


Geotechnical Engineering Design Study

Foothills Trail Phase II

King County, Washington

Prepared for

King County Parks and Recreation

Revised
February 9, 2016
17633-03

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Prepared by

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1.0 INTRODUCTION

This report presents the results of our subsurface explorations and geotechnical engineering design study for the proposed Phase II of the Foothills Trail project, located in unincorporated King County and the City of Buckley in Washington State.

Our scope of work for this study included:

- Completing 18 test pit explorations along the proposed trail alignment.
- Completing four ultrasonic borings and two auger borings along the proposed trail alignment, including the two bridge locations.
- Collecting soil samples and performing laboratory tests on representative samples.
- Analyzing geotechnical engineering aspects of:
  - Soil bearing capacity and passive resistance of the existing Boise Creek Arch Bridge footing elements;
  - Potential liquefaction at the two White River Bridge abutment pier locations;
  - General condition of the existing concrete railroad abutment and backfill material north of the Boise Creek Arch Bridge; and
  - Stability of the proposed trail alignment along the portion where Boise Creek is encroaching and actively undercutting the adjacent steep slope.
- Providing geotechnical recommendations for:
  - Subgrade preparation and pavement for on-grade trail sections;
  - Allowable vertical and uplift capacity and lateral resistance for shallow mat footing support at the proposed White River Bridge abutments;
  - General design of mechanically stabilized earth (MSE) walls;
  - Temporary and permanent cut/fill slopes;
  - Structural fill; and
  - Geotechnical construction.
- Summarizing our findings in this geotechnical engineering report.

We completed this work in general accordance with Amendment No. 3 (and subsequent relevant amendments) to our subconsultant agreement with Huitt-Zollars dated November 13, 2015. This report is for the exclusive use of Huitt-Zollars, King County, and their design consultants and construction contractors for specific application to the subject project and site. We performed our work in general accordance with geotechnical engineering practices accepted for work done in the same or similar localities, related to the nature of the work we accomplished here, and done at the time our services were performed. No other warranty, express or implied, is made.

2.0 SITE AND PROJECT DESCRIPTION

The proposed Phase II of the Foothills Trail system begins at the existing Stream 5 trail bridge southwest of 252nd Avenue Southeast and continues southwesterly along the old railroad corridor that runs parallel to State Route 410 before turning south across the old Boise Creek Bridge. The proposed trail alignment continues southwest through the previous Nagel property and across Mud Mountain Road, then crosses the White River into Buckley along the old SR 410 route(Figure 1).

A previous geotechnical design study by Hart Crowser (2011) evaluated the northeast portion of the trail, which ends with the Stream 5 Bridge southwest of 252nd Avenue Southeast (where the Phase II trail begins). The southern end of the proposed Phase II trail alignment connects with a completed segment of the Foothills Trail that runs through the City of Buckley.

Figure 2 shows the general Phase II trail alignment, and Figures 3 through 9 show the details of the field located topography and exploration locations. These figures also present stick log representations of the soil stratigraphy encountered in each exploration.

In general, the 12-foot-wide asphalt-paved trail will be supported on grade. Trail use will typically be limited to pedestrians and bicycles, but the pavement surface will be subject to loads from occasional use by light maintenance or emergency vehicles. Additionally, if the bridge for State Route 410 is closed for an emergency, trail design will accommodate emergency vehicles that could use the trail's White River Bridge crossing.

Generally, structural retaining walls, open cuts, and/or embankment fills will be used to accommodate the trail grades and surrounding terrain along the trail alignment. The three Phase II trail sections are summarized below.

2.1 Old Railroad Grade Trail Alignment

This segment extends about 3,600 feet (from approximately Station 105+00 at the Stream 5 Crossing to approximate Station 141+00) over generally level and gently sloping terrain before sloping down to transition across the Boise Creek Bridge. Currently a walking trail is in place along the route of the old railroad alignment. Site explorations showed varying depths of fill along the entire alignment. Remnants of the railroad system such as railroad ties and old track lengths were not encountered during field explorations, but could be found along this portion of the project.

The trail through this segment will consist primarily of on-grade sections of pavement. The existing walking trail along the old railroad alignment minimizes the need for site clearing or grading. A portion of the existing railroad alignment (from approximately Station 120+00 to Station 130+00) crosses a wetlands area. To reduce disturbance of the wetlands, the trail will be routed onto the adjacent embankment, requiring vegetation clearing and possible grading (cuts and fills).

The trail also needs to be rerouted onto the adjacent embankment to allow adequate setback to address slope stability issues where Boise Creek encroaches on the existing trail alignment, as addressed in Section 5 of this report. Additionally, significant grading cuts are required over several hundred feet to bring the proposed trail from the old railroad grade to the lower elevation of the existing Boise Creek Bridge deck.

2.2 Boise Creek Bridge to Mud Mountain Road

The Boise Creek Bridge to Mud Mountain Road segment is about 1,000 feet long, extending from approximately Station 142+00 to Station 151+00. The trail stationing shown from the Boise Creek Bridge (142+00) to the City of Buckley (160+00) is based on the old SR 410 road alignment and to be used as reference only. Most of this trail section lies within the Nagel property adjacent to Mud Mountain Road. Existing structures on the Nagel property include the residence home, tennis courts, and a swimming pool. A gravel road provides access from the house to Mud Mountain Road along both the south and northeast ends of the property.

The trail through this segment will primarily be paved and on-grade, but elevation changes may require site grading. Specifically, the approach to the proposed Mud Mountain Road crossing runs through the southwest corner of the property and will be located atop a sharp drop in grade. Grading cuts may be needed in this area to maintain maximum allowable trail slope inclinations, depending on the final trail alignment.

2.3 Mud Mountain Road to White River Bridge

The final trail segment runs from Mud Mountain Road to the remaining south abutment of the former SR 410 White River Bridge crossing (on Buckley side of river), extending from approximately Station 151+00 to Station 158+75. Prior to the former bridge demolition, a trestle structure supported the north approach to the bridge, which roughly aligns with the proposed trail. Subgrade construction work along this part of the trail may encounter old concrete footings which were found along the former SR 410 road alignment.

Currently, construction of a fill embankment is recommended as likely the most appropriate and cost-effective approach to support the trail approach between Mud Mountain Road and the north side of the bridge, as detailed in Section 5. An approach embankment already exists on the south side of the proposed White River bridge crossing.

The current proposal, and our assumption for this geotechnical evaluation, is that the old SR 167 Puyallup River bridge structure will be relocated and reused as the White River bridge crossing for this project.

3.0 GENERALIZED SUBSURFACE CONDITIONS

Our understanding of the subsurface conditions along the proposed trail alignment is based on data from our soil explorations and laboratory tests, as well as from explorations completed in the past by others. Our field exploration program consisted of advancing two hollow-stem auger borings (BA-1 through BA-2), four ultrasonic continuous borings (BS-1 through BS-4), and 18 test pits (TP-1 through TP-18). These were completed between November 16 and December 2, 2015. The borings were drilled to depths ranging from 12 to 130 feet below ground surface and the test pits were excavated to depths ranging from 5 to 14 feet below ground surface. The locations of these explorations are shown on Figures 3 through 9.

Subsurface conditions were logged by field geologists and a geotechnical engineer from Hart Crowser and recorded on detailed boring and test pit logs (Appendix A). Results of the soil laboratory tests are in Appendix B. Explorations completed by others are in Appendix C.

The explorations performed for this study reveal subsurface conditions only at discrete locations across the project site; actual conditions in other areas could vary. Furthermore, the nature and extent of any variations may not become evident until additional explorations are performed or until construction activities have begun. If significant variations are observed at that time, we may need to modify our conclusions and recommendations accordingly to reflect actual site conditions.

3.1 Soil Conditions

We found seven main soil types in explorations along the trail alignment. The locations and depths of each soil type are shown on stick logs for each exploration on Figures 3 through 9. Soil types encountered are described below in the general order (from the surface down) in which they were encountered, and starting from the northeast explorations and moving to the southwest. Simplified names are used in the following descriptions and in the text of this report. Because the soils varied and grade changed along the relatively long trail, we interpret and generalize some of the results. Appendices A and C contain more detailed and specific subsurface information.

3.1.1 Surficial Organic Soil

We encountered Surficial Organic Soil (or Forest Duff) at the ground surface in most of our explorations along the alignment. In general, this layer of loose fill containing organics varied in thickness along the trail alignment between approximately 6 inches and 3 feet. This material is not suitable for support of any structural elements.

3.1.2 Fill or Disturbed Native Soil

Historic fill and Disturbed Native soils across the site varied significantly in density and material. Fill has been placed along the proposed trail at different times and for different purposes, so it will likely be highly variable across the project alignment. Previous site development included fill for the railroad subgrade, which ran over old State Route 410, across the Boise Creek Bridge, and up to road elevation above the Boise Creek Bridge arch; foundation fill for the trestle approach to the old White River Bridge; and embankment fill for the South approach.

Generally, the Fill and Disturbed Native soils ranged from loose to medium dense silty, gravelly Sand and silty, sandy Gravel. These fill soils may be suitable to support proposed pavement sections if they are adequately dense in a natural state, or can be compacted in place to achieve a suitable pavement subgrade (see the Structural Fill section of this report). They would not be suitable for footing support.

3.1.3 Medium Dense Silty, Gravelly Sand

We encountered a near-surface medium dense to dense silty, gravelly sand in most test pits along the trail alignment, as well as in BA-1 (at the south end of the Boise Creek Bridge). In general, we found this layer beneath surficial organic soils and the fill or disturbed native soil. We generally encountered this layer to be about 1 to 5 feet thick except at the location of boring BA-1 where it was 10 feet thick. Figure 10 shows the stratigraphy of the Boise Creek Bridge and the variations in this layer across this portion of the project site. This unit would be suitable for on-grade pavement or footing support provided that it can be compacted in-place to a dense condition.

3.1.4 Osceola Mudflow

The Osceola Mudflow originated during a period of Mount Rainier eruptions approximately 5,600 years ago. Lahar flows resulting from the eruption descended down the White River past present-day Enumclaw. We encountered deposits from the Osceola Mudflow in the four sonic borings ranging in thickness from 24 to 53 feet. The Osceola Mudflow is predominately composed of medium dense to very dense silty/clayey gravelly Sand to sandy Gravel with scattered cobbles. Depositional forces of the mudflow resulted in highly non-uniform distribution of soil particles. Physical and engineering properties of this unit likely vary at different locations of the site. We performed laboratory tests on mudflow samples; results are in Appendix B. This material is not expected to be encountered directly beneath pavement sections or footings.

3.1.5 Loose to Medium Dense Sandy Gravel (Pit Run Fill)

Pit Run fill material was encountered near the old buried railroad bridge abutment where the trail alignment turns southwest to cross the Boise Creek Bridge. The Pit Run material was loose to medium dense sandy Gravel between 5 and 8 feet thick. This near surface material will be the primary soil encountered during grading cuts for the Boise Creek crossing approach. This soil unit would be suitable for on-grade pavement support provided that it can be compacted in-place to a dense condition.

3.1.6 Dense Silty, Sandy Gravel

This layer consists of dense to very dense silty, sandy Gravel to very sandy Gravel. We encountered this unit in general beneath the Osceola mudflow and medium dense sand units. Some near-surface deposits of dense gravel were observed in the trail approach to the proposed White River Bridge abutment. Thickness of this unit ranges from 1 to 13 feet. It would generally be suitable for on-grade pavement and embankment fill support.

3.1.7 Very Dense Slightly Silty Sand

The sonic borings on either side of the White River Bridge encountered a very dense, slightly silty to slightly gravelly Sand below both the Osceola mudflow and the dense gravel units. This was the deepest unit encountered during this project's exploration program and is interpreted to be glacially overconsolidated. This material is not expected to be encountered directly beneath pavement sections or footings.

3.2 Groundwater Conditions

Groundwater levels across the site varied between the higher-elevation trail section in the northeast and the lower-elevation White River section at the southwest end. In general, groundwater was not encountered in the test pit explorations along the majority of the trail alignment. However, we encountered groundwater at various depths during drilling at the Boise Creek Bridge and White River crossing locations.

Note that measured groundwater levels are representative of the times that the measurements were taken. Fluctuations in groundwater levels may be caused by variations in rainfall, temperature, seasons, and other factors. Also, the duration that a test pit remains open may affect the volume of water that flows into the excavation.

3.2.1 Old Railroad Grade Trail Alignment

We observed no groundwater in the test pits or BS-1 advanced along the northeast portion of the trail. These explorations were relatively shallow and did not penetrate the deeper strata beneath the elevation of the existing creek.

3.2.2 Boise Creek Bridge to Mud Mountain Road

We encountered a deeper groundwater zone below elevations 650 and 652 feet in borings BA-1 and BS-2, respectively (on either side of the Boise Creek Bridge). This corresponds to approximately 35 feet below the bridge roadway surface, or about 10 feet below the bottom of the existing bridge foundations. Perched groundwater seepage was also encountered at the time of drilling atop the buried, old SR 410 concrete pavement surface in BS-2. We did not observe groundwater seepage in the test pits on the Nagel property southwest of the bridge.

3.2.3 Mud Mountain Road to White River Bridge

No groundwater was observed in either the tests pits or the relatively shallow auger boring (BA-2) within the north approach to the White River Bridge abutments. However, we did encounter multiple water levels in the deeper sonic borings BS-3 and BS-4 at the abutment locations. BS-3 (on north side of the river) encountered perched groundwater at a depth of 14 feet (above the Osceola Mudflow) and artesian groundwater zones within and below the mudflow at depths of 32 and 68 feet, respectively. BS-4 (on south side of the river) encountered perched groundwater at a depth of 20 feet (above the Osceola Mudflow), and artesian groundwater zones within and below the mudflow at depths of 48 and 63 feet. The artesian groundwater zone within the mudflow deposit correspond to an elevation of 600 feet on both sides of the river. The lower artesian groundwater zone is confined by the bottom of the mudflow deposit at elevations ranging from 564 to 585 feet on the north and south sides of the river, respectively.

We installed three vibrating wire piezometers in each of the river abutment explorations to verify groundwater levels and monitor the water pressure fluctuations at various depths over time. Figure 11 shows the hydraulic head measured at the various elevations spanning the one to two week monitoring period. One of the installed piezometers (75-foot depth in BS-3) was malfunctioning after installation and is therefore not shown on Figure 11. The recorded data indicate that the artesian groundwater zones within (and perhaps also below) the mudflow deposit may be interconnected, resulting in a similar hydraulic head at the various levels measured (above the groundsurface near the river).

4.0 SEISMIC CONSIDERATIONS

The site is in a seismically active area. In this section, we describe the seismic setting at the project site and discuss seismically induced geotechnical hazards.

4.1 Seismic Setting

The seismicity of western Washington is dominated by the Cascadia Subduction Zone (CSZ), in which the offshore Juan de Fuca plate is subducting beneath the continental North American plate. Three types of earthquakes are typically associated with subduction zone environments: interface subduction, intraslab subduction, and crustal. Seismic records in the Puget Sound area clearly indicate a distinct shallow zone of crustal seismicity (e.g., the Seattle Fault) that may have surficial expressions and can extend to depths of up to 15 to 18 miles. A deeper zone is associated with the subducting Juan de Fuca plate. This deeper zone produces intraslab subduction earthquakes at depths of 24 to 42 miles beneath the Puget Sound region (e.g., the 1949, 1965, and 2001 earthquakes) and interface subduction earthquakes at shallow depths near the Washington coast (e.g., the 1700 earthquake, with an approximate magnitude of 9.0).

4.2 Seismic Parameters

To evaluate the seismic stability of slopes and liquefaction potential of soil, the appropriate hazard level must be selected to estimate the peak ground acceleration (PGA) associated with a design earthquake event (according to governing code or design criteria). The USGS predicted PGA is highly dependent on selection of an appropriate seismic hazard level, i.e., earthquake return period. The longer the return period assumed in the design, the larger the resulting earthquake magnitude input and seismic force used in the design.

Although a structural design is not yet available for the White River bridge crossing, we have assumed that the relocated bridge main structural project elements for this trail project should be designed in accordance with the American Association of State Highway and Transportation Officials (AASHTO) LRFD Bridge Design Specifications (AASHTO 2014). The basis of design for AASHTO is the event with a 7 percent probability of exceedance in 75 years, which corresponds to a return period of 975 years. We obtained parameters for this event (Table 1) from the US Seismic Design Maps web application (http://earthquake.usgs.gov/designmaps/us/application.php; USGS 2008), accessed on September 23, 2015. If needed, these parameters may also be used for the Boise Creek Bridge evaluation.

Table 1 – Seismic Design Parameters According to 2014 AASHTO

Parameter	Value	
Latitude	47.175 °N	
Longitude	122.019 °W	
Site class – North White River bridge abutment (BS-3)	C	
Site class – South White River bridge abutment (BS-4)	D	
Site class – Boise Creek bridge (BA-1)	D	
Spectral response acceleration at short periods, S_S	0.824 g	
Spectral response acceleration at 1-second periods, S_1	0.271 g	
Mapped peak ground acceleration, PGA	0.370 g	
	Site Class C	**Site Class D**
Site coefficient at short periods, F_a	1.070	1.170
Site coefficient at 1-second periods, F_v	1.529	1.858
Seismic coefficient, F_{PGA}	1.030	1.130

4.3 Seismically Induced Geotechnical Hazards

Potential seismically induced geotechnical hazards along the trail alignment we considered were surface rupture, liquefaction, lateral spreading, and landslides. Our assessment of these hazards is based on the soils encountered in our explorations, regional experience, and our knowledge of local seismicity.

4.3.1 Surface Rupture

We are not aware of any known faults that intersect the trail alignment, so we consider the potential for surface rupture to be very small. Rather than attempting to design against potential surface rupture, it would be reasonable to plan to repair any damage potential surface rupture may cause.

4.3.2 Liquefaction Potential

When cyclic loading occurs during an earthquake, the shaking can increase the pore pressure in loose to medium dense saturated sand and silt, and certain low-plasticity clay. Increased shaking results in liquefaction and temporary loss of soil strength. This can lead to surface settlement, lateral spreading, or slope displacement, depending on the site-specific topography.

To estimate liquefaction potential at this site, we used empirical methods (i.e., simplified procedures). The loose to medium dense soil layers that would be potentially susceptible to liquefaction at this site are all above the groundwater levels encountered at the time of drilling. Therefore, we do not expect liquefaction at this site.

4.3.3 Lateral Spreading

Based on the topography and anticipated soil conditions along the trail alignment, the potential for lateral spreading is very small and does not warrant special design considerations.

4.3.4 Seismically Induced Landsliding

In general, some potential for seismically induced landslides exists for any relatively steep slope. As discussed later in this report, we analyzed slope stability at selected critical locations along the trail alignment to evaluate the stability of nearby steep slopes for both static and seismic conditions.

5.0 GEOTECHNICAL ENGINEERING RECOMMENDATIONS

This section presents our conclusions and recommendations for geotechnical aspects of design and construction along the trail alignment. We developed our engineering analyses and provide these geotechnical recommendations based on our current understanding of the project and trail alignment, subsurface conditions encountered by our explorations at discrete locations, and laboratory tests. If the nature or location of the proposed trail or bridges are different than we have assumed, we should be notified so we can confirm or re-evaluate our analyses and recommendations.

5.1 Site Preparation

The trail alignment generally runs through areas that are currently undeveloped; some segments are moderately wooded. Initial site preparation will involve stripping and grubbing existing vegetation and visible organic material and overexcavating unsuitable soils. We estimate that the stripping depth will vary from less than a foot up to a few feet, depending on the thickness of root material and organic matter at specific locations. Generally, visible organic material (sod, humus, roots larger than 1/4 inch in diameter, and/or other decaying plant material), debris, and other unsuitable materials should be removed from the subgrade areas. Material in areas of historical fill will be highly variable. Buried railroad structures and trestle timber and footings may be encountered in the old railroad alignment and the White River Bridge approach, respectively.

Along most of the alignment, topsoil, sod, or forest duff was typically encountered to a depth of 6 inches to 3 feet; however, organic matter and unsuitable fill material was encountered in variable amounts below this depth in several borings and test pits. A Hart Crowser representative should be on-site during construction to assess the required stripping depth during construction.

Site preparation should provide a firm and unyielding subgrade beneath on-grade trail portions, retaining walls, and embankment fill areas. Structural or pavement subgrade areas should be compacted to a firm and unyielding surface and should be observed and approved by a Hart Crowser representative. The prepared subgrade should be inspected for soft areas, if necessary, by observing or proof-rolling with a fully loaded tandem-axle dump truck. Any identified soft areas should be overexcavated to a firm subgrade and backfilled with properly compacted structural fill under the observation of a Hart Crowser field representative. Alternatively, it may be possible to bridge soft subgrade areas with a suitable large-aggregate material such as quarry spalls, riprap, or ballast rock in combination with a suitable geotextile, if necessary. Below structural foundation elements (or rockeries), such overexcavation should extend a distance outside the edge of the footing (or rockery) equal to the depth of the overexcavation.

It may be necessary to relocate or abandon some utilities within the construction area. Abandoned underground utilities should be removed or completely grouted. The ends of remaining abandoned utility lines should be sealed to keep soil and water out. Soft or loose backfill materials should be removed and the area backfilled according to the structural fill recommendations in this report. Coordination with the utility owners is generally required when addressing existing utilities.

Any existing concrete elements that are encountered should be removed if they are within 2 feet vertically of the bottom of pavement sections. The purpose of this is to eliminate "hard spots" that could lead to "bumps" in the trail surface.

5.2 Retaining Walls

At the time of this report, the project is still in the preliminary planning phase and the final requirements for the wall system has not yet been determined. Therefore, we provide recommendations for both Mechanically Stabilize Earth (MSE) walls and cast-in-place (CIP) concrete cantilevered walls. We recommend consulting Hart Crowser further when design progresses and the final wall types are selected. At that time, we will be able to refine our design recommendations and provide further design assistance.

The type of retaining wall to use depends on numerous factors including construction access, cost, and aesthetics. The advantages and disadvantages of each system should be carefully weighed to account for cost and construction benefits that may be lost or gained with the selection of a specific retaining system for the project.

5.2.1 Mechanically Stabilized Earth Walls

Construction of MSE walls generally consists of compacting a block or mass of soil in lifts with reinforcing strips in between. The reinforcing strips are connected to wall-facing panels or vegetation baskets along the face of the wall. Numerous systems are available that generally use the same basic design principle. The successful construction and performance of MSE walls depends on several factors, such as:

- Suitability of supporting subgrade soils;
- Presence and quantity of water and the ability to drain water from behind the wall;
- Type, length, and spacing of reinforcement strips used;
- Type and installation method of wall facing;
- Surcharge loads and compaction effort near the wall face during construction;
- Consistency of the fill soil; and
- Attention to construction details, especially the connection of the facing to the reinforcement strips.

5.2.1.1 MSE Wall Design

It is typically economical for the vendor of the MSE wall materials to design the MSE wall for internal stability, with our input and review. We recommend the following for MSE wall design:

- Design the MSE walls in general accordance with the Federal Highway Administration (FHWA) design manual, Mechanically Stabilized Earth Walls and Reinforced Soil Slope Design and Construction Guidelines (Publication FHWA-NHI-00-043, March 2001). This publication is available online (http://www.fhwa.dot.gov/engineering/geotech/library_listing.cfm).

- Design the length and spacing of reinforcing layers so that the MSE wall does not slide or overturn; maintains its bearing capacity; resists overall slope instability; and remains internally stable (that is, reinforcement does not break or pull out).

- Use a soil friction angle of 35 degrees and a unit weight of 125 pounds per cubic foot (pcf) for the compacted structural fill that makes up the MSE wall. These values assume granular fill, free of organic material, placed and compacted to the degree presented in the Structural Fill section of this report. The frictional strength of the fill material will need to be determined early in the MSE design stage. Soil properties should be confirmed and the design modified, if necessary, once actual fill materials are identified.

- For lateral earth pressure acting on the reinforced soil prism of the MSE wall system, use an equivalent fluid density (EFD) of 35 pcf, assuming level backfill and active soil pressure conditions for yielding wall systems (minimum wall movement of about 0.001 times the height of the wall).

- Design the MSE facing members for an appropriate lateral surcharge condition resulting from the anticipated occasional traffic loading from emergency vehicles. This can be modeled as an additional 2 feet of vertical soil surcharge or a uniform lateral surcharge pressure on the back side of the wall facing of 75 pounds per square foot (psf).

- Seismic surcharge loads will act over the entire back of the MSE wall and vary with the backslope inclination, the design PGA, and the wall height. Assuming a level ground surface behind the wall and a design PGA of 0.190g, use a uniform horizontal seismic pressure of 8H psf (assuming a yielding wall), where H is the height of the wall.

- To improve stability and reduce risk of future erosion, use a minimum MSE wall embedment depth of 1 foot below the ground surface.

We recommend retaining Hart Crowser to review the MSE wall designer's design calculations, specifications, and plans for conformance with geotechnical recommendations. Hart Crowser can also help analyze global stability for the MSE wall system if the vendor does not analyze it.

5.2.1.2 MSE Wall Drainage

To provide adequate drainage and reduce the risk of hydrostatic pressure on the MSE wall, the wall backfill should consist of free-draining sand or sand and gravel with less than 3 percent by weight passing the No. 200 mesh sieve, based on the minus 3/4-inch fraction of the material. If excavated on-site soil (which may not be free-draining) will be used within the MSE backfill zone, we recommend placing a curtain drain of at least 18 inches of imported, free-draining soil directly behind the MSE face blocks, and as a blanket drain behind the reinforced soil zone. Gravel borrow as described in WSDOT

Standard Specifications, Section 9-03.14(1), with the added requirement that fines content should not exceed 3 percent (based on the minus ¾-inch fraction of the material) may be used for this purpose.

5.2.2 Cast-in-Place Concrete Cantilever Walls

This section addresses geotechnical recommendations for cast-in-place (CIP) concrete cantilever retaining walls, in case they are included in the design.

5.2.2.1 CIP Wall Foundations

We recommend supporting retaining walls on shallow foundations (footings), as follows:

- Design and construct footings to bear on medium dense to very dense natural granular soils that do not contain organic material and that are compacted to a dense condition during construction, or on compacted structural fill placed immediately above these natural soils.

- Use a maximum allowable bearing pressure of 2,500 pounds per square foot (psf) for the prescribed bearing surface. This allowable bearing pressure may be increased by up to one-third for loads of short duration (e.g., wind or seismic loads).

- Design footings with a minimum embedment depth of 18 inches below ground surface and a minimum width of 3 feet.

- Use an allowable coefficient of friction of 0.3 (with a resistance factor of 0.67) for CIP footings placed on the prescribed dense granular bearing soil. A higher allowable base friction coefficient value of 0.4 may be used for design if a minimum 6-inch-thick layer of compacted crushed rock is placed below the face blocks.

5.2.2.2 CIP Lateral Earth Pressures

Lateral earth pressures on backfilled retaining walls depend on the ability of the wall to deform. If the top of the wall is allowed to yield about 0.001 to 0.002 times the height, and if no settlement-sensitive structures or utilities are located in the zone of deformation, the wall may be designed using active earth pressures. If settlement-sensitive structures or utilities exist within the potential zone of deformation, or where the wall system is too stiff to allow sufficient lateral movement to develop an active condition, at-rest earth pressures should be used to design the wall.

To estimate lateral pressures acting on the wall, we recommend:

- Use an active or at-rest earth pressure represented by an EFD of 35 and 55 pcf, respectively, for walls backfilled with well-compacted structural fill. These values assume level backfill and a fully drained condition behind the wall (no buildup of hydrostatic pressure).

- Seismic surcharge loads on backfilled walls act over the entire back of the wall and vary with the backslope inclination, the design PGA, and the wall height. With a level ground surface behind the wall and a PGA of 0.19g we recommend using a uniform, horizontal seismic pressure of 8H psf (assuming a yielding wall condition) where H is the height of the wall.

- Where necessary, design walls to support surcharge loading from adjacent structures (including other walls), traffic loading, or other surface loads located within a horizontal distance equal to the wall height. The anticipated occasional traffic loading from emergency vehicles can be modeled as an additional 2 feet of vertical soil surcharge, or as a uniform lateral surcharge pressure on the back side of the wall facing of 75 psf. Evaluate surcharge loads from adjacent structures case by case. We can help evaluate if needed.

5.2.2.3 CIP Wall Backfill

Backfill soil should consist of structural fill placed in 8- to 10-inch thick loose lifts and compacted to at least 95 percent of the maximum dry density as determined by the modified Proctor test method (ASTM D1557). Within 2 feet of the wall, soil should be compacted with small, hand-operated equipment to avoid imparting excess horizontal stresses on the wall. Within this zone, compaction may be reduced to 92 percent.

5.2.2.4 CIP Wall Drainage

To reduce the risk of potential hydrostatic pressure buildup, we recommend using a free-draining granular material (less than 3 percent passing the US No. 200 sieve based on the minus ¾-inch fraction of the material) as structural fill within an 18-inch-wide zone immediately behind the wall. This curtain drain should be continuous and hydraulically connected to a footing drain collection system at the base of the wall. The free-draining material should be capped with at least 6 inches of fine-grained soil at the surface (or impervious pavement) to reduce surface water infiltration into the subsurface drainage system.

The wall footing drain system should incorporate a minimum 4-inch-diameter perforated drain pipe surrounded by at least 6 inches of free-draining material. The drain pipe should have cleanouts, and the drain holes or slots should be compatible with the surrounding drainage material. Drainage should discharge to a municipal storm drain, sewer system, or other suitable location by gravity flow.

5.3 On-Grade Trail Pavement

We understand that the on-grade portions of the trail will have an asphaltic concrete pavement (ACP) surface. The planned pavement section can likely be supported on the near-surface soils anticipated along most of the alignment, provided that the soils are primarily granular (i.e., silty sand, sand, or and/gravel) and that the trail subgrade areas are prepared as recommended in this report. If moisture-sensitive, fine-grained soils (i.e., sandy silt, silt, or clay) are encountered in pavement subgrade areas, they may need to be overexcavated and replaced with structural fill if found to be yielding during proof-rolling, as discussed in Section 5.1.

Assuming well-compacted, granular fill soil or structural fill subgrade, the asphalt pavement section of the on-grade trail should consist of 2 inches ACP over 4 inches of crushed surfacing base course (CSBC) for light-duty traffic including occasional emergency vehicles. Class B asphalt is typically suitable for ACP courses. The CSBC layer should consist of imported crushed surfacing top course or base course according to WSDOT Standard Specifications, Section 9-03.9(3). We generally recommend against the use of recycled or pulverized concrete as CSBC.

Specific pavement subgrade preparation includes removal of surficial organic material and compaction of near-surface granular subgrade soil to a minimum density of 95 percent of the maximum dry density as determined by the modified Proctor method (ASTM D1557). The subgrade should then be proof-rolled with a loaded dump truck to verify a firm and unyielding subgrade. Any localized zones of yielding subgrade disclosed during proof-rolling should be overexcavated to a depth to be determined in the field, and replaced with compacted structural fill (granular subbase course). A suitable geofabric may be required to stabilize the soft subgrade below the overexcavation and minimize silt migration into the structural fill and pavement section, based on a field evaluation of subgrade conditions. Overexcavation below pavement subgrade areas should extend a distance outside the edge of the pavement equal to the depth of the overexcavation.

5.4 Slope Stability Evaluation

To evaluate the stability at critical locations along the steep slope portions of the proposed trail alignment, we completed slope stability analyses using the Slope/W computer program, which uses limit equilibrium methods. Stability was evaluated using the Morgenstern-Price analysis method, which satisfies both moment and force equilibrium. Slip surfaces were defined using the auto-locate feature of the program to select a circular slip surface with the lowest factor of safety. We ignored surficial slip surfaces (up to 3 feet deep) in our analyses, as we considered them part of the surficial slope erosion/sloughing process and not likely to significantly impact trail system function. Ignoring these surfaces allows the slope stability analysis to focus on identifying potential deeper slope failures and associated safety factors more critical to the trail design.

Generally, minimum static and seismic slope stability factors of safety (FS) of 1.4 and 1.1 are typically used as appropriate design criteria for trail projects. However, for the trail location and subsurface conditions analyzed for this project, we recommend using a minimum static FS of 1.3, which follows the guidelines in the Washington State Department of Transportation (WSDOT) Geotechnical Design Manual (GDM), Section 7.4. This design standard allows for an FS of 1.3 static for slopes adjacent to, but not supporting, structures. Given this portion of the trail location is in a rural environment and is not supporting a structure, we believe this slightly lower FS is justified in this specific design case.

The seismic analysis case models the dynamic impact of the earthquake as a pseudo-static force based on the expected PGA as described in the Seismic Parameters section of this report. The PGA used in the seismic (dynamic) analysis case depends on the design earthquake hazard. For the 975-year return period we used a pseudo-static force equal to 0.19 g (half of the PGA multiplied by the site coefficient and an alpha value of 0.963).

