Boston Coastal Storm Risk Management Study

Draft Integrated Feasibility Report and Programmatic Environmental Assessment

Photos: Top and Middle- 2018 Nor'easter courtesy of businessinsider.com
Bottom- Flooding beneath Evelyn Moakley Bridge. Courtesy of Alison Brizius, December 2022

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USACE New England District | City of Boston

US Army Corps of Engineers

Contents

1 INTRODUCTION

1.1 Background

This geotechnical engineering appendix was prepared in support of the Boston Coastal Storm Risk Management Study Draft Integrated Feasibility Report which was prepared to investigate potential responses to existing and increasing vulnerability to coastal flooding and storm damage risks within the City of Boston. The study area comprised the City of Boston, Massachusetts, with its approximate 47 miles of coastline, divided into the neighborhoods of Charlestown, Dorchester, Downtown, East Boston, and South Boston.

The study is structured in two phases. The first is an initial Assessment and Evaluation phase that advanced typical designs for the entire system to approximately 10% design maturity to select a preferred alternative. The Assessment and Evaluation phase is documented in this report. After the preferred alternative is selected, the second phase will provide Feasibility Level Analysis and Design to advance selected portions of the project to approximately 35% design maturity.

Due to the availability of a large volume of existing subsurface information and the extensive footprint of the potential project, no new subsurface investigations were carried out during the Assessment and Evaluation phase. This appendix compiles some of the available information to develop an understanding of likely subsurface conditions for each of the five neighborhoods. Feasibility Level Analysis and Design to 35% design maturity, as well as Pre-construction Engineering Design to plans and specifications for bid will require site specific information derived from site investigations for design.

1.2 Scope of Geotechnical Study

The scope of the Assessment and Evaluation phase geotechnical study comprised:

A. Collection of existing subsurface information
B. Geotechnical evaluation of subsurface information
C. Conceptual geotechnical designs at approximately 10% design maturity

This appendix summarizes available geotechnical information and provides the basis of design for typical sections used to evaluate alternatives at a 10% level of design maturity. The appendix begins with a qualitative understanding of subsurface conditions and geotechnical considerations at the Boston coastline in Section 2. Section 3 documents the analysis of over 270 existing boreholes that were located and digitized in support of this study. A neighborhood-wide approach was applied to develop broad understanding of the varying stratigraphy throughout the City of Boston and to develop typical soil profiles.

The typical sections were then used to develop typical design sections. The typical sections and rationale for each design element are provided in Section 4. Quantitative typical section design focused primarily on foundation design due to the cost magnitude and potential cost risk associated with foundation designs. Other design elements of lower risk were provided based on experience and engineering judgement to capture project scope with limited detail at a 10% design maturity. This report documents the rationale for the selected design of each design element. Design risks are identified in this report and documented in the project’s Electronic Risk Register (ERR) and the Abbreviated Risk Assessment (ARA) supporting the cost estimate.

2 EXISTING GEOTECHNICAL INFORMATION

2.1 Regional Geology

The City of Boston is located within the Northeast Coastal Zone of the United States which extends from northeastern New Jersey to the southern coast of Maine. The city itself is located within the Boston Basin subregion which extents along the coast of Massachusetts from Cohasset to north of Beverly and as far inland as the Framingham area. The Boston Basin is ringed by low hills to the south and an extensive escarpment to the west and north. Topography within the basin is low and rolling glacial landforms with till-covered bedrock hills (Griffith, 2009).

2.2 Local Geology

A typical soil profile was developed based on an extensive literature review as well as existing boring logs. This profile is intended to provide an overview of typical strata encountered within the project area. Representative profiles for each neighborhood are provided in Section 3.2.2, showing the variation encountered in different regions of the City. In general, soil profiles near the Boston coastline will consist of Fill placed over the existing Organic Marsh Sediments. Marshes were typically formed on the Marine Outwash deposit commonly known as the Boston Blue Clay, but in some areas a layer of sand and gravel outwash is found in place of the Organic Marsh sediments and the clay. The Boston Blue Clay is extensive throughout much of the City and is commonly very thick. Below the Blue Clay are glacial outwash and Glacial Till deposits over the Bedrock. Each of these are discussed briefly below.

2.2.1 Fill

The fill soils may consist of reworked glacial deposits as existing hills were leveled during development of the city, or marsh sediments from nearby harbor dredging. Waste and construction debris, especially ashes and cinders, are commonly found in these reworked soils (Ty, 1987). The nature of the fill and procedures by which it was placed vary considerably, resulting in a varied and mostly unsuitable subsurface foundation for major structures (Brankman and Baise, 2008; Johnson, 1989). In this report, fill is broadly categorized as either granular, clay, or unknown depending upon the information available. Granular fill is expected to be relatively free-draining based upon the descriptions provided. Any fills that identify clay as a constituent have been categorized as clay fills and should be expected to have relatively low permeability. Limited hydraulic conductivity data was included in the historic data reviewed.

2.2.2 Organic Marsh Sediments, River Sediment, and Outwash Sand and Gravel

The organic marsh sediments and outwash sands are normally consolidated and often have high water and organic content. The result is a compressible layer of low bearing capacity (Johnson, 1989). Locally the organic marsh sediments may be intermingled with fluvial delta deposits of sand and gravel. The organic silt and outwash sands may be presented together, or one or both may be absent (Hawkes, 1987). The sands and gravels can form a suitable bearing layer for shallow foundations if suitably thick and if the structural loads are fairly small.

2.2.3 Marine Sediments (Boston Blue Clay)

Marine deposits (Boston Blue Clay) are typically found below fill and marsh/fluvial deposits near the shoreline throughout the City of Boston. The Boston Blue Clay layer is normally consolidated, except for an upper crust layer which is present in many locations. Where present, the upper crust is stiff, oxidized, and over consolidated and provides a competent bearing layer for some structures. At some locations sand/silt layers are present within the clay stratum.

2.2.4 Glacial Outwash and Glacial Till

Below the Boston Blue Clay are complex glacial and glaciofluvial deposits. The deposits are varied and appear to be the result of multiple glacier advances and retreats. The repeated progression and regression from multiple directions results in a mix of glacial tills and interbedded outwash sands and gravels of varying thickness. The clayey tills are typically oxidized, heavily overconsolidated and contain cobbles and boulders.

2.2.5 Bedrock

Rock of metamorphic and sedimentary origin typically underlies the glacial deposits. This is commonly known as the Cambridge Argillite (Zen, 83); a shale or siltstone whose age has been the topic of debate. While the Argillite is typically hard and lightly fractured, it often exhibits an upper crust of weathered and decomposed rock, with a gradual transition to the hard rock below.

2.3 Typical Soil Properties

Centuries of construction and many decades of engineering and research have resulted in a wealth of available information on the behavior of subsoils in the vicinity of Boston. Typical soil parameters have been presented in numerous publications. Significant geotechnical work was conducted specifically for the alignment of the Central Artery Tunnel (CA/T) “Big Dig” project, in the 1989-2002 period. This information has been retrieved and will be consulted with additional data gathered during future design phases. The following summary is compiled from Cullen et al. (1982) and Johnson (1989) who provide general guidance regarding soil properties throughout the City of Boston. Key engineering parameters for the typical layers described in Section 2.2 are provided in Table 1 below. Where multiple ranges are provided, the common range is listed, and the full range is given in parentheses.

2.4 Groundwater

Much of the project is located at or near the coastline where the tidal ocean and groundwater systems meet. NOAA’s tide station at Boston experiences about 10 feet of tide range, which represents a significant change to hydrostatic pressures at the boundary. The presence of seawalls, surface pavement, and drainage systems contributes to a complex groundwater regime. Designs must consider groundwater levels and gradients during storm and non-storm conditions. Designs must be considered both with and without the temporary hydrostatic loads. Groundwater flows must be understood in terms of tidal and seasonal variations, which must be considered in design. For example, complete cutoffs may require groundwater management on the protected side.

Groundwater wells monitored by the Boston Groundwater Trust suggest that groundwater levels near the shoreline are generally at or above mid-tide range, and at some locations at or above high tide levels. Local presence of pervious fill and natural sands and gravels, as well as presence of utility trenches and infrastructure cause some localities inland of the ocean to exhibit tidal influence in the groundwater.

Table 1: Typical Soil Parameters

Soil Type	Saturated Unit Weight (pcf)	Effective Friction Angle (deg)	Undrained Shear Strength (psf)	Allowable Bearing Pressure[1] (tsf)
Fill	110–120 (100–125)	28–33	—	1.0
Organic/Marsh Silts	90–110	—	300–800	—
Outwash Sand and Gravel	110–125 (110–135)	32–36	—	2.0–2.5 (1.0–5.0)
Boston Blue Clay (Upper)	115–120 (114–125)	24	600–1,200	1.5–2.0 (1.0–4.0)
Boston Blue Clay (Lower)	113–118	24	400–800	0.5–1.0
Glacial Till	125–140	36	2,000–8,000	4.0–5.0 (3.0–10.0)
Bedrock	165–170	45	—	10.0–20.0

[1] Presumptive values. Bearing capacity is dependent on several factors.

