GEOTECHNICAL ENGINEERING REPORT
ALBANY WATERFRONT REDEVELOPMENT
ALBANY, OREGON

PREPARED FOR

WALKER MACY &
THE CITY OF ALBANY

GEOTECHNICS PROJECT NO. 19-008-1

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INTRODUCTION AND PROJECT DESCRIPTION

Geotechnics LLC is pleased to submit this geotechnical report to support design and construction of the Albany Waterfront Redevelopment project. The project extends from Monteith Park on the west and extends eastward along the Dave Clark Trail for approximately ¾-mile and also eastward along Water Avenue for a distance of 14 blocks, from Washington Street to Main Street. The project location is indicated on Figure 1. The project owner is the City of Albany and the design effort is being led by Walker Macy.

Geotechnically, the project consists of two primary elements:

<u>Park Improvements</u> will include relocation of concrete walkways, a new boardwalk pathway traversing a wetland, structural improvements to the stage, and very minor site grading, generally cuts and fills of less than 3 feet in thickness. Figure 2a shows the Monteith Park area. To the east of the park, along the Dave Clark Trail, improvements will generally not require geotechnical support and these include repurposing of two piers, improvements to an existing boardwalk, and pathway access improvements. However, this report does include a general discussion of conditions along the existing trail, including slope stability discussions for reference in case rebuilding of the trail is planned for the future.

<u>Roadway improvements</u> consist of repaving the 14-block alignment of Water Avenue shown on Figure 2b. Related streetscape improvements include curbs, pull-ins, railroad crossings, and possibly concrete unit pavers for a portion of the alignment. The roadway is currently asphalt-paved and new paving will be predominantly asphalt, but with concrete considered possible for pull-ins. The team will also consider grind & inlay options for possible re-use of a portion of the existing pavement sections.

The following report provides our geological and geotechnical assessment of the site as well as our geotechnical engineering recommendations. Our work was completed in general accordance with our subconsultant agreement with Walker Macy dated December 4, 2019.

SCOPE OF SERVICES

The purpose of our services is to evaluate soil and groundwater conditions as a basis for developing geotechnical design and construction recommendations. We completed the following specific services:

- Reviewed existing available subsurface soil and groundwater information, geologic maps, hazard maps, aerial photographs, and other information pertinent to the site.
- Performed a geologic reconnaissance to observe existing surficial slope, soil, ground, and surface water conditions at the site. Performed a trail reconnaissance.
- Explored subsurface soil and groundwater conditions at the site by drilling twelve borings: eleven machine-borings and one hand-auger boring.
- Obtained samples at representative intervals from the borings, observed groundwater conditions, performed Standard Penetration Testing, and maintained detailed logs. Performed laboratory tests on selected soil samples.
- Performed an infiltration test to assess the possibility of on-site stormwater infiltration.

- Performed pavement corings and Dynamic Cone Penetrometer (DCP) tests.
- Performed geotechnical evaluations and analyses and prepared the design recommendations presented in this geotechnical report.

This study was preceded by two previous geotechnical documents: 1) a review of background documents including geotechnical reports, geologic maps, well logs, etc. (Geotechnics, 2020a), and 2) a pavement design memorandum presenting our roadway pavement recommendations (Geotechnics, 2020b). Document 1) is summarized herein and 2) is duplicated within this report.

SITE DESCRIPTION

As described above, the project includes 14 blocks of streetscape improvements along Water Avenue and various improvements along the adjacent Willamette River waterfront. Water Avenue is at approximately Elev. 205 to 208-ft, extending east-west across the project corridor as shown on Figure 1. The waterfront park and riverbanks slope downwards to the river, with an approximate water level of Elev. 170-ft. Slope inclination is variable, including flatter park space as well as steeper banks. The steepest banks are inclined approximately 1H:1V (45 deg.) and these are generally below Elev. 195-ft.

The proposed boardwalk will cross a low-area wetland and the borders of this wetland are shown on Figure 2a. Ground elevation at the low point of this proposed crossing will be approximately 178 ft. Much of Monteith Park is within the mapped floodplain, which corresponds to approximately Elev. 200 ft according to the project survey. Surveyors have also recorded the 1996 flood level as marked on the stage structure, as Elev. 197 ft.

A pavement survey along the 14-block segment of Water Avenue was completed and our observations are presented separately below in the *Pavements* section.

BACKGROUND DATA

GEOLOGY AND SOILS

Our prior study (Geotechnics, 2020a) included a review and summary of geologic maps, regional geology, soils maps, hazard maps, and existing geotechnical data. Generally, this review showed that expected near-surface native soils are alluvial flood deposits and alluvial terrace deposits. Fill soils were also anticipated to some extent.

HAZARDS

For a preliminary assessment of geologic hazards, we primarily relied on the document, "*Geologic Hazards, Earthquake and Landslide Hazard Maps, and Future Earthquake Damage Estimates for Six Counties*….." (Burns et al., 2008). Maps within Appendix D of that document are intended to provide general estimates of the degree of hazard to be expected from ground motion amplification, liquefaction, and landslide susceptibility. These maps indicate the site likely falls within the following hazard levels:

- Ground Motion Amplification - High
- Liquefaction Susceptibility - Moderate
- Landslide Susceptibility - Moderate

The online Hazvu (DOGAMI, 2020) presents maps showing relative hazard levels for the site:

- Cascadia Earthquake - Very Strong Shaking
- Earthquake - Very Strong Shaking
- Landslide - Areas of Moderate and High
- Flooding - Yes, flood hazard zone in north portion of Monteith Park including the stage.

Liquefaction: Liquefaction is a phenomenon caused by a rapid increase in pore water pressure that reduces the effective stress between soil particles, resulting in the sudden loss of shear strength in the soil. Granular soils, which rely on interparticle friction for strength, are susceptible to liquefaction until the excess pore pressures can dissipate. In general, loose, saturated sand soils with low silt and clay contents are the most susceptible to liquefaction. Silty soils with low plasticity are moderately susceptible to liquefaction under relatively higher levels of ground shaking. The silt soils encountered at this site have moderate plasticity and are relatively stiff. These soils are very unlikely to liquefy and are generally above the groundwater table. The granular soils along the roadway alignment are predominantly dense to very dense, thus these soils are also unlikely to liquefy. In the north portion of the park, some potentially liquefiable gravels were encountered (see boring B-10 below), loose to medium dense gravels with minimal silt content. Our borings did not extend to great depth in this area, thus the hazard is undefined. But the project contains no high value structures in that area that would warrant designing against liquefaction, so the hazard can remain undefined.

Ground Rupture: We reviewed the USGS online Fault and Fold database (USGS, 2020) which includes all active and potentially active known faults. This mapping shows no known faults passing through the site, with the closest being the Owl Creek Fault, approximately 5 miles to the southwest. Based on the distance to the mapped fault, we consider the potential for ground rupture at this site to be remote.

Slope Instability: The relatively steep riverbank slopes have the potential for shallow slope instability along much of the Dave Clark Trail. This hazard is discussed in more detail below.

ENCOUNTERED SUBSURFACE CONDITIONS

We completed field explorations on October 12th and 13th, 2020 consisting of eleven machine-borings to depths ranging from 10 to 16½ feet below ground surface (bgs). The borings were completed by Dan Fischer Excavating of Forest Grove, Oregon using a trailer-mounted drill rig advancing 4-inch diameter solid-stem augers. Approximate boring locations are shown on Figures 2a and 2b, and boring logs are included in Appendix A. Standard Penetration Tests (SPT) were completed in general conformance with ASTM Test Method D1586, "*Standard Method for Penetration Test and Split-Barrel Sampling of Soils*." In addition to the machine borings, we completed one hand auger boring in the proposed boardwalk area.

Samples were collected from the borings and returned to our soils laboratory for further examination and testing. Testing included Moisture Content (44 tests in accordance with ASTM D2216), Fines Content (3 tests, ASTM D1140), and Grain Size Distribution (1 test, ASTM D6913). Laboratory test results are presented in Appendix A on the boring logs (Figures A1 - A12) and on Figure A16.

Generally in agreement with the published geologic maps and other documents reviewed (Geotechnics, 2020a), native near-surface soils at the site consist of alluvial deposits. The roadway alignment exhibited some fill soils towards the west end while the park area was dominated by fill soils. Soils in the wetland at the location of the proposed boardwalk consisted of recent alluvium. Others have documented the deeper siltstone bedrock underlying the area, but our borings were too shallow to encounter this layer. Encountered soil and groundwater conditions are described separately for the three areas below.

ROADWAY

Fill: Fill soils were encountered in borings B-6, B-7, and B-11 as well as some probable trench-backfill material in boring B-5. The four eastern borings, B-1 through B-4, did not encounter fill soils. Fill soil depths in the three western borings were 5, 9½, and 2½ ft bgs for B-6, B-7, and B-11 respectively. Fill was highly variable from elastic silt to clayey gravel. Metal debris was encountered in boring B-6. SPT blow counts ranged from 11 to 31 blows per foot (bpf). The trench backfill in boring B-5 was looser, from 8 to 13 bpf.

Alluvium: These reworked alluvial deposits transition from fine deposits (generally silt) to coarse deposits (sands and gravels) at depths greater than 7 feet bgs.

> Fine Alluvium: Near surface soils beneath pavement and fill consist of fine alluvium, generally low to moderate plasticity silt. The soil was generally stiff, with SPT blow counts from 8 to 20 bpf with an average of 12.6 bpf. Moisture contents varied from 25 to 37 percent with an average of 30.7 percent. These soils possess low to moderate shear strength, moderate compressibility, and low permeability.
>
> Coarse Alluvium: In seven of the eight borings, sand and gravel alluvium was encountered beneath the fine alluvium and/or fill. The depth of this contact varied from 7 to 10¼ ft bgs with an average of 9.0 ft bgs. These soils are generally dense to very dense gravel with minor silt and sand.

Groundwater: Groundwater was not encountered although some of the deeper soils were described as very moist. We found that groundwater is deeper than 11 ft bgs along this road alignment, at the time of our explorations. This is consistent with our data review (Geotechnics, 2020a) which found that groundwater in this area only approaches 10 ft bgs during the wettest months.

PARK

Fill: Borings B-8 and B-9 encountered fill to total depth (greater than 14 ft bgs) while boring B-10 transitioned from fill to alluvium at a depth of 10½ ft. Fill was quite variable with gravelly silt predominating in B-8 and B-10, while B-9 was mostly silty gravel. Consistency and density was also variable, with SPT blow counts ranging from 2 to 33 bpf, with an average of 12 bpf. Moisture contents varied from 10 to 36 percent with an average of 19.8 percent.

Coarse Alluvium: The northernmost machine-boring, B-10, encountered coarse-grained alluvial soils below 10½ ft bgs and extending to the final boring depth of 16½ ft. The material was gravel with silt and sand, generally medium dense.

Groundwater: Groundwater was not encountered in the southernmost boring, B-8, with a total drilled depth of 14 ft bgs. So, this upper-park area may be appropriate for the siting of shallow stormwater infiltration facilities. The northern two borings did encounter rapid groundwater seepage from granular soils, at depths of 11½ and 12 ft for B-9 and B-10 respectively. This corresponds to groundwater elevations of 176’ and 171½’ respectively. In the B-10 boring, this elevation is very similar to the level of the Willamette River surface (Elev. 171’ at the time of the project survey).

BOARDWALK

Alluvium: Boardwalk area soils are considerably different, consisting of recent alluvium. Above 7-ft depth in boring HA-1, soils were very loose silty sand. One fines content test in this material resulted in 25% fines (materials finer than US No. 200 sieve). Below 7-ft, soils transition to very soft silt, becoming organic silt below 8.4 ft.

We had to terminate our boring at 9¼-ft bgs in these very soft soils. To determine their extent and thickness, we returned to the site in December and performed an adjacent *Wildcat Cone* test, a type of DCP using a larger hammer and threaded 1-meter rods, appropriate for greater depths than the pavement DCP testing described below. The log for this DCP-HA1 is included as Figure A13, with a total depth of 15¾-ft. Additionally, the converted N-values (blows per foot, bpf) from the Wildcat Cone are shown graphically on the HA-1 boring log, Figure A12.

From the DCP data, we can see that the very soft layer ends at about 10 ft bgs. The underlying soils between 10 and 13 ft are not dense, but much stiffer/denser than those above, and will have some foundation bearing capacity (see *Boardwalk Foundations* section, below).

Groundwater: Groundwater was encountered in HA-1 at a depth of 6.9 ft bgs. This is approximately Elev. 171½ ft, nearly coincident with the river surface level.

INFILTRATION

We completed one infiltration test at the approximate location shown on Figure 2a. Test I-1 was completed adjacent to boring B-8, approximately 3 feet away. The test was conducted using the falling head percolation test procedure. The test involves embedding a 6-inch plastic pipe 6 inches into the soil at the test depth, pre-soaking the soil, then measuring the drop in water head over a period of two hours. Three test iterations were performed and the average was selected as the unfactored infiltration rate. For continuous data collection, we utilized a submerged data logger and checked this with periodic manual readings.

Infiltration test results are summarized in the table below. Plots of the transducer readouts are provided as Figure D1. At the test location, we extended an adjacent boring (B-8) deeper than test level in order to assess soil and groundwater conditions within the primary infiltration zone. The B-8 boring log is included as Figure A8.

Infiltration Test	**Location**	**Depth**	**USCS Material Type**	**Percent Fines**	**Field Measured Infiltration Rate (in/hr) [1]**
I-1	North of Sr. Center parking	2’6”	Sandy SILT with Gravel (ML)	59.5	0.41

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

PAVEMENT EVALUATION AND SUBGRADE TESTING

DCP TESTING

To supplement the visual observations, lab testing, and SPT testing and further evaluate subgrade soils for pavement design, we performed Dynamic Cone Penetrometer (DCP) testing. After coring through existing pavement and removing existing base course rock, a cone was driven into the soil using a 575-mm drop of an 8-kg hammer. The penetration versus blow count (mm/blow) was recorded as the DCP value. Standard correlations provided by ODOT Pavement Services (ODOT, 2019b) provide resilient modulus (M_R) and California Bearing Ratio (CBR) values for use in pavement design. The apparatus and testing procedures are in accordance with ASTM D6951.

We performed DCP tests in the three locations shown on Figure 2b. The table below summarizes the results of the tests. Complete DCP logs are included as Figures A14 and A15.

DCP Test	Top Test Depth (inches)	Distance Driven (inches)	Material Tested	Average DCP (mm/blow)	[1]Average M_R (psi)	[2]Corrected Average M_R (psi)	[3]Correlated CBR (%)
DCP-B1	29	10	Silt (ML)	13.4	18,083	6,329	4.2
DCP-B4	23	14	Silt (ML)	24.2	14,224	4,978	3.3
DCP-B6	33	10	Elastic Silt (MH)	25.4	13,884	4,859	3.2
DCP-B11	10	30	Sandy Silt to Silt (ML)	23.4	15,100	5,285	3.5

1. M_r value based on ODOT recommended correlation: $M_r = 49{,}022.76{*}(DCP)^{-0.39}$, rounded to nearest 100 psi.
2. Corrected M_r value based on ODOT recommended correction factor of 0.35 for fine-grained subgrade soil, rounded to nearest 100 psi.
3. California Bearing Ratio Correlation: CBR = M_R/1,500.
4. All values based on upper, weaker soils above 43-inch depth and as shown on Figures A9 and A10.

PAVEMENT CORING

All eight of our borings included pavement corings to determine pavement and base rock thickness. Findings are included in the attached boring logs and summarized here. We did not encounter any geotextiles within or below the base aggregate (base rock). The quality of the base rock was variable, as described in the boring logs.

Boring	Station	AC Thickness (inches)	Base Rock Thickness (inches)
B1	52+17	4.5	23
B2	45+46	5.0	25
B3	39+03	3.0	24
B4	32+04	2.5	22.5
B5	26+21	5.0	24+
B6	20+57	5.0	26
B7	14+47	6.0	18
B11	08+64	2.0	8
Average	---	4.1	21.3

PAVEMENT CONDITION

We performed a pavement condition assessment using the methods presented in the *Pavement Data Collection Manual* (ODOT, 2019a). We referenced our field survey to the project stationing provided by KPFF, shown on Figure 2b. We summarized our findings in five project segments defined by significant changes in pavement condition. The pavement evaluation is supplemented with photographs of pavement distress, presented as Figures B1 through B7.

Our results are summarized in the table below.

