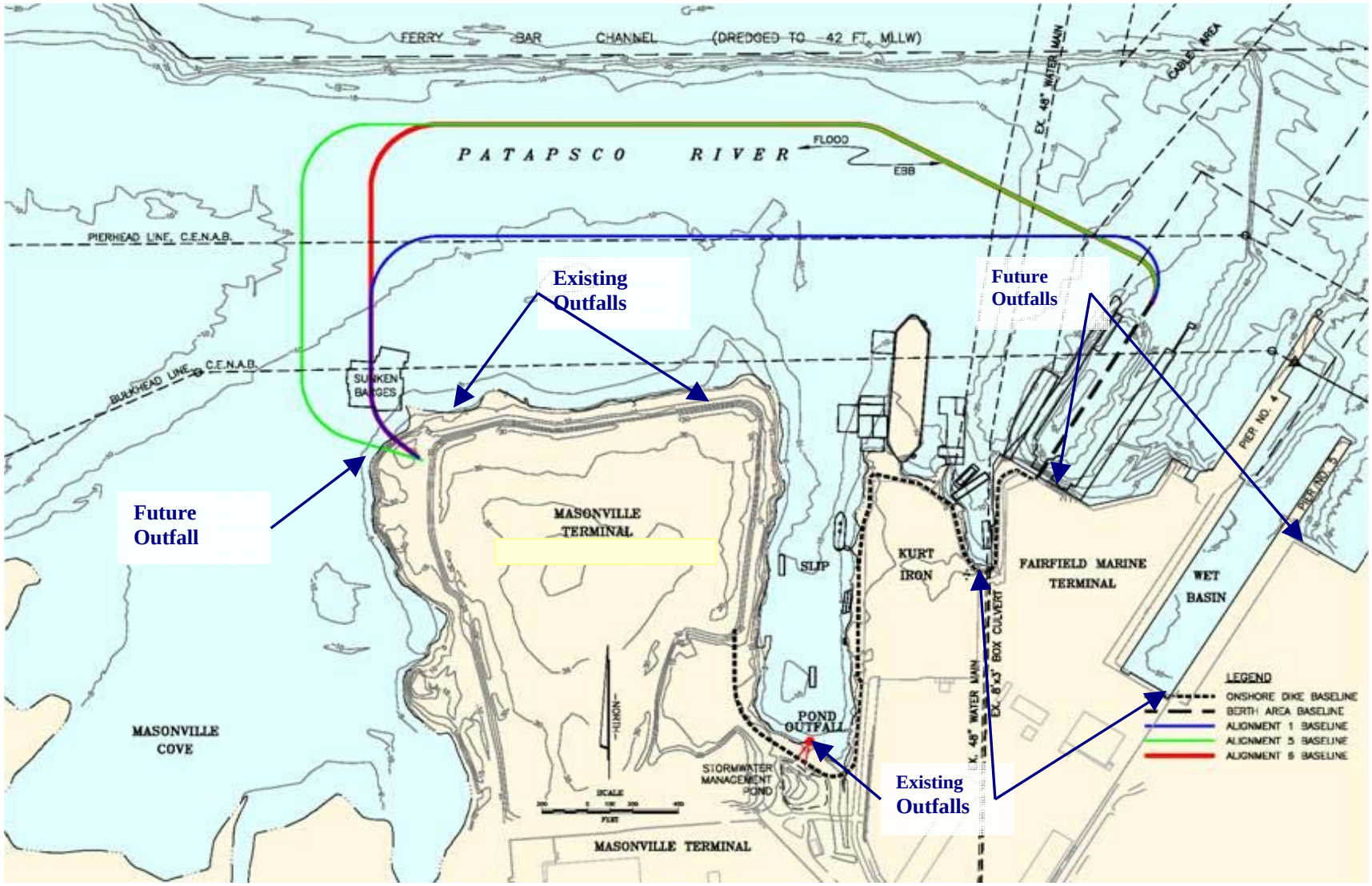


Figure 4-16. Location of Stormwater Outfalls

Phase II, Baltimore City Storm Drain Relocation and Masonville Marine Terminal (MMT) Phase 2/Kurt Iron and Metal Site (KIM) Stormwater Outfall Relocation

The Masonville stormwater management pond and outlet pipes from the proposed KIM and MMT Phase 2 storage lots would be closed off with the construction of the proposed DMCF. A new storm drain system would be installed to convey storm drainage collected by these lines to the relocated storm drain (from Phase I). In Phase II, the existing Baltimore City storm drain system would also be tied into the newly placed culvert from Phase I (Figure 4-16).