5.4.1 Slope Stability at the Encroaching Boise Creek Trail Location

The primary slope stability concern along the steep slope portion of the trail occurs where Boise Creek encroaches on the existing walking trail (approximate station 137+50 to 138+50). The creek is actively undercutting the steep slope adjacent to this trail location. This has resulted in a near-vertical, exposed soil face approximately 16 feet high above the creek level (about 14 feet from top of slope). Slope stability analyses served two goals: (1) to determine the feasibility of designing the trail along the existing alignment and within the required factor of safety, and (2) to establish setback criteria if the existing alignment is determined to be unacceptable. We analyzed both situations under static and pseudostatic loading conditions.

Results of these slope stability analyses are shown on Figures 12 through 14. For the currently proposed trail alignment and slope geometry (Figure 12), our analyses indicate inadequate stability under both static and seismic loading conditions. Therefore, we recommend against constructing the trail as shown along the old railroad alignment.

To evaluate potential other safe trail locations relative to the undercut portion of the trail, we analyzed several cases for different cut setbacks and elevations. Possible trail locations are limited by both the close proximity to the near-vertical slope on the Boise Creek side and an old City of Tacoma water main pipeline roughly 100 feet to the northwest (parallel to State Route 410). Due to the unknown condition of this old pipeline and the potential for vibration damage from construction activity, we understand that the trail will not be planned closer than 30 feet to this old water line. Potential effects of construction on the pipeline should be considered and accounted for by the contractor.

The results of our slope stability analyses indicate that the trail will need to be relocated approximately 60 feet back from the edge of the slope at the current trail elevation, to maintain adequate, minimum static and seismic safety factors relative to the near-vertical slope face on the Boise Creek side. At this location the trail grade will be lowered from its existing elevation as it approaches the Boise Creek Bridge crossing. In order to maintain the trail at the design elevation, a retaining wall up to 12 feet tall may be required as shown in Figures 13 and 14. We recommend a 2V:1H slope from the top of the retaining wall to the existing ground surface to satisfy the 30 foot exclusion criteria from the pipeline.

Although the mudflow deposit appears to contain some clay and therefore likely exhibits cohesive strength characteristics, we did not rely on this in our slope stability analysis (i.e. analysis based only on soil friction). The primarily reason for this was the uncertainty of estimating the cohesive component of the soil strength for the mudflow deposit, given the inconsistent depositional nature of this soil unit. A second reason is that the current stability of the near-vertical, undercut slope face may be due to apparent cohesion within the mudflow, which cannot be relied upon for long-term stability calculations. If the trail location suggested by our current slope stability evaluation is not feasible, it may be possible to perform further exploration and soil laboratory testing (as suggested in Section 6) to determine the cohesive soil strength with more certainty. This may allow a higher soil strength to be used in the slope stability analysis, which in turn would result in higher slope stability safety factors and may allow the trail to be located closer to the originally proposed alignment.

5.4.2 Slope Stability along Remainder of Trail Alignment

Other than the Boise Creek encroachment trail location discussed above, the proposed trail along the old railroad grade is generally far enough away from the steep slope area along Boise Creek to be considered adequately stable. However, the proposed trail portion between stations 130+00 and 131+50 appears to be a fill embankment constructed across a former creek crossing, with relatively steep side slopes (on the order of 1H:1V). Based on the subsurface conditions observed in TP-6, this fill embankment is likely constructed of loose to medium dense pit run fill. This existing embankment trail portion is not considered adequately stable in its current configuration. We recommend that this portion of the trail (and others with a similar side slope configuration) be regraded to meet the 2H:1V permanent slope configuration recommended in the Permanent Slope section of this report. This can be accomplished by adding fill to the existing sideslopes (keyed in a stair-step fashion) or by lowering the trail elevation to below a projected 2H:1V plane from the toe of the existing embankment slopes.

We understand the remaining portions of the trail as they exist are (or will be) within our recommendations for stable permanent cuts and slopes. Periodic slope erosion may occur along portions of the trail over the design life of the project. Such potential future slope erosion may require additional trail maintenance or slope stability analysis.

5.5 Existing Concrete Railroad Abutment

The existing railroad abutment northwest of Boise Creek Bridge (buried when SR 410 was backfilled) will be partially exposed when cuts are made to ramp down the north trail approach to the bridge level on the northeast side of the abutment. As such, a structural stability evaluation of this abutment will be required. The following sections provide a summary of our field observations of abutment geometry and backfill conditions, along with geotechnical design recommendations for lateral resistance of the abutment.

5.5.1 Abutment Backfill Material

We observed the abutment backfill material in three test pits (TP-10 through TP-12) excavated to depths ranging from 8 to 14 feet around the existing railroad abutment (see figure 15 for more detailed exploration locations). Test pits TP-10 and TP-11 on southwest side (originally backfilled side of railroad bridge abutment) encountered a loose, sandy Gravel (pit run fill) in the upper 8 feet, underlain by a loose to medium dense, silty, gravelly Sand fill, which extended to the maximum explored depth of 14 feet. Test pit TP-12 on northeast side (originally exposed side of railroad bridge abutment) encountered a loose, sandy Gravel (pit run fill) to the maximum depth explored of 8 feet. The test pit depth was limited by excessive caving during excavation. However, based on subsurface conditions observed in nearby boring BS-2 (about 30 feet to the northeast), the loose pit run backfill likely extends downward to the original SR 410 road surface below the explored depth of TP-12.

5.5.2 Abutment Geometry

We observed the geometry of the abutment in the test pit excavations, as depicted on Figure 15. Test pits TP-10 and TP-12 were dug along the main abutment structure, whereas TP-11 was excavated along the southern wing wall. The upper portion of the main abutment wall appears to be about 4 feet thick, while the bottom portion steps out to about 6.5 feet thickness at a depth of 12 feet. Figure 15 shows approximate plan and section views of the abutment, based on hand measurements in the field.

While it was not feasible to excavate deep enough to reveal the base of the abutment, it is likely that the abutment was originally extended deep enough to provide at least a minimal embedment depth below the old SR 410 road surface, and into medium dense to dense, native bearing soils (as was observed near the bottom of TP-10 and TP-11). Based on the soil stratigraphy and old road surface observed in boring BS-2, and assuming a minimum 2-foot embedment below the old road surface, the overall abutment height may be on the order of 17 feet, or more. Our field observations are generally consistent with old King County design drawings for this RR crossing, which shows an abutment height of about 15 feet above the old road surface, and an embedment of 4 feet below the old road surface.

5.5.3 Abutment Lateral Resistance

Resistance against lateral sliding of subsurface structures is typically provided by a combination of passive earth pressure and frictional resistance along the base. We make the following geotechnical recommendations for lateral stability analysis of the existing concrete railroad abutment (assuming LRFD analysis method), based on estimated strength properties for the fill material encountered in the adjacent test pits and nearby sonic boring:

- For the active earth pressures on the abutment, use an EFD of 50 psf in a triangular pressure distribution. A load factor of 1.5 has been applied to this value.

- For the passive earth pressure resistance on the abutment, use an EFD of 350 psf in a triangular pressure distribution (assuming level ground conditions). A resistance factor of 0.75 has been applied to this value. The height of soil assumed for passive resistance in front of the abutment should be determined after final trail elevations have been designed.

- Use an allowable coefficient of friction of 0.2 to resist sliding along the base of the existing abutment. This includes a resistance factor of 0.67, and is conservatively estimated for a smooth concrete surface to account for the unknown nature of the original abutment construction method.

5.6 Boise Creek Arch Bridge

Our understanding of the Boise Creek Arch Bridge is based on our field observations and the original King County as-built structural drawings dated June 29, 1915. The drawings indicate that the bridge concrete arch is supported by strip footings measuring 7 feet wide by 24 feet long. For the bridge to be reused as a trail crossing, a structural evaluation is required. An estimate of the bearing capacity and passive resistance of the existing bridge footings will be needed for this evaluation, along with the estimated density of the soil fill within the bridge core above the concrete arch.

Field investigation of the arch bridge led to the discovery that the north footing of the bridge has been substantially undermined by creek scour. The undermined footing appears to have been poured at a higher elevation than that shown on the as-built drawings. In order to rely on this footing for future structural support, remediation of the existing footing needs to occur. This can be accomplished through supporting the existing footing element with an extended concrete pier or deep foundations embedded into suitable bearing soils below the creek level (to account for potential future scour) or by effectively widening the footing with additional footing elements that are structurally connected to the existing footing. Soil ground improvement of the bearing material may also be required.

For the structural evaluation of the existing Boise Creek Bridge, we recommend the following geotechnical design parameters (assuming LRFD analysis method):

- For the strength limit state, use a maximum allowable soil bearing pressure of 5.5 kips per square foot (ksf) for the existing footings. This value includes a resistance factor of 0.45.
- For extreme limit states (seismic and impact forces), use an allowable bearing pressure of 11 ksf. This includes a resistance factor of 0.9.
- Use an allowable coefficient of friction of 0.3 to resist sliding for the existing footings. This includes a resistance factor of 0.67.
- Use a fill soil density of 125 pcf when evaluating the loading on the concrete bridge arch.
- For the passive resistance finite element modelling required to evaluate the lateral stability of the concrete arch, use the spring constant (load vs. deflection) depicted in Figure 16. This is based on a Hyperbolic Force-Displacement (HFD) curve fitting model derived from field testing and numerical modeling. It shows the relationship between deflection of the footing and the associated passive resistance force of the soil body on both the footing and the structural arch. For the associated passive soil resistance value (when soil resistance is fully mobilized), we recommend using a passive EFD of 600 pcf in a triangular pressure distribution. No resistance factor has been applied to this value, assuming service limit state deflection analysis.

If higher allowable soil bearing pressure is required it may be possible to increase the allowable bearing pressure through refinement of the geotechnical parameters for the in-situ soil. Additional subsurface explorations on or immediately adjacent to the northern footing would be necessary to assess this possibility. In this case an increased bearing pressure would only be realized if the strength properties of the soils encountered in the new boring were higher than those assumed using the currently available information Alternatively it may be feasible to retrofit the existing footing with supplemental foundation elements as described above.

5.7 White River Bridge

The proposed trail crossing over the White River will consist of the old SR 167 Puyallup River bridge superstructure placed on newly constructed abutments. At the time this report was written, the structural engineer had not yet been selected, and the loads on the foundation elements were not available. However, Hart Crowser was the geotechnical engineer for the temporary foundation support system for the old bridge. Based on this experience, we assumed an unfactored load of 4800 kips per abutment for our current evaluation. When actual design loads have been determined, the foundation design should be re-evaluated based on updated bridge loading criteria.

We initially examined the feasibility of both a deep foundation system (driven open-ended steel pipe piles or drilled shafts) and a shallow mat foundation system. However, the presence of abundant cobbles and scattered boulders within the very dense mudflow deposit (up to 68 feet deep), along with artesian groundwater conditions, could pose significant construction challenges for a deep foundation system. Driving piles through cobbles and boulders can be extremely difficult and risks damaging the piles or premature refusal. Additionally, the artesian groundwater conditions encountered in the borings could result in soil heave and other challenges during drilled shaft installation. Given this, and since soil liquefaction is not a foundation design concern at this site, we recommend use of a shallow mat foundation system as the most appropriate and cost-effective foundation system for the proposed bridge.

5.7.1 Mat Foundations

Mat foundations require proper embedment on suitable bearing material to provide uniform and suitable bearing resistance. On the north side of the White River, suitable abutment mat foundation bearing soils (very dense, silty, sandy Gravel) are is expected at about 3 feet below the existing ground surface. However, the south White River abutment will be located near the end of the existing embankment, which is composed of loose to medium dense fill of unknown quality and uniformity, and therefore not dependable for foundation bearing support. Suitable bearing soils (very dense, sandy Gravel) are anticipated at a depth of 19 feet below the top of the existing embankment at this location, or about 6 feet below the ground surface east of the existing old abutment. Consequently, temporary excavation of the end of the existing embankment will be required to install the south White River abutment mat foundation.

For design and construction of the mat footings (assuming LRFD analysis method), we recommend:

- Design mats so that:
  - Width of the mat is equal to the proposed width of the bridge superstructure.
  - All footings have a minimum embedment depth of 24 inches below the lowest adjacent grade (to account for frost depth and suitable bearing material).
- For the service limit state, use a maximum allowable soil bearing pressure of 6 ksf for mat footings constructed on dense native material or compacted structural fill.
- For the strength limit state, use a maximum allowable soil bearing pressure of 35 ksf for the mat footings. This value includes a resistance factor of 0.45.
- For extreme limit states, i.e. seismic and impact forces, use an allowable bearing pressure of 70 ksf. This value includes a resistance factor of 0.9.
- Increase the allowable soil bearing pressure by up to 1/3 for loads of short duration, such as those caused by wind or seismic forces.
- For footing resistance to lateral loads, use an EFD to represent the passive resistance of the soil. For footings poured against neat cut dense native soil or compacted structural fill we recommend an allowable passive EFD of 300 pcf in a triangular pressure distribution. A resistance

factor of 0.5 has been applied to this value. The contribution from the uppermost 2 feet of soil should be ignored when calculating passive resistance.

- Use an allowable coefficient of friction of 0.3 to resist sliding for footings poured neat on dense granular soil or compacted granular structural fill. A resistance factor of 0.67 has been applied to this value.

- Ensure that mats bear on dense native soil or compacted structural fill. The south abutment mat foundation should be placed on the dense native material underlying the existing embankment fill.

- Backfill any excavation extending below the planned foundation elevation with lean or structural concrete.

- Before concrete for mats is placed, ensure that subgrade soil is in a very dense, non-yielding condition. Remove any disturbed soil or standing water.

- Have a Hart Crowser representative observe exposed subgrades before mat construction to verify design assumptions about subsurface conditions and subgrade preparation.

At the time this report was written, no allowable structural settlement criteria was available. Our recommendations are based on an expected post-construction mat foundation settlement of 1.5 inches. This value corresponds to the anticipated settlement due only to the weight of the bridge superstructure. Additional loading may be imposed on the foundation due to traffic loading and the earth fill embankment approach, as subsequently discussed in the White River Bridge Approach section of this report. Our foundation design recommendations should be re-evaluated when final bridge design loading conditions become available. We expect settlement to occur elastically (i.e., essentially as the loads are applied). These values assume proper subgrade preparation. Any loosening of the subgrade materials during construction could result in more settlement.

According to King County flood plain mapping (White River Work Map Zone 4, dated June 28, 2012), the trail alignment and proposed new White River bridge abutment locations are outside of the 100 year flood plain. As a result, scour analysis has not been performed. Footing recommendations are based on no anticipated scour to occur at the White River Bridge portion of the site. Should the bridge location change to be within the 100 year flood plain, additional analysis to determine the capacity and feasibility of the mat footings under scour condition will need to be performed.

For the recommended mat foundation depth and required temporary excavation depths, we do not expect significant groundwater issues to be encountered during construction. However, this may depend on the time of year and the river level.

5.8 White River Bridge Approach

Two options were considered for the approach to the north abutment of the White River Bridge. One was a structurally supported elevated deck and the other was a soil embankment. Pile foundation elements were considered for support of the elevated deck. However, test pit and boring explorations

(TP-16 through TP-18, BA-2, and BS-3) advanced between Mud Mountain Road and the White River encountered numerous randomly distributed cobbles, boulders, and abandoned concrete foundation elements from the historical trestle structure in the proposed pin pile locations. These large obstructions could result in irregular installation conditions for piles, leading to premature refusal and possible damage to the piles. Furthermore, it may not be feasible to install the piles deep enough to develop adequate lateral capacity required for the elevated structure.

5.8.1 Earth Embankment

Because of these potential constructability issues associated with a pin pile foundation design approach, an earth fill embankment is recommended as a more feasible and likely more cost-effective approach for the north bridge approach. The earth embankment would extend from the existing grade of Mud Mountain Road to the bridge deck elevation. The slopes of the embankment should follow recommendations stated in the Permanent Slope section of this report.

We generally recommend embankment slopes no steeper than 2H:1V, to minimize long-term erosion and to facilitate revegetation. However, if the County can accept a slope design FS of 1.25 and the risk of potential localized surficial erosion (requiring future maintenance), the embankment side slopes may be constructed at a slightly steeper inclination, 1.75H:1V, to reduce the project footprint and construction cost. For this steeper inclination, any utility line within the embankment should be placed below a projected 2H:1V plane from the bottom of the embankment side slope, to minimize risk of damage from potential surficial side slope instability.

Settlement of the embankment relative to the surrounding ground surface could be a controlling factor in the design. Based on an assumed 2H:1V side slope configuration and an embankment height of 16 feet and a top width of 30 feet, we estimate for preliminary design purposes that total settlement may be approximately 1 to 2 inches. The majority of this settlement is expected to be elastic, occurring during construction. For an embankment constructed at a uniform grade from Mud Mountain Road to the White River Bridge, this settlement estimate may vary slightly based on the differential fill depths along the old trestle approach. Our preliminary fill embankment settlement estimates should be re-evaluated as part of the final bridge approach design.

Near the proposed bridge abutment, we recommend staging the construction of the fill embankment as early as possible to avoid potential settlement impacts to the mat foundation after the bridge is in place. Allowing for settlement due to the embankment loading to occur prior to bridge placement will also reduce the potential risk of post-construction settlement damage to the bridge. Refer to the Mat Foundation section of this report for discussion of expected settlement of the mat foundations due to the bridge.

Because a portion of the embankment is expected to be near Boise Creek, we recommend that slope stability analysis of the east bank of Boise Creek be performed during final design. The analysis should be based on the geometry of the Boise Creek bank, the height and geometry of the new embankment, and additional subsurface boring information in this specific area. We recommend that at least one boring be advanced and a groundwater monitoring well installed at this location. The results of the slope stability analysis will be a factor in determining the final geometry and location of the new embankment relative to Boise Creek.

5.8.2 Shallow Foundations

An elevated deck supported by shallow foundations provides an alternative approach to an earth embankment. Spread footings provide a means of support with reduced constructability issues related to the cobbles, boulders, and abandoned foundation elements previously described. We recommend the following for the design and construction of spread footings:

- Footings should bear on dense native granular soil or compacted structural fill.
- Design footings so that:
  - Isolated footings have minimum dimensions necessary to support the design loads at the strength and extreme limit states. Figure 17 presents required dimensions for square footings at varying loads and limit states.
  - All footings have a minimum embedment depth of 18 inches below the lowest adjacent grade for consideration of frost depth.
- Use an increase in the allowable soil bearing pressure of up to 1/3 for loads of short duration, such as those caused by wind or seismic forces.
- Footings should be founded outside of an imaginary 1H:1V plane projected upward from the bottom edge of adjacent footings or utility trenches.
- For footing resistance to lateral loads, use an EFD to represent the passive resistance of the soil. For footings poured against neat cut dense native soil or compacted structural fill we recommend an allowable passive EFD of 300 pcf in a triangular pressure distribution. A resistance factor of 0.5 has been applied to this value. The contribution from the uppermost 2 feet of soil should be ignored when calculating passive resistance.
- Use an allowable coefficient of friction of 0.3 to resist sliding for footings poured neat on glacial soil or compacted structural fill. A resistance factor of 0.67 has been applied to this value.
- Before placing concrete for footings, subgrade soil should be in a dense, non-yielding condition. Remove any disturbed soil or standing water.
- Have a Hart Crowser representative observe exposed subgrades before footing construction to verify design assumptions about subsurface conditions and subgrade preparation.

At the time of this report information on the anticipated loading due to the elevated deck is not available. Settlements should be estimated after footing sizes and loads are available.

5.9 Rockeries

For planning purposes, rockeries up to 4 feet high may be considered on this project to protect cut slopes in native soil. The stability of all slopes behind rockeries should be assessed case by case.

For rockery design and construction, we recommend the following:

- Design rockeries in general accordance with King County standard rockery details.
- Embed the base of rockeries at least 12 inches below adjacent grade and found the lowest rockery course on undisturbed dense or compacted native soil or structural fill.
- Design and construct rockeries with an overall rock face inclination of from 1H:6V to 1H:4V.
- Provide a drain rock curtain composed of 2- to 4-inch spalls or angular gravel at least 12 inches thick directly behind the rockery, to preclude buildup of hydrostatic pressure. The drain rock should be capped with at least 6 inches of silty soil at the surface (or pavement) and should be hydraulically connected with a 4-inch-diameter perforated PVC drain pipe installed behind the lowest rock course at the base of the rockery.
- Require inspection of rockery construction to check that the basic design details are in place, including proper subgrade preparation, type of rock and rock size, rock placement, drain rock layer and drainage pipe installation, embedment, and face inclination.

5.10 Structural Fill

Structural fill is required for backfill in open cut and overexcavated areas, beneath footings and pavement, behind retaining walls, and above utility installations. The suitability of soil for structural fill depends primarily on its grain size distribution and moisture content when placed. As the fines content (fraction passing the US No. 200 sieve) increases, soil becomes more sensitive to small changes in moisture. With more than about 5 percent fines (by weight), soil cannot be consistently compacted to a firm, relatively unyielding condition when the moisture content is more than 2 percent above or below optimum. Structural fill must also be free of organic matter and other debris.

Generally, any fill material with moisture content at or near optimum can be compacted as structural fill provided it is placed on a firm and relatively unyielding subgrade surface. However, if fill is to be placed during wet weather, we recommend using clean fill, that is, soil with a fines content (fraction passing the US No. 200 sieve) of 5 percent or less (by weight). Clean fill should meet the requirements specified in Section 5.10.2.

For structural fill placement and compaction we recommend:

- Place and compact all structural fill in lifts with a loose thickness no greater than 8 to 10 inches. If small hand-operated compaction equipment is used to compact structural fill within 12 inches of utility pipes or other structures, the lifts should not exceed 4 to 6 inches in loose thickness, depending on the equipment used. The maximum particle size within the structural fill should be no more than two-thirds of the loose lift thickness to allow full compaction of the soil surrounding the large particles.

- Generally, compact structural fill that is beneath footings, behind walls, and within two feet of the bottom of pavement sections to a minimum of 95 percent of the modified Proctor maximum dry density, as determined by the ASTM D1557 test procedure.
- Structural fill that is more than 2 feet below pavement sections, and within 2 feet of the back of subgrade walls should be compacted to 92 percent.
- Within 2 feet of subgrade walls, use hand compaction equipment to avoid overstressing the wall.
- Control the moisture content of the fill to within 2 percent of the optimum moisture based on laboratory Proctor tests. The optimum moisture content corresponds to the maximum attainable Proctor dry density.
- Generally, place structural fill only on dense and relatively unyielding subgrade (see Section 5.1). If subgrade areas are wet, clean material with at least 30 to 35 percent gravel content (material coarser than a US No. 4 sieve) may be needed to bridge moisture-sensitive subsoils. In some cases, clean crushed rock or quarry spalls may be needed to stabilize weak or wet subgrade soil.
- Where free-draining material is required, such as behind retaining walls or around drainage pipes, use a well-graded sand and gravel with less than 3 percent passing the No. 200 sieve (based on the minus ¾-inch fraction of the material).
- Perform a representative number of in-place density tests to verify adequate compaction. A Hart Crowser representative should verify each structural fill lift and the subgrade area below it.
- Before using any material as structural fill, have it sampled and tested to determine its maximum dry density and gradation.

5.10.1 Use of On-Site Soil as Structural Fill

Some site soils may be suitable for reuse as structural fill. Much of the near-surface soil (below surficial organics) encountered in our explorations was moist granular soil with relatively low silt content (i.e. pit run fill), and may be suitable for use as structural fill or re-compaction given favorable weather and moisture conditions. However, most of the soil samples contained more than 5 percent fines and would thus be moisture-sensitive; such soils are difficult to compact if they are wet when excavated, become wet when stockpiled, or are placed during wet weather.

If the site soils are or become wet, it may be possible to use them as structural fill if fill is placed during the summer, and the material can be moisture-conditioned to near its optimum level. Typically, relatively long periods of dry, warm weather and large open areas to spread and rototill the soil are required for successful aeration of soil to reduce its moisture content.

Because the soils along the trail alignment vary, as will the weather, the suitability of on-site soils for use as structural fill should be determined in the field during construction. We recommend separately stockpiling the excavated soil intended for reuse as structural fill and having the on-site geotechnical engineer or geologist review it for suitability. Stockpiles should be protected with plastic sheeting so they do not get overly wet during rainy weather. On-site soil is typically not considered suitable for use as free-draining material, unless a large deposit of consistently clean (silt-free) sandy soil is found.

5.10.2 Imported Structural Fill

Imported structural fill should be well-graded sand or sand and gravel with a low fines content, free of organic and other unsuitable materials. Generally, imported structural fill for most applications should meet the requirements in WSDOT Standard Specifications, Section 9-03.14(1), with the added requirement that the fines content not exceed 5 percent.

5.11 Permanent Slopes

Permanent cut and fill slopes should be adequately inclined and revegetated to minimize long-term raveling, sloughing, and erosion. A vegetative groundcover should be established as soon as possible after grading, to further protect the slope from runoff-water erosion. Permanent slopes should not be steeper than 2H:1V, to minimize long-term erosion and to facilitate revegetation. Final grading near the top of permanent slopes should direct surface water away from the slope face.

5.12 Temporary Open Cuts

Temporary soil cuts for site excavations more than 4 feet deep should be adequately sloped back to prevent sloughing and collapse in accordance with Washington State Department of Labor & Industries (L&I) Division of Occupational Safety and Health (DOSH) guidelines (WAC Chapter 296-155 Part N). The stability and safety of cut slopes depend on a number of factors, including:

- Type and density of the soil;
- 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; and
- Care and methods used by the contractor.

Because of the variables involved, slope angles required for stability in temporary cut areas can be only estimated, not determined precisely, before construction. It is the contractor's responsibility to ensure that the excavation is properly sloped or braced for worker protection in accordance with DOSH guidelines and other applicable local or federal safety requirements.

Because soil conditions vary along the trail alignment, the contractor should anticipate encountering all the soil types described in DOSH guidelines (Types A, B, and C), which may require temporary slope inclinations ranging from 0.75H:1V to 1.5H:1V.

- For planning purposes only, assume a temporary slope inclination of 1.5H:1V or flatter, until actual soil conditions can be verified in the field during construction. If groundwater seepage is encountered within the excavation slopes, the cut slope may need to be inclined flatter than 1.5H:1V.

- Protect the slope from erosion with plastic sheeting for the duration of the excavation to reduce the risk of surface erosion and raveling.

- Limit the open excavation to the shortest time possible.

- Place no surcharge loads (such as from equipment or materials) within 10 feet of the top of the slope, in general. However, more or less stringent requirements may apply depending on field conditions and actual surcharge loads.

- Temporary or permanent cuts should not extend into existing steep slopes or bluffs near portions of the proposed trail alignment, especially the steep hillside near the northern end of the trail.

- If adequate sloping or slot cutting is not feasible because of site spatial constraints or other factors, temporary excavations should be supported by an appropriate shoring system.

5.13 Placement of Excess Materials on Site

We understand that there will be some excess materials that will be placed on site rather than hauled off site. There will be no structural elements or loads on these materials. We make the following recommendations for placement of these materials:

- Place material in lifts not exceeding 2 feet in thickness;

- Compact materials to at least 85 percent of the modified proctor maximum dry density (ASTM D 1557) to provide a stable surface; and

- Slope the sides of these materials no steeper than 4H:1V.

6.0 RECOMMENDED ADDITIONAL SERVICES

Recommendations discussed in this report should be reviewed and modified as needed during the project's final design stages. We also recommend incorporating geotechnical construction observation into the construction plans. The following sections present our recommended post-report geotechnical engineering services for this project.

6.1 Geotechnical Design Services

We recommend that Hart Crowser continue interacting with the design team periodically as the design documents become more complete, and review geotechnical aspects of the final design plans and specifications to confirm that our recommendations were properly understood and implemented in the design. Specifically, we recommend the following additional design services:

- Work with the structural and civil engineers as the final design of the Boise Creek Bridge and the White River Bridge are developed.

- Evaluate global stability of retaining walls as their locations and heights are finalized.

- Review final slope setback and design of the retaining wall/rockery for the portion of the trail where Boise Creek encroaches on the alignment.

- Perform additional explorations and soil laboratory tests on the mudflow deposit to further optimize the slope stability design and refine soil properties, if required to realign the trail closer to Boise Creek than recommended in this report.

- Provide geotechnical engineering support to the civil/structural engineer during preparation of project plans and specifications.

- Prepare geotechnical review letters in response to geotechnical plan review comments by the building department as needed during the permitting process.

6.2 Geotechnical Construction Services

Because the future performance and integrity of the structural elements of the project will depend largely on proper site preparation, drainage, fill placement, and construction procedures, monitoring and testing by experienced geotechnical personnel should be an integral part of construction.

Our observations will verify compliance with design concepts and recommendations, and allow design changes or evaluation of appropriate construction methods if subsurface conditions differ from those anticipated before construction begins. We recommend retaining Hart Crowser to provide the following construction support services:

- Review geotechnical-related construction submittals from the contractor to verify compliance with the construction plans and the recommendations of this report.

- Attend a pre-construction conference with the contractor and King County to discuss important geotechnical-related construction issues.

- Observe exposed wall footing, trail pavement, and embankment fill subgrade areas after completion of excavation, to confirm that suitable soil conditions have been reached or determine appropriate subgrade preparation methods, if needed.

- Observe proof-rolling of pavement subgrade before paving.

- Observe installation of the White River Bridge approach embankment to confirm conformance with the geotechnical design recommendations and construction plans.

- Monitor the placement of and perform in-place density tests on structural fill soil to verify conformance with construction specifications.

- Observe installation of retaining wall and rockery drains to verify their conformance with construction plans.
- Observe excavation and construction of the slope setback and retaining structure along the unstable slope portion of the northeast trail segment.
- Observe construction of MSE walls to verify adequate wall embedment, fill compaction, drainage installation, and placement of reinforcing elements in accordance with the plans and specifications.

