Preliminary Design and Construction Recommendations – Revised

Floodwalls and Embankment Levees
North Shore Levee West Segment
Hoquiam, Washington

for
KPFF Consulting Engineers, Inc.

March 17, 2020

GeoEngineers
Earth Science + Technology

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Hoquiam, Washington

File No. 23944-001-00

March 17, 2020

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1.0 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 West Segment project in Hoquiam, Washington (City). 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 September 16, 2019 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 using high ground tie-ins. 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 from approximate intersection of Washington Street and Tyler Street and continuing south along Levee Street in downtown Hoquiam. Floodwalls are planned to continue around the Hoquiam Police Station and along 11th Street to the K Street Pump Station. Additional sections of the floodwalls are planned to continue along the southern alignment from the Simpson Avenue Bridge to the K Street Pump Station where earth embankment levees connect to roadway crossing closure gates. An additional floodwall section is planned for the southwest high ground tie-in near the intersection of Paulson Road and West Emerson Avenue. We understand that the top of the floodwalls will be established at Elevation 15.2 feet. This results in a wall height of about 2 to 5 feet at the tallest that tapers down to the ground surface at the high ground tie-ins along the planned alignment. 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.

2.0 BASE FLOOD ELEVATIONS

The design Base Flood Elevations (BFEs) have been established by the team hydraulics engineer, Watershed Science & Engineering (WSE). WSE analyzed two potential flooding sources along the proposed North Shore Levee West Segment, including coastal flooding (Grays Harbor) and riverine flooding (Hoquiam River). WSE conducted a hydraulic analysis and found the highest water levels along the Hoquiam River were controlled by the coastal BFE, as a result the controlling BFE throughout the project area is coastal flooding from Grays Harbor.

The stillwater elevation within the project area has been determined to be Elevation 13.1 feet. Wave run up and total water levels (TWL) under the design storm has been predicted to range between Elevation 13.8 feet and 15.1 (WSE, 2017). The top of the levees and floodwalls will be established at Elevation 15.2 feet to provide at least 2 feet of freeboard over the stillwater elevation. All elevations referenced in this report use the National American Vertical Datum of 1988 (NAVD88). The BFEs for the project area are based on coastal flooding in Grays Harbor, as a result, are driven in a large part by the tide cycles. Based on a previous study by WSE for the City of Aberdeen, 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.

3.0 SUBSURFACE CONDITIONS

Our understanding of the subsurface conditions is based on our subsurface exploration program and review of previous subsurface explorations performed by us and others. For this project, we explored the subsurface conditions along the North Shore Levee West Segment alignment by advancing six geotechnical borings, eight Cone Penetration Tests (CPTs) and three Electric Field Vane Shear Tests (eVST). This exploration program and other reviewed exploration programs will be described in more detail in our Levee Certification report. The approximate location of EACH exploration is shown in Figure 2, Site Plan. From our explorations, we identified two primary soil units, fill and alluvium.

The fill was observed within the upper 5 to 10 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 has been considered when evaluating potential seepage paths.

The alluvium was observed throughout the alignment and consisted primarily of soft to medium stiff 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 design groups. We named the design groups based on location and extent along the proposed alignment. The extents of these groups have been delineated by street name along the proposed levee and floodwall alignment, as depicted in Table 1 below. Additionally, the extents of each design group have been delineated and are shown in Figure 2. The recommendations in this report shall be applied to all analyses groups unless stated otherwise.

