REPORT OF GEOTECHNICAL ENGINEERING SERVICES

St. Helens Riverwalk Phase I
3 Strand Street
St. Helens, Oregon

For
Mayer/Reed, Inc.
December 16, 2021

Project: StHelens-4-01

NV5

December 16, 2021

Mayer/Reed, Inc.
319 SW Washington Street, Suite 820
Portland, OR 97204

Attention: Jeramie Shane

Report of Geotechnical Engineering Services
St. Helens Riverwalk Phase I
3 Strand Street
St. Helens, Oregon
Project: StHelens-4-01

NV5 is pleased to submit this geotechnical engineering report for the proposed St. Helens Riverwalk Phase I project, including the associated Columbia View Park Amphitheater, located at 3 Strand Street in St. Helens, Oregon. Our services for this project were conducted in accordance with our subconsultant agreement with Mayer/Reed, Inc. executed on September 21, 2021.

We appreciate the opportunity to be of service to you. Please call if you have questions regarding this report.

Sincerely,

NV5

Shawn M. Dimke, P.E., G.E.
Principal Engineer

cc: Shannon Simms, Mayer/Reed, Inc. (via email only)
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SMD:kt
Attachments
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EXECUTIVE SUMMARY

This section provides a summary of the subsurface conditions and a discussion of the geotechnical considerations associated with development of the proposed St. Helens Riverwalk Phase I project, including the associated Columbia View Park Amphitheater, at the location shown on Figure 1 in St. Helens, Oregon. This summary is an overview and the report should be referenced for a thorough discussion of the subsurface conditions and geotechnical recommendations for the project.

SUBSURFACE CONDITIONS

Subsurface conditions generally consist of 20 to 35 feet of loose to medium dense sand with silt to silty sand fill underlain by alluvial silt, with basalt or weathered basalt gravel beginning at depths between 35 and 78.5 feet BGS. The alluvial silt varies from very soft to medium stiff. The depth to basalt increases with proximity to the Columbia River. The groundwater depth is expected to correspond closely with the water level in the Columbia River.

CONCLUSIONS

The primary geotechnical considerations for the site are summarized as follows:

- We estimate liquefaction-induced settlement of up to 3 inches at the ground surface from design-level seismic events. We also estimate lateral spreading of up to 8 feet near the edge of the riverbank toward the Columbia River for a design-level subduction zone event. The estimated maximum lateral spreading potential decreases farther inland from the riverbank to approximately 2.5 feet near the location of the existing gazebo stage structure. Buried remnant wood piling may reduce the lateral spreading potential at the site, but the condition and spacing of the wood piling are unknown; therefore, its effect cannot be quantified. Large lateral spreading close to the edge of the riverbank will also increase the settlement potential to greater than the estimated liquefaction-induced settlement.

- The estimated lateral spreading exceeds allowable tolerances for buildings prescribed by ASCE 7-16, so mat foundations are recommended for support of the new stage and pavilion structures. The mat foundations should include longitudinal reinforcement in both directions at the top and bottom to limit differential movement and protect life and safety. Alternately, ground improvement can be conducted to reduce the liquefaction-induced lateral spreading potential and/or the structures can be supported on deep foundations designed for the liquefaction downdrag and lateral spreading-induced loading.

- If more substantial structures will be designed to be serviceable after a significant seismic event, they will require deep foundations designed for liquefaction-induced downdrag and lateral spreading loads and/or with ground improvement to mitigate the liquefaction and lateral spreading potential. If deep foundations are used, they may need to extend down to the underlying basalt, which was encountered at depths between 35 and 78.5 feet BGS, particularly if they will be designed for liquefaction-induced loading.

- We understand grid-reinforced walls with rock-filled gabion or similar style facings, designed not to be serviceable after liquefaction and lateral spreading from a significant seismic event, are planned to construct the two overlooks in the Riverwalk Phase I area. Based on our analyses, the grid reinforcement lengths measured from the face of the walls should extend back 1.4 times the adjacent wall heights to satisfy static global stability requirements. We recommend embedding the face of the walls a minimum of 18 inches or as required to achieve a minimum horizontal offset of 5 feet. The walls will likely fail from seismic-induced liquefaction and lateral spreading, but the reinforcing geogrids and structural backfill are expected to limit deformations to satisfy life safety requirements. Further recommendations are provided in the "Retaining Structures" section.

- Imported granular material is recommended for the retaining wall backfill. We recommend surcharging all wall backfill areas where finish grades will be more than 1 foot above existing grades to limit post-construction settlement. The surcharge should consist of a minimum of 4 feet of fill material above finish grades and should be in place for a minimum duration of four weeks or until settlement data indicates no further settlement. As an alternate to surcharging, cellular concrete with a maximum density of 35 pcf and minimum unconfined compressive strength of 80 psi could alternately be used for the wall backfill to limit post-construction settlement to less than 1 inch. If cellular concrete is used, a minimum 8-inch-thick zone of drain rock sandwiched in geotextile drainage fabric should extend along the base and cut slope for the backfill.

- We recommend waiting a minimum of four weeks or until survey data indicates no further settlement after placement of the planned lawn fill before constructing any settlement-sensitive hardscapes or other features within 10 feet of the lawn fill berm to reduce the potential for post-construction settlement.

- Ground improvement methods such as stone columns, RAPs, vibro compaction, and deep soil mixing are potential methods to mitigate liquefaction and lateral spreading. Buried remnant wood pilings are anticipated in the former dock area shown on Figure 2. The buried pilings will obstruct drilling and penetrations making ground improvement more challenging, particularly for deep soil mixing. Stone columns or RAPs may be the most feasible liquefaction mitigation method for the site. If requested, NV5 can provide the names of several specialty contractors who can be consulted on the best and least expensive options for liquefaction mitigation.

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APPENDICES

ACRONYMS AND ABBREVIATIONS

ASCE	American Society of Civil Engineers
ASTM	American Society for Testing and Materials
BGS	below ground surface
CPT	cone penetration test
g	gravitational acceleration (32.2 feet/second^2)
GPS	global positioning system
H:V	horizontal to vertical
MCE	maximum considered earthquake
MSE	mechanically stabilized earth
OSHA	Occupational Safety and Health Administration
OSSC	Oregon Standard Specifications for Construction (2021)
pcf	pounds per cubic foot
pci	pounds per cubic inch
psf	pounds per square foot
psi	pounds per square inch
RAP	rammed aggregate pier
SPT	standard penetration test

1.0 INTRODUCTION

NV5 is pleased to submit this geotechnical engineering report for the proposed St. Helens Riverwalk Phase I project, including the Columbia View Park Amphitheater improvements, located at 3 Strand Street in St. Helens, Oregon. Figure 1 shows the site relative to existing topographic and physical features. Figure 2 shows the existing site features and topography and the approximate locations of our explorations. Acronyms and abbreviations used herein are defined above, immediately following the Table of Contents. All elevations referenced in this report are relative to North American Vertical Datum 88.

We understand plans include new hard surface paths and improvements and two new overlooks along the Columbia River, which will be constructed with grid-reinforced walls that have rock-filled gabion or similar style facing. The existing gazebo stage will also be removed and replaced with a new multi-purpose pavilion and reoriented stage structure with an expanded lawn seating berm. The extent of Riverwalk Phase I includes all of the section through Columbia View Park and a small section on the Veneer Property south of Columbia View Park. The existing Columbia View Park Amphitheater seating, which was constructed into the hillside of the park, will remain.

Cuts and fills for Riverwalk Phase I and improvements to the Columbia View Park Amphitheater are expected to be less than a few feet each, except for the new berm seating that may include fills of up to approximately 4 feet and fills at the new overlooks that may range up to approximately 13 feet. Loads for the new stage structure and ancillary structures are expected to be relatively light, with concentrated and/or continuous loads of less than 25 kips and 3 kips per foot, respectively.

2.0 PURPOSE AND SCOPE

The purpose of our geotechnical engineering services was to provide an understanding of the subsurface conditions and geotechnical engineering recommendations for use in design and construction of the proposed project. Specifically, we have performed the following tasks:

- Reviewed available geotechnical and geologic information for the site area from our in-house project files.
- Reviewed historical aerial photographs to help identify the potential locations of buried remnant wood piling at the site.
- Coordinated and managed the field explorations, including private and public utility locates, access preparation, and scheduling contractors and NV5 staff.
- Conducted the following subsurface explorations at the site:
  - Advanced three CPT probes (CPT-1 and CPT-2 [April] and CPT-1 [September]) to refusal at depths between 40.4 and 78.9 feet BGS. Performed pore pressure dissipation testing in each CPT probe to assist in evaluating the groundwater depth.
  - Drilled two borings (B-1 and B-2) to depths between 85.4 and 105 feet BGS.
- Completed laboratory analyses on disturbed and undisturbed soil samples collected from the explorations as follows:
  - Twenty-two moisture content determinations in general accordance with ASTM D2216
  - Seven dry density determinations in general accordance with ASTM D7263

    - Three Atterberg limits tests in general accordance with ASTM D4318
    - Four particle-size analyses in general accordance with ASTM D1140
    - Two consolidation tests in general accordance with ASTM D2435
- Provided recommendations for a mat foundation to support the new amphitheater stage and pavilion and shallow foundations to support other lightly loaded non-building structures.
- Provided design criteria recommendations for retaining walls, including lateral earth pressures, backfill, compaction, and drainage.
- Provided design calculations for a grid-reinforced, rock-filled gabion or similar style-faced retaining wall.
- Provided recommendations for site preparation, grading and drainage, stripping depths, fill type for imported material, compaction criteria, trench excavation and backfill, use of on-site soil, and wet/dry weather earthwork.
- Estimated consolidation settlement potential and provided surcharge recommendations for fill areas to limit post-construction settlement.
- Provided recommendations for permanent and temporary slopes.
- Provided recommendations for preparation of the subgrade for hardscapes.
- Provided recommendations for managing identified groundwater conditions that may affect the performance of structures.
- Evaluated the potential for liquefaction and lateral spreading at the site and provided mitigation options should mitigation be required.
- Provided seismic design parameters in accordance with the ASCE 7-16.
- Prepared this geotechnical engineering report summarizing the results of our geotechnical evaluation and recommendations.

3.0 SITE CONDITIONS

3.1 GEOLOGIC SETTING

The site is located in the northwestern portion of the Portland Basin physiographic province, which is bound by the Tualatin Mountains to the west and south and the Cascade Range to the east and north. The near-surface geologic unit is mapped as alluvial deposits left by the Missoula and Bonneville floods. Based on the development history of the riverbank and exploration results, undocumented fill material overlies the alluvial deposits. The fill and alluvium are underlain by basalt flows belonging to the Sentinel Bluffs member of the Columbia River Basalt Group. The Miocene aged (20 million to 10 million years ago) Columbia River Basalt Group is a series of basalt flows that originated from southeastern Washington and northeastern Oregon (Evarts, 2004). The Sentinel Bluffs basalt flows are reported to be up to 300 feet thick and are considered the geologic basement unit for this report.

3.2 SURFACE CONDITIONS

The site is located on the western bank of the Columbia River in St. Helens, Oregon. Strand Street is west of the site; the Columbia County Courthouse is northwest of the site; a parking lot borders the northern side of the site; and vacant, City of St. Helens-owned property borders the southern side of the site. The site includes the eastern portion of Columbia View Park and a waterfront area extending approximately 100 feet into the vacant waterfront area south of the park. A dock is accessed by a ramp at the northeastern corner of the site. An existing gazebo stage structure is in the north-central portion of the site adjacent to stepped seating built into a bank on the western side of the park. A ramp slopes down gently from Strand Street to a playground and splash pad at the southern end of the site. The flat portion of the site extending south of Columbia View Park is surfaced with gravel. The park includes restrooms at the northwestern corner and a concrete patio at the southwestern corner, which are not considered part of the project area. The site is located east of Strand Street below the stepped amphitheater seating and a retaining wall. Most of the site is a relatively flat bench for the park with elevations ranging from 27 to 30 feet. The riverbank at the eastern edge of the site slopes down to an elevation of 9 feet at grades between approximately 1.5H:1V and 2H:1V. The park is vegetated with grass, shrubs, and trees. The riverbank is vegetated with grass, brush, and blackberries. Variable sizes of riprap are present along the lower section of the riverbank. The tops of remnant wooden piles protrude through the surface of the riverbank. The outline of the former dock based on a 1948 aerial photograph at the site is shown on Figure 2.

3.3 SUBSURFACE CONDITIONS

We explored subsurface conditions at the site by drilling two borings (B-1 and B-2) to depths between 85.4 and 105 feet BGS and advancing three CPT probes (CPT-1 and CPT-2 [April] and CPT-1 [September]) to refusal at depths between 40.4 and 78.9 feet BGS. Three borings (B-1 through B-3) were drilled at the site in 2003 by West Coast Geotech, Inc. Exploration locations are shown on Figure 2. A description of our boring explorations and laboratory testing program, the boring logs, and the results of laboratory testing are presented in the Appendix A. A description and the results of the CPT probes are presented in Appendix B. The 2003 boring logs are presented in Appendix C.

Subsurface conditions generally consist of 20 to 35 feet of loose to medium dense sand with silt to silty sand fill underlain by alluvial silt, with basalt or weathered basalt gravel beginning at depths between 35 and 78.5 feet BGS. The alluvial silt varies from very soft to medium stiff. The depth to basalt increases with proximity to the Columbia River. A basalt face is exposed between the parking lot and courthouse building immediately north of the site, so basalt is likely very shallow near the western edge of the site.

3.3.1 Fill

Undocumented fill was encountered to depths of up to 35 feet BGS. We encountered 4.5 to 5.5 feet of medium dense gravel with cobbles and boulders at the surface in borings B-1 and B-2. Below the gravel, the undocumented fill generally consists of very loose to medium dense sand with silt. Stiff silt and loose, silty sand fill were also encountered from 18 to 22 feet BGS and 22 to 28 feet BGS in boring B-2, respectively. The tested moisture content of the silt and sand fill ranged from 24 to 52 percent at the time of our explorations.

3.3.2 Native Silt and Silty Sand

Very soft to medium stiff silt was generally encountered underlying the fill. Loose, silty sand was also encountered from approximately 37 to 42.5 feet BGS in boring B-1. Atterberg limits testing indicates the silt generally exhibits moderate plasticity but varies from non-plastic to moderately plastic. Consolidation testing indicates the silt is slightly over consolidated and moderately compressible. The tested moisture content of the silt and silty sand ranged from 35 to 59 percent at the time of our explorations.

3.3.3 Basalt

Basalt underlies the silt at depths that increase with proximity to the river. The depths to refusal for the CPTs or to basalt or dense gravel (decomposed to weathered basalt) varied from 35 feet to 105 feet BGS as indicated for each exploration location on Figure 2.

3.4 GROUNDWATER

Groundwater was measured through pore pressure dissipation testing in CPT probes CPT-1 and CPT-2 (April) and CPT-1 (September). The depths to groundwater measured from the CPTs ranged between 20.5 and 22.2 feet BGS (approximately 7 and 6 feet in elevation) at the time of the explorations. Mud rotary drilling methods prevented groundwater observations in the recent borings. Groundwater was measured at a depth of approximately 18 feet BGS in the borings drilled at the site in May 2003. The depth to groundwater is expected to correspond closely with the adjacent Columbia River level, which has an ordinary low water elevation of 2.9 feet and an ordinary high water elevation of 14.6 feet.

4.0 SEISMIC CONSIDERATIONS

4.1 LIQUEFACTION POTENTIAL

Liquefaction is a phenomenon caused by a rapid increase in pore water pressure that reduces the effective stress between soil particles to near zero. The excessive buildup of pore water pressure results in the sudden loss of shear strength in a soil. Granular soil, which relies on interparticle friction for strength, is susceptible to liquefaction until the excess pore pressures can dissipate. Sand boils and flows observed at the ground surface after an earthquake are the result of excess pore pressures dissipating upwards, carrying soil particles with the draining water. In general, loose, saturated sand soil with low silt and clay content is the most susceptible to liquefaction. Low plasticity, sandy silt may be moderately susceptible to liquefaction under relatively high levels of ground shaking.

We performed our liquefaction analysis using the CPT probe data and CLiq 2.0 software. For our analysis, we assumed groundwater depths ranging from 14 to 15.5 feet BGS, corresponding to the approximate ordinary Columbia River high water elevation.

Based on our analysis, sand and sandy soil below the groundwater table is subject to liquefaction from design-level earthquake events. We estimate liquefaction-induced settlement of up to 3 inches at the ground surface from design-level seismic events.

4.2 LATERAL SPREADING

Lateral spreading is a liquefaction-related seismic hazard and occurs on gently sloping or flat sites underlain by liquefiable sediment adjacent to an open face, such as a riverbank, seawall, or pond. Liquefied soil adjacent to an open face can flow toward the open face, resulting in lateral ground displacement. Liquefied soil flows downslope or to an exposed bank much like a viscous fluid.

Our analysis indicates significant potential for lateral spreading at the site toward the Columbia River during a design-level seismic event. We estimate maximum lateral spreading of approximately 5 to 8 feet near the top of the riverbank (Bray et al., 2018) for a design-level subduction zone earthquake. The lateral spreading potential decreases farther inland from the bank to an estimated maximum of approximately 2.5 feet near the location of the existing gazebo stage structure (Dickenson, 2018). We evaluated the slope stability of the riverbank and the yield accelerations for estimating the lateral spreading potential using the limit equilibrium computer program SLOPE/W. Plots showing the input and results of our SLOPE/W analyses are presented in Appendix D. Buried remnant wood piling may reduce the lateral spreading potential at the site, but the condition and spacing of the wood piling are unknown; therefore, its effect cannot be quantified. The estimated lateral spreading exceeds the allowable tolerances prescribed by ASCE 7-16, so a mat foundation is recommended for support of the new stage structure and pavilion. Mitigation for lateral spreading is discussed in the "Design and Mitigation for Liquefaction Hazards" section.

4.3 SEISMIC DESIGN PARAMETERS

Seismic design is prescribed by ASCE 7-16. Table 1 presents the site design parameters prescribed by ASCE 7-16 for the site assuming structures are supported as recommended in the "Foundation Support" section and the risk of liquefaction-induced settlement and lateral spreading is acceptable or the lateral spreading potential is mitigated. Due to the presence of liquefiable soil, the Site Class is F; however, the design parameters for Site Class D provided below can be used per ASCE 7-16, provided the fundamental period of the structures is 0.5 second or less.

Table 1. Seismic Design Parameters

Parameter	Short Period (T_s = 0.2 second)	1 Second Period (T_1 = 1.0 second)
MCE Spectral Acceleration, S	S_s = 0.829 g	S_1 = 0.398 g
Site Class	F*	
Site Coefficient, F	F_a = 1.20	F_v = 1.90
Adjusted Spectral Acceleration, S_M	S_{MS} = 0.995 g	S_{M1} = 0.757 g
Design Spectral Response Acceleration Parameters, S_D	S_{DS} = 0.664 g	S_{D1} = 0.505 g

* The above parameters provided for Site Class D can be used, provided the structures have a fundamental period of 0.5 second or less per ASCE 7-16 Section 20.3.1 and the seismic response coefficient (C_s) is determined according to the exception in ASCE 7-16 Section 11.4.8 or else a site-specific response analysis will be required.

5.0 DESIGN AND MITIGATION FOR LIQUEFACTION HAZARDS

Considering the liquefaction and lateral spreading potential, we recommend supporting the new stage and pavilion structures on mat foundations as recommended in this report. If more substantial structures will be designed to be serviceable after a significant seismic event, they will require deep foundations designed for liquefaction-induced downdrag and lateral spreading loads and/or ground improvement to mitigate the liquefaction and lateral spreading potential. An open cell sheet pile structure may be another way to potentially retain fill material near the riverbank and limit lateral spreading.

As discussed previously, sand below the groundwater level at the site will likely liquefy and cause lateral spreading of the site soil toward the Columbia River. Ground improvement methods such as stone columns, RAPs, compaction, and deep soil mixing may be used to mitigate liquefaction and lateral spreading. Buried remnant wood pilings at the site will obstruct the indicated ground improvement methods and are expected to be even more challenging for deep soil mixing. Vibro compaction consists of densifying granular soil with a vibrating probe. Vibro compaction is typically only feasible in clean, granular soil. The silt content of the sand at the site is likely too high for vibro compaction. Additional drilling and laboratory testing could be conducted to better evaluate the silt content of the sand.

Stone columns or RAPs may be the most feasible liquefaction mitigation methods for the site. RAPs are installed by excavating columns of soil and replacing them with compacted gravel. This system is typically limited to depths between 20 and 30 feet BGS. Stone columns are constructed by inserting a vibrating probe into the subsurface to the desired depth. When the probe is extracted, the void is backfilled with crushed rock aggregate. Stone columns densify the surrounding matrix soil, reducing the potential for liquefaction. Stone columns are typically placed at spacings of 6 to 10 feet on-center. Where obstructions require stone columns or RAPs to be moved, more elements will be required to maintain minimum spacings between the columns/piers. Cement can be mixed into the crushed rock for stone columns and RAPs for added strength and stiffness to resist lateral spreading.

Ground improvement is typically designed and constructed by a specialty contractor. If requested, NV5 can provide the names of several specialty contractors who can be consulted on the best and least expensive options for liquefaction mitigation. The specialty contractor will also provide associated design and construction services. We recommend that NV5 review the ground improvement design.

6.0 FOUNDATION SUPPORT

6.1 GENERAL

We recommend supporting the new stage and pavilion structures on mat foundations over a minimum 6-inch-thick gravel pad, provided the risk of liquefaction-induced settlement and lateral spreading potential from a seismic event are acceptable. Alternately, ground improvement can be conducted to reduce the liquefaction-induced lateral spreading potential and/or the structures can be supported on deep foundations designed for liquefaction and lateral spreading-induced loading. Similarly, if more substantial structures will be included, deep foundations and/or ground improvement will likely be required.

Based on the results of our explorations and analysis, only non-building structures (e.g., equipment foundations and small retaining walls) for which the estimated liquefaction-induced settlement and lateral spreading discussed in the “Seismic Considerations” section is acceptable without presenting a life and safety hazard can be supported by conventional spread footings constructed on minimum 6-inch-thick gravel pads.

Deep foundations can be installed to support foundation loads. However, liquefied soil will impose downdrag forces and large lateral spreading forces along the shafts of piles unless mitigation is conducted via ground improvement. If deep foundations are used, they may need to extend down to the underlying basalt, which was encountered at depths between 35 and 78.5 feet BGS, particularly if they will be designed for liquefaction-induced loading. If the team would like to explore the option of using deep foundations, NV5 can be contacted to conduct further explorations and provide more detailed recommendations.

6.2 MAT FOUNDATIONS

Mat foundations can be used to support the new stage structure and pavilion near the location of the existing structure, provided the risk of liquefaction-induced settlement and lateral spreading potential is acceptable. The mat foundations should include longitudinal reinforcement in both directions at the top and bottom and should be detailed in accordance with the requirements of Section 18.6.3.1 of American Concrete Institute 318 and to limit differential movement and protect life and safety. Mats should be founded on a 6-inch-thick gravel pad. We estimate that post-construction consolidation-induced settlement of the mat foundation will be less than 1 inch. Liquefaction-induced settlement during the design-level earthquake is expected as discussed in the “Seismic Considerations” and “Design and Mitigation for Liquefaction Hazards” sections. We estimate that differential settlement for mat foundations will be less than one-third of the total liquefaction-induced settlement.

