Geotechnical Engineering Report

Salem Cannery 6-Story Mixed-use Development
Salem, Oregon

for
Future of Neighborhood Development

March 24, 2023

GEOENGINEERS

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File No. 26595-001-00

March 24, 2023

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1.0 INTRODUCTION

GeoEngineers, Inc. (GeoEngineers) is pleased to submit this geotechnical engineering report for proposed Future of Neighborhood Development Salem Cannery 6-Story Mixed-use Development (Salem Cannery) Project. The Salem Cannery is located on four blocks along Front Street NE between Belmont Street NE to Shipping Street NE in Salem, Oregon. The location of the site is shown on the Figure 1, Vicinity Map.

A preliminary site development drawing for the project by LRS Architects, was provided to the project team. The plan is titled "Salem Cannery Prelim Design" dated January 1, 2023. Based on discussions with the project team and the preliminary site plan, the project will consist of constructing one 3-story, concrete parking structure (block 5), four new 6-story mixed-use buildings (Blocks 1 through 4), associated underground utilities and paved parking areas and drive aisles, and one below grade parking level spanning beneath blocks 3, 4 and 5 in the first phase of development. Future phases of development will extend northward to Blocks 1 and 2.

In addition, the project will include off-site improvements to Front Street NE, site access drives that are extensions of east-west trending city streets at each block, and adaptive reuse of some existing structures along the west side of proposed blocks 3 and 4 and east of Front Street SE southeast of block 4.

Existing site conditions are presented on the Figure 2, Site Plan. Our recommendations for structural development for the site are based on estimated column and wall loads on the order of 575 and 10 kips per lineal foot (klf), respectively, and slab loads of 250 pounds or less as provided by F4OELICH Engineers (Structural Engineer). If design loads exceed these values our recommendations may need to be revised.

2.0 SCOPE OF SERVICES

Our specific scope of services is detailed in our January 31, 2023, proposal to you. Our original scope of services was authorized on February 1, 2023. In general, our scope of services included: reviewing selected geotechnical information about the site; performing a geologic reconnaissance; exploring subsurface soil and groundwater conditions; collecting representative soil samples; completing infiltration testing at the site; completing relevant laboratory testing and geotechnical analyses; conducting a site-specific seismic hazard evaluation for the proposed project; and providing this geotechnical report with our conclusions, findings and design recommendations.

3.0 SITE DESCRIPTION

3.1. Surface Conditions

The project site comprises approximately 11 acres located between the east bank of the Willamette River and Front Street NE between Belmont Street NE just north of the bridge over Mill Creek on Front Street NE, and Shipping Street NE in Salem, Oregon. The site is occupied by buildings and equipment formerly used for industrial food processing plants dating back to the early 1900s.

The property is bounded by a slope that grades down to the Willamette River to the west, Front Street NE to the east, commercial properties to the north and a gabion basket retaining wall overlooking a creek to the south. The project site is currently developed with the existing industrial food processing facility (Truitt Cannery), administrative support buildings and paved parking and associated underground utilities located in the southern two-thirds of the site as shown on Figure 2. Site grades are generally flat or sloping gently away from the existing structures as part of the previous site grading, except along the west margin of the site where it slopes down more steeply toward the Willamette River (west) and the southern portion of the site where a nearly vertical face retained by a gabion basket wall is located. At the time of this report, the team did not have design or construction information with respect to the gabion wall. Asphalt concrete (AC) and portland cement concrete (PCC) hardscape pavements or gravel parking areas and grassy equipment laydown and general storage areas border the buildings.

3.2. Site Geology

The geology of the site is mapped by the *Geology of the Rickreall, Salem West, Monmouth and Sidney Quadrangles, Marion, Polk, and Linn Counties, Oregon* (Bela 1981) as stretching across the contact of two geologic units.

The southern portion of the site – from roughly the alignment of Hood Street NE to the southwestern edge of the site along Mill Creek - is mapped as underlain by Pleistocene-age Linn Gravels. These materials typically consist of "…stratified fine to coarse fluvial gravels deposited as an alluvial fan…by an early stage of the Santiam River." Our explorations suggest that this coarse alluvium underlies the entire site at depth, ranging from less than 5 feet below ground surface (bgs) near the center of the site to as much as 20 to 25 feet bgs near the northeast and southwest perimeters.

Northeast of Hood Street to the northeast edge of the site along Shipping Street NE the published mapping and our investigation suggests that the Linn Gravel is mantled by what Bela (1981) terms "Middle Terrace Deposits" and describes as "…10-30 feet of light brown silty clay and interbedded very fine sand and silt…" that the mapping equates with the Willamette Silt flood deposit alluvium typically encountered as valley fill across the lower Willamette Valley. Our subsurface investigation suggests that the actual contact between the shallow Linn Gravels and the mantle of Middle Terrace Deposits is further southwest than is mapped, probably between the alignments of Market and Gaines Streets.

Although not mapped by Bela (1981), our explorations and experience in the area indicates that the site is mantled by fill of variable thickness that generally increases to the south as a result of historical site grading.

Our review of the site geology, together with on-site observations, suggests that the site geology is generally consistent with the published mapping and our experience in the area, except for the fill as noted above.

3.3. Subsurface Conditions

Subsurface conditions at the site were explored by advancing 13 drilled borings (B-1 through B-13), 4 cone penetration soundings (CPT-1 through CPT-4), and 4 ground penetrating radar soundings completed between February 20, and February 25, 2023. Drilled borings were advanced to a final depth between 16.5 and 41.5 feet bgs and the cone penetrometer tests (CPTs) were advanced to refusal approximately between 6.5 and 19 feet bgs. The approximate locations of the explorations completed at the site are shown on Figure 2. Logs of GeoEngineers' explorations completed for this study are presented in Appendix A, Field Explorations and Laboratory Testing.

Soil samples obtained during site exploration were taken to GeoEngineers' laboratory for further evaluation. Selected samples were tested for determination of moisture content and Atterberg Limit Determinations. A description of the laboratory testing, and the test results are presented in Appendix A.

3.3.1. Soil Conditions

The site soils can be generally divided into three general categories: Man-made Fill, Middle Terrace Deposits and Linn Gravels. Some blending or alluvial soil interfaces may be present, but we consider the descriptions below to be the dominant soils present at the site.

3.3.1.1. Man-made Fill

A highly variable mix of silt, sand and gravel fill was encountered in four of the borings and one CPT located in the southwestern portions of the site. The materials ranged from 4 to 5 feet of soft to medium stiff silt encountered in B-1 and B-4, roughly 9½ feet of loose silty gravel and silty sand in B-2, approximately 13 feet of soft to medium stiff silt in B-4, and up to 23 feet of loose to very dense silty gravel and soft silt in B-13. This material was likely used to level the low-lying portions of the site that formed the former Mill Creek and Willamette River confluence and are likely to include an even wider range of materials including possibly wood and man-made debris.

3.3.1.2. Middle Terrace Deposits

The central and northeastern portions of the site are mantled by a thickening to the northeast wedge of soft to stiff silt and loose to medium dense fine sand we interpret as the mapped Middle Terrace Deposits. The thickness of these deposits ranges from roughly 6 to 7 feet in B-5 and B-6 to approximately 18 to 20 feet in B-7, B-12 and B-10.

3.3.1.3. Linn Gravels

Underlying the fill to the southwest and the Middle Terrace Deposits to the northeast we encountered medium dense to very dense silty gravel and poorly-graded gravel with sand and silt that we interpret as the mapped Linn Gravels to the maximum depth explored. Several borings encountered layers of silt or sand that we interpret as natural interbeds of alluvial materials deposited during low-energy episodes of Willamette and Santiam River deposition.

3.3.2. Groundwater Conditions

Groundwater was encountered at approximately 30 feet bgs in B-6 and B-13. Based on our experience at nearby sites the regional groundwater level is likely related to the level of the Willamette River, although shallow seasonal ("perched") groundwater may be encountered at shallower depths during the wet winter and spring months of the year.

4.0 INFILTRATION TESTING

As requested by the project team, we conducted two on-site infiltration tests to assist in the evaluation of the site for stormwater infiltration design at the exploration location noted as IT-1 and IT-2 on Figure 2. The testing was conducted at a depth of 5 feet bgs.

On site testing was conducted in general accordance with the professional encased falling head procedure outlined in development design standards of multiple Oregon jurisdictions. Our general procedure included drilling a 4-inch-diameter hole to insert a polyvinyl chloride (PVC) pipe for the encased falling head procedure at a depth of 4 feet bgs.

The encased PVC pipe was filled with clean water to approximately 1 foot above the soil at the bottom of the drilled hole. The initial fill of water did not drain into the soil within 10 minutes, so the water level was maintained, and the soil allowed to saturate for 4 hours at the test locations. The levels were checked, and the pipes were refilled to 12 inches above the soil in the bottom of the pipe at the end of each hour and for multiple days after initiating the test. The drop-in water level was measured during three, hour-long iteration periods at the test locations. Field test results are summarized in Table 1.

TABLE 1. FIELD MEASURED INFILTRATION RESULTS

Infiltration Test No.	Location	Depth (feet)	USCS Material Type	Field Measured Infiltration Rate[1] (in/hr)
IT-1	See Site Plan	5	GM (Fill)	12
IT-2	See Site Plan	5	ML (Middle Terrace)	0 – 0.1

Notes:

[1] Appropriate factors should be applied to the field-measured infiltration rate, based on the design methodology and specific system used.

USCS = Unified Soil Classification System; in/hr = inches per hour

Infiltration rates shown in Table 1 represent a field-measured infiltration rate. The rates summarized for IT-2 and IT-3 indicate effectively 0 in/hr because minimal to no infiltration (drop in water levels) was observed during the testing period. Field measurements are limited to the accuracy of equipment employed to conduct the test. Actual long-term infiltration rates of the on-site soils are likely greater than 0 in/hr if measured out over very long-time frames (much longer than the time frames prescribed in the testing standards). A field-measured rate of 0 in/hr generally indicates infiltration less than ⅛ inch per hour, which is about the limit of the field measuring equipment.

In addition, field-measured rates represent a relatively short-term infiltration rate, and factors of safety have not been applied for the type of infiltration system being considered or for variability that may be present across large areas in the on-site soil. In our opinion, and consistent with the state of the practice, correction factors should be applied to this measured rate to reflect the localized area of testing relative to the field sizes.

Appropriate correction factors should also be applied by the project civil engineer to account for long-term infiltration parameters. From a geotechnical perspective, we recommend a factor of safety (correction factor) of at least 2 be applied to the field infiltration values to account for potential soil variability with depth and location within the area tested. In addition, the stormwater system design engineer should determine and apply appropriate remaining correction factor values, or factors of safety, to account for repeated wetting and drying that occur in this area, degree of in-system filtration, frequency and type of system maintenance, vegetation, potential for siltation and bio-fouling, etc., as well as system design correction factors for overflow or redundancy, and base and facility size.

Actual depths, lateral extent and estimated infiltration rates can vary from the values presented above. Field testing/confirmation during construction is often required in large or long systems or other situations where soil conditions may vary within the area where the system is constructed. The results of this field testing might necessitate that the infiltration locations be modified to achieve any appreciable design infiltration rate. In no case, however, do we recommend infiltration within 50 feet of the adjacent slopes to the west. The infiltration flow rate of a focused stormwater system like a drywell or small infiltration box or pond typically diminishes over time as suspended solids and precipitates in the stormwater further clog the void spaces between the soil particles or cake on the infiltration surface or in the engineered media. The serviceable life of an infiltration media in a stormwater system can be extended by pre-filtering or with on-going accessible maintenance. Eventually, most systems will fail and will need to be replaced or have media regenerated or replaced.

We recommend that infiltration systems not be located within 50 feet of the adjacent slope to the west. In addition, for infiltration systems located anywhere on site an overflow that is connected to a suitable discharge point should be provided. Also, infiltration systems can cause localized, high groundwater levels and should not be located near basement walls, retaining walls or other embedded structures unless these are specifically designed to account for the resulting hydrostatic pressure. Infiltration locations should not be located on sloping ground, unless it is approved by a geotechnical engineer, and should not be infiltrated at a location that allows for flow to travel laterally toward a slope face, such as a mounded water condition or too close to a slope face that could cause instability of the slope.

4.1. Suitability of Infiltration System

Successful design and implementation of stormwater infiltration systems and whether a system is suitable for development depends on several site-specific factors. Stormwater infiltration systems are generally best suited for sites having sandy or gravelly soil with saturated hydraulic conductivities greater than 2 in/hr. Sites with silty or clayey soil, are generally not well- suited for long-term stormwater infiltration or as a sole method of stormwater infiltration. Soils that have fine-grained matrices are susceptible to volumetric change and softening during wetting and drying cycles. Fine-grained soils also have large variations in the magnitude of infiltration rates because of bedding and stratification that occurs during alluvial deposition, and often have thin layers of less permeable or impermeable soil within a larger layer. As a result of fine-grained soil conditions and relatively low field measured infiltration rates over portions of the site mantled by fine-grained fill and middle terrace deposits, and proximity to existing slopes in portions of the site mantled with fill, we recommend infiltration of stormwater not be used in as the sole method of stormwater management.

We understand that stormwater infrastructure on the site will include vegetated swales (rain gardens) that will treat site stormwater before being discharged to a suitable drainage system. Where located within 50 feet of the crest of existing slopes, vegetated swales for stormwater treatment should be lined with an impervious geomembrane to prevent infiltration of stormwater that could negatively affect slope stability.

5.0 CONCLUSIONS

Based on our explorations, testing and analyses, it is our opinion that the site is generally suitable for the proposed development from a geotechnical engineering standpoint, provided the recommendations in this report are included in design and construction. As a result of relatively high column and wall structural loads, building foundations should be supported on a system of compacted aggregate piers (CAPs) or extended-depth foundations. Fill encountered in the southwest portion of the site will also require structural loads to be supported on inclusions (CAPs or extended-depth foundations) to depths below the fill layer and to depths that transfer loads sufficiently deep so that additional vertical or lateral loads are not applied to the existing gabion wall. The existing gabion wall was observed to have been compromised over a portion of its extent (blowout of facing) and will require repair or replacement as part of site development.

A summary of the primary geotechnical considerations is provided below. The summary is presented for introductory purposes only and should be used in conjunction with the complete recommendations presented in this report.

- Due to the fines content of the upper soils at the site, they will likely become disturbed by construction traffic from earthwork occurring during periods of wet weather or when the moisture content of the soil is more than a few percentage points above optimum. Wet weather construction practices will be required, except during the dry summer months.
- On-site soils may be reused as structural fill; however, the material is considered moisture sensitive and may not be suitable for reuse except during the driest of summer months. On-site material will be practically unworkable as structural fill during the wet season or when prolonged wet weather persists. In general, the most persistent wet weather in the area occurs from early October to mid-May. The blackish upper silt material observed is generally not recommended for reuse as structural fill.
- The site is generally poorly suited for stormwater infiltration as a method of handling site stormwater. Infiltration should not be used as the sole method for handling site stormwater and should not be used adjacent to slopes. Vegetated swales (rain gardens) for stormwater treatment should be lined with an impervious geomembrane to prevent infiltration of site stormwater where it could negatively affect slope stability.
- Based on the maximum design column and wall loads provided by the project structural engineer, we recommend that proposed structures with column loads greater than 180 kips and wall loads greater than 5 klf be supported on shallow spread foundations bearing on subgrade improved by compacted aggregate piers or rigid inclusions, or be founded on extended-depth foundations such as driven piles or drilled shafts. Ground improvements using CAPs or rigid inclusions used as ground improvement should be designed to a performance criterion that is acceptable to the structural engineer and project team, typically less than 1 inch of allowable settlement.
- Based on our discussions with the project team, block 5 may be constructed at a zero offset from the south and west property corner, near an approximately 20-foot-tall (in some spots) gabion wall along Mill Creek on the southwest corner boundary of the project site. As detailed in the Geologic Hazard Assessment included in Appendix B, the existing wall has blown out (broken) in at least one area and the remaining wall is considered marginally stable as a result of foundation undermining and will require repair or demolition and replacement based on the preferred configuration for block 5.
- New foundations for the proposed block 5, whichever alternative is selected, should be designed and constructed in a manner that loads downward beyond a depth that would impose additional vertical or lateral load to the existing gabion wall if it is to remain in place. In addition, any structural elements extending behind the gabion wall should not be designed for passive lateral resistance from soil retained by the gabion system within a 2H:1V (horizontal to vertical) projection back from the base of the wall.
- If the existing gabion wall is not removed and shallow foundations are the preferred alternative for block 5, the existing wall should be repaired and foundation elements for block 5 should be located outside of a 2H:1V projection from the bottom of the gabion wall. Spread footings should be founded on subgrade improved by rammed aggregate piers.

  Alternatively, the existing wall could be repaired and block 5 could be satisfactorily founded on rigid inclusions or shallow foundations over subgrade improved by CAPs located within the recommended setback distance, provided that they can be designed to impose no additional lateral load on the existing gabion wall.

- If the existing gabion wall is demolished (excavated down and regraded) or encased and replaced by a permanent wall to facilitate the configuration of block 5 as a zero-offset structure, the new wall should be designed for at-rest earth pressures as it will be restrained top and bottom by horizontal structural elements, and should account for additional loading from newly constructed foundations.
- If ground improvement is not completed at the site, post-construction settlement from the underlying compressible soils under the design loads are anticipated to exceed 1-inch total. We estimate settlements on the order of 2.5 to 3.5 inches, with about ½ of that magnitude occurring as differential settlement over a distance of approximately 50 feet.
- Relatively lightly loaded floor slabs (250 pounds per square foot [psf] loads or less) can be supported on aggregate base placed on native medium stiff/medium dense or stiffer/denser soils or on structural fill placed over native soils. Structural slabs should be supported on a minimum 6-inch-thick compacted crushed rock base.
- Standard pavement sections prepared as described in this report will suitably support the estimated traffic loads, provided the site subgrade is prepared as recommended. Maintenance and repair will likely be required following the design level earthquake.
- We observed groundwater at a depth of approximately 30 feet bgs.

6.0 EARTHWORK RECOMMENDATIONS

6.1. Site Preparation

In general, site preparation and earthwork for site development will include demolition and removal of existing structures and hardscapes, removal or relocation of existing site utilities where present beneath proposed building footprints, excavation for removal of existing foundation elements, tree and tree root removal, grading the site and excavating for utilities and foundations.

6.1.1. Demolition

All existing structural elements should be excavated and removed from proposed structural areas including above-ground structures, below-grade basement structures, concrete flatwork, rail lines or conduit, stripping of site vegetation in the north blocks, and removal or known and potentially buried and previously abandoned subsurface support elements. If present, existing utilities that will be abandoned on site should be identified prior to project construction. Abandoned utility lines larger than 4 inches in diameter that are located beneath proposed structural areas should be completely removed or filled with grout if abandoned and left in place in order to reduce potential settlement or caving in the future.

In general, demolished material should be transported off site for disposal. Excavations left from demolition of existing development should be backfilled with compacted structural fill as recommended in this report. The bottom of the excavations should be excavated to expose firm subgrade. The sides of the excavations should be cut into firm material and sloped a minimum of 1H:1V. Excavations should not undermine adjacent foundations, walkways, streets or other hardscapes unless special shoring or underpinning is provided. Excavations should not be conducted within an outward and downward projection of a 1H:1V line starting at least 2 feet outside the edge of an adjacent structural feature.

6.1.2. Stripping

Areas to receive fill, structures or pavements should be cleared of vegetation and stripped of topsoil. Based on our observations at the site, we estimate that the depth of stripping will generally be on the order of 2 to 6 inches where vegetation is present with increased depths in areas of thicker vegetation.

Greater stripping depths may be required to remove localized zones of loose or organic soil. The actual stripping depth should be based on field observations at the time of construction. Stripped material should be transported off site for disposal unless otherwise allowed by project specifications for other uses such as landscaping. Clearing and grubbing recommendations provided below should be used in areas where moderate to heavy vegetation are present, or where surface disturbance from prior use has occurred.

