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**CITY OF AUMSVILLE**

**Project Manual:  
SPECIFICATIONS AND CONTRACT DOCUMENTS**

**For**

**AUMSVILLE WASTEWATER TREATMENT PLANT  
IMPROVEMENTS**



RENEWS: 12/31/27

JO 2599.3220.0

May 2026

**WESTECH ENGINEERING, INC.**  
**3841 Fairview Industrial Dr. SE. ~ Suite 100**  
**Salem, OR 97302-1192**  
**(503) 585-2474**

**THE CITY OF AUMSVILLE  
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# **APPENDIX A**

## **GEOTECHNICAL REPORT**

The attached geotechnical report was utilized in the preparation of the Contract Documents and project design, but was not prepared for purposes of bid development, and its accuracy may be limited based on locations of field observations and tests indicated in the report. The Contractor is cautioned to make a complete evaluation of site soil conditions, including consultation with consultants or geotechnical professionals as applicable. During bidding, the Contractor shall make his/her own deductions and conclusions as to the nature of the materials to be excavated and the difficulties that may arise from subsurface conditions.

Contractor construction submittals shall include adequate information to demonstrate that how the recommendations contained herein are addressed. Where applicable, submittal of the sheeting/shoring/dewatering/coffer dam plans shall demonstrate conformance with the specification requirements and/or recommendations of the geotechnical report.



## Geotechnical Engineering Report:

Aumsville WWTP  
955 Olney Street SE  
Aumsville, Oregon

Central Project: Westech-17-01



### Prepared For:

Westech Engineering, Inc  
3841 Fairview Industrial Drive, SE  
Salem, Oregon 97325



Date: May 15, 2026



7662 SW Mohawk Street  
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(503) 616-9419  
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May 15, 2026

Westech Engineering, Inc.  
3841 Fairview Industrial Drive SE #100  
Salem, OR 97302

Attention: Christopher J. Brugato, PE (cbrugato@westech-eng.com)

Subject Geotechnical Engineering Design Services Report  
Westech-17-01  
Aumsville WWTP Improvements Project  
955 Olney Street SE  
Aumsville, Oregon 97325

Central Geotechnical Services LLC (Central) is pleased to submit this report of geotechnical engineering services for the proposed City of Aumsville Wastewater Treatment Plant Improvements Project in Aumsville, Oregon. Our report has been prepared for conformance with the signed agreement dated October 30, 2023, and authorized May 17, 2024, and in accordance with Contract Amendment #1 dated and authorized June 21, 2024. Our scope of services was further amended based on conditions encountered in the field and ongoing discussions with Westech Engineering, Inc. We appreciate the opportunity to be of service to the project team for this project. Please feel free to call our office with questions about this report.

Respectfully,

Central Geotechnical Services, LLC



Julio C. Vela, PhD, PE, GE  
Principal Engineer

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## 1.0 PROJECT DESCRIPTION

Central Engineering Services (Central) is pleased to submit this report of geotechnical engineering services to Westech Engineering, Inc. (Westech) for the proposed City of Aumsville WWTP Improvements project located at 955 Olney Street SE in Aumsville, Oregon. The overall site is developed with the existing wastewater lagoons, wastewater treatment plant, pump stations, and other wastewater support structures. The location of the site is shown in the Vicinity Map, Figure 1.

Our understanding of the project was developed based on our experience in the vicinity of the project site, and on information provided to us by Chris Brugato of Westech including a preliminary plan set titled "*Drawings for Wastewater Treatment Plant Improvements (preliminary design drawings)*" by Westech dated March 13, 2023; and a geotechnical report previously prepared for an adjacent project on the WWTP site titled "*Geotechnical Engineering Report – City of Aumsville Wastewater Treatment Plant*" dated October 1, 2021 also prepared under the direction of Julio Vela. The previous geotechnical report is included in Appendix D for completeness of data in this report as discussed with Westech.

Based on information provided to us, we understand that the project will include new sequencing batch reactor (SBR) basins, headworks, and blower facility including two 20 foot high SBR basins, a new 40 foot by 60 foot footprint single story storage building, a below grade plant pump station for miscellaneous sanitary drainage to the headworks, a trenchless 24-inch-diameter horizontal SBR discharge pipe beneath Beaver Creek, and various small-diameter piping across Beaver creek that will be installed using either horizontal directional drilling or an auger bore. The 24 inch horizontal-discharge pipe will connect to a new chlorine contact chamber and a DAF building that will be constructed on new embankment fill within the lagoon along the southern bank.

Column and wall structural loads were not available at the time that this report was prepared, but we understand it is planned to be relatively light buildings on slab. We assumed structural loads generally consistent with the proposed structure type and size of up to 50-kip column loads, 2 kips per lineal foot wall loads, and 100 psf floor loads, excluding loads imposed by the SBRs. If it is determined that structural loads will exceed our assumptions, we should be contacted to update our recommendations.

## 2.0 PURPOSE AND SCOPE OF WORK

The purpose of our services was to evaluate soil and groundwater conditions as a basis for developing geotechnical engineering design criteria for the proposed project. Geotechnical work completed under this scope of services was directly supervised by a geotechnical engineer licensed in the state of Oregon, and included the following:

1. Reviewed information regarding subsurface soil and groundwater conditions in the vicinity of the site, including reports in our files, selected geologic maps, and other geotechnical engineering-related information.
2. Coordinated and managed the field investigation, including public utility notification and scheduling of subcontractors and Central field staff. Additionally, we coordinated a private locate company to access the site and mark site utilities.
3. Explored subsurface soil and groundwater conditions at site by completing one drilled boring exploration to a depth of 60 feet below ground surface (bgs) within the footprint of the proposed SBR basins and headworks facility, three borings to final depths ranging from 76.5 feet bgs to 86.5 feet bgs on either side of Beaver Creek near the lateral extents of the proposed horizontal auger bore and Horizontal Directionally installed 4-inch sludge pipe, and two cone penetration tests (CPT's) to final depths between 35 to 50 feet bgs.

4. Obtained samples at representative intervals from the explorations, observed groundwater conditions and maintained detailed logs in general accordance with ASTM International (ASTM) Standard Practices Test Method D 2488. The field activities were observed by qualified staff from our office.
5. Performed laboratory tests on selected soil samples obtained from the explorations to evaluate pertinent engineering characteristics. Specific laboratory tests included moisture content determinations, Atterberg limit determinations, gradation tests, percent fines determinations, and one 1-Dimensional Consolidation test.
6. Completed a site-specific seismic hazard analysis and ground motion hazard analysis in accordance with the requirements of the Oregon State Structural Specialty Code (OSSC) and Chapter 21 of ASCE7, respectively. This included:
  - Coordinated subcontracted geophysical consulting to complete surface wave geophysical testing to develop subsurface shear wave velocity profiles for ASCE7 site class determination.
  - Developed a site-specific risk targeted maximum-considered earthquake response (MCER) spectrum consistent with the methodology of ASCE7-16.
  - Summarized the results of a site-specific seismic hazard analysis completed in accordance with the requirements of the 2022 OSSC.
  - Summarized the results of the seismic evaluation in an appendix to this report.
7. Provided a geotechnical evaluation of the site and provided design recommendations in a geotechnical report that addresses the following geotechnical components:
  - A general description of site topography, geology and subsurface conditions.
  - An opinion as to the adequacy of the proposed development from a geotechnical engineering standpoint.
  - Recommendations for site preparation measures, including disposition of undocumented fill and unsuitable native soils, recommendations for temporary cut slopes and constraints for wet weather construction.
  - Recommendations for temporary excavation and temporary excavation protection, such as excavation sheeting and bracing.
  - Recommendations for earthwork construction, including use of on-site and imported structural fill and fill placement and compaction requirements, and relating to in-lagoon grading for the proposed DAF building and Chlorine contact chamber including soil parameters for cofferdam design and recommendations for placing fill “in the wet”.
  - Recommendations relating to construction of the below grade influent pump station structure including lateral earth pressures and buoyancy and uplift considerations.
  - Recommendations for foundations to support the proposed structures, including minimum width and embedment, design soil bearing pressures, settlement estimates (total and differential), coefficient of friction and passive earth pressures for sliding resistance. Excluding SBR basins, we have assumed that shallow foundations can be used to adequately support the structures.
  - Recommendations for supporting on-grade slabs, including base rock, capillary break, and modulus of subgrade reaction.

### **3.0 SITE CONDITIONS**

#### **3.1 SITE GEOLOGY**

Near surface site geologic conditions are mapped by Walker and Duncan (1989) as consisting of predominately Holocene to Pleistocene-age "Older Alluvium". This unit is described as poorly consolidated deposits of clay, silt, sand, and gravel that are typically mantled with fine grained late-stage facies of the Missoula Flood Sediment better known as Willamette Silt. Although not shown on the above-referenced map, young Holocene aged alluvial deposits from Beaver creek are present locally. In addition, it appears that significant grading has been completed between the south bank of beaver creek and north of the existing structures.

In addition to information from published geology, based on our discussions with an employee onsite during our subsurface explorations, it was indicated that in addition to significant fill that had been placed south of Beaver Creek, the area may also experience a long-term (multiple year) subsurface fire from an accidental ignition post construction. While not directly observed in our explorations that were conducted near the extents of the supposed fill, organic rich soils capable of burning (subsurface smoldering) could be present within site fills.

#### **3.2 SURFACE CONDITIONS**

The overall site consists of developed areas in the southern and northern thirds, and a generally underdeveloped central section where Beaver Creek bisects the site. Existing development consists of four single-story metal storage and maintenance structures along the north embankment of Lagoon Cell 1, and a pump station on the south embankment of Lagoon Cell 4. Site vegetation generally consists of short grasses in maintained areas, and areas of isolated thick shrubbery and blackberry bush and deciduous and coniferous trees along the margins near the proposed auger bore location on the south side of Beaver Creek. Site grades are generally flat or sloping gently down to Beaver Creek, except for slopes along the constructed lagoon embankments. Approximate Locations of our explorations are provided in the Site Map, Figure 2.

#### **3.3 SUBSURFACE CONDITIONS**

Subsurface conditions at the site were explored by drilling five geotechnical borings (B-1 through B-5) and by completing two CPT's on July 30, through August 1, 2024. Exploratory borings were completed to final depths of 60 to 86.5 feet bgs in the vicinity of the proposed structures. CPT's were advanced to final depths of between 35 to 50 feet bgs. Additionally, geophysical surface wave testing was completed by a subcontracted geophysical consultant in the vicinity of the proposed improvements. Approximate locations of explorations completed at the site are presented in Figure 2. The logs of CGS's explorations completed for this study are presented in Appendix A. The geophysical report is presented in Appendix C.

Soil samples obtained during our explorations were taken to Central's laboratory for further evaluation. Selected samples were tested for determination of moisture content, fines content, and Atterberg limits. A description of laboratory testing and test results are presented in Appendix A.

##### **3.3.1 Soil Conditions**

###### **3.3.1.1 SBR Basins**

Soil conditions observed within our explorations in the vicinity of the proposed SBR basins consists of 9 feet of stiff silt (ML) with varying gravel content underlain by dense to very dense sandy gravel observed to extend up to 25 to 35 feet bgs, underlain by very stiff to hard elastic silt (MH) with varying clay content observed to extend up to 50 feet bgs; underlain by R2 sedimentary rock (siltstone). Geophysical surface wave testing results were consistent with the observation of bedrock at an approximate depth of 50 feet bgs within the footprint of the proposed SBR basins.

### 3.3.1.2 Auger Bore/Horizontal Directional and Chlorine Contact Chamber

Soil conditions observed within our explorations near the extents of the proposed auger bore and horizontal directional drill locations were observed to be variable. Within B-3 subsurface conditions consist of the following:

- Medium-stiff light-brown to brown, sandy silt, trace gravel and organics (Fill; 0 -5.5 feet bgs)
- Soft to medium-stiff gray to brown, moist to wet Clay (CL) and silt with clay (MH) (5.5 – 40 feet bgs)
- Very-stiff to hard clay (CL) and silt (MH) (40 – 76.5 feet bgs)

Within B-4 subsurface soil conditions consist of the following:

- Medium-stiff light-brown silt (ML) and medium-dense gravel with sand (GP), trace silt (Fill; 0 – 8.5 feet bgs)
- Very-stiff light-brown gravelly silt (ML; 8.5 – 15.5 feet bgs)
- Medium-dense brown sand (SP; 15.5 – 22 feet bgs)
- Stiff to hard-brown to orange silt and clay (CL&MH; 22 – 81.5 feet bgs)

Within B-5 Subsurface conditions consist of the following:

- Stiff to hard, brown-gray elastic silt (MH; 0 – 10 feet bgs)
- Stiff-brown elastic silt and clay (MH & CL; 10- 19 feet bgs)
- Medium-stiff brown silt with sand and gravel (ML; 19 – 37 feet bgs)
- Stiff to hard multi-color silt and clay (CL; 37 – 86.5 feet bgs)

### 3.3.2 Groundwater Conditions

Groundwater was not directly observed within our explorations because of the mud-rotary drilling method. However, based on the groundwater elevation in nearby Beaver Creek, and on the results of the CPT's, we anticipate areal groundwater may be encountered at depths of about 5 to 15 feet bgs at the locations of our explorations. Groundwater levels observed were relatively shallow and may be perched within the upper soils on top of fine-grained facies. Areal depths to groundwater may be as deep as 40 to 50 feet based on CPT data.

Based on observations on site and data from nearby sites, groundwater sufficient to infiltrate construction excavations may be present at relatively shallow depths in this perched condition on harder underlying layers as it moves downslope during wet times of the year or during extended periods of wet weather. Groundwater conditions at the site are expected to vary seasonally due to rainfall events and other factors not observed in our explorations.

## 4.0 CONCLUSIONS AND RECOMMENDATIONS

Based on our explorations, testing, and analyses, it is our opinion that the site is suitable for the proposed project from a geotechnical standpoint, provided the recommendations in this report are incorporated into project design and implemented during construction. We offer the following conclusions regarding geotechnical engineering design and construction at the site.

- Upper silty-site soils are considered to be highly susceptible to moisture and will likely become disturbed from earthwork during periods of wet weather or when the moisture content of the soil is more than a few percentage points above optimum. Wet weather construction practices will be required, except during the dry summer months.

- The proposed blower building, storage structure, and pump station can be satisfactorily supported on continuous and isolated shallow foundations or mat foundations (as applicable) supported on stiff to dense native site soils or on properly structural fill overlying native soils, or on the underlying dense gravels.
- Soils at the location the proposed tank structures (SBRs, digestors, and equalization basins) generally consists of up to 10 feet of medium stiff or better fine-grained soils underlain by dense gravels. While generally suitable for shallow foundation support of relatively lightly loaded structures, the fine grained soils are considered moderately compressible under the full operating design loads of the proposed tanks. We estimate 1 to 3 inches of long-term settlement could occur for the currently proposed configuration.
- The proposed tank structures may be suitably supported using mat foundations that extend to bear directly on the underlying dense gravels. For this scenario, we anticipate less than 1 inch of total long-term settlement and less than ½-inch differential settlement between adjacent foundation elements up to 50 feet apart.
- Alternatively, the proposed tank structures may be suitably supported on mat foundations constructed over existing subgrade improved with ground improvement such as rigid inclusions or rammed aggregate piers. The settlement of structures supported on rammed aggregate piers or other ground improvements will vary by the contractor's design, but typically are designed to performance specifications of less than 1 inch of total long-term settlement.
- The proposed in-lagoon structures planned to be constructed on approximate 10-foot fill prisms (wedge of fill that becomes deeper downslope to maintain a level top elevation) can be satisfactorily supported using shallow foundations provided the recommendations for preloading or surcharging described in this report are included in the project.
- We anticipate in-lagoon fill prisms could experience 3 to 5 inches of long-term post-construction settlement as they are to be constructed on in-place soils prone to settlement under areal loading. If preloading or surcharging are not incorporated into project construction, structures with foundations constructed within the fill prism footprint would be subject to the same magnitudes of settlement.
- General surface conditions at the site consist of gravel surfaced drives and access spaces, and vegetated areas covered with grasses and shrubs. Minor clearing, stripping, and grubbing will be required. We anticipate a stripping depth of approximately 4 to 6 inches bgs to remove topsoil and organic upper soils. Grubbing and deeper excavations up to several feet will be required to remove root zones of shrubs and trees where encountered. Cleared, stripped, and grubbed materials should be hauled off site and properly disposed of unless otherwise allowed by project specifications for other uses such as landscaping or stockpiling.
- Groundwater was not directly observed during our explorations but is anticipated as shallow as 6 feet bgs. Based on our experience and our observations, groundwater shallower than areal groundwater depths may be present during periods of persistent rainfall. Significant dewatering using sump pumping and wells or well points (may be intensive use depending on extent and duration of excavation) could be required where excavations extend beneath the groundwater table depending on subsurface conditions at the time of construction, and the time of year of construction. If more extensive dewatering is required, such as wells or well points, they should be designed by a registered professional engineer or hydrogeologist, and submitted to the project team for approval prior to implementation.
- Existing site structures and structural features designated for removal from within the footprint of proposed improvements should be demolished and completely removed from the site.

- If possible, existing utilities below proposed new structural areas should be relocated or abandoned and grouted full if left in place.
- Soils at the site below the topsoil are suitable to use as structural fill if they are properly moisture conditioned and compacted. Based on our subsurface observations and laboratory testing, and near surface fine-grained site soils are considered to be highly susceptible to moisture and will be very difficult to work with when wet of optimum. Wet weather construction practices will be required over exposed native soils and to protect exposed subgrades.
- Slabs on grade for the proposed structures can be satisfactorily supported on aggregate base that is founded on firm native soils or on structural fill that extends to the firm native soils. We recommend that slabs on-grade be provided with proper moisture control by constructing the aggregate base as a capillary break and providing a vapor barrier for moisture sensitive applications.

## **5.0 EARTHWORK RECOMMENDATIONS**

### ***SITE PREPARATION***

In general, initial site preparation and primary earthwork operations will include demolition of existing site structures, grading to construct the fill prism for the proposed in-lagoon chlorine contact chamber and DAF building, stripping and grubbing of upper organics where present, removal or relocation of existing site utilities if present beneath proposed development, excavating and filling for foundations and utilities, excavation for construction of a horizontal auger bore, and fine grading to establish final grades.

All existing utilities in proposed earthwork construction areas should be identified prior to excavation. Live utility lines beneath proposed structures should be completely removed or filled with grout to reduce potential settlement of new structures. Soft or loose soil encountered in utility line excavations should be removed and replaced with structural fill where it is located within structural areas.

Debris materials generated during demolition of existing improvements or relocation of utilities should be transported off site for disposal. Existing voids and new depressions created during site preparation, and resulting from removal of existing utilities, or other subsurface elements, should be cleaned of loose soil or debris down to firm soil and backfilled with compacted structural fill. Disturbance to a greater depth should be expected if site preparation and earthwork are conducted during period of wet weather.

### ***5.1 DEMOLITION***

All structures and below ground structural elements to be demolished should be completely removed from proposed structural areas and for a margin of at least 5 feet around proposed structural areas. Proposed structural areas are areas where new structures will be built, including foundations and pavements. Existing utilities that will be abandoned on site should be identified prior to construction. Abandoned utility lines should be completely removed or filled with grout if abandoned and left in place to reduce potential settlement or caving in the future. Materials generated during demolition should be transported off site and properly disposed.

### ***5.2 STRIPPING***

Based on our observations at the site, we estimate that the depth of stripping should be about 8 to 12 inches, where required. Greater stripping depths may be required to remove localized zones of loose or organic soil, and in areas where moderate to heavy vegetation are present, or where surface disturbance from prior use has occurred. Actual stripping depths should be based on field observations at the time of construction. Stripped material should be transported off site for disposal unless otherwise allowed by project specifications for other uses such as landscaping.

### **5.3 CLEARING AND GRUBBING**

Site clearing may be required to remove site vegetation, including grass, shrubs and root zones that are designated for removal. Following clearing, grubbing and excavations up to several feet may be required to remove the root zones of shrubs and trees. In general, roots larger than ½ inch in diameter should be removed. Excavations to remove root zones should be done with a smooth bucket to minimize subgrade disturbance. Grubbed materials should be hauled off site and properly disposed of unless otherwise allowed by the project specifications for other uses such as landscaping, stockpiling or on-site burning.

Existing voids and new depressions created during demolition, clearing, grubbing or other site preparation activities, should be excavated to firm soil and backfilled with imported Select Structural Fill material as defined in this report. Greater depths of disturbance should be expected if site preparation and earthwork are conducted during periods of wet weather.

### **5.4 SUBGRADE PREPARATION AND EVALUATION**

Disturbed materials may be present after demolition and site stripping are complete. Subgrade areas should be prepared to be in a uniform unyielding condition prior to placement of structural fill or structural elements. As discussed in the subgrade protection and wet weather considerations section of this report, because of the fines content, surficial silty and clayey soil can be sensitive to small changes in moisture content and will be difficult, or not possible, to compact adequately during wet weather. While tilling and compacting the subgrade is the economical method for subgrade improvement, it will likely only be possible during extended dry periods and following moisture-conditioning of the soil or on the relatively small overall site if large equipment is required.

Upon completion of site preparation activities, exposed subgrades should be proof-rolled with a fully loaded dump truck or similar heavy rubber-tired construction equipment where space allows to identify soft, loose or unsuitable areas. Probing may be used for evaluating smaller areas or where proof-rolling is not practical. Proof-rolling and probing should be conducted prior to placing fill and should be performed by a representative of Central who will evaluate the suitability of the subgrade and identify areas of yielding that are indicative of soft or loose soil. If soft or loose zones are identified during proof-rolling or probing, these areas should be excavated to the extent indicated by our representative and replaced with structural fill.

During wet weather, or when the exposed subgrade is wet or unsuitable for proof-rolling, the prepared subgrade should be evaluated by observing excavation activity and probing with a steel foundation probe. Observations, probing and compaction testing should be performed by a member of our staff. Wet soil that has been disturbed due to site preparation activities or soft or loose zones identified during probing should be removed and replaced with compacted structural fill.

### **5.5 SUBGRADE PROTECTION AND WET WEATHER CONSIDERATIONS**

Upper fine-grained site soils are considered highly susceptible to moisture. Wet weather construction practices will be necessary if construction takes place during wet weather conditions. It may be necessary to use track-mounted equipment, load removed material into trucks supported on gravel haul roads, use gravel working pads and employ other methods to reduce ground disturbance. The contractor should be responsible for protecting the subgrade during construction.

Earthwork planning should include considerations for minimizing subgrade disturbance. We provide the following recommendations if wet weather construction is considered:

- The ground surface in and around the work area should be sloped so that surface water is directed to a sump or discharge location. The ground surface should be graded such that areas of ponded water do not develop. Measures should be taken by the contractor to prevent surface water from

collecting in excavations and trenches. Measures should be implemented to remove surface water from the work areas.

- Earthwork activities should not take place during periods of heavy precipitation.
- Slopes with exposed soils should be covered with plastic sheeting or similar means.
- The site soils should not be left in a disturbed or uncompacted state and exposed to moisture. Sealing the surficial soils by rolling with a smooth-drum roller prior to periods of precipitation may reduce the extent to which these soils become wet or unstable.
- Construction activities should be scheduled so that the length of time that soil is left exposed to moisture is reduced to the extent practicable.
- Construction traffic should be restricted to specific areas of the site, preferably areas that are not susceptible to wet weather disturbance such as haul roads and areas that are adequately surfaced with working pad materials.
- When on-site, moisture sensitive soils are wet of optimum, they are easily disturbed and will not provide adequate support for construction traffic nor for the proposed development. The use of granular haul roads and staging areas will be necessary to support heavy construction traffic. Generally, a 12- to 16-inch-thick mat of Imported Select Structural Fill should be sufficient for light staging areas for the building pad and light staging activities but is not expected to be adequate to support repeated heavy equipment or truck traffic. The thickness of the Imported Select Structural Fill for haul roads and areas with repeated heavy construction traffic should be increased to between 18 and 24 inches. The actual thickness of haul roads and staging areas should be determined at the time of construction and based on the contractor's approach to site development and the amount and type of construction traffic.
- During periods of wet weather, concrete should be placed as soon as practical after preparing foundation excavations. Foundation bearing surfaces should not be exposed to standing water. Should water infiltrate and pool in the excavation, the water should be removed, and the foundation subgrade should be re-evaluated before placing reinforcing steel or concrete. Foundation subgrade protection, such as a 3- to 4 inch thickness of Aggregate Base/Aggregate Subbase or lean concrete, may be necessary if footing excavations are exposed to extended wet weather conditions.

During wet weather, or when the exposed subgrade is wet or unsuitable for proof-rolling, the prepared subgrade should be evaluated by observing excavation activity and probing with a steel foundation probe. Observations and probing should be performed by a member of our staff. Wet soil that has been disturbed due to site preparation activities, or soft or loose zones identified during probing, should be removed, and replaced with Imported Select Structural Fill.

## **5.6 SHORED EXCAVATIONS**

We understand that the proposed SBR's will be constructed at or near existing grade. However, we offer the following recommendations for shored excavations in the vicinity of the proposed SBR basins should they be required. Where sloping back excavations is not practical or economical compared to shoring, excavations should be shored to support deep excavations. Depending on proposed depths, below-grade well construction may also be completed using a shored excavation in order to limit the lateral extent of a laid-back excavation. Because of the diversity of available shoring systems and construction techniques, the design of temporary shoring is most appropriately left up to the contractor proposing to complete the installation. We recommend that the shoring be designed by a Professional Engineer (PE) licensed in Oregon, and that the PE stamped shoring plans and calculations be submitted to the project team for review prior to construction.

The contractor and shoring designer should be provided with this report and are in the best position to be responsible for selecting an appropriate shoring system that can be suitably constructed for the project. The following paragraphs present recommendations for the type of shoring system and design parameters appropriate for the subsurface conditions at the project.

The soils within the project area consist of medium stiff to very stiff silt and medium dense to very dense sand and gravel with varying silt content observed as shallow as 10 feet bgs . It is likely these soils can be retained using conventional shoring systems such as soldier piles or sheet piles or a box system with internal slide bracing. The following information is not intended to direct the selection of a shoring system, but to provide design parameters for shoring systems.

Temporary shoring should be selected by the contractor performing the work and should be designed for lateral pressures resulting from: the retained soil; surcharge loads resulting from traffic; surcharge loads resulting from construction equipment and temporary stockpiles adjacent to the excavation; and other lateral and surcharge loads that will be imposed on the shoring. Lateral load resistance can be mobilized through the use of braces, tiebacks, anchor blocks, and passive pressures on structural shoring members that extend below the base of the excavation. Recommended soil parameters for temporary shoring within the footprint of the proposed SBR basins are presented in Table 1 below. We can provide additional soil parameters for shoring at different locations at the project site upon request.

**Table 1: Soil Parameters for Design of Temporary Shoring**

Soil Unit	Moist Unit Weight (pcf)	Cohesion (psf)	Internal Angle of Friction (degrees)
Silt and Clay	115	50	28
Sand and Gravel (SP)	130	0	34

Pcf = pounds per cubic foot

Psf = pounds per square foot

Excavation configurations and maintenance of safe working conditions, including temporary excavation stability, should be the sole responsibility of the contractor. All applicable local, state, and federal safety codes should be followed. The temporary shoring design should be submitted to the City’s team for review prior to construction. Additionally, where shored excavations are required adjacent to or within the influence zone of existing improvements, shoring should be designed to limit deformations to existing structures to a suitable performance criteria. If shoring and excavation support cannot be designed to limit deformations, existing improvements should be underpinned to prevent damaging deformations consistent with section 1804.2 of the IBC.

**5.7 UNDERPINNING**

Based on discussions with Westech, we understand that excavations adjacent to or in the vicinity of existing building foundations or utilities may be required to develop the site as planned. Unsupported excavations should be maintained outside of a 1H:1V downward projection from a 5 foot horizontal offset established from the bottom outside edge of existing footings or utility trenches. Excavations within this zone should be supported as described in the Shored Excavations section to support the lateral load imposed by the adjacent structure or utility and to prevent settlement. If shoring is not designed to resist those loads and mitigate

potential settlement, existing foundations should be underpinned in a manner consistent with Section 1804.2 of the IBC.

We recommend that the contractor be responsible for final selection, design, and construction of an appropriate shoring or support system as they are best positioned to know the system types available to them. The design should be stamped by a qualified engineer registered in the State of Oregon. We also recommend that the condition of utilities and adjacent structures be documented with photographs before and after construction to confirm adequate performance of the shoring and to confirm any existing settlement or damage prior to adjacent earthwork did not cause or further any structural distress. Survey points should be established and monitored to detect or document possible settlement or lateral movement of nearby improvements.

### **5.8 DEWATERING**

As discussed in the "Groundwater" section of this report, groundwater was not directly observed in our explorations. However, we do not expect groundwater to be a major factor during shallow excavations and earthwork. Excavations that extend into saturated/wet soils, or excavations that extend into perched groundwater, should be dewatered. Sump pumps are expected to adequately address groundwater encountered in shallow excavations. In addition to groundwater seepage, surface water inflow to the excavations during the wet season can be problematic. Provisions for surface water control during earthwork and excavations should be included in the project plans and should be installed prior to commencing earthwork.

If excavations beneath the groundwater table, or during or after extended periods of wet weather are required, significant dewatering other than sump pumps should be expected. Open pumping, pumped wells, and well points are dewatering options typically used. Maximum drawdown of the water table in the immediate vicinity of pumped wells or well points is often achieved several hours after the start of pumping. However, complete dewatering of the saturated zone within the cone of depression may require several days or weeks. This lag reflects the time it takes for vertical drainage of the water stored in the saturated zone. If used, we recommend pumping from wells or well points at least 3 to 5 days prior to extending deep excavations below the depth of anticipated groundwater.

As noted, shallow excavations that extend into saturated/wet soils should be dewatered. Sump pumps are expected to adequately address light groundwater inflow that may be encountered in relatively shallow excavations or during times following extended dry periods. However, excavations beneath the static groundwater table at the time of construction will require larger pumps and/or systems to seal off the excavation walls from the groundwater. In addition to groundwater seepage and upward groundwater flow into excavations, surface water inflow to the excavations during the wet season can be problematic. Provisions for surface water control during earthwork and excavations should be included in the project plans and should be installed prior to commencing earthwork.

If sandy soil is present at the base of the excavation as anticipated, dewatering should be extended deeper than the base to prevent upward inflow of water that will cause upward heaving from the base. This may also require a section of stabilization material (4 inch- or 2 inch-minus crushed rock) up to 3 feet thick to be placed at the base of the excavation as a working and firm foundation surface. In addition, inflowing water into excavations may result in saturated soils to flow in, commonly referred to as "running sand" conditions. This would necessitate filtered dewatering methods or dewatering externally from the excavation.

The level of effort required for dewatering and degree of base stabilization required will depend to a great extent on the time of year during which construction is accomplished. In general, we recommend that construction be completed in the late summer or early autumn months when the groundwater level is typically at its lowest elevation, or that planning include more robust dewatering systems to be used in deeper

excavations, as necessary. The contractor is in the best position to understand the required construction sequence and should be responsible for the design, installation, monitoring, and maintenance of any required dewatering system(s). If wells or well points are necessary, the contractor should submit a dewatering plan prepared by a registered professional engineer or geologist for review by the project team and the City if required.

Additionally, Lowering the groundwater table beneath existing structures can result in settlement. If a dewatering plan is required, it should be designed to address potential settlement of nearby structures as a result of dewatering.