A subgrade reaction modulus of 150 pci can be used to design the mat. Lateral loads can be resisted by passive earth pressure on the sides of the mat foundations and by friction on the bearing surface as discussed in the ”Resistance to Sliding” section.

6.3 SHALLOW FOUNDATIONS

Non-building structures for which the estimated liquefaction-induced settlement and lateral spreading discussed in the “Seismic Considerations” section are acceptable and structures in areas where the lateral spreading hazard has been mitigated can be supported on conventional spread footings constructed on minimum 6-inch-thick gravel pads. If constructed, continuous and isolated spread footings should be at least 16 and 20 inches wide, respectively. The bottoms of exterior footings should be at least 18 inches below the lowest adjacent exterior grade. The bottoms of interior footings should be established at least 12 inches below the base of the slab. Footings established on on-site soil or structural fill soil and prepared as recommended above should be sized based on an allowable bearing pressure of 2,000 psf. This is a net bearing pressure; the weight of the footing and overlying backfill can be ignored in calculating footing sizes. The recommended allowable bearing pressure applies to the total of dead plus long-term live loads and can be increased by one-half for short-term loads such as those resulting from wind or seismic forces.

Based on our analysis and experience with similar soil, total post-construction consolidation-induced settlement under static conditions should be less than 1 inch, with differential settlement of less than ½ inch between footings. This does not include liquefaction-induced settlement that may occur during the design-level earthquake.

6.4 RESISTANCE TO SLIDING

Lateral loads on foundations (excluding deep foundations) can be resisted by passive earth pressure on the sides of the structure and by friction on the base. Our analysis indicates that the available passive earth equivalent fluid pressure for footings confined by on-site soil and structural fill is 325 pcf. Typically, the movement required to develop the available passive resistance may be relatively large; therefore, we recommend using a reduced passive equivalent fluid pressure of 250 pcf. Adjacent floor slabs, pavement, or the upper 12-inch depth of adjacent unpaved areas should not be considered when calculating passive resistance. In addition, in order to rely on passive resistance, a minimum of 10 feet of horizontal clearance must exist between the face of the footings and adjacent downslopes.

For foundations/grade beams in contact with imported granular material, a coefficient of friction equal to 0.40 may be used when calculating resistance to sliding. This value should be reduced to 0.35 for structural elements established over the on-site soil.

7.0 HARDSCAPES

The anticipated liquefaction-induced settlement and lateral spreading could cause severe cracking and likely result in slabs and hardscapes that are unusable after a design-level seismic event. Alternatively, hardscapes can be structurally supported using grade beams spanning to deep foundations and/or the potential liquefaction-induced settlement and lateral spreading can be mitigated via ground improvement as previously discussed.

A minimum 6-inch-thick layer of imported granular material should be placed and compacted over the prepared subgrade for hardscapes. The base rock should be crushed rock or crushed gravel and sand meeting the requirements outlined in the "Structural Fill" section. The base rock should be placed in one lift and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557. Base rock contaminated with excessive fines (greater than 5 percent by dry weight passing the U.S. Standard No. 200 sieve) should be replaced.

8.0 RETAINING STRUCTURES

8.1 GENERAL

Construction of any conventional structural (greater than 4 feet in height) retaining walls will likely require mitigation of the liquefaction-induced lateral spreading hazard as discussed in this report. The wall design parameters provided for structural walls below assume the liquefaction-induced lateral spreading hazard has been mitigated. MSE retaining walls are an option that may be able to be constructed without ground improvement but would not be serviceable after a significant seismic event, as discussed in the "Executive Summary."

8.2 GRID-REINFORCED RETAINING WALLS

Grid-reinforced (MSE) walls with rock-filled gabion or similar style facings, designed not to be serviceable after liquefaction and lateral spreading from a significant seismic event, are planned to construct the two overlooks in the Riverwalk Phase I area. Based on our analyses, the grid reinforcement lengths measured from the face of the walls should extend back 1.4 times the adjacent wall heights to satisfy static global stability requirements. Imported granular material should be used or alternately cellular concrete as detailed in the "Surcharging and Lightweight Fill" section could be used for retaining wall backfill. The soil parameters and output of our stability analyses are presented in Appendix D. We recommend using minimum geogrid spacings of 1.5 feet and Synteen SF35 or an engineer-approved alternative reinforcing geogrid. The results of our internal analysis for the MSE walls are presented in Appendix E. The contractor will need to submit the planned materials and details for the gabion, SierraScape®, or similar style stone-filled wall facing if more detailed requirements are not provided. NV5 can provide design details for a SierraScape® wall if requested. We recommend embedding the face of the walls a minimum of 18 inches or as required to achieve a minimum horizontal offset of 5 feet. The walls will likely fail from seismic-induced liquefaction and lateral spreading, unless the seismic event occurs when the river level and associated groundwater level are low, but the reinforcing geogrids and structural backfill are expected to limit deformations to satisfy life safety requirements. Ground improvement will likely be necessary if movement of the retaining walls will be limited to serviceable amounts after a significant seismic event.

8.3 CONVENTIONAL WALL DESIGN PARAMETERS

Retaining structures free to rotate slightly around the base should be designed for active earth pressures using an equivalent fluid unit pressure of 35 pcf. If retaining walls are restrained against rotation during backfilling, they should be designed for an at-rest earth pressure of 55 pcf. This value is based on the assumptions that (1) the retained soil has a slope flatter than 4H:1V, (2) the backfill is drained, and (3) the walls are less than 8 feet in height. Seismic lateral forces can be calculated using a dynamic force equal to $7H^2$ pounds per linear foot of wall, where H is the wall height. The seismic force should be applied as a distributed load with the centroid located at 0.6H from the wall base. Footings for retaining walls should be designed as recommended for shallow foundations.

If surcharges (e.g., retained slopes, building foundations, vehicles, steep slopes, terraced walls, etc.) are located within a horizontal distance from the back of a wall equal to the height of the wall, additional pressures will need to be accounted for in the wall design. Our office should be contacted for appropriate wall surcharges based on the actual magnitude and configuration of the applied loads.

8.4 CONVENTIONAL WALL DRAINAGE AND BACKFILL

The above design parameters have been provided assuming that back-of-wall drains will be installed to prevent buildup of hydrostatic pressure behind all walls. If a drainage system is not installed, our office should be contacted for revised design forces.

The backfill material placed behind the walls and extending a horizontal distance of ½H, where H is the height of the retaining wall, should consist of imported granular material placed and compacted in conformance with the "Structural Fill" section.

A minimum 4-inch-diameter, perforated collector pipe should be placed at the base of the walls. The pipe should be embedded in a minimum 2-foot-wide zone of angular drain rock that is wrapped in a drainage geotextile fabric and extends up the back of the wall to within 1 foot of the finished grade. The drain rock and drainage geotextile fabric should meet the specifications provided in the "Materials" section. The perforated collector pipes should discharge at an appropriate location away from the base of the wall. The discharge pipe(s) should not be tied directly into stormwater drain systems, unless measures are taken to prevent backflow into the drainage system of the wall.

Settlement of up to 1 percent of the wall height commonly occurs immediately adjacent to the wall as the wall rotates and develops active lateral earth pressures. Consequently, we recommend that construction of flatwork adjacent to retaining walls be postponed at least four weeks after backfilling of the wall, unless survey data indicate that settlement is complete sooner.

9.0 SURCHARGING AND LIGHTWEIGHT FILL

Our analysis indicates backfill for the overlooks may result in up to 2 inches of settlement. We recommend surcharging all wall backfill areas where finish grades will be more than 1 foot above existing grades to limit post-construction settlement. The surcharge should consist of a minimum of 4 feet of fill material above finish grades and should be in place for a minimum duration of four weeks or until settlement data indicates no further settlement. As an alternate to surcharging the wall area, cellular concrete with a maximum density of 35 pcf and minimum unconfined compressive strength of 80 psi could alternately be used for the wall backfill to limit post-construction settlement to less than 1 inch. Plots of our settlement analysis are presented in Appendix D. If cellular concrete is used, it should be placed in maximum lift thicknesses of 4 feet and a minimum 8-inch-thick zone of drain rock sandwiched in geotextile drainage fabric should extend along the base and cut slope for the wall backfill.

10.0 PERMANENT SLOPES

Permanent cut and fill slopes should be no steeper than 2H:1V. Newly constructed fill slopes should be over-built by at least 12 inches and then trimmed back to the required slope to maintain a firm face.

Access roads and pavement should be located at least 5 feet from the top of cut and fill slopes. The setback should be increased to 10 feet for buildings, unless special foundation considerations are implemented. Slopes should be planted with appropriate vegetation to provide protection against erosion as soon as possible after grading. Surface water runoff should be collected and directed away from slopes to prevent water from running down the face of the slope.

11.0 DRAINAGE CONSIDERATIONS

11.1 TEMPORARY

During earthwork at the site, the contractor should be made responsible for temporary drainage of surface water as necessary to prevent standing water and/or erosion at the working surface.

11.2 SITE DRAINAGE

We recommend that all roof drains be connected to a tightline leading to storm drain facilities. Pavement surfaces and open space areas should be sloped such that surface water runoff is collected and routed to suitable discharge points. We also recommend sloping ground surfaces adjacent to structures to facilitate surface drainage away from the structures. Trapped planter areas should not be created adjacent to pavement and structures without providing means for positive drainage (e.g., swales or catch basins).

12.0 SITE DEVELOPMENT RECOMMENDATIONS

12.1 SITE PREPARATION

12.1.1 Demolition

Demolition includes complete removal of existing site improvements within 5 feet of areas to receive new pavement, buildings, retaining walls, or engineered fills. Underground vaults, tanks, manholes, and other subsurface structures should be removed in areas of new improvements. Utility lines can be completely removed or grouted full if left in place. Voids resulting from removal of existing improvements should be backfilled with compacted structural fill, as discussed in the “Structural Fill” section. The bottoms of such excavations should be excavated to expose a firm subgrade before filling and their sides sloped at a minimum of 1.5H:1V to allow for more uniform compaction at the edges of the excavations. Material generated during demolition should be transported off site for disposal or stockpiled in areas designated by the owner. In general, this material will not be suitable for re-use as engineered fill.

12.1.2 Subgrade Evaluation

After required demolition and site cutting have been completed, we recommend proof rolling the subgrade with a fully loaded dump truck or similarly heavy, rubber tire construction equipment to identify areas of excessive yielding, which may be indicative of underlying soft, loose, or unsuitable soil. A member of our geotechnical staff should observe proof rolling to evaluate yielding of the ground surface. Soft or loose zones identified during proof rolling should be excavated and replaced with compacted structural fill.

12.2 CONSTRUCTION CONSIDERATIONS

Sandy soil is prone to raveling under construction and other traffic that will cause the surface sand to become loose. Loose sand and silty soil will provide inadequate support for construction equipment. Haul roads and staging areas can be constructed to support construction traffic over the exposed soil. A 6-inch-thick layer of imported granular material generally should be sufficient for light staging areas and the building pad but generally is not expected to be adequate to support heavy equipment or truck traffic. Haul roads and areas with repeated heavy construction traffic should be constructed with a minimum 12-inch-thick layer of imported granular material. The imported granular material should be placed in one lift over the prepared undisturbed subgrade and compacted using a smooth-drum roller without the use of vibratory action. The recommended thicknesses are intended to be guidelines. Selecting the actual thickness should be the responsibility of the contractor, who has control of the construction traffic loads and frequency.

12.3 EXCAVATION

Conventional earthmoving equipment in proper working condition should be capable of making necessary excavations for pavement, foundations, and utilities. We recommend that excavation be performed by a track-mounted excavator using a smooth-blade bucket.

Excavations in the on-site sand will be prone to raveling. In addition, caving, sloughing, and “running sand” are likely for excavations below the water table. Raveling, caving, sloughing, and “running sand” will result in undermining of adjacent utilities or structures. We recommend that excavations be laid back at an inclination of 1.5H:1V or flatter. Shoring will be required where flattened excavation side slopes are not possible. It may be necessary to use tight-joint, driven sheet piling to control groundwater seepage and loss of ground in trench areas adjacent to existing improvements. If shoring is used, we recommend that the type and design of the shoring system be the responsibility of the contractor, who is in the best position to choose a system that fits the overall plan of operation.

We anticipate that groundwater levels will fluctuate somewhat based on the season and the water levels in the river. Dewatering will be required in excavations that extend below the water table. Because of the tendency for sand and sandy soil to “run,” dewatering measures will likely require well points or pump wells located outside of the trench excavation. However, it may be possible to use a sump located within trench excavations to dewater isolated zones of perched water or shallow limited excavations below the water table.

If groundwater is present in the excavations, we recommend placing at least 1 foot of stabilization material at the base of the excavation. Stabilization material should consist of well-graded gravel, crushed gravel, or crushed rock meeting the requirements outlined in the “Structural Fill” section. Stabilization material should be placed in one lift.

Excavations should be made in accordance with applicable OSHA and state regulations. While this report describes certain approaches to excavation and dewatering, the contractor should be responsible for selecting excavation and dewatering methods, monitoring the excavations for safety, and providing shoring as required to protect personnel and adjacent utilities and structures.

12.4 MATERIALS

12.4.1 Structural Fill

Structural fill should be free of organic material and other deleterious material and, in general, should consist of particles no larger than 3 inches in diameter. Existing concrete debris or remnant concrete structural elements, asphalt concrete pavement, or base rock can be used as structural fill, provided it is environmentally acceptable, is adequately processed as described below for recycled concrete or broken into particles no greater than 3 inches in greatest dimension, and can be incorporated into well-graded structural fill and adequately compacted.

12.4.1.1 On-Site Native Soil

The on-site material is suitable for use as general structural fill, provided it is properly moisture conditioned and free of debris, organic material, and particles over 6 inches in diameter. It should be possible to adequately compact the near-surface sand during periods of light precipitation, but adequate compaction will not likely be achieved during moderate to heavy precipitation. Some moisture conditioning (drying) may be required after periods of moderate to heavy precipitation. Water may need to be added to the on-site sand during the dry summer months to achieve adequate compaction.

When used as structural fill, on-site material should be placed in lifts with a maximum uncompacted thickness of 12 inches and compacted to not less than 92 percent of the maximum dry density, as determined by ASTM D1557.

12.4.1.2 Imported Granular Material

Imported granular material used for structural fill should be pit- or quarry-run rock, crushed rock, or crushed gravel and sand. Imported granular material should be fairly well graded between coarse and fine material, should have less than 5 percent by dry weight passing the U.S. Standard No. 200 sieve, and should have at least two mechanically fractured faces. Material with higher fines content is permissible, provided compaction can be achieved. When used as structural fill, imported granular material should be placed in lifts with a maximum uncompacted thickness of 12 inches and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557.

12.4.1.3 Recycled Concrete

Recycled concrete can be used for structural fill, provided it is environmentally suitable for the proposed application and the concrete is broken to a maximum particle size of 3 inches. This material can be used as trench backfill and pavement base rock if it meets the requirements for imported granular material, which would require a smaller maximum particle size. The material should be placed in lifts with a maximum uncompacted thickness of 12 inches and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557.

12.4.1.4 Aggregate Base Rock

Imported granular material used as base rock for building floor slabs and pavement should consist of ¾-inch-minus material. The aggregate should have less than 5 percent by dry weight passing the U.S. Standard No. 200 sieve and at least two fractured faces. The aggregate base should be compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557.

12.4.1.5 Trench Backfill

Trench backfill for the utility pipe base and pipe zone should consist of well-graded, durable, crushed granular material with a maximum particle size of ¾ inch and less than 5 percent by dry weight passing the U.S. Standard No. 200 sieve. The material should be free of roots, organic material, and other unsuitable material. Backfill for the pipe base and pipe zone should be compacted to at least 90 percent of the maximum dry density, as determined by ASTM D1557, or as recommended by the pipe manufacturer.

Within building, pavement, and other structural areas, trench backfill placed above the pipe zone should consist of imported granular material as specified above. The backfill should be compacted to at least 92 percent of the maximum dry density, as determined by ASTM D1557, at depths greater than 2 feet below the finished subgrade and 95 percent of the maximum dry density, as determined by ASTM D1557, within 2 feet of finished subgrade. In all other areas, trench backfill above the pipe zone should be compacted to at least 90 percent of the maximum dry density, as determined by ASTM D1557.

12.4.1.6 Stabilization Material

Stabilization material should consist of pit- or quarry-run rock, crushed rock, or crushed gravel and sand that consist of 4- to 6-inch-minus material. It should have less than 5 percent by dry weight passing the U.S. Standard No. 4 sieve and at least two mechanically fractured faces. The material should be free of organic material and other deleterious material. Stabilization material should be placed in one lift and compacted to a firm condition.

Where the stabilization material is used to stabilize soft subgrade beneath pavement or construction haul roads, a geotextile should be placed as a barrier between the soil subgrade and the imported granular material. The geotextile fabric should meet the specifications provided below for subgrade geotextiles. Geotextile is not required where stabilization material is used at the base of utility trenches.

12.4.1.7 Drain Rock

Drain rock should consist of granular material that meets the specifications provided in OSSC 00430.11 (Granular Drain Backfill Material). In addition, the drain rock should be angular, should be well graded between coarse and fine material, should have less than 2 percent by dry weight passing the U.S. Standard No. 200 sieve, and should have at least two mechanically fractured faces. The drain rock should be wrapped in a drainage geotextile that meets the specifications provided below for drainage geotextiles.

12.4.2 Geotextile Fabric

12.4.2.1 Separation Geotextile Fabric

A separation geotextile fabric can be placed as a barrier between silty subgrade and granular material in staging areas, haul road areas, or in areas of repeated construction traffic. The subgrade geotextile should meet the requirements in OSSC 02320 (Geosynthetics) for subgrade geotextiles and be installed in conformance with OSSC 00350 (Geosynthetic Installation).

12.4.2.2 Drainage Geotextile Fabric

Drain rock and other granular material used for subsurface drains should be wrapped in a geotextile fabric that meets the specifications provided in OSSC 00350 (Geosynthetic Installation) and OSSC 02320 (Geosynthetics) for drainage geotextiles and installed in conformance with OSSC 00350 (Geosynthetic Installation).

12.5 EROSION CONTROL

The site soil is moderately susceptible to erosion; therefore, erosion control measures should be carefully planned and in place before construction begins. Surface water runoff should be collected and directed away from slopes to prevent water from running down the slope face.

Erosion control measures (such as straw bales, sediment fences, and temporary detention and settling basins) should be used in accordance with local and state ordinances.

13.0 OBSERVATION OF CONSTRUCTION

Satisfactory foundation performance depends to a large degree on the quality of construction. Sufficient observation of the contractor's activities is a key part of determining that the work is completed in accordance with the construction drawings and specifications. We recommend that an NV5 representative be retained to observe excavation, fill placement, and subgrade preparation.

Subsurface soil and groundwater conditions observed during construction should be compared with those encountered during the subsurface explorations. Recognition of changed conditions often requires experience; therefore, qualified personnel should visit the site with sufficient frequency to detect whether subsurface conditions change significantly from those anticipated.

14.0 LIMITATIONS

We have prepared this report for use by the City of St. Helens; Mayer/Reed, Inc.; and other members of the design and construction teams for the proposed development. The data and report can be used for bidding or estimating purposes, but this report and our conclusions and interpretations should not be construed as warranty of the subsurface conditions and are not applicable to other sites.

Soil exploration observations indicate soil conditions at specific locations and to the depths explored. They do not necessarily reflect soil strata or water level variations that may exist between exploration locations. If subsurface conditions differing from those described are noted during the course of excavation and construction, re-evaluation will be necessary.

The site development plans and design details were preliminary at the time this report was prepared. When the design has been finalized and if there are changes in the site grades or location, utility plans, configuration, design loads, or type of construction, the conclusions and recommendations presented may not be applicable. If design changes are made, we request that we be retained to review our conclusions and recommendations and to provide a written modification or verification.

The scope of our services does not include services related to construction safety precautions, and our recommendations are not intended to direct the contractor's methods, techniques, sequences, or procedures, except as specifically described in this report for consideration in design.

Within the limitations of scope, schedule, and budget, our services have been executed in accordance with generally accepted practices in this area at the time this report was prepared. No warranty, express or implied, should be understood.

♦ ♦ ♦

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

Sincerely,

NV5

Shawn M. Dimke, P.E., G.E.
Principal Engineer

REGISTERED PROFESSIONAL ENGINEER
63114PE
OREGON
March 14, 2006
SHAWN DIMKE
EXPIRES: 12/31/21

REFERENCES

ASCE, 2016. *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*. ASCE Standard ASCE/SEI 7-016.

Bray, J. D., and Travasarou, T., 2007. "Simplified Procedure for Estimating Earthquake-Induced Deviatoric Slope Displacements." *Journal of Geotechnical & Geoenvironmental Engineering*, vol. 133(4), pp. 381 – 392.

Bray, J. D., Macedo, J., and Travasarou, T., 2018. "Simplified Procedure for Estimating Seismic Slope Displacements for Subduction Zone Earthquakes." *Journal of Geotechnical & Geoenvironmental Engineering*, vol. 144(3).

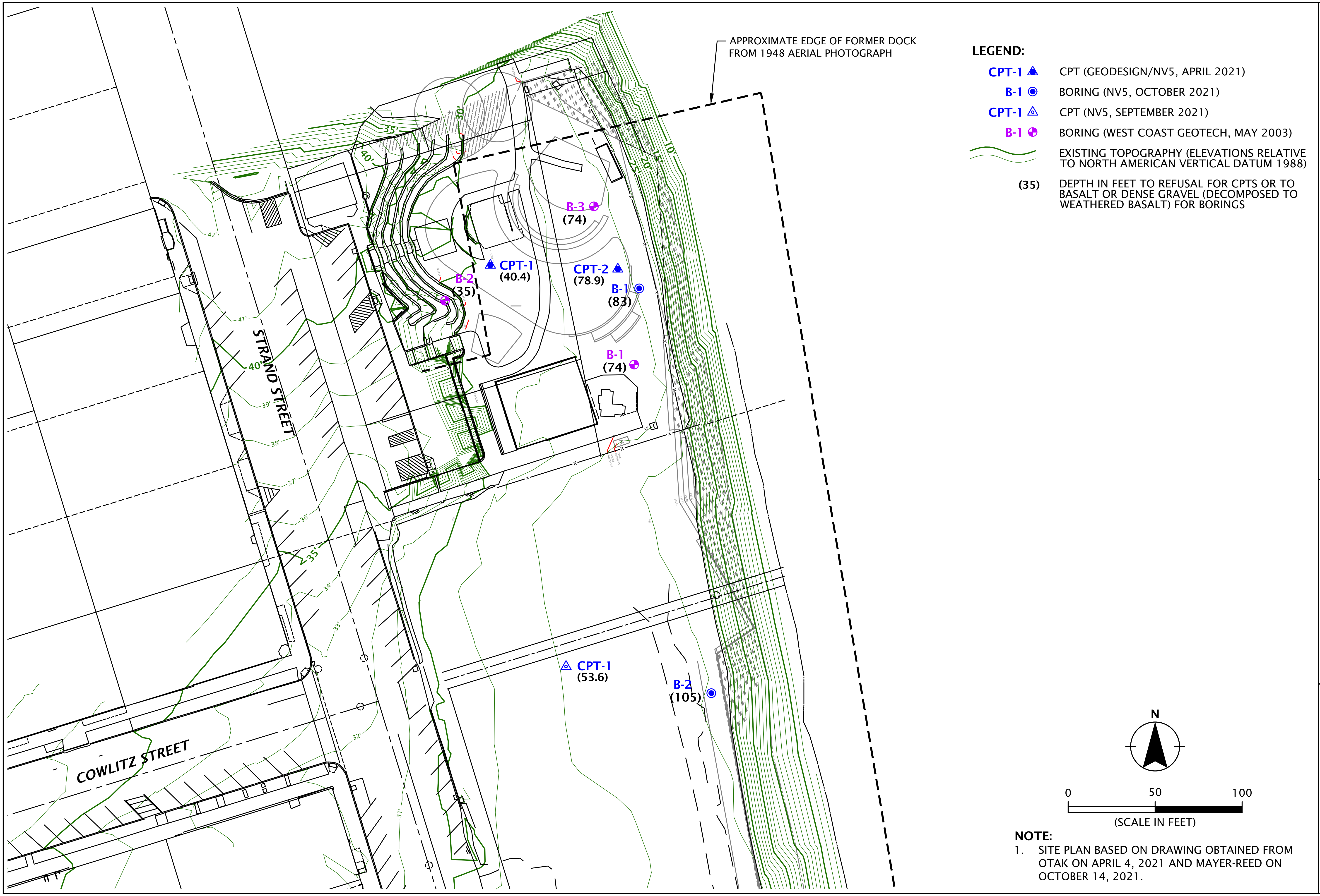
Dickenson, S., 2018. *ASCE Spring Seminar 2018: Waterfront Engineering*.

