Geotechnical Investigation and Seismic Hazard Study

East 40th Avenue Storage Tanks

Eugene, Oregon

Prepared for:

Eugene Water & Electric Board
Eugene, Oregon

July 2, 2021

Foundation Engineering, Inc.

*Professional
Geotechnical
Services*

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Laura Farthing, P.E.
Senior Engineer – Water
Eugene Water & Electric Board
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Eugene, Oregon 97440

East 40th Avenue Storage Tanks
Geotechnical Investigation and Seismic Hazard Study
Eugene, Oregon

Project No.: 2201086

Dear Ms. Farthing:

We have completed the requested geotechnical investigation and seismic hazard study for the above-referenced project. Our report includes a description of our work, a discussion of the site conditions, a summary of laboratory testing, and a discussion of engineering analyses. Recommendations for site preparation and foundation design and construction are also provided.

A seismic hazard study was also completed to identify potential geologic and seismic hazards and evaluate the effect those hazards may have on the proposed site. The study fulfills the requirements presented in the 2019 Oregon Structural Specialty Code (OSSC 2019) for site-specific seismic hazard reports for essential and hazardous facilities, and major and special-occupancy structures. The 2019 OSSC is based on the 2018 International Building Code and ASCE 7-16. Results of the study (provided in Appendix D) indicate there are no geologic or seismic hazards that require special design consideration or would preclude construction of the proposed reservoir. The study was completed by Brooke Running, R.G., C.E.G.

There are numerous values in geotechnical investigations that are approximate including calculated parameters, measured lengths, soil layer depths, elevations, and strength measurements. For brevity, the symbol "±" is used throughout this report to represent the words approximate or approximately when discussing these values.

It has been a pleasure assisting you with this phase of your project. Please do not hesitate to contact us if you have any questions or if you require further assistance.

Sincerely,

FOUNDATION ENGINEERING, INC.

Mallory McAdams

Mallory L. McAdams, E.I.T.
Geotechnical Staff

David L. Running

David L. Running, P.E., G.E.
Senior Geotechnical Engineer

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GEOTECHNICAL INVESTIGATION
AND SEISMIC HAZARD STUDY

EAST 40TH AVENUE STORAGE TANKS
EUGENE, OREGON

BACKGROUND

The Eugene Water & Electric Board (EWEB) is planning to construct two new reservoir tanks on a currently undeveloped property located at the south terminus of Patterson Street, south of E 40th Avenue in Eugene, Oregon. The site location is shown on Figure 1A (Appendix A). The proposed site layout is shown on Figure 2A (Appendix A). The new 7.5-million-gallon tanks will have a diameter of 210 feet and will extend up to 35 feet below the current grades with a bottom of tank elevation of El. 577.

A preliminary investigation of the site was conducted by Branch Engineering. That investigation included five borings advanced using a track-mounted air-rotary drill rig. The drilling was able to confirm the presence of bedrock, but the use of air-rotary precluded the ability to obtain rock core samples. Therefore, additional exploratory drilling was required to provide more detailed information for design and construction.

EWEB is the project owner and Murraysmith is the lead design consultant. EWEB retained Foundation Engineering, Inc. to conduct a geotechnical investigation for the project. Our original scope of work was outlined in a proposal dated October 1, 2020, and authorized by Personal Services Contract # 20-200-Q.

We completed an initial field investigation in November 2020. The geotechnical investigation included exploratory drilling and laboratory testing. The findings were summarized in a draft report dated March 12, 2021, to assist EWEB with their preliminary planning and selection of the tank locations.

Subsequent to our initial work, EWEB requested additional exploratory drilling be completed near the location of the west tank to provide supplemental subsurface information and to help determine the transition between the softer and harder bedrock encountered in the previous explorations. Our scope of work for the additional drilling was outlined in a proposal dated April 28, 2021, and authorized by a signed amendment. The additional field investigation was completed in May 2021. The findings of the additional field explorations and laboratory testing are described in this report. This report also includes analyses and design and construction recommendations.

LOCAL GEOLOGY

Detailed discussions of the local and regional geology, tectonic setting, local faulting, historical seismicity, seismic hazards, and design earthquakes are included in the Site-specific Seismic Hazard Study report (Appendix D). References cited in this section are found in Appendix D. An abbreviated discussion of the local geology is provided below.

The project site is located within the southern Willamette Valley, ±3 miles south of the Willamette River in South Eugene. Local geologic mapping indicates the project site is underlain by bedrock of the Fisher Formation (Yeats et al., 1996; Madin and Murray, 2006; McClaughry et al., 2010). The Fisher Formation consists of volcaniclastic sedimentary rock and tuff with interfingering andesitic to basaltic flows. The rock can be deeply weathered or hydrothermally altered (Walker and Duncan, 1989; Yeats et al., 1996; Madin and Murray, 2006).

The subsurface conditions encountered in our explorations are consistent with the mapped local geology. The bedrock encountered within the explorations are interpreted to be the Fisher Formation based on the local geologic mapping. Details of the subsurface conditions are provided in the Subsurface Conditions section below and in the exploration logs (Appendix B).

FIELD EXPLORATION

We drilled seven exploratory borings (BH-1 through BH-7) at the site between November 9 and November 15, 2020. BH-1 through BH-3 were drilled near the proposed east tank, BH-5 through BH-7 were drilled near the proposed west tank, and BH-4 was drilled between the two tanks. The explorations extended to depths of ±30 to 52 feet.

We returned to the site on May 12 and May 13, 2021, and completed three additional borings (BH-8 through BH-10). The additional explorations were completed within the footprint of the proposed west tank. Those borings extended to depths of ±40 to 47 feet.

The individual drilling depths were selected to extend below the planned bottom of tank elevations. The boring locations are shown on Figure 2A. The ground elevations at the boring locations were estimated based on the topographic survey contours.

The borings were drilled using a CME-55 track-mounted drill rig with mud-rotary drilling and HQ-sized wire-line coring methods. Soil samples were obtained at 2½-foot intervals using a split-spoon sampler in conjunction with the Standard Penetration Test (SPT). The SPT provides an indication of the relative stiffness or density of the soil. Continuous, HQ-wire line rock coring was completed once coreable rock was encountered.

The borings were continually logged during drilling. The final logs (Appendix B) were prepared based on a review of the field logs and the results of the laboratory testing, and an examination of the soil and rock samples in our office. Upon completion of drilling, the boreholes were backfilled with bentonite chips and bentonite grout, in accordance with Oregon Water Resources Department (OWRD) guidelines.

LABORATORY TESTING

The laboratory testing included moisture content, percent fines, and Atterberg Limits tests to help classify the soils according to the Unified Soil Classification System (USCS) and estimate their overall engineering properties. Non-tested samples were visually classified in accordance with ASTM D 2487 and ASTM D 2488. USCS symbols shown on the boring logs for untested samples should be considered approximations. The test results are summarized in Table 1C (Appendix C). The moisture contents are also shown on the boring logs (Appendix B).

Twenty (20) unconfined compression (q_u) tests were completed on rock core samples to evaluate the bedrock strength. Seven tests were conducted with continuous stress-strain measurements to evaluate the elastic properties of the rock in addition to the peak q_u values. The other thirteen tests focused on the maximum q_u values only. The stress-strain curves are plotted on Figures 1C through 7C (Appendix C) and the q_u values for each of the tests and core sample information are summarized in Table 2C (Appendix C). The test results indicate unconfined compressive strengths ranging from ±8,216 to 26,388 psi, consistent with strong (R4) to very strong (R5) rock.

The test on CS-9-2 from ±13.8 to 14.3 feet indicated a q_u of 4,999 psi, which corresponds to medium strong (R3) rock. However, our field identification, based on the methods outlined in the ODOT Soil and Rock Manual (1987), indicates the rock is typically strong to very strong (R4 to R5). It was noted the test sample failed along existing jointing. Therefore, we believe the q_u test results for this sample underestimated the rock strength.

SUMMARY OF SURFACE AND SUBSURFACE CONDITIONS

Surface Conditions

The site is located on the northern slope of an undeveloped, 10-acre parcel south of E 40th Avenue. Survey information provided by EWEB indicates the crest of the hill lies at ±El. 620. At the planned tank locations, the ground surface slopes down to the northeast with ±5:1(H:V) to ±10:1(H:V) slopes. South of the planned tank locations the ground surface slopes more steeply down to the southwest with slopes as steep as ±2.5:1(H:V). The ground surface is predominately covered in grass and several large trees. A meadow occupies the northern extent of the site.

Subsurface Conditions

We developed a series of cross-sections across the site utilizing topographic data provided by EWEB and subsurface information from the borings. The cross-section locations are shown on Figure 2A. The cross-sections, shown on Figures 3A through 7A (Appendix A), indicate the site is underlain by a thin mantle of topsoil followed by residual soil (i.e., bedrock that has decomposed in place to the consistency of soil) and bedrock of the Fisher Formation. The topsoil consists of soft to stiff sandy silt. The residual soil typically includes medium dense to very dense silty sand with rock fragments, medium dense to very dense, silty sandy gravel, and hard clayey silt with rock fragments. The exception was BH-8, which encountered residual soil consisting predominately of stiff to hard silt with some sand and cobbles.

Bedrock was encountered at depths of ±0.5 to 11.5 feet in most of the borings. The exceptions were BH 7, which encountered bedrock at ±32.5 feet, and BH 10, which encountered bedrock at ±17.5 feet. The estimated ground surface elevations, exploration depths, and bedrock elevations for each of the borings are shown on the boring logs and cross-sections. The data is also summarized in Table 1B (Appendix B).

The bedrock is predominantly comprised of medium strong to very strong (R3 to R5) basalt. Extremely weak to very weak (R0 to R1) silty sandstone was encountered above the basalt in BH 3. Very weak to weak (R1 to R2) tuff underlies the basalt in BH-6 at ±38 feet. In BH 7, very weak (R1) sandy siltstone was encountered below the residual soil from ±32.5 to 38 feet, followed by very weak (R1) tuff to ±39.9 feet and weak (R2) basalt breccia to ±44.6 feet. The basalt breccia in BH 7 is underlain by strong to very strong (R4 to R5) basalt to the bottom of the boring. Very weak (R1) tuff was encountered below the residual soil in BH-8 from ±11.5 to 34.2 feet, followed by very weak (R1) sandstone from ±34.2 to 43.8 feet, and very weak (R1) tuff from ±43.8 feet to the bottom of the boring (i.e., ±47 feet). In BH-9, very weak to weak (R1 to R2) tuff is interbedded in the basalt below ±35 feet. In BH-10, the residual soil is underlain by extremely weak (R0) tuff from ±17.5 to 26.8 feet, followed by very weak to weak (R1 to R2) silty sandstone from ±26.8 to 29.3 feet, and extremely weak to very weak (R0 to R1) tuff from ±29.3 feet to the bottom of the boring (i.e., ±40 feet).

The joint spacing ranges from very close to moderately close. Rock Quality Designation (RQD) values vary with location and depth and range from 0 to 100%. The overall average RQD value is ±46.5%. An information sheet is included in Appendix B providing descriptions of the weathering, rock hardness, jointing, and RQD criteria used in our evaluation of the bedrock. Photos of the rock core are shown in Photos 1B through 39B (Appendix B).

Ground Water

Mud-rotary drilling techniques precluded measurement of ground water levels in the borings during drilling. Based on the subsurface conditions, we anticipate water perches on the shallow bedrock in the wet, winter months. The perched water may disappear in the dry, summer months.

DISCUSSION

Rock excavation will be the key geotechnical consideration. We understand the planned finish floor (FF) elevation is El. 577 for both tanks. The excavations for the tanks will extend 5 feet below the FF elevation (i.e., to El. 572) to provide room beneath the floor slab for utilities, a granular leveling layer, a leak detection layer, and a foundation drain layer.

Surface elevations at the planned tank locations range from ±El. 584 to El. 606. Therefore, excavation depths ranging from ±12 to 34 feet will be required. Based on the subsurface information from our boings, we anticipate excavations will extend ±0.5 to 24 feet below the bedrock surface at the west tank location and ±14 to 26 feet below the bedrock surface at the east tank location.

The unconfined compression (q_u) test results indicate q_u values ranging from ±4,999 to 26,357 psi, with an average of ±18,933 psi. The joint spacing in the bedrock ranges from very close (i.e., less than 2 inches) to moderately close (i.e., 1 to 3 feet).

Based on the rock hardness and the joint spacing, we anticipate it will not be practical to excavate the rock by digging with an excavator bucket alone. We believe it will be necessary to fracture the rock prior to excavating. Potential rock fracturing methods include hammering the rock with a hydraulic ram, drilling and splitting, or controlled blasting. Considering the overall volume of the bedrock to removed, we believe controlled blasting will be the most practical method for breaking up the bedrock.

The excavations will extend through topsoil and residual soil and into the underlying bedrock. The topsoil and residual soil include soft to stiff sandy silt, stiff to hard silt, and medium dense to very dense silty sand. We anticipate the topsoil will correspond to an OR-OSHA Class B soil due to its undrained shear strength and plasticity.

Much of the residual soil is described on the boring logs as sandy soil, which is typically categorized as an OR-OSHA Class C soil. However, we anticipate the residual soil will correspond most closely to an OR-OSHA Class B soil due to its stiffness/density and relict bedrock texture. OR-OSHA recommends a maximum temporary cut slope of 1:1(H:V) in Class B soil. Suitable cut slopes in these soils will have to be confirmed in the field at the time of construction. The soil may degrade when exposed to rainfall. Therefore, during wet weather, the cut slopes should be covered in plastic sheeting and inspected daily.

The bedrock will likely satisfy the OR-OSHA criterion for stable rock where it is not disturbed. OR-OSHA allows vertical cuts in stable rock. The site layout and subsurface conditions should provide sufficient room to grade the temporary cut slopes to OR-OSHA standards without the need for shoring.

ENGINEERING ANALYSIS

Seismic Design

A detailed seismic hazard study was completed for the site and the findings are summarized in Appendix D. The study concluded there are no seismic hazards that would preclude construction of the proposed reservoir tanks, provided the earthwork is completed as recommended herein.

Site Response Spectra. We developed site response spectra for the site in accordance with the AWWA D110-13 (R18) Section 4.3. The AWWA D110-13 site response is separated into components with an impulsive component representing the structure with 5% damping and a convective component with 0.5% damping representing the fluid contents.

Based on the interpreted cross-sections, we anticipate the tanks will be underlain by medium strong to very strong (R3 to R5) basalt or by weak (R2) basalt breccia or very weak to weak (R1 to R2) tuff or sandy siltstone followed by medium strong to very strong (R3 to R5) basalt. We have concluded the subsurface conditions correspond most closely to an AWWA Site Class B.

AWWA D110-13 references ASCE 7-05 for seismic design. Seismic design in ASCE 7-05 utilizes USGS 2002 seismic maps. For our evaluation of the tank site, we used the updated USGS 2014 maps referenced in ASCE 7-16 and OSSC 2019 to provide the spectral accelerations consistent with the current building codes. Risk-targeted maximum considered earthquake (MCE_R) ground motions on bedrock were obtained using modified USGS 2014 maps with 2% probability of exceedance in 50 years (i.e., a ±2,475-year return period). The modifications include factors to adjust the spectral accelerations to account for directivity and risk. Murraysmith also requested maximum considered earthquake (MCE) ground motions for a 10% probability of exceedance in 50 years (i.e., a ±475-year return period). Spectral accelerations for this return period were obtained from the USGS interactive deaggregation website (USGS, 2014) using maps which include modification for directivity.

To develop the site response spectra, spectral accelerations at the ground surface are adjusted using F_a and F_v values selected from ASCE 7-16 Tables 11-4-1 and 11-4-2. ASCE 7-16 stipulates F_a and F_v values be taken as 1.0 for rock conditions consistent with a Site Class B, where site-specific velocity measurements are not completed.

The AWWA D110-13 site response spectra for impulsive and convective components with MCE_R ground motions with 2% probability of exceedance in 50 years are shown on Figure 8A (Appendix A). The site response spectra with MCE ground motions with 10% probability of exceedance in 50 years are shown on Figure 9A (Appendix A).

Vertical Accelerations. Vertical accelerations may be analyzed based on AWWA D110-13 Section 4.5 and Equation 4-36, with a B coefficient taken as 2/3. The coefficient of vertical acceleration (C_v) may be calculated from Equations 4-37 and 4-38 using spectral accelerations (S_{DS} and S_{D1}) from Figures 8A and 9A.

Liquefaction. Liquefiable soils typically consist of saturated, loose sands and non-plastic or low plasticity silt (i.e., a PI of less than 8). The site is underlain by medium dense to very dense residual soil followed by relatively shallow bedrock. These materials are not susceptible to liquefaction. Therefore, there is no liquefaction hazard at the site.

Bearing Capacity and Settlement

We anticipate the new tank will have a concrete floor and a ring footing supporting the perimeter wall. Interior column footings may also be required. We anticipate the east tank foundations will bear on compacted crushed rock underlain by bedrock consisting of predominately medium strong to very strong (R3 to R5) basalt. We anticipate the west tank foundations will bear on compacted crushed rock underlain by medium strong to very strong (R3 to R5) basalt or very weak to weak (R1 to R2) sandstone, sandy siltstone, or tuff.

We recommend assuming a conservative allowable bearing pressure of 30 ksf for foundation design. The allowable bearing pressure may be increased by one-third for the evaluation of transient loads (i.e., seismic and wind).

The foundation excavations will extend ±12 to 34 feet below the original ground surface and the excavated material (which will be mostly bedrock) will be replaced with predominantly water. Therefore, the net increase in vertical pressure on the foundation subgrade (if any) will be small. Most of the excavations will terminate in medium strong to very strong (R3 to R5) basalt with very weak to weak (R1 to R2) sandy siltstone, sandstone, or tuff in some areas. If the sedimentary rock exposed at the subgrade elevation is determined to be appreciably more compressible than the basalt, overexcavation and replacement of the softer material with additional compacted crushed rock may be required to reduce the risk of differential settlement. The need for overexcavation (if any) should be made by a Foundation Engineering representative.

The bedrock is elastic material that may rebound when unloaded and compress when reloaded. Based on the loading conditions and the low compressibility of the bedrock and compacted crushed rock below the foundations, we anticipate the total foundation settlement will be less than ½ inch and the differential settlement will be ¼-inch or less in 50 feet. We anticipate the settlement will occur during the construction and initial filling of the tanks.

Sliding Coefficient and Passive Resistance for Footings

The footings and slab will bear on a leveling course of compacted crushed rock. For sliding analysis, we recommend using a coefficient of friction of 0.5 between the base of the concrete and the crushed rock.

Passive resistance of the backfill in front of the buried footings was calculated as an equivalent fluid density equal to γ^*K_p, where γ is the unit weight of the backfill and K_p is the passive earth pressure coefficient. We anticipate the footings will be backfilled with compacted Select Fill surrounded by bedrock. For these conditions, we calculated the passive pressure on the footings assuming a soil unit weight (γ) of 130 pcf an internal friction angle of (ϕ) of 36 degrees. The calculations indicate the ultimate passive resistance may be modeled using an equivalent fluid density of ±500 pcf.

The passive resistance may be combined with the sliding resistance at the base of the footings to evaluate the overall lateral resistance, however, the sliding and ultimate passive resistances will develop with different displacements. The sliding resistance will develop with very small transitional movement. Development of the ultimate passive resistance on the footings may require a lateral displacement corresponding to 1% of the buried footing height, assuming dense, compacted crushed rock backfill.

Lateral Earth Pressures for Buried Walls

Lateral earth pressures will be imparted on the buried tank walls from the backfill. We assume the backfill will consist of compacted Select Fill extending a minimum of 10 feet beyond the tank wall, surrounded by compacted, native backfill. To calculate lateral earth pressure on the buried walls, we assumed a γ of 130 pcf, ϕ of 36 degrees, and a wall friction angle (δ) of 22 degrees. Both static and seismic loading conditions were analyzed, as discussed below.

<u>Static Loading</u>. For load combinations where static loading is evaluated, the wall deflection may not be sufficient to fully mobilize active earth pressure conditions. Therefore, we recommend designing the walls using at-rest earth pressures. The static lateral earth pressure on the walls may be calculated as $k_o{*}\gamma$, assuming an at-rest earth pressure coefficient (k_o) of 0.41 and a γ of 130 pcf. This corresponds to an equivalent fluid density of 53 pcf. The resultant of the at-rest pressure acts at H/3 above the base of the wall, where H is the buried height of the wall.

Seismic Loading. For load combinations where seismic loading is considered, it is customary to assume the wall deflection will be sufficient to mobilize active earth pressures. A study of seismic earth pressures on deep building basements (Lew et al. 2010) concluded, total dynamic earth pressure on buried walls may be modeled as triangular distribution calculated as $k_{ae}*\gamma$. The total dynamic earth pressure ($k_{ae}*\gamma$) may be divided into a static active earth pressure component ($k_a*\gamma$) and a seismic thrust component ($\Delta k_{ae}*\gamma$), where: $\Delta k_{ae} = k_{ae} - k_a$. The resultants of both the static and seismic thrust components act at H/3 above the base of the wall.

We completed Generalized Limit Equilibrium (GLE) analysis using Slide 5.0 software to back-calculate $k_{ae}*\gamma$. A pseudo-static horizontal acceleration coefficient (k_h) of 0.3g was assumed for the analysis based on the USGS Maximum Credible Earthquake (MCE) peak ground acceleration and a Site Class B. No reduction in k_h was assumed for displacement. We believe this is a conservative assumption. The failure surface was assumed to extend through the wall backfill. A horizontal line load was applied at the wall location at a height of H/3 above the base of the wall to represent the lateral resistance provided by the wall. The line load corresponding to a FS of 1.0 was used to back-calculate $k_{ae}*\gamma$.

The results of the GLE analysis indicate the total dynamic earth pressure can be modeled using an equivalent fluid density of 49 pcf, which corresponds to a k_{ae} of 0.38, assuming a γ of 130 pcf for the wall backfill. The total dynamic earth pressure may be divided into a static active earth pressure component ($k_a*\gamma$) modeled using a k_a of 0.24 and an equivalent fluid density of 31 pcf and a seismic thrust component modeled using a Δk_{ae} of 0.14 and an equivalent fluid density of 18 pcf. Table 1 summarizes the recommended lateral earth pressures for static and seismic design.

Table 1. Lateral Earth Pressure Parameters for Buried Walls

Parameter	Source	Value
At-Rest Earth Pressure Coefficient, k_o	1-sinϕ	0.41
At-Rest Equivalent Fluid Density (Static Design)	$k_o*\gamma_{backfill}$	53 pcf
Total Dynamic Earth Pressure Coefficient, k_{ae}	GLE Analysis	0.38
Active Earth Pressure Coefficient, k_a	$\tan^2(45 - \phi/2)$	0.24
Active Equivalent Fluid Density (Static Component of Total Dynamic Earth Pressure)	$k_a*\gamma_{backfill}$	31 pcf
Seismic Thrust Earth Pressure Coefficient, Δk_{ae}	$\Delta k_{ae} = k_{ae} - k_a$	0.14
Seismic Thrust Equivalent Fluid Density (Seismic Component of Total Dynamic Earth Pressure)	$\Delta k_{ae}*\gamma_{backfill}$	18 pcf

RECOMMENDATIONS

Design and construction recommendations are provided in the following sections. We recommend contractors be provided a copy of this report to review the site conditions and recommendations for site preparation and foundation construction.

General Earthwork

1. Select Fill as defined in this report should consist of ¾ or 1-inch minus, clean (i.e., less than 5% passing the #200 U.S. Sieve), well-graded, angular crushed rock. We should be provided a gradation sheet for this material for approval prior to delivery to the site.

2. Granular Site Fill should consist of approved soil and rock taken from on-site excavations that are free of construction debris, organics, or other deleterious materials. This material may be used for general site grading outside foundation areas and as backfill around the tank beginning 10 feet (measured horizontally) from the tank perimeter. Rock fragments in the fill should be limited to a maximum diameter of 6 inches. The suitability of Site Fill for reuse should be confirmed by a Foundation Engineering representative at the time of construction.

3. Drain Rock should consist of ¾ to 1½-inch, clean (less than 2% passing the #200 sieve), open-graded, angular, crushed quarry rock. Other gradations may be acceptable, provided the rock is durable and free draining. We should be provided a gradation sheet for this material for approval prior to delivery to the site.

4. Subsurface Drainage Geotextile should be a non-woven geotextile with Mean Average Roll Value (MARV) strength properties meeting the requirements of an AASHTO M 288-17 Class 3 geotextile (Subsurface Drainage Geotextile), with a maximum AOS of 0.3 mm (max average roll value) and a permittivity greater than 0.1 sec^{-1}. We should be provided a specification sheet on the selected geotextile for approval prior to delivery to the site.

5. Compact all fill in loose lifts not exceeding 12 inches. The lift thickness should be reduced to 6 inches where light or hand-operated equipment is used. Compact all fill to a minimum of 95% relative compaction. The maximum dry density of ASTM D 698 should be used as the standard for estimating relative compaction. The moisture content of the fill should be adjusted to within ±2% of its optimum value prior to compaction.

Field density tests should be run frequently to confirm adequate compaction of the fill. Compaction of granular fill that contains open-graded rock or aggregate too coarse for density testing should be evaluated by observation of the compaction method and by proof-rolling, where practical, using a loaded 10-yd^3 dump truck or other heavy construction vehicle approved by Foundation Engineering. Areas of pumping or deflection observed beneath the truck wheels may be reworked or overexcavated and replaced with compacted Select Fill and proof-rolled again.

Foundation Design and Construction

6. Design the tank using the seismic design parameters and response spectrum shown on Figures 8A and 9A and the lateral earth pressures in Table 1.

7. Design the footings using an allowable bearing pressure of 30 ksf. This value may be increased by one-third for transient loads. The allowable bearing pressure assumes the footings will bear on bedrock or on compacted Select Fill underlain by bedrock. Assume the total foundation settlement will be less than ½ inch and the differential settlement will be ¼ inch or less in 50 feet.

8. Use a coefficient of sliding friction of 0.5 between the bottom of the footings and slab and the compacted Select Fill. Calculate the ultimate passive resistance for the buried tank footings using an equivalent fluid density of 500 pcf. Assume it may require a lateral displacement of 1% of the buried footing height to mobilize the ultimate passive resistance.

9. Use a modulus of subgrade reaction, k_s of 400 pci, for floor slab design. This value assumes the floor slab will be constructed on compacted Select Fill underlain by bedrock.

Foundation Preparation

10. Use controlled blasting to fracture the bedrock within the tank footprints. If current construction will be limited to one tank, blasted rock within the future tank footprint may be left in place. The design and sequencing of the blasting should be provided by a qualified blasting contractor.

11. Excavate the tank footprint to the planned finish subgrade elevation. We understand the excavation will extend at least 10 feet beyond the tank perimeter to provide room for construction. Remove all loose rock and debris exposed at the subgrade level prior to backfilling. Siltstone, sandstone, or tuff encountered at the subgrade level that is determined to be appreciably more compressible than the basaltic bedrock encountered elsewhere in the foundation footprint should be

overexcavated and replaced with compacted Select Fill to reduce the risk of differential settlement. The need for overexcavation and replacement (if any) should be evaluated by a Foundation Engineering representative. Use Select Fill to backfill any low-lying areas left following the excavation of the foundation area. Place and compact the Select Fill in lifts as recommended in Item 5.

12. Install a perimeter foundation drain and place Drain Rock to construct the foundation drain layer and tank leak detection layer beneath the tank as shown on the plans. Compact the Drain rock until it is visibly dense and unyielding. The adequacy of the compaction should be verified by a Foundation Engineering representative.

13. Cap the tank leak detection layer with a minimum of 6 inches of compacted Select Fill to provide a leveling layer beneath the footings and floor slab. Place and compact the Select Fill in lifts as recommended in Item 5.

Excavations/Shoring/Dewatering/Backfilling/Site Grading

14. Excavations should be shored or sloped in accordance with OR-OSHA requirements to protect workers. The excavations around the perimeter of the existing tank are expected to encounter topsoil and/or residual soil underlain by basalt, sandstone, siltstone, or tuff.

    The topsoil and residual soil include soft to stiff sandy silt, stiff to hard silt, and medium dense to very dense silty sand. We anticipate the topsoil will correspond to an OR-OSHA Class B soil due to its undrained shear strength and plasticity. We anticipate the residual soil will also correspond most closely to an OR-OSHA Class B soil due to its stiffness/density and relict bedrock texture. OR-OSHA recommends a maximum temporary cut slope of 1:1(H:V) in Class B soil. Suitable cut slopes will have to be confirmed in the field by a Foundation Engineering representative at the time of construction. The soil may degrade when exposed to rainfall. Therefore, during wet weather, the cut slopes should be covered in plastic sheeting and inspected daily.

    The bedrock will likely satisfy the OR-OSHA criterion for stable rock, where it is not disturbed. OR-OSHA allows vertical cuts in stable rock. The configuration of suitable rock cut slopes will need to be confirmed at the time of construction. Loose material should be scaled from the cut slopes, as needed, to protect workers from falling rock.

