Geotechnical Investigation and Seismic Hazard Study

College Hill 607 Storage Tanks

Eugene, Oregon

Prepared for:

Eugene Water & Electric Board
Eugene, Oregon

June 12, 2024

Foundation Engineering, Inc.

Professional
Geotechnical
Services

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Laura Farthing, P.E.
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4200 Roosevelt Boulevard
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June 12, 2024

College Hill 607 Storage Tanks
Geotechnical Investigation and Seismic Hazard Study
Eugene, Oregon

Project No.: 2201012

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.

The site-specific seismic hazard study was 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 2022 Oregon Structural Specialty Code (OSSC, 2022) for site-specific seismic hazard reports for essential and hazardous facilities, and major and special-occupancy structures. The 2022 OSSC is based on the 2021 International Building Code (IBC) and ASCE 7-16. Results of the seismic hazard 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 seismic hazard 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 and 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, P.E.
Project Engineer

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GEOTECHNICAL INVESTIGATION AND SEISMIC HAZARD STUDY

COLLEGE HILL 607 STORAGE TANKS
EUGENE, OREGON

BACKGROUND

The Eugene Water & Electric Board (EWEB) is planning to replace the existing College Hill 607 Reservoir. The site location is shown in Figure 1A (Appendix A), and the current site layout is shown in Figure 2A. The existing reservoir is a 15 MG rectangular concrete tank that was built in 1940. The site also includes a 2.5 MG rectangular reservoir to the north of the 15 MG reservoir. The smaller reservoir was built in 1916 and is designated as College Hill 603 Storage Tank.

It was initially planned to construct one new 7.5 MG circular concrete tank at the site. Two possible tank locations were considered for that tank:

- One location was in the north half of the existing 15 MG reservoir footprint
- One location was north of the existing 15 MG reservoir, overlapping the 2.5 MG reservoir footprint

It is currently planned to demolish both of the existing reservoirs and to construct two new, 7.5 MG, 212-foot diameter, circular concrete tanks at the site with both of the new tanks sited within the existing 15 MG rectangular tank footprint. The current proposed tank locations and the previously proposed alternate tank location north of the existing 15 MG tank are shown in Figure 2A.

EWEB retained Foundation Engineering, Inc. to conduct a geotechnical investigation for the project. Our scope of work was outlined in a proposal dated January 31, 2020, and authorized by Personal Services Contract # 20-047-Q.

Foundation Engineering completed a geotechnical investigation for the previously proposed tank locations and presented the findings in a draft report dated August 11, 2023. That investigation included five exploratory borings, which were focused on the north half of the site. Subsequent to that work, EWEB requested Foundation Engineering complete additional borings to investigate the subsurface conditions adjacent to the south end of the existing 15 MG reservoir. Our previous report was updated to include the additional subsurface information and updated design and construction recommendations for the current planned tank locations.

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 Seismic Hazard Study report (Appendix D). References cited in this section are in Appendix D. An abbreviated discussion of the local geology is provided below.

The project site is located at the top of College Hill in southwest Eugene. Local geologic mapping indicates the project site is underlain by the Eugene Formation (Yeats et al., 1996; Madin and Murray, 2006; McClaughry et al., 2010). The Eugene Formation consists of tuffaceous sandstone and siltstone deposited in a shallow marine environment (Yeats et al., 1996; Orr and Orr, 1999; O'Connor et al., 2001; Madin and Murray, 2006). Eugene Formation outcrops in the Coburg Hills, ±7 miles northeast of the site, and is estimated to be ±1,800 feet thick (Yeats et al., 1996).

The subsurface conditions encountered in our explorations are consistent with the mapped local geology. Sandstone was encountered in each of the explorations. The sandstone was interpreted to be the Eugene Formation based on the 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 five exploratory borings (BH-1 to BH-5) at the site between March 2 and March 4, 2020, and two additional two borings (BH-6 and BH-7) between January 18 and 23, 2024. The approximate boring locations are shown in Figure 2A. These locations were established using a measuring wheel and tape measure referencing existing landmarks. The ground elevations at the boring locations were estimated based on the topographic survey contours shown in Figure 2A. The ground elevations reference the National Geodetic Vertical Datum of 1929 (NGVD 29) which is being used for this project.

The borings were drilled using a CME 55 track-mounted drill rig with mud-rotary drilling and HQ wire-line coring techniques. Soil and bedrock samples were obtained at 2½-foot intervals. Samples were obtained with a split-spoon sampler in conjunction with Standard Penetration Testing (SPT). The SPT provides an indication of the relative stiffness or density of the foundation soils. HQ-sized, wire line rock coring was completed once coreable bedrock was encountered. Upon completion of drilling, the boreholes were backfilled with bentonite chips in accordance with Oregon Water Resources Department (OWRD) guidelines.

The borings were continuously logged during drilling. The final logs (Appendix B) were prepared based on a review of the field logs, the results of the laboratory testing, and an examination of the soil and rock samples in our office. Important information about the boring logs and definitions of symbols and descriptive terms used in the logs are provided in the Symbol Key and the Soil and Rock Descriptions and Common Terms Key sheets included in Appendix B. The Important Information sheet also includes a discussion of the interpretation of the subsurface profile at the boring locations and the potential for inherent variations in the subsurface conditions across the site.

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 D2487 and ASTM D2488. The test results are summarized in Table 1C (Appendix C). The moisture contents are also shown in the boring logs (Appendix B).

Fourteen unconfined compression (q_u) tests were completed on rock core samples to evaluate the strength of the bedrock. Two tests were conducted with continuous stress-strain measurements to evaluate the elastic properties of the bedrock in addition to the peak q_u values. The other twelve tests focused on the maximum q_u values only. The stress-strain curves are plotted in Figures 1C and 2C (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 ±1,478 to 4,247 psi, consistent with weak (R2) to medium strong (R3) rock.

SITE CONDITIONS

Surface Conditions

The existing reservoirs are located east of Lawrence Street at the top of College Hill. The ground surface surrounding the existing 15 MG reservoir slopes down to the northeast. The ground surface surrounding the existing 2.5 MG reservoir slopes down to the west on the west side of the structure. There is a relatively flat area immediately east of the 2.5 MG reservoir. The ground surface to the east of the flat area slopes down to the east and south. The topographic survey indicates surface elevations ranging from ±El. 585 to ±El. 620 across the property. The site is landscaped with grass and several deciduous trees near the property boundaries.

Subsurface Conditions

We developed a series of cross-sections across the site utilizing topographic data provided by EWEB and subsurface information gathered during exploration. The cross-sections locations are shown in Figure 2A. The cross-sections, shown in Figures 3A through 6A, indicate the site is underlain by medium dense to very dense residual soil over relatively shallow sandstone and silty sandstone (Eugene Formation).

In most of our borings, bedrock was encountered at depths of ±5 to 8.5 feet. The exception was BH-1, which encountered bedrock at ±21 feet. The estimated ground surface elevations, exploration depths, and bedrock elevations for each of the borings are shown in the boring logs. The data is also summarized in Table 1B (Appendix B).

The upper ±2 to 6.5 feet of the bedrock encountered in the borings is decomposed to highly weathered and extremely weak to very weak (R0 to R1). The bedrock becomes moderately weathered to fresh and very weak (R1) to medium strong (R3) at greater depths. The variation in bedrock elevations between the borings suggests the bedrock surface slopes down to the east and north across the footprint of the existing 15MG reservoir. Based on our cross-sections (Figures 3A and 6A) and a comparison of the existing 15MG reservoir bottom elevations to the top of bedrock elevations in the adjacent borings, we anticipate the upper, more highly weathered, extremely weak to very weak (R0 to R1) bedrock was removed from at least the west half of the existing reservoir footprint, and the bedrock underlying the west half of the existing reservoir is predominantly slightly weathered to fresh and weak to medium strong (R2 to R3). Extremely weak to very weak (R0 to R1) bedrock may be encountered beneath the east half of the reservoir footprint underlain by weak to medium strong (R2 to R3) bedrock.

The quality of the rock is defined in part by a Rock Quality Designation (RQD) value which is calculated by summing the lengths of retained rock core that are at least 4 inches long and dividing that summed length by the total length of the core run. For example, a core run with an RQD of 100% may be comprised of a series of core segments that are all at least 4 inches long. A core run with an RQD of 100% could also be comprised of one continuous rock core segment with no joints. Not all of the breaks in the retained rock core are natural joints in the bedrock. There are also mechanical breaks (i.e., breaks due to drilling operation or due to using a hammer to break the rock core as needed to fit in the core boxes). Mechanical breaks are excluded from the calculation of RQD.

Overall RQD values in the borings range from ±18% to 100%, with the exception of a RQD of 0% in the upper rock in BH-6 (core run CS-6-2). The borings closest to the 15MG reservoir (i.e., BH-3, BH-4, BH-5, BH-6, and BH-7) were used to estimate the bedrock conditions within the planned excavation limits for the new tanks. Within the elevation range associated with the planned rock excavation for the new tanks (i.e., between ±El. 572 and ±El. 586), the RQDs in BH-3, BH-4, BH-5, BH-6, and BH-7 range from ±59% to 100% with an average RQD of ±89%.

These RQD values indicate fair to excellent rock quality. The joints in the extremely weak to very weak (R0 to R1) rock are typically open with very close to close spacing and the joints in the weak to medium hard (R2 to R3) rock are typically closed with close to wide spacing.

Photos of the rock core are shown in Photos 1B through 26B (Appendix B). For reference, the length of the inside of the core boxes is ±2 feet. As noted above, not all of the breaks visible in the rock core are natural joints in the bedrock. Mechanical breaks are designated by a MB written on the core box dividers next to the core. As an example, in the rock core for Box 2 of 4 from BH-3 (Photo 8B), all of the breaks in core run CS-3-4 are mechanical. That core run has an RQD of 100% and the actual length of the continuous rock core in CS-3-4 is ±5 feet. There were several core runs in the borings with continuous rock core segments that were more than 2 feet long and some of the deeper core runs had continuous rock core segments that were more than 4 feet long. Our more recent borings on Lincoln Street to the east and northeast of the tanks also had core runs with continuous rock core lengths exceeding 4 feet. This information suggests much of the weak to medium hard (R2 to R3) bedrock beneath the existing 15MG reservoir will typically have joint spacings of at least 2 feet, and in some locations and depths, the bedrock will be massive with joint spacings exceeding 4 feet.

Groundwater

Mud-rotary drilling techniques precluded measurement of groundwater levels 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

The new tanks will be constructed within the existing 15 MG reservoir footprint. A drawing for the existing 15MG reservoir is included in Appendix A. The drawing shows the existing reservoir is divided into two cells. The finished floor at the center of each cell is at ±El. 585.5 and the floors slope upward toward the sides of the cells with a FFE of ±El. 586.6 at the edges.

The new tanks will have an outside diameter of 212 feet and 34-foot-tall walls. The finished floor elevations (FFE) for the new tanks will vary across the footprint with a FFE of El. 579.1 at the center of each tank and a FFE of 577.0 at the inside of the perimeter walls. A subslab drainage layer will be constructed beneath the bottoms of the tanks. The subslab drainage will include a minimum of 2 feet of open-graded crushed drain rock. The drain rock will be capped with ¾ or 1-inch minus, well-graded crushed rock (Base Aggregate). The Base Aggregate will be at least 6 inches thick beneath the perimeter ring footing. The Base Aggregate capping layer will be thicker beneath the floor slab. The footings and columns will be constructed on top of the floor slab to support the roof.

The excavations for the new tanks will extend to ±El. 574 at the center of each tank and to ±El. 572 a few feet beyond the edges of the tanks. Therefore, the excavations will extend ±11.5 feet below the current FFE at the center of the existing reservoir cells and ±14.6 feet below the current FFE at the edges of the existing reservoir. The existing reservoir was constructed on rock. Therefore, site preparation for the new tanks will include demolishing the existing reservoir and excavating into the underlying bedrock.

A control valve vault is planned to the east of the northern tank. The 60% plans indicate a vault base elevation of ±El. 567. However, we understand the vault layout may change as the design progresses. Drilling near the proposed vault location (BH-5) encountered dense to very dense residual soil followed by extremely weak to very weak (R0 to R1) silty sandstone below ±8.4 feet (±El. 578.6). The bedrock becomes weak (R2) below ±13 feet (±El. 574.0).

Rock excavation will be an important geotechnical consideration. The unconfined compression (q_u) test results indicate q_u values ranging from 1,478 to 4,247 psi, with an average of 2,644 psi. The joint spacing in the bedrock in our borings ranged from very close (<2 inches) to wide. The joints in the upper extremely weak to very weak (R0 to R1) rock are typically open with very close to close spacing. In the deeper weak to medium hard (R2 to R3) rock, the joints are typically closed with close to wide spacing. Joint spacing the weak to medium hard (R2 to R3) rock may typically exceed 2 feet. The bedrock has relatively high RQD values as described in the Subsurface Conditions section of this report.

Based on the rock hardness and the spacing and condition of the joints, we anticipate it will not be practical to excavate the weak to medium hard (R2 to R3) bedrock by digging with an excavator bucket alone, and it will be necessary to fracture the rock prior to excavating. Potential fracturing methods include blasting, hammering the rock with a hydraulic ram, or drilling and splitting. We presume the contractor will select the rock excavation method. Given the overall volumes and depths of the required excavation in weak to medium hard (R2 to R3) bedrock beneath the new tanks, we anticipate blasting will be the most practical approach to fracture the rock for the bulk of the tank excavation. Fracturing the rock with mechanical means alone (i.e., using a hydraulic ram) is likely to be slow and best suited for chipping the rock in smaller areas or for fine grading the bottom and sides of the larger excavation for the new tanks. Mechanical means may also be suitable for fracturing extremely weak to very weak (R0 to R1) rock encountered in the eastern portion of the tank excavations.

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

Based on the interpreted cross-sections, we anticipate the new tanks will be underlain by very weak to medium strong (R1 to R3) sandstone and silty sandstone. We have concluded the subsurface conditions correspond to an OSSC Site Class B.

AWWA D110-13(R18) 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 2022 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.

Maximum considered earthquake (MCE) ground motions for a 10% probability of exceedance in 50 years (i.e., a ±475-year return period) may be required. Therefore, we also prepared site response spectra for that return interval. Spectral accelerations for a ±475-year return period were obtained from the USGS interactive deaggregation website (USGS, 2014) using maps, which include modification for directivity.

To develop the design 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(R18) site response spectra for impulsive and convective components with MCE_R ground motions with 2% probability of exceedance in 50 years are shown in Figure 7A. The site response spectra with MCE ground motions with 10% probability of exceedance in 50 years are shown in Figure 8A.

Vertical Accelerations. Vertical accelerations may be analyzed based on AWWA D110-13(R18) 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 7A and 8A.

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

The new tanks will have a concrete floor slab, a ring footing supporting the perimeter wall, and interior columns supporting the roofs. The interior columns will be supported by footings bearing on the floor slabs. The ring footings and floor slabs will be underlain by a leveling course of compacted Base Aggregate and ±2 feet of compacted crushed drain rock over weak to medium strong (R2 to R3) sandstone or silty sandstone. We recommend assuming a conservative allowable bearing pressure of 30 ksf for footing design. The allowable bearing pressure may be increased by one-third for the evaluation of transient loads (i.e., seismic and wind).

The bedrock is an 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 Base Aggregate, 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

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

The tank walls will be backfilled with a combination of crushed Drain Rock and compacted Base Aggregate extending a minimum of 10 feet beyond the tank walls. Outside this zone, the backfill will consist of additional compacted Base Aggregate or Granular Site Fill. The backfill will extend to the sidewalls of the excavation which will be predominantly in bedrock. The ring footings will be backfilled with an 18-inch thick layer of sand wrapped in a non-woven geotextile fabric. The sand layer will serve as a filter media between the Drain Rock used to backfill the tank sidewalls and the Drain Rock in the drainage layer beneath the tanks.

Passive resistance from the backfill against the ring footings and tank sidewalls 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. The backfill in direct contact with the tank walls and ring footing will be crushed Drain Rock and sand. Base Aggregate will be used to backfill outside these materials. For these conditions, we calculated the passive pressure assuming a ϕ of 36 degrees and a γ of 130 pcf. 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 and slabs 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 ring footing may require a lateral displacement corresponding to 1% of the buried height, assuming the excavation is backfilled with compacted granular fill as described above.

Lateral Earth Pressures for Buried Walls

Lateral earth pressures will be imparted on the buried portions of the tank walls from the backfill. We assume the walls will be backfilled with crushed Drain Rock and Base Aggregate extending at least 10 feet beyond the tank walls, surrounded by compacted, Granular Site Fill or additional Base Aggregate.

To calculate lateral earth pressure on the buried walls, we assumed a γ of 130 pcf, a ϕ of 36 degrees, and a wall friction angle (δ) of 24 degrees. Both static and seismic loading conditions were analyzed, as discussed below.

Static Loading. 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 $k_{ae} * \gamma$. We calculated a USGS Maximum Credible Earthquake (MCE) peak ground acceleration (k_{max}) of 0.34g at the base of the tank assuming a Site Class B. This value was adjusted to a k_{av} of 0.35g following the guidelines in Section 7.5 of NCHRP Report 611 (TRB, 2008) to account for the wall height and wave scattering. The calculated k_{av} value assumes no wall deflection. Based on the wall height, we anticipate it would be unlikely there would be no wall deflection. We used a reduced pseudo-static horizontal acceleration coefficient (k_h) of 0.30g for the GLE analysis accounting for modest wall deflection (i.e., less than ½ inch).

The failure surface was assumed to extend through the wall backfill. A line load was applied at the wall location at a height of H/3 above the base of the wall to represent the resultant of the lateral resistance provided by the wall. The line load was angled at δ = 24 degrees below the horizontal to account for friction between the wall and backfill. 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 63 pcf, which corresponds to a k_{ae} of 0.49, 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 0.25 and an equivalent fluid density of 32 pcf. The resultants of both the static and seismic thrust components act at H/3 above the base of the wall.

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	0.49
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}	$\Delta k_{ae} = k_{ae} - k_a$	0.25
Seismic Thrust Equivalent Fluid Density (Seismic Component of Total Dynamic Earth Pressure)	$\Delta k_{ae}*\gamma_{backfill}$	32 pcf

Traffic Surcharge. The new tanks will be 45 feet from the edge of pavement on Lawrence Street and 80 feet from the edge of pavement on Lincoln Street. At those distances, there will be no traffic surcharge on the buried tank sidewalls from traffic on the adjacent streets. However, a surcharge from construction traffic or future maintenance vehicles driving next to the tanks and other buried structures is likely.

We modeled the construction traffic surcharge as an 8-foot-wide uniformly loaded strip with a vertical pressure of 250 psf, centered 7 feet from the edge of the buried structures. Using this configuration, we calculated a lateral surcharge pressure that can be modeled starting from zero at the ground surface and ramping up to 120 psf at a depth of 2 feet, staying uniform at 120 psf to a depth of 4 feet, and then ramping down to zero at a depth of 15 feet. The recommended traffic surcharge pressure configuration is summarized in Table 2.

Table 2. Construction Traffic Surcharge Pressures

Depth (ft)	Lateral Surcharge Pressure (psf)
0	0
2	120
4	120
15	0

The construction/maintenance vehicle traffic surcharge pressure should be combined with at-rest pressure for static loading. A construction/maintenance vehicle traffic surcharge is not required for seismic design.

RECOMMENDATIONS

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

General Earthwork and Materials

1. Base Aggregate 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 fragments from imported sources or on-site excavations that are free of organics and construction debris. This material may be used for general site grading outside foundation areas and for backfilling outside the Base Aggregate wall backfill zone (i.e., 10 feet beyond the tank walls). The suitability of reusing on-site materials as Granular Site Fill 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 rock. Other gradations may be acceptable, provided the rock is durable, free draining, and suitably angular to allow dense compaction. 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 D698 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 Base Aggregate and proof-rolled again.

