Preliminary Design and Construction Recommendations

Floodwalls and Embankment Levees
North Shore Levee
Aberdeen and Hoquiam, Washington

for
KPFF Consulting Engineers, Inc.

April 26, 2017

GeoEngineers
Earth Science + Technology

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GeoEngineers

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North Shore Levee
Aberdeen and Hoquiam, Washington

File No. 0201-008-01

April 26, 2017

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INTRODUCTION

The purpose of this report is to provide foundation design recommendations for new floodwalls that will be constructed as part of the North Shore Levee Project in Aberdeen and Hoquiam, Washington. The floodwalls will be designed to about a 60 percent level as part of a project to obtain a Conditional Letter of Map Revision (CLOMR) from the Federal Emergency Management Administration (FEMA). Once the complete flood protection works have been constructed the City will apply for a Letter of Map Revision (LOMR). Our services for this project are being completed in accordance with our May 6, 2016 agreement with KPFF Consulting Engineers and include subsurface explorations and a geotechnical analysis of the proposed levee system, which will be documented in more detail in our Levee Certification report.

The project area and the location of the proposed levee and floodwall alignment are shown in the Vicinity Map and Site Plan, Figures 1 and 2, respectively. Most of the levee will consist of earth embankments. Floodwalls will be used in areas where space constraints do not allow for earth embankments or where closures are required. Concrete walls (T-Walls) and sheet pile walls (I-Walls) are being considered.

The floodwalls are proposed mostly along the Hoquiam River in East Hoquiam and the Wishkah River in North Aberdeen. Shorter sections of floodwalls are planned to be used parallel to the train tracks from about one block south of Bay Avenue to the Highway 101 bridge. Additional sections of the floodwalls are also proposed where earth embankment levees connect to roadway crossing closure gates. We understand that the top of the floodwalls will be established at Elevation 15.2 feet. This results in a wall height of about 3 to 4 feet at the tallest that tapers down to the ground surface at the high ground tie-ins to the north. The floodwalls will mostly be founded on level ground. Floodwalls founded near or adjacent to slopes will be evaluated for global stability and the results presented in our Levee Certification report.

The design Base Flood Elevations (BFEs) have been established by the team hydraulics engineer, Watershed Science & Engineering (WSE). The stillwater BFE (100-year return period) within the project area has been determined to be Elevation 13.2 feet. Wave run up under the design storm has been predicted to be Elevation 14.2 feet. The top of the levees and floodwalls will be established at Elevation 15.2 feet. All elevations referenced in this report use the National American Vertical Datum of 1988 (NAVD88). The BFEs are based on coastal flooding and, as a result, are driven in a large part by the tide cycles. Accordingly, high water events that rise above the surrounding ground surface, at about Elevation 10 feet, are anticipated to recede with the tide cycle, within 4 to 6 hours.

The 100-year base flood stillwater Elevation is 13.2 feet. Based on hydraulic analysis of recorded tides, a 500-year event and 1,000-year event have a stillwater Elevation of 14.1 feet and 14.5 feet, respectively. From this data, WSE has estimated that a 750-year event would have a stillwater Elevation of about 14.4 feet. The design recommendations presented in this report are provided as part of Task 6.02 Design Document Support.

SUBSURFACE CONDITIONS

Our understanding of the subsurface conditions is based on our subsurface exploration and review of previous subsurface explorations performed by us and others. For this project, we explored the subsurface conditions along the North Shore Levee alignment by advancing 8 geotechnical borings and 10 Cone

Penetration Tests (CPTs). This exploration program and other reviewed exploration programs will be described in more detail in our Levee Certification report. The approximate locations OF EACH exploration is shown on Figure 2. From our explorations, we identified two primary soil units, fill and alluvium. One exploration (B-7-16) also encountered soft siltstone or sandstone (Montesano Formation; Tmss) at about Elevation -40 feet. In our opinion, this geologic unit is too deep to affect the anticipated levee walls and is not considered in this analysis.

The fill was observed within the upper 4 to 7 feet and consisted of a wide range of materials. Our explorations were in roads or other public right-of-ways; we expect more fill in these areas than in vacant lots or undeveloped areas. In all cases, the fill was observed to be stronger or denser than the underlying alluvium. Based on this observation, we have conservatively assumed for the purposes of developing soil parameters for structural design that no fill is present over the alluvium. However, the fill is expected to be more permeable than the underlying alluvium and will be considered when evaluating potential seepage paths.

The alluvium was observed throughout the alignment and consisted primarily of very soft to soft fine-grained soils (silts and clays) with occasional organic materials and larger layers of loose silty fine sand.

Based on the observed strength profiles of the alluvium, we divided the project alignment into five analysis groups. We named the analysis groups or design reaches after the nearest body of water. The extents of these groups have been delineated by street name along the proposed levee and floodwall alignment, as depicted in Table 1. The recommendations in this report shall to all analyses groups unless stated otherwise:

TABLE 1. DESCRIPTION OF ANALYSIS GROUP LOCATIONS

Analysis Group	Description of Analysis Location (West to East along Proposed Alignment)
Hoquiam	Broadway Street to 25th Street
Grays Harbor	25th Street to Myrtle Street
Chehalis West	Myrtle Street to Washington Street
Chehalis East	Washington Street to South Broadway Street
Wishkah	South Broadway Street to Arthur Street

LEVEE DESIGN AND CONSTRUCTION RECOMMENDATIONS

General

The design and construction recommendations provided in this report are based on our understanding of concept level design and are intended to help advance the design to the 60 percent level. Additional recommendations will be provided, where appropriate, as the design progresses and during construction. We must be retained to review project plans and to monitor the geotechnical aspects of levee construction in order to confirm that soil conditions in the field are as we assumed in our analysis and we must be given an opportunity to revise our recommendations as needed.

Levee Embankment Fill

Gradation

Fill material used to construct embankment levees and to backfill inspection and seepage cut-off trenches or other overexcavations must consist of a homogeneous low permeability material that can be compacted to a firm and unyielding condition. The material must be adequately blended and compacted during placement so that no preferential seepage paths are created.

We recommend material for the levee embankment be a silty sand or clayey sand conforming to the following material specification:

TABLE 2. MATERIAL SPECIFICATION FOR LEVEE EMBANKMENT

Sieve Size	Percent Passing
4-inch	100
3/4-inch	80-100
#4	60-100
#200	30-60[1]

Note:

[1] The percent passing the #200 sieve divided by the percent passing the ¾-inch sieve shall be greater or equal to 0.3.