2.5 Existing Structures

Much of the proposed alignment, particularly in the Downtown neighborhood follows the existing harborwalk park, with paved walkways and adjacent seawall bulkhead structures. Numerous drainage outfalls across a variety of public and private owners are present. These represent perforations in the coastal defense, and it will be essential to manage the potential intrusion at every outfall. Interior drainage must be managed to prevent interior flooding. Landward of the Harborwalk are existing buildings of various age, construction, and sea level rise resiliency. Multiple tunnels operated by several different agencies pass below the project design elevation.

The structures are briefly discussed in each subsection below.

2.5.1 Bulkheads

Much of the project shoreline consists of seawalls. These are often constructed of granite block founded on timber piles. This type of construction can be centuries old and is prevalent in the downtown neighborhood. Steel construction, and occasionally concrete, have been preferred since the early 1900s and are prevalent outside of downtown. Examples of each type of construction are shown in Figure 1 below.

Granite block structures are typically dry fit construction, which provides limited hydraulic sealing. Over time some of these have settled, shifted, and experienced loss of retained soil. This construction is often very old, so design and construction details are unknown. It is typical engineering practice to avoid applying additional loads to these structures or within the zone of influence landward of the wall. In the recent reconstruction at Langone Park, for example, new coastal resilience features were designed so that they do not interact with the existing granite block seawall. Thus, the upper structure was cantilevered over the wall and foundations were supported below the wall on deep piles. A schematic cross section from an informational panel at the park is shown in Figure 1 below.

Steel and concrete panel seawall structures came to prominence with the advent of high strength steel and modern geotechnical methods. The interlock mechanism of steel panels results in minimal seepage through the wall. Concrete panels can be sealed to a similar degree as steel when properly constructed; however, small differential movements can open voids that provide direct flow pathways, making older concrete walls more leaky than comparable steel structures. Both steel and concrete seawalls are easily adapted to provide perforations for outfalls. These perforations are also prone to leakage at the seal and could provide a direct hydraulic connection to the protected area. These potential seepage pathways must be addressed.

Figure 1: Schematic cross section from an informational panel at Langone Park

2.5.2 Outfalls and Interior Drainage

Major outfalls owned by the Boston Sewer and Water Authority and MassDOT are readily identifiable. However, there are also likely many private outfalls that will require identification and assessment.

Every perforation of the existing seawall structures will need to be documented and suitable mitigation determined. Design considerations may include:

- Back pressurization of drainage lines,
- Potential for artesian head at storm drains, and
- Susceptibility to leakage at seawall gaps or pipe damage on the landward side.

Potential management of outfalls may include some combination of:

- Check valves,
- Gate structures,
- Gap sealing, and/or
- Abandonment.

These systems must work in conjunction with plans for interior drainage. The extensive hardened surfaces within the protected area will produce near 100% runoff of precipitation. The runoff will have to be managed until the storm surge recedes.

2.5.3 Existing Buildings

There are numerous instances where existing buildings are located in close proximity to proposed alignments. This may present potential conflicts with foundation structures, basements, or other facilities. Structures of historic and/or cultural significance will likely need to be preserved. However, structures that are not in a protected category may present an opportunity to incorporate the existing structure as a part of the coastal defense. In this approach, the project would fill gaps between existing buildings and provide waterproofing and reinforcement to the existing building as necessary. This was a common practice during the river flood risk management projects constructed in New England from the 1930s through the 1970s. Existing mill buildings were often built up to the water's edge. Floodwalls or levees were connected to each end of the structure, and the structure itself was floodproofed and reinforced as necessary. An example below (Figure 2) from West Warren, Massachusetts shows a floodwall meeting a previously existing mill building. Inside the structure windows were bricked-in, a reinforced concrete abutment was provided, and floor drains installed to relieve hydrostatic pressures. A similar approach could be used at some locations to reduce the Boston CSRM project's construction footprint.

Figure 2: Example of a flood wall integrated with an existing building

2.5.4 Tunnels

Multiple transportation tunnels cross the proposed alignments and/or link neighborhoods together. Tunnels identified in proximity to proposed alignments are summarized in the following Table 2 by location. Though the tunnels are typically deep in the vicinity of the shoreline, the elevation of the tunnels at the project crossings should be verified during future design.

2.6 Existing Utilities

Utility data has been compiled by USACE to identify major corridors and facilities. These are incorporated in the Feasibility Study. Some of the information was obtained via a non-disclosure agreement and is not presented in the report but are retained in project files.

Underground utilities can present significant challenges to this project. All effort should be made to locate and avoid existing utilities.

Table 2:Tunnels Identified in Proximity to Proposed Alignments

Location	Managed By	Name
Downtown — Charlestown	MBTA	Orange Line
Downtown — Charlestown	MassDOT	Zakim/Bunker Hill Bridge (I-93)
Downtown North End	MassDOT	Central Artery Tunnel (I-93)
Downtown — East Boston	MassDOT	Callahan/Sumner Tunnels
Downtown — East Boston	MBTA	Blue Line
Downtown Waterfront	MassDOT	Central Artery Tunnel
Downtown — South Boston	MBTA	Red Line
South Boston — East Boston	MassDOT	Ted Williams Tunnels (I-90)

3 GEOTECHNICAL ASPECTS OF STUDY ALTERNATIVES

3.1 General Considerations

Design will be conducted in accordance with standard engineering practice and applicable USACE regulations and guidance. Key Engineering Regulations and Engineering Manuals, with significant geotechnical implications are indicated below:

- ER 1110-2-1150: Engineering and Design for Civil Works Projects (8/31/1999)
- EM 1110-2-1913: Design and Construction of Levees (4/30/2000)
- EM 1110-2-2502: Flood Walls and other Hydraulic Retaining Walls (8/1/2022)
- EM 1110-2-1901: Seepage Analysis and Control for Dams (9/30/1986, revised 4/30/1993)
- EM 1110-2-1902: Slope Stability (10/31/2003)
- EM 1110-1-1804: Geotechnical Investigations (1/1/2001)
- ECB 2023-9 – Civil Works Design Milestone Checklist

Geotechnical design will be required for vertical and horizontal bearing capacity, settlement, seepage, and slope stability. Several key geotechnical concerns will apply throughout the project. These include but are not limited to:

- Compressible soils and settlement potential.
- Control of seepage through non-engineered, randomly placed fill.
- Proximity of adjacent buildings and structures.
- Need for erosion protection.

3.2 Subsurface Profiles

3.2.1 Data Gathering and Interpretation

Existing borehole information was gathered by USACE from a variety of sources, primarily the City of Boston, the Boston Inspectional Services Database, and data compilations from the Boston Society of Civil Engineers (BSCE) journals published between the 1940s and the 1970s. The BSCE logs provided an extensive database of general written soil type descriptions.

Locations within proximity of proposed alignments were identified and digitized using Open Ground Cloud (OGC) database software. Digitization of soil type required some interpretation but is generally consistent with the Unified Soil Classification System, with emphasis on soil type and incorporating descriptors indicative of physical properties and/or geologic formation when available. Test results indicative of strength properties, such as Standard Penetration Tests (SPT) and Pocket Penetrometer tests were entered in the database when available.

The resulting database consists of over 270 borehole logs of varying depth and detail. The extent of coverage is illustrated in Figure 3, where each blue marker indicates one boring location. The database was consulted throughout initial formulation to understand soil conditions as various alignments were considered. It provides a broad statistical basis for this report that will be used throughout future design and construction.

