# Appendix D
# Geotechnical Memorandum

# Port of Anchorage Intermodal Expansion Project – Geotechnical Recommendations for the 15% Concept Design

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DATE: February 28, 2013

PROJECT NUMBER: 462130.01.04.A3

# Contents

# 1 Introduction

CH2M HILL was contracted by the United States Army Corps of Engineers (USACE) to conduct a 15% concept design of new pile-supported wharves at the North Expansion area and the existing terminals of the Port of Anchorage (POA). This technical memorandum provides an overview of the analyses carried out to support the geotechnical recommendations for the 15% concept design. This memorandum also documents the design basis, performance criteria, and analysis details, which include the underlying assumptions that were used in the geotechnical calculations. Results of the geotechnical analyses described herein were used by CH2M HILL's structural and civil engineers to support design and develop cost estimates for the construction. The accompanying figures are present at the end of the memorandum.

## 1.1 Project Descriptions

New pile-supported wharves were evaluated as possible replacements for the existing OPEN CELL® sheet pile (OCSP®) system at the North Expansion area and the existing pile-supported wharves at Terminals 2 and 3. The following options were considered in the 15% concept design:

- **Option 1.** The major structural components of Option 1 would consist of two pile-supported wharves, six access trestles, and a cellular steel sheet pile bulkhead. Other ancillary structural components to support port operations would include heavy-duty fenders, mooring bollards, quick release hooks along the wharf face, and a stevedore building. The pile-supported wharves would provide a total of 1,800 linear feet of new dock face and two new berths: (1) a barge berth to support containerized, break bulk, or bulk cargo operations, and (2) a RO/RO Berth to support containerized RO/RO operations. The top of the deck and upland would be at elevation +38 feet mean low lower water (MLLW). This alternative would require removal of the existing OCSP® system and some of its associated fill. After removal of the existing OCSP® system, existing fill would be dredged to a 5H:1V (horizontal:vertical) embankment slope, extending landward from the face of the wharf until it caught a new cellular sheet pile bulkhead at elevations between elevation +18 feet Mean Lower Low Water (MLLW) and elevation +8 feet MLLW. The main structural components of this alternative are shown in Figure 1.

- **Option 5:** The main structural components of Option 5 would consist of three pile-supported wharves, nine access trestles, a cellular steel sheet pile bulkhead, and several retaining walls. Other ancillary structural components to support port operations would include heavy-duty fenders, mooring bollards, quick release hooks along the wharf face, two stevedore buildings, and container-crane-supporting infrastructure. The pile-supported wharves would provide 2,900 linear feet of new dock face and three new berths: (1) a barge berth to support containerized, break bulk, or bulk cargo operations; (2) an RO/RO berth to support containerized RO/RO operations; and (3) a container cargo berth to support LO/LO container cargo operations. The top of the deck and upland would be at elevation +38 feet MLLW. After removal of the existing OCSP® system, existing gravel fill would be dredged to form an embankment, extending landward from the face of the wharf until it caught either a new cellular sheet pile bulkhead or a master pile retaining wall at elevations between elevation +18 feet MLLW and elevation +8 feet MLLW. The embankment slope would be 5H:1V at the Wet Barge Berth and 4.25H:1V at the RO/RO and Container Berths. The main structural components of this alternative are shown in Figure 2.

- **Option 5-1 Hybrid:** The main structural components of Option 5-1 Hybrid would consist of three pile-supported wharves, nine access trestles, a cellular steel sheet pile bulkhead, and four mooring dolphins. Other ancillary structural components to support port operations would include heavy-duty fenders, mooring bollards, and quick-release hooks along the wharf face, three stevedore buildings, and container-crane-supporting infrastructure. The pile-supported wharves would provide 2,405 linear feet of new dock face and three new berths: (1) a hybrid berth to support containerized, break bulk, bulk, and RO/RO cargo operations; (2) an RO/RO berth to support containerized RO/RO operations; and (3) a container cargo berth to support LO/LO container cargo operations. The top of the deck and upland would be at elevation +38 feet MLLW. After removal of the existing OCSP® system, a 5H:1V embankment would be constructed, extending landward from the face of the wharf until it intersected the new cellular sheet pile bulkhead at elevation +8 feet MLLW (Hybrid Berth) or the existing mudline (RO/RO and Container Berths). The main structural components of this alternative are shown in Figure 3.

## 1.2 Scope of Work

The following topics will be discussed in this technical memorandum:

- Relevant subsurface and groundwater conditions
- Seismic design considerations
- Dredge and scour depths
- Geotechnical design criteria
- Geotechnical analyses and design recommendations
- Design and construction issues to be considered in the final design

## 1.3 Limitations

This technical memorandum has been prepared exclusively for the 15% concept design of new pile-supported wharves at the POA in accordance with generally accepted geotechnical engineering practice. No other warranty, expressed or implied, is made. It is assumed that the information collected from the borings conducted within the project limits is representative of the subsurface condition at the project site. Interpretations and assumptions that have been made about site conditions are intended for concept design purposes only.

Geotechnical recommendations provided in this technical memorandum are not sufficient for the final design of the three wharf options. Additional field explorations, laboratory soil testing, and foundation design studies will be necessary to complete the design of any of the options.

# 2 Basis of Design

Design guidelines, specifications, methodologies, and underlying assumptions that were used in the geotechnical analyses conducted for the 15% concept design are summarized in this section.

## 2.1 Design Guidelines and Specifications

The following design guidelines and specifications were used to conduct the analyses:

- **Electric Power Research Institute (EPRI; 1990)**. *Manual on Estimating Soil Properties for Foundation Design,* EPRI EL-6800, Final Report.
- **Federal Highway Administration (FHWA; 2002)**. *Evaluation of Soil and Rock Properties, Geotechnical Engineering Circular No. 5*. Report No. FHWA-IF-02-034.
- **Naval Facilities Engineering Command (NAVFAC; 1986)**. *Soil Mechanics*, Design Manual DM-7.01.
- **American Association of State Highway and Transportation Officials (AASHTO; 2012)**. *LRFD Bridge Design Specifications.*
- **American Petroleum Institute (API; 2003)**. *Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms – Load and Resistance Factor Design*.

## 2.2 Design Methodologies and Tools

The following methodologies and tools were used to conduct the analyses described in this technical memorandum:

- Calculations of seismic-induced permanent deformation follow the methods developed by Ambraseys and Menu (1988), Hynes-Griffin and Franklin (1984), Bray and Travasarou (2007), and Rathje and Saygili (2008).
- Computer program SLIDE (Rocscience, Version 6.02) for evaluating global slope stability and conducting transient seepage analyses
- Computer program APILE (Ensoft, Version 5.0.22) for calculating axial capacity and settlement of steel pipe piles
- Computer program GRLWEAP (GRL, Version 2010) conducting for drivability analysis of driven pipe piles

## 2.3 Key Assumptions

Key assumptions made in the geotechnical analyses are as follows:

- 48-inch open-ended steel pipe piles would be used for the pile-supported wharves and trestles. The wall thickness of the pipe piles would be 1 inch.
- The drag load on piles due to settlement of the Bootlegger Cove Formation (BCF) clay would be negligible. This is believed to be a reasonable assumption considering that the BCF clay would be “unloaded” when the existing OCSP® system and the contained fill were removed to construct the new pile-supported wharves and the embankment slopes.
- The open-ended pipe piles could be either “plugged” or “unplugged” when subjected to structural loads. The nominal pile capacity would be calculated based on the minimum end-bearing resistance obtained from the “plugged” and “unplugged” conditions.
- The capacity of the piles at the service limit state would be determined based on a pile settlement of 1 inch.
- The 48-inch pipe piles would be driven by using a Pileco D-280 hammer (or hammers with similar capacity) without the use of the driving shoe.
- Side resistance during pile driving is the same as the static long-term side resistance (that is, no strength loss during pile installation).
- Both “plugged” and “unplugged” conditions are considered in the drivability analysis. For the “unplugged” case, the inner side resistance of the pipe piles equals half of the outer side resistance.
- Micropiles[1] used to stabilize the slope under the wharves and the retaining walls at the existing terminals (Options 5 and 5H) would have a lateral capacity of 75 kip. The micropiles would be drilled at least 5 feet into the medium–dense-to-dense sand below the estuarine silt. The shear force mobilized in the micropiles would be parallel to the base of the failure surface at the pile locations. The center-to-center spacing between the micropiles (in both directions) would be 5 feet.
- The global failure surface would have a non-circular shape. The factor of safety (FS) of the global failure surface would be calculated using the general limit equilibrium (GLE)/Morgenstern-Price method.

# 3 Relevant Subsurface and Groundwater Conditions

Subsurface conditions at the proposed facilities were obtained primarily from two sources: (1) *Geotechnical Analysis Report* (2008) and additional appendices (2010a and 2010b) issued by PND Engineers, Inc., (PND) for the

[1] Micropiles are typically small diameter (12 to 18 inch) piles that are drilled and grouted in place. Arrays of micropiles are used to stabilize embankments. See Section 6 of *Micropile Design and Construction Reference Manual* (FHWA NHI 05-039), December 2005 and Section 10.9 of the 2012 *AASHTO LRFD Bridge Design Specifications* for additional discussion of the design and construction of micropiles.

design of the original OCSP® system; and (2) supplemental field and laboratory testing conducted as part of CH2M HILL's suitability study (2013). CH2M HILL did not conduct any new field or laboratory work for the 15% concept design. The following subsections provide an overview of the subsurface profiles, soil properties, and groundwater conditions in the vicinity of the proposed facilities.

## 3.1 Design Soil Profiles and Soil Properties

Typical subsurface profiles at the Wet Barge Berth, North Extension, and existing terminals consist of a relatively thick layer of Controlled Fill placed over native materials. Below the existing mudline, the native soils encountered in the upper 200 feet include the Estuarine Deposits, the BCF clay, and the Glaciofluvial Deposits. The soil units are described below.

**Controlled Fill.** According to PND (2008), the fill above the native soils at the Wet Barge Berth and North Extension areas includes compacted Granular Fill and un-compacted Common Fill. The Granular Fill was used within the footprint of the existing open cells (that is, approximately 200 feet from the face of the OCSP® system). The Common Fill was used outside the footprint of the existing open cells. The compacted Granular Fill consists mainly of clean sand and gravel with a fines content of less than 10%. The Common Fill was described as well-graded granular soil with less than 50% fines. As shown in the 15% concept design plans, the existing OCSP® system and a majority of the Granular Fill within the cells would be removed as part of the construction for the new pile-supported wharves. Because of disturbance during removal of the existing fill, the Granular Fill that remained within the embankment slope body is assumed to be in a loose state. The characteristics of the fill material used at Terminals 2 and 3 are currently unknown. For design purposes, the properties of the Common Fill were also assumed for the fill at the existing terminals.

**Estuarine Deposits.** At the Wet Barge Berth and North Extension areas, the Estuarine Deposits consist of low-plasticity silt with fines content up to 90%. Results from Atterberg limits tests indicate that the Estuarine Deposits in these areas have liquid limits (LL) ranging from 21% to 40% and plasticity indices (PI) ranging from non-plastic to about 18%. In-situ Vane Shear Testing (VST) conducted by USACE (2008) indicated a range of undrained shear strengths from 600 to 1,950 pounds per square foot (psf) with the peak over residual undrained shear strength ratio ranging from 1.8 to 4.3. The thickness of the Estuarine Deposits increases from about 5 feet at the north end of the Wet Barge Berth to slightly more than 20 feet at the south end of the North Extension area. Unlike the Estuarine deposits encountered at the Wet Barge Berth and North Extension existing areas, the Estuarine Deposits encountered at the existing Terminals consist of approximately 10 to 30 feet of gravelly silt overlying about 30 feet of dense to very dense silty sand and gravel. Based on the average blow counts obtained from the standard penetration test (SPT), the gravelly silt at the existing Terminals appears to be medium dense to dense.

**BCF Clay.** This soil unit mainly consists of overconsolidated silty clay with interbedded lenses of dense fines, clay and silt (CL and ML). The BCF Clay can be classified as stiff to very stiff silty clay of low plasticity. The BCF clay layer is typically located under the Estuarine Deposits and extends to about elevation -150 feet MLLW at the Wet Barge Berth and North Extension areas, and to about elevation -200 feet MLLW at the existing terminal area. The shear strength characteristics of the BCF Clay were studied extensively by both PND (2008) during the original design of the OCSP® wall system and CH2M HILL during the suitability study (2013). According to the study, the following SHANSEP relationships can be used to correlate the undrained shear strength of the BCF Clay ($S_u$) with the effective overburden stress ($\sigma'_v$) and the over-consolidation ratio (OCR):

- Triaxial compression loading condition (CIUC): $S_u/\sigma'_v = 0.34*(OCR)^{0.79}$ [Equation 1]
- Direct simple shear loading condition (DSS): $S_u/\sigma'_v = 0.25*(OCR)^{0.79}$ [Equation 2]
- Triaxial extension loading condition (CIUE): $S_u/\sigma'_v = 0.17*(OCR)^{0.79}$ [Equation 3]

**Glaciofluvial Deposits (glacial drift).** This soil unit consists mainly of dense to very dense sand and gravel with interbedded hard clay layers that were consolidated under the weight of glaciers. These deposits are present under the BCF Clay and generally extend to over 600 feet in depth. The glacial drift is underlain by undifferentiated pre-Quaternary deposits or metamorphic bedrock.

Design soil profiles at the Wet Barge Berth, North Extension, and the existing terminals are shown in Figures 4 though 7. Engineering properties of all soil units are shown in Table 1.

TABLE 1
**Engineering Properties of All Soil Units**

| Soil Unit | Total Unit Weight, $\gamma$ (pcf) | Effective Cohesion, c' (psf) | | Effective Friction Angle, $\phi'$ (degree) | | Undrained Shear Strength Ratio, $Su/\sigma'_v$ | | Permeability, k (ft/sec) |
|---|---|---|---|---|---|---|---|---|
| | | Static | Pseudo-Static | Static | Pseudo-Static | Static | Pseudo-Static | |
| Surface (5 feet below pavement) | 130 | 0 | 100 | 36 | 36 | – | – | 3.3e-4 |
| Common Fill | 125 | 0 | 0 | 32 | 27 | – | – | 3.3e-6 |
| Compacted Common Fill | 130 | 0 | 100 | 40 | 40 | – | – | 3.3e-6 |
| Loose Granular Fill | 125 | 0 | 0 | 34 | 28 | – | – | 3.3e-3 |
| Estuarine Silt | 120 | – | – | – | – | 0.55 | 0.44 | 3.3e-8 |
| Dense Silty Sand and Gravel (terminals) | 130 | 0 | 100 | 38 | 38 | – | – | 3.3e-4 |
| BCF Clay | 125 | – | – | 30 | 30 | SHANSEP(*) | SHANSEP(*) | 3.3e-12 |
| Glaciofluvial Deposits | 135 | 0 | – | 45 | – | – | – | 3.3e-4 |
| Rock Armor and Riprap | 140 | 0 | 100 | 40 | 40 | – | – | 3.3e-1 |

Notes:

* See Equations 1 through 3

ft/sec = feet per second

pcf = pounds per cubic foot

psf = pounds per square foot

## 3.2 Groundwater Conditions

According to information obtained from piezometers installed in the backfill layer and the BCF clay, the landside groundwater table, measured approximately 500 feet behind the face of the existing OCSP® system, is approximately at elevation +20 feet MLLW.