15. The bedrock underlying the proposed tank is typically medium strong (R3) to very strong (R5), very close to moderately close-jointed, and slightly weathered. Based on the rock hardness and the joint spacing, it should be assumed it will not be practical to excavate the rock by digging with an excavator bucket alone, and it will be necessary to fracture the rock prior to excavating. The contractor should select the appropriate rock excavation method. The laboratory testing completed to date on rock core samples indicates q_u values ranging from ±4,999 to 26,388 psi. However, harder rock may be encountered.

16. Water is likely to perch above the bedrock during wet weather. Therefore, the need for dewatering should be anticipated if the work is completed in the wet winter or spring months.

17. Use Select Fill to backfill around the tank within ±10 feet of the walls. Granular Site Fill may be used to backfill outside this zone. Compact the backfill as recommended in Item 5.

18. Grade the finished ground surface surrounding the tanks to promote positive drainage away from the foundations. Finished cut and fill slopes in soil should be graded at 2:1(H:V) or flatter. Slopes constructed using fill comprised of angular basalt fragments may be graded at 1.5:1(H:V).

19. Finished cut slopes in sedimentary rock (sandstone, siltstone, or tuff) should be graded at 1.5:1(H:V) or flatter. Finished cut slopes in basalt may be graded at 1:1(H:V). Cut slopes in bedrock will likely ravel over time due to drying or wetting cycles or frost. Therefore, it should be assumed some future maintenance will be required for these slopes to remove raveled material.

20. Soil that is left exposed on slopes will also be susceptible to raveling and erosion. Therefore, following construction, all exposed ground surfaces should be vegetated as soon as practical so that a mature vegetation cover is in place prior to the onset of wet weather. Residual soil and bedrock exposed in the cut slopes may be relatively sterile for growing vegetation. Therefore, it may be necessary to dress the finished surfaces with topsoil (if practical) or use an appropriate fertilizer and erosion control blanket to help establish vegetation. We assume specific recommendations of the type of vegetation will be provided by others.

Foundation Drainage

Water from surface runoff will collect within the granular backfill around the perimeter of the tank and beneath the tank, which may result in hydrostatic pressure on the floor slab and sidewalls. A perimeter drain is recommended to remove the perched water in the event the tank needs to be drained. The perimeter drain should be constructed as described below:

21. Install a foundation drain along the perimeter of the tank. The drain should consist of a 6-inch diameter, perforated HDPE or PVC pipe. The flowline of the pipe should be set near the bottom of the excavation. The pipe should be bedded in at least 4 inches of Drain Rock and backfilled to within 6 inches of the ground surface with Drain Rock. The mass of Drain Rock should be wrapped in a Subsurface Drainage Geotextile that laps at least 12 inches at the top.

22. Provide clean-outs at appropriate locations for future maintenance of the drainage system.

23. Discharge the water from the drain system away from the tank in a manner that will not cause local erosion or ponding at the outlet of the drainpipe.

DESIGN REVIEW/CONSTRUCTION OBSERVATION/TESTING

We should be provided the opportunity to review all drawings and specifications that pertain to site preparation and foundation construction. Site preparation will require field confirmation of the subgrade conditions beneath the tanks. That confirmation should be done by a Foundation Engineering representative. Mitigation of any subgrade pumping will also require engineering review and judgment. Frequent field density tests should be run on all engineered fill. Compaction of fill that is too coarse or variable for density testing should be evaluated by observation of the compaction method and proof-rolling with a loaded dump truck or other approved vehicle.

VARIATION OF SUBSURFACE CONDITIONS, USE OF THIS REPORT, AND WARRANTY

The analysis, conclusions, and recommendations contained herein assume the subsurface profiles observed in the borings are representative of the site conditions. The above recommendations assume we will have the opportunity to review final drawings and be present during construction to confirm the assumed soil and ground water conditions in the excavations. No changes in the enclosed recommendations should be made without our approval. We will assume no responsibility or liability for any engineering judgment, inspection, or testing performed by others.

This report was prepared for the exclusive use of the Eugene Water & Electric Board and their design consultants for the East 40th Avenue Storage Tanks project in Eugene, Oregon. Information contained herein should not be used for other sites or for unanticipated construction without our written consent. This report is intended for planning and design purposes as described herein. Contractors using this information to estimate construction quantities or costs do so at their own risk. Our services do not include any survey or assessment of potential surface contamination or contamination of the soil or ground water by hazardous or toxic materials. We assume those services, if needed, have been completed by others.

Our work was done in accordance with generally accepted soil and foundation engineering practices. No other warranty, expressed or implied, is made.

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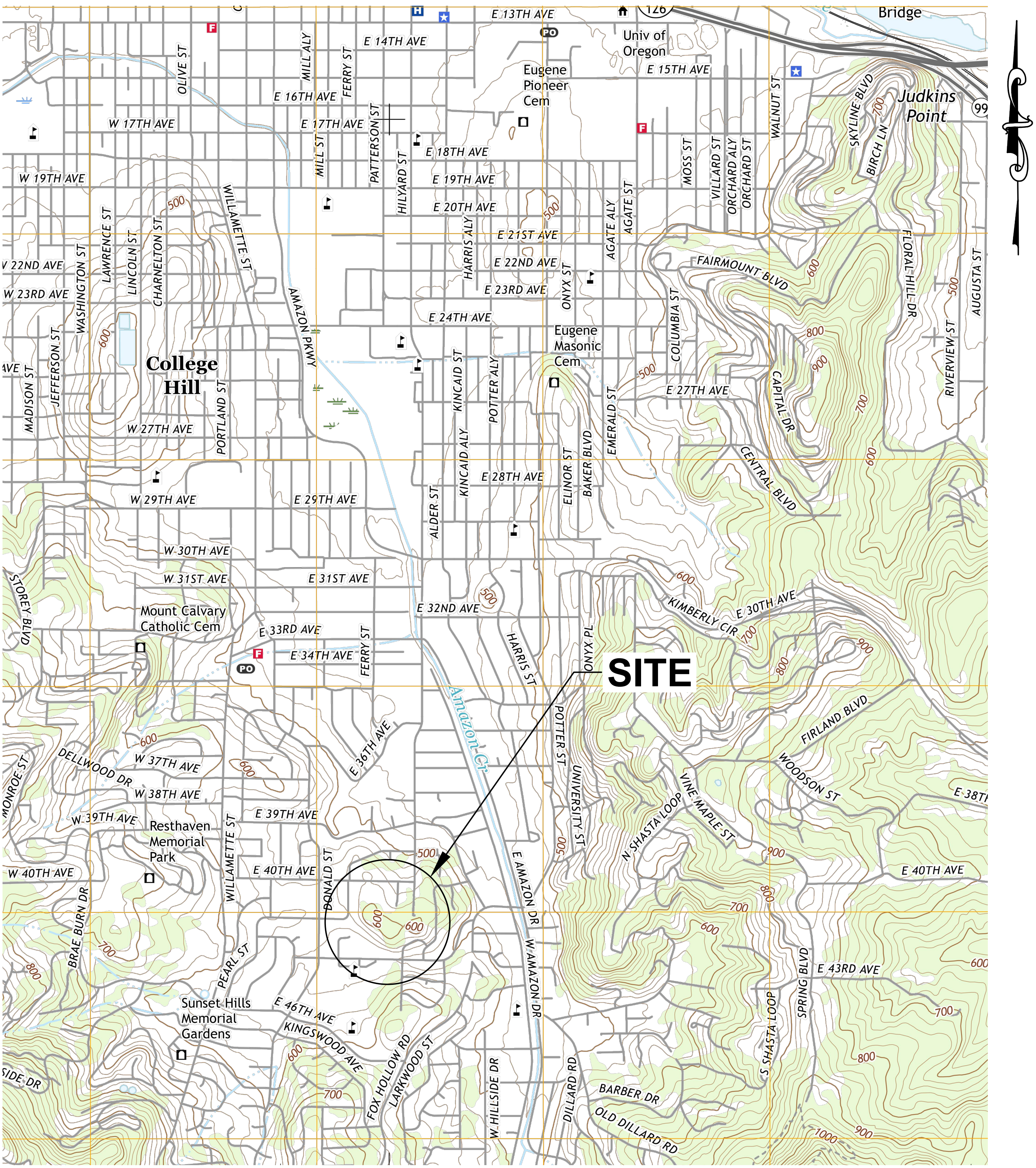
Appendix A

Figures

Foundation Engineering, Inc.
Professional Geotechnical Services



SCALE IN FEET

0 1000 2000 4000

Note: Base map obtained from the USGS website (https://ngmdb.usgs.gov).

Foundation Engineering, Inc.
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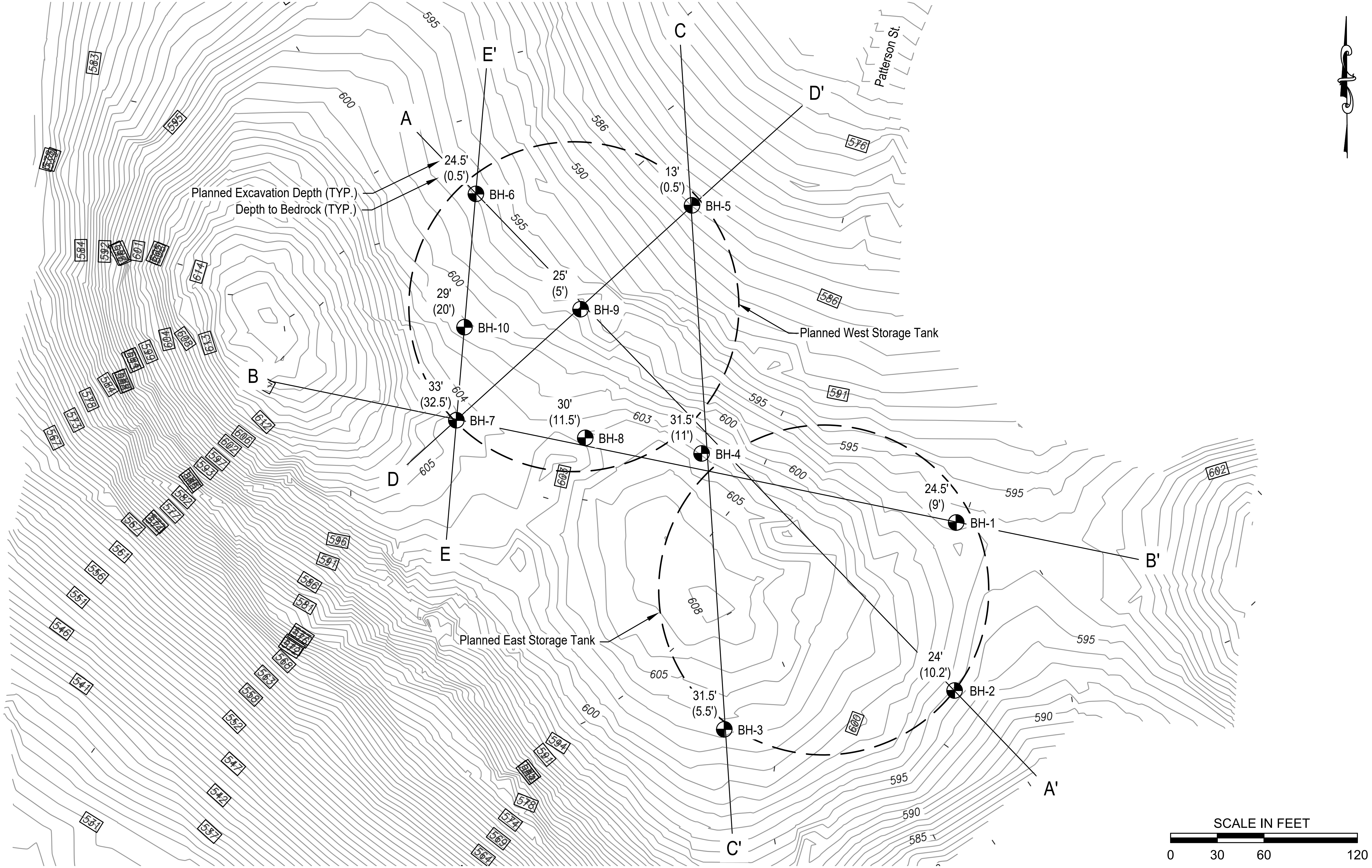
VICINITY MAP

EAST 40TH AVE STORAGE TANKS
EUGENE, OREGON

FIGURE NO.
1A

PROJECT NO.	DATE:	DRAWN BY:
2201086	Nov. 24, 2020	MLM

FILE:Figure 1A (11-23-20).dwg



NOTES:

1. BORING LOCATIONS WERE ESTABLISHED REFERENCING EXISTING LANDMARKS AND ARE APPROXIMATE.
2. SEE REPORT FOR A DISCUSSION OF SUBSURFACE CONDITIONS.
3. BASE MAP WAS PROVIDED BY MURRAYSMITH. ELEVATIONS REFERENCE NGVD29.

Foundation Engineering, Inc.
Professional Geotechnical Services

PROJECT NO.
2201086

DATE:
May. 28, 2021

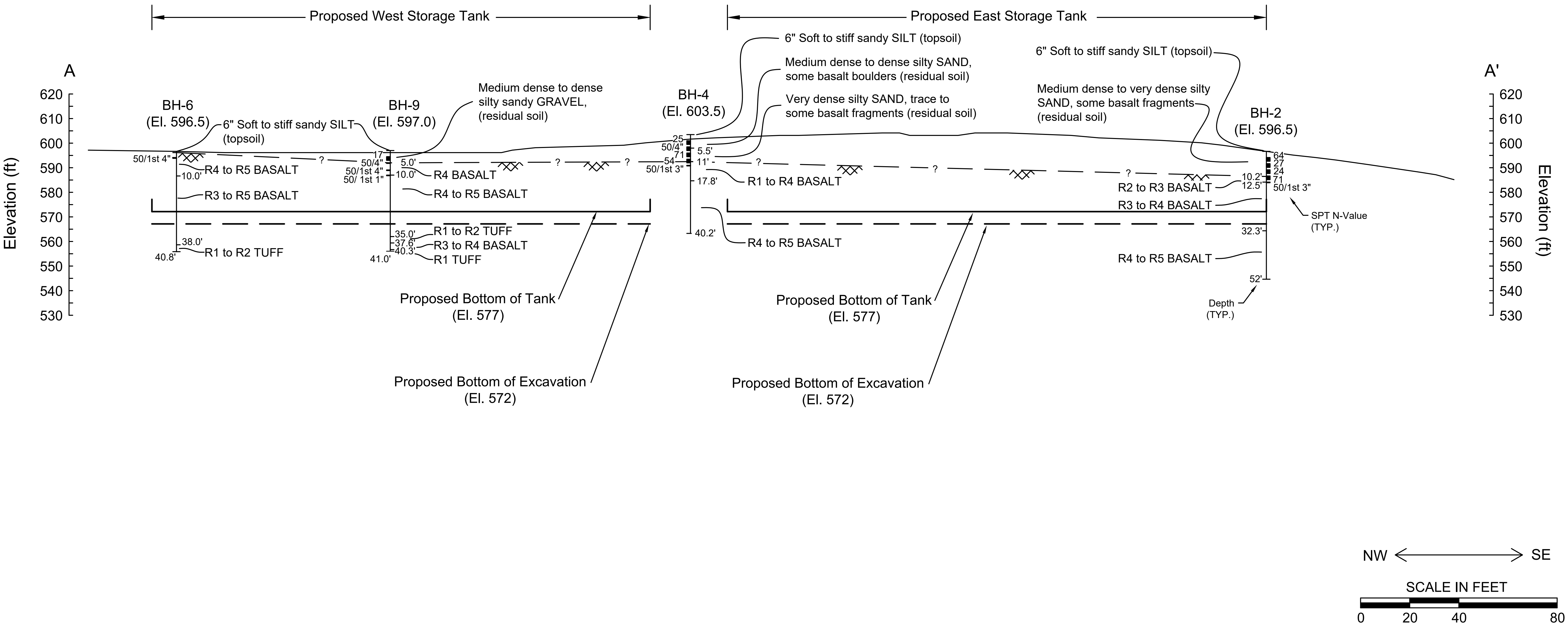
DRAWN BY:
MLM

SITE LAYOUT AND BORING LOCATIONS

EAST 40TH AVENUE STORAGE TANKS
EUGENE, OREGON

FIGURE NO.
2A

FILE:Figure 2A NGVD29 (5–28–21).dwg



NOTES:

1. SURFACE PROFILE IS BASED ON TOPOGRAPHIC DATA PROVIDED BY EWEB. ELEVATIONS REFERENCE NGVD29.
2. SUBSURFACE CROSS-SECTION WAS INTERPRETED BASED ON THE SOIL AND ROCK CONDITIONS ENCOUNTERED IN BH-2, BH-4, BH-6, AND BH-9.
3. SEE REPORT FOR A DISCUSSION OF SITE CONDITIONS.

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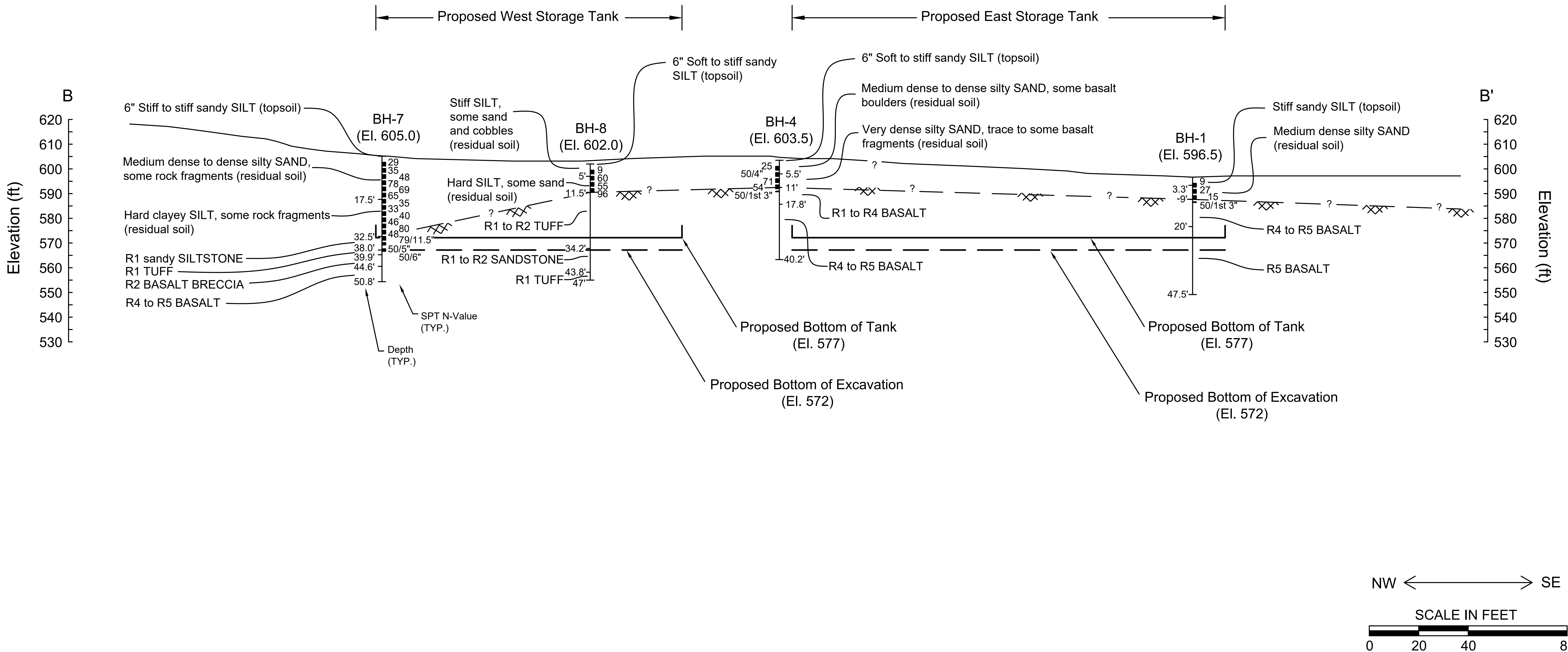
PROJECT NO.	DATE:	DRAWN BY:
2201086	Jun. 24, 2021	MLM

CROSS-SECTION A-A'

EAST 40TH AVENUE STORAGE TANKS
EUGENE, OREGON

FIGURE NO.
3A

FILE: 40th Ave X–sections (6–25–21).dwg



NOTES:

1. SURFACE PROFILE IS BASED ON TOPOGRAPHIC DATA PROVIDED BY EWEB. ELEVATIONS REFERENCE NGVD29.
2. SUBSURFACE CROSS-SECTION WAS INTERPRETED BASED ON THE SOIL AND ROCK CONDITIONS ENCOUNTERED IN BH-1, BH-4, BH-7, AND BH-8.
3. SEE REPORT FOR A DISCUSSION OF SITE CONDITIONS.

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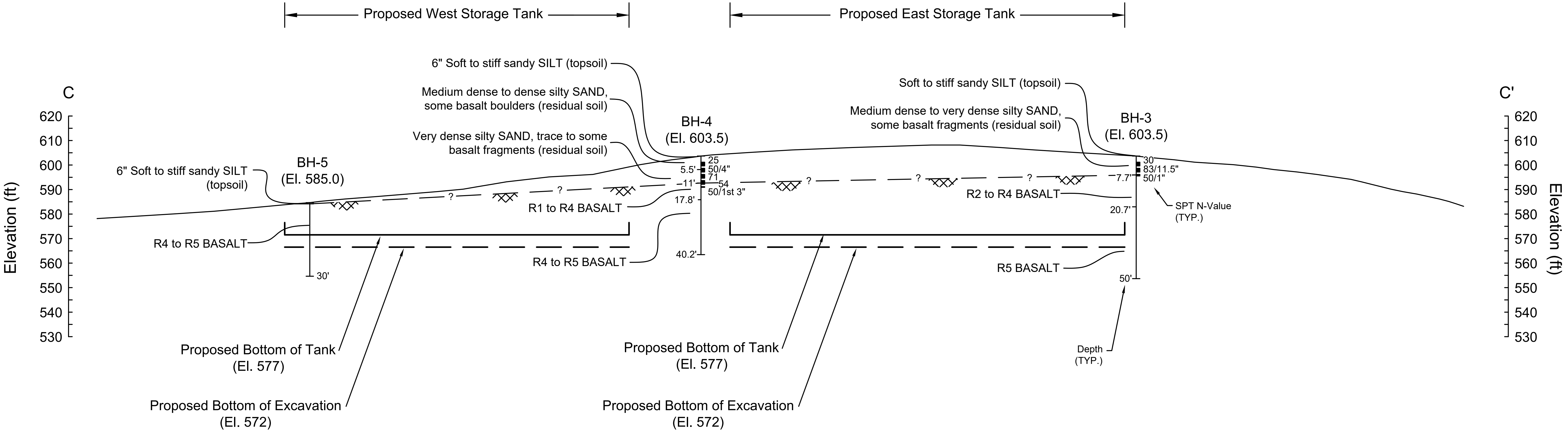
PROJECT NO.	DATE:	DRAWN BY:
2201086	Jun. 24, 2021	MLM

CROSS-SECTION B-B'

EAST 40TH AVENUE STORAGE TANKS
EUGENE, OREGON

FIGURE NO.
4A

FILE: 40th Ave X-sections (6-25-21).dwg



N S

SCALE IN FEET

0 20 40 80

NOTES:

1. SURFACE PROFILE IS BASED ON TOPOGRAPHIC DATA PROVIDED BY EWEB. ELEVATIONS REFERENCE NGVD29.
2. SUBSURFACE CROSS-SECTION WAS INTERPRETED BASED ON THE SOIL AND ROCK CONDITIONS ENCOUNTERED IN BH-3, BH-4, AND BH-5.
3. SEE REPORT FOR A DISCUSSION OF SITE CONDITIONS.

FILE: 40th Ave X-sections (6-25-21).dwg

Foundation Engineering, Inc.
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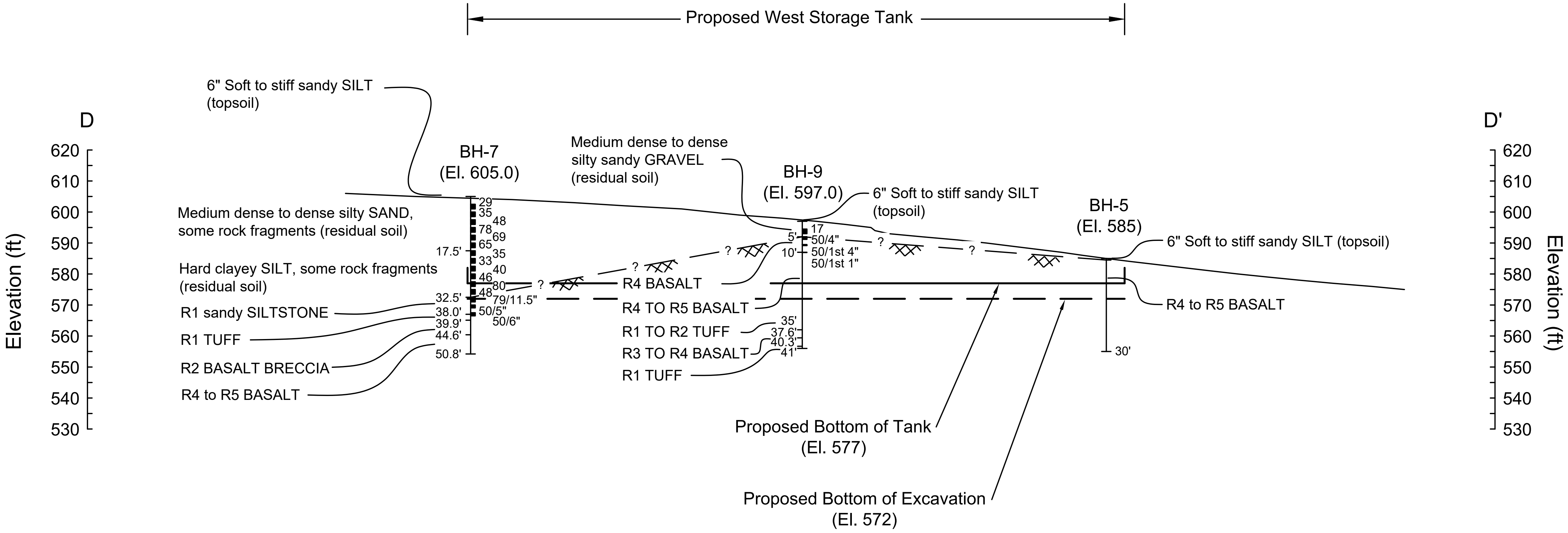
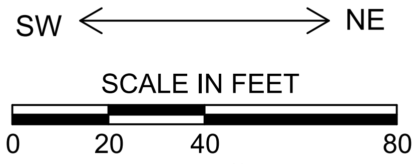
PROJECT NO.	DATE:	DRAWN BY:
2201086	Jun. 24, 2021	MLM

CROSS-SECTION C-C'

EAST 40TH AVENUE STORAGE TANKS
EUGENE, OREGON

FIGURE NO.
5A





NOTES:

1. SURFACE PROFILE IS BASED ON TOPOGRAPHIC DATA PROVIDED BY EWEB. ELEVATIONS REFERENCE NAVD88.
2. SUBSURFACE CROSS-SECTION WAS INTERPRETED BASED ON THE SOIL AND ROCK CONDITIONS ENCOUNTERED IN BH-5, BH-7, AND BH-9.
3. SEE REPORT FOR A DISCUSSION OF SITE CONDITIONS.