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Sources: Esri, HERE, DeLorme, USGS, Intermap, increment P Corp., NRCAN, Esri Japan, METI, Esri China (Hong Kong), Esri (Thailand), TomTom, MapmyIndia, © OpenStreetMap contributors, and the GIS User Community

2,000 1,000 0 2,000

Scale in Feet

N

Foothills Trail Phase II
King County, Washington

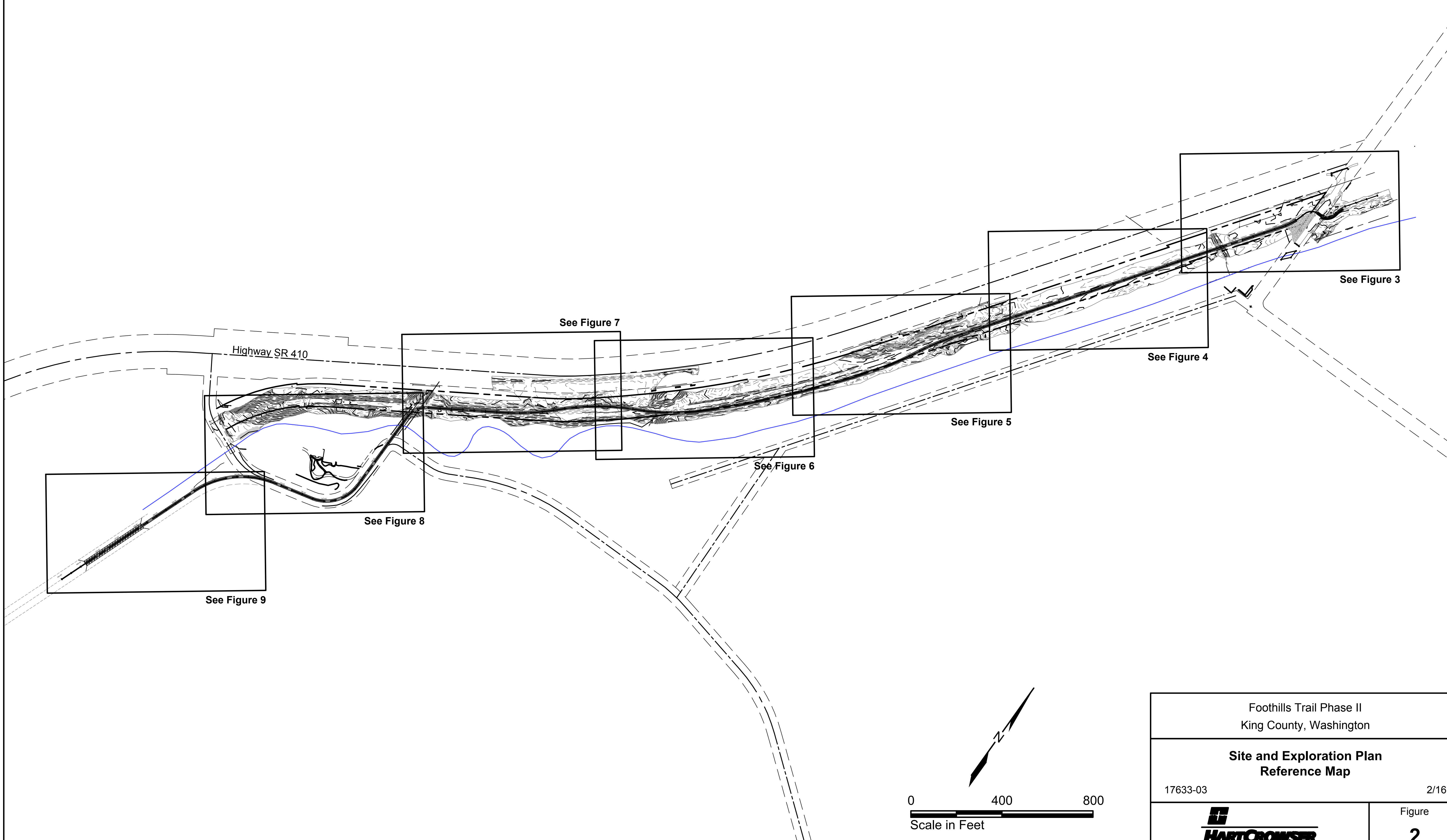
Vicinity Map

17633-03 12/15

HARTCROWSER

Figure
1

EAL 12/22/15 1763303-AA (VMap).mxd



Foothills Trail Phase II
King County, Washington

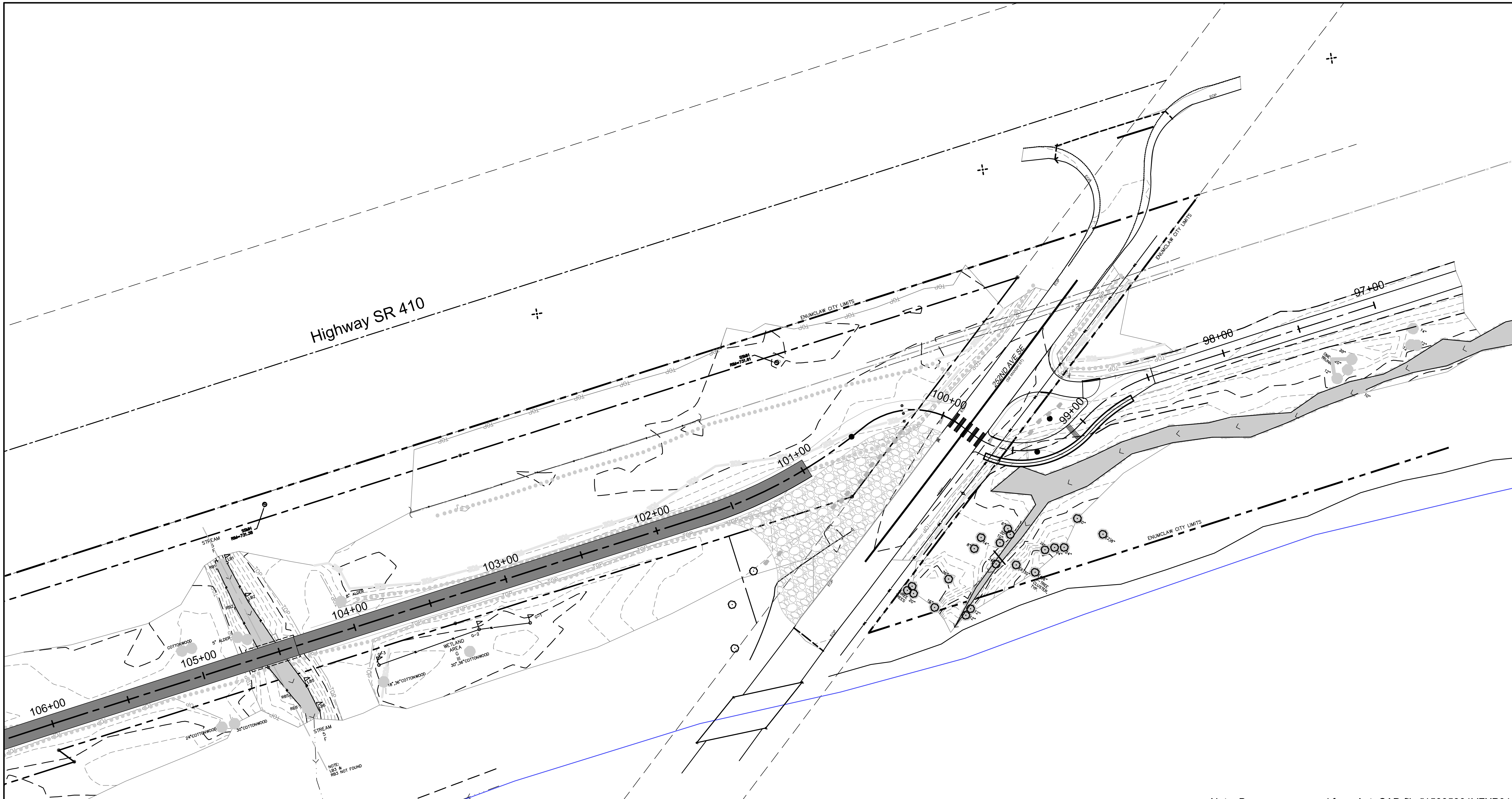
Site and Exploration Plan
Reference Map

17633-03 2/16

HARTCROWSER

Figure
2

EAL 02/1/16 1763303-002 (SPlan).dwg



Legend

TP-1 Test pit (Hart Crowser 2015)

BA-1 Auger boring (Hart Crowser 2015)

Trail alignment (reference only)

Graphic Exploration Log Descriptions
(depth to soil layer boundaries indicated in feet)

0

- Surficial organic soil
- Fill or disturbed native soil
- Medium dense to dense, gravelly silty SAND
- Pit Run Fill: medium dense, very sandy GRAVEL
- Osceola Mudflow: medium dense to dense, silty/clayey, very gravelly SAND
- Dense to very dense, slightly silty, sandy GRAVEL
- Very dense, slightly gravelly SAND

26

N

0 60 120
Scale in Feet

Note: Basemap prepared from AutoCAD file "15095301VFXP01 - 2015 updated.dwg," provided by Huit-Zollars, dated 11/11/15.

Foothills Trail Phase II
King County, Washington

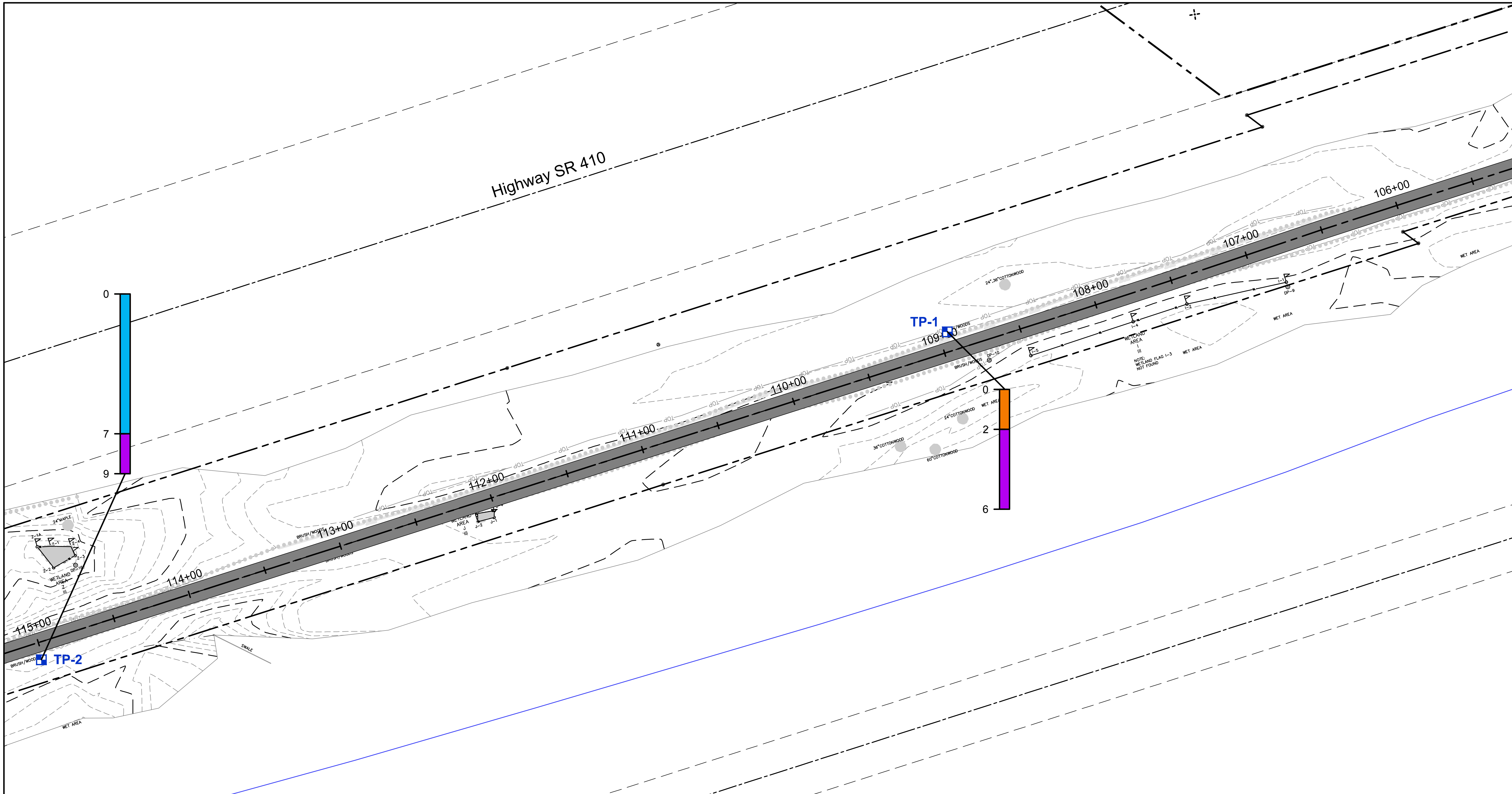
Site and Exploration Plan:
Approximately Station 101+00 to 106+50

17633-03 2/16

HARTCROWSER

Figure
3

EAL 02/1/16 1763303-002 (SPlan).dwg



Legend

TP-1 Test pit (Hart Crowser 2015)

BA-1 Auger boring (Hart Crowser 2015)

Trail alignment (reference only)

Graphic Exploration Log Descriptions
(depth to soil layer boundaries indicated in feet)

0

- Surficial organic soil
- Fill or disturbed native soil
- Medium dense to dense, gravelly silty SAND
- Pit Run Fill: medium dense, very sandy GRAVEL
- Osceola Mudflow: medium dense to dense, silty/clayey, very gravelly SAND
- Dense to very dense, slightly silty, sandy GRAVEL
- Very dense, slightly gravelly SAND

26

0 60 120
Scale in Feet

Note: Basemap prepared from AutoCAD file "15095301VFXP01 - 2015 updated.dwg," provided by Huit-Zollars, dated 11/11/15.

Foothills Trail Phase II
King County, Washington

Site and Exploration Plan:
Approximately Station 106+00 to 115+50

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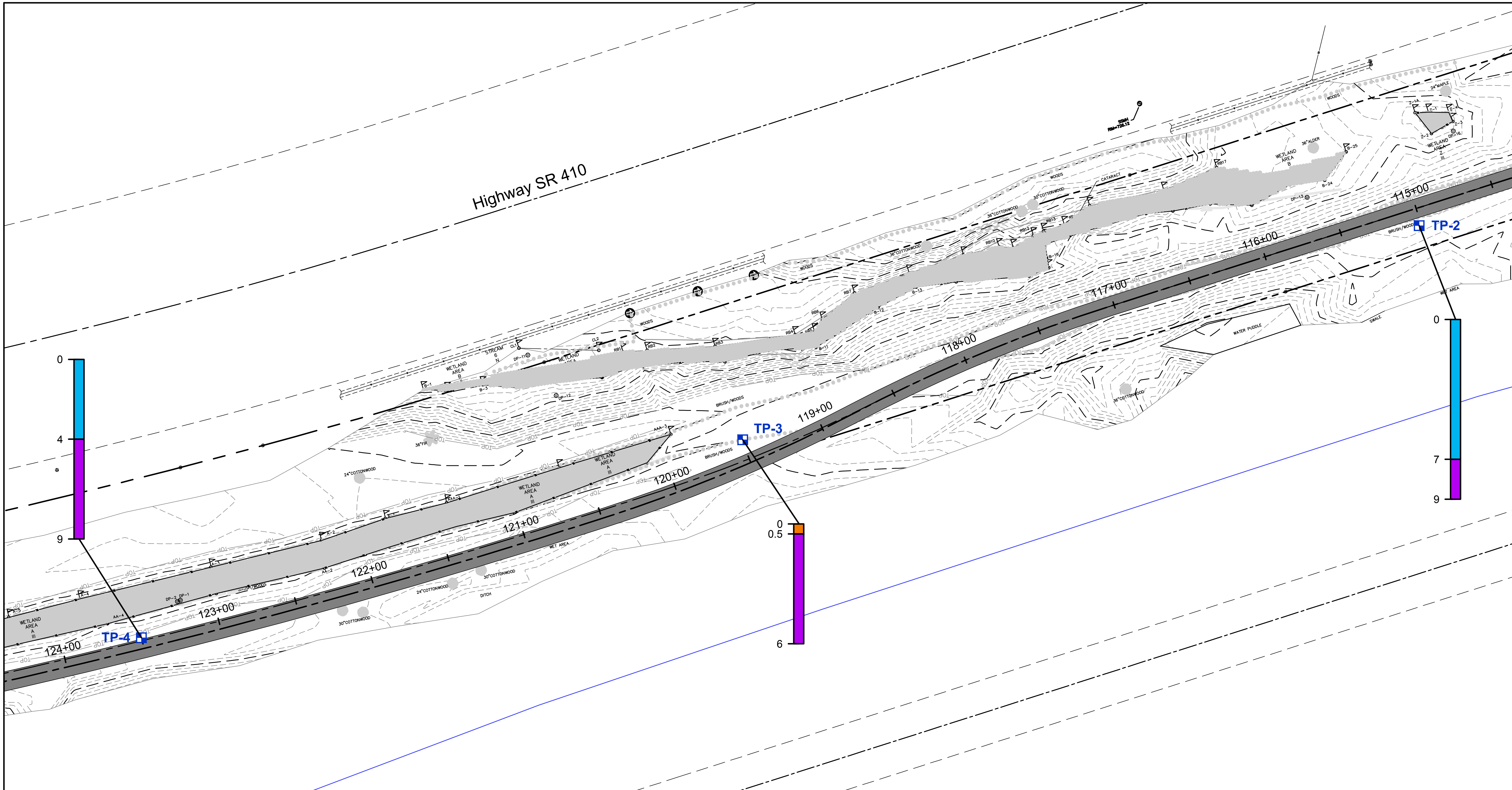
HARTCROWSER

Figure
4

EAL 02/1/16 1763303-002 (SPlan).dwg



Legend

TP-1 Test pit (Hart Crowser 2015)

BA-1 Auger boring (Hart Crowser 2015)

Trail alignment (reference only)

Graphic Exploration Log Descriptions
(depth to soil layer boundaries indicated in feet)

0
- Surficial organic soil
- Fill or disturbed native soil
- Medium dense to dense, gravelly silty SAND
- Pit Run Fill: medium dense, very sandy GRAVEL
- Osceola Mudflow: medium dense to dense, silty/clayey, very gravelly SAND
- Dense to very dense, slightly silty, sandy GRAVEL
- Very dense, slightly gravelly SAND

26

0 60 120
Scale in Feet

Note: Basemap prepared from AutoCAD file "15095301VFXP01 - 2015 updated.dwg," provided by Huit-Zollars, dated 11/11/15.

Foothills Trail Phase II
King County, Washington

Site and Exploration Plan: Approximately Station 114+00 to 124+50

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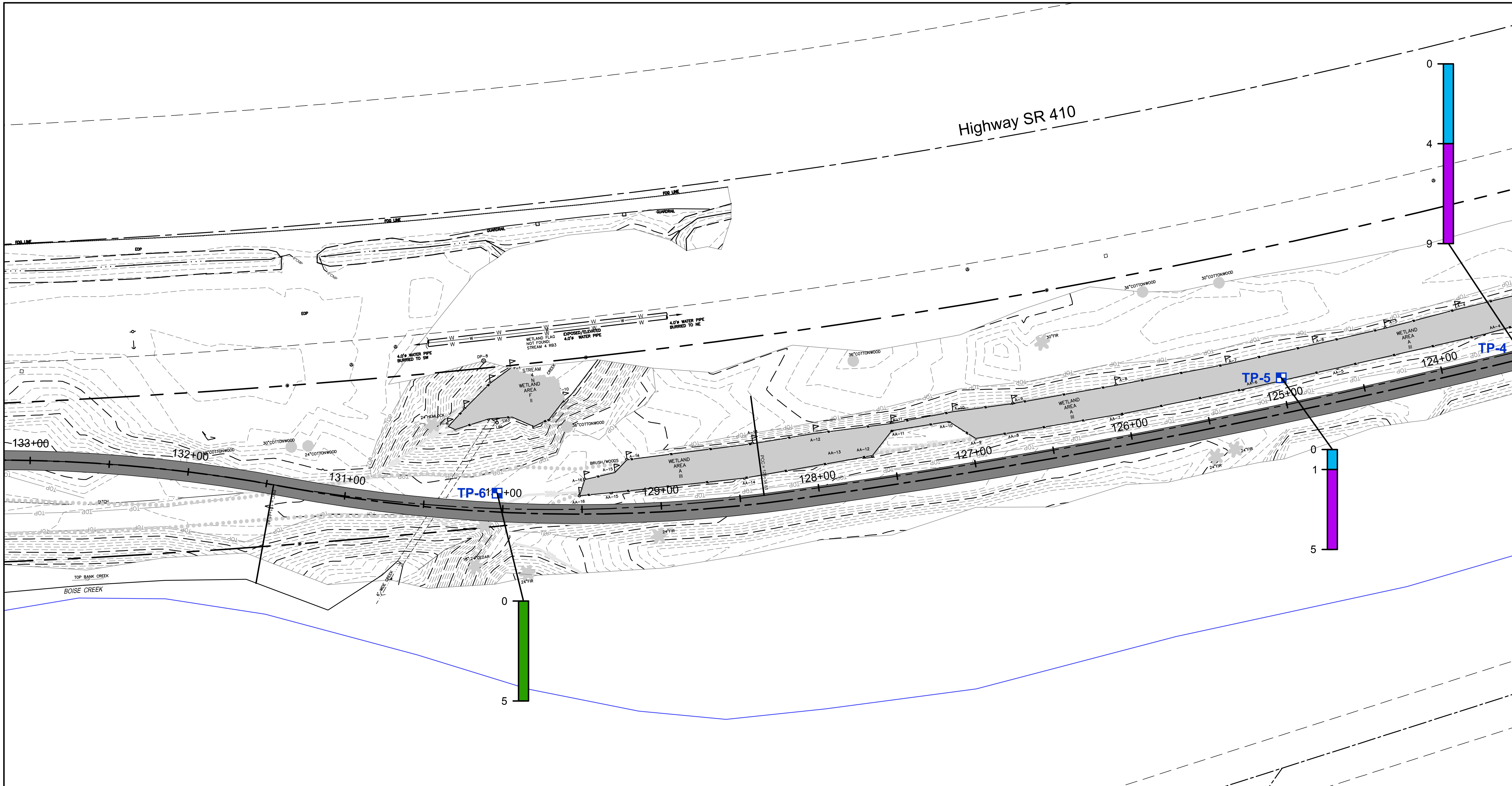
HARTCROWSER

Figure
5

EAL 02/1/16 1763303-002 (SPlan).dwg



Note: Basemap prepared from AutoCAD file "15095301VFXP01 - 2015 updated.dwg," provided by Huit-Zollars, dated 11/11/15.

Legend

- TP-1 Test pit (Hart Crowser 2015)
- BA-1 Auger boring (Hart Crowser 2015)
- Trail alignment (reference only)

Graphic Exploration Log Descriptions

(depth to soil layer boundaries indicated in feet)

0 – 26

- Surficial organic soil
- Fill or disturbed native soil
- Medium dense to dense, gravelly silty SAND
- Pit Run Fill: medium dense, very sandy GRAVEL
- Osceola Mudflow: medium dense to dense, silty/clayey, very gravelly SAND
- Dense to very dense, slightly silty, sandy GRAVEL
- Very dense, slightly gravelly SAND

N

0 60 120

Scale in Feet

Foothills Trail Phase II
King County, Washington

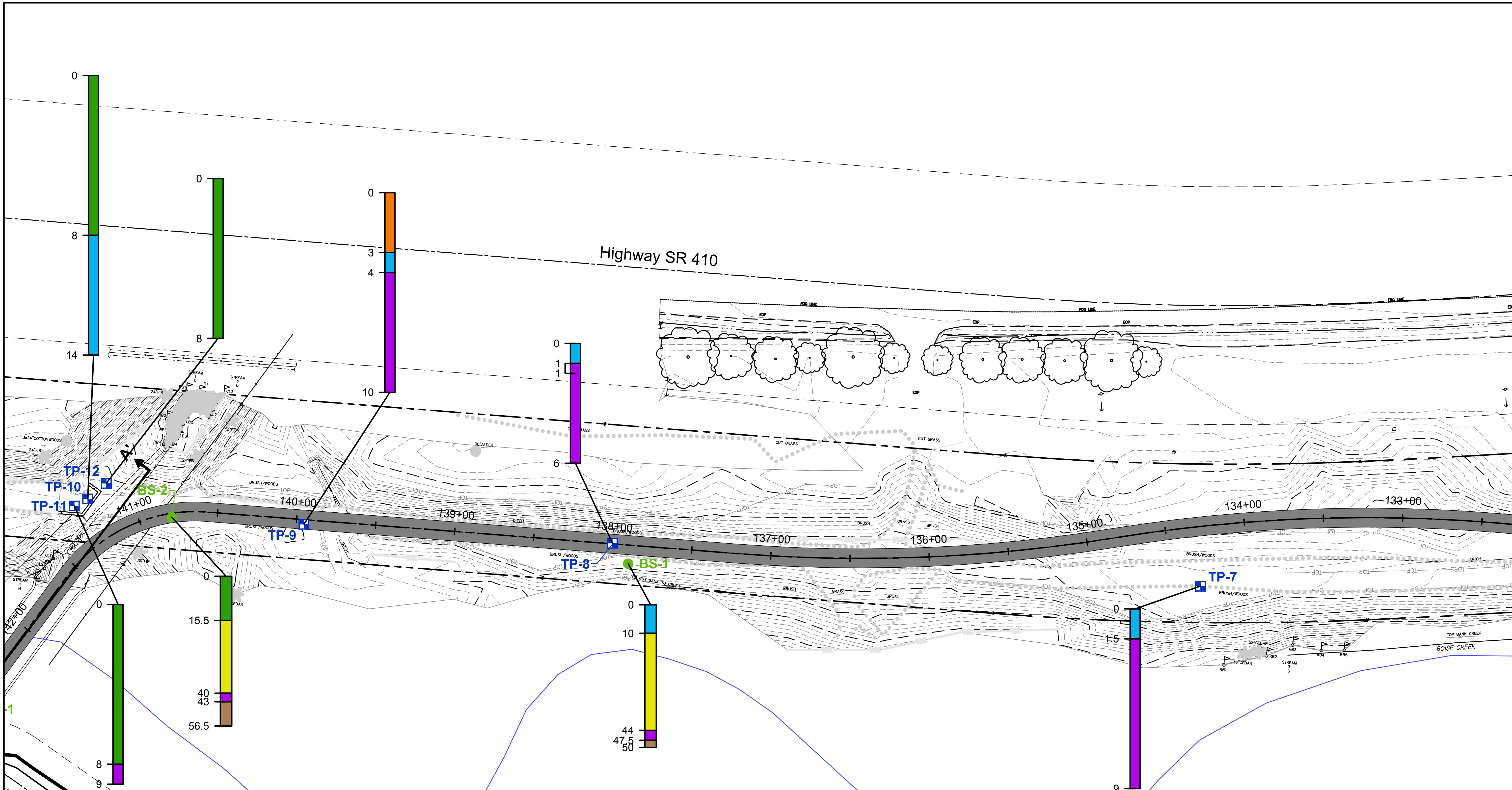
Site and Exploration Plan: Approximately Station 123+50 to 133+50

17633-03 2/16

HARTCROWSER

Figure **6**

EAL 02/1/16 1763303-002 (SPlan).dwg



Note: Basemap prepared from AutoCAD file "15095301VFXP01 - 2015 updated.dwg," provided by Huit-Zollars, dated 11/11/15.

Legend

- TP-1 Test pit (Hart Crowser 2015)
- BA-1 Auger boring (Hart Crowser 2015)
- Trail alignment (reference only)
- A A' Generalized subsurface cross section

Graphic Exploration Log Descriptions
(depth to soil layer boundaries indicated in feet)

0
- Surficial organic soil
- Fill or disturbed native soil
- Medium dense to dense, gravelly silty SAND
- Pit Run Fill: medium dense, very sandy GRAVEL
- Osceola Mudflow: medium dense to dense, silty/clayey, very gravelly SAND
- Dense to very dense, slightly silty, sandy GRAVEL
- Very dense, slightly gravelly SAND

26

Foothills Trail Phase II
King County, Washington

Site and Exploration Plan: Approximately Station 132+00 to 142+00

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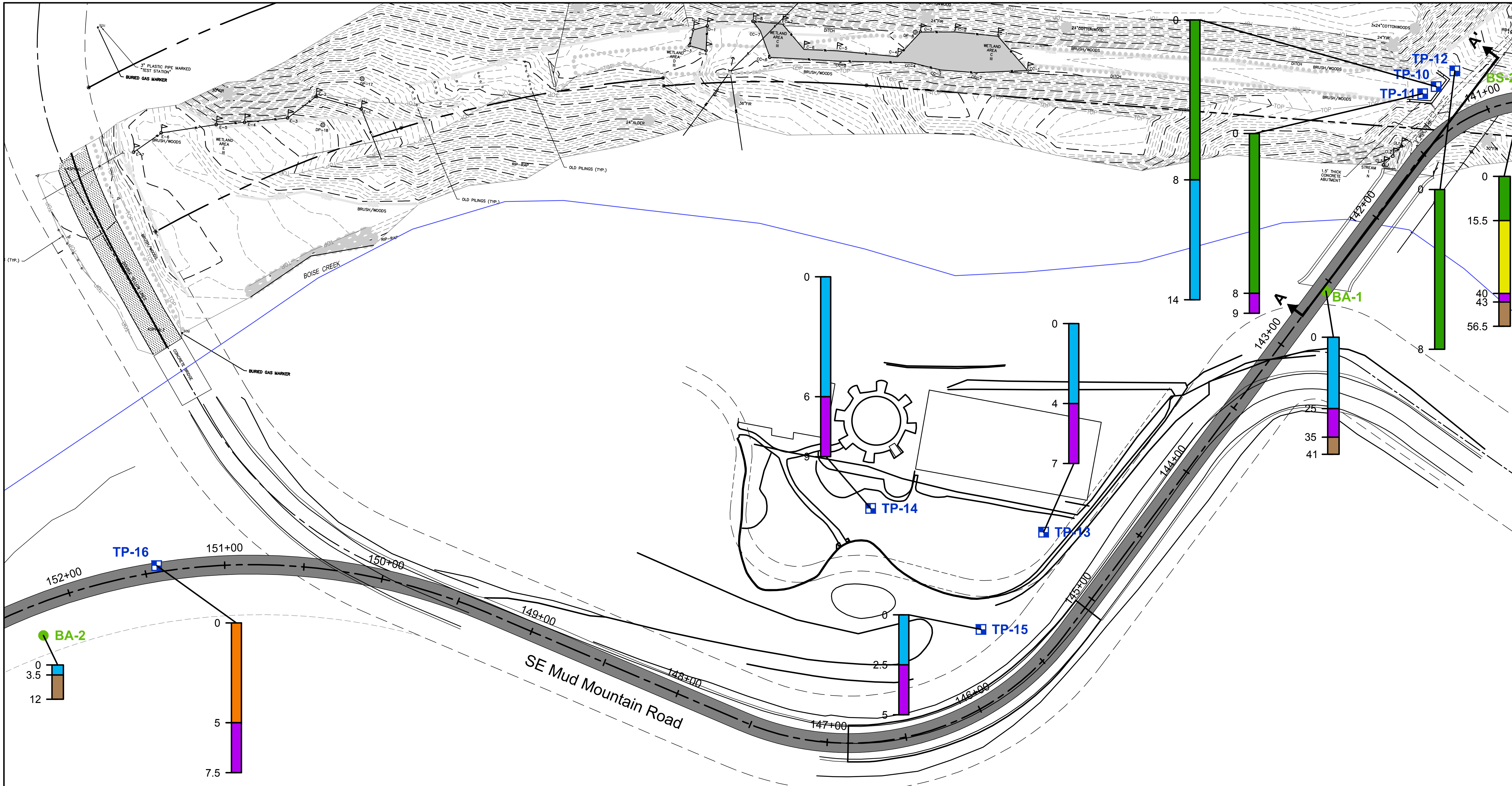
HARTCROWSER

Figure
7

EAL 02/1/16 1763303-002 (SPlan).dwg



Legend

- TP-1 — Test pit (Hart Crowser 2015)
- BA-1 — Auger boring (Hart Crowser 2015)
- Trail alignment (reference only)
- A A' — Generalized subsurface cross section

Graphic Exploration Log Descriptions
(depth to soil layer boundaries indicated in feet)

- Surficial organic soil
- Fill or disturbed native soil
- Medium dense to dense, gravelly silty SAND
- Pit Run Fill: medium dense, very sandy GRAVEL
- Osceola Mudflow: medium dense to dense, silty/clayey, very gravelly SAND
- Dense to very dense, slightly silty, sandy GRAVEL
- Very dense, slightly gravelly SAND

0 60 120
Scale in Feet

Note: Basemap prepared from AutoCAD file "15095301VFXP01 - 2015 updated.dwg," provided by Huit-Zollars, dated 11/11/15.