Evarts, R. C., 2004. *Geologic Map of the Saint Helens Quadrangle, Columbia County, Oregon, and Clark and Cowlitz Counties, Washington.* U.S. Geological Survey Scientific Investigations Map 2834, scale: 1:24,000.

FIGURES

VICINITY MAP BASED ON AERIAL PHOTOGRAPH OBTAINED FROM GOOGLE EARTH PRO®

NV5	STHELENS-4-01	**VICINITY MAP**	
	DECEMBER 2021	ST. HELENS RIVERWALK PHASE I ST. HELENS, OR	**FIGURE 1**



LEGEND:

CPT-1 ▲	CPT (GEODESIGN/NV5, APRIL 2021)
B-1 ◉	BORING (NV5, OCTOBER 2021)
CPT-1 △	CPT (NV5, SEPTEMBER 2021)
B-1 ●	BORING (WEST COAST GEOTECH, MAY 2003)
〰	EXISTING TOPOGRAPHY (ELEVATIONS RELATIVE TO NORTH AMERICAN VERTICAL DATUM 1988)
(35)	DEPTH IN FEET TO REFUSAL FOR CPTS OR TO BASALT OR DENSE GRAVEL (DECOMPOSED TO WEATHERED BASALT) FOR BORINGS

NOTE:

1. SITE PLAN BASED ON DRAWING OBTAINED FROM OTAK ON APRIL 4, 2021 AND MAYER-REED ON OCTOBER 14, 2021.

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FIGURE 2

SITE PLAN

ST. HELENS RIVERWALK PHASE I
ST. HELENS, OR

STHELENS-4-01

DECEMBER 2021

NV5

APPENDIX A

APPENDIX A

FIELD EXPLORATIONS

GENERAL

We explored subsurface conditions at the Riverwalk Phase I site by drilling two borings (B-1 and B-2) on October 4 and 5, 2021. The borings were drilled to depths between 85.4 and 105 feet BGS by Western States Soil Conservation, Inc. of Hubbard, Oregon, under the supervision of NV5 personnel. The borings were completed using mud rotary drilling methods.

We chose the locations of the explorations based on information provided by the design team. The locations of the explorations were determined using a GPS application on a mobile phone. Some locations were adjusted slightly relative to nearby surrounding features. This information should be considered accurate only to the degree implied by the methods used. Approximate exploration locations are shown on Figure 2. The exploration logs are presented in this appendix.

SOIL SAMPLING

Samples were collected from the borings using a 1½- to 3-inch-inside-diameter split-spoon SPT sampler in general accordance with ASTM D1586. The split-spoon sampler was driven into the soil with a 140-pound hammer free falling 30 inches. The sampler was driven a total distance of 18 inches. The number of blows required to drive the sampler the final 12 inches is recorded on the boring logs, unless otherwise noted. Higher quality, relatively undisturbed samples were collected using a standard Shelby tube in general accordance with ASTM D1587. Sampling methods and intervals are shown on the exploration logs.

The average efficiency of the automatic SPT hammer used by the drill rig was 87.4 percent, as shown on the exploration logs. The calibration testing results are presented at the end of this appendix.

SOIL CLASSIFICATION

The soil samples were classified in the field in accordance with the “Exploration Key” (Table A-1) and “Soil Classification System” (Table A-2), which are presented in this appendix. The exploration logs indicate the depths at which the soil characteristics change, although the change actually could be gradual. If the change occurred between sample locations, the depth was interpreted. Classifications are shown on the exploration logs.

LABORATORY TESTING

Laboratory testing was conducted on select soil samples to confirm field classifications and determine the index engineering properties and strength characteristics. Descriptions of the testing completed are presented below.

MOISTURE CONTENT

We tested the natural moisture content of select soil samples in general accordance with ASTM D2216. The natural moisture content is a ratio of the weight of the water to dry soil in a test sample and is expressed as a percentage. The test results are presented in this appendix.

ATTERBERG LIMITS TESTING

The plastic limit and liquid limit (Atterberg limits) of select soil samples were determined in accordance with ASTM D4318. The Atterberg limits and the plasticity index were completed to aid in the classification of the soil. The plastic limit is defined as the moisture content (in percent) where the soil becomes brittle. The liquid limit is defined as the moisture content where the soil begins to act similar to a liquid. The plasticity index is the difference between the liquid and plastic limits. The test results are presented in this appendix.

PARTICLE-SIZE ANALYSIS

Particle-size analysis were performed on select soil samples in general accordance with ASTM D1140. This test is a quantitative determination of the amount of material finer than the U.S. Standard No. 200 sieve expressed as a percentage of soil weight. The test results are presented in this appendix.

CONSOLIDATION TESTING

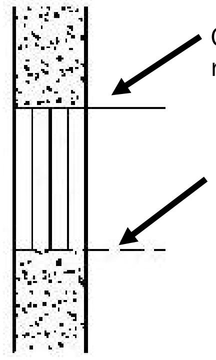
Consolidation testing was performed on select soil samples in general accordance with ASTM D2435. The test measures the volume change of a soil sample under predetermined loads. The test results are presented in this appendix.

DRY DENSITY

We tested the in-situ dry density of select soil samples in general accordance with ASTM D7263. The dry density of the ratio between the mass of the soil (not including water) and the volume of the intact sample. The density is expressed in units of pcf. The test results are presented in this appendix.

SYMBOL	SAMPLING DESCRIPTION
▮	Location of sample collected in general accordance with ASTM D1586 using Standard Penetration Test (SPT) with recovery
▮	Location of sample collected using thin-wall Shelby tube or Geoprobe® sampler in general accordance with ASTM D1587 with recovery
▮	Location of sample collected using Dames & Moore sampler and 300-pound hammer or pushed with recovery
▮	Location of sample collected using Dames & Moore sampler and 140-pound hammer or pushed with recovery
▮	Location of sample collected using 3-inch-outside diameter California split-spoon sampler and 140-pound hammer with recovery
☒	Location of grab sample
▮	Rock coring interval
▽	Water level during drilling
▼	Water level taken on date shown

Graphic Log of Soil and Rock Types



Observed contact between soil or rock units (at depth indicated)

Inferred contact between soil or rock units (at approximate depths indicated)

GEOTECHNICAL TESTING EXPLANATIONS

ATT	Atterberg Limits	P	Pushed Sample
CBR	California Bearing Ratio	PP	Pocket Penetrometer
CON	Consolidation	P200	Percent Passing U.S. Standard No. 200 Sieve
DD	Dry Density		
DS	Direct Shear	RES	Resilient Modulus
HYD	Hydrometer Gradation	SIEV	Sieve Gradation
MC	Moisture Content	TOR	Torvane
MD	Moisture-Density Relationship	UC	Unconfined Compressive Strength
NP	Non-Plastic	VS	Vane Shear
OC	Organic Content	kPa	Kilopascal

ENVIRONMENTAL TESTING EXPLANATIONS

CA	Sample Submitted for Chemical Analysis	ND	Not Detected
P	Pushed Sample	NS	No Visible Sheen
PID	Photoionization Detector Headspace Analysis	SS	Slight Sheen
		MS	Moderate Sheen
ppm	Parts per Million	HS	Heavy Sheen

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EXPLORATION KEY

TABLE A-1

RELATIVE DENSITY - COARSE-GRAINED SOIL

Relative Density	Standard Penetration Test (SPT) Resistance	Dames & Moore Sampler (140-pound hammer)	Dames & Moore Sampler (300-pound hammer)
Very loose	0 – 4	0 – 11	0 – 4
Loose	4 – 10	11 – 26	4 – 10
Medium dense	10 – 30	26 – 74	10 – 30
Dense	30 – 50	74 – 120	30 – 47
Very dense	More than 50	More than 120	More than 47

CONSISTENCY - FINE-GRAINED SOIL

Consistency	Standard Penetration Test (SPT) Resistance	Dames & Moore Sampler (140-pound hammer)	Dames & Moore Sampler (300-pound hammer)	Unconfined Compressive Strength (tsf)
Very soft	Less than 2	Less than 3	Less than 2	Less than 0.25
Soft	2 – 4	3 – 6	2 – 5	0.25 – 0.50
Medium stiff	4 – 8	6 – 12	5 – 9	0.50 – 1.0
Stiff	8 – 15	12 – 25	9 – 19	1.0 – 2.0
Very stiff	15 – 30	25 – 65	19 – 31	2.0 – 4.0
Hard	More than 30	More than 65	More than 31	More than 4.0

PRIMARY SOIL DIVISIONS			GROUP SYMBOL	GROUP NAME
COARSE-GRAINED SOIL (more than 50% retained on No. 200 sieve)	GRAVEL (more than 50% of coarse fraction retained on No. 4 sieve)	CLEAN GRAVEL (< 5% fines)	GW or GP	GRAVEL
		GRAVEL WITH FINES (≥ 5% and ≤ 12% fines)	GW-GM or GP-GM	GRAVEL with silt
			GW-GC or GP-GC	GRAVEL with clay
		GRAVEL WITH FINES (> 12% fines)	GM	silty GRAVEL
			GC	clayey GRAVEL
			GC-GM	silty, clayey GRAVEL
	SAND (50% or more of coarse fraction passing No. 4 sieve)	CLEAN SAND (<5% fines)	SW or SP	SAND
		SAND WITH FINES (≥ 5% and ≤ 12% fines)	SW-SM or SP-SM	SAND with silt
			SW-SC or SP-SC	SAND with clay
		SAND WITH FINES (> 12% fines)	SM	silty SAND
			SC	clayey SAND
			SC-SM	silty, clayey SAND
FINE-GRAINED SOIL (50% or more passing No. 200 sieve)	SILT AND CLAY	Liquid limit less than 50	ML	SILT
			CL	CLAY
			CL-ML	silty CLAY
			OL	ORGANIC SILT or ORGANIC CLAY
		Liquid limit 50 or greater	MH	SILT
			CH	CLAY
			OH	ORGANIC SILT or ORGANIC CLAY
HIGHLY ORGANIC SOIL			PT	PEAT

MOISTURE CLASSIFICATION

Term	Field Test
dry	very low moisture, dry to touch
moist	damp, without visible moisture
wet	visible free water, usually saturated

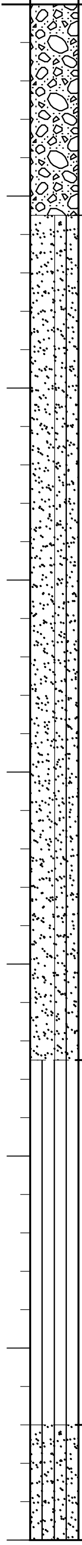
ADDITIONAL CONSTITUENTS

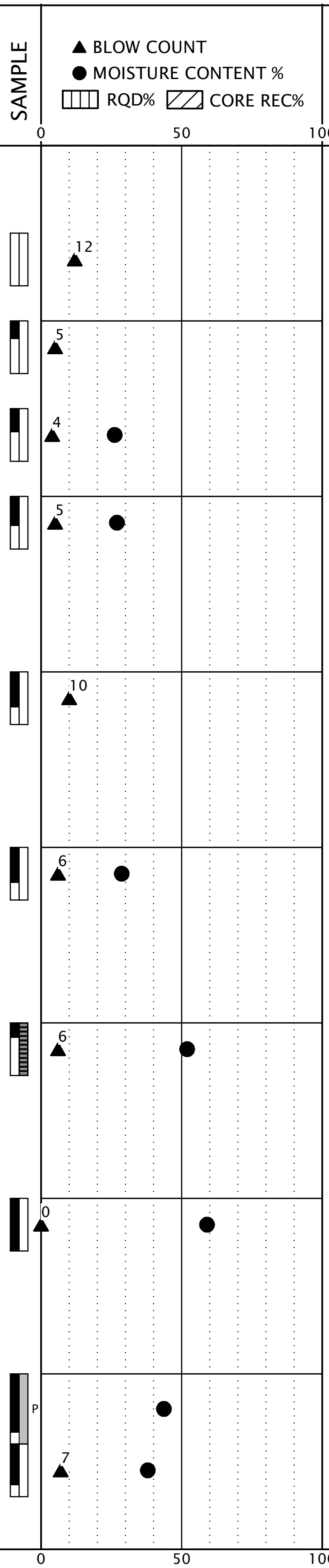
Secondary granular components or other materials such as organics, man-made debris, etc.					
Percent	Silt and Clay In:		Percent	Sand and Gravel In:	
	Fine-Grained Soil	Coarse-Grained Soil		Fine-Grained Soil	Coarse-Grained Soil
< 5	trace	trace	< 5	trace	trace
5 – 12	minor	with	5 – 15	minor	minor
> 12	some	silty/clayey	15 – 30	with	with
			> 30	sandy/gravelly	Indicate %

NV5

SOIL CLASSIFICATION SYSTEM

TABLE A-2





DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
0	Medium dense, gray GRAVEL with cobbles and boulders (GP); moist, cobbles are approximately 40%, boulders are approximately 20% (4-inch-thick root zone) - **FILL**.			12	
5.5	Loose, gray SAND with silt (SP-SM); moist, sand is fine to medium - **FILL**.	5.5		5	
7.5	very loose to loose at 7.5 feet			4	
10	loose at 10.0 feet		P200	5	P200 = 7%
15	loose to medium dense at 15.0 feet			10	
20	loose; wet at 20.0 feet			6	
25	very loose at 25.0 feet		DD	6	DD = 69 pcf
27.5	Very soft, gray SILT (ML), minor sand; wet, silt has low plasticity, sand is fine, interbedded with very loose, gray SAND with silt (SP-SM); wet, sand is fine to medium.	27.5		0	
35			CON		
37	Loose, dark gray, silty SAND (SM), trace organics (wood fibers); wet, sand is fine to medium.	37.0	OC	7	OC = 38%
40					

DRILLED BY: Western States Soil Conservation, Inc.

LOGGED BY: T. Pierce

COMPLETED: 10/04/21

BORING METHOD: mud rotary (see document text)

BORING BIT DIAMETER: 4 7/8 inches and 3 7/8 inches

NV5

STHELENS-4-01

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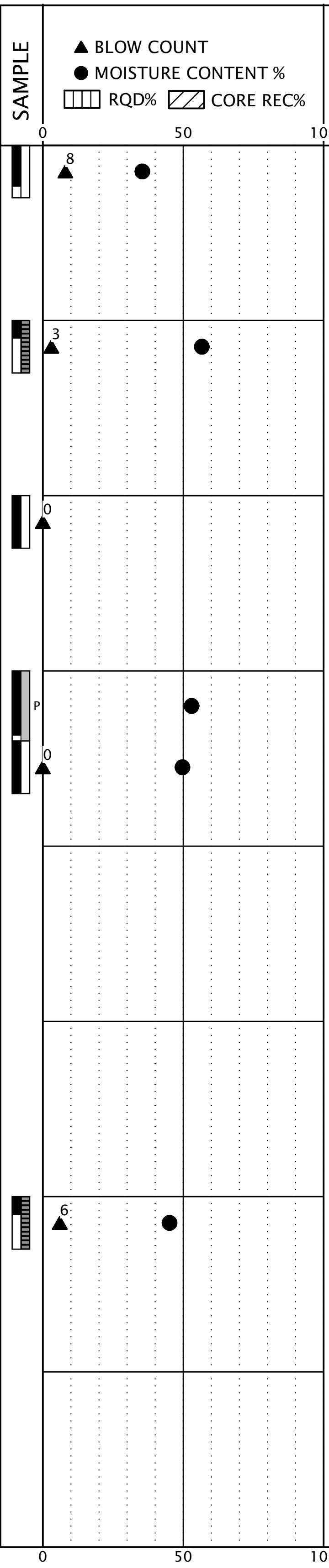
BORING B-1

ST. HELENS RIVERWALK PHASE I
ST. HELENS, OR

FIGURE A-1

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
40	without organics at 40.0 feet		P200	P200 = 19%
	Very soft, dark gray SILT (ML), trace sand and clay; wet, silt has medium plasticity, sand is fine.	42.5		
45			DD	DD = 66 pcf
50	lenses of SAND (<1 inch thick) at 50.0 feet			
55			DD	DD = 68 pcf
			ATT	LL = 50% PL = 35%
60				
	trace gravel at 62.5 feet			
	without gravel at 64.0 feet			
65				
70	soft at 70.0 feet		DD	DD = 74 pcf
75				
	Loose to medium dense, gray, silty GRAVEL (GM); wet, gravel is coarse.	76.0		
80				



DRILLED BY: Western States Soil Conservation, Inc.

LOGGED BY: T. Pierce

COMPLETED: 10/04/21

BORING METHOD: mud rotary (see document text)

BORING BIT DIAMETER: 4 7/8 inches and 3 7/8 inches

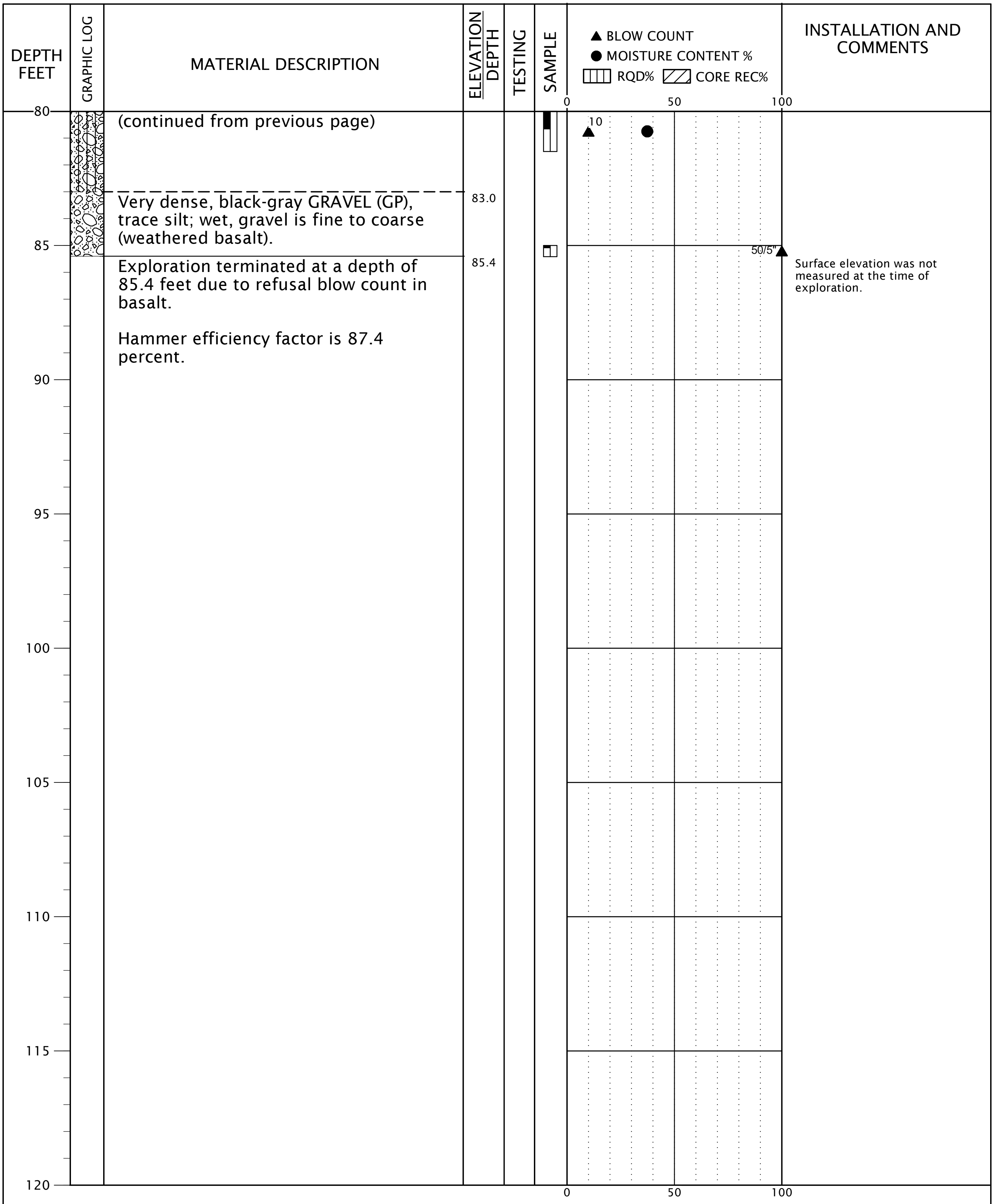
STHELENS-4-01

DECEMBER 2021

BORING B-1
(continued)

ST. HELENS RIVERWALK PHASE I
ST. HELENS, OR

FIGURE A-1



(continued from previous page)

Very dense, black-gray GRAVEL (GP), trace silt; wet, gravel is fine to coarse (weathered basalt).

Exploration terminated at a depth of 85.4 feet due to refusal blow count in basalt.

Hammer efficiency factor is 87.4 percent.

Surface elevation was not measured at the time of exploration.