FILE: 40th Ave X-sections (6-25-21).dwg

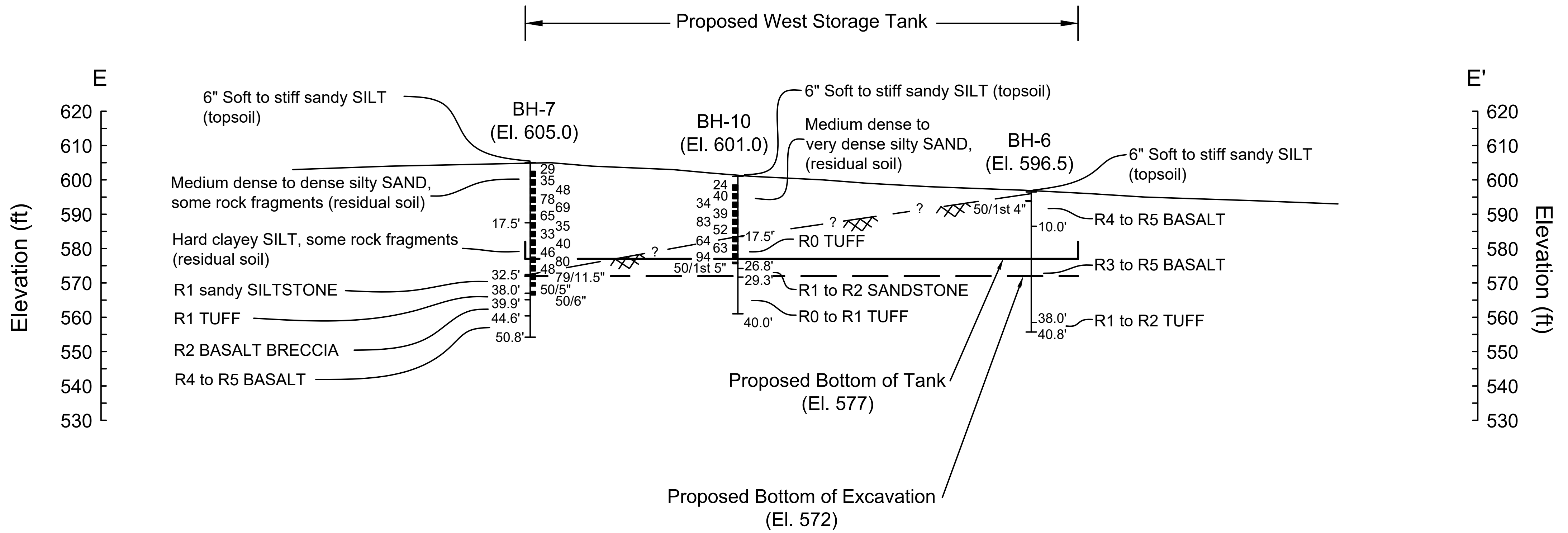
Foundation Engineering, Inc.
Professional Geotechnical Services

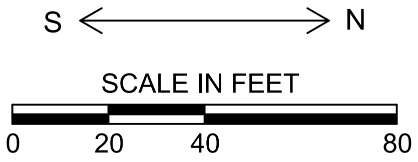
PROJECT NO.	DATE:	DRAWN BY:
2201086	Jun. 24, 2021	MLM

CROSS-SECTION D-D'

EAST 40TH AVENUE STORAGE TANKS
EUGENE, OREGON

FIGURE NO.
6A





NOTES:

1. SURFACE PROFILE IS BASED ON TOPOGRAPHIC DATA PROVIDED BY EWEB. ELEVATIONS REFERENCE NAVD88.
2. SUBSURFACE CROSS-SECTION WAS INTERPRETED BASED ON THE SOIL AND ROCK CONDITIONS ENCOUNTERED IN BH-6, BH-7, AND BH-10.
3. SEE REPORT FOR A DISCUSSION OF SITE CONDITIONS.

FILE: 40th Ave X-sections (6-25-21).dwg

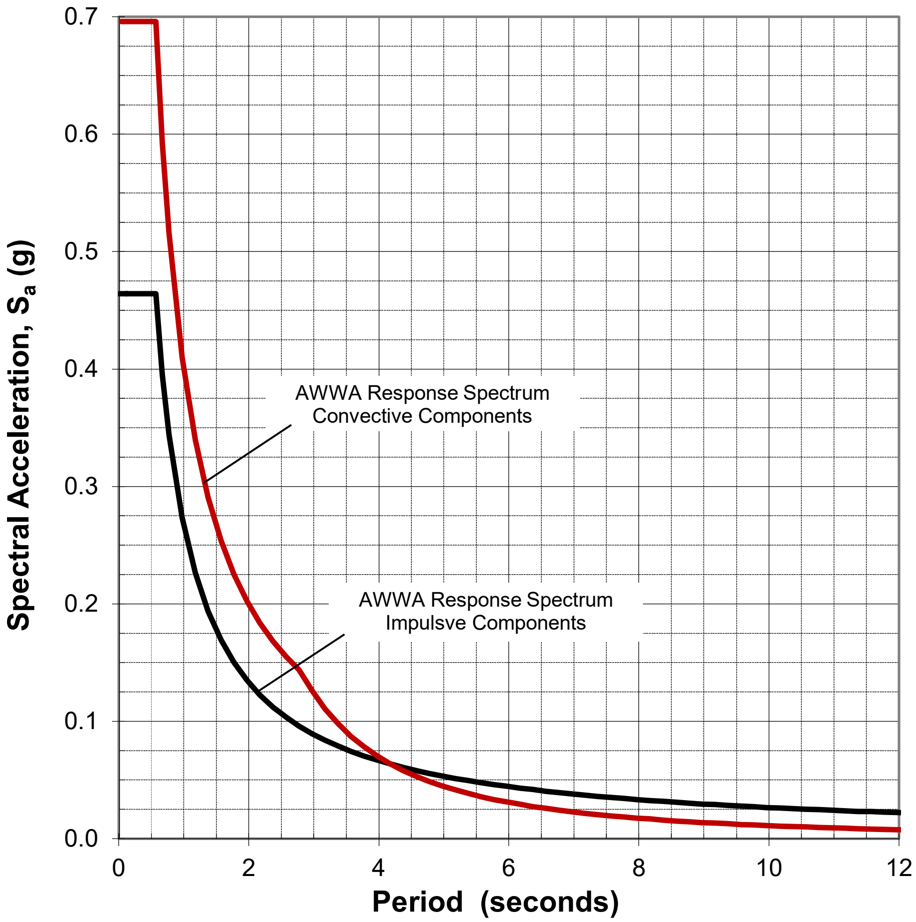
Foundation Engineering, Inc.
Professional Geotechnical Services

PROJECT NO.	DATE:	DRAWN BY:
2201086	Jun. 24, 2021	MLM

CROSS-SECTION E-E'

EAST 40TH AVENUE STORAGE TANKS
EUGENE, OREGON

FIGURE NO.
7A



Notes:

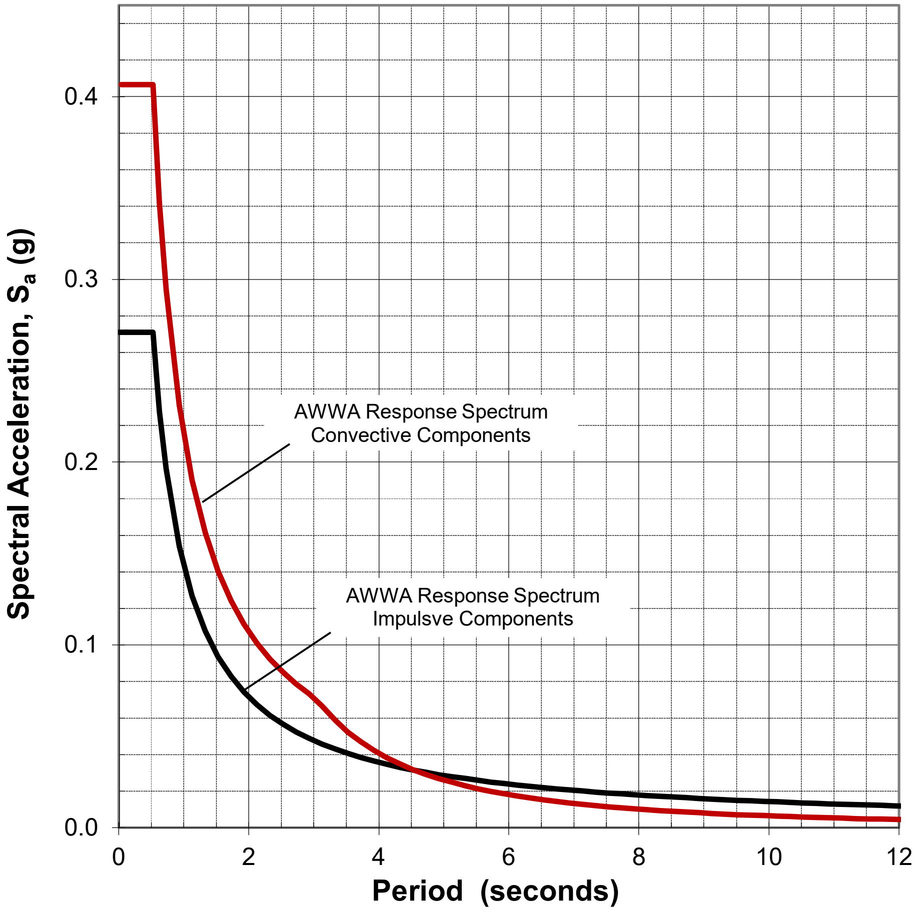
1. The Design Response Spectra are based on the General Procedure in AWWA D110-13 Section 4.3 with a 2% probability of exceedence in 50 years.
2. The following parameters were used for the impulsive component response spectrum:

Site Class= B Damping = 5%

S_S =	0.70	F_a =	1.00	S_{MS} =	0.70	S_{DS} =	0.46
S_1 =	0.40	F_v =	1.00	S_{M1} =	0.40	S_{D1} =	0.27

3. S_S and S_1 values indicated in Note 2 are USGS 2014 risk-targeted MCE spectral accelerations available from https//:seismicmaps.org.
4. F_a and F_v were selected from ASCE 7-16 Tables 11.4-1 and 11.4-2 based on the S_S and S_1 values. S_{DS} and S_{D1} values include a 2/3 reduction on S_{MS} and S_{M1} as discussed in AWWA Section 4.3.
5. The response spectrum for the conductive components was calculated based on AWWA D110-13 Eqs. 4-19 and 4-20.
6. Site location is: Latitude 44.0099, Longitude -123.0835.

FIGURE 8A
AWWA D110-13 SITE RESPONSE SPECTRA
2% Probability of Exceedence in 50 years
East 40th Avenue Storage Tanks
Eugene, Oregon
Project No.: 2201086



Notes:

1. The Design Response Spectra are based on the General Procedure in AWWA D110-13 Section 4.3 with a 10% probability of exceedence in 50 years.
2. The following parameters were used for the impulsive component response spectrum:

Site Class=	B	Damping = 5%			
S_S =	0.27	F_a =	1.00	S_{XS} =	0.27
S_1 =	0.14	F_v =	1.00	S_{X1} =	0.14

3. S_S and S_1 values indicated in Note 2 are USGS 2014 MCE spectral accelerations corrected for directivity available from https//:seismicmaps.org.
4. F_a and F_v were selected from ASCE 7-16 Tables 11.4-1 and 11.4-2 based on the S_S and S_1 values.
5. The response spectrum for the conductive components was calculated based on AWWA D110-13 Eqs. 4-19 and 4-20.
6. Site location is: Latitude 44.0099, Longitude -123.0835.

FIGURE 9A
AWWA D110-13 SITE RESPONSE SPECTRA
10% Probability of Exceedence in 50 years
East 40th Avenue Storage Tanks
Eugene, Oregon
Project No.: 2201086

Appendix B

Boring Logs and Corebox Photos

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DISTINCTION BETWEEN FIELD LOGS AND FINAL LOGS

A field log is prepared for each boring or test pit by our field representative. The log contains information concerning sampling depths and the presence of various materials such as gravel, cobbles, and fill, and observations of ground water. It also contains our interpretation of the soil conditions between samples. The final logs presented in this report represent our interpretation of the contents of the field logs and the results of the sample examinations and laboratory test results. Our recommendations are based on the contents of the final logs and the information contained therein and not on the field logs.

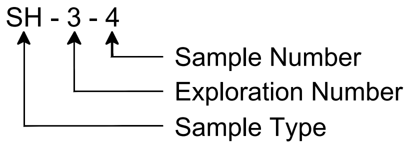
VARIATION IN SOILS BETWEEN TEST PITS AND BORINGS

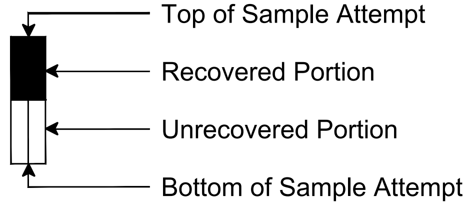
The final log and related information depict subsurface conditions only at the specific location and on the date indicated. Those using the information contained herein should be aware that soil conditions at other locations or on other dates may differ. Actual foundation or subgrade conditions should be confirmed by us during construction.

TRANSITION BETWEEN SOIL OR ROCK TYPES

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

SAMPLE OR TEST SYMBOLS





C - Pavement Core Sample
CS - Rock Core Sample
OS - Oversize Sample (3-inch O.D. split-spoon)
S - Grab Sample
SH - Thin-walled Shelby Tube Sample
SS - Standard Penetration Test Sample
(2-inch O.D. split-spoon)

▲ Standard Penetration Test Resistance equals the number of blows a 140 lb. weight falling 30 in. is required to drive a standard split-spoon sampler 1 ft. Practical refusal is equal to 50 or more blows per 6 in. of sampler penetration.

● Water Content (%)

FIELD SHEAR STRENGTH TEST

Shear strength measurements on test pit side walls, blocks of soil or Shelby tube samples are typically made with Torvane or Field Vane shear devices.

WATER TABLE

▼ Water Table Location

(1/31/16) Date of Measurement

TYPICAL SOIL/ROCK SYMBOLS

▩ Concrete
⌇ Organics
▨ Clay
▥ Silt
⁘ Sand
⁙ Gravel
▧ Basalt
⁚ Sandstone
▤ Siltstone

UNIFIED SOIL CLASSIFICATION SYMBOLS

G - Gravel
S - Sand
M - Silt
C - Clay
Pt - Peat

W - Well Graded
P - Poorly Graded
L - Low Plasticity
H - High Plasticity
O - Organic

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SYMBOL KEY

EXPLORATION LOGS

Explanation of Common Terms Used in Soil Descriptions

Field Identification	Cohesive Soils			Granular Soils	
	SPT*	S_u** (tsf)	Term	SPT*	Term
Easily penetrated several inches by fist.	0 - 2	< 0.125	Very Soft	0 - 4	Very Loose
Easily penetrated several inches by thumb.	2 - 4	0.125 - 0.25	Soft	4 - 10	Loose
Can be penetrated several inches by thumb with moderate effort.	4 - 8	0.25 - 0.50	Medium Stiff	10 - 30	Medium Dense
Readily indented by thumb but penetrated only with great effort.	8 - 15	0.50 - 1.0	Stiff	30 - 50	Dense
Readily indented by thumbnail.	15 - 30	1.0 - 2.0	Very Stiff	> 50	Very Dense
Indented with difficulty by thumbnail.	> 30	> 2.0	Hard		

* SPT N-value in blows per foot (bpf)
** Undrained shear strength

Term	Soil Moisture Field Description
Dry	Absence of moisture. Dusty. Dry to the touch.
Damp	Soil has moisture. Cohesive soils are below plastic limit and usually moldable.
Moist	Grains appear darkened, but no visible water. Silt/clay will clump. Sand will bulk. Soils are often at or near plastic limit.
Wet	Visible water on larger grain surfaces. Sand and cohesionless silt exhibit dilatancy. Cohesive soil can be readily remolded. Soil leaves wetness on the hand when squeezed. Soil is wetter than the optimum moisture content and above the plastic limit.

Term	PI	Plasticity Field Test
Non-plastic	0 - 3	Cannot be rolled into a thread at any moisture.
Low Plasticity	3 - 15	Can be rolled into a thread with some difficulty.
Medium Plasticity	15 - 30	Easily rolled into thread.
High Plasticity	> 30	Easily rolled and re-rolled into thread.

Term	Soil Structure Criteria
Stratified	Alternating layers at least ¼ inch thick.
Laminated	Alternating layers less than ¼ inch thick.
Fissured	Contains shears and partings along planes of weakness.
Slickensided	Partings appear glossy or striated.
Blocky	Breaks into small lumps that resist further breakdown.
Lensed	Contains pockets of different soils.

Term	Soil Cementation Criteria
Weak	Breaks under light finger pressure.
Moderate	Breaks under hard finger pressure.
Strong	Will not break with finger pressure.

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SOIL DESCRIPTIONS

COMMON TERMS

Explanation of Common Terms Used in Rock Descriptions

Field Identification		UCS (psi)	Strength	Hardness (ODOT)
Indented by thumbnail.	R0	< 100	Extremely Weak	Extremely Soft
Crumbles under firm blows with geological hammer. Can be peeled by a pocket knife.	R1	100 - 1,000	Very Weak	Very Soft
Can be peeled by a pocket knife with difficulty, shallow indentations made by firm blow with geological hammer.	R2	1,000 - 4,000	Weak	Soft
Cannot be scraped or peeled with a pocket knife, specimen can be fractured with a single blow of geological hammer.	R3	4,000 - 8,000	Medium Strong	Medium Hard
Specimen requires more than one blow of geological hammer to fracture it.	R4	8,000 - 16,000	Strong	Hard
Specimen requires many blows of geological hammer to fracture it.	R5	> 16,000	Very Strong	Very Hard

Term (ODOT)	Weathering Field Identification
Fresh	Crystals are bright. Discontinuities may show some minor surface staining. No discoloration in rock fabric.
Slightly Weathered	Rock mass is generally fresh. Discontinuities are stained and may contain clay. Some discoloration in rock fabric.
Moderately Weathered	Significant portions of rock show discoloration and weathering effects. Crystals are dull and show visible chemical alteration. Discontinuities are stained and may contain secondary mineral deposits.
Highly Weathered (Predominately Decomposed)	Rock can be excavated with geologist's pick. All discontinuities exhibit secondary mineralization. Complete discoloration of rock fabric. Surface of core is friable and usually pitted due to washing out of highly altered minerals by drilling water.
Decomposed	Rock mass is completely decomposed. Original rock "fabric" may be evident (relict texture). May be reduced to soil with hand pressure.

Spacing (metric)	Spacing (imperial)	Spacing Term	Bedding/Foliation
< 6 cm	< 2 in.	Very Close	Very Thin (Laminated)
6 cm - 30 cm	2 in. - 1 ft.	Close	Thin
30 cm - 90 cm	1 ft. - 3 ft.	Moderately Close	Medium
90 cm - 3.0 m	3 ft - 10 ft.	Wide	Thick
> 3.0 m	> 10 ft.	Very Wide	Very Thick (Massive)

Vesicle Term	Volume
Some vesicles	5 - 25%
Highly vesicular	25 - 50%
Scoriaceous	> 50%

Stratification Term	Description
Lamination	< 1 cm (0.4 in.) thick beds
Fissile	Preferred break along laminations
Parting	Preferred break parallel to bedding
Foliation	Metamorpic layering and segregation of minerals

RQD %	Designation	RQD %	Designation
0 - 25	Very Poor	75 - 90	Good
25 - 50	Poor	90 - 100	Excellent
50 - 75	Fair		

Rock Quality Designation (RQD) is the cumulative length of intact rock core pieces 4 inches or longer excluding breaks caused by drilling and handling divided by run length, expressed as a percentage.

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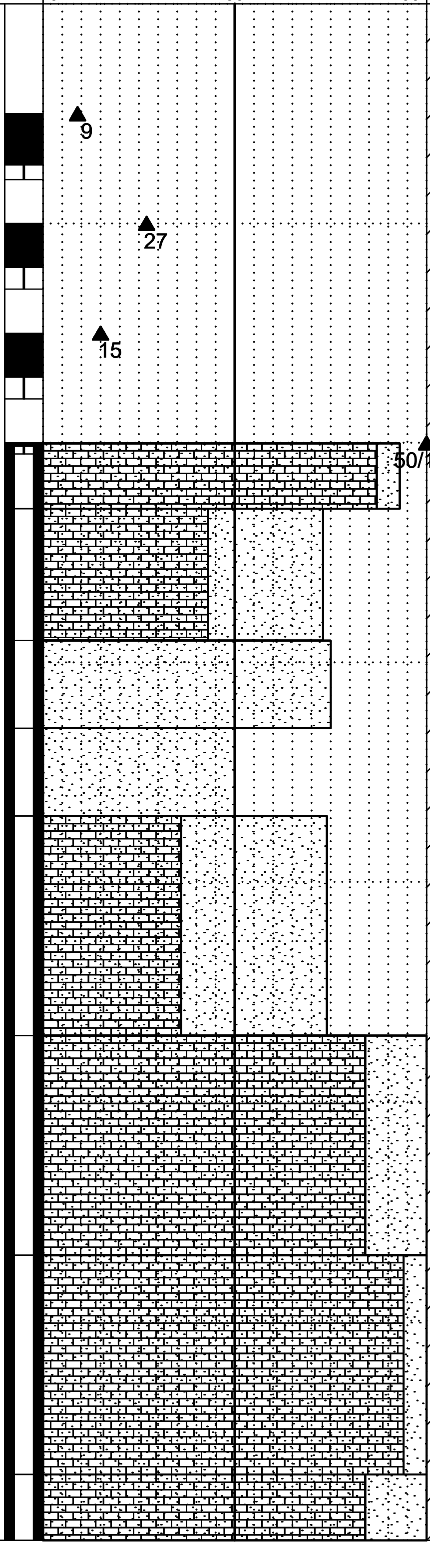
ROCK DESCRIPTIONS

COMMON TERMS

Table 1B. Summary of Boring and Bedrock Elevations

Boring	Estimated Ground Surface Elevation (ft)	Maximum Depth of Boring (ft)	Estimated Bottom of Boring Elevation (ft)	Depth to Bedrock (ft)	Estimated Bedrock Elevation (ft)
BH-1	±El. 596.5	±47.5	±El. 549.0	±9.0	±El. 587.5
BH-2	±El. 596.0	±52.0	±El. 544.0	±10.2	±El. 585.8
BH-3	±El. 603.5	±50.0	±El. 553.5	±7.7	±El. 595.8
BH-4	±El. 603.5	±40.2	±El. 563.3	±11.0	±El. 592.5
BH-5	±El. 585.0	±30.0	±El. 555.0	±0.5	±El. 584.5
BH-6	±El. 596.5	±40.8	±El. 555.7	±0.5	±El. 596.0
BH-7	±El. 605.0	±50.8	±El. 554.2	±32.5	±El. 572.5
BH-8	±El. 602.0	±47.0	±El. 555.0	±11.5	±El. 590.5
BH-9	±El. 597.0	±41.0	±El. 556.0	±5.0	±El. 592.0
BH-10	±El. 601.0	±40.0	±El. 561.0	±17.5	±El. 583.5

Depth Feet	Soil and Rock Description and Comments	Log	Elev. Depth 596.5	Samples	▲ SPT, N-Value ● Moisture, % ⬚ Recovery ⊟ RQD., % (0 50 100)	Backfill/ Installations/ Water Table
1 2 3	Stiff sandy SILT, scattered organics (ML); brown and iron-stained, damp to moist, low plasticity, fine sand, organics consist of fine roots and wood fragments, (topsoil).		0.0	SS-1-1	9	Backfilled with bentonite chips
4 5 6 7 8 9	Medium dense silty SAND (SM); light grey and iron-stained, damp, non-plastic to low plasticity silt, fine sand, (residual soil). Fine to coarse sand-sized basalt rock fragments below ±5.3 feet. Relict rock texture below ±7.5 feet.		593.2 3.3	SS-1-2 SS-1-3	27 15	
10 11 12 13 14 15 16 17 18 19 20	Strong to very strong (R4 to R5) BASALT; dark grey-brown, moderately to slightly weathered, very close to close joints are planar to irregular, rough, open, and iron-stained, (Fisher Formation). Occasional ±30 to 60 degree joints from ±10.5 to 15.5 feet. Occasional vertical joints from ±17.5 to 18.5 feet.		587.5 9.0	SS-1-4 CS-1-1 CS-1-2 CS-1-3 CS-1-4 CS-1-5	50/1st 3"	
21 22 23 24 25 26 27 28 29 30 31 32 33 34	Very strong (R5) BASALT; dark grey, slightly weathered to fresh, very close to close joints are planar to irregular, rough, and open to closed, (Fisher Formation). Occasional ±30 to 60 degree joints from ±20.5 to 44 feet. Joint calcite mineralization below ±21.5 feet. Vertical joint from ±23.5 to 24.5 feet. Close joints are planar to irregular, rough, open to closed. Some calcite-healed joints below ±23.5 feet. Dark pinkish-grey from ±29 to 30.5 feet.		576.5 20.0	CS-1-6 CS-1-7 CS-1-8		



Project No.: 2201086

Surface Elevation: 596.5 feet (Approx.)

Date of Boring: November 9, 2020

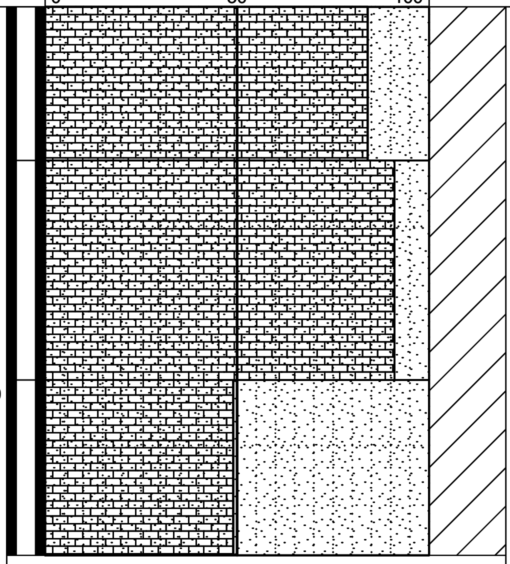
Boring Log: BH- 1

East 40th Avenue Storage Tanks

Eugene, Oregon

Foundation Engineering, Inc.

Depth Feet	Soil and Rock Description and Comments	Log	Elev. Depth 561.5	Samples	▲ SPT, N-Value ☐ Recovery ● Moisture, % ▤ RQD., % 0 50 100	Backfill/ Installations/ Water Table
36						
37						
38						
39				CS-1-9		
40						
41						
42						
43	Some iron- and manganese-stained joints below ±42.7 feet.					
44				CS-1-10		
45	Vertical joints at ±43.5 and 46.5 feet.					
46						
47			549.0 47.5			
	BOTTOM OF BORING					



Project No.: 2201086

Surface Elevation: 596.5 feet (Approx.)

Date of Boring: November 9, 2020

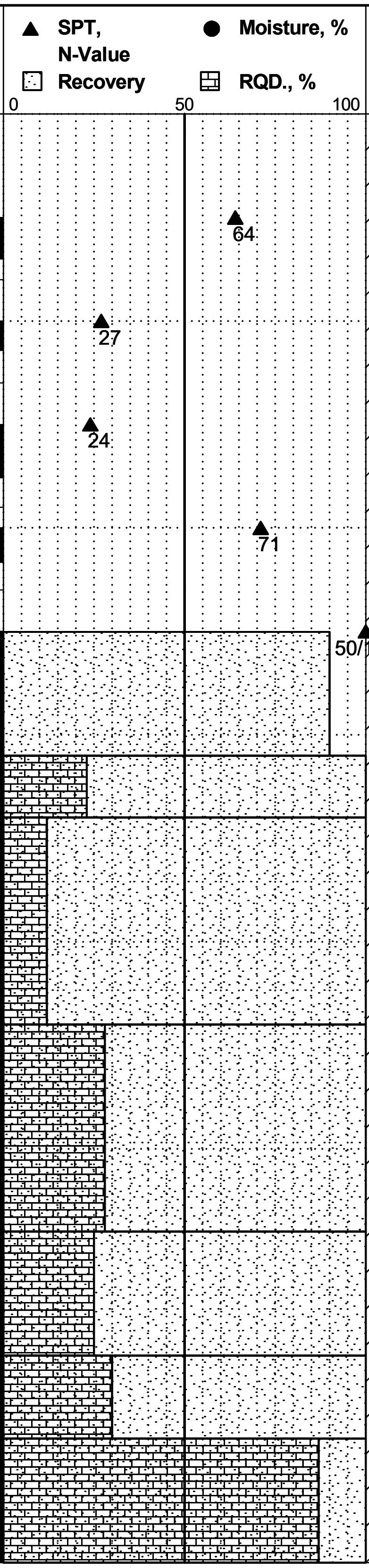
Foundation Engineering, Inc.

Boring Log: BH- 1

East 40th Avenue Storage Tanks

Eugene, Oregon

Depth Feet	Soil and Rock Description and Comments	Elev. Depth	Samples	SPT, N-Value	Backfill/ Installations/ Water Table
		596			
1	Soft to stiff sandy SILT, scattered organics (ML); brown, moist to wet, low to medium plasticity, fine sand, organics consist of fine roots, (topsoil).	595.5 0.5			Backfilled with bentonite chips
2	Medium dense to very dense silty SAND, some rock fragments (SM); light brown and iron-stained, damp to moist, non-plastic to low plasticity silt, fine sand, fine sand- to coarse angular to subangular gravel-sized basalt rock fragments, (residual soil).				
3			SS-2-1	64	
5			SS-2-2	27	
8	Relict rock texture at ±8 feet.		SS-2-3	24	
9	Iron-stained, high plasticity sandy clay lens with fine sand at ±10 feet.				
10	Weak to medium strong (R2 to R3) BASALT; dark grey, slightly weathered, (Fisher Formation).	585.8 10.2	SS-2-4	71	
12	Medium strong to strong (R3 to R4); BASALT; dark grey, moderately to slightly weathered, very close to close joints are planar to irregular, rough, open to closed, and iron-stained, (Fisher Formation).	583.5 12.5	SS-2-5 CS-2-1	50/1st 3"	
15	Occasional ±30 to 60 degree joints below ±12.6 feet.				
16			CS-2-2		
17			CS-2-3		
19	Occasional vertical joints from ±18.5 to 22.5 feet and ±27 to 32 feet.				
22			CS-2-4		
27			CS-2-5		
29	Moderately to highly weathered from ±29 to 29.2 feet.				
30			CS-2-6		
32	Strong to very strong (R4 to R5) BASALT; dark grey, slightly weathered to fresh, close to moderately close joints are planar, rough, and open to closed, some iron-stained joints, (Fisher Formation).	563.7 32.3	CS-2-7		



Project No.: 2201086

Surface Elevation: 596.0 feet (Approx.)