### **5.9 TRENCH SHORING**

All trench excavations should be made in accordance with applicable Occupational Safety and Health Administration (OSHA) and state regulations. In our opinion, fine grained native site soils are generally OSHA Types B and site granular soils are type C, provided there is no seepage, and excavations occur during periods of dry weather. Temporary excavations deeper than 4 feet should be shored or laid back at an inclination of 1H:1V or flatter for type B and 1.5H:1V for type C if workers are required to enter. Excavations made to construct footings, or other structural elements should be laid back or shored at the surface as necessary to prevent soil from falling into excavations.

It should be expected that unsupported cut slopes will experience some sloughing and raveling if exposed to water. Plastic sheeting, placed over the exposed slope and directing water away from the slope, will reduce the potential for sloughing and erosion of cut slopes during wet weather.

The contractor is responsible for shoring methods and shoring system design. Shoring systems should be designed by a professional engineer before installation.

In our opinion, the contractor will be in the best position to observe subsurface conditions continuously throughout the construction process and to respond to the soil and groundwater conditions. Construction site safety is generally the sole responsibility of the contractor, who also is solely responsible for the means, methods, and sequencing of the construction operations and choices regarding excavations and shoring.

Under no circumstances should the information provided by Central be interpreted to mean that Central is assuming responsibility for construction site safety or the contractor's activities; such responsibility is not being implied and should not be inferred.

### **5.10 COFFERDAM SHORING**

We understand that cofferdams may be employed to construct fill prisms "in the dry" for the proposed in-lagoon fill for the chlorine contact chamber. Cofferdams are considered a special type of temporary shoring structure that are typically contractor-designed in order to effectively dewater an excavation in a body of water. Some types of commonly used cofferdam structures in this area include traditional internally braced interlocking, water-tight steel sheet piles, and proprietary fluid filled membrane type systems such as aquadam®.

If cofferdams are required in order to construct in the dry, they should be designed by a registered professional engineer licensed in the state of Oregon, and the design should be completed in general accordance with the requirements of the 2024 Oregon Standard Specifications for Construction and the Oregon Department of Transportation Bridge Design Manual as necessary to successfully place and compact fill "in the dry" or remain as permanent support. Additionally, they should be designed to account for hydrostatic pressures corresponding to the maximum water level in the lagoon, and for active earth pressures induced by structural fill and structural loads, as necessary.

Soil conditions observed within our explorations near potential cofferdams generally consist of soft to very stiff clay to the maximum extents of our explorations. While not directly observed, some very soft lagoon sediment requiring overexcavation is likely to be encountered at subgrade elevation for the proposed fill.

We have provided general soil parameters in Table 2 that can be used for preliminary planning of cofferdam structures. However, the contractor is solely responsible for design and construction of cofferdams, and other shoring structures as previously noted. They should be provided a copy of this report in its entirety in order to formulate their own design parameters as appropriate for their selected system. A full, general design subsurface profile is provided in the "Steel Sheet Piles" section of this report that can be applied to cofferdam design where required.

**Table 2: Preliminary Soil Parameters for Cofferdam Design**

Soil Unit	Moist Unit Weight (pcf)	Cohesion (psf)	Internal Angle of Friction (degrees)
Clay	105	50	28

### 5.11 STRUCTURAL FILL AND BACKFILL

Structural areas include areas beneath foundations, floor slabs, pavements, and any other areas intended to support structures or within the influence zone of structures. Fill intended for use in structural areas should meet the criteria for structural fill presented below. All structural fill soils should be free of debris, clay balls, roots, organic matter, frozen soil, human-made contaminants, particles with greatest dimension exceeding 4 inches (3-inch-maximum particle size in building footprints) and other deleterious materials.

The suitability of soil for use as structural fill will depend on the gradation and moisture content of the soil. As the amount of fines in the soil matrix increases, the soil becomes increasingly sensitive to minor changes in moisture content and achieving the required degree of compaction becomes more difficult or impossible. Recommendations for suitable fill material are provided in the following sections.

#### 5.11.1 Reuse of On-Site Soil

On-site near surface soil generally consists of native silt or gravel fill with varying silt content. On-site soils can be used as structural fill, provided the material meets the above requirements. Materials with appreciable fines with high moisture will likely be unsuitable as structural fill during most of the year. If the soil is too wet to achieve satisfactory compaction, moisture conditioning by drying back the material will be required. If the material cannot be properly moisture conditioned (dried back), we recommend using imported material for structural fill.

An experienced field representative from Central should determine the suitability of on-site soil encountered during earthwork activities for reuse as structural fill.

#### 5.11.2 Imported Select Structural Fill

Imported Select Structural Fill may be used as structural fill and should consist of pit or quarry run rock, crushed rock, or crushed gravel and sand that is fairly well-graded between coarse and fine sizes (approximately 25 to 65 percent passing the U.S. No. 4 sieve). It should have less than 5-percent passing the U.S. No. 200 sieve and have a minimum of 75 percent fractured particles according to American Association of State Highway and Transportation Officials (AASHTO) T-335.

### 5.11.3 Aggregate Base

Aggregate base material located under floor slabs crushed rock used in footing overexcavations should consist of imported clean, durable, crushed angular rock. Such rock should be well-graded, have a maximum particle size of 1 inch and have less than 5 percent passing the U.S. No. 200 sieve (3 percent for retaining walls). In addition, aggregate base shall have a minimum of 75 percent fractured particles according to AASHTO T-355 and a sand equivalent of not less than 30 percent based on AASHTO T-176.

### 5.11.4 Trench Backfill

Backfill for pipe bedding and in the pipe zone should consist of well-graded granular material with a maximum particle size of  $\frac{3}{4}$  inch and less than 5 percent passing the U.S. No. 200 sieve. The material should be free of organic matter and other deleterious materials. Further, the backfill should meet the pipe manufacturer's recommendations. Above the pipe zone backfill, Imported Select Structural Fill may be used as described above.

### 5.11.5 Open-Graded Base Rock

Open-Graded Base rock material for "choking" of quarry spalls should consist of durable, angular (fractured) fine gravel with sand and negligible fines content. Material meeting gradation criteria for open-graded aggregate in Section 02630.11 of the Oregon Department of Transportation (ODOT) Standard Specifications or ASTM No. 57 should be suitable.

### 5.11.6 Fill Placement and Compaction

Structural fill should be compacted at moisture contents that are within 3 percent of the optimum moisture content as determined by ASTM International (ASTM) Test Method D 1557 (Modified Proctor). The optimum moisture content varies with gradation and should be evaluated during construction. Fill material that is not near the optimum moisture content should be moisture conditioned prior to compaction.

Fill and backfill material should be placed in uniform, horizontal lifts and compacted with appropriate equipment. The appropriate lift thickness will vary depending on the material and compaction equipment used. Fill material should be compacted in accordance with Table 3. It is the contractor's responsibility to select appropriate compaction equipment and place the material in lifts that are thin enough to meet these criteria. However, in no case should the loose lift thickness exceed 18 inches.

While not anticipated, in addition to the recommendations in Table 3 fill placed on slopes steeper than 5H:1V should be "keyed" into the existing ground surface by cutting horizontal benches into the existing slope for fill placement. Bench widths should be at minimum wide enough to accommodate compaction equipment and should be cut at the intersection of the existing slope and the vertical cut of the previous benches.

Fill placed to construct permanent slopes should be placed and compacted beyond the line of the finished slope and cut back to finished grade such that the finished face of the slope consists of material compacted by vertical compaction and not down the face of the slope.

**Table 3. Compaction Criteria**

Fill Type.	Compaction Requirements		
	Percent Maximum Dry Density Determined by ASTM Test Method D 1557 at ± 3% of Optimum Moisture		
	0 to 2 Feet Below Subgrade	> 2 Feet Below Subgrade	Pipe Zone
Fine-grained soils (Non expansive)	92	92	-----
Imported Granular, maximum particle size < 1¼ inch	95	95	-----
Imported Granular, maximum particle size 1¼ inch to 6 inches (3-inch-maximum under building footprints)	n/a (proof-roll)	n/a (proof-roll)	-----
Retaining Wall Backfill*	92	92	-----
Nonstructural Zones	90	90	90

Note:

\* Measures should be taken to prevent over-compaction of the backfill behind retaining walls. We recommend placing the zone of backfill located within 5 feet of the wall in lifts not exceeding about 6 inches in loose thickness and compacting this zone with hand-operated equipment such as a vibrating plate compactor or a jumping jack.

If conditions require that fill be placed and compacted in-the-wet, quarry spalls should be used to fill excavations to the elevation of the existing water. Compaction of quarry spalls could be completed by tamping with an excavator attached bucket.

After the quarry spalls have established a stable working surface above the water table, the quarry spalls should be “choked” with 8 to 12 inches of Aggregate for Open Graded Base rock as define previously in this report to reduce potential migration of finer borrow soil into the void spaces of the coarser underlying quarry spalls. Alternatively, a geotextile separation fabric could be installed between the quarry spalls and overlying structural fill to prevent migration of material.

A representative from Central should evaluate the compaction of each lift of fill. Compaction should be evaluated by compaction testing unless other methods are proposed for oversized materials and are approved by Central during construction. These other methods typically involve procedural placement and compaction specifications together with verification requirements such as proof-rolling.

**6.0 STRUCTURAL DESIGN RECOMMENDATIONS**

Based on our review of the preliminary plan set, we understand that the proposed DAF clarifier/chemical feed buildings will be constructed on new fill prisms within Lagoon Cell 4, and that storage building, blower building, and the SBRs, digestors, and equalization basin (tanks) will be constructed at or near existing grade.

## **6.1 STEEL SHEET PILE WALLS**

The configuration and geometry of steel sheet coffer dams for the chlorine contact chamber were not available at the time that this report was prepared. We can provide additional information or review for specific configurations or design methodologies when additional information relating to these items is available. Recommendations provided below are generally applicable to conditions observed on site, but should be confirmed by the shoring designer to be applicable to their selected system.

### **6.1.1 General**

Steel sheet pile walls are typically used for temporary construction applications such as cofferdams. They generally consist of interlocking z-type sheet piles in a water-tight configuration that can be cantilevered or anchored.

In our experience, cantilevered sheet pile walls can be used where the height of retained soil is about 10 feet or less assuming that anticipated outward displacements are acceptable for nearby existing structures, if any. Based on the information provided to us, we anticipate that sheet pile shoring walls for the chlorine contact chamber will be constructed as cantilever walls. We should be notified to provide additional recommendations as necessary if the project will require anchored sheet piles.

Sheet pile walls are typically contractor designed and furnished. We have provided general design and construction considerations for preliminary planning purposes. The contractor should be responsible for reviewing this report in its entirety and providing a design that is suitable for the project including the use of alternative design parameters to those presented herein.

The wall contractor should be required to submit a design report to Central and the project team to review prior to the commission of construction of the wall. The design should be completed and sealed by a professional engineer registered in the state of Oregon.

As part of their wall design package, the design should include global stability analyses for the wall in both short term and long term, with considerations for the potential for excess pore pressure generation during installation.

We completed a suite of corrosivity tests using subcontracted lab services that included ASTM G57 Standard Test Method for Measurement of Soil Resistivity, EPA 300 Determination of Inorganic Anions by Ion Chromatography for sulfate and Chloride contents, and ASTM G51 Standard Test Method for Measuring pH of Soil. The summary results of the corrosivity lab testing are presented in Appendix A, and can be used for planning sheetpile walls if corrosivity considerations are required.

### **6.1.2 Construction Considerations**

Sheet pile walls are typically installed using high-energy vibratory hammers that can result in slope failures, damaging ground settlement, and excessive vibrations in a widespread area around the point of installation if not accounted for in design. Design of sheet pile walls should consider the potential negative effects of pile installation to adjacent features such as utilities and slope stability of the existing dike.

### **6.1.3 Sheet Pile Wall Design Parameters**

The following preliminary recommendations can be considered for wall design, based on the following assumptions: (1) walls consist of a cantilevered sheet pile system; (2) walls are less than 12 feet in total exposed height; (3) wall backfill consists of free draining imported select structural fill; (4) permanent groundwater buildup behind walls will be mitigated by systematic drainage (perforated pipes connected and properly routed away) or a similar system to prevent buildup of hydrostatic pressures in the retained fill, or hydrostatic pressures will be included in design in addition to the design pressures provided below, and (5) backfill is

generally level behind the wall (no upslope). Re-evaluation of our recommendations will be required if the retaining wall design criteria for the project varies from these assumptions.

In general, there are two methodologies that are commonly employed for design of sheet pile retaining walls including limit equilibrium analyses, which include the use of traditional lateral earth pressure theories such as Rankine or Coulomb, and a soil-structure interaction method using modern design software such as CSHTSSI, PY-WALL, CWALSSI, etc. to include non-linear stress-strain soil-structure characteristics and deformations.

Both methods have provided suitable wall designs historically and are considered feasible for the proposed sheet pile wall. The designer should be responsible for selecting an appropriate design method that is suitable for the proposed and existing improvements.

#### **6.1.3.1 Lateral Earth Pressures**

Lateral earth pressures for use in design of sheet pile walls will vary based on the degree to which the retaining structure is allowed to deflect or rotate around the base. Structures that are allowed to rotate slightly around the base (i.e. displace 0.001 to 0.002 times the exposed wall height) should be designed for active earth pressure conditions using an equivalent fluid density (efd) of 41 pcf above the design water elevation and 21 pcf below assuming the ground surface extends behind the wall a distance of at least twice the wall height. For active earth conditions and an inclined backslope of 2H:1V, an efd of 72 pcf above the design water elevation and 33 below can be used for design. For lesser slopes between flat and 2H:1V, the efp can be linearly interpolated between the recommended values. Fluid pressures below the level of groundwater consider the depth of groundwater the same on both sides of the wall. If depth of water is unequal on the soil support side of the wall, additional hydrostatic pressure from the depth of water should be included as part of design.

Retaining walls, including foundation walls that are restrained against rotation during backfilling, should be designed for an at-rest equivalent fluid unit weight of 61 pcf above the design water elevation and 28 pcf when below the design water elevation assuming that the ground surface extends level behind the wall equal to a distance of at least twice the height of the wall. At-rest retaining walls with an inclined backslope of 2H:1V can be designed using an efd 88 pcf above the water elevation and 40 pcf below the design water elevation. For lesser slopes between flat and 2H:1V, the efp can be linearly interpolated between the recommended values.

Seismically induced lateral forces on sheet pile retaining walls can be calculated using a dynamic force equal to 10H psf, where H is the wall height. This seismic force should be applied with the centroid located at 0.6H from the base of the wall. These values the wall is vertical and unrestrained and the backfill behind the wall is horizontal.

Surcharge loads applied closer than one-half of the wall height should be considered as uniformly distributed horizontal pressures equal to one-third of the distributed vertical surcharge pressure. Backfill for sheet pile walls should be placed and compacted as recommended for structural fill. If differing water elevations could occur between the front and back of the sheet pile wall and wall drainage is not included, hydrostatic pressures equivalent to the full depth of the difference should be included behind the wall.

Passive pressures for cantilever stability can be calculated using an equivalent fluid density of 122 pcf. The recommended efd for passive pressure calculations includes a factor of safety of 1.5 and assumes a sloping ground in front of the wall of approximately 4H:1V, and a submerged condition. We recommend excluding the upper 2-feet of soil from passive pressure calculations due to the potential for disturbance during sheet pile installation and subsequent construction activities.

#### **6.1.3.2 SSI Soil Parameters**

Sheet pile retaining wall design including soil structure interaction effects can be estimated using computer based design programs like Ensoft's PY-Wall. PY-Wall uses lateral soil reaction (p) and lateral deflection (y) curves generalized from field tests of laterally loaded piles and soil input properties to approximate lateral wall

deflections and moments. General input parameters for lateral capacity calculations under static conditions are provided in Table 4 below. Recommended parameters are based on conditions observed in nearby explorations not completed within the lagoon or the existing dike due to access restrictions. Based on site construction history, soils at the base of the existing lagoon and submerged portions of the dike are anticipated to be similar to those observed in out-of-water explorations.

**Table 4. Recommended PY-WALL Soil Parameters <sup>(1)</sup>**

Soil Model	Applicability	Undrained Shear Strength (ksf) <sup>(2)</sup>	Effective Unit Weight (pci) <sup>(1)</sup>	Strain Factor $\epsilon_{50}$	K (pci)
Stiff Clay with No Free Water	Fine grained soils beneath the dike	2.0	0.069	0.010	-
Sand above the GW table	Retained granular structural fill	-	0.078	-	225
Sand below the GW table		-	0.042	-	125

Note:

1 pci = pounds per cubic inch

2 ksf = kips per square foot

## 6.2 FOUNDATION SUPPORT ALTERNATIVES – IN LAGOON STRUCTURES

The planned building structures located over top of new fill prisms within lagoon cell 4 include the DAF clarifier/chemical feed building and the chlorine contact chamber. Recommendations for preloading and surcharge are provided for structures over fill placed in-lagoon that will include significant fills. If lesser fill depths than those noted below are anticipated based on inclusion of permanent retaining walls preloading or surcharging may not be necessary, provided the associated settlement (from shallower fill depths) is tolerable for site structures.

Where required, and based on the information provided to us fill prisms are anticipated to have a maximum fill height of 10 to 12 feet from existing grade, decreasing in depth up the existing detention berm slope (hence, a fill wedge or prism).

Near surface soil conditions observed within our explorations at these locations consist of approximately 40 feet of soft to medium stiff clay that is moderately to highly susceptible to long-term post-construction settlement. For the proposed fill heights, we estimate post construction consolidation settlements of about 3 to 5 inches could occur under the thicker fill sections.

To limit post-construction consolidation settlement of the buildings constructed on new fill prisms, preloading or surcharging can be completed where structures cannot be designed to tolerate the estimated magnitudes of fill-induced consolidation settlement.

While preloading is a technically feasible method to mitigate post-construction settlement, special considerations should be made for tolerance to settlement of the existing embankments that would be induced by preload or surcharge fill. We anticipate about 1 to 3 inches of post construction consolidation settlement could occur along the lagoon edge of the existing embankment (thickest section of fill placement) from the

proposed surcharge fill. In addition to potential settlement, the existing embankment will be sensitive to excess pore pressures generated as a function of the rate of placement and compaction, and magnitudes of surcharge fill.

Excess pore pressures can temporarily reduce the shear strength of soils beneath the fill which can result in localized slope instability. As such, the pace of placement and compaction during construction should allow for dissipation of buildup of pore pressures in a manner to prevent a “mud wave” scenario where shear strengths drop and contribute to slope failure into the lagoon, which typically occurs in new fill embankments when founded on submerged or saturated soft or sensitive soils. An open-standpipe piezometer should be installed within the dike adjacent to the location of proposed fill(s) to monitor excess pore pressure generation during construction. The contractor should also visually monitor the dike during fill placement for signs of distress such as longitudinal cracking, bulging ground, or settlement.

Assuming a surcharge/preload program is successfully completed, proposed lagoon-side structures can be suitably supported using shallow foundation elements using the recommendations presented in the Shallow Foundation Support section of this report. Alternatively, lagoon-side structures could be founded on deep foundation elements if surcharge/preloading are not incorporated into project construction.

### 6.2.1 Preloading/Surcharging

As discussed, areas of planned fill prisms of about 10 to 12 feet in added height (deep side depth) placed near the southern embankment of lagoon cell 4 could be subject to up to 3 to 5 inches of post construction settlement as a result of the fill and application of loads from foundations.

Preloading and surcharging are common methods for mitigating or lessening potential post construction settlement. These methods generally include placing fill on compressible soils in order to load them to the approximate final design loads or beyond. Compressible soils are then allowed to settle (consolidate) for a sufficient period of time prior to construction of foundations, floor slabs, and other settlement sensitive structural elements. Surcharging includes loading compressible soils to a magnitude greater than the anticipated final loads to accelerate consolidation settlement relative to preloading.

The purpose of preloading or surcharging is to pre-induce a portion of the settlements that would otherwise occur when project loads are applied. The preload/surcharge program will also significantly reduce post-construction settlements and potential differential settlements due to variability in areal loading and thickness of compressible soils.

### 6.2.2 Surcharge Material and Configuration

In general, we recommend that preload or surcharge fill be placed to a minimum height that will result in the compressible soils being loaded to a degree of no less than that posed by the final project design area loads (fill and floor loads) sufficient to induce settlements prior to construction. If surcharge material is incorporated as permanent structural fill for fill prisms, it should be placed and compacted in accordance with the fill placement and compaction recommendations of this report. Surcharge fill should be placed over the entire structural footprint and a minimum of 5-feet laterally beyond those edges, where space allows.

In general, we recommend surcharge materials consist of select granular fill as described in the *Structural fill and Backfill* section of this report. Select granular fill is well suited for surcharge fill due to the minimized difficulty in handling and compaction during wet weather and ready availability in the area. Select granular fill can be used in grading portions of the site after the period of surcharge is complete. While select granular fill is generally well suited as surcharge material, oversized materials may be considered for a portion of the fill as necessary to construct the fill prism if constructed “in the wet”. However, if fill is placed in the wet, or if a portion of the surcharge fill is to remain underwater during the surcharge period, the height of the surcharge should be adjusted to account for the buoyancy of the fill.

### 6.2.3 Surcharge Monitoring

During and after placement, surcharge should be monitored to determine the magnitude and rate of settlement. Data will be used to determine whether consolidation of underlying soils has slowed sufficiently to allow removal of the preload. The preloading program should be maintained until well after the primary settlement has been achieved. The time to complete primary settlement will vary as previously noted, but for preliminary planning purposes the primary settlement should occur between 9 to 12 weeks after placement of fill.

To monitor settlement, we recommend placing settlement plates within the area to be preloaded. The plate should be placed on the existing ground surface prior to placing any preload, surcharge, or fill needed to raise site grades. Elevations of the settlement plates should be referenced to a stable benchmark, at least 100 feet outside the fill area to reduce the potential influence of fill placement. This may require establishing a benchmark several hundred feet away from the nearest site filling. Initial elevation readings of the settlement indicators must be obtained when they are installed. Settlement indicators should be surveyed using equipment that is accurate to 1/100 ( $\pm 0.005$  feet). If a settlement indicator or monitoring point is damaged, it should be repaired and resurveyed immediately.

Settlement indicators should be surveyed every day during filling, and one per week thereafter until the preload fill is removed so that settlement progress can be monitored. The settlement data should be provided to us immediately after the readings are taken so that we may review and comment as appropriate. The duration of preloading or surcharging period will be determined based on the settlement readings. For preliminary planning purposes, we expect that required settlement should occur within 6 to 8 weeks following placement of fill. Construction of site improvements should not be started until after the fill settlements are generally complete. The construction documents should emphasize the importance of protecting the settlement indicators from disturbance. The time to sufficiently complete primary settlement may be significantly reduced by the installation of rammed aggregate piers, assuming that they are installed in a manner that allows drainage into the aggregate.

## 6.3 FOUNDATION SUPPORT ALTERNATIVES – TANK STRUCTURES

Based on our review of information provided to us we have assumed that proposed tanks will be supported using mat foundations. We have assumed maximum tributary loads of 1,500 psf to account for the SBR's and Digester tank structures with a maximum ponded water depth of 18 feet, and a maximum tributary load of 900 psf for the proposed equalization basin. If it is determined that loads will exceed the assumed values, we should be contacted to update our recommendations.

Soils observed in the vicinity of these structures consists of a surface section of up to 10 feet of fine-grained silt and clay fill over dense native gravels. These soils are generally suitable for shallow foundation support of the proposed lightly loaded building structures. However, under the relatively heavy tank loads they are moderately susceptible to long-term post construction, consolidation settlements on the order of 2 to 3 inches.

It is our opinion that the underlying dense gravels are suitable for shallow foundation support of the proposed tanks. However, due to the depth to competent gravels and the effect to the hydraulic design, it is likely more economically feasible to support the proposed tank structures on shallow foundations constructed over ground improvements such as rammed-aggregate piers (RAP's) or rigid inclusions.

### 6.3.1 Mat foundations on Dense Gravels

Structural concrete over aggregate base section mat foundations for tank structures founded on the dense underlying gravels, observed at approximately 10 feet bgs, may have concrete sections with flat bottoms or may be thickened at the perimeter and interior where walls or areas of concentrated loading are to be supported.

A structural mat foundation as noted above can be evaluated assuming a subgrade modulus of 125 pounds per cubic inch (pci) if required for isolated load distribution. Local bearing pressures below concentrated loads can be evaluated assuming an allowable soil bearing capacity of 2,500 psf. This bearing value considers combined dead and long-term live loads and may be increased by up to one-third to account for short term loads such as wind or seismic.

### **6.3.2 Ground Improvement/Aggregate Piers**

Shallow foundations supported by RAPs or rigid inclusions can provide higher bearing capacity and reduce total and differential settlement under design loads by creating a stiffened soil matrix subgrade. Ground improvement methods typically include RAPS and rigid inclusion systems designed and constructed by a specialty foundation construction company. Other ground improvement systems/contractors may be considered but should be reviewed and approved by the project team.

RAPs or rigid inclusion systems are typically designed and constructed by the specialty contractor to a performance specification. In our experience they typically range from 18- to 30-inch-diameter piers spaced in a triangular distribution with center-to-center spacing ranging from 6 to 8 feet depending on design loads and tolerable settlement requirements. The piers typically extend from the design bearing stratum elevation to the bottom of a granular load transfer platform beneath foundation elements.

The specialty contractor should be given a copy of our geotechnical report and the opportunity to complete additional explorations if they choose. They should submit a ground improvement design that has been completed and stamped by a registered professional engineer with experience in such projects. We recommend the geotechnical engineer of record review the design on behalf of the Owner, although the specialty contractor will retain responsibility for the design and construction of the ground improvements to the specified performance criteria.

The underlying dense gravel within our explorations conducted in the vicinity of the proposed tanks was encountered at varying depths of about 8 to 10 feet bgs. We anticipate that RAP's or rigid inclusions would extend from the bottom of the shallow foundations to the underlying dense gravels or to a minimum design depth required to meet the specified performance criteria.

Lengths of ground improvements may vary across the site. Ground improvements should be designed to meet the final bearing capacity and settlement tolerance provided by the structural engineer. The specialty contractor would provide final design and in-house quality assurance for the owner during the construction process.

### **6.3.3 Aggregate Piers Bearing Capacity**

Allowable design bearing capacity of the compacted aggregate pier/improved subgrade matrix would be determined by the specialty contractor and will be dependent on actual tank loads and acceptable settlement magnitudes. Typically, bearing capacities of about 4,000 to 5,000 psf in the soil/pier matrix is achievable for soils similar to those observed at the site.

### **6.3.4 Foundation Settlement**

Settlement of foundations supported on aggregate pier improved subgrade, as described above, would depend on the specialty contractor's design. Typically, systems are designed to a performance specification that is normally 1 inch total.

## 6.4 SHALLOW FOUNDATION SUPPORT

Detailed structural loading was not available at the time that this report was prepared. We assumed structural loads generally consistent with the proposed structure type and size; up to 50-kip column loads, 2 kips per lineal foot wall loading, and 100 psf floor loads, excluding the SBR basins. If it is determined that loads will exceed the assumed values, we should be contacted to update our recommendations.

The proposed blower and storage buildings and the below grade pump station can be satisfactorily founded on firm native soils or on compacted structural fill extending to firm native soils. Structures that will be constructed within lagoon cell 4 including the proposed DAF clarifier/chemical feed building, and a chlorine contact chamber can also be suitably supported with shallow foundations provided that the recommendations for preloading/surcharging are incorporated into the project design.

Exterior footings should be established at least 18 inches below the lowest adjacent grade. The recommended minimum footing depth is greater than the anticipated frost depth. Interior footings can be found a minimum of 12 inches below the top of slab. Isolated column and continuous wall footings should have minimum widths of 24- and 18 inches, respectively.

### 6.4.1 Foundation subgrade preparation

Subgrades beneath proposed structural elements should be prepared as described below and in the "Earthworks" section of this report. We recommend loose or disturbed soils resulting from foundation excavation be removed before placing reinforcing steel and concrete. Foundation bearing surfaces should not be exposed to standing water. If water infiltrates and pools in the excavation, the water, along with any disturbed soil, should be removed before placing reinforcing steel and concrete.

We recommend Central observe all foundation subgrades before placing concrete forms and reinforcing steel to determine that bearing surfaces have been adequately prepared and the soil conditions are consistent with those observed during our explorations.

### 6.4.2 Isolated Spread Footings

We recommend conventional footings be proportioned using a maximum allowable bearing pressure of 2,000 psf if supported on firm, fine-grained native soils or on structural fill placed over firm native soils. These bearing pressures apply to the total of dead and long-term live loads and may be increased by one-third when considering earthquake or wind loads. This is net bearing pressure. The weight of the footing and overlying backfill can be ignored in calculating footing sizes.

### 6.4.3 Foundation Settlement

Foundations designed and constructed as recommended, including preloading/surcharging of in-lagoon fill prisms, are expected to experience total settlements of less than 1 inch. If pre-loading or surcharging is not completed for the structures located on the up to 10 feet tall fill prisms within the lagoon, total long term post construction settlements of about 3 to 5 inches could occur. Differential settlements of up to one half of the total settlement magnitude can be expected between adjacent footings supporting comparable loads.

### 6.4.4 Lateral resistance

We recommend **against** including lateral resistance from passive earth pressures for the chlorine contact chamber. Lateral resistance for the chlorine contact chamber should consider only the frictional resistance at its base, using a coefficient of friction between concrete and soil of 0.35. For other structures supported by shallow foundations, passive lateral resistance may be included as recommended below.

Lateral loads can be resisted by a combination of friction between the footing and the supporting soil, and by the passive lateral resistance of the soil surrounding the embedded portions of the footings. A coefficient of friction between the concrete and soil of 0.35 and a passive lateral resistance corresponding to an equivalent fluid density of 250 pcf may be used for design. These values are appropriate for foundation elements that are poured directly against the native soils or surrounded by compacted structural fill.

The passive earth pressure and friction components may be combined, provided the passive component does not exceed two-thirds of the total. Additionally, The passive earth pressure value is based on the assumptions that the adjacent grade is level, and static groundwater remains below the base of the footing throughout the year. The top 1 foot of soil should be neglected when calculating passive lateral earth pressures unless the adjacent area is covered with pavement. The lateral resistance values do not include safety factors.

#### **6.4.5 Drainage Considerations**

We recommend the ground surface be sloped away from the buildings at least 5 percent for a minimum distance of 10 feet measured perpendicular to the face of the wall in accordance with section 1804.4 of the 2018 IBC. All downspouts should be tightlined away from the building foundation areas and should also be discharged into a stormwater disposal system. Downspouts should not be connected to footing drains.

Because of the potential for shallow groundwater, we recommend that perimeter drains be included in the design and be installed at the base of the exterior footings. The perimeter footing drains should be provided with cleanouts and should consist of at least 4-inch-diameter perforated pipe placed on a 3-inch bed of and surrounded by 6 inches of drainage material enclosed in a non-woven geotextile such as Mirafi 140N (or approved equivalent) to prevent fine soil from migrating into the drain material. We recommend against using flexible tubing for footing drainpipes. The perimeter drains should be sloped to drain by gravity to a suitable discharge point, preferably a storm drain. We recommend that the cleanouts be covered and placed in flush-mounted utility boxes. Water collected in roof downspout lines must not be routed to the footing drain lines.

If elevator pits, utility vaults, or other subterranean open structural elements are installed below the expected level of groundwater, we recommend foundation drains be installed as described above. Active dewatering or tightline routing of draining water will be required during wet times of the year at these locations in order to provide a removal pathway.