According to Terracon (2011), tidal fluctuation up to 35 feet (that is, between elevations +30 feet MLLW and elevation -5 feet MLLW) can be observed during the day. A typical daily tidal curve is shown in Figure 8. With the highly fluctuating tidal elevations and the seaward flow of groundwater, the porewater pressure distribution within the embankment slopes will be constantly in flux (that is, in a transient state). Details of the transient seepage analyses are discussed later in this memorandum.

# 4 Seismic Design Considerations

Seismic design parameters and liquefaction potential of the geotechnical materials at the proposed facilities are discussed in this section. The design parameters used in the structural and pseudo-static global stability analyses were obtained from the site-specific probabilistic seismic hazard analyses (PSHAs) conducted as part of the OCSP® system design (PND, 2008) and seismic ground response analysis conducted by CH2M HILL for the 2013 suitability study. Details of the seismic ground response and liquefaction potential assessments can be found in the suitability study.

## 4.1 Seismic Design Parameters

The following seismic parameters were used in the 15% design:

- Peak ground acceleration (PGA) at the ground surface
  - Operating level earthquake (OLE) seismic event (72-year return period) PGA = 0.17 g
  - Contingency level earthquake (CLE) seismic event (475-year return period) PGA = 0.31 g
  - Maximum considered earthquake (MCE) seismic event (2,475-year return period) PGA = 0.39 g
- Seismic lateral coefficient: $k_h$ = 0.5*PGA

## 4.2 Liquefaction Potential and Shear Strength Degradation

The loose granular soils (granular fill, common fill, and the estuarine silt) encountered at the proposed facilities are assumed to undergo partial liquefaction during seismic loading. The development of excess porewater pressure will cause shear strength degradation. As a result, the friction angle and undrained shear strength of these soils were assumed to decrease in the pseudo-static global stability analyses. The reduced shear strength parameters used in the pseudo-static global stability analyses can be found in Table 1. The basis of the reduced shear strength is provided in the suitability study (CH2M HILL, 2013). The undrained shear strength of the BCF clay was assumed to be unchanged under seismic loading conditions.

# 5 Considerations for Over-Dredging and Scour Depths

The following assumptions were used for the over-dredging and scour depths at the proposed facilities:

- Over-dredging depth
  - Wet Barge Berth From elevation -31 feet MLLW at the north end to elevation -41 feet MLLW at the south end
  - North Extension Elevation -51 feet MLLW
  - Existing terminals Elevation -51 feet MLLW

- Scour depth – An additional 5-foot scour allowance was used to account for localized scour around piles if stone armor is not used.

# 6 Geotechnical Design Criteria

This section presents the geotechnical design criteria used in the 15% concept design. These criteria are intended to either improve the long-term performance of the facilities during normal operations or to provide higher confidence in the performance of these facilities during future seismic events.

## 6.1 Service Life

The new facilities will be designed for a 75-year service life, which is longer than the service life of 50 years typically used for other ports and waterfront structures around the United States including the OCSP® system and the ports of Los Angeles, Long Beach, Seattle, and Houston. The 75-year service life is consistent with the service life used in the current AASHTO *LRFD Bridge Design Specifications* for the design of most new bridges.

## 6.2 Design Criteria

Table 2 provides the design criteria for the embankment slope, cellular sheet pile bulkheads, and retaining walls. Allowable bulkhead and retaining wall deformations for short- and long-term static loading are the same as used for the OCSP®, as discussed in PND (2008). Additional analyses will be required to confirm these displacements during later phases of design based on risk of damage and through discussions with the POA. Allowable displacements during the OLE are the same as defined for the OCSP® (PND, 2008). However, allowable displacements for the CLE and MCE have been increased. The amount of increase is based on a qualitative assessment of consequences of movement, as well as judgment. These levels of deformation should also be confirmed in later phases of design.

TABLE 2
**Performance and Global Stability Criteria for Slopes, Sheet Pile Bulkheads, and Retaining Walls**

| Loading Condition | Deformation-Based Criteria Description | Allowable Bulkhead and Retaining Wall Deformation (inches) | Global Stability FS for the Embankment Slope |
|---|---|---|---|
| Short-term static | Moderate bulkhead movement without overstressing of structural components | Less than 18 | 1.3 |
| Long-term static | Moderate bulkhead movement of structural components | Less than 18 | 1.5 |
| Seismic: OLE | Very little additional bulkhead movement beyond static loading condition – damage repairable in a short time period and no interruption to wharf operations | Less than 3 (permanent)[a] | N/A [b] |
| Seismic: CLE | Small additional bulkhead movement beyond static loading condition – damage repairable with minimal interruption to wharf operations | Less than 12 (permanent)[a] | N/A [b] |
| Seismic: MCE | Moderate additional bulkhead movement beyond static loading condition – moderate damage but economically repairable with some significant interruption to damaged portions of wharf operation | Less than 30 (permanent)[a] | N/A [b] |

Notes:

[a] Displacements under seismic conditions are in addition to those from static conditions. Temporary wall movements during a seismic event might exceed permanent wall displacements at the end of an earthquake.

[b] Bulkhead performance is controlled by deformation criteria.

CLE = contingency level earthquake

FS = Factor of Safety

MCE = maximum considered earthquake

OLE = operating level earthquake

# 7 Geotechnical Analyses and Recommendations

The static global stability of the embankment slopes depends largely on the porewater pressure distribution within the slope body, which will be in a transient state (that is, time-dependent) due to the fluctuation of sea level. The following subsections provide a summary of the transient seepage analysis and the global stability analyses of the embankment slopes under both static and seismic loading conditions.

## 7.1 Transient Seepage Analyses

The global stability of the slope, sheet pile bulkhead, and retaining walls depends on many factors, such as geometry, loading, soil conditions, and porewater pressure distribution. As summarized in Section 3.2, the seaward flow of groundwater combined with a fluctuating sea level creates a transient flow of groundwater in and out of the embankment slope that results in a time-dependent porewater pressure distribution. To calculate the porewater pressure in the slope body, a transient seepage analysis is required to model the flow of groundwater at any time during the day. In addition, the transient seepage analysis can be used to estimate the exit gradient of the groundwater at the toe of the embankment slope and to develop mitigation measures to prevent a critical flow gradient from occurring at the slope surface.

Two-dimensional seepage analyses were conducted for the 15% concept design using the finite element package included in the computer program SLIDE. Four cross sections were considered in the transient seepage analyses: (1) Section C of Option 1, (2) Section B of Option 5, (3) Section C of Option 5, and (4) Section C of Option 5-1 Hybrid. These sections were selected for analysis because they occur at the location of structures for each of the facilities. The effects of piles on seepage were not considered, as these effects were expected to be minor based on the relative volume of slope to pile. To prevent a critical flow condition at the slope surface, it was assumed that the entire embankment slope was covered by a layer of armor rock that is 5 feet or more in thickness. The assumed engineering properties of the rock are shown in Table 1.

The armor rock in the North Expansion area is shown over the full slope embankment height. Further study may show that the armor rock can be limited to the area where embankment fill material occurs – which is the zone most susceptible to critical flow from groundwater seepage and where scour from wave and current effects is most likely. In general the existing soil below elevation -10 feet MLLW would not be susceptible to erosion from wave or ship propeller wash and therefore armor rock may not be required. Armor rock is not planned for slopes at the existing terminals, because of the fine-grained characteristics of these soils.

Inputs for the transient seepage analyses include the permeability of all soil units (Table 1) and the hydraulic boundary conditions, which were applied on the landside edge of the finite element model and also along the surface of the embankment slope. Total head of 20 feet was assumed along the landside boundary of the model to represent the constant seaward flow of the groundwater. Along the surface of the embankment slope, a transient hydraulic boundary condition was used in which the total head (sea level) was considered as a function of time (see Figure 8). It was also assumed that the horizontal hydraulic permeability of the soils is twice as much as the permeability in the vertical direction. The ratio of 2 between horizontal and vertical permeability is based on ratios typically observed in granular soils. Additional evaluation of this ratio is appropriate during later phases of design.

Results obtained from the transient seepage analyses included the time-dependent porewater pressure distribution and the gradient contours within the embankment slope. The porewater pressure distribution was then used as input for the global stability analyses, which is discussed in the next section. Figure 9 shows the uplift gradient contours at Section C (Option 1) for the most critical condition when the sea level drops to elevation -5 feet MLLW. As can be seen in Figure 9, the uplift gradient in the loose granular fill at the toe of the slope can be greater than 1.0, which indicates that a critical flow conditions will likely occur without mitigation measures. With the use of rock armor, the average uplift gradient at the slope surface was about 0.67. This gradient corresponds to a factor of safety (FS) of 1.5 against piping conditions, which is deemed acceptable. Similar results were observed from the transient seepage analyses conducted for other cross sections.

Only armor rock and embankment fill were considered in this seepage analysis. During later phases of design, it will be necessary to design filter layers that will keep finer material from filtering through the armor rock, while at the same time not inhibiting drainage. If drainage is inhibited, global stability could be affected through buildup in porewater pressures in the embankment slope.

## 7.2 Global Stability Analyses

As for the transient seepage analyses, static global stability analyses were conducted for four cross sections: (1) Section C of Option 1, (2) Section B of Option 5, (3) Section C of Option 5, and (4) Section C of Option 5-1 Hybrid. The global stability analyses were performed using a conventional limit-equilibrium methodology, in which the most critical slip surface with the lowest FS was identified from numerous trial slip surfaces. The critical slip surface was assumed to have a non-circular shape that consists of multiple linear segments defined by a finite number of vertices. The FS of the failure surface was calculated using the GLE/Morgenstern-Price method. Pinning effects of piles were not considered in the stability analyses.

### 7.2.1 Static Global Stability Analyses

Static global stability analyses were conducted for both long-term undrained and long-term drained conditions. The end-of-construction case was not considered in the global stability analyses as the long-term FS of the embankment slopes was expected to be less than the short-term FS due to the removal of the overlying fill, which leads to reduction of the undrained shear strength of the BCF clay.

For the long-term undrained condition, the BCF clay was divided into three zones: (1) under the new sheet pile bulkhead, (2) under the embankment slope, and (3) in front of the embankment slope. The SHANSEP undrained shear strength of the BCF clay under the embankment slope and sheet pile bulkhead was calculated using the average overburden stress associated with each zone. The pre-construction undrained shear strength – after dredging to the final dredge elevations – was used for the BCF clay in front of the slope. For the long-term drained case, the shear strength of the BCF clay was modeled using a friction angle of 30 degrees with zero cohesion.

Due to the transient flow in the slope body, the shear strength of the estuarine silt was characterized by using the undrained shear strength ratio ($Su/\sigma'_v$) with $\sigma'_v$ varying with tidal elevation. The porewater pressure distribution used in both undrained and drained cases was obtained from the transient seepage analyses, which were previously described in Section 7.1.

An external surcharge equal to 1,000 psf was assumed on top of the sheet pile bulkhead in the static global stability analyses. Figures 10 and 11 show the variation of the FS obtained from the long-term undrained and the long-term drained cases for the embankment slopes considered in the analyses. As shown in these figures, the FS of the embankment slopes changed quite noticeably during the day as the tidal elevation varied. The lowest FS value for both long-term undrained and long-term drained cases was approximately 1.4, which was calculated when the sea level dropped to the lowest point (elevation -5 feet MLLW).

This minimum computed FS value in Figures 10 and 11 is slightly less than the targeted value of 1.5. Although there is an increase in the potential for instability when the FS drops below the target value of 1.5, this condition only exists for a small percentage of the year when extreme tidal conditions occur. Given this short duration and since the margin of capacity versus demand is still 40%, the FS of 1.4 is considered acceptable for this 15% level of design. Additional evaluations should be conducted in future phases of design to confirm that adequate stability occurs. Factors such as friction value in the embankment fill and the pinning effects of piles in the embankment will likely contribute to better stability than estimated at this concept level.

### 7.2.2 Pseudo-Static Global Stability Analyses

The global stability of the embankment slopes during the design seismic events (that is, OLE, CLE, and MCE) was evaluated using the pseudo-static stability approach. In the pseudo-static analysis, the lateral seismic coefficient was assumed to be half of the peak horizontal ground acceleration (that is, 0.5 x PGA). The 50% reduction in the PGA for pseudo-static global stability analyses is based on guidance provided in AASHTO (2012) and Anderson et al. (2008). Designs based on this reduction will have small amounts of permanent slope movement when the FS for global stability is approximately 1.0.

All fine-grained soils including the BCF clay and the estuarine silt were assumed to behave in an undrained manner during the design seismic events. A small cohesion of 100 psf was considered in modeling the shear strength of the non-liquefied cohesionless soils to account for the short-term undrained behavior of these soils in a seismic event. An external surcharge equal to 20% of the live load (that is, 200 psf) was assumed on top of the sheet pile bulkhead in the pseudo-static analyses. The sea level was assumed to be at elevation +7.5 feet MLLW, whereas the landside groundwater was assumed to be at elevation +20 feet MLLW. The basis of the tidal and groundwater elevations was discussed in more detail in the suitability study (CH2M HILL, 2013).

For the pseudo-static loading cases, the cyclic degradation of the shear strength was considered for soils that were potentially affected by cyclic loading. The following assumptions were made for the shear strength reduction of these soils in the cyclic loading condition:

- **BCF clay.** The cyclic and static undrained shear strength were assumed to be the same (that is, no cyclic strength degradation).
- **Estuarine Deposits.** The $S_u/\sigma'_v$ ratio of the Estuarine Deposits was assumed to decrease by 20% (from 0.55 to 0.44).

- **Loose Granular and Common Fill.** The friction angle of the loose granular fill was assumed to decrease from 34 to 28 degrees to account for the shear strength reduction due to excess porewater pressure buildup. This reduction in the friction angle is equivalent to about 20% reduction in the shear strength of the soil. Similarly, the friction angle of the common fill was assumed to decrease from 32 to 27 degrees.

Results from the pseudo-static global stability analyses indicate that the FS for the OLE case was equal to or greater than 1.2 for all embankment slopes. This finding suggests that earthquake-induced slope movement, if any, will likely be very small.

The FS values of all embankment slopes under the CLE and MCE-level loading ranged from 0.8 to 1.1, indicating that minor to considerable slope movement would be expected during the CLE and MCE events. Table 3 provides the summary of the FS values obtained from the pseudo-static global stability analyses. The seismic-induced permanent deformation of the embankment slopes is discussed in the next section.