Wet Basin Storm Drain Relocation

Filling the Wet Basin would require the relocation of a 9.5 ft by 4.5 ft box culvert. The preferred option is to construct a new ten ft by five ft box culvert around the east side of the Wet Basin prior to fill placement (Figure 4-16).

Baltimore City Waterline

The 48-inch waterline is a secondary line providing backup to the Baltimore City water distribution system, which serves over 300,000 customers in southeast Baltimore City and northern Anne Arundel County. Baltimore City has stated that construction of the DMCF over the existing waterline is not acceptable. The waterline must be relocated prior to portions of the DMCF construction to allow for access to the line for maintenance. Potential options being evaluated include routing the line around the site or routing the line within the dikes and sand berm along the cofferdam cells. The existing waterline and the proposed reroutes of the waterline are shown in Figure 4-17. Baltimore City's preferred reroute is the option that extends farthest to the north (alignment 3). Due to construction logistics, it is anticipated that this project will be combined with the cofferdam project (4.5.2.2) during the construction phase.

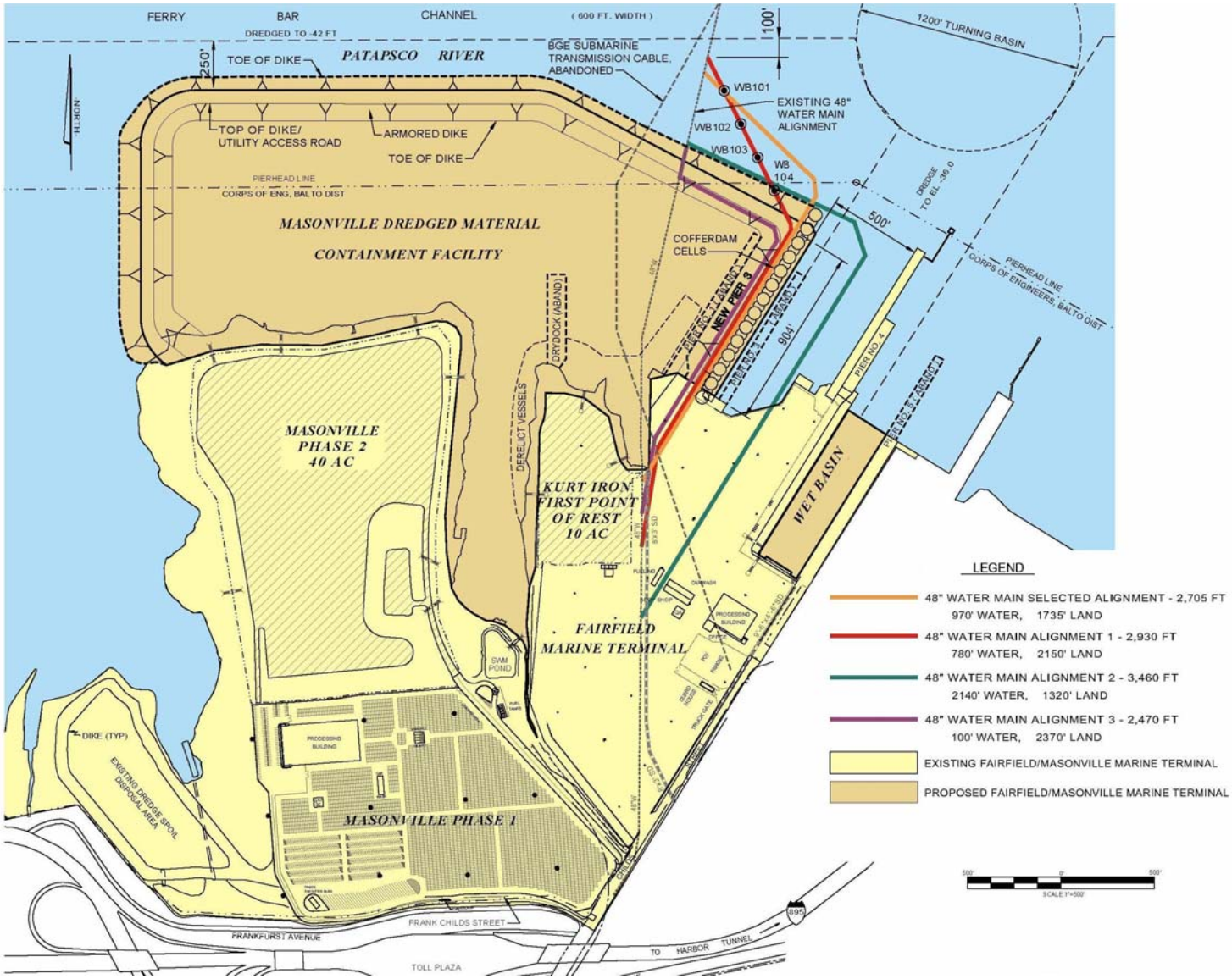


Figure 4-17. Watermain Relocation Options

Derelict Vessel Remediation at the Former Kurt Iron and Metal Facility

Derelict vessels are generally located within the dashed line in the water near the former KIM facility in Figure 4-17. MPA will remove any hazardous and regulated wastes from the vessels for proper disposal in licensed landfills. The MPA will also remove and properly dispose of any solid wastes generated in the hazardous waste removal effort. Following remediation, Barge 4 will be taken offsite to be salvaged.

This work will not impact wetlands, thus no mitigation for this work is proposed. An agreement has been made with the Maryland Department of the Environment (MDE) and this action does not require mitigation. If the Masonville DMCF is not constructed, there would be no regulatory reason to remediate the derelict vessels on the western side of the proposed DMCF site. The funding currently allocated for site development would be released to other Maryland Department of Transportation (MDOT) efforts and the remediation of the 25 derelict vessels would be deferred.

The MPA would relocate barges interfering with DMCF construction to new locations within the proposed DMCF footprint. Barges 1 and 2, which had been dislocated during a storm and the Coast Guard had moored to Pier No. 3, would be relocated to the KIM Channel (Figure 4-18). Barge 3 and the Crane Barge would also require relocation to the KIM Channel (Figure 4-18).

