Geotechnical Engineering and Ground Improvement Design Services Report

Waterfront Place Central PGL Site
Everett, Washington

for
Port of Everett
c/o PND Engineers, Inc.

October 15, 2021

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Waterfront Place Central PGL Site
Everett, Washington

File No. 0676-030-00

October 15, 2021

Prepared for:

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1.0 INTRODUCTION

This report presents the results of GeoEngineers Inc.'s (GeoEngineers') geotechnical engineering and rigid inclusion ground improvement design services for the proposed development at the PGL site as part of Port of Everett Waterfront Place Center in Everett, Washington (Vicinity Map, Figure 1).

We understand that the proposed development at the PGL site includes the design and construction of two one-story retail buildings located at the corner of 18th Street and the West Marine View Drive (State Route 529). The north boundary of the project site is approximately 80 feet south to the bulkhead at the South Marina, next to the Inn at Port Gardner; and the retail building footprints are approximately 200 feet south to the bulkhead. An existing retail store building is on the west side of the project site, as shown in the Site Plan, Figure 2. Based on our communication with the Port and PND Engineers, Inc. (PND, project structural engineer), and review of the available engineering drawings within the site vicinity, we understand that the bulkhead north of the project site is basically a rock dike built in the mid-1970s, with marine sediment fill behind the dike, and a concrete retaining wall, about 3 feet high, is sitting on top of the rock dike.

Based on the review of available geotechnical information within the site vicinity and our previous project experience at the Port of Everett, lateral spreading and ground settlements caused by soil liquefaction may have significant impacts on the planned foundations of the proposed two retail buildings at the PGL site. To support the new construction of the two retail buildings, the ground improvement with rigid inclusion is considered as the foundation support for this project.

The objectives of GeoEngineers' geotechnical engineering and rigid inclusion ground improvement services are to provide PND geotechnical seismic design criteria, foundation recommendations in support of the building design and construction at the site, and develop a ground improvement design consisting of rigid inclusion elements to mitigate the ground deformations (e.g., lateral spreading and vertical settlements) of the building foundations for the proposed new buildings to meet the building code criteria.

2.0 SITE CONDITIONS

The site conditions were evaluated by reviewing the available geotechnical, geological, groundwater within the site vicinity, and the design and construction information for the existing rock dike at the South Marina corner. The related documents that we reviewed included the following:

- "Report Soils Investigation for Proposed Norton Avenue Terminal, Everett, Washington," prepared for Reid Middleton, by Roger Lowe Associates Inc., dated November 30, 1977.
- "Augercast Pile Capacities for Pioneer Marina Plaza, Everett, Washington," prepared for Commercial Design Associates, by Earth Consultants Inc., dated March 31, 1988.
- "Preliminary Environmental Site Assessment for Pioneer Marine Plaza, West Marine View Drive, Everett, Washington," prepared for First Western Development, by Earth Consultants Inc., dated November 30, 1989.
- "Public Access and Transient Moorage Drawings," prepared for Port of Everett, by Reid Middleton, dated August 28, 1997.
- "Geotechnical Engineering Services Report for Proposed South Marina Improvements, Port of Everett, Everett, Washington," prepared for Moffat & Nichol, by Landau Associates, dated February 12, 2007.

- “Port of Everett Waterfront Place Central PGL Site 30% Design Drawings,” prepared for Port of Everett, by PND Engineers, Inc., dated September 1, 2021.

As shown in Figure 2, there is no existing geotechnical information (e.g., standard penetration tests [SPT], cone penetration tests [CPT], or test pits) available within the PGL building footprint; therefore, a geophysical survey was performed at the project site to correlate with the existing geotechnical information near the building footprint to evaluate the subsurface conditions within the building footprint.

2.1. Bulkhead Conditions

The PGL site is located at the corner of 18th Street and the West Marine View Drive (State Route 529). The north boundary of the project site is approximately 80 feet south to the bulkhead at the South Marina, next to the Inn at Port Gardner; and the proposed retail building footprints are approximately 200 feet south of the bulkhead. An existing retail store building is on the west side of the project site.

Based on our communication with the Port and PND, we understand that the bulkhead north of the project site is basically a rock dike built in the mid-1970s, with marine sediment fill behind the dike, and a concrete retaining wall, about 3 feet high, is sitting on top of the rock dike. Based on the review of available engineering drawings, nearby geotechnical information (e.g., boring logs), and survey drawings within the site vicinity, we assumed a constant existing ground surface across the project site at about Elevation +18.00 feet. The slope in front of the dike was assumed to slope down at an approximate 1.5H:1V (horizontal to vertical) slope from Elevation +16.00 feet to Elevation +1.00 feet, and an approximate 5.5H:1V slope from Elevation +1.0 feet to Elevation -12.00 feet. The elevation of -12.00 feet was the dredge elevation in front of the bulkhead from the nearby project. Note that the project vertical datum is Mean Lower Low Water (MLLW).

2.2. Subsurface Conditions

Based on our review of the nearby subsurface information, the soils at the project site generally consist of hydraulic fill, recent deposits, and glacially consolidated soils.

Hydraulic fill generally consists of soft sandy silt (ML) and loose to medium dense silty sand (SM) containing wood debris, organics, and shell fragments at some locations.

Recent deposits were encountered below the hydraulic fill and generally consist of soft to medium stiff sandy silt and loose silty sand containing wood debris, organics, and shell fragments at some locations. Some medium dense sands (SP) were observed at B-1 and P-3 under the above soft to medium stiff sandy silty and loose silty sand layer, which is also part of the recent deposits.

Glacially consolidated soils were encountered below the recent deposits and generally consist of dense to very dense sand, extended to the depths explored. Glacially consolidated soils are competent bearing soils.

The locations of the test pits, SPT, and CPT borings that were reviewed and used in our analyses are shown in Figure 2. The corresponding test pits, SPT, and CPT logs are presented in Appendix A.

2.2.1. Geophysical Survey

As shown in Figure 2, the existing test pits, SPT, and CPT borings are all located outside of the proposed building footprint. Therefore, a geophysical survey was performed at the project site and also at some nearby locations that overlay with some previous borings (e.g., B-1 and B-4), so that correlations between the geophysical survey results and the nearby geotechnical information can be done to characterize the subsurface conditions at the site, including site class and the depth of the glacially consolidated soils.

The geophysical survey was completed by conducting multichannel analysis of surface waves (MASW) along five transects (Line 1 through Line 5) and one microtremor array measurements (MAM) across the project site (shown as blue stars), as shown in the Geophysical Survey Arrays, Figure 3. Through the MASW, five two-dimensional (2D) shear wave velocity (Vs) profiles were developed along the five transects, going down to about 70 feet below ground surface (bgs); and a deeper one-dimensional (1D) Vs profile (about 300 feet deep) was developed through the MAM. After calibrated with the nearby boring information (B-1 and B-4), the geophysical survey indicates that the depth of the glacially consolidated soils ranges from 27.5 feet to 35.0 feet bgs across the building footprint. The correlations between the geophysical survey results and the nearby geotechnical information are used in our geotechnical analyses completed as the basis of the geotechnical recommendations.

The locations of the survey arrays are shown in Figure 3 and the geophysical survey report is presented in Appendix B.

2.3. Groundwater Conditions

Based on the reviewed information, the design groundwater level was assumed at Elevation 0.00 feet on the water side in front of the bulkhead and at Elevation +11.11 feet on the land side behind the bulkhead.

3.0 DESIGN CROSS SECTION

Cross Section A-A’, the location of which is shown in Figure 2, was selected for use in our geotechnical analyses. The soil delineation for Cross Section A-A’ is presented in Figure 4. Due to the lack of survey and subsurface exploration information on the water side, based on our communication with the Port and PND, and review of the previous engineering drawings for the nearby sites, we assumed a riprap rock slope in front of the dike with a 1.5H:1V slope from Elevation +16.00 feet to Elevation +1.00 feet, and with a 5.5H:1V slope from Elevation +1.00 feet to Elevation -12.00 feet. The soil profile behind the dike was horizontally extended to the water side.

4.0 EARTHQUAKE ENGINEERING

4.1. ASCE 7-16 Seismic Design Information

The project site is classified as a Site Class F due to the presence of the liquefiable soils below the proposed buildings. Based on our discussions with PND, it is our understanding that the building vibration period will be less than 0.5 seconds. Consequently, the project can be completed using the code-based seismic design per American Society of Civil Engineers (ASCE) 7-16; and a site-specific response analysis is not required.

Based on the geophysical survey completed at the site, the project site is best characterized as a Site Class D per ASCE 7-16. We recommend the use of the following ASCE 7-16 mapped parameters for Site Class, short-period mapped MCE_R spectral response acceleration (S_S), 1-second period mapped MCE_R spectral response acceleration (S_1), and seismic coefficients (F_A and F_V) for the project site (Table 1).

TABLE 1. ASCE 7-16 MAPPED SEISMIC DESIGN PARAMETERS

ASCE 7-16 Parameter	Recommended Value
Site Class	F
Short-period mapped MCE_R spectral response acceleration, S_S (g)	1.229[1]
Long-period mapped MCE_R spectral response acceleration, S_1 (g)	0.437[1]
Short-period site coefficient, F_A	1.01
Long-period site coefficient, F_V	1.86

Notes:

[1] The project site is classified as a Site Class F. But S_{ds} and S_{d1} are based on Site Class D per the code exception for building vibration period less than 0.5 seconds.

4.2. Liquefaction Potential

Liquefaction is a phenomenon where soils experience a rapid loss of internal strength as a consequence of strong ground shaking. Ground settlement, lateral spreading and sand boils may result from liquefaction. In general, the soil that is susceptible to liquefaction include very loose to medium dense, clean to silty sands and some silts that are below the groundwater level.

The structures supported on liquefied soils could suffer foundation settlement, downdrag loads, or lateral movement that could be severely damaging to the structures. The evaluation of liquefaction potential is complex and dependent on numerous parameters, including soil type, grain distribution, soil density, depth to groundwater, in-situ static ground stresses, earthquake-induced ground stresses and excess pore water pressure generated during seismic shaking.

We evaluated the liquefaction potential of the site soils at the maximum-considered earthquake (MCE) level based on the previous SPTs (B-1, B-2, B-4, and B-101 through B-103). Table 2 below presents the seismic input parameters used in our liquefaction analyses, where the magnitude is the maximum magnitude among the mean and mode magnitudes from the deaggregation performed at the peak ground acceleration (PGA) using the 2014 United States Geological Survey (USGS) seismic source characterization (SSC) model; and the PGA was derived from ASCE 7-16 for Site Class D.

TABLE 2. SEISMIC INPUT PRAMETERS FOR LIQUEFACTION EVALUATION

Design Earthquake	Mw	PGA (g)
MCE	7.22	0.580

The simplified triggering criteria proposed by Idriss and Boulanger (2008) and Youd et al. (2001) were used to evaluate the liquefaction potential of the site soils. An SPT hammer efficiency of 60 percent was assumed due to the lack of information on hammer type. Based on our simplified liquefaction analyses, the hydraulic fill and recent deposits are susceptible to liquefaction. Therefore, the depth of liquefaction is along the top of the glacially consolidated soils, as shown in Figure 4.

Liquefaction-induced free-field ground settlement of the potentially liquefiable zones was estimated using the semi-empirical approaches proposed by Tokimatsu and Seed (1987), Ishihara and Yoshimine (1992), and Idriss and Boulanger (2008) at the MCE level on B-1, B-2, and B-4 since these three borings went down to the glacially consolidate soils while the other three borings (B-101 through B-103) stopped at a shallower depth. Our analyses indicate that the site (under existing conditions) could experience liquefaction-induced free-field ground settlement on the order of 16 to 24 inches during an MCE seismic event, as shown in the Liquefaction-Induced Free-Field Ground Settlement, Figure 5. The differential liquefaction-induced settlements are anticipated to be on the order of 0.3 to 3.2 inches over a horizontal distance of 50 feet.

4.3. Lateral Spreading and Kinematic Loads

Lateral spreading involves lateral displacements of large volumes of liquefied soil. Lateral spreading can occur on near-level ground as blocks of surface soils are displaced relative to adjacent blocks. Lateral spreading can also occur as blocks of surface soils are displaced towards a nearby slope or free-face by movement of underlying liquefied soil. In the case of this project site, lateral spreading could occur during earthquakes resulting in the excessive movement of the buildings.

4.3.1. Earthquake-Induced Lateral Ground Deformations

Earthquake-induced lateral ground deformations were evaluated by performing slope stability analyses and simplified Newmark analyses for the Cascadia Subduction Zone (CSZ) intraslab source since the intraslab source has the highest percent contribution per deaggregation.

4.3.1.1. Slope Stability Analysis

Slope stability analyses were completed on Cross Section A-A' using Limit Equilibrium Method (LEM) with a commercial software, Slope/W, developed by GEO-SLOPE International, Ltd.

The slope stability was evaluated under static, seismic, and post-earthquake conditions.

4.3.1.2. Soil Properties

The soil properties that were used in the slope stability analyses are listed in Table 3, where static strengths were used in static condition for all soil units. We assumed that liquefaction occurs during the earthquake; therefore, in seismic and post-earthquake conditions, residual friction angles were used in the liquefied soils (below groundwater table and above liquefaction depth); 80 percent of static strengths were used in the soils above groundwater table; and full static strengths were used in the soils (non-liquefiable) below liquefaction depth.

TABLE 3. SOIL PROPERTIES IN SLOPE/W ANALYSIS

Soil Unit	Unit Weigth (pcf)	Friction Angle (deg)	Residual Friction Angle (deg)
Hydraulic Fill (Soft ML/Loose to Medium Dense SM)[1]	120	28	3
Recent Deposits (Soft to Medium Stiff ML/Loose SM)[1]	125	28	5
Recent Deposits (Medium Dense SP)[1]	125	32	9
Glacially Consolidated Soils (Dense to Very Dense SP)[1]	130	39	-
Riprap Rock[2]	125	45	-

Notes:

[1] The soil properties for hydraulic fill, recent deposits, and glacially consolidated soils were estimated based on the correlations on SPTs.

[2] The soil properties for rip-rap rock were assumed.

pcf = pound per cubic foot; deg = degree

4.3.1.3. Surcharge Loads

Based on the collaboration with PND, the surcharge load within the building footprint is 190 pound per square foot (psf) in static condition and 115 psf in seismic and post-earthquake conditions.

4.3.1.4. Slope Stability Results

Figures 6 through 8 present the slope stability analysis results for Cross Section A-A'. In static condition, the factor of safety (FOS) was evaluated generally over 3.0 for the slip surfaces going underneath the potential building footprints (Figure 6), which indicates a stable condition.

In post-earthquake condition, most of the slip surfaces going underneath the potential building footprints have the FOS over 1.1 (Figure 7); while a small portion near the north edge of the building footprints that has slip surfaces with FOS around 1.0 may experience slope failure with excessive slope movement.

In seismic condition (Figure 8), with a failure wedge starting from about one-third of the potential building footprints with a FOS of 1.0, the yield acceleration was estimated as 0.01g. The corresponding earthquake-induced lateral ground deformation was estimated at about 53 inches using the simplified displacement approach developed by Bray and Travasarou (2007) and Bray et al. (2018), which exceeds the allowable lateral displacement criterion of 18 inches per Table 12.13.2 of ASCE 7-16. Ground improvement is designed to reduce the lateral displacement to meet the allowable lateral displacement criterion per ASCE 7-16. Please note that the ground improvement does not mitigate the effects of liquefaction to the existing bulkhead, which is not part of our current scope of work.

5.0 GROUND IMPROVEMENT RECOMMENDATIONS

Based on the collaboration with PND, we understand that due to the potential liquefaction issue at the site, ground improvement with rigid inclusion is selected as the foundation support for the buildings at PGL site to mitigate the ground deformations, that include static (long-term consolidation) and seismic (liquefaction-induced) settlements, earthquake-induced lateral ground deformation, etc., and prevent slope failure from occurring within the potential building footprints.

5.1. Rigid Inclusions

Rigid inclusions are generally unreinforced, grouted, or concrete columns installed in weak foundation soils to improve the overall stiffness and strength so that the foundation soils can meet the settlement criteria and achieve the bearing capacity required for support of shallow foundations of a structure.

Rigid inclusion elements are formed by constructing unreinforced concrete columns using a continuous-flight, hollow-stem auger attached to a set of leads supported by a crane or installed with a fixed-mast drill rig. The first step in the rigid inclusion placement process consists of drilling the auger into the ground to the specified tip elevation of the rigid inclusion. Concrete is then pumped through the hollow-stem during steady withdrawal of the auger, replacing the soils on the flights of the auger. One benefit of using the augercast method for rigid inclusion installation is that the auger provides support for the soils during the installation process, thus eliminating the need for temporary casing or drilling fluid.