Foundation Design and Construction

6. Design the tank using the seismic design parameters and response spectra shown in Figures 7A and 8A 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 seismic loads. The allowable bearing pressure assumes the footings will bear on compacted Base Aggregate and Drain Rock 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 slabs and footings and the compacted Base Aggregate. Calculate the ultimate passive resistance for the buried tanks using an equivalent fluid density of 500 pcf. Assume it may require a lateral displacement of 1% of the buried 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 slabs will be constructed on compacted Base Aggregate and Drain Rock underlain by bedrock.

Foundation Preparation

Foundation preparation recommendations are provided below.

10. Demolish the existing reservoirs and remove the slabs and foundations and existing backfill materials. Haul all construction debris off-site.

11. Excavate the tank foundation areas to the finish subgrade elevations. The excavation will accommodate at least 24 inches of Drain Rock Capped with Base Aggregate. The Base Aggregate will be at least 6 inches thick beneath the ring footings and thicker beneath the floor slabs. The excavation should extend at least 10 feet beyond the tank footprints to provide construction access.

12. Remove all loose rock and debris exposed at the subgrade level. Place Drain Rock over the approved subgrade and install a perimeter foundation drain in the Drain Rock. Place the Drain Rock in lifts up to 12 inches thick and compact it until it is visibly dense and unyielding. A Foundation Engineering representative should verify the adequacy of the compaction.

13. Cover the Drain Rock with a Subsurface Drainage Geotextile and a minimum of 6 inches of compacted Base Aggregate to serve as a leveling course beneath the slabs and footings. The Base Aggregate should be compacted 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 are expected to encounter topsoil, existing fill, and/or residual soil underlain by sandstone and silty sandstone. The topsoil, existing fill, and residual soil includes soft to medium stiff silty clay and loose to medium dense silty sand. The silty clay corresponds to an OR-OSHA Type B soil, and the silty sand corresponds to an OR-OSHA Type C soil. OR-OSHA recommends excavating temporary cut slopes no steeper than 1:1(H:V) in Type B soil and 1½:1(H:V) in Type C soil.

The upper ±2 to 6.5 feet of the bedrock in the borings is decomposed to highly weathered and extremely weak (R0). This material corresponds most closely to an OR-OSHA Type A soil. OR-OSHA recommends excavating temporary cut slopes no steeper than ¾:1(H:V) in Type A soil. The deeper bedrock is less weathered and harder. This material should satisfy the OR-OSHA requirements for Stable Rock where it is not closely jointed or disturbed. OR-OSHA allows up to vertical cuts in stable rock. Loose material should be scaled from the cut slopes, as needed, to protect workers from falling rock. Covering the cut slopes with chain link fencing or other measures may also be employed to help direct raveled surface material to the toe of the slopes and reduce risks to workers.

We anticipate the presence of shallow bedrock will preclude using sheet piles for shoring. Other potential shoring options for retaining the soil above the bedrock include conventional trench boxes, slide rail systems, or soldier pile walls with the soldier piles drilled and grouted into the bedrock.

15. Suitable cut slopes in the soil and bedrock will have to be confirmed in the field by a Foundation Engineering representative at the time of construction. The soil and decomposed to highly weathered bedrock may degrade when exposed to rainfall. Therefore, during wet weather, the cut slopes should be covered in plastic sheeting and inspected daily.

16. The contractor should select the appropriate rock excavation method. We anticipate the bedrock directly underlying at least the west half of the existing 15MG reservoir is slightly weathered to fresh and weak to medium strong (R2 to R3) with close to moderately close joint spacings and closed joints. Extremely weak to very weak (R0 to R1) bedrock may be encountered beneath the east half of the existing reservoir followed by weak to medium strong (R2 to R3) bedrock.

    Based on the rock hardness and the spacing and condition of the joints, we anticipate it will not be practical to excavate the weak to medium hard (R2 to R3) bedrock by digging with an excavator bucket alone, and it will be necessary to fracture the rock prior to excavating. Given the overall volumes and depths of the required excavation in weak to medium hard (R2 to R3) rock beneath the new tanks, we anticipate blasting will be the most practical approach to fracture the rock for the bulk of the tank excavation. Fracturing the rock with mechanical means alone (i.e., using a hydraulic ram) is likely to be slow and best suited for chipping the rock in smaller areas or for fine grading the bottom and sides of the larger excavation for the tanks. Mechanical means may also be suitable for fracturing extremely weak to very weak (R0 to R1) bedrock encountered in the upper excavations beneath the eastern portions of the new tanks.

The laboratory testing completed to date on rock core samples indicates q_u values ranging from 1,478 to 4,247 psi. However, harder bedrock may be encountered. The selected method should be capable of removing bedrock with unconfined compressive strengths of up to 8,000 psi (i.e., the upper limit for R3 rock).

17. 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.

18. Use Drain Rock and Base Aggregate to backfill around the tank within ±10 feet of the walls as shown in the plans. Approved Granular Site Fill may be used to backfill outside this zone. Compact the backfill as recommended in Item 5.

19. Where possible, 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 Base Aggregate may be graded at 1½:1(H:V).

20. Finished cut slopes in bedrock that will remain exposed permanently should be graded at 1½:1(H:V) or flatter. 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.

21. 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 tanks and beneath the tanks, which may result in hydrostatic pressure on the floor slab and sidewalls. A perimeter foundation 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:

22. Install a foundation drain along the perimeter of the tanks. The drain should consist of a 6-inch diameter, perforated HDPE or PVC pipe. The flowline of the pipe should be set in the Drain Rock layer near the bottom of the excavation.

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

24. 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 unstable cut slopes 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 to 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 College Hill 607 Storage Tanks project in Eugene, Oregon. The 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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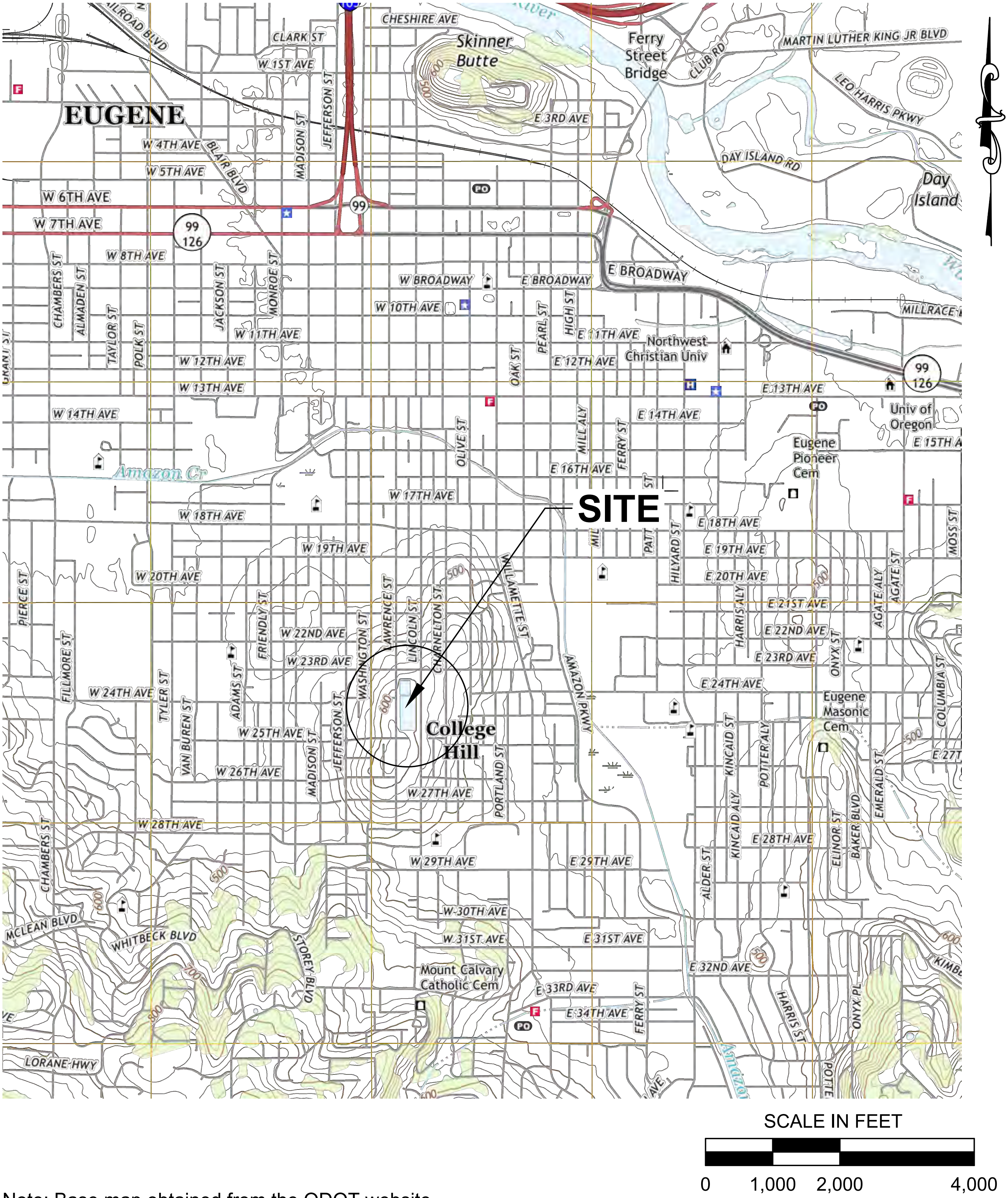
Appendix A

Figures

Foundation Engineering, Inc.
Professional Geotechnical Services



Note: Base map obtained from the ODOT website.

Foundation Engineering, Inc.
Professional Geotechnical Services

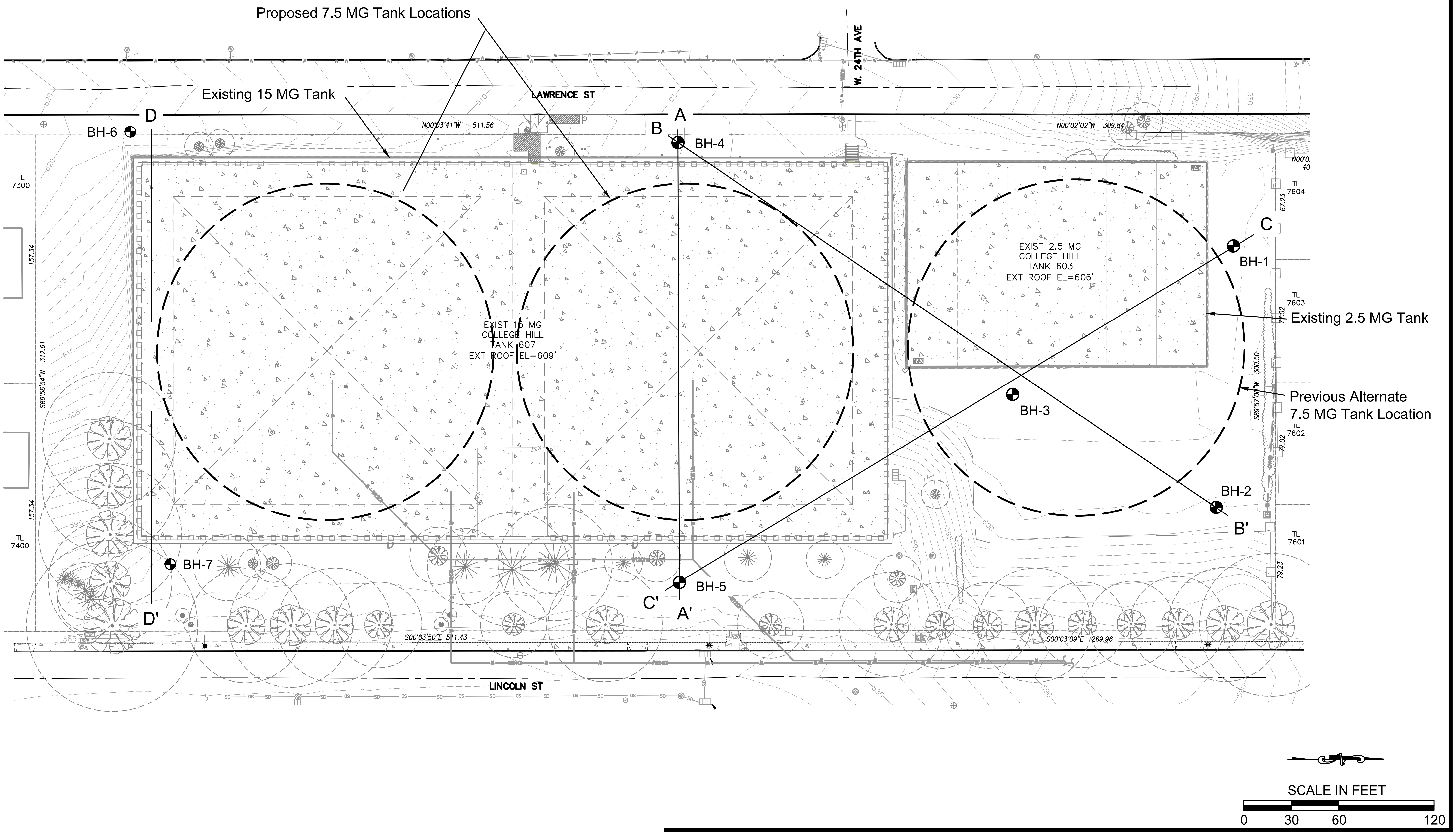
PROJECT NO.	DATE:	DRAWN BY:
2201012	Aug. 11, 2023	MLM

VICINITY MAP

COLLEGE HILL 607 STORAGE TANKS
EUGENE, OREGON

FIGURE NO.

1A



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 CONSOR.
4. GROUND ELEVATIONS REFERENCE NGVD 29.

Foundation Engineering, Inc.
Professional Geotechnical Services

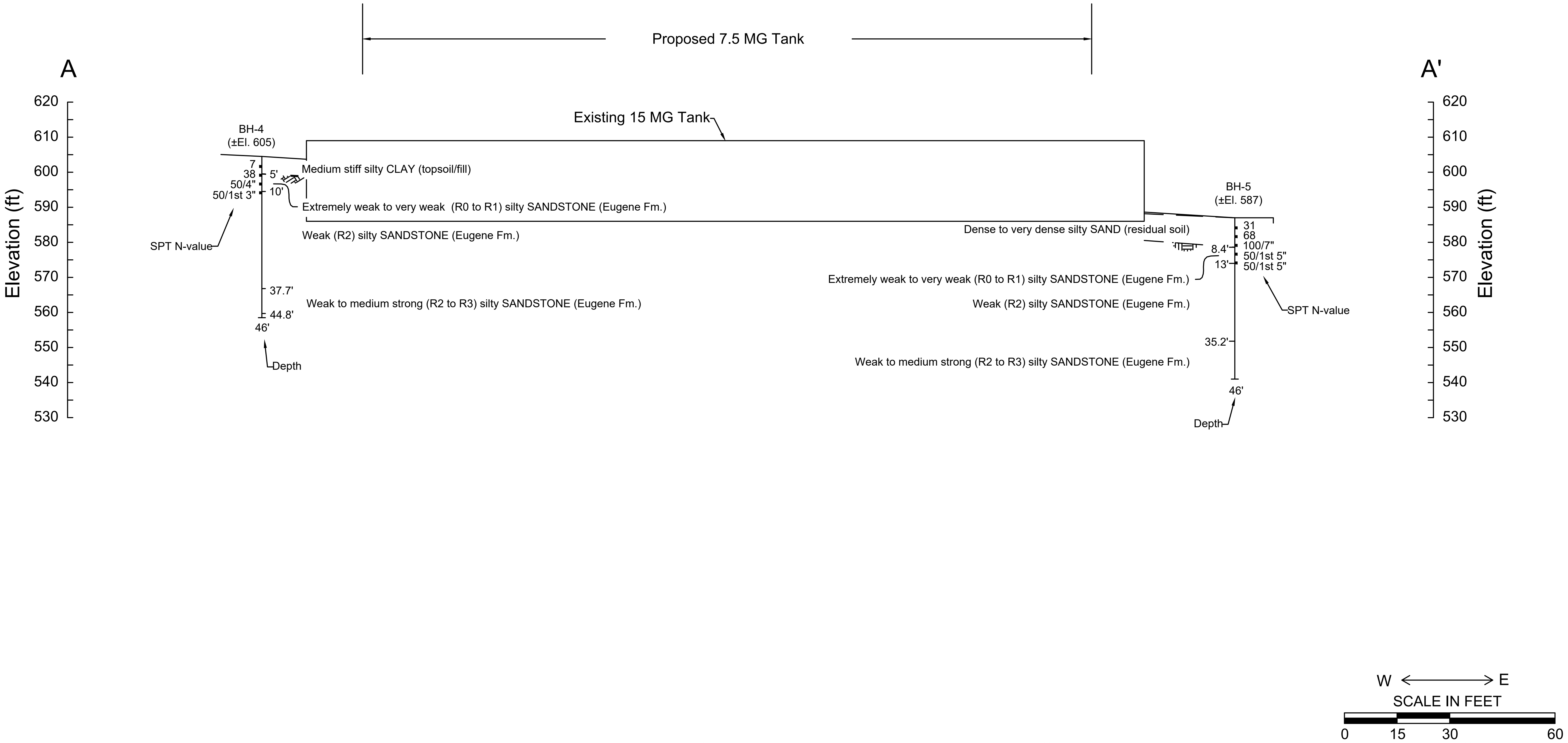
PROJECT NO.	DATE:	DRAWN BY:
2201012	Mar. 29, 2024	MLM

SITE LAYOUT AND BORING LOCATIONS

EXISTING AND PROPOSED RESERVOIRS
COLLEGE HILL 607 STORAGE TANKS
EUGENE, OREGON

FIGURE NO.
2A

FILE: Figure 2A – College Hill Boring Locations (3–27–24).dwg



NOTES:
1. SURFACE CROSS-SECTION IS BASED ON TOPOGRAPHIC DATA PROVIDED BY CONSOR.
2. SUBSURFACE PROFILE WAS INTERPRETED BASED ON THE CONDITIONS ENCOUNTERED IN BH-4 AND BH-5.
3. SEE REPORT FOR A DISCUSSION OF SITE CONDITIONS.
4. GROUND ELEVATIONS REFERENCE NGVD 29.

