

PHASE 1 GEOTECHNICAL REPORT

Alexandria Waterfront Flood Mitigation Alexandria, Virginia

Schnabel Reference: 16C12012
January 25, 2017



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Mr. Jeffrey Lohr, PE
Stantec
4500 Daly Drive, Suite 100 Fairfax
Chantilly, VA 20151

**Subject: Project 16C12012, Alexandria Waterfront Flood Mitigation
Phase 1 Geotechnical Engineering Report, Alexandria, Virginia**

Dear Mr. Lohr:

SCHNABEL ENGINEERING, LLC (Schnabel) is pleased to present this Phase 1 Geotechnical Engineering Report (GER) providing our preliminary geotechnical recommendations. The Phase 1 GER includes our preliminary recommendations to help in evaluating design concepts and options for the proposed construction. In addition, the Phase 1 GER addresses potential geologic hazards, geotechnical-related design considerations, preliminary pavement recommendations, earthwork considerations, construction dewatering considerations, and an assessment of potential ground improvement techniques that may be used for this project. Furthermore, we have provided recommendations for the Phase 2 Geotechnical Investigation.

INTRODUCTION

In 2012, the City of Alexandria, Virginia, approved the Alexandria Waterfront Small Area Plan (Waterfront Plan), which provides a 20- to 30-year vision for development of the Alexandria waterfront. The Waterfront Plan includes a framework for revitalizing Alexandria's waterfront by incorporating Alexandria's history, expanding and enhancing public open spaces, improving public access and connectivity, promoting the waterfront as an arts and cultural destination, and ensuring compatible development. The area of the proposed Waterfront Plan extends from Wilkes Street on the south to Canal Center Plaza on the north, and between the Potomac River to the east and Union Street (from Wilkes Street to Pendleton Street) and North Lee Street (from Pendleton Street to Canal Center Plaza) on the west.

Phase I of the planned waterfront development will include construction of a promenade, a bulkhead, and a flood mitigation system in the "Core Area" of the Waterfront Plan. The Core Area of the proposed development, bounded by the Potomac River to the east, Queen Street to the north, Union Street to the west, and Duke Street to the south, is shown on Figure 1, Site Vicinity Map.

DESCRIPTION OF SITE AND PROPOSED CONSTRUCTION

Site Description

The City of Alexandria, Virginia (City) is a municipality with a population of approximately 140,000, located on the west bank of the Potomac River, near Washington, DC. The site is located in Old Town Alexandria, near the waterfront on reclaimed land¹ as shown in Figure 2. The site is generally flat and highly developed with a variety of buildings, roadways, parks, piers, and other structures.

The Core Area of the project is located in an urban area with more than 200 years of waterfront development history. At some point between the late 1700's and the mid-1800's, the river channel was filled in from N. Lee Street to where the shoreline presently exists today. A variety of methods have been used to stabilize the new shoreline over time including stone, wood beams, metal bulkheads, concrete, and vegetation. Based on our exploration program, the materials used to fill in the river channel appear to be an uncontrolled mixture of coarse- and fine-grained soils with wood, metal, ceramic, glass, brick, and other miscellaneous debris placed directly on top of very soft river muds.

We obtained this site information from a site plan dated May 2016, prepared by Stantec, and through our site visits.

Proposed Construction

We understand that the following improvements will be performed as part of the Phase I flood mitigation system and Potomac River shoreline improvements project:

- A new structural bulkhead will be installed to at least EL 6.0 along the Potomac River within the project area. The proposed bulkhead will be generally located east of the existing shoreline and in some portion east of the existing bulkhead line;
- A new riverfront promenade, 20 ft to 25 ft in width, adjacent to the new structural bulkhead. The promenade includes a landside component and a riverside component. The landside component will include a paved walking path and areas with a stepped bulkhead, or grand steps, into the water. The waterside component will include a wooden boardwalk constructed on pilings over the river;
- Two proposed pump stations, including screens, wet wells, pumps, backup generators, backup fuel sources, discharge piping, mechanical equipment, controls, and all related infrastructure. Each pump station site will include public restroom facilities;
- A new storm sewer network to convey upstream runoff directly to the river, bypassing the pump stations; and
- A new storm sewer inlet and pipe network to collect and convey runoff from the proposed development area to the proposed pump station wet wells.

¹ City of Alexandria Department of Planning and Zoning, "Alexandria Historic Shoreline Map," 2009

The locations, extents, and elevations of these improvements are currently conceptual and will be developed during future design phases. Additional information on proposed construction is presented in the geotechnical recommendations sections.

SUBSURFACE CONDITIONS

Subsurface conditions at the site were explored by Schnabel in August 2016. The results of our exploration are presented in our report titled "*Alexandria Waterfront Flood Mitigation Phase 1 Geotechnical Data Report, Alexandria, Virginia*," dated October 26, 2016. A brief description of subsurface conditions is presented below.

The subsurface stratigraphy at the site generally consists of the following strata: Stratum A (Fill) was encountered throughout the site at the surface and consists of both coarse-grained and fine-grained soils with varying amounts of mica, shells, and other debris including wood, metal, and brick to a depth up to 43.5 ft below ground surface (ft-bgs). Below Stratum A, where present, is Stratum B (Recent Alluvium) that primarily consists of very soft to stiff fine-grained soils. Underlying Stratum B, where present, Stratum C (Old Town Terrace) deposits consist of coarse-grained soils that are very loose to loose in compaction. Underlying all the above Strata, Stratum D (Potomac Group) consists of medium stiff to hard elastic silts and lean to fat clays to the maximum depth explored during our investigation.

SITE HAZARDS

Flooding

The City of Alexandria Energy and Climate Change Action Plan², prepared by the Office of Environmental Quality, Department of Transportation and Environmental Services and dated June 2011, projects the impacts of climate change on the Potomac River water levels through the year 2100. The Action Plan projects that the median sea level rise over this period will range from 1.33 ft to 3.35 ft and the median sea level rise during high tide will range from 2.94 ft to 4.96 ft.

The 2011 Federal Emergency Management Agency (FEMA) Flood Insurance Rate Map (FIRM) estimates that project site is located in special flood hazard area subject to inundation by the 1 percent annual chance flood (100-year flood). The base flood elevation along the Alexandria waterfront is predicted to reach EL 10.

Seismic Hazards

The City is located in a low seismicity area, far from any active tectonic plate boundaries. However, large magnitude earthquake events have been recorded in the eastern and southeastern US, such as the 1886 Charleston, South Carolina and the 1897 Giles County, Virginia earthquakes, but the historical record for earthquakes is limited. These large magnitude events are infrequent and the frequency and mechanism of these earthquakes are not well understood. Of the recorded earthquakes within a 300 mile radius of

² *City of Alexandria Energy and Climate Change Action Plan, Local Actions to Save Energy, Reduce Greenhouse Gas Emissions, and Prepare for the Impacts of Climate Change 2012-2020*. Rep. Alexandria: Office of Environmental Quality Department of Transportation and Environmental Services, 2011. Print.

the project site, only seven were of moment magnitude M4.5 or greater. The largest modern earthquake in proximity to the project site was the moment magnitude 5.8 earthquake that occurred in Mineral, VA on August 23, 2011, approximately 45 miles southwest of the project area.