The above levee embankment fill shall consist of granular material either naturally occurring or processed, and shall meet the above requirements for grading and quality. The material shall not contain more than 2 percent organic material by weight. Recycled material such as asphalt, concrete rubble, recycled glass, or slag shall not be used.

Placement and Compaction

The levee embankment fill shall be placed with a moisture content within 2 percentage points below or 4 percentage points above the optimum moisture content, which should be adjusted as necessary in order to achieve the specified compaction criteria. In general, the levee embankment fill shall be compacted to a minimum of 92 percent of the maximum dry density (MDD). In areas where the levee embankment also provides support to other structural elements, such as where roadways cross over the levee, the fill shall be compacted to a minimum of 95 percent of MDD. MDD and optimum moisture content shall be determined by Modified Proctor (ASTM International [ASTM] D 1557).

Utility Penetrations and Seepage Control

The embankment levee and floodwalls are anticipated to cross over multiple underground utilities. This can create a seepage path from the flood side to the protected side of the levee. Seepage can occur through the utility pipes themselves and also around the utilities within the utility backfill. Seepage of floodwater through the utility pipes must be evaluated by the project civil engineer and could be controlled with passive closure systems like check valves or active closure systems like gate valves.

We recommend seepage around the utilities and through the utility backfill be controlled using filter drains. Filter drains consist of specially graded backfill installed around and over utilities where they cross the levee or floodwall footprint. To construct a filter drain, levee embankment fill is placed around the utility on the flood side of the utility crossing and a drainage layer is placed around the utility on the landside of the utility crossing. Under embankment levees, the drainage layer should be placed for a distance equal to one-third of the width of the levee. An illustration of this layout is provided in Figure 8-1 of United States Army Corps of Engineers (USACE) EM 1110-2-1913 “Design and Construction of Levees”.

The drainage layer should consist of granular material meeting the filter design criteria outlined in Appendix D of USACE EM 1110-2-1901 “Seepage Analysis and Control for Dams”. The filter criterion is based on the gradation of the surrounding soil, and must be confirmed in the field. In our experience, material conforming to the gradation requirements of Washington State Department of Transportation (WSDOT) Standard Specification 9-03.1(2) Fine Aggregate for Portland Cement Concrete (Class 1) meets the filter criterion in most cases. We recommend project plans specify this material but also allow for a field change should the conditions warrant modifying the specification.

Where the levee crosses existing utilities, the existing trench backfill should be removed to within 6 inches of the top of the utility and replaced with the filter drain. The extent and details of the filter drains must be determined in the field so that the actual as-built conditions can be accounted for. Deeper utilities may not require a filter drain detail depending on the depth of the utility and the depth of the predicted floodwaters. This must be evaluated on a case-by-case basis.

Shallow Foundation Recommendations

General

We recommend that flood walls be designed in accordance with the guidelines provided in USACE EM 1110-2-2502 “Retaining and Flood Walls” and USACE EM 1110-2-6066 “Design of I-Walls”. Bearing capacity and lateral resistance of shallow foundations is dependent on the shape, width, and depth of the footing. The stability of floodwalls is also dependent on the seepage forces under the wall. We recommend that GeoEngineers review the wall plans to confirm that our recommendations are interpreted as we intended and that conditions that affect potential seepage forces are as anticipated. For analysis of floodwall bearing capacity we have conservatively assumed that footings bear on the weaker alluvium underlying the fill.

Bearing Surface Preparation

The soil in the vicinity of the proposed floodwall is expected to consist of either fill or alluvium. Based on our explorations and our experience in the area, we expect these soils to have a high fines content and to be easily disturbed, especially during periods of wet weather. To limit disturbance of subgrade soils we recommend that footing excavations be excavated using a smooth-edge bucket (no teeth). If subgrades become disturbed, we recommend that they be compacted to a firm and unyielding condition using hand-operated compaction equipment or overexcavated and replaced with compacted levee embankment fill.

The footing bearing surface should be observed and evaluated by a member of our firm to confirm that no soft, compressible, organic, highly permeable soil, or material otherwise deleterious to the function of the floodwall is present. Overexcavation may be required to remove deleterious material. Overexcavated soil must be replaced with levee embankment fill compacted to at least 95 percent of MDD.

Foundation bearing surfaces are to be thoroughly compacted to a dense, non-yielding condition. Loose or disturbed materials present at the base of footing excavations must be removed or compacted. Foundation bearing surfaces are not to be exposed to standing water. Should water infiltrate and pool in the excavation, it must be removed and the bearing surface re-evaluated before placing structural fill or reinforcing steel.

Bearing Capacity and Settlement

We recommend that footings founded as recommended be evaluated using the soil bearing pressures presented in Table 3, below. These are ultimate soil bearing pressures and an appropriate factor of safety must be applied. Guidance in USACE EM 1110-2-2502 “Retaining and Flood Walls” Table 4-2 “Inland Flood Wall Stability Criteria” states that a minimum factor of safety of 3.0 is required for the “Design flood” loading condition and a minimum factor of safety of 2.0 is required for the “Water to top of wall” loading condition. The values presented are “net” bearing pressures. The weight of soil over the top of the footing can be neglected.

TABLE 3. FLOODWALL DESIGN ULTIMATE BEARING PRESSURES

Footing Embedment Depth (ft)	Allowable Bearing Capacity (psf)				
	Hoquiam	Grays Harbor	Chehalis West	Chehalis East	Wishkah
2	770	810	1,190	1,160	730
3	820	860	1,240	1,200	780
4	870	910	1,280	1,240	820

The dead weight of the floodwall is expected to be relatively low, less than 1,000 pounds per linear foot (plf). Accordingly, we expect long-term settlement due to the weight of the wall to be minor. If floodwalls are to be constructed adjacent to or connecting to large embankment fills, the adjacent fill could induce settlement of the wall. We recommend that construction be staged such that the walls adjacent to large fills are not constructed until after most of the expected settlement from the embankment fill has occurred.

The wall foundation could experience elastic settlement when loaded by floodwaters, which would result in tipping of the wall. We estimate that settlements of footings under the design flood load will be less than 1 inch provided that the loading is applied for less than one day. Differential settlements between comparably loaded sections of the wall are expected to be less than ½ inch.

