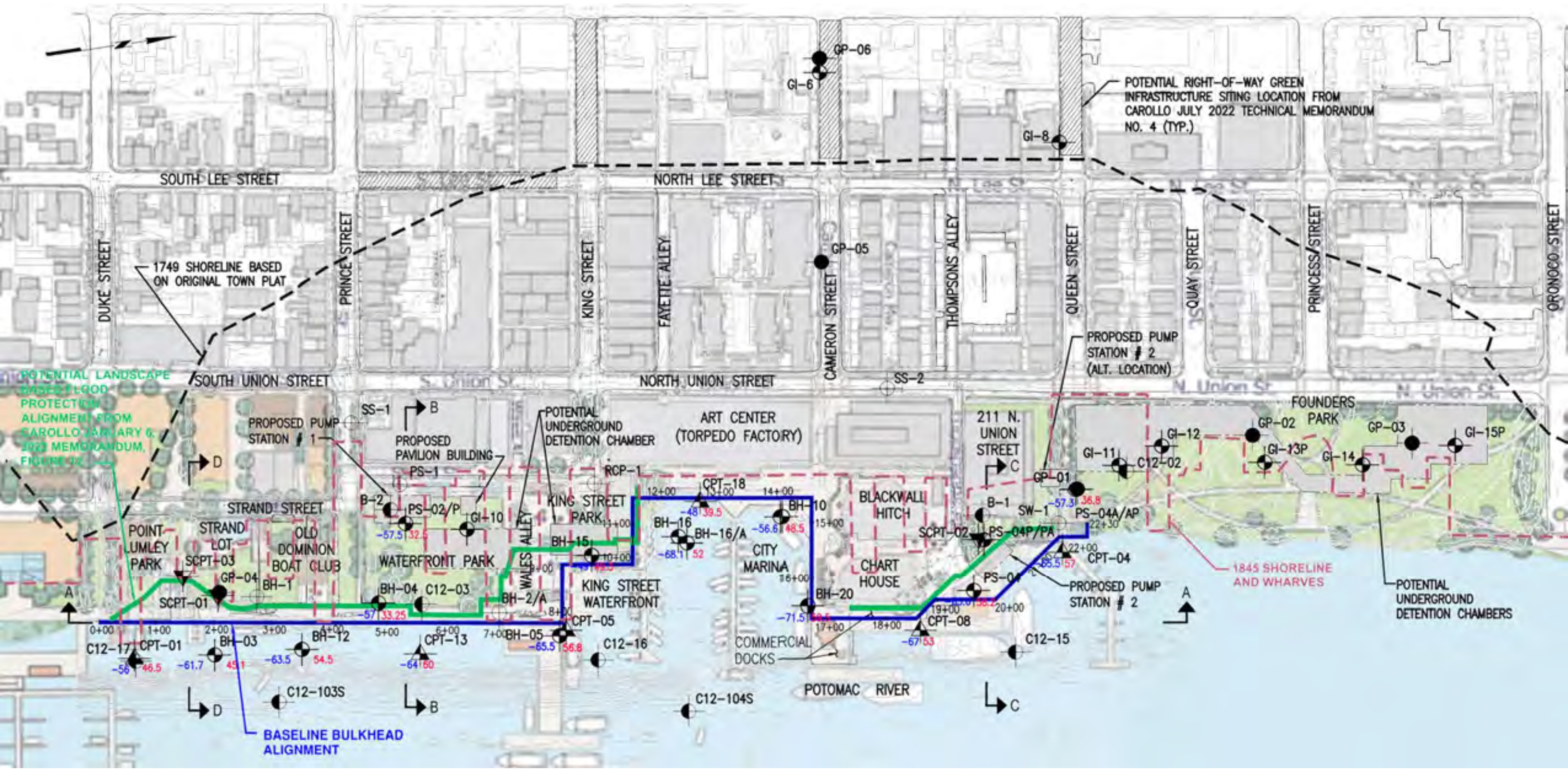
GEOTECHNICAL DESIGN MEMORANDUM



City of Alexandria, VA Waterfront Implementation Project
Phase II Geotechnical Exploration

Carollo Engineers, Inc.
530 Seventh Avenue, Suite 2402
New York, NY 10018

September 16, 2022

MRCE
built on firm foundations

NEW YORK CITY | PHILADELPHIA | WASHINGTON, DC

built on firm foundations

September 16, 2022

Carollo Engineers Inc.
530 Seventh Avenue, Suite 2402
New York, New York, 10018

Attn: Ms. Sara Igielski, PE

Re: **Geotechnical Design Memorandum**
City of Alexandria Waterfront Implementation
Phase II Geotechnical Exploration
Alexandria, Virginia
MRCE File 14123

Greetings:

Mueser Rutledge Consulting Engineers (MRCE) is pleased to submit this Geotechnical Design Memorandum (GDM) for the City of Alexandria Waterfront Implementation Project, Phase II Geotechnical Exploration.

This GDM is prepared in accordance with authorized Task Order No. 2. The engineering analysis and recommendations herein are based on geotechnical data collected in the 2021-2022 Phase II Geotechnical Exploration and records of prior investigations, as presented in the Geotechnical Data Report (GDR).

Very truly yours,

MUESER RUTLEDGE CONSULTING ENGINEERS PLLC

Walter E. Kaeck

Walter E. Kaeck, Principal, P.E.

Raj S. Chinthamani, Associate

Aaron L. Sacks, Associate

1.0 TABLE OF CONTENTS

1.0 TABLE OF CONTENTS 2
2.0 EXHIBITS 4

PART 1 – PURPOSE AND PROJECT BASELINE 5

3.0 PURPOSE 5
4.0 ELEVATION DATUM AND HORIZONTAL CONTROL 5
5.0 PROJECT AREA DESCRIPTION 5
6.0 WATERFRONT HISTORY AND PRESENT CONDITION 6
7.0 POTENTIAL WATERFRONT FLOOD MITIGATION IMPROVEMENTS 8
8.0 RIVERINE FLOOD PROTECTION ALIGNMENT 8

PART 2 – SUBSURFACE CONDITIONS AND SOIL PARAMETERS 9

9.0 SUBSURFACE INVESTIGATIONS 9
9.1. Phase II Geotechnical Exploration 9
9.2. Prior Subsurface Investigations 9
9.3. Waterfront Site Walk 9
10.0 SUBSURFACE CONDITIONS 10
10.1. Stratum F – Fill 10
10.2. Stratum A – Recent Alluvial Deposits 10
10.3. Stratum T – Pleistocene Terrace Deposits 13
10.4. Stratum P – Cretaceous Deposits 13
11.0 GROUNDWATER AND DESIGN WATER LEVEL 15
11.1. Measured Groundwater Level 15
11.2. Flood Hazard and Design Groundwater Level 17
11.3. Future Sea Level Rise 17
12.0 SOIL PARAMETERS FOR CONCEPTUAL DESIGN 17
13.0 SEISMIC DESIGN PARAMETERS 18
13.1. Liquefaction Evaluation 18
13.2. Seismically Induced Settlement 18
13.3. Site Class and Recommended Design Spectrum 18
13.4. Seismic Lateral Pressures 18

PART 3 – BULKHEAD REPLACEMENT / REUSE OPTIONS 20

14.0 BULKHEAD OPTIONS & BASIS OF ANALYSIS 20
14.1. Objectives 20
14.2. Existing Bulkhead Conditions 20
14.3. Bulkhead Improvement Options 21
14.4. Bulkhead Design Criteria 22
14.5. Representative Bulkhead Design Sections 22
14.6. Conceptual Bulkhead Design Analyses 23
15.0 BULKHEAD REPLACEMENT OPTIONS 23
15.1. Baseline Design Concept - Batter Pile Bulkhead 24
15.2. Option 1 - Cantilever Bulkhead (Combined Wall System) 25
15.3. Option 2 - Anchored Sheet Pile Bulkhead 27
15.4. Design Life and Maintenance Considerations 29
15.5. Offset Bulkhead Alignment and Waterside Fill Placement 30
15.6. General Bulkhead Replacement Considerations 31
15.7. Bulkhead Replacement Constraints Specific to Individual Sections 32

15.8. Future-Proofing and Outboard Promenade *32*
16.0 REUSE OF EXISTING BULKHEAD 32
16.1. Localized Bulkhead Repairs *33*
16.2. Inland Landscape-Based Options *33*
16.3. Addition of Deployable Barriers to Top of Existing Bulkhead *35*
16.4. Raising Existing Bulkhead to El. +6 .... *35*
16.5. Limitations of Bulkhead Reuse Options *36*
17.0 GENERAL BULKHEAD CONSIDERATIONS 36
17.1. Settlement under Grade Raise Fill *36*
17.2. Boating Accessibility *36*
17.3. Permitting Considerations *37*
17.4. Summary of Conceptual Bulkhead Replacement / Reuse Options *38*

PART 4 – PUMP STATIONS AND OTHER STRUCTURES 40

18.0 PUMP STATIONS 40
18.1. Foundation Type and Capacity *40*
18.2. Grade Raise and Downdrag at Pump Stations *42*
18.3. Pile Load Testing *42*
18.4. Bottom Slab Design *42*
18.5. Foundation Wall Design *43*
18.6. Foundation Uplift & Waterproofing *43*
18.7. Construction Considerations *43*
18.8. Pump Station No. 2 (Alternative Location): *45*
18.9. Alternative Approaches at Pump Stations *45*
19.0 STORMWATER ATTENUATION FEATURES 46
19.1. Right-of-Way Green Infrastructure *46*
19.2. Underground Detention Chambers *47*
20.0 GENERAL FOUNDATION DESIGN CONSIDERATIONS 48
20.1. Lightly Loaded Foundations *48*
20.2. Corrosion Potential and Chemical Attack *48*
20.3. Subgrade Preparation for Shallow Foundations and Pavements *49*
20.4. Backfill *49*
21.0 PAVEMENT DESIGN 49
22.0 INSTRUMENTATION AND MONITORING 51
23.0 SUPPLEMENTAL GEOTECHNICAL INVESTIGATION 53
24.0 CLOSURE 53
25.0 REFERENCES 55
25.1. Project References *55*
25.2. Literature References *55*

2.0 EXHIBITS

The following exhibits are included in this Report:

Table 1	Conceptual Design Soil Parameters
Table 2	Conceptual Design Lateral Earth Pressures
Table 3a	Baseline Bulkhead Design (Moffat & Nichol)
Table 3b	Baseline Bulkhead Design (MRCE Evaluation)
Table 4	Conceptual Cantilever Bulkhead Design
Table 5	Conceptual Anchored Bulkhead Design
Table 6	Summary of Conceptual Bulkhead Replacement & Reuse Options
Table 7	Summary of Conceptual Pump Station Pile Types (50-ton Allowable Load)
Table 8	Right-of-Way Green Infrastructure Suitability
Table 9	Conceptual Flexible Pavement Section
Table 10	Conceptual Monitoring Plan
Table 11	Recommended Supplemental Geotechnical Investigation
Figure 1	Project Area Location Plan
Figure 2	Historic Shoreline
Figure 3	Soil Properties Profile – Stratum A1 (Land Borings)
Figure 4	Soil Properties Profile – Stratum A1 (Water Borings)
Figure 5	Stratum P1 Plasticity Data
Figure 6	Stratum P1 Undrained Shear Strength Data
Figure 7	Groundwater Level Data (Manual Readings)
Figure 8	Groundwater Level and Tide Data Comparison
Figure 9	Existing Bulkhead Sections Identified in January 2022 Site Walk
Figure 10	New Bulkhead Alignment and Landscape-Based Flood Protection Alignment
Figure 11	Baseline Bulkhead Concept
Figure 12	New Cantilever Bulkhead Concept
Figure 13	New Cantilever Bulkhead Concept (Offset Alignment)
Figure 14	New Anchored Bulkhead Concept
Figure 15	Landscape-Based Flood Protection Concept
Drawing BLP-1	Boring Location Plan
Drawing GS-1	Geologic Section A-A (Sheet 1 of 4)
Drawing GS-2	Geologic Section A-A (Sheet 2 of 4)
Drawing GS-3	Geologic Section A-A (Sheet 3 of 4)
Drawing GS-4	Geologic Section A-A (Sheet 4 of 4)
Drawing GS-5	Geologic Section B-B
Drawing GS-6	Geologic Section C-C
Drawing GS-7	Geologic Section D-D
Drawing GS-R	Geotechnical Reference Standards
Appendix A	January 10, 2022 Bulkhead Site Walk Findings
Appendix B	Typical Monitoring Instrumentation Details

PART 1 – PURPOSE AND PROJECT BASELINE

3.0 PURPOSE

The purpose of this Geotechnical Design Memorandum (GDM) is to:

- Discuss and interpret collected geotechnical data included in the Geotechnical Data Report (GDR, Ref. 1)
- Evaluate how the subsurface conditions may affect alternative approaches to project design and construction
- Evaluate project geotechnical risks as a function of alternative construction approaches
- Assess potential construction impacts on adjacent facilities
- Provide preliminary geotechnical design criteria for permanent and temporary structures
- Identify needs for additional investigation

A Phase I Geotechnical Engineering Report (GER) was prepared by Schnabel in 2017 (Ref. 3) and included general design and construction recommendations. This Phase II GDM builds on the Phase I GER to incorporate results of the Phase II Exploration and provide additional recommendations suited to the current conceptual design. Recommendations in the Phase I GER are referenced where applicable. Phase I GER recommendations remain applicable except where specifically noted herein. Where discrepancies exist between the GDM and the GER, the GDM takes precedence.

Progressive design-build (PDB) is the project delivery method. This GDM is intended to provide suitable interpretation and analysis of the geotechnical data collected in the Phase I and Phase II Geotechnical Explorations to allow the Owner to develop the conceptual design included in the PDB procurement documents. Conceptual level designs for alternative bulkhead improvements and foundations for proposed structures are presented herein to assist the Owner in that effort and memorialize the alternative concepts considered in developing the PDB documents. The design-builder is responsible for performing all necessary additional geotechnical investigation and analysis to progress the conceptual design shown in the PDB procurement documents to final design and construction. The GDM is not intended for use as a basis for PDB cost proposals or to establish differing site conditions during construction.

The GDM is focused on geotechnical design and construction considerations. We refer to the environmental and geoarchaeological reports provided in the GDR (Ref. 1) for environmental and geoarchaeological conditions affecting design and construction.

4.0 ELEVATION DATUM AND HORIZONTAL CONTROL

Elevations refer to the North American Vertical Datum of 1988 (NAVD88). Coordinate locations reference Virginia State Plane North, NAD83 (2011).

5.0 PROJECT AREA DESCRIPTION

The evaluated project area encompasses several city blocks that span approximately one half mile of the Potomac River shoreline in Old Town Alexandria, Virginia. The project area is bounded by Duke Street to the south, Oronoco Street to the north, Fairfax Street to the west, and the Potomac River to the east as shown in Figure 1. The project area consists of residential buildings, commercial buildings, public parks, public roadways, and public and private docks/piers, and includes approximately 2,200 linear feet of manmade bulkhead and shoreline. Ground surface elevation ranges from approximately El. +2 to +7 at the waterfront and slopes up to the west (inland), reaching approximately El. +30 at Fairfax Street near the western limit of the project area. Elevation of the river bottom (mudline) outboard of the shoreline ranges from El. 0 to El. -20 and generally slopes down from the bulkhead/shoreline toward the pier line. Landside surface topography and bathymetry data showing the variation in mudline are depicted on Drawing BLP-1 and the enlarged plans accompanying the geologic sections on Drawings GS-1 through GS-7.

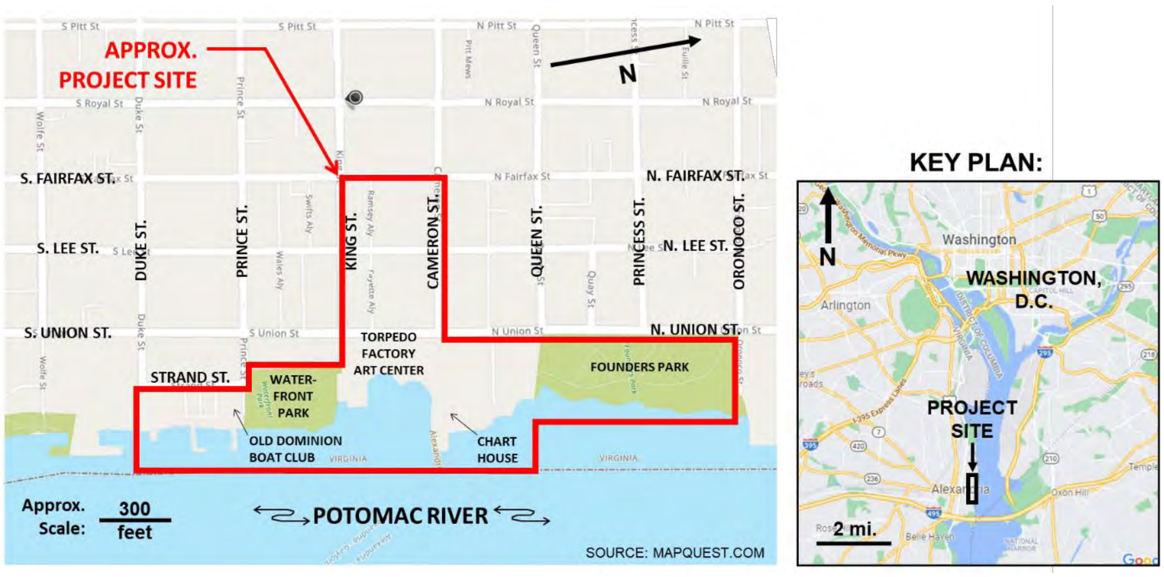


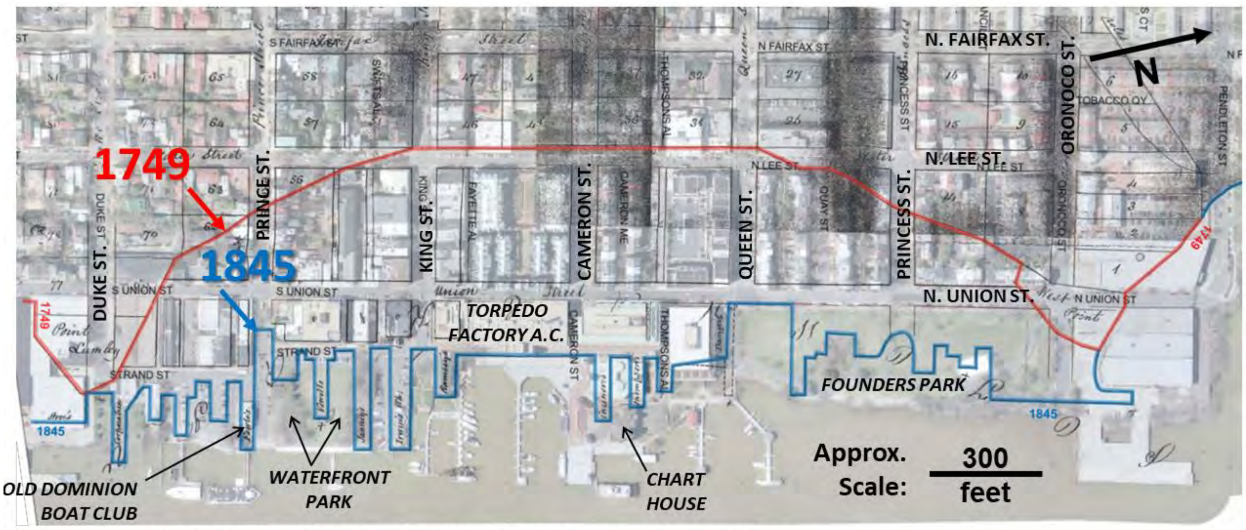
Figure 1. Project Area Location Plan

The GDM is not a scoping document and therefore does not dictate the area of disturbance for the project. Rather, it analyzes and interprets the field data collected and reported in the GDR. Note that the data collection area for the GDR extended outside the "Core Area" designated as the project area in the City of Alexandria's Waterfront Small Area Plan (WSAP).

6.0 WATERFRONT HISTORY AND PRESENT CONDITION

An understanding of the history of the waterfront is key to understanding the likely extent and characteristics of existing fill materials, potential relict structures within the fill, and to inform risk-based decisions on future development. As noted in the Stantec Initial Archaeological Assessment (Ref. 17), the remnants of as many as 11 historic wharves and 22 associated structures and their uses may remain preserved within the fill. The presence and potential archaeological significance of these structures is a factor in the design, construction, and cost of alternative plans for waterfront construction. Historic facts relevant to current geotechnical conditions are summarized in this section. Refer to Ref. 17 for a more detailed history.

Figure 2 shows the historic (1749) shoreline relative to the modern-day waterfront. In Alexandria's earliest days, the shoreline lay well inland of the current waterfront. Development of the waterfront progressed in increments by "*cutting the high riverfront banks and placing the excavated soils along the river behind wood pilings to prevent erosion. This effort leveled the shoreline and increased waterfront access and acreage. The City then gave adjacent landowners the rights to develop the* *newly made land" (Ref. 17).* As a result, much of the area east of Lee Street (east of the red line in Figure 2) is filled land, with some of the fill soil expected to be derived from upland areas.



SOURCE: CITY OF ALEXANDRIA DEPARTMENT OF PLANNING AND ZONING
www.alexandriava.gov/special/waterfront/default.aspx?id=24244 (Accessed Feb. 14, 2022)

Figure 2. Historic Shorelines in 1749 (red line) and 1845 (blue line)

In the 1800s, numerous finger piers, bulkheads and other structures were constructed on an alignment inland of the present-day shoreline as shown by the blue outline in Figure 2. Remnants of these former structures likely remain buried beneath the waterfront and may include timber piles, cribbing, rip-rap / stone, concrete, and other debris.

The modern-day waterfront came to form in the 20th century. The United States government built the **Torpedo Factory** at the end of King Street to produce torpedoes as part of the World War I effort. With the onset of World War II, the Torpedo Factory grew to include 10 buildings. After World War II, the Alexandria waterfront largely transformed to support leisure activities such as tourism and recreation. The **Old Dominion Boat Club** was established at the foot of Prince Street, and the Torpedo Factory was purchased by the City and repurposed to the current **Torpedo Factory Art Center**. Today, the Alexandria waterfront is a thriving destination for tourism and leisure activities with notable restaurants such as the **Chart House**, city owned and privately operated marinas, and large swaths of parkland and public space including **Founders Park** and **Waterfront Park**.

The present-day shoreline is formed by a mix of various types of steel and concrete retaining walls, stone filled crib walls, and partially armored slopes (referred to collectively hereafter as bulkheads) of various age and condition that support and partially protect the shoreline and inland areas from water action and elevated water levels of the Potomac River. However, ground surface along most of the shoreline ranges from only El. +2 to +4, except in the limited length along the Torpedo Factory and City Marina wharves where ground surface is at El +7.

The low present-day surface elevation along the shoreline allows inland areas to routinely flood during seasonal high tides and storm surges due to both overtopping and backflow through existing storm sewer outfalls in low lying areas along the shoreline (since outfalls are not equipped with backflow prevention). Additionally, rainfall frequently induces flooding in Old Town Alexandria because the existing storm sewer network is both undersized and tidally influenced. As a result, frequent nuisance flooding of lower King Street, segments of North and South Union Streets, and Strand Street have troubled Alexandria for years and caused damage to property and disruption to businesses. A potentially higher future Potomac River elevation (further discussed in Section 11.3) will further exacerbate these flooding conditions.

7.0 POTENTIAL WATERFRONT FLOOD MITIGATION IMPROVEMENTS

The Waterfront Implementation Project is intended to mitigate high frequency flooding and enhance access to the waterfront through a series of planned improvements. The planned progressive design-build (PDB) delivery method facilitates a “scope to budget” approach. Therefore, multiple alternative improvement types are under consideration. The goal of the PDB process is to engage the creativity and innovation of the Design Builder during initial (“Phase 1”) services to optimize the design within available funding. Flood mitigation design elements under consideration include:

- construction of a new bulkhead to El. +6 between Duke Street and Queen Street to mitigate flooding and provide a continuous promenade along the waterfront.
- re-use of portions of the existing bulkhead combined with landscape-based flood protection measures inland of the bulkhead
- construction of two new below-grade pump stations (Nos. 1 and 2) with surface access buildings located approximately in Waterfront Park and at the foot of Thompson’s Alley
- implementation of Virginia Best Management Practices (BMPs) including:
    - right-of-way (ROW) green infrastructure (GI), e.g. permeable pavement
    - greener park features such as bioretention and underground detention chambers
- storm sewer capacity and conveyance improvements

Drawing BLP-1 illustrates the approximate locations of the potential new bulkhead, landscape-based flood protection alignment, pump stations, ROW GI siting, and underground detention chambers. The reader is referred to “Technical Memorandum 4 – Parkspace and Streetscape Stormwater Attenuation Solutions” prepared by Carollo (Ref. 18) for the fully envisioned plan for ROW GI implementation. For the location of planned storm sewer improvements, refer to the Master Storm Water Management Plan prepared by Stantec (Ref. 4).

This report independently evaluates the geotechnical aspects of various flood mitigation design components but does not prioritize options. The reader is referred to the Conceptual Design Report by Carollo for scope prioritization relative to available funding.

8.0 RIVERINE FLOOD PROTECTION ALIGNMENT

A new or improved bulkhead is an important component of the Waterfront Implementation Project because it provides flood mitigation while enhancing waterfront access by providing a continuous promenade along the waterfront. Drawing BLP-1 shows the existing and proposed (blue) bulkhead alignment according to the Baseline Project Plan (Refs. 9, 11). The **Baseline Bulkhead Alignment** is intended to simplify and straighten the shoreline and reduce zones of stagnant water and debris collection. However, the bulkhead realignment will require significant offset east of the existing bulkhead in some areas, particularly near Thompsons Alley and Point Lumley Park, and may incur increased regulatory scrutiny to permit infilling of the Potomac River to raise grade behind the new bulkhead (further discussed in Section 17.3).

Alternative Landscaped Flood Protection Alignments (green line on Drawing BLP-1) are also proposed to reduce the disruption and cost of construction. Landscaping options improve flood protection by raising inland grades to El. +6 using sloping terrain or low height retaining walls (‘Ha-Ha’ walls), stairs, or other architectural features but rely on extending the service life of the existing bulkhead by repair and maintenance.

PART 2 – SUBSURFACE CONDITIONS AND SOIL PARAMETERS

9.0 SUBSURFACE INVESTIGATIONS

9.1. Phase II Geotechnical Exploration

MRCE performed a Phase II Geotechnical Exploration along the waterfront between November 2021 and January 2022. The Phase II exploration consisted of 16 soil borings and Cone Penetrometer Test (CPT) probes made landside of the existing bulkhead and 15 soil borings and CPT probes advanced waterside of the bulkhead. The exploration also included archaeological geoprobes, borehole infiltration testing, geotechnical laboratory testing and a one-day site walk of the waterfront to observe the condition of visible elements of the existing bulkhead. We refer to the Phase II GDR (Ref. 1) for a more detailed description of the Phase II Geotechnical Exploration and summary of the results of field and laboratory testing.

9.2. Prior Subsurface Investigations

Records of prior subsurface investigations made in the project area were collected and incorporated in the findings of the Phase II Geotechnical Exploration. These prior investigations include:

- 2014 preliminary geotechnical investigation for the Alexandria Waterfront Project consisting of 2 borings at the approximate pump station locations, made by URS (Ref. 2)
- 2016 Phase I investigation for the Alexandria Waterfront Project consisting of 7 land-based borings made by Schnabel Engineering (Ref. 2)
- 2018/2019 borings made for Alexandria RiverRenew Tunnel System project (2 land borings and 5 water borings in project area)

Locations of the borings made in the Phase II Geotechnical Exploration and prior investigations made in the project area are shown on Drawing BLP-1. Data obtained from these investigations are the basis of the engineering analysis and conceptual design recommendations described herein.

9.3. Waterfront Site Walk

MRCE performed a one-day site walk of the project area waterfront with Carollo Engineers and the City of Alexandria Project Implementation Team on January 10, 2022. The objective of the site walk was to observe the condition of visible elements of the existing bulkhead and facilitate an evaluation of potential options for bulkhead reuse or replacement based on observed conditions. The walk covered the waterfront between Duke Street to the south and Queen Street to the north. We refer to Appendix A for a record of our site walk observations and findings.

10.0 SUBSURFACE CONDITIONS

Our interpretation of the subsurface conditions is illustrated on the geologic sections on Drawing Nos. GS-1 to GS-7 based on the Phase II Geotechnical Exploration and prior investigations. Boring information shown on the sections includes sample number and position, SPT resistance (N-Value) in blows per foot (bpf) and the Unified Soil Classification System (USCS) soil group symbol for each soil sample. Measured cone tip resistance with depth is also graphically summarized at the CPT probe locations.

General descriptions of principal soil strata encountered in the borings are described below in order of increasing depth.

10.1. Stratum F – Fill

Fill (Stratum F) placed to create land outboard of the historical shoreline covers most of the project area. Stratum F consists of loose to very compact brown, black, gray, and dark gray fine to coarse sand with varying amounts of gravel, silt, clay, brick, concrete, wood and roots. Some previous investigation borings also noted metal, glass, and ceramics in the fill. Standard Penetration Test hammer blow counts (termed “N-values”), a measure of soil density and stiffness, range from 1 (loose) to 152 (very dense) blows per foot (bpf), with an average of 17 bpf. The thickness of Stratum F in borings made along the waterfront in the Phase II Geotechnical Exploration ranges from 10 feet to more than 20 feet. Phase I borings SS-1 and BH-2A encountered greater fill thicknesses of 28.5 and 43.5 feet, respectively. Borings GI-6 and GI-8, made inland at higher elevations, encountered only 1 to 2 feet of fill.

Obstructions were encountered in many of the borings performed during the Phase II geotechnical exploration and prior investigations as summarized in Table D of the GDR (Ref. 1). Petroleum odor and/or staining in the fill were also noted in some borings. Where field screening indicated the potential presence of contaminants, a sample from this interval was selected for analytical testing by the environmental consultant. We refer to the Phase II Environmental Assessment Report attached to the GDR (Ref. 1) for the results of soil sample environmental testing.

Both obstructions and the presence of contaminants in Stratum F are expected due to the industrial history and varied use of the waterfront over the last 250 years. With this historical context in mind, excavations in this area will likely encounter buried remnant foundations, pilings, bulkheads, crib structures, pier structures, pavements, railways, etc., which add both construction cost and schedule risk.

10.2. Stratum A – Recent Alluvial Deposits

Alluvial river deposits (Stratum A) of relatively recent origin underlie the Stratum F fill landside and form the river bottom outboard of the shoreline. The alluvial soil deposits are divided into two separate strata (A1 and A2). Stratum A1 consists of primarily fine grained organic clay that exhibits plasticity and is moderately to highly compressible. Stratum A2 consists of predominantly loose to medium compact granular soils with slight amounts of organic matter.

10.2.1. <u>Stratum A1 – Alluvial Clay/Silt:</u>

Stratum A1 consists of very soft to medium gray, gray brown, and dark gray clay and organic clay with varying amounts of sand, silt, shells, wood, and roots. Stratum A1 typically consists of high plasticity clay (USCS soil group symbol OH). N-values in landside borings range from sampler penetration under the weight of the hammer (WH), indicating very soft consistency to 8 bpf (medium consistency) with an average of about 2 bpf (soft). N-values in waterside borings are somewhat lower with a range from sampler penetration under the weight of the drill rods (WR) to 10 bpf with an average of 1 bpf. Stratum A1 is present in all water borings and all land borings that penetrated Stratum F and varied between approximately 30 feet and 60 feet in thickness in borings that fully penetrated the stratum.

Natural moisture content of Stratum A1 samples in both land and water borings is typically near the liquid limit. In land borings, the liquidity index (calculated as the difference between a sample's natural water content and its plastic limit divided by the plasticity index) ranged from 0.6 to 1.3 with an average of 0.9. In water borings, the liquidity index ranges from 0.6 to 2.5 and averages 1.2.

Undrained shear strength (Su), equal to one-half of the measured compressive strength, measured in the laboratory generally increases with depth in Stratum A1 with a range of about 330 to 1,220 pounds per square foot (psf) in landside borings and about 100 psf to 930 psf in waterside borings.

Overconsolidation ratio (OCR), the ratio of the maximum past pressure measured in laboratory consolidation tests to the present effective overburden stress, ranges from 1.0 to 1.5 with an average of 1.2 in land borings (5 tests across 3 borings) and from 1.6 to 1.8 with an average of 1.7 in water borings (3 tests in 1 boring). The limited amount of overconsolidation in some land borings is likely related to historic use of the land which may have included stockpiling of bulk materials and may not be uniform throughout the project area. The reason for overconsolidation measured on the waterside in Boring PS-04 may relate to historic dredging which removed overburden load. Measured overconsolidation generally correlated well with liquidity index which should be useful in evaluating soil compressibility and stress history between borings with consolidation testing results.

A comparison of undrained shear strength, preconsolidation stress, natural water content and plasticity limits for Stratum A1 in land and water borings is provided in Figures 3 and 4, respectively. The undrained shear strength profiles recommended for conceptual design based on the test data are indicated on Figures 3 and 4.

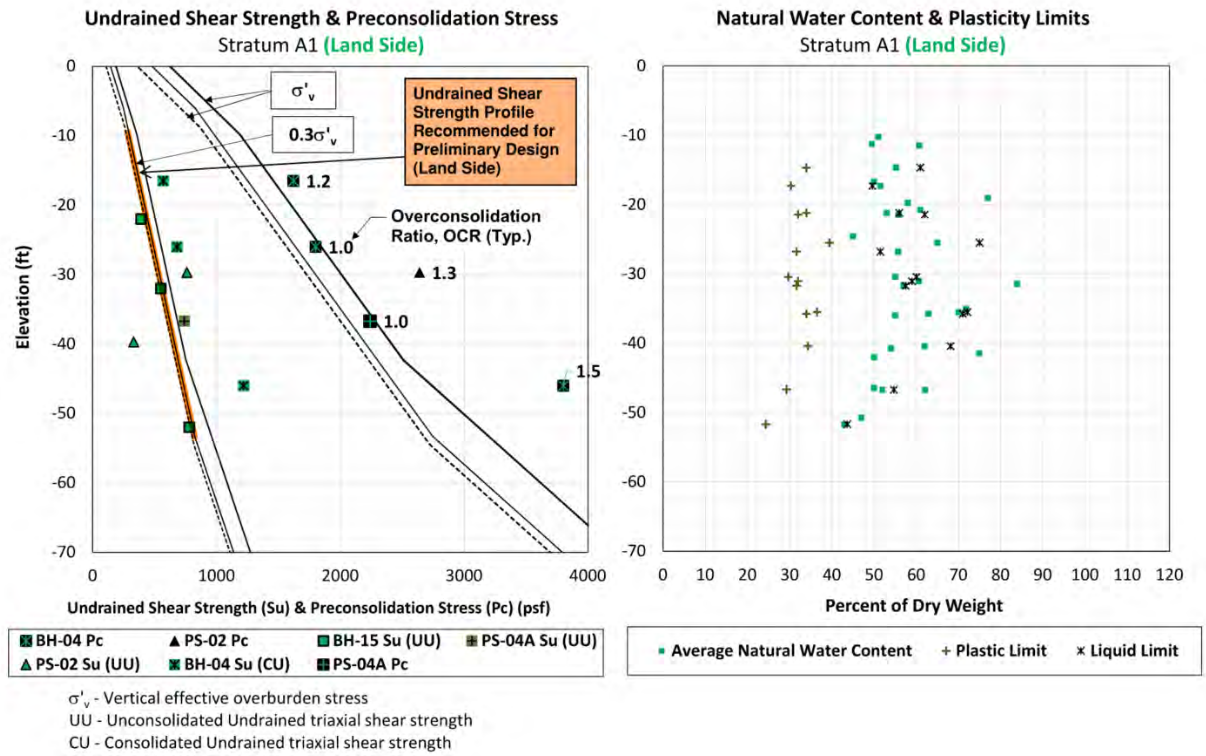


Figure 3. Soil Properties Profile – Stratum A1 (Land Borings)

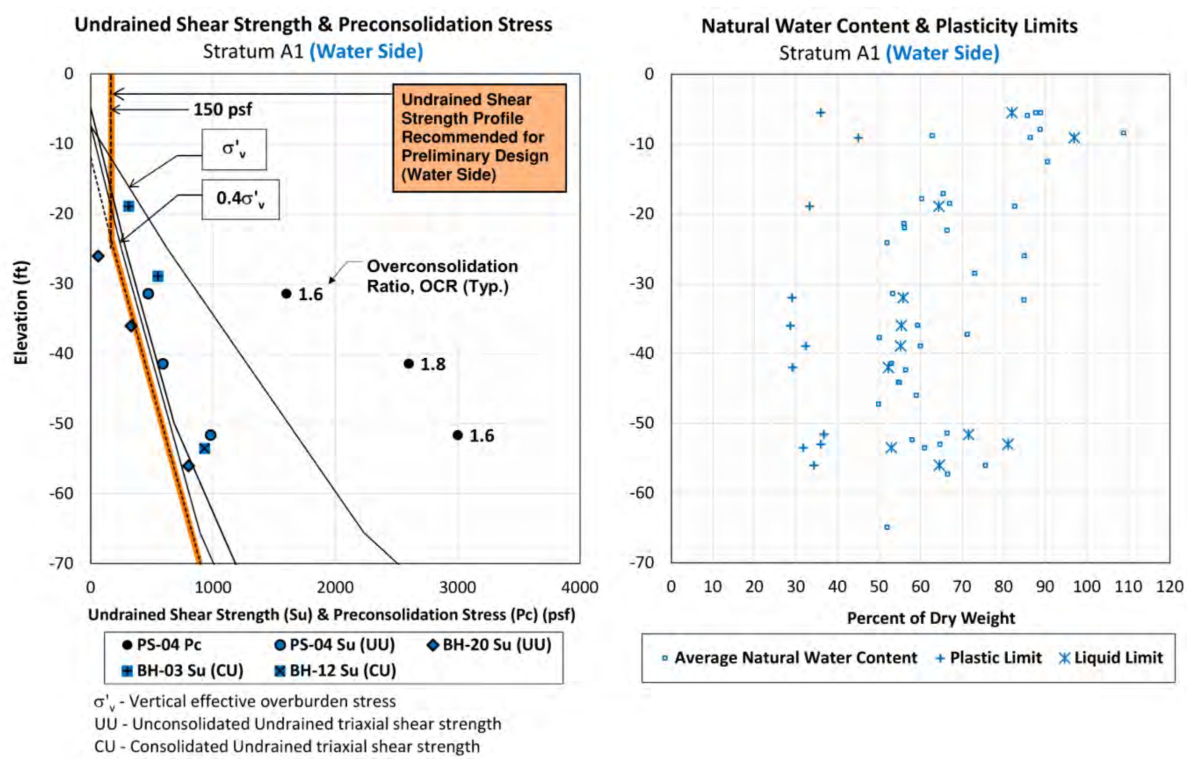


Figure 4. Soil Properties Profile – Stratum A1 (Water Borings)

Environmental sampling and testing performed in waterside borings in the Phase II exploration confirmed the presence of contaminants in the upper Stratum A1 sediment in some borings. The reader is referred to the Phase II Environmental Assessment Report attached to the GDR (Ref. 1) for environmental sediment sampling results and handling and disposal options for dredged soil.