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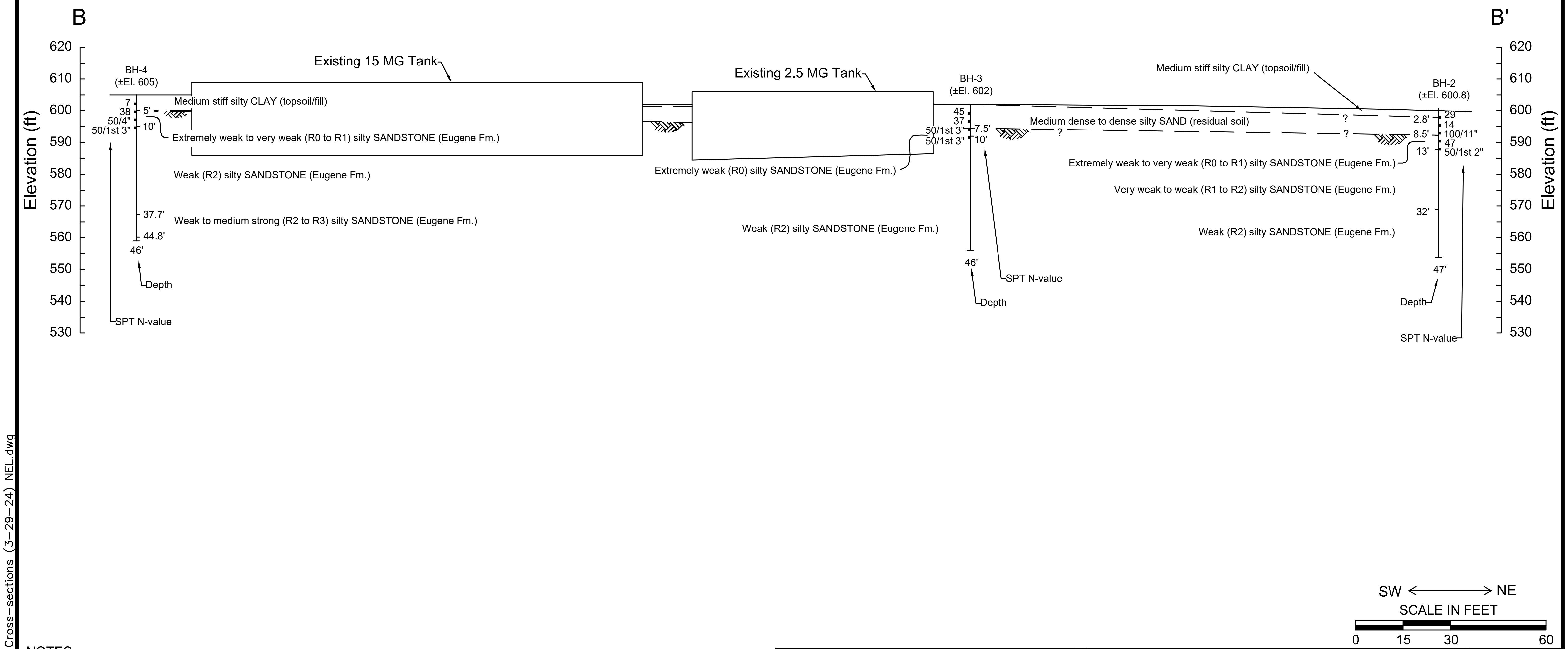
PROJECT NO.	DATE:	DRAWN BY:
2201012	Mar. 29, 2024	MLM

CROSS-SECTION A-A'

COLLEGE HILL 607 STORAGE TANKS
EUGENE, OREGON

FIGURE NO.
3A

FILE: College Hill Cross-sections (3-29-24) NEL.dwg



NOTES:

1. SURFACE CROSS-SECTION IS BASED ON TOPOGRAPHIC DATA PROVIDED BY CONSOR.
2. SUBSURFACE PROFILE WAS INTERPRETED BASED ON THE CONDITIONS ENCOUNTERED IN BH-2, BH-3, AND BH-4.
3. SEE REPORT FOR A DISCUSSION OF SITE CONDITIONS.
4. GROUND ELEVATIONS REFERENCE NGVD 29.

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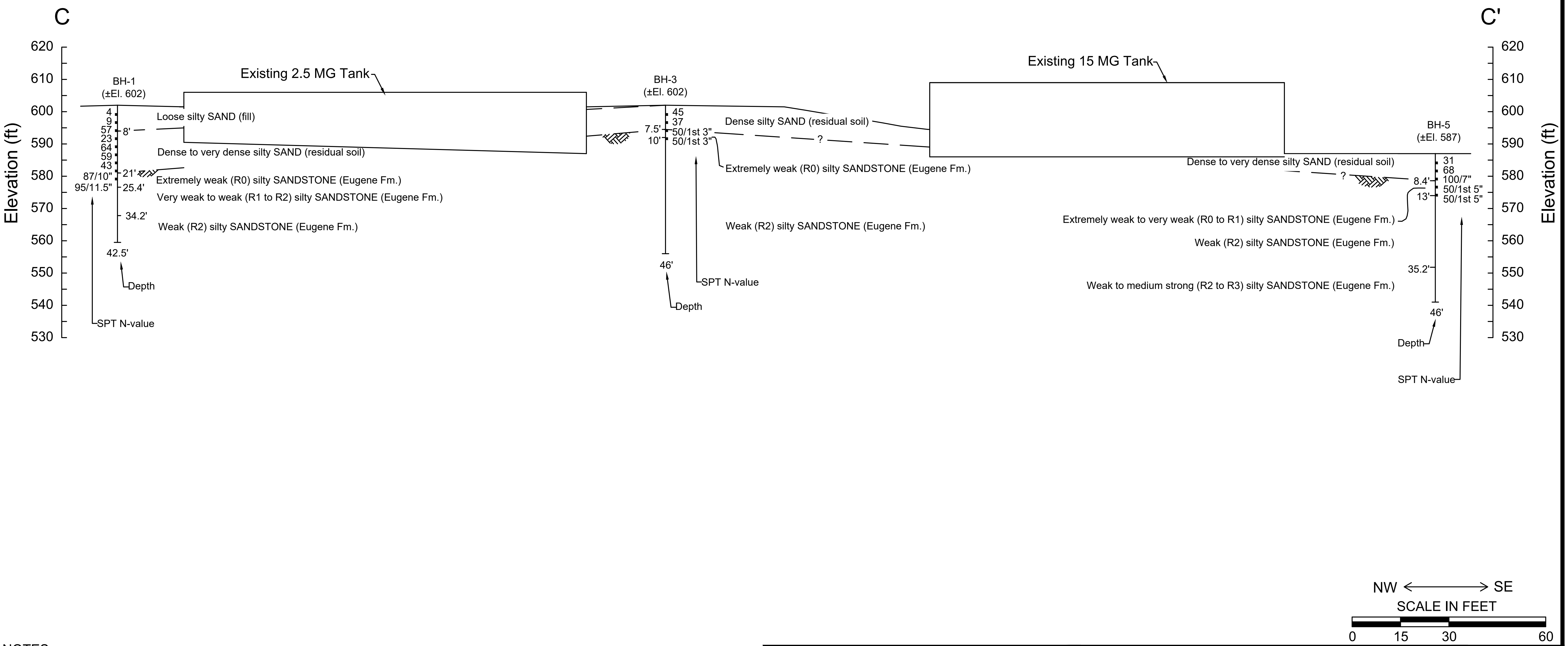
PROJECT NO.	DATE:	DRAWN BY:
2201012	Mar. 29, 2024	MLM

CROSS-SECTION B-B'

COLLEGE HILL 607 STORAGE TANKS
EUGENE, OREGON

FIGURE NO.
4A

FILE: College Hill Cross-sections (3-29-24) NEL.dwg



NOTES:
1. SURFACE CROSS-SECTION IS BASED ON TOPOGRAPHIC DATA PROVIDED BY CONSOR.
2. SUBSURFACE PROFILE WAS INTERPRETED BASED ON THE CONDITIONS ENCOUNTERED IN BH-1, BH-3, AND BH-5.
3. SEE REPORT FOR A DISCUSSION OF SITE CONDITIONS.
4. GROUND ELEVATIONS REFERENCE NGVD 29.

FILE: College Hill Cross-sections (3-29-24) NEL.dwg

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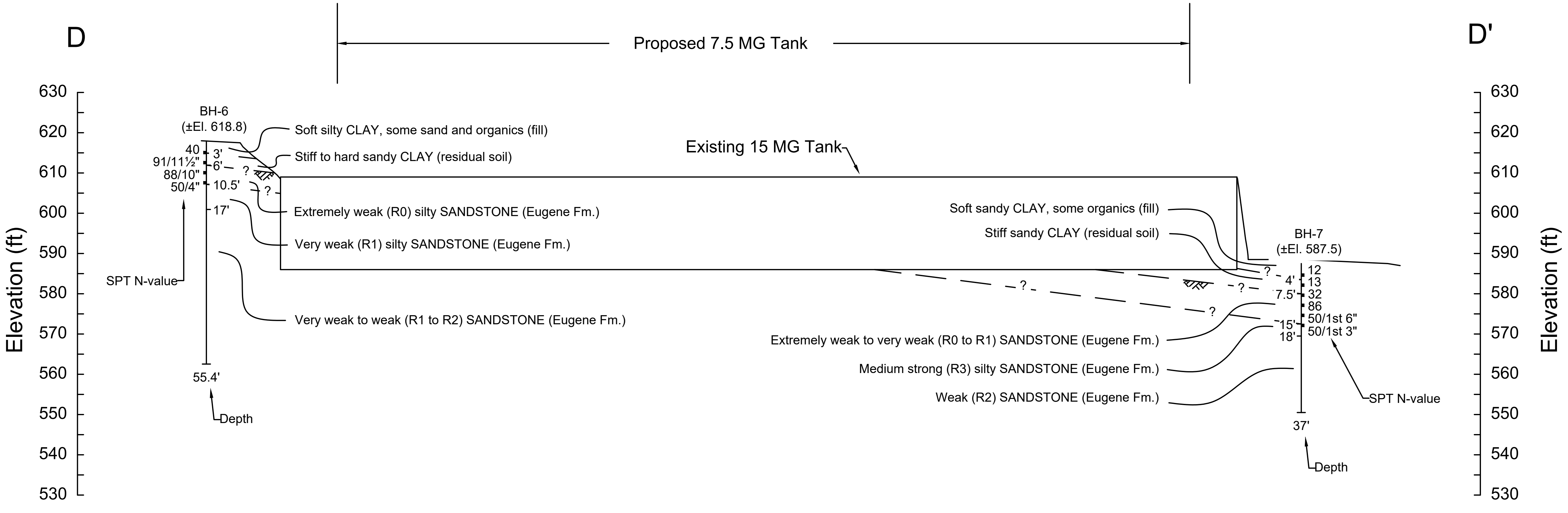
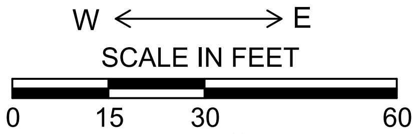
PROJECT NO.	DATE:	DRAWN BY:
2201012	Mar. 29, 2024	MLM

CROSS-SECTION C-C'

COLLEGE HILL 607 STORAGE TANKS
EUGENE, OREGON

FIGURE NO.
5A





NOTES:
1. SURFACE CROSS-SECTION IS BASED ON TOPOGRAPHIC DATA PROVIDED BY CONSOR.
2. SUBSURFACE PROFILE WAS INTERPRETED BASED ON THE CONDITIONS ENCOUNTERED IN BH-6 AND BH-7.
3. SEE REPORT FOR A DISCUSSION OF SITE CONDITIONS.
4. GROUND ELEVATIONS REFERENCE NGVD 29.

Foundation Engineering, Inc.
Professional Geotechnical Services

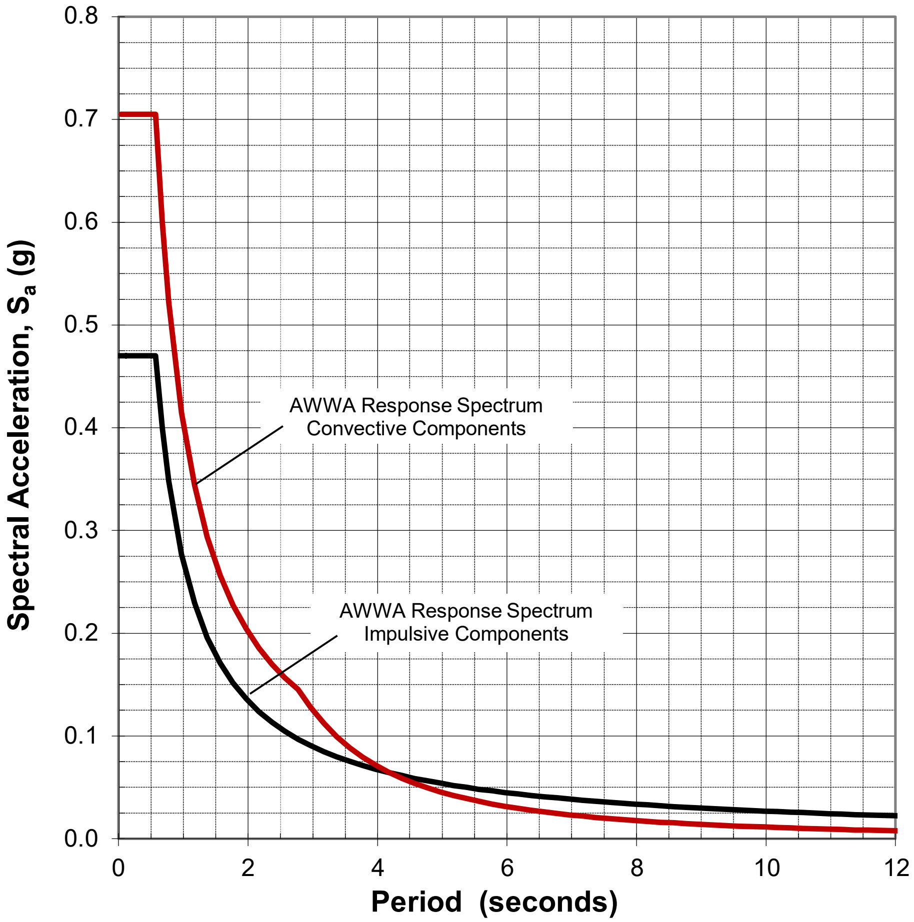
PROJECT NO.	DATE:	DRAWN BY:
2201012	Mar. 29, 2024	MLM

CROSS-SECTION D-D'

COLLEGE HILL 607 STORAGE TANKS
EUGENE, OREGON

FIGURE NO.
6A

FILE: College Hill Cross-sections (3-29-24) NEL.dwg



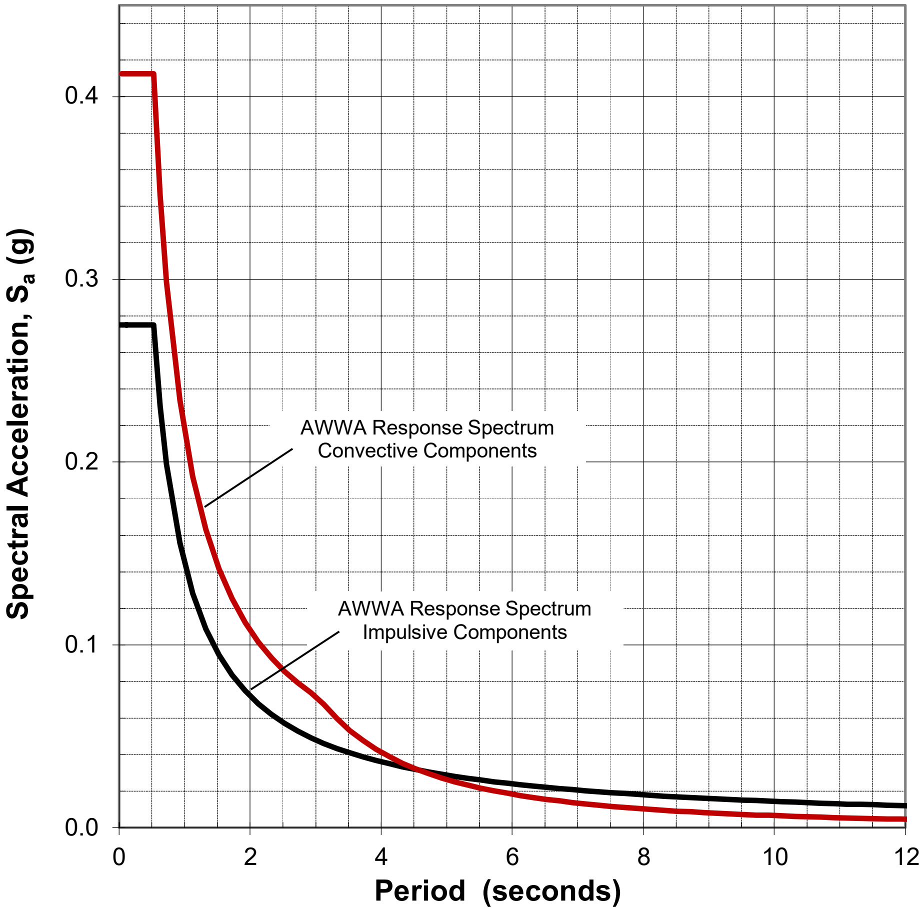
Notes:

1. The Design Response Spectra are based on the General Procedure in AWWA D110-13(R18) 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 =	0.705	F_a =	1.000	S_{MS} =	0.705	S_{DS} =	0.470
S_1 =	0.403	F_v =	1.000	S_{M1} =	0.403	S_{D1} =	0.269

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 convective components were adjusted for 0.5% damping using AWWA D110-13 Eqs. 4-19 and 4-20.
6. Site location is: Latitude 44.0329, Longitude -123.0978.

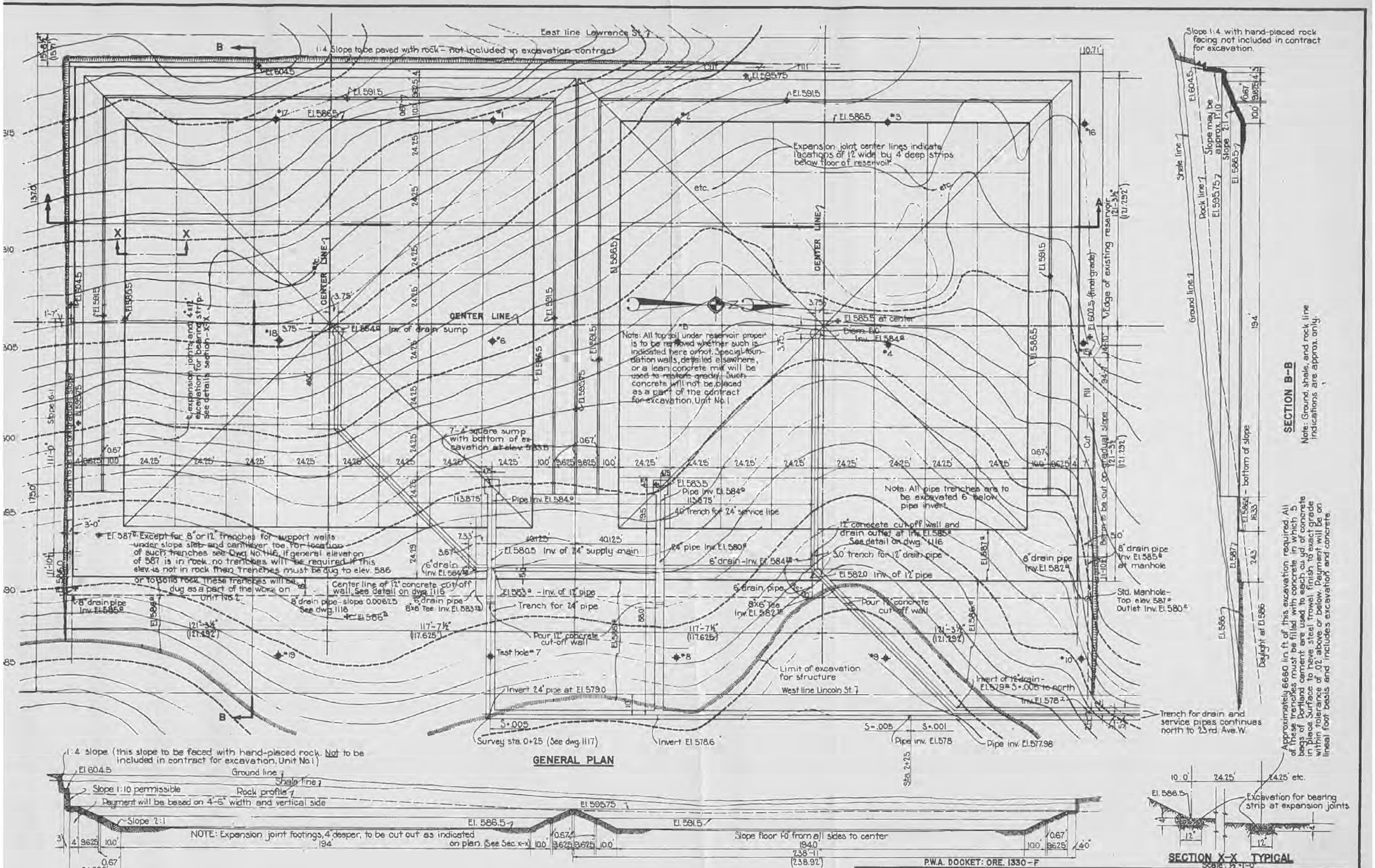
FIGURE 7A
AWWA D110-13 SITE RESPONSE SPECTRA
2% Probability of Exceedence in 50 years
College Hill 607 Storage Tanks
Eugene, Oregon
Project No.: 2201012



Notes:

1. The Design Response Spectra are based on the General Procedure in AWWA D110-13(R18) 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 = 0.275 F_a = 1.000 S_{XS} = 0.275
    S_1 = 0.144 F_v = 1.000 S_{X1} = 0.144
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 convective components were adjusted for 0.5% damping using AWWA D110-13 Eqs. 4-19 and 4-20.
6. Site location is: Latitude 44.0329, Longitude -123.0978.