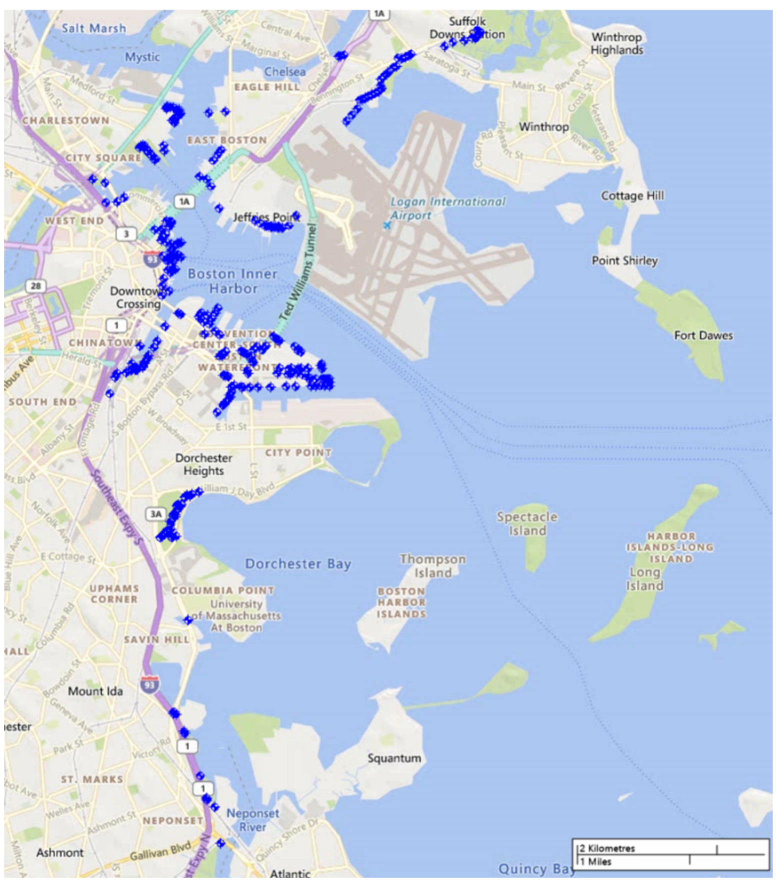

Figure 3: The Extent of Borehole Coverage Throughout the Project.

Due to the limited and generic soil descriptions provided in the historic information, the USCS symbols provided on the cross sections do not represent determination of USCS soil types. Rather, these symbols were generally related to the soils described as given in Table 3 below.

Table 3 Soil Descriptions

Log Description	USCS Symbol Assigned
Gravel	GP
Sand	SP
Gravel and Sand	GW
Sand and Gravel	SW
Gravel, Sand, and Silt	GW-GM
Gravel, Sand, Silt, and Clay	GW-GC
Sand and Silt	SM
Sand, Silt and Clay	SC
Clay (Hard and/or Yellow)	CL (Boston Blue — Upper)
Clay (Soft and/or Blue)	CH (Boston Blue — Lower)

The logs provide limited context, so it was often necessary to interpret based on the soil types identified and the relative stratigraphy. General guidelines used for this interpretation are illustrated below.

3.2.1.1 Fill

Fill was readily identified as fill in most logs. Sometimes this was the only indicator for this stratum. If other soil types were provided, they are logged accordingly and illustrated on the cross sections by a single descriptor in the middle of the stratum.

The descriptor "Fill" was also applied to upper layers that contained descriptors of artificial materials (such as bricks, cinders, lumber) since these are indicative of artificial placement or reworking.

3.2.1.2 Marsh, Riverine, and Outwash Sediments

When located near the surface, below fill, or above clay, soils with organic material were identified as marsh sediments. Sands, gravels, and silts that occupied a similar position in the stratigraphy that were not described as organic were identified as riverine or outwash sediments based upon grain size. Coarser or mixed sediments were identified as outwash, whereas more uniform sands were identified as riverine.

3.2.1.3 Marine Sediments (Boston Blue Clay)

Boston blue clay was identified as a thick clay or sand/silt/clay deposit, hard near the upper surface and often softer at depth. Sand and silt seams were identified as blue clay when bound on both sides by clay strata. The Boston Blue Clay was relatively easily identified because it is a very thick layer, predominantly identified as clay, or occasionally silt. In some cases, stiff clays at the base were difficult to interpret as this may have been a blue clay over till interface. The stiff layer was identified as till where indicators of till were recorded, such as cobbles or a well graded grain size distribution. The blue clay description was continued if no indicators of till were identified on the log.

3.2.1.4 Glacial Outwash and Glacial Till

In this report, these deposits were identified as till or outwash depending upon the descriptors provided in the historic logs. Till or till-type descriptors, along with hard or very hard indications on the boring logs were identified as till. More uniform grain size and/or less-hard deposits were identified as outwash.

3.2.1.5 Bedrock

Bedrock is readily identified in most logs and by most drilling methods. Many test holes were terminated before reaching bedrock. Hardpan was noted in several logs, and lacking other information, was logged as bedrock if depths were similar to other information in the area. It is possible that the bedrock identified in these logs could be hardpan, till, boulders, or weathered bedrock regolith.

3.2.2 Typical Soil Profiles by Neighborhood

For simplicity, typical profiles have been developed for each of the five neighborhoods. The number of borehole locations in each neighborhood is given in Table 4 along with the total length of potential alignment. The number of boreholes per mile is provided as an indicator of data density, however the boreholes are not evenly spaced throughout the alignments. Charlestown and East Boston had extensive areas where no existing information was retrieved. Similarly, data was limited along an extensive segment at Harbor Point in Dorchester.

Table 4: Number of Boreholes in Each Neighborhood

Neighborhood	Number of Locations	Miles of Alignment	Average Boreholes/Mile
Charlestown	29	2.1	14
Dorchester	38	2.8	14
Downtown	42	2.6	16
East Boston	61	4.7	13
South Boston	113	4.1	28

Each neighborhood was evaluated to determine a typical pattern of geology, using the groups identified in Section 2.2. Each typical profile is summarized graphically and a brief explanation provided in the following subsections. The mean thickness of each layer is provided. Soil profiles indicate the average depth below ground surface of recorded transitions. Variability is identified by a typical range of depths, which represent one standard deviation (σ) to each side of the mean. This range represents approximately 67% (or the middle two-thirds) of the expected values, leaving roughly about 16.5% of samples higher and 16.5% of samples lower.

3.2.2.1 Charlestown

A typical Charlestown soil profile consists of about 15 feet of Fill over a thin layer of Marsh and Riverine Sediments (Table 5). Below the Marsh and Riverine Sediments is the Boston Blue Clay layer which averaged 36 feet in thickness where the bottom was identified. It should be noted that the top of the Clay ranges from 8 feet to 26 feet below ground surface for the middle two thirds of data. The Lower Outwash and Till layers were identified at 53 feet below surface on average.

There is a noted data gap at the alignment along the Mystic River at Schrafft's Center where no existing information was retrieved.

Table 5: Typical Charlestown Soil Profile.

Stratum	Log Description	Interpreted Geology	Stratum Thickness (ft)	Average Depth to Bottom (ft)	Typical Depth Range (ft)
1	Fill	Fill	15	15	8 – 22
2	Sand, Silt, and/or Organic	Marsh and Riverine Deposits	2	17	8 – 26
3	Clay	Boston Blue Clay (BBC)	36	53	26 – 80
4	Gravel/Till	Glacial Outwash and Till	17	70 (End of Boring)	N/A

3.2.2.2 Dorchester

A typical Dorchester soil profile consists of about 10 feet of Fill over a thick layer of Marsh and Riverine sediments mixed with Upper Outwash Sediments (

Table 6). When taken together, the Marsh, Riverine and Outwash are on average 48 feet thick before encountering the Boston Blue Clay at 58 feet below ground, although this ranged from 36 to 80 feet for the middle two thirds of data. Dorchester borings typically ended in the Clay at an average of about 78 feet below ground surface.

Dorchester has the shortest total distance of proposed alignment due to a generally higher topography that limits potential water encroachment over land and minimal alternative alignments to consider. Dorchester includes a noted data gap at the Harbor Point neighborhood, where no existing information could be retrieved. Given the relatively short alignments scattered over a large area of variable geology, the typical profile provides general guidance but is not applicable at any specific location.

Table 6: Typical Dorchester Soil Profile.

Stratum	Log Description	Interpreted Geology	Stratum Thickness (ft)	Avg. Depth to Bottom (ft)	Typical Depth Range (ft)
1	Fill	Fill	10	10	4 – 16
2	Silt and Clay (occasional Gravel/Organic)	Marsh, Riverine, and Outwash	48	58	36 – 80
3	Clay	Boston Blue Clay	20	78 (End of Boring)	N/A

3.2.2.3 Downtown

A typical Downtown soil profile consists of 16 feet of Fill over a thin layer of Marsh and Riverine sediments (Table 7). The Boston Blue Clay is about 21 feet below ground, although this ranged from 9 to 32 feet for the middle two thirds of data. The Boston Blue Clay layer is 32 feet thick on average, though this varies considerably. The upper surface of the Lower Outwash and Till can be 31 to 74 feet below ground in the middle two thirds range of data. Downtown borings typically ended in the Boston Blue Clay or the Outwash and Till at typical depth of 73 feet.

Table 7: Typical Downtown Soil Profile.