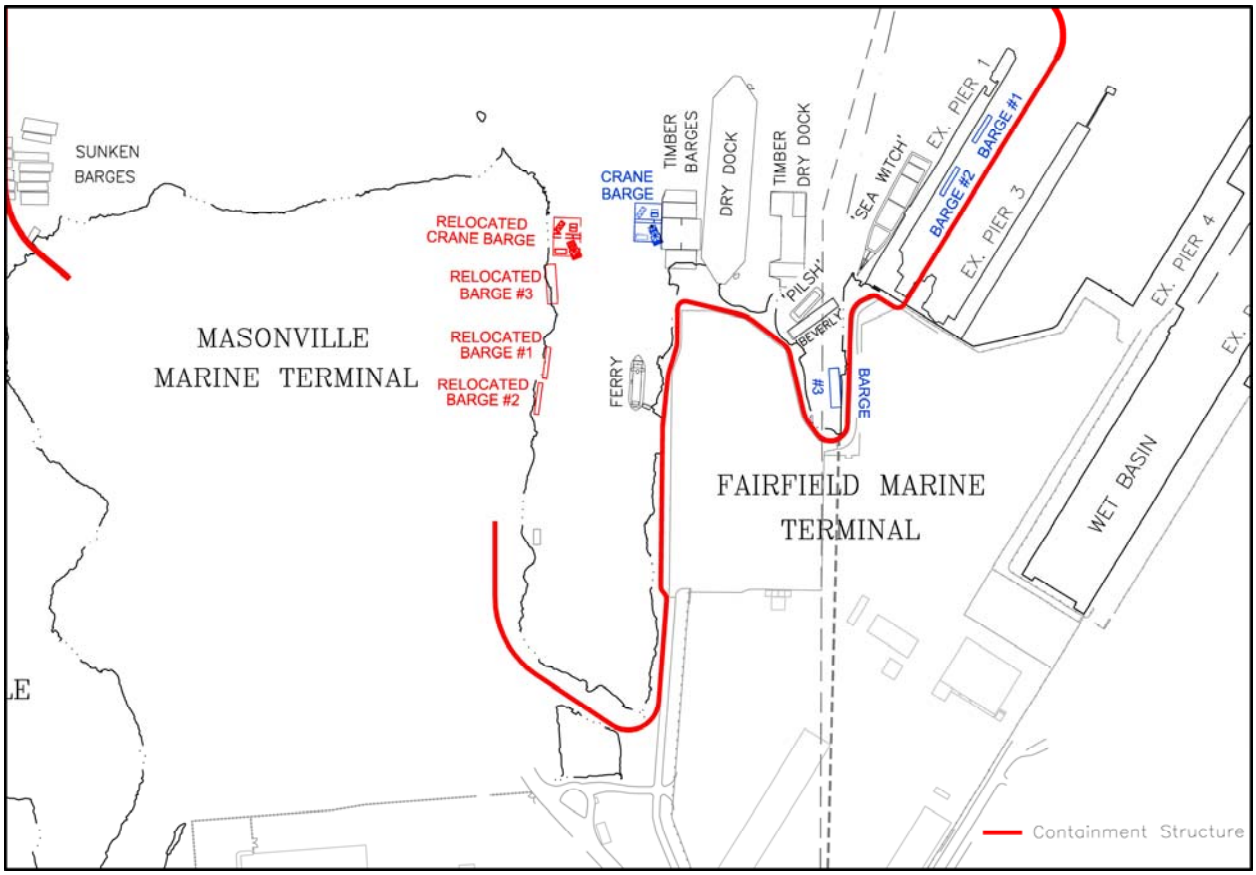


Figure 4-18. Onsite Relocation of Barges

4.5.1.2 Pre-Dredging (Removal of Overburden)

Pre-dredging involves the mechanical removal of geotechnically unsuitable construction materials (generally silts and soft clays) from beneath the containment structure and overlying onsite borrow areas. The material would be loaded into scows for transport to the HMI DMCF, where it would be hydraulically unloaded. The total estimated volume of material to be removed from the site and placed at the HMI DMCF is approximately 1.7 mcy. This includes 1.1 mcy of overburden from the borrow area and 0.6 mcy of overburden from the containment structure area.

4.5.2 Containment Structure Construction

There are four construction components required for the completion of the containment structure at Masonville (Figure 4-1). Their designs are described in detail in Section 4.4.1, and their construction is described in this section.

4.5.2.1 General Construction Sequence

A sufficient quantity of borrow material for construction of the containment structure is believed to exist onsite above elevation -60 ft MLLW. The sand and clay would be dug with a large hydraulic cutterhead dredge and pumped into section. The sand portion of the containment structure would be placed into sections prior to dredging of the clay, effectively closing off the interior of the site. This would reduce potential turbidity issues involving dredging of the clay. Following completion of the clay dike placement, dredging equipment would exit the dike through a "key way" excavated to approximately –8 ft MLLW. The key way would then be filled by mechanical placement of material. If onsite volumes are determined to be insufficient, offsite upland sources may be required.