FIGURE 8A
AWWA D110-13 SITE RESPONSE SPECTRA
10% Probability of Exceedence in 50 years
College Hill 607 Storage Tanks
Eugene, Oregon
Project No.: 2201012



Note: All top soil under reservoir proper is to be removed whether such is indicated here or not. Special foundation walls, detailed elsewhere, or a lean concrete mix will be used to restore grade. Such concrete will not be placed as a part of the contract for excavation, Unit No. 1

El. 587.0 - Except for 8" or 12" trenches for support walls under slope slab and cantilever toe. For location of such trenches see Dwg. No. 1116. If general elevation of 587 is in rock no trenches will be required. If this elev. is not in rock then trenches must be dug to elev. 586 or to solid rock. These trenches will be dug as a part of the work on Unit No. 2.

SECTION A-A

Note: Ground, shale, and rock line indications are approximate only - based on "Log of Test Holes" shown on Dwg. 1114.

P.W.A. DOCKET: ORE. 1330-F

1:13:39	Minor notes added
	REVISIONS

COLLEGE HILL RESERVOIR No. 2
EUGENE - OREGON
RESERVOIR EXCAVATION
Scale: 1" = 20'-0"

Dr: J.C.G. | Tr: G.M.M. | Ch: R.E.K.

R. E. Koon

Approved by Eugene Water Board
J. M. McBrhu
General Supt. - Secretary

STEVENS & KOON
CONSULTING ENGINEERS PORTLAND OREGON

Date: 9-28-38 | Set: 2 of 15 | File: 699-1115

2

Appendix B

Boring Logs and Rock Core Photos

Foundation Engineering, Inc.
Professional Geotechnical Services

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

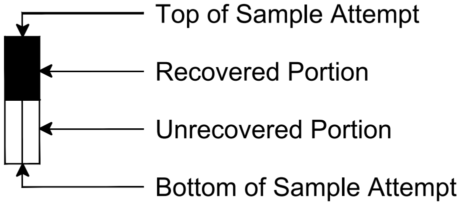
The final log and related information depict subsurface conditions only at the specific location and on the date indicated. Those using the information contained herein should be aware that soil conditions at other locations or on other dates may differ. Actual foundation or subgrade conditions should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

SH - 3 - 4
4 ← Sample Number
3 ← Exploration Number
SH ← Sample Type



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	▥ Silt	▧ Basalt
▢ Organics	░ Sand	░ Sandstone
▨ Clay	▦ Gravel	▤ 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.

Foundation Engineering, Inc.
Professional Geotechnical Services

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.

Foundation Engineering, Inc.
Professional Geotechnical Services

ROCK DESCRIPTIONS

COMMON TERMS

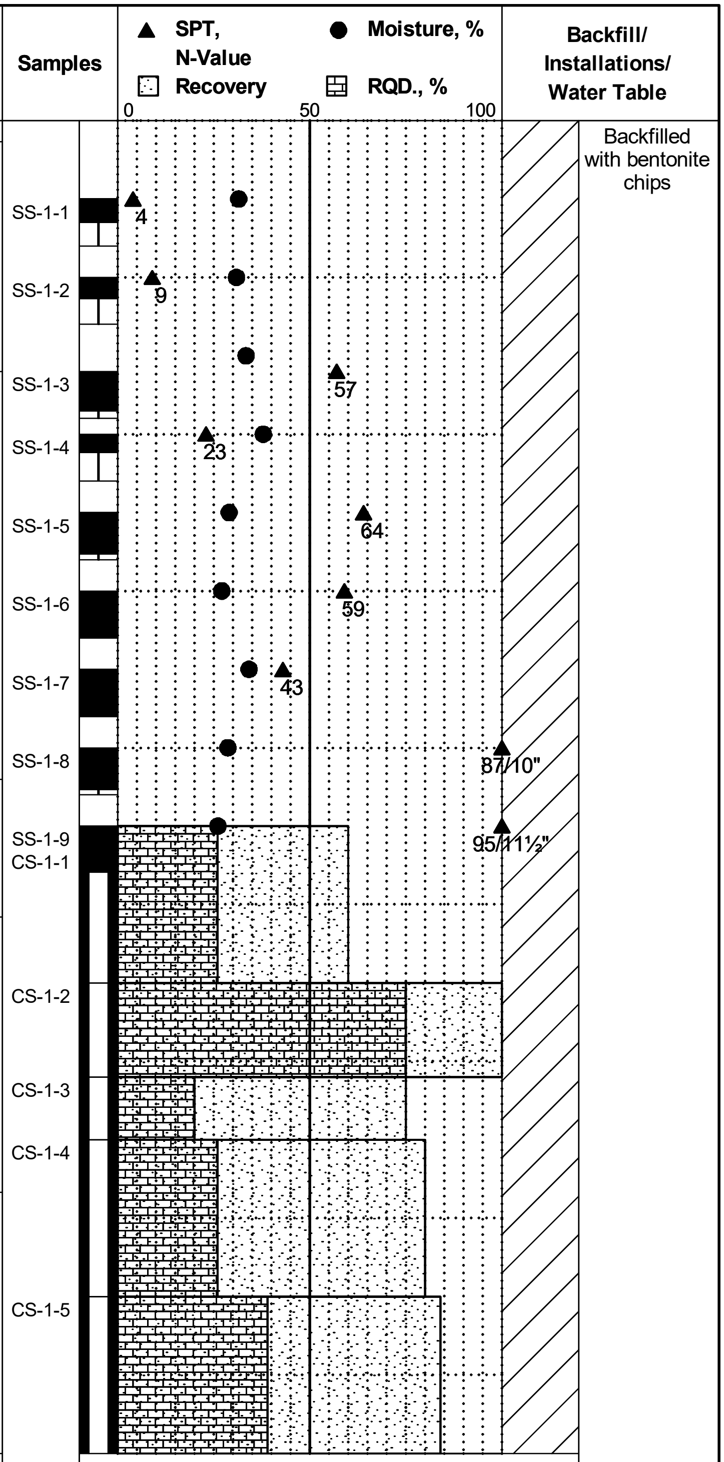
Table 1B. Summary of Boring and Bedrock Elevations

Boring	Approximate Ground Surface Elevation (ft)	Maximum Depth of Boring (ft)	Approximate Bottom of Boring Elevation (ft)	Depth to Bedrock (ft)	Approximate Bedrock Elevation (ft)
BH-1	±El. 602.0	±42.5	±El. 559.5	±21.0	±El. 581.0
BH-2	±El. 600.8	±47.0	±El. 553.8	±8.5	±El. 592.3
BH-3	±El. 602.0	±46.0	±El. 556.0	±7.5	±El. 594.5
BH-4	±El. 605.0	±46.0	±El. 559.0	±5.0	±El. 600.0
BH-5	±El. 587.0	±46.0	±El. 541.0	±8.4	±El. 578.6
BH-6	±El. 618.8	±55.4	±El. 563.4	±6.0	±El. 612.8
BH-7	±El. 587.5	±37.0	±El. 550.5	±7.5	±El. 580.0

Notes:

1. Elevations reference the NGVD 29 datum.
2. Ground surface elevations are approximate and were interpolated from the available topographic map.

Depth Feet	Soil and Rock Description and Comments	Elev. Depth
		602
1	Soft silty CLAY, some sand (CL); dark brown, moist, low plasticity, fine sand, (topsoil).	601.3 0.7
2 3 4 5 6 7 8	Loose silty SAND (SM); brown and iron-stained, moist, low plasticity silt, fine to coarse sand, (fill). Lost ±180 gallons of drilling fluid at ±5 feet.	594.0 8.0
9 10 11 12 13 14 15 16 17 18 19 20 21	Dense to very dense silty SAND (SM); light brown and iron-stained, moist, low plasticity silt, fine sand, (residual soil). Medium dense from ±10 to 11.5 feet. Silt, some sand lens at ±11 feet.	581.0 21.0
22 23 24 25	Extremely weak (R0) silty SANDSTONE; light brown and iron-stained, decomposed to highly weathered, fine sand, (Eugene Formation).	576.6 25.4
26 27 28 29 30 31 32 33 34	Very weak to weak (R1 to R2) silty SANDSTONE: brown and iron-stained, highly weathered, very close to close joints are irregular, rough to very rough, open to closed, and iron and manganese-stained, fine sand, (Eugene Formation).	567.8 34.2
35 36 37 38 39 40 41 42	Weak (R2) silty SANDSTONE; dark grey, moderately to slightly weathered, close joints are planar to irregular, rough, and open to closed, fine sand, (Eugene Formation). Iron-stained joint at ±36.5 feet. Slightly weathered to fresh below ±38.1 feet.	559.5 42.5
	BOTTOM OF BORING	



Project No.: 2201012

Surface Elevation: 602.0 feet (Approx.)

Date of Boring: March 3, 2020

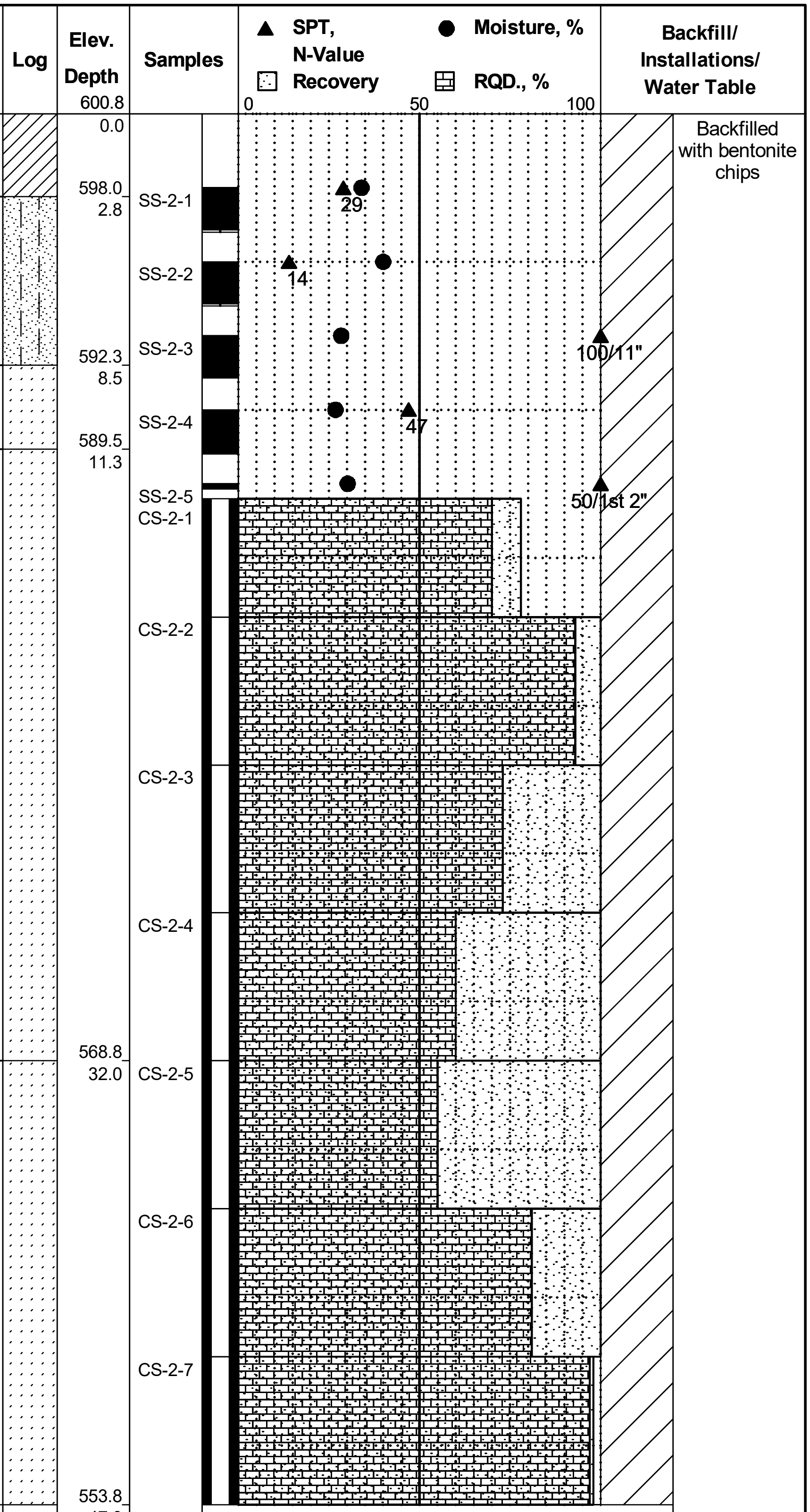
Foundation Engineering, Inc.

Boring Log: BH-1

College Hill 607 Storage Tanks

Eugene, Oregon

Depth Feet	Soil and Rock Description and Comments	Elev. Depth	Samples
		600.8	
1 2	Medium stiff silty CLAY, trace sand (CL); dark brown, wet, low to medium plasticity, fine sand, (topsoil).	0.0	
3 4 5 6 7 8	Medium dense silty SAND (SM); brown and iron-stained, moist, low plasticity silt, fine sand, (residual soil).	598.0 2.8	SS-2-1 SS-2-2 SS-2-3
9 10 11	Extremely weak to very weak (R0 to R1) silty SANDSTONE; brown and iron-stained, highly to moderately weathered, fine sand, (Eugene Formation).	592.3 8.5	SS-2-4
12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31	Very weak (R1) silty SANDSTONE; dark grey, moderately to slightly weathered, fine sand, (Eugene Formation). Organics consisting of wood fragments from ±13 to 17 feet. Weak (R2), close to moderately close joints are irregular, rough, and closed, and scattered shells below ±13 feet. Some iron-stained joints below ±17 feet. Grey, open to closed joints, and fine to medium sand below ±22 feet. Extremely weak to very weak (R0 to R1), light brown, and decomposed to highly weathered sandy siltstone interbed from ±27 to 28.9 feet.	589.5 11.3	SS-2-5 CS-2-1 CS-2-2 CS-2-3 CS-2-4
32 33 34 35 36 37 38 39 40 41 42 43 44 45 46	Weak (R2) silty SANDSTONE; dark grey, slightly weathered to fresh, close to moderately close joints are planar to irregular, rough, and closed, fine to medium sand, scattered shells, (Eugene Formation). Healed joints at ±34 feet. Healed joint at ±45.4 feet.	568.8 32.0	CS-2-5 CS-2-6 CS-2-7
47	BOTTOM OF BORING	553.8 47.0	



Project No.: 2201012

Surface Elevation: 600.8 feet (Approx.)

Date of Boring: March 4, 2020

Foundation Engineering, Inc.

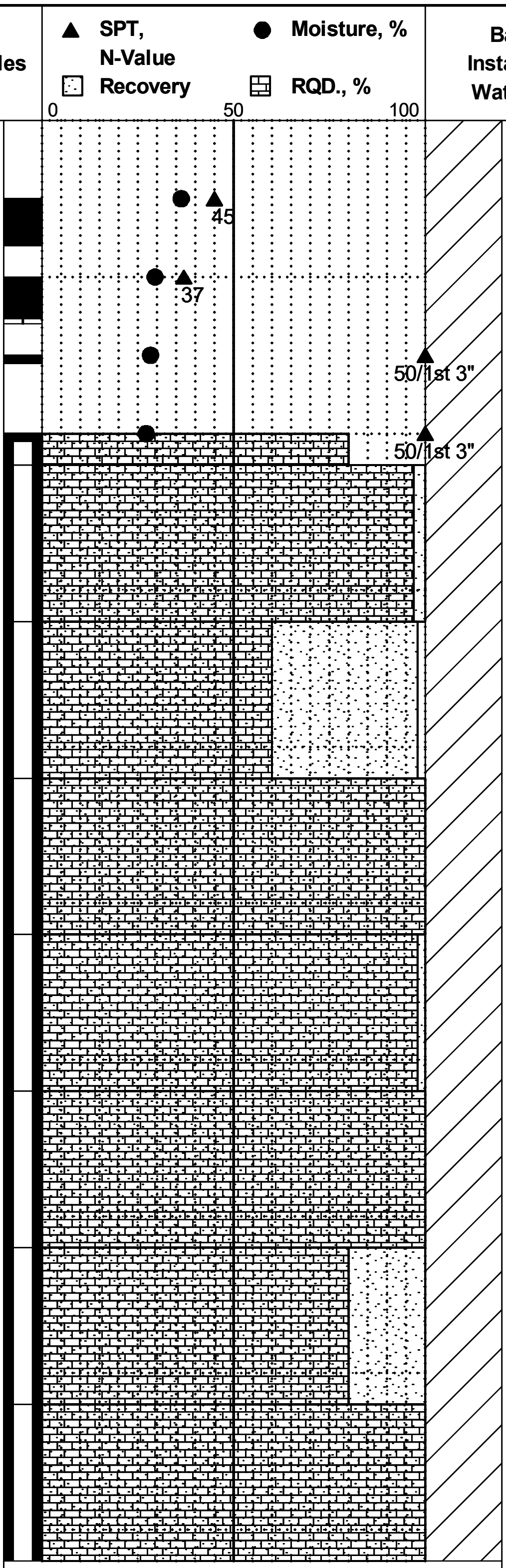
Boring Log: BH-2

College Hill 607 Storage Tanks

Eugene, Oregon

Depth Feet	Soil and Rock Description and Comments	Elev. Depth 602	Samples	Backfill/ Installations/ Water Table
1	Dense silty SAND (SM); brown and iron-stained, moist, low plasticity silt, fine sand, relict sandstone texture, (residual soil).	0.0		Backfilled with bentonite chips
3			SS-3-1	
6			SS-3-2	
8	Extremely weak (R0) silty SANDSTONE; dark grey, moderately weathered, fine sand, (Eugene Formation).	594.5 7.5	SS-3-3	
10	Weak (R2) silty SANDSTONE; dark grey, slightly weathered to fresh, close to moderately close joints are planar to irregular, rough to very rough, and closed, fine sand, scattered shells, (Eugene Formation).	592.0 10.0	SS-3-4 CS-3-1 CS-3-2	
14	Fine to medium sand from ±13.7 to 14 feet and from ±15.3 to 15.5 feet.			
16			CS-3-3	
17	Weak (R2) and dark grey sandy siltstone interbed from ±16.4 to 18.7 feet.			
21			CS-3-4	
26			CS-3-5	
31			CS-3-6	
36			CS-3-7	
41			CS-3-8	
43	Some healed joints below ±42.5 feet.			
46	BOTTOM OF BORING	556.0 46.0		

▲ SPT, N-Value ● Moisture, %
▨ Recovery ▤ RQD., %



Project No.: 2201012

Surface Elevation: 602.0 feet (Approx.)