Stratum	Log Description	Interpreted Geology	Stratum Thickness (ft)	Avg. Depth to Bottom (ft)	Typical Depth Range (ft)
1	Fill	Fill	16	16	8 – 24
2	Sand, Silt, and/or Organics	Marsh and Riverine	5	21	9 – 32
3	Clay	Boston Blue Clay	32	53	31 – 74
4	Sand and Boulders (Till)	Outwash and Till	20	73 (End of Boring)	N/A

3.2.2.4 East Boston

A typical East Boston soil profile consists of 16 feet of Fill over a thin layer of Marsh and Riverine sediments (Table 8). The Boston Blue Clay is about 16 feet below ground, although this ranges from 6 to 26 feet for the middle two thirds of data. East Boston borings typically end in the Boston Blue Clay at about 37 feet. East Boston borings frequently terminated at hardpan or rock. The hardness of this material might be a relative function of the drilling equipment available at the time of the potentially very old drilling. It is also likely that bedrock and hard till are present.

The depth to end of boring is much shallower in East Boston than the other neighborhoods, which is consistent with the presence of shallow hard till and bedrock.

Table 8: Typical East Boston Soil Profile.

Stratum	Log Description	Interpreted Geology	Stratum Thickness (ft)	Avg. Depth to Bottom (ft)	Typical Depth Range (ft)
1	Fill	Fill	12	12	6 – 18
2	Sand, Silt, and/or Organics	Marsh and Riverine	4	16	6 – 26
3	Clay	Boston Blue Clay	21	37 (End of Boring)	N/A

3.2.2.5 South Boston

A typical South Boston soil profile consists of 20 feet of Fill, the greatest in the project area with a range of 9 to 32 feet for the middle two-thirds of data (Table 9). Below the Fill is a layer of Marsh and Riverine sediments at about 8 feet thick. The Boston Blue Clay is about 28 feet below ground, although this ranges from 16 to 40 feet for the middle two thirds of data. South Boston borings typically end in the Boston Blue Clay at a depth of about 63 feet.

Table 9: Typical South Boston Soil Profile.

Stratum	Log Description	Interpreted Geology	Stratum Thickness (ft)	Avg. Depth to Bottom (ft)	Typical Depth Range (ft)
1	Fill	Fill	20	20	9 – 32
2	Sand and Silt	Marsh and Riverine	8	28	16 – 40
3	Clay	Boston Blue Clay	35	63 (End of Boring)	N/A

3.2.3 Cross Section Development

After the information was entered into the database, the OGC software was used to produce cross sections throughout the proposed alignments, which were manually refined. Typical cross sections for all areas considered are presented in Figure 4) Stick log cross sections are provided as an attachment to this Appendix. These illustrate the extent of coverage along each alignment. The alignments compiled the existing grades from the Central Eastern Massachusetts LiDAR data set collected by the US

Geological Survey (USGS) in 2021 with borehole logs referenced to NAVD88. Some logs begin above or below the existing grade. Possible causes for this have been considered, including:

- age of some borehole logs and uncertainties regarding datum,
- continued development and redevelopment throughout the City of Boston (excavation or fill),
- smoothing of the topographic data,
- offset from the alignment to the profile baseline (in the third dimension).

It was decided to display the logs based on the recorded top of hole elevations regardless of the relative position of the topography in order to remain consistent with the recorded data. Interpretation could then be made from the overall context. In some cases, no surface elevation is given on the logs. These logs were incorporated if they were generally consistent with adjacent data. Logs that could not be reconciled were not included.

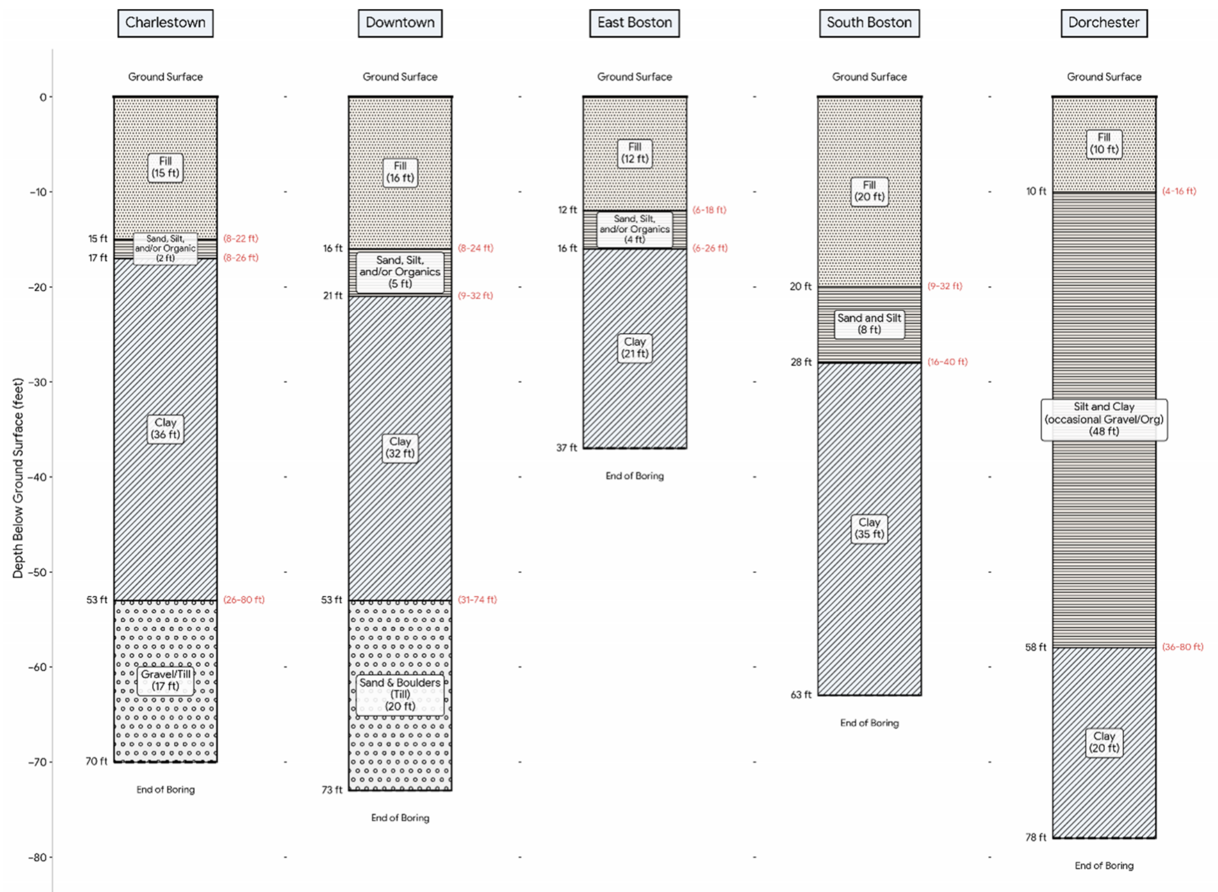

Figure 4: Typical subsurface stratigraphy for the five areas in Boston (produced using GenAI with the tabular information summarized in this document.)

3.3 Seismicity and Liquefaction

Liquefaction of soils occurs when saturated cohesionless soils experience a transient loss of shear strength under dynamic loading or seismic vibration. The dynamic loading or seismic vibration generates excess pore water pressure in the skeleton of the soil which reduces or eliminates the effective confining stress and internal friction, resulting in loss of shear strength and bearing capacity. This hazard most commonly exists in loose, poorly graded sands associated with aeolian or fluvial deposits, as well as artificial fills that have not been adequately compacted. Liquefaction requires several conditions to occur simultaneously:

1. Soil must be saturated.
2. Soil must have susceptible fabric (relative density and predominantly cohesionless or low plasticity granular material).
3. The site must be subjected to cyclic loading (e.g. seismic shaking) of sufficient intensity and duration to generate high pore water pressures.

Boston is located in an area with a 5 to 25% chance of Modified Mercalli Intensity VI (MMI) or greater earthquake events in 100 years. Probable peak ground accelerations (PGA) for a 2,450 year return period (2% chance of occurrence in 50 years) are approximately 0.23g per the USGS dynamic hazard curves based on the Conterminous U.S. 2023 National Hazard Maps R2. Research on behalf of the USGS (Brankman and Baise, 2008) investigated over 10,000 soil samples at over 1,800 locations throughout the Boston area and determined that approximately 7.5% of samples in each of the fill and marsh deposits were susceptible to liquefaction. Nearly 30% of locations with artificial fill had at least one sample susceptible to liquefaction under design earthquake conditions and 22% of locations with Marsh deposits. This led the authors to identify the fill and marsh deposits as highly susceptible to liquefaction. However, because these sediments exhibit extreme spatial and compositional heterogeneity, liquefaction potential is highly variable and must be evaluated on a site-specific basis. The deeper Boston Blue Clay and fluvioglacial/till soils are not susceptible to liquefaction.

4 Preliminary Geotechnical Designs for Assessment and Evaluation (10% Design Maturity)

4.1 Introduction

Preliminary geotechnical designs were prepared to approximately a 10% level of maturity to support Class IV cost estimates. Geotechnical consultation and design was provided for floodwall and levee cross sections throughout the city. The typical soil profiles developed from existing data discussed in Section 3.2.2 were used to develop neighborhood specific recommendations that incorporated variability in stratigraphy and soil strength parameters.