TABLE 1. HOQUIAM NORTH SHORE LEVEE DESIGN GROUPS

Design Group	Description of Design Group Extent (North to South along Proposed Alignment)	Type of Flood Protection
North Alignment	Perry Avenue/Endressen Road intersection (North High Ground Tie-in) to Queen Street Pump Station	Concrete floodwalls (T-walls) and earthen levee embankment
Hoquiam River	Queen Street Pump Station to Tyler Street	Earthen levee embankment and existing high ground tie-in
North Hoquiam	Tyler Street to Riverside Avenue Bridge	Sheet pile floodwalls (I-walls), concrete floodwalls (T-walls) and earthen levee embankment.
South Hoquiam	Riverside Avenue Bridge to Simpson Avenue Bridge	Concrete floodwalls (T-walls)

Design Group	Description of Design Group Extent (North to South along Proposed Alignment)	Type of Flood Protection
Grays Harbor	Simpson Avenue Bridge to West Emerson Avenue/Paulson Road (Southwest High Ground Tie-in)	Earthen levee embankment (Access Road near K Street Pump Station and Southwest tie-in). Sheet pile floodwalls (I-walls) at southwest tie-in

The soil properties for the Grays Harbor Design Group were developed based on subsurface explorations near the east end of this design group. We understand that some I-Walls will be required near the Southwest High Ground Tie-in, nearly a mile to the west of the closest exploration. The parameters for the Grays Harbor Design Group are assumed to be conservative and can be applied to these areas west of CPT-9-19 for preliminary or 30-percent design level analysis. However, additional explorations could be required for final design. We should review preliminary wall designs and discuss potential risks with the design team to determine if additional subsurface explorations are warranted.

4.0 LEVEE DESIGN AND CONSTRUCTION RECOMMENDATIONS

4.1. General

The design and construction recommendations provided in this report are based on our understanding of the 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 the 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.

4.2. Levee Embankment Fill

4.2.1. 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
¾-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 than 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 two percent (2%) organic material by weight. Recycled material such as asphalt, concrete rubble, recycled glass, or slag shall not be used.

4.2.2. 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).

4.3. 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 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.

4.4. Shallow Foundation Recommendations

4.4.1. 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.

4.4.2. 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.

4.4.3. Minimum Setback for Concrete Walls

We recommend a minimum setback from the existing embankment or slope crest of at least 15 feet for the concrete walls. If concrete walls are planned to be within 15 feet of the existing embankment crest, we recommend those sections be evaluated on a case-by-case basis or that sheet pile I-Walls be considered.

4.4.4. 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 "ultimate" 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)	Ultimate Bearing Capacity (psf)		
	North Alignment	South Hoquiam	Grays Harbor
2	2,529	1,738	1,436[1]
3	2,637	1,836	1,537[1]
4	2,745	1,934	1,638[1]

Note:

[1] Allowable bearing capacity based on soil properties presented in the report titled "Geotechnical Design Study, WSDOT SR 520 Pontoon Construction Project" in Grays Harbor, Washington by Landau Associates, dated March 25, 2009. We reviewed these properties with the previously completed boring logs and agree with the presented engineering soil parameters.

The net or additional dead weight of the floodwall is expected to be relatively low, less than 1,000 pounds per linear foot (plf). Net dead weight or additional dead weight is the weight of the wall and backfill minus the weight of the soil excavated for its construction. 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 up to 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.

4.4.5. 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 240 psf[3]
Levee Embankment Fill below Water Table	18[2]	190[1]	0.6 up to 240 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. The recommended maximum friction value is limited to the estimated cohesion for the near surface soil conditions. The friction resistance should not exceed this recommended value.

4.4.6. Lateral Load Resistance for Sheet Pile Walls

The proposed sheet pile floodwalls will be predominantly in an at-rest condition most of the time because the ground level (mudline) elevation will be approximately equal on both sides of the wall. However, during flood events the cantilevered floodwall will need to resist the floodwater that will rise on only one side of the floodwall. For this cantilevered condition that occurs during flood events, we recommend the sheet pile walls be designed using the USACE design program CWALSHT or similar analysis program using the soil parameters in Tables 5 and 6. This program does not allow for soil layers with variable properties (i.e., strength increase with depth). To account for this, we have divided the soil units into smaller increments and used average properties over depth.