6.1.3. Clearing and Grubbing

Where thicker vegetation (brush and trees) is present, more extensive site clearing will be required to remove site vegetation, including thick grass, shrubs and trees that are designated for removal. Following clearing, grubbing and excavations up to several feet will be required to remove the root zones of thick shrubs and trees. Deeper excavations, up to 5 or 6 feet may be required to remove the root zones of large trees if encountered. In general, roots larger than ½ inch in diameter should be removed. Excavations to remove root zones should be done with a smooth-bucket to minimize subgrade disturbance. Portions of the site are heavily vegetated and previously buried roots may be present, even in the current grassy areas of the site. Grubbed materials should be hauled off site and properly disposed of unless otherwise allowed by the project specifications for other uses such as landscaping, stockpiling or on-site burning.

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

6.2. Subgrade Preparation and Evaluation

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

As discussed in Section 6.3 Subgrade Protection and Wet Weather Considerations of this report, because of the fines content native clayey soil can be sensitive to small changes in moisture content and will be difficult, or not possible, to compact adequately during wet weather. While tilling and compacting the subgrade is the economical method for subgrade improvement, it will likely only be possible during extended dry periods and following moisture-conditioning of the soil.

During wet weather, or when the exposed subgrade is wet or unsuitable for proof-rolling, the prepared subgrade should be evaluated by observing excavation activity and probing with a steel foundation probe.

Observations, probing and compaction testing should be performed by a member of our staff. Wet soil that has been disturbed due to site preparation activities or soft or loose zones identified during probing should be removed and replaced with compacted structural fill.

6.3. Subgrade Protection and Wet Weather Considerations

The upper clayey soils at the site are extremely susceptible to moisture. Wet weather construction practices will be necessary if work is performed during periods of wet weather. If site grading will occur during wet weather conditions, it will be necessary to use track-mounted equipment, load material into trucks supported on gravel work pads and employ other methods to reduce ground disturbance. The contractor should be responsible to protect the subgrade during construction reflective of their proposed means and methods and time of year.

Earthwork planning should include considerations for minimizing subgrade disturbance. The following recommendations can be implemented if wet weather construction is considered:

- The ground surface in and around the work area should be sloped so that surface water is directed to a sump or discharge location. The ground surface should be graded such that areas of ponded water do not develop. Measures should be taken by the contractor to prevent surface water from collecting in excavations and trenches. Measures should be implemented to remove surface water from the work area.
- Earthwork activities should not take place during periods of heavy precipitation.
- Slopes with exposed soils should be covered with plastic sheeting or similar means.
- The site soils should not be left uncompacted and exposed to moisture. Sealing the surficial soils by rolling with a smooth-drum roller prior to periods of precipitation will reduce the extent to which these soils become wet or unstable.
- Construction activities should be scheduled so that the length of time that soils are left exposed to moisture is reduced to the extent practicable.
- Construction traffic should be restricted to specific areas of the site, preferably areas that are surfaced with working pad materials not susceptible to wet weather disturbance such as haul roads and rocked staging areas.
- When on-site fine-grained soils are wet of optimum moisture, they are easily disturbed and will not provide adequate support for construction traffic or the proposed development. The use of granular haul roads and staging areas will be necessary for support of construction traffic. Generally, a 12- to 16-inch-thick mat of imported granular base rock aggregate material is sufficient for light staging areas for the building pad and light staging activities but is not expected to be adequate to support repeated heavy equipment or truck traffic. The granular mat for haul roads and areas with repeated heavy construction traffic should be increased to between 18 and 24 inches. The actual thickness of haul roads and staging areas should be based on the contractor's approach to site development and the amount and type of construction traffic.
- During periods of wet weather, concrete should be placed as soon as practical after preparation of the footing excavations. Foundation bearing surfaces should not be exposed to standing water. If water collects in the excavation, it should be removed before placing structural fill or reinforcing steel.

Subgrade protection for foundations consisting of a lean concrete mat may be necessary if footing excavations are exposed to extended wet weather conditions.

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

6.4. Soil Amendment with Cement

As an alternative to using Imported Select Structural Fill material for wet weather structural fill, an experienced contractor may be able to amend the on-site soil with portland cement concrete (PCC) to obtain suitable support properties. It is often less costly to amend on-site soils than to remove and replace soft soils with imported granular materials. Single pass tilling depths for cement amendment equipment is typically 18 inches or less. However, multiple tilling passes may be required to adequately blend in the cement with the soils and to sufficiently process the soils. It may also be necessary to place the recommended cement quantities in multiple passes between tilling passes, which requires intermediate compaction.

The contractor should be responsible for selecting the means and methods to construct the amended soil without disturbing exposed subgrades. We recommend low ground-pressure (such as balloon-tired) cement spreading equipment be required. We have observed other methods used for spreading that have resulted in significant site disturbance and high remedial costs. For example, we have observed amendment efforts using a spreader truck equipped with road tires pulled by track-mounted equipment that resulted in significant disturbance to the work area and required re-working large areas of cement-amended product at additional expense.

Areas of standing water, or areas where traffic patterns are concentrated and disturbing the subgrade, will also create a need for higher amounts of cement to be applied and additional tilling for better mixing and cement hydration prior to final compaction.

Successful use of soil amendment depends on the use of correct mixing techniques, the soil moisture content at the time of amendment and amendment quantities. Specific recommendations, based on exposed site conditions for soil amending, can be provided if necessary. However, for preliminary planning purposes, it may be assumed that a minimum of 5 percent cement (by dry weight, assuming a unit weight of 100 pounds per cubic foot [pcf]) will be sufficient for improving on-site soils. Treatment depths of 12 to 16 inches are typical (assuming a 7-day unconfined compressive strength of at least 80 pounds per square inch [psi]), although they may be adjusted in the field depending on site conditions. Soil amending should be conducted in accordance with the specifications provided in Oregon Structural Specialty Code (OSSC) 00344 (Treated Subgrade).

We recommend a target strength for cement-amended soils of 80 psi. The amount of cement used to achieve this target generally varies with moisture content and soil type. It is difficult to predict field performance of soil-to-cement amendment due to variability in soil response and we recommend laboratory testing to confirm expectations. However, for preliminary design purposes, 4 to 5 percent cement by weight of dry soil can generally be used when the soil moisture content does not exceed approximately 20 percent. If the soil moisture content is in the range of 20 to 35 percent, 5 to 7 percent by weight of dry soil is recommended. The amount of cement added to the soil should be adjusted based on field observations and performance.

PCC-amended soil is hard and has low permeability; therefore, this soil does not drain well nor is it suitable for planting. Future landscape areas should not be cement amended, if practical, or accommodations should be planned for drainage and planting. Cement amendment should not be used if runoff during construction cannot be directed away from adjacent low-lying wet areas and active waterways and drainage paths.

When used for constructing pavement, staging or haul road subgrades, the amended surface should be protected from abrasion by placing a minimum 4-inch thickness of base rock material (Aggregate Base/Aggregate Subbase). To prevent strength loss during curing, cement-amended soil should be allowed to cure for a minimum of 4 days prior to placing the base rock. The base rock typically becomes contaminated with soil during construction. Contaminated base rock should be removed and replaced with clean base rock in pavement areas to meet the required thickness(es) in Section 8.0 Pavement Recommendations of this report.

It is not possible to amend soil during heavy or continuous rainfall. Work should be completed during suitable weather conditions.

6.5. Shoring and Temporary Slopes

All excavations should be made in accordance with applicable Occupational Safety and Health Administration (OSHA) and state regulations. Site soils within expected excavation depths typically range from very soft to medium stiff clay or silt, or medium dense gravel and sand fill. In our opinion, fine-grained native soils are generally OSHA Type B (OSHA 2018) and sandy native soils are Type C, provided there is no seepage and excavations occur during periods of dry weather. Excavations deeper than 4 feet should be shored or laid back at an inclination of 1H:1V for Type B soils and 1½H:1V for Type C soils. Flatter slopes may be necessary if workers are required to enter. Excavations made to construct footings or other structural elements should be laid back or shored at the surface as necessary to prevent soil from falling into excavations.

Temporary cut slopes should not exceed a gradient appropriate for the soil type being excavated. However, because of the variables involved, actual slope angles required for stability in temporary cut areas can only be estimated before construction. The stability and safety of cut slopes depend on a number of factors, including:

- The type and density of the soil.
- The presence and amount of any seepage.
- Depth of cut.
- Proximity and magnitude of the cut to any surcharge loads, such as stockpiled material, traffic loads or structures.
- Duration of the open excavation.
- Care and methods used by the contractor.

We recommend that stability of the temporary slopes used for construction be the responsibility of the contractor, since the contractor is in control of the construction operation and is continuously at the site to observe the nature and condition of the subsurface. If groundwater seepage is encountered within the excavation slopes, the cut slope inclination may have to be flatter than 1.5H:1V. However, appropriate inclinations will ultimately depend on the actual soil and groundwater seepage conditions exposed in the cuts at the time of construction. It is the responsibility of the contractor to ensure that the excavation is properly sloped or braced for worker protection, in accordance with applicable guidelines. To assist with this effort, we make the following recommendations regarding temporary excavation slopes:

- Protect the slope from erosion with plastic sheeting for the duration of the excavation to minimize surface erosion and raveling.
- Limit the maximum duration of the open excavation to the shortest time period possible.
- Place no surcharge loads (equipment, materials, etc.) within 10 feet of the top of the slope.

More restrictive requirements may apply depending on specific site conditions, which should be continuously assessed by the contractor.

If temporary sloping is not feasible based on site spatial constraints, excavations could be supported by internally braced shoring systems, such as a trench box or other temporary shoring. There are a variety of options available. We recommend that the contractor be responsible for selecting the type of shoring system to apply.

Additionally, in our opinion, the contractor will be in the best position to observe subsurface conditions continuously throughout the construction process and to respond to the soil and groundwater conditions. Construction site safety is generally the sole responsibility of the contractor, who also is solely responsible for the means, methods and sequencing of the construction operations and choices regarding excavations and shoring. Under no circumstances should the information provided by GeoEngineers be interpreted to mean that GeoEngineers is assuming responsibility for construction site safety or the contractor's activities; such responsibility is not being implied and should not be inferred.

6.6. Permanent Slopes

Permanent cut or fill slopes should not exceed a gradient of 2H:1V. Where access for landscape maintenance is desired, we recommend a maximum gradient of 3H:1V. Fill slopes should be overbuilt by at least 12 inches and trimmed back to the required slope to maintain a firm face.

To reduce erosion, newly constructed slopes should be planted or hydroseeded shortly after completion of grading. Until the vegetation is established, some sloughing and raveling of the slopes should be expected. This may necessitate localized repairs and reseeding. Temporary covering, such as clear heavy plastic sheeting, jute fabric or erosion control blankets (such as American Excelsior Curlex 1 or North American Green SC150) could be used to protect the slopes during periods of rainfall.

6.7. Dewatering

As discussed in Section 3.3.2 Groundwater Conditions of this report, groundwater was encountered in our explorations, but is expected to typically be below the anticipated excavation depths. Excavations that extend into saturated/wet soils should be dewatered. Sump pumps are expected to adequately address groundwater encountered in shallow excavations. In addition to groundwater seepage and upward confining flow, surface water inflow to the excavations during the wet season can be problematic. Provisions for surface water control during earthwork and excavations should be included in the project plans and should be installed prior to commencing earthwork.

Deep wells or well points will likely be necessary where excavations extend below the groundwater table. The contractor should be required to submit a dewatering plan prepared by a registered professional engineer or hydrogeologist for review by the project team including GeoEngineers. Additionally, it should be noted that dewatering near the existing structure using wells or well points could result in settlement in addition to the long-term static settlement estimates presented in Section 7.3 Foundation Support Alternatives.

6.8. Structural Fill and Backfill

6.8.1. General

Materials used to support building foundations, floor slabs, hardscape, pavements and any other areas intended to support structures or within the influence zone of structures are classified as structural fill for the purposes of this report.

All structural fill soils should be free of debris, clay balls, roots, organic matter, frozen soil, man-made contaminants, particles with greatest dimension exceeding 4 inches and other deleterious materials. The suitability of soil for use as structural fill will depend on the gradation and moisture content of the soil. As the amount of fines in the soil matrix increases, the soil becomes increasingly more sensitive to small changes in moisture content and achieving the required degree of compaction becomes more difficult or impossible. Recommendations for suitable fill material are provided in the following sections.

6.8.2. Use of On-site Soil

As discussed in Section 3.3 Subsurface Conditions, on-site near surface soil generally consists of native silt and granular fill. On-site soils can be used as structural fill, provided the material meets the above requirements, although due to moisture sensitivity it could be challenging or impossible to use during periods of wet weather. If the soil is too wet to achieve satisfactory compaction, moisture-conditioning by drying back the material will be required. If the material cannot be properly moisture-conditioned, we recommend using imported material for structural fill.

An experienced geotechnical engineer from GeoEngineers should determine the suitability of on-site soil encountered during earthwork activities for reuse as structural fill.

6.8.3. Imported Select Structural Fill

Imported Select granular material may be used as structural fill. The imported material should consist of pit or quarry run rock, crushed rock or crushed gravel and sand that is fairly well-graded between coarse and fine sizes (approximately 25 to 65 percent passing the U.S. No. 4 sieve). It should have less than 5 percent passing the U.S. No. 200 sieve. During dry weather, the fines content can be increased to a maximum of 12 percent.

6.8.4. Aggregate Base

Aggregate base material located under floor slabs and crushed rock used in footing overexcavations should consist of imported clean, durable, crushed angular rock. Such rock should be well-graded, have a maximum particle size of 1-inch and have less than 5 percent passing the U.S. No. 200 sieve (3 percent for retaining walls). In addition, aggregate base shall have a minimum of 75 percent fractured particles according to American Association of State Highway and Transportation Officials (AASHTO) TP-61 and a sand equivalent of not less than 30 percent based on AASHTO T-176.

6.8.5. Aggregate Leveling Course

Aggregate leveling coarse material located under Portland cement concrete (PCC) pavement sections should consist crushed rock used in footing overexcavations should consist of imported clean, durable, crushed angular rock. Such rock should be well-graded, have a maximum particle size of ¾ -inch and have less than 5 percent passing the U.S. No. 200 sieve (3 percent for retaining walls). In addition, aggregate leveling course shall have a minimum of 75 percent fractured particles according to American Association of State Highway and Transportation Officials (AASHTO) TP-61 and a sand equivalent of not less than 30 percent based on AASHTO T-176.

6.8.6. Trench Backfill

Backfill for pipe bedding and in the pipe zone should consist of well-graded granular material with a maximum particle size of ¾-inch and less than 5 percent passing the U.S. No. 200 sieve. The material should be free of organic matter and other deleterious materials. Further, the backfill should meet the pipe manufacturer's recommendations. Above the pipe zone, Imported Select Structural Fill may be used as described above.

6.9. Fill Placement and Compaction

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

Fill and backfill material should be placed in uniform, horizontal lifts and compacted with appropriate equipment. The appropriate lift thickness will vary depending on the material and compaction equipment used. Fill material should be compacted in accordance with Table 2 below. It is the contractor's responsibility to select appropriate compaction equipment and place the material in lifts that are thin enough to meet these criteria. However, in no case should the loose lift thickness exceed 18 inches.

TABLE 2. COMPACTION CRITERIA

Fill Type	Compaction Requirements Percent Maximum Dry Density Determined by ASTM Test Method D 1557 at ± 3% of Optimum Moisture		
	0 to 2 Feet Below Subgrade	> 2 Feet Below Subgrade	Pipe Zone
Fine-grained soils (non-expansive)	92	92	----
Imported Granular, maximum particle size < 1¼ inch	95	95	----
Imported Granular, maximum particle size 1¼ inch to 4 inches (3-inch maximum under building footprints)	n/a (proof-roll)	n/a (proof-roll)	----
Retaining Wall Backfill*	92	92	-----
Nonstructural Zones	90	90	90
Trench Backfill	95	90	90

Notes:

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

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

7.0 STRUCTURAL DESIGN RECOMMENDATIONS

7.1. Six-Story Mixed use Residential/Commercial Structures (Blocks 1 through 4)

We understand development will consist of a five-story wood-framed structure over a one-story concrete podium. Blocks 3 and 4 in the initial phase will also include a one-story below grade parking level that will extend beneath the concrete podium and under Block 5. Blocks 1 and 2 are proposed for similar development but it was not confirmed at the time of this report if the below-grade parking level would also be included.

Based on information provided to us by F4OELICH Engineers (Structural Engineer), we understand that column loads will be on the order of up to 575 kips; wall loads will be on the order of up 10 klf; and floor loads on the order of 250 psf or less. We have developed our recommendations based on the design loads provided.

As a result of the anticipated loads, we estimate that static consolidation settlement of site soils overlying the competent dense to very dense gravel (absent ground improvement or rigid inclusions) could be up to 2.5- to 3.5-inches total with half that magnitude occurring as differential settlement over a horizontal distance of 50 feet.

To limit potential post-construction settlement, we recommend that proposed building loads be supported on spread footings over subgrade improved with ground improvements such as compacted aggregate piers or rigid inclusions, on spread footings founded directly on the underlying competent gravels or compacted crushed rock fill over the dense gravels, or on extended-depth pile type foundations where it may not be economically feasible to excavate to the competent gravel bearing layer.

7.2. Parking Structure (Block 5)

Block 5 is proposed to be occupied by a concrete parking structure comprising up to three stories above grade, and one below grade along the south margin of the site. Based on information provided by F4OELICH Engineers, column loads on the order of 534 kips; wall loads on the order of 10.2 kips, and floor loads of up to 250 psf are anticipated.

Based on our discussions with the project team, we anticipate that Block 5 may be constructed with zero offset from the southwest corner property line near an approximately 20-foot-tall gabion wall along Mill Creek and on the bank of the Willamette River. As detailed in the Geologic Hazard Assessment included in Appendix B, the existing wall has at least one broken (compromised) face section and is currently considered as marginally stable as a result of foundation undermining. The wall will require repair if block 5 is offset from top of the wall as discussed below, or removal/reconstruction or structural encasement if block 5 is extended to the edge of the property (i.e., zero offset). Suitable foundation options for block 5 depend on the preferred plan for the existing wall, but may include the following:

- If the existing wall is left in place and repaired, block 5 may be satisfactorily founded on shallow spread footing foundations over ground improvements such as CAPs, rigid inclusions or other ground improvements to support proposed building loads, provided that ground improvement and foundations can be designed such that they do not impose additional vertical or lateral load on the existing gabion wall after repair. Additionally, a scour analysis may be necessary as part of construction at the zero offset line to determine if additional setback from the stream and river adjacent slope should be considered so that new footings are not subject to instability as a result of scour.
- If the construction of block 5 with zero offset precludes the methods above, it may be satisfactorily founded on extended-depth foundations, or spread footings over CAPs that extend into the competent Linn Gravels or sufficiently deep such that the foundation system **can be designed so that no additional load is imposed vertically or horizontally** on the existing wall if it is to remain in place or rebuilt.

7.2.1.1. Block 5 Construction Considerations

Based on discussions with the project team, and assuming zero-offset of block 5, construction of an additional temporary or permanent wall along the face of the existing gabion wall would be necessary. If a new wall is constructed at the property boundary, the gabion wall could be removed or encased. Furthermore, the new wall could be incorporated into the exterior foundations of block 5 as a permanent below grade wall.

New foundations or permanent walls along Mill Creek should not be founded within a potential zone of scour. If project development includes constructing a permanent wall along Mill Creek to transfer structural loads to the underlying Linn Gravels, the wall should be designed for the at-rest earth pressures presented in Section 7.10 Retaining Walls of this report since it would be restrained against rotation by the structure, and should also account for additional loading that may be transferred to the back of the wall from foundation and floor loads.

7.3. Foundation Support Alternatives

7.3.1. Shallow Foundations on Linn Gravels

It is our opinion that the underlying dense to very dense gravels are suitable for shallow foundation support. However, because of the increasing depth from south to north to the competent bearing gravels across the site, it will likely be more economically feasible to support proposed building loads on spread footings over subgrade improved with ground improvements, or to found buildings on extended-depth foundations where the competent gravels are not readily exposed at more shallow depths of excavation depths during construction.