The next phases of containment structure construction would be the dike raisings to +28 ft MLLW and from +28 to +42 ft MLLW. The dike raising using common borrow from +28 ft MLLW would be constructed by truck-haul of the common borrow into place and mechanically shaping the dike section. The incremental raisings from +28 to +42 ft MLLW would be constructed using dried dredged material. Dried dredged material (crust) would be pulled to the sides of the dike each year to establish a berm of dredged material with increased strengths near the containment dike. The dike raising would then be constructed via typical mechanical methods with dried dredged material. Should the dried dredge material available onsite not be sufficient to construct the incremental dike raisings, material would be obtained from offsite.

4.5.2.2 Cofferdam

To begin cofferdam construction, a two level template would be positioned, and steel sheet piles would be vibrated in place to form the cofferdam. After the cofferdam walls are completed, granular fill would be mechanically placed in the cofferdam cell via truck haul or barge. Once the cell is partially filled to a stable level, the template would be removed and moved to the next position. After the fill is placed in the cell, the fill would be densified by vibrocompaction or vibroflotation. Following densification of the cell fill, a sand berm would be constructed in the area immediately inboard of the cells. Due to construction logistics, it is anticipated that this project will be combined with the 48 inch waterline relocation. Predredging of Stratum I material (Figures 4-3 to 4-5) within the cofferdam construction area would be completed prior to cofferdam construction.

4.5.2.3 Armored Sand Dike

Training dikes would be constructed by hydraulic placement of the sand mined from onsite using a large (approximately 30 inch hydraulic dredge). The sand portion of the dike would then be constructed through continuing hydraulic placement and would be shaped with dozers. Hydraulic placement methods would likely include training dikes and turbidity curtains to minimize impacts from turbidity. As the dike is constructed to grade, it would be armored using an initial layer of geotextile and subsequent stone placement, as specified in Section 4.4.1.2. The stone would be offloaded from transport barges and set in place using cranes mounted on work barges.