TABLE 3
**Results from the Pseudo-Static Global Stability Analyses for All Considered Embankments**

| Case No. | Seismic Event | Global Factor of Safety | | | | Range of Seismic-Induced Slope Movement |
|---|---|---|---|---|---|---|
| | | Option 1 | Option 5 | | Option 5-1 Hybrid | |
| | | Section C | Section B | Section C | Section C | |
| 1 | OLE | 1.2 | 1.2 | 1.2 | 1.3 | No to minor movement |
| 2 | CLE | 0.9 | 0.9 | 1.1 | 1.0 | Minor to considerable movement |
| 3 | MCE | 0.8 | 0.8 | 1.1 | 0.9 | Minor to considerable movement |

Notes:

CLE = contingency level earthquake

MCE = maximum considered earthquake

OLE = operating level earthquake

## 7.3 Seismic-Induced Permanent Deformation

In this section, the seismic-induced permanent deformation estimated by simplified methods is discussed. The simplified methods used in this study are based on the Newmark sliding-block methodology. The simplified Newmark sliding-block methods used in the 15% concept design include Hynes-Griffin and Franklin (1984), Ambraseys and Menu (1988), Bray and Travasarou (2007), and Rathje and Saygili (2008). The final seismic-induced permanent deformation was estimated as a weighted average of those calculated by the four methods using the following equation:

$$D = 0.15*D_1 + 0.15*D_2 + 0.35*D_3 + 0.35*D_4 \qquad \text{[Equation 4]}$$

Where:

$D$ = weighted average seismic-induced permanent deformation

$D_1$ = seismic-induced permanent deformation estimated by Hynes-Griffin and Franklin (1984)

$D_2$ = seismic-induced permanent deformation estimated by Ambraseys and Menu (1984)

$D_3$ = seismic-induced permanent deformation estimated by Bray and Travasarou (2007)

$D_4$ = seismic-induced permanent deformation estimated by Rathje and Saygili (2008)

The yield accelerations calculated from the pseudo-static global analysis for all embankment slopes are:

- Option 1, Section C $k_y$ = 0.12 g
- Option 5, Section B $k_y$ = 0.12 g
- Option 5 – Section C: $k_y$ = 0.12 g
- Option 5-1 Hybrid – Section C: $k_y$ = 0.15 g

Results obtained from the deformation analyses using simplified Newmark sliding-block methods are summarized in Table 4.

TABLE 4
**Seismic-Induced Permanent Deformation Calculated by Simplified Rigid-Block Methods**

| Seismic Event | Magnitude | Avg. PHGA from SHAKE 2000 (g) | Estimated Seismic-Induced Permanent Deformation (inch) | | | |
|---|---|---|---|---|---|---|
| | | | Option 1 | Option 5 | | Option 5-1 Hybrid |
| | | | Section C | Section B | Section C | Section C |
| OLE | 6.1 | 0.16 | 1 | 1 | 1 | Minimal |
| CLE | 6.3 – 7.5 | 0.31 | 3 | 3 | 3 | 2 |
| MCE | 6.6 – 9.2 | 0.39 | 6 | 6 | 6 | 4 |

Compared with the deformation criteria shown in Table 2, the seismic-induced permanent deformations calculated for all embankment slopes appear to be acceptable.

## 7.4 Foundation Design Recommendations

This section provides the geotechnical recommendations for the pile foundation design. Both axial and lateral resistances of the piles are discussed. Also provided in this section are the geotechnical inputs for the lateral response analysis of piles subjected to seismic-induced slope movement.

### 7.4.1 Recommended Resistance Factors for LRFD Pile Design

Resistance factors for side, base, and uplift resistance components of the steel pipe piles are provided in Table 5. These resistance factors are consistent with those recommended by AAHSTO *LRFD Bridge Design Specifications* (2012) for driven piles. Higher resistance factors could be used for pile design if static and/or dynamic load testing (that is, pile driving analyzer [PDA]) is conducted in the future.

TABLE 5
**Resistance Factors for LRFD Design of Pipe Piles**

| Limit State | Resistance Factor, Φ | | | |
|---|---|---|---|---|
| | Side Resistance | Base Resistance | Uplift Resistance | Lateral Resistance |
| Strength | 0.45 | 0.45 | 0.35 | 1.0 |
| Extreme | 1.0 | 1.0 | 0.80 | 1.0 |
| Service | 1.0 | 1.0 | N/A | 1.0 |

### 7.4.2 Axial Capacity of Piles

The nominal and service capacities of the 48-inch steel pipe piles were calculated using the API method (2003). The service capacity of the piles was determined as the pile load that induces 1 inch of settlement. Because the piles are driven to the planned toe elevation without a bottom plate (that is, open-ended), both “plugged” and “unplugged” conditions were considered in the axial capacity calculations. The end-bearing resistance of the piles was considered as the lesser of the following components:

- Sum of the end-bearing resistance from the pile annular area and the skin friction along the inner surface of the piles, or
- End-bearing resistance calculated using the gross toe area of the piles.

The following soil profiles were considered in the axial pile capacity calculations:

- **Soil profile 1**. This soil profile consists of the BCF Clay overlying the Glaciofluvial Deposits. This profile is suitable for the piles supporting the wharves or the trestle piles that are located near the toe of the embankment slope.
- **Soil profile 2.** This soil profile consists of a layer of fill overlying the BCF Clay and the Glaciofluvial Deposits. The thickness of the fill layer was estimated based on the mid-slope elevation. This soil profile is suitable for the piles located near the midsection of the trestle.
- **Soil profile 3.** This soil profile consists of a layer of fill overlying the BCF Clay and the Glaciofluvial Deposits. The thickness of the fill layer was estimated using the crest elevation of the embankment slope (that is., toe elevation of the bulkhead). This profile is suitable for the piles located near the back of the trestle.

The nominal axial capacity charts associated with each design option are shown in Figures 12 through 16. The service capacities associated with each design option can be estimated from load-displacement curves, which are shown in Figures 17 through 20.

The following factors should be considered when estimating the pile length:

- The manner in which loads are developed during construction was not considered in the evaluation of the service capacity. Depending on the construction sequencing, it may be possible to accept larger settlements, and hence larger service capacity, if some of the settlement occurs before the placement of the deck panels or other critical components of wharves or trestles.
- Group effects can be ignored when the center-to-center spacing between adjacent piles is greater than 6 pile diameters. The group reduction factors ($\eta$) should be taken as 1.0 and 0.65 for center-to-center spacings of 6 and 2.5 pile diameters, respectively. For intermediate spacings, the group reduction factor should be determined by linear interpolation.
- The nominal uplift resistance of piles is assumed to be equal to the nominal side resistance. The nominal side resistance should be used in combination with the uplift resistance factors provided in Table 5.

### 7.4.3 Geotechnical Parameters for the Lateral Response Analysis of Piles

The lateral response of the 48-inch pipe piles will be analyzed using the program LPILE (Wang and Reese, 1993), which is based on the P-y spring approach. In this approach, the surrounding soils are modeled as a series of springs, which are represented by non-linear P-y curves. The initial slope of the P-y curves is either defined by the modulus of subgrade reaction ($k$) when the soils are cohesionless or by the secant shear strain ($\varepsilon_{50}$) when the soils are cohesive. The modulus of subgrade reaction and the secant shear strain can be estimated using the soil friction angle ($\phi$) and the undrained shear strain ($S_u$), respectively.

Geotechnical parameters for use in the lateral response analysis of the 48-inch pipe piles are provided in Table 6. These parameters were developed following guidance provided by Wang and Reese (1993). When using the LPILE properties, sensitivity studies should be conducted to account for variations in P-y parameters specific to project conditions, including the difference between upslope and downslope response of the piles. A variation of plus 100% and minus 50% is considered at this level of design. Sensitivity checks should also be conducted to evaluate the effects of different unit weights and friction angles, particularly in the armor rock and riprap layer once the type of armor or riprap has been identified. For example, a uniformly grade rock that is placed carefully could have a higher unit weight and a higher friction angle. These conditions could result in increased pinning at the top of the pile during lateral loading.

TABLE 6
**Inputs for the Lateral Response Analyses**

| Soil Unit | Description | Effective Unit Weight, $\phi$ (pci) | Friction Angle (degrees) | Undrained Shear Strength, $S_u$ (psi) | Subgrade Modulus Reaction, k (pci) | Secant Shear Strain, $\varepsilon_{50}$ (--) |
|---|---|---|---|---|---|---|
| 1 | Rock Armor and Riprap (cohesionless) | 0.044 | 40 | -- | 140 | -- |
| 2 | Estuarine Silt at North Extension and WBB (cohesionless) | 0.032 | 32 | -- | 28 | -- |
| 3 | Estuarine Silt and Sand at Terminals (cohesionless) | 0.035 | 36 | -- | 75 | -- |
| 4 | BCF Clay (cohesive) | 0.032 | -- | 18.75 to 34.72 (*) | -- | 0.005 to 0.004 (**) |
| 5 | Glaciofluvial Deposits (cohesionless) | 0.038 | 45 | -- | 185 | -- |

Notes:

* Undrained shear strength increases in a linear fashion from the top to the bottom of the BCF clay.

** Secant shear strain decreases in a linear fashion from the top to the bottom of the BCF clay.

pci = pounds per cubic inch

psi = pounds per square inch

### 7.4.4 Geotechnical Parameters for the Lateral Spreading Analysis

Seismic-induced permanent deformation of the slope (that is, lateral spreading) can create an additional lateral load on the piles. In addition to its use for analyzing the lateral response of the 48-inch pipe piles, the program LPILE can be also used to evaluate the impact of seismic-induced slope movement on the pile foundation. In this analysis, the free-field displacement (that is, lateral spreading) was applied to the soils surrounding the piles. The magnitude and depth of lateral spreading are provided in Table 7. The lateral spreading was assumed to be uniform over the depth of the moving soil. The depth of lateral spreading was estimated using the location of the failure surface obtained from the pseudo-static global stability analyses. The lateral load induced by the moving soil should be treated as an external load in Extreme Limit State (that is, resistance factor = 1.0). Generally, the resistance of the pile to lateral soil response should develop within approximately 5 pile diameters of the depth at which sliding occurs.

TABLE 7
**Soil Movement (Lateral Spreading) under Seismic Loading**

| Design Option | Cross Section | Yield Acceleration | Magnitude of Lateral Spreading (inches) | Elevation at Top of Stationary Layer (feet, MLLW) |
|---|---|---|---|---|
| Option 1 | C | 0.12 *g* | 6 | -55 |
| Option 5 | B | 0.12 *g* | 6 | -40 |
| | C | 0.12 *g* | 6 | -25 |
| Option 5-1 Hybrid | C | 0.15 *g* | 4 | -25 |

## 7.5 Drivability Analyses

The assumptions used to evaluate the drivability of the 48-inch steel pipe piles are provided in Section 2.3. Findings from the evaluation were:

- Results suggested that it would be possible to drive the 48-inch pipe piles to the bottom of the BCF Clay layer (elevation -150 feet MLLW at the North Extension; elevation -180 feet MLLW at the terminals) using a Pileco D280 hammer. However, it would be challenging to drive the 48-inch pipe piles into the underlying Glaciofluvial Deposits with this hammer. Indicator pile testing with PDA measurements would be required to further refine identification of the depth that could be achieved with different hammer sizes.
- The axial capacity of the 48-inch pipe piles estimated from the drivability analyses ranged from 1,500 to 2,000 kilo-pounds (kips) when the piles were terminated in the BCF Clay. The axial capacity increased significantly when the piles were driven into the Glaciofluvial Deposits. If axial capacity of more than 2,000 kips would be required, it is likely that the piles will have to be driven into the Glaciofluvial Deposits.
- Compressive stress in the piles during driving could be up to 30 ksi. The tensile stresses during driving ranged from 5 to 10 kilo-pounds per square inch (ksi).

## 7.6 Lateral Earth Pressure Coefficients for Wall Design

The lateral earth pressures for the design of the sheet pile bulkheads and retaining walls were estimated assuming a vertical wall and a level ground surface behind the wall. For the cellular sheet pile bulkhead design, the fill within the circular cells was assumed to be compacted with unit weight of 130 pounds per cubic foot (pcf) and friction angle of 40 degrees. For the retaining walls at the terminal sites (Option 5), the fill behind the wall was assumed to be un-compacted Common Fill with unit weight of 125 pcf and friction angle of 32 degrees.

Because lateral movement of the compacted fill within the circular cells would be restricted, the at-rest earth pressure coefficient ($K_0$) should be used to estimate the lateral earth pressure inside the cellular sheet pile bulkhead. For the retaining walls at the existing terminals (Option 5), the active earth pressure coefficient ($K_a$) should be used to estimate the lateral earth pressure on the wall. A surcharge load of 1,000 psf should be used during these computations.

For seismic loading conditions, the seismic component of the active lateral earth pressure can be estimated using the coefficient ($\Delta K_{ae}$), which can be calculated by the Mononobe-Okabe method. Under seismic loading, the cellular sheet pile bulkhead should have sufficient flexibility to be similar to a cantilevered retaining wall. It was also assumed that both retaining structures may translate back and forth several inches during the seismic event. As a result, the horizontal seismic coefficient ($k_h$), which was used in calculating $\Delta K_{ae}$, was assumed to be half the PGA value. The recommended lateral earth pressure coefficients are summarized in Table 8. A surcharge load of 200 psf should be used when estimating the lateral earth pressure for seismic loading.

TABLE 8
**Lateral Earth Pressure Coefficients for Retaining Wall Design**

| Parameter | Value |
|---|---|
| Total unit weight of compacted fill within the cellular bulkhead, $\gamma$ (pcf) | 130 |
| Total unit weight of Common Fill, $\gamma$ (pcf) | 125 |
| At-rest lateral earth pressure coefficient, $K_0$ | 0.36 |
| Active lateral earth pressure coefficient, $K_a$ | 0.28 |
| Incremental seismic lateral earth pressure coefficient ($\Delta K_{ae}$) for OLE event | 0.05 |
| Incremental seismic lateral earth pressure coefficient ($\Delta K_{ae}$) for CLE event | 0.10 |
| Incremental seismic lateral earth pressure coefficient ($\Delta K_{ae}$) for MCE event | 0.14 |

Notes: Passive earth pressure in front of the retaining walls and sheet pile bulkhead was assumed to be negligible.

pcf = pounds per cubic foot

# 8 Issues to be Considered in the Final Design

The geotechnical analysis and discussions presented in the previous section, as well as other supporting information in this technical memorandum, are based on a 15% level of design. This level of design is sufficient for geotechnical feasibility assessments and preliminary cost evaluations; however, additional geotechnical evaluations will be required for final design. These future evaluations could include additional field exploration and laboratory testing, final embankment and foundation design studies, and pre-production pile load testing. A number of issues related to the planned construction approach will also need to be discussed to confirm that geotechnical design methods being used for final design are consistent with planned construction methods. The followings subsections identify a number of these future design and construction issues.