	Distress Level **L = Low M = Moderate H = High**					
Station	Fatigue Cracking	Longitudinal Cracking	Transverse Cracking	Potholes	Raveling	**Overall**
6+80 to 9+70	H	H	M	M	H	**H**
9+70 to 16+00	L	M	M	L	L-M	**L-M**
16+00 to 27+00	L-M	M	L-M	L	L-M	**L-M**
27+00 to 41+50	L-M	L-M	L	L	M	**L-M**
41+50 to 53+00	L-M	M	M	L	L	**L-M**

SUMMARY OF PAVEMENT CORING AND SURFACE CONDITIONS

Pavement condition is a function of pavement profile, past traffic, subgrade soils, drainage conditions, and age. It is not surprising that the poorest pavement condition coincides with the thinnest pavement (see boring B-11). Subgrade soils are generally low quality, requiring a relatively thick pavement section for a moderate traffic level. We are unaware of the pavement age and it is probably highly variable along the alignment. In general however, pavement distress appears excessive for its apparent age, and this suggests under-design. It is likely that the design did not properly account for traffic volumes along this alignment.

In considering a grind-inlay option, we consider condition and thickness of current paving materials. The first zone in the above table disqualifies for grind-inlay based on pavement condition and section thickness. The next three zones disqualify based on inadequate thickness of high-quality, angular base rock. East of Sta. 41+50 could be considered for a grind-inlay option, provided enough of the distressed AC is removed and desired grades allow for the design thickness required (see *Pavements* section).

CONCLUSIONS AND RECOMMENDATIONS SUMMARY

Based on our explorations, testing, and analyses, it is our opinion that the site is suitable for the proposed redevelopment provided the recommendations in this report are incorporated in design and construction. We offer the following summary of findings and conclusions. The following report sections present our recommendations in greater detail.

- The recommended asphalt pavement section is 7 inches AC over 12 inches of aggregate base. We recommend a geotextile separator, placed below the aggregate base. Grind/inlay may be used for an eastern segment of the Water Avenue alignment. Concrete pavers can be used on the west end. For paved pathways, a lighter pavement section will be appropriate, and concrete surfacing can be used provided our subgrade support recommendations are followed.
- In our opinion, the site is marginal for on-site infiltration of stormwater within fill soils in the upper park. Our measured infiltration rate of 0.4 in/hr may not be sufficient, depending on the size of the facility. If used in design of on-site infiltration facilities, note that the presented value is unfactored and appropriate safety factors will be required. A minimum safety factor of 2 is necessary, but additional safety factors may be applicable depending on the facility design method, the potential for long-term siltation, and other factors.
- Because the silty soils are moisture sensitive, we recommend scheduling the work for dry-season construction.
- The wetland-boardwalk area is underlain by very loose and very soft soils. We recommend deep support of boardwalk foundations, preferably with helical piles.
- For any future new or replacement segments of the Dave Clark Trail, we recommend further setback from the steep riverbank, in accordance with the recommendations following.

RECOMMENDATIONS

PAVEMENTS

Water Avenue

We understand all roadway pavements will be asphalt with the possible exception of the western three blocks (Sta. 6+80 to 17+00), which is being considered for pavers.

Jurisdictional Guidelines. The City of Albany requirements for minimum sections and pavement design are detailed in their document, *Division D - Street and Alley Engineering Standards* (City of Albany, 2019). They provide a minimum section, which for Collector streets is 7” AC over 12” CRB (crushed rock base). The document additionally requires a geotextile separator below the CRB. The document notes that pavement design calculations are required, but only used to check whether the required section (7”/12”) is sufficient (only to see whether additional thickness is necessary, with reductions not allowed). The Albany Standard also requires, “Design of the A.C. pavement structural section shall follow the latest edition of Asphalt Pavement Asssociation of Oregon (APAO) *Asphalt Pavement Design Guide*.”

Design. We have prepared flexible pavement design recommendations in accordance with the APAO document (APAO, 2003). We also performed calculations for ESALs utilizing methods presented in *AASHTO Guide for Design of Pavement Structures* (AASHTO, 1993).

Our recommendations are based on a 20-year performance period with 90 percent reliability. Inputs for design include our pavement condition survey, pavement corings to determine current pavement section, borings to assess subgrade soils, and subgrade DCP testing. The shallow-soil conditions along the 14-block-long alignment are relatively consistent, and hence we provide a single AC pavement section for the entire project.

Subgrade Soil: Soil borings indicate that pavement subgrade soil consists generally of medium stiff to stiff silt along the entire alignment. DCP testing indicates the resilient modulus is about 4,800 to 6,300 psi which corresponds to a California Bearing Ratio (CBR) of 3.2 to 4.2 percent. We have used a resilient modulus of 5,000 psi which corresponds to ‘fair’ soil as defined in the APAO Guide. The design of the recommended pavement section is based on the assumption that construction will be completed during an extended period of dry weather.

Traffic: The City has confirmed that site-specific traffic counts are unavailable for this roadway but that it should be classified as a “Collector”. We estimated traffic loading using guidance in the *Asphalt Pavement Design Guide* (APAO, 2003). Table 3.1 of this reference provides six traffic classes from ‘Very Light’ (Level I) to ‘Heavy’ (Level VI). The Collector classification points to Level V which is ‘high-moderate’ traffic. The table lists the 20-year Equivalent Axle Loads (EAL) as varying from 250,000 to 500,000. We selected the higher value (500,000 EAL) as our design input for the AASHTO methodology. This conservatism accounts for the possibility of higher truck percentages than assumed in the table, appropriate for the high proportion of bus traffic this roadway might experience.

As a reality check on this design-life EAL of 500,000, we calculated EAL’s using the AASHTO method (AASHTO, 1993). For a collector roadway, we have assumed current traffic of 4,000 vehicles per day and a truck percentage of 4%. To project traffic over the design period, we have assumed an annual traffic growth rate of 3.0 percent. Our breakdown of truck traffic for the 160 daily trucks uses typical percentages as follows: 2-axle (118), 3-axle (7), 4-axle (6), 5-axle (14), and bus (15). With these data and assumptions, we calculated an equivalent of roughly 487,000 Equivalent Single Axle Loads (ESALs) over a 20-year period, very similar to our design value of 500,000 selected from the APAO Guide.

Design Calculations: Using the AASHTO method, we make the following additional assumptions:

- Performance period: 20 years
- Reliability: 90%
- Standard Deviation: 0.45
- Initial to Terminal Serviceability: 4.2 to 2.5
- Layer coefficients of 0.06 for angular, high-quality aggregate and 0.42 for asphalt.

Our resulting required structural number is 3.62. Using a 12-inch thickness of base aggregate as recommended in the APAO Guide, 7 inches of asphalt will be required. We recommend an asphalt concrete (AC) pavement section that consists of **7 inches of AC over 12 inches of aggregate base**. This corresponds exactly with the minimum requirements of the City of Albany. Our calculations have confirmed the appropriateness of their pavement section and determined that additional thickness should not be necessary.

Geotextile: We concur with the City’s recommendation for use of a geotextile separator. This product is intended only for separation from the silty subgrade soils and does not contribute to the structural capacity of the pavement section. Therefore, the material can be a relatively lightweight non-woven geotextile such as Mirafi 140N. The material should conform to ODOT Std Specification 2320.2 – *Type 1 Nonwoven Drainage Geotextile* (ODOT, 2021). Rolls should be placed with minimum 6-inch overlap.

Grind/Inlay: The existing pavement east of Sta. 41+50 is appropriate for modification by grinding and inlaying. Assuming a **2-inch grind** with 2.5 inches existing AC remaining, 4.5 inches of new AC would be required to obtain the 7" AC thickness. The resulting increase in elevation would be 2.5 inches. If grades can be raised in this area, the method could provide significant cost savings. We have again used the AASHTO method to confirm the need for 7" of asphalt, using a layer coefficient of 0.35 for existing AC and assuming 15-inch thick existing aggregate base. We do not recommend an overlay without grinding because in our opinion, the distressed upper-layer of pavement should be removed.

Parking lots and Pull-Ins: For parking pull-ins and parking lots, we assume traffic will be lower-volume with a lower truck percentage. Concrete (PCC) pavements may also be allowed for pull-ins. We have prepared AC and PCC pavement recommendations based on 150,000 ESALs with a required structural number of 2.77. We recommend a concrete section of **5 inches of PCC over 4 inches of aggregate base** or an asphalt section of **5.5 inches AC over 8 inches of aggregate base.** The concrete design was prepared in accordance with *Guide for Design and Construction of Concrete Parking Lots* (ACI, 2008).

The AC pavement should conform to City of Albany requirements which are ½-inch dense graded HMAC (hot-mix asphalt) for upper 2 inches and ¾-inch HMAC for base courses (City of Albany, 2019). Asphalt should be compacted to 91 percent of Rice density. Our recommendations for base-course aggregate is provided below under *Fill and Backfill Materials*. Aggregate base should extend a minimum of 6 inches beyond the edge of the AC. We do not recommend planning on the re-use of existing base rock as part of the structural design section. However, in areas where the excavation for new pavement subgrade does not remove all existing rock, the rock may remain. Although this is mostly described as low-quality aggregate (see boring logs), the strength/deflection behavior is still superior to existing subgrade silt soils, so it can remain as subgrade soil.

The PCC used to construct the recommended rigid pavement section should have a minimum 28-day flexural strength of not less than 600 psi as determined by ASTM C 78. Typically, concrete with a compressive strength of at least 4,000 psi will achieve the above recommended flexural strength.

Concrete Pathways

Pavements for site pathways and sidewalks will consist of Portland Cement Concrete (PCC). Aggregate base material for PCC pathways should conform to the recommendations below in the report section, *Fill and Backfill Materials*.

We assume pathways will be subject to traffic primarily from pedestrians but may also occasional be subject to loading from maintenance trucks.

For design, we utilized the American Concrete Institute manual 330R-08, *Guide for Design and Construction of Concrete Parking Lots* (ACI, 2008) in conjunction with results from exploratory borings and our experience. The recommended pavement section for the anticipated light traffic is **4 inches of PCC over 8 inches of aggregate base**. An appropriate alternative flexible pavement section for pathways is 3 inches of AC over 12 inches of aggregate base.

The recommendation is for unreinforced concrete, sometimes referred to as jointed-plain-concrete pavement (JPCP). The PCC used to construct the recommended rigid pavement section should have a minimum 28-day flexural strength of not less than 600 psi as determined by ASTM C 78. Typically, concrete with a compressive strength of at least 4,000 psi will achieve the above recommended flexural strength. For control of shrinkage cracks, spacing of contraction joints should be a maximum of 10 feet.

SLOPE & TRAIL

We performed a visual reconnaissance of the Dave Clark Trail which parallels the riverfront slope, in some areas coming within a few feet of the slope crest. We evaluated the current condition of the trail surface and adjacent slope. The reconnaissance extended from approximately Washington Street to Main Street.

Appendix C contains 16 photographs illustrating conditions of the trail and surroundings, presented from west to east. Distress features observed related to slope movement include concrete cracking parallel to the slope, concrete slab segments tilting towards the slope, and undermining of exposed concrete-edge at the slope face. We additionally observed numerous features on the slopes suggesting past slope movement, generally slope creep (slow, shallow movement as opposed to deeper landsliding). These include tilting and curved tree trunks, tilting and distressed structures such as abandoned posts and foundations, and offset drain pipe connections. The eastern half of the 14-block segment is generally in much better condition than the western half. The eastern portions are newer and there are several segments that provide structural support to the trail (piles and retaining walls).

In general, the trail is located closer than optimal from the crest of a potentially unstable slope. In addition to the visual evidence of past movement, the slope height (20 to 30 ft) and inclination (average of approx. 1.5H:1V), as well as the likely soil conditions (undocumented fill or unconsolidated native alluvium) are indications of potential instability.

We understand the trail will remain as-is, without any realignment planned at this time. The trail is functional and the risk of significant and abrupt landsliding is relatively low. However, if any significant trail segments will be replaced, we would recommend locating these a minimum distance of 10 feet from the slope crest to the nearest edge of concrete. This setback will minimize future trail surface distress caused by slope movements. Wherever setbacks are unachievable, retaining walls or boardwalk-segments should be considered. Design of such structures should be overseen by a geotechnical engineer.

BOARDWALK

The proposed boardwalk will span across an area of very loose and very soft soils (see discussion in report section *Encountered Subsurface Conditions*). These soils are highly compressible. In order to limit settlement, provide some lateral restraint, and prevent scour loss during floodwater flow, we recommend against the use of shallow foundations. Helical piles will be an appropriate and cost-effective alternative deep-foundation option for the boardwalk. We recommend helical piles bearing in the soils between 10 and 15 feet below grade, applicable to the entire length of boardwalk shown on Figure 2a, approximately 65-ft in length.

Axial Load

Design loading is unknown at this time. To cover a range of conditions, we have calculated required helix configurations of round-shaft piles for three design loads (single-pile allowable loads) that might be appropriate for this project. As shown in the following table, lead-sections with a single-helix are appropriate for the lighter loads. For axial loads over 4 kips, double-helix configurations should be considered.

Helical Anchor Design Recommendations

Allowable Load (kips)	Helix Configuration	Required Torque (ft-lb), 2-⅞" diam.	Required Torque (ft-lb), 3" diam.
2	8"	450	500
3	10"	670	750
5	8"/10"	1,120	1,250

In the field, capacity is verified by torque measurements with equipment provided by the contractor. The table above provides the required minimum torque for each of the selected piles, based on a factor of safety of 2.0 applied to the calculated capacity.

These recommendations assume minimum shaft outside-diameter is 2-⅞ or 3 inches. Square shafts should not be used. If a diameter greater than 3 inches is selected, we should be consulted for modification of the above torque and load capacity values. Wall thickness of the circular section should be 0.25-inch minimum.

To achieve bearing in the denser deposits and to establish depth of fixity for lateral constraint, we have established a recommended minimum pile depth of 10 feet. Even if required torque is achieved above this depth, the installation should be continued to the required minimum depth.

Based on results of the field exploration program, our estimated helical pile depth to achieve required torque is 13 ft. This estimate is solely for cost estimating purposes and the contractor should be prepared to add additional shaft extensions as necessary.

Lateral Load

We have recommended circular shaft helical piles which will provide greater stiffness and lateral resistance than equivalent-size square-shaft piles. We have reviewed a paper by Howard Perko (2003) which presents the findings of lateral load tests performed on 3-inch diameter helical piles in a variety of soil types. Perko shows that even for very loose sands and soft clays, such piles can be loaded laterally to over 1,000 lb with less than 0.5 inch deflection. We have confirmed this in the past with lateral loading analyses (LPile) on several projects. In our opinion, lateral loads will be adequately resisted by the circular-shaft helical piles with acceptable levels of deflection. However, if it is determined that additional lateral resistance is required, battered helical piles could be installed.

OTHER STRUCTURES

Possible small structures include signage, playground equipment, and seating walls. These relatively low ground pressure structures can be placed on a minimum 6-inch layer of compacted crushed rock (see Aggregate Base below) over the prepared subgrade as described herein.

The stage structure might be modified and this work may include additional foundation support. Subgrade soils in this area are expected to be fill soils consisting generally of medium dense silty gravel. Foundation support elements for the stage can consist of shallow continuous or isolated concrete footings. Due to the fill, we recommend overexcavation of foundation excavations by 12 inches and replacement with compacted crushed rock. Structure foundations can be proportioned using a maximum allowable bearing pressure of 2,500 psf. Such foundations are expected to experience settlements of less than ¾-inch.

Lateral loads on footings can be resisted by passive earth pressure on the sides of footings and by friction on the bearing surface. We recommend that passive earth pressures be calculated using an equivalent unit weight of 280 pounds per cubic foot (pcf) for foundations confined by medium stiff or better native soils or compacted imported granular fill. We recommend using a friction coefficient of 0.35 for foundations placed directly on site soils. The passive earth pressure and friction components may be combined provided that the passive component does not exceed two-thirds of the total.

Foundations for light poles should consist of reinforced concrete piers. These are typically 24 inches in diameter and 4 to 6 feet in depth, depending on the pole height. Standard foundation details, provided by the manufacturer will likely provide adequate support at this site. Geotechnics should review the preliminary plans to verify adequate lateral and vertical support.

EARTHWORKS

Site and Subgrade Preparation

Existing site vegetation including roots should be removed from all work areas. Stripped material should be transported off site for disposal or placed in stable, non-settlement-sensitive areas. Grubbing should include removal of all trees, brush and their trunks within structure and pavement areas. Roots up to 1 inch in diameter should also be grubbed from such areas. Low or disturbed areas from grubbing should be backfilled and compacted with structural fill as described later in this report.

After stripping and grubbing, the existing subgrade of pavements, walkways, or areas to receive structural fill soil should be proofrolled with a loaded dump truck or heavy drum roller to identify remaining soft, loose or unsuitable areas. The proofrolling should be observed by a member of our staff, who should evaluate the suitability of the subgrade and identify any areas of yielding that are indicative of soft or loose soil. If soft or loose zones are identified during proofrolling, these areas should be excavated to the extent indicated by the geotechnical engineer and replaced with compacted structural fill. For roadway pavements, if work is performed during wet subgrade conditions and/or widespread excessive deflection is noted during proofrolling, we may recommend placement of a higher-strength woven geotextile or geogrid on the soil subgrade.