Foothills Trail Phase II
King County, Washington

Site and Exploration Plan: Approximately Station 141+00 to 152+50

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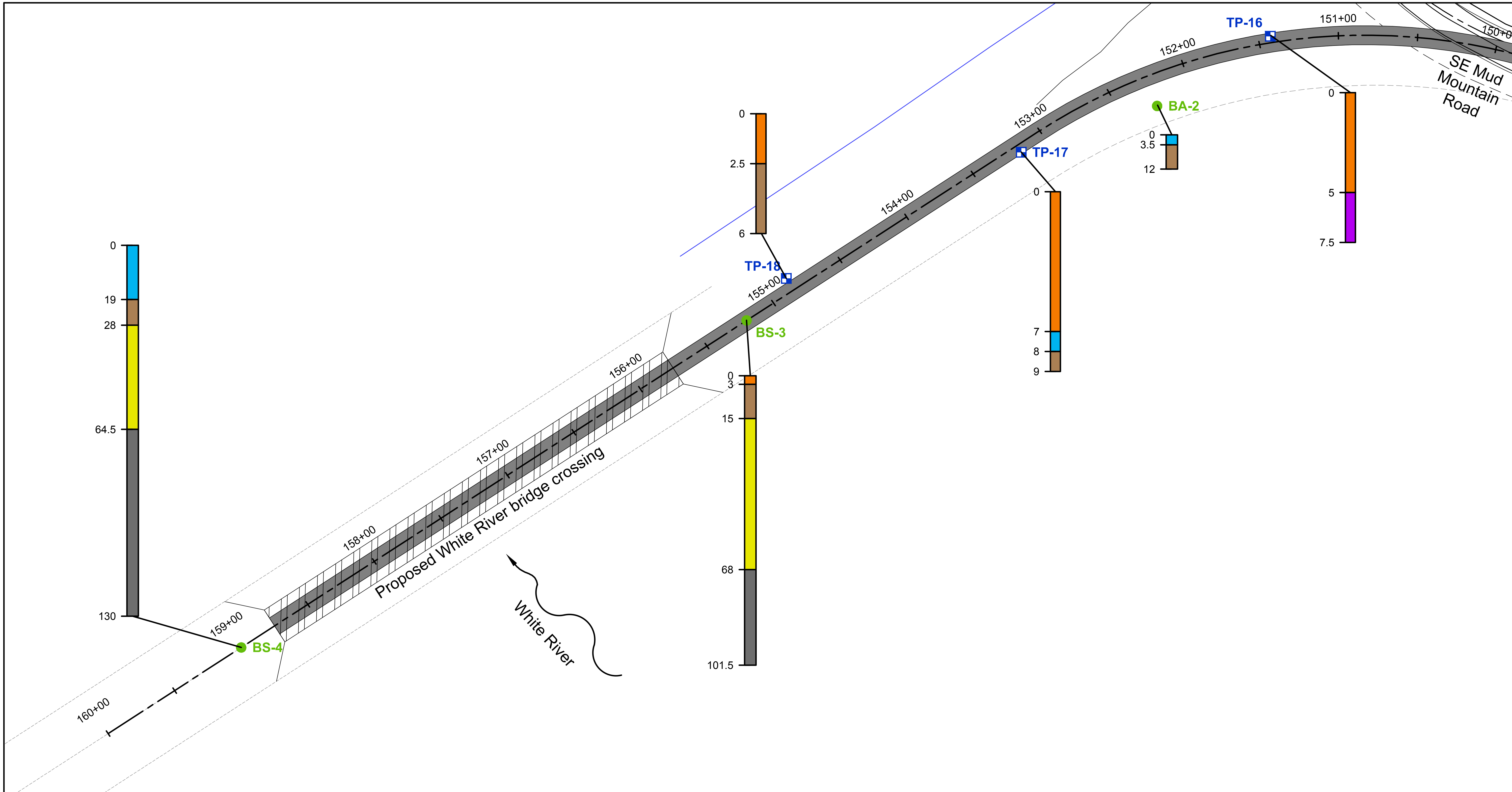
HARTCROWSER

Figure **8**

EAL 02/1/16 1763303-002 (SPlan).dwg



Legend

TP-1 Test pit (Hart Crowser 2015)

BA-1 Auger boring (Hart Crowser 2015)

Trail alignment (reference only)

Graphic Exploration Log Descriptions
(depth to soil layer boundaries indicated in feet)

0 – 26

- Surficial organic soil
- Fill or disturbed native soil
- Medium dense to dense, gravelly silty SAND
- Pit Run Fill: medium dense, very sandy GRAVEL
- Osceola Mudflow: medium dense to dense, silty/clayey, very gravelly SAND
- Dense to very dense, slightly silty, sandy GRAVEL
- Very dense, slightly gravelly SAND

N

0 60 120
Scale in Feet

Note: Basemap prepared from AutoCAD file "15095301VFXP01 - 2015 updated.dwg," provided by Huit-Zollars, dated 11/11/15.

Foothills Trail Phase II
King County, Washington

Site and Exploration Plan:
Approximately Station 150+00 to 160+50

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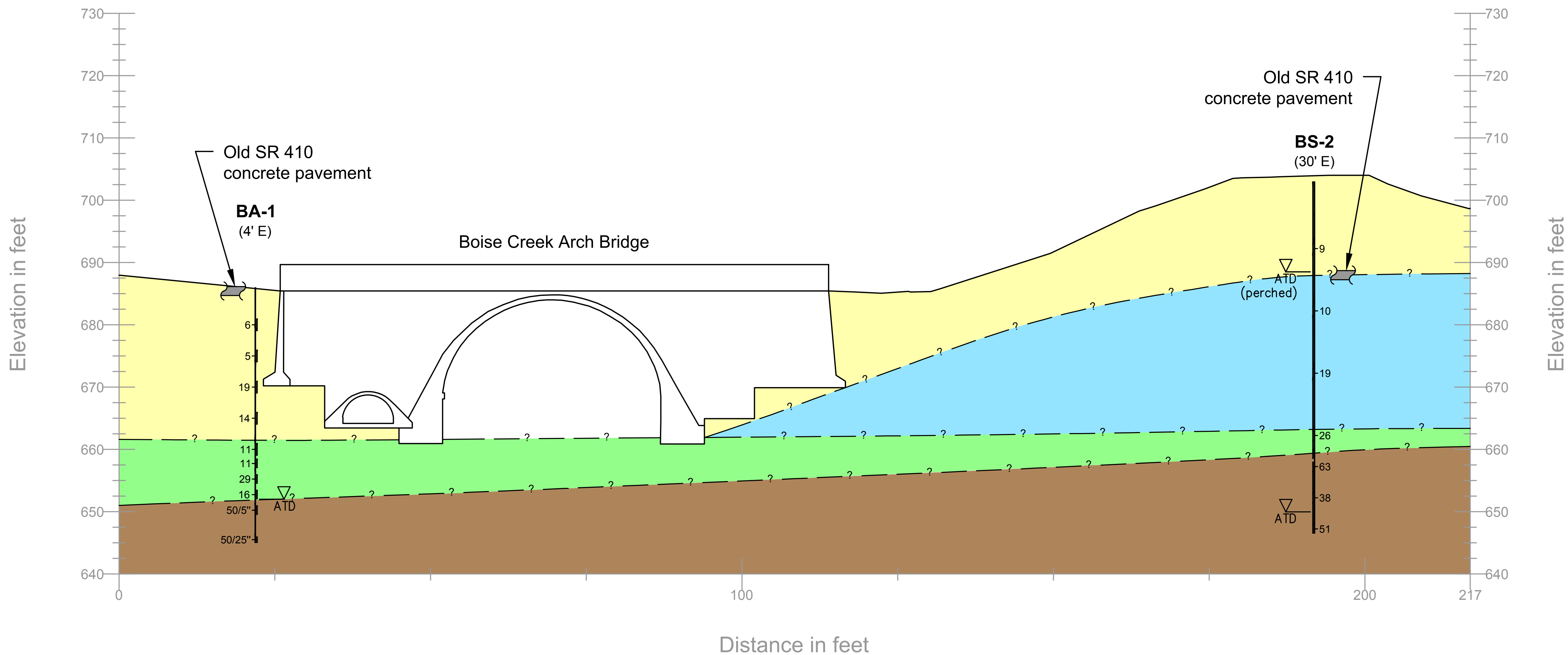
Figure
9

EAL 02/1/16 1763303-002 (SPlan).dwg



BA-1 (4' E) Exploration number (offset distance and direction)

Exploration location

ATD Groundwater level (at time of drilling)

9 Standard penetration resistance in blows per foot

Loose to medium dense, gravelly SAND to sandy GRAVEL (Fill)

Stiff, slightly gravelly, sandy SILT

Medium dense, gravelly, silty SAND

Very dense, slightly silty, sandy GRAVEL

0 20 40

Horizontal and Vertical Scale in Feet

Foothills Trail Phase II
King County, Washington

Generalized Subsurface Cross Section A-A' (Boise Creek Arch Bridge)

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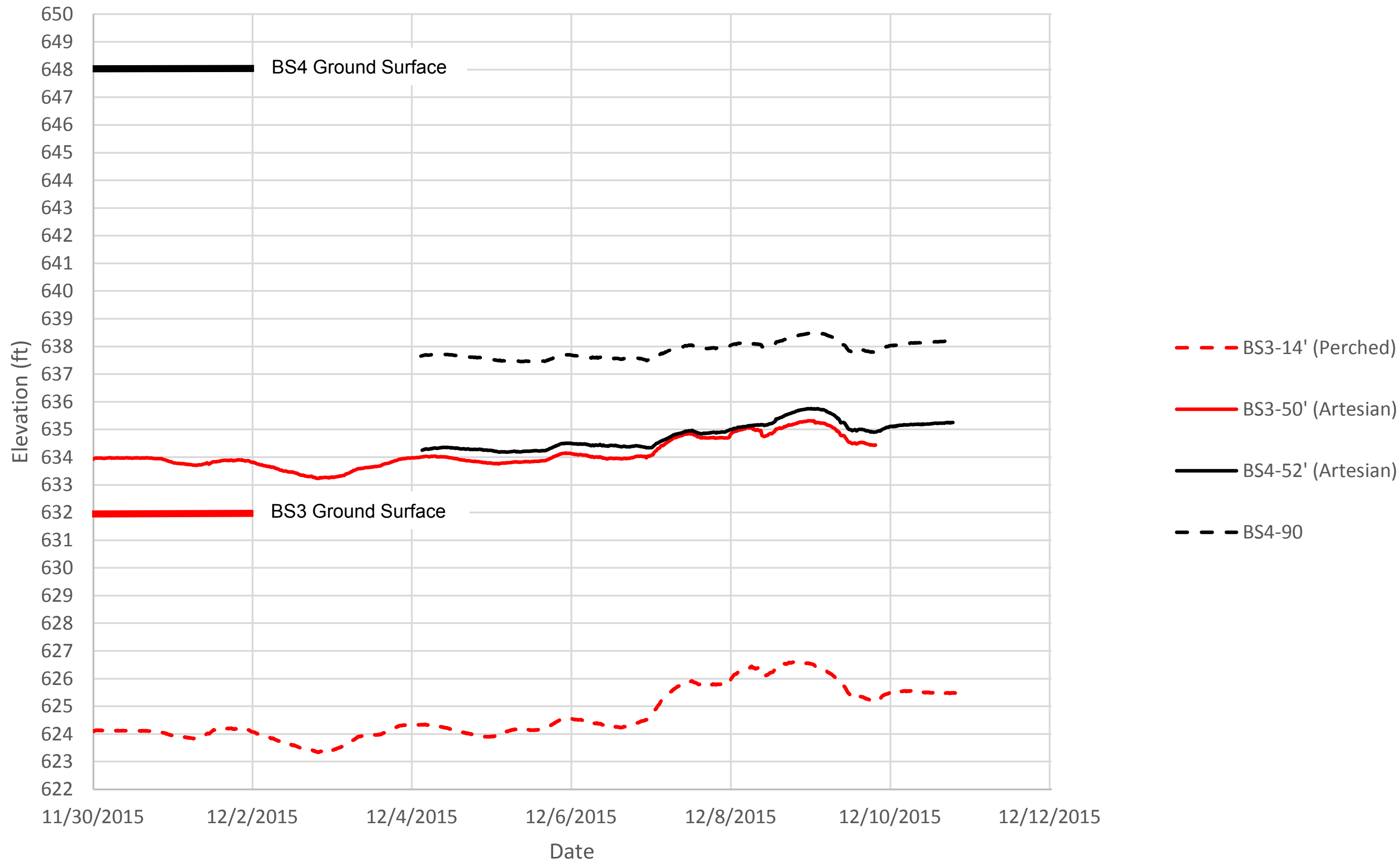
Figure **10**

EAL 12/29/15 1763303-001 (XSec).dwg



Foothills Trail Phase II
Enumclaw, WA

Vibrating Wire Piezometer Hydraulic Head Measurements

17633-03

12/15

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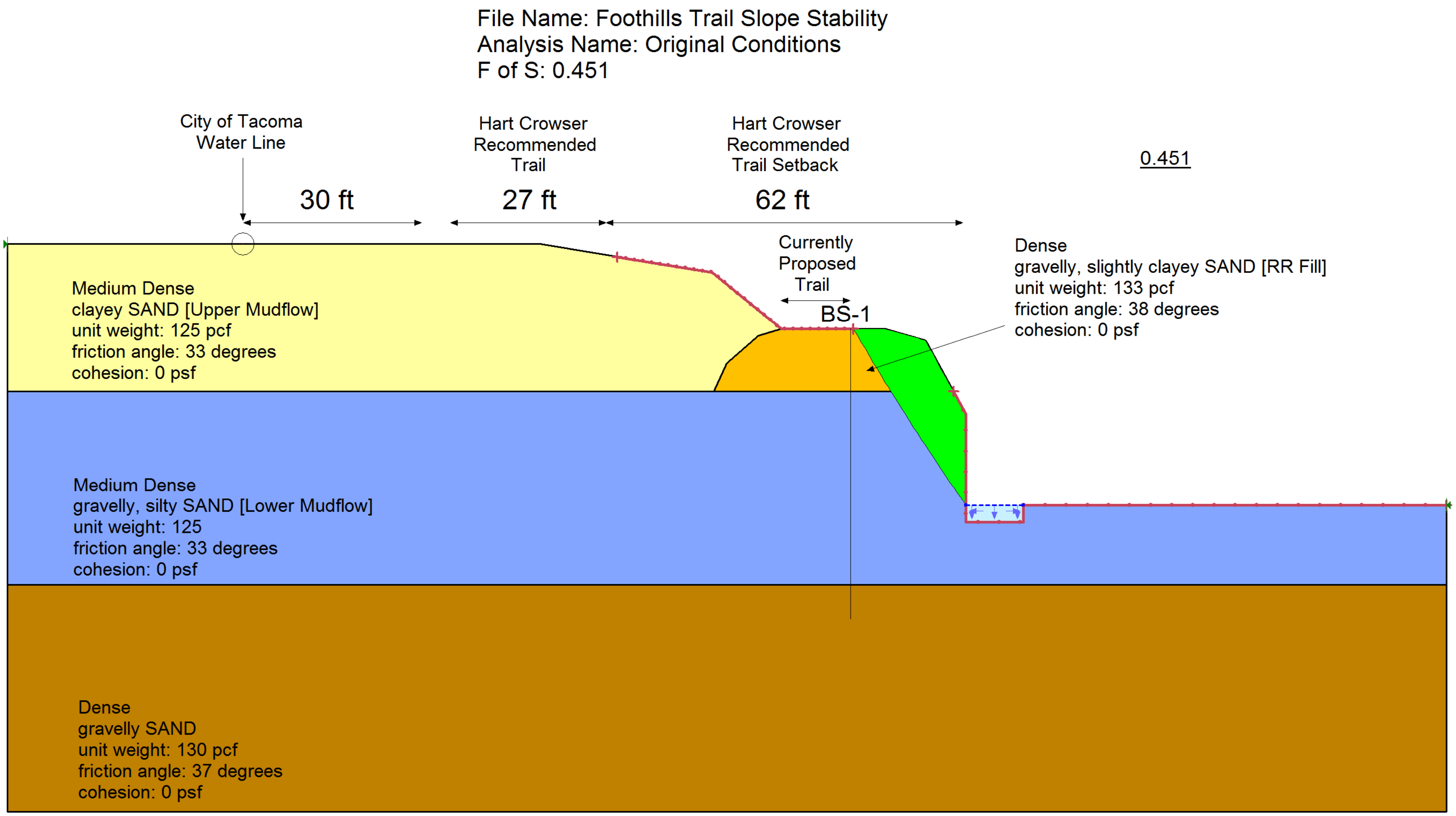
Figure

11



Foothills Trail Phase II
King County, WA

Existing Slope Geometry and Stability Analysis (Boise Creek Undercut Location at BS-1)

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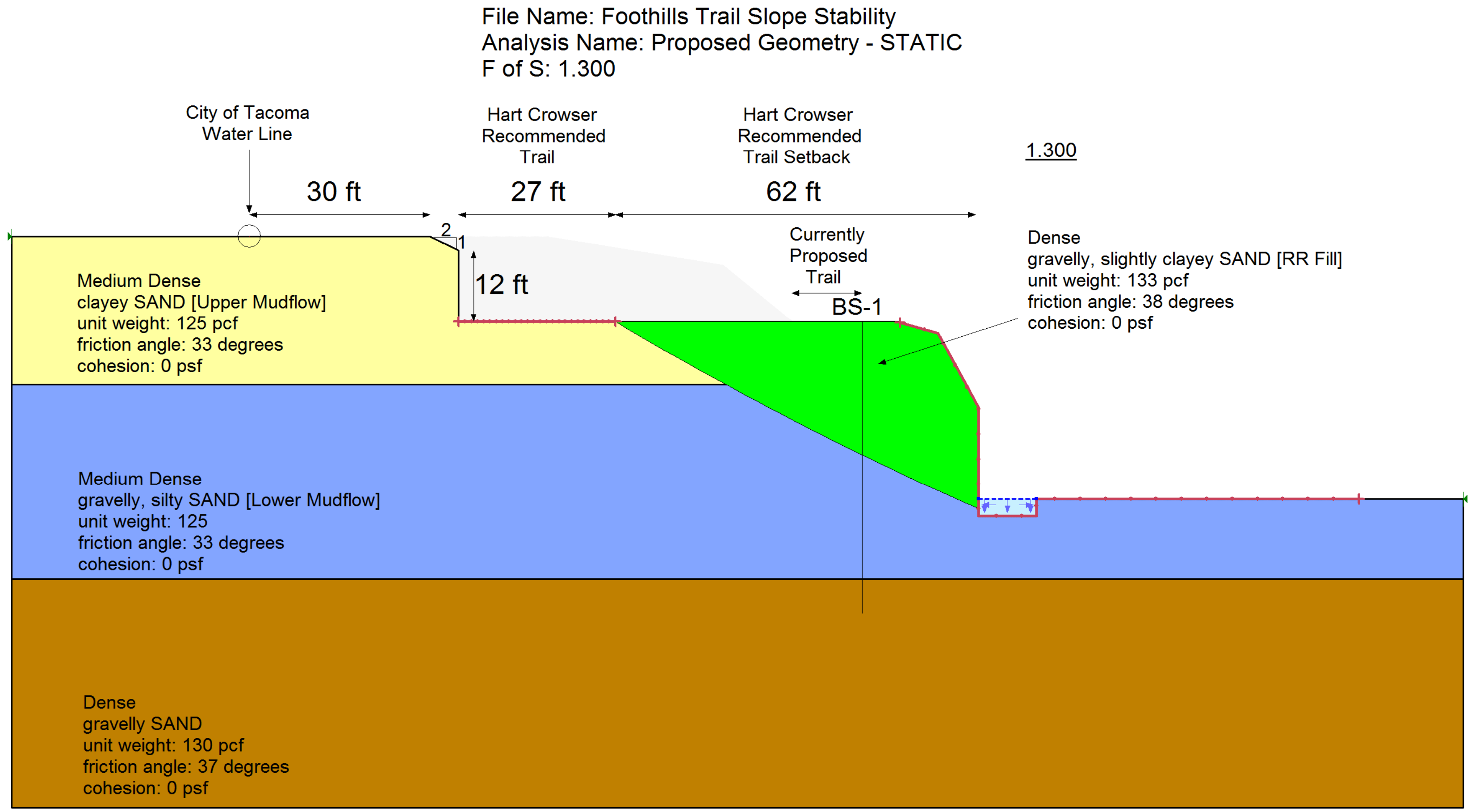
HARTCROWSER

Figure **12**



Foothills Trail Phase II
King County, WA

Recommended Slope Geometry - Static Analysis (Boise Creek Undercut Location at BS-1)

17633-03

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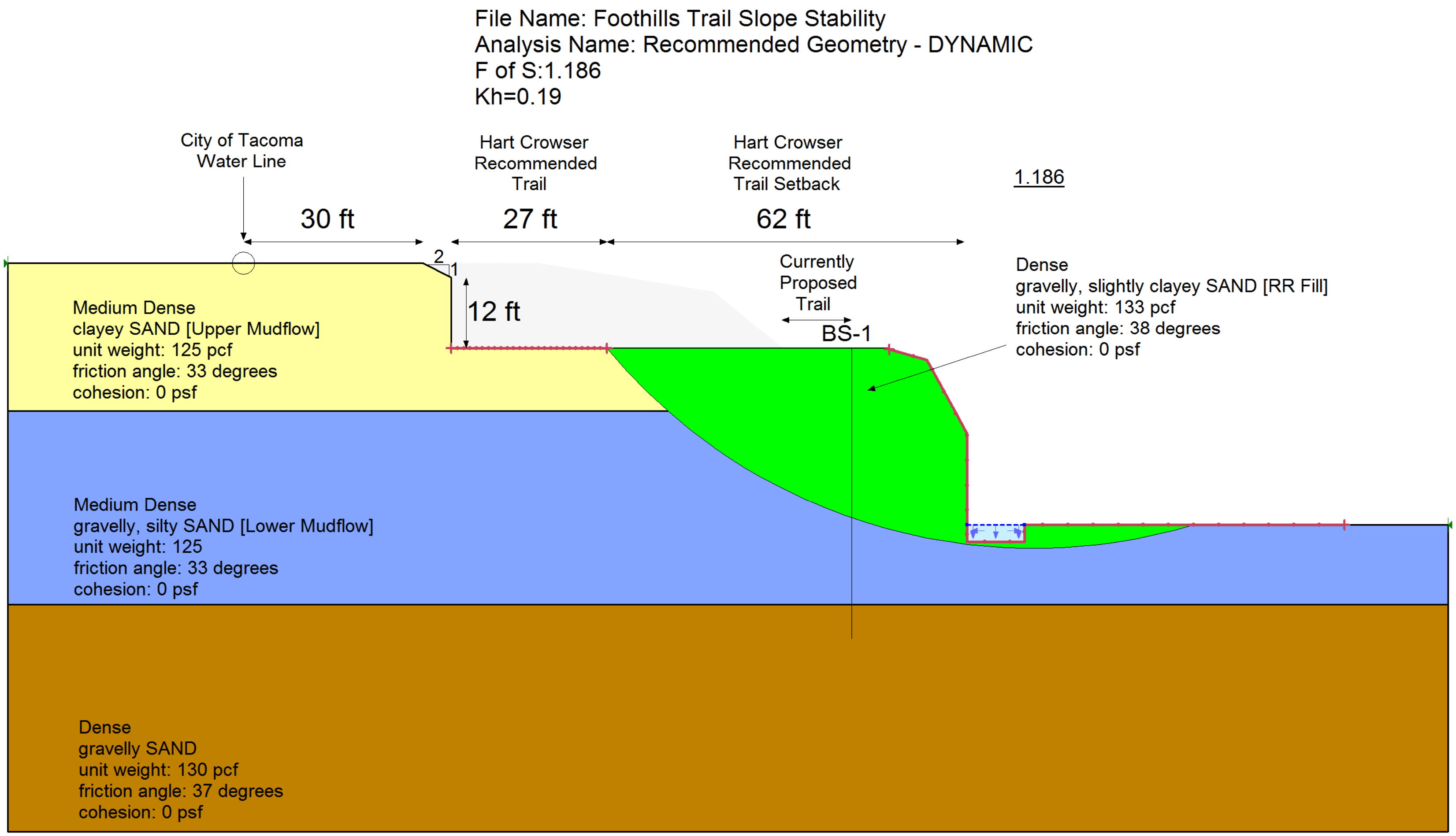
Figure

13



Foothills Trail Phase II
King County, WA

Recommended Slope Geometry - Dynamic Analysis (Boise Creek Undercut Location at BS-1)

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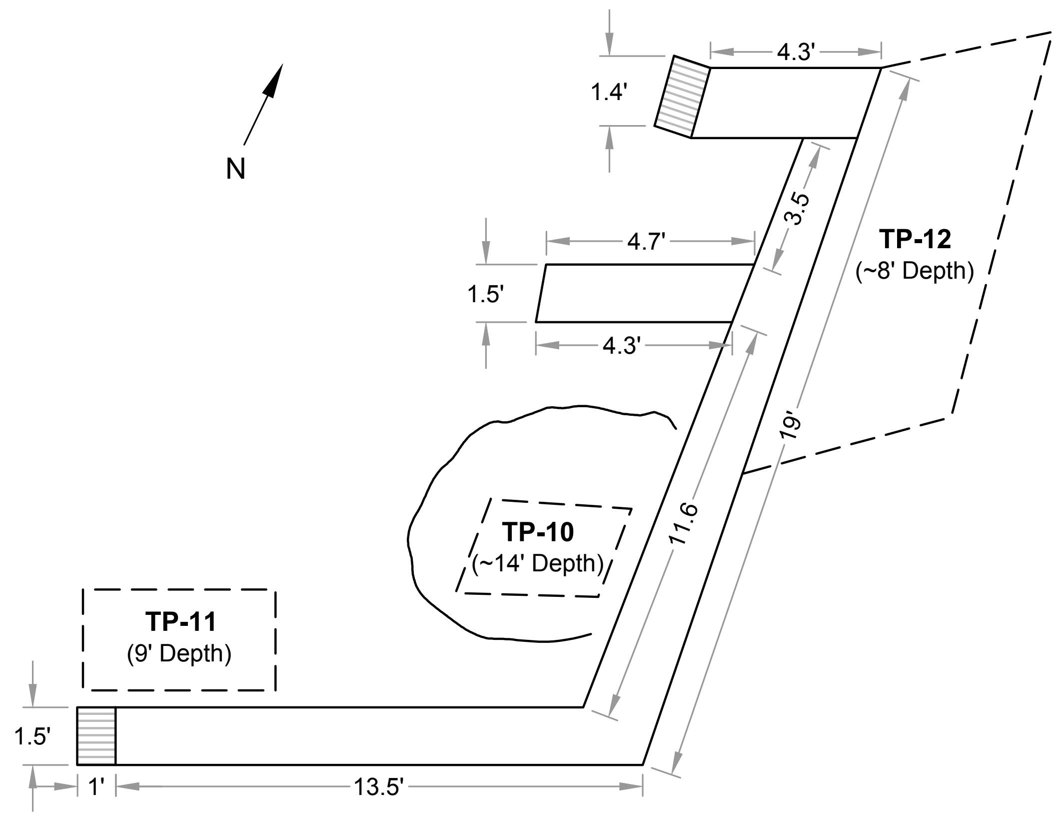
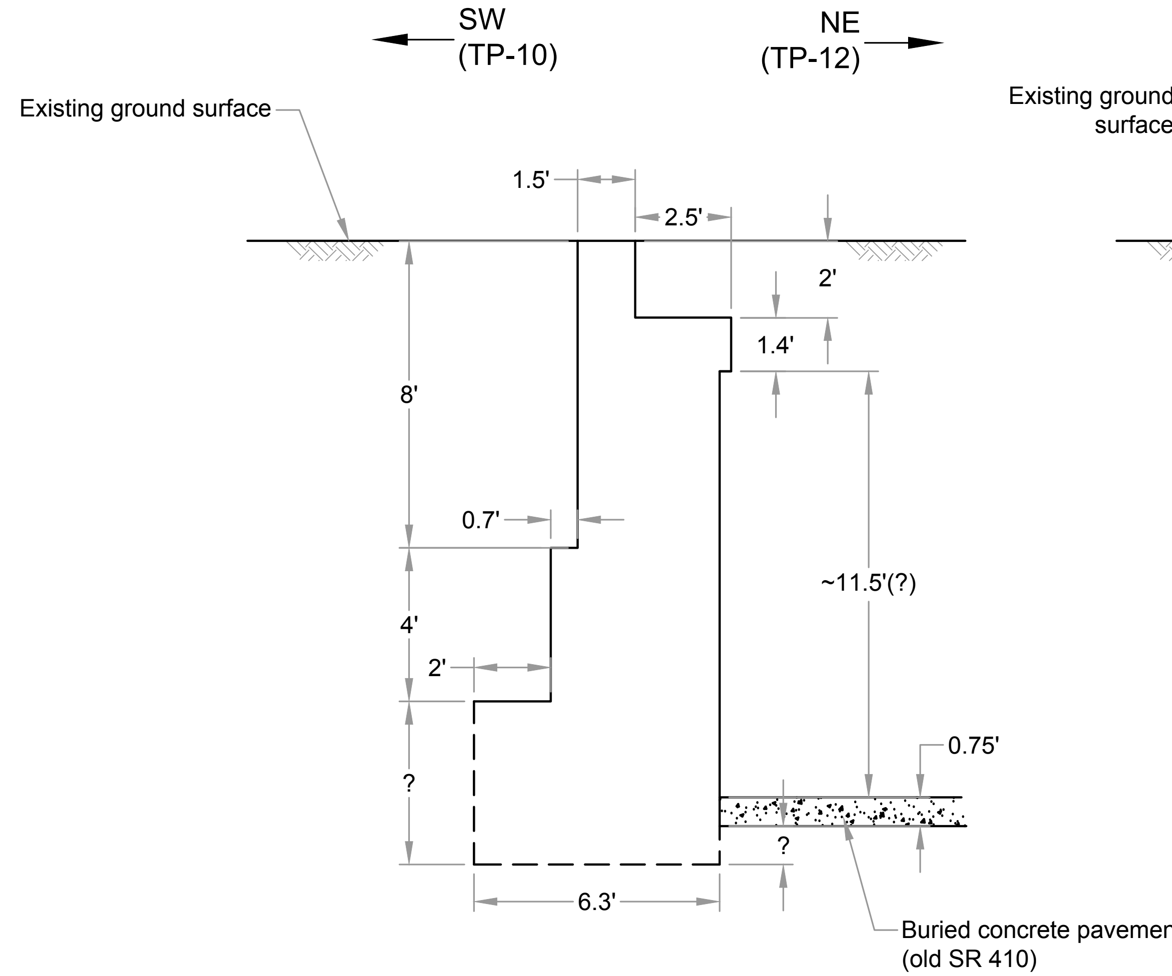
12/15

Figure

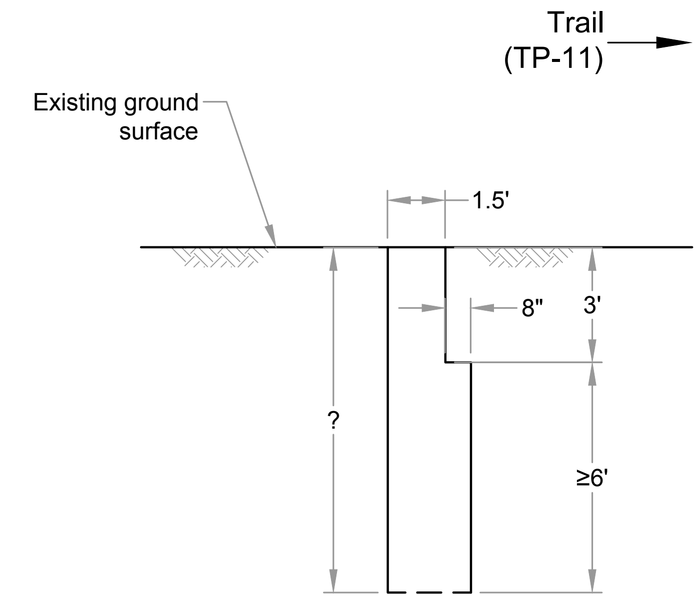
14



Plan view
Not to Scale



Approximate abutment section view based on observations in TP-10 and TP-12
Not to Scale



Approximate south wingwall section view based on observations in TP-11
Not to Scale

Note: All dimensions are approximate based on hand measurements in the field.