7.3.1.1. Bearing Capacity – Spread Footings on Linn Gravels

We recommend that new conventional footings be proportioned using a maximum allowable bearing pressure of 4,000 psf if supported on the underlying dense to very dense gravel or structural fill bearing on these materials. The recommended bearing pressure applies to the total of dead and long-term live loads and may be increased by one-third when considering earthquake or wind loads. This is a net bearing pressure. The weight of the footing and overlying backfill can be ignored in calculating footing sizes.

7.3.1.2. Foundation Settlement – Spread Footings on Linn Gravels

Assuming that subgrade is prepared in accordance with Section 7.5 Shallow Foundation Subgrade Preparation of this report, foundations designed and constructed on the underlying dense to very dense gravels as recommended are expected to experience total static settlements of less than 1-inch. Static differential settlements of up to one-half of the total settlement magnitude can be expected between adjacent footings supporting comparable loads.

7.3.2. Ground Improvement/Aggregate Piers

Shallow spread and continuous footings supported on CAPs or rigid inclusions can provide higher bearing capacity and reduce total and differential settlement under design loads by creating a stiffened soil matrix subgrade. Ground improvement methods typically considered in the region include rammed aggregate piers (RAP) or Geopiers, and rigid inclusion systems designed and constructed by specialty foundation construction companies. Other ground improvement systems/contractors may be considered, but should be reviewed and approved by the project team.

CAP or rigid inclusion systems are typically designed and constructed by the specialty contractor to a performance specification. In our experience they typically range from 18- to 30-inch-diameter piers spaced in a triangular distribution with center-to-center spacing ranging from 6 to 8 feet depending on design loads and tolerable settlement requirements. The specialty contractor should be given a copy of our geotechnical report and the opportunity to complete additional explorations if they choose. They should submit a ground improvement design that has been completed and stamped by a registered professional engineer with experience in such projects. We recommend the geotechnical engineer of record review the design on behalf of the Owner, although the specialty contractor will retain responsibility for the design and construction of the ground improvements to the specified performance criteria.

The underlying dense to very dense gravel of the Linn Gravel Formation was encountered at varying depths of approximately 4 to 24 feet bgs in our explorations. We anticipate that compacted aggregate piers would extend from the bottom of shallow foundations to this very dense Linn Gravel Formation or to a minimum design depth required to meet allowable bearing capacity for design loads as well as settlement tolerances for the project. Granular pads beneath shallow foundations should be discussed with the specialty contractor if required as part of load transfer to the underlying ground improvement.

The length of compacted aggregate piers may vary across the site. Compacted aggregate piers should be designed to meet the final bearing capacity and settlement tolerance provided by the structural engineer. The specialty contractor would provide final design and in-house quality control for the piers. We recommend that GeoEngineers provide construction quality assurance for the Owner during the construction process.

Structural fill to raise site grades should be placed after construction of ground improvements to reduce the overall depth of installation since improvements are typically extended to ground surface during construction.

7.3.2.1. Aggregate Piers Bearing Capacity

Allowable design bearing capacity of the compacted aggregate pier/improved subgrade matrix would be determined by the specialty contractor and will be dependent on actual building loads and acceptable settlement magnitudes. We typically see a bearing capacity of approximately 4,000 to 6,000 psf in the soil/pier matrix for soils similar to those we observed at the site that have been improved with compacted aggregate piers.

7.3.2.2. Foundation Settlement

Settlement for shallow foundations supported on an aggregate pier improved subgrade, as described above, would depend on the specialty contractor's design. Typically, systems are designed to a performance specification that is normally on the order of approximately 1-inch. Differential settlements of up to half the total magnitude can be expected between individual footings.

7.3.3. Deep Foundations

Deep foundations can be considered as a suitable option to support foundations and to transfer structural loads to the underlying, competent gravels. In addition to building loads. We anticipate driven piles (open-ended pipe of H-pile sections) or drilled and cast-in-place piles will likely be the most efficient deep foundation methods for this site.

If deep foundations are the preferred foundation alternative, they should be designed to extend through the upper middle terrace and fill deposits encountered in our explorations to underlying dense coarse-grained deposits. The top of these relatively dense layers was encountered at depths of approximately 4 to 24 feet bgs in our explorations.

7.4. Shallow Foundation Recommendations

Where shallow foundations are planned for the project, exterior footings should be established at least 18 inches below the lowest adjacent grade. The recommended minimum footing depth is greater than the anticipated frost depth. Interior footings can be founded a minimum of 12 inches below the top of the floor slab. Isolated column and continuous wall footings should have minimum widths of 24 and 18 inches, respectively. We have assumed that the maximum isolated column loads will be on the order of 40 kips, wall loads will be 2 klf or less and floor loads for slabs on grade will be 100 psf or less for the proposed development. If design loads exceed these values, we should be notified as our recommendations may need to be revised.

7.5. Shallow Foundation Subgrade Preparation

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

We recommend loose or disturbed soils resulting from foundation excavation be removed before placing reinforcing steel and concrete. Foundation bearing surfaces should not be exposed to standing water. If water infiltrates and pools in the excavation, the water, along with any disturbed soil, should be removed before placing reinforcing steel. A thin layer of crushed rock can be used to provide protection to the subgrade from weather and light foot traffic. Compaction should be performed as described in Section 6.6 Permanent Slopes.

We recommend a representative of the geotechnical engineer of record observe all foundation excavations before placing concrete forms and reinforcing steel to determine that bearing surfaces have been adequately prepared and the soil conditions are consistent with those observed during our explorations. Additionally, we recommend overexcavating and placing a minimum of 2-foot-thick granular bearing pad consisting of crushed rock, structural fill compacted in accordance with Section 6.3. Overexcavation should extend laterally 1-foot beyond the edges of footings.

7.6. Shallow Foundation Lateral Resistance

Lateral loads on footings can be resisted by passive earth pressures on the sides of footings and by friction on the bearing surface. We recommend that passive earth pressures be calculated using an equivalent fluid unit weight of 240 pcf for foundations confined by native medium stiff or stiffer silt and 350 pcf if confined by a minimum of 2 feet of imported granular fill.

We recommend using a friction coefficient of 0.35 for foundations placed on the native medium stiff or stiffer silt, or 0.50 for foundations placed on a minimum 2-foot thickness of compacted crushed rock. The passive earth pressure and friction components may be combined provided the passive component does not exceed $^{2}/_{3}$ of the total.

The passive earth pressure value is based on the assumptions that the adjacent grade is level and static groundwater remains below the base of the footing throughout the year. The top 1-foot of soil should be neglected when calculating passive lateral earth pressures unless the adjacent area is covered with pavement. The lateral resistance values include a safety factor of approximately 1.5.

7.7. Drainage

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

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

7.8. Slab on Grade Floors

7.8.1. Design Parameters

Satisfactory subgrade support for floor slabs of up to 250 psf can be obtained provided the floor slab subgrade is prepared as recommended in Section 6.0 Earthwork Recommendations of this report, including compaction of the upper exposed subgrade. Slabs should be reinforced according to their proposed use and per the structural engineer's recommendations. Load-bearing concrete slabs should be designed assuming a modulus of subgrade reaction (k) of 100 pci.

The intent of supporting on-grade slabs on a minimum 6-inch-thick compacted crushed rock base is that it acts as a capillary break and provides adequate subgrade support for slab design (develop the recommended modulus of subgrade reaction). The crushed rock base material should consist of Aggregate Base material as described in Section 6.8 Structural Fill and Backfill of this report. The material should be placed as recommended in Section 6.9 Fill Placement and Compaction. If dry slabs are required (e.g., where adhesives are used to anchor carpet or tile to the slab or for other moisture-sensitive situations), a waterproof liner may be placed as a vapor barrier below the slab. The vapor barrier should be selected by the structural engineer and should be accounted for in the design floor section and mix design selection for the concrete, to accommodate the effect of the vapor barrier on concrete slab curing.

We estimate that concrete slabs constructed as recommended will settle less than 1 inch for slabs over native soils (no ground improvement required.

7.9. Seismic Design

7.9.1. 2018 IBC Seismic Design Parameters

Parameters provided in Table 3 are based on the conditions encountered during our subsurface exploration program and the procedure and requirements outlined in the 2018 IBC and the 2019 OSSC Chapters 1 and 18. Per American Society of Civil Engineers (ASCE) 7-16 Section 11.4.8, a site specific response analysis is required for site class F sites, and a ground motion hazard analysis or site-specific response analysis is required to determine the design ground motions for structures on Site Class D and E sites with S_1 greater than or equal to 0.2g. For this project, the site is classified as site class C; therefore, in our opinion the provisions of 11.4.8 are not applicable. The parameters listed on Table 3 may be used to determine the design ground motions if the

TABLE 3. MAPPED 2018 IBC SEISMIC DESIGN PARAMETERS

Parameter	Recommended Value[1]
Site Class	C
Mapped Spectral Response Acceleration at Short Period (S_S)	0.828 g
Mapped Spectral Response Acceleration at 1 Second Period (S_1)	0.415 g
Site Modified Peak Ground Acceleration (PGA_M)	0.462 g
Site Amplification Factor at 0.2 second period (F_a)	1.2
Site Amplification Factor at 1.0 second period (F_v)	1.5
Design Spectral Acceleration at 0.2 second period (S_{DS})	0.663 g
Design Spectral Acceleration at 1.0 second period (S_{D1})	.415 g

Notes:

[1] Parameters developed based on Latitude 45.0935° and Longitude -123.389137° using the Applied Technology Council (ATC) Hazards online tool.

7.9.2. Liquefaction Potential

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

As discussed in Section 3.3.2 of this report, groundwater was encountered during our explorations at approximately 30 feet bgs. The site soils below the groundwater table are expected to include dense to very dense gravel and sand, that is not considered susceptible to liquefaction for the design earthquake event. Therefore, it is our opinion that the risk for liquefaction at the site is very low.

7.9.3. Lateral Spreading Potential

Lateral spreading related to seismic activity typically involves lateral displacement of large, surficial blocks of non-liquefied soil when a layer of underlying soil loses strength during seismic shaking. Lateral spreading usually develops in areas where sloping ground or large grade changes (including retaining walls) are present. Based on our understanding of the subsurface conditions at the site, it is our opinion the risk of lateral spreading impacting the site is low.

7.10. Retaining Walls

7.10.1. Drainage

Positive drainage is imperative behind retaining structures. This can be accomplished by providing a drainage zone behind the wall consisting of free-draining material and perforated pipes to collect and dispose of the water. The drainage material should consist of Aggregate Base having less than 3 percent passing the U.S. No. 200 sieve. The wall drainage zone should extend horizontally at least 18 inches from the back of the wall.

A perforated smooth-walled rigid drainpipe having a minimum diameter of 4 inches should be placed at the bottom of the drainage zone along the entire length of the wall, with the pipe invert at or below the base of the wall footing. The drainpipes should discharge to a tightline leading to an appropriate collection and disposal system. An adequate number of cleanouts should be incorporated into the design of the drains to provide access for regular maintenance. Roof downspouts, perimeter drains or other types of drainage systems should not be connected to retaining wall drain systems.

7.10.2. Concrete Retaining Walls Design Parameters

Retaining structures free to rotate slightly around the base should be designed for active earth pressures using an equivalent fluid unit weight (efp) of 40 pcf when the ground surface extends level behind the wall equal to a distance of at least twice the height of the wall, and 65 pcf for an inclined slope of 2H:1V above the wall. For lesser slopes between flat and 2H:1V, the efp can be linearly interpolated between the recommended values. The efp value is based on the following assumptions.

- The walls will not be restrained against rotation when the backfill is placed.
- Walls are 12 feet or less in total wall support height.
- The backfill within 2 feet of the wall consists of free-draining granular materials.
- Grades above the top of the walls are no steeper than a 2H:1V slope.
- Total wall heights are determined based on a level front slope from the base of the wall.
- Hydrostatic pressures do not develop, and drainage will be provided behind the wall.

Seismically induced lateral forces on permanent below-grade building walls can be calculated using a dynamic force equal to 10.6H psf, where H is the wall height. This seismic force should be applied with the centroid located at 0.6H from the wall base. These values assume that the wall is vertical and unrestrained and the backfill behind the wall is horizontal.

For site retaining walls, seismic lateral earth pressures should be computed as a part of retaining wall design using the Mononobe-Okabe equation or another method appropriate to the selected wall system.

Retaining walls, including foundation walls that are restrained against rotation during backfilling, should be designed for an at-rest equivalent fluid unit weight of 64 pcf when the ground surface extends level behind the wall equal to a distance of at least twice the height of the wall, and 96 pcf for an inclined slope of 2H:1V above the wall. For lesser slopes between flat and 2H:1V, the efp can be linearly interpolated between the recommended values.

Surcharge loads applied closer than one-half of the wall height should be considered as uniformly distributed horizontal pressures equal to one-third of the distributed vertical surcharge pressure. Footings for retaining walls should be designed as recommended for shallow foundations. Backfill should be placed and compacted as recommended for structural fill.

Re-evaluation of our recommendations will be required if the retaining wall design criteria for the project vary from these assumptions.

We recommend that GeoEngineers be retained to review the retaining wall design to confirm that it meets the requirements in our report. The retaining wall designer should perform global stability analysis of the proposed wall.

8.0 PAVEMENT RECOMMENDATIONS

Our pavement recommendations are based on the results of our field testing and analysis. The recommended pavement sections assume that final improvements surrounding the pavement will be designed and constructed such that stormwater or excess irrigation water from landscape areas does not infiltrate below the pavement section into the base rock materials.

Standards used for pavement design for asphalt pavement design and adapted for gravel section design by deleting the upper AC section are listed below:

- *Oregon Department of Transportation (ODOT) Pavement Design Guide* (ODOT 2019)
- *AASHTO Guide for Design of Pavement Structures* (AASHTO 1993).
- Supplement to AASHTO 93 Part II Rigid Pavement Design & Rigid Pavement Joint Design.

8.1. Drainage

Long-term performance of pavements is influenced significantly by drainage conditions beneath the pavement section. Positive drainage can be accomplished by crowning the subgrade and establishing grades to promote drainage.

8.2. On-Site Asphalt Concrete (AC) Pavement Sections

Pavement subgrades should be prepared in accordance with Section 6.2 of this report. Our pavement recommendations assume that traffic at the site will consist of occasional truck traffic and passenger cars. We do not have specific information on the frequency and type of vehicles that will use the area; however, we have based our design analysis on traffic loading consistent with heavy trucks to account for delivery- and service-type vehicles and passenger car traffic for the heavy-duty pavement sections, and passenger car traffic only for the light-duty pavement sections and the assumed equivalent single axle loads (ESALS) presented in Table 4.

Our pavement recommendations are based on the following assumptions:

- The on-site soil subgrade below proposed fill placed to raise site grades or below aggregate base sections has been prepared as described in Section 6.2 Subgrade Preparation and Evaluation of this report, and observations indicate that subgrade is in a firm and unyielding condition.
- A resilient modulus of 20,000 psi was estimated for base rock prepared and compacted as recommended.

- A resilient modulus of 4,500 psi was estimated for firm in-place soils or structural fill placed on firm native soils for the proposed parking lot and drive aisles.
- Initial and terminal serviceability indices of 4.2 and 2.0, respectively.
- Reliability and standard deviations of 75 percent and 0.45, respectively.
- Structural coefficients of 0.41 and 0.10 for the asphalt and base rock, respectively.
- A 20-year design life.

If any of the noted assumptions vary from project design use, our office should be contacted with the appropriate information so that the pavement designs can be revised or confirmed adequate.

The recommended minimum pavement sections are provided in Table 4. Pavement recommendations for "On-Site Local Roads" are for roadways within the development only.

An alternate pavement section using Aggregate Subbase material is provided below because it may be more applicable during wet-weather construction where a gravel haul road or working surface is needed to support construction traffic. Wet weather construction recommendations are provided in Section 6.0 of this report. The subbase material can be incorporated into the gravel working blankets and haul roads provided the material meets the minimum thickness in Table 4 and meets the specifications for Aggregate Subbase. Working blanket and haul road materials that pump excessively, or have excessive fines from construction traffic, should be removed and replaced with specified materials prior to constructing roadways over those areas.

TABLE 4. MINIMUM ON-SITE PAVEMENT SECTION THICKNESS

Section	Minimum Asphalt Thickness (inches)	Minimum Aggregate Base Thickness (inches)	Minimum Aggregate Subbase Thickness (inches)	Assumed Traffic Loading (Design Life ESAL's)
Light Duty (general automobile parking areas)	2.5	10	-	<10,000
	2.5	4	12	
Heavy Duty (drive aisles and heavy delivery areas, or City designated local cul-de-sac)	3.5	10	-	<50,000
	3.5	4	12	

The recommended minimum pavement sections are provided in Table 4. Pavement recommendations for "On-Site Local Roads" are for roadways within the development only.

The aggregate base course should conform to Section 6.8.4 Aggregate Base of this report and be compacted to at least 95 percent of the maximum dry density (MDD) determined in accordance with AASHTO T-180/ASTM Test Method D 1557. The AC pavement should conform to Section 00745 of the most current edition of the *ODOT Standard Specifications for Highway Construction*. The Job Mix Formula should meet the requirements for a ½-inch Dense Graded Level 2 Mix. The AC should be PG 64-22 grade meeting the *ODOT Standard Specifications for Asphalt Materials*. AC pavement should be compacted to 92.0 percent at Maximum Theoretical Unit Weight (Rice Gravity) of AASHTO T-209.

If cement amendment is used during site development, as described in Section 6.0 of this report, it may be possible to reduce the amount of aggregate base for the pavement sections. This will depend on several factors, including the prevailing weather conditions, depth of amendment and condition of the subgrade after amendment. GeoEngineers can provide additional information for on-site pavement sections if cement amendment will be used during construction.

The recommended pavement sections assume that final improvements surrounding the pavement will be designed and constructed such that stormwater or excess irrigation water from landscape areas does not infiltrate below the pavement section into the crushed base.

TABLE 5. PAVEMENT SECTION RECOMMENDATIONS WITH CEMENT AMENDED SUB-BASE

Section	Minimum Asphalt Thickness (inches)	Minimum Aggregate Base Thickness (inches)	Minimum Cement Amended Subgrade Thickness (inches)
Light Duty (general automobile parking areas)	3.0	4.0	12
Heavy Duty (drive aisles and heavy delivery areas)	3.0	4.0	12

Cement amendment may be used during site development, as described above, or to reduce the pavement section thickness. The exact design of the amount of cement to be used should be determined based on the condition of the subgrade at the time of construction and the prevailing weather conditions but should likely be between 3 and 6 percent. We recommend the minimum thickness of amendment be 12 inches. GeoEngineers can provide additional information regarding cement volumes at the time of construction. The minimum pavement sections, with a 12-inch-thick cement amended soil section, are provided in Table 5 above.

8.3. Front Street NE

8.3.1. Existing Pavement Section

The existing pavement section thickness along Front Street NE was observed using ground penetrating radar (GPR) at locations GPR-1 through GPR-7 and with a cored location at boring location B-6 as shown on Figure 2. A summary of existing pavement section thickness at is presented in Table 6.

TABLE 6. EXISTING PAVEMENT SECTION

Exploration Designation	Approximate Asphalt Concrete thickness (Inches)	Approximate Base Coarse Thickness (Inches)
GPR-1	4	14
GPR-2	4	14
B-6/GPR-3	6	12
GPR-4	6	12
GPR-5	8	22
GPR-6	6	12

Exploration Designation	Approximate Asphalt Concrete thickness (Inches)	Approximate Base Coarse Thickness (Inches)
GPR-7	8	24

8.3.2. Asphalt Concrete Pavement Design

Project development includes widening Front Street NE to accommodate increased traffic in the area from the proposed development. Widening the roadway will involve raising the current grade to match the existing roadway elevation. Fill placement to raise subgrade elevations and pavement subgrades should be prepared in accordance with Section 6.2 of this report.

AC pavement recommendations for the widening of Front Street NE are provided in Table 7. The recommended pavement sections are provided in Table 7. If any of the noted assumptions vary from project design use, our office should be contacted with the appropriate information so that the pavement designs can be revised or confirmed adequate.