DRILLED BY: Western States Soil Conservation, Inc.
LOGGED BY: T. Pierce
COMPLETED: 10/04/21

BORING METHOD: mud rotary (see document text)
BORING BIT DIAMETER: 4 7/8 inches and 3 7/8 inches

NV5

STHELENS-4-01

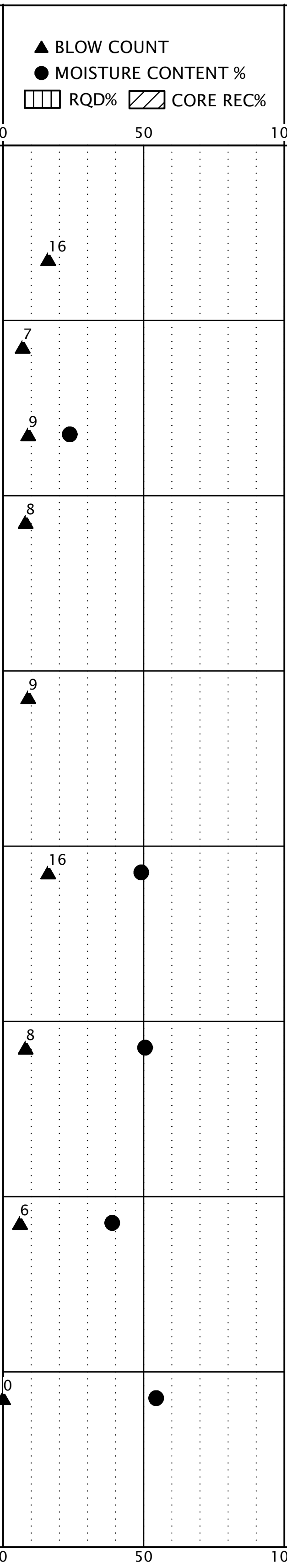
DECEMBER 2021

BORING B-1
(continued)

ST. HELENS RIVERWALK PHASE I
ST. HELENS, OR

FIGURE A-1

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
0	Medium dense, gray GRAVEL with boulders and cobbles (GP); moist, boulders are approximately 40%, cobbles are approximately 30% - **FILL**.			16	Hit boulder at 3.0 feet. Had to reset hole.
5	Loose, gray SAND with silt (SP-SM); wet, sand is fine to medium - **FILL**.	4.5		7	
				9	
10				8	
15				9	
	Stiff, dark gray SILT (ML), trace sand; moist to wet, sand is fine - **FILL**.	18.0			
20			P200	16	P200 = 97%
	Loose, dark gray, silty SAND (SM), trace gravel; wet, sand is fine - **FILL**.	22.0			
25			P200	8	P200 = 37%
	Very loose, dark gray SAND with silt (SP-SM); wet, sand is fine to medium, interbedded with soft, dark gray SILT (ML), trace sand; wet, sand is fine to medium (1 to 4 inches thick).	28.0			
30			DD	6	DD = 84 pcf
	Very soft, dark gray SILT (ML), trace sand; wet, sand is fine, lenses of fine to medium SAND (<1 inch thick).	33.0			
35				0	
40					

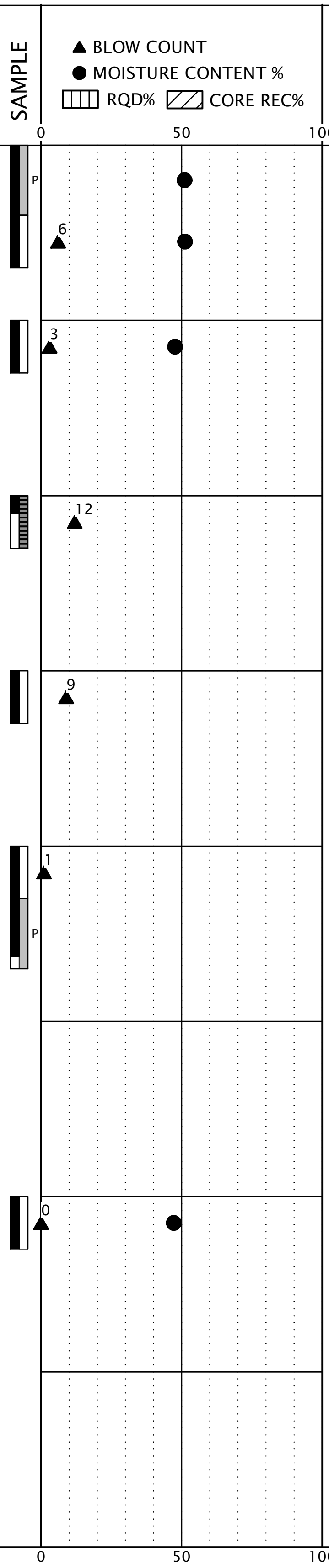


DRILLED BY: Western States Soil Conservation, Inc.
LOGGED BY: T. Pierce
COMPLETED: 10/05/21

BORING METHOD: mud rotary (see document text)
BORING BIT DIAMETER: 4 7/8 inches and 3 7/8 inches

NV5	STHELENS-4-01	**BORING B-2**	
	DECEMBER 2021	ST. HELENS RIVERWALK PHASE I ST. HELENS, OR	**FIGURE A-2**

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
40	(continued from previous page)		CON	
	medium stiff at 42.0 feet			
45	soft at 45.0 feet			LL = NP PL = NP
50	medium stiff; interbedded with loose, dark gray SAND with silt (SP-SM); wet, sand is fine to medium at 50.0 feet			
55	without SAND interbeds at 55.0 feet			
60	very soft, trace organics (wood debris) at 60.0 feet			
65				
70	without organics at 70.0 feet		ATT	LL = 42% PL = 31%
75				
80				



DRILLED BY: Western States Soil Conservation, Inc.
LOGGED BY: T. Pierce
COMPLETED: 10/05/21

BORING METHOD: mud rotary (see document text)
BORING BIT DIAMETER: 4 7/8 inches and 3 7/8 inches

NV5

STHELENS-4-01

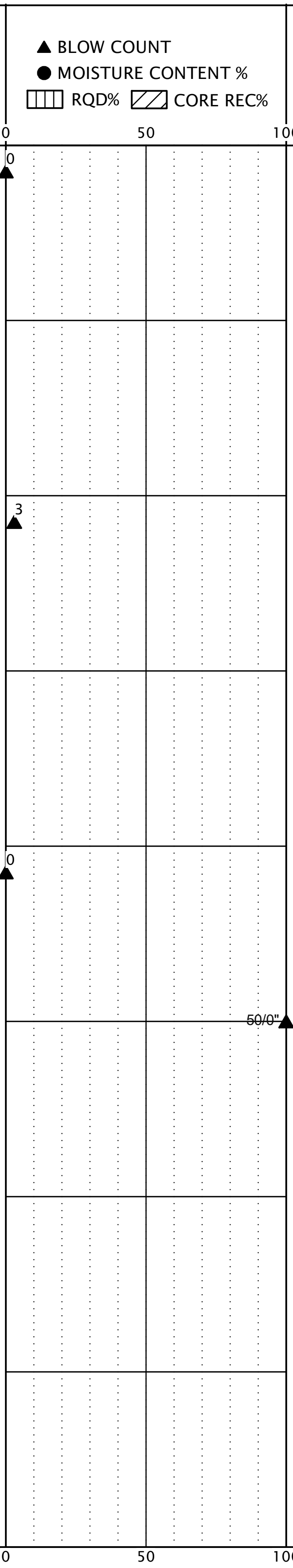
DECEMBER 2021

BORING B-2
(continued)

ST. HELENS RIVERWALK PHASE I
ST. HELENS, OR

FIGURE A-2

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	INSTALLATION AND COMMENTS
80		(continued from previous page)				
100		trace organics (wood fibers) at 100.0 feet				
105		Exploration terminated at a depth of 105.0 feet due to refusal blow count in basalt. Hammer efficiency factor is 87.4 percent.	105.0			Surface elevation was not measured at the time of exploration.



DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: T. Pierce COMPLETED: 10/05/21

BORING METHOD: mud rotary (see document text) BORING BIT DIAMETER: 4 7/8 inches and 3 7/8 inches

NV5

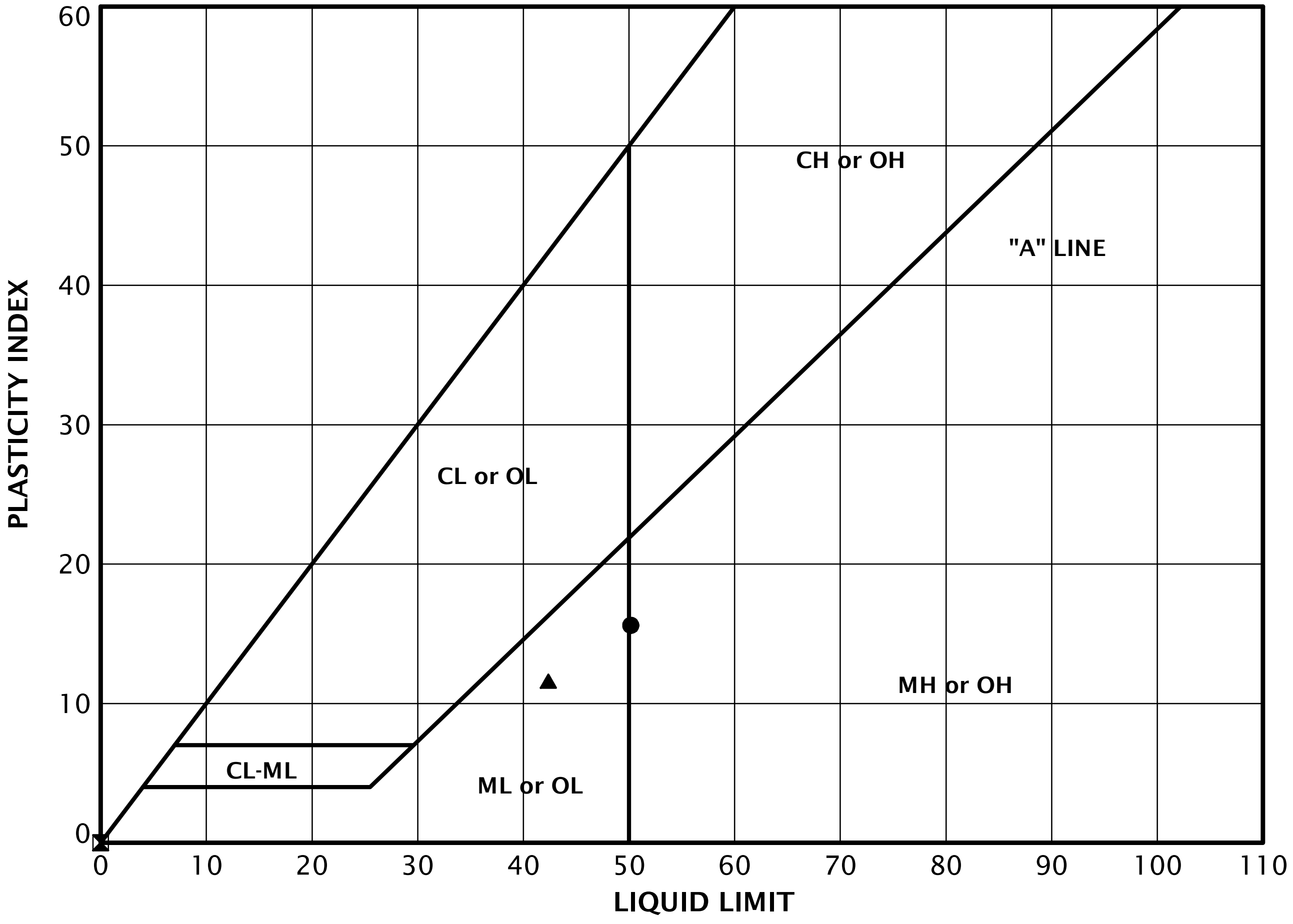
STHELENS-4-01

DECEMBER 2021

BORING B-2
(continued)

ST. HELENS RIVERWALK PHASE I
ST. HELENS, OR

FIGURE A-2



KEY	EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	MOISTURE CONTENT (PERCENT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
●	B-1	57.0	50	50	35	15
⧗	B-2	45.0	48	NP	NP	NP
▲	B-2	70.0	47	42	31	11

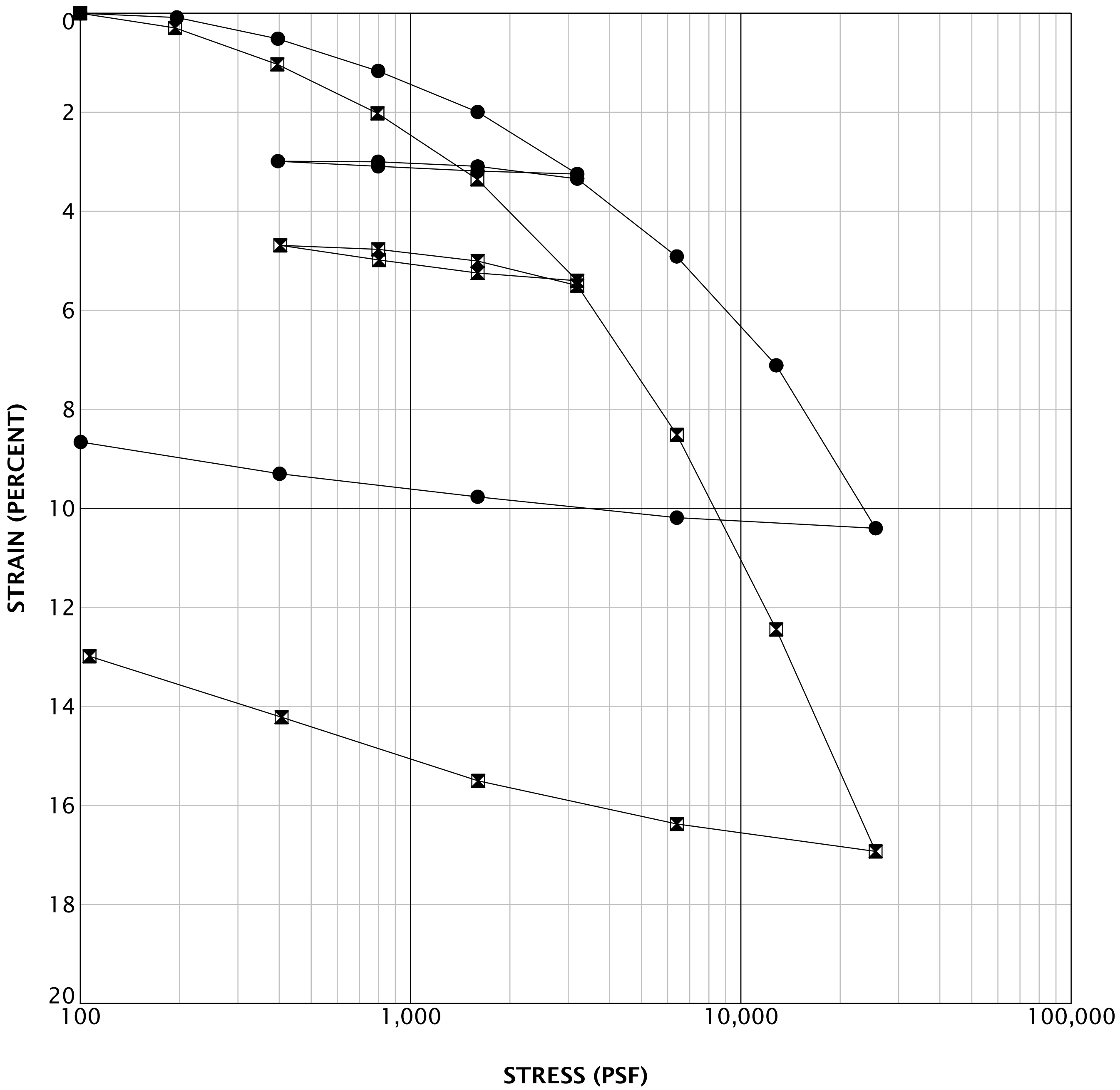
STHELENS-4-01

ATTERBERG LIMITS TEST RESULTS

DECEMBER 2021

ST. HELENS RIVERWALK PHASE I
ST. HELENS, OR

FIGURE A-3



KEY	EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	MOISTURE CONTENT (PERCENT)	DRY DENSITY (PCF)
●	B-1	35.0	44	74
☒	B-2	40.0	51	71

STHELENS-4-01	**CONSOLIDATION TEST RESULTS**	
DECEMBER 2021	ST. HELENS RIVERWALK PHASE I ST. HELENS, OR	**FIGURE A-4**

SAMPLE INFORMATION			MOISTURE CONTENT (PERCENT)	DRY DENSITY (PCF)	SIEVE			ATTERBERG LIMITS		
EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	ELEVATION (FEET)			GRAVEL (PERCENT)	SAND (PERCENT)	P200 (PERCENT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
B-1	7.5		26							
B-1	10.0		27				7			
B-1	20.0		29							
B-1	25.0		52	69						
B-1	30.0		59							
B-1	35.0		44	74						
B-1	37.0		38							
B-1	40.0		35				19			
B-1	45.0		57	66						
B-1	55.0		53	68						
B-1	57.0		50					50	35	15
B-1	70.0		45	74						
B-1	80.0		37							
B-2	7.5		24							
B-2	20.0		49				97			
B-2	25.0		50				37			
B-2	30.0		39	84						
B-2	35.0		54							
B-2	40.0		51	71						
B-2	42.0		51							
B-2	45.0		48					NP	NP	NP
B-2	70.0		47					42	31	11

LAB SUMMARY - GDI-NV5 STHELENS-4-01-B1_2.GPJ GDI_NV5.GDT PRINT DATE: 12/8/21:KT

NV5

STHELENS-4-01

DECEMBER 2021

SUMMARY OF LABORATORY DATA

ST. HELENS RIVERWALK PHASE I
ST. HELENS, OR

FIGURE A-5

Summary of SPT Test Results

Project: WSSC-8-05, Test Date: 4/13/2020

EMX: Maximum Energy

ETR: Energy Transfer Ratio - Rated

Start Depth ft	Final Depth ft	**N Value**	**N60 Value**	Average EMX ft-lb	Average ETR %
42.50	44.00	**18**	**26**	306.23	87.5
45.00	46.50	**17**	**24**	304.53	87.0
50.00	51.50	**12**	**17**	305.90	87.4
52.50	54.00	**26**	**37**	306.91	87.7
			Overall Average Values:	306.02	87.4
			Standard Deviation:	4.49	1.3
			Overall Maximum Value:	313.51	89.6
			Overall Minimum Value:	294.12	84.0

APPENDIX B

APPENDIX B

CONE PENETRATION TESTING

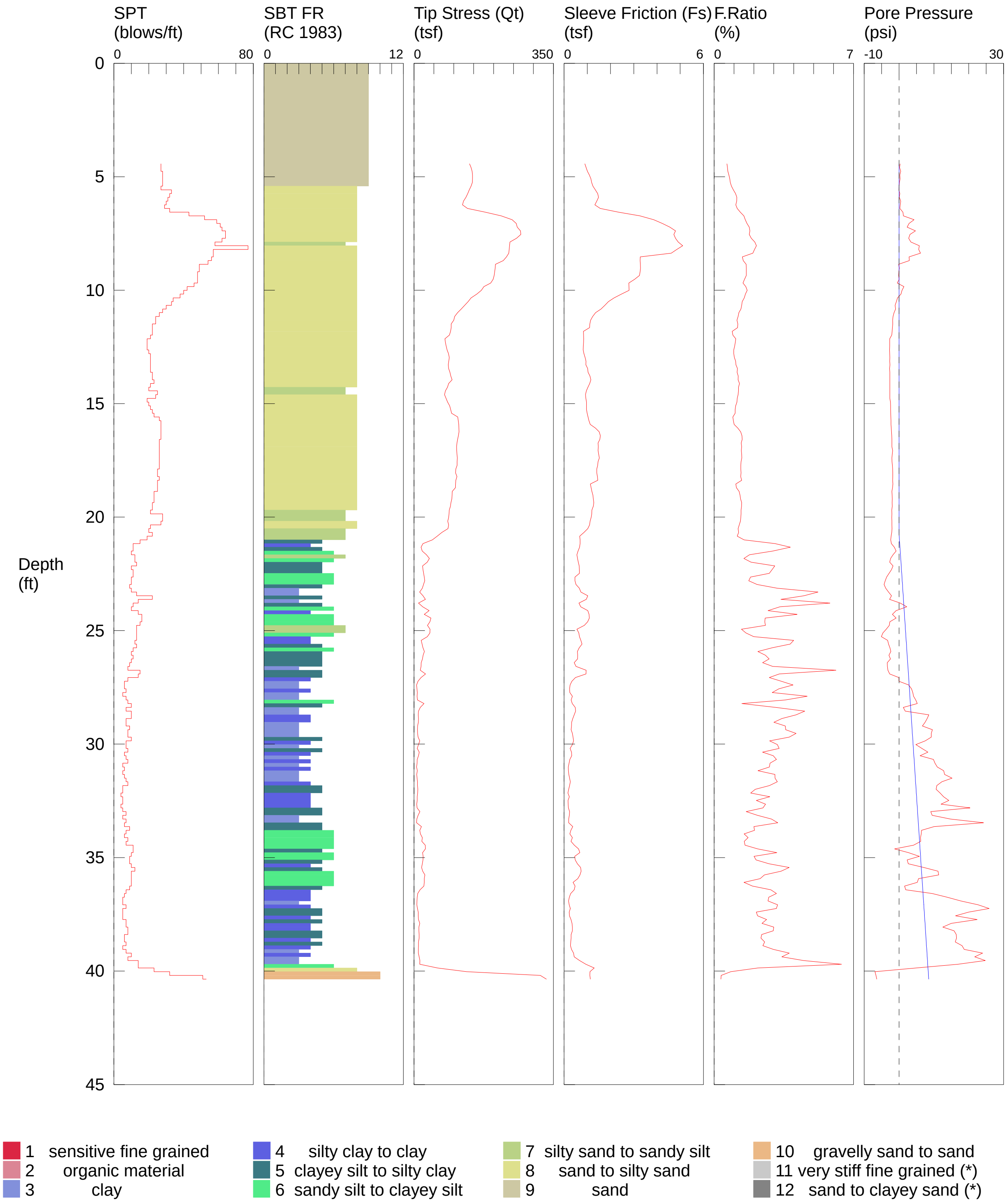
The CPTs were performed in general accordance with ASTM D5778 by Oregon Geotechnical Explorations, Inc. of Keizer, Oregon. CPT-1 and CPT-2 (April) were advanced on April 22, 2021, and CPT-1 (September) was advanced on September 30, 2021, at the approximate locations shown on Figure 2. The CPTs were advanced to refusal at depths ranging from 40.4 to 78.9 feet BGS.

The CPT is an in-situ test that provides assistance in characterizing subsurface stratigraphy. The test includes advancing a 35.6-millimeter-diameter cone equipped with a load cell, friction sleeve, strain gauges, porous stone, and geophone through the soil profile. The cone is advanced at a rate of approximately 2 centimeters per second. Tip resistance, sleeve friction, and pore pressure are typically recorded at 0.1-meter intervals. At select depths, the CPT advancement can be suspended and pore water dissipation rates measured. The results of the CPTs performed for this project are presented in this appendix.

The CPT locations were determined using a GPS application on a mobile phone. Some locations were adjusted slightly relative to nearby surrounding features. This information should be considered accurate to the degree implied by the method used.