Foothills Trail Phase II
King County, Washington

Existing Railroad Abutment Schematics

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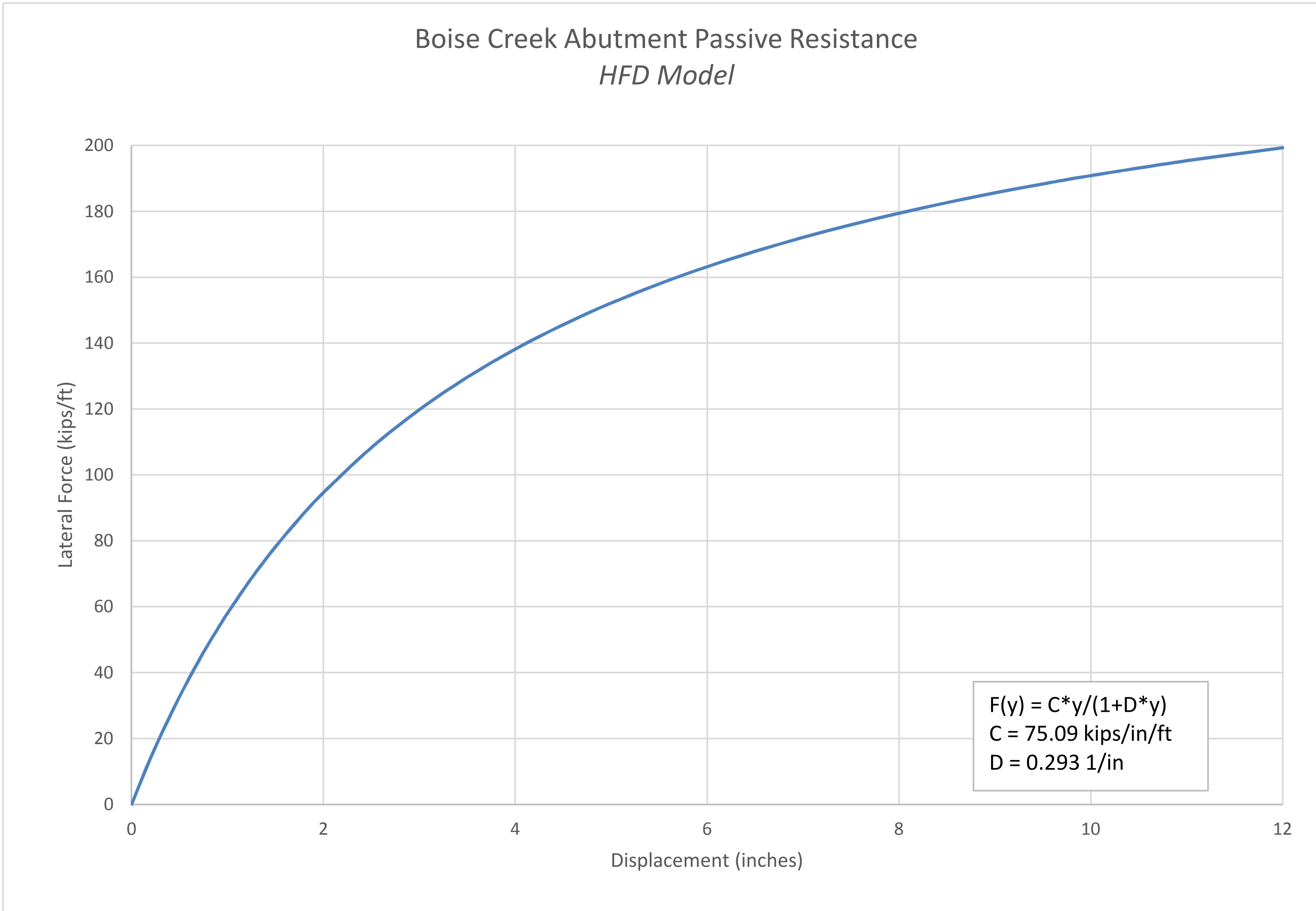
Figure
15

File: L:\Notebooks\1763303_Foothills Trail Phase II\CAD\1763303_Schematic.dwg Layout:11x17 - H Date: 12-31-2015 Author: ericlindquist



Foothills Trail Phase II
Enumclaw, WA

Boise Creek Bridge Abutment Passive Resistance

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12/15

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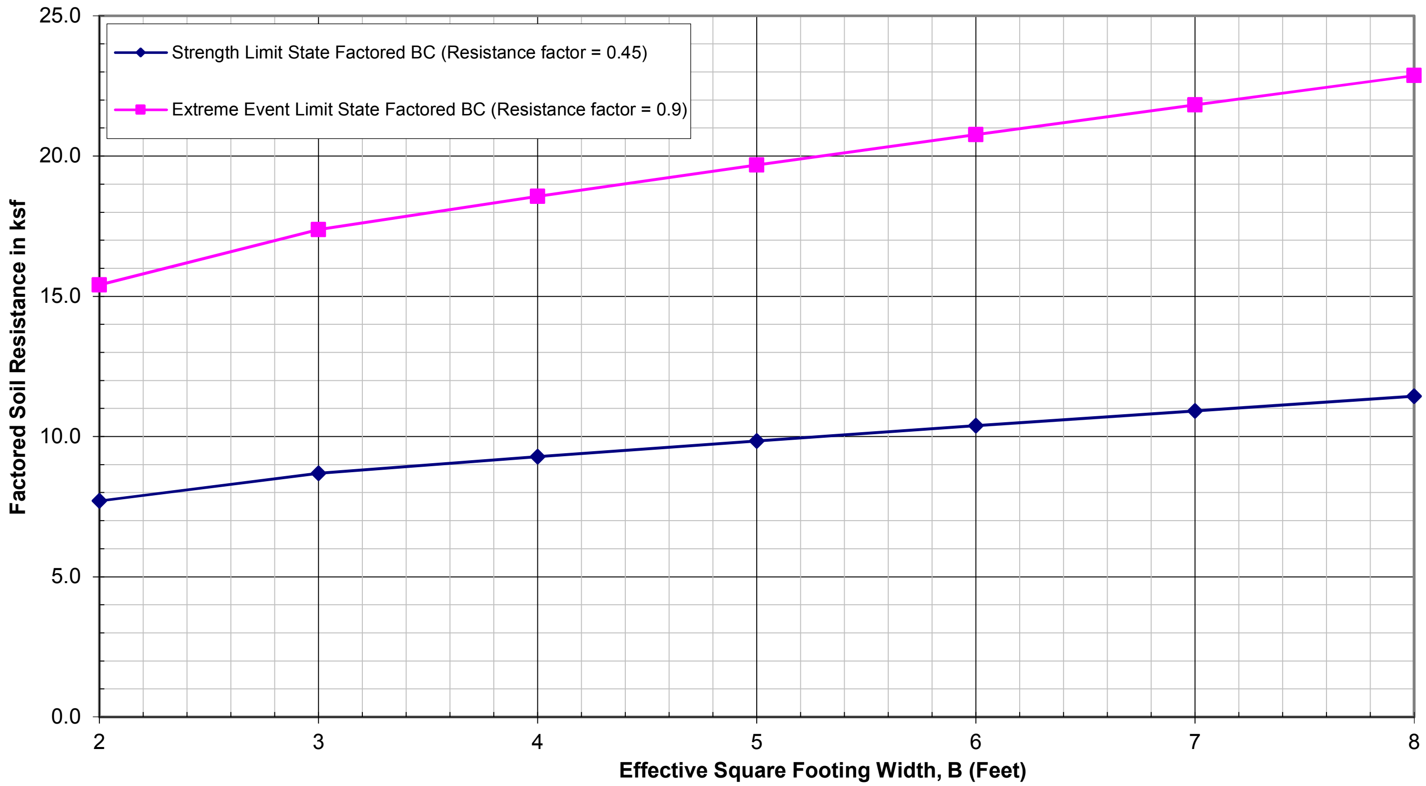
Figure

16

Factored Soil Resistance For Square Footing



Assumptions
1. Footings to be constructed having a minimum footing depth of 18 inches below the lowest adjacent grade.
2. Disturbed soil and any standing water has been removed prior to concrete placement.
3. Excavation sufficient for footing to bear on dense native soil or compacted structural fill.

Foothills Trail
Enumclaw, Washington

Soil Resistance

White River Bridge Approach Shallow Foundations

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Figure **17**

APPENDIX A
Field Exploration Methods and Analysis

APPENDIX A

Field Exploration Methods and Analysis

This appendix documents the processes Hart Crowser used to determine the nature of the site soils. Sections are:

- Explorations and Their Location,
- Hollow-Stem Auger Borings,
- Ultrasonic Borings,
- Standard Penetration Test Procedures, and
- Excavation of Test Pits.

Explorations and Their Location

Explorations. Subsurface explorations for this project were two hollow-stem auger borings, four ultrasonic borings, and 18 test pits. The exploration logs in this appendix show our interpretation of the drilling, sampling, and testing data. They indicate the depth where the soils change; the change may be gradual. In the field, we classified the samples taken from the explorations according to the methods on Figure A-1, Key to Exploration Logs; the legend explains the symbols and abbreviations used in the logs and tables.

Locations. Figures 3 through 9 show the location of the explorations. Several locations are based on a survey of the explorations from Huitt-Zollars. Where survey data was not available, locations are based off field measurements referenced with GIS topographic maps. Elevations shown on the logs have been interpolated from either the provided survey or topographic contour lines from the King County GIS database. The measurement methods used determine the accuracy of the location and elevation information for the explorations.

Hollow-Stem Auger Borings

Two hollow-stem auger borings (BA1 through BA2) were drilled between November 23 and 24, 2015, to depths of 12 to 41 feet below existing ground surface. Using a 4-inch-inside-diameter hollow-stem auger, borings were advanced with a LAR track drill rig subcontracted by Hart Crowser. A geotechnical engineer or geologist from Hart Crowser continuously observed the drilling. Detailed field logs were prepared of each boring. Using the standard penetration test (SPT), we obtained samples at depth intervals of 2.5 to 5 feet.

The borings logs are presented on Figures A-2 through A-7 at the end of this appendix.

Ultrasonic Borings

Four ultrasonic borings (BS1 through BS4) were drilled between November 20 and December 2, 2015, to depths of 50 to 130 feet below existing ground surface. An 8-inch-inside-diameter hollow-stem auger was advanced with a sonic drill rig subcontracted by Hart Crowser. A geotechnical engineer or geologist from Hart Crowser continuously observed the drilling. Detailed field logs were prepared of each boring. Using the SPT, we obtained samples at 5- to 10-foot depth intervals.

The borings logs are presented on Figures A-2 through A-7 at the end of this appendix.

Standard Penetration Test Procedures

The SPT is an approximate measure of soil density and consistency. To be useful, the results must be used with engineering judgment in conjunction with other tests. The SPT (as described in ASTM D1586) was used to obtain disturbed samples. This test employs a standard 2-inch-outside-diameter split-spoon sampler. A 140-pound hammer free-falling 30 inches drives the sampler into the soil for 18 inches. The number of blows required to drive the sampler the last 12 inches only is the standard penetration resistance. This resistance, or blow count, measures the relative density of granular soils and the consistency of cohesive soils. The blow counts are plotted on the boring logs at their respective sample depths. Soil samples are recovered from the split-barrel sampler, field classified, placed into watertight jars, and taken to Hart Crowser's laboratory for further testing, as described in Appendix B.

Occasionally, very dense materials preclude driving the total 18-inch sample. When this happens, the penetration resistance is entered on logs as follows:

Penetration less than 6 inches. The log indicates the total number of blows over the number of inches of penetration.

Penetration greater than 6 inches. The blow count noted on the log is the sum of the total number of blows completed after the first 6 inches of penetration. This sum is expressed over the number of inches driven that exceed the first 6 inches. The number of blows needed to drive the first 6 inches is not reported. For example, a blow count series of 12 blows for 6 inches, 30 blows for 6 inches, and 50 (the maximum number of blows counted within a 6-inch increment for SPT) for 3 inches would be recorded as 80/9.

Excavation of Test Pits

We subcontracted a trackhoe and excavated 18 test pits (TP1 through TP18) across the site between November 16 and 19, 2015. The sides of the pits offer direct observation of the subgrade soils. The test pits were located by and excavated under the direction of a geologist from Hart Crowser. The geologist observed the soil exposed in the test pits and reported the findings on a field log. We obtained representative samples of soil types for visual classification at Hart Crowser's laboratory. Groundwater levels and/or seepage were noted during excavation. The density/consistency of the soils (in parentheses on the test pit logs to indicate they are estimates) is based on visual observation only as disturbed soils cannot be measured for in-place density in the laboratory.

The test pit logs are presented on Figures A-8 through A-16 at the end of this appendix.

In the case where a test pit was logged at the bottom of a slope, the general consistency of the toe of that slope (above depth = 0 feet) was logged and a note was added to the test pit.

Key to Exploration Logs

Sample Description

Classification of soils in this report is based on visual field and laboratory observations which include density/consistency, moisture condition, grain size, and plasticity estimates and should not be construed to imply field nor laboratory testing unless presented herein. Visual-manual classification methods of ASTM D 2488 were used as an identification guide.

Soil descriptions consist of the following:
Density/consistency, moisture, color, minor constituents, MAJOR CONSTITUENT, additional remarks.

Density/Consistency

Soil density/consistency in borings is related primarily to the Standard Penetration Resistance. Soil density/consistency in test pits and probes is estimated based on visual observation and is presented parenthetically on the logs.

SAND or GRAVEL Density	Standard Penetration Resistance (N) in Blows/Foot	SILT or CLAY Consistency	Standard Penetration Resistance (N) in Blows/Foot	Approximate Shear Strength in TSF
Very loose	0 to 4	Very soft	0 to 2	<0.125
Loose	4 to 10	Soft	2 to 4	0.125 to 0.25
Medium dense	10 to 30	Medium stiff	4 to 8	0.25 to 0.5
Dense	30 to 50	Stiff	8 to 15	0.5 to 1.0
Very dense	>50	Very stiff	15 to 30	1.0 to 2.0
		Hard	>30	>2.0

Sampling Test Symbols

- 1.5" I.D. Split Spoon
- Grab (Jar)
- 3.0" I.D. Split Spoon
- Shelby Tube (Pushed)
- Bag
- Cuttings
- Core Run

SOIL CLASSIFICATION CHART

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

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

Moisture

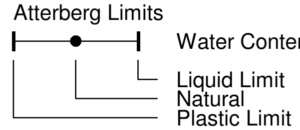
Dry	Little perceptible moisture
Damp	Some perceptible moisture, likely below optimum
Moist	Likely near optimum moisture content
Wet	Much perceptible moisture, likely above optimum

Minor Constituents

Minor Constituents	Estimated Percentage
Trace	<5
Slightly (clayey, silty, etc.)	5 - 12
Clayey, silty, sandy, gravelly	12 - 30
Very (clayey, silty, etc.)	30 - 50

Laboratory Test Symbols

GS	Grain Size Classification
CN	Consolidation
UU	Unconsolidated Undrained Triaxial
CU	Consolidated Undrained Triaxial
CD	Consolidated Drained Triaxial
QU	Unconfined Compression
DS	Direct Shear
K	Permeability
PP	Pocket Penetrometer Approximate Compressive Strength in TSF
TV	Torvane Approximate Shear Strength in TSF
CBR	California Bearing Ratio
MD	Moisture Density Relationship
AL	Atterberg Limits



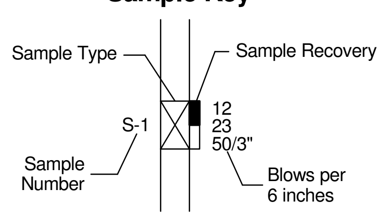
PID	Photoionization Detector Reading
CA	Chemical Analysis
DT	In Situ Density in PCF
OT	Tests by Others

Groundwater Indicators

Groundwater Level on Date or (ATD) At Time of Drilling

Groundwater Seepage (Test Pits)

Sample Key



HARTCROWSER

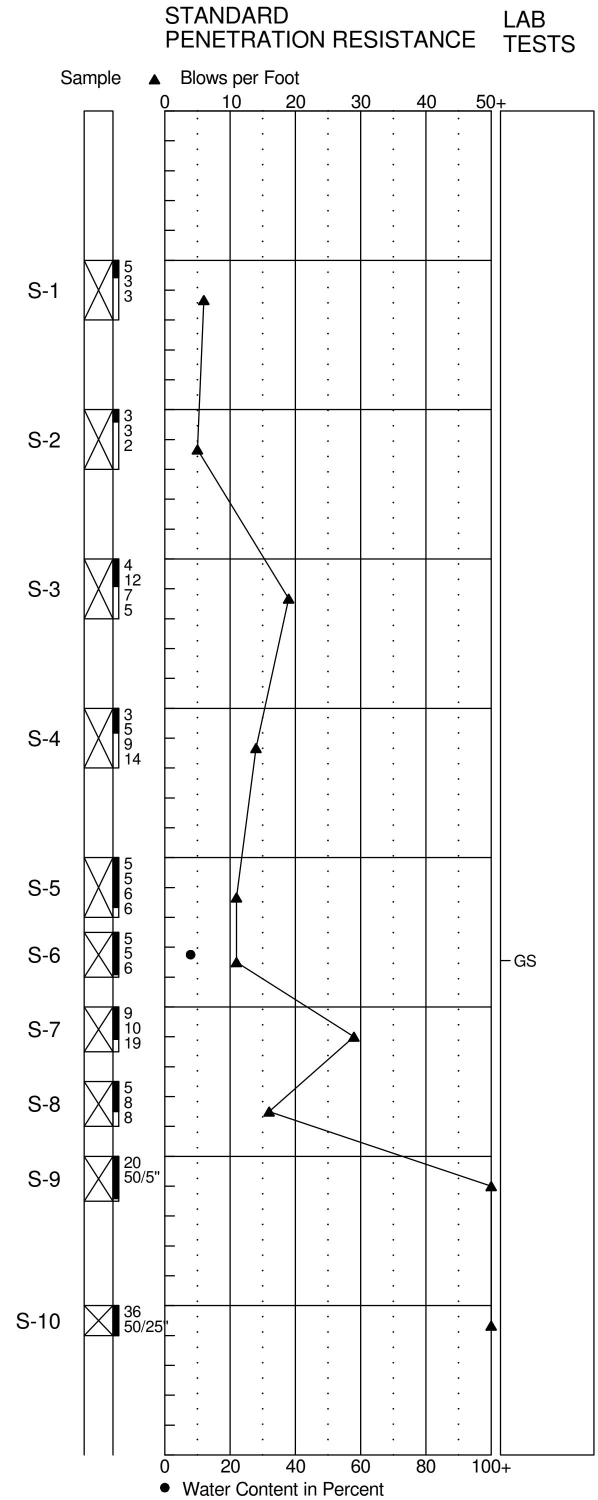
17633-03 11/15

Figure A-1

Boring Log BA-1

Location: N 66974.9119 E 1346208.7348	Drill Equipment: LAR Track HSA
Approximate Ground Surface Elevation: 686.4 Feet	Hammer Type: SPT
Horizontal Datum:	Hole Diameter: 4 inches
Vertical Datum: NAVD88	Logged By: J. Bruce Reviewed By: N. Campbell

USCS Class	Soil Descriptions	Depth in Feet
GP-GM	9 inches of Concrete over loose, moist, brown, silty, very sandy GRAVEL. (FILL)	0
SM	Loose, moist, brown, silty SAND with gravel. (FILL)	
GP	Medium dense, moist, gray, sandy GRAVEL with cobbles. (FILL)	
GP-GM	Grades to damp, slightly silty, sandy GRAVEL. (FILL)	20
SM	Medium dense, damp, brown, gravelly, silty SAND with occasional gravel.	25
SM	Becomes gray to brown.	30
	Gravel in cuttings.	
GP	Very dense, wet, gray-brown, silty, sandy GRAVEL with wood fragments.	35 (ATD)
CL	Hard, moist, gray, sandy CLAY with trace gravel.	40
	Bottom of Boring at 41.0 Feet. Started 11/23/15. Completed 11/23/15.	



1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

NEW BORING LOG 1763303-BL.GPJ HC_CORP.GDT 12/22/15

HARTCROWSER

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Figure A-2

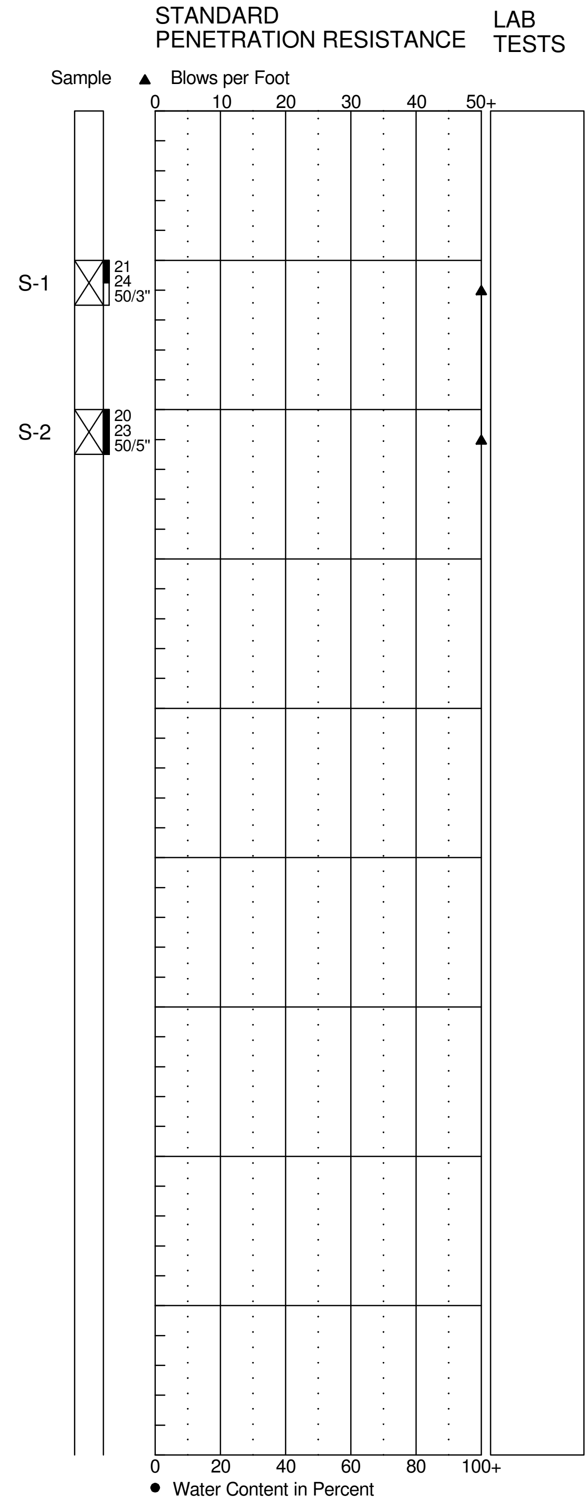
Boring Log BA-2

Location: N 66320.5686 E 1345685.7663
Approximate Ground Surface Elevation: 639.7 Feet
Horizontal Datum:
Vertical Datum: NAVD88

Drill Equipment: LAR Track HSA
Hammer Type: SPT
Hole Diameter: 4 inches
Logged By: J. Bruce   Reviewed By: N. Campbell

USCS Class	Graphic Log	Soil Descriptions	Depth in Feet
SP-SM		Light brown to brown, slightly silty SAND. (FILL)	0
GM		Very dense, damp to wet, brown to dark gray, silty, sandy GRAVEL and crushed rock with occasional metal fragments.	3.5
		Bottom of Boring at 12.0 Feet. Started 11/24/15. Completed 11/24/15.	12.0

Depth in Feet: 0, 5, 10, 15, 20, 25, 30, 35, 40, 45



1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

HARTCROWSER

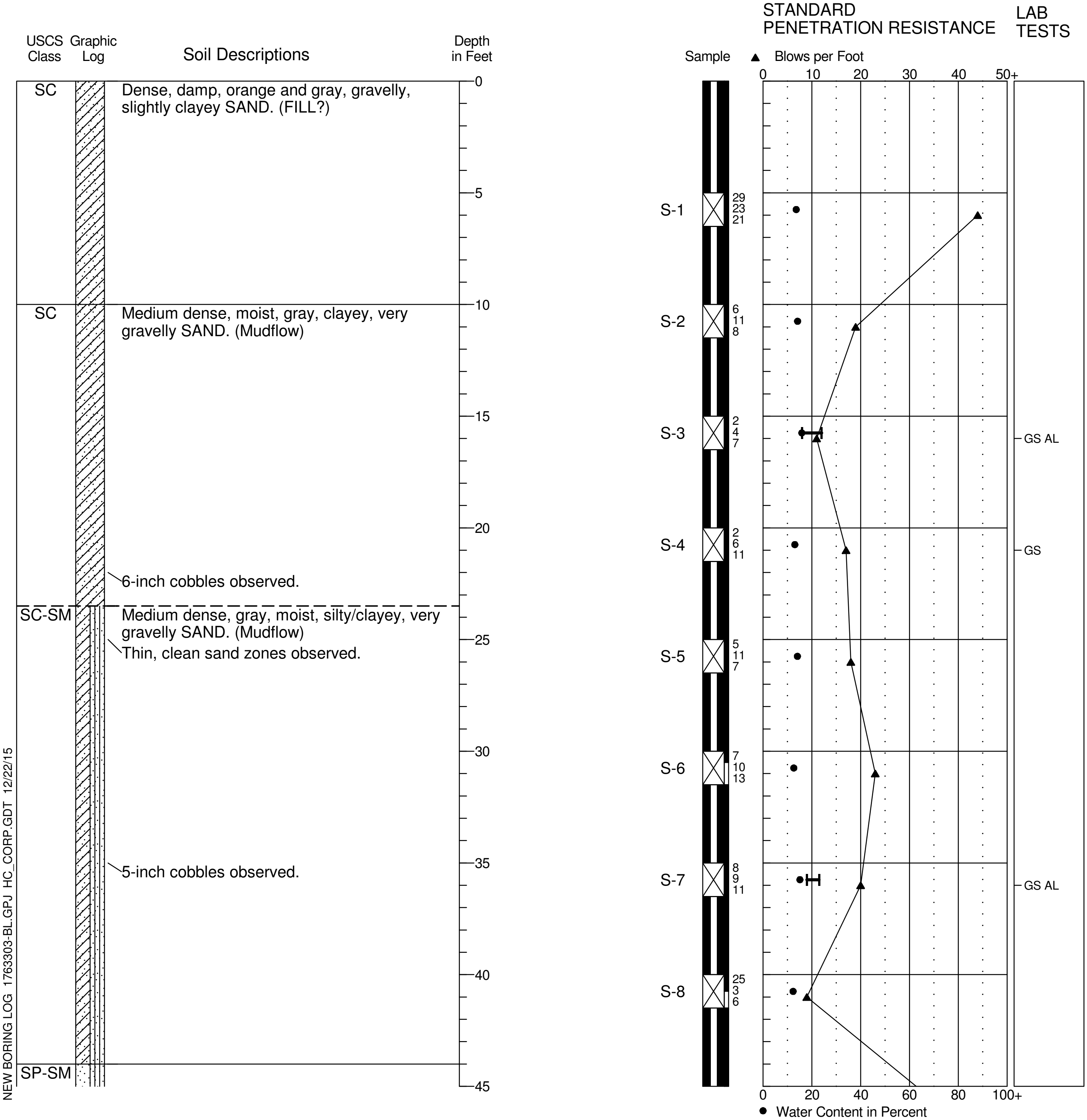
17633-03   11/15
Figure A-3

NEW BORING LOG 1763303-BL.GPJ HC_CORP.GDT 12/22/15

Boring Log BS-1

Location: See Site Exploration Plan.
Approximate Ground Surface Elevation: 707 Feet
Horizontal Datum:
Vertical Datum: NAVD88

Drill Equipment: Sonic Rig
Hammer Type: SPT
Hole Diameter: 8 inches
Logged By: M. Miller   Reviewed By: J. Bruce



1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

HARTCROWSER

17633-03   11/15

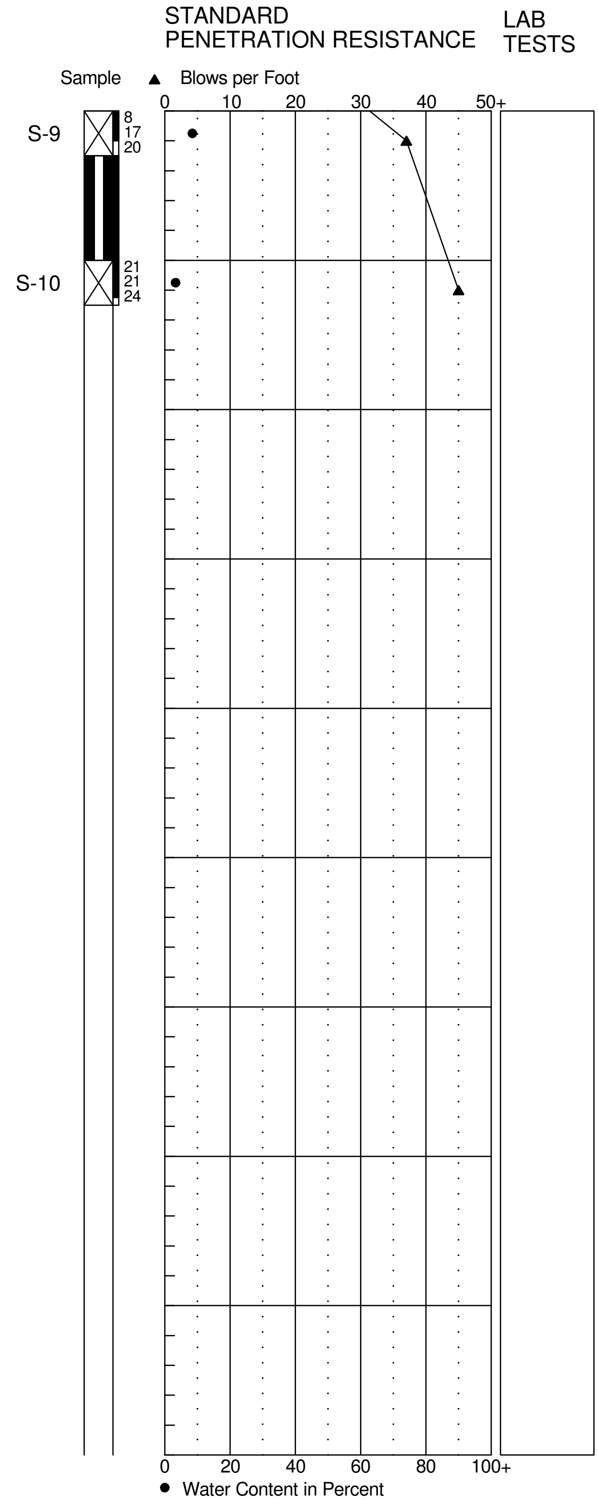
Figure A-4   1/2

Boring Log BS-1

Location: See Site Exploration Plan.
Approximate Ground Surface Elevation: 707 Feet
Horizontal Datum:
Vertical Datum: NAVD88

Drill Equipment: Sonic Rig
Hammer Type: SPT
Hole Diameter: 8 inches
Logged By: M. Miller Reviewed By: J. Bruce

USCS Class	Graphic Log	Soil Descriptions	Depth in Feet
SP-SM		Dense, dark brown, moist, gravelly, silty SAND. (cont'd)	45
GP		Dense, light brown, damp, very sandy GRAVEL.	
		Bottom of Boring at 50.0 Feet. Started 11/20/15. Completed 11/20/15.	50



NEW BORING LOG 1763303-BL.GPJ HC_CORP.GDT 12/22/15

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

HARTCROWSER

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Figure A-4 2/2

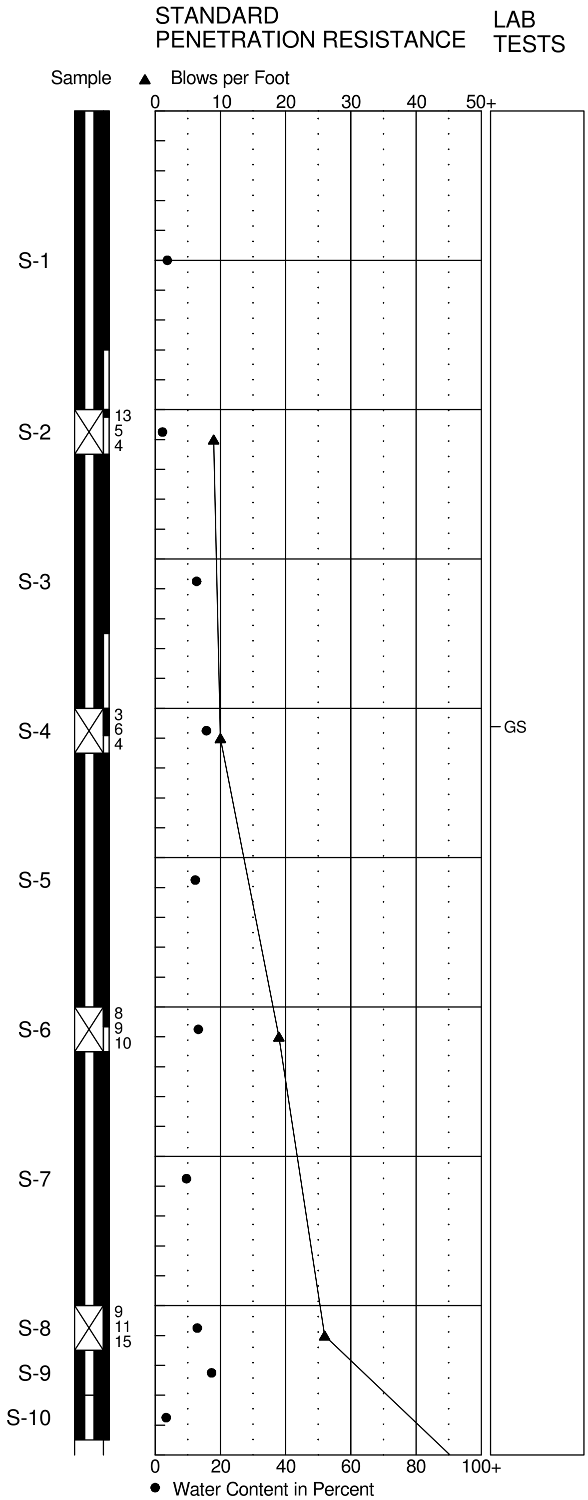
Boring Log BS-2

Location: See Site Exploration Plan.
Approximate Ground Surface Elevation: 703 Feet
Horizontal Datum:
Vertical Datum: NAVD88

Drill Equipment: Sonic Rig
Hammer Type: SPT
Hole Diameter: 8 inches
Logged By: W. McDonald   Reviewed By: J. Bruce

USCS Class	Soil Descriptions	Depth in Feet
SP	Medium dense, moist, gray, gravelly SAND with roots. (FILL)	0
GP	Loose, moist, brown, silty, sandy GRAVEL with cobbles. (FILL)	5
	Trace silt.	10
	Concrete.	14.5
GM	Wet, gray, very silty, clayey, sandy GRAVEL.	15
SC-SM	Medium dense to dense, moist to wet, gray, gravelly, very silty/clayey SAND. (Osceola Mudflow)	18
	Grades to gravelly, very sandy, with cobbles.	25
SM	Medium dense, moist, dark brown to red-brown, gravelly, very silty SAND with organics.	40
GP	Very dense, moist, gray-brown, slightly silty, sandy GRAVEL with cobbles.	43