5.2. Rigid Inclusions Design

In this project, the rigid inclusions are designed to mitigate the ground deformations and the risk of slope failure within the potential building footprints due to earthquake by stiffening the liquefiable fill and recent deposits so that the foundation design of the buildings meets the performance criteria for bearing pressure, static and seismic settlements, and lateral spreading.

The design bearing capacity of the improved foundation soils was estimated based on the axial capacity of each individual rigid inclusion column and the load transfer between the columns and the surrounding soils. Design analyses were completed to estimate the total amount (i.e., number and depth) of the rigid inclusion columns that are required under the foundation; and therefore, the static and seismic settlements meet the performance criteria.

The post-improvement earthquake-induced lateral ground deformation was estimated by performing global slope stability and Newmark analyses that include the improved area with an increased overall strength.

5.2.1. Rigid Inclusion Layout

A preliminary rigid inclusion design layout (dated October 7, 2021) was developed for the two buildings at the PGL site based on the foundation plan provided by PND on September 30, 2021. The preliminary rigid inclusion design layout will be updated based on the potential updates in the foundation and utility plans.

The preliminary rigid inclusion layout is summarized below. And the detail ground improvement plans, and construction specifications will be included in separate design drawing submittals for PGL site.

- Two-foot-diameter rigid inclusions.
- Typical center-to-center spacing is 5 feet 4 inches and 7 feet in a triangular pattern below the building foundations, and 10 feet 6 inches in a square pattern under the slab. Refer to the preliminary rigid inclusion design layout for additional details.
- Rigid inclusions should extend a minimum of 3 feet into the non-liquefiable layer. According to the geophysical survey results (Section 2.2.1) that indicate a depth of glacially consolidated soils ranging from 27.5 feet to 35.0 feet bgs, by including an extra 2 feet to consider the potential variability in soil condition, the tip elevations are anticipated approximately at -22.0 feet.

- The concrete mixture for the rigid inclusions should have a minimum compressive strength (f_c') at 28 days of 3,000 pounds per square inch (psi). Refer to ground improvement specifications that will be included in separate design drawing submittals for PGL site for additional details.
- Rigid inclusions should be overlain by a 2-foot-thick layer of crushed surfacing base course (CSBC) to act as a load transfer pad (LTP) between the foundation elements and the rigid inclusions (compacted in accordance with the geotechnical engineer's recommendations).
- The CSBC should comply with Washington State Department of Transportation (WSDOT) Standard Specification Section 9-03.9(3), Base Course, and restrict fines to maximum 5 percent passing Number 200 sieve.

5.2.2. Post-Improvement Recommendations

5.2.2.1. Design Bearing Capacity

We recommend an allowable bearing capacity of 3,000 psf with a FOS of at least 3.0 under static conditions and a modulus of subgrade reaction of 200 pounds per cubic inch (pci) be used for the foundations bearing on the improved ground.

5.2.2.2. Static and Seismic Settlements

We anticipated that the static settlement will be less than 1 inch with the implementation of rigid inclusion ground improvement, and the differential static settlements between building foundations should be less than ½ inch over a horizontal distance of 50 feet.

The seismic settlements are estimated based on the settlement reduction ratio determined based on the area replacement ratio of the rigid inclusions and the stiffness ratio between the soils and rigid inclusions. The post-improved seismic soil settlements are anticipated to be less than 6.0 inches; and the differential seismic settlements are anticipated to be on the order of 0.1 to 0.8 inches over a horizontal distance of 50 feet.

5.2.2.3. Earthquake-Induced Lateral Ground Deformation

The post-improvement earthquake-induced lateral ground deformation was estimated by performing global slope stability and Newmark analyses that include the improved area with an increased overall strength.

Based on the preliminary rigid inclusion design layout, a conservative area replacement ratio of 3 percent was used in the post-improvement global slope stability evaluation. With an area replacement ratio of 3 percent and a compressive strength of 3,000 psi, the increased composite strength of the improved ground was estimated as 6.5 kilopound per square foot (ksf) for the improved area. The improved area was conservatively applied to the north building only.

As shown in the Post-Improvement Slope Stability - Post-Earthquake Condition, Figure 9, the FOS corresponding to the slip surfaces going underneath the potential building footprint is higher than 1.9, which indicates a stable post-earthquake condition after ground improvement. With a stable post-earthquake condition, the post-improvement earthquake-induced lateral spreading is mainly due to the deformations happening during earthquakes.

As shown in the Post-Improvement Slope Stability - Seismic Condition, Figure 10, with a failure wedge going underneath the potential building footprint with a FOS of about 1.0, the yield acceleration was estimated as 0.25g. The corresponding earthquake-induced lateral ground deformation was estimated at about 3.1 inches, which is within the allowable lateral displacement criterion of 18 inches per ASCE 7-16.

5.3. Construction Considerations

Rigid inclusion installation should be completed in general accordance with the best practices per the industry as detailed in the specifications that will be included in a separate design drawing submittal for the PGL site. The anticipated construction sequencing is presented below:

- Excavate to the bottom of building foundation elevation as appropriate.
- Install rigid inclusions according to plans and specifications to the depth required to achieve 3-foot embedment into the non-liquefiable layer. Top of the rigid inclusions should be 2 feet below the bottom of foundations.
- Overexcavate by 2 feet and replace with CSBC.

The rigid inclusions should be installed using a continuous-flight hollow-stem auger. As is standard practice, the rigid inclusion concrete must be pumped under pressure through the hollow stem as the auger is withdrawn. Maintenance of adequate concrete pressure at the auger tip is critical to reduce the potential for encroachment of adjacent native soils into the concrete column. The rate of withdrawal of the auger must remain constant throughout the installation of the rigid inclusions in order to reduce the potential for necking of the rigid inclusions. Failure to maintain a constant rate of withdrawal of the auger should result in immediate rejection of that rigid inclusion element.

The contractor should adhere to a waiting period of at least 12 hours between the installation of rigid inclusion elements spaced closer than 6 feet, center-to-center. This waiting period is necessary to avoid disturbing the curing concrete in previously cast elements.

Concrete pumps must be fitted with a volume-measuring device and pressure gauge so that the volume of concrete placed in each rigid inclusion and the pressure head maintained during pumping can be observed. A minimum line pressure of 100 psi should be maintained. And the rate of auger withdrawal should be controlled during concrete placement such that the volume of concrete pumped is equal to 100 percent of the theoretical rigid inclusion volume per 60-inch segments. A minimum head of 5 feet of concrete should be maintained above the auger tip during withdrawal of the auger to maintain a full column of concrete and to prevent hole collapse.

A qualified geotechnical engineer should: observe the drilling operations and instrumentation output; monitor concrete injection procedures; record the volume of concrete placed in each rigid inclusion element relative to the calculated volume of the hole; and evaluate the adequacy of individual rigid inclusion installations to ensure that embedment requirements are met and that they are installed in accordance with the ground improvement plans and specifications.

6.0 LIMITATIONS

We have prepared this report for the exclusive use of Port of Everett, PND, and their authorized agents for the Port of Everett Waterfront Place Central PGL Site project in Everett, Washington.

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. No warranty or other conditions, express or implied, should be understood.

Please refer to Appendix C “Report Limitations and Guidelines or Use“ for additional information pertaining to use of this report.

7.0 REFERENCES

American Association of State Highway and Transportation Officials (AASHTO). 2017. “AASHTO LRFD Bridge Design Specifications.”

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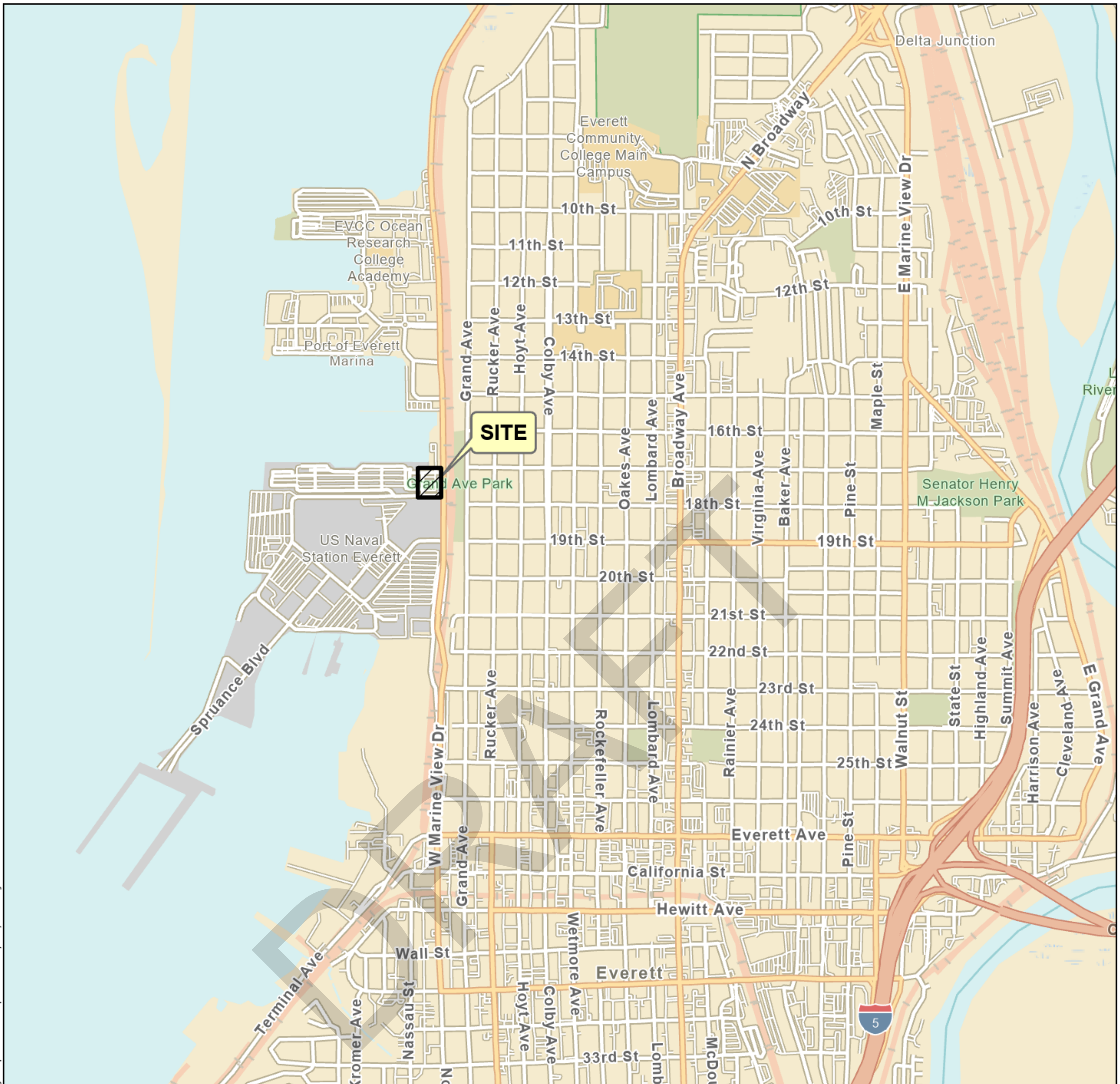

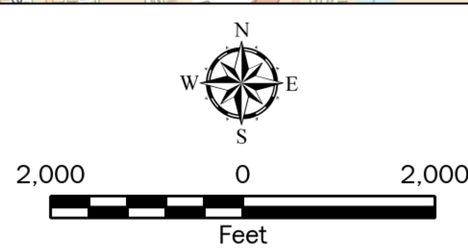

Notes:

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

Data Source: ESRI

Projection: NAD 1983 UTM Zone 10N

Vicinity Map

Port of Everett Waterfront Place Central PGL Site
Everett, Washington

GeoEngineers

Figure 1

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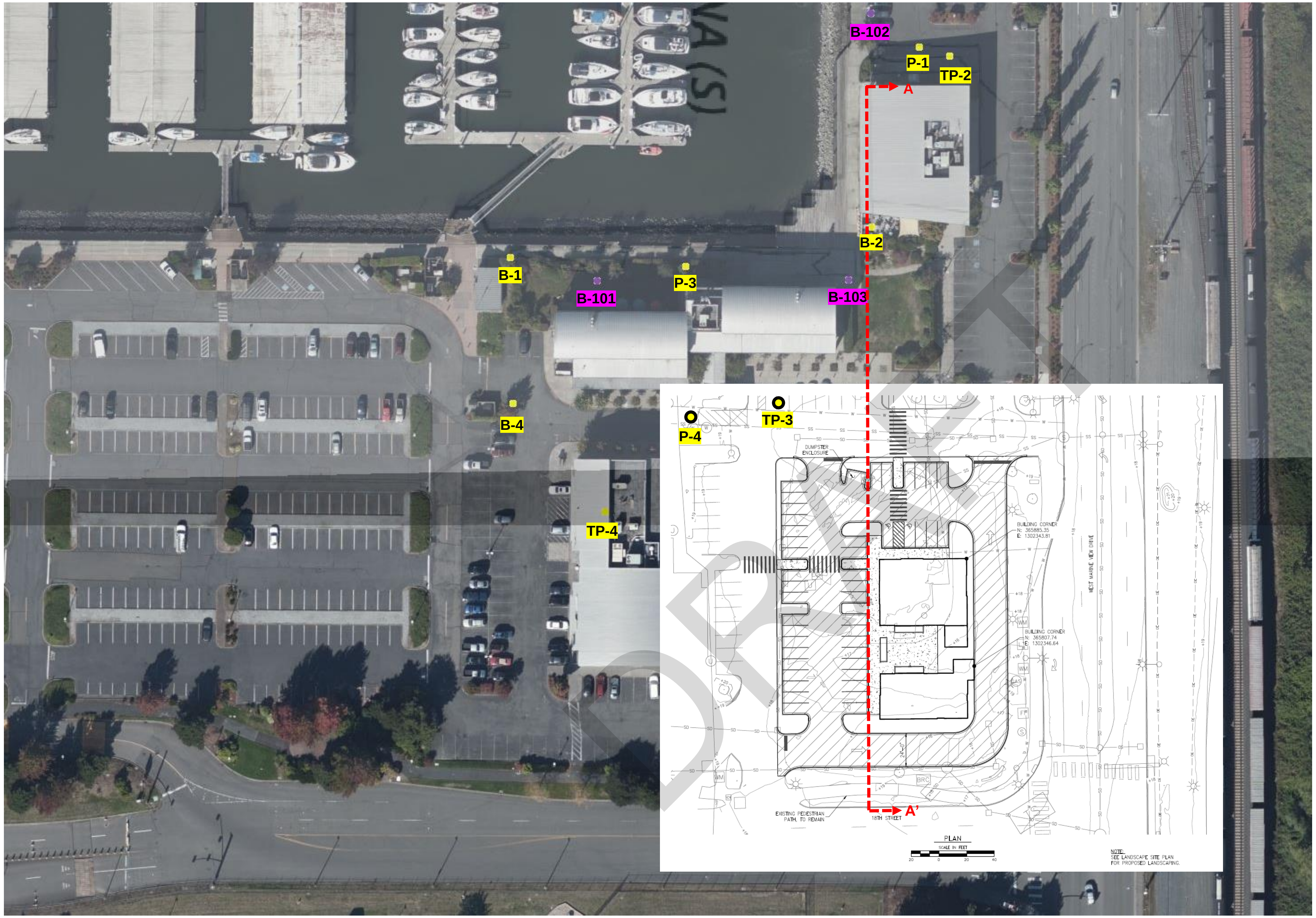

Note: Project site limits from "Port of Everett Waterfront Place Central PGL Site 30% Design Drawings," prepared for Port of Everett, by PND Engineers, Inc., dated September 1, 2021.