Liquefaction is a phenomenon where loose, granular soils below the groundwater table lose strength during strong and extended earthquake shaking over successive cycles of ground motion. Liquefaction is most likely to occur in areas with shallow groundwater, where the subsurface profile consists of relatively thick layers of loose, granular material (generally SPT N-values less than about 10), and when the site is subjected to strong, and sufficiently long, ground motions. Recent research has indicated that, while less likely, fine-grained soils can also be susceptible to liquefaction (Bray and Sancio, 2006³). Therefore, it is our opinion that the soil liquefaction potential at the site is low to moderate due to the presence of a shallow water table, the presence of loose granular material in the upper 15 ft, and some deposits of fine-grained soils.

PRELIMINARY GEOTECHNICAL RECOMMENDATIONS

The following sections present our geotechnical recommendations for the proposed construction. We based our geotechnical engineering analysis on the information developed from our subsurface exploration and soil laboratory testing, along with the project development plans, site plans, and structural loading furnished to our office by Stantec and Moffatt & Nichol. The recommendations presented herein are preliminary and will be revised and refined based on how the design process progresses.

The wooden boardwalk is currently in the concept design phase and no specific design information is available at this time. In addition, there is currently no subsurface data within the river channel for the boardwalk location since barge drilling will not occur until Phase 2. Therefore, recommendations regarding the boardwalk will be provided in the Phase 2 geotechnical recommendations.

General

Design Groundwater Elevation

We recommend that the 100-year flood level of EL 10 be used for permanent structures as well as the temporary support of excavation (SOE) systems for the project.

Seismic Site Classification

We evaluated the Seismic Site Class and Seismic Site Coefficients for this project according to the 2012 Edition of the Virginia Construction Code, Part I of the Virginia Uniform Statewide Building Code, which adopts IBC Section 1615 [2012]. Our evaluation for Site Class was based on correlations with corrected SPT values and shear strength values from pocket penetrometer test results. The seismic Site Class and coefficients for this site are presented below:

³ Bray, J. and Sancio, R. (2006). "Assessment of the Liquefaction Susceptibility of Fine-Grained Soils." *J. Geotech. Geoenviron. Eng.*, 10.1061/(ASCE)1090-0241(2006)132:9(1165), 1165-1177.

Site Class:	E
F _a :	2.5
F _v :	3.5

Geotechnical Challenges

- Due to the long and varied use of the waterfront over the years, we anticipate that historic foundations, pilings, bulkheads, utilities, debris and contamination from previous site development may remain on site. These relic structures may be encountered as obstructions either during excavation or during installation of deep foundations. The contractor should be prepared to encounter and remove such obstructions during their work. Contaminated soils encountered during excavation will need to be properly characterized, handled, and transported to an appropriately licensed disposal facility.
- The site is also underlain by a very soft layer of under-consolidated alluvial soils or river mud (Stratum B). These soils are very soft and weak, with high water contents. The alluvial soils and any structures supported by them will slowly settle over time due to the weight of the fill layer above as well as any surcharge loads. These alluvial soils are also sensitive to disturbance during construction and may trigger basal heave when deep excavations get close to this stratum.
- The project is located in a historic area with many buildings and structures sensitive to movement. Care will be needed during excavation and construction to make sure that the planned work will not damage existing structures. Information about nearby structures, including foundation type and size, the presence of basements and their construction, drainage systems and waterproofing, and utilities will be needed as the design progresses to Phase 2.
- Excessive vibrations, unexpected horizontal or vertical ground movements, and tieback installations all are potential sources of structural damage to existing facilities. In addition, the project is located in a high-visibility area of the waterfront that is a tourist attraction and receives a significant number of visitors throughout the year, which could be negatively impacted by noise. We recommend that construction methods and foundation systems that generate loud noises and vibrations, such as driven piles, be avoided, if possible.
- The groundwater table is shallow and the site is located next to the Potomac River, which floods periodically. Groundwater and surface water controls will be needed during construction to maintain a stable and dry excavation. These controls may include well points, recharge wells, sumps and pumps, cut off walls, diversion structures and cofferdams.

Corrosion Potential and Chemical Attack

As discussed in our Preliminary Geotechnical Data Report, the soils at the site can be corrosive to ductile iron according to AWWA Specification C105. Additional sacrificial steel or coating using protective materials is expected to be required for buried metal structures. In addition, the soils of Strata A and B exhibit a Class I sulfate exposure per American Concrete Institute (ACI) Publication 318, Section 4.3 (as designated in Section 1904.3 of the IBC 2009 manual), which can be detrimental to concrete. In accordance with ACI, we recommend that Type II or equivalent sulfate-resistant cement be used in construction.

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Results of laboratory testing presented in our October 26, 2016 Geotechnical Data Report and the results of the Phase 2 investigation should be reviewed by the project corrosion consultant to develop specific recommendations with respect to corrosion protection of proposed structures and buried utilities.

Bulkhead

Proposed Construction

A new bulkhead is proposed to be constructed along the Potomac River waterfront from Duke Street to Queen Street. We understand that the ground surface elevation behind the bulkhead will be EL 6. The river directly in front of the bulkhead wall will be dredged to EL -6 and the bottom will slope down to EL -10 at a distance of 10 ft away from the bulkhead. We understand that an easement of only 25 ft to 30 ft is available for bulkhead anchorages due to nearby existing structures supported on deep foundations. Therefore, permanent easements for long tiebacks/anchors behind the bulkhead are not possible.

Preliminary design requirements provided by Moffat & Nichol include a surcharge load of 250 psf which is consistent with an HS-20 vehicle with a truck axle load of 32,000 pounds. Acceptable lateral deformation of the bulkhead is tentatively set as 1 percent to 2 percent of the free height. Information on surcharge loads due to boat impact are not available at the time of this report.

Design Parameters

We provide anticipated generalized subsurface conditions and recommended soil parameters for bulkhead design in Table 1.

Table 1: Estimated Bulkhead Design Subsurface Conditions and Parameters

Top Depth (ft)	Stratum	USCS Classification	Unit Weight (pcf)		Angle of Internal Friction (°)	Wall Friction Angle (°)	Undrained Shear Strength (psf)	Adhesion (psf)	Strain Values for Clay, e_{50}	Horizontal Soil Modulus, k (pci)
			Moist	Buoyant						
0	A	SM	120	63	28	14	-	-	-	45
15	A	CH/MH	95	33	-	-	250	250	0.02	20
40.1	B	CL/OH	95	33	-	-	250	250	0.02	20
63.5	D	CH	115	53	-	-	1,000	700	0.01	500
73.5	D	CL	120	58	-	-	2,000	720	0.005	750
83.5	D	CH	135	73	-	-	3,000	735	0.005	1,000
New Fill		See Table 4	125	68	32	16	-	-	-	45

Recommended type of bulkhead system

Tiebacks/anchors are often used to resist the lateral movement of bulkheads. These anchors transfer lateral loads on the bulkhead into deeper soils beyond the active wedge/zone behind the bulkhead. This allows the anchors to obtain capacity from frictional resistance in stiff/dense soils. We anticipate that multiple rows of tiebacks would likely be needed. However, as discussed above, we also understand that tiebacks/anchors greater than 25 ft in length behind the bulkhead are not possible.