NV5 / CPT-1 / Columbia View Park St. Helens

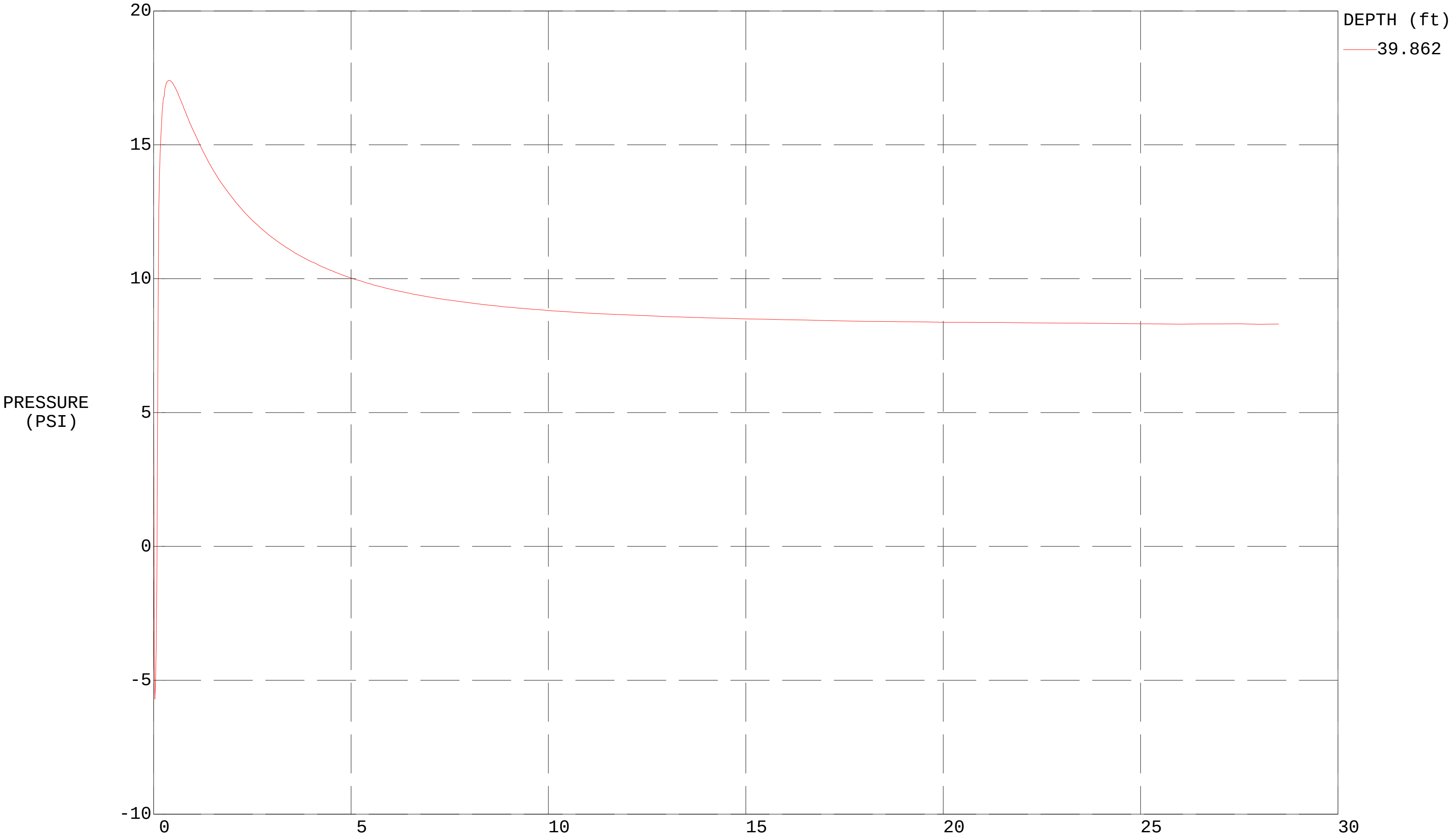
OPERATOR: OGE BAK
CONE ID: DPG1211
HOLE NUMBER: CPT-1
TEST DATE: 4/22/2021 11:06:36 AM
TOTAL DEPTH: 40.354 ft



*SBT/SPT CORRELATION: UBC-1983

COMMENT: NV5 / CPT-1 / Columbia View Park St. Helens

TEST DATE: 4/22/2021 11:06:36 AM

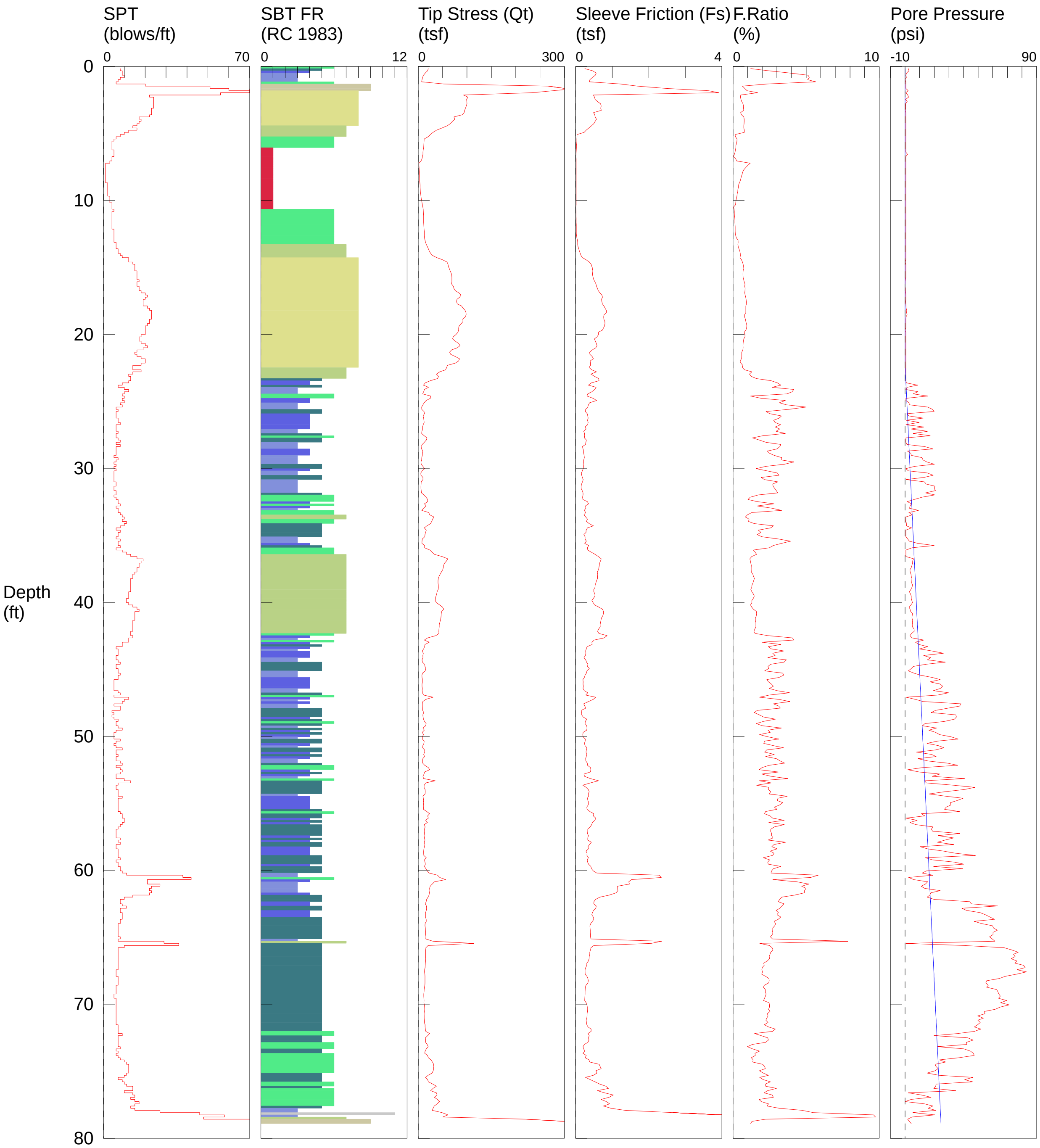


MAXIMUM PRESSURE = 17.41 (PSI) TIME: (MINUTES)

HYDROSTATIC PRESSURE = 8.308 (PSI), WATER TABLE: 20.69 ft

NV5 / CPT-2 / Columbia View Park St. Helens

OPERATOR: OGE BAK
CONE ID: DPG1211
HOLE NUMBER: CPT-2
TEST DATE: 4/22/2021 9:21:52 AM
TOTAL DEPTH: 78.904 ft

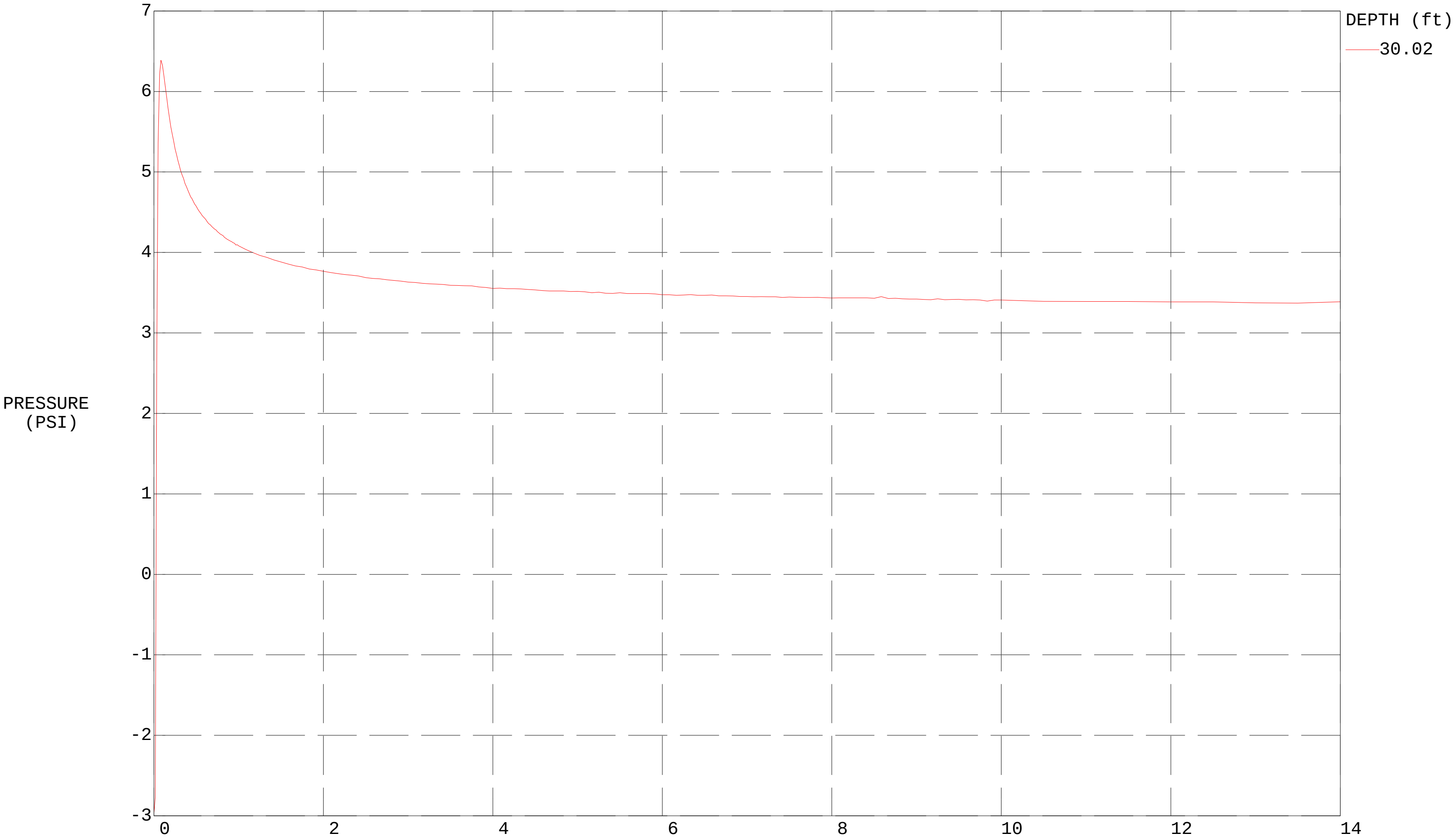


■ 1 sensitive fine grained
■ 2 organic material
■ 3 clay
■ 4 silty clay to clay
■ 5 clayey silt to silty clay
■ 6 sandy silt to clayey silt
■ 7 silty sand to sandy silt
■ 8 sand to silty sand
■ 9 sand
■ 10 gravelly sand to sand
■ 11 very stiff fine grained (*)
■ 12 sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

COMMENT: NV5 / CPT-2 / Columbia View Park St. Helens

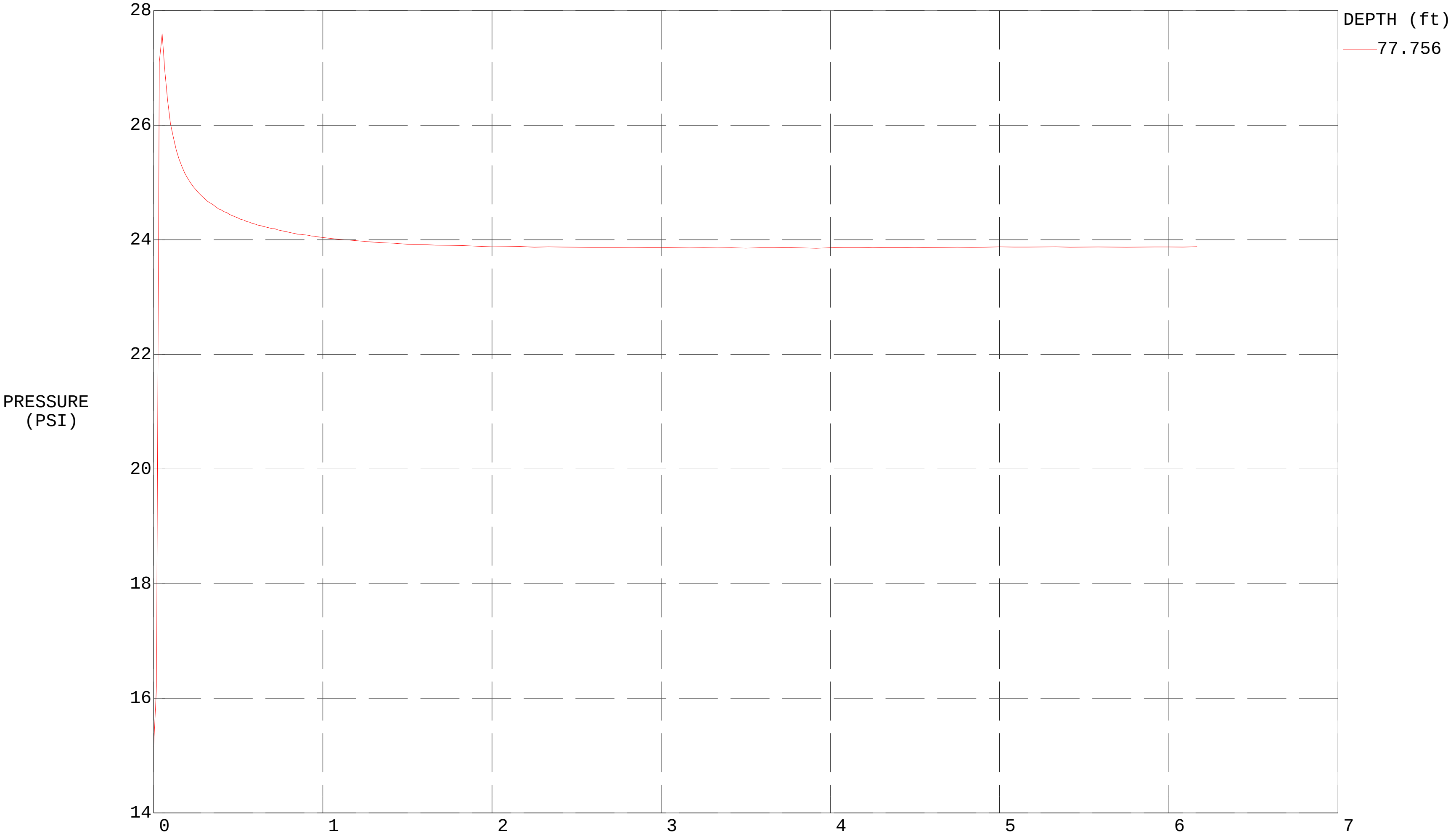
TEST DATE: 4/22/2021 9:21:52 AM



MAXIMUM PRESSURE = 6.388 (PSI)
HYDROSTATIC PRESSURE = 3.403 (PSI), WATER TABLE: 22.17 ft

COMMENT: NV5 / CPT-2 / Columbia View Park St. Helens

TEST DATE: 4/22/2021 9:21:52 AM



MAXIMUM PRESSURE = 27.593 (PSI)

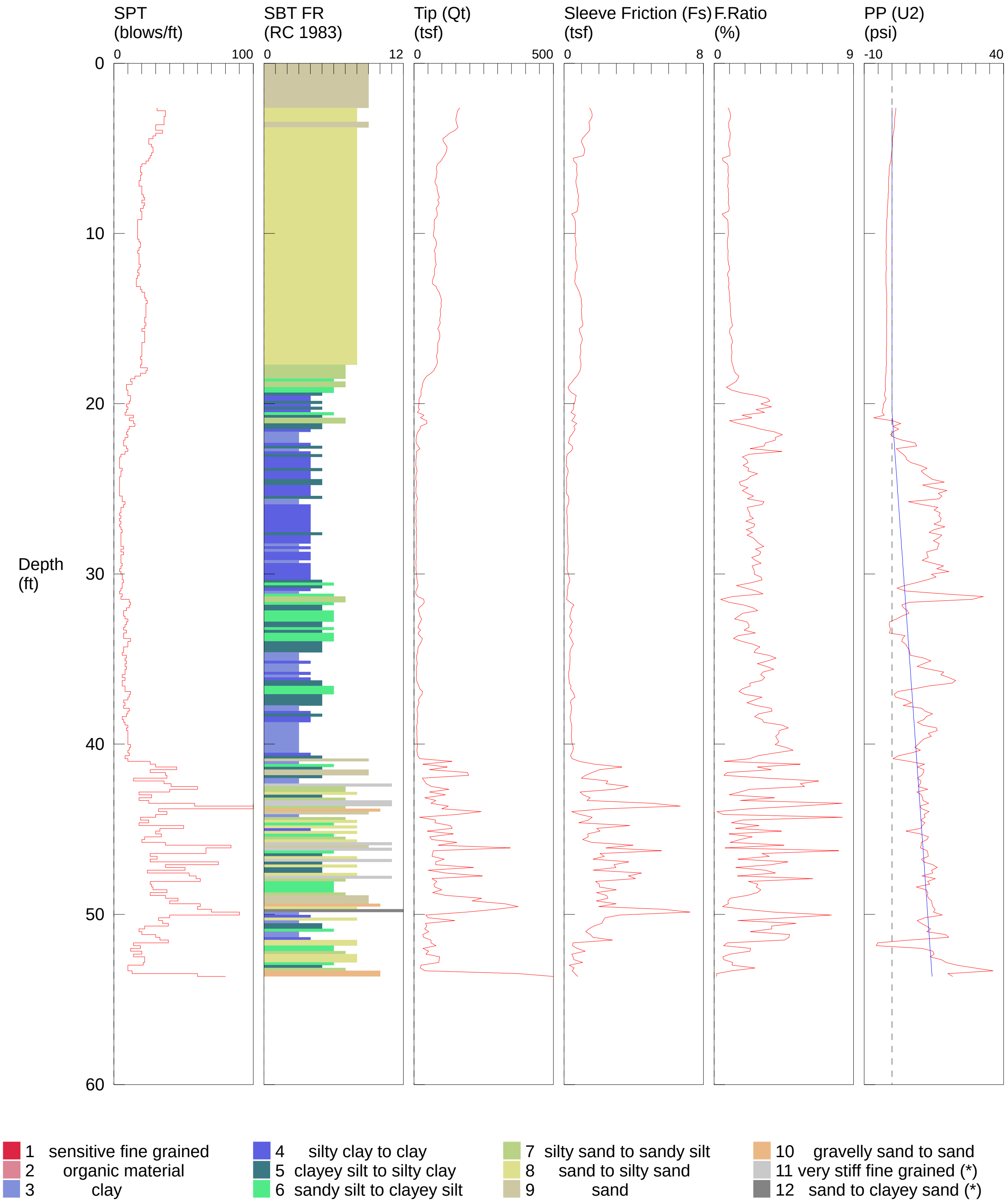
TIME: (MINUTES)

HYDROSTATIC PRESSURE = 24.091 (PSI), WATER TABLE: 22.17 ft

CPT-1 (SEPTEMBER 2021)

NV5 / CPT-1 / 490 S 1st St St. Helens

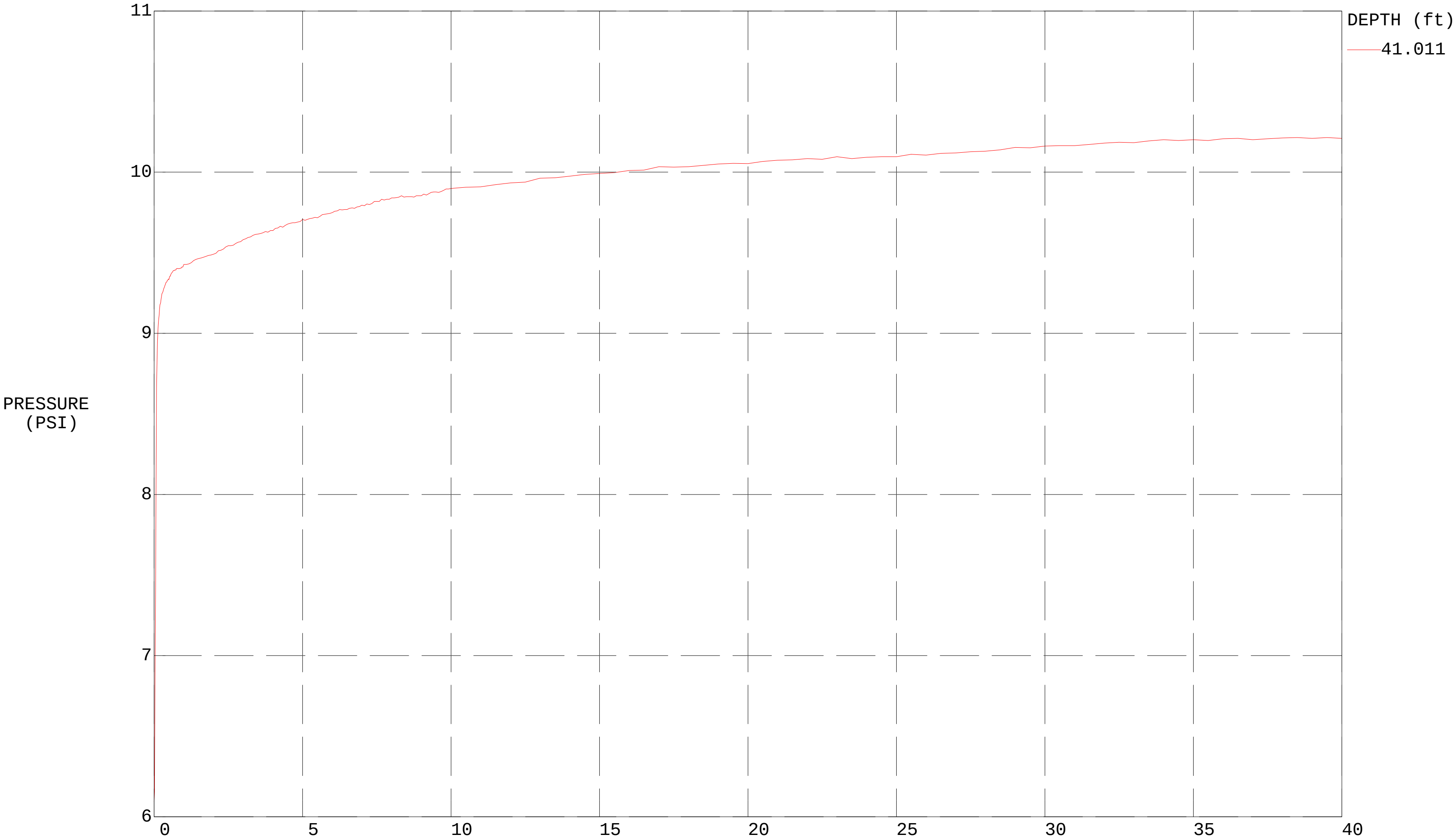
OPERATOR: OGE BAK
CONE ID: DDG1532
HOLE NUMBER: CPT-1
TEST DATE: 9/30/2021 9:35:58 AM
TOTAL DEPTH: 53.642 ft