Date of Boring: March 5, 2020

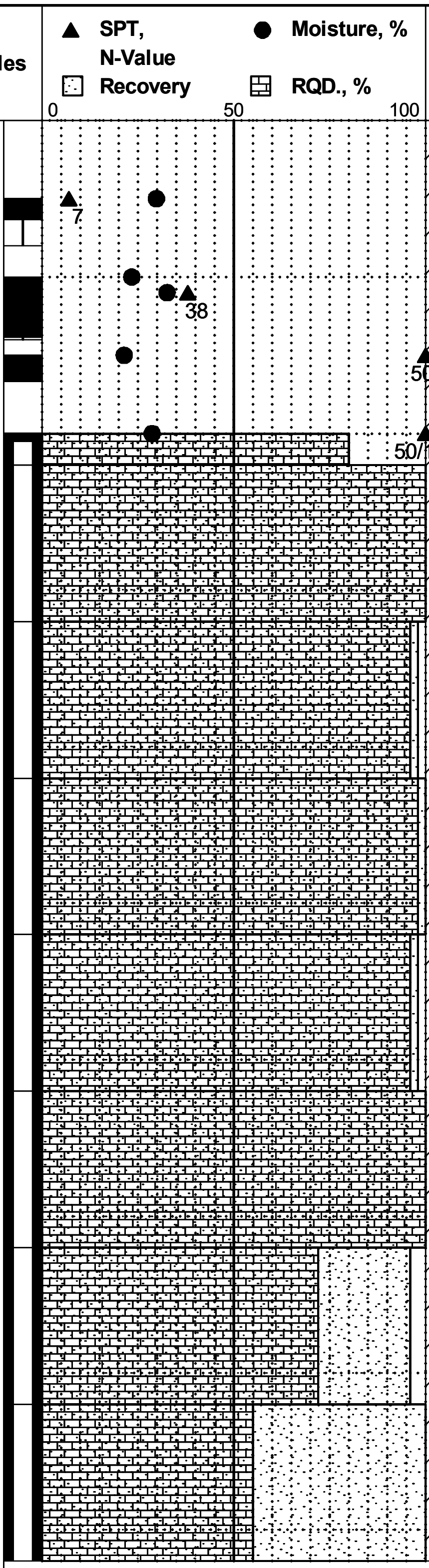
Foundation Engineering, Inc.

Boring Log: BH-3

College Hill 607 Storage Tanks

Eugene, Oregon

Depth Feet	Soil and Rock Description and Comments	Log	Elev. Depth 605	Samples	Backfill/ Installations/ Water Table
1	Medium stiff silty CLAY, trace to some sand (CL); dark brown, moist, low to medium plasticity, fine sand, (topsoil/fill).		0.0		Backfilled with bentonite chips
2					
3				SS-4-1	
4					
5	Extremely weak to very weak (R0 to R1) silty SANDSTONE; brown and iron-stained, decomposed to highly weathered, fine sand, (Eugene Formation).		600.0 5.0	SH-4-2	
6				SS-4-3	
7					
8	Light brown and iron-stained below ±7.5 feet.			SS-4-4	
9					
10	Weak (R2) silty SANDSTONE; dark grey, slightly weathered, fine sand, (Eugene Formation).		595.0 10.0	SS-4-5	
11				CS-4-1	
12				CS-4-2	
13	Slightly weathered to fresh, close to moderately close joints are planar to irregular, rough, and closed, and scattered shells below ±11 feet.				
14					
15					
16				CS-4-3	
17					
18					
19					
20					
21				CS-4-4	
22					
23					
24					
25					
26				CS-4-5	
27					
28					
29					
30	Fine to medium sand below ±29.2 feet.				
31				CS-4-6	
32					
33					
34					
35					
36				CS-4-7	
37					
38	Weak to medium strong (R2 to R3) silty SANDSTONE; grey, slightly weathered, close to moderately close joints are planar to irregular, rough to very rough, and closed, fine to coarse sand, (Eugene Formation).		567.3 37.7		
39					
40					
41				CS-4-8	
42					
43					
44					
45	Weak (R2), dark grey, and close joints are irregular to planar, rough, and closed, and some vertical below ±44.8 feet.				
46			559.0 46.0		
	BOTTOM OF BORING				



Project No.: 2201012

Surface Elevation: 605.0 feet (Approx.)

Date of Boring: March 5, 2020

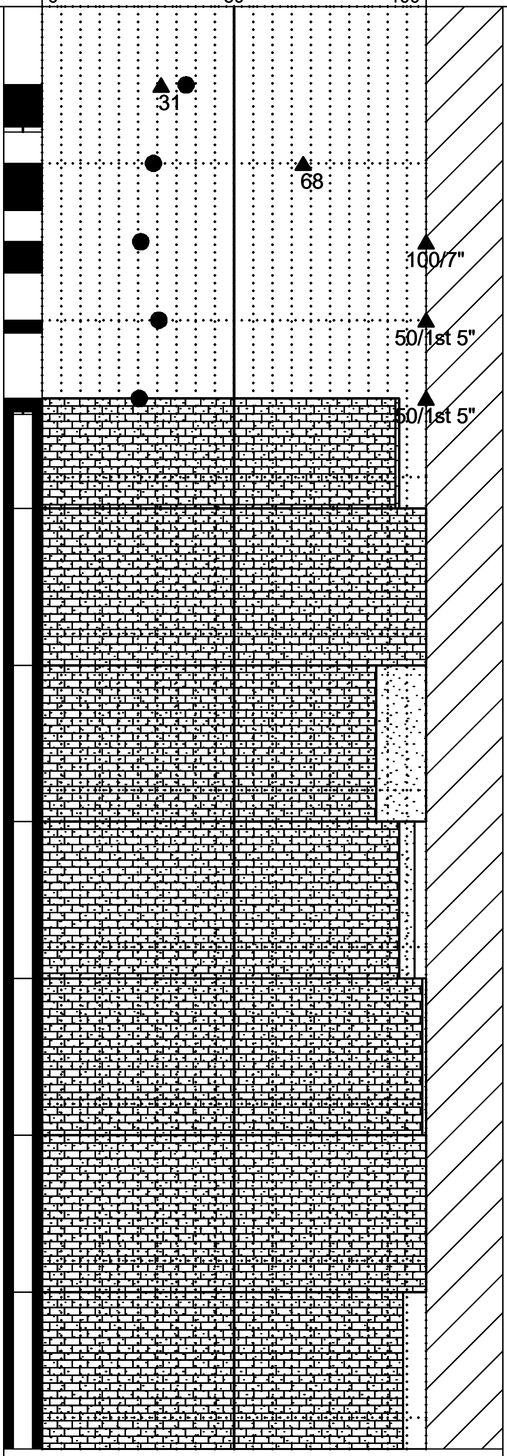
Foundation Engineering, Inc.

Boring Log: BH-4

College Hill 607 Storage Tanks

Eugene, Oregon

Depth Feet	Soil and Rock Description and Comments	Log	Elev. Depth 587	Samples	▲ SPT, N-Value ● Moisture, % ⬚ Recovery ⊟ RQD., %	Backfill/ Installations/ Water Table
1	Dense to very dense silty SAND (SM); light brown and iron-stained, moist, low plasticity silt, fine sand, relict sandstone texture, (residual soil).		0.0	SS-5-1 SS-5-2 SS-5-3		Backfilled with bentonite chips
9	Extremely weak to very weak (R0 to R1) silty SANDSTONE; dark grey, moderately to slightly weathered, fine sand, (Eugene Formation).		578.6 8.4	SS-5-4		
14	Weak (R2) silty SANDSTONE; dark grey, slightly weathered to fresh, close to wide joints are irregular, rough, and closed, fine sand, scattered shells, (Eugene Formation).		574.0 13.0	SS-5-5 CS-5-1 CS-5-2		
22	Some healed joints from ±22 to 23.7 feet.			CS-5-3 CS-5-4		
34	Scattered organics consisting of wood fragments below ±33 feet.			CS-5-5		
36	Weak to medium strong (R2 to R3) silty SANDSTONE; dark grey, slightly weathered to fresh, moderately close to wide joints are planar to irregular, rough to very rough, and closed, fine to coarse sand, (Eugene Formation).		551.8 35.2	CS-5-6 CS-5-7		
46	BOTTOM OF BORING		541.0 46.0			



Project No.: 2201012

Surface Elevation: 587.0 feet (Approx.)

Date of Boring: March 2, 2020

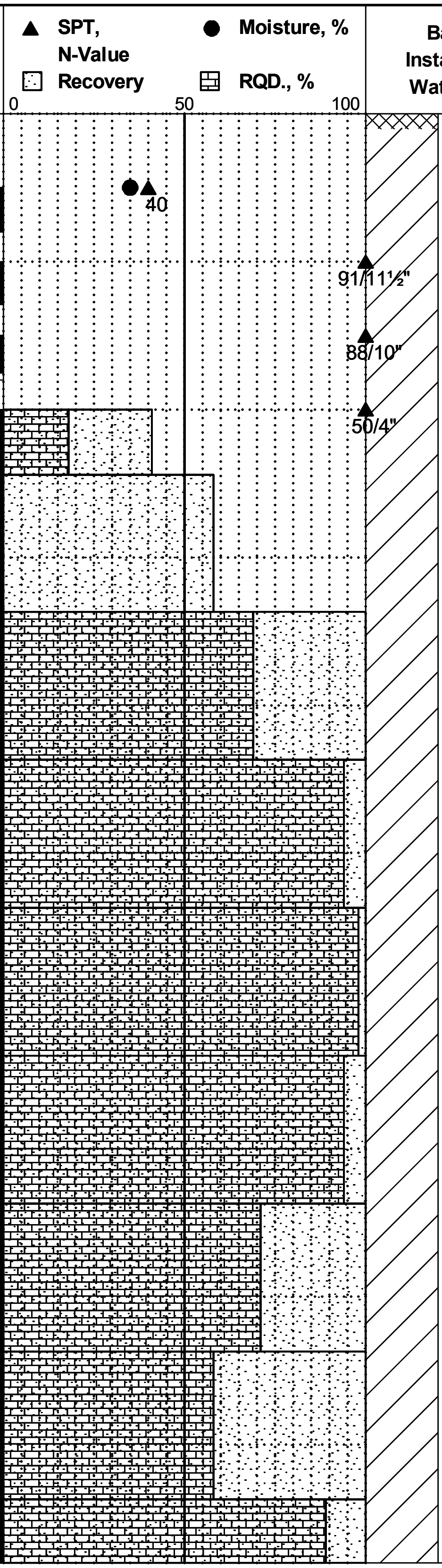
Foundation Engineering, Inc.

Boring Log: BH-5

College Hill 607 Storage Tanks

Eugene, Oregon

Depth Feet	Soil and Rock Description and Comments	Elev. Depth	Samples
0	Soft silty CLAY, some sand and organics (CL); dark brown, moist, medium plasticity, fine sand, organics consist of roots and wood fragments, (topsoil/fill).	618.8 0.0	
3	Stiff to hard sandy CLAY (CL); light brown and iron-stained, moist, low to medium plasticity, fine sand, (residual soil).	615.8 3.0	SS-6-1 SS-6-2
6	Extremely weak (R0) silty SANDSTONE; light brown and iron-stained, decomposed grading to moderately weathered, iron-stained joints, fine to medium sand, (Eugene Formation).	612.8 6.0	SS-6-3
10.5	Very weak (R1) silty SANDSTONE; light brown and iron-stained, moderately weathered, very close to close joints are planar to irregular, rough to very rough, open, and iron- and manganese-stained, fine sand, (Eugene Formation). Extremely weak (R0) and highly weathered claystone interbeds from ±14.5 to 16 feet.	608.3 10.5	SS-6-4 CS-6-1 CS-6-2
17	Very weak to weak (R1 to R2) SANDSTONE; dark grey-green, fresh, close to moderately close joints are planar to irregular, rough, and closed, fine sand, scattered shells, (Eugene Formation). Fine to coarse sand at ±19 feet. Scattered healed hairline joints below ±19 feet. Fine to medium sand at ±22.6 feet. Weak (R2) and wide joints from ±27 to 38 feet. Close to moderately close joints from ±38 to 48 feet. Very weak (R1), silty, grey, and slightly weathered to fresh interbed from ±39 to 43 feet. Calcite veining from ±43 to 46 feet. Moderately close to wide joints below ±48 feet.	601.8 17.0	CS-6-3 CS-6-4 CS-6-5 CS-6-6 CS-6-7 CS-6-8 CS-6-9



Backfill/ Installations/ Water Table

Capped with soil

Backfilled with bentonite chips

Project No.: 2201012

Surface Elevation: 618.8 feet (Approx.)

Date of Boring: January 23, 2024

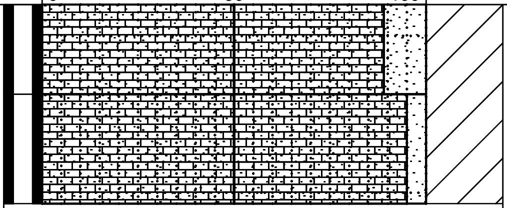
Boring Log: BH-6

College Hill 607 Storage Tanks

Eugene, Oregon

Foundation Engineering, Inc.

Depth Feet	Soil and Rock Description and Comments	Log	Elev. Depth	Samples	▲ SPT, N-Value ● Moisture, % ⬚ Recovery ⊞ RQD., %	Backfill/ Installations/ Water Table
			569.8		0 50 100	
50						
51						
52				CS-6-10		
53						
54						
55			563.5 55.4			
	BOTTOM OF BORING					



Project No.: 2201012

Surface Elevation: 618.8 feet (Approx.)

Date of Boring: January 23, 2024

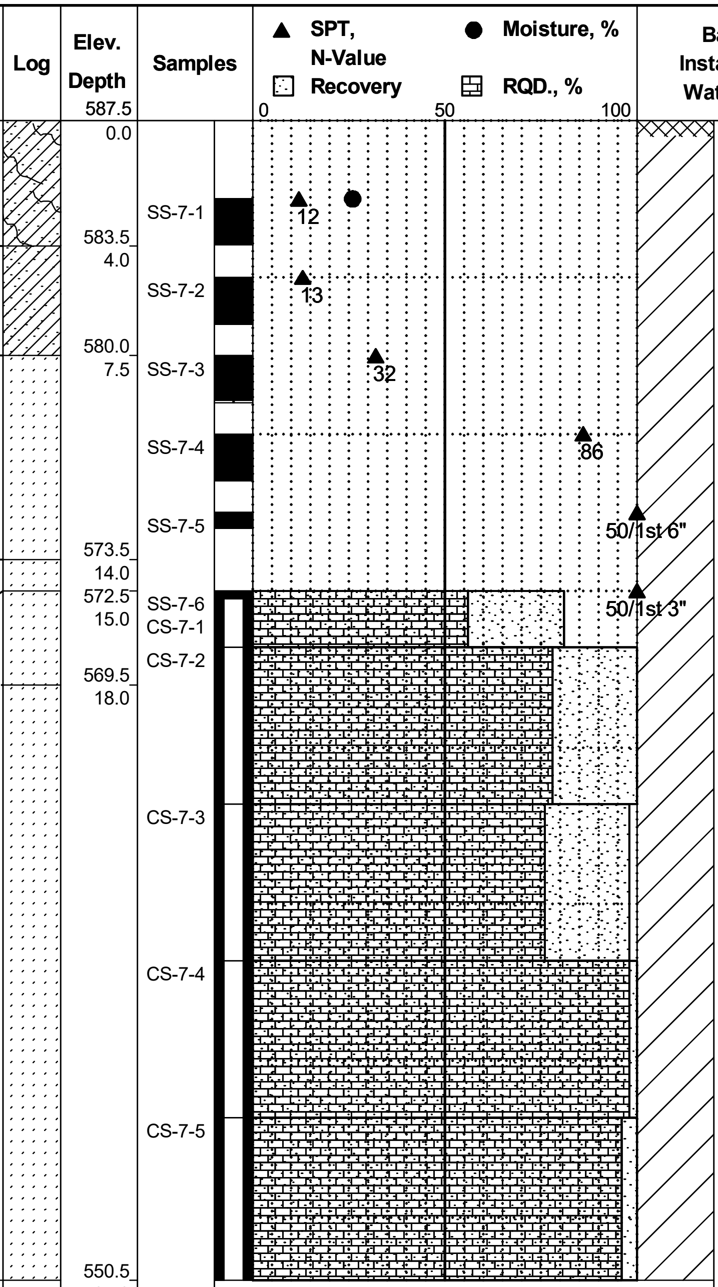
Foundation Engineering, Inc.

Boring Log: BH-6

College Hill 607 Storage Tanks

Eugene, Oregon

Depth Feet	Soil and Rock Description and Comments	Elev. Depth	Samples	SPT, N-Value	Backfill/ Installations/ Water Table
0	Soft to stiff sandy CLAY, some organics (CL); brown to dark brown, moist, medium plasticity, fine to medium sand, organics consist of wood fragments, (topsoil/fill).	587.5 0.0	SS-7-1	12	Capped with soil Backfilled with bentonite chips
4	Stiff sandy CLAY (CL); light brown, moist, medium plasticity, fine to medium sand, (residual soil).	583.5 4.0	SS-7-2	13	
7.5	Extremely weak to very weak (R0 to R1) SANDSTONE; light brown, decomposed to highly weathered, iron-stained joints, fine sand, (Eugene Formation).	580.0 7.5	SS-7-3 SS-7-4	32 86	
12.5	Moderately weathered below ±12.5 feet.		SS-7-5	50/1st 6"	
14	Very weak (R1) SANDSTONE; blue-grey, slightly weathered, fine sand, (Eugene Formation).	573.5 14.0			
15	Medium strong (R3) silty SANDSTONE; grey, slightly weathered, close joints are planar, rough, and open, fine to coarse sand, (Eugene Formation).	572.5 15.0	SS-7-6 CS-7-1	50/1st 3"	
18	Weak (R2) SANDSTONE; dark grey, slightly weathered to fresh, close joints are planar, rough, and closed, fine sand, (Eugene Formation).	569.5 18.0	CS-7-2 CS-7-3		
25	Scattered shells below ±25 feet.				
26	Moderately close to wide joints and healed hairline joints below ±26 feet.		CS-7-4		
31	Lenses of silt and fine sand from ±31 to 33.5 feet.		CS-7-5		
37	BOTTOM OF BORING	550.5 37.0			



Project No.: 2201012

Surface Elevation: 587.5 feet (Approx.)

Date of Boring: January 19, 2024

Foundation Engineering, Inc.