Foundation Subgrade

We recommend that Geotechnics observe the base of prepared foundation excavations before placing any concrete forms, reinforcing steel, and/or replacement crushed rock. 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. We will evaluate whether the bearing surface has been adequately prepared and that the soil conditions are consistent with those observed during our explorations.

Dry Weather Construction

The silty soils at the site can be expected to become disturbed during periods of wet weather or when the moisture content of the material is more than a few percentage points above optimum. This will likely be the case in all but mid-summer through early fall. When wet, the on-site soils are susceptible to disturbance and generally will provide inadequate support for construction equipment.

We recommend earthwork be scheduled for the dry summer months. As noted above, our recommendations for flexible pavement design are contingent on dry-weather construction and the resultant ability to adequately prepare the subgrade soils. If earthwork is scheduled for the wet season or significant precipitation occurs during construction, special techniques may be needed to minimize disturbance to the subgrade from construction traffic. This could include constructing a temporary working pad of 12 to 18 inches of crushed rock over a geotextile fabric. Tracked equipment can be used to reduce loading on the subgrade. Construction access and staging can be planned to reduce traffic over soft subgrade areas.

Utility Trenches

We assume the project will include the placement of utilities in trenches. Lateral support should be provided to prevent loss of ground support. Excavations deeper than 4 feet bgs should be shored or sloped if workers are required to enter. Excavations made to construct footings or other structural elements should be laid back at the surface as necessary to prevent soil from falling into excavations. All excavations should be made in accordance with applicable Occupational Safety and Health Administration (OSHA) and state regulations. Site soils are generally OSHA Type B.

The contractor should be responsible for reviewing the boring logs, selecting and designing the specific shoring methods, monitoring the excavations for safety, and providing shoring required to protect personnel and adjacent structural elements. Shoring deeper than 6 feet should be designed by a registered engineer who should be provided with a copy of this report. Shoring should be designed and constructed to support an equivalent fluid pressure of 40 pcf, plus surcharge loads from construction equipment, construction materials, excavated soils, or vehicular traffic.

The majority of soils encountered should be suitable for support of utility pipes. Pipe bedding materials should be placed on relatively undisturbed soils. Trench bottoms should be free of debris, organics, and standing water. If subgrade soils are very loose or disturbed, the soils should be compacted in place, or removed and replaced with compacted bedding material or larger aggregate.

We recommend a minimum 4-inch thickness of bedding material beneath pipes. Bedding material should be used as pipe zone backfill and placed in layers and compacted around the pipe to obtain complete contact. Bedding material should extend at least 12 inches above the top of the pipe. Pipe bedding material, placement, compaction, and shaping should be in accordance with the manufacturer's specifications.

During the dry season, groundwater is not likely to occur within the depths of expected excavations. During the wet season, however, perched groundwater could rise to within excavation depths. If groundwater is encountered, sump pumps placed in the excavations should be sufficient for dewatering. In addition to groundwater seepage, surface water inflow to the excavations during the wet season could be problematic. In addition to groundwater seepage, surface water inflow to the excavations during the wet season could be problematic. Provisions for temporary surface water control should be included in the project plans and should be installed prior to commencing work (see below).

Surface Drainage

Temporary: Surface runoff can be controlled during construction by careful grading practices. Typically, these include the construction of shallow, upgrade perimeter ditches or low earthen berms and the use of temporary sumps to collect runoff and prevent water from damaging exposed subgrades. Also, measures should be taken to avoid ponding of surface water during construction.

Erosion at the site during construction can be minimized by judicious use of straw bales, silt fences and plastic sheets. The erosion control devices should be in place and remain in place throughout site preparation and construction. Maintaining appropriate erosion control is the responsibility of the contractor and should be carried out in accordance with the project plans and specifications and applicable regulations.

Permanent: A well-designed permanent surface water control plan should be included in the design documents. Adequate surface gradients and drainage systems should be incorporated into the design such that roof drains and parking lot runoff are directed away from structures and into swales, pipes, or other controlled drainage devices that discharge to a suitable outlet.

Fill and Backfill Materials

Fill beneath pathways and other structures should be placed and compacted as structural fill. Any fill placed on or at slopes steeper than 5H:1V should also be constructed as structural fill. Following are recommendations for structural fill. On-site soils, placed during dry weather, may be suitable for use as structural fill provided debris, organics, and oversized particles are removed, as described below. A Geotechnics representative should evaluate on-site and imported fill materials prior to use at the site.

General Structural Fill: Structural fill soils should be free of debris, roots, organic matter, frozen soil, man-made contaminants, particles with greatest dimension exceeding 3 inches, and other deleterious materials. The suitability of soil for use as structural fill will also 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. If the soil is too wet to achieve satisfactory compaction, moisture conditioning such as disking or tilling will be required. If the material cannot be properly moisture conditioned, we recommend using imported material for structural fill.

Select imported 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. The material should have less than 5 percent passing the U.S. No. 200 Sieve, but during dry weather the fines content can be increased to a maximum of 20 percent. The material should have a maximum particle size of 3 inches.

Aggregate Base: This rock product should be used under roadway and pathway pavements. Aggregate base should also be used for backfill of overexcavated zones beneath foundations, roadways, and pathways. The material should consist of imported clean, durable, crushed angular rock. Such rock should be well-graded and have a maximum particle size of 1½ inches, and less than 5 percent passing the U.S. No. 200 Sieve. The material should additionally conform to Section 2630.10 of the ODOT Standard Specifications for Construction (ODOT, 2021) for 1½"-minus dense-graded base aggregate.

<u>Trench Backfill:</u> Utility trench 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 8 percent passing the U.S. No. 200 Sieve. The pipe bedding and fill in the pipe zone should meet the pipe manufacturer's recommendations. Above the pipe zone, imported granular fill or aggregate base may be used as described above.

Fill Placement and Compaction

Structural fill material should be placed and compacted in thin lifts to the percentage of Maximum Dry Density (MDD) as listed below. MDD is based on ASTM Test Method D1557 (Modified Proctor).

Mass Fill (imported): 92
Mass Fill (site soils): 92

Pavement Aggregate Base: 95
Trench Backfill: 92
Nonstructural Trench Backfill: 88

Structural fill should be placed and compacted in lifts in accordance with the following:

- Place all fill and backfill on a prepared subgrade that consists of firm, inorganic native soils or approved fill soils. When placed on sloping ground, the ground should be benched and keyed such that soils are placed on a level surface.
- Place all fill or backfill in uniform horizontal lifts with a thickness appropriate for the material type and compaction equipment. Unless otherwise directed by the geotechnical engineer, maximum thickness of loose lifts shall be 8 inches.
- Place fill at a moisture content within about 3 percent of optimum as determined in accordance with ASTM Test Method D1557. Moisture condition fill soil to achieve a uniform moisture content within the specified range before compacting.
- Do not place fill and backfill until tests and evaluation of the underlying materials have been made and the appropriate approvals have been obtained.
- Grade the surface of the fill at the end of each working shift so that surface water can drain readily.

During structural fill placement and compaction, a sufficient number of in-place density tests should be completed to verify that the specified degree of compaction is being achieved.

DOCUMENT REVIEW AND CONSTRUCTION SUPPORT

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 exploration program. Recognition of changed conditions often requires experience; therefore, the project geotechnical engineer or their representative should visit the site with sufficient frequency to detect whether subsurface conditions change significantly from those anticipated. Geotechnics should also review the final plans and specifications to verify that the recommendations presented herein have been interpreted as intended.

LIMITATIONS

We have prepared this report for the exclusive use of the City of Albany and the Walker Macy design team. Our report is intended to provide our opinion of geotechnical parameters for design and construction of the proposed project based on exploration locations that are believed to be representative of site conditions. However, conditions can vary significantly between exploration locations and our conclusions should not be construed as a warranty or guarantee of subsurface conditions or future site performance. If soil conditions are encountered during construction that differ from those described herein, we should be notified immediately to assess the implications and provide any necessary design supplements or modifications. If the scope of proposed construction, including the structure locations, changes from that described herein, our recommendations should also be reviewed.

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

* * *

We appreciate the opportunity to submit this report. Please contact us if you have any questions or need additional information.

Sincerely,

REGISTERED PROFESSIONAL ENGINEER
19463
OREGON
SEPT. 16, 1997
ANDRÉ DAVID MARÉ
EXPIRES: 12/31/22

André D. Maré, P.E., G.E.
Geotechnical Engineer

Document ID: Albany-Geotech.docx

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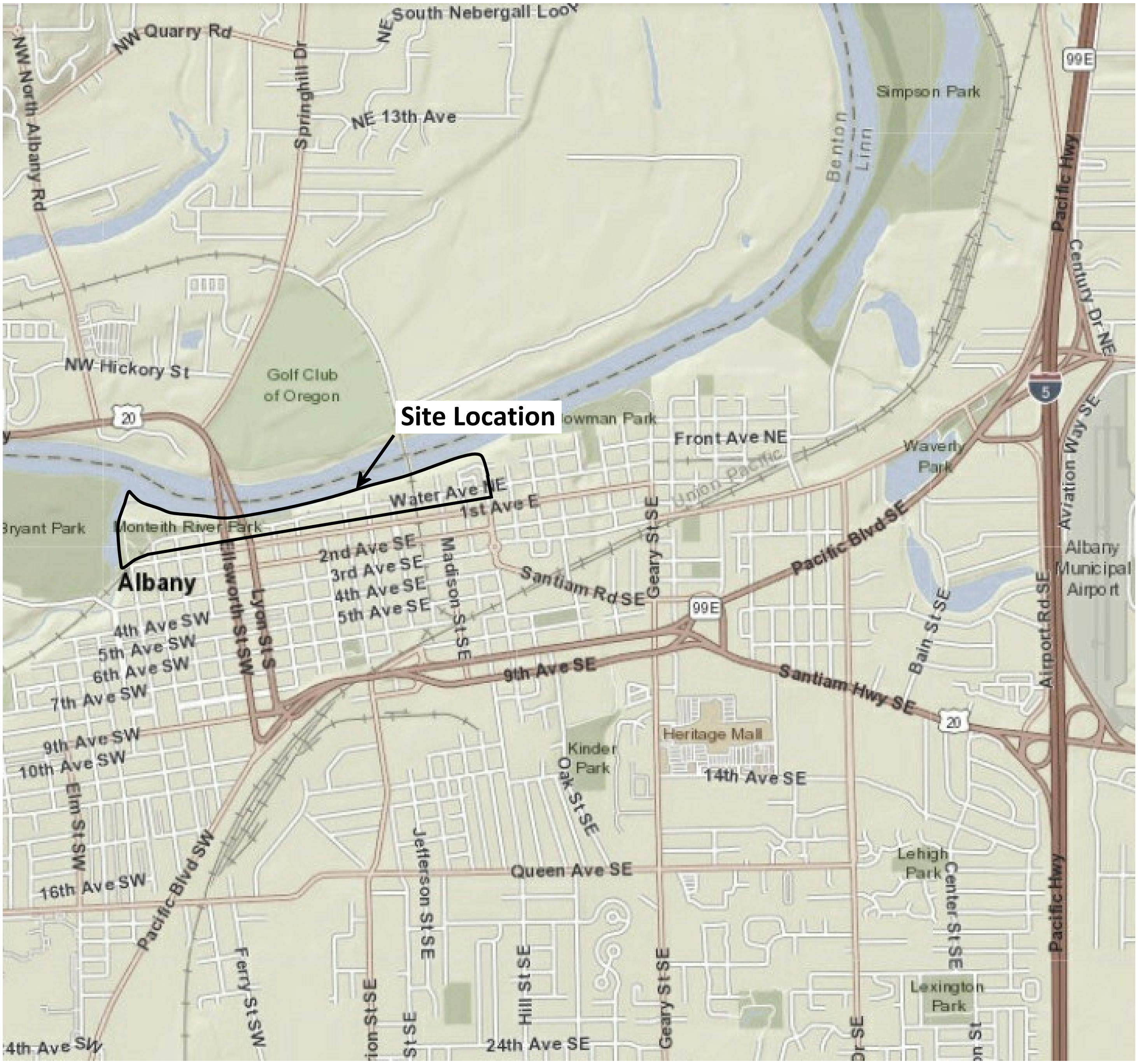