Engineering judgement was used to manage design risk at the 10% level through a combination of calculations, strategic choices, and assumptions. Existing data were gathered, and analyses were performed to determine and/or confirm significant components or uncertain assumptions. For example, pile axial capacity is a significant component of the wall design and so was determined by calculation. Pile lateral resistance was confirmed by a few calculations. Seepage, settlement, and global stability considerations are addressed by application of broad engineering principles and engineering experience to develop the scope of the project to approximately a 10% design maturity.

For simplicity, three typical sections were prepared for arbitrary pre-determined stickup heights of 3 feet, 6 feet, and 9 feet. In this way a suitable typical design could be applied based on location specific topography and geometry. This strategic choice allowed assumptions to be made regarding suitability of some seepage, global stability, and settlement considerations.

Geotechnical design for wall segments was primarily focused on pile design. Levee designs are primarily geometric, with some back-calculating of permeabilities for seepage control through the structure itself. Settlement was not investigated for preliminary designs primarily due to the site-specific nature of these calculations and variability in tolerable limits with respect to subsurface utilities. Settlement of wall segments is anticipated to be controlled by the deep foundations due to the relatively low stresses applied but must be determined thoroughly during future design phases. Settlement of levee segments will be highly dependent on location-specific stratigraphy, site preparation, and applied surcharge. Settlement may be a significant design consideration at some levee locations but insignificant at others. Levee settlement will require thorough design calculations during future design phases.

The significance of geologic variability along miles of alignment scattered throughout a major US metropolitan area with over 300 years of urban modification cannot be understated. Thorough investigations by qualified geotechnical professionals are essential to engineering reliable coastal storm risk management systems. Detailed subsurface investigations and characterization will be required for all proposed alignments throughout the City of Boston.

Preliminary designs for Walls and Levees at approximately 10% design maturity are presented in the following sections. Strength, stability, seepage, settlement and seismic questions are addressed with calculations and/or assumptions and supporting rationale.

4.2 Wall Designs

Conventional T-Wall designs were prepared by the structural engineer based upon anticipated hydrostatic and wave forces, in addition to self-weight. Due to the loose and variable nature of the fill stratum and the potential high compressibility of the marsh and riverine stratum, deep foundation piles were selected for all preliminary wall foundation designs. This is considered conservative from a cost perspective as some locations may prove suitable for shallow foundations and/or ground improvement with shallow foundations during future design phases, which would reduce cost. A robust pile system was selected and applied throughout the project. This is also conservative from a cost perspective and presents an opportunity for smaller, lighter piles to be substituted on a case-by-case basis during future design phases.

Typical floodwall cross sections with stickup heights of three feet, six feet, and nine feet were developed assuming construction on land. In some areas, the floodwall is to be constructed as an over-water pier-type structure parallel to shore. Typical sections were prepared for the same three stickup elevations for construction over water, with three (3) feet added to the top of pile based on keeping the top of the foundation approximately at grade. Schematic cross sections are shown in Figure 5below.

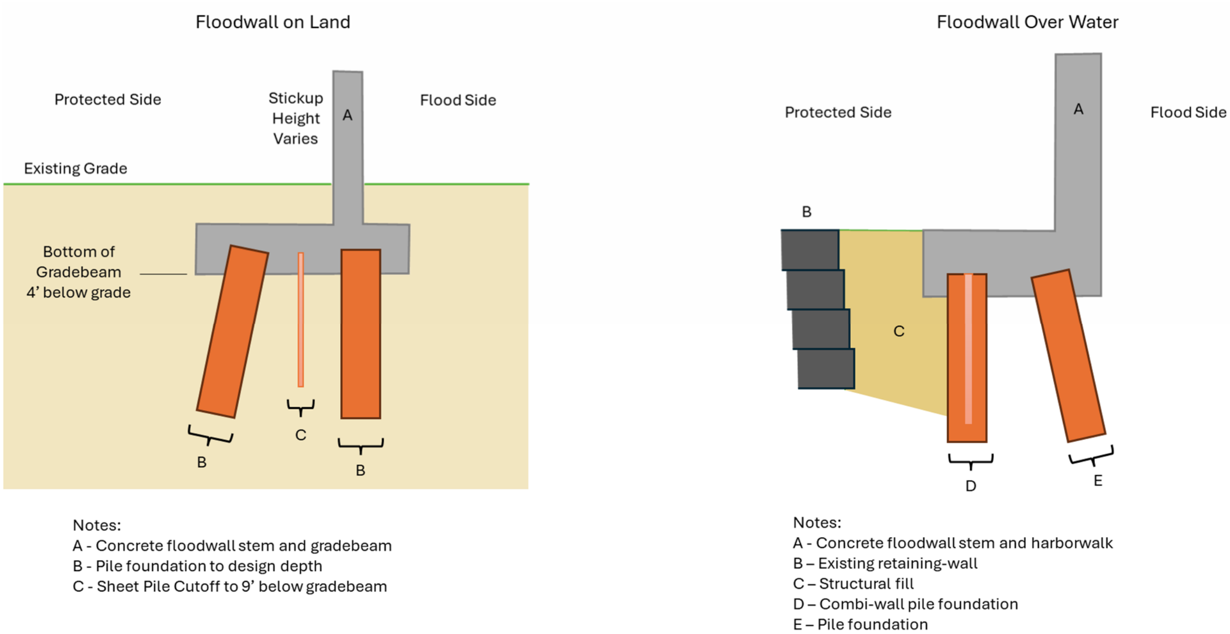

Figure 5: Schematic floodwall cross sections

Both the loaded and unloaded conditions were considered to determine design loads at each pile location for each wall height. A seepage cutoff was included in compliance with EM 1110-2-2502. Global stability is also briefly discussed.

4.2.1 Pile Design Parameters

For simplicity a 24” diameter, concrete-filled, driven steel pipe pile was selected and applied throughout the project. This was selected based on recent design phase work for a project with similar soil stratigraphy in New England. This approach was considered an efficient first pass to establish a basis for project cost. The design is highly conventional, using methods and materials that have been in common use for decades.

Preliminary estimates of pile dimensions were developed to approximately 10% design in support of a Class IV cost estimate. Piles were designed for axial capacity, then lateral deflection was verified. Design risks are managed by "first pass" calculations that provide an estimate of axial pile capacity for axial sizing. Lateral deflections are calculated based on typical soil parameters for the typical soil profiles developed in each neighborhood.

A wide range of deep foundation materials, sizes, and installation methods exist, in addition to rapidly advancing ground improvement technologies. As a result, a wide range of options are available for the design of this project. Several options should be considered for each alignment after site specific data are collected and processed. Efficiencies and simplifications are anticipated during future design phases.

Anticipated service loads were developed by the structural engineer in an editable spreadsheet. Loads consisted of self-weight, applied hydrostatic, and applied wave forces. Designs considered both the self weight and with-wave-and-water load cases. Pile reaction forces were determined from the spreadsheet and are summarized in Table 10 and Table 11 below. Axial loads were rounded up to the nearest 5 kips. Horizontal loads were rounded up to the nearest kip.

Table 10: Pile Axial Load Requirements.

Loading	**Units**	**Required Pile Capacities**		
Wall Height	**feet**	**3**	**6**	**9**
Spacing	**feet**	**10**	**8**	**6**
Vertical	kip	40	40	40
Battered	kip	45	50	50

Table 11: Pile Lateral Load Requirements.

Loading	**Units**	**Required Pile Capacities**	
Wall Height	**feet**	**6**	**9**
Spacing	**feet**	**8**	**6**
Vertical	kip	11	14
Battered	kip	19	22

4.2.2 Axial Pile Design

4.2.2.1 Assumptions

Pile length was determined based on axial capacity requirements. Capacities were estimated based on several key decisions and assumptions summarized below:

- Driven steel pipe was selected (24" concrete filled)
- Most piles were designed on the basis of skin friction within the Boston Blue Clay layer, neglecting end bearing. Some piles (only in Dorchester neighborhood) were designed using end bearing only.

- Some Dorchester alignments were designed based on end bearing due to the presence of competent sands.
- Piles were designed to bear all axial load within the Boston Blue Clay formation (a conservative assumption)
- Fill and Marsh/Riverine strata were assumed to contribute no axial reaction due to strain incompatibilities and potential for settlement.
- Downdrag was neglected since no fill after the pile installation is anticipated in association with floodwalls in the federal project. This assumption may need to be revisited if fill is incorporated in the future.