Lateral Load Resistance for Concrete Walls

The ability of the soil to resist lateral loads is a function of frictional resistance, which can develop at the base of footings, and passive resistance, which can develop on the face of below-grade elements, such as the face of the footing, as these elements move into the soil. Concrete floodwalls or floodwalls that will be located on generally flat ground and founded in accordance with our recommendations are to be designed using the values in Table 4 below. When evaluating the wall for a flooded loading condition (the “Design flood” loading condition or the “Water to top of wall” loading condition) the values provided for “Fill Below the Water Table” are to be used.

TABLE 4. DESIGN LATERAL PRESSURES AND RESISTANCES AGAINST WALLS

Soil Condition	Active Equivalent Fluid Pressure (pcf)	Ultimate Passive Equivalent Fluid Pressure (pcf)	Coefficient of Friction for Concrete Cast Directly on Soil
Levee Embankment Fill above Water Table	40	390[1]	0.6 up to 250 psf[3]
Levee Embankment Fill below Water Table	18[2]	190[1]	0.6 up to 250 psf[3]

Notes:

[1] An appropriate factor of safety must be applied based on the loading condition being analyzed in accordance with USACE EM 1110-2-2502.

[2] This value must be combined with hydrostatic pressure.

[3] Friction resistance is based on the vertical dead load and must include the effects of buoyancy and uplift pressures.

Lateral Load Resistance for Sheet Pile Walls

The proposed cantilevered sheet pile floodwalls will be in an at-rest condition most of the time because the ground level (mudline) will be approximately equal on both sides of the wall. However, during flood events the cantilevered floodwall will need to resist the floodwaters that will rise on only one side of the floodwall. For this cantilevered condition that occurs during flood events, we recommend that the wall be designed using the USACE design program CWALSHT or similar analysis program using the soil parameters in Tables 5 through 9. This program does not allow for soil layers with variable properties (i.e., strength increase with depth). To account for this we have divided soil units into smaller increments and used average properties over that depth.

TABLE 5. HOQUIAM ANALYSIS GROUP – SOIL PARAMETERS FOR I-WALL ANALYSIS

Geologic Layer	Bottom Elevation of Layer	Unit Weight (PCF)		Q-Strength Short Term Loading		S-Strength Long Term Loading	
		Saturated	Moist	Angle of Internal Friction (Deg)	Cohesion (PSF)	Angle of Internal Friction (Deg)	Cohesion (PSF)
Clay Alluvium (260 psf)	El. -2 feet	102	102	0	260	18.0	0
Clay Alluvium (260 psf at top, increasing by 8 psf per foot of depth)	El. -7 feet	109	109	0	284	18.2	0
	El. -12 feet	109	109	0	324	18.6	0
	El. -17 feet	109	109	0	364	19.0	0
	El. -22 feet	109	109	0	404	19.4	0

TABLE 6. GRAYS HARBOR ANALYSIS GROUP – SOIL PARAMETERS FOR I-WALL ANALYSIS

Geologic Layer	Bottom Elevation of Layer	Unit Weight (PCF)		Q-Strength Short Term Loading		S-Strength Long Term Loading	
		Saturated	Moist	Angle of Internal Friction (Deg)	Cohesion (PSF)	Angle of Internal Friction (Deg)	Cohesion (PSF)
Clay Alluvium (280 psf)	El. 2 feet	102	102	0	280	14.0	0
Clay Alluvium (280 psf at top, increasing by 28 psf per foot of depth)	El. -4 feet	109	109	0	378	15.8	0
	El. -10 feet	109	109	0	546	18.8	0
	El. -16 feet	109	109	0	714	21.8	0
Sand Alluvium (29 Deg)	El. -22 feet	109	109	29	0	29	0

TABLE 7. CHEHALIS WEST ANALYSIS GROUP – SOIL PARAMETERS FOR I-WALL ANALYSIS

Geologic Layer	Bottom Elevation of Layer	Unit Weight (PCF)		Q-Strength Short Term Loading		S-Strength Long Term Loading	
		Saturated	Moist	Angle of Internal Friction (Deg)	Cohesion (PSF)	Angle of Internal Friction (Deg)	Cohesion (PSF)
Clay Alluvium (430 psf)	El. -8 feet	90	90	0	430	22	0
Clay Alluvium (350 psf)	El. -18 feet	90	90	0	350	22	0
Clay Alluvium (700 psf)	El. -36 feet	107	107	0	700	25	0

TABLE 8. CHEHALIS EAST ANALYSIS GROUP – SOIL PARAMETERS FOR I-WALL ANALYSIS

Geologic Layer	Bottom Elevation of Layer	Unit Weight (PCF)		Q-Strength Short Term Loading		S-Strength Long Term Loading	
		Saturated	Moist	Angle of Internal Friction (Deg)	Cohesion (PSF)	Angle of Internal Friction (Deg)	Cohesion (PSF)
Clay Alluvium (420 psf)	El. -2 feet	83	83	0	420	16	0
Clay Alluvium (500 psf)	El. -22 feet	99	99	0	500	23	0

TABLE 9. WISHKAH ANALYSIS GROUP – SOIL PARAMETERS FOR I-WALL ANALYSIS

Geologic Layer	Bottom Elevation of Layer	Unit Weight (PCF)		Q-Strength Short Term Loading		S-Strength Long Term Loading	
		Saturated	Moist	Angle of Internal Friction (Deg)	Cohesion (PSF)	Angle of Internal Friction (Deg)	Cohesion (PSF)
Clay Alluvium (250 psf)	El. -2 feet	93	93	0	250	14.0	0
Clay Alluvium (250 psf at top, increasing by 7 psf per foot of depth)	El. -7 feet	94	94	0	271	14.0	0
	El. -12 feet	94	94	0	306	14.2	0
	El. -17 feet	94	94	0	341	14.3	0
	El. -22 feet	94	94	0	376	14.3	0

Input parameters not included in this table, such as Angle of Wall Friction and Adhesion, should be set to zero. We recommend that all soil layers are assumed to be flat, not sloped. Provided that the modeled ground surface is relatively flat a “Fixed Surface” analysis rather than a “Sweep Search” should be used when determining the failure wedge. We recommend that groundwater be modeled at the ground surface for all design cases. (Except on the flood side of the wall during flood cases when the water level is above the ground surface.) Accordingly, recommended “saturated” and “moist” soil unit weights are the same.