10.2.2. Stratum A2 – Alluvial Sand:

Stratum A2 consists of loose to medium compact dark gray and gray silty fine sand and clayey fine sand with varying amounts of medium to coarse sand, shells, and gravel. N-values range from 1 (loose) to 27 (medium compact) bpf with an average of 8 bpf (loose). Stratum A2 is present in Phase II Borings BH-03, BH-04, BH-05, BH-12, BH-16/A, PS-02, and PS-04A and varies between approximately 5 feet and 15 feet in thickness.

10.3. Stratum T – Pleistocene Terrace Deposits

Pleistocene Terrace deposits (Stratum T) underlie the alluvial deposits and are divided into coarser grained Stratum T and finer grained Stratum T0.

10.3.1. Stratum T – Old Town Terrace:

Stratum T consists of loose to compact gray fine to coarse sand with varying amounts of silt, clay, and gravel, gray fine sandy clay, and gray gravel. N-values range from 3 (loose) to 50 (compact) bpf with an average of 17 bpf (medium compact). Stratum T is present in all Phase II borings except BH-03, BH-16/A, PS-04P, and GI-10 through GI-15 (shallow borings) and varies between approximately 3 feet and 33 feet in thickness.

In Borings GI-6 and GI-8 located inland of the historic Potomac River shoreline, Stratum T is subdivided into sub-strata T1 and T2. Stratum T1 is predominantly cohesive and consists of medium to stiff brown and red brown clay with varying amounts of fine sand and occasional sand layers. Stratum T2 is predominantly granular and consists of red brown fine sand with varying amounts of silt, gravel, and clay layers.

10.3.2. Stratum T0 – Organic Member of Terrace**:**

Stratum T0 consists of stiff gray fine sandy silt, trace medium to coarse sand, and stiff gray silt, trace fine sand and decomposed wood. N-values range from 11 to 17 bpf with an average of 14 bpf. Stratum T0 is only present in Boring PS-02 and is approximately 12 feet thick.

10.4. Stratum P – Cretaceous Deposits

10.4.1. Stratum P1 – Potomac Clay:

Cretaceous Clay (Stratum P1) is present beneath Stratum A2 or Stata T / T0 in all deep borings except BH-05, which was terminated in Stratum T. Stratum P1 consists of stiff to hard mottled gray, gray brown, red brown, gray, light brown and purple clay and silt with varying amounts of sand and gravel. Stratum P1 is generally a high plasticity clay (CH). N values range from 5 to 60 bpf with an average of 28 bpf. All deep borings were terminated in Stratum P1. A distinctive feature of the Stratum P1 clay is the presence of a marked secondary structure (slickensides and fissures) as the result of past soil movements and pressure release with a consequent wide variation in soil strength.

The results of Atterberg Limits performed on samples of the P1 clay are summarized in Figure 5. All samples plot as a high plasticity clay (CH). The Plasticity Index (PI) ranges from 38% to 78% with an average of 61%. Natural water contents of the P1 clays range from 24% to 39% and average 33%

with the water content typically plotting at or near the plastic limit. The liquidity index is an excellent indicator of a soil's geologic history and relative soil properties. A liquidity index near zero indicates that a soil is heavily overconsolidated and of low compressibility. The liquidity index of the Cretaceous clays generally ranges from 0.0 to 0.2 and averages 0.1.

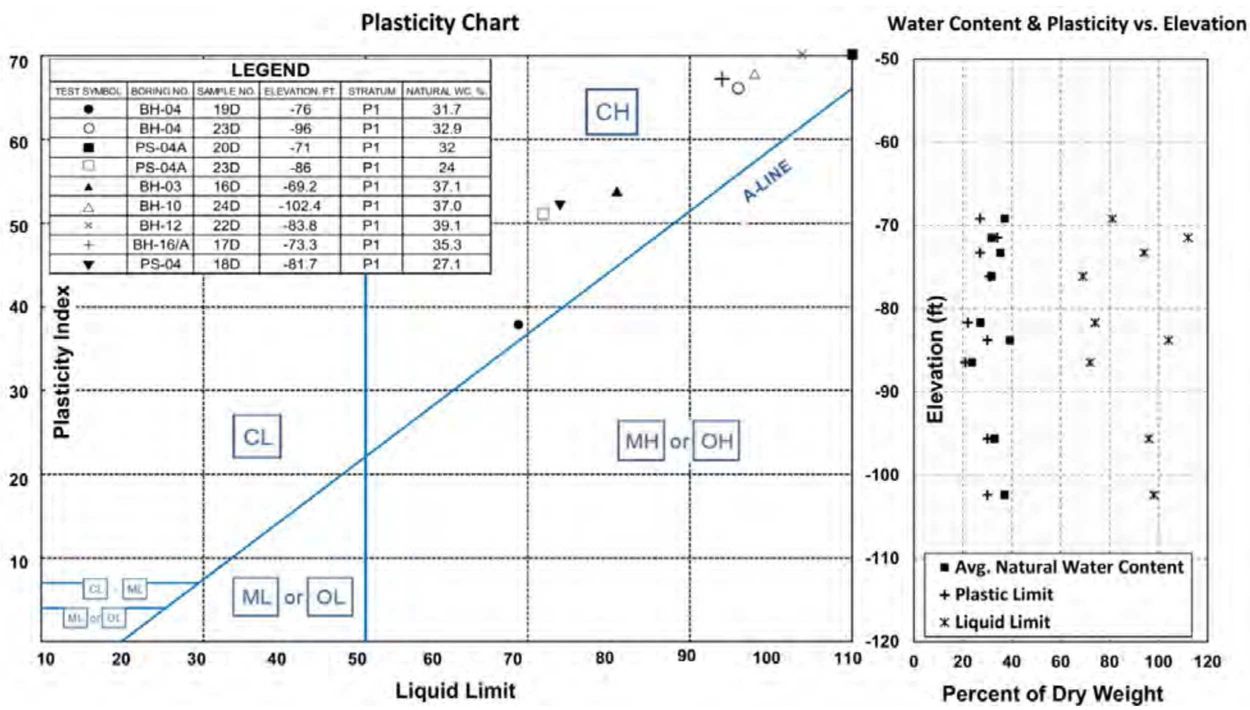


Figure 5. Stratum P1 Plasticity Data

Undrained shear strengths in the P1 clay are typically in the range of 4 to 10 ksf (4,000 to 10,000 psf) or higher (stiff to hard). This range has been directly measured by in-situ field testing (CPT and dilatometer) at the nearby Woodrow Wilson bridge and the DC Water Blue Plains Wastewater Treatment Plant, and is confirmed within this project area by correlation with SPT N-values from the Phase II Geotechnical Exploration. A comparison between the undrained shear strength measured at nearby sites and correlated with SPT N-values in the Phase II Geotechnical Exploration is provided in Figure 6.

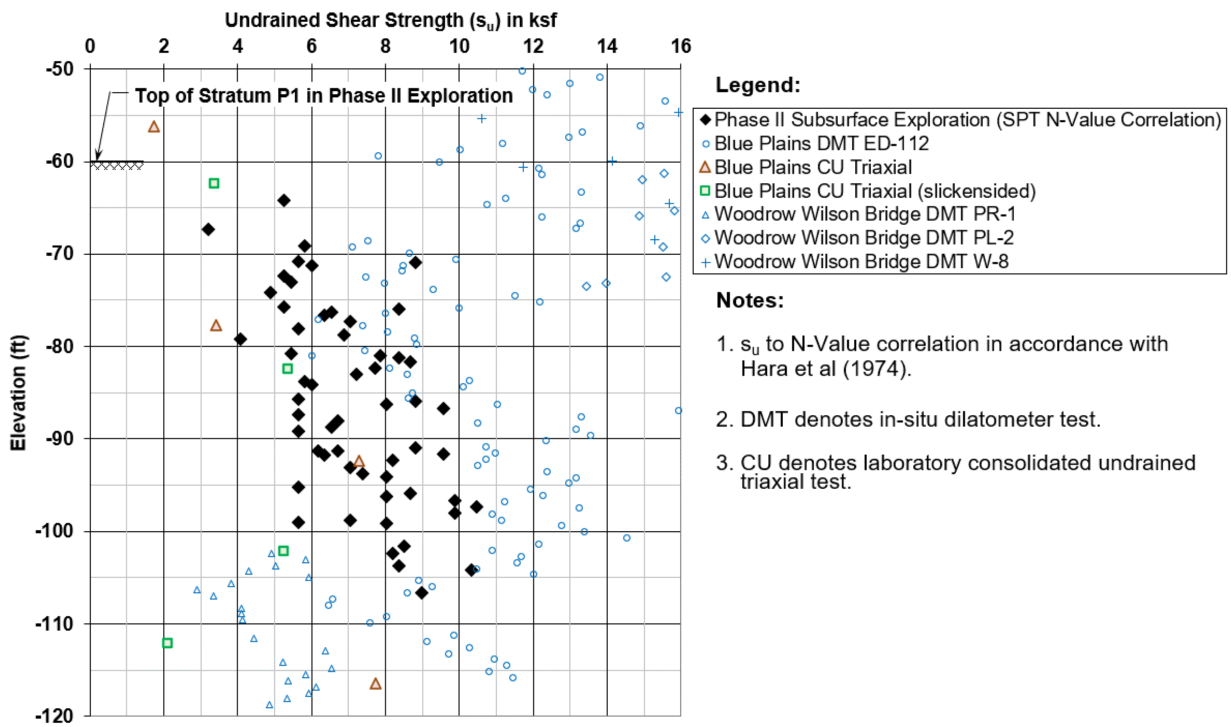


Figure 6. Stratum P1 Undrained Shear Strength Data

11.0 GROUNDWATER AND DESIGN WATER LEVEL

11.1. Measured Groundwater Level

Groundwater level measurements collected in five open-standpipe piezometers over the 8-month period from November 5, 2021 through June 27, 2022 ranged from El. -1.1 to +3.6, as shown in Figure 7. In addition to periodic manual measurements, data loggers were installed in three piezometers (GI-13P, PS-02P, and PS-04P) to collect continuous groundwater level data beginning in February 2022. A comparison of collected groundwater data and tide data from the Potomac River, Old Town Alexandria tide gauge is provided in Figure 8.

Groundwater levels showed tidal influence, becoming more muted with distance from the shoreline. At PS-04AP, approximately 20 feet inland, groundwater level nearly matched the Potomac River level ranging from Elev. -1.0 to +2.1 with a diurnal cycle. At PS-02P, approximately 175 feet inland, groundwater level reflected weekly tidal highs and lows but varied less overall, ranging from El. -0.1 to +1.3 during the monitoring period. At GI-15P, located in Founders Park about 180 feet inland of the shoreline, the groundwater level appears to be perched 1 to 2 feet above River level, ranging from El. +2.5 to +3.6. The reason for the perched water is unknown but could relate to the presence of buried remnant structures that act as an artificial barrier to downward migration of surface water infiltration.

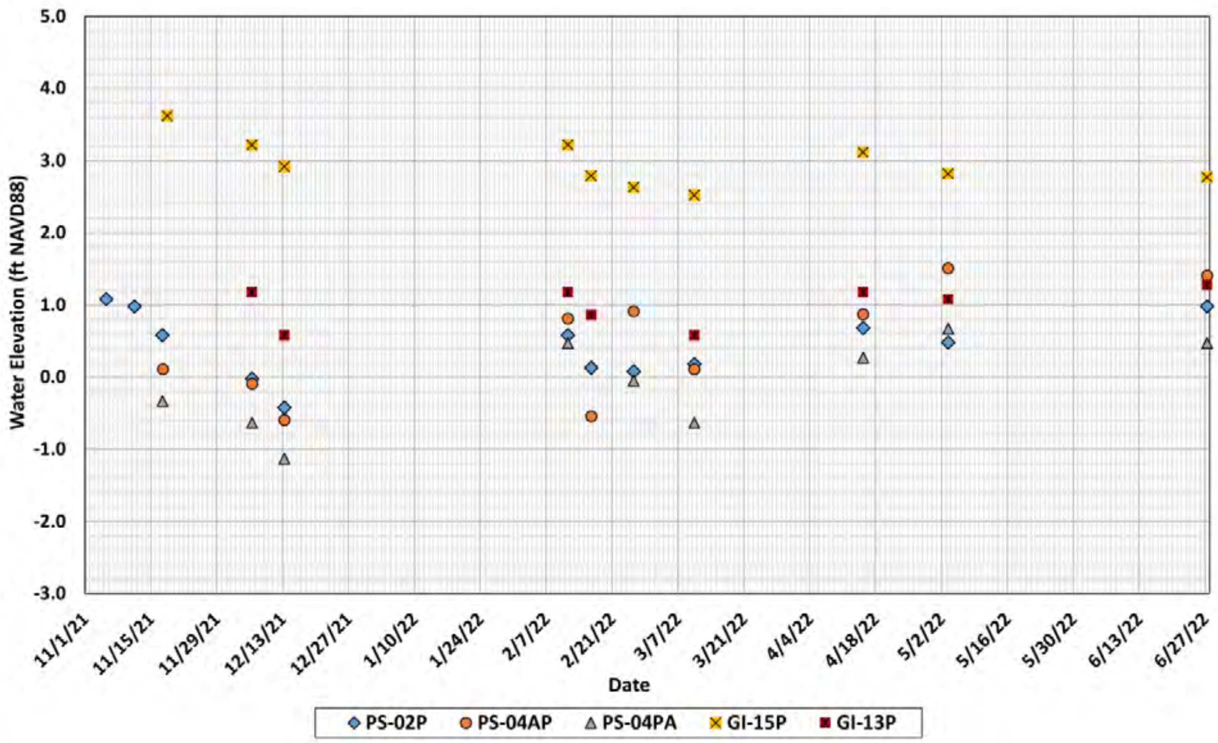


Figure 7. Groundwater Level Data (Manual Readings)

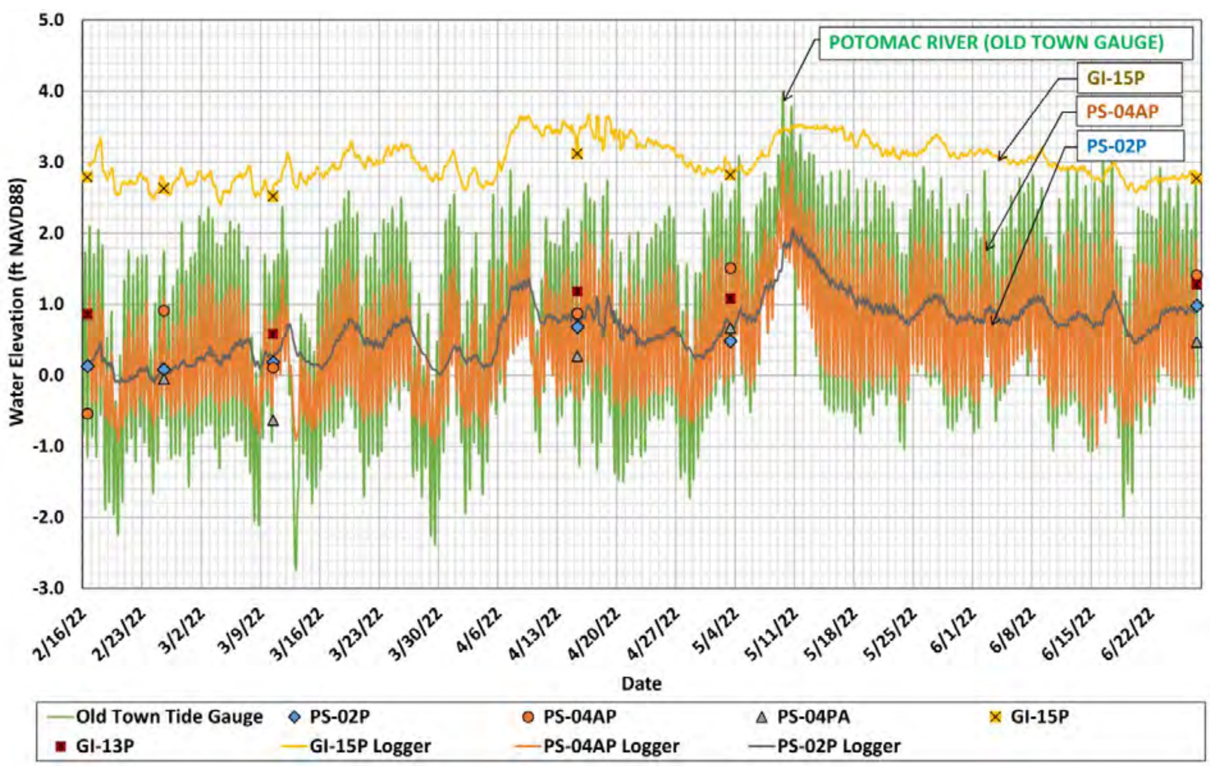


Figure 8. Groundwater Level and Tide Data Comparison

11.2. Flood Hazard and Design Groundwater Level

According to the Federal Emergency Management Agency (FEMA) Flood Insurance Rate Map (FIRM) No. 5155190041E (effective 6/16/2011), the evaluated project area is located within a mapped Special Flood Hazard Area, Zone AE with base flood elevation (BFE) +10. Permanent structures should be designed to withstand, as a minimum, the BFE +10.

11.3. Future Sea Level Rise

Future Sea Level Rise (SLR) is a consideration for design of flood protection infrastructure. A detailed discussion of SLR considering a range of recent projections is provided in Technical Memorandum 2, (TM2) Potomac River Flood Frequency Analysis prepared by Carollo (Ref. 19). In summary, TM2 recommends considering 2 feet of future sea level rise in development of the Waterfront Implementation Project.

12.0 SOIL PARAMETERS FOR CONCEPTUAL DESIGN

Soil parameters developed from the results of the Phase II geotechnical exploration for use in conceptual design of the waterfront improvements are summarized in Table 1.

Table 1 – Conceptual Design Soil Parameters

<table>
<tr><th rowspan="2">Stratum</th><th rowspan="2">Total Unit Weight (pcf)</th><th rowspan="2">Submerged (Buoyant) Unit Weight (pcf)</th><th colspan="2">Shear Strength</th><th colspan="2">LPILE Parameters for Conceptual Design (Note 2)</th></tr>
<tr><th>Drained Friction Angle, ϕ (°)</th><th>Undrained Shear Strength, s_u (psf)</th><th>Strain Values for Clay, ε_{50}</th><th>Horizontal Soil Modulus, k (pci)</th></tr>
<tr><td>New Fill (Normal Wt.)</td><td>125</td><td>61</td><td>30</td><td>-</td><td rowspan="3">n/a</td><td rowspan="3">30 - 60</td></tr>
<tr><td>New Fill (Lightweight)</td><td>70</td><td>6</td><td>30</td><td>-</td></tr>
<tr><td>F</td><td>120</td><td>56</td><td>30</td><td>-</td></tr>
<tr><td>A1 (Landside)</td><td>103</td><td>39</td><td rowspan="3">-</td><td>$s_u = 0.3\sigma'_v$</td><td rowspan="3">0.02</td><td rowspan="3">n/a</td></tr>
<tr><td>A1 (Waterside above El. -25)</td><td>98</td><td>34</td><td>150</td></tr>
<tr><td>A1 (Waterside below El. -25)</td><td>102</td><td>38</td><td>$s_u = 0.4\sigma'_v$</td></tr>
<tr><td>A2</td><td>125</td><td>61</td><td>32</td><td>-</td><td>n/a</td><td>90 - 100</td></tr>
<tr><td>T / T0</td><td>125</td><td>61</td><td>34</td><td>-</td><td>n/a</td><td>90 - 125</td></tr>
<tr><td>P1</td><td>125</td><td>61</td><td>-</td><td>4,000</td><td>0.005</td><td>n/a</td></tr>
</table>

Notes:

1. σ'_v = Vertical effective overburden stress
2. Ranges in parameters given reflect expected variability in soil properties and construction methods (i.e. degree of disturbance). Parameters should be refined during detailed design based on selected construction type at each location.

13.0 SEISMIC DESIGN PARAMETERS

Geotechnical and Structural Design of the proposed improvements must comply with the 2018 Virginia Construction Code (Code). The Code requires an evaluation of the seismic Site Class to determine the seismic design parameters and an assessment of the potential hazard of soil liquefaction under the seismic event specified by the Code.

13.1. Liquefaction Evaluation

Typically, loose granular soils and low-plasticity cohesive soils below the groundwater table are subject to potential liquefaction. Based on the presence of soils matching this description in the evaluated project area, we determined that a liquefaction evaluation was necessary. Soil plasticity data indicates that Stratum A1 is not susceptible to liquefaction according to Bray & Sancio (2006). Groundwater observations indicate that the depth to groundwater in the evaluated project area is only several feet below grade.

A simplified liquefaction assessment was performed to determine potential liquefaction susceptibility and to estimate the potential seismic induced settlements of granular deposits below the groundwater including the Stratum F fill and underlying Strata A2 and T sands. We evaluated liquefaction potential using the Seed and Idriss (1971) simplified procedure, as modified by the NCEER Workshops (Youd et al, 2001). The available SPT data was used to evaluate liquefaction resistance. Based on the presumed site class (neglecting liquefaction), the Code specifies analyzing liquefaction potential using a Peak Ground Acceleration adjusted for Site Class effects PGA_M of 0.165g for the Geometric Mean Maximum Considered Earthquake (MCE_G). The simplified liquefaction analysis shows that pockets of relatively loose sands exist in Strata F & A2 that may either liquefy or soften resulting in seismically induced settlement. However, available data indicate these looser pockets are discontinuous and limited in extent. Therefore, liquefaction will not impact site response.

13.2. Seismically Induced Settlement

We have estimated the magnitude of seismically induced settlement of the soil deposit (free-field), using the Tokimatsu & Seed (1987) and Ishihara & Yoshimine (1992) procedures to range between 0.25 and 2 inches for the purposes of designing connections with shallow supported utilities. However, it should be noted that accurately estimating seismically induced ground settlement is difficult especially using SPT data, and estimates should be considered approximate. Seismically induced settlements should be considered additional to any other settlement caused under static loading.

13.3. Site Class and Recommended Design Spectrum

Based on the Phase I and Phase II geotechnical exploration data, Site Class E is appropriate in accordance with the Code. The design spectral response acceleration parameters for the project area, S_{DS} and S_{D1}, are 0.213g and 0.121g, respectively. Assuming a Risk Category of I, II, or III, Seismic Design Category B is appropriate.

13.4. Seismic Lateral Pressures

The dynamic seismic lateral earth pressures on below-grade walls and bulkheads due to design earthquake ground motions should be based on the PGA_M. The Seed and Whitman (1970) or simplified Mononobe-Okabe method may be used to estimate the dynamic lateral earth pressures (with adjustments according to Gerali and Sitar, 2013 and Candia and Sitar, 2013). Estimated incremental seismic lateral pressures to be added to the static component of the lateral earth pressure are provided in Table 2.

Table 2 – Conceptual Design Lateral Earth Pressures

Stratum	Lateral Earth Pressure Coefficients		
	Active (wall top free to rotate)	**At-Rest (non-displacing walls)**	**Passive**
New Fill	0.33	0.50	3.0
F	0.33	0.50	3.0
A1	1.0	1.0	1.0
A2	0.31	0.47	3.25
T / T0	0.28	0.44	3.54
P1	n/a	n/a	n/a

<u>Notes:</u>

1. To compute lateral earth pressure in level ground, multiply vertical effective stress by the applicable lateral earth pressure coefficient at that depth. Add hydrostatic pressure to the computed lateral earth pressure.

2. Design foundation walls to resist lateral pressures due to the greater of:
   a. Prevailing groundwater level plus the greater of surcharge / seismic
   b. Base Flood Elevation +10 without surcharge / seismic

3. For 600 psf surcharge loading, add the following lateral pressures:
   a. 240 psf from 0 to 10 feet depth
   b. 100 psf from 10 to 20 feet depth

4. For seismic lateral loading, add the following equivalent fluid pressures:

Non-displacing walls:	Cantilever walls:
12 pcf for granular soils above groundwater level 6 pcf for granular soils below groundwater level 2 pcf for cohesive soils below groundwater level	5 pcf for granular soils above groundwater level 2.5 pcf for granular soils below groundwater level 0.5 pcf for cohesive soils below groundwater level

PART 3 – BULKHEAD REPLACEMENT / REUSE OPTIONS

14.0 BULKHEAD OPTIONS & BASIS OF ANALYSIS

14.1. Objectives

The proposed waterfront improvements span from Duke Street to Queen Street, a distance of approximately 2,230 linear feet. The existing shoreline is comprised of a mix of various types of steel and concrete retaining walls, crib walls, and partially armored slopes (referred to collectively hereafter as bulkheads) of various age that support and protect the shoreline and inland areas from water action and elevated water levels of the Potomac River. Ground surface elevation along most of the shoreline (except the Torpedo Factory and City Marina wharves) ranges from +2 to +4. This low shoreline elevation allows inland areas to routinely flood during seasonal high tides and storm surges.

Primary objectives of the proposed shoreline improvements defined by the City of Alexandria are to:

- Improve flood protection to El. +6
- Improve pedestrian access via a continuous promenade along the waterfront
- Accommodate future dredging outboard of the bulkhead to improve boating access
- Include allowances for extending flood protection to a higher elevation in the future, e.g. by the addition of flood barriers

14.2. Existing Bulkhead Conditions

Multiple prior investigations have documented conditions along the shoreline. Michael Baker Jr., Inc. (2013) performed a detailed structural inspection of 1,057 feet of the northern half of the bulkhead from the Torpedo Factory to Founders Park (Ref. 6). Moffatt & Nichol (2017) evaluated two segments of the shoreline (1,185 linear feet from Point Lumley to the Torpedo Factory and 548 linear feet from the Commercial Docks at the City Marina to Founders Park) and developed a preliminary design for a new bulkhead (Ref. 5) that was carried forward in the Project Baseline design for bulkhead replacement.

In January 2022, MRCE performed a 1-day landside visual investigation and subdivided the shoreline into 11 sections characterized by different bulkhead types and conditions (Ref. 12). The MRCE report dated February 14, 2022 in Appendix A includes a plan (reproduced as Figure 9 below) depicting the limits of the bulkhead sections with descriptions of visually observable bulkhead conditions at each section. Observations made in the January 2022 site walk are the basis of the potential bulkhead reuse options described in Section 16.0.

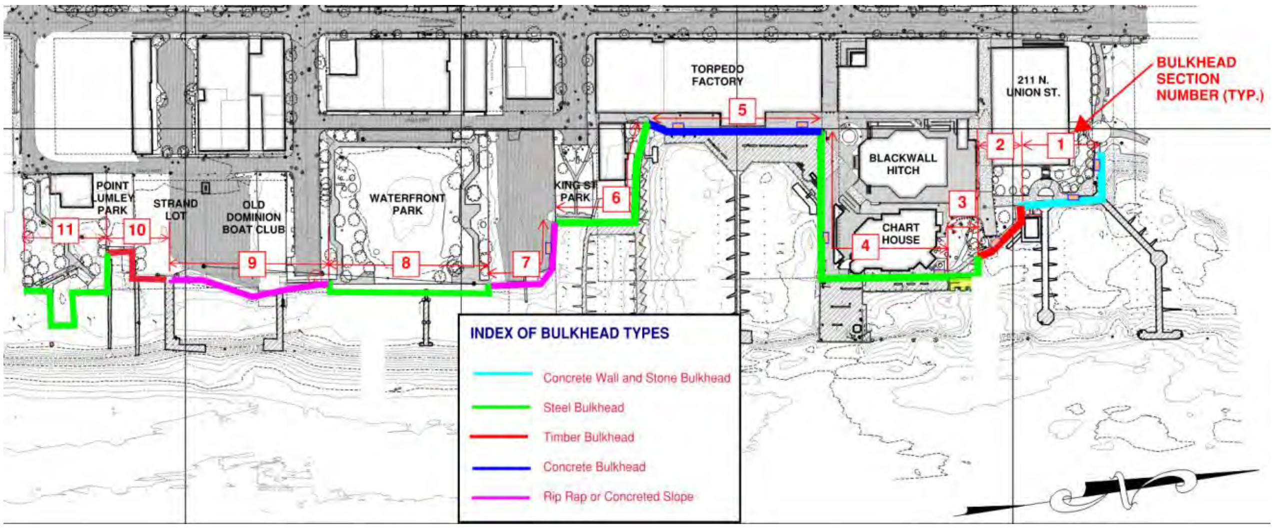


Figure 9. Existing Bulkhead Sections Identified in January 2022 Site Walk.

14.3. Bulkhead Improvement Options

Options for bulkhead improvements fall broadly into two categories:

1. **Replace bulkhead** to El. +6 outboard of the existing bulkhead or shoreline.
2. **Re-use the existing bulkhead** and add inland, landscape-based flood protection features (e.g. raised grade, shallow retaining walls, planter box walls, etc.) to El. +6. For any re-used sections of bulkhead, a detailed condition assessment is recommended to establish the existing bulkhead geometry and condition, understand remaining service life, and identify necessary repairs to extend the service life in accordance with that of the inland improvements.

The City of Alexandria Project Implementation Team is considering a range of scenarios for application of shoreline improvements. The Baseline Project would provide a new bulkhead over the full 2,230 linear feet of shoreline. Value Engineering options identified by the team would combine new bulkhead in some shoreline sections with re-use of the existing bulkhead and landscape-based flood protection in other sections. Conceptual plan alignments for each option (new bulkhead and landscape-based flood protection) as summarized by Carollo (Ref. 11) are shown on Drawing BLP-1 and Figure 10.

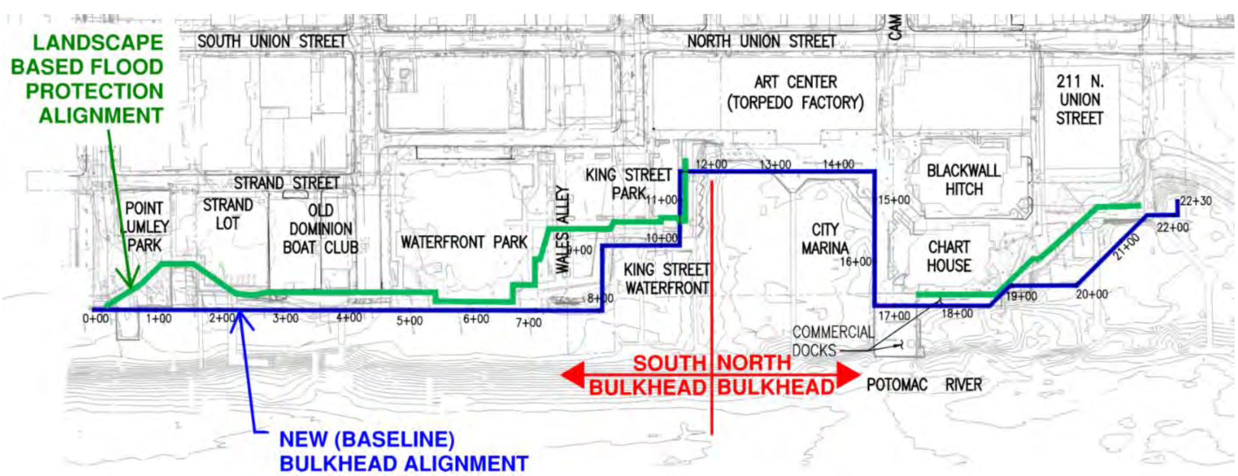


Figure 10. New Bulkhead Alignment (blue) and Landscape-Based Flood Protection Alignment (green)

For reference in this report, station numbering is assigned to the Baseline proposed bulkhead alignment from Sta. 0+00 at the foot of Duke Street to Sta. 22+30 at the foot of Queen Street. Station numbering is shown on Drawing BLP-1.

14.4. Bulkhead Design Criteria

In evaluating feasibility of new bulkhead options, similar design criteria as used by Moffatt & Nichol (Ref. 5) were considered. These include:

- 50-year design life
- Top of bulkhead grade El. +6.0 feet
- 300 psf live load surcharge allowance to accommodate construction and emergency vehicles. This is approximately 20% higher than AASHTO HS-20 loading.
- 2-foot differential water level between groundwater inboard of the bulkhead and water level in the River
- Dredge El. –6.0 on the south bulkhead segment and -10.0 on the north segment, with a 1.5-foot over-dredge allowance at each segment
- Maximum allowable bulkhead lateral deflection of 6 inches[1] under full live load surcharge and assuming no contribution from the existing bulkhead (further discussed below)
- Boating usage / accessibility: Mix of commercial and recreational boating similar to the current usage, to be maintained as a minimum*

*Mooring and berthing loads were not considered for conceptual bulkhead evaluation but must be considered for final design.

14.5. Representative Bulkhead Design Sections

Numerous factors govern the geotechnical design and constructability of each improvement option along the shoreline. Key factors include:

- Geotechnical subsurface conditions
- Condition of the existing bulkhead / shoreline
- Available space inland of the shoreline
- Conditions outboard of the shoreline (e.g. water depth, presence of existing piers or pile fields, debris at mudline, etc.)

To capture and "bracket" the range of these factors, we selected three cross-sections for geotechnical analysis and evaluation of shoreline improvement options:

1. **South Bulkhead (Station 0+00 to Station 12+00)** - Section B at approx. Sta. 5+50 adjacent to Waterfront Park is representative of conditions in the southern portion of bulkhead alignment. Key factors captured at Section B include:
    a. Design dredge El. -6.0
    b. "Typical" bottom of soft Alluvial Clay stratum (A1) at approx. El. -58
    c. New bulkhead alignment is just outboard of existing bulkhead

[1] Actual tolerable deflections must be determined by the Design-Builder based on the proposed construction sequencing and proximity of existing structures and their tolerance to movement.

   d. Existing (steel sheet pile) bulkhead appears to be in fair condition[2]
   e. Inland space exists for landscape-based improvements

2. **North Bulkhead (Station 12+00 to Station 22+30)** - Section C at approx. Sta. 19+50, near the foot of Thompsons Alley is representative of conditions in the northern portion of the bulkhead alignment. Key factors captured at Section C include:
   a. Design dredge El. -10.0
   b. “Deep” Bottom of soft Alluvial Clay stratum (A1) at approx. El. -70
   c. Existing (timber) bulkhead appears to be in deteriorated condition

3. **North Bulkhead Offset Alignment** – As a variation to Section B and C (where the new bulkhead is constructed just outboard of the existing bulkhead), the Offset alignment evaluates design requirements where the bulkhead alignment is shifted to the east and significant fill placement is necessary to raise grade and create land behind the new bulkhead. Such offset alignments are proposed to build Pump Station No. 2 near Thompsons Alley and straighten the bulkhead alignment by infilling the existing sloped revetment near the Strand Lot.

The South and North Bulkhead Design Sections (Sections B and C) are illustrated on Drawings GS-5 and GS-6, respectively.

14.6. Conceptual Bulkhead Design Analyses

Conceptual design analyses were performed at each design section to evaluate the feasibility of the alternative bulkhead improvement options and provide a basis for more detailed evaluation and design and updated cost estimates for bulkhead improvements. Analyses included:

1. Structural analysis of typical bulkhead design section to evaluate minimum dimensions and depths (i.e. pile tip elevations) of bulkhead elements to support applied soil, water and surcharge loads under normal and flood conditions. The analysis also included an evaluation of anticipated bulkhead deflections under design loading.

2. Global stability analysis using limit equilibrium methods to evaluate the minimum bulkhead embedment depth to prevent a potential deep seated failure beneath the bulkhead tip. Analyses were performed using the Spencer method for a circular failure surface and targeted a minimum factor of safety of 1.5 in bulkhead design.

3. Settlement analyses to estimate potential ground surface settlement caused by consolidation of the underlying compressible Stratum A1 clay from the weight of backfill placed behind bulkhead to raise grades to the design flood El. +6.

Soil parameters used in the conceptual design analyses were developed from the Phase II geotechnical exploration and are presented in Table 1.

15.0 BULKHEAD REPLACEMENT OPTIONS

---

[2] The Section B evaluation and conceptual bulkhead design is also applicable to sections of the South Bulkhead where the existing bulkhead is in visibly poor condition, although initial deflection of the new bulkhead may be higher. See discussion of bulkhead deflection in Section 15.2. Where no bulkhead exists and the baseline bulkhead alignment is shifted outboard of the existing shoreline, the North Bulkhead Offset Alignment is applicable.

The current concept for bulkhead replacement (Baseline Bulkhead Design) consists of a batter pile bulkhead with seaward battered piles bracing new sheet piling installed waterside of the existing bulkhead alignment as shown in Figure 11.

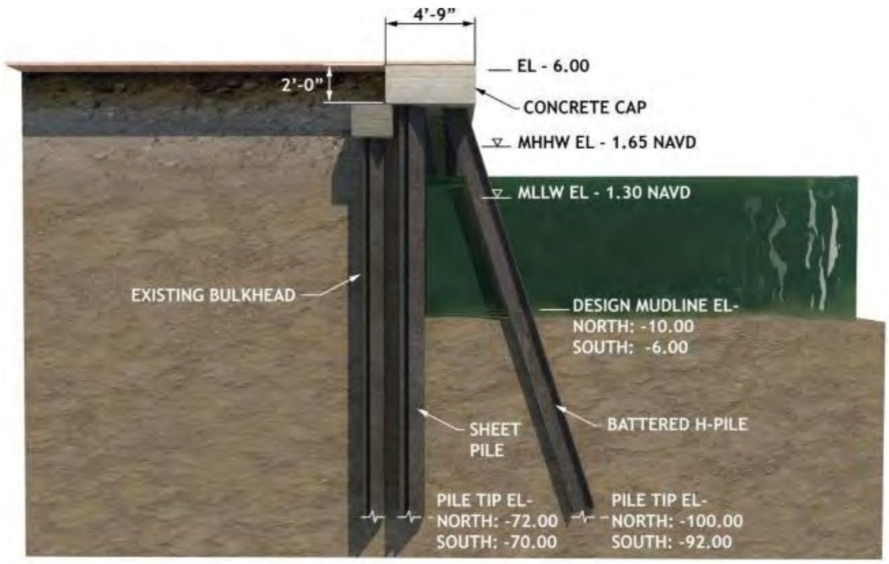


Figure 11 – Baseline Bulkhead Design Concept (reproduced from Ref. 5)

The primary advantage of the baseline concept is the all-waterside construction and consequently, reduced impact to adjacent structures and business/recreational activities landside of the wall. The waterside construction also mitigates the risks of encountering unforeseen buried structures and contaminated ground associated with landside construction. However, the baseline concept is relatively expensive and has certain drawbacks related to the batter pile design, namely that the outboard batter piles obstruct boat mooring directly adjacent to the bulkhead and introduce maintenance concerns arising from the potential for river borne debris to become trapped between the batter piles and accumulate along the waterfront. Both of these drawbacks could be addressed by the addition of a pile-supported timber boardwalk with integrated debris screen outboard of the bulkhead as presented in Ref. 5, although at increased cost and potentially increased regulatory scrutiny.

Alternative bulkhead options were therefore developed with anticipated potential to improve constructability, permitting demand, and/or cost. Conceptual designs of the bulkhead options are presented in this section and compared to the Baseline design.