Date of Boring: November 10, 2020

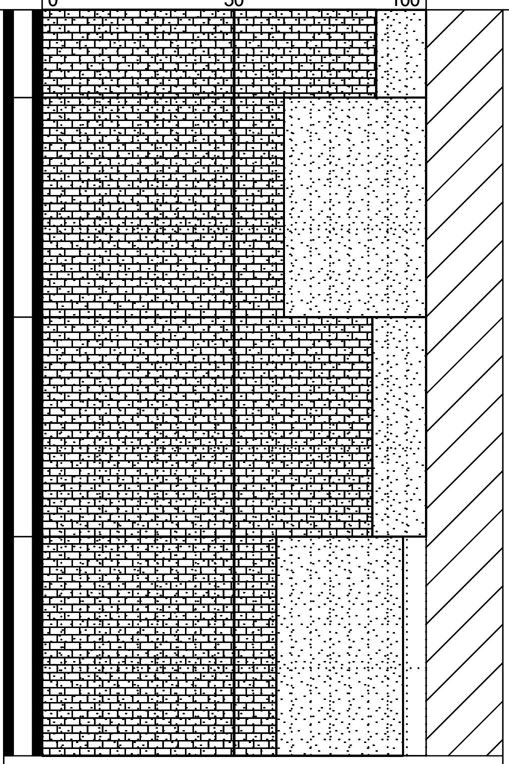
Foundation Engineering, Inc.

Boring Log: BH- 2

East 40th Avenue Storage Tanks

Eugene, Oregon

Depth Feet	Soil and Rock Description and Comments	Log	Elev. Depth 561	Samples	▲ SPT, N-Value ☐ Recovery ● Moisture, % ⊟ RQD., % (0 50 100)	Backfill/ Installations/ Water Table
36						
37				CS-2-8		
38						
39	Occasional ±30 to 60 degree joints from ±38.5 to 51 feet.					
40						
41						
42	Occasional iron-stained joints below ±42 feet.			CS-2-9		
43						
44						
45						
46						
47				CS-2-10		
48						
49						
50						
51						
52	BOTTOM OF BORING		544.0 52.0			



Project No.: 2201086

Surface Elevation: 596.0 feet (Approx.)

Date of Boring: November 10, 2020

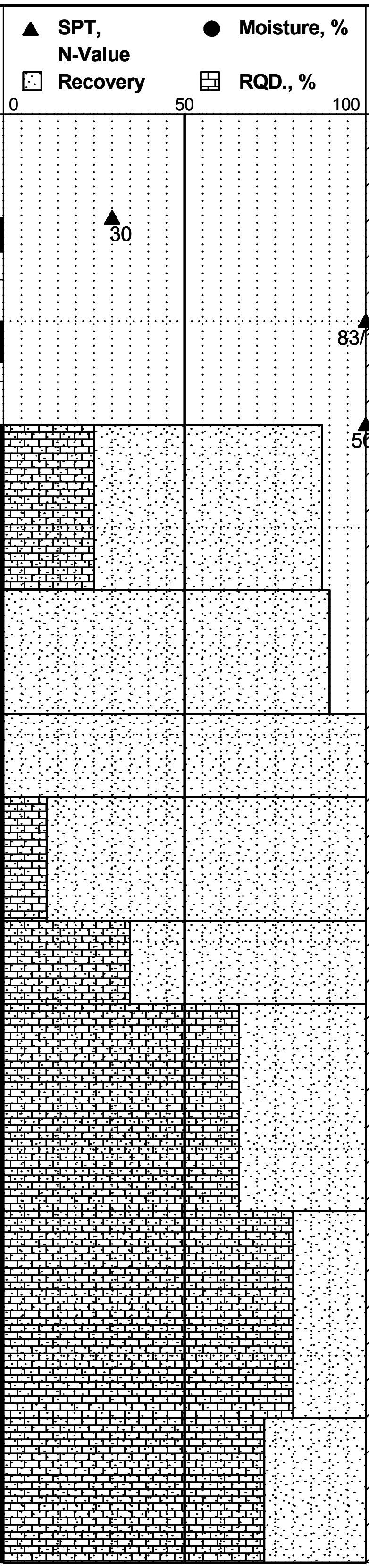
Foundation Engineering, Inc.

Boring Log: BH- 2

East 40th Avenue Storage Tanks

Eugene, Oregon

Depth Feet	Soil and Rock Description and Comments	Elev. Depth	Samples	SPT, N-Value	Backfill/ Installations/ Water Table
		603.5			
1	Soft to stiff sandy SILT, scattered organics (ML); brown, moist to wet, low to medium plasticity, fine sand, organics consist of fine roots, (topsoil).	603.0 0.5			Backfilled with bentonite chips
2	Medium dense to very dense silty SAND, some rock fragments (SM); whitish-grey to brown and iron-stained, damp, non-plastic to low plasticity silt, fine sand, fine sand- to coarse angular to subangular gravel-sized basalt rock fragments, tuffaceous, (residual soil).				
3			SS-3-1	30	
4					
5			SS-3-2	83/11½"	
6					
7					
8	Medium strong (R3) BASALT; dark grey-brown, highly to moderately weathered, very close to close joints are irregular, rough, open, iron-stained, and silt-infilled, (Fisher Formation).	595.8 7.7	SS-3-3 CS-3-1	50/1"	
9					
10					
11	Strong (R4), dark grey, fresh, and moderately close joints from ±8.3 to 9.3 feet. Occasional vertical joints below ±11.5 feet.				
12			CS-3-2		
13					
14					
15	Weak to medium strong (R2 to R3), grey, and moderately weathered silty sandstone from ±14.1 to 16.5 feet and ±17.6 to 18.0 feet.		CS-3-3		
16					
17			CS-3-4		
18					
19	Dark pinkish-grey below ±18.2 feet.				
20			CS-3-5		
21	Very strong (R5) BASALT; dark grey, slightly weathered to fresh, close to moderately close joints are planar, rough, open, and iron-stained, (Fisher Formation).	582.8 20.7			
22			CS-3-6		
23					
24					
25					
26					
27			CS-3-7		
28					
29					
30	Occasional healed joints below ±30 feet.				
31					
32			CS-3-8		
33	Very close to close joints are planar to irregular, rough, and open to closed. Some silt-infilled joints from ±30.1 to 45.3 feet.				
34					



Project No.: 2201086

Surface Elevation: 603.5 feet (Approx.)

Date of Boring: November 11, 2020

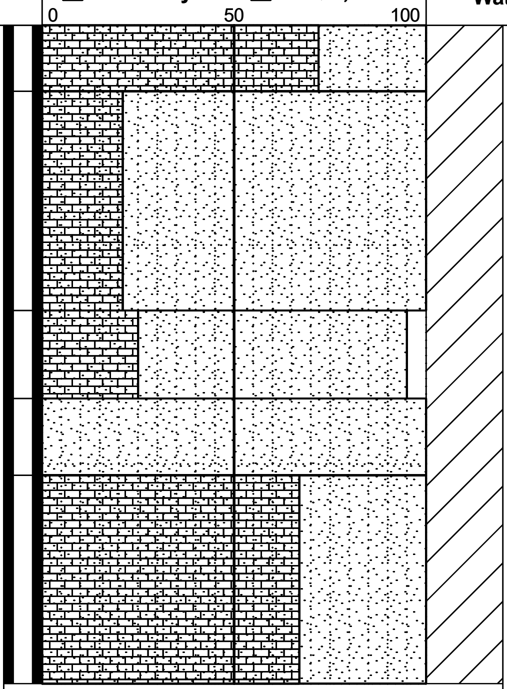
Foundation Engineering, Inc.

Boring Log: BH- 3

East 40th Avenue Storage Tanks

Eugene, Oregon

Depth Feet	Soil and Rock Description and Comments	Log	Elev. Depth	Samples	▲ SPT, N-Value ⬚ Recovery ● Moisture, % ⊟ RQD., %	Backfill/ Installations/ Water Table
			568.5			
36						
37				CS-3-9		
38						
39						
40						
41						
42	Vertical joint from ±41.5 to 42.5 feet.			CS-3-10		
43						
44				CS-3-11		
45						
46	Very close to moderately close joints below ±45.5 feet.			CS-3-12		
47						
48						
49						
50	BOTTOM OF BORING		553.5 50.0			



Project No.: 2201086

Surface Elevation: 603.5 feet (Approx.)

Date of Boring: November 11, 2020

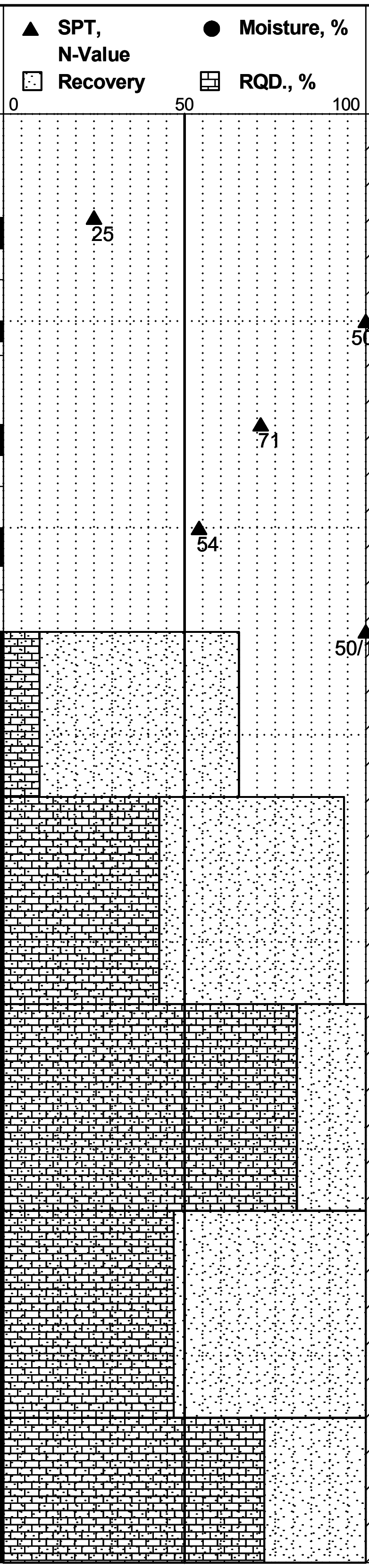
Foundation Engineering, Inc.

Boring Log: BH- 3

East 40th Avenue Storage Tanks

Eugene, Oregon

Depth Feet	Soil and Rock Description and Comments	Elev. Depth	Samples	▲ SPT, N-Value	Backfill/ Installations/ Water Table
		603.5			
1	Soft to stiff sandy SILT, scattered organics (ML); brown, moist to wet, low to medium plasticity, fine sand, organics consist of fine roots, (topsoil).	603.0 0.5			Backfilled with bentonite chips
2	Medium dense to dense silty SAND, some boulders (SM); brown to light brown and iron-stained, damp, non-plastic to low plasticity silt, fine sand, basalt boulders, (residual soil).				
3			SS-4-1	25	
4					
5		598.0 5.5	SS-4-2	50/4"	
6	Very dense silty SAND, trace to some rock fragments (SM); light brown and iron-stained, damp, non-plastic to low plasticity silt, fine sand, fine to coarse angular to subangular gravel-sized basalt rock fragments, tuffaceous, (residual soil).				
7					
8			SS-4-3	71	
9					
10			SS-4-4	54	
11	Medium strong to strong (R3 to R4) BASALT; dark grey-brown, moderately to slightly weathered, (Fisher Formation).	592.5 11.0			
12					
13	Very weak (R1), light brown, and highly to moderately weathered sandstone with fine sand along vertical joint from ±13.9 to 16.3 feet.		SS-4-5 CS-4-1	50/1st 3"	
14					
15	Very close to close joints are planar to irregular, rough, and open to closed below ±13.9 feet.				
16					
17	Occasional ±30 to 60 degree joints below ±17.3 feet.		CS-4-2		
18	Strong to very strong (R4 to R5) BASALT; dark grey to pinkish-grey, slightly weathered, very close to close joints are planar to irregular, rough, open to closed, and iron-stained, (Fisher Formation).	585.7 17.8			
19					
20					
21					
22			CS-4-3		
23					
24					
25					
26					
27			CS-4-4		
28					
29					
30	Silt-infilled joint at ±29.5 feet.				
31					
32	Close to moderately close joints from ±31.5 to 35.5 feet.		CS-4-5		
33					
34					



Project No.: 2201086

Surface Elevation: 603.5 feet (Approx.)

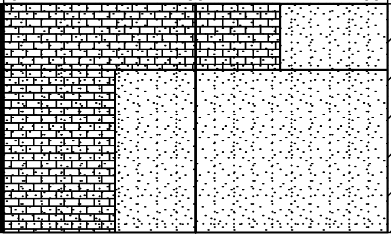
Date of Boring: November 12, 2020

Boring Log: BH- 4

East 40th Avenue Storage Tanks

Eugene, Oregon

Foundation Engineering, Inc.

Depth Feet	Soil and Rock Description and Comments	Log	Elev. Depth	Samples	▲ SPT, N-Value ☐ Recovery ● Moisture, % ⊞ RQD., %	Backfill/ Installations/ Water Table
			568.5		0 50 100	
36 37 38 39 40	Dark pinkish-grey and some healed and silt-infilled joints below ±36.7 feet.		563.3 40.2	CS-4-6		
	BOTTOM OF BORING					

Project No.: 2201086

Surface Elevation: 603.5 feet (Approx.)

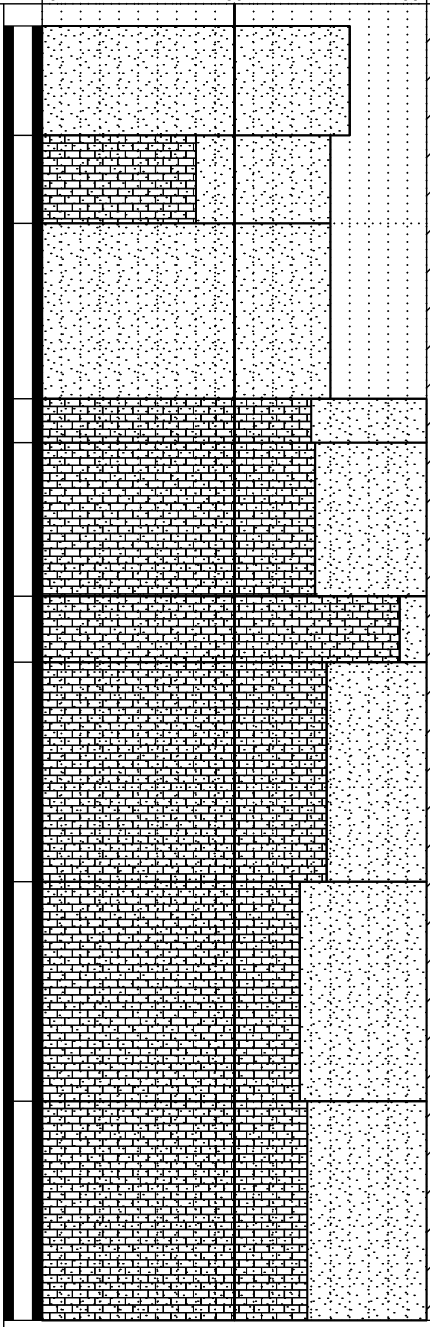
Date of Boring: November 12, 2020

Foundation Engineering, Inc.

Boring Log: BH- 4

East 40th Avenue Storage Tanks

Eugene, Oregon

Depth Feet	Soil and Rock Description and Comments	Log	Elev. Depth 585	Samples	▲ SPT, N-Value ⊡ Recovery ● Moisture, % ⊟ RQD., % 0 50 100	Backfill/ Installations/ Water Table
1	Soft to stiff sandy SILT, scattered organics (ML); brown, moist to wet, low to medium plasticity, fine sand, organics consist of fine roots, (topsoil).		584.5 0.5	CS-5-1		Backfilled with bentonite chips
2	Strong to very strong (R4 to R5) BASALT; dark grey, moderately to slightly weathered, very close to close joints are irregular, rough, open to closed, and iron-stained, some silt-infilled joints, (Fisher Formation).					
3				CS-5-2		
4						
5	Occasional ±60 degree joints from ±0.5 to 3.7 feet.			CS-5-3		
6	Planar to irregular joints below ±6 feet.					
7						
8	Dark pinkish-grey from ±8 to 8.5 feet and ±12.3 to 13 feet.					
9				CS-5-4		
10	Occasional ±30 to 60 degree joints below ±9 feet.			CS-5-5		
11						
12						
13						
14				CS-5-6		
15	Slightly weathered to fresh and very close to moderately close joints below ±15 feet.			CS-5-7		
16						
17						
18						
19						
20				CS-5-8		
21						
22						
23						
24	Some healed joints below ±24 feet.					
25				CS-5-9		
26						
27						
28						
29						
30	BOTTOM OF BORING		555.0 30.0			

Project No.: 2201086

Surface Elevation: 585.0 feet (Approx.)

Date of Boring: November 15, 2020

Foundation Engineering, Inc.

Boring Log: BH- 5

East 40th Avenue Storage Tanks

Eugene, Oregon

Depth Feet	Soil and Rock Description and Comments	Log	Elev. Depth	Samples	▲ SPT, N-Value ⊡ Recovery ● Moisture, % ⊟ RQD., %	Backfill/ Installations/ Water Table

Soft to stiff sandy SILT, scattered organics (ML); brown, moist to wet, low to medium plasticity, fine sand, organics consist of fine roots, (topsoil).

Strong to very strong (R4 to R5) BASALT; dark grey, moderately to slightly weathered, (Fisher Formation).

Slightly weathered and very close to moderately close joints are planar to irregular, rough, open to closed, and iron-stained below ±5 feet.

Occasional ±30 to 60 degree joints from ±6 to 9.7 feet.

Vertical joint from ±6 to 8 feet.

Strong to very strong (R4 to R5) BASALT; dark grey, slightly weathered, close joints are planar to irregular, rough, open to closed, iron-stained, and silt-infilled, (Fisher Formation).

Dark pinkish-grey from ±10 to 11 feet.

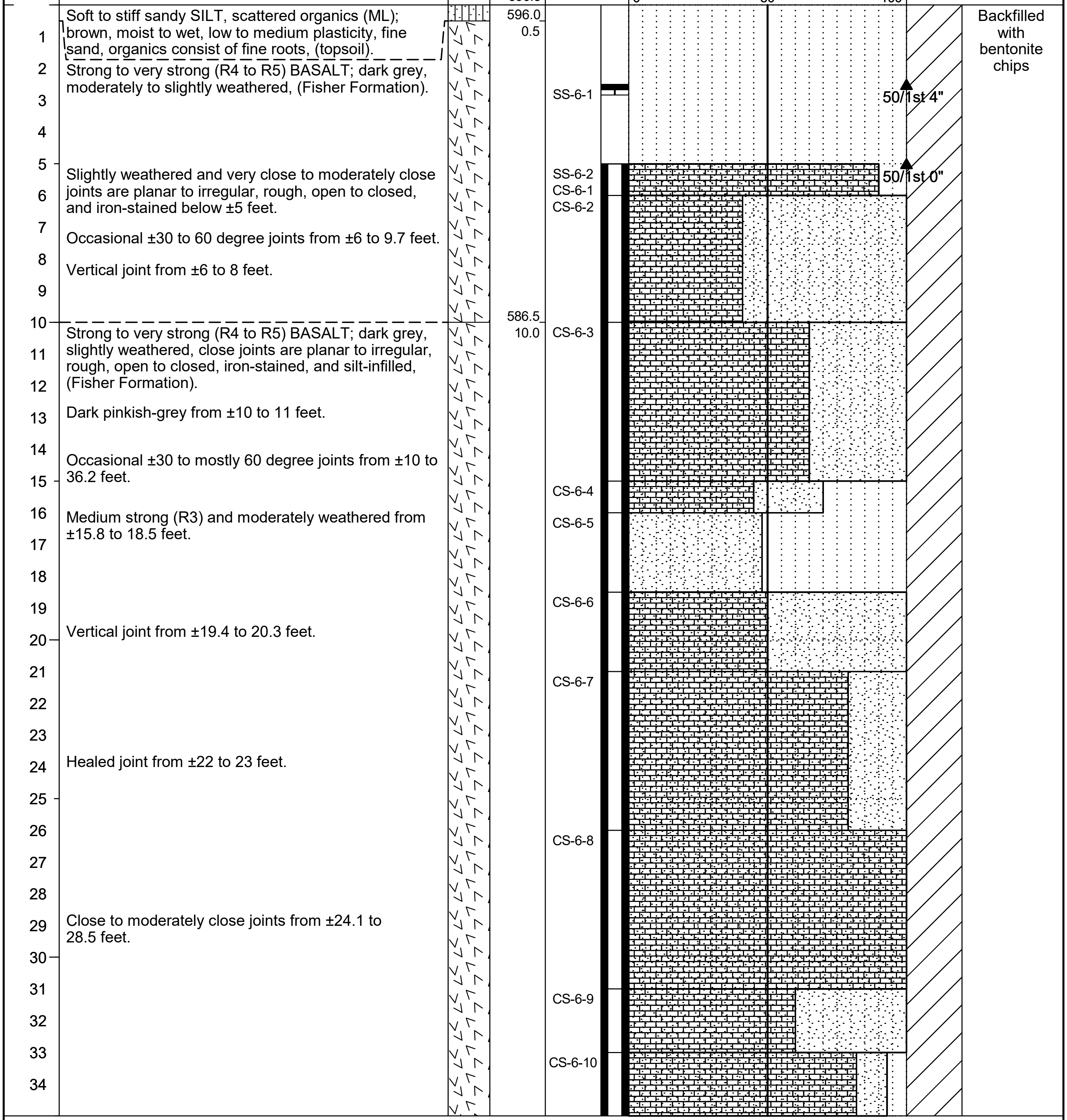
Occasional ±30 to mostly 60 degree joints from ±10 to 36.2 feet.

Medium strong (R3) and moderately weathered from ±15.8 to 18.5 feet.

Vertical joint from ±19.4 to 20.3 feet.

Healed joint from ±22 to 23 feet.

Close to moderately close joints from ±24.1 to 28.5 feet.



Project No.: 2201086

Surface Elevation: 596.5 feet (Approx.)

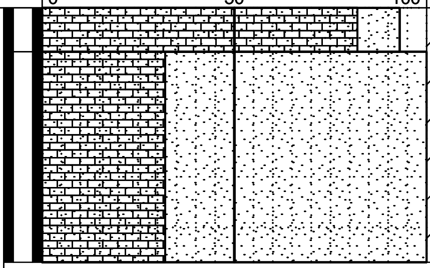
Date of Boring: November 14, 2020

Foundation Engineering, Inc.

Boring Log: BH- 6

East 40th Avenue Storage Tanks

Eugene, Oregon

Depth Feet	Soil and Rock Description and Comments	Log	Elev. Depth	Samples	▲ SPT, N-Value ● Moisture, % ⬚ Recovery ⊟ RQD., % (0 50 100)	Backfill/ Installations/ Water Table
			561.5			
36	Dark grey to dark pinkish-grey below ±35.6 feet.			CS-6-11		
37						
38	Healed joint at ±37.4 feet.		558.5 38.0			
39	Very weak to weak (R1 to R2) TUFF; pinkish-grey, moderately weathered, close joints are irregular, rough, and open, (Fisher Formation).					
40	Volcanic gravel-sized clasts below ±40 feet.		555.7 40.8			
	BOTTOM OF BORING					

Project No.: 2201086

Surface Elevation: 596.5 feet (Approx.)

Date of Boring: November 14, 2020

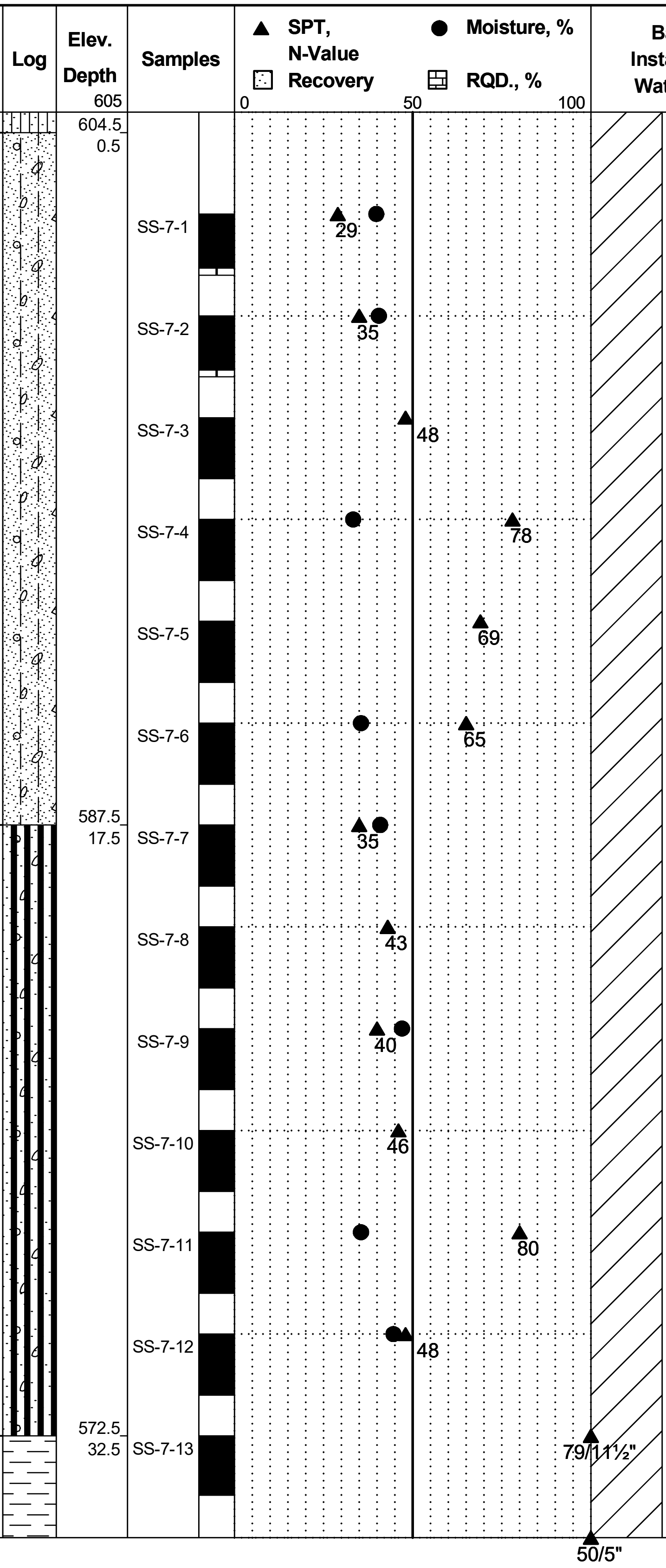
Boring Log: BH- 6

East 40th Avenue Storage Tanks

Eugene, Oregon

Foundation Engineering, Inc.

Depth Feet	Soil and Rock Description and Comments	Elev. Depth	Samples	SPT, N-Value	Backfill/ Installations/ Water Table
		605			
1	Soft to stiff sandy SILT, scattered organics (ML); brown, moist to wet, low to medium plasticity, fine sand, organics consist of fine roots, (topsoil).	604.5 0.5			Backfilled with bentonite chips
2	Medium dense to dense silty SAND, some rock fragments (SM); dark grey and iron-stained, damp, low to medium plasticity silt, fine sand, fine angular gravel-sized rock fragments, relict tuff texture, tuffaceous, (residual soil).				
3			SS-7-1	29	
5	Light brown from ±2.7 to 3 feet.		SS-7-2	35	
8			SS-7-3	48	
10	Yellowish-brown from ±10 to 12.5 feet.		SS-7-4	78	
13			SS-7-5	69	
15			SS-7-6	65	
18	Hard clayey SILT, some rock fragments (MH) orange-brown and iron-stained, damp, medium plasticity, sand- to gravel-sized rock fragments, relict tuff texture, tuffaceous, (residual soil).	587.5 17.5	SS-7-7	35	
20			SS-7-8	43	
23	Light grey and iron-stained from ±22.5 to 23.2 feet and from ±27.5 to 30.5 feet.		SS-7-9	40	
25	Orange-brown from ±23.2 to 25 feet.		SS-7-10	46	
27	Pinkish-brown from ±25 to 27.5 feet.		SS-7-11	80	
30			SS-7-12	48	
31	Light brown and iron-stained from ±30.5 to 32.5 feet.				
33	Very weak (R1) sandy SILTSTONE; blue-grey, highly to moderately weathered, fine sand, tuffaceous, (Fisher Formation).	572.5 32.5	SS-7-13	79/11½" 50/5"	



Project No.: 2201086

Surface Elevation: 605.0 feet (Approx.)