We have also performed independent calculations of lateral earth pressures to check earth pressures calculated using design software. These earth pressures were calculated using simplified Coulomb lateral earth pressure theory and assume a ground surface elevation of 12 feet. These pressures will not exactly match earth pressures calculated using a more detailed strain wedge analysis like the CWALSHT program performs. The earth pressures should, however, be similar. Walls that are not founded on flat level ground must also be checked for global slope stability. This will be provided in our Levee Certification report. A graphical presentation of these net (passive minus active) design earth pressures are included in Figures 3 through 7. We recommend factors of safety be based in part on USACE minimums for the loading conditions described in Table 6-2. “I-Wall Loading Conditions, Classification, and Criteria” of USACE 1110-2-6066 “Design of I-Walls” with some exceptions as described herein. We recommend that the factors of safety for an “Ordinary” understanding of subsurface conditions be used.

The USACE guidance recommends that I-Walls be designed for overtopping (water to the top of the floodwall) with a factor of safety of 1.5 for an “unusual” loading condition and 1.3 for an “extreme” loading condition considering both Q (Short Term) soil strengths and S (Long Term) soil strengths. A “usual” loading condition is defined as an event with a return period less than 10 years. An “unusual” loading condition is defined as an event with a return period greater than 10 years but less than 750 years. An “extreme” loading condition is defined as an event with a return period greater than 750 years.

In our opinion it is appropriate to consider Q (Short Term) soil strengths for short duration loading conditions, about one day or less. The overtopping case is both an extreme loading condition and is of short duration. Accordingly, it is our opinion that only Q (Short Term) soil strengths should be used in evaluating the overtopping case. If the overtopping case is treated as an extreme event with a factor of safety of 1.3, we recommend that the wall also be checked with the estimated 750-year stillwater elevation (14.4 feet) using a factor of safety of 1.5. If the flood wall is expected to be loaded above the design flood elevation for longer than one day we should be contacted for revised recommendations.

We do not recommend any changes be made to the USACE guidance for the design water level. This case should be evaluated using both Q (Short Term) soil strengths and S (Long Term) soil strengths.

LIMITATIONS

We have prepared this report for the exclusive use of KPFF Consulting Engineers, Inc. KPFF may distribute copies of this report to the City of Aberdeen, the City's authorized agents, and regulatory agencies including FEMA and FEMA's designated reviewers, as may be required for the project.

Levee certification within the context of this or other reports follows the definition provided in 44 CFR 65.2, which states that "certification by a registered professional engineer or other party does not constitute a warranty or guarantee of performance, expressed or implied. Certification of data is a statement that the data is accurate to the best of the certifier's knowledge. Certification of analyses is a statement that the analyses have been performed correctly and in accordance with sound engineering practices."

Qualified engineering and construction practices can help mitigate flooding risks, but they cannot completely eliminate those risks. Favorable performance of structures in the recent past provides useful information for anticipating likely near-term future performance, but it cannot predict or imply a certainty of similar long-term performance. Levee systems require periodic inspection to confirm that all critical components continue functioning as intended. Confirmation that design flood flows and/or elevations have not significantly changed also requires the periodic review of design criteria and other potential contributing factors including, but not limited to, changes in surrounding development, weather patterns, system operations policies, or sedimentation.

Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practices in the field of geotechnical engineering in this area at the time this report was prepared. The conclusions, recommendations, and opinions presented in this report are based on our professional knowledge, judgement, and experience. No warranty or other conditions, express or implied, should be understood.

Please refer to Appendix A titled "Report Limitations and Guidelines for Use" for additional information pertaining to use of this report.

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Feet

Notes:

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source: Mapbox Open Street Map, 2015
Projection: NAD 1983 2011 StatePlane Washington South FIPS 4602 Ft US

Vicinity Map

North Shore Levee
Aberdeen and Hoquiam, Washington

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Figure 1

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Notes:

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source: Aerial from ESRI, 2015

Projection: NAD 1983 StatePlane Washington South FIPS 4602 Feet

Legend

Levee System Alignment

Approximate Location of CPT

- GeoEngineers, 2016
- GeoEngineers, 2015

Approximate Location of Boring

- GeoEngineers, 2016
- GeoEngineers, 2015
- WSDOT, 2010
- Shannon and Wilson, 1999
- WSDOT, 1983
- WSDOT, 1976
- Lowe, 1977
- WSDOT, 1968
- WSDOT, 1952

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Feet

Site Plan

North Shore Levee
Aberdeen and Hoquiam, Washington

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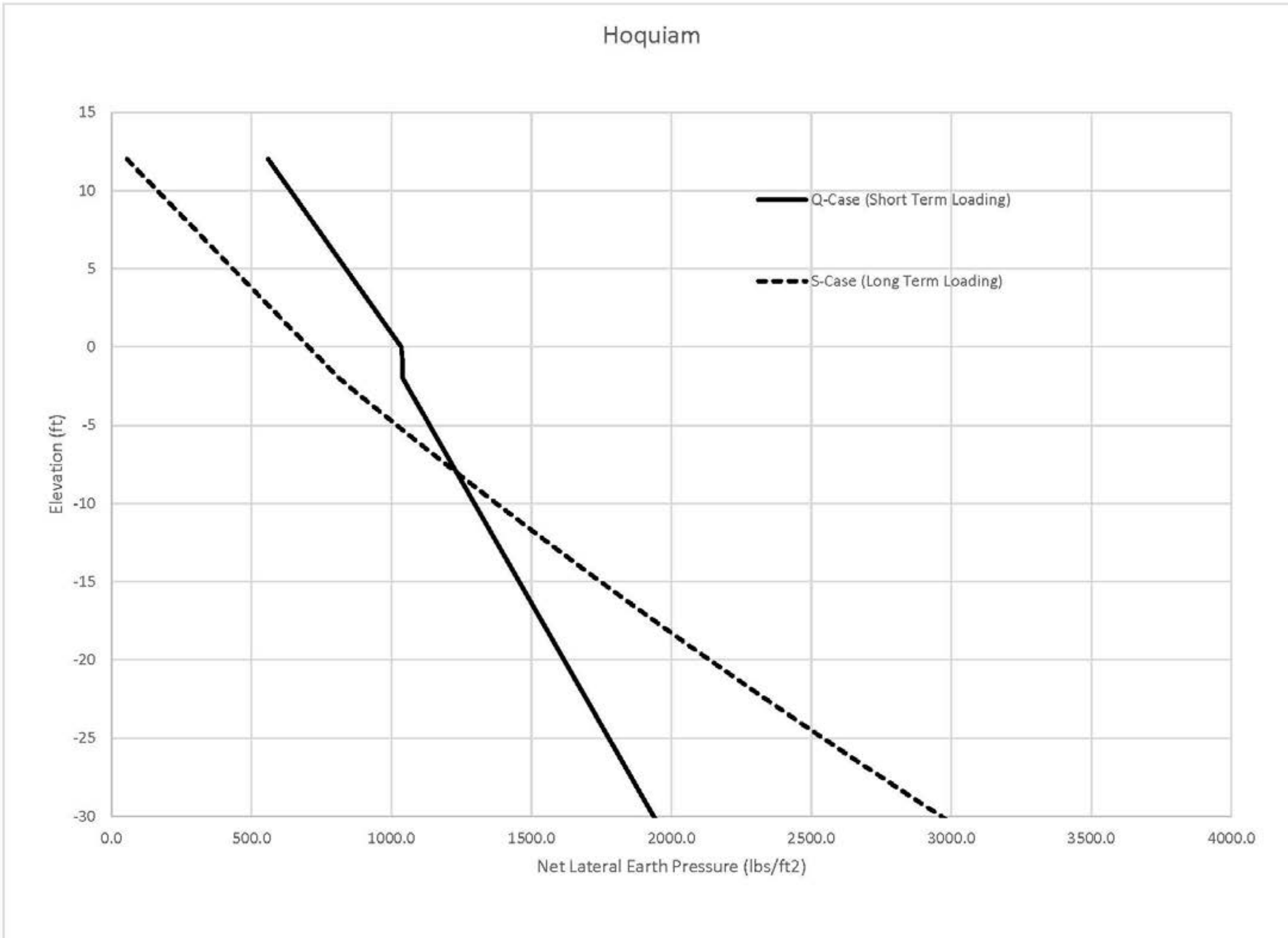
Figure 2