#### **6.5 FLOOR SLABS**

Satisfactory subgrade support for floor slabs on grade supporting up to 150 psf floor loads can be obtained provided the floor slab subgrade is described in the "Earthwork Recommendations" section of this report. Slabs should be reinforced according to their proposed use and per the structural engineer's recommendations. Subgrade support for concrete slabs can be obtained from the firm native soils underlying the topsoil or on structural fill placed over firm native soils.

We recommend that on-grade slabs be underlain by a minimum 6-inch-thickness of aggregate base in order to provide the structural design support for subgrade reaction as described below and to function as a capillary break material to reduce the potential for moisture migration into the slab. The aggregate base section should be placed as recommended in the "Fill Placement and Compaction" section of this report.

If dry on grade slabs are required, for example at interior spaces where adhesives are used to anchor carpet or tile to the slab, a waterproof liner may be placed as a vapor barrier below the slab. The vapor barrier should be selected by the structural engineer and should be accounted for in the design floor section and mix design selection for the concrete, to accommodate the effect of the vapor barrier on concrete slab curing.

Load-bearing concrete slabs prepared as recommended should be designed assuming a modulus of subgrade reaction (k) of 150 psi per inch. This value is for a 1-foot by 1-foot square plate. The coefficient of subgrade reaction for a foundation varies based on its minimum width according to the following equation:

$$k_s = k_{s1} \left[ \frac{B+1}{2B} \right]^2$$

Where  $k_s$  is the coefficient of subgrade reaction,  $k_{s1}$  is the coefficient of subgrade reaction for a 1-foot by 1-foot plate, and B is the minimum width or lateral dimension of the mat.

We estimate that concrete slabs constructed as recommended will settle less than ½ inch. Floor slab subgrades should be evaluated according to the “Subgrade Evaluation” section of this report.

## **6.6 BELOW GRADE STRUCTURES**

Planned below grade structures for the project include the chlorine contact chamber, effluent pumps and transfer piping, and an influent pump station. Recommendations provided for below grade structures are based on the subsurface conditions observed within CPT-2, those noted in boring B-1 completed by others in the vicinity of the proposed below-grade chlorine contact chamber, and on the preliminary project drawings.

We have based our recommendations on an estimated height of 10 to 12 feet for below grade walls for the chlorine contact chamber and up to 25 feet for the proposed influent pump station.

In addition to recommendations in the Limitations section of this report, Central should be retained to complete sufficient geotechnical construction observations, testing, and consultation during construction to confirm that the conditions encountered at the locations of below grade structures to confirm that they are consistent with those assumed for the recommendations of this section. Foundation recommendations for the proposed chlorine contact chamber are provided in the Foundation Alternatives – In Lagoon Structures section of this report. Foundation recommendations provided in the mat foundations on dense gravels and shallow foundation support sections of this report can be applied to the proposed pump station.

### **6.6.1 Subgrade Preparation**

In addition to the subgrade preparation recommendations in the earthworks section of this report We recommend that the subgrade for the below-grade structures beneath the water table be excavated at least 18 inches deeper than design foundation elevation and filled with ballast material consisting of free-draining crushed rock having a maximum particle size of 4 inches, or 4 inch minus quarry spalls.

### **6.6.2 Buoyancy and Uplift**

Below-grade structures will experience upward buoyancy forces when the water level around the structure is higher than the fluid level inside the structure. Resistance to uplift can be developed by the dead weight of the structure and friction along sides of structures, where appropriate. Frictional resistance can be computed using a friction coefficient of 0.30 applied to the lateral soil pressures. We recommend that lateral soil pressure for uplift resistance be computed using an equivalent fluid density of 40 pcf. In addition to friction and deadweight of the structure, uplift resistance may also be provided by constructing footings that extend beyond structure walls so that the weight of the overlying soil resists a portion of the uplift. For this purpose, the overlying backfill may be assumed to have a submerged unit weight of 48 pcf.

### 6.6.3 Lateral Earth Pressures

Walls for below-grade structures will be restrained from movement during backfilling if constructed using temporary open cuts and temporary shoring. Additionally, below-grade walls will be subject to permanent hydrostatic pressures from static groundwater or lagoon effluent. Therefore, below-grade walls should be designed for lateral earth pressures corresponding to at-rest soil pressures and full hydrostatic pressures. Groundwater levels were not directly observed within our explorations because of the mud rotary drilling method. However, we recommend for design purposes that the static groundwater level for design of below grades walls of the chlorine contact chamber and effluent pumpstation as the high-water elevation within the lagoon cell 4 and 10 feet bgs, respectively. Groundwater levels are expected to fluctuate throughout the year.

For the conditions noted above, we recommend using the design lateral earth pressures for static loading conditions above the ground water table using the equivalent fluid densities summarized in Tables 5 and 6, assuming level ground conditions behind the wall.

**Table 5: At-Rest Lateral Earth Pressures – Influent Pump Station**

Soil Unit	Backslope	Lateral Earth Pressure		
		At Rest Earth Pressure Coefficient	Equivalent Fluid Density above groundwater (pcf)	Equivalent Fluid Density Below groundwater (pcf)
Soft to Medium Stiff Silt	Level	0.53	59	88
Dense Sand and Gravel	Level	0.44	55	90

**Table 6: At-Rest Lateral Earth Pressures – Chlorine Contact Chamber**

Soil Unit	Backslope	Lateral Earth Pressure		
		At Rest Earth Pressure Coefficient	Equivalent Fluid Density above groundwater (pcf)	Equivalent Fluid Density Below groundwater (pcf)
Soft to Medium Stiff Silt and Clay	Level	0.53	59	88

Lateral loads that affect the below-grade structures during seismic loading conditions will result in additional lateral soil pressures on subject walls. If below grade walls are designed for seismic forces, we recommend that the seismic loading be approximated using a uniform lateral pressure equal to 6H psf, where H is the depth (in feet) of the structure below grade. Seismic lateral pressure is in addition to and should be superimposed upon the static soil and hydrostatic pressures.

Lateral soil pressures from traffic or other surcharge loads are excluded from these soil pressures. They should be added separately, as appropriate. Typically, below-grade walls are designed for a surcharge pressure for traffic loading. For traffic loading from passenger vehicles and light trucks, we recommend that below-grade

walls be designed for a uniform surcharge pressure determined by increasing the design height of the wall by 2 feet. *Other surcharge loads should be included as appropriate.*

**SEISMIC DESIGN**

Recommended seismic parameters are based on the conditions encountered during our subsurface exploration program and procedures and requirements outlined in the 2022 IBC and the 2022 Oregon State Structural Specialty Code (OSSC).

We completed a site-specific seismic hazard analysis in general accordance with the requirements of the 2022 IBC as required for risk category 3 or 4 structures which is presented in Appendix B.

**6.6.4 Site Class North Portion of the Project**

Per American Society of Civil Engineers (ASCE) 7-16 a site-specific ground motion hazard analysis is required for structures located on site classes D and E sites with  $S_1$  greater than or equal to 0.2 g and a site response analysis is required for structures located on site Class F.

Subsurface geology at the site is highly variable as previously noted. Upper fine-grained soils increase in thickness toward the north of the site toward the pond and across the creek. For structures located north of the proposed SBRs and blower building the site is classified as a Site Class D. Therefore, the provisions of ASCE7-16 are applicable to those structures, only. The parameters listed in Table 7 may be used to determine the design ground motions for those structures provided the exceptions of ASCE7-16 supplement 3, are met. The applicable exceptions are provided below.

*From ASCE 7-16 Supplement 3*  
 Exception: A ground motion hazard analysis not required:

1. Where the values of the parameter  $S_{M1}$  determined by Eq. (11.4-2) is increased by 50% for all applications of  $S_{M1}$  in the standard. And:
2. The resulting value of the parameter  $S_{D1}$  determined by Eq. (11.4-4) shall be used for all applications of  $S_{D1}$  in the standard.

If it is desirable to avoid these exceptions, a ground motion hazard analysis would need to be completed for this portion of the site. However, based on the anticipated fundamental periods of the subject structures, performing a ground motion hazard analysis is unlikely to significantly reduce design accelerations.

**Table 7. Mapped 2022 IBC Seismic design parameters – Site Class D**

Parameter	Values <sup>1,2</sup>
Site Class	D
Mapped Spectral Response Acceleration at Short Period ( $S_s$ )	0.755 g
Mapped Spectral Response Acceleration at 1 Second Period ( $S_1$ )	0.381 g
Site Modified Peak Ground Acceleration ( $PGA_M$ )	0.436 g
Site Amplification Factor at 0.2 second period ( $F_a$ )	1.198

Site Amplification Factor at 1.0 second period ( $F_v$ )	1.919
Design Spectral Acceleration at 0.2 second period ( $S_{DS}$ )	0.603 g
Design Spectral Acceleration at 1.0 second period ( $S_{D1}$ ) <sup>(3)</sup>	0.731 g

Note:

<sup>1</sup> Parameters developed based on Latitude 44.854211° and longitude -122.876306 using the ATC Hazards online tool.

<sup>2</sup> These values are only valid if the structural engineer utilizes Exception 1 of ASCE 7-16 Supplement 3 Exception 1.

<sup>3</sup> Increased by a factor of 1.5 per ASCE 7-16 Supplement 3 Exception 1.

### 6.6.5 Site Class South Portion of the Project

For this project site, at the location of the proposed tanks, blower building, influent pump station, and the new storage building (flat area near existing structures and south of Beaver Creek) the site is classified as Site Class C based on geophysical surface wave testing completed at these locations as shown in Figure 2. Geophysical testing was included in this project as part of the original scope of services in order to gather data for a seismic site response analysis as discussed with the project team. The geophysical report is presented in Appendix C.

In addition to the site-specific seismic hazard analysis, we completed a ground motion hazard analysis in accordance with the procedures of Chapter 21 of ASCE7-16 to estimate site-specific ground motion hazards and response spectra for design of the structures noted above. Standard code-based parameters are provided in Table 8 for reference, only. The response spectra generated using the parameters in Table 6 and that obtained from the ground motion hazard analysis are presented in Appendix B. We recommend that applicable structures be designed using the response spectra from the **ground motion hazard analysis** site-specific procedures as shown in Figures B-7 and B-8 as applicable to the period of the proposed structures.

**Table 8. Mapped 2022 IBC Seismic design parameters – Site Class C**

Parameter	Site Class C Parameters
Site Class	C
Mapped Spectral Response Acceleration at Short Period ( $S_S$ )	0.755 g
Mapped Spectral Response Acceleration at 1 Second Period ( $S_1$ )	0.381 g
Site Modified Peak Ground Acceleration ( $PGA_M$ )	0.418 g
Site Amplification Factor at 0.2 second period ( $F_a$ )	1.2

Parameter	Site Class C Parameters
Site Amplification Factor at 1.0 second period ( $F_v$ )	1.5
Design Spectral Acceleration at 0.2 second period ( $S_{DS}$ )	0.604 g
Design Spectral Acceleration at 1.0 second period ( $S_{D1}$ ) <sup>(3)</sup>	0.381 g

Note:

<sup>1</sup> Parameters developed based on Latitude 44.854211° and longitude -122.876306 using the ATC Hazards online tool.

<sup>2</sup> Parameters are not recommended for design and are provided for reference, only. Response spectra

### 6.6.6 Liquefaction and Lateral Spreading

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. 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 contents is the most susceptible to liquefaction. Low plasticity, silty sand may be moderately susceptible to liquefaction under relatively higher levels of ground shaking.

As discussed in this report's groundwater section, relatively shallow groundwater is expected at the site. However, the predominately fine-grained site soils encountered below the groundwater are minimally susceptible to liquefaction during a design-level earthquake. Accordingly, lateral spreading or liquefaction induced deformations are not expected.

## 7.0 LIMITATIONS OF REPORT

We have prepared this report for the exclusive use of Westech Engineering and the City of Aumsville and their authorized parties for the project specifically identified in this report only. The report should be provided in its entirety to prospective contractors for bidding and estimating purposes; however, the conclusions and interpretations presented should not be construed as a warranty of the subsurface conditions. Experience has shown that soil and groundwater conditions can vary significantly over small distances. Inconsistent conditions can occur between explorations that may not be detected by a geotechnical study. If, during future site operations, subsurface conditions are encountered which vary appreciably from those described herein, Central should be notified for review of the recommendations of this report, and revision of such if necessary.

This report is not intended for use by others, and the information contained herein is not applicable to other sites. No other party may rely on the product of our services unless we agree in advance and in writing to such reliance.

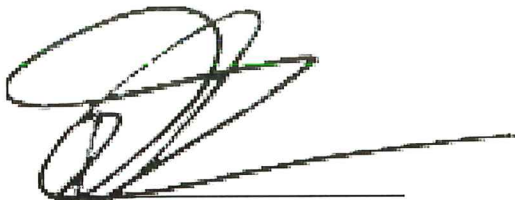
We recommend that Central be retained to review the plans and specifications and verify that our recommendations have been interpreted and implemented as intended. Sufficient geotechnical monitoring, testing and consultation should be provided during construction to confirm that the conditions encountered are consistent with those indicated by explorations. Recommendations for design changes will be provided should conditions revealed during construction differ from those anticipated. Should Central not be retained for Design or Construction-related services further into the development process, this report and its recommendations should be considered void, as we cannot take on responsibility for construction operations that were unobserved by our office.

Within the limitations of scope, schedule and budget, the analysis, conclusions, and recommendations presented in this report were prepared in accordance with generally accepted professional principles and practices in the fields of geotechnical engineering and engineering geology in this area at the time the report was prepared.

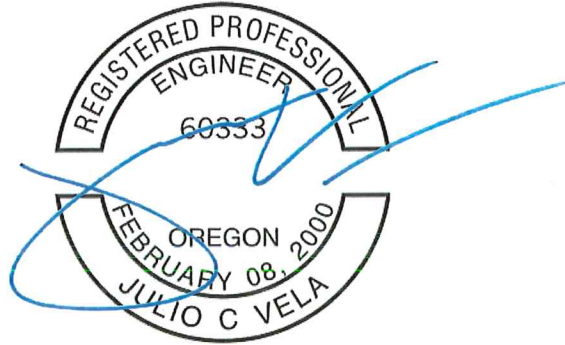
**8.0 SIGNATURES**

Thank you very much for the opportunity to collaborate with you. If you feel obliged, we welcome referrals from our previous clients and would enjoy the opportunity to work with others in your professional and personal networks.

Central Geotechnical Services, LLC



Blayne Sandau, PE, GIT  
Project Engineer



EXPIRES: 6/30/2026  
Julio Vela, PHD, PE, GE  
Principal Engineer

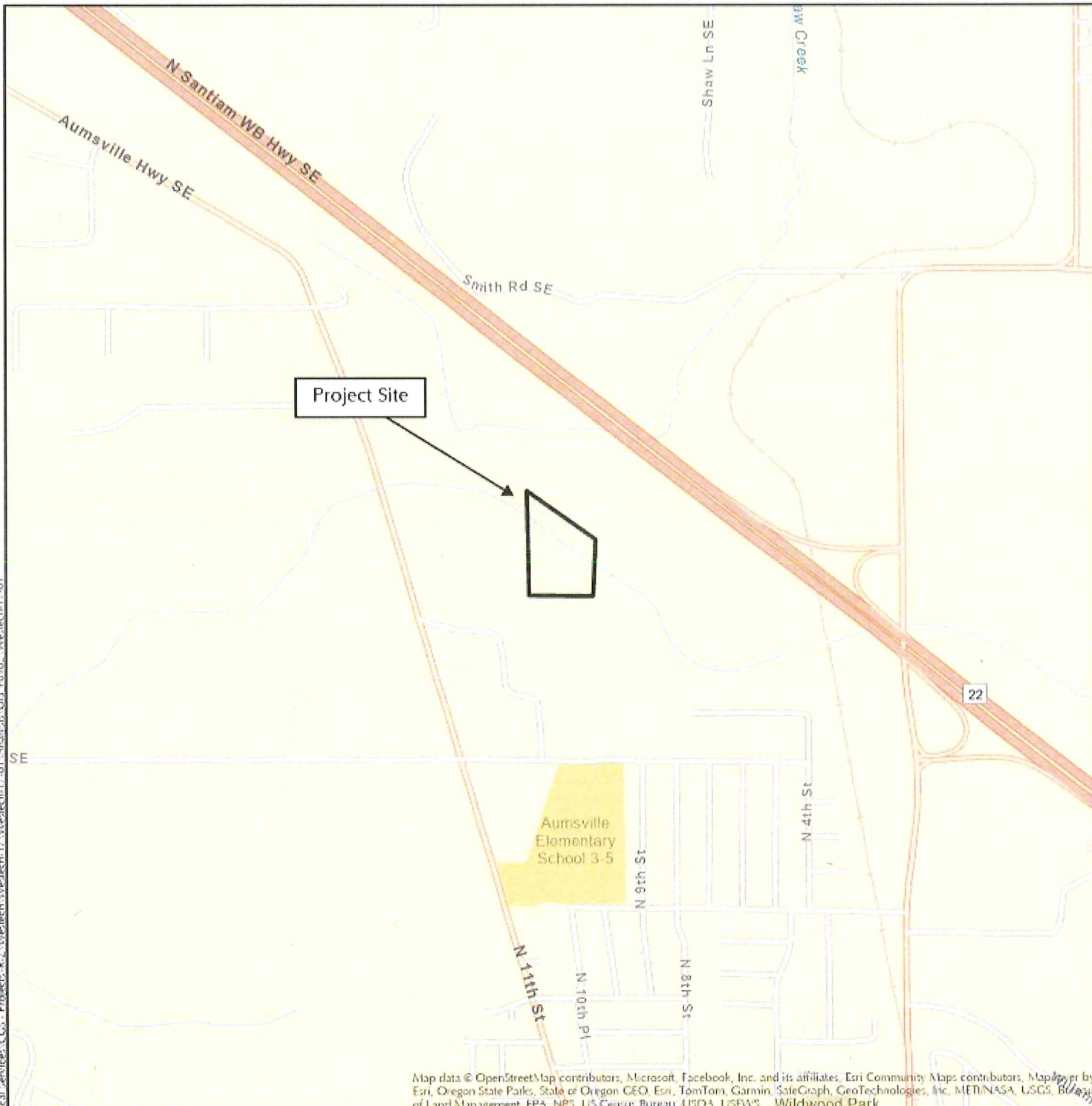


Adrian Cadena, GIT  
Staff Geologist

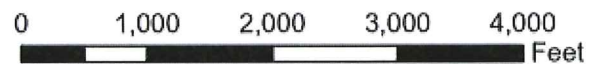


Figures

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Map data © OpenStreetMap contributors, Microsoft, Facebook, Inc. and its affiliates, Esri Community Maps contributors, Mapbox by Esri, Oregon State Parks, State of Oregon GEO, Esri, TomTom, Garmin, SafeGraph, GeoTechnologies, Inc, METINASA, USGS, Bureau of Land Management, EPA, NPS, US Census Bureau, USDA, USFWS, Wildwood Park



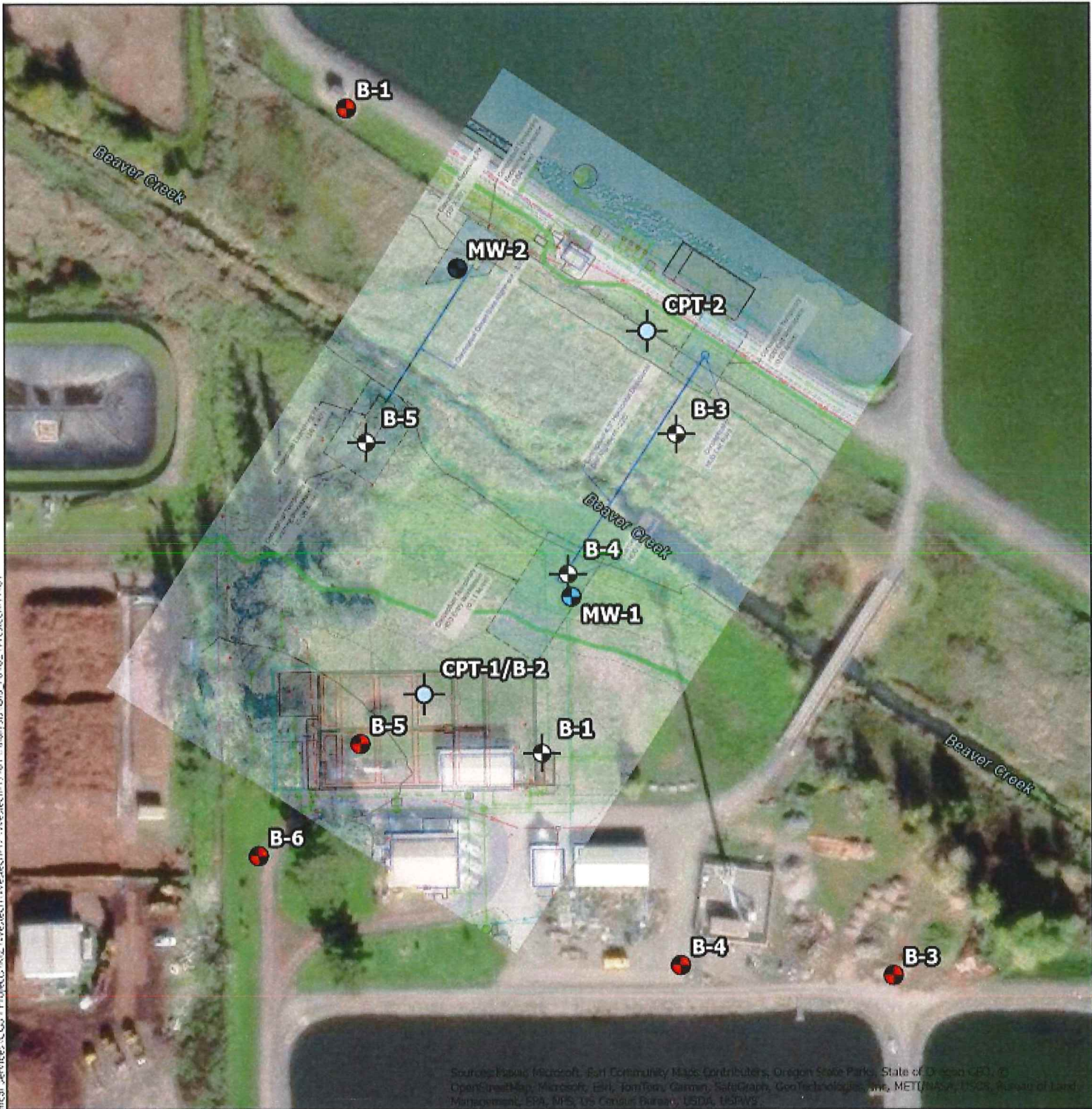
Westech-17-01

Project Vicinity



Figure-1

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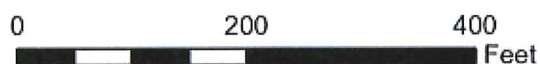
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Sources: Esri, Microsoft, Earth Community Made Contributors, Oregon State Parks, State of Oregon GIS, © OpenStreetMap contributors, CNR, TomTom, Garmin, SafeGraph, GeoTechnology, Inc, METUNAS, USGS, Bureau of Land Management, EPA, NPS, US Census Bureau, USDA, USFWS

## Legend

### Exploration Designation and Approximate Locations

-  Boring
-  CPT
-  Previous boring completed by others
-  Monitoring Well



Westtech-17-01	
Site Plan	
	Figure-2

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**APPENDIX A: Field Explorations and Laboratory Testing**

## **APPENDIX A**

### **FIELD EXPLORATIONS AND LABORATORY TESTING**

#### **Field Explorations**

Subsurface soil and groundwater conditions at the site were explored by drilling five geotechnical borings (B-1 through B-5) and by completing two CPT's on July 30 through August 1, 2024 at the approximate locations shown on the Site Plan, Figure 2. Exploratory borings were completed to final depths of 60 to 86.5 feet bgs in the vicinity of the proposed structures. CPT's were advanced to final depths of between 35 to 50 feet bgs. The exploratory borings were advanced using mud rotary drilling techniques. The borings were completed with a drill rig owned and operated by Western States Soil Conservation, and the cone penetration tests were completed using a CPT rig owned and operated by Oregon Geotechnical Explorations Inc.

The drilling and cone penetration tests were continuously monitored by qualified staff from our office who maintained detailed logs of subsurface explorations, visually classified the soil encountered and obtained representative soil samples from the borings. Representative soil samples were obtained from each boring at approximate 2½- and 5 foot depth- intervals using a 1-inch-inside diameter, standard split spoon sampler. The samplers were driven into the soil using a 140-pound hammer, freefalling 30 inches on each blow. The number of blows required- to drive the sampler each of three, 6-inch increments of penetration were recorded in the field. The sum of the blow counts for the last two, 6-inch increments of penetration is reported on the boring logs as the ASTM International (ASTM) Test Method D 1556 Standard Penetration Test (SPT) N-value. Relatively undisturbed samples were collected at select depth intervals using relatively thin-walled Shelby tube samplers.

Recovered soil samples from exploratory borings and test pits were visually classified in the field in general accordance with ASTM D 2488 and the classification chart listed in Key to Exploration Logs. Logs of the borings are presented in this Appendix. The logs are based on interpretation of the field and laboratory data and indicate the depth at which subsurface materials, or their characteristics change, although these changes might actually be gradual.

#### **Laboratory Testing**

Soil samples obtained from the explorations were visually classified in the field and in our laboratory using the USCS and ASTM classification methods. ASTM Test Method D 2488 was used to visually classify the soil samples, while ASTM D 2487 was used to classify the soils based on laboratory test results. Moisture content tests were performed in general accordance with ASTM D 221605. Results of the moisture contents testing are presented in the appropriate exploration logs at the respective sample depths.

Atterberg limits testing was performed on selected fine-grained soil samples. The tests were used to classify the soil as well as to evaluate index properties. The liquid limit and the plastic limit were estimated through a procedure performed in general accordance with ASTM D 4318. The results of the Atterberg limits testing are summarized in Figure A-10

One-dimension Consolidation tests were completed on relatively undisturbed soil samples extruded from Shelby tubes obtained from Boring B-14. The tests were used to aid in evaluating consolidation characteristics of the soft fine-grained soil deposits present in the site's northwestern portion. The consolidation tests were performed in general accordance with ASTM D 2435, using a fixed ring consolidometer. The results of the consolidation tests are summarized in Figure A-12 and A-13.

Sieve analyses were performed on select samples in general accordance with ASTM C136. The wet sieve analysis method was used to determine the percentage of soil greater than the U.S. No. 200 mesh sieve. The results of the sieve analyses were plotted and were classified in general accordance with the Unified Soil Classification System and are presented in Figure A-11

Selected samples were “washed” through the U.S. No. 200 mesh sieve to estimate the relative percentages of coarse- and fine-grained particles in the soil. The percent passing value is the percentage by weight of the sample finer than the U.S. No. 200 sieve. These tests were conducted to verify field descriptions and to estimate the fines content for analysis purposes. The tests were conducted in accordance with ASTM D 1140, and the results are shown on the exploration logs in Appendix A at the respective sample depths.