N

NOT TO SCALE

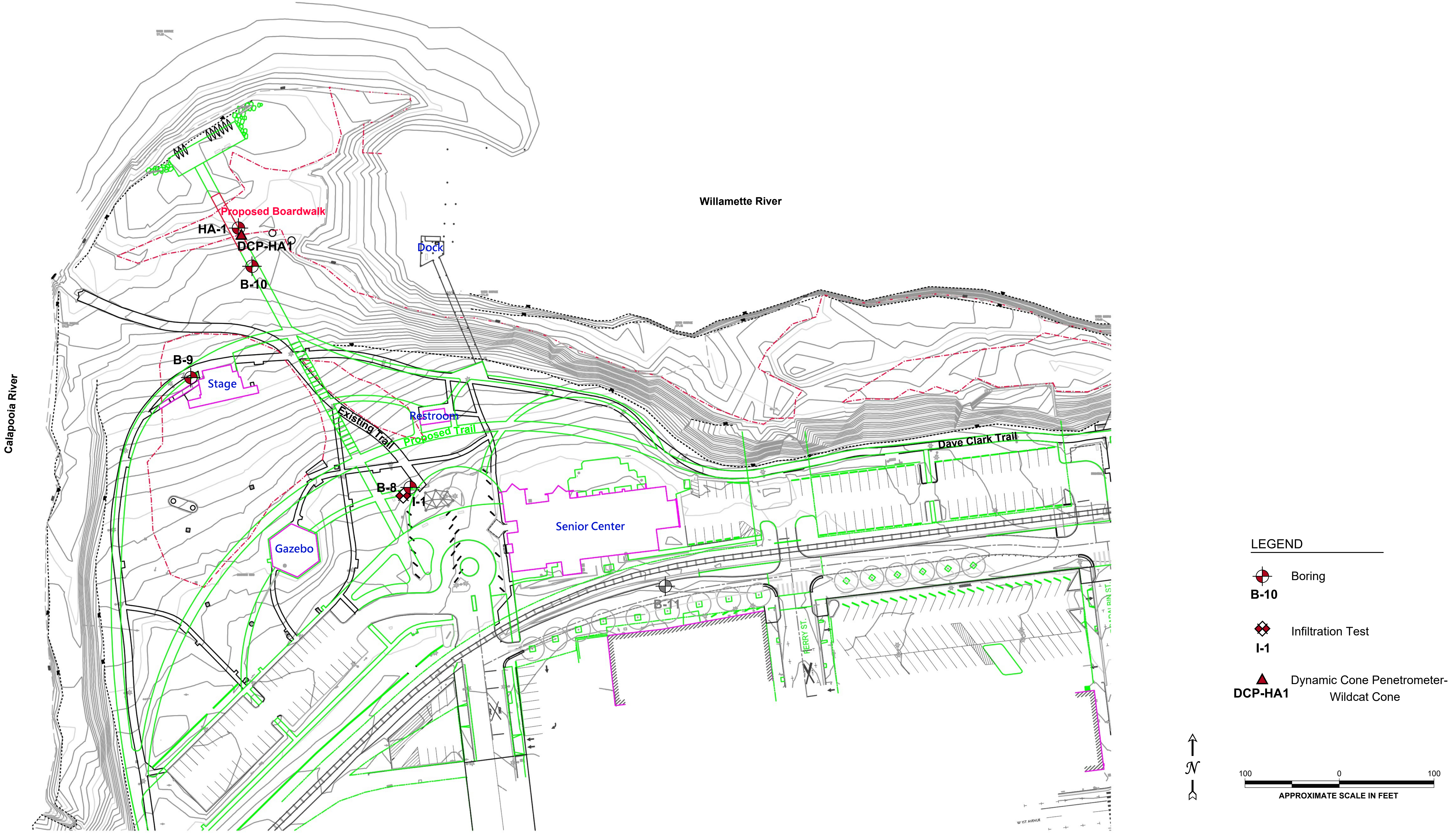
GEOTECHNICS

VICINITY MAP

Albany Waterfront Redevelopment
Albany, Oregon

Project No. 19-008-1

Figure 1

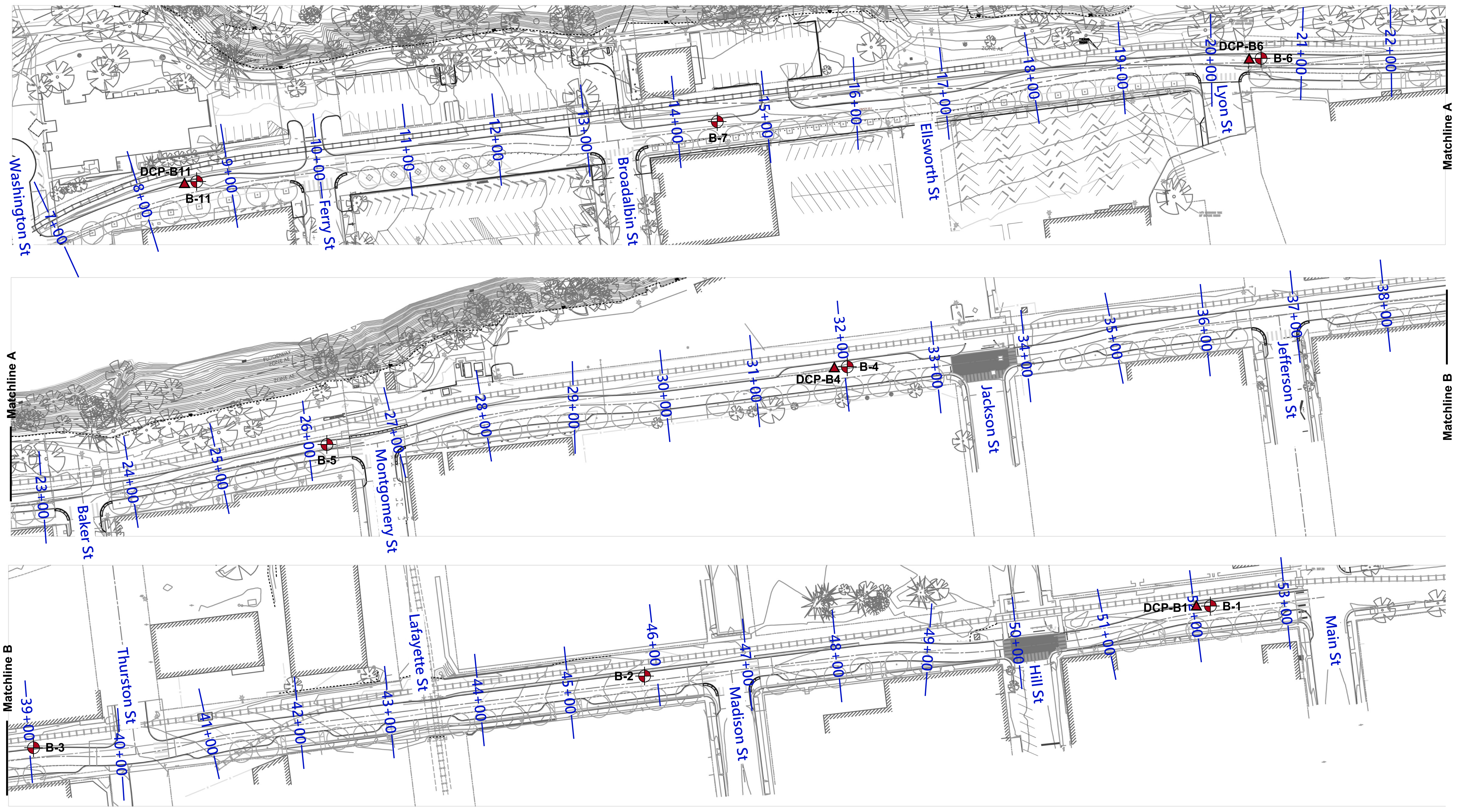


SITE & EXPLORATION PLAN - PARK

Albany Waterfront Redevelopment
Albany, Oregon

Project No. 19-008-1

Figure 2a



N

100 0 100

APPROXIMATE SCALE IN FEET

LEGEND

⊕ B-11	Boring
▲ DCP-B4	Dynamic Cone Penetrometer

GEOTECHNICS

SITE & EXPLORATION PLAN - PAVEMENTS

Albany Waterfront Redevelopment
Albany, Oregon

Project No. 19-008-1

Figure 2b

Appendix A

FIELD EXPLORATIONS AND LABORATORY TESTING

RELATIVE DENSITY OR CONSISTENCY VERSUS SPT N-VALUE

COHESIONLESS SOILS			COHESIVE SOILS		
Density	N (blows/ft)	Approximate Relative Density (%)	Consistency	N (blows/ft)	Approximate Undrained Shear Strength (psf)
Very Loose	0 to 4	0 to 15	Very Soft	0 to 2	<250
Loose	4 to 10	15 to 35	Soft	2 to 4	250 - 500
Medium Dense	10 to 30	35 to 65	Medium Stiff	4 to 8	500 - 1000
Dense	30 to 50	65 to 85	Stiff	8 to 15	1000 - 2000
Very Dense	over 50	85 to 100	Very Stiff	15 to 30	2000 - 4000
			Hard	over 30	>4000

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP DESCRIPTIONS		
Coarse Grained Soils More than 50% Retained on No. 200 Sieve Size	**Gravel and Gravelly Soils** More than 50% of Coarse Fraction Retained on No. 4 Sieve	Clean Gravel (little or no fines)	▨	GW	Well-graded GRAVEL
			▨	GP	Poorly-graded GRAVEL
		Gravel with Fines (appreciable amount of fines)	▨	GM	Silty GRAVEL
			▨	GC	Clayey GRAVEL
	Sand and Sandy Soils 50% or More of Coarse Fraction Passing No. 4 Sieve	Clean Sand (little or no fines)	▨	SW	Well-graded SAND
			▨	SP	Poorly-graded SAND
		Sand with Fines (appreciable amount of fines)	▨	SM	Silty SAND
			▨	SC	Clayey SAND
Fine Grained Soils 50% or More Passing No. 200 Sieve Size	**Silt and Clay**	Liquid Limit Less than 50%	▨	ML	SILT
			▨	ML	Sandy SILT
			▨	CL	Lean CLAY
			▨	CL	Sandy CLAY
	Silt and Clay	Liquid Limit 50% or More	▨	MH	Elastic SILT
			▨	CH	Fat CLAY

COMPONENT DEFINITIONS

COMPONENT	SIZE RANGE
Boulders	Larger than 12 in
Cobbles	3 in to 12 in
Gravel	3 in to #4 (5 mm)
Coarse Gravel	3 in to ¾ in
Fine Gravel	¾ in to #4 (5 mm)
Sand	#4 (5 mm) to #200 (0.075 mm)
Coarse Sand	#4 (5 mm) to #10 (2 mm)
Medium Sand	#10 (2 mm) to #40 (0.4 mm)
Fine Sand	#40 (0.4 mm) to #200 (0.075 mm)
Silt and Clay	Smaller than #200 (0.075 mm)

ABBREVIATIONS

Laboratory Tests:

AL	Atterberg Limits
PL	Plastic Limit
LL	Liquid Limit
%F	Fines Content
GSD	Grain Size Distribution
DD	Dry Density
MD	Moisture/Density Relationship
-S	Standard Proctor (ASTM D-698)
-M	Modified Proctor (ASTM D-1557)
SG	Specific Gravity
CBR	California Bearing Ratio
RM	Resilient Modulus
K	Permeability
CN	Consolidation
DS	Direct Shear
TX	Triaxial Shear
-UU	Unconsolidated Undrained
-CU	Consolidated Undrained

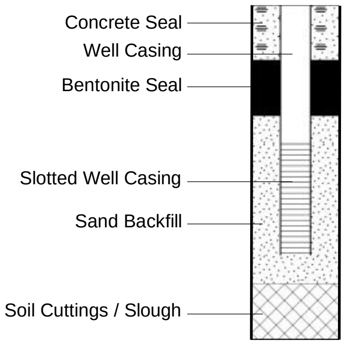
Field Tests:

PP	Pocket Penetrometer
TV	Torvane

Sample Type:

SPT	Standard Penetration Test (2.0" OD)
D&M	Ring Sampler (3.25" OD)
C-MOD	California Modified Sampler (3.0" OD)
SH	Thin-Walled Shelby Tube (3.0" OD)
GRAB	Disturbed Sample collected from auger cuttings or test pit

WELL DETAIL



NOTES

Soil descriptions are based on the general approach presented in ASTM D-2488 (Visual-Manual Procedure). Where laboratory data are available, soil classifications are in accordance with ASTM D-2487.

Solid lines between soil unit descriptions indicate change in interpreted geologic unit. Dashed lines indicate stratigraphic change within the geologic unit.

Blowcount (N) is recorded for driven samplers as the number of blows required to advance sampler 12 inches (or distance noted) per ASTM D-1586. See exploration log for hammer weight and drop.

Please also refer to the discussion in the report for a general description of subsurface conditions.

GEOTECHNICS

KEY TO LOG SYMBOLS AND TERMS

Albany Waterfront Redevelopment
Albany, Oregon

Client: Walker Macy

Project Number: 19-008-1

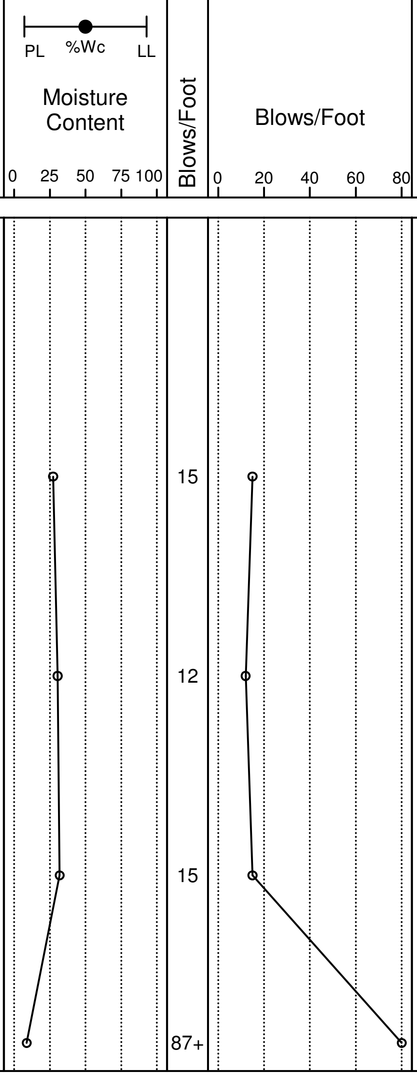
LOG OF BORING B-1

Surface Elevation:	208.0 feet
Northing:	365,528
Easting:	7,527,317
Coordinate System:	OR State Plane North, NAD 83
Start Date:	October 12, 2020
End Date:	October 12, 2020
Logged By:	ADM
Contractor:	Dan Fischer Excavating, Inc.
Drilling Method:	Solid-stem auger
Drilling Equipment:	Trailer-mount
Hammer Weight:	140 lb.
Hammer Drop:	30 in.

SAMPLE TYPE

- ◪ SPT Standard Penetration Test
- ◧ CMOD California Modified Split-Barrel
- ▥ SHELBY Thin-Walled Tube - 3"
- ☒ GRAB Bag or Bucket

Depth in Feet	Sample Type	Sample Number	USCS	MATERIAL DESCRIPTION	Water Level, ATD	Moisture Content, %	Blows/Foot	Other Tests
0	☒	S-1	GP	4.5 inches Asphalt over: 23 inches high quality CRB (crushed-rock base).				
	◪	S-2	ML	Gray SILT, moist, stiff. Low plasticity. (FINE ALLUVIUM)		27.4	15	
5	◪	S-3	ML	@4.5', becomes non-plastic, micaceous.		30.5	12	
	◪	S-4	ML	@7.5', becomes gray mottled reddish brown, low plasticity, no mica.		31.8	15	
10	◪	S-5	GP-GM	Brown mottled light yellowish brown, gray, and black, Poorly Graded GRAVEL with Silt and Sand, moist, very dense. (COARSE ALLUVIUM)		8.9	87+	
15								



Total Depth = 10.7'.
No Groundwater Encountered.

Adjacent Dynamic Cone Penetrometer

GEOTECHNICS

Boring B-1

Albany Waterfront Redevelopment
Albany, Oregon

Sta. 52+17

Figure A1

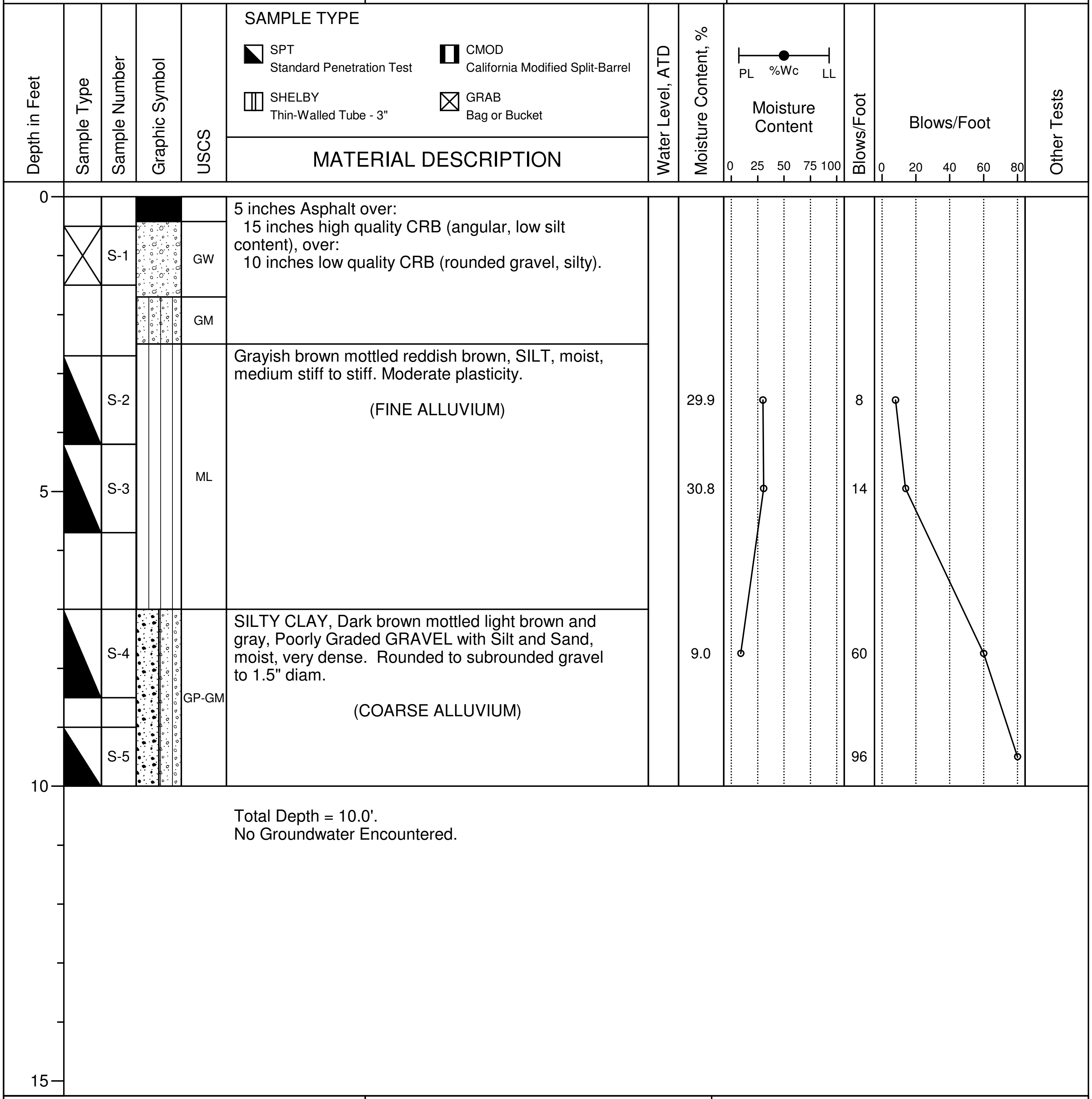
Albany Waterfront Redevelopment
Albany, Oregon

Client: Walker Macy

Project Number: 19-008-1

LOG OF BORING B-2

Surface Elevation:	205.0 feet	Start Date:	October 12, 2020	Drilling Method:	Solid-stem auger
Northing:	365,452	End Date:	October 12, 2020	Drilling Equipment:	Trailer-mount
Easting:	7,526,692	Logged By:	ADM	Hammer Weight:	140 lb.
Coordinate System:	OR State Plane North, NAD 83	Contractor:	Dan Fischer Excavating, Inc.	Hammer Drop:	30 in.



Depth in Feet	Sample Type	Sample Number	USCS	MATERIAL DESCRIPTION	Water Level, ATD	Moisture Content, %	Blows/Foot	Other Tests
0				5 inches Asphalt over:				
	GRAB	S-1	GW	15 inches high quality CRB (angular, low silt content), over:				
			GM	10 inches low quality CRB (rounded gravel, silty).				
	SPT	S-2	ML	Grayish brown mottled reddish brown, SILT, moist, medium stiff to stiff. Moderate plasticity. (FINE ALLUVIUM)		29.9	8	
5	SPT	S-3				30.8	14	
	SPT	S-4	GP-GM	SILTY CLAY, Dark brown mottled light brown and gray, Poorly Graded GRAVEL with Silt and Sand, moist, very dense. Rounded to subrounded gravel to 1.5" diam. (COARSE ALLUVIUM)		9.0	60	
	SPT	S-5					96	
10								

Total Depth = 10.0'.
No Groundwater Encountered.