Our pavement recommendations are based on the following assumptions and design parameters included in the *ODOT Pavement Design Guide*:

- The pavement subgrades, fill subgrades and site earthwork used to establish road grades below the Aggregate Subbase and Aggregate Base materials have been prepared as described in Section 6.0 of this report.
- A resilient modulus of 20,000 psi has been estimated for compacted Aggregate Base.
- A resilient modulus of 4,500 psi was estimated for subgrade prepared and compacted as recommended.
- Initial and terminal serviceability indices of 4.2 and 2.5, respectively.
- Reliability and standard deviations of 90 percent and 0.49, respectively.
- Structural coefficients of 0.41 and 0.10 for the asphalt and base rock, respectively.
- A 25-year design life.
- Estimated traffic levels (4,000,000 ESAL's) based on City of Salem Administrative Rules Division 006 Default ESALS based on a Minor Arterial Classification.

TABLE 7. MINIMUM PAVEMENT SECTIONS FOR FRONT STREET NE WIDENING

Minimum Asphalt Thickness (inches)	Minimum Aggregate Base Thickness (inches)	Minimum Aggregate Subbase Thickness (inches)
7.0	21	0.0
7.0	16	12.0

The aggregate base course should conform to Section 6.8.4 of this report and be compacted to at least 95 percent of the MDD determined in accordance with AASHTO T-180/ASTM Test Method D 1557.

The AC pavement should conform to Section 00745 of the most current edition of the *ODOT Standard Specifications for Highway Construction*. The Job Mix Formula should meet the requirements for a ½-inch Dense Graded Level 3 Mix. The AC should be PG 70-22 grade meeting the *ODOT Standard Specifications for Asphalt Materials*. AC pavement should be compacted to 92.0 percent at Maximum Theoretical Unit Weight (Rice Gravity) of AASHTO T-209.

8.3.3. Portland Cement Concrete Pavement Design

PCC pavement section recommendations for the widening of Front Street NE are provided in Table 8 and based on the assumptions below. If any of the noted assumptions vary from project design use, our office should be contacted with the appropriate information so that the pavement designs can be revised or confirmed adequate.

Our pavement recommendations are based on the following assumptions and design parameters included in the ODOT Pavement Design Guide and City of Salem Administrative Rules Section 006:

- The pavement subgrades, fill subgrades and site earthwork used to establish road grades below the Aggregate Subbase and Aggregate Base materials have been prepared as described in Section 6.0 of this report.
- A modulus of subgrade reaction (k) of 150 psi was estimated for subgrade prepared and compacted as recommended.
- A concrete rupture modulus of 600 psi was estimated based on a 28-day compressive strength of concrete equal to 4500 psi.
- A drainage coefficient of 0.9 was estimated for site silty soils.
- A joint load coefficient of 3.2 was estimated for PCC reinforced using plain dowel bars.
- Initial and terminal serviceability indices of 4.2 and 2.5, respectively.
- Reliability and standard deviations of 90 percent and 0.49, respectively.
- A 50-year design life.
- Estimated traffic levels (4,000,000 ESAL's) based on City of Salem Administrative Rules Division 006 Default ESALS based on a Minor Arterial Classification.

TABLE 8. MINIMUM PCC PAVEMENT SECTIONS FOR FRONT STREET NE WIDENING

Minimum Portland Cement Concrete Thickness (inches)	Minimum Leveling course Thickness (inches)
8.5	8

Joint spacing for PCC pavements should be designed in accordance with section 6.26(d) of the City of Salem Administrative Rules. Longitudinal spacing of joints should not exceed two times the slab thickness in feet up to a maximum distance of 15 feet.

The leveling course should conform to Section 6.8.5 Aggregate Leveling Coarse of this report and be compacted to at least 95 percent of the MDD determined in accordance with AASHTO T-180/ASTM Test Method D 1557.

9.0 DESIGN REVIEW AND CONSTRUCTION SERVICES

Recommendations provided in this report are based on the assumptions and preliminary design information stated herein. We welcome the opportunity to review and discuss construction plans and specifications for this project as they are being developed. In addition, GeoEngineers should be retained to review the geotechnical-related portions of the plans and specifications to evaluate whether they are in conformance with the recommendations provided in this report.

Satisfactory foundation and earthwork performance depends to a large degree on quality of construction. Sufficient monitoring of the contractor's activities is a key part of determining that the work is completed in accordance with the construction drawings and specifications. Subsurface conditions observed during construction should be compared with those encountered during the subsurface explorations. Recognition of changed conditions often requires experience; therefore, qualified personnel should visit the site with sufficient frequency to detect whether subsurface conditions change significantly from those anticipated.

We recommend that the geotechnical engineer of record be retained to observe construction at the site to confirm that subsurface conditions are consistent with the site explorations, and to confirm that the intent of project plans and specifications relating to earthwork, pavement and foundation construction are being met.

10.0 LIMITATIONS

We have prepared this report for the exclusive use of Future of Neighborhood Development, and their authorized agents and/or regulatory agencies for the proposed Salem Cannery 6-Story Mixed-use Development Project located along Front Street NE between Belmont Street NE and Shipping Street NE in Salem, Oregon.

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

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

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

11.0 REFERENCES

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FIGURES

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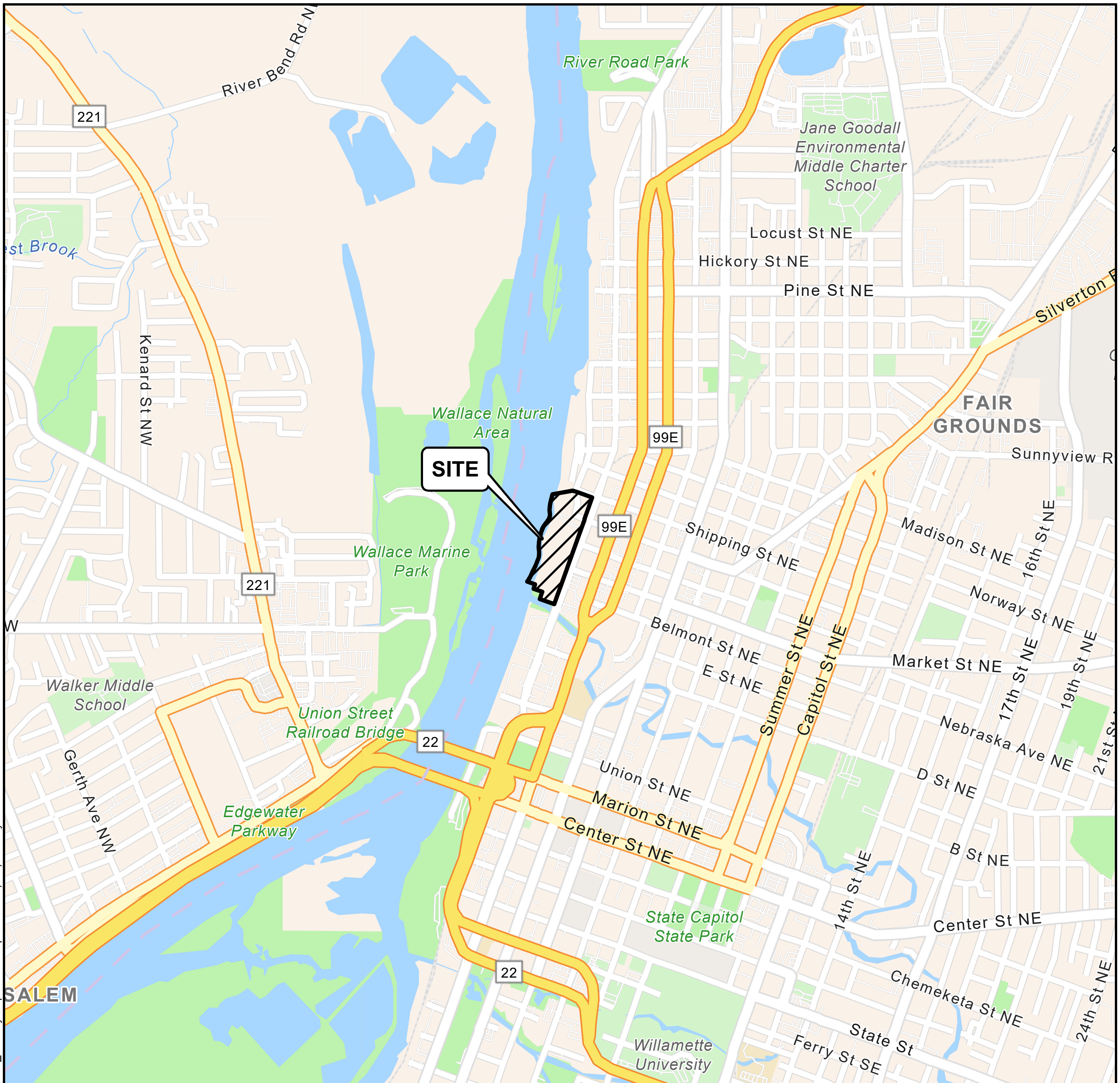

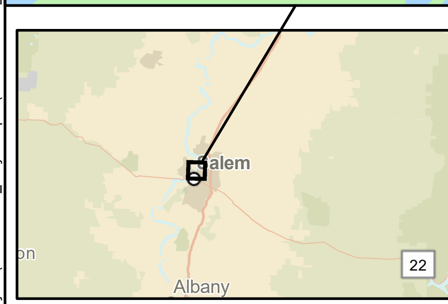

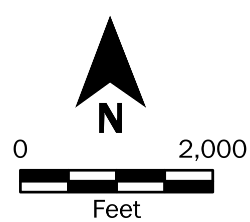

Source(s):
- ESRI

Coordinate System: NAD 1983 UTM Zone 10N

Disclaimer: This figure was created for a specific purpose and project. Any use of this figure for any other project or purpose shall be at the user's sole risk and without liability to GeoEngineers. The locations of features shown may be approximate. GeoEngineers makes no warranty or representation as to the accuracy, completeness, or suitability of the figure, or data contained therein. The file containing this figure is a copy of a master document, the original of which is retained by GeoEngineers and is the official document of record.

Vicinity Map

Salem Cannery 6-story Mixed-use Development
Salem, Oregon

GeoEngineers

Figure 1

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Legend

- Boring Number and Approximate Location
- Boring/IT Number and Approximate Location
- Boring/GPR Number and Approximate Location
- CPT Number and Approximate Location
- GPR Number and Approximate Location
- Cross Section

Source(s):
- Marion County roads

Coordinate System: NAD 1983 HARN StatePlane Oregon North FIPS 3601 Feet Intl

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N

0 140

Feet

Site Plan

Salem Cannery 6-story Mixed-use Development
Salem, Oregon

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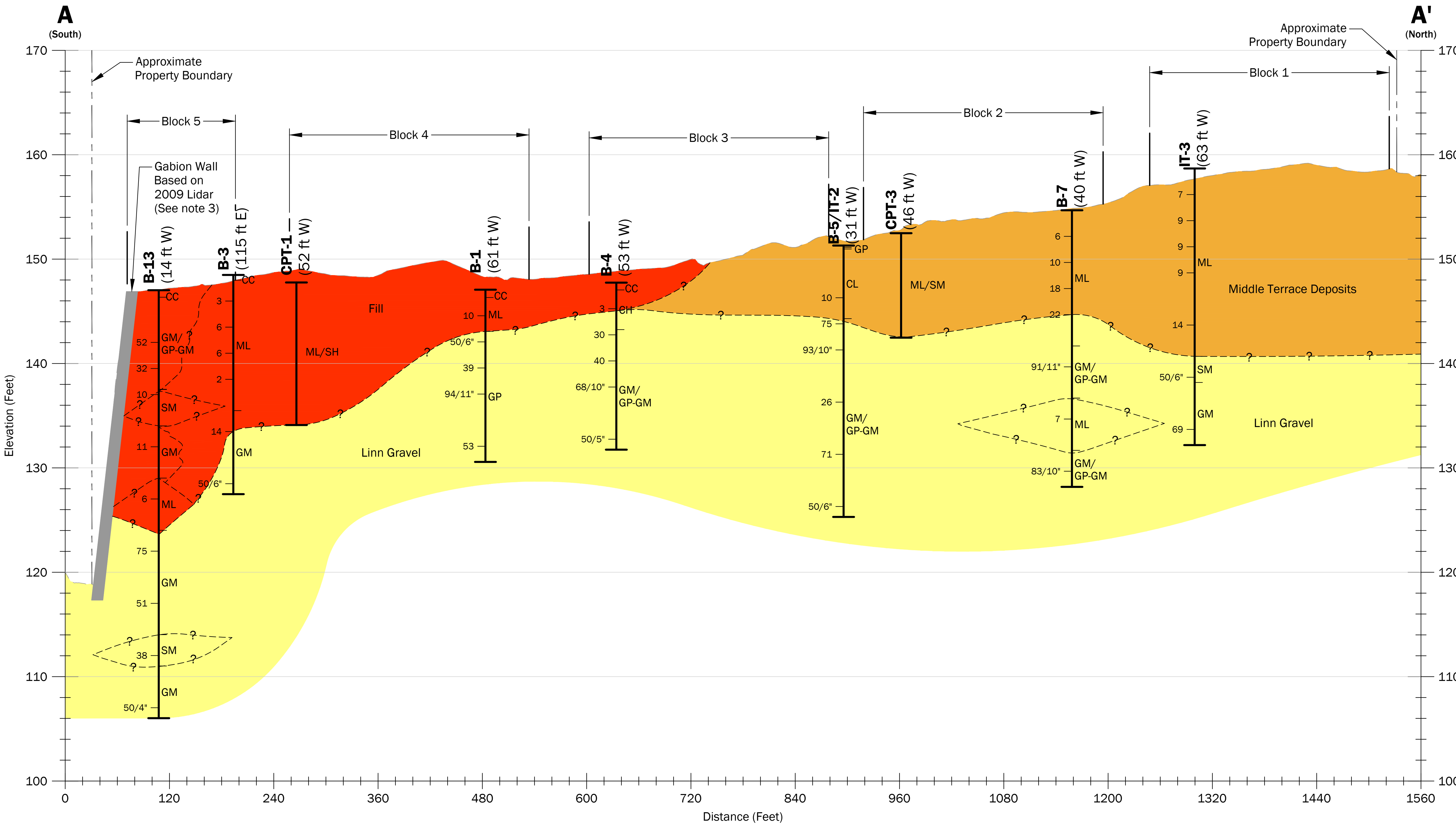
Figure 2



Notes:

1. The subsurface conditions shown are based on interpolation between widely spaced explorations and should be considered approximate; actual subsurface conditions may vary from those shown.
2. This figure is for informational purposes only. It is intended to assist in the identification of features discussed in a related document. Data were compiled from sources as listed in this figure. The data sources do not guarantee these data are accurate or complete. There may have been updates to the data since the publication of this figure. This figure is a copy of a master document. The hard copy is stored by GeoEngineers, Inc. and will serve as the official document of record.
3. Gabion wall failed between August 2022 and March 16, 2023 (Based on aerial photo review and site observation). Therefore the topography shown in the vicinity of the Gabion Wall is not accurate

Data Source: Lidar from Oregon Department of Geology and Mineral Industries Lidar Data Quadrangle Series Collected in 2009.

Datum: NAVD 88, unless otherwise noted.

Legend

Boring ID (Offset)

Boring

Inferred Soil Contact

SM Soil Classification

20 Blow Count

Fill

Middle Terrace Deposits

Linn Gravel

120 0 120
Horizontal Scale in Feet

10 0 10
Vertical Scale in Feet
Vertical Exaggeration = 12X

Cross Section A-A'

Salem Cannery 6-story Mixed-use Development
Salem, Oregon

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Figure 3

APPENDICES

APPENDIX A
Field Explorations and Laboratory Testing

APPENDIX A
FIELD EXPLORATIONS AND LABORATORY TESTING

Field Explorations

Soil and groundwater conditions at the site were explored between February 20 and February 25, 2023, by completing 13 drilled borings (B-1 to B-13), 3 infiltration tests, 6 ground penetration radar (GPR) soundings and 4 cone penetration tests (CPT's) at the approximate locations shown on Figure 2, Site Plan. The drilled borings were advanced using a track-mounted drill rig owned and operated by Western States Soil Conservation Inc. and the CPT's were advanced using a truck-mounted rig owned and operated by Oregon Geotechnical Explorations.

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

Recovered soil samples were visually classified in the field in general accordance with ASTM D 2488 and the classification chart listed in Figure A-1, Key to Exploration Logs. Logs of the borings are presented in Figures A-2 through A-14, Logs of Drilled Borings. Logs of the CPTs are presented in Figures A-15 through A-18, Logs of CPT Soundings. The logs are based on interpretation of the field and laboratory data and indicate the depth at which subsurface materials, or their characteristics change, although these changes might actually be gradual.

Laboratory Testing

Soil samples obtained from the explorations were visually classified in the field and in our laboratory using the Unified Soil Classification System (USCS) and ASTM classification methods. ASTM Test Method D 2488 was used to visually classify the soil samples. A discussion relating to the laboratory tests performed is provided below.

Moisture Content

Moisture content tests were completed in general accordance with ASTM D 2216 for representative samples obtained from the explorations. The results of these tests are presented on the exploration logs in Appendix A at the depths at which the samples were obtained.

Atterberg Limits Testing

Atterberg limits testing was performed on selected fine-grained soil samples. The tests were used to classify the soil as well as to evaluate index properties. The liquid limit and the plastic limit were estimated through a procedure performed in general accordance with ASTM D 4318. The results of the Atterberg limits testing are summarized in Figure A-19, Atterberg Limits Test Results.