Notes: 1. Ground surface assumed to be at Elevation 12 feet
2. See report text for additional details.

Hoquiam Analysis Group - Lateral Earth Pressure

North Shore Levee
Aberdeen and Hoquiam, Washington

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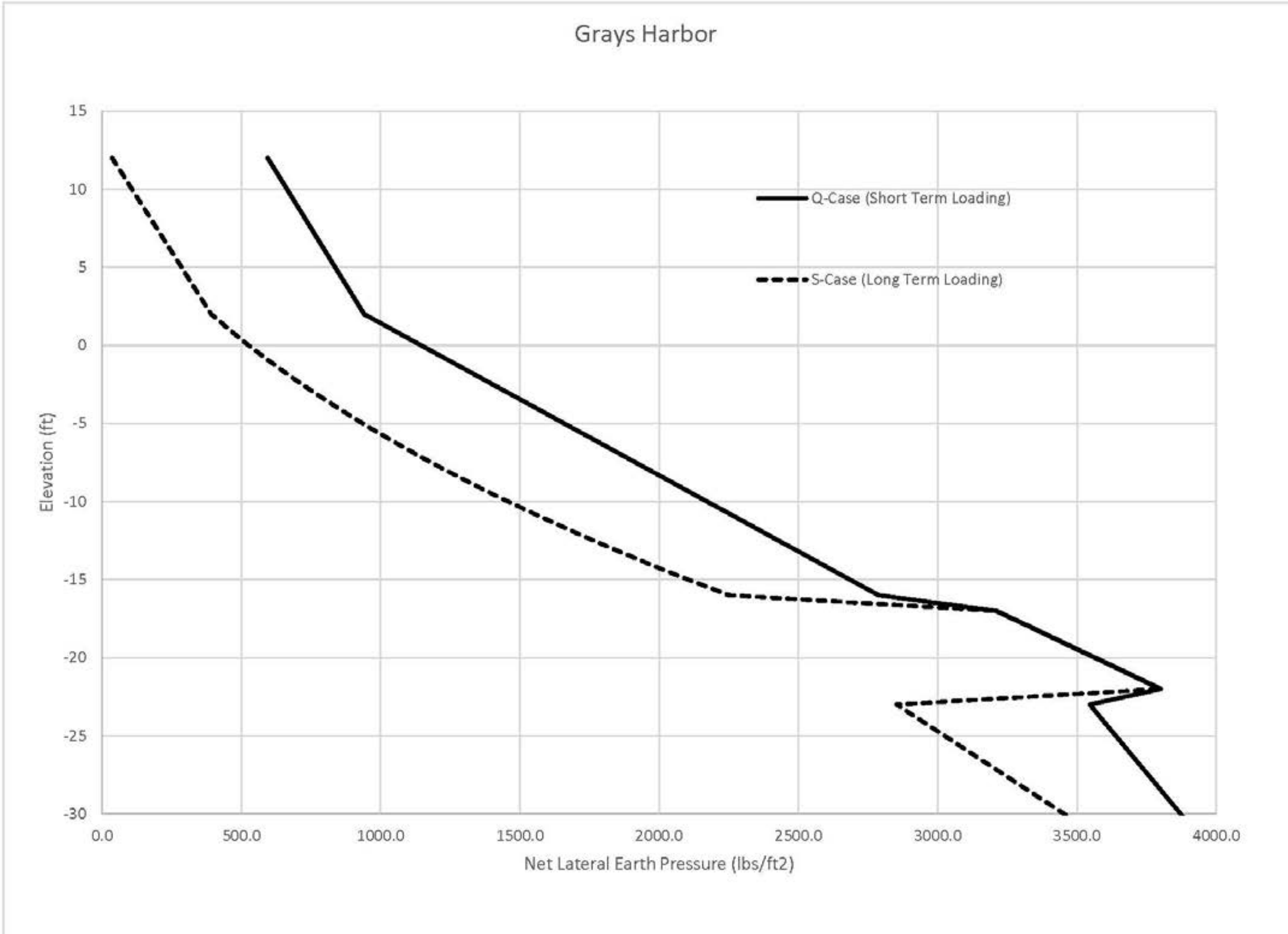
Figure 3



Notes: 1. Ground surface assumed to be at Elevation 12 feet
2. See report text for additional details.