Relative Density - Coarse-Grained Soil						GEOTECHNICAL TESTING EXPLANATIONS	
Term	SPT (140-lb Hammer)*	D&M Sampler (140-lb Hammer)*		D&M Sampler (300-lb Hammer)*	ATT	Atterberg Limits	
Very-loose	0-4	0-11		0-4	CBR	California Bearing Ratio	
Loose	4-10	11-26		4-10	CON	Consolidation	
Medium-dense	10-30	26-74		10-30	DD	Dry Density	
Dense	30-50	74-120		30-47	DS	Direct Shear	
Very-dense	>50	>120		>47	HYD	Hydrometer Gradation	
Consistency - Fine-Grained Soil						LL	Liquid Limit
Term	SPT (140-lb Hammer)*	Sampler (140-lb Hammer)*	Sampler (300-lb Hammer)*	Pocket Pen (tsf)	Torvane (tsf)	PL	Plastic Limit
						PI	Plasticity Index
Very-soft	0-2	0-3	0-2	<0.25	<0.13	MC	Moisture Content
Soft	2-4	3-6	2-5	0.25-0.5	0.13-0.25	MD	Moisture-Density
Medium-stiff	4-8	6-12	5-9	0.5-1	0.25-0.5	NP	Non-Plastic
Stiff	8-15	12-25	9-19	1.0-2.0	0.5-1.0	OC	Organic Content
Very-stiff	15-30	25-65	19-31	2.0-4.0	1.0-2.0	P	Pushed Sample
Hard	>30	>65	>31	>4.0	>2.0	PP	Pocket Penetrometer
SPT N-value correlation based off ASTM D1586						Passing No.200	Percent Passing U.S. Std. No.200 Sieve
Unified Soil Classification System (USCS)						RES	Resilient Modulus
USCS Symbols	Graph	Typical Descriptions				SIEV	Sieve Gradation
GP		Poorly graded GRAVEL, <5% fines				TOR	Torvane
GP-GM/GP-GC		Poorly graded GRAVEL w/ silt/clay, 5 to 12% fines				UC	Unconfined Compressive Strength
GM		silty GRAVEL, over 12% fines				VS	Vane Shear
GC		clayey GRAVEL, over 12% fines				<b>CONTACT LINES</b>	
GW		well graded GRAVEL, <5% fines				Distinct contact between soil strata (approximate location)	
SP		poorly graded SAND, <5% fines				Approximate contact between soil strata	
SP-SM/SP-SC		poorly graded SAND w/ silt/clay, 5 to 12% fines				<b>WATER LEVELS</b>	
SM		silty SAND, over 12% fines				Water Level at Time of Drilling, or as labeled	
SC		clayey SAND, over 12% fines				Water Level at End of Drilling, or as labeled	
SW		well graded SAND, <5% fines				Static Water Level, or as labeled	
ML		SILT, low plasticity				<b>Moisture (ASTM D2488)</b>	
MH		SILT, high plasticity				Dry	Very low moisture, dry to touch
CL		CLAY, low plasticity				Moist	Damp, without visible moisture
CH		CLAY, high plasticity				Wet	Visible free water, usually saturated
OL		ORGANIC SILT, low plasticity				<b>ADDITIONAL MATERIALS</b>	
OH		ORGANIC CLAY, medium to high plasticity				AC	ASPHALT CONCRETE
PT		PEAT				CC	CEMENT CONCRETE
<b>ADDITIONAL CONSTITUENTS</b>						CR	CRUSHED ROCK
Silt/Clay in:			Sand/Gravel in:			SOD	SOD/FOREST DUFF
Percent*	Fine-Grained	Coarse-Grained	Percent*	Fine-Grained	Coarse-Grained	FILL	FILL
<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	with		
SYMBOL	SAMPLER DESCRIPTIONS				SYMBOL	SAMPLER DESCRIPTIONS	
	Location of grab sample (GS)					Location of sample collected using Standard Penetration Test with recovery (SS)	
	No Recovery					Location of sample collected using Shelby tube/Geoprobe sample with recovery (ST)	
	Location of rock coring interval (RC)					Location of sample collected using Dames & Moore sampler or pushed with recovery (D&M)	

BORING TEMPLATE - GINT STD US LAB.GDT - 1/13/25 13:00 - C:\USERS\CGS\USER\GIS\WORKSTATION\CENTRAL GEOTECHNICAL SERVICES\CGS - PROJECTS\R-Z\WESTECH\WESTECH-17\WESTECH-17-01\FIELD EXPLORATION 2 FIELD AND DRAFT LOG



Central Geotechnical Services  
 10240 SW Nimbus Ave, Suite L6  
 Portland, OR 97223  
 Telephone: (503) 616-9419

**Project No:**  
**Westech-17-01**

# BORING LOG B-1

PAGE 1 OF 2

**Project:** Aumsville Waste Water Treatment Plant  
**Location:** 955 Olney St SE, Aumsville, OR 97325  
**Client:** City of Aumsville

**Date Started:**  
 7/30/24  
**Date Completed:**  
 7/30/24

**Approximate Ground Elevation:**  
**Groundwater first encountered:** ---  
**Groundwater at end of drilling:** ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	Elevation:	SAMPLE TYPE NUMBER	RECOVERY (in.)	MOISTURE (%)	BLOW COUNTS	LAB RESULTS/REMARKS		
0										
1		Stiff, brown, orange, black and dark gray, gravelly SILT with sand (ML), moist, sand is medium to coarse, gravel is angular and subangular (FILL)								
2										
3										
4						SS S-1	5	25	6-5-4	
5										
6						SS S-2	3		6-5-5	
7										
8						SS S-3	5		2-3-7	
9										
10			9.50							
11		Very-stiff, dark gray, light brown and orange, gravelly elastic SILT (MH), minor sand, moist to wet, gravel is angular to subangular, sand is medium to coarse								
12										
13										
14										
15										
16				Becomes hard at 15 feet bgs		SS S-5	5		15-22-42	
17										
18										
19										
20										
21						SS S-6	10		12-12-15	
22										
23										
24										
25										
26			25.50							
27		Very-stiff, gray, brown, orange, red, white, black, yellow, elastic SILT (MH), minor to trace sand and gravel, moist to wet, sand is fine to medium, relict rock structures								
28										
29										
30										
31						SS S-7	12	49	14-8-8	Passing No. 200 = 87.6% LL = 69 PL = 40 PI = 28
32										
33										
34										
35						SS S-8	18		6-7-11	

(Continued Next Page)

**Operator:** Western States Soil Conservation Inc.  
**Drilling Method:** 4 7/8" Mud Rotary  
**Equipment:** CME 850XR Track Rig

**Logged By:** Adrian C.  
**Checked By:** Blayne S.  
**Approximate Location Coordinates:**  
 Lat: Long:

**Remarks:**

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**Project No:**  
**Westech-17-01**

# BORING LOG B-1

PAGE 2 OF 2

**Project:** Aumsville Waste Water Treatment Plant  
**Location:** 955 Olney St SE, Aumsville, OR 97325  
**Client:** City of Aumsville

**Date Started:**  
 7/30/24  
**Date Completed:**  
 7/30/24

**Approximate Ground Elevation:**  
**Groundwater first encountered:** ---  
**Groundwater at end of drilling:** ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	Elevation:	SAMPLE TYPE NUMBER	RECOVERY (in.)	MOISTURE (%)	BLOW COUNTS	LAB RESULTS/REMARKS		
35										
36		Very-stiff, gray, brown, orange, red, white, black, yellow, elastic SILT (MH), minor to trace sand and gravel, moist to wet, sand is fine to medium, relict rock structures <i>(continued)</i>  Becomes hard at 40 feet bgs Grades to minor to trace sand at 40 feet		SS S-9	18	34	4-7-18	Passing No. 200 = 94.0% LL = 74 PL = 40 PI = 34		
37										
38										
39										
40										
41						SS S-10	18	38	6-9-27	Passing No. 200 = 96.6% LL = 56 PL = 30 PI = 26
42										
43										
44										
45										
46				SS S-11	18	45	7-12-22			
47										
48										
49										
50			50.00							
51		Hard, bluish gray, SILTSTONE (R2), visible laminations, sample broken up into shards		SS S-12	1		50/1"			
52										
53										
54										
55										
56						SS S-13	3		50/3"	
57										
58										
59										
60					60.00					

Boring completed at 60 feet bgs.  
 Groundwater not observed due to the drilling method  
 Hammer efficiency factor is 80.4%

<b>Operator:</b> Western States Soil Conservation Inc. <b>Drilling Method:</b> 4 7/8" Mud Rotary <b>Equipment:</b> CME 850XR Track Rig	<b>Logged By:</b> Adrian C. <b>Checked By:</b> Blayne S.	<b>Remarks:</b>
	<b>Approximate Location Coordinates:</b> Lat:                      Long:	

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 Portland, OR 97223  
 Telephone: (503) 616-9419

**Project No:**  
**Westech-17-01**

**BORING LOG B-2**

PAGE 1 OF 1

**Project:** Aumsville Waste Water Treatment Plant  
**Location:** 955 Olney St SE, Aumsville, OR 97325  
**Client:** City of Aumsville

**Date Started:**  
 7/29/24  
**Date Completed:**  
 7/29/24

**Approximate Ground Elevation:**  
**Groundwater first encountered:** ---  
**Groundwater at end of drilling:** ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	Elevation:	SAMPLE TYPE NUMBER	RECOVERY (in.)	MOISTURE (%)	BLOW COUNTS	LAB RESULTS/REMARKS	
0									
1		Medium-stiff, light brown, brown, orange, gray, black, sandy SILT (ML), trace gravel, moist, sand is fine to coarse (FILL)							
2									
3									
4			SS S-1	4			3-3-2		
5									
6			SS S-2	3	42		2-2-8		
7									
8			8.00						
9		Stiff, dark-gray and light brown, gravelly, elastic SILT with sand (MH), trace silt, moist, gravel is subrounded to subangular, sand is medium to coarse							
10									
11			SS S-3	10			5-5-6		
12									
13									
14									
15									
16			SS S-4	5			13-8-10		
17									
18									
19									
20									
21			SS S-5	12	15		16-27-33		
22									
23									
24									
25									
26	SS S-6	5			3-6-30				
27									
28									
29									
30									
31	SS S-7	8			1-8-11				
32									
33									
34									
35	35.00								

Boring completed at 35.0 feet bgs.  
 Groundwater not observed due to the drilling method.  
 Hammer efficiency factor is 80.4%

**Operator:** Western States Soil Conservation Inc.  
**Drilling Method:** 3 7/8" Mud Rotary  
**Equipment:** CME 850XR Track Rig

**Logged By:** Adrian C.  
**Checked By:** Blayne S.  
**Approximate Location Coordinates:**  
 Lat: Long:

**Remarks:**

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**Project No:**  
**Westech-17-01**

# BORING LOG B-3

PAGE 1 OF 3

**Project:** Aumsville Waste Water Treatment Plant  
**Location:** 955 Olney St SE, Aumsville, OR 97325  
**Client:** City of Aumsville

**Date Started:**  
 7/29/24  
**Date Completed:**  
 7/30/24

**Approximate Ground Elevation:**  
**Groundwater first encountered:** ---  
**Groundwater at end of drilling:** ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	Elevation:	SAMPLE TYPE	NUMBER	RECOVERY (in.)	MOISTURE (%)	BLOW COUNTS	LAB RESULTS/REMARKS	
0										
1		Medium-stiff, light brown, brown, orange, gray, black, sandy SILT (ML), trace gravel and organics, moist, sand is fine to coarse (FILL)								
2										
3					SS S-1	10			2-4-5	
4										
5										
6		Soft to very-stiff, gray, brown, orange, red, white, black, yellow, lean CLAY (CL) with sand and gravel, moist to wet  Becomes medium-stiff at 7.5 feet bgs	5.50						Passing No. 200 = 76.2% LL = 46 PL = 25 PI = 20	
7					SS S-2	18	35	4-1-2		
8										
9					SS S-3	18				1-1-3
10					10.00					
11		Medium-stiff, brown-orange with yellow, gray, black, elastic SILT with sand (MH), moist to wet, sand is fine to medium, relict rock structures  Becomes soft at 20 feet bgs								
12										
13										
14										
15										
16										
17										
18										
19										
20										
21										
22										
23										
24										
25										
26										
27										
28										
29										
30										
31										
32										
33										
34										
35										

(Continued Next Page)

<b>Operator:</b> Western States Soil Conservation Inc.	<b>Logged By:</b> Adrian C.	<b>Remarks:</b>
<b>Drilling Method:</b> 4 7/8" Mud Rotary	<b>Checked By:</b> Blayne S.	
<b>Equipment:</b> CME 850XR Track Rig	<b>Approximate Location Coordinates:</b>	
	Lat: Long:	

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**Project No:**  
**Westech-17-01**

# BORING LOG B-3

PAGE 2 OF 3

<b>Project:</b> Aumsville Waste Water Treatment Plant	<b>Date Started:</b> 7/29/24	<b>Approximate Ground Elevation:</b>
<b>Location:</b> 955 Olney St SE, Aumsville, OR 97325	<b>Date Completed:</b> 7/30/24	<b>Groundwater first encountered:</b> ---
<b>Client:</b> City of Aumsville		<b>Groundwater at end of drilling:</b> ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	Elevation:	SAMPLE TYPE NUMBER	RECOVERY (in.)	MOISTURE (%)	BLOW COUNTS	LAB RESULTS/REMARKS	
35									
36		Medium-stiff, brown-orange with yellow, gray, black, elastic SILT with sand (MH), moist to wet, sand is fine to medium, relict rock structures <i>(continued)</i>		SS S-10	18	43	1-2-0		
37									
38									
39									
40									
41					ST S-11	24			
42									
43		Becomes very-stiff at 42 feet bgs			SS S-12	18		11-10-16	
44									
45									
46									
47									
48									
49									
50									
51	Becomes stiff at 50 feet bgs			SS S-13	18		4-4-4		
52									
53									
54									
55									
56	Becomes very-stiff at 55 feet bgs			SS S-14	18		8-11-10		
57									
58									
59									
60									
61				SS S-15	18		7-10-12		
62									
63									
64									
65									
66				SS S-16	18	48	5-6-10	Passing No. 200 = 93.6% LL = 100 PL = 45 PI = 55	
67									
68									
69									
70									

(Continued Next Page)

<b>Operator:</b> Western States Soil Conservation Inc.	<b>Logged By:</b> Adrian C.	<b>Remarks:</b>
<b>Drilling Method:</b> 4 7/8" Mud Rotary	<b>Checked By:</b> Blayne S.	
<b>Equipment:</b> CME 850XR Track Rig	<b>Approximate Location Coordinates:</b>	
	Lat: Long:	

BORING TEMPLATE - GINT STD US LAB.GDT - 1/13/25 13:00 - C:\USERS\CGS\USER\GIS-WORKSTATION\CENTRAL GEOTECHNICAL SERVICES\CGS - PROJECTS\R-Z\WESTECH\WESTECH-17\WESTECH-17-01\1 FIELD EXPLORATION\2 FIELD AND DRAFT LOG



Central Geotechnical Services  
 10240 SW Nimbus Ave, Suite L6  
 Portland, OR 97223  
 Telephone: (503) 616-9419

**Project No:**  
**Westech-17-01**

# BORING LOG B-3

PAGE 3 OF 3

**Project:** Aumsville Waste Water Treatment Plant  
**Location:** 955 Olney St SE, Aumsville, OR 97325  
**Client:** City of Aumsville

**Date Started:**  
 7/29/24  
**Date Completed:**  
 7/30/24

**Approximate Ground Elevation:**  
**Groundwater first encountered:** ---  
**Groundwater at end of drilling:** ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	Elevation:	SAMPLE TYPE NUMBER	RECOVERY (in.)	MOISTURE (%)	BLOW COUNTS	LAB RESULTS/REMARKS
70								
71		Medium-stiff, brown-orange with yellow, gray, black, elastic SILT with sand (MH), moist to wet, sand is fine to medium, relict rock structures ( <i>continued</i> )		SS S-17	18		5-6-10	
72								
73								
74								
75								
76		Becomes stiff at 75 feet bgs 76.50			SS S-18	18		

Boring completed at 76.5 feet bgs.  
 Groundwater not observed due to the drilling method.  
 Hammer efficiency factor is 80.4%

<b>Operator:</b> Western States Soil Conservation Inc.	<b>Logged By:</b> Adrian C.	<b>Remarks:</b>
<b>Drilling Method:</b> 4 7/8" Mud Rotary	<b>Checked By:</b> Blayne S.	
<b>Equipment:</b> CME 850XR Track Rig	<b>Approximate Location Coordinates:</b>	
	Lat: Long:	

BORING TEMPLATE - GINT STD U.S. LAB. GDT - 1/13/25 13:00 - C:\USERS\CGS\USER\GIS\WORKSTATION\1\CENTRAL GEOTECHNICAL SERVICES\CGS - PROJECTS\R-Z\WESTECH\WESTECH-17\WESTECH-17-01\FIELD EXPLORATION\2\FIELD AND DRAFT LOG



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 Portland, OR 97223  
 Telephone: (503) 616-9419

**Project No:**  
**Westech-17-01**

**BORING LOG B-4**

<b>Project:</b> Aumsville Waste Water Treatment Plant	<b>Date Started:</b> 7/31/24	<b>Approximate Ground Elevation:</b>
<b>Location:</b> 955 Olney St SE, Aumsville, OR 97325	<b>Date Completed:</b> 7/31/24	<b>Groundwater first encountered:</b> ---
<b>Client:</b> City of Aumsville		<b>Groundwater at end of drilling:</b> ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	Elevation:	SAMPLE TYPE NUMBER	RECOVERY (in.)	MOISTURE (%)	BLOW COUNTS	LAB RESULTS/REMARKS		
0										
1		Medium-stiff, light brown, brown, orange, gray, black, sandy SILT (ML), trace gravel, moist, sand is fine to coarse (FILL)								
2										
3										
4						SS S-1	4	29	2-2-3	
5			5.00							
6		Medium-dense to very-dense, dark-gray and light brown, GRAVEL (GP) with sand, trace silt, moist, gravel is subrounded to subangular, sand is medium to coarse, gravel and cobble up to 3-inches in diameter (FILL)								
7										
8						SS S-2	10		6-9-15	
9					8.50					
10		Very-stiff, brown, light brown, dark brown, orange, yellow, elastic SILT with sand (MH), trace gravel, moist, gravel is subrounded, sand is fine to coarse, relict rock structures								
11										
12						SS S-3	8		7-16-8	
13										
14										
15										
16		Medium-dense, light and dark brown, SAND (SP), moist to wet, sand is medium								
17										
18										
19										
20										
21										
22			21.75							
23		Stiff, brown, light brown, dark brown, orange, yellow, gravelly SILT (ML), moist to wet, gravel is subrounded, relict rock structures								
24										
25						SS S-4	15		6-8-7	
26					25.00					
27		Very-stiff, reddish-orange and white with pockets of white, light green, black, elastic SILT (MH) minor sand, moist to wet, sand is fine to medium, relict rock structures								
28										
29										
30										
31										
32										
33										
34										
35										

(Continued Next Page)

<b>Operator:</b> Western States Soil Conservation Inc.	<b>Logged By:</b> Adrian C.	<b>Remarks:</b>
<b>Drilling Method:</b> 4 7/8" Mud Rotary	<b>Checked By:</b> Blayne S.	
<b>Equipment:</b> CME 850XR Track Rig	<b>Approximate Location Coordinates:</b>	
	Lat: Long:	

Passing No. 200  
 = 88.9%  
 LL = 61  
 PL = 36  
 PI = 24

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 Telephone: (503) 616-9419

**Project No:**  
**Westech-17-01**

# BORING LOG B-4

PAGE 2 OF 3

**Project:** Aumsville Waste Water Treatment Plant  
**Location:** 955 Olney St SE, Aumsville, OR 97325  
**Client:** City of Aumsville

**Date Started:**  
 7/31/24  
**Date Completed:**  
 7/31/24

**Approximate Ground Elevation:**  
**Groundwater first encountered:** ---  
**Groundwater at end of drilling:** ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	Elevation:	SAMPLE TYPE NUMBER	RECOVERY (in.)	MOISTURE (%)	BLOW COUNTS	LAB RESULTS/REMARKS
35								
36		Very-stiff, reddish-orange and white with pockets of white, light green, black, elastic SILT (MH) minor sand, moist to wet, sand is fine to medium, relict rock structures <i>(continued)</i> Becomes stiff at 35 feet bgs		SS S-9	18	33	2-3-6	Passing No. 200 = 90.8% LL = 90 PL = 42 PI = 49
37								
38								
39								
40								
41		Becomes hard at 35 feet bgs		SS S-10	18		8-9-23	
42								
43								
44								
45								
46		Grades to trace sand at 45 feet Becomes very-stiff at 45 feet bgs		SS S-11	18	53	8-14-9	
47								
48								
49								
50								
51				SS S-12	18		3-8-8	
52								
53								
54								
55								
56		Becomes stiff at 55 feet bgs		SS S-13	16		3-5-8	
57								
58								
59								
60		60.00						
61		Stiff, gray with white, SILT (ML), moist to wet, relict rock structures		SS S-14	18		1-5-5	
62								
63								
64								
65								
66				SS S-15	18		4-6-7	
67								
68								
69								
70								

(Continued Next Page)

<b>Operator:</b> Western States Soil Conservation Inc.	<b>Logged By:</b> Adrian C.	<b>Remarks:</b>
<b>Drilling Method:</b> 4 7/8" Mud Rotary	<b>Checked By:</b> Blayne S.	
<b>Equipment:</b> CME 850XR Track Rig	<b>Approximate Location Coordinates:</b>	
	Lat: Long:	

BORING TEMPLATE - GINT STD US LAB.GDT - 1/13/25 13:00 - C:\USERS\CGS\USER\GIS\WORKSTATION\1\CENTRAL GEOTECHNICAL SERVICES\CGS - PROJECTS\R-Z\WESTECH\WESTECH-17\WESTECH-17-01\FIELD EXPLORATION\2\_FIELD AND DRAFT LOG



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 Telephone: (503) 616-9419

**Project No:**  
**Westech-17-01**

**BORING LOG B-4**

**Project:** Aumsville Waste Water Treatment Plant  
**Location:** 955 Olney St SE, Aumsville, OR 97325  
**Client:** City of Aumsville

**Date Started:**  
 7/31/24  
**Date Completed:**  
 7/31/24

**Approximate Ground Elevation:**  
**Groundwater first encountered:** ---  
**Groundwater at end of drilling:** ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	Elevation:	SAMPLE TYPE NUMBER	RECOVERY (in.)	MOISTURE (%)	BLOW COUNTS	LAB RESULTS/REMARKS
70								
71		Stiff, gray with white, SILT (ML), moist to wet, relict rock structures ( <i>continued</i> )		SS S-16	0		9-12-14	
72								
73								
74			74.00					
75		Stiff, gray with black and yellow, CLAY (CL), trace organics, moist to wet		SS S-17	0		6-6-7	
76								
77								
78								
79								
80								
81			81.50	SS S-18	18		3-5-9	

Boring completed at 81.5 feet bgs.  
 Groundwater not observed due to the drilling method.  
 Hammer efficiency factor is 80.4%

**Operator:** Western States Soil Conservation Inc.  
**Drilling Method:** 4 7/8" Mud Rotary  
**Equipment:** CME 850XR Track Rig

**Logged By:** Adrian C.  
**Checked By:** Blayne S.  
**Approximate Location Coordinates:**  
 Lat: Long:

**Remarks:**

BORING TEMPLATE - GINT STD US LAB.GDT - 1/13/25 13:00 - C:\USERS\CGS\USER\GIS\WORKSTATION\CENTRAL\_GEOTECHNICAL\_SERVICES\CGS - PROJECTS\17-01\WESTECH\17-01\FIELD EXPLORATION\2\_FIELD AND DRAFT LOG



Central Geotechnical Services  
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 Portland, OR 97223  
 Telephone: (503) 616-9419

**Project No:**  
**Westech-17-01**

# BORING LOG B-5

PAGE 1 OF 3

**Project:** Aumsville Waste Water Treatment Plant  
**Location:** 955 Olney St SE, Aumsville, OR 97325  
**Client:** City of Aumsville

**Date Started:**  
 8/1/24  
**Date Completed:**  
 8/1/24

**Approximate Ground Elevation:**  
**Groundwater first encountered:** ---  
**Groundwater at end of drilling:** ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	Elevation:	SAMPLE TYPE NUMBER	RECOVERY (in.)	MOISTURE (%)	BLOW COUNTS	LAB RESULTS/REMARKS		
0										
1		Stiff, brown-gray with orange, elastic SILT (MH) or CLAY (CL), moist, trace sand and organics, relict rock structures								
2										
3										
4						SS S-1	10	24	5-6-7	
5										
6						SS S-2	14		3-3-17	
7										
7		7.00								
8		Hard, brown, orange, dark gray, white, black, gravelly SILT (MH), minor sand, moist, gravel is subrounded, sand is medium to coarse								
9										
10										
10		10.00								
11		Stiff, brown-gray with orange, elastic SILT (MH), trace sand and organics, moist to wet, relict rock structures								
12										
13										
14										
15										
16				SS S-5	10		3-4-5			
17										
18										
19										
19		19.00								
20		Medium-stiff, brown, green-gray, orange, black, dark gray, SILT with some sand and gravel (ML), moist, gravel is subrounded to rounded, sand is medium to coarse, some pockets more weathered than others								
21										
22										
23										
24										
25										
26						SS S-7	0		1-2-2	
27										
28										
29										
30										
31				ST S-8	4					
32										
33				SS S-9	4	50	7-9-12			
34										
35										

Becomes very-stiff at 32 feet bgs

(Continued Next Page)

<b>Operator:</b> Western States Soil Conservation Inc. <b>Drilling Method:</b> 4 7/8" Mud Rotary <b>Equipment:</b> CME 850XR Track Rig	<b>Logged By:</b> Adrian C. <b>Checked By:</b> Blayne S.	<b>Remarks:</b>
	<b>Approximate Location Coordinates:</b>	
	Lat:                      Long:	

BORING TEMPLATE - GINT STD US LAB.GDT - 1/13/25 13:00 - C:\USERS\CGS\USER\GIS-WORKSTATION\CENTRAL GEOTECHNICAL SERVICES\CGS - PROJECTS\R-Z\WESTECH\WESTECH-17\WESTECH-17-01\FIELD EXPLORATION\2\_FIELD AND DRAFT LOG



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 Telephone: (503) 616-9419

**Project No:**  
**Westech-17-01**

# BORING LOG B-5

PAGE 2 OF 3

**Project:** Aumsville Waste Water Treatment Plant  
**Location:** 955 Olney St SE, Aumsville, OR 97325  
**Client:** City of Aumsville

**Date Started:**  
 8/1/24  
**Date Completed:**  
 8/1/24

**Approximate Ground Elevation:**  
**Groundwater first encountered:** ---  
**Groundwater at end of drilling:** ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	Elevation:	SAMPLE TYPE NUMBER	RECOVERY (in.)	MOISTURE (%)	BLOW COUNTS	LAB RESULTS/REMARKS
35								
36				SS S-10	4		8-11-14	
37			37.00					
38		Very-stiff, red, orange-red, CLAY (CL), moist						
39								
40								
41				SS S-11	18	41	4-6-9	
42								
43								
44								
45								
46				SS S-12	14		4-6-10	
47								
48			48.00					
49		Stiff, light gray, light brown, white and orange, SILT (ML), moist, relict rock structures						
50								
51				SS S-13	18		4-4-5	
52								
53								
54								
55								
56				SS S-14	18		5-5-6	
57								
58								
59								
60								
61		Becomes very-stiff at 60 feet bgs		SS S-15	18		3-9-16	
62								
63			63.00					
64		Stiff, gray, light green and orange-brown, elastic SILT (MH), moist to wet						
65								
66				SS S-16	18	56	2-4-9	Passing No. 200 = 98.2% LL = 132 PL = 60 PI = 72
67								
68								
69								
70								

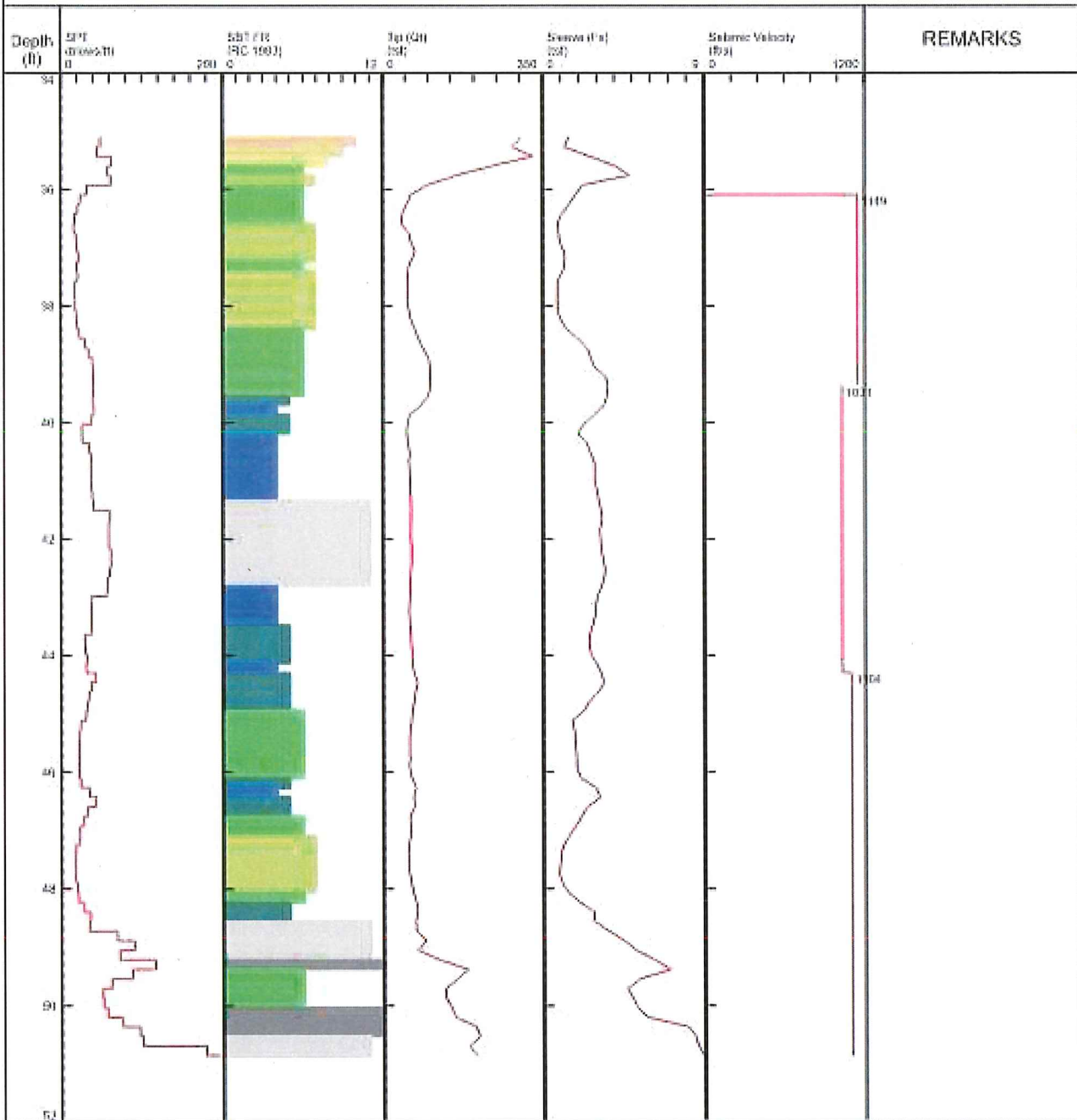
(Continued Next Page)

<b>Operator:</b> Western States Soil Conservation Inc.	<b>Logged By:</b> Adrian C.	<b>Remarks:</b>
<b>Drilling Method:</b> 4 7/8" Mud Rotary	<b>Checked By:</b> Blayne S.	
<b>Equipment:</b> CME 850XR Track Rig	<b>Approximate Location Coordinates:</b>	
	Lat: Long:	



# Central Geotech / CPT-1 / 945 Olney Street SE Aumsville

OPERATOR: OGE BAK  
 CONE ID: DOG1296  
 TEST DATE: 7/29/2024 1:10:07 PM  
 TOTAL DEPTH: 50.853 ft

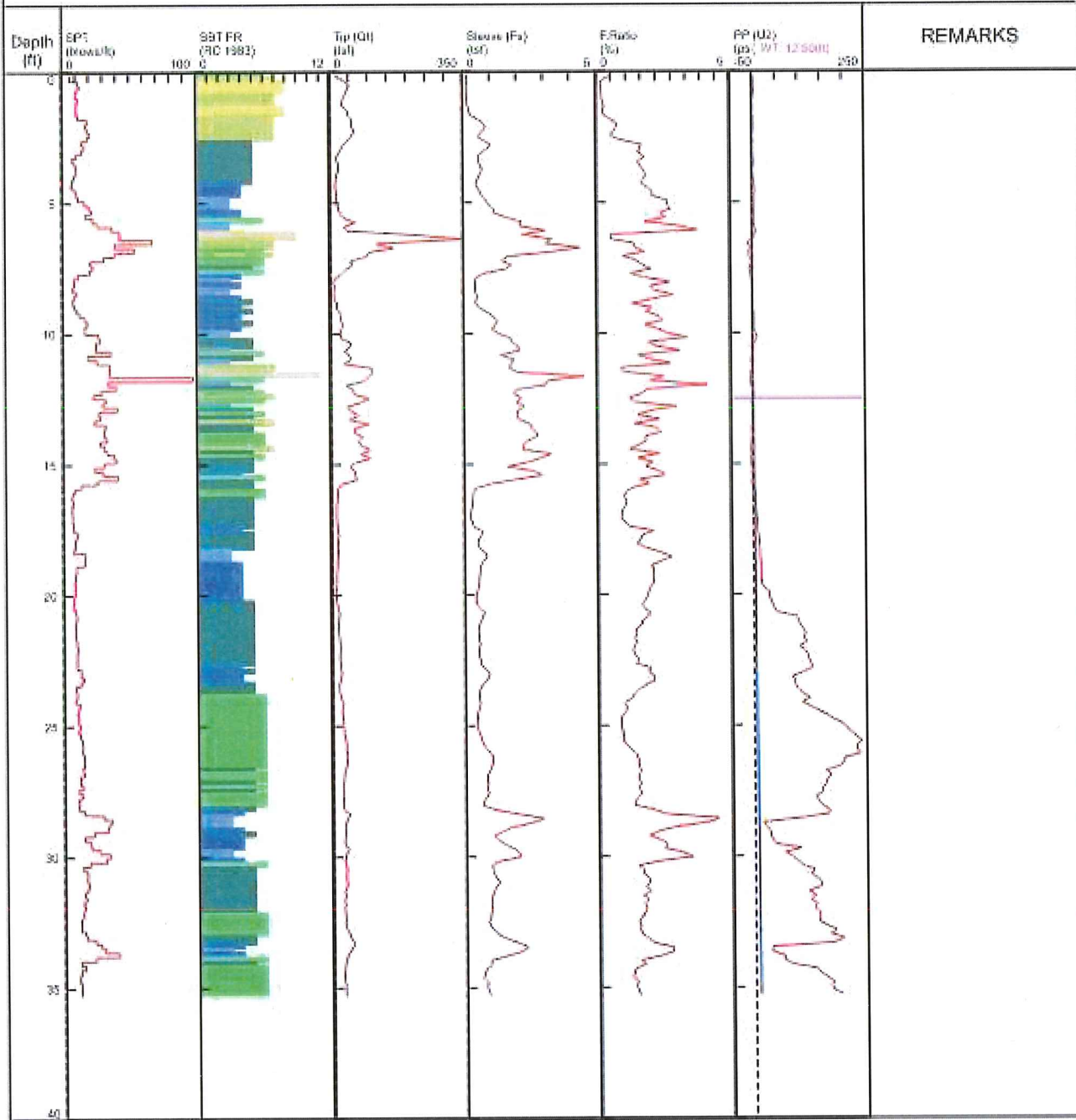


- |                          |                             |                            |                                |
|--------------------------|-----------------------------|----------------------------|--------------------------------|
| 1 sensitive fine grained | 4 silty clay to clay        | 7 silty sand to sandy silt | 10 gravelly sand to sand       |
| 2 organic material       | 5 clayey silt to silty clay | 8 sand to silty sand       | 11 very stiff fine grained (*) |
| 3 clay                   | 6 sandy silt to clayey silt | 9 sand                     | 12 sand to clayey sand (*)     |
- \*SBT/SPT CORRELATION: UBC-1983