In our opinion, the following types of bulkheads could be considered based on the limitation on the easement behind bulkhead and the encountered subsurface conditions:

- Sheet pile bulkhead with battered steel H-piles at regular horizontal distance with a capping beam at the top to provide stability;
- Sheet pile bulkhead with a parallel row of steel H-piles to form a frame such that the overturning moments from the retained soils could be resisted within the frame;
- Sheet pile bulkhead with tiebacks anchored to deadmen supported on battered micropiles or drilled shafts. Note that for deadmen located within the active zone of soil, the deep foundations will need to be designed for reduced capacities;
- T-panel secant pile bulkhead wall with a capping beam at the top to provide stability;
- T-panel press-in sheet pile bulkhead wall with a capping beam at the top to provide stability;
- Sheet pile bulkhead with excavation behind the bulkhead and replacement with lightweight fill such as geofoam, cellular concrete, or other lightweight material;
- Pre-cast concrete sheet piles (pre-tensioned or post-tensioned); and
- Cellular cofferdam.

No specific recommendation are provided for the above options at this time. Recommendations for the above options should be discussed in the Phase 2 Geotechnical Report.

Pump Stations

Proposed Construction

Two new pump stations are proposed to be constructed. The pump stations will consist of one above-grade level and one below-grade level. Pump Station 1 will be located along Strand Street adjacent to Waterfront Park near the intersection of Strand Street and Prince Street. We understand that the main floor level will be at EL 5 and have an approximate footprint of 20 ft by 37 ft. A portion of the below-grade level at both pump stations will extend beyond the footprint of the above-grade level. The pump station below-grade will have a lowest floor level at approximately EL -12.5.

Pump Station 2 will be located in an area that is currently under the Potomac River, adjacent to Founders Park. The pump station will be located on new fill that will be placed behind the realigned proposed bulkhead. We understand that the lowest floor elevation of the pump station building will be at EL 7 and have an approximate footprint of 20 ft by 59 ft. The pump station's below-grade level will have a lowest floor level at approximately EL -13.5. Various alternatives for the layout and dimensions of the pump station are under consideration. For the purposes of our analysis, it was assumed that the bulkhead would be constructed first. This would allow for fill to be placed behind the bulkhead to raise the site grades to design elevations for the construction of the Pump Station. After construction of the below-grade level is completed, additional fill would be placed to reach the design grades.

For both pump stations, anticipated maximum column and wall loads are 60 kips and 9 kips/ft, respectively. We understand that the design team is considering a concrete mat foundation with an approximate average pressure of 1.15-1.5 kips/ft for the below-grade bottom slab. The design team is considering tolerable settlement for the pump stations as 1 inch with tolerable differential settlement between columns of 0.5 inches.

Below-Grade Walls

Wall Types

We have considered two different feasible approaches to the construction of the below-grade walls.

Option 1: Conventional cast-in-place concrete walls are generally constructed by excavating the building footprint to the bearing elevation, installing concrete forms, and then placing concrete. At the pump station sites, this approach will require significant construction dewatering and a robust SOE system. Accordingly, the SOE systems such as temporary sheet piles will be needed. Sheet piles should extend down into Strata C/D to achieve sufficient resistance and fixity at the base and groundwater cutoff. The soft/loose alluvial soils of Stratum B will not provide axial or lateral resistance for SOE. Due to easement restrictions on tiebacks, we anticipate that internal bracing will be required to limit horizontal deformations at the top of the wall. This option has the advantage of exposing and removing any obstructions or debris that may be present within the excavation limits and could complicate other construction approaches.

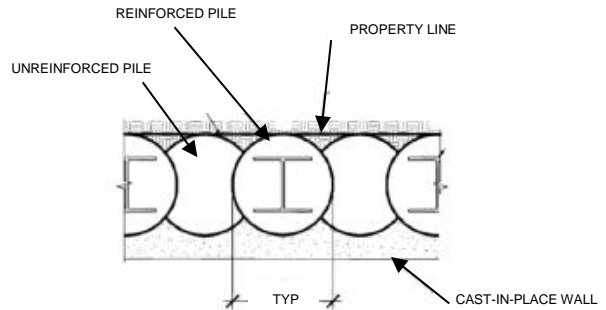
Local groundwater levels will need to be lowered to at least 3 ft below the bottom of excavation to limit the potential for basal heave at the bottom of the excavation. Lowering of groundwater level could also result in unacceptable settlement of nearby ground, utilities, or structures supported on shallow foundations, especially if groundwater cutoff is not achieved by the SOE. Water pumped from the excavation may also need special handling and disposal if it is contaminated. Due to the shallow groundwater levels, a soldier pile and lagging system would not be appropriate as an SOE.

If sheet piles cannot be installed deep enough to cutoff groundwater, another approach would be to install a combined wall system composed of longer “king piles” and shorter sheet piles. The shorter sheet piles would only extend 5 ft to 10 ft below the bottom of the excavation to provide lateral restraint to the soil at the sides of the excavation. The shorter sheet piles would transfer the soil and water pressures to the king piles, which would be installed deeper, into Strata C/D. The king piles would also reduce the potential for sheet piles to settle within the softer Strata A and B. The king piles (HZM or Pipe) are designed to carry all the vertical loads and lateral loads of the SOE system. An interlocking or welded connector is used to connect the king pile to the sheet pile. An approximately 3-ft to 5-ft thick concrete tremie slab could be placed in the wet at the bottom of the excavation to seal the groundwater inflow into the bottom of the excavation.

Option 2: An alternative approach for wall construction is to construct the below-grade walls as secant pile or diaphragm walls. Secant pile walls and diaphragm walls are walls cast or formed in the ground prior to the start of general excavation. These walls are designed to perform up to four different functions; groundwater cutoff, temporary excavation support, permanent below-grade walls, and deep foundations.

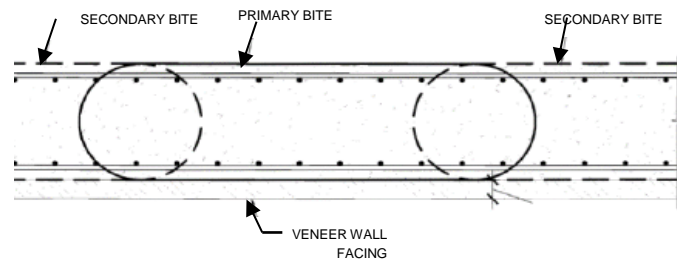
Secant pile walls consist of overlapping reinforced and unreinforced drilled shafts. The unreinforced primary shafts are drilled first and the reinforced secondary shafts are drilled only after adjacent primary shafts are backfilled and allowed to set.

The primary shafts function as a water-tight lagging that restrains soil, cuts off groundwater flow and prevent basal instability of the in-situ soils at the bottom of the excavation. We estimate the primary shafts would need to be drilled to approximately EL -40. The reinforced secondary shafts provide foundation support and lateral structural support. The secondary shafts will need to be socketed into the stiffer soils of Strata C and D.