TABLE 5. NORTH HOQUIAM DESIGN 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)
Silty Sand Alluvium (24 Deg.)	El. -2 feet	108	108	24	0	24	0
Silt Alluvium with organic interbeds (400 psf)	El. -23 feet	114	114	0	400	23[1]	0
Silt and Clay Alluvium with organic interbeds (400 psf at top, increasing by 25 psf per foot of depth.)	El. -27 feet	114	114	0	500	25[1]	0
	El. -31 feet	114	114	0	600	24[1]	0
	El. -35 feet	114	114	0	700	24[1]	0

Note:

[1] EPRI EL-6800, Manual on Estimating Soil Properties for Foundation Design, Cornell University. Critical void ration friction angle (φ_{cv}) for Normally Consolidated (NC) Clays, Figure 4-20, Page 4-22.

TABLE 6. GRAYS HARBOR DESIGN 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)
Silt and Clay Alluvium (240[1] psf)	El. -30 feet	101[1]	101[1]	0	240[1]	18[2]	0

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)
Silty Sand Alluvium (28 Deg.)	El. -70 feet	110[1]	110[1]	28	0	28	0

Notes:

[1] Soil properties obtained from report titled "Geotechnical Design Study, WSDOT SR 520 Pontoon Construction Project" in Grays Harbor, Washington by Landau Associates, dated March 25, 2009.

[2] EPRI EL-6800, Manual on Estimating Soil Properties for Foundation Design, Cornell University. Critical void ration friction angle (φ_{cv}) for Normally Consolidated (NC) Clays, Figure 4-20, Page 4-22.

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 using design software. The 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 found on flat level ground must also be checked for global stability. This will be provided in our Levee Certification report. A graphical presentation of these net (passive minus active pressures) design earth pressures are included in Figures 3 and 4. 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 of greater than 10 years. An "unusual" event 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 short duration. Accordingly, it is our opinion that only Q (Short Term) soil strengths should be used in evaluating the overtopping case.

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.

5.0 LIMITATIONS

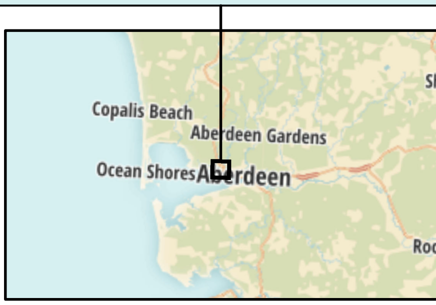
We have prepared this report for the exclusive use of KPFF Consulting Engineers, Inc. KPFF may distribute copies of this report to the City of Hoquiam (City), the City's authorized agents, and regulatory agencies including FEMA and FEMA's designated reviewers, as may be required for the project.

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 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 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, expressed or implied, should be understood.

Please refer to Appendix A titled "Report Limitations and Guidelines for Use" for additional information pertaining to the 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, 2016

Projection: NAD 1983 UTM Zone 10N

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Vicinity Map

North Shore Levee West Segment
Hoquiam, Washington

GEOENGINEERS

Figure 1

Legend

- Hoquiam Flood Protection Alignment
- Extent of Design Group
- CPT-1-19 — CPT Soil Boring by GeoEngineers, Inc., 2019
- GEI-1-19 — Boring by GeoEngineers, Inc., 2019
- VST-2-19 — Electric Vane Shear Testing by GeoEngineers, Inc., 2019
- TH-34-08 — Boring by Landau, 2008
- TH-3-03 — Boring by WA State Dept. of Trans., 2003
- **CPT-2-03** — CPT by WA State Dept. of Trans., 2003
- B-3 — Boring by AGRA Earth and Environmental, inc., 1997
- H-2 — Boring by WA State Dept. of Trans., 1967
- H-1 — Borings by WA State Dept. of Trans., 1957

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 Google Earth Pro dated 7/21/2018.
Hoquiam flood protection alignment obtained on 12/06/2019 from KPFF Consulting Engineers.