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% RETAINED ON NO. 200 SIEVE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS
				SP	POORLY-GRADED SANDS, GRAVELLY SAND
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% PASSING NO. 200 SIEVE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS, ROCK FLOUR, CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS AND SILTS OF MEDIUM TO HIGH PLASTICITY
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

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

Sampler Symbol Descriptions

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

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

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

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

ADDITIONAL MATERIAL SYMBOLS

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

Groundwater Contact

Measured groundwater level in exploration, well, or piezometer

Measured free product in well or piezometer

Graphic Log Contact

Distinct contact between soil strata

Approximate contact between soil strata

Material Description Contact

Contact between geologic units

Contact between soil of the same geologic unit

Laboratory / Field Tests

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

Sheen Classification

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

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

Key to Exploration Logs

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Figure A-1

Drilled Start 2/20/2023 End 2/20/2023	Total Depth (ft) 16.5	Logged By SLG Checked By	Driller Western States Soil Conservation	Drilling Method Hollow-stem Auger
Surface Elevation (ft) 147 Vertical Datum NAVD88		Hammer Data Autohammer 140 (lbs) / 30 (in) Drop		Drilling Equipment CME 850
Easting (X) 7545339 Northing (Y) 479024		System Datum OR State Plane NAD83 (feet)		Groundwater not observed at time of exploration

Notes:

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							CC	Approximately 9 inches cement concrete pavement			
145								ML	Dark brown silt with occasional gravel (stiff to very stiff, moist) (fill)			
			6	10		1 / MC				19		
								GP	Brown silty sandy gravel with sand interbeds (very dense, moist) (Linn gravels)			
	5		6	50/6"		2						
140												
			12	39		3			Becomes dense			
	10		12	94/11"		4			Becomes very dense			
135												
	15		12	53		5						

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

Log of Boring B-1

GEOENGINEERS

Project: Salem Cannery 6 Story Mixed-use Development
Project Location: Salem, Oregon
Project Number: 26595-001-00

Figure A-2
Sheet 1 of 1

Drilled	Start 2/25/2023	End 2/25/2023	Total Depth (ft) 30.75	Logged By JLL; Checked By	Driller Western States Soil Conservation	Drilling Method Hollow-stem Auger

Surface Elevation (ft) 149 Vertical Datum NAVD88	Hammer Data Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment CME 850
Easting (X) 7545550 Northing (Y) 478991	System Datum OR State Plane NAD83 (feet)	Groundwater not observed at time of exploration
Notes:		

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
149	0							GM	Gray-brown silty gravel with sand (loose, moist) (fill)			
			6	4		1						
145	5		10	44		2						
								GP-GM	Gray-brown gravel with silt and sand (dense, moist)			
								SM	Gray-brown silty fine to medium sand with occasional gavel (loose, moist to wet)			
			10	7		3						
140	10							GP-GM	Brown gravel with silt and sand (dense, moist) (Linn gravels)			
			14	51		4						
135	15		6	50/6"		5						
130	20		6	50/5"		6						
125								GM	Brown silty gravel with sand (medium dense, moist to wet)			
	25		9	24		7						
120								GP-GM	Brown gravel with silt and sand (very dense, moist)			
	30		6	50/3"		8						

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

Log of Boring B-2

GeoEngineers

Project: Salem Cannery 6 Story Mixed-use Development
Project Location: Salem, Oregon
Project Number: 26595-001-00

Figure A-3
Sheet 1 of 1

Drilled	Start 2/20/2023	End 2/20/2023	Total Depth (ft) 21	Logged By SLG Checked By	Driller Western States Soil Conservation	Drilling Method Hollow-stem Auger

Surface Elevation (ft) 148 Vertical Datum NAVD88	Hammer Data Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment CME 850	
Easting (X) 7545397 Northing (Y) 478689	System Datum OR State Plane NAD83 (feet)	Groundwater not observed at time of exploration	

Notes:

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							CC	Approximately 6 inches cement concrete pavement			
								ML	Brown silt (soft, moist) (fill)			
145			4	3		1 / AL				31		AL (LL = 38, PI = 18)
	5		3	6		2			Becomes medium stiff			
140			6	6		3			Becomes dark brown			
	10		12	2		4 / MC			Becomes soft	28		
135								GM	Gray-brown silty gravel with sand (medium dense, moist) (Linn gravels)			
	15		12	14		5						
130												
	20		12	50/6"		6			Becomes very dense			

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

Log of Boring B-3

GeoEngineers

Project: Salem Cannery 6 Story Mixed-use Development
Project Location: Salem, Oregon
Project Number: 26595-001-00

Drilled	Start 2/21/2023	End 2/21/2023	Total Depth (ft) 16	Logged By SLG / Checked By	Driller: Western States Soil Conservation	Drilling Method: Hollow-stem Auger

Surface Elevation (ft) / Vertical Datum	147 / NAVD88	Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	CME 850
Easting (X) / Northing (Y)	7545401 / 479161	System Datum	OR State Plane NAD83 (feet)	Groundwater not observed at time of exploration	

Notes:

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							CC	Approximately 8½ inches cement concrete pavement			
145								CH	Dark brown clay (soft, moist) (fill)			
			8	3		1 / AL				38		AL (LL = 54, PI = 31)
	5		6	30		2		GM	Brown silty sandy gravel (medium dense to dense, moist) (Linn gravels)			
140												
			6	40		3			Becomes dense			
	10		6	68/10"		4			Becomes gray, very dense, moist to wet			
135												
	15		6	50/5"		5						

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

Log of Boring B-4

GeoEngineers

Project: Salem Cannery 6 Story Mixed-use Development
Project Location: Salem, Oregon
Project Number: 26595-001-00

Drilled	Start 2/21/2023	End 2/21/2023	Total Depth (ft)	26	Logged By SLG / Checked By	Driller	Western States Soil Conservation	Drilling Method	Hollow-stem Auger

Surface Elevation (ft) / Vertical Datum	151 / NAVD88	Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	CME 850
Easting (X) / Northing (Y)	7545517 / 479397	System Datum	OR State Plane NAD83 (feet)	Groundwater not observed at time of exploration	

Notes:

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
150	0							AC	Approximately 2 inches asphalt concrete pavement			
								GP	Approximately 3 inches base course			
								CL	Brown clay (stiff, moist) (Middle Terrace deposits)			
145	5	X	18	10		1 / AL				35		AL (LL = 37, PI - 15)
		X	12	75		2		GM	Brown silty sandy gravel (very dense, moist to wet) (Linn gravels)			
140	10	X	12	93/10"		3						
135	15	X	16	26		4			Becomes medium dense			
130	20	X	8	71		5			Becomes very dense			
125	25	X	8	50/6"		6						

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

Log of Boring B-5/IT-2

GeoEngineers

Project: Salem Cannery 6 Story Mixed-use Development
Project Location: Salem, Oregon
Project Number: 26595-001-00

Drilled	Start: 2/28/2023	End: 2/28/2023	Total Depth (ft): 31.5	Logged By: JLL Checked By:	Driller: Western States Soil Conservation	Drilling Method: Hollow-stem Auger

Surface Elevation (ft): 152 Vertical Datum: NAVD88	Hammer Data: Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment: CME 850
Easting (X): 7545693 Northing (Y): 479327	System Datum: OR State Plane NAD83 (feet)	See "Remarks" section for groundwater observed

Notes:

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name Testing	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0						AC	Approximately 6 inches asphalt concrete pavement			
150							GP	Approximately 12 inches base course			
							ML	Gray silt with trace sand and mica (stiff, moist) (Middle Terrace Deposits)			
			16	12		1					
	5		16	14		2					
145							GM	Gray-brown silty gravel with sand (very dense, moist) (Linn gravels)			
			9	50/5"		3					
							GP-GM	Gray-brown gravel with silt and sand (very dense, moist)			
	10		12	77		4					
140											
			6	50/3"		5					
	15			50/3"		6					
135											
							GM	Brown silty gravel with sand (very dense, moist to wet)			
	20		10	52		7					
130											
							GP-GM	Brown gravel with silt and sand (very dense, moist)			
	25		10	50/5"		8					
125											
	30		12	50		9		Becomes wet			Groundwater observed at approximately 30½ feet during drilling

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

Log of Boring B-6/GPR-3

GEOENGINEERS

Project: Salem Cannery 6 Story Mixed-use Development
Project Location: Salem, Oregon
Project Number: 26595-001-00

Figure A-7
Sheet 1 of 1

Drilled	Start 2/21/2023	End 2/21/2023	Total Depth (ft) 26.5	Logged By SLG / Checked By	Driller Western States Soil Conservation	Drilling Method Hollow-stem Auger

Surface Elevation (ft) 154 / Vertical Datum NAVD88	Hammer Data: Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment: CME 850	
Easting (X) 7545605 / Northing (Y) 479644	System Datum: OR State Plane NAD83 (feet)	Groundwater not observed at time of exploration	

Notes:

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							ML	Dark brown silt (medium stiff, moist) (Middle Terrace deposits)			
150		X	8	6		1 / MC				24		
	5	X	18	10		2			Becomes stiff			
145		X	18	18		3 / MC			Becomes very stiff	34		
	10	X	18	22		4						
140								GM	Brown silty sandy gravel (very dense, moist) (Linn gravels)			
	15	X	12	91/11"		5						
135								ML	Brown silt with occasional gravel (medium stiff, moist)			
	20	X	12	7		6 / MC				46		
130								GM/GP-GM	Brown silty sandy gravel (very dense, moist)			
	25	X	12	83/10"		7						

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

Log of Boring B-7

GeoEngineers

Project: Salem Cannery 6 Story Mixed-use Development
Project Location: Salem, Oregon
Project Number: 26595-001-00

Figure A-8
Sheet 1 of 1

Drilled	Start 2/22/2002	End 2/22/2002	Total Depth (ft) 25.75	Logged By SLG / Checked By	Driller: Western States Soil Conservation	Drilling Method: Hollow-stem Auger

Surface Elevation (ft) / Vertical Datum	156 / NAVD88	Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	CME 850
Easting (X) / Northing (Y)	7545766 / 479691	System Datum	OR State Plane NAD83 (feet)	Groundwater not observed at time of exploration	

Notes:

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
155	0							ML	Brown with rust mottling silt (stiff, moist) (Middle Terrace deposits)			
			18	10		1 / MC				27		
150	5		18	16		2			Becomes very stiff			
			18	11		3			Becomes stiff			
145	10		18	12		4 / MC				28		
								SP	Brown fine to medium sand with occasional gravel (dense, moist)			
140	15		12	34		5						
								GM	Brown silty gravel with sand (very dense, moist) (Linn gravels)			
135	20		6	50/3"		6						
	25		6	50/3"		7						

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

Log of Boring B-8

GeoEngineers

Project: Salem Cannery 6 Story Mixed-use Development
Project Location: Salem, Oregon
Project Number: 26595-001-00

Figure A-9
Sheet 1 of 1

Drilled	Start 2/20/2023	End 2/20/2023	Total Depth (ft) 21.5	Logged By SLG / Checked By	Driller: Western States Soil Conservation	Drilling Method: Hollow-stem Auger

Surface Elevation (ft) / Vertical Datum: 153 / NAVD88	Hammer Data: Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment: CME 850
Easting (X) / Northing (Y): 7545697 / 479408	System Datum: OR State Plane NAD83 (feet)	Groundwater not observed at time of exploration

Notes:

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
153	0							AC	Approximately 2 inches asphalt concrete pavement			
								GP	Approximately 3 inches base course			
								ML	Dark brown silt (stiff, moist) (Middle Terrace deposits)			
150			14	9		1 / MC				35		
	5		18	8		2			Becomes medium stiff			
145			18	46		3						
						4		GM	Brown silty gravel with sand (dense, moist) (Linn gravels)			
	10		18	63		5						
140								SM/ML	Dark brown silty fine sand to sandy silt (medium dense to very stiff, moist)			
	15		18	16		6						
						7		GM	Brown silty gravel with sand (medium dense, moist)			
135												
	20		12	43		8			Becomes dense			

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

Log of Boring B-9

GeoEngineers

Project: Salem Cannery 6 Story Mixed-use Development
Project Location: Salem, Oregon
Project Number: 26595-001-00

Drilled: Start 2/22/2002, End 2/22/2002	Total Depth (ft) 26.5	Logged By SLG / Checked By	Driller: Western States Soil Conservation	Drilling Method: Hollow-stem Auger	
Surface Elevation (ft) / Vertical Datum: 159 / NAVD88		Hammer Data: Autohammer 140 (lbs) / 30 (in) Drop		Drilling Equipment: CME 850	
Easting (X) / Northing (Y): 7545652 / 479807		System Datum: OR State Plane NAD83 (feet)		Groundwater not observed at time of exploration	

Notes:

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							ML	Brown with rust colored mottling silt (medium stiff, moist) (Middle Terrace deposits)			
155			18	7		1 / MC				29		
	5		18	9		2			Becomes stiff			
150			18	9		3						
	10		18	9		4 / MC				33		
145												
	15		18	14		5						
140								SM	Brown silty fine to medium sand (dense, moist)			
	20		12	50/6"		6 / 7		GM	Brown silty gravel with sand (very dense, moist) (Linn gravels)			
135												
	25		12	69		8						

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

Log of Boring B-10/IT-3

GEOENGINEERS

Project: Salem Cannery 6 Story Mixed-use Development
Project Location: Salem, Oregon
Project Number: 26595-001-00

Drilled	Start 2/22/2002	End 2/22/2002	Total Depth (ft)	26	Logged By SLG	Checked By	Driller	Western States Soil Conservation	Drilling Method	Hollow-stem Auger

Surface Elevation (ft) / Vertical Datum	159 / NAVD88	Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	CME 850
Easting (X) / Northing (Y)	7545700 / 479947	System Datum	OR State Plane NAD83 (feet)	Groundwater not observed at time of exploration	

Notes:

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							ML	Brown with rust colored mottling silt (stiff, moist) (Middle Terrace deposits)			
155			18	10		1 / MC				28		
	5		18	9		2 / MC				29		
150			18	11		3						
	10		18	11		4 / MC				33		
145												
	15		18	30		5			Becomes hard			
								SM	Brown silty fine to medium sand (dense, moist) (Linn gravels)			
140								GM	Brown silty gravel with sand (very dense, moist)			
	20		12	60		6						
135												
	25		6	50/4"		7						

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

Log of Boring B-11

GeoEngineers

Project: Salem Cannery 6 Story Mixed-use Development
Project Location: Salem, Oregon
Project Number: 26595-001-00

Drilled	Start 2/22/2002	End 2/22/2023	Total Depth (ft) 26.5	Logged By SLG Checked By	Driller Western States Soil Conservation	Drilling Method Hollow-stem Auger

Surface Elevation (ft) 156 Vertical Datum NAVD88	Hammer Data: Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment: CME 850	
Easting (X) 7545863 Northing (Y) 479902	System Datum: OR State Plane NAD83 (feet)	Groundwater not observed at time of exploration	

Notes:

Elevation (feet)	Depth (feet)	FIELD DATA: Interval / Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
155	0						ML	Brown with rust colored mottling silt (soft, moist) (Middle Terrace deposits)			
		12	3		1 / MC				29		
150	5	18	10		2			Becomes stiff			
		18	6		3 / MC			Becomes medium stiff	33		
145	10	18	15		4			Becomes stiff			
140	15	18	18		5			With occasional silty fine sand interbedded approximately 2 to 3 inches thick, becomes very stiff			
							GM	Brown silty gravel with sand (dense, moist) (Linn gravels)			
135	20	12	41		6 / 7						
130	25	12	85		7			Becomes very dense			

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

Log of Boring B-12

GeoEngineers

Project: Salem Cannery 6 Story Mixed-use Development
Project Location: Salem, Oregon
Project Number: 26595-001-00

Figure A-13
Sheet 1 of 1

Drilled	Start 2/21/2023	End 2/21/2023	Total Depth (ft)	41	Logged By SLG	Checked By	Driller	Western States Soil Conservation	Drilling Method	Hollow-stem Auger

Surface Elevation (ft) / Vertical Datum	147 / NAVD88	Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	CME 850
Easting (X) / Northing (Y)	7545245 / 478657	System Datum	OR State Plane NAD83 (feet)	See "Remarks" section for groundwater observed	

Notes:

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							CC	Approximately 7 inches cement concrete pavement			
145								GM	Brown silty gravel (very dense, moist) (fill)			
	5		12	52		1						
140			8	32		2			Becomes dense			
	10		3	10		3		SM	Brown silty sand with gravel (loose to medium dense, moist)			
135								GM	Brown silty gravel with sand (medium dense, moist)			
	15		6	11		4						
130								ML	Brown silt with occasional gravel (soft, moist)			
	20		18	6		5 6			Becomes gray to black with occasional organic matter			
125								GM	Brown silty gravel with sand (very dense, moist) (Linn gravels)			
	25		12	75		7						
120												
	30		6	51		8			Becomes wet			Groundwater observed at approximately 30 feet during drilling
115								SM	Gray silty fine to medium sand with trace gravel (dense, wet)			
	35											

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

Log of Boring B-13/IT-1

GeoEngineers

Project: Salem Cannery 6 Story Mixed-use Development
Project Location: Salem, Oregon
Project Number: 26595-001-00

Figure A-14
Sheet 1 of 2

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	35		18	38		9						
110						10		GM	Brown silty gravel with sand (dense, wet)			
	40		4	50/4"		11			Becomes very dense			

Log of Boring B-13/IT-1 (continued)

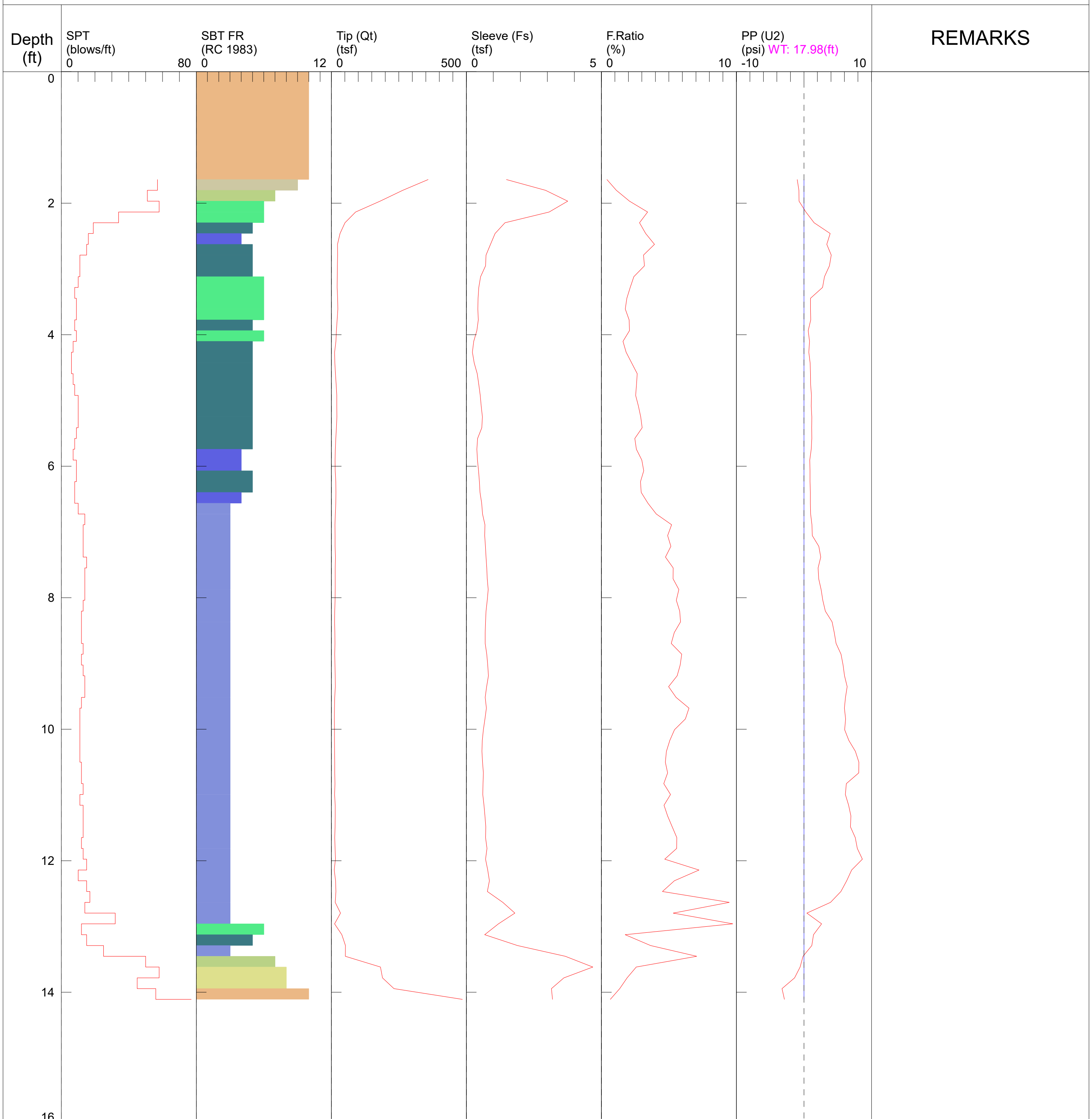
GeoEngineers

Project: Salem Cannery 6 Story Mixed-use Development
Project Location: Salem, Oregon
Project Number: 26595-001-00

Figure A-14
Sheet 2 of 2

GeoEngineers / CPT-1 / 1105 Front St NE Salem

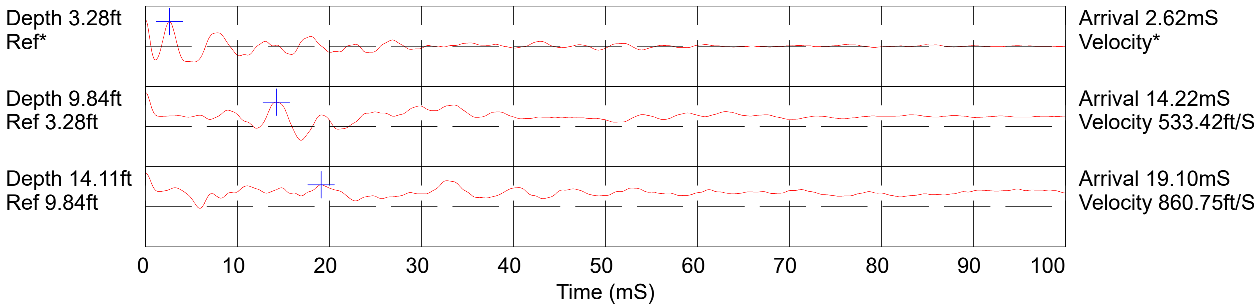
OPERATOR: OGE DMM
CONE ID: DDG1296
TEST DATE: 2/25/2023 10:44:59 AM
TOTAL DEPTH: 14.108 ft