Perched ▽ (approx. 15 ft)



NEW BORING LOG 1763303-BL.GPJ HC_CORP.GDT 12/22/15

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

HARTCROWSER

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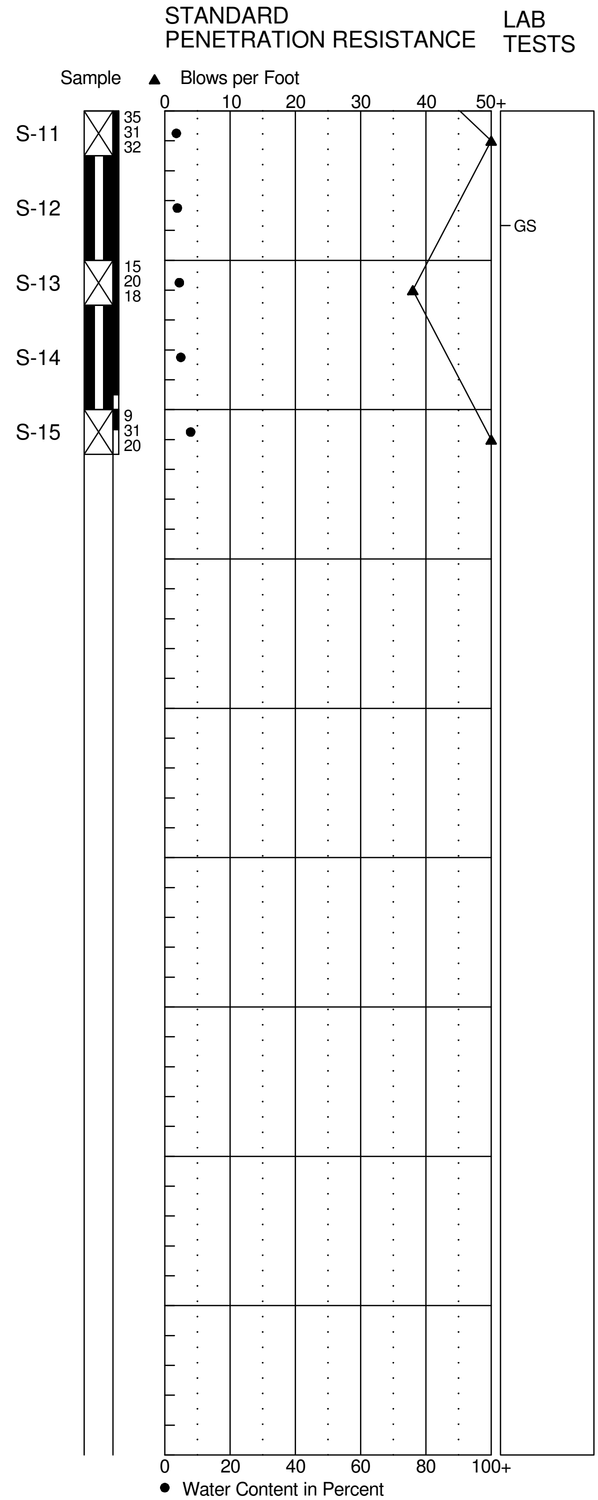
Figure A-5 1/2

Boring Log BS-2

Location: See Site Exploration Plan.
Approximate Ground Surface Elevation: 703 Feet
Horizontal Datum:
Vertical Datum: NAVD88

Drill Equipment: Sonic Rig
Hammer Type: SPT
Hole Diameter: 8 inches
Logged By: W. McDonald   Reviewed By: J. Bruce

USCS Class	Graphic Log	Soil Descriptions	Depth in Feet
GP		Very dense, moist, gray-brown, slightly silty, sandy GRAVEL with cobbles. (cont'd)	45
			50
		Layer of dense, gravelly sand.	∇ ATD
			55
		Bottom of Boring at 56.5 Feet. Started 11/19/15. Completed 11/19/15.	
			60
			65
			70
			75
			80
			85
			90



NEW BORING LOG 1763303-BL.GPJ HC_CORP.GDT 12/22/15

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

HARTCROWSER

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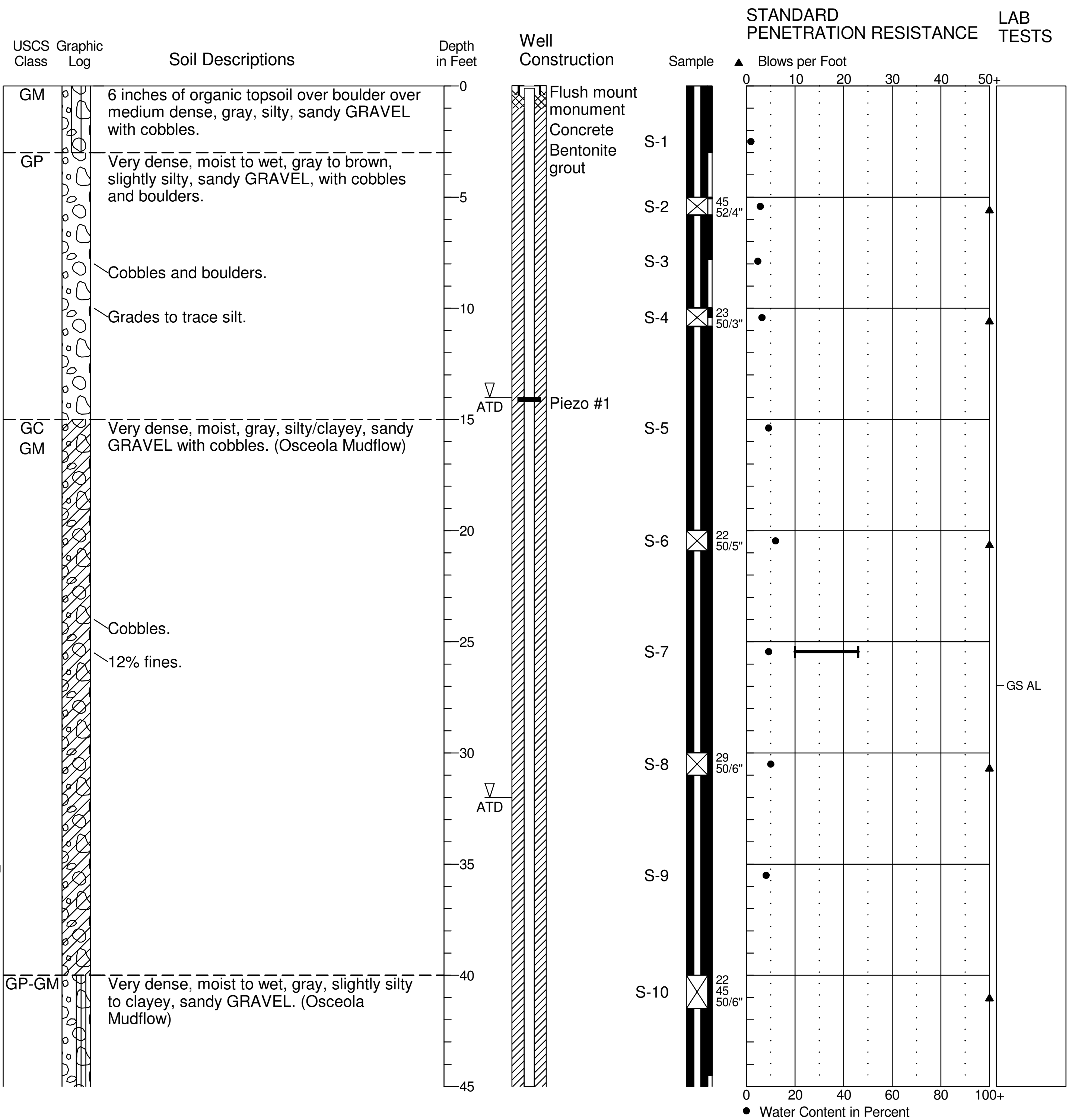
Figure A-5 2/2

Boring Log BS-3

Location: See Site Exploration Plan.
Approximate Ground Surface Elevation: 632 Feet
Horizontal Datum:
Vertical Datum: NAVD88

Drill Equipment: Terra Sonic Track
Hammer Type: Sonic + SPT
Hole Diameter: 8 inches
Logged By: W. McDonald Reviewed By: J. Bruce



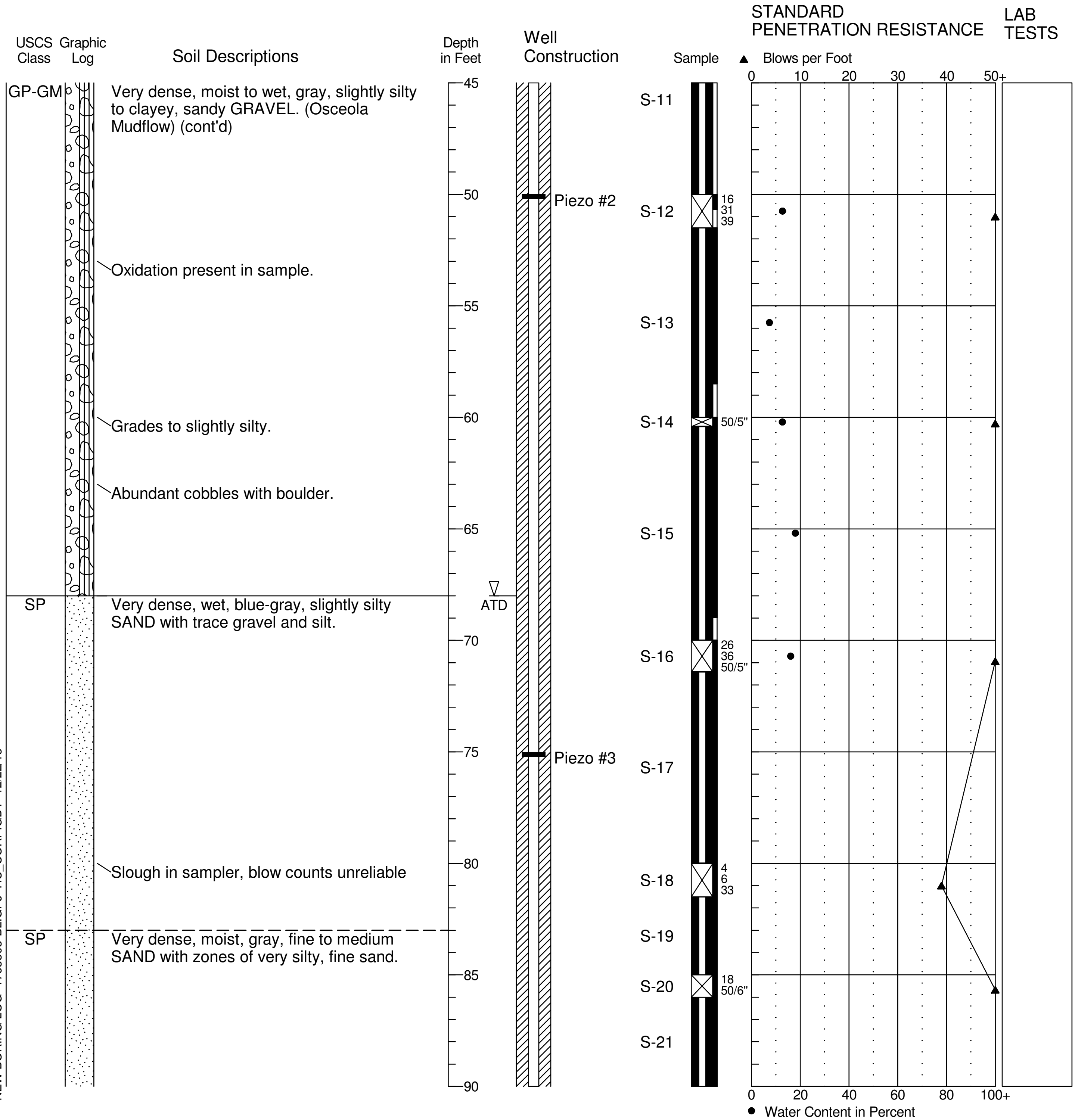
1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

HARTCROWSER

Boring Log BS-3

Location: See Site Exploration Plan.
Approximate Ground Surface Elevation: 632 Feet
Horizontal Datum:
Vertical Datum: NAVD88

Drill Equipment: Terra Sonic Track
Hammer Type: Sonic + SPT
Hole Diameter: 8 inches
Logged By: W. McDonald   Reviewed By: J. Bruce



NEW BORING LOG 17633303-BL.GPJ HC_CORP.GDT 12/22/15

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

HARTCROWSER

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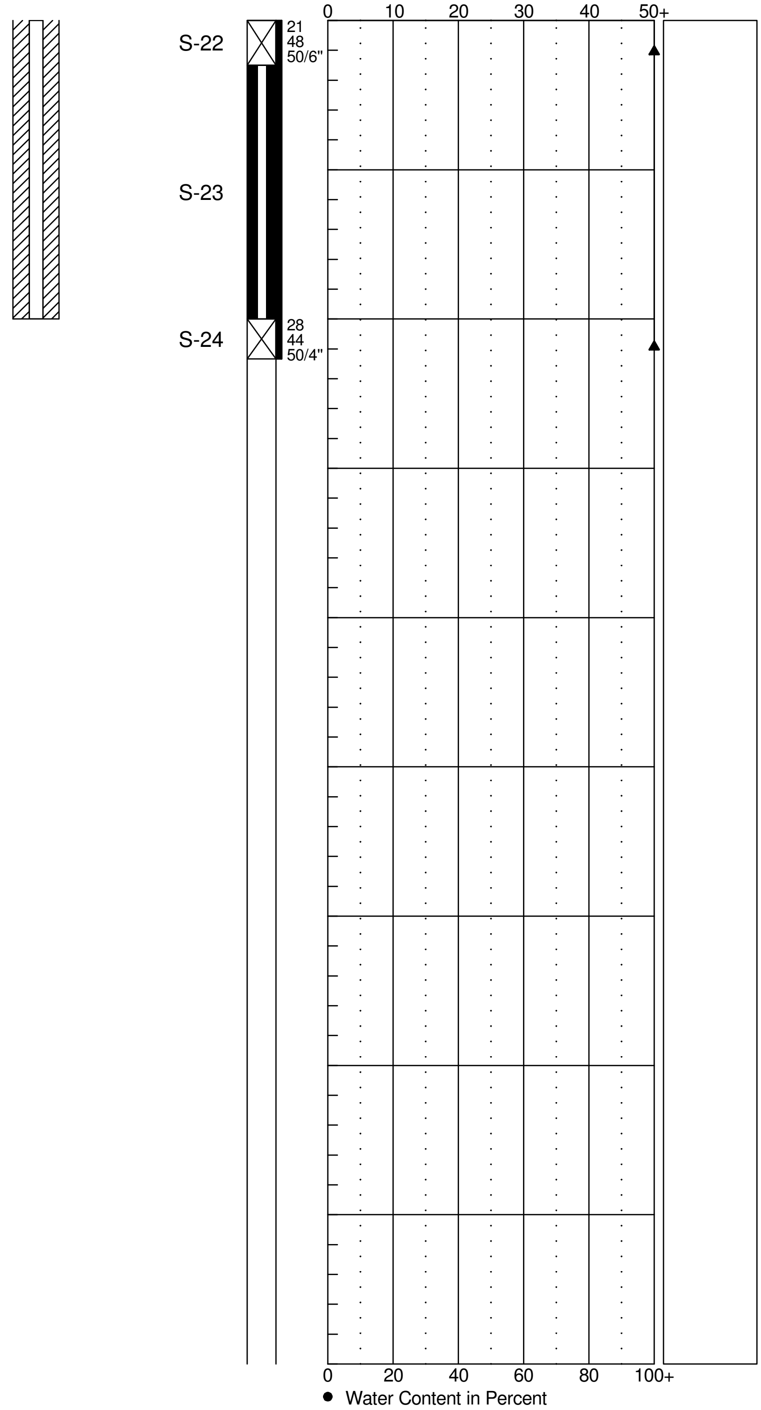
Figure A-6 *2/3*

Boring Log BS-3

Location: See Site Exploration Plan.
Approximate Ground Surface Elevation: 632 Feet
Horizontal Datum:
Vertical Datum: NAVD88

Drill Equipment: Terra Sonic Track
Hammer Type: Sonic + SPT
Hole Diameter: 8 inches
Logged By: W. McDonald   Reviewed By: J. Bruce

USCS Class	Soil Descriptions	Depth in Feet	Sample	Blows
SP	Very dense, moist, gray, fine to medium SAND with zones of very silty, fine sand. (cont'd) Scattered gravels.	90	S-22	21 48 50/6"
		95	S-23	
		100	S-24	28 44 50/4"
	Bottom of Boring at 101.3 Feet. Started 11/23/15. Completed 11/24/15. Ecology Well Tag #BJZ-005			



NEW BORING LOG 1763303-BL.GPJ HC_CORP.GDT 12/22/15

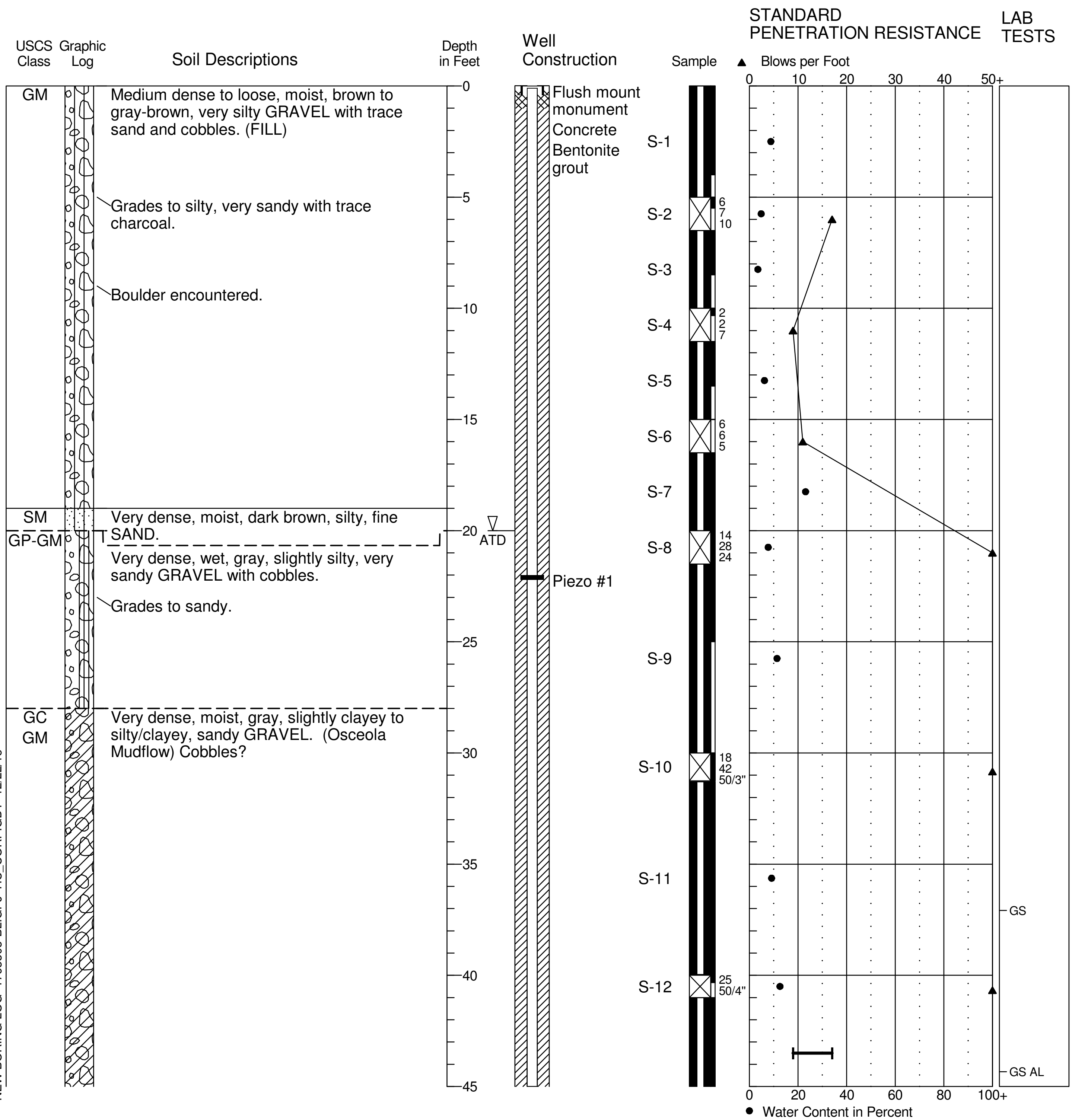
1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

HARTCROWSER

Boring Log BS-4

Location: See Site Exploration Plan.
Approximate Ground Surface Elevation: 648 Feet
Horizontal Datum:
Vertical Datum: NAVD88

Drill Equipment: Terra Sonic Track
Hammer Type: Sonic + SPT
Hole Diameter: 8 inches
Logged By: W. McDonald   Reviewed By: J. Bruce



1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

HARTCROWSER

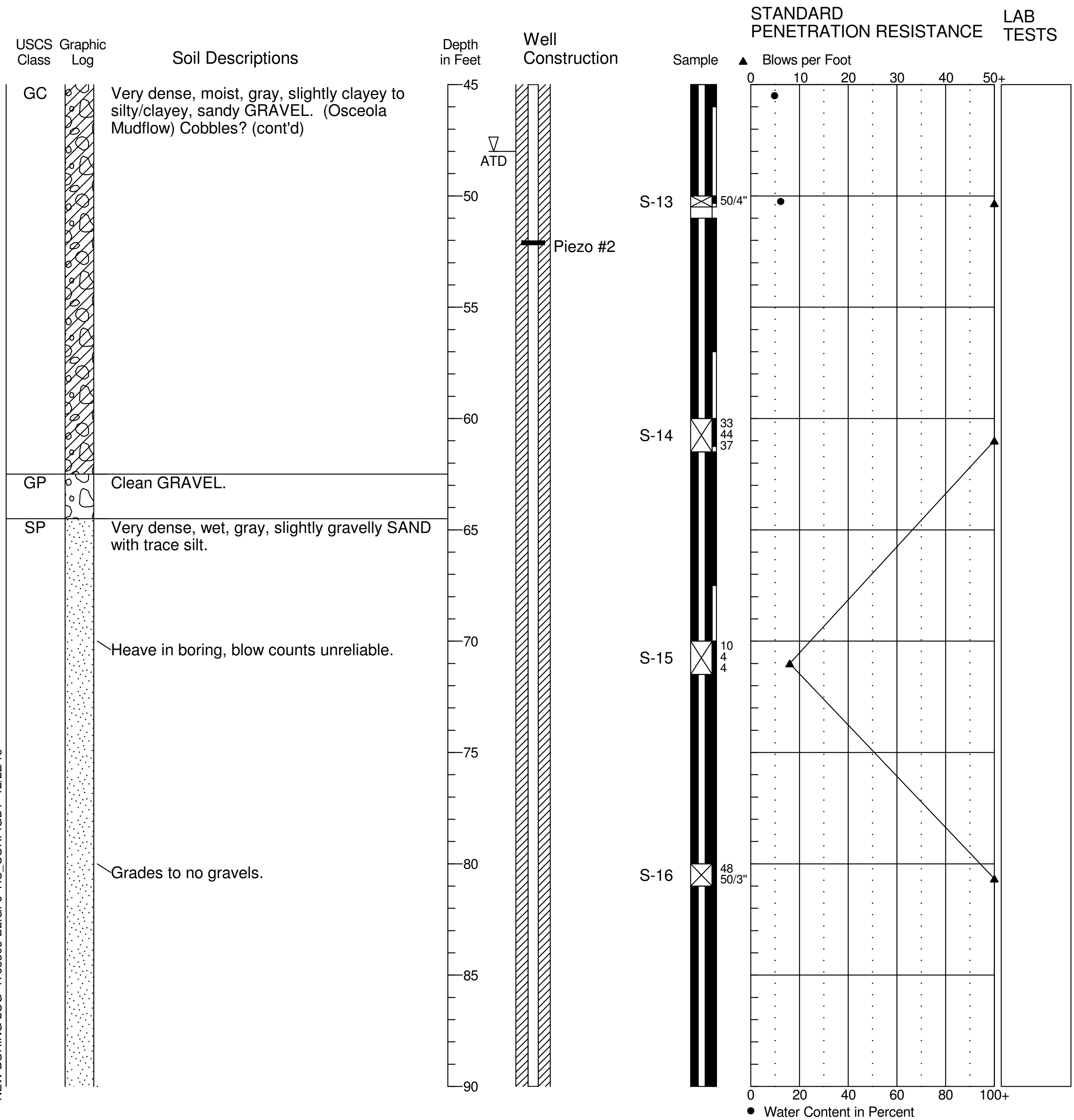
17633-03 *12/15*

Figure A-7 *1/3*

Boring Log BS-4

Location: See Site Exploration Plan.
Approximate Ground Surface Elevation: 648 Feet
Horizontal Datum:
Vertical Datum: NAVD88

Drill Equipment: Terra Sonic Track
Hammer Type: Sonic + SPT
Hole Diameter: 8 inches
Logged By: W. McDonald Reviewed By: J. Bruce



NEW BORING LOG 1763303-BL.GPJ HC_CORP.GDT 12/22/15

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

HARTCROWSER

Boring Log BS-4

Location: See Site Exploration Plan.
Approximate Ground Surface Elevation: 648 Feet
Horizontal Datum:
Vertical Datum: NAVD88

Drill Equipment: Terra Sonic Track
Hammer Type: Sonic + SPT
Hole Diameter: 8 inches
Logged By: W. McDonald   Reviewed By: J. Bruce

USCS Class	Soil Descriptions	Depth in Feet	Well Construction	Sample	Blows
SP	Very dense, wet, gray, slightly gravelly SAND with trace silt. (cont'd)	90	Piezo #3	S-17	5, 34, 50/4"
	Grades to slightly silty.				
		95			
	Grades to slightly gravelly with trace silt.	100		S-18	4, 22, 50/3"
	4-inch silt lenses observed.	105			
	Trace gravel.	110		S-19	4, 14, 50/5"
		115			
	Grades to no gravel.	120		S-20	4, 17, 37
	Trace silt and wood debris.	125			
	Bottom of Boring at 130.0 Feet.	130			
	Started 11/30/15.				
	Completed 12/02/15.				
	Ecology Well Tag #BJZ-006	135			

STANDARD PENETRATION RESISTANCE
▲ Blows per Foot: 0 10 20 30 40 50+
● Water Content in Percent: 0 20 40 60 80 100+

LAB TESTS

NEW BORING LOG 17633303-BL.GPJ HC_CORP.GDT 12/22/15

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

HARTCROWSER

17633-03 *12/15*

Figure A-7 *3/3*

Test Pit Log TP- 1

Location: See Site Exploration Plan.
Approximate Ground Surface Elevation: 732 Feet
Logged By: M. Miller Reviewed By: J. Thomas

Horizontal Datum:
Vertical Datum: NAVD88

USCS Class	Graphic Log	Soil Descriptions	Depth in Feet	Sample	Water Content in Percent	PID	LAB TESTS
SM		(Loose), moist to wet, black, silty SAND with organics. (FILL?)	0				
SM		(Dense), wet, light brown, silty, gravelly SAND with cobbles and oxidation.	5				
		Bottom of Test Pit at 6.0 Feet. Started 11/19/15. Completed 11/19/15.	10 15				

Test Pit Log TP- 2

Location: See Site Exploration Plan.
Approximate Ground Surface Elevation: 728 Feet
Logged By: M. Miller Reviewed By: J. Thomas

Horizontal Datum:
Vertical Datum: NAVD88

USCS Class	Graphic Log	Soil Descriptions	Depth in Feet	Sample	Water Content in Percent	PID	LAB TESTS
SW		(Loose), moist, dark brown, slightly silty, gravelly SAND. (FILL?)	0	1	13		GS
		Becomes silty.	5				
SM		(Dense), damp, brown, silty, gravelly to very gravelly SAND.					
		Bottom of Test Pit at 9.0 Feet. Started 11/19/15. Completed 11/19/15.	10 15				

NEW TEST PIT LOG 1763303-TP.GPJ HC_CORP.GDT 12/23/15

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater conditions, if indicated, are at time of excavation. Conditions may vary with time.

HARTCROWSER

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Figure A-8

Test Pit Log TP- 3

Location: See Site Exploration Plan.
Approximate Ground Surface Elevation: 725 Feet
Logged By: M. Miller   Reviewed By: J. Thomas

Horizontal Datum:
Vertical Datum: NAVD88

USCS Class	Graphic Log	Soil Descriptions	Depth in Feet	Sample	Water Content in Percent	PID	LAB TESTS
SM		(Medium dense), moist to wet, dark brown, silty SAND with organics. (FILL?)	0				
SW		(Very dense), moist, light brown, slightly silty, very gravelly SAND.	5				
		Bottom of Test Pit at 6.0 Feet. Started 11/19/15. Completed 11/19/15.	10				
			15				

Test Pit Log TP- 4

Location: See Site Exploration Plan.
Approximate Ground Surface Elevation: 727 Feet
Logged By: M. Miller   Reviewed By: J. Thomas

Horizontal Datum:
Vertical Datum: NAVD88

USCS Class	Graphic Log	Soil Descriptions	Depth in Feet	Sample	Water Content in Percent	PID	LAB TESTS
GM		(Loose), moist, dark brown, silty, very sandy GRAVEL. (FILL?)	0	1	16		GS
SM		(Medium dense to dense), moist, light brown to light gray, silty, very gravelly SAND.	5				
		Becomes orange and very silty.					
		Bottom of Test Pit at 9.0 Feet. Started 11/19/15. Completed 11/19/15.	10				
			15				

NEW TEST PIT LOG 1763303-TP.GPJ HC_CORP.GDT 12/23/15

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater conditions, if indicated, are at time of excavation. Conditions may vary with time.

HARTCROWSER

17633-03 *11/15*

Figure A-9

Test Pit Log TP- 5

Location: See Site Exploration Plan.
Approximate Ground Surface Elevation: 722 Feet
Logged By: M. Miller   Reviewed By: J. Thomas

Horizontal Datum:
Vertical Datum: NAVD88

USCS Class	Graphic Log	Soil Descriptions	Depth in Feet	Sample	Water Content in Percent	PID	LAB TESTS
GM		(Loose), damp, black, silty, sandy GRAVEL.	0				
SM		(Dense), damp, light brown, silty, very gravelly SAND with cobbles.					
		Bottom of Test Pit at 5.0 Feet. Started 11/19/15. Completed 11/19/15.	5				
			10				
			15				

Test Pit Log TP- 6

Location: See Site Exploration Plan.
Approximate Ground Surface Elevation: 716 Feet
Logged By: M. Miller   Reviewed By: J. Thomas

Horizontal Datum:
Vertical Datum: NAVD88

USCS Class	Graphic Log	Soil Descriptions	Depth in Feet	Sample	Water Content in Percent	PID	LAB TESTS
GP		(Medium dense), damp, brown, very sandy GRAVEL (Pit Run) with trace silt and organics.	0				
				1	7		GS
		Bottom of Test Pit at 5.0 Feet. Started 11/19/15. Completed 11/19/15.	5				
			10				
			15				

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater conditions, if indicated, are at time of excavation. Conditions may vary with time.

Test Pit Log TP- 7

Location: See Site Exploration Plan.
Approximate Ground Surface Elevation: 711 Feet
Logged By: M. Miller   Reviewed By: J. Thomas

Horizontal Datum:
Vertical Datum: NAVD88

USCS Class	Graphic Log	Soil Descriptions	Depth in Feet	Sample	Water Content in Percent	PID	LAB TESTS
SM		(Loose), damp, dark brown, slightly silty SAND with organics and trace gravel. (FILL?)	0				
SM		(Dense), damp, brown, silty, gravelly SAND with cobbles.					
		Organics and water running into hole.	5				
		Bottom of Test Pit at 9.0 Feet. Started 11/19/15. Completed 11/19/15.	10				
			15				

Test Pit Log TP- 8

Location: See Site Exploration Plan.
Approximate Ground Surface Elevation: 707 Feet
Logged By: M. Miller   Reviewed By: J. Thomas

Horizontal Datum:
Vertical Datum: NAVD88

USCS Class	Graphic Log	Soil Descriptions	Depth in Feet	Sample	Water Content in Percent	PID	LAB TESTS
SM		(Medium dense), moist, dark brown, silty SAND.	0				
SM		(Dense to very dense), moist, brown, silty, gravelly SAND with oxidation and cobbles.		1	14		GS
		Bottom of Test Pit at 6.0 Feet. Started 11/19/15. Completed 11/19/15.	5				
			10				
			15				

NEW TEST PIT LOG 1763303-TP.GPJ HC_CORP.GDT 12/23/15

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater conditions, if indicated, are at time of excavation. Conditions may vary with time.