GEOTECHNICS

Boring B-2

Albany Waterfront Redevelopment
Albany, Oregon

Sta. 45+86

Figure A2

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Albany Waterfront Redevelopment
Albany, Oregon

Client: Walker Macy

Project Number: 19-008-1

LOG OF BORING B-3

Surface Elevation:	207.0 feet	Start Date:	October 12, 2020	Drilling Method:	Solid-stem auger
Northing:	365,373	End Date:	October 12, 2020	Drilling Equipment:	Trailer-mount
Easting:	7,526,013	Logged By:	ADM	Hammer Weight:	140 lb.
Coordinate System:	OR State Plane North, NAD 83	Contractor:	Dan Fischer Excavating, Inc.	Hammer Drop:	30 in.

SAMPLE TYPE

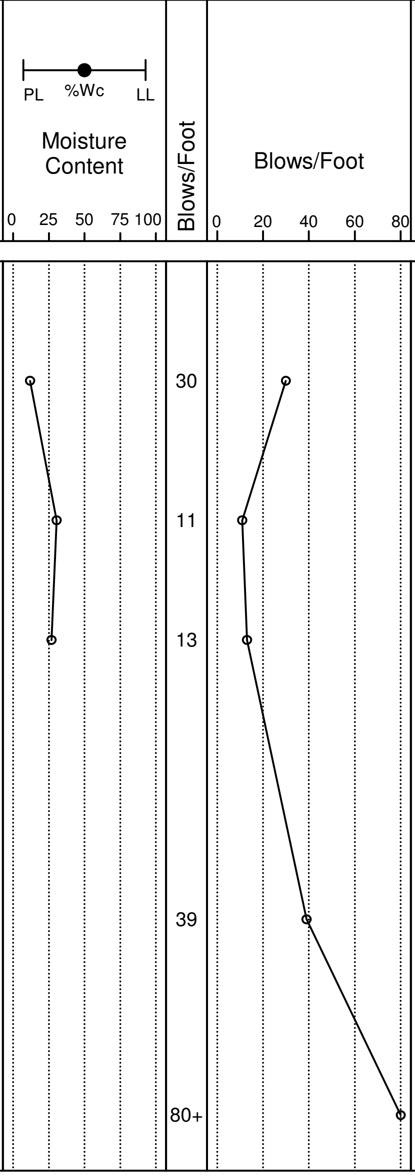
◩ SPT Standard Penetration Test

◨ CMOD California Modified Split-Barrel

▥ SHELBY Thin-Walled Tube - 3"

☒ GRAB Bag or Bucket

Sample Type	Sample Number	USCS	MATERIAL DESCRIPTION	Water Level, ATD	Moisture Content, %	Blows/Foot	Other Tests
			3 inches Asphalt over:				
		GW	6 inches high quality CRB (angular, low silt content), over:				
◩	S-1	GM	18 inches low quality CRB (rounded gravel, silty).		11.9	30	
		ML	Brown SILT, moist, stiff. Moderate plasticity. (FINE ALLUVIUM)				
◩	S-2				30.4	11	
			@4', becomes light brown, low plasticity.				
◩	S-3				26.8	13	
			@7.5', becomes light brown mottled gray and dark brown.				
◩	S-4					39	
		GM	Brown mottled reddish brown and black, Silty GRAVEL, moist, dense. Mostly fine rounded gravel to 0.5" diam. (COARSE ALLUVIUM)				
◩	S-5	SP-ML	Light brown, Poorly Graded SAND (SP) layered with SILT (ML) and minor Poorly Graded GRAVEL (GP), moist, very dense.			80+	



Depth in Feet: 0, 5, 10, 15

Total Depth = 11.4'.
No Groundwater Encountered.

GEOTECHNICS

Boring B-3

Albany Waterfront Redevelopment
Albany, Oregon

Sta. 39+03

Figure A3

Albany Waterfront Redevelopment
Albany, Oregon

Client: Walker Macy

Project Number: 19-008-1

LOG OF BORING B-4

Surface Elevation:	206.0 feet	Start Date:	October 12, 2020	Drilling Method:	Solid-stem auger
Northing:	365,277	End Date:	October 12, 2020	Drilling Equipment:	Trailer-mount
Easting:	7,525,323	Logged By:	ADM	Hammer Weight:	140 lb.
Coordinate System:	OR State Plane North, NAD 83	Contractor:	Dan Fischer Excavating, Inc.	Hammer Drop:	30 in.

SAMPLE TYPE

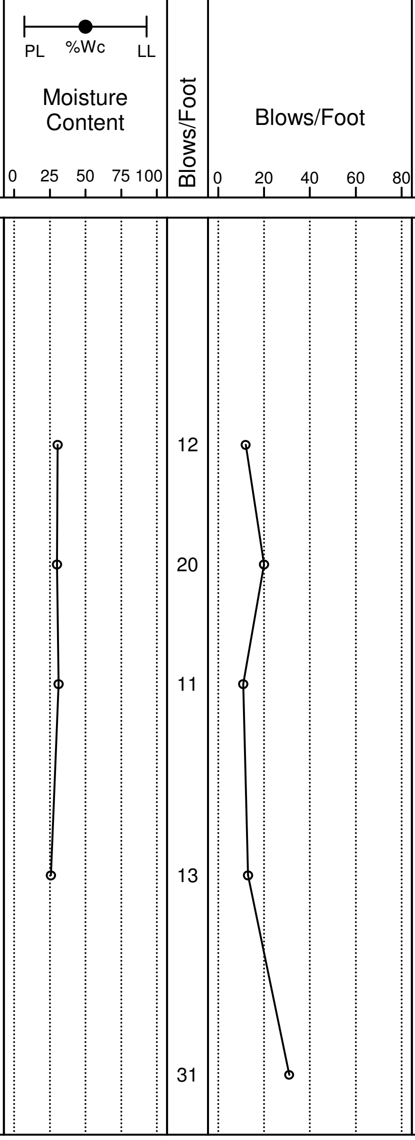
◩ SPT
Standard Penetration Test

▮ CMOD
California Modified Split-Barrel

▥ SHELBY
Thin-Walled Tube - 3"

☒ GRAB
Bag or Bucket

Depth in Feet	Sample Type	Sample Number	Graphic Symbol	USCS	MATERIAL DESCRIPTION	Water Level, ATD	Moisture Content, %	Blows/Foot	Other Tests
0	☒	S-1		GW	2.5 inches Asphalt over: 6.5 inches high quality CRB (angular, low silt content), over: 16 inches low quality CRB (rounded gravel, silty).				
				GM					
	◩	S-2		ML	Grayish brown, SILT, moist, stiff. Low to moderate plasticity. (FINE ALLUVIUM)		30.5	12	
	◩	S-3			@3.6', becomes light brownish gray mottled reddish brown, very stiff.		30.0	20	
5	◩	S-4			@5.1', becomes very moist, stiff.		31.1	11	
	◩	S-5			@7.5', becomes light brown mottled gray, moderate plasticity.		25.6	13	
10	◩	S-6		GM-ML	Gray mottled reddish brown SILT (ML) layered with gray Poorly Graded SAND (SP) and Silty GRAVEL (GM), very moist, dense. Sand is fine to medium grained. Gravel in sample tip only. (COARSE ALLUVIUM)			31	
15									



Total Depth = 11.5'.
No Groundwater Encountered.

Adjacent Dynamic Cone Penetrometer

GEOTECHNICS

Boring B-4

Albany Waterfront Redevelopment
Albany, Oregon

Sta. 32+04

Figure A4

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Albany Waterfront Redevelopment
Albany, Oregon

Client: Walker Macy

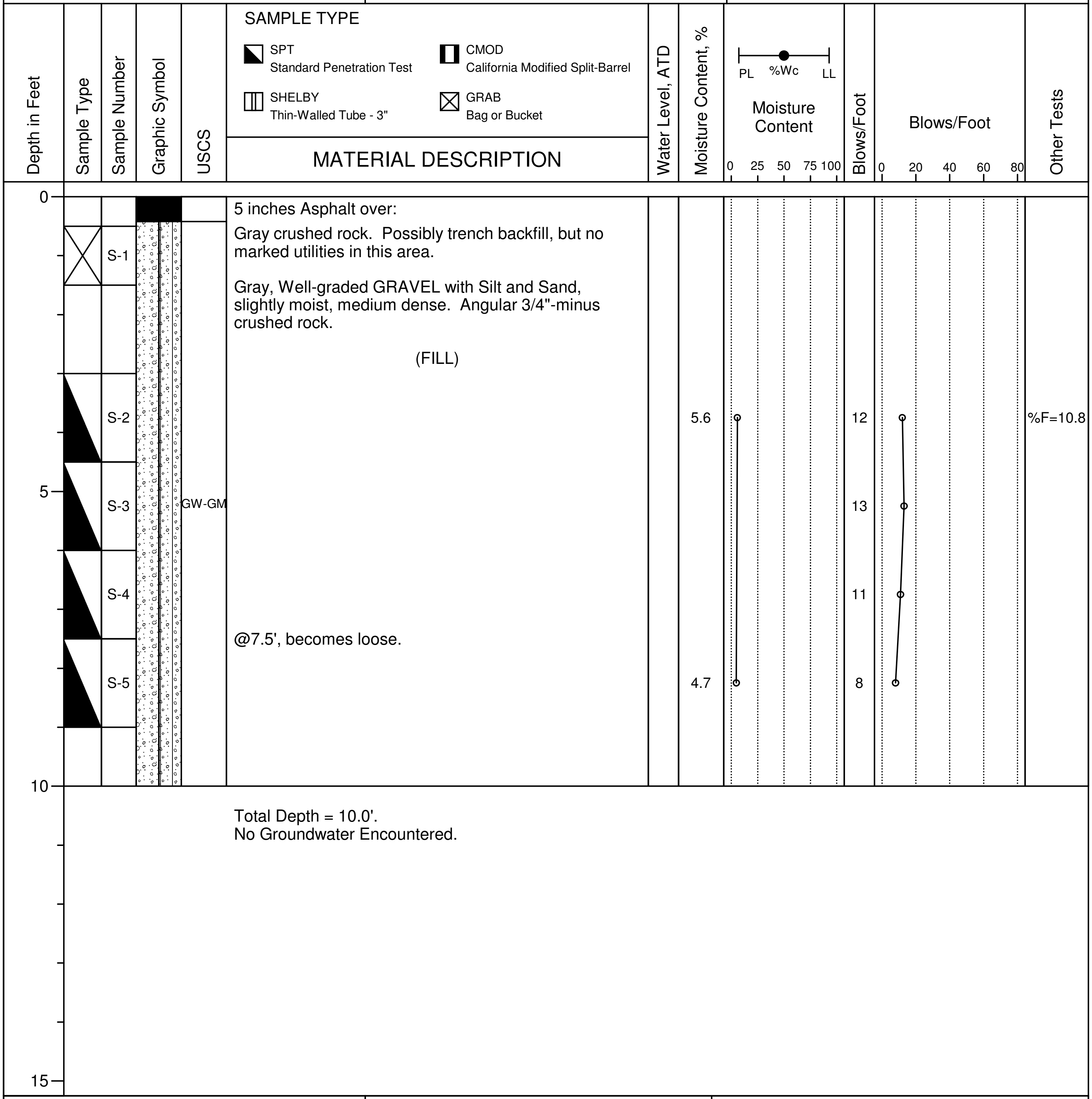
Project Number: 19-008-1

LOG OF BORING B-5

Surface Elevation:	205.0 feet	Start Date:	October 12, 2020
Northing:	365,191	End Date:	October 12, 2020
Easting:	7,524,745	Logged By:	ADM
Coordinate System:	OR State Plane North, NAD 83	Contractor:	Dan Fischer Excavating, Inc.
Drilling Method:	Solid-stem auger		
Drilling Equipment:	Trailer-mount		
Hammer Weight:	140 lb.		
Hammer Drop:	30 in.		

SAMPLE TYPE

- ◩ SPT Standard Penetration Test
- ◫ CMOD California Modified Split-Barrel
- ▥ SHELBY Thin-Walled Tube - 3"
- ☒ GRAB Bag or Bucket



MATERIAL DESCRIPTION

5 inches Asphalt over:

Gray crushed rock. Possibly trench backfill, but no marked utilities in this area.

Gray, Well-graded GRAVEL with Silt and Sand, slightly moist, medium dense. Angular 3/4"-minus crushed rock.

(FILL)

@7.5', becomes loose.

Total Depth = 10.0'.
No Groundwater Encountered.

Sample Type	Sample Number	USCS	Moisture Content, %	Blows/Foot	Other Tests
GRAB	S-1	GW-GM			
SPT	S-2	GW-GM	5.6	12	%F=10.8
SPT	S-3	GW-GM		13	
SPT	S-4	GW-GM		11	
SPT	S-5	GW-GM	4.7	8	

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GEOTECHNICS

Boring B-5

Albany Waterfront Redevelopment
Albany, Oregon

Sta. 26+21

Figure A5

Albany Waterfront Redevelopment
Albany, Oregon

Client: Walker Macy

Project Number: 19-008-1

LOG OF BORING B-6

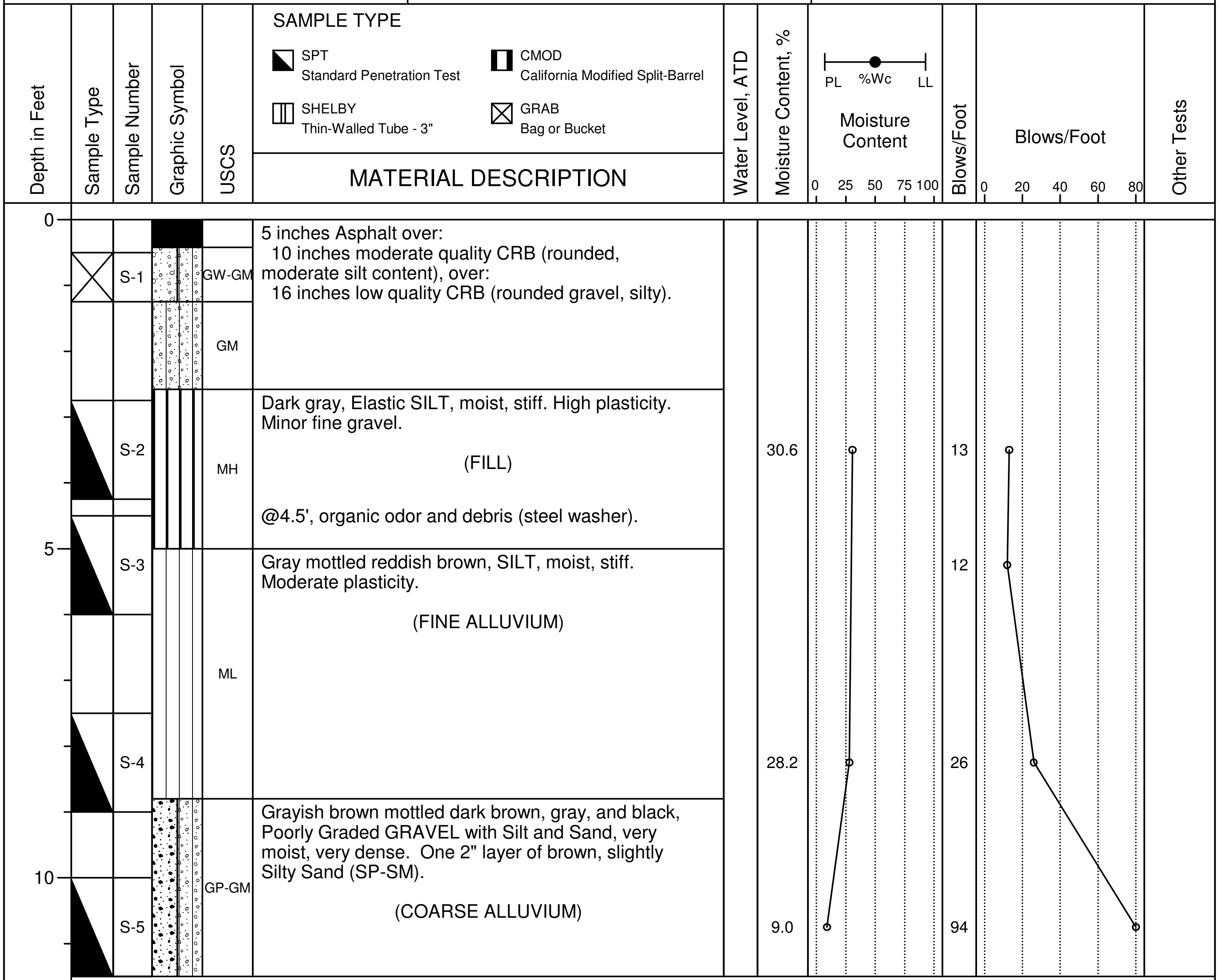
Surface Elevation: 202.0 feet
Northing: 365,126
Easting: 7,524,187
Coordinate System: OR State Plane North, NAD 83

Start Date: October 12, 2020
End Date: October 12, 2020
Logged By: ADM
Contractor: Dan Fischer Excavating, Inc.