4.2.2.2 Geometry

Pile lengths were developed by identifying the length to the clay layer and required embedment within the clay layer. The length of pile to clay was determined by subtracting the depth to the bottom of the concrete grade beam footing, from the depth to clay, then adding back the required length of embedment within the grade beam footing. Embedment in the clay layer was determined based on the length required to develop reaction forces, plus an additional 5 feet to allow for:

- Transition zone from upper layer into Boston Blue Clay
- Allowance from the tip for skin friction mobilization
- Stratigraphic and strength variability.

The depth to the clay layer and standard deviations determined in Section 3.2.2 for each neighborhood were used to estimate required pile lengths in four groups as identified below.

- Shallow – upper 30% of piles – represented by 1σ shallower than mean
- Average – middle 40% of piles – represented by mean depth
- Deep – lower 25% of piles - represented by 1σ deeper than mean
- Very Deep – lowest 5% of piles – represented by 2σ deeper than mean.

A neighborhood average was then calculated as a weighted average for the four depths to clay, then a weighted average of those was determined for the three pile capacities discussed in Section 4.2.1..

4.2.2.3 Pile Design

Skin friction capacities were estimated using Schmertmann's correlations with SPT as given in Paikowsky et al. (2004) Appendix D. For each neighborhood, available SPT data were compiled for the Boston Blue Clay. Weighted averages were compiled to develop an overall average skin friction for the layer as described below. The Schmertmann method provides ultimate capacity, so allowable skin friction values were estimated using a Factor of Safety of three (FS=3). Average strengths compared reasonably well with generally recommended allowable skin friction capacities of 500 psf (Johnson, 1989).

Schmertmann identified four soil types. Since the Boston Blue Clay is a low plasticity mixture of primarily silt and clay sized particles, the Plastic Clay and Clay, Silt, Sand Mixture skin frictions (Type 1 and Type 2 respectively) were averaged. Weighted averages were calculated based on the frequency of occurrence within 5 blow count increments (0 to 5, 5 to 10, etc). The midpoint of each range was converted to a skin friction using Schmertmann's equations, the factor of safety was applied, then the weighted average was taken.

SPT data were readily available and applied to the entire neighborhood of Charlestown, Downtown, and South Boston. Sufficient SPT data were not available for East Boston, so the City-wide dataset was used. The results of the skin friction calculations and the average length of pile for land side alignments are given in Table 12 below. Designs were prepared on an individual basis for each Dorchester alignment due to several reasons identified below:

- Significant variability was observed throughout the neighborhood.
- The neighborhood is geographically large.
- Alignments are relatively short and isolated throughout the neighborhood.

Dorchester designs were prepared based on skin friction or end bearing based upon the stratigraphy and SPT values identified at each location.

Table 12: Skin Friction and Average Pile Length for Land Side Alignments

Neighborhood	Number of SPT Samples in Boston Blue Clay	Average of Schmertmann's Allowable Skin Friction (psf)	Average Length of Pile — Land Side (feet)
Charlestown	43	640	31
Downtown	36	650	35
South Boston	221	580	43
East Boston/City Wide	343	620	30

To support over-water structures, the top of clay layer elevation had to be verified against potential dredge depths. If the adjacent channel had been cut to below the top of clay layer, site specific pile lengths were determined by lowering the top of clay layer to five feet below the channel bottom to allow for muck, then adding the required embedment for that neighborhood. An additional three (3) feet were added to account for the change in elevation of the top of the pile. The effect on pile design ranged from no change to an additional 50 feet due to the proximity of deep channel. These adjustments were required at select locations in Charlestown, Downtown, and South Boston, and are summarized in Table 13 below. Dorchester and East Boston do not have over-water floodwall segments.

4.2.2.4 Summary and Discussion

Driven steel pipe piles (24" concrete filled) were selected for 10% design of floodwall foundations. The piles were assumed to bear entirely on skin friction in the Boston Blue Clay. Depth to Clay was determined for each of the five (5) neighborhoods based on four statistical categories using neighborhood specific data from the database of over 270 borehole locations discussed in Section 3.2.1. Based on the loads and spacings for each stickup height, pile capacity requirements of 40, 45, and 50 kip were identified. Skin friction values for each neighborhood were determined using Schmertmann's SPT correlations and a Factor of Safety of three (3). Required embedment was determined for each pile capacity and added to the other pile geometry. Estimated neighborhood average pile lengths ranged from 30 to 43 feet.

Table 13: Water Side-Floodwall Pile Lengths

Neighborhood	Location	Interpreted Dredge Elevation (feet MLLW)	Pile Length (feet)
Charlestown	Navy Yard	-10 and -20	50 and 60
Charlestown	Little Mystic	-20	60
Downtown	Downtown – Water Side	-20	60
South Boston	Fan Pier	-15	50
South Boston	Outer Fan Pier Marina	-35	70
South Boston	Inner Fan Pier Marina	-15	50
South Boston	Pier 4 Slip	-35	70

Several additional comments are made based upon this approach to 10% pile design.

- Pile length distribution within neighborhoods varied much more than the differences between neighborhoods.
- Variations in depth to the Boston Blue Clay within and between neighborhoods affected pile length much more than variations in average strength parameters of the Clay between neighborhoods.
- Depths to the clay layer appear to be represented reasonably well by a normal distribution.
- Log-normal distribution appears to describe the SPT dataset much better than a linear distribution, which is typical for geotechnical parameters.
- Based on limited calculations using Schmertmann’s SPT correlations for SPT in fill and marine sands, incorporating skin friction from these layers would provide sufficient capacity such that piles would not extend to the Boston Blue Clay. However, settlement would need to be determined for any proposed foundation in the fill or marine sands. This more complex analysis was beyond the scope and considered to have significant cost risk for these preliminary designs. Therefore, the more conservative decision to extend into the Boston Blue Clay was taken.

Design risks related to axial pile capacity have been managed by selection of a robust foundation pile system. Pile design lengths have been determined by calculation using a statistical evaluation of available field data for geometry and skin friction capacity. Skin friction capacities were calculated based on a simple “first pass” approach and compared well with readily available literature.

4.2.2.5 Recommendations

The 10% feasibility designs presented in this report were developed to provide a basis for estimating project costs in support of economic analysis. Developing site specific designs for each alignment was considered impractical and unnecessary during this feasibility stage.

Several recommendations are made for consideration during the advancement to 35% design maturity. These recommendations in no particular order are:

- Suitable site investigations must be conducted along each proposed alignment to identify local stratigraphy and suitable soil parameters. These site investigations must include focus on settlement and seepage considerations.
- The laboratory soil testing program may need to occur in stages. First, initial testing may be used to identify likely soil parameters based on relatively simple and fast tests. Based on this information a suitable foundation type could be selected. Then a second round of additional and confirmatory laboratory tests that are more complex and slower that are tailored to the selected foundation type can be conducted.
- Shallow foundations should be evaluated in areas where suitable soils are identified near surface. A tradeoff analysis is recommended between over-excavating and replacing unsuitable surficial soils versus the cost of a deep foundation system.
- Ground improvement may also prove practical in areas where surficial soils are near suitable or readily improved. The relatively high water table and granular fills may be conducive to in-situ compaction or injection of grout. Numerous controlled modulus systems are also available for organic and fine grained soils. These may consist of driven concrete piles, direct push concrete columns, injected, or augered concrete piles. Systems that require aggregate may enhance seepage potential and should be avoided. In-situ soil improvement would provide the added benefit of minimizing spoil materials requiring disposal.
- Driven steel pipe piles with concrete fill were selected. This provided a combined benefit of not producing spoils requiring disposal and an incremental increase in soil strength. Drilled displacement piles may provide another option to achieve both of these benefits if deflection criteria can be met.
- It will be necessary to evaluate the potential for consolidation settlement regardless of the selection of deep or shallow foundations. The Fill, Marine Organic, and Boston Blue Clay layers are all susceptible to consolidation settlement. The Boston Blue Clay was selected as the bearing stratum because it is least susceptible to settlement and applied loads are considered sufficiently low to control settlement.
- During the 30% design process, other pile support mechanisms should be considered, such as bearing in the fill, end bearing on clay or sands, or ground improvement on a case by case basis.
- Pile load tests are strongly recommended. These should consider both capacity and settlement if time allows. Load tests often result in reduction of required pile lengths. Developing a database of load tests can inform both current designs and future work throughout the City of Boston.

4.2.3 Lateral Pile Capacity and Deflection

4.2.3.1 Given Loads and Geometry

Lateral pile deflection was checked to determine if the deflections are acceptable with the selected pile design. LPile software by EnSoft was used to determine deflections. The EnSoft technical manual was used to determine typical soil parameters for the soil types identified in the typical profiles. For simplicity, single pile analyses were conducted at this time to verify the assessment and evaluation designs at 10% design maturity. A thorough sensitivity analysis was conducted on the Charlestown neighborhood to identify key drivers of deflection.