15.1. Baseline Design Concept - Batter Pile Bulkhead

The batter pile bulkhead alternative recommended by Moffatt & Nichol (Ref. 5) consists of a steel sheet pile bulkhead buttressed by battered H-piles driven outboard of the bulkhead, as summarized in Table 3a.

Table 3a – Moffatt & Nichol Baseline Bulkhead Design (Ref. 5)

Design Section	Batter Pile Size*			Sheet Pile Size	
	Section	Tip Elev.	Spacing (ft)	Section	Tip Elev.
South	not provided	-92	9	AZ14	-70
North	not provided	-100	9	AZ24	-72

* batter angle 1H:3V scaled from graphic in Figure 11.

MRCE re-visited the Baseline Bulkhead Design using soil parameters developed from the Phase II geotechnical exploration at Design Sections B and C. The Baseline Bulkhead Design as evaluated by MRCE is summarized in Table 3b. As shown in Table 3b, the improved soil parameters developed from the Phase II exploration should result in some reduced element sizes and increased batter pile spacing, reducing the total number of batter piles.

Table 3b – Baseline Bulkhead Design (MRCE Evaluation)

<table>
<tr><th colspan="2" rowspan="2">Design Section</th><th colspan="3">Estimated Batter Pile Size[1]</th><th colspan="2">Sheet Pile Size</th></tr>
<tr><th>Section</th><th>Tip Elev.</th><th>Spacing (ft)</th><th>Section</th><th>Tip Elev.</th></tr>
<tr><td colspan="2">South (B)</td><td>HP14x89</td><td>-90</td><td>12</td><td>AZ12-770</td><td>-60</td></tr>
<tr><td colspan="2">North (C)</td><td>HP14x89</td><td>-100</td><td>9</td><td>AZ18-700</td><td>-72</td></tr>
<tr><td rowspan="2">North C Offset Alignment</td><td>LWF[2]</td><td>HP14x89</td><td>-100</td><td>7</td><td>AZ18-700</td><td>-83</td></tr>
<tr><td>NWF[2]</td><td colspan="5">Not recommended</td></tr>
</table>

Notes:
1. Based on 100 ton allowable axial load per pile at 1H:3V batter (requires confirmation by load testing)
2. NWF = Normal Weight Fill, LWF = Lightweight Fill (applies to fill placed to Elev. +6).
3. New bulkhead assumed to be immediately outboard of existing bulkhead, except at Offset Alignment.

For general construction considerations, see Section 15.6. Construction considerations specific to the Baseline Bulkhead Design include:

- A pile driving template will be required to install batter piles.
- Construction of batter pile installation from waterside with the batter direction toward the barge may be slow and pose issues in maintaining pile alignment. Pile driving equipment will need to be positioned at the edge of the barge with the top of the pile-driving lead angled outward.
- At Segments 1, 2, and 3 (see Figure 9), installing the seaward batter piles from the west (landside) of the new bulkhead alignment appears to be an option.

15.2. Option 1 - Cantilever Bulkhead (Combined Wall System)

The option of a cantilever bulkhead was explored since a cantilever system will provide the significant benefit of eliminating the need for either landside anchorage or waterside bracing. Figure 12 shows a possible cantilever bulkhead consisting of a combined (combi-) wall system comprised of interlocking large diameter steel pipe piles connected with intermediate lighter weight sheet piles. The large diameter pipe piles provide the system with high bending strength and stiffness and allow the combi-wall to perform in cantilever without the need for landside anchorage or waterside bracing. At the north bulkhead alignment, the cantilever wall consists of continuous interlocking steel pipe piles (without intermediate sheet piles) to provide additional strength and stiffness to accommodate the deeper dredge depth and depth to more competent soil strata. Similar to the baseline design concept, the cantilever bulkhead provides the advantage of all waterside construction but alleviates the issues with boating access and debris accumulation in finished construction. The simplified cantilever bulkhead profile with all vertical elements avoids the need for installation of more complicated batter pile bracing and makes bulkhead construction more efficient.

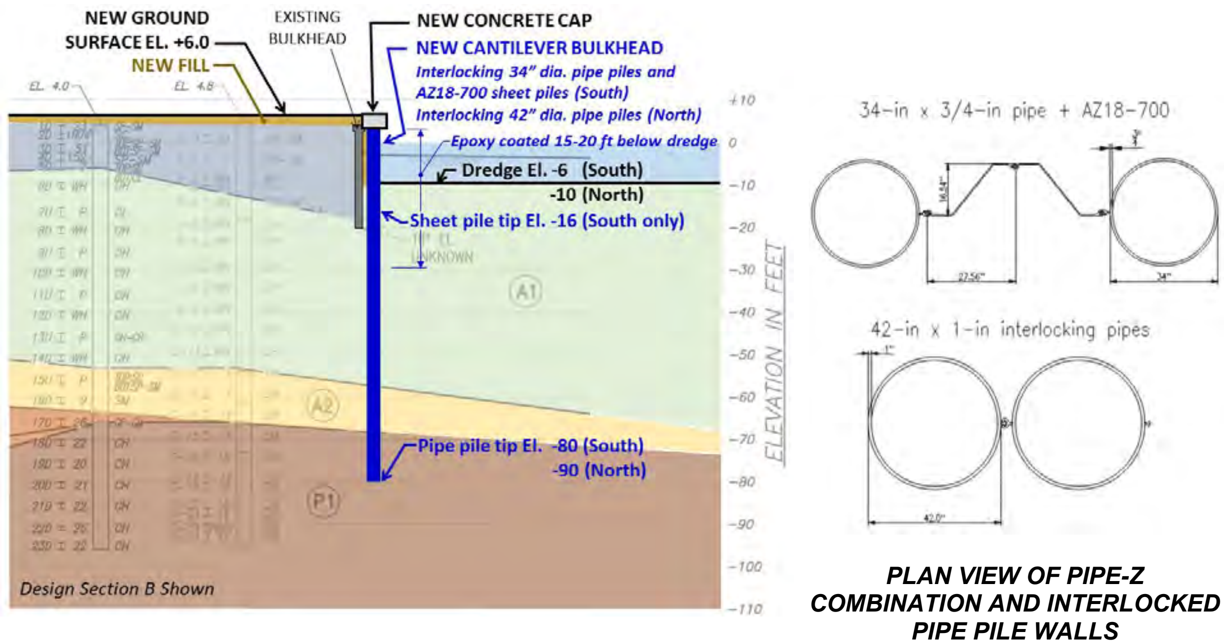


Figure 12. New Cantilever Bulkhead Concept

The cantilever bulkhead concept includes a concrete cap to connect the tops of the interlocking pipe and sheet piles and provide a continuous top surface and outboard edge. The concrete cap must be designed to accommodate attachments for mooring points, gangways and walkways, and/or attachments for deployable flood walls as applicable to the planned usage (refer to Section 15.8).

Table 4 provides a comparative summary of the conceptual design requirements developed for a cantilever bulkhead at the North and South Bulkhead Design Sections (B and C). Table 4 also summarizes the estimated wall performance including lateral deflection at the top of the wall, factor of safety (FS) against a global stability failure beneath the bulkhead tip, and ground surface settlement behind the bulkhead due to backfill placement to raise grade to the design flood El. +6.

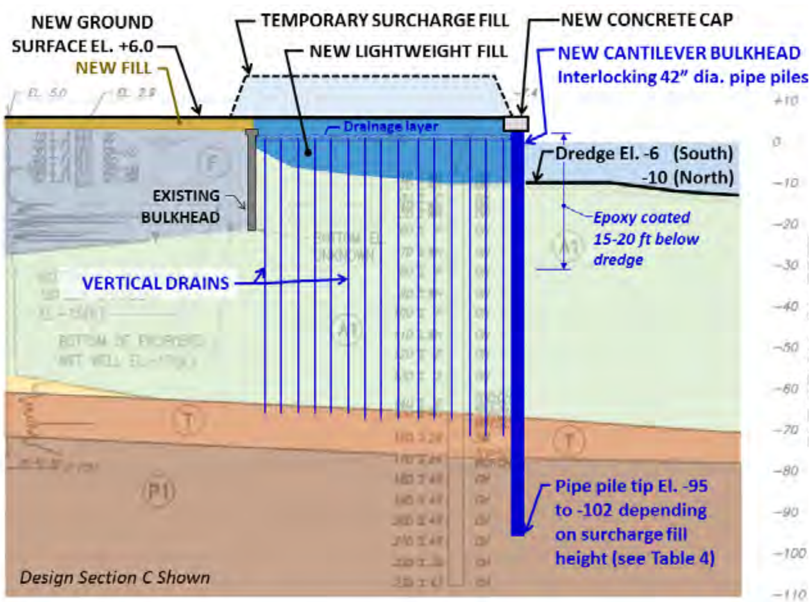
Table 4 - Conceptual Cantilever Bulkhead Design

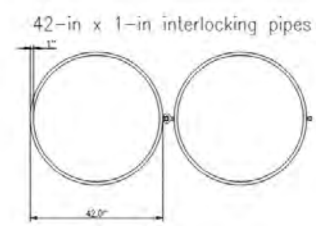
Design Section		Surch. Fill Ht. (ft)	Pipe Pile Size		Sheet Pile Size		Estimated Deflection (in.)	Global FS	Estimated Long-Term Settlement (in.)
			Dia. x Wall (in.)	Tip Elev.	Section	Tip Elev.			
South B		n/a	34x3/4	-80	AZ18-700	-16	5	>1.5	3-4 (NWF[1]) 1½ -3 (LWF[1])
North C		n/a	42x1	-90	n/a (interlocking pipes)		5	>1.5	2-4 (NWF[1]) 1-2 (LWF[1])
North C Offset Alignment	LWF[1]	5	42x1	-95			7	>1.5	4-6[2]
		10	42x1	-102			11	>1.5	2-3[2]
	NWF[1]	Not recommended							

Notes:
1. NWF = Normal Weight Fill, LWF = Lightweight Fill (applies to fill placed to Elev. +6).
2. Long-term settlement after surcharge fill removal. Does not include primary consolidation during surcharge preloading, which will be on the order of several feet. See Section 15.5.
3. New bulkhead assumed to be immediately outboard of existing bulkhead, except at Offset Alignment.

Estimated lateral (outward) deflection at the top of the cantilever bulkhead shown in Table 4 assumes backfill placement to El. +6 landside of the bulkhead and dredging to El. -6 (south) or El. -10 (north) waterside. Most of this deflection will occur in conjunction with fill placement inboard and dredging outboard of the bulkhead. These deflection estimates assume no contribution from the existing bulkhead inboard of the new bulkhead. As such, initial deflection will be less than estimated where the existing bulkhead is in fair condition and supports some of the load of the new fill. Where possible, dredging should occur prior to restoration of landside grades and construction of the cap, to minimize impacts to finishes. Over time, as the existing bulkhead deteriorates, an additional portion of the estimated deflection may occur.

The proposed reconfiguration of the bulkhead to a straighter alignment as shown on Figure 10 requires offset of the bulkhead to the east and significant fill placement waterside of the existing bulkhead in some locations, namely at the Strand Street Lot (Sta. 1+25 to 3+00 [Sections 9 and 10]) and Pump Station No. 2 (Sta. 19+00 to 21+50 [Sections 1 and 2]). At these locations, the new bulkhead must also support a temporary surcharge fill above El. +6 to pre-load the alluvial clay (see Section 15.5 for further discussion) and limit post-construction ground settlement to tolerable values. This temporary surcharge results in additional bulkhead deflection as summarized in Table 4. Lightweight fill material such as expanded shale aggregate must be used for the fill to El. +6 to reduce demand on the bulkhead, also shown in Table 4. The offset cantilever bulkhead concept is illustrated in Figure 13.





PLAN VIEW OF INTERLOCKING PIPE PILE WALL

Figure 13. New Cantilever Bulkhead Concept (Offset Alignment)

15.3. Option 2 - Anchored Sheet Pile Bulkhead

Where inland space is sufficient, installation of an anchored bulkhead is possible. This system would consist of a continuous wall of interlocking steel sheet piles anchored to a continuous sheet pile deadman set back 35 to 40 feet from the bulkhead (the exact setback distance depends on the final bulkhead design, selected so that the passive earth pressure zone of the deadman does not intersect the active earth pressure zone of the bulkhead). The bulkhead is anchored to the deadman by a series of steel tie rods, typically at 10 to 15 feet spacing. The anchored bulkhead design includes a concrete cap. As for the cantilever bulkhead concept, the concrete cap is designed to provide a uniform horizontal surface and vertical face for the bulkhead and accommodate mooring points, gangway / walkways and/or deployable flood wall attachments as applicable. It may also be designed to include connections to the tie rod anchors, eliminating the need for a separate wale. The anchored bulkhead concept is illustrated in Figure 14.

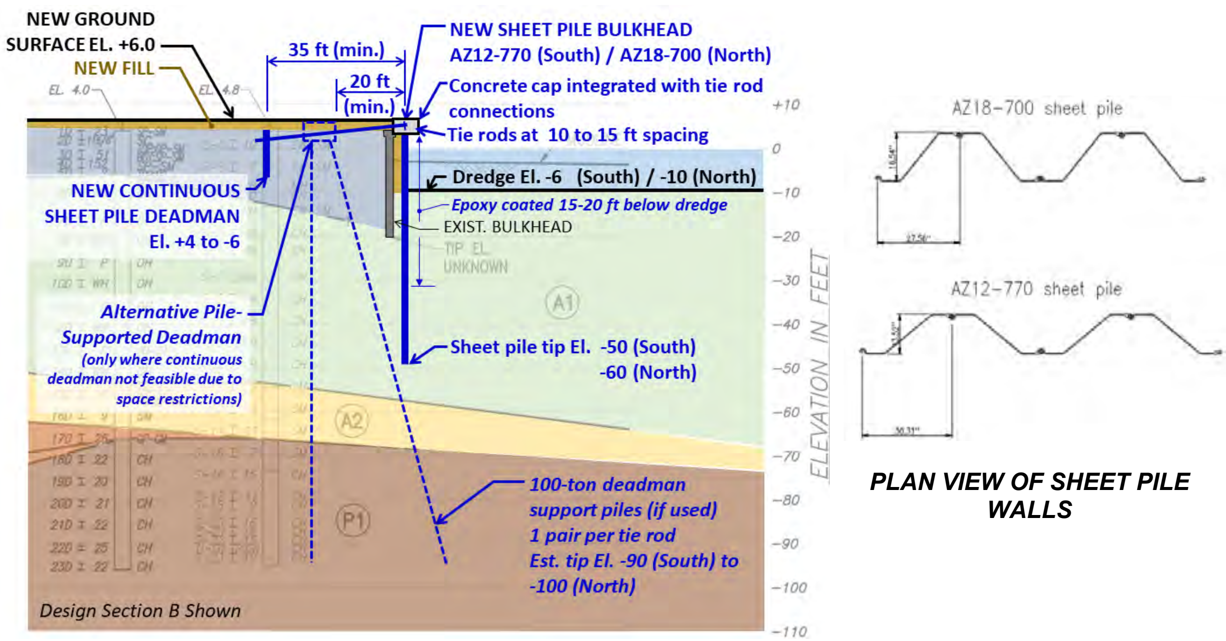


Figure 14. New Anchored Bulkhead Concept

Table 5 provides a comparative summary of the conceptual design requirements developed for the anchored bulkhead at the North and South Bulkhead Design Sections (B and C). Table 5 also summarizes the estimated wall performance including lateral deflection, factor of safety (FS) against a global stability failure beneath the bulkhead tip, and ground surface settlement behind the bulkhead due to backfill placement to raise grade to the design flood El. +6. The use of an anchored bulkhead at an offset alignment where fill placement waterside of the existing bulkhead is required was considered but eliminated as a viable option due to the potential impacts to anchorage performance from the large ground settlement caused by backfill placement and surcharge.

Table 5 - Conceptual Anchored Bulkhead Design

Design Section	Anchorage Type	Sheet Pile Size		Estimated Deflection (in.)	Global FS	Estimated Long-Term Settlement
		Section	Tip Elev.			
South B	10-ft. (min.) deep[2] continuous sheet pile deadman	AZ12-770	-50	2	>1.5	3-4 (NWF[1]) 1½ -3 (LWF[1])
North C		AZ18-700	-60[3]	3	1.5	2-4 (NWF[1]) 1-2 (LWF[1])
North C Offset Alignment	Not recommended					

Notes:
1. NWF = Normal Weight Fill, LWF = Lightweight Fill (applies to fill placed to Elev. +6)
2. Top El. +4 assumed
3. Governed by global stability

Where less than 40 feet of inland space is available, the deadman could be situated approximately 20 feet inland of the bulkhead but would require pile support. Deadman piles would be installed in an “A-Frame” configuration as illustrated in Figure 14 and consist of driven steel H-piles or closed-end steel pipe piles at approximately 100 tons allowable load each, installed in pairs at each tie rod. Due to the relatively steep batter angle, deadman support piles must have high axial capacity to resolve the horizontal anchor loads and be designed to resist drag due to settlement that will occur when fill is placed to raise grade. These demands result in a deep estimated pile tip elevation as shown on Figure 14.

Compared with the cantilevered bulkhead, benefits of the anchored bulkhead option include:

- Lighter weight (less costly) and narrower profile wall
- Shallower required tip elevation
- Lower estimated deflection

These benefits must be weighed against the following design and construction constraints:

- Viable only where at least 25 feet (and preferably 40 or more feet) of inland area is available. This precludes the length of bulkhead along the Torpedo Factory, City Marina, and Commercial Pier (Sta. 11+50 to 18+60 [Sections 4 and 5]) and in front of Old Dominion Boat Club (Sta. 3+50 to 4+25 [Section 9]).
- Land-side construction access is needed to install the bulkhead anchorage.
- Where a pile-supported deadman is used, pile installation will require significant inland space and introduce noise and vibration.
- Installation of the deadman and tie-rod system requires removal of trees and landscaping between the bulkhead and deadman, unless the tie rods and deadman can be designed to avoid removal or damage of the trees. Buried utilities in this zone will need to be worked around or relocated.
- Tie rods must be designed with oversized sleeves to accommodate settlement of the ground inboard of the bulkhead (see Table 5) and not exert sag load on the tie rods.
- The sheet pile deadman alignment crosses the footprint of old piers and will likely encounter obstructions that will require pre-drilling to clear.
- Due to their shallower toe embedment, sheet piles will undergo settlement when new is placed to raise grade, as indicated in Table 5. This should be taken into account when determining pile procurement length and cutoff elevation, and in the design of outfalls that penetrate the bulkhead.

Use of tiebacks to provide bulkhead anchorage is an option and would mitigate the disturbance and risks associated with landside installation of deadman anchorage. However, the tiebacks would have to achieve capacity largely in the underlying Stratum P1 high plasticity clay and thus require very long anchor lengths with potential for creep and relaxation in tieback load with time. The tieback option is expected to have higher cost than the deadman option and therefore not considered further in conceptual design.

15.4. Design Life and Maintenance Considerations

To provide the intended (50-year) design life, steel pipe piles and sheet piles must be epoxy coated from the top to a depth of 15 to 20 feet below the dredge line and inspected and maintained periodically.

Typical epoxy coatings include high build coal tar epoxy or glass flake epoxy built up to a minimum total of 16 mil dry film thickness (DFT) on the steel surface prepared to SSPC SP 10 standard. While the concept designs presented herein include 1/16-inch sacrificial steel thickness, the design-builder may consider adding additional sacrificial steel to provide increased durability and extend the design life, considering potential future reconfiguration of the bulkhead with deployable barriers to extend effectiveness against raised water levels.

Using weathering steel grades ASTM A 588 and A 690 for rolled steel shapes, pipe piles and sheet piles in addition to epoxy coating is an additional potential design measure to help prolong the service life of the bulkhead elements.

A regular inspection and maintenance program is required to identify and repair any conditions of distress in a timely manner and achieve the intended 50 year bulkhead design life. A 5-year inspection frequency is recommended to assess conditions and develop repair plans as required. For planning purposes, a reasonable maintenance and repair schedule is provided in Ref. 5. The final inspection and maintenance program should be developed by the Design-Builder and tailored to the final bulkhead design.

Any post-installed connections and attachments to the steel bulkhead elements in the tidal zone will tend to compromise the coatings, and require prompt patching of the coatings. Since field applied patches are not as durable as the shop installed coating, post installed connections should be avoided or kept to a minimum.

15.5. Offset Bulkhead Alignment and Waterside Fill Placement

Where the new bulkhead line is outboard of the existing shoreline, it is necessary to place fill into the Potomac River to make land in the intervening area. The area of greatest proposed landfill is the area at the foot of Thompsons Alley surrounding proposed Pump Station No. 2 and foot of Queen Street (Bulkhead Sections 1 and 2). Significant landfill is also proposed at the old boat ramp and shoreline at the Strand Street Lot (Bulkhead Sections 9 and 10). The proposed landfill has significant design and construction implications due to the presence of the underlying highly compressible and low-strength organic silt and clay (Stratum A1) and consequent ground settlement caused by consolidation (compression) of these soils under the weight of the new backfill.

Primary consolidation settlement under the weight of normal-weight fill to El. +6 is expected to be on the order of 3 to 4 feet, with additional secondary (creep) settlement occurring over long periods. The magnitude of this settlement can be reduced by the use of lightweight fill material such as expanded shale aggregate or cellular concrete, but will still be significant, on the order of feet. A program of surcharge preloading with vertical drains is necessary to accelerate settlement within a manageable construction timeframe and keep the magnitude of future settlement within an acceptable range.

In general, the required construction sequence for landfill placement and surcharge preloading includes, as a minimum, the following steps:

1. Install new bulkhead (the bulkhead must be designed to handle the temporary surcharge fill load, discussed in the preceding sections).
2. Place fill into the water between the existing and new bulkheads to achieve a working platform above the mean high water level. The elevation of the initial fill placement must consider the anticipated settlement. Alternatively, the area enclosed by the new bulkhead could be dewatered and new fill placed in the dry.
3. Install prefabricated vertical drains on a pattern and spacing designed to achieve a sufficient amount of consolidation within a manageable time period. Preliminary calculations indicate

that drains placed in a 4 foot center-to-center square pattern will enable the majority of primary consolidation to occur in an approximately 6-month time period.

4. Install a network of drainage pipes and/or a drainage layer to collect effluent from the vertical drains and convey it to a permitted discharge or detention location. Typically, the drainage layer consists of a 6- to 12-inch thick layer of crushed stone sandwiched between top and bottom geotextile layers to prevent migration of fine-grained soil into the stone.
5. Place fill to El. +6 while monitoring settlement and pore water pressure in Stratum A1.
6. Place additional (surcharge) fill to an elevation higher than finish grade. The additional (surcharge) fill height must be designed to achieve, within a reasonable time period, e.g. 6 to 12 months, a sufficient degree of consolidation to maintain post-surcharge-removal settlement to acceptable ranges. Preliminary calculations indicate that an approximate 5-foot surcharge fill kept in place for 6 months should limit post-surcharge-removal settlement to on the order of 5 inches over 100 years. Similarly, a 10-foot surcharge fill kept in place for 6 months would limit post-surcharge-removal settlement on the order of 2 inches in 100 years. Note that the new bulkhead must be designed to handle the load of the temporary surcharge fill above final grade.

It is important to note that the settlement under the new landfill, which could be on the order of several feet, will extend some distance inland, dragging down above- or below-grade structures in the zone of influence. A two- or three-dimensional settlement analysis would be needed to estimate the extent of the settlement influence zone once landfill areas are finalized. Any pipes or outfalls within the zone of influence will require temporary re-routing and reconstruction after settlement is substantially complete. This consideration likely prohibits performing landfill outboard of existing sensitive structures near the shoreline such as the Chart House and Torpedo Factory.

15.6. General Bulkhead Replacement Considerations

The following design and construction considerations are common to all replacement bulkhead types considered:

- All bulkhead options include long steel sections, which makes waterside work advantageous. Long sections can be delivered by barge, avoiding the need to splice shorter steel segments delivered by land.
- Where existing mudline is shallow and the water depths do not provide sufficient draft for construction barges, dredging in advance of bulkhead construction to allow barge access will be required. The required draft is a function of the builder's selected equipment. Stability of the existing bulkhead to support this pre-construction dredging needs to be evaluated. Where the existing bulkhead is not capable to support additional dredge depth, the pre-construction dredging must be set back from the existing bulkhead by a sufficient distance to maintain the existing bulkhead stability and compatible with the reach of the construction equipment, as determined by the design-builder.
- The existing piers will need to be partially or fully demolished to make way for construction of the new bulkhead.
- Potential obstructions in the path of the new bulkhead such as concrete fill, rip-rap, or other debris at mudline will require removal by excavation (spudding) or pre-drilling, a significant effort.
- All of the proposed bulkhead types are installed by impact driving, which produce noise and vibration. Recommended monitoring of nearby structures for movement and vibration is summarized in Section 22.0. Use of alternative low-impact (e.g. "press-in") methods to install the bulkhead can be considered where the new bulkhead alignment is close to existing structures sensitive to vibration and settlement.

- Some inland area along bulkhead will be required during construction. This will require sequencing the work to minimize disruption to waterfront property owners, City Marina / Water Taxi, and the public.
- A number of outfalls exist along the current shoreline including two in Point Lumley Park and several below the City Marina wharf (Ref. 6). In addition, new outfalls are proposed at each pump station. The bulkhead must be designed with suitable openings to accommodate any existing outfalls to be preserved and new outfalls, including watertight penetrations and backflow preventers as applicable. Outfall design must consider the potential for differential movement at the bulkhead penetration due to settlement caused by raising inland grades.

15.7. Bulkhead Replacement Constraints Specific to Individual Sections

In front of the Old Dominion Boat Club between Sta. 3+00 and 4+00 [Section 9], significant rip-rap and uncontrolled mass concrete fill are present in the path of the proposed bulkhead line. This material will require pre-excavation or pre-drilling in advance of new bulkhead construction, a significant effort. As an alternative to a new bulkhead, a built-up rip-rap revetment with its toe at the bulkhead line could be an option in this area. However, a rip-rap revetment would not be readily compatible with construction of a pile-supported platform to provide a continuous 20-foot wide promenade along the waterfront as envisioned in the WSAP, and would not accommodate future dredging.

In front of the Chart House Restaurant between Sta. 17+35 and 18+60 [Section 4], it is not possible to raise grade at the new bulkhead due to the presence of the existing building close to the bulkhead with entrances below El. +6.0. Here, it may be necessary to terminate the new bulkhead at the existing wharf elevation and extend flood protection to +6.0 by means of flood barriers installed on top of the bulkhead.

Additional design and construction constraints and considerations specific to each bulkhead section are listed in the MRCE January 10, 2022 Bulkhead Site Walk Findings report in Appendix A.

15.8. Future-Proofing and Outboard Promenade

Addition of fixed or deployable flood barriers on top of the new bulkhead to extend flood protection to a higher elevation in the future is an option for all replacement bulkhead types. In general, elevated flood loads act opposite to the normal service loads and therefore do not increase structural demand on the bulkhead system. Suitable structural connections must however be designed to transfer flood loads to the bulkhead. Concrete caps atop the new bulkhead sections can be designed to accommodate such structural connections.

The cantilever bulkhead (Replacement Bulkhead Option 1) is better suited for future raising of the inland ground surface (above El. +6) to accommodate higher water levels, which could be achieved by adding an anchorage and extended cap. The baseline (sheet pile with outboard batter piles) and anchored bulkhead (Replacement Bulkhead Option 2) would be more difficult to retrofit for increased earth loads.

Future construction of a pile-supported outboard promenade is also an option for all replacement bulkhead types described above.

16.0 REUSE OF EXISTING BULKHEAD

Where the existing bulkhead has significant remaining service life or its life can be prolonged by localized repairs, continued use of the bulkhead with the addition of inland or shoreline features to raise flood protection to El. +6 is possible.

16.1. Localized Bulkhead Repairs

Localized repairs to extend the life of the existing bulkhead may consist of adding welded steel plates in areas of localized corrosion or holes, application of protective coatings in the tidal/splash zone at steel sheet pile segments, and concrete repairs or shotcreting at concrete bulkhead segments. The scope and design of the repairs must be determined by the design-builder based on a detailed bulkhead condition assessment.

Since the top of the bulkhead at the Torpedo Factory Wharf and City Marina Wharf (Sta. 12+00 to 17+35, Sections 4 and 5) is already at El. +7, above the target flood protection elevation, localized repairs in these sections may be sufficient to extend the service life and postpone bulkhead replacement. In 2013, Michael Baker Jr., Inc. identified the concrete and steel bulkheads in this area to be in satisfactory and fair condition, respectively (Ref. 6).

Level of disturbance to the public associated with localized bulkhead repairs will depend on their scope and nature as determined by the design-builder based on condition assessment. Where the bulkhead is not fronted by a pile-supported platform, it may be possible to perform most of the repair work from the water side with minimal pedestrian disruption. At the King Street Park, Torpedo Factory, City Marina, and Chart House wharves (Segments 4, 5, and 6), some of the repair work may be possible to perform from below the existing wharves by divers. However, the repairs may require sequential removal and reconstruction of portions of the wharf boardwalks with associated need for temporary closure of portions of the boardwalk and pedestrian detours.

16.2. Inland Landscape-Based Options

Where sufficient inland space exists, landscape-based solutions are possible. Landscaping options improve flood protection by raising inland grades to EL. +6 using sloping terrain, low height retaining walls (‘Ha-Ha’ walls), planter boxes, stairs, or other architectural features as described in the January 2022 Carollo Assessment Report (Ref. 11).

Based on the condition of the existing bulkhead and available inland space, landscape-based options appear most viable in the southern half of the alignment at the following bulkhead sections:

- King Street Waterfront, Sta. 9+20 to 11+50 [Section 6]
- Wales Alley Waterfront, Sta. 7+20 to 9+20 [Section 7]
- Waterfront Park, Sta. 4+70 to 7+20 [Section 8]
- Point Lumley, Sta. 0+00 to 1+25 [Section 11]

Geotechnical engineering considerations for the landscape-based solutions include: settlement under the weight of new fill placed to raise grade, global stability, and foundation design.

A simplified typical cross-section was analyzed as illustrated in Figure 15. This analysis considered a 2-foot high grade raise set back 20 feet from the shoreline, leaving a 20-foot wide promenade at the current shoreline elevation.

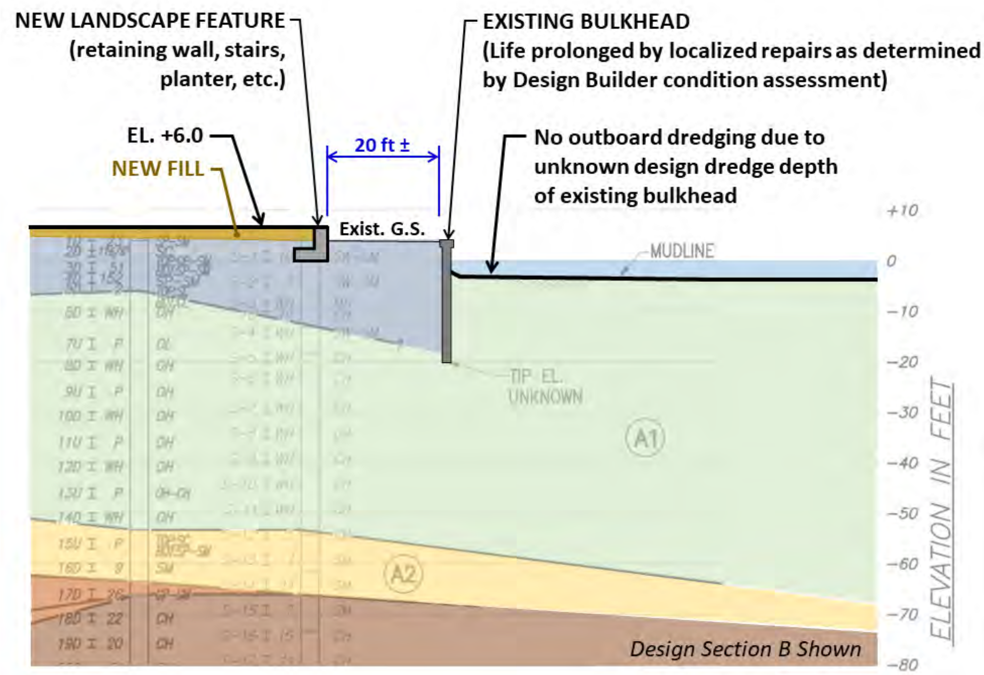


Figure 15. Landscape-Based Flood Protection Concept

Estimated consolidation settlement under the weight of the new fill ranges from 2 to 4 inches over an approximately 20-year period if normal-weight fill is used, or 1 to 3 inches if lightweight fill (e.g. expanded shale aggregate) is used. These estimates are based on the assumption that the fill is placed uniformly over a 50-foot wide strip parallel to the shoreline. By optimizing the design, the magnitude and effects of the estimated settlement can be reduced: for example, by narrowing the width (and hence weight) of the raised area, and/or by tapering the edges of the raised area so that settlement is distributed gradually over a wide area. These options should be evaluated in final design. Additional considerations related to settlement are discussed in Section 16.6 below.

Global stability analyses of the raised-grade cross section shown in Figure 14 using parameters from the Phase II Geotechnical Exploration indicate that the added weight of soil does not significantly impact the existing global stability safety factor.

Foundations for landscape-based flood protection features such as retaining walls, stairs, and planter boxes may be designed in general accordance with the recommendations for Lightly Loaded Foundations in Section 20.1. Retaining walls must be checked for both internal (structural) stability and external stability (sliding, bearing, and overturning) under normal (gravity), flood, and rapid drawdown cases. The "Active" lateral earth pressures given in Table 3 may be used for preliminary design of retaining walls, and surcharge loads from walkways, streets, and construction equipment (as applicable) added to the lateral earth pressures. For stand-alone structures designed to act as flood barriers, a seepage analysis should be performed to confirm significant seepage through or under the structure will not occur when the outboard water level rises to the design flood protection elevation. This analysis may result in the need to install a cutoff wall or trench filled with impervious fill below the structures.

16.3. Addition of Deployable Barriers to Top of Existing Bulkhead

Where the existing bulkhead is confirmed to have significant remaining service life and structural capacity, the addition of fixed or deployable flood barriers on top of the existing bulkhead is an option. Based on preliminary available information including the Michael Baker Jr., Inc (2013) structural condition assessment (Ref. 6) and the MRCE 1-day site walk (Ref. 12), this option appears to have potential for further consideration at the following sections:

- City Marina Wharf and Commercial Docks, Sta. 17+35 to 18+80 [Section 4]
- Waterfront Park, Sta. 4+70 to 7+20 [Section 8]
- Point Lumley, Sta. 0+00 to 1+25 [Section 11]

To confirm the feasibility of this option along any length of bulkhead, a detailed bulkhead condition assessment is required to evaluate the strength of the existing bulkhead system to support new loads imposed by the barrier. Suitable structural connections must be designed to transfer flood lateral and overturning loads from the new flood barrier to the existing bulkhead.

16.4. Raising Existing Bulkhead to El. +6

The option of raising the existing bulkhead to El. +6 has been considered by the Implementation Team (Ref. 11). This option is feasible only where (a) the existing bulkhead is confirmed to have significant structural capacity and remaining service life, and (b) sufficient inland space is available to supplement the existing bulkhead cap and anchorage. Based on preliminary available information, this option appears to have the potential for further consideration at the following sections:

- Waterfront Park, Sta. 4+70 to 7+20 [Section 8]
- Point Lumley, Sta. 0+00 to 1+25 [Section 11]

To confirm the feasibility of this option along any length of bulkhead, a detailed condition assessment is required to confirm the strength of the existing system to support increased earth, hydrostatic, and surcharge loading on the raised bulkhead. In particular, the upper portion of the bulkhead and existing anchorage (if present) would require investigation by test pitting. The test pits must extend deep enough to expose the existing anchorage (if any), potentially below the groundwater table.

Where the existing bulkhead is found to have significant capacity and remaining life, one or more of the following modifications would be necessary to support a raised elevation:

- Installation of a new steel and/or concrete cap with a raised top
- Addition of supplemental tie rods and deadman anchorage inland of the bulkhead or tiebacks from the waterside
- Excavation of existing fill and replacement with lightweight fill (i.e. expanded shale aggregate or cellular concrete) in the approximately 20-foot wide “active” earth pressure zone behind the bulkhead, to avoid increasing load on the bulkhead.

All the above concepts will require further evaluation based on the results of a detailed condition inspection to confirm feasibility along any given length of shoreline.

The existing bulkhead at Point Lumley Park, Sta. 0+00 to 1+25 [Section 11] appears to have an anchorage consisting of tie rods set just above high water. There may be a potential to re-use or supplement this anchorage to support a raised bulkhead, but this would require a detailed water-side inspection and land-side test pit investigation to confirm feasibility.

16.5. Limitations of Bulkhead Reuse Options

Since all continued use options rely on the capacity of the existing bulkhead for a significant future period, a more detailed bulkhead condition assessment is necessary to provide a quantitative estimate of remaining service life, define the need for local repairs, and confirm viability at each section considered. The more detailed assessment would include: detailed water-side bulkhead inspection, diver survey, ultra-sound / pulse echo / GPR testing to determine thickness and depth of steel elements, concrete strength / soundness testing, test pits to evaluate existing anchorages, and any additional investigation determined necessary by the design-builder.

It is important to note that flood protection will only be effective if the elevations of flood protection between various sections and along the entire waterfront seamlessly provide a continuous barrier up to the desired elevation (El. +6). Both the Michael Baker Jr., Inc. (2013) condition inspection (Ref. 5) and MRCE 1-day site walk (Ref. 12) documented numerous pipe penetrations and other holes/discontinuities in the existing bulkheads that would require repair and sealing under all continued use approaches. In addition, modifications to the storm drainage system to prevent backflow through existing outfalls is a required component of the flood protection strategy.

Outboard dredging is generally not possible for the continued use options since the design dredge depth of the original bulkhead is not known and the existing bulkhead tip elevations are difficult or impossible to determine.

17.0 GENERAL BULKHEAD CONSIDERATIONS

17.1. Settlement under Grade Raise Fill

When new fill is placed to raise grade inboard of the existing shoreline or new bulkhead, settlement will occur due to compression of the Stratum A1 organic clay under the weight of the fill. Preliminary land-side settlement estimates indicate on the order of 2 to 4 inches over an approximately 20-year period if normal-weight fill is used. If lightweight fill (e.g. expanded shale aggregate) is used to raise grade, the estimated primary consolidation settlement decreases to approximately 1 to 3 inches. These estimates are based on the assumption that fill to raise grade is placed over an approximately 50-foot wide zone parallel to the shoreline.

Settlement will affect existing buried utilities, surface structures, and new structures and finishes in the area of the grade raise. We therefore recommend a program of monitoring of these structures and utilities, including pre-construction and post-construction condition surveys and contingency plans to repair damage to structures or buried utilities in the area of grade raise be included in the contract documents. See Section 22 for additional discussion of monitoring recommendations.

These settlement estimates apply to locations where new fill is placed over existing land. Where new fill will be placed in the River, significant additional estimated settlement and additional design and construction considerations apply (see Section 15.5).

17.2. Boating Accessibility

All options that raise the bulkhead elevation must consider boating accessibility. To accommodate small boats and other vessels with low freeboard, the use of stepped wharves or floating piers accessed by articulating gangways will be required.