Drilling Method: Solid-stem auger
Drilling Equipment: Trailer-mount
Hammer Weight: 140 lb.
Hammer Drop: 30 in.

SAMPLE TYPE

- ◪ SPT Standard Penetration Test
- ◨ CMOD California Modified Split-Barrel
- ▥ SHELBY Thin-Walled Tube - 3"
- ☒ GRAB Bag or Bucket



Sample Type	Sample Number	USCS	MATERIAL DESCRIPTION	Moisture Content, %	Blows/Foot
			5 inches Asphalt over:		
GRAB	S-1	GW-GM	10 inches moderate quality CRB (rounded, moderate silt content), over:		
		GM	16 inches low quality CRB (rounded gravel, silty).		
SPT	S-2	MH	Dark gray, Elastic SILT, moist, stiff. High plasticity. Minor fine gravel. (FILL) @4.5', organic odor and debris (steel washer).	30.6	13
SPT	S-3	ML	Gray mottled reddish brown, SILT, moist, stiff. Moderate plasticity. (FINE ALLUVIUM)		12
SPT	S-4	ML		28.2	26
SPT	S-5	GP-GM	Grayish brown mottled dark brown, gray, and black, Poorly Graded GRAVEL with Silt and Sand, very moist, very dense. One 2" layer of brown, slightly Silty Sand (SP-SM). (COARSE ALLUVIUM)	9.0	94

Total Depth = 11.5'.
No Groundwater Encountered.

Adjacent Dynamic Cone Penetrometer

GEOTECHNICS

Boring B-6

Albany Waterfront Redevelopment
Albany, Oregon

Sta. 20+57

Figure A6

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Albany Waterfront Redevelopment
Albany, Oregon

Client: Walker Macy

Project Number: 19-008-1

LOG OF BORING B-7

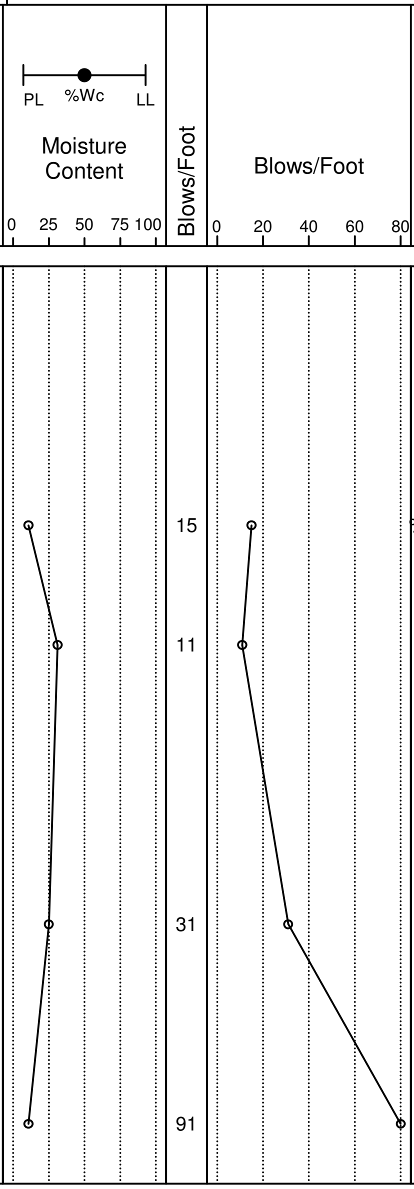
Surface Elevation:	202.0 feet	Start Date:	October 12, 2020	Drilling Method:	Solid-stem auger
Northing:	365,058	End Date:	October 12, 2020	Drilling Equipment:	Trailer-mount
Easting:	7,523,584	Logged By:	ADM	Hammer Weight:	140 lb.
Coordinate System:	OR State Plane North, NAD 83	Contractor:	Dan Fischer Excavating, Inc.	Hammer Drop:	30 in.

SAMPLE TYPE

◣	SPT Standard Penetration Test	◨	CMOD California Modified Split-Barrel
▥	SHELBY Thin-Walled Tube - 3"	☒	GRAB Bag or Bucket

MATERIAL DESCRIPTION

Depth in Feet	Sample Type	Sample Number	Graphic Symbol	USCS	Material Description	Water Level, ATD	Moisture Content, %	Blows/Foot	Other Tests
0					6 inches Asphalt over: 18 inches low quality CRB (rounded gravel, silty).				
	☒	S-1		GM					
	☒	S-2		GC	Dark gray, to bluish gray, Clayey GRAVEL, moist, medium dense. Rounded gravel to 2.5" diam. (FILL) @4', organic odor.				
	◣	S-3					10.7	15	%F = 14.4
5	◣	S-4		ML-MH	Gray, SILT to Elastic SILT, moist, stiff. Moderate to high plasticity.		31.2	11	
				ML	Gray mottled reddish brown, SILT, moist, stiff. Moderate plasticity.				
				GP-GM	Brown mottled yellowish brown, gray, and black, Poorly Graded GRAVEL with Silt, moist, medium dense. Rounded gravel to 1" diam.				
	◣	S-5		ML	Grayish brown mottled reddish brown SILT, moist, very stiff. Minor fine gravel and coarse sand.		24.9	31	
10	◣	S-6		GP-GM	Grayish brown mottled yellowish brown, light gray, and dark brown, Poorly Graded GRAVEL with Silt, moist, very dense. Rounded to subrounded gravel to 1" diam. (COARSE ALLUVIUM)		10.6	91	
15									



Total Depth = 11.5'.
No Groundwater Encountered

GEOTECHNICS

Boring B-7

Albany Waterfront Redevelopment
Albany, Oregon

Sta. 14+47

Albany Waterfront Redevelopment
Albany, Oregon

Client: Walker Macy

Project Number: 19-008-1

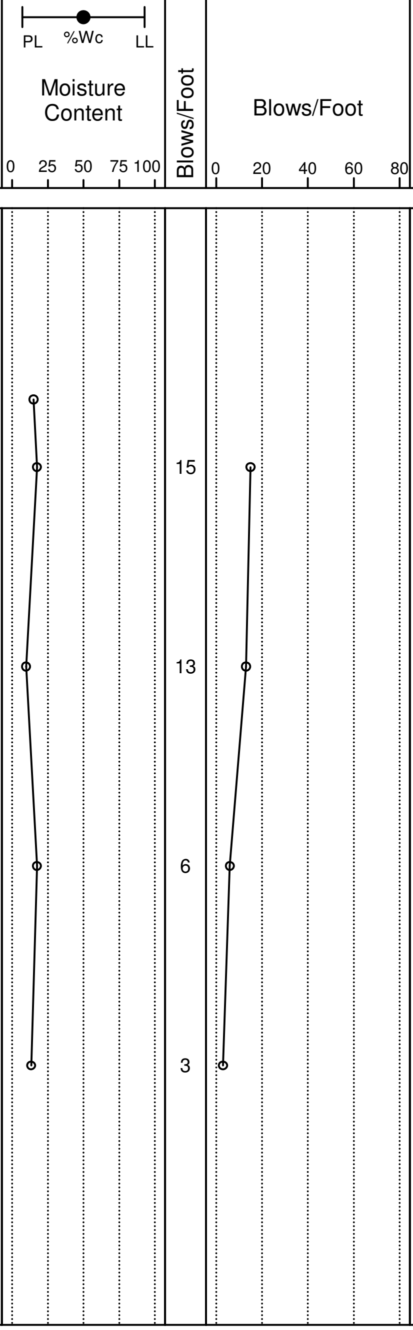
LOG OF BORING B-8

Surface Elevation:	201.5 feet	Start Date:	October 12, 2020	Drilling Method:	Solid-stem auger
Northing:	365,095	End Date:	October 12, 2020	Drilling Equipment:	Trailer-mount
Easting:	7,522,740	Logged By:	ADM	Hammer Weight:	140 lb.
Coordinate System:	OR State Plane North, NAD 83	Contractor:	Dan Fischer Excavating, Inc.	Hammer Drop:	30 in.

SAMPLE TYPE

- ◩ SPT Standard Penetration Test
- ▮ CMOD California Modified Split-Barrel
- ▥ SHELBY Thin-Walled Tube - 3"
- ☒ GRAB Bag or Bucket

Depth in Feet	Sample Type	Sample Number	USCS	MATERIAL DESCRIPTION	Water Level, ATD	Moisture Content, %	Blows/Foot	Other Tests
0			ML	Dark brown mottled tan, Sandy SILT, with gravel, slightly moist, stiff. (FILL)				
	☒	S-1		@2.5', becomes dark brown. Minor wood chips.		15.0		GSD %F=59.5
	◩	S-2				17.3	15	Inf Test @ 2' 6"
5				@5', becomes brown mottled black and tan.				
	◩	S-3				9.8	13	
	◩	S-4		@8.7', becomes light reddish brown mottled dark brown and tan, becomes medium stiff.		17.4	6	
			GM	Dark brown mottled reddish brown, Silty GRAVEL with Sand, moist, very loose.				
10	◩	S-5				13.3	3	
				Refusal on wood.				
15				Total Depth = 14'. No Groundwater Encountered Adjacent Infiltration Test				



GEOTECHNICS

Boring B-8

Albany Waterfront Redevelopment
Albany, Oregon

Figure A8

Albany Waterfront Redevelopment
Albany, Oregon

Client: Walker Macy

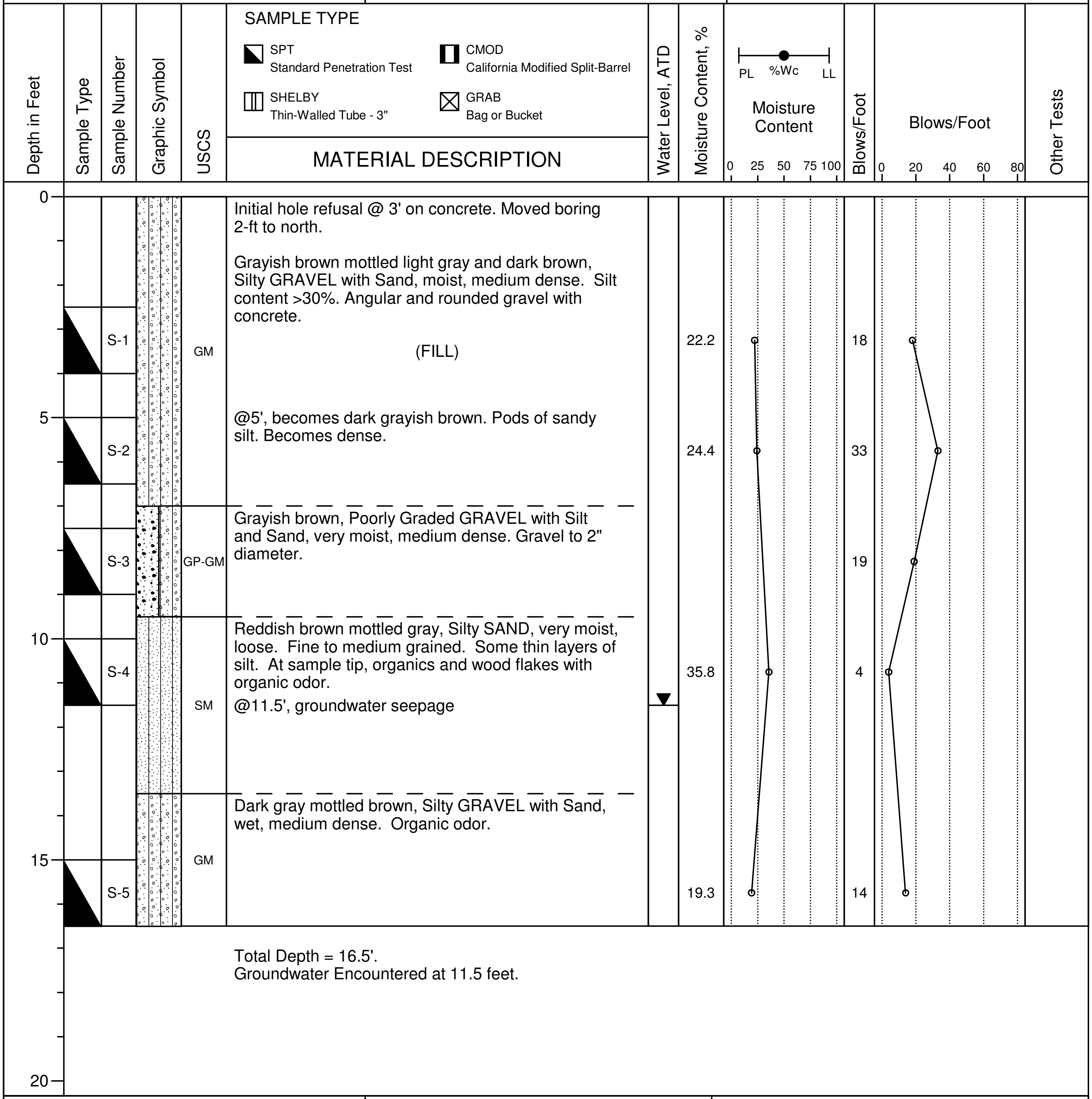
Project Number: 19-008-1

LOG OF BORING B-9

Surface Elevation:	187.5 feet	Start Date:	October 13, 2020	Drilling Method:	Solid-stem auger
Northing:	365,211	End Date:	October 13, 2020	Drilling Equipment:	Trailer-mount
Easting:	7,522,511	Logged By:	ADM	Hammer Weight:	140 lb.
Coordinate System:	OR State Plane North, NAD 83	Contractor:	Dan Fischer Excavating, Inc.	Hammer Drop:	30 in.

SAMPLE TYPE

- ◪ SPT Standard Penetration Test
- ◫ CMOD California Modified Split-Barrel
- ◫ SHELBY Thin-Walled Tube - 3"
- ☒ GRAB Bag or Bucket



MATERIAL DESCRIPTION

Initial hole refusal @ 3' on concrete. Moved boring 2-ft to north.

Grayish brown mottled light gray and dark brown, Silty GRAVEL with Sand, moist, medium dense. Silt content >30%. Angular and rounded gravel with concrete.

(FILL)

@5', becomes dark grayish brown. Pods of sandy silt. Becomes dense.

Grayish brown, Poorly Graded GRAVEL with Silt and Sand, very moist, medium dense. Gravel to 2" diameter.

Reddish brown mottled gray, Silty SAND, very moist, loose. Fine to medium grained. Some thin layers of silt. At sample tip, organics and wood flakes with organic odor.

@11.5', groundwater seepage

Dark gray mottled brown, Silty GRAVEL with Sand, wet, medium dense. Organic odor.

Total Depth = 16.5'.
Groundwater Encountered at 11.5 feet.