<b>Summary Log of CPT-1</b>	
Aumsville WWTP	
 <b>CENTRAL</b> GEOTECHNICAL SERVICES	Figure A-7

# Central Geotech / CPT-2 / 945 Olney St SE Aumsville

OPERATOR: OGE DMM  
 CONE ID: DDG1296  
 TEST DATE: 7/29/2024 9:10:01 AM  
 TOTAL DEPTH: 35.269 ft

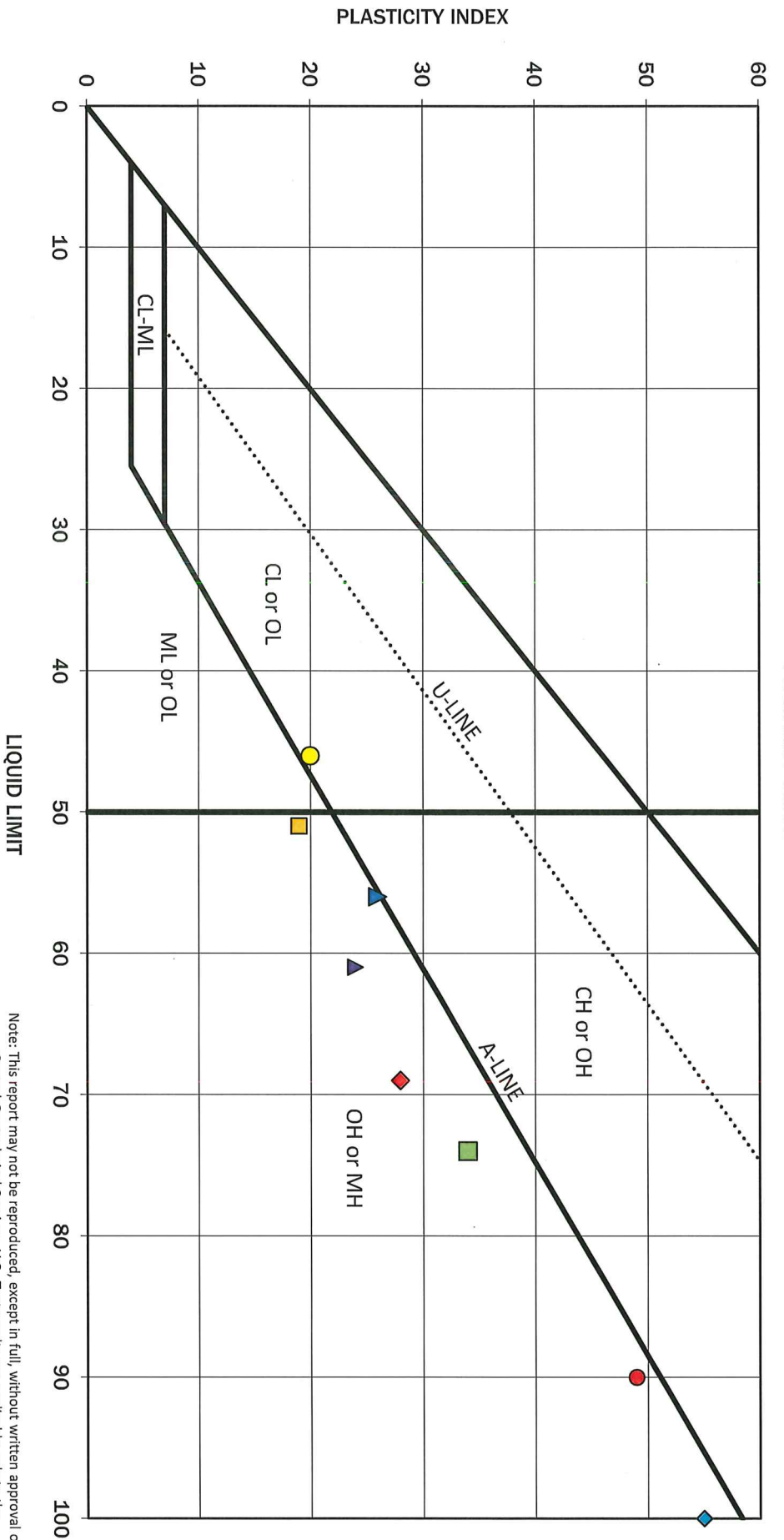


- |  |  |  |  |
|--|--|--|--|
| <ul style="list-style-type: none"> <li>1 sensitive fine grained</li> <li>2 organic material</li> <li>3 clay</li> </ul> | <ul style="list-style-type: none"> <li>4 silty clay to clay</li> <li>5 clayey silt to silty clay</li> <li>6 sandy silt to clayey silt</li> </ul> | <ul style="list-style-type: none"> <li>7 silty sand to sandy silt</li> <li>8 sand to silty sand</li> <li>9 sand</li> </ul> | <ul style="list-style-type: none"> <li>10 gravelly sand to sand</li> <li>11 very stiff fine grained (*)</li> <li>12 sand to clayey sand (*)</li> </ul> |
|--|--|--|--|

\*SBT/SPT CORRELATION: UBC-1983

<b>Summary Log of CPT-2</b>	
Aumsville WWTP	
	<b>Figure A-8</b>

PLASTICITY CHART



Symbol	Boring Number	Depth (feet)	Liquid Limit (%)	Plasticity Index (%)	Soil Description
▲	B-4	30	61	24	Elastic Silt (MH)
◆	B-1	25	69	28	Elastic Silt (MH)
◆	B-1	35	74	34	Elastic Silt (MH)
▲	B-1	40	56	26	Elastic Silt (MH)
●	B-3	5	46	20	Clay (CL)
■	B-3	30	51	20	Elastic Silt (MH)
■	B-3	65	100	55	Elastic Silt (MH)
●	B-4	45	90	45	Elastic Silt (MH)

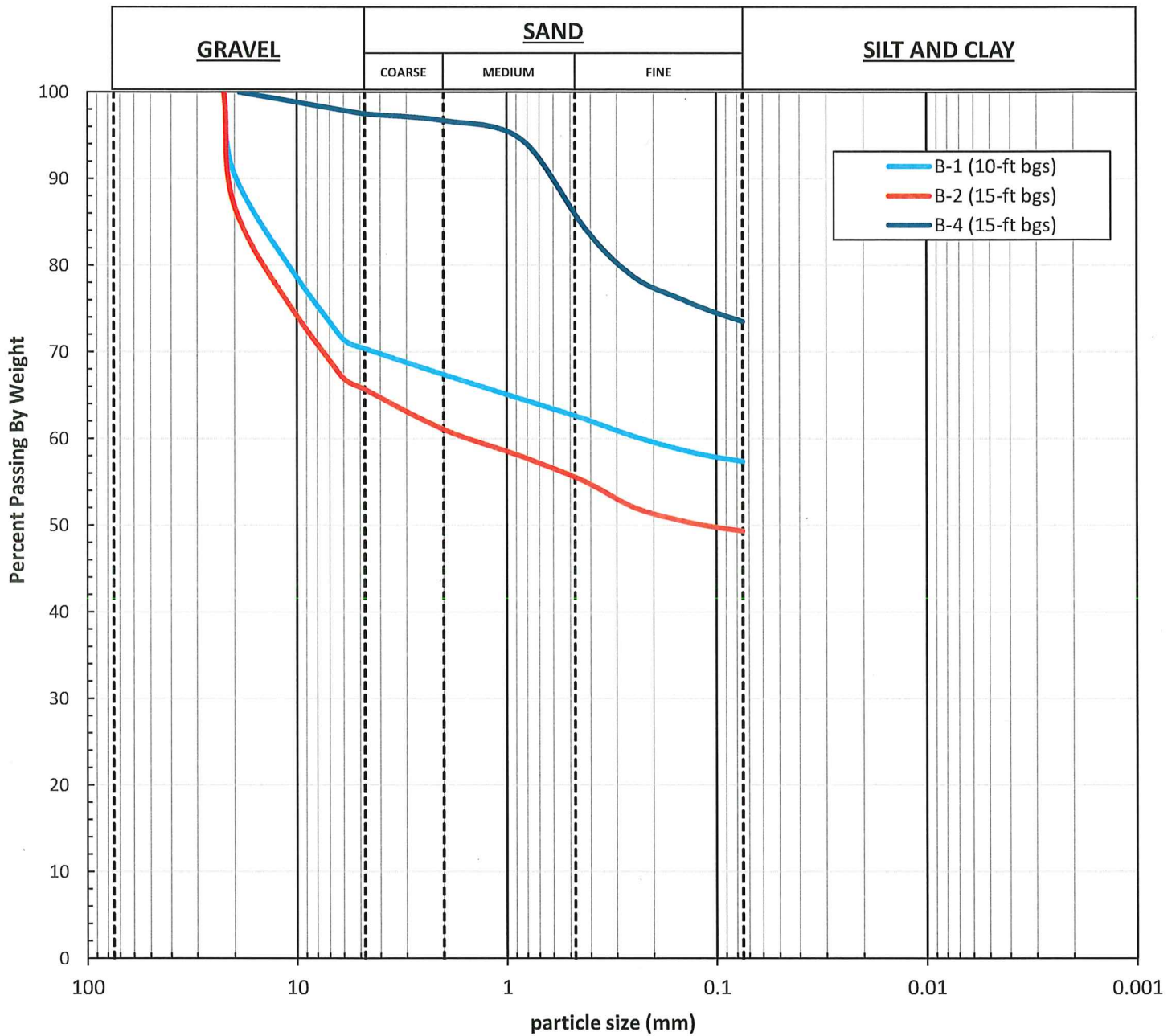
Note: This report may not be reproduced, except in full, without written approval of Central Geotechnical Services, LLC. Test results are applicable only to the specific sample on which they were performed and should not be interpreted as representative of any other samples obtained at other times, depths or locations, or generated by separate operations or processes.  
 The liquid limit and plasticity index were obtained in general accordance with ASTM D 4318.

Atterberg Limits Test Results

Aumsville WWTP  
 Aumsville, Oregon



Figure A-9



Boring Number	Depth of Sample (ft)	Gravel Total (%)	Sand Total (%)	Fines Total (%)	USCS
B-1	10	30	13	57	MH
B-2	15	34	16	49	GM
B-4	15	2.5	24	74	MH

### Grain Size Analysis ASTM C136 with D1140

**Note:** Test results are only applicable to the specific sample on which they were performed and should not be interpreted as representative of other samples obtained at other times, depths or location or generated by separate operations or processes. This report may not be reproduced, except in full, without written approval of Central Geotechnical Services LLC.



Aumsville WWTP  
Aumsville, Oregon

**Figure A-11**

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-3	25.0		42	78						
B-5	30.0		42	74						



LAB SUMMARY - GDI-NV5 CENTRALGEO-2-01.CPJ GDI\_NV5.GDT PRINT DATE: 9/4/24:



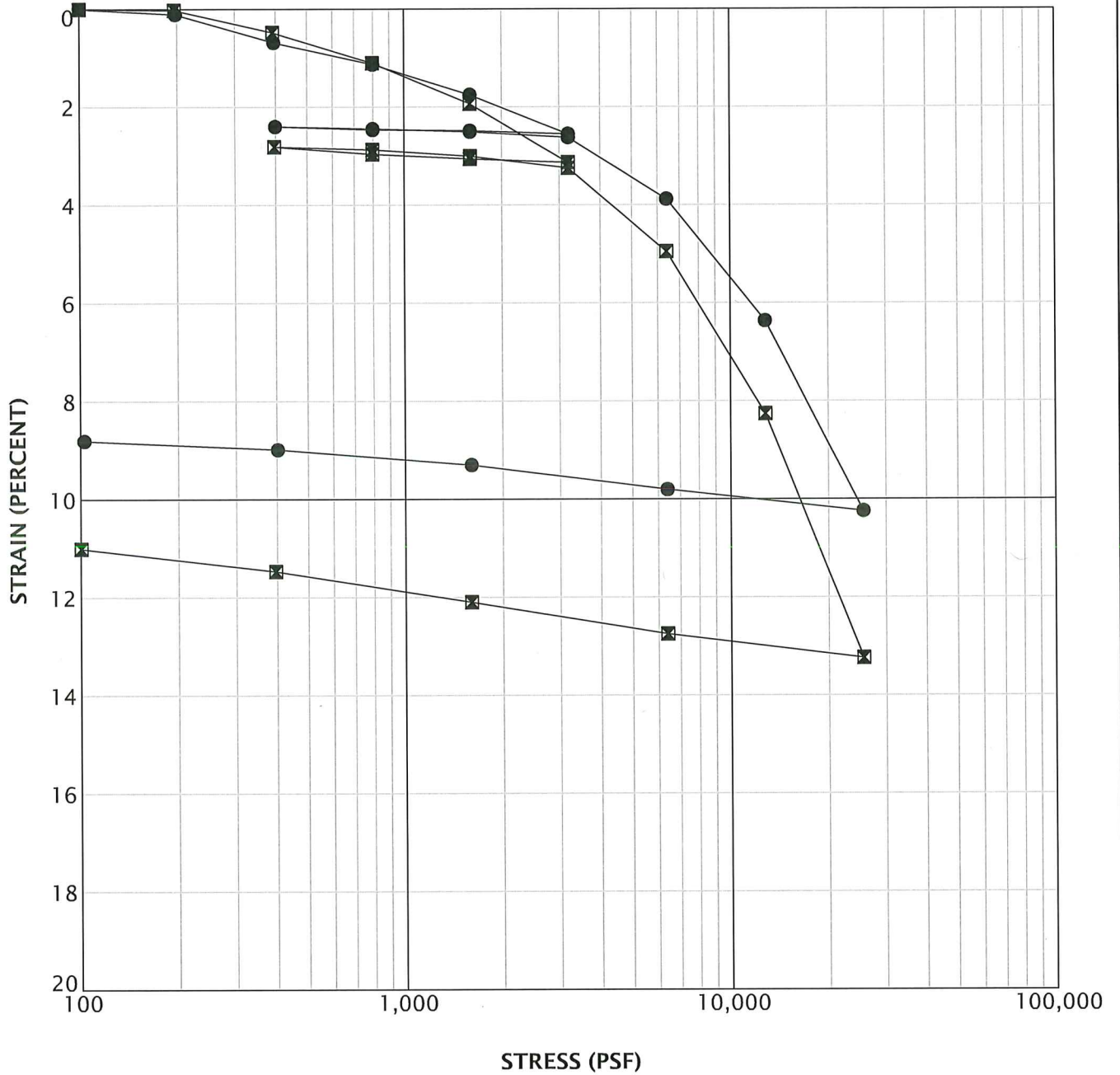
WESTERN-17-01

**SUMMARY OF LABORATORY DATA**

SEPTEMBER 2024

CENTRAL GEOTECH LAB SERVICES  
7662 SW MOHAWK STREET

**FIGURE**



KEY	EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	MOISTURE CONTENT (PERCENT)	DRY DENSITY (PCF)
●	B-3	25.0	42	78
⊠	B-5	30.0	42	74



WESTERN-17-01

**CONSOLIDATION TEST RESULTS**

SEPTEMBER 2024

CENTRAL GEOTECH LAB SERVICES  
7662 SW MOHAWK STREET

**FIGURE**





**APPENDIX B: Site Specific Seismic Analyses**

## **APPENDIX B**

### ***SITE SPECIFIC SEISMIC HAZARD ANALYSIS (2022 OSSC)***

#### **Introduction**

This appendix presents the results of Central's site-specific seismic hazard analysis for the proposed project located at. This report is prepared to meet the requirements of the Oregon Structural Specialty Code (OSSC) Section 1803 and the International Building Code (IBC).

The details of our field exploration program, subsurface conditions and measured groundwater levels are provided in the main report. The approximate field exploration locations are shown in the Site Plan, Figure-2.

#### **Regional Seismicity**

##### ***Earthquake Source Zones***

Tectonic conditions in the Aumsville area are similar to those typical of western Oregon; potential seismic sources include the convergent continental boundary known as the Cascadia Subduction Zone (CSZ), an approximately 720-mile-long thrust fault that extends along the Pacific Coast from mid Vancouver Island to Northern California. The CSZ is the zone where the westward advancing North American Plate is overriding the subducting Juan de Fuca Plate. The interaction of these two plates results in two potential seismic source zones: (1) the CSZ subduction-zone (or "interplate") source zone; and (2) the subducting slab/Benioff (or "intraslab") source zone. A third seismic source zone, referred to as the shallow crustal source zone, is associated with the regional convergent tectonic stresses. A discussion of these potential sources is provided below.

##### ***Cascadia Subduction Zone (CSZ)/Interplate Sources***

The closest part of the CSZ to the site are several strands of the "Cascadia Fold and Thrust Belt" mapped approximately 62 miles west of the site (Personius 2002a). This structure is a forearc-portion of the roughly 720-milelong zone of active tectonic convergence where oceanic crust of the Juan de Fuca Plate is subducting beneath the North American continent at a rate of 5 centimeters per year (DeMets et al. 2010). Most of these folds and faults are mapped as active in the Holocene or late Pleistocene, but some are mapped as active in the middle and late Quaternary (<700-780 ka) or Plio-Pleistocene (Personius 2002a).

##### ***Intraslab Earthquake Sources***

Typically, the region of maximum curvature of the subducting Juan de Fuca slab is where large intraslab earthquakes are expected to occur; in northern and central Oregon, this is assumed to be roughly 30 miles below the Oregon Coast Range. This area is about 27 miles west of the site. However, Wong (2005) notes that "...no moderate-to-large intraslab earthquakes of moment magnitude (M) 5.5 or greater have occurred within the subducting Juan de Fuca plate of the central CSZ from south of the Puget Sound in northwestern Washington to the Oregon-California border. Also, very few intraslab earthquakes as small as M3 have been instrumentally located within the central CSZ since 1960, and a Wadati-Benioff zone is not apparent." Therefore, we conclude that the hazard to the site from intraslab earthquakes is low.

##### ***Crustal Earthquake Sources***

The closest mapped fault to the site is the Turner and Mill Creek Faults, with the closest part of the fault about 0.6 miles west of the site (Personius 2002n). The second closest mapped fault is the Waldo Hills Fault, about 4.8-miles northwest of the site (Personius 2002m). Both are mapped as having low activity rates of less than 0.2 millimeters/year (mm/yr) (Personius 2002m) and Personius 2002n). The closest fault with documented historical activity is the Mount Angel Fault, about 29 miles southeast of the site (Black 1996). However, per the

USGS seismic deaggregation performed for the site used the 2023 National Seismic Hazard Model, the largest contributing hazard from crustal sources is the Turner and Mill Creek Faults. Therefore, these are considered the dominant crustal seismic sources for the site. The historical seismicity in relation to the project site is shown in Historical Seismicity Map, Figure B-1. The location and relative distance of Quaternary faults within a 50-mile radius from the site have been summarized from geologic publications and are shown in Quaternary Fault Map, Figure B-2. The estimated distance, displacement, and relative age of faults within a 50-mile radius from the site is shown in Table B-1 (Personius 2002b-q).

**Table B-1. Faults within site vicinity**

Fault Name	Proximity to the Site (surface projection in miles)	Estimated Displacement Description	Estimated Age	Estimated Slip Rate
Turner and Mill Creek faults	0.6	Offsets Columbia River Basalt flows and Miocene age deposits	Quaternary (<1.6 million years before present)	Less than 0.2 mm/yr
Waldo Hills fault	4.8	Offsets Columbia River Basalt flows. Does not offset surficial Quaternary deposits.	Quaternary (<1.6 million years before present)	Less than 0.2 mm/yr
Salem-Eola Hills homocline	7.1	Offsets Columbia River Basalt (CRB) flows and Miocene age deposits.	Quaternary (<1.6 million years before present)	Less than 0.2 mm/yr
Mount Angel fault	14.5	Offsets late Pleistocene and Holocene deposits. Associated with earthquake swarms near Woodburn (1990) and ML 5.6 earthquake near Scotts Mills (1993).	Latest Quaternary (<15,000 years before present)	Less than 0.2 mm/yr
Owl Creek fault	23.2	The middle to late Pleistocene Rowland Formation appears to be offset across the Creek fault, but no evidence of offset is apparent in the latest Pleistocene Willamette Formation.	Late to middle Quaternary (750,000 years before present)	Less than 0.2 mm/yr

Fault Name	Proximity to the Site (surface projection in miles)	Estimated Displacement Description	Estimated Age	Estimated Slip Rate
Canby-Molalla fault	24.3	Probable offset of Missoula deposits flood	Late Quaternary(<1.6 million years before present)	Less than 0.2 mm/yr
Newberg fault	30.7	Quaternary (<1.6 million years before present)	Quaternary (<1.6 million years before present)	Less than 0.2 mm/yr
Gales Creek fault zone	37.5	Offsets CRB flows	Quaternary (<1.6 million years before present)	Less than 0.2 mm/yr
Oatfield fault	38.6	Offsets CRB flows and Miocene age deposits	Quaternary (<1.6 million years before present)	Less than 0.2 mm/yr
Portland Hills fault	39.3	Offsets CRB flows	Quaternary (<1.6 million years before present)	Less than 0.2 mm/yr
Damascus-Tickle Creek fault zone	39.9	Offsets Pliocene Troutdale Formation, Pliocene-Pleistocene Springwater Formation and Pleistocene Boring Lava	Late to middle Quaternary (750,000 years before present)	Less than 0.2 mm/yr
Beaverton fault zone	42.9	Offsets CRB flows and Miocene age deposits	Late to middle Quaternary (750,000 years before present)	Less than 0.2 mm/yr
Grant Butte fault	46.3	Offsets Columbia River Basalt flows and overlain by of Missoula flood deposits	Late to middle Quaternary (750,000 years before present)	Less than 0.2 mm/yr

Fault Name	Proximity to the Site (surface projection in miles)	Estimated Displacement Description	Estimated Age	Estimated Slip Rate
East Bank fault	47.7	Offsets CRB flows and Miocene to Pliocene Troutdale Formation sediment	Late to middle Quaternary (750,000 years before present)	Less than 0.2 mm/yr
Helvetia fault	48.4	Probable offset of Miocene CRB Group	Quaternary (<1.6 million years before present)	Less than 0.2 mm/yr
Corvallis fault zone	48.6	Possible offsets in late Pleistocene age deposits. Does not offset Missoula flood deposits.	Quaternary (>15,000 years before present)	Less than 0.2 mm/yr

### Seismic Hazard Mapping

Madin and Wang (1998) generally maps the relative earthquake hazard in the vicinity of the project site as within Hazard Zone A on a four-category scale from Zone A (highest hazard) to D (lowest). Category B represents a moderate relative hazard. The hazard level is associated with liquefaction, amplification and landslide hazards.

### Liquefaction

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. 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 contents is the most susceptible to liquefaction. Low plasticity, silty sand may be moderately susceptible to liquefaction under relatively higher levels of ground shaking.

As discussed in the groundwater section of this report, relatively shallow groundwater is anticipated at the site. However, the predominately fine-grained site soils encountered below the groundwater are minimally susceptible to liquefaction during a design-level earthquake. Accordingly, lateral spreading or liquefaction induced deformations are not expected. Additionally, Madin and Wang map the site as having the lowest susceptibility to liquefaction or "No Liquefaction Hazard".

### Fault Surface Rupture

The closest mapped fault to the site capable of surface rupture is the Turner and Mill Creek Faults located approximately 0.6 miles north of the site (Personius 2002b). Other local crustal faults are listed in Table B-1. No faults are mapped as crossing the site, and the potential for site fault surface rupture is therefore, low.

### **Amplification**

Based on relative earthquake hazard mapping completed in the site vicinity (Madin et al. 1999), the site is mapped as having a low to medium amplification hazard on a four-tier classification system ranging from “highest amplification hazard” to “no amplification hazard.”

### **Earthquake Induced Landslides**

Based on the relative hazard mapping (Madin 1998) the site is mapped as presenting a “Low Landslide Hazard”. Site grades are generally flat or sloping gently at inclinations that in our opinion, present a low earthquake induced landslide hazard.

### **Tsunami Inundation/Seiche/Subsidence**

The site is inland and elevated away from tsunami inundation and subsidence zones. Portions of the site are anticipated to be subject to hazards from seiches that could develop within the effluent lagoons during the design earthquake event.

### **Design Earthquakes**

We have evaluated three potential earthquake sources that could impact building design at the site. These include CSZ interface earthquakes, CSZ intraplate earthquakes (also referred to as Benioff Zone or intraslab) and local crustal earthquakes.

Geologic evidence indicates that CSZ interface earthquakes occur approximately every 100 to 1,000 years with average recurrence intervals of approximately 400 years. Based on postulated rupture lengths, widths and displacements (as well as historical Pacific tsunamis), coastal subsidence and liquefaction evidence, magnitudes for such earthquakes are estimated to range from approximately Mw= 7.5 to 9.0.

CSZ intraplate earthquakes have occurred in the Puget Sound area and are also postulated to occur beneath Western Oregon. These earthquakes have a deep focus of 40 to 80 kilometers in the subducted Juan de Fuca Plate and theoretical magnitudes of up to 7.8. These earthquakes are expected to have epicenters from 25 to 35 miles from the site.

### **Ground Motion Estimation**

Peak ground accelerations (PGAs) were estimated for each of the events discussed above. The PGA values for the CSZ events and specific and random crustal events were computed with the ground motion models presented in Table B-1 for soils that have a shear wave velocity of approximately 220 meters per second. Figure B-3 shows the computed response spectra for the postulated ground motion events. We recommend against using the design spectrum

Table B-2 shows the postulated earthquakes and weighted amounts in the model; Table B-3 shows the computed PGAs. Response spectra for the 2022 OSSC hazard earthquake events are presented in Figure B-3.

Table B-2. Ground Motion models and weights

Earthquake Source	Ground Motion Model	Weight
Crustal	Abrahamson et al. (2014) [ASK14]	0.25
	Boore et al. (2014) [BSSA14]	0.25

	Campbell and Bozorgnia (2014) [CB14]	0.25
	Chiou and Youngs (2014) [CY14]	0.25
CSZ Benioff/Intraslab	Kuehn et al. (2020) – Cascadia [KBCG20-C]	0.40
	Parker et al. (2021) – Cascadia [PSBAH21-C]	0.20
	Abrahamson and Gülerce (2022) – Cascadia [AG22-C]	0.40
CSZ Interface	Kuehn et al. (2020) – Cascadia [KBCG20-C]	0.33
	Parker et al. (2021) – Cascadia [PSBAH21-C]	0.33
	Abrahamson and Gülerce (2022) – Cascadia [AG22-C]	0.34

**Table B-3. Calculated PGA Values**

Earthquake Source	Magnitude	Focal Depth (miles)	Anticipated PGA (g) (Site Class C)	Anticipated PGA (g) (Site Class D)
CSZ Interface	9.1	20	0.233	0.271
CSZ Intraslab	7.0	30	0.323	0.370
Crustal (GCFZ)	6.5	-	0.45	0.42

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**SITE SPECIFIC SEISMIC GROUND MOTION HAZARD ANALYSIS (ASCE7-16 CHAPTER 21)**

We completed a site-specific ground motion hazard analysis (GMHA) per ASCE7-16 to develop site-specific horizontal risk-targeted maximum considered earthquake ( $MCE_R$ ) and design earthquake (DE) response spectra and design acceleration parameters for the planned tank structures located at approximately 44.854232° N - 122.876894° W in Aumsville Oregon. The information presented below was completed using the USGS 2023 US-50-State National Seismic Hazard Model (NSHM).

**Ground Motion Hazard Analysis Procedure**

We completed the GMHA using the following procedure:

8. Determine the ASCE7-16 site class and design acceleration parameters based on the results of geophysical surface wave testing at the site (Appendix C.)
9. Complete a site-specific probabilistic seismic hazard analysis (PSHA) to compute the uniform hazard spectrum (UHS) corresponding to the site class determined in step 1 for the maximum considered earthquake (MCE) (i.e., 2-percent probability of exceedance in 50-years, or a 2475-year return period).
10. Determine Scaling factors and risk coefficients to convert the probabilistic MCE response spectral coordinates to the  $MCE_R$  spectral coordinates.
11. Perform a Deterministic seismic hazard analysis (DSHA) to develop the deterministic ( $MCE_R$ ) horizontal response spectrum.
12. Evaluate the site-specific  $MCE_R$  and DE response spectra as the lesser of the probabilistic and deterministic ground motions but not less than the ASCE7-16 section 21.3 minimum response spectra.

**Geophysical Testing**

We completed a geophysical testing program consisting of one active-source multichannel analysis of surface waves (MASW) survey and three passive source refraction microtremor arrays (MAM) to estimate shear velocity profiles as shown in Figure 2, and to determine the ASCE7-16 site classification. A summary of the seismic site class from the shear wave velocities measured in the upper 30 meters ( $Vs_{30}$ ) is provided in Table B-4, and the full geophysical testing report is included in Appendix C.

**Table B-4. ASCE 7-16 Site Class**

Line	$Vs_{30}$ (ft/sec)	ASCE7-16 Site Class
MAM + MASW 1	1293.6	C
MAM + MASW 2	1556.9	C
MAM + MASW 3	1422.8	C

**Site-Specific Probabilistic Seismic Hazard Analysis**

We completed a site specific PSHA to develop the MCE UHS adjusted for site class effects. Inputs to the PSHA include the seismic source characterization model (SSC) and ground motion characterization models (GMC).

### Seismic Source Characterization Model

We performed the PSHA using the 2023 United States Geological Survey (USGS) SSC model (Petersen et al. 2024), which contains seismic source and recurrence model parameters. The 2023 SSC model includes substantial changes from the previous 2018 model including more data, updated earthquake rupture forecasts, and ground-motion components. Specific updates relevant to this project location include updated crustal fault data, the inclusion of new ground motion models (GMM's) for subduction zone events, and to a lesser degree development of basin amplification factors for the Portland and Tualatin sedimentary basins.

### Ground Motion Characterization Model

The ground motion characterization model consists of the suite of ground motion models (GMM's) used to compute the ground motions for the three source-types: crustal, Cascadia subduction zone (CSZ) intraslab and CSZ interface. Tables B-5 and B-6 represents the median GMM logic-tree adopted for this project based on the 2023 NSHM for sites within and outside of the Western United States Basins considered by the NSHM.