Date of Boring: November 14, 2020

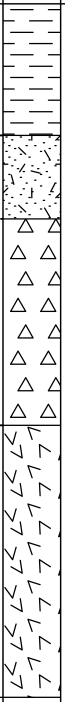
Boring Log: BH- 7

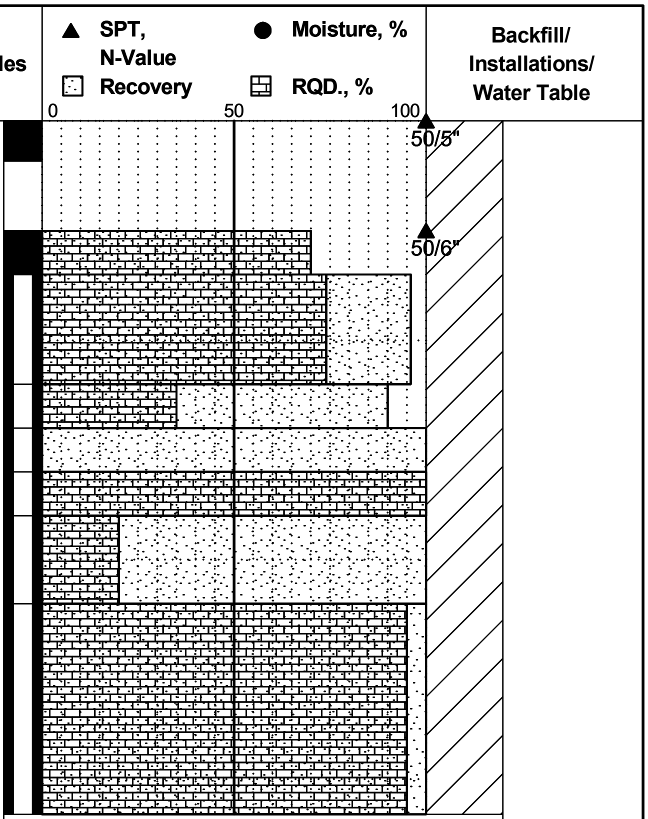
East 40th Avenue Storage Tanks

Eugene, Oregon

Foundation Engineering, Inc.

Depth Feet	Soil and Rock Description and Comments	Log	Elev. Depth 570	Samples
36 37	Dark grey below ±35 feet. Shaly below ±37.5 feet.			SS-7-14
38 39 40 41	Very weak (R1) TUFF; blue-grey and iron-stained, moderately weathered, very close to close joints are irregular, rough, and closed, fine to medium sand, (Fisher Formation). Occasional gravel-sized angular basalt clasts below ±38.5 feet. Slickensides at ±39 feet.		567.0 38.0	SS-7-15 CS-7-1 CS-7-2
42 43 44	Weak (R2) BASALT BRECCIA; dark grey, slightly weathered, very close to close joints are irregular, rough to very rough, and open to closed, (Fisher Formation).		565.1 39.9	CS-7-3 CS-7-5 CS-7-4
45 46 47 48 49 50	Strong to very strong (R4 to R5) BASALT; dark grey, slightly weathered, very close to moderately close joints are irregular, rough, and open to closed, (Fisher Formation). Slickensides at ±44.6 to 44.9 feet. Calcite veining from ±45 to 46 feet.		560.4 44.6	CS-7-6 CS-7-7
	BOTTOM OF BORING		554.2 50.8	





Project No.: 2201086

Surface Elevation: 605.0 feet (Approx.)

Date of Boring: November 14, 2020

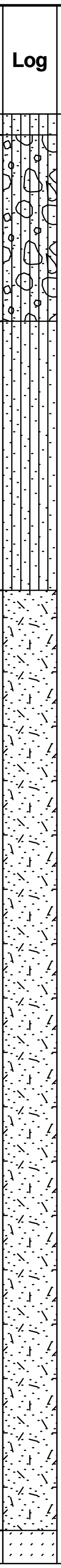
Boring Log: BH- 7

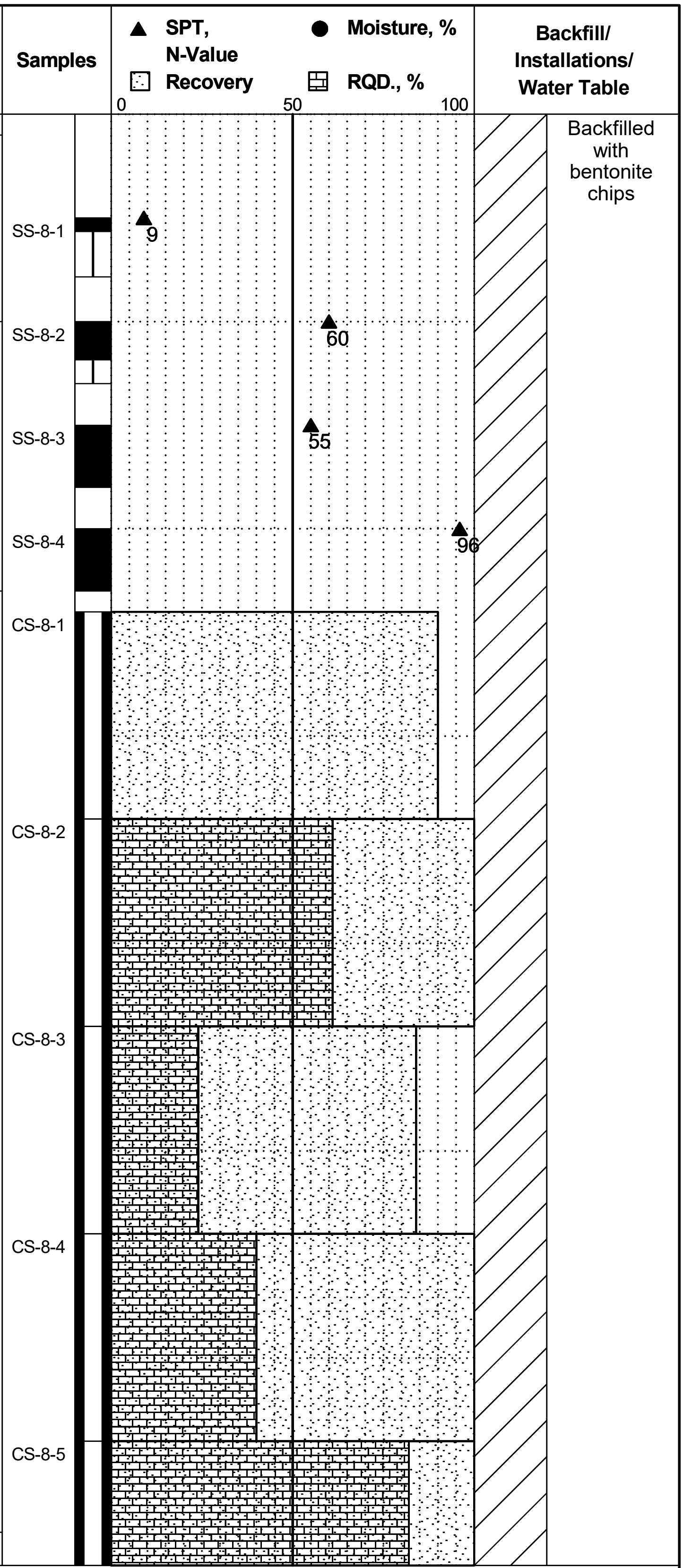
East 40th Avenue Storage Tanks

Eugene, Oregon

Foundation Engineering, Inc.

Depth Feet	Soil and Rock Description and Comments	Elev. Depth 602
1	Soft to stiff sandy SILT, scattered organics (ML); brown, moist to wet, low to medium plasticity, fine sand, organics consist of fine roots, (topsoil).	601.5 0.5
2	Stiff SILT, some sand and cobbles (ML); brown, moist to wet, low plasticity, fine to coarse sand, subangular to subrounded cobbles, (residual soil).	597.0 5.0
5	Hard SILT, some sand (ML); light brown to pinkish-grey, damp, low plasticity, fine sand, relict tuff texture, tuffaceous, (residual soil).	590.5 11.5
12	Very weak (R1) TUFF; pinkish-grey, slightly weathered, very close joints are irregular, rough to very rough, open to closed, and iron-stained, sand and some fine gravel-sized volcanic clasts, (Fisher Formation).	
22	Reddish from ±16.7 to 27.5 feet.	
27	Slickensided from ±26.8 to 27.5 feet.	
29	Very weak to weak (R1 to R2) and dark grey below ±28.8 feet.	
31	Some healed joints and calcite veining below ±31.1 feet.	567.8 34.2
34	Very weak (R1) SANDSTONE; brownish-grey,	





Project No.: 2201086

Surface Elevation: 602.0 feet (Approx.)

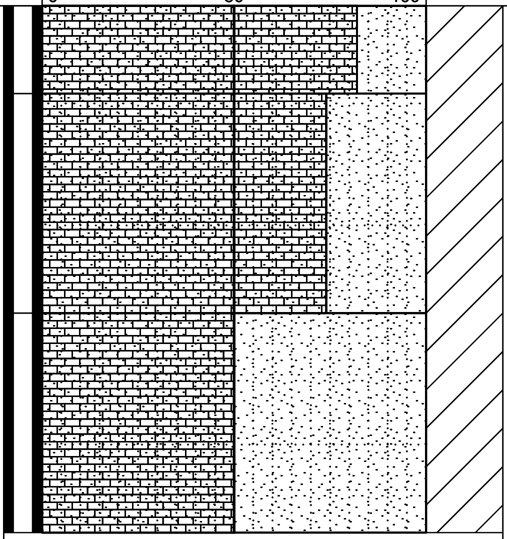
Date of Boring: May 13, 2021

Boring Log: BH- 8

East 40th Avenue Storage Tanks

Eugene, Oregon

Foundation Engineering, Inc.

Depth Feet	Soil and Rock Description and Comments	Log	Elev. Depth 567	Samples	▲ SPT, N-Value ● Moisture, % ⊡ Recovery ⊟ RQD., % 0 50 100	Backfill/ Installations/ Water Table
36	moderately to slightly weathered, very close to close joints are planar to irregular, rough to very rough, open to closed, and iron-stained, fine sand, tuffaceous, (Fisher Formation).					
37				CS-8-6		
38						
39	Weak (R2) and grey from ±38.5 to 41.1 feet.					
40						
41						
42				CS-8-7		
43			558.2 43.8			
44	Very weak (R1) TUFF; dark grey, moderately weathered, very close joints are irregular, rough, and open, sand and some fine gravel-sized volcanic clasts, (Fisher Formation).					
45						
46						
47	BOTTOM OF BORING		555.0 47.0			

Project No.: 2201086

Surface Elevation: 602.0 feet (Approx.)

Date of Boring: May 13, 2021

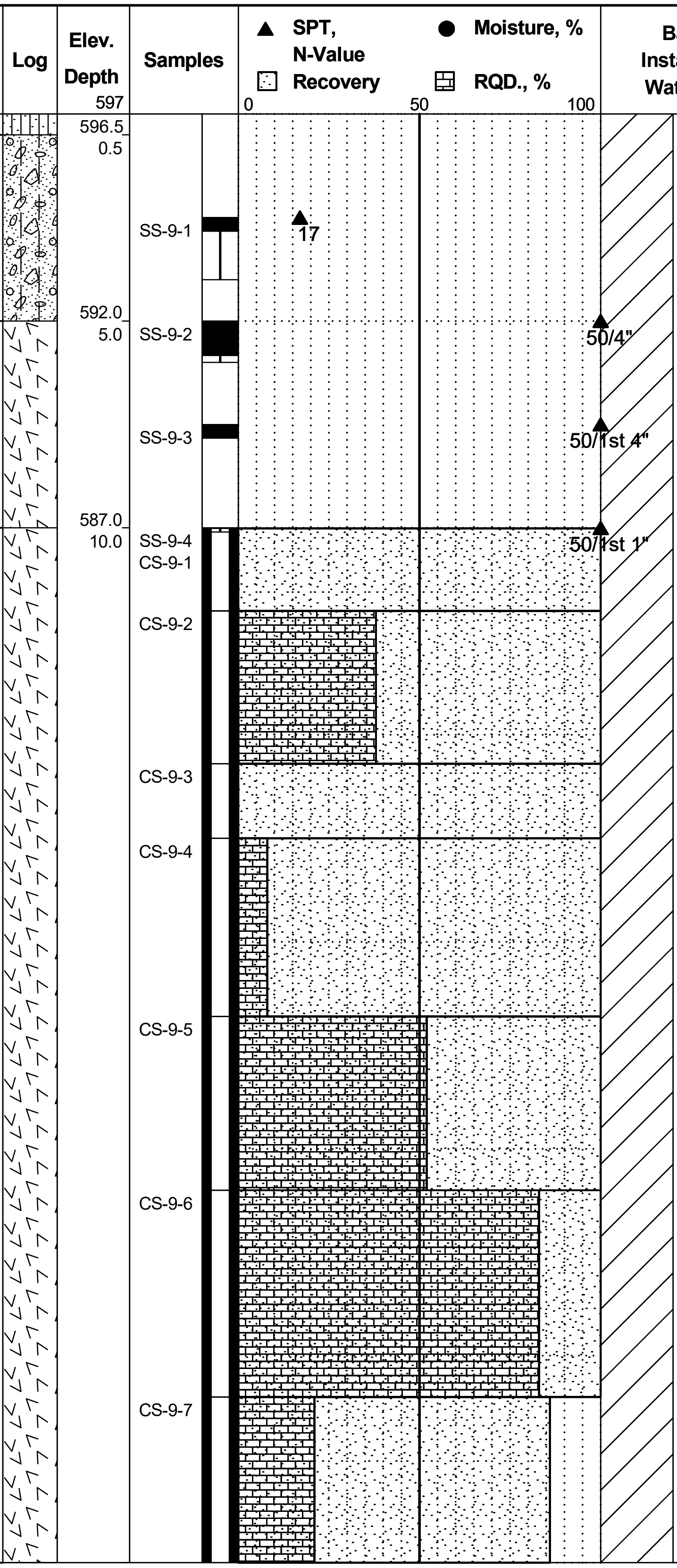
Foundation Engineering, Inc.

Boring Log: BH- 8

East 40th Avenue Storage Tanks

Eugene, Oregon

Depth Feet	Soil and Rock Description and Comments	Elev. Depth 597	Samples	SPT, N-Value	Backfill/ Installations/ Water Table
	Soft to stiff sandy SILT, scattered organics (ML); brown, moist to wet, low to medium plasticity, fine sand, organics consist of fine roots, (topsoil).	596.5 0.5			Backfilled with bentonite chips
2	Medium dense silty sandy GRAVEL, some cobbles (GM); brown, moist to wet, low plasticity silt, fine to coarse sand, fine to coarse angular basaltic gravel, subangular to subrounded basaltic cobbles, (residual soil).	592.0 5.0	SS-9-1	17	
5	Strong (R4) BASALT; dark grey, slightly weathered, (Fisher Formation).	587.0 10.0	SS-9-2 SS-9-3	50/4" 50/1st 4"	
10	Strong to very strong (R4 to R5) BASALT; dark grey, slightly weathered, very close to close joints are planar to irregular, rough, open to closed, iron-stained, and silt-infilled, (Fisher Formation).		SS-9-4 CS-9-1 CS-9-2 CS-9-3 CS-9-4 CS-9-5 CS-9-6	50/1st 1"	
31	Some healed joints below ±31 feet.		CS-9-7		
33	Altered and secondary mineralization below ±32.8 feet with very close, irregular, rough to very rough, and open joints.				



Project No.: 2201086

Surface Elevation: 597.0 feet (Approx.)

Date of Boring: May 13, 2021

Foundation Engineering, Inc.

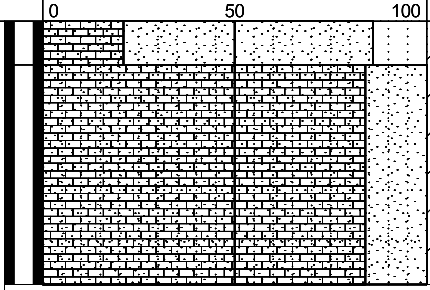
Boring Log: BH- 9

East 40th Avenue Storage Tanks

Eugene, Oregon

Depth Feet	Soil and Rock Description and Comments	Log	Elev. Depth	Samples
			562	
	Slickened joint at ±35 feet.		35.0	
36 37	Very weak to weak (R1 to R2) TUFF; dark reddish-brown, moderately weathered, close joints are irregular, rough, and open to closed, sand and some fine gravel sized volcanic clasts, (Fisher Formation).		559.4 37.6	CS-9-8
38 39 40	Medium strong to strong (R3 to R4) BASALT; dark grey, slightly weathered, close to moderately close joints are planar to irregular, rough to very rough, and open to closed, some calcite veining, (Fisher Formation).		556.7 40.3	
41	Very weak (R1) TUFF; dark reddish-brown, moderately weathered, close joints are irregular, rough, and open, sand-sized volcanic clasts. (Fisher Formation).		556.0 41.0	
	BOTTOM OF BORING			

▲ SPT, N-Value ● Moisture, %
☐ Recovery ▦ RQD., %



Backfill/ Installations/ Water Table

Project No.: 2201086

Surface Elevation: 597.0 feet (Approx.)

Date of Boring: May 13, 2021

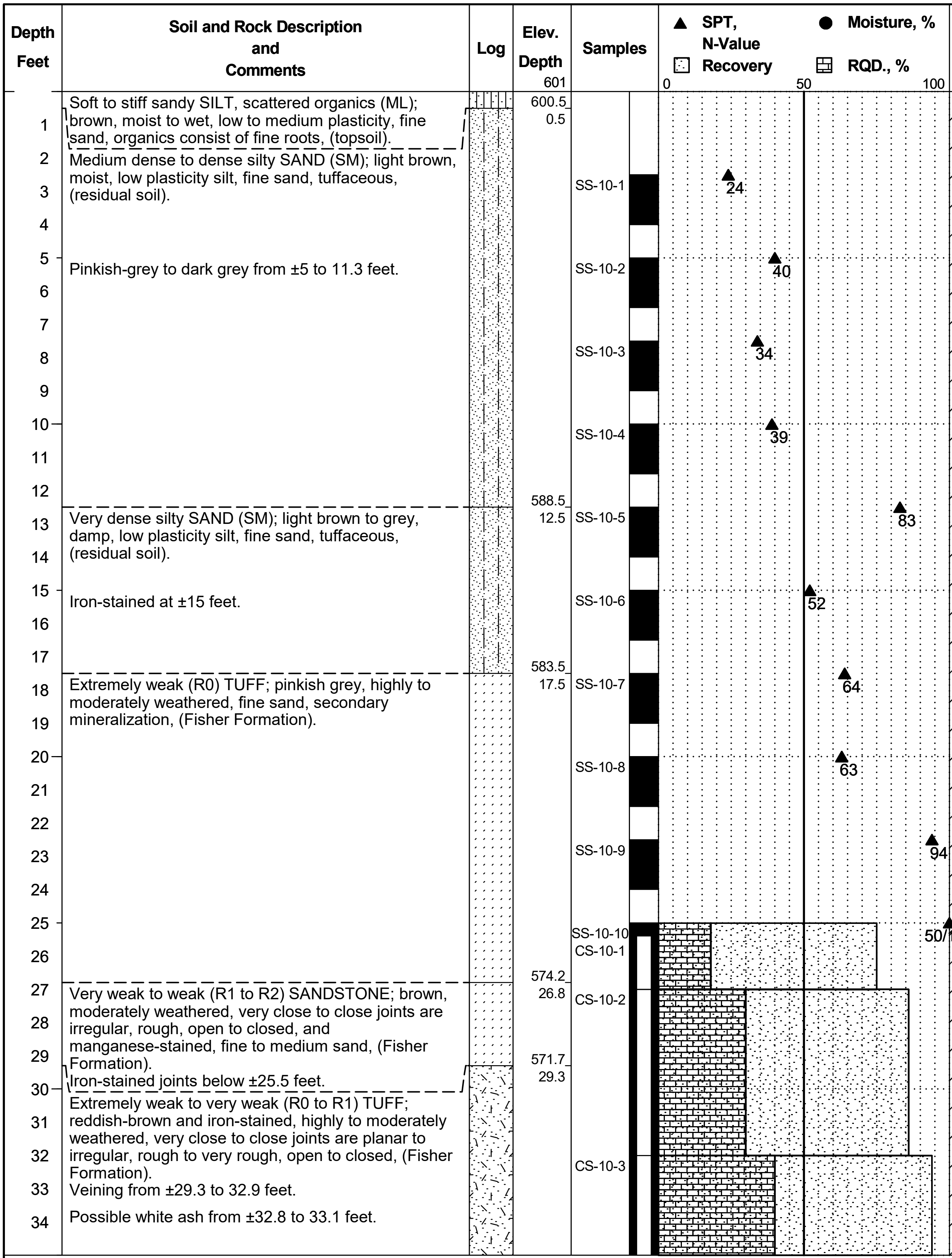
Foundation Engineering, Inc.

Boring Log: BH- 9

East 40th Avenue Storage Tanks

Eugene, Oregon

Soil and Rock Description and Comments	Elev. Depth 601	Samples	SPT, N-Value	Backfill/ Installations/ Water Table
Soft to stiff sandy SILT, scattered organics (ML); brown, moist to wet, low to medium plasticity, fine sand, organics consist of fine roots, (topsoil).	600.5 0.5			Backfilled with bentonite chips
Medium dense to dense silty SAND (SM); light brown, moist, low plasticity silt, fine sand, tuffaceous, (residual soil). Pinkish-grey to dark grey from ±5 to 11.3 feet.	588.5 12.5	SS-10-1 SS-10-2 SS-10-3 SS-10-4	24 40 34 39	
Very dense silty SAND (SM); light brown to grey, damp, low plasticity silt, fine sand, tuffaceous, (residual soil). Iron-stained at ±15 feet.	583.5 17.5	SS-10-5 SS-10-6	83 52	
Extremely weak (R0) TUFF; pinkish grey, highly to moderately weathered, fine sand, secondary mineralization, (Fisher Formation).	574.2 26.8	SS-10-7 SS-10-8 SS-10-9 SS-10-10 CS-10-1	64 63 94 50/1st 5"	
Very weak to weak (R1 to R2) SANDSTONE; brown, moderately weathered, very close to close joints are irregular, rough, open to closed, and manganese-stained, fine to medium sand, (Fisher Formation). Iron-stained joints below ±25.5 feet.	571.7 29.3	CS-10-2		
Extremely weak to very weak (R0 to R1) TUFF; reddish-brown and iron-stained, highly to moderately weathered, very close to close joints are planar to irregular, rough to very rough, open to closed, (Fisher Formation). Veining from ±29.3 to 32.9 feet. Possible white ash from ±32.8 to 33.1 feet.		CS-10-3		



Project No.: 2201086

Surface Elevation: 601.0 feet (Approx.)

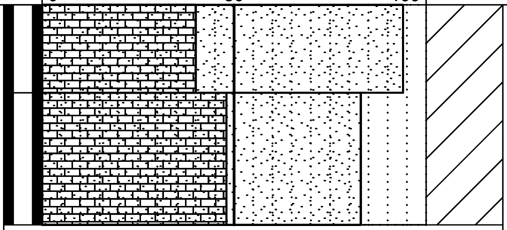
Date of Boring: May 13, 2021

Foundation Engineering, Inc.

Boring Log: BH-10

East 40th Avenue Storage Tanks

Eugene, Oregon

Depth Feet	Soil and Rock Description and Comments	Log	Elev. Depth 566	Samples	▲ SPT, N-Value ⊡ Recovery ● Moisture, % ⊞ RQD., % 0 50 100	Backfill/ Installations/ Water Table
36 37 38 39	Fine to coarse sand-sized volcanic clasts from ±32.8 to 35 feet. Dark reddish-brown from ±35.4 to 37.4 feet and below ±38.4 feet. Brown from ±37.5 to 38.4 feet. Blue-grey from ±38.25 to 38.4 feet. Slickened joint at ±39 feet.			CS-10-4		
40	BOTTOM OF BORING		561.0 40.0			

Project No.: 2201086

Surface Elevation: 601.0 feet (Approx.)

Date of Boring: May 13, 2021

Foundation Engineering, Inc.