Sample Type	Sample Number	USCS	Moisture Content, %	Blows/Foot
SPT	S-1	GM	22.2	18
SPT	S-2		24.4	33
SPT	S-3	GP-GM		19
SPT	S-4	SM	35.8	4
SPT	S-5	GM	19.3	14

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GEOTECHNICS

Boring B-9

Albany Waterfront Redevelopment
Albany, Oregon

Figure A9

Albany Waterfront Redevelopment
Albany, Oregon

Client: Walker Macy

Project Number: 19-008-1

LOG OF BORING B-10

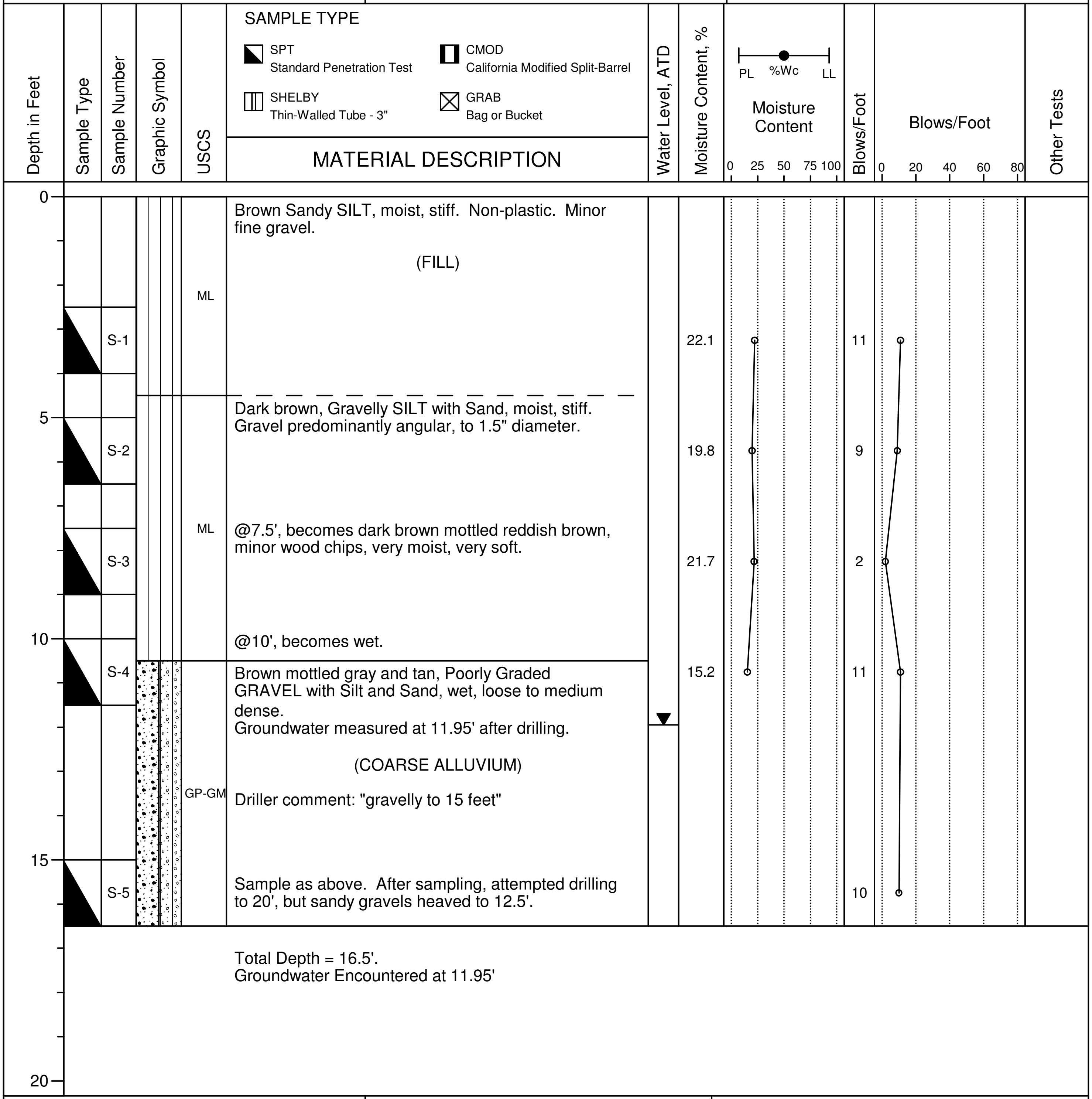
Surface Elevation: 183.5 feet
Northing: 365,328
Easting: 7,522,572
Coordinate System: OR State Plane North, NAD 83

Start Date: October 13, 2020
End Date: October 13, 2020
Logged By: ADM
Contractor: Dan Fischer Excavating, Inc.

Drilling Method: Solid-stem auger
Drilling Equipment: Trailer-mount
Hammer Weight: 140 lb.
Hammer Drop: 30 in.

SAMPLE TYPE

- ◩ SPT Standard Penetration Test
- ◧ CMOD California Modified Split-Barrel
- ▥ SHELBY Thin-Walled Tube - 3"
- ☒ GRAB Bag or Bucket



Depth in Feet	Sample Type	Sample Number	USCS	MATERIAL DESCRIPTION	Water Level, ATD	Moisture Content, %	Blows/Foot	Other Tests
0			ML	Brown Sandy SILT, moist, stiff. Non-plastic. Minor fine gravel. (FILL)				
	SPT	S-1				22.1	11	
5	SPT	S-2	ML	Dark brown, Gravelly SILT with Sand, moist, stiff. Gravel predominantly angular, to 1.5" diameter.		19.8	9	
				@7.5', becomes dark brown mottled reddish brown, minor wood chips, very moist, very soft.				
	SPT	S-3				21.7	2	
10				@10', becomes wet.				
	SPT	S-4	GP-GM	Brown mottled gray and tan, Poorly Graded GRAVEL with Silt and Sand, wet, loose to medium dense. Groundwater measured at 11.95' after drilling. (COARSE ALLUVIUM) Driller comment: "gravelly to 15 feet"	▼	15.2	11	
15	SPT	S-5		Sample as above. After sampling, attempted drilling to 20', but sandy gravels heaved to 12.5'.			10	
20								

Total Depth = 16.5'.
Groundwater Encountered at 11.95'

Albany Waterfront Redevelopment
Albany, Oregon

Client: Walker Macy

Project Number: 19-008-1

LOG OF BORING B-11

Surface Elevation:	206.0 feet	Start Date:	October 12, 2020	Drilling Method:	Solid-stem auger
Northing:	364,991	End Date:	October 12, 2020	Drilling Equipment:	Trailer-mount
Easting:	7,523,008	Logged By:	ADM	Hammer Weight:	140 lb.
Coordinate System:	OR State Plane North, NAD 83	Contractor:	Dan Fischer Excavating, Inc.	Hammer Drop:	30 in.

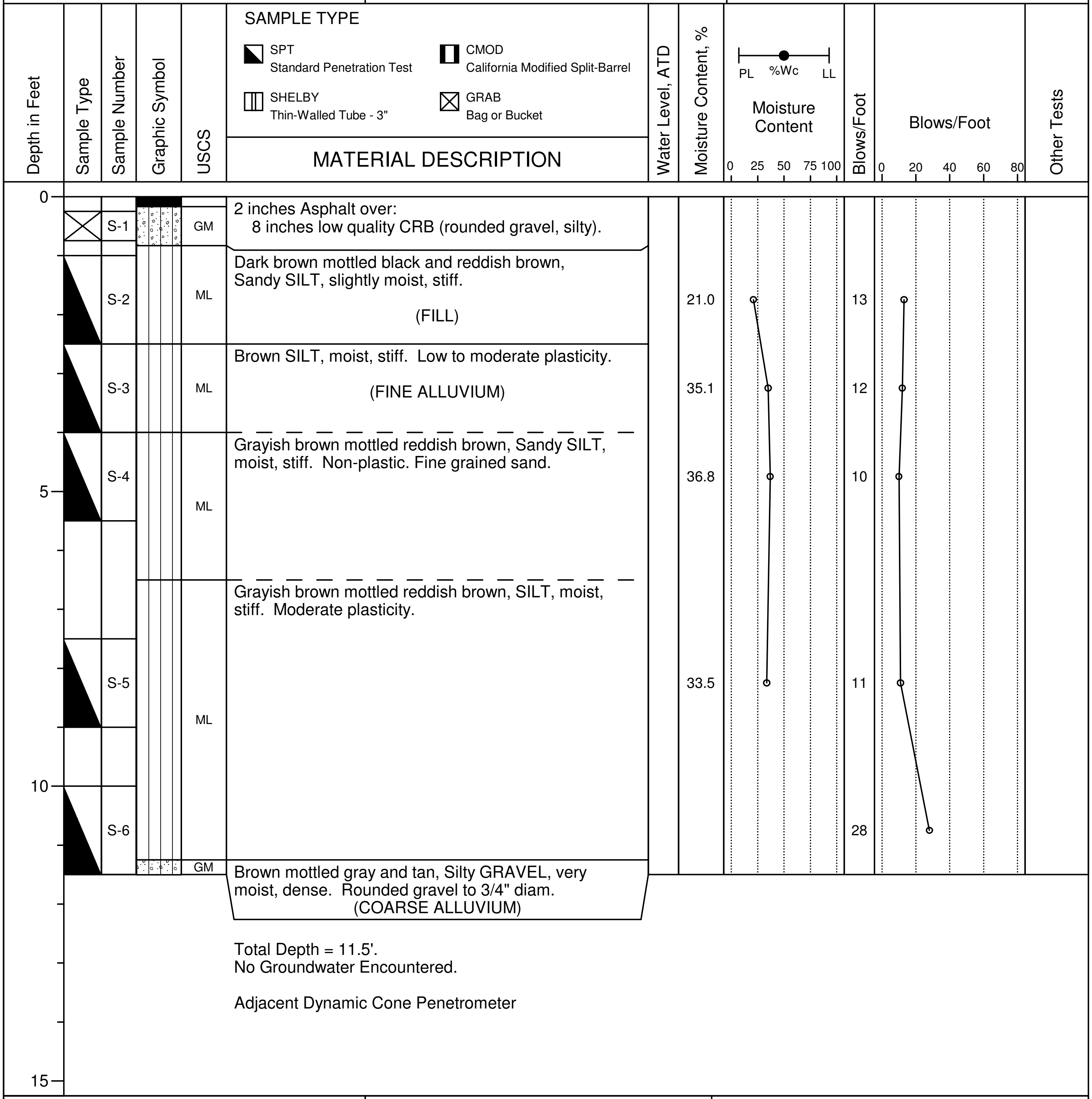
SAMPLE TYPE

◩ SPT Standard Penetration Test

◨ CMOD California Modified Split-Barrel

▥ SHELBY Thin-Walled Tube - 3"

☒ GRAB Bag or Bucket



2 inches Asphalt over:
8 inches low quality CRB (rounded gravel, silty).

Dark brown mottled black and reddish brown, Sandy SILT, slightly moist, stiff.

(FILL)

Brown SILT, moist, stiff. Low to moderate plasticity.

(FINE ALLUVIUM)

Grayish brown mottled reddish brown, Sandy SILT, moist, stiff. Non-plastic. Fine grained sand.

Grayish brown mottled reddish brown, SILT, moist, stiff. Moderate plasticity.

Brown mottled gray and tan, Silty GRAVEL, very moist, dense. Rounded gravel to 3/4" diam.
(COARSE ALLUVIUM)

Total Depth = 11.5'.
No Groundwater Encountered.

Adjacent Dynamic Cone Penetrometer

Sample Type	Sample Number	USCS	Moisture Content, %	Blows/Foot
☒	S-1	GM		
◩	S-2	ML	21.0	13
◩	S-3	ML	35.1	12
◩	S-4	ML	36.8	10
◩	S-5	ML	33.5	11
◩	S-6	GM		28

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GEOTECHNICS

Boring B-11

Albany Waterfront Redevelopment
Albany, Oregon

Sta. 08+64

Figure A11

Albany Waterfront Redevelopment
Albany, Oregon

Client: Walker Macy

Project Number: 19-008-1

LOG OF BORING HA-1

Surface Elevation: 178.5 feet
Northing: 365,369
Easting: 7,522,558
Coordinate System: OR State Plane North, NAD 83

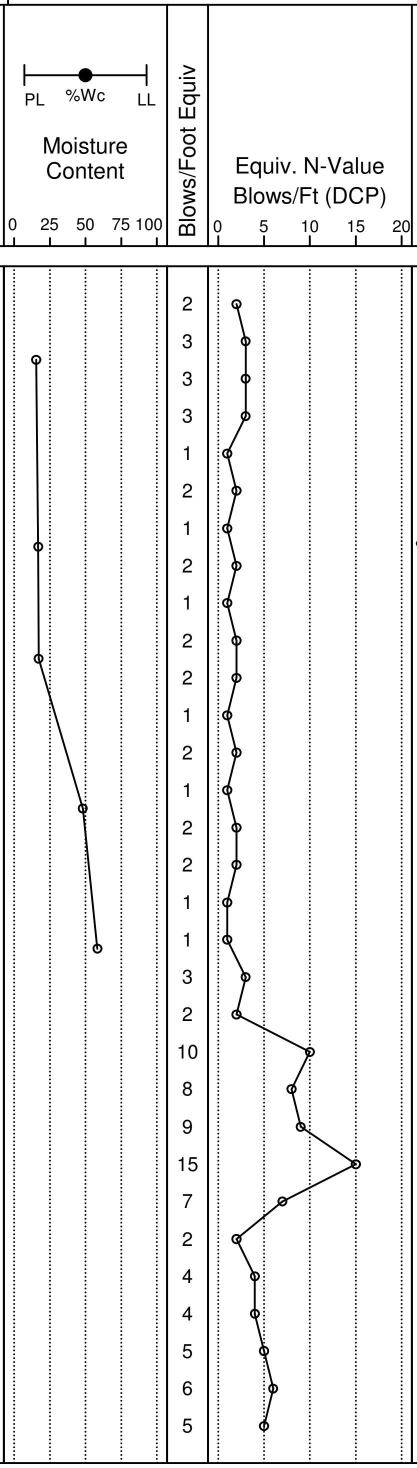
Start Date: October 13, 2020
End Date: October 13, 2020
Logged By: ADM
Contractor: Geotechnics

Drilling Method: Hand-Auger
Drilling Equipment: 3" diam HA + Wildcat cone
Hammer Weight: 35#
Hammer Drop: 15-inch

SAMPLE TYPE

- ◩ SPT Standard Penetration Test
- ◨ CMOD California Modified Split-Barrel
- ▥ SHELBY Thin-Walled Tube - 3"
- ☒ GRAB Bag or Bucket

Depth in Feet	Sample Type	Sample Number	USCS	MATERIAL DESCRIPTION	Water Level, ATD	Moisture Content, %	Other Tests
0			SM	Brown Silty SAND, slightly moist, very loose. Fine grained. 30-50% Silt. Some roots/rootlets. @0.5', becomes tan. (RECENT ALLUVIUM)			
	☒	S-1				15.5	
	☒	S-2					
				@3.5', becomes moist, lower silt content (20-30%).			
	☒	S-3				16.8	%F=25.0
5	☒	S-4				17.2	
				@5.75', becomes tan mottled light gray.			
	☒	S-5					
			ML	Gray SILT, very moist, soft to very soft. Low to moderate plasticity. Abundant roots. Groundwater measured at 6.9 ft bgs after drilling.	▼		
	☒	S-6				48.2	
			OH	Gray to dark gray, Organic SILT, wet, very soft. Strong organic odor.			
	☒	S-7				58.4	
10				Total Depth = 9.25'. Groundwater Encountered at 6.9'			
15							



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GEOTECHNICS

Boring HA-1

Albany Waterfront Redevelopment
Albany, Oregon

Figure A12

WILDCAT DYNAMIC CONE LOG

Geotechnics LLC
30110 E Woodard Rd
Troutdale, OR 97060

PROJECT NUMBER: 19-008-1
DATE STARTED: 12-04-2020
DATE COMPLETED: 12-04-2020

HOLE #: DCP-HA1
CREW: ADM
PROJECT: Albany Waterfront
ADDRESS: Montieth Park
LOCATION: Albany, Oregon

SURFACE ELEVATION: 179
WATER ON COMPLETION:
HAMMER WEIGHT: 35 lbs.
CONE AREA: 10 sq. cm

DEPTH	BLOWS PER 10 cm	RESISTANCE Kg/cm²	GRAPH OF CONE RESISTANCE 0 50 100 150	N'	TESTED CONSISTENCY SAND & SILT	CLAY
-	1	4.4	•	1	VERY LOOSE	VERY SOFT
-	2	8.9	••	2	VERY LOOSE	SOFT
- 1 ft	3	13.3	•••	3	VERY LOOSE	SOFT
-	2	8.9	••	2	VERY LOOSE	SOFT
-	3	13.3	•••	3	VERY LOOSE	SOFT
- 2 ft	3	13.3	•••	3	VERY LOOSE	SOFT
-	3	13.3	•••	3	VERY LOOSE	SOFT
-	1	4.4	•	1	VERY LOOSE	VERY SOFT
- 3 ft	2	8.9	••	2	VERY LOOSE	SOFT
- 1 m	1	4.4	•	1	VERY LOOSE	VERY SOFT
-	1	3.9	•	1	VERY LOOSE	VERY SOFT
- 4 ft	2	7.7	••	2	VERY LOOSE	SOFT
-	3	11.6	•••	3	VERY LOOSE	SOFT
-	1	3.9	•	1	VERY LOOSE	VERY SOFT
- 5 ft	2	7.7	••	2	VERY LOOSE	SOFT
-	1	3.9	•	1	VERY LOOSE	VERY SOFT
-	2	7.7	••	2	VERY LOOSE	SOFT
- 6 ft	1	3.9	•	1	VERY LOOSE	VERY SOFT
-	2	7.7	••	2	VERY LOOSE	SOFT
- 2 m	2	7.7	••	2	VERY LOOSE	SOFT
- 7 ft	2	6.8	•	1	VERY LOOSE	VERY SOFT
-	3	10.3	••	2	VERY LOOSE	SOFT
-	3	10.3	••	2	VERY LOOSE	SOFT
- 8 ft	3	10.3	••	2	VERY LOOSE	SOFT
-	4	13.7	•••	3	VERY LOOSE	SOFT
-	2	6.8	•	1	VERY LOOSE	VERY SOFT
- 9 ft	1	3.4		0	VERY LOOSE	VERY SOFT
-	3	10.3	••	2	VERY LOOSE	SOFT
-	4	13.7	•••	3	VERY LOOSE	SOFT
- 3 m 10 ft	3	10.3	••	2	VERY LOOSE	SOFT
-	8	24.5	•••••••	6	LOOSE	MEDIUM STIFF
-	12	36.7	••••••••••	10	LOOSE	STIFF
-	12	36.7	••••••••••	10	LOOSE	STIFF
- 11 ft	10	30.6	••••••••	8	LOOSE	MEDIUM STIFF
-	11	33.7	•••••••••	9	LOOSE	STIFF
-	14	42.8	••••••••••••	12	MEDIUM DENSE	STIFF
- 12 ft	20	61.2	•••••••••••••••••	17	MEDIUM DENSE	VERY STIFF
-	8	24.5	•••••••	6	LOOSE	MEDIUM STIFF
-	3	9.2	••	2	VERY LOOSE	SOFT
- 4 m 13 ft	3	9.2	••	2	VERY LOOSE	SOFT
-	6	16.6	••••	4	VERY LOOSE	SOFT
-	6	16.6	••••	4	VERY LOOSE	SOFT
- 14 ft	6	16.6	••••	4	VERY LOOSE	SOFT
-	7	19.4	•••••	5	LOOSE	MEDIUM STIFF
-	7	19.4	•••••	5	LOOSE	MEDIUM STIFF
- 15 ft	8	22.2	••••••	6	LOOSE	MEDIUM STIFF
-	7	19.4	•••••	5	LOOSE	MEDIUM STIFF
-	7	19.4	•••••	5	LOOSE	MEDIUM STIFF