**Table B-5. Ground Motion models and weights outside of WUS Basins**

Earthquake Source	Ground Motion Model	Weight
Crustal	Abrahamson et al. (2014) [ASK14]	0.25
	Boore et al. (2014) [BSSA14]	0.25
	Campbell and Bozorgnia (2014) [CB14]	0.25
	Chiou and Youngs (2014) [CY14]	0.25
CSZ Benioff/Intraslab	Kuehn et al. (2020) – Cascadia [KBCG20-C]	0.25
	Parker et al. (2021) – Cascadia [PSBAH21-C]	0.25
	Abrahamson and Gülerce (2022) – Cascadia [AG22-C]	0.25
	Zhao et al. (2006) – Cascadia [Zhao06]	0.25
CSZ Interface	Kuehn et al. (2020) – Cascadia [KBCG20-C]	0.25
	Parker et al. (2021) – Cascadia [PSBAH21-C]	0.25
	Abrahamson and Gülerce (2022) – Cascadia [AG22-C]	0.25
	Zhao et al. (2006) – Cascadia [Zhao06]	.125
	Atkinson and Macias (2009) – Cascadia [AM09]	.125

**Table B-6. Ground Motion models and weights within WUS Basins**

Earthquake Source	Ground Motion Model	Weight
Crustal	Abrahamson et al. (2014) [ASK14]	0.25
	Boore et al. (2014) [BSSA14]	0.25
	Campbell and Bozorgnia (2014) [CB14]	0.25
	Chiou and Youngs (2014) [CY14]	0.25
CSZ Benioff/Intraslab	Kuehn et al. (2020) – Cascadia [KBCG20-C] + basin	0.25
	Parker et al. (2021) – Cascadia [PSBAH21] + basin	0.25
	Abrahamson and Gülerce (2021) – Cascadia intraslab model [AG22-C]	0.0825
	Abrahamson and Gülerce - Intraslab model Adjusted (2021) Cascadia [AG22-C] + basin	0.1675
	Zhao et al. (2006) – Cascadia [Zhao06]	0.25
CSZ Interface	Kuehn et al. (2020) – Cascadia [KBCG20-C] + basin	0.25
	Parker et al. (2021) – Cascadia [PSBAH21-C] + basin	0.25
	Abrahamson and Gülerce (2022) – Cascadia [AG22-C] + basin	0.25
	Zhao et al. (2006) – Cascadia [Zhao06] + basin	.125
	Atkinson and Macias (2009) – Cascadia [AM09] + basin	.125

The suite of GMMs and their associated epistemic weights represent the practicing state-of-art in ground motion in the Pacific Northwest (PNW) region. The Next-Generation Attenuation-West-2 (NGA-West2) GMMs (ASK14, BSSA14, CB14, and CY14) are the latest and most comprehensive GMMs published for crustal sources. We assigned equal weights to the ASK14, BSSA14, CB14, and CY14 GMMs in accordance with the weighting scheme of the 2023 NSHM. The two older models AM09 and Zhao06 were included in the suite of GMM's for the 2023 NSHM to allow for additional epistemic uncertainty particularly for long-period ground motions (Peterson et al. 2023).

### Median Ground Motion Adjustments

We considered adjustments to the median ground motions produced from the PSHA for near-field directivity and sedimentary basin effects. However, the site is not located within a sedimentary basin that is considered by the 2023 NSHM and is not located within 15km of a mapped fault thought to be capable of producing a Mw 7 or larger event. Therefore, we did not include adjustment factors for these effects.

### Probabilistic (MCE<sub>R</sub>) Horizontal Response Spectrum

Table B-6 presents the spectral ordinates of the site specific probabilistic 5-percent damped MCE UHS and risk targeted MCE<sub>R</sub>. The spectral ordinates of the MCE have been adjusted using maximum horizontal component modification factors calculated using the USGS Earthquake Hazard Toolbox that are also presented in Table B-8. In addition to the UHS generated, dynamic hazard curves for annual frequency of exceedance versus spectral accelerations were obtained for a 2475-year return period for each period. Risk targeted spectral ordinates were then calculated at each spectral period by converting the UHS spectral ordinate to risk targeted coordinates using the USGS Risk-Targeted Ground Motion Calculator, which calculates these values by iterative integration of the site-specific hazard curve with a lognormal probability density function representing collapse fragility. The probabilistic MCE<sub>R</sub> response spectrum is also provided in Figure B-4.

**Table B-6. Probabilistic MCE And MCE<sub>R</sub>**

Period (sec)	5% Damped Spectral Acceleration, Sa (g) (UHS)	5% Damped Spectral Acceleration, Sa (g) MCE <sub>R</sub>
PGA	0.523	0.455
0.01	0.632	0.552
0.02	0.660	0.568
0.03	0.708	0.604
0.05	0.824	0.717
0.075	1.031	0.888
0.1	1.187	1.032
0.15	1.420	1.216
0.2	1.500	1.283
0.25	1.492	1.275
0.3	1.447	1.238
0.4	1.268	1.104
0.5	1.144	0.985

Period (sec)	5% Damped Spectral Acceleration, Sa (g) (UHS)	5% Damped Spectral Acceleration, Sa (g) MCE <sub>R</sub>
0.75	0.884	0.764
1.00	0.682	0.598
1.5	0.453	0.395
2.0	0.335	0.286
3.0	0.202	0.173
4.0	0.140	0.120
5.0	0.104	0.090
7.5	0.061	0.055
10	0.046	0.042

#### Deterministic (MCE<sub>R</sub>) Horizontal Response Spectrum

Per ASCE 7-16 Section 21.2.3, the site-specific MCE<sub>R</sub> horizontal ground motions are taken as the lesser of the probabilistic MCE<sub>R</sub> and deterministic (MCE<sub>R</sub>) ground motions. We evaluated the 84<sup>th</sup> percentile deterministic (MCE<sub>R</sub>) horizontal response spectrum per ASCE 7-16 Section 21. We computed maximum-direction deterministic response spectra for the controlling earthquake scenarios associated with the CSZ intraslab and CSZ interface sources and the controlling crustal sources at short periods using the suite of GMMs and weights in Table B-5 and the BE Vs30 value of 1,424 ft/sec (434 m/sec). Controlling events were determined by deaggregation of the PSHA using the USGS Earthquake Hazard toolbox and the 2023 NSHM.

In our experience, deterministic ground motions are higher than probabilistic ground motions for sites in Western Oregon, which are located near seismic sources that can produce large but infrequent earthquakes. Completing DSHA based on Vs30-based site factors is approximate, but it adequately demonstrates probabilistic ground motions control as shown in figures B-4 through B-8 and as discussed in the following section.

Table B-7 summarizes the magnitude and distance parameters for the deterministic scenarios. Figure B-5 presents the development of the recommended deterministic (MCE<sub>R</sub>) horizontal response spectrum, which is taken as the envelope of the 84<sup>th</sup> percentile (RotD50) deterministic response spectra from the three primary source-types and then adjusted to be in the orientation of maximum horizontal response using the maximum component adjustment factors provided in Table B-8.

**Table B-7. Deterministic Earthquake GMM Parameters**

Parameter	Turner and Mill Creek Faults	CSZ Intraslab	CSZ Interface
Mw	6.62	7.5	9.1

Parameter	Turner and Mill Creek Faults	CSZ Intraslab	CSZ Interface
Vs30 (m/sec)	434	434	434
R <sub>RUP</sub> (km)	3.92	64.03	73.74
R <sub>JB</sub> (km)	3.92	-	-
R <sub>X</sub> (km)	-3.92	-	-
Style of Faulting	Reverse	-	-
Dip (degrees)	60	-	-
Z <sub>TOR</sub> (km)	0	54	50
Z <sub>HYP</sub> (km)	-	54	50
Z <sub>2.5</sub> (km)	-	5	5
W (km)	15	-	-

**Table B-8. Maximum Component Modification Factor**

Period (sec)	Maximum Component Adjustment Factor
PGA	1.1
0.01	1.1
0.02	1.1
0.03	1.1
0.05	1.1
0.075	1.1
1	1.1
0.15	1.1

Period (sec)	Maximum Component Adjustment Factor
0.2	1.1
0.3	1.1125
0.4	1.125
0.5	1.15
0.75	1.175
1	1.2375
1.5	1.3
2	1.325
3	1.35
4	1.4
5	1.45
7.5	1.5
10	1.5

#### Site Specific Design Acceleration Parameters

Per ASCE 7-16 Section 21.4, the site-specific  $S_{DS}$  is taken as 90 percent of the maximum design  $S_a$  between  $T=0.2$  to  $5.0$  sec. The site-specific  $S_{D1}$  is taken as the maximum value of the product,  $T \times S_a$ , between  $T=1.0$  to  $2.0$  sec for  $V_{s30} \geq 1,200$  ft/sec. The site-specific  $S_{DS}$  and  $S_{D1}$  values shall not be less than 80% of the site-specific minimum parameters provided in Table E-2. Table E-13 presents the site specific-  $S_{DS}$  and  $S_{D1}$  values.

Per ASCE 7-16 Section 21.4, for use with the ELF procedure, the site-specific  $S_a$  at some structural period  $T$  shall be permitted to replace  $S_{D1}/T$  in Eq. (12.8-3). The site-specific  $S_{DS}$  (Table B-9) shall be permitted to be used in Eqs. (12.8-2), (12.8-5), (15.4-1), and (15.4-3). The mapped value of  $S_1$  (table 5) shall be used in Eqs. (12.8-6), (15.4-2), and (15.4-4).

**Table B-9. Site-specific Spectral Parameters**

Parameter	Acceleration (g)	Mapped Value (g)
$S_{Ds}$	0.770	0.755 g
$S_{D1}$	0.399	0.381 g

**Site Specific Recommended  $PGA_M$**

We computed the site-specific geometric mean MCE peak ground acceleration ( $PGA_M$ ) by taking the lesser of the probabilistic and deterministic geometric mean peak ground accelerations that were computed in accordance with ASCE7-16 section 21.5. Site specific and prescriptive  $PGA_M$  values are provided in Table B-10.

**Table B-10. Site-specific  $MCE_G$  and  $PGAM$**

Parameter	Acceleration (g)
Mapped $MCE_G$ Peak Ground Acceleration, $PGA$ (g)	0.348
$PGA$ Site Coefficient, $F_{PGA}$	1.20
ASCE7-16 $PGA_M$ (g)	0.435
80% ASCE 7-16 $PGA_M$ (g) [Code Minimum]	0.334
Site Specific $MCE_G$ $PGA_M$ (g)	0.470
Recommended $PGA_M$	0.470

**Site-Specific  $MCE_R$  and DE Response Spectra and Parameters**

As noted previously, ASCE 7-16 Section 21.2.3, the site-specific  $MCE_R$  horizontal ground motions are taken as the lesser of the probabilistic  $MCE_R$  and deterministic ( $MCE_R$ ) ground motion, but no less than the 80 percent of the Spectral acceleration determined in accordance with 11.4.6. The design spectrum was determined by taking two-thirds of the  $MCE_R$  at the respective periods per ASCE7 21.3. The Site specific  $MCE_R$  and recommended design response spectra are provided in Figures B-6, and B-7 and B-8, respectively.

**References**

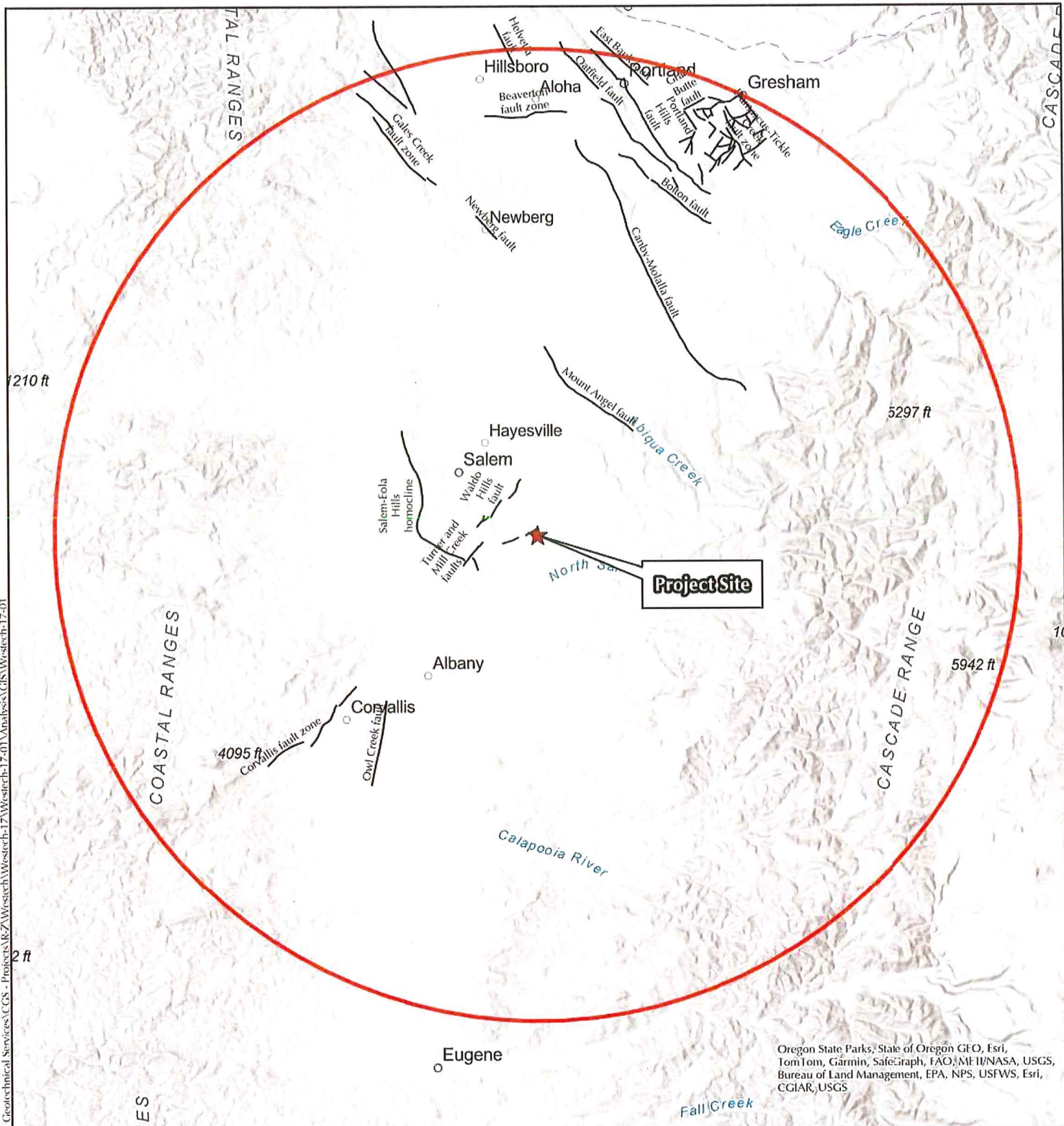
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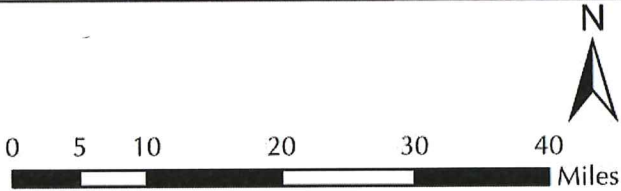
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**Legend**

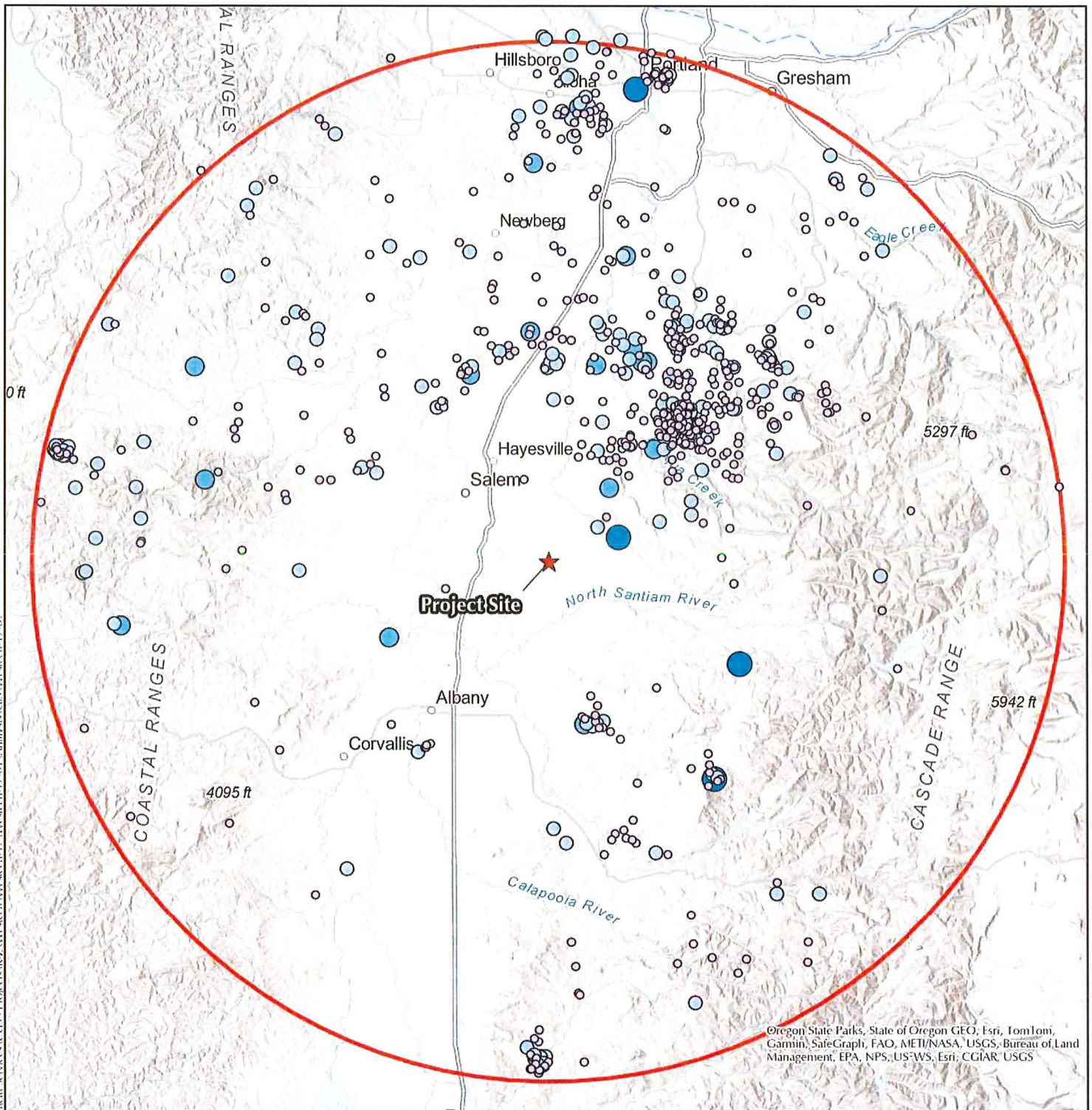
- ★ Project Site
- 50-Mile Radius
- USGS Quaternary Folds and Faults

**Notes:**

1. The locations of all features shown are approximate.  
 2. This drawing is for informational purposes. It is intended to assist in showing features discussed in the attached document. Central Geotechnical Services, LLC, cannot guarantee the accuracy and content of electronic files. The master file is stored by Central Geotechnical Services, LLC, and will serve as the official record of this communication.  
 Quaternary Faults from USGS, <http://earthquake.usgs.gov/hazards/qafaults/>  
 Projection: NAD 1983 UTM Zone 10N



Westech-17-01	
Quaternary Faults	
<b>CENTRAL</b> GEOTECHNICAL SERVICES	<b>Figure B-1</b>



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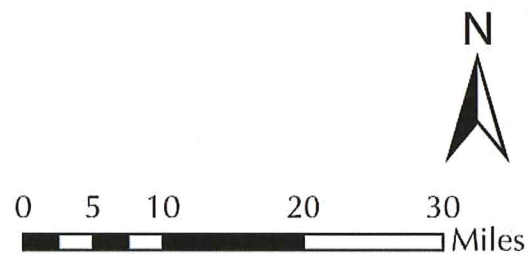
© Oregon State Parks, State of Oregon GEO, Esri, TomTom, Garmin, SafeGraph, FAO, METI/NASA, USGS, Bureau of Land Management, EPA, NPS, USFS, Esri, CGIAR, USGS

**Legend**

- Project Site
- 50-Mile Radius

**USGS Earthquakes by Magnitude**

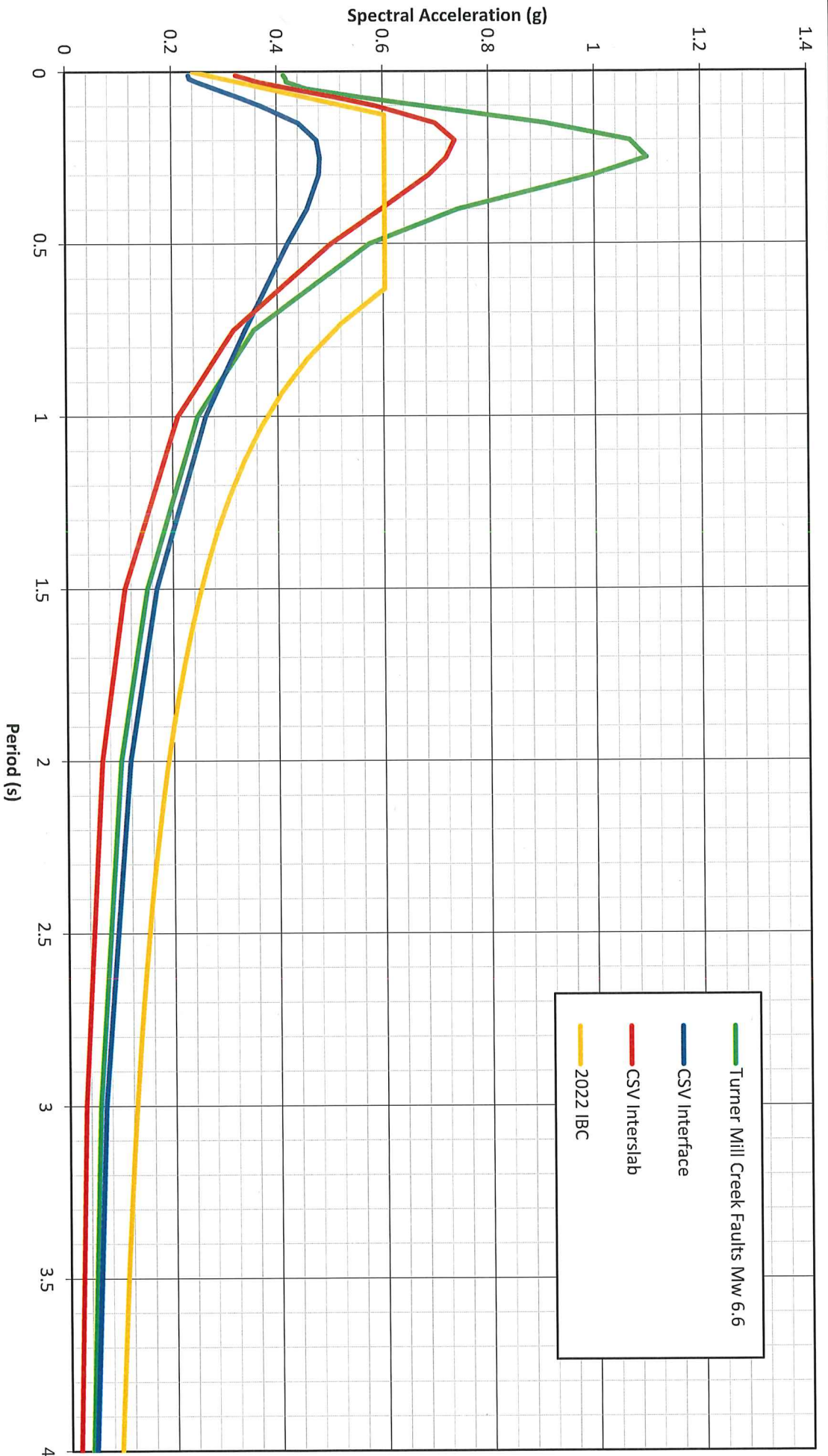
- < 2
- 2 - 3
- 3 - 4
- 4 - 5
- > 5



**Notes:**


1. The locations of all features shown are approximate.
2. This drawing is for informational purposes. It is intended to assist in showing features discussed in the attached document. Central Geotechnical Services, LLC. cannot guarantee the accuracy and content of electronic files. The master file is stored by Central Geotechnical Services, LLC, and will serve as the official record of this communication. Quaternary Faults from USGS, <http://earthquake.usgs.gov/hazards/qafaults/> Projection: NAD 1983 UTM Zone 10N

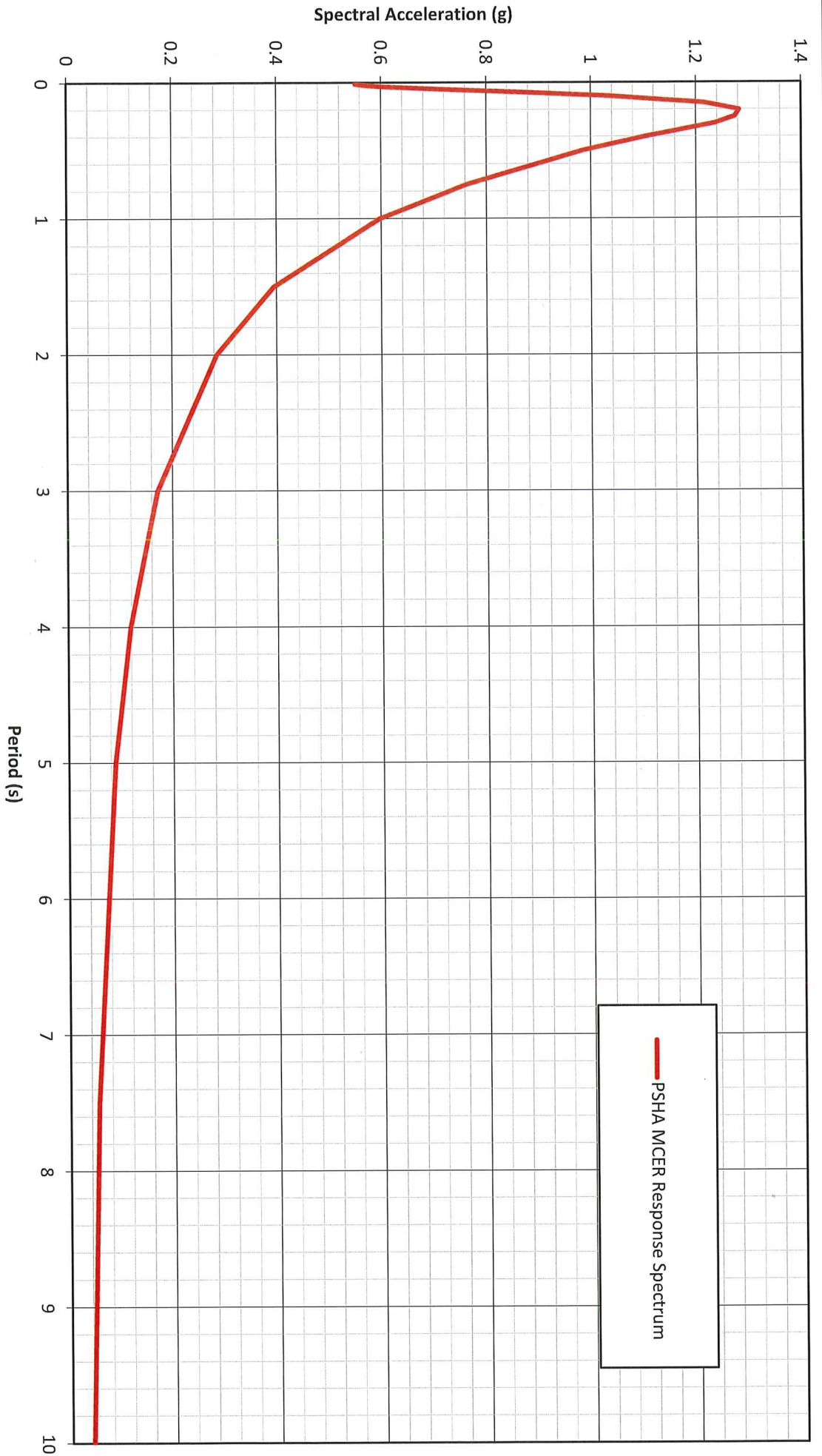
Westch-17-01	
Historical Seismicity	
	<b>CENTRAL</b> GEOTECHNICAL SERVICES
<b>Figure B-2</b>	




**Notes:**

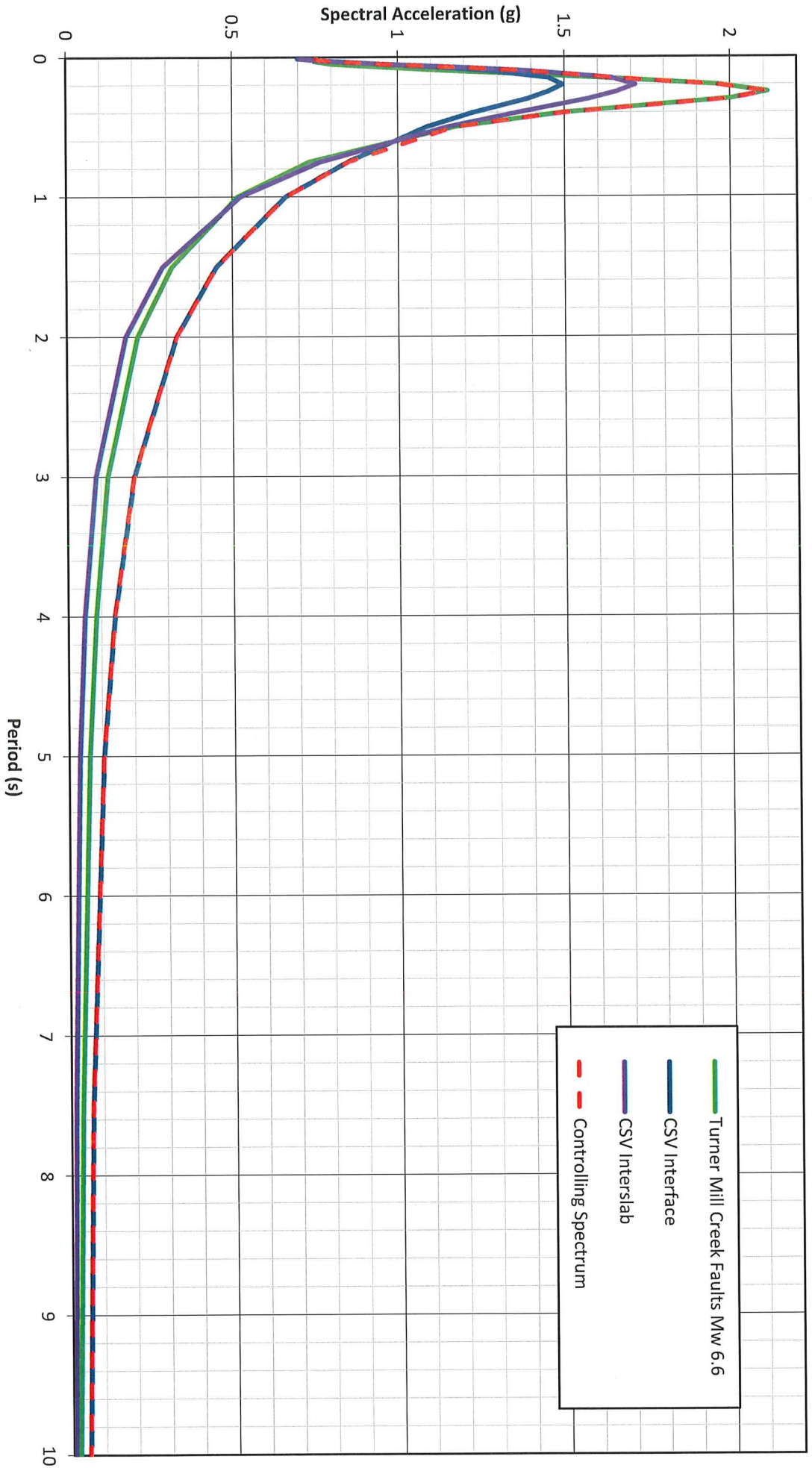
1. Deterministic Seismic Hazard Analysis response spectra are for illustration only. Final recommended site-specific MCER and DE response spectra are provided in Figures B-6, and B-7 and B-8, respectively.
2. IBC response spectra is for site class C conditions.