Once the sand portion of the dike is constructed and the containment structure encloses the site from the Patapsco River, the clay portion of the dike would be constructed. The clay would be excavated hydraulically and pumped into section as "clay balls". Clay balls are created through the process of hydraulically excavating and pumping stiff to hard clays. No mechanical compaction of the clay would occur below water. Once the clay portion of the dike reaches +2 ft MLLW, dozers would be used for grading and shaping the clay section.

4.5.2.4 Beach Sand Dike

Training dikes would be constructed by hydraulic placement of the sand mined from onsite or from an outside source. The sand dike would then be constructed to grade through hydraulic placement and then shaped with dozers. It is anticipated that turbidity curtains would be used to minimize turbidity impacts. See Section 4.4.1.3 for the design of the beach sand dike.

Once the sand portion of the dike is constructed and the containment structure encloses the site from the Patapsco River, the clay portion of the dike would be constructed. The clay would be excavated hydraulically and pumped into section as "clay balls". Clay balls are created through the process of hydraulically excavating and pumping stiff to hard clays. No mechanical compaction of the clay would occur below water. Once the clay portion of the dike reaches +2 ft MLLW, dozers would be used for grading and shaping the clay section.

4.5.2.5 Onshore Dike

The onshore dike would be constructed using conventional earthwork construction methods. Material would be hauled via trucks from an offsite location or onsite stockpile. See Section 4.4.1.4 for the design of the onshore dike segment.

4.5.2.6 Ancillary Items

Ancillary items required to prepare the site for dredged material acceptance would be constructed following completion of the containment structure. The structure and location of these would be decided upon in the design phase. The ancillary items include spillways, access roads, and site monitoring equipment. These items are not yet in the design phase.

4.5.3 Wet Basin Rock Dike Construction

The rock dike would be constructed in three lifts using mechanical placement of stone from a barge. Sand would be placed behind each stone lift and then construction of the next lift would begin. Armament transported to the site by barge would be placed on the upper 18 ft of the dike slope by a crane.

Selective demolition of existing bulkheads would be required prior to fill placement. The selective demolition would involve removal of the earth fill and concrete deck/pedestal overlying the sheet pile walls inside the Wet Basin. The material for filling the Wet Basin would be excavated from within the Masonville DMCF footprint, and placed into the Wet Basin.

4.6 SITE OPERATIONS AND MAINTENANCE

The annual quantity of dredged material placed at the site is anticipated to be approximately 0.5 to 1.0 mcy (Table 1-2). During placement, monitoring of site conditions, periodic discharging of water, relocation of inflow pipes, and other activities would be required. Further, placement operations at the site would necessitate maintenance of the dikes and site facilities. Following placement, activities such as crust management may be required. A Site Operations Manual, detailing procedures and methods, would be developed as part of the future site design efforts.

4.7 SITE CLOSURE

The end use of the site is anticipated to be an expansion of the MMT. The area would serve as additional storage facility for Roll On-Roll Off (RO-RO) cargo or automobiles. Closure of the DMCF would require consolidation of the placed dredged material, which may be facilitated by wick installation and a rolling surcharge. A wick drain is a series of plastic tubes surrounded by a permeable membrane inserted into a soil medium to relieve pore pressures and provide a path for water to escape. The surcharge applies a load to the soil, creating excess pore pressure, which is relieved by the wick drains. The surcharge speeds up the consolidation process. The rolling portion of the term refers to the fact that the surcharge moves around the area of soil being consolidated. There would be no significant impacts as a result of the rolling surcharge. Following consolidation of the dredged material, the site would likely be prepared and paved. The pavement would include all areas that are not steep slopes, or required for stormwater management facilities.

4.8 REAL ESTATE

The proposed facility abuts the existing Masonville and Fairfield Marine Terminals and the Masonville Cove, all of which are owned by the MDOT and managed by the MPA. The Cove, which is the proposed site of the majority of the mitigation projects associated with the proposed DMCF (Chapter 6), is bound by Frankfurt Ave to the south, Arundel Corporation to the west, and MMT to the east. Land access to the site would be available through the Masonville and Fairfield Marine Terminals.

4.9 ADDITIONAL ENVIRONMENTAL BENEFITS

Construction of the proposed Masonville DMCF would have environmental benefits in addition to those included in the mitigation plan (Chapter 6). These benefits are described below.