17.3. Permitting Considerations

This project involves shoreline stabilization with the construction of new bulkheads outboard of existing bulkheads in need of repairs and raising of grades for flood protection in tidal waters. The permitting process will require preparation of a Tidewater Joint Permit Application (JPA). The JPA will be reviewed by the US Army Corps of Engineers (USACE), Virginia Marine Resources Commission (VMRC) and the Virginia Department of Environmental Quality (VA DEQ) who may identify additional agencies that need to review the application and permit requirements.

The bulkhead portions of the project fall within the purview of Nationwide Permit NWP 3 – Maintenance. NWP 3 permits repair, rehabilitation or replacement of any previously authorized, currently serviceable structure or fill authorized by Code of Federal Regulations (CFR) 330.3. Provided the project conforms to all requirements of NWP 3 requirements under CFR 330 and FEMA approved state and local floodplain management requirements, the USACE should allow the project to proceed under NWP 3.

The Project application will need to be submitted to VMRC, who will then assign an application number and forward to the other agencies for separate but concurrent review. The local Wetlands board may also review the application if any part of the shoreline proposed for rehabilitation includes areas designated as wetlands. If the project falls under the purview of the most recent State Program General Permit and is below the impact thresholds and meets all the limitations and conditions, DEQ is authorized to issue the USACE general permit. Public notice, especially to adjacent and riparian property owners, comment period, and public hearing will be required.

There are other scoping elements such as new outfall structures that may trigger regulatory requirements outside of the GDM scope of work. Actual permitting requirements will depend on the final design developed by the Design-Builder.

17.3.1. Permitting Implications for New Bulkheads

In general, projects where deteriorated bulkheads are replaced with landward revetments are considered most favorably for permitting, followed by new bulkheads installed immediately outboard of the existing bulkhead alignment with minimal incursion into water. Since sections of the existing bulkhead are deteriorated and require rehabilitation to support flood protection measures, new bulkheads are required which may need to be bolstered in the future to adapt to rising water levels. Several alternative bulkhead replacement options were discussed in the preceding sections with each option imposing somewhat different permitting implications as described below. In general the options minimizing incursion into the waterway will be most favored by the regulatory agencies. The Design Builder will need to provide a strong technical basis for the recommended bulkhead type.

Sheet pile bulkhead with outboard batter piles (Baseline option)

In this system, the outboard batter piles will be an incursion into navigable waterway and as such may be objectionable to the permitting agencies. This system would also allow for debris to accumulate between the sheet piles and batter piles which may add to scrutiny and potential objection from the public.

Cantilevered (Pipe-Z combination or interlocked pipes wall)

A new cantilever bulkhead system installed immediately outboard of the existing deteriorated bulkhead with a minimal annulus for flowable fill should be more favorable for permitting given its reduced incursion into the water relative to the batter pile option.

In the northern portion of the shoreline, between Queen Street and Thompsons Alley, the baseline bulkhead alignment is approximately 30 ft outboard of the existing bulkhead. The design builder and the City will need to provide a strong technical basis for this required offset. For example, due to the presence of mass concrete pour and un-engineered riprap outboard of the existing line of bulkheads in some sections, installing new bulkhead structures immediately outboard of the existing bulkhead will be nearly impossible. The new bulkhead will need to be located outboard of the obstructions. This offset alignment will also provide a straight edge for any future resiliency measures as well as a continuous promenade for the public. If the project cannot be approved under NWP 3, it will have to be processed under the normal USACE / Virginia Tidewater Joint Permit application. Off-site wetland replacement may be a condition of permit approval with this option.

Anchored sheet pile bulkhead

An anchored sheet pile bulkhead installed immediately outboard of existing deteriorating bulkhead with minimal annulus for flowable should be more favorable than both the baseline and cantilever options given its relatively minimal incursion into the water. However, upland space required to install anchorages with sufficient offset will limit its application to a few sections of the shoreline.

17.3.2. Permitting Implications for Dredging

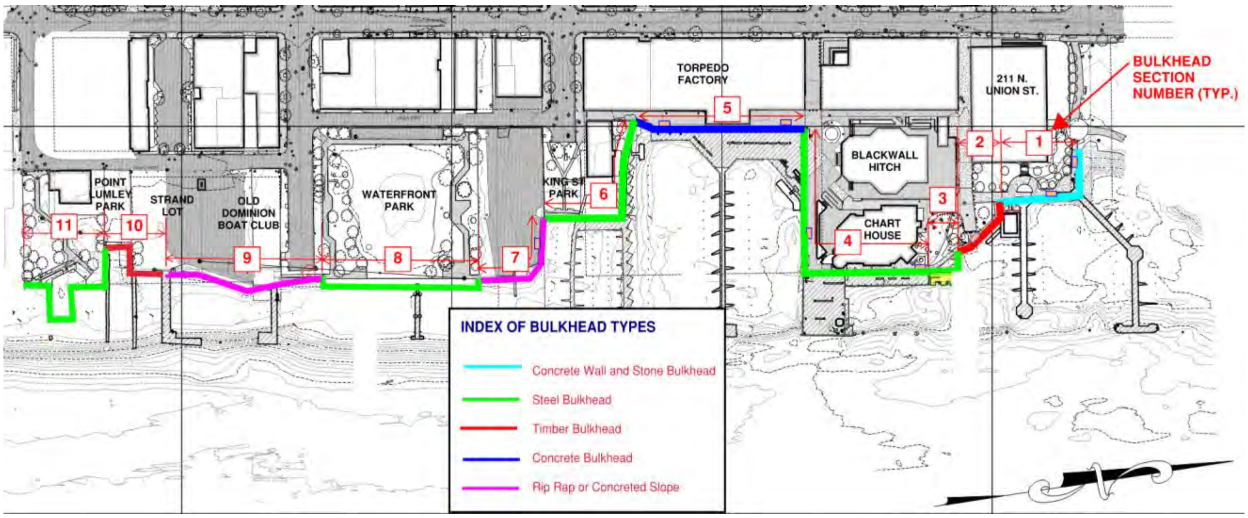
The dredging proposed for this project is required to meet the project design criteria and objectives of improving waterfront access, and also to provide barge access for bulkhead installation. While portions of the proposed dredging may be covered under NWP 3, the overall dredging scope requires coordination with the standard Tidewater JPA for the project. The permit application will need to include information on the extents of dredging, contamination (if any), and proposed options for treatment and disposal of dredged spoil.

We refer to the Phase II Environmental Site Assessment Report attached to the GDR (Ref. 1) for information on contaminants identified in the Potomac River sediment and recommendations regarding characterization and disposal requirements for dredged materials.

17.4. Summary of Conceptual Bulkhead Replacement / Reuse Options

Table 6 summarizes the conceptual bulkhead replacement or reuse options that appear geotechnically feasible by bulkhead section, based on currently known conditions and the conceptual design analyses described above. For ease of comparison, a figure illustrating bulkhead section locations is included with Table 6.

Table 6. Summary of Conceptual Bulkhead Replacement & Reuse Options



Section (Sta.)	Bulkhead Replacement Option	Bulkhead Reuse Option	Notes
1 (21+00 to 22+30)	Baseline or Cantilever (Offset Alignment)	-	Inland space is available but existing bulkhead is near end of service life.
2 (19+30 to 21+00)		-	
3 (18+80 to 19+30)		-	
4 (14+55 to 18+80)	Baseline or Cantilever	Repair with addition of deployable barriers (17+35 to 18+80 only)	Bulkhead currently at El. +7± (12+15 to 17+35). No inland space available.
5 (12+15 to 14+55)	Baseline or Cantilever	Repair existing bulkhead	
6 (9+20 to 12+15)	Baseline, Cantilever, or Anchored	Repair / supplement existing bulkhead and add inland landscape-based flood protection	
7 (7+20 to 9+20)			
8 (4+70 to 7+20)			
9 (2+25 to 4+70)	Baseline or Cantilever (Offset Alignment)	Build up existing rip-rap revetment and add inland landscape-based flood protection (integrate with existing flood wall at Old Dominion Boat Club)	Significant rip-rap and uncontrolled mass concrete present in path of replacement bulkhead.
10 (1+25 to 2+25)		-	Existing bulkhead is near end of service life.
11 (0+00 to 1+25)	Baseline, Cantilever, or Anchored	Repair existing bulkhead and add inland landscape-based flood protection	

PART 4 – PUMP STATIONS AND OTHER STRUCTURES

18.0 PUMP STATIONS

Project plans propose construction of two Pump Stations (Nos. 1 and 2) for stormwater management at the approximate locations shown on Drawing BLP-1:

- **Pump Station No. 1** is in Waterfront Park at the northeast corner of Prince and Strand Streets near the south end of the project area. Concept plans (Ref. 4) indicate that the pump station will consist of a reinforced concrete structure built mostly below grade. The below-grade structure includes a screen chamber, wetwell, and check valve chamber occupying a combined footprint of approximately 32 by 95 feet. The lowest point is at approximate El. -17, or about 22 feet below grade. The above-grade structure consists of a 30-foot by 30-foot by 20 feet high building housing a trash rack room, generator, and control room. Drawing GS-5 illustrates the approximate configuration of Pump Station No. 1 in plan and section relative to surrounding structures and subsurface conditions revealed in the Phase II Geotechnical Exploration.

- **Pump Station No. 2** is at the foot of Thompsons Alley near the north end of the project area. The new pump station will be built partially outboard of the current shoreline with a portion of the pump station constructed within new fill placed in the Potomac River inboard of a new bulkhead. Similar to Pump Station No. 1, Pump Station No. 2 will consist of a reinforced concrete structure built mostly below grade. The below-grade structure includes a screen chamber, wetwell, and check valve chamber occupying a combined footprint of approximately 20 by 95 feet. The lowest point will be at approximate El. -19. The above-grade structure consists of a 30-foot by 30-foot by 20 feet high building housing a trash rack room, generator, and control room. Drawing GS-6 illustrates the approximate configuration of Pump Station No. 1 in plan and section relative to surrounding structures and subsurface conditions revealed in the Phase II Geotechnical Exploration.

- **Pump Station No. 2 (Alternate Inland Location)** - An alternative more inland location for Pump Station No. 2 is possible without alteration to the shoreline. The alternate location is at the foot of Queen Street just north of the building at 211 North Union Street as shown on Drawing GS-6.

18.1. Foundation Type and Capacity

The proposed pump stations are underlain by relatively loose Stratum F fill and thick deposits of Stratum A1 silt/clay as shown on Drawings GS-5 and GS-6. These soils are relatively weak and compressible and not suitable to support the pump stations. Deep foundations are therefore required to carry structure loads to suitable bearing strata and resist uplift. Pump stations will extend 20 or more feet below the groundwater table.

Driven or drilled piles developing capacity in the underlying Strata A2, T and P1 soils are viable for support of the proposed structure. Piles must be designed to carry both compressive load and tension to resist hydrostatic uplift. For conceptual design purposes, we considered three types of driven piles and one drilled pile, each providing a 50-ton design capacity, as summarized in Table 7. More economical drilled piles such as continuous flight auger (CFA) piles or partial displacement piles may be viable if further investigation confirms that significant remnant structures or other obstructions are not present at pump station locations.

Table 7 – Summary of Conceptual Pump Station Pile Types (50-ton allowable load)[1]

Pump Station	Pile Type	Top of Bearing Stratum El.	Estimated Tip El.	Estimated Down-Drag Load (tons)	Estimated Allowable Tension (tons)[2]
Pump Station No. 1	Driven HP12x74	-45	-70	19	35
	Driven 12-in. dia. steel pipe (closed end)		-75	16	35
	Driven 18-in. dia. steel pipe (closed end)		-65	19	35
	Drilled, pressure grouted 12-in. dia. micropile		-70	19	50
Pump Station No. 2	Driven HP12x74	-65	-90	33	35
	Driven 12-in. dia. steel pipe (closed end)		-95	25	35
	Driven 18-in. dia. steel pipe (closed end)		-85	38	35
	Drilled, pressure grouted 12-in. dia. micropile		-90	29	50
Pump Station No. 2 (Alt. Location)	Driven HP12x74	-55	-80	25	35
	Driven 12-in. dia. steel pipe (closed end)		-85	19	35
	Driven 18-in. dia. steel pipe (closed end)		-75	27	35
	Drilled, pressure grouted 12-in. dia. micropile		-80	22	50

Notes:

1. Estimated pile tip elevations, drag loads, and tension capacity are provided for concept comparison purposes only. Pile lengths and allowable load require re-evaluation for the final selected pile types and verification by load tests during construction.

Axial pile capacity of driven piles is estimated in accordance with the methods recommended in the FHWA driven pile design manual (Ref. 35) and considering available experience with pile load testing in similar subsurface conditions local to the project area. The capacity of micropiles is calculated in general accordance with the design procedures for pressure grouted micropiles in the FHWA micropile design manual (Ref. 36). Micropile shaft resistance is calculated in accordance with recommended grout to ground bond values for soil types within the anticipated pile bond zone. Micropile toe resistance is ignored due to the small pile toe area and potential disturbance of soils below the pile toe during pile installation.

Down-drag load in Stratum A1 was considered as an additional load for structural evaluation but not a reduction in geotechnical capacity, as recommended by Fellenius (2004). To confirm piles would not undergo excessive settlement due to down-drag, a check was performed that the neutral plane is within the relatively incompressible bearing stratum where soil settlement is small. A factor of safety (FS) of 2 on geotechnical compression capacity was used to estimate pile lengths, premised on the performance of pile load tests during construction to confirm allowable pile design loads. Tension design loads were estimated based on a FS of 3. Higher tension loads may be possible with load testing.

Non-displacement driven piles, such as H-piles, are preferred to limit vibrations produced during pile driving. However, if downward migration of contamination is identified as a concern during environmental waste disposal characterization testing by the design builder, driven closed-end pipe piles may be more favorable than H-piles because they form a seal with adjacent soil and prevent migration of contamination along the length of the pile.

Exact pile tip elevations and design loads must be determined based on a detailed analysis of the selected pile type.

18.2. Grade Raise and Downdrag at Pump Stations

If fill is placed to raise grade adjacent to the pump station, the added load will cause settlement of the underlying Stratum A1 soil and impose downward drag load on the piles. Drag load must be considered as an increase in axial load in the structural design of piles where settlement will occur. Estimated drag load is summarized in Table 7. To minimize differential settlement between the pile-supported pump station and the adjacent ground, it is advisable to avoid raising grade adjacent to the pump station or use lightweight fill to avoid increasing the load on Stratum A1 (with appropriate hold-down restraint on the lightweight fill to resist buoyancy during floods).

Design of utility connections should anticipate some differential settlement between the pile-supported foundation and the surrounding soil. The magnitude of the differential settlement will be proportional to the weight of fill added to raise grade. Pipelines connecting to the pump stations will require pile support if the estimated settlement cannot be accommodated by flexible connections.

18.3. Pile Load Testing

Pile design loads must be substantiated by pile load testing during construction. Two piles at each pump station should be tested for axial capacity. Both test piles should be tested in accordance with ASTM D 4945 (high-strain dynamic test) and one should also be tested in accordance with ASTM D 1143 (static load test). Dynamic tests should be performed on initial drive and on re-strike a minimum of 7 days after pile installation. A tension test in accordance with ASTM D 3689 is also recommended depending on the magnitude of tension loads and safety factor used in pile design. Piles may be designed for an allowable load equal to one-half of the ultimate test load where successful load testing is performed. Wave equation analysis should be performed to establish driving criteria for driven piles at ultimate capacity and demonstrate that the selected hammer type and energy will not overstress the pile during driving.

18.4. Bottom Slab Design

The pump stations are underlain by thick deposits compressible Stratum A1 clay soils that will experience settlement with time particularly if surrounding grades are raised. The pump station slab should therefore be designed as a monolithic structural slab spanning between piles, with appropriate connections to transfer both compression and uplift loads to the piles.

18.5. Foundation Wall Design

Design of foundation walls must consider a combination of earth, water, seismic, and surcharge pressure. Earth pressures should be calculated using the at-rest pressures provided in Table 2. Foundation walls must also accommodate temporary increases in lateral pressure such as may occur from equipment surcharge (crane, etc.) adjacent to the structure, or seismic loading. Preliminary recommendations for these pressures are provided in Table 2. Either the surcharge load or seismic lateral load should be selected, whichever is larger. We recommend the surcharge load with prevailing groundwater level be compared to the case of no surcharge and flood level pressures for selection of the design lateral load.

18.6. Foundation Uplift & Waterproofing

The entire pump station foundation should be waterproofed and designed to resist hydrostatic pressures up to the design flood elevation. Waterproofing may consist of a membrane such as manufactured by WR Grace or a concrete admixture such as KIM, manufactured by Kryton International Inc. A waterstop such as Swellseal manufactured by DeNeef should be added to all joints.

18.7. Construction Considerations

18.7.1. Excavation and Support-of-Excavation (SOE)

The locations of Pump Station No. 1 and alternate Pump Station 2 overlap with the historic shoreline as shown on Drawing BLP-1. Buried remnants of the historic shoreline, including potential buried wharves and associated structures (Ref. 17) may be encountered during excavation for the pump stations and require removal to construct the pump stations. Boring B-2 made at Pump Station No. 1 encountered a concrete obstruction at 4 feet depth. Borings PS-04P and PS-04PA made near Pump Station No. 2 both encountered wood obstructions at 13 to 14 feet depth. We recommend old timber piles be cut off below subgrade and not pulled, to avoid creating new pathways for migration of contaminated soil and groundwater.

A watertight cofferdam consisting of interlocking steel sheet piles penetrating into the relatively impervious Stratum A1 clay is a suitable option to support the pump station excavation. The watertight cofferdam will limit the volume of pumped groundwater requiring treatment and disposal and will mitigate off-site drawdown and possible ground settlement due to groundwater lowering. The sheet piles must be driven to a sufficient depth to provide structural toe restraint, groundwater cutoff, and prevent base heave or blow-in at the bottom of the excavation. Two to 3 levels of internal bracing consisting of cross-lot struts and corner braces will likely be required to brace the excavation. Alternatively, tiebacks may be considered where conflicts with buried utilities and sensitive buildings or above-ground infrastructure can be avoided but will likely need to be long and steeply angled to reach suitable soils to develop bond. It may be possible to eliminate brace levels and/or shallower tip elevation if the lower stage of the excavation is made “in the wet” and a tremie concrete plug used to brace the base of the excavation prior to fully dewatering.

Pre-excavation and/or pre-drilling along the sheet pile alignment may be necessary to clear potential obstructions in the fill.

As noted in the Phase II Environmental Assessment Report attached to the GDR (Ref. 1), excavated soil will require proper characterization, handling, treatment, and/or off-site disposal (some excavated soil may be suitable for on-site reuse if it meets engineering and environmental criteria discussed in Section 20.4).

18.7.2. Construction Dewatering

Dewatering ahead of the excavation is necessary to maintain dry subgrade conditions. Assuming use of watertight shoring, shallow wells, vacuum wellpoints and/or pumping from sumps within the cofferdam should suffice for draining and controlling groundwater in the soils above excavation subgrade. Pumping to depressurize the underlying granular soils of Strata A2 and T using deep wells or ejectors may also prove necessary to lower groundwater pressures and prevent heave in the excavation bottom depending on the final pump station configuration and temporary shoring system design. Water pumped from the excavation will require pre-treatment for disposal as further discussed in the Phase II Environmental Assessment Report attached to the GDR (Ref. 1).

18.7.3. Pile Installation

Pile installation is expected to occur from at or near existing ground surface to avoid placement of pile equipment in the relatively narrow excavation. Pile installation from the ground surface will also avoid the potential for artesian conditions (upward water flow) and excavation instability that may occur when the pile penetrates through the A1 clay into the underlying granular water bearing deposits if the pile is installed from lower grade. If installed from ground surface, the piles may be installed with a sacrificial upper section or use of a follower (if driven). Pile driving should not occur within 50 ft of sensitive structures or utilities without assessment of structure condition and potential for damage caused by vibrations during pile driving. Noise may also be an issue for driven piles.

Given the development history of the project area, pre-excavation to remove shallow obstructions and/or pre-auguring or spudding of pile locations to clear deeper obstructions may be required in the Stratum F fill prior to pile installation.

18.7.4. Subgrade Preparation

The subgrade for the proposed pump station excavation is within soft / loose soil that is sensitive to disturbance and therefore requires careful handling and stabilization to allow construction of the foundation slab. Excavation to final subgrade should be made with smooth-edged digging tools operating from at least two feet above final subgrade. Any excessively soft, wet, or frozen soil, debris, or otherwise unsuitable material should be over-excavated and replaced with geotextile-wrapped crushed stone, compacted fill, flowable cementitious fill, or concrete. Hard points or obstructions (e.g. old timber piles) should be cut off or removed to a depth of 1 foot below subgrade and similarly backfilled.

Subgrade stabilization may consist of a heavy nonwoven geotextile (8 oz. / sq. yd. or higher) placed over the soil subgrade with a 6-inch layer of crushed stone placed over the geotextile with the geotextile wrapping the sides of the crushed stone layer. More robust stabilization measures such as structural geogrid and a thicker load transfer layer will likely be necessary if the subgrade will be used as a working platform to support pile installation or other equipment, depending on the contractor's selected installation methods and sequence.

18.7.5. Coordination of Pump Station No. 2 with New Fill Placement

A conceptual sequence for placement of the new fill at Pump Station No. 2 is described in Section 15.5. There are multiple possible options to coordinate construction of the pump station with the new fill. One approach is to install the pump station SOE and foundations from an interim work platform, before surcharge fill is placed. This scenario could potentially reduce the overall construction schedule but the support-of-excavation (SOE) and foundations would need to be designed to accommodate and/or resist the additional surcharge pressures and ground settlement during and after surcharge fill placement. An alternative approach is to install the pump station after the surcharge preloading improvement is complete and the surcharge fill is removed.

Differential settlement between Pump Station No. 2 and the surrounding fill is expected even after surcharge pre-loading. Flexible connections between the pump station and connecting utilities are therefore required. Estimates of the magnitude of the expected long-term settlement, assuming a program of surcharge preloading is performed, are provided in Section 15.5. Alternatively, adjoining utilities could be pile-supported and designed to not settle with the adjacent ground.

18.8. Pump Station No. 2 (Alternative Location):

A key constraint on foundation design and construction at the alternate Pump Station No. 2 location is imposed by the proximity of the existing office building at 211 North Union Street. The foundation type is not known from currently available information. If the building columns and slabs are supported on deep foundations, the use of driven piles may be feasible to construct the pump station. If either the columns or the slab (or both) of 211 North Union Street are supported on shallow foundations, it will be necessary to use drilled piles to construct the new pump station to avoid pile driving and excessive vibration that could cause settlement of the adjacent structure.

If the support of the adjacent structure on deep foundations is confirmed, SOE for the new pump station could consist of a sheet pile wall similar to that described for Pump Station No. 1. Otherwise, a stiff, reinforced concrete secant pile wall will likely be necessary to minimize both wall deflection and vibration.

Monitoring of the adjacent building for movement and vibration will be required during pump station construction.

18.9. Alternative Approaches at Pump Stations

There are other design and construction approaches that could prove less costly than the approach described above, subject to design and pricing by the design-builder. Such approaches could include, but are not limited to:

- Use of structural concrete walls such as secant pile or slurry diaphragm walls to support the pump station excavations and also form the foundation walls of the permanent structure
- Excavating in the wet and placement of an underwater tremie plug at the bottom of the cofferdam to limit dewatering during excavation
- Use of ground improvement such as deep soil mixing to create the SOE walls and stabilize the base of the excavation.

19.0 STORMWATER ATTENUATION FEATURES

Stormwater Best Management Practices (BMPs) reduce the volume of runoff entering the storm drain system through both infiltration and temporary storage. We understand that the City of Alexandria is considering two types of BMPs in the Waterfront Implementation plan:

- Right-of-way green infrastructure installed in the parking lane or sidewalk strip
- Underground stormwater storage and above-ground bioretention installed in park space

Approximate potential locations of the BMPs under consideration are shown on Drawing BLP-1.

19.1. Right-of-Way Green Infrastructure

Right-of-way green infrastructure (ROW GI) consists of curb-side infiltration trenches, dry swales, and permeable pavement. According to the Virginia Department of Environmental Quality (VA DEQ) Stormwater Design Specification No. 8, (Ref. 16), infiltration practices may be used in residential and urban areas where measured soil permeability rates exceed ½ inch per hour. Furthermore, Ref. 16 prohibits construction of infiltration-based practices above fill soils, restricting potential ROW GI locations to those inland of the historic shoreline.

Soil infiltration tests to evaluate the feasibility of ROW GI were performed at two locations between Fairfax and Lee Streets, one on Cameron Street (Boring GI-6) and one on Queen Street (Boring GI-8). Both locations are on relatively high ground inland of the original shoreline. The borings indicated a 1 to 2 foot thick layer of fill underlain by natural interlayered Terrace clay and sand (Strata T1 and T2, respectively).

Infiltration tests were performed in accordance with VA DEQ procedures at multiple depths below the ground surface, as described in the GDR (Ref. 1). Based on measured infiltration rates, infiltration practices appear potentially viable on both Cameron and Queen Streets. However, the testing indicated significant variability in suitability by both depth and location, summarized as follows:

Table 8 - Right-of-Way Green Infrastructure Suitability

Boring	Street	Test Depth (ft)	Measured Avg. Infiltration Rate (in/hr)	Summary of Suitability based on Infiltration Test Results and Boring Log
GI-6	Cameron	5	1.0	Both shallow (e.g. permeable pavement) and deeper (e.g. infiltration trench) practices appear possible. However, the clayey layer between 4 and 9 feet depth could inhibit downward flow and limit efficiency of infiltration practices.
		10	1.4	
GI-8	Queen	5	0	Shallow infiltration practices do not appear possible but deep-bottomed infiltration practices such as infiltration trenches with deep-bottomed stone chambers could be effective but would require significant excavation to install.
		9	5.3	

Infiltration test locations on King and South Lee Streets were also considered but were eliminated due to equipment access limitations and density of buried utilities.

Infiltration practices should be designed, constructed, and maintained in accordance with the VA DEQ criteria provided in Ref. 16. Given the expected variation in measured infiltration rates due to the naturally occurring spatial variation of the Terrace soil deposit, additional testing is necessary to refine locations and appropriate types of ROW GI practices in the final design. VA DEQ recommends a minimum test frequency of one test per 1,000 square feet of infiltration surface area.

19.2. Underground Detention Chambers

Underground detention chambers capture and temporarily store excess surface stormwater to delay its release into the storm drainage system. Three detention chamber locations are currently conceived, two in Waterfront Park and one in Founders Park. Plan dimensions of the proposed chambers range from approximately 60 x 120 feet to over 100 x 700 feet, as shown on Drawing BLP-1.

Stormwater detention chambers are often designed to both store water and allow it to percolate into the underlying soils. However, due to the siting of the potential chamber locations on fill, infiltration is not allowed according to VA DEQ (Ref. 16). Therefore, the chambers must be sealed and designed to drain collected stormwater to the downstream pump stations.

Due to the high target storage volume and shallow water table, the bottom elevation of the proposed chambers is below the groundwater table. Conceptual design information provided by Carollo (Ref. 15) indicates a bottom of storage elevation -5.7 feet, approximately 6 to 9 feet below the water table. Due to the need for a thick structural slab to resist uplift forces, the required subgrade elevation will be lower and will fall within the highly variable Stratum F (fill) or the soft Stratum A1 silt/clay.

Installing the underground detention chambers on variable, soft subgrade conditions below the water table introduces significant design and construction considerations, summarized below. The intent of this summary is not to provide specific design or construction criteria but rather to identify key factors that impact feasibility and cost for use in cost-benefit evaluation.

19.2.1. Design Considerations

Key design considerations for the underground detention chambers include:

1. chamber subgrade depth relative to the groundwater level. In general, maintaining chambers as shallow as possible will improve efficiency and reduce cost of construction.
2. the potential need for hold-down piles, anchors, and/or a thickened base slab to resist buoyancy and avoid excessive settlement, depending on depth of cover
3. the need for robust structural design to transfer hydrostatic uplift load to the piles or anchors
4. full waterproofing of chambers and measures to manage groundwater that may leak into the chambers
5. the potential for differential settlement between the underground chambers and connecting utility pipes

19.2.2. Construction Considerations

Key construction considerations for the underground detention chambers include:

1. the need for a watertight support-of-excavation (SOE) system to limit the volume of pumped groundwater requiring treatment and disposal and prevent off-site drawdown and migration of groundwater contamination. Note that the presence of the thick, soft Stratum A1 soil deposit below the excavation will result in heavy SOE sections and deep required toe embedment.
2. the need for a construction dewatering program to lower the groundwater level by up to 10 feet or more within the excavation, and treatment of discharge prior to disposal.

3. the difficulty of preparing a suitable bearing subgrade for the chambers on the loose and easily disturbed soils of Strata F and A1.

Additional construction considerations for the underground detention chambers include:

- As shown on Drawing BLP-1, the planned detention chamber footprints overlap the historic shoreline, including potential buried wharves and associated structures (Ref. 17). Remnants of these structures may be encountered during excavation for the chambers and will need to be removed to construct the chambers. Boring GI-14, located in the footprint of one of the planned detention chambers, repeatedly encountered a concrete obstruction at 4 feet depth. It is possible that voids exist beneath buried wharves that will require filling when exposed.

- As noted in the Phase II Environmental Assessment Report attached to the GDR (Ref. 1), excavated soil will require proper characterization, handling, treatment, and/or off-site disposal (some excavated soil may be suitable on-site reuse if it meets engineering and environmental criteria discussed in Section 20.4).

20.0 GENERAL FOUNDATION DESIGN CONSIDERATIONS

20.1. Lightly Loaded Foundations

Support of foundations for lightly loaded structures such as the pavilion / restroom and landscaping features on shallow spread or strip footings bearing in the Stratum F fill is viable provided the footings are constructed on compacted structural fill placed in an undercut replacing the existing fill. These foundations may be designed for a maximum net allowable bearing pressure of 500 psf, increased by 33% for wind and seismic loads. Use of higher bearing pressures is possible if tolerable settlements are demonstrated by analyses. Footings should bear at a depth of at least 30 inches below finish grade for frost protection with a minimum 2.5-foot thick layer of compacted structural fill below the bearing elevation.

Where the existing fill at the footing locations is confirmed (e.g. by test pits or borings) to be granular and free of voids, organic material, trash, or debris, the need for an undercut and replacement fill can be eliminated. This determination must be made on a footing-by-footing basis and approved by the responsible geotechnical engineer.

Floor slabs for lightly loaded structures may be designed as slabs-on-grade provided the subgrade is prepared as described in Section 20.3. A non-woven separation geotextile (minimum 6 oz. / sq. yd.) should be placed over the prepared subgrade followed by a 4-inch thick capillary break layer of crushed stone (e.g. VDOT No. 57 stone) and nylon-reinforced plastic vapor barrier. Slab edges should be turned down a minimum of 30 inches below finish grade to avoid frost effects.

20.2. Corrosion Potential and Chemical Attack

Corrosion potential and chemical testing were performed on bulk composite samples obtained from three borings (GI-10, GI-11 and GI-14) made in the Phase II Geotechnical Exploration. Test results are consistent with the findings of the Phase I geotechnical investigation, indicating that the Fill exhibits low to moderate sulfate exposure. Therefore, Type II or equivalent sulfate-resistant cement should be used for below-grade construction. As noted in the Phase I GER (Ref. 3), the soils in the evaluated project area are also potentially corrosive to ductile iron according to AWWA Specification C105. Use of additional sacrificial steel or protective coatings are expected for buried metal structures.

We refer to Refs. 2 and 3 for further detail regarding the corrosion potential and potential for chemical attack on concrete of the soils in the evaluated project area. The results of corrosion potential and chemical testing presented in the both the Phase I and Phase II GDR (Refs. 1 and 2) should be reviewed by the design-builder's corrosion consultant to develop specific recommendations with respect to corrosion protection of proposed structures and buried utilities.

20.3. Subgrade Preparation for Shallow Foundations and Pavements

Subgrades for pavements, shallow foundations, slabs, and structural fill supporting those elements should be prepared using smooth-edged digging tools operating from at least 2 feet above the subgrade. Subgrades should be proof-rolled with two passes of a smooth drum, non-vibratory roller to identify excessively soft areas. Any wet, frozen, or excessively soft, organic material, debris, trash, or other unsuitable material at subgrade should be removed and replaced with structural fill, flowable cementitious fill, or concrete. Hard points or obstructions should be removed flush to subgrade before placing overlying materials.

20.4. Backfill

Recommendations for Fill Materials, Compaction, Site Grading, and Earthwork provided in the Phase I GER (Ref. 3) remain applicable.

Some excavated fill may be suitable for on-site reuse as fill based on testing performed on bulk samples of Stratum F collected from five borings in the Phase II Geotechnical Exploration. Specifically:

- The bulk sample from Boring BH-04 (located in Waterfront Park) met geotechnical engineering criteria for reuse as Structural Backfill, Structural Fill, or Common Fill.
- Bulk samples from Borings PS-02 and GI-10 (Waterfront Park) met geotechnical criteria for Structural Fill or Common Fill.
- Bulk samples from Borings GI-11 and GI-14A/B (Founders Park) are suitable only for reuse as Common Fill based on moisture-density and CBR characteristics.

Bulk sample test results are presented in Table C of the Phase II GDR (Ref. 1).

Additional classification, moisture-density, and plasticity testing are needed during final design and construction to confirm geotechnical suitability of excavated fill for re-use, once excavation locations are finalized. At least one set of tests should be required for each 500 cubic yards of excavated soil with additional tests at any visible change in excavated material type. Excavated soil will require processing through a 3-inch screen for reuse as Structural Fill or a 4-inch screen for reuse as Common Fill and may require drying to remove excess moisture. Oversized particles, organics, trash, debris, or other unsuitable material must be removed and disposed off-site. Cost and space needs for processing excavated fill must be balanced against the cost of off-site disposal and importing new fill.

Contaminants are present in the existing fill that will require characterization testing to define suitability for reuse and, if found unsuitable, to satisfy disposal facility criteria and State of Virginia requirements as further discussed in the Phase II Environmental Assessment Report attached to the GDR (Ref. 1). As noted in that report, it is anticipated that the majority of excavated soil will be characterized as Category 2 – Contaminated Soil requiring testing and off-site disposal at a regulated facility.

21.0 PAVEMENT DESIGN

We have analyzed and designed flexible asphalt pavement sections for new pavements following the procedure in "Pavement Design Guide for Subdivision and Secondary Roads in Virginia", by Virginia

Department of Transportation, August 2018 revision. The input average California Bearing Ratio (CBR) is based on our review of tests performed throughout the project area in the Phase II geotechnical exploration, presented in Table C of the GDR (Ref. 1). Input parameters for Annual Average Daily Traffic (AADT) and Heavy Commercial Vehicles (HCV) are based on our review of the 2019 and 2020 versions of “Virginia Department of Transportation Daily Traffic Volume Estimate Including Vehicle Classification Estimates, Special Locality Report 100, City of Alexandria”.

Additional testing to confirm the subgrade CBR at new roadway locations should be performed once those locations are established and the pavement design updated as necessary.

Our design is based on the following parameters:

- Design for Secondary Road with 20 year service life
- Average CBR = 12
- Resiliency Factor = 2.0
- Growth Rate = 5%
- Prince, Fairfax and Duke Streets AADT less than or equal to 4,000 and HCV less than or equal to 5 percent
- Cameron Street AADT equal to 5,000 and HCV less than or equal to 5 percent
- King Street AADT equal to 4,400 and HCV equal to 13 percent

Recommended pavement sections are given below. Sections are based on the design parameters listed herein and assume proper construction, inspection, and maintenance. Pavement drainage must be designed so that water is drained away both from the pavement surface and the pavement subgrade. Failure to provide adequate drainage will shorten pavement life.

Pavement subgrades should be prepared in accordance with recommendations given in the Subgrade Preparation section. Unsuitable subgrade materials observed during proof-rolling (i.e. excessive rutting or weaving) should be over-excavated and replaced with fill meeting the requirements for Structural Fill.

Table 9 – Recommended Flexible Pavement Section

Course	Material	Thickness for: Prince, Fairfax, Duke Streets	Thickness for: Cameron and King Street
Surface	Asphalt Concrete VDOT SM-9.5A	1.5 inches	2.0 inches
Base	Asphalt Concrete VDOT BM-25.0A	6.0 inches	6.0 inches
Subbase	VDOT Aggregate Base Material	6.0 inches	6.0 inches

Tack coat must be applied between all courses. All pavement materials must conform with and be placed in accordance with the applicable Virginia State and local DOT specifications. Variations of the recommended thickness of the courses can be evaluated by the design-builder based on in-place unit costs of all components.

For new pavement sections in existing roadways, the pavement design should match or exceed the existing pavement section. Pavement cores should be obtained to confirm the existing pavement section at these locations.

22.0 INSTRUMENTATION AND MONITORING

An instrumentation and monitoring program is required during construction to assess the effects of the proposed construction on adjacent structures and permit timely remedy of construction methods and procedures when necessary to avoid potential damage. Recommendations for geotechnical instrumentation and monitoring are outlined in Table 10 based on the conceptual layout and design of proposed structures described herein. Typical monitoring instrumentation details are provided in Appendix B. The design-builder is responsible for development of a more detailed monitoring program specific to the final project and proposed methods of construction.

Table 10 - Conceptual Monitoring Plan

Instrument / Monitoring Type	Application	Location / Number / Frequency	Monitoring Schedule / Duration	Typical Detail Figure (App. B)
Pre-construction condition survey	Existing buildings	Key structures within 100 feet of proposed construction	At least 2 weeks prior to construction within 100 feet; repeat upon completion of construction	n/a
Ground Surface Settlement Monitoring Point	Monitor hardscape features (sidewalks, roadways, etc.) for settlement	Install in arrays at 50 feet spacing on key features within zone of influence of new fill placement	At least 2 weeks prior to construction within 100 feet; monthly thereafter through construction completion	B1
Optical prism (deformation monitoring point)	Existing buildings	Install at 25-50 feet horizontal spacing on structures within 50 feet of construction		B2
	Existing buried utilities	Install at 50-100 feet horizontal spacing on crown of significant utilities within 50 feet of construction		B3
	Support-of-Excavation at new pump stations and detention chambers (if used)	Install at 25feet horizontal spacing on top of SOE and each brace level	Weekly during excavation; monthly through structure backfill to grade	B2
	New Bulkhead	Install at 50 feet horizontal spacing on top of new bulkhead	Weekly during duration of fill placement and dredging	B2
Seismograph (vibration monitoring)	Existing buildings	Place in structures located within 50 feet of pile driving activity	Continuous; program instrument to produce waveform when triggering peak particle velocity (PPV) reached	n/a
Inclinometer	Lateral ground movement at key buried utilities	Install adjacent to key buried utilities within 25 feet of proposed excavations, new fill, or new bulkhead	Baseline 2 weeks prior to construction within 25 feet, read monthly through construction completion	B4
	Lateral ground movement at bulkhead at edge of new fill	Install in a row at 100 foot horizontal spacing, 5-10 feet inland of new bulkhead where new fill is placed (including surcharge fill)	Baseline 2 weeks prior to placement of new fill, read weekly during fill placement and every 2-3 months for 1 year after fill completion / surcharge fill removal (if used)	B4
Piezometer (open standpipe or fully grouted vibrating wire)	Dewatering for pump station excavations	At least two within each dewatered excavation, spaced midway between dewatering wells / wellpoints, plus two outside the SOE	Weekly during duration of dewatering program	B5 or B6
Piezometer (fully grouted vibrating wire)	Surcharge preloading at new landfill for offset bulkhead	Install in vertical arrays at 3 depths within compressible layer; one array per 1000 ft^2	Weekly for duration of surcharge preload, until target degree of consolidation reached	B6
Settlement Plate		Install at base of surcharge fill, one point per 1000 ft^2		B7

Note: Threshold and limiting values for each type of instrument should be developed during final design, specific to the construction type and structure being monitored.