<b>2022 OSSC Hazard Response Spectra</b>	
Aumsville WWTP Aumsville, Oregon	
	<b>Figure B-3</b>




- Notes:**
1. Probabilistic Seismic Hazard Analysis response spectra are for illustration only. Final recommended site-specific MCER and DE response spectra are provided in Figures B-6, and B-7 and B-8, respectively.

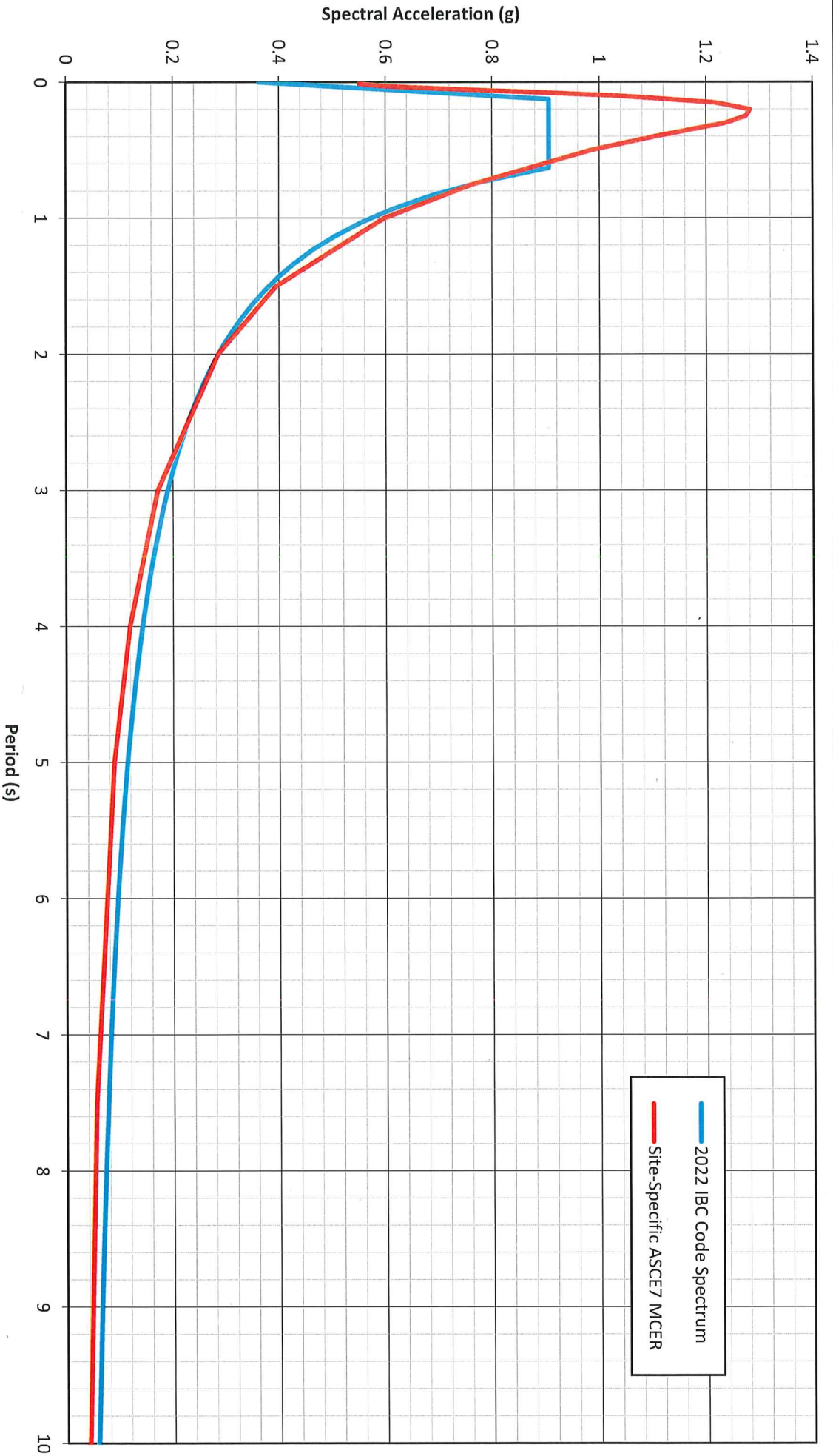
<b>GMHA PSHA Response Spectrum</b>	
Aumsville WWTP Aumsville, Oregon	
	<b>Figure B-4</b>



**Notes:**

1. Deterministic Seismic Hazard Analysis response spectra are for illustration only. Final recommended site-specific MCER and DE response spectra are provided in Figures B-6, and B-7 and B-8, respectively.

<b>GMHA DSHA Response Spectra</b>	
Aumsville WWTP Aumsville, Oregon	
	Figure B-5



**Notes:**

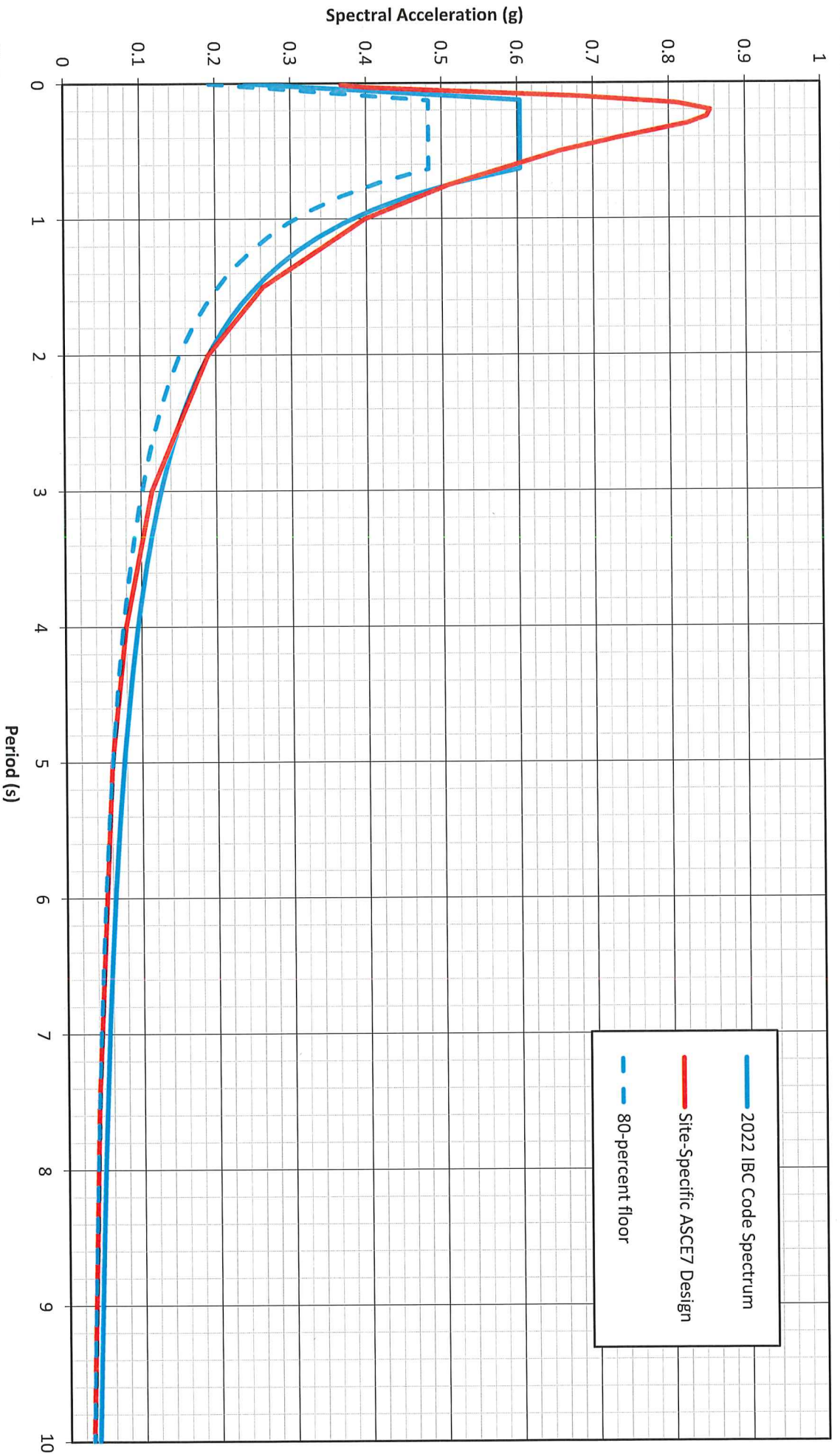
1. Final site-specific spectrum is recommended where a risk-targeted spectra is required.

**Site Specific MCER Response Spectra**


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Aumsville, Oregon

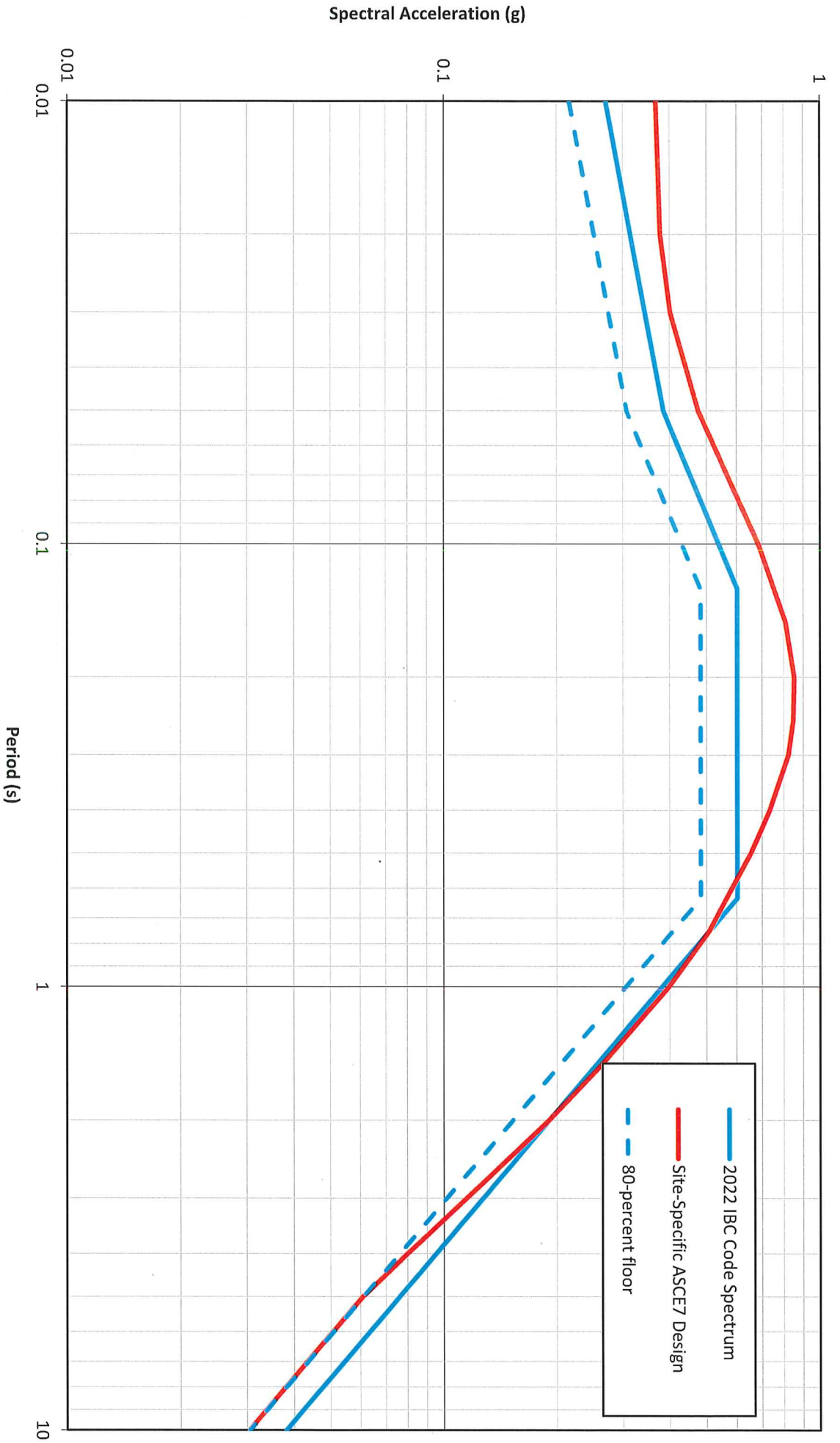


Figure B-6




- Notes:**
- Final site-specific spectrum is recommended where a design response spectrum is required.

<b>Site-Specific Design Response Spectra</b>	
Aumsville WWTP Aumsville, Oregon	
	<b>Figure B-7</b>



**Notes:**

1. Final site-specific spectrum is recommended where a design response spectrum is required.

<b>Site-Specific Design Response Spectra</b>	
Aumsville WWTP Aumsville, Oregon	
	<b>Figure B-8</b>



**APPENDIX C: Geophysical Report**

Geophysical Survey LLC  
711 S Tacoma Street  
Kennewick, Washington 99336

August 21, 2024

Blayne Sandau, PE, GIT  
Central Geotechnical Services  
10240 SW Nimbus Ave Ste L6,  
Portland, OR 97223

**Re:** *Seismic Survey  
Aumsville WWTP  
Aumsville, Oregon*

Mr. Sandau:

Geophysical Survey LLC conducted a seismic survey at the Aumsville Wastewater Treatment Plant in Aumsville, Oregon on July 29 through August 3, 2024. Seismic surface wave data were collected along three alignments (RL-1 through RL-3) at the project site (Figure 1). Additionally, shear wave data was collected along a 2D profile and HVSR data was recorded onsite. The objectives of the survey were to characterize subsurface conditions and calculate the seismic site classification.

## **Methodology**

### ***Seismic Surface Waves***

#### ***Microtremor Array Measurements***

The Microtremor Array Measurements (MAM) method is based on the change in phase velocity with frequency of seismic surface waves. Shear wave velocity ( $V_s$ ) is calculated by mathematical inversion of the dispersive phase velocity of surface waves. There are multiple types of surface waves, MAM utilizes Rayleigh waves which is the dominant component of ground roll. MAM utilizes a 'passive source' for seismic energy, also referred to as 'microtremor surveying'. The sources of seismic energy are wind, wave motion or cultural noise. The seismic energy creates Rayleigh waves which are recorded over a line of receivers in a linear array linked to a seismograph.

Surface wave energy decays exponentially with depth beneath the surface. Longer wavelength data (longer period and lower frequency) surface waves travel deeper and contain more information about deeper velocity structure. MAM data is most useful for lower frequency, deeper velocity structure.

#### ***Multi-channel Analysis of Surface Waves***

The Multi-channel Analysis of Surface Wave (MASW) method is based on the change in phase velocity with frequency of seismic surface waves. Shear wave velocity ( $V_s$ ) is calculated by mathematical inversion of the dispersive phase velocity of surface waves. There are multiple types of surface waves, MASW utilizes Rayleigh waves which is the dominant component of ground roll. MASW utilizes an 'active source' for seismic energy that is generated at a specific location and recording begins when source energy is imparted into the ground. The seismic energy creates Rayleigh waves which are recorded over a line of receivers in a linear array linked to a seismograph.

Surface wave energy decays exponentially with depth beneath the surface. Longer wavelength data (longer period and lower frequency) surface waves travel deeper and contain more information about deeper velocity structure. MASW data is most useful for higher frequency, shallow velocity structure.

### ***Seismic Surface Waves 2D***

The Multi-channel Analysis of Surface Wave (MASW) method, as previously discussed, is incorporated in producing a 2D profile. One dimensional (1-D) shear-wave (S-wave) velocity ( $V_s$ ) profiles were generated for a series of shot locations along a linear array. Each individual 1-D profile is spatially plotted at the center of each geophone array. A 2-D color gradient model was then created from the 1-D models using the SeisImager interpolation scheme. It should be emphasized that the 2-D profile represents the represent the average condition across the length of the geophone array

### ***Horizontal/Vertical Spectral Ratio***

Horizontal/Vertical Spectral Ratio (HVSR) data was collected at the site. The HVSR method uses a single, broadband, three-component seismometer that measures seismic noise in three-orthogonal directions (i.e., two horizontal and one vertical). The raw seismic-trace data show the amplitudes of acoustic noise in the time domain (i.e., over time) for each direction. The two-horizontal components are averaged merged into a single frequency spectrum, which is used to compute the ratio of horizontal-to-vertical (H/V) spectra.

## **FIELD SURVEY**

### ***Mapping Control***

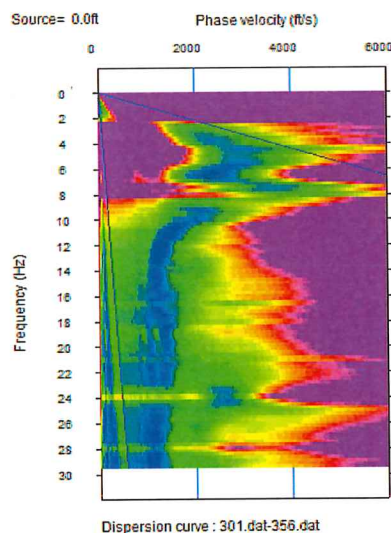
Line shotpoints were mapped with a Trimble Pro6H GNSS (Global Navigation Satellite System) receiver with decimeter accuracy (<10cm).

### ***Seismic Surface Wave Acquisition***

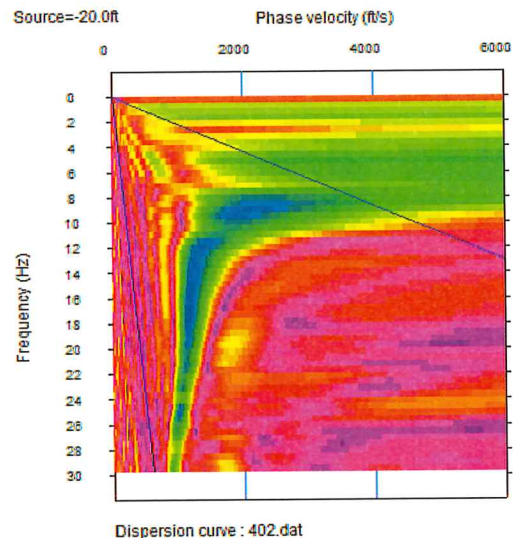
MASW data were recorded on twenty-four 4.5Hz geophones spaced 10 feet apart using a Geometrics Geode seismic controller. Three shotpoints along the 24-geophone spread were collected and digitally recorded on a laptop computer. Data were recorded at 0.5 millisecond intervals for 2 second records at each shot point.

MAM data were recorded on twenty-four 4.5Hz geophones spaced 10 feet apart using a Geometrics Geode seismic controller. 30 records, each 32 seconds long with a 2.0 millisecond sample interval, were recorded.

Seismic data was interpreted using SeisImager SW software V4.0. from Geometrics. A phase velocity -frequency plot was made from each MASW and MAM wave form and phase velocities were picked on amplitude. MAM and MASW phase velocity dispersion curves were combined and used to create an initial Vs model with depth. The initial model was inverted using a least squares method to determine the best fit of the model to the data. Seismic surface wave results are results are presented on Figure 3.



**MAM Phase Velocity-Frequency Plot**



**MASW Phase Velocity-Frequency Plot**

### ***Seismic Surface Waves 2D Acquisition***

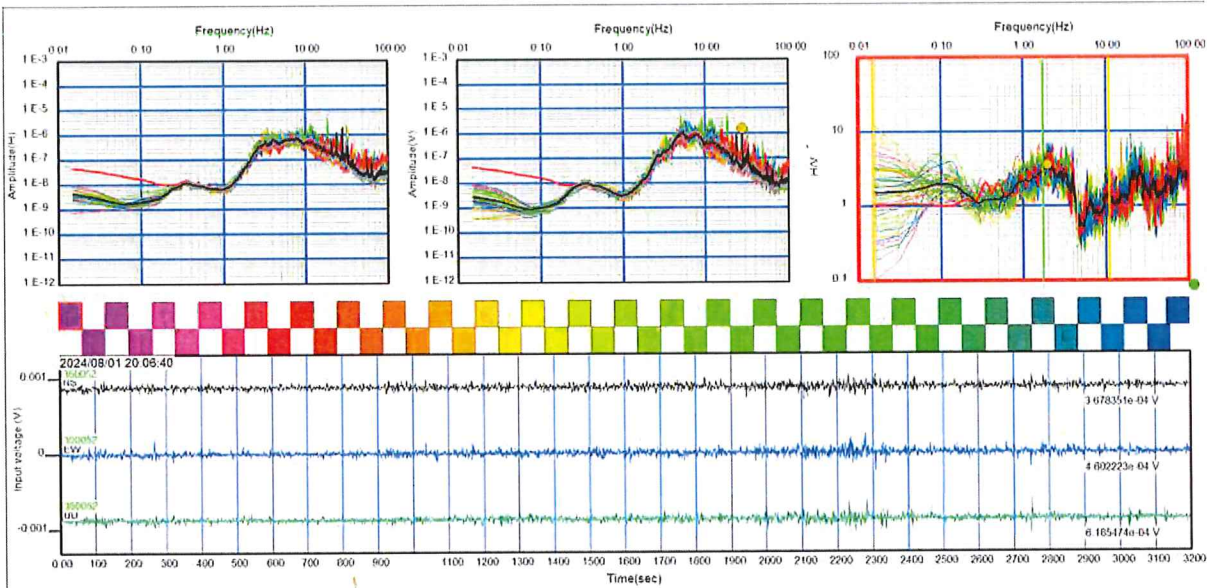
The MASW method for generating a 2D profile of rock, utilizes an active seismic signal generated by a hammer and plate (shot). Data were recorded using a 24-channel Geometrics Geode seismograph and 24, 4.5-Hz vertical component geophones. The geophones were coupled to the ground surface stationed 10 feet apart. Shots were conducted off the end of the lines. Prior to the collection of surface wave data, near and far field effects were evaluated for several shot offset distances at each traverse. The results indicated that the optimum offset distance for the shot point of the MASW study was 20 feet for the traverse.

Three records, each two seconds in length, were recorded at each shot location. After each shot, the shot location and geophones were shifted 20 feet longitudinally along the profile direction. Seismic data was interpreted using SeisImager SW/2D software V5.2 from Geometrics.

### ***Horizontal/Vertical Spectral Ratio***

Horizontal/Vertical Spectral Ratio (HVSr) data was collected using a Geometrics Atom 3-C triaxial geophone. The Atom 3-C was deployed onsite, and coupled with the ground, to collect ambient ground vibration data. Data was recorded using 4 millisecond intervals for 20 second record lengths. This process was repeated at multiple locations to establish an average HVSr for the entire site.

Seismic data was interpreted using SeisImager SPAC+ software V4.0. from Geometrics. A common time block window is selected from the raw time series data. The two-horizontal components are averaged merged into a single frequency spectrum, which is used to compute the ratio of horizontal-to-vertical (H/V) spectra.



**HVSr Time Series Plot**

## RESULTS AND INTERPRETATION

Figure 1 shows the location of the seismic tests. The seismic VS100 soundings captured some variability in subsurface conditions. However, the data is generally consistent with regards to the calculation of the IBC Seismic Site response. All results indicate the site response as a **Seismic Site Class C** (See figures 2a through 2c, and Table 1).

**Table 1: Seismic Site Class**

Table: NEHRP seismic site classification based on shear-wave velocity (Vs) ranges.

Site Class	S-Velocity (Vs) (ft/sec)	S-Velocity (Vs) (m/sec)
A (Hard Rock)	> 5,000	> 1500
B (Rock)	2,500 – 5000	760 – 1500
C (Very Dense Soil and Soft Rock)	1,200 – 2,500	360 – 760
D (Stiff Soil)	600 – 1,200	180 – 360
E (Soft Clay Soil)	< 600	< 180
F (Soils requiring additional response)	< 600, and meeting some additional conditions.	< 180, and meeting some additional conditions.

The results for the 2D MASW seismic evaluation are depicted on Figure 3. Relatively soft material (i.e., alluvium or clay) is present in the near surface. More competent material is encountered at approximately 10 feet BGS. A lens of low velocity material (i.e., unconsolidated sediment or clay) is pronounced on the eastern edge of the profile at a depth of 25ft to 40ft BGS. This feature tapers towards the central/western portion of the profile. More competent rock is detected at depth.

The HVSR data reported on figure 4 is the compiled average results from three test locations. The results have fulfilled Sesame criteria.

## CLOSURE

Geophysical surveys performed as part of this survey may or may not successfully detect or delineate any or all subsurface objects or features present. Locations, depths and scale of buried objects or subsurface features mapped as a result of this survey are a result of geophysical interpretation only, and should be considered as confirmed, actual, or accurate only where recovered by excavation or drilling.

Aumsville WWTP  
Aumsville, Oregon  
August 21, 2024

Page 7

Geophysical Survey LLC performed this work in a manner consistent with the level of skill ordinarily exercised by members of the profession currently practicing under similar conditions. No warranty, express or implied, beyond exercise of reasonable care and professional diligence, is made. This report is intended for use only in accordance with the purposes of the study described within.

Respectfully,

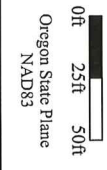
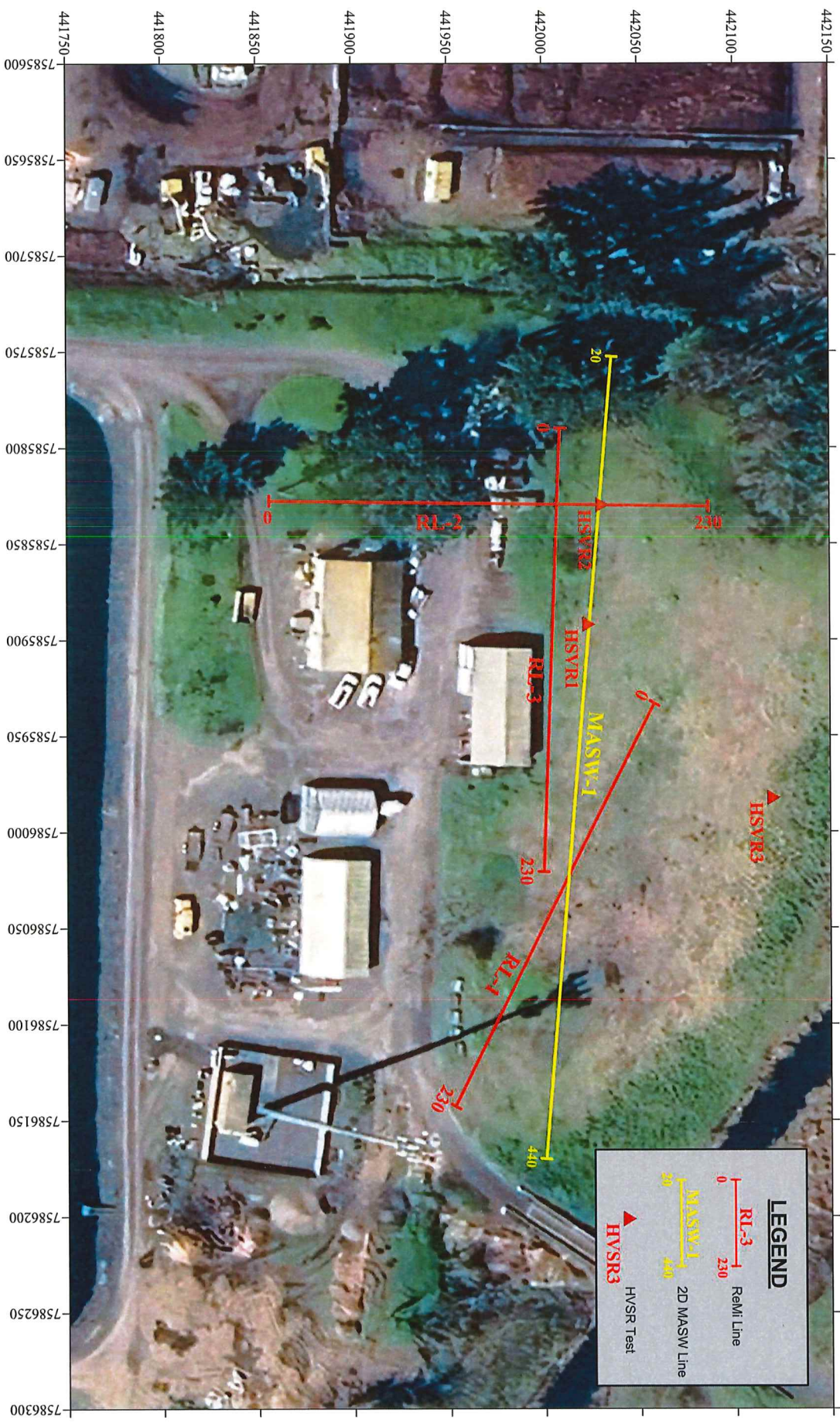
A handwritten signature in cursive script that reads "Andrew Baird".