Boring Log: BH-7

College Hill 607 Storage Tanks

Eugene, Oregon

Photo 1B. BH-1 from 22.5 to 34.3 ft - Box 1 of 2

Photo 2B. BH-1 from 34.3 to 42.5 ft - Box 2 of 2

Photo 3B. BH-2 from 13.0 to 22.0 ft - Box 1 of 4

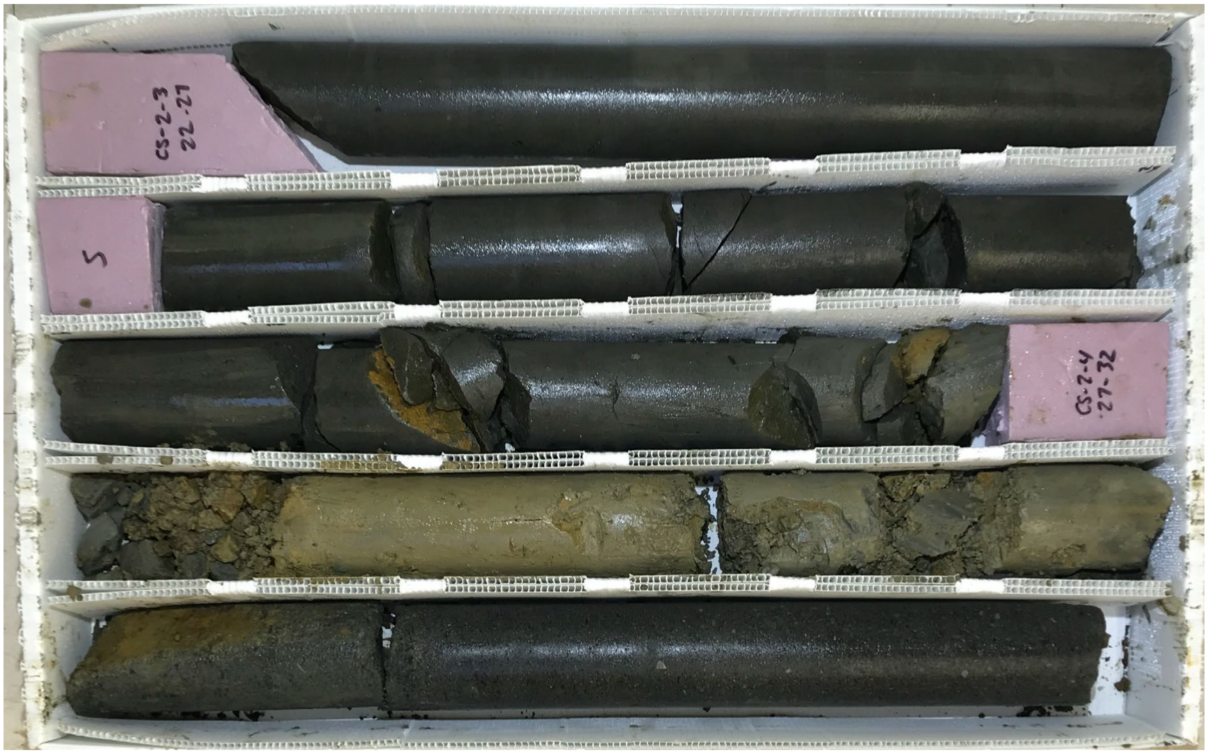


Photo 4B. BH-2 from 22.0 to 30.9 ft - Box 2 of 4

Photo 5B. BH-2 from 30.9 to 38.5 ft - Box 3 of 4

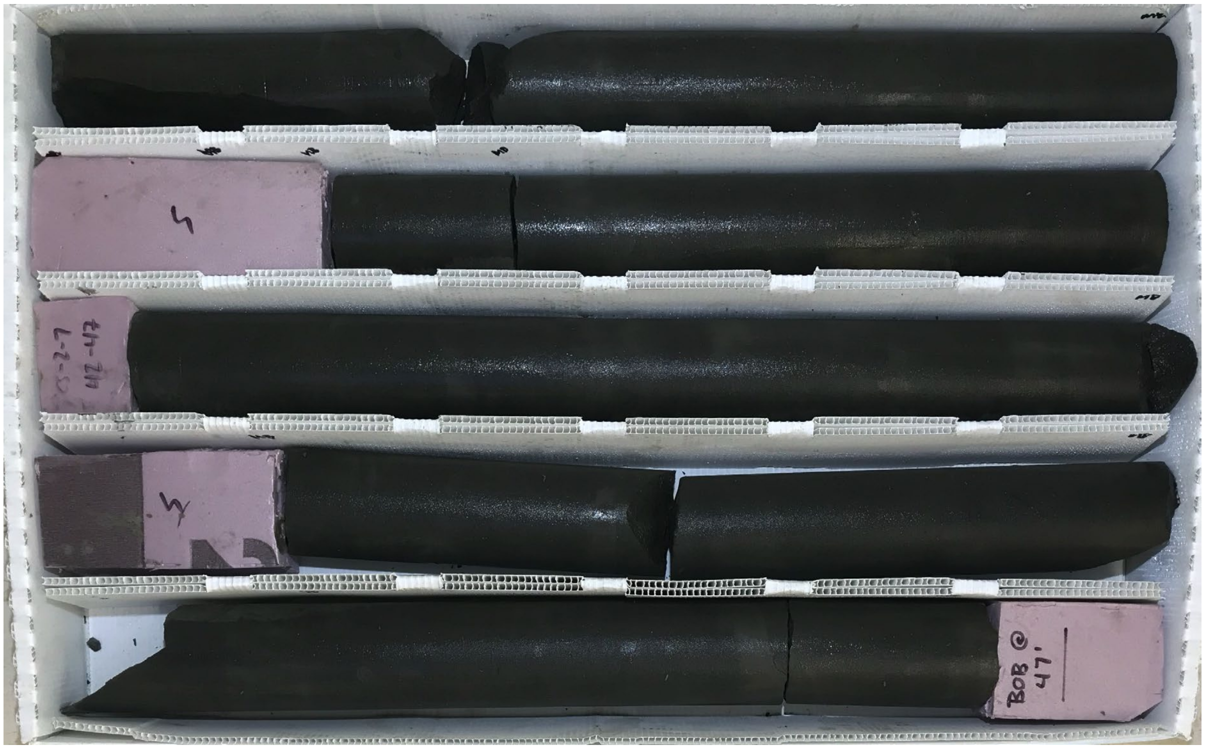


Photo 6B. BH-2 from 38.5 to 47.0 ft - Box 4 of 4

Photo 7B. BH-3 from 10.0 to 20.3 ft - Box 1 of 4

Photo 8B. BH-3 from 20.3 to 27.8 ft - Box 2 of 4

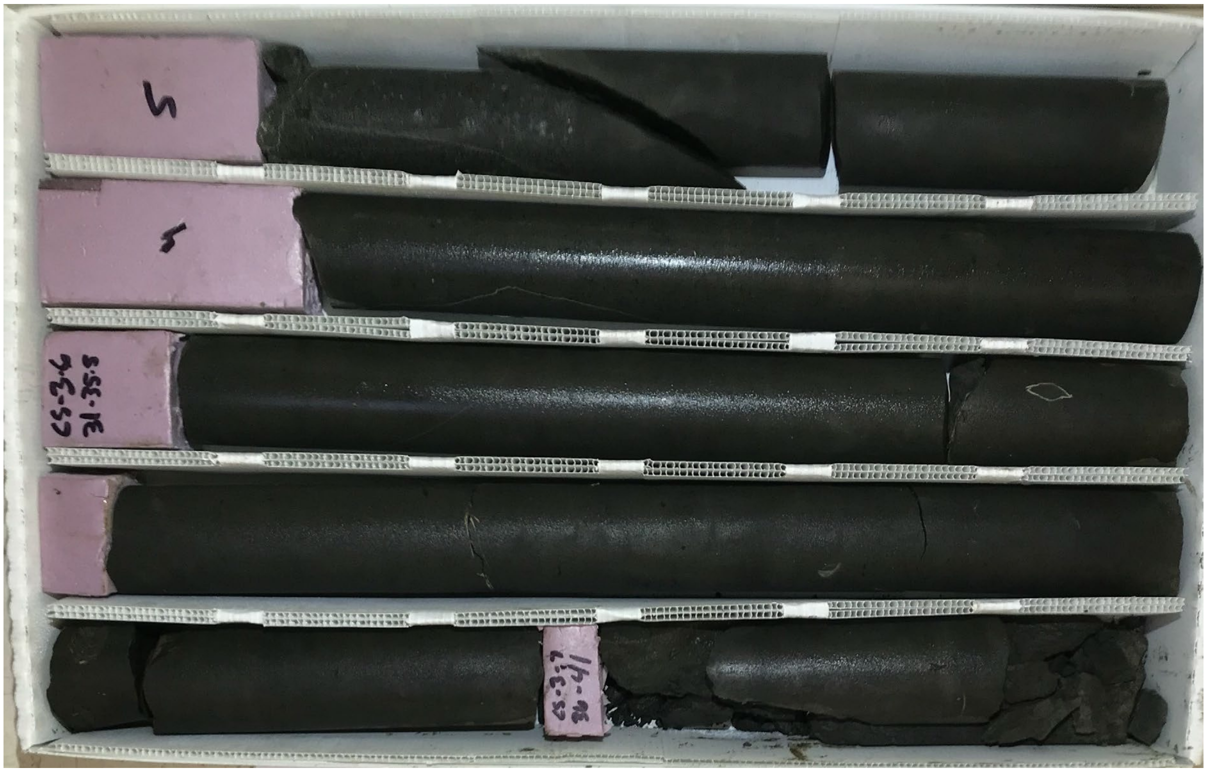


Photo 9B. BH-3 from 27.8 to 37.0 ft - Box 3 of 4

Photo 10B. BH-3 from 37.0 to 46.0 ft - Box 4 of 4

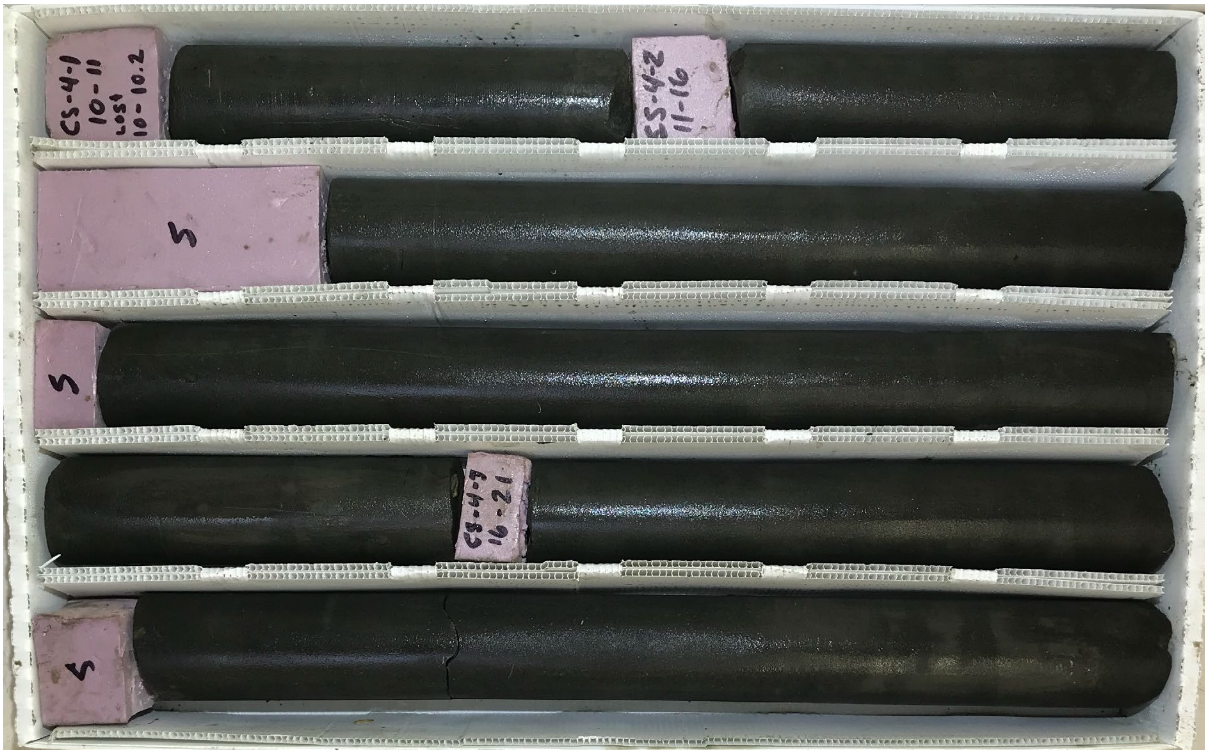


Photo 11B. BH-4 from 10.0 to 19.2 ft - Box 1 of 4

Photo 12B. BH-4 from 19.2 to 28.5 ft - Box 2 of 4

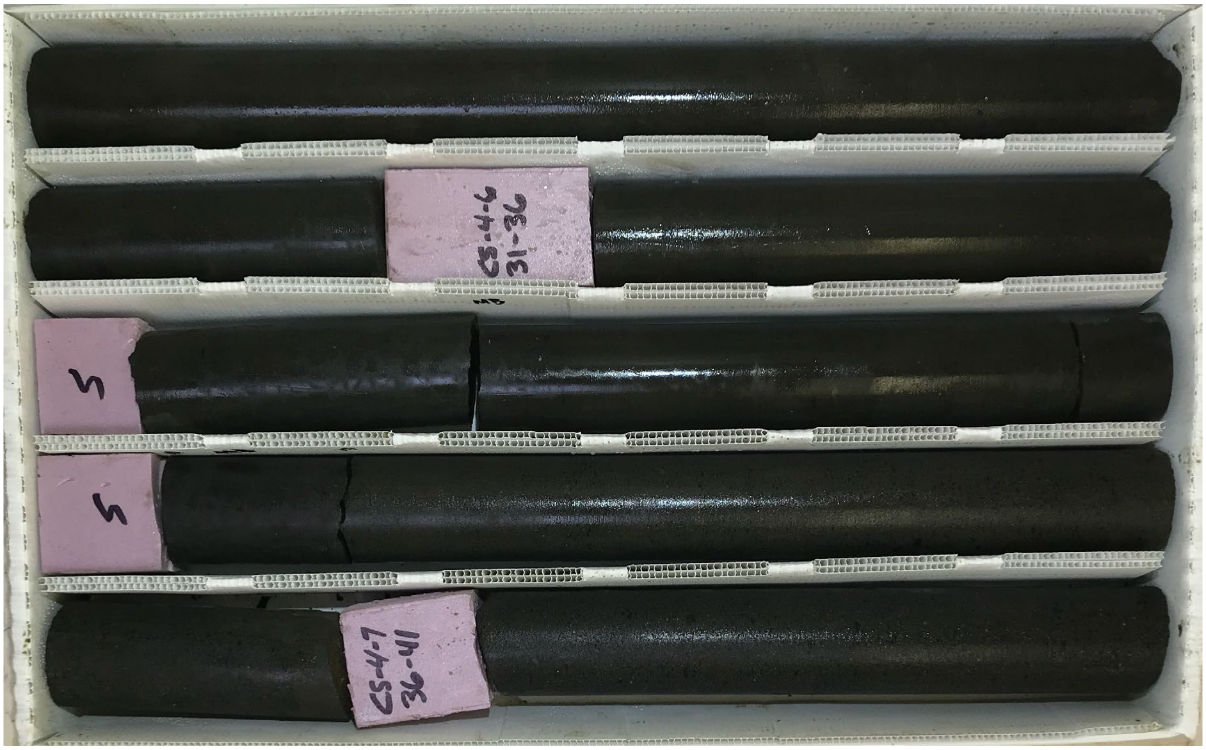


Photo 13B. BH-4 from 28.5 to 37.3 ft - Box 3 of 4

Photo 14B. BH-4 from 37.3 to 46.0 ft - Box 4 of 4

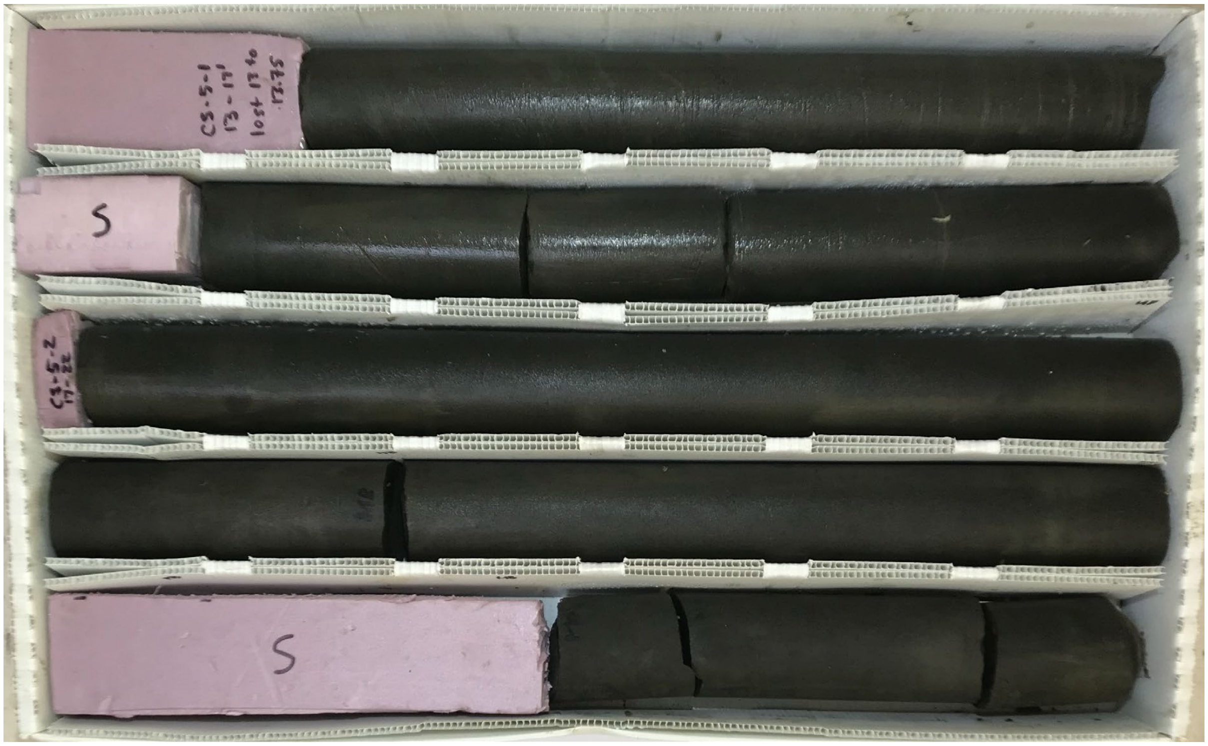


Photo 15B. BH-5 from 13.0 to 22.0 ft - Box 1 of 4

Photo 16B. BH-5 from 22.0 to 31.6 ft - Box 2 of 4

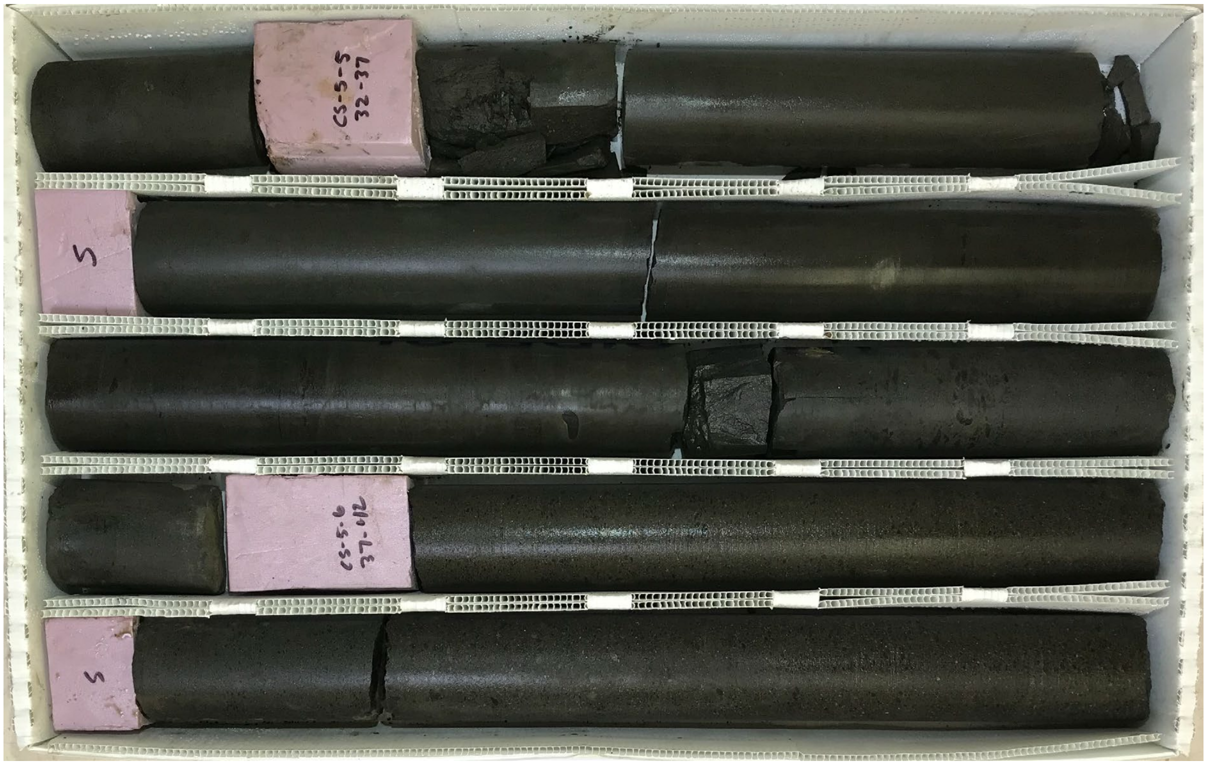


Photo 17B. BH-5 from 31.6 to 40.1 ft - Box 3 of 4

Photo 18B. BH-5 from 40.1 to 47.0 ft - Box 4 of 4

Foundation Engineering, Inc.
College Hill 607 Storage Tanks
Project No.: 2201012

Photo 19B. BH-6 from 10.0 to 21.85 ft - Box 1 of 5

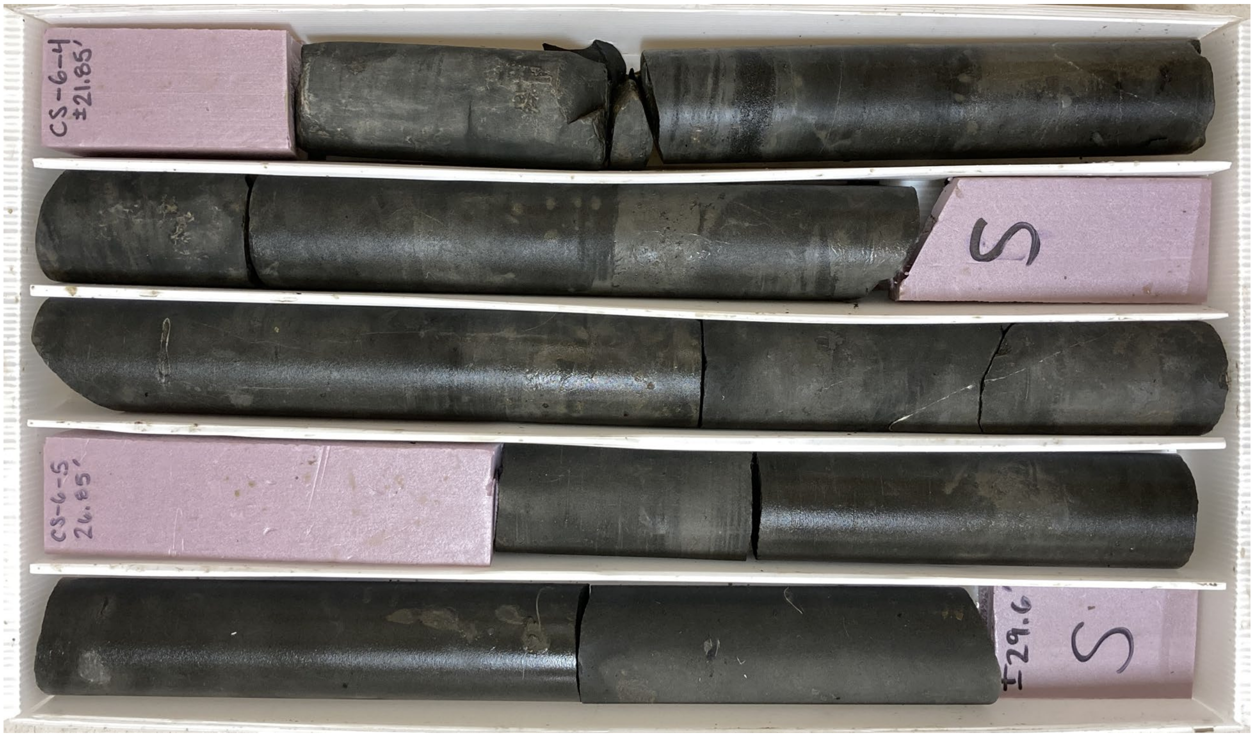


Photo 20B. BH-6 from 21.85 to 29.6 ft - Box 2 of 5

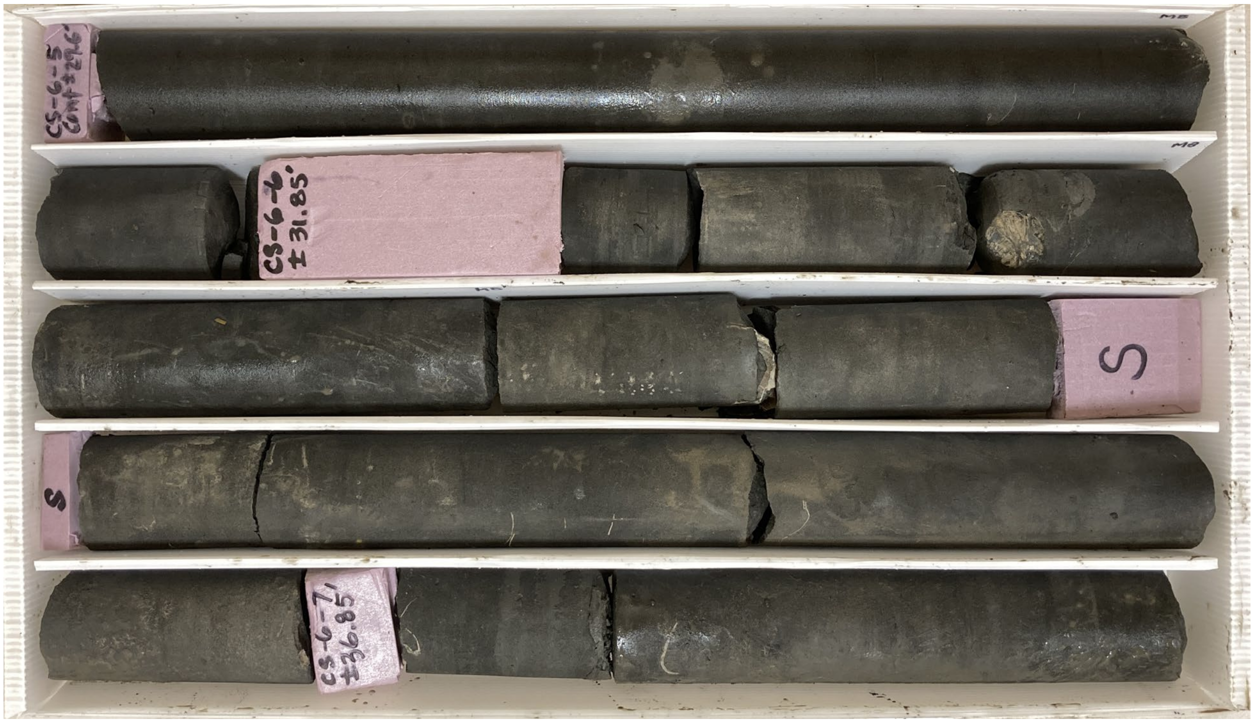


Photo 21B. BH-6 from 29.6 to 38.2 ft - Box 3 of 5

Photo 22B. BH-6 from 38.2 to 46.7 ft - Box 4 of 5

Photo 23B. BH-6 from 46.7 to 55.35 ft - Box 5 of 5