Typical Layout of Secant Pile Wall

The **diaphragm wall** is a reinforced concrete wall formed and cast in a slurry trench, prior to the start of general excavation. The excavation for the diaphragm wall is performed in alternating panels or 'bites'. The trench excavation is temporarily kept open and stable with bentonite or polymer based slurries that balance soil and water pressures within the trench. Once a full panel has been excavated, the reinforcing cage is inserted and the slurry is replaced by concrete using tremie methods. Slurry wall construction requires specialized equipment designed for excavating slurry trenches. This equipment could include a hydraulic clamshell or a hydromill.



Typical Layout of Diaphragm Wall

Both the secant pile wall and diaphragm wall approaches allow the excavation to proceed without construction dewatering and recharging, which may reduce the risk of settlement of nearby utilities, pavements, buildings, and other structures. However, these methods may develop complications including penetration difficulties and groundwater seepage through leaks if significant obstructions are present in the Stratum A fill soils. The secant pile wall and diaphragm wall will require waterproofing to maintain dry excavation conditions. The use of these systems will require that a finite element analysis be performed to model the behavior of the secant pile wall or diaphragm wall to confirm the minimum wall depths for stability and to determine the best construction sequence to manage deformations.

Lateral Earth Pressures

The proposed below-grade walls should be designed to resist lateral earth pressures from soils (both static and seismic effects), surcharge loads, and hydrostatic pressures. Due to shallow groundwater and the potential for flooding, we recommend that the below-grade walls be designed to resist full hydrostatic conditions. The amount of hydrostatic pressure should be calculated using a design groundwater at EL 10 ft for permanent construction based on results of the City of Alexandria Energy and Climate Change Action Plan.

The lateral earth pressure coefficients for both static and earthquake loading for use in the design of below-grade walls, are provided in Figure 3. The additive seismic force (P_E) acting on the retaining and below-grade walls may be applied as a trapezoidal lateral earth loading as shown in Figure 4. The below-grade walls will be subjected to at-rest (K_0) conditions from the retained soil. For purposes of lateral earth pressure calculations, we recommend a moist unit weight value of 130 pcf for backfill to account for possible variations in the source of backfill materials from potential off-site borrow sources. Where the backfill materials from the potential off-site borrow sources will be submerged below the groundwater table, a buoyant soil unit weight of 65 pcf may be used. For Stratum B soils, moist and buoyant unit weights of 100 pcf and 33 pcf, respectively, may be used.

The design horizontal acceleration coefficient (k_h) to be used for lateral seismic loading of retaining walls and below-grade walls is a function of:

- the design PGA (PGA_D),
- height and type of the wall, and
- allowable lateral movement of the wall.

Foundations

Shallow foundations for support of the pump stations are not recommended due to the potential for excessive settlement of the structures due to the presence of very soft to soft clay and silt (locally with considerable organics) extending to as deep as about EL -60. In general, the fill soils of Stratum A and the very soft to soft silt and clay layers in Stratum B are not suitable for foundation support. We understand that the design team is considering a concrete mat foundation for the pump stations. However, our analysis indicates the weight of the excavated soils does not fully compensate for the new building loads and would result in settlement. A pile-supported raft may be used instead of a mat foundation. The mat foundation can be supported on micropiles or drilled shafts to transfer the loads to the stiff clays of Stratum D.

The deep foundations for Pump Station 1 are expected to develop the required capacity within the medium dense cohesionless soils of Stratum C. The deep foundations of Pump Station 2 are expected to develop the required capacity within the medium stiff to stiff cohesive soils of Stratum D.

Straight drilled shafts (without bells) using the wet construction method are recommended. We do not recommend the use of auger-cast piles due to potential for necking and/or structural defects in the piles that could occur in soft-ground conditions. If micropiles are used, we anticipate that a minimum of four micropiles per column will need to be installed a minimum of 10 ft into the Stratum C/D soils to support the design column loads.

If noise and vibration from a driven pile system is acceptable, driven steel H-piles may also be used. Steel H-piles will likely need to be driven to EL -90. We estimated pile capacities for 12x53 steel H-piles but alternative, heavier sections may be used as well.

A factor of safety of 3 was used during our preliminary analysis of axial capacity. For conceptual planning purposes, we estimate the following allowable capacities:

Table 2: Preliminary Deep Foundation Recommendations

Structure	Foundation Type	Size	Tip Elevation	Allowable Capacity (kips)	Lowest level of very soft soils Elevation
Pump Station 1 (Strand Street)	Straight Drilled Shaft	36 inch dia.	EL -65 to -70	70	EL -43.5
	Micropiles	5.5 inch dia.	EL -55 to -60	25	
	Steel H-piles	12x53	EL -88 to -90	65	
Pump Station 2 (Founders Park)	Straight Drilled Shaft	36 inch dia.	EL -70 to -75	75	EL -59.5
	Micropiles	5.5 inch dia.	EL -70 to -75	25	
	Steel H-piles	12x53	EL -90 to 95	60	

A site grading plan is not available at this time but based on information provided by the design team, Pump Station 1 will require approximately 1 ft of fill. The main floor elevation is approximately the same as the existing ground surface elevation. However, Pump Station 2 will require up to 14 ft of fill since the structure is proposed to be constructed on new reclaimed land currently within the Potomac River. The significant amount of fill required to raise the grade will cause consolidation of Strata A and B, creating downdrag loads on the pile foundation in Strata A and B. The deep foundations of Pump Station 1 will also experience downdrag due to the under-consolidated nature of Stratum B soils. Downdrag has been considered in our analyses for allowable capacity of the drilled shafts in Table 2 for the pump stations.

The below-grade levels of the pump stations should be designed to resist transient uplift conditions that may occur during flooding event until the pumps start working. This can be accomplished by using the deep foundations as tie downs. The foundations in Table 2 will have sufficient uplift capacity to resist buoyancy of the below-grade levels if it were to become dry during high groundwater conditions.

Floor Slab

Based on test boring data, the soils at the proposed floor slab elevations are expected to consist of loose existing fill of Stratum A or very soft to soft clay soils of Stratum B. These soils are not considered suitable for direct floor slab support. Therefore, we recommend that the lowest level floor slabs be structurally supported. The floor slabs should be designed to resist the hydrostatic uplift pressures and must be waterproofed.

Site Grading and Earthwork

Placement and compaction of all fill materials should be conducted in the dry. New fill lifts should not be placed until the underlying subgrade or prior lift has been approved by a qualified Geotechnical Engineer or his/her representative.

The natural moisture contents of imported soils should be expected to vary. Moisture conditioning of these soils is likely to be required to achieve proper compaction. Moisture conditioning might include drying of excessively moist soils (by scarification or other methods) and adding moisture to excessively dry soils. Careful planning of fill operations may be required to allow moisture conditioning of individual fill

lifts. Some delays due to moisture conditioning or difficulties achieving compaction should be expected. High plasticity fine-grained soils are susceptible to moisture changes, will be easily disturbed, and will be difficult to compact under wet weather conditions. Drying and reworking of the soils are likely to be difficult during periods of wet months. We recommend that the earthwork phases of this project be performed during the warmer, drier times of the year to limit the potential for disturbance of on-site soils.