Geophysical Survey LLC  
Andrew Baird R.G.  
Project Geophysicist

**Seismic Survey  
Aumsville WWTP  
Aumsville, Oregon**

**LIST OF FIGURES**

Figure 1	Site Map
Figure 2a	Seismic Surface Wave Profile
Figure 2b	Seismic Surface Wave Profile
Figure 2c	Seismic Surface Wave Profile
Figure 3	Seismic Data Profile
Figure 4	HVSR Results



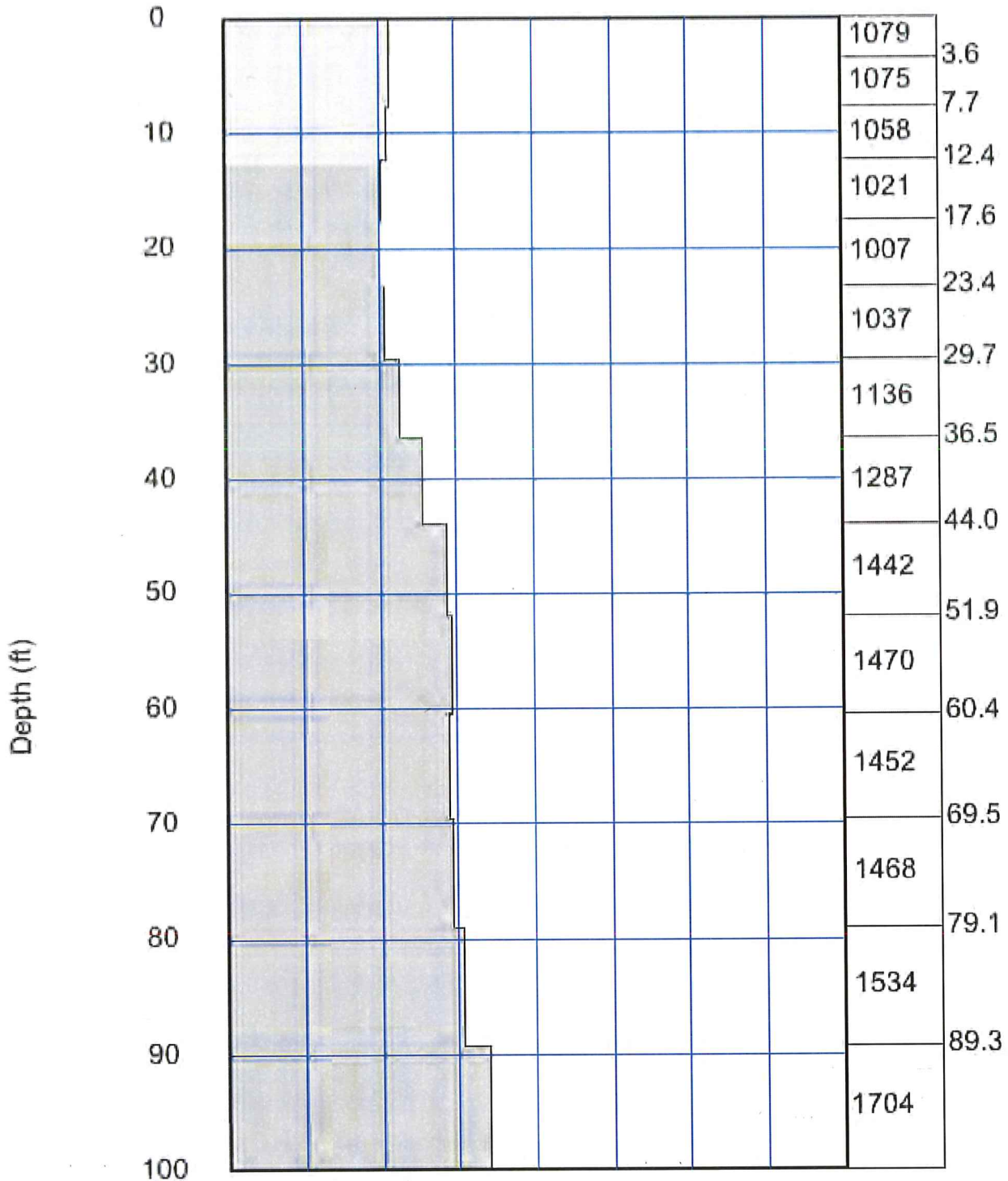
Aumsville WWTP  
Aumsville, Oregon



**FIGURE 1**  
Site Map

### S-wave velocity (ft/s)

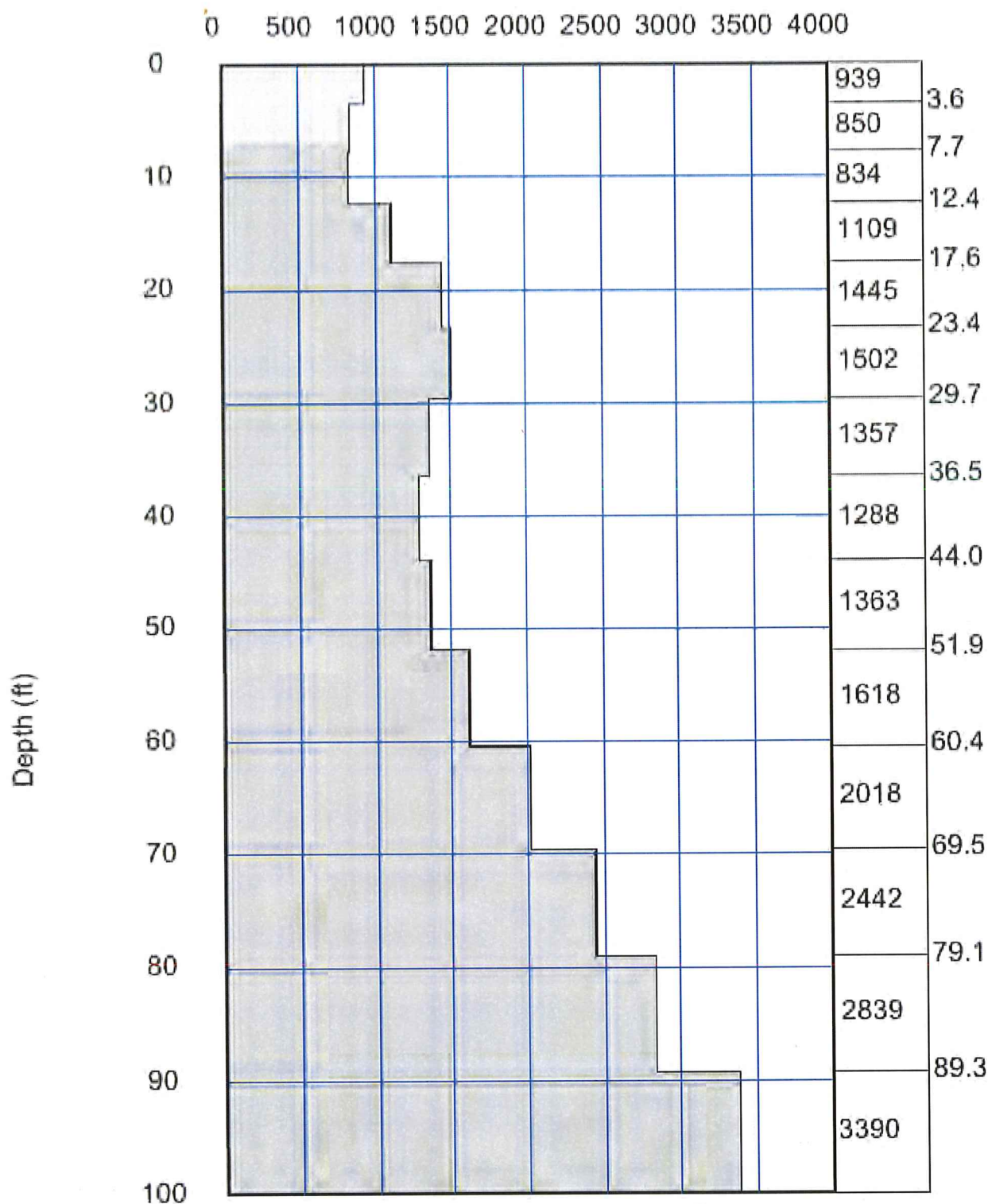
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S-wave velocity model : MAM + MASW Line 1

Average Vs 100ft = 1293.6 ft/sec

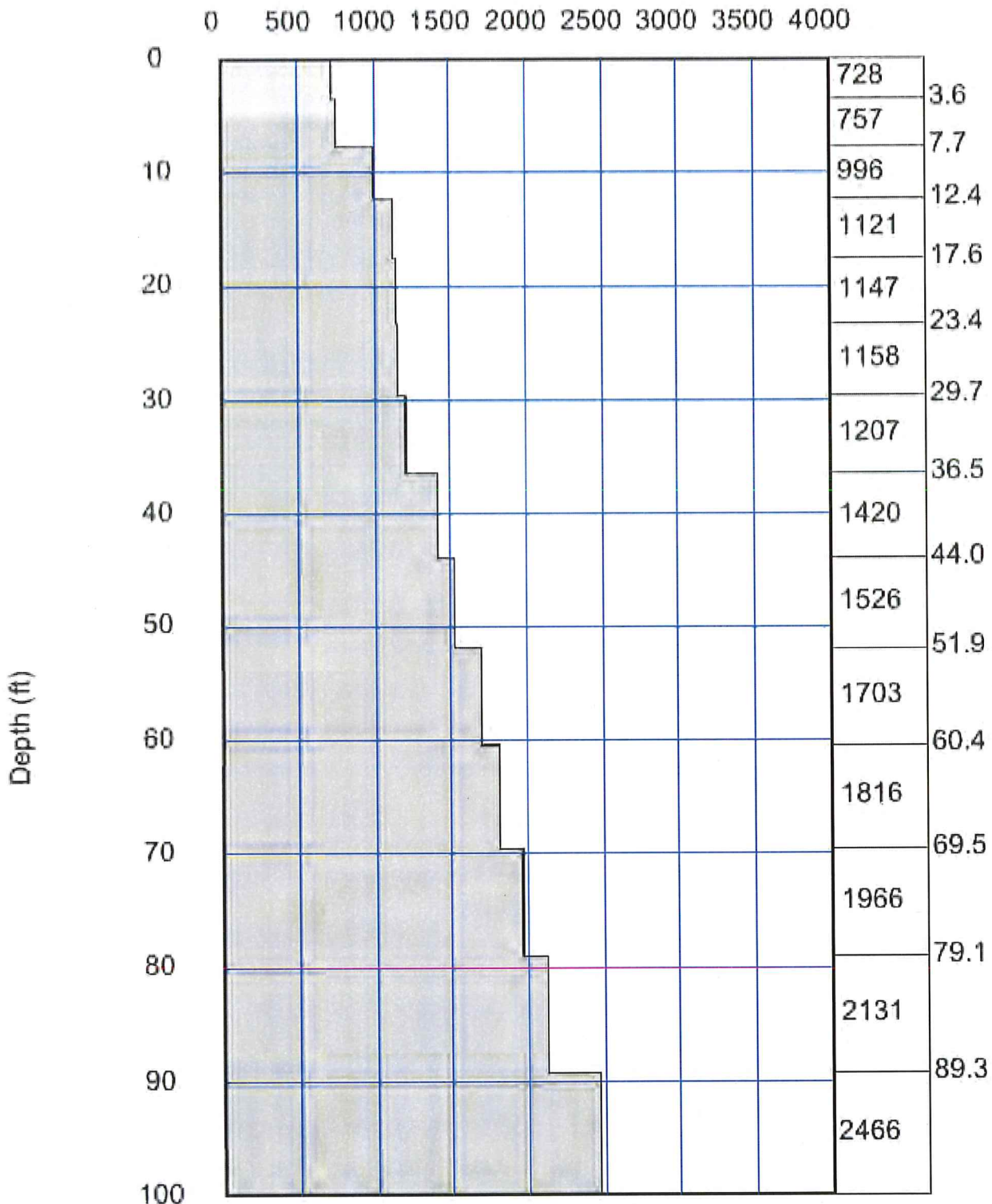
### S-wave velocity (ft/s)



S-wave velocity model : MAM +MASW Line 2.

Average Vs 100ft = 1556.9 ft/sec

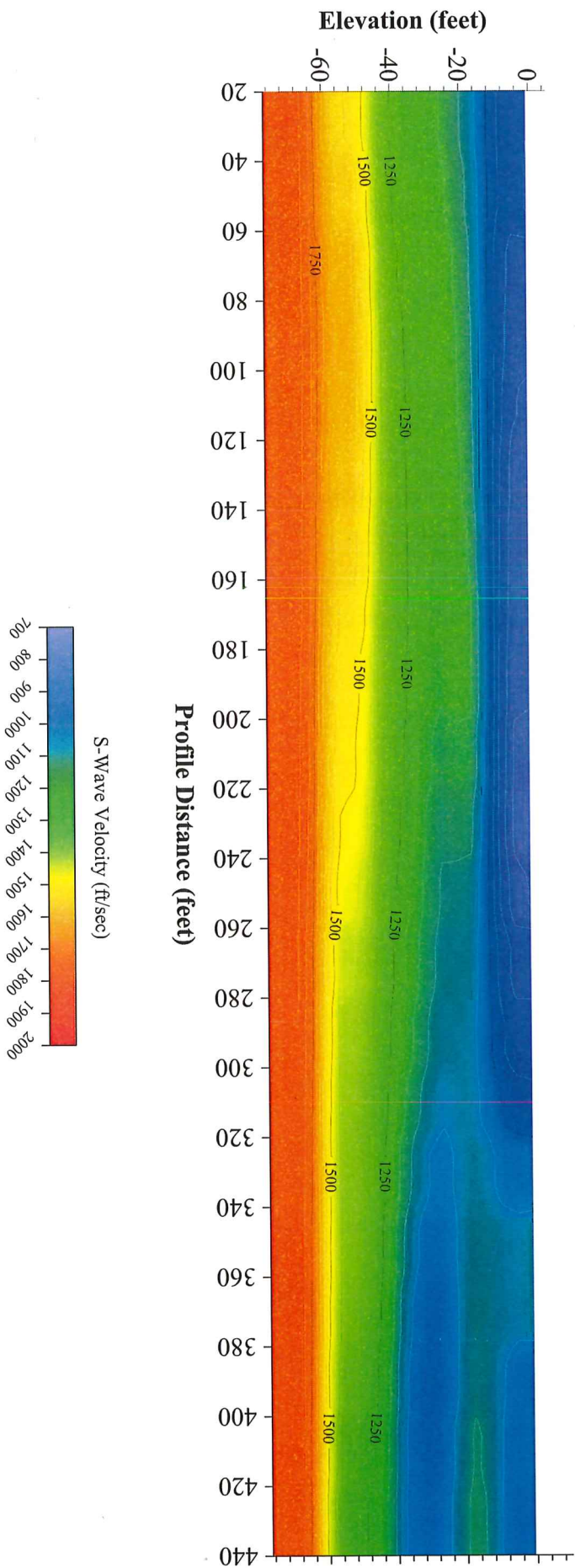
### S-wave velocity (ft/s)

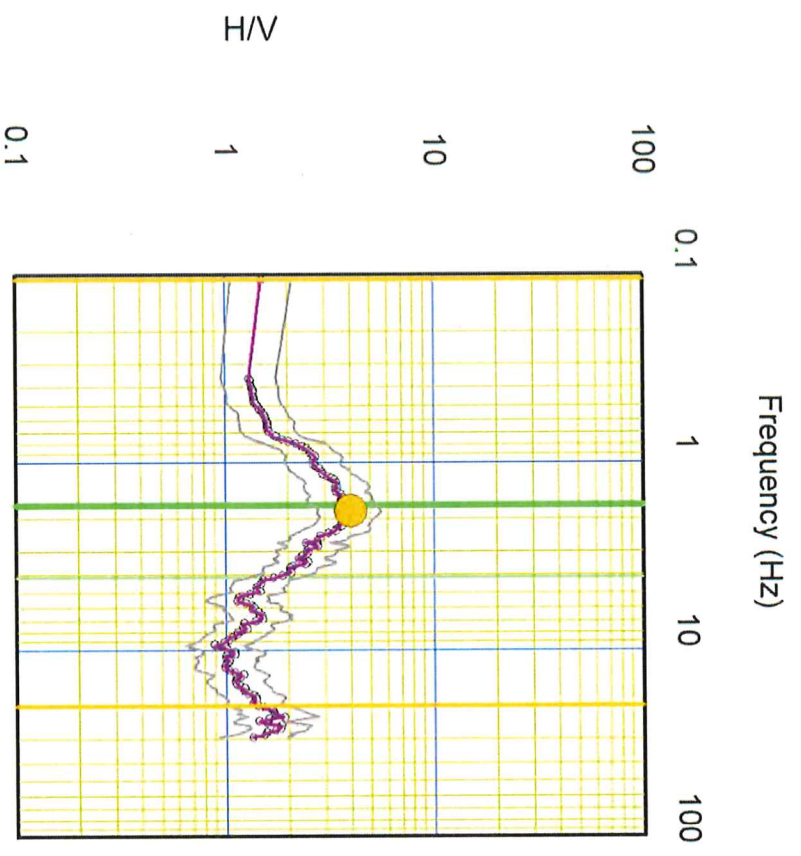


S-wave velocity model : MAM + MASW Line 3

Average Vs 100ft = 1422.8 ft/sec

# MASW Line 1





H/V spectrum : Aumsville WWTP

H/V peak frequency = 1.816 Hz

H/V peak amplitude = 3.919

H/V peak clarity = 3.464



**APPENDIX D: Previous Explorations by Others**

## SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS		TYPICAL DESCRIPTIONS
			GRAPH	LETTER	
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS <small>(LITTLE OR NO FINES)</small>		<b>GW</b>	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES
		GRAVELS WITH FINES <small>(APPRECIABLE AMOUNT OF FINES)</small>		<b>GP</b>	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES
		CLEAN SANDS <small>(LITTLE OR NO FINES)</small>		<b>GM</b>	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
		SANDS WITH FINES <small>(APPRECIABLE AMOUNT OF FINES)</small>		<b>GC</b>	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS	CLEAN SANDS <small>(LITTLE OR NO FINES)</small>		<b>SW</b>	WELL-GRADED SANDS, GRAVELLY SANDS
		SANDS WITH FINES <small>(APPRECIABLE AMOUNT OF FINES)</small>		<b>SP</b>	POORLY-GRADED SANDS, GRAVELLY SAND
FINE GRAINED SOILS	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		<b>ML</b>	INORGANIC SILTS, ROCK FLOUR, CLAYEY SILTS WITH SLIGHT PLASTICITY
		LIQUID LIMIT GREATER THAN 50		<b>CL</b>	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
		LIQUID LIMIT GREATER THAN 50		<b>OL</b>	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		<b>MH</b>	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS SILTY SOILS
		LIQUID LIMIT GREATER THAN 50		<b>CH</b>	INORGANIC CLAYS OF HIGH PLASTICITY
		LIQUID LIMIT GREATER THAN 50		<b>OH</b>	ORGANIC CLAYS AND SILTS OF MEDIUM TO HIGH PLASTICITY
HIGHLY ORGANIC SOILS				<b>PT</b>	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: Multiple symbols are used to indicate borderline or dual soil classifications

### Sampler Symbol Descriptions

	2.4-inch I.D. split barrel
	Standard Penetration Test (SPT)
	Shelby tube
	Piston
	Direct-Push
	Bulk or grab
	Continuous Coring

Blowcount is recorded for driven samplers as the number of blows required to advance sampler 12 inches (or distance noted). See exploration log for hammer weight and drop.

"P" indicates sampler pushed using the weight of the drill rig.

"WOH" indicates sampler pushed using the weight of the hammer.

NOTE: The reader must refer to the discussion in the report text and the logs of explorations for a proper understanding of subsurface conditions. Descriptions on the logs apply only at the specific exploration locations and at the time the explorations were made; they are not warranted to be representative of subsurface conditions at other locations or times.

## ADDITIONAL MATERIAL SYMBOLS

SYMBOLS		TYPICAL DESCRIPTIONS
GRAPH	LETTER	
	<b>AC</b>	Asphalt Concrete
	<b>CC</b>	Cement Concrete
	<b>CR</b>	Crushed Rock/ Quarry Spalls
	<b>SOD</b>	Sod/Forest Duff
	<b>TS</b>	Topsoil

### Groundwater Contact



Measured groundwater level in exploration, well, or piezometer



Measured free product in well or piezometer

### Graphic Log Contact



Distinct contact between soil strata



Approximate contact between soil strata

### Material Description Contact



Contact between geologic units



Contact between soil of the same geologic unit

### Laboratory / Field Tests

%F	Percent fines
%G	Percent gravel
AL	Atterberg limits
CA	Chemical analysis
CP	Laboratory compaction test
CS	Consolidation test
DD	Dry density
DS	Direct shear
HA	Hydrometer analysis
MC	Moisture content
MD	Moisture content and dry density
Mohs	Mohs hardness scale
OC	Organic content
PM	Permeability or hydraulic conductivity
PI	Plasticity index
PL	Point load test
PP	Pocket penetrometer
SA	Sieve analysis
TX	Triaxial compression
UC	Unconfined compression
VS	Vane shear

### Sheen Classification

NS	No Visible Sheen
SS	Slight Sheen
MS	Moderate Sheen
HS	Heavy Sheen

## Key to Exploration Logs

Start Drilled 7/20/2021	End 7/20/2021	Total Depth (ft)	21.5	Logged By Checked By	NWW TG	Driller Holt Services	Drilling Method	Mud Rotary	
Surface Elevation (ft) Vertical Datum		352 NAVD88		Hammer Data		Auto Hammer 140 (lbs) / (in) Drop		Drilling Equipment	Mobile B-57
Latitude Longitude		44.855732 -122.876999		System Datum		WGS84 (feet)		Groundwater not observed at time of exploration	

Notes: D&M and California values reduced using Lacroix-Horn conversion to correlate with SPT N-values.

Elevation (feet)	Depth (feet)	FIELD DATA				Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
		Interval Recovered (in)	Blows/foot	Collected Sample	Sample Name Testing						
350	0					GP	Angular basalt gravel roadway (fill)				
345	5	9	4		1	ML	Gray-brown silt with occasional gravel and sand (soft to medium stiff, moist)				
340	10	18	7		2		Grades without gravel, medium stiff				
335	15	2	39		3	GM	Gray silty gravel with cobbles (dense, moist) (Alluvium)				
	20		9		4	ML	Brown silt with occasional sand and gravel (stiff, moist)				

Note: See Figure A-1 for explanation of symbols.  
Coordinates Data Source: Horizontal approximated based on Google Earth. Vertical approximated based on Google Earth.

### Log of Boring B-1



Project: Aumsville Wastewater Treatment Plant  
Project Location: Aumsville, Oregon  
Project Number: 15356-003-00

Figure A-2  
Sheet 1 of 1

Date: 9/21/21 Path: C:\ESTES\DOCUMENTS\SHAREPOINT DRAFTS\1535600300\GP1 DBLibrary\Library\GEOENGINEERS\_DF\_STD\_US\_JUNE\_2017.GLB\GEB\_GEO TECH\_STANDARD\_MF\_NO\_GW

Start Drilled	7/20/2021	End	7/20/2021	Total Depth (ft)	31.5	Logged By	NW	Checked By	TG	Driller	Holt Services	Drilling Method	Mud Rotary
Surface Elevation (ft) Vertical Datum	352 NAVD88			Hammer Data	Auto Hammer 140 (lbs) / (in) Drop			Drilling Equipment	Mobile B-57				
Latitude	44.855301			System Datum	WGS84 (feet)			Groundwater not observed at time of exploration					
Longitude	-122.876103												

Notes: D&M and California values reduced using Lacroix-Horn conversion to correlate with SPT N-values.

Elevation (feet)	FIELD DATA					Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	Depth (feet)	Interval Recovered (in)	Blows/foot	Collected Sample	Sample Name Testing					
0						GP	Angular gravel road/pad (fill)			
350						CL	Black and dark gray clay with organic matter (very soft to soft, moist) (Alluvium)			
345	5	10	2		1 MC			34		
340	10	16	8		2 AL		Grades to gray with orange mottling, occasional sand, gravel, medium stiff to stiff	26		AL (LL = 42; PI = 21)
335	15	17	15		3 %F	SM	Gray silty sand with gravel (medium dense, moist)	27	34	
330	20		6		4	CL	Red-brown clay with occasional sand (medium stiff, moist)			
325	25	0	9		5		Becomes stiff			
	28	18	7		6		Becomes medium stiff			
	30	18	9		7		Becomes stiff			

Note: See Figure A-1 for explanation of symbols.  
Coordinates Data Source: Horizontal approximated based on Google Earth. Vertical approximated based on Google Earth.

### Log of Boring B-2



Project: Aumsville Wastewater Treatment Plant  
Project Location: Aumsville, Oregon  
Project Number: 15356-003-00

Figure A-3  
Sheet 1 of 1

Date: 9/22/21 Path: C:\USERS\CESTES\DOCUMENTS\SHAREPOINT DRAFTS\1535600300.GPJ DBLibrary\Library\GEOENGINEERS\_DF\_STD\_US\_JUNE\_2017.GLB\GEB8\_GEOTECH\_STANDARD\_%F\_NO\_SW

Start Drilled	7/19/2021	End	7/19/2021	Total Depth (ft)	41.5	Logged By	NW	Checked By	TG	Driller	Holt Services	Drilling Method	Mud Rotary
Surface Elevation (ft) Vertical Datum	352 NAVD88			Hammer Data	Auto Hammer 140 (lbs) / (in) Drop			Drilling Equipment	Mobile B-57				
Latitude	44.853777			System Datum	WGS84 (feet)			Groundwater not observed at time of exploration					
Longitude	-122.875237												

Notes: D&M and California values reduced using Lacroix-Horn conversion to correlate with SPT N-values.

Elevation (feet)	FIELD DATA					Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	Depth (feet)	Interval Recovered (in)	Blows/foot	Collected Sample	Sample Name Testing					
350	0					GP	Poorly graded fine gravel, with wood fragments (very loose to loose, moist) (fill)			
345	5	2	4		1					
340	10	18	7		2	ML	Black silt with gravel and organic matter (medium stiff, moist) (fill)			
335	15	10	50/5"		3	GP	Gray fine to coarse to fine rounded to angular gravel with sand (very dense, moist) (Alluvium)			
330	20	18	37		4		Grades to rounded gravel, dense			
325	25	18	8		5	CH	Brown clay with occasional sand (medium stiff to stiff, moist)	44		AL (LL = 57; PI = 33)
320	30	18	12		6	MD	Grades with red and gray mottling, stiff	39		Dry Density = 85 pcf



Note: See Figure A-1 for explanation of symbols.  
Coordinates Data Source: Horizontal approximated based on Google Earth. Vertical approximated based on Google Earth.

### Log of Boring B-3



Project: Aumsville Wastewater Treatment Plant  
Project Location: Aumsville, Oregon  
Project Number: 15356-003-00

Date: 9/21/21 Path: C:\Users\jester\Documents\SHAREPOINT DRAFTS\15356003000.GPJ DBLibrary\Library\GEOENGINEERS\_DP\_STD\_US\_JUNE\_2017.GLB\GEB\_GEO TECH\_STANDARD\_MF\_NO\_GW

Elevation (feet)	FIELD DATA					MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	Depth (feet)	Interval Recovered (in)	Blows/foot	Collected Sample	Sample Name Testing				
35	18	9		7					Becomes gray with mottling
40	18	9		8					

**Log of Boring B-3 (continued)**



Project: Aumsville Wastewater Treatment Plant  
 Project Location: Aumsville, Oregon  
 Project Number: 15356-003-00

Start Drilled 7/19/2021	End 7/19/2021	Total Depth (ft) 16.5	Logged By Checked By NWW TG	Driller Holt Services	Drilling Method Mud Rotary
Surface Elevation (ft) Vertical Datum	353 NAVD88	Hammer Data	Auto Hammer 140 (lbs) / (in) Drop	Drilling Equipment	Mobile B-57
Latitude Longitude	44.853811 -122.875877	System Datum	WGS84 (feet)	Groundwater not observed at time of exploration	
Notes: D&M and California values reduced using Lacroix-Horn conversion to correlate with SPT N-values.					

Elevation (feet)	FIELD DATA					Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	Depth (feet)	Interval Recovered (in)	Blows/foot	Collected Sample	Sample Name Testing					
0						GM	Gray silty gravel with trace organic matter (very loose, moist) (fill)			
350										
5	5	2	1							
345										
10	12	23	2			ML	Brown silt with gravel and occasional sand (stiff, moist) (Alluvium)			
340										
15	9	37/10"	3			GP	Rounded gravel with sand (dense, moist)			

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Note: See Figure A-1 for explanation of symbols.  
Coordinates Data Source: Horizontal approximated based on Google Earth. Vertical approximated based on Google Earth.

### Log of Boring B-4



Project: Aumsville Wastewater Treatment Plant  
Project Location: Aumsville, Oregon  
Project Number: 15356-003-00

Figure A-5  
Sheet 1 of 1

Drilled	Start 7/20/2021	End 7/20/2021	Total Depth (ft)	21.5	Logged By Checked By	NW TG	Driller	Holt Services	Drilling Method	Mud Rotary	
Surface Elevation (ft) Vertical Datum	355 NAVD88			Hammer Data	Auto Hammer 140 (lbs) / (in) Drop			Drilling Equipment	Mobile B-57		
Latitude Longitude	44.854299 -122.877006			System Datum	WGS84 (feet)			Groundwater not observed at time of exploration			

Notes: D&M and California values reduced using Lacroix-Horn conversion to correlate with SPT N-values.

Elevation (feet)	FIELD DATA					Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	Depth (feet)	Interval Recovered (in)	Blows/foot	Collected Sample	Sample Name Testing					
0						CL	Brown lean clay with occasional sand and gravel (stiff, moist) (Alluvium)			
350	5	4	12		1 AL			44		AL (LL = 49; PI = 25)
345	10	14	8		2	GP	Gray gravel with sand and trace silt (loose, moist)			
340	15	16	20		3		Becomes medium dense			
335	20		27		4					

Note: See Figure A-1 for explanation of symbols.  
Coordinates Data Source: Horizontal approximated based on Google Earth. Vertical approximated based on Google Earth.

### Log of Boring B-5



Project: Aumsville Wastewater Treatment Plant  
Project Location: Aumsville, Oregon  
Project Number: 15356-003-00

Figure A-6  
Sheet 1 of 1

Date: 9/22/21; Path: C:\USERS\CESTES\DOCUMENTS\SHAREPOINT DRAFTS\1535600300.GPJ; DBL library/Library; GEOENGINEERS\_DP\_STD\_US\_JUNE\_2017.GLB/GEIB\_GEOTECH\_STANDARD\_MF\_NO\_GW

Start Drilled	7/19/2021	End	7/20/2021	Total Depth (ft)	51.5	Logged By	NWW	Checked By	TG	Driller	Holt Services	Drilling Method	Mud Rotary
Surface Elevation (ft) Vertical Datum	355 NAVD88			Hammer Data	Auto Hammer 140 (lbs) / (in) Drop			Drilling Equipment	Mobile B-57				
Latitude	44.853986			System Datum	WGS84 (feet)			Groundwater not observed at time of exploration					
Longitude	-122.877242												

Notes: D&M and California values reduced using Lacroix-Horn conversion to correlate with SPT N-values.

Elevation (feet)	FIELD DATA					Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	Depth (feet)	Interval Recovered (in)	Blows/foot	Collected Sample	Sample Name Testing						
0							ML	Brown silt with angular to rounded gravel (stiff, moist) (fill)			
350	5	10	9		1						
345	10	11	13		2		GP	Gray gravel with sand, trace silt (medium dense, moist) (Alluvium)			
340	15	10	28		3			Becomes dense			
335	20		39/10"		4						
330	25	12	32		5						
325	30	18	9		6		CL	Brown clay with red/white mottling (stiff, moist)			
320	35										

Note: See Figure A-1 for explanation of symbols.  
Coordinates Data Source: Horizontal approximated based on Google Earth. Vertical approximated based on Google Earth.

### Log of Boring B-6



Project: Aumsville Wastewater Treatment Plant  
Project Location: Aumsville, Oregon  
Project Number: 15356-003-00

Date: 8/21/21 Path: C:\USERS\CESTES\DOCUMENTS\SHAREPOINT\DRAPTS\15356003000.GPJ DBLibrary\Library\GEOENGINEERS\_DF\_STD\_US\_JUNE\_2017.GLB\GEB\_GEO TECH\_STANDARD\_MF\_NO\_GW

Date: 9/21/21 Path: C:\USERS\CESTES\DOCUMENTS\SHAREPOINT DRAFTS\1535600300.GPJ DBLibrary\Library\GEOENGINEERS\_DF\_STD\_US\_JUNE\_2017.GLB\GEB\_GEO TECH\_STANDARD\_#E\_NO\_GW

Elevation (feet)	FIELD DATA					Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	Depth (feet)	Interval Recovered (in)	Blows/foot	Collected Sample	Sample Name Testing					
35	18	13		7			Becomes reddish with brown mottling			
315	40	0	15		8					
310	45	18	15		9	ML	Dark black/brown silt (stiff, moist)			
305	50	18	10		10		Becomes lighter brown and gray mottled			

**Log of Boring B-6 (continued)**



Project: Aumsville Wastewater Treatment Plant  
 Project Location: Aumsville, Oregon  
 Project Number: 15356-003-00