Photo 24B. BH-7 from 15.0 to 23.9 ft - Box 1 of 3

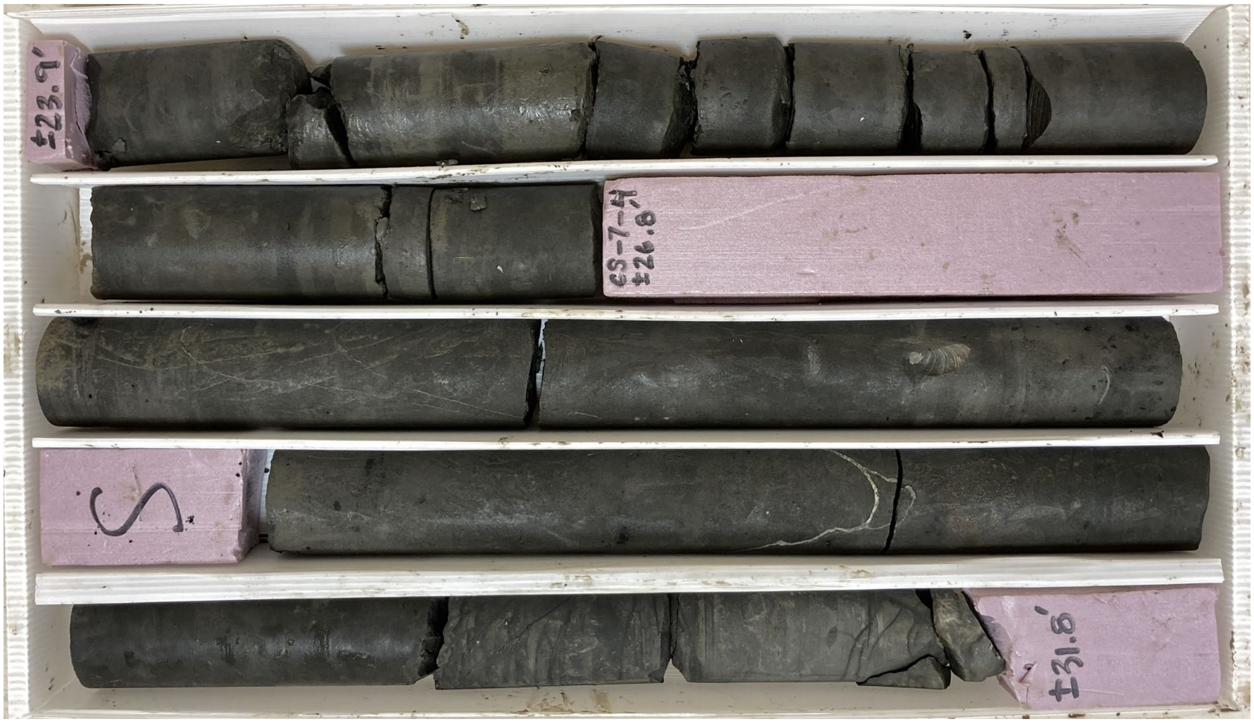


Photo 25B. BH-7 from 23.9 to 31.8 ft - Box 2 of 3

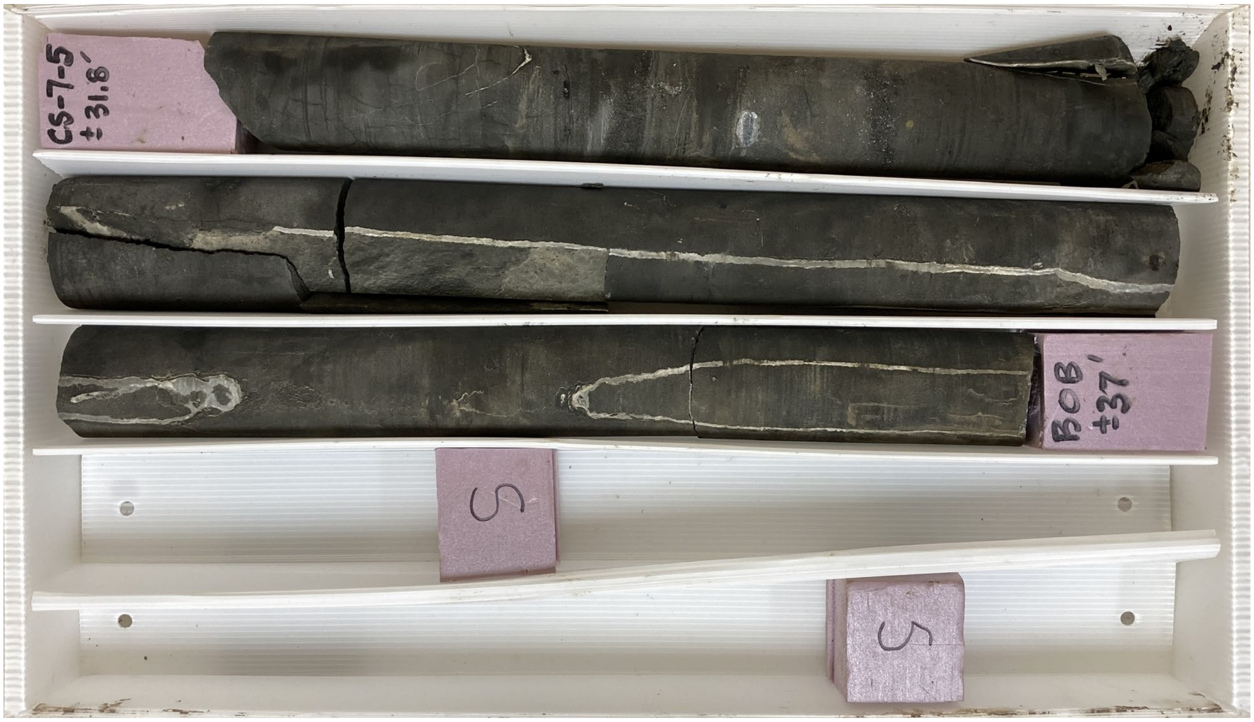


Photo 26B. BH-7 from 31.8 to 37.0 ft - Box 3 of 3

Appendix C

Laboratory Testing

Foundation Engineering, Inc.
Professional Geotechnical Services

Table 1C. Moisture Contents (ASTM D2216), Percent Fines (ASTM D1140), & Atterberg Limits (ASTM D4318)

Sample Number	Sample Depth (feet)	Moisture Content (percent)	% Fines	Atterberg Limits			USCS Classification
				LL	PL	PI	
SS-1-1	2.5 - 4.0	31.5					
SS-1-2	5.0 - 6.5	30.9	24.3				
SS-1-3	7.5 - 9.0	33.4					
SS-1-4	10.0 - 11.5	37.9	80.5				
SS-1-5	12.5 - 14.0	29.0					
SS-1-6	15.0 - 16.5	27.1					
SS-1-7	17.5 - 19.0	34.2					
SS-1-8	20.0 - 21.3	28.7					
SS-1-9	22.5 - 24.0	26.1					
SS-2-1	2.5 - 4.0	34.0					
SS-2-2	5.0 - 6.5	40.0	49.3				
SS-2-3	7.5 - 8.9	28.4					
SS-2-4	10.0 - 11.5	26.8					
SS-2-5	12.5 - 12.7	30.2					
SS-3-1	2.5 - 4.0	36.3	46.3				
SS-3-2	5.0 - 6.5	29.5					
SS-3-3	7.5 - 7.8	28.3					
SS-3-4	10.0 - 10.3	27.2					
SS-4-1	2.5 - 4.0	29.8		38	21	17	CL
SH-4-2	5.0 - 5.5	23.4					
SS-4-3	5.5 - 7.0	32.6	58.8				
SS-4-4	7.5 - 9.0	21.4					
SS-4-5	10.0 - 11.5	28.7					

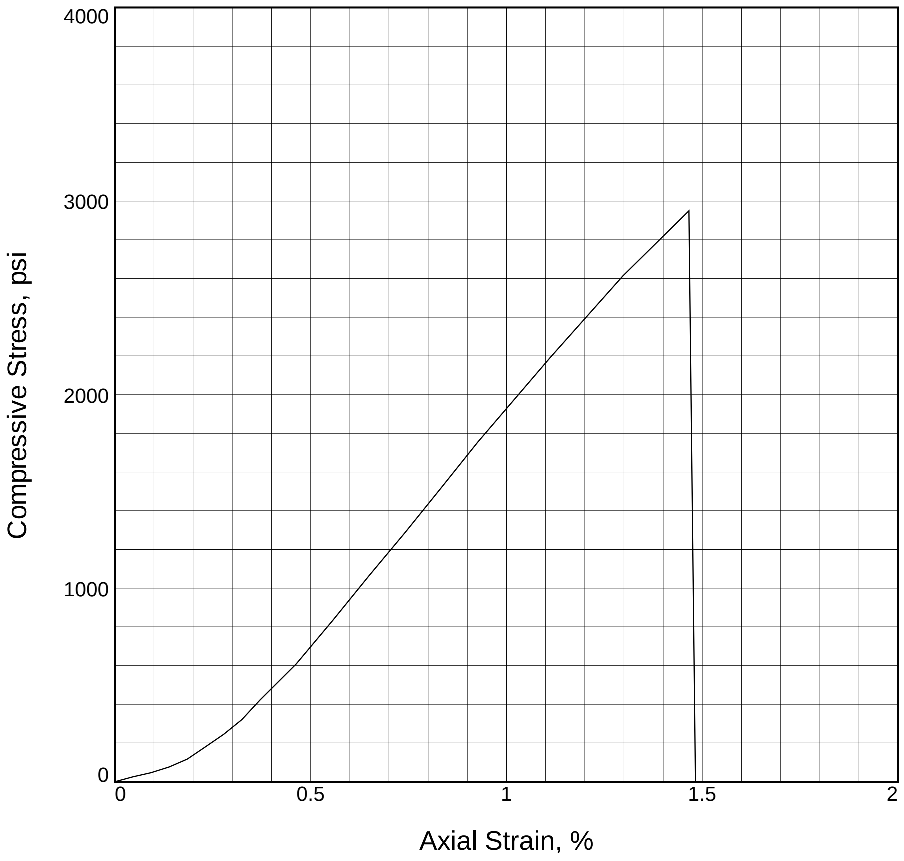
Table 1C. Moisture Contents (ASTM D2216), Percent Fines (ASTM D1140), & Atterberg Limits (ASTM D4318)

Sample Number	Sample Depth (feet)	Moisture Content (percent)	% Fines	Atterberg Limits			USCS Classification
				LL	PL	PI	
SS-5-1	2.5 - 4.0	37.5	49.3				
SS-5-2	5.0 - 6.5	29.0					
SS-5-3	7.5 - 9.0	25.7					
SS-5-4	10.0 - 11.5	30.4					
SS-5-5	12.5 - 14.0	25.3					
SS-6-1	2.5 – 4.0	35.0					
SS-7-1	2.5 – 4.0	26.0					