Stratum B soils are very soft, moderate to high plasticity clays and silts. These soils are moisture sensitive, and will become readily disturbed by construction and vibration.

Seasonal Shade Structure

Proposed Construction

A Seasonal Shade Shelter (SSS) will be located adjacent to Waterfront Park at the current location of the Old Dominion Boat Club. We understand that the SSS will be wood construction and have a floor elevation of EL 3 and an approximate footprint of 30 ft by 30 ft. The maximum column and wall loads for the SSS are 15 kips and 1.5 kips/ft, respectively. Tolerable settlements and deflections between columns are 1½ inches and ½ inch, respectively.

Shallow Foundations

Boring BH-2/2A, located approximately 200 ft from the proposed location for the SSS encountered existing fill to EL -40. We consider shallow foundations such as spread footings and strip footings suitable for support of the proposed seasonal shade structure if the existing fill soils are undercut and replaced with compacted structural fill. The undercut should be to a depth of 2.5 ft below the bearing elevation of the footings. To avoid frost effects, shallow foundations should be installed at least 30 inches below the ground surface.

We recommend spread and strip footings supported on structural fill be designed for a net allowable soil bearing pressure of 500 psf. This bearing pressure provides a factor of safety against general bearing capacity failure of at least 3.0.

The above allowable soil bearing pressure may be increased by 33 percent for wind and seismic loads when used in conjunction with load combinations defined in IBC 2012 Section 1605.3.2, Alternative Basic Load Combinations for use with allowable stress design. This increase is not applicable for other allowable stress load combinations, strength design, or load and resistance factor design.

Floor slabs

The floor slabs of the SSS may be supported on-grade if the subgrade is undercut and the slab-on-grade is supported on at least a 2-ft thick layer of structural fill. A 4-inch thick, crushed stone (such as VDOT No. 57 stone) capillary break should be placed below the floor slabs-on-grade. The Contractor should compact the stone in place with at least two passes of suitable vibratory compaction equipment. We recommend that the edges of the slab be turned down to at least 30 inches below the adjacent finished grade to protect against frost heave.

Storm Drains

Proposed Construction

New by-pass storm drains are proposed to be constructed throughout the overall project site. The summary table below describes each proposed storm drain. At the time of this report, no information was available on the foundation systems that currently support the existing City of Alexandria storm drains in the area; and their performance to date. In addition, no information is currently available on tolerable settlements or deflections for the proposed storm drains.

Table 3: Summary of Proposed Storm Drains

Proposed Storm Drains	Diameter (inches)	Invert Elevation (ft)	Total Length (ft)
Prince St. By-Pass	36	2.84 to -0.21	654
King St. By-Pass	48	0.07 to -2.42	541
Duke St. By-Pass	24	-0.02 to -2.10	188
Queen St. By-Pass	36	-1.47 to -2.73	252
Prince St. to Pump Station 1	15 to 36	3.04 to -6.95	1,468
Cameron St. to Pump Station 2	15 to 24	1.75 to -5.18	1,100

It is our understanding that the proposed storm drain outfalls will pass through the proposed bulkhead and discharge into the Potomac River near the channel bottom. The bulkhead will move, flex, and settle relative to the outfall pipes as conditions in the river change over time and the structure settles. We recommend that the connection between the outfall pipes and bulkhead be a very flexible connection that tolerates movement in all three axes (x, y, and z) to prevent shearing stresses or negative slopes from developing in the drain pipe. This also requires that the outfall elevations should consider the potential settlement of the storm drains.

Foundations

We do not recommend supporting the proposed storm drains on shallow foundations due to potential for excessive total and differential settlement of the structures as the thick layer of soft alluvial soils of Stratum B consolidate. We recommend that bored deep foundations such as drilled shafts and micropiles be used to transfer the structure loads to the firm soils of Strata C and D. If information is located about the foundation support of the existing City of Alexandria storm drains and their performance to date, we will revisit our foundation recommendations.

If a sudden flood event were to occur and the storm drain pipes were empty, buoyant forces causing uplift could develop if the storm drain pipes were to slow or delayed to fill with water. Therefore, we also recommend the use of straps to secure the storm sewer drain to the deep foundation system to resist uplift or buoyant forces that may develop during flood events. Refer to the Pump Station section for unit capacities for deep foundation systems.

For the portions of the storm drain system that will flow under pressure, thrust blocks will be needed at locations where the storm drains flowing under pressure changes direction. The thrust blocks will need to be supported by deep foundations due to the weak underlying soils present throughout the site.

Fill Materials

New fills at the site may consist of structural fill, backfill behind retaining and below-grade structures, drainage fill, utility backfill, low permeability fill, pavement base course, and common fill. Table 4 presents our recommendations for these various fill types:

Table 4: Fill Material and Compaction Recommendations

Fill Type	Use	Suitable Material Type/ Maximum Particle Size	Liquid Limit	Plasticity Index	Minimum Relative Compaction Requirement (per ASTM D698)^{1,2}
Structural Fill	Below structures and trench backfill in paved areas ³	USCS Classification SC or coarser / 3 inch	<40	<20	95%
Structural Backfill	Behind below-grade walls	USCS Classification SM or coarser / 3 inch	<40	<10	95% ⁴
Pavement Base Courses	Vehicular pavement base courses	Dense-graded aggregate (equivalent to VADOT 21A)	<40	<10	100% in the upper 1 ft of base course 95% (below this depth)
Common Fill	General landscape areas, non-structural fill, utility trench backfill above pipe bedding in non-pavement areas	USCS Classification CL or coarser / 4 inch	<45	<30	90%
Pipe Bedding	Pipe bedding material for utilities	VDOT No. 57	NA ⁵	NA ⁵	See Note 6

- Notes:
- (1) Fill should be placed in maximum 8-inch thick (unless otherwise noted) horizontal, loose lifts and compacted to the percentage indicated. In confined areas, such as excavations or trenches where large compaction equipment cannot be used, light, hand-operated equipment must be used and the fill should be placed in 4-inch thick horizontal, loose lifts.
 - (2) Soil moisture contents at the time of compaction should be within 2 percentage points of the soils' optimum moisture content.
 - (3) Areal structural fills below building footprints should extend laterally beyond the building footprint a distance equal to the height of the fill, or 5 ft, whichever is less.
 - (4) Fill being placed within a distance equal to the height of the below-grade walls or within 5 ft of any wall, should be placed in maximum 4-inch thick horizontal, loose lifts and compacted using light, hand-operated equipment to the percentage indicated.
 - (5) Not applicable.
 - (6) Pipe bedding should be placed in a maximum 6-inch thick horizontal, loose lift and compacted with two passes of compaction equipment.

The existing fill soils of Stratum A are not suitable for reuse as structural fill since it contains fine-grained soils and debris like ceramic, metal, wood, glass, and brick. In addition, we understand that environmental screening and testing performed by GeoConcepts indicated that the fill is contaminated with low levels of diesel and gasoline-range organics, arsenic, barium, chromium, lead, and mercury.

Temporary Excavation Support

Temporary excavations up to 20 ft deep may be required to construct the below-grade level of the pump station and install the new the storm drains.