Site Plan	
Port of Everett Waterfront Place Central PGL Site Everett, Washington	
GeoEngineers	**Figure 2**

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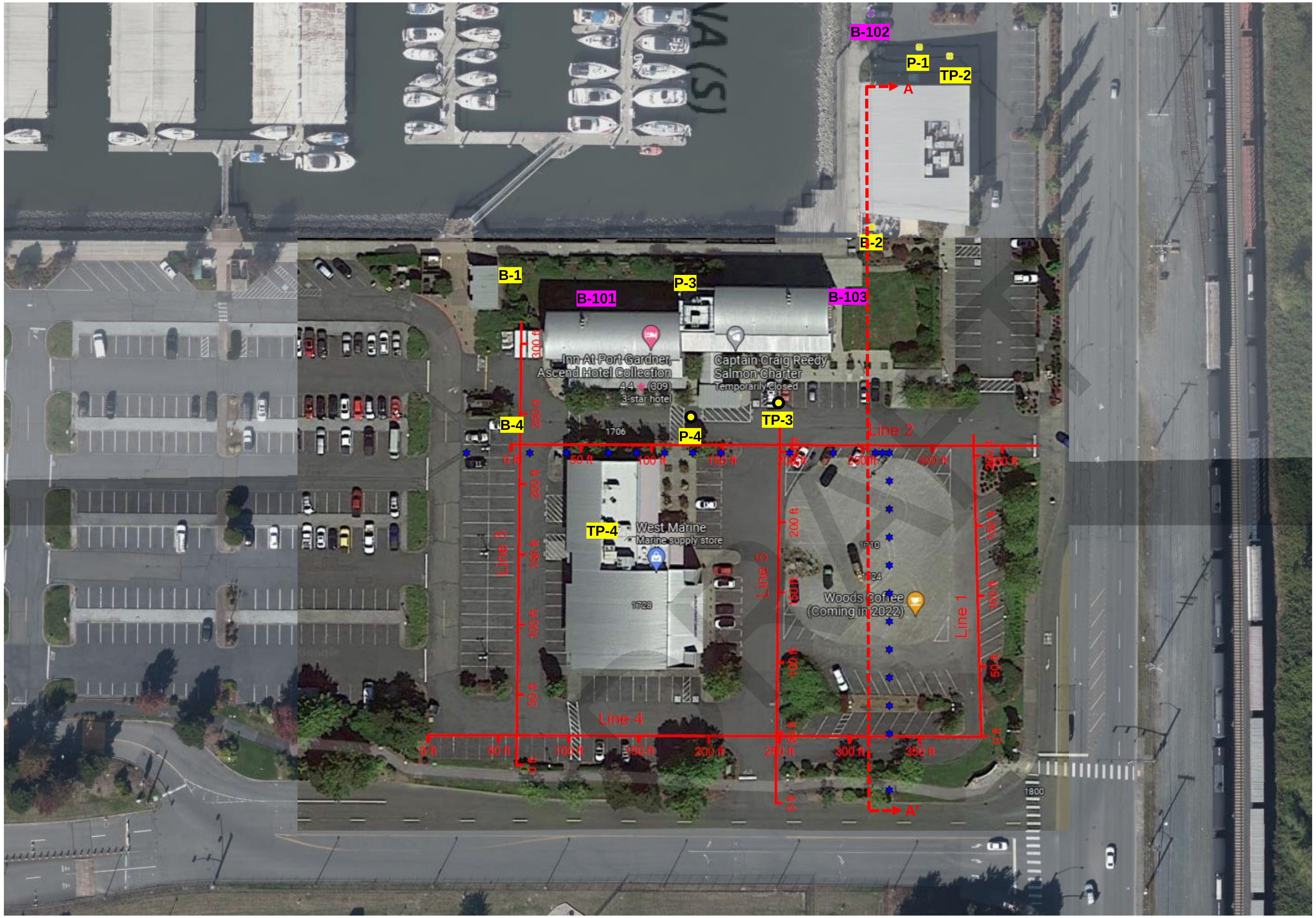

Notes:

- Line 1 through Line 5 – Multichannel Analysis of Surface Waves (MASW)
- Blue Stars – Microtremor Array Measurements (MAM)

Geophysical Survey Arrays	
Port of Everett Waterfront Place Central PGL Site Everett, Washington	
GeoEngineers	Figure 3

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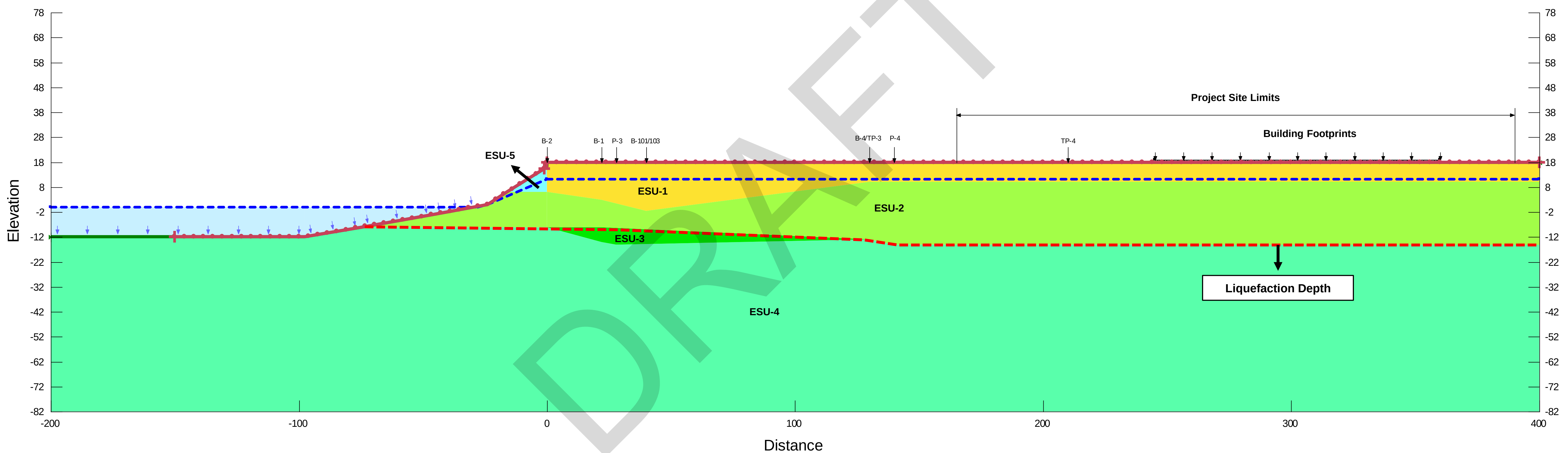
- ESU-1: Hydraulic Fill (Soft ML/Loose to Medium Dense SM)
- ESU-2: Recent Deposits (Soft to Medium Stiff ML/Loose SM)
- ESU-3: Recent Deposits (Medium Dense SP)
- ESU-4: Glacially Consolidated Soils (Dense to Very Dense SP)
- ESU-5: Rip-Rap Rock



Cross Section A-A' with Liquefaction Depth

Port of Everett Waterfront Place Central PGL Site
Everett, Washington

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Figure 4

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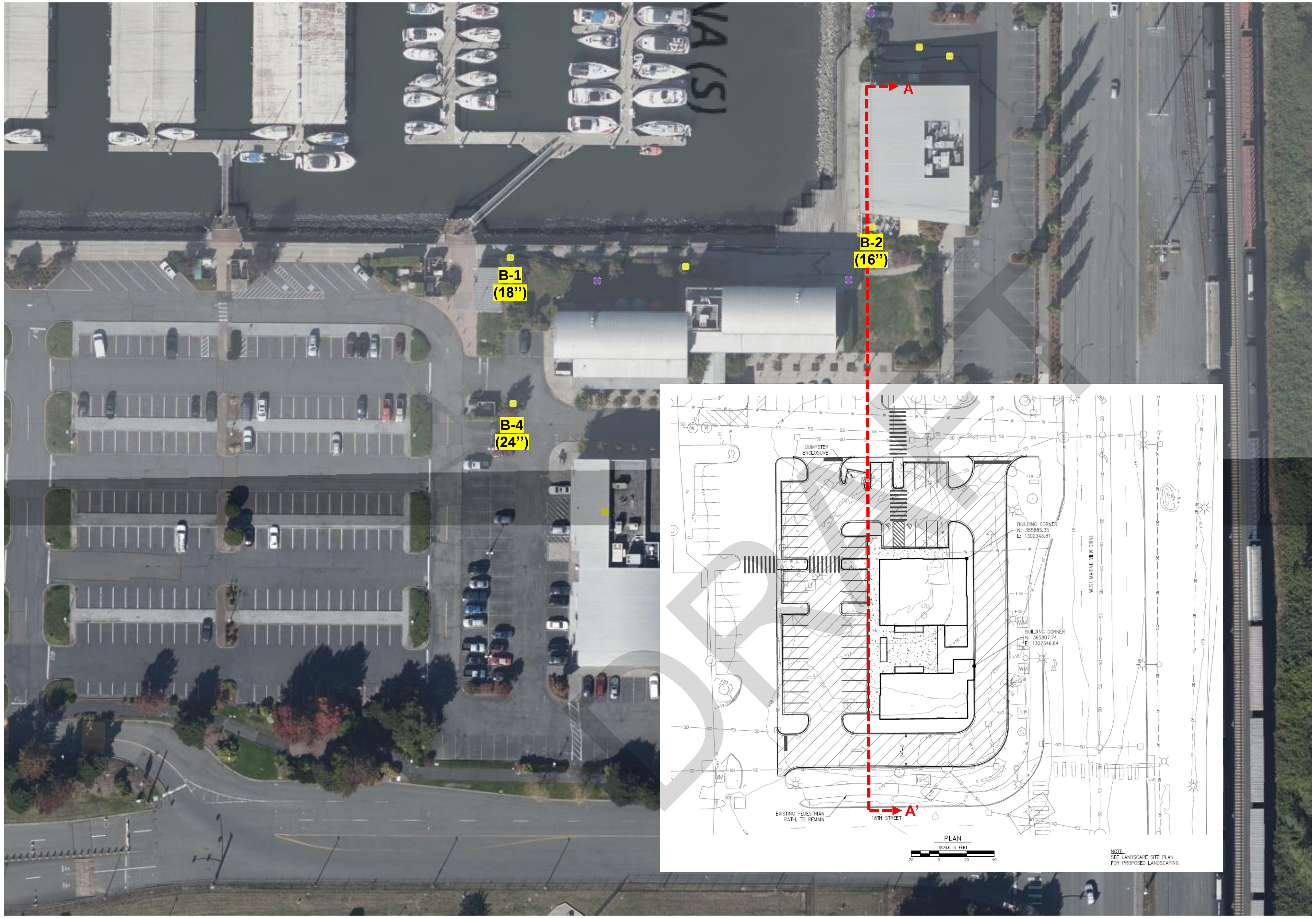

Liquefaction-Induced Free-Field Ground Settlement

Port of Everett Waterfront Place Central PGL Site
Everett, Washington

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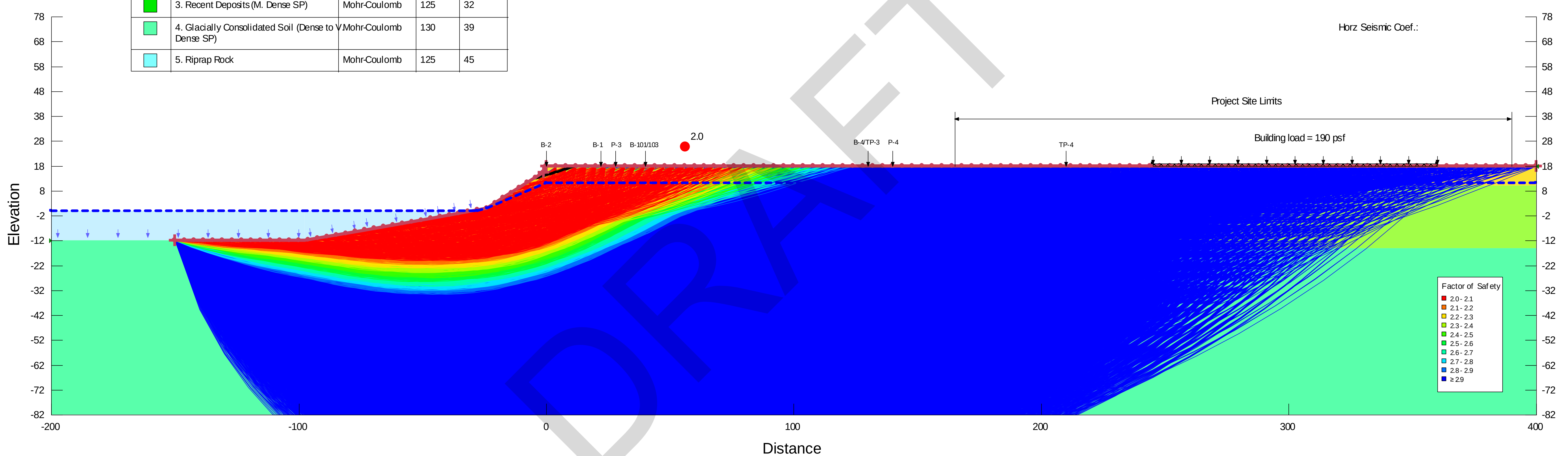
Figure 5

00676-030-00 Date Exported: 10/13/2021



Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Friction Angle (°)
	1. Hydraulic Fill (Soft ML/Loose to M. Dense SM)	Mohr-Coulomb	120	28
	2. Recent Deposits (Soft to M. Stiff ML/Loose SM)	Mohr-Coulomb	125	28
	3. Recent Deposits (M. Dense SP)	Mohr-Coulomb	125	32
	4. Glacially Consolidated Soil (Dense to V. Dense SP)	Mohr-Coulomb	130	39
	5. Riprap Rock	Mohr-Coulomb	125	45

Slope Stability - Static Condition

Port of Everett Waterfront Place Central PGL Site
Everett, Washington

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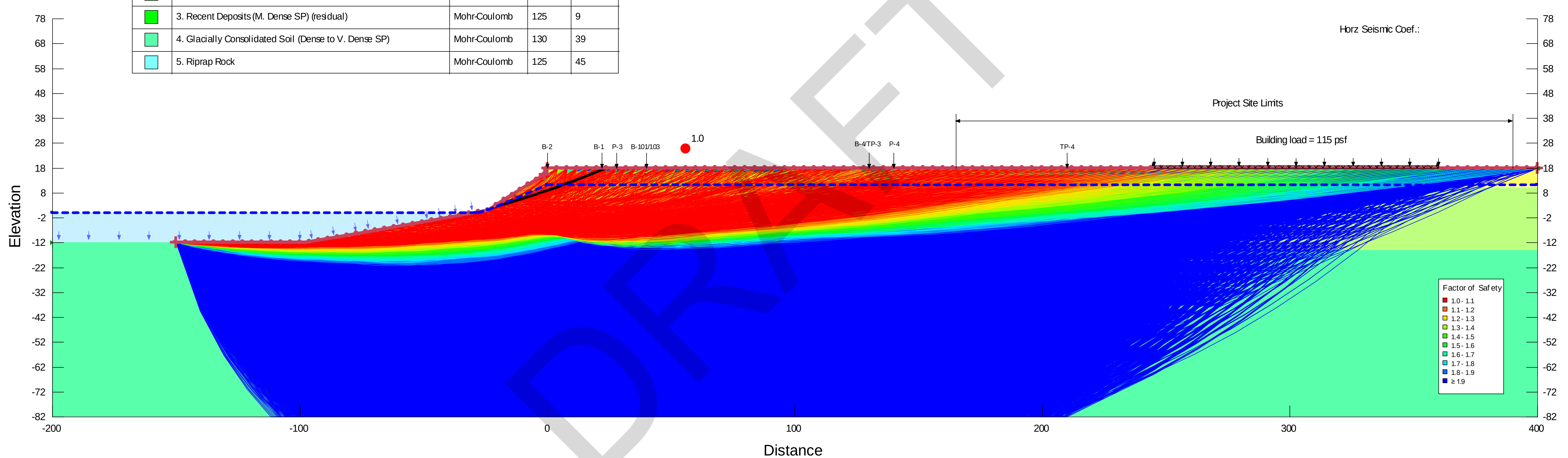
Figure 6

00676-030-00 Date Exported: 10/13/2021

Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Friction Angle (°)
	1. Hydraulic Fill (Soft ML/Loose to M. Dense SM) (80%)	Mohr-Coulomb	120	22.4
	1. Hydraulic Fill (Soft ML/Loose to M. Dense SM) (residual)	Mohr-Coulomb	120	3
	2. Recent Deposits (Soft to M. Stiff ML/Loose SM) (residual)	Mohr-Coulomb	125	5
	3. Recent Deposits (M. Dense SP) (residual)	Mohr-Coulomb	125	9
	4. Glacially Consolidated Soil (Dense to V. Dense SP)	Mohr-Coulomb	130	39
	5. Riprap Rock	Mohr-Coulomb	125	45