*SBT/SPT CORRELATION: UBC-1983

COMMENT: NV5 / CPT-1 / 490 S 1st Street St. Helens

TEST DATE: 9/30/2021 9:35:58 AM



MAXIMUM PRESSURE = 10.215 (PSI)

HYDROSTATIC PRESSURE = 8.887 (PSI), WATER TABLE: 20.51 ft

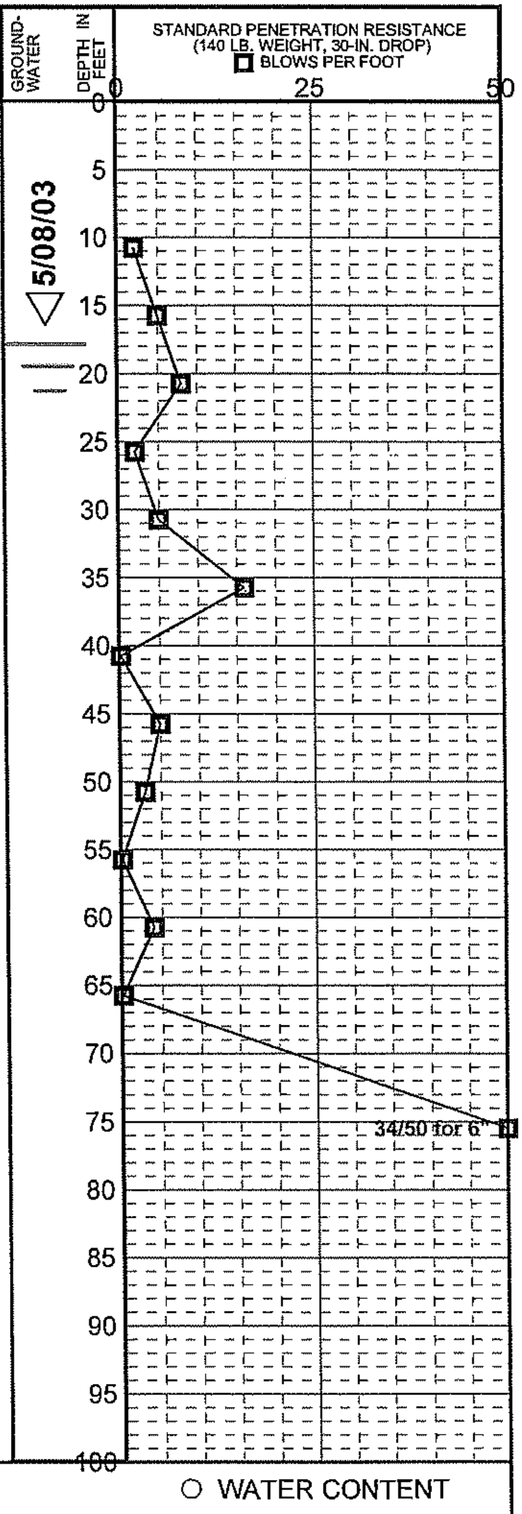
APPENDIX C

APPENDIX C

2003 WEST COAST GEOTECH BORINGS

Boring logs for three borings (B-1 through B-3) drilled at the site in May 2003 are presented in this appendix.

USC	SOIL DESCRIPTION S. Elevation - N.A.	DEPTH IN FEET	SAMPLE NO.
	FINE SAND (FILL) - Very loose to loose, gray, with embedded wood and debris, and also containing existing vertical treated wood timber piles of unknown lengths		S-1 ‡ S-2 ‡ S-3 ‡ S-4 ‡ S-5 ‡
	SILT - Soft, gray, varying amounts of fine sand from trace to fine sandy, with sand/ gravel layer at an approximate depth of 38 feet beneath current ground surface, also containing scattered organic material in form of wood and reeds	35.0	S-6 ‡ S-7 ‡ S-8 ‡ S-9 ‡ S-10 ‡ S-11 ‡ S-12 ‡
	GRAVEL - Dense becoming more dense, gray, possibly cemented at drilling refusal depth of 79 feet or basalt rock	74.0	S-13 ‡
	DRILLING REFUSAL **DRILLING COMPLETED 5/08/03**	79.0	



NOTES:
1. SOIL DESCRIPTIONS AND INTERFACES ARE INTERPRETIVE AND ACTUAL CHANGES MAY BE GRADUAL.
2. WATER LEVEL IS FOR DATE SHOWN AND MAY VARY WITH TIME OF YEAR.

LEGEND

‡ 2.0 " O.D. SPLIT SPOON SAMPLE
3.0" O.D. THIN-WALL SAMPLE
* SAMPLE NOT RECOVERED
P SAMPLE PUSHED
USC UNIFIED SOIL CLASSIFICATION

IMPERVIOUS SEAL
▽ WATER LEVEL
SLOTTED TIP

ATTERBERG

LIQUID LIMIT
NATURAL WATER
PLASTIC LIMIT

CONVENTION CENTER (RIVER SITE)
St. Helens, Oregon

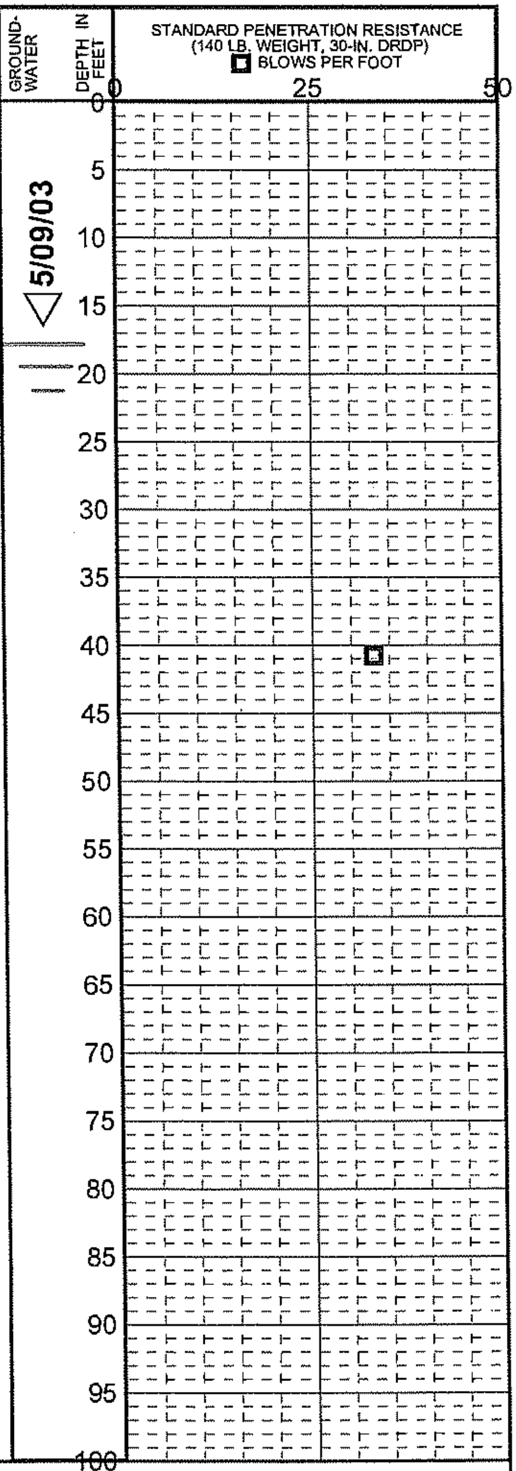
LOG OF BORING B-1

May, 2003 **W-1744**

WEST COAST GEOTECH, INC
Geotechnical Consultants
West Linn, OR

FIG. 2

USC	SOIL DESCRIPTION S. Elevation - N.A.	DEPTH IN FEET	SAMPLE NO.
	FINE SAND (FILL) - Very loose to loose, gray, with embedded wood and debris, and also containing existing vertical treated wood timber piles of unknown lengths		
	GRAVEL - Dense becoming more dense, gray, possibly cemented at drilling refusal depth of 44 feet or basalt rock	35.0	S-1 ‡
	DRILLING REFUSAL **DRILLING COMPLETED 5/09/03**	44.0	



○ WATER CONTENT

NOTES:
1. SOIL DESCRIPTIONS AND INTERFACES ARE INTERPRETIVE AND ACTUAL CHANGES MAY BE GRADUAL.
2. WATER LEVEL IS FOR DATE SHOWN AND MAY VARY WITH TIME OF YEAR.

LEGEND

‡ 2.0 " O.D. SPLIT SPOON SAMPLE

3.0" O.D. THIN-WALL SAMPLE

* SAMPLE NOT RECOVERED

P SAMPLE PUSHED

USC UNIFIED SOIL CLASSIFICATION

IMPERVIOUS SEAL

▽ WATER LEVEL

SLOTTED TIP

ATTERBERG

LIQUID LIMIT

NATURAL WATER

PLASTIC LIMIT

CONVENTION CENTER (RIVER SITE)

St. Helens, Oregon

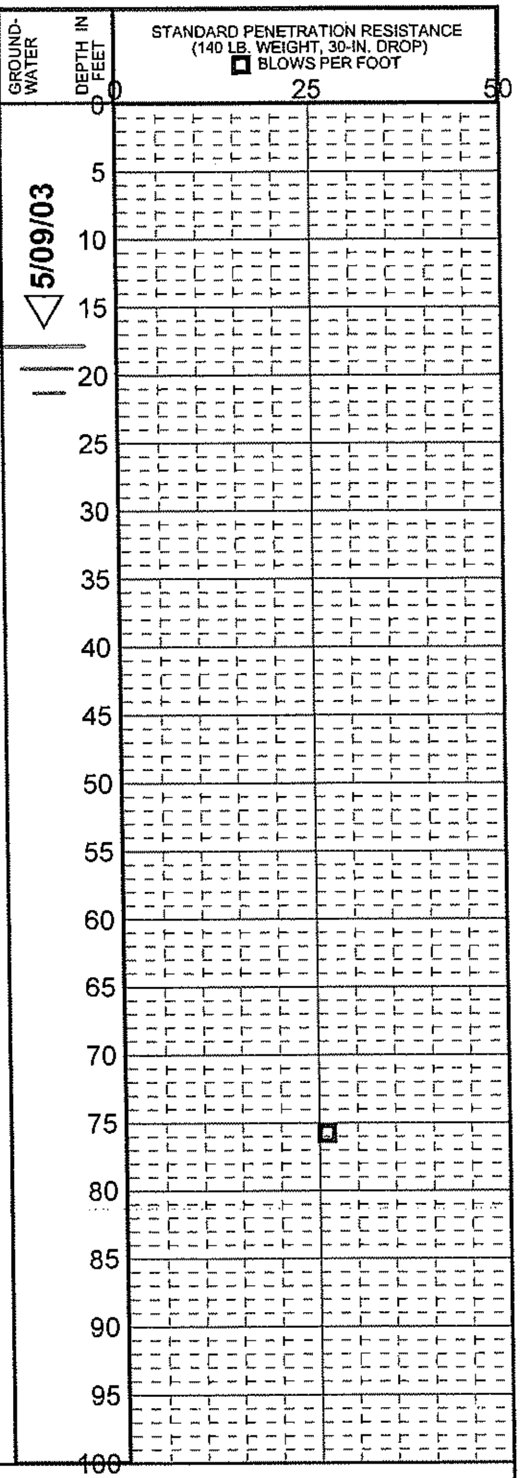
LOG OF BORING B-2

May, 2003 **W-1744**

WEST COAST GEOTECH, INC
Geotechnical Consultants
West Linn, OR

FIG. 3

USC	SOIL DESCRIPTION S. Elevation - N.A.	DEPTH IN FEET	SAMPLE NO.
	FINE SAND (FILL) - Very loose to loose, gray, with embedded wood and debris, and also containing existing vertical treated wood timber piles of unknown lengths		
	SILT - Soft, gray, varying amounts of fine sand from trace to fine sandy, with sand/ gravel layer at an approximate depth of 35 feet beneath current ground surface, also containing scattered organic material in form of wood and reeds	32.0	
	GRAVEL - Dense becoming more dense, gray, possibly cemented at drilling refusal depth of 80 feet or basalt rock	74.0	S-1 ‡
	DRILLING REFUSAL **DRILLING COMPLETED 5/09/03**	80.0	



○ WATER CONTENT

NOTES:
1. SOIL DESCRIPTIONS AND INTERFACES ARE INTERPRETIVE AND ACTUAL CHANGES MAY BE GRADUAL.
2. WATER LEVEL IS FOR DATE SHOWN AND MAY VARY WITH TIME OF YEAR.

LEGEND

‡ 2.0 " O.D. SPLIT SPOON SAMPLE
3.0" O.D. THIN-WALL SAMPLE
* SAMPLE NOT RECOVERED
P SAMPLE PUSHED
USC UNIFIED SOIL CLASSIFICATION

IMPERVIOUS SEAL
▽ WATER LEVEL
SLOTTED TIP

ATTERBERG
LIQUID LIMIT
NATURAL WATER
PLASTIC LIMIT

CONVENTION CENTER (RIVER SITE)
St. Helens, Oregon

LOG OF BORING B-3

May, 2003 **W-1744**

WEST COAST GEOTECH, INC
Geotechnical Consultants
West Linn, OR

FIG. 4

APPENDIX D

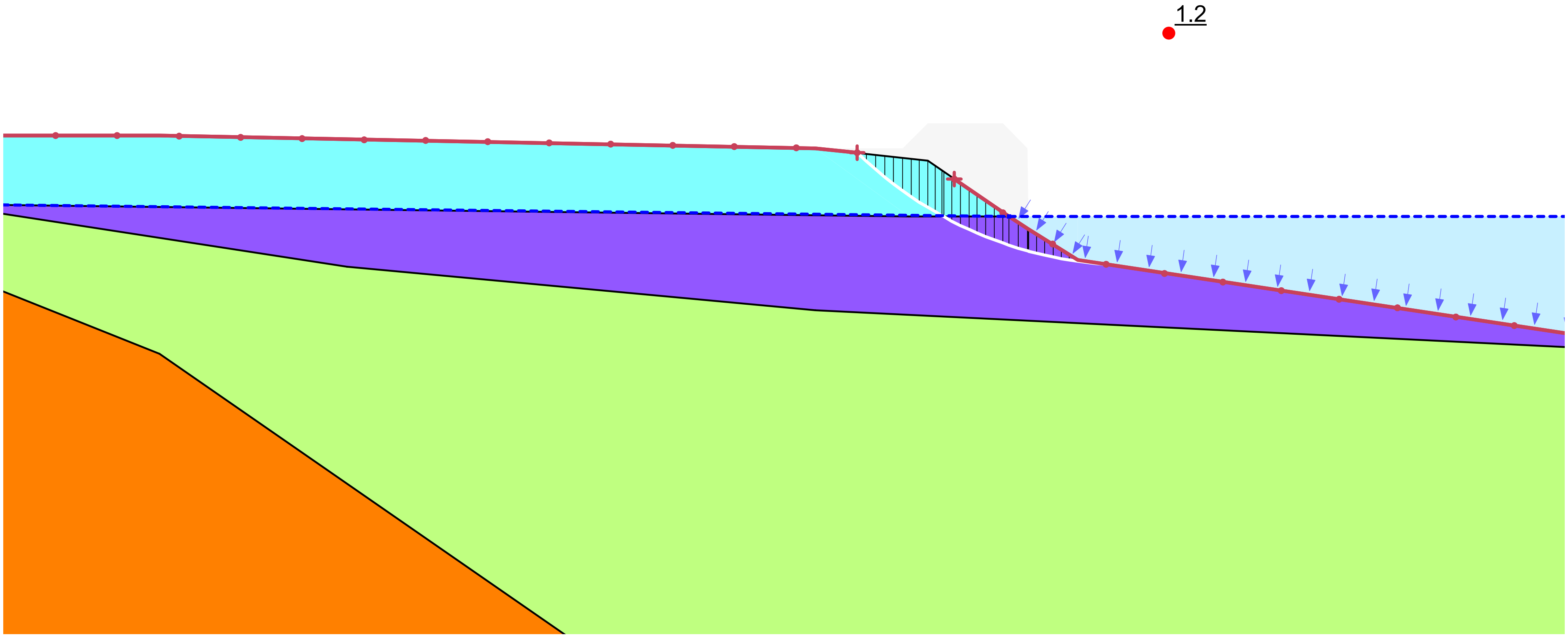
APPENDIX D

SLOPE/W AND SIGMA/W ANALYSIS

Plots providing the input parameters, cross section, and results of our stability and settlement analyses are presented in this appendix.

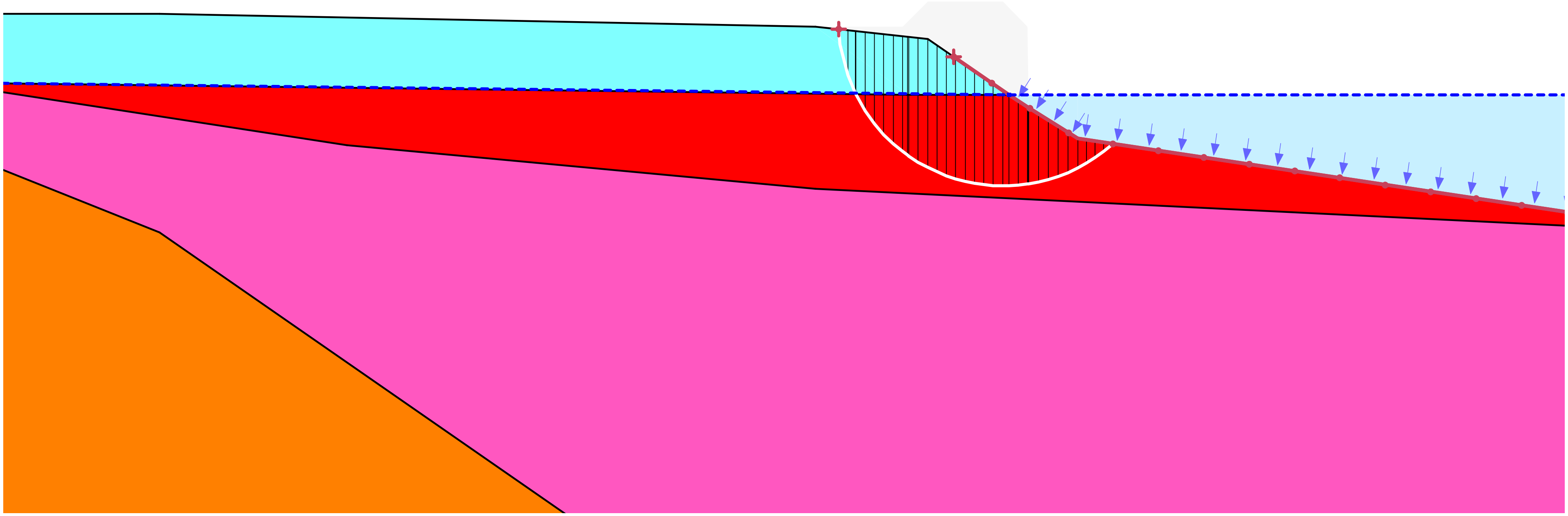
Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟧	Basalt			
🟪	Liquefiable Saturated Sand	105	0	32
🟩	Medium Stiff Silt	105	100	29
🟦	Sand and Gravel Fill	105	0	33
🟩	Soft Silt	105	100	25

Static Stability Analysis



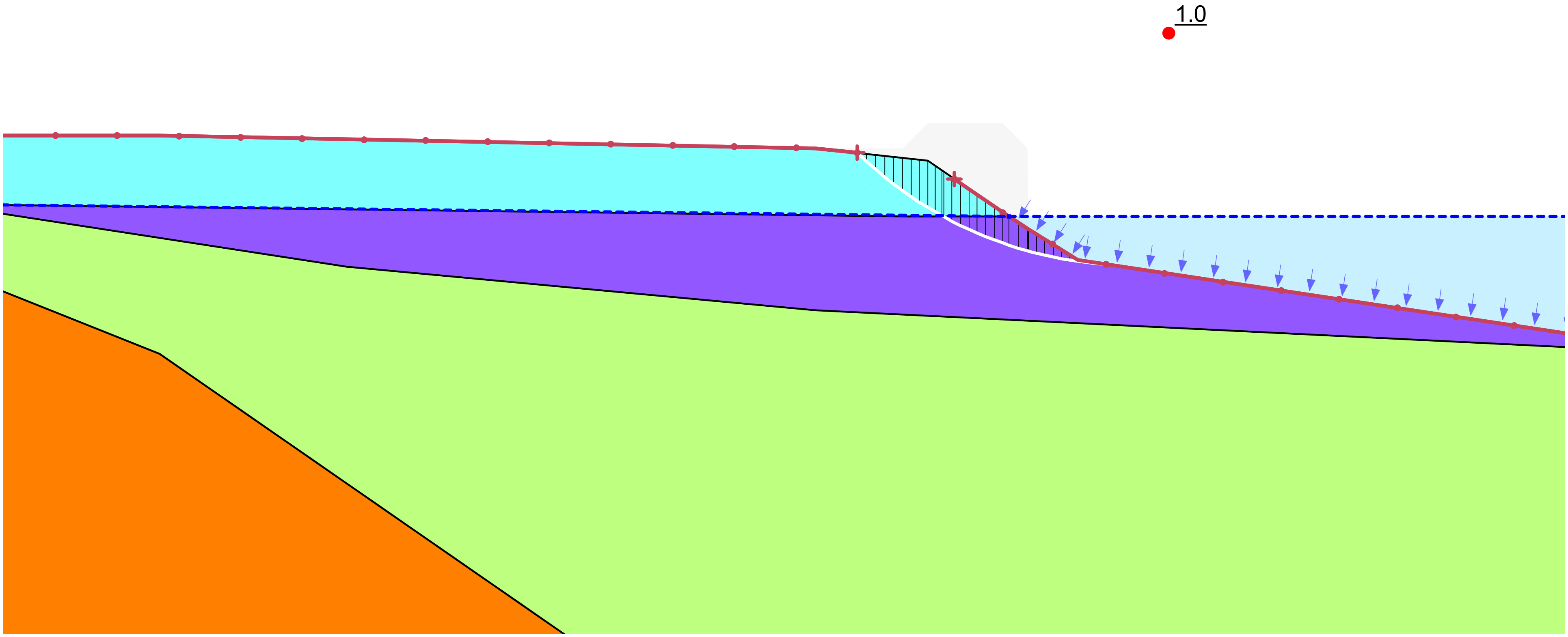
Post Liquefaction Stability Analysis

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Cohesion (psf)
🟧	Basalt				
🟪	Cyclic Softened Silt	105			500
🟥	Liquified Sand	105			250
🟩	Medium Stiff Silt	105	100	29	
🟦	Sand and Gravel Fill	105	0	33	