## 8.1 Design-Related Issues

The following issues should be considered in the final design:

- The transient seepage analysis should be refined. The current analysis made some broad assumptions about the hydraulic properties of the soils, as well as boundary conditions (that is, the total head on both seaside and landside boundaries). These assumptions must be re-evaluated and adjusted if necessary in the final design.
- Soil-structure interaction under static and dynamic loading conditions should be considered in modeling the pile-supported wharves and the embankment slopes. The soil-structure interaction analyses should be conducted using either 2D or 3D finite element or finite difference models. These methods are better able to evaluate static and earthquake-induced slope deformations, particularly issues such as inertial response of the wharf and kinematic loading from ground movement.
- The shear strength reduction of the BCF Clay, especially under large earthquake-induced slope deformation, should be confirmed for final design. The BCF Clay encountered at the project site has a high potential for strain-softening (that is, loss of strength at large shear strain) during large soil displacements. It is critical that design confirm that such deformations do not develop during OLE, CLE, or MCE events.
- Liquefaction potential and the impact of liquefaction on the piles and bulkheads should be re-evaluated and adjusted in the final design to confirm that liquefaction within the granular backfill does not result in unacceptable loading to the piles or loss of support to the bulkhead wall. These analyses will require additional soil-structure interaction analyses and may require further evaluation of the effects of granular fill removal on the fill making up the final embankment slopes.

- The slope protection design (against a critical flow gradient and erosion) should be refined. These refinements include the gradation and thickness of bedding layers to meet filter requirements, as well as required sizes of armor to meet wave and ice loading conditions.
- A number of additional studies will be required to finalize pile selection. For example, axial capacity calculations for pipe piles should be refined considering other capacity predictive methods such as CPT-based analysis. Various pile size and types could be considered and optimized if necessary. The resistance factors used in the pile foundation analysis should also be adjusted if static and dynamic load tests are to be conducted. Using higher resistance factors will result in shorter or smaller diameter piles. Drivability analysis should be refined considering various hammer types and sizes. The capacity (both skin and end bearing) mobilized during pile driving should be re-evaluated.
- Other alternatives for bulkhead and retaining walls should be considered in the final design. In addition, the suitability of the existing sheets (salvaged from the existing OCSP® system) for sheet pile bulkhead and retaining wall construction must be re-evaluated. Other alternatives for stabilization of the bulkhead and retaining wall should be considered in the final design.

## 8.2 Construction-Related Issues

The following issues should be considered in the development of the design concept and construction planning in the final design:

- The 15% concept design currently assumes that the existing OCSP® system and retained fill would be removed prior to construction of the embankment slope and pile-supported wharves. Construction methodology and staging for removal of the existing OCSP® system and the retained fill must be fully reviewed for feasibility in the final design. At least a part of the fill contained within the cells must be removed before the sheets can be pulled or cut.
- The 15% concept design assumes that the embankment slopes under the new wharves will be protected by a layer of riprap to prevent a critical flow condition and erosion of granular soil at the slope surface. The placement of riprap will likely have to be done from a barge (hydraulic fill). Sequencing of bedding layer and armor rock placement must be evaluated relative to installing piles and the potential for pile damage. The 15% concept design assumes that the 48-inch steel pipe piles will be driven through the embankment fill without having significant toe damage and that riprap or armor rock above the embankment fill will be placed after the piles are driven. Contract documents will have to include specific wording regarding removal of exposed obstructions from existing armor rock or boulders that could exist at the site. These provisions should cover spudding or drilling to knock buried rock or boulder obstructions out of the way, if encountered, and payment for removal of obstructions.
- Field pile-load testing involving both static (top down) capacity tests and dynamic tests (PDA with CAPWAP) during pile installation should be planned. These tests have an important benefit relative to the length and size of production piles. These pre-installation tests can be used to establish pile hammer requirements, pile drivability, formation of plugs with the pile, and ultimate side and toe capacities. Load tests also allow use of higher resistance factors in the design, which results in better optimization of the production piles.
- The 15% concept design assumes that up to 50% of the required sheets for the cellular sheet pile bulkhead and retaining wall will come from new sheets existing at the POA that were purchased for the OCSP® structures. Undamaged sheets removed during demolition of the existing OCSP® structure may also be used for construction of the cellular sheet pile bulkhead and retaining wall, depending on their conditions after removal. These assumptions must be re-evaluated in the final design, particularly relative to installation requirements. The 15% concept design also assumes that the fill within the cellular sheet pile bulkhead and the Estuarine Deposits under the footprint of the bulkhead can be vibro-compacted so that a friction angle of 40 degrees will be achieved. The suitability of the vibro-compaction method must be re-evaluated in the final design as the fine content of the fill and Estuarine Deposits can be quite high. Ground improvement using

stone columns may be considered if vibro-compaction is deemed unsuitable for the type of material encountered within and under the bulkhead.

- The 15% concept design assumes that micropiles will be used to stabilize the retaining wall (Option 5) and the embankment slopes (Option 5-1 Hybrid). The constructability and suitability of micropiles must be re-evaluated in the final design. Field testing may be required to evaluate the lateral capacity of the micropiles. Additional background describing the use of micropiles can be found in Section 6 of *Micropile Design and Construction Reference Manual* (FHWA NHI 05-039), December 2005 and Section 10.9 of the 2012 *AASHTO LRFD Bridge Design Specifications.*

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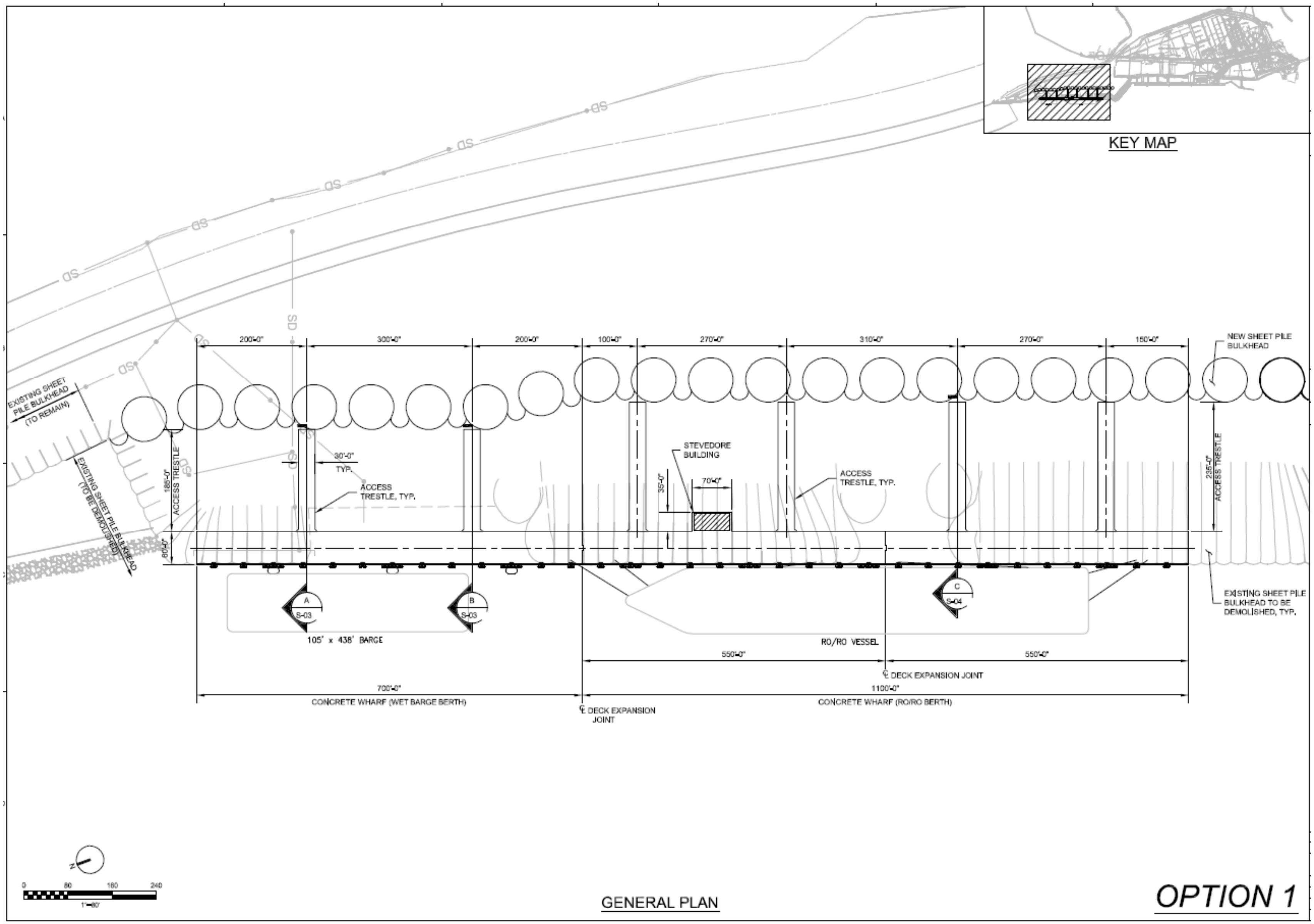


**Figure 1.** Proposed design option 1.

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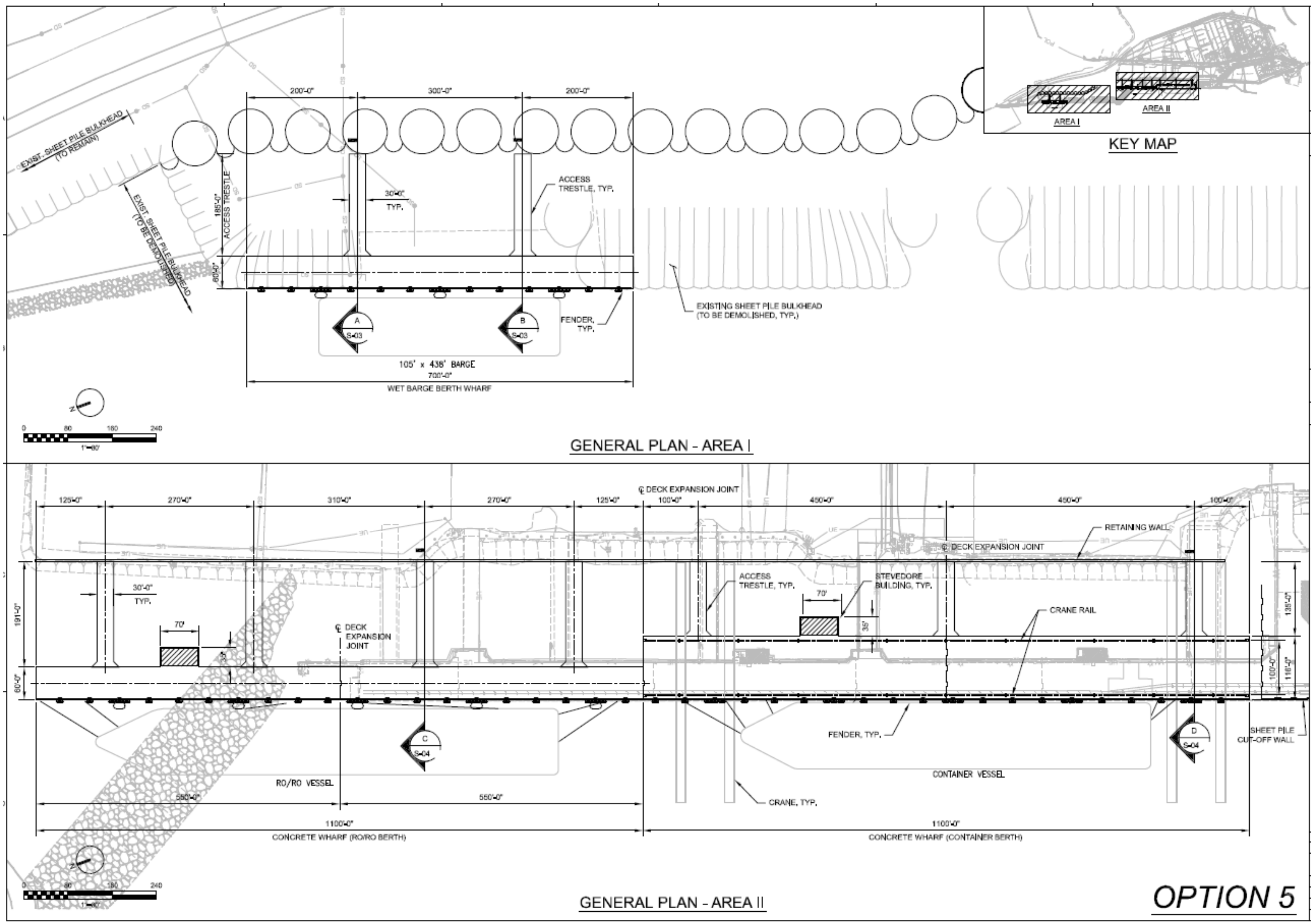


**Figure 2.** Proposed design option 5.

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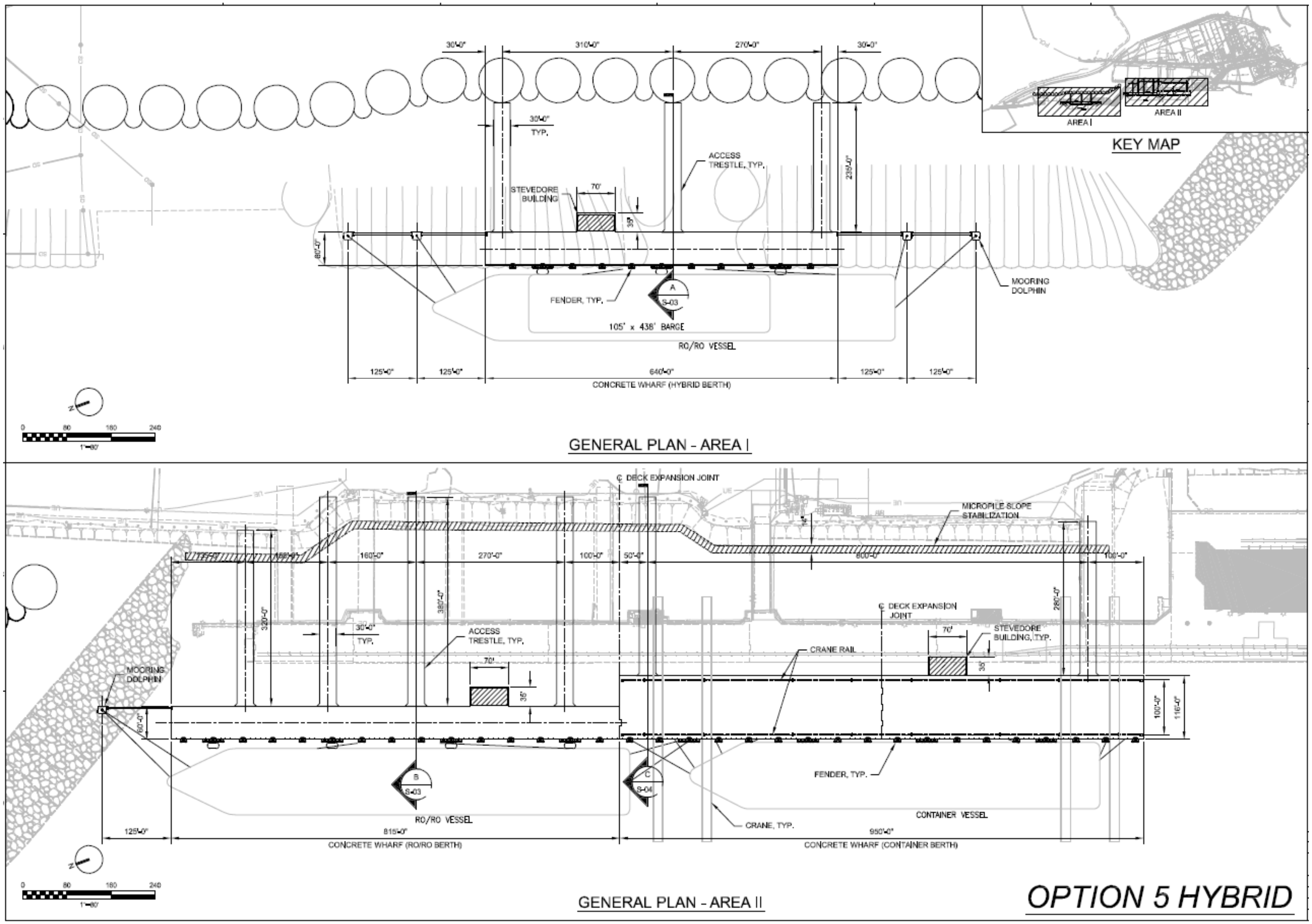