Temporary excavation methods such as trench boxes, soldier piles and lagging with tiebacks, and steel sheet piles may be used to support shallow excavations in the existing fill soils of Stratum A. However, in our opinion these systems may not be practical for excavations deeper than 5 ft to 6 ft for the following reasons:

- Significant dewatering using well points and recharge wells will be necessary
- Potential for basal heave due to Stratum B soils
- Soldier piles and/or sheet piles may need to be embedded in Strata C and D soils to achieve fixity at base
- Limited right-of-way to install tie-backs or anchors
- Risk of damaging movements/settlements to nearby historic structures due to groundwater well point/recharge system.

Deeper excavations for the below-grade levels and deep storm drains near the pump stations and bulkhead may be accomplished using more rigid structural support such as secant pile walls or diaphragm walls. These wall systems can also be used as permanent below-grade walls for the pump stations. Refer to the section “Below-Grade Walls” for pump stations for further discussion and recommendations.

Groundwater Control

The use of temporary support of excavation systems such as trench boxes and soldier pile and lagging will require the lowering of the groundwater levels to allow for a dry working environment at the base of the excavation. In our opinion, well point dewatering systems will be necessary for deeper excavations to lower the groundwater around the perimeter of the excavation. However, dewatering will cause consolidation settlement of the soft alluvial soils underlying the adjacent buildings, roads, and utilities unless recharge wells to maintain the groundwater levels away from the excavation and reduce the potential for damage to the adjacent structures. Therefore, we do not recommend the use of well points. Instrumentation monitoring of adjacent buildings and utilities is recommended. Refer to Instrumentation and Monitoring section later in this report.

Ground Improvement

As an alternative approach to managing basal heave and groundwater, ground improvement may be used to decrease the permeability and to increase the shear strength of Stratum B soils during construction of the below-grade levels of the pump station. Ground improvement methods such as deep soil mixing and jet grouting could be feasible ground improvement methods.

Deep soil mixing is the process of mechanically mixing in-situ soils with a dry cement or wet mix cement grout to improve the engineering properties. Due to the high water content and low strength of all alluvial soils, the dry reinforcement approach is recommended.

Jet grouting is another alternative method. Both of these approaches may be technically feasible but cost-prohibitive. In addition, jet grouting operation generates significant amount of waste and spoils that will need to be managed.

Promenade and Boardwalk

As discussed in the Bulkhead section, a promenade and boardwalk will be constructed behind and in front of the bulkhead along the waterfront from Duke Street to Queen Street. In the Bulkhead section, additional information on ground surface elevations and loading conditions is presented. The wooden boardwalk is currently in the concept design phase and specifics are unavailable at this time. In addition, no borings were drilled in the river, so no subsurface information is available at this time. Geotechnical recommendations for the promenade and the boardwalk will be developed during Phase 2.

Pavements

Pavement sections should be designed per VDOT standard pavement sections for an assumed California Bearing Ratio (CBR) of 3. For pavement sections in existing roadways, the pavement design should match the existing pavement sections. Recommendations for pavement sections should be developed in the next phase of the project after CBR tests have been completed, design traffic loading is available, and pavement cores obtained to confirm the existing pavement sections.

Instrumentation and Monitoring

We recommend that an instrumentation and monitoring program be established during excavation and construction of this project. The program will help monitor horizontal and vertical movements for movements for sensitive structures so that steps can be taken to prevent damage. Potential instrumentation includes building monitoring points, surface settlement points, inclinometers, tiltmeters, crack gauges, strain gauges, and observation wells.

CONSTRUCTION CONSIDERATIONS

Fill Material

When excavation of unsuitable materials is required, it should be performed in a manner to limit disturbance of the underlying suitable material. The excavation should be performed under the observation of the Geotechnical Engineer to evaluate required excavation depths.

Fill Subgrades

Compacted structural fill subgrades should be kept free of ponded water. If springs or other flowing water are present at the compacted structural fill subgrade level, the Contractor should direct water to discharge beyond the fill limits. Recommendations for discharging springs should be provided by the Geotechnical Engineer.

Compacted structural fill subgrades should be free of snow, ice, and frozen soils. If snow, ice, or frozen soils are present at subgrade levels, these materials should be removed as recommended by the Geotechnical Engineer.

Compacted structural fill subgrades should not be steeper than about 4H:1V. If steeper slopes are present, subgrades should be benched to permit placement of horizontal lifts of fill.

Below-Grade Walls

For the installation of permanent secant pile or diaphragm walls, a guide wall is required to provide a template such that the drilled shafts are installed with proper alignment and overlapping, ensuring that a minimum overlap is maintained throughout the entire length of the shaft including the adverse effects of the installation tolerances. The actual overlap should be determined by the wall designer based on the planned installation methods and associated drilling tolerances.

Construction Dewatering

It is our understanding that based on environmental testing results, special environmental measures will be needed during the dewatering process due to the presence of diesel and gasoline produce, arsenic, chromium, mercury, lead, naphthalene, and other potential contaminants. The contractor must be made aware of the soil and groundwater environmental conditions at the site.

If well points are used during the dewatering process, monitoring points/equipment should be installed near adjacent structures to monitor for potential settlement. Construction dewatering systems using well points and recharge wells should be designed by an experienced contractor specializing in these systems.

RECOMMENDATIONS FOR PHASE 2 GEOTECHNICAL INVESTIGATION

After conceptual site plans are finalized, including establishing site grading, lowest level floor elevations, and structural loading, additional subsurface investigations should be performed. Future field investigations should include additional test borings along the proposed bulkhead, in building areas, bioretention basins, and pavement areas. Test borings, combined with other in-situ testing methods, as available and as appropriate, are needed to better define the depth, thickness, and design parameters for the soil strata at actual building and structure locations. Table 5 presents the proposed range of depths for additional borings for the Phase 2 Geotechnical Investigation. The specific number of borings will be determined as the concept design is advanced.

Table 5: Proposed Additional Test Borings for Phase 2 of Geotechnical Investigation

Structure	Barge or Land Boring	Anticipated Range of Boring Depths (ft)
Bulkhead	Barge	100-110
Bulkhead	Land	100
Pump Stations	Land	90-110
24- to 36-inch Diameter Storm Sewer	Land	65-70
18-inch Diameter Storm Sewer	Land	35-45
Cistern	Land	20-25
Bioretention Basins	Land	10-15
BMP Ponds	Land	10-15

Additional laboratory testing should also be performed to characterize the on-site soils and verify their engineering and corrosion potential properties at actual building and structure locations. This will allow the development of recommendations for economical alternatives for foundations as well as other geotechnical-related design issues. Additional laboratory testing for shrink/swell potential of site soils and California Bearing Ratio (CBR) testing is recommended as part of a more detailed Phase 2 Geotechnical Study investigation.

Pavement cores should be obtained to determine the existing pavement condition. We also recommend environmental characterization of sampled soils.

Due to the low to moderate potential for liquefaction at the site, the potential for seismic-induced settlement and lateral spreading should be evaluated during the final design.