23.0 SUPPLEMENTAL GEOTECHNICAL INVESTIGATION

The Phase II Geotechnical Exploration has improved the characterization of soils and groundwater along the Alexandria waterfront and also identified gaps in the available data that may require further exploration depending on the final project design and proposed methods of construction. Recommendations for the types, location, and frequency of this supplemental geotechnical investigation are provided in Table 11 based on the conceptual design described herein. The design-builder is responsible for developing the exact scope of any supplemental investigations as the design is further progressed since the type, location, and number of explorations are dependent on the nature and extent of the improvements implemented.

24.0 CLOSURE

This report is prepared for the sole use of the client and is specific to the project design contemplated at the time of report issue.

Table 11 - Recommended Supplemental Geotechnical Investigation

Investigation Type	Location	Application	Location / Number / Frequency	Depth / Remarks
Deep soil borings or CPT probes	Proposed bulkhead	Define soil profile along new bulkhead and anchorage	Provide 100-ft. avg. spacing along final bulkhead alignment; add'l borings spaced at 100-200 feet in anchorage zone for anchored bulkhead	• Met by Phase II and previous borings for current proposed bulkhead alignment, except in vicinity of City Marina and Commercial Docks. If cantilevered or battered pile bulkheads are selected, additional deep borings should be considered to better define character and depth of pile bearing strata (T and P1) particularly at transitions identified in Phase II investigation such as between Stations 9+00 and 12+00 • Extend 20 feet deeper than anticipated bulkhead / deadman bottom elevation, or at least 20 feet into Potomac soils
Shallow borings	Proposed lightly loaded structures on shallow foundations	Define fill properties to determine need for undercut and replacement	Minimum two per structure or one per 2500 square feet of built-over area	• Extend to base of Stratum F (Fill)
Test pits	Proposed pump stations, underground detention chambers, or bulkhead anchorages	Define below-grade obstructions and/or remnant structures in path of proposed structures	Minimum 5 per proposed pump station or detention chamber (4 corners and middle), or at 50-foot maximum spacing	• Supplement test pits with ground penetrating radar (GPR) and/or seismic refraction surveys to add definition to buried obstruction type, depth, and spatial distribution
Test pits	211 N. Union Street building	Confirm existing building foundation type	As needed to confirm existing building columns and slab within 50 feet of proposed construction are pile supported	• Only required for Pump Station No. 2 (Alternate Location) • Test pits outside and/or inside the building (ground floor parking level) • Perform non-destructive pulse echo testing on exposed piles to estimate length
Bulk sampling for fill properties testing	Pump station and underground detention chamber excavations	Confirm geotechnical engineering properties for reuse as fill	One set of tests per 500 cubic yards of excavated fill, plus additional at any observed change in material	• Laboratory testing to include: sieve analysis (ASTM D422 / D6913), classification (ASTM D 2488), Atterberg Limits (ASTM D4318), Moisture Density (ASTM D698)
Bulk sampling for Pavement support	New Roadways	Confirm subgrade properties at proposed new pavements	As defined in VDOT Pavement Design Guide for Subdivision and Secondary Roads	• Laboratory testing to include: sieve analysis (AASHTO T 27), classification (ASTM D2488 / AASHTO M145) compaction (AASHTO T99) California Bearing Ratio (AASHTO T193)
Pavement cores	Any location where the existing pavement will be reused	Confirm existing pavement section	See Remark	• Minimum of 1 core for roadway segments up to 200 feet, or 2 cores for segments up to 500 feet in length
Infiltration rate tests	Final locations selected for infiltration-based infrastructure	Confirm soil infiltration rates consistent with design	As per VA DEQ requirement	• Perform in accordance with VA DEQ Stormwater Design Specification No. 8 • Perform at a depth approximately 2 feet below bottom of proposed infiltration practice
Detailed bulkhead condition survey	Full length of any segment of bulkhead planned for reuse or modification			Potential inspection types (not limited to): • detailed water-side bulkhead inspection • diver survey • ultra-sound / pulse echo / GPR testing to determine thickness and depth of steel elements • concrete strength / soundness testing • test pits to evaluate existing anchorages • any additional investigation determined necessary by the design-builder

Notes:

1. For recommended supplemental environmental characterization sampling and testing, refer to the Environmental Assessment Report attached to the Geotechnical Data Report (Ref. 1).

25.0 REFERENCES

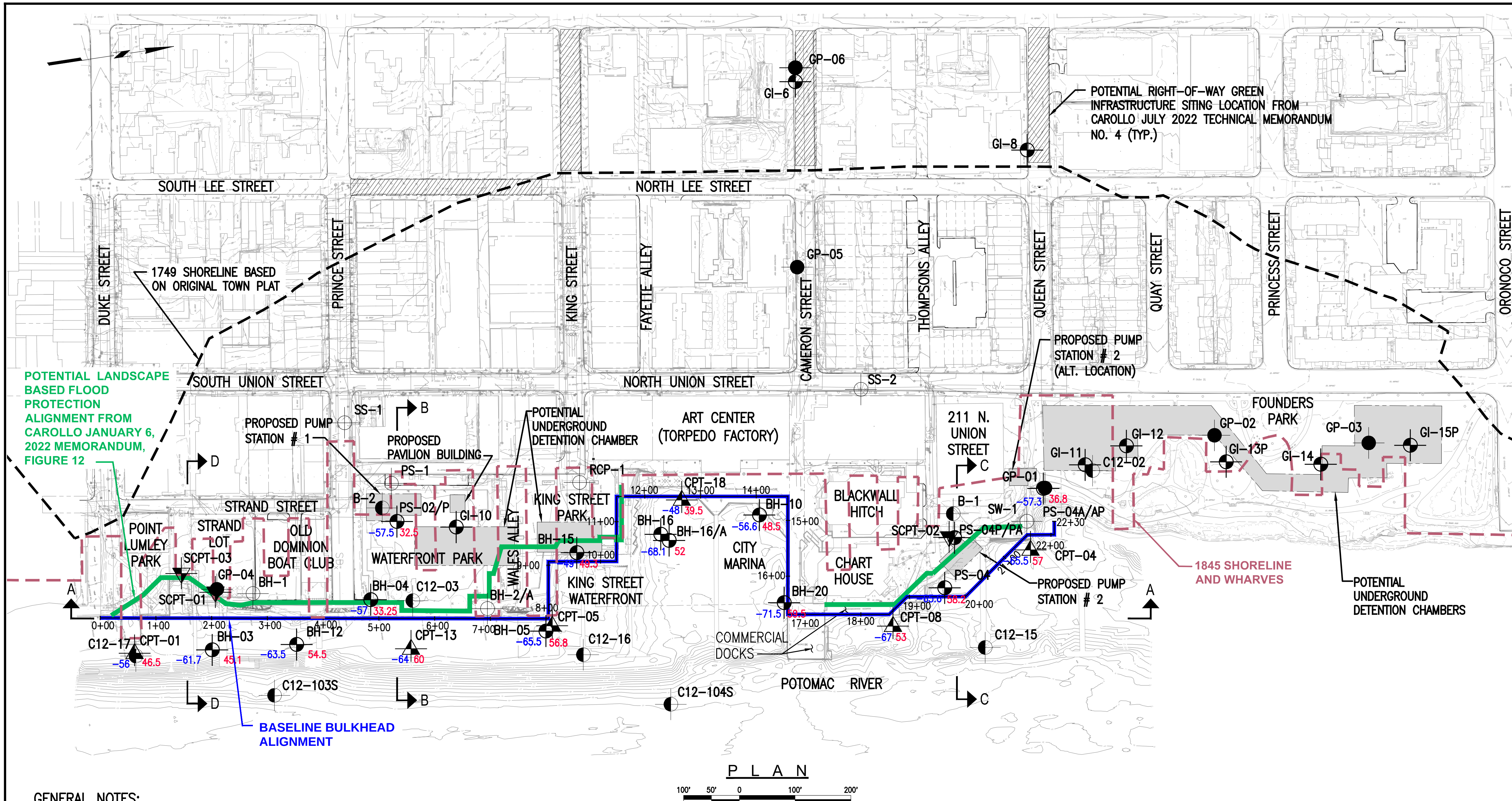
25.1. Project References

1. Geotechnical Data Report, City of Alexandria, VA Waterfront Implementation Project, Phase II Geotechnical Exploration, prepared by MRCE, dated June 23, 2022
2. Phase 1 Geotechnical Data Report Alexandria Waterfront Flood Mitigation, Alexandria Virginia, prepared by Schnabel, dated October 26, 2016.
3. Phase 1 Geotechnical Report, Alexandria Waterfront Flood Mitigation, Alexandria, Virginia, prepared by Schnabel, dated January 25, 2017.
4. Master Storm Water Management Plan prepared by Stantec, dated November 5, 2018.
5. Bulkhead Evaluation prepared by Moffatt & Nichol, dated December 19, 2017.
6. Waterfront Dock & Marina Maintenance & Repair Assessment prepared by Michael Baker Jr., Inc. dated October 2013.
7. 15% Concept Design Submission drawings prepared by URS, dated 2014.
8. 15%-30% Schematic Design drawings prepared by Olin, dated 2014.
9. Alexandria Waterfront Bulkhead Technical Design Manual Draft prepared by Olin, Moffatt & Nichol, and Stantec dated September 24, 2019.
10. Preliminary Archaeological Assessment Response provided by Alexandria Archaeology dated 11/05/2021.
11. “Bulkhead Existing Condition Assessment and Recommendations,” prepared by Carollo, dated January 6, 2022.
12. “January 10, 2022 Bulkhead Site Walk Findings,” prepared by MRCE, dated February 14, 2022.
13. PowerPoint slides illustrating Cost Based Waterfront Plan – 1 and Current Project Alternative provided by Carollo, April 4, 2022.
14. Annotated documents illustrating pump station design concepts provided by Carollo, April 4, 2022.
15. Drawings entitled “City of Alexandria Flood Control, Alexandria, VA” prepared by Advanced Drainage Systems, Inc. illustrating conceptual underground detention chamber design, dated June 19, 2021 (8 sheets).
16. Virginia DEQ Stormwater Design Specification No. 8, Infiltration Practices, Version 1.9, dated March 1, 2011.
17. Initial Archaeological Assessment of the Proposed Waterfront Flood Management Project, Alexandria, Virginia, prepared by Stantec, dated November 2018.
18. Technical Memorandum 4, Parkspace and Streetscape Stormwater Attenuation Solutions, prepared by Carollo, dated July 2022 (Draft Final).
19. Technical Memorandum 2, Potomac River Flood Frequency Analysis, prepared by Carollo, dated May 2022.

25.2. Literature References

20. Hara, A., Ohta, T., Niwa, M., Tanaka, S., and Banno, T. “Shear Modulus and Shear Strength of Cohesive Soils.” *Soils and Foundations* 14.3 (1974): 1-12.
21. Bray, Jonathan D., and Rodolfo B. Sancio. "Assessment of the liquefaction susceptibility of fine-grained soils." *ASCE Journal of Geotechnical and Geoenvironmental Engineering* 132.9 (2006): 1165-1177.
22. Seed, H. Bolton, and Izzat M. Idriss. "Simplified procedure for evaluating soil liquefaction potential." *ASCE Journal of the Soil Mechanics and Foundations Division* 97.9 (1971): 1249-1273.

23. Youd, T. Leslie, and Izzat M. Idriss. "Liquefaction resistance of soils: summary report from the 1996 NCEER and 1998 NCEER/NSF workshops on evaluation of liquefaction resistance of soils." *ASCE Journal of Geotechnical and Geoenvironmental Engineering* 127.4 (2001): 297-313.
24. Tokimatsu, Kohji, and H. Bolton Seed. "Evaluation of settlements in sands due to earthquake shaking." *ASCE Journal of Geotechnical Engineering* 113.8 (1987): 861-878.
25. Ishihara, Kenji, and Mitsutoshi Yoshimine. "Evaluation of settlements in sand deposits following liquefaction during earthquakes." *Soils and Foundations* 32.1 (1992): 173-188.
26. Seed, H. B., and Whitman, R. V. 1970. "Design of Earth Retaining Structures for Dynamic Loads", ASCE Specialty Conference, Lateral Stresses in the Ground and Design of Earth Retaining Structures, pp 103-147.
27. Mikola, Roozbeh Geraili, and Nicholas Sitar. "Seismic earth pressures on retaining structures in cohesionless soils." *California Department of Transportation* (2013).
28. Agusti, G. C., and N. Sitar. "Seismic earth pressure on retaining structures in cohesive soils, Report submitted to the California Department of Transportation (Caltrans)." *Report No. UCB GT* (2013): 13-102.
29. Fellenius, B.H.. "Unified design of piled foundations with emphasis on settlement analysis. Honoring George G. Goble — "Current Practice and Future Trends in Deep Foundations" Geo-Institute Geo-TRANS Conference, Los Angeles, July 27-30, 2004, Edited by J.A. DiMaggio and M.H. Hussein. ASCE Geotechnical Special Publication, GSP 125, pp. 253 - 275.
30. Craig, Robert F. *Craig's Soil Mechanics*. CRC press, 2004.
31. Johnson, Stanley J. "Precompression for improving foundation soils." *ASCE Journal of the Soil Mechanics and Foundations Division* 96.1 (1970): 111-144.
32. Mesri, G., and T. W. Feng. "Surcharging to reduce secondary compression." *Journal of Geotechnical Engineering for Coastal Development-Theory to Pratice, Yokohama* 1 (1991): 359-364.
33. Unified Facilities Criteria (UFC) Soil Mechanics (DM 7.1). United States of America Department of Defense UFC 3-220-10 (2022).
34. Foundations and Earth Structures Design Manual (DM 7.2). United States Department of the Navy Facilities Engineering Command, (1982).
35. U.S. Department of Transportation Federal Highway Administration (FHWA) Publication No. FHWA-NHI-16-009, Design and Construction of Driven Pile Foundations – Volume 1 (2016).
36. U.S. Department of Transportation Federal Highway Administration (FHWA) Publication No. FHWA NHI-05-039, Micropile Design and Construction Reference Manual (2005).
37. APILE v2015 – Technical Manual by ENSOFT, INC.



	COORDINATES OF PHASE II BORINGS/CPTS			
	BORING/CPT	NORTHING	EASTING	GROUND SURFACE/ MUDLINE ELEVATION
LANDSIDE	BH-04*	6978822	11899218	4±
	BH-15*	6979200	11899190	3±
	PS-02P	6978889	11899087	5.28
	PS-04P/PA	6979876	11899264	2.87
	PS-04A/AP	6980048	11899201	3.31
	GI-6*	6979714	11898413	23±
	GI-8*	6980107	11898597	18±
	GI-10*	6978993	11899112	6±
	GI-11	6980089	11899126	4.98
	GI-12	6980204	11899148	7.03
	GI-13P	6980357	11899199	7.78
	GI-14	6980543	11899233	7.81
	GI-15P	6980707	11899223	8.12
	GP-01	6980045	11899202	3.25
	GP-02	6980363	11899153	8.05
	GP-03	6980634	11899208	8.48
	GP-04	6978551	11899158	3.25
	GP-05*	6979671	11898703	8±
	GP-06*	6979718	11898389	25±
	SCPT-01	6978548	11899163	3.19
	SCPT-02	6979868	11899263	2.87
	SCPT-03	6978493	11899121	3.75
WATERSIDE	BH-03	6978527	11899265	−4.9
	BH-05	6979124	11899321	−3.7
	BH-10	6979534	11899172	−8.1
	BH-12	6978678	11899278	−4.5
	BH-16	6979354	11899180	−7.8
	BH-16A	6979367	11899193	−8.0
	BH-20	6979554	11899335	−12.0
	PS-04	6979843	11899351	−7.4
	CPT-01*	6978389	11899250	−9.5
	CPT-04	6980005	11899307	−8.5
	CPT-05*	6979667	11898743	−3.5
	CPT-08	6979741	11899405	−14
	CPT-13	6980376	11899203	−14.1
	CPT-18	6979400	11899125	−8.5

* INDICATES BORING OR CPT LOCATIONS WITHOUT GPS OR SURVEY COORDINATES. COORDINATES ARE BASED ON PROPOSED LOCATIONS, ADJUSTED FOR FIELD MEASUREMENTS WHERE AVAILABLE.

GENERAL NOTES:

1. PHASE II LAND BORING/CPT LOCATIONS SHOWN ARE BASED ON SURVEY COORDINATES PROVIDED BY GORDON, UNLESS OTHERWISE NOTED. WATER BORING/CPT LOCATIONS SHOWN ARE BASED ON HAND HELD GPS COORDINATES TAKEN BY MRCE AND ARE APPROXIMATE, UNLESS OTHERWISE NOTED.
2. BASE SURVEY TAKEN FROM "TOPOGRAPHICAL SURVEY OF ALEXANDRIA WATERFRONT IMPROVEMENTS" DATED 12/9/2021, PREPARED BY GORDON.
3. BATHYMETRY TAKEN FROM CITY OF ALEXANDRIA – WATERFRONT FLOOD MITIGATION IMPLEMENTATION PROJECT, TOPOGRAPHIC SURVEY, DUKE ST. TO QUEEN ST. & UNION ST. TO POTOMAC RIVER, PREPARED BY STANTEC, DATED 9/12/2016.
4. COORDINATES REFERENCE VIRGINIA STATE PLANE NORTH NAD83 (2011) HORIZONTAL DATUM. ELEVATIONS REFERENCE NAVD88 DATUM.
5. PRIOR BORING LOCATIONS WERE PROVIDED BY CAROLLO ENGINEERS INC., AND ARE APPROXIMATE.
6. PHASE II LAND BORINGS WERE MADE BY FREE STATE DRILLING INC., OF FREDERICK MARYLAND BETWEEN NOVEMBER 1, AND NOVEMBER 18, 2021 UNDER THE CONTINUOUS INSPECTION OF MUESER RUTLEDGE CONSULTING ENGINEERS, PLLC (MRCE).
7. PHASE II CPTS WERE PERFORMED BY INSITU SOIL TESTING, L.C. OF LANCASTER VIRGINIA (INSITU) ON NOVEMBER 23, 2021 (LANDSIDE), AND BETWEEN DECEMBER 23 AND DECEMBER 28, 2021 (WATER SIDE) UNDER THE INSPECTION OF MRCE.
8. PHASE II GEOPROBES WERE MADE BY GSI MID-ATLANTIC INC. OF BEL AIR MARYLAND ON NOVEMBER 17, AND DECEMBER 6 2021 UNDER THE CONTINUOUS INSPECTION OF HAYES & MONAGHAN, GEOARCHAEOLOGISTS, LLC.
9. PHASE II WATER BORINGS WERE MADE BY CONNELLY AND ASSOCIATES INC. OF FREDERICK MARYLAND BETWEEN NOVEMBER 30, 2021 AND JANUARY 12, 2022 UNDER THE CONTINUOUS INSPECTION OF MRCE.
10. PHASE I BORINGS WERE MADE BY FREE STATE STATE DRILLING INC., OF FREDERICK MARYLAND FOR SCHNABEL ENGINEERING, LLC IN 2016. BORING LOCATIONS ARE APPROXIMATE.
11. HISTORIC SHORELINES TRACED FROM MAP OBTAINED FROM CITY OF ALEXANDRIA DEPARTMENT OF PLANNING AND ZONING WEBSITE (ACCESSED FEB. 2022). OVERLAY IS APPROXIMATE.
12. ALL PROPOSED FEATURE LOCATIONS SHOWN ARE APPROXIMATE.
13. FOR GEOLOGIC SECTIONS, SEE DRAWING NOS. GS-1 TO GS-7.

LEGEND:

BH-03
−61.7 | 45.1
– PHASE II SOIL BORING
– "BH" INDICATES 91–105 FT BULKHEAD BORING
– "PS" INDICATES 100 FT PUMP STATION BORING
– "GI" INDICATES 20 FT GREEN INFRASTRUCTURE BORING
– "P" INDICATES 14–20 FT OPEN STANDPIPE MONITORING WELL INSTALLED IN BORING
THICKNESS OF STRATUM A1 (FT) (TYP.)
ELEVATION OF TOP OF STRATUM T/P1 (FT) (TYP.)

GP-01
– PHASE II GEOPROBE BORING FOR GEO-ARCHAEOLOGICAL SAMPLING

CPT-01
−56 | 46.5 – PHASE II CPT PROBE
THICKNESS OF STRATUM A1 (FT) (TYP.)
ELEVATION OF TOP OF STRATUM T/P1 (FT) (TYP.)

SCPT-01
– PHASE II SEISMIC CPT PROBE

SS-1
– PHASE I BORING PERFORMED BY OTHERS

B-1
– PRIOR BORING PERFORMED BY OTHERS

1+00 – STATION WITH STATION NUMBER

– 1749 SHORELINE

– 1845 SHORELINE AND WHARVES

– PROPOSED BASELINE BULKHEAD ALIGNMENT

– PROPOSED LANDSCAPE BASED FLOOD PROTECTION ALIGNMENT

REV.	DATE	BY	DESCRIPTION

CITY OF ALEXANDRIA WATERFRONT IMPLEMENTATION PROJECT PHASE II GEOTECHNICAL INVESTIGATION
ALEXANDRIA VIRGINIA

CAROLLO ENGINEERS, INC.
NEW YORK NEW YORK

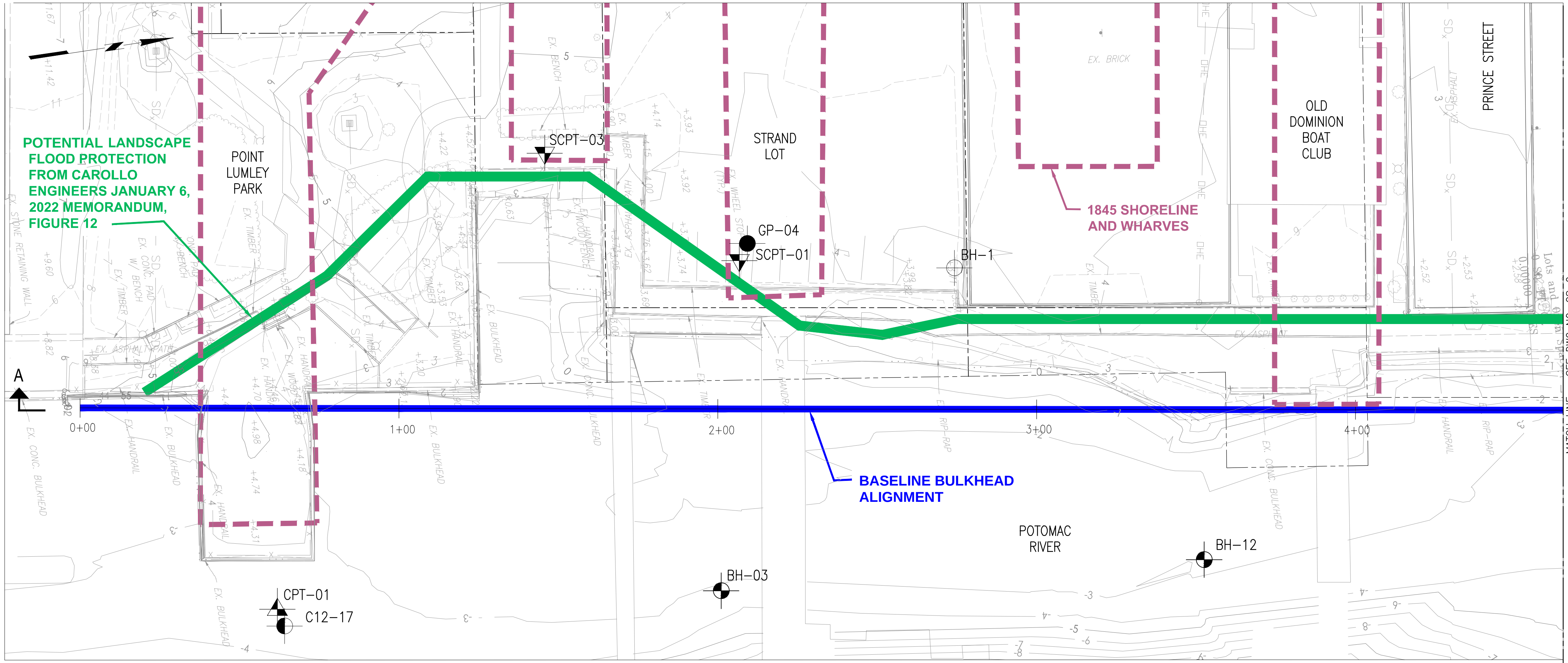
MUESER RUTLEDGE CONSULTING ENGINEERS PLLC
14 PENN PLAZA – 225 WEST 34TH STREET, NEW YORK, NY 10122

SCALE	MADE BY: B.B.	DATE: 09-15-2022	FILE NUMBER
GRAPHIC	CH'KD BY: A.L.S.	DATE: 09-15-2022	14123

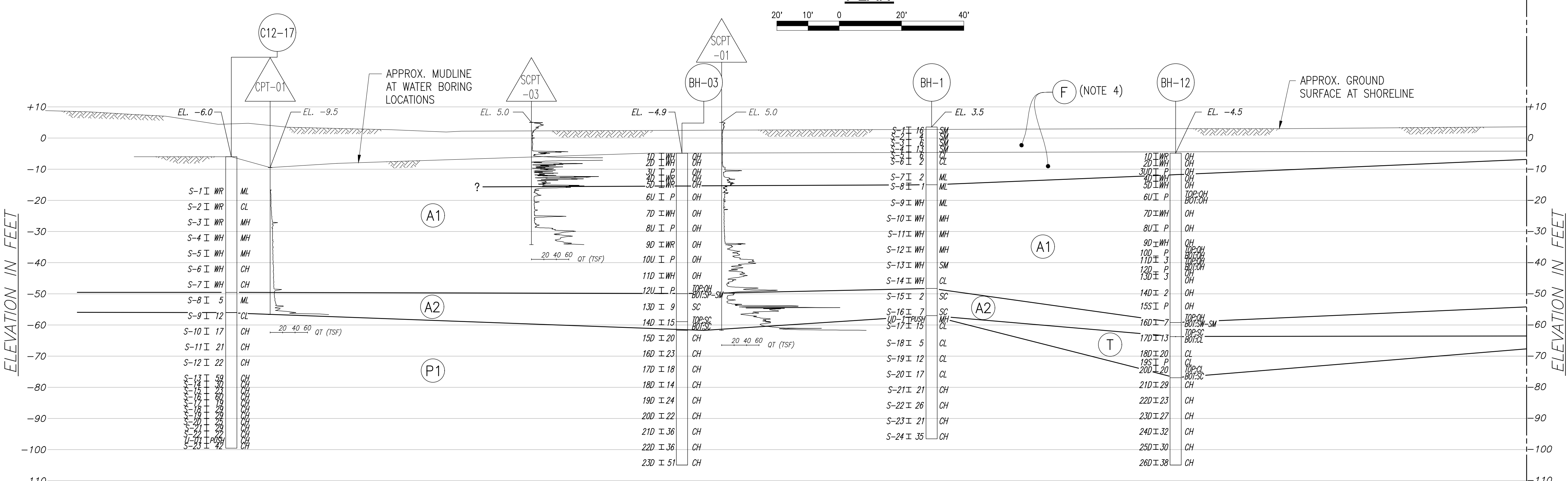
BORING LOCATION PLAN

DRAWING NUMBER
BLP-1

Last saved by: lreva on Thursday, Sep 15, 2022 – 4:04:39 PM
Printed on: Thursday, Sep 15, 2022 – 04:06:59 PM
Printed by: Larisa Reva
G:\DWGS\141\14123\BLP-1.dwg



PLAN



SECTION A-A

NOTES:

1. SEE DRAWING NO. BLP-1 FOR GENERAL NOTES AND LEGEND.
2. BORINGS ILLUSTRATED ON GEOLOGIC SECTIONS ARE PROJECTED TO THE SECTION.
3. STRATIFICATION SHOWN IS AN INTERPRETATION OF THE BORINGS AND MAY NOT REPRESENT ACTUAL SUBSURFACE CONDITIONS.
4. FILL ONLY APPLICABLE AT LAND-SIDE BORINGS.
5. SEE DRAWING NO. GS-R FOR GEOLOGIC SECTION SYMBOL DEFINITIONS.

GENERAL STRATA DESCRIPTIONS:

(F) FILL – LOOSE TO VERY COMPACT FINE TO COARSE SAND WITH VARYING AMOUNTS OF GRAVEL, SILT, CLAY, BRICK, CONCRETE, WOOD AND ROOTS

(A1) ALLUVIAL CLAY/SILT – SOFT TO MEDIUM CLAY AND ORGANIC CLAY WITH VARYING AMOUNTS OF SAND, SILT, SHELLS, AND ROOTS

(A2) ALLUVIAL SAND – LOOSE TO MEDIUM COMPACT SILTY FINE SAND AND CLAYEY FINE SAND WITH VARYING AMOUNTS OF MEDIUM TO COARSE SAND, SHELLS, AND GRAVEL

(T) "OLD TOWN TERRACE" – LOOSE TO MEDIUM COMPACT FINE TO COARSE SAND WITH VARYING AMOUNTS OF SILT, CLAY, AND GRAVEL

(TO) "OLD TOWN TERRACE" (ORGANIC MEMBER) – STIFF FINE SANDY SILT AND SILT, TRACE MEDIUM TO COARSE SAND AND DECOMPOSED WOOD

(P1) POTOMAC CLAY – STIFF TO HARD CLAY AND SILT WITH VARYING AMOUNTS OF SAND AND GRAVEL

REV.	DATE	BY	DESCRIPTION

CITY OF ALEXANDRIA WATERFRONT IMPLEMENTATION PROJECT PHASE II GEOTECHNICAL INVESTIGATION
ALEXANDRIA VIRGINIA

CAROLLO ENGINEERS, INC.
NEW YORK NEW YORK

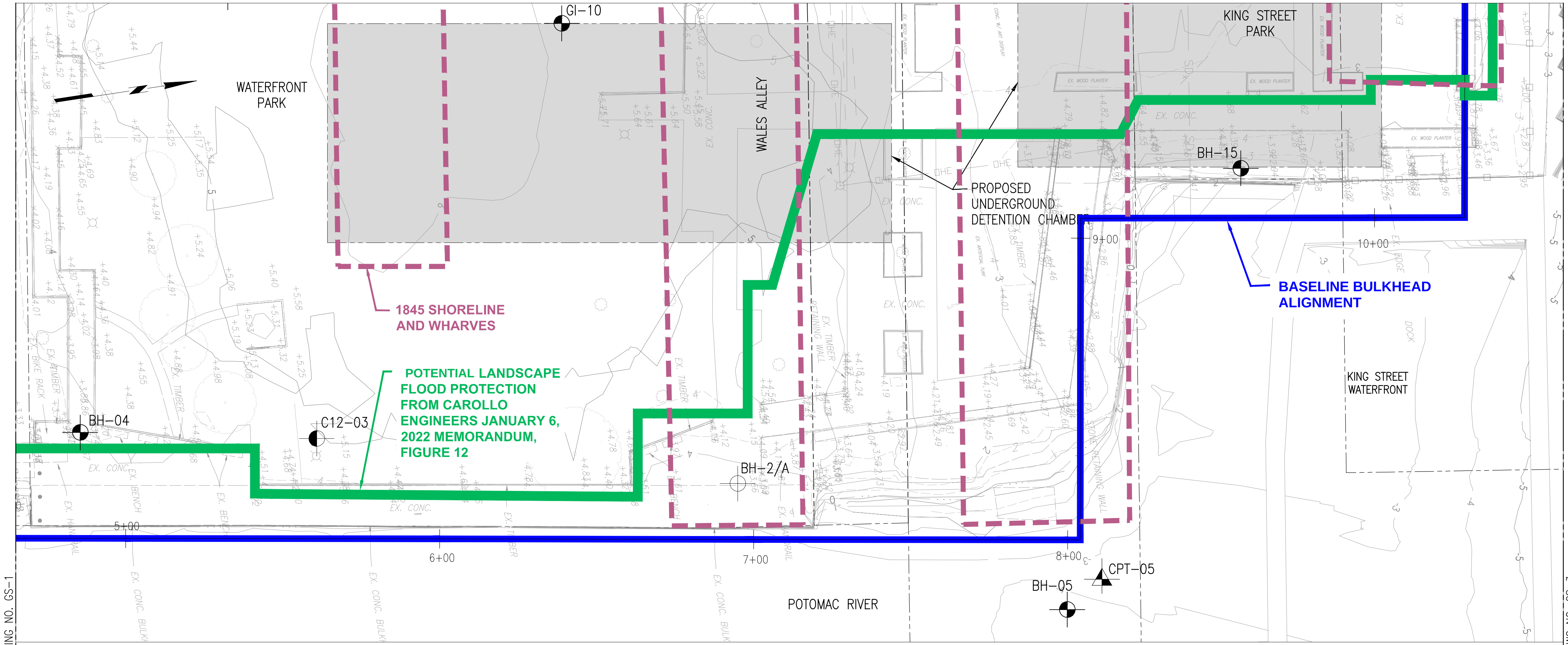
MUESER RUTLEDGE CONSULTING ENGINEERS PLLC
14 PENN PLAZA – 225 WEST 34TH STREET, NEW YORK, NY 10122

SCALE	MADE BY: L.R.	DATE: 09-15-2022	FILE NUMBER
GRAPHIC	CH'KD BY: A.L.S.	DATE: 09-15-2022	14123

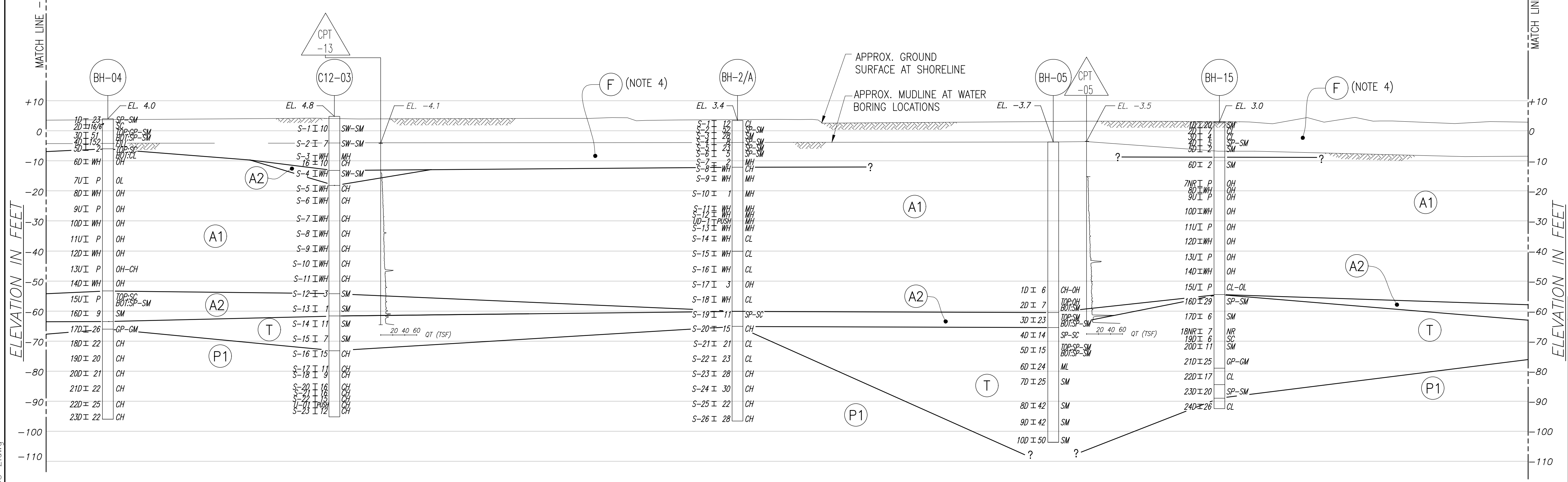
PLAN AND GEOLOGIC SECTION A-A SHEET 1 OF 4

DRAWING NUMBER GS-1

Last saved by: lreva on Thursday, Sep 15, 2022 – 1:07:27 PM
Printed on: Thursday, Sep 15, 2022 – 01:08:32 PM
Printed by: Larisa Reva
G:\DWGS\141\14123\GS-1.dwg



PLAN



SECTION A-A

NOTES:

1. SEE DRAWING NO. BLP-1 FOR GENERAL NOTES AND LEGEND.
2. BORINGS ILLUSTRATED ON GEOLOGIC SECTIONS ARE PROJECTED TO THE SECTION.
3. STRATIFICATION SHOWN IS AN INTERPRETATION OF THE BORINGS AND MAY NOT REPRESENT ACTUAL SUBSURFACE CONDITIONS.
4. FILL ONLY APPLICABLE AT LAND-SIDE BORINGS.
5. SEE DRAWING NO. GS-R FOR GEOLOGIC SECTION SYMBOL DEFINITIONS.

GENERAL STRATA DESCRIPTIONS:

(F) FILL – LOOSE TO VERY COMPACT FINE TO COARSE SAND WITH VARYING AMOUNTS OF GRAVEL, SILT, CLAY, BRICK, CONCRETE, WOOD AND ROOTS

(A1) ALLUVIAL CLAY/SILT – SOFT TO MEDIUM CLAY AND ORGANIC CLAY WITH VARYING AMOUNTS OF SAND, SILT, SHELLS, AND ROOTS

(A2) ALLUVIAL SAND – LOOSE TO MEDIUM COMPACT SILTY FINE SAND AND CLAYEY FINE SAND WITH VARYING AMOUNTS OF MEDIUM TO COARSE SAND, SHELLS, AND GRAVEL

(T) "OLD TOWN TERRACE" – LOOSE TO MEDIUM COMPACT FINE TO COARSE SAND WITH VARYING AMOUNTS OF SILT, CLAY, AND GRAVEL

(TO) "OLD TOWN TERRACE" (ORGANIC MEMBER) – STIFF FINE SANDY SILT AND SILT, TRACE MEDIUM TO COARSE SAND AND DECOMPOSED WOOD

(P1) POTOMAC CLAY – STIFF TO HARD CLAY AND SILT WITH VARYING AMOUNTS OF SAND AND GRAVEL

REV.	DATE	BY	DESCRIPTION

CITY OF ALEXANDRIA WATERFRONT IMPLEMENTATION PROJECT PHASE II GEOTECHNICAL INVESTIGATION

ALEXANDRIA VIRGINIA

CAROLLO ENGINEERS, INC.