**Figure 3.** Proposed design option 5-1 Hybrid.

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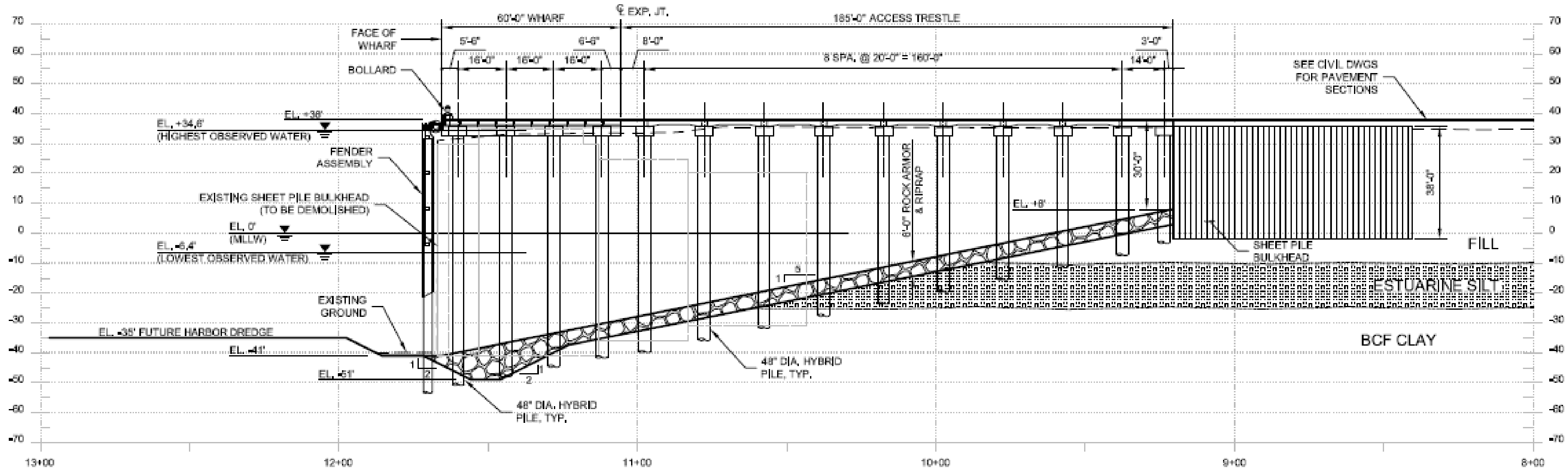


**Figure 4.** Design soil profile at the Wet Barge Berth area (Section B, Option 5).

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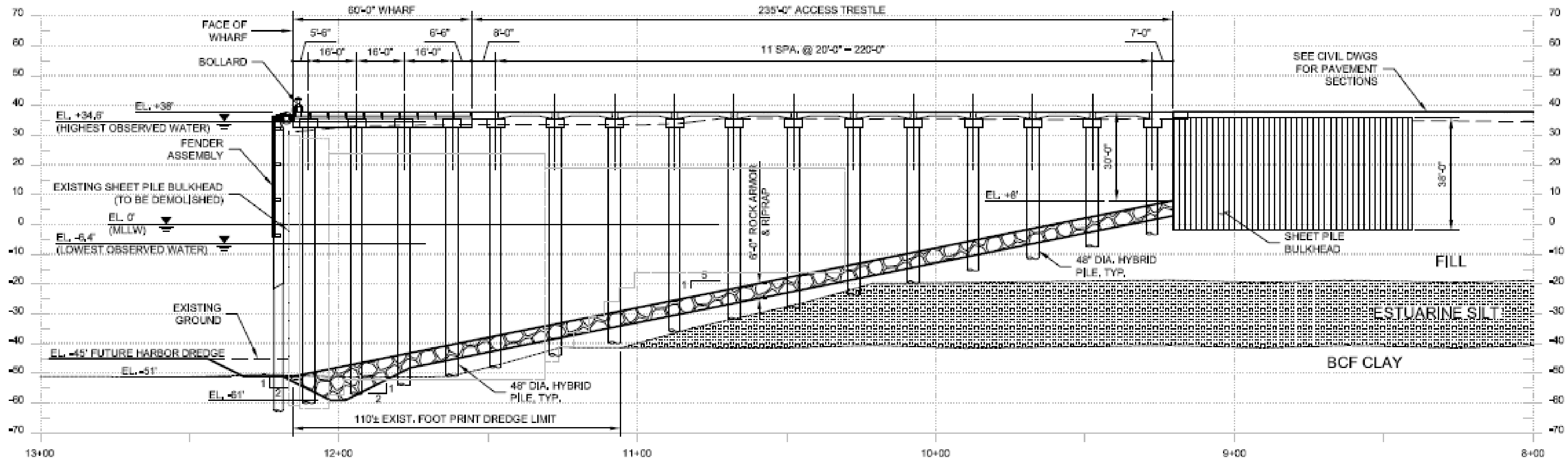


**Figure 5.** Design soil profile at the North Extension area (Section C, Option 1).

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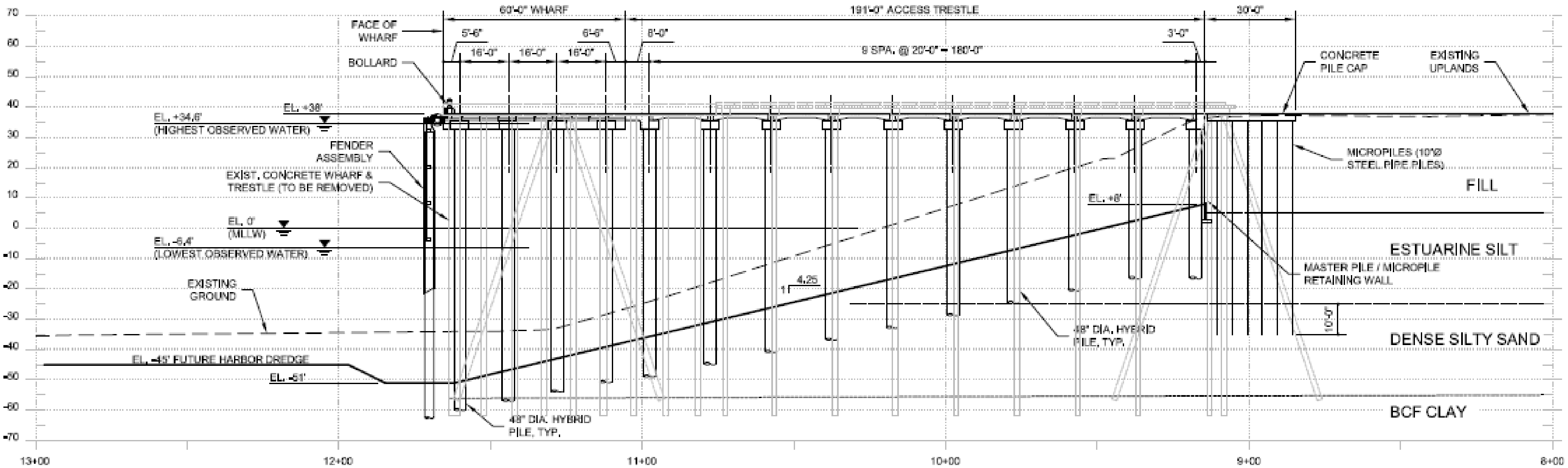


**Figure 6.** Design soil profile at Terminal 2 area (Section C, Option 5).

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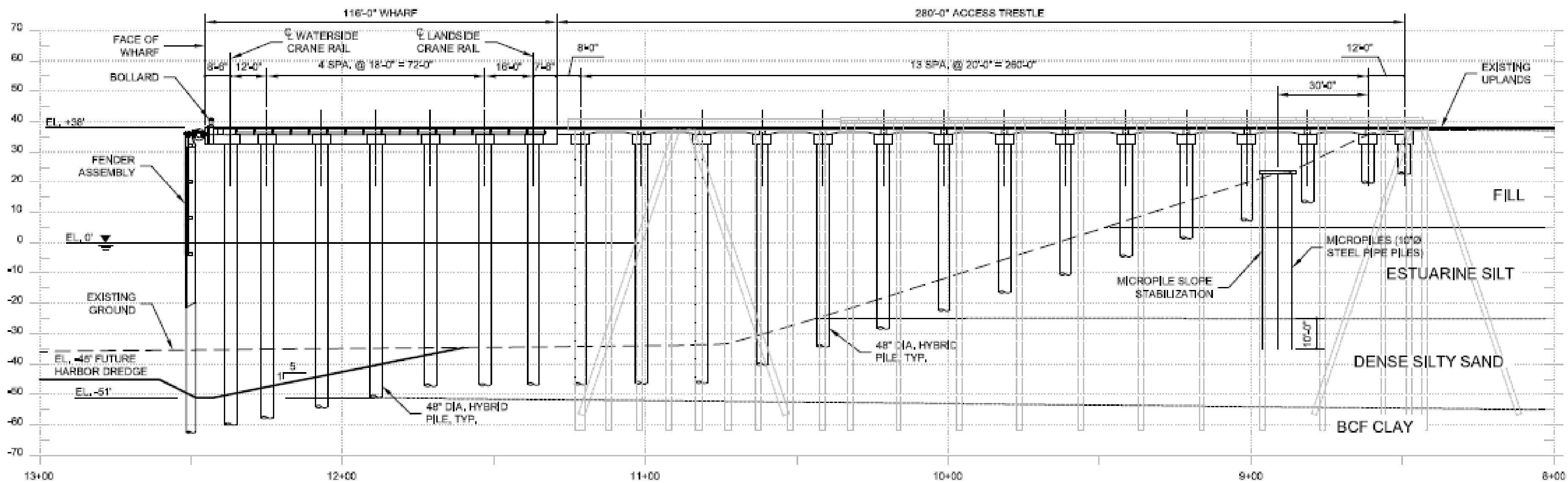