Comprehensive geotechnical engineering analysis and reporting is required to provide final foundation recommendations for the final building layouts, floor grades and structural loads. The engineering analysis and report should also include final design recommendations for foundation and floor slab design, lateral earth pressures, foundation subdrainage, earthwork requirements, pavement support, and comments on other geotechnical aspects that should be considered during design and in construction.

LIMITATIONS

We based the analyses and recommendations submitted in this Phase I Geotechnical Engineering Report on the information revealed by our exploration. We attempted to provide for normal contingencies, but the possibility remains that unexpected conditions may be encountered during construction.

This report has been prepared to aid in the evaluation of this site and to assist in the design of the project. It is intended for use concerning this specific project. We based our recommendations on information on the site and proposed construction as described in this report. Substantial changes in loads, locations, or grades should be brought to our attention so we can modify our recommendations as needed. We would appreciate an opportunity to review the plans and specifications as they pertain to the recommendations contained in this report, and to submit our comments to you based on this review.

We have endeavored to complete the services identified herein in a manner consistent with that level of care and skill ordinarily exercised by members of the profession currently practicing in the same locality and under similar conditions as this project. No other representation, express or implied, is included or intended, and no warranty or guarantee is included or intended in this report, or other instrument of service.

Stantec
Alexandria Waterfront Flood Mitigation

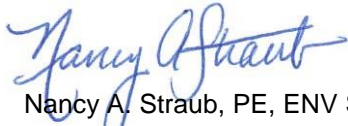
We appreciate the opportunity to be of service for this project. Please call us if you have any questions regarding this report.

Sincerely,

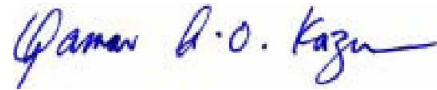
SCHNABEL ENGINEERING, LLC



Tim Hastings, EIT
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Qamar A. O. Kazmi, PE
Principal

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FIGURES

- Figure 1: Site Vicinity Map
- Figure 2: Alexandria Historic Shoreline
- Figure 3: Lateral Earth Pressure Diagram for Design of Below-Grade Walls
- Figure 4: Trapezoidal Lateral Earth Pressure Diagram



**APPROXIMATE
SITE LOCATION**

Sources: Esri, HERE, DeLorme, USGS, Intermap, INCREMENT P, NRCan, Esri Japan, METI, Esri China (Hong Kong), Esri Korea, Esri (Thailand), MapmyIndia, NGCC, © OpenStreetMap contributors, and the GIS User Community
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NOT TO SCALE



**WATERFRONT FLOOD MITIGATION
ALEXANDRIA , VIRGINIA**

**SITE VICINITY
MAP**

PROJECT NO. 16C12012.00

FIGURE 1

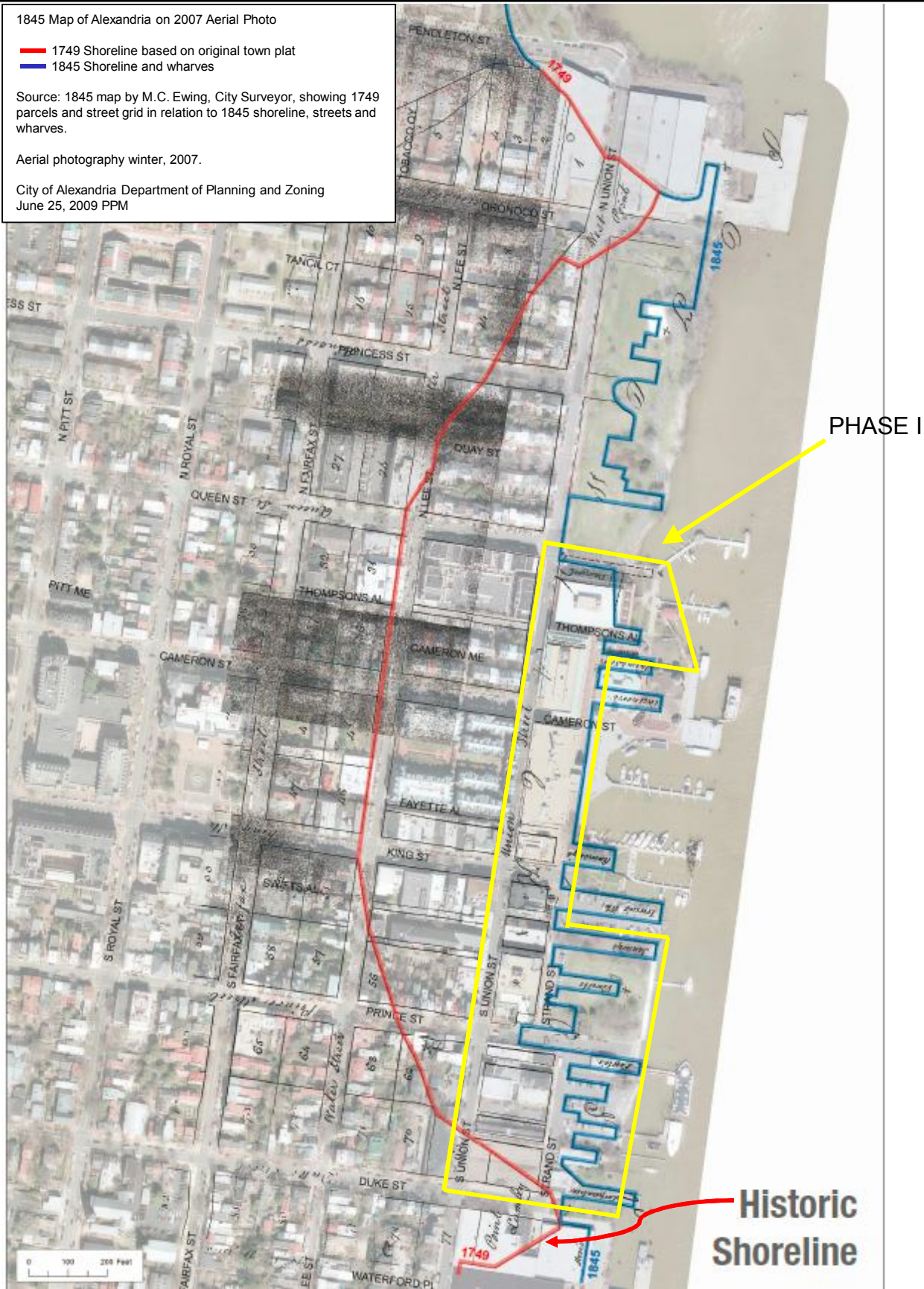
1845 Map of Alexandria on 2007 Aerial Photo

- 1749 Shoreline based on original town plat
- 1845 Shoreline and wharves

Source: 1845 map by M.C. Ewing, City Surveyor, showing 1749 parcels and street grid in relation to 1845 shoreline, streets and wharves.

Aerial photography winter, 2007.