Slope Stability - Post-Earthquake Condition

Port of Everett Waterfront Place Central PGL Site
Everett, Washington

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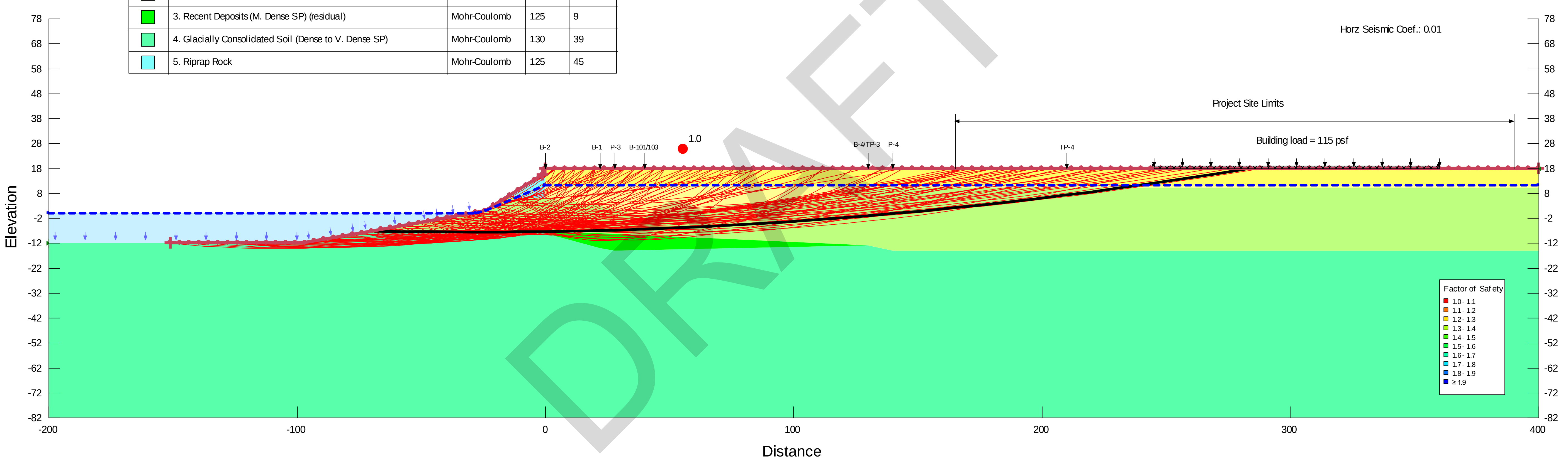
Figure 7

00676-030-00 Date Exported: 10/13/2021

Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Friction Angle (°)
	1. Hydraulic Fill (Soft ML/Loose to M. Dense SM) (80%)	Mohr-Coulomb	120	22.4
	1. Hydraulic Fill (Soft ML/Loose to M. Dense SM) (residual)	Mohr-Coulomb	120	3
	2. Recent Deposits (Soft to M. Stiff ML/Loose SM) (residual)	Mohr-Coulomb	125	5
	3. Recent Deposits (M. Dense SP) (residual)	Mohr-Coulomb	125	9
	4. Glacially Consolidated Soil (Dense to V. Dense SP)	Mohr-Coulomb	130	39
	5. Riprap Rock	Mohr-Coulomb	125	45



- ❑ **Failure Wedge Height = 26 feet**
- ❑ **Yield Acceleration = 0.01g**
- ❑ **Lateral Spreading = 53 inches**

Slope Stability - Seismic Condition

Port of Everett Waterfront Place Central PGL Site
Everett, Washington

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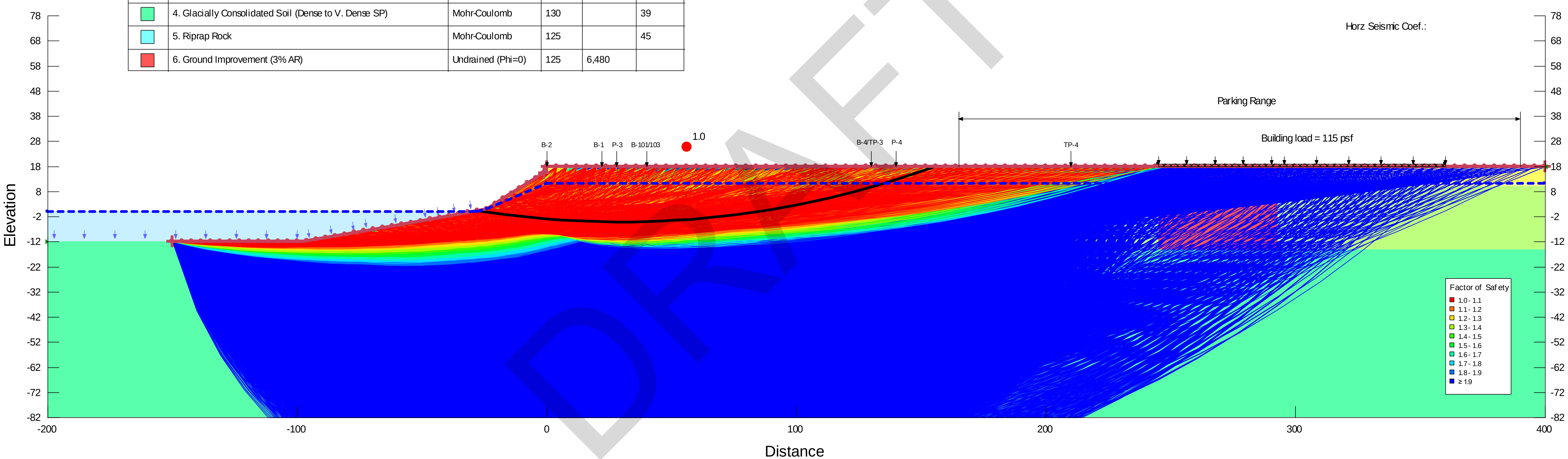
Figure 8

00676-030-00 Date Exported: 10/13/2021



Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Total Cohesion (psf)	Effective Friction Angle (°)
	1. Hydraulic Fill (Soft ML/Loose to M. Dense SM) (80%)	Mohr-Coulomb	120		22.4
	1. Hydraulic Fill (Soft ML/Loose to M. Dense SM) (residual)	Mohr-Coulomb	120		3
	2. Recent Deposits (Soft to M. Stiff ML/Loose SM) (residual)	Mohr-Coulomb	125		5
	3. Recent Deposits (M. Dense SP) (residual)	Mohr-Coulomb	125		9
	4. Glacially Consolidated Soil (Dense to V. Dense SP)	Mohr-Coulomb	130		39
	5. Riprap Rock	Mohr-Coulomb	125		45
	6. Ground Improvement (3% AR)	Undrained (Phi=0)	125	6,480	

Post-Improvement Slope Stability - Post-Earthquake Condition

Port of Everett Waterfront Place Central PGL Site
Everett, Washington

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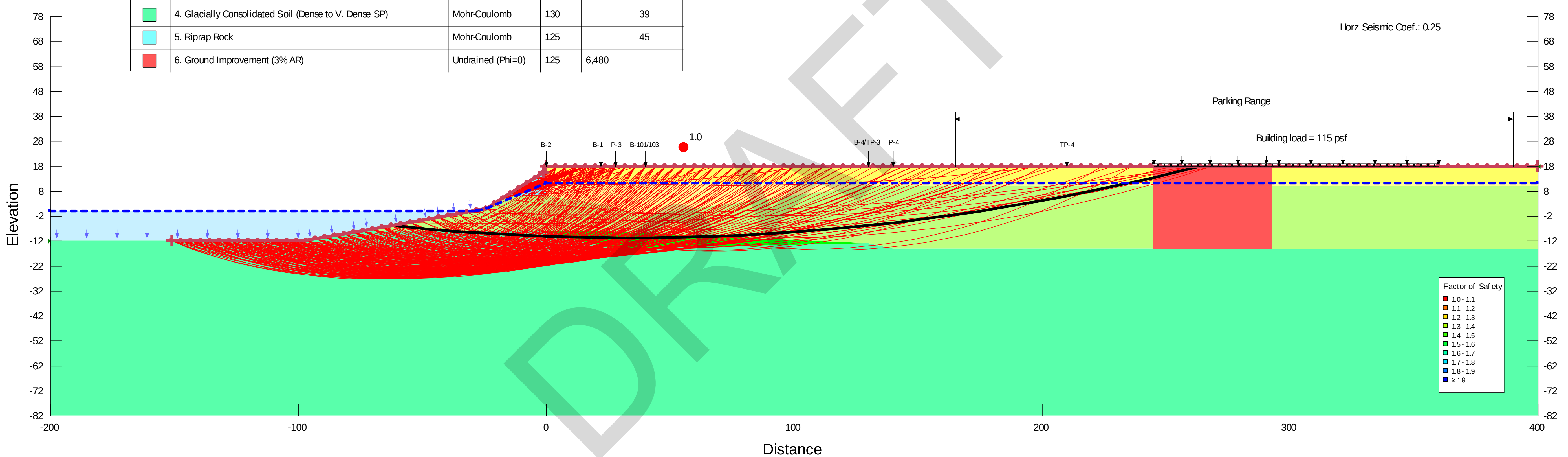
Figure 9

00676-030-00 Date Exported: 10/13/2021

Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Total Cohesion (psf)	Effective Friction Angle (°)
	1. Hydraulic Fill (Soft ML/Loose to M. Dense SM) (80%)	Mohr-Coulomb	120		22.4
	1. Hydraulic Fill (Soft ML/Loose to M. Dense SM) (residual)	Mohr-Coulomb	120		3
	2. Recent Deposits (Soft to M. Stiff ML/Loose SM) (residual)	Mohr-Coulomb	125		5
	3. Recent Deposits (M. Dense SP) (residual)	Mohr-Coulomb	125		9
	4. Glacially Consolidated Soil (Dense to V. Dense SP)	Mohr-Coulomb	130		39
	5. Riprap Rock	Mohr-Coulomb	125		45
	6. Ground Improvement (3% AR)	Undrained (Phi=0)	125	6,480	



- Failure Wedge Height = 29 feet
- Yield Acceleration = 0.25g
- Lateral Spreading = 3.1 inches

00676-030-00 Date Exported: 10/13/2021

Post-Improvement Slope Stability - Seismic Condition

Port of Everett Waterfront Place Central PGL Site
Everett, Washington

GeoEngineers

Figure 10

APPENDIX A
Previous Subsurface Investigations

BORING NO. 1

Logged By DA

Date 12/15/87

ELEV. 18'±*

Graph	USCS	Soil Description	Depth (ft.)	Sample	(N) Blows Ft.	W (%)
	SM	(6" brown SAND, little gravel) Dark gray silty SAND, trace to a little wood, trace rubble, wet to saturated, loose to medium dense (Dike fill)		I	12	33
				I	14	29
			5	II	P	26 100
				I	12	44
			10	I	7	29
		Sand lenses		I	4	36
	ml	Dark gray organic sandy SILT, little wood debris, wet, soft	15			
	sm	Gray silty SAND with wood debris, wet, loose		I	5	64
			20			
				II	P	28
			25			
	SP	Gray SAND, little organics, some shells and wood, trace silt, saturated, medium dense, becoming very dense		I	21	24
			30			
				I	43	28
			35			
				I	65	22
			40			
				I	80	22
		Gravel lense	45			
				I	83	24

Boring terminated at 48 feet below existing grade. Groundwater encountered at 14 feet during drilling. Boring backfilled with cuttings.

* Boring, test pit, and probe elevations determined by interpolating between contour lines shown on the topographic survey drawing by Tim Hanson and Associates, Project No. 86013, Sheet 1 of 1, dated May, 1986.

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BORING LOG
PIONEER MARINE PLAZA
EVERETT, WASHINGTON

Proj. No. 3691	Date Jan'88	Plate 4

BORING NO. 2

Logged By DA

Date 12/15/87

ELEV. 17'±*

Graph	US CS	Soil Description	Depth (ft.)	Sample	(N) Blows Ft.	W (%)	
	sm	Gray and brown silty SAND, little gravel, wet, very dense			87	11	
	gm	Gray-brown silty sandy GRAVEL cobbles and boulders, very dense, armor rock			50/6"	9	
	ml	Gray sandy SILT and wood debris, soft to very stiff, wet (Dike fill)	5		18		
					3	91	
			10		P	31	
	ML	Gray fine sandy clayey SILT, little wood fiber, wet, soft to medium stiff			4	37	
			15				LL=39 PI=5
					P	59	
			20				
	sm	Gray silty SAND, saturated, loose, shell fragments			4	21	
			25				
	sp	Gray SAND, trace gravel, trace wood, very dense, saturated			61	18	
			30		50	31	
			35				
					50	22	

Boring terminated at 38.5 feet below existing grade. Groundwater encountered at 19 feet during drilling. Boring backfilled with cuttings.

Subsurface conditions depicted represent our observations at the time and location of this exploratory hole, modified by engineering tests, analysis, and judgement. They are not necessarily representative of other times and locations. We cannot accept responsibility for the use or interpretation by others of information presented on this log.

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BORING LOG
PIONEER MARINE PLAZA
EVERETT, WASHINGTON

Proj. No. 3691 | Date Jan'88 | Plate 5

BORING NO. 3

Logged By DA

Date 12/15/87

ELEV. 17.5'±*

Graph	USCS	Soil Description	Depth (ft.)	Sample	(N) Blows Ft.	W (%)	
	sp	Brown SAND, little gravel, wet (fill)			22	15	
	sm	Gray silty SAND, wet, medium dense (fill)	5		15	27	
	ml	Dark gray sandy SILT, soft, low plasticity (fill)			P 5	11 32 64 43	
	SM	Gray silty SAND, little wood, saturated, medium dense	10		17	32	
	ml	Dark gray sandy SILT with some wood fragments, soft to medium stiff, saturated, low plasticity	15		5 P 11	41 79 46	qu 0.5tsf
	sp	Gray-brown SAND, trace gravel, trace silt, saturated, medium dense to very dense - gravel	20 25 30		36 50/5" 50/5"	21 20 48	

Boring terminated at 33.5 feet below existing grade. No groundwater encountered during drilling. Boring backfilled with cuttings.

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BORING LOG
PIONEER MARINE PLAZA
EVERETT, WASHINGTON

Proj. No. 3691 | Date Jan'88 | Plate 6

BORING NO. 4

Logged By DA

Date 12/16/87

ELEV. 19'±*

Graph	USCS	Soil Description	Depth (ft.)	Sample	(N) Blows Ft.	W (%)
	sp	Brown SAND, little gravel, wet, medium dense (fill)			11	11
	sm	Gray silty fine to medium SAND, moist, medium dense (fill)			11	22
	ML	Gray sandy SILT and shell fragments, loose (fill) (Dike fill)	5		5	30
					P	53 35
	sm	Gray silty fine SAND, trace sawdust, saturated, very loose	10		1	38
					2	34
			15			
	ml	Gray SILT, saturated, very loose			1	67
			20			
	sm	Dark gray silty SAND, trace gravel, trace wood, saturated, loose			10	26
			25			
	ml	Dark gray sandy SILT and wood, saturated medium dense, non-plastic			16	29
			30			
	SP	Gray SAND, saturated, medium dense to very dense			46	24
			35			
					68	24
			40			
					50/6"	24
			45			
					79/11"	20
		- gravel lense	50			
					50/3"	19
			55			
					50/5"	

Boring terminated at 58 feet below existing grade. Groundwater encountered at 10 feet during drilling. Boring backfilled with cuttings.

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BORING LOG
PIONEER MARINE PLAZA
EVERETT, WASHINGTON

Proj. No. 3691	Date Jan'88	Plate 7

BORING NO. 101

Logged By STL

Date 9-20-89

Elev. 99±*

Graph	USCS	Soil Description	Depth (ft.)	Sample	(N) Blows Ft.	W (%)
		"Fill" 1" minus crushed rock				
		"Fill" 4" crushed rock				
	ml	"Fill" gray/black sandy SILT, some gravel, moist to wet with roots and wood fragments, loose to medium dense		I	9	
			5	I	7	
		-with pieces of rock to 3" at 7½'		I	6	
			10			
		-medium dense below 12½'		I	18	
			15			
				I	11	
			20			
	sm	Gray silty SAND with wood chips		I	10	

Boring terminated at 24 feet below existing grade.
No groundwater encountered during drilling.
Well as built:
Screen size: 2" Casing size: 2'
Screen location: 22½ - 5'
Locking cap: No
Bentonite seal: 3' - 2'
Concrete: 2' - surface
Surface casing: Steel flush mount monument
Backfill: sand 22½ - 3'

*Elevation data: Centerline of West Marine View Drive, assumed elevation 100.00 feet.