Boring Log: BH-10

East 40th Avenue Storage Tanks

Eugene, Oregon

Photo 1B. BH-1 from 10.0 to 23.3 ft - Box 1 of 5

Photo 2B. BH-1 from 23.3 to 30.9 ft - Box 2 of 5

Photo 3B. BH-1 from 30.9 to 38.5 ft - Box 3 of 5

Photo 4B. BH-1 from 38.5 to 46.3 ft - Box 4 of 5

Photo 5B. BH-1 from 46.3 to 48.5 ft - Box 5 of 5

Photo 6B. BH-2 from 12.5 to 21.3 ft - Box 1 of 5

Photo 7B. BH-2 from 21.3 to 30.7 ft - Box 2 of 5

Photo 8B. BH-2 from 30.7 to 39.8 ft – Box 3 of 5

Photo 9B. BH-2 from 39.8 to 48.8 ft - Box 4 of 5

Photo 10B. BH-2 from 48.8 to 52.0 ft - Box 5 of 5

Photo 11B. BH-3 from 7.5 to 17.1 ft - Box 1 of 5

Photo 12B. BH-3 from 17.1 to 25.4 ft - Box 2 of 5

Photo 13B. BH-3 from 25.4 to 33.5 ft - Box 3 of 5

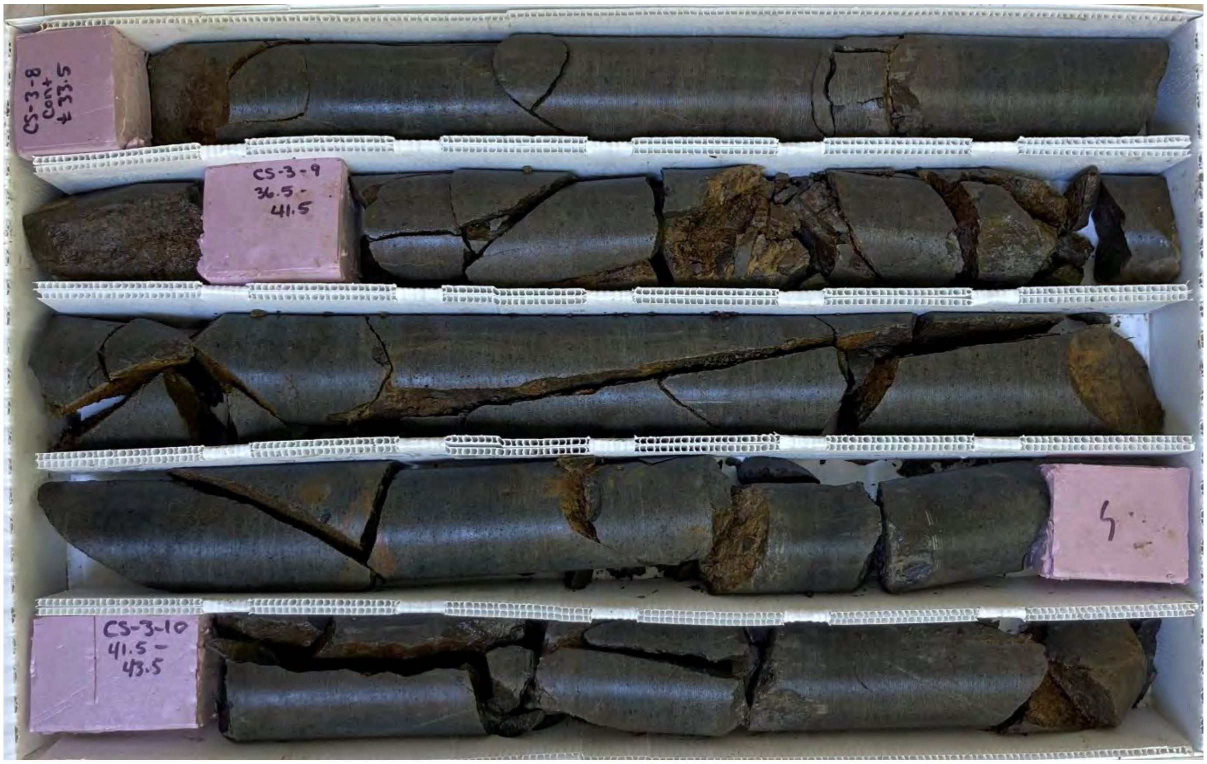


Photo 14B. BH-3 from 33.5 to 43.5 ft - Box 4 of 5

Photo 15B. BH-3 from 43.5 to 50.0 ft - Box 5 of 5

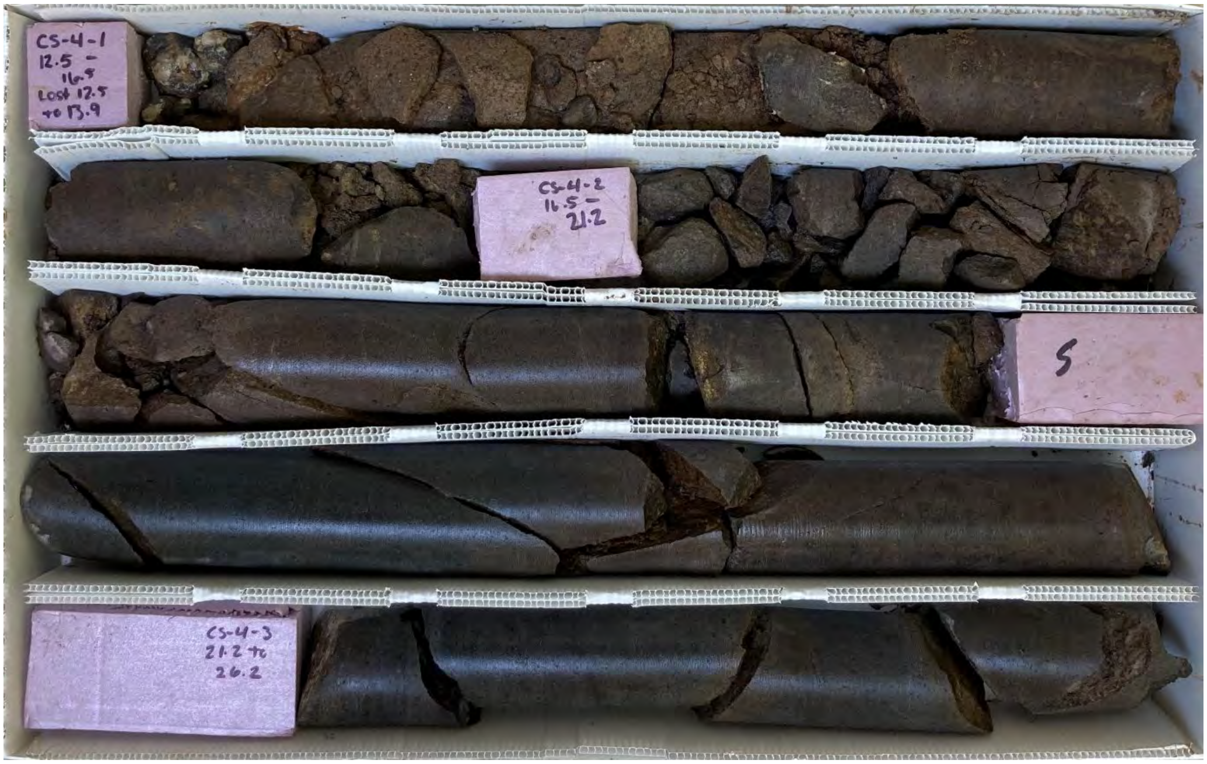


Photo 16B. BH-4 from 12.5 to 23.0 ft - Box 1 of 3

Photo 17B. BH-4 from 23.0 to 31.5 ft - Box 2 of 3

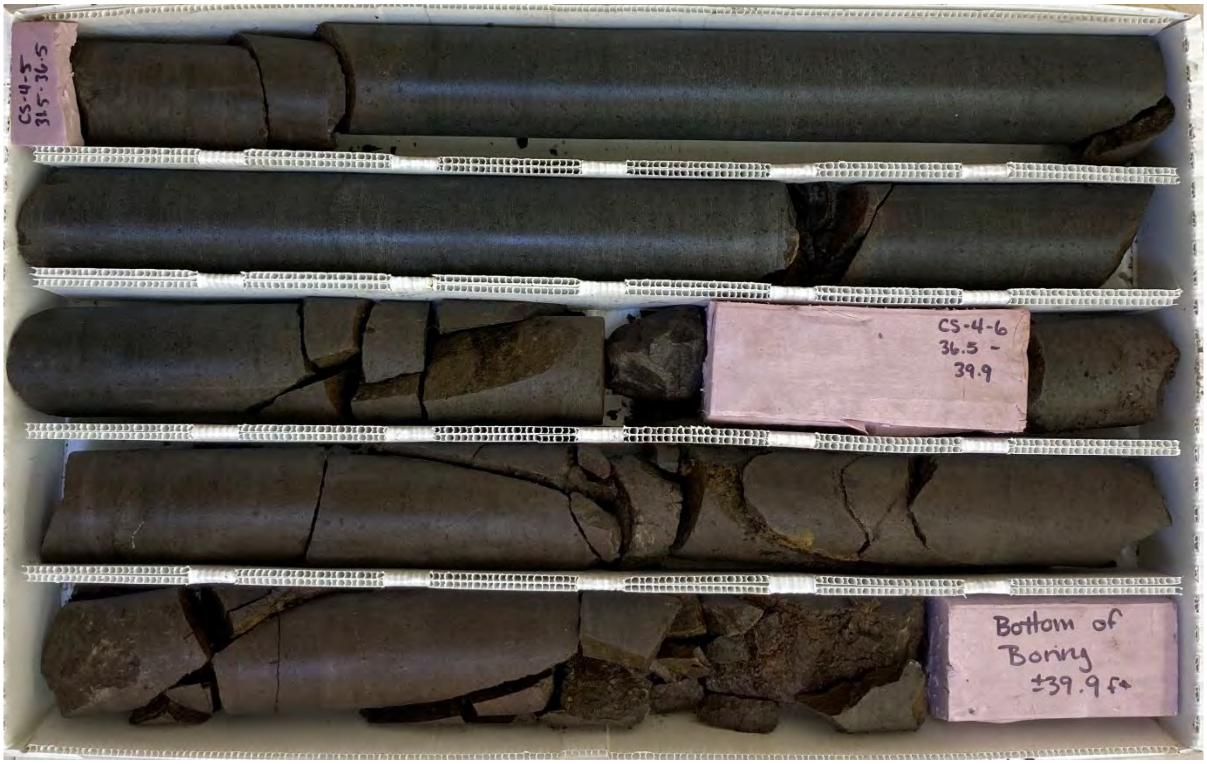


Photo 18B. BH-4 from 31.5 to 39.9 ft - Box 3 of 3

Photo 19B. BH-5 from 0.5 to 11.8 ft - Box 1 of 4

Photo 20B. BH-5 from 11.8 to 20.0 ft - Box 2 of 4

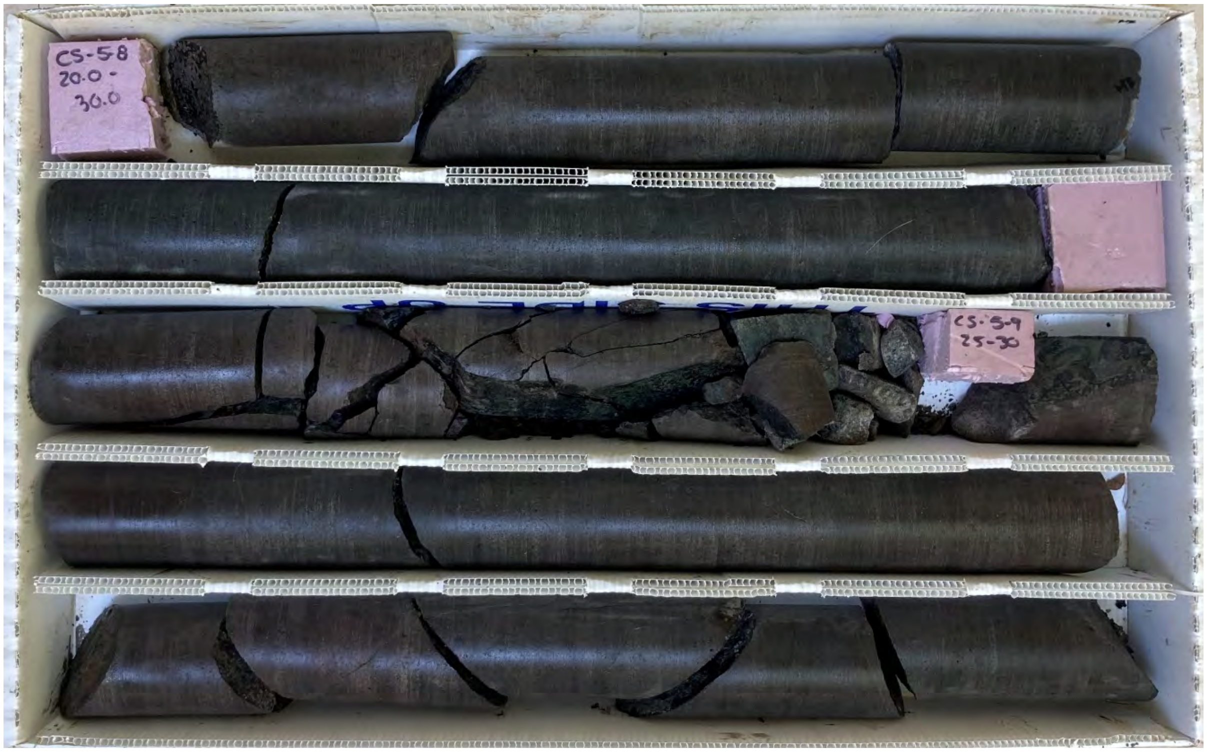


Photo 21B. BH-5 from 20.0 to 29.0 ft - Box 3 of 4

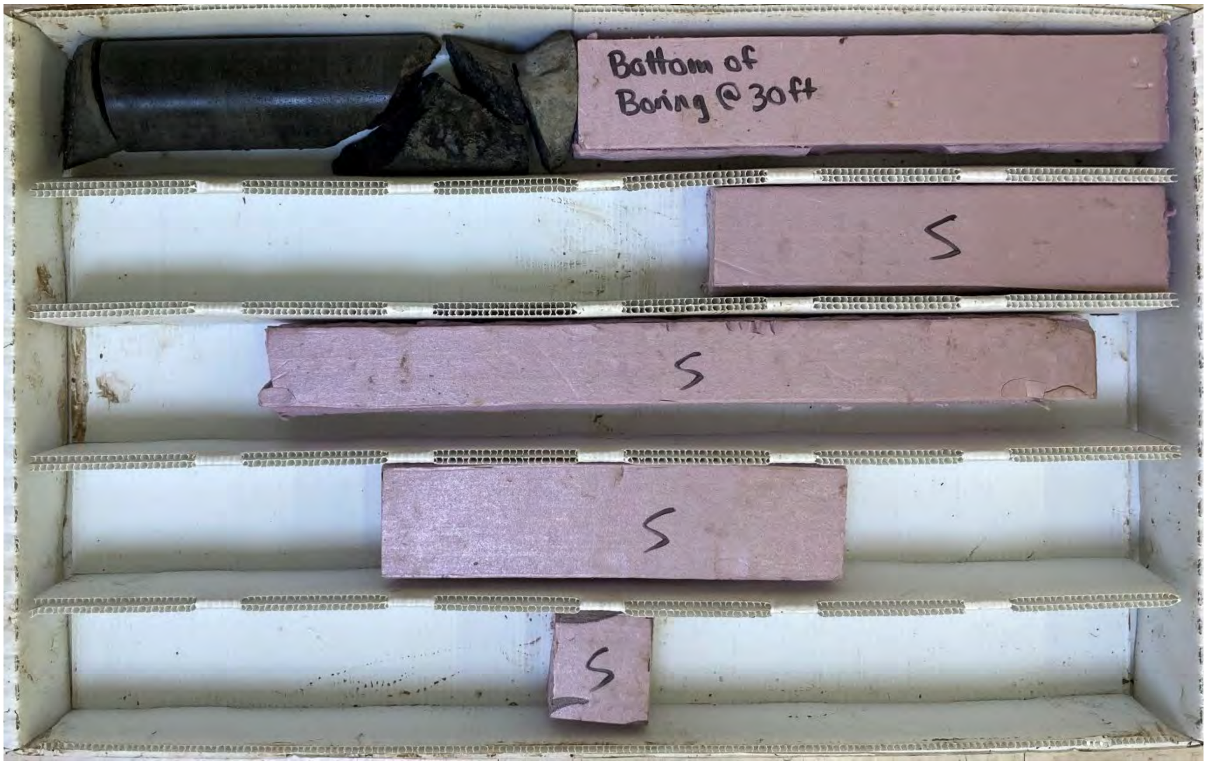


Photo 22B. BH-5 from 29.0 to 30.0 ft - Box 4 of 4

Photo 23B. BH-6 from 5.0 to 11.8 ft - Box 1 of 5

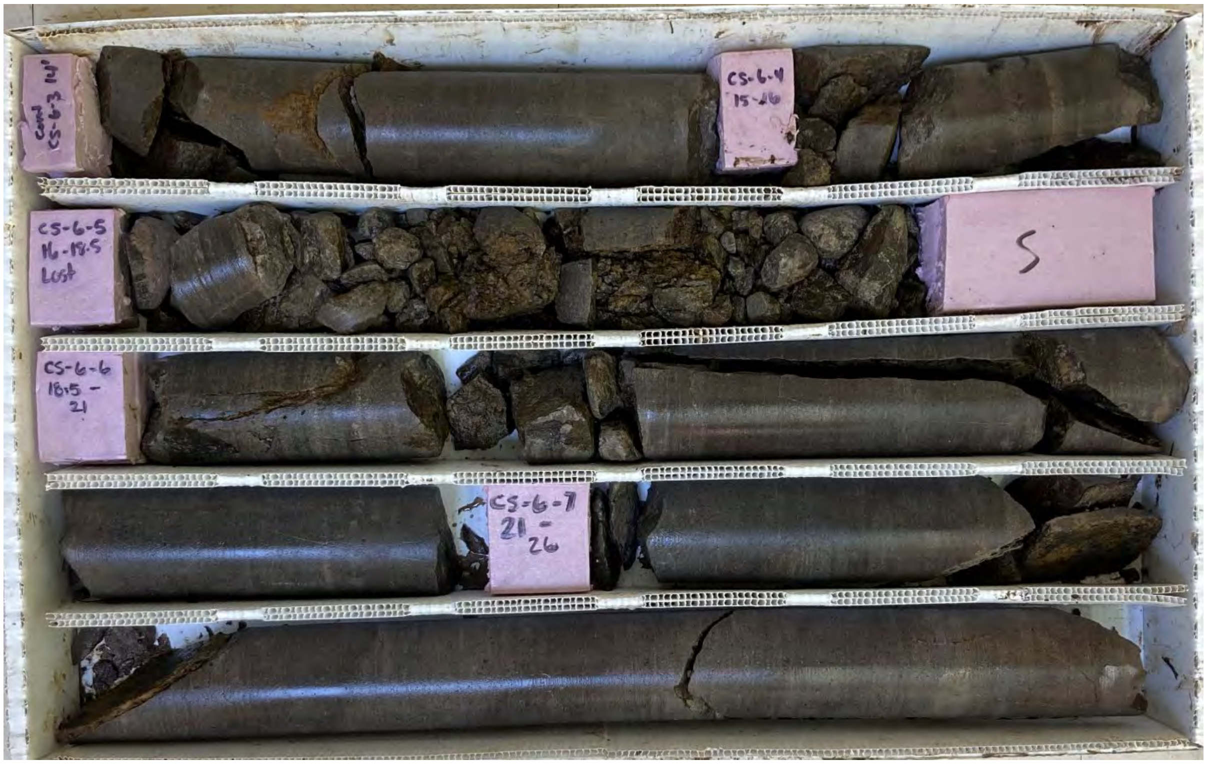


Photo 24B. BH-6 from 11.8 to 23.2 ft - Box 2 of 5

Photo 25B. BH-6 from 23.2 to 31.0 ft - Box 3 of 5

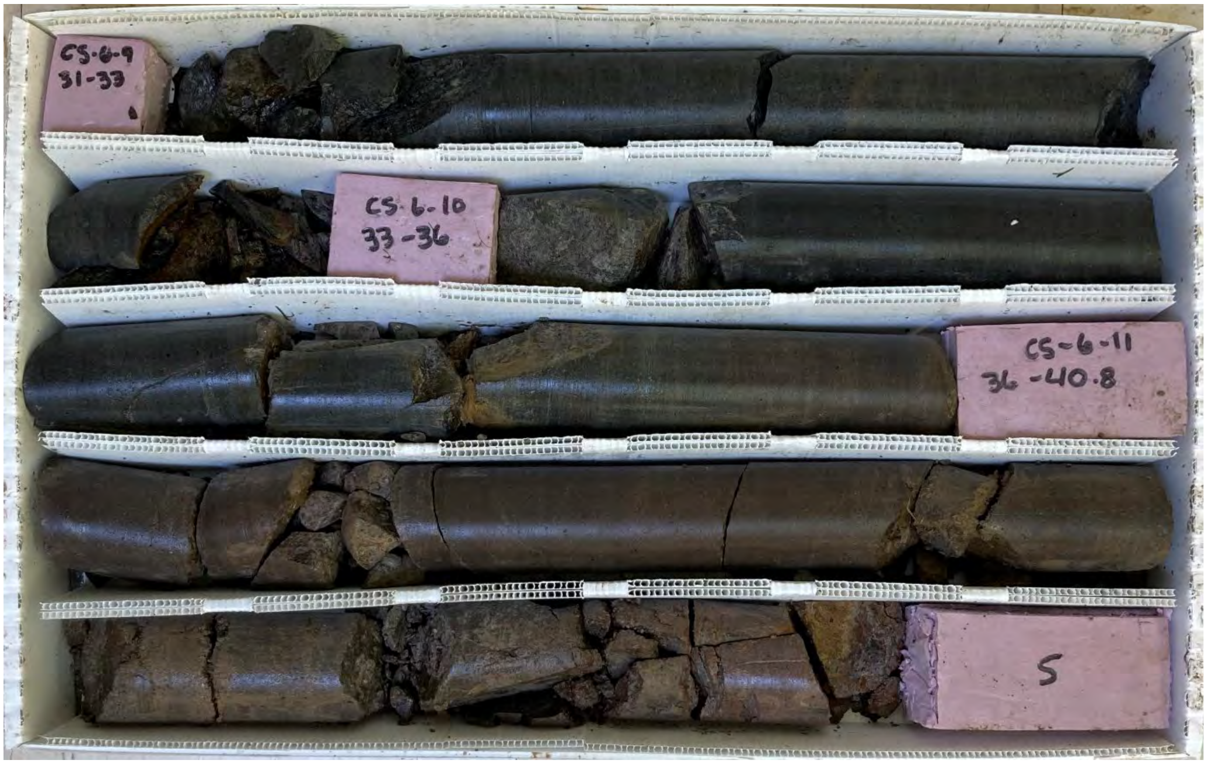


Photo 26B. BH-6 from 31.0 to 39.4 ft - Box 4 of 5

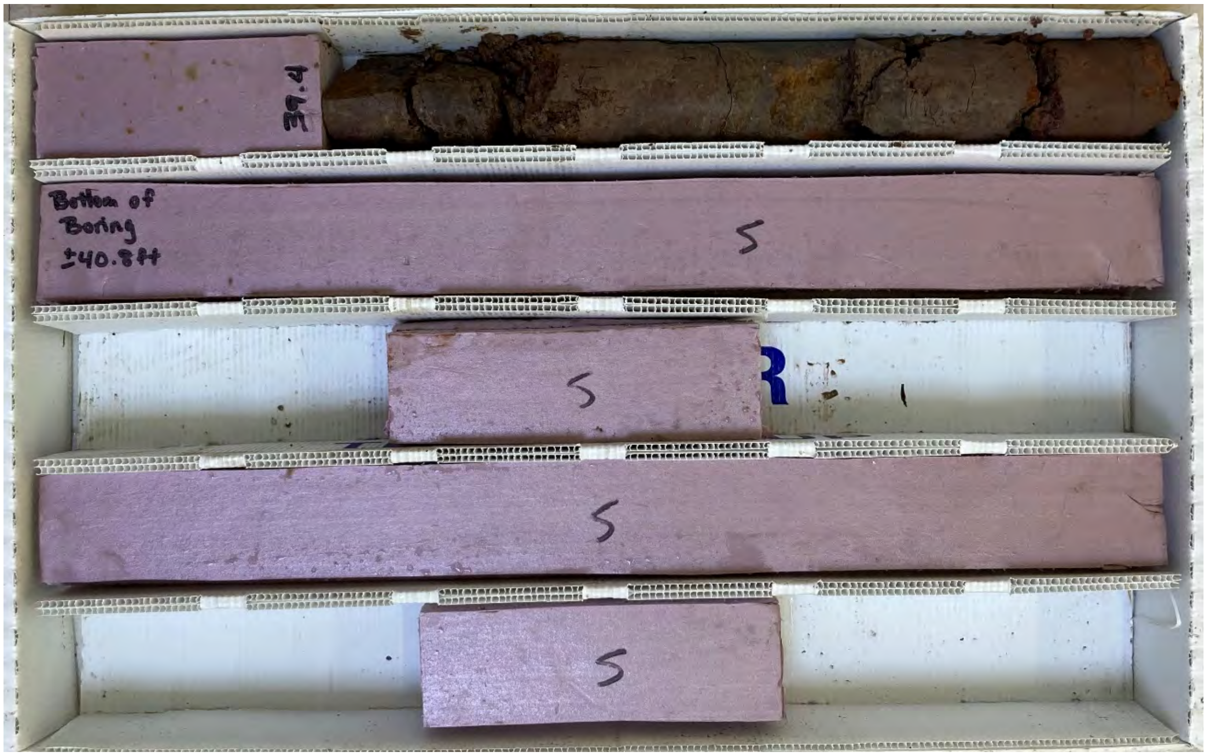


Photo 27B. BH-6 from 39.4 to 40.8 ft - Box 5 of 5

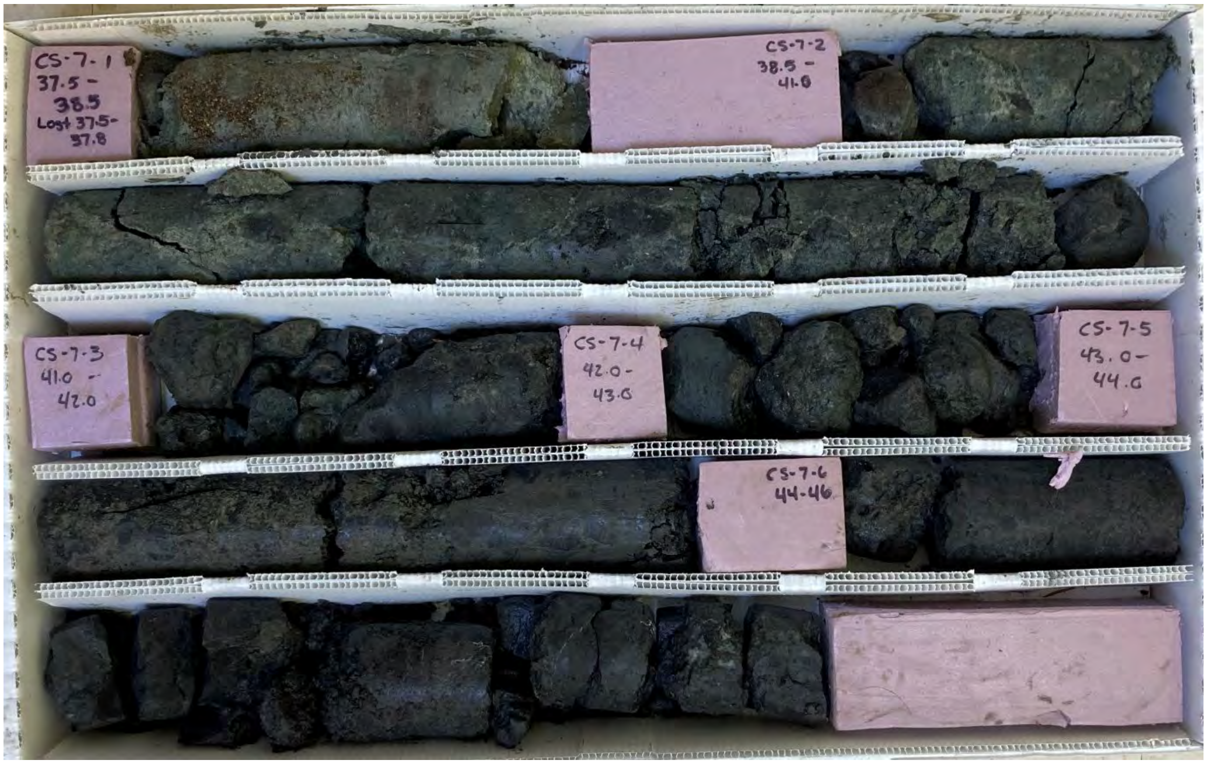


Photo 28B. BH-7 from 37.5 to 46.0 ft - Box 1 of 2

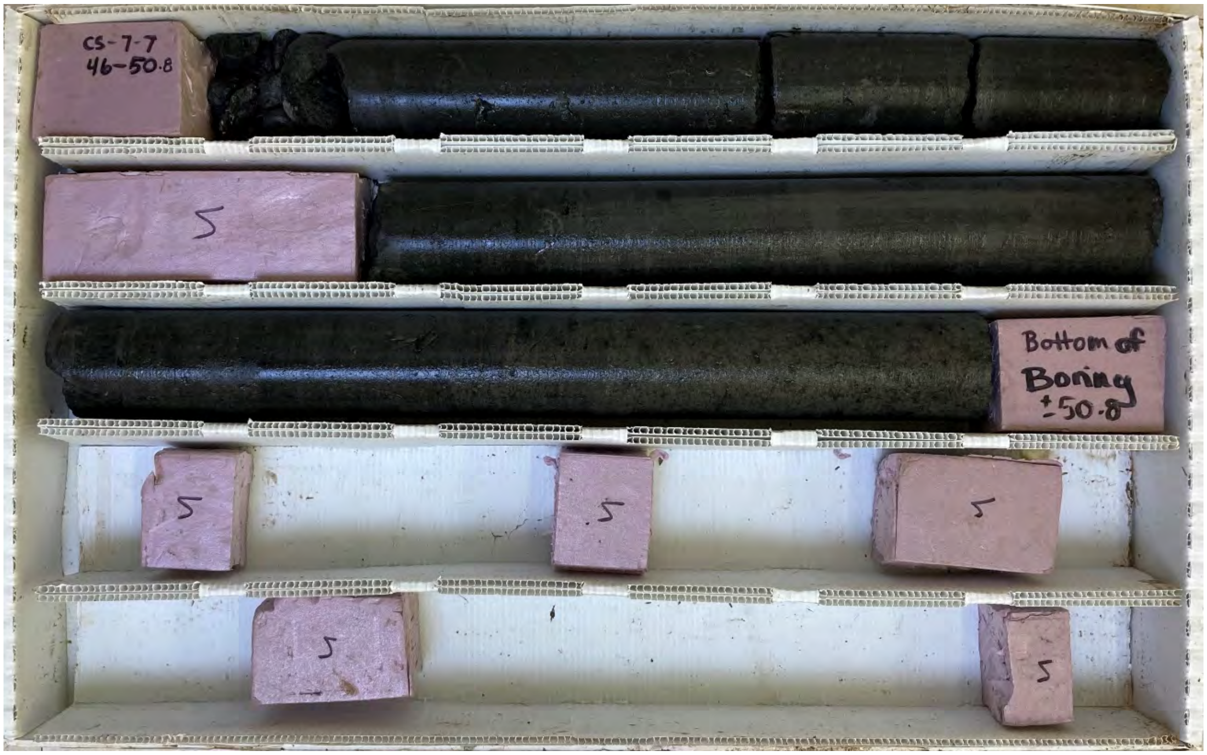


Photo 29B. BH-7 from 46.0 to 50.8 ft - Box 2 of 2

Photo 30B. BH-8 from 12.0 to 22.0 ft - Box 1 of 4

Photo 31B. BH-8 from 22.0 to 32.0 ft - Box 2 of 4

Photo 32B. BH-8 from 32.0 to 41.1 ft - Box 3 of 4

Photo 33B. BH-8 from 41.1 to 47.0 ft - Box 4 of 4

Photo 34B. BH-9 from 10.0 to 18.5 ft - Box 1 of 4

Photo 35B. BH-9 from 18.5 to 27.1 ft - Box 2 of 4

Photo 36B. BH-9 from 27.1 to 36.0 ft - Box 3 of 4

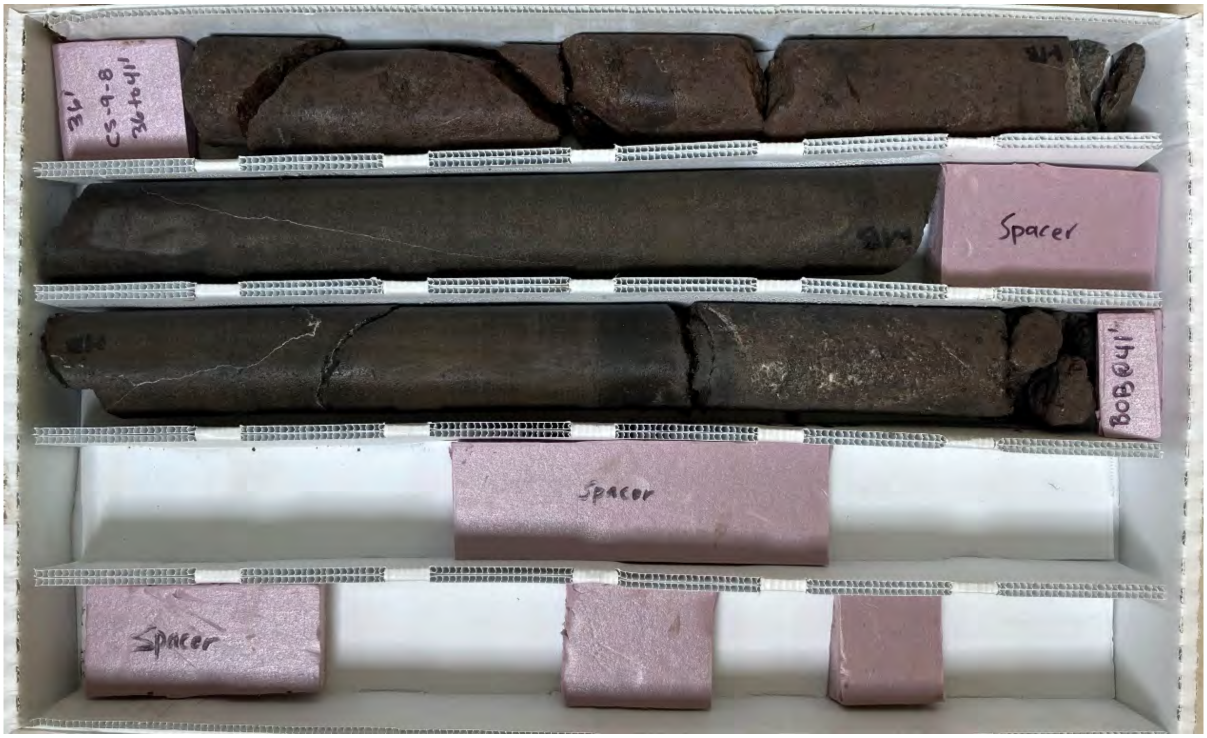


Photo 37B. BH-9 from 36.0 to 41.1 ft - Box 4 of 4

Photo 38B. BH-10 from 25.0 to 35.0 ft - Box 1 of 2

Photo 39B. BH-10 from 35.0 to 40.0 ft - Box 2 of 2

Appendix C

Laboratory Testing

Foundation Engineering, Inc.
Professional Geotechnical Services

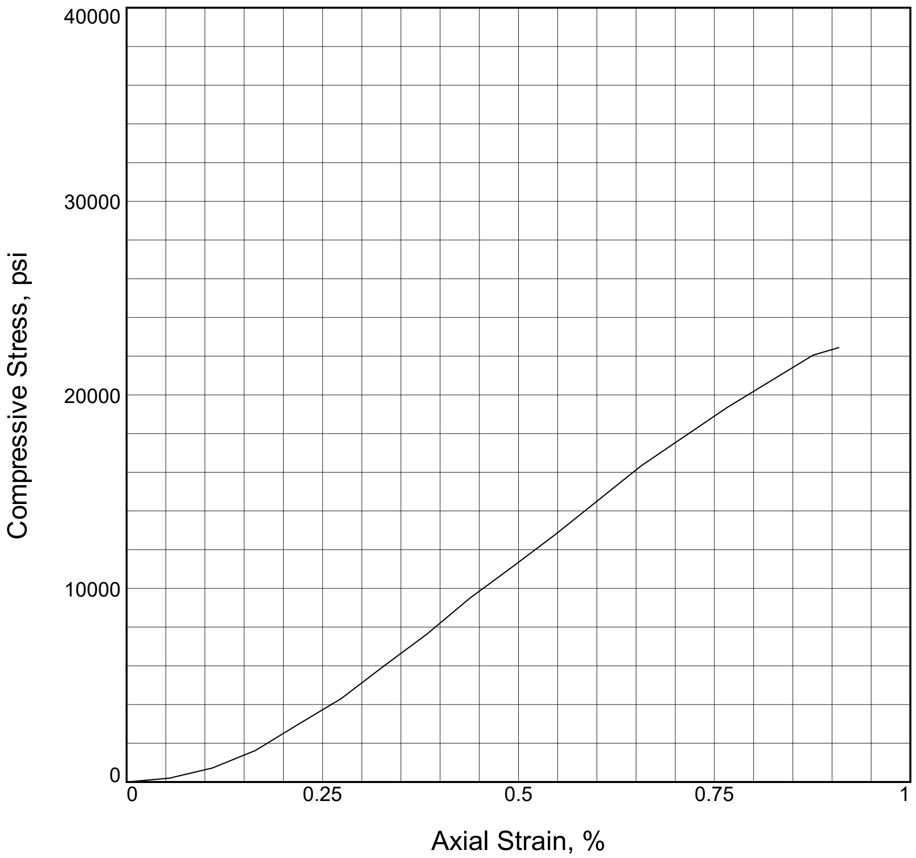
Table 1C. Moisture Contents (ASTM D 2216)

Sample Number	Sample Depth (ft)	Moisture Content (percent)
SS-7-1	2.5 – 4.0	39.8
SS-7-2	5.0 – 6.5	40.5
SS-7-4	10.0 – 11.5	33.3
SS-7-6	15.0 – 16.5	35.5
SS-7-7	17.5 – 19.0	40.9
SS-7-9	22.5 – 24.0	47.0
SS-7-11	27.5 – 29.0	35.5
SS-7-12	30.0 - 31.5	44.6

Table 2C. Summary of Unconfined Compressive Strengths

Boring	Sample Number	Sample Depth (ft)	Rock Description	Wet Density (pcf)	Unconfined Compressive Strength (psi)
BH-1	CS-1-1	11.3 – 11.7	R5 BASALT	172.0	22,922
BH-1	CS-1-5	21.3 - 21.7	R5 BASALT	176.1	18,854
BH-1	CS-1-6	25.8 - 26.2	R5 BASALT	174.9	22,444
BH-2	CS-2-4	23.5 - 23.9	R3 - R4 BASALT	174.4	8,216
BH-3	CS-3-1	7.7 - 8.1	R4 BASALT	175.5	10,623
BH-3	CS-3-6	24.3 - 24.7	R5 BASALT	176.3	22,753
BH-3	CS-3-7	27.1 - 27.5	R5 BASALT	176.7	26,388
BH-3	CS-3-8	32.0 - 32.4	R5 BASALT	177.0	24,092
BH-4	CS-4-3	23.1 - 23.5	R5 BASALT	175.5	23,395
BH-4	CS-4-4	28.4 - 28.8	R4 - R5 BASALT	177.6	16,853
BH-4	CS-4-5	33.6 - 34.0	R5 BASALT	178.3	24,787
BH-5	CS-5-2	3.8 - 4.2	R5 BASALT	176.7	26,357
BH-5	CS-5-5	11.1 - 11.5	R4 BASALT	173.8	10,320
BH-5	CS-5-8	20.7 - 21.1	R5 BASALT	175.7	23,548
BH-6	CS-6-2	8.1 - 8.5	R5 BASALT	175.7	20,029
BH-6	CS-6-3	11.1 - 11.5	R4 - R5 BASALT	175.0	16,049
BH-6	CS-6-7	24.1 - 24.5	R5 BASALT	176.3	19,948
BH-6	CS-6-8	27.5 - 27.9	R5 BASALT	175.9	19,677
BH-7	CS-7-7	50.3 - 50.7	R4 - R5 BASALT	173.2	16,398
BH-9	CS-9-2	13.8 – 14.3	R3 BASALT	173.5	4,999

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, psi	22444			
Failure strain, %	0.9			
Strain rate, %/min.	0.25			
Water content, %	1.4			
Wet density, pcf	174.9			
Dry density, pcf	172.5			
Specimen diameter, in.	2.38			
Specimen height, in.	4.57			
Height/diameter ratio	1.92			

Description: BASALT

Type: Rock Core

Project No.: 2206001-638

Date Sampled:

Remarks:
Weak jointing reduced viability of sample.
L/D Ratio - 1:1.923

Figure 1C

Client: Foundation Engineering, Inc. (Project No.:2201086)

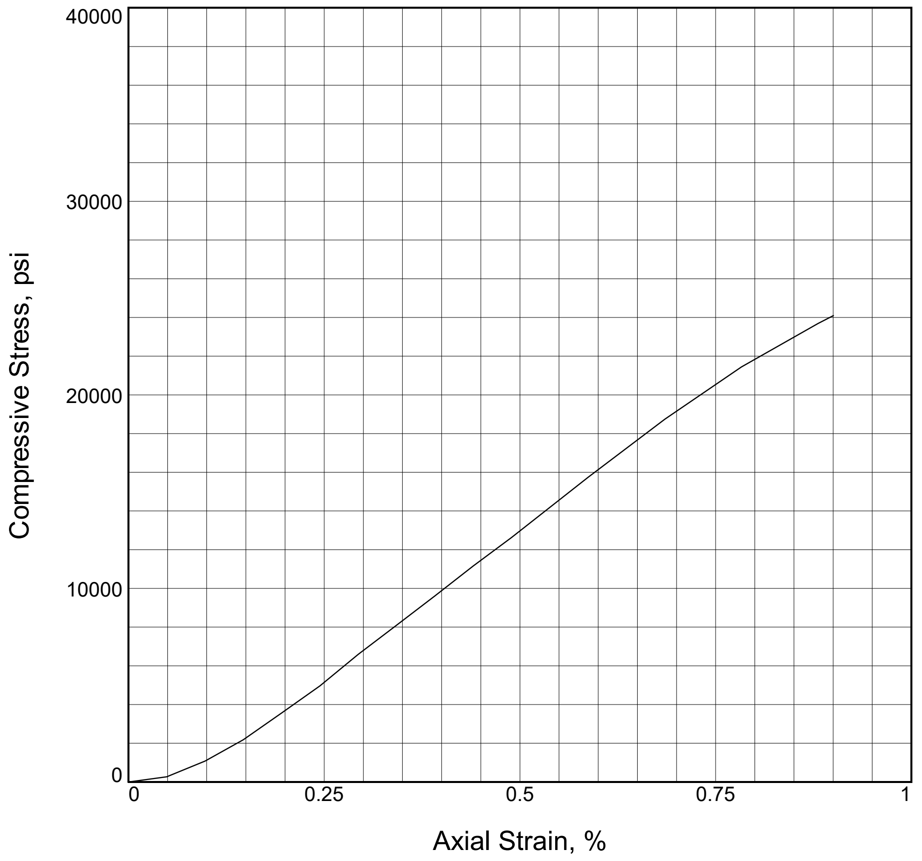
Project: East 40th Avenue Storage Tanks

Source of Sample: 8077 **Depth:** 25.8'-26.2'

Sample Number: CS-1-6

UNCONFINED COMPRESSION TEST
FEI Testing & Inspection, Inc.
Corvallis, OR

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, psi	24092			
Failure strain, %	0.9			
Strain rate, %/min.	0.25			