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

*SBT/SPT CORRELATION: UBC-1983

COMMENT: GeoEngineers / CPT-1 / 1105 Front St NE Salem



Hammer to Rod String Distance (ft): 2.04
* = Not Determined

GeoEngineers / CPT-1 / 1105 Front St NE Salem

OPERATOR: OGE DMM
CONE ID: DDG1296
TEST DATE: 2/25/2023 10:44:59 AM
TOTAL DEPTH: 14.108 ft

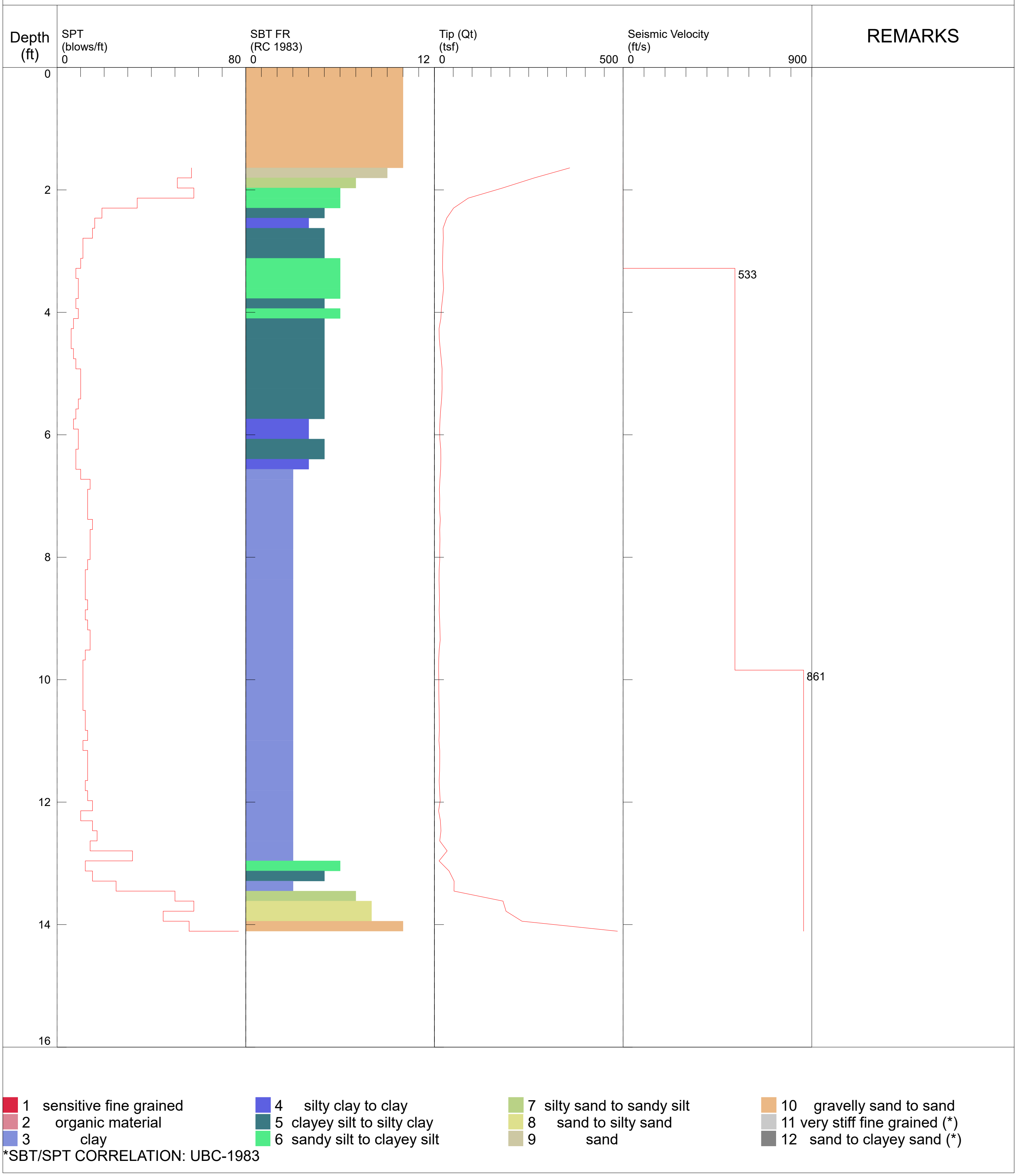
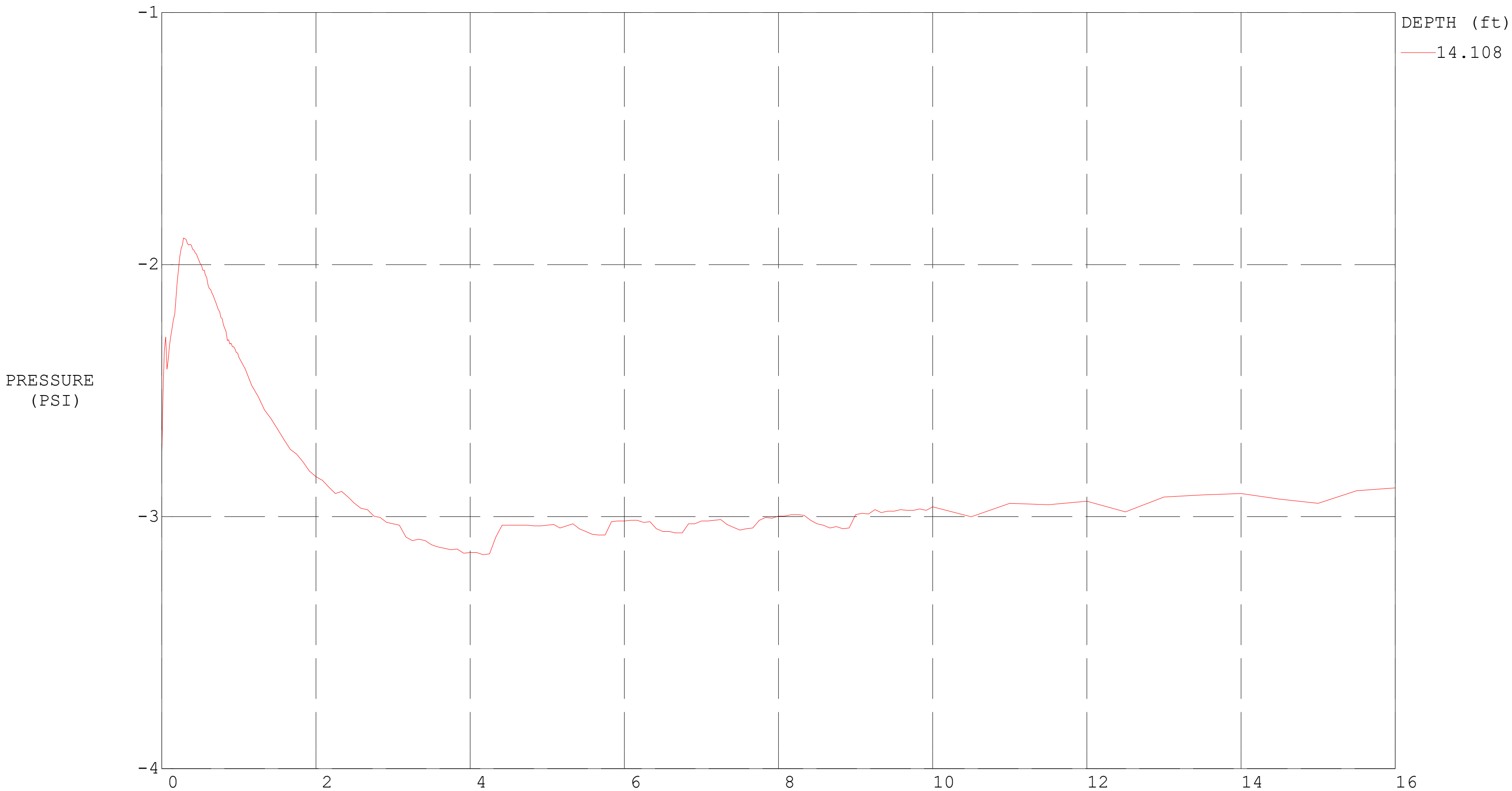


Figure A-15 (3/4)

COMMENT: GeoEngineers / CPT-1 / 1105 Front St NE Salem

CONE ID: DDG1296
TEST DATE: 2/25/2023 10:44:59 AM

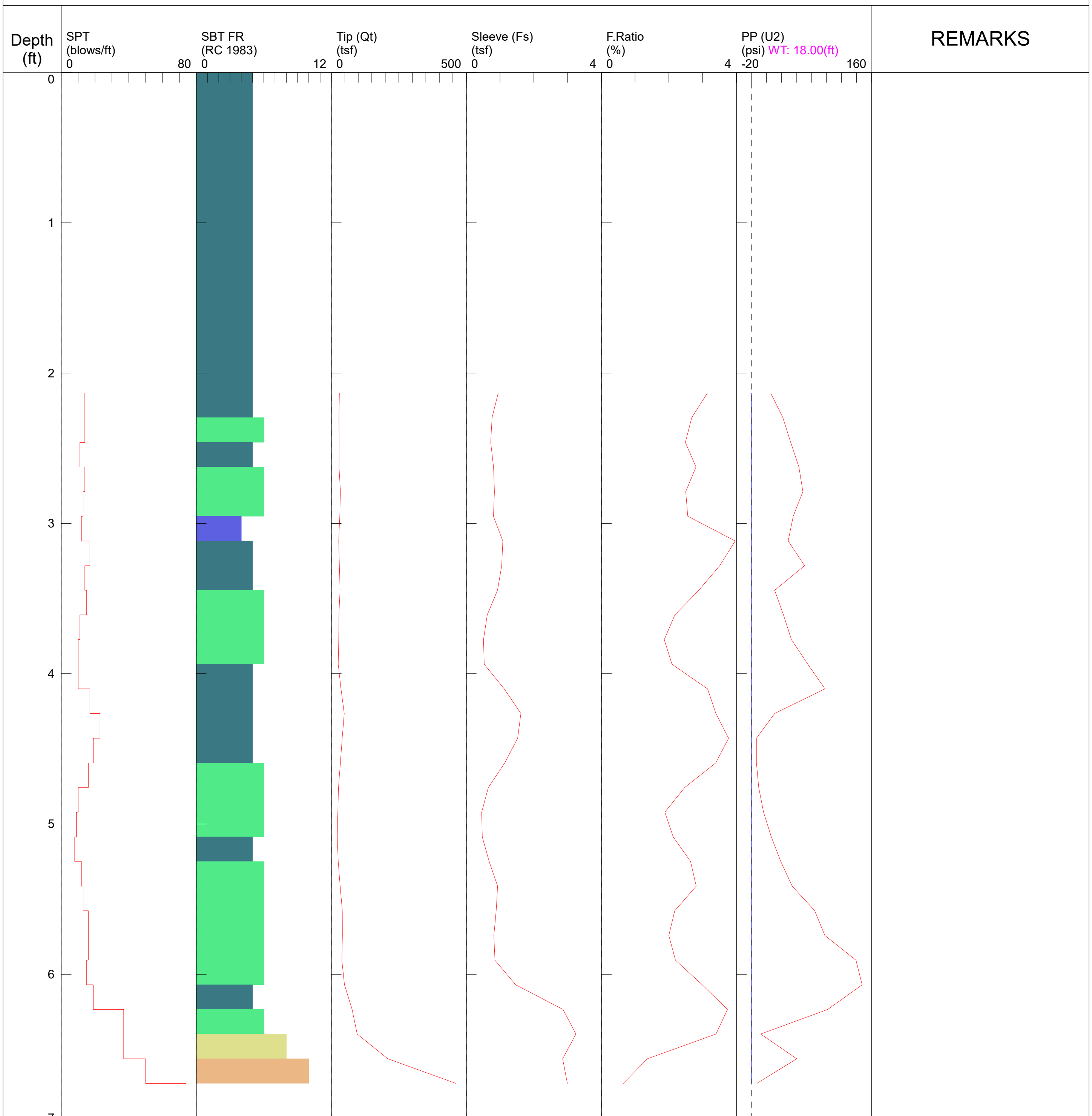


MAXIMUM PRESSURE = -1.894 (PSI) TIME: (MINUTES)
HYDROSTATIC PRESSURE = 0.0 (PSI), WATER TABLE: 17.98 ft

GeoEngineers Figure A-15 (4/4)

GeoEngineers / CPT-2 / 1105 Front St NE Salem

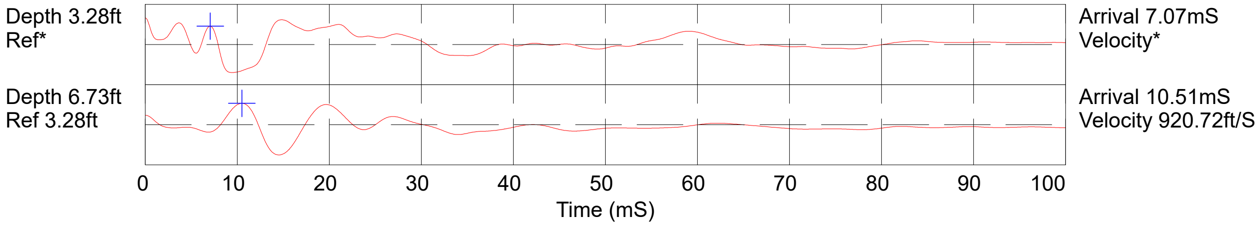
OPERATOR: OGE DMM
CONE ID: DDG1296
TEST DATE: 2/25/2023 9:52:27 AM
TOTAL DEPTH: 6.726 ft



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

*SBT/SPT CORRELATION: UBC-1983

COMMENT: GeoEngineers / CPT-2 / 1105 Front St NE Salem

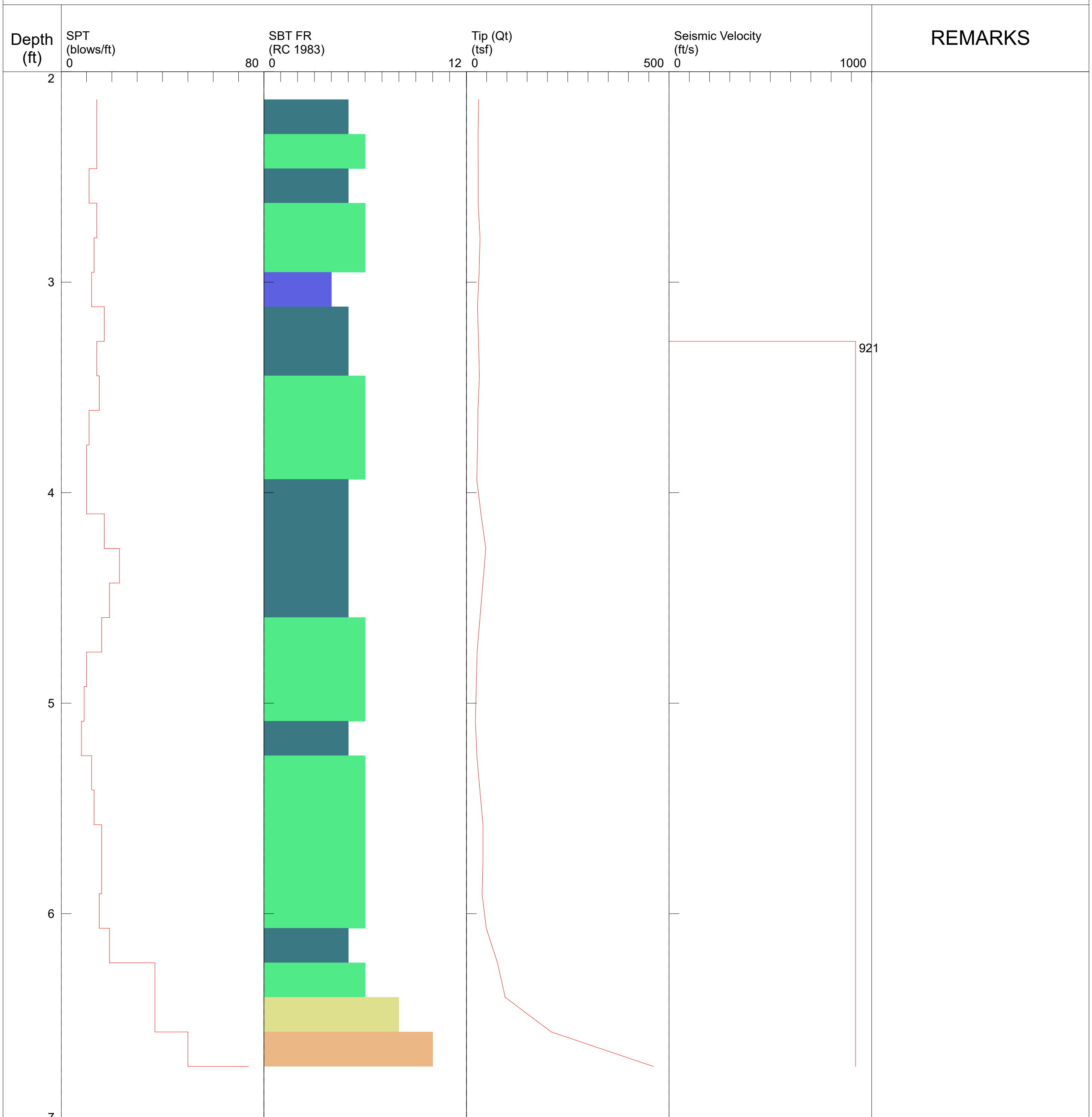


Hammer to Rod String Distance (ft): 2.04

* = Not Determined

GeoEngineers / CPT-2 / 1105 Front St NE Salem

OPERATOR: OGE DMM
CONE ID: DDG1296
TEST DATE: 2/25/2023 9:52:27 AM
TOTAL DEPTH: 6.726 ft

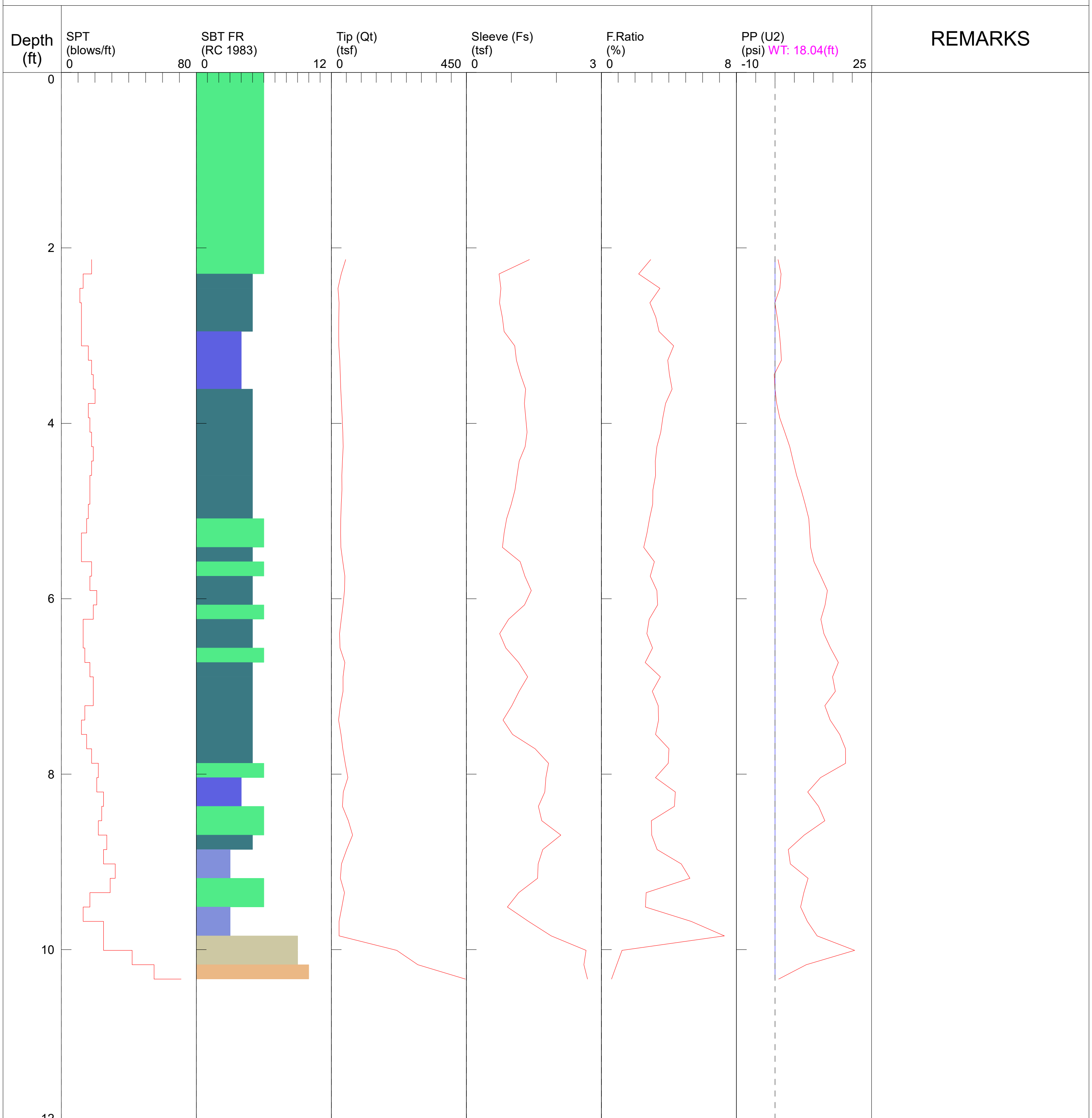


1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

GeoEngineers / CPT-3 / 1105 Front St NE Salem

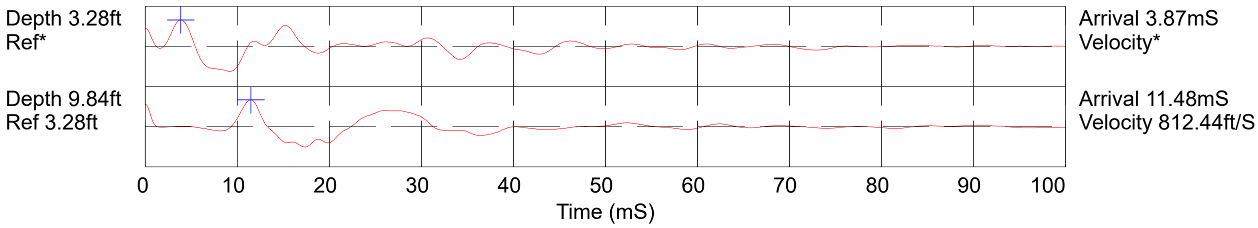
OPERATOR: OGE DMM
CONE ID: DDG1296
TEST DATE: 2/25/2023 12:10:55 PM
TOTAL DEPTH: 10.335 ft