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BORING LOG
PIONEER MARINE PLAZA
EVERETT, WASHINGTON

BORING NO. 102

Logged By STL

Elev. 99±*

Date 9-20-89

Graph	USCS	Soil Description	Depth (ft.)	Sample	(N) Blows Ft.	W (%)
		"Fill" 1" minus rock				
	ml-sm	"Fill" Gray sandy SILT with wood chips, moist to wet, medium dense and very loose		I	13	
			5			
		-silty SAND at 7½' -very loose 7½ - 14'		I	2	
			10			
		-wet and dilatant below 12½'		I	2	
			15			
				I	12	
			20			
	sp	Gray medium SAND with silt		I	13	

Boring terminated at 24 feet below existing grade.
Groundwater encountered at 12½ feet during drilling.
Well as built:
Screen size: #10   Casing size: 2"
Screen location: 22½ - 5'
Locking cap: No
Bentonite seal: 3' - 2'
Concrete: 2' - surface
Surface casing: Steel flush casing
Backfill: sand 22½ - 3'

Subsurface conditions depicted represent our observations at the time and location of this exploratory hole, modified by engineering tests, analysis, and judgement. They are not necessarily representative of other times and locations. We cannot accept responsibility for the use or interpretation by others of information presented on this log.

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BORING LOG

PIONEER MARINE PLAZA
EVERETT, WASHINGTON

BORING NO. 103

Logged By STL

Date 9-20-89

Elev. 99±*

Graph	USCS	Soil Description	Depth (ft.)	Sample	(N) Blows Ft.	W (%)
		"Fill" 4" of 1" minus rock Brown silty SAND, moist				
		"Fill" Gray silty SAND, moist, medium dense to loose		I	20	
			5			
		-loose below 7½'				
		-with wood chips and some gravel at 7½'		I	4	
			10			
				I	4	

Boring terminated at 14 below existing grade.
No groundwater encountered during drilling.
No well built.
Backfill: Cutting and bentonite

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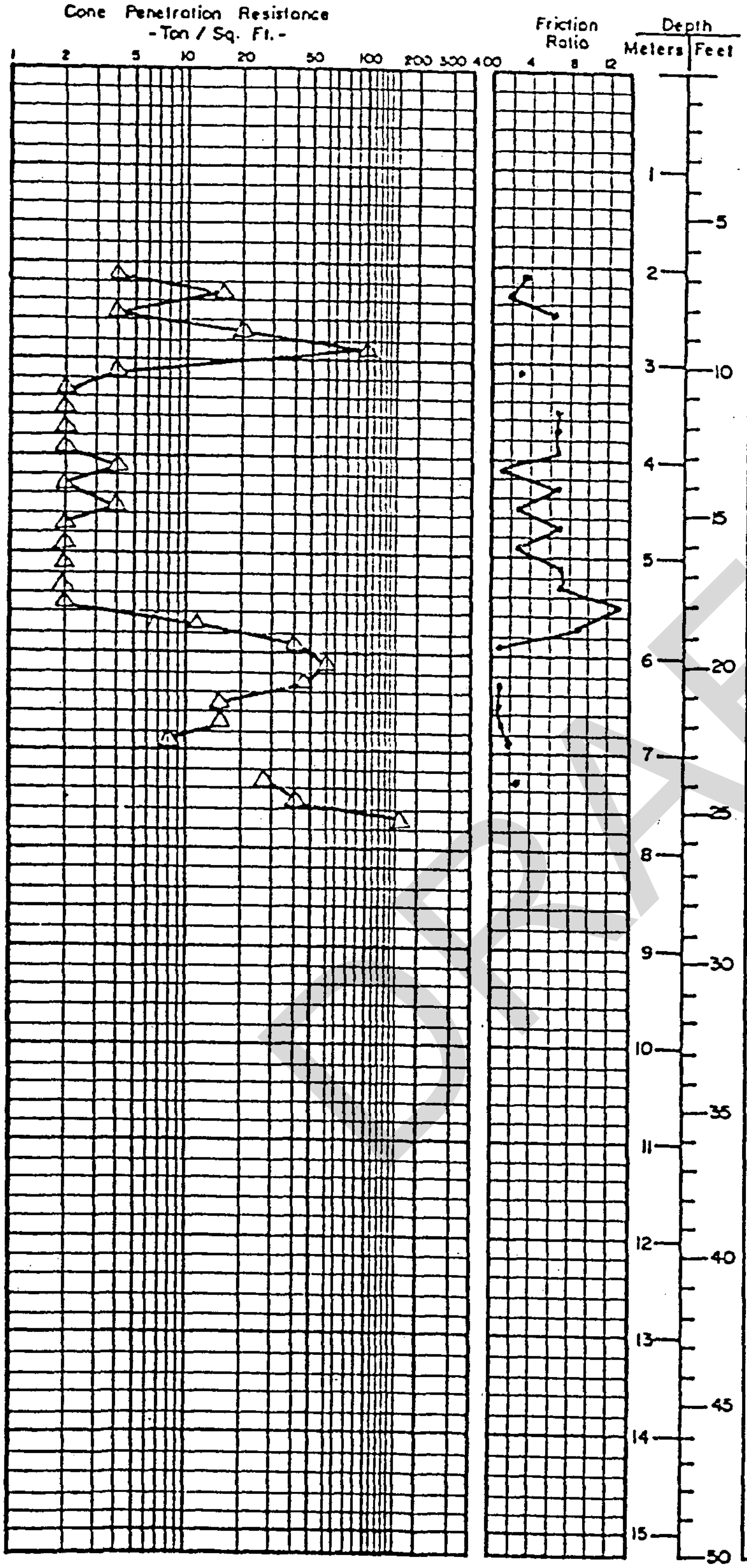
BORING LOG

PIONEER MARINE PLAZA
EVERETT, WASHINGTON

Dutch Cone Log P-1



Soil Interpretation

sm	Gray and brown SAND wet, medium dense - rubble at 3 feet (upper 6' predrilled
ml	Gray sandy clayey SILT trace organics, wet, soft to med. stiff
gm	rubble
ml	Dark gray to gray bedded sandy clayey SILT, saturated , very soft to soft
sp	Gray SAND, saturated, loose to medium dense

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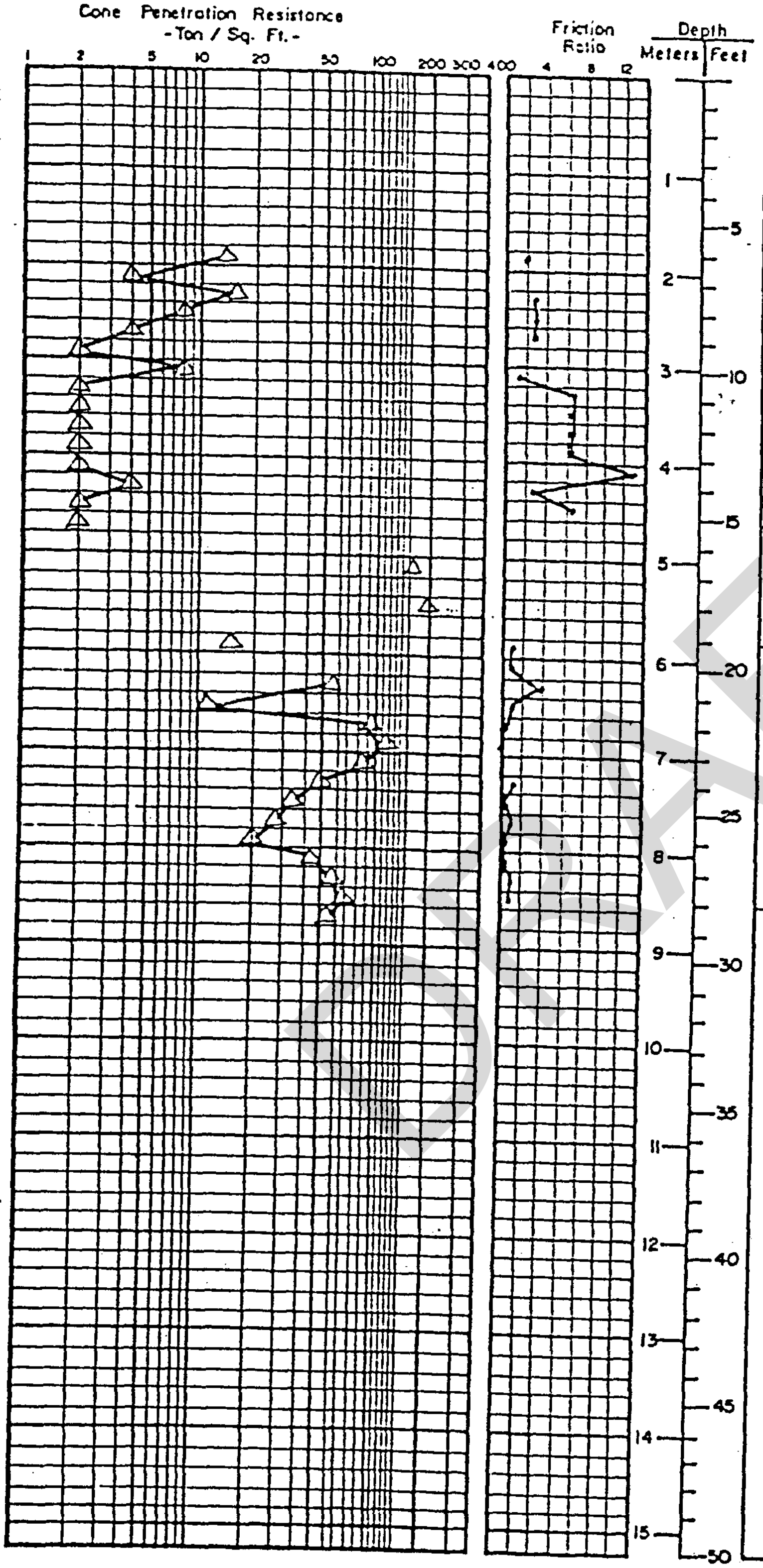
PIONEER MARINE PLAZA
EVERETT, WASHINGTON

Proj. No. 3691	Date Dec'87	Plate 10

Dutch Cone Log P-2



Soil Interpretation

sm	Brown and gray silty SAND, trace gravel, wet, medium dense - rubble 1 to 2 feet (upper 6' predrilled)
sm	Gray silty SAND, wet, medium dense
ml	Dark gray sandy clayey SILT trace organics, very soft to soft, saturated
gm	GRAVEL
sp	Gray silty SAND, trace organics dense

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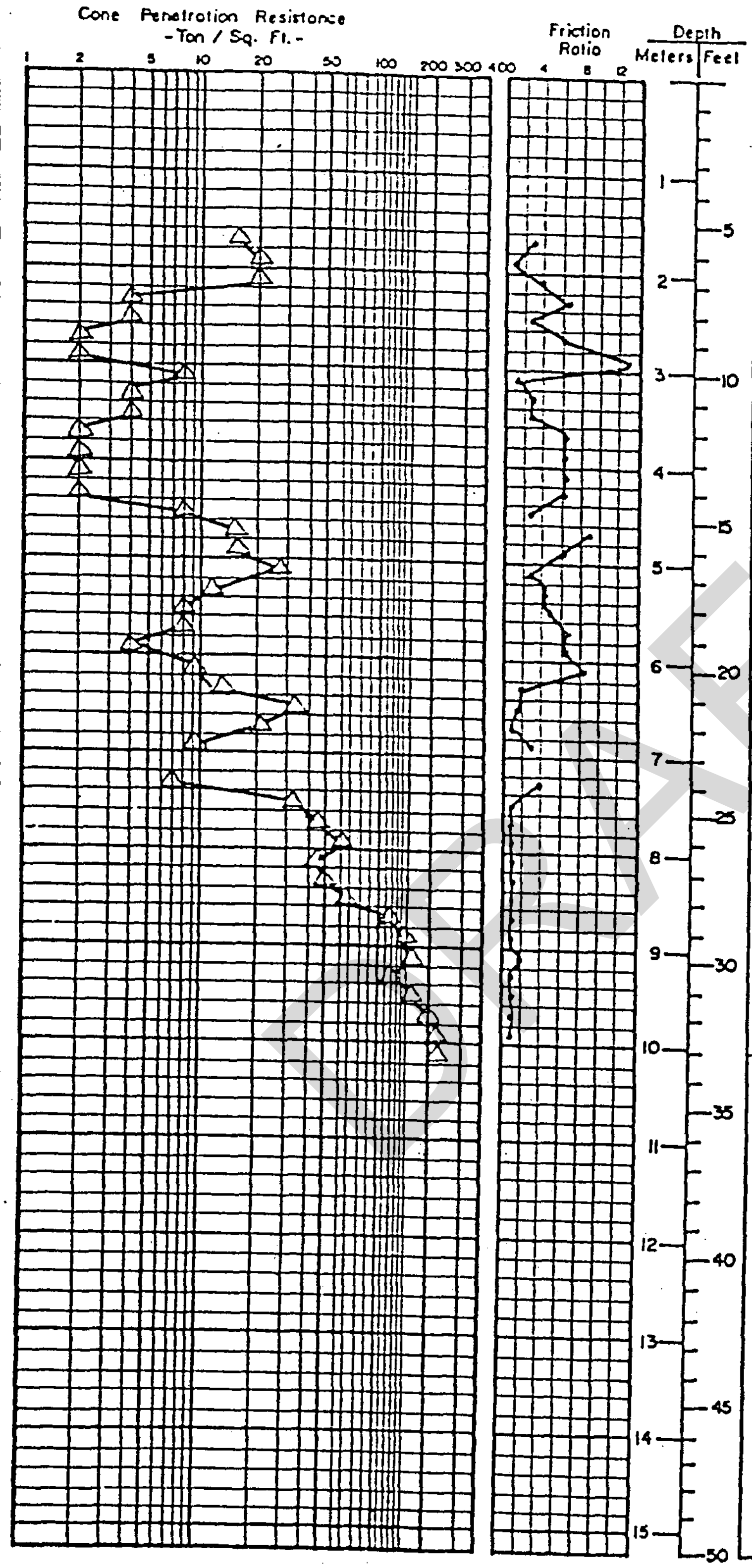
PIONEER MARINE PLAZA
EVERETT, WASHINGTON

Proj. No. 3691	Date Dec'87	Plate 11

Dutch Cone Log P-3



Soil Interpretation

sm	Gray and brown SAND and silty SAND, wet, medium dense - gravel at 1.5 feet (upper 5 feet predrilled)
sm	Gray silty SAND trace shell fragments, saturated, loose
ml	Dark gray sandy clayey SILT trace to little wood, very soft to soft
cl	Dark gray sandy silty organic to inorganic CLAY, soft to stiff - with sand lenses below 20 feet
sp	Gray SAND, saturated, loose to very dense