Pile designs consist of bents of two piles in plane, orthogonal to the wall face. The stem is eccentric on the foundation grade beam, near the water-side pile. The land side pile is located landward of the T-Wall stem, at 3 diameters on center to minimize group effects. The land side pile is battered at 1V:4H to provide additional lateral resistance.

The first investigation was the distribution of the lateral reactions between the vertical pile and the battered pile. Dividing the horizontal reaction forces equally between the two piles resulted in non-compatible deflections of the two individual pile analyses, assuming each responds only in bending. A second set of predictions were made considering the bending of each pile, and the horizontal reaction component contributed by skin friction along the axis of the battered pile. Reactions were determined by finding the axial component contributed by the battered pile from geometry and then distributing the remaining force equally between the two piles in bending. The second approach resulted in near-congruent deflections. This approach was selected and applied to the analyses for the remaining neighborhoods.

Deflections were calculated for the 6-foot and 9-foot wall typical sections. Deflections of the 3-foot typical section are expected to be less than those of the larger typical sections and therefore were not computed directly.

4.2.3.2 Sensitivity Analysis

The sensitivity analysis was conducted using pile parameters for the 6-foot wall design in Charlestown. For the sensitivity analysis, the base case of the average and/or expected conditions was applied and horizontal deflections were calculated. Then each parameter was individually modified to a sensitivity value selected to assess less conservative conditions while keeping all others at the base case values. In this way, the effects of varying a single parameter were assessed. The percent change (100*(sensitivity – base)/base) was then calculated for the parameter and the vertical and battered pile horizontal deflections. Finally, the ratio was determined by dividing the average of the percent changes of the deflection by the percent change of the parameter. Input parameters, deflections, percent change, and percent change ratios are provided in

Table 14 and

Table 15 below.

A review of the percent changes and ratios indicates that changes in lateral load, pile diameter, and fill properties have relatively large effects on the predicted deflection. Neglecting the concrete fill had a moderate effect on the predicted deflection, increasing the predicted deflection on the order of 20 to 40%. Minimal changes to deflection were observed when the pile wall thickness, depth to Boston Blue Clay, and Clay strength properties were varied.

Table 14: Sensitivity Analysis Results for 6-foot Wall Design in Charlestown

Parameter	Units	Base Case	Deflection Vertical (in)	Deflection Battered (in)	Sensitivity Values	Deflection Vertical (in)	Deflection Battered (in)
Lateral Load (Vertical)	kip	14.8	0.17	—	11.4	0.13	—
Lateral Load (Battered)	kip	14.8	—	0.09	18.6	—	0.13
Pile Diameter	inches	24	0.17	0.09	18	0.26	0.12
Pile Wall Thickness	inches	0.75	0.17	0.09	0.5	0.18	0.09
Concrete Fill	psi	4000	0.17	0.09	0	0.20	0.13
Depth to Boston Blue Clay	feet	17	0.17	0.09	26	0.17	0.08
Fill Properties (Friction)	phi	28	0.17	0.09	30	0.08	0.04
Fill Properties (Modulus)	pci	25/20	0.17	0.09	90/60	0.08	0.04
Clay Properties (Cohesion)	psf	1000	0.17	0.09	600	0.17	0.09
Clay Properties (50% Strain)	E50	0.01	0.17	0.09	0.02	0.17	0.09

Table 15: Sensitivity Analysis Results as Percent Change and Percent Change Ratios

Parameter	Change in Parameter (%)	Change in Vertical Deflection (%)	Change in Battered Deflection (%)	Ratio (Avg Chg in Def / Chg in Parameter)
Lateral Load (Vertical)	-23	-24	--	1.0
Lateral Load (Battered)	26	--	44	1.7
Pile Diameter	-25	53	33	-1.7
Pile Wall Thickness	-33	6	0	-0.1
Concrete Fill	-100	18	44	-0.3
Depth to Boston Blue Clay	53	0	-11	-0.1
Fill Properties (Friction)	7	-53	-56	-0.5
Fill Properties (Modulus)	200			
Clay Properties (Cohesion)	-40	0	0	0.0
Clay Properties (50% Strain)	100			

The selected parameters and reasons for selection are summarized below.

- Lateral load distribution accounting for the battered component produced deflections that indicated a congruent system and was selected. It was considered to provide a more appropriate prediction than an equal distribution of the horizontal load between the two piles.
- A pile diameter of 24 inches was selected for the project and selected after the sensitivity analysis as providing the best constructable design over a broad range of site and installation conditions.
- A pile wall thickness of 0.5 inches was selected based upon the minimal change and significant reduction in steel quantity relative to a 0.75 inch thickness.
- Average depths to Boston Blue Clay were selected since adding one standard deviation to the average resulted in essentially no change in deflection.
- Loose sand was selected for the fill properties since it produced a much greater deflection than more dense parameters, as expected. The loose sand parameters selected represent granular soils with SPT values of 4 or less. SPT values in the granular fill averaged about 10 with two standard deviations to each side (~95% of the sample) ranging from 5 to 21 based on 23 samples of granular fill in Charlestown on a log-normal distribution.
- Clay properties representing a soft to medium clay without free water were selected for the Boston Blue Clay layer. It was found that changing these to soft clay did not change the predicted deflection. This is reasonable given that the lateral load is likely taken up by strain in the granular fill layer, resulting in minimal strain and associated load transfer in the Clay layer at

depth. A similar reasoning can be applied to the water-side scenarios, where the piles could be braced and lateral loads transferred to the upland via the combi-wall construction.

4.2.3.3 Calculated Deflections

Predicted deflections for the 6-foot wall were 0.16 inches on the vertical pile and 0.13 inches on the battered pile for all neighborhoods for the selected parameters identified in Section 4.2.3.2. this is likely attributable to the rigidity of the concrete filled pipe pile and the relatively thick layer of granular fill modeled at surface. It is interpreted that the surficial fill takes up the lateral load, minimizing dependence on deeper layers. The 9-foot wall produced deflections of 0.20 inches on the vertical pile and 0.21 inches on the battered pile and was also consistent across all neighborhoods.

The predicted deflections were well within the generally accepted tolerance of 1 inch. It is extrapolated that deflections can be expected to remain within tolerance when group effects are incorporated. The design is adequate and represents a feasible approach to construction. It is anticipated that foundation designs will be evaluated and optimized for each alignment based on site specific investigations and analysis prior to the 35% design milestone.

Design risks associated with lateral capacity and deflection were managed by calculating deflections for typical profiles and evaluating potential variability. Calculated deflections were well within tolerable limits. Variability and group effects are likely to increase deflections, but it is anticipated that they would remain within tolerable limits. It is noted that each foundation design will require engineering design and analysis.

4.2.3.4 Recommendations

Lateral capacity and deflection are critical design parameters for the hydrostatically loaded condition. Several recommendations related to lateral capacity design and analysis are provided below.

- Design for lateral load should have a significant influence on the scope of the geotechnical investigation and geotechnical laboratory testing.
- It is recommended that the field investigation identify elastic parameters by shear wave velocity (or other suitable methods) for lateral deflection analysis where possible.
- Ground improvement should be considered to provide sufficient vertical and lateral bearing capacity, while also reducing seepage and minimizing spoils and excavation.

4.2.4 Seepage Under Walls

EM 1110-2-2502 requires that all USACE designs include a minimum cutoff wall; however, the length of the wall or depth of cutoff are not specified. It is likely that some form of seepage control will be required along wall segments due to the narrow base and differential head. The extent must be investigated on a case by case basis by thorough geotechnical investigation.

Seepage analysis was considered highly location specific across the project due to spatial and geologic variability. Typical analyses would not be applicable at any specific location. Seepage was also considered to be a minor component of design due to the short loading durations caused by the large tide cycle.

As such, a place holder design was developed as an input to develop approximate costs at this 10% design level throughout the large and variable project. It is intended to be representative of the overall project, identifying a component of the overall scope and conceptual design.

Design risks related to potential seepage were assumed to be managed by a 10-foot-deep steel sheetpile cutoff with 1 foot of embedment in the wall foundation. This design extends the minimum flow path by approximately 50% from about 20 feet to 30 feet. It also provides protection against preferential seepage pathways that are likely to develop between the bottom of the grade beam and the backfill below.

4.2.4.1 Recommendations

Several comments and recommendations regarding seepage design below wall segments are provided below.

- The presence of hardened surfaces such as steel seawalls and surface pavements will significantly alter seepage in urban areas. Although this is likely to reduce seepage volumes in most areas, it may also act to channelize the flow and must be evaluated on a case-by-case basis.
- Injection grouting should be considered in areas where voids in soils or structures are anticipated.
- Drainage outfalls represent a significant risk to project design, with the potential to hydrostatically charge the pipe and leak into the protected area. Adequate sealing of drainage outfalls will be critical to ensure that the system performs as intended.
- A thorough investigation of hydraulic conductivity should be incorporated in the geotechnical investigation at each alignment. Adequate understanding of conductivity and variability across the site can be anticipated to provide opportunities to optimize the design. Field and laboratory methods should be employed to establish typical conductivity and quantify the extent of variability. These may include vertical infiltration tests, well tests such as slug tests, or pump tests, downhole tests such as the Hydraulic Profiling Tool (HPT), or laboratory tests as appropriate.