Grays Harbor Analysis Group - Lateral Earth Pressure

North Shore Levee
Aberdeen and Hoquiam, Washington

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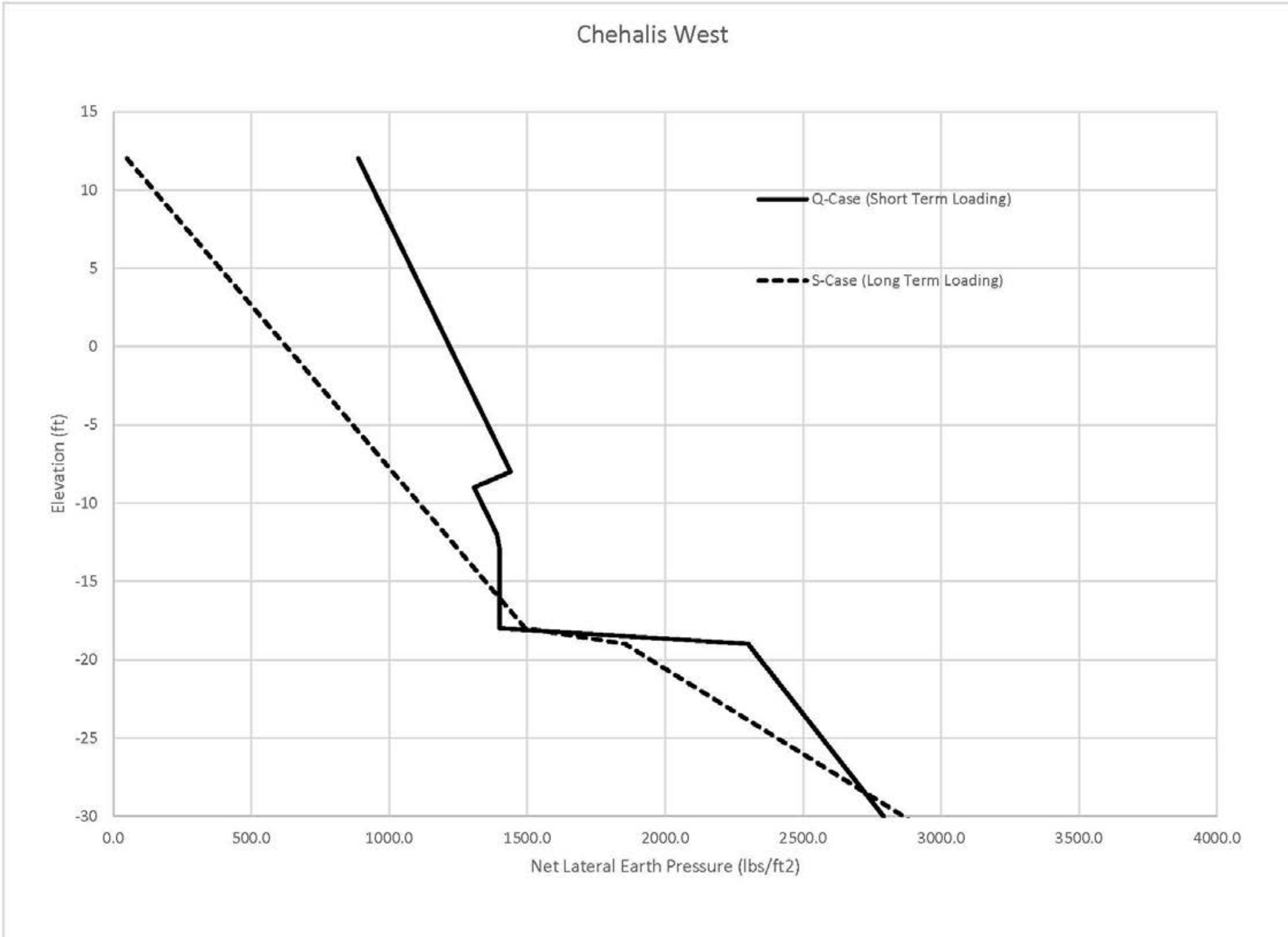
Figure 4



Notes: 1. Ground surface assumed to be at Elevation 12 feet
2. See report text for additional details.

Chehalis West Analysis Group - Lateral Earth Pressure

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Aberdeen and Hoquiam, Washington