Water content, %	1.0			
Wet density, pcf	177.0			
Dry density, pcf	175.3			
Specimen diameter, in.	2.37			
Specimen height, in.	5.11			
Height/diameter ratio	2.15			

Description: BASALT

Type: Rock Core

Project No.: 2206001-638
Date Sampled:
Remarks:

Client: Foundation Engineering, Inc. (Project No.: 2201086)

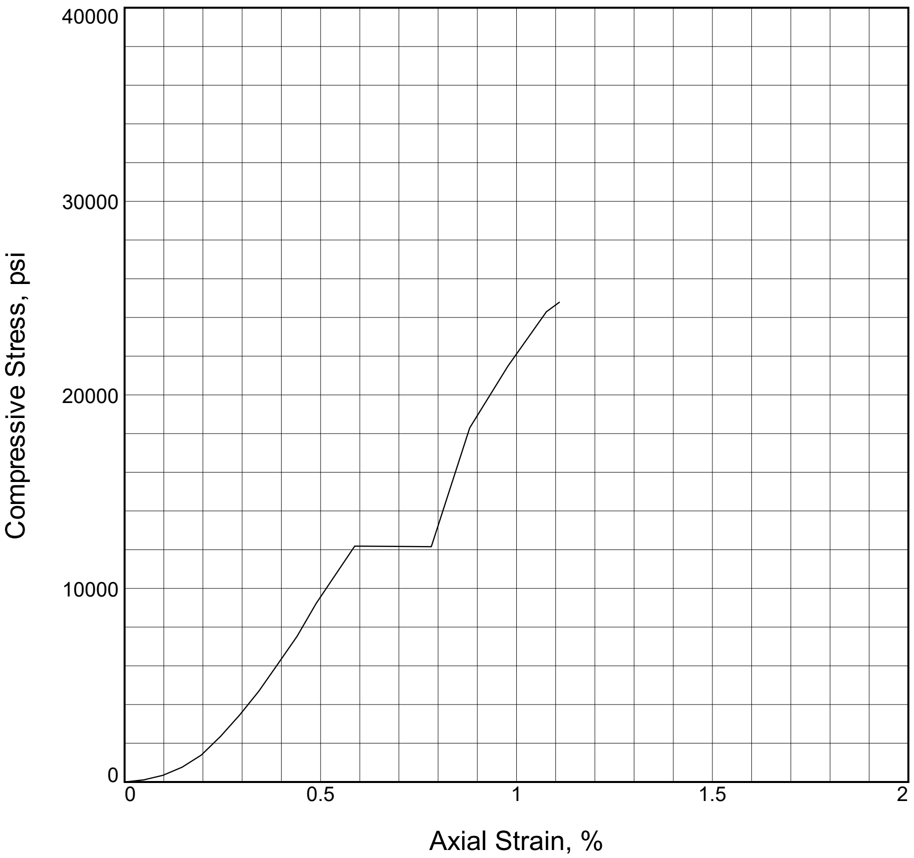
Project: East 40th Avenue Storage Tanks

Source of Sample: 8077 **Depth:** 32.0'-32.4'
Sample Number: CS-3-8

UNCONFINED COMPRESSION TEST
FEI Testing & Inspection, Inc.
Corvallis, OR

Figure 2C

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, psi	24787			
Failure strain, %	1.1			
Strain rate, %/min.	0.25			
Water content, %	0.7			
Wet density, pcf	178.3			
Dry density, pcf	177.1			
Specimen diameter, in.	2.38			
Specimen height, in.	5.11			
Height/diameter ratio	2.15			

Description: BASALT

Type: Rock Core

Project No.: 2206001-638

Date Sampled:

Remarks:
Initial fracture caused dial guage to jump approximately 0.0050 in., from 0.0350-0.0400. Unable to collect data from this portion of test.

Figure 3C

Client: Foundation Engineering, Inc. (Project No.: 2201086)

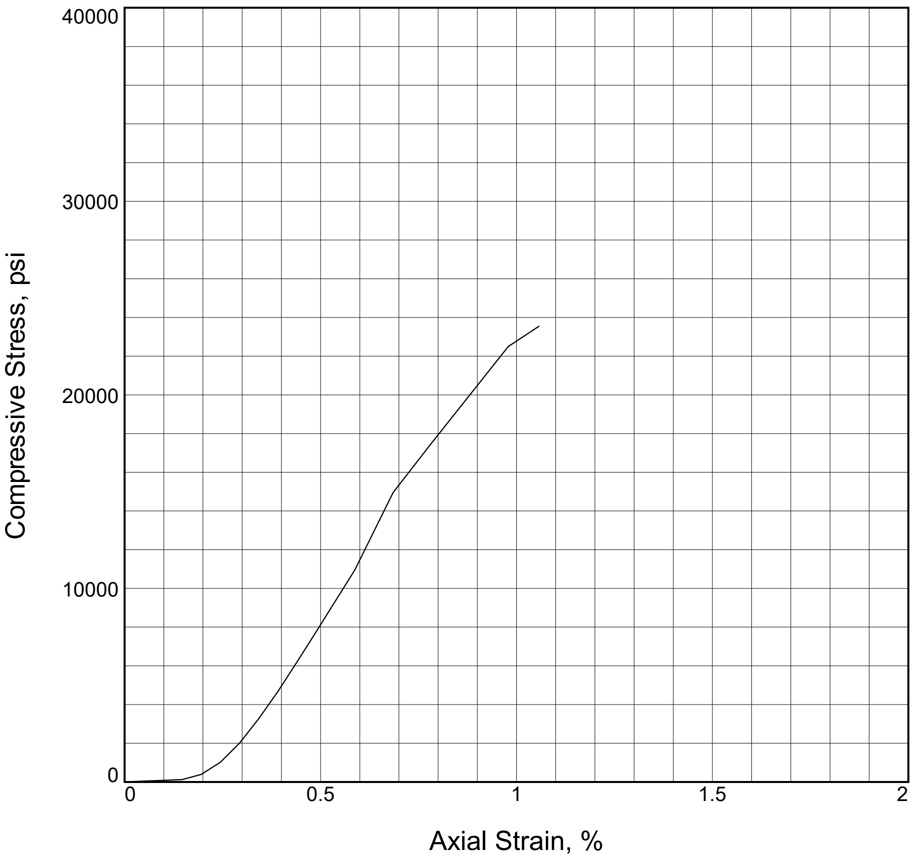
Project: East 40th Avenue Storage Tanks

Source of Sample: 8077 **Depth:** 33.6'-34.0'

Sample Number: CS-4-5

UNCONFINED COMPRESSION TEST
FEI Testing & Inspection, Inc.
Corvallis, OR

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, psi	23548			
Failure strain, %	1.1			
Strain rate, %/min.	0.25			
Water content, %	1.2			
Wet density, pcf	175.7			
Dry density, pcf	173.6			
Specimen diameter, in.	2.38			
Specimen height, in.	5.11			
Height/diameter ratio	2.15			

Description: BASALT

Type: Rock Core

Project No.: 2206001-638
Date Sampled: 11/09/20-11/14/20
Remarks:

Figure 4C

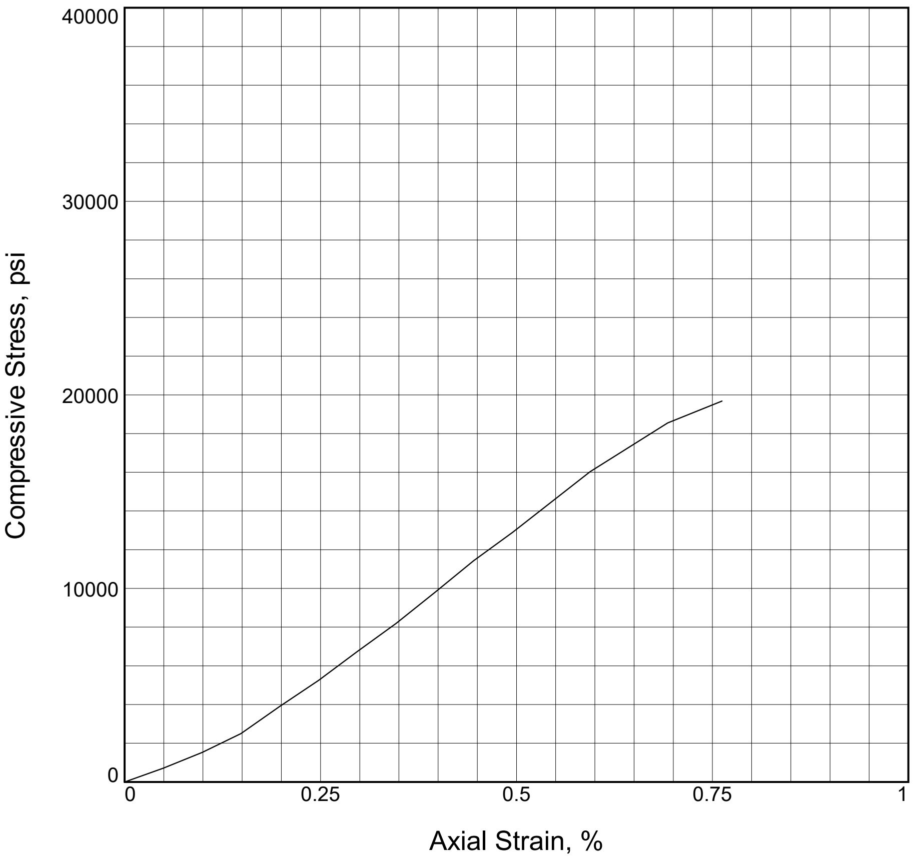
Client: Foundation Engineering, Inc. (Project No.: 2201086)

Project: East 40th Avenue Storage Tanks

Source of Sample: 8077 **Depth:** 20.7'-21.1'
Sample Number: CS-5-8

UNCONFINED COMPRESSION TEST
FEI Testing & Inspection, Inc.
Corvallis, OR

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, psi	19677			
Failure strain, %	0.8			
Strain rate, %/min.	0.25			
Water content, %	0.9			
Wet density, pcf	175.9			
Dry density, pcf	174.3			
Specimen diameter, in.	2.38			
Specimen height, in.	5.05			
Height/diameter ratio	2.12			

Description: BASALT

Type: Rock Core

Project No.: 2206001-638
Date Sampled:
Remarks:

Client: Foundation Engineering, Inc. (Project No.: 2201086)

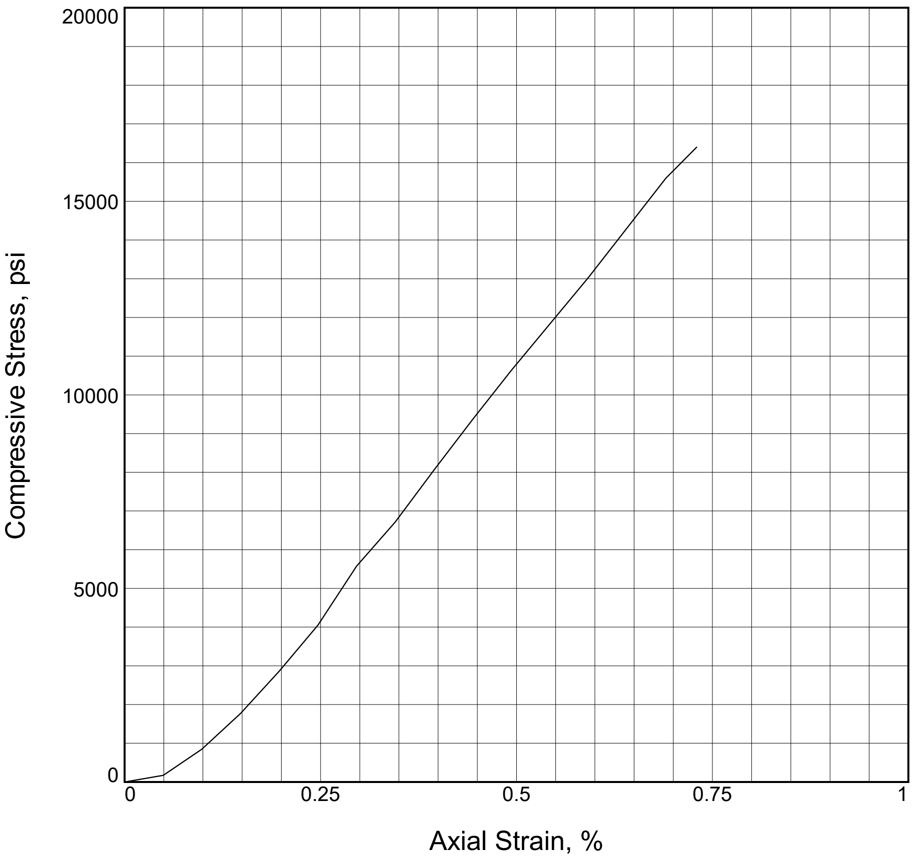
Project: East 40th Avenue Storage Tanks

Source of Sample: 8077 **Depth:** 27.5'-27.9'
Sample Number: CS-6-8

UNCONFINED COMPRESSION TEST
FEI Testing & Inspection, Inc.
Corvallis, OR

Figure 5C

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, psi	16398			
Failure strain, %	0.7			
Strain rate, %/min.	0.25			
Water content, %	2.3			
Wet density, pcf	173.2			
Dry density, pcf	169.4			
Specimen diameter, in.	2.38			
Specimen height, in.	5.07			
Height/diameter ratio	2.13			

Description: BASALT

Type: Rock Core

Project No.: 2206001-638
Date Sampled:
Remarks:

Client: Foundation Engineering, Inc. (Project No.: 2201086)

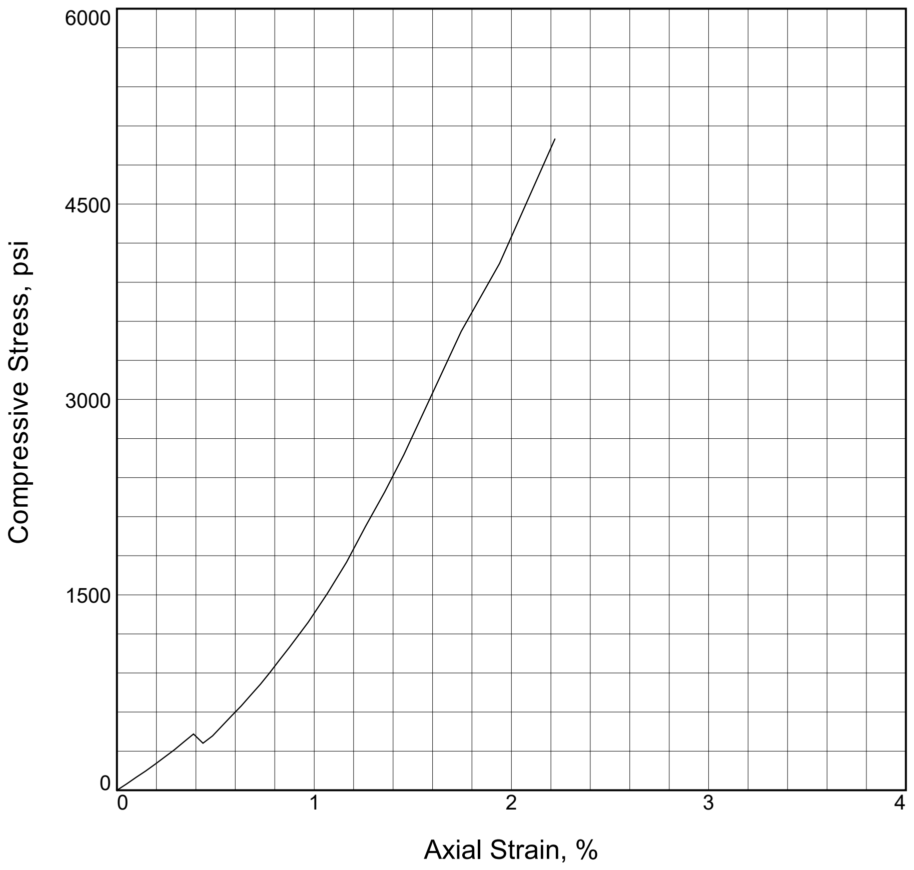
Project: East 40th Avenue Storage Tanks

Source of Sample: 8077 **Depth:** 50.3'-50.7'
Sample Number: CS-7-7

UNCONFINED COMPRESSION TEST
FEI Testing & Inspection, Inc.
Corvallis, OR

Figure 6C

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, psi	4999.15			
Failure strain, %	2.2			
Strain rate, %/min.	0.25			
Water content, %	1.0			
Wet density, pcf	173.5			
Dry density, pcf	171.7			
Specimen diameter, in.	2.40			
Specimen height, in.	5.16			
Height/diameter ratio	2.15			

Description: BASALT

Type: Rock Core

Project No.: 2216001-606

Date Sampled:

Remarks:
sample failed on existing jointing

Client: Foundation Engineering, Inc. (Project No.: 2201086)

Project: East 40th Avenue Storage Tanks

Source of Sample: 8200 **Depth:** 13.8'-14.3'

Sample Number: CS-9-2

UNCONFINED COMPRESSION TEST
FEI Testing & Inspection, Inc.
Corvallis, OR

Figure 7C

Appendix D

Seismic Hazard Study

Foundation Engineering, Inc.
Professional Geotechnical Services

SEISMIC HAZARD STUDY

EAST 40TH AVENUE STORAGE TANKS
EUGENE, OREGON

INTRODUCTION

This seismic hazard study was completed to identify potential geologic and seismic hazards and evaluate the effect those hazards may have on the proposed project. The study fulfills the requirements presented in the 2019 Oregon Structural Specialty Code (OSSC), Section 1803 for site-specific seismic hazard reports for essential and hazardous facilities and major and special-occupancy structures (OSSC, 2019).

The following sections provide a discussion of the local and regional geology, seismic and tectonic setting, earthquakes, and seismic hazards. A detailed discussion of the subsurface conditions at the project location, including exploration logs, is provided in the main report.