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

*SBT/SPT CORRELATION: UBC-1983

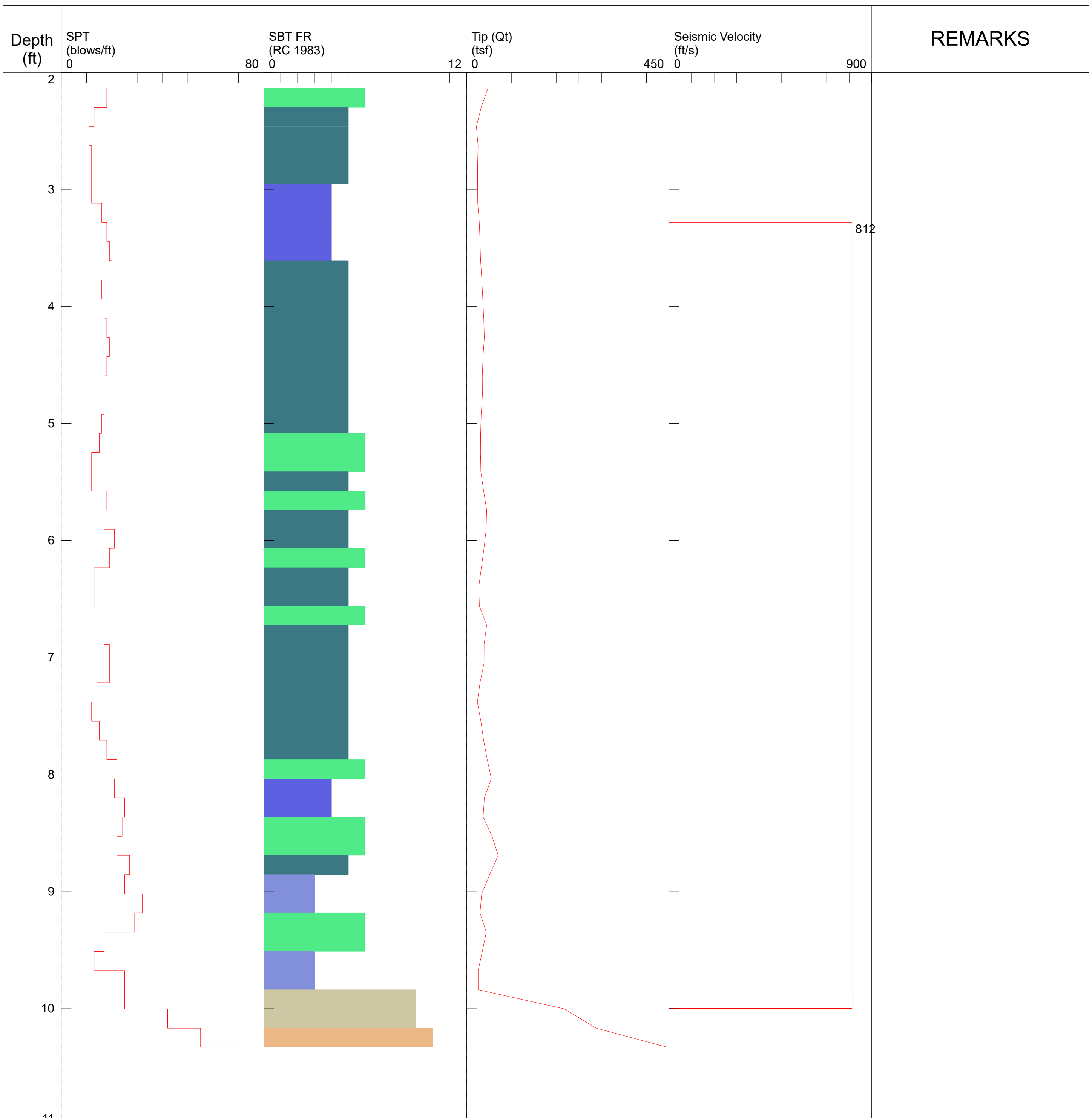
COMMENT: GeoEngineers / CPT-3 / 1105 Front St NE Salem



Hammer to Rod String Distance (ft): 2.04
* = Not Determined

GeoEngineers / CPT-3 / 1105 Front St NE Salem

OPERATOR: OGE DMM
CONE ID: DDG1296
TEST DATE: 2/25/2023 12:10:55 PM
TOTAL DEPTH: 10.335 ft

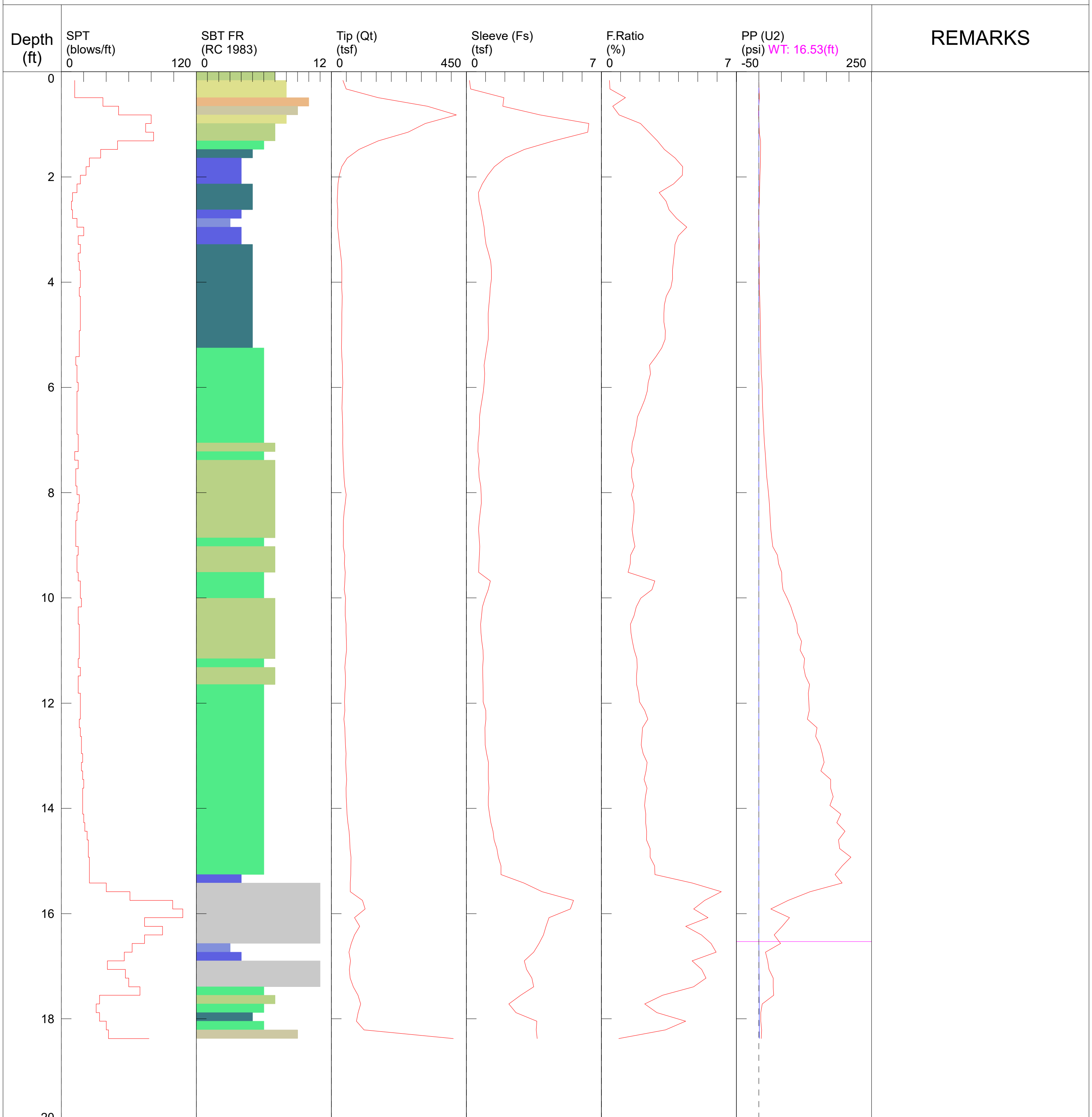


1 sensitive fine grained	4 silty clay to clay	7 silty sand to sandy silt	10 gravelly sand to sand
2 organic material	5 clayey silt to silty clay	8 sand to silty sand	11 very stiff fine grained (*)
3 clay	6 sandy silt to clayey silt	9 sand	12 sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

GeoEngineers / CPT-4 / 1105 Front St NE Salem

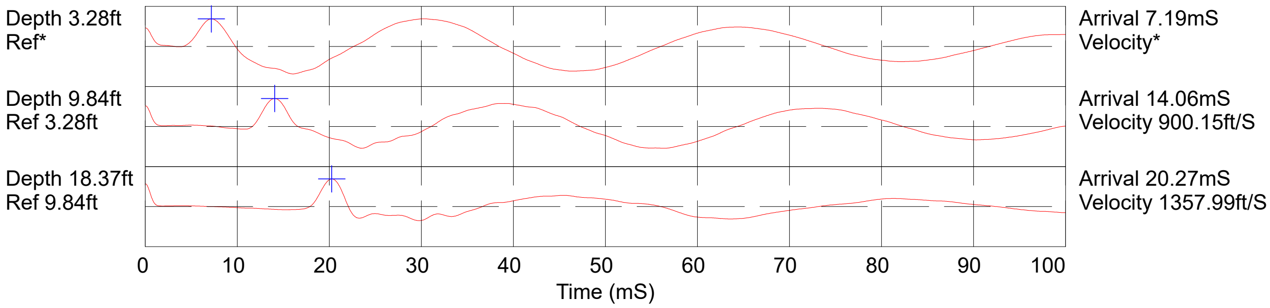
OPERATOR: OGE DMM
CONE ID: DDG1296
TEST DATE: 2/25/2023 12:41:11 PM
TOTAL DEPTH: 18.373 ft



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

*SBT/SPT CORRELATION: UBC-1983

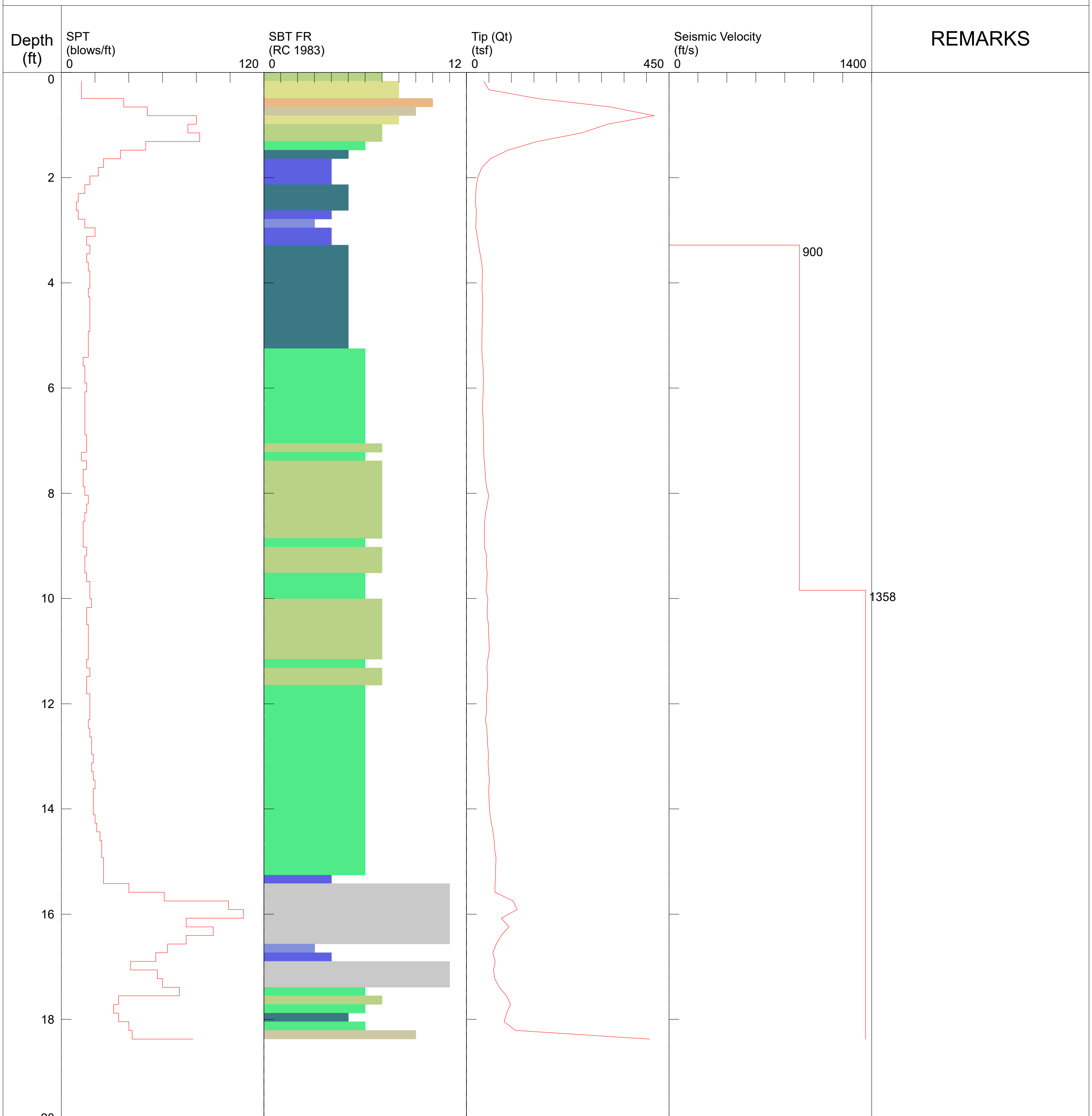
COMMENT: GeoEngineers / CPT-4 / 1105 Front St NE Salem