DYNAMIC CONE LOG DCP-HA1

Albany Waterfront Redevelopment
Albany, Oregon

Project No. 19-008-1

Figure A13

DCP-B1:

Soil Description	Bottom Depth Below Paved Surface (in)	Blows	Cumulative Blows	Accumulative Penetration (mm)	Incremental Penetration (mm)	DCP (mm/blow)	Uncorrected M_r (psi)	Corrected M_r (psi)
SILT	31.0	3	3	51	51	16.9	16,263	5,692
	33.0	3	6	102	51	16.9	16,263	5,692
	35.0	5	11	152	51	10.2	19,848	6,947
	37.0	4	15	203	51	12.7	18,193	6,368
	39.0	5	20	254	51	10.2	19,848	6,947
	41.0	6	26	305	51	8.5	21,310	7,459
	43.0	6	32	356	51	8.5	21,310	7,459
	45.0	7	39	406	51	7.3	22,631	7,921
	47.0	7	46	457	51	7.3	22,631	7,921

Asphalt: 4.5"

Base: 23"

Above 39" Average M_r: 18,083 6,329

Corresponding CBR: 4.2

DCP-B4:

Soil Description	Bottom Depth Below Paved Surface (in)	Blows	Cumulative Blows	Accumulative Penetration (mm)	Incremental Penetration (mm)	DCP (mm/blow)	Uncorrected M_r (psi)	Corrected M_r (psi)
SILT	25.0	2	2	51	51	25.4	13,884	4,859
	27.0	2	4	102	51	25.4	13,884	4,859
	29.0	2	6	152	51	25.4	13,884	4,859
	31.0	2	8	203	51	25.4	13,884	4,859
	33.0	2	10	254	51	25.4	13,884	4,859
	35.0	3	13	305	51	16.9	16,263	5,692
	37.0	2	15	356	51	25.4	13,884	4,859
	39.0	5	20	406	51	10.2	19,848	6,947
	41.0	5	25	457	51	10.2	19,848	6,947
	43.0	2	27	508	51	25.4	13,884	4,859
	45.0	5	32	559	51	10.2	19,848	6,947
	47.0	8	40	610	51	6.3	23,841	8,344
	49.0	8	48	660	51	6.3	23,841	8,344

Asphalt: 2.5"

Base: 22.5"

Above 37" Average M_r: 14,224 4,978

Corresponding CBR: 3.3

Testing in accordance with ASTM D6951

GEOTECHNICS

DYNAMIC CONE PENETROMETER LOGS

Albany Waterfront Redevelopment
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Figure A14

DCP-B6:

Soil Description	Bottom Depth Below Paved Surface (in)	Blows	Cumulative Blows	Accumulative Penetration (mm)	Incremental Penetration (mm)	DCP (mm/blow)	Uncorrected M_r (psi)	Corrected M_r (psi)
	35.0	2	2	51	51	25.4	13,884	4,859
Elastic SILT	37.0	2	4	102	51	25.4	13,884	4,859
	39.0	2	6	152	51	25.4	13,884	4,859
	41.0	2	8	203	51	25.4	13,884	4,859
	43.0	2	10	254	51	25.4	13,884	4,859
	45.0	3	13	305	51	16.9	16,263	5,692
	47.0	2	15	356	51	25.4	13,884	4,859

Asphalt:	5"	**Above 43" Average M_r:**	13,884	4,859
Base:	26"	**Corresponding CBR:**		3.2

DCP-B11:

Soil Description	Bottom Depth Below Paved Surface (in)	Blows	Cumulative Blows	Accumulative Penetration (mm)	Incremental Penetration (mm)	DCP (mm/blow)	Uncorrected M_r (psi)	Corrected M_r (psi)
	12.0	1	1	51	51	50.8	10,595	3,708
Sandy SILT	14.0	1	2	102	51	50.8	10,595	3,708
	16.0	3	5	152	51	16.9	16,263	5,692
	18.0	2	7	203	51	25.4	13,884	4,859
	20.0	2	9	254	51	25.4	13,884	4,859
	22.0	2	11	305	51	25.4	13,884	4,859
	24.0	2	13	356	51	25.4	13,884	4,859
	26.0	3	16	406	51	16.9	16,263	5,692
	28.0	3	19	457	51	16.9	16,263	5,692
SILT	30.0	4	23	508	51	12.7	18,193	6,368
	32.0	3	26	559	51	16.9	16,263	5,692
	34.0	2	28	610	51	25.4	13,884	4,859
	36.0	3	31	660	51	16.9	16,263	5,692
	38.0	4	35	711	51	12.7	18,193	6,368
	40.0	4	39	762	51	12.7	18,193	6,368
	42.0	5	44	813	51	10.2	19,848	6,947
	44.0	6	50	864	51	8.5	21,310	7,459
	46.0	5	55	914	51	10.2	19,848	6,947
Sandy SILT	48.0	6	61	965	51	8.5	21,310	7,459
	50.0	7	68	1016	51	7.3	22,631	7,921

Asphalt:	2"	**Above 40" Average M_r:**	15,100	5,285
Base:	8"	**Corresponding CBR:**		3.5

Testing in accordance with ASTM D6951

GEOTECHNICS

DYNAMIC CONE PENETROMETER LOGS

Albany Waterfront Redevelopment
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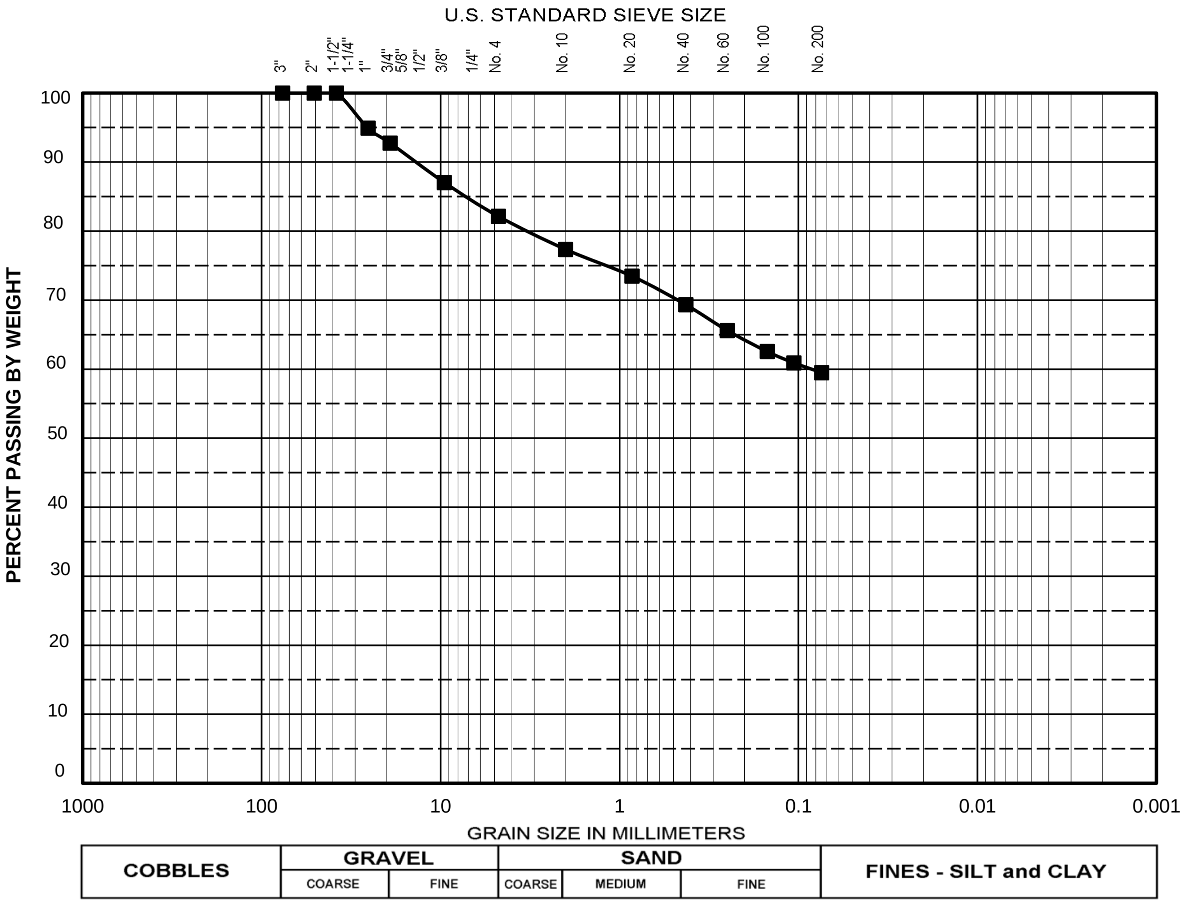
Figure A15



Symbol	Sample Location	% MC	% Gravel	% Sand	%Fines	Classification
■	B-8 ; @ 2.3 - 2.5ft	15.0	17.9	22.7	59.5	Sandy SILT with Gravel (ML)

Grain Size Distribution determined in accordance with ASTM D-6913

GEOTECHNICS

GRAIN SIZE DISTRIBUTION

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Figure A16

Appendix B

PHOTOGRAPHS - ROADWAY PAVEMENTS

WASHINGTON ST TO FERRY ST

8+30

9+00

FERRY ST TO BROADALBIN ST

11+70

13+40

BROADALBIN ST TO ELLSWORTH ST

15+00

16+80

ELLSWORTH ST TO LYON ST

18+40

20+00

LYON ST TO BAKER ST

21+00

21+70

BAKER ST TO MONTGOMERY ST

25+10

26+80

MONTGOMERY ST TO RAILROAD ST

28+50

30+30

RAILROAD ST TO JACKSON ST

31+90

33+60

JACKSON ST TO JEFFERSON ST

35+00

36+90

JEFFERSON ST TO THURSTON ST

38+50

40+20

THURSTON ST TO LAFAYETTE ST

41+80

43+50

LAFAYETTE ST TO MADISON ST

45+10

46+80

MADISON ST TO HILL ST

48+50

50+10

HILL ST TO MAIN ST

51+70

53+30

Appendix C

PHOTOGRAPHS - TRAIL

Between Washington & Ferry
cracks to 3/4” width

End of Ferry Street,
drain pipe offset

Between Ferry & Broadalbin,
slab panel tilt towards slope

Between Ferry & Broadalbin,
parallel crack & slab tilt

Between Broadalbin & Ellsworth,
trail undermining, 9” lateral

Between Broadalbin & Ellsworth,
existing boardwalk

Between Ellsworth & Lyon,
slab tilt towards slope

Between Ellsworth & Lyon,
slab panel lateral offset

End of Lyon Street,
overlook undermining, 28” lateral

End of Lyon Street,
overlook undermining, 28” lateral

Between Lyon & Baker,
cracks to 1” width

Between Montgomery & Railroad,
trail transition to pile-support

Between Jackson & Jefferson,
tilting posts, pile-supported trail

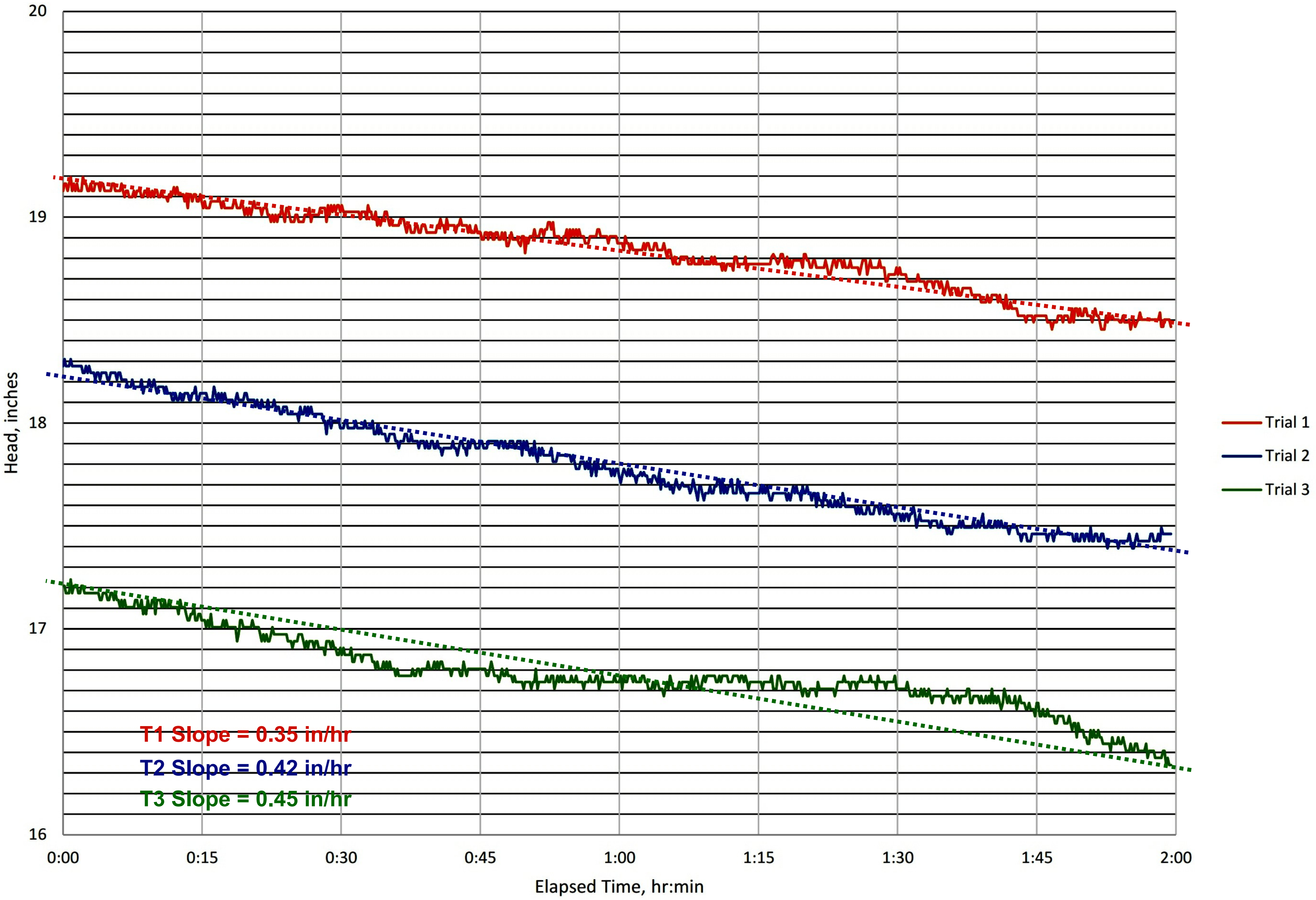
Between Jefferson & Thurston,
good condition

Between Thurston & Lafayette,
good condition

Between Madison & Hill,
modular-block retaining wall, good trail condition

Appendix D

INFILTRATION



Unfactored Rate = 0.41 in/hr

GEOTECHNICS

INFILTRATION I-1 @ 2.5’

Albany Waterfront Redevelopment
Albany, Oregon

Project No. 19-008-1

Figure D1