Table 2C. Unconfined Compressive Strength Test Results

Boring	Core Run	Sample Depth (feet)	Rock Description	Wet Density (pcf)	Unconfined Compressive Strength (psi)
BH-1	CS-1-5	41.8 – 42.3	R2 Silty SANDSTONE	141.2	2,580
BH-2	CS-2-2	18.6 – 19.1	R2 Silty SANDSTONE	142.9	1,478
BH-3	CS-3-2	11.6 - 12.1	R2 Silty SANDSTONE	140.2	1,877
BH-3	CS-3-6	31.8 – 32.3	R2 Silty SANDSTONE	142.6	2,950
BH-4	CS-4-2	11.9 – 12.4	R2 Silty SANDSTONE	142.8	2,078
BH-4	CS-4-5	28.5 – 28.9	R2 Silty SANDSTONE	140.8	2,559
BH-5	CS-5-3	24.2 – 24.7	R2 Silty SANDSTONE	142.7	2,727
BH-5	CS-5-5	32.6 – 33.1	R2 Silty SANDSTONE	135.7	1,907
BH-6	CS-6-4	23.8 – 24.2	R2 SANDSTONE	140.1	3,156
BH-6	CS-6-6	33.1 – 33.5	R2 SANDSTONE	140.6	2,724
BH-6	CS-6-9	47.5 – 47.9	R2 SANDSTONE	139.6	3,007
BH-7	CS-7-2	16.8 – 17.3	R3 Silty SANDSTONE	143.9	4,247
BH-7	CS-7-4	28.0 – 28.4	R2 SANDSTONE	139.1	2,578
BH-7	CS-7-5	32.4 – 32.8	R2 SANDSTONE	135.2	3,143

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, psi	2950			
Failure strain, %	1.5			
Strain rate, %/min.	0.25			
Water content, %	12.6			
Wet density, pcf	142.6			
Dry density, pcf				
	0.3066			
Specimen diameter, in.	2.39			
Specimen height, in.	5.40			
Height/diameter ratio	2.26			

Description: Silty SANDSTONE

Type: Rock Core

Project No.: 2206001-611

Date Sampled:

Remarks:

Figure 1C

Client: Foundation Engineering, Inc.; Project No. 2201012

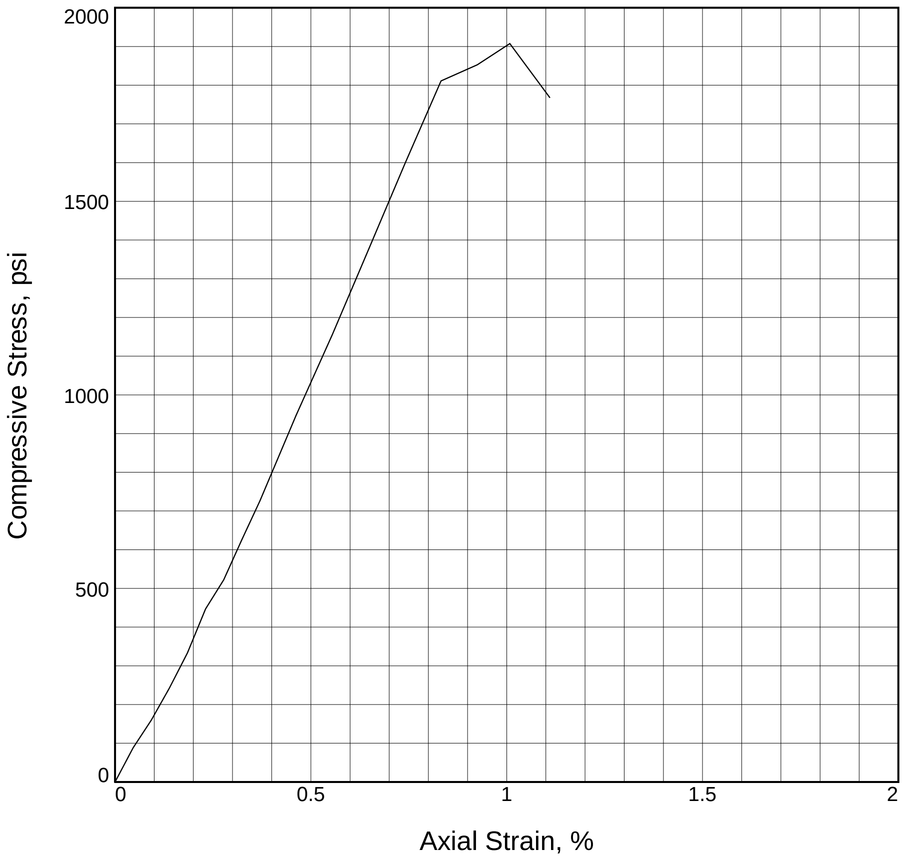
Project: College Hill 607 Storage Tanks

Source of Sample: 7759 **Depth:** 31.8-32.3'

Sample Number: CS-3-6

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

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, psi	1907			
Failure strain, %	1.0			
Strain rate, %/min.	0.25			
Water content, %	16.8			
Wet density, pcf	135.7			
Dry density, pcf	116.2			
Specimen diameter, in.	2.38			
Specimen height, in.	5.41			
Height/diameter ratio	2.27			

Description: Silty SANDSTONE

Type: Rock Core

Project No.: 2206001-611

Date Sampled:

Remarks:

Figure 2C

Client: Foundation Engineering, Inc.; Project No. 2201012

Project: College Hill 607 Storage Tanks

Source of Sample: 7759 **Depth:** 32.6-33.1'

Sample Number: CS-5-5

UNCONFINED COMPRESSION TEST

FEI Testing & Inspection, Inc.

Corvallis, OR

Appendix D

Seismic Hazard Study

Foundation Engineering, Inc.
Professional Geotechnical Services

SEISMIC HAZARD STUDY

COLLEGE HILL 607 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 2022 Oregon Structural Specialty Code (OSSC), Section 1803 for site-specific seismic hazard reports for major structures (ORS 455.447) or Risk Category III or IV buildings, which include essential and hazardous facilities and major and special-occupancy structures (OSSC, 2022).

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. Geologic time and age relations of soil and rock units referred to in this document are based on the 1983 Geologic Time Scale unless otherwise noted (Palmer, 1983).

LITERATURE REVIEW

Available geologic and seismic publications and maps were reviewed to characterize the local and regional geology and evaluate relative seismic hazards at the site. Information from other geotechnical and seismic hazard investigations previously conducted by Foundation Engineering in the surrounding area were also reviewed. A nearby boring and shear wave velocity log, which is part of a seismic hazard mapping project for the Eugene-Springfield area completed by the Oregon Department of Geology and Mineral Industries (DOGAMI), was also reviewed.

Regional Geology

Most of Oregon is geologically young, especially areas west of the Cascade Mountain Range, which are less than 50 million years old. 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).

Much of the State is formed as a result of CSZ movement. Volcanism, earthquakes, folding, faulting, creation of the volcanic arc, accumulation of marine sediments along the ocean floor including exotic terranes west of the subduction zone, uplift of oceanic and other sediments on the western margin of the North American plate, accretion, and erosion of these rocks have all helped form Western Oregon.

The project site is located within the Willamette Valley, near its southern extent. The Willamette Valley is a broad, north-south-trending basin separating the Coast Range to the west from the Cascade Range to the east. 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). These sediments typically overlie, but are also interbedded with, basalt and volcanics of the Siletz River Volcanics and 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; 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 during a period of time 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 (i.e., Willamette Silt) (Orr and Orr, 1999).

Local Geology

The existing and proposed tanks are at the top of College Hill in southwest Eugene. Local geologic mapping indicates the site is underlain by the Eugene Formation (Yeats et al., 1996; Madin and Murray, 2006; McClaughry et al., 2010). The Eugene Formation consists of tuffaceous sandstone and siltstone deposited in a shallow marine environment (Yeats et al., 1996; Orr and Orr, 1999; O'Connor et al., 2001; Madin and Murray, 2006).

A boring (ES5) and accompanying shear wave velocity log were completed at Roosevelt Middle School (± ¾ miles east of the College Hill Storage Tank) in 1996 by DOGAMI. This boring was part of a shear wave velocity study of the Eugene-Springfield area (Wang et al., 1998). Subsurface conditions at ES5 included clay and sand with trace rounded gravel underlain by sandstone (Eugene Formation). The recorded shear wave velocities at ES5 were 325 to 748 feet/second (ft/s) in the soil extending to ±16 feet and 1,703 to 5,758 ft/s in the underlying sandstone extending to ±32.5 feet (Wang et al., 1998). Although the ES5 site is some distance from the College Hill site, both sites encountered Eugene Formation sandstone. The shear wave velocities at ES5 within the sandstone should be fairly comparable to the sandstone at College Hill tank.

The subsurface conditions encountered in our exploratory borings are consistent with the mapped local geology. Silty sandstone, interpreted to be the Eugene Formation based on the local geologic mapping, was encountered in each of the borings. 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 reviewed information pertaining to the CSZ and nearby crustal 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). A discussion of these earthquake sources is provided below.

Cascadia Subduction Zone (CSZ). Eugene is located ±110 miles inland from the surface expression of the CSZ. It is estimated that 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).

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). Intraplate (Intraslab or Wadati-Benioff Zone) earthquakes occur at depths of ±21 to 43 miles ((Petersen et al., 2014). The fault rupture may occur along a segment or the entire length of the CSZ (Weaver and Shedlock, 1996). The estimated maximum magnitude of a CSZ interface earthquake is up to a moment magnitude (M_w) 9.3 (Petersen et al., 2014). The maximum estimated magnitude of an intraplate earthquake is about M_w 7.5 (Petersen et al., 2014).

Crustal Faults. Crustal faults are fractures within the North American plate. Earthquakes on crustal faults 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).

Numerous crustal faults are shown 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, 2017). The Class C Harrisburg anticline is ±17 miles north-northwest of the site. The USGS does not consider any features in Oregon as Class D (USGS, 2017).

A review of nearby faults was completed to evaluate the seismic setting and the potential seismic sources. A few concealed and inferred crustal faults are located 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, 2017).

Four potentially active Quaternary Class A and B crustal fault zones have been mapped by the USGS within ±50 miles of the site (Palmer, 1983; Geomatrix Consultants, 1995; Personius et al., 2003; USGS, 2017). 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 ± 50-mile Radius of the College Hill Tank (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	±26 SE	<1.6 million	<0.20
Owl Creek (A)	870	±9	±31 N-NW	<750,000	<0.20
Corvallis (B)	869	±25	±37 NW	<1.6 million	<0.20
Unnamed faults near Sutherlin (B)	862	±17	±40 SW	<750,000	<0.20

(1) Fault data based on Personius et al., 2003 and USGS, 2006a, 2014a, and 2020.

(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

CSZ Interface Earthquakes. No significant interface (subduction zone or megathrust) 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).

There is much debate and uncertainty regarding the size, rupture type, and frequency of CSZ earthquakes. Numerous detailed studies of coastal subsidence, tsunami, and turbidite deposits estimate a wide range of CSZ earthquake recurrence intervals. Turbidite deposits in the Cascadia Basin have been investigated to help develop a paleoseismic record for the CSZ and estimate recurrence intervals for interface earthquakes (Adams, 1990; Goldfinger et al., 2012).

A 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 ±324 years ago (January 26, 1700) (Nelson et al., 1995; Satake et al., 1996).

Additional research suggests multiple repeated M8 earthquakes might have occurred along the southern portion of the CSZ (including the southern Oregon coast) between the larger, M9 earthquakes, which are thought to rupture the entire length of the CSZ (Frankel et al., 2015).

CSZ Intraplate Earthquakes. There have been no documented intraplate earthquakes in Oregon. However, the Puget Sound region of Washington State has experienced three intraplate events in the last ±75 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. 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 ±191 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 44.033351, Longitude -123.097787.

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

Seismic events in Oregon were inconsistently documented until 1928 (Wong and Bott, 1995). Earthquake epicenters located in Oregon from 1928 to 1962 were limited due to the number of and the distance between seismographs, the number of recording stations, and the uncertainty in travel times (PNSN, 1998). 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 ≤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 the 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 1803.6.1 of the 2022 OSSC 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, 2022). The tanks are an essential facility and are considered within a Risk Category IV per the 2022 OSSC.

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, 2018, 2021a, b). The above-mentioned maps refer to some, but do not cover all of the seismic hazards. The reviewed information 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 mostly within Zone B (intermediate to high hazard) for the overall, relative earthquake hazard with a small outer ring of Zone C (low to intermediate hazard) (Black et al., 2000).

Landslides and Earthquake-Induced Landslides. The existing tank is located near the top of College Hill. LiDAR imagery shows gentle slopes for most of the site (DOGAMI, 2021a). There are no historic or mapped landslides at the site (Burns et al., 2008; Calhoun et al., 2018; DOGAMI, 2021b). The regional landslide hazard map indicates no deep landslide susceptibility (>15 feet) at the site, and a shallow (<15 feet) landslide susceptibility is considered moderate to high along the north and south ends of the site (DOGAMI, 2018, 2021b).

The relative earthquake hazard maps for the Eugene-Springfield metropolitan area indicate a moderate to high landslide hazard at the site (Black et al., 2000; Burns et al., 2008; Calhoun et al., 2018). However, this assessment appears to be based predominantly on slope and not on site-specific subsurface information. The site is underlain by stiff/dense residual soil and shallow bedrock. Based on the site conditions and the absence of mapped or historic landslides and instability features, we have concluded the risk of landslides or earthquake-induced landslides is low. The existing and new tanks will be supported on bedrock. Therefore, 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.

Loose silty sand (fill) was encountered to a depth of ±8 feet in BH-1, but this soil is not considered liquefiable due to the absence of shallow groundwater. The underlying residual soil is typically medium dense to very dense or very stiff to hard and is not expected to be liquefiable due to its density and strength. The new tanks will be supported on bedrock. Therefore, the risk of liquefaction impacting the tanks is negligible. The HazVu site indicates a low to moderate 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 negligible 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. The more recently developed ASCE online tsunami tool (ASCE, 2021) indicates a ground subsidence at the site of 0.45 feet for the ASCE 7-16 criteria and 0.36 feet for the ASCE 7-22 criteria. Ground subsidence cannot be mitigated. Therefore, it should be assumed the site and surrounding area could drop by up to 0.5 feet during a large magnitude CSZ earthquake. Subsidence has widespread regional effects and is not considered a site-specific hazard.

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; McClaughry et al., 2010; USGS, 2017, 2020). The closest potentially active (Class A) crustal fault is the Owl Creek fault, which is ±31 miles from 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 groundwater 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 relatively limited fill and residual soil followed by shallow extremely weak to medium strong (R0 to R3) bedrock, it is our opinion the amplification hazard is low and is consistent with an OSSC/IBC Site Class B soil profile (rock, 2,500 to 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 2022 OSSC, 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 2022 Section 1613.
Intraplate:	A deep CSZ intraplate earthquake with a M_W greater than 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 2022 OSSC, 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. Crustal earthquakes were included in the studies but were not considered to be a principal seismic hazard at this site. Seismic sources representing at least 4.5% of the overall hazard are summarized in Table 4D.

Table 4D. Principal Seismic Sources based on USGS (2014b) Seismic Hazard Maps

Source	Mean Moment Magnitude, M_w	Mean Source-to-Site Distance, R (miles)	Percent Contribution
CSZ Megathrust (Interface)	9.10	38	±44.2
CSZ Megathrust (Interface)	8.92	68	±18.3
CSZ Megathrust (Interface)	8.82	76	±4.7
CSZ Coastal Oregon (Intraslab)	6.94	39	±4.5

The earthquake magnitudes and source-to-site distances used to generate the 2014 USGS maps satisfy the requirements of 2022 OSSC. Seismic design parameters are discussed in the Site Response section of the main report and the design response spectra are shown in Figures 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. The most significant geologic or seismic hazard is related to the proximity of the site to the Cascadia Subduction Zone (CSZ) and the earthquake effects impacting the site. This site-specific hazard investigation for the College Hill 607 Storage Tanks, Eugene, Oregon, was prepared by Brooke Running, R.G., C.E.G.

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Expires: 6/1/24

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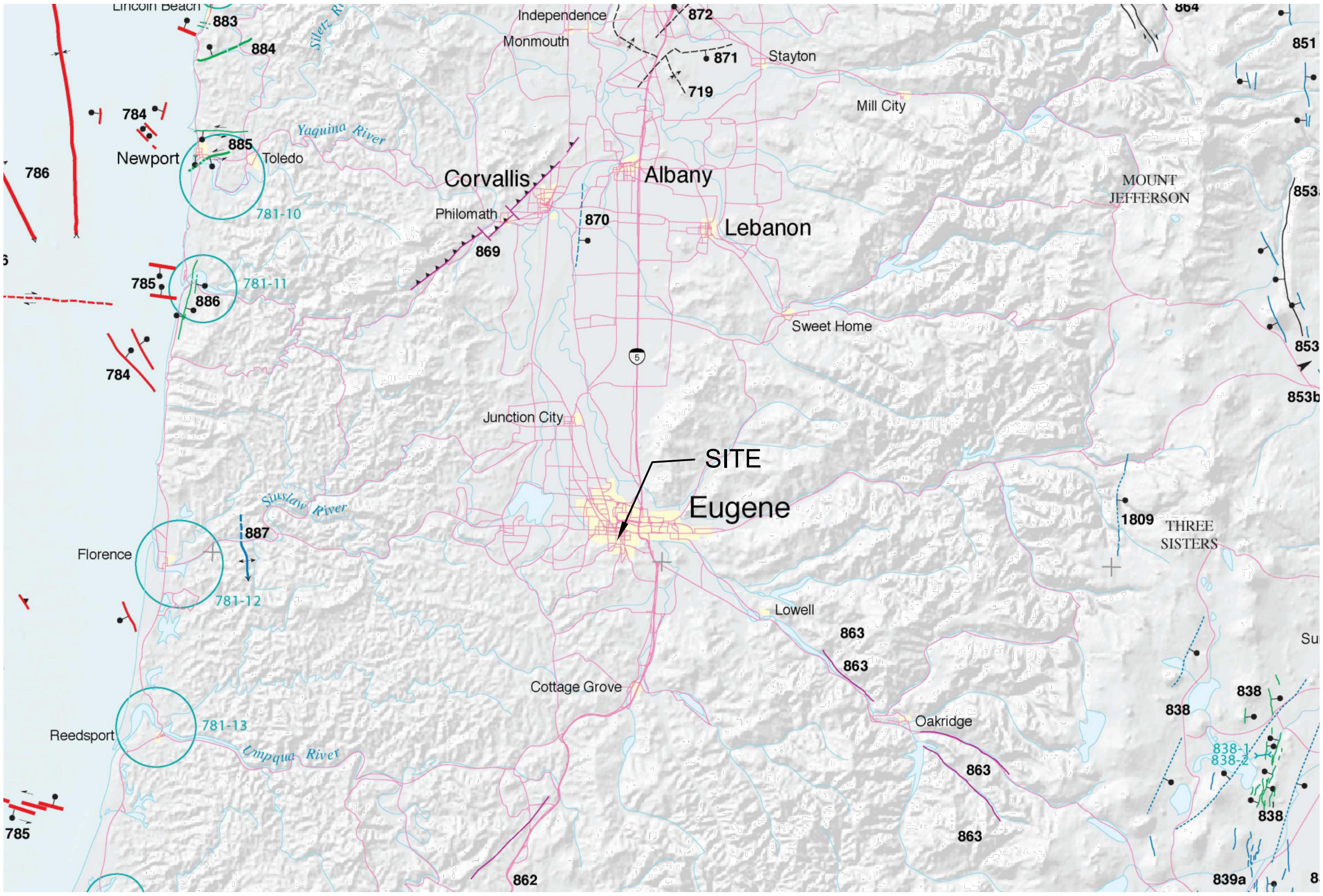

NOTES:

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: #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

QUARTERNARY CRUSTAL FAULT MAP

COLLEGE HILL 607 STORAGE TANKS
EUGENE, OREGON

PROJECT NO.	DATE:	DRAWN BY:
2201012	Aug. 11, 2023	BKR

FIGURE NO.
1D