DRAFT

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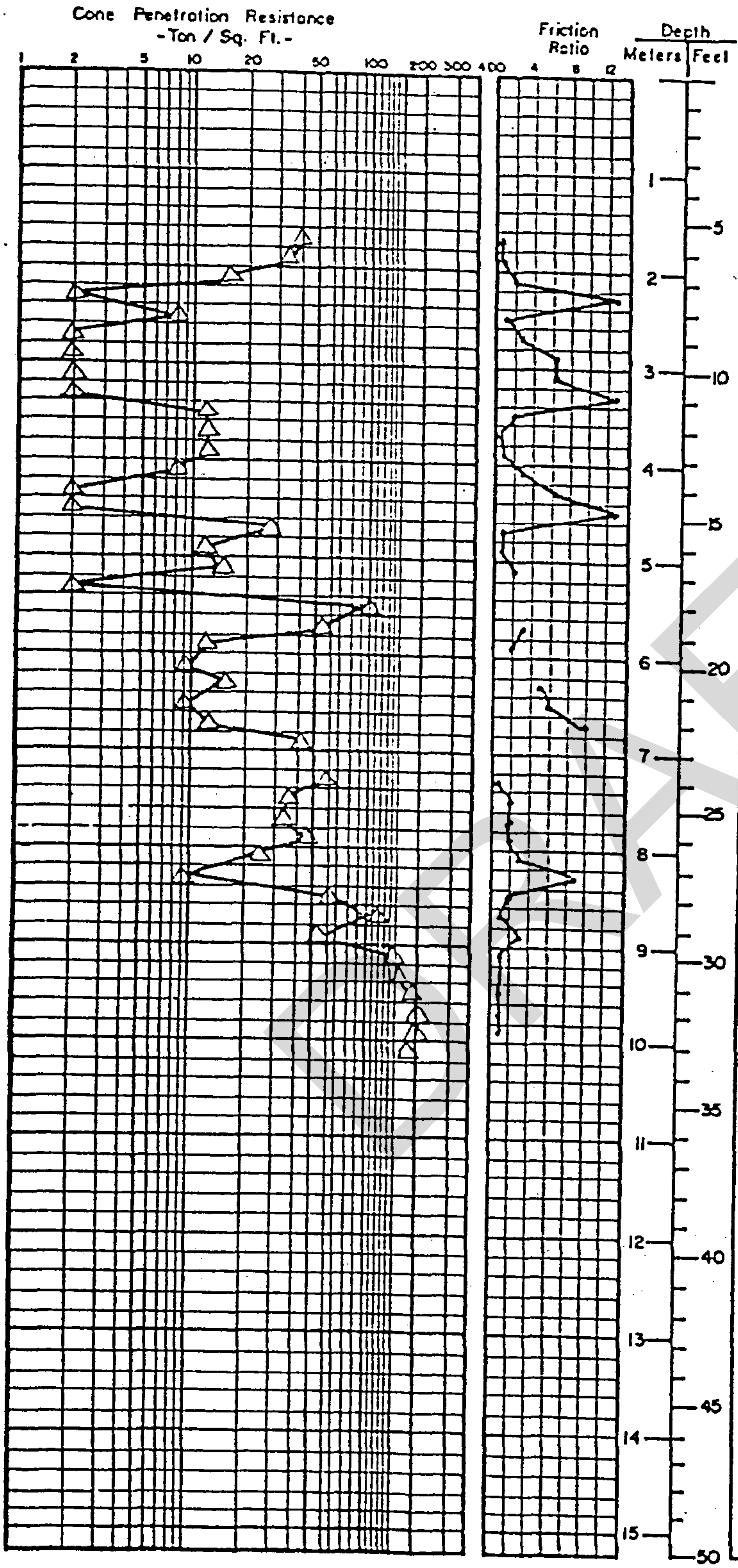
PIONEER MARINE PLAZA
EVERETT, WASHINGTON

Proj. No. 3691	Date Dec'87	Plate 12

Dutch Cone Log P-4



Soil Interpretation

sm	Gray and brown silty SAND, trace gravel, medium dense - rubble at 1 to 2 feet (upper 5' predrilled)
ml	Dark gray sandy SILT, trace clay saturated, very soft to soft
sm	Gray silty SAND, wet to saturated loose
ml	Soft SILT
gm ml	GRAVEL and sandy GRAVEL, interbedded with organic SILT
sp	Gray silty SAND with silt interbeds, wet to saturated, medium dense to very dense

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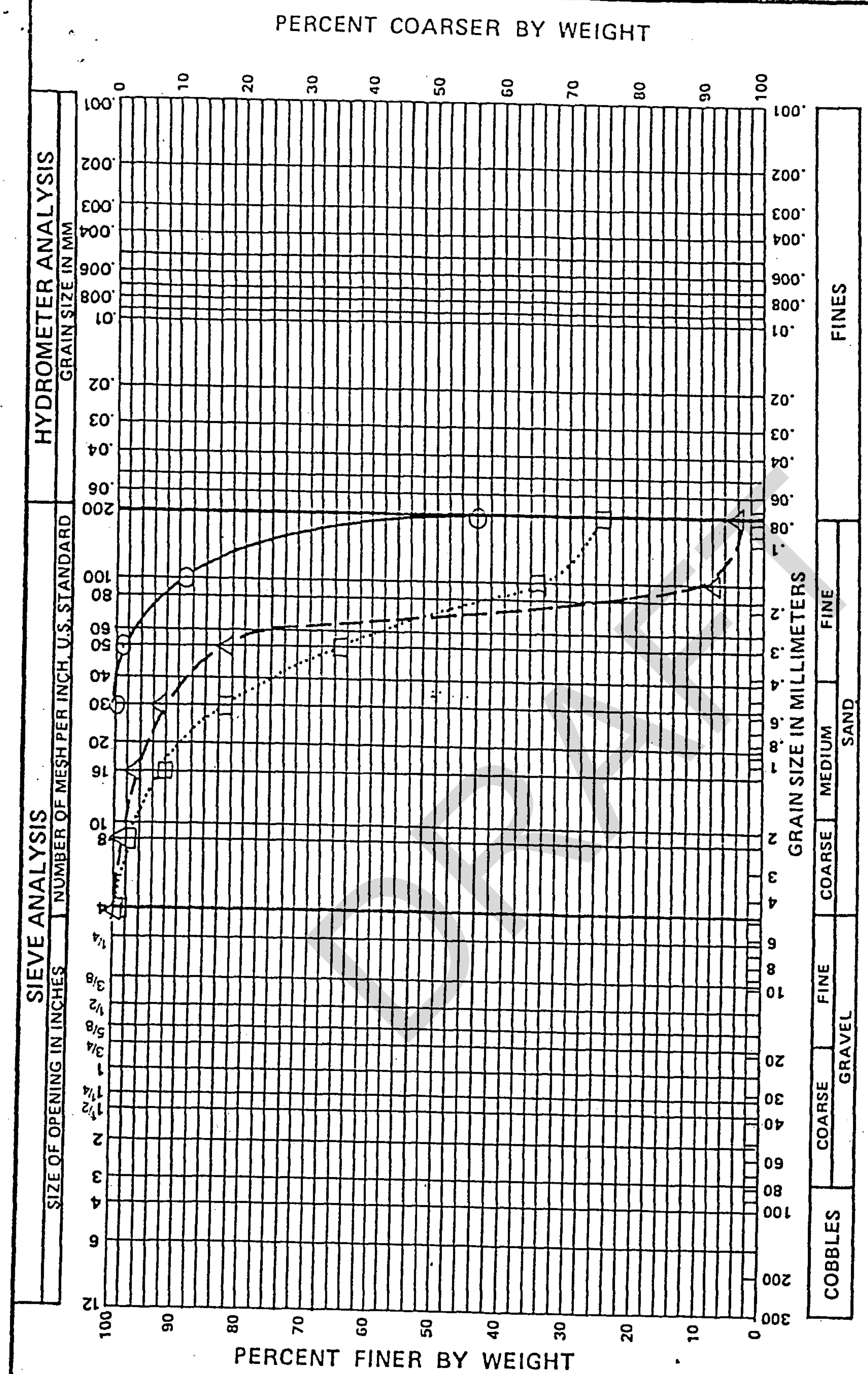
PIONEER MARINE PLAZA
EVERETT, WASHINGTON

Proj. No. 3691	Date Dec'87	Plate 13



KEY	Boring or Test Pit No.	DEPTH (ft.)	USCS	DESCRIPTION	Moisture Content (%)	LL	PL
○———	B-1	12.5	SM	silty SAND	36		
△— — —	B-1	32.5	SP	poorly graded SAND	28		
□........	B-3	10.0	SM	silty SAND	32		

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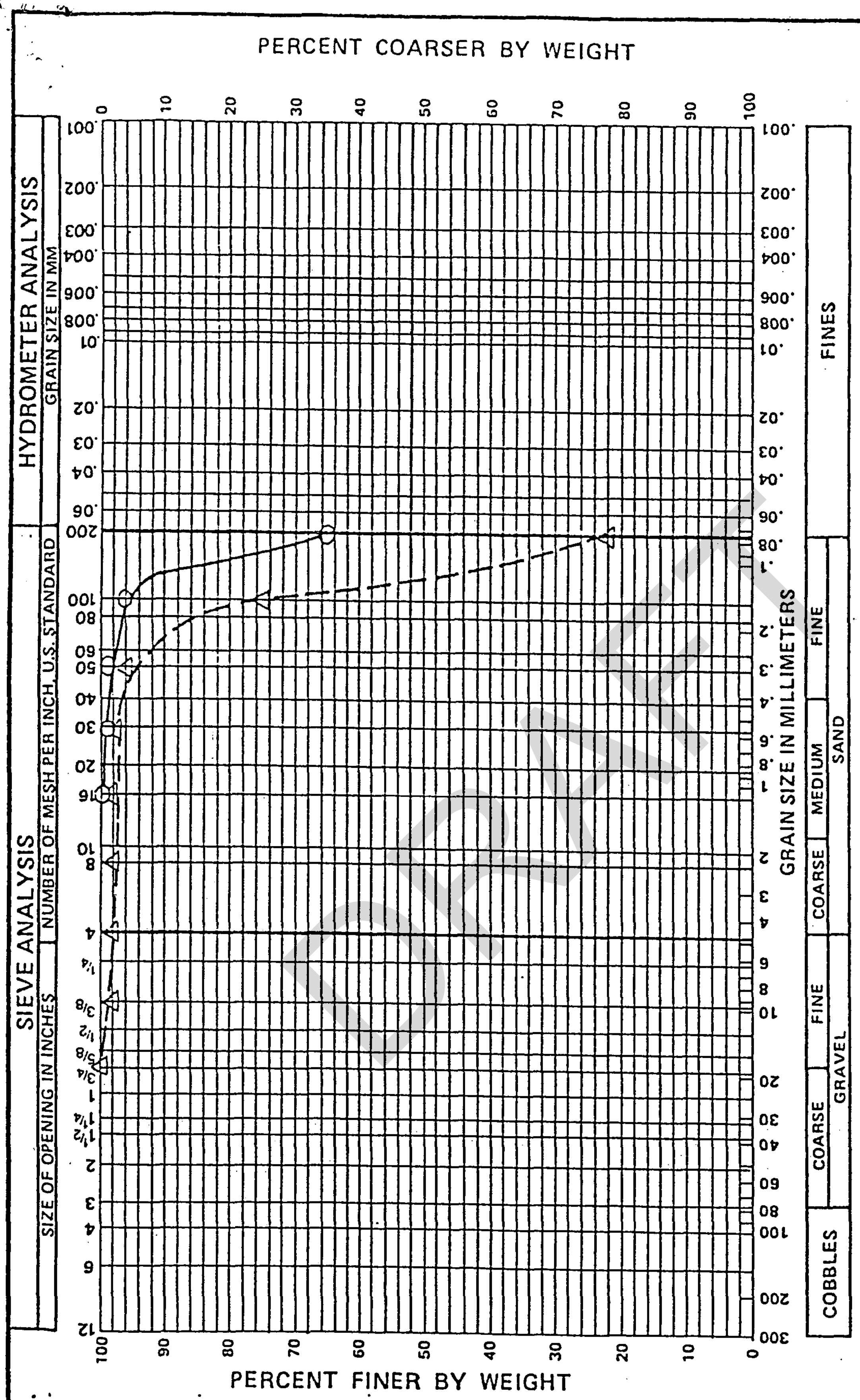
GRAIN SIZE ANALYSES
PIONEER MARINE PLAZA
EVERETT, WASHINGTON

Proj. No. 3691 | Date Jan'88 | Plate 14



KEY	Boring or Test Pit No.	DEPTH (ft.)	USCS	DESCRIPTION	Moisture Content (%)	LL	PL
○——	B-4	10.0	ML	sandy SILT	35		
△- - -	TP-3	4.5	SM	silty SAND	28		

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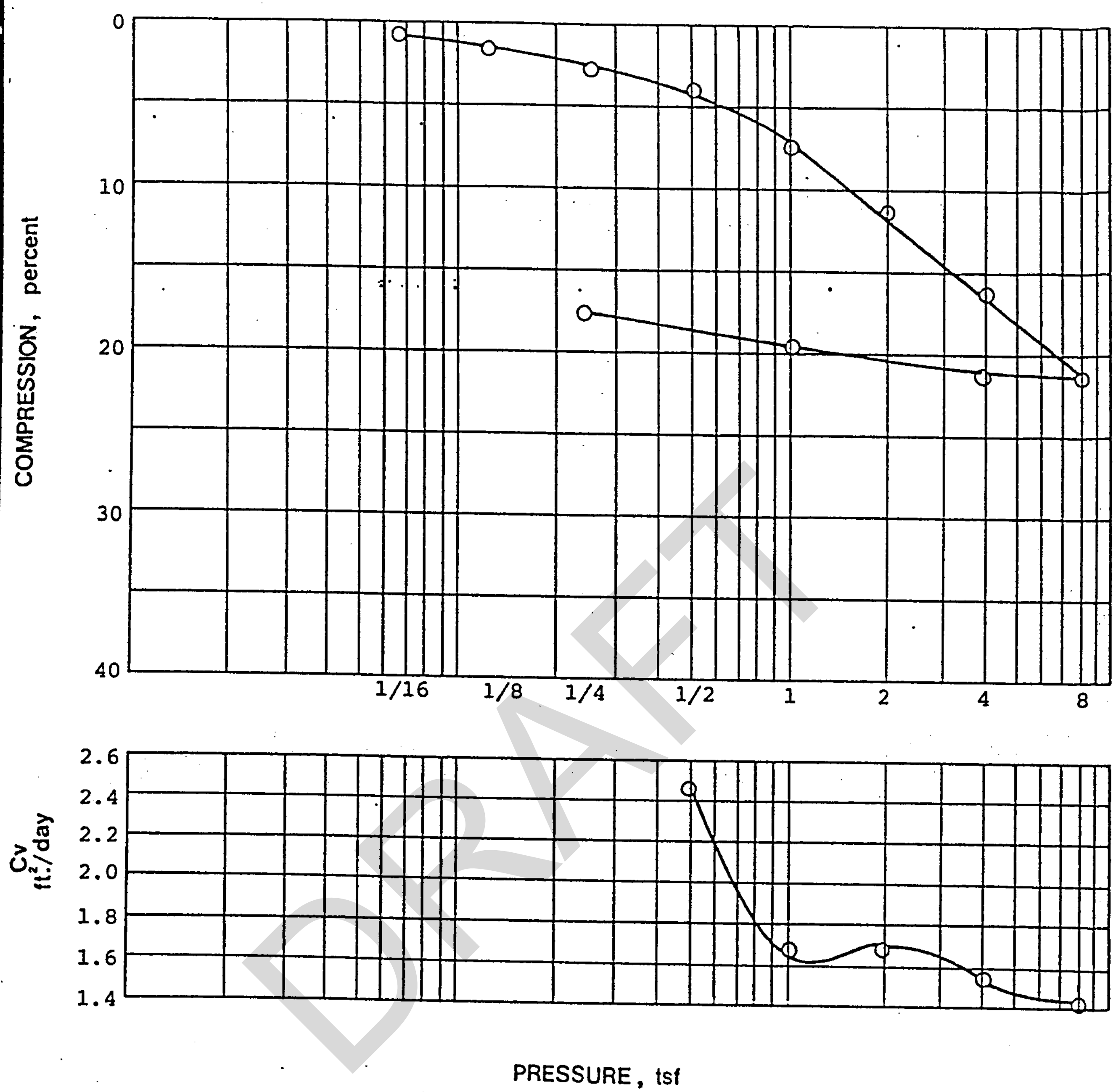
GRAIN SIZE ANALYSES
PIONEER MARINE PLAZA
EVERETT, WASHINGTON

Proj. No. 3691	Date Jan'88	Plate 15



Key	Boring No.	Depth (ft.)	USCS	Soil Description	Liquid Limit %	Plastic Limit %	Plasticity Index %	Moisture Content, W% Before	Moisture Content, W% After	Dry Density (pcf)
O	B-5	5	sm/ pt	mix of silty SAND with PEAT	--	--	--	63.9	47.8	58.8