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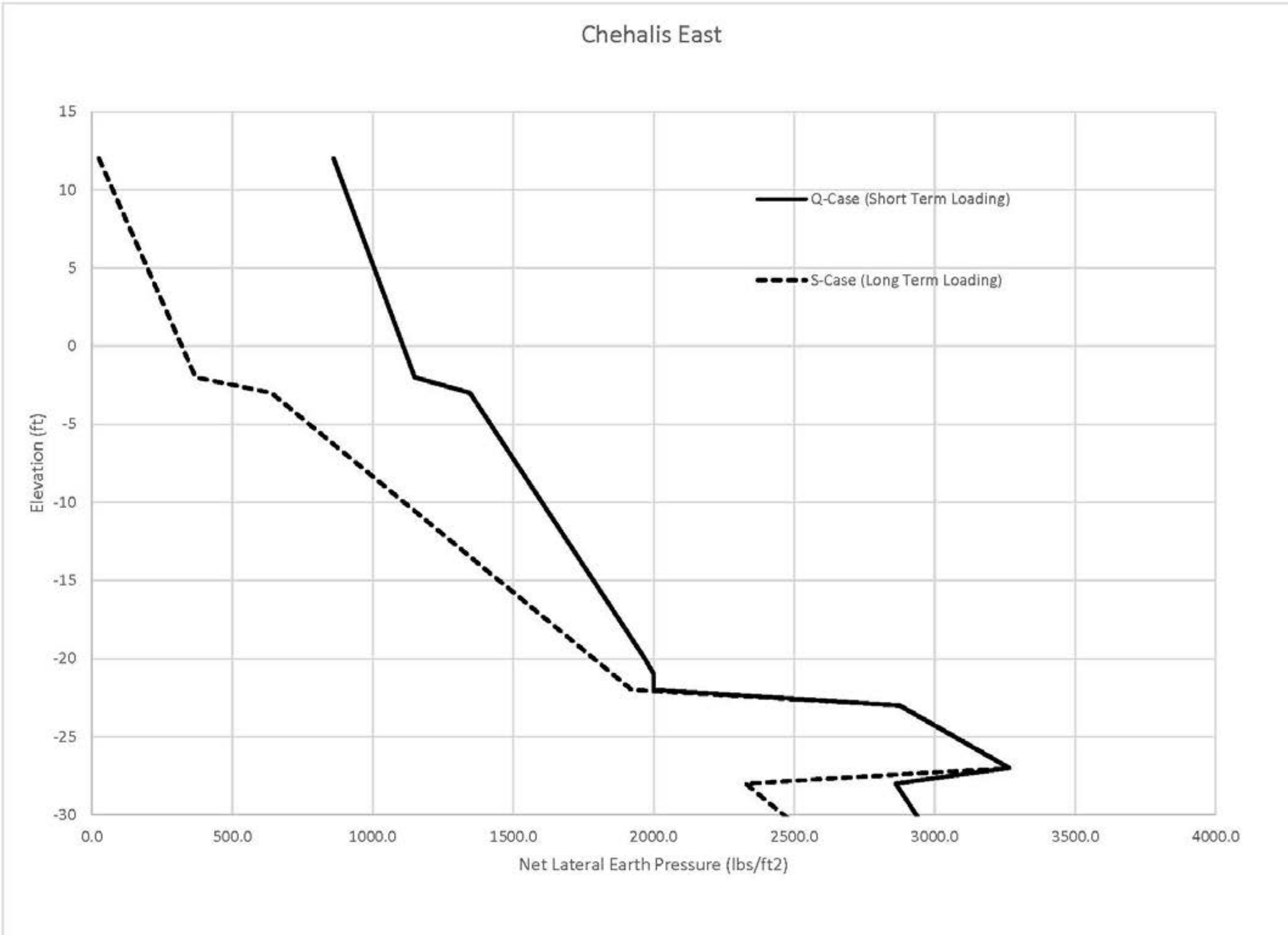
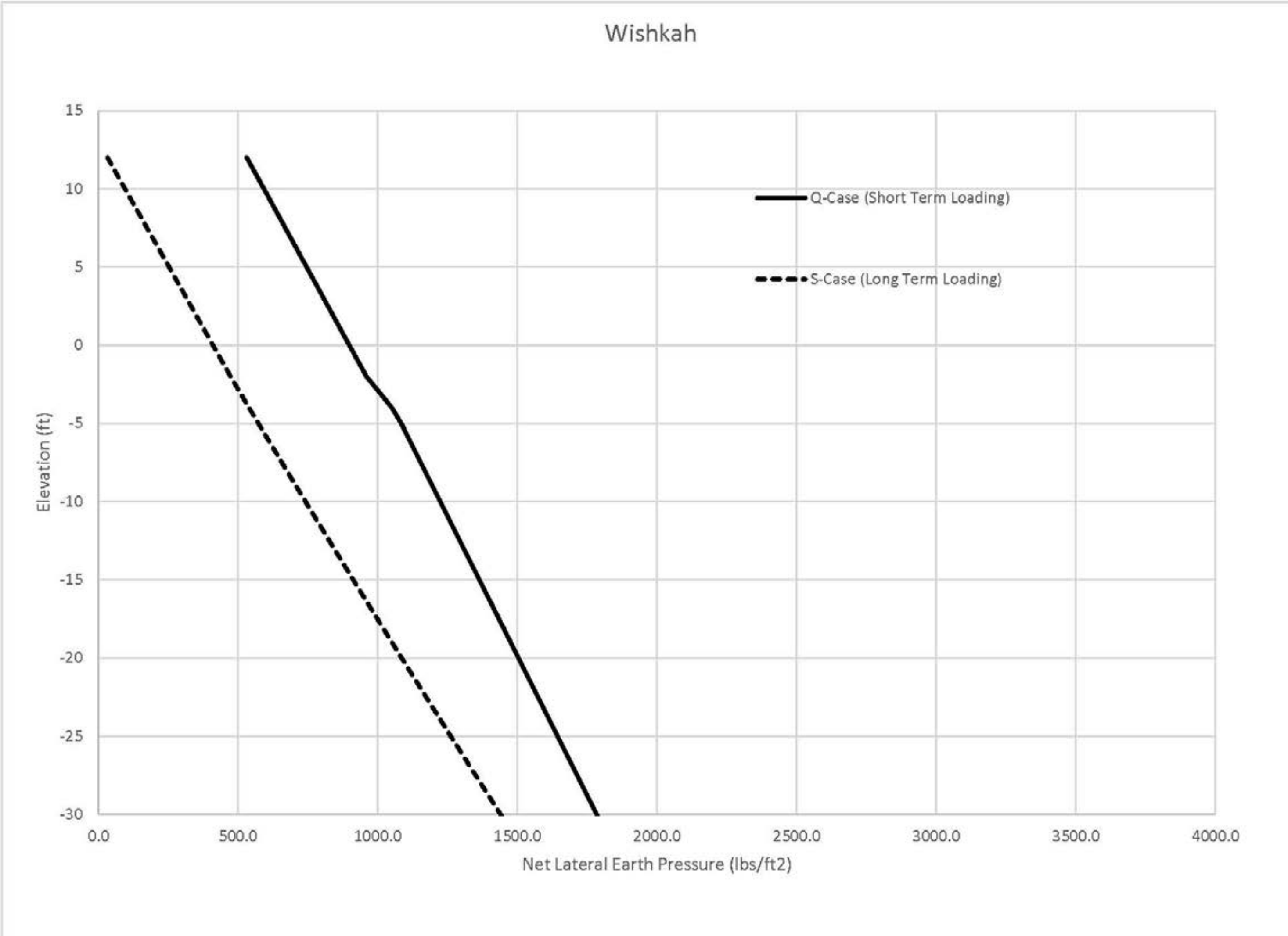
Figure 5



Notes: 1. Ground surface assumed to be at Elevation 12 feet
2. See report text for additional details.

Chehalis East Analysis Group - Lateral Earth Pressure	
North Shore Levee Aberdeen and Hoquiam, Washington	
GeoEngineers	**Figure 6**



Notes: 1. Ground surface assumed to be at Elevation 12 feet
2. See report text for additional details.

Wishkah Analysis Group - Lateral Earth Pressure

North Shore Levee
Aberdeen and Hoquiam, Washington

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Figure 7

0201-008-01 Date Exported: 4/25/17

APPENDIX A
Report Limitations and Guidelines for Use

APPENDIX A
REPORT LIMITATIONS AND GUIDELINES FOR USE[1]

This appendix provides information to help you manage your risks with respect to the use of this report.

Read These Provisions Closely

It is important to recognize that the geoscience practices (geotechnical engineering, geology and environmental science) rely on professional judgment and opinion to a greater extent than other engineering and natural science disciplines, where more precise and/or readily observable data may exist. To help clients better understand how this difference pertains to our services, GeoEngineers includes the following explanatory "limitations" provisions in its reports. Please confer with GeoEngineers if you need to know more how these "Report Limitations and Guidelines for Use" apply to your project or site.

Geotechnical Services are Performed for Specific Purposes, Persons and Projects

This report has been prepared for KPFF Consulting Engineers, Inc. and for the Project(s) specifically identified in the report. The information contained herein is not applicable to other sites or projects.