**Figure 7.** Design soil profile at Terminal 3 area (Section C, Option 5-1 Hybrid).

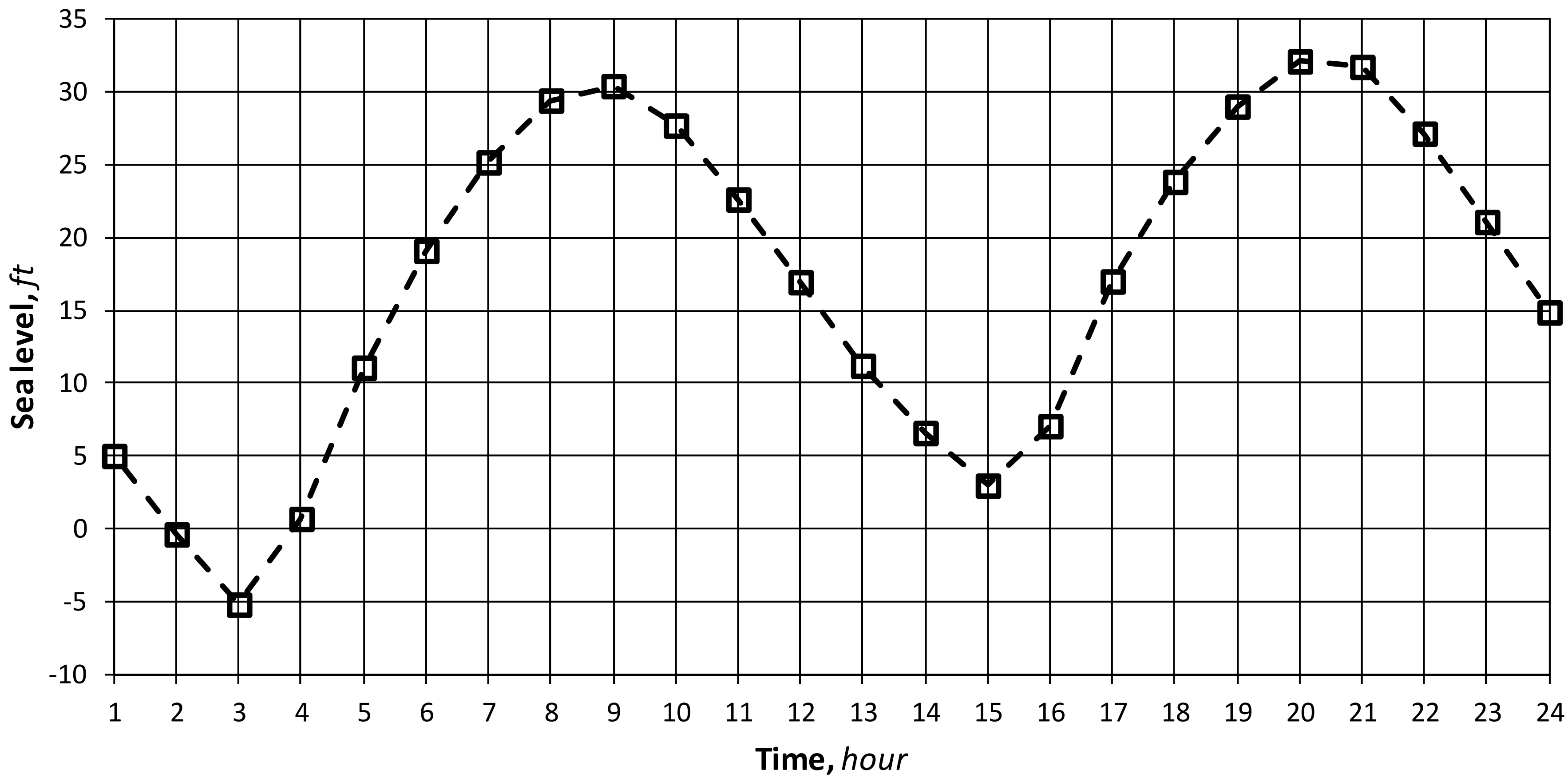


**Figure 8.** Typical tidal curve in a 24-hour period.

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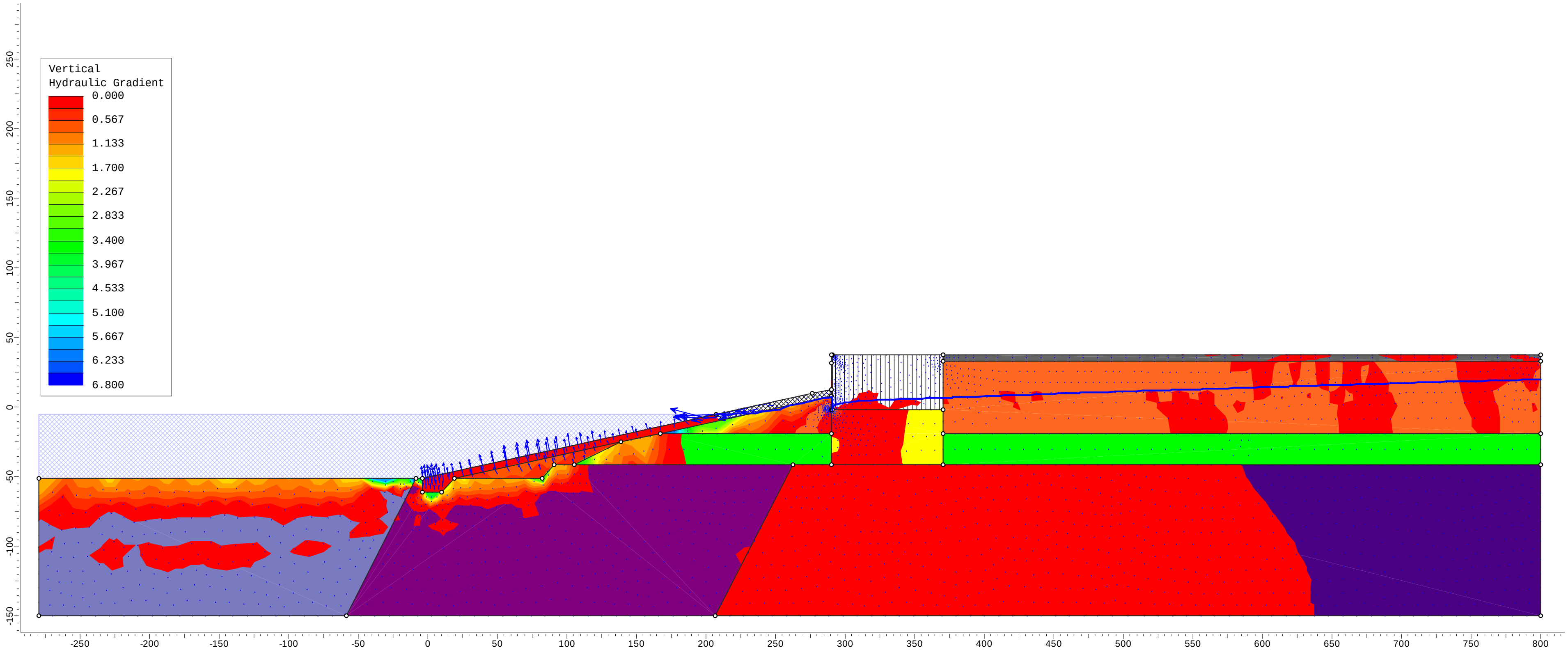


**Figure 9.** Uplift gradient contours at Section C (Option 1) when sea level dropped to EL. -5 ft (MLLW).

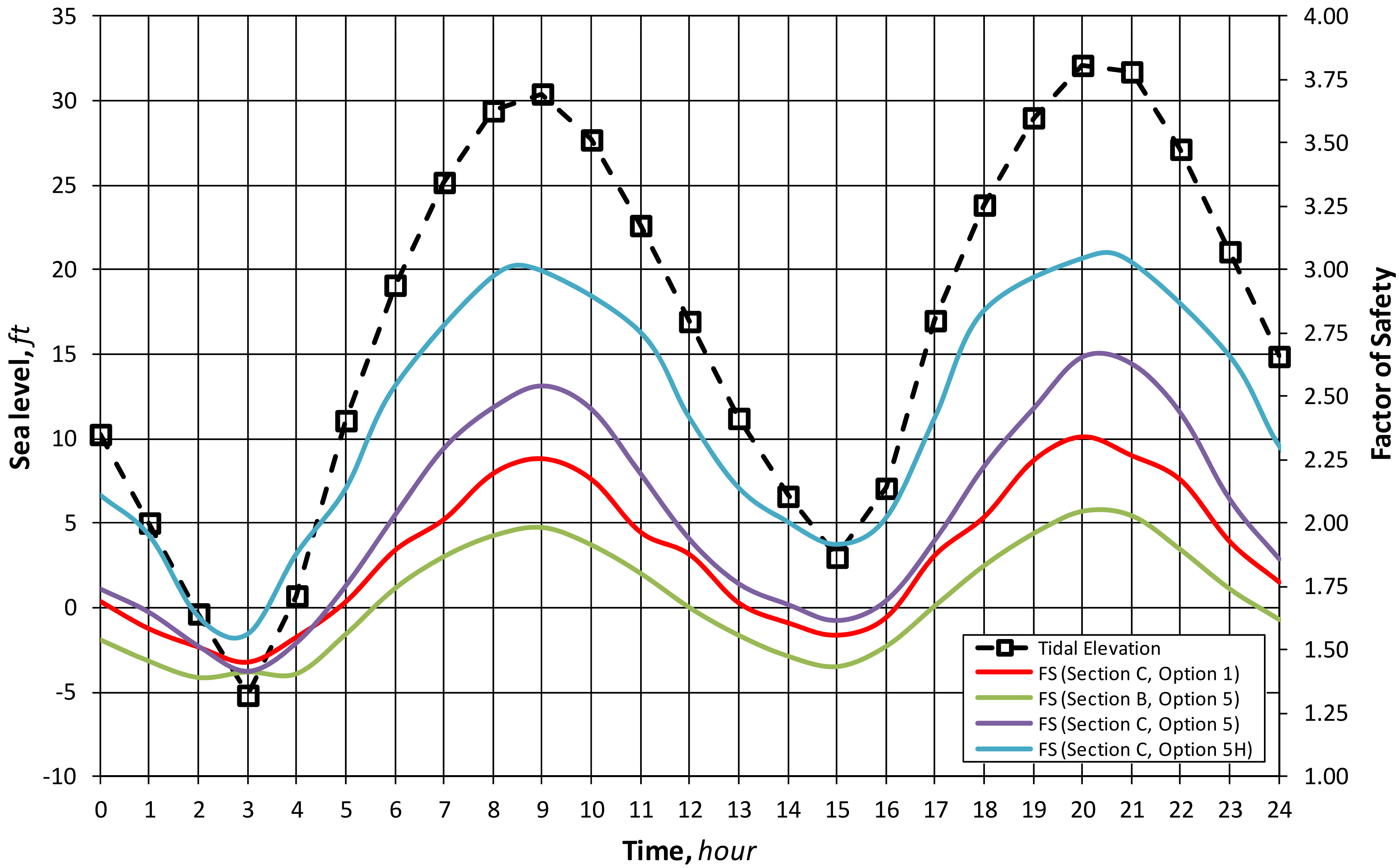


**Figure 10.** Variation of long-term undrained FS as tidal elevation changes during the day.

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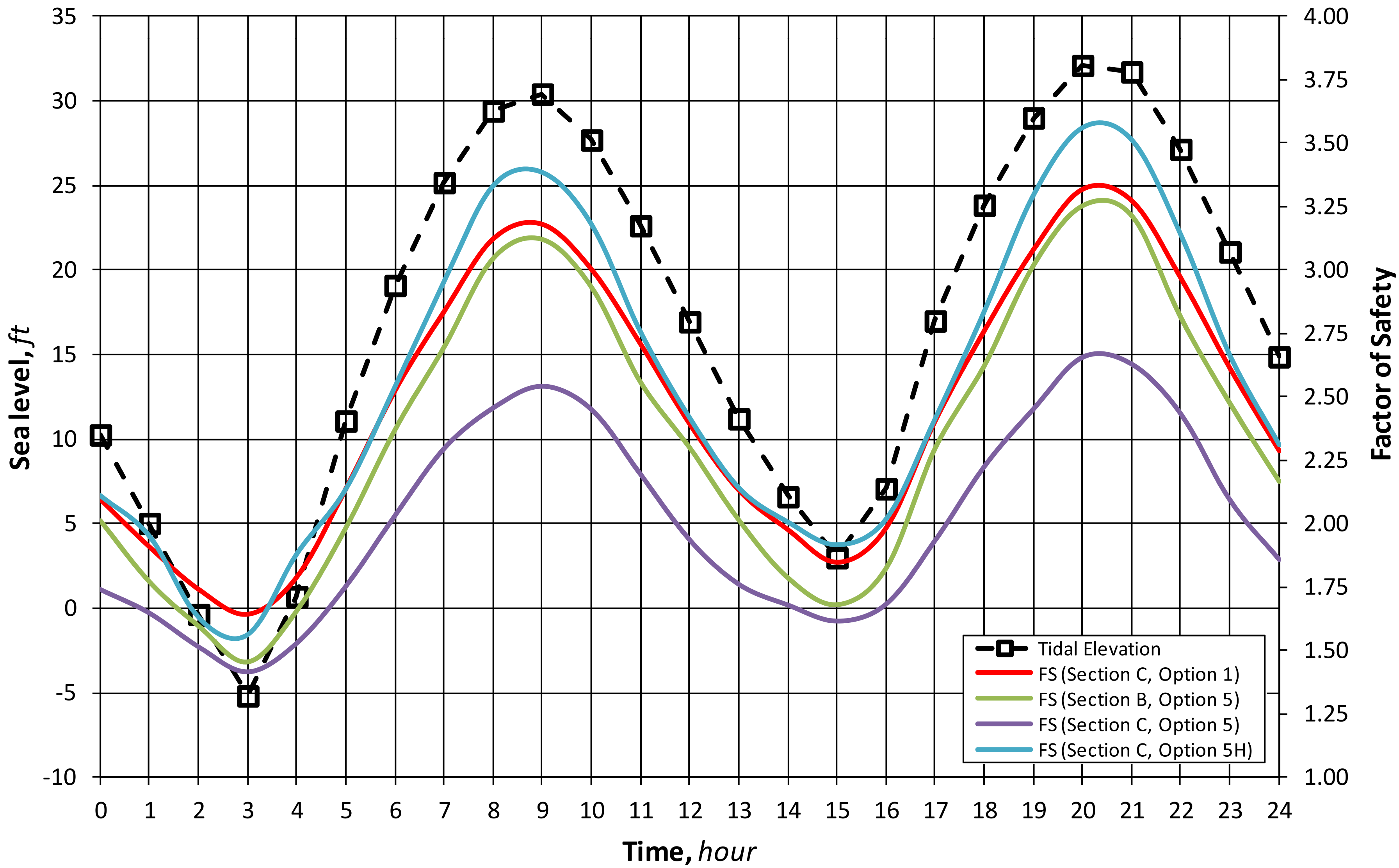


**Figure 11.** Variation of long-term drained FS as tidal elevation changes during the day.

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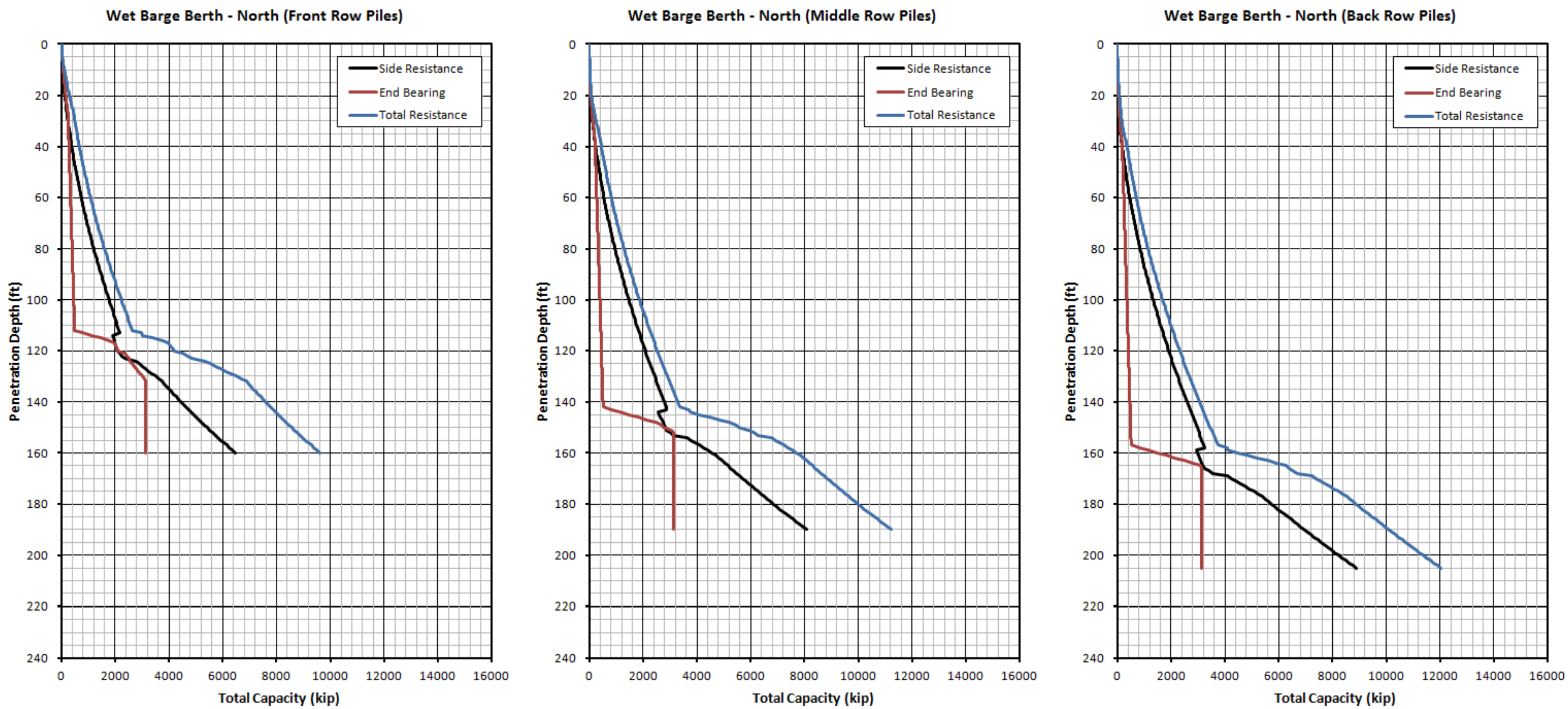