4.2.5 Comments on Global Stability and Liquefaction

Deep foundations were selected for all floodwall alignments at this 10% feasibility level of design. In this way, the deep foundations bear on strata that are below the mudline and therefore do not induce a surcharge in the upper layers, making the designs inherently stable. If shallow foundations are pursued during future design development, it will be necessary to consider global stability of the feature during loaded and unloaded conditions, especially if the feature is in proximity to natural or constructed banks or bulkheads.

The deep soils, in this case the Boston Blue Clay, exceed the common threshold of 15% to 20% fines, above which soils are considered non-liquifiable. However, the surficial fill and marine sands are frequently liquifiable and will require thorough investigation if shallow foundations in these layers are pursued.

At the 10% level of design maturity, global stability and liquefaction risks have been managed by selection of deep foundations in non-liquifiable soils.

4.3 Levee Designs

Levee designs for preliminary estimates of quantities were developed to approximately 10% design maturity in support of a Class IV cost estimate. Levees represent less than three miles out of approximately 20 miles of protection, and therefore represent a relatively small proportion of cost and overall cost risk. Levee design consisted primarily of geometric requirements, permeability, and seepage considerations. Assumptions were made regarding settlement and global stability as discussed in the following sections. Similar to the pile designs, major features were determined at a preliminary level of design and simplified typical sections were applied throughout the proposed alignment. It will be essential for designers to collect site specific data for suitable analysis during the development of designs for construction.

4.3.1 Geometry

USACE (EM 1110-2-1913) recommends levee side slopes of 1V:3H or flatter, with a crest width adequate for maintenance needs. This guidance is prepared primarily based on levees of the Mississippi Basin and large western rivers such as the Columbia that often hold water for weeks or months at a time. Guidance is not provided for the relatively low crested coastal levee systems that only hold surge for a few days at most.

The crest width is required to be adequate for maintenance, which is typically interpreted as wide enough for a pickup truck to comfortably travel. Levee crest width is often in the range of 8 to 10 feet wide. The guidance does not address space constraints.

Levee designs were prepared as 3-, 6-, and 9-foot-high typical sections. The 6' and 9' sections were designed with 1V:3H sideslopes, while the 3-foot section was designed with 1V:2H sideslopes due to the very small height and low associated risk. These were also much more likely to be located in places with space constraints. Layout and quantity calculations were performed by the project Civil Engineer.

4.3.2 Seepage

Seepage must be evaluated both through and below the levee. Placeholder designs at 10% design maturity were developed to support a Class IV cost estimate. For simplicity, preliminary manual calculations were made to determine a suitable bulk soil permeability to control seepage through the levee. For simplicity, a seepage cutoff was selected using quantities consistent with wall designs. Toe drains and other drainage features were not provided at this preliminary design phase. It is assumed that toe drains are not likely to be needed for the relatively short loading durations on low height and broad base levee designs prepared at this phase. Toe drains also commonly serve the same function as the seepage cutoff and may represent design alternatives. In this way the risk to scope has been captured.

4.3.2.1 Through-Seepage Design

Term through-seepage here is used for water seepage through the body of the levee itself. At its basic concept, the levee soil must have sufficient resistance to flow to limit seepage to acceptable levels. The project will be subjected to relatively short duration loading of not more than 3 or 4 days, during which the water levels will vary 8 to 10 feet with the tide cycle. Soil permeabilities were assumed such that they would be representative of steady-state hydrostatic conditions, consistent with EM 1110-2-1913 and EM 1110-2-1901. This analysis is anticipated to overestimate the actual seepage that would be experienced through a levee with these design parameters.

Fixed boundary conditions for the calculations consisted of the levee geometry and a flood condition at 2 feet below the crest, which is the typical wave runup across the project. An assumed flow rate and the elevation of the phreatic surface on the land side of the levee are required. The phreatic surface within the levee was arbitrarily set to ½ foot above grade on the protected side. This condition ensures that the conductivity will be sufficiently low to dissipate almost all of the pressure head applied on the flood side. The flow rate was varied to assess the effects on the required conductivity.

It was found that the required acceptable soil hydraulic conductivities decreased as the height of the levee increased. Required hydraulic conductivities also decreased as the specified flowrate decreased. A flowrate of 0.1 cubic feet per minute per linear foot (10 cubic feet per second per mile) was assigned to the 3-foot and 6-foot levees, while a more conservative 0.01 cubic feet per minute per linear foot (1 cubic feet per second per mile) was assigned to the 9 foot levee. Based on these flowrates, maximum hydraulic conductivities of $1x10^{-3}$, $1x10^{-4}$, and $1x10^{-5}$ centimeters per second were assigned to levees of 3 feet, 6 feet, and 9 feet in height respectively.

4.3.2.2 Underseepage Design

Underseepage refers to the water seepage through foundation soils beneath a levee. Underseepage considerations were addressed by including a 10-foot deep sheetpile cutoff wall. Need for this feature should be investigated for each alignment as designs progress to 35%. It is possible that some locations will not require the seepage cutoff for the anticipated flood durations while some locations may require deeper cutoff depths. It is also possible that any ground improvement provided will densify the soil or provide sufficient cutoff to eliminate the need for the seepage cutoff wall. This cutoff wall element may also be a placeholder for toe drains or other seepage related features of work not captured in this 10% design effort.

4.3.2.3 Summary and Discussion

This preliminary design provides a feasible, constructible approach to achieve the design objective. It is intended as a placeholder in support of a Class IV cost estimate. These designs are intended to provide a 10% design maturity concept and should not be considered a final levee design for the project. The levee design process for seepage will require determining appropriate zoning, filter compatibility, conductivity, and potential failure modes, among other considerations. Each levee must be designed to work in conjunction with the local geography and geology. Appropriate underseepage measures must be considered and evaluated in conjunction with through-seepage.

Seepage design risks were managed by calculating soil properties sufficient to provide water retention through the structure. The same underseepage cutoff was provided to match the floodwall underseepage design based upon similar assumptions for simplicity. These designs are intended to provide a constructable and functional design at a conceptual 10% level of design maturity. Levee cross sections and underseepage must be designed on a case-by-case basis in response to site specific geometry and stratigraphy.

4.3.3 Comments on Other Levee Design Aspects

4.3.3.1 Settlement

Levee construction requires adequate preparation of the subgrade. In the case of Boston, most of the low lying coastal fringe is loose to medium dense random fill, over highly compressible organic marsh deposits, over the less compressible Boston Blue Clay. In short, the entire soil profile can be expected to compress in response to surcharge. The greatest surcharge occurs in the immediate subsurface, down to a depth of about double the bottom width of the embankment. Below this, the increment of pressure decreases meaningfully, resulting in a reduction in contribution to settlement from the deeper layers.

Some form of ground improvement will likely be required below levee segments represented by the 6 foot and 9 foot typical sections. The type and intensity of treatment will vary from site to site, and therefore could not be readily designed as a typical section. In its place, it was assumed that removing a mirror image cross-section of sediment and replacing it with levee material would provide a representative placeholder for the scope and costs associated with ground improvement. Replacing the existing loose fill with engineered compacted fill would reduce the settlement potential in the vicinity of the greatest stress increment. It was assumed that the remaining settlement would be within acceptable limits and could be incorporated in design.

Where levees and shallow foundations are used, settlement must be evaluated by thorough site characterization, site specific laboratory analysis, and suitable engineering design. Tolerable settlements at utility crossings will need to be established and suitable designs developed to remain in compliance.

4.3.3.2 Global Stability

Levees constructed at 1V:3H side slopes are unlikely to encounter internal slope stability concerns when suitable soils and compaction are provided. Global stability assessments were not conducted for these 10% feasibility designs. Global stability risks were minimized by the design approach as described in the following paragraph.

Levees situated immediately adjacent to natural banks would extend the slope, resulting in potential increase in driving forces and global stability risk. These have been avoided in preliminary layouts by setting the levee back from the crest of the natural bank or seawall. If future alignments result in levees in close proximity to natural banks or seawalls, global stability calculations must be conducted per EM 1110-2-1902 to verify stability.

Resistance to sliding was assumed to be adequate given the broad base and relatively shallow water depths. The soil replacement approach below the levee would also provide a key into the existing soils.

4.3.3.3 Armoring

Armoring on levee slopes may be required at some locations to prevent erosion due to wave attack. Site specific designs must also consider potential overtopping and associated crest and backslope erosion.

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