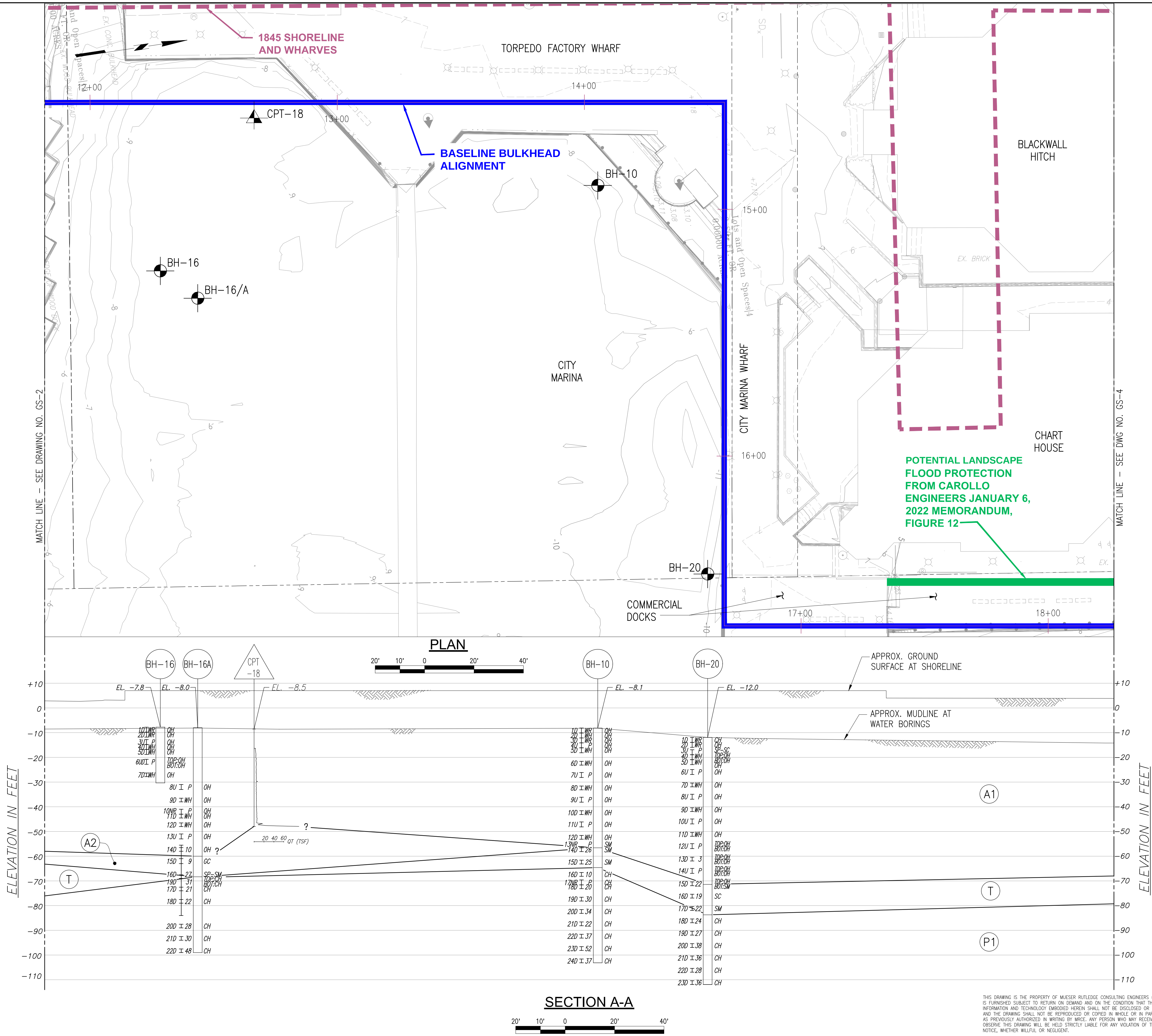
NEW YORK NEW YORK

MUESER RUTLEDGE CONSULTING ENGINEERS PLLC
14 PENN PLAZA – 225 WEST 34TH STREET, NEW YORK, NY 10122

SCALE	MADE BY: L.R.	DATE: 09-15-2022	FILE NUMBER
GRAPHIC	CH'KD BY: A.L.S.	DATE: 09-15-2022	14123

PLAN AND GEOLOGIC SECTION A-A SHEET 2 OF 4

DRAWING NUMBER GS-2



NOTES:

1. SEE DRAWING NO. BLP-1 FOR GENERAL NOTES AND LEGEND.
2. BORINGS ILLUSTRATED ON GEOLOGIC SECTIONS ARE PROJECTED TO THE SECTION.
3. STRATIFICATION SHOWN IS AN INTERPRETATION OF THE BORINGS AND MAY NOT REPRESENT ACTUAL SUBSURFACE CONDITIONS.
4. FILL ONLY APPLICABLE AT LAND-SIDE BORINGS.
5. SEE DRAWING NO. GS-R FOR GEOLOGIC SECTION SYMBOL DEFINITIONS.

GENERAL STRATA DESCRIPTIONS:

(F) FILL – LOOSE TO VERY COMPACT FINE TO COARSE SAND WITH VARYING AMOUNTS OF GRAVEL, SILT, CLAY, BRICK, CONCRETE, WOOD AND ROOTS

(A1) ALLUVIAL CLAY/SILT – SOFT TO MEDIUM CLAY AND ORGANIC CLAY WITH VARYING AMOUNTS OF SAND, SILT, SHELLS, AND ROOTS

(A2) ALLUVIAL SAND – LOOSE TO MEDIUM COMPACT SILTY FINE SAND AND CLAYEY FINE SAND WITH VARYING AMOUNTS OF MEDIUM TO COARSE SAND, SHELLS, AND GRAVEL

(T) "OLD TOWN TERRACE" – LOOSE TO MEDIUM COMPACT FINE TO COARSE SAND WITH VARYING AMOUNTS OF SILT, CLAY, AND GRAVEL

(TO) "OLD TOWN TERRACE" (ORGANIC MEMBER) – STIFF FINE SANDY SILT AND SILT, TRACE MEDIUM TO COARSE SAND AND DECOMPOSED WOOD

(P1) POTOMAC CLAY – STIFF TO HARD CLAY AND SILT WITH VARYING AMOUNTS OF SAND AND GRAVEL

REV.	DATE	BY	DESCRIPTION

CITY OF ALEXANDRIA WATERFRONT IMPLEMENTATION PROJECT PHASE II GEOTECHNICAL INVESTIGATION
ALEXANDRIA VIRGINIA

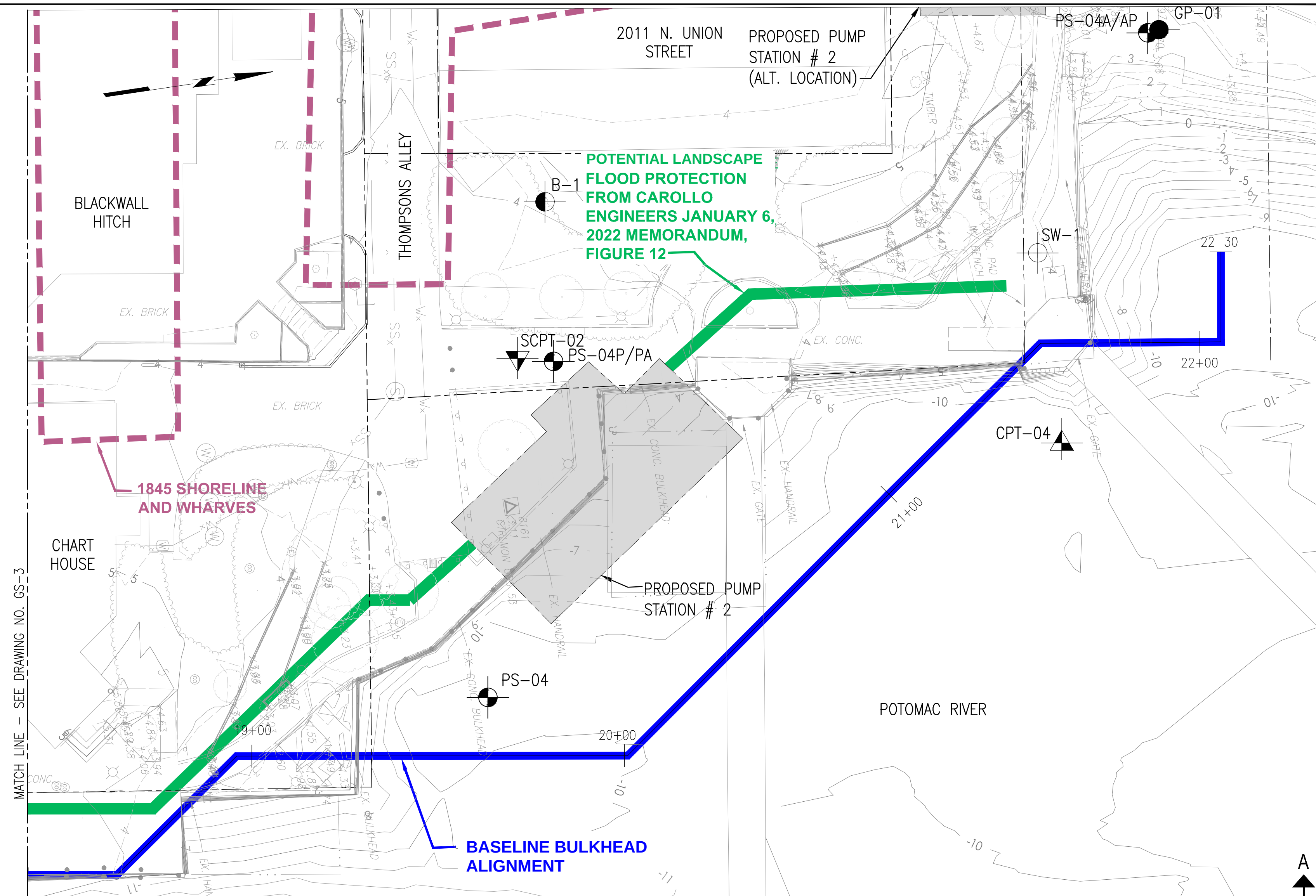
CAROLLO ENGINEERS, INC.
NEW YORK NEW YORK

MUESER RUTLEDGE CONSULTING ENGINEERS PLLC
14 PENN PLAZA – 225 WEST 34TH STREET, NEW YORK, NY 10122

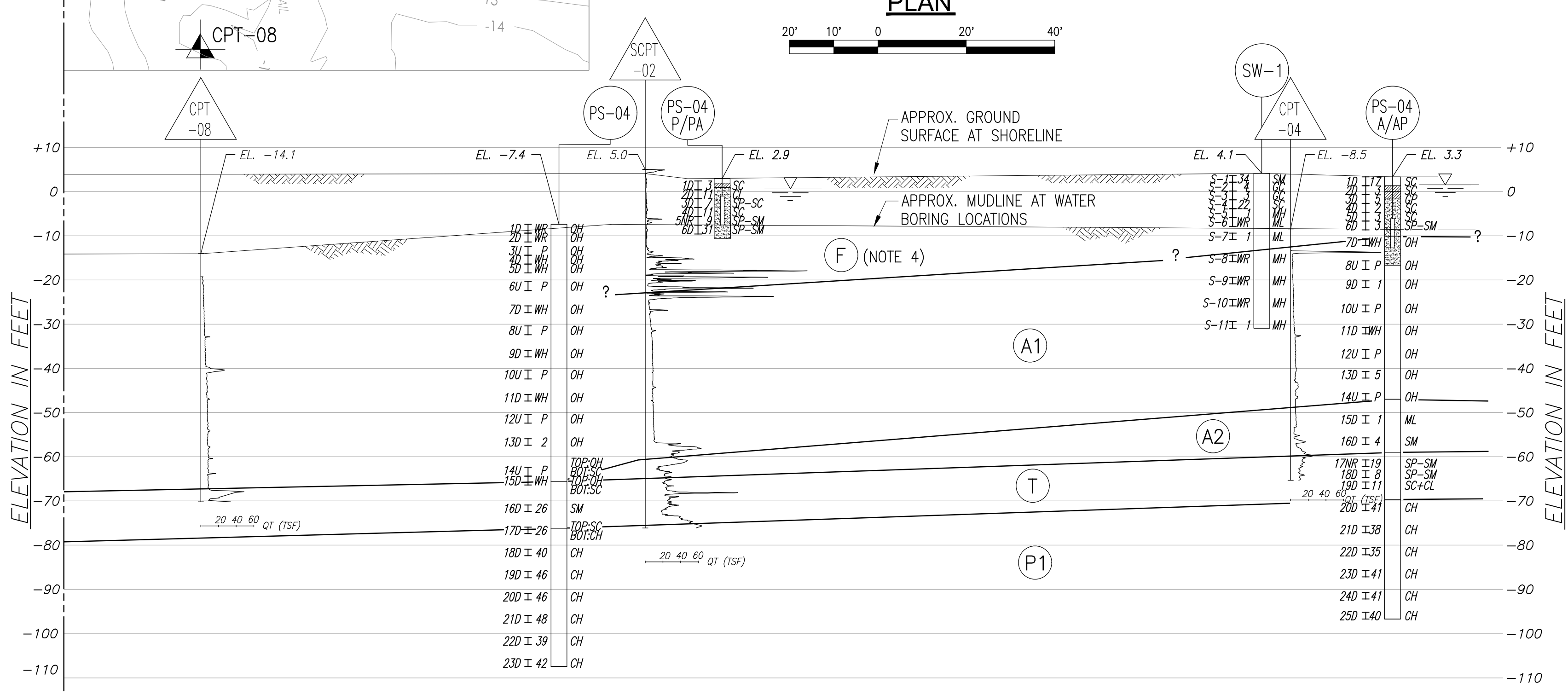
SCALE	MADE BY: L.R.	DATE: 09-15-2022	FILE NUMBER
GRAPHIC	CH'KD BY: A.L.S.	DATE: 09-15-2022	14123

PLAN AND GEOLOGIC SECTION A-A
SHEET 3 OF 4

DRAWING NUMBER
GS-3



PLAN



SECTION A-A

NOTES:

1. SEE DRAWING NO. BLP-1 FOR GENERAL NOTES AND LEGEND.
2. BORINGS ILLUSTRATED ON GEOLOGIC SECTIONS ARE PROJECTED TO THE SECTION.
3. STRATIFICATION SHOWN IS AN INTERPRETATION OF THE BORINGS AND MAY NOT REPRESENT ACTUAL SUBSURFACE CONDITIONS.
4. FILL ONLY APPLICABLE AT LAND-SIDE BORINGS.
5. SEE DRAWING NO. GS-R FOR GEOLOGIC SECTION SYMBOL DEFINITIONS.

GENERAL STRATA DESCRIPTIONS:

F — FILL – LOOSE TO VERY COMPACT FINE TO COARSE SAND WITH VARYING AMOUNTS OF GRAVEL, SILT, CLAY, BRICK, CONCRETE, WOOD AND ROOTS

A1 — ALLUVIAL CLAY/SILT – SOFT TO MEDIUM CLAY AND ORGANIC CLAY WITH VARYING AMOUNTS OF SAND, SILT, SHELLS, AND ROOTS

A2 — ALLUVIAL SAND – LOOSE TO MEDIUM COMPACT SILTY FINE SAND AND CLAYEY FINE SAND WITH VARYING AMOUNTS OF MEDIUM TO COARSE SAND, SHELLS, AND GRAVEL

T — "OLD TOWN TERRACE" – LOOSE TO MEDIUM COMPACT FINE TO COARSE SAND WITH VARYING AMOUNTS OF SILT, CLAY, AND GRAVEL

TO — "OLD TOWN TERRACE" (ORGANIC MEMBER) – STIFF FINE SANDY SILT AND SILT, TRACE MEDIUM TO COARSE SAND AND DECOMPOSED WOOD

P1 — POTOMAC CLAY – STIFF TO HARD CLAY AND SILT WITH VARYING AMOUNTS OF SAND AND GRAVEL

REV.	DATE	BY	DESCRIPTION

CITY OF ALEXANDRIA WATERFRONT IMPLEMENTATION PROJECT PHASE II GEOTECHNICAL INVESTIGATION

ALEXANDRIA — VIRGINIA

CAROLLO ENGINEERS, INC.

NEW YORK — NEW YORK

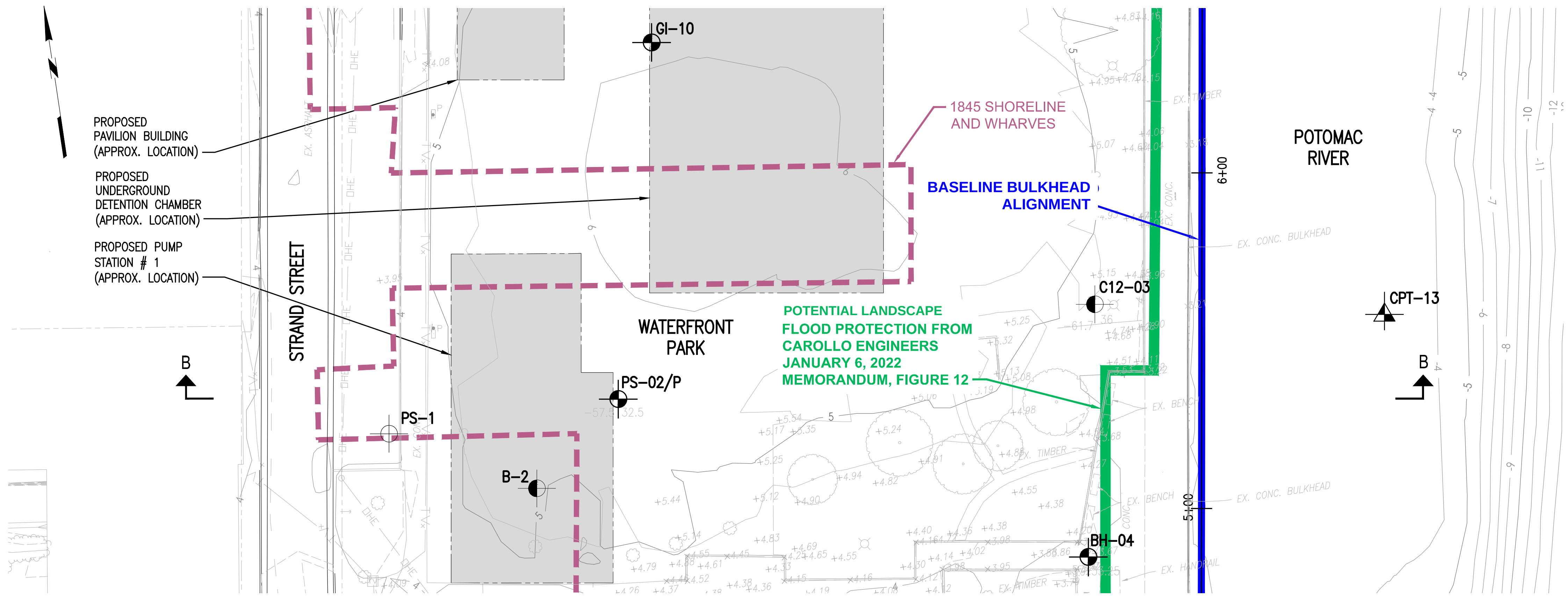
MUESER RUTLEDGE CONSULTING ENGINEERS PLLC

14 PENN PLAZA – 225 WEST 34TH STREET, NEW YORK, NY 10122

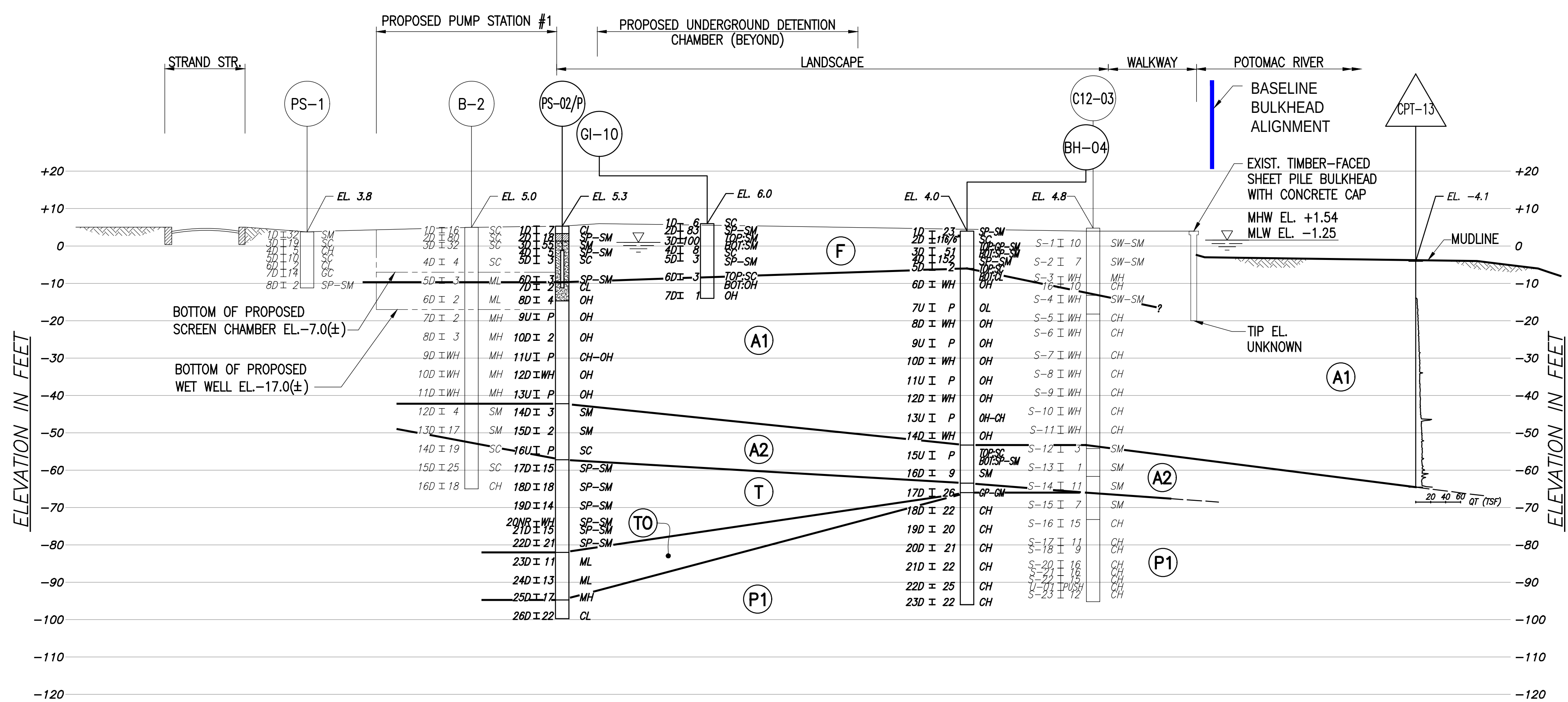
SCALE			FILE NUMBER
GRAPHIC	MADE BY: L.R.	DATE: 09-15-2022	14123
	CH'KD BY: A.L.S.	DATE: 09-15-2022	

PLAN AND GEOLOGIC SECTION A-A SHEET 4 OF 4

DRAWING NUMBER: GS-4



PLAN



SECTION B-B

NOTES:

1. SEE DRAWING NO. BLP-1 FOR GENERAL NOTES AND LEGEND.
2. BORINGS ILLUSTRATED ON GEOLOGIC SECTIONS ARE PROJECTED TO THE SECTION.
3. STRATIFICATION SHOWN IS AN INTERPRETATION OF THE BORINGS AND MAY NOT REPRESENT ACTUAL SUBSURFACE CONDITIONS.
4. SEE DRAWING NO. GS-R FOR GEOLOGIC SECTION SYMBOL DEFINITIONS.

GENERAL STRATA DESCRIPTIONS:

(F) FILL – LOOSE TO VERY COMPACT FINE TO COARSE SAND WITH VARYING AMOUNTS OF GRAVEL, SILT, CLAY, BRICK, CONCRETE, WOOD AND ROOTS

(A1) ALLUVIAL CLAY/SILT – SOFT TO MEDIUM CLAY AND ORGANIC CLAY WITH VARYING AMOUNTS OF SAND, SILT, SHELLS, AND ROOTS

(A2) ALLUVIAL SAND – LOOSE TO MEDIUM COMPACT SILTY FINE SAND AND CLAYEY FINE SAND WITH VARYING AMOUNTS OF MEDIUM TO COARSE SAND, SHELLS, AND GRAVEL

(T) "OLD TOWN TERRACE" – LOOSE TO MEDIUM COMPACT FINE TO COARSE SAND WITH VARYING AMOUNTS OF SILT, CLAY, AND GRAVEL

(TO) "OLD TOWN TERRACE" (ORGANIC MEMBER) – STIFF FINE SANDY SILT AND SILT, TRACE MEDIUM TO COARSE SAND AND DECOMPOSED WOOD

(P1) POTOMAC CLAY – STIFF TO HARD CLAY AND SILT WITH VARYING AMOUNTS OF SAND AND GRAVEL

REV.	DATE	BY	DESCRIPTION

CITY OF ALEXANDRIA WATERFRONT IMPLEMENTATION PROJECT PHASE II GEOTECHNICAL INVESTIGATION

ALEXANDRIA VIRGINIA

CAROLLO ENGINEERS, INC.

NEW YORK NEW YORK

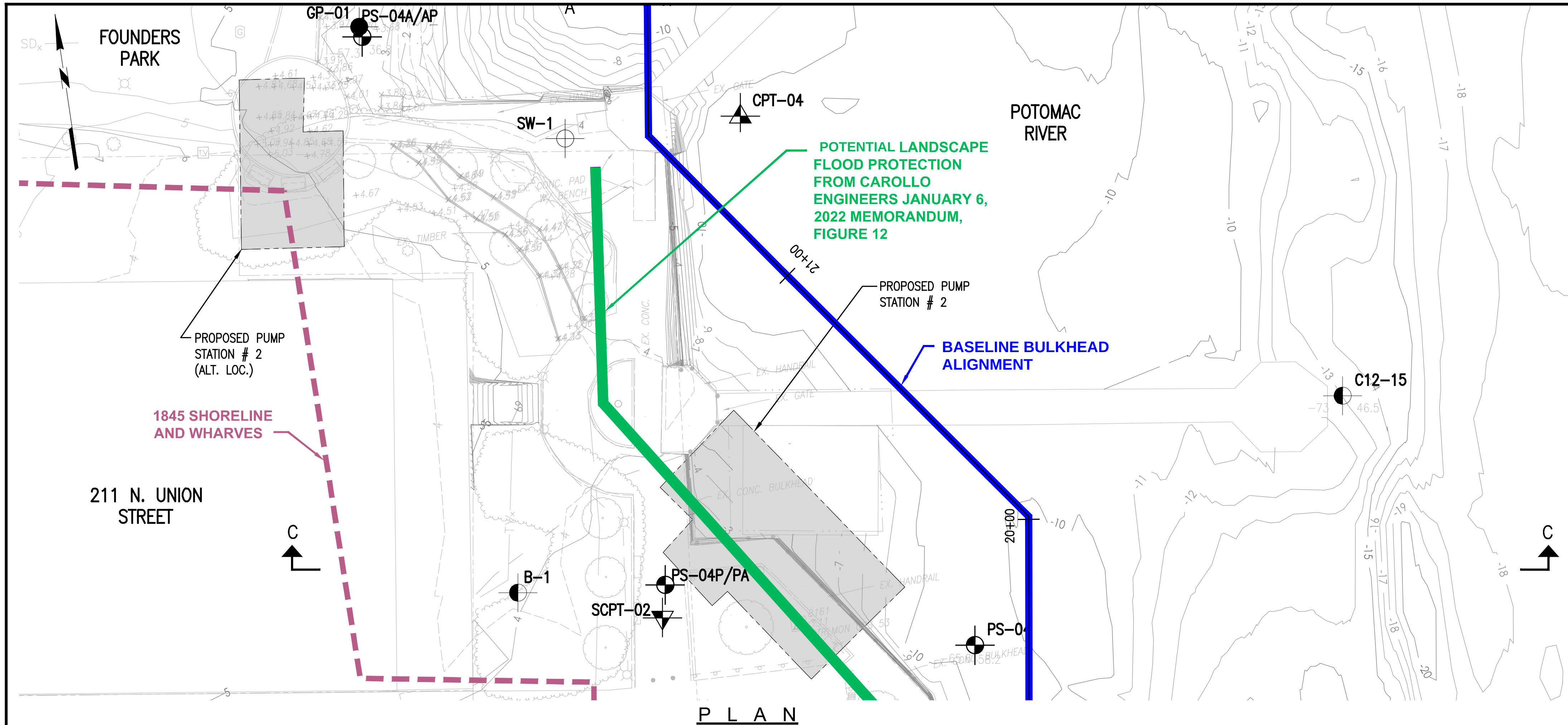
MUESER RUTLEDGE CONSULTING ENGINEERS PLLC
14 PENN PLAZA – 225 WEST 34TH STREET, NEW YORK, NY 10122

SCALE	MADE BY: L.R.	DATE: 09-15-2022	FILE NUMBER
GRAPHIC	CH'KD BY: A.L.S.	DATE: 09-15-2022	14123

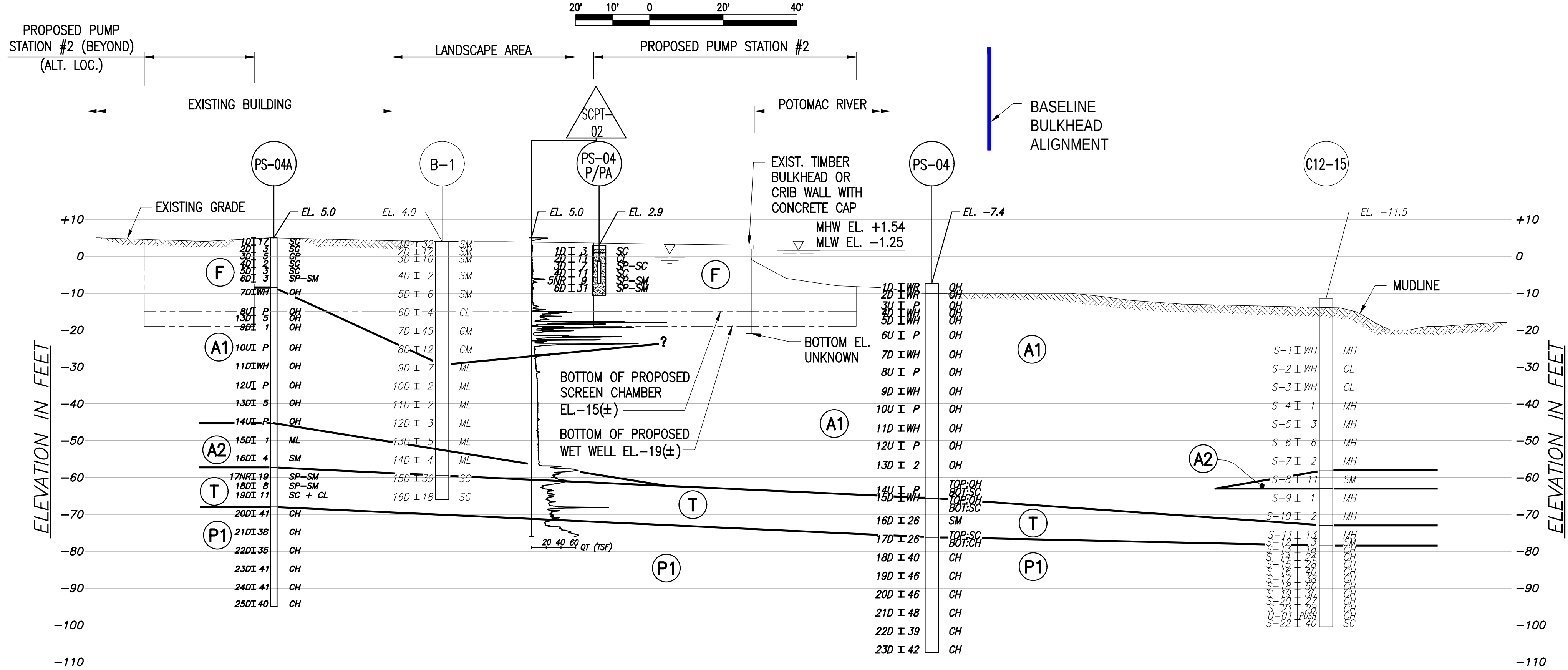
PLAN AND GEOLOGIC SECTION B-B

DRAWING NUMBER
GS-5

Last saved by: lreva on Thursday, Jun 30, 2022 - 6:30:44 PM
Printed on: Thursday, Sep 15, 2022 - 01:16:06 PM
Printed by: Larisa Reva
G:\DWGS\141\14123\GS-5.dwg



P L A N



SECTION C-C

NOTES:

1. SEE DRAWING NO. BLP-1 FOR GENERAL NOTES AND LEGEND.
2. BORINGS ILLUSTRATED ON GEOLOGIC SECTIONS ARE PROJECTED TO THE SECTION.
3. STRATIFICATION SHOWN IS AN INTERPRETATION OF THE BORINGS AND MAY NOT REPRESENT ACTUAL SUBSURFACE CONDITIONS.
4. SEE DRAWING NO. GS-R FOR GEOLOGIC SECTION SYMBOL DEFINITIONS.

GENERAL STRATA DESCRIPTIONS:

(F) **FILL** – LOOSE TO VERY COMPACT FINE TO COARSE SAND WITH VARYING AMOUNTS OF GRAVEL, SILT, CLAY, BRICK, CONCRETE, WOOD AND ROOTS

(A1) **ALLUVIAL CLAY/SILT** – SOFT TO MEDIUM CLAY AND ORGANIC CLAY WITH VARYING AMOUNTS OF SAND, SILT, SHELLS, AND ROOTS

(A2) **ALLUVIAL SAND** – LOOSE TO MEDIUM COMPACT SILTY FINE SAND AND CLAYEY FINE SAND WITH VARYING AMOUNTS OF MEDIUM TO COARSE SAND, SHELLS, AND GRAVEL

(T) **"OLD TOWN TERRACE"** – LOOSE TO MEDIUM COMPACT FINE TO COARSE SAND WITH VARYING AMOUNTS OF SILT, CLAY, AND GRAVEL

(TO) **"OLD TOWN TERRACE" (ORGANIC MEMBER)** – STIFF FINE SANDY SILT AND SILT, TRACE MEDIUM TO COARSE SAND AND DECOMPOSED WOOD

(P1) **POTOMAC CLAY** – STIFF TO HARD CLAY AND SILT WITH VARYING AMOUNTS OF SAND AND GRAVEL

REV.	DATE	BY	DESCRIPTION

CITY OF ALEXANDRIA WATERFRONT IMPLEMENTATION PROJECT PHASE II GEOTECHNICAL INVESTIGATION

ALEXANDRIA VIRGINIA

CAROLLO ENGINEERS, INC.

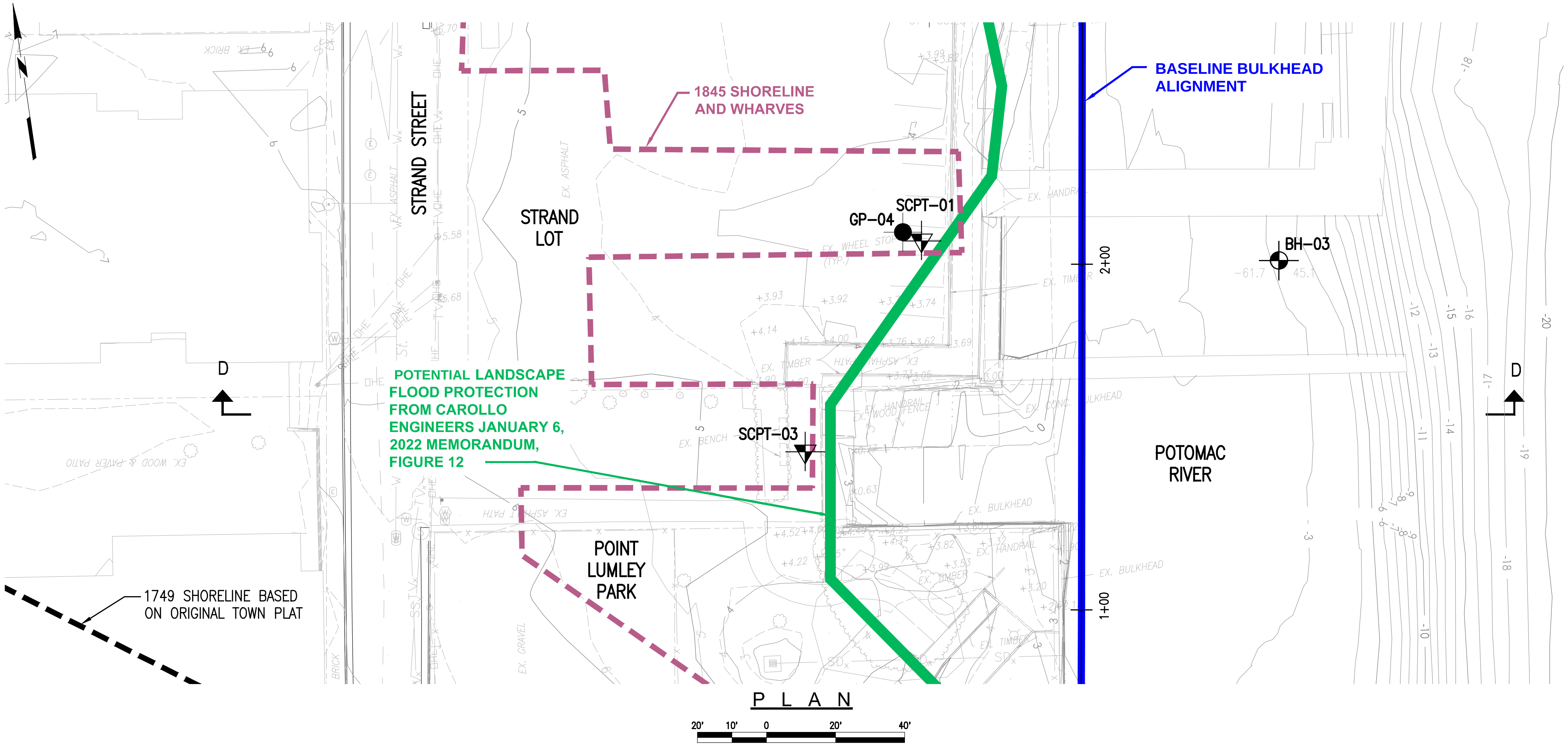
NEW YORK NEW YORK

MUESER RUTLEDGE CONSULTING ENGINEERS PLLC
14 PENN PLAZA – 225 WEST 34TH STREET, NEW YORK, NY 10122

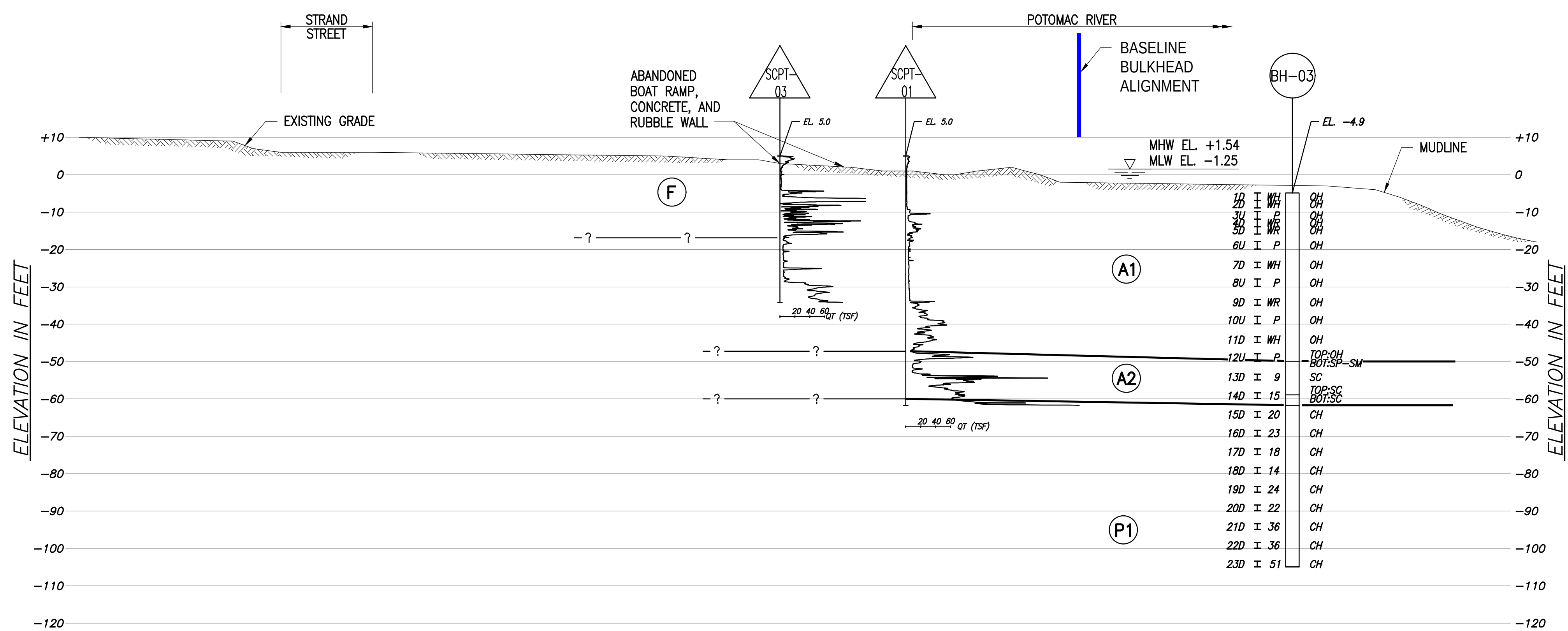
SCALE	MADE BY: L.R.	DATE: 09-15-2022	FILE NUMBER
GRAPHIC	CH'KD BY: A.L.S.	DATE: 09-15-2022	14123

PLAN AND GEOLOGIC SECTION C-C

DRAWING NUMBER GS-6



PLAN



SECTION D-D

NOTES:

1. SEE DRAWING NO. BLP-1 FOR GENERAL NOTES AND LEGEND.
2. BORINGS ILLUSTRATED ON GEOLOGIC SECTIONS ARE PROJECTED TO THE SECTION.
3. STRATIFICATION SHOWN IS AN INTERPRETATION OF THE BORINGS AND MAY NOT REPRESENT ACTUAL SUBSURFACE CONDITIONS.
4. SEE DRAWING NO. GS-R FOR GEOLOGIC SECTION SYMBOL DEFINITIONS.

