GEO CONSULTANTS NORTHWEST

May 25, 2018 R-3

Tetra Tech, Inc.
1020 SW Taylor Street, Suite 530
Portland, OR 97205

Attention: Kevin Coulton

Subject: Geotechnical Engineering Report
Hungry Harbor Restoration
GCN – 1239

GEO Consultants Northwest (GCN) is pleased to submit this Geotechnical Engineering Report for the proposed Hungry Harbor Restoration project that will be constructed along Washington State Route 401 in Pacific County, Washington. We were provided with 60 percent design level engineering drawings for the project that we used to form our understanding of the project. Portions of the drawings used in the figures contained in this report are from the 60 percent design set.

This report was prepared in accordance with our Scope, Project Fee and Schedule, dated June 15, 2017. This report summarizes the work accomplished and provides our recommendations for site development.

PROJECT INFORMATION

The Columbia River Estuary Study Taskforce (CREST) is working with partners and funding agencies to accomplish the Hungry Harbor Restoration project that will restore fish passage to spawning and rearing habitat. Central to the recovery is an improved passage through a perennial stream that crosses under Washington State Route 401. The site relative to surrounding features is shown in Figure 1.

The existing passage runs in a small culvert that is undersized and dilapidated and has been accumulating sediment both behind and in front of the roadway embankment. In addition to the sediment accumulation, the existing culvert sits several feet above the optimal elevation for passage of spawning and juvenile salmon.

The existing culvert will be replaced with a pre-cast concrete box culvert that will span just under 20 feet and will be about 10 feet tall. The replacement culvert will be located approximately 80 to 100 feet west of the existing culvert. Grading of the upstream and downstream channels will provide diversion of the existing creek flow to the new culvert new outfall into the Columbia river. The 60% design site plan is shown in Figure 2.

Roadway elevation is about 19 feet above mean sea level and the culvert invert elevation is about 1 foot. Cover above the culvert top will be about 4 feet. The 60 percent design shows that fill to channel thalweg will be about 3 feet thick.

The replacement roadway embankment is expected to be constructed with native soil that meets WSDOT fill specifications. The Columbia channel side of the embankment is to be protected with rip-rap and quarry spoils.

We expect the box culvert installation will require a cofferdam and that shoring at the ends of the culvert will be needed to reduce cofferdam length. The 60 percent inlet and outlet cross sections are shown in Figure 3.

SCOPE OF WORK

The purpose of our services was to explore the site conditions and provide recommendations for design and construction to the point of 60% design. Our site investigation followed the guidelines presented in the WSDOT Bridge Design Manual and Geotechnical Design Manual and referenced publications. The following describes our specific scope of work:

- Review information presented in the project RFP, the 2010 Habitat Enhancement Feasibility report with attachments and other information compiled from the design information provided to us. We also reviewed available geological and geotechnical information for the site and vicinity.
- Evaluate subsurface soil conditions by observing drilling of two mud-rotary borings, to depths of 21- ½ and 86- ½ feet below the existing ground surface (bgs) in the vicinity of the proposed new culvert. The explorations were backfilled with hydrated bentonite chips at the end of drilling.
- Conduct Standard Penetration Testing (SPT) in general accordance with ASTM D1586 using a split spoon sampler. In general, SPT tests were made at 2.5-foot intervals within the upper 10 feet of soil and at 5-foot intervals below 10 feet.
- Maintain a log of soil, rock, and groundwater conditions encountered in the borings, and obtain soil samples for laboratory testing. The samples obtained were classified in the field and returned to our laboratory for additional evaluation and testing. We classified the soil in general accordance with the Unified Soil Classification System (USCS).
- Drill four hand-auger borings within the upstream lowland area to evaluate the accumulation of sediment behind the existing culvert and highway embankment.
- Determine the moisture content, dry unit weight, Atterberg limits, and fines content of selected soil samples in general accordance with ASTM D2216, D2937, D4318, and D1140 respectively.
- Provide a written Geotechnical Report summarizing our explorations, geotechnical analysis, conclusions, and recommendations that include:
    - A discussion on the regional geology and the seismic setting including the general geologic features of the surface and underlying deposits and tectonic faulting in the area and seismic design criteria in accordance with the WSDOT and AASHTO requirements.
    - Description of subsurface conditions including estimated depth to groundwater.
    - Design ground motions and seismic classification analysis for the 1,000-year event.
    - Design recommendations for vertical and lateral support of the box culvert including soil bearing capacity, sliding resistance factors, vertical and lateral soil loads, and settlement.
    - Recommendations and design parameters for excavation for temporary and permanent slopes, shoring, and construction dewatering.

- Design recommendations for cast-in-place and segmental retaining walls that may be used for wing walls the culvert wing walls.

SITE CONDITIONS

SITE GEOLOGY

The project site is located along the north banks of the Columbia River, approximately 10 miles upstream from the river mouth at the Pacific Ocean. The stream that flows to the site flows downhill from the north, between two ridgelines that rise to the northeast and northwest of the site. The ridges consist of Eocene-age sandstone and siltstone. Sediment originating from the slopes has accumulated both behind and in front of the existing roadway embankment, conveyed through meandering streams to the culvert location. Sandstone bedrock is exposed alongside SR 401 west of the site to make way for the highway alignment.

The sediment infill of the creek channel is expected to be soil, sand, and gravel derived from the cliff point sandstone bedrock that is interbedded with marine and terrestrial materials of the Youngs Bay and Columbia River systems.

SEISMIC SETTING

In accordance with 2017 WSDOT BDM 8.3.3. E, "Seismic design need not be considered for buried structures with span lengths of less than 20 feet." Seismic design factors have been omitted from this report. A brief discussion of conditions is provided in the conclusions section below. There are no Quaternary faults within 10 miles of the site.

SEISMIC CONSIDERATIONS

CODE-BASED RESPONSE SPECTRUM

We understand the culvert structure will be designed to withstand forces from a 1000-year return interval seismic event (probability of exceedance of 7% in 75-years) in accordance with AASHTOLRFD Seismic Analysis and Design of Bridges Reference Manual (FHWA-NHI-15-004),. The AASHTO design methodology uses two spectral response coefficients, S_S and S_1, corresponding to periods of 0.2 and 1.0 second, to develop the earthquake response spectrum. The spectral response coefficients were obtained from the U.S. Geological Survey (USGS) Uniform Hazard Response Spectra Curves for the coordinates of 46.26076° N latitude and 123.84973° W longitude. The S_S and S_1 coefficients identified for the site are 0.909 and 0.427 g, respectively, for Site Class B conditions. These bedrock spectral ordinates are adjusted for Site Class with the 0.2- and 1.0-second period site coefficients, F_a and F_v, based on the soil profile in the upper 100 ft. This spectrum is designated the MCE_R-level spectrum. The design-level response spectrum is calculated as two-thirds of the Site Class- adjusted MCE_R spectrum.

SEISMIC HAZARDS

We have performed a preliminary liquefaction triggering analysis for a magnitude 9.0 earthquake and peak ground acceleration of 0.39s (A_s, Site Class E). We also performed a preliminary evaluation of a cross-section with respect to earthquake induced lateral spreading, one cross-section through the site into the Columbia River to the south. A detailed discussion of our preliminary analysis is summarized below.

LIQUEFACTION POTENTIAL

A preliminary liquefaction hazard assessment for the site has been completed. Liquefaction is a mechanism by which loose, saturated, granular materials, such as sands and low-plasticity silts, temporarily lose strength during and immediately after a seismic event. Liquefaction occurs when saturated granular soils are subjected to cyclic loading, which distorts the soil structure and causes loosely packed groups of particles to collapse, increasing porewater pressure in the soil mass. As pore water pressure increases, the soil begins to lose strength and may even behave as a viscous liquid in the most extreme cases. Liquefaction may result in ground surface settlement, decrease bearing capacity and settlement of shallow foundations, and lateral deformations towards the Columbia river.

The simplified procedure presented by Youd, et al. (2001), was used to evaluate the liquefaction hazard, with additional screening recommendations for fine-grained soils presented by Boulanger and Idriss (2006) and Bray and Sancio (2006). In this procedure, the demand imposed by the seismic loading is determined for different combinations of earthquake magnitude and peak ground acceleration (PGA), or from a suite of scaled earthquake records used in the site-response modeling. The demand, defined as the cyclic stress ratio (CSR), is then compared with the soil resistance, defined as the cyclic resistance ratio (CRR). Further discussion regarding determination of CRR based on different soil types is provided in the following sections. The factor of safety against liquefaction can then be calculated as the CRR/CSR. As the factor of safety against liquefaction approaches 1.0, there is an increased risk of cyclic strength loss and liquefaction induced settlement.

The simplified procedure is best calibrated for sandy soils, and the seismic behavior of silty soils is less well understood. However, recent research has begun to focus on the potential loss of strength and associated volumetric strain of fine-grained soils, such as silt, during cyclic loading. The recent research by Boulanger and Idriss and Bray and Sancio indicates the behavior of fine-grained soils during cyclic loading is highly dependent on soil plasticity and natural water content. Bray and Sancio suggest that soils with a plasticity index (PI) less than 18% and natural water content greater than 80% of the liquid limit (LL) are susceptible to liquefaction. Furthermore, Boulanger and Idriss suggest that during cyclic loading, soils with a PI less than 7% demonstrate “sand-like” behavior, and soils with PI greater than 7% demonstrate a “clay-like” behavior. The Atterberg limits testing completed for this study indicates portions of the fine-grained materials encountered at the site are susceptible to liquefaction based on at least one of the screening techniques.

The liquefaction potential of soils that demonstrate “clay-like” and “sand-like” behavior was evaluated using the simplified procedure of Youd, et al. and, as implemented in the LiqIT v.4.7.7.5 program by Geologismiki Software. For the purpose of liquefaction studies, we have conservatively estimated the groundwater table is located at an elevation of 10 feet below the top of the embankment. The CSR in this analysis was defined considering one earthquake magnitude and distance (M-R) pair, with an emphasis on the hazards that dominate the USGS probabilistic seismic hazard analysis (PSHA) for the site, a Cascadia Subduction Zone event.

Our liquefaction analyses used magnitude 9.0 earthquake and peak ground acceleration A_s of 0.39g. In the simplified procedure, CRR is determined on the basis of Standard Penetration Test (SPT) N-values, Cone Penetrometer Test (CPT) resistance values, and/or measured shear wave velocities, with correction factors that account for actual fines content.

It should be noted that the simplified method used to evaluate the liquefaction potential is based on historical databases of liquefied sites with shallow liquefaction, i.e., less than about 50 feet. For this reason, the simplified method may not be able to accurately predict liquefaction at depths greater than 50 to 60 feet. Accordingly, the depth of liquefaction described above, and the potential settlement and lateral spreading displacements are likely less than predicted. For preliminary planning, it should be assumed that potentially liquefiable soil extends to a depth of 80 to 100 feet below existing site grades. Available geotechnical data indicate variability in the soil conditions could locally affect the depth of liquefaction. For final design, we recommend that seismic cone penetration testing, and downhole shear wave velocity measurements coupled with an effective stress non-linear site response analysis be completed to further evaluate the potential for deep liquefaction.

LIQUEFACTION INDUCED SETTLEMENT

The results of our parametric studies, when the liquefaction depth is not limited, indicate that about 20 to 24 inches of liquefaction-induced settlements could occur within the sand and silt layers during a major seismic event. The results of our parametric studies when the liquefaction depth is limited to 50 feet are on the order of 10 to 15 inches of liquefaction settlement. These would represent the upper and lower bounds of potential liquefaction settlement at the site. Detailed results are attached in Attachment B.

LATERAL SPREAD

The potential for liquefaction-induced lateral spread was assessed using the empirical procedures described in Youd, et al. (2002). A “level-ground” condition was assumed to occur along the Columbia River channel below the embankment, the magnitude of lateral spread displacement was estimated as a function of the slope of the river bottom. The results indicate that the leading edge of the lateral spread zone could displace roughly 100 to 200 inches for the MCE_R using this approach.

SITE RECONNAISSANCE & SURFACE CONDITIONS

We conducted a walking reconnaissance of the site and surrounding area on September 26 and 27, 2017. During the reconnaissance we observed vegetation and topographic conditions. We also drilled four shallow hand auger borings in the near-surface sediment on the north (upstream) side of the roadway embankment.

The roadway embankment supporting SR 401 rises about 13 feet above the native ground surface in the site vicinity, the location of an unnamed creek that passes beneath the embankment. The native surface, predating the roadway, represents the north beach of the Columbia River at the confluence of the creek. The native slope was created through a combination of sediment deposition from the Columbia River and from the creek.

Our reconnaissance confirms the topographic mapping data available on the 60% design drawings. The drawings show the native surface on the upstream (north) side of the embankment is nearly flat. The native ground surface on the downstream (south) side is inclined downward at an inclination ranging from about 2 to 10 percent. The inclination decreases with distance toward the Columbia River. Bathymetric mapping of the Columbia channel shows the nearly flat slopes extend a significant distance farther south below the river's waterline.

SUBSURFACE CONDITIONS

We explored subsurface conditions at the site by drilling two mud-rotary borings (B-1 and B-2) to depths of 81½ and 21½ feet below ground surface (bgs), respectively. We also drilled four hand-auger boring (HA-1 to HA-4) to depths up to 6 feet bgs. The approximate location of the borings is shown in Figure 2.

Soil samples obtained from the borings were returned to our soil laboratory for additional evaluation. Selected samples were used to determine in-situ soil moisture, dry unit weight, fines content, and Atterberg Limits (clay behavior). Descriptions of field and laboratory procedures, exploration logs, and laboratory test results are included in Attachment A. Test results are also shown in the exploration logs.

We encountered an approximate 1-1/2-foot-thick section of asphalt pavement that was underlain by an approximate 11 to 12-foot-thick fill embankment. The fill was underlain to a depth of 86 feet by alternating layers of sand and gravel and silt. Based on the 60 percent design, the culvert will be founded near the top of an approximate 12-foot-thick layer of medium dense silty gravel.

A summary of the subsurface materials is provided below. Additional details of the subsurface layers are provided in the boring logs presented in Attachment A.

PAVEMENT SECTION

In boring B-1 we encountered an 18-inch thick asphalt pavement that was underlain by a two-foot-thick layer of silty sand with subangular to angular gravel that we interpreted to be the pavement base. The total pavement section was about 3-1/2 feet thick. In boring B-2 we encountered a similar 18-inch thick asphalt pavement. The fill beneath the asphalt pavement was indistinguishable from the embankment fill below.

EMBANKMENT FILL

Below the pavement section we encountered medium dense to dense silty gravel fill with sand, which extended to depths of 11 to 12 feet bgs. The gravel portion of this unit was angular, implying that crushed material was used as fill. The depth of the fill roughly correlates with the height of the existing embankment. Standard penetration test "N" values in the fill varied from 16 to 47 and the in-situ moisture content in the fill ranged from 18 to 31 percent.

NATIVE SILTY FINE SAND AND FINE SAND

Immediately below the fill we encountered an approximate 2-foot thick layer of medium dense silty sand. We infer this to be the original ground surface material before embankment construction began.

The silty sand was underlain by an approximate 3- to 4-foot thick layer of loose fine sand.

Standard penetration test "N" values in the silty sand and sand unit varied from 4 to 20 and the in-situ moisture content in the fill ranged from 14- to 16 percent.

SILTY GRAVEL

We encountered medium dense, subrounded, silty gravel with sand at approximate depths of 15 feet bgs in both borings. This unit extended to 28-1/2 feet in boring B-1 and beyond the depth penetrated in B-2 at 21-1/2 feet. This unit will directly support the planned culvert and is expected to be about 13 feet thick. The gravel and coarse sand fractions of the unit are primarily sandstone that can be carved with a knife.

Standard penetration test "N" values in the silty gravel sand and sand unit varied from 17 to 30 and the in-situ moisture content in the fill ranged from 23 to 30 percent.

SILT

We encountered an approximate 8-foot thick layer of stiff silt with fine to coarse subrounded sand underlying the silty gravel layer in boring B-1. The silt extended to a depth of about 36 feet.

Standard penetration test blow counts in the silt unit varied from 10 to 14 and the in-situ moisture content in the fill ranged from 23 to 26 percent.

DEEPER SOIL LAYERS

Boring B-1 was extended to a depth of 86-1/2 feet. Below the near surface layers described above, we encountered an additional eight distinct layers of soil that alternated between gravel and silt and varied in thickness from 2 to 13 feet thick. The gravel and silt units were like the unit descriptions above. The dry unit weight of a gravel sample obtained at 45 feet was 115 pounds per cubic foot. The fines content of three gravel samples varied from 15 to 39 percent. An Atterberg limits test of a silt sample obtained at 50 feet provided a Liquid Limit value of 46 percent and a Plastic Limit of 40 percent, supporting the field classification of this unit as silt (ML).

GROUNDWATER

Mud-rotary boring B-1 was flushed and bailed after drilling and left open overnight to equalize pressure. The static groundwater level was measured the following morning at 13-1/2 feet bgs, corresponding to the approximate level of the adjacent Columbia river.

WETLAND FINE SEDIMENT THICKNESS

We drilled four hand auger borings in the wetland area behind the roadway embankment to measure the thickness of the fine sediment near the ground surface. The borings penetrated to the depth of practical hand auger refusal in gravel. The boring locations are shown in Figure 2. The measurements obtained are provided below in Table 1.

TABLE 1 – WETLAND FINE SEDIMENT THICKNESS

EXPLORATION	FINE SEDIMENT THICKNESS (IN)	DEPTH DRILLED (FT)
HA - 1	36	4.5
HA - 2	48	6.0
HA - 3	30	4.5
HA - 4	54	6.0

CONCLUSIONS AND RECOMMENDATIONS

Design of the box culvert and site grading plans through the 60 percent stage of design were provided to us by Tetra Tech. The plans show that the culvert is expected to be founded about nineteen feet below existing highway grade. At the planned base elevation, the culvert will be founded on a relatively thick layer of medium dense gravel with silt and sand that will provide suitable bearing support. We expect that post construction settlement will be within acceptable tolerances given the strength of the gravel layer and immediate underlying silt layer and considering the effect of pre-loading that has occurred since the existing roadway embankment was constructed.

We expect that open cut construction methods with solider pile and lagging or sheet piling shoring can be successfully performed. The near surface soil units encountered in our borings suggest that placement of a cofferdam and shoring during construction should not require unusual methods.

Suitable native borrow material for backfill of the culvert may be found in the planned excavations on the upstream and downstream sides of the roadway prism. However, because of limited area for moisture conditioning and limited duration of dry weather conditions, it may not be a practical material for project use. Regardless of native borrow suitability, imported granular material should be used for backfill of the culvert wing walls and headwalls for a distance inboard lateral distance equal to the wall height.

In accordance with 2017 WSDOT BDM 8.3.3. E, “Seismic design need not be considered for buried structures with span lengths of 20 feet or less.” We considered whether conditions at the site call for seismic design, even if not required. We conclude that the site area is in a location where the greater roadway embankment will likely be severely compromised during a design level seismic event. Evaluation and remediation of the greater roadway embankment is outside the limited scope of the planned fish passage improvement.

Design of the retaining walls that support the roadway embankment does not fall under the provisions of BDM 8.3.3. E. Seismic design of these elements is required. Earth pressures to be used in the design are provided in the recommendations below.

Construction methods and materials should follow WSDOT M 41-10, “Standard Specifications for Road, Bridge, and Municipal Construction”, 2018 edition.

Supplemental recommendations for design and construction are provided below.

EXCAVATIONS AND SHORING

We expect that excavations on the order of 20 feet deep will be to reach the desired depth to allow for placement of the box culvert and associated walls. A combination of shoring and temporary excavation slopes may be used to complete the excavation, depending on the location of the cut with respect to the portion of the roadway that will be left operational. The shoring system may require tie-back anchors where the excavation is adjacent to settlement-sensitive structures.

Where the construction staging sequence permits, temporary excavation slopes above the water line can be constructed as steep as 1H:1V. To protect temporary excavation slopes from erosion caused by rainfall and subsequent runoff, the slopes should be covered with waterproof sheeting and all surface drainage directed away from the excavation. In addition, we recommend that surcharge loads due to construction traffic, material laydown, excavation spoils, etc., not be allowed within a horizontal distance of an unshored cut equal to H/2 ft from the top of the cut, where H is the height of the cut.

SHORING CRITERIA

We recommend the use of shoring to support the excavation where sloped excavations are not possible. Widespread practice in Washington state is to use shoring systems consisting of soldier piles and lagging, either cantilevered or with tieback anchors or internal bracing. The pattern and intensity of the lateral earth pressures on embedded walls and shoring will be governed by the height of the wall, soil type, the degree to which the walls are structurally supported, and whether the walls are drained. For the design of braced shoring systems and basement walls in areas supported by shoring, we recommend using the soil strength factors shown on Table 2. We recommend the construction contractor provide an independent shoring design, choosing lateral earth pressures and anchor adhesion based on the soil properties shown in Table 2.

TABLE 2– SOIL PROPERTIES FOR SHORING DESIGN

SOIL MATERIAL	MOIST UNIT WEIGHT (PCF)	COHESION INTERCEPT (PSF)	INTERNAL FRICTION ANGLE (DEG)
EXISTING EMBANKMENT SOIL	145*	0	32
IMPORTED CRUSHED ROCK	150	0	45
ON-SITE GRAVEL BORROW	145	0	35

* *Provide based on silty gravel embankment fill.*

CONSTRUCTION DEWATERING

Because of the location and the presence of the Columbia River and associated tidal fluctuations, the excavation will most likely require construction inside a cofferdam. In addition, a dewatering system will most likely be required as well to dewater the cofferdam and control groundwater.

The type and design of the cofferdam and dewatering systems should be the responsibility of the contractor, who is in the best position to choose systems that fit their overall plan of operation. Because of the importance of the cofferdam and dewatering system we recommend that the construction contract include separate line item be included for the preparation of the cofferdam and dewatering system by and experience engineering license to practice in the State of Washington. We further recommend that the design be submitted for comment by the project design team prior to payment for that item.

PERMANENT SLOPES

Permanent cut and fill slopes should not exceed a grade of 2H:1V (Horizontal to Vertical). Slopes that will be maintained by mowing should not be constructed steeper than 3H:1V. Structures and paved surfaces should be located at least 5 feet from the slope face.

The slopes should be planted with appropriate vegetation to provide protection against erosion. Surface water runoff should be collected and directed away from slopes steeper than 3H:1V to prevent water from running down the face of the slope.

STRUCTURAL FILL / PAVEMENT BASE

Structural fill is any material used for the roadway embankment, structure foundations, and mass grading fill.

ON-SITE SOIL

The on-site embankment fill and material excavated from the upstream and downstream areas of the project site area suitable for use as structural fill provided it can be moisture-conditioned, separated from unsuitable material, and compacted to the specified density. Particles larger than 6-inches should be excluded from the fill.

IMPORTED SOIL

Imported soil for structural fill should meet the requirements of WSDOT M 41-10 Section 9-03-17, Class A Foundation Material.

PAVEMENT BASE

Pavement base should meet the requirements of WSDOT M 41-10 Section 9-03-9(3).

SHALLOW FOUNDATIONS / CULVERT FOUNDATION

BEARING CAPACITY

It is expected that the box culvert and wall foundations will bear on medium dense silty gravel with sand that underlies the existing roadway embankment at a depth of about 12 feet. Based on the properties of the soil at this elevation, design of foundations for the culvert and walls may use an ultimate bearing capacity of 5,000 pounds per square foot (psf). A resistance factor (RF) of 0.45 is applicable for strength limit state design.

Settlement of the structures is expected to be controlled by elastic compression and will be less than 1 inch in total and less than ½ that value over a span length of 20 feet.

LATERAL RESISTANCE

The proposed box culvert will be supported equally on both sides and will not require lateral support. Foundation elements for walls will require lateral support and can be designed based on the resistance factors shown in Table 3.

TABLE 3 – LATERAL RESISTANCE FACTORS

SOIL TYPE	EQUIVALENT FLUID PRESSURE (γ – PCF)	FRICTION COEFFICIENT (μ)
CLASS A FOUNDATION MATERIAL	350	0.45

A resistance factor (RF) of 0.80 is applicable for strength limit state design.

FOUNDATION SUBGRADE PREPARATION

We recommend that foundations be underlain by a minimum 6-inch thickness of Class A Foundation Material compacted to not less than 95 percent of the maximum density as determined by AASHTO T-99.

RETAINING WALLS

The following recommendations assume that the walls consist of conventional, cantilevered retaining walls, the walls are not more than 20 feet in height, the backfill is drained, and the wall backfill consists of free-draining, imported angular crushed quarry rock (Class A Foundation Material). Re-evaluation of our recommendations will be required if retaining walls vary from these assumptions.

In general, cantilever retaining walls, such as the culvert wing walls, yield under lateral loads and should be designed with active lateral earth pressures. Restrained walls, such as the culvert sidewalls, should be designed to withstand at-rest lateral earth pressures. We recommend using the lateral earth pressures shown in Table 4. The loads are provided as equivalent fluid density (G).

Static lateral earth pressures acting on a retaining wall should be increased to account for surcharge loadings resulting from traffic, construction equipment, material stockpiles, or structures. The surcharge component provided is based on surcharge loading of 250 pounds per square foot.

TABLE 4 – EQUIVALENT FLUID DENSITY (G) ACTING ON RETAINING WALLS

WALL TYPE	BACKFILL CONDITION	BACKFILL COMPONENT (PCF)	SURCHARGE COMPONENT (PSF)	SEISMIC COMPONENT (PCF)
YIELDING WALL	FLAT	30	80	20
	3H:1V	45		25
NON-YIELDING WALL	FLAT	50	120	30
	3H:1V	60		35

Backfill behind the project retaining walls should be equivalent to Class A Foundation Material for a minimum distance of H, the wall height, behind the wall. Geotextile filter fabric should be placed between the Class A Foundation Material and any embankment fill that includes a fines fraction greater than 2 percent of the dry unit weight of the material.

Backfill should be placed and compacted as recommended for structural fill, except for backfill placed immediately adjacent to walls. To reduce pressure on the walls, backfill located within a horizontal distance of 3 feet from the retaining walls should be compacted to approximately 90% of the maximum dry density, as determined by ASTM D1557. Backfill placed within 3 feet of the wall should be compacted in lifts less than 6 inches thick using hand-operated tamping equipment (such as a jumping jack or vibratory plate compactor).

SLOPE STABILITY

We used the software SLOPE/W developed by GEO-SLOPE International, Ltd., of Calgary, Alberta, Canada to evaluate a cross section through the wing walls. The software uses Spencer's method of slices to evaluate the static equilibrium of the model for both force and moment equilibrium while assuming that resultant inter-slice forces are of constant orientation throughout the sliding mass. This method of analysis is valid for circular and non-circular failure surfaces.

The factor of safety against sliding is defined as the ratio of the forces resisting movement to the forces driving movement. Factors of safety less than 1 infer that the model is not in equilibrium, and movement is likely to occur. Slopes with static factors of safety greater than about 1.3 are generally considered to have an acceptable risk against slope failure. The results of our static, seismic (pseudo-static), and post-liquefaction (residual-strength) analysis are provided in Attachment C.

LIMITATIONS

This report was prepared for the exclusive use of CREST and members of the design team for this specific project. It should be made available to prospective contractors for information on the factual data only, and not as a warranty of subsurface conditions, such as those interpreted from the explorations and discussed in this report.

The recommendations contained in this report are preliminary, and are based on information derived through site reconnaissance, subsurface testing, and knowledge of the site area. Variation of conditions within the area and the presence of unsuitable materials are possible and cannot be determined until exposed during construction. Accordingly, GCN's recommendations can be finalized only through GCN's observation of the project's earthwork construction. GCN accepts no responsibility or liability for any party's reliance on GCN's preliminary recommendations.

Unanticipated soil conditions are commonly encountered and cannot fully be determined by exploratory methods. Such unexpected conditions frequently require that additional expenditures be made to attain properly-constructed projects. Therefore, a contingency fund is recommended to accommodate the potential for extra costs.

Within the limitations of the scope of work, schedule, and budget, the analyses, conclusions, and recommendations presented in this report were prepared in accordance with generally-accepted professional geotechnical engineering principles and practice in this area at the time this report was prepared. We make no warranty, either express or implied.

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We appreciate the opportunity to be of continued service to you. Please call if you have questions concerning this report or if we can provide additional services.

Sincerely,
GEO Consultants Northwest, Inc.

EXPIRES 02/25/2019

EXPIRES 6/30/2019

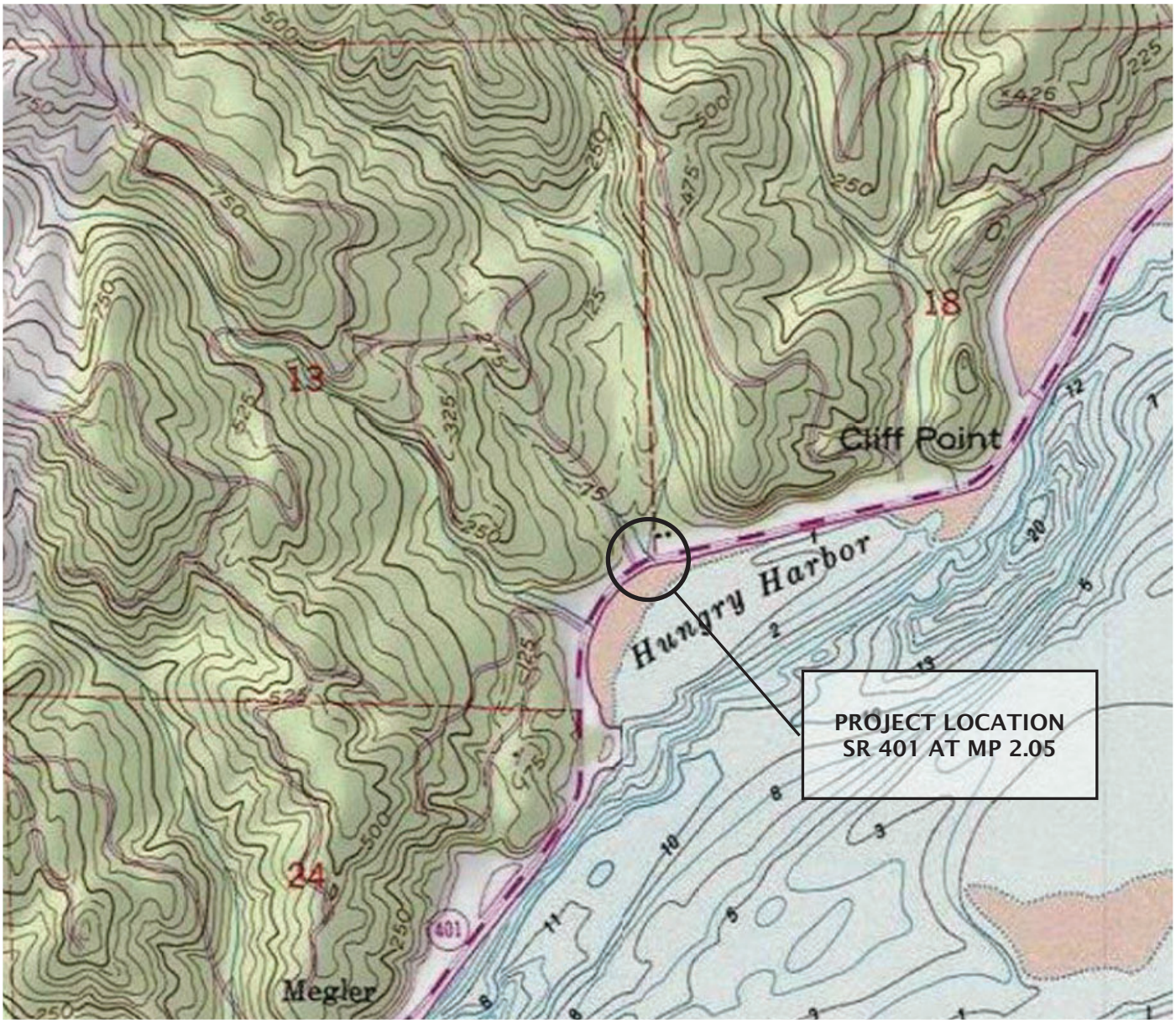
David A. Rankin, LEG
Principal Engineering Geologist

Britton W. Gentry, PE, GE
Principal Engineer

Figures: Figure 1 – Site Vicinity
Figure 2 –Site Layout with Explorations

Attachments: Attachment A – Field Exploration and Laboratory Testing

Record of Revision: Original Version: January 26, 2018
R-1 – April 10, 2018 - Added Seismic Analysis
R-2 – Revised reference to BDM 8.3.3 E vice superseded design memorandum, added note to Table 2, added paragraph to recommendations introduction section concerning seismic design of retaining walls.
R-3 – May 25, 2018 Slope stability analysis on cross-section through wing wall.



USGS KNAPPTON QUADRANGLE PROVIDED BY ACME MAPPER. 21

LAT 46.261 N, LON 123.849 W; TOWNSHIP 9N RANGE 10W SECTION 13

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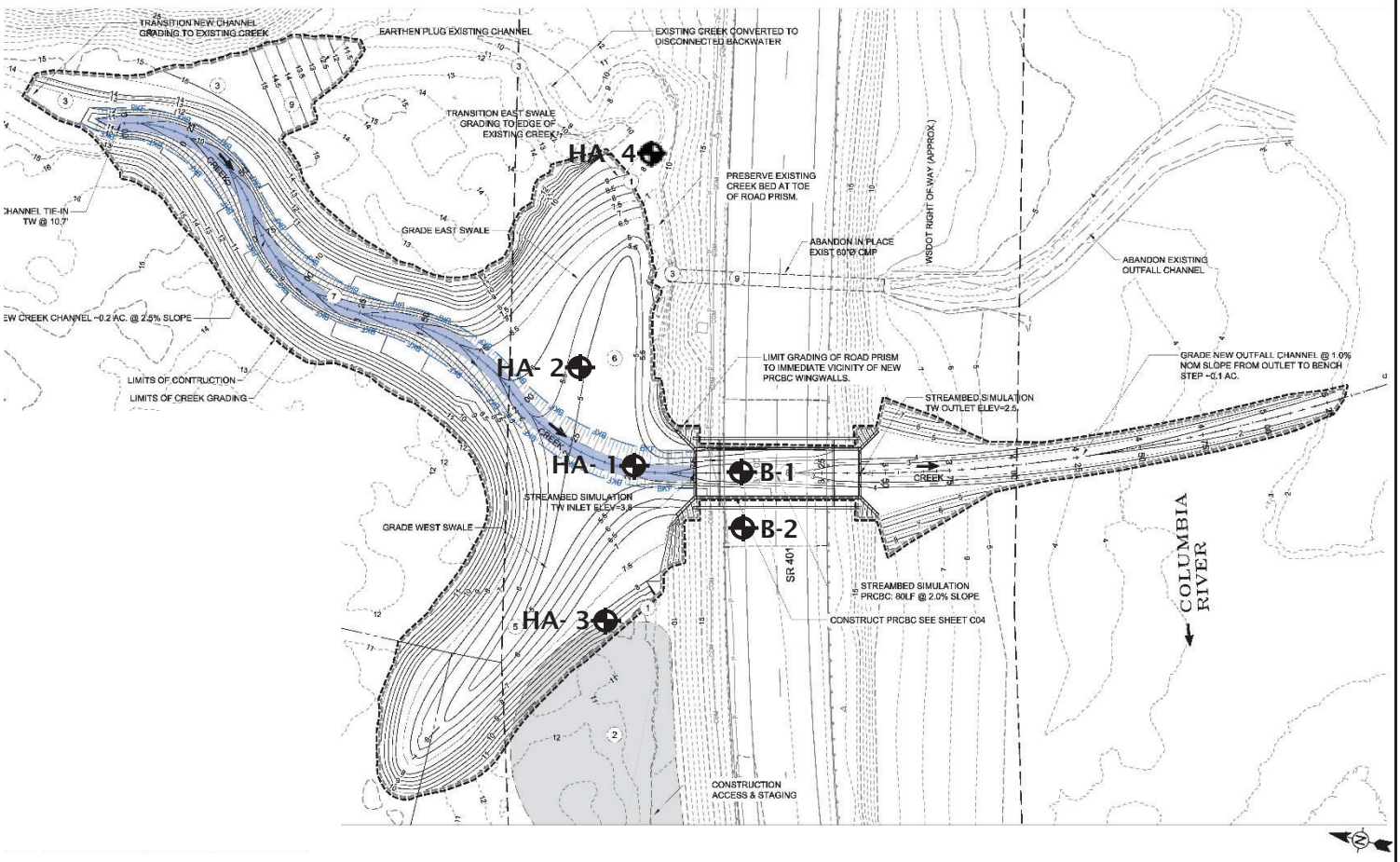
GEO CONSULTANTS NORTHWEST	PROJECT 1239	TETRA TECH HUNGRY HARBOR SR 401 UNDERCROSSING	
824 SE 12th Avenue Portland, OR 97214	JAN 2018 Drawn By: TAC	SITE VICINITY	FIGURE 1



B-1 / HA-1 ⊕ MUD ROTARY AND HAND AUGER BORINGS DRILLED SEPTEMBER 26TH – 27TH, 2017
LOCATIONS APPROXIMATE

BASE DRAWING: CREEK GRADING PLAN - C02.1- 60 PERCENT PLAN SET
PREPARED BY TETRA TECH - JANUARY 22, 2018

GEO CONSULTANTS NORTHWEST	PROJECT 1239	TETRA TECH HUNGRY HARBOR SR 401 UNDERCROSSING	
824 SE 12th Avenue Portland, OR 97214	JAN 2018 Drawn By: BLH	SITE LAYOUT & EXPLORATIONS	FIGURE 2

ATTACHMENT A

FIELD EXPLORATION PROCEDURES

LABORATORY TESTING PROCEDURES

KEY TO EXPLORATION LOGS

BORING LOGS B1 & B2

LABORATORY TEST RESULTS

FIELD EXPLORATION PROCEDURES

GENERAL

We explored subsurface conditions at the site by drilling two mud-rotary borings (B-1 and B-2) to depths of 86½ and 21½ feet below the ground surface (bgs), respectively, on September 26 and 27, 2017. The borings were drilled with a truck-mounted drill rig operated by Holt Drilling of Puyallup, Washington. The approximate boring locations are shown in Figure 2.

SOIL SAMPLING

A member of GCN's geotechnical staff observed subsurface explorations to record the soil, rock, and groundwater conditions encountered, and to obtain soil samples for laboratory testing.

Relatively undisturbed soil samples were obtained using a standard 3-inch outside diameter Shelby tube, in general accordance with ASTM D1587, *Standard Practice for Thin-Walled Tube Sampling of Soils for Geotechnical Purposes*. A disturbed soil sample was obtained from a mechanical boring using a 2-inch outside diameter split spoon sampler in general conformance with guidelines presented in ASTM D1586, *Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils*.

Soil samples were sealed to retain moisture, and returned to our laboratory for additional examination and testing.

FIELD CLASSIFICATION

Soil samples were initially classified visually in the field. Consistency, color, relative moisture, degree of plasticity, peculiar odors, and other distinguishing characteristics of the soil samples were noted. The terminology used is described in the key and glossary that follow.

STANDARD PENETRATION TESTING

For standard penetration testing, the split spoon sampler was driven into the soil 18 inches or to refusal using a 140-pound hammer free falling 30 inches. Typically, the tests are performed using an auto hammer. The sum of the blows required to drive the split spoon sampler the final two increments of 6 inches is recorded in the boring summary exploration log. If the sampler met refusal, the number of inches driven, and the number of blows is recorded. Blow counts shown in the log are uncorrected blow counts as recorded in the field.

SUMMARY EXPLORATION LOGS

Results from the test pits, borings, and cone penetrometer testing are show in the summary exploration logs. The left-hand portion of a log provides our interpretation of the soil encountered, sample depths, and groundwater information. The right-hand, graphic portion of a log shows the results of pocket penetrometer and laboratory testing. Atterberg limits are shown in the Atterberg Limits Results Chart. Soil descriptions and interfaces between soil types shown in summary logs are interpretive, and actual transitions may be gradual.

LABORATORY TESTING PROCEDURES

Soil samples obtained during field explorations are examined in our laboratory, and representative samples may be selected for further testing. The testing program may include visual-manual classification, natural moisture content, dry unit weight (in-place dry density), or Atterberg limits.

VISUAL-MANUAL CLASSIFICATION

Soil samples are classified in general accordance with guidelines presented in ASTM D2488, *Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)*. The physical characteristics of the samples are noted, and the field classifications are modified, where necessary, in accordance with ASTM terminology, though certain terminology that incorporates current local engineering practice may be used. The term which best described the major portion of the sample is used to describe the soil type.

NATURAL MOISTURE CONTENT

Natural moisture content is determined in general accordance with guidelines presented in ASTM D2216, *Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass*. The natural moisture content is the ratio, expressed as a percentage, of the weight of water in a given amount of soil to the weight of solid particles.

DRY UNIT WEIGHT (IN-PLACE DRY DENSITY)

Dry unit weight (in-place dry density) testing is performed in general accordance with guidelines presented in ASTM D2937, *Standard Test Method for Density of Soil in Place by the Drive-Cylinder Method*. The dry unit weight is defined as the ratio of the dry weight of the soil sample to the volume of that sample. The dry unit weight typically is expressed in pounds per cubic foot.

FINES CONTENT

Fines content testing is performed in general accordance with guidelines presented in ASTM D1140, *Standard Test Methods for Determining the Amount of Material Finer than 75-μm (No. 200) Sieve in Soils by Washing*. The fines content is the fraction of soil that passes the U.S. Standard Number 200 Sieve. This sieve differentiates fines (silt and clay) from fine sand. Soil material that remains on the 200 sieve is sand. Material that passes the sieve is fines. The test is used to refine soil type.

ATTERBERG LIMITS

The Atterberg limits are a basic measure of the critical water contents of a fine-grained soil. The behavior of fine-grained soil can change markedly at different water contents, and this analysis aids in soil classification. Atterberg Limits are determined in general accordance with guidelines presented in ASTM D4318, *Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils*.

BORING AND TEST PIT LOGS

DISTINCTION BETWEEN FIELD LOGS AND FINAL LOGS

A field log is prepared for exploration by our field representative. The log contains information concerning soil and groundwater encountered, sampling depths, sampler types used and identification of samples selected for laboratory analysis. The final logs presented in this report represent our interpretation of subsurface conditions based on the contents of the field logs, observations made during explorations, and the results of laboratory testing. Our recommendations are based on the contents of the final logs and the information contained therein, and not on the field logs.

SOIL CLASSIFICATION SYSTEM

Soil samples are classified in the field in general accordance with the United Soil Classification System (USCS) presented in ASTM D2488 "Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)." Final logs reflect field soil classifications and laboratory testing results. A summary of the USCS is provided on page 3. Classifications and sampling intervals are shown in the logs.

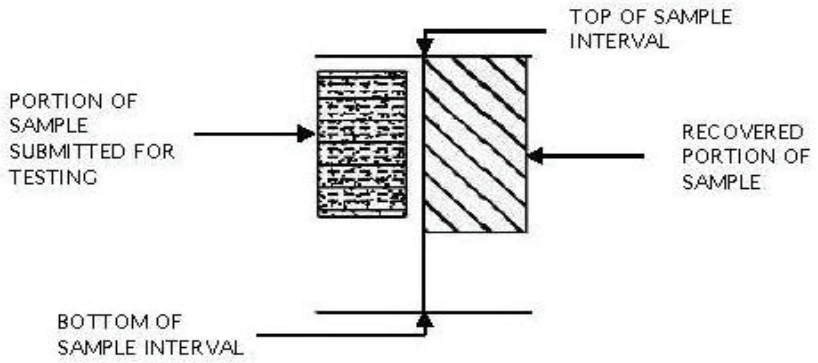
VARIATION OF SOIL BETWEEN EXPLORATIONS

The final logs and related information depict subsurface conditions only at the specific location and on the date(s) indicated. Those using the information contained herein should be aware that soil conditions at other locations or on other dates may differ.

TRANSITION BETWEEN SOIL AND ROCK CLASSIFICATIONS

The lines designating the interface between soil, fill, or rock on the final logs and on the subsurface profiles presented in the report are determined by interpolation and are, therefore, approximate. The transition between the materials may be abrupt or gradual. Only at specific exploration locations should profiles be considered as reasonably accurate and then only to the degree implied by the notes.

BORING LOG SAMPLES



GEO CONSULTANTS NORTHWEST	2017	KEY TO BORING AND TEST PIT LOGS	
824 SE 12th Avenue Portland, OR 97214	Drawn By: GCN	GENERAL INFORMATION	1/5

EXPLORATION LOG SYMBOLS

- Sample Location with No Sample Recovery
- Sample Location Using Direct Push Sampler (ASTM D 6282)
- Sample Location Using Ring-Lined Barrel Sampler (ASTM D 3550)
- Sample Location Using Split-Barrel Sampler (ASTM D 1586)
- Sample Location Using Thin-Walled Tube Sampler (ASTM D 1587)
- Rock Core Interval
- Grab Sample Location
- Soil Sample Submitted for Chemical Testing
- Soil Sample Submitted for Physical Property Testing
- Water Sample Screened Interval
- Water Sample Submitted for Chemical Testing
- Water Sample Tested in the Field
- Groundwater Level Encountered While Drilling
- Static Groundwater Level
- Perched Groundwater
- Groundwater Level at Time of Sampling

SOIL CHARACTER

Granular Soil		Cohesive Soil		
Density	Standard Penetration Test *	Consistency	Standard Penetration Test*	Unconfined Compressive Strength (tsf)
Very Loose	0 - 4	Very Soft	Less Than 2	Less Than 0.25
Loose	4 - 10	Soft	2 - 4	0.25 - 0.5
Medium Dense	10 - 30	Medium Stiff	4 - 8	0.50 - 1.0
Dense	30 - 50	Stiff	8 - 16	1.0 - 2.0
Very Dense	Greater Than 50	Very Stiff	16 - 32	2.0 - 4.0
Blows Required to Drive a Split-Barrel Sampler 12 inches		Hard	Greater Than 32	Greater Than 4.0

DEFINITIONS AND ABBREVIATIONS

AT	ATTERBERG LIMITS TEST	ND	NON DETECT	PPB	PARTS PER BILLION
BGS	BELOW GROUND SURFACE	NEG	NEGATIVE RESULT	PPM	PARTS PER MILLION
CO	CONSOLIDATION TEST	NS	NO VISIBLE SHEEN	PSF	POUNDS PER SQUARE FOOT
DS	DIRECT SHEAR TEST	OC	ORGANIC CONTENT	RS	SOIL RESISTIVITY TEST
DW	DRY UNIT WEIGHT	P	PUSHED SAMPLE	S4	SUDAN IV SOIL TEST
GS	MECHANICAL GRAIN SIZE TEST	P200	P200 FINES CONTENT TEST	SG	SPECIFIC GRAVITY TEST
HS	HEAVY SHEEN	PCF	POUNDS PER CUBIC FOOT	SPT	STD. PENETRATION TEST
HYD	HYDROMETER TEST	PH	SOIL pH	SS	SLIGHT SHEEN
MC	MOISTURE CONTENT	PID	PHOTOIONIZATION DETECTOR	TO	TOREVANE
MG/KG	MILLIGRAMS PER KILOGRAM	POS	POSITIVE RESULT	TSF	TONS PER SQUARE FOOT
MS	MODERATE SHEEN	PP	POCKET PENETROMETER	UV	ULTRAVIOLET LIGHT TEST

GRAIN SIZE DEFINITIONS			MINOR FRACTIONS IN FINE GRAINED SOIL		GROUNDWATER SEEPAGE	
SAND	FINE	No. 200 to No. 40	No Mention (CLAY, SILT)	< 15 percent	Slow	< 1 gpm
	MEDIUM	No. 40 to No. 10	With Sand, With Gravel	15 to 30 percent	Moderate	1-3 gpm
	COARSE	No. 10 to No. 4	Sandy, Gravelly	30 to 49 percent	Rapid	> 3 gpm
GRAVEL	FINE	No. 4 to 3/4-inch	FIELD MOISTURE OBSERVATION		CAVING	
	COARSE	3/4- to 3-inch	Dry	Absence of moisture, dusty, dry to touch	Minor	
COBBLE		3-inches to 12-inches	Moist	Damp but no visible water.	Moderate	
BOULDER		> 12-inches	Wet	Saturated, below groundwater	Severe	

GEO CONSULTANTS NORTHWEST	2017	KEY TO BORING AND TEST PIT LOGS	
824 SE 12th Avenue Portland, OR 97214	Drawn By: GCN	SYMBOLS AND ABBREVIATIONS	2/5

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

GEO CONSULTANTS NORTHWEST	2017	KEY TO BORING AND TEST PIT LOGS	
824 SE 12th Avenue Portland, OR 97214	Drawn By: GCN	SOIL CLASSIFICATION	3/5

ROCK CLASSIFICATION GUIDELINES

HARDNESS		DESCRIPTION
Very soft	(RH-0)	For plastic material only
Soft	(RH-1)	Carved or gouged with a knife
Moderate	(RH-2)	Scratched with a knife
Hard	(RH-3)	Difficult to scratch with a knife
Very hard	(RH-4)	Rock scratches metal; rock cannot be scratched with a knife

STRENGTH	DESCRIPTION
Plastic	Easily deformable with finger pressure
Friable	Crumbles by rubbing with fingers
Weak	Crumbles only under light hammer blows
Moderately Strong	Few heavy hammer blows before breaking
Strong	Withstands few heavy hammer blows and yields large fragments
Very Strong	Withstands many heavy hammer blows, yields dust and small fragments

WEATHERING	DESCRIPTION
Severe	Rock decomposed; thorough discoloration; all fractures extensively coated with clay, oxides, or carbonates.
Moderate	Intense localized discoloration of rock; fracture surfaces coated with weathering minerals.
Little	Slight and intermittent discoloration of rock; few stains on fracture surfaces.
Fresh	Rock unaffected by weathering

FRACTURING	FRACTURE SPACING
Crushed	Less than 5/8 inch to contains clay
Highly Fractured	5/8 inch to 2 inches
Closely Fractured	2 inches to 6 inches
Moderately fractured	6 inches to 1 foot
Little Fractured	1 foot to 4 feet
Massive	Greater than 4 feet

JOINT SPACING	DESCRIPTION
Papery	Less than 1/8 inch
Shaley or Platey	1/8 inch to 5/8 inch
Very Close	5/8 inch to 3 inches
Close	3 inches to 2 feet
Blocky	2 to 4 feet
Massive	Greater than 4 feet

GLOSSARY

Alluvial – Made up of or found in the materials that are left by the water of rivers, streams, floods, etc.

Bearing pressure – The total stress transferred from the structure to the foundation, then to the soil below the foundation.

Bulk density (Soil density) – The total mass of water and soil particles contained in a unit volume of soil: lb/ft^3.

Coefficient of active earth pressure – The ratio of the minimum horizontal effective stress of a soil to the vertical effective stress at a single point in a soil mass retained by a retaining wall as the wall moves away from the soil.

Cohesive soil – Clay type soil with angles of internal friction close to zero. Cohesion is the force that holds together molecules or like-particles within a substance.

Colluvium – A loose accumulation of soil and rock fragments deposited through the action of gravity, such as erosion and soil creep.

Differential settlement – The vertical displacement due to settlement of one point in a foundation with respect to another point of the foundation.

Engineered fill – Soil used as fill, such as retaining wall backfill, foundation support, dams, slopes, etc., that are to be placed in accordance with engineered specifications. These specifications may delineate soil grain-size, plasticity, moisture, compaction, angularity, and many other index properties depending on the application.

Excess pore pressure – That increment of pore water pressures greater than hydro-static values, produced by consolidation stresses in compressible materials or by shear strain; excess pore pressure is dissipated during consolidation.

Factor of safety – The ratio of a limiting value of a quantity to the design value of that quantity.

Fines – Material by weight passing the U.S. Standard No. 200 Sieve by washed analysis.

Fluvial – Produced by the action of rivers or streams.

Homogenous soil – A mass of soil where the soil is of one characteristic having the same engineering and index properties.

In situ – Undisturbed, existing field conditions.

Lacustrine – Of a lake, e.g., the depositional environment of a lake.

Liquefaction – The sudden, large decrease of shear strength of cohesionless soil caused by collapse of the soil structure, produced by small shear strains associated with sudden but temporary increase of pore water pressure. Usually a problem in submerged, poorly graded sands within the upper 50 feet of subgrade in earthquake-prone environments.

Maximum dry density – A soil property obtained in the laboratory from a Proctor test. Density of soil at 100% compaction.

Overbank deposit – Sediment that has been deposited on the floodplain of a river or stream by flood waters that have broken through or overtopped the banks.

Permeability – A measure of continuous voids in a soil. The property which allows the flow of water through a soil. See also coefficient of permeability.

Porosity (Pore space) – The ratio of the volume of voids to the total volume: unitless or expressed as a percentage.

Residual soil – Soil that has been formed in place by rock decay.

Shear strength – The maximum shear stress which a soil can sustain under a given set of conditions. For clay, shear strength = cohesion. For sand, shear strength = the product of effective stress and the tangent of the angle of internal friction.

Surcharge – An additional force applied at the exposed upper surface of a restrained soil.

Tuff – An igneous rock (from molten material) that forms from the debris ejected by an explosive volcanic eruption.

Unit weight – The ratio of the total weight of soil to the total volume of a unit of soil: lb/ft^3.

DEPTH (ft bgs)	GRAPHIC LOG	USCS SYMBOL	SOIL DESCRIPTION	SAMPLE	BLOW COUNT SPT N VALUE	MOISTURE CONTENT (%)	GROUNDWATER	FIELD TESTING	TESTING AND LABORATORY DATA
0		ACC	ASPHALT PAVEMENT - 18 inches thick						
		SM	Dense, Brown-gray, SILTY SAND FILL with subangular to angular gravel; moist.	1	21-19-27	18			
5		GM	Medium dense, brown-gray SILTY GRAVEL FILL with sand; moist.	2	10-10-9	31			
				3	5-7-11	21			
10				4	5-7-17	21			
		SM	Medium dense, gray, fine to medium SILTY SAND; moist.	5	7-12-8	17			
15		SP	Loose, poorly graded SAND with trace organics (wood fragments); moist.	6	5-2-2				
		GM	Medium dense, gray to gray-brown SILTY GRAVEL with sand, trace organics (wood fragments); moist to wet.	7	5-9-8	30			
20				8	10-11-15	23			
25			Becomes red-brown at 25 feet.	9	13-15-15	23			
30		ML	Stiff, gray SILT with fine gravel and sand; wet.	10	9-7-7	26			
35				11	5-5-5	23			
40			Loose to very loose, red-brown GRAVEL with fine sand and silt; wet.						

BORING METHOD: Mud Rotary
BOREHOLE DIAMETER: 5 7/8
DRILL RIG: Mobile B-54
CONTRACTOR: Holt Drilling Services
LOGGED BY: Tim North, PE GE

ELEVATION REFERECE:
GROUND SURFACE ELEVATION:
CASING ELEVTATION:
LOCATION: See Figure 2
DRILLING DATES: 9/26/17 9/27/17

START CARD/TAG ID:
REMARKS:

Tetra Tech - Hungry Harbor, WA

1239

GEO Consultants Northwest
824 SE 12th Avenue
Portland OR 97214
Tel 503-616-9425
Fax 1-866-293-9037

GEO CONSULTANTS NORTHWEST

LOG OF BORING
B-1

DEPTH (ft bgs)	GRAPHIC LOG	USCS SYMBOL	SOIL DESCRIPTION	SAMPLE	BLOW COUNT SPT N VALUE	MOISTURE CONTENT (%)	GROUNDWATER	FIELD TESTING	TESTING AND LABORATORY DATA
		GM	Becomes loose, loss of circulation - no recovery on all samples between 40 and 42 feet.	12	0-0-0				
				13	P				DD = 115 pcf, FC = 15%
45				14	P	17			FC = 16%
		GP	Medium dense, dark gray, poorly graded, subangular GRAVEL with sand; wet.	15	10-7-11	28			
50		ML	Stiff, dark gray SILT; wet.	16	4-5-4	47			LL = 46, PL=40, PI = 6
55									
60		GM	Medium dense, dark gray-brown SILTY GRAVEL with sand; wet.	17	7-8-12	31			FC = 39%
		ML	Stiff, dark gray SILT; wet.						
65				18	3-6-12	36			
		GM	Medium dense, red-brown, subrounded SILTY GRAVEL with sand; wet.						
70				19	7-11-4	26			
75				20	5-7-9	36			
			Stiff, dark gray SILT with fine sand and trace subrounded gravel; wet.						
80									

BORING METHOD: Mud Rotary	**ELEVATION REFERECE:**	**START CARD/TAG ID:**
BOREHOLE DIAMETER: 5 7/8	**GROUND SURFACE ELEVATION:**	**REMARKS:**
DRILL RIG: Mobile B-54	**CASING ELEVTATION:**	
CONTRACTOR: Holt Drilling Services	**LOCATION:** See Figure 2	
LOGGED BY: Tim North, PE GE	**DRILLING DATES:** 9/26/17 9/27/17	

DEPTH (ft bgs)	GRAPHIC LOG	USCS SYMBOL	SOIL DESCRIPTION	SAMPLE	BLOW COUNT SPT N VALUE	MOISTURE CONTENT (%)	GROUNDWATER	FIELD TESTING	TESTING AND LABORATORY DATA
		ML							
85		GM	Medium dense, red-brown, rounded GRAVEL with silt and sand; wet. End at 86 1/2 feet in medium dense native gravel	21	10-11-11	28			
90			Boring backfilled with bentonite chips.						
95									
100									

BORING METHOD: Mud Rotary	**ELEVATION REFERECE:**	**START CARD/TAG ID:**
BOREHOLE DIAMETER: 5 7/8	**GROUND SURFACE ELEVATION:**	**REMARKS:**
DRILL RIG: Mobile B-54	**CASING ELEVTATION:**	
CONTRACTOR: Holt Drilling Services	**LOCATION:** See Figure 2	
LOGGED BY: Tim North, PE GE	**DRILLING DATES:** 9/26/17 9/27/17	

GEO Consultants Northwest
824 SE 12th Avenue
Portland OR 97214
Tel 503-616-9425
Fax 1-866-293-9037

GEO CONSULTANTS NORTHWEST

LOG OF BORING
B-1

DEPTH (ft bgs)	GRAPHIC LOG	USCS SYMBOL	SOIL DESCRIPTION	SAMPLE	BLOW COUNT SPT N VALUE	MOISTURE CONTENT (%)	GROUNDWATER	FIELD TESTING	TESTING AND LABORATORY DATA
0		ACC	ASPHALT PAVEMENT - 18 inches thick						
		GM	Dense to medium dense, brown-gray, angular SILTY GRAVEL FILL with sand; moist to wet.	1	22-25-22	17			
5				2	7-9-7	23			
				3	7-17-18	19			
10		SM	Medium dense, gray, fine to medium SILTY SAND; moist.	4	11-9-5	16			
		SP	Loose, poorly graded SAND with trace organics (wood fragments); moist.	5	P				
15			Loss of circulation at 15 feet.	6	11-7-8	16			
		GP	Medium dense, gray, poorly graded GRAVEL with sand and silt; moist to wet.	7	5-10-11	26			
20				8					
			End at 21 1/2 feet in medium dense native gravel.						
			Boring backfilled with bentonite chips.						
25									
30									
35									
40									

BORING METHOD: Mud Rotary	**ELEVATION REFERECE:**	**START CARD/TAG ID:**
BOREHOLE DIAMETER: 5 7/8	**GROUND SURFACE ELEVATION:**	**REMARKS:**
DRILL RIG: Mobile B-54	**CASING ELEVTATION:**	
CONTRACTOR: Holt Drilling Services	**LOCATION:** See Figure 2	
LOGGED BY: Tim North, PE GE	**DRILLING DATES:** 9/27/17 9/27/17	

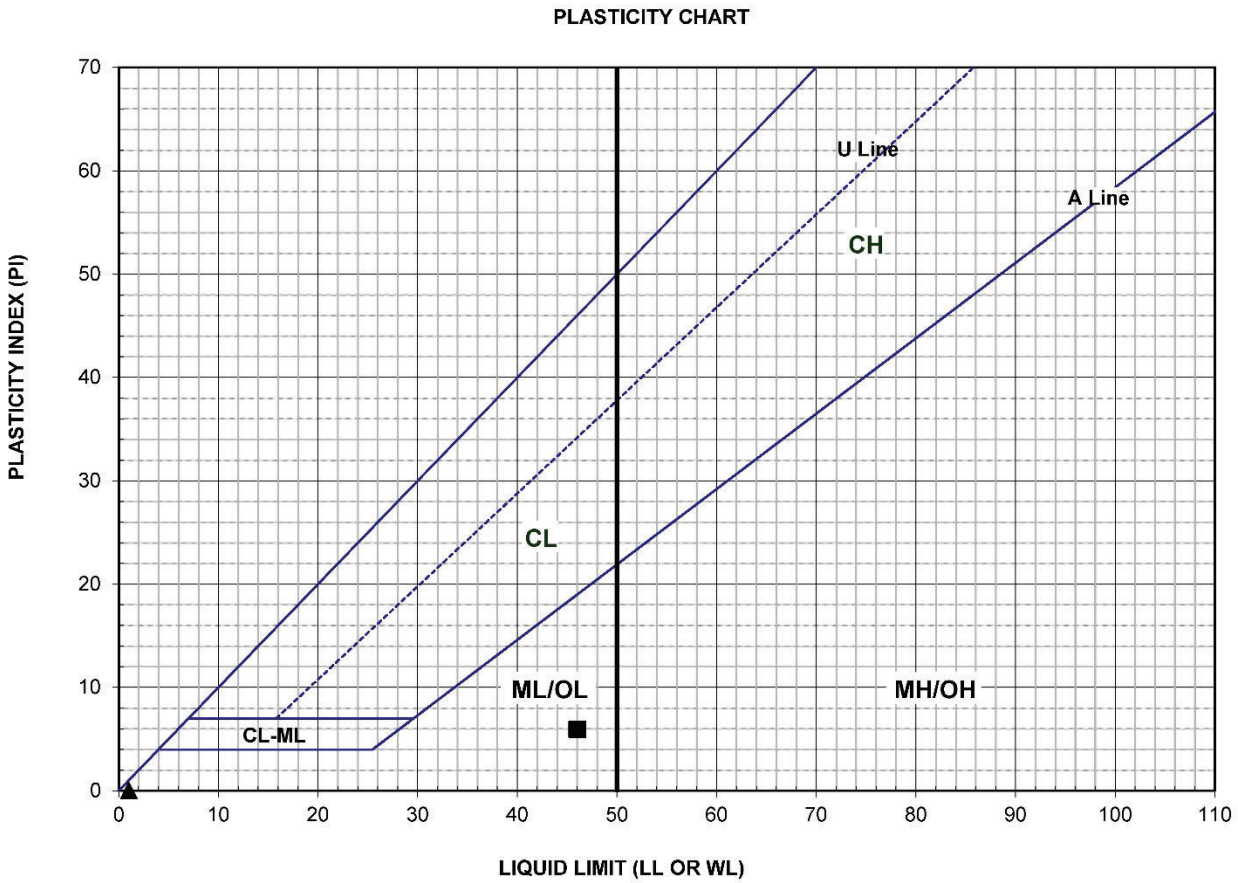
Tetra Tech - Hungry Harbor, WA

1239

GEO Consultants Northwest
824 SE 12th Avenue
Portland OR 97214
Tel 503-616-9425
Fax 1-866-293-9037

GEO CONSULTANTS NORTHWEST

LOG OF BORING
B-2



B/TP	DEPTH FT	USCS SYMBOL	SOIL DESCRIPTION	LL %	PL %	PI %	IN-SITU MOISTURE	% FINES	SYMBOL
B-1	50.0FT	ML	SILT	46	40	6	39	100	■

GEO CONSULTANTS NORTHWEST	PROJECT 1239	TETRA TECH HUNGRY HARBOR SR 401 UNDERCROSSING	
824 SE 12th Avenue Portland, OR 97214	JAN 2018 Drawn By: TJN	ATTERBERG LIMITS TEST RESULTS	FIGURE A1

ATTACHMENT B

LIQUEFACTION ANALYSIS

GEO COnsultants Northwest, Inc.
824 SE 12th Avenue
Portland, Oregon 97214
url: http://www.gcnweb.com

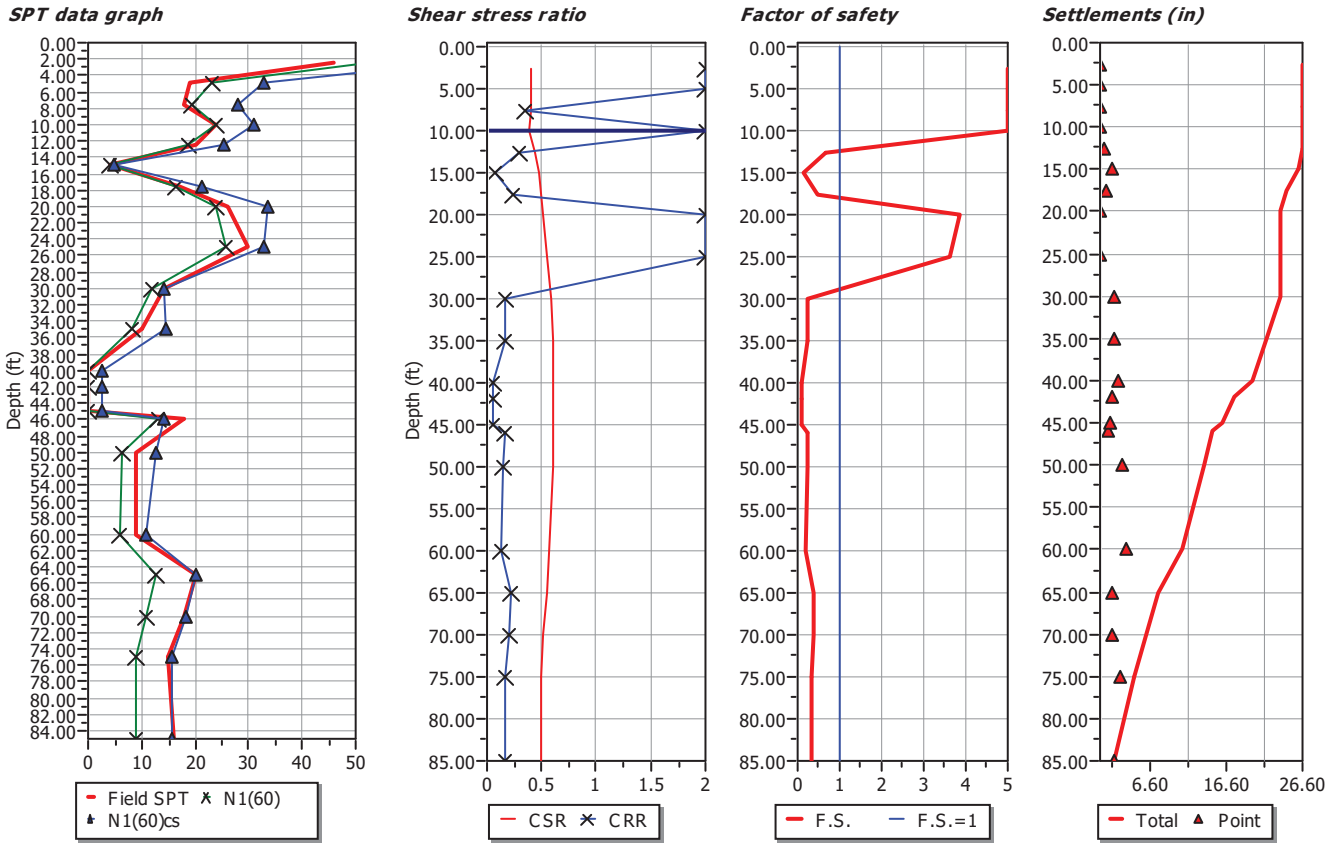
LIQUEFACTION ANALYSIS REPORT

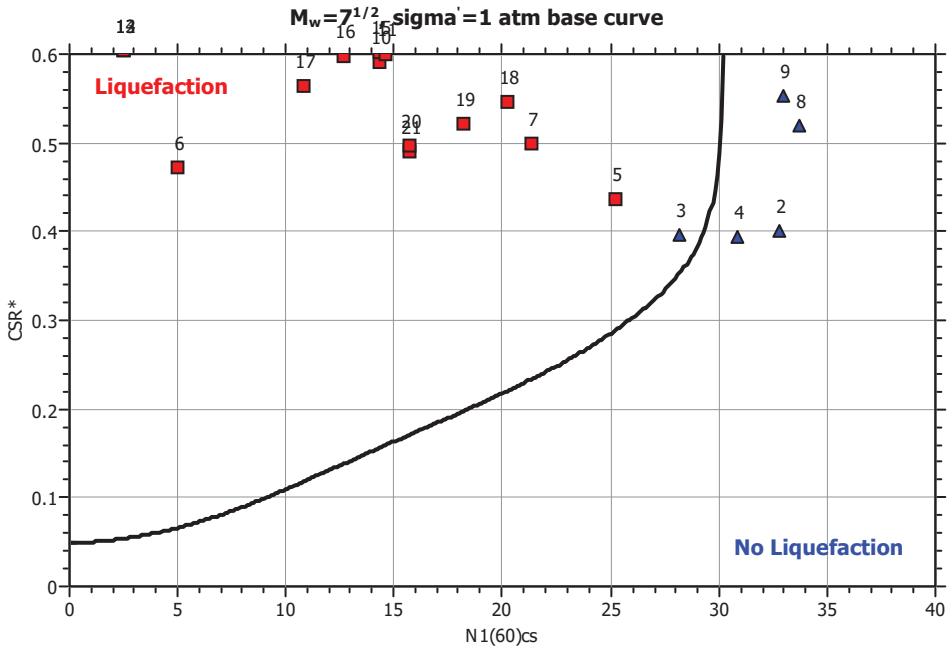
Project title : 1239 - Hungry Harbor

Project subtitle : B-1

Input parameters and analysis data

In-situ data type:	Standard Penetration Test	Depth to water table:	10.00 ft
Analysis type:	Deterministic	Earthquake magnitude M_w:	9.00
Analysis method:	NCEER 1998	Peak ground accelaration:	0.39 g
Fines correction method:	Idriss & Seed	User defined F.S.:	1.00





:: Field input data ::

Point ID	Depth (ft)	Field N_{SPT} (blows/feet)	Unit weight (pcf)	Fines content (%)
1	2.50	46.00	130.00	35.00
2	5.00	19.00	125.00	35.00
3	7.50	18.00	120.00	35.00
4	10.00	24.00	120.00	25.00
5	12.50	20.00	120.00	25.00
6	15.00	4.00	110.00	10.00
7	17.50	17.00	120.00	20.00
8	20.00	26.00	120.00	35.00
9	25.00	30.00	120.00	25.00
10	30.00	14.00	110.00	13.00
11	35.00	10.00	125.00	35.00
12	40.00	0.00	110.00	15.00
13	42.00	0.00	110.00	15.00
14	45.00	0.00	110.00	15.00
15	46.00	18.00	120.00	10.00
16	50.00	9.00	105.00	75.00
17	60.00	9.00	125.00	25.00
18	65.00	20.00	115.00	39.00
19	70.00	18.00	125.00	40.00
20	75.00	15.00	115.00	75.00
21	85.00	16.00	125.00	35.00

Depth : Depth from free surface, at which SPT was performed (ft)
Field SPT : SPT blows measured at field (blows/feet)
Unit weight : Bulk unit weight of soil at test depth (pcf)
Fines content : Percentage of fines in soil (%)

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Point ID	Depth (ft)	Sigma (tsf)	u (tsf)	Sigma' (tsf)	r_d	CSR	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*
1	2.50	0.16	0.00	0.16	0.99	0.25	0.63	0.40	1.00	0.40
2	5.00	0.32	0.00	0.32	0.99	0.25	0.63	0.40	1.00	0.40
3	7.50	0.47	0.00	0.47	0.98	0.25	0.63	0.40	1.00	0.40
4	10.00	0.62	0.00	0.62	0.98	0.25	0.63	0.39	1.00	0.39
5	12.50	0.77	0.08	0.69	0.97	0.27	0.63	0.44	1.00	0.44
6	15.00	0.91	0.16	0.75	0.97	0.30	0.63	0.47	1.00	0.47
7	17.50	1.06	0.23	0.82	0.96	0.31	0.63	0.50	1.00	0.50
8	20.00	1.21	0.31	0.89	0.95	0.33	0.63	0.52	1.00	0.52
9	25.00	1.51	0.47	1.04	0.94	0.35	0.63	0.55	1.00	0.55
10	30.00	1.78	0.62	1.16	0.93	0.36	0.63	0.58	0.98	0.59
11	35.00	2.09	0.78	1.31	0.89	0.36	0.63	0.57	0.96	0.60
12	40.00	2.37	0.94	1.43	0.85	0.36	0.63	0.57	0.94	0.61
13	42.00	2.48	1.00	1.48	0.83	0.35	0.63	0.56	0.93	0.61
14	45.00	2.64	1.09	1.55	0.81	0.35	0.63	0.56	0.92	0.60
15	46.00	2.70	1.12	1.58	0.80	0.35	0.63	0.55	0.92	0.60
16	50.00	2.91	1.25	1.66	0.77	0.34	0.63	0.54	0.91	0.60
17	60.00	3.54	1.56	1.98	0.69	0.31	0.63	0.50	0.88	0.56
18	65.00	3.83	1.72	2.11	0.65	0.30	0.63	0.47	0.87	0.55
19	70.00	4.14	1.87	2.27	0.61	0.28	0.63	0.45	0.86	0.52
20	75.00	4.43	2.03	2.40	0.56	0.26	0.63	0.42	0.85	0.50
21	85.00	5.05	2.34	2.71	0.54	0.25	0.63	0.40	0.83	0.49

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Point ID	Depth (ft)	Sigma (tsf)	u (tsf)	Sigma' (tsf)	r_d	CSR	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*

Depth : Depth from free surface, at which SPT was performed (ft)
Sigma : Total overburden pressure at test point, during earthquake (tsf)
u : Water pressure at test point, during earthquake (tsf)
Sigma' : Effective overburden pressure, during earthquake (tsf)
r_d : Nonlinear shear mass factor
CSR : Cyclic Stress Ratio
MSF : Magnitude Scaling Factor
$CSR_{eq,M=7.5}$ CSR adjusted for M=7.5
K_{sigma} Effective overburden stress factor
CSR^* CSR fully adjusted

:: Cyclic Resistance Ratio calculation $CRR_{7.5}$ ::

Point ID	Field SPT	C_n	C_e	C_b	C_r	C_s	$N_{1(60)}$	DeltaN	$N_{1(60)cs}$	$CRR_{7.5}$
1	46.00	1.70	0.90	1.00	0.75	1.00	52.78	15.38	68.16	2.00
2	19.00	1.70	0.90	1.00	0.80	1.00	23.26	9.56	32.82	2.00
3	18.00	1.49	0.90	1.00	0.80	1.00	19.34	8.79	28.13	0.35
4	24.00	1.30	0.90	1.00	0.85	1.00	23.85	7.03	30.88	2.00
5	20.00	1.23	0.90	1.00	0.85	1.00	18.81	6.45	25.26	0.29
6	4.00	1.18	0.90	1.00	0.95	1.00	4.04	0.96	4.99	0.07
7	17.00	1.13	0.90	1.00	0.95	1.00	16.38	4.92	21.30	0.23
8	26.00	1.08	0.90	1.00	0.95	1.00	24.02	9.71	33.74	2.00
9	30.00	1.00	0.90	1.00	0.95	1.00	25.73	7.25	32.97	2.00
10	14.00	0.95	0.90	1.00	1.00	1.00	11.97	2.33	14.30	0.16
11	10.00	0.89	0.90	1.00	1.00	1.00	8.03	6.56	14.58	0.16
12	0.00	0.85	0.90	1.00	1.00	1.00	0.00	2.50	2.50	0.05
13	0.00	0.84	0.90	1.00	1.00	1.00	0.00	2.50	2.50	0.05
14	0.00	0.82	0.90	1.00	1.00	1.00	0.00	2.50	2.50	0.05
15	18.00	0.81	0.90	1.00	1.00	1.00	13.17	1.15	14.32	0.16
16	9.00	0.79	0.90	1.00	1.00	1.00	6.41	6.28	12.70	0.14
17	9.00	0.73	0.90	1.00	1.00	1.00	5.89	4.97	10.85	0.12
18	20.00	0.70	0.90	1.00	1.00	1.00	12.67	7.53	20.20	0.22
19	18.00	0.68	0.90	1.00	1.00	1.00	11.00	7.20	18.20	0.20
20	15.00	0.66	0.90	1.00	1.00	1.00	8.91	6.78	15.69	0.17
21	16.00	0.62	0.90	1.00	1.00	1.00	8.94	6.74	15.68	0.17

C_n : Overburden corretion factor
C_e : Energy correction factor
C_b : Borehole diameter correction factor
C_r : Rod length correction factor
C_s : Liner correction factor
$N_{1(60)}$: Corrected N_{SPT}
DeltaN : Addition to corrected N_{SPT} value due to the presence of fines
$N_{1(60)cs}$: Corected $N_{1(60)}$ value for fines
$CRR_{7.5)}$: Cyclic resistance ratio for M=7.5

:: Settlements calculation for saturated sands ::

Point ID	$N_{1(60)}$	N_1	FS_L	e_v (%)	Settle. (in)
1	68.16	56.80	5.00	0.00	0.00
2	32.82	27.35	5.00	0.00	0.00
3	28.13	23.44	5.00	0.00	0.00
4	30.88	25.74	5.00	0.00	0.00
5	25.26	21.05	0.66	2.06	0.62
6	4.99	4.16	0.14	5.07	1.52
7	21.30	17.75	0.47	2.39	0.72
8	33.74	28.11	3.84	0.00	0.00
9	32.97	27.48	3.62	0.00	0.00
10	14.30	11.92	0.26	3.16	1.90
11	14.58	12.15	0.26	3.12	1.87

:: Settlements calculation for saturated sands ::

Point ID	$N_{1(60)}$	N_1	FS_L	e_v (%)	Settle. (in)
12	2.50	2.08	0.09	5.50	2.31
13	2.50	2.08	0.09	5.50	1.65
14	2.50	2.08	0.09	5.50	1.32
15	14.32	11.94	0.26	3.16	0.95
16	12.70	10.58	0.23	3.40	2.85
17	10.85	9.04	0.21	3.72	3.34
18	20.20	16.83	0.40	2.49	1.50
19	18.20	15.17	0.38	2.67	1.60
20	15.69	13.08	0.34	2.96	2.67
21	15.68	13.07	0.35	2.96	1.78

Total settlement : 26.60

$N_{1,(60)}$: Stress normalized and corrected SPT blow count
N_1: Japanese equivalent corrected value
FS_L: Calculated factor of safety
e_v: Post-liquefaction volumentric strain (%)
Settle.: Calculated settlement (in)

:: Liquefaction potential according to Iwasaki ::

Point ID	F	w_z	I_L
1	0.00	9.62	0.00
2	0.00	9.24	0.00
3	0.00	8.86	0.00
4	0.00	8.48	0.00
5	0.34	8.10	2.08
6	0.86	7.71	5.06
7	0.53	7.33	2.98
8	0.00	6.95	0.00
9	0.00	6.19	0.00
10	0.74	5.43	6.09
11	0.74	4.67	5.23
12	0.91	3.90	5.42
13	0.91	3.60	2.00
14	0.91	3.14	2.62
15	0.74	2.99	0.67
16	0.77	2.38	2.23
17	0.79	0.86	2.06
18	0.60	0.09	0.09

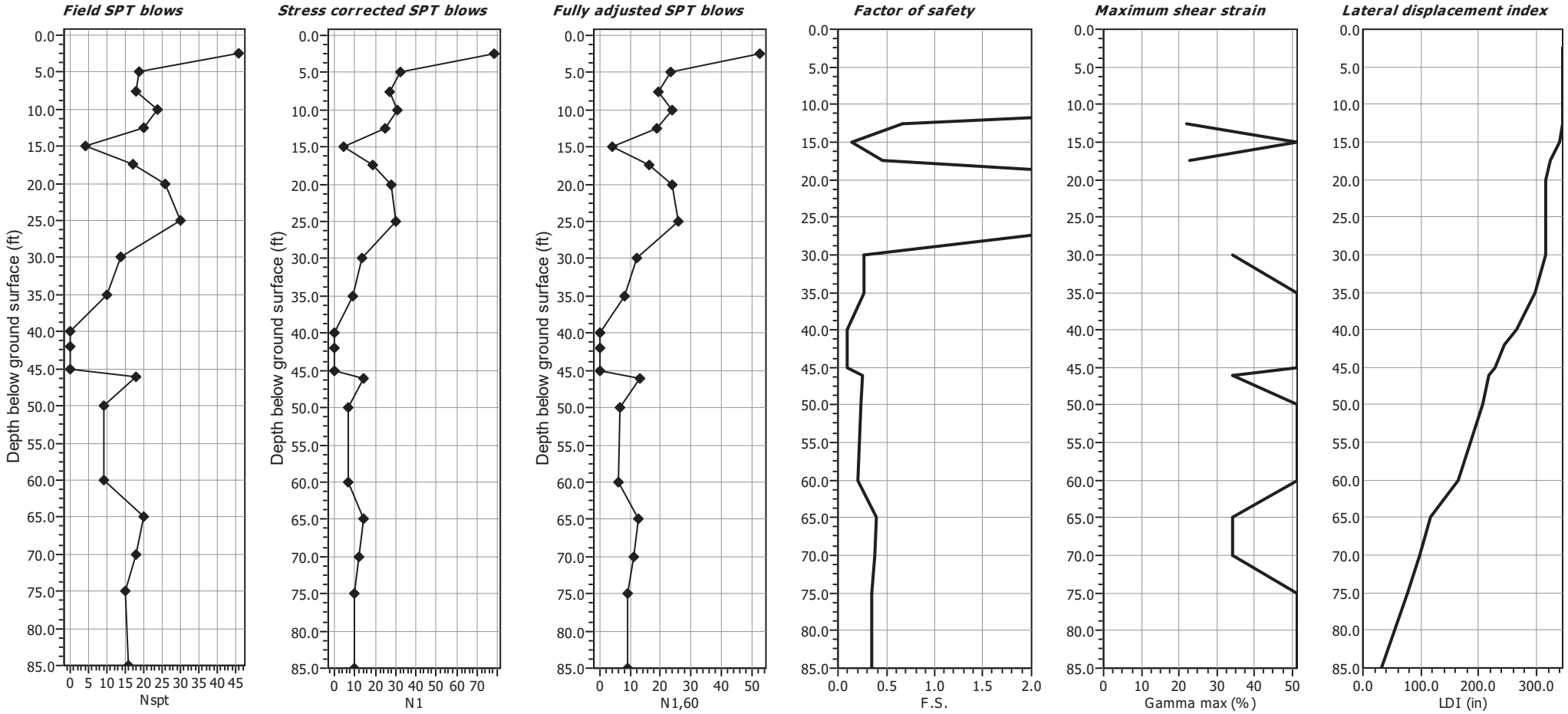
Overall potential I_L : 36.51

I_L = 0.00 - No liquefaction
I_L between 0.00 and 5 - Liquefaction not probable
I_L between 5 and 15 - Liquefaction probable
I_L > 15 - Liquefaction certain

GEO COnsultants Northwest, Inc.
824 SE 12th Avenue
Portland, Oregon 97214
url: http://www.gcnweb.com

LATERAL DISPLACEMENTS ESTIMATION DUE TO SOIL LIQUEFACTION[1]

Geometric parameters: Gently sloping ground without free face
Total lateral displacement estimation: 242.32 in



N_{spt}: Measured SPT blows
N_1: Stress adjusted SPT blows
$N_{1,60}$: Fully adjusted SPT blows

F.S.: Factor of safety
Gamma max: Maximum cyclic shear strain
LDI: Lateral displacement index

[1] This method was developed using the NCEER methods (SPT and CPT) and other methods will produce slightly different results

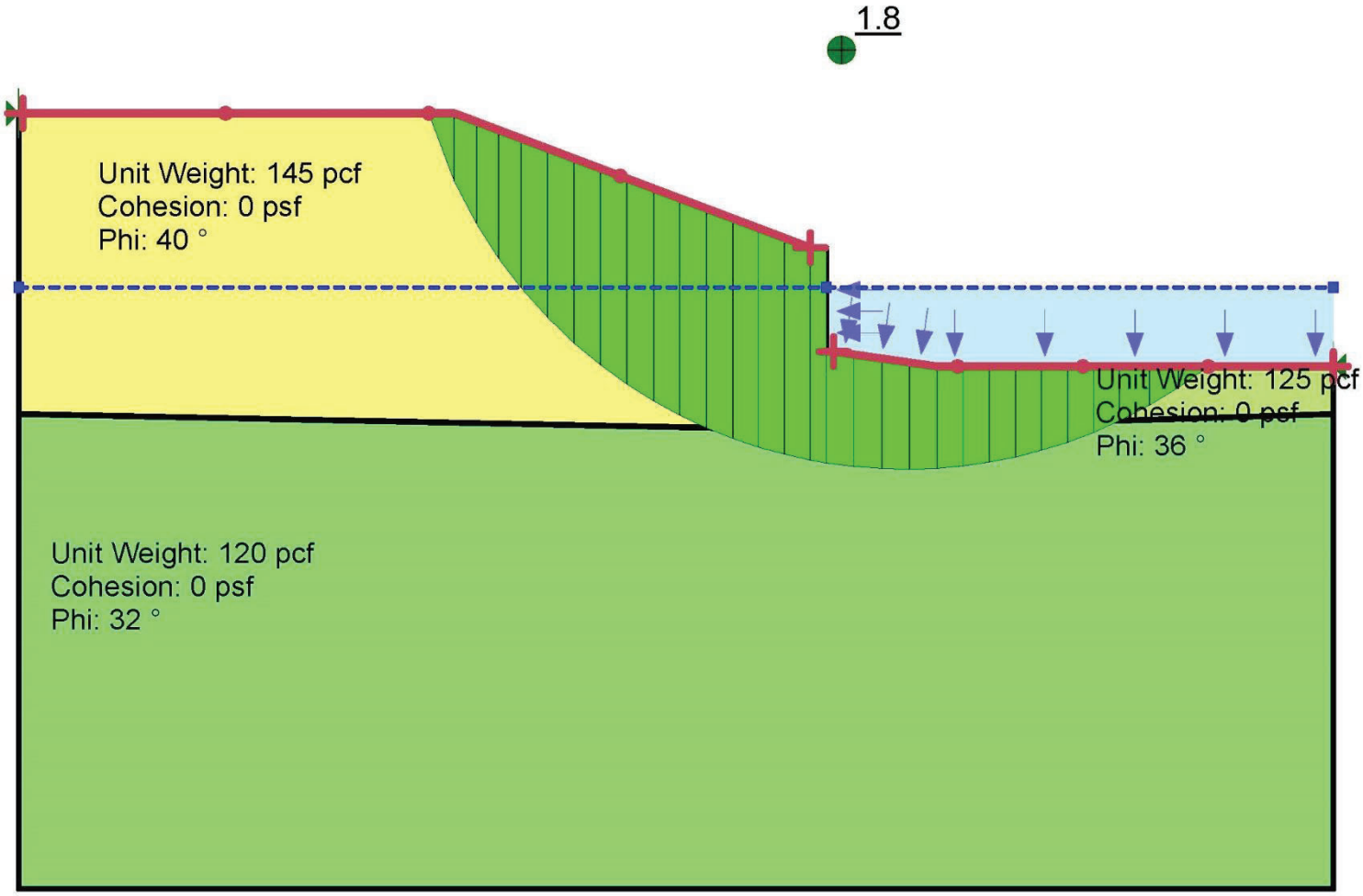
:: Lateral displacement index calculation ::

Point ID	N_{SPT}	N_1	$N_{1,60}$	F.S.	D_r	$Gamma_{max}$ (%)	LDI (in)
1	46.00	41.40	52.78	5.00	100.00	0.00	0.00
2	19.00	17.10	23.26	5.00	67.51	0.00	0.00
3	18.00	16.20	19.34	5.00	61.57	0.00	0.00
4	24.00	21.60	23.85	5.00	68.37	0.00	0.00
5	20.00	18.00	18.81	0.66	60.72	22.10	6.63
6	4.00	3.60	4.04	0.14	28.12	51.20	15.36
7	17.00	15.30	16.38	0.47	56.66	22.70	6.81
8	26.00	23.40	24.02	3.84	68.62	0.00	0.00
9	30.00	27.00	25.73	3.62	71.01	0.00	0.00
10	14.00	12.60	11.97	0.26	48.44	34.10	20.46
11	10.00	9.00	8.03	0.26	39.66	51.20	30.72
12	0.00	0.00	0.00	0.09	0.00	51.20	21.50
13	0.00	0.00	0.00	0.09	0.00	51.20	15.36
14	0.00	0.00	0.00	0.09	0.00	51.20	12.29
15	18.00	16.20	13.17	0.26	50.81	34.10	10.23
16	9.00	8.10	6.41	0.23	35.46	51.20	43.01
17	9.00	8.10	5.89	0.21	33.96	51.20	46.08
18	20.00	18.00	12.67	0.40	49.82	34.10	20.46
19	18.00	16.20	11.00	0.38	46.43	34.10	20.46
20	15.00	13.50	8.91	0.34	41.79	51.20	46.08
21	16.00	14.40	8.94	0.35	41.86	51.20	30.72

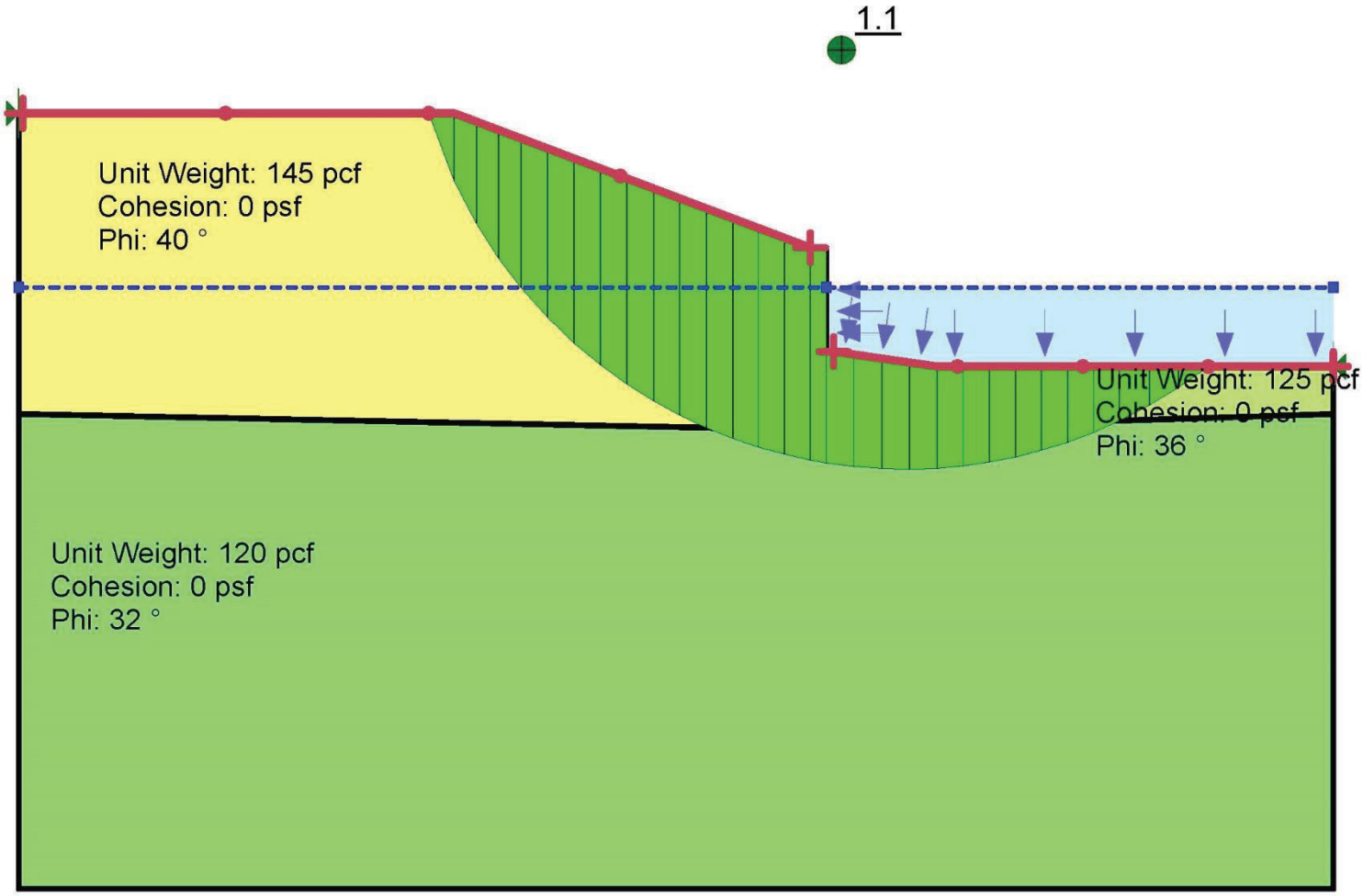
N_{SPT} : Measured SPT blows
N_1 : Adjusted SPT blows to an effective overburden stress of 100 kPa
$N_{1,60}$: Fully adjusted SPT blows
F.S. : Calculated factor of safety against liquefaction
D_r : Calculated relative density
$Gamma_{max}$: Calculated maximum cyclic shear strain
LDI : Lateral displacement index

ATTACHMENT C

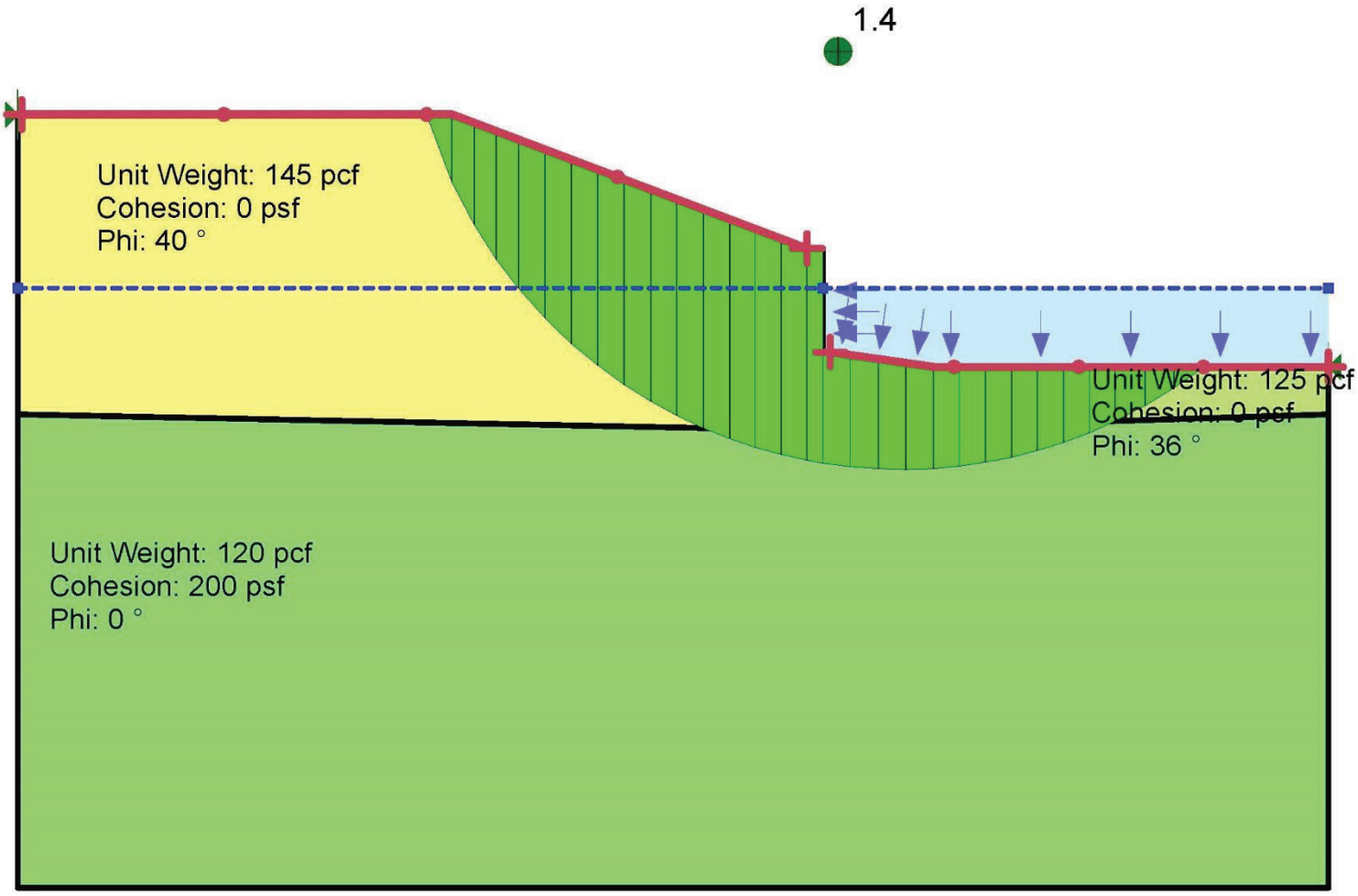
SLOPE STABILITY ANALYSIS



GEO CONSULTANTS NORTHWEST	PROJECT 1239	TETRA TECH HUNGRY HARBOR SR 401 UNDERCROSSING	
824 SE 12th Avenue Portland, OR 97214	MAY 2018 Drawn By: BWG	STATIC SLOPE STABILITY	FIGURE C1



GEO CONSULTANTS NORTHWEST	PROJECT 1239	TETRA TECH HUNGRY HARBOR SR 401 UNDERCROSSING	
824 SE 12th Avenue Portland, OR 97214	MAY 2018 Drawn By: BWG	SEISMIC (PSEUDO-STATIC) SLOPE STABILITY	FIGURE C2



GEO CONSULTANTS NORTHWEST	PROJECT 1239	TETRA TECH HUNGRY HARBOR SR 401 UNDERCROSSING	
824 SE 12th Avenue Portland, OR 97214	MAY 2018 Drawn By: BWG	POST-SEISMIC (RESIDUAL STRENGTH) SLOPE STABILITY	FIGURE C3