GeoEngineers structures its services to meet the specific needs of its clients. No party other than the party to whom this report is addressed may rely on the product of our services unless we agree to such reliance in advance and in writing. Within the limitations of the agreed scope of services for the Project, and its schedule and budget, our services have been executed in accordance with our Agreement with KPFF Consulting Engineers, Inc. dated May 5, 2016 and generally accepted geotechnical practices in this area at the time this report was prepared. We do not authorize, and will not be responsible for, the use of this report for any purposes or projects other than those identified in the report.

A Geotechnical Engineering or Geologic Report is based on a Unique Set of Project-Specific Factors

This report has been prepared for Floodwalls and Embankment Levees North Shore Levee located in Aberdeen and Hoquiam, Washington. GeoEngineers considered a number of unique, project-specific factors when establishing the scope of services for this project and report. Unless GeoEngineers specifically indicates otherwise, it is important not to rely on this report if it was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

For example, changes that can affect the applicability of this report include those that affect:

- the function of the proposed structure;
- elevation, configuration, location, orientation or weight of the proposed structure;

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[1] Developed based on material provided by ASFE, Professional Firms Practicing in the Geosciences; www.asfe.org.

- composition of the design team; or
- project ownership.

If changes occur after the date of this report, GeoEngineers cannot be responsible for any consequences of such changes in relation to this report unless we have been given the opportunity to review our interpretations and recommendations. Based on that review, we can provide written modifications or confirmation, as appropriate.

Environmental Concerns are Not Covered

Unless environmental services were specifically included in our scope of services, this report does not provide any environmental findings, conclusions, or recommendations, including but not limited to, the likelihood of encountering underground storage tanks or regulated contaminants.

Information Provided by Others

GeoEngineers has relied upon certain data or information provided or compiled by others in the performance of our services. Although we use sources that we reasonably believe to be trustworthy, GeoEngineers cannot warrant or guarantee the accuracy or completeness of information provided or compiled by others.

Subsurface Conditions Can Change

This geotechnical or geologic report is based on conditions that existed at the time the study was performed. The findings and conclusions of this report may be affected by the passage of time, by man-made events such as construction on or adjacent to the site, new information or technology that becomes available subsequent to the report date, or by natural events such as floods, earthquakes, slope instability or groundwater fluctuations. If more than a few months have passed since issuance of our report or work product, or if any of the described events may have occurred, please contact GeoEngineers before applying this report for its intended purpose so that we may evaluate whether changed conditions affect the continued reliability or applicability of our conclusions and recommendations.

Information Provided by Others

GeoEngineers has relied upon certain data or information provided or compiled by others in the performance of our services. Although we use sources that we reasonably believe to be trustworthy, GeoEngineers cannot warrant or guarantee the accuracy or completeness of information provided or compiled by others.

Geotechnical and Geologic Findings are Professional Opinions

Our interpretations of subsurface conditions are based on field observations from widely spaced sampling locations at the site. Site exploration identifies the specific subsurface conditions only at those points where subsurface tests are conducted or samples are taken. GeoEngineers reviewed field and laboratory data and then applied its professional judgment to render an informed opinion about subsurface conditions at other locations. Actual subsurface conditions may differ, sometimes significantly, from the opinions presented in this report. Our report, conclusions and interpretations are not a warranty of the actual subsurface conditions.

Geotechnical Engineering Report Recommendations are Not Final

We have developed the following recommendations based on data gathered from subsurface investigation(s). These investigations sample just a small percentage of a site to create a snapshot of the subsurface conditions elsewhere on the site. Such sampling on its own cannot provide a complete and accurate view of subsurface conditions for the entire site. Therefore, the recommendations included in this report are preliminary and should not be considered final. GeoEngineers' recommendations can be finalized only by observing actual subsurface conditions revealed during construction. GeoEngineers cannot assume responsibility or liability for the recommendations in this report if we do not perform construction observation.

We recommend that you allow sufficient monitoring, testing and consultation during construction by GeoEngineers to confirm that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes if the conditions revealed during the work differ from those anticipated, and to evaluate whether earthwork activities are completed in accordance with our recommendations. Retaining GeoEngineers for construction observation for this project is the most effective means of managing the risks associated with unanticipated conditions. If another party performs field observation and confirms our expectations, the other party must take full responsibility for both the observations and recommendations. Please note, however, that another party would lack our project-specific knowledge and resources.

A Geotechnical Engineering or Geologic Report Could Be Subject to Misinterpretation

Misinterpretation of this report by members of the design team or by contractors can result in costly problems. GeoEngineers can help reduce the risks of misinterpretation by conferring with appropriate members of the design team after submitting the report, reviewing pertinent elements of the design team's plans and specifications, participating in pre-bid and preconstruction conferences, and providing construction observation.

Do Not Redraw the Exploration Logs

Geotechnical engineers and geologists prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. The logs included in a geotechnical engineering or geologic report should never be redrawn for inclusion in architectural or other design drawings. Photographic or electronic reproduction is acceptable, but separating logs from the report can create a risk of misinterpretation.

Give Contractors a Complete Report and Guidance

To help reduce the risk of problems associated with unanticipated subsurface conditions, GeoEngineers recommends giving contractors the complete geotechnical engineering or geologic report, including these "Report Limitations and Guidelines for Use." When providing the report, you should preface it with a clearly written letter of transmittal that:

- advises contractors that the report was not prepared for purposes of bid development and that its accuracy is limited; and
- encourages contractors to confer with GeoEngineers and/or to conduct additional study to obtain the specific types of information they need or prefer.

Contractors are Responsible for Site Safety on Their Own Construction Projects

Our geotechnical recommendations are not intended to direct the contractor's procedures, methods, schedule or management of the work site. The contractor is solely responsible for job site safety and for managing construction operations to minimize risks to on-site personnel and adjacent properties.

Biological Pollutants

GeoEngineers' Scope of Work specifically excludes the investigation, detection, prevention or assessment of the presence of Biological Pollutants. Accordingly, this report does not include any interpretations, recommendations, findings or conclusions regarding the detecting, assessing, preventing or abating of Biological Pollutants, and no conclusions or inferences should be drawn regarding Biological Pollutants as they may relate to this project. The term "Biological Pollutants" includes, but is not limited to, molds, fungi, spores, bacteria and viruses, and/or any of their byproducts.

A Client that desires these specialized services is advised to obtain them from a consultant who offers services in this specialized field.