Projection: NAD83 Washington State Planes, South Zone, US Foot

N
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Feet

Site Plan

North Shore Levee West Segment
Hoquiam, Washington

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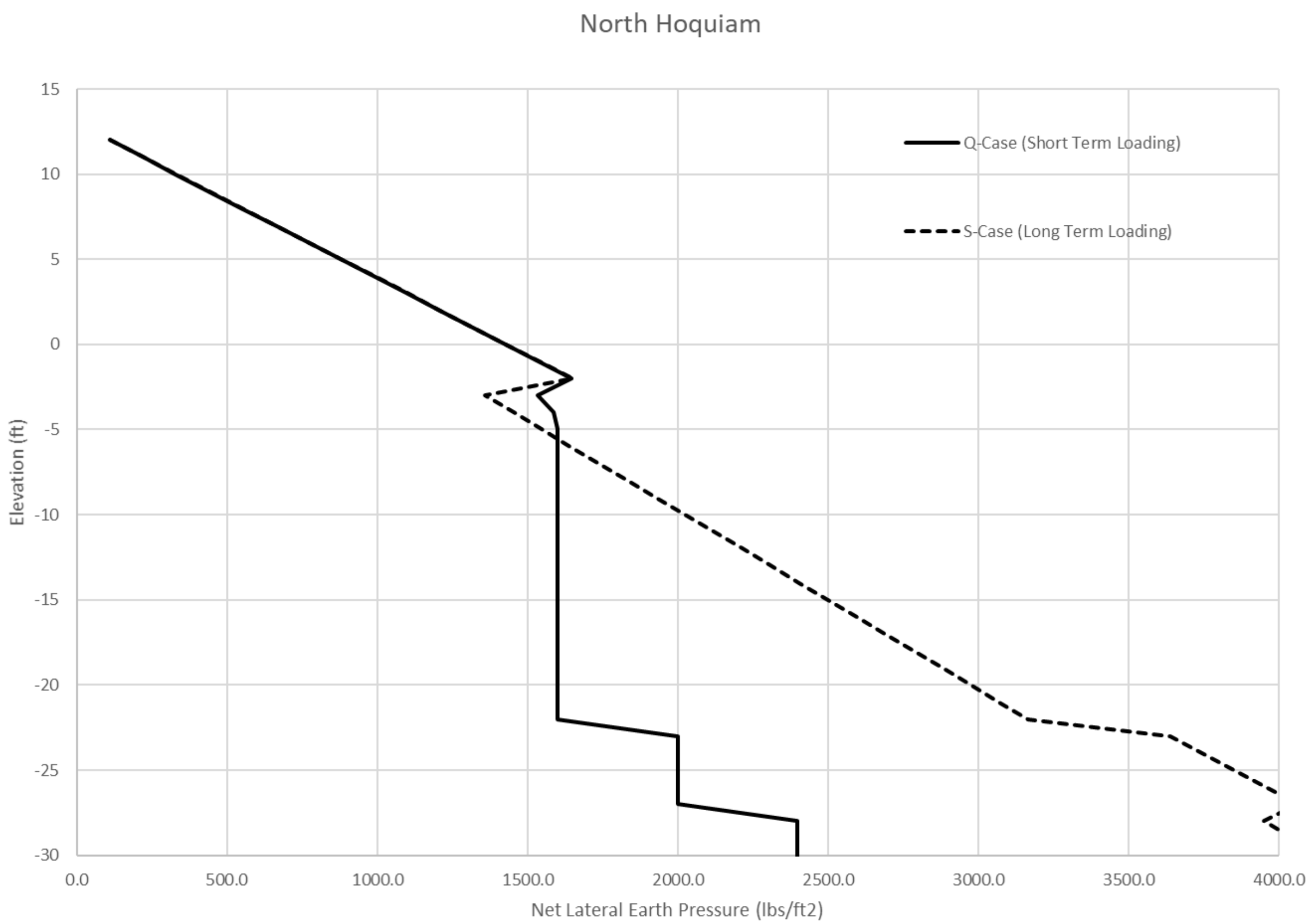
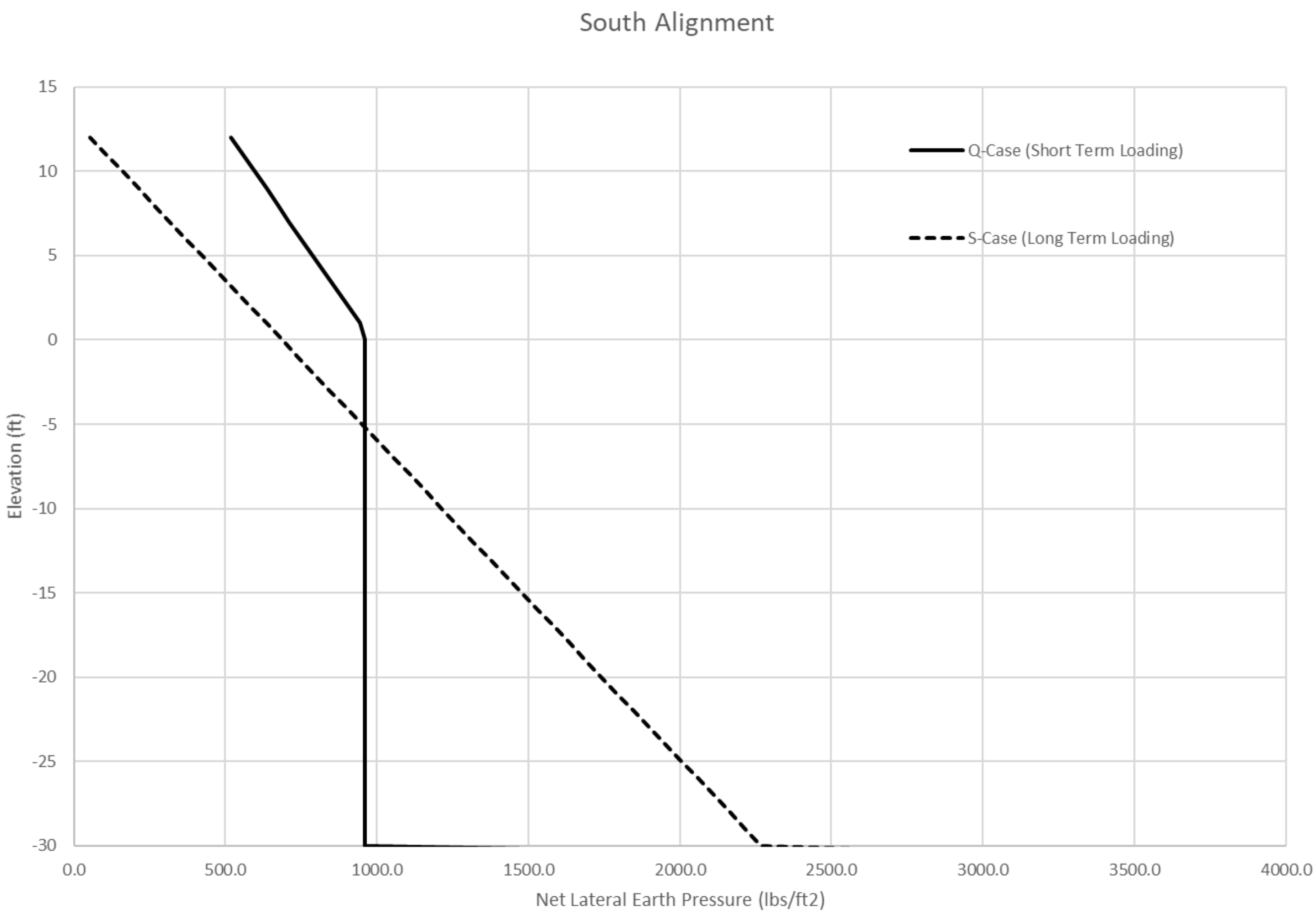
Figure 2