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Figure A-11

Test Pit Log TP- 9

Location: See Site Exploration Plan.
Approximate Ground Surface Elevation: 704 Feet
Logged By: M. Miller   Reviewed By: J. Thomas

Horizontal Datum:
Vertical Datum: NAVD88

USCS Class	Soil Descriptions	Depth in Feet	Sample	Water Content in Percent	PID	LAB TESTS
SP-SM	(Medium dense to loose), moist, dark brown, slightly silty, gravelly SAND with organics. (FILL?)	0	1	12		GS
SM	(Medium dense), damp, orange, silty, gravelly SAND.					
SM/ML	(Dense), moist, gray, slightly silty, gravelly SAND (well-graded) to sandy SILT.	5				
	Bottom of Test Pit at 10.0 Feet. Started 11/19/15. Completed 11/19/15.	10				
		15				

Test Pit Log TP-10

Location: See Site Exploration Plan.
Approximate Ground Surface Elevation: 702 Feet
Logged By: M. Miller   Reviewed By: J. Thomas

Horizontal Datum:
Vertical Datum: NAVD88

USCS Class	Soil Descriptions	Depth in Feet	Sample	Water Content in Percent	PID	LAB TESTS
GW	(Loose), moist, brown, sandy GRAVEL with trace silt. (FILL?)	0				
		5				
SM	(Loose to medium dense), moist, light brown, silty, gravelly SAND. (FILL?)	10				
	Bottom of Test Pit at 14.0 Feet. Started 11/19/15. Completed 11/19/15.	15				

NEW TEST PIT LOG 1763303-TP.GPJ HC_CORP.GDT 12/23/15

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater conditions, if indicated, are at time of excavation. Conditions may vary with time.

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Figure A-12

Test Pit Log TP-11

Location: See Site Exploration Plan.
Approximate Ground Surface Elevation: 702 Feet
Logged By: M. Miller   Reviewed By: J. Thomas

Horizontal Datum:
Vertical Datum: NAVD88

USCS Class	Graphic Log	Soil Descriptions	Depth in Feet	Sample	Water Content in Percent	PID	LAB TESTS
GW		(Loose), moist, brown, sandy GRAVEL with trace silt. (FILL?)	0				
			5				
SM		(Dense), moist to damp, light brown, silty, gravelly SAND.					
		Bottom of Test Pit at 9.0 Feet. Started 11/19/15. Completed 11/19/15.	10				
			15				

Test Pit Log TP-12

Location: See Site Exploration Plan.
Approximate Ground Surface Elevation: 702 Feet
Logged By: M. Miller   Reviewed By: J. Thomas

Horizontal Datum:
Vertical Datum: NAVD88

USCS Class	Graphic Log	Soil Descriptions	Depth in Feet	Sample	Water Content in Percent	PID	LAB TESTS
GW		(Loose), wet, sandy GRAVEL. (Pit Run)	0				
			5				
		Bottom of Test Pit at 8.0 Feet. Started 11/19/15. Completed 11/19/15.	10				
			15				

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater conditions, if indicated, are at time of excavation. Conditions may vary with time.

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Figure A-13

Test Pit Log TP-13

Location: N 66746.9767 E 1346155.2994
Approximate Ground Surface Elevation: 700 Feet
Logged By: M. Miller Reviewed By: J. Thomas

Horizontal Datum:
Vertical Datum: NAVD88

USCS Class	Graphic Log	Soil Descriptions	Depth in Feet	Sample	Water Content in Percent	PID	LAB TESTS
SM		(Loose), damp, brown, slightly silty SAND. (FILL?)	0				
SW		(Very dense), damp, very gravelly SAND with trace silt.	5				
		Large cobbles.					
		Bottom of Test Pit at 7.0 Feet.					
		Started 11/19/15.					
		Completed 11/19/15.	10				
			15				

Test Pit Log TP-14

Location: N 66694.3993 E 1346058.5413
Approximate Ground Surface Elevation: 700 Feet
Logged By: M. Miller Reviewed By: J. Thomas

Horizontal Datum:
Vertical Datum: NAVD88

USCS Class	Graphic Log	Soil Descriptions	Depth in Feet	Sample	Water Content in Percent	PID	LAB TESTS
SM		(Loose), damp, light brown to dark brown,slightly gravelly, very silty SAND. (FILL?)	0				
				1	17		GS
			5				
SW		(Very dense), damp, light brown, gravelly SAND with trace silt, organics, and occasional cobbles.					
		Bottom of Test Pit at 9.0 Feet.					
		Started 11/19/15.	10				
		Completed 11/19/15.					
			15				

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater conditions, if indicated, are at time of excavation. Conditions may vary with time.

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Test Pit Log TP-15

Location: N 66674.4038 E 1346159.8957
Approximate Ground Surface Elevation: 700 Feet
Logged By: M. Miller Reviewed By: J. Thomas

Horizontal Datum:
Vertical Datum: NAVD88

USCS Class	Graphic Log	Soil Descriptions	Depth in Feet	Sample	Water Content in Percent	PID	LAB TESTS
SW		(Dense), damp, brown, gravelly SAND with trace silt.	0				
SM		(Loose), damp, brown, slightly silty SAND.					
SM		(Very dense), damp, brown, very gravelly, slightly silty SAND with organics.					
		Bottom of Test Pit at 5.0 Feet. Started 11/19/15. Completed 11/19/15.	5				
			10				
			15				

Test Pit Log TP-16

Location: N 66398.3492 E 1345717.2175
Approximate Ground Surface Elevation: 640 Feet
Logged By: M. Miller Reviewed By: J. Thomas

Horizontal Datum:
Vertical Datum: NAVD88

USCS Class	Graphic Log	Soil Descriptions	Depth in Feet	Sample	Water Content in Percent	PID	LAB TESTS
SW		(Loose), damp, brown, slightly gravelly SAND with trace silt and organics. (FILL?)	0				
SW-SM		(Very dense), moist to wet, brown, silty, very gravelly SAND to sandy GRAVEL.	5	1	15		GS
		Bottom of Test Pit at 7.5 Feet. Started 11/19/15. Completed 11/19/15.					
			10				
			15				

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater conditions, if indicated, are at time of excavation. Conditions may vary with time.

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Figure A-15

Test Pit Log TP-17

Location: N 66246.2517 E 1345634.1099
Approximate Ground Surface Elevation: 638 Feet
Logged By: M. Miller   Reviewed By: J. Thomas

Horizontal Datum:
Vertical Datum: NAVD88

USCS Class	Graphic Log	Soil Descriptions	Depth in Feet	Sample	Water Content in Percent	PID	LAB TESTS
SM		(Loose), damp, brown, slightly silty SAND with organics. (FILL?)	0		19		GS
		Hit water at 6 feet.	5				
SC		(Loose), wet, gray, clayey SAND.					
GP		(Very dense), wet, gray, sandy GRAVEL.					
		Bottom of Test Pit at 9.0 Feet. Started 11/19/15. Completed 11/19/15.	10				
			15				

Test Pit Log TP-18

Location: See Site Exploration Plan.
Approximate Ground Surface Elevation: 632 Feet
Logged By: M. Miller   Reviewed By: J. Thomas

Horizontal Datum:
Vertical Datum: NAVD88

USCS Class	Graphic Log	Soil Descriptions	Depth in Feet	Sample	Water Content in Percent	PID	LAB TESTS
SM		(Medium dense to loose), moist, brown, silty SAND with organics. (FILL?)	0				
GW		(Medium dense to very dense), moist, brown, sandy GRAVEL with cobbles.	5				
		Bottom of Test Pit at 6.0 Feet. Started 11/19/15. Completed 11/19/15.					
			10				
			15				

NEW TEST PIT LOG 1763303-TP.GPJ HC_CORP.GDT 12/23/15

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater conditions, if indicated, are at time of excavation. Conditions may vary with time.

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Figure A-16

APPENDIX B
Geotechnical Laboratory Tests

APPENDIX B

Geotechnical Laboratory Tests

Laboratory tests were performed to evaluate the basic index and geotechnical engineering properties of the site soils. Only disturbed samples from test pit excavations and SPT split spoons were tested. The tests performed and the procedures followed are outlined below.

Soil Classification

Soil samples from the explorations were visually classified in the field; classifications were verified in our relatively controlled laboratory environment. Field and laboratory observations were density/consistency, moisture, and grain size and plasticity estimates. We used laboratory tests such as Atterberg limits determinations and grain size analysis to check classifications of selected samples. Soil was classified in general accordance with the Unified Soil Classification (USC) System, ASTM D2487, as presented on Figure B-1.

Water Content Determination

Water content was determined for a representative number of samples recovered in the explorations in general accordance with ASTM D2216 as soon as possible after their arrival in our laboratory. The results of these tests are plotted at the respective sample depths on the exploration logs. In addition, water content is routinely determined for samples subjected to other testing. These results are also presented on the exploration logs.

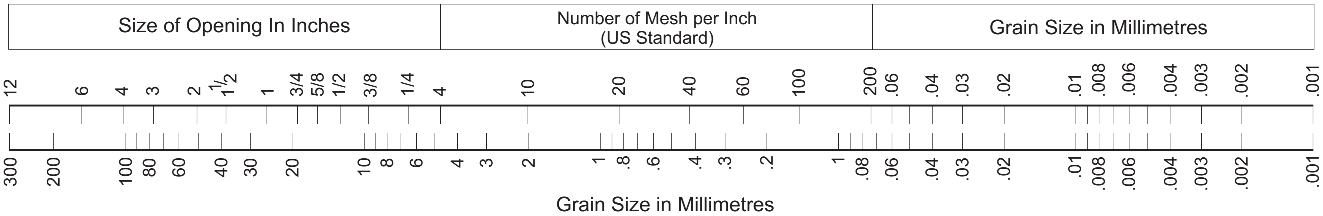
Atterberg Limits

We determined Atterberg limits for selected fine-grained soil samples. The liquid limit and plastic limit were determined in general accordance with ASTM D4318-84. The results of the Atterberg limits analyses and the plasticity characteristics are summarized in Figures B-8. This figure relates the plasticity index (liquid limit minus the plastic limit) to the liquid limit for the purpose of fine grained soil classification. The results of the Atterberg limits tests are shown graphically on the boring logs as well as, where applicable, on figures presenting other test results.

Grain Size Analysis

Grain size distribution was analyzed on representative samples in general accordance with ASTM D422. Wet sieve analysis was used to determine the size distribution greater than the U.S. No. 200 mesh sieve. The results of the tests are presented as curves on Figures B-2 through B-7, which plot percent finer by weight versus grain size.

Unified Soil Classification (USC) System
Soil Grain Size



COBBLES	GRAVEL	SAND	SILT and CLAY
Coarse-Grained Soils			Fine-Grained Soils

Coarse-Grained Soils

G W	G P	G M	G C	S W	S P	S M	S C
Clean GRAVEL <5% fines		* GRAVEL with >12% fines		Clean SAND <5% fines		* SAND with >12% fines	
GRAVEL >50% coarse fraction larger than No. 4				SAND >50% coarse fraction smaller than No. 4			
Coarse-Grained Soils >50% larger than No. 200 sieve							

G W and S W $\left(\frac{D_{60}}{D_{10}}\right)$ >4 for G W, >6 for S W & $1 \leq \left(\frac{(D_{30})^2}{D_{10} \times D_{60}}\right) \leq 3$

G P and S P Clean GRAVEL or SAND not meeting requirements for G W and S W

G M and S M Atterberg limits below A line with PI <4

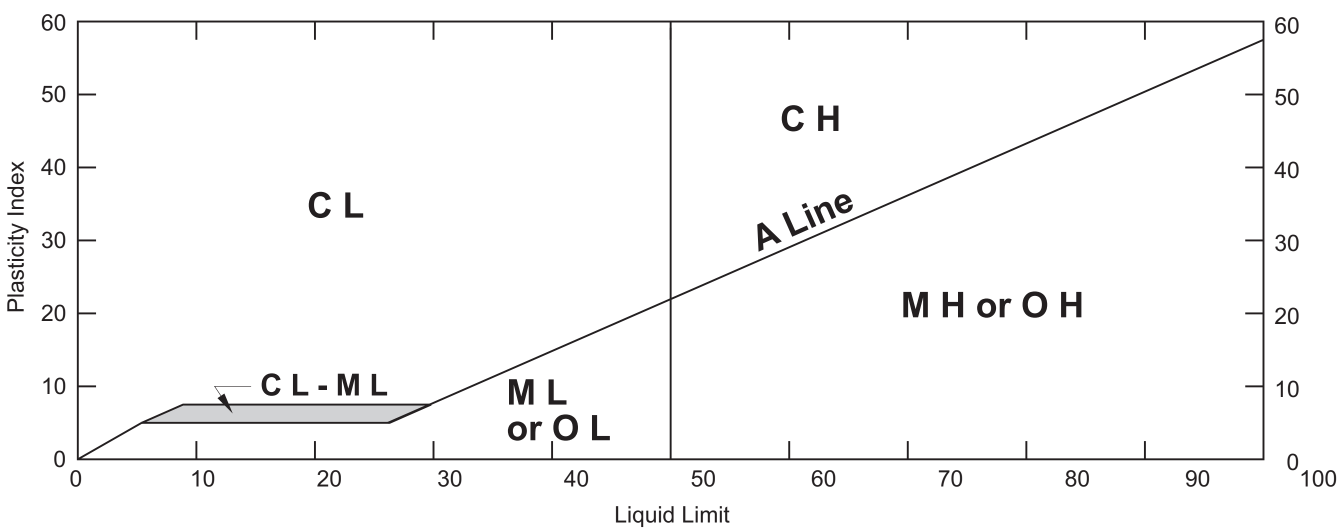
G C and S C Atterberg limits above A Line with PI >7

* Coarse-grained soils with percentage of fines between 5 and 12 are considered borderline cases requiring use of dual symbols.

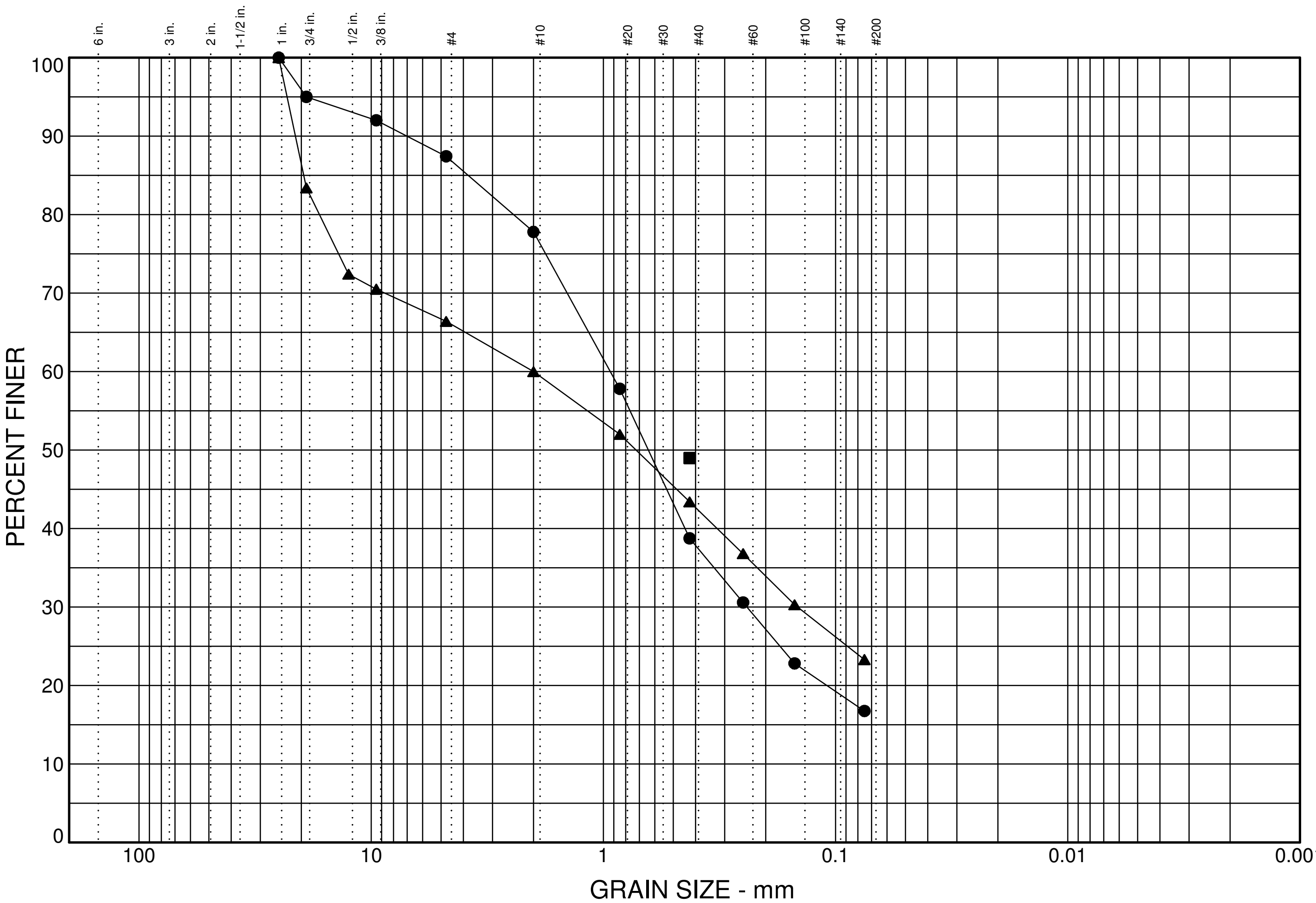
D_{10}, D_{30}, and D_{60} are the particles diameter of which 10, 30, and 60 percent, respectively, of the soil weight are finer.

Fine-Grained Soils

M L	C L	O L	M H	C H	O H	Pt
SILT	CLAY	Organic	SILT	CLAY	Organic	Highly Organic Soils
Soils with Liquid Limit <50%			Soils with Liquid Limit >50%			
Fine-Grained Soils >50% smaller than No. 200 sieve						



Particle Size Distribution Test Report



	% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	12.6	70.7	16.8	
■	0.0	0.0			
▲	0.0	33.6	43.1	23.3	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●			3.816	0.933	0.64	0.24				
■	24	8								
▲			19.509	2	0.723	0.146				

MATERIAL DESCRIPTION	USCS	NAT. MOIST.
● gravelly silty SAND	SM	7.9%
■ gravelly clayey SAND	SC	15.9%
▲ silty, very gravelly SAND	SM	13.0%

Remarks:
●

■

▲

Project: Foothills Trail Phase II

Client: Huitt-Zollars

● Source: BA-1 Sample No.: S-6 Depth: 27.5 to 29.0
■ Source: BS-1 Sample No.: S-3 Depth: 15.0 to 16.5
▲ Source: BS-1 Sample No.: S-4 Depth: 20.0 to 21.5

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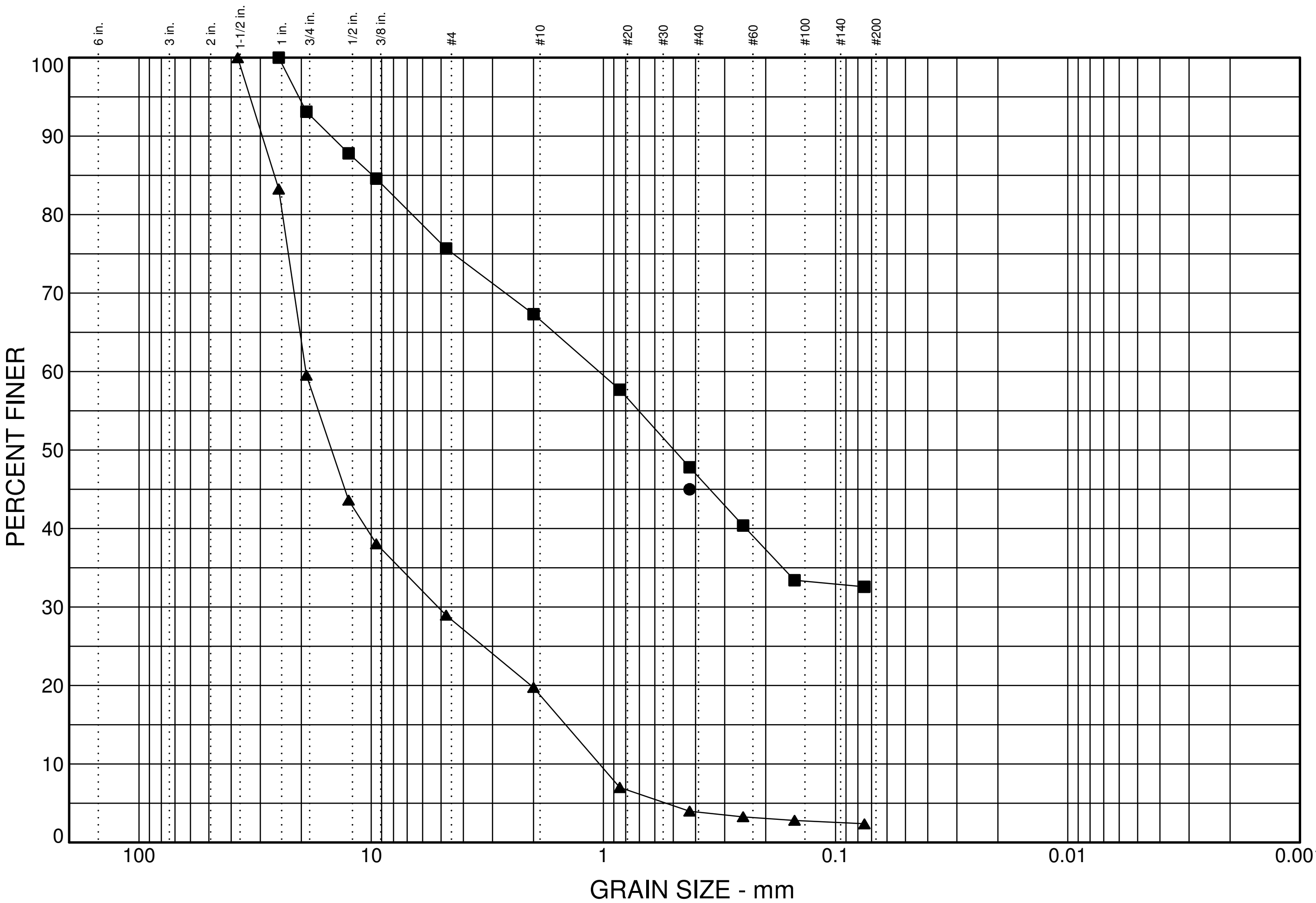
Figure B-2

GRAIN SIZE 1763303-BL.GPJ HC_CORP.GDT 12/22/15

Particle Size Distribution Test Report



	% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	0.0			
■	0.0	24.3	43.1	32.6	
▲	0.0	71.0	26.6	2.4	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●	23	5								
■			9.832	1.043	0.496					
▲			26.072	19.101	14.785	5.14	1.453	1.039	1.33	18.39

	MATERIAL DESCRIPTION	USCS	NAT. MOIST.
●	silty, very gravelly SAND	SM	15.1%
■	gravelly, very silty SAND	SM	15.7%
▲	sandy GRAVEL, trace silt	GW	3.8%

Remarks:

●

■

▲

Project: Foothills Trail Phase II

Client: Huitt-Zollars

●	Source: BS-1	Sample No.: S-7	Depth: 35.0 to 36.5
■	Source: BS-2	Sample No.: S-4	Depth: 20.0 to 21.5
▲	Source: BS-2	Sample No.: S-12	Depth: 46.5 to 50.0

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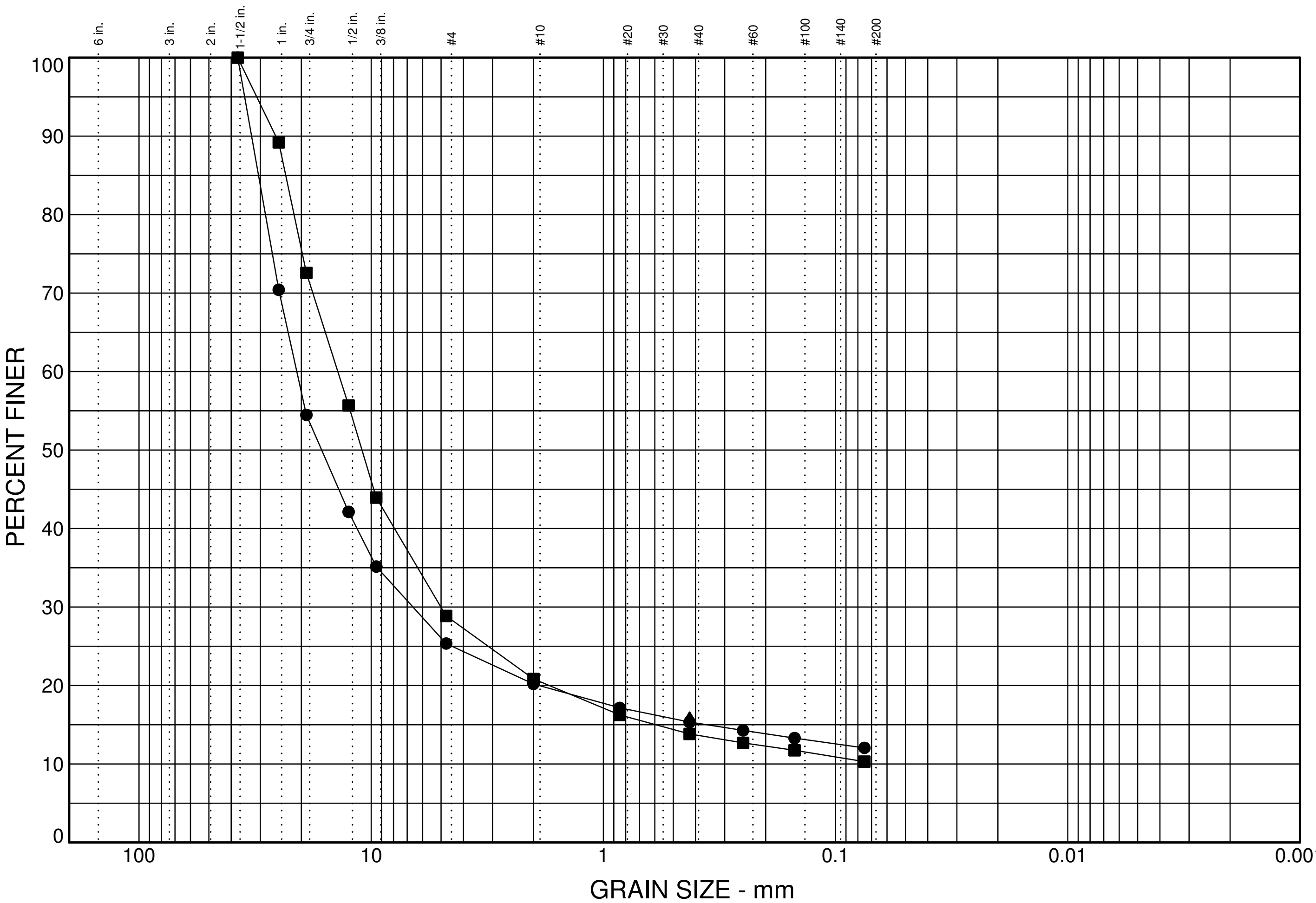
Figure B-3

GRAIN SIZE 1763303-BL.GPJ HC_CORP.GDT 12/22/15

Particle Size Distribution Test Report



	% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	74.6	13.3	12.1	
■	0.0	71.1	18.6	10.3	
▲	0.0	0.0			

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●	46	26	30.53	20.891	16.318	6.593	0.359		88.30	886.42
■			23.321	13.914	10.945	5.005	0.593		27.84	215.17
▲	34	16								

MATERIAL DESCRIPTION	USCS	NAT. MOIST.
● clayey sandy GRAVEL	GC	9.2%
■ slightly clayey, sandy GRAVEL	GP-GC	9.1%
▲ clayey sandy GRAVEL	GC	9.7%

Remarks:

●

■

▲

Project: Foothills Trail Phase II

Client: Huitt-Zollars

● Source: BS-3 Sample No.: S-7 Depth: 20.9 to 30.0
■ Source: BS-4 Sample No.: S-11 Depth: 31.3 to 40.0
▲ Source: BS-4 Sample No.: Depth: 41.0 to 50.0

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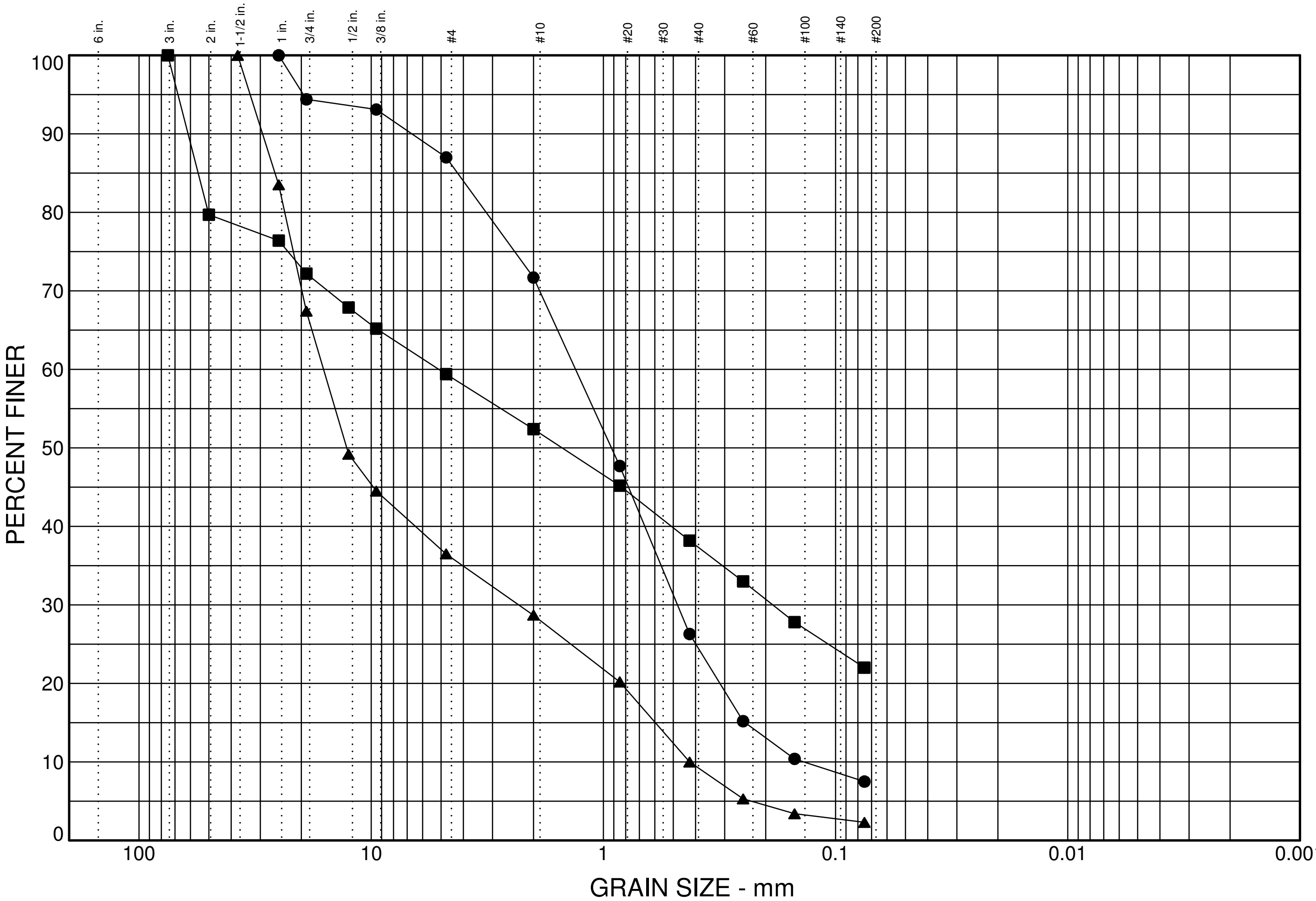
Figure B-4

GRAIN SIZE 1763303-BL.GPJ HC_CORP.GDT 12/22/15

Particle Size Distribution Test Report



	% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	13.0	79.5	7.5	
■	0.0	40.6	37.4	22.0	
▲	0.0	63.5	34.2	2.3	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●			4.242	1.318	0.923	0.479	0.245	0.136	1.28	9.67
■			55.583	5.103	1.504	0.186				
▲			25.939	16.026	12.732	2.31	0.597	0.425	0.78	37.71