**Figure 12.** Nominal capacity charts of 48-inch pipe piles at the north end of the Wet Barge Berth.

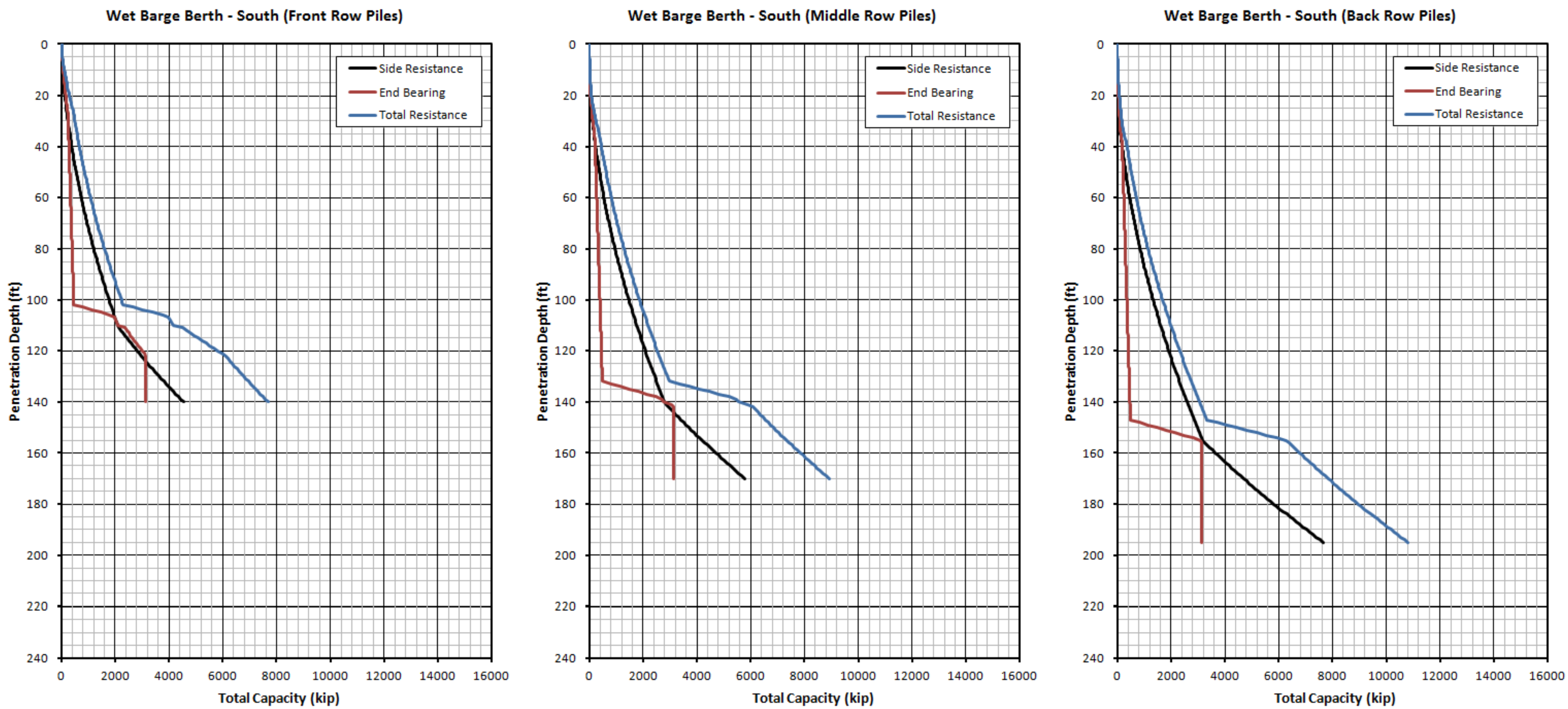


**Figure 13.** Nominal capacity charts of 48-inch pipe piles at the south end of the Wet Barge Berth.

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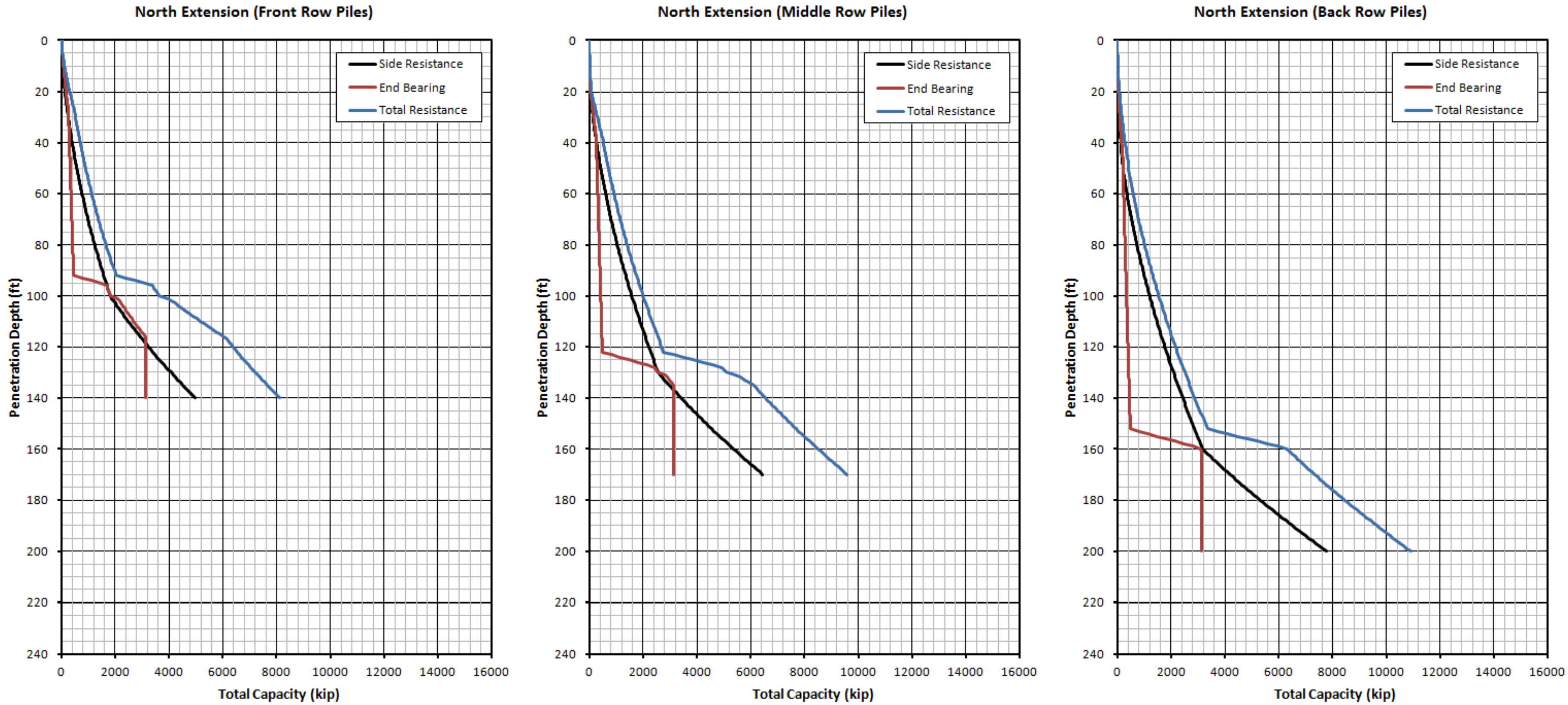


**Figure 14.** Nominal capacity charts of 48-inch pipe piles at the North Extension.

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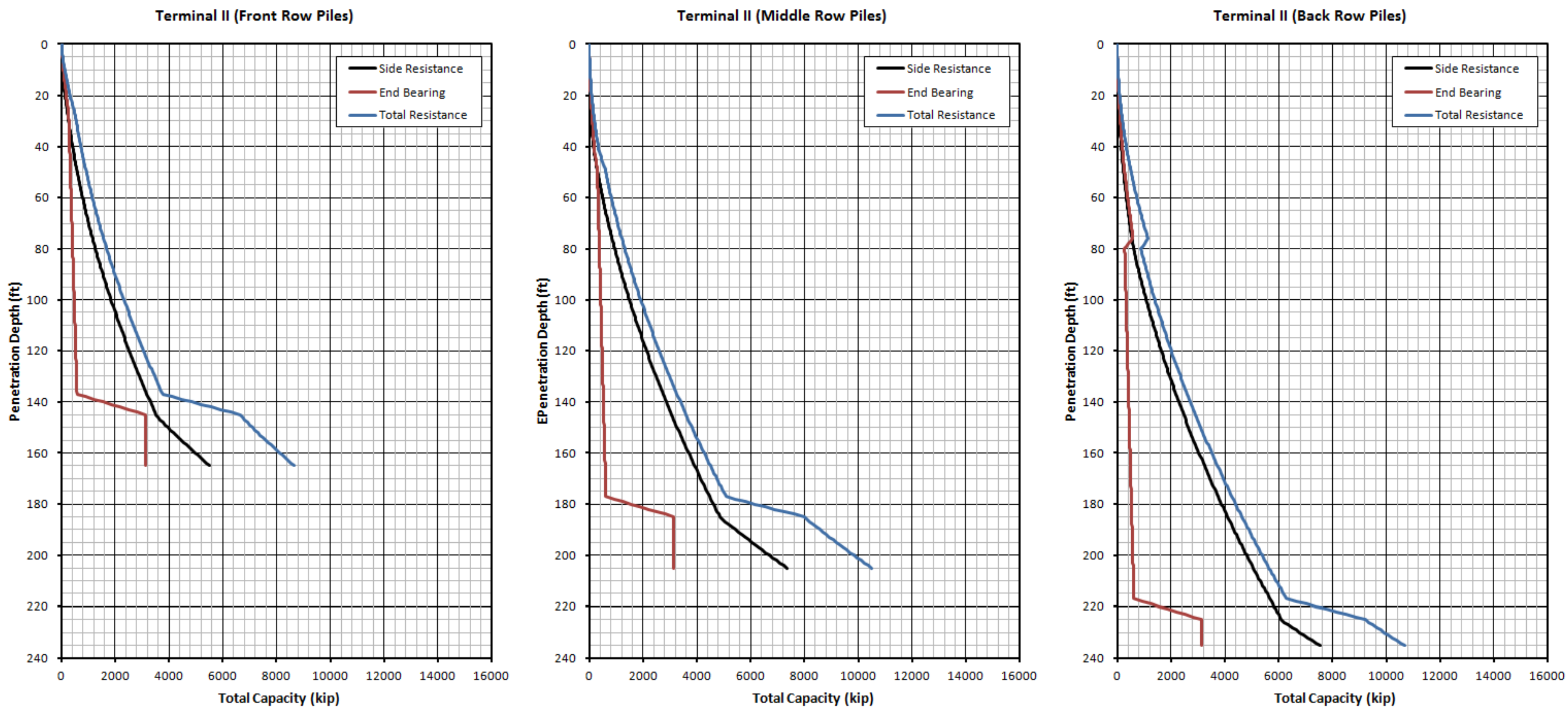