LITERATURE REVIEW

We reviewed available geologic, seismic, and hazard publications and maps to characterize the local and regional geology and evaluate relative seismic hazards at the site. Information from geotechnical and seismic hazard investigations previously conducted by others at the site and by Foundation Engineering in the surrounding area were also reviewed.

Regional Geology

The site is located within the central Southern Willamette Valley, which is a broad, north-south-trending basin separating the Coast Range to the west from the Cascade Range to the east.

At the western margin of Oregon is the Cascadia Subduction Zone (CSZ). The CSZ is a converging, oblique plate boundary where the Juan de Fuca oceanic plate is being subducted beneath the western edge of the North American continental plate (Geomatrix Consultants, 1995). The CSZ extends from central Vancouver Island, in British Columbia, Canada, through Washington and Oregon to Northern California in the United States (Atwater, 1970). The movement of the subduction zone has resulted in accretion, folding, faulting, and uplift of oceanic and other sediments on the western margin of the North American plate.

In the early Eocene (±55 million years ago), the present location of the Willamette Valley was part of a broad continental shelf extending west from the Western Cascades beyond the present coastline (Orr and Orr, 1999). Basement rock underlying most of the north-central portion of the Valley includes the Siletz River Volcanics (early to middle Eocene, ±50 to 58 million years old), which erupted as part of a submarine oceanic island-arc (Bela, 1979; Yeats et al., 1996). The thickness of the basement volcanic rock is unknown; however, it is estimated to be ±3 to 4 miles thick (Yeats et al., 1996).

The island-arc collided with, and was accreted to, the western margin of the converging North American plate near the end of the early Eocene. Volcanism subsided and a forearc basin was created and infilled to the south with marine sediments of the Eugene Formation and terrestrial sedimentary and volcanic deposits of the Fisher Formation and Little Butte Volcanics throughout the late Eocene and Oligocene (Orr and Orr, 1999; Wiley, 2008). These sediments typically overlie but are also interbedded with younger Tertiary volcanics in the Eugene area.

After emerging from a gradually shallowing ocean, the marine sediments and volcanic formations were covered by the terrestrial Columbia River Basalt (CRB). The CRB poured through the Columbia Gorge from northeastern Oregon and southeastern Washington and spread as far south as Salem, Oregon (±17 to 10 million years ago, middle to late Miocene) (Tolan et al., 2000). Uplift and folding of the Coast Range and the Western Cascades during the late Miocene formed the trough-like configuration of the Willamette Valley (Orr and Orr, 1999; O'Connor et al., 2001; Wiley, 2008; McClaughry et al., 2010).

Following the formation of the Willamette Valley, thick layers of Pliocene gravel filled the Southern Valley (McClaughry et al., 2010). The deposits were then incised by the Willamette River, forming alluvial terraces. In the Pleistocene (±1.6 million to 10,000 years ago), the Central and Southern Valley was refilled with fan-delta gravel, originating from the melting glaciers in the Cascade Range. The Willamette and McKenzie Rivers in the Eugene area incised deeply into the fan-delta deposits during the Quaternary and deposited recent alluvium adjacent to the river banks and major tributaries (Madin and Murray, 2006).

Also, during the Pleistocene (over 15,000 years ago), catastrophic flood deposits mantled the Willamette Valley floor as far south as Eugene (Hampton, 1972; Yeats et al., 1996; O'Connor et al., 2001; McClaughry et al., 2010). These deposits originated from a series of glacial-outburst floods that periodically drained Glacial Lake Missoula in western Montana (Allen et al., 2009). The older flood deposits, typically found within the Portland Basin, usually consist of layers of cobbles/boulders, gravel, and sand deposited during a time period when the river(s) had sufficiently high flow to move large boulders (i.e., erratics) and coarser material. In the Southern Willamette Valley, turbid floodwater eventually settled, depositing a relatively thick layer (50 to 100 feet) of silt and clay, which has been named Willamette Silt (Orr and Orr, 1999).

Local Geology

The reservoir site is near the top of a ridge composed of northwestern-trending mounds with a shallow saddle between. Local geologic mapping indicates the site is underlain by bedrock of the Fisher Formation (Yeats et al., 1996; Madin and Murray, 2006; McClaughry et al., 2010). The Fisher Formation consists of volcaniclastic sedimentary rock and tuff with interfingering andesitic to basaltic flows. The rock can be deeply weathered or hydrothermally altered (Walker and Duncan, 1989; Yeats et al., 1996; Madin and Murray, 2006).

The subsurface conditions encountered in our exploratory borings are consistent with the mapped local geology. The bedrock encountered within the explorations was interpreted to be the Fisher Formation based on the local geologic mapping. Details are provided in the Subsurface Conditions section of the main report and on the boring logs in Appendix B.

Seismic Setting and Local Faulting

We completed a literature review of nearby faults to evaluate the seismic setting and identify the potential seismic sources. The US Geological Survey (USGS) website includes an interactive deaggregation tool, which allows evaluation of the contribution of the various seismic sources to the overall seismic hazard (USGS, 2014). The USGS interactive deaggregation indicates the seismic hazard at the site is dominated by the CSZ (USGS, 2014). Crustal faults also represent a potential seismic hazard. A discussion of these earthquake sources is provided below.

Cascadia Subduction Zone (CSZ). The site is ±110 miles east of the surface expression of the CSZ. The CSZ is a converging, oblique plate boundary where the Juan de Fuca plate is being subducted beneath the western edge of the North American plate. It is estimated the average rate of subduction of the Juan de Fuca plate under the North American plate is ±37 mm/year northeast, based on Pacific and Mid-Ocean Ridge velocities, geodetic studies of convergence, and magnetic anomalies of the Juan de Fuca plate (Personius and Nelson, 2006; DeMets et al., 2010). The CSZ extends ±700 miles from central Vancouver Island in British Columbia, Canada, through Washington and Oregon to Northern California (Atwater, 1970).

Crustal Faults. Crustal faults are fractures within the North American plate. Numerous faults are presented on local and regional geologic maps. However, not all faults are considered to be active. Because the historical earthquake record is so short, active faults are identified by geologic mapping and seismic surveys.

The USGS has defined four fault classifications based on evidence for displacement within the Quaternary (<1.6 million years) in their US fault database (Palmer, 1983; Personius et al., 2003). The fault classes are defined as follows:

- ***Class A*** – Faults with geologic evidence supporting tectonic movement in the Quaternary known or presumed to be associated with large-magnitude earthquakes.

- ***Class B*** – Faults with geologic evidence that demonstrates the existence of a fault or suggests Quaternary deformation, but either: 1) the fault might not extend deep enough to be a potential source of significant earthquakes or 2) the current evidence is too strong to confidently classify the fault as a Class C but not strong enough to classify it as a Class A.

- ***Class C*** – Faults with insufficient evidence to demonstrate 1) the existence of a tectonic fault, or 2) Quaternary movement or deformation associated with the feature.

- ***Class D*** – Geologic evidence indicates the feature is not a tectonic fault or feature.

Class A and B faults are included in the USGS fault database and interactive fault map. USGS considers 17 features in Oregon to be Class C faults (USGS, 2006a). The Class C Harrisburg anticline is ±19 miles north-northwest of the site. The USGS does not consider any features in Oregon as Class D (USGS, 2006a).

Local geologic maps indicate no faults are mapped beneath the site (Walker and Duncan, 1989; Yeats et al., 1996; Madin and Murray, 2006). A few concealed and inferred crustal faults have been mapped within ±10 miles of the site; however, none of the nearby faults show any evidence of movement in the last ±1.6 million years (Palmer, 1983; Geomatrix Consultants, 1995; Personius et al., 2003; USGS, 2006a).

Four potentially active Quaternary Class A and B crustal fault zones have been mapped by the USGS within ±40 miles of the site (Palmer, 1983; Geomatrix Consultants, 1995; Personius et al., 2003; USGS, 2006a). These faults are listed in Table 1D. Figure 1D shows the approximate surface projection locations of these faults.

Table 1D. USGS Class A and Class B Crustal Faults within a ± 40-mile Radius of the Site (1)

Fault Name and Class	Fault Number	Approximate Length (miles)	Approximate Distance and Direction from Site (miles) (2)	Last Known Deformation (years) (3)	Slip Rate (mm/yr)
Upper Willamette River (B)	863	±27	±24 SE	<1.6 million years	<0.20
Owl Creek (A)	870	±9	±33 N-NW	<750,000	<0.20
Corvallis (B)	869	±25	±38 NW	<1.6 million years	<0.20
Unnamed faults near Sutherlin (B)	862	±17	±39 SW	<750,000	<0.20

(1) Fault data based on Personius et al., 2003 and USGS, 2006a and b.

(2) Distance and direction from site to nearest surface projection of the fault.

(3) Quaternary time period defined at <1.6 million years based on the 1983 Geologic Time Scale (Palmer, 1983).

Historic Earthquakes

Available information indicates the CSZ is capable of generating earthquakes along the inclined interface between the two plates (interface) and within the descending Juan de Fuca plate (intraplate) (Weaver and Shedlock, 1996). The fault rupture may occur along a portion or the entire length of the CSZ (Weaver and Shedlock, 1996).

CSZ Interface Earthquakes. The estimated maximum magnitude of a CSZ interface earthquake is up to a moment magnitude (M_w) 9.3 (Petersen et al., 2014). No significant interface (subduction zone) earthquakes have occurred on the CSZ in historic times. However, several large-magnitude (>M ~8.0, M = unspecified magnitude scale) subduction zone earthquakes are thought to have occurred in the past few thousand years. This is evidenced by tsunami inundation deposits, combined with evidence for episodic subsidence along the Oregon and Washington coasts (Peterson et al., 1993; Atwater et al., 1995).

Numerous detailed studies of coastal subsidence, tsunami, and turbidite deposits have been conducted to develop a better understanding of CSZ earthquakes. The studies include investigations of turbidite deposits in the offshore Cascadia Basin that were used to help develop a paleoseismic record for the CSZ and estimate recurrence intervals for interface earthquakes (Adams, 1990; Goldfinger et al., 2012). Study of offshore turbidites from the last ±10,000 years suggests the return period for interface earthquakes varies with location and rupture length. That study estimated an average recurrence interval of ±220 to 380 years for an interface earthquake on the southern portion of the CSZ, and an average recurrence interval of ±500 to 530 years for an interface earthquake extending the entire length of the CSZ (Goldfinger et al., 2012). Older, deep-sea cores have been re-examined more recently, and the findings may indicate greater Holocene stratigraphy variability along the Washington coast (Atwater et al., 2014). Additional research by Goldfinger for the northern portion of the CSZ suggests a recurrence interval of ±340 years for the northern Oregon Coast (Goldfinger et al., 2016). The most recent CSZ interface earthquake occurred ±321 years ago (January 26, 1700) (Nelson et al., 1995; Satake et al., 1996).

CSZ Intraplate Earthquakes. Intraplate (Intraslab or Wadati-Benioff Zone) earthquakes occur within the Juan de Fuca plate at depths of ±28 to 37 miles (Weaver and Shedlock, 1996). The maximum estimated magnitude of an intraplate earthquake is about M_w 7.5 (Petersen et al., 2014). The available record for intraplate earthquakes in Oregon is limited. The available data indicates a M_b = 4.6 (compressional body wave magnitude) event occurred in 1963, located ±23 miles east of Salem at a depth of ±29 miles (Barnett et al., 2009). Based on its depth, this earthquake may be considered an intraplate event. The Puget Sound region of Washington State has experienced three intraplate events in the last ±72 years, including a surface wave magnitude (M_s) 7.1 event in 1949 (Olympia), a M_s 6.5 event in 1965 (Seattle/Tacoma) (Wong and Silva, 1998), and a M_w 6.8 event in 2001 (Nisqually) (Dewey et al., 2002).

Crustal Earthquakes. Crustal earthquakes dominate Oregon's seismic history. Crustal earthquakes occur within the North American plate, typically at depths of ±6 to 12 miles. The estimated maximum magnitude of a crustal earthquake in the Willamette Valley and adjacent physiographic regions is about M_w 7.0 (Petersen et al., 2014). Only two historic crustal events in Oregon have reached Richter local magnitude (M_L) 6 (the 1936 Milton-Freewater M_L 6.1 earthquake and the 1993 Klamath Falls M_L 6.0 earthquake) (Wong and Bott, 1995). The majority of Oregon's larger crustal earthquakes are in the M_L 4 to 5 range (Wong and Bott, 1995).

Table 2D summarizes earthquakes with a M of 4.0 or greater or Modified Mercalli Intensity (MMI) of V or greater, that have occurred within a ±50-mile radius of Eugene in the last ±188 years (Johnson et al., 1994; USGS, 2013). Note that the referenced earthquake catalogs are a composite of different earthquake catalogs and seismic networks; therefore, data errors may exist. Complete historic earthquake records may not yet be included in the referenced earthquake catalogs. Therefore, it is possible some earthquakes may not be included in Table 2D.

Table 2D. Historic Earthquakes Within a ± 50-mile Radius of Eugene (1)

Year	Month	Day	Hour	Minute	Latitude	Longitude	Depth (miles)	Magnitude or Intensity (2)
1921	02	25	20	00	44.4	-122.4	unknown	MMI = V
1942	05	13	01	52	44.5	-123.3	unknown	MMI = V
1961	08	19	04	56	44.7	-122.5	unknown	M = 4.5
2015	07	04	15	42	44.1	-122.8	5.0	M_L = 4.1

(1) The site is located at Latitude 44.009714, Longitude -123.083273.

(2) M = unspecified magnitude, M_b = compressional body wave magnitude, M_c = primary coda magnitude, M_L = local Richter magnitude, and MMI = Modified Mercalli Intensity at or near epicenter.

Seismic events in Oregon were not comprehensively documented until the 1840s (Wong and Bott, 1995). Earthquake epicenters located in Oregon from the late 1920s to 1962 were limited due to the number of and the distance between seismographs, the number of recording stations, and uncertainty in travel times. Therefore, information recorded during that time suggests only earthquakes with magnitudes >5 would be recorded in Oregon (Bela, 1979). Oregon State University (OSU) likely had the first station installed in 1946, and the first modern seismograph was installed at OSU in 1962 (Wong and Bott, 1995; Barnett et al., 2009). According to Wong and Bott (1995), seismograph stations sensitive to smaller earthquakes ($M_L \leq 4$ to 5) were not implemented in northwestern Oregon until 1979 when the University of Washington expanded their seismograph network to Oregon. The local Richter magnitude (M_L) of events occurring prior to the establishment of seismograph stations have been estimated based on correlations between magnitude and MMI. Some discrepancy exists in the correlations.

Table 3D summarizes distant, strong earthquakes felt in the Eugene area (Bott and Wong, 1993; Stover and Coffman, 1993; Wiley et al., 1993; Dewey et al., 1994; Wong and Bott, 1995; Black, 1996; Dewey et al., 2002). None of these events caused significant, reportable damage in Eugene or surrounding area.

Table 3D. Distant Earthquakes Felt in the Eugene Area

Earthquake	Modified Mercalli Intensities (MMI)
2001 Nisqually, Washington	II to III
1993 Klamath Falls, Oregon	IV
1993 Scotts Mills, Oregon	IV
1965 Seattle – Tacoma, Washington	I to IV
1962 Portland, Oregon	I to IV
1961 Lebanon/Albany, Oregon	IV
1949 Olympia, Washington	IV
1873 Crescent City, California	V

Seismic and Geologic Hazards

Section 1803.7 of the OSSC 2019 requires the evaluation of risks from a range of seismic hazards including landslides, earthquake-induced landslides, liquefaction and lateral spread, seismic-induced settlement or subsidence, fault rupture, earthquake-induced flooding and inundation, and local ground motion amplification (OSSC, 2019).

We have developed conclusions regarding the seismic hazards based on the subsurface profiles encountered in our borings at the project site. The conclusions are also based on our knowledge of the site geology, a review of previous geotechnical and seismic studies performed in the area, and available geologic hazard maps (including information available from DOGAMI).

DOGAMI has completed geologic and seismic hazard studies, which include Lane County (Burns et al., 2008), and provides online hazard information through HazVu, LiDAR, and SLIDO viewers (Black et al., 2000; DOGAMI, 2016, 2017, 2018). The above-mentioned maps and viewers refer to some, but do not cover all of the seismic hazards. The information available from DOGAMI is only considered a guide and does not have precedence over site-specific evaluations. In the following sections, information from the available seismic hazard maps is provided along with our site-specific evaluations for comparison.

The relative earthquake hazard is based on the combined effects of ground shaking amplification and earthquake-induced landslides with a range in hazard from Zone A (highest hazard) to Zone D (lowest hazard). Based on the DOGAMI mapping, the site is within Zone D (lowest hazard) for the overall, relative earthquake hazard (Black et al., 2000).

Landslides and Earthquake-Induced Landslides. The proposed tanks will be located near the top of a tree-covered ridge with minor undergrowth. Steep to gentle slopes below the ridge are mostly grass-covered. LiDAR imagery shows smooth, gentle slopes for most of the site with the north portion of the site being relatively flat (DOGAMI, 2017). There are no historic or mapped landslides at the site (Burns et al., 2008; DOGAMI, 2016; Calhoun et al., 2018). The regional landslide hazard map indicates no deep landslide susceptibility (>15 feet deep) at the site, and the susceptibility for shallow (<15 feet deep) landslides is considered low to moderate along the ridgeline (DOGAMI, 2016, 2018).

The site is underlain topsoil/residual soil followed by shallow, predominately very weak (R1) to very strong (R5) bedrock. Based on the site conditions and the absence of mapped or historic landslides and instability features, we believe the risk of landslides or earthquake-induced landslides is very low. The new tanks will be supported on bedrock. Therefore, we believe the risk of slope instability impacting the tanks is negligible.

Liquefaction, Settlement, and Lateral Spread. Soil liquefaction occurs when loose, saturated cohesionless soil experiences a significant loss of strength during strong ground shaking. The strength loss is associated with rapid densification of the soil and corresponding development of high pore water pressure, which can lead to the soil behaving like a viscous fluid. Liquefiable soils typically consist of saturated, loose, clean sand and non-plastic to low plasticity silt with a plasticity index (PI) typically less than 8.

A very thin topsoil mantle overlies residual soil followed by shallow, weak to moderately strong bedrock. The underlying residual soil is typically medium dense to very dense or hard and is not expected to be liquefiable due to its density and strength, and the absence of shallow groundwater.

The new tanks will be supported on bedrock. Therefore, the risk of liquefaction impacting the tanks is nil. The HazVu site indicates no liquefaction susceptibility in the project area; (Burns et al., 2008; DOGAMI, 2018).

Lateral spread is a liquefaction-induced hazard, which occurs when soil or blocks of soil are displaced down slope or toward a free face (such as a riverbank) along a liquefied layer. The lateral spread hazard at this site is considered nil due to the absence of a liquefaction hazard.

Subsidence. Ground subsidence is a regional phenomenon resulting from a large magnitude CSZ earthquake. It occurs because the subduction of the oceanic crust beneath the continental crust compresses the continental crust and pushes it upward. Prior to the earthquake, the continental crust is held in this position by friction at the CSZ interface. When the earthquake occurs, that frictional bond breaks allowing the continental crust to drop. The subsidence hazard map included in the Oregon Resilience Plan (OSSPAC, 2013), indicates the ground subsidence in the Eugene area during a M_w 9 CSZ earthquake could be up to 1 foot. Ground subsidence cannot be mitigated. Therefore, it should be assumed the site and surrounding area could drop by up to 1 foot during a large magnitude CSZ earthquake.

***Fault Rupture*.** The risk of fault rupture is expected to be low due to the lack of known active faulting beneath the site (Personius et al., 2003; Madin and Murray, 2006; USGS, 2006b, a; McClaughry et al., 2010). The closest potentially active (Class A) crustal fault is the Owl Creek fault, which is ±33 miles north of the site.

***Tsunami / Seiche/ Earthquake-Induced Flooding*.** Tsunami are waves created by a large-scale displacement of the sea floor due to earthquakes, landslides, or volcanic eruptions (Priest, 1995). Tsunami inundation is not applicable to this site because Eugene is not on the Oregon Coast. Seiche (the back and forth oscillations of a water body during a seismic event) is also not a local hazard due to the absence of large bodies of water near the site.

According to HazVu, there is no localized flood potential for the Effective FEMA 100-year flood at or near the site (DOGAMI, 2018). Earthquake-induced flooding related to the failure of other structures (e.g., dams) or shallow ground water and subsidence does not apply to the site.

***Local Ground Motion Amplification*.** Ground motion amplification is the influence of a soil deposit on the earthquake motion. As seismic energy propagates up through the soil strata, the ground motion is typically increased (i.e., amplified) or decreased (i.e., attenuated) to some extent. Based on the presence of limited topsoil and residual soil followed by shallow, very weak (R1) to very strong (R5) bedrock, it is our opinion the amplification hazard is low and is consistent with an OSSC/IBC Site Class B (i.e., bedrock with a shear wave velocity (V_s) between 2,500 and 5,000 ft/s). The DOGAMI hazard studies also indicate the amplification susceptibility for the site is low (NEHRP Site Class B) (Black et al., 2000; Burns et al., 2008). The site is expected to experience strong ground shaking during a CSZ earthquake due to its proximity to the CSZ (DOGAMI, 2018). See the main report for more discussion on the site response.

SEISMIC DESIGN

Design Earthquakes

The OSSC 2019, Section 1803.3.2.1, requires the design of structures classified as essential or hazardous facilities and of major and special occupancy structures to address, at a minimum, the following earthquakes:

Crustal:	A shallow crustal earthquake on a real or assumed fault near the site with a minimum M_W 6.0 or the design earthquake ground motion acceleration determined in accordance with the OSSC 2019 Section 1613.
Intraplate:	A CSZ intraplate earthquake with M_W of at least 7.0.
Interface:	A CSZ interface earthquake with a M_W of at least 8.5.

The design maximum considered earthquake ground motion maps provided in the OSSC 2019, are based on modified (risk-targeted) 2014 maps prepared by the USGS for an earthquake with a 2% probability of exceedance in 50 years (i.e., a ±2,475-year return period) for design spectral accelerations (USGS, 2014). The modifications include factors to adjust the spectral accelerations to account for directivity and risk.

The 2014 USGS maps were established based on probabilistic studies and include aggregate hazards from a variety of seismic sources. The USGS interactive deaggregation for a 2,475-year return period indicates the seismic hazard at the site is dominated by the CSZ, contributing ±82% to the overall aggregate hazard. Crustal earthquakes were included in the studies but were not considered to be a principal seismic hazard at this site. The CSZ scenarios considered ranged from M_w 8.5 to 9.3, located ±43 to 68 miles west of the site.

The earthquake magnitudes and source-to-site distances used to generate the 2014 USGS maps satisfy the requirements of OSSC 2019. Seismic design parameters and AWWA D110-13 design response spectra are discussed in the Site Response Spectra section of the main report and are shown on Figure 6A and 7A (Appendix A).

CONCLUSION

Based on the findings presented herein, it is our opinion there are no geologic or seismic hazards that would preclude the design and construction of the proposed project. This site-specific seismic hazard investigation for the East 40th Avenue Storage Tanks, Eugene, Oregon, was prepared by Brooke Running, R.G., C.E.G.

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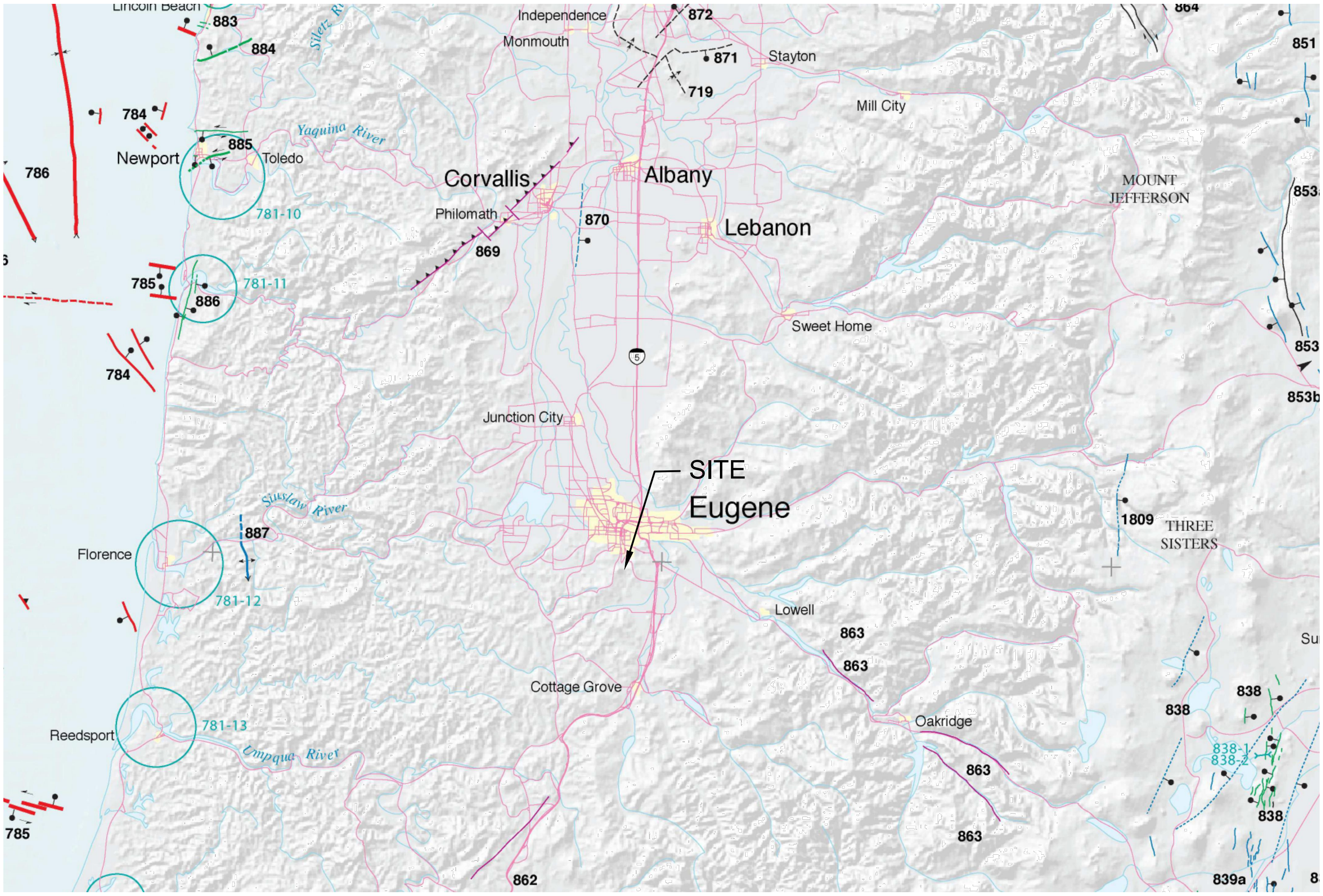

NOTES:

1. PORTION OF MAP BASED ON MAP OF QUATERNARY FAULTS AND FOLDS IN OREGON (PERSONIUS ET AL., 2003).
2. SEE SITE-SPECIFIC SEISMIC HAZARD STUDY FOR A DISCUSSION OF LOCAL FAULTING.
3. FAULTS: #862 = UNNAMED FAULTS NEAR SUTHERLIN; #863 = UPPER WILLAMETTE RIVER, #869 = CORVALLIS, AND #870 = OWL CREEK.
4. MAP IS NOT TO SCALE.

MAP LEGEND:

TIME OF MOST RECENT SURFACE RUPTURE

- Holocene (<10,000 years) or post last glaciation (<15,000 years); no historic ruptures in Oregon to date
- Late Quaternary (<130,000 years; post penultimate glaciation)
- Late and middle Quaternary (<750,000 years)
- Quaternary, undifferentiated (<1,600,000 years)
- Class B structure (age or origin uncertain)

SLIP RATE

- >5mm/year
- 1.0-5.0 mm/year
- 0.2-1.0 mm/year
- <0.2mm/year

TRACE

- Mostly continuous at map scale
- Mostly discontinuous at map scale
- Inferred or concealed

STRUCTURE TYPE & RELATED FEATURES

- Normal or high-angle reverse fault
- Strike-slip fault
- Thrust fault
- Anticlinal fold
- Synclinal fold
- Monoclinal fold
- Plunge direction of fold
- Fault section marker

CULTURAL AND GEOGRAPHIC FEATURES

- Divided highway
- Primary or secondary road
- Permanent river or stream
- Intermittent river or stream
- Permanent or intermittent lake

DETAILED STUDY SITES

- 731-2 Trench site
- 781-2 Subsuction zone study site

Foundation Engineering, Inc.
Professional Geotechnical Services

PROJECT NO.	DATE:	DRAWN BY:
2201086	Mar. 9, 2021	BKR

QUARTERNARY CRUSTAL FAULT MAP

SOUTHERN WILLAMETTE VALLEY
EAST 40TH AVENUE STORAGE TANKS
EUGENE, OREGON

FIGURE NO.
1D