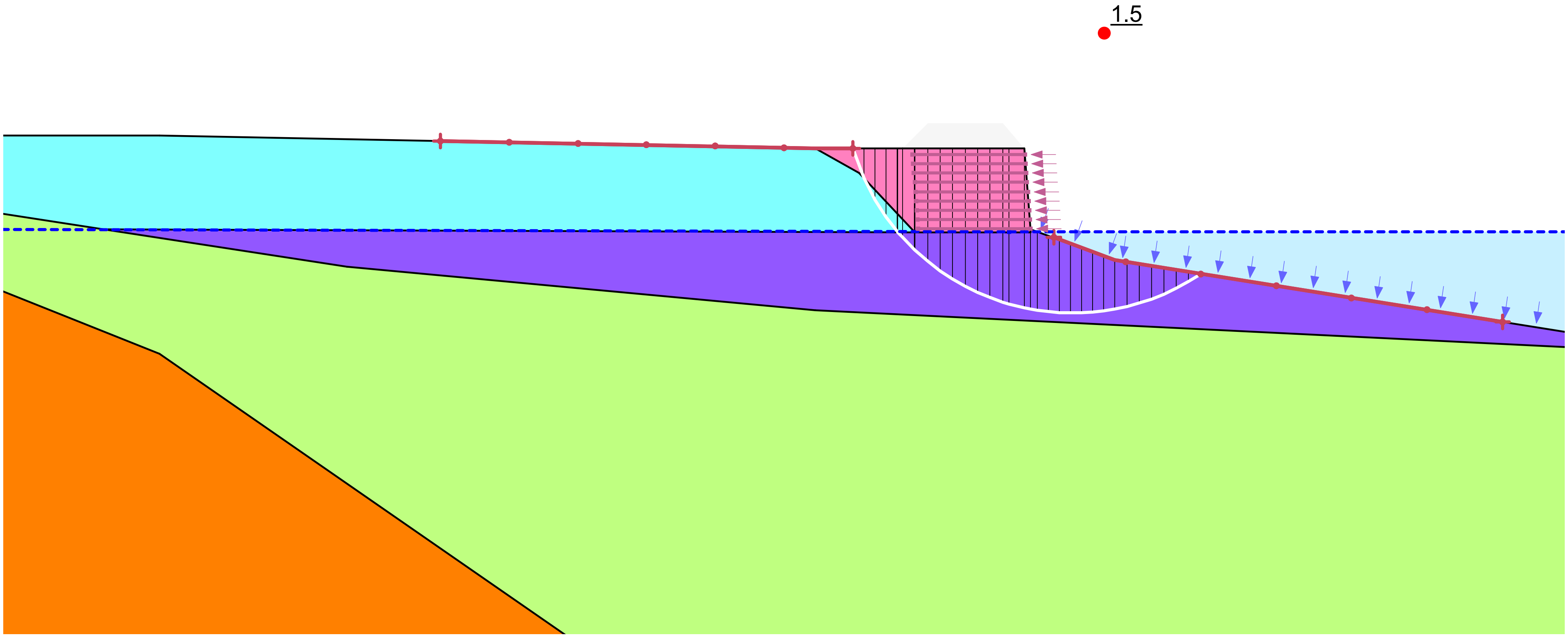
Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟧	Basalt			
🟪	Liquefiable Saturated Sand	105	0	32
🟩	Medium Stiff Silt	105	100	29
🟦	Sand and Gravel Fill	105	0	33
🟩	Soft Silt	105	100	25

Static Stability Analysis
Yield PGA = 0.08g



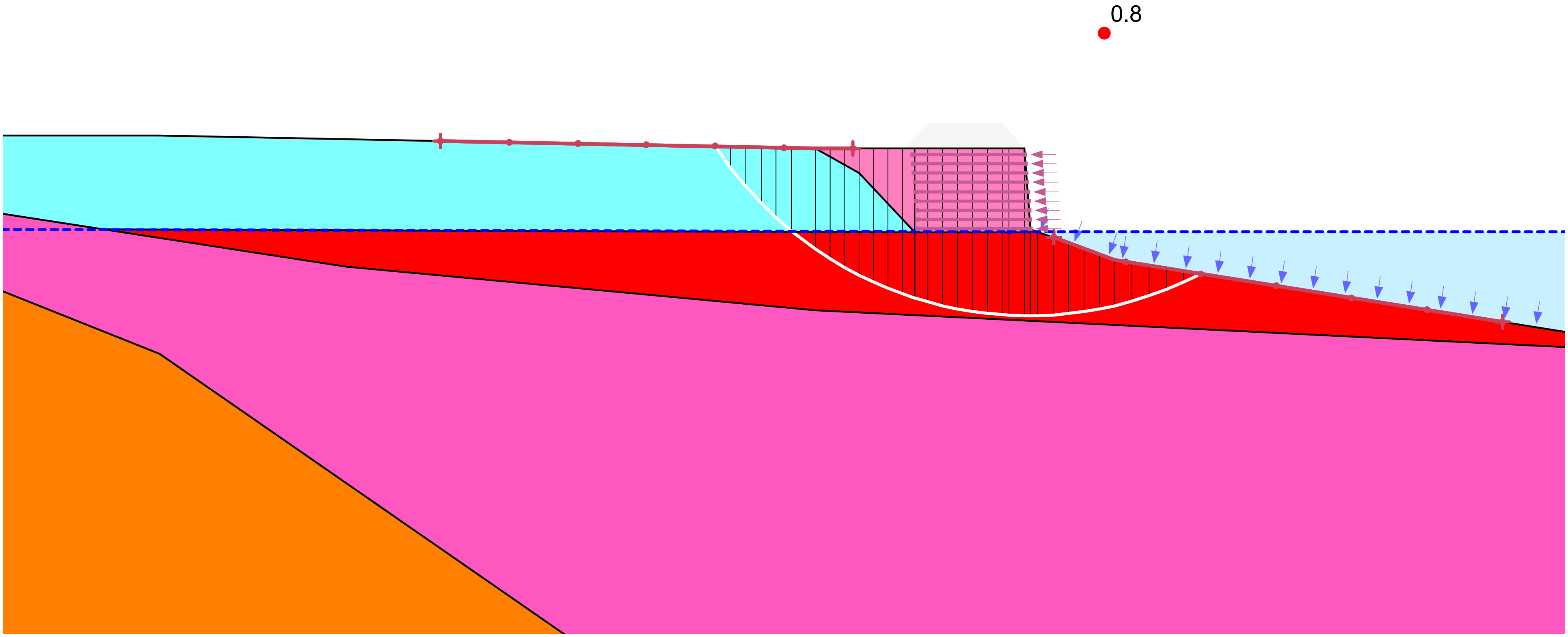
Static Stability Analysis with Wall

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟧	Basalt			
🟥	Crushed Rock Fill	128	0	39
🟪	Liquefiable Saturated Sand	105	0	32
🟩	Medium Stiff Silt	105	100	29
🟦	Sand and Gravel Fill	105	0	33
🟩	Soft Silt	105	100	25



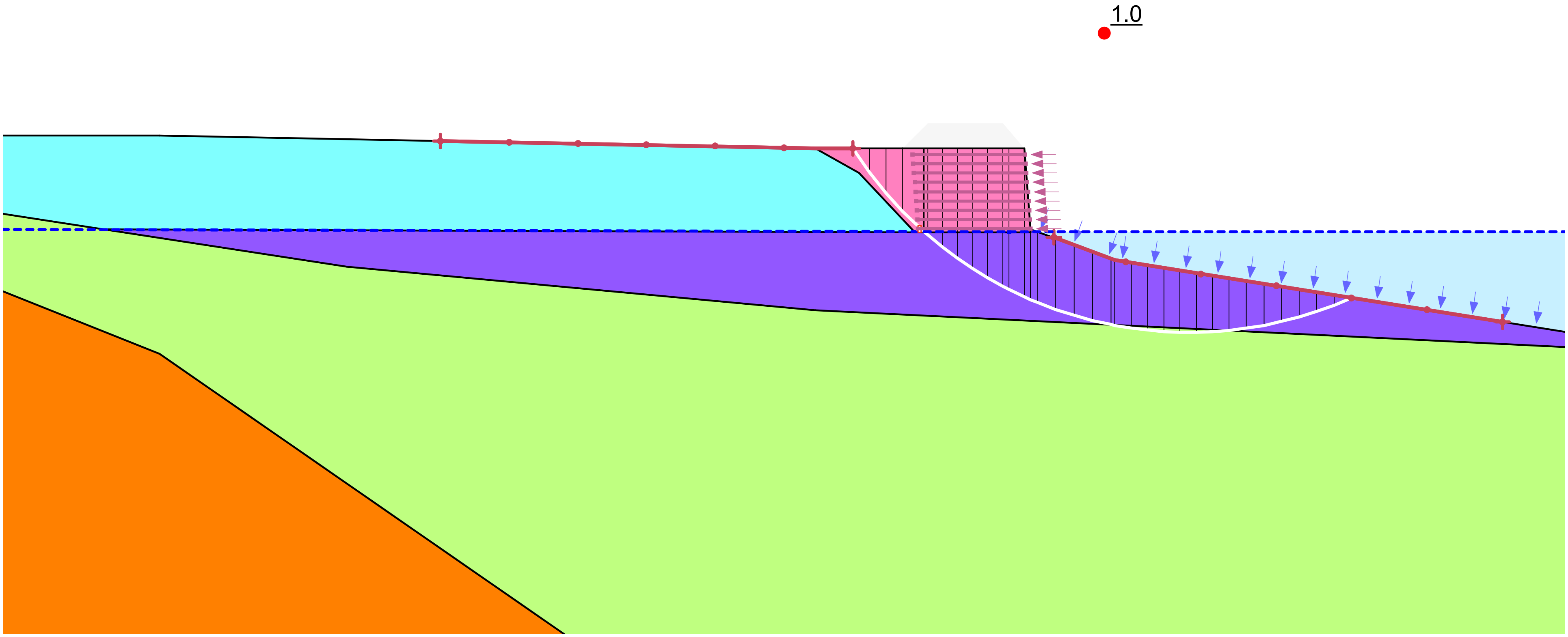
Color	Name	Unit Weight (pcf)	Cohesion (psf)	Cohesion' (psf)	Phi' (°)
🟧	Basalt				
🟪	Crushed Rock Fill	128		0	39
🟪	Cyclic Softened Silt	105	500		
🟥	Liquified Sand	105	250		
🟩	Medium Stiff Silt	105		100	29
🟦	Sand and Gravel Fill	105		0	33

Post Seismic Wall Stability



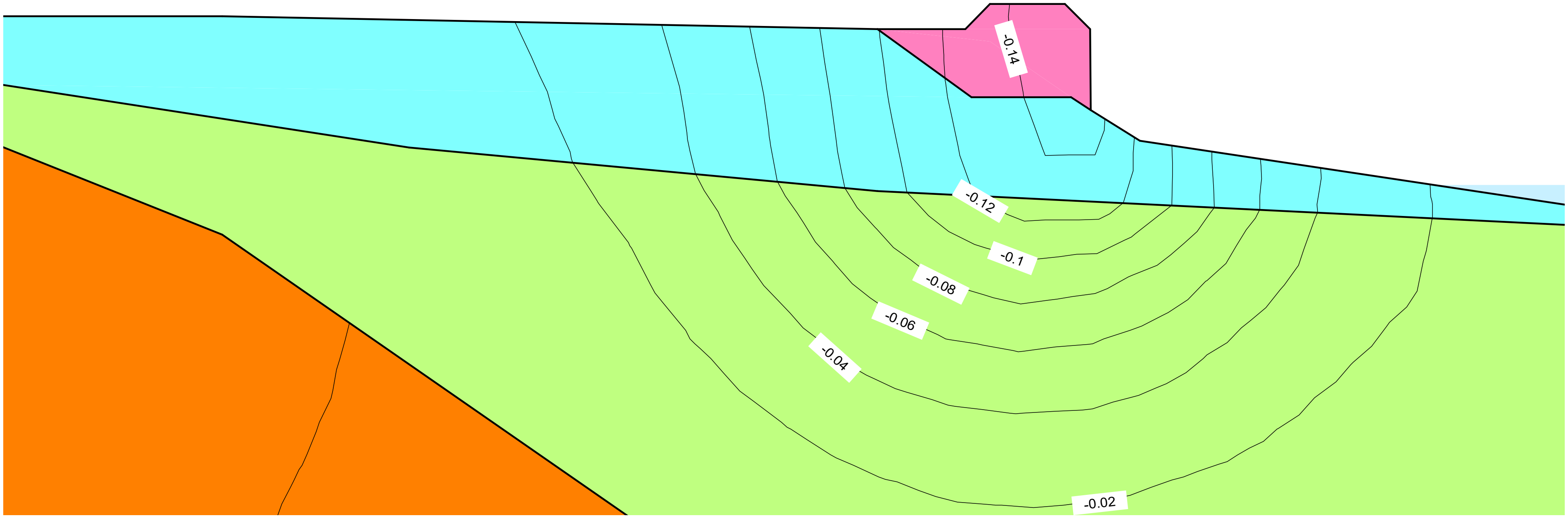
Color	**Name**	**Unit Weight (pcf)**	**Cohesion' (psf)**	**Phi' (°)**
🟧	Basalt			
🟥	Crushed Rock Fill	128	0	39
🟪	Liquefiable Saturated Sand	105	0	32
🟩	Medium Stiff Silt	105	100	29
🟦	Sand and Gravel Fill	105	0	33
🟩	Soft Silt	105	100	25

Wall Stability Analysis -Yield acceleration 0.15g with Static Strengths



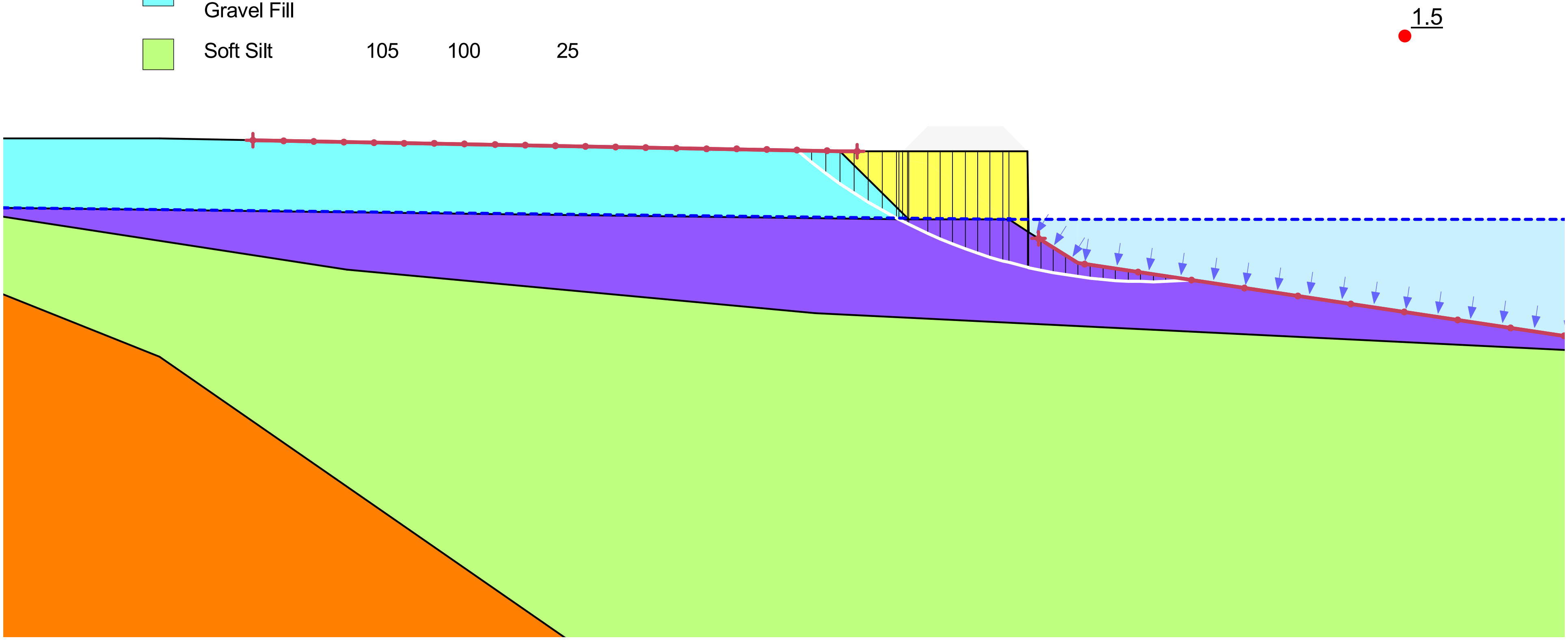
Color	Name	O.C. Ratio	Effective Young's Modulus (E') (psf)	Unit Weight (pcf)	Poisson's Ratio	Lambda	Kappa
🟧	Basalt		5e+08	125	0.334		
🟪	Crushed Rock Fill		2,000,000	128	0.334		
🟦	Sand and Gravel Fill		900,000	105	0.35		
🟩	Soft Silt	1.05		105	0.35	0.13	0.008

Settlement in feet from Wall and Surcharge Load



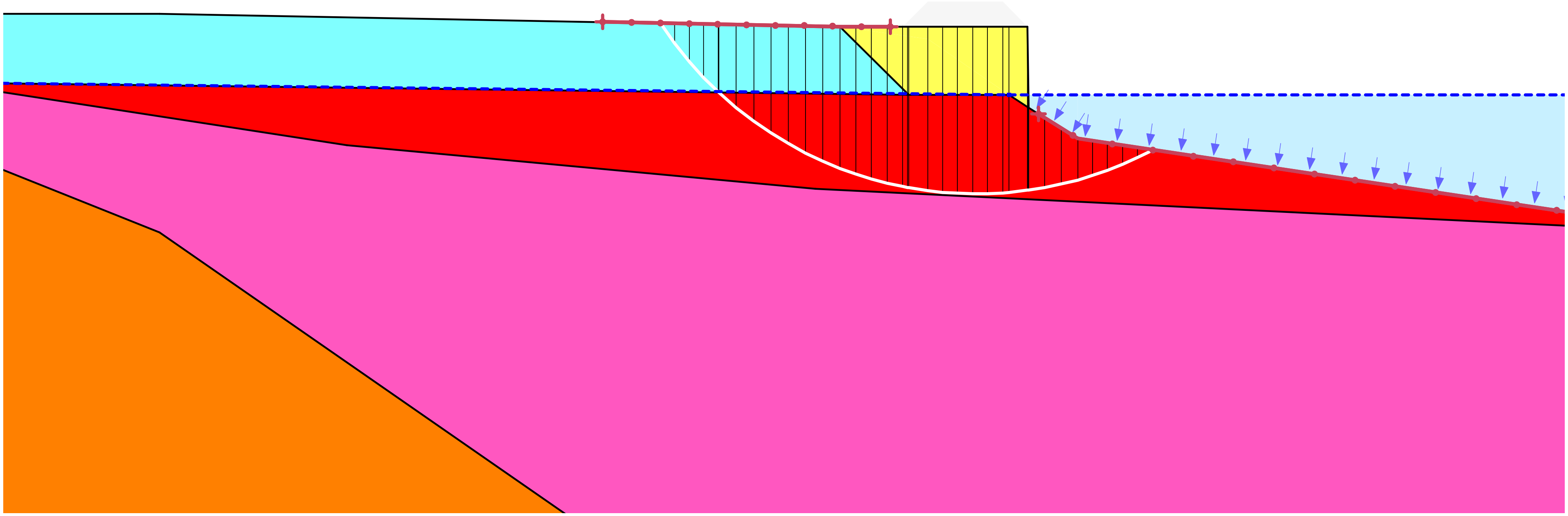
Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟧	Basalt			
🟨	Cellular Concrete	35	7,000	0
🟪	Liquefiable Saturated Sand	105	0	32
🟩	Medium Stiff Silt	105	100	29
🟦	Sand and Gravel Fill	105	0	33
🟩	Soft Silt	105	100	25

Static Stability Analysis with Cellular Concrete Backfill



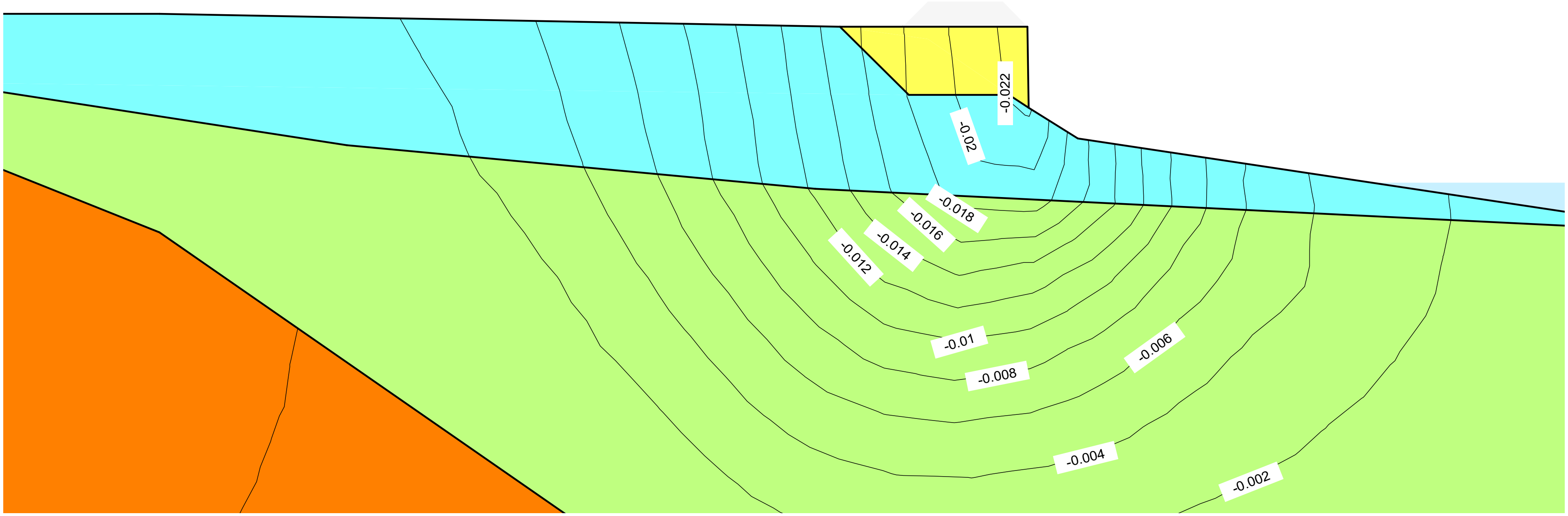
Color	Name	Unit Weight (pcf)	Cohesion (psf)	Cohesion' (psf)	Phi' (°)
🟧	Basalt				
🟨	Cellular Concrete	35		7,000	0
🟪	Cyclic Softened Silt	105	500		
🟥	Liquified Sand	105	250		
🟩	Medium Stiff Silt	105		100	29
🟦	Sand and Gravel Fill	105		0	33

Post Liquefaction Stability Analysis with Cellular Concrete Backfill



Color	Name	O.C. Ratio	Effective Young's Modulus (E') (psf)	Unit Weight (pcf)	Poisson's Ratio	Lambda	Kappa
🟧	Basalt		5e+08	125	0.334		
🟨	Cellular Concrete		4,000,000	35	0.334		
🟦	Sand and Gravel Fill		900,000	105	0.35		
🟩	Soft Silt	1.05		105	0.35	0.13	0.008

Settlement in feet from Wall with Cellular Concrete Backfill



APPENDIX E

APPENDIX E

MSEW ANALYSIS

Results from our MSEW analysis are presented in this appendix.