City of Alexandria Department of Planning and Zoning
June 25, 2009 PPM



REFERENCE: CITY OF ALEXANDRIA DEPARTMENT OF PLANNING AND ZONING, JUNE 25,2009 PPM

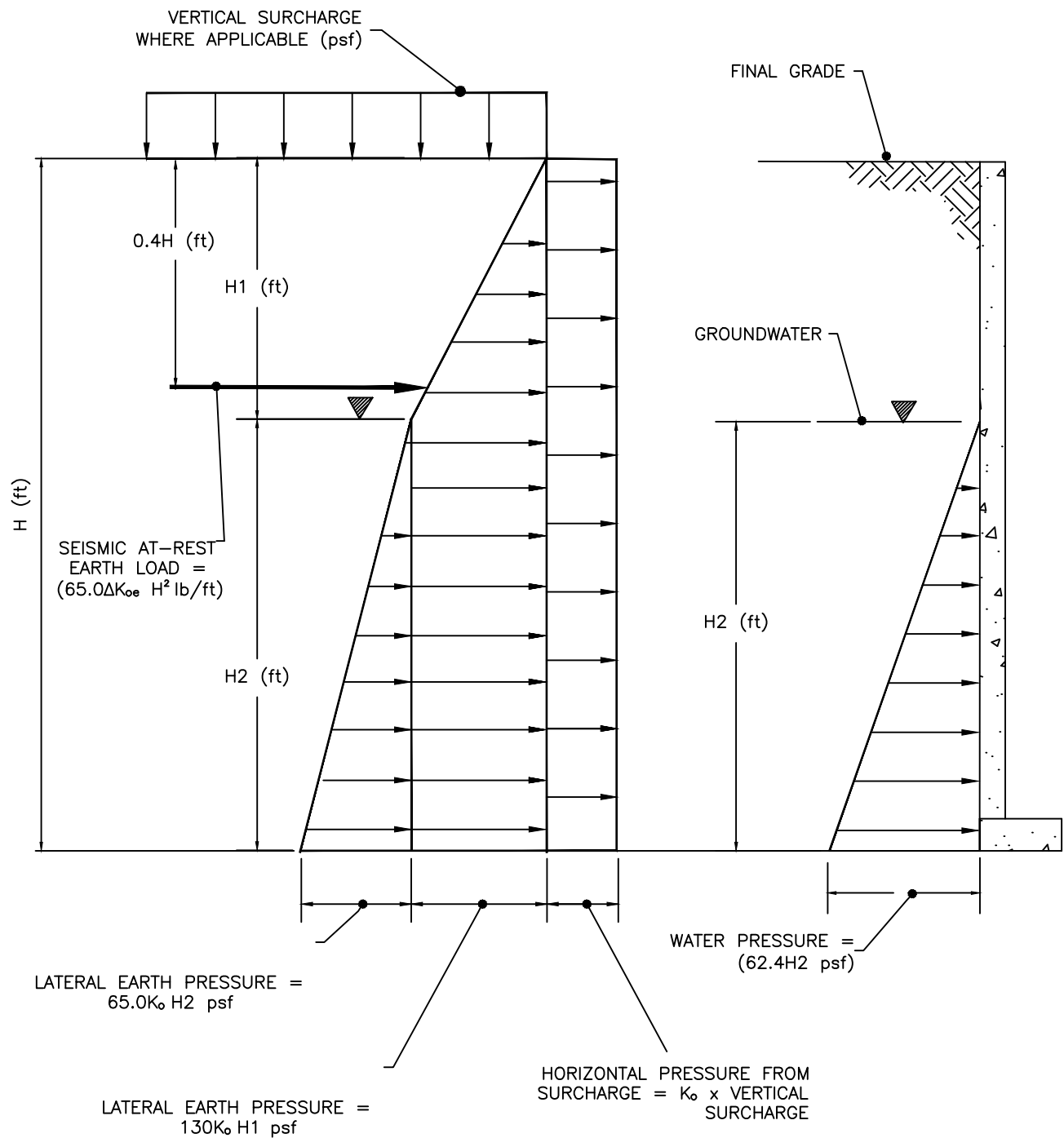


**WATERFRONT FLOOD MITIGATION
ALEXANDRIA, VIRGINIA**

PROJECT NO. 16C12012.00

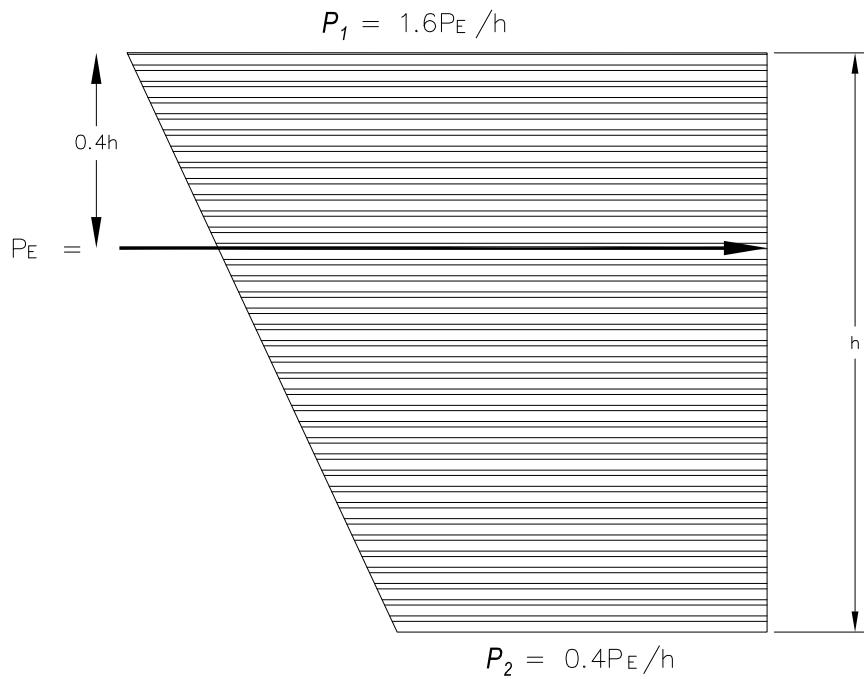
**ALEXANDRIA HISTORIC
SHORELINE**

FIGURE 2



EARTH PRESSURE NOTES

1. WHERE K_0 IS THE AT-REST LATERAL EARTH PRESSURE COEFFICIENT = 0.47
2. WHERE ΔK_{0e} IS THE ADDITIVE DYNAMIC AT-REST LATERAL EARTH PRESSURE COEFFICIENT = 0.11
3. SEE REPORT FOR BACKFILL MATERIAL REQUIREMENTS.



P_E = ADDITIVE SEISMIC FORCE
(FROM FIGURE 3)

h = RETAINED HEIGHT OF SOIL
(SEE FIGURE 3)

P_1, P_2 = MAGNITUDE OF PRESSURE
AT TOP AND BOTTOM,
RESPECTIVELY, OF THE
RETAINED SOIL HEIGHT

NOTE

- PLEASE REFER TO FIGURE 3 FOR DETERMINATION OF ADDITIVE SEISMIC FORCE, P_E .

NOT TO SCALE



ALEXANDRIA WATERFRONT FLOOD
MITIGATION
ALEXANDRIA, VIRGINIA
PROJECT NO. 16C12012

TRAPEZOIDAL SEISMIC LATERAL
LOADING FOR BELOW-GRADE AND
RETAINING WALLS

FIGURE 4