Hammer to Rod String Distance (ft): 2.04
* = Not Determined

GeoEngineers / CPT-4 / 1105 Front St NE Salem

OPERATOR: OGE DMM
CONE ID: DDG1296
TEST DATE: 2/25/2023 12:41:11 PM
TOTAL DEPTH: 18.373 ft



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

*SBT/SPT CORRELATION: UBC-1983

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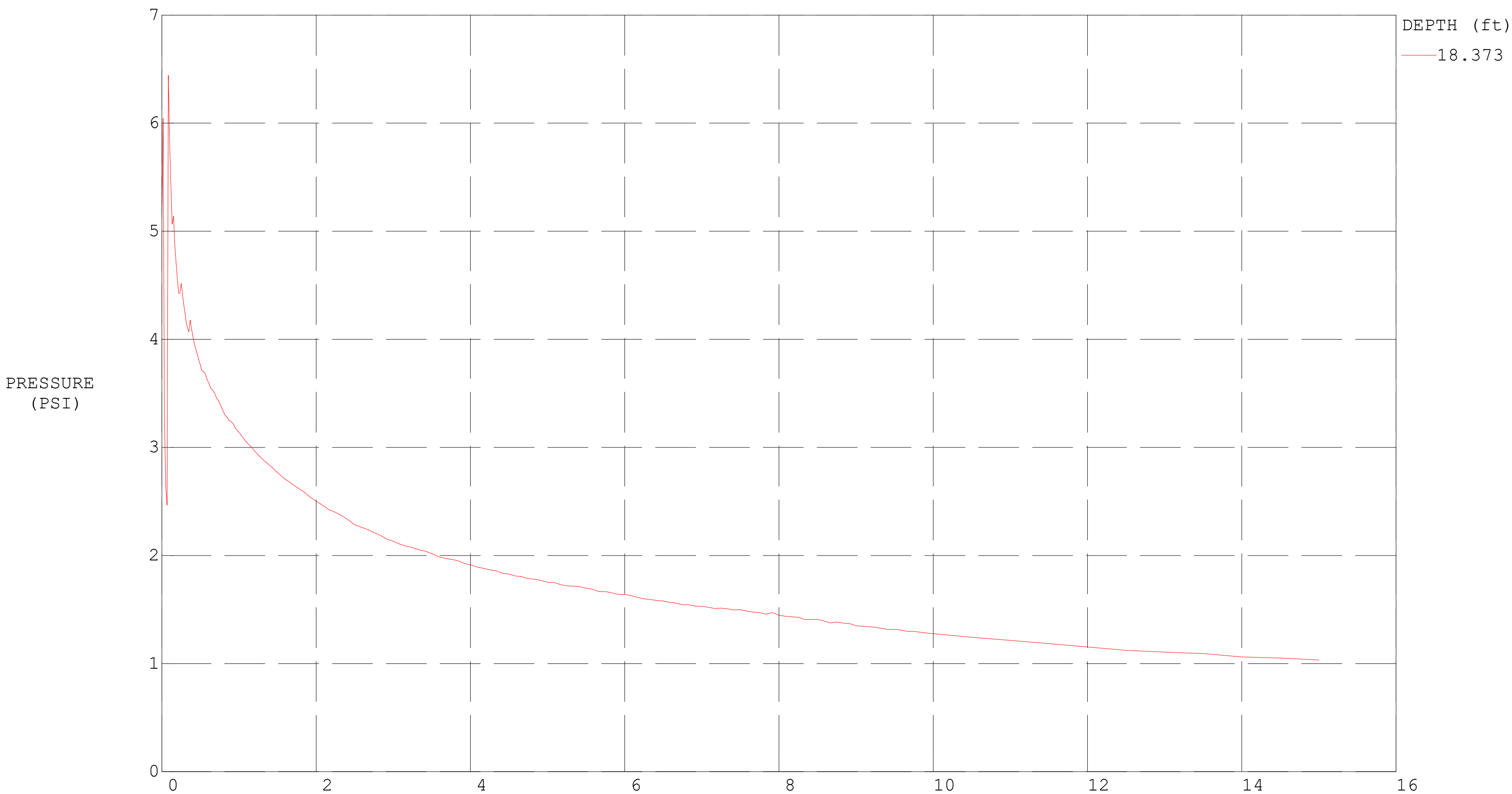
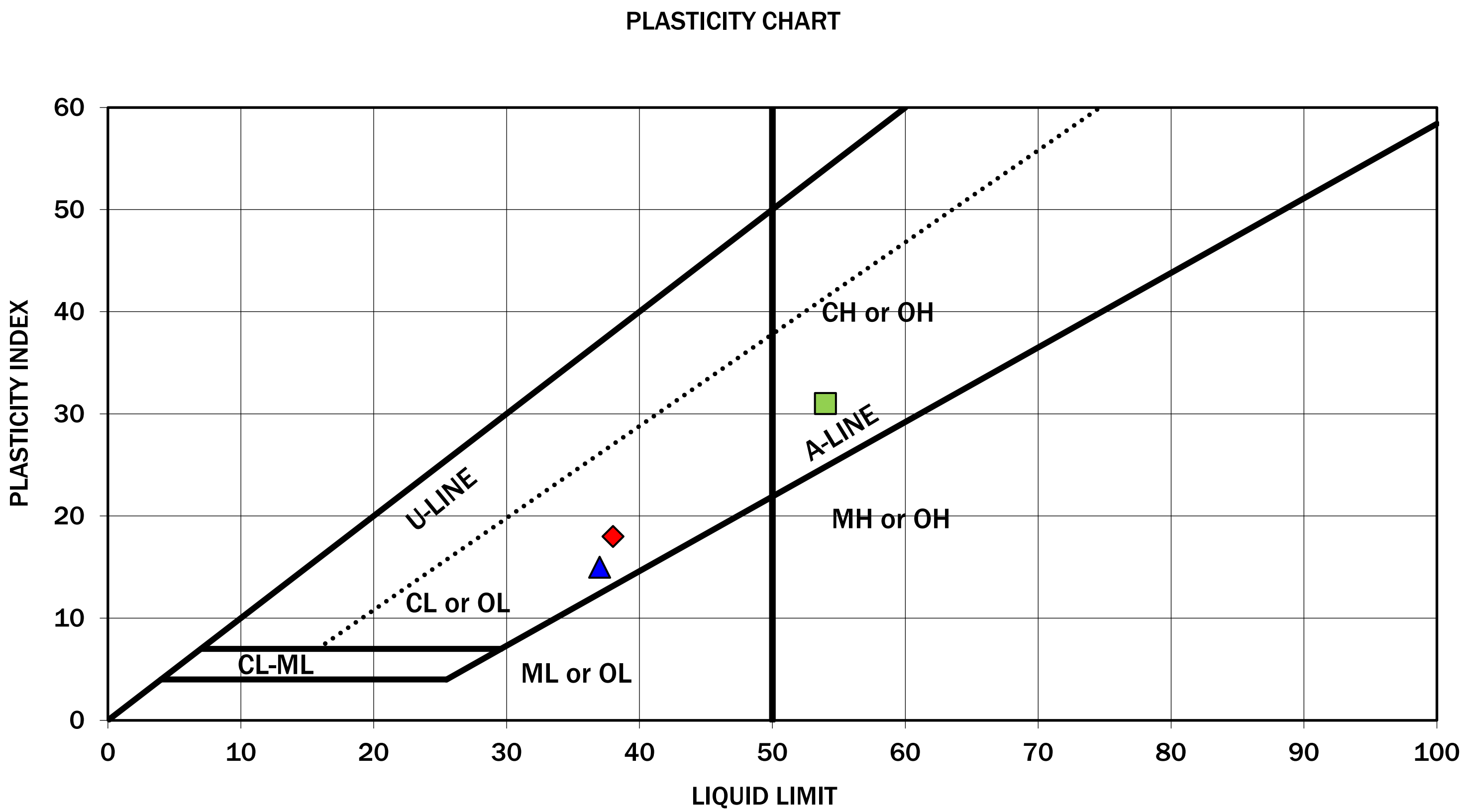
COMMENT: GeoEngineers / CPT-4 / 1105 Front St NE Salem

CONE ID: DDG1296
TEST DATE: 2/25/2023 12:41:11 PM



MAXIMUM PRESSURE = 6.444 (PSI)
HYDROSTATIC PRESSURE = 0.799 (PSI), WATER TABLE: 16.53 ft



Symbol	Boring Number	Depth (feet)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	Soil Description
◆ (red diamond)	B-3	2.5	31	38	18	Lean clay (CL)
■ (green square)	B-4	2.5	38	54	31	Fat clay (CH)
▲ (blue triangle)	B-5/IT-2	5	35	37	15	Lean clay (CL)

Note: This report may not be reproduced, except in full, without written approval of GeoEngineers, Inc. Test results are applicable only to the specific sample on which they were performed and should not be interpreted as representative of any other samples obtained at other times, depths or locations, or generated by separate operations or processes. The liquid limit and plasticity index were obtained in general accordance with ASTM D 4318. GeoEngineers 17425 NE Union Hill Road Ste 250, Redmond, WA 98052

Atterberg Limits Test Results

Salem Cannery 6 Story Mixed Use Development
Salem, Oregon

GEOENGINEERS

Figure A-19

APPENDIX B
Geologic Hazard Assessment

APPENDIX B
GEOLOGIC HAZARD ASSESSMENT

GeoEngineers, Inc. (GeoEngineers) is pleased to submit this summary of our geological assessment completed in general accordance with City of Salem Revised Code Section 810.030(a) and (b) for the proposed Salem Cannery 6-Story Mixed-use Development located along Front Street NE between Belmont Street NE to Shipping Street NE in Salem, Oregon.

To perform the geological assessment, our scope included: reviewing geologic hazard maps and selected geotechnical and geological information about the site including our subsurface investigation, performing a geologic reconnaissance to observe surface conditions at the site and preparing this appendix providing a summary of our evaluation and conclusions and recommendations.

The site is located just east of the Willamette River and bounded by Shipping Street NE to the north, Mill Creek to the south, a flood plain of the Willamette River to the west and Front Street NE to the east. The site is relatively flat. However, adjacent slopes range from approximately 50 percent on the west side adjacent to the Willamette River and vertical where an existing gabion basket wall is located on the south side of the site adjacent to Mill Creek and a small flood plain of the Willamette River. Site surface conditions are described in further detail in the “Site Reconnaissance” section of this appendix.

Desktop Review

We completed a desktop review of the site prior to our site reconnaissance. Our desktop review included a landslide hazard risk assessment in accordance with the City of Salem revised code 810.025, geologic maps of the site, the Oregon Department of Geology and Mineral Industries (DOGAMI) Statewide Landslide Information Database for Oregon (SLIDO) (DOGAMI 2023) and a Light Detection and Ranging (LiDAR) hillshade model of the site (also viewed on the SLIDO).

Landslide Risk Assessment (SRC 810.025)

Based on our desktop review of the criteria in SRC 810.025 (a) and (b):

1. Table 810-1A (Earthquake-Induced Landslide Susceptibility Ratings [Hofmeister and Wang 2000) requires review of IMS-17 and IMS-18. Most of the site is mapped as having a “Low” hazard rating. However, slopes within the property boundaries on the west and south sides of the site are mapped as the “Moderate” category. As such we assign **2 points** to the Earthquake Induced Landslide susceptibility rating.
2. Table 810-1B (Water-Induced Landslide Susceptibility Ratings [Harvey and Peterson 1998) requires review of IMS-5, IMS-6 and IMS-22. The subject site is outside the study area boundary of IMS-5 and IMS-6 and is not mapped as a “potential landslide hazard zone” in IMS-22. However, Table 810-1B specifies that since the site is outside the mapped hazard area of IMS-5 and IMS-6 and is between 15 and 25 percent slopes, it must be assigned **2 points**.
3. Table 810-1C (Activity Susceptibility Ratings) – Since the project is planned for a multi-use development, we assumed it would classify as “installation or construction of any structure greater than 500 square feet in area” and would be considered a “multiple family building permit” in accordance with Table 810-C. Therefore we assigned **2 points** to the Activity Susceptibility Rating.

4. Table 810-1D (Cumulative Score) totals the cumulative score for the subject site. As we interpret the first three tables above, the cumulative score for the site is as follows: Step 1 (2 points) plus Step 2 (2 points) plus Step 3 (2 points) **Total = 6 points**

Per Table 810-1E (Total Landslide Hazard Risk), a cumulative score of 5 to 8 points falls under the moderate landslide hazard risk (Category B), which specifies that *"…a geological assessment shall be submitted for all regulated activities. If the geological assessment indicates that mitigation measures are necessary to safely undertake the regulated activity, a geotechnical report prepared by a certified engineering geologist and geotechnical engineer shall be submitted."*

Geologic Mapping

See Section 3.2 Site Geology of the main body of this report. We note that Bela (1981) did not map any landslides at the site.

Landslide Hazard Mapping – SLIDO Review

Landslide mapping and landslide hazards for the site are compiled by the DOGAMI SLIDO (DOGAMI 2023). The SLIDO does not map landslides within the subject property, although it shows the western and southern slopes of the site as having a high regional landslide susceptibility.

LiDAR Hillshade Model Review

We reviewed a Light Detection and Ranging (LiDAR) bare earth hillshade model of the site on the SLIDO (DOGAMI 2023). We did not see obvious indications of landsliding within the site boundaries in the hillshade model.

Site Reconnaissance

We conducted a site reconnaissance on March 16, 2023. Most of the site is currently developed as an industrial cannery property with several buildings, paved parking and landscape areas, as shown on Figure 2. For the most part the development is located on the flat portion of the site between Front Street NE and slopes to the west and south. However, two of the buildings were constructed on the crest of and above the slope on the west side of the site. These buildings are founded on steel piles with the building floors above the slope. The remaining buildings appear to be founded on shallow foundations. We did not observe indications of slope movement within existing hardscape features (patio's, foundations, pavements) located on the flat portion of the site such as arcuate shaped ground cracks, significantly cracked foundations or sunken pavements.

The flat portion of the site is bounded to the west by an approximately 20- to 25-foot-high slope that terminates in a small flat floodplain of the Willamette River. In general, the slope is relatively planar and vegetated with a thick covering of blackberry and ivy and deciduous trees. We observed asphalt and concrete in portions of this slope indicating it is likely a fill slope associated with the existing development. An existing stormwater culvert daylights on this slope just north and west of the northernmost buildings, as shown on Figure 2. Stormwater flow from this culvert has eroded the slope resulting in a very steep to vertical approximately 8-foot-high slope on the south side of the culvert and undercutting of the concrete apron at the face of the culvert. Stormwater from this culvert currently falls about 5 feet between the concrete apron and asphalt/concrete/basalt boulder placed just below the apron.

The banks of the Willamette River (outside the site boundaries) are between approximately 4 and 8 feet high and vertical in many locations. Several deciduous trees are growing on these banks and just above them. We did not observe indications of recent or past landsliding on this slope, although thick blackberry cover precluded direct observation of the slopes' ground surface.

Mill Creek is located just south of the flat developed surface of the site. A gabion basket wall had been constructed on the southwest corner of the site. Ivy and blackberry was growing over the wall; however, we estimate the wall may be up to about 25 feet high. A portion of the wall on the southwest corner of the site failed resulting in an approximately 20-foot-high vertical slope. We observed gravel within the slope where the wall failed. The thalweg of Mill Creek is located on the north bank of the creek by this wall suggesting that creek erosion during flood conditions may have undercut the wall.

Geologic Hazard Conclusions

Based on our geologic hazard evaluation as presented herein, most of the slopes we observed surrounding the existing development appear relatively stable in their current configuration. However, the gabion wall on the south side of the site has failed indicating that the gabion wall, at least on the south side of the site adjacent to Mill Creek, is marginally stable. This wall likely failed because of erosion of the toe of the slope by Mill Creek. In our opinion, it would be beneficial to conduct a scour analysis to determine the likely future scour/migration of Mill Creek and how it would affect the proposed development.

A storm sewer outfall (assumed to be owned by the City of Salem) is actively eroding a portion of the slope bounding the west side of the site (see Figure 2). In our opinion, continued erosion around this outfall presents a moderate hazard of future erosion and/or landsliding adversely affecting the proposed development.

We recommend that development not encroach on any of the slopes surrounding the site. In addition, the planned development should not impart building loads on any of the site slopes and particularly the slopes in the southwest corner and south side of the site where the gabion walls are located. We recommend that new structures be placed sufficiently distant from top of slope or sufficiently deep as to maintain at least a 1.5H:1V (horizontal to vertical) set back from the base of the existing site slopes. We anticipate that the failed gabion wall will have to be mitigated in conjunction with construction of the proposed development. However, we recommend that development not alter the current configuration of the other slopes surrounding the site.

REFERENCES

Harvey, A.F. and G.I. Peterson. 1998. Water Induced Landslide Hazards, Western Portion of the Salem Hills, Marion County, Oregon: Oregon Department of Geology and Mineral Industries, Interpretive Map Series IMS-6, 13p, 1 plate, 1:24,000 scale

Hofmeister, J.R and Y. Wang. 2000. Earthquake Induced Slope Instability: Relative Hazard Map Western Portion of the Salem Hills, Marion County, Oregon: Oregon Department of Geology and Mineral Industries Interpretive Map Series IMS-17, 1 plate, 1:24,000 scale

Oregon Department of Geology and Mineral Industries (DOGAMI) 2023. Statewide Landslide Information Database for Oregon, Version 4.4, November 29, 2021. Accessed at https://www.oregongeology.org/slido/ on March 17, 2023.

APPENDIX C
Report Limitations and Guidelines for Use

APPENDIX C
REPORT LIMITATIONS AND GUIDELINES FOR USE[1]

This appendix provides information to help you manage your risks with respect to the use of this report.

Read These Provisions Closely

It is important to recognize that the geoscience practices (geotechnical engineering, geology and environmental science) rely on professional judgment and opinion to a greater extent than other engineering and natural science disciplines, where more precise and/or readily observable data may exist. To help clients better understand how this difference pertains to our services, GeoEngineers includes the following explanatory "limitations" provisions in its reports. Please confer with GeoEngineers if you need to know more about how these "Report Limitations and Guidelines for Use" apply to your project or site.

Geotechnical Services Are Performed for Specific Purposes, Persons and Projects

This report has been prepared for the Future of Neighborhood Development for the Project specifically identified in the report. The information contained herein is not applicable to other sites or projects.

GeoEngineers structures its services to meet the specific needs of its clients. No party other than the party to whom this report is addressed may rely on the product of our services unless we agree to such reliance in advance and in writing. Within the limitations of the agreed scope of services for the Project, and its schedule and budget, our services have been executed in accordance with our Agreement with the Future of Neighborhood Development and DAY CPM dated July 25, 2022 (authorized December 14, 2022) and generally accepted geotechnical practices in this area at the time this report was prepared. We do not authorize, and will not be responsible for, the use of this report for any purposes or projects other than those identified in the report.

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

This report has been prepared for the Future of Neighborhood Development Salem Cannery 6-Story Mixed-use Development Project located at Front Street NE in Salem, Oregon. GeoEngineers considered a number of unique, project-specific factors when establishing the scope of services for this project and report. Unless GeoEngineers specifically indicates otherwise, it is important not to rely on this report if it was:

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

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

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

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

If changes occur after the date of this report, GeoEngineers cannot be responsible for any consequences of such changes in relation to this report unless we have been given the opportunity to review our interpretations and recommendations. Based on that review, we can provide written modifications or confirmation, as appropriate.

Environmental Concerns Are Not Covered

Unless environmental services were specifically included in our scope of services, this report does not provide any environmental findings, conclusions, or recommendations, including but not limited to, the likelihood of encountering underground storage tanks or regulated contaminants.

Subsurface Conditions Can Change

This geotechnical or geologic report is based on conditions that existed at the time the study was performed. The findings and conclusions of this report may be affected by the passage of time, by man-made events such as construction on or adjacent to the site, new information or technology that becomes available subsequent to the report date, or by natural events such as floods, earthquakes, slope instability or groundwater fluctuations. If more than a few months have passed since issuance of our report or work product, or if any of the described events may have occurred, please contact GeoEngineers before applying this report for its intended purpose so that we may evaluate whether changed conditions affect the continued reliability or applicability of our conclusions and recommendations.

Geotechnical and Geologic Findings Are Professional Opinions

Our interpretations of subsurface conditions are based on field observations from widely spaced sampling locations at the site. Site exploration identifies the specific subsurface conditions only at those points where subsurface tests are conducted or samples are taken. GeoEngineers reviewed field and laboratory data and then applied its professional judgment to render an informed opinion about subsurface conditions at other locations. Actual subsurface conditions may differ, sometimes significantly, from the opinions presented in this report. Our report, conclusions and interpretations are not a warranty of the actual subsurface conditions.

Geotechnical Engineering Report Recommendations Are Not Final

We have developed the following recommendations based on data gathered from subsurface investigation(s). These investigations sample just a small percentage of a site to create a snapshot of the subsurface conditions elsewhere on the site. Such sampling on its own cannot provide a complete and accurate view of subsurface conditions for the entire site. Therefore, the recommendations included in this report are preliminary and should not be considered final. GeoEngineers' recommendations can be finalized only by observing actual subsurface conditions revealed during construction. GeoEngineers cannot assume responsibility or liability for the recommendations in this report if we do not perform construction observation.

We recommend that you allow sufficient monitoring, testing and consultation during construction by GeoEngineers to confirm that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes if the conditions revealed during the work differ from those anticipated, and to evaluate whether earthwork activities are completed in accordance with our recommendations. Retaining GeoEngineers for construction observation for this project is the most effective means of managing the risks associated with unanticipated conditions. If another party performs field observation and confirms our expectations, the other party must take full responsibility for both the observations and recommendations. Please note, however, that another party would lack our project-specific knowledge and resources.

A Geotechnical Engineering or Geologic Report Could Be Subject to Misinterpretation

Misinterpretation of this report by members of the design team or by contractors can result in costly problems. GeoEngineers can help reduce the risks of misinterpretation by conferring with appropriate members of the design team after submitting the report, reviewing pertinent elements of the design team's plans and specifications, participating in pre-bid and preconstruction conferences, and providing construction observation.

Do Not Redraw the Exploration Logs

Geotechnical engineers and geologists prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. The logs included in a geotechnical engineering or geologic report should never be redrawn for inclusion in architectural or other design drawings. Photographic or electronic reproduction is acceptable but separating logs from the report can create a risk of misinterpretation.

Give Contractors a Complete Report and Guidance

To help reduce the risk of problems associated with unanticipated subsurface conditions, GeoEngineers recommends giving contractors the complete geotechnical engineering or geologic report, including these "Report Limitations and Guidelines for Use." When providing the report, you should preface it with a clearly written letter of transmittal that:

- Advises contractors that the report was not prepared for purposes of bid development and that its accuracy is limited; and
- Encourages contractors to confer with GeoEngineers and/or to conduct additional study to obtain the specific types of information they need or prefer.

Contractors Are Responsible for Site Safety on Their Own Construction Projects

Our geotechnical recommendations are not intended to direct the contractor's procedures, methods, schedule or management of the work site. The contractor is solely responsible for job site safety and for managing construction operations to minimize risks to on-site personnel and adjacent properties.

Biological Pollutants

GeoEngineers' Scope of Work specifically excludes the investigation, detection, prevention or assessment of the presence of Biological Pollutants. Accordingly, this report does not include any interpretations, recommendations, findings or conclusions regarding the detecting, assessing, preventing or abating of Biological Pollutants, and no conclusions or inferences should be drawn regarding Biological Pollutants as they may relate to this project. The term “Biological Pollutants” includes, but is not limited to, molds, fungi, spores, bacteria and viruses, and/or any of their byproducts.

A Client that desires these specialized services is advised to obtain them from a consultant who offers services in this specialized field.

GeoEngineers