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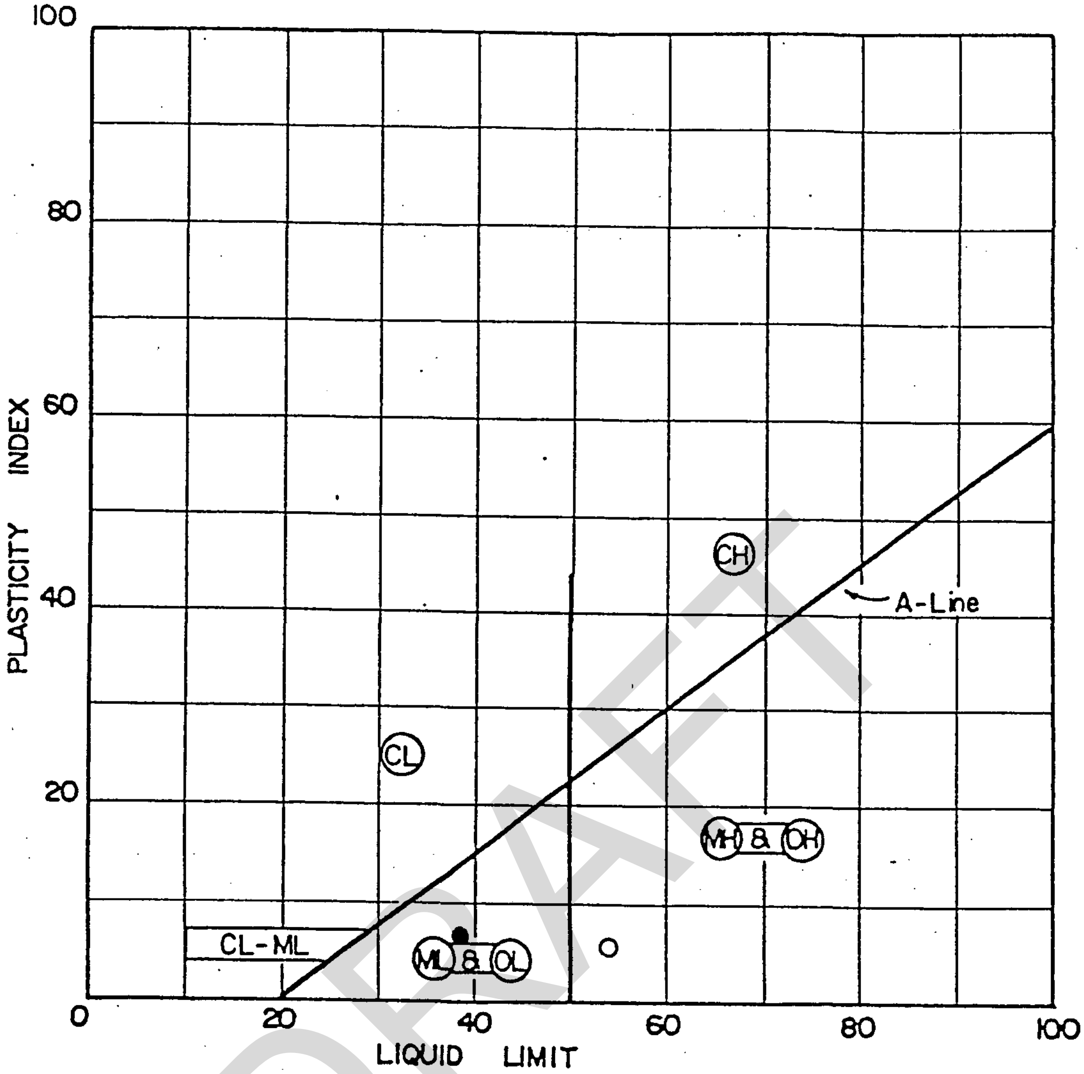
CONSOLIDATION TEST DATA
PIONEER MARINE PLAZA
EVERETT, WASHINGTON

Proj. No. 3691 | Date Jan'88 | Plate 16



Key	Boring/ Test Pit	Depth (ft)	Soil Classification	USCS	L.L.	P.L.	P.I.	Natural Water Content
●	B-2	17.5		ML	39	34	5	
○	TP-1	5.0		MH	53	49	4	

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Atterberg Limits Test Data
PIONEER MARINE PLAZA
EVERETT, WASHINGTON

Proj. No. 3691 | Date Jan'88 | Plate 17

TEST PIT NO. 3

Logged By DA

Date 12/9/87

Elev. 18'±*

Depth (ft.)	USCS	Soil Description	W (%)	
0	sp	(3" crushed rock) Brown and gray SAND, little gravel, trace silt, wet, medium dense	7	
	SM	Dark gray silty SAND, trace wood, wet, medium dense	9	
5			28	
			34	
			◁	
10	ml ol	Dark gray and sandy organic SILT, little sawdust, wet, soft	65	
				qu 0.5 tsf
			99	

Test pit terminated at 12.5 feet below existing grade. Groundwater seepage encountered at 9 feet during excavation.

15

Logged By DA

Date 12/9/87

TEST PIT NO. 4

Elev. 19'±*

Depth (ft.)	USCS	Soil Description	W (%)	
0		(3" crushed rock)		
	sp	Brown and gray SAND fill		
			25	
	sm	Dark gray to black silty SAND, little gravel, wood, rubble and bricks, wet, dense (fill) - slight H_2S odor	18	
5			18	
			49	
10			31	

Test pit terminated at 12 feet below existing grade. No groundwater seepage encountered during excavation.

15

Subsurface conditions depicted represent our observations at the time and location of this exploratory hole, modified by engineering tests, analysis, and judgement. They are not necessarily representative of other times and locations. We cannot accept responsibility for the use or interpretation by others of information presented on this log.

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TEST PIT LOGS
PIONEER MARINE PLAZA
EVERETT, WASHINGTON

Proj. No. 3691	Date Dec'87	Plate 9

TEST PIT NO. 1

Logged By DA

Date 12/9/87

Elev. 17'±*

Depth (ft.)	USCS	Soil Description	W (%)	
0	sp	(3" crushed rock) Brown fine to medium SAND, little gravel, trace silt, medium dense (fill) Geofabric at 3'	16 9	
	sm	Gray silty fine SAND and sawdust, wet, medium dense (fill)	53	
5	MH oh	Dark gray organic SILT, little sand, soft (fill)	75 71	qu<0.5tsf LL=53 PI=4
10	ml	Gray sandy SILT, trace sawdust, wet, soft, low plasticity	97	qu<0.5tsf
15		Test pit terminated at 12 feet below existing grade. Groundwater seepage encountered at 1 foot during excavation.		

Logged By DA

Date 12/9/87

TEST PIT NO. 2

Elev. 16'±*

Depth (ft.)	USCS	Soil Description	W (%)	
0	sp	(3" crushed rock) Brown and gray SAND, little gravel, wet, medium dense (fill)	12	
5 10	ml	Dark gray to gray organic SILT, little sand, little sawdust, wet, soft, low plasticity (Pit caved in: sand filled ditch line.)	68 67 13 75	qu<0.5 tsf
15		Test pit terminated at 11.5 feet below existing grade. No groundwater seepage encountered during excavation.		

Subsurface conditions depicted represent our observations at the time and location of this exploratory hole, modified by engineering tests, analysis, and judgement. They are not necessarily representative of other times and locations. We cannot accept responsibility for the use or interpretation by others of information presented on this log.

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TEST PIT LOGS
PIONEER MARINE PLAZA
EVERETT, WASHINGTON

Proj. No. 3691	Date Jan'88	Plate 8

Appendix B
Geophysical Survey

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Report on MASW and MAM Survey

Port of Everett

1716 West Marine View Drive, Everett, WA

Global Geophysics Project No. 111-0920.000

Prepared for
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1. OBJECTIVE

Global Geophysics LLC conducted a MASW survey along 5 transects and one MAM on September 20, 2021 around 1716 Marine View Drive, Everett, WA. The goal of this investigation was to study the lateral variation of the soft sediment and the average shear wave velocity to a depth of 100 feet. This report provides the methods, instrumentation, data collection and processing procedures, results, and analysis of this investigation.

2. INTRODUCTION

Surface waves are a special type of seismic wave whose propagation is confined to the near surface medium. The depth of subsurface penetration of a surface wave is directly proportional to its wavelength. In a non-homogeneous medium, surface waves are dispersive, i.e. each wavelength has a characteristic velocity stemming from subsurface variations in the soils and rocks. The velocity that the surface waves' wavelengths propagate through the subsurface is related to the shear wave (S-Wave) velocity of the subsurface. If the S-Wave velocity varies with depth, so will the surface wave's wavelength velocity. Analysis of how the wavelength varies, or dispersion, allows us to estimate the S-Wave velocity as it passes through the subsurface. The S-Wave velocity of the subsurface can then be used to infer useful characteristics such as the rock/soil type, stratigraphy, and soil conditions.

Average S-Wave velocities to a depth of 100ft (30m) are known as V_{S100} (V_{S30}) and are sorted into classes by the International Building Code (IBC) to provide valuable earthquake engineering design information. These classes are shown here:

Class Name	Ground Description	V_{S100}	V_{S30}
A	Hard Rock	>5000ft/s	>1500m/s
B	Rock	5000ft/s to 2500ft/s	1500m/s to 760m/s
C	Dense Soil or Soft Rock	2500ft/s to 1200ft/s	760m/s to 360m/s
D	Stiff Soil	1200ft/s to 600ft/s	360m/s to 180m/s
E	Soft Soil	<600ft/s	<360m/s
F	Needs site specific evaluation	NA	NA

Surface waves can be utilized in both active and passive deployments. Multichannel Analysis of Surface Waves (MASW) comprises most active deployments while Microtremor Array Measurements (MAM) are the primary method to collect passive data. MASW arrays are typically linear while MAM arrays can be linear (often known as refraction microtremor, or ReMi, when linear) but generally perform better when deployed in 2D orientations (triangular, circular, T-shaped, or L-shaped arrays). Another passive method employed is the Horizontal over Vertical Spectral Ratio (HVSR) which utilizes a single geophone sensitive to motion in three directions (vertical, east-west, and north-south).

For this project, the seismic survey was deployed at the location shown on Figure 1.

3. METHODOLOGY AND INSTRUMENTATION

3.1 MASW and Microtremor Array Measurements (MAM) Methods

The Multichannel Analysis of Surface Waves (MASW) method determines variations in surface wave velocities with increasing distances and wavelengths. The data from these measurements are used to model the shear wave velocities of the subsurface. This information can then be used to infer rock/soil types, stratigraphy and soil conditions.

The MASW survey requires a seismic source, to generate surface-waves, and at least 24 geophones, to measure the ground response at increasing distances from the source. Surface waves are a special type of seismic wave whose propagation is confined to the near surface medium. The depth of subsurface penetration of a surface-wave is directly proportional to its wavelength. In a non-homogeneous medium, surface-waves are dispersive, i.e. each wavelength has a characteristic velocity stemming from subsurface heterogeneities. The relationship between surface-wave velocity and wavelength is used to calculate the shear-wave velocity of the medium with increasing depth.

MAM survey uses the ambient vibrations. A detailed description of the MAM method can be found in Okada, 2003. MAM arrays generally have a greater degree of flexibility with their design and in addition to linear arrays, can be deployed in 2D arrays such as the circular, triangular, T and L arrays. Since this is a passive survey, the ambient vibrations of the surroundings are utilized rather than deliberately generated. These passive sources can come from all directions and include traffic, ocean waves, cultural noise, and construction. MAM arrays should utilize an array size equal to or greater than the depth of investigation (Geometrics, 2009) and record the ambient vibrations for a minimum of 30 seconds and collect a minimum of 10 minutes of data.

ReMi (Refraction Microtremor) surveys are linear MAM surveys, commonly collected with wired geophones and can be quickly set up and executed. These are useful for collecting data in locations where space is limited or where wireless geophones are not practical, such as inside buildings where GPS signals can be poor or non-existent.

3.2 Surface Wave Dispersion Curve Modeling

Dispersion curves are useful for determining S-Wave velocities of the subsurface and are generated with the help of specialized software. Data files are added to the software and their traces displayed by location versus time, showing the seismic waves that arrive at each geophone over the course of the record.

For MAM surveys, the data are transformed with a fast Fourier transform to the frequency domain. Then the coherence (or similarity between traces or waveforms) is calculated. If the coherences are averaged over a long period of time or over many data blocks, the data is considered to be Spatially Auto-Correlated (SPAC) ((Aki, 1957). From here, the phase velocity can be calculated from each frequency and fundamental and higher modes can be picked. From the fundamental mode, the dispersion curve can be created and edited

(Roesset, 1991). The dispersion curve is used to create an inversion model that displays the S-Wave velocities at the desired range of depths (Xia, Miller, & Park, 1999). Theoretical dispersion curves are generated via a matrix method (Saito & Kabasawa, 1993) and compared against the observed dispersion curve. The model is updated until the observed and theoretical dispersion curves converge. The resulting model is the delivered S-Wave velocity model for the array.

4. INSTRUMENTS AND EQUIPMENT

4.1 MASW/MAM

For this investigation, Global Geophysics used a 24-channel land streamer array of 4.5 Hz geophones at a spacing of 5 feet (distances between geophones were shortened for the soundings inside the storage facility) connected to a Geometrics Geode seismograph. MAM data were collected using ambient seismic waves recorded from the surroundings. Data processing was done with Geometrics SeisImager software package.

5. PROCEDURES

5.1 Field Deployment

MASW 2D S-wave profile

A landstreamer with geophone array was laid out along the transects, KEG-40 was used to pound on the ground to generate vibrations. After on screen QA/QC, the array was moved 20 ft to a new location, and used the source to pound on the ground again.

MAM Sounding

The sounding was located in the parking lot. The survey had an array size of 301 feet Geophone deployment can be seen in Figure 1.

5.2 Data Processing

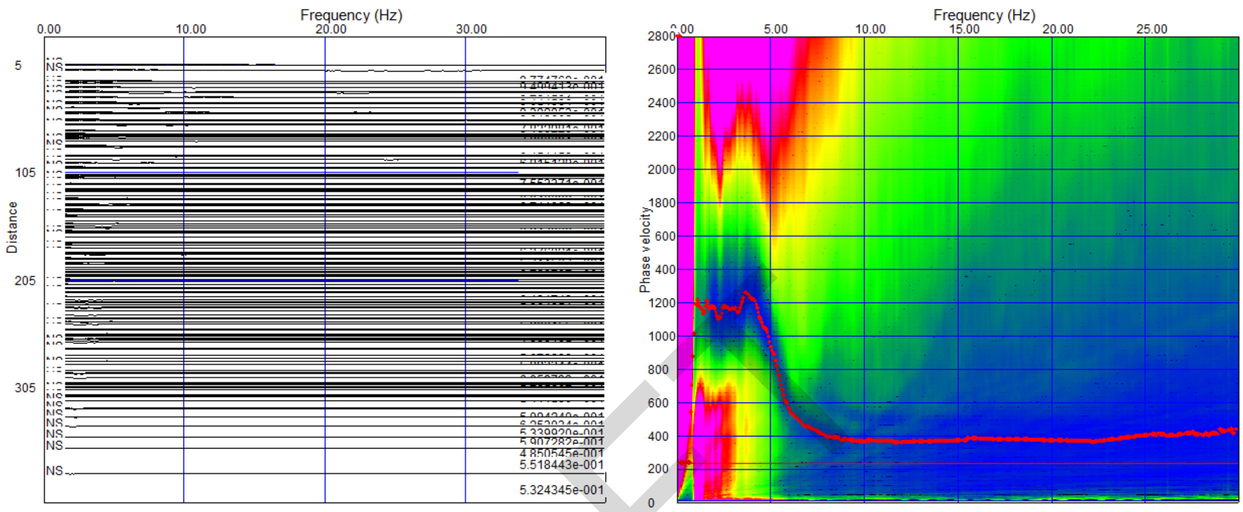
5.2.1 Data Processing

Data were processed using SeisImager. A dispersion curve is generated from the data and the fundamental mode is picked with the assistance of the software. Uncertain data at high and low frequencies are clipped. The dispersion curve is inverted with the Wave EQ program within SeisImager and an initial model is generated. The model is improved by using a Least Square Method inversion with at least 5 iterations. A series of 1D soundings along the transects are combined to make 2D profiles. While 1D deep sounding was processed separately to obtain Vs100.

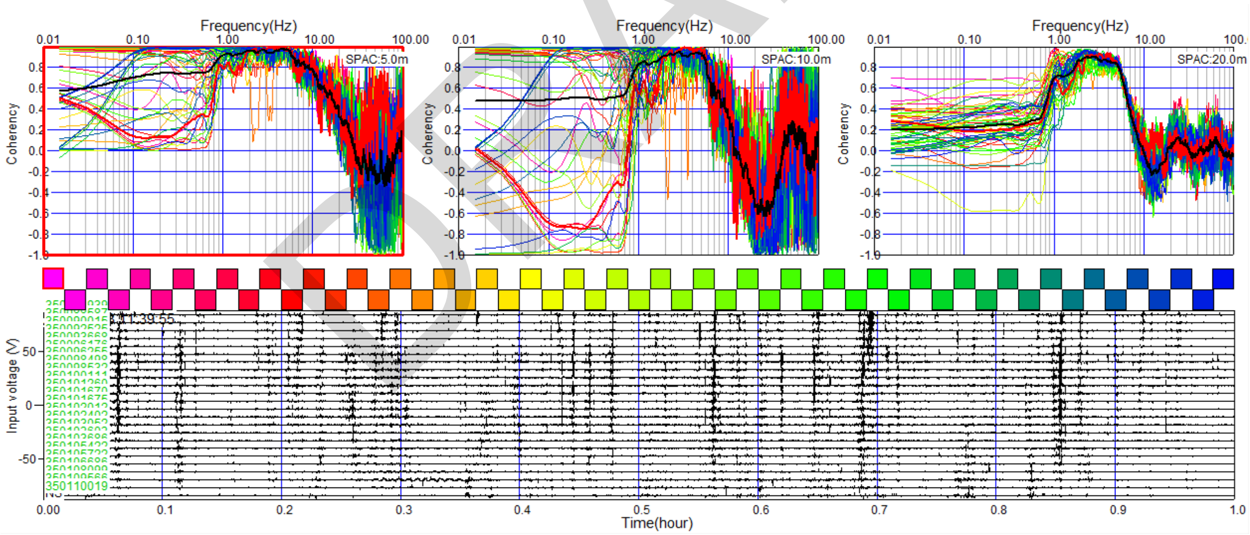
6. RESULTS

The S-wave velocity profiles along Lines 1-5 are presented in Figure 2.