AASHTO 98 ASD DESIGN METHOD
St Helens Riverwalk Wall

MSEW(3.0): Update # 14.95

PROJECT IDENTIFICATION

Title: St Helens Riverwalk Wall
Project Number: StHelens-4-03
Client: St Helens
Designer: TAP
Station Number:

Description:

Company's information:

Name: NV5
Street: 703 Broadway Street
Suite 650
Vancouver, WA 98660
Telephone #: 360.693.8416
Fax #: 360.693.8426
E-Mail: tyler.pierce@nv5.com

Original file path and name: J:\S-Z\StHelens\StHelens-4\StHelens-4-03\Analysis\SstHe.....
.....tHelens MSE Wall.BEN
Original date and time of creating this file: Tue Nov 30 13:59:13 2021

PROGRAM MODE: ANALYSIS
of a SIMPLE STRUCTURE
using GEOGRID as reinforcing material.

SOIL DATA

REINFORCED SOIL

Unit weight, γ	135.0 lb/ft ³
Design value of internal angle of friction, ϕ	38.0 °

RETAINED SOIL

Unit weight, γ	110.0 lb/ft ³
Design value of internal angle of friction, ϕ	30.0 °

FOUNDATION SOIL (Considered as an equivalent uniform soil)

Equivalent unit weight, $\gamma_{equiv.}$	110.0 lb/ft ³
Equivalent internal angle of friction, $\phi_{equiv.}$	30.0 °
Equivalent cohesion, $c_{equiv.}$	0.0 lb/ft ²

Water table is at wall base elevation

LATERAL EARTH PRESSURE COEFFICIENTS

Ka (internal stability) = 0.2379 (if batter is less than 10°, Ka is calculated from eq. 15. Otherwise, eq. 38 is utilized)
Inclination of internal slip plane, $\psi = 64.00°$ (see Fig. 28 in DEMO 82).
Ka (external stability) = 0.3333 (if batter is less than 10°, Ka is calculated from eq. 16. Otherwise, eq. 17 is utilized)

BEARING CAPACITY

Bearing capacity coefficients (calculated by MSEW): Nc = 30.14 N γ= 22.40

SEISMICITY

Maximum ground acceleration coefficient, A = 0.154
Design acceleration coefficient in Internal Stability: Kh = Am = 0.200
Design acceleration coefficient in External Stability: Kh_d = 0.200 => Kh = Am = 0.200

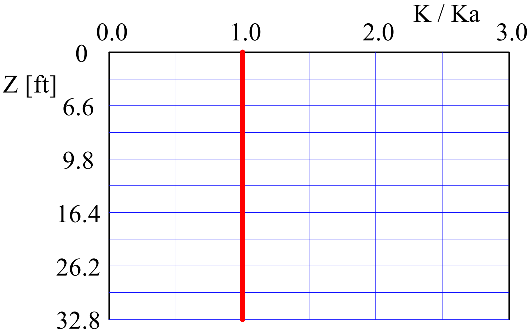
Kae (Kh > 0) = 0.4277 Kae (Kh = 0) = 0.2878 Δ Kae = 0.1399
Seismic soil-geogrid friction coefficient, F* is 80.0% of its specified static value.

INPUT DATA: Geogrids
(Analysis)

D A T A	Geogrid type #1	Geogrid type #2	Geogrid type #3	Geogrid type #4	Geogrid type #5
Tult [lb/ft]	3600.0	5000.0	6200.0	2025.0	
Durability reduction factor, RFd	1.30	1.30	1.30	1.30	
Installation-damage reduction factor, RFid	1.19	1.19	1.19	1.19	
Creep reduction factor, RFc	1.51	1.51	1.51	1.51	N/A
Fs-overall for strength	N/A	N/A	N/A	N/A	
Coverage ratio, Rc	1.000	1.000	1.000	1.000	
Friction angle along geogrid-soil interface, ρ	21.33	21.33	21.33	21.33	
Pullout resistance factor, F*	0.80·tanϕ	0.80·tanϕ	0.80·tanϕ	0.80·tanϕ	N/A
Scale-effect correction factor, α	0.8	0.8	0.8	0.8	

Variation of Lateral Earth Pressure Coefficient With Depth

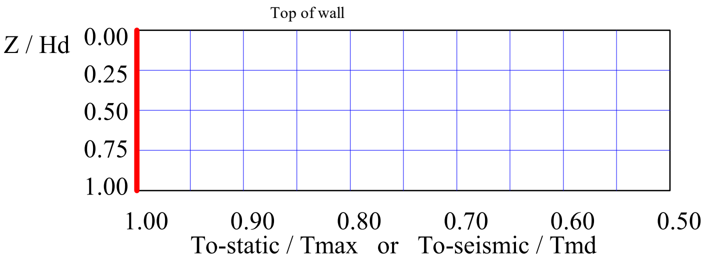
Z	K / Ka
0 ft	1.00
3.3 ft	1.00
6.6 ft	1.00
9.8 ft	1.00
13.1 ft	1.00
16.4 ft	1.00
19.7 ft	1.00



INPUT DATA: Facia and Connection
(Analysis)

FACIA type: Facing enabling frictional connection of reinforcement (e.g., modular concrete blocks, gabions)
Depth/height of block is 1.50/1.50 ft. Horizontal distance to Center of Gravity of block is 0.75 ft.
Average unit weight of block is $\gamma_f = 135.00$ lb/ft^3

Z / Hd	To-static / Tmax or To-seismic / Tmd
0.00	1.00
0.25	1.00
0.50	1.00
0.75	1.00
1.00	1.00



Geogrid Type #1		Geogrid Type #2		Geogrid Type #3		Geogrid Type #4		Geogrid Type #5	
σ (1)	CRu (2)	σ	CRu	σ	CRu	σ	CRu	σ	CRu
1044.2	0.90	1044.2	0.90	1044.2	0.90	1044.2	0.90		
2506.1	0.90	2506.2	0.90	2506.2	0.90	2506.2	0.90	N/A	

Geogrid Type #1)		Geogrid Type #2		Geogrid Type #3		Geogrid Type #4		Geogrid Type #5	
σ	CRs	σ	CRs	σ	CRs	σ	CRs	σ	CRs
0.0	0.00	0.0	0.00	0.0	0.00	0.0	0.00		
1044.2	0.90	1044.2	0.90	1044.2	0.90	1044.2	0.90	N/A	

(1) σ = Confining stress in between stacked blocks [lb/ft^2]
(2) CRu = Tult-c / Tult
(3) CRs = Tpo-c / Tult

In seismic analysis, Tc-pullout is reduced to 80% of its static value.

D A T A (for connection only)	Type #1	Type #2	Type #3	Type #4	Type #5
Product Name	SF35	SF55	SF65	SF20	N/A
Durability reduction factor, RFd	1.30	1.30	1.30	1.30	N/A
Creep reduction factor, RFc	1.51	1.51	1.51	1.51	N/A
Overall factor of safety: connection break, Fs	N/A	N/A	N/A	N/A	N/A
Overall factor of safety: connection pullout, Fs	N/A	N/A	N/A	N/A	N/A

INPUT DATA: Geometry and Surcharge loads (of a SIMPLE STRUCTURE)

Design height, Hd	15.00	[ft]	{ Embedded depth is E = 1.50 ft, and height above top of finished bottom grade is H = 13.50 ft }
Batter, ω	7.1	[deg]	
Backslope, β	0.0	[deg]	
Backslope rise	0.0	[ft]	Broken back equivalent angle, I = 0.00° (see Fig. 25 in DEMO 82)

UNIFORM SURCHARGE

🟦 Uniformly distributed dead load is 500.0 [lb/ft ^{2}]

ANALYZED REINFORCEMENT LAYOUT:

SCALE:

0 2 4 6 8 10 [ft]

ANALYSIS: CALCULATED FACTORS (Static conditions)

Bearing capacity, Fs = 4.10, Meyerhof stress = 2455 lb/ft².

Foundation Interface: Direct sliding, Fs = 4.045, Eccentricity, e/L = 0.0153, Fs-overturning = 12.14

GEOGRID				CONNECTION							
#	Elevation [ft]	Length [ft]	Type #	Fs-overall [pullout resistance]	Fs-overall [connection break]	Fs-overall [geogrid strength]	Geogrid strength Fs	Pullout resistance Fs	Direct sliding Fs	Eccentricity e/L	Product name
1	0.00	19.50	1	7.34	3.74	3.49	3.491	105.220	2.736	0.0153	SF35
2	1.50	19.50	1	3.91	1.99	1.86	1.860	51.548	2.997	0.0103	SF35
3	3.00	19.50	1	4.28	2.18	2.04	2.037	51.177	3.314	0.0060	SF35
4	4.50	19.50	1	4.74	2.41	2.25	2.252	50.066	3.708	0.0024	SF35
5	6.00	19.50	1	5.29	2.70	2.52	2.518	48.542	4.212	-0.0005	SF35
6	7.50	19.50	1	5.82	3.06	2.86	2.856	47.005	4.888	-0.0028	SF35
7	9.00	19.50	1	5.38	3.53	3.30	3.297	45.481	5.850	-0.0042	SF35
8	10.50	19.50	1	4.77	4.18	3.90	3.900	43.957	7.364	-0.0049	SF35
9	12.00	19.50	1	1.87	2.46	2.29	2.294	20.396	10.208	-0.0047	SF35

ANALYSIS: CALCULATED FACTORS (Seismic conditions)

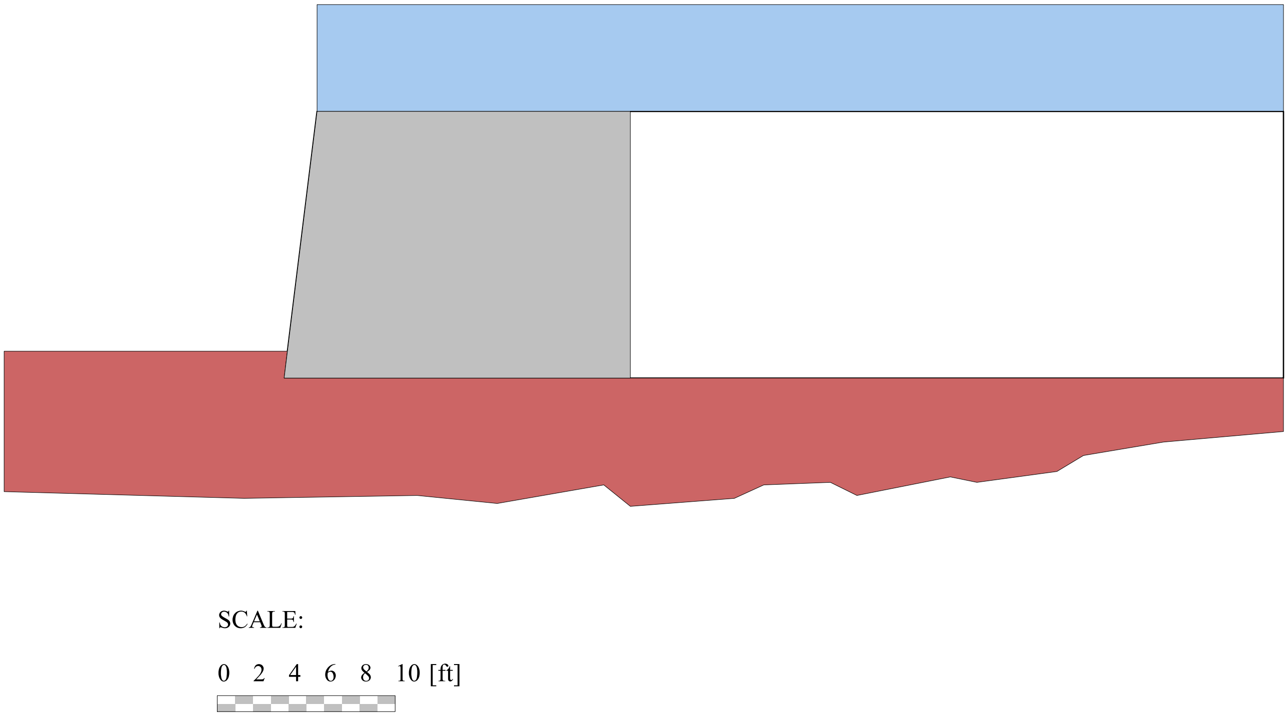
Bearing capacity, Fs = 3.62, Meyerhof stress = 2615 lb/ft².

Foundation Interface: Direct sliding, Fs = 2.642, Eccentricity, e/L = 0.0449, Fs-overturning = 7.23

GEOGRID				CONNECTION							
#	Elevation [ft]	Length [ft]	Type #	Fs-overall [pullout resistance]	Fs-overall [connection break]	Fs-overall [geogrid strength]	Geogrid strength Fs	Pullout resistance Fs	Direct sliding Fs	Eccentricity e/L	Product name
1	0.00	19.50	1	4.48	3.10	2.89	2.893	64.162	1.787	0.0449	SF35
2	1.50	19.50	1	2.69	1.80	1.68	1.680	35.504	1.985	0.0336	SF35
3	3.00	19.50	1	2.92	1.96	1.83	1.829	34.937	2.231	0.0239	SF35
4	4.50	19.50	1	3.20	2.15	2.01	2.007	33.817	2.544	0.0155	SF35
5	6.00	19.50	1	3.53	2.38	2.22	2.224	32.365	2.957	0.0087	SF35
6	7.50	19.50	1	3.82	2.67	2.49	2.493	30.835	3.527	0.0032	SF35
7	9.00	19.50	1	3.45	3.04	2.84	2.836	29.220	4.368	-0.0007	SF35
8	10.50	19.50	1	2.98	3.52	3.29	3.289	27.467	5.742	-0.0032	SF35
9	12.00	19.50	1	1.29	2.22	2.08	2.075	14.077	8.425	-0.0041	SF35

BEARING CAPACITY for GIVEN LAYOUT

	STATIC	SEISMIC	UNITS
(Water table is at wall base elevation)			
Ultimate bearing capacity, q-ult	10068	9454	[lb/ft ²]
Meyerhof stress, σ_V	2455.4	2615	[lb/ft ²]
Eccentricity, e	0.30	0.88	[ft]
Eccentricity, e/L	0.015	0.045	
Fs calculated	4.10	3.62	
Base length	19.50	19.50	[ft]



DIRECT SLIDING for GIVEN LAYOUT (for GEOGRID reinforcements)

Along reinforced and foundation soils interface: Fs-static = 4.045 and Fs-seismic = 2.642

#	Geogrid Elevation [ft]	Geogrid Length [ft]	Fs Static	Fs Seismic	Geogrid Type #	Product name
1	0.00	19.50	2.736	1.787	1	SF35
2	1.50	19.50	2.997	1.985	1	SF35
3	3.00	19.50	3.314	2.231	1	SF35
4	4.50	19.50	3.708	2.544	1	SF35
5	6.00	19.50	4.212	2.957	1	SF35
6	7.50	19.50	4.888	3.527	1	SF35
7	9.00	19.50	5.850	4.368	1	SF35
8	10.50	19.50	7.364	5.742	1	SF35
9	12.00	19.50	10.208	8.425	1	SF35

ECCENTRICITY for GIVEN LAYOUT

At interface with foundation: e/L static = 0.0153, e/L seismic = 0.0449; Overturning: Fs-static = 12.14, Fs-seismic = 7.23

#	Geogrid Elevation [ft]	Geogrid Length [ft]	e / L Static	e / L Seismic	Geogrid Type #	Product name
1	0.00	19.50	0.0153	0.0449	1	SF35
2	1.50	19.50	0.0103	0.0336	1	SF35
3	3.00	19.50	0.0060	0.0239	1	SF35
4	4.50	19.50	0.0024	0.0155	1	SF35
5	6.00	19.50	-0.0005	0.0087	1	SF35
6	7.50	19.50	-0.0028	0.0032	1	SF35
7	9.00	19.50	-0.0042	-0.0007	1	SF35
8	10.50	19.50	-0.0049	-0.0032	1	SF35
9	12.00	19.50	-0.0047	-0.0041	1	SF35

RESULTS for STRENGTH

Live Load included in calculating Tmax

#	Geogrid Elevation [ft]	Tavailable [lb/ft]	Tmax [lb/ft]	Tmd [lb/ft]	Specified minimum Fs-overall static	Actual calculated Fs-overall static	Specified minimum Fs-overall seismic	Actual calculated Fs-overall seismic	Product name
1	0.00	1541	441.46	137.70	N/A	3.491	N/A	2.893	SF35
2	1.50	1541	828.72	133.86	N/A	1.860	N/A	1.680	SF35
3	3.00	1541	756.47	130.01	N/A	2.037	N/A	1.829	SF35
4	4.50	1541	684.21	126.16	N/A	2.252	N/A	2.007	SF35
5	6.00	1541	611.95	122.32	N/A	2.518	N/A	2.224	SF35
6	7.50	1541	539.70	118.47	N/A	2.856	N/A	2.493	SF35
7	9.00	1541	467.44	114.62	N/A	3.297	N/A	2.836	SF35
8	10.50	1541	395.18	110.78	N/A	3.900	N/A	3.289	SF35
9	12.00	1541	671.83	106.93	N/A	2.294	N/A	2.075	SF35

RESULTS for PULLOUT

Live Load included in calculating Tmax

#	Geogrid Elevation [ft]	Coverage Ratio	Tmax [lb/ft]	Tmd [lb/ft]	Le [ft]	La [ft]	Avail.Static Pullout, Pr [lb/ft]	Specified Static Fs	Actual Static Fs	Avail.Seism. Pullout, Pr [lb/ft]	Specified Seismic Fs	Actual Seismic Fs
1	0.00	1.000	441.5	137.7	19.50	0.00	46450.4	N/A	105.220	37160.3	N/A	64.162
2	1.50	1.000	828.7	133.9	18.96	0.54	42718.8	N/A	51.548	34175.1	N/A	35.504
3	3.00	1.000	756.5	130.0	18.41	1.09	38713.4	N/A	51.177	30970.8	N/A	34.937
4	4.50	1.000	684.2	126.2	17.87	1.63	34255.9	N/A	50.066	27404.7	N/A	33.817
5	6.00	1.000	612.0	122.3	17.32	2.18	29705.7	N/A	48.542	23764.5	N/A	32.365
6	7.50	1.000	539.7	118.5	16.78	2.72	25368.6	N/A	47.005	20294.9	N/A	30.835
7	9.00	1.000	467.4	114.6	16.23	3.27	21259.8	N/A	45.481	17007.8	N/A	29.220
8	10.50	1.000	395.2	110.8	15.69	3.81	17371.2	N/A	43.957	13897.0	N/A	27.467
9	12.00	1.000	671.8	106.9	15.14	4.36	13702.9	N/A	20.396	10962.3	N/A	14.077

RESULTS for CONNECTION (static conditions)

Live Load included in calculating Tmax

#	Geogrid Elevation [ft]	Connection force, To [lb/ft]	Reduction factor for connection break, CRu	Reduction factor for connection pullout, CRs	Available connection strength, Tc-break criterion [lb/ft]	Available connection strength, Tc-pullout criterion [lb/ft]	Available Geogrid strength, Tavailable [lb/ft]	Fs-overall connection break		Fs-overall connection pullout		Fs-overall Geogrid strength		Product name
								Specified	Actual	Specified	Actual	Specified	Actual	
1	0.00	441	0.90	0.90	1651	3240	1541	N/A	3.74	N/A	7.34	N/A	3.49	SF35
2	1.50	829	0.90	0.90	1651	3240	1541	N/A	1.99	N/A	3.91	N/A	1.86	SF35
3	3.00	756	0.90	0.90	1651	3240	1541	N/A	2.18	N/A	4.28	N/A	2.04	SF35
4	4.50	684	0.90	0.90	1651	3240	1541	N/A	2.41	N/A	4.74	N/A	2.25	SF35
5	6.00	612	0.90	0.90	1651	3240	1541	N/A	2.70	N/A	5.29	N/A	2.52	SF35
6	7.50	540	0.90	0.87	1651	3142	1541	N/A	3.06	N/A	5.82	N/A	2.86	SF35
7	9.00	467	0.90	0.70	1651	2513	1541	N/A	3.53	N/A	5.38	N/A	3.30	SF35
8	10.50	395	0.90	0.52	1651	1885	1541	N/A	4.18	N/A	4.77	N/A	3.90	SF35
9	12.00	672	0.90	0.35	1651	1257	1541	N/A	2.46	N/A	1.87	N/A	2.29	SF35

RESULTS for CONNECTION (seismic conditions)

Live Load included in calculating Tmax

#	Geogrid Elevation [ft]	Connection force, To [lb/ft]	Reduction factor for connection break, CRu	Reduction factor for connection pullout, CRs	Available connection strength, Tc-break criterion [lb/ft]	Available connection strength, Tc-pullout criterion [lb/ft]	Available Geogrid strength, Tavailable [lb/ft]	Fs-overall connection break		Fs-overall connection pullout		Fs-overall Geogrid strength		Product name
								Specified	Actual	Specified	Actual	Specified	Actual	
1	0.00	579	0.90	0.72	1651	2592	1541	N/A	3.10	N/A	4.48	N/A	2.89	SF35
2	1.50	963	0.90	0.72	1651	2592	1541	N/A	1.80	N/A	2.69	N/A	1.68	SF35
3	3.00	886	0.90	0.72	1651	2592	1541	N/A	1.96	N/A	2.92	N/A	1.83	SF35
4	4.50	810	0.90	0.72	1651	2592	1541	N/A	2.15	N/A	3.20	N/A	2.01	SF35
5	6.00	734	0.90	0.72	1651	2592	1541	N/A	2.38	N/A	3.53	N/A	2.22	SF35
6	7.50	658	0.90	0.70	1651	2513	1541	N/A	2.67	N/A	3.82	N/A	2.49	SF35
7	9.00	582	0.90	0.56	1651	2011	1541	N/A	3.04	N/A	3.45	N/A	2.84	SF35
8	10.50	506	0.90	0.42	1651	1508	1541	N/A	3.52	N/A	2.98	N/A	3.29	SF35
9	12.00	779	0.90	0.28	1651	1005	1541	N/A	2.22	N/A	1.29	N/A	2.08	SF35

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