4.9.1 Sediment and Contaminant Capping and Removal

Up to 2 mcy of contaminated overburden would be removed and placed at the HMI DMCF. These sediments would be removed from approximately 41 acres within the proposed alignment. Contaminated sediments from the remaining 88 acres within the alignment of the proposed Masonville DMCF would be capped as part of the construction and operation of the DMCF. The surficial sediment quality within the alignment is degraded as a result of elevated levels of some

contaminants (Section 2.1.5). Capping and the removal of sediments would make contaminants less available to the aquatic environment. The action would also make the contaminants less bioavailable for accumulation in fish tissue, possibly lowering the potential human health and ecological risks associated with the consumption of contaminated fish.

4.9.2 Derelict Vessel Removal and Remediation

The former KIM site lies in the eastern portion of the alignment (Figure 4-1) and there are currently 25 sunken and derelict vessels, a steel dry dock, and numerous barges with various materials on board associated with the site. Only three of the vessels were legally transferred with the property and are currently owned by MPA. The ownership of the other vessels is unknown. The MPA would remove deregulated and hazardous wastes from the ships and drydocks and dispose of them in properly licensed landfills. The solid wastes that remain would be minimally processed and relocated as necessary inside the footprint of the proposed Masonville DMCF. Depending on costs, some derelict vessels may be processed offsite. The remediation plan for the vessels has been developed (Section 7.3) and is moving forward. An agreement has been made with the MDE and this action does not require mitigation.

Remediation of the derelict vessels would remove a significant source of toxic substances within the area and reduce the toxic substances burden in this part of the Patapsco River. This would make the contaminants less available to the aquatic environment, and could directly benefit the benthic community and fish forage availability. Indirectly, the action could also make contaminants less bioavailable for accumulation in fish tissue, possibly lowering the potential human health and ecological risks associated with consumption of contaminated fish. Current cost estimates for the remediation and relocation of derelict vessels range from $5 to $10 million.

4.10 TOTAL PROJECT COST

The total project cost for the operational life of the Masonville site is the sum of the 1) study and design costs, 2) initial construction costs, 3) infrastructure and mitigation costs, 4) site development costs, 5) second dike raising costs, and 6) dredging, transport and placement costs during the life of the facility. The following list describes the project costs.

- Study and Design Costs: These costs were estimated based on study and design costs from past projects.
- Initial Construction Costs: These include site preparation costs, excavation and placement costs for overburden and borrow material, containment structure construction costs, and installation of spillways/outlet structures and site infrastructure.
- Infrastructure and Derelict Vessel Remediation: These include the estimated costs for the relocation and demolition of existing infrastructure (outfalls, watermain, piers, etc.), the remediation and relocation of the derelict vessels, and the mitigation projects associated with the DMCF.
- Mitigation Costs: These include the estimated costs for the mitigation and community enhancement projects associated both with Masonville Cove and off-site.

- Site Operational Costs: These include the estimated costs of annual dredged material management, site maintenance, and site monitoring/reporting for the operational life of the site. The cost estimates for site development are generated based on historical data for similar sites.
- Second Dike Raising Costs: These costs include estimates for second dike raising using suitable material (on-site dried dredged material and/or common borrow material).
- Dredging, Transport, and Placement (DTP) Costs: These include estimated costs for mobilization and demobilization, dredging, transport to the placement site, and unloading of the dredged material at the placement site for the operational life of the site.

The total site cost is the sum of the above plus a contingency cost equal to 20 percent of the sum of all above listed costs. Table 4-3 summarizes the costs for the project. A detailed breakdown of project costs, with the exception of the Wet Basin, is presented in Appendix F. The costs for the Wet Basin are included in the initial construction costs in Table 4-3. Refer to M&N (2005b) for the calculation of Wet Basin construction costs. Costs are presented in 2005 dollars.

Table 4-3. Project Costs

Line Item	Cost
Study and Design (millions)	$3
Initial Construction (millions)	$54
Infrastructure (millions)	$16.5
Mitigation (millions)	$12.5
Site Operations (millions)	$18
Second Dike Raising (millions)	$20
Dredging, Transportation, and Placement for 16 mcy (millions)	$122
Total (millions)	$246
Approximate Total Unit Cost ($/cy capacity)	$15

Note: Values presented in 2005 dollars. The total does not equal the sum of line items due to rounding.

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