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

North Hoquiam Design Group – Lateral Earth Pressure
North Shore Levee West Segment Hoquiam, Washington
GeoEngineers — **Figure 3**



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

Source:
Soil properties obtained and confirmed from report titled, “Geotechnical Design Study, WSDOT SR 520 Pontoon Construction Project, Grays Harbor, Washington”. Landau Associates. Dated March 25, 2009.

Grays Harbor Design Group – Lateral Earth Pressure

North Shore Levee West Segment
Hoquiam, Washington

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Figure 4

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 sciences) 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 West Segment located in 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;

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

- elevation, configuration, location, orientation or weight of the proposed structure;
- Project ownership.

If important changes are made after the date of this report, GeoEngineers should be given the opportunity to review our interpretations and recommendations and provide written modifications or confirmation, as appropriate.

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 manmade events such as construction on or adjacent to the site, or by natural events such as floods, earthquakes, slope instability or groundwater fluctuations. Always contact GeoEngineers before applying a report to determine if it remains applicable.

Most 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 subsurface conditions only at those points where subsurface tests are conducted or samples are taken. GeoEngineers reviewed field and laboratory data and then applied our professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ, sometimes significantly, from those indicated in this report. Our report, conclusions and interpretations should not be construed as a warranty of the subsurface conditions.

Geotechnical Engineering Report Recommendations Are Not Final

Do not over-rely on the preliminary construction recommendations included in this report. These recommendations are not final, because they were developed principally from GeoEngineers' professional judgment and opinion. GeoEngineers' recommendations can be finalized only by observing actual subsurface conditions revealed during construction. GeoEngineers cannot assume responsibility or liability for this report's recommendations if we do not perform construction observation.

Sufficient monitoring, testing and consultation by GeoEngineers should be provided during construction to confirm that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes should the conditions revealed during the work differ from those anticipated, and to evaluate whether or not earthwork activities are completed in accordance with our recommendations. Retaining GeoEngineers for construction observation for this project is the most effective method of managing the risks associated with unanticipated conditions.

A Geotechnical Engineering or Geologic Report Could be Subject to Misinterpretation

Misinterpretation of this report by other design team members can result in costly problems. You could lower that risk by having GeoEngineers confer with appropriate members of the design team after submitting the report. Also retain GeoEngineers to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering or geologic report. Reduce that risk by having GeoEngineers participate in pre-bid and preconstruction conferences, and by 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. To prevent errors or omissions, the logs included in a geotechnical engineering or geologic report should never be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, but recognize that separating logs from the report can elevate risk.

Give Contractors a Complete Report and Guidance

Some owners and design professionals believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering or geologic report, but preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with GeoEngineers and/or to conduct additional study to obtain the specific types of information they need or prefer. A pre-bid conference can also be valuable. Be sure contractors have sufficient time to perform additional study. Only then might an owner be in a position to give contractors the best information available, while requiring them to at least share the financial responsibilities stemming from unanticipated conditions. Further, a contingency for unanticipated conditions should be included in your project budget and schedule.

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 to adjacent properties.

Read These Provisions Closely

Some clients, design professionals and contractors may not recognize that the geoscience practices (geotechnical engineering or geology) are far less exact than other engineering and natural science disciplines. This lack of understanding can create unrealistic expectations that could lead to disappointments, claims and disputes. GeoEngineers includes these explanatory "limitations" provisions in our reports to help reduce such risks. Please confer with GeoEngineers if you are unclear how these "Report Limitations and Guidelines for Use" apply to your project or site.

Geotechnical, Geologic and Environmental Reports Should Not Be Interchanged

The equipment, techniques and personnel used to perform an environmental study differ significantly from those used to perform a geotechnical or geologic study and vice versa. For that reason, a geotechnical engineering or geologic report does not usually relate any environmental findings, conclusions or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. Similarly, environmental reports are not used to address geotechnical or geologic concerns regarding a specific project.

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.

If Client desires these specialized services, they should be obtained from a consultant who offers services in this specialized field.