**Figure 15.** Nominal capacity charts of 48-inch pipe piles at Terminal 2.

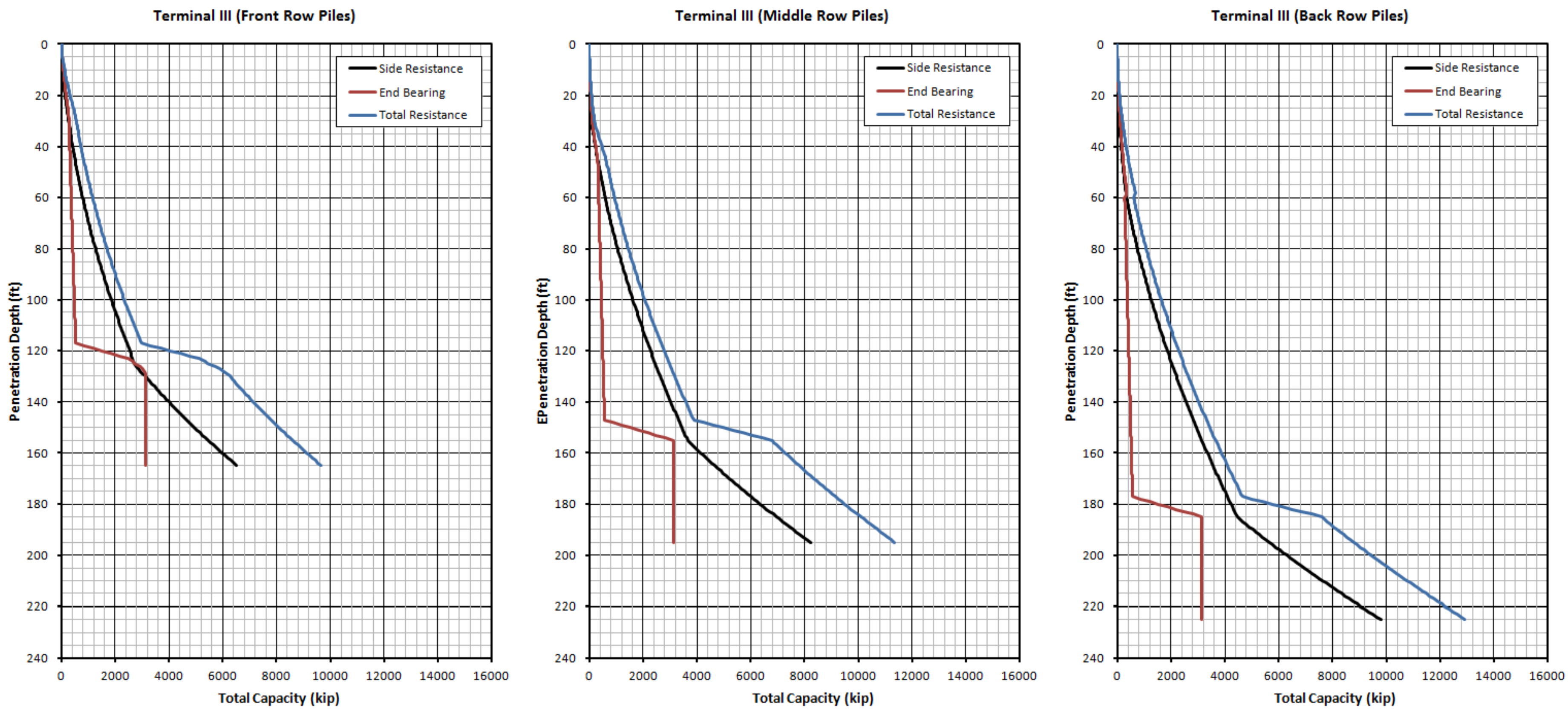


**Figure 16.** Nominal capacity charts of 48-inch pipe piles at Terminal 3.

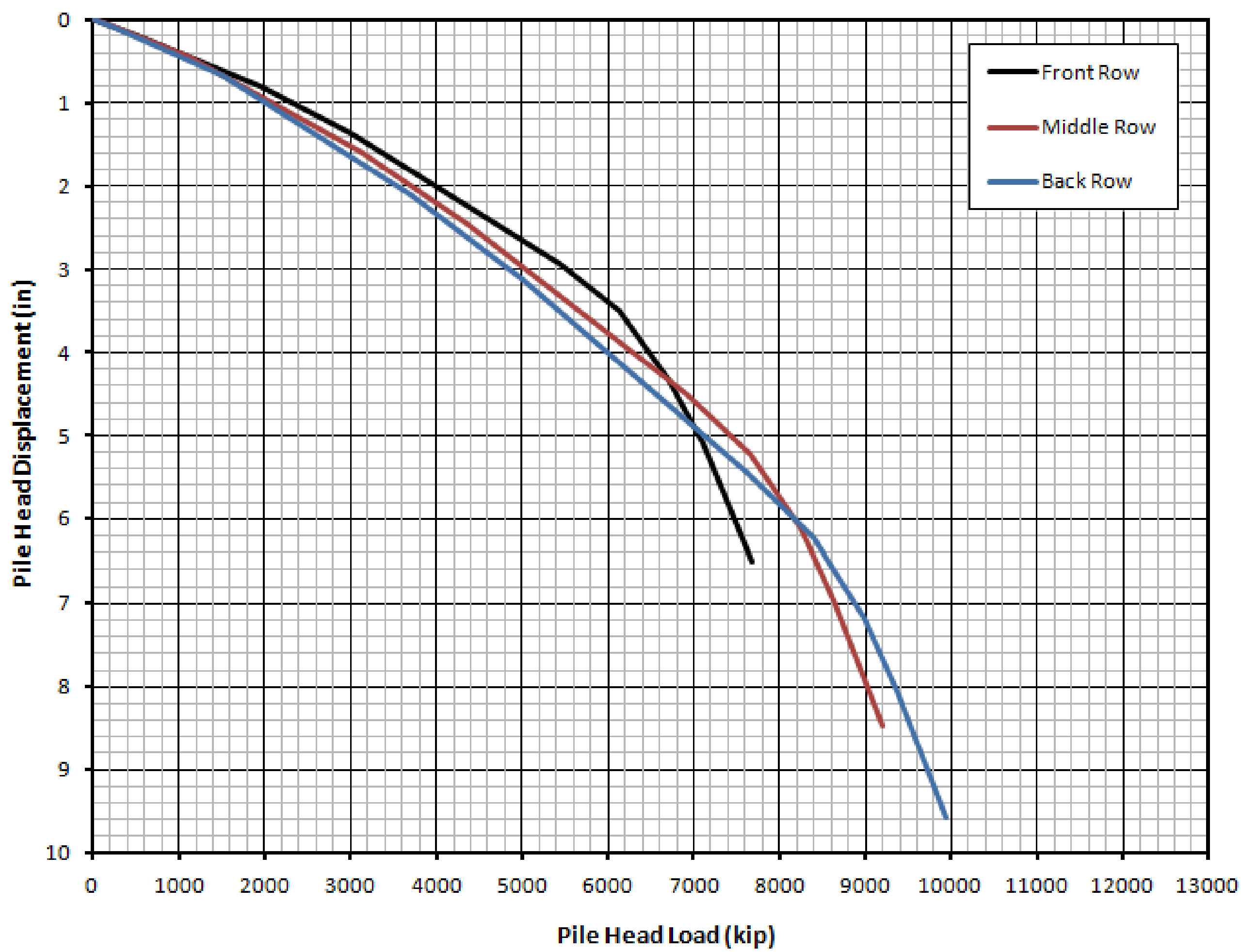


**Figure 17.** Predicted load-displacement curves of 48-inch pipe piles at the Wet Barge Berth.

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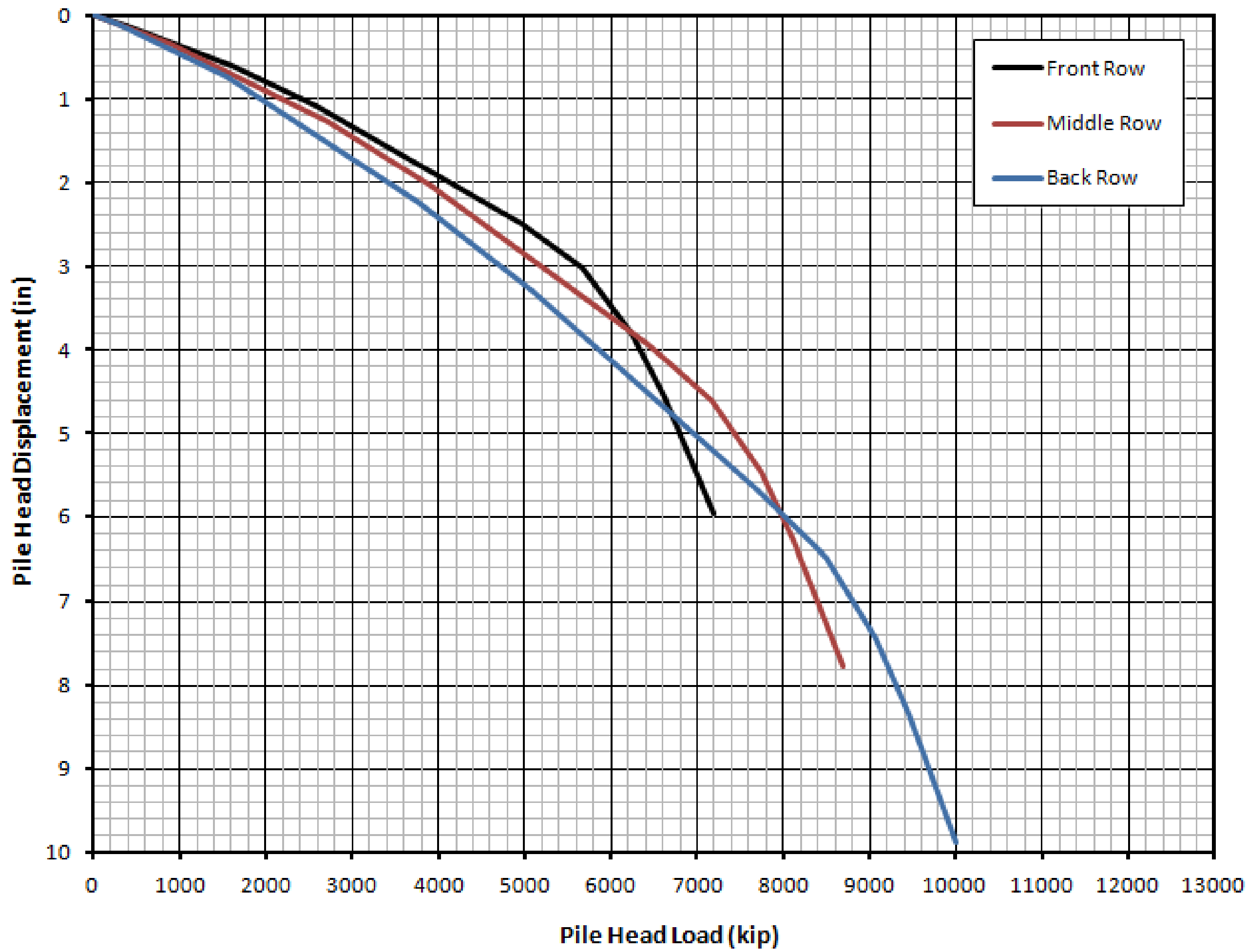


**Figure 18.** Predicted load-displacement curves of 48-inch pipe piles at the North Extension.

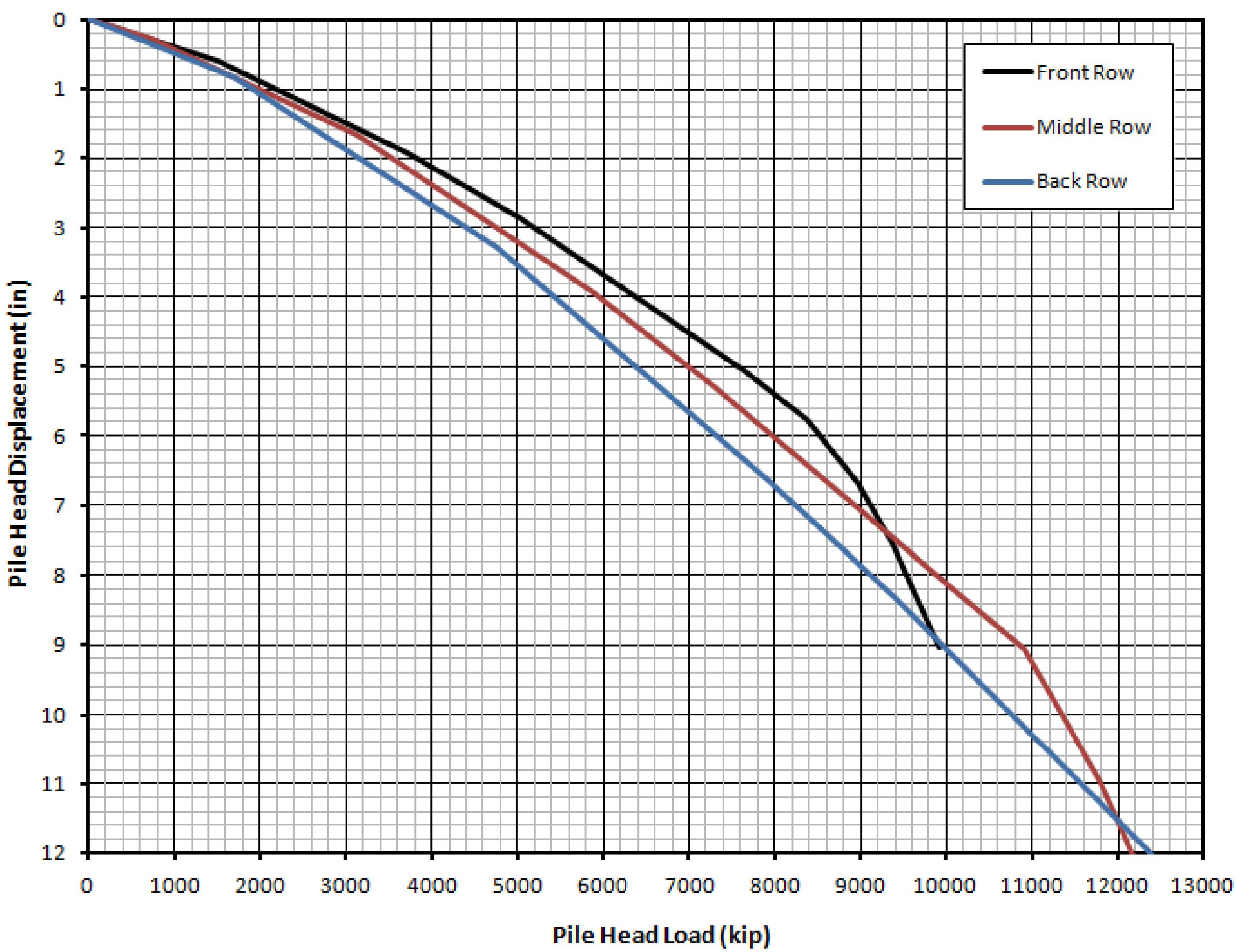


**Figure 19.** Predicted load-displacement curves of 48-inch pipe piles at Terminal 2.

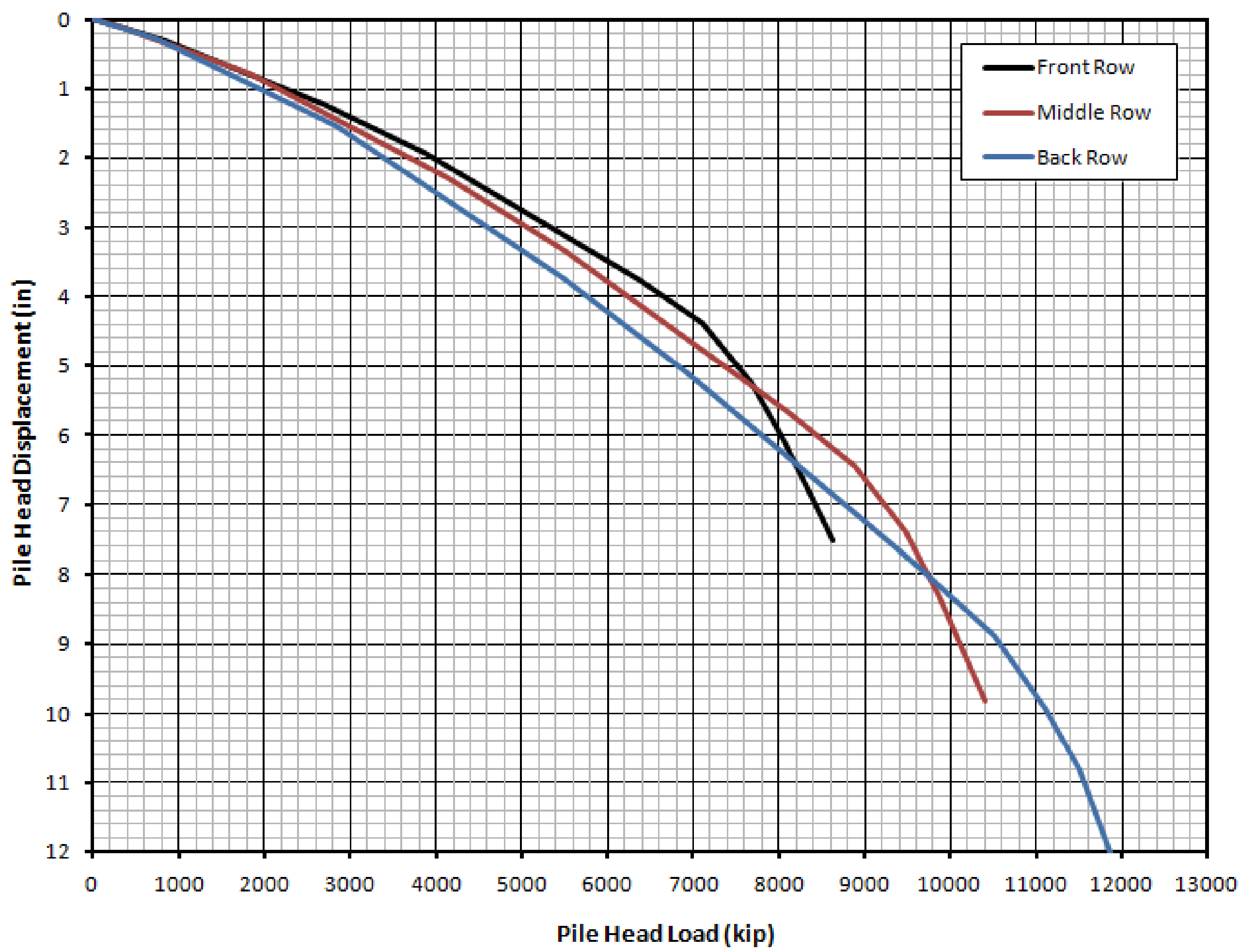


**Figure 20.** Predicted load-displacement curves of 48-inch pipe piles at Terminal 3.

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