MATERIAL DESCRIPTION	USCS	NAT. MOIST.
● slightly silty, gravelly SAND	SW	13.3%
■ silty, very sandy GRAVEL	GM	15.9%
▲ very sandy GRAVEL, trace silt	GP	6.9%

Remarks:
●

■

▲

Project: Foothills Trail

Client: Huitt-Zollars

● Source: TP- 2	Sample No.: 1	Depth: 0.5 to 1.0
■ Source: TP- 4	Sample No.: 1	Depth: 1.5 to 2.3
▲ Source: TP- 6	Sample No.: 1	Depth: 1.5 to 2.5

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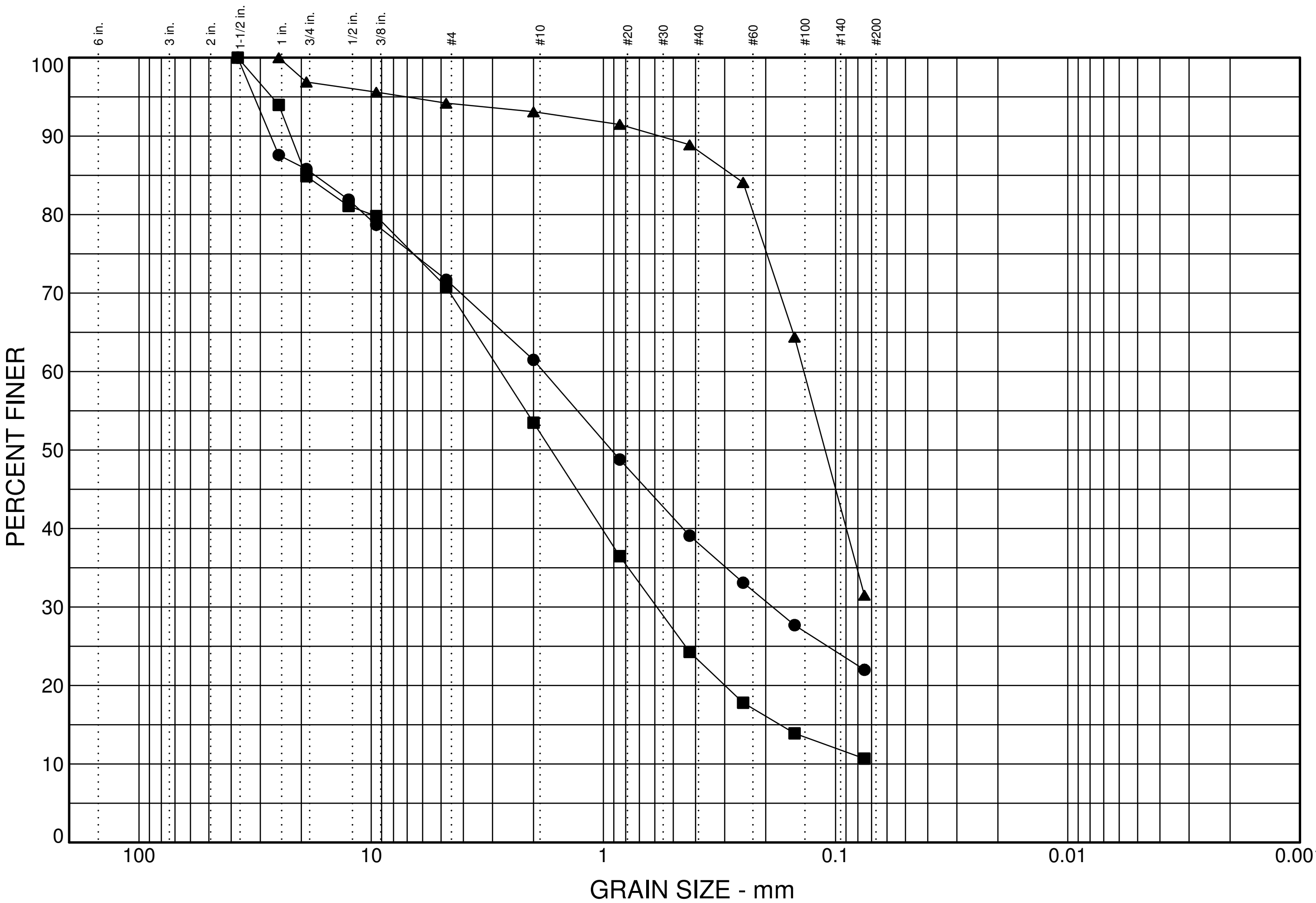
Figure B-5

Particle Size Distribution Test Report



	% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	28.3	49.7	22.0	
■	0.0	29.2	60.1	10.7	
▲	0.0	5.8	62.7	31.5	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●			17.436	1.808	0.922	0.186				
■			19.057	2.768	1.677	0.588	0.173		1.94	42.95
▲			0.276	0.137	0.111					

MATERIAL DESCRIPTION	USCS	NAT. MOIST.
● silty gravelly SAND	SM	13.9%
■ slightly silty, gravelly SAND	SP-SM	12.0%
▲ slightly gravelly, very silty SAND	SM	17.2%

Remarks:
●

■

▲

Project: Foothills Trail

Client: Huitt-Zollars

● Source: TP- 8	Sample No.: 1	Depth: 0.0 to 5.0
■ Source: TP- 9	Sample No.: 1	Depth: 1.5 to 2.5
▲ Source: TP-14	Sample No.: 1	Depth: 0.0 to 5.0

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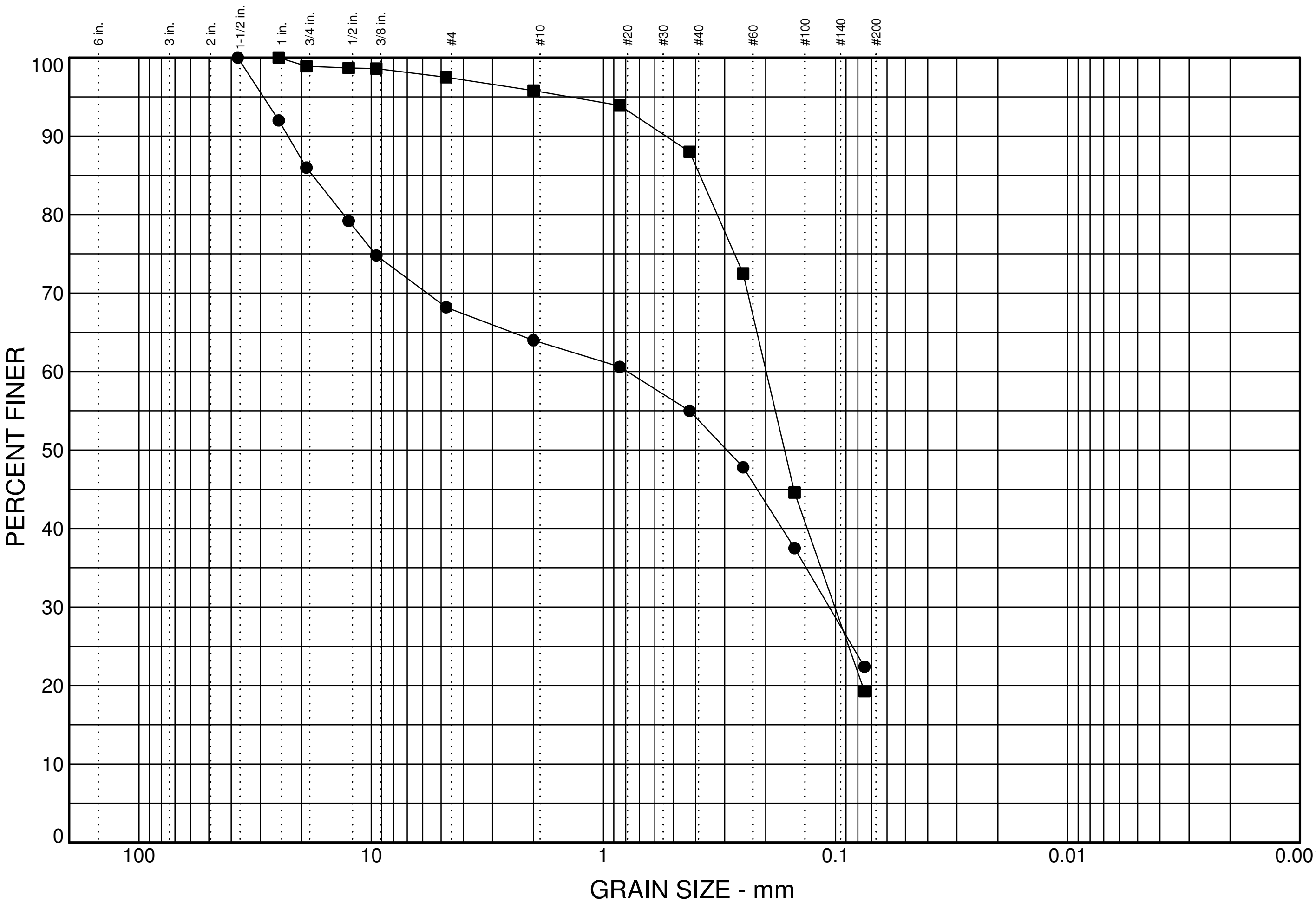
Figure B-6

Particle Size Distribution Test Report



	% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	31.8	45.8	22.4	
■	0.0	2.5	78.2	19.3	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●			17.865	0.789	0.294	0.106				
■			0.384	0.199	0.166	0.101				

MATERIAL DESCRIPTION	USCS	NAT. MOIST.
● silty, very gravelly SAND	SM	15.4%
■ silty SAND, trace gravel	SM	18.6%

Remarks:
●

■ minor organic contents

Project: Foothills Trail

Client: Huitt-Zollars

● Source: TP-16 Sample No.: 1 Depth: 4.0 to 6.0
■ Source: TP-17 Sample No.: Depth: 0.0 to 3.0

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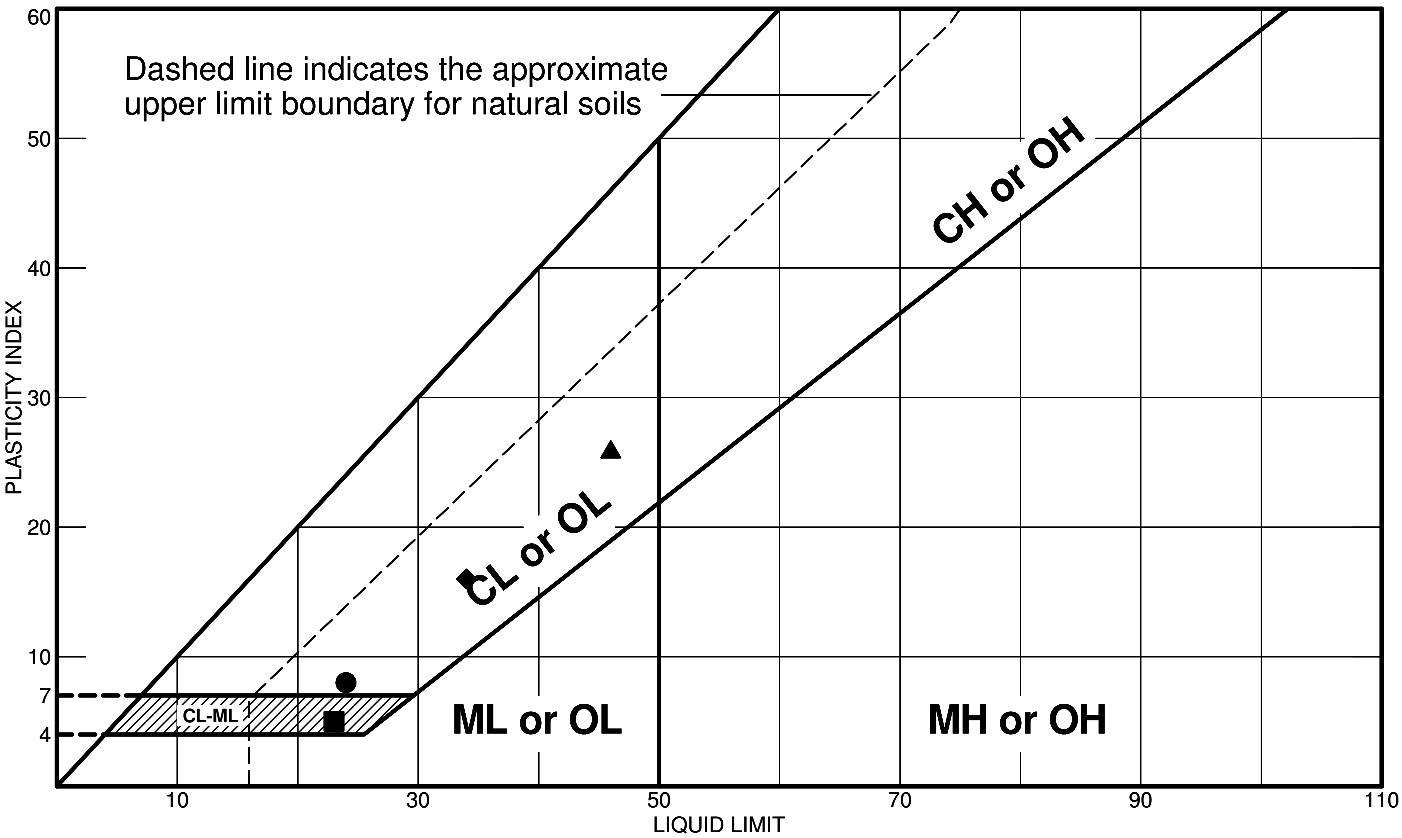
Figure B-7

Liquid and Plastic Limits Test Report



Location + Description	LL	PL	PI	-200	USCS
● Source: BS-1 Sample No.: S-3 Depth: 15 gravelly clayey SAND	24	16	8		SC
■ Source: BS-1 Sample No.: S-7 Depth: 35 silty, very gravelly SAND	23	18	5		SM
▲ Source: BS-3 Sample No.: S-7 Depth: 20.9 clayey sandy GRAVEL	46	20	26	12	GC
◆ Source: BS-4 Sample No.: Depth: 41 clayey sandy GRAVEL	34	18	16		GC

Remarks:
●
■
▲
◆

Project: Foothills Trail Phase II

Client: Huitt-Zollars

Location: Enumclaw, WA

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Figure B- 8

ATTERBERG LIMITS 1763303-BL.GPJ HC_CORP.GDT 2/1/16

APPENDIX C
Historical Explorations

APPENDIX C

Historical Explorations

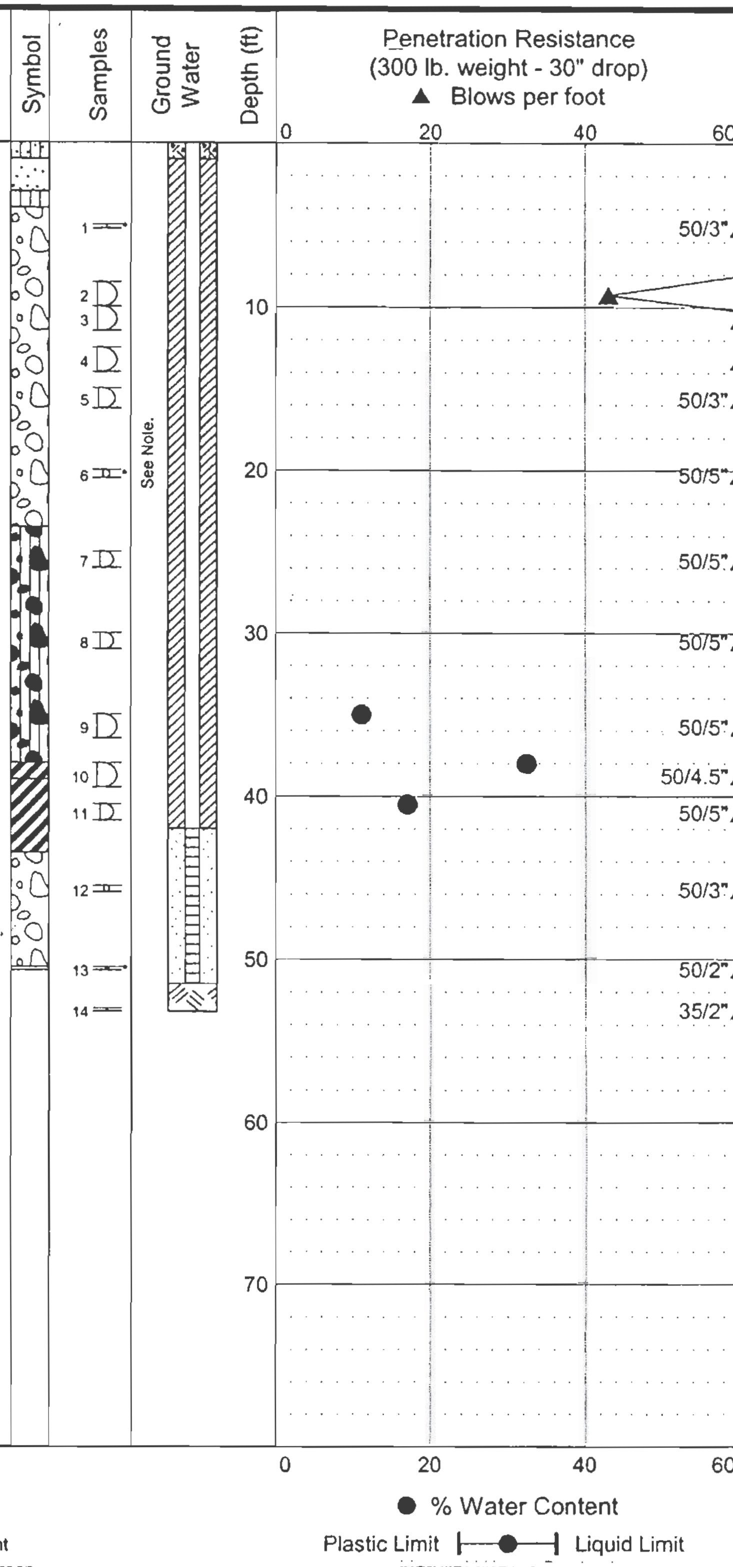
In addition to the explorations and laboratory test results presented in Appendices A and B, we reviewed historical test pit explorations completed by Shannon and Wilson in 2000 and Converse Davis Dixon in 1977 to gain a better understanding of the subsurface conditions in unexplored portions of the site.

Logs of these previous explorations that were selected as relevant to this project are presented in this appendix for reference only; Hart Crowser is not responsible for the accuracy or completeness of the information presented in these logs.

SOIL DESCRIPTION Ground Surface: Approx. 627 Ft.	Depth (ft)
Loose, dark brown, silty, gravelly, fine to medium SAND; moist; organics; (Topsoil) SM.	1.0
Loose, dark gray with red flecks, fine to medium SAND; trace of silt; moist; SP.	3.0
Loose, brown, sandy SILT; trace clay and gravel; moist; scattered organics; ML.	4.0
Medium dense to very dense, gray, gravelly, sandy COBBLES and BOULDERS; trace of silt; wet to saturated; gravel, cobbles, and boulders subangular to rounded; (White River alluvium/reworked Osceola Mudflow) GP.	23.5
Very dense, gray to gray-green, silty, sandy, clayey GRAVEL; moist; scattered cobbles and possibly boulders; scattered organics; (intact Osceola Mudflow) GM/GC.	38.0
Very stiff, blue-green with white flecks, CLAY; trace of silt and sand; dry; volcanic glass shards, ash, frothy texture; (Mudflow deposit; Pre-Osceola) CH.	39.0
Very dense/stiff, gray, gravelly, sandy SILT/CLAY; dry to moist; scattered organics, volcanic glass shards, ash, and possibly coal or obsidian clasts; (Mudflow deposit; Pre-Osceola) CH/MH.	43.5
Very dense, cobbly gravel (no sample recovered on two attempts; description inferred from drilling action); (Alluvium or glacial outwash) GP.	50.5
No recovery	50.7

BOTTOM OF BORING
COMPLETED 5/5/2000

Note: **Water below 43.5 feet under artesian condition; Flow rate = 5 gpm. Pressure head is at 14.8 feet above ground surface.**



LEGEND

- • Sample Not Recovered
- 2-inch O.D. Split Spoon Sample
- 3-inch O.D. Shelby Tube Sample
- 3.5" O.D. Split Spoon Sample
- Surface Seal
- Annular Sealant
- Piezometer Screen
- Grout
- Ground Water Level ATD
- Ground Water Level in Well

NOTES

1. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
2. The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
3. Groundwater level, if indicated above, is for the date specified and may vary.
4. Refer to KEY for explanation of "Symbols" and definitions.
5. USCS designation is based on visual-manual classification and selected laboratory index testing.

White River Pipeline Crossing
Buckley, Washington

LOG OF BORING B-1

May 2000 21-1-09002-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 5

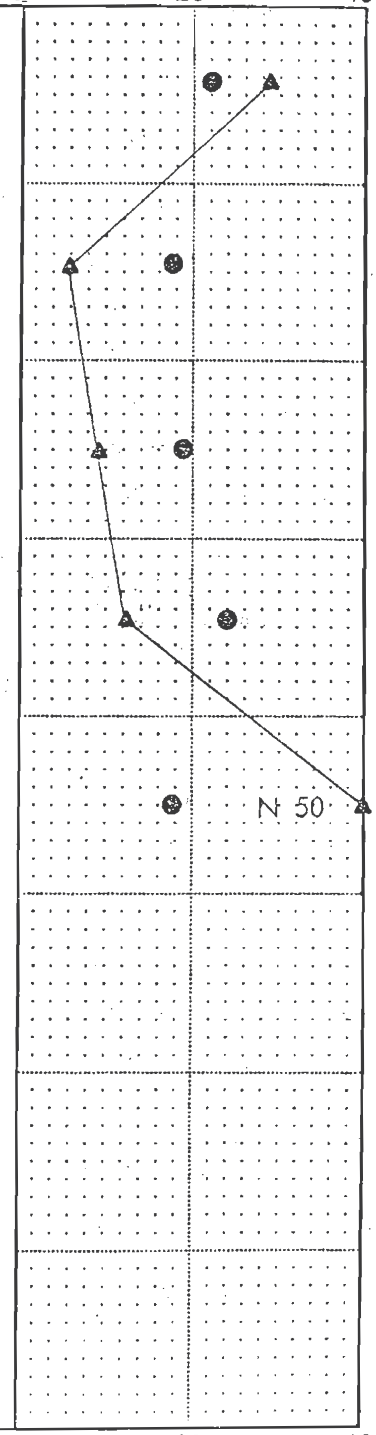
MASTER LOG 21-09002.GPJ SHAN WIL.GDT 5/21/00 Log: TJS Rev: Typ: EET

DATE DRILLED 5/31/77

LOG OF BORING NO. 4D

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED ARE A SIMPLIFICATION OF ACTUAL CONDITIONS ENCOUNTERED.

ELEVATION	DEPTH IN FEET	SAMPLES *	SYMBOL	DESCRIPTION	BLOWS/6"	DRY DENSITY LB/CU FT.
717			GW	road ballast, sand & gravel fill		
		1-A	SC	Mottled orange & purplish-gray, fine to med., clayey SAND w/occ. gravel to 1/2" dia.; m. dense, moist (Osceola Mudflow)	6 16 13	
712	5					
		2-A		changes to purplish-gray, med., clayey SAND w/occ. gravel to 3/4" dia.; loose, wet smooth, easy drilling 7.5'-12.5'	4 2 4	
707	10					
		3-A		grades slightly finer	6 5 4	
702	15					
		4-A		w/ 3/4" layers of gray, fine slightly silty sand & gravel to 3/4" dia.; med. dense	14 7 5	
697	20			contains clean sand layers 1/2" thick tip on rock at 23.7', trace of organics at 23.4'		
		5-A			3 6 50/3"	
				Bottom of boring at depth 23.8' Piezometer installed in hole w/ tip at 23.8', backfilled w/pea gravel, sealed w/bentonite, & cast iron monument set in concrete plug. Completed 5/31/77		
692	25					



A. 2" O.D. split-spoon sampler
B. 3" O.D. thin-wall sampler
C. 3-1/4" O.D. x 2-1/2" liner
D. 3-1/2" O.D. split barrel sampler
X. sample not recovered

** Standard Penetration Resistance except for 2" O.D. split-spoon samples estimated using non-standard procedures.

water level
impervious seal
piezometer tip

ENUMCLAW WASTEWATER FACILITY IMPROVEMENTS
for
Kramer, Chin & Mayo

Project No. 77-5109-01

ConverseDavisDixon Geotechnical Consultants

Drawing No. 6

FORM NO. D58/77

Approved for publication 3/16/78 by EKM

DATE DRILLED 5/27/77

LOG OF BORING NO. 5D

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED ARE A SIMPLIFICATION OF ACTUAL CONDITIONS ENCOUNTERED

ELEVATION	DEPTH IN FEET	SAMPLES *	SYMBOL	DESCRIPTION	BLOWS/6"	DRY DENSITY LB./CU FT.	STANDARD PENETRATION ▲ BLOWS PER FOOT ** (0 – 20 – 40)
671			GM	Brown, gravelly SAND w/cobbles			
		1-A	SM	Gray, clayey SAND w/gravel; med. dense, very moist gravel & cobbles below 3'	4 75/3"		▲
666	5						
				Bottom of boring at depth 6' due to refusal of auger; unable to penetrate cobbles in 2 redrill attempts			
661	10						

0 20 40
WATER CONTENT - % dry weight

A. 2" O.D. split-spoon sampler
B. 3" O.D. thin-wall sampler
C. 3-1/4" O.D. x 2-1/2" liner
D. 3-1/2" O.D. split barrel sampler
X. sample not recovered

** Standard Penetration Resistance except for 2" O.D. split-spoon samples estimated using non-standard procedures.

water level
impervious seal
piezometer tip

plastic limit — liquid limit
natural water content

ENUMCLAW WASTEWATER FACILITY IMPROVEMENTS
for
Kramer, Chin & Mayo

Project No. 77-5109-01

ConverseDavisDixon Geotechnical Consultants

Drawing No. 7

Approved for publication 3/16/78 by EKM

RM NO. D58/77

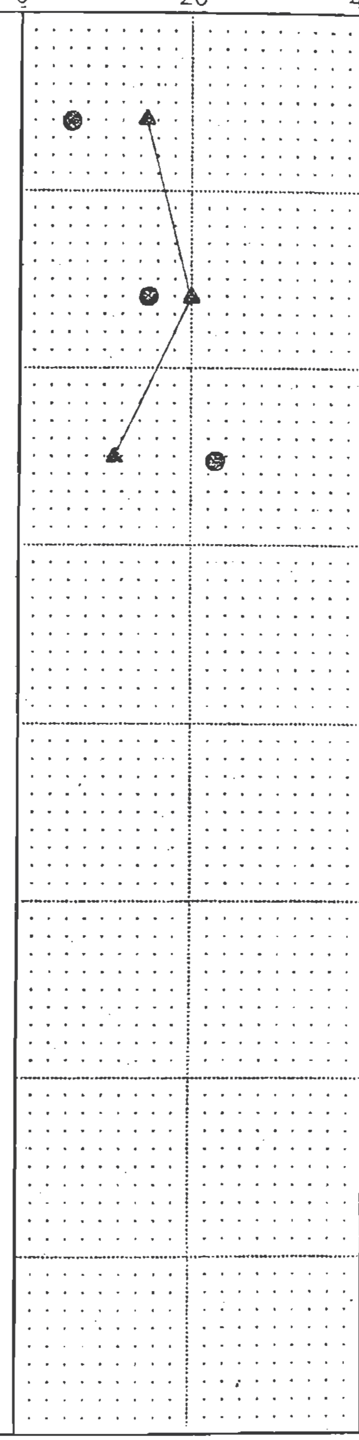
DATE DRILLED 2/24/78

LOG OF BORING NO. 9D

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED ARE A SIMPLIFICATION OF ACTUAL CONDITIONS ENCOUNTERED.

ELEVATION	DEPTH IN FEET	SAMPLES*	SYMBOL		BLOWS/6"	DRY DENSITY LB/CU FT.
713			GP	brown to dark gray, fine to coarse sandy GRAVEL to 1" dia.; med. dense, moist (fill)		
		1-A			20 9 6	
708	5					
		2-A	SC	Mottled gray-brown to purplish-gray clayey SAND w/gravel to 1/2" dia.; med. dense, very moist (Osceola Mudflow)	6 11 9	
703	10					
		3-A		very moist to wet at 13'	7 6 5	
698	15			Bottom of boring at depth 14.5' Piezometer installed in hole w/ tip at 14.5', backfilled w/pea gravel, sealed w/bentonite, & cast iron monument set in concrete plug at surface. Completed 2/24/78		

STANDARD PENETRATION
▲ BLOWS PER FOOT **



WATER CONTENT - % dry weight

plastic limit — natural water content — liquid limit

* A. 2" O.D. split-spoon sampler
B. 3" O.D. thin-wall sampler
C. 3-1/4" O.D. x 2-1/2" liner
D. 3-1/2" O.D. split barrel sampler
X. sample not recovered

** Standard Penetration Resistance except for 2" O.D. split-spoon samples estimated using non-standard procedures.

water level
impervious seal
piezometer tip

ENUMCLAW WASTEWATER FACILITY IMPROVEMENTS
for
Kramer, Chin & Mayo

Project No. 77-5109-01

ConverseDavisDixon Geotechnical Consultants

Drawing No. 11

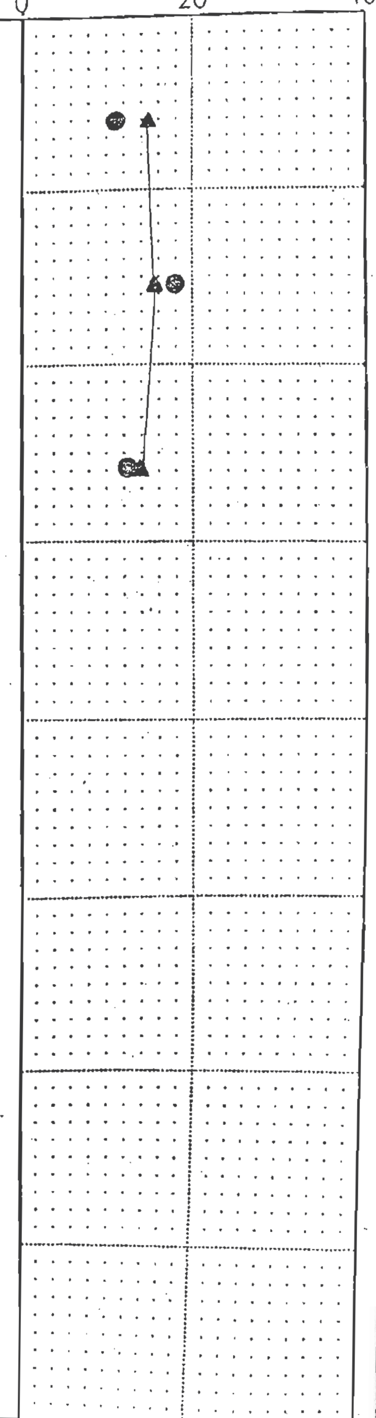
FORM NO. D58/77 Approved for publication 3/16/78 by ERM

DATE DRILLED 2/24/78

LOG OF BORING NO. 10D

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED ARE A SIMPLIFICATION OF ACTUAL CONDITIONS ENCOUNTERED

ELEVATION	DEPTH IN FEET	SAMPLES *	SYMBOL	DESCRIPTION	BLOWS/6"	DRY DENSITY LB./CU FT.
689			GP	brown, sandy GRAVEL w/numerous cobbles; med. dense, moist (fill)		
		1-A			24 8 7	
684	5		SC	purplish-gray, fine to coarse, clayey SAND w/gravel to 1" dia.; med. dense, very moist (Osceola Mudflow)		
		2-A		wet; (heavy cobbles encountered from 8' to 14.5')	6 8 8	
679	10					
		3-A			3 6 8	
674	15			Bottom of boring at depth 14.5' Piezometer installed w/tip at 14.5', backfilled w/pea gravel, sealed w/bentonite at 4' depth, & cast iron monument set in concrete plug at surface. Completed 2/24/78		



A. 2" O.D. split-spoon sampler
B. 3" O.D. thin-wall sampler
C. 3-1/4" O.D. x 2-1/2" liner
D. 3-1/2" O.D. split barrel sampler
X. sample not recovered

** Standard Penetration Resistance except for 2" O.D. split-spoon samples estimated using non-standard procedures.

water level
impervious seal
piezometer tip

plastic limit — liquid limit
natural water content

ENUMCLAW WASTEWATER FACILITY IMPROVEMENTS
for
Kramer, Chin & Mayo

Project No. 77-5109-01

Drawing No. 12

ConverseDavisDixon Geotechnical Consultants

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