GENERAL STRATA DESCRIPTIONS:

- (F) **FILL** – LOOSE TO VERY COMPACT FINE TO COARSE SAND WITH VARYING AMOUNTS OF GRAVEL, SILT, CLAY, BRICK, CONCRETE, WOOD AND ROOTS
- (A1) **ALLUVIAL CLAY/SILT** – SOFT TO MEDIUM CLAY AND ORGANIC CLAY WITH VARYING AMOUNTS OF SAND, SILT, SHELLS, AND ROOTS
- (A2) **ALLUVIAL SAND** – LOOSE TO MEDIUM COMPACT SILTY FINE SAND AND CLAYEY FINE SAND WITH VARYING AMOUNTS OF MEDIUM TO COARSE SAND, SHELLS, AND GRAVEL
- (T) **"OLD TOWN TERRACE"** – LOOSE TO MEDIUM COMPACT FINE TO COARSE SAND WITH VARYING AMOUNTS OF SILT, CLAY, AND GRAVEL
- (TO) **"OLD TOWN TERRACE" (ORGANIC MEMBER)** – STIFF FINE SANDY SILT AND SILT, TRACE MEDIUM TO COARSE SAND AND DECOMPOSED WOOD
- (P1) **POTOMAC CLAY** – STIFF TO HARD CLAY AND SILT WITH VARYING AMOUNTS OF SAND AND GRAVEL

REV.	DATE	BY	DESCRIPTION

CITY OF ALEXANDRIA WATERFRONT IMPLEMENTATION PROJECT PHASE II GEOTECHNICAL INVESTIGATION
ALEXANDRIA VIRGINIA

CAROLLO ENGINEERS, INC.
NEW YORK NEW YORK

MUESER RUTLEDGE CONSULTING ENGINEERS PLLC
14 PENN PLAZA – 225 WEST 34TH STREET, NEW YORK, NY 10122

SCALE	MADE BY: L.R.	DATE: 09-15-2022	FILE NUMBER
GRAPHIC	CH'KD BY: A.L.S.	DATE: 09-15-2022	14123

PLAN AND GEOLOGIC SECTION D-D

DRAWING NUMBER GS-7

UNIFIED SOIL CLASSIFICATION (INCLUDING IDENTIFICATION AND DESCRIPTION)

MAJOR DIVISIONS 1	MAJOR DIVISIONS 2		GROUP SYMBOLS 3	TYPICAL NAMES 4	FIELD IDENTIFICATION PROCEDURES (EXCLUDING PARTICLES LARGER THAN 3 IN. AND BASING FRACTIONS ON ESTIMATED WEIGHTS) 5
COARSE-GRAINED SOILS MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVELS MORE THAN HALF OF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE SIZE.	CLEAN GRAVELS (LITTLE OR NO FINES)	GW	WELL GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES.	WIDE RANGE IN GRAIN SIZES AND SUBSTANTIAL AMOUNTS OF ALL INTERMEDIATE PARTICLE SIZES.
			GP	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES.	PREDOMINANTLY ONE SIZE OR A RANGE OF SIZES WITH SOME INTERMEDIATE SIZES MISSING.
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)	GM	SILTY GRAVELS, GRAVEL-SAND-SILT-MIXTURES.	NONPLASTIC FINES OR FINES WITH LOW PLASTICITY (FOR IDENTIFICATION PROCEDURES SEE ML BELOW)
			GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES.	PLASTIC FINES (FOR IDENTIFICATION PROCEDURES SEE CL BELOW)
	SANDS MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE SIZE.	CLEAN SANDS (LITTLE OR NO FINES)	SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES.	WIDE RANGE IN GRAIN SIZES AND SUBSTANTIAL AMOUNTS OF ALL INTERMEDIATE PARTICLE SIZES.
			SP	POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES.	PREDOMINANTLY ONE SIZE OR A RANGE OF SIZES WITH SOME INTERMEDIATE SIZES MISSING.
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)	SM	SILTY SANDS, SAND-SILT-MIXTURES.	NONPLASTIC FINES OR FINES WITH LOW PLASTICITY (FOR IDENTIFICATION PROCEDURES SEE ML BELOW)
			SC	CLAYEY SANDS, SAND-CLAY MIXTURES.	PLASTIC FINES (FOR IDENTIFICATION PROCEDURES SEE CL BELOW)

(FOR VISUAL CLASSIFICATION, THE 1/4 -IN. SIZE MAY BE USED AS EQUIVALENT TO THE NO. 4 SIEVE SIZE)

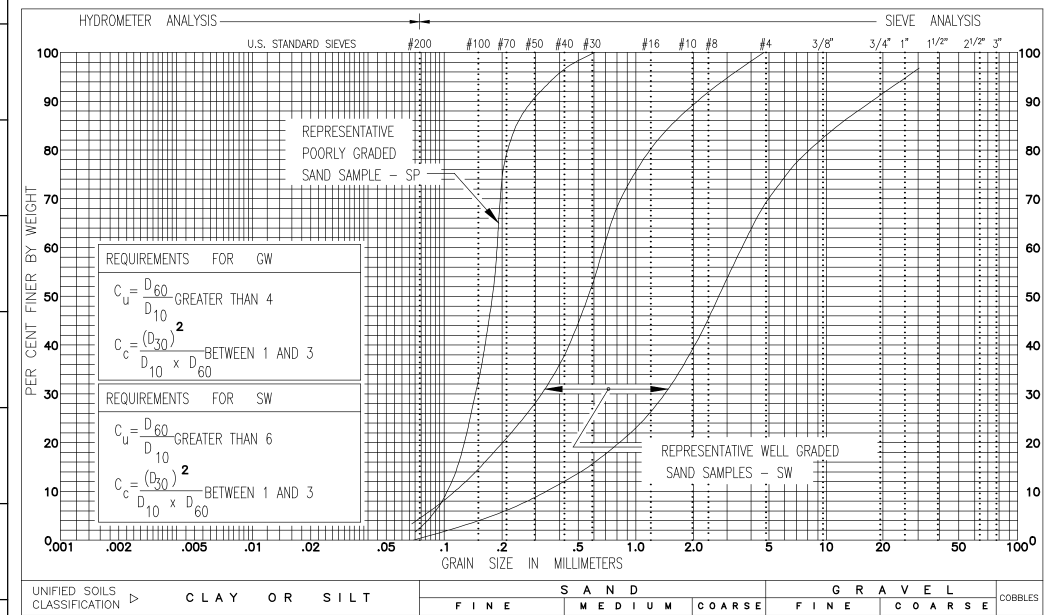
THE NO. 200 SIEVE SIZE IS ABOUT THE SMALLEST PARTICLE VISIBLE TO THE NAKED EYE

IDENTIFICATION PROCEDURES ON FRACTION SMALLER THAN NO. 40 SIEVE SIZE

MAJOR DIVISIONS 1	MAJOR DIVISIONS 2	GROUP SYMBOLS	TYPICAL NAMES	DRY STRENGTH (CRUSHING CHARACTERISTICS)	DILATANCY (REACTION TO SHAKING)	TOUGHNESS (CONSISTENCY NEAR PL)
FINE-GRAINED SOILS MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS LIQUID LIMIT IS LESS THAN 50	ML	INORGANIC SILTS, SANDY SILTS, ROCK FLOUR, OR CLAYEY SILTS WITH SLIGHT PLASTICITY.	NONE TO SLIGHT	QUICK TO SLOW	NONE
		CL	INORGANIC CLAYS, OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS.	MEDIUM TO HIGH	NONE TO VERY SLOW	MEDIUM
		OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY.	SLIGHT TO MEDIUM	SLOW	SLIGHT
	SILTS AND CLAYS LIQUID LIMIT IS GREATER THAN 50	MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS.	SLIGHT TO MEDIUM	SLOW TO NONE	SLIGHT TO MEDIUM
		CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS.	HIGH TO VERY HIGH	NONE	HIGH
		OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS.	MEDIUM TO HIGH	NONE TO VERY SLOW	SLIGHT TO MEDIUM
HIGHLY ORGANIC SOILS		Pt	PEAT AND OTHER HIGHLY ORGANIC SOILS.	READILY IDENTIFIED BY COLOR, ODOR, SPONGY FEEL AND FREQUENTLY BY FIBROUS TEXTURE.		

BOUNDARY CLASSIFICATIONS: SOILS POSSESSING CHARACTERISTICS OF TWO GROUPS ARE DESIGNATED BY COMBINATIONS OF GROUP SYMBOLS, I.E.: SP-SC POORLY GRADED SAND WITH CLAY BINDER.

LABORATORY CLASSIFICATION CRITERIA

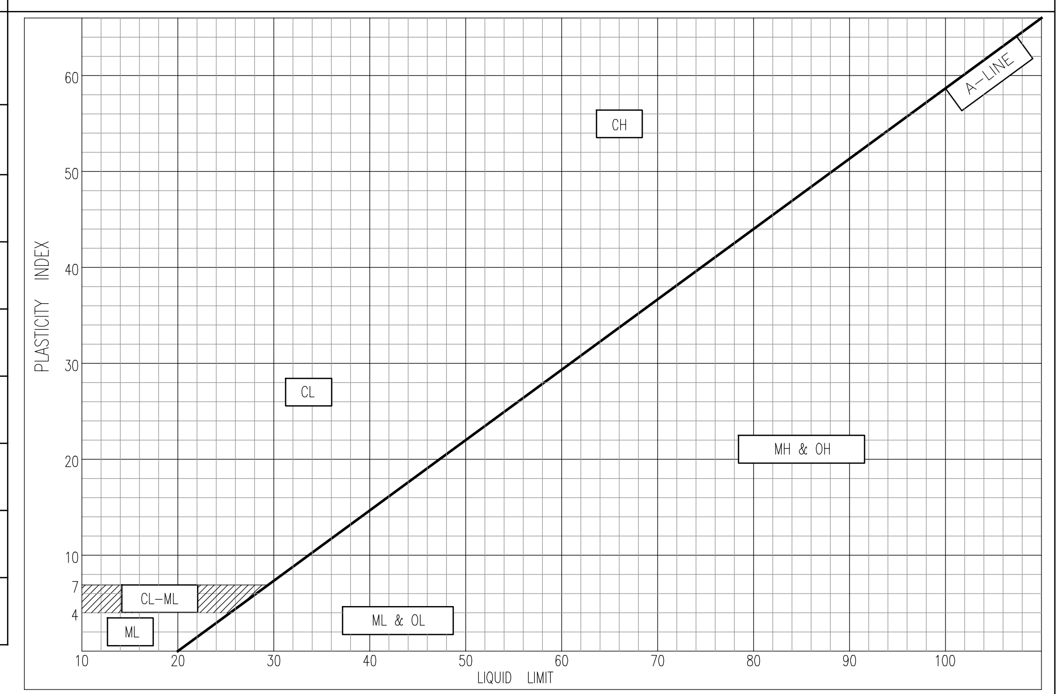


GRAIN SIZE PLOT

COBBLE 3-12"
BOULDER > 12"

DEPENDING ON PERCENTAGE OF FINES (FRACTION SMALLER THAN NO. 200 SIEVE SIZE) COARSE GRAINED SOILS ARE CLASSIFIED AS FOLLOWS:

LESS THAN 5%	GW, GP, SW, SP
MORE THAN 12%	GM, GC, SM, SC
5% TO 12%	BORDERLINE CASES REQUIRING USE OF DUAL SYMBOLS, I.E.: SP-SM, GP-GM.



PLASTICITY CHART FOR CLASSIFICATION OF FINE GRAINED SOILS

TERMINOLOGY USED IN MRCE SOIL DESCRIPTIONS

DEGREE OF COMPACTION FOR NON-PLASTIC SOIL

DEGREE OF COMPACTION	BLOWS* PER FOOT
LOOSE	0 TO 10
MEDIUM COMPACT	11 TO 29
COMPACT	30 TO 50
VERY COMPACT	GREATER THAN 50

* STANDARD PENETRATION RESISTANCE USING 140 LB. HAMMER FREE FALLING 30 INCHES TO DRIVE A 2 INCH O.D. SPLIT-SPOON SAMPLER.

CONSISTENCY OF CLAY AND CLAYEY SILT+

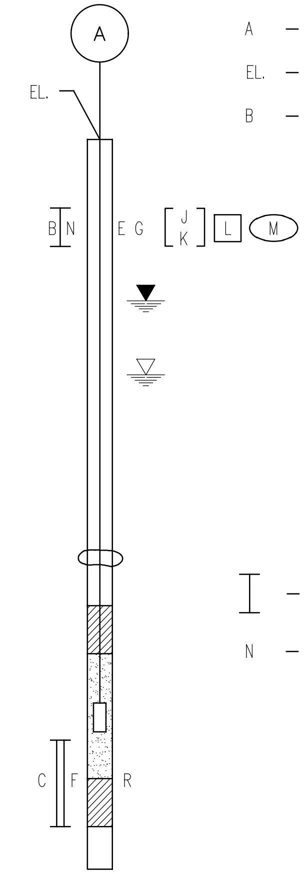
CONSISTENCY	UNCONFINED COMPRESSIVE STRENGTH (TSF)	IDENTIFICATION CHARACTERISTICS
SOFT	LESS THAN 0.5	EASILY REMOLDED WITH SLIGHT FINGER PRESSURE
MEDIUM	0.5 TO 1.0	REQUIRES SUBSTANTIAL PRESSURE FOR REMOLDING
STIFF	1.0 TO 4.0	DIFFICULT TO REMOLD WITH FINGERS
HARD	GREATER THAN 4.0	CANNOT BE REMOLDED WITH FINGERS

\+ NONPLASTIC SILTS ARE DESCRIBED USING DEGREE OF COMPACTION AS PRESENTED FOR NON-PLASTIC SOIL.

DESCRIPTION OF CONSTITUENT PERCENTAGES AS USED IN SOIL SAMPLE CLASSIFICATIONS

1% TO 12%	"TRACE"
13% TO 30%	"SOME"
31% TO 49%	ADJECTIVE FORM OF SOIL GROUP (EG. SANDY)
EQUAL AMOUNT	"AND" (EG. SAND AND GRAVEL)

BORING LEGEND



A — NUMBER, TYPE AND LOCATION OF BORING

EL. — GROUND SURFACE ELEVATION AT BORING

B — NUMBER AND TYPE OF SAMPLE

- HA – HAND AUGER SAMPLE
- D – DRY SAMPLE TAKEN WITH 2 INCH O.D. SPLIT SPOON
- U – UNDISTURBED SAMPLE TAKEN WITH 3 INCH O.D. FIXED PISTON TYPE SAMPLER
- UD – UNDISTURBED SAMPLE EXTRUDED IN FIELD AND PLACED IN JAR DUE TO POOR RECOVERY OR DISTURBANCE
- W – WASH SAMPLE
- S – THIN TUBE SAMPLE TAKEN WITH SHELBY TUBE SAMPLER
- N – THIN TUBE SAMPLE TAKEN WITH DENISON BARREL SAMPLER
- P – THIN TUBE SAMPLE TAKEN WITH PITCHER BARREL SAMPLER
- NR – NO RECOVERY

— LENGTH OF SAMPLE ATTEMPT

N — STANDARD PENETRATION RESISTANCE. NUMBER OF BLOWS FROM 140 LB. HAMMER FREE FALLING 30 INCHES REQUIRED TO DRIVE 2 INCH O.D. SPLIT SPOON SAMPLER ONE FOOT AFTER INITIAL PENETRATION OF 6 INCHES, UNLESS A SPECIFIC PENETRATION IS INDICATED.

- P – PRESSED OR PUSH SAMPLE
- WH – SAMPLE TAKEN UNDER WEIGHT OF HAMMER AND RODS
- WR – SAMPLE TAKEN UNDER WEIGHT OF RODS

E — AVERAGE NATURAL WATER CONTENT OF SAMPLE, IN PERCENT OF DRY WEIGHT

G — UNIFIED SOIL CLASSIFICATON GROUP SYMBOL OF SAMPLE

[J = ATTERBERG LIQUID LIMIT VALUE

K] = ATTERBERG PLASTIC LIMIT VALUE

L — COMPRESSIVE STRENGTH IN TSF DETERMINED FROM UNCONFINED COMPRESSION TEST

M — COMPRESSIVE STRENGTH IN TSF DETERMINED FROM UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST

— GROUNDWATER LEVEL OBSERVED IN BORING
*– MUD LEVEL

— GROUNDWATER LEVEL OBSERVED IN PIEZOMETER

C — ROCK CORE NUMBER

— LENGTH OF CORE RUN

F — LENGTH OF CORE RECOVERED EXPRESSED AS A PERCENT OF THE LENGTH OF CORE RUN

R — ROCK QUALITY DESIGNATION-THE SUM OF THE LENGTHS OF PIECES OF RECOVERED CORE WHICH ARE EQUAL TO OR GREATER THAN FOUR INCHES IN LENGTH, EXPRESSED AS A PERCENTAGE OF THE TOTAL LENGTH OF CORE RUN. LENGTHS ARE MEASURED BETWEEN IN-SITU SEPARATIONS AND MECHANICAL BREAKS RESULTING FROM CORING ARE IGNORED.

IMPERVIOUS SEAL

SAND FILTER SURROUNDING PIEZOMETER INTAKE ELEMENT

INTAKE ELEMENT

COBBLE OR BOULDER

REVISED 5-12-2020

MRCE built on firm foundations

MUESER RUTLEDGE CONSULTING ENGINEERS PLLC
14 PENN PLAZA – 225 WEST 34TH STREET, NEW YORK, NY 10122

GEOTECHNICAL REFERENCE STANDARDS

DRAWING NUMBER GS-R

APPENDIX A

January 10, 2022 Bulkhead Site Walk Findings

MRCE
built on firm foundations

PRINCIPALS
Peter W. Deming
Roderic A. Ellman, Jr.
Francis J. Arland
David R. Good
Walter E. Kaeck
Sitotaw Y. Fantaye
Jan Cermak

ASSOCIATE PRINCIPALS
Tony D. Canale
Anthony DeVito

SENIOR ASSOCIATES
Domenic D'Argenzio
Ketan H. Trivedi
Hiren J. Shah
Alice Arana
Joel L. Volterra
Steven R. Lowe
Andrew R. Tognon
James M. Tantalla
T. C. Michael Law
Andrew Pontecorvo
Renzo D. Verastegui
Srinivas Yenamandra
Jesse L. Richins

ASSOCIATES
Douglas W. Christie
Farid Vastani
Jong W. Choi
Raj S. Chinthamani
Andrew R. Klaetsch
Peter L. Madarasz
Aaron L. Sacks
Sung H. Kong
Fathey N. Elsaid
Colleen Liddy
Adam M. Dyer
Alessandro Ciamei
David Z. Chen
Matthew T. Ruehle

TECHNICAL SPECIALISTS
David M. Cacoilo
Alfred H. Brand
George J. Tamaro
Hugh S. Lacy
Joel Moskowitz
James L. Kaufman
Robert K. Radske

FINANCE DIRECTOR
Eric A. Hoffman

MARKETING DIRECTOR
Martha J. Huguet

February 14, 2022

Carollo Engineers Inc.
530 Seventh Avenue, Suite 2402
New York, New York, 10018

Attn: Sara Igielski
Lead Engineer

Re: January 10, 2022 Bulkhead Site Walk Findings
City of Alexandria Waterfront Implementation Project
Alexandria, Virginia
MRCE File 14123

Dear Ms. Igielski,

MRCE performed a one-day site walk of the project area waterfront on January 10, 2022. The walk was attended by Raj Chinthamani and Aaron Sacks of MRCE and representatives of Carollo Engineers and the Project Implementation Team of the City of Alexandria. The objective of the site walk was to observe visible bulkhead elements and discuss potential options for improvements to address the City's goals of improved flood mitigation and enhancement of the waterfront area. The walk covered the waterfront between Duke Street to the south and Queen Street to the north.

This report summarizes observations made at the time of our site visit and presents our evaluation of waterfront improvement options that meet project objectives and offer potential savings in project cost either by alternate bulkhead design or by use of landscaping solutions for flood control where existing waterfront conditions are favorable in the near term. We also provide our recommendations for next steps in assessment of these potentially more cost effective options.

Exhibits

The following exhibits are attached to this report:

Figure 1	Site Plan showing Bulkhead/Waterfront Sections
Table 1	Recommended Options for Further Consideration
Appendix	Bulkhead Observations and Options (Sections 1 to 11)

Basis of Site Observations and Option Evaluation

The primary objective of the waterfront improvements is to improve flood protection to EL. +6. Dredging outboard of the bulkhead is also planned to improve waterfront access. These inboard and outboard changes will add load to the inland side of the bulkhead and remove resistance on the waterside and are therefore primary design considerations in the evaluation of any alternative bulkhead option.

Alternative options for bulkhead improvements were evaluated with respect to the Bulkhead Design Manual and associated bulkhead design prepared by Moffatt and Nichol (2017) and the VE alternatives developed by Carollo Engineers and The Olin Studio (Carollo, 1/6/22). We also reviewed the 2013 Michael Baker Jr., Inc. structural condition report of the existing piers and bulkhead at the City Dock and Marina.

Bulkhead Observations

For the purposes of cataloging our observations and discussion of alternative options herein, we divided the bulkhead into 11 sections (Nos. 1 through 11) primarily based on bulkhead type as shown on Figure 1 starting with Section 1 at the north end. Figure 1 shows the section numbering and approximate section limits.

The site walk commenced from the foot of Duke Street at the southern limit of the project waterfront (Section 11) and progressed northward. Observations focused on the top elevation of the bulkhead relative to EL. +6, type of bulkhead, bulkhead condition and imminent need for repairs where observed, land use behind the bulkhead, and proximity of existing structures. Representative photographs and a summary of observed conditions at each bulkhead section are summarized in the Appendix.

Bulkhead Improvement Options

Potential repair options for achieving the project objectives at each bulkhead section accompany the summary of observed conditions in the Appendix**.** Options are grouped broadly into two categories based primarily on observed bulkhead condition:

1. **Continued Use Options** where the remaining service life of the existing bulkhead is significant (i.e. 10 years or more) or can be prolonged by localized repairs and upland area is available for landscaping to raise inland grades for flood protection. Landscaping options improve flood protection by raising inland grades to EL. +6 using sloping terrain, or low height retaining walls ('Ha-Ha' walls) or other architectural features as detailed in the January 2022 Carollo Assessment Report. With these continued use options, replacement of the existing bulkhead with a new bulkhead is deferred to the future. Landscaping options appear viable at Bulkhead Sections 6, 7, 8 and 11.

   The existing bulkhead segments at Sections 3, 4 and 5 also appear to have significant service life remaining. However, landscaping options are not viable for flood protection to EL. +6 at these sections owing to the proximity of existing structures and limited access due to the presence of marinas. As a result, we recommend consideration of deployable barriers for flood protection at Sections 3 and 4 to match the elevations at the ends with adjacent section flood proofing measures. Deployable barriers are not necessary at Section 5 since the top of the bulkhead is at EL. +7 and above the project flood control objective. Continued use of the existing bulkhead at Section 5 is expected

Since all of these continued use options rely on the capacity of the existing bulkhead for a significant future period (10 or more years), a more detailed bulkhead condition assessment is required to provide a quantitative estimate of remaining service life, need for local repairs, and confirm option viability. This more detailed condition assessment would include: detailed water-side bulkhead inspection, diver survey, ultra-sound / pulse echo / GPR testing to determine thickness and depth of steel elements, concrete strength / soundness testing, test pits to evaluate existing anchorages, etc.

2. **Replacement Options** where the existing bulkhead is failed or severely deteriorated with limited remaining service life. Bulkhead replacement appears necessary in the relatively near future at Sections 1 and 2 at the north end and 9 and 10 at the south end. Section 1 could potentially change to a landscaping solution if the project objectives are relaxed by limiting upland land use (the area remains a park) and deferring dredging. In this case, rehabilitation of the existing stone and timber crib would consist of infilling of existing voids and shotcreting of the bulkhead face to prolong the service life. (Note however that a new bulkhead in Sections 1 and 2 is necessary to accommodate the landfill for the Thompson's Alley pump station location considered in the Master Stormwater Management Plan.)

   In these sections, we recommend evaluation of the following more cost-effective replacement alternatives relative to the current bulkhead design:

   a. New cantilever (masterpile or interlocked pipe pile) bulkhead outboard of the existing bulkhead
   b. New anchored bulkhead outboard of the existing bulkhead, with pile-supported (A-frame) deadman

It is important to note that flood protection will only be effective if the elevations of flood protection between various sections and along the entire waterfront seamlessly provide a continuous barrier up to the desired elevation (EL. +6). In addition, modifications to the storm drainage system to prevent backflow through existing outfalls is a required component of the flood protection strategy.

Summary and Next Steps

Our assessment of potential bulkhead improvements to achieve flood control objectives and provide more cost effective bulkhead design are listed below and summarized in more detail at each bulkhead section on Table 1. Ballpark cost estimates for the bulkhead options recommended for further consideration are also provided in Table 1. Cost estimates are intended for concept planning purposes only and do not include dredging, demolition, excavation, spoils handling / disposal, clearing obstructions, fill import and placement, deployable barriers, mobilization and demobilization, contingency or any unforseen conditions encountered during construction.

1. Continued use of the existing bulkhead with landscaping solutions for flood protection where the remaining service life of the existing bulkhead is significant, including:

   a. Raise grade inland of the bulkhead as proposed in the January 2022 Carollo/Olin Report
   b. at Sections 6, 7, 8 and 11.

c. Addition of deployable flood barriers on top of the existing bulkhead at Sections 3 and 4 where existing structures restrict access for upland improvements.
d. Bulkhead repairs at Section 5 where the existing bulkhead is already at EL. +7.

Since these continued use solutions rely on the capacity of the existing bulkhead, these sections will require further investigation (as described above) to confirm that the remaining bulkhead service life is significant and may need additional localized repairs to prolong service life. Localized repairs may consist of adding welded steel plates in areas of localized corrosion and application of protective coating in the tidal/splash zone at steel sheet pile segments and concrete repairs or shotcreting at concrete bulkhead segments.

2. Bulkhead replacement at Sections 1, 2, 9 and 10. More cost-effective alternatives relative to the current bulkhead design are anticipated to consist of:

   a. New cantilever (masterpile or interlocked pipe pile) bulkhead outboard of the existing bulkhead
   b. New anchored bulkhead outboard of the existing bulkhead, with pile-supported (A-frame) deadman

   We recommend that these replacement alternatives with the benefit of the Phase II geotechnical investigations become the focus of analysis of potential bulkhead stabilization and flood improvement options in the Geotechnical Design Memorandum (GDM), if authorized.

Limitations

The observations and potential repair options presented herein are based only on a one-day visual inspection performed from land, preliminary discussions with Carollo and the City's Project Implementation team, and review of previous reports prepared by others. MRCE has not performed any physical measurements to verify existing bulkhead conditions and remaining service life.

Please feel free to contact us if you have questions.

Very truly yours,
MUESER RUTLEDGE CONSULTING ENGINEERS PLLC

Rajnikanth CT

Raj S. Chinthamani

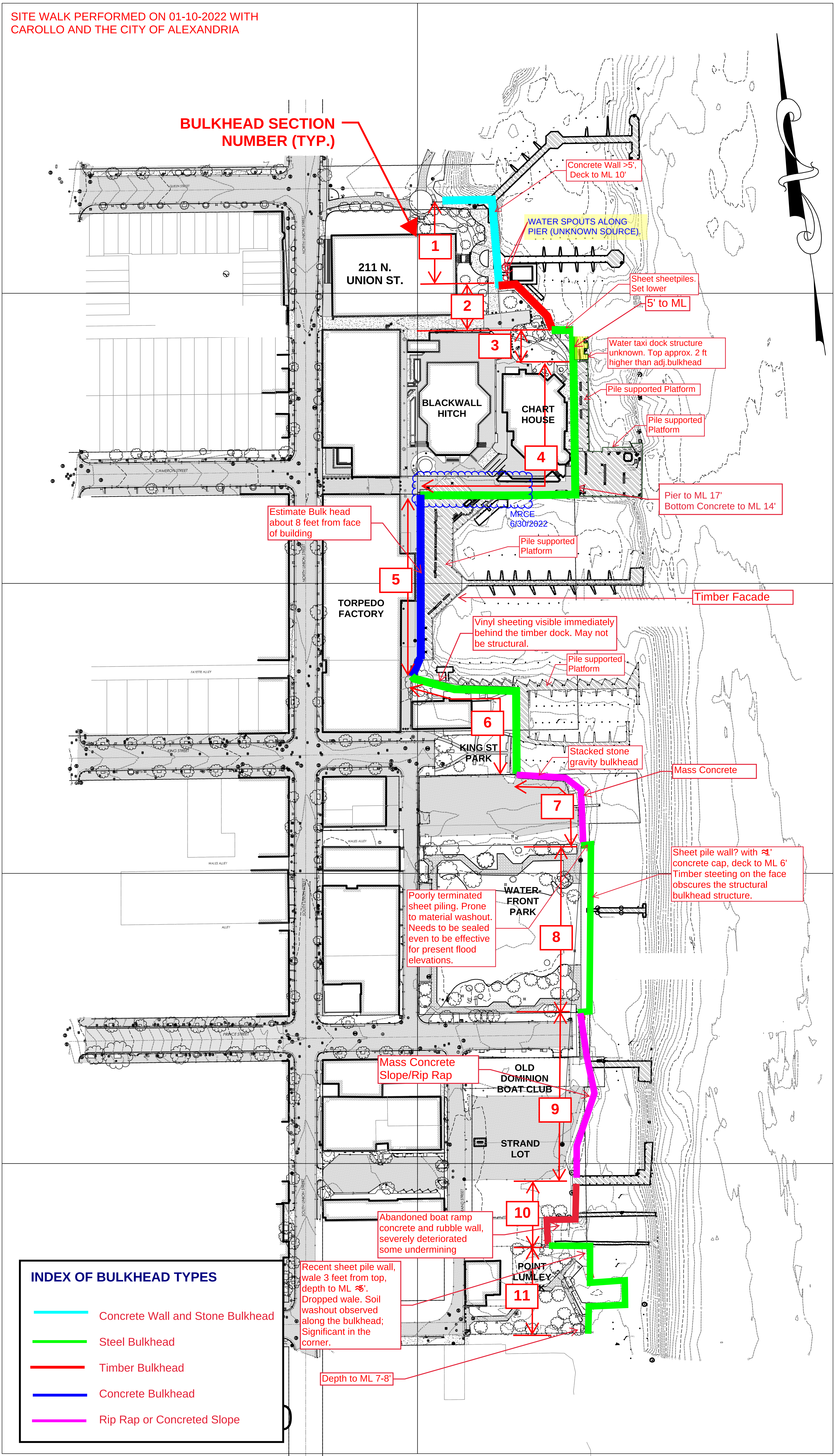
Walter E. Kaeck

Walter E. Kaeck, Principal

RC:ALS:WEK:F:\141\14123\Bulkhead Site Walk Jan 2022\Jan22 Sitewalk_MRCE Letter_fnl.docx

EXHIBITS

ALEXANDRIA WATERFRONT, SITE PLAN



MRCE | ALEXANDRIA, VA. | 2/14/2022

WATERFRONT REPAIR / REPLACEMENT OPTIONS

TABLE 1

SECTION	APPROX. LENGTH (LF)	RECOMMENDED OPTION FOR FURTHER CONSIDERATION		APPROXIMATE COST (RANGE) CLASS 4 ESTIMATE
SECTION 1	225	BULKHEAD REPLACEMENT	NEW MASTERPILE WALL SET SET TO EL. +6 MIN. FLOW FILL THE ANNULUS. RAISE GRADE INBOARD	$9,000 / LF
SECTION 2	115	BULKHEAD REPLACEMENT	NEW MASTERPILE WALL SET SET TO EL. +6 MIN. FLOW FILL THE ANNULUS. RAISE GRADE INBOARD	$9,000 / LF
SECTION 3	80	CONTINUED USE OF EXISTING BULKHEAD	REPAIR EXISTING BULKHEAD. INSTALL DEPLOYABLE BARRIERS FOR FLOOD RESILIENCY UP TO EL. +6 MIN.	$2,000 / LF
SECTION 4	MRCE 6/30/2022 ~~240~~ 430	CONTINUED USE OF EXISTING BULKHEAD	REPAIR EXISTING BULKHEAD. INSTALL DEPLOYABLE BARRIERS FOR FLOOD RESILIENCY UP TO EL. +6 MIN.	$2,000 / LF
SECTION 5	~~470~~ 280	CONTINUED USE OF EXISTING BULKHEAD	REMOVE CLADDING. REPAIR BULKHEAD AS NEEDED WITH REINFORCED CONCRETE FASCIA. REPLACE TIMBER CLADDING AND SCREENS	$4,000/ LF
SECTION 6	300	CONTINUED USE OF EXISTING BULKHEAD	REPAIR EXISTING BULKHEAD AS NEEDED. RAISE GRADES UPLAND TO EL. +6 MIN BY MEANS OF LANDSCAPING AND LOW HEIGHT HA-HA WALLS.	$1,500 / LF
SECTION 7	220	CONTINUED USE OF EXISTING BULKHEAD	INSTALL LOW HEIGHT RETAINING WALL AND LANDSCAPING TO CONNECT ADJACENT SECTIONS AND PROVIDE FLOOD RESILIENCY UP TO EL. +6 MIN.	$1,000 / LF
SECTION 8	285	CONTINUED USE OF EXISTING BULKHEAD	REPAIR EXISTING BULKHEAD. RAISE GRADES UPLAND TO EL. +6 MIN BY MEANS OF LANDSCAPING AND LOW HEIGHT HA-HA WALLS.	$1,000 / LF
SECTION 9	255	BULKHEAD REPLACEMENT	NEW BULKHEAD - INTERLOCKED PIPE PILE / MASTERPILE WALL OR RIPRAP REVETMENT SET TO EL. +6 MIN. FILL INBOARD. RAISE GRADES TO EL. +6 MINIMUM.	$10,000 / LF
SECTION 10	145	BULKHEAD REPLACEMENT	NEW BULKHEAD - INTERLOCKED PIPE PILE / MASTERPILE WALL OR RIPRAP REVETMENT SET TO EL. +6 MIN. FILL INBOARD. RAISE GRADES TO EL. +6 MINIMUM.	$10,000 / LF
SECTION 11	285	CONTINUED USE OF EXISTING BULKHEAD	REPAIRS TO BULKHEAD. RAISE GRADES UPLAND TO EL. +6 MIN BY MEANS OF LANDSCAPING AND LOW HEIGHT HA-HA WALLS.	$1,500 / LF

NOTES:

1. FOR ALL OPTIONS THAT UTILIZE THE EXISTING BULKHEAD, A BULKHEAD CONDITION ASSESSMENT NEEDS TO BE PERFORMED IN ORDER TO BETTER EVALUATE THE REPAIR / REPLACEMENT NECESSITY AND HENCE THE COST ESTIMATES.

2. COST ESTIMATES ARE INTENDED FOR CONCEPT PLANNING PURPOSES ONLY AND DO NOT INCLUDE DREDGING, DEMOLITION, EXCAVATION, SPOILS HANDLING / DISPOSAL, CLEARING OBSTRUCTIONS, FILL IMPORT AND PLACEMENT, DEPLOYABLE BARRIERS OR ANY UNFORSEEN CONDITIONS ENCOUNTERED DURING CONSTRUCTION.

3. COST ESTIMATES DO NOT INCLUDE MOBILIZATION AND DEMOBILIZATION, PERMITTING AND CONTINGENCY.

APPENDIX

Note: Refer to attached Site Plan for delineation of bulkhead sections

SECTION 1

GENERAL OBSERVATIONS

- Bulkhead consists of a concrete retaining structure over a timber and stone crib wall.
- The condition of the retaining wall is poor and will need replacement in the near term with a new bulkhead installed outboard of the existing bulkhead.
- Upland space is available for raising grades gradually if required.
- Several water spouts (see photograph above) were observed along the timber pier north of the house barge. The source of the spouts is unknown. These spouts, if resulting from a damaged pipeline exiting the bulkhead, could cause significant material washout if continued over time.

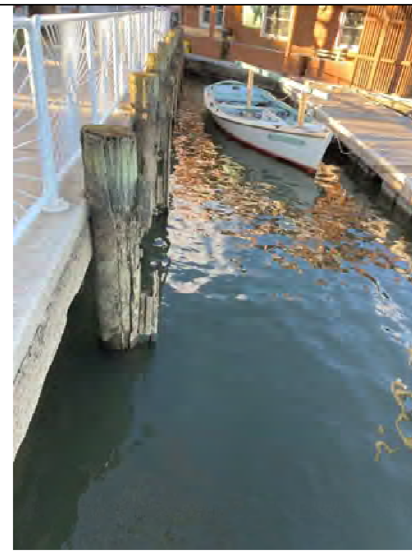
RECOMMENDED OPTION FOR FURTHER CONSIDERATION

- Due to the deteriorated condition of bulkhead in this section, a new bulkhead will be required to support raised grades to EL. +6, and protect inland space against projected flood water levels and accommodate continued use of esplanade.
- Potential New Bulkhead options: Interlocked pipe pile wall or a Masterpile wall with light weight fill placed inboard of the bulkhead to raise grades. A-frame deadman or a concrete deadman could be installed inboard of the existing bulkhead to offer anchorage to the new bulkhead.
- However, if the bulkhead piles are sized adequately, the bulkhead could be designed as a cantilever system.

POTENTIAL CONSTRAINTS AND CONSIDERATIONS

- Insufficient water depth for barges. May need dredging first or use of low draft floats to mobilize from waterside.
- Bulkhead does not appear to have capacity currently to support construction surcharge for landside mobilization and bulkhead installation
- Use of the marina will be affected for the duration of construction

SECTION 2



GENERAL OBSERVATIONS

- Timber sheeting observed along the bulkhead.
- It is not clear if the timber boards are installed in front of an older bulkhead-crib wall or a low level relieving platform.
- Structural capacity of the bulkhead is unknown.

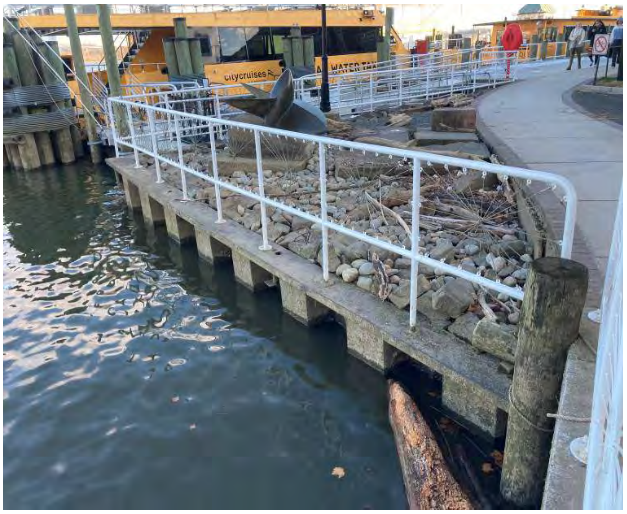
RECOMMENDED OPTION FOR FURTHER CONSIDERATION

- Install new bulkhead to support raised grades and protect against projected flood water levels.
- Though upland area is available for landscaping option to raise grades, extending the new bulkhead from section 1 to cover this section will be beneficial to raise grades upland to EL. +6.
- Potential new bulkhead options: Interlocked pipe pile wall or a Masterpile wall with light weight fill placed inboard of the bulkhead to raise grades. A-frame deadman or a concrete deadman could be installed inboard of the existing bulkhead to offer anchorage to the new bulkhead.
- However, if the bulkhead piles are sized adequately, the bulkhead could be designed as a cantilever system.

POTENTIAL CONSTRAINTS AND CONSIDERATIONS

- Water depths along this section may be insufficient for construction barges. May need dredging first or use of low draft floats to mobilize from waterside.
- Bulkhead does not appear to have capacity to support construction surcharge for landside mobilization.
- Existing piers and house barge will need to be removed for construction.

SECTION 3



GENERAL OBSERVATIONS

- Steel Sheetpile bulkhead. Large holes were visible in the sheets indicating probable significant material loss over time.
- The water taxi dock structure needs to be verified for anchorage including any guide piles and their proximity to the bulkhead structure.
- Top elevation of the bulkhead and the esplanade are set low.
- The Steel sheetpile bulkhead continues below the timber pile supported platform for most of this segment.

RECOMMENDED OPTION FOR FURTHER CONSIDERATION

- If general bulkhead condition is confirmed to have sufficient remaining service life, repair sheets with welded plates and coating, raise grade with low height retaining walls such as Ha-Ha walls and light weight fill set back from bulkhead.
- Install deployable flood barriers to match elevations at the northern interface with section 2 and southern end (pending bulkhead condition assessment) for flood resiliency up to EL. +6 in the near term.

POTENTIAL CONSTRAINTS AND CONSIDERATIONS

- Bulkhead does not appear to have capacity to support construction surcharge for landside mobilization.
- The water taxi dock will need to be relocated prior to commencement of repairs.

SECTION 4

GENERAL OBSERVATIONS

- Appears to be steel sheetpile bulkhead. Visible only at a gap between timber cladding along the marina. To be confirmed by future inspection and condition assessment.
- Top elevation of esplanade lower along the eastern face of Chart House building.
- Due to proximity of existing structures behind the bulkhead and openings at platform level, raising grade or bulkhead will be impossible without losing access to building from platform level.

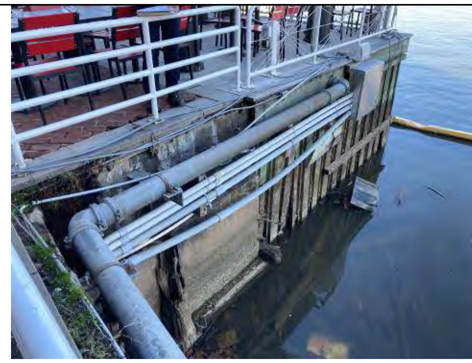
RECOMMENDED OPTION FOR FURTHER CONSIDERATION

- If bulkhead condition is confirmed to have reasonable remaining service life and capacity, repair any holes in the steel sheetpiles.
- Install deployable flood barriers to match elevations at the southern end.

POTENTIAL CONSTRAINTS AND CONSIDERATIONS

- Water depth may be sufficient in this portion to accommodate construction barges but existing Marina operations will be affected during construction.
- Area inboard of the bulkhead is insufficient to support construction activities and surcharge from landside mobilization.
- Access to bulkhead for repairs will be limited due to presence of timber pile supported platform outboard of the bulkhead.

SECTION 5



GENERAL OBSERVATIONS

- Probable concrete over timber cribbing structure. Not visible over most of segment due to outboard pile supported platform and timber sheet cladding / screen. Top elevation set higher, EL. +7 (Per the available topographical survey), than most of the waterfront.
- Minimal offset observed between the bulkhead and the Torpedo Factory building.
- Concrete bulkhead exposed at the southern end of this section is in poor condition. Timber crib wall underneath the concrete cap is severely deteriorated and is allowing material loss.

RECOMMENDED OPTION FOR FURTHER CONSIDERATION

- If the bulkhead condition is found to in same the condition as the southern end, repair in place with shotcrete or formed concrete repairs. Timber fascia may need replacement.

POTENTIAL CONSTRAINTS AND CONSIDERATIONS

- Access to bulkhead for repairs will be limited due to presence of timber pile supported platform outboard of the bulkhead.
- Existing Marina operations will be affected during construction;
- Area inboard of the existing bulkhead is insufficient to support construction activities.

SECTION 6

GENERAL OBSERVATIONS

- Steel sheet pile with steel channel cap visible along the eastern edge (to the south of the existing docks). Appears to be in fair condition
- It is unclear if the vinyl sheeting along the northern edge (running east-west) is a structural bulkhead.
- Top of bulkhead elevation is relatively low and the existing grades rise away from the bulkhead but are still below EL. +6.

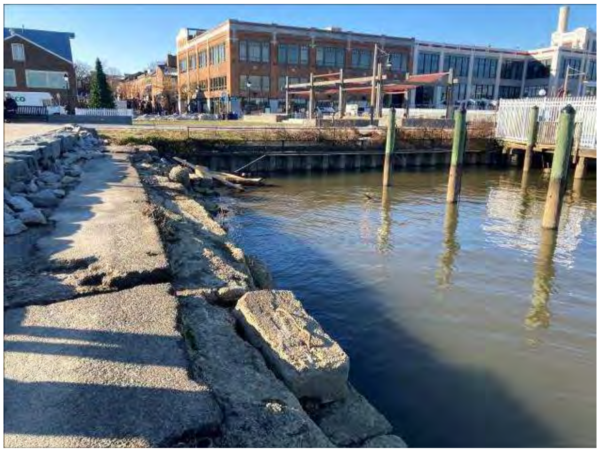
RECOMMENDED OPTION FOR FURTHER CONSIDERATION

- The bulkhead appears to be in fair condition from the visual inspection of the portion above water. If the service life of this section of bulkhead can be extended to 25 years with maintenance repairs such as welded steel plates and coating, raising inland grades by means of landscaping is possible.
- The availability of space inland of the bulkhead offers the opportunity to raise grades to a minimum of EL. +6 using sustainable slopes or low level retaining walls like Ha-ha walls.

POTENTIAL CONSTRAINTS AND CONSIDERATIONS

- Area inboard of the existing bulkhead could be available for construction staging. Surcharge loading will need to be analyzed.
- Existing Marina operations will be affected during construction.

SECTION 7



GENERAL OBSERVATIONS

- Stacked stone gravity bulkhead. Top of bulkhead elevation is relatively low – approximately EL +3.
- Water depths outboard of the bulkhead appear to be shallow, including at the slips in the marina.
- The eastern edge of this section is comprised of an uncontrolled mass concrete pour bulkhead in the northern portion and dumped riprap in the southern portion.
- The edge of this section of waterfront is very uneven.

RECOMMENDED OPTION FOR FURTHER CONSIDERATION

- The stacked stone gravity bulkhead appears to be a fix for a previously built bulkhead. It will be difficult to evaluate the capacity of the bulkhead without understanding the full cross section.
- Due to availability of space inboard of the bulkhead, landscaping solution could be adopted to raise grades using low height retaining walls such as Ha-Ha walls and regrading to EL. +6 minimum to improve flood resiliency.

POTENTIAL CONSTRAINTS AND CONSIDERATIONS

- Existing Marina operations will be affected during construction.
- Landscaping / Ha-Ha wall will need tie-in to the grade change in sections 6 and 8 to provide a continuous protection up to EL. +6

SECTION 8

GENERAL OBSERVATIONS

- Timber facing over what appears to be a steel sheet pile bulkhead with concrete cap.
- Top of bulkhead is set at low elevation of approximately EL. +3. Mooring bollards are installed along the edge and appear to be in fair condition.

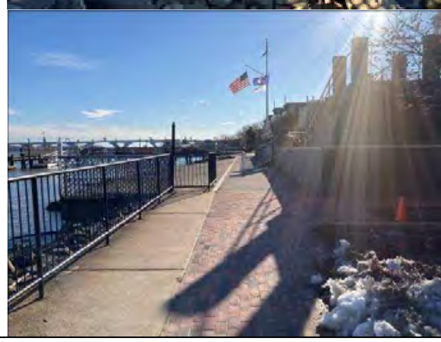
RECOMMENDED OPTION FOR FURTHER CONSIDERATION

- If the steel bulkhead is determined to have sufficient service life, perform any necessary repairs with welded plates and coating.
- Since the upland area is available for offset and regrading, landscaping solution could be adopted to raise grades to EL. + 6 using low height retaining walls such as Ha-Ha walls and regrading to provide flood resiliency up to EL. +6.

POTENTIAL CONSTRAINTS AND CONSIDERATIONS

- Capacity of the dock to support construction surcharge loading is unknown.
- Landscaping / Ha-Ha wall will need tie-in to the grade change in sections 7 and 9 to provide a continuous protection up to EL. +6

SECTION 9



GENERAL OBSERVATIONS

- Uncontrolled riprap in the northern portion probably placed to shore up a damaged bulkhead.
- Mass concrete edge in the middle portion. Stacked stone gravity bulkhead section in southern portion (underlying structure unknown)
- Old Dominion boathouse parking lot and the first floor slab appears to be set at EL +6.

RECOMMENDED OPTIONS FOR FURTHER CONSIDERATION

- Option 1 -New bulkhead along the new bulkhead line with top elevation at EL. +6 minimum. Tied back sheetpile bulkhead with steel pile A-frame deadman east of existing bulkhead or cantilever interlocked pipe pile wall bulkhead capable of supporting. Backfill up inboard of new bulkhead.
- Option 2 - Riprap revetment built up to required elevation and with toe at the new bulkhead line. Alternatively, the revetment can be installed against existing bulkhead and well within the new bulkhead line – used less fill but creates to less available esplanade area relative to option 1.

POTENTIAL CONSTRAINTS AND CONSIDERATIONS

- Existing bulkhead is uneven in plan.
- Repair options limited due to minimal upland space and proximity to structures.
- Water depth may not be sufficient for construction from waterside. May need dredging first or use of low draft floats to mobilize from waterside.
- Upland area is very limited to support construction staging.
- Installing piles for A-frame anchorage could be challenging due to presence of mass concrete.

SECTION 10

GENERAL OBSERVATIONS

- Bulkhead appears to be concrete cap over timber cribbing. Concrete cap is severely deteriorated; will need replacement or shoring in the near term to prevent loss of material.
- Concrete boat ramp severely deteriorated.

RECOMMENDED OPTIONS FOR FURTHER CONSIDERATION

- Demolish and remove existing boat ramp.
- Install new bulkhead along the new bulkhead line set to a min. EL. +6. Install backfill and raise grades inboard to EL +6 minimum.
- New Bulkhead Options:

  Option 1 -New bulkhead along the new bulkhead line with top elevation at EL. +6 minimum. Tied back sheetpile bulkhead with steel pile A-frame deadman east of existing bulkhead or cantilever interlocked pipe pile wall bulkhead capable of supporting. Backfill up inboard of new bulkhead.

  Option 2 - Riprap revetment built up to required elevation and with toe at the new bulkhead line. Alternatively, the revetment can be installed against existing bulkhead and well within the new bulkhead line – used less fill but creates to less available esplanade area relative to option 1.

POTENTIAL CONSTRAINTS AND CONSIDERATIONS

- Water depth may insufficient for construction from waterside.
- Bulkhead does not have capacity to support construction surcharge.
- If built along new bulkhead line east of existing bulkhead, more options and area available for use and offsetting from floods.

SECTION 11

GENERAL OBSERVATIONS

- Steel sheet pile bulkhead with steel cap.
- Top of bulkhead elevation is relatively low and the existing grades rise away from the bulkhead but are still below EL. +6.
- Material loss was observed at several locations of along the bulkhead. Gaps and holes were observed in the bulkhead, which is the most likely cause of material washout and ground subsidence. Dislocated wale and missing tie rod / hardware 1 location.

RECOMMENDED OPTIONS FOR FURTHER CONSIDERATION

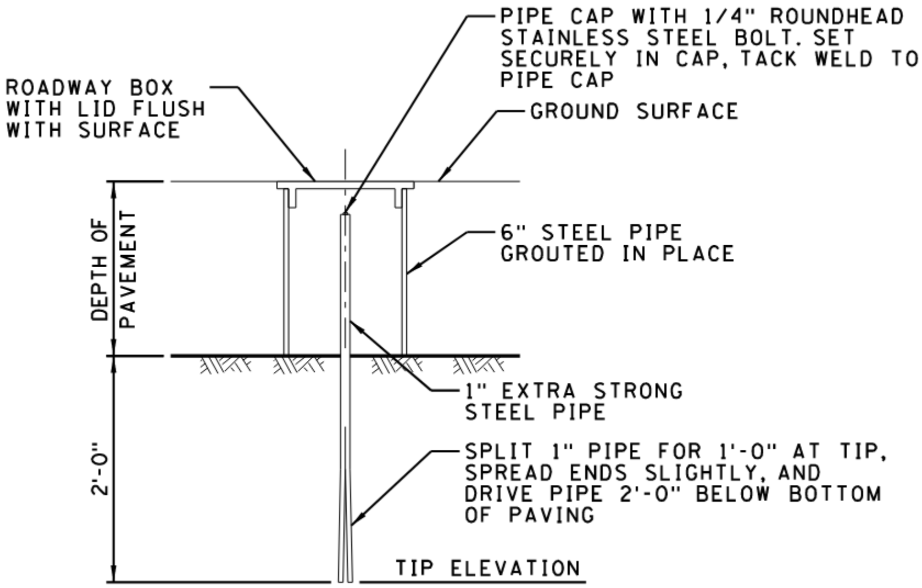
- Repair any openings or gaps in the bulkhead with welded plates, repair steel wales and anchorage where disengaged and install coating as required.
- Landscape upland area to raise grades to EL. +6 min. using low height retaining walls such as Ha-Ha walls as needed.

POTENTIAL CONSTRAINTS

- The existing steel sheet pile bulkhead may not have sufficient capacity to support construction surcharge.

APPENDIX B

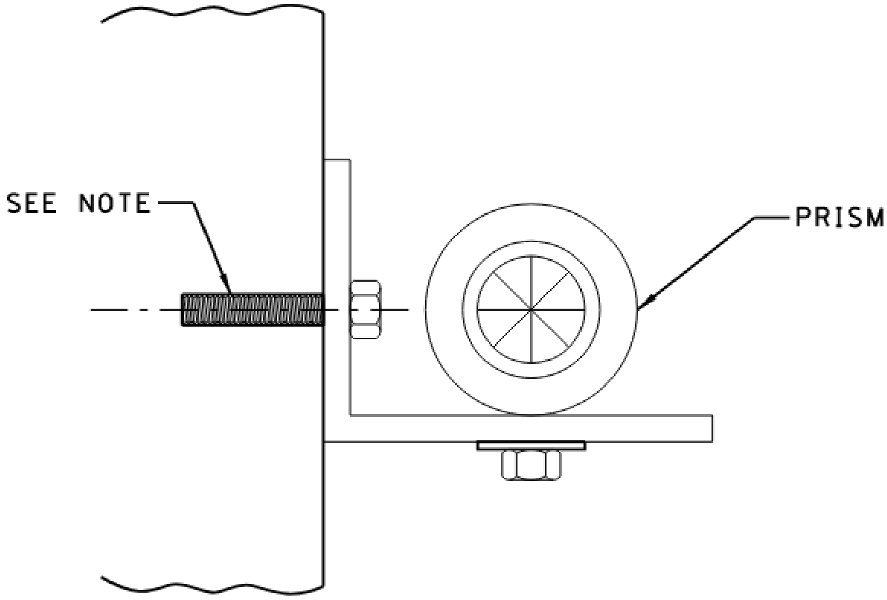
Typical Monitoring Instrumentation Details



TYPICAL SURFACE SETTLEMENT MONITORING POINT DETAIL

Note: Typical detail provided for conceptual planning purposes only.

CITY OF ALEXANDRIA WATERFRONT IMPLEMENTATION PROJECT ALEXANDRIA VIRGINIA	
GROUND SURFACE SETTLEMENT MONITORING POINT	FIGURE NO. **B1**



NOTES:

1. ANCHOR EITHER INTO SOUND CONCRETE USING EXPANSION ANCHORS OR ONTO STEEL MEMBERS USING CLAMPS.

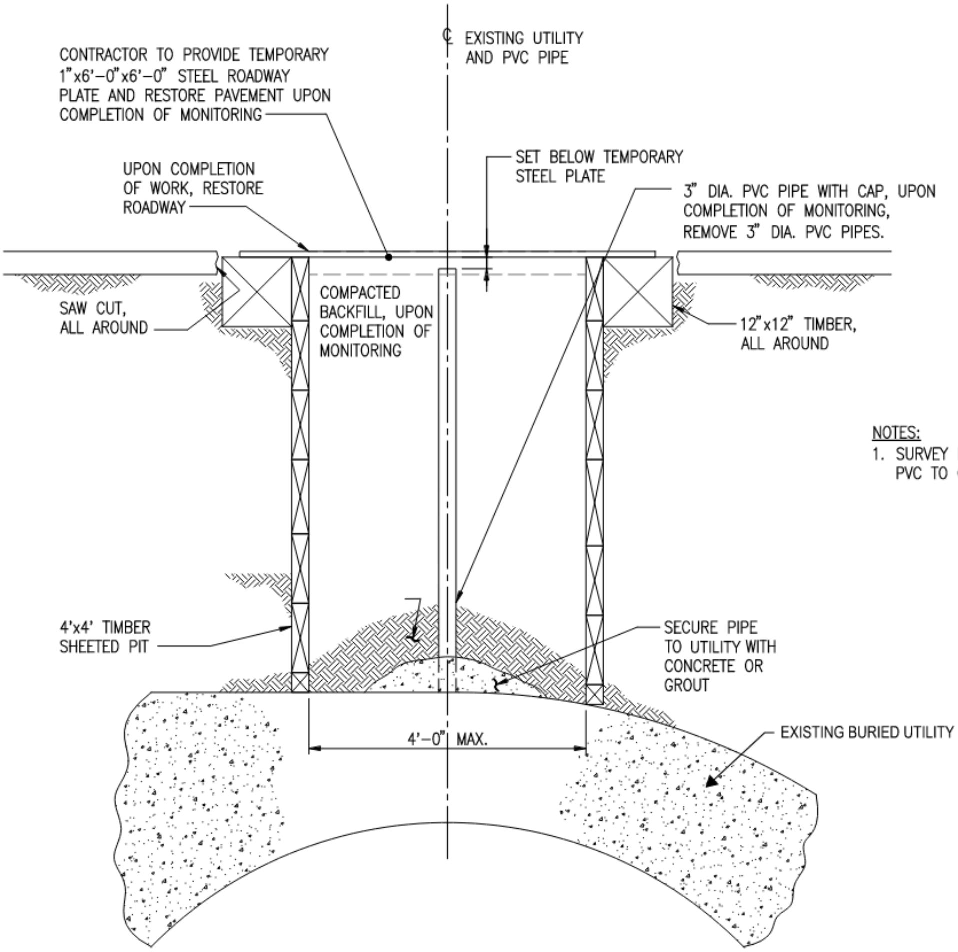
TYPICAL SURVEY PRISM ON STRUCTURE DETAIL

Note: Typical detail provided for conceptual planning purposes only.

CITY OF ALEXANDRIA WATERFRONT IMPLEMENTATION PROJECT
ALEXANDRIA VIRGINIA

DEFORMATION MONITORING POINT ON STRUCTURE

FIGURE NO.
B2



NOTES:
1. SURVEY ROD TO BE INSERTED INTO 3" PVC TO OBTAIN READING.

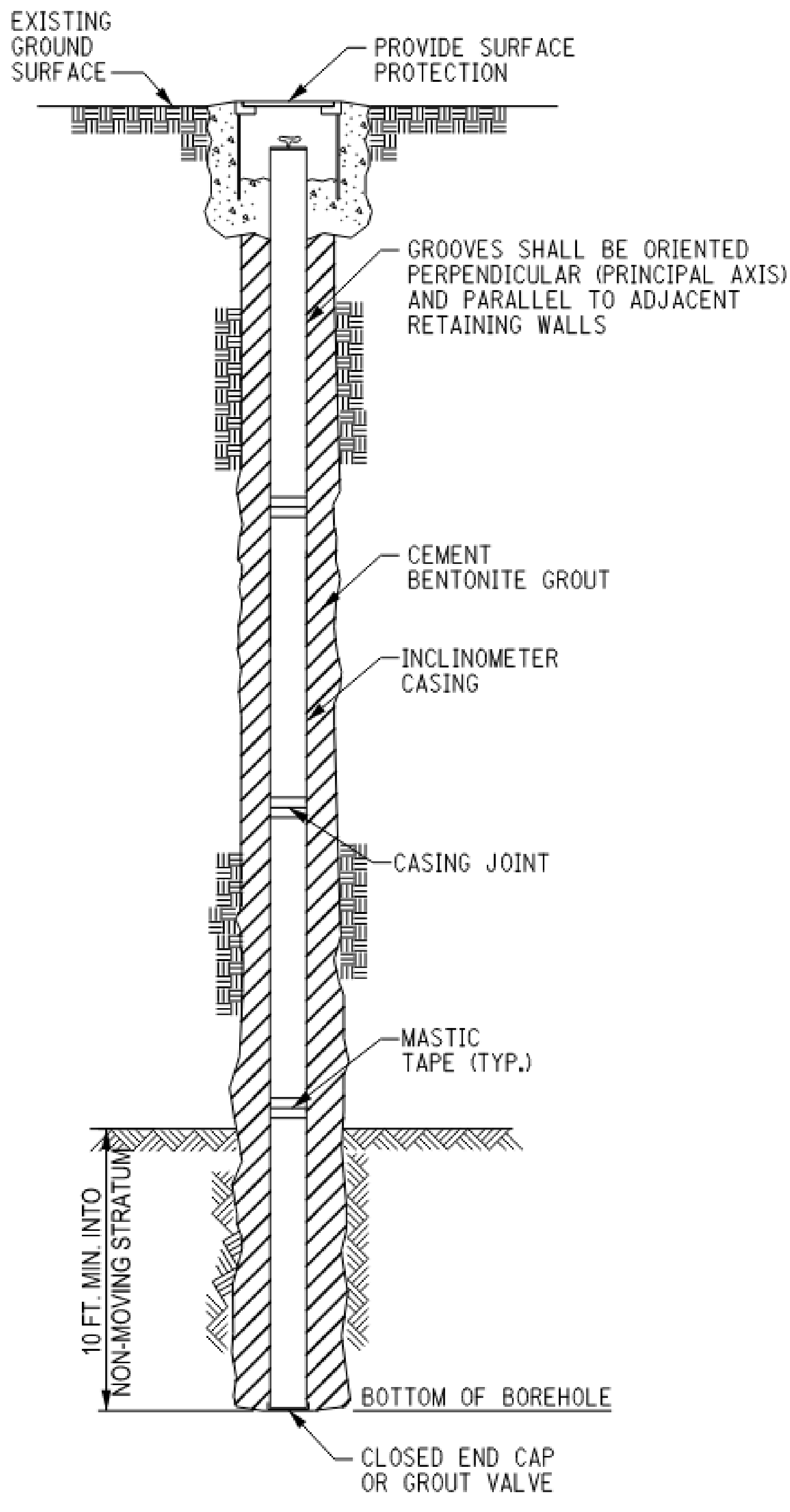
Note: Typical detail provided for conceptual planning purposes only.

CITY OF ALEXANDRIA WATERFRONT IMPLEMENTATION PROJECT

ALEXANDRIA VIRGINIA

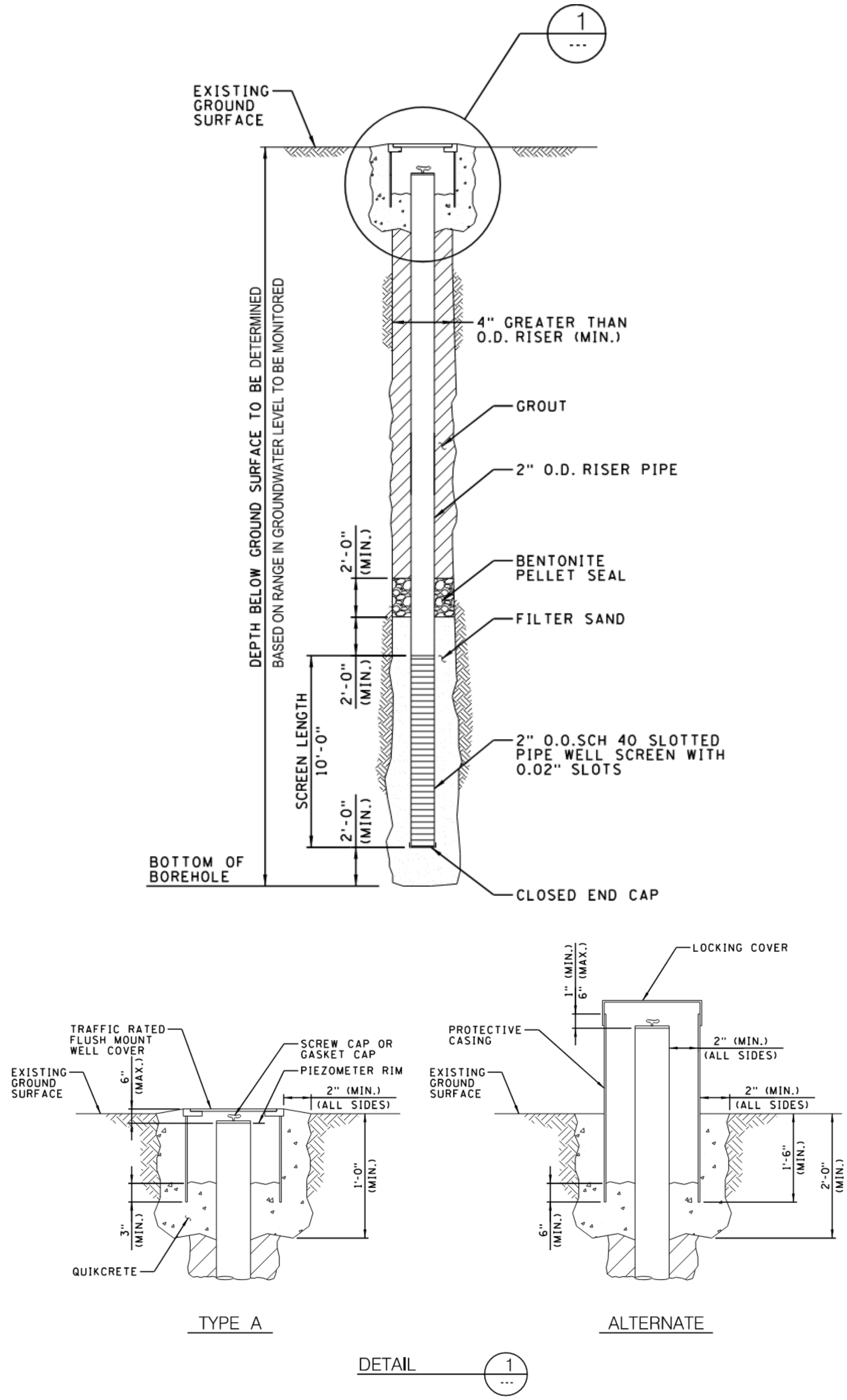
DEFORMATION MONITORING POINT ON BURIED UTILITY

FIGURE NO.
B3



Note: Typical detail provided for conceptual planning purposes only.

CITY OF ALEXANDRIA WATERFRONT IMPLEMENTATION PROJECT ALEXANDRIA VIRGINIA	
TYPICAL INCLINOMETER INSTALLATION DETAIL	FIGURE NO. **B4**



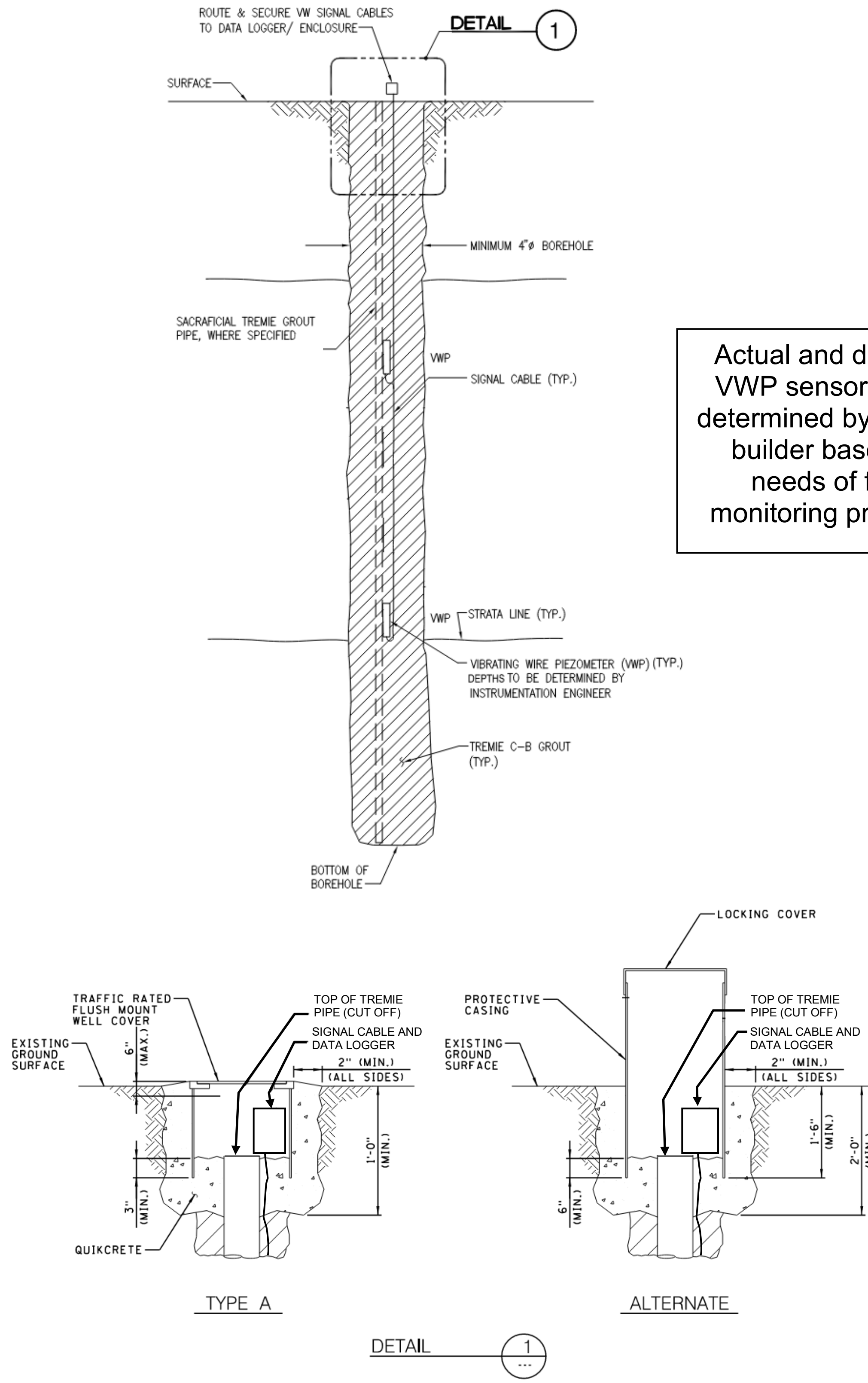
Note: Typical detail provided for conceptual planning purposes only.

CITY OF ALEXANDRIA WATERFRONT IMPLEMENTATION PROJECT

ALEXANDRIA VIRGINIA

TYPICAL OPEN STANDPIPE PIEZOMETER DETAIL

FIGURE NO.
B5



Actual and depth of VWP sensors to be determined by design-builder based on needs of final monitoring program.

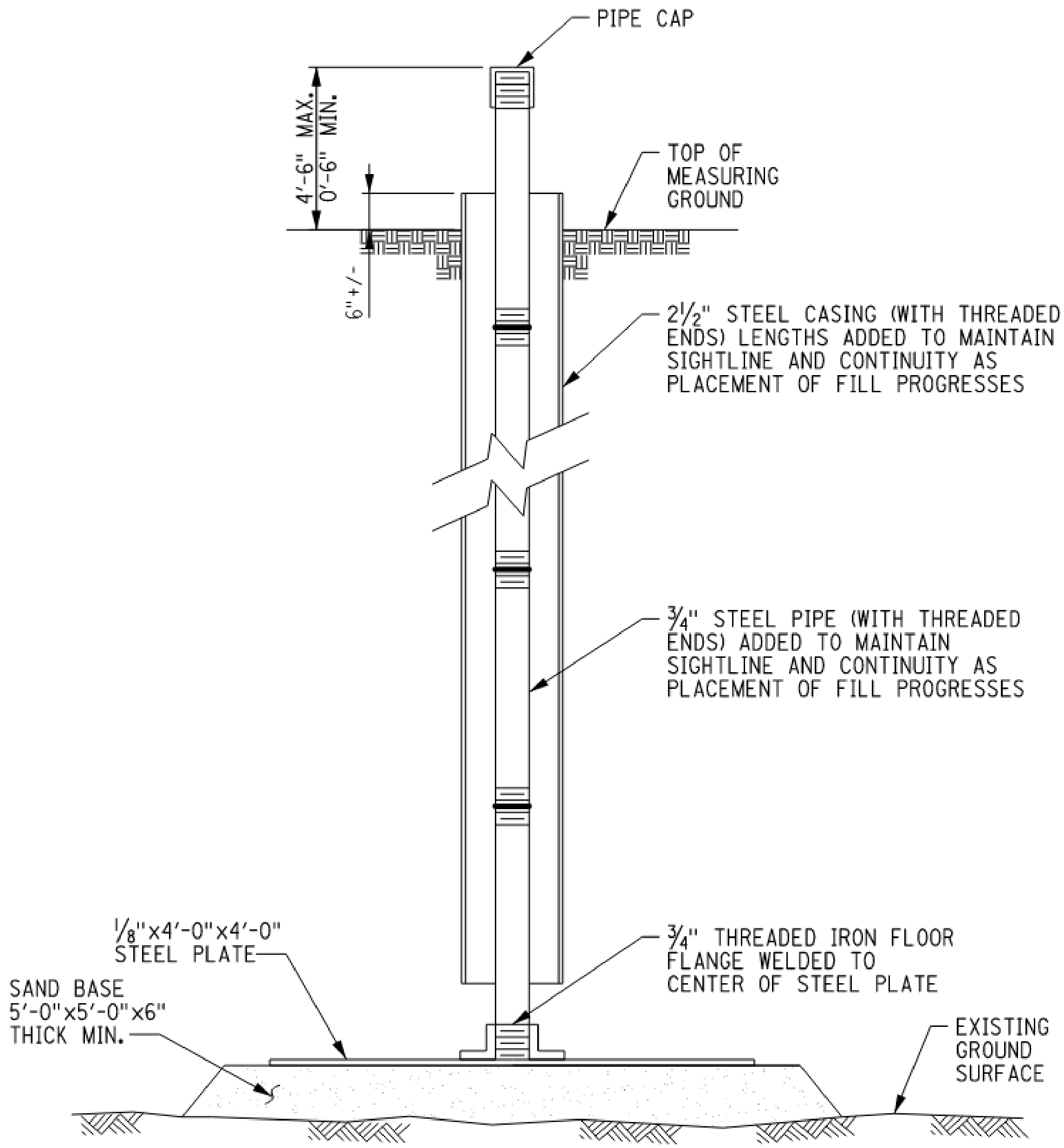
Note: Typical detail provided for conceptual planning purposes only.

CITY OF ALEXANDRIA WATERFRONT IMPLEMENTATION PROJECT

ALEXANDRIA VIRGINIA

TYPICAL GROUTED VIBRATING WIRE PIEZOMETER

FIGURE NO. **B6**



Note: Typical detail provided for conceptual planning purposes only.

CITY OF ALEXANDRIA WATERFRONT IMPLEMENTATION PROJECT

ALEXANDRIA VIRGINIA

SETTLEMENT PLATE TYPICAL DETAIL

FIGURE NO.
B7