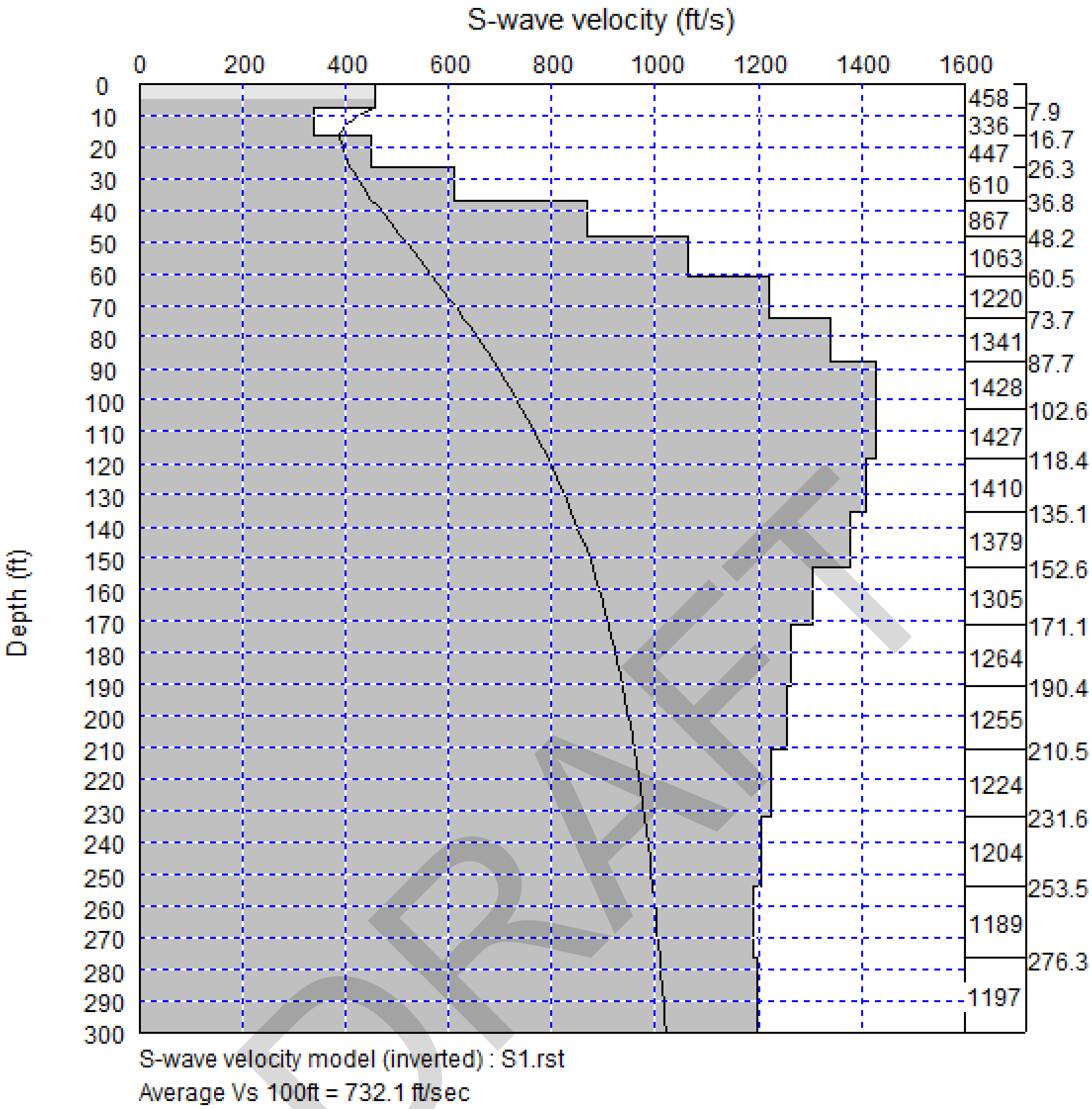
The dispersion curve, coherency and model for the 1D MAM sounding are shown below:



1D MAM Sounding Dispersion Curve



1D MAM Sounding Coherency



1D MAM Sounding Model

Depth(ft)	S-wave velocity(ft/s)
0	458
8	337
17	448
26	611
37	867
48	1063
61	1220
74	1342
88	1428
103	1428
118	1410
135	1379
153	1306
171	1264
190	1256
211	1225
232	1205
254	1190
276	1198
300	1198

Table 1. S-Wave Velocities by Depth for Sounding

7. ANALYSIS

MASW

The majority of penetration with and KEG-40 source is approximately 60-70 ft. The profiles show a layer of soft material across the whole site.

MAM

The dispersion curve with the fundamental mode was picked in red. The coherency is greater than 0.8 near 1 Hz. The shear wave velocity profile to a depth of 300 feet. Table 1 shows the calculated shear wave velocities of the subsurface at specific depths. The V_{S100} value for this sounding was 732.1 ft/s.

8. LIMITATIONS OF THE GEOPHYSICAL METHOD

Global Geophysics' services are conducted in a manner consistent with the level of care and skill ordinarily exercised by other members of the geophysical community currently practicing under similar conditions and are subject to the time limits, financial and physical constraints applicable to the services. MASW and MAM are remote sensing geophysical methods that may not detect all subsurface conditions due to the limitations of the method, soil conditions, size of features, and their depths.

Sincerely,

Global Geophysics, LLC.

Alex Kover
Staff Geophysicist

John Liu, Ph.D., R.G.
Principal Geophysicist

9. REFERENCES

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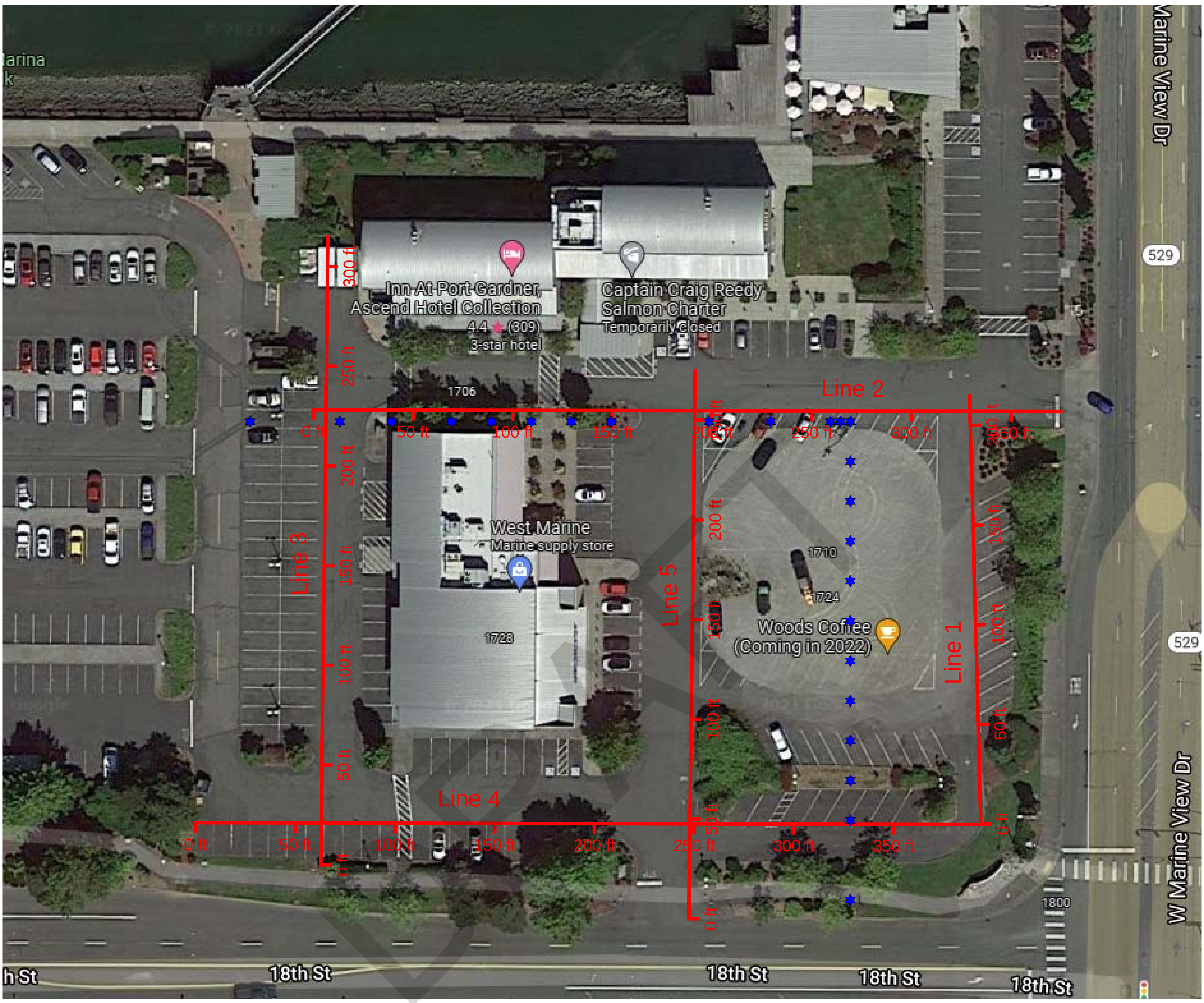


Legend:

MASW Transect

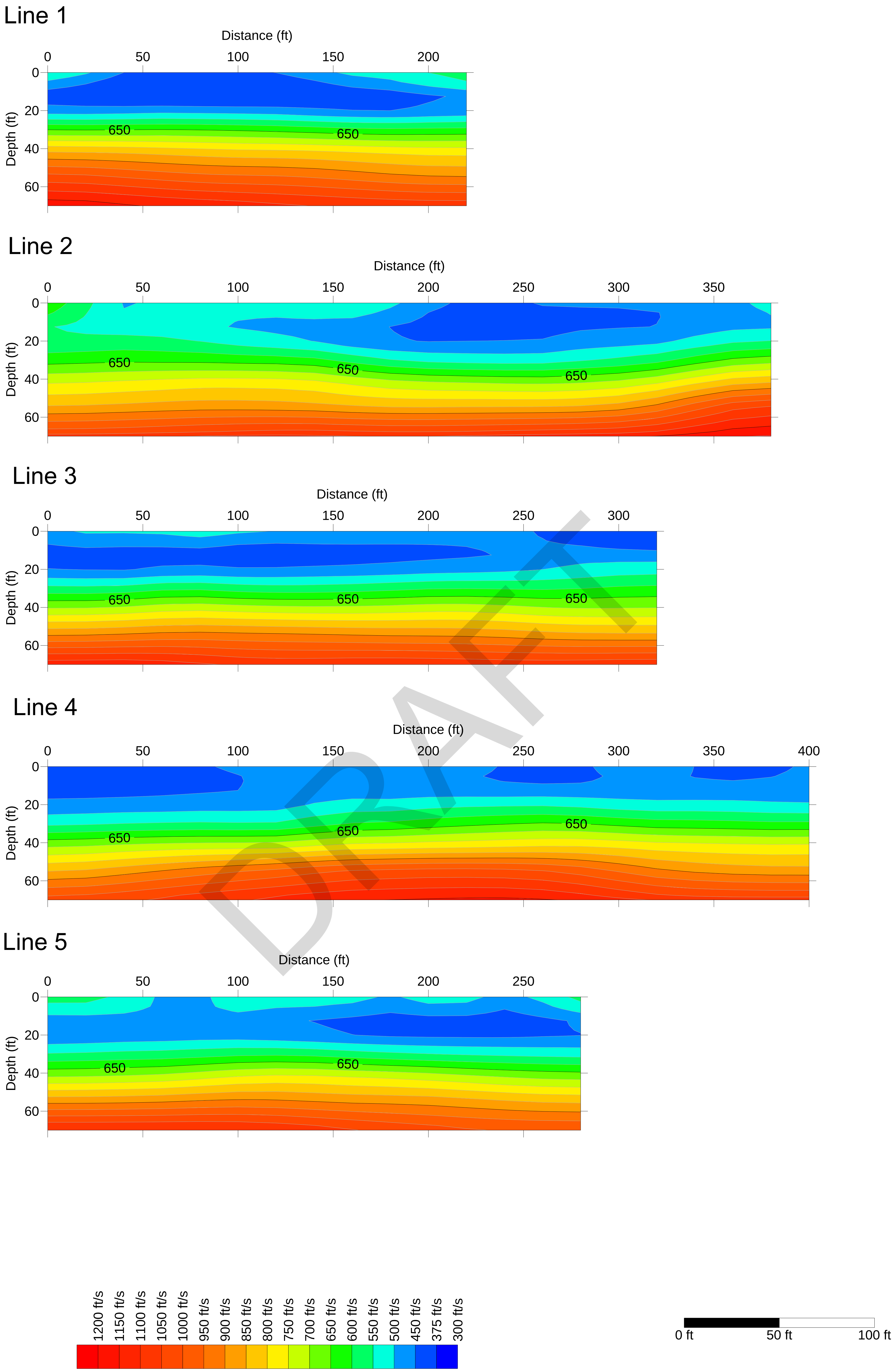
Wireless sensor location

0 ft 60 ft 120 ft

PROJECT	**MASW and ReMi Surveys at 1716 West Marine View Drive, Everett, WA**
TITLE	**Site Plan**

Global Geophysics
P.O. Box 2229
Redmond, WA 98073-2229
Tel: 425-890-4321

Project #: 111-0922.000		FILE No.
DESIGN	--	SCALE AS SHOWN REV.
CADD	AK	**FIGURE 1**
CHECK	JL	
REVIEW	--	



PROJECT

MASW and ReMi Surveys at
1716 West Marine View Drive, Everett, WA

TITLE

S-wave Velocity Profiles

Global Geophysics
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Project #: 111-0922.000		FILE No.	
DESIGN	--	SCALE AS SHOWN	REV.
CADD	AK		
CHECK	JL		
REVIEW	--		

FIGURE 2

APPENDIX C
Report Limitations and Guidelines for Use

APPENDIX C
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.

Geotechnical Services Are Performed for Specific Purposes, Persons and Projects

This report has been prepared for the exclusive use of Port of Everett and PND Engineers, Inc. This report may be made available to prospective contractors for their bidding or estimating purposes, but our report, conclusions and interpretations should not be construed as a warranty of the subsurface conditions. This report is not intended for use by others, and the information contained herein is not applicable to other sites.

GeoEngineers structures our services to meet the specific needs of our clients. For example, a geotechnical or geologic study conducted for a civil engineer or architect may not fulfill the needs of a construction contractor or even another civil engineer or architect that are involved in the same project. Because each geotechnical or geologic study is unique, each geotechnical engineering or geologic report is unique, prepared solely for the specific client and project site. Our report is prepared for the exclusive use of our Client. No other party may rely on the product of our services unless we agree in advance to such reliance in writing. This is to provide our firm with reasonable protection against open-ended liability claims by third parties with which there would otherwise be no contractual limits to their actions. Within the limitations of scope, schedule and budget, our services have been executed in accordance with our Agreement with the Client and generally accepted geotechnical practices in this area at the time this report was prepared. This report should not be applied for any purpose or project except the one originally contemplated.

A Geotechnical Engineering or Geologic Report Is Based on a Unique Set of Project-Specific Factors

This report has been prepared for the proposed Port of Everett Waterfront Place Central PGL Site project in Everett, 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, do not rely on this report if it was:

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

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

- The function of the proposed structure,
- Elevation, configuration, location, orientation or weight of the proposed structure,

[1] Developed based on material provided by GBA, Geoprofessional Business Association; www.geoprofessional.org.

